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PERFORMANCE AND STABILITY OF LOW-CRESTED BREAKWATERS

BY

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LIST OF SYMBOLS

a - 0.5926 (Equations 3-10, 11, 4-9, 12, 18, 21, 22, 23, 25)

a_j - wave amplitude of frequency band in Equation 3.4

A_d - area of original breakwater that displaced during test

A_t - total cross-sectional area of structure

B - breakwater crest width

d - water depth

D.D. - dimensionless damage $= \dfrac{A_d}{(D_{50})^2}$

D_{50} - median stone diameter $= \left(\dfrac{w_{50}}{w_r}\right)^{1/3}$

f - wave frequency in Equation 3.4

F - freeboard, $h - d$, in Seelig's formula for K_t (Equation 3-12)

h - structure crest height in Seelig's formula for K_t (Equation 3-12)

h_f - maximum damaged crest height

$\overline{h}_f$ - average final crest height

h_i - initial structure crest height

h_f/d - final relative crest height

H_c - incident zero-moment wave height used in the calculation of K_t

H_s - incident zero-moment wave height

H_t - transmitted significant wave height calculated as average of two transmission gages

K - Hudson's dimensionless stability coefficient $= \dfrac{H_s^3}{D_{50}^3 \left(\dfrac{w_r}{w_w} - 1\right) \cot \alpha}$

K_d - energy dissipation coefficient

K_r - energy reflection coefficient

K_t - energy transmission coefficient

L_p - wave length corresponding to T_p

M - modified spectral stability number = $N_s^* \left(\frac{h_i}{d}\right)^{1.5}$

N - Hudson's stability number = $\dfrac{H_s}{D_{50}\left(\frac{w_r}{w_w}\right) - 1)}$

N_s^* - spectral stability number = $\dfrac{(H_s^2 \; L_p)^{1/3}}{D_{50} \left(\frac{w_r}{w_w} - 1\right)}$

P - $\left(\frac{A_t}{D_{50}^2}\right)\left(\frac{H_s}{L_p}\right)$

Q_p - Goda's spectral peakedness parameter defined by Equations 3-3 and 3-4

R - relative freeboard = $\dfrac{h_f - d}{H_s}$

$S(f)$ - value of energy density spectrum (Equation 3-3)

$\overline{T}$ - average wave period of spectrum

T_p - peak wave period of spectrum

T_s - significant wave period of spectrum

w_{50} - median stone mass

w_r - mass density of stone

w_w - mass density of water

Δf - frequency band width (Equation 3-4)

Δh - change in crest height = $h_i - \bar{h}_f$

α - slope of structure face

CHAPTER 1 INTRODUCTION

1.1 BACKGROUND

Traditional ideas about shore protection works embrace the philosophy that damage to a structure is to be avoided for all but catastrophic conditions. In the case of offshore breakwaters, this usually means specifying the crest elevation such that little to no overtopping occurs, since the volume of water overtopping the crest has been found to be an important parameter in determining rear slope stability (Graveson et al., 1980). This approach often results in cost-prohibitive shore protection, because structure cost is integrally related to the volume of material required for construction and maintenance. Thus, any reduction in crest height results in a cost savings and an increase in project feasibility.

Recent field observations and subsequent laboratory studies indicate that adequate shore protection can be achieved in some instances through the use of low-crested and/or "sacrificial" breakwaters. In 1976, combined wave and surge action due to Cyclone David caused severe damage to a breakwater at Rosslyn Bay in Australia. Despite the fact that its crest was battered to below mean water level, the breakwater continued to function effectively for two and a half years until the structure was repaired (Bremner et al. 1980). The unexpected success of this failed breakwater prompted the concept of a "sacrificial" offshore structure which is used to protect an inner breakwater or revetment and is designed to fail under extreme wave conditions. Model tests on such a structure proposed for Townsville Harbor, Australia were conducted, and it was shown that this approach would save 40 percent over a conventional design (Bremner et al. 1980). Interestingly, these tests also suggested that the wave transmission may not be very dependent upon the amount of structural

damage, because the increased energy transmission resulting from a lower crest is balanced by the increased energy dissipation resulting from a wider crest. Thus, the design parameters associated with these structures are the prediction of damage levels and the subsequent performance of the "failed" breakwater. Additional research is required in order to better understand the influence of structural and wave parameters on these criteria.

1.2 BRIEF SUMMARY OF PREVIOUS WORK

Ahrens (1984) investigated the stability and to some extent the performance of low-crested breakwaters, with regard to certain structure parameter and incident wave conditions. His data and findings are the basis for this report and are discussed in greater detail in Chapter 2. A brief summary of recent studies on the low-crested design concept constitutes the remainder of this section. A more comprehensive list and annotated bibliography of research on submerged and low-crested structures and related topics is included in Appendix A.

Foster and Haradasa (1977) conducted model tests on the original Rosslyn Bay breakwater and on a proposed modification. Irregular incident wave conditions were simulated using monochromatic waves with the same height and period as the significant wave height and peak period of the prototype spectrum. The tests closely reproduced the mode of damage seen in the prototype structure, except that the initiation of damage occurred earlier and the rate of damage after initiation was slower in the model than in the prototype.

Foster and Khan (1984) studied overtopped structures in an attempt to determine the variables most influencing their stability. They conclude that the relationships between the parameters governing stability of an

overtopped structure are more complex than for a non-overtopped structure. They recommended that rigorous physical model testing with the full range of expected wave conditions and water depths be conducted prior to prototype construction.

Seelig (1979) studied wave transmission by overtopping of regular and irregular waves for subaerial and submerged smooth, impermeable, trapezoidal structures. Seelig found that the dimensionless parameter, freeboard divided by the incident significant wave height, is an important factor governing energy transmission by overtopping. In a later investigation, tests were extended to include rubble mound and dolos armored structures (Seelig 1980). An empirical method for determining wave transmission by overtopping that includes the effects of structure width and wave runup, in addition to freeboard and wave height, was developed.

Allsop (1983) studied transmission, overtopping, and damage to low-crested, multi-layered trapezoidal structures. He found that wave transmission, which was largely due to overtopping, is a function of wave steepness. Although he did not find a similar period dependence in the damage data, he notes that since stability is closely related to overtopping, it is possible that stability of overtopped structures is a function of the wave period.

1.3 PURPOSE AND ORGANIZATION OF THIS REPORT

The purposes of this investigation were to provide additional insights into the stability and wave transmission data from studies of homogeneous, low-crested breakwaters conducted at the Coastal Engineering Research Center (Ahrens 1984), and to develop an interactive computer program to assist in the design of these structures. In addition, an extensive literature search was conducted. Pertinent papers and reports

are summarized in a series of abstracts which are presented in Appendix C of this report.

Chapter 2 of this report outlines the experimental approach and analytical techniques used by Ahrens (1984). Chapter 3 describes the analyses used here and presents the results which are the basis for the design program summarized in Chapter 4. Summary, conclusions and recommendations for future work are given in Chapter 5. Contained in the three appendices are: the design aid program, the experimental data analyzed in this report and the abstracts of related reports and technical papers.

CHAPTER 2 EXPERIMENTAL APPROACH

2.1 DESCRIPTION OF EXPERIMENTS

Laboratory experiments on the performance of low-crested breakwaters were conducted (by Ahrens) in the wave flume at the U.S. Army Corps of Engineers Waterways Experiment Station in Vicksburg, Mississippi (Ahrens 1984). Structures were tested in a 61-cm wide channel within a 1.2 m by 4.6 m by 42.7 m tank. Signals for the generation of irregular waves were stored on magnetic tape and transferred to the wave paddle using a data acquisition computer system (DAS). Four files with periods of peak energy density ranging from 1.45 to 3.60 sec were used. A total of five wire-resistance wave gages recorded wave conditions in front of and behind the structure. Records from three unequally spaced gages in front of the structure were used to resolve the incident and reflected wave fields using the method of Goda and Suzuki (1976). Two gages behind the structure recorded the transmitted wave conditions. The DAS sampled the gages sixteen times per second for 256 seconds.

A ten-turn potentiometer in a voltage divider network was used to regulate the signal amplitude to the wave blade. The signal amplitude is related to the wave heights that are generated. An undamped signal produced the depth-limited energy spectrum as described by Vincent (1981, 1982). The theoretical basis for this spectrum is taken from the work of Phillips (1958) who proposed an expression for the upper bound on energy density for deep water waves based on wave steepness. Phillips' limit is proportional to f^{-5} where f is wave frequency. Using Phillips' expression as a starting point, Kitaigordoskii et al. (1975) derived an equation for the depth-controlled maximum energy density. The depth-dependent limit on energy density is proportional for f^{-3}. Other characteristics of these

spectra are a sharp drop in energy density at frequencies below the peak, and, in the shallow water limit, wave heights that are proportional to the square root of depth.

To ensure the most severe wave conditions possible at the structure, waves were shoaled on a 1:15 slope from a water depth 25 cm greater than the depth at the breakwater. Incident significant wave heights ranged from one to eighteen centimeters. Details of the test setup are shown in Figure 2.1. Incident wave conditions are summarized in Table 2.1.

Two types of tests were performed. The purpose of the first type was to determine expected levels of damage under different wave conditions, both mild and severe. The second type sought to evaluate the performance of the damaged breakwaters under more typical, less severe wave attack. Wave action for Type 1 tests lasted between 1.5 hr for File 1 spectra to 3.5 hr for File 4 spectra. Wave data were collected several times during each run. Tests on the previously damaged structures lasted about 30 minutes and data were collected two to three times.

Structures were built with homogeneous stone. Two different stone sizes were tested. Specific gravity and median mass were 2.63 and 17 grams, respectively, for the smaller stone, and 2.83 and 71 grams for the larger stone. The undamaged structures were trapezoidal with front and rear slopes of 1 on 1.5. The initial profile for a Type 2 test was the same as the final profile of the preceeding test. Starting crest heights ranged from 24.11 to 36.09 cm in a water depth of 25 cm and from 31.55 to 32.06 cm in a depth of 30 cm. Using test type, initial structure height, stone size, and water depth as criteria, the 205 experiments were divided into ten subsets. A summary of pertinent parameters for each subset is given in Tables 2.2 and 2.3.

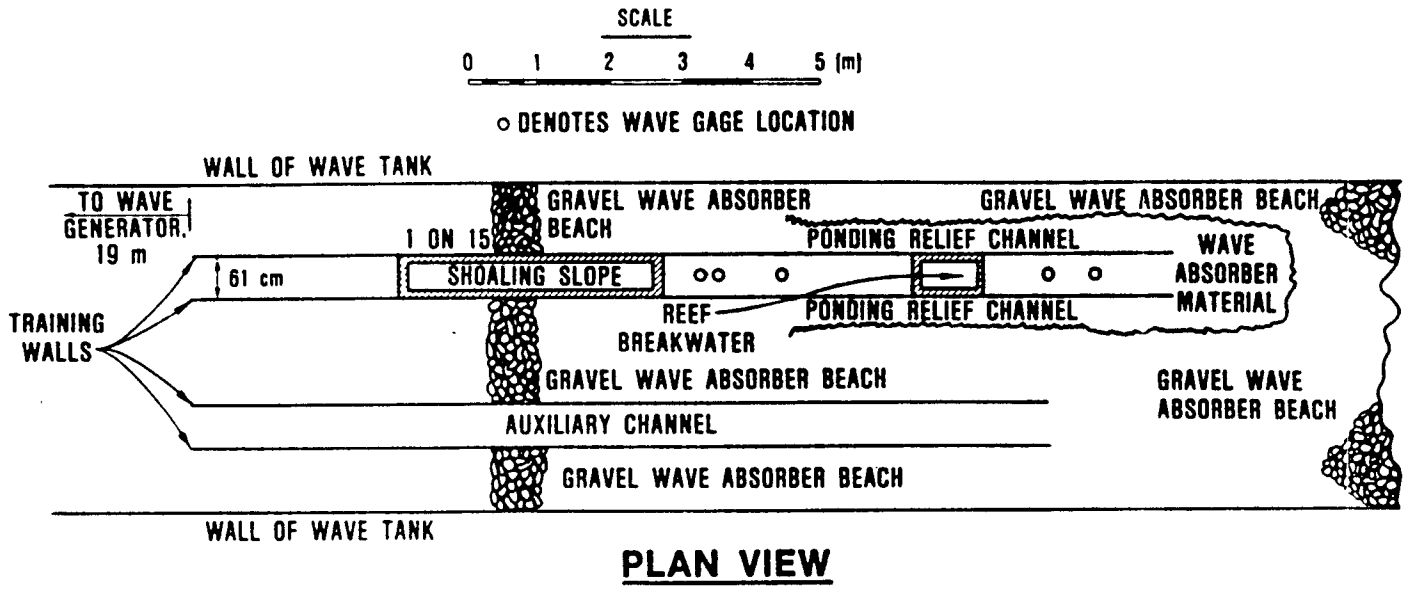


Figure 2.1 Details of Experimental Setup (after Ahrens 1984).

Table 2.1. Summary of incident wave conditions for low-crested breakwater test.

File No.	Water Depth (cm)	Approx Peak Period (Sec)	Range of Incident Wave Height (cm)
1	25	1.45	1.09 - 11.47
	30	1.45	5.76 - 12.63
2	25	2.25	1.16 - 13.43
	30	2.25	2.58 - 14.46
3	25	2.86	1.62 - 15.78
	30	2.86	8.20 - 18.17
4	25	3.60	2.25 - 16.10
	30	3.60	5.22 - 17.60

Table 2.2. Summary of structural and incident wave conditions for type 1 (Stability) Tests

Subset No.	No. of Tests	Median Stone Diameter (cm)	Water Depth (cm)	Cross-Sectional Area (cm^2)	File No.	Range of Incident Wave Height (cm)	Range of Initial Crest Height (cm)
1	27	1.86	25	1170	1	2.87 - 11.45	24.11 - 25.39
					2	2.91 - 13.43	24.41 - 25.48
					3	3.89 - 15.78	24.44 - 25.73
					4	5.46 - 16.10	24.14 - 25.12
3	29	1.86	25	1560	1	2.82 - 11.36	29.02 - 30.48
					2	2.89 - 13.38	29.29 - 29.81
					3	3.68 - 15.63	28.74 - 29.84
					4	2.59 - 15.84	28.86 - 30.08
5	41	1.86	25	2190	1	2.75 - 11.35	34.38 - 35.57
					2	4.03 - 13.02	34.41 - 36.06
					3	1.81 - 15.61	34.93 - 36.09
					4	2.56 - 15.99	34.59 - 36.03
7	38	2.93	25	1900	1	2.60 - 11.44	31.36 - 32.00
					2	2.72 - 13.11	31.49 - 32.34
					3	1.65 - 15.66	31.24 - 32.52
					4	2.35 - 16.04	31.39 - 32.80
9	13	2.93	30	1900	1	5.76 - 12.63	31.55 - 31.82
					2	5.80 - 14.46	31.58 - 31.67
					3	8.20 - 18.17	31.61 - 32.06
					4	5.22 - 17.60	31.61 - 32.13

Table 2.3. Summary of Structural and Incident Wave Conditions for Type 2 (Previously Damaged) Tests

Subset No.	No. of Tests	Median Stone Diameter (cm)	Water Depth (cm)	Cross-Sectional Area (cm^2)	File No.	Range of Incident Wave Height (cm)	Range of Initial Crest Height (cm)
2	3	1.86	25	1170	1	N/A	N/A
					2	5.87 - 5.95	15.88 - 19.99
4	12	1.86	25	1560	1	3.17 - 11.19	17.56 - 18.01
					2	2.72 - 13.27	17.80 - 19.45
6	11	1.86	25	2190	1	2.84 - 11.47	19.54 - 19.81
					2	2.49 - 12.88	19.78 - 19.96
8	26	2.92	25	1900	1	1.09 - 11.03	28.19 - 28.35
					2	1.16 - 13.31	28.16 - 28.29
					3	1.62 - 13.32	27.58 - 28.01
					4	2.25 - 12.26	27.55 - 28.01
10	5	2.92	30	1900	1	N/A	N/A
					2	2.58 - 14.41	24.96 - 25.21

2.2 BRIEF SUMMARY OF AHRENS' ANALYSES AND RESULTS

Structural stability is defined by Ahrens (1984) in terms of both the volumetric damage and the change in crest height. Volumetric damage is described by a dimensionless damage parameter,

$$D.\ D. = \frac{A_d}{\left(\frac{w_{50}}{w_r}\right)^{2/3}} = \frac{A_d}{(D_{50})^2} \qquad (2\text{-}1)$$

where

A_d = cross-sectional area of the portion of the original breakwater that was displaced;

w_{50} = median mass of the stone;

w_r = mass density of the stone; and

D_{50} = the median stone effective diameter.

The change in crest height is represented by the final relative crest height or ratio of final crest height to the water depth. Final crest height is measured at the highest point on the structure.

Hudson's stability number defined as,

$$N_S = \frac{H_s}{D_{50}\left(\frac{w_r}{w_w} - 1\right)} \qquad (2.2)$$

where

H_s = incident significant wave height; and

w_w = mass density of water,

was initially used by Ahrens as a means of predicting the stability of the structure. Plots of final relative crest height and dimensionless damage as a function of N_s are presented in Figures 2.2 - 2.6(a,b) for the Type 1 tests. The expected trends are obvious; larger values of stability number

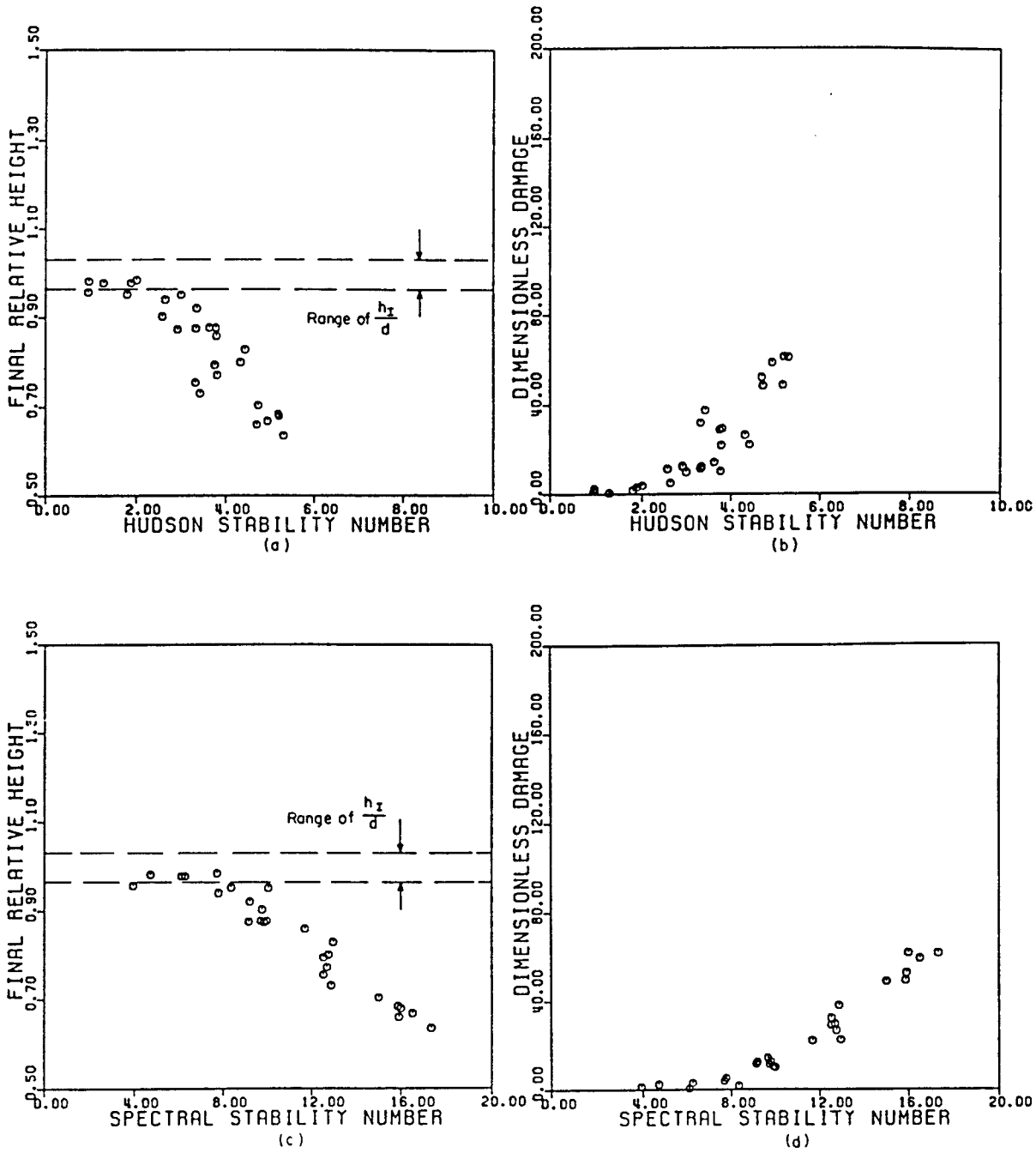


Figure 2.2 Damage parameters as a function of the Hudson stability number and the spectral stability number for subset 1.

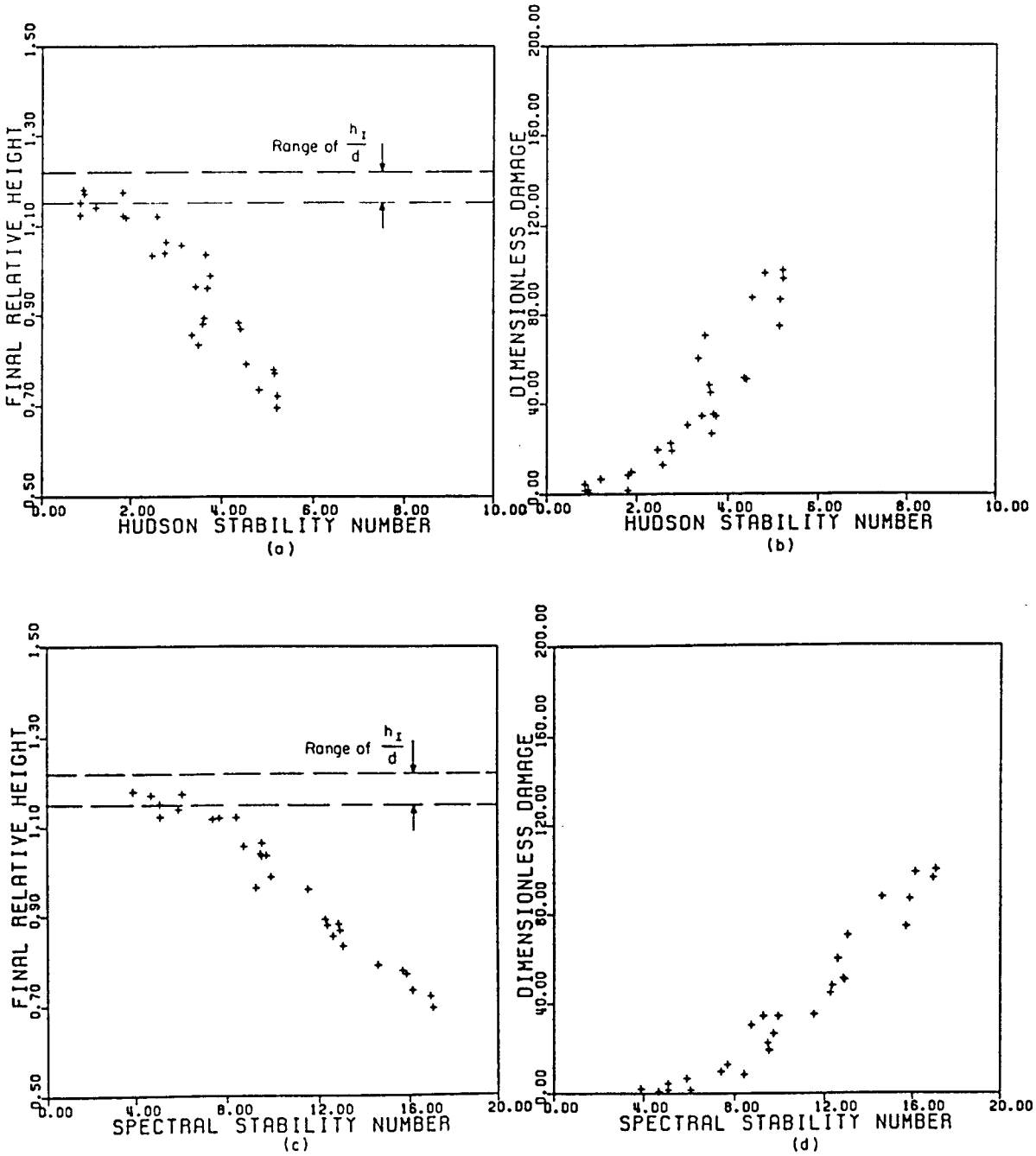


Figure 2.3 Damage parameters as a function of the Hudson stability number and the spectral stability number for subset 3.

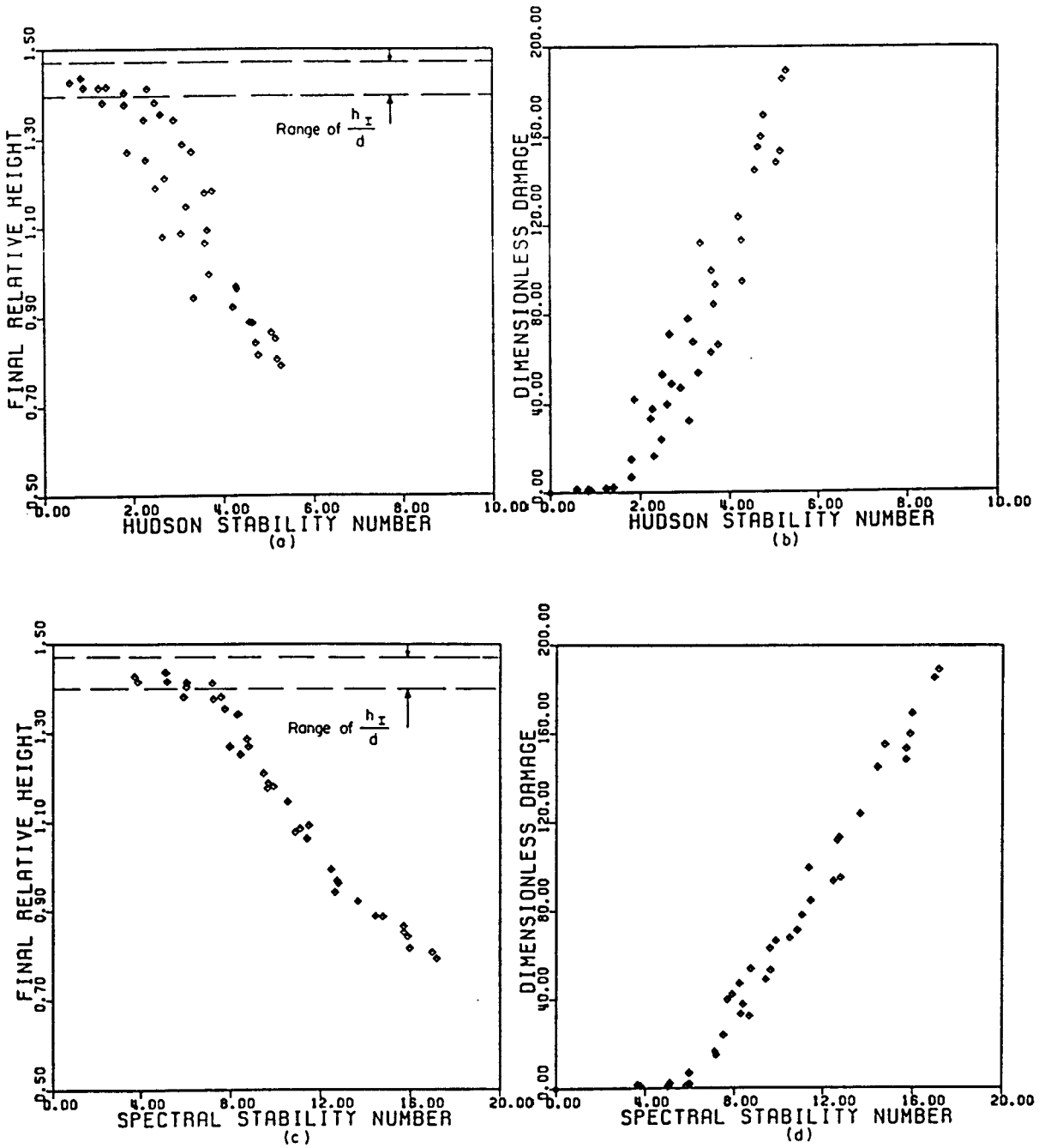


Figure 2.4 Damage parameters as a function of the Hudson stability number and the spectral stability number for subset 5.

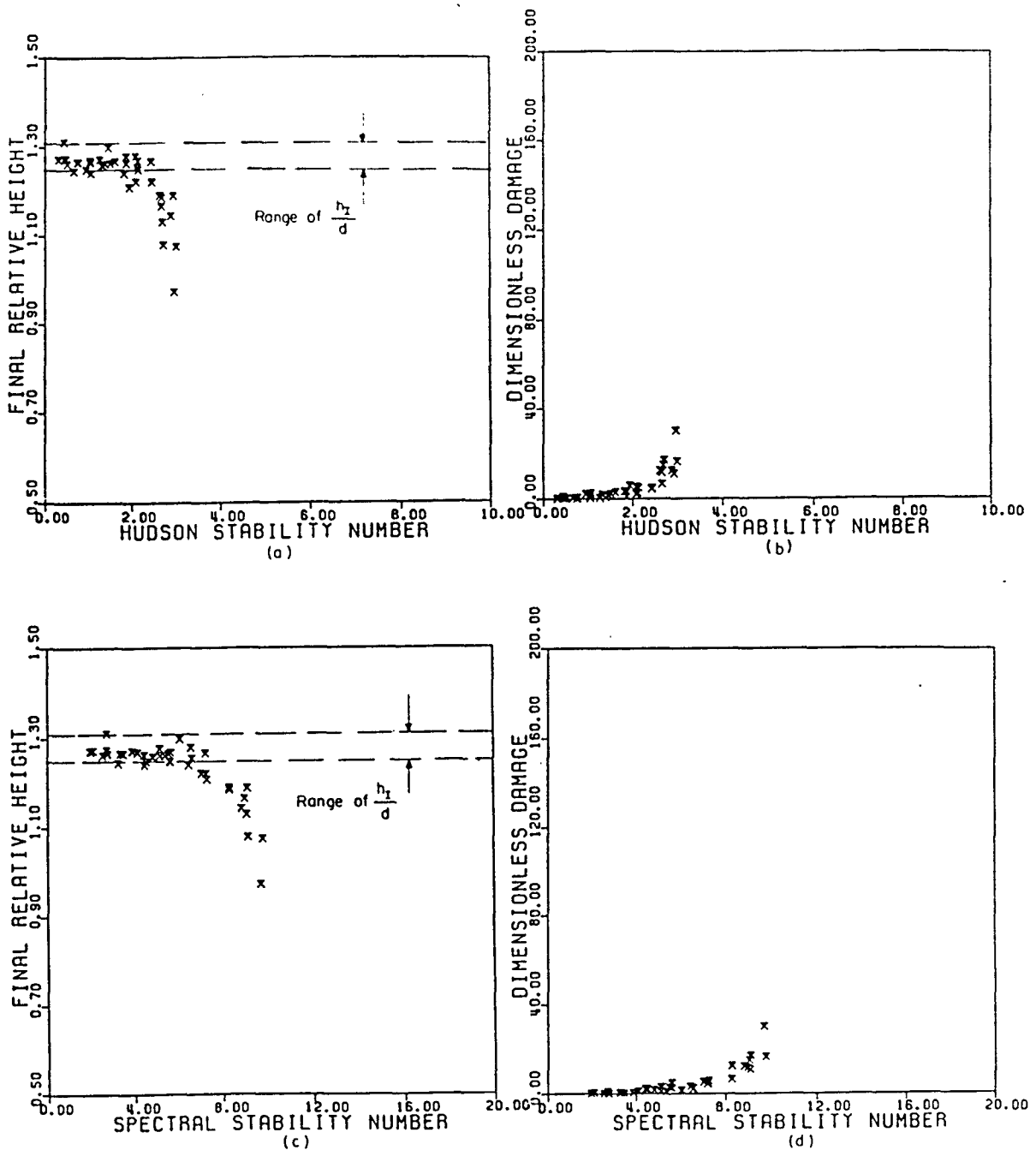


Figure 2.5 Damage parameters as a function of the Hudson stability number and the spectral stability number for subset 7.

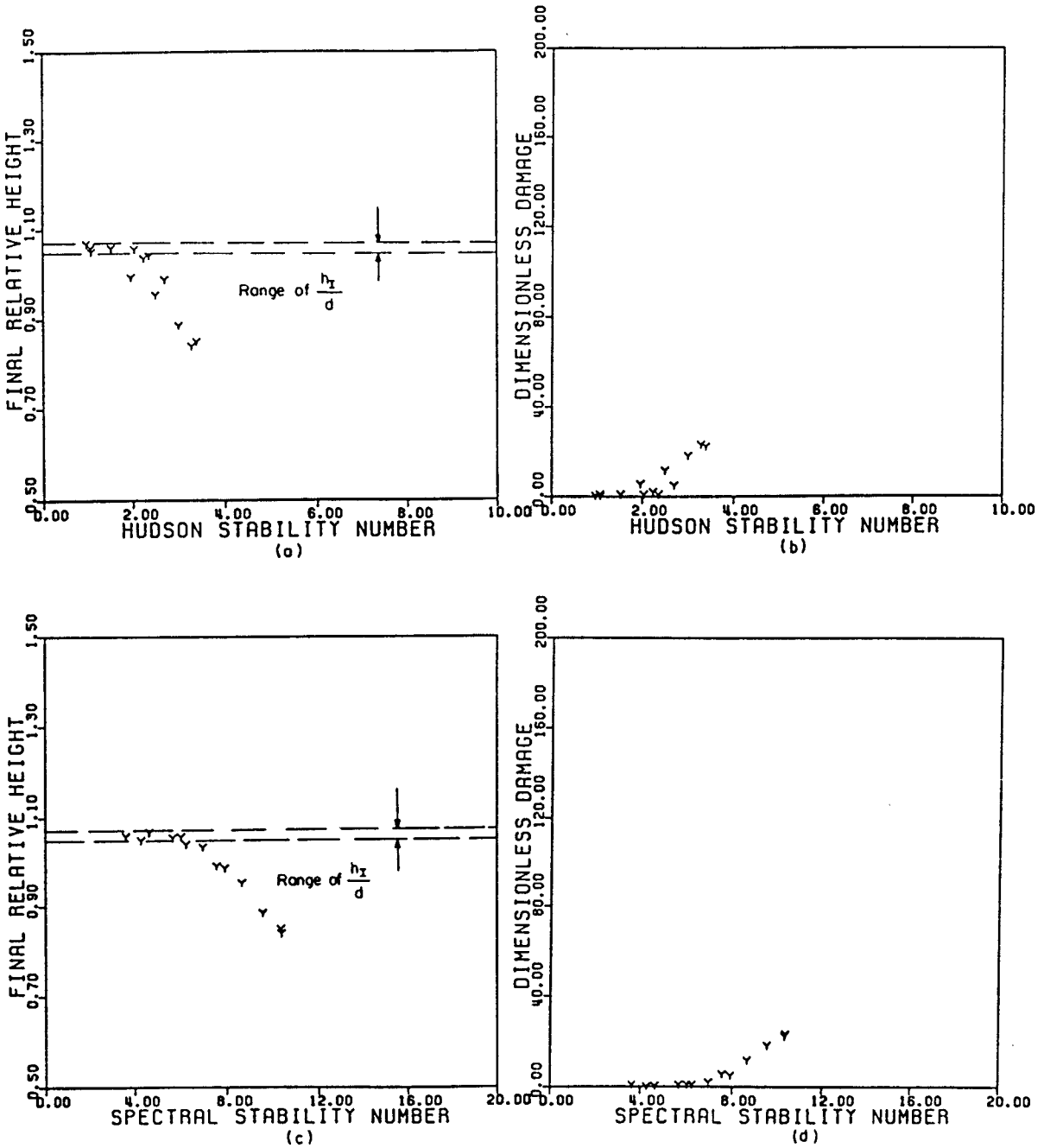


Figure 2.6 Damage parameters as a function of the Hudson stability number and the spectral stability number for subset 9.

lead to greater damage and lower crest heights. The data, however, exhibited considerable scatter. Scrutiny of the data with respect to peak incident wave period suggested that some of the scatter might be eliminated by the inclusion of a term to account for wave period effects. Graveson et al. (1980) present results of several different rubble mound stability studies conducted at the Danish Hydraulic Institute (DHI). They note that the dimensionless stability coefficient defined by

$$K = \frac{H_s^{\,3}}{D_{50}^{\,3}\left(\frac{w_r}{w_w} - 1\right)\cot\alpha} \tag{2-3}$$

where

α = slope angle,

is proportional to wave steepness, H_s/L_p where L_p is the wave length corresponding to the peak period. Thus, Hudson's stability number was modified to the following:

$$N_s^{\,*} = \frac{(H_s^2 L_p)^{1/3}}{D_{50}\left(\frac{w_r}{w_w} - 1\right)} \tag{2-4}$$

Using this parameter, Ahrens found that substantial reduction in scatter could be achieved (Figures 2.2 to 2.6(c,d)). A general trend seen in all the subsets is that the onset of damage occurs at about $N_s^* = 6$. Expected damage increases slowly as N_s^* approaches 8, and increases rapidly for $N_s^* > 8$.

The manner in which wave energy is distributed can be described by the equation

$$K_t^2 + K_r^2 + K_d^2 = 1 \tag{2-5}$$

where

K_t = transmission coefficient;

K_r = reflection coefficient; and

K_d = energy dissipation coefficient.

For these tests, K_r is given as the reflection coefficient measured during the last period of wave sampling in a test; the K_t value is the average transmission of all the sampling periods. Traditionally, K_t is defined by the ratio of the measured transmitted wave height to the measured incident significant wave height. This approach can lead to artificially low values of K_t, however, since some energy is lost due to internal and bottom friction between the wave gauges on the forward side of the breakwater and the wave gages measuring transmission. In order to ascertain the amount of energy transmission due to the breakwater only, the transmission coefficient was defined as

$$K_t = \frac{H_t}{H_c} . \tag{2-6}$$

where

H_t = the average value of significant wave height as measured at the back gages, and

H_c = the average incident significant wave height at the locations of the transmitted gages without the structure in place.

This definition gives a more conservative estimate of K_t than the traditional approach. Figure 2.7 illustrates the difference between the incident and calibrated wave heights.

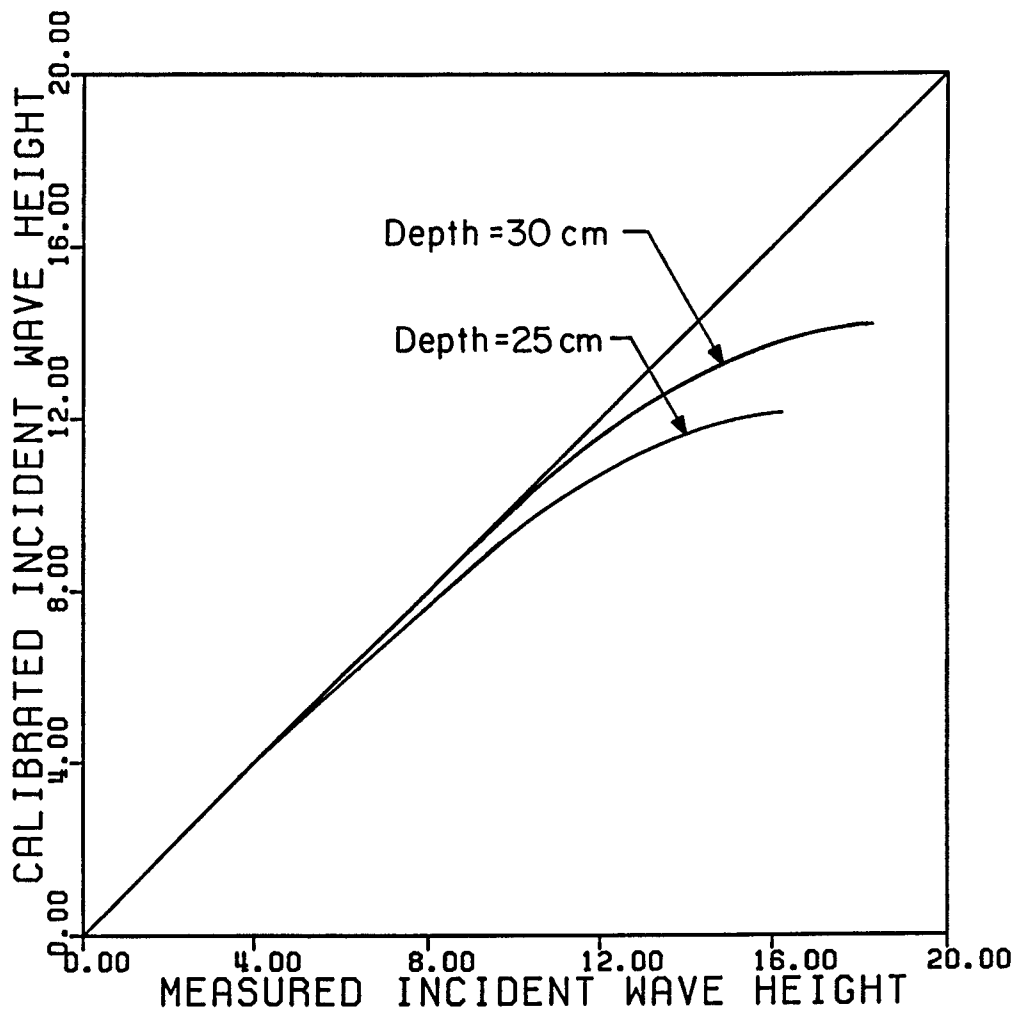


Figure 2.7 Comparison of the measured and calibrated incident wave heights.

