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TECHNICAL REPORT CERC-89-15

CRITERIA FOR EVALUATING COASTAL FLOOD-PROTECTION STRUCTURES

by

Todd L. Walton, Jr., John P. Ahrens

Coastal Engineering Research Center

DEPARTMENT OF THE ARMY

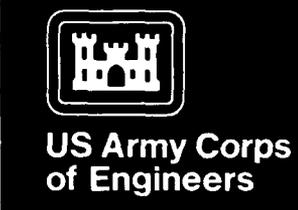
Waterways Experiment Station, Corps of Engineers
3909 Halls Ferry Road, Vicksburg, Mississippi 39180-6199

Clifford L. Truitt

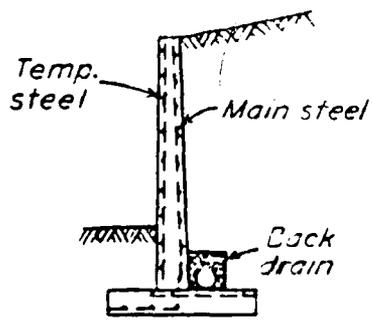
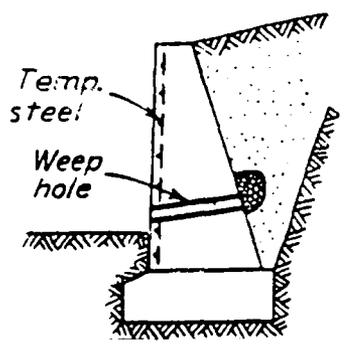
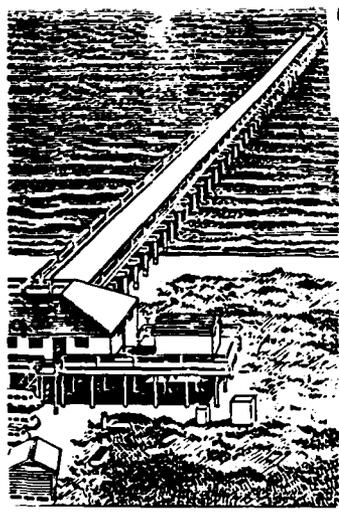
Coastal Planning and Engineering, Inc.
1605 Main Street, Sarasota, Florida 34236

Robert G. Dean

Coastal and Oceanographic Engineering Department
University of Florida
Gainesville, Florida 32611



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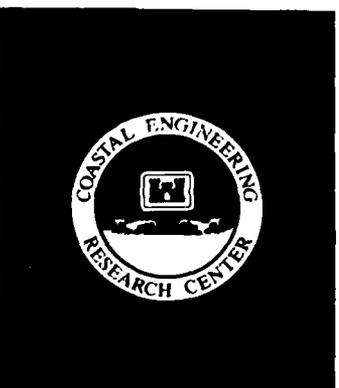


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SUMMARY

Crude state-of-the art methods are presented for predicting wave runup, overtopping, and wave transmission past coastal flood-protection structures. The runup methodology presented provides equations for calculation of runup on smooth sloped structures along with correction factors to adjust the smooth slope runup value in the case of rough slopes. The runup methodology can be used along with the existing Federal Emergency Management Agency (FEMA) methodology for runup on beaches to assess adequacy of a given coastal flood-protection structure to prevent flooding. A first crude approach to the problem of predicting wave transmission behind the structure via overtopping has been presented. The method is preliminary, and insufficient testing of the method for general usage has been made because of lack of available data. If necessary, the method could be used as an interim procedure to determine the level of wave protection afforded buildings behind the coastal flood-protection structure should runup exceed the crest height of the coastal flood-protection structure.

The survivability of the coastal flood-protection structure through the 100-year frequency of occurrence storm is also addressed in this report. Common types of coastal flood-protection structures have been categorized by functionality, and modes of failure for these structures have been discussed. The primary types of coastal flood-protection structures are gravity seawalls, pile-supported seawalls, anchored bulkheads, and dikes (or "levees"). The first three of these categories can be assessed for stability in a reasonable manner where sufficient design details of the structure and adequate information as to present condition and state of maintenance are known. There are no adequate stability assessment criteria for application to general types of coastal dikes, although limited information on some design aspects for a few specific cases does exist. A flowchart check list has been provided to assess adequacy of coastal flood-protection structures to withstand the 100-year return period coastal storm. Literature pertinent to such checks has been provided to enable a qualified coastal and geotechnical engineer to address the coastal flood-protection structure survivability issue. One approach to assessing adequacy of coastal flood-protection structures (where design information is limited or where inadequate knowledge of present condition is known) is via history of past performance of similar type structures. A review of

literature on historical performance of various coastal flood-protection structures is provided in this report, and it is concluded that only one type of coastal flood-protection structure (anchored bulkhead) has shown repeated history of serious failures in large coastal storms. It is recommended that FEMA not consider anchored bulkheads adequate for coastal flood protection unless the applicant can prove with sufficient documented engineering evidence the structure(s) in question will survive the FEMA postulated 100-year return period coastal storm scenario.

It must be realized that, due to the complexity of the typical coastal flood-protection structure, definitive answers to the question of structure adequacy cannot always be made without great expense and numerous engineering calculations. Even then, many questions such as the internal structural strength of the coastal flood-protection structure (i.e. adequacy and condition of reinforcing steel) will still be left unanswered. In such cases, good engineering judgment and common sense will still have to be an important part of any evaluation of such structures.

PREFACE

This report examines various methodologies for assessing the adequacy of coastal flood-protection structures to survive a large coastal storm. The report was prepared at the Coastal Engineering Research Center (CERC) of the US Army Engineer Waterways Experiment Station (WES) in response to a request from the Federal Emergency Management Agency (FEMA). Dr. Frank Tsai was FEMA's contract monitor.

Dr. Todd L. Walton, Jr., Mr. John P. Ahrens, Dr. Clifford L. Truitt, and Dr. Robert G. Dean prepared the report under the general supervision of Mr. Thomas W. Richardson, Chief, Engineering Development Division; and Dr. James R. Houston and Mr. Charles C. Calhoun, Jr., Chief and Assistant Chief, CERC, respectively. This report was edited by Ms. Lee Byrne, Information Technology Laboratory, WES.

Commander and Director of WES upon publication of this report was COL Larry B. Fulton, EN. Dr. Robert W. Whalin was Technical Director.

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CONVERSION FACTORS, NON-SI TO SI (METRIC)
UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

<u>Multiply</u>	<u>By</u>	<u>To Obtain</u>
cubic feet	0.02831685	cubic metres
degrees (angle)	0.01745329	radians
feet	0.3048	metres
inches	25.4	millimetres
miles (US statute)	1.609347	kilometres
pounds (force) per square inch	6.894757	kilopascals
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
tons (2,000 pounds, mass)	907.1847	kilograms

CRITERIA FOR EVALUATING COASTAL FLOOD-PROTECTION STRUCTURES

PART I: OBJECTIVES OF STUDY

1. The Federal Emergency Management Agency (FEMA) is tasked with the job of establishing potential flood zones along the coast for use in determining property insurance rates within the structure of the Federal Insurance Administration. Within the high hazard coastal area, many coastal flood-control structures (such as seawalls) have been built to protect property from erosion as well as to prevent flooding of the upland areas. At present FEMA does not have any guidance to determine adequacy of these structures: (a) to prevent flooding from a major storm event or (b) to withstand the major storm event without some type of structural failure occurring.

2. At the onset of the study, specific requests by FEMA were made considering the philosophical approach to the problem of evaluating a coastal flood-control structure's potential in providing flood protection. These requests can be summarized as follow: (a) the methodology recommended be consistent with the state of the art in coastal engineering, (b) the methodology be generic in the sense it can be used for any structure, and (c) the methodology be consistent with existing FEMA guidelines and procedures in evaluating coastal flood zones. In a preliminary meeting, it was noted that these three recommendations might not all be capable of being fulfilled completely. As an example, if it was determined that the state-of-the-art method for determining wave runup on a structure was via utilization of a random wave method, this would not be consistent with the present FEMA approach of utilizing depth-limited breaking wave height for developing coastal high hazard zones (the V-zone determination). As a result of discussions in these meetings, it was determined that monochromatic wave theory and depth-limited breaking waves should be the governing approach. One justification for this is that present knowledge of wave data around the Nation's coastline is not adequate to justify usage of a more complex random wave theory for general evaluation of coastal structures at all locations. This statement does not conflict with the fact that present site-specific design of coastal protection is based on random wave input into a laboratory tank with the measured response providing

design conditions. Rather, this statement suggests that where sufficient wave climate knowledge is available and where site-specific laboratory tank tests of the structures can be made, exemptions to a proposed methodology should be allowed. Further justification for utilizing monochromatic wave theory for a unified FEMA approach will be provided in the various sections of the report.

3. This report is organized into seven sections. Part I states the objectives with the next three sections addressing the hydrodynamic recommendations for calculation of wave runup, wave overtopping, and wave transmission. This methodology is necessary to identify if a coastal structure that is intact through a major storm event will provide the necessary level of flood and wave protection to upland property. Part V addresses the calculation of wave forces on the coastal flood-control structure. Knowledge of the wave forces on the structure is required to assess the adequacy of the structure against wave damage during a major storm. Parts II through V address methodology that can be utilized in a generic sense to evaluate any coastal structure where laboratory tank testing of the structure for the major storm under consideration does not exist. Part VI addresses the problem of structural integrity of the coastal flood-protection structure, i.e., its ability to withstand the major storm intact. In this section, guidelines are provided for checking various potential failure modes of coastal flood-control structures. Part VII provides a flowchart along with recommendations for determining adequacy of design.

4. The present report is a first attempt to provide guidance to FEMA for use in assessing adequacy of coastal flood-control structures (primarily seawalls) for flood protection against the extreme event. This report is not meant to address design of coastal flood-protection structures and should not be used for design purposes. Considerable in-depth studies and tests would be needed for proper design. The subject of design is beyond the scope of this report.

5. It should be recognized that the state of the art in understanding coastal hydrodynamics is very crude and often belies theoretical attempts to characterize waves and water levels for coastal areas with a desired high level of accuracy. Present state-of-the-art attempts at characterization of water waves on structures is highly dependent on laboratory testing done under controlled environmental conditions to determine empirical knowledge of the resultant interaction between wave and structure. Results from such tests

provide knowledge for use in similar situations in the real world but must be recognized as only providing an approximate crude answer for the real world situation. Until considerable further research is done on nearshore wave interaction with structures, attempts to predict nature's response to a major storm event will only be a rough guess at what may really occur. In light of this inadequate knowledge of nature, the attempt is to err on the side of conservatism where possible.

PART II: WAVE RUNUP ON STRUCTURES

Background

6. Design of coastal flood-protection structures for which no overtopping or flooding landward thereof is allowable should use a maximum wave runup criterion. The maximum wave runup criterion is consistent with a desire of FEMA to have a clearly defined pass/fail type approach to evaluate whether existing structures should be credited for flood protection. A methodology to predict maximum wave runup under the existing FEMA approach of depth-limited waves would allow FEMA to directly evaluate a structure for flood-protection purposes by evaluating the maximum runup elevation to be above or below the structure crest elevation. In particular, a reasonable approach for use by FEMA in evaluating flooding potential behind a structure should consist of the following points:

- a. The methodology should be sufficiently robust to work on all structure slopes, structure roughnesses, and structure types.
- b. The methodology should be independent of existing bathymetry leading to the structure, i.e., decoupled from wave transformation effects prior to encountering the structure.
- c. The methodology should be consistent with existing theory and verified by physical model testing at a scale sufficient to ensure that scale effects are minimized in the data (or provide a rationale to correct for such scale effects).
- d. The methodology should be consistent with existing FEMA monochromatic depth-limited breaking wave criteria.
- e. The methodology should provide consistent answers with existing knowledge of coastal flooding events at structure sites.

7. Present state of the art in runup prediction is not sufficient to adequately address all of these points. The primary reason for this inadequate state of knowledge is the fact that (in this country as well as in other countries with coastal flooding problems) generic research sets of runup data for various structure types, locations, slopes, bathymetry, roughnesses, scales, etc., do not exist. The primary countries where limited research efforts have been made advancing the state of knowledge of runup on structures are the European countries (mainly the United Kingdom, Germany, and the Netherlands), Japan, and the United States. In the majority of runup studies within these countries, the studies were made with a limited objective of

designing a site-specific structure of a given type, slope, roughness, off-shore bathymetry, and with wave conditions consistent with the site. Limited studies exist that have addressed the physics of runup. These are either verified (or more likely calibrated) on very limited sets of field or laboratory data or not verified at all. A majority of the recent research on wave runup on structures consists of irregular wave input at laboratory tank wave generators with corresponding irregular wave runup measurements on the structure, again for very limited conditions. This recent research has progressed with irregular wave input (typically for site-specific conditions) in spite of the fact that the simpler problem of monochromatic wave runup is still not well understood.

8. In view of these comments, it must be realized that any approach to the problem of runup prediction which meets FEMA needs will not be without limitations, but rather a pragmatic approach to the problem out of need for a cost justifiable approach which engineers can use to provide a state-of-the-art answer to a very complex problem. For an improved answer to the runup problem, it will still be necessary, as in present coastal design, to do laboratory testing for site-specific cases. Those applicants to FEMA requiring credit for their coastal flood-protection structures should be allowed the option of providing independent physical (laboratory) testing on their structure in lieu of any proposed FEMA suggested approach. A short review of existing available data sets and runup research on both monochromatic and irregular wave runup is presented followed by discussion of various methodologies for prediction of wave runup.

9. Much of the existing state-of-the-art approach to computing wave runup on structures is summarized in nine documents: Koh and Le Méhauté (1966); Le Méhauté, Koh, and Hwang (1968); the Technical Advisory Committee on Protection Against Inundation (TACPI) (1974); Stoa (1978a, 1978b, 1979); Horikawa (1978); the Shore Protection Manual (SPM) (1984); and Allsop, Franco, and Hawkes (1985).

10. The SPM (1984) as amended by various Coastal Engineering Technical Aids (CETA's) (primarily those by Stoa (1978b, 1979)) along with Stoa (1978a) summarizes the present design methodology used by the US Army Corps of Engineers (USACE) in providing preliminary guidance for runup calculations in the United States. In this approach, runup calculations are made primarily by means of runup curves based on physical modeling results for various input

wave conditions and a specific type of structure and offshore bathymetry. No guidance has been provided for situations outside of specific combinations of offshore bathymetry and structure tested. Design practice dictates that physical model tests must be made for structures and offshore bathymetry outside of those special conditions tested. Limited guidance is also provided in SPM (1984) for scaling up laboratory results of runup to prototype conditions. Unfortunately such guidance is derived from only three data points* and cannot be used for steep sloped structures where the runup scale correction factor approaches infinity. Stoa (1978a) provides a refined suggested curve for scale correction based on his intuition and the same limited data. An untested methodology for using the results of the runup curves for computing irregular wave runup values is also provided in SPM (1984).

11. The TACPI (1974) report provides a review of various data sets and results of laboratory testing on regular and irregular wave runup, as well as considerable information on laboratory results of runup effects due to structure roughness, structure slope(s) (in the case of composite structure slope), and various features of the offshore bathymetry (such as berm). Again, no generic runup methodology is provided that would cover all instances, but rather the approach is predicated based on the use of existing laboratory and theoretical information for specific structures or structure features where physical model results exist. A review of both nonbreaking wave and breaking wave runup theoretical aspects is included, but no guidance is provided that would cover all instances in design. In particular, most of the literature reviewed pertains to nonsteep structures typical of dikes rather than steep sloped structures such as near-vertical walled flood-protection structures. Dutch methodology in design of coastal structures also utilizes site-specific physical model testing for obtaining runup. The TACPI (1974) provides information that would help in optimizing a model testing program for design of a flood-protection structure.

12. Horikawa (1978) reviews the Japanese approaches to runup calculations. Japanese design is also predicated on the results of site-specific physical modeling with no clear attempts to provide a predictive approach for calculation of runup under all circumstances of wave action, structure

* T. Saville, Jr., 1960, "Scale Effect in Wave Runup," unpublished paper presented at the American Society of Civil Engineers Convention, Boston, MA.

configuration, etc. Horikawa notes design runup curves derived from physical model tests for specific beach and structure configurations (Toyoshima, Shuto, and Hashimoto (1966)) that could be considered analogous to SPM (1984) runup curves used in the United States.

13. Koh and Le Méhauté (1966) and Le Méhauté, Koh, and Hwang (1968) provide a general review of theoretical approaches to the runup problem along with a discussion of wave runup under both breaking and nonbreaking waves. Intuitive approaches for blending of the two different physical types of runup (breaking and nonbreaking) to provide a coherent and consistent approach to presenting runup data are given.

14. Research on runup conducted in other countries (primarily the United Kingdom and Germany) is confined mostly to various technical papers in journals, papers presented at conferences, and laboratory reports. Allsop, Franco, and Hawkes (1985) provide an updated review of much of this research.

15. It is important to note that in all of these countries, there is no attempt to define a predictive runup computation strategy for all possible scenarios because all flood-protection structures undergo site-specific physical modeling tests to optimize structure design based on the known bathymetry at the site and the known or postulated (irregular or depth-limited) wave conditions that would occur during design event(s). This approach to design substantiates the statement made earlier that present state of the art in wave runup prediction does not allow an engineer to compute wave runup from a theoretical basis over all conditions of wave action and structure type/configuration with any degree of confidence. This statement is reinforced for the case of irregular waves, the reasons for which will be noted later.

16. Some of the more pertinent literature on the topic of runup (with emphasis on literature not incorporated in the previous nine primary references) will be noted in the following paragraphs along with a short discussion of the theoretical aspects of periodic wave runup on smooth slopes under breaking and nonbreaking waves. A summary of attempts at periodic wave runup prediction on sloped structures will be reviewed along with results of testing potentially useful smooth slope methods. A robust approach to the calculation of a maximum wave runup on structures will be presented for the case of periodic waves on smooth sloping structures. Recent results of measurements of periodic wave runup on nonsmooth sloped structures will also be presented along with a recommended approach to modifying smooth slope wave runup to

account for roughness and scale effects. Such a methodology will allow these smooth slope predictions to be extended to more complex structures by means of simple correction factors to develop a robust approach for maximum wave runup under depth-limited breaking waves consistent with present FEMA flood zone evaluation. A review of attempts to extend the results of periodic wave runup theoretical calculations or physical modeling results to the irregular wave domain problem will be made along with a rational explanation of why such approaches may be of limited use in practical problems at present.

Regular Wave Runup on Smooth Linear Slopes

17. As noted previously, many of the attempts at predicting runup are physical modeling based. In the United States, past physical model experiments by Grantham (1953); Saville (1955, 1956, 1958, 1962); Savage (1957, 1958); and Hudson, Jackson, and Cuckler (1957) for periodic wave runup on smooth slopes (and sand roughened slopes in the case of Savage (1957, 1958)) appear to be the most comprehensive data sets available. The only data set on periodic wave runup that approximates a generic data set is the smooth slope data of Saville (1956, 1962) and Savage (1957, 1958), which is the basis for most of the runup curves in Stoa (1978a) and in SPM (1984) and which extends over several slopes for a limited number of offshore bathymetric conditions. These data will be discussed further in a later section of the report. The data sets of Saville (1956) and Hudson, Jackson, and Cuckler (1957) appear to be the primary data sets used by Hunt (1959) in his classic empirical approach to evaluate wave runup under breaking waves. The so-called Hunt equation was the first attempt at a prediction equation for runup under breaking waves. The Hunt equation is given in its original form as follows (in non-SI units):

$$R = 2.3(HT^2)^{1/2} \tan(\theta) \quad (1)$$

where

- R - wave runup (feet)
- H - wave height (feet)
- T - wave period (seconds)
- θ - structure slope

Battjes (1974b) noted that this expression is not dimensionally correct and could be nondimensionalized to a form as follows:

$$\frac{R}{H} = \frac{\tan(\theta)}{\left(\frac{H}{L_0}\right)^{1/2}} = I_r \quad (2)$$

where L_0 is the deepwater wavelength* and I_r is the Iribarren number. Numerous investigators have noted that the Hunt expression leads to reasonable predictions of runup under breaking wave conditions. A particular problem with the expression given in the form above is that $\tan(\theta)$ goes to infinity as structure slope increases toward a vertical walled structure.

18. Runup theories for nonbreaking inviscid fluid waves on smooth (frictionless) slopes are typically of a form as noted by Koh and Le Méhauté (1966) given by:

$$\frac{R}{H} = \left(\frac{1}{K_s}\right) \left(\frac{\pi}{2\theta}\right)^{1/2} + (\text{higher order non linear terms}) \quad (3)$$

where K_s is a linear shoaling coefficient. The first term on the right-hand side of Equation 3 is often attributed to Miche (1951). It should be noted that the shoaling coefficient dependency is missing in Koh and Le Méhauté (1966), most likely because Miche's results are derived for deep water. The higher order nonlinear terms in Equation 3 vary from researcher to researcher but are typically of a form:

$$(\text{Higher order nonlinear terms}) = \left(\frac{\pi H}{L}\right) \left[\frac{1}{\tanh\left(\frac{2\pi d}{L}\right)} \right] (\text{1 + more nonlinear terms}) \quad (4)$$

where

L = wavelength

d = depth

The higher order terms are typically a small portion of the overall nonbreaking (standing) wave runup on the structure.

* For convenience, symbols and abbreviations are listed in the Notation (Appendix D).

19. The critical wave/slope combination at which the transition from standing wave to initiation of breaking occurs is typically of a form due to Miche (1951):

$$\frac{H}{L} = \left(\frac{2\theta}{\pi}\right)^{1/2} \left[\frac{\sin^2(\theta)}{\pi}\right] \quad (5)$$

It should be noted that Keller (1961) has derived a similar form of Equation 5 only with $\sin(\theta)$ replaced by " θ " and a different constant.

20. Other researchers, notably Jackson (1968), Dai and Kamel (1969), Ahrens (1975a, 1975b), and Gunbak (1979), have measured periodic wave runup on a variety of rubble mound, riprap, and concrete-armored block-layered structures. Ahrens and McCartney (1975), Gunbak (1979), and Coastal Engineering Research Center (CERC) (1985) (CETN-I-37, 12/85) present equations for runup that encompass the entire breaking/nonbreaking range of wave conditions for these type structures with empirical equations of the form:

$$\frac{R}{H} = \frac{a I_r}{1 + b I_r} \quad (6)$$

where a, b are empirical coefficients dependent on structure type and slope. These equations are meant to provide a fitting of runup data over the entire breaking/nonbreaking wave runup zone, but unfortunately address only a limited number of structure types and slopes. The philosophy behind this empirical equation approach can be seen by assuming small I_r (breaking wave region) where the equation reduces to a form:

$$\frac{R}{H} = a I_r \quad (7)$$

Equation 7 is of a Hunt equation form. For the nonbreaking region ($I_r \gg 0$), the equation reduces to:

$$\frac{R}{H} = \frac{a}{b} \quad (8)$$

Equation 8 is a constant similar to nonbreaking (standing wave) theory for a given slope and no nonlinear terms.

21. Many of these same runup data sets on nonsmooth (permeable and impermeable) sloped structures are summarized as well as reworked in Losada and Gimenez-Curto (1981), who have provided empirical expressions for calculating wave runup in the form:

$$\frac{R}{H} = a[1 - \exp(-b I_r)] \quad (9)$$

where a, b are empirically fit coefficients for the data, again slope and structure dependent.

22. As these empirical equation approaches to calculate periodic wave runup were fit to runup measurements from a variety of researchers using different measurement techniques, laboratory equipment, and scales of testing, any attempts to interpolate or extrapolate such results to structures outside the range or scale of those tested should be done with extreme caution.

23. Losada and Gimenez-Curto (1981), as well as other researchers, note that runup on smooth slope structures does not follow the trend of the roughened, permeable structures. The smooth slope structure periodic wave runup data as noted in runup curves of various researchers do not follow a smooth monotonically increasing value of relative runup (R/H) with increasing Iribarren number (or increasing slope) as does the roughened permeable structure runup data. Many researchers, notably Gunbak (1979) and Sawaragi, Iwata, and Kobayashi (1982), attribute this phenomenon to a resonance on the slope, although it would appear that this is more likely due to a more clearly defined transition from the breaking wave runup to the nonbreaking wave runup process under the less turbulent conditions on a smooth slope.

24. Losada and Gimenez-Curto (1981) present a series of expressions for calculation of wave runup on smooth slopes over the full range of Iribarren number as follows:

$$\frac{R}{H} = I_r \quad 0 < I_r < 2.5 \quad (10a)$$

$$\frac{R}{H} = 2.5 - \frac{I_r - 2.5}{3.0} \quad 2.5 < I_r < 4.0 \quad (10b)$$

$$\frac{R}{H} = 2.0 \quad 4.0 < I_r \quad (10c)$$

These expressions are not consistent with Saville's (1956) and Savage's (1957) data presented in a revised format in Appendix A.

25. Tautenhaim, Kohlase, and Partenscky (1982) propose an equation for periodic wave runup of the form:

$$R = 1.29(HL_0)^{1/2} \frac{1 - K_r}{\cos(\theta)} \quad (11)$$

where K_r is the reflection coefficient of the structure. For the case of complete breaking ($K_r = 0.0$), this equation can be rewritten as:

$$\frac{R}{H} = \frac{1.29 I_r}{\sin(\theta)} \quad (12)$$

Equation 12 shows an additional slope effect beyond what is already included in the Hunt equation. This equation predicts a finite runup effect on horizontal slopes that increases toward an infinite runup value on vertical slopes (assuming breaking waves and no reflection). Such effects are contrary to physical intuition. As noted in previous paragraphs and will be shown in reanalysis of Saville (1956) and Savage (1957) data, the Hunt equation appears to be very reliable for mild slopes, suggesting that the additional slope effect in Equation 12 is not physically correct. For the case of nonbreaking wave runup, i.e., runup under pure standing waves with $K_r = 1.0$, the Tautenhaim, Kohlase, and Partenscky (1982) expression would predict zero runup, a physically incorrect answer. As Tautenhaim, Kohlase, and Partenscky (1982) do not clearly explain the rationalization for this equation that appears to be amenable to both breaking and nonbreaking wave regions, it will be dismissed.

26. Chue (1980) also provides an expression for periodic wave runup with claimed wide applicability. The form of his equation is as follows:

$$\frac{R}{H} = 1.8 \left(1 - \frac{3.11H_o}{L_o} \right) \left\{ \frac{1 - \exp \left[- \left(\frac{\pi}{2\theta} \right)^{1/2} \right]}{I_{rc}} \right\} I_{rc} \quad (13)$$

where

$$I_{rc} = \left[\frac{\tan(\theta)}{\frac{H}{L_o}} \right]^{0.4} \quad (14)$$

where

H_o = deepwater wave height

I_{rc} = Chue number

This expression is criticized by Ahrens and Titus (1985) because of its incorrect trend of relative runup of nonbreaking waves decreasing with increasing wave steepness. Due to its lack of physical basis, Chue's equation will also be dismissed.

27. Ahrens and Titus (1985) used a selected portion of the data from periodic wave tests by Saville (1956) and Savage (1958) to derive a general empirical equation for periodic wave runup on plane smooth slopes under both nonbreaking and breaking periodic wave runup. This equation for transitional waves (between breaking and nonbreaking) is of the form given below. For $2.0 \leq I_r \leq 3.5$, use:

$$\frac{R}{H} = \frac{I_r - 2.0}{1.5} \quad (\text{nonbreaking relative runup equation}) \\ + \frac{3.5 - I_r}{1.5} \quad (\text{breaking relative runup equation}) \quad (15a)$$

For $I_r \leq 2.0$, a breaking wave runup equation is used. The breaking relative runup equation is given by:

$$\frac{R}{H} = 1.002 I_r \quad (\text{breaking}) \quad (15b)$$

Equation 15b is the Hunt equation for all practical purposes. For $I_r \geq 3.5$, a nonbreaking wave runup equation is used. The nonbreaking relative runup equation given in Ahrens and Titus (1985) necessitates calculating the crest

elevation of the incident wave via stream function theory. Ahrens* has more recently superseded the nonbreaking relative runup expression with a new empirical expression of the following form:

$$\frac{R}{H} = 1.087 \left(\frac{\pi}{2\theta} \right)^{1/2} + 0.775G \quad (\text{nonbreaking}) \quad (15c)$$

where G is the Goda (1983) nonlinearity parameter given by:

$$G = \frac{\frac{H}{L}}{\tanh^3 \left(\frac{2\pi d}{L} \right)} \quad (16)$$

This expression was tested with the full set of Saville's (1956) and Savage's (1958) smooth slope periodic wave runup data and found to give reasonable agreement over much of the data although considerable scatter remained within the data sets (Figures 1-7).

28. Van Dorn (1966) has postulated an approach to the calculation of periodic wave runup on smooth impermeable sloped structures/beaches that incorporates wave nonlinearities for the nonbreaking wave runup portion of the equation. The methodology incorporates a decision to use breaking or non-breaking theory based on a modified form of the Hunt equation, as follows:

$$\frac{R}{H} = \frac{\theta}{\left(\frac{H}{L_0} \right)^{1/2}} = I_{rv} \quad (17)$$

where

$$I_{rv} = \frac{\theta}{\left(\frac{H}{L_0} \right)^{1/2}}$$

Relative runup is first calculated assuming that breaking process dominates. If $I_{rv} < \pi$, then Equation 17 is assumed to hold; otherwise a nonbreaking decision process is implemented. The nonbreaking wave runup equation used is Ursell number dependent and uses either Cnoidal wave theory or Stokes

* Personal Communication, 1987, John P. Ahrens, Oceanographer, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

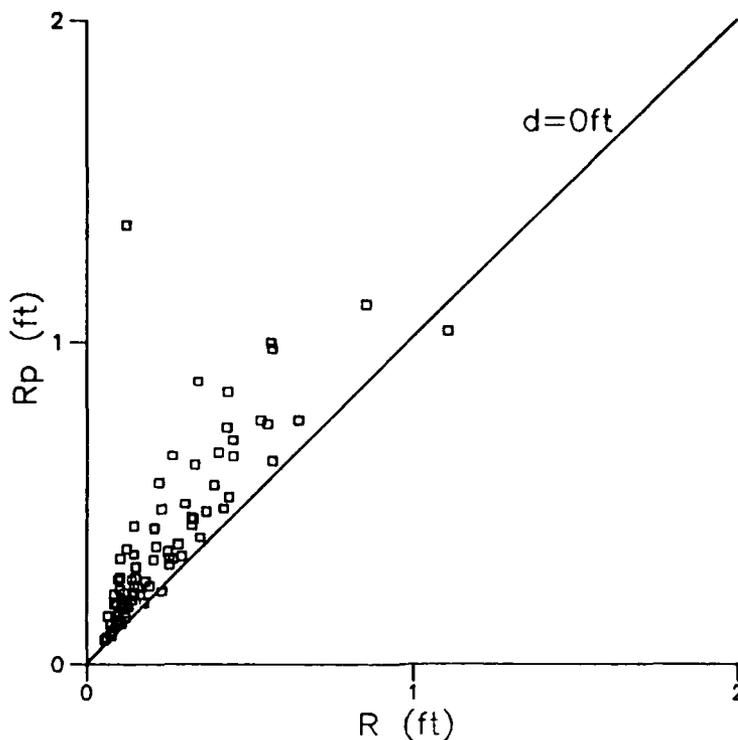


Figure 1. Predicted (R_p) versus observed (R) runup (Ahrens method), depth = 0 ft*

second-order wave theory. This methodology was also tested using the smooth slope periodic wave runup data of Saville (1956) and Savage (1958). Results of this testing are provided in Figures 8-14, where it can be seen that as in Ahrens' method, there is also considerable scatter in the data.

29. Nagai and Takada (1972) and Takada (1974) have proposed runup formulae for use on smooth slope impermeable structures that have been compared with a limited set of Japanese periodic wave runup laboratory measurements as well as some of Saville's (1956) smooth slope data. Takada (1974) is concerned only with vertical walled structures. Takada (1974) provides periodic wave runup formulae of a type similar to that proposed by Koh and Le Méhauté (1966), where a decision process is made based on critical slope or critical Iribarren number as to whether the runup process is a breaking or nonbreaking wave process. The runup equation for the breaking wave process deviates considerably from the Hunt equation and in fact resembles his nonbreaking type of equation with a minor slope effect change. Although Takada shows supporting

* A table of factors for converting non-SI units of measurement to SI units is presented on page 6.

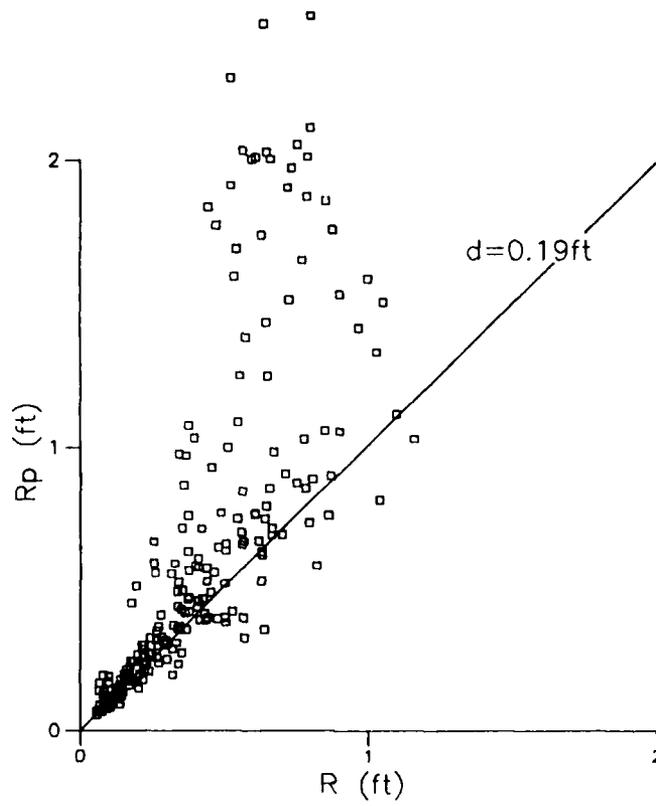


Figure 2. Predicted versus observed runup (Ahrens method), $d = 0.19$ ft

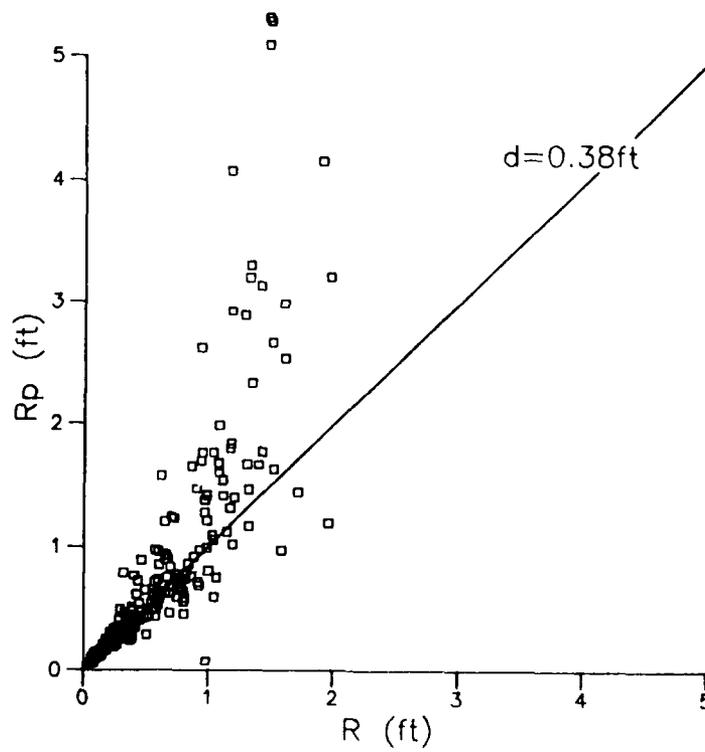


Figure 3. Predicted versus observed runup (Ahrens method), $d = 0.38$ ft

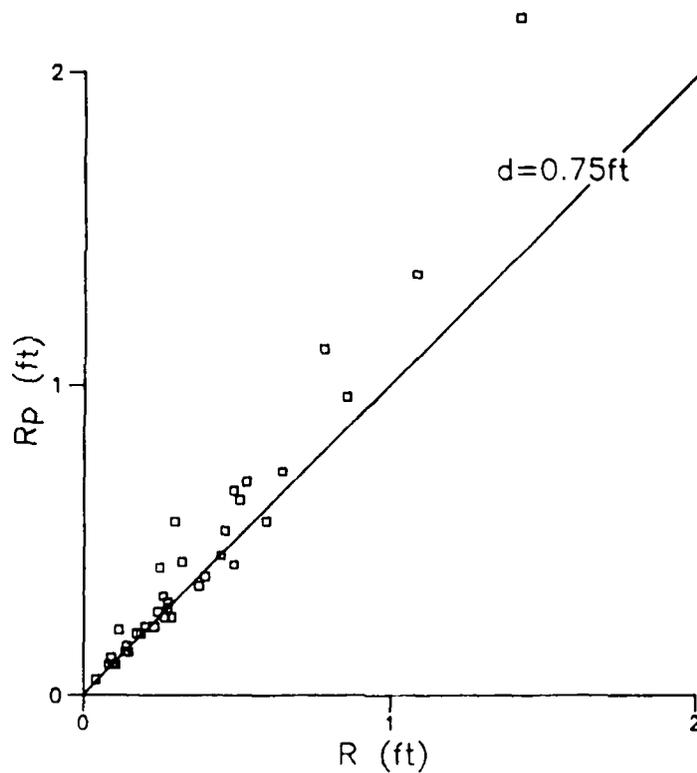


Figure 4. Predicted versus observed runup (Ahrens method), $d = 0.75$ ft

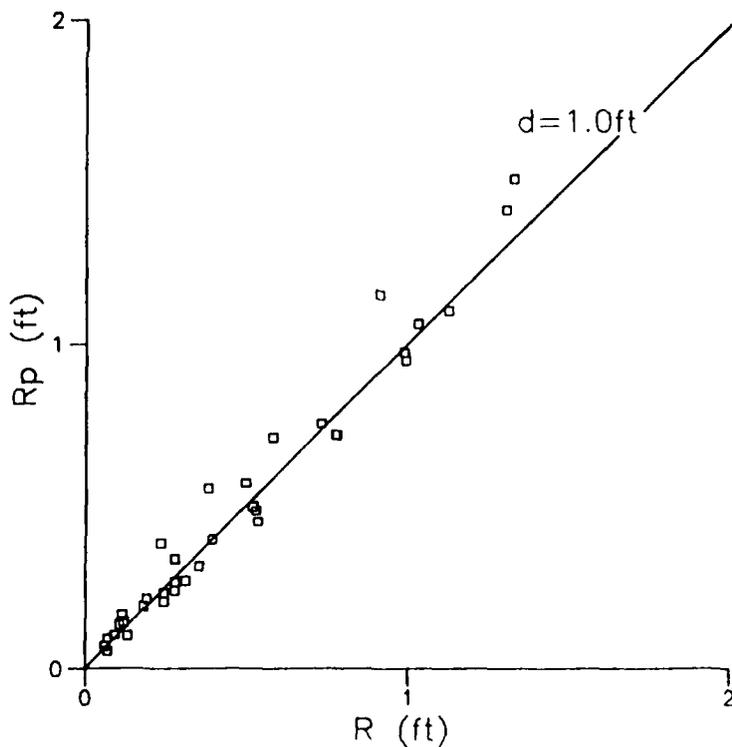


Figure 5. Predicted versus observed runup (Ahrens method), $d = 1.0$ ft

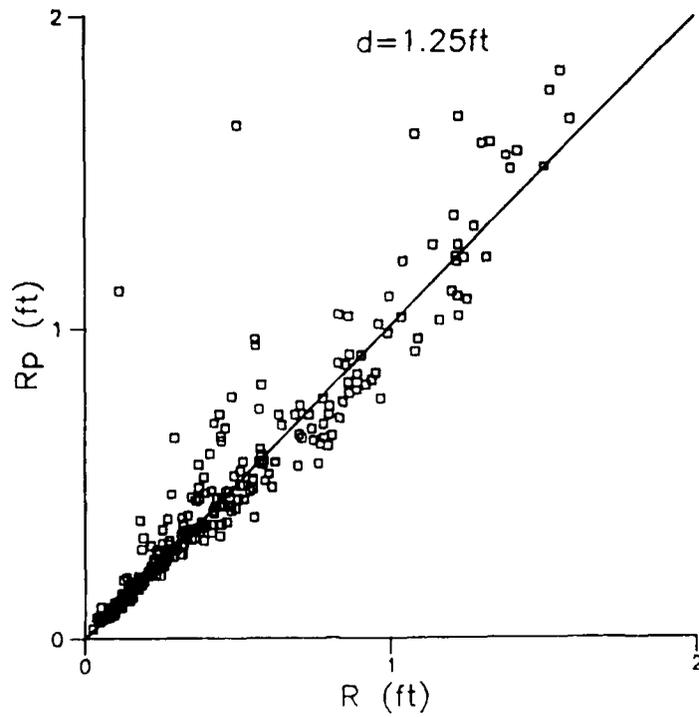


Figure 6. Predicted versus observed runup (Ahrens method), $d = 1.25$ ft

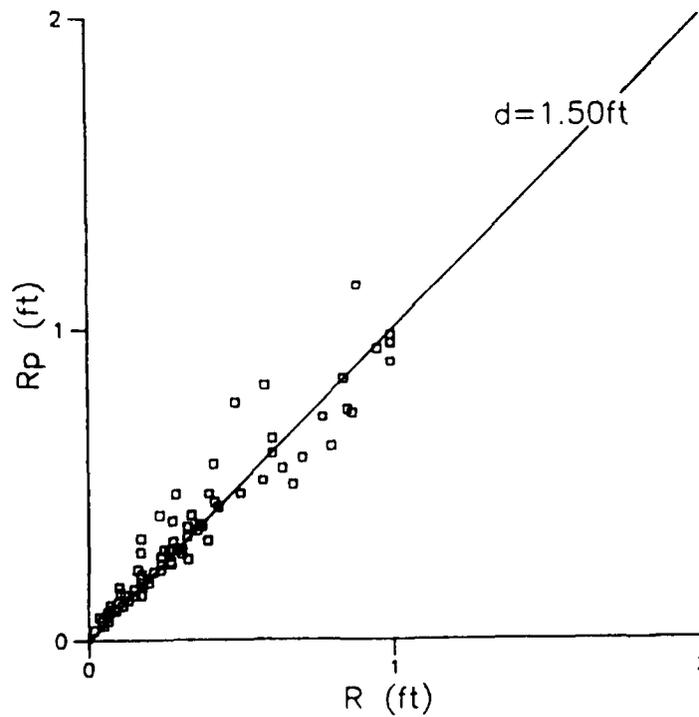


Figure 7. Predicted versus observed runup (Ahrens method), $d = 1.50$ ft

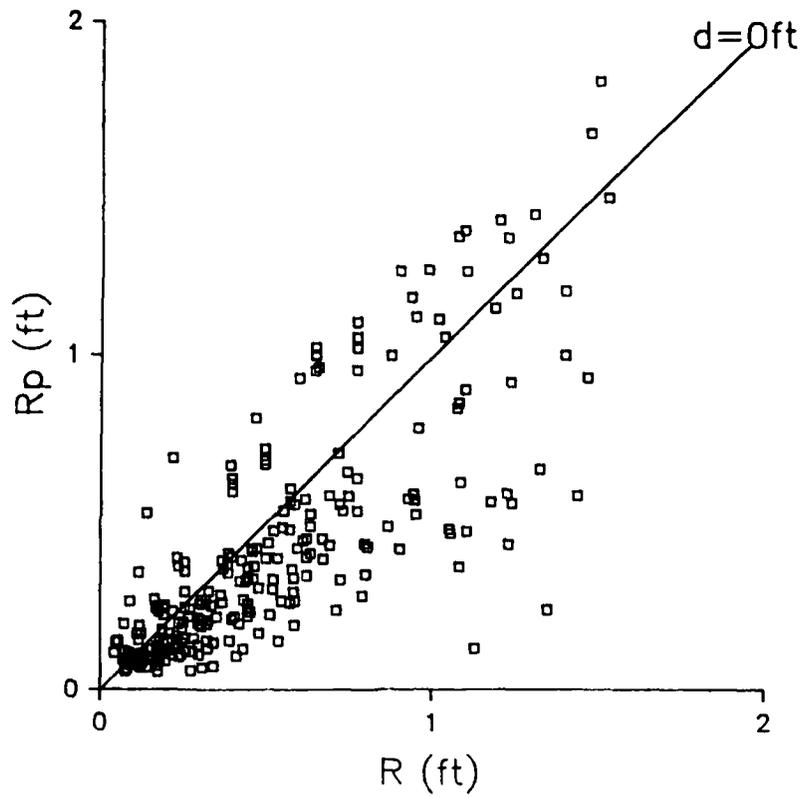


Figure 8. Predicted versus observed runup (Van Dorn method), $d = 0$ ft

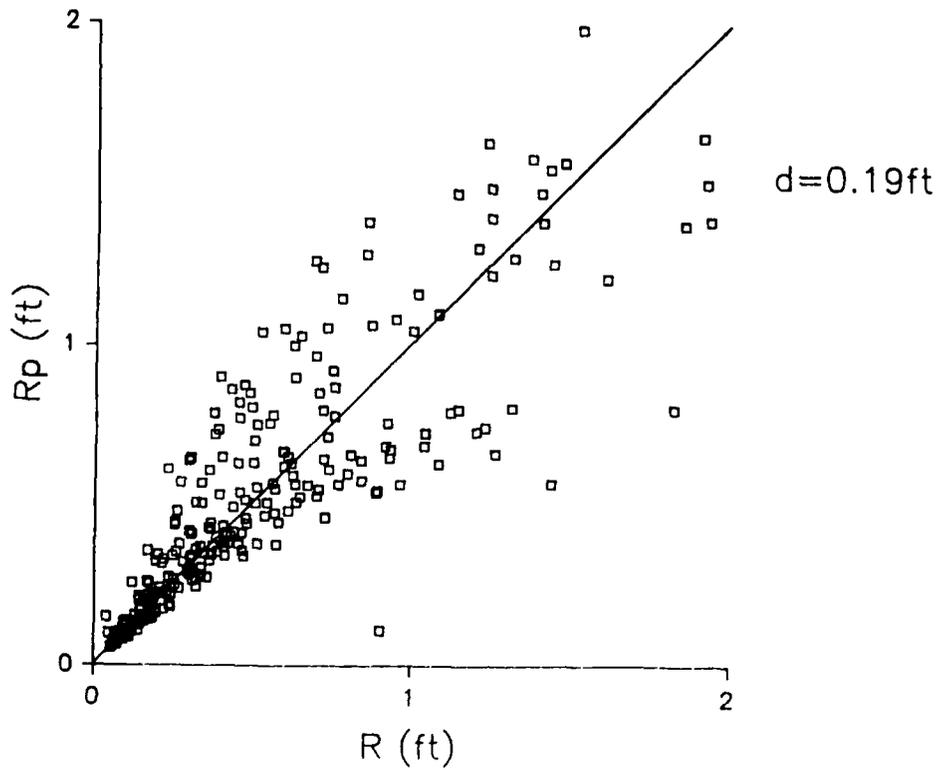


Figure 9. Predicted versus observed runup (Van Dorn method), $d = 0.19$ ft

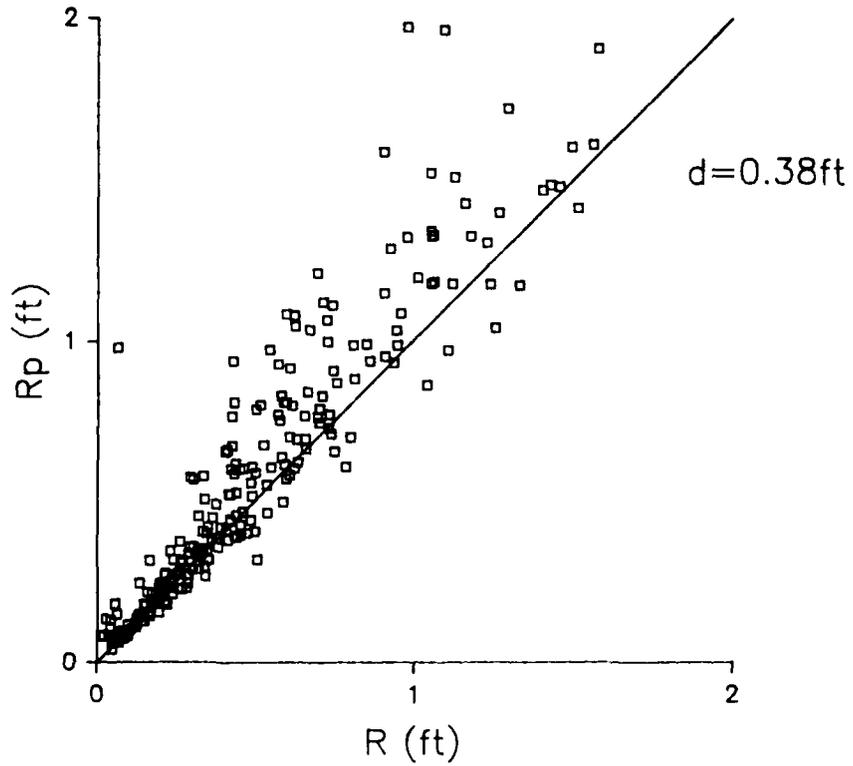


Figure 10. Predicted versus observed runup (Van Dorn method), $d = 0.38$ ft

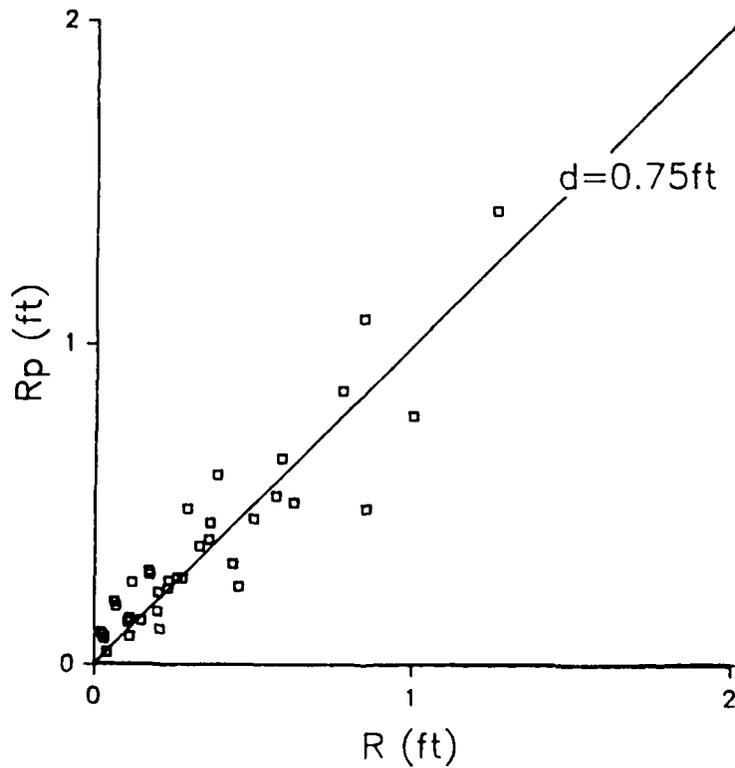


Figure 11. Predicted versus observed runup (Van Dorn method), $d = 0.75$

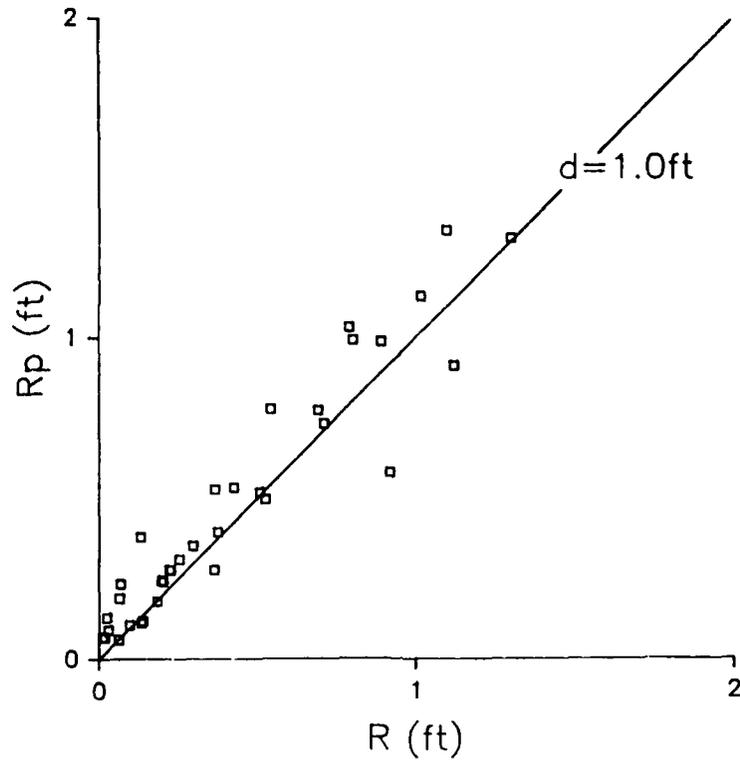


Figure 12. Predicted versus observed runup (Van Dorn method), $d = 1.0$ ft

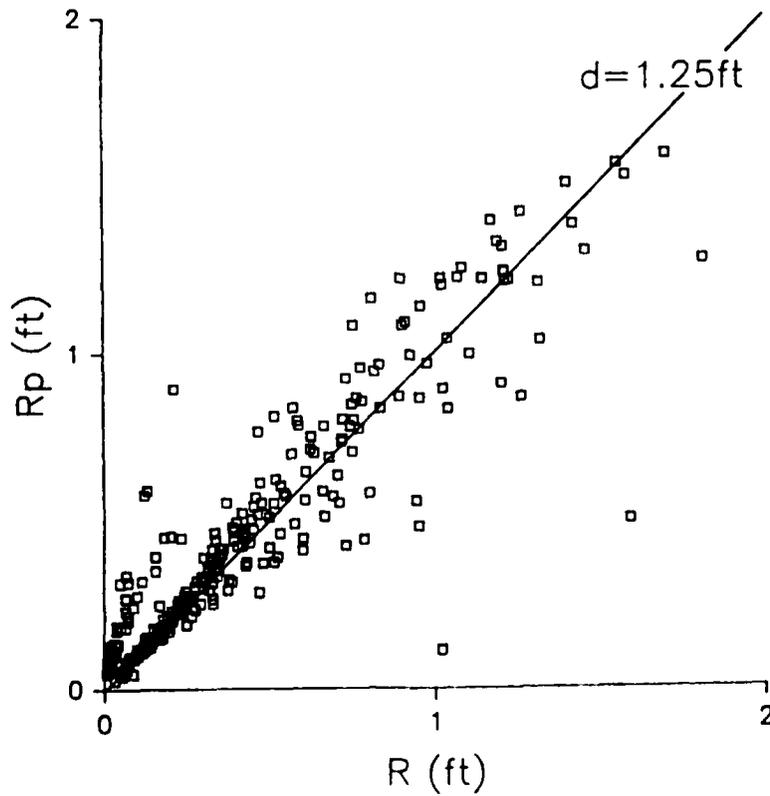


Figure 13. Predicted versus observed runup (Van Dorn method), $d = 1.25$ ft

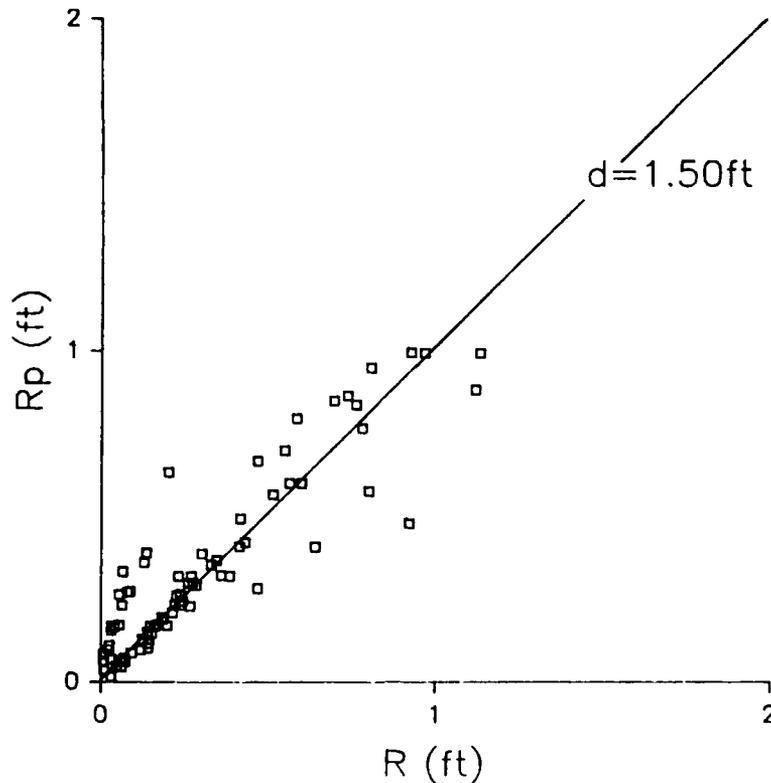


Figure 14. Predicted versus observed runup (Van Dorn method), $d = 1.5 \text{ ft}$

evidence for such an equation through laboratory data, numerous investigators who have shown Hunt's equation to be valid in form suggest a dismissal of Takada's equations from further consideration. Nagai and Takada (1972) show a meaningful refinement of the runup formulae by using a form of the Hunt equation in the breaking wave runup process although they have incorrectly specified the shoaling coefficient dependency. In the nonbreaking wave region, they use a form of relative runup equation contributed by Miche (1944) but again misspecify the shoaling coefficient dependency. Their postulated runup equations are fit to limited data with reasonable success in spite of the minor shoaling coefficient misspecifications. As their formula is similar to the approach used by Van Dorn (1966) and Ahrens,* their expressions will not be discussed further. Takada (1974) addresses the wave runup problem on vertical walls fronted by a 1 on 10 beach slope exclusively. He presents formulae different from previous papers for the case of vertical walls as might be

* Personal Communication, 1987, John P. Ahrens, Oceanographer, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

expected since Hunt's equation will not work on vertical walls because of the slope tangent dependency. His formulae also have provision to account for wave spray effects but do not address model scale, which can be critical for consideration of surface tension effects. As his data are presented in terms of relative runup scaled by deepwater wave height, the runup on the structure cannot be decoupled from the wave transformation that exists for his particular laboratory setup, thus making practical application of his equations impossible for a FEMA depth-limited wave height approach to the runup problem.

Regular Wave Runup on Smooth Composite Slopes

30. There has been very limited research on wave runup on composite slope structures such as structures with convex or concave profiles. Saville (1958) compared laboratory results of wave runup on one type of composite slope structure with an iterative equivalent slope approach to runup calculation. This method is the recommended method in SPM (1984). A good discussion of the method and its limitations is provided in TACPI (1974), which concludes that the equivalent gradient method of Saville may have some merits, although there is uncertainty as to the limits of its validity. The TACPI (1974) also notes that "due to the strongly empirical nature of the method, considerable caution is necessary in applying it to slope shapes which differ clearly from the shapes for which the method has been found to be valid. This is all the more true since the real runup tends to be underestimated rather than overestimated." More recently, Kobayashi and Jacobs (1985) have proposed an alternative empirical method to the calculation of runup on composite slope structures with berms. The methodology is tested on a limited set of laboratory data with results similar to those found by Saville (1958). Although the method appears promising (as does Saville's method), the limited testing provided to date does not provide sufficient confidence for the method to be recommended for general use. There is at present no analytical method based on sound physical principles to compute wave runup on composite slope structures that will provide an answer with an acceptable degree of accuracy for varying structure, wave, and bathymetric conditions. This is especially true in the case of bermed structures where the width of the structure may be on the same order as the shallow-water wavelength. The only present recourse for complex structure shapes is to do laboratory tank testing to find a realistic

answer to wave runup. If the width of the flood-protection structure is short compared with the shallow-water wavelength, it may be a sufficient approximation to consider using a linear slope from the toe of the flood-protection structure to the top of the structure.

Nonsmooth Slope Effects

31. As noted previously, for FEMA purposes a seawall should be defined as a hardened impermeable structure constructed to prevent flooding behind it. Consistent with this philosophy, any credit for runup reduction by roughened structures (as opposed to smooth surface seawalls/dikes) should reflect either theoretical or laboratory findings for impermeable type surfaces or low permeability surfaces such as a hardened impermeable surface covered by a limited surface layer of rock, rubble, or concrete armor shapes. Runup tests on permeable breakwaters where considerable wave energy can be transmitted through the structure as well as dissipated within the structure should not be considered as a valid basis for reducing runup from the smooth slope runup results. As the state of the art in theoretical wave runup prediction does not allow for an approach to the problem of runup reduction for roughened structures, any recommendations for the reduction of runup on roughened structures must be made based on laboratory tests.

32. Due to limited information on runup reduction caused by roughened surfaces, only crude attempts to account for it have been made, primarily by a runup reduction factor r applied to smooth slope runup (TACPI 1974, Stoa 1978a, and SPM 1984). The reduction factor r is a combination of effects caused by both turbulent boundary layer energy dissipation on the face of the structure (because of roughness effects) and turbulent energy dissipation within the structure (because of permeability of the structure). A possible form of the reduction factor might be posed as:

$$r = (\text{roughness factor}) \times (\text{permeability factor}) \quad (18)$$

Existing literature on the subject has not tried to separate these two effects sufficiently for the purposes of engineering prediction of runup. For the purposes previously stated, the present attempt to provide guidance for FEMA

on seawall runup suggests that r be determined based on impermeable or limited permeability structures considering only roughness factor reduction.

33. A considerable amount of information on runup reduction on impermeable structures (as well as permeable structures) has been summarized in TACPI (1974) and Stoa (1978a). For an impermeable structure, the roughness reduction factor r has been noted by TACPI (1974) to be a function of both the form and shape of the roughness elements as well as other factors such as wave steepness, structure slope, and relative roughness factor k_r/H_o , where k_r is a roughness height. Additional unquantified effects expected to be important include Reynolds number and a dimensionless number $R/l\sin(\theta)$ representing the projected number of roughness elements in the flow field (where l = length between roughness elements) for structures where the roughness is not intrinsic but created by the casting of concrete (or other material) projected roughness forms. The most complete set of runup data for understanding the effects of roughness is that of Savage (1957, 1958), which indicates that r decreases with decreasing wave steepness and decreasing slope. Unfortunately, Savage's (1957, 1958) data were limited primarily to sand-covered slopes and therefore have limited usefulness for practical considerations. The TACPI (1974) report includes considerable information on numerous European studies to provide roughness runup reduction coefficients, along with a number of empirical equations for calculating a roughness reduction factor r . Unfortunately, the empirical approach to this problem prevents such equations from being extended beyond their laboratory data base with any confidence. Much of this information has been summarized in three tables in TACPI (1974) with roughness reduction coefficients ranging from 0.5 to 1.0.

34. Stoa (1978a) summarizes the more pertinent laboratory studies for the types of seawalls that are most common along US coastlines. For rubble surface impermeable structures, information on the runup reduction is limited to runup tests on sloping structures (1 on 1.5 to 1 on 2) with armor layers one to three stones thick on a hard impermeable surface. Results of roughness coefficients calculated as a function of slope and dimensionless roughness length = k_r/H_o are presented in a series of graphs with relative roughness varying from 0.58 to 0.75 for Hudson and Jackson (1962), 0.5 to 0.6 for Raichlen and Hammack (1974), 0.60 to 0.62 for Saville (1956), and 0.5 to 0.7 for Ahrens (1975a). Stoa (1978a) summarizes two sets of runup tests of concrete armor units on an impermeable base. Vanoni and Raichlen (1966) tests

suggested runup reduction for tribars underlain by two filter layers of stone to be 0.38 to 0.40, while McCartney and Ahrens (1975) runup tests on Gobi blocks suggested a runup reduction factor of 0.93. For stepped structures, Stoa (1978a) summarized the results of three runup investigations (Saville 1955, Jachowski 1964, Nussbaum and Colley 1971) that provided runup reduction factors ranging from 0.66 to 0.90 depending on relative roughness, step size, relative depth, and slope.

35. Due to a paucity of data on comparisons between runup on smooth slopes and runup on rough slopes under similar wave and bathymetric conditions, information on runup reduction factors is inadequate to provide more than an approximation of the reduction offered by a rough (and possibly limited permeability in the case of stone on a hard surface) surface. Suggested roughness runup reduction factors r for various surfaces are provided in Table 1 and are similar to those in Table 7-2 of SPM (1984) as well as summarized literature from the TACPI (1974) and Stoa (1978a). The primary difference between Table 1 provided here and the tables and information in the above reports is reflected in the use of an r on the high end of the laboratory measurements because of the present lack of confidence in extrapolating such information to field conditions.

Table 1
Suggested Roughness Runup Reduction Factors for Various Surfaces

<u>Surface Characteristic</u>	<u>r</u>
Smooth	1.00
Concrete blocks, Gobi blocks	0.90
Grass	0.90
Quarystone, rubble	0.80
Stepped surface	0.80

Scale effects

36. Literature on scale effects in runup laboratory testing is scarce, and coverage of the subject is limited to very few references (Stoa 1978a, Fuhrboter 1986, Saville* 1987). This subject has been examined by only two

* T. Saville, Jr., "Scale Effect in Wave Runup," unpublished report, American Society of Civil Engineers Convention, Boston, MA.

investigators (Fuhrboter 1986; Saville* 1987) when considering only those investigations that have duplicated small scale runup tests in larger (prototype) laboratory facilities.

37. The only extensive review of scale effects on monochromatic wave runup found is that of Stoa (1978a) who discusses numerous laboratory tests, primarily of a site-specific nature, and the effects of a water wave defined Reynolds number on runup results. Stoa (1978a) also provides a suggested revision of a runup scale correction factor relationship provided in the SPM (1984). This runup scale correction factor originates from Saville* (1987) and was in earlier versions of the SPM as well as the most recent (1984) version and reflects a very limited amount of data on only three structure slopes (1:3, 1:6, and 1:15). As noted by both Stoa (1978a) and Saville (1987), there is also some ambiguity as to whether such scale effects are true scale effects (i.e., due to modeling at different flow Reynolds numbers) or due (at least in part) to different relative roughnesses between the large scale tests and the small scale tests. Stoa's (1978a) scale correction curve based on Saville's* work reflects an attempt to provide for this ambiguity by reducing the scale correction factors (as given by Saville* and included in the present edition of SPM (1984)) to be applied to runup predictions based on small scale tests. An additional problem addressed by Stoa (1978a) is that the Saville* runup scale factor correction curve provides enormously high runup correction factors (approaching infinity) for steep sloped structures. Stoa's (1978a) revised runup scale factor correction curve is (in part) a hypothetical curve in which runup scale correction factor reduces to zero (for vertical walls) as structure slope increases to account for the steep slope problem in Saville's* original curve.

38. A more recent look into scale effects has been provided by Fuhrboter (1986) for one relatively smooth slope (1:4) in which waves are breaking on the slope. Fuhrboter's (1986) conclusions, based on this limited data set, show little scale effect and contradict the findings of Saville* and Stoa (1978a).

39. In the course of the present study, a reanalysis was made of both Fuhrboter's (1986) and Saville's* (1987) data, but due to scatter in both data sets, no clear recommendations can be provided addressing scale factor

* Saville, op. cit.

corrections in runup. The present recommendations provided in SPM (1984) as noted previously are to use the Saville* scale correction factor as a multiplier for the small scale laboratory runup based predictions to obtain a postulated runup for prototype conditions. Although such recommendation provides a conservative approach to coastal design, it is not sufficiently supported in the literature or by other laboratory findings to justify the use of a scale correction factor for FEMA considerations in evaluating runup on existing seawalls. The additional problem provided by the Saville* curves (i.e., the SPM (1984) curves) for the steep slope structures reinforce the previous statement that the scale correction factor not be used in a FEMA-based approach to wave runup. Although the Stoa (1978a) scale correction factor could be used, it is felt that justification for adoption by FEMA of such a curve is lacking from both a theoretical and experimental viewpoint. Until considerable laboratory work on wave runup at both small scales and near prototype scales (of identical structure situations) is completed, it is felt best (for FEMA purposes) to neglect any scale correction factor, realizing that the predicted runup may be on the unconservative side in such an approach. Unfortunately the subject of scale effects in wave runup is an area where research is deficient for providing good guidance for coastal design or evaluation of structures.

Irregular Wave Runup

40. Two different approaches to the problem of computing irregular wave runup have been taken, the first of which is based on physical model testing of site-specific structures, offshore bathymetry, and incident wave conditions typically posed as a site-specific nearshore design wave spectra. Runup probability distributions are then fit to the runup measurement series using a best fit probability distribution found by trial and error. Design runup value used is typically equated to R (2 percent), the value of runup which is exceeded by only 2 percent of the tested runup values. The use of R (2 percent) is totally arbitrary but seems to be the standardized value selected in a number of European countries as well as the United States. Various probability distributions have been used by numerous researchers in this approach. Kamphuis and Mohamed (1978), Ahrens (1979), and Allsop (1983)

* Saville, op. cit.

have all fitted Rayleigh distributions to irregular wave runup measurements on smooth slopes and concluded that while the Rayleigh distribution fits the data very well over most of the measurements, it tends to underpredict the extreme runup levels to varying degrees. Ahrens (1979, 1983) has tried the Gamma distribution and the two-parameter Weibull distribution as well as the Rayleigh distribution. Two points should be noted in any probability distribution approach:

- a. Probability distributions with higher numbers of parameters will typically fit measurements better than one parameter distributions such as the Rayleigh distribution because probability distribution fitting is just another form of curve fitting exercise in which better fits are provided by increasing the number of parameters (degrees of freedom) of the curve and hence its flexibility to adapt to the measured values.
- b. A probability distribution found to be a good fit for one site does not guarantee an underlying universal fit of the same distribution to other sites.

As most probability distribution fits are not based on physics of the problem but simply curve fitting exercises for a site-specific problem, a cautious approach must be used in attempting to extrapolate random wave runup probability distribution information to another site with differing conditions.

Researchers, notably Van Oorschot and d'Angremond (1968) and Gunbak (1979), have attempted alternative approaches to the probability distribution fitting by the prediction of R (2 percent) (or R (p percent)) based on gross parameters of the incident wave spectra such as peak wave period T_p , significant wave height H_s , and spectral width parameter ϵ . As an example of this approach for breaking waves on slopes, Van Oorschot and d'Angremond (1968) suggested a modified version of the Hunt equation for the runup value exceeded by 2 percent of the measured values as follows:

$$R(2\%) = C(2\%) I_{rr} \quad (19)$$

where

C = dimensionless coefficient

I_{rr} = random Iribarren number

where

$$I_{rr} = \frac{\tan(\theta)}{\left(\frac{2\pi H_s}{g T_p^2}\right)^{1/2}} \quad (20)$$

The coefficient C (2 percent) is then defined as a function of the spectral width parameter ϵ and found through laboratory tests. An important point to note in this approach is that wave conditions at a coastal structure may be modified considerably by nearshore bathymetry, which can radically alter the measured runup. Attempts of defining irregular wave characteristics in such a lumped parameter type method must do so via wave spectra parameters at the structure itself and not at the wave generator if such data are to be extrapolated to other sites (which must then have similar nearshore wave characteristics and spectra).

41. A second approach to the prediction of irregular wave runup is by means of probability distribution transformation using the "hypothesis of equivalency" (Saville 1962) coupled with a physical law valid for periodic wave runup and a knowledge of the incident wave probability distribution. The method consists of first defining the incident wave probability distribution (which should be a joint probability distribution if the runup process is wave height and wave period dependent). This distribution is then transformed to a new distribution (analytically or numerically) via use of the known physical law governing the runup process or using physical model runup measurement results of periodic wave runup for given wave heights and periods. This approach is based on a "hypothesis of equivalency" which states that each wave can be treated as an independent entity that will create a runup according to the assumed known physical law for runup used (see Battjes (1974b)). This approach has been used with limited success in describing irregular wave runup by Saville (1962) and Battjes (1974b). A particular problem with this approach is that when the runup process follows two different laws (i.e., breaking and nonbreaking), an analytical solution cannot be obtained. The approach can still be used numerically, although such a technique has not yet been tested on data sets that cover both the breaking and nonbreaking wave runup domains.

42. Additional investigations into irregular wave runup not discussed in the above paragraphs include Webber and Bullock (1968, 1970), Whalin (1971), Hashimoto (1974), Erchinger (1976) (field measurements), Burcharth (1979), Owen (1980, 1982a, 1982b), and Grune (1982) (field measurements).