In agreement with other researchers (Seelig 1979), Ahrens notes that the relative freeboard defined by

$$R = \frac{h_f - d}{H_s} \tag{2-7}$$

where

h_f = final crest height, and

d = water depth at structure site,

is the primary variable in the determination of K_t for overtopped and submerged structures, i.e., situations in which the dominant mode of transmission is overtopping. As R gets large, however, the dominant mode shifts from overtopping to transmission through the structure. Ahrens (1984) suggests that this transmission occurs at about R = 1.5. As the mode of transmission changes, so do the variables affecting K_t. Wave steepness, for example, becomes more important as R increases. Figure 2.8 shows the general trend exhibited by the data from these tests. The dashed line indicates the region in which transmission through the structure dominates. One should not interpret the dashed line to mean that wave transmission increases as the freeboard increases--this is clearly contrary to intuition--but rather that for a constant freeboard, smaller wave heights give larger transmission coefficients.

Finally, Ahrens (1984) presents a schematic graph of the general relationship between energy reflection, transmission and dissipation as a function R (Figure 2.9). One notes that the energy of long waves is not as easily dissipated as the energy in short waves. The difference is particularly obvious for wave reflection for larger values of R.

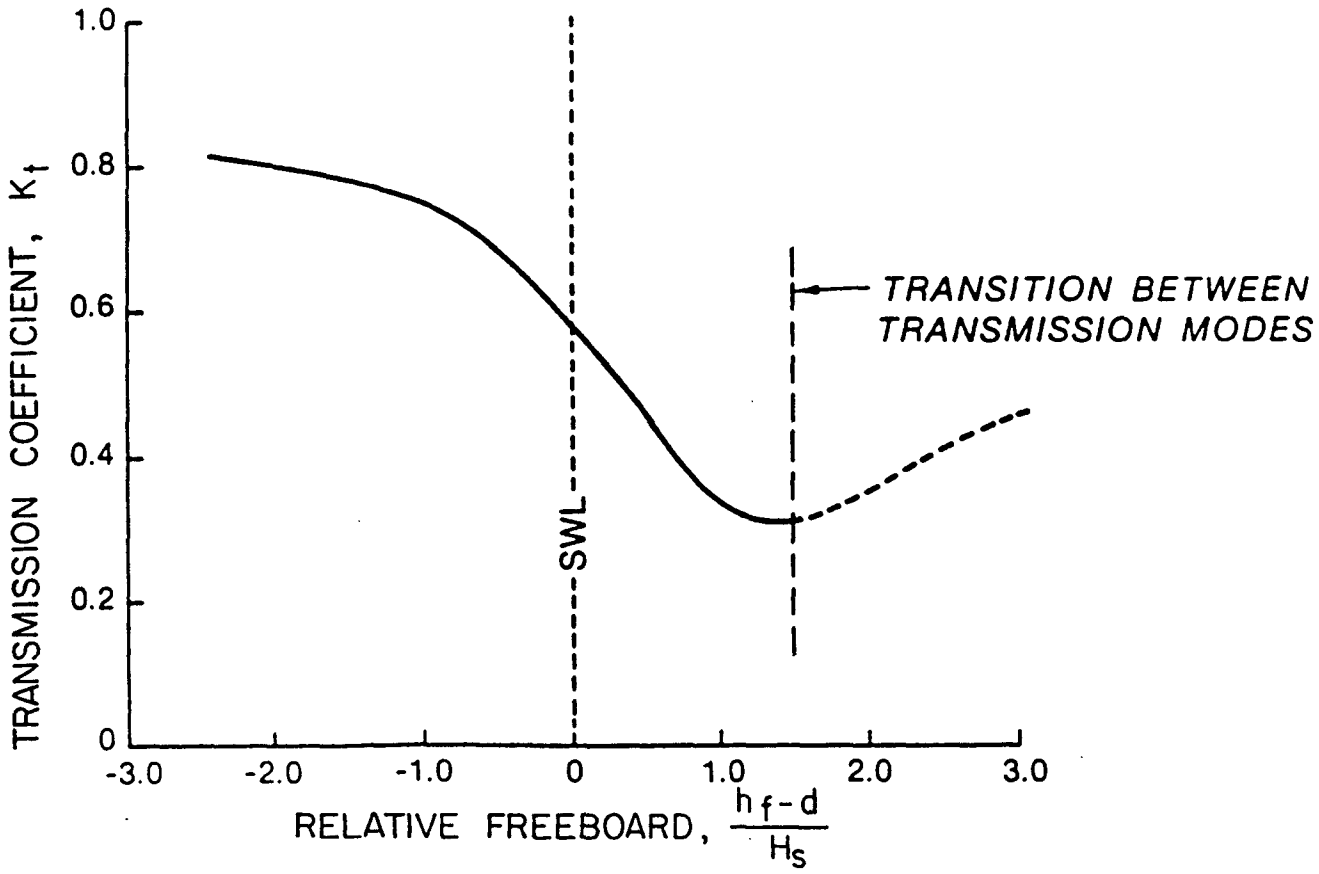


Figure 2.8 General trend for transmission coefficient vs. relative freeboard (after Ahrens 1984).

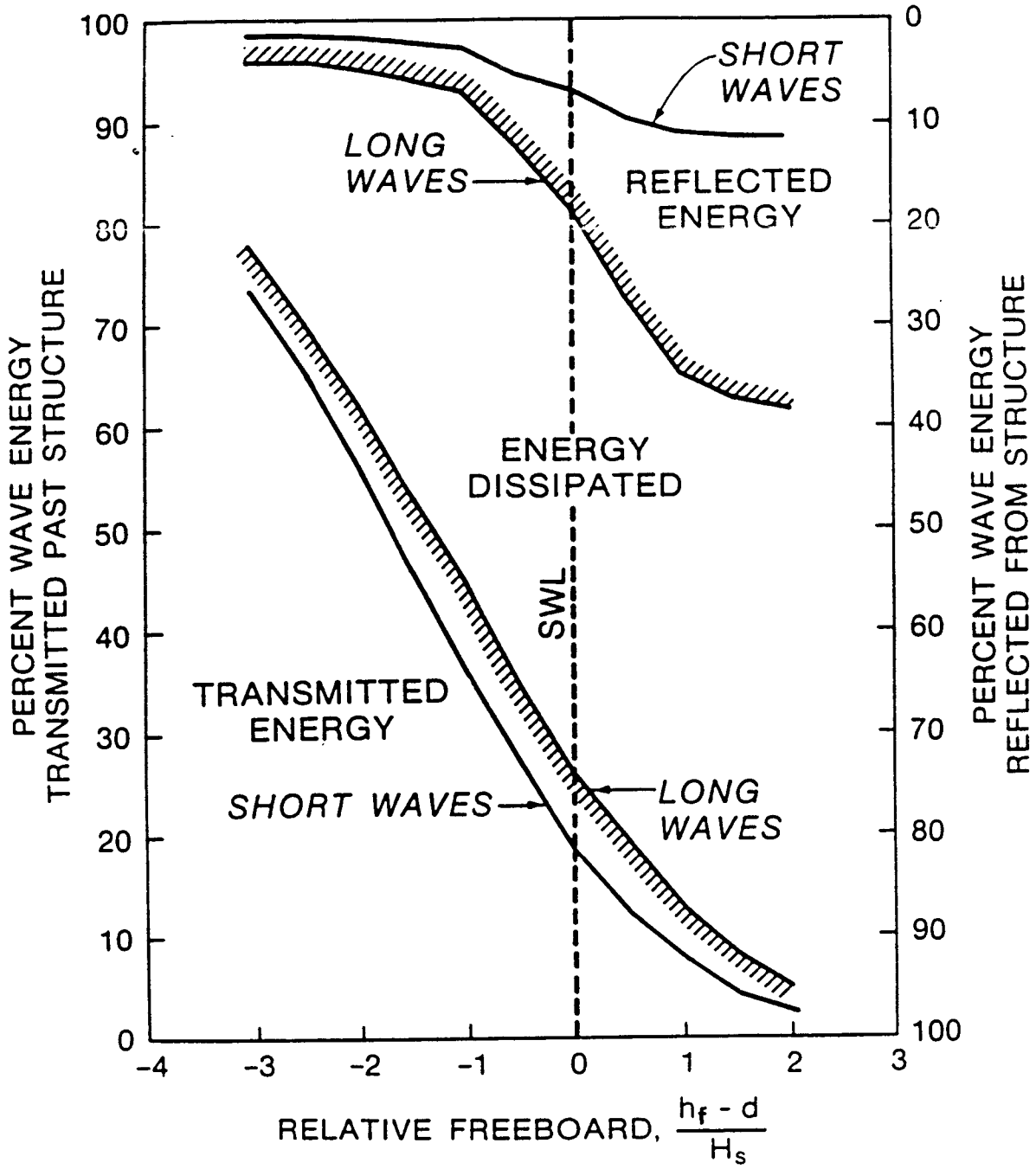


Figure 2.9 General relationship between energy reflection, transmission and dissipation as a function of relative freeboard (after Ahrens 1984).

CHAPTER 3 ANALYSES AND RESULTS

CERC provided the summarized damage and transmission data on 5 1/4 inch floppy disks in spreadsheet format for use with a personal computer. In addition, some of the computer printouts from which the summaries were compiled were supplied. Because it was faster, the spreadsheet files were transferred to the Coastal and Oceanographic Engineering Department's VAX 750 computer.

The spreadsheet summary includes the following information for each of the 205 tests.

- Subset number (1-10);
- Test type (stability or previously damaged);
- File number and wave maker signal amplification;
- Median stone mass, w_{50};
- Stone mass density, w_r;
- Cross-sectional area of breakwater, A_t;
- Water depth, d;
- Average incident significant wave height, H_s;
- Average incident peak period, T_p;
- Average transmitted significant wave height, H_t;
- Average reflection coefficient, K_r;
- Calibrated significant wave height, H_c;
- As built structure height, h_i;
- Damaged structure height, h_f;
- Area of damage, A_d;
- Peak, significant, and average incident and transmitted wave periods,

T_p, T_s, and $\overline{T}$, respectively, for the final sampling period of a test;

- Goda's spectral peakedness parameter, Q_p, for the final sampling period of a test;
- Fraction of displaced stone found seaward of the structure;
- Unsubmerged area of the damaged breakwater.

Discussion of the data analyses and results is in two parts. Section 3.1 deals with the mechanisms governing overall structure stability. Section 3.2 reviews the parameters influencing spectral changes and energy redistribution due to the breakwater.

3.1 STRUCTURAL STABILITY

Structural stability of overtopped breakwaters is complexly related to many factors, including stone shape, density, and median mass; incident wave height and period; ratio of structure height to water depth; storm hydrograph; and currents. As discussed by Foster and Khan (1984), the relationship between governing variables and stability is much more complex for overtopped than for non-overtopped breakwaters, and careful modeling of proposed structures is still the best way to obtain information about an individual structure's behavior. It is, however, beneficial to be able to predict the general performance of the structure in order to expedite testing.

Ahrens' data were examined exhaustively in an attempt to enhance existing understanding of stability of low-crested structures and to develop a viable preliminary design procedure. Prior to a discussion of analyses and results, it is enlightening to examine what is meant by "stability" of low-crested structures and how the expression of stability is related to structure shape and size. As discussed in Chapter 1, the

design problem associated with stability of a low-crested structure is not necessarily prevention of damage, but rather prediction of damage for a given set of incident wave conditions. For maintenance purposes, the volume of material displaced is needed. For prediction of energy transmission, the reduction in crest height, and, to a lesser degree, final structure width are important. It is obvious that structures of many different shapes may have the same total volume of material (cross-sectional area), but entirely different relationships between that volume and structural dimensions. For example, a rectangle with a given area has an infinite number of height to width ratios, and a rectangle and isosceles triangle with the same area and base dimensions have quite different heights. Similarly the damage area associated with a given reduction in crest height is a function of the initial structure size and shape. Thus, the application of results that utilize quantities such as damage area, total area, and structure height are necessarily restricted to structures of like shape. The structures used in this study were trapezoidal with front and back slopes of 1:1.5. The relationship between the total area and initial structure height was linear (Figure 3.1) and is described by the equation,

$$A_t = m\ h_i + b \tag{3-1}$$

where

$m = 98.7769\ cm^2/cm$; and

$b = -1285.44\ cm^2$.

Converting to prototype units,

$m = S\ (0.987769)\ m^2/m$; and

$b = S^2(-0.128544)\ m^2$

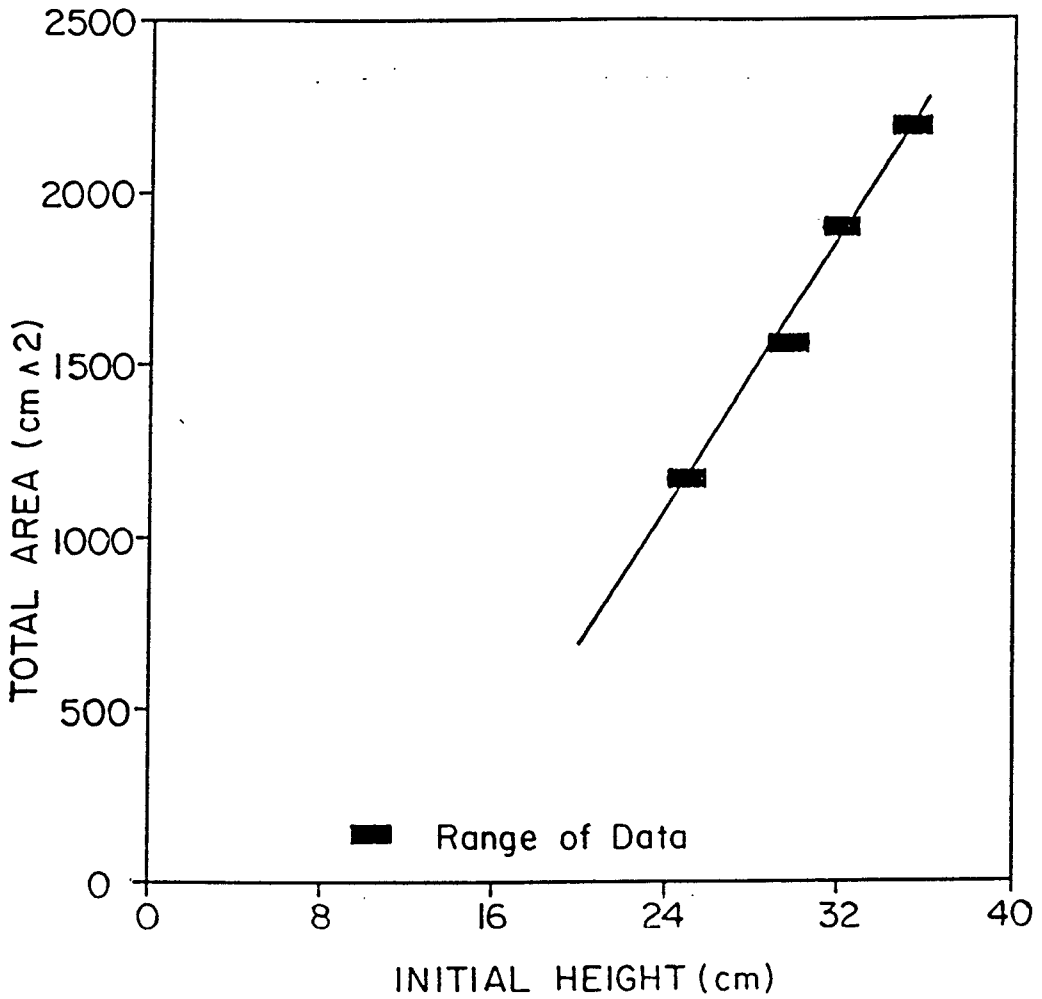


Figure 3.1 Relationship between initial structure height and total cross-sectional area.

and

$m = S\ (3.24067)\ ft^2/ft$; and

$b = S^2(1.38260)\ ft^2$

where

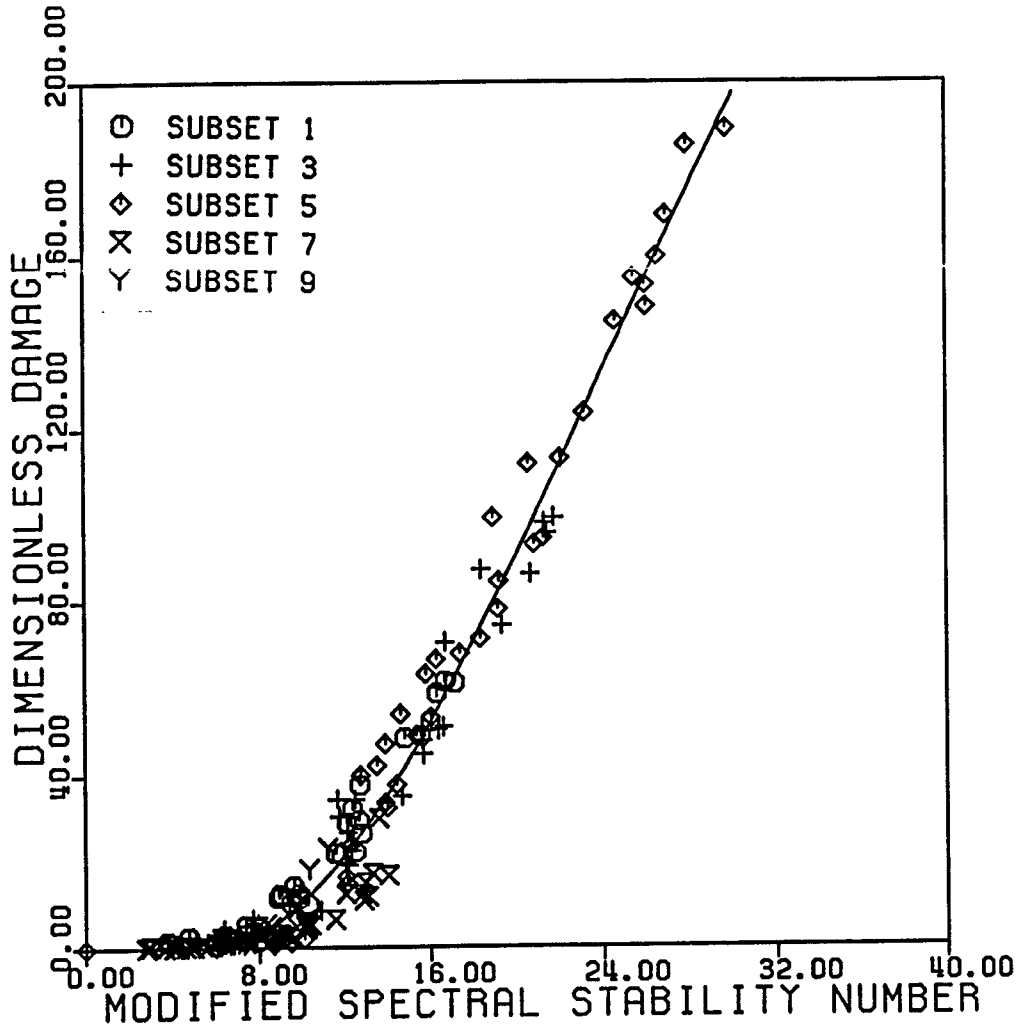
S = prototype to model length scale.

Based on Ahrens' observation that N_s* provides better definition of stability than N_s, the relationship between damage and N_s was not considered in the analyses. Instead, an extension and hopefully improvement of the Ahrens (1984) work were sought.

3.1.1 Volumetric Changes

Examination of Figures 2.2 - 2.6(d) reveals that the rate at which dimensionless damage increases as N_s* increases is dependent upon the ratio of initial structure height to water depth, h_i/d. Ahrens calls this the "exposure parameter" (personal communication, 1986). This observation is a reflection of the fact that the more of the structure that is exposed to direct wave attack the greater the volume of displaced material will be for the same N_s*. Often the structure stabilizes once the crest becomes submerged and the water acts as a protective cushion (Bremner, et al., 1980). Since dimensionless damage is a function of both N_s* and h_i/d, a new relationship is proposed in which dimensionless damage is related to the product of N_s* and $(h_i/d)^n$. It was found that the best correlation, using a least squares curve-fitting technique, is obtained for n = 1.5 (Figure 3.2). The curve shown is described by the equation,

$$D.D. = 19.4458 - 7.4546\ m + 0.760505\ m^2 - 0.010478\ m^3, \qquad (3\text{-}2)$$



$$N_s^* = \frac{(H_s^2 L_p)}{D_{50}(\frac{w_r}{w_w} - 1)}$$

Figure 3.2 Dimensionless damage as a function of the modified spectral stability number.

where

$$M = N_s^* \, (h_i/d)^{1.5},$$

and is valid within the limits $6 < M < 29$.

3.1.2 Crest Height Changes

The typical final structure profile was irregular in that the crest height varied along its length. For the purposes of this study, the final crest height, h_f, was specified by the highest surveyed point on the crest; h_f cannot, therefore, be geometrically related to A_d. For this reason, an estimation of h_f was made independent of A_d. Figures 2.2 - 2.6(c) show that h_f varies approximately linearly with N_s^* for $N_s^* \geqslant 6$, and constant h_i/d. All data from Figures 2.2 - 2.6(c) are shown on Figure 3.3(a). In a manner consistent with the observations in the last section, the rate of decrease of h_f/d increases as h_i/d increases. The data from subsets 7 and 9 appear to drop off more rapidly than is expected based on results from subsets 1, 3, and 5. This does not suggest that the structures built with larger stone suffered more damage. Rather, it may be an artificial effect resulting from the difference in stone size, i.e., the removal of one large stone shows up as a greater decrease in height than the removal of several smaller stones. Because the range of N_s^* tested and the number of tests where damage was measured were less for the large stone structures, the average trend is not as well-defined. It was, therefore, decided to use only data from subsets 1, 3, and 5 for this analysis. Based on these data, and a least squares analysis, a set of design curves is proposed as shown in Figure 3.3(b). Each line is for a constant value of h_i/d; interpolation is required for intermediate values of h_i/d. A summary of slopes and y-intercepts for each line is given in Table 3.1.

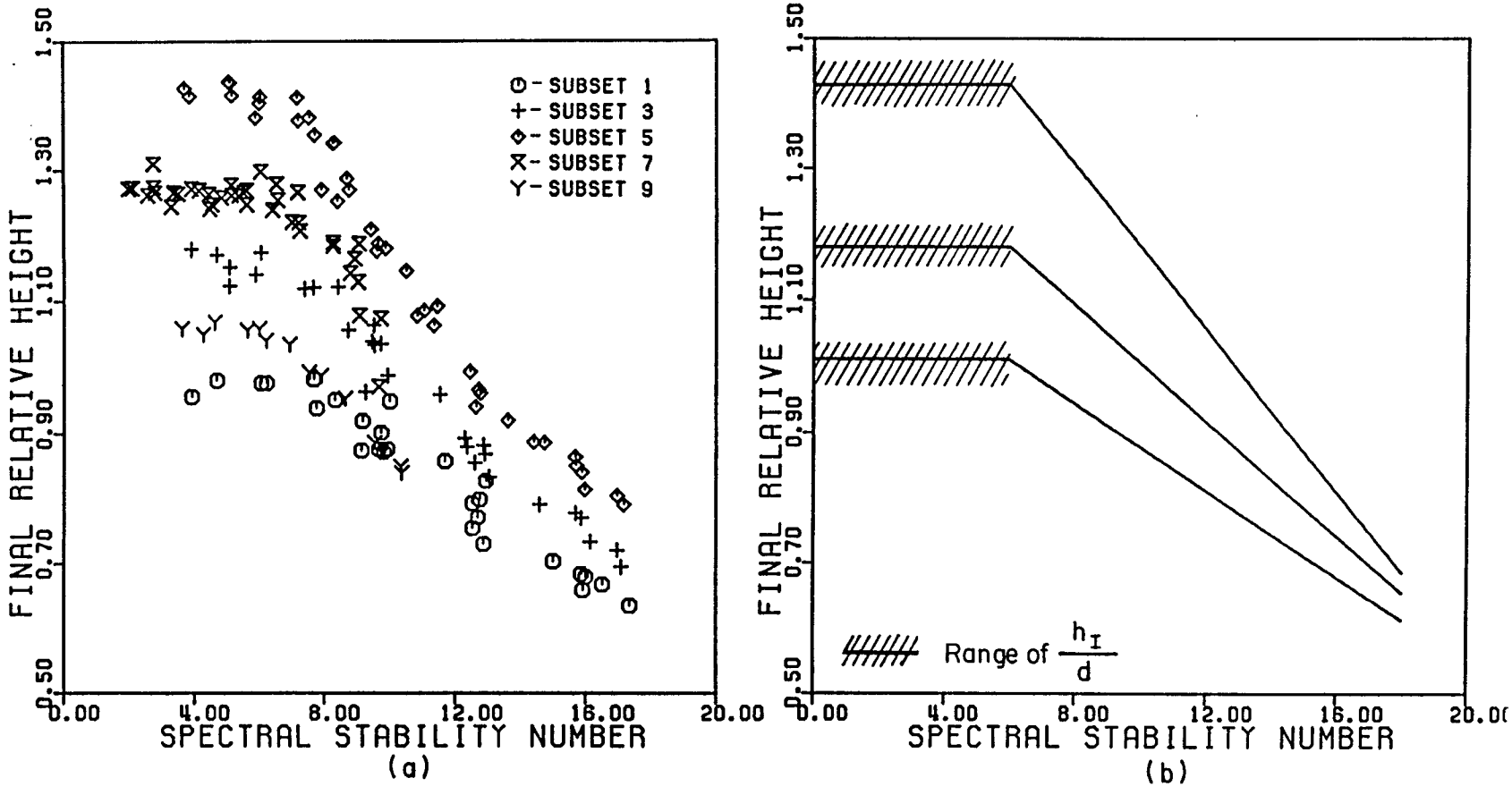


Figure 3.3 Relationship between final relative height and spectral stability number -- (a) data from type 1 tests; (b) design curves based on subsets 1, 3, and 5.

Table 3.1. Summary of slopes and y-intercepts for design curves in Figure 3.3(b)

Subset #	Range of $\frac{h_i}{d}$	Slope 10^2	y-intercept
1	0.9644 - 1.029	-3.32322	1.21020
3	1.150 - 1.219	-4.39221	1.44490
5	1.375 - 1.444	-6.19724	1.80112

3.2. WAVE FIELD MODIFICATION DUE TO THE BREAKWATER

Several types of changes to the wave field due to the presence of the breakwater were examined. Emphasis was placed on energy transmission, but attention was also given to energy reflection, shifts in the peak, significant, and average wave periods, and changes in the spectral peakedness parameter, as defined by Goda (1970). In addition, data from the two wave gages in the lee of the structure, gages 4 and 5, were compared. The data were also compared to the predictive methods of Seelig (1980) and Madsen and White (1976).

Calculation of the transmitted Q_p and the comparison of gages 4 and 5 are based on data from four runs from subset 3, all of subsets 5, 6, 8 and 9, and all but a few runs in subset 7.

3.2.1 Comparison of Transmission Gages

3.2.1(a) Wave height

Figure 3.4 shows the significant wave heights measured by gages 4 and 5 for each of the four data files. It is clear from these plots that the magnitude of the discrepancy between the gages is in part a function of the incident T_p. In general, the measured difference increases as T_p increases and within each file as wave height increases. The differences are probably caused by reflection from the absorbing material at the end of the wave tank. As the wave period (wave length) increases, so do the amount of reflection for a given wave height and the difference in wave heights measured by gages 4 and 5. Little if any difference is seen in the File 1 data. File 2 data show only small absolute differences--up to about 3/4 cm--but these can translate into substantial percent differences, particularly for the smaller waves. Usually, but not always, the larger H_t was measured by gage 4. The File 3 and 4 data have about

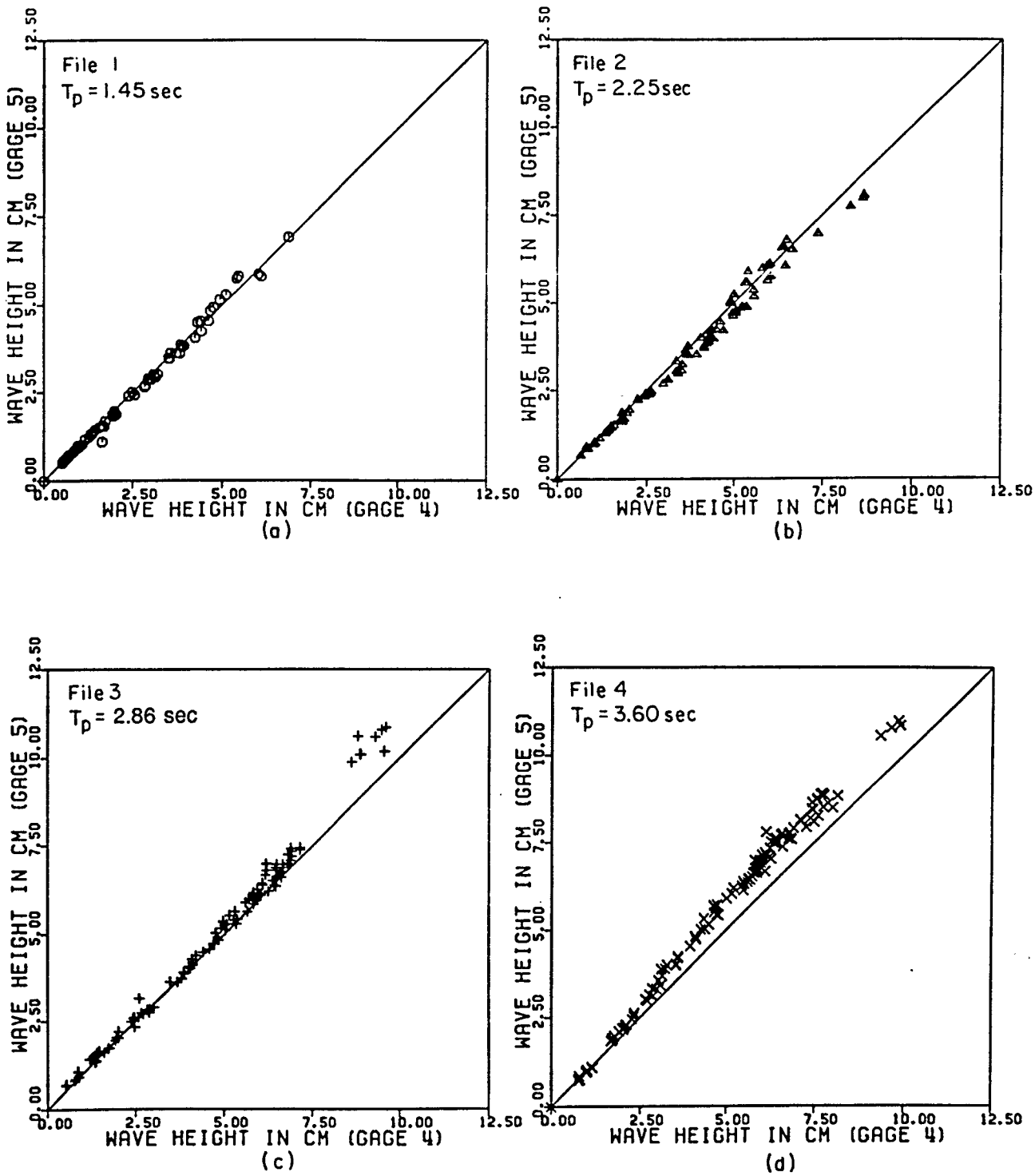


Figure 3.4 Comparison of transmitted wave heights measured at gages 4 and 5.

the same maximum absolute difference ($1\frac{1}{2}$ - 2 cm); H_t at gage 5 was always greater than H_t at gage 4. Note also that the absolute difference for File 3 increases gradually as wave height increases, but the discrepancy in File 4 data grows rapidly to about H_t = 3 cm and more slowly thereafter. The result is that the percent differences for File 4 are greater than for File 3.

It is difficult to assess the error due to reflection that is introduced into the calculated value of H_t because the measured values of H_t depend upon the location of the wave gages with respect to the partial standing wave. Goda's resolution procedure should be used in order to ensure that the most accurate transmitted wave height is obtained.

3.2.1(b) Goda's spectral peakedness parameter

Goda (1970) defines spectral peakedness as

$$Q_p = \frac{2 \int_0^\infty fS(f)^2 df}{\left(\int_0^\infty S(f)df\right)^2} \tag{3-3}$$

where,

f = frequency; and

S(f) = value of the energy density spectrum.

In differential notation,

$$Q_p = \frac{2}{\Delta f} \frac{\sum_{j=1}^{\infty} f_j a_j^4}{\left(\sum_{j=1}^{\infty} a_j^2\right)^2} \tag{3-4}$$

where

f_j = frequency at the midpoint of the band; and

Δf = spectral band width.

The higher the value of Q_p, the more peaked the spectrum.

The Q_p values for the wave spectra measured by gages 4 and 5 were calculated and are presented in Figure 3.5. To maintain consistency for comparison with the incident Q_p, all Q_p values were calculated using the range of frequencies spanned by the incident spectrum. Except for File 1, the trends observed are generally consistent with the observations of Section 3.2.1(a). Figure 3.5(a) suggests that the spectral peak at gage 4 is greater than the peak at gage 5, but no corresponding difference in wave height is observed (Figure 3.4(a)). Plots of the File 1 energy spectra show only small differences in the energy densities measured by the two gages. Thus it seems that the Q_p values are more sensitive than the wave heights to small differences in energy density.

3.2.2 Energy Transmission

3.2.2(a) Variation of K_t within a test

As discussed in Section 2.2, the K_t value obtained by Ahrens for use in subsequent analysis was calculated using,

$$K_t = \frac{1}{m} \sum_{n=1}^{m} \frac{(K_{t4} + K_{t5})_m}{2} \qquad (3\text{-}5)$$

where

m = number of sampling periods;

K_{t4} = transmission coefficient obtained using H_t from gage 4; and

K_{t5} = the transmission coefficient obtained using H_t from gage 5.

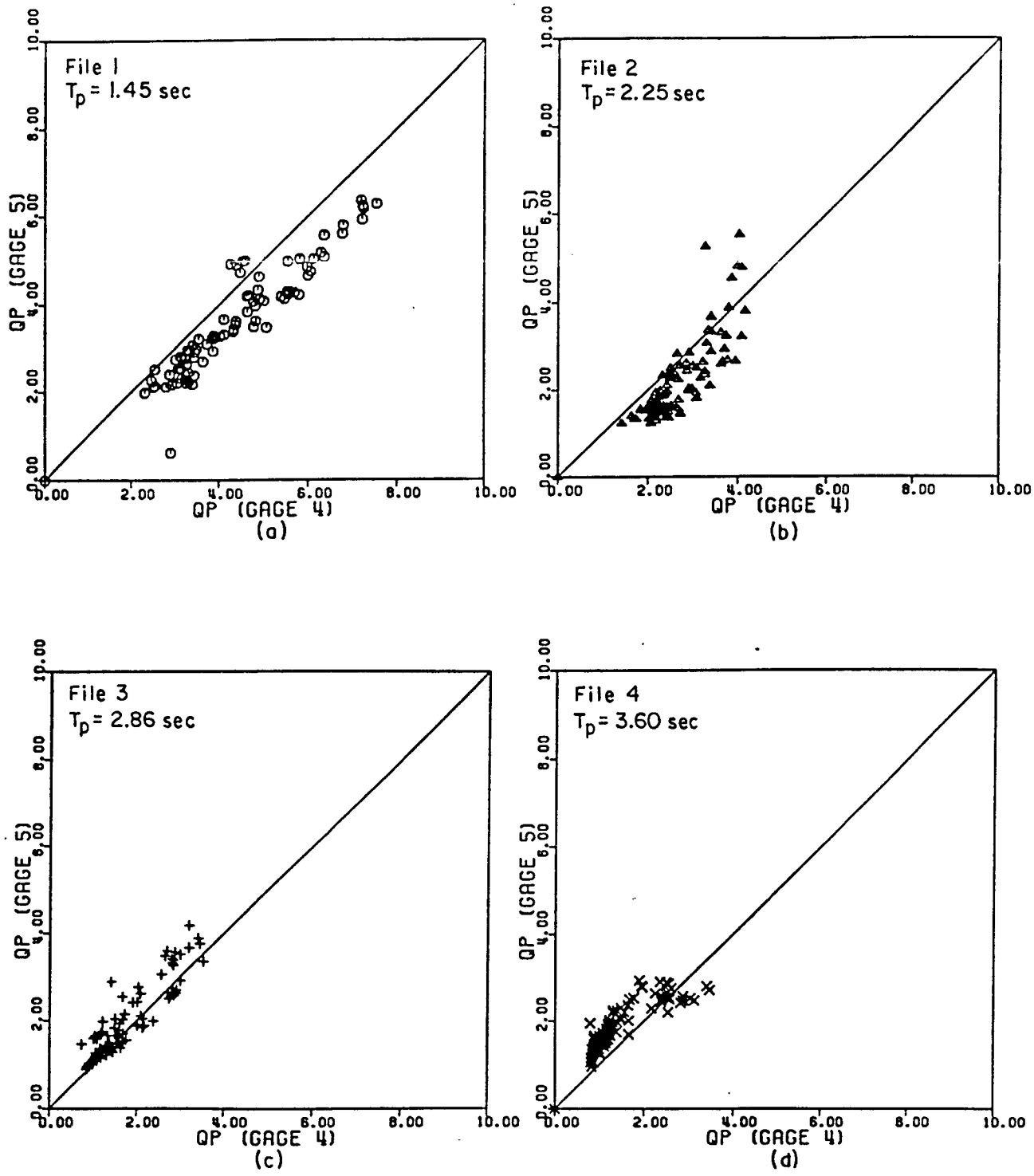


Figure 3.5 Comparison of spectral peakedness parameters (Q_p) measured at gages 4 and 5.

This value gives the average K_t that can be expected for the given storm event, but if K_t changes substantially from the beginning to the end of a test, it may not be adequate for prediction of the maximum transmission.

Figure 3.6 presents typical examples of K_t vs. sampling period for one test from each of the four files. The difference between the gages is due to the difference in the measured H_t (Section 3.2.1(a). Due to the reduction in freeboard, energy transmission increased as the tests progressed. The increase is smaller, however, than would be expected if the crest height had been reduced without the accompanying increase in structure width. This observation is consistent with those of Bremner, et al. (1980). The maximum absolute difference in K_t for the four files ranged from about 0.06 to 0.17, but the percent change was as high as 46 percent. These changes are the same order of magnitude as the difference between the values of K_t measured by gages 4 and 5. Without resolving the discrepancy between the gages, it is inappropriate to do a detailed transmission analysis based on the "maximum" K_t. Even greater inaccuracies could be introduced, because the existing errors may tend to cancel one another. Figure 3.6 suggests, however, that the increase in K_t should be addressed in future studies.

3.2.2(b) Prediction of K_t

The relative importance of the parameters governing energy transmission and, therefore, the method used to predict K_t change as the dominant mode of transmission shifts from overtopping to flow through the structure. Figure 3.7 presents the average K_t as a function of relative freeboard, R. The relationship between K_t and R changes at about R = 1.0 because as R increases, transmission by overtopping approaches zero, and the importance of freeboard in determining K_t is diminished. Ahrens

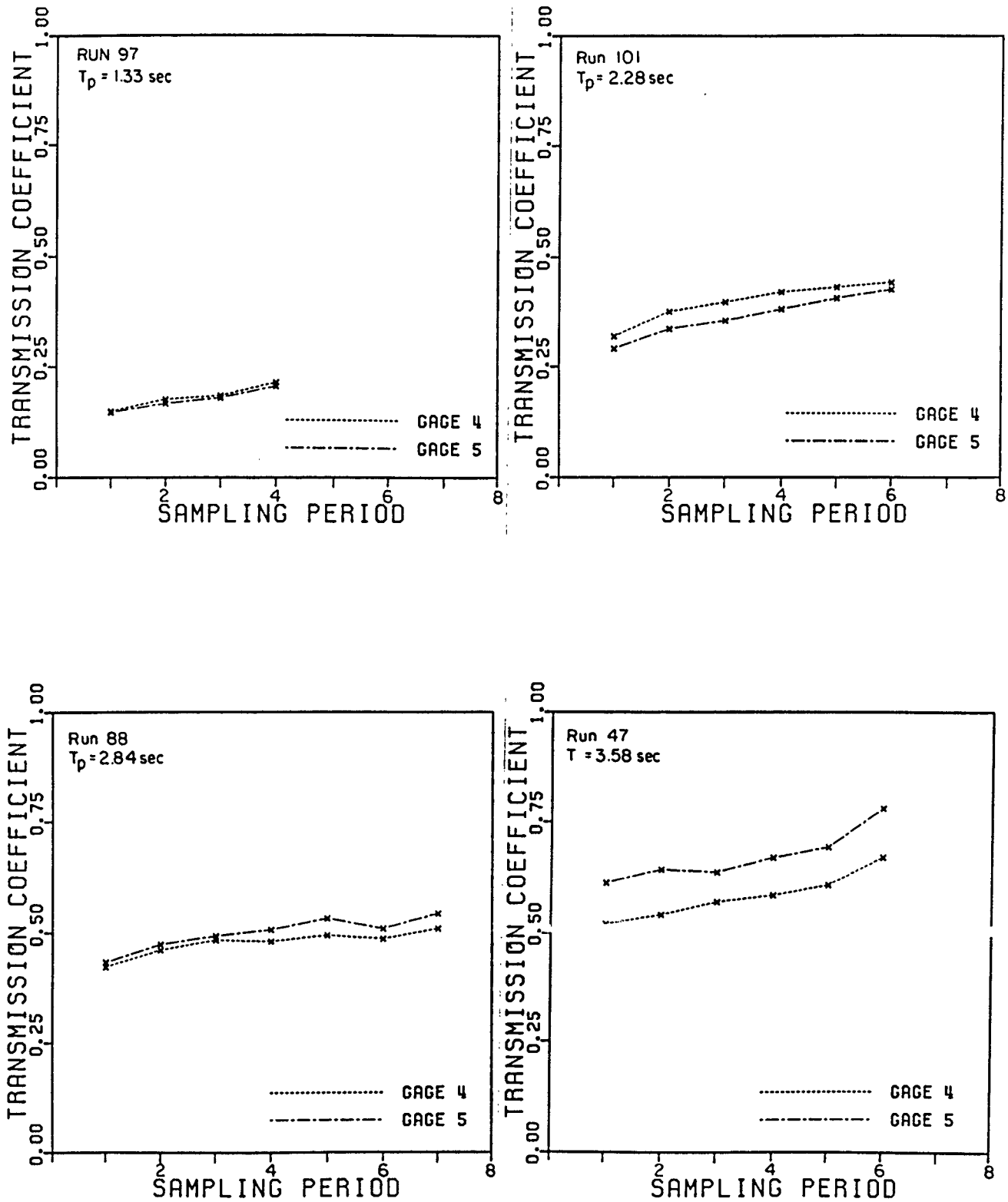


Figure 3.6 Change in transmission coefficient from the beginning to the end of damage tests.