43. Particular problems with an irregular wave runup approach for use by FEMA (beyond those already discussed) are noted as follows: (a) There is no well-defined upper cutoff of runup that would provide a suitable answer as

to whether flooding occurs behind a coastal flood-protection structure; (b) an irregular wave runup approach would be inconsistent with present FEMA depth-limited breaking wave and wave transformation methodology as defined in FEMA (1981); and (c) critical incident wave spectra are known in very few coastal locations, and knowledge on shallow-water wave transformation from gage sites to structure locations is beyond the present state of the art (due to shallow-water nonlinearity) to be applied with any level of confidence.

Runup Calculation Conclusions

44. For the previously stated reasons, the present conclusion is that a monochromatic wave runup approach is best suited for FEMA needs in evaluating the flood protection offered by a given coastal flood-protection structure. The two most promising state-of-the-art runup predictive equations for smooth linear slope structures are due to Van Dorn (1966) and Ahrens.* These approaches are not capable of predicting periodic wave runup on smooth slope structures with a high degree of confidence, but appear better than alternative approaches. A partial reason for this inability to accurately predict runup appears to be a poor ability to predict runup in the transition and nonbreaking wave region. Both methods use Hunt's (1959) equation for the breaking waves, which limits treatment of steep slope structures to a non-breaking wave approach.

45. An alternative robust method of predicting maximum wave runup on any smooth linear sloped structure is proposed for FEMA considerations in Appendix A. This alternative method unifies various approaches and overcomes the difficulty of applying Hunt's equation on steep slope structures. This proposed method is verified with the same smooth slope laboratory data of Savage (1957, 1958) and Saville (1956) that was used in the Van Dorn (1966) and Ahrens* approaches. The method consists of a modified Hunt equation in the region of breaking/broken waves and an upper limit to the runup process that will be suitable for state-of-the-art computation of monochromatic wave runup in situations of nonbreaking waves on smooth linear slopes. Further refinements on runup methodology for smooth linear slopes must await good

* Personal Communication, 1987, John P. Ahrens, Oceanographer, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

generic data sets on periodic wave runup and improved understanding of laboratory scale effects.

46. Corrections to the smooth slope data should be made via a "roughness factor" as suggested in Table 1 applied to the smooth slope answer. The inherent complexities of rough turbulent flow make an analytical solution to this problem impossible, and state-of-the-art methods for prediction of runup on rough slopes will remain heavily dependent on laboratory measurements. It is recommended that no scale correction adjustment be used because of the paucity of data on laboratory scale effects and conflicting results among such sets. No guidance can be provided on composite slope structure runup as state-of-the-art methodology concerning runup on such structures has not provided complete guidance or consistent trends. Here again, state-of-the-art methodology dictates that runup prediction must rely heavily on laboratory test results.

47. A state-of-the-art method to runup calculation for coastal flood-control structures dictates a two-step approach. The first step is to calculate the wave transformation and runup on the natural coastal profile as if no coastal structure was present. This can be done using existing FEMA methodology. The second step is to calculate the runup on the coastal flood-control structure assuming the wave conditions at the toe of the structure are the same as those without the structure present (i.e., no wave-structure interaction). Although it is apparent that the true situation is far more complex than this (because of wave/structure interaction), present state-of-the-art coastal engineering does not provide a tested methodology which can address this complex situation.

Recommendations

48. It is recommended that FEMA use a monochromatic wave runup criterion to assess whether or not flooding occurs behind a coastal flood-protection structure. Due to the present state of the art in runup prediction and the need for FEMA to use a method consistent with depth-limited breaking wave criteria, it is recommended that a monochromatic wave runup approach be used.

49. Numerous empirical criteria exist for determining wave runup on structures. Most of these criteria are not reasonable for FEMA use because of

the lack of underlying physics or wide applicability to various situations. An approach to runup prediction on smooth sloped structures consistent with FEMA criteria of depth-limited wave heights is discussed in Appendix A and shown to agree well with existing theory and laboratory data. The runup prediction for smooth slopes is given as follows:

$$\frac{R}{H} = \frac{\sin(\theta)}{\sqrt{\frac{H}{L_0}}} \text{ for } \frac{\sin(\theta)}{\sqrt{\frac{H}{L_0}}} \leq \sqrt{2\pi} \sqrt[4]{\frac{\pi}{2\theta}} \quad (21)$$

and

$$\frac{R}{H} = \sqrt{2\pi} \sqrt[4]{\frac{\pi}{2\theta}} \text{ for } \frac{\sin \theta}{\sqrt{\frac{H}{L_0}}} > \sqrt{2\pi} \sqrt[4]{\frac{\pi}{2\theta}} \quad (22)$$

where H = wave height = $0.78d$ at the toe of the structure. The runup should be modified by a factor r provided in Table 1 for rough slopes. Evidence is insufficient at present to determine scale effects of various size laboratory tests; hence, it is recommended that scale effects be ignored, recognizing that an unconservative answer will result. The runup calculation on the structure can be made after calculation of runup and corresponding water depth on existing bathymetry via present FEMA techniques assuming the structure does not exist.

50. As present state-of-the-art methodology for runup prediction on structures is extremely crude, data from applicants who provide runup prediction based on physical model studies of their structures under critical scenario conditions should be used as a possible alternative to the proposed methodology.

PART III: OVERTOPPING OF STRUCTURES

Background

51. For the purpose of calculating the flood-prevention potential offered by a coastal flood-protection structure, a predictive calculation of the overtopping rate under a given set of wave conditions is desirable. Overtopping prediction can be considered a first step in the calculation of wave transmission over a coastal flood-protection structure necessary to delineate the type of hazard zone (V or A zone) for FEMA insurance rate maps. Research on the prediction of wave overtopping has proceeded primarily in a series of increments based on site-specific laboratory studies. There has never been a comprehensive generic (one laboratory) set of overtopping data for a variety of offshore bathymetry, seawall/revetment configurations and slopes, and wave conditions. Such a data set would be most desirable to better understand the physics of overtopping. Most existing laboratory data on overtopping consist of smooth sloped structures for a few structure configurations. Very few field measurements (primarily Japanese) exist on overtopping. The most comprehensive set of laboratory data on overtopping taken to date is the data set of Owen (1980, 1982a), which was for a few specific types of seawalls or sea-dikes, offshore bathymetry, and irregular wave conditions, primarily of relevance to the English coastline. The model scale for these tests was 1:25.

52. The ideal overtopping prediction equation consistent with a FEMA-adopted approach of a depth-limited wave height criterion would have the same five properties discussed earlier in the section on runup. It will be shown that none of the existing methods for calculation of overtopping are sufficient to adequately address these desirable qualities of a predictive overtopping method.

53. The primary countries that have studied overtopping are those which (a) have a coastal flooding problem, (b) have expensive coastal development that cannot (due to lack of land) or will not (due to desirability of living on the coast) relocate inland, and (c) have the technology and resources to provide expensive coastal protection. These are the countries of northern Europe, the United States, and Japan. Other countries have also pursued research and development in this area, but research reviews of the overtopping subject failed to turn up useful material. A short review of the present

state of the art for preliminary design in each of these countries and methodology for calculating overtopping is discussed in the following paragraphs along with a critical appraisal of problems encountered when trying to extend past overtopping studies to a methodology for calculating overtopping for establishment of flood insurance credit under FEMA guidelines. It should be recognized that none of these countries appear to use overtopping calculation for final design but rather do extensive site-specific laboratory testing to determine final design overtopping quantities. This reflects a lack of confidence in applying any general approach for overtopping calculation to a specific site.

54. In the United Kingdom, the present state of the art in overtopping (for design) is reflected in publications by Owen (1980, 1982a). An equation of the following type is used by Owen for calculation of overtopping:

$$q^* = A \exp (-B F^*) \quad (23)$$

where

$$q^* = q / (T_z g H)$$

q = average rate of overtopping in volume rate per unit length of structure

T_z = zero crossing wave period

g = acceleration of gravity

A, B = coefficients found by laboratory testing

$$F^* = F / [T_z (g H_s)^{1/2}]$$

F = freeboard, i.e., crest height of structure above still-water level

H_s = significant wave height

55. In an extensive testing program by Owen, A, B coefficients were fit to laboratory data for a few types of seawalls of a design typical on the UK coastline. Testing was usually done with irregular waves with a JONSWAP type spectrum. A method of correction for roughened seawall slopes was also suggested within these studies.

56. Particular problems exist in using this methodology with either (a) a coastal structure having different geometry and nearshore bathymetry than the tested structure or (b) wave conditions different from the assumed spectrum used in the tests. To calculate overtopping for a structure different from the one tested, the coefficients A and B must be interpolated,

which is difficult at best since two coefficients should be estimated simultaneously in a nonlinear equation of the form used. The method used by Owen allows each coefficient to be interpolated separately, but this approach does not guarantee results consistent with the original data. An additional problem is that the spectrum tested is not presented at the base of the structure but rather at the wave generator prior to possible complex wave transformation in the nearshore zone. Additional problems with adopting Owen's method to a FEMA approach are (a) using irregular wave data is not consistent with past FEMA criteria for a limiting monochromatic wave at the coast under a given 100-year water level (i.e., $H = 0.78d$ where d is the 100-year return period water depth at the coast), (b) the method of presenting the overtopping as an exponential decreasing function of dimensionless freeboard suggests that there will always be some water over the wall no matter what the structure crest height, and (c) the bathymetry modeled is not generic to all sites; therefore wave transformation and hence overtopping might be very different for sites with similar structures but different nearshore bathymetry. The third problem mentioned is common to all laboratory tests in which the waves are measured prior to transformation by the nearshore bathymetry. Although an irregular wave approach may be a more realistic method provided the true spectrum is known, such a method provides considerable problems to a methodology that would be consistent with what FEMA has established for the depth-limited breaking wave height approach. The additional problems caused by the inability to accurately extrapolate coefficients A, B for seawalls/coastal flooding structures not tested and the uncertainty in the wave transformation process make this method undesirable for FEMA needs.

57. Recently, laboratory tests of irregular wave overtopping of various seawalls and seawall/revetment configurations have been conducted at the CERC, US Army Engineer Waterways Experiment Station (WES) (Ahrens and Heimbaugh 1986). Although the geometry of structures and the nearshore bathymetry tested at CERC are different from those tested by Owen (1982a) at Wallingford, the approach to predicting overtopping rates is quite similar. The method of Ahrens and Heimbaugh uses wave conditions at the toe of the structure to estimate wave overtopping rates; otherwise the critique applied to Owen's approach can be applied to this method.

58. A method used in Japan for preliminary estimation of wave overtopping at coastal structures is presented in Goda (1970, 1985), Goda, Kishira,

and Kamiyama (1975) and Goda and Kishira (1976). This method appears to have evolved from earlier work by Tsuruta and Goda (1968). The method is summarized in a series of dimensionless overtopping graphs with the following functional forms:

$$\frac{q}{\sqrt{2gH'_0}} H'_0 = f \left(\frac{F}{H'_0}, \frac{d}{H'_0} \right) \quad (24)$$

where

- q = overtopping rate
- g = acceleration of gravity
- H'_0 = refracted deepwater wave height
- F = height of seawall crest above still-water level (freeboard)
- d = water depth at toe of structure

59. The provided charts are again for very limited cases of vertical seawalls with and without protective armor blocks in front of the wall. Goda (1985) has noted that the design charts are based on irregular wave tests; this observation does not appear to be consistent with the use of the monochromatic wave parameter H'_0 used in the graphs. Original data are not shown on the graphs of Goda but only lines showing expected data trends. Tsuruta and Goda (1968) had previously provided similar graphs based on limited regular wave tests (which show data points) and graphs for irregular waves based on analysis of regular wave overtopping combined with a probability transformation to obtain an expected value of overtopping. Tsuruta and Goda (1968) also provide some results of laboratory measurements on irregular wave overtopping tests using a spectral wave generator with an input of 10 sinusoidal components. Goda (1971) states that regular wave data can be used to estimate irregular wave overtopping by a direct probability transformation approach with reasonable results; it is noted that some of his earlier design charts for irregular waves are actually derived from regular wave data with an assumption of Rayleigh distributed wave heights. Goda states that scatter does exist in laboratory data for irregular waves and further postulates taking such scatter into account in design as it is also evidenced in limited field data. Such scatter might be expected in irregular waves because of the groupiness effect (i.e., frequency modulation) and the inherent difficulty in measuring overtopping and wave conditions in the field.

60. As noted, Goda's method appears to be a suggested method of calculating preliminary estimates of overtopping in Japan where vertical wall monolithic seawalls are commonplace. His design charts are provided in the Japanese Port Design Manual. Unfortunately his data are limited to specific types of seawall design (i.e., vertical walls with and without armor block protection) and to offshore bathymetry typical of the Japanese coastline. The postulated methodology does not clarify how the transition from irregular wave data to design is made, although Goda (1985) presents example use of the graphs for what appear to be monochromatic wave input. This weak point along with the fact that irregular waves are not consistent with the present FEMA approach of depth-limited wave height criteria make the Goda method of limited use for FEMA purposes. The earlier regular wave data overtopping charts by Tsuruta and Goda (1968) consist of a limited data base and thus are also of limited value. One additional problem with the Goda method is that the incident wave heights are formulated in terms of offshore conditions; thus the additional complex transition of waves from offshore to the structure site are implicitly contained in the method. This is undesirable for FEMA methodology where the wave conditions at the coast are provided via the FEMA limiting depth breaking wave height $H = 0.78d$.

61. Another Japanese method proposed by Kikkawa, Shi-igai, and Kono (1968) is based on an extension of a steady-state weir flow equation of the form:

$$q = 0.667 m F (2gF)^{1/2} \quad (25)$$

where

m = discharge coefficient (assumed = 0.5)

F = freeboard

62. By extending the method to the dynamic (unsteady) case and assuming a triangular wave form, a solution was proposed of the form

$$q = H_o (2gH_o)^{1/2} \left(\frac{2}{15}\right) mk^{3/2} \left(1 - \frac{F}{kH_o}\right)^{5/2} \quad (26)$$

where k is a dimensionless coefficient fit to data.

63. To obtain the coefficient k , a very limited set of data is used consisting of a few Japanese monochromatic tests on vertical and sloping walls, some of the Beach Erosion Board (BEB)/WES (BEB 1956) monochromatic overtopping data, and the wind wave flume overtopping data of Sibul and Tickner (1956). Although the approach appears to be a promising one, problems for its adoption to FEMA are that input wave conditions are again in deep water and involve wave transformation uncertainty and the postulated nonlinear form of equation makes the empirical constant difficult to fit and difficult to interpolate outside the boundaries of the fitted data. As the basic theoretical approach appears more sound than alternative methods, this method was modified somewhat and was investigated with the entire BEB/WES (BEB 1956) data set. Although results of this approach appear promising in some data cases, the poor fit of the method to other data cases suggests that successful use of the methodology must await more high quality overtopping data for a sufficient number of coastal flood-protection structures. A similar method is discussed further in the section on wave transmission.

64. The calculation method used for preliminary design wave overtopping calculations in the United States is that given in the SPM (1984). This method is originally due to Weggel (1976) and is based on the equation:

$$q = \left(g Q_0^* H_3' \right)^{1/2} \exp \left[- \left(\frac{0.217}{\alpha} \right) \tanh^{-1} \left(\frac{F}{R} \right) \right] \quad (27)$$

where

q = overtopping rate in volume per unit time (per unit width of structure)

Q_0^* = fitted empirical coefficient

α = fitted empirical coefficient

F = freeboard (structure crest height above still-water level)

R = wave runup defined in SPM curves

Although this method is essentially a monochromatic wave approach to calculating wave overtopping, it relies on a semiempirical nonlinear equation in which two coefficients must be fitted, thus making an interpolation between coefficients very difficult for conditions not covered by the limited data base. Weggel (1976) fit the parameters by eye rather than by using a statistically based parameter estimation technique. An additional limitation in the approach was the use of measured runup data to fit empirical coefficients of

the equation; this limitation provides additional uncertainty in estimating the overtopping, i.e., that of estimating the runup. This method also uses deepwater wave input, therefore involving wave transformation uncertainty.

65. In view of the particular limitations in this approach, a test was made to assess the ability of calculating q with original data sets. Figures 15 and 16 show measured q versus calculated q for BEB/WES (BEB 1956) data (the data used to formulate the method) for two slopes. The calculations were made by utilizing the SPM curves for runup without scale correction. As seen in these figures, the method does not provide a highly accurate calculation of overtopping for the data used in its calibration. This problem is most probably due to the use of an estimated runup from the SPM runup curves. These problems make the Weggel/SPM method for calculation of overtopping undesirable from a FEMA standpoint where the methodology would have to be extended to a variety of structure types for which there will be no data.

66. In an earlier US study, Cross and Sollitt (1971) used laboratory data from an extensive data set on one structure slope to formulate a semiempirical approach to predicting structure overtopping and consequent wave transmission behind the structure. To evaluate the overtopping, the runup profile is considered to be parabolic in form and the total runup to be as provided by Saville's (1956) data. The overtopping amount is essentially that portion of the wave profile above the structure crest assuming the runup is the potential runup that would have occurred if the structure slope extended to the runup limit. As the primary objective of the study was to evaluate wave transmission, no information on ability of the method to predict wave overtopping was provided. Further discussion on this method is provided in the section on wave transmission.

67. Dutch methods for calculation of wave overtopping are limited to a narrow range of tests with slopes typical of dikes built in The Netherlands. Pappe (1960) presented a series of tests for the overtopping of dikes with plane smooth slopes and a horizontal foreshore. These tests, which consisted of slopes varying from 1 on 2 to 1 on 8, have been resummarized in TACPI (1974) and provide a measure of irregular wave overtopping in dimensionless form as follows:

$$\frac{2\pi q \hat{T}}{H_{50} \bar{L}} = f \left(\frac{\cot^{3/2} \theta}{H_{50}} \right) \quad (28)$$

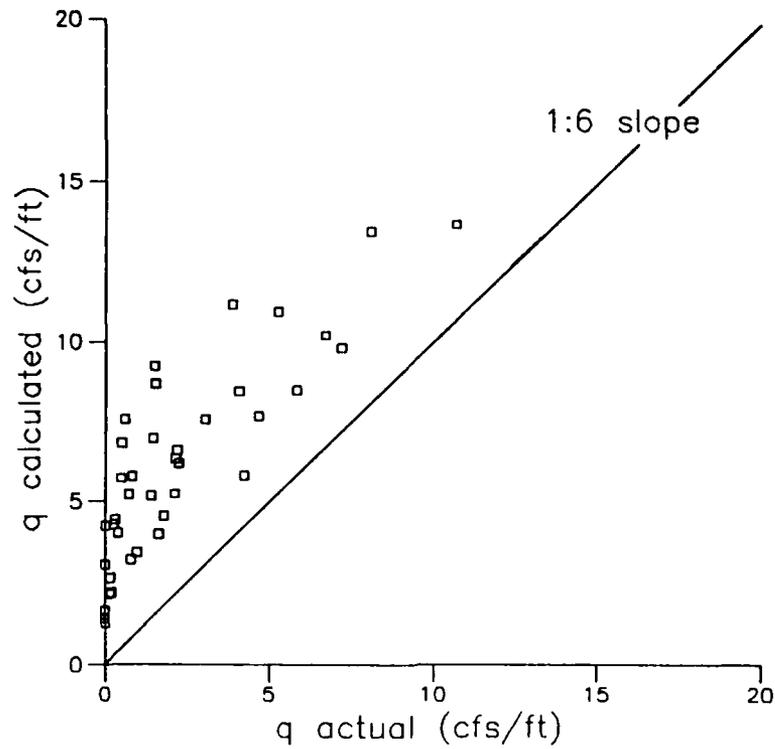


Figure 15. Predicted versus actual overtopping (Weggel method), 1:6 slope

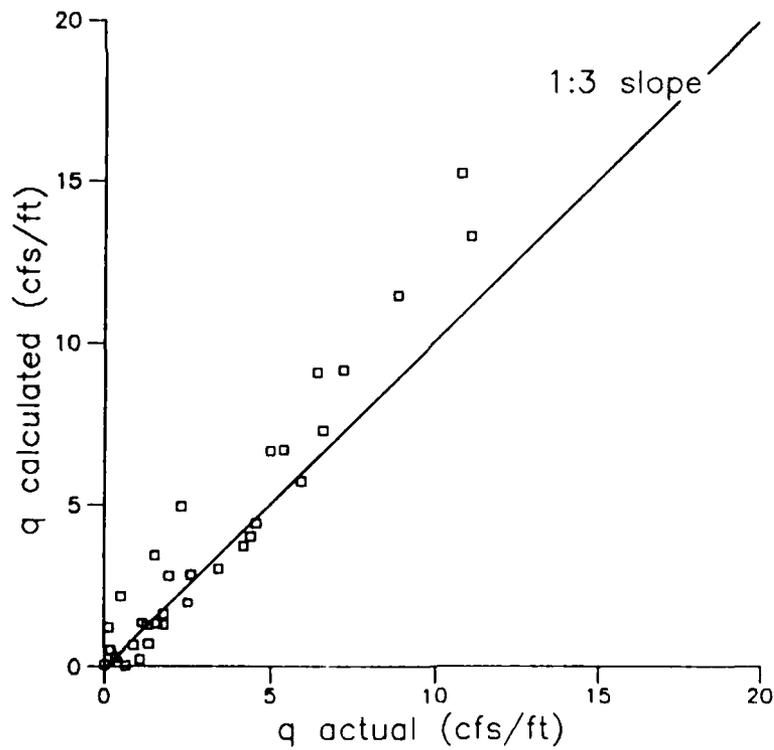


Figure 16. Predicted versus actual overtopping (Weggel method), 1:3 slope

where

- \hat{T} = average period
- H_{50} = median wave height
- \tilde{L} = wavelength calculated via linear theory using \hat{T}
- θ = slope of structure

Particular problems with this set of tests are that the tests were done in a wind wave facility prior to more modern laboratory spectral wave generators and no control was exerted on the wave spectra so produced. Again the approach is of an irregular wave type and has the further complication that the spectra generated may not resemble true wave spectra. The functional form is of an exponentially decreasing type and thus does not show a cutoff (i.e., no clear crest height of structure at which zero overtopping occurs) desirable for a FEMA methodology. Considerable scatter was also noted in the data similar to other irregular wave data overtopping. This may be a result of testing of too short a sequence of waves (i.e., groupiness effects).

68. Battjes (1974b) proposed a semiempirical equation for calculation of overtopping based on a limited set of monochromatic smooth linear slope data taken at Delft and on the similar smooth slope BEB/WES (BEB 1956) data. The equation proposed is of the form:

$$q T = H L_o (\tan\theta) A \left(1 - \frac{F}{R_h}\right)^2 \quad (29)$$

where

- T = wave period
- A = empirically fit constant (= 0.1)
- $R_h = \tan\theta (H/L_o)^{-1/2}$ = wave runup based on Hunt's (1959) runup expression for breaking waves

69. The equation was fit to smooth linear slope monochromatic breaking wave data with slopes ranging from 1 on 3 to 1 on 7, and an empirical coefficient A was recommended. This equation appears to be reasonable within the limited slope range of data considered. Again its weak point is the limited data set used to fit the empirical constant A . Battjes (1974b) proposes a methodology for the calculation of irregular wave overtopping based on this approach. The Battjes approach is somewhat similar to the earlier Japanese method of Kikkawa, Shi-igai, and Kono (1968) and could be used in the

development of a FEMA methodology but has not been tested for steep slopes. As the equation is less well founded in fundamental physics than that of Kikkawa, Shi-igai, and Kono (1968), it was felt that a modification of the Japanese equation held more promise for use in overtopping calculation.

Overtopping Calculation Conclusions

70. None of the existing methods of calculating overtopping fulfill the previously mentioned requirements of an ideal overtopping methodology for FEMA adoption. The UK method (Owen (1980, 1982a)), the US method of Ahrens and Heimbaugh (1986), the Dutch method of Pappe (1960), and the Japanese method of Goda (1985) use irregular wave input. In addition to specific approach dependent problems, the use of irregular waves is not consistent with FEMA adopted criteria of depth-limited wave at the coastline, therefore these methods will not be discussed further.

71. In the equation dependent regular wave methods discussed, the overtopping calculation was dependent on empirically fit constant(s). In many of the approaches, these constants were fit by eye to a nonlinear expression for q rather than using a nonlinear least squares parameter fitting technique. Very limited data exist for fitting the overtopping equations as can be noted by the fact that all of the monochromatic wave overtopping equations use the same set of data in their formulation (i.e., the BEB/WES (BEB 1956) data).

72. The actual design method of a coastal flood-protection structure in all countries relies on either monochromatic or irregular wave tank testing with wave transformation accomplished by physical modeling of site bottom bathymetry. No designs use the crude overtopping prediction equations given here, but rather site-specific testing is required for the given bottom bathymetry and the particular structure configuration used. The reasons for this are as follows: (a) overtopping is extremely sensitive to structure geometry, water levels, and bottom topography; (b) nearshore transformation of waves over complex topography and consequent breaking wave transformation is not within the state of the art to solve either numerically or analytically; and (c) no extensive set of overtopping data has ever been made for the numerous types of flood-protection structures that exist. As a result, the types of calculations made for overtopping are only of a preliminary design nature.

73. Until such time as an extensive generic set of monochromatic overtopping data becomes available, it is recommended that the FEMA criteria for determining flooding behind a seawall/coastal flood structure be determined based on runup. Runup is still a complex process and a first-step calculation necessary in the prediction of overtopping. More data exist on runup than on overtopping and as has been shown, although predictive equations for runup are crude, they are inherently better and exist for a wider variety of cases than those for overtopping. This is a logical conclusion as runup is a necessary ingredient (explicit or implicit) in a good overtopping equation.

Recommendation

74. Present state of the art in wave overtopping prediction is not sufficient to provide calculation of overtopping flows with any confidence using existing theoretical or empirical equations except in a very few site-specific structure and bathymetry cases. It is recommended that FEMA use a runup criteria to determine whether or not flooding occurs behind a coastal flood-protection structure (see Part II), therefore negating the need to calculate overtopping in the flooding determination process.

PART IV: WAVE TRANSMISSION OVER COASTAL STRUCTURES

Background

75. During severe storms, water elevations are increased and coastal flood-protection structures such as revetments and seawalls may be overtopped by high waves. These waves will reform landward of the structures and, depending on their height, have the potential of causing substantial damage to upland structures including dwellings. In order for FEMA to determine the effectiveness of various structures in providing upland damage reduction, it is necessary to know the heights of the transmitted waves. The process of wave regeneration behind the coastal flood-protection structure is complicated because of effects of wave nonlinearities, reflection, energy dissipation, wind, etc.

76. Limited research has been accomplished on this subject. As noted in the preceding sections of this report, design of an actual coastal flood-protective structure is accomplished through laboratory tank testing. The reason for this situation is twofold: first, the problem is very complex; and, secondly, research has not generated sufficient data to test or fit potential generic theoretical solutions and/or semiempirical methods.

77. The ideal wave transmission prediction equation consistent with a FEMA-adopted approach of a depth-limited breaking wave height criterion would have the same properties as those discussed in the preceding sections on runup and overtopping. Also, as in the overtopping case, none of the existing methods for the calculation of wave transmission by overtopping are sufficient in and of themselves to address the desirable qualities of a generic predictive overtopping method that can be applied uniformly to all scenarios.

78. A very limited number of studies have been carried out in an attempt to develop an improved understanding of wave generation by overtopping of coastal structures. Most of these studies have been in the form of fitting empirical curves and/or relationships to laboratory data for various types of coastal structures. A difficulty in assessing the relative validity of various methods is that no consistent objective "goodness of fit" has been proposed and implemented. A brief review of a number of these studies is presented here.

79. Cross and Sollitt (1971) have used the data set of Lamarre (1967) consisting of numerous wave scenarios and one structure slope (1 on 1.5). They developed a semianalytical approach to predicting transmitted wave heights due to structure overtopping. The method considers, as a source of energy for the generated waves, the potential energy of the runup that would have occurred if the structure had been sufficiently high to prevent overtopping. To evaluate the potential energy of the runup above the structure crest, the runup profile is considered to be parabolic in form. The method evaluates the conversion of this potential energy to that of the transmitted wave. A total of five unknowns are involved with four equations. The necessary fifth source of information is based on wave runup data by Saville (1956). In addition, two empirical coefficients are required, one relating to energy losses as the water descends on the downwave side of the structure and the second, an intrinsic reflection coefficient.

80. The comparison presented by Cross and Sollitt appears reasonable for the one structure slope investigated, although for low breakwaters, their theory underestimated the transmission coefficient and for high breakwaters, the transmission coefficient was overestimated. No quantification of the agreement of their method with the data was presented.

81. Seelig (1980) conducted tests of wave transmission over and through structures of various cross sections (17 in total) and permeabilities. Regular and irregular waves were included in the tests. An empirical procedure was developed for estimating wave heights due to overtopping of permeable and impermeable structures. For FEMA purposes, only the results of the impermeable structures are of concern.

82. The method first estimates the wave runup on smooth impermeable slopes for no overtopping. The equation used involves three empirical coefficients which depend on structure slope. A different equation is employed for rough slopes. Remaining equations involve the ratio of structure crest width to structure total vertical height and the ratio of structure freeboard (defined as crest height of structure above still-water level), F , to maximum potential runup at zero crest phase angle R (0 deg). This ratio is defined as the "relative freeboard." The transmission coefficient was found to be related to the ratio of structure freeboard to wave height in a reasonably linear manner. One disadvantage of using the relative freeboard as the only independent variable is that overtopping is a nonlinear process and the

transmission coefficient increases with increasing wave height. The recommended expression for predicting the wave transmission coefficient, κ_T , by overtopping in this method is

$$\kappa_T = C \left[1 - \frac{F}{R(0^\circ)} \right] \quad (30)$$

in which κ_T is the ratio of transmitted to incident wave heights (H_t/H_i) and C is a coefficient that depends on the ratio of structure crest width, B_c , to total structure height, h_s , as

$$C = 0.51 - \frac{0.11 B_c}{h_s} \quad (31)$$

Comparisons presented by Seelig appear favorable; however, in some cases it is found necessary to use different C values for positive and negative freeboards.

83. Goda (1969) carried out a reanalysis of existing overtopping data for vertical walled coastal flood-protection structures and has developed the following empirical equation for the transmission coefficient, where α and β are empirical coefficients.

$$\kappa_T = 0.5 \left\{ 1 - \sin \frac{\pi}{2\alpha} \left[\frac{R(0^\circ)}{H_i} + \beta \right] \right\}, \quad \beta - \alpha \leq \frac{R(0^\circ)}{H_i} \leq \alpha - \beta \quad (32)$$

where

$$\alpha = 2.0 \text{ and } \beta = \begin{cases} 0.1 & \text{for high mound breakwaters} \\ 0.3 & \text{for medium mound breakwaters} \\ 0.5 & \text{for low mound breakwaters} \end{cases}$$

84. Goda found that the amount of wave energy dissipated by a vertical seawall was between 30 to 35 percent with the maximum occurring when the crest was slightly submerged.

85. Equation 3 was found to bracket the data for seawall structures with the following pairs of α and β ,

$$\left. \begin{array}{l} \alpha = 2.2 \\ \beta = 0.8 \end{array} \right\} \text{ lower limit of } \kappa_T$$

$$\begin{aligned} \alpha &= 2.2 \\ \beta &= 0 \end{aligned} \quad \text{upper limit of } \kappa_T$$

The general fit of Goda's empirical relationship to the data is encouraging, although these empirical constants cannot be extrapolated to any other type of structure with any confidence.

86. Cox and Machemehl (1986) have proposed an analytical procedure for calculating the wave height distribution with distance after the wave propagates onto a horizontal bed that is initially dry. The method would be applicable to a structure cross section, such as a gravel island, which does not have standing water behind the structure.

87. It is assumed that the initial wave height at the top of the structure equals the excess maximum potential runup height $[R(0^\circ) - F]$ at the wave crest, $\theta = 0^\circ$, and that the effective water depth is 10 percent of the local wave height. The energy losses are considered to be the result of bore dissipation as originally proposed by Le Méhauté (1963). The resulting wave height distribution with distance, x , is given by

$$H(x) = \left[\sqrt{R(0^\circ) - F} - \frac{5x}{T\sqrt{g}} \right]^2 \quad (33)$$

where T is the wave period and g is gravity.

88. One problem in Equation 33 is that the wave height does not approach a stable limit but in fact, strictly interpreted, decreases to zero and then increases with additional distance. Of course the intent is not to use the equation beyond the location of zero wave height. No example calculations nor comparisons with data were provided. Because treatment by Cox and Machemehl (1986) does not apply to the case of standing water behind the structure nor does it consider a stabilized wave height, it was not deemed appropriate for further consideration.

89. Due to the above noted limitations in the existing state-of-the-art methodology, no recommendations can be provided at present for computing wave transmission over coastal flood-protection structures. In Appendix B, an alternative method for calculating wave overtopping and consequent wave transmission via overtopping is provided and compared with existing limited laboratory data. At present, this methodology is preliminary and requires more

testing with a generic set of laboratory data prior to any final recommendations for its use. The approach in Appendix B may be useful as an interim measure for the calculation of wave overtopping and wave transmission for FEMA purposes in delineating the type of flood zone (V or A zone) behind a coastal flood-protection structure.

Recommendation

90. Present state-of-the-art methodology for prediction of wave transmission past coastal structures is inadequate to determine the wave height landward of a coastal flood-protection structure with any level of confidence. A preliminary methodology has been presented in Appendix B for calculation of the wave transmission past a coastal flood-protection structure. The method has been tested with a limited amount of data. Recommendation for general usage of the methodology cannot be made without further testing, although the methodology could be used as an interim measure to determine the wave height in the lee of a coastal flood-protection structure. As an interim measure, the methodology could be used along with present FEMA inland wave routing models to determine the velocity zone behind a coastal flood-protection structure.

PART V: WAVE FORCES ON COASTAL FLOOD-PROTECTION STRUCTURES

91. A major question of importance in any evaluation of coastal flood-protection structures is the survivability of the structure through a major storm event. This survivability is dependent on a number of factors, many of which are covered in a later section of this report. The primary requirement for survivability from an engineering point of view is the ability of the wall to withstand the wave forces to which it is subjected. For this reason, an evaluation of wave forces on sloping and vertical wall structures is necessary.

92. The ideal methodology for calculation of wave forces on coastal flood-protection structures from a FEMA standpoint would consist of the following points:

- a. The methodology would be consistent with present FEMA guidelines in evaluating waves and water levels; i.e., it should be consistent with a depth-limited breaking wave height approach.
- b. The methodology should be capable of providing a completely defined pressure distribution over the vertical such that shear and moment diagrams necessary for structural evaluation can be made.
- c. The methodology should be supported by field and laboratory data for all possible types of structure under all types of wave conditions and bathymetry.

93. It will be shown in the following section that none of the methods completely fulfills the above ideal requirements. In the United States, as in other countries, the design of coastal flood-protection structures is done after extensive laboratory testing of site-specific conditions. Wave force calculation methods such as those provided in the following section would be used only in preliminary design of a structure. No comprehensive set of wave force measurements covering all types of structures, wave conditions, and bathymetry exists. As such, any method chosen will be open to questions concerning its accuracy. Where site-specific laboratory data exist for a given coastal flood-protection structure, they should be considered and possibly adopted in lieu of any FEMA recommended approach.

94. The study of wave forces on walls in quantitative terms goes back into the 19th century (see Blackmore and Hewson (1984) for a brief discussion

of this early work). Green* divides the early work on this subject from the recent with the publication of the results of Rundgren (1958) research. Rundgren provides a thorough review of the earlier work including both field measurements and laboratory studies. Early field measurements of wave forces were made using spring dynamometers. Because of the high inertia of these devices, it was impossible to measure the high-impact but short-duration loads (often referred to as shock loadings) caused by waves breaking directly on the walls; as a consequence, many of the early studies did not record the high pressures found in later studies.

95. Improvements in strain gages, pressure transducers, and electronics in general have made it possible to measure wave pressures reliably both in the laboratory and field. By using high frequency response transducers and high sampling rates, it is now possible to accurately record the extremely high impact pressures caused by breaking waves. Important advances are also being made in numerical modeling of wave forces, and when these methods are coupled with improved laboratory investigations and field studies, they can be expected to produce techniques to estimate wave pressures and forces on walls substantially better than obtained in early studies. Due to the stated limitations of the early work, much of it is of limited value for realistic engineering assessments.

Methods in Use

96. Recent literature on wave forces (since Rundgren (1958)) can be logically categorized as follows:

- a. Experiments using monochromatic waves.
- b. Field measurements of wave pressures.
- c. Theoretical work on standing waves.
- d. Work on breaking wave pressures.
- e. Work on the response of vertical-walled breakwaters to wave forces.

97. To provide perspective on wave force calculations, a schematic approach to the subject based on present policy of the USACE is shown in Figure 17. The SPM (1984) divides wave forces into three classes based on

* T. Green, "Recent Work on Wave Forces on Vertical Walls," unpublished report, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

ANGLE OF WAVE APPROACH (ALPHA)	WAVE CHARACTERISTICS			
	NONBREAKING WAVES	WAVES BREAK DIRECTLY ON WALLS	BROKEN WAVES	
			WALL SEAWARD OF SWL*	WALL SHOREWARD OF SWL
90-70 DEG. FORCE REDUCTION SIN	SAINFLOU, MICHE, AND RUNDGREN AS GIVEN IN SPM (1984)	MINIKIN, AS GIVEN IN SPM (1984)	SPM (1984)	CAMFIELD CETA-III-29 (1986)
70-45 DEG.	WEGGEL CETA 81-1 (1981)			
45-0 DEG. MACH-STEM EFFECT	WEGGEL CETA 81-1 (1981)			

Figure 17. Overview of methods to compute wave forces on walls

nonbreaking waves, waves breaking directly on the wall, and waves breaking before encountering the wall. This classification is used in Figure 17 with the broken wave class divided into two parts, a wall seaward of the still-water line and a wall landward of the still-water line. To further compartmentalize the methods of computing wave forces, the angle of wave approach to the wall is used. This classification is useful because it provides a logical way to separate forces caused by the traditional three classes of wave conditions and those caused by Mach-stem waves. Mach-stem waves are formed when waves approach obliquely and reflect from vertical walls. For Mach-stem effects to be important, the angle of approach of the waves to the wall must be less than about 45 deg.

98. When the angle of wave approach to a wall is less than about 70 deg, wave impact loads probably do not occur, i.e., loads caused by

extremely high but short-duration pressures occurring when a wave breaks directly against a wall.

99. The problem of the angle of approach to a vertical wall is generally not relevant to the design of seawalls on the open coast since seawalls typically run parallel to shore. Wave refraction usually causes the angle of approach to a seawall to be about 90 deg. Since the SPM-recommended correction for wave forces is $\sin^2\alpha$ where α is the angle of approach, small differences from an angle of approach of 90 deg are inconsequential. It should be recognized that the SPM correction factor for wave angle of approach is based on an engineering approximation and has not been verified by either field or laboratory studies. Review of the wave force literature indicates that it is not within the current state-of-the-art ability to accurately predict wave forces on walls other than assuming perpendicular approach. It is recommended that FEMA consider only structure perpendicular (90 deg) wave force loading in its evaluation of coastal flood-protection structures.

100. The primary emphasis in wave force research has been on laboratory studies with waves breaking directly on vertical walls or caisson-type breakwaters. Much of this work has been conducted in Japan, the European countries, and the United States.

101. A substantial portion of recent research has been conducted in Japan. Such research is very valuable but must be interpreted carefully. Much of the Japanese effort is directed toward the design of large caisson-type breakwaters often sited in deep or intermediate water depth on rubble foundations. This situation is in contrast to the typical coastal flood-protection structure such as a bulkhead/seawall or revetment located near the shoreline with relatively shallow water offshore. Because of large mass, a typical caisson is generally not vulnerable to failure under the very high but short-duration pressures caused by waves breaking directly on the wall. For a typical "American coast," nonengineer-designed, low-mass bulkhead/seawall that has lost its backfill, these short-duration loads create the possibility of failure of the wall by shear or inward bending moment (Walton and Sensabaugh 1979). Fatigue and corrosion also contribute to the vulnerability of thin, low-mass sheet-pile walls to these short-duration wave forces. Figure 18 shows pressure records from laboratory tests of wave impact loads on a seawall (Grace and Carver 1985). In Figure 18 the peak temporal pressures (often referred to as "shock" pressures) are caused by the wave breaking directly on

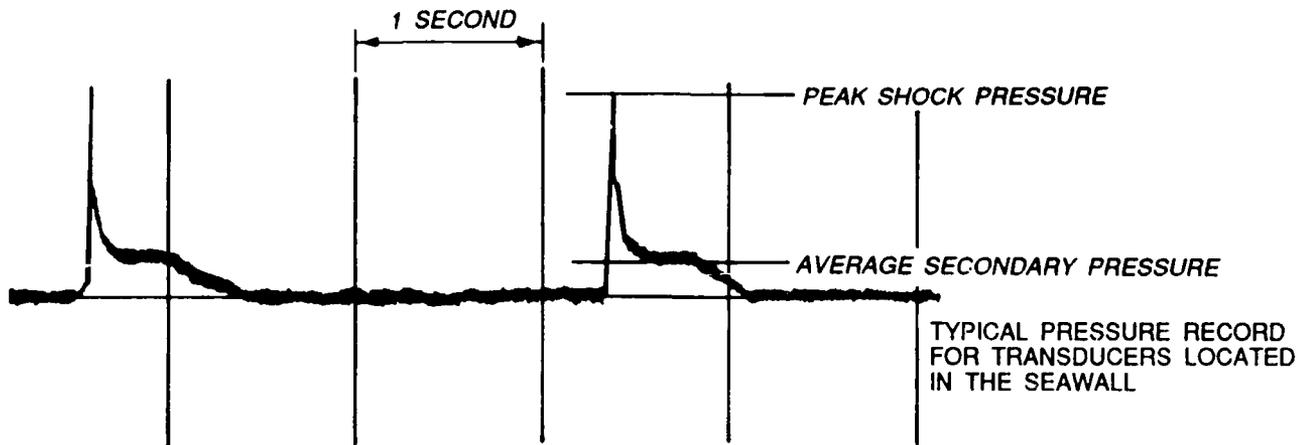


Figure 18. Typical pressure-time record for waves breaking on the structure (Grace and Carver 1985)

the seawall while the secondary or surge pressures are a longer duration dynamic pressure normally associated with hydrodynamic momentum transfer. It appears that most formulae to predict maximum pressures are predicting secondary pressures rather than "shock" pressures; this tendency is particularly true of the Japanese literature. In fact it is difficult to base a design criterion strictly on "shock" loads because their short duration make them hard to measure, resulting in much scatter in the data and controversy in the literature about their actual magnitude. In addition, it is found that waves breaking directly on a vertical wall is a relatively rare event in the field. Blackmore and Hewson (1984) found that approximately 1 in 2,500 waves impinging on the walls that they monitored actually produced "shock" loads. This discussion suggests that a method of computing wave forces based on a conservative interpretation of secondary nonshock pressures might be a logical approach for coastal flood-protection structures with substantial mass. For nonengineered structures with little mass (i.e., sheet-pile seawalls), the primary "shock" load should be considered.

102. A considerable amount of research on wave forces on walls has been conducted at Osaka City University, Japan, by Nagai and colleagues. This work has been primarily directed at the design of vertical-walled caissons placed in intermediate depths of water and includes extensive laboratory and field studies. Two good summaries of Nagai's findings are given in Nagai (1973) and Nagai (1976). Generally Nagai's work is not directly applicable to the design of many coastal flood-protection structures because of the extensive rubble

foundation and because of the depths of water considered. In Nagai (1973), there is a formula for the total dynamic force on a seawall, but its origin is unclear and it does not provide a definition of the force distribution. This formula may have resulted from research by Mitsuyasu (1962), but the formula does not lend itself to unambiguous interpretation and can even give negative forces for steep wave conditions. Nagai (1973) notes that high shock pressures were rarely observed during laboratory experiments on vertical walls when offshore slopes were 1 on 50 or flatter; however, no data are shown or other references cited.

103. Another important source of research in Japan is at the Port and Harbour Research Institute (PHRI), Yokosuka, Japan, by Goda and colleagues. These studies are oriented toward goals similar to those of Nagai, i.e., design of caisson breakwaters in the intermediate water depths. Goda (1974) has developed general guidelines that provide the force distribution on walls. The model can be applied to nonbreaking, breaking, and broken wave conditions and to vertical walls with and without rubble foundations. Goda's model has been developed and refined from laboratory and field studies. Although the model is for irregular wave conditions, it will be compared with monochromatic approaches in the following section. Goda (1972) shows some monochromatic wave data for a traditional seawall, i.e., a vertical wall without a rubble foundation. The highest pressures observed were rather low with the largest dimensionless value given by $P_{\max}/wH = 1.25$, where P_{\max} is the maximum pressure caused by a wave of height H and w is the unit weight of the fluid. In contrast, Nagai (1973) found values of the dimensionless maximum pressures considerably higher (from 5 to 20), apparently caused by unfortunate combinations of wave conditions incident on caissons with high rubble foundations. According to Goda (1972), the low values of pressure observed are due to the flat, 1 on 100, offshore slope and the high relative depths used, $0.10 \leq d_s/L \leq 0.30$, where d_s = depth at toe of structure. Goda indicates that his data are consistent with earlier data collected at PHRI by Mitsuyasu (1962) for similar conditions but somewhat smaller scale. The monochromatic tests of Goda (1970) are important because they confirm some of the earlier findings of Mitsuyasu and because they were conducted at relatively large scales, i.e., $1.15 \leq d_s \leq 1.48$ ft, $0.22 \leq H \leq 1.36$ ft, and $1.01 \leq T \leq 2.01$ sec. These tests indicate that peak pressures on a seawall should increase with decreasing d_s/L , and they confirm unpublished findings

mentioned by Nagai (1973) that peak impact pressures are low when the offshore slope is flat. Best references for this research at PHRI are Goda (1974, 1985).

104. Other research centers for wave forces on walls are located in Tokyo at Tokyo University and at Nihon University. Studies at these universities are directly related to the design of seawalls and not caissons on rubble foundations. Hom-ma and Horikawa (1964, 1965) developed a method (hereafter referred to as the H and H method) for the pressure distribution and total force on a seawall. The method accounts for both vertical and inclined walls. Their model was calibrated with laboratory tests. These tests included both pressure transducers to obtain the pressure distribution on the wall and strain gage measurements to obtain the total force. They compared their model with data collected by Mitsuyasu (1962) and generally found good but not necessarily conservative agreement for breaking wave conditions. Their method gives surprisingly good estimates of wave pressures on inclined walls when compared with their laboratory data. This finding is surprising because the pressure reduction of the H and H method for an inclined wall is quite substantial, but clearly a wave breaking directly on an inclined slope can cause very high pressures if there is little overlying water to cushion the impact, as has been measured by Fuhrboter (1986). Without further research, it is impossible to reconcile some of the differences in findings between the studies of Hom-ma and Horikawa and Fuhrboter. Further laboratory tests of the total force on a vertical wall at Nihon University by Takezawa (1979) provides additional support for the H and H method. Interestingly, Takezawa found that the Hom-ma/Horikawa model provided relatively conservative results even for waves breaking directly on the wall. Takezawa developed his own model for the total force on a wall apparently to provide a somewhat less conservative envelope for forces caused by nonbreaking wave conditions than would be obtained using the H and H method. Takezawa's work is important because it gives independent support for the H and H method based on extensive laboratory tests at a larger scale. Wave heights used in Takezawa's tests went up to 0.72 ft, and water depths at the toe of the wall were as great as 0.97 ft. Unfortunately, there are little contemporary field data to support the H and H method as noted in Hom-ma and Horikawa (1965).

105. In a discussion of some of the early research conducted in Japan on wave forces, Goda (1985) mentions the work of Hiroi, who made field

measurements of wave forces in the early 20th century and developed a simple relation for the pressure distribution. In Japan, Hiroi's method has been used to compute breaking wave forces, whereas Sainflou's method has been used to compute standing wave forces, but there is a discontinuity in pressure and force estimates at the location of the transition between the two methods. It was dissatisfaction with this illogical discontinuity that encouraged Goda to develop his more general method (Goda 1985). Goda's method appears to be the one accepted for calculation of wave forces on walls (primarily caisson-type structures) in Japan today.

106. In summarizing the Japanese literature on wave forces, it appears that Goda's method might be useful for the calculation of nonshock wave forces on vertical-walled massive structures, but it may be unconservative for low mass structures where high shock loads could cause failure. In view of the fact that no independent testing of Goda's method could be found, it is not mentioned further. The Hom-ma and Horikawa wave force calculation method appears reasonable for use in nonshock loading on coastal flood-protection structures and will be discussed further later.

107. In the United States, the primary recommended method of wave force calculation for nonbreaking waves are the Miche, Rundgren, and Sainflou methods as discussed in SPM (1984). These methods are for nonbreaking wave forces that might thus be interpreted in the present context as nonimpact loading. These methods have undergone considerable laboratory testing and appear quite useful for structures in intermediate or deep water where the assumption of partial standing wave is reasonable. For use in evaluating coastal flood-protection structures, the validity of the assumptions necessary to use the methods (i.e., partial standing waves) is questionable.

108. One of the most widely used methods to estimate the pressures and forces of waves breaking on walls is Minikin's method (Minikin 1963). Minikin based his approach on field observations and limited laboratory data collected by Bagnold (1939). The SPM (1984) recommends Minikin's method for calculating wave pressures and force due to breaking waves. Minikin's approach has been criticized as yielding forces that are unreasonably large (e.g., Blackmore and Hewson 1984), but the method has also found support from some field observations by Myers, Dunwoody, and Kirley (1983). Some of Nagai's (1960) laboratory data are consistent with Minikin's method, but the condition described by Nagai as "extraordinary breaking waves" produces pressures that exceed

prediction by Minikin. No clear definition of extraordinary breaking waves is given by Nagai. Kirgoz (1982) also provides laboratory data which exceeds the pressures predicted by Minikin, but these data could be influenced by scale effects, as noted below. An unusual aspect of Minikin's method is that pressures and forces decrease with increasing wave period. This trend is at variance with intuition and most other methods. This rather anomalous trend may explain some of the controversy related to Minikin's method. Blackmore and Hewson's pressure observations were for short period waves where Minikin's formula might be expected to give excessively high values.

109. The study of Blackmore and Hewson (1984) is based on field data and a thorough review of data collected in the laboratory and field by other investigators. Nine pressure transducers mounted in a seawall at Ilfracombe, on Bristol Channel, England, permits evaluation of the pressure distribution as well as peak pressures for one specific site and type of structure. Other instrumented seawalls were mentioned as part of the study, but most of the data discussed come from the one site at Ilfracombe. Blackmore and Hewson's data summary indicates that with few exceptions the wave conditions had periods less than 4.7 sec. This means that their wave force model is not calibrated for longer period swell waves. They make some interesting comments regarding scale effects, in that laboratory tests should produce conservative results on peak pressure measurements. The reason is that surface tension is more important at small scale, and this causes less air entrained in the laboratory tests when compared with field conditions. With less air entrained in the laboratory, the scaled peak pressures are higher. These conclusions are strongly supported by laboratory work by Fuhrboter (1986) which included tests conducted in both large and small wave tanks. Between the two facilities, the range of wave heights Fuhrboter investigated was from 0.28 to 6.89 ft. Although some of the conclusions reached by Fuhrboter relating to scale effects seem weak, his findings regarding peak pressures provide very strong support to Blackmore and Hewson's interpretation that small-scale tests produce greater peak impact pressure than would be expected at prototype. This finding is also consistent with the interpretation of field measurements of wave impact pressures made by Miller et al. (1974). Miller examined results from his own field work and the field work of other investigators and found that it consistently produced lower wave impact pressures than would be expected based on laboratory tests.

110. As noted previously, Kirgoz (1982) measured very high impact pressures on vertical walls in his laboratory tests. These pressures are greater than would be predicted by Minikin's formula. There are several unusual aspects to his tests; they included: (a) very steep beaches were in front of the wall, ranging from 1 on 4.45 to 1 on 15; (b) the instrumented wall was dropped in place into an existing wave field; and (c) the scales were quite small (e.g., breaker heights ranged from 0.13 to 0.27 ft). These test setup conditions may account for the high pressures observed. Kirgoz's work yields a pressure distribution model that is dependent on the offshore slope.

Suggested Methodology

111. Information from the preceding discussion can be distilled for comparison into tabular form. The criteria used by FEMA for depth-limited breaking waves are given as

$$H_b = 0.78d_s \quad (34)$$

Using the definition given in Equation 34, a number of methods developed by different researchers for computing the total force on a vertical wall can be reduced to the form

$$F_t = C w d_s^2 \quad (35)$$

where

F_t = total force per unit length of wall, including both the static and dynamic components

C = dimensionless force coefficient that is a constant for some methods and a function of water depth and/or wave period for other methods

w = unit weight of water, i.e., 62.4 lb/ft³ for fresh water and 64.0 lb/ft³ for seawater

The static component must be included for comparison since it is calculated from the upper limit of runup for some methods and from the mean water level (MWL) for other methods. The value or range of the force coefficient C is tabulated for various methods of calculating the total force in Table 2. Also shown in Table 2 is the dimensionless maximum pressure for the method. For the methods shown in Table 2, the maximum dynamic pressure occurs at the

Table 2
Methods to Calculate Wave Forces Reducible to the Common Form

$$\underline{F_t} = C w d_s^2$$

<u>Method/Reference</u>	<u>Functional Form of Force Coeff., C</u>	<u>Range of C</u>	<u>Dimensionless Peak Pressure $\frac{P_{max}}{w h_b}$</u>
Hiroi/Goda (1985)	constant	4.26	1.50
H&H/Hom-ma and Horikawa (1964, 1965)	constant	4.18	2.05
Goda (1974, 1985)*	$f\left(\frac{d_s}{L}\right)$	1.03-1.86	0.60-1.10
Minikin/SPM (1984)*	$0.97 + 41\left(\frac{d_s}{L}\right)$	1.18-21.47	$202\left(\frac{d_s}{L}\right)$
Blackmore and Hewson (1984)**	$(0.97 + 0.38T)$	1.73-8.57	0.62T

* Range of relative depth $0.05 \leq d_s/L \leq 0.50$ used for Goda's and Minikin's method.

** Range of wave period, $2.0 \leq T \leq 20$, used for Blackmore and Hewson's method.

MWL. In Table 3 the dimensionless maximum pressure is tabulated for two field studies and one study using a large wave tank.

112. In comparing the methods of calculating wave forces shown in Table 2, it is apparent that Goda's (1985) method gives conspicuously low values, suggesting that it is primarily intended for the design of massive caisson breakwaters that are not very vulnerable to high short-duration shock loads. Another problem in using Goda's method is that it was developed for irregular wave conditions and therefore not consistent with a depth-limited breaking wave height approach. Problems related to the choice of wave heights to be used in a method always occur when findings based on monochromatic waves are applied to irregular waves or vice-versa. A similar problem occurs when trying to adapt the method of Blackmore and Hewson (1984), which is based on field observations, to the essentially monochromatic approach of FEMA consistent with existing guidelines. When the formula for the dynamic pressure of Blackmore and Hewson was adapted to monochromatic wave conditions, the relation for wave celerity used was

$$C_b = \sqrt{gd_s(1 + 0.7H_b)} \quad (36)$$

or considering Equation 34

$$C_b = \sqrt{gd_s(1.55)} \quad (37)$$

where C_b is the celerity of the breaking wave. Using wave periods in the range of 4 to 5 sec (within the range of Blackmore and Hewson's data) for the peak pressure relation in Table 2 gives pressures greater than observed in their study as shown in Table 2 of their report.

Table 3

Values of $\frac{P_{\max}}{w H_o}$ from Recent Field Studies and Large-Scale Laboratory Tests

Source	Typical Value	Range
Blackmore and Hewson (1984), Table 2	1.24	0.87-2.26
Miller et al. (1974), Figure 5	2.70	1.46-8.10
Fuhrboter (1986), Table 3	2.2	1.8-2.6

113. Table 2 also shows a rather surprising aspect of Minikin's method of calculating wave forces. Minikin's methods predict decreasing wave forces with increasing wavelength. Since breaking wave forces might be expected to be proportional to wave celerity squared, it is difficult to envision a force equation with an inverse relation to wavelength. In some ways, Goda's (1974, 1985) model for wave forces appears more intuitively correct, and his model shows a gradually increasing force with increasing wavelength as would be expected. Minikin's method gives extremely high forces for short period waves, $T = 3$ to 5 sec, as noted by Blackmore and Hewson (1984) but may give more realistic values for longer period waves.

114. The remaining two methods, Hiroi's and Hom-ma and Horikawa's, shown in Table 2 give rather similar wave forces. Because of differences in the pressure distributions used by the two methods, Hiroi gives slightly higher total force, and Hom-ma and Horikawa give higher maximum pressures. When the pressures of Table 2 are compared with field and large-scale data in Table 3, the Hom-ma and Horikawa method shows values within the range of all

three data sets. Although the agreement may be fortuitous, it indicates that the Hom-ma and Horikawa method provides realistic estimates of prototype wave loads. Because of the basic simplicity of the method, the independent confirmation of the method by the laboratory work of Takezawa (1979), and the agreement with prototype scale measurements, the Hom-ma and Horikawa method is recommended for use to compute wave pressures, forces, and moments on walls by FEMA.

The Hom-ma and Horikawa Method

115. A definition sketch for the pressure distribution of the method of Hom-ma and Horikawa (1964, 1965) is shown in Figure 19. The upper limit for both the static and dynamic pressure is $1.2 d_s$ above the MWL. Dynamic pressure increases linearly from zero at the upper limit to the maximum value at the MWL of

$$P_{\max}(\text{dynamic}) = 1.6 w d_s \quad (38)$$

Dynamic pressure then decreases linearly from the maximum value to zero at the toe of the wall. This pressure distribution produces a total force on the wall, i.e., static plus dynamic, of

$$F_t = 4.18 w d_s^2 \quad (39)$$

This acts just below the MWL at $z = -0.126 d_s$, where z is the vertical axis and the minus sign indicates below the MWL.

116. Hom-ma and Horikawa (1964) suggest a reduction in the pressure against the wall for plane, inclined walls of $\sin^2 \theta$, where θ is the angle of inclination with the horizontal. This reduction is supported by their laboratory data. The general pressure distribution relation for plane walls is

$$p_u = 0, \text{ at } z = 1.2 d_s$$

$$p_{\max} = 2.8 w d_s \sin^2 \theta, \text{ at } z = 0$$

$$p_t = 2.2 w d_s, \text{ at } z = -d_s \quad (40)$$

where

p_u = pressure at the estimated upper limit of runup

p_t = pressure at the toe of the wall

The pressure distribution shown will give the total force indicated by Equation 39 when $\theta = 90$ deg , i.e., a vertical wall.

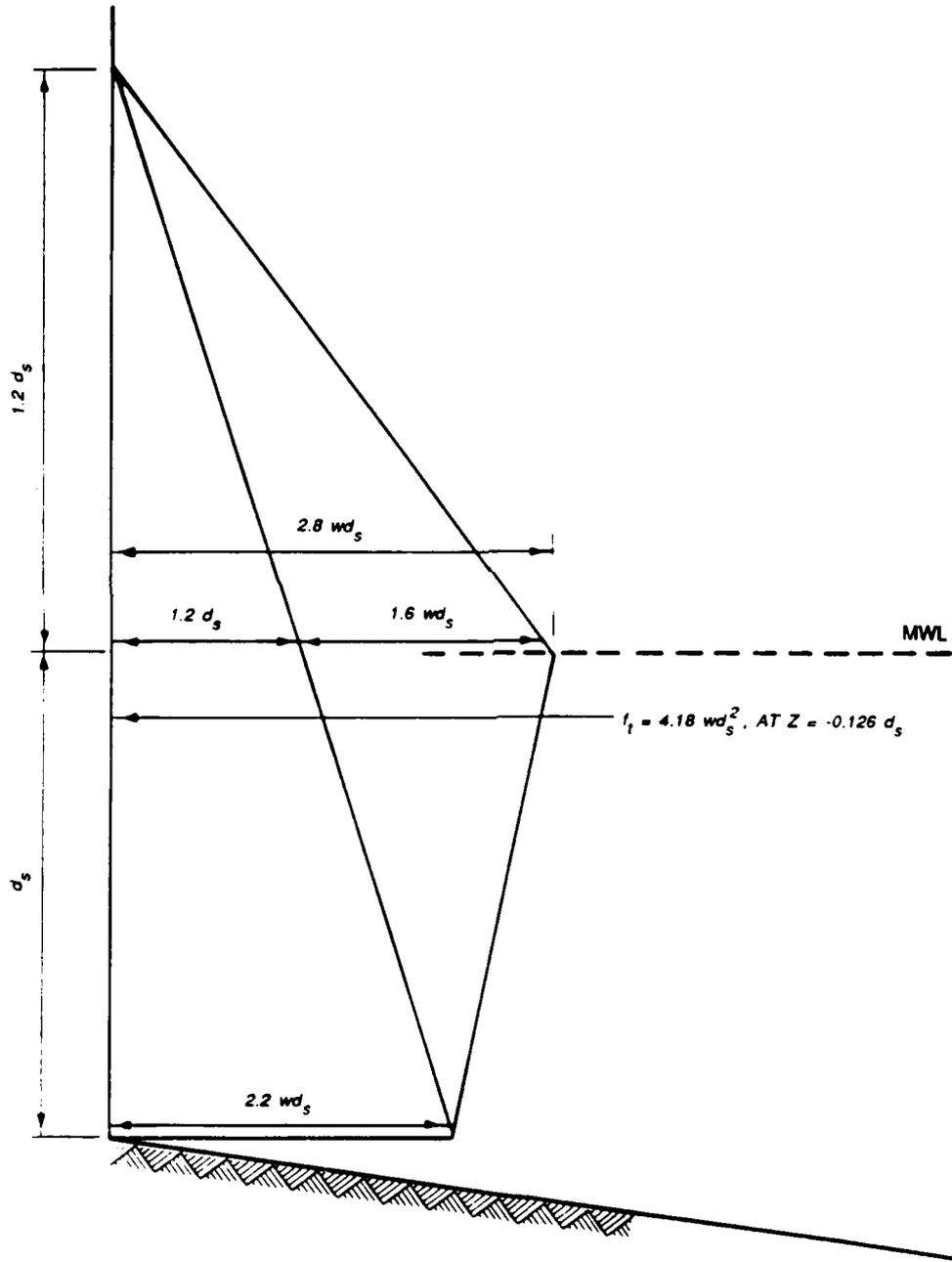


Figure 19. Pressure distribution for Hom-ma and Horikawa method

117. Due to the wide scatter seen in laboratory wave force measurements for the seemingly identical wave-structure-bathymetry conditions, it may be appropriate to adjust the coefficients upward for the higher shock pressures and forces. Although Hom-ma and Horikawa do not provide information for such an adjustment, results from another study could be used to infer these coefficients. Fuhrboter (1986) shows that impact pressures have a log-normal distribution. Fuhrboter's findings suggest that if the maximum pressure in the vertical plane given by Equation 38 is regarded as the median value of many impact loads, then the coefficient in the equation can be regarded as a probabilistic function as shown in Table 3. Data plots by Hom-ma and Horikawa (1964) indicate that it is reasonable to regard the value of the coefficient as approximately a median value. Table 3 could be used to increase the maximum dynamic pressure given by Equation 38 if the structure is of low mass and might fail by short-duration wave force. The total maximum pressure can be written

$$P_{\max}(\text{total}) = \begin{array}{c} \text{dynamic} \\ \text{component} \end{array} C_p w d_s + \begin{array}{c} \text{static} \\ \text{component} \end{array} 1.2 w d_s \quad (41)$$

where the dimensionless coefficient C_p can be chosen from Table 3 based on the degree of conservatism required for a specific type of structure.

118. There are ambiguities in comparing the findings of Hom-ma and Horikawa (1964, 1965) and Fuhrboter (1986). The method used to apply Fuhrboter's findings in Table 3 should provide a reasonable guess as to maximum pressure. However, the H and H method gives maximum pressures that decrease as the angle of inclination decreases, which is opposite to the findings of Fuhrboter. It is difficult to reconcile these differences between the results of two important studies. Further research is required to supply answers to problems such as noted above to achieve a higher level of confidence for methods of calculating wave pressures and forces.*

* At the time of writing this report an important paper entitled "Dynamic Forces Due to Waves Breaking at Vertical Coastal Structures" by H. W. Partensky was not available. This paper provides information on large scale laboratory measurements of breaking wave forces on vertical walls.

Table 3

Value of Coefficient for Equation 37 as a Function of
Probability of Exceedance Based on Fuhrboter (1986)

<u>Coefficient</u> <u>C_p</u>	<u>Probability of</u> <u>Exceedance</u>
1.60	0.50
2.18	0.10
2.84	0.01
3.49	0.001

Recommendation

119. Numerous empirical wave force prediction equations exist for calculating the hydrodynamic force exerted on a coastal flood-protection structure. The limitations of these equations are not known; therefore, none of the equations can be used with any level of confidence outside the limited conditions for which they were formulated. The best equation for FEMA purposes is the wave force equation of Homma and Horikawa (1964, 1965), which can be used for all structure slopes in a manner consistent with the FEMA depth-limited breaking wave criteria. It should be recognized that forces calculated via the Homma and Horikawa method may be low compared with many measured shock forces, but reasonable guidance on determination of such shock forces is still not within state-of-the-art coastal engineering predictive ability.

PART VI: COASTAL FLOOD-PROTECTION STRUCTURE INTEGRITY

Description and Classification of Coastal Flood-Protection Structures

120. Seawalls, dikes, bulkheads, revetments, and other types of coastal flood-protection structures can be grouped or classified in several ways. Three of the most commonly used groupings are functional intent, design or construction method, and construction material(s). Although the classification into which a particular structure might be placed using these systems is somewhat arbitrary and overlapping, it is useful to discuss the groupings to provide insight into considerations that should be a part of proper engineering design and/or the technical evaluation of such structures. Further details on specific key factors in the design, construction, and evaluation of shore protection structures will be provided in a subsequent section.

121. Functional intent is probably the primary means used to distinguish various types of coastal structures. Two functional objectives typically cited are retaining upland soil and intercepting wave action. Since many coastal structures perform both of these functions (and others) to some degree, definitions tend to focus on a "principal" function.

122. As an example, both seawalls and bulkheads retain upland property and intercept wave forces. However, sources generally distinguish the two structures on the basis that seawalls are intended principally to intercept wave action. Further, such wave action is recognized in the design process to be severe, and the resulting hydrodynamic forces are meant to be resisted largely within the seawall structure itself. Intercepting wave action is a secondary objective in the design of bulkheads, and the design wave conditions considered are less extreme. Bulkheads are often constructed without considering wave force loadings. In such cases their success depends on transferring dynamic loadings to the soil behind the bulkhead.

123. In contrast, dikes, revetments, and breakwaters are intended principally to intercept wave action, even severe wave action. Dikes, revetments, and breakwaters are distinguished from seawalls and bulkheads, however, by the fact that they do not function directly to retain upland soil. In the case of revetments, the upland soil mass provides support for the structure and serves to resist wave forces transmitted through the flexible armor layers.

Revetments alone are not meant to prevent flooding. They are used in combination with or as a functional part of a flood-protection structure such as a dike. Dikes may or may not be in contact with the upland soil. Breakwaters are typically not in contact with the upland soil that they protect. They function by dissipating energy at some distance offshore.

124. One of the difficulties with functional classification is that two structures which appear physically similar could be classified differently depending on design intent. This is especially a problem when evaluating existing construction for which the original design intent may be uncertain.

125. There is no general agreement on the severity of design wave action that requires a seawall design as opposed to a bulkhead design. The Naval Facilities Engineering Command (NAVFAC) (1981a) suggests that seawalls should be selected where the site is exposed to (significant) wave heights greater than 3 to 4 ft. Other references (e.g., SPM (1984), USACE 1985) avoid stating a precise wave height and simply suggest that bulkheads (and revetments) are not appropriate for "severe" or "high" wave conditions. Perhaps the most succinct statement of both the definitional problem and at least one pragmatic solution is found in USACE (1985): "The use [of bulkhead materials] is limited to those areas where wave action can be resisted by such materials."

126. Flood-protection structures are frequently described by modifiers that convey details about the engineering design and/or construction approach used. Examples are pile-supported seawall, counterfort gravity seawall, or free-earth support anchored bulkhead. The proper use, recording, and understanding of such descriptors is much more than an exercise in semantics; these grouping terms can contain valuable information about the design intent of a structure, the forces and/or problems that the designer felt were important at the site, and likely types of potential damage or modes of failure in the structure. Some caution must be used in interpretation, however. Unfortunately, terminology has not been adequately standardized, nor does construction always follow the intended plans or designs.

127. The factors that allow certain structures to be grouped by design approach generally include such things as the way in which the structure itself is supported, how applied loads are resisted or transmitted, or the degree of structural flexibility or rigidity (i.e. assumptions about deformation). A preliminary discussion of the more common terms and design

approaches follows. Several references and handbooks are also available for additional information (see Peck, Hanson, and Thornburn (1974); US Steel Corporation (1975); Merritt (1976); or NAVFAC (1982a)).

Gravity seawalls

128. Gravity seawalls are those which rest directly on a soil substratum and rely on that soil for support in the same manner as conventionally designed shallow foundations. Gravity walls may be constructed of different types of material and in several configurations. However, as with all gravity-type structures, these walls depend principally on their own weight for stability against all loading conditions including lateral loads such as waves. The ability to mobilize that weight for stability is in turn dependent on the bearing capacity and continued integrity of the underlying soil mass.

129. Since weight is required for stability, gravity structures typically tend to be massive, monolithic in construction, often unreinforced, and not particularly efficient in economy of material. The overturning moments induced in a wall by soil and wave lateral forces are proportionate to the moment arm length or height at which the forces act above the base. As a result, height is generally the limiting factor for the use of conventionally designed rigid gravity walls. As a very rough approximation, true gravity walls are generally not considered economical for heights much above 15 ft (Merritt 1976). In coastal applications where a seawall must be high enough to extend from below some scour depth to above a design overtopping elevation, this economical height is often exceeded, and variations in gravity wall designs are frequently seen.

130. Techniques used to increase wall height without proportionate increases in weight and cross-sectional area (and cost) have been borrowed from the design of upland retaining walls and applied to seawalls. These techniques give rise to the use of terms such as semigravity, cantilever, counterfort, and others (Figure 20) to describe structures that have characteristics generally common to gravity walls, but are not strictly rigid. In each case an increasing percentage of the lateral thrust is resisted by bending moments developed within the vertical stem and an extended section of the base, and by cantilever action between them. Such walls may be quite high, yet relatively slender compared with conventional gravity structures. Reinforcement is required in concrete walls of these types, and external stiffeners are necessary for the more rarely used steel or timber gravity walls.

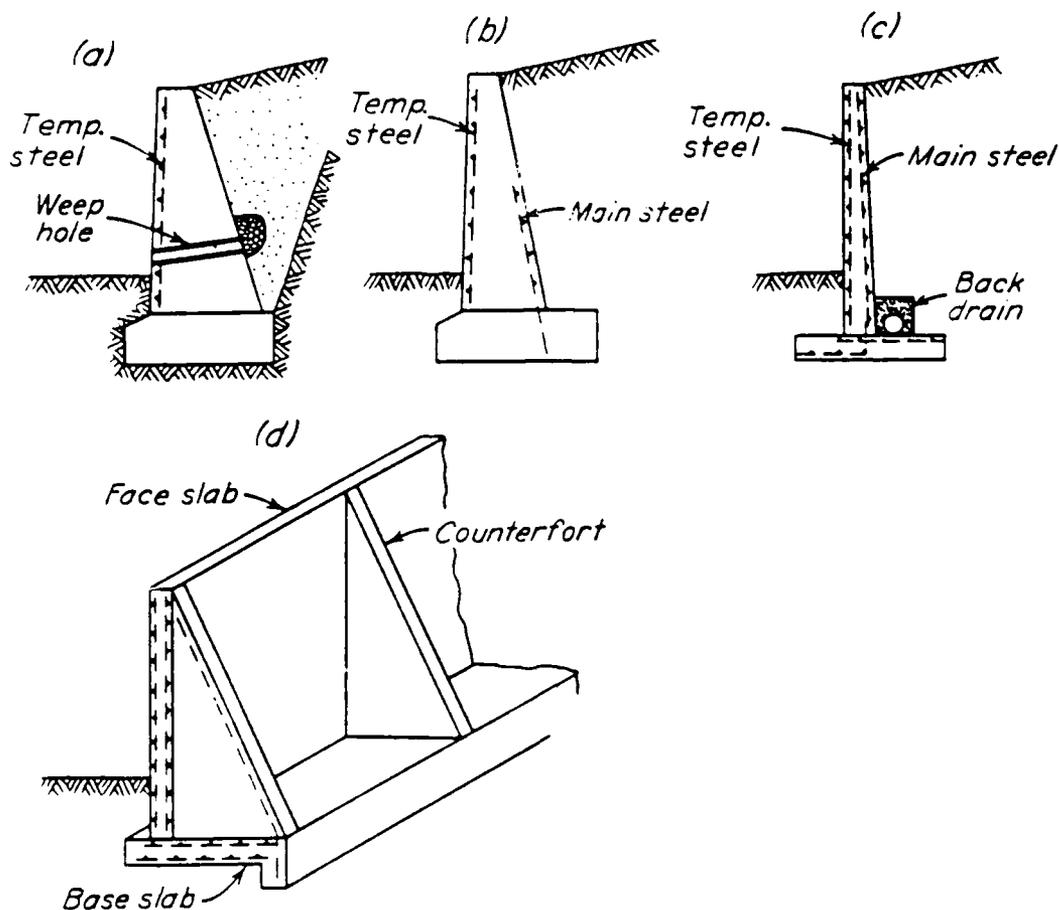


Figure 20. Types of walls; (a) gravity section, (b) semigravity section, (c) cantilever section, (d) counterfort wall

131. Other gravity-type seawalls occasionally seen are variations of caissonlike structures. These can include true caissons or the expedient use of precast units such as large-diameter pipe. The design approach used in these seawalls is similar to conventional gravity walls in that the cross-sectional areas and weights remain greater than used in cantilevered sections and the structures are generally considered rigid for the purposes of preliminary analysis.