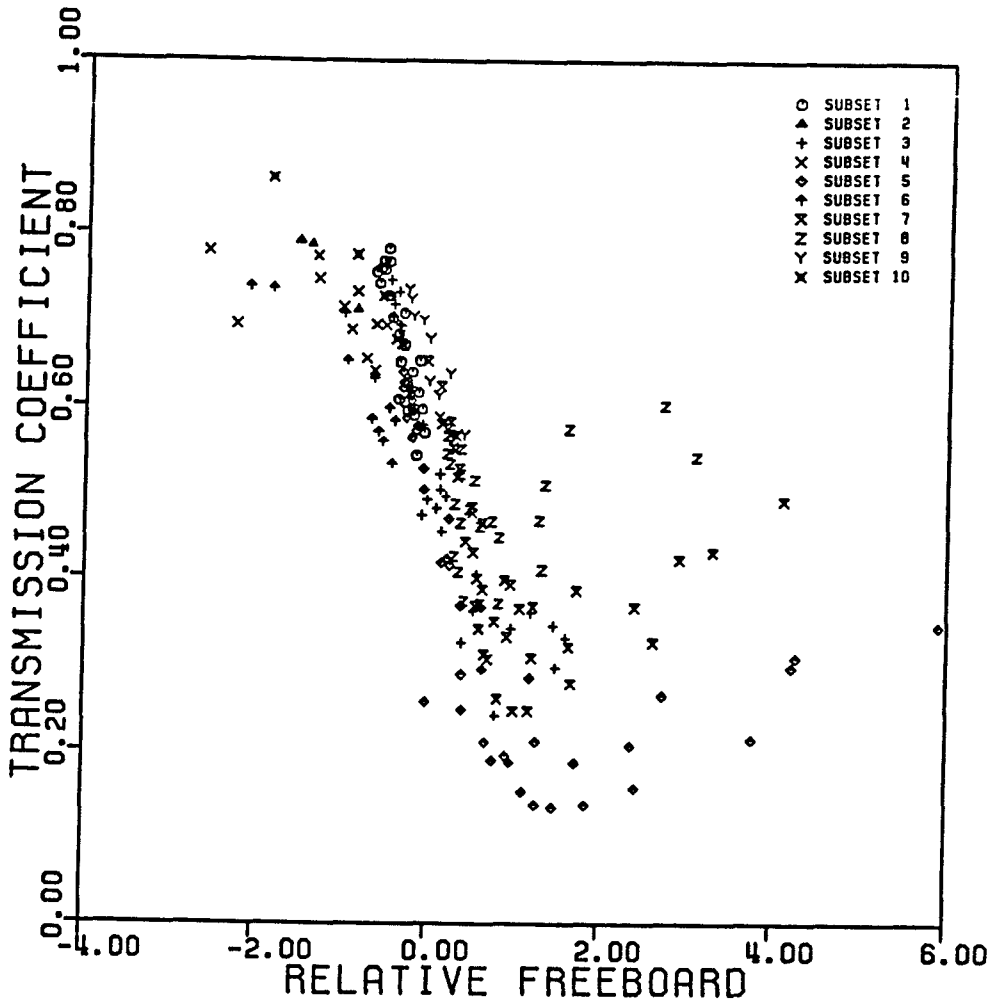


Figure 3.7. Transmission coefficient as a function of relative freeboard.

(personal communication, 1986) found that K_t is a function of the parameter, P, for R > 1.0, where

$$P = \frac{H_s A_t}{L_p (D_{50})^2}, \tag{3-6}$$

(Figure 3.8). Note that, in this formulation, K_t is independent of freeboard for R > 1.0. At values of R ≤ 1.0, K_t depends upon both P and R, i.e., transmission is by a combination of the two mechanisms. The relative importance of P decreases as R gets smaller.

The parameter, P incorporates the influences of wave period, stone size, structure area or width, and H_s all of which can be examined separately. Figures 3.9(a) and 3.9(b) show the transmission data for R ≤ 1.0. From Figure 3.9(a), it is clear that the longer waves tend to produce a higher K_t than the short waves, all else being equal. Unfortunately, there are no long wave data for R less than about -0.6. The available data indicate, however, that the K_t for Files 3 and 4 and low R values would be higher than for Files 1 and 2. Figure 3.9(b) shows that the K_t for subsets 7-10 is consistently higher than the K_t for subset 1-6. The difference is probably due to the larger void spaces resulting from the use of larger stones in the structures of subsets 7-10. Also, K_t is generally smaller for subsets with larger cross-sectional areas, all else being equal, suggesting additional energy dissipation across the crest and/or through the structure.

The approach used to predict K_t varies depending upon the value of R. For R ≤ 0.0, K_t is assumed to be a function of R only and is predicted using an exponential curve of the form

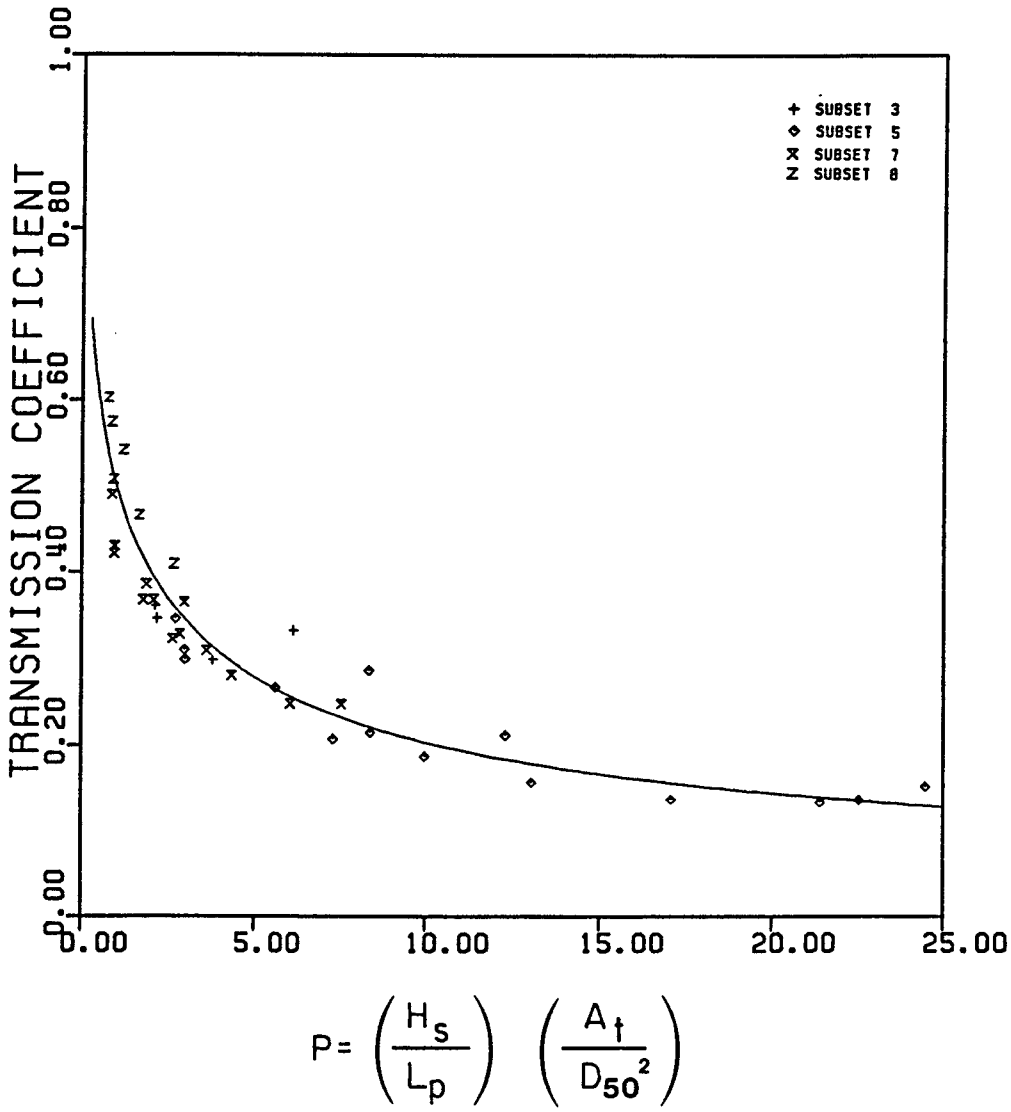


Figure 3.8 Transmission coefficient as a function of P for relative freeboards greater than 1.0.

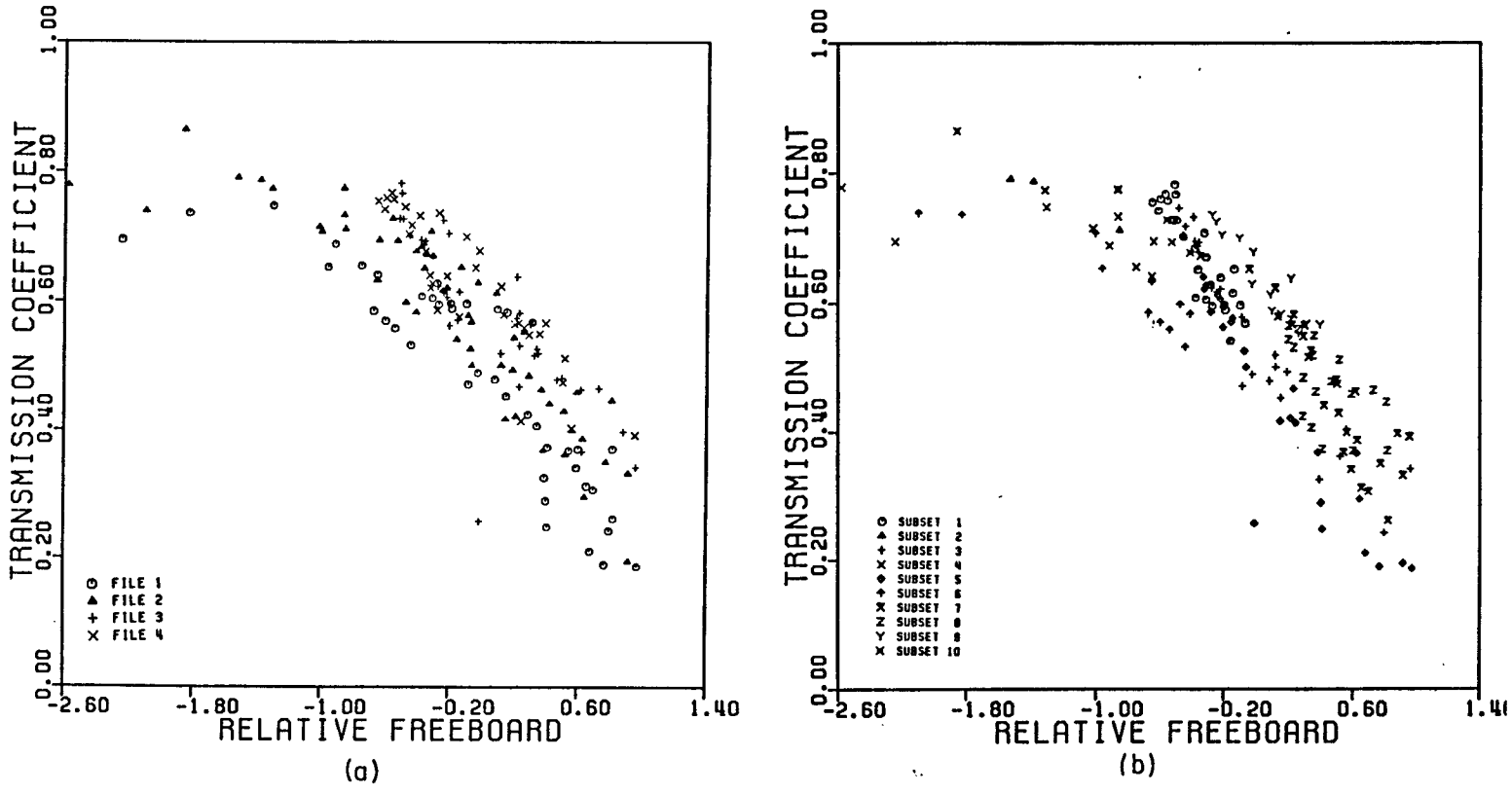


Figure 3.9 Transmission coefficient as a function of relative freeboard, R, for R ≤ 1.0. Data are separated by both peak incident wave period (a) and subset (b).

$$K_t = A11 + A21\ e^R \tag{3-7}$$

where

A11 = 0.9 and

A21 = -0.358.

For $0.0 \leq R \leq 1.0$

$$K_t = \frac{1.0}{A12 + A22\ R} \tag{3-8}$$

where

$$A12 = \frac{1.0}{A11 + A21} = 1.845 \text{ and} \tag{3-9}$$

$$A22 = (1 - A12) + P^a$$

$$= -0.845 + P^a \tag{3-10}$$

where

a = 0.5926

For $R > 1.0$, K_t is a function of P only. The relationship shown in Figure 3.8 is given by

$$K_t = \frac{1}{1 + P^a} \tag{3-11}$$

(Ahrens, personal communication, 1986). Design curves for different values of P are shown in Figure 3.10. Details of this development are discussed in Chapter 4.

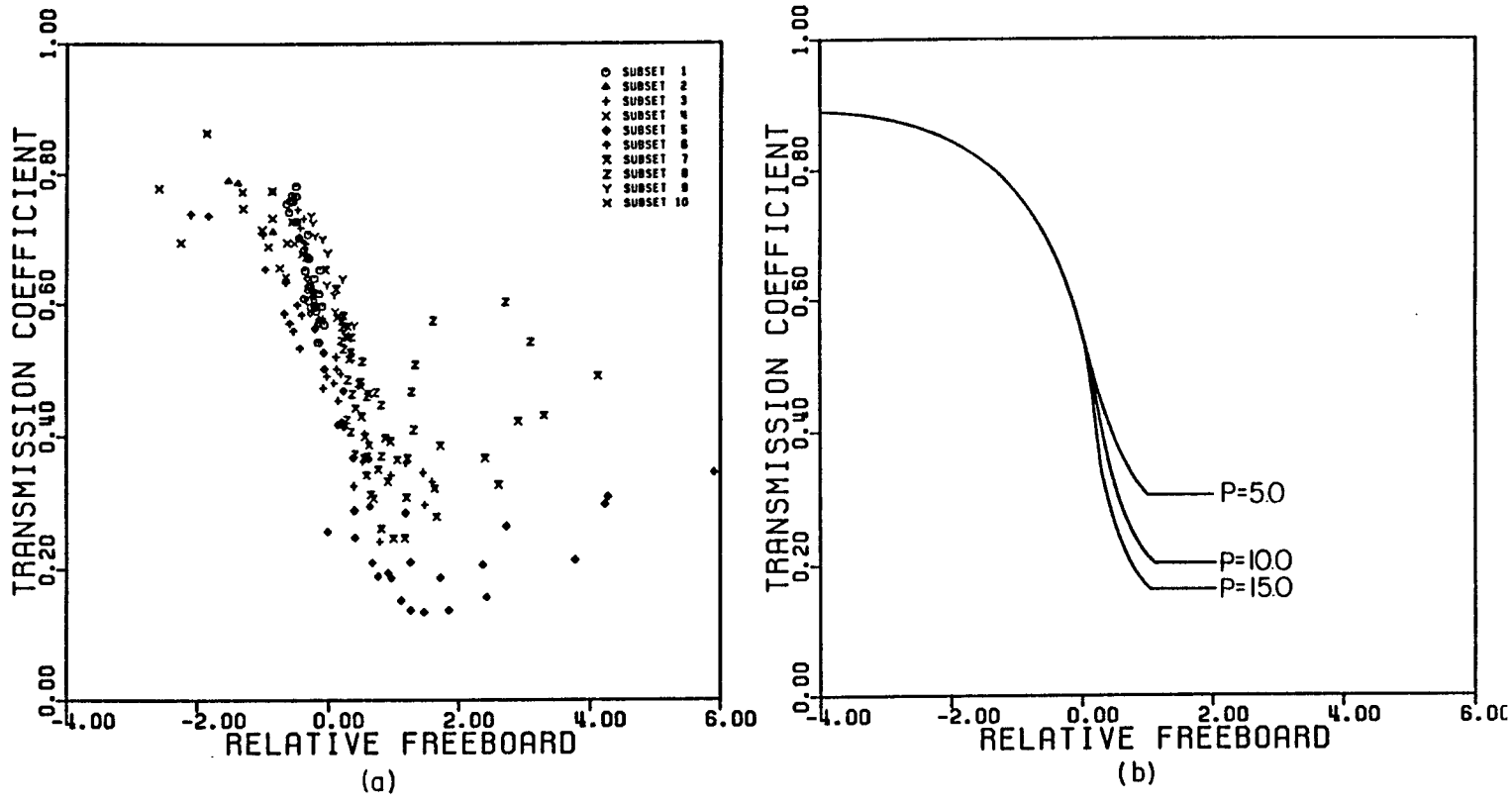


Figure 3.10 Design curves for the prediction of transmission coefficient as a function of relative freeboard and P.

3.2.2(c) Comparison of data with predictive approaches of other researchers

Seelig (1979, 1980) found that the transmission coefficient due to overtopping for a structure fronted by a 1:15 slope is given by

$$K_t = C\ (1 - F/U) - (1 - 2C)\ F/U \text{ and} \qquad (3\text{-}12)$$
$$C = 0.51 - 0.11\ B/h$$

where

F = freeboard;

U = Wave runup;

B = crest width; and

h = structure height.

Madsen and White (1976) derived an analytical solution for transmission through trapezoidal, permeable, multi-layered structures. CERC program, MADSEN, (presented in Seelig, 1980) calculates the total transmission combining both approaches using

$$K_t\ (\text{total})^2 = K_t\ (\text{overtopping})^2 + K_t\ (\text{through})^2. \qquad (3\text{-}14)$$

In order to compare these approaches with the data, the width of the damaged structure had to be estimated. This was done by assuming that the final structure shape is a trapezoid similar to the initial shape, e.g., parallel on top and bottom with side slopes of 1:1.5. The material removed from the top is redistributed at the front and back sides. The increased area at the sides is equal to the area of damage, A_d (Figure 3.11). The total cross-sectional area, A_t, is the same for both profiles, so that A_t is expressed in terms of initial conditions as

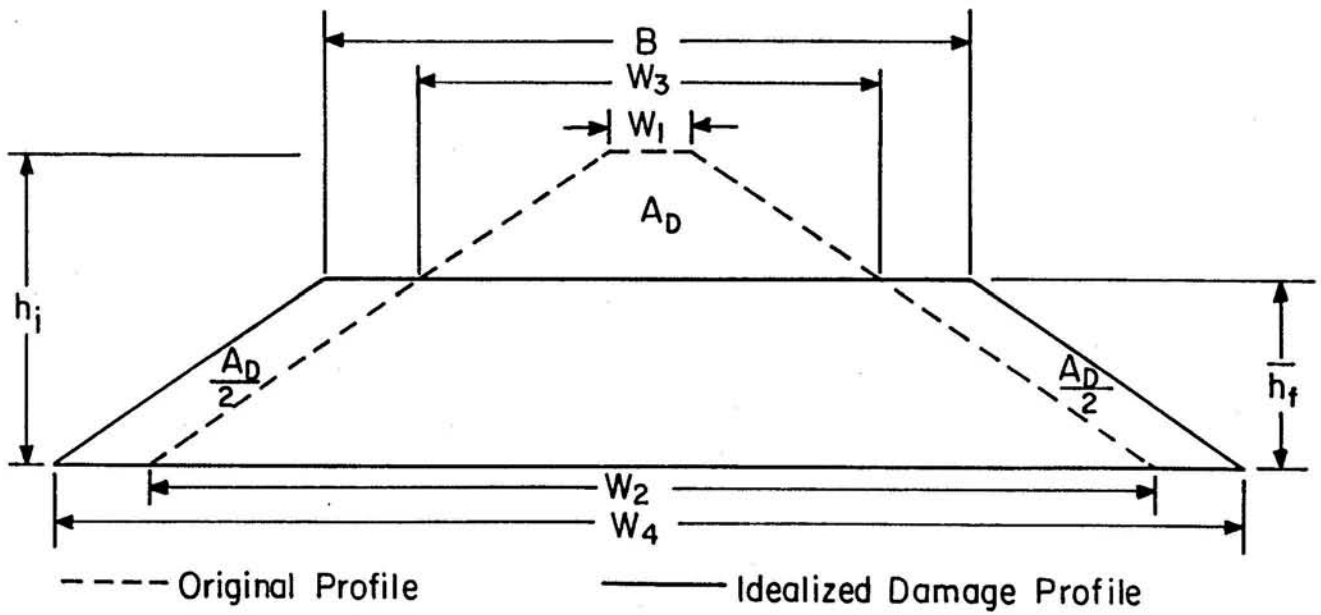


Figure 3.11. Definition sketch of idealized damaged structure.

$$A_t = \left(\frac{B + W_4}{2}\right) \overline{h}_f. \tag{3-15}$$

Damage area is given by

$$A_d = \left(\frac{W_1 + W_3}{2}\right) \Delta h \tag{3-16}$$

where

Δh = the change in crest height $(h_i - \overline{h}_f)$.

The values W_1, W_2, W_3, W_4, and B are given by

$$W_1 = \frac{A_t}{h_i} - 1.5\ h_i, \tag{3-17}$$

$$W_2 = W_1 + 3h_i, \tag{3-18}$$

$$W_3 = W_1 + 3\Delta h, \tag{3-19}$$

$$W_4 = B + 3\overline{h}_f \text{ and} \tag{3-20}$$

$$B = \frac{A_t}{h_f} - 1.5\ \overline{h}_f. \tag{3-21}$$

All variables in the above equations are known except $\overline{h}_f$, which represents the average final crest height of the structure. The measured final crest height is not used because it is the highest point on the crest and would give a value for B that is inconsistent with the known values of A_t and A_d. To solve for $\overline{h}_f$, equations 3 and 6 were combined to obtain

$$1.5(\Delta h)^2 + W_1 \Delta h - A_d = 0 \tag{3-22}$$

which is a quadratic in Δh. The solution is the positive root of

$$\Delta h = \frac{-W_1 \pm (W_1^{\,2} + 6\,A_d)^{\frac{1}{2}}}{3}. \tag{3-23}$$

The average final crest height is thus, $\overline{h}_f = h_i - \Delta h$.

Comparison of the data with the predicted K_t by overtopping (Seelig, 1980) and with the predicted total K_t (Madsen and White, 1976) are shown in Figures 3.12(a, b) and (c, d), respectively. Predicted K_t is plotted against R in Figure 3.13. As expected, the predicted K_t due to overtopping is less than the total measured K_t, with the discrepancy increasing as T_p increases. Overprediction occurs at low relative freeboards, and no transmission is accounted for when $R > 1.0$. Consideration of transmission through the structure gives some improvement, especially for cases of $R > 1.0$, but there is still considerable scatter in the data. In particular, when the subsets are considered individually the predicted values are unsatisfactory. Predicted K_t has much less variation than the measured data show. A possible explanation is that Seelig's formulation is valid for $0.88 < B/h < 3.2$, but the range of B/h for these tests is 0.23 to 5.9. For larger values of B/h, the relationship may overaccount for structure width.

3.2.3 Energy Reflection

The reflection coefficient as a function of R is shown in Figure 3.14(a). There is a clear separation of the data into two groups based on T_p, with the longer waves of Files 2, 3, and 4 having a higher K_r than the

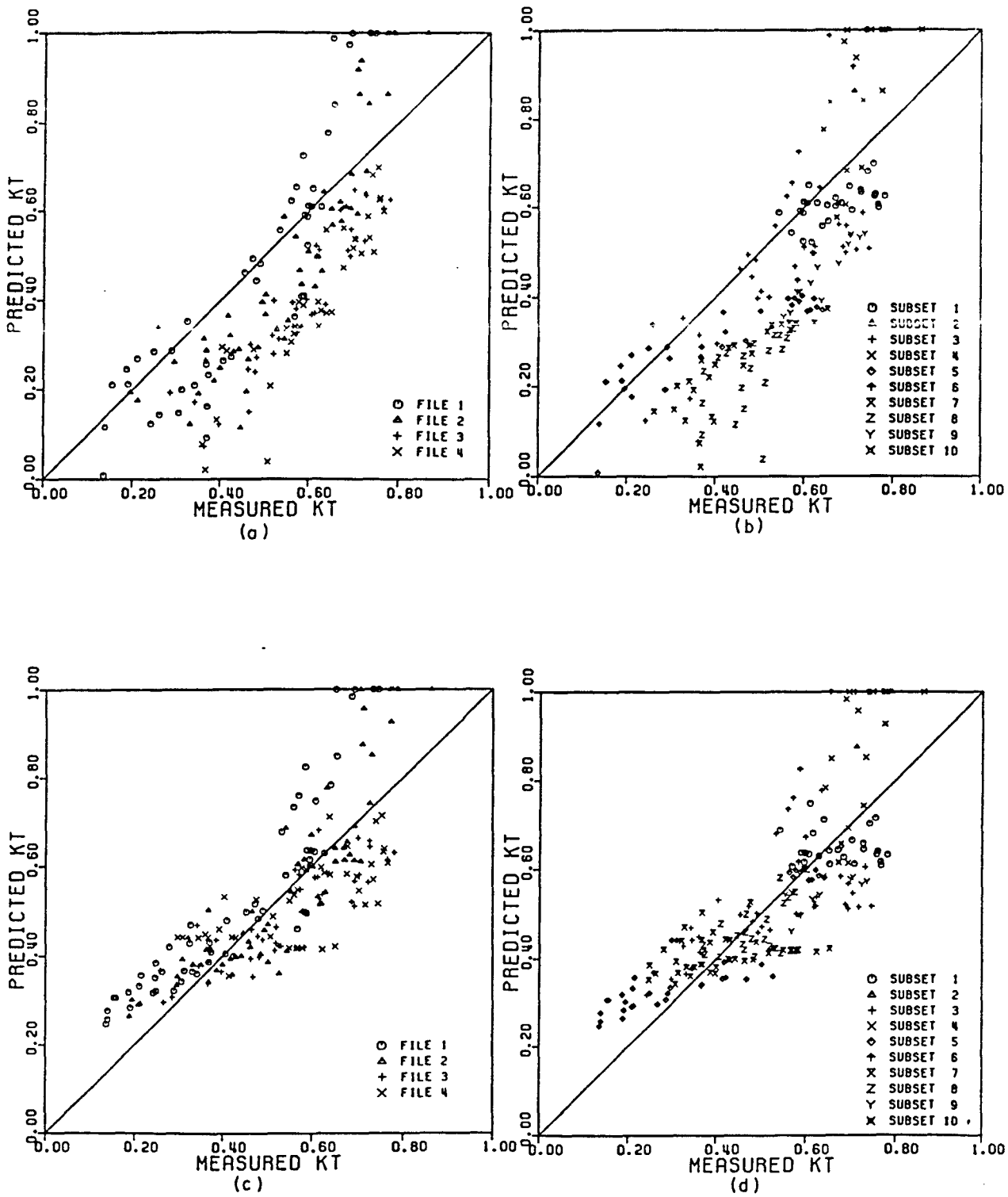


Figure 3.12. Comparison of measured transmission coefficients with those predicted by the approaches of Seelig (a, b) and Madsen and White (c, d). Seelig (1980) accounts for transmission by overtopping only. Madsen and White (1975) consider both overtopping and transmission through the structure.

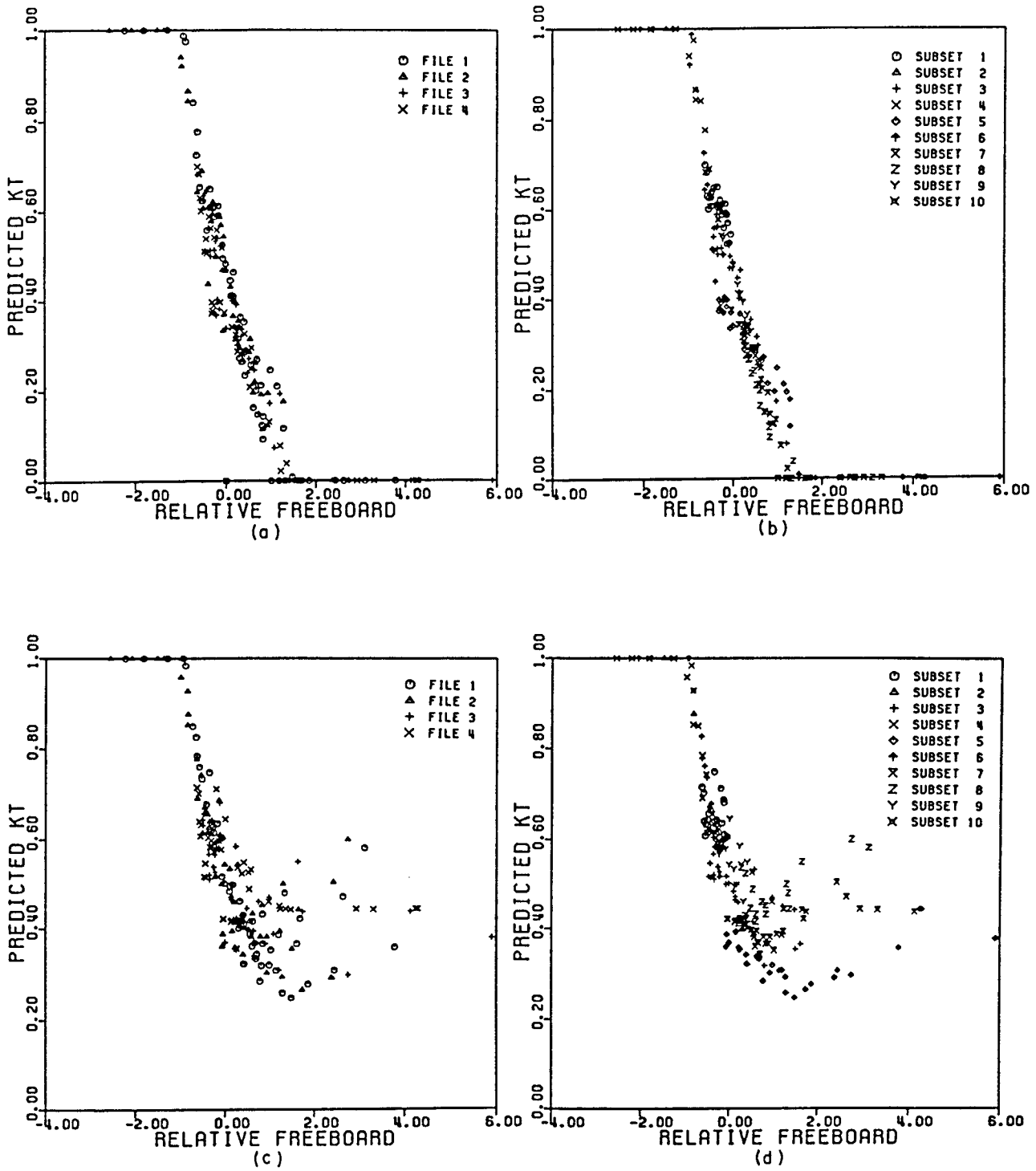


Figure 3.13 Transmission coefficients predicted by the approaches of Seelig (a, b) and Madsen and White (c, d) as a function of relative freeboard. Seelig (1980) accounts for transmission by overtopping only. Madsen and White (1975) consider both overtopping and transmission through the structure.

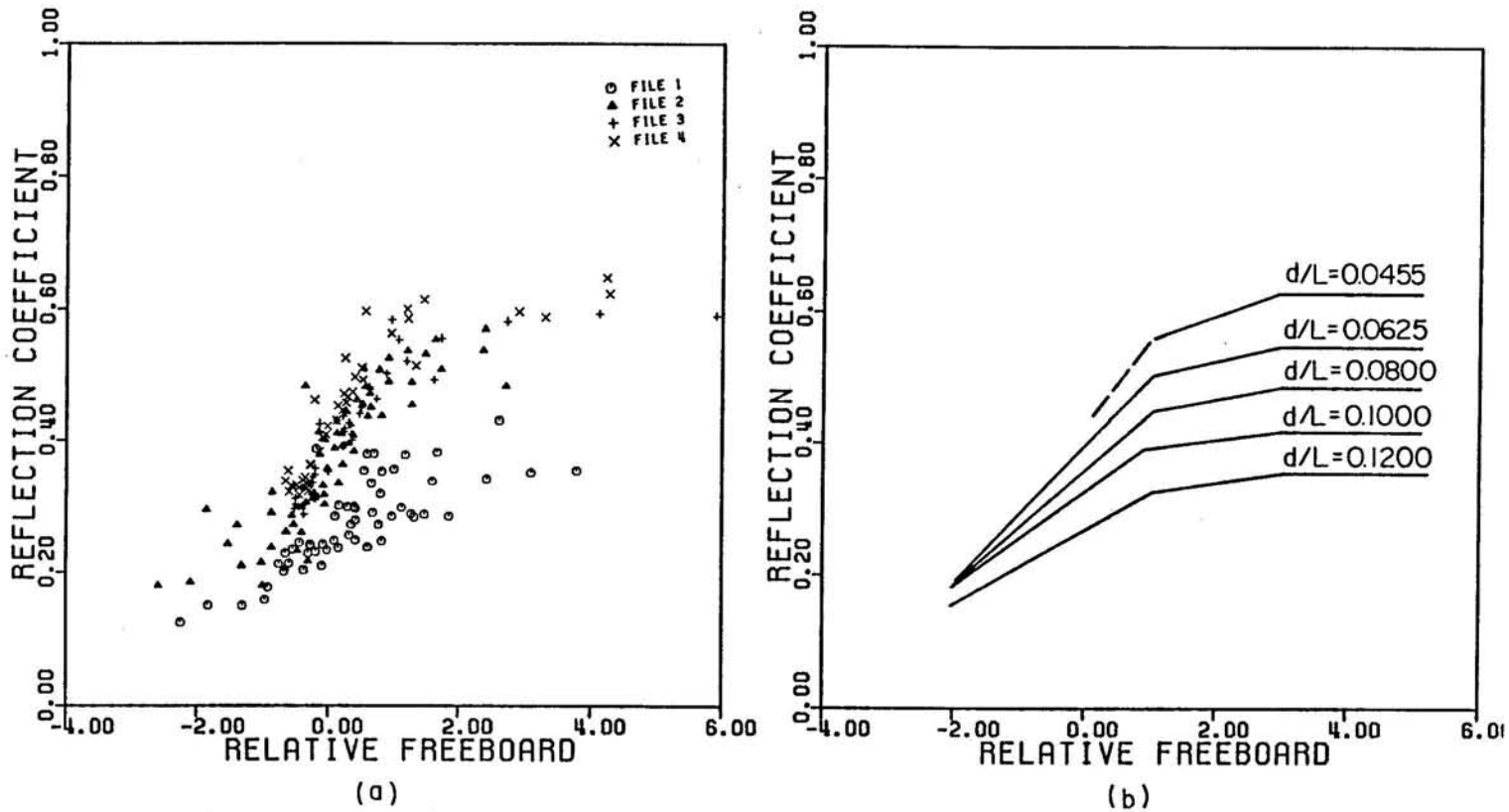


Figure 3.14 Design curves for the prediction of reflection coefficient as a function of relative freeboard and relative depth.

short waves in File 1. As R increases, K_r approaches a constant whose value is a function of T_p only. K_r is predicted by fitting three lines within the ranges of $R \leq 1.0$, $1.0 < R < 3.0$, and $R > 3.0$ (Figure 3.14(b). The lines were matched at $R = 1.0$ and $R = 3.0$ such that the overall goodness of fit was as high as possible. The effects of T_p are included in the linear coefficients which are a function of d/L. A detailed discussion of the predicton of K_r is given in Chapter 4.

3.2.4 Changes in Wave Period and Spectral Peakedness

3.2.4(a) Change in T_p

As discussed earlier, T_p is the wave period in the spectrum that contains the most energy. It is expected that energy will be lost at the peak and redistributed to higher and lower frequencies, but that, in the absence of breaking, the frequency at which the peak is located will not change. This is generally the case for these data. The ratio of the transmitted to incident T_p as a function of R is shown in Figure 3.15.

3.2.4(b) Change in T_s and $\overline{T}$

T_s is the average wave period of the one-third highest waves. $\overline{T}$ is the average of all waves. Unlike T_p, T_s and $\overline{T}$ are expected to change because they are directly related to the wave heights. As higher frequencies are introduced or filtered out, the change should be reflected in T_s and $\overline{T}$. The ratios of transmitted to incident T_s and $\overline{T}$ vs. R are shown in Figures 3.16 and 3.18, respectively. The data in Figure 3.16(a-d) are plotted together in Figure 3.17. There is a distinct pattern to these data that corresponds to the shift in transmission modes. For R less than about 1.0, the ratio is less than one. This means that higher harmonics are being introduced as waves pass over the structure. For $R > 1.0$, the

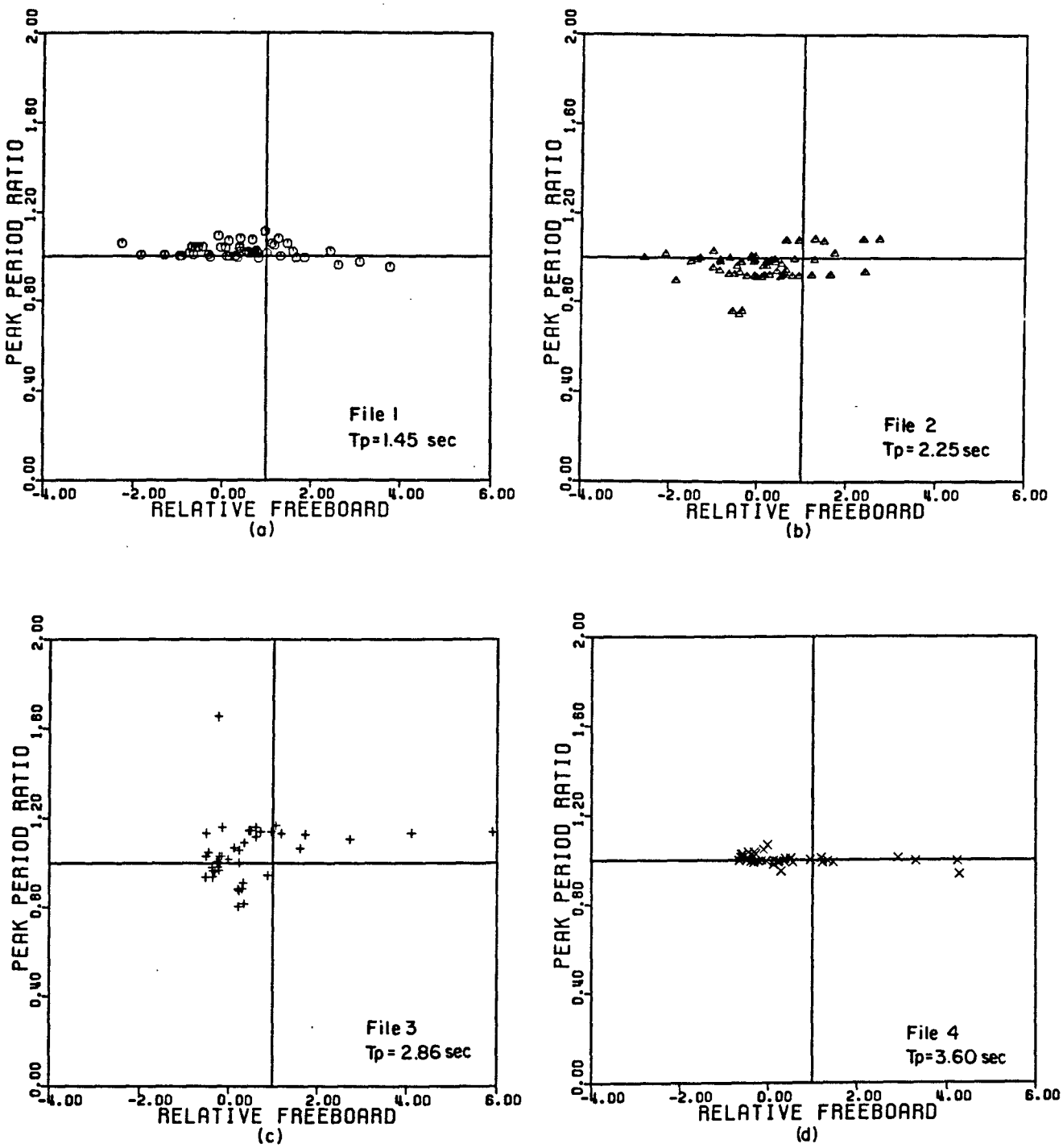


Figure 3.15 Ratio of incident to transmitted peak period as a function of relative freeboard.

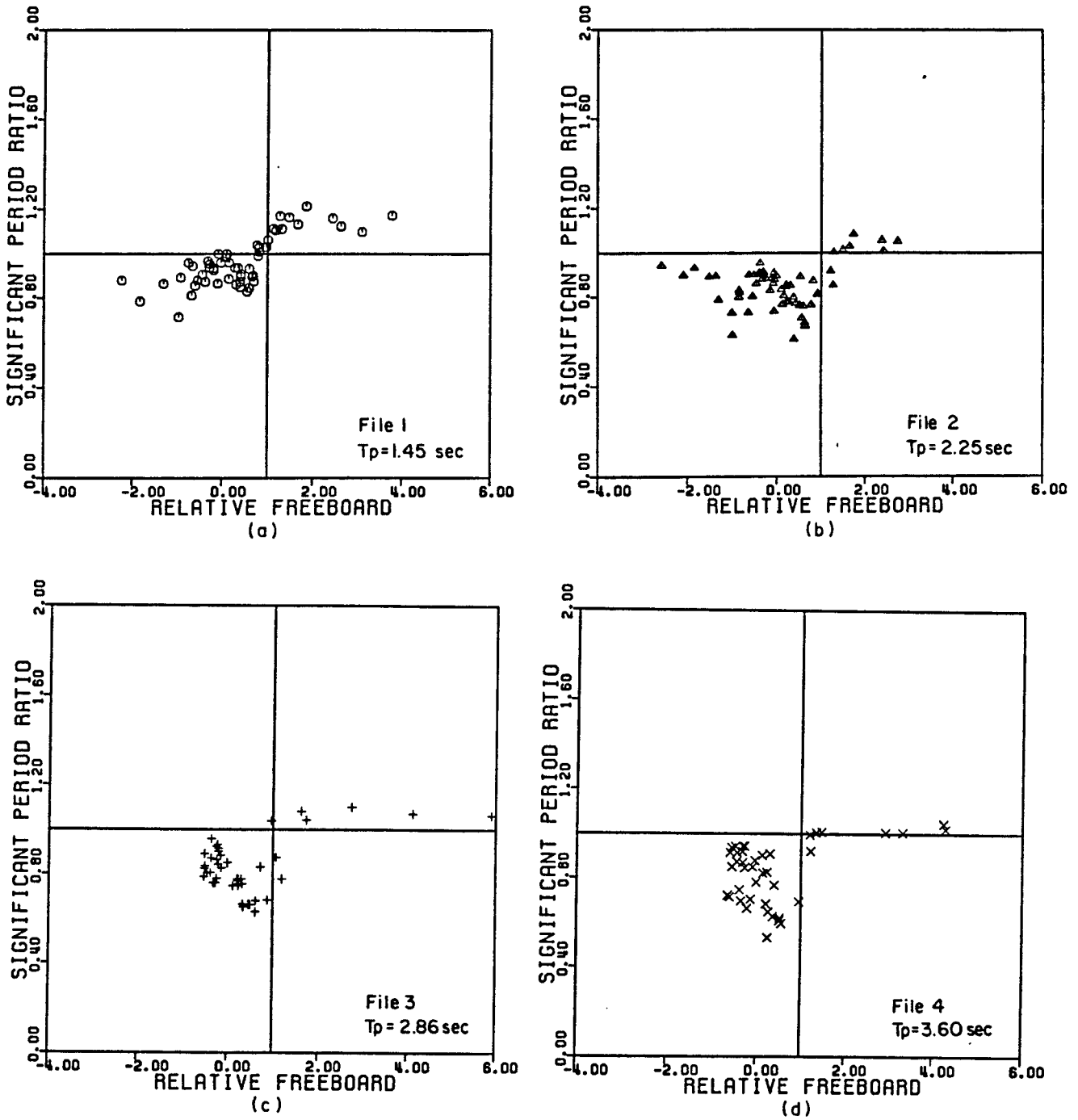


Figure 3.16 Ratio of transmitted to incident significant wave period as a function of relative freeboard.

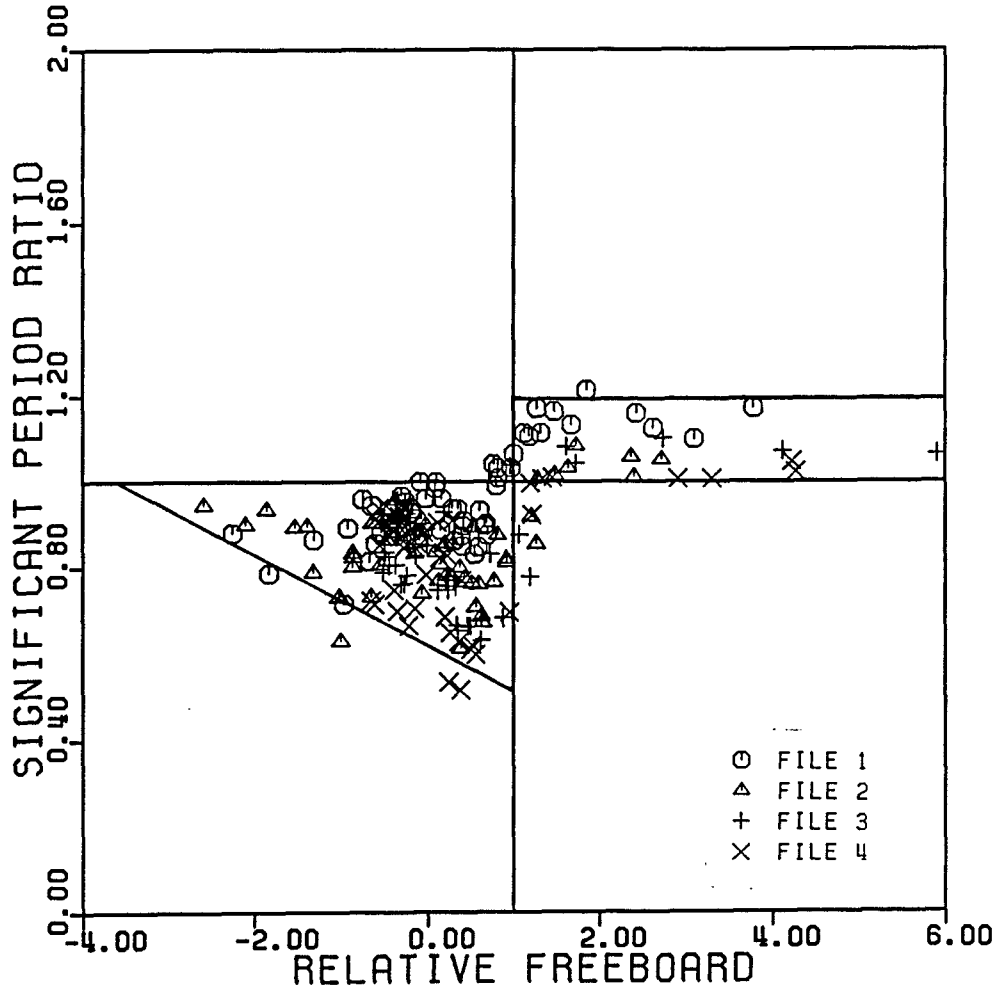


Figure 3.17 Ratio of transmitted to incident significant wave period as a function of relative freeboard. The limits used in the design program (Chapter 4) to determine the upper and lower bounds on the ratio are shown.

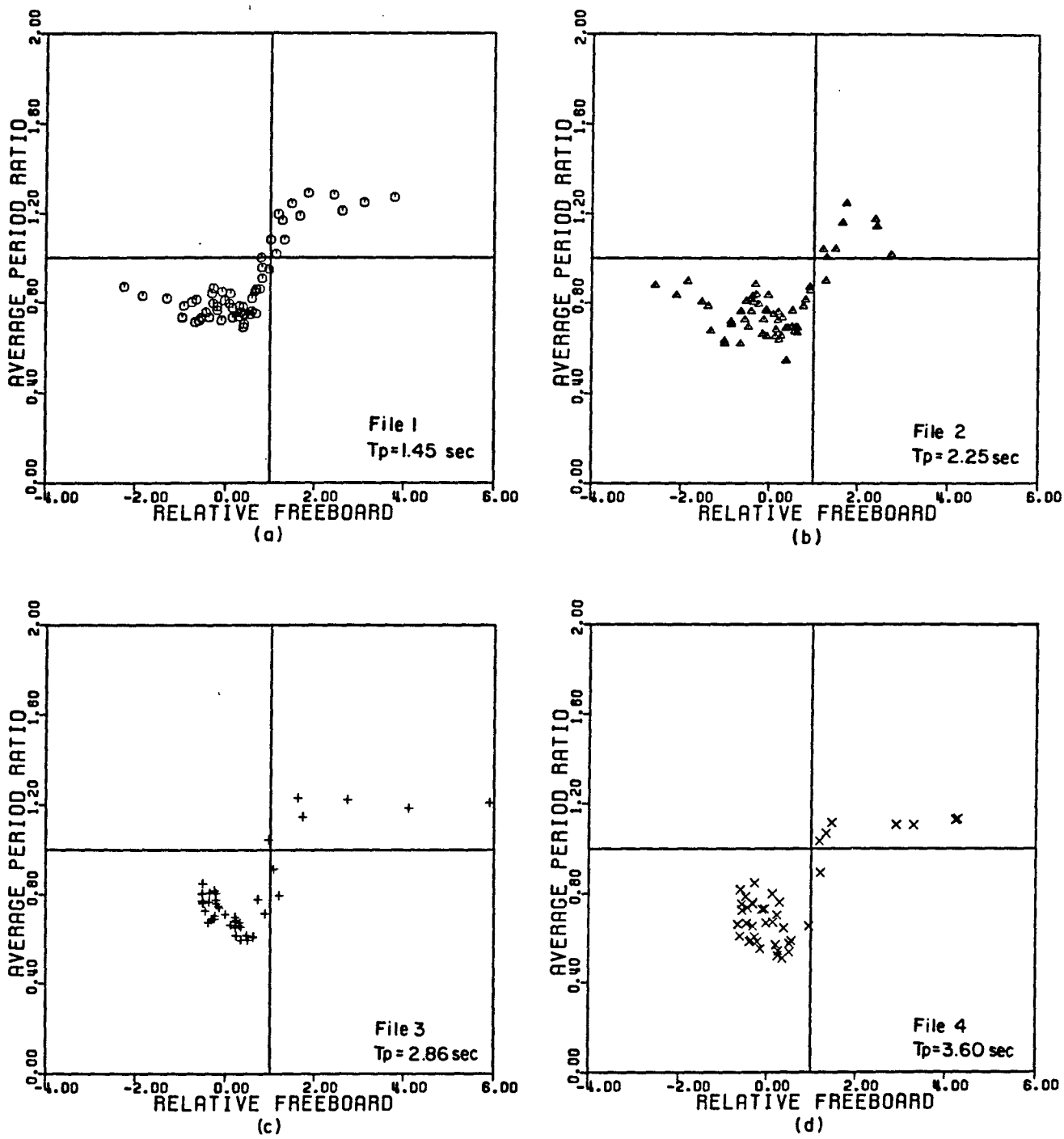


Figure 3.18 Ratio of transmitted to incident average wave period as a function of relative freeboard.

ratio is greater than one, indicating that higher frequencies are being filtered out. This is characteristic of waves passing through a structure. The magnitude of the change is in part dependent on T_p. Overall, longer waves experience a greater change in T_s and $\overline{T}$.

3.2.4(c) Change in Q_p

The Q_p ratios are plotted in Figure 3.19 and 3.20. Changes in the Q_p ratio are attributed to the same factors as the changes in T_s and $\overline{T}$.

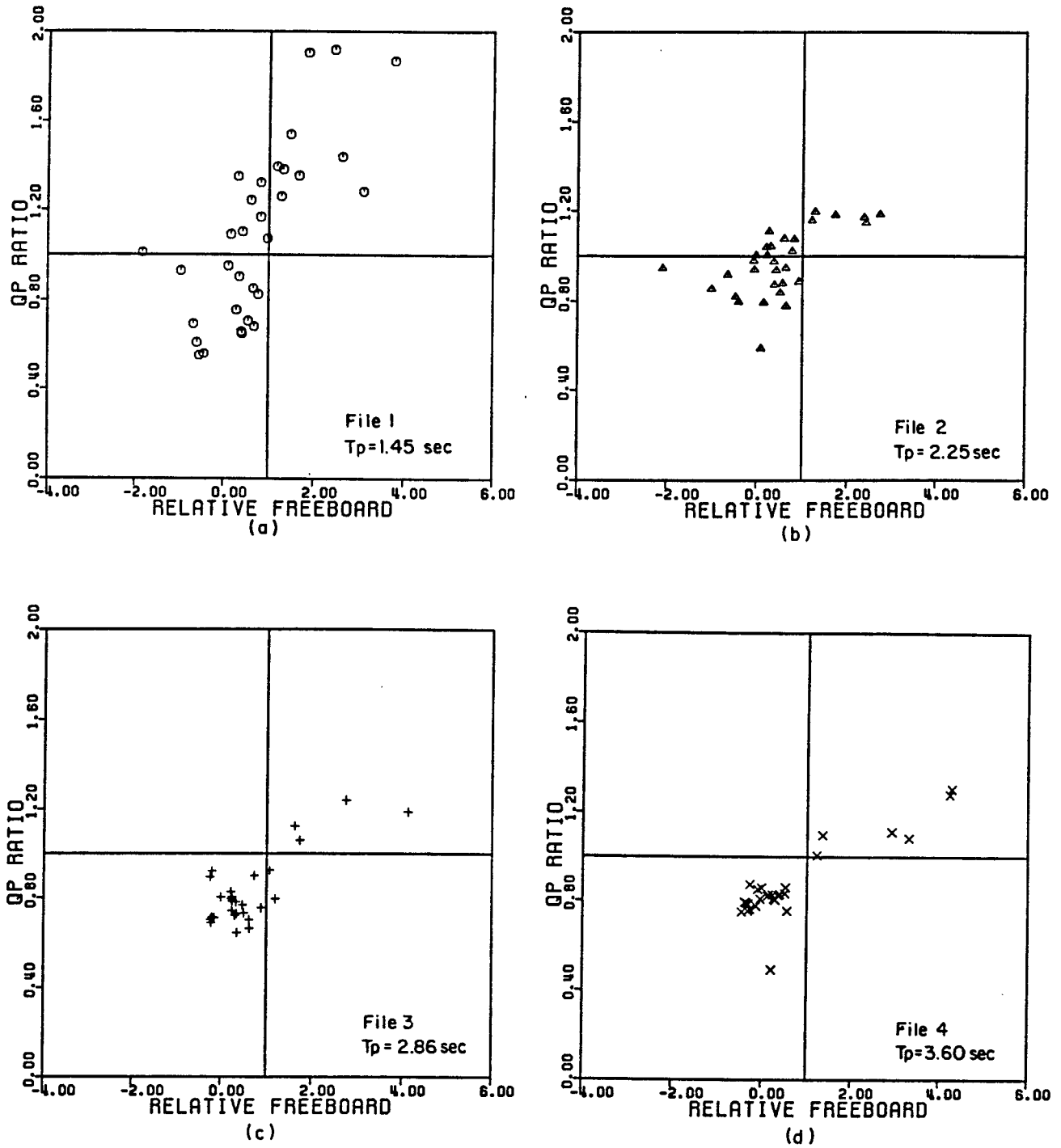


Figure 3.19 Ratio of transmitted to incident spectral peakedness parameter as a function of relative freeboard for each of the four wave files.

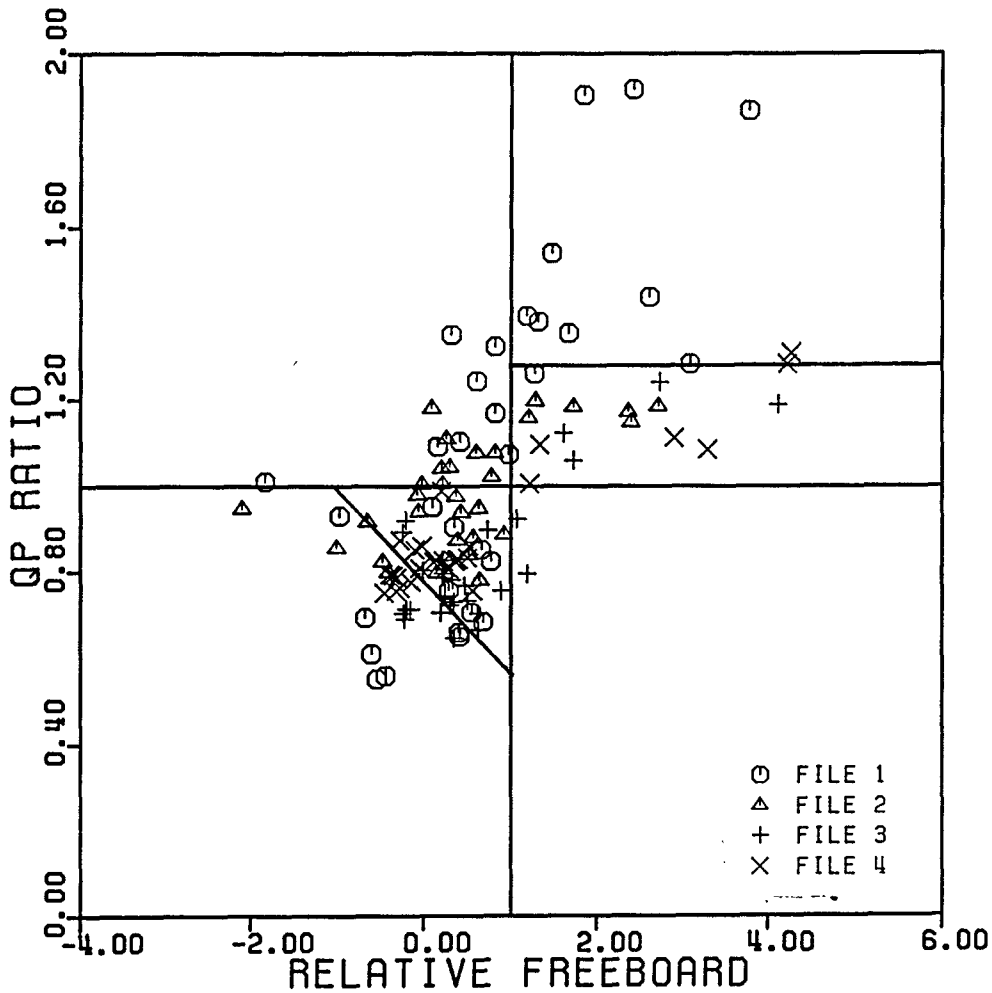


Figure 3.20 Ratio of transmitted to incident spectral peakedness parameter as a function of relative freeboard. The limits used in design program (Chapter 4) to determine the upper and lower bounds on the ratio as shown.

CHAPTER 4 DESIGN AID PROGRAM

This chapter describes in detail the computer program, LCBDGN, which is to be used as an aid in designing low-crested breakwaters. Version 1.0 of this program is based on laboratory data only. Future versions will incorporate field data as well.

The assumption is made that the designer knows the incident wave conditions for which the breakwater is to be subjected and the desired transmitted or reflected significant wave height for specified incident wave conditions. Two sets of incident wave conditions must be specified (referred to here as operational and extreme) as well as which of these conditions are to be used as a basis for design. The program computes the structure height needed in order product the desired results and the height to which the structure must be constructed in order to achieve that final height. The constructed height may or may not be the same as the final height depending on the specified design conditions and structure parameters.

Any two sets of incident wave conditions for which the designer would like stability and performance information is acceptable as long as they are within the range of the present data. As more and better data are available the program can be upgraded and extended to include a wider range of conditions. If sufficient statistical information is known about the incident wave climate, the "operational" sea state may be taken as the conditions (significant wave height, peak period, peakedness parameter) that are not exceeded a high percentage (say 95%) of the time. "Extreme" conditions refer to what is often called "design conditions" and are the

most severe conditions anticipated during the life of the structure. LCBDGN computes the performance (i.e., transmitted significant wave height, peak period, significant period range and peakedness parameter range and reflected significant wave height) of the breakwater for both sets of conditions before and after it has been subjected to the extreme sea state.

Least squares curve fits to laboratory data have been made regarding the stability and performance of low-crested breakwaters. A description of how these curves are used to compute structure heights and damage and transmitted and reflected wave parameters is presented below.

Final Structure Height

The structure height required to produce the desired transmitted or reflected significant wave height for a specified set of initial conditions is computed using lease squares curve fit equations of the data shown in Figures 3.10, 3.3(b), and 3.14(b). Figures 3.10 and 3.3(b) are used when transmitted wave conditions are specified and Figure 3.14(b) when specific reflected wave conditions are desired.

First consider the case where transmitted waves are specified (refer to Figure 3.10)

For $R < 0.0$, the transmission coefficient is given by

$$K_t = A11 + A21\ e^{R} \tag{4-1}$$

A11 is fixed at 0.9 and A21 is determined using least squares curve fit techniques

$$A11 = 0.9 \tag{4-2}$$

Thus

$$A21 = \left(\frac{\sum_{i=1}^{n} e^{R_i} K_{t_i} - A11 \sum_{i=1}^{n} e^{R_i}}{\sum_{i=1}^{n} e^{2R_i}}\right) = -0.358 \qquad (4\text{-}3)$$

$$K_t = 0.9 - 0.358\, e^{R} \qquad (4\text{-}4)$$

Solving for h_f in Equation 4-4 we get

$$R \equiv \frac{h_f - d}{H_{s_i}} = \frac{0.9 - K_t}{0.358} \text{ and} \qquad (4\text{-}5)$$

$$h_f = d + H_{s_i} \ln\left(\frac{0.9 - K_t}{0.358}\right) . \qquad (4\text{-}6)$$

For $0.0 < R < 1.0$, the transmission coefficient is given by

$$K_t = \frac{1.0}{A12 + A22\, R} \quad . \qquad (4\text{-}7)$$

In order to make the K_t vs. R curve continuous at R = 0.0, A12 is expressed as

$$A12 = \frac{1.0}{A11 + A21} = 1.845 \quad . \qquad (4\text{-}8)$$

A22 is chosen so that the K_t vs. R curves will be continuous at R = 1.0,

$$A22 = (1 - A12) + P^a \qquad (4\text{-}9)$$

where, as defined in Chapter 3,

$$P \equiv \frac{H_s A_t}{L_p (D_{50})^2} \qquad (4\text{-}10)$$

and

$$A_t = h_i \left(\frac{h_i}{\tan\theta} + b\right) \tag{4-11}$$

Substituting these expressions into the K_t equation results in

$$K_t = \frac{1.0}{A12 + \left\{1 - A12 + \left(\frac{H_s \; h_i}{L_p \; (D_{50})^2} \left(\frac{h_i}{\tan\theta} + b\right)\right)^a\right\} R} \; . \tag{4-12}$$

h_i can be expressed in terms of h_f by using the curves from Figure 3.3(b).

For $N_s^* > 6.0$

$$\frac{h_f}{d} = A13 - A23 \; N_s^* \tag{4-13}$$

where

$$A13 = A113 + A213 \left(\frac{h_i}{d}\right) \tag{4-14}$$

and

$$A23 = A123 + A223 \left(\frac{h_i}{d}\right) \tag{4-15}$$

or

$$\frac{h_f}{d} = \left[A113 + A213\left(\frac{h_i}{d}\right)\right] + \left[A123 + A223\left(\frac{h_i}{d}\right)\right] N_s^* \tag{4-16}$$

Solving for h_i we have

$$h_i = \frac{d[(\frac{h_f}{d}) - A113 - A123\ N^*_s]}{[A213 + A223\ N^*_s]} \tag{4-17}$$

Substituting this expression into the K_t equation yields

$$\{k_6 + [k_5(\frac{h_f - k_1}{k_2\ k_3})(\frac{h_f - k_1}{k_2\ k_3} + k_4)]^a\}(\frac{h_f - d}{H_s} + k_7) = 0 \tag{4-18}$$

where

$$k_1 \equiv A113 + A123\ N^*_s\ ,$$

$$k_2 \equiv A213 + A223\ N^*_s\ ,$$

$$k_3 \equiv \tan\theta\ ,$$

$$k_4 \equiv b\ ,$$

$$k_5 \equiv H_i\ /L_p(D_{50})^2\ ,$$

$$k_6 \equiv 1.0 - A12\ , \tag{4-19}$$

$$k_7 \equiv A12 - \frac{1}{K_t}\ ,$$

$$A113 = -0.2338\ ,$$

$$A213 = 1.436\ ,$$

$$A123 = 0.03737 \quad \text{and}$$

$$A223 = -0.06997\ .$$

This transcendental equation can be solved using a Newton-Raphson scheme, i.e.,

$$h_{f(j+1)} = h_{f(j)} - \frac{f(h_{f(j)})}{f'(h_{f(j)})} \tag{4-20}$$

where

$$f(h_f) \equiv$$

$$\{k_6 + [k_5(\frac{h_f - k_1}{k_2\ k_3})(\frac{h_f - k_1}{k_2\ k_3})]^a\}(\frac{h_f - d}{H_s} + k_7) = 0 \tag{4-21}$$

and

$$f'(h_f) \equiv \frac{d\ f(h_f)}{d\ h_f} =$$

$$\{k_6 + [k_5(\frac{h_f - k_1}{k_2\ k_3})(\frac{h_f - k_1}{k_2\ k_3} + k_4)]^a\}(\frac{1}{H_s})$$

$$+ a[k_5(\frac{h_f - k_1}{k_2\ k_3})(\frac{h_f - k_1}{k_2\ k_3} + k_4)]^{a-1}$$

$$[k_5(\frac{h_f - k_1}{k_2\ k_3})(\frac{1.0}{k_2\ k_3}) + (\frac{k_5}{k_2\ k_3})(\frac{h_f - k}{k_2\ k_3} + k_4)](\frac{h_f - d}{H_s}) \tag{4-22}$$

For R > 1.0

$$K_t = \frac{1.0}{1.0 + P^a} \tag{4-23}$$

where

$$P = \frac{H_s}{L_p(D_{50})^2} \left[h_i\left(\frac{h_i}{\tan\theta} + b\right)\right] \tag{4-24}$$

Substituting this expression for P into the above K_t equation and solving for h_i results in

$$h_i = \frac{-b \tan\theta \pm \sqrt{(b \tan\theta)^2 + \dfrac{L_p(D_{50})^2 K^* \tan\theta}{H_s}}}{2} \tag{4-25}$$

where

$$K^* \equiv \left(\frac{1}{K_t} - 1\right)^{\frac{1}{a}}$$

Next, equate h_i in this expression to h_i in Equation 4.17.

$$d\left\{\frac{\left[\dfrac{h_f}{d} - A113 - (A123)\, N_s^*\right]}{(A213 + (A223)\, N_s^*)}\right\} =$$

$$\frac{-b \tan\theta}{2} + \frac{1}{2}\sqrt{(b \tan\theta)^2 + \frac{4L_p(D_{50})^2 K^* \tan\theta}{H_s}} \tag{4-27}$$

Solving for h_f we get

$$h_f = A113 - (A123)\ N_s^*$$

$$+ [A213 + (A223)\ N_s^*][\frac{-b \tan \theta}{2d} + \frac{1}{2d}\ (b \tan \theta)^2 + \frac{4Lp(D_{50})^2\ K^* \tan \theta}{H_i} \tag{4-28}$$

$$a = 0.5926 \tag{4-29}$$

For the case where reflected significant wave height is specified, Figure 3.14(b) must be used.

For $R < 1.0$

$$K_R = A14 + (A24)\ R \tag{4-30}$$

where

$$A14 = A114 + A214\ (\frac{d}{L_p})$$

$$A24 = \frac{1.0}{A124 + A224\ (\frac{d}{L_p})}, \tag{4-31}$$

$$R = \frac{K_r - A14}{A24} \quad \text{and} \tag{4-32}$$

$$h_f = d + H_s\ \frac{(K_r - A14)}{A24} \tag{4-33}$$

or

$$h_f = d + \frac{H_s\ \{K_r - [A114 + A214(\frac{d}{L_p})]\}}{[A124 + A224\ (\frac{d}{L_p})]} \tag{4-34}$$

where

$$A114 = 0.5085 ,$$

$$A214 = -2.018 , \qquad (4\text{-}35)$$

$$A124 = 1.019 \text{ and}$$

$$A224 = 137.6 .$$

For $1.0 < R < 3.0$

$$K_r = A15 + (A25)\, R \qquad (4\text{-}36)$$

$$A15 = A115 + A215\left(\frac{d}{L_p}\right) \qquad (4\text{-}37)$$

$$A25 = \frac{1.0}{A125 + A225\left(\frac{d}{L_p}\right)} \qquad (4\text{-}38)$$

$$h_f = d + \frac{H_s \left\{K_r - \left[A114 + A214\left(\frac{d}{L_p}\right)\right]\right\}}{\dfrac{1.0}{\left[A124 + A224\left(\frac{d}{L_p}\right)\right]}} \qquad (4\text{-}39)$$

where

$$A115 = 0.7195 ,$$

$$A215 = -3.400 , \qquad (4\text{-}40)$$

$$A125 = 48.70 \text{ and}$$

$$A225 = 268.1 .$$

For $R > 3.0$

$$h_f(R = 3.0)$$

$$= A15 + A25(3.0) \qquad (4\text{-}41)$$

Initial Structure Height

Once the final structure crest height has been determined then the initial or constructed crest height can be obtained from Figure 3.3(b) (Eq. 4-17).

$$h_i = d\left\{\frac{\left(\frac{h_f}{d}\right) - A113 - (A123)N_s^*}{(A213 + (A223)N_s^*}\right\}$$

Knowing the initial structure height, the "area of damage" can be found from the least squares curve fit to the data in Figure 3.2.

$$A_d = (D_{50})^2[A16 + A26M + A36M^2 + A46M^3] \qquad (4\text{-}42)$$

where

$$M \equiv N_s^*\left(\frac{h_i}{d}\right)^{1.5} \qquad (4\text{-}43)$$

and

$$\begin{aligned} A16 &= 19.45\ , \\ A26 &= -7.455\ , \\ A36 &= 0.7605 \quad \text{and} \\ A46 &= -0.01048\ . \end{aligned} \qquad (4\text{-}44)$$

With the initial and final structure heights established, the breakwater performance can be computed for the circumstances and conditions of interest. As stated earlier in this report, the performance parameters for this version of the program include:

1. Transmitted significant wave height
2. Transmitted significant wave period range
3. Transmitted peakedness parameter range
4. Reflected significant wave height

These parameters are computed for the following circumstances:

1. Operational Incident Waves
   a. Previous wave conditions not exceeding operational sea state.
   b. Previous wave conditions reaching but not exceeding extreme sea state.
2. Extreme Incident Waves
   a. Previous wave conditions reaching but not exceeding extreme sea state.

Transmitted Significant Wave Period

The data for transmitted significant wave period is not sufficient to allow prediction of specific values. Bounds on the range values for given set of conditions can, however, be obtained from the data as shown in Figure 55. The equations for these bounds are as follows:

$$\frac{T_{s_t}}{T_{s_i}} = \begin{cases} 1.0 & R < -3.5 \\ T11 + (T21)R & -3.5 \leq R \leq 1.0 \\ T13 + (T23)R & 1.0 < R \end{cases} \qquad (4\text{-}45)$$

where

$T11 = 0.6320$ and

$T21 = -0.106$. (4-46)

Transmitted Peakedness Parameter

As with transmitted significant wave height, there is not enough data to predict specific values of the peakedness parameter. Bounds on the range of values were established by eye and equations fit to these bounds. These equations are given below:

$$\frac{Q_{p_t}}{Q_{p_i}} = \begin{cases} 1.0 & R < -3.5 \\ Q11 + (Q21)R & -3.5 \leq R \leq 1.0 \\ 1.28 & 1.0 < R \end{cases} \tag{4-47}$$

where

$$Q11 = 0.67 \quad \text{and}$$

$$Q21 = -0.095 \; . \tag{4-48}$$

CHAPTER 5 SUMMARY AND RECOMMENDATIONS

Data from experiments on low-crested breakwaters conducted at the Coastal Research Engineering Center were analyzed with respect to structural damage and modifications of the wave field by the breakwater. The results were incorporated into an interactive breakwater design program use with a personal computer. In addition, observations from these analyses were used to develop recommendations for future studies.

Structural damage was assessed in terms of both crest height changes and volumetric changes. The volumetric change, expressed as dimensionless damage, was found to be related to a modified spectral stability number defined by

$$M = N_s^* \left(\frac{h_i}{d}\right)^{1.5}$$

For values of $6 < M < 29$ this relationship is given by a third order polynomial (Equation 3-2). For $M < 6$, the damage is effectively zero. Crest height changes are determined using a linear relationship between $\frac{h_f}{d}$ and N_s^* for a specified value of $\frac{h_i}{d}$. Three design curves are based on data from subsets 1, 3, and 5 having values of $\frac{h_i}{d}$ of about 1.0, 1.2, and 1.4, respectively, see Figure 3.3.

The modification of the wave field due to the breakwater is a complex function of many variables including wave height, wave period, stone characteristics, freeboard, and cross-sectional area of the structure. The relative importance of these paramterrs in the determination of Kt and Kr depends upon the relative freeboard, R, which also serves to define the transition from transmission by overtopping to transmission by flow through the structure. Prediction of Kt is divided into three zones. For

$R > 1.0$, transmission is predicted using the parameter, P, which is a function of wave steepness, cross-sectional area of the structure, and stone diameter, but is independent of R. The region $0 < R < 1$ is a transition zone in which Kt is a function of both P and R. At negative R, Kt is primarily a function of R. The influences of stone and structure parameters are less obvious in the reflection data. Below $R = 3.0$, Kr is a function of R and incident wave period expressed as the relative depth, d/L. At higher values of R, Kr depends of d/L only.

The shift in dominant mode of transmission also effects the ratio between the transmitted and incident wave periods and spectral peakedness parameters. When transmission is by overtopping ($R < 1.0$), higher harmonics are introduced into the transmitted wave and the wave period and spectral peakedness ratios are less than one. For higher relative freeboards, high frequency (short period) waves are filtered out and the ratio is greater than one.

Based on observations from these data and analyses, the following recommendations for future studies are made:

1) Apply Goda's method for the resolution of reflected and incident waves behind the structure as well in front of the structure;
2) Examine the increase in K_t as the damage tests progress in order to determine the relationship between the average K_t and the maximum K_t that can be expected during an extreme event;
3) Conduct experiments specifically designed to determine the relationship between transmission at low relative freeeboards and stone characteristics, structure size, and wave period; and
4) Conduct three-dimensional tests so that a) (structure) end effects can be examined and (b) non-orthogonal incident waves can be tested.

A computer program has been written as part of the work reported here. The program name is Low Crested Breakwater Design, LCBDGN, and its purpose is to aid in the design of low crested breakwaters. The initial version (Version 1.00) has certain limitations which need to be pointed out at this point.

1. The data on which the design equations used in the program are based are two dimensional wave tank data. There has been no attempt to account for scale effects in going from the laboratory to the prototype (field).
2. No attempt has been made to account for 3-dimensional effects, i.e., refraction and diffraction at the end(s) of the structure.
3. Incident reflected and transmitted waves are assumed to approach and leave the structure at right angles to the structure (i.e., wave crests are assumed to be parallel to the structure).

In spite of the above qualifications, LCBDGN should be helpful to the engineer designing a low-crested permeable rubble mound breakwater. The intent is to incorporate scale and other effects in later versions of the program as field data (and its analysis) becomes available. In the meantime, the program can be used as long as the results are used along with "engineering judgment" and additional information regarding the factors discussed above.

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Appendix A

DOCUMENTATION FOR LCBDGN.FOR

"LCBDGN.FOR" is an interactive FORTRAN program the purpose of which is to aid in the design of low crested breakwaters. Input data can be entered 1) from the keyboard, 2) from a disk file or 3) by a user modified disk file (i.e. any number of the 18 input quantities can be changed from within LCBDGN. An input data file is created each time the program is run under the name LCBIN.DAT.