132. Gravity-type structures designed to withstand wave forces without a need for a backfill buffer can fail in four primary ways: monolithic overturning, sliding, loss of bearing capacity, or differential settlement. Each of these problems can ultimately be related either to inadequate analysis, preparation, and protection of the foundation soil or to poor selection and placement of the backfill. Each of these failure modes must be checked in a design to assure stability of the structure.

133. The base of a gravity-type wall must extend below the potential scour depth, and/or scour protection must be provided if the wall is to perform successfully. Scour can directly undermine the structure, causing overturning. It can also affect the bearing capacity of the foundation stratum and the sliding stability of the wall and its retained soil mass by reducing the passive resistance contribution from overburden soils.

134. Design scour depth at a site is discussed in Appendix C. Various types of scour protection are described in detail in references including USACE (1985), Eckert and Callender (1987), and others. Careful evaluation of potential scour and its effect is especially important in reviewing existing structures because profile elevations may have changed since the original design.

135. Richart and Schmertmann (1958) and Eckert and Callender (1987) discuss the importance of backfill in wall design. They note that free-draining soils that exhibit large angles of internal friction (i.e. clean sands) are the preferred backfill.

136. If the soil behind the seawall is not free-draining and if positive drainage measures are not provided, hydrostatic pressures can build up from tidal lag, high ground-water levels, and/or overtopping (Walton and Sensabaugh 1979). The increase in lateral pressure on the wall can be as great as 50 percent above the drained case (Eckert and Callender 1987). This increase can be accommodated in the structural design, but it must be recognized in the design process. If a positive drainage alternative is selected, the drainage media must be capable of removing the excess water without concurrent loss of any backfill. Methods for designing adequate drainage of backfill are provided in Cedegren (1960), USACE (1983), and others.

137. Conventionally designed gravity seawalls rely principally on their own weight for stability and on satisfactory foundation soil conditions to support that weight. Protection of the toe soils from scour is critical to the performance of gravity walls. Free-draining backfill or special positive-drainage filter media are also important. New construction of true rigid gravity seawalls is less common in the last 50 years (possibly because of costs), but many such structures that were previously built for shore protection are still in service today.

Pile-supported seawalls

138. Pile-supported seawalls and similar structures rely on deep-pile foundations for stability. Figures 6-1 through 6-3 of the SPM (1984) show examples of this type of seawall. Piles transfer the weight of the structure to deeper soil strata through axial compression and resist the overturning moments partially by bending and partially by a tension-compression couple. Although the portion of a pile-supported seawall exposed to wave action may appear very similar in cross section, materials, and construction to a gravity seawall, it should be emphasized that the pile structure is designed (or evaluated) in a fundamentally different manner.

139. Since pile-supported structures are subjected to bending moments and to shear at the pile connections, the pile caps and any vertical stiffening stems tend to be highly reinforced. The cross section can be relatively slender and high compared with gravity structures. However, the wall bases are frequently designed to be wider than would otherwise be necessary simply to allow for installation of multiple pile rows. By increasing the number and spacing of piles in the perpendicular direction, the unit load (especially in tension) on individual piles can be reduced. Smaller, shorter piles can be used, and installation costs tend to be less. This redundancy also increases the overall factor of safety.

140. It is actually possible for pile-supported structures to fail in the same four modes as gravity structures: overturning, sliding, bearing, or settlement. The use of piles, however, allows the designer to rely to a greater extent on the more predictable properties of the construction materials and to minimize design problems caused by the uncertainties in foundation soil response. Piles reduce the potential catastrophic effects of undermining typical in gravity walls. Other problems such as differential settlement and the difficulty of properly preparing foundation soils near the ground-water surface are also reduced.

141. As a practical result, reasonably designed pile-supported seawalls generally do not fail from monolithic overturning, settlement, or bearing. Instead, the more common problems are deterioration and damage to the structural connection between the piles and the wall base and loss of backfill. Both problems can contribute to slidinglike displacements and complete separation and overturning of the wall section above the piles. Toe scour protection similar to that used for gravity walls, and/or sheet-pile cutoff walls

are added to pile-supported seawalls to reduce exposure of the pile-to-wall connections and to prevent loss of backfill and upland property.

Anchored bulkhead

142. Anchored bulkhead refers to a large class of structures grouped by a generally common structural design approach, whether or not they are actually intended to function within the earlier definition of a bulkhead. These walls are typically constructed of some type of sheet piles driven into the soil and connected by a cap and possibly one or more stiffeners or wales placed within the plane of the wall.

143. A wall constructed only as described would actually be considered a cantilevered structure. The pile penetration would have to be sufficient to resist the entire lateral load by passive resistance in the soils below the bottom and by bending within the sheet piles. Since this limits the height of the wall for practical penetration depths and sheet-pile thicknesses, additional lateral resistance is usually provided by placing some type of anchorage system behind the bulkhead. The result, an anchored bulkhead, is probably the most commonly used retaining and shore protection structure in the United States. It is more typically designed to hold upland soil and not to serve as a flood-control structure. Figure D-1 in Engineer Manual 1110-2-1614 (USACE 1985) provides an example of this type of structure.

144. The two basic design approaches for determining the required depth of sheet-pile penetration are anchorage resistance and pile section properties. These approaches are termed free-earth support and fixed-earth support. The principal variations in method result from two different assumptions regarding the elastic deformation of the wall under loading. A number of references are available that describe the details of the several variations (e.g., Rowe (1952), Terzaghi (1954), US Steel Corporation (1975), NAVFAC (1981a), USACE (1983), and others). No further explanation of the design procedures or their variations will be provided here, except to note some specific characteristics of typical anchorage systems as part of the discussion on failure modes.

145. The sheet piles used in bulkheads vary in material and type. Engineer Manual 1110-2-1614 (USACE 1985) describes several typical materials and installation configurations.

146. Eckert and Callender (1987), citing work by Terzaghi (1954), Sowers and Sowers (1971), and others, note that there is a relatively high

frequency of failure of anchored bulkheads. The principal reasons for failure which they list include incorrect estimates of backfill loads, poorly designed anchorage systems, and construction procedures resulting in excessive earth pressures. The mode of failure from each of these problems would typically be outward rotation of the wall cap (i.e. away from the retained soil mass). Although this type of failure is undoubtedly the classic case for upland retaining walls and dredged/filled harbor quays, Clark (1986a, 1986b) suggests that the failure mode is quite different for coastal structures under wave attack.

147. Clark's suggestion will be examined more fully in later discussions of case studies. However, the potential explanation for such a view can be understood by briefly looking at the traditional manner in which anchorage systems are designed for upland installations. In these designs, anchors (e.g. "deadmen," batter piles, etc.) are placed in the upland soil mass a sufficient distance from the wall to be outside the zone of active pressure. These anchors are then connected to the wall by some type of tieback system. The tiebacks resist the tendency of the wall to rotate away from the anchors. Therefore, the tiebacks are intended to be loaded in tension and are frequently only slender steel rods or possibly cables. If properly designed, this type of anchorage will perform well to resist lateral earth pressure from behind the wall.

148. However, when a wall is subjected to large lateral pressures from waves on its face, the resultant thrust is toward the upland soil and the anchors. Initially the tendency of the wall to rotate landward is resisted by the soil mass. But, if soil support is lost, either from deformation in the soil or from direct washout of the backfill, the tiebacks are placed in compression, a function for which they are not typically designed. One resulting failure mode can occur under wave action with landward rotation of the wall and, in extreme cases, shear of the piles at the mudline.

149. Other problems frequently contribute to failures because of the unique nature of bulkhead design and installation. Scour can reduce the penetration of the sheet piles and lead to toe-out rotation failures. The cyclic loading from waves causes vibrations and nonuniform deflections of individual piles which can separate the pile-edge joints and damage the cap. Failure of the wall at one point can precipitate a raveling failure along greater lengths. Also, since the piles and caps are generally relatively thin

sections, any eventual corrosion, spalling, or similar deterioration often causes a serious reduction in the designed factor of safety against failure.

150. Anchored bulkheads are typically composed of sheet piles installed into the soil and connected in the parallel direction by a cap and wales and braced in the perpendicular direction by a tieback and anchorage system. It is possible that anchored "bulkheadlike" structures can be designed and constructed with material and section properties such that they might be classified as seawalls based on their functional ability to resist wave forces. This is not the usual case, however. Most conventional bulkheads are designed primarily as soil-retaining structures to resist lateral earth pressures from behind the wall. Wave forces load the structure in the opposite manner and occur during times when the resisting potential in the upland soil mass can be weakened by overtopping, seepage, and washout. As a result, bulkheads frequently fail when used in exposed coastal installations.

151. The term "dike" or "levee" refers to a large class of structures typically of a soil mass (clay or sand) covered by a hardened layer consisting of rock, interlocking concrete shapes, or asphalt underlain by a filter consisting of geotextile or graded rock. The front face of a dike is in essence a "revetment" and must be structurally designed with the same principles as a revetment, although dikes often have the additional property of an impermeable layer or core to prevent flooding for the upland area protected. The primary difference between a dike and a revetment is the fact that a revetment only defines the hardened front face of an earthen backfill whereas the dike refers to both the hardened face and the earthen mound supporting the hardened face.

152. As a practical result, reasonably designed dikes do not fail as would a massive concrete structure but are more susceptible to geotechnical type failures such as slip circle, wedge, or piping type failures within the earthen core material of the structure (see Thorn and Roberts (1981)). Additional modes of failure to consider in this type of structure are cover (hardened) layer failure resulting from uplift pressure and slope instability caused by wave action. Dike structures are common on the low coastlines of European countries, although they are not common in the United States.

153. Presently little guidance exists for design of dikes because of the complexity of adequately defining the geotechnical properties involved in their design and the complex nature of their failure modes. Although The Netherlands has been building dikes for centuries, there is still no adequate

design manual or procedure for dike design to resist wave activity as well as flood water. Present dike design in European countries exposed to severe wave activity is done in part using large physical modeling facilities for testing of geophysical as well as structural adequacy of dike components. In certain cases, though, the facing layer of a dike might be evaluated as would a revetment.

154. Some structures either do not fit precisely into any of the above categories or could be placed in more than one. The largest group of these difficult-to-classify structures is made up of various patented designs and components. The second major group consists of a number of combinations, composites, or variations of the basic structure types.

155. The majority of patented structural wall systems do not fit the basic structure-type descriptions almost by definition. They are deliberately designed compromises that attempt to capitalize on the desirable features of conventional designs while eliminating or reducing the undesirable features. The degree to which these compromises are successful varies considerably.

156. Two general criticisms of patented systems are often made: there is not enough prototype experience in most cases to draw supportable conclusions, and suppliers tend to claim universal applicability based on limited success in very specific installations.

157. The first situation is being addressed gradually by increasing numbers of test installations monitored by the manufacturers, regulatory agencies, and independent research organizations such as universities and CERC. An example is the Low Cost Shore Protection Demonstration Program conducted by the USACE (1981). This program included some type of assessment of over 20 proprietary structures or components. Unfortunately for the present discussion, most of these devices were installed in some type of revetment or offshore breakwater rather than as a seawall.

158. The second situation, claims of universal applicability, may continue as a criticism. It would be rare for a design engineer to claim that any one type of coastal flood-protection structure would be suitable for all sites or conditions. Yet, under marketing pressures, something close to that very claim is frequently made by the suppliers of patented devices.

159. In a number of cases, a particular nonproprietary structure may seem to fit into more than one of the structure-type categories, often because it is literally a combination of two or more structure-types. Examples

include cantilevered wall sections cast on top of bulkhead sheet piles, gravity walls underpinned with structural piles, or rubble mounds grouted so that they act structurally like rigid gravity walls.

160. Many times these combinations result from an existing structure's evolution over a series of repair or rehabilitation efforts. The point is simply that care must be used to evaluate such structures on the basis of how they react structurally to applied forces, rather than on their superficial appearance. This requires analysis on a case-by-case basis and relies on engineering judgment and experience.

161. Another way of describing a structure is by the material from which it is constructed. This approach potentially provides some information about the strength and durability of the structure. It can also suggest likely behavior and effects during failure. For example, because some materials are more ductile, structures built from them might offer a degree of residual protection during a prolonged progressive failure. In addition, certain materials may be more suitable, or at least less likely to require maintenance, under particular environmental effects such as ice, marine borers, or corrosion.

162. There is a problem, however, in considering the material properties without regard to the design or construction method in which they are used. Equally strong bulkheads might be constructed of timber, steel, aluminum, or concrete simply by choosing appropriate sections, thicknesses, and connection details. Some of these bulkheads might not be appropriate in all deterioration environments, and some may not be economical, but all could be constructed to equal design strengths. Each material has its own unique advantages and disadvantages, and it is not feasible to universally recommend one construction material over another.

163. Whatever material is selected, it is more important that the properties are accurately considered in the design process, it conforms to appropriate governing quality standards, and it is handled and installed properly during construction. Brief discussions of the materials commonly used in coastal protection and their applicable specifications are provided in the following paragraphs. The references cited, especially Moffatt and Nichol Engineers (1983), should be consulted for further information.

164. Structural timber has long been recognized as a viable and economical material for marine use. It is frequently used in sheet-pile bulkheads

and as cutoff walls for gravity seawalls. Round timber piles are probably the most common structural piles used to support seawalls and similar structures. Timber is also used as cribs and in other caissonlike installations.

165. Timber has a number of advantages as a construction material. It has a relatively high strength-to-weight ratio; can be readily field cut, shaped, and connected; is easy to repair or replace; and is generally available throughout the country. It is a flexible material and tends to be used in installations that are structurally flexible and redundant. As a result, timber structures can typically withstand considerable deformation and damage to individual members before complete collapse. Timber cannot, however, survive the extremely high unit stresses allowable in structural concrete or steel. Longevity can also be cited as a concern.

166. Marine borers are the principal cause of deterioration of timber structures in salt or brackish water. Generally, only timber that has been treated to resist borer attack should be used in these environments. The American Wood Preservers Bureau (AWPB) has established several standards defining the treatment chemicals, processing methods, and quality control criteria for treating timber to resist specific classes of exposure. For example, the AWPB Quality Mark MP-1 (dual treatment) specifies a treated timber suitable for use in areas of extreme borer hazard and in waters where both limnoria and pholad attack may be expected. Other AWPB standards are listed in the references. Additional information about timber treatment is contained in Naval Civil Engineering Laboratory (NCEL) (1978) and NAVFAC (1982b).

167. Timber can also be damaged by abrasion from ice, debris, and sand, and by direct impact (e.g., the case studies reported in NAVFAC (1981b)). These problems are usually addressed by jacketing individual round piles or by providing fender wales and/or toe revetments along bulkheads. Another common problem is damage during construction. Timber piles are particularly prone to splitting and "brushing" if not driven with care. Abrasion, impact, and construction handling cannot only damage the timber directly, but can reduce the effectiveness of the preserving treatment by making the damaged area more susceptible to borer intrusion.

168. Concrete is used in virtually every type of coastal structure including massive gravity-type seawalls. There are two principal forms of concrete: cast-in-place and precast (including prestressed). The design methods, reinforcement requirements, and construction techniques are quite

different for each form. Cast-in-place concrete was traditionally used for the monolithic gravity structures because of its mass and ability to be batched at the construction site. However, the increased costs of labor-intensive onsite forming and the concurrent advances in precast component quality and availability have lead to greater use of precast units. Precast components are now commonly used as structural piles, sheet-pile panels, tie-back beams, and even as recurved or stepped-face seawall stem sections.

169. Concrete is a good alternative where abrasion, corrosion, or marine-borer attack might limit the use of other materials. Studies have shown that conventional concrete made with Type I cement can perform satisfactorily in a saltwater environment without significant leaching or loss of section (e.g. by Portland Cement Association (PCA) and NAVFAC (1981b)). When failure does occur, it can usually be related to salt penetration into the concrete matrix and subsequent corrosion and spalling of the reinforcing steel. Two factors affect this type of failure: the depth of concrete cover over the reinforcing and the density (impermeability or watertightness) of the concrete mix.

170. The minimum depth of concrete covering any reinforcing is recommended to be from 3 in. (PCA 1979, NAVFAC 1981b) to 4 in. (Merritt 1976) in order to reduce salt intrusion. Ensuring that this minimum is achieved in the field is much more difficult than simply incorporating it into specifications. Problems frequently occur at points where the reinforcing is overlapped, tied to other members, or bent to change direction. The total amount of steel in the section at these areas is greater, and it tends to be pushed into the cover dimension.

171. A denser concrete mixture will also assist in reducing salt intrusion and reinforcement corrosion. This watertightness depends primarily on the mixing water-cement ratio in the paste (PCA 1979). Lower ratios are denser and more watertight. Since the compressive strength of concrete is also largely dependent on the same water-cement ratio, some references (e.g., NAVFAC 1981b) address the problem of permeability indirectly by simply specifying a higher strength concrete, 4000 psi, for exposure to salt water.

172. Structural steel is used in coastal protection structures in several ways, but principally as support piles. Steel's great unit strength makes it ideal for high capacity bearing piles that must be driven into dense sands or consolidated clays. These bearing piles are typically fabricated

from structural steel shapes or structural steel pipe sections. Steel sheet piles are also used in bulkheads, although somewhat less frequently on the open coast (because of a shorter service life) than in harbors or inland waterways.

173. One of the great advantages of steel, in addition to strength, is its availability in a wide range of standard shapes, alloys, and section properties applicable to almost any need. It is generally a very uniform and predictable material for which there exists considerable service data and design experience. Steel components are easily field cut, spliced, or reinforced to meet variable conditions. Typical steel structures can tolerate relatively large deformations before failing and continue to exhibit some residual strength during and after yielding.

174. Steel is probably the simplest material to evaluate for residual service strength and life in existing structures. If the basic steel grade is known, the remaining load-carrying capacity can be reasonably estimated from measurements of section geometry, thickness, and condition of any sections. Several American Society for Testing and Materials (ASTM) standards governing structural steel are listed in the references. Section properties and similar data are tabulated in handbooks such as American Institute of Steel Construction (AISC) (1980).

175. Experience has shown that conventional carbon steel exposed to the marine environment may have a limited service life if not protected from corrosion and otherwise inspected and maintained. The factors affecting corrosion of steel piling include geographical location, range of tidal or wave inundation, soil or other cover, abrasion conditions, and protective coating (NAVFAC 1981b, Moffatt and Nichol Engineers 1983). A number of studies (e.g., NAVFAC 1981a and 1981b) suggest that corrosion is a greater problem in the zone of cyclic wet-dry conditions, rather than below the permanent water level. As a result, bearing piles driven to below grade and capped are frequently not coated or are given only minimal protection. Conversely, sheet piles and other members extending above grade into the corrosive zone should be provided with some protective coating or other system (and periodically inspected). Moffatt and Nichol Engineers (1983), NAVFAC (1981b) and other references have extensive discussions of coatings and cathodic protection systems.

176. Rubble-mound seawalls can be constructed of rock, quarystone, riprap, and precast concrete armor units. These materials may be used singly or in combination, and they may be placed in a variety of cross sections ranging from a single uniform layer to graded multilayers. Design recommendations for these types of structures are provided in the SPM (1984) and other references. Those discussions will not be repeated here since, in general, the high void ratio and resulting permeability of typically designed rubble-mound structures would preclude them from consideration as coastal flood-protection structures. However, rubble-mound structures which have an impermeable section (either because of grouting and/or sheet-pile diaphragm) are exceptions to conventional designs that should be considered and evaluated.

177. The design or construction variations that are of interest are those that result in either low water and wave transmissibility and/or significant structural rigidity similar to gravity-type walls. These characteristics can be designed into the structure, such as specifying a core material with very low permeability in a multilayer cross section or placing sheet piles or a similar diaphragm as a barrier in the mound. The situation can also occur as previously mentioned through successive maintenance or rehabilitation attempts usually culminating in a structure composed of several materials finally grouted into a rigid mass.

178. In summary, this section has introduced descriptions of the principal shore protection structures likely to be considered for flood protection. Each of these structures can be described in more than one way using various classification systems. Three systems discussed were classification by functional intent, design or construction method, and construction material.

179. A functional description of a structure cites several performance objectives and associated design factors. The two objectives commonly stated for shore protection are intercepting wave action and retaining upland property. Similar structures may be distinguished by defining a principal function and establishing threshold criteria. In this way, for example, a seawall could be defined as a structure whose principal function is to resist the effects of waves with heights greater than 3 to 4 ft. The term "bulkhead" might refer to a structure that is designed principally to retain soil and is capable of withstanding direct attack from waves with heights less than 3 ft.

180. Since similar appearing structures could be classified differently by functional intent, additional descriptors are frequently included that group structures by common approaches in engineering design. These descriptions usually reflect the ways in which structures are supported, transmit or resist loads, and undergo deformations. Examples discussed included gravity-type walls, pile-supported walls, anchored bulkheads, and combination structures. Typical failure modes for each type of structure were noted. In addition, important common design considerations such as backfill selection and placement, drainage, and scour protection were introduced and will be considered more fully in subsequent sections.

181. A third consideration in describing coastal structures is the construction material. All of the common construction materials, timber, concrete, steel, and rock or stone are used in shore protection. Since each material has both advantages and disadvantages, no generalization can be made about a preferred material. Selection depends on availability, construction cost, and service life under the environmental conditions at a specific site. Examples governing specifications were cited for each material.

Case Histories During Storms

182. The preceding sections have described and summarized the functions, designs, and materials used in "idealized" coastal protection structures. However, it is important to look beyond the theoretical performance of a design or material and consider experiences with actual structures.

183. In this section, existing literature on seawall case histories is reviewed for illustrative examples of protective structure performance under storm conditions. In each case reviewed, emphasis was placed on those structural aspects that could be considered as representing "successes" or "failures" of a particular functional intent, design, or material. General recommended design features for acceptable structures will then be presented in a later section based on these examples and other experiences. The cases reviewed do not represent an exhaustive set of examples, but appear to be reasonably representative. The information presented about each case is the minimum necessary to illustrate a particular point. Further details on these structures can be found in the referenced literature. It should be noted that these reviews extend only to the performance of the structure in accomplishing

its protective goal and do not address any possible impacts or other problems that might be associated with hardening the shoreline. The topic of impacts to the beach or adjacent coastal areas is a subject with its own specialized literature (e.g., the Journal of Coastal Research, Special Issue No. 4, Autumn 1988).

184. Prior to presenting these summaries, some difficulties encountered in developing the case studies will be mentioned, since the problems associated with the research may be similar to those eventually faced by FEMA in evaluating existing coastal flood-protection structures. Difficulties were found in describing the type and assessing the condition of the structures reviewed and in characterizing the natural forces that had impacted them. In many cases there was simply little information available about the type of construction or details on the materials used. During construction of large or complex projects, field modifications to the originally designed, typical sections are common, and "as-built" plans are not always prepared. There is also a particular problem with large public works structures which have undergone repeated repair, modification, and expansion to the extent that the present structure may bear little resemblance to the original.

185. In almost no case was it possible to establish the details of the design conditions (water level, wave forces, scour, etc.) on which the structure was planned. Similarly, there are often little actual data available (or it is speculative) about the conditions associated with storms impacting a structure. The result is that meaningful or complete conclusions about the performance of a structure in comparison with its design intent are not always possible. However, some trends are evident, and the following summaries should be illustrative.

186. The Galveston, TX, seawall is one of the best known examples of a major shore protection and flood-control project. As described in references such as US Army Engineer Division (USAED), Galveston (1981), the wall began with a 3-mile-long segment constructed in 1902. The original wall was a composite structure consisting of a curved-face, reinforced concrete, gravity wall placed on untreated timber foundation piles. It included a wooden sheet-pile cutoff wall and riprap for toe-scour protection. Several modifications were made as a result of performance problems during storms (e.g. hurricanes in 1909 and 1915). Extensions were also constructed in 1920, 1926, and 1953 through 1962. The wall length now totals 10 miles.

187. The wall can generally be considered a successful structural design. There has been some long-term settlement because of consolidation of the foundation soils under the piles. There have also been repeated problems with scour, sinking of the riprap layer, exposure of the cutoff wall, and occasional loss of fill under and/or through the wall. However, there has never been any true structural failure of the wall, and the materials are in excellent condition, especially for the age of the construction.

188. The general performance of the Galveston wall can be considered a mixed success. In comparison with the devastation and loss of life in the city prior to construction, the wall has successfully functioned to offer a degree of protection to the upland. However, the wall has not been as successful, at least in certain key design features. In particular, there was a serious and recurring problem with overtopping of the wall, scour of fill behind the wall, and continued (although reduced) flooding.

189. The cap elevation was originally +17 ft (MLW), but a 1909 hurricane with a storm-surge elevation reported at +6 to +7 ft (MLW) caused overtopping and serious loss of fill behind the wall from runoff. The land behind the wall was subsequently raised a foot and stabilized with pavement over an area extending even farther landward. The 1914 hurricane was even more severe with a reported surge elevation of +14 ft (MLW) and combined wave action reaching to +21 ft (MLW). The result was continuous sheet flow over the wall 2 ft deep and serious scour of the landward embankment to a depth of 7 to 8 ft in some areas. The project required extensive repair including raising the land elevation again (to +21 ft) and installing a concrete sheet-pile bulkhead approximately 100 ft landward of the main wall to further contain the fill areas.

190. The beach erosion control project at Virginia Beach has included a number of structures and beach-fill sections. However, the principal structure is a 28-block-long concrete bulkhead built by local interests in 1927. The design of the wall, known typically as a king-pile or pile-and-panel wall, consists of precast rectangular concrete bearing piles installed on 14-ft centers with concrete slabs or panels set horizontally between the king-piles. A massive cast-in-place cap rests on the wall at elevation +11.5 ft National Geodetic Vertical Datum (NGVD) with a concrete promenade/splash apron extending landward for 20 ft. In addition, a sand berm was placed seaward of the

wall in 1952-53 at elevation +5.4 ft (NGVD) and has been periodically renourished to provide additional protection.

191. The Virginia Beach wall performance is particularly representative of experiences with large public works projects of the period. The project has been exposed to severe storm conditions and suffered major damage. However, in each case repairs and modifications have extended the service life of the structure well beyond its original intent.

192. Waves associated with northeasters have repeatedly damaged the panels forming the wall face. For example, in 1948 many panels were completely knocked out by a storm and buried, and 44 king-piles were sheared-off or otherwise severely damaged. The return period for this storm water level, based only on the height of combined surge and tide, was in the range of 2 to 5 years (USAED, Norfolk 1970). In a 1962 extratropical storm (the "Ash Wednesday" storm), 4,100 ft of wall was so badly damaged that it had to be completely rebuilt. Water levels in that storm were the second highest recorded in the area, +7.4 ft mean sea level (MSL), but represent only a 20-year return period water level (USAED, Norfolk 1970).

193. Loss of fill from behind the wall has been a recurring problem in these and lesser events. Also, scour along the face of the wall was severe during many storms. Beach profile modeling by CERC, described in USAED, Norfolk (1970), demonstrated the positive effects of the fill berm placed in 1953 in reducing even greater damage to the wall (and upland structures) in the 1962 storm.

194. Unusually complete information has been compiled (USAED, Norfolk 1970) about the storms impacting the Virginia Beach wall. For example, gage records and a storm hydrograph calculated for a 1933 hurricane documented a very rapid rise in still-water level. Although the water level reached over 8 ft (MSL), it rose so rapidly that waves had little time to erode or impact the base of the wall, and structural damage was minimized. On the other hand, considerable water overtopped the wall and deposited large quantities of sand on the upland area.

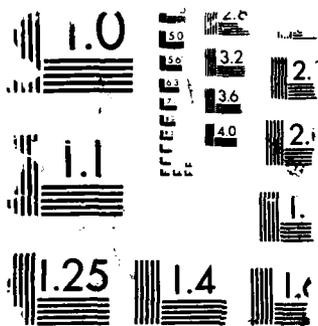
195. The maximum water level associated with the 1962 storm was +7.4 ft (MSL), but tides remained well above normal for 4 days. The overtopping during this storm was described as resulting from broken waves and surf being propelled over the cap by winds and following waves. Flooding up to 1 ft deep occurred behind the wall, but no direct wave transmission inland was reported.

196. In general, the Virginia Beach wall has successfully reduced upland flooding to only moderate overtopping during relatively severe storms. However, the major exceptions to this success have been in situations associated with structural failure of wall segments leading to more significant upland impacts. These seawall failures occurred during storm events which had associated return period water levels considerably less than 100 years.

197. The O'Shaughnessy Seawall in San Francisco, CA, built between 1915 and 1920, is another example of composite construction. As described by O'Shaughnessy (1924), the wall is basically a reinforced concrete stepped revetment supported at the toe on a concrete sheet-pile cutoff wall. The stepped section is then topped by a recurved concrete parapet wall also supported along the landward edge by pedestal foundations. The stepped section extends from approximately elevation +7 to +17 ft mean lower low water (MLLW), and the top of the recurve is at +27 ft (MLLW). Cross-walls, or returns, were constructed on 150-ft centers perpendicular to the main wall alignment to provide compartmentalization. A concrete splash apron placed above a foot-thick layer of impervious clay prevents seepage of any overspray landward of the wall.

198. The O'Shaughnessy Seawall is probably one of the more successful shore protection structures. It has never been significantly damaged, even under the effects of extreme storm events. The only problem with the structure has been an increasing deterioration of the concrete and corrosion of the underlying reinforcing steel. This problem is largely the result of abrasion of the concrete by coarse-grained sediments and the suspension of periodic repair and maintenance for the last 20 years because of funding shortages. The wall has performed its protective function equally well and has not been significantly overtopped, nor scoured. The structure's success is undoubtedly related to its sheer mass and size: over 12 tons per lineal foot and an elevation of +27 ft (MLLW).

199. Escoffier (1951) reported on his investigation of shore protection structures along approximately 40 miles of coastal Mississippi. His work focused on the performance of various designs of seawalls and bulkheads, particularly under the impact of a major hurricane in 1947. In his analysis, he grouped structures into nine types based on cross-sectional geometry. These groups will not be described in detail here, but they included several



variations of pile-supported walls, buttressed gravity walls, anchored bulkheads, and stepped and smooth face revetment-bulkhead combinations.

200. The 1947 storm had water levels ranging from +7.7 ft (MSL) in eastern Mississippi to +15.2 ft (MSL) at Bay Saint Louis. Associated waves were 6 to 8 ft high. Although the wall sections varied in elevation somewhat, generally the water levels were at, or above, the wall caps so that there was no direct breaking wave impact on the wall but considerable flooding behind the wall. The following discussion of comparative performance is summarized from Escoffier (1951).

201. The more massive wall sections, such as pile-supported recurved concrete walls and buttressed gravity walls, survived the storm with little or no damage. The anchored bulkheads, even those anchored to soldier piles, suffered frequent failures, which generally involved loss of soil from behind the wall, anchorage collapse, and subsequent wall rotation (seaward in some cases, and landward in others). In some reaches, the scour behind the wall was great enough to expose anchor piles to a length sufficient for them to then shear under cantilever loading. Prior corrosion of steel tieback rods was also noted as a major contribution to anchorage collapse and wall rotation. The stepped faced, sloping walls generally performed well and significantly better than similar smooth slab revetments. However, this was likely not the direct result of the slightly greater roughness, but because the stepped slabs had to be cast in thicker sections, almost twice as thick, to allow for the steps. Where the slab-type structures failed, they did so as a result of loss of backfill, differential settlement, cyclic stresses in the unsupported spans, and finally, cracking of the joints along the supports.

202. Fulton-Bennett and Griggs (1986) and Griggs and Fulton-Bennett (1988) have provided a comprehensive perspective on the effectiveness of coastal protection structures based largely on their examination of a variety of types of structures (including rock revetments) in central California. Their work was in much greater detail than can be described here, but they confirm a number of trends found in other case histories and suggest some general conclusions as follow.

203. The structures reviewed in Fulton-Bennett and Griggs (1986) and Griggs and Fulton-Bennett (1988) tended to be (with some important exceptions) designed and constructed within the last 25 years. During that period, rock revetments began to replace concrete and timber walls as the most common

protective structure in their study area. They concluded that this replacement was largely based on lower first costs for the revetments. However, they state that revetments required frequent maintenance as a result of settlement, scour, and destabilization of the slope associated with structure rotation. These same problems can all reoccur relatively quickly under storm attack with the result that the revetment may offer greatly reduced protection to the upland. Further, when revetments or similar rubble-mound structures do begin to fail, either from wave overtopping or flanking, the effect can be a very rapid, progressive collapse as reported, for example, by Smith and Chapman (1982).

204. Fulton-Bennett and Griggs (1986) and Griggs and Fulton-Bennett (1988) found that, in general, well-engineered concrete walls with adequate height to prevent overtopping and sufficient depth to tolerate scour provided the more effective protection, especially during the winter storms of 1983, which were noted to be very severe. The most common problems, other than toe scour, noted with the concrete walls in the California study area included loss of upland fill and foundation support. Piping of fill occurred through interlocking joints if the joints were not carefully grouted. The authors stress the benefits of a continuous, coherent approach to structure design and placement based on damage observed following the 1983 storms, during which an entire group of different, unconnected walls failed at one site and flanking problems were found at several other sites.

205. Pinellas County is located on the west-central gulf coast of Florida and includes the major population centers of St. Petersburg and Clearwater. The majority of the more than 30-mile-long shoreline is fronted by low-lying, intensively developed barrier islands. Vertical concrete bulkheads were the most common coastal protective structure in the county (Clark 1986a) with occasional reaches of restored protective beaches.

206. It is reasonable to state, based on the discussion by Clark (1986a), that most of the bulkheads (with some exceptions) in the county were older structures, inadequately designed for the exposed conditions which they increasingly faced as a result of continued long-term erosion. The walls had very low cap elevations (e.g. +6 ft (NGVD)), insufficient toe penetration for the eroded profile, and missing or inadequate return walls. Although these bulkheads may not be representative of proper, present state-of-the-art engineering, experiences with their performance are instructive in supporting

suggested, desirable features in any proposed structures. This type of wall is also representative of coastal protective structures which currently exist on much of the individual residential property along the Florida gulf coast.

207. Hurricane Elena passed offshore of the Florida gulf coast during 29 August through 1 September 1985 and particularly impacted the Pinellas County barrier islands. Tropical storm Juan passed through the western gulf between 26 and 31 October 1985, further damaging the west coast of Florida. Clark (1986a) summarized the coastal damage resulting from these storms (including over 150 photographs) and provides several general recommendations for design of proposed reconstruction.

208. Clark (1986a) reported that a total of approximately 17,600 linear ft of bulkheads in Pinellas County were destroyed or damaged to the extent of requiring major reconstruction as a result of the two storms. The report concludes that the most significant problems shared by the damaged walls were a lack of toe-scour protection and absence of positive measures to contain upland fill. Specific comparisons are noted between the survival of walls with toe-scour protection and the collapse of immediately adjacent walls without such protection (e.g. Figures 97-100 in Clark (1986a)).

209. Clark's (1986a) discussion of the second problem, loss of upland fill and its effect on structure survivability, parallels the description of failure modes for anchored bulkheads described previously in this report. He confirms that overtopping waves removed large quantities of upland sediment, leaving the tiebacks exposed, which then failed in compression; thus, wall sheet piles were left unsupported and incapable of withstanding impact loads. Figures 132 and 134, among others, in Clark (1986a) clearly demonstrate this situation and show sheet piles sheared at the "mud" line and knocked landward, rather than in a more conventionally supposed toe-out rotational soil failure. His recommendations for proposed new structures include carefully containerizing the upland fill by using geotextile filters, adequate return walls at each end and at intermediate intervals along longer, continuous walls and by avoiding any fill other than clean sand with high permeability. He also suggests that tiebacks be encased in concrete to resist corrosion and compressive loads.

210. Reviews of other cases suggest very similar experiences with shore protection structures in other areas of the country. Oertel, Fowler, and Pope (1985) discuss the history of erosion control measures at Tybee Island,

Georgia. Essentially all of the 3.5 mile-long Atlantic shoreline of Tybee Island had been armored with a succession of seawalls, bulkheads, and revetments dating to the early 1900s. Segments of the various walls failed repeatedly and were repaired or replaced until the area received an initial beach restoration project in the 1970s and subsequent renourishments. There were exceptions, but Oertel, Fowler, and Pope (1985) note that the typical succession of structures was from short lifespan steel sheet-pile bulkheads; to concrete structures that were more durable, but continued to suffer from toe scour; to bulkheads with rock revetments for toe protection.

211. Clark (1986b) also described the impacts to the Florida panhandle associated with hurricane Kate in November 1985. This section of Florida did not have shore armoring structures to nearly the extent of Pinellas County, but the damage to those that did exist was similar to that previously discussed. A total of approximately 2,000 ft of bulkheads was destroyed in the area. A 1,275-ft section of sloping concrete slab revetment was destroyed in Port St. Joe, FL, during this storm. Water levels during the storm were on the order of +8 ft (NGVD), reaching to +9 to +10 ft (NGVD) where wave action was significant.

212. The Town of Palm Beach, FL, recently completed a comprehensive reevaluation of shore protection structures along the town's Atlantic shoreline.* The structures in the area include a number of designs, construction periods, and private, group, and public projects. However, one predominant structure is a 6,000-plus-ft steel sheet-pile wall built in 1929. Experiences with this structure are similar to many other large public works projects of the period in that, through maintenance and repairs, it has far outlived its original design life and may now be inadequate for present exposure conditions. Structural problems began to appear in the wall in the 1950s, and extensive repairs, including a concrete (gunite) overlay, were made at that time. Additional repairs, new tiebacks, and replacement of some entire segments are now proposed. One major problem with the existing structure is inadequate toe penetration (or conversely, scour protection). The original

* Coastal Planning & Engineering, Inc., 1987, "Town of Palm Beach Roadway Protective Seawall Evaluation," unpublished letter report to the Town of Palm Beach, FL, Boca Raton, FL.

Cubit Engineering Ltd., 1986, "Town of Palm Beach Roadway Protective Seawall Repairs Design Study Evaluation," unpublished letter report to the Town of Palm Beach, FL, Charleston, SC.

design was based on a fairly stable sand berm along the wall face at the time of construction at approximately elevation +5 ft (MSL) with piles penetrating to -8 to -12 ft (MSL). Since that time, the profile has lowered to the extent that elevations frequently drop to 0 to -1 ft (NGVD) during even minor storms and spring tides, leaving the piles underpenetrated and the tiebacks overstressed. Substantial corrosion of the sheet-pile section and the tieback rods was also documented; the section modulus of the sheet piles has been reduced by 43 percent, and the tie-rod cross-sectional area has been reduced by 48 to 58 percent.*

213. This section has presented very brief summaries of the structural and functional performance of several case history shore protection structures or regional groups of structures. The case histories reviewed have not presented a significantly different view of shore protective structure design from that used in the present state-of-the-art coastal engineering practice, and the review has confirmed the importance of certain basic design features. Those features will be discussed further in a subsequent section.

Recommendation of Design Features for Acceptable Structures

214. Previous sections of this report have reviewed the basic engineering properties of the materials commonly used in coastal protection structures, the factors upon which such structure designs are based, the theoretical failure modes for these structures, and experiences with actual projects reported in the literature. The results have strongly confirmed the importance of certain basic design features that should be a part of any acceptable structure. Those features are summarized in this section.

215. It is clear to the authors of this report and supported by the case histories previously reviewed that proper consideration of toe scour and overtopping (resulting in loss of upland) are the two most significant features of successful shore protection structures. In almost every case of failure, including those in which the construction materials themselves actually failed, the underlying cause was inadequately anticipating the degree and effect of profile lowering along the wall face and/or the loss of fill behind the structure. Appendix C discusses "state-of-the-art" design scour criteria

* Cubit Engineering Ltd., op. cit.

(in the absence of toe protection), while previous sections have discussed runup and consequent overtopping.

216. Obviously, structures under evaluation must be investigated to ensure that foundation piles or wall sheet piles have a minimum embedment sufficient to tolerate the expected scour at the site (in the absence of toe protection). Appendix C notes that the maximum scour design criterion at the base of a structure (in the absence of toe protection) should be the maximum wave height at that location. The more successful structures will also include positive provisions for directly controlling that potential scour along the face. Such provisions provide an additional factor of safety under design conditions and reduce the effects of more severe conditions. The most common approach has been placement of a toe revetment, and considerable guidance exists for designing, or evaluating the design, of scour revetments and aprons (e.g. SPM (1984), USACE (1985), Eckert and Callender (1987), and others). Other projects have relied on providing an overfilled profile or storm berm seaward of a structure to erode sacrificially and reduce scour effects.

217. The case histories show that either approach can be effective, but both frequently result in high maintenance requirements to replace lost rock or counter the effects of revetment settlement, or to replenish the berm volume. The evaluation of an existing structure should, therefore, probably focus on the additional protection offered by the scour treatment in a condition something less than the original full design section since the degree of maintenance cannot be guaranteed.

218. Similar to the case of scour, evaluating a structure for overtopping would begin by investigating its elevation relative to expected water levels, the area bathymetry and the wall's position on the profile, and certain features of its cross-sectional geometry. However, the evaluation should then expand to include positive provisions to control the effects of overtopping, especially the loss of backfill. This is a desirable feature of a well-designed coastal flood-protection structure, even though the structure has been designed not to be overtopped during the design storm.

219. Case histories and other experiences (see especially Richart and Schmertmann (1958), Clark (1986a), or Fulton-Bennett and Griggs (1986)) have shown that backfill material can be lost seaward by rundown over the structure cap, landward by drainage of flood water, or laterally around inadequate return walls. Sediment can also be piped through joints or seams in the

structure, through drainage openings (weep holes, storm sewers, etc.), under the toe, and through direct breaches in the wall's face. The condition of the backfill is further affected by ground-water levels elevated by precipitation and/or tidal response. Each of these paths must be investigated, and each usually requires a combination of solutions to be fully effective.

220. For example, grouting or otherwise sealing joints should be considered a minimum provision against piping. However, grouting alone is usually not fully effective because of deterioration, differential movements at the joints, or inadequate initial depth of the seal. A filter medium such as a geotextile fabric, preformed drain, or graded stone filter should be provided behind the wall to reduce the hydrostatic gradient and isolate the fill from the joint areas. In addition, the more successful structures will also include surface treatments landward of the wall cap such as positive drainage, stabilized splash aprons, and/or impermeable layers to reduce the volume of water entering the backfill. Experience has confirmed that this type of redundancy is necessary to provide an improved factor of safety and control damage when it occurs. Unfortunately, there is no methodology established that would lead to a good check for adequate drainage for a coastal flood-control structure. This is a complex problem requiring good engineering judgment on a case-by-case approach.

221. As a further measure toward successful performance, the basic plan alignment of a shore protection project should present a continuous structure. Isolated, individual walls or revetments along staggered alignments have not performed nearly as well. Waves forces tend to concentrate on the multiple corners or be diffracted into pocket areas; reflected energy can strike adjacent return wall sections producing greater damage; and the failure of one structure may contribute to failures in adjacent structures if they do not have adequate return walls or otherwise depend on each other for fill containment. Examples of these situations are discussed by Fulton-Bennett and Griggs (1986) at their "Site No. 26," by Clark (1986b) (e.g. Figures 7, 8, and 11), and by Clark (1986a) (e.g. Figures 66, 67, 94, and 95). As Clark (1986a) notes, even structures with coherent, contiguous alignments should also incorporate redundant return walls at frequent intervals. These intermediate return walls compartmentalize the upland fill and help ensure that breaching or other failure at one point will be confined and not ravel into larger sections.

222. It is difficult to exclude, a priori, a particular type of construction material or design geometry as completely unacceptable from a stability standpoint, although numerous failures of bulkheads and similar nonmassive structures during storms with water levels having return periods less than 100 years may be sufficient evidence for FEMA to decide a priori that such structures will not be considered for flood credit. It is clear that some materials and designs have generally performed better than others. Revetments alone, for example, have not proved particularly useful for protection against flooding (although they can be used in combination with upland walls). Slabs, panels, sheet piles, and similar thin-section, "many-pieced" structures have typically been structurally damaged more frequently than monolithic (massive concrete) sections (e.g. failures at Virginia Beach and Pinellas County versus little structural damage at Galveston or San Francisco). Properly reinforced and proportioned concrete appears to be more durable than steel for coastal exposures, although this durability can depend on the degree of initial protection and subsequent maintenance given to each (see, for example, NAVFAC 1981b; Moffatt and Nichol Engineers (1983); and previously discussed case histories at San Francisco and the Town of Palm Beach).

223. Based on case histories and the previous discussion in this report, a model of a typically successful coastal flood-protection structure might be a composite design structure with a relatively massive concrete gravity-type wall section, but supported on a pile foundation and protected by a scour revetment and/or sheet-pile cutoff toe wall. It would also be provided with positive measures to control seepage and loss of backfill and be regularly inspected, maintained, and repaired. Other materials or geometries might certainly be shown to be equally acceptable, if the basic design features discussed were adequately addressed. Unfortunately, most nonmassive seawalls invariably experience failure to some degree in a major storm because of some design inadequacy. It can be noted that the "model" seawall described is rarely built today, and if it were, it would likely be prohibitively expensive* (Clark 1986a, Fulton-Bennett and Griggs 1986).

* Coastal Planning & Engineering, Inc., op. cit.

Summary and Recommendation

224. There are four primary types of coastal flood-protection structures from a functional standpoint: gravity seawalls, pile-supported seawalls, anchored bulkheads, and dikes (also referred to as "levees"). Typically a coastal structure may contain composite features of one or more of these four broad classes of structure. Properly designed and maintained, any of these coastal flood-protection structures can survive a 100-year return period storm and remain functionally intact (in theory). In practice though, anchored bulkheads have numerous instances of failure in severe coastal storms. It is recommended that as a general policy, FEMA not consider anchored bulkheads for flood-protection credit because of extensive failures of anchored bulkheads during large storms and difficulty in checking the present conditions of anchored bulkheads to resist design storms.

PART VII: EVALUATING EXISTING COASTAL FLOOD-PROTECTION STRUCTURES

Flowchart/Outline Approach

225. In summary, a flowchart/outline approach will be presented which a qualified coastal engineer might use to evaluate the adequacy of a particular existing coastal flood-protection structure. This evaluation should be conducted as an independent, in-depth engineering review of the design, rather than simply verification of the original designer's certification. Such a review would need to be made by applicants to FEMA requiring consideration of protective benefits offered by their coastal flood-control structure. Not surprisingly, the basic information and approach required for such a review is similar to that which is necessary for the original design process.

226. This flowchart is presented as Figure 21 and incorporates criteria similar to information used in design procedures outlined in USACE (1985), NAVFAC (1981a), and Allsop (1986). For the in-depth structural stability aspects of the coastal flood-control structure, it is assumed that a certified professional engineer with coastal engineering experience would be required to do the analysis. The burden of such an analysis should be put on the person applying for reduced insurance costs.

227. The flowchart in Figure 21 may be used, as follows, to:

- a. Determine the cross section of a structure and cap elevation along with water-level range and wave conditions for the site; prestorm profile at the site; and bathymetry/topography at the site.
 - (1) Water levels would range from a maximum of 100-year return period water level to low water level at the site.
 - (2) Waves for the site might be determined as depth-limited breaking waves in accordance with present FEMA criteria.
- b. Evaluate wave runup on a structure to determine if runup is above the crest of the structure (functional failure) or below the crest of the structure. Runup could be calculated via Appendix A methodology and adjusted for roughness of the structure.
- c. Collect design details on a seawall; cross-section properties; history of any structural damage or failures, maintenance, modifications, etc.; record of performance in various large storms (i.e. overtopping or breaching of the structure, toe failure due to scour, etc.). Also collect soil information (laboratory tests, borings, etc.) to determine soil properties. If the structure has experienced failure from storms

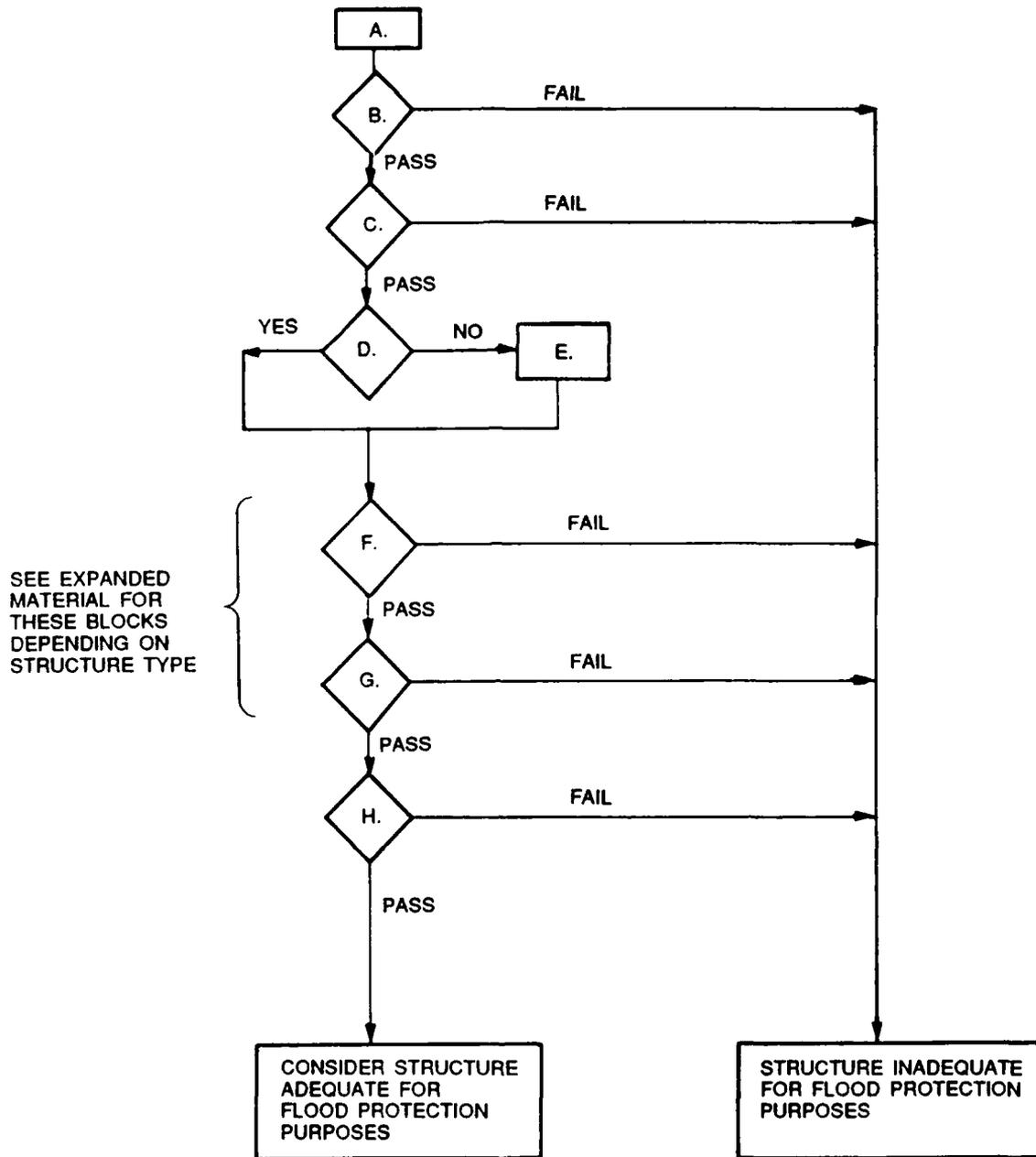


Figure 21. Flowchart/outline for structure evaluation

less than design storm (100-year storm), then failure under design storm should be assumed.

- d. Check for adequacy of toe protection in front of the structure to determine potential for scour development (see USACE (1985), Eckert and Callender (1987)).
- e. Evaluate scour potential in front of the structure in accordance with Appendix C.
- f. Evaluate structure stability for minimum seaward water level, no wave height, and saturated soil conditions behind the structure. Stability check is to be made with or without

scour (step e) depending on adequacy of toe protection (step d).

- (1) Required checks for dikes/revetments. Geotechnical checks are to be made for potential dike failure in landward direction by rotational gravity slip of dike (see Terzaghi and Peck (1967), Sowers and Sowers (1970), Department of Navy (1971), Thorn and Roberts (1981), Spangler and Handy (1982)).
 - (2) Required checks for gravity/pile-supported seawall.
 - (a) Check for sliding toward sea (assume rigid foundation).
 - (b) Check for overturning toward sea (assume rigid foundation).
 - (c) Check adequacy of foundation based on maximum pressure developed in sliding and overturning calculations (see Terzaghi and Peck (1967), Sowers and Sowers (1970), Winterkorn and Fang (1975), Department of Navy (1971), Spangler and Handy (1982)).
 - (3) Required checks for anchored bulkheads.
 - (a) Check for shear failure in bulkhead.
 - (b) Check for moment failure in bulkhead.
 - (c) Check adequacy of tiebacks to resist tension load.
 - (d) Check adequacy of deadman size and allowable soil pressure to resist deadman loadings. (See Walton and Sensabaugh (1979) for guidelines on these checks).
- g. Evaluate structure stability for critical seaward water level (which might be any water level from minimum to the maximum 100-year return period water level) to include hydrostatic and hydrodynamic (wave) loading on the seaward side of the structure (with the assumption of soil deformation (i.e. no soil pressure) on the landward side in the case of seawalls or bulkheads). Use a condition of scour in front of the structure if inadequate toe protection is provided.
- (1) Required checks for dikes/revetment.
 - (a) Geotechnical checks are to be made for potential dike failure in seaward direction by rotational gravity slip of dike (see Terzaghi and Peck (1967), Sowers and Sowers (1970), Department of Navy (1971), Thorn and Roberts (1981), Spangler and Handy (1982)) or by failure of the foundation because of inadequate bearing strength under wave and water loading.
 - (b) Check revetment stability (if applicable) for the following items:
 - o Stability of rock/riprap/armor blocks to wave action (via procedures given in Chapter 7, of the

SPM (1984), the section on stability of rubble structures and material in EM 1110-2-1614 (USACE 1985), on revetment design in Zanen (1978), or Ahrens (1981).

- o Toe stability of structures via material given in SPM (1984), Chapter 7; EM 1110-2-1614 (USACE 1985); Pilarczyk (1987); and Eckert and Callender (1987).
- o Uplift forces on blocks or impervious surfaces via material given in Delft University Press (1984) or Pilarczyk (1987).
- o Adequacy of geotechnical filter via material provided in Calhoun (1972) and USACE (1977).
- o Adequacy of graded rock filter via material provided in Ahrens (1975a), Engineer Manual 1110-2-1913 (USACE 1978), and Zanen (1978).

(2) Checks for gravity seawall/pile-supported seawall.

- (a) Check for sliding toward land (assume rigid foundation).
- (b) Check for overturning toward land (assume rigid foundation).
- (c) Check adequacy of foundation based on maximum bearing pressures developed in sliding and overturning calculations (see Terzaghi and Peck (1967), Sowers and Sowers (1970), Department of Navy (1971), Winterkorn and Fang (1975), and Spangler and Handy (1982)).

A reasonable example calculation for sliding and overturning is given in Chapter 16 of Delft University of Technology (1976) for a caisson with water on both sides and in Chapter 8 of the SPM (USACE 1977). This basic approach can be used for a gravity seawall only with water level at the seawall base on the landward side.

(3) Required checks for anchored bulkhead.

- (a) Check for shear failure in bulkhead.
- (b) Check for moment failure in bulkhead.

Assume tiebacks do not provide support in these calculations.

- h. Check for material adequacy (i.e., has expected life of material been exceeded?).
 - (1) See US Steel Corporation (1975) and AISC (1980) for evaluating adequacy of steel in structure.

- (2) See AWPB (periodically updated) for evaluating adequacy of wood in structure.
- (3) See ASTM (periodically updated) for evaluating all material standards.
- (4) See Moffatt and Nichol Engineers (1983) for general guidance on construction materials in the coastal zone.

Summary

228. A flowchart approach to check adequacy of the primary types of coastal flood-protection structures to survive the 100-year return period storm is provided. References to information necessary to do detailed checking of structures are provided where they exist.

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APPENDIX A: MAXIMUM PERIODIC WAVE RUNUP ON SMOOTH SLOPES

1. A robust approach to data presentation is given here for the case of short monochromatic waves on smooth surface sloping structures with an intent to unify various existing theories and approaches for wave runup and to provide a reasonable means to calculate an upper limit of runup. The reanalysis of periodic wave runup data provided here shows the raw data points in a new format. An additional variant in the present reanalysis of runup data is to provide wave runup in terms of wave height at the structure toe depth as opposed to using the deepwater wave height. The advantage of using transformed wave height rather than deepwater wave height is that wave height transformation uncertainty from deep water to the structure site becomes a separate problem, uncoupled from the problem of runup on the structure caused by a given wave condition at the toe of the structure. Although it is recognized that the type (shape) of wave existing at the site is important to the ultimate problem (i.e., the transformation prior to the structure and the ultimate runup are not entirely uncoupled), the presentation of data in terms of wave conditions at the base of the slope should be of benefit in dealing with various types of nearshore bathymetry. Since wave period is considered invariant throughout the transformation process, deepwater wavelength is still used in the analysis.

2. The data sources for this runup reanalysis are from earlier tests at the Coastal Engineering Research Center (CERC) on smooth slope runup. These data are discussed in length by Saville (1956)* and Savage (1958). For purposes of clarification, a short discussion of this data set follows. Further information on these tests can be found in Saville (1956) and Savage (1958).

3. The test procedure involved placing a smooth surface plywood test slope in the end of the wave tank and propagating periodic waves of known characteristics toward the slope. The waves in each test were measured after the initial unsteady wave transients died down but prior to rereflection of waves from the wave generator. An average of 6 to 15 waves were visually measured by reading the runup on a scale marked on the face of the slope to the nearest hundredth of a foot in vertical elevation. In all the data presented in this paper, the water depth was constant (≈ 0.38 m). Saville (1956) noted

* See References at the end of the main text.

that varying the water depth at the toe of the structure had a negligible effect on the relative wave runup when the water depth at the toe of the structures was equal to or greater than three times the deepwater wave height.

4. Wave characteristics were determined by calibrating the wave generator for the 0.38-m water depth. The generator was calibrated by placing a wave absorber in the beach end of the tank and generating a wave train with known and reproducible settings on the generator. The average height of the wave train so generated was measured with a parallel wire gage at 2-m intervals along the tank beginning near the wave generator. Wave heights were plotted versus distance along the tank, and the wave height value obtained from a smooth curve drawn through the points at a distance coinciding with the toe of the test slope was interpolated as the wave height value for that particular generator setting and structure slope. Using the wave height at the structure toe, water depth, and wave period, deepwater wave height was computed from linear wave theory via an inverse transformation. Original runup results were plotted using the deepwater wave height rather than the measured wave height. Wave periods for the test data ranged from 0.72 to 5.00 sec while wave height ranged from 0.01 to 0.19 m.

5. A listing of the data test conditions is provided in Table A1.

Table A1
Summary of Test Conditions

<u>Structure Slope</u>	<u>Wave Height cm</u>	<u>Wave Period sec</u>	<u>R* cm</u>	<u>H/d</u>	<u>d/L_o</u>	<u>Number of Observations</u>
vertical	0.9-12.2	0.72-4.71	1.3-15.5	0.024-0.321	0.011-0.471	26
1 on 0.5	0.9-18.6	0.72-4.71	1.1-37.2	0.025-0.488	0.011-0.471	33
1 on 1.0	0.9-18.6	0.72-4.71	0.9-42.8	0.022-0.488	0.011-0.471	32
1 on 1.5	1.2-17.1	0.72-4.71	1.2-39.6	0.030-0.448	0.011-0.658	45
1 on 2.25	0.9-17.4	0.72-4.71	1.5-45.8	0.028-0.456	0.011-0.658	51
1 on 3	0.9-17.7	0.72-4.71	2.4-48.4	0.023-0.464	0.011-0.658	49
1 on 4	0.9-18.3	0.72-4.71	2.3-47.5	0.028-0.480	0.011-0.658	51
1 on 6	0.8-17.8	0.72-4.71	1.8-37.2	0.021-0.468	0.011-0.658	51
1 on 10	0.5-12.1	0.72-4.71	0.8-8.9	0.013-0.480	0.011-0.471	28

* R = runup; H = wave height at toe of structure; d = water depth at toe of structure; L_o = deepwater wavelength.

Analysis Procedure

6. As in all approaches to evaluating laboratory data, there are two methods of determining the important dimensionless groupings of variables for data presentation. A fundamental method for obtaining important dimensionless groupings of parameters often used where the physics of the processes are unknown or not well understood is the Buckingham Pi method (e.g., White (1979)). This method or a variation of it has been used in various studies of runup (e.g., Technical Advisory Committee on Protection Against Inundation (1974)). The resulting dimensionless groupings of runup variables for the case of normal wave incidence and linear smooth slope structures of unknown functional form is as follows:

$$\frac{R}{H_o} = f\left(\frac{H}{L_o}, \frac{H}{d}, \frac{\rho H^3}{\sigma T^2}, \frac{\rho H^2}{\mu T}, \theta\right) \quad (A1)$$

where

- R - runup
- H - wave height at toe of structure
- L_o - deepwater wavelength
- d - water depth at toe of structure
- ρ - density of fluid
- σ - surface tension
- μ - dynamic viscosity
- θ - structure slope

The third dimensionless grouping on the right-hand side of Equation A1 is the Weber Number for oscillatory flow, which is of negligible importance for models of reasonable size. The fourth dimensionless grouping on the right-hand side of Equation A1 is the Reynolds Number for oscillatory flow. Projected effects of Reynolds Number are unknown in the present data set. Actual scale effect in runup studies has been investigated by various authors (Saville 1958, Fuhrboter 1986), but results to date have been insufficient to define such effects well.

7. Of the remaining three parameters in Equation A1, Iribarren and Nogales (1949) first noted the importance of the combination of wave steepness

(H/L_o) and structure slope (θ) in one parameter, the Iribarren number $\tan(\theta)/(H/L_o)^{1/2}$. Various researchers (Galvin 1972, Battjes 1974b, Hunt 1959) have noted the importance of this parameter in both the breaking process and resulting runup on beaches. For mild slope structures, Hunt (1959) recognized that the relative runup was proportional to the Iribarren number under breaking wave conditions. Battjes (1974b) provides a physical explanation for the relationship between runup and the Iribarren number for the case of mild slope structures. A limitation to this equation can be seen for the case of steep slope structures where the Iribarren number approaches infinity. As it is desired to provide a unified approach for wave runup on both steep and mild slope structures, a $\sin(\theta)$ term was used in place of the $\tan(\theta)$ of the Iribarren number; hence this modified dimensionless grouping will be referred to as the modified Iribarren number. It should be noted that a slight refinement of Battjes' (1974b) arguments (i.e., considering wavelength defined along the slope rather than in the horizontal plane) will lead to the $\sin(\theta)$ term used here. This modification of the Iribarren number is consistent with various criteria for delineation of the zone between breaking and nonbreaking. For example, Munk and Wimbush (1969) provide an expression based on linear wave theory for breaking on a slope in which the downslope component of the particle acceleration cannot exceed $g \sin(\theta)$. As Battjes (1974b) noted, with proper accounting of the reflected wave height, the Munk and Wimbush (1969) criteria can be written as:

$$\left[\frac{\sin(\theta)}{\left(\frac{H}{L_o}\right)^{1/2}} \right]_c = (2\pi)^{1/2} \tag{A2}$$

where the subscript c refers to incipient breaking. Miche (1951) using linear waver theory also derived a kinematic criterion for the limiting conditions of nonbreaking on a plane slope extending to deep water. His criterion is given by:

$$\left(\frac{H_o}{L_o}\right)_c = \left[\frac{\sin^2(\theta)}{\pi} \right] \left(\frac{2\theta}{\pi}\right)^{1/2} \quad \text{for } \theta \leq \frac{\pi}{4} \tag{A3}$$

This expression can be reformulated as:

$$\left[\frac{\sin(\theta)}{\left(\frac{H}{L_o}\right)^{1/2}} \right]_c = \left(\frac{\pi}{K_s}\right)^{1/2} \left(\frac{\pi}{2\theta}\right)^{1/4} \quad (A4)$$

where K_s is the linear shoaling coefficient if one assumes the expression good for all depths. Keller (1961) finds a similar expression for limiting conditions for nonbreaking where:

$$\left(\frac{H}{L_o}\right)_c = K_s \left(\frac{\theta^2}{2\pi}\right) \left(\frac{2\theta}{\pi}\right)^{1/2} \quad (A5)$$

Equation A5 can be rewritten as:

$$\left[\frac{\theta}{\left(\frac{H}{L_o}\right)^{1/2}} \right]_c = \left(\frac{2\pi}{K_s}\right)^{1/2} \left(\frac{\pi}{2\theta}\right)^{1/4} \quad (A6)$$

Keller's expression is based on a nonlinear shallow-water theory and therefore might be considered more valid than the expression of Miche. For small slopes $\theta = \sin(\theta)$; hence Keller's criterion, Equation A6, is within a constant of Miche's expression, Equation A4. For large slopes (i.e., limiting case $\theta = \pi/2$), the factor $\sin(\theta)$ differs from θ by 50 percent.

8. An additional dimensionless grouping d/L_o can be formed by dividing the first right-hand side parameter grouping by the second parameter grouping. This grouping of parameters has the advantage of delineating the relative water depth in which the structure is situated. An important parameter for free surface flows not explicitly mentioned is the Froude number, which for oscillatory flows in deep water can be represented as the multiplication of the first two right-hand side groupings in Equation A1.

9. A second method of obtaining dimensionless variable groupings of importance is by casting the physical equations into dimensionless form. The basic equations of fluid dynamics would point out the importance of the

Reynolds number, Froude number, and Weber number as before. To obtain further groupings of importance, a direct look at physical equations for runup because of nonbreaking waves is called for.

10. Koh and Le Méhauté (1966) have suggested a runup equation for sloped structures of the form:

$$\frac{R}{H} = \left(\frac{\pi}{2\theta}\right)^{1/2} + \left(\frac{\pi H}{L}\right) \left[\frac{1}{\tanh(kd)}\right] \left[1 + 3/4 \sinh^2(kd) - 1/4 \cosh^2(kd)\right] \quad (A7)$$

The first term is based on an earlier linear expression derived by Miche (1951) for deepwater conditions, while the following terms are based on Miche's (1951) approximation to nonbreaking runup on vertical walls for non-linear wave theory. Except for a missing linear shoaling coefficient, the first part of this expression agrees with that of Keller (1961), which was derived for nonplane beds with nonsteep slopes.

11. Keller and Keller (1965) derived an expression for the case of a plane slope and horizontal bottom using linear long wave theory with the result:

$$\frac{R}{H} = \left\{ J_0 \left[\frac{(k_o d)^{1/2}}{\theta} \right] + J_1 \left[\frac{(k_o d)^{1/2}}{\theta} \right] \right\}^{-1/2} \quad (A8)$$

where J_0 and J_1 are Bessel functions of the first kind of zero and first order, respectively.

12. In both Equations A7 and A8 for nonbreaking wave runup, the relative runup is seen to be of the functional form:

$$\frac{R}{H} = f\left(\theta, \frac{d}{L_o}, \frac{H}{L_o}\right) \quad (A9)$$

The grouping of the parameters on the right side of Equation A9 can also be combined and reexpressed as before:

$$\frac{R}{H} = f \left[\frac{\theta}{\left(\frac{H}{L_o} \right)^{1/2}}, \frac{d}{L_o} \right] \quad (A10)$$

where the first dimensionless grouping is of a similar form to the modified Iribarren number used previously.

13. In an attempt to unify the presentation of relative runup on smooth slopes, the primary independent variable of importance was chosen to be the modified Iribarren number.

Results

14. Relative runup plots for nine slopes ranging from vertical to 1 on 10 are presented in Figures A1 through A9. The modified Hunt (1959) expression is:

$$\frac{R}{H} = \left[\frac{\sin(\theta)}{\frac{H}{L_o}} \right]^{1/2} \quad (A11)$$

The expected criterion for breaking waves given by Equation A11 is superimposed on the plots along with an upper limit expression of the Miche-Keller form (Equations A2 and A4) for nonbreaking wave limit. The expression for this upper limit found to be most consistent with the data is given by:

$$\frac{R}{H} = (2\pi)^{1/2} \left(\frac{\pi}{2\theta} \right)^{1/4} \quad (A12)$$

If the shoaling coefficient K_s is assumed to be unity, this expression is consistent with Equation A4 (except for a constant (= 2.0)) and with Equation A6 (using $\sin(\theta)$ rather than θ). Table A2 presents this relative runup upper limit versus structure slope for the slopes investigated in this study.