In order to make changes in the input data file you must know the "sequence" number of the quantities you wish to change. These numbers are given below:

SEQUENCE NUMBER	QUANTITY	DESCRIPTION
1	HSO	Incident Operational Significant Wave Height (m)
2	TSO	Incident Operational Significant Wave Period (sec)
3	TPO	Incident Operational Peak Period (sec)
4	QPO	Incident Operational Peakedness Parameter
5	HSE	Incident Extreme Significant Wave Height (m)
6	TSE	Incident Extreme Significant Wave Period (sec)
7	TPE	Incident Extreme Peak Period (sec)
8	QPE	Incident Extreme Peakedness Parameter
9	BS	Bottom Slope at Structure (del z)/(del x)
10	WD	Water Depth at Structure (m)
11	WIDTH	Width of Structure Crest (Constructed) (m)
12	SS	Structure Slope (forward and back) (del z)/del x)
13	WW	Mass Density of Water (kg/m**3)
14	WR	Mass Density of Stone (kg/m**3)
15	W50	Mass of Mean Stone (kg)
16	IDGN1	Denotes Design Wave Condition 1 - Operational Wave Conditions 2 - Extreme Wave Conditions
17	IDGN2	Denotes Design for Transmitted or Reflected Wave 1 - Transmitted 2 - Reflected
18	HSD	Desired (Transmitted or Reflected) Significant Wave Height (m)

```
C*********************************************************************C
C---------------------------------------------------------------------C
C        PROGRAM LCBDGN  (VERSION 1.00) 15 NOVEMBER 1986              C
C        LOW CRESTED BREAKWATER DESIGN                                C
C                                                                     C
C        D.M. SHEPPARD ((904) 392-1570)                               C
C                                                                     C
C        PURPOSE:                                                     C
C        The purpose of this interactive FORTRAN program is to aid in C
C        the design of Low Crested Breakwaters. The structure of the  C
C        program is such that new data and information regarding the  C
C        performance and stability of Low Crested Breakwaters may be  C
C        added with a minimum of effort.                              C
C                                                                     C
C        PROGRAM DESCRIPTION                                          C
C        The following assumptions are made:                          C
C        1. OPERATIONAL incident wave conditions are known            C
C        2. EXTREME incident wave conditions are known                C
C        3. The design is based on a desired transmitted or           C
C           reflected significant wave height for either              C
C           OPERATIONAL or EXTREME conditions.                        C
C                                                                     C
C        PROGRAM OUTPUT                                               C
C        1. Constructed structure height                              C
C        2. Structure height after structure experiences OPERATIONAL  C
C           waves                                                     C
C        3. Structure height after structure experiences EXTREME      C
C           waves                                                     C
C        4. Transmitted and reflected wave parameters for OPERATIONAL C
C           and EXTREME conditions                                    C
C                                                                     C
C---------------------------------------------------------------------C
C
      DIMENSION DATA(18),ANDATA(18)
      COMMON HSO,TPO,QPO,HSE,TPE,QPE,WIDTH,SS,WW,WR,W50,
     1SSND,WD,WLO,WLE,D50,HCIOOT,HCIOOR,HCIEET,HCIEER
      INTEGER ITEM(18)
C
      OPEN(11,FILE='LCBOUT.DAT',STATUS='UNKNOWN')
C
   10 WRITE(*,20)
   20 FORMAT(3X,'TYPE THE NUMBER IN FRONT OF THE DESIRED OPTION',
     1/4X,'1) INPUT DATA FROM KEYBOARD ',
     2/4X,'2) INPUT DATA FROM DATAFILE LCBIN.DAT AS IS',
     2/4X,'3) MODIFY DATA IN DATAFILE LCBIN.DAT',/)
      READ(*,*)IOPT
      IF(IOPT.EQ.1)GO TO 105
      IF(IOPT.EQ.2)GO TO 30
      IF(IOPT.EQ.3)GO TO 40
      GO TO 10
C
   30 OPEN(9,FILE='LCBIN.DAT',STATUS='OLD')
      READ(9,*)HSO,TSO,TPO,QPO,HSE,TSE,TPE,QPE,BS,WD,WIDTH,
     1SS,WW,WR,W50,IDGN1,IDGN2,HSD
      GO TO 280
   40 OPEN(9,FILE='LCBIN.DAT',STATUS='OLD')
      WRITE(*,50)
   50 FORMAT(3X,'REFER TO THE "INPUT DATA LIST" IN THE DOCUMENTATION',
     1/1X,'FOR THIS PROGRAM !',//,3X,
     2'HOW MANY QUANTITIES WOULD YOU LIKE TO CHANGE ?',/)
      READ(*,*)INCNG
      DO 70 I=1,INCNG
```

```
      WRITE(*,60)I
   60 FORMAT(3X,'TYPE THE NUMBER IN FRONT OF ITEM',I2,1X,
     1'TO BE CHANGED',/1X,'FOLLOWED BY THE NEW VALUE OF THE ITEM (SEPARA
     2TED BY A COMMA)',/)
      READ(*,*)ITEM(I),ANDATA(I)
   70 CONTINUE
      READ(9,*)(DATA(J),J=1,18)
      CLOSE(9,STATUS='DELETE')
      DO 90 K=1,INCNG
      JK= ITEM(K)
      DATA(JK)= ANDATA(K)
   90 CONTINUE
      OPEN(9,FILE='LCBIN.DAT',STATUS='NEW')
      DO 100 L=1,18
      WRITE(9,*)DATA(L)
  100 CONTINUE
      CLOSE (9)
      GO TO 30
C
  105 WRITE(*,110)
  110 FORMAT(3X,'TYPE THE FOLLOWING "OPERATIONAL" INCIDENT WAVE',/1X,
     1'INFORMATION SEPARATED BY COMMAS',/4X,
     2'1. SIGNIFICANT WAVE HEIGHT (in meters)',/4X,
     3'2. SIGNIFICANT WAVE PERIOD (in seconds)',/4X,
     4'3. PEAK PERIOD (in seconds)',/4x,
     5'4. Qp SPECTRAL PEAKEDNESS (Qp = 2.0 for PIERSON-MOSKOWITZ)',
     6//)
      READ(*,*)HSO,TSO,TPO,QPO
      WRITE(*,120)
  120 FORMAT(3X,'TYPE THE FOLLOWING "EXTREME" INCIDENT WAVE',/1X,
     1'INFORMATION SEPARATED BY COMMAS',/4X,
     2'1. SIGNIFICANT WAVE HEIGHT (in meters)',/4X,
     3'2. SIGNIFICANT WAVE PERIOD (in seconds)',/4X,
     4'3. PEAK PERIOD (in seconds)',/4x,
     5'4. Qp SPECTRAL PEAKEDNESS (Qp = 2.0 for PIERSON-MOSKOWITZ)',
     6//)
      READ(*,*)HSE,TSE,TPE,QPE
      WRITE(*,130)
  130 FORMAT(3X,'TYPE BOTTOM SLOPE AT LOCATION OF STRUCTURE.',/1X,
     1'( (DELTA Z)/(DELTA X) )'/)
      READ(*,*)BS
      IF(BS.GT.0.07)GO TO 135
       WRITE(*,133)
  133   FORMAT(/3X,'CAUTION! DATA USED IN THIS VERSION OF THIS PROGRAM',
     1/1X,'IS BASED ON A BOTTOM SLOPE OF 1(VERT) ON 15(HOR) AT',/1X,
     2'THE STRUCTURE')
  135 WRITE(*,140)
  140 FORMAT(3X,'TYPE WATER DEPTH (in meters) AT THE STRUCTURE SITE',
     1/)
      READ(*,*)WD
      WRITE(*,150)
  150 FORMAT(3X,'TYPE WIDTH OF STRUCTURE CREST (in meters)'/)
      READ(*,*)WIDTH
      WRITE(*,160)
  160 FORMAT(3X,'TYPE STRUCTURE SLOPE (DELTA Z)/(DELTA X)',/)
      READ(*,*)SS
      IF(SS.LT.0.7.AND.SS.GT.0.6)GO TO 165
        WRITE(*,163)
  163   FORMAT(/3X,'CAUTION! DATA USED IN THIS VERSION OF THIS PROGRAM',
     1/1X,'IS BASED ON A STRUCTURE SLOPE OF 1(VERT) ON 1.5(HOR)')
```

```
  165 WRITE(*,170)
  170 FORMAT(/3X,'TYPE WATER DENSITY (in kg/m**3)',/)
      READ(*,*)WW
      WRITE(*,180)
  180 FORMAT(/3X,'TYPE STONE DENSITY (in kg/m**3)',/)
      READ(*,*)WR
      WRITE(*,190)
  190 FORMAT(/3X,'TYPE MEDIAN STONE WEIGHT, W50,(in kg)',/)
      READ(*,*)W50
  200 WRITE(*,210)
  210 FORMAT(3X,'TYPE THE NUMBER IN FRONT OF THE CONDITIONS',
     1/3X,'TO BE USED FOR DESIGN ',//4X,
     2'1. OPERATIONAL CONDITIONS',/4X,
     3'2. EXTREME CONDITIONS',/)
      READ(*,*)IDGN1
      IF(IDGN1.EQ.1.OR.IDGN1.EQ.2.)GO TO 220
      GO TO 200
  220 WRITE(*,230)
  230 FORMAT(3X,'TYPE THE NUMBER IN FRONT OF THE CONDITIONS',
     1/3X,'TO BE USED FOR DESIGN ',//4X,
     2'1. TRANSMITTED SIGNIFICANT WAVE HEIGHT, HST,',/4X,
     3'2. REFLECTED SIGNIFICANT WAVE HEIGHT, HSR,',/)
      READ(*,*)IDGN2
      IF(IDGN2.EQ.1.OR.IDGN2.EQ.2.)GO TO 240
      GO TO 220
  240 GO TO(250,260),IDGN2
  250 WRITE(*,255)
  255 FORMAT(/3X,'TYPE DESIRED VALUE OF TRANSMITTED',/3X,
     1'SIGNIFICANT WAVE  HEIGHT (in meters)',/)
      READ(*,*)HSD
      GO TO 270
  260 WRITE(*,265)
  265 FORMAT(/3X,'TYPE DESIRED VALUE OF REFLECTED',/3X,
     1'SIGNIFICANT WAVE  HEIGHT (in meters)',/)
      READ(*,*)HSD
  270 CONTINUE
C
      OPEN(9,FILE='LCBIN.DAT',STATUS='UNKNOWN')
C
C        WRITE INPUT DATA TO FILE
C
      WRITE(9,*)HSO,TSO,TPO,QPO,HSE,TSE,TPE,QPE,BS,WD,WIDTH,
     1SS,WW,WR,W50,IDGN1,IDGN2,HSD
C        END INPUT
C-----------------------------------------------------------------------C
C                          START COMPUTATION
C-----------------------------------------------------------------------C
C
C        COMPUTE DESIRED TRANSMISSION OR REFLECTION COEFFICIENT
C
  280 IF(IDGN1.EQ.1)THEN
       AK= HSD/HSO
      ELSE
       AK= HSD/HSE
C      IF(AK.GE.0.75)THEN
C       WRITE(*,281)AK
C 281   FORMAT(1X,'THE VALUE OF Kt (Kt = ',F8.3,') IS TOO LARGE FOR',
C    1  /1X,'THE RANGE OF VALIDITY OF THIS PROGRAM')
C       GO TO 500
C      ENDIF
```

```
      ENDIF
C
C        COMPUTE OPERATIONAL AND EXTREME WAVE LENGTHS, WLO AND WLE
C
      CALL WAVLEN(WD,TPO,WLO)
      CALL WAVLEN(WD,TPE,WLE)
      D50= (W50/WR)**.333333
      SSND= D50*((WR/WW)-1.)
      SSNO= (((HSO**2)*WLO)**.3333333)/SSND
      SSNE= (((HSE**2)*WLE)**.3333333)/SSND
C
C        COMPUTE STRUCTURE HEIGHT
C
C-----------------------------------------------------------------------C
C        IDGN1 = 1  FOR OPERATIONAL DESIGN
C              = 2  FOR EXTREME DESIGN
C        IDGN2 = 1  DESIGN BASED ON TRANSMITTED HS
C              = 2  DESIGN BASED ON REFLECTED HS
C
C        ICOND = 1  FOR OPERATIONAL CONDITIONS
C              = 2  FOR EXTREME CONDITIONS
C
C        IPREV = 1  FOR OPERATIONAL CONDITIONS NOT YET EXCEEDED
C              = 2  FOR EXTREME CONDITIONS REACHED
C-----------------------------------------------------------------------C
C
C        COMPUTE STRUCTURE INITIAL AND FINAL HEIGHTS AND
C                AREA OF DAMAGE FOR FOLLOWING SITUATIONS:
C
C                          TRANSMITTED
C
      IFLAG= 0
      IF(IDGN2.EQ.2)GO TO 310
C
      IF(IDGN1.EQ.2)GO TO 290
C
C        1. DESIGN - OPERATIONAL CONDITIONS
C                  - TRANSMITTED HS
C           CONDITIONS - OPERATIONAL
C
      IDGN1= 1
      IDGN2= 1
      ICOND= 1
      CALL STRSTA(IDGN1,IDGN2,ICOND,AK,HSO,WLO,HI,HF,AD,RI,RF,IFLAG)
      IF(IFLAG.GT.0)GO TO 500
      HCIOOT= HI
      HCFOOT= HF
      DAMOOT= AD
      RIOOT= RI
      RFOOT= RF
C
C        2. DESIGN - OPERATIONAL CONDITIONS
C                  - TRANSMITTED HS
C           CONDITIONS - EXTREME
C
      IDGN1= 1
      IDGN2= 1
      ICOND= 2
      CALL STRSTA(IDGN1,IDGN2,ICOND,AK,HSE,WLE,HI,HF,AD,RI,RF,IFLAG)
      IF(IFLAG.GT.0)GO TO 500
```

```
      HCIOET= HI
      HCFOET= HF
      DAMOET= AD
      RIOET= RI
      RFOET= RF
      GO TO 340
C
C        3. DESIGN - EXTREME CONDITIONS
C                  - TRANSMITTED HS
C           CONDITIONS - EXTREME
C
  290 IDGN1= 2
      IDGN2= 1
      ICOND= 2
      CALL STRSTA(IDGN1,IDGN2,ICOND,AK,HSE,WLE,HI,HF,AD,RI,RF,IFLAG)
      IF(IFLAG.GT.0)GO TO 500
      HCIEET= HI
      HCFEET= HF
      DAMEET= AD
      RIEET= RI
      RFEET= RF
C
C
C        4. DESIGN - EXTREME CONDITIONS
C                  - TRANSMITTED HS
C           CONDITIONS - OPERATIONAL
C
  300 IDGN1= 2
      IDGN2= 1
      ICOND= 1
      CALL STRSTA(IDGN1,IDGN2,ICOND,AK,HSO,WLO,HI,HF,AD,RI,RF,IFLAG)
      IF(IFLAG.GT.0)GO TO 500
      HCIEOT= HI
      HCFEOT= HF
      DAMEOT= AD
      RIEOT= RI
      RFEOT= RF
      GO TO 340
C
C
C                          REFLECTED
C
C        1. DESIGN - OPERATIONAL CONDITIONS
C                  - REFLECTED HS
C           CONDITIONS - OPERATIONAL
C
  310 IF(IDGN1.EQ.2)GO TO 320
      IDGN1= 1
      IDGN2= 2
      ICOND= 1
      CALL STRSTA(IDGN1,IDGN2,ICOND,AK,HSO,WLO,HI,HF,AD,RI,RF,IFLAG)
      IF(IFLAG.GT.0)GO TO 500
      HCIOOR= HI
      HCFOOR= HF
      DAMOOR= AD
      RIOOR= RI
      RFOOR= RF
C
C        2. DESIGN - OPERATIONAL CONDITIONS
C                  - REFLECTED HS
```

```
C           CONDITIONS - EXTREME
C
      IDGN1= 1
      IDGN2= 2
      ICOND= 2
      CALL STRSTA(IDGN1,IDGN2,ICOND,AK,HSE,WLE,HI,HF,AD,RI,RF,IFLAG)
      IF(IFLAG.GT.0)GO TO 500
      HCIOER= HI
      HCFOER= HF
      DAMOER= AD
      RIOER= RI
      RFOER= RF
      GO TO 340
C
C        3. DESIGN - EXTREME CONDITIONS
C                  - REFLECTED HS
C           CONDITIONS - EXTREME
C
  320 IDGN1= 2
      IDGN2= 2
      ICOND= 2
      CALL STRSTA(IDGN1,IDGN2,ICOND,AK,HSE,WLE,HI,HF,AD,RI,RF,IFLAG)
      IF(IFLAG.GT.0)GO TO 500
      HCIEER= HI
      HCFEER= HF
      DAMEER= AD
      RIEER= RI
      RFEER= RF
C
C        4. DESIGN - EXTREME CONDITIONS
C                  - REFLECTED HS
C           CONDITIONS - OPERATIONAL
C
  330 IDGN1= 2
      IDGN2= 2
      ICOND= 1
      CALL STRSTA(IDGN1,IDGN2,ICOND,AK,HSO,WLO,HI,HF,AD,RI,RF,IFLAG)
      IF(IFLAG.GT.0)GO TO 500
      HCIEOR= HI
      HCFEOR= HF
      DAMEOR= AD
      RIEOR= RI
      RFEOR= RF
C
C-----------------------------------------------------------------------C
C        COMPUTE THE PERFORMANCE OF THE BREAKWATER FOR
C                THE FOLLOWING SITUATIONS:
C
  340 IF(IDGN1.EQ.2)GO TO 350
C
C
C                          TRANSMITTED
C
C        1. DESIGN - OPERATIONAL
C           CONDITIONS - OPERATIONAL
C           PREVIOUS CONDITIONS - OPERATIONAL NOT EXCEEDED
C
      HSI= HSO
      TSI= TSO
      QPI= QPO
```

```
      IF(IDGN2.EQ.1)THEN
       HCP= HCFOOT
       HCON= HCIOOT
      ELSE
       HCP= HCFOOR
       HCON= HCIOOR
      ENDIF
      AT= HCON*((HCON/SS)+WIDTH)
      ALP= WLO
      CALL STRPER(HSI,TSI,QPI,HCP,AT,ALP,HST,TST,QPT,HSR)
      IF(IDGN2.EQ.1)THEN
       HTTOO1= HST
       TTTOO1= TST
       QTTOO1= QPT
       HTROO1= HSR
      ELSE
       HRTOO1= HST
       TRTOO1= TST
       QRTOO1= QPT
       HRROO1= HSR
      ENDIF
C
C        2. DESIGN - OPERATIONAL
C           CONDITIONS - OPERATIONAL
C           PREVIOUS CONDITIONS - EXTREME CONDITIONS REACHED
C
      HSI= HSO
      TSI= TSO
      QPI= QPO
      IF(IDGN2.EQ.1)THEN
       HCP= HCFOET
       HCON= HCIOET
      ELSE
       HCP= HCFOER
       HCON= HCIOER
      ENDIF
      AT= HCON*((HCON/SS)+WIDTH)
      ALP= WLO
      CALL STRPER(HSI,TSI,QPI,HCP,AT,ALP,HST,TST,QPT,HSR)
      IF(IDGN2.EQ.1)THEN
       HTTOO2= HST
       TTTOO2= TST
       QTTOO2= QPT
       HTROO2= HSR
      ELSE
       HRTOO2= HST
       TRTOO2= TST
       QRTOO2= QPT
       HRROO2= HSR
      ENDIF
C
C        3. DESIGN - OPERATIONAL
C           CONDITIONS - EXTREME
C           PREVIOUS CONDITIONS - EXTREME CONDITIONS REACHED
C
      HSI= HSE
      TSI= TSE
      QPI= QPE
      IF(IDGN2.EQ.1)THEN
       HCP= HCFOET
```

```
      HCON= HCIOET
     ELSE
      HCP= HCFOER
      HCON= HCIOER
     ENDIF
     AT= HCON*((HCON/SS)+WIDTH)
     ALP= WLE
     CALL STRPER(HSI,TSI,QPI,HCP,AT,ALP,HST,TST,QPT,HSR)
     IF(IDGN2.EQ.1)THEN
      HTTOE= HST
      TTTOE= TST
      QTTOE= QPT
      HTROE= HSR
     ELSE
      HRTOE= HST
      TRTOE= TST
      QRTOE= QPT
      HRROE= HSR
     ENDIF
     GO TO 360
C
C        1. DESIGN - EXTREME
C           CONDITIONS - OPERATIONAL
C           PREVIOUS CONDITIONS - OPERATIONAL NOT EXCEEDED
C
  350 HSI= HSO
     TSI= TSO
     QPI= QPO
     IF(IDGN2.EQ.1)THEN
      HCP= HCIEOT
      HCON= HCIEET
     ELSE
      HCP= HCIEOR
      HCON= HCIEER
     ENDIF
     AT= HCON*((HCON/SS)+WIDTH)
     ALP= WLO
     CALL STRPER(HSI,TSI,QPI,HCP,AT,ALP,HST,TST,QPT,HSR)
     IF(IDGN2.EQ.1)THEN
      HTTEO1= HST
      TTTEO1= TST
      QTTEO1= QPT
      HTREO1= HSR
     ELSE
      HRTEO1= HST
      TRTEO1= TST
      QRTEO1= QPT
      HRREO1= HSR
     ENDIF
C
C        2. DESIGN - EXTREME
C           CONDITIONS - EXTREME
C           PREVIOUS CONDITIONS - EXTREME CONDITIONS REACHED
C
     HSI= HSE
     TSI= TSE
     QPI= QPE
     IF(IDGN2.EQ.1)THEN
      HCP= HCFEET
      HCON= HCIEET
```

```
      ELSE
       HCP= HCFEER
       HCON= HCIEER
      ENDIF
      AT= HCON*((HCON/SS)+WIDTH)
      ALP= WLE
      CALL STRPER(HSI,TSI,QPI,HCP,AT,ALP,HST,TST,QPT,HSR)
      IF(IDGN2.EQ.1)THEN
       HTTEE= HST
       TTTEE= TST
       QTTEE= QPT
       HTREE= HSR
      ELSE
       HRTEE= HST
       TRTEE= TST
       QRTEE= QPT
       HRREE= HSR
      ENDIF
C
C
C        3. DESIGN - EXTREME
C           CONDITIONS - OPERATIONAL
C           PREVIOUS CONDITIONS - EXTREME CONDITIONS REACHED
C
      HSI= HSO
      TSI= TSO
      QPI= QPO
      IF(IDGN2.EQ.1)THEN
       HCP= HCFEET
       HCON= HCIEET
      ELSE
       HCP= HCFEER
       HCON= HCIEER
      ENDIF
      AT= HCON*((HCON/SS)+WIDTH)
      ALP= WLO
      CALL STRPER(HSI,TSI,QPI,HCP,AT,ALP,HST,TST,QPT,HSR)
      IF(IDGN2.EQ.1)THEN
       HTTEO2= HST
       TTTEO2= TST
       QTTEO2= QPT
       HTREO2= HSR
      ELSE
       HRTEO2= HST
       TRTEO2= TST
       QRTEO2= QPT
       HRREO2= HSR
      ENDIF
C
C-----------------------------------------------------------------------C
C                                OUTPUT
C
C        INPUT DATA
C
  360 WRITE(11,370)
  370 FORMAT(1X,'_____________________________________________________________
     1_____',//10X,'DATA INPUT TO PROGRAM',/1X,'_____________________________
     2_____________________________________',/)
      WRITE(11,380)HSO,TSO,TPO,QPO,HSE,TSE,TPE,QPE,BS,WD,WIDTH,
     1SS,WW,WR,W50
```

```
  380 FORMAT(4X,'OPERATIONAL INCIDENT WAVE CONDITIONS',/1X,
     1'SIGNIFICANT WAVE HEIGHT (meters) ------------------- ',F7.2,/1X,
     2'SIGNIFICANT WAVE PERIOD (seconds) ------------------- ',F6.2,/1X,
     3'PEAK PERIOD (seconds) ------------------------------- ',F6.2,/1X,
     4'SPECTRICAL PEAKEDNESS PARAMETER (Qp) ---------------- ',
     5F6.2,//4X,
     6'EXTREME INCIDENT WAVE CONDITIONS',/1X,
     7'SIGNIFICANT WAVE HEIGHT (meters) ------------------- ',F7.2,/1X,
     8'SIGNIFICANT WAVE PERIOD (seconds) ------------------- ',F6.2,/1X,
     9'PEAK PERIOD (seconds) ------------------------------- ',F6.2,/1X,
     1'SPECTRICAL PEAKEDNESS PARAMETER (Qp) ---------------- ',
     2F6.2,//4X,'GENERAL PARAMETERS',/1X,
     3'BOTTOM SLOPE AT STRUCTURE -------------------------  ',
     4F6.4,/1X,
     5'WATER DEPTH AT STRUCTURE (meters) ------------------ ',F7.1,/1X,
     6'INITIAL WIDTH OF STRUCTURE CREST (meters) ----------- ',F6.1,/1X,
     7'STRUCTURE SLOPE ------------------------------------ ',
     8F6.3,/1X,
     9'MASS DENSITY OF WATER (kg/m**3) -------------------- ',F7.1,/1X,
     1'MASS DENSITY OF STONE (kg/m**3) ------------------- ',F8.1,/1X,
     2'MEDIAN STONE MASS (kg) ---------------------------- ',F8.1,/1X,
     3'_____________________________________________________',//)
C
C        OUTPUT DATA
C
      WRITE(11,390)
  390 FORMAT(4X,'THE COMPUTATIONS MADE IN THIS PROGRAM ARE BASED',/2X,
     1'PRIMARILY ON THE RESULTS OF LABORATORY TESTS WITH',/2X,
     2'TRAPAZOIDAL STRUCTURES WITH FORWARD AND BACK SLOPES',/2X,
     3'OF 1.5 (HORIZONTAL) ON 1 (VERTICAL). IN THIS VERSION ',/2X,
     4'OF THE PROGRAM NO ATTEMPT HAS BEEN MADE TO ACCOUNT FOR',/2X,
     5'SCALE EFFECTS.',//)
      WRITE(11,400)
  400 FORMAT(1X,'_____________________________________________________
     1_____',/1X,'_____________________________________________________
     2_______',//2X,'STRUCTURE DESIGN BASED ON:')
      IF(IDGN1.EQ.1)THEN
       WRITE(11,410)
  410  FORMAT(6X,'1) OPERATIONAL WAVE CONDITIONS')
      ELSE
       WRITE(11,420)
  420  FORMAT(6X,'1) EXTREME WAVE CONDITIONS')
      ENDIF
      IF(IDGN2.EQ.1)THEN
       WRITE(11,430)HSD
  430  FORMAT(6X,'2) DESIRED TRANSMITTED SIGNIFICANT WAVE HEIGHT',/10X,
     1 'HST (meters) = ',F6.2)
      ELSE
       WRITE(11,440)HSD
  440  FORMAT(6X,'2) DESIRED REFLECTED SIGNIFICANT WAVE HEIGHT',/10X,
     1 'HSR (meters) = ',F6.2)
      ENDIF
C
  450 IF(IDGN1.EQ.1.AND.IDGN2.EQ.1)THEN
       WRITE(11,460)HSO,TSO,TPO,WLO,QPO,HCIOOT,RIOOT,HCFOOT,RFOOT,AT,
     1SSNO,DAMOOT,HTTOO1,TSO,TTTOO1,TPO,QPO,QTTOO1,HTROO1,HCIOET,
     2HCFOET,AT,DAMOET,HTTOO2,TSO,TTTOO2,TPO,QPO,QTTOO2,HTROO2,HSE,
     3TSE,TPE,WLE,QPE,HCIOET,HCFOET,RFOET,AT,SSNE,DAMOET,HTTOE,TSE,
     4TTTOE,TPE,QPE,QTTOE,HTROE
       GO TO 500
```

```
      ENDIF
      IF(IDGN1.EQ.1.AND.IDGN2.EQ.2)THEN
       WRITE(11,460)HSO,TSO,TPO,WLO,QPO,HCIOOR,RIOOR,HCFOOR,RFOOR,AT,
     1SSNO,DAMOOR,HRTOO1,TSO,TRTOO1,TPO,QPO,QRTOO1,HRROO1,HCIOER,
     2HCFOER,AT,DAMOER,HRTOO2,TSO,TRTOO2,TPO,QPO,QRTOO2,HRROO2,HSE,
     3TSE,TPE,WLE,QPE,HCIOER,HCFOER,RFOER,AT,SSNE,DAMOER,HRTOE,TSE,
     4TRTOE,TPE,QPE,QRTOE,HRROE
       GO TO 500
      ENDIF
      IF(IDGN1.EQ.2.AND.IDGN2.EQ.1)THEN
       WRITE(11,460)HSO,TSO,TPO,WLO,QPO,HCIEET,RIEET,HCFEOT,RFEOT,AT,
     1SSNO,DAMEOT,HTTEO1,TSO,TTTEO1,TPO,QPO,QTTEO1,HTREO1,HCIEET,
     2HCFEET,AT,DAMEET,HTTEO2,TSO,TTTEO2,TPO,QPO,QTTEO2,HTREO2,HSE,
     3TSE,TPE,WLE,QPE,HCIEET,HCFEET,RFEET,AT,SSNE,DAMEET,HTTEE,TSE,
     4TTTEE,TPE,QPE,QTTEE,HTREE
       GO TO 500
      ENDIF
      IF(IDGN1.EQ.2.AND.IDGN2.EQ.2)THEN
       WRITE(11,460)HSO,TSO,TPO,WLO,QPO,HCIEER,RIEER,HCFEOR,RFEOR,AT,
     1SSNO,DAMEOR,HRTEO1,TSO,TRTEO1,TPO,QPO,QRTEO1,HRREO1,HCIEER,
     2HCFEER,AT,DAMEER,HRTEO2,TSO,TRTEO2,TPO,QPO,QRTEO2,HRREO2,HSE,
     3TSE,TPE,WLE,QPE,HCIEER,HCFEER,RFEER,AT,SSNE,DAMEER,HRTEE,TSE,
     4TRTEE,TPE,QPE,QRTEE,HRREE
      ENDIF
C
C        OUTPUT FORMAT
C
  460 FORMAT(/10X,'FOR OPERATIONAL INCIDENT WAVES ',/2X,
     1'SIGNIFICANT WAVE HEIGHT (meters) -------------------- ',F6.2,/2X,
     2'SIGNIFICANT WAVE PERIOD (seconds) -------------------- ',F5.1,/2X,
     3'PEAK PERIOD (seconds) ------------------------------- ',F5.1,/2X,
     4'WAVE LENGTH (peak period) (meters)------------------- ',F6.1,/2X,
     5'PEAKEDNESS PARAMETER, Qp, --------------------------- ',
     6F5.1,///2X,
     7'THE FOLLOWING STRUCTURE AND WAVE PARAMETERS ARE FOR',/2X,
     8'A STRUCTURE THAT HAS ONLY EXPERIENCED WAVE CONDITIONS',/,2X,
     9'NO GREATER THAN OPERATIONAL',//2X,
     1'CONSTRUCTED STRUCTURE CREST HEIGHT (meters) --------- ',F5.1,/2X,
     2'INITIAL DIMENSIONLESS FREEBOARD, R,----------------- ',F6.2,/2X,
     3'STRUCTUE CREST HEIGHT AFTER OPERATIONAL WAVES(meters) ',F5.1,/2X,
     4'DIMENSIONLESS FREEBOARD AFTER OPERATIONAL WAVES ,R,- ',F6.2,/2X,
     5'TOTAL STRUCTURE CROSS-SECTIONAL AREA (meters**2) -- ',F7.1,/2X,
     6'OPERATIONAL SPECTRICAL STABILITY NO. ,Ns*,---------  ',F6.2,/2X,
     7'AREA OF DAMAGE (meters**2) ------------------------ ',F7.1,//10X,
     8'TRANSMITTED WAVES',/2X,
     9'SIGNIFICANT WAVE HEIGHT (meters) -------------------- ',F6.2,/2X,
     1'SIGNIFICANT WAVE PERIOD (seconds) --- BETWEEN',F5.1,' AND',
     2F5.1,/2X,
     3'PEAK PERIOD (seconds) ------------------------------- ',F5.1,/2X,
     4'PEAKEDNESS PARAMETER, Qp, ----------- BETWEEN',F5.1,' AND',
     5F5.1,//10X,
     6'REFLECTED WAVES',/2X,
     7'SIGNIFICANT WAVE HEIGHT (meters) -------------------- ',F6.2,/1X,
     8'------------------------------------------------------------ ',
     9///2X,
     1'THE FOLLOWING STRUCTURE AND WAVE PARAMETERS ARE FAR',/2X,
     2'A STRUCTURE THAT HAS EXPERIENCED EXTREME WAVE CONDITIONS',//2X,
     3'CONSTRUCTED STRUCTURE CREST HEIGHT (meters) --------- ',F5.1,/2X,
     4'STRUCTUE CREST HEIGHT AFTER EXTREME WAVES(meters) --- ',F5.1,/2X,
     5'TOTAL STRUCTURE CROSS-SECTIONAL AREA (meters**2) -- ',F7.1,/2X,
```

```
     6'AREA OF DAMAGE (meters**2) ------------------------ ',F7.1,//10X,
     7'TRANSMITTED WAVES',/2X,
     8'SIGNIFICANT WAVE HEIGHT (meters) ------------------- ',F6.2,/2X,
     9'SIGNIFICANT WAVE PERIOD (seconds) --- BETWEEN',F5.1,' AND',
     1F5.1,/2X,
     2'PEAK PERIOD (seconds) ------------------------------ ',F5.1,/2X,
     3'PEAKEDNESS PARAMETER, Qp, ----------- BETWEEN',F5.1,' AND',
     4F5.1,//10X,
     5'REFLECTED WAVES',/2X,
     6'SIGNIFICANT WAVE HEIGHT (meters) ------------------- ',F6.2,//1X,
     7'------------------------------------------------------------ ',/1X
     8,'------------------------------------------------------------ ',
     9///,10X,
     1'FOR EXTREME INCIDENT WAVES ',/2X,
     2'SIGNIFICANT WAVE HEIGHT (meters) ------------------- ',F6.2,/2X,
     3'SIGNIFICANT WAVE PERIOD (seconds) ------------------- ',F5.1,/2X,
     4'PEAK PERIOD (seconds) ------------------------------ ',F5.1,/2X,
     5'WAVE LENGTH (peak period) (meters)------------------ ',F6.1,/2X,
     6'PEAKEDNESS PARAMETER, Qp, --------------------------- ',
     7F5.1,///2X,
     8'CONSTRUCTED STRUCTURE CREST HEIGHT (meters) --------- ',F5.1,/2X,
     9'STRUCTUE CREST HEIGHT AFTER EXTREME WAVES(meters) --- ',F5.1,/2X,
     1'DIMENSIONLESS FREEBOARD AFTER EXTREME WAVES,R,------ ',F6.2,/2X,
     2'TOTAL STRUCTURE CROSS-SECTIONAL AREA (meters**2) -- ',F7.1,/2X,
     3'EXTREME SPECTRICAL STABILITY NO. ,Ns*,-------------  ',F6.2,/2X,
     4'AREA OF DAMAGE (meters**2) ------------------------ ',F7.1,//10X,
     5'TRANSMITTED WAVES',/2X,
     6'SIGNIFICANT WAVE HEIGHT (meters) ------------------- ',F6.2,/2X,
     7'SIGNIFICANT WAVE PERIOD (seconds) --- BETWEEN',F5.1,' AND',
     8F5.1,/2X,
     9'PEAK PERIOD (seconds) ------------------------------ ',F5.1,/2X,
     1'PEAKEDNESS PARAMETER, Qp, ----------- BETWEEN',F5.1,' AND',
     2F5.1,//10X,
     3'REFLECTED WAVES',/2X,
     4'SIGNIFICANT WAVE HEIGHT (meters) ------------------- ',F6.2,//1X,
     5'____________________________________________________',///)
  500 STOP
      END
C       * END MAIN *
C*********************************************************************C
```

```
C*********************************************************************C
C
      SUBROUTINE WAVLEN(WD,T,L)
C
C---------------------------------------------------------------------C
C         SUBROUTION WAVLEN                                           C
C         WAVE LENGTH                                                 C
C                                                                     C
C         PURPOSE:                                                    C
C         The purpose of this subroutine is to compute the wave length, C
C         L, (linear theory) of a wave in water depth, WD, and with a C
C         period, T.                                                  C
C                                                                     C
C---------------------------------------------------------------------C
C
      REAL L,LN
      PI=3.14159
      ACC= .001
      G= 9.800
   10 RTEST= WD/(G*T**2)
      IF(RTEST.LT..0025)THEN
        L= T*SQRT(G*WD)
        GO TO 20
      ENDIF
      IF(RTEST.GT..08)THEN
        L= (G*T**2)/(2.*PI)
        GO TO 20
      ENDIF
      L= (T*SQRT(G*WD)+ (G*T**2)/(2.*PI))/2.
   20 A= G*T**2
      B= A/(2.*PI)
      C= A*WD
      E= 2.*PI*WD
      DEL= 1.0
   30 IF(DEL.GE.ACC)THEN
        CL= C/L**2
        EL= E/L
        SECH2= (1.0/(COSH(EL)))**2
        LN= L-((L-B*TANH(EL))/(1.+CL*SECH2))
        DEL= ABS((LN-L)/L)
        L= LN
        GO TO 30
      ENDIF
      RETURN
      END
C       * END SUBROUTINE WAVLEN *
C*********************************************************************C
```

```
C*********************************************************************C
C
      SUBROUTINE STRSTA(IDGN1,IDGN2,ICOND,AK,HS,WL,HI,HF,AD,RI,RF,IFLAG)
C
C-------------------------------------------------------------------C
C                                                                   C
C        SUBROUTINE STRSTA                                          C
C        STRUCTURE STABILITY                                        C
C                                                                   C
C        PURPOSE:                                                   C
C        The purpose of this subroutine is to compute initial and   C
C        final (after damage) structure crest heights for given     C
C        incident and desired transmitted/reflected wave conditions. C
C        The program also computes the area of damage.              C
C                                                                   C
C-------------------------------------------------------------------C
C
      COMMON HSO,TPO,QPO,HSE,TPE,QPE,WIDTH,SS,WW,WR,W50,
     1SSND,WD,WLO,WLE,D50,HCIOOT,HCIOOR,HCIEET,HCIEER
C
C
      ICT= 1
      D50S= D50**2
      A11= 0.9
      A21= -.358
      A12= 1.0/(A11+A21)
      A113= -.2338
      A213= 1.436
      A123= .03737
      A223= -.06997
      AEX= .5926
      A114= .5085
      A214= -2.018
      A124= -1.019
      A224= 137.6
      A115= .7195
      A215= -3.4
      A125= 48.7
      A225= 268.1
      A16= 19.45
      A26= -7.455
      A36= .7605
      A46= -.01048
C
      SSNN= ((HS**2)*WL)**.333333
      SSN= SSNN/SSND
      IF(SSN.LE.6.0)THEN
       SSNM= 6.0
      ELSE
       SSNM= SSN
      ENDIF
      IF(IDGN1.EQ.1.AND.ICOND.EQ.2)GO TO 120
      IF(IDGN1.EQ.2.AND.ICOND.EQ.1)GO TO 130
      IF(IDGN2.EQ.2)GO TO 100
C
C                          TRANSMITTED
C
C        FOR AK > 0.542  (i.e. R < 0.0)
C
      IF(AK.LT.0.542)GO TO 50
```

```
      SPAT= (AK-A11)/A21
      HF= WD+HS*ALOG(SPAT)
      IF(HF.LT.0.0)HF= 0.0
C     WRITE(*,10)SPAT,HF
C  10 FORMAT(1X,'ARGUMENT = ',F6.3,/1X,'HF = ',F8.2)
       IF(IDGN1.EQ.1)THEN
        RF= (HF-WD)/HSO
       ELSE
        RF= (HF-WD)/HSE
       ENDIF
      GO TO 110
C
C        ITERATIVE PROCEDURE FOR DETERMINING HF
C        (PROCEEDURE USED WHEN AK < .542 i.e. WHEN
C         0.0 < R < 1.0)
C
   50 HCF= WD
      BK1= A113+A123*SSNM
      BK2= A213+A223*SSNM
      BK3= SS
      BK4= WIDTH
      BK5= HS/(WL*D50S)
      BK6= 1.0-A12
      BK7= A12-(1.0/AK)
      BK23= 1.0/(BK2*BK3)
      BK523= BK5*BK23
   55 HCF1= (HCF-BK1)/(BK2*BK3)
      HCF2= HCF1+BK4
      R= (HCF-WD)/HS
      IC= 1
   60 IF(IC.GE.100)THEN
       WRITE(*,70)
   70  FORMAT(1X,'*** NEWTON RAPHSON SCHEME FOR COMPUTING FINAL STRUCTUR
     1E',/1X,'CREST HEIGHT DID NOT CONVERGE AFTER 100 ITERATIONS ***',
     2 //1X,'IDGN1 = ',I3,/1X,'IDGN2 = ',I3,/1X,'ICOND = ',I3)
       GO TO 90
      ENDIF
      F=(BK6+(BK5*HCF1*HCF2)**AEX)*R+BK7
      AM1= AEX-1.0
      FP= (BK6+(BK5*HCF1*HCF2)**AEX)*(1.0/HS)+
     1(AEX*(BK5*HCF1*HCF2)**AM1)*(BK5*HCF1*BK23+BK523*HCF2)*R
      HCN= HCF-(F/FP)
      TEST= ABS((HCN-HCF)/HCF)
      IF(TEST.LE.0.005)THEN
       HF= HCN
       IF(HF.LT.0.0)HF= 0.0
       IF(IDGN1.EQ.1)THEN
        RF= (HF-WD)/HSO
       ELSE
        RF= (HF-WD)/HSE
       ENDIF
       GO TO 110
      ELSE
       HCF= HCN
       IC= IC+1
       GO TO 55
      ENDIF
C
C        FOR R > 1.0
C
```

```
   90 ICT= ICT+1
      AEXR= 1.0/AEX
      AKS= ((1.0/AK)-1.0)**AEXR
      TM1= (WIDTH*SS)**2
      TM2= 4.0*WL*D50S*AKS*SS/HS
      SARG= SQRT(TM1+TM2)
      TM3= SARG/2.0
      TM4= -WIDTH*SS/2.0
      HF= (A113+A123*SSN)*WD+(A213+A223*SSN)*(TM4+TM3)
      IF(HF.LE.0.0)HF= 0.0
      GO TO 110
C
C                              REFLECTED
C
  100 DL= WD/WL
      A14= A114+A214*DL
      A24= 1.0/(A124+A224*DL)
      A15= A115+A215*DL
      A25= 1.0/(A125+A225*DL)
      TK1= A14+A24*1.0
      TK2= A15+A25*3.0
      IF(TK1.GE.AK)THEN
       HF= HS*((AK-A14)/A24)+WD
       IF(HF.LT.0.0)HF= 0.0
       IF(IDGN1.EQ.1)THEN
        RF= (HF-WD)/HSO
       ELSE
        RF= (HF-WD)/HSE
       ENDIF
       GO TO 110
      ENDIF
      IF(TK1.LT.AK.AND.TK2.GE.AK)THEN
       HF= HS*((AK-A15)/A25)+WD
       IF(HF.LT.0.0)HF= 0.0
       IF(IDGN1.EQ.1)THEN
        RF= (HF-WD)/HSO
       ELSE
        RF= (HF-WD)/HSE
       ENDIF
       GO TO 110
      ELSE
       AKK= (A15+A25*3.0)
       HRR= HS*AKK
       HF= HS*((AKK-A15)/A25)+WD
       IF(HF.LT.0.0)HF= 0.0
       IF(IDGN1.EQ.1)THEN
        RF= (HF-WD)/HSO
       ELSE
        RF= (HF-WD)/HSE
       ENDIF
       WRITE(*,105)HRR
  105  FORMAT(3X,' THE MAXIMUM REFLECTED Hsr OBTAINABLE FOR THE GIVEN',
     1 /1X,' INCIDENT Hsi AND Tpi IS Hsr = ',F6.2)
      ENDIF
C
C        COMPUTE INITIAL CREST HEIGHT
C
C        hf = hc(final)
C        hi = hc(initial)
C        hf/ds = A13 + A23*SSN
```

```
C         hf = ds*(A13 + A23*SSN)
C         A13 = A113 + A213*(hi/ds)
C         A23 = A123 + A223*(hi/ds)
C         hi = ds*((hf/ds)-A113-A123*SSN)/(A213+A223*SSN)
C
  110 IF(SSN.LE.6.0)THEN
       HI= HF
       RI= RF
       AD= 0.0
       GO TO 115
      ENDIF
      IF(HF.EQ.0.0)THEN
       HI= 0.0
       GO TO 114
      ENDIF
      HI= WD*((HF/WD)-A113-A123*SSN)/(A213+A223*SSN)
  114 IF(IDGN1.EQ.1)THEN
       RI= (HI-WD)/HSO
      ELSE
       RI= (HI-WD)/HSE
      ENDIF
      IF(AK.GE.0.542.OR.IDGN2.EQ.2)GO TO 150
  115 AT= HI*((HI/SS)+WIDTH)
      P= HS*AT/(WL*D50S)
      PE= P**AEX
      A22= (1.0-A12)+PE
      HST= HS*(1.0/(A12+A22*RF))
      AAR= (AK-(HST/HS))/AK
      DH= ABS(AAR)
      IF(DH.GT.0.1.AND.ICT.EQ.1)THEN
       ICT= ICT+1
       GO TO 90
      ENDIF
      IF(DH.GT.0.1.AND.ICT.GT.1)THEN
       WRITE(*,117)
  117  FORMAT(1X,'***** ITERATIVE PROCEDURE FOR HF FAILED TO CONVERGE **
     1***',/1X,'     CHANGE INPUT PARAMETERS AND TRY AGAIN')
       IFLAG= 1
       GO TO 200
      ENDIF
      GO TO 150
C
C         END INITIAL HEIGHT COMPUTATIONS
C
C         OPERATIONAL DESIGN - EXTREME CONDITIONS
C
  120 IF(IDGN2.EQ.1)THEN
       HI= HCIOOT
      ELSE
       HI= HCIOOR
      ENDIF
      RI= (HI-WD)/HSO
      GO TO 135
C
C         EXTREME DESIGN - OPERATIONAL CONDITIONS
C
  130 IF(IDGN2.EQ.1)THEN
       HI= HCIEET
      ELSE
       HI= HCIEER
```

```
      ENDIF
      RI= (HI-WD)/HSE
C
  135 IF(SSN.LE.6.0)THEN
       HF= HI
       AD= 0.0
       RF= RI
       GO TO 200
      ENDIF
      DHI= HI/WD
      A13= A113+A213*DHI
      A23= A123+A223*DHI
      HF= WD*(A13+A23*SSNM)
      IF(HF.GT.HI)HF= HI
      IF(HI.EQ.0.0)HF= 0.0
      IF(IDGN1.EQ.1)THEN
       RF= (HF-WD)/HSO
      ELSE
       RF= (HF-WD)/HSE
      ENDIF
C
C        COMPUTE AREA OF DAMAGE
C
  150 ET= (HI/WD)**1.5
      X= SSN*ET
      IF(X.GT.30.)THEN
       WRITE(*,160)
  160  FORMAT(1X,'*** VALUE OF PRODUCT OF STRUCTURAL STABILITY NUMBER',
     1 /1X,'AND (INITIAL CREST HEIGHT/WATER DEPTH)**1.5 HAS EXCEEDED',
     2 /1X,'30. THIS PRODUCT IS USED IN COMPUTING THE AREA OF DAMAGE.',
     3 /1X,'EXISTING DATA DOES NOT ALLOW COMPUTATION FOR VALUES GREATER
     4 ',/1X,'THAN 30.')
       AD= 999999
       GO TO 200
      ENDIF
      IF(HI.EQ.0.0)THEN
       AD= 0.0
       GO TO 200
      ENDIF
      IF(X.LT.6.0)THEN
       AD= 0.0
      ELSE
       X2= X**2
       X3= X**3
       AD= (D50**2)*(A16+A26*X+A36*X2+A46*X3)
      ENDIF
C
  200 RETURN
      END
C      * END SUBROUTINE STRSTA *
C*****************************************************************C
```

```
C*********************************************************************C
C
      SUBROUTINE STRPER(HSI,TSI,QPI,HCP,AT,ALP,HST,TST,QPT,HSR)
C
C---------------------------------------------------------------------C
C                                                                     C
C        SUBROUTINE STRPER                                            C
C        STRUCTURE PERFORMAMCE                                        C
C                                                                     C
C        PURPOSE:                                                     C
C        The purpose of this program is to compute the TRANSMITTED    C
C        and REFLECTED wave parameters (significant height, peak      C
C        period and spectral peakedness Qp) for a specified structure C
C        and incident wave conditions.                                C
C                                                                     C
C---------------------------------------------------------------------C
      COMMON HSO,TPO,QPO,HSE,TPE,QPE,WIDTH,SS,WW,WR,W50,
     1SSND,WD,WLO,WLE,D50,HCIOOT,HCIOOR,HCIEET,HCIEER
C
      A11= 0.9
      A21= -.358
      A12= 1.0/(A11+A21)
      AEX= .5926
      A114= .5085
      A214= -2.018
      A124= -1.019
      A224= 137.6
      A115= .7195
      A215= -3.4
      A125= 48.7
      A225= 268.1
C
      R= (HCP-WD)/HSI
C
C        TRANSMITTED HS
C
      IF(R.LT.-3.5.OR.HCP.EQ.0.0)THEN
       HST= HSI
       GO TO 20
      ENDIF
      IF(R.LE.0.0)THEN
       HST= HSI*(A11+A21*EXP(R))
       GO TO 20
      ENDIF
       P= HSI*AT/(ALP*(D50**2))
       PE= P**AEX
      IF(R.GT.0.0.AND.R.LE.1.0)THEN
       A22= (1.0-A12)+PE
       HST= HSI*(1.0/(A12+A22*R))
       GO TO 20
      ELSE
       HST= HSI/(1.0+PE)
      ENDIF
C
C        TRANSMITTED TS
C
   20 IF(R.LT.-3.5.OR.HCP.EQ.0.0)THEN
       TST= TSI
       GO TO 30
      ENDIF
```

```
      IF(R.LE.1.0.AND.R.GE.-3.5)THEN
       TST= TSI*(0.63-0.106*R)
       GO TO 30
      ENDIF
      IF(R.GT.1.0)THEN
       TST= TSI*1.2
       GO TO 30
      ENDIF
C
C        TRANSMITTED QP
C
   30 IF(R.LT.-3.5.OR.HCP.EQ.0.0)THEN
       QPT= QPI
       GO TO 40
      ENDIF
      IF(R.LE.1.0.AND.R.GE.-3.5)THEN
       QPT= QPI*(0.67-0.095*R)
       GO TO 40
      ENDIF
      IF(R.GT.1.0)THEN
       QPT= QPI*1.28
       GO TO 40
      ENDIF
C
C        REFLECTED HS
C
   40 DL= WD/ALP
      IF(R.LT.-3.5.OR.HCP.EQ.0.0)THEN
       HSR= 0.0
       GO TO 50
      ENDIF
      IF(R.LE.1.0)THEN
       A14= A114+A214*DL
       A24= 1.0/(A124+A224*DL)
       HSR= HSI*(A14+A24*R)
       GO TO 50
      ENDIF
       A15= A115+A215*DL
       A25= 1.0/(A125+A225*DL)
      IF(R.GT.1.0.AND.R.LE.3.0)THEN
       HSR= HSI*(A15+A25*R)
      ELSE
       HSR= HSI*(A15+A25*3.0)
      ENDIF
   50 RETURN
      END
C      * END SUBROUTINE STRPER *
C*****************************************************************C
```

EXAMPLE PROBLEM:

A longshore, detached, low crested breakwater is to be constructed to reduce the wave heights landward of the breakwater for routine navigation of these waters. The relatively steady winds in that area produce a somewhat consistent sea state characterized by a significant wave height Hs = 2.5 meters, peak period Tp = 7 seconds and peakedness parameter Qp = 2.5. The most severe wave conditions expected during the life of the structure are: Hs = 8.0 meters, Tp = 16 seconds and Qp = 4.0. The desired significant wave height "inside" the breakwater is less than or equal to 1.0 meter (i.e. Ht = 1.0 meter) during "operational" conditions. The water depth at the structure site is 10 meters and the bottom slope is 1 on 25. The water mass density is 1025 kg/cubic meter. Two stone densities are available for use on the breakwater: wr1 = 2500 kg/cubic meter and wr2 = 2850 kg/cubic meter. The median stone mass W50 = 2030 kg, the constructed crest width is to be 2 meters and the structure slope 1 on 1.5.

Determine:

1. The constructed structure crest height that is necessary to achieve the 1.0 meter transmitted significant wave height for each of the stone densities.
2. The damage experienced by the structure during a) operational waves and b) extreme waves for each stone density.
3. The transmitted and reflected significant wave heights for operational and extreme wave conditions for each stone density.

SOLUTION:

The above information is input into the program LCBDGN. Output files for the two cases (two stone densities) are given on the following pages.

For stone 1 (density = 2500 kg/cubic meter)

1. Constructed structure crest height = 10.4 meters

2. No damage experienced during operational waves

3. Area of damage during extreme waves = 53.5 square meters

4. Operational Incident Waves
   a. Transmitted significant wave height = 1.07 meters
   b. Reflected significant wave height = 0.45 meters

5. Operational Incident Waves (after extreme wave damage)
   a. Transmitted significant wave height = 1.99 meters
   b. Reflected significant wave height = 0.29 meters

6. Extreme Incident waves
   a. Transmitted significant wave height = 5.24 meters
   b. Reflected significant wave height = 2.64 meters

(continued on next page)

For stone 2 (density = 2850 kg/cubic meters)

1. Constructed structure crest height = 10.4 meters

2. No damage experienced during operational waves

3. Area of damage during extreme waves = 30.3 square meters

4. Operational Incident Waves
   a. Transmitted significant wave height = 1.07 meters
   b. Reflected significant wave height = 0.45 meters

5. Operational Incident Waves (after extreme wave damage)
   a. Transmitted significant wave height = 1.87 meters
   b. Reflected significant wave height = 0.33 meters

6. Extreme Incident waves
   a. Transmitted significant wave height = 5.02 meters
   b. Reflected significant wave height = 2.75 meters

STONE 1 (DENSITY = 2500 KG/CUBIC METER)

DATA INPUT TO PROGRAM

OPERATIONAL INCIDENT WAVE CONDITIONS

SIGNIFICANT WAVE HEIGHT (meters)	2.50
SIGNIFICANT WAVE PERIOD (seconds)	7.50
PEAK PERIOD (seconds)	7.00
SPECTRICAL PEAKEDNESS PARAMETER (Qp)	2.50

EXTREME INCIDENT WAVE CONDITIONS

SIGNIFICANT WAVE HEIGHT (meters)	8.00
SIGNIFICANT WAVE PERIOD (seconds)	16.00
PEAK PERIOD (seconds)	16.00
SPECTRICAL PEAKEDNESS PARAMETER (Qp)	4.00

GENERAL PARAMETERS

BOTTOM SLOPE AT STRUCTURE	0.0400
WATER DEPTH AT STRUCTURE (meters)	10.0
INITIAL WIDTH OF STRUCTURE CREST (meters)	2.0
STRUCTURE SLOPE	0.667
MASS DENSITY OF WATER (kg/m**3)	1025.0
MASS DENSITY OF STONE (kg/m**3)	2500.0
MEDIAN STONE MASS (kg)	2030.0

THE COMPUTATIONS MADE IN THIS PROGRAM ARE BASED PRIMARILY ON THE RESULTS OF LABORATORY TESTS WITH TRAPAZOIDAL STRUCTURES WITH FORWARD AND BACK SLOPES OF 1.5 (HORIZONTAL) ON 1 (VERTICAL). IN THIS VERSION OF THE PROGRAM NO ATTEMPT HAS BEEN MADE TO ACCOUNT FOR SCALE EFFECTS.

STRUCTURE DESIGN BASED ON:
1) OPERATIONAL WAVE CONDITIONS
2) DESIRED TRANSMITTED SIGNIFICANT WAVE HEIGHT
HST (meters) = 1.00

FOR OPERATIONAL INCIDENT WAVES

SIGNIFICANT WAVE HEIGHT (meters)	2.50
SIGNIFICANT WAVE PERIOD (seconds)	7.5
PEAK PERIOD (seconds)	7.0
WAVE LENGTH (peak period) (meters)	59.8
PEAKEDNESS PARAMETER, Qp,	2.5

THE FOLLOWING STRUCTURE AND WAVE PARAMETERS ARE FOR A STRUCTURE THAT HAS ONLY EXPERIENCED WAVE CONDITIONS NO GREATER THAN OPERATIONAL

CONSTRUCTED STRUCTURE CREST HEIGHT (meters)	10.4
INITIAL DIMENSIONLESS FREEBOARD, R,	0.18
STRUCTUE CREST HEIGHT AFTER OPERATIONAL WAVES(meters)	10.4
DIMENSIONLESS FREEBOARD AFTER OPERATIONAL WAVES ,R,-	0.18
TOTAL STRUCTURE CROSS-SECTIONAL AREA (meters**2) --	184.4
OPERATIONAL SPECTRICAL STABILITY NO. ,Ns*,	5.36
AREA OF DAMAGE (meters**2)	0.0

STONE 2 (DENSITY = 2850 KG/CUBIC METER)

DATA INPUT TO PROGRAM

OPERATIONAL INCIDENT WAVE CONDITIONS

SIGNIFICANT WAVE HEIGHT (meters)	2.50
SIGNIFICANT WAVE PERIOD (seconds)	7.50
PEAK PERIOD (seconds)	7.00
SPECTRICAL PEAKEDNESS PARAMETER (Qp)	2.50

EXTREME INCIDENT WAVE CONDITIONS

SIGNIFICANT WAVE HEIGHT (meters)	8.00
SIGNIFICANT WAVE PERIOD (seconds)	16.00
PEAK PERIOD (seconds)	16.00
SPECTRICAL PEAKEDNESS PARAMETER (Qp)	4.00

GENERAL PARAMETERS

BOTTOM SLOPE AT STRUCTURE	0.0400
WATER DEPTH AT STRUCTURE (meters)	10.0
INITIAL WIDTH OF STRUCTURE CREST (meters)	2.0
STRUCTURE SLOPE	0.667
MASS DENSITY OF WATER (kg/m**3)	1025.0
MASS DENSITY OF STONE (kg/m**3)	2850.0
MEDIAN STONE MASS (kg)	2030.0

THE COMPUTATIONS MADE IN THIS PROGRAM ARE BASED PRIMARILY ON THE RESULTS OF LABORATORY TESTS WITH TRAPAZOIDAL STRUCTURES WITH FORWARD AND BACK SLOPES OF 1.5 (HORIZONTAL) ON 1 (VERTICAL). IN THIS VERSION OF THE PROGRAM NO ATTEMPT HAS BEEN MADE TO ACCOUNT FOR SCALE EFFECTS.

STRUCTURE DESIGN BASED ON:
1) OPERATIONAL WAVE CONDITIONS
2) DESIRED TRANSMITTED SIGNIFICANT WAVE HEIGHT
HST (meters) = 1.00

FOR OPERATIONAL INCIDENT WAVES

SIGNIFICANT WAVE HEIGHT (meters)	2.50
SIGNIFICANT WAVE PERIOD (seconds)	7.5
PEAK PERIOD (seconds)	7.0
WAVE LENGTH (peak period) (meters)	59.8
PEAKEDNESS PARAMETER, Qp,	2.5

THE FOLLOWING STRUCTURE AND WAVE PARAMETERS ARE FOR A STRUCTURE THAT HAS ONLY EXPERIENCED WAVE CONDITIONS NO GREATER THAN OPERATIONAL

CONSTRUCTED STRUCTURE CREST HEIGHT (meters)	10.4
INITIAL DIMENSIONLESS FREEBOARD, R,	0.17
STRUCTUE CREST HEIGHT AFTER OPERATIONAL WAVES(meters)	10.4
DIMENSIONLESS FREEBOARD AFTER OPERATIONAL WAVES ,R,-	0.17
TOTAL STRUCTURE CROSS-SECTIONAL AREA (meters**2) --	183.6
OPERATIONAL SPECTRICAL STABILITY NO. ,Ns*,	4.53
AREA OF DAMAGE (meters**2)	0.0

(continued)

STONE 1 (DENSITY = 2500 KG/CUBIC METER)

TRANSMITTED WAVES

SIGNIFICANT WAVE HEIGHT (meters) ---------- 1.07
SIGNIFICANT WAVE PERIOD (seconds) --- BETWEEN 7.5 AND 4.6
PEAK PERIOD (seconds) ---------- 7.0
PEAKEDNESS PARAMETER, Qp, ---------- BETWEEN 2.5 AND 1.6

REFLECTED WAVES

SIGNIFICANT WAVE HEIGHT (meters) ---------- 0.45

THE FOLLOWING STRUCTURE AND WAVE PARAMETERS ARE FAR
A STRUCTURE THAT HAS EXPERIENCED EXTREME WAVE CONDITIONS

CONSTRUCTED STRUCTURE CREST HEIGHT (meters) --------- 10.4
STRUCTUE CREST HEIGHT AFTER EXTREME WAVES(meters) --- 7.0
TOTAL STRUCTURE CROSS-SECTIONAL AREA (meters**2) -- 184.4
AREA OF DAMAGE (meters**2) ---------- 53.5

TRANSMITTED WAVES

SIGNIFICANT WAVE HEIGHT (meters) ---------- 1.99
SIGNIFICANT WAVE PERIOD (seconds) --- BETWEEN 7.5 AND 5.7
PEAK PERIOD (seconds) ---------- 7.0
PEAKEDNESS PARAMETER, Qp, ---------- BETWEEN 2.5 AND 2.0

REFLECTED WAVES

SIGNIFICANT WAVE HEIGHT (meters) ---------- 0.29

FOR EXTREME INCIDENT WAVES

SIGNIFICANT WAVE HEIGHT (meters) ---------- 8.00
SIGNIFICANT WAVE PERIOD (seconds) ---------- 16.0
PEAK PERIOD (seconds) ---------- 16.0
WAVE LENGTH (peak period) (meters)---------- 154.2
PEAKEDNESS PARAMETER, Qp, ---------- 4.0

CONSTRUCTED STRUCTURE CREST HEIGHT (meters) --------- 10.4
STRUCTUE CREST HEIGHT AFTER EXTREME WAVES(meters) --- 7.0
DIMENSIONLESS FREEBOARD AFTER EXTREME WAVES,R,------ -1.22
TOTAL STRUCTURE CROSS-SECTIONAL AREA (meters**2) -- 184.4
EXTREME SPECTRICAL STABILITY NO. ,Ns*,---------- 15.98
AREA OF DAMAGE (meters**2) ---------- 53.5

TRANSMITTED WAVES

SIGNIFICANT WAVE HEIGHT (meters) ---------- 5.24
SIGNIFICANT WAVE PERIOD (seconds) --- BETWEEN 16.0 AND 10.7
PEAK PERIOD (seconds) ---------- 16.0
PEAKEDNESS PARAMETER, Qp, ---------- BETWEEN 4.0 AND 2.8

REFLECTED WAVES

SIGNIFICANT WAVE HEIGHT (meters) ---------- 2.64

(continued)

STONE 2 (DENSITY = 2850 KG/CUBIC METER)

TRANSMITTED WAVES

SIGNIFICANT WAVE HEIGHT (meters) ---------- 1.07
SIGNIFICANT WAVE PERIOD (seconds) --- BETWEEN 7.5 AND 4.6
PEAK PERIOD (seconds) ---------- 7.0
PEAKEDNESS PARAMETER, Qp, ---------- BETWEEN 2.5 AND 1.6

REFLECTED WAVES

SIGNIFICANT WAVE HEIGHT (meters) ---------- 0.45

THE FOLLOWING STRUCTURE AND WAVE PARAMETERS ARE FAR
A STRUCTURE THAT HAS EXPERIENCED EXTREME WAVE CONDITIONS

CONSTRUCTED STRUCTURE CREST HEIGHT (meters) --------- 10.4
STRUCTUE CREST HEIGHT AFTER EXTREME WAVES(meters) --- 7.8
TOTAL STRUCTURE CROSS-SECTIONAL AREA (meters**2) -- 183.6
AREA OF DAMAGE (meters**2) ---------- 30.3

TRANSMITTED WAVES

SIGNIFICANT WAVE HEIGHT (meters) ---------- 1.87
SIGNIFICANT WAVE PERIOD (seconds) --- BETWEEN 7.5 AND 5.4
PEAK PERIOD (seconds) ---------- 7.0
PEAKEDNESS PARAMETER, Qp, ---------- BETWEEN 2.5 AND 1.9

REFLECTED WAVES

SIGNIFICANT WAVE HEIGHT (meters) ---------- 0.33

FOR EXTREME INCIDENT WAVES

SIGNIFICANT WAVE HEIGHT (meters) ---------- 8.00
SIGNIFICANT WAVE PERIOD (seconds) ---------- 16.0
PEAK PERIOD (seconds) ---------- 16.0
WAVE LENGTH (peak period) (meters)---------- 154.2
PEAKEDNESS PARAMETER, Qp, ---------- 4.0

CONSTRUCTED STRUCTURE CREST HEIGHT (meters) --------- 10.4
STRUCTUE CREST HEIGHT AFTER EXTREME WAVES(meters) --- 7.8
DIMENSIONLESS FREEBOARD AFTER EXTREME WAVES,R,------ -0.87
TOTAL STRUCTURE CROSS-SECTIONAL AREA (meters**2) -- 183.6
EXTREME SPECTRICAL STABILITY NO. ,Ns*,---------- 13.49
AREA OF DAMAGE (meters**2) ---------- 30.3

TRANSMITTED WAVES

SIGNIFICANT WAVE HEIGHT (meters) ---------- 5.02
SIGNIFICANT WAVE PERIOD (seconds) --- BETWEEN 16.0 AND 10.5
PEAK PERIOD (seconds) ---------- 16.0
PEAKEDNESS PARAMETER, Qp, ---------- BETWEEN 4.0 AND 2.8

REFLECTED WAVES

SIGNIFICANT WAVE HEIGHT (meters) ---------- 2.75

Appendix B

SUBSET NO.	TEST NO.	LP	TP	INC. HS	W50	WR	INITIAL CRST HT	FINAL HT	WATER DEPTH	REL FREEBD	TRANS COEFF	FINAL QP	AREA OF DAMAGE	GODA ARGMENT	GODA STPNS	REF COEF	FINAL TRANS TP	HUDSON STAB NO	SPECTRAL STAB. NO	CRST HT RATIO	DLESS DAMAGE	DEL Hc/ ds
1.	1	208.77	1.45	11.01	17	2.63	24.90	21.92	25	-.280	.629	3.85	49.52	.752	.204	.242	0	3.626	9.6693	.8768	14.27	.1192
1.	2	210.67	1.46	10.14	17	2.63	24.72	23.01	25	-.196	.598	3.31	42.92	.746	.190	.231	0	3.340	9.1807	.9204	12.37	.0684
1.	3	205.41	1.43	8.00	17	2.63	24.11	23.50	25	-.188	.590	3.14	17.74	.765	.146	.387	0	2.635	7.7729	.9400	5.11	.0244
1.	4	208.77	1.45	5.73	17	2.63	25.39	24.44	25	-.098	.597	3.16	10.59	.752	.106	.210	0	1.887	6.2562	.9776	3.05	.0380
1.	5	207.09	1.44	2.87	17	2.63	24.26	23.93	25	-.373	.609	3.18	4.83	.759	.053	.204	0	.945	3.9351	.9572	1.39	.0132
1.	6	337.58	2.23	13.43	17	2.63	24.41	20.70	25	-.320	.708	2.20	77.20	.465	.485	.332	0	4.423	12.9566	.8280	22.25	.1484
1.	7	337.58	2.23	11.50	17	2.63	24.84	21.46	25	-.308	.671	2.58	75.99	.465	.416	.217	0	3.787	11.6835	.8584	21.90	.1352
1.	8	340.50	2.25	9.07	17	2.63	25.48	23.77	25	-.136	.653	2.52	34.75	.461	.332	.379	2.23	2.987	10.0021	.9508	10.01	.0684
1.	9	344.06	2.27	6.09	17	2.63	25.09	24.60	25	-.066	.570	2.48	13.39	.457	.227	.401	2.23	2.006	7.6961	.9840	3.86	.0196
1.	10	345.69	2.28	2.91	17	2.63	24.99	24.54	25	-.158	.543	2.58	8.64	.454	.109	.413	2.27	.958	4.7113	.9816	2.49	.0180
1.	11	337.58	2.23	13.13	17	2.63	25.05	19.99	25	-.382	.684	2.08	91.79	.465	.474	.327	2.21	4.324	12.7629	.7996	26.45	.2024
1.	13	460.82	3.00	15.78	17	2.63	25.73	16.98	25	-.508	.782	1.08	213.03	.341	.968	.311	0	5.197	16.0040	.6792	61.39	.3500
1.	14	460.82	3.00	14.35	17	2.63	24.78	17.59	25	-.516	.728	1.16	168.53	.341	.880	.296	2.79	4.726	15.0219	.7036	48.57	.2876
1.	15	425.69	2.78	11.38	17	2.63	24.44	19.84	25	-.453	.702	1.61	100.61	.369	.607	.299	2.98	3.748	12.5345	.7936	28.99	.1840
1.	16	422.48	2.76	7.81	17	2.63	25.27	22.56	25	-.312	.671	1.87	39.58	.372	.411	.337	2.65	2.572	9.7278	.9024	11.41	.1084
1.	17	420.89	2.75	3.89	17	2.63	24.66	24.44	25	-.144	.616	2.37	2.04	.373	.203	.425	2.83	1.281	6.1048	.9776	.59	.0088
1.	18	453.29	2.95	15.72	17	2.63	24.69	17.10	25	-.503	.767	1.07	170.01	.347	.937	.303	3.03	5.177	15.8760	.6840	48.99	.3036
1.	20	545.64	3.53	5.46	17	2.63	24.14	23.80	25	-.220	.640	2.07	5.95	.288	.455	.461	3.58	1.798	8.3447	.9520	1.71	.0136
1.	21	543.50	3.52	10.07	17	2.63	24.78	18.87	25	-.609	.742	1.60	111.86	.289	.833	.354	3.56	3.316	12.5334	.7548	32.24	.2364
1.	22	556.75	3.60	14.25	17	2.63	25.12	16.49	25	-.597	.760	1.11	181.72	.282	1.232	.322	3.82	4.693	15.9249	.6596	52.37	.3452
1.	23	562.54	3.64	16.10	17	2.63	24.84	15.88	25	-.566	.768	1.19	212.38	.279	1.419	.331	3.63	5.302	17.3347	.6352	61.20	.3584
1.	25	208.77	1.45	11.45	17	2.63	24.23	21.92	25	-.269	.596	3.94	35.77	.752	.213	.240	2.82	3.771	9.9252	.8768	10.31	.0924
1.	26	210.67	1.46	10.08	17	2.63	24.57	21.88	25	-.310	.606	3.39	40.04	.746	.189	.229	1.48	3.320	9.1444	.8752	11.54	.1076
1.	27	339.21	2.24	8.83	17	2.63	25.09	21.82	25	-.360	.652	2.40	43.29	.463	.322	.483	2.21	2.908	9.8125	.8728	12.48	.1308
1.	28	429.31	2.80	11.55	17	2.63	24.99	19.29	25	-.494	.727	1.58	102.66	.366	.625	.312	2.94	3.804	12.6948	.7716	29.59	.2280
1.	29	554.61	3.59	10.38	17	2.63	24.81	18.26	25	-.649	.755	1.71	131.18	.283	.891	.338	3.61	3.419	12.8758	.7304	37.80	.2620
1.	30	561.52	3.63	14.98	17	2.63	24.81	16.70	25	-.554	.758	1.55	204.11	.280	1.316	.330	3.66	4.934	16.5111	.6680	58.82	.3244
2.	12	339.21	2.24	5.87	17	2.63	19.99	19.99	25	-.853	.712	2.38	1.77	.463	.214	.321	2.21	1.933	7.4742	.7996	.51	0
2.	19	337.58	2.23	5.87	17	2.63	17.10	16.86	25	-1.387	.787	2.60	1.58	.465	.212	.271	2.23	1.933	7.4622	.6744	.46	.0096
2.	24	337.58	2.23	5.95	17	2.63	15.88	15.91	25	-1.528	.791	2.57	.65	.465	.215	.243	2.21	1.960	7.5298	.6364	.19	-.0012
3.	31	212.37	1.47	11.36	17	2.63	29.17	24.72	25	-.025	.491	3.96	119.38	.740	.215	.234	1.53	3.741	9.9296	.9888	34.40	.1780
3.	32	208.77	1.45	9.46	17	2.63	30.48	26.43	25	.151	.454	3.73	105.72	.752	.176	.237	1.52	3.116	8.7391	1.0572	30.47	.1620
3.	33	207.09	1.44	7.82	17	2.63	29.60	28.04	25	.389	.326	3.28	44.87	.759	.144	.299	1.48	2.575	7.6767	1.1216	12.93	.0624
3.	34	207.09	1.44	5.50	17	2.63	29.63	29.38	25	.796	.243	3.03	5.65	.759	.101	.319	1.46	1.811	6.0712	1.1752	1.63	.0100
3.	35	207.09	1.44	2.82	17	2.63	29.63	29.50	25	1.596	.332	3.24	7.34	.759	.052	.338	1.46	.929	3.8893	1.1800	2.12	.0052
3.	36	458.08	2.98	15.63	17	2.63	29.81	19.26	25	-.367	.688	1.23	299.89	.343	.949	.303	2.96	5.148	15.8709	.7704	86.42	.4220
3.	37	460.82	3.00	13.76	17	2.63	29.17	19.78	25	-.379	.695	1.42	303.42	.341	.844	.288	2.93	4.532	14.6073	.7912	87.44	.3756

SUBSET NO.	TEST NO.	LP	TP	INC. HS	W50	WR	INITIAL CRST HT	FINAL HT	WATER DEPTH	REL FREEBD	TRANS COEFF	FINAL QP	AREA OF DAMAGE	GODA ARGMENT	GODA STPNS	REF COEF	FINAL TRANS TP	HUDSON STAB NO	SPECTRAL STAB. NO	CRST HT RATIO	DLESS DAMAGE	DEL Hc/ ds
3.	38	430.49	2.81	10.98	17	2.63	29.44	22.34	25	-.242	.618	1.66	155.89	.365	.597	.319	2.80	3.616	12.2849	.8936	44.93	.2840
3.	39	432.08	2.82	7.49	17	2.63	29.26	25.88	25	.117	.521	1.85	67.73	.364	.410	.430	2.74	2.467	9.5314	1.0352	19.52	.1352
3.	40	427.28	2.79	3.68	17	2.63	29.84	28.53	25	.959	.343	2.49	23.23	.368	.198	.584	2.83	1.212	5.9127	1.1412	6.69	.0524
3.	41	337.58	2.23	13.38	17	2.63	29.29	21.73	25	-.244	.618	2.16	175.40	.465	.484	.311	2.21	4.407	12.9244	.8692	50.55	.3024
3.	42	344.06	2.27	11.17	17	2.63	29.38	24.02	25	-.088	.579	2.15	122.26	.457	.416	.331	2.21	3.679	11.5317	.9608	35.23	.2144
3.	43	340.50	2.25	8.35	17	2.63	29.81	26.00	25	.120	.503	2.70	77.76	.461	.306	.411	2.17	2.750	9.4655	1.0400	22.41	.1524
3.	44	346.96	2.29	5.72	17	2.63	29.29	27.98	25	.521	.363	3.01	34.00	.453	.216	.510	2.23	1.884	7.4018	1.1192	9.80	.0524
3.	45	345.69	2.28	2.89	17	2.63	29.44	29.29	25	1.484	.298	2.98	2.69	.454	.109	.532	2.25	.952	4.6896	1.1716	.78	.0060
3.	46	549.86	3.56	5.51	17	2.63	29.63	28.07	25	.557	.404	2.34	29.64	.286	.466	.597	3.48	1.815	8.4172	1.1228	8.54	.0624
3.	47	553.58	3.58	10.61	17	2.63	29.50	20.85	25	-.391	.732	1.56	244.61	.284	.908	.339	3.69	3.494	13.0572	.8340	70.49	.3460
3.	48	543.50	3.52	10.17	17	2.63	29.69	21.40	25	-.354	.678	1.56	208.01	.289	.841	.344	3.66	3.349	12.6162	.8560	59.95	.3316
3.	49	551.43	3.57	14.61	17	2.63	30.08	18.35	25	-.455	.704	1.62	341.42	.285	1.241	.328	3.66	4.812	16.1403	.7340	98.39	.4692
3.	50	556.75	3.60	15.82	17	2.63	29.38	17.37	25	-.482	.746	1.15	345.88	.282	1.368	.332	3.66	5.210	17.0741	.6948	99.68	.4804
3.	51	543.50	3.52	2.61	17	2.63	28.99	28.80	25	1.456	.346	2.46	5.39	.289	.216	.615	3.58	.860	5.0950	1.1520	1.55	.0076
3.	52	340.50	2.25	13.23	17	2.63	29.69	22.07	25	-.221	.622	2.15	177.54	.461	.485	.320	0	4.357	12.8645	.8828	51.16	.3048
3.	54	445.31	2.90	15.59	17	2.63	28.74	19.45	25	-.356	.694	1.10	258.27	.353	.900	.305	2.80	5.134	15.6952	.7780	74.43	.3716
3.	56	543.50	3.52	15.84	17	2.63	29.23	18.01	25	-.441	.718	1.27	332.96	.289	1.310	.317	3.80	5.217	16.9519	.7204	95.95	.4488
3.	67	205.41	1.43	10.42	17	2.63	29.14	24.11	25	-.085	.473	3.79	119.66	.765	.190	.242	1.53	1.347	.5901	.9644	3.80	10.4706
3.	68	210.67	1.46	11.06	17	2.63	29.02	25.91	25	.082	.481	4.18	92.53	.353	1.348	.248	1.52	1.351	.5959	1.0364	1.75	11.0976
3.	69	340.50	2.25	8.43	17	2.63	29.44	26.61	25	.191	.495	2.58	66.43	.533	.213	.410	2.24	1.337	.6814	1.0644	2.67	8.4293
3.	70	446.46	2.91	10.89	17	2.63	29.23	22.01	25	-.275	.624	1.36	166.39	-.594	-.161	.317	3.08	1.344	.7462	.8804	-2.96	10.8727
3.	71	554.61	3.59	2.59	17	2.63	28.86	28.10	25	1.197	.361	2.55	15.14	2.095	.005	.600	3.63	1.356	.8075	1.1240	10.35	2.5898
4.	53	.00	.00	.00	0	2	22.07	22.04	25	.000	.000	.00	1.67	#DIV/0!	#DIV/0!	.000	3.56	0	.0000		#VALUE	DIV/0!
4.	55	342.10	2.26	5.51	17	2.63	19.45	19.39	25	-1.018	.715	2.31	2.42	-2.830	-.016	.215	2.92	1.795	.9162	.7756	-10.74	5.5108
4.	57	339.21	2.24	2.72	17	2.63	18.01	17.98	25	-2.581	.779	2.65	1.02	-7.240	-.008	.180	2.23	1.898	.9657	.7192	-26.11	2.7199
4.	58	335.63	2.22	5.45	17	2.63	17.98	17.83	25	-1.316	.774	2.54	.74	-3.725	-.016	.210	2.23	1.900	.9640	.7132	-13.42	5.4483
4.	59	337.58	2.23	8.35	17	2.63	17.83	17.80	25	-.862	.733	2.22	.56	-2.429	-.026	.238	2.21	1.912	.9714	.7120	-8.70	8.3527
4.	60	337.58	2.23	11.18	17	2.63	17.80	17.86	25	-.639	.695	2.03	-.65	-1.800	-.039	.261	3.82	1.914	.9726	.7144	-6.44	11.1737
4.	61	337.58	2.23	13.27	17	2.63	17.86	18.01	25	-.527	.694	1.78	.19	-1.485	-.053	.272	2.09	1.909	.9702	.7204	-5.32	13.2638
4.	62	207.09	1.44	3.17	17	2.63	18.01	17.89	25	-2.243	.695	3.12	1.49	-9.787	-.015	.125	1.47	1.898	.8335	.7156	-35.29	3.1699
4.	63	207.09	1.44	5.56	17	2.63	17.89	17.74	25	-1.306	.748	3.28	.84	-5.698	-.027	.150	1.47	1.907	.8375	.7096	-20.46	5.5590
4.	64	207.09	1.44	7.99	17	2.63	17.74	17.68	25	-.916	.689	3.26	.19	-3.997	-.039	.178	1.47	1.919	.8427	.7072	-14.27	7.9913
4.	65	207.09	1.44	9.92	17	2.63	17.68	17.56	25	-.750	.656	4.29	.46	-3.272	-.048	.213	1.47	1.924	.8448	.7024	-11.66	9.9200
4.	66	210.67	1.46	11.19	17	2.63	17.56	17.71	25	-.651	.642	4.06	-1.11	-2.802	-.054	.229	1.48	1.933	.8529	.7084	-9.93	11.1982
5.	72	210.67	1.46	10.86	17	2.63	34.87	29.44	25	.409	.249	4.10	219.44	1.760	.062	.279	1.48	1.188	.5241	1.1776	9.86	10.8557
5.	73	208.77	1.45	9.38	17	2.63	34.38	32.19	25	.767	.190	3.48	113.16	3.324	.045	.272	1.48	1.200	.5281	1.2876	18.44	9.3742
5.	74	203.92	1.42	7.91	17	2.63	35.05	33.89	25	1.124	.153	3.50	138.70	4.973	.039	.298	1.48	1.184	.5174	1.3556	27.96	7.9093
5.	75	210.67	1.46	7.52	17	2.63	34.78	34.56	25	1.271	.138	3.38	83.43	5.470	.036	.289	1.47	1.190	.5251	1.3824	30.59	7.5216
5.	76	202.21	1.41	5.46	17	2.63	34.56	35.11	25	1.852	.138	3.13	23.88	8.253	.027	.285	1.46	1.195	.5213	1.4044	45.96	5.4590
5.	77	553.58	3.58	15.72	17	2.63	34.78	20.12	25	-.310	.629	1.45	644.19	-.544	-.233	.322	3.71	1.190	.7080	.8048	-3.04	15.7419

SUBSET NO.	TEST NO.	LP	TP	INC. HS	W50	WR	INITIAL CRST HT	FINAL HT	WATER DEPTH	REL FREEBD	TRANS COEFF	FINAL QP	AREA OF DAMAGE	GODA ARGMENT	GODA STPNS	REF COEF	FINAL TRANS TP	HUDSON STAB NO	SPECTRAL STAB. NO	CRST HT RATIO	DLESS DAMAGE	DEL Hc/ ds
5.	78	202.21	1.41	8.82	17	2.63	35.45	33.56	25	.971	.187	3.39	163.97	4.327	.044	.285	1.47	1.174	.5122	1.3424	24.51	8.8157
5.	79	207.09	1.44	2.75	17	2.63	35.27	35.39	25	3.778	.215	3.00	3.90	16.485	.013	.354	1.45	1.179	.5176	1.4156	93.05	2.7501
5.	80	344.06	2.27	12.96	17	2.63	36.06	24.20	25	-.062	.503	2.17	393.26	-.172	-7.674	.303	2.27	1.161	.5932	.9680	-.98	12.9032
5.	81	345.69	2.28	10.89	17	2.63	35.17	26.61	25	.148	.418	2.13	345.97	.408	.545	.335	2.10	1.181	.6045	1.0644	2.30	10.8784
5.	82	348.57	2.30	9.64	17	2.63	35.05	28.68	25	.382	.369	2.29	235.70	1.044	.058	.384	2.25	1.184	.6077	1.1472	5.87	9.6335
5.	83	348.57	2.30	6.79	17	2.63	35.27	33.59	25	1.265	.212	2.73	116.69	3.456	.020	.489	2.31	1.179	.6050	1.3436	19.51	6.7905
5.	84	348.57	2.30	4.03	17	2.63	34.41	34.56	25	2.372	.208	3.31	4.09	6.480	.012	.538	2.29	1.199	.6155	1.3824	35.98	4.0304
5.	85	460.82	3.00	15.34	17	2.63	34.99	21.61	25	-.221	.607	1.22	514.59	-.463	-.412	.312	2.98	1.185	.6647	.8644	-2.60	15.3394
5.	86	454.90	2.96	14.08	17	2.63	35.91	22.16	25	-.202	.595	1.33	538.37	-.429	-.468	.311	3	1.164	.6499	.8864	-2.45	14.0594
5.	87	438.91	2.86	12.75	17	2.63	35.54	23.04	25	-.154	.572	1.42	429.58	-.338	-.839	.314	3	1.172	.6472	.9216	-1.92	12.7273
5.	88	435.72	2.84	11.16	17	2.63	35.23	24.87	25	-.012	.258	1.44	324.14	-.027	********	.352	2.87	1.179	.6496	.9948	-.15	10.8333
5.	89	436.87	2.85	7.58	17	2.63	35.14	29.72	25	.623	.367	1.81	184.97	1.373	.026	.477	2.83	1.182	.6515	1.1888	7.73	7.5762
5.	90	425.69	2.78	3.78	17	2.63	35.36	35.36	25	2.741	.266	2.54	6.97	6.195	.009	.581	2.78	1.176	.6433	1.4144	35.03	3.7796
5.	91	549.86	3.56	14.29	17	2.63	35.17	21.03	25	-.278	.588	1.33	555.19	-.491	-.276	.364	3.69	1.181	.7013	.8412	-2.76	14.2806
5.	92	551.43	3.57	10.13	17	2.63	34.59	23.56	25	-.142	.578	1.64	389.08	-.250	-1.252	.383	3.56	1.195	.7101	.9424	-1.39	10.1408
5.	93	549.86	3.56	5.33	17	2.63	34.81	27.01	25	.377	.447	1.85	231.33	.665	.049	.524	3.53	1.189	.7063	1.0804	3.72	5.3316
5.	94	551.43	3.57	2.58	17	2.63	35.87	35.91	25	4.229	.299	1.97	5.02	7.443	.005	.648	3.53	1.165	.6924	1.4364	42.49	2.5798
5.	95	207.09	1.44	4.28	17	2.63	35.57	35.42	25	2.435	.158	3.35	8.36	10.625	.021	.341	1.46	1.172	.5146	1.4168	60.31	4.2793
5.	96	207.09	1.44	7.02	17	2.63	35.54	35.33	25	1.472	.135	3.30	56.86	6.423	.034	.288	1.47	1.172	.5149	1.4132	36.44	7.0177
5.	97	188.54	1.33	9.99	17	2.63	35.17	31.76	25	.677	.211	3.33	187.20	3.198	.054	.290	1.43	1.181	.5051	1.2704	18.02	9.9852
5.	98	208.77	1.45	11.35	17	2.63	34.96	29.54	25	.400	.290	4.30	231.05	1.733	.066	.297	1.45	1.186	.5220	1.1816	9.73	11.3500
5.	99	346.96	2.29	5.48	17	2.63	35.48	34.44	25	1.723	.188	2.76	51.56	4.727	.016	.509	2.16	1.174	.6017	1.3776	26.79	5.4788
5.	100	346.96	2.29	8.22	17	2.63	34.63	30.27	25	.641	.296	2.44	170.38	1.759	.028	.450	2.29	1.194	.6119	1.2108	9.81	8.2215
5.	101	345.69	2.28	11.03	17	2.63	35.30	27.34	25	.212	.422	2.07	293.85	.584	.220	.363	2.23	1.178	.6029	1.0936	3.30	11.0377
5.	102	346.96	2.29	6.91	17	2.63	35.91	31.33	25	.916	.195	2.63	131.36	2.513	.021	.490	2.29	1.164	.5966	1.2532	14.36	6.9105
5.	103	345.69	2.28	13.02	17	2.63	35.08	24.05	25	-.073	.528	1.70	329.25	-.201	-4.815	.318	2.09	1.183	.6055	.9620	-1.13	13.0137
5.	104	425.69	2.78	1.81	17	2.63	35.81	35.69	25	5.906	.346	2.57	5.67	13.348	.004	.590	2.78	1.166	.6377	1.4276	76.11	1.8100
5.	105	430.49	2.81	5.68	17	2.63	35.75	31.76	25	1.190	.286	2.39	146.60	2.661	.014	.521	2.83	1.168	.6407	1.2704	15.16	5.6807
5.	106	436.87	2.85	9.31	17	2.63	36.09	27.16	25	.232	.469	1.63	271.37	.511	.204	.413	2.81	1.160	.6396	1.0864	2.93	9.3103
5.	107	438.91	2.86	13.87	17	2.63	35.63	22.19	25	-.203	.564	1.32	502.70	-.446	-.431	.357	3	1.170	.6460	.8876	-2.53	13.8424
5.	108	446.46	2.91	15.61	17	2.63	34.93	21.28	25	-.238	.614	1.30	531.87	-.514	-.330	.335	3.31	1.187	.6588	.8512	-2.88	15.6303
5.	109	549.86	3.56	2.56	17	2.63	35.91	35.94	25	4.273	.310	1.91	2.69	7.542	.005	.624	3.32	1.164	.6912	1.4376	43.08	2.5603
5.	110	549.86	3.56	8.06	17	2.63	35.54	26.97	25	.244	.415	1.85	247.40	.431	.219	.526	0	1.172	.6962	1.0788	2.44	8.0738
5.	111	546.68	3.54	14.46	17	2.63	35.36	20.36	25	-.321	.622	1.34	588.17	-.570	-.193	.335	3.75	1.176	.6973	.8144	-3.22	14.4548
5.	112	553.58	3.58	15.99	17	2.63	36.03	19.78	25	-.326	.641	1.33	656.82	-.572	-.209	.327	3.56	1.161	.6909	.7912	-3.28	16.0123
6.	113	205.41	1.43	2.84	17	2.63	19.78	19.81	25	-1.827	.736	3.40	-.84	-8.028	-.014	.151	1.45	1.774	.7771	.7924	-30.82	2.8407
6.	114	205.41	1.43	9.04	17	2.63	19.81	19.60	25	-.597	.571	4.24	4.37	-2.623	-.045	.214	1.50	1.772	.7762	.7840	-10.08	9.0452
6.	115	200.50	1.40	5.59	17	2.63	19.60	19.63	25	-.961	.654	3.27	.37	-4.313	-.028	.159	1.39	1.785	.7767	.7852	-16.46	5.5879
6.	116	207.09	1.44	8.12	17	2.63	19.63	19.54	25	-.672	.586	3.92	2.14	-2.932	-.040	.201	1.50	1.783	.7832	.7816	-11.20	8.1250
6.	117	207.09	1.44	9.98	17	2.63	19.54	19.63	25	-.538	.560	4.39	.00	-2.347	-.051	.235	1.50	1.789	.7858	.7852	-8.94	9.9814