15. The rationalization for this approach to maximum wave runup is that within the realm of breaking waves (on the slope) the relative runup should follow the modified Iribarren number. As the modified Iribarren number is

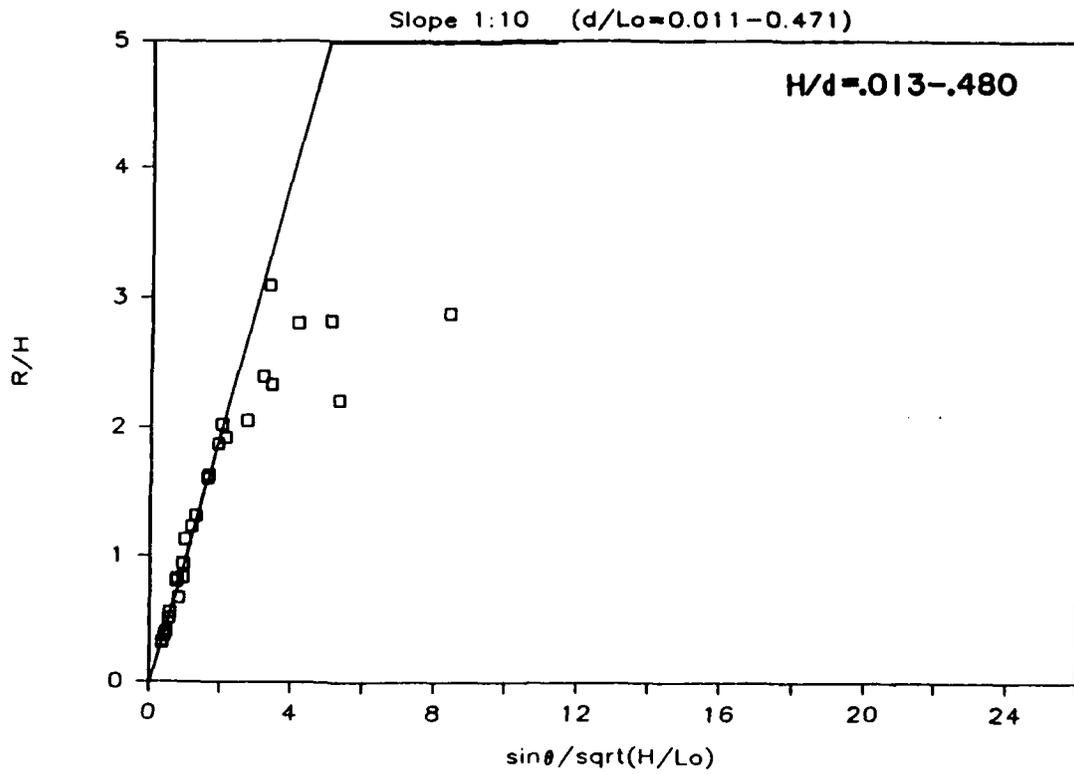


Figure A1. Relative runup versus modified Iribarren number, slope 1:10

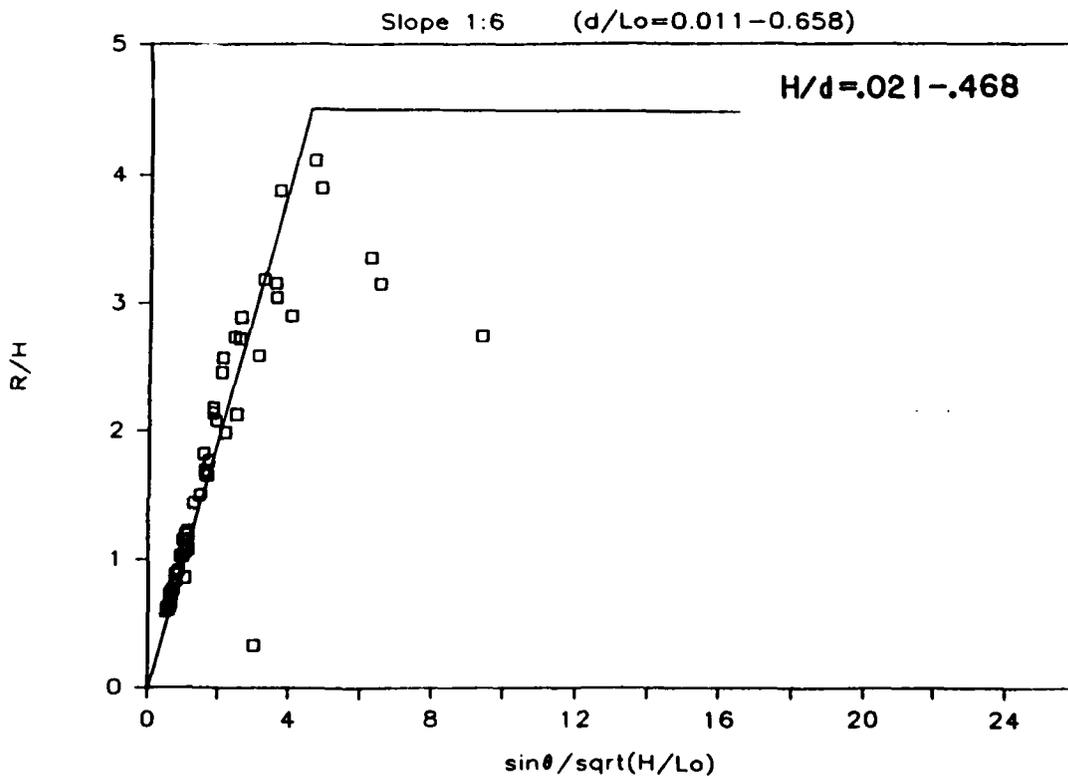


Figure A2. Relative runup versus modified Iribarren number, slope 1:6

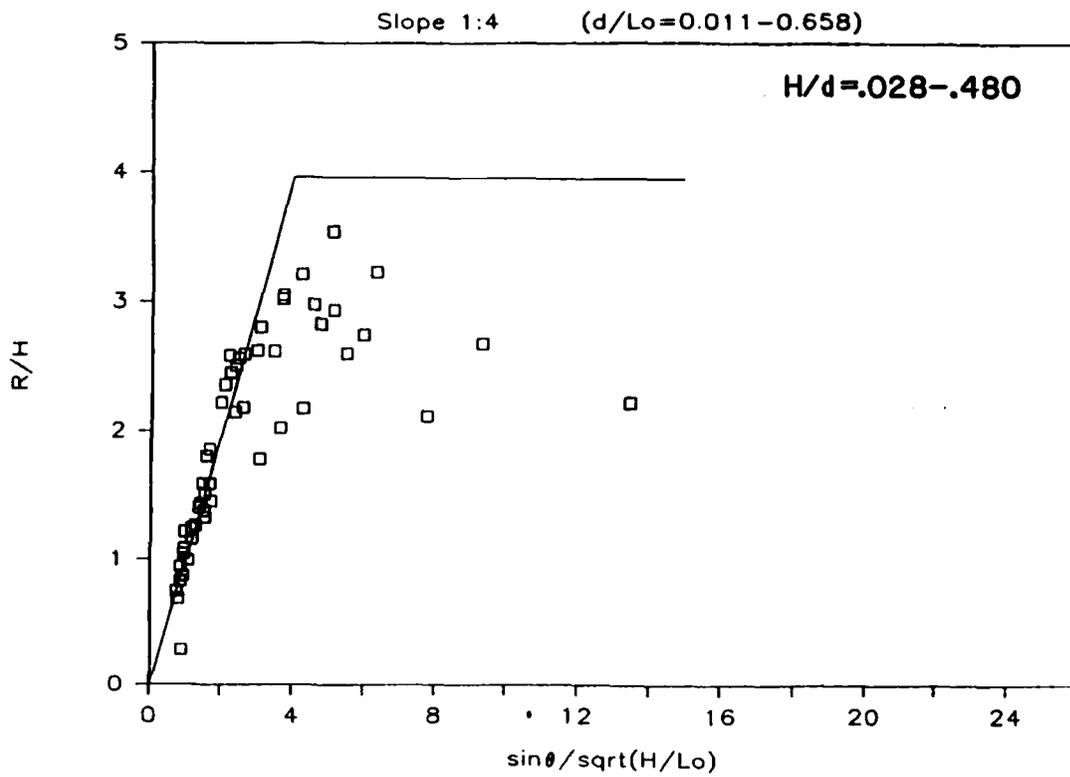


Figure A3. Relative runup versus modified Iribarren number, slope 1:4

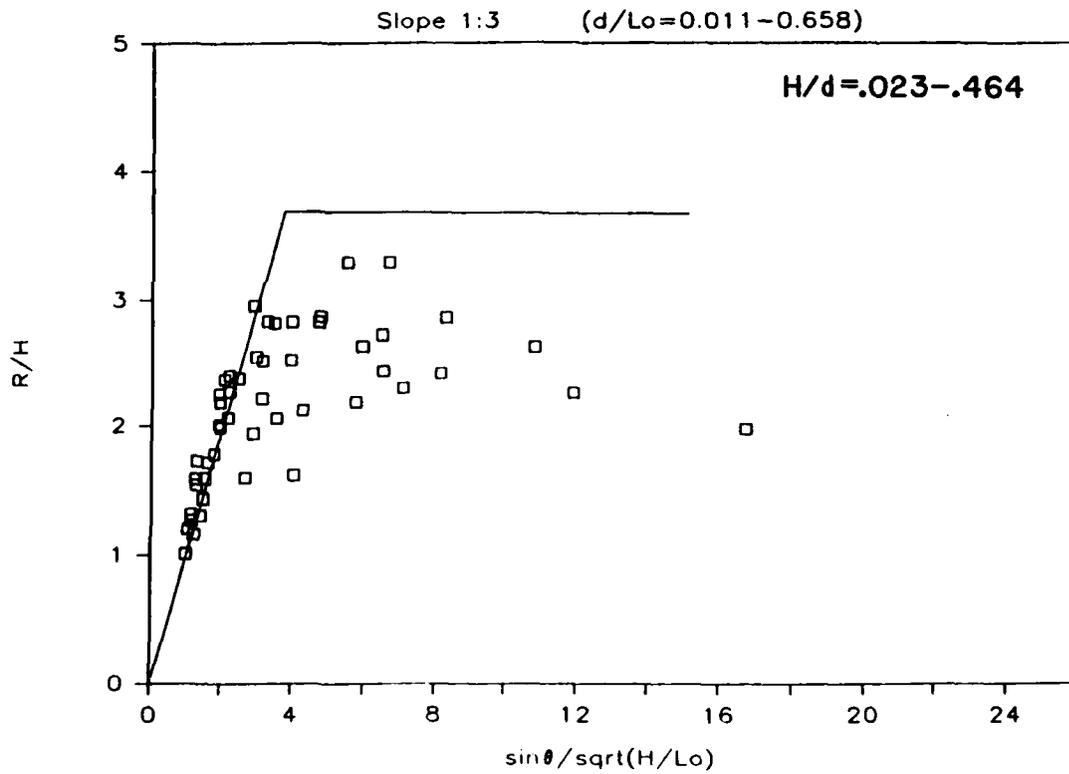


Figure A4. Relative runup versus modified Iribarren number, slope 1:3

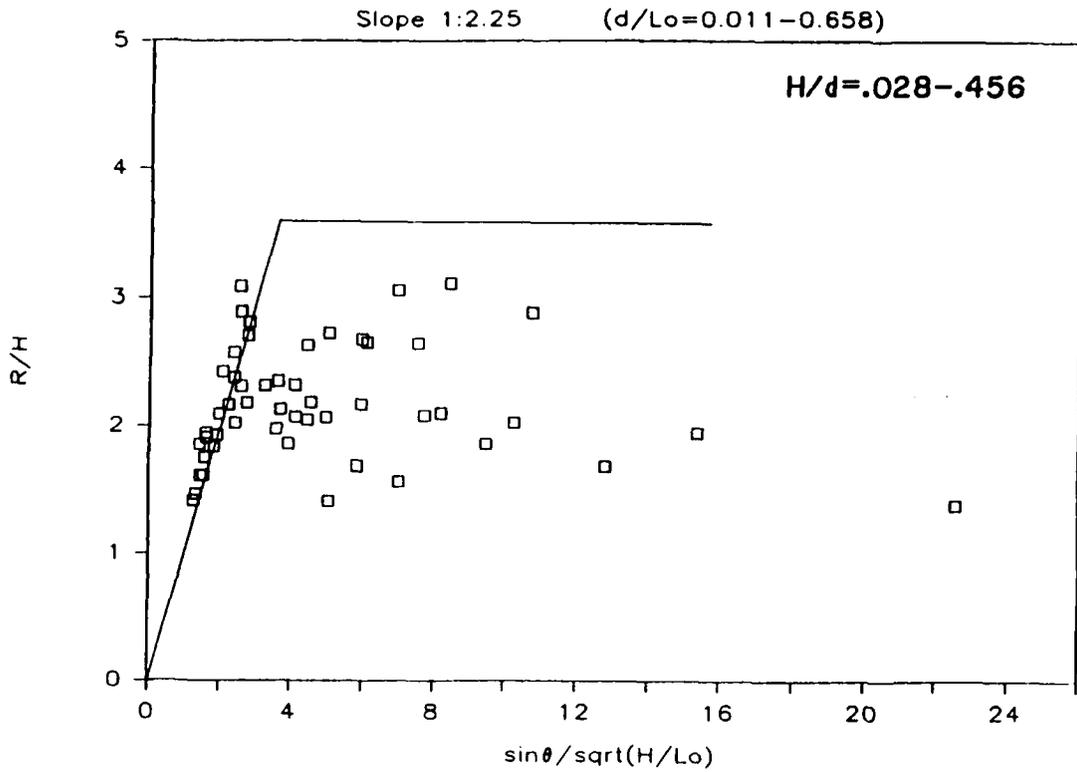


Figure A5. Relative runup versus modified Iribarren number, slope 1:2.25

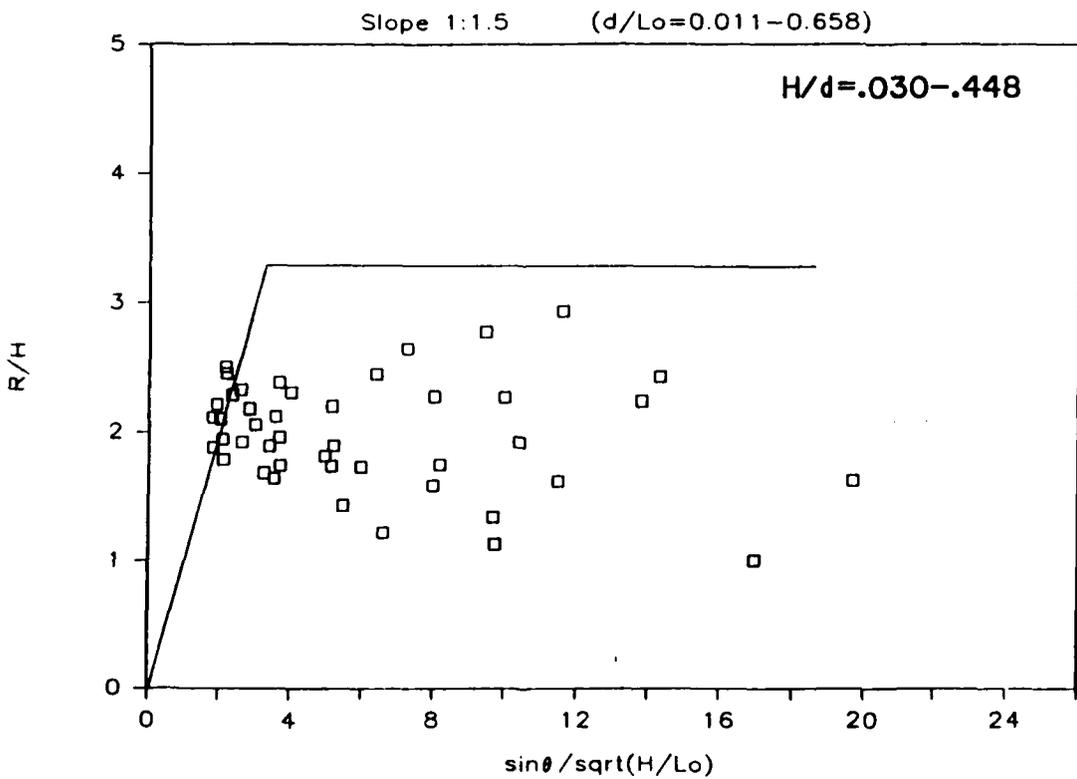


Figure A6. Relative runup versus modified Iribarren number, slope 1:1.5

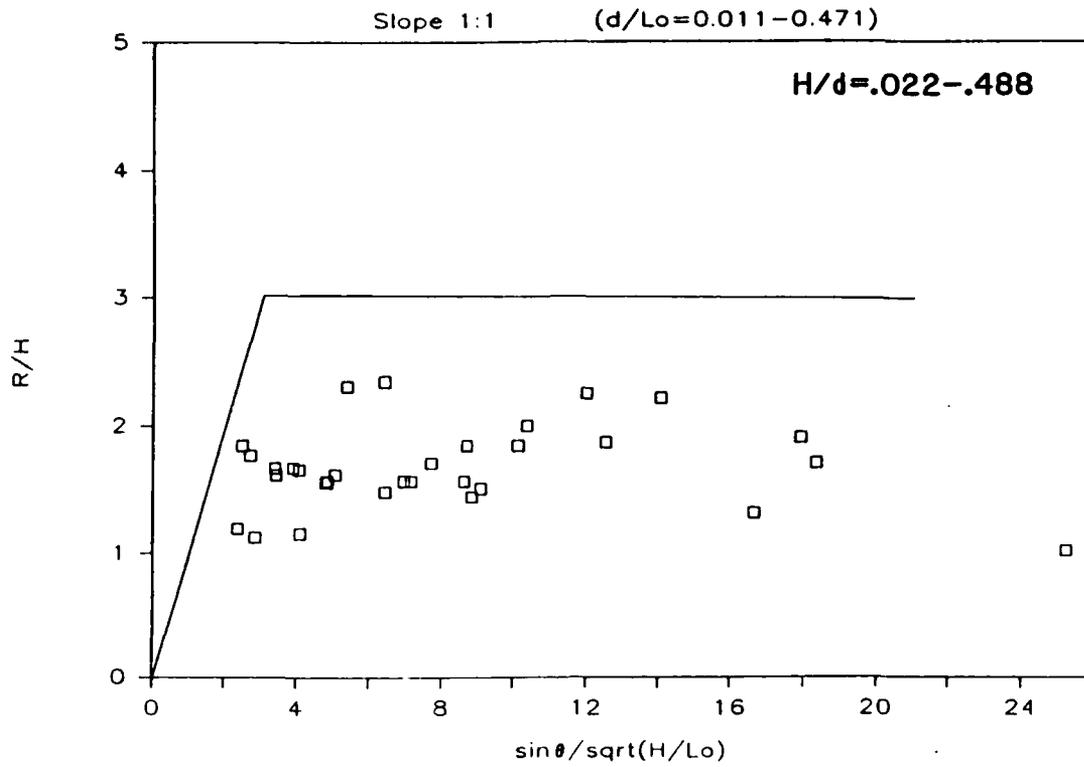


Figure A7. Relative runup versus modified Iribarren number, slope 1:1

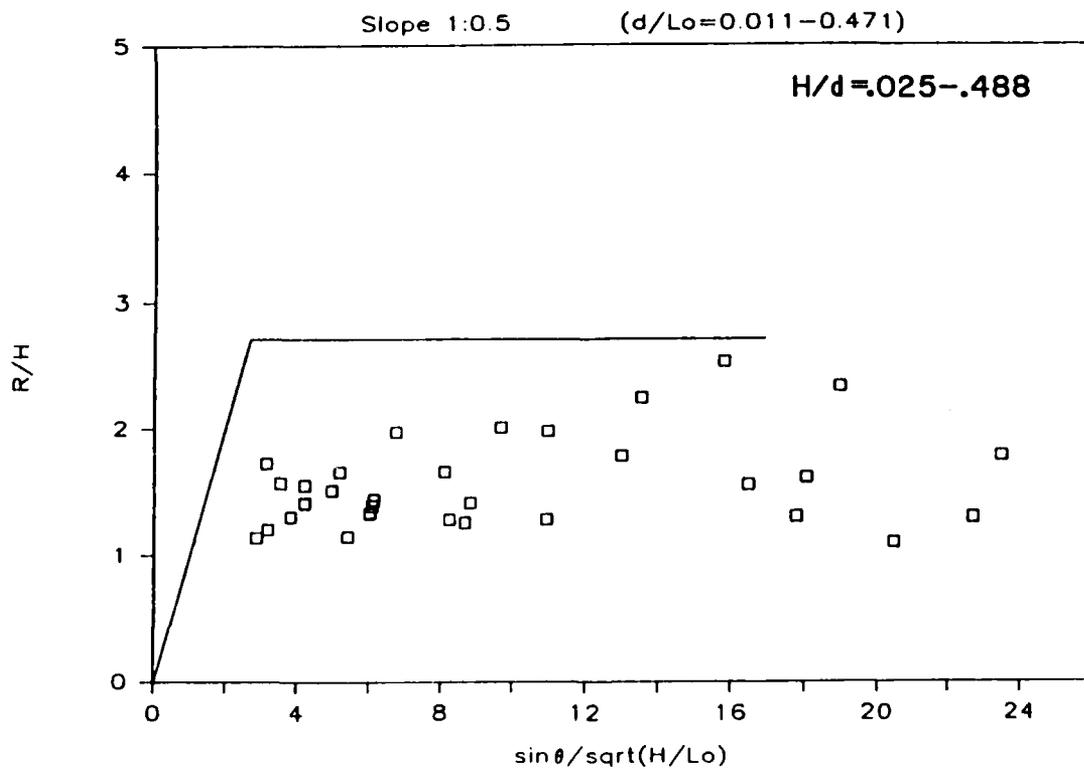


Figure A8. Relative runup versus modified Iribarren number, slope 1:0.5

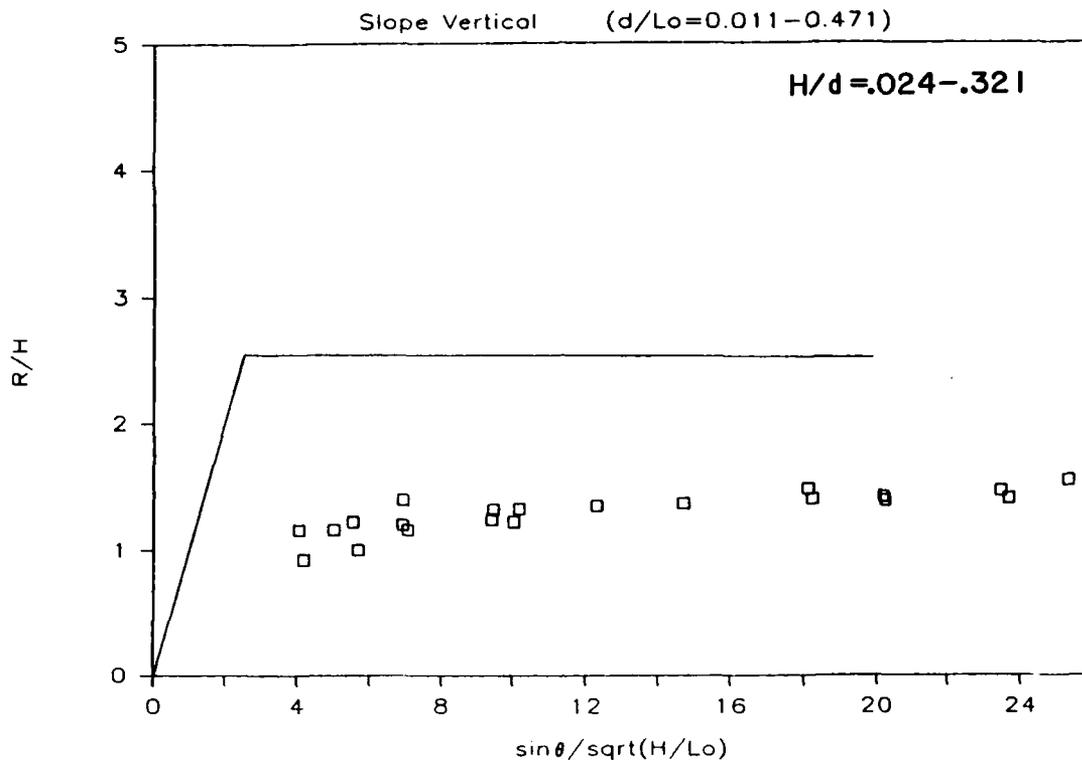


Figure A9. Relative runup versus modified Iribarren number, slope vertical

Table A2

Maximum Relative Runup Versus Structure Slope

<u>Slope</u>	<u>R/H (Equation A12)</u>
Vertical	2.50
1:0.5	2.73
1:1	2.97
1:1.5	3.19
1:2.25	3.49
1:3	3.70
1:4	3.95
1:6	4.39
1:10	5.00

increased and enters the zone of nonbreaking, the relative runup should decrease; therefore limits provided should envelope runup data on the conservative side.

16. In all cases, the data in the breaking zone portion of the curve follow the modified Iribarren number relationship well. Additionally, the critical transition point for nonbreaking describes well the upper limit of the relative runup except for the 1 on 10 and vertical slope where it overestimates. The reason for this overestimation on the 1 on 10 slope is not entirely known but believed to be due in part to the greater effect of friction as slope gets milder and the consequent opportunity for a viscous boundary layer to develop. As typical coastal flood-control structures have slopes steeper than 1 on 10, this discrepancy should not be a major concern.

17. The upper limit for relative runup on a smooth vertical slope according to Equation A12 is:

$$\frac{R}{H} = (2\pi)^{1/2} = 2.5 \quad (\text{A13})$$

A simple periodic linear standing wave on a vertical slope would produce an expression with relative runup $R/H = 1.0$. Wallace (1964) provided a method to numerically calculate the runup for solitary waves (which can be considered to be a limiting case of periodic waves in shallow water). For vertical walls with $H/d > 0.15$, his method produces an estimated $R/H = 2.5$ in accord with the above proposed criterion. A value of relative runup $R/H = 2.5$ on vertical waves is also consistent with the laboratory findings of Takada (1974) and with the original recommended value of $R/H = 3.0$ proposed by Hunt (1959) for surging (nonbreaking) waves.

18. Although graphs have not been included here, the approach has been used on one set of data with a 1 on 3 slope in 29.5-cm water depth and is consistent with the results provided herein.

APPENDIX B: PRELIMINARY METHOD FOR COMPUTATION OF
WAVE OVERTOPPING AND WAVE TRANSMISSION

Methodology

1. Several methods were investigated to evaluate their applicability for predicting overtopped wave heights. With the exception of the method selected, these included equating that component of candidate overtopped variables to the same variable in the transmitted wave. These candidate variables included the following: volume flux, momentum flux, and energy flux. The portion of the variable in the incident wave crest lying above the structure crest was equated to the quantity in the transmitted wave, thereby determining its height.

2. These evaluations established the desirability of using a nonlinear wave and the necessity of accounting for a localized enhancement of the incident wave through reflection or convergence for a sloping structure.

3. The method presented here equates the volumetric flow over the coastal flood-protection structure to that under the crest phase of the transmitted waves. As described later, the volumetric flow rate, q_2 , per unit length of structure is calculated by either a critical flow weir equation or a submerged flow weir equation, depending on the relative submergence of the structure crest. Referring to Figure B1, the empirically adopted criterion for the type of weir flow formula is

$$\text{Submerged Weir Flow Equation: } \frac{F}{H_i} < 0.5 \quad (B1)$$

$$\text{Critical Flow Equation: } \frac{F}{H_i} \geq 0.5$$

The equations for each of these two flow conditions are discussed below.

Submerged Weir Flow
Equation: $F/H_i < 0.5$

4. For this case the head driving the flow is $\eta_1 - \eta_2$ and the flow area through which the flow occurs is $[(\eta_1 + \eta_2)/2 - F]$, where η_1, η_2 = wave water level above still-water level seaward and shoreward of structure,

respectively. Expressing the Bernoulli equation between 1 and 2 (Figure B1),

$$\eta_1 + \frac{v_1^2}{2g} = \eta_2 + \frac{v_2^2}{2g} + \text{Losses} \quad (\text{B2})$$

Neglecting the approach velocity head ($v_1^2/2g$) and losses, the velocity, v_2 , over the structure crest is

$$v_2 = \sqrt{2g(\eta_1 - \eta_2)} \quad (\text{B3})$$

The discharge per unit width, q , is

$$q_2 = v_2 \left(\frac{\eta_1 + \eta_2}{2} - F \right) = \sqrt{2g(\eta_1 - \eta_2)} \left(\frac{\eta_1 + \eta_2}{2} - F \right) \quad (\text{B4})$$

The water level η_1 is affected by the incident, reflected, and transmitted waves.

5. To determine η_2 , it is noted that the instantaneous water surface displacement is related to the water transport, q_2 , by linear water wave theory as

$$\eta_2 = \frac{q_2}{C_2} \quad (\text{B5})$$

where C_2 is the downwave celerity. Without overtopping, the water level η_1 would be the result of enhancement of the incident wave by reflection and in the case of a sloping structure, a convergence effect. The reflected wave η_r will be reduced by the overtopping volume and can be calculated similar to the transmitted wave as,

$$\eta_r = \eta_i - \frac{q_2}{C_1} \quad (\text{B6})$$

where

C_1 = wave celerity at section 1 on the upwave side

In the absence of convergence against the upwave side of a sloping structure, η_1 would be the superposition of the incident and reflected waves. However the convergence (Figure B1) causes an additional potential flow enhancement as documented by runup, $R(0^\circ)$, on a sloping structure. To account for the reflection and convergence effects, a potential runup, $R(0^\circ)$, is defined which is the value that would occur in the absence of overtopping. The actual upwave water surface elevation is reduced by the overtopping in accordance with Equation B6.

$$\eta_1(\theta) = R(0^\circ) \frac{\eta_i(\theta)}{\eta_i(0^\circ)} - U \frac{R(0^\circ)}{\eta_i(0^\circ)} \frac{q_2(\theta)}{C_1} \quad (B7)$$

where θ is the wave phase angle and U is an empirical constant. It is seen that in the case of no overtopping ($q_2 = 0$), the maximum water surface elevation is $R(0^\circ)$ at the wave crest and for other phases, and the runup has the same shape as the water surface displacement of the incident wave. For the case of overtopping, the upwave water surface displacement is reduced by the second term. In the second term, the factor $R(0^\circ)/\eta_i(0^\circ)$ represents a first approximation of the local effects of convergence and the shape of the structures. The quantity U is an empirical constant to be determined from data. The constant U is expected to be nearly unity, and if it were unity, the upwave elevation would be reduced by a factor the same as the enhancement factor occurring to the incident wave in the case of no overtopping.

6. The downwave local water surface displacement, η_2 , is based on Equation B5 with the same empirical constant, U

$$\eta_2(\theta) = U \frac{R(0^\circ)}{\eta_i(0^\circ)} \frac{q_2(\theta)}{C_2} \quad (B8)$$

Equations B4, B7, and B8 may now be combined to yield

$$q_2(\theta) = \sqrt{2g \frac{R(0^\circ)H_i}{\eta_i(0^\circ)} \left[\frac{\eta_i(\theta)}{H_i} - \frac{Uq_2(\theta)}{C_1H_i} \left(1 + \frac{C_1}{C_2} \right) \right]} \times \left\{ \frac{R(0^\circ)H_i}{2\eta_i(0^\circ)} \left[\frac{\eta_i(\theta)}{H_i} - \frac{Uq_2(\theta)}{C_1H_i} \left(1 - \frac{C_1}{C_2} \right) \right] - F \right\} \quad (B9)$$

This can be expressed in nondimensional form as

$$q_2'(\theta) = \frac{1}{2} \frac{\sqrt{2gH_i}}{C_i} \sqrt{\frac{R'(0^\circ)}{\eta_i'(0^\circ)} \left[\eta_i'(\theta) - Uq_2'(\theta) \left(1 + \frac{C_1}{C_2} \right) \right]} \times \left\{ \frac{R'(0^\circ)}{\eta_i'(0^\circ)} \left[\eta_i'(\theta) - Uq_2'(\theta) \left(1 - \frac{C_1}{C_2} \right) \right] - 2 F' \right\} \quad (B10)$$

where

q_2' = dimensionless discharge over structure

R' = dimensionless runup

η_i' = dimensionless water surface elevation

F' = dimensionless freeboard

where

$$R' = \frac{R}{H_i}$$

$$\eta_i'(\theta) = \frac{\eta_i(\theta)}{H_i}$$

$$q_2'(\theta) = \frac{q_2(\theta)}{C_1H_i}$$

$$F' = \frac{F}{H_i}$$

(B11)

7. With the discharges per unit width $q_2(\theta)$ over the structure now determined, it remains to establish the transmitted wave height, H_t . The volumetric flux per wave, V_t , under the crest phase positions for a linear wave, is

$$V_t = \frac{H_t}{2} \frac{T}{\pi} C_2 \quad (B12)$$

Equating V_t and the overtopping volume, H_t is found as

$$H_t = H_i \frac{C_1}{C_2} \int q_2'(\theta) d\theta \quad (B13)$$

where, as noted previously, the integration occurs over the phase angles of positive transport. Simpson's rule was used to carry out the numerical integration of $q_2(\theta)$, which was calculated at phase angle increments, $\Delta\theta$, of 0.1745 radians (= 10 deg).

Critical Flow

Equation: $F/H_i \geq 0.5$

8. The critical flow discharge over sharp- and broad-crested weirs can be derived through application of the Bernoulli equation between the upstream section and a section over the weir (e.g., Streeter and Wylie (1979)*, pp 357-361). The results are

$$\text{Sharp-Crested: } q_2 = \frac{2}{3} \sqrt{2g} (\eta_1 - F)^{3/2} \quad (B14)$$

$$\text{Broad-Crested: } q_2 = \sqrt{2g} \left(\frac{2}{3} \eta_1 - F \right)^{3/2} \quad (B15)$$

For purposes here, the sharp-crested weir equation is considered valid with an "efficiency factor" of 0.62 determined for steady-state (Streeter and Wylie (1979), p 358).

* See References at the end of the main text.

9. Using the reduction in reflected wave, the convergence enhancement factor, and the reduction in upstream water surface elevation due to overtopping, the discharge equation can be expressed as

$$q_2(\theta) = 0.62 \frac{2}{3} \sqrt{2g} \left[R(0^\circ) \frac{\eta_i(\theta)}{\eta_i(0^\circ)} - U \frac{R(0^\circ)}{\eta_i(0^\circ)} \frac{q_2(\theta)}{C_1} - F \right]^{3/2} \quad (B16)$$

In dimensionless form, Equation B16 becomes

$$q_2' = 0.62 \frac{2}{3} \frac{\sqrt{2gH_i}}{C_1} \left\{ \frac{R'(0^\circ)}{\eta_i'(0^\circ)} \left[\eta_i'(\theta) - Uq_2'(\theta) \right] - F' \right\}^{3/2} \quad (B17)$$

As for the case of the submerged weir flow, Equation B17 is solved by iteration for all phase angles for which the term inside the brackets is positive. With the positive discharge determined, the transmitted wave height is established by Equation B13.

10. Results of applying and evaluating this method against two fairly extensive sets of laboratory measurements are presented in the next section.

Results

11. Lamarre (1967) carried out a fairly extensive set of laboratory measurements for a breakwater with side slopes of 1:1.5. A total of six wave periods were investigated; for each period, various combinations of structure crest elevations and wave heights were tested. For each test, the incident and transmitted wave heights were measured. The water depth for all 200 tests was 1.5 ft. Table B1 summarizes the test results of Lamarre, and Figure B2 portrays the dimensionless wave characteristics.

12. In the computations, stream function wave profiles for Cases 6-A, 7-A, and 8-A (USACE 1974) were used depending on the relative water depth d/L_0 (Figure B2), as follows:

$$\text{Case 6-A, } \frac{d}{L_0} < 0.15$$

$$\text{Case 7-A, } 0.15 \leq \frac{d}{L_o} \leq 0.35$$

$$\text{Case 8-A, } \frac{d}{L_o} > 0.35$$

Table B1
Summary of Characteristics of Lamarre Laboratory Experiments (Total
of 200 Tests, Water Depth = 1.5 ft. Side Slopes = 1:1.5)

Wave Period T, sec	Relative Structure Crest Elevation, F/H_i		Incident Wave Height H_i	
	No. of Values	Range	No. of Values	Range, ft
0.8	9	-2.022 to 1.880	4	0.090 to 0.211
0.9	9	-1.925 to 1.852	4	0.094 to 0.226
1.0	9	-1.833 to 1.710	4	0.091 to 0.218
1.1	9	-2.160 to 1.780	4	0.082 to 0.204
1.25	10	-2.500 to 1.605	4	0.075 to 0.198
1.50	9	-3.220 to 1.610	4	0.063 to 0.162

13. Three bases for evaluating the overall success of the method will be presented. One basis is simply a plot of measured versus predicted transmitted wave heights for the best-fit value of U . The second is an overall dimensionless standard deviation, defined as

$$\delta = \sqrt{\frac{\sum_j (H_{tmj} - H_{tcj})^2}{\sum_j (H_{tmj}^2)}} \quad (\text{B18})$$

where the subscript m refers to measured, the subscript c refers to computed, and the subscript j is an index. In the above equation, the summation is carried out over the entire set of 200 run results. It is clear that

if the fit were perfect, $\delta = 0$, and that a worst-fit should be $\delta = 1$, as this would occur, for example, for the case of all $H_{tcj} = 0$.

14. Finally, the correlation coefficient squared, r^2 , was calculated

$$r^2 = \frac{\left[\sum_j (H_{tmj} - \bar{H}_{tm}) (H_{tcj} - \bar{H}_{tc}) \right]^2}{\sum_j (H_{tmj} - \bar{H}_{tm})^2 \sum_j (H_{tcj} - \bar{H}_{tc})^2} \quad (B19)$$

15. An optimum maximum relative runup of 2.5 was used for the test condition slope of 1:1.5. Table B2 presents the associated best-fit u values, the dimensionless standard deviations as defined by Equation B18 and the correlation coefficient defined by Equation B19.

Table B2
Summary of Best-Fit Results Lamarre Data

$R(0^\circ)/H_1$		r^2	\bar{H}_{tm}	\bar{H}_{tc}	δ
<u>Assumed</u>	<u>Best-Fit U</u>	<u>ft</u>	<u>ft</u>	<u>ft</u>	<u>ft</u>
2.5	0.80	0.803	0.070	0.067	0.206

16. Based on these results and the chosen relative runup of 2.5, it appears that the best-fit value of U is

$$U = 0.80 \quad (B20)$$

It is encouraging that, as anticipated, the value of U is of order unity.

17. Figure B3 presents a plot of measured and calculated wave heights for the best-fit value U presented in Equation B20. The scatter about the line of equivalency appears to be reasonable.

18. In considering possible reasons for the scatter in Figure B3, the character of the experimental data should be noted. First, it seems reasonable that the scatter in transmitted wave heights cannot be expected to be significantly better than the runup values. Additionally, the Lamarre data

included values of the transmitted wave heights at two locations along the wave tank. The value used in the comparisons presented in Figure B3 is the average of these two heights. In some cases, the two transmitted height values were in good agreement, and in others the agreement was fairly poor. In 12 of the 200 runs, at least one of the two measured transmitted wave heights was greater than the incident wave height. To quantify the differences between the two measured transmitted wave heights, a nondimensional standard deviation, similar to Equation B18, is defined

$$\delta_m = \frac{\sqrt{\sum_{j=1}^{200} (H_{tm_1} - H_{tm_2})^2}}{\sqrt{\sum_{j=1}^{200} \frac{(H_{tm_1} + H_{tm_2})^2}{2}}} \quad (B21)$$

As before, a value of $\delta_m = 0$ would indicate that in all runs, the two measured wave heights were identical. The value of δ_m for the Lamarre data is 0.23.