SUBSET NO.	TEST NO.	LP	TP	INC. HS	W50	WR	INITIAL CRST HT	FINAL HT	WATER DEPTH	REL FREEBD	TRANS COEFF	FINAL QP	AREA OF DAMAGE	GODA ARGMENT	GODA STPNS	REF COEF	FINAL TRANS TP	HUDSON STAB NO	SPECTRAL STAB. NO	CRST HT RATIO	DLESS DAMAGE	DEL Hc/ ds
6.	118	208.77	1.45	11.47	17	2.63	19.63	19.96	25	-.439	.534	4.81	4.74	-1.902	-.063	.245	1.49	1.783	.7850	.7984	-7.27	11.4806
6.	119	335.63	2.22	2.49	17	2.63	19.96	19.78	25	-2.096	.739	2.38	.37	-5.932	-.007	.185	2.23	1.762	.8940	.7912	-22.91	2.4905
6.	120	337.58	2.23	5.18	17	2.63	19.78	19.81	25	-1.002	.708	2.34	1.30	-2.823	-.016	.180	2.23	1.774	.9011	.7924	-10.84	5.1796
6.	121	335.63	2.22	7.96	17	2.63	19.81	19.81	25	-.652	.634	2.23	1.49	-1.845	-.028	.206	2.07	1.772	.8988	.7924	-7.09	7.9601
6.	122	340.50	2.25	10.66	17	2.63	19.81	19.96	25	-.473	.599	2.10	3.44	-1.321	-.048	.233	2.15	1.772	.9028	.7984	-5.08	10.6554
6.	123	337.58	2.23	12.88	17	2.63	19.96	19.75	25	-.408	.584	1.87	3.16	-1.150	-.070	.260	2.09	1.762	.8953	.7900	-4.44	12.8676
7.	124	208.77	1.45	11.44	71	2.83	31.46	31.21	25	.543	.369	4.01	42.46	2.353	.058	.354	1.53	5.202	1.4220	1.2484	11.72	11.4365
7.	125	208.77	1.45	10.02	71	2.83	31.70	31.55	25	.654	.313	3.74	26.57	2.834	.049	.335	1.53	5.175	1.4144	1.2620	14.19	10.0153
7.	126	207.09	1.44	8.03	71	2.83	31.36	31.58	25	.819	.262	3.26	20.07	3.574	.039	.352	1.49	5.214	1.4219	1.2632	17.76	8.0342
7.	127	205.41	1.43	5.60	71	2.83	31.70	31.61	25	1.180	.248	3.35	1.77	5.185	.027	.378	1.50	5.175	1.4079	1.2644	25.96	5.6017
7.	128	205.41	1.43	2.60	71	2.83	31.67	31.82	25	2.623	.328	3.30	3.53	11.525	.013	.430	1.46	5.178	1.4088	1.2728	57.66	2.6001
7.	129	336.60	2.23	13.03	71	2.83	31.67	31.67	25	.512	.430	1.93	37.44	1.445	.054	.455	2.08	5.178	1.6328	1.2668	7.23	13.0273
7.	130	348.57	2.30	11.11	71	2.83	32.34	31.97	25	.627	.387	2.06	18.95	1.713	.039	.471	2.15	5.103	1.6267	1.2788	8.69	11.1164
7.	131	345.69	2.28	8.63	71	2.83	31.91	31.67	25	.773	.351	2.32	24.90	2.130	.027	.508	2.09	5.151	1.6372	1.2668	10.71	8.6287
7.	132	345.69	2.28	5.58	71	2.83	32.06	31.73	25	1.206	.309	2.67	9.66	3.323	.016	.537	2.11	5.134	1.6318	1.2692	16.76	5.5804
7.	133	342.10	2.26	2.72	71	2.83	31.67	31.55	25	2.408	.368	3.09	1.39	6.695	.008	.570	2.13	5.178	1.6410	1.2620	33.49	2.7201
7.	134	467.66	3.04	15.66	71	2.83	31.64	29.72	25	.301	.566	1.29	93.74	.622	.198	.426	2.61	5.181	1.8127	1.1888	3.11	15.6811
7.	135	441.67	2.88	14.03	71	2.83	32.16	29.75	25	.339	.528	1.60	106.84	.740	.128	.409	2.28	5.123	1.7601	1.1900	3.74	14.0118
7.	136	427.28	2.79	11.17	71	2.83	32.52	30.54	25	.496	.483	1.75	45.24	1.117	.050	.449	2.95	5.083	1.7281	1.2216	5.69	11.1694
7.	137	432.08	2.82	7.42	71	2.83	31.67	31.54	25	.881	.398	1.98	7.90	1.963	.019	.502	2.67	5.178	1.7667	1.2616	9.82	7.4234
7.	138	425.69	2.78	3.55	71	2.83	31.30	31.12	25	1.724	.386	2.56	2.97	3.896	.008	.556	2.80	5.221	1.7728	1.2448	19.34	3.5499
7.	139	553.58	3.58	15.86	71	2.83	31.39	24.32	25	-.043	.653	1.61	258.36	-.075	-67.035	.409	3.65	5.210	1.9249	.9728	-.38	15.8140
7.	140	543.50	3.52	14.23	71	2.83	32.25	29.14	25	.291	.550	1.18	100.61	.519	.241	.466	3.51	5.113	1.8782	1.1656	2.63	14.2268
7.	141	548.81	3.55	10.38	71	2.83	31.39	30.21	25	.502	.476	1.64	50.45	.888	.053	.511	3.58	5.210	1.9195	1.2084	4.42	10.3785
7.	142	551.43	3.57	5.10	71	2.83	32.22	31.21	25	1.218	.368	2.34	18.21	2.144	.010	.586	3.56	5.116	1.8883	1.2484	10.85	5.0985
7.	143	556.75	3.60	2.35	71	2.83	31.67	31.85	25	2.915	.422	2.34	9.01	5.088	.004	.596	3.61	5.178	1.9165	1.2740	25.45	2.3499
7.	144	203.92	1.42	3.98	71	2.83	31.97	31.64	25	1.668	.280	3.26	1.11	7.381	.020	.382	1.50	5.144	1.3963	1.2656	37.16	3.9808
7.	145	198.59	1.39	6.74	71	2.83	31.85	31.79	25	1.007	.247	3.13	3.16	4.552	.034	.356	1.53	5.158	1.3900	1.2716	22.86	6.7428
7.	146	208.77	1.45	9.98	71	2.83	31.82	31.94	25	.695	.307	3.80	9.29	3.012	.049	.380	1.53	5.161	1.4107	1.2776	15.11	9.9856
7.	147	208.77	1.45	11.42	71	2.83	32.00	31.73	25	.589	.342	3.95	23.23	2.552	.057	.379	1.53	5.141	1.4051	1.2692	12.86	11.4261
7.	148	346.96	2.29	4.07	71	2.83	31.82	31.64	25	1.631	.322	2.95	4.46	4.475	.012	.554	2.12	5.161	1.6428	1.2656	22.46	4.0711
7.	149	346.96	2.29	7.07	71	2.83	31.61	31.46	25	.914	.333	2.56	15.33	2.508	.021	.526	2.09	5.185	1.6504	1.2584	12.53	7.0678
7.	150	342.10	2.26	11.32	71	2.83	31.49	31.36	25	.562	.400	2.12	23.04	1.562	.043	.482	2.11	5.199	1.6476	1.2544	7.79	11.3167
7.	151	337.58	2.23	13.11	71	2.83	31.61	30.51	25	.420	.442	1.89	40.88	1.183	.068	.461	2.12	5.185	1.6359	1.2204	5.91	13.1190
7.	152	425.69	2.78	1.65	71	2.83	31.70	31.79	25	4.115	.491	2.45	.93	9.300	.004	.593	2.78	5.175	1.7571	1.2716	46.56	1.6501
7.	153	427.28	2.79	5.66	71	2.83	31.82	31.03	25	1.065	.365	2.23	20.25	2.398	.014	.554	2.85	5.161	1.7546	1.2412	12.04	5.6620
7.	154	429.31	2.80	9.75	71	2.83	31.30	31.00	25	.615	.464	1.81	27.87	1.380	.033	.481	2.88	5.221	1.7771	1.2400	6.85	9.7561
7.	155	430.49	2.81	14.24	71	2.83	31.24	29.63	25	.325	.518	1.45	56.30	.727	.138	.423	2.63	5.228	1.7816	1.1852	3.60	14.2462
7.	156	441.67	2.88	15.42	71	2.83	32.13	28.59	25	.233	.583	1.28	106.65	.508	.339	.418	2.61	5.126	1.7613	1.1436	2.57	15.4077
7.	157	553.58	3.58	2.35	71	2.83	32.80	32.77	25	3.306	.431	2.26	1.30	5.802	.004	.588	3.56	5.053	1.8667	1.3108	29.72	2.3503

SUBSET NO.	TEST NO.	LP	TP	INC. HS	W50	WR	INITIAL CRST HT	FINAL HT	WATER DEPTH	REL FREEBD	TRANS COEFF	FINAL QP	AREA OF DAMAGE	GODA ARGMENT	GODA STPNS	REF COEF	FINAL TRANS TP	HUDSON STAB NO	SPECTRAL STAB. NO	CRST HT RATIO	DLESS DAMAGE	DEL Hc/ ds
7.	158	556.75	3.60	7.81	71	2.83	32.74	32.46	25	.955	.393	1.89	13.56	1.667	.017	.564	3.58	5.059	1.8725	1.2984	8.53	7.8115
7.	159	548.81	3.55	14.51	71	2.83	32.22	26.97	25	.136	.580	1.47	146.42	.241	2.007	.452	3.56	5.116	1.8848	1.0788	1.22	14.4853
7.	160	553.58	3.58	16.04	71	2.83	31.94	26.88	25	.117	.624	1.41	142.42	.205	3.489	.430	3.56	5.147	1.9016	1.0752	1.03	16.0684
7.	161	546.68	3.54	14.42	71	2.83	31.86	28.25	25	.225	.568	1.54	129.51	.399	.483	.471	3.56	5.156	1.8979	1.1300	2.01	14.4444
8.	162	205.41	1.43	1.09	71	2.83	28.25	28.38	25	3.101	.543	3.63	1.58	13.625	.005	.351	1.45	5.610	1.5264	1.1352	63.17	1.0900
8.	163	205.41	1.43	2.43	71	2.83	28.38	28.19	25	1.313	.410	3.19	.56	5.769	.012	.284	1.45	5.592	1.5215	1.1276	26.83	2.4296
8.	164	207.09	1.44	3.99	71	2.83	28.19	28.25	25	.815	.371	3.14	.00	3.556	.019	.247	1.44	5.618	1.5322	1.1300	16.46	3.9877
8.	165	207.09	1.44	5.38	71	2.83	28.25	28.22	25	.599	.371	3.20	.56	2.614	.027	.238	1.43	5.610	1.5299	1.1288	12.12	5.3756
8.	166	208.77	1.45	7.80	71	2.83	28.22	28.19	25	.409	.374	3.18	1.76	1.772	.044	.249	1.47	5.614	1.5346	1.1276	8.21	7.7995
8.	167	210.67	1.46	9.78	71	2.83	28.19	28.35	25	.343	.407	3.69	1.11	1.476	.064	.272	1.49	5.618	1.5393	1.1340	6.83	9.7668
8.	168	208.77	1.45	11.03	71	2.83	28.35	28.16	25	.286	.425	4.12	1.67	1.239	.087	.299	1.48	5.596	1.5297	1.1264	5.76	11.0490
8.	169	345.69	2.28	1.16	71	2.83	28.16	28.16	25	2.724	.603	3.16	1.76	7.507	.003	.483	2.27	5.623	1.7872	1.1264	34.73	1.1601
8.	170	344.06	2.27	2.55	71	2.83	28.16	28.25	25	1.275	.467	2.83	.65	3.529	.007	.454	2.27	5.623	1.7846	1.1300	16.33	2.5490
8.	171	342.10	2.26	3.94	71	2.83	28.25	28.19	25	.810	.447	2.85	.46	2.252	.012	.437	2.09	5.610	1.7779	1.1276	10.44	3.9383
8.	172	348.57	2.30	5.44	71	2.83	28.19	28.22	25	.592	.460	2.93	1.67	1.617	.020	.436	2.14	5.618	1.7911	1.1288	7.49	5.4392
8.	173	342.10	2.26	8.73	71	2.83	28.22	28.22	25	.369	.464	2.51	1.11	1.026	.055	.408	2.27	5.614	1.7793	1.1288	4.75	8.7263
8.	174	345.69	2.28	11.26	71	2.83	28.22	28.29	25	.292	.486	2.30	.19	.805	.110	.395	2.25	5.614	1.7845	1.1316	3.73	11.2671
8.	175	339.21	2.24	13.31	71	2.83	28.29	27.65	25	.199	.545	1.84	6.04	.558	.302	.391	2.17	5.605	1.7709	1.1060	2.59	13.3166
8.	176	425.69	2.78	1.62	71	2.83	27.65	27.61	25	1.611	.575	2.41	.65	3.641	.004	.493	2.68	5.695	1.9340	1.1044	16.64	1.6201
8.	177	429.31	2.80	3.55	71	2.83	27.61	27.58	25	.727	.466	2.21	-.65	1.631	.010	.463	2.83	5.701	1.9406	1.1032	7.45	3.5488
8.	178	429.31	2.80	5.60	71	2.83	27.58	27.61	25	.466	.480	2.02	1.11	1.046	.027	.440	2.83	5.706	1.9421	1.1044	4.77	5.6009
8.	179	434.11	2.83	7.59	71	2.83	27.61	27.65	25	.349	.521	2.29	3.16	.775	.064	.400	2.81	5.701	1.9475	1.1060	3.54	7.5931
8.	180	435.72	2.84	11.34	71	2.83	27.65	27.65	25	.234	.533	1.51	2.04	.518	.241	.393	2.64	5.695	1.9478	1.1060	2.37	11.3248
8.	181	429.31	2.80	14.16	71	2.83	27.65	28.01	25	.213	.575	1.46	.37	.478	.375	.391	2.60	5.695	1.9386	1.1204	2.18	14.1315
8.	182	267.38	1.80	13.32	71	2.83	28.01	27.55	25	.191	.533	2.20	2.60	.667	.252	.350	1.69	5.644	1.6580	1.1020	3.07	13.3508
8.	183	553.58	3.58	2.25	71	2.83	27.55	28.01	25	1.338	.509	2.16	.37	2.348	.004	.515	3.56	5.710	2.1095	1.1204	10.71	2.2496
8.	184	554.61	3.59	5.01	71	2.83	28.01	27.58	25	.515	.514	2.01	2.42	.901	.025	.493	3.61	5.644	2.0870	1.1032	4.16	5.0097
8.	185	549.86	3.56	7.50	71	2.83	27.58	27.68	25	.357	.551	1.84	1.11	.630	.078	.474	3.56	5.706	2.1040	1.1072	2.87	7.5070
8.	186	546.68	3.54	9.93	71	2.83	27.68	27.58	25	.260	.561	1.63	3.81	.461	.226	.457	3.56	5.691	2.0947	1.1032	2.11	9.9231
8.	187	546.68	3.54	12.26	71	2.83	27.58	27.49	25	.203	.566	1.52	2.60	.360	.544	.445	3.56	5.706	2.1000	1.0996	1.64	12.2660
9.	188	600.72	3.56	10.54	71	2.83	32.00	29.81	30	-.018	.679	1.59	47.66	-.032	********	.422	3.70	5.141	1.8956	.9937	-.16	10.5556
9.	189	220.97	1.43	5.76	71	2.83	31.82	31.79	30	.311	.569	3.44	8.55	1.366	.039	.256	1.50	5.161	1.4042	1.0597	6.86	5.7556
9.	190	234.11	1.50	10.94	71	2.83	31.55	31.70	30	.155	.584	4.13	7.62	.649	.251	.301	1.50	5.192	1.4353	1.0567	3.24	10.9677
9.	191	234.11	1.50	12.63	71	2.83	31.73	31.24	30	.098	.589	4.74	7.84	.411	.917	.285	1.50	5.171	1.4296	1.0413	2.06	12.6531
9.	192	361.49	2.20	5.80	71	2.83	31.58	31.52	30	.262	.554	2.47	1.39	.748	.063	.443	2.03	5.188	1.6296	1.0507	3.74	5.8015
9.	193	359.72	2.19	12.02	71	2.83	31.67	31.06	30	.088	.614	2.11	17.28	.252	2.211	.388	2.02	5.178	1.6239	1.0353	1.26	12.0455
9.	194	365.42	2.22	14.46	71	2.83	31.58	29.66	30	-.024	.630	1.87	42.55	-.068	********	.357	2.02	5.188	1.6345	.9887	-.34	14.1667
9.	195	501.11	2.99	8.20	71	2.83	32.00	31.76	30	.215	.639	1.96	7.25	.452	.216	.436	2.25	5.141	1.7885	1.0587	2.28	8.1860
9.	196	516.90	3.08	16.09	71	2.83	31.61	26.61	30	-.211	.705	1.54	156.26	-.430	-.466	.348	3.08	5.185	1.8218	.8870	-2.15	16.0664
9.	197	513.38	3.06	18.17	71	2.83	32.06	25.51	30	-.247	.725	1.42	191.29	-.507	-.346	.344	3.03	5.134	1.8000	.8503	-2.56	18.1781

SUBSET NO.	TEST NO.	LP	TP	INC. HS	W50	WR	INITIAL CRST HT	FINAL HT	WATER DEPTH	REL FREEBD	TRANS COEFF	FINAL QP	AREA OF DAMAGE	GODA ARGMENT	GODA STPNS	REF COEF	FINAL TRANS TP	HUDSON STAB NO	SPECTRAL STAB. NO	CRST HT RATIO	DLESS DAMAGE	DEL Hc/ ds
9.	198	568.16	3.37	5.22	71	2.83	32.13	32.06	30	.395	.568	2.25	3.16	.736	.037	.497	3.37	5.126	1.8560	1.0687	3.72	5.2152
9.	199	557.68	3.31	13.38	71	2.83	32.00	28.64	30	-.102	.700	1.52	99.31	-.194	-3.430	.405	3.52	5.141	1.8502	.9547	-.98	13.3333
9.	200	551.88	3.28	17.60	71	2.83	31.61	25.21	30	-.272	.736	.00	198.63	-.521	-.291	.362	3.53	5.185	1.8604	.8403	-2.60	17.6103
10.	201	363.27	2.21	2.58	71	2.83	25.21	25.21	30	-1.857	.864	.00	1.86	-5.280	-.007	.295	2.23	6.079	1.9124	.8403	-22.69	2.5794
10.	202	365.42	2.22	5.57	71	2.83	25.21	25.18	30	-.865	.775	.00	2.23	-2.448	-.016	.290	2.08	6.079	1.9152	.8393	-10.52	5.5723
10.	203	365.42	2.22	8.75	71	2.83	25.18	25.12	30	-.558	.728	.00	.06	-1.579	-.031	.286	1.68	6.085	1.9169	.8373	-6.78	8.7455
10.	204	365.42	2.22	12.25	71	2.83	25.12	24.96	30	-.411	.678	.00	2.97	-1.163	-.060	.299	1.60	6.095	1.9201	.8320	-4.99	12.2628
10.	205	365.42	2.22	14.41	71	2.83	24.96	25.02	30	-.346	.673	.00	2.23	-.979	-.092	.306	1.67	6.122	1.9288	.8340	-4.18	14.3931

Appendix C

Ahrens, J.P. Reef type breakwaters.

Proceedings of the 19th Coastal Engineering Conference. Houston, Texas. August 1984. pp. 2648-2662.

This paper discusses laboratory experiments conducted on low-crested/submerged, trapezoidal, homogeneous, rubblemound breakwaters. The study provides information on structure stability, wave transmission and reflection, and energy dissipation characteristics of both damaged and undamaged structures. Two different sizes of stone were tested using depth-limited wave spectra. In general, waves did not break on the structure. The peak energy density of the four spectra tested ranged from 1.45 to 3.60 sec. Stability is expressed in terms of relative crest height, dimensionless damage, which is a function of the area of damage, median stone weight, and stone density, and Hudson's stability number, N, modified according to Graveson et al. to include the effects of wave period. Structures showed little damage for N<=6, but noticeable damage for N>=8. Wave transmission was found to be primarily a function of relative freeboard and wave steepness expressed using significant wave height and wave length at the peak energy period. An estimate of energy dissipated as a function of relative freeboard is given.

BREAKWATER
SUBMERGED
EXPERIMENTAL
LABORATORY

STABILITY
PERFORMANCE

Allsop, N.W.H. Low-crest breakwater, studies in random waves.

Proceedings of Coastal Structures '83. Arlington, Virginia. March 1983. pp. 94-107.

Rate of wave overtopping and wave transmission were measured for several low-crested, nonhomogeneous, trapezoidal, rubblemound breakwaters. Tests were conducted with two different irregular wave regimes. The first had a constant mean wave period, with the wave height being increased for each successive part of a test. The second used a constant mean sea steepness (Hs/Lo). Spectra were of the general Moskowitz form. Stability was characterized using three damage categories based on the distance of movement of the center of an armor unit. Dimensionless damage was defined as the percent of units moving >1m but <2m relative to the total number of units. Stability curves were obtained by plotting this number against Hudson's stability number. The number of waves overtopping was assessed visually and expressed as a percentage of the total number of waves in a test. Transmitted waves were measured using a twin-wire probe located between the test section and an absorbing beach. For these tests, wave overtopping was best described by the dimensionless freeboard parameters- freeboard divided by depth and incident significant wave height, with no dependency on wave period or steepness. Conversely, transmitted waves were dependent on steepness. Total damage for Hs/Lo greater than 0.03 depended on stability number but not freeboard. For waves steeper than 0.03, percent damage increased for increasing freeboard.

BREAKWATER
EXPERIMENTAL
LABORATORY
PERFORMANCE
STABILITY
PERMEABLE
WAVE OVERTOPPING

Bade, P.
Kaldenhoff, H.

Energy transmission over breakwater- a design criterion?

Proceedings of the 17th Coastal Engineering Conference. Sydney, Australia. March 1980. pp. 1885-1897.

Laboratory tests were conducted with a permeable, trapezoidal breakwater with an armor layer of concrete cubes in order to investigate the characteristics of the incoming and transmitted wave spectra. The study stemmed from an earlier investigation of wave transmission across a proposed breakwater at Scharhoern/Neuwark, Germany. The first study did not address adequately the question of wave transmission, so the present study was undertaken to examine changes in spectral characteristics. The experimental flume was 4 m wide, 1.6 m deep, and 90 m long. Incoming waves were generated by the superposition of different wave packets. Water depths were varied from 90 to 110 percent of the structure height. Wave transmission was calculated three ways- using the average, significant, and highest 10 percent wave heights. In general, the transmission coefficient depends upon the distribution of incident wave heights and the peak period of the spectrum. It was also found that wave groups lead to higher transmission coefficients.

BREAKWATER
SUBMERGED
EXPERIMENTAL
LABORATORY
PERFORMANCE
PERMEABLE
SPECTRAL ANALYSIS
WAVE GROUPING

Black, K.P. Wave transformation over a coral reef (summary).

Look Lab Hawaii. Dept. of Ocean Engineering, Univ. of Hawaii, Honolulu, Hawaii. Vol. 7, No. 2. pp. 30-58.

Wave transformation across a coral reef was measured in the field along a 1650 ft, shore perpendicular transect on the south side of the island of Oahu, Hawaii from July to Sept. 1976. Data were analyzed with respect to wave height probability density function, spectral shape, statistical changes of wave height and period, and wave phase velocity in shallow water. Pertinent observations include:
1) Both wave height and period density functions were best described by a Weibull distribution; the correlation coefficient exceeded 0.98 in nearly all cases;
2) The significant wave period decreased as the wave traveled across the reef due to the introduction of harmonics and decreasing correlation between wave height and period;
3) Waves in shallow water were non-dispersive below about 0.25 Hz;
4) The probability of the occurrence of wave groups decreased as the waves entered shallow water; and
5) Maximum correlation between wave height and period occurred at the offshore station (outside the reef); minimum correlation occurred in shallow water with onshore winds.

REEFS
EXPERIMENTAL
FIELD
PERFORMANCE

WAVE TRANSFORMATIONS

Bremner, W.
Foster, D.N.
Miller, C.A.
Wallace, B.C.

The design concept of dual breakwaters and its application to Townsville, Australia.

Proceedings of the 17th Coastal Engineering Conference. Sydney, Australia. March 1980. Vol. 2. pp. 1898-1908.

In this paper the concept of using an offshore breakwater which is designed to fail under extreme wave conditions to protect an inner breakwater or revetment is examined and the results applied to Townsville Harbour where cost savings on the order of 40 percent were achieved over a conventional design. The design concept arose from studies and observations of the damage to the Rosslyn Bay breakwater over which waves broke. The structure provided adequate protection to the harbor during the 2 1/2 years after failure and prior to reconstruction. Breakwater failure was closely simulated by model tests. Tests were conducted for both constant and tidally varying water levels. In general, the mode of failure was similar in both types of tests, but more damage was observed when tidal influence was included. Transmitted wave height was not very dependent upon the amount of structural damage. Apparently, the increased energy transmission resulting from a lower crest is balanced by the increased energy dissipation resulting from the widened crest. This provides a considerable degree of safety against the design conditions being exceeded. Three dimensional tests indicated that wave diffraction around the ends of the structure had little influence on the transmitted wave heights.

BREAKWATER
SUBMERGED
EXPERIMENTAL
FIELD

STABILITY
PERFORMANCE

Bruun, P.
Johannesson, P.

Parameters affecting stability of rubble mounds.

ASCE Journal of the Waterways, Harbors, and Coastal Engineering Division. Vol. 102, No. WW2. May 1976. pp. 141-164.

A synthesis of existing information on the parameters affecting the stability of rubble-mounds is given, particularly with respect to forces by uprush and downrush, permeability, resonance, friction between armor blocks and between armor and sublayer, and slope geometry. This paper is a condensed version of a comprehensive report published by the Norwegian Institute of Technology (Institute Report RE3, 1974). Based on the available data and information, the following design principles are given:
1) Streamline the exposed portion of the armor units to avoid high inertia and drag forces;
2) Minimize downrush velocities to reduce uplift forces. Uprush should therefore be reduced or retained;
3) Avoid resonance between incident wave period and downrush period. At resonance downrush is at its lowest position when the wave breaks on the structure, thus exposing the breakwater to maximum forces. In addition the uplift forces (hydrostatic pressure inside mound) are greatest when downrush is at a minimum;
4) Construct with highest practical permeability to decrease the buildup of internal hydrostatic forces;
5) Maximize friction between armor blocks and between these blocks and the sublayer. Friction increases in importance as the thickness of the armor layer increases; and
6) Use an S-geometry for breakwater profile so that the slope is minimum where the destructive forces are maximum. This is particularly important at sites where the tidal range is low.

BREAKWATER
THEORETICAL
DATA SUMMARY
STABILITY
PERFORMANCE

Calhoun, R.J. Field study of wave transmission through a rubble-mound breakwater.

Master Thesis. United States Naval Postgraduate School. March 1971. 88 pp.

Characteristics of sea and swell incident at a permeable rubblemound breakwater located in Monterey Harbor, California, are resolved into reflected and transmitted wave components. The wave characteristics are studied by analyzing synchronized wave records of three underwater sensors, two placed seaward and one placed landward of the breakwater. Energy spectra and cross spectra are calculated for various characteristic sea states selected from three months of observations. Amplitude and phase are determined for the spectral wave components comprising the partial standig wave phenomena. This study was unique in that it entails experiments conducted in the field on a prototype structure in the natural environment. Transmission has been studied heretofore exclusively with scale models. The coefficients of transmission are considerably less for the prototype than predicted by a Corps of Engineers model study of the Monterey Breakwater. The differences are apparently related to more wave energy scattering and dissipation due to turbulence than predicted by the model.

BREAKWATER
EXPERIMENTAL
FIELD
PERFORMANCE
PERMEABLE

Carver, R.D.
Markle, D.G.

Rubble-mound breakwater stability and wave-attenuation tests, Port Ontario Harbor, New York.

U.S. Army Corps of Engineers Waterways Experiment Station. Tech. Rpt. HL-81-5. April 1981. 13 pp. + figures.

An undistorted-scale hydraulic model study was conducted to investigate the armor stability and wave-transmission design of three rubble-mound breakwater cross sections for Port Ontario Harbor, New York. Plan 1 was constructed to a crown elevation of +10 ft lwd and used armor slopes of 1V-on-2H and 1V-on-1.5H lake-side and harbor-side, respectively. A crown width of 16 ft, equivalent to three armor-stone diameters, was used and the slopes and crown were armored with two thicknesses of 7.8-ton stone. Plan 1A was the same as Plan 1 except that the crown elevation was lowered to +9 ft lwd. Plan 2 was similar to Plan 1 except that the armor weight was reduced to 5.3 tons and the crown width was narrowed to 14 ft. Stability was tested using regular waves and a nine-step storm surge hydrograph. Water level was varied from +1.0 to +4.6 and back to +1.0 ft lwd. Wave periods used were 7-, 9- and 11-sec and wave heights corresponded to the most severe breaking wave height that could be produced for the given wave period and water depth. Wave steepness, H/T^2, ranged from 0.1020 to 0.1612. Stability was assessed qualitatively based on the degree of displacement and the stability of the final configuration. Transmitted waves were measured at one wavelength and one-half wavelength behind the centerline of the breakwater for all wave periods and water elevation of +4.6 ft lwd. Based on results of model tests, it was concluded that Plans 1 and 2 meet the designated wave-transmission criteria of significant transmitted wave height <=3 ft and are stable designs for the maximum wave heights for 7- and 11-sec waves at swl's of +1.0 and +4.6 ft lwd. Plan 1 was the most stable of all plans. Maximum significant transmitted wave heights were 2.5, 3.0 and 2.4 ft for Plans 1, 1A, and 2, respectively.

BREAKWATER
EXPERIMENTAL
LABORATORY
STABILITY
PERFORMANCE
PERMEABLE

Civil Engineering (London)

Submerged breakwaters.

Civil Engineering (London). May 1980. pp. 34-39.

See Dattatri et al. 1978. Proceedings of the 16th International Coastal Engineering Conference.

Cross, R.H.
Sollitt, C.K.

Wave transmission by overtopping.

Massachusetts Institute of Technology.
Technical Note No.15. July 1971. 37 pp.

See Cross and Sollitt. 1972. ASCE Waterways Journal, Vol. 98. pp. 295-309.

Cross, R.H.
Sollitt, C.K.

Wave transmission by overtopping.

ASCE Journal of the Waterways, Harbors, and Coastal Engineering Division. Vol. 98, No. WW3. August 1972. pp. 295-309.

This paper outlines a theoretical method for predicting wave transmission across impermeable breakwaters based on the energy content of the overtopping water. Theoretical results are compared to experiments. The energy content and volume of the overtopping water is assumed to be the same as those of the portion of the runup wedge that would exeed the breakwater crest elevation were the structure face extended such that no overtopping could occur. The total runup wedge extends from the trough of the first seaward wave to the point of maximum runup and is defined by an nth degree parabola. An analytical solution for the transmission coefficient is given based on conservation of energy over one wave period. For a control volume on the seaward face and enclosing the entire runup wedge, the energy provided by the incident wave must equal the sum of the energy of the reflected wave, overtopping water and losses resulting from friction and regeneration. For a control volume on the landward side, the overtopping energy divided by the wave period gives the incoming energy flux. This must equal the sum of the rundown losses and the outgoing energy flux given by the transmitted wave. The fundamental mode of the transmitted wave is the same as the incident wave. Previous studies at MIT indicate that most of the energy is contained at this mode, so as a first approximation higher harmonics are neglected and energy transmitted is calculated based on the incident wave period. Lab experiments were conducted with regular waves for a range of relative depth, d/L, of 0.16 to 0.45 and wave steepness, H/L, 0.064. Both smooth and rough slopes of 1:1.5 were tested. The comparison of lab results to analytical solutions requires the assumption of reflection and loss coefficients and of a runup ratio (runup above still water divided by wave height).

Cross, R.H.
Sollitt, C.K.

Wave transmission by overtopping (cont).

The assumed runup ratio was based on other studies by the authors. Reflection and loss coefficients were based on the work of Miche and Sy, respectively. Data show a considerable amount of scatter when transmission coefficient is plotted against relative freeboard. This is attributed to wave gage inaccuracies; non-uniformity of wave crests across the channel; and breaking of waves on the structure. The data are not separated according to wave steepness. This might reduce the scatter. Another problem with the data is that they do not cover the full range of expected prototype conditions. The general results indicate that the theory tends to overpredict the transmission coefficient ,K, for values of relative freeboard, F, greater than 0.5 and underestimates K for F<0.3. An envelope curve representing an upper bound on K is proposed.

BREAKWATER
THEORETICAL
EXPERIMENTAL
LABORATORY

PERFORMANCE
IMPERMEABLE
WAVE OVERTOPPING

Dai, Y.B.
Jackson, R.A.

Designs for rubble-mound breakwaters, Dana Point Harbor, California.

U.S. Army Corps of Engineers Waterways Experiment Station. Technical Report 2-725. 1966. 23 pp. + figures.

A small-craft harbor, protected by two breakwaters separated by a navigation opening and containing a mole paralleling one breakwater, is proposed for Dana Point, California. Tests were conducted on two-dimensional, 1:50- and 1:100-scale models of rubble-mound breakwater sections constructed of quarrystone to 1) obtain data for design of a stable rubble-mound breakwater which will allow a minimum of wave energy to pass through and over the structure into the protected area of the marina; 2) obtain design data for wave absorbers on the fairway side of the interior mole; and 3) determine the relation between wave transmission in a 1:50-scale model and in a 1:100-scale model. This relation was to be used in evaluating data obtained in a three-dimensional, 1:100-scale model of the entire harbor.

The initial breakwater plan developed by the Los Angeles District (LAD) was not adequate to withstand 12- and 18-sec waves 16 ft high when the cover stones were random-placed. Stability was improved slightly when the stone was placed in accordance with a special technique developed by LAD; and a modification of the original design was developed by LAD which was stable for the largest breaking waves which could reach the structure.

A mole-revetment and breakwater combination that satisfied the selected wave-height criterion for the fairway between the mole and breakwater was developed. The crown elevation of the breakwater was +18 ft mllw. The wave absorber on the mole extended from +12 to -8 ft mllw and had a 25-ft wide berm elevation at +4 ft mllw.

The 1:50 model produced higher transmitted waves than the 1:100 model, so 1:100 scale sections that would reproduce characteristics of the 1:50 sections were used in the three-dimensional harbor tests.

BREAKWATER
EXPERIMENTAL
LABORATORY
MODEL
STABILITY
PERFORMANCE
PERMEABLE

Dattatri, J.
Raman, H.
Shankar, N.J.

Performance characteristics of submerged breakwaters.

Proceedings of the 16th Coastal Engineering Conference. Hamburg, Germany.
August/September 1978. pp. 2153-2171.

This paper presents the results of laboratory investigations of energy transmission of regular waves across both porous and non-porous submerged breakwaters. Shapes of the impermeable structures tested include a thin vertical wall, triangular, rectangular, trapezoidal with a sloping face on both the seaward and landward sides, and trapezoidal with a sloping face on the seaward side and vertical face on the beach side. A fixed horizontal plate, rectangular, and trapezoidal with front and back sloping faces were used for the tests on permeable structures. The influence of incident wave steepness, relative water depth, relative crest width, relative depth of crest submergence, structure shape and slope, and porosity on the wave transmission coefficient is discussed. Crest width and depth of submergence are found to be the most important parameters with respect to the amount of energy transferred. In addition, transmitted waves were found to contain higher order harmonics, the generation of which is more pronounced when the crest of the breakwater is close to the still water level.

BREAKWATER
SUBMERGED
EXPERIMENTAL
LABORATORY

PERFORMANCE
PERMEABLE
IMPERMEABLE

Davidson, D.D.

Stability and transmission tests of tribar breakwater section proposed for Monterey Harbor, California.

U.S. Army Corps of Engineers Waterways Experiment Station. Technical Report H-69-11. 1969.

Tests were conducted on a 1:10-scale model of a twin-pontoon floating breakwater, proposed for construction at Oak Harbor, Washington, to determine its effectiveness in attenuating wave action and to determine the magnitude of the mooring forces. Chain- and pile-type mooring systems were tested in conjunction with the proposed floating breakwater to determine:
1) the effectiveness of the proposed structure in reducing the existing wave heights;
2) the mooring forces for both the chain- and pile-type mooring systems;
3) whether the proposed breakwater and mooring system would oscillate in resonance with the existing wave conditions; and
4) the natural period of oscillation of the proposed breakwater, while unrestrained in still water.
Based on test results, the following conclusions were made:
1) The breakwater proposed by the Seattle District will reduce 2 ft, 2.5 sec incident waves to 0.5 ft using either type of proposed mooring system;
2) The effectiveness of the proposed breakwater in attenuating wave energy is more a function of wave period than water depth for both mooring systems tested;
3) No consistent differences in wave attenuation were observed for the two different mooring systems;
4) Mooring forces were considerably less for the chain mooring system than for the pile system;
5) No resonant condition of oscillation was noted for the chain system, but may have been a factor for 3-sec waves with a pile system.
6) The natural period of oscillation for the prototype breakwater was found to be 7.1 sec.

BREAKWATER
FLOATING
EXPERIMENTAL
LABORATORY

MODEL
PERFORMANCE

De St. Q. Isaacson, M.

Wave dampening due to rubblemound breakwaters.

ASCE Journal of the Waterway, Port, Coastal, and Ocean Division. Vol. 104, No. 4. November 1978. pp. 391-405.