Comparison of Method with Hall (1940) Data

19. Hall reported wave tank measurements of transmitted waves due to overtopping of structures of three different cross sections including 18 tests on a vertical plate structure which is similar to a seawall, with the exception that the water depths on both sides of the structure are the same. The range of variables encompassed in these tests is presented in Table B3, and the wave characteristics are portrayed graphically in Figure B4.

Table B3
Range of Variables Included in Hall's 18 Tests
with a Vertical Plate

Wave Period <u>T, sec</u>	Relative Structures Crest Elevation <u>F/H_i</u>	Water Depth, <u>h, ft</u>	Incident Wave Height <u>H_i, ft</u>
1.24 to 2.64	-4.93 to 0.785	1.35 to 2.25	0.122 to 0.476

20. An approximate $R(0^\circ)/H$ of 1.2 was used for a vertical wall. This value and the previously determined U value of 0.8 were used to calculate transmitted wave heights for the Hall data. Table B4 presents various results from these computations. Figure B5 provides a graphical comparison of measured and calculated measured wave heights. It is encouraging that in an evaluation model (i.e., no free coefficients), the method provides measures of fit (δ and r^2) which are comparable to those obtained from the calibration phase which included one free coefficient.

Table B4
Summary of Comparisons with Hall (1940) Data (18 Runs)

$R(0^\circ)/H_1$	U	\bar{H}_{tm}	\bar{H}_{tc}	δ	r^2
<u>Assumed</u>	<u>U</u>	<u>ft</u>	<u>ft</u>	<u>δ</u>	<u>r^2</u>
1.2	0.8	0.137	0.125	0.23	0.814

Summary and Conclusions

21. A rational method has been developed for predicting the transmitted wave resulting from overtopping of a smooth-sided sloping or vertically faced coastal structure such as a breakwater or seawall. The potential for overtopping is based on the runup that would occur if the structure were of sufficient height to prevent overtopping. Weir equations are employed to represent overtopping volumes, and the resulting transmitted wave height is based on a volumetric equivalency. The method requires determination of two empirical constants, representing the potential runup and the effect that overtopping has on the local water surface. The method is tested against the rather extensive set of laboratory tests by Lamarre (1967) for overtopping of a smooth structure with side slopes of 1:1.5 and the smaller number of tests by Hall (1940) for a vertical wall. A dimensionless standard deviation, δ , is defined as an objective measure of the goodness of fit of the calculated transmitted waves. Best-fit values of $\delta = 0.21$ and 0.23 , respectively were found for the Lamarre and Hall comparisons. The standard correlation coefficient values, r^2 , were 0.80 and 0.81, respectively.

22. Further testing using a generic set of data taken under both breaking and nonbreaking wave conditions would be necessary prior to final recommendation for Federal Emergency Management Agency use.

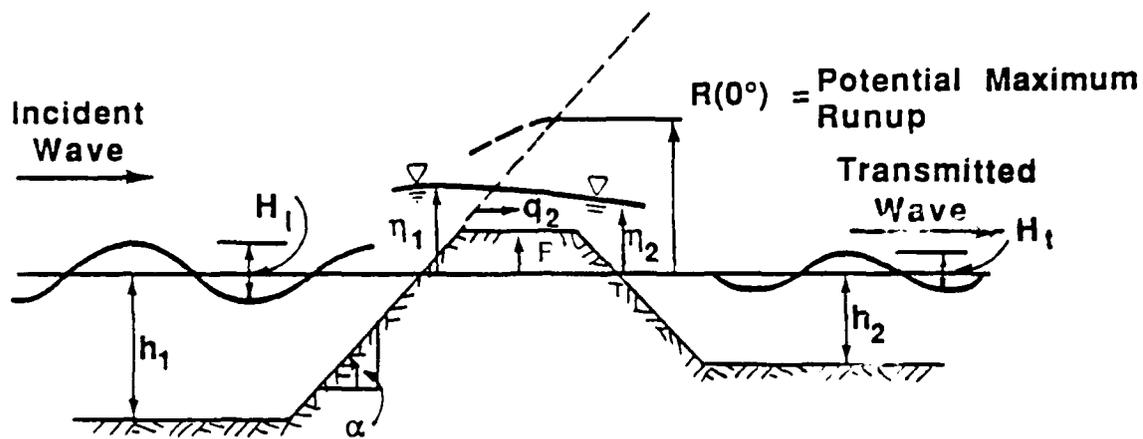


Figure B1. Definition sketch

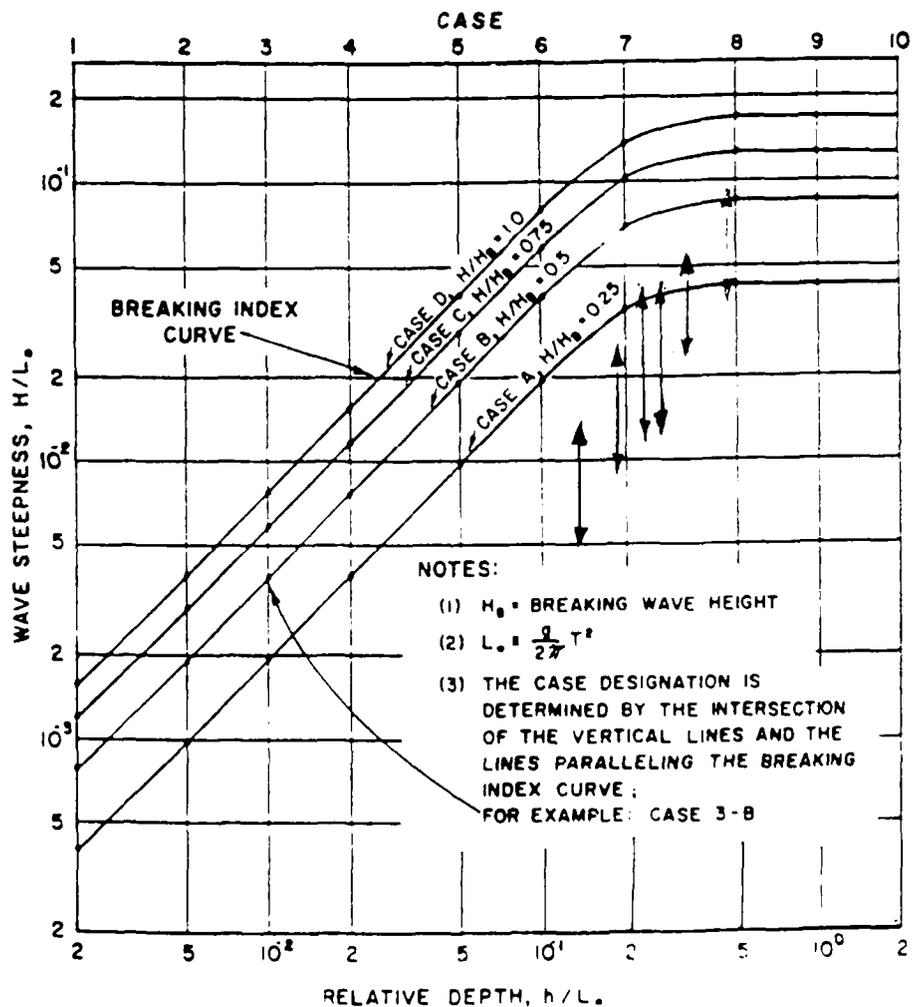


Figure B2. Ranges of dimensionless wave steepnesses and relative depths of Lamarre data, indicated by vertical arrows

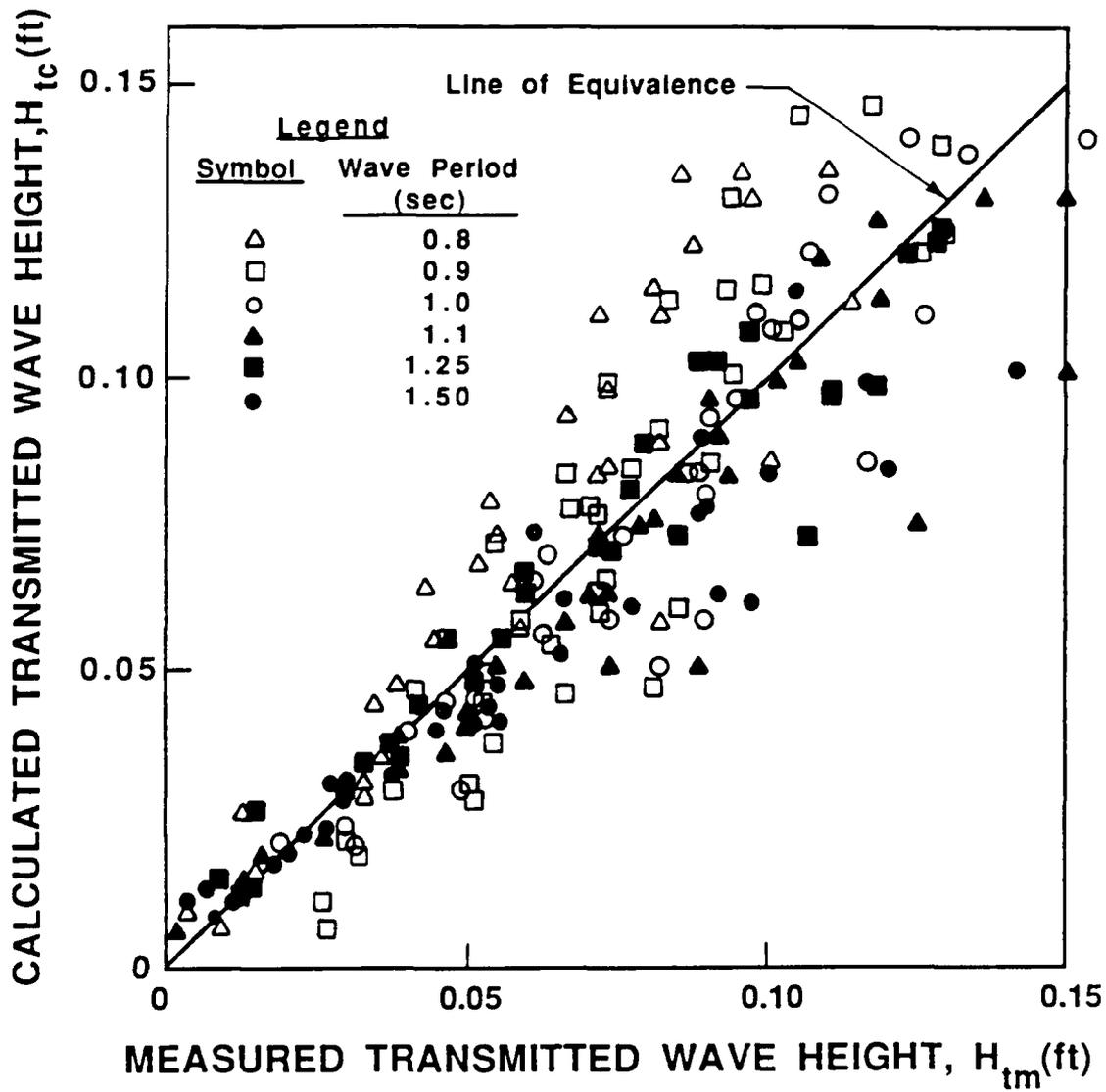


Figure B3. Plot of measured versus calculated transmitted wave heights by overtopping, Lamarre data

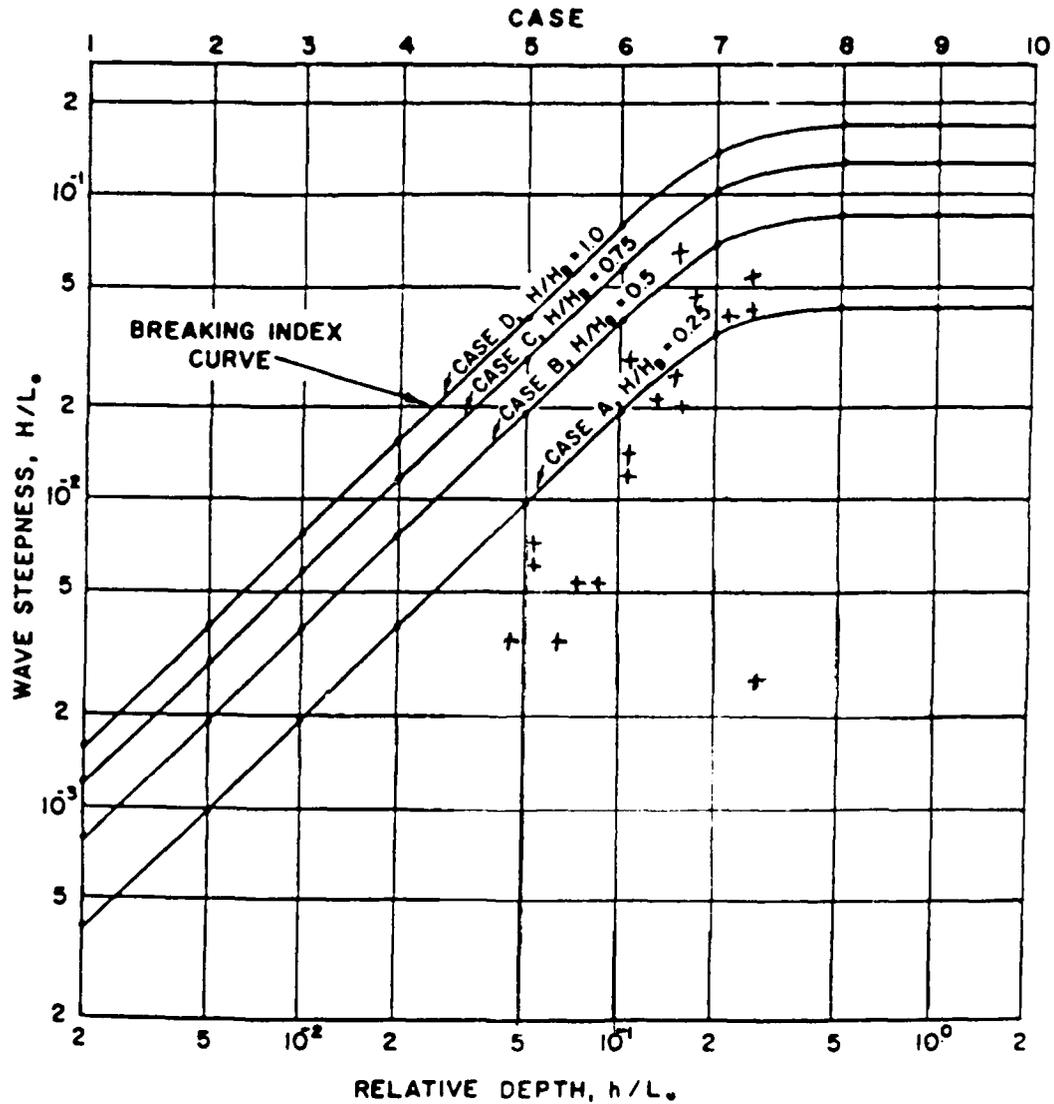


Figure B4. Dimensionless wave steepnesses and relative depths of Hall data, shown as crosses (+); refer to Figure B2 for description of figure format

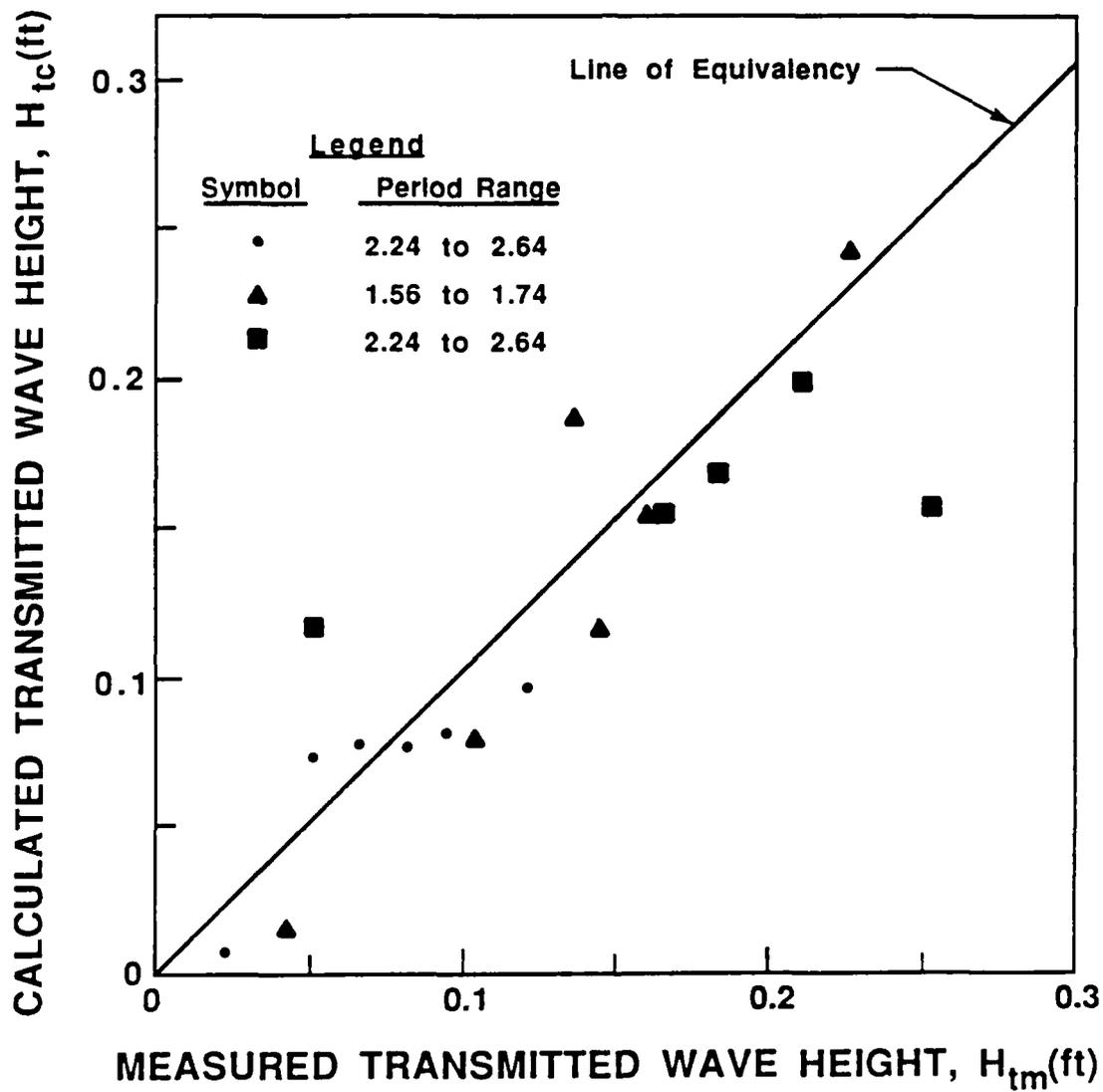
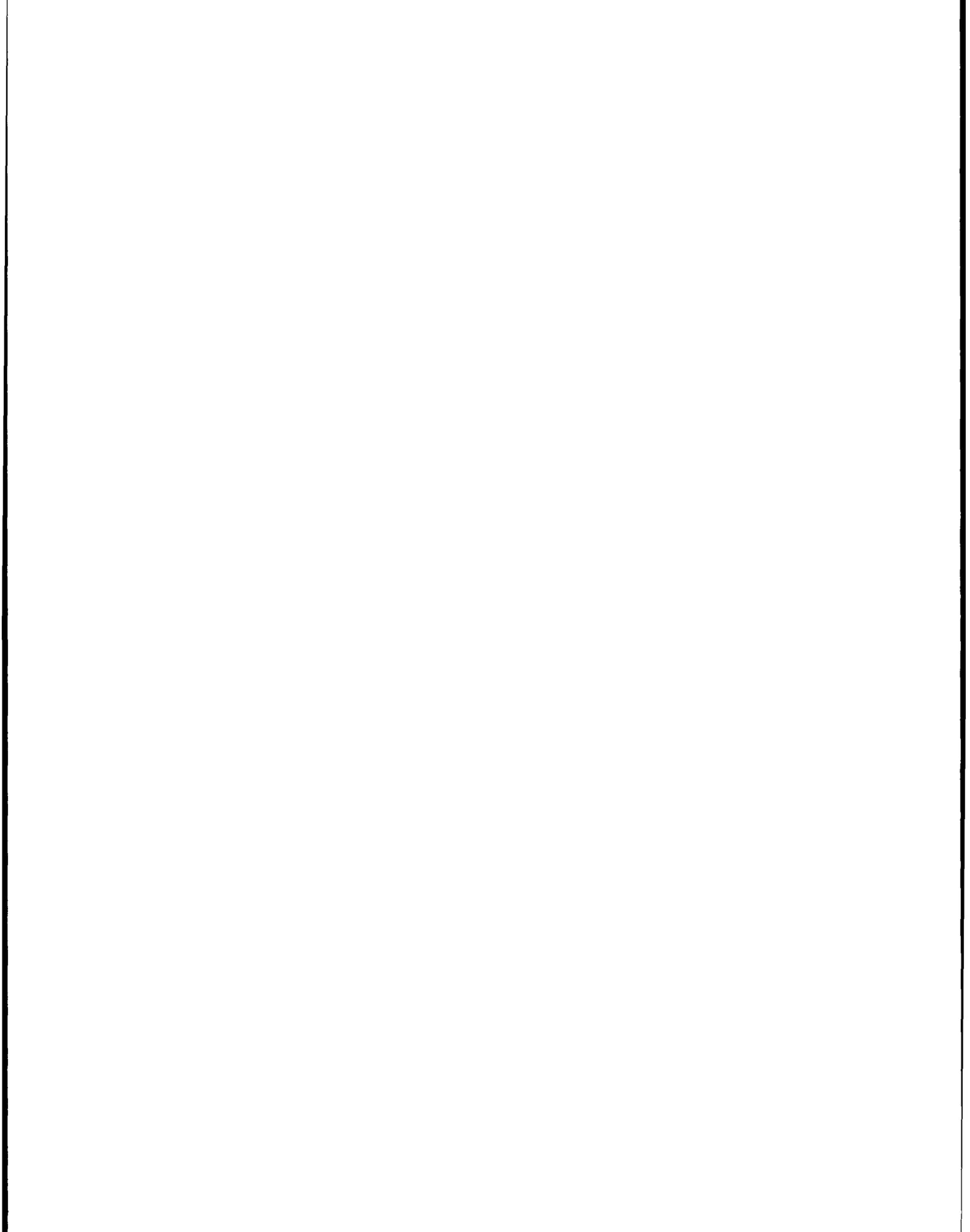


Figure B5. Measured versus calculated transmitted wave heights, Hall data



APPENDIX C: SCOUR CRITERIA

1. An important criterion for stability of a seawall or coastal flood-protection dike is the amount of scour that will occur at the toe of the structure (Figure C1). If adequate measures have not been taken to protect the foundation in front of the structure, failure may occur by three possible modes: (a) inadequate foundation stability if the structure is free standing such as a massive gravity wall, (b) inadequate resisting soil pressure on the seaward side of the structure to prevent a seaward sliding failure (in the case of a free-standing gravity structure), or (c) inadequate soil pressure on the seaward side to prevent a resisting shear/moment failure (in the case of a cantilevered structure).

2. Most of the research to date on scour at coastal flood-protection structures (primarily impermeable vertical walled structures) is based on small-scale physical modeling studies. As the state of the art in coastal engineering is still inadequate to predict the scale effects of small-scale movable bed models, the results of such studies must be interpreted with caution. Field studies provide data that are more believable scalewise but suffer from the inability to adequately monitor scour throughout storm conditions, therefore providing possible unconservative scour data since scour troughs may fill in prior to measurement. Both laboratory and field studies typically define a quantity called scour depth as the vertical distance from the prestorm equilibrium profile to the minimum profile elevation reached at or very near to the structure toe during the storm/wave event. This definition will be used in the following paragraphs to delineate from scour defined in other ways.

3. In an early study of scour, Russell and Inglis (1953)* placed a vertical wall in the swash zone of a laboratory beach that had previously attained an equilibrium shape under given wave and tide conditions. The same wave and tide action was then continued and scour at the toe of the wall measured. Russell and Inglis concluded from their tests that the scour that occurred was due to the reflection of wave action from the wall and that maximum scour as measured from low water to the after-storm profile in the vicinity of the structure toe would approximately equal the incident wave height.

* See References at the end of the main text.

Because of the limited number of tests performed and the complicating effect of tides, no predictive measure of scour depth as defined in paragraphs 2 and 3 is given. As Russell and Inglis' experimental setup was in a very small laboratory tank (16.4 m), the results may have considerable scale effects included.

4. Numerous other laboratory measurements of scour since the Russell and Inglis (1953) study have suggested similar inconclusive scour relationships based upon small-scale laboratory tests. Kadib (1963) included the effects of grain size of sand within his experiments with the result that scour depth decreased for the coarser sand size in agreement with physical intuition. Kadib (1963) noted greater scour depths than found in earlier laboratory studies. Maximum scour depths in his experiments were two to three times the incident wave height. Kraus (1987) notes that Kadib's (1963) experiments are difficult to interpret because of the unknown influence of ponding and seepage from behind the wall and possible return flow over the wall.

5. Sawaragi and Kawasaki (1960) found the maximum laboratory scour depth to be approximately equal to the incident deepwater wave height based on small-scale laboratory measurements with various sand grain sizes. Sawaragi and Kawasaki (1960) also compiled field data on scour from two storms at seawall/seadikes on Ise Bay, Japan. They found that the average maximum scour depth from eight measurements was approximately equal to the water depth at the toe of the seadikes with the ratio of scour depth to water depth ranging from 0.25 to 1.68. In a later laboratory study, Sawaragi (1967) measured scour in front of permeable sloping seawalls (consisting of wood boards with holes drilled into them) and found that as permeability increased, reflectivity of the wall decreased and consequent scour decreased. Sawaragi (1967) presents an empirical relationship for the reflection coefficient versus scour depth (slope not provided) and noted that maximum scour depth approached 0.6 times the incident deepwater wave height.

6. Sato, Tanaka, and Irie (1969) conducted a comprehensive set of scour measurements in two laboratory wave tanks (the largest being 105 m long, 3 m wide, and 2.5 m deep). They ran their tests for a variety of incident wave conditions that included cases where wave breaking took place on the bottom profile prior to reaching the seawall providing a reasonable idea of what might happen under breaking wave conditions at a real seawall during a storm. The results of these tests provide scour at the toe of the wall ranging from

$S/H_o = 0.25$ to 2.5 , where S = scour depth. The scour was found to be dependent on the location of the wall in the surf zone and on incident wave steepness, as well as wall inclination, composition, and shape. They found that relative scour depth (S/H_o) decreased with increasing wave steepness. For deepwater wave steepness in the storm range (0.02 to 0.04), the maximum scour predicted by them is equal to the incident deepwater wave height. Sato, Tanaka, and Irie (1969) also noted that inclined walls produced less scour, and they allude to field measurements made along a breakwater at the Port of Kashima, which faces the Pacific Ocean. They note that ". . . the maximum scour depth under the stormy conditions may be considered to be nearly equal to the maximum significant wave height during the storm." Although it is impossible to directly compare the results of scour that occurs at shore perpendicular structures (such as breakwaters that may be heavily influenced by coastal currents) with seawalls/seadikes that run parallel to the coast, Sato, Tanaka, and Irie (1969) suggest that such field measurement of scour coincide with their laboratory testing of vertical walled seawalls/seadikes. Sato, Tanaka, and Irie's (1969) tests are especially relevant to Federal Emergency Management Agency (FEMA) design criteria due to the fact that their tests were run under breaking waves consistent with FEMA depth-limited breaking wave approach to design.

7. Hotta and Marui (1976) testing permeable and impermeable shore parallel breakwaters also found a maximum scour depth on the order of the incident wave height in deep water. When plotted as a function of depth at the structure, their results showed a scour depth approximately equal to 0.6 times the water depth, in reasonable agreement with Katayama, Irie, and Kawakami's (1974) field findings discussed below.

8. During the late sixties and early seventies, a number of experimental beach profile responses to structure physical model studies were made at Texas A&M University (see Herbich, Murphy, and Van Weele (1965); Herbich and Ko (1969); Chestnutt and Schiller (1971); and Song and Schiller (1977)). The results of these studies are discussed in Herbich et al.'s (1984) textbook on scour. Herbich et al. (1984) present general conclusions on these studies which often consist of empirical or semiempirical relationships for scour depth S . Because of the small scale of the tests and the often limited conditions for which the equations were developed, no universal conclusions can be drawn. As an example, Herbich and Ko (1969) present a semitheoretical

relationship for scour based upon partial standing wave action which is slope and structure permeability dependent. In the case of a vertical wall, their equation would predict a negative scour (i.e. accretion). Another example of the particular problems that empirical expressions for scour provide to a university approach needed by FEMA is given by an equation for scour suggested by Song and Schiller (1977) as follows:

$$\frac{S}{H_o} = 1.94 + 0.57 \ln\left(\frac{X}{X_b}\right) + 0.72 \ln\left(\frac{H}{L}\right) \quad (C1)$$

For reasonable values of storm wave steepness parameters ($H/L = 0.02$ to 0.04) and assuming wave breaking at the structure toe ($X/X_b = 1.0$), a negative scour depth is calculated that is contrary to many physical findings. Walton and Sensabaugh (1979) also present an equation for scour credited to Jones (1975) which has many of the same limitations as those noted above.

9. In a limited study of scour depth at seawalls, Barnett (1987) found scour depths ranging from 0.36 to 1.46 times the incident deepwater wave height depending on wave steepness and structure location on the profile.

10. Xie (1985) studied scour at vertical breakwaters in the laboratory and developed an empirical expression for scour depth at the structure toe. Kraus (1987) notes that Xie's (1985) relationship for small values of wave steepness gives a scour depth approximately equal to the incident wave height at the site. Xie (1985) also did limited testing with irregular waves and concluded that scour depth could be conservatively estimated by substituting significant wave height into his empirical expression for scour.

11. Katayama, Irie, and Kawakami (1974) made measurements of scour at permeable rubble-mound detached breakwaters on the Niigata (Sea of Japan) coastline. They installed rods with movable steel rings to measure maximum scour depth and found that maximum scour depths (2 to 4 m) were on the same order as the prestorm water depth (3 to 4 m). Unfortunately lack of knowledge of wave and water level climate during the measurement period limits conclusions of the study.

12. The Shore Protection Manual (SPM) (1984) presently recommends that "the maximum depth of a scour trough below the natural bed is about equal to the height of the maximum unbroken wave that can be supported by the original depth of water at the toe of the structure." This criterion is believed to be

derived from literature research of the problem as well as expert opinion of various engineers.

13. In final analysis, it is felt that the present state of knowledge is lacking sufficient answers in this area. Laboratory tests provide information on scour, but no existing scour formula or equation has been justified beyond the limited set of data for which it was proposed. Additional scale factor uncertainty in laboratory experiments make laboratory results open to question. Until further advancements are made in this area, a prudent approach to design suggests using the SPM criteria with a change in wording from "original depth of water" to "maximum storm depth of water." This approach suggests considering toe scour depth for seawalls/seadikes with no toe protection to be equal to incident storm wave height (0.78 times water depth in existing FEMA guidelines) at maximum storm water level. It should be recognized that this criterion may be overly conservative, and FEMA may wish to relax this criterion somewhat.

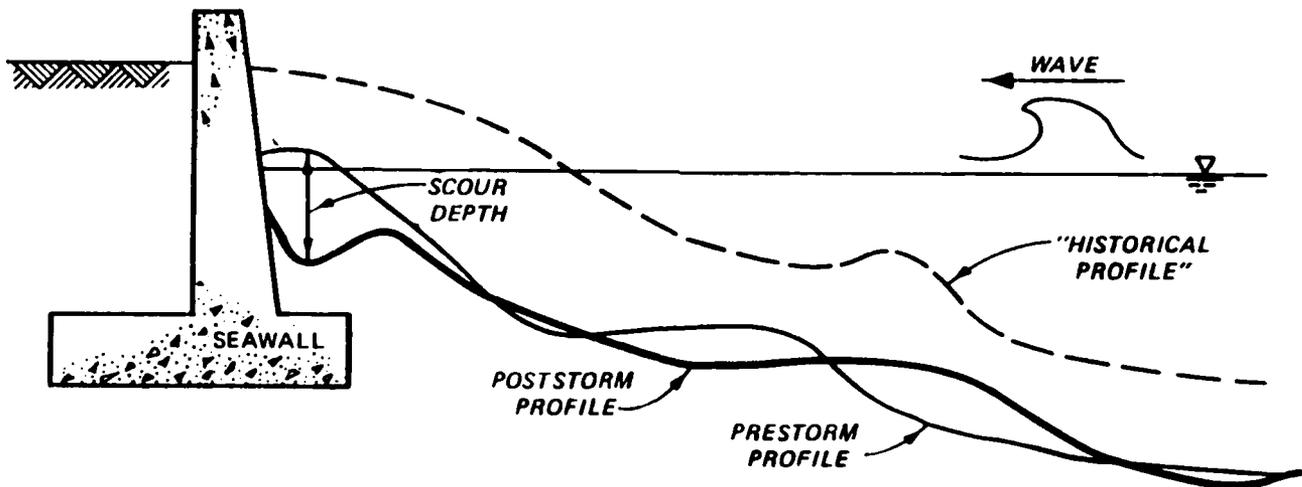


Figure C1. Scour definition at seawall

APPENDIX D: NOTATION

- a - empirical coefficient
- A - empirical coefficient
- b - empirical coefficient
- B - empirical coefficient
- B_c - structure crest width
- C - dimensionless coefficient
- C_b - wave celerity at breaking
- C_p - dimensionless coefficient
- C_1 - wave celerity at section 1
- C_2 - wave celerity at section 2 (downwave)
- d - depth
- d_s - depth at toe of structure
- $f()$ - function of
- F - freeboard
- F_t - force per unit length of wall
- F' - dimensionless freeboard
- F^* - dimensionless freeboard
- G - Goda number
- g - gravitational acceleration constant
- h_s - structure height
- H - wave height
- H_b - wave height at breaking
- H_o - deepwater wave height
- H'_o - refracted deepwater wave height
- H_s - significant wave height
- H_{50} - median wave height
- H_i - incident wave height
- H_t - transmitted wave height
- H_{tm} - measured transmitted wave height
- H_{tc} - computed transmitted wave height
- I_r - Iribarren number
- I_{rc} - Chue number
- I_{rv} - Van Dorn number
- I_{rr} - random Iribarren number

k - dimensionless coefficient
 k_r - roughness height
 K_r - reflection coefficient
 K_s - shoaling coefficient
 l - length between roughness elements
 L - wavelength
 \bar{L} - wavelength calculated via linear theory using \hat{T}
 L_o - deepwater wavelength
 m - discharge coefficient
 P_{max} - maximum pressure
 P_u - pressure at estimated upper limit of runup
 P_t - pressure at toe of structure
 q - overtopping rate (discharge)
 q_2 - volumetric flow rate
 q'_2 - dimensionless discharge over structure
 q^* - dimensionless overtopping rate
 Q_g^* - dimensionless fitted empirical overtopping coefficient
 r - correlation coefficient; also, roughness coefficient
 R - wave runup
 R_p - predicted wave runup
 R' - dimensionless runup
 R_h - wave runup calculated by Hunt expression
 T - wave period
 \hat{T} - average wave period
 T_p - peak wave period
 T_z - zero crossing wave period
 U - empirical constant
 V_t - volumetric flux under crest phase
 V_1, V_2 - velocity of water at upwave and downwave end of structure respectively
 w - specific weight of water
 x - distance
 z - vertical axis
 α - angle of approach; also empirical constant
 β - empirical coefficient
 δ - dimensionless standard deviation
 δ_m - nondimensional standard deviation

- ϵ = spectral width parameter
- κ_T = wave transmission
- θ = structure slope; also, wave phase angle
- μ = dynamic viscosity
- η_i = incident water wave level above still-water level
- η_r = reflected water wave level above still-water level
- η_1, η_2 = wave water level above still-water level seaward and shoreward of structure respectively
- η'_i = dimensionless water surface elevation
- ρ = density of fluid
- σ = surface tension

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