Laboratory tests were conducted to investigate the decay of wave height for waves propagating along a straight channel bounded by a pair of parallel rubblemound breakwaters. Structure slopes of 1:1.5, 1:2, and 1:3 were tested. Water depth, d, ranged from 20 to 40 cm. Wave periods, T, were selected from the range 0.4 to 1.5 sec such that the depth parameter, d/gT^2 vared from 0.01 to 0.10 in increments of 0.01. The ratio of w/d, where w was the distance between rubble mounds measured at still water level, ranged from 3.6 to greater than 6.6. The attenuation rate is derived analytically based on the assumption that the change in wave height with distance traveled, dH/dx, is proportional to H raised to some power n. For turbulent damping, n=2. An attenuation coefficient, A, given as a function of dH/dx, $H^{(-n)}$, d, and, w is defined. A, calculated assuming turbulent damping, is plotted as a function of d/gT^2 for the three different structure slopes. A modified A that contains a term to account for slope, is also plotted as a function of d/gT^2 with good results. Because the experimental data may be expressed in terms of a single function of d/gT^2, the results provide a particularly simple design procedure for the problem at hand. The effect of size and shape of breakwater units requires further investigation, and in this respect the proposed procedure remains tentative.

BREAKWATER
EXPERIMENTAL
LABORATORY
PERFORMANCE
PERMEABLE
OBLIQUE WAVES

Dean, W.R. On the reflexion of surface waves by a submerged plane barrier.

Proceedings of the Cambridge Philosophical Society. Vol. 41. March 1945. pp. 231-238.

An analytical solution for the reflection and transmission of periodic waves from a thin, vertical, submerged barrier extending from the horizontal bottom to a distance, A, below the surface is presented. Flow is assumed to be governed by the Laplace and unsteady Bernoulli equations. The theory does not account for any dissipative losses, so that the sum of the squares of the reflection and transmission coefficients is 1.0. Theoretical curves of coefficients as a function of M, where M is A times the frequency squared divided by the gravitational constant, are given, but no experimental data are shown for comparison. To achieve a 75 percent reduction in reflected energy, M must equal 0.5; or A=(g*T^2)/80. Smaller values of A will give a higher reflection coefficient.

BREAKWATER
SUBMERGED
THEORETICAL
PERFORMANCE

IMPERMEABLE

Dick, T.M.
Brebner, A.

Solid and permeable submerged breakwaters.

Proceedings of the 11th Coastal Engineering Conference. London, England. September 1968. pp. 1141-1158.

The behavior of thin and rectangular solid submerged breakwaters is re-examined. Dean's theory (Dean 1945) for reflection and transmission from a thin barrier in infinitely deep water is found to be correct. Empirical and theoretical relationships for the reflection coefficient of a thin breakwater as a function of the wave number times the water depth is proposed. Other general conclusions include:

1) Rectangular solid and permeable breakwaters have maximum reflection when the incident wave has the same period as the standing wave on the structure crest;
2) A submerged permeable breakwater with a depth of submergence greater than 5 percent of the total depth transmits less wave energy than the solid over a certain frequency range;
3) Both permeable and solid rectangular breakwaters cause a substantial loss in wave energy, and at least 50 percent of the incident energy is lost to turbulence; and
4) Thirty-six to 64 percent of the energy transmitted is transferred to frequencies higher than that of the incident wave.

BREAKWATER
SUBMERGED
EXPERIMENATAL
LABORATORY
PERFORMANCE
PERMEABLE
IMPERMEABLE

Diskin, M.H.
Vajda, M.L.
Amir, I.

Piling-up behind low and submerged permeable breakwaters.

ASCE Journal of the Waterways and Harbors Division. Vol. 96, No. WW2. May 1970. pp. 359-372.

Laboratory measurements of the difference in water levels on the seaward and shoreward sides of a permeable, trapezoidal, low or submerged breakwater using regular waves are discussed and an empirical equation relating the height of pile-up to the wave parameters is proposed. Piling up behind breakwaters occurs particularly when and area is completely enclosed by a breakwater or when the breakwater is parallel to shore and very long such that the water flow out of the protected zone is maintained by a difference in head on the lee and seaward sides of the breakwater. In the laboratory tests the maximum value of pile-up occurred for breakwaters with a freeboard equal to 50 to 90 percent of the incident wave height. Pile-up decreased gradually to 10 percent of the wave height as freeboard increased to twice the wave height. For a breakwater with a crest at still water level, the set-up decreased to about 35 percent of the wave height. Pile-up for structures submerged to one-half the wave height and more than one wave height were about 15 and 5 percent, respectively.

BREAKWATER
SUBMERGED
EXPERIMENTAL
LABORATORY

PERMEABLE
WATER LEVEL SET-UP

Finnigan, T.D.
Yamamoto, T.

Analysis of semi-submerged porous breakwaters.

Civil Engineering in the Oceans/4.
San Francisco, California. September 1979.
pp. 380-397.

A numerical model is developed for the study of floating (semi-submerged) porous and solid breakwaters. The influence of viscous damping and porosity on wave transmission characteristics across porous structures is investigated. The model assumes incompressible, irrotational flow and small amplitude waves, implying that Laplace's equation for velocity potential is valid everywhere. The effect of porosity and added mass of the porous medium are included by means of a virtual porosity term. Viscous damping is assumed to be linearly related to velocity by a proportionality constant. Computations are done for rectangular breakwaters, both solid and permeable, with breakwater width to water depth and submerged depth to water depth ratios of 2.0 and 0.25, respectively. The viscous damping term, D, and virtual porosity, V, are varied independently. For D=1.0, V varies from 0.5 to 0.9. D varies from 0.5 to 1.5 for V= 0.75. Model results indicate that 1) the reflection coefficient for porous breakwaters is substantially lower than for solid breakwaters and the transmission coefficient is slightly greater; 2) an appreciable amount of energy is dissipated by porous structures; 3) energy loss increases with increasing V and D; 4) reflection coefficient decreases substantially for increasing V and increases slightly for increasing D; 5) transmission coefficient decreases substantially for increasing D, but only varies about 5% over the range of V tested; and 6) uplift forces decrease as V and D increase.

BREAKWATER
FLOATING
NUMERICAL
PERFORMANCE
PERMEABLE
IMPERMEABLE

Foster, D.N.
Haradasa, D.

Rosslyn Bay Boat Harbour breakwater model studies.

University of New South Wales Water Research Laboratory. Technical Report No. 77/6. April 1977. 12 pp. + figures and tables.

A 1:27.59 linear scale model reproduction of storm conditions that caused the failure of the breakwater at Rosslyn Bay, Queensland, and 1:30 scale tests on the proposed modified design are discussed. The original structure was severely damaged in early 1976 by the combination of wave and surge conditions created by Cyclone David. The purpose of the laboratory storm simulation was twofold: to determine the sequence of structural failure and to compare damage of the scale model by regular waves to the actual damage observed in the prototype. The wave height and period (H and T) were assumed to correspond to the significant wave height and peak period of the measured spectra. A Froudian time scale was used. H ranged from 1.29 to 3.84 m and T from 6.2 to 9.6 sec. Tidal range was about 1 m to 5 m above low water datum (LWD). The structure crest was at 7.32 m LWD. It was observed that damage in the model was initiated earlier than in the prototype, and once initiated progressed at a slower rate. The earlier onset of damage, and perhaps the difference in rate of damage, is explained by the fact that the prototype armor has settled and become more strongly interlocked than in the model. The equilibrium profiles in the model were very similar to the final profiles in the prototype, but it is not known for certain if the prototype had reached equilibrium. Failure in both model and prototype resulted from waves overtopping the crest and moving stone from the crest to the harbor side. The original design was modified by flattening the seaward and leeward slopes; increasing the armor unit weight, particularly on the crest and harbor side slope; and extending the filter from the front slope, across the crest to the toe of the back slope. No change in crest elevation was suggested. The proposed structure was tested for

Foster, D. N.
Haradasa, D.

Rosslyn Bay Boat Harbour breakwater model studies (cont).

a design wave of 4.8 m and T= 9 to 16 sec. Maximum simulated tide ranged from 3 m to 8.6 m LWD. Maximum damage occurred at surge levels between 5 and 7.5 m LWD. Wave forces for higher surges are reduced by the layer of water over the crest. Because the crest is the weakest zone during overtopping, it is recommended that the largest armor units be placed on the crest and particular care be taken to achieve the best possible interlocking. Also, damage can be minimized by orienting the long axis of the stone so it is perpendicular to the wave direction.

BREAKWATER
EXPERIMENTAL
LABORATORY
FIELD

STABILITY
PERMEABLE

Foster, D. N.
Khan, S. P.

Stability of overtopped rock armoured breakwaters.

University of New South Wales Water Research Laboratory. Report No. 161. February 1984. 38 pp. + figures.

Parameters governing the stability of overtopped breakwaters are examined. A review of other researches and existing design practices is presented, including a discussion of rock stability under steady flow. Several types of laboratory experiments on two-layer rock structures were conducted in the Water Research Laboratory's 0.9 m wide by 1.5 m deep by 35 m long wave flume. Stability of the structures under steady flow was assessed by establishing flow over the crest and lowering the elevation of the tailwater until failure occurred. The flow rate, Q, was varied from 250 to 600 cubic meters per second per meter of width, and the upstream head from about 5.5 to 12 cm. Failure was usually sudden rather than progressive, and its onset appeared to be determined by the head difference across the structure. This result suggest that breakwater stability will be a function of the parameters that govern overtopping, ie; incident wave height and period (Hi and T), structure and offshore slopes, and wave runup (RU). Next an investigation of RU, wave rundown (RD), and percent damage to a non-overtopped breakwater was undertaken. Structure slope was 1:1.5, T= 1.70 sec, and Hi= 10.4 to 13.5 cm. Within the limits of experimental accuracy, the agreement with results of other researchers is judged to be reasonable. Two types of overtopping test were done. In the first series, structure height (h), and T remained constant at 33.5 cm and 1.70 sec, respectively. For a fixed water depth (d), the structure was subjected to attack by a periodic wave train, the height of which was increased incrementally until failure occurred. Water depth was varied from 21.2 to 38.7 cm. Hi, transmitted wave height (Ht), RD, level of overtopping (OT), and average Q were measured. Degree and location of damage were noted. Hi vs. (d-h)

Foster, D. N.
Khan, S. P.

Stability of overtopped rock armoured breakwaters (cont).

is plotted, and the relative damage corresponding to each data point is shown. In the second series, the wave paddle setting remained constant, but d was increased or decreased very slowly. Values of T= 1.34, 2.20, and 2.69 sec were tested. Hi, Ht, (d-h), OT, and degree of damage were measured at different points in time. Plots of Hi, Ht, (d-h), and OT vs. time are presented. Qualitative notations about the progression of damage are included. Hi vs. (d-h) is also shown. Some of the observations of the researchers include:
1) Interaction between parameters governing stability is more complex for overtopped breakwaters than for non-overtopped breakwaters;
2) Dependence of failure on tailwater levels and head difference across the structure supports observations that long period swell is more damaging than shorter period waves;
3) Rock size over the crest may have to be increased when overtopping is allowed; and
4) Rigorous model testing with a full range of incident wave conditions remains the best way to determine behavior of overtopped structures.

BREAKWATER
EXPERIMENTAL
LABORATORY
STABILITY

PERMEABLE
WAVE OVERTOPPING

Giles, M.L.
Sorensen, R.M.

Determination of mooring loads and wave transmission for a floating tire breakwater.

Coastal Structures '79. Alexandria, Virginia. March 1979. pp. 1069-1086.

Prototype scale laboratory tests to determine the mooring load and wave transmission characteristics of a floating tire breakwater (FTB) were conducted in the US Army Coastal Engineering Research Center's large wave tank. FTB's are typically constructed of scrap automobile or truck tires filled with plastic foam or some other bouyant material, and connected to form modules of various configurations. Eighteen-tire modules, in an arrangement proposed by the Goodyear Tire and Rubber Co., were connected to form breakwaters 4 and 6 (8.5 and 12.8 m) wide in the direction of wave advance. These breakwaters were tested in water depths of 2 and 4 m using monochromatic waves with periods ranging from 2.64 to 8.25 sec and wave heights up to 1.4 m. Test results indicate that the wave transmission coefficient decreases as the ratio of breakwater width to wavelength, W/L, increases, and increases as the incident wave height, H, increases. The mooring forces are mainly a function of H with only a slight dependence of L and W. Recommended design curves for the wave transmission coefficient as a function of W/L and mooring load as a function of H are presented.

BREAKWATER
FLOATING
EXPERIMENTAL
LABORATORY
PERFORMANCE

Goda, Y.
Suzuki, Y.

Estimation of incident and reflected waves in random wave experiments.

Proceedings of the 15th Coastal Engineering Conference. Honolulu, Hawaii. July 1976. pp. 828-845.

A technique to resolve the incident and reflected waves from the records of composite waves is presented. It is applicable to both regular and irregular trains of waves. Two simultaneous wave records are taken at adjacent locations, and all the amplitudes of Fourier components are analyzed by the FFT technique. The amplitudes of incident and reflected wave components are estimated from the Fourier components, and the incident and reflected wave spectra are constructed by smoothing the estimated periodograms. The wave resolution is effective in the range outside the condition of the gauge spacing being even integer of half wavelength. The ratio of incident and reflected wave energies in the effective resolution range is employed in estimating the overall reflection coefficient. The incident and reflected wave heights are estimated from the composite wave heights by consideration of total energy in the incident and reflected spectra. Laboratory test were conducted with both regular and irregular waves in order to verify the theory. Inaccuracies are attributed to nonlinear effects, transversal waves and other disturbances in the flume, and signal noise.

WAVE RESOLUTION
THEORETICAL
WAVE REFLECTION

Graveson, H.
Jensen, O.J.
Sorensen, T.

Stability of rubble mound breakwaters II.

Danish Hydraulic Institute, Technical University of Denmark. 1980.

A summary of data collected by the Danish Hydraulic Institute (DHI) on the stability of surface-piercing rubble mound breakwaters is given. Topics discussed include:
1) Specification of design criteria;
2) Effect of wave friction and internal friction on structure stability;
3) Design of filter layers; and
4) Effect of oblique incident waves.
It is demonstrated using four wave height exceedance curves, each with a different shape but the same 100-yr wave height, that long term accumulated damage is sometimes more critical than the single extreme event for determining overall structure stability. Also, a method for evaluating the risk of local damage and its relationship to total damage is discussed. The relative merits of the Hudson and Iribarren stability formulas are considered, and based on observations at DHI indicating that the stability coefficient is proportional to wave steepness, the Iribarren formula is modified to include wave period effects. Iribarren's formula requires knowledge of the natural angle of repose of the armor units, so this angle is determined for several natural and artificial materials. Experimental data showing that a thick permeable filter layer gives a more stable structure than a thin, impermeable layer are presented. Finally, stability of the breakwater's seaward and back slopes under the attack of obliquely incident waves is discussed. Qualitative front slope damage development patterns are given for dolosse and quarry stone. It is shown that the required freeboard for the same level of rear slope damage and the same overtopping volume is 70 percent less for a 70 to 80 deg angle of incidence than for 0 deg (normal) incidence. Thus, rear slope stability is strongly dependent on overtopping volume.

BREAKWATER
THEORETICAL
DATA SUMMARY
STABILITY
PERMEABLE

Grune, J.
Kohlhase, S.

Wave transmission through vertical slotted walls.

Proceedings of the 14th Coastal Engineering Conference. Copenhagen, Denmark. June 1974. pp. 1906-1923.

Experiments were conducted with regular waves to determine the wave transmission characteristics of vertical slotted walls with respect to 1) the ratio of the impermeable part of the wall to the total wall, W; 2) the shape and relative dimensions of wall elements (ratio of width to thickness, b/t); 3) the wave approach direction (B= 0, 45, 67.5 and 90 degrees); and 4) the incident wave steepness (H/L= 1:12 to 1:40). Rectangular wall elements were tested for W= 0.4 to 0.75 and b/t= 0.67 to 10. An H-beam shape was tested using W= 0.425 and 0.61 and b/t= 0.5. The authors state that element shape is more important for B>0 than B=0, but the data are inconclusive in this regard. Trends evident from the data are that increasing B, W, and/or H/L causes a decrease in transmission coefficient. There is some evidence that thicker wall elements provide greater wave damping at higher values of W, but there are not enough data presented to substantiate this. Test results for B=0 are compared with previous investigations and theoretical derivations.

BREAKWATER
VERTICAL
EXPERIMENTAL
LABORATORY

PERFORMANCE
PERMEABLE

Hall, W.C.
Hall, J.V.

A model study of the effects of submerged breakwaters on wave action.

U.S. Army Corps of Engineers, Beach Erosion Board. Technical Memo 1. 1940. 33 pp.

This study examines the changes produced in a wave field by interaction with submerged breakwaters. Smooth, impermeable, triangular, trapezoidal, and thin vertical structures are tested in the laboratory. Wave height and length are measured landward and seaward of the breakwater; wave period and velocity are measured seaward of the breakwater. All data presented are averages for a given run. Relationships between the ratio of water depth to structure height, which is varied from 0.9 to 1.5, and the ratio of wave height landward to wave height seaward of the structure are presented for the different shapes, for two different values of Hs, one large and one small, and for two different wave length to wave height ratios, one representing storm conditions and the other representing calm sea conditions. Graphs showing the range of values of the wave height ratios and wave power ratios are also given for different depth to structure height ratios. No quantitative conclusions are drawn based on the test results, however the following qualitative conclusions are made:
1) Submerged, shore-parallel structures reduce wave energy and cause sand accretion;
2) A vertical wall is the most effective shape for attenuating wave energy;
3) Depth of submergence influences reduction in wave energy; and
4) For protection from storm waves the depth to structure height ratio should not exceed 1.2.

BREAKWATER
SUBMERGED
EXPERIMENTAL
LABORATORY
PERFORMANCE
IMPERMEABLE

Hamer, D.G.
Hamer, F.C.

Laboratory experiments on wave transmission by overtopping.

Coastal Engineering. Vol. 6, No. 3. 1982. pp. 209-215.

Laboratory experiments with regular waves were used to investigate wave transmission by overtopping for a smooth, impermeable breakwater. The front and back faces of the test structure were composite slopes. The front face was 1:4 at the toe and crest, separated by a horizontal ledge below water level. The back face was 1:6 from the crest down to a horizontal ledge at the same elevation as the front face ledge, and 1:4 at the toe. Incident wave heights ranged from 9 to 15 cm, and wave period from 0.98 to 2.19 sec. Water depth varied from 32.5 to 37.5 cm. Test results show that the amount of wave transmission behind the breakwater is linearly related to the breakwater freeboard divided by the theoretical wave run-up height, where wave run-up is computed according to the empirical formula of Hunt. Transmitted waves were found to contain higher harmonics which became more clearly distinguished as the amount of wave overtopping increased. The transmitted wave height was defined by the average of the highest one-third waves. The experimental results are not applicable to irregular waves.

BREAKWATER
EXPERIMENTAL
LABORATORY
PERFORMANCE

WAVE OVERTOPPING

Hattori, M.

Transmission of water waves through perforated wall.

Coastal Engineering in Japan. Vol. 15. 1972. pp. 69-79.

An analytical solution to the transmission of small amplitude, shallow-water waves through a vertical, perforated wall is given. It is assumed that wall thickness and hole diameter are small compared to other significant length scales such as water depth and incident wave length. The continuity, Bernoulli, and momentum equations are applied at the wall, and reflection and transmission coefficients, Kr and Kt, are calculated for normally incident waves. A theoretical expression for the coefficient of energy loss due to the perforated wall is obtained based on continuity between the front and back of the wall. Based on theory, wave attenuation is a function of porosity, E, which is defined by ratio of hole area to total wall area, wave steepness, relative water depth, h/L, and the discharge coefficient of the hole, D. D depends on the ratio of wall thickness to hole diameter, t/d, Reynolds number with respect to jet velocity, and hole shape. Correction factors are used to account for the use of of average values in the derivations. Experiments were conducted with regular waves on three different walls with E's, d's, and t/d's of 0.25, 10 mm, 2.0; 0.125, 40 mm, 0.5; and 0.0625, 20 mm, 1.0; respectively; h/L varied from 0.15 to 0.25. D for each wall was determined experimentally and assumed constant throughout the tests. The theoretical and experimental Kt and Kr are plotted versus the ratio of water depth to incident wave height, h/H. Nearly all experimental data are within the range of $4<h/H<12$, the assymptotic part of the curve. At $h/H<4$ the curves change rapidly. Therefore, the data set does not provide a good verification of the theory. At best, application should be limited to the range of values tested, after regression analyses have shown that the test data are well-correlated to this part of the curve.

BREAKWATER
VERTICAL
THEORETICAL
EXPERIMENTAL
LABORATORY
PERFORMANCE
PERMEABLE

Hom-ma, M.
Sakou, T.

An experimental study on the submerged breakwater.

Coastal Engineering in Japan. Vol. 2. 1959. pp. 103-109.

Laboratory tests were conducted with vertical walls constructed of concrete and wooden blocks imbedded in a sand bed in order to determine the two-dimensional changes in the beach profile resulting from placement of the wall. An initial beach slope of 1:15 was used. Wall construction was such that there was no flow in the uniformly-sized sand beneath the wall. The ratio of wall height to water depth, h/d, varied from 0.5 to 1.0. Deep water wave steepness ranged from 0.0205 to 0.0490. For each run, the wall was placed at the breakpoint; inshore one-quarter of the distance from the breakpoint to shore; and offshore one-half the breaking wavelength from the breakpoint. Profiles were measured at 15, 30, 60, and 100 minutes.
Based on the data and analyses presented, general conclusions from the study are:
1) Maximum scour occurs when the wall is placed at the breakpoint;
2) Most scour occurs in the first 15 to 30 minutes;
3) A small area of accretion adjacent to the shoreward side of the wall can be expected;
4) The value of h/d that gives maximum scour is not fixed, but seems to depend upon the wall's location with respect to the breakpoint. In these tests, maximum scour for the breakpoint and shoreward locations occurred for h/d=1.0. At the seaward site, the maximum occurred at h/d=0.75.
The authors also noted a rise in the sea level shoreward of the wall due to the presence of the wall.

BREAKWATER
SUBMERGED
EXPERIMENTAL
LABORATORY
PERFORMANCE
PERMEABLE
WAVE SCOUR

Hudson, R.Y.
Jackson, R.A.

Stability of rubble-mound breakwaters, hydraulic model investigation.

U.S. Army Corps of Engineers Waterways Experiment Station. Technical Memo 2-365. 1953. 66 pp. + appendices.

Laboratory tests were conducted on trapezoidal breakwater sections in a 5 ft by 18 ft by 119 ft concrete flume. Tests were designed to provide information about several topics related to the design and construction of rubble mound breakwaters. First the validity of model-prototype ratios based on Froude's law was determined. Several different sections were subjected to the same prototype waves at test scales of 1:30, 1:45, and 1:60. There were no substantive differences between the scales in the way the rate of damage, time to stabilization, and final stable profile were modelled. Next tests were conducted to determine 1) the stability of component breakwater materials during various stages of construction, including completed sections; 2) the accuracy of the Iribarren and Epstein-Tyrell formulas for design of rubble breakwaters; 3) coefficients of reflection from rubble breakwaters; 4) the effect of angle of incidence of wave attack on the stability of rubble breakwaters; 5) the increase in design wave height that can be realized by allowing slight damage to the breakwater; and 6) the ability of a wall of air bubbles to reduce wave heights. Test results are summarized as follows:

1) Protective cap rock should be placed on both the seaward and shoreward sides of the structure as soon as feasible during construction to prevent erosion of the underlying materials by large waves;

2) Care should be taken that the cap rock is extended a sufficient distance below still water level to prevent scour at the toe of the cap rock slope;

3) Crest erosion increases when crest height increases and/or crest width decreases;

4) The Iribarren formula is a valuable design tool when used in conjunction with an experimentally determined stability coefficient, K.

Hudson, R. Y.
Jackson, R. A.

Stability of rubble-mound breakwaters, hydraulic model investigation (cont).

The value of K for a particular type of rock varies with breakwater slope, wave steepness, and relative depth, d/L;
5) The Epstein-Tyrell formula is not an improvement over the Iribarren formula for rubble breakwater design;
6) Reflection and absorption by rubble structures varies with breakwater side slope, incident wave characteristics, and water depth;
7) Allowance of slight structural damage can increase the design wave height more than 50 percent;
8) Cut stone or molded block breakwaters can be more stable than dumped rubble; and
9) Reduction in wave height by the compressed air curtain appears to be a function of d/L. In these tests reduction ranged from almost none for d/L= 0.25 to 25 percent for d/L= 0.37. The curtain seemed to be more effective with steeper waves, but no quantitative conclusions could be drawn.

BREAKWATER
EXPERIMENTAL
LABORATORY
STABILITY

PERMEABLE

Iwagaki, Y.
Asano, T.
Honda, T.

Combination effect of pneumatic breakwater and other type breakwater on wave damping.

Proceedings of the 16th Coastal Engineering Conference. Hamburg, Germany.
August/September 1978. pp. 2172-2190.

Laboratory tests were conducted to determine the improvement to wave damping that can be achieved by combining a pneumatic breakwater with a vertical submerged or floating breakwater. Four basic types of submerged structures were tested:
1) impermeable and smooth;
2) impermeable with roughness on the crest;
3) permeable with void ratio = 98%; and
4) permeable with void ratio = 45%
Tests were done in a water depth of 45 cm with structure heights of 30, 35, and 40 cm for all types of structures. Additional tests were done on 45 cm, impermeable structures. Crest width was always 20 cm except for the impermeable, 30 cm high models which were also tested for widths of 40 and 60 cm. The floating breakwater was a solid plate between 30 and 90 cm wide. Both anchor lines and fixed supports were tested. The effectiveness of adding a vertical or inclined plate to the bottom of the floating plate was checked. Both monochromatic and irregular waves were tested. Regular waves varied in height from 4 to 6 cm and in frequency from 0.4 to 2 Hz. Spectral characteristics of the incident irregular waves are not clearly defined. General results from these experiments show that the performance of a pneumatic breakwater is most greatly enhanced when used in conjunction with a submerged structure. A pneumatic breakwater is least effective for low frequency waves. The submerged structure tends to redistribute the low frequency energy into higher frequencies where it can be effectively damped by the pneumatic breakwater. The floating breakwater is also ineffective at low frequencies so combination with the pneumatic breakwater only improves damping at high frequencies. Use of guide plates and fixed supports also gives greater damping.

BREAKWATER
PERFORMANCE
EXPERIMENTAL
LABORATORY
PERMEABLE
IMPERMEABLE

Iwasaki, T.
Numata, A.

Experimental studies on wave transmission of a permeable breakwater constructed by artificial blocks.

Coastal Engineering in Japan. Vol. 13. 1970. pp. 25-29.

Experiments were conducted using regular waves to determine the energy transmission characteristics of trapezoidal, 1:1.5 sloped, permeable breakwaters constructed with manufactured concrete blocks. Three structures were tested, with freeboards ranging from 6 to 16 cm and widths from 50.8 to 77.6 cm. The structures were built on a 1:6 slope, with 1:30 slopes in front and behind, and located slightly offshore of where the breakpoint would be without the structure in place. The water depths in front of and behind the structure averaged 41.3 and 22.3 cm, respectively. Wave steepness, H/L, ranged from 0.016 to 0.048.
Test results indicate that, in general, wave transmission decreases as H/L and relative freeboard, h/H, increase. Energy dissipation increases as H/L and h/H increase. Using a relationship similar to Goda's expression for wave transmission through wire mesh screens, it is also shown that for values of h/H>1, wave transmission is mostly dependent on wave steepness. Data from the Niigata Investigation and Planning Office tend to support this finding.

BREAKWATER
EXPERIMENTAL
LABORATORY
PERFORMANCE
PERMEABLE

Jackson, R.A.

Stability of rubble-mound breakwaters, Nassau Harbor Nassau, New Providence, Bahamas.

U.S. Army Corps of Engineers Waterways Experiment Station. Technical Report 2-697. 1965. 21 pp. + appendices.

Tests were conducted on 1:36, 1:45, 1:48, and 1:56 scale models of breakwater trunks and heads constructed of quadripods, tetrapods, and tribar armor layers to obtain data from which alternate designs could be developed for construction of breakwaters at Nassau Harbor, Nassau, New Providence, Bahamas. It was determined that because of the very large cross section required it would be uneconomical to construct a mound of rubble on the seaside of an impermeable vertical wall capable of reducing the maximum wave-generated shock pressures to values obtained from the Sainflou theory. Coefficients for the WES breakwater stability equation were determined for design of tetrapod and tribar armor layers for both the trunk and head of the west breakwater. Stability of the east breakwater proposed for Nassau Harbor was not investigated; design coefficients for the west breakwater are applicable to geometrically similar structures for the east breakwater. A method was developed for placing armor units in a rectangular trench at the toe of the slope of the cover layer to prevent en masse sliding of the armor units.

BREAKWATER
EXPERIMENTAL
LABORATORY
MODEL

STABILITY
PERMEABLE

Johnson, J.W.
Fuchs, R.A.
Morison, J.R.

The damping action of submerged breakwaters.

Transactions of the American Geophysical Union. Vol. 32, No. 5. October 1951. pp. 704-718.

The results of an experimental investigation on the damping action of submerged, impermeable, rectangular breakwaters are presented. The experimental data are compared with published theories. A new theory is presented which compares more favorably with the experiments than previous theories. Also given is a summary of all available published theoretical and experimental information on the damping action of trapezoidal and triangular breakwaters, reefs of various configurations, and plane barriers of various orientations. Test variables are height and width of the barrier, water depth, and wave height and length. Tests are conducted for both a horizontal bottom and a beach slope of 19.2:1. In the latter tests, the position of the breakwater relative to the breaker line for undisturbed conditions is also varied. General conclusions drawn from the study are:
1) For a given barrier dimension and relative depth, more damping occurs for relatively steep waves than for relatively flat waves;
2) For a give barrier dimension and wave steepness, more damping occurs for smaller relative depths; and
3) A wide barrier has better damping action than a narrow barrier, especially for steeper waves.

BREAKWATER
SUBMERGED
EXPERIMENTAL
LABORATORY

PERFORMANCE
IMPERMEABLE

Kamel, A.M. Water wave transmission through and reflection by pervious coastal structures.

U.S. Army Corps of Engineers Waterways Experiment Station. Research Report H-69-1. 1969. 27 pp. + figures.

An expression is given for the resistance of free surface flow in a porous medium, such as a rubblemound breakwater, in terms of the density of the fluid, the approach velocity, the grain diameter of the material in the breakwater structure, and a resistance coefficient which is a function of Reynolds number of the flow. Tests indicated that the length of the structure relative to the grain diameter is an important factor in determining the resistance of free surface flow in a porous medium. A certain ratio between the length of the structure and its grain diameter must be reached before the structure can be treated as a porous medium. An expression derived by Dr. G. H. Keulegan for damping of translation waves by screen filters was modified to give the transmission coefficient of surface waves propagated through a porous structure. The constants in this expression were also found to be a function of the length of the structure relative to the grain diameter. Measured and computed values of wave transmission coefficients agreed well.

BREAKWATER
THEORETICAL
EXPERIMENTAL
PERFORMANCE

PERMEABLE

Katayama, T.
Irie, I.
Kawakami, T.

Performance of offshore breakwaters of the Niigata Coast.

Coastal Engineering in Japan. Vol. 17. 1974. pp. 129-139.

The Shinano River is a major supplier of sediment to the Niigata Coast. Following the construction of jetties at the river's mouth and the subsequent dredging of a diverging channel, the amount of sediment being supplied to the adjacent coastlines decreased, resulting in progressive beach erosion. Initial attempts to slow this erosion consisted of T-groins along the west coast. Eventually the T's were connected to form a continuous offshore breakwater. A continuous breakwater was built on the east coast several years later. This report examines the effect of these breakwaters on the coastline. Field observations of annual variations in water depth on- and offshore of the breakwater; scour-related subsidence of the breakwater; increase in water level behind the structure; energy transmission; and vertical distribution of suspended sediment are discussed.
After construction of the breakwaters, the water depths offshore increased, probably due to ground subsidence. There was only a slight variation onshore of the west coast structure, indicating that there was enough material passing through and over the breakwater to offset the effect of ground subsidence. In contrast, the onshore depth along the east coast increased abruptly following construction of the breakwater. This is presumably because the bottom was scoured by longshore currents that were prevented on the west coast by the connecting groins. Water depths on the east coast increased after groins were added. Local scour and settlement were found to vary with crest elevation, with a lower crest resulting in greater local scour and settlement. Maximum settlement occurred in the section 100 to 200 m from the end. Additional field observations were made during the winter of 1972-73 to measure

Katayama, T.
Irie, I.
Kawakami, T.

Performance of offshore breakwaters of the Niigata Coast (continued).

energy transmission, vertical sediment suspension, and water level set-up behind the breakwater. The incident significant wave height and wave period ranged from 0.96 to 2.03 m and 7.1 to 8.5 sec, respectively. Spectral analyses indicate that the transmission coefficient is higher at lower frequencies, and that at the dominant frequency the transmission coefficient decreases as incident wave height increases. There is also a slight increase in transmission coefficient at very high frequencies. The concentration of sediment found in suspension onshore of the breakwater increased with the incident wave height. Since no substantial overtopping took place for this range of wave heights, this can be taken as further evidence that material passes through the structure. Water level set-up behind the breakwater varied from about 4 to 19 cm for the range of wave heights given. Undistorted 1:100-scale model tests with regular waves overpredicted the actual set-up observed for spectral waves.

BREAKWATER
EXPERIMENTAL
FIELD
LABORATORY
MODEL
PERFORMANCE

Kato, J.
Noma, T.

On the wave damping effect of double curtain breakwater.

Coastal Engineering in Japan. Vol. 12. 1969. pp. 41-45.

Laboratory experiments were conducted with regular waves to determine the wave damping effect of two 12 mm wide, rigid vertical barriers placed a distance, l, apart. Experimental results are compared to the theories of Ursell and Wiegel for a single vertical barrier. The theory for the double curtain is based on an extension of Wiegel's single wall theory. The ranges of wave steepness, H/L, and relative depth, h/L, were 0.027 to 0.114 and 1.071 to 0.291, respectively. The distance, l, was varied in 30 cm increments from zero to 300 cm. Values of d = 0, 8.5, 20.0, and 30.0 cm were used, where d is the distance that the plate extends below the surface. l/L ranged from 0 to 2.14 and d/L from 0 to 0.214. The effect of wave reflection in front of and behind the walls is not considered. The transmission coefficient, K, is plotted versus l/L for different values of d/L and different values of H/L at d/L=0. K is also plotted versus d/L for l/L = 0, 0.510 and 0.856. The data as presented are fairly scattered and do not provide a very clear picture of the relationship between K and d/L, l/L, and H/L. Some general trends that seem to emerge, however, include: 1) The double curtain is more effective than the single barrier; 2) larger values of d/L provide better wave damping; 3) for d/L=0, K increases as H/L increases; and 4) the degree of influence of l/L on K depends upon the value of d/L. Since H/L appears to influence K, the data might be collapsed somewhat if they were separated consistently according to incident wave steepness. It should also be considered that reflected waves may have a substantial influence on the wave records.

BREAKWATER
EXPERIMENTAL
LABORATORY
PERFORMANCE
IMPERMEABLE

Keulegan, G.H. Wave transmission through rock structures.

U.S. Army Corps of Engineers Waterways Experiment Station. Research Report H-73-1. 1973. 67 pp.

A theoretical expression for the wave transmission coefficient, Kt, for a permeable prismatic structure built with homogeneous size rock is derived for long waves traversing the structure. First the solution for wave attenuation in a porous medium is presented for the case where energy losses are due to viscous forces only. The governing dynamic and continuity equations are modified to include an energy loss term that accounts for dissipation by turbulence. Continuity is expressed in terms of density and kinematic viscosity of water, seepage velocity, stone diameter, energy loss per unit volume of the structure per unit time, and and experimentally determined loss coefficient. When dissipation in the voids is due partly to turbulence, Kt is given by a power formula. The formula is modified such that it can be used for waves other than shallow water waves and is in the form of a linear equation, where Kt is the dependent variable, and the independent variable is a function of water depth, structure width, and incident wave height and length. The slope of the line is a term that includes the effects of experimental coefficients, porosity, wave period, water depth, stone diameter, and the kinematic viscosity of water. This slope is determined experimentally for several different rock structures (rock diameter= 1.3 to 4.8 cm) and wave conditions (wave period= 0.71 to 1.94 sec). Results indicate that this linear relationship works quite well. Wave reflection is not dealt with theoretically. Rather, all results are combined onto one curve, and the reflection coefficient, Kr, is given as a function of the ratio of structure length to wavelength, l/L. Although there is considerable scatter in the data, which does not seem to be due to differences in rock size, trend shows that Kr is maximum at about l/L= 0.2.

BREAKWATER
THEORETICAL
PERFORMANCE
PERMEABLE

Kogami, Y.

Researches on stability of rubble-mound breakwater.

Coastal Engineering in Japan. Vol. 21. 1978. pp. 75-93.

The results from several types of experimental and theoretical studies on the stability of armor units on rubble-mound breakwaters are presented. The first set of experiments seeks to determine the effects of different wave parameters on the critical Hudson's stability number, Nc, for a multi-layered, submerged, 1:2-sloped rubble structure fronting an impermeable caisson. Tests were conducted at a scale of 1:20 in a 80 cm wide by 100 cm deep by 28 m long wave tank. Parameters tested include wave steepness (H/L= 0.017 to 0.108); ratio of water depth in front of the structure to wavelength (d/L= 0.064 to 0.181); ratio of freeboard to water depth (s/d= 0.33 to 0.80); and ratio of top width to wavelength (B/L= 0.05 to 0.25). Nc was defined as the condition at which less than one percent of the precast concrete armor units were rocked or moved. All of the above parameters had some influence on stability. A second set of experiments evaluates the critical stability of armor units near and below mean water level. Tests on 1:2 sloped rubble mounds were carried out in a 1 m by 1 m by 30 m flume in a water depth of 60 cm. Slope deformation and the water depth to which armor layer erosion occurred were measured for a range of wave steepness 0.010 to 0.072. Tests on the movement of armor stones at the lower end of a protective "sand mastic" layer are discussed with respect to wave steepness and depth of the protection below mean water. The third discussion is a theoretical analysis of scale effects in rubble mound stability and wave transmission models. Finally, results of tests on the uplift force exerted on the sand-mastic layer are presented. Maximum uplift was measured just before wave uprush. The validity of an uplift force factor calculated by other researchers is evaluated.

BREAKWATER
EXPERIMENTAL
LABORATORY
STABILITY
PERFORMANCE

Kondo, H. An analytical approach to wave transmission through permeable structures.

Coastal Engineering in Japan. Vol. 13. 1970. pp. 31-42.

The wave transmission coefficient, Kt, for homogeneous, rectangular rubble structures is derived theoretically based on small amplitude, long wave theory. First, the equations for unsteady, laminar flow in a permeable medium are solved. From this solution it is seen that water surface elevation and particle velocity are no longer in phase, and the phase difference is a function of wave period and medium characteristics. Theory predicts that the wave height within a porous medium decreases exponentially and that the ratio of wave height at some point, x, to the original wave height decreases if wave period, water depth, or permeability decrease. The results for laminar flow are extended to turbulent flow using a linearized friction term and an expression for steady, turbulent flow in a porous medium given by Ward (ASCE Jour. of the Hydraulics Div., #5, September 1964). Theory is extended further to the case of wave transmission and reflection at the interface of two permeable media, and finally to transmission through an idealized structure. Continuity of pressure and conservation of mass and mean energy at interfaces are assumed. Experimental results of other researchers are compared to the theoretical results. Wave periods ranged from 1.4 to 3.5 sec; stone diameter from 0.91 to 5.9 cm; structure width from 15.2 to 124 cm; and structure height from 30.5 to 100 cm. Results agreed well with theory in some cases and not so well in others. In general, theory underpredicted Kt for larger wave periods and over predicted for the smaller diameter stone and narrow top width. Predictions were best for the shorter wave periods and larger stone sizes.

BREAKWATER
THEORETICAL
PERFORMANCE

Kondo, H.
Toma, S.

Breaking wave transformation by porous breakwaters.

Coastal Engineering in Japan. Vol. 17. 1974. pp. 81-91.

Experimental observations of the reflection and transmission characteristics of a variety of subaerial porous breakwaters are presented. Tests were conducted in a 40 cm wide by 100 cm deep by 18.5 m long wave tank with breaking, broken, and non-breaking waves. Structures were fronted by a 1:20 sloped beach. Types of structures tested include trapezoidal and varying width rectangular lattice structures, where the lattice is constructed with PVC pipe glued together horizontally and vertically, and a trapezoidal rubble mound. Incident wave periods were 0.8 to 1.4 sec and water depth at the structure toe ranged from 8.2 to 13.2 cm. Some of the data are compared to theories derived by the author in earlier works. Since the theoretical approach is based on rectangular structures, the trapezoids are approximated by an equivalent rectangle. The data comparisons presented are confusing and incomplete, so it is difficult to comment on the validity of the theory. It appears, however, that for the situations shown, theory predicts general trends for Kt and Kr, but there is a large discrepancy between the actual values that are predicted and those that are measured. The discrepancy increases as the ratio of structure width to wavelength increases. General observations based on tests are:
1) Minimum Kt occurs when the wave breaks at the base of the structure; and
2) Wave reflection from rectangular structures is independent of wave height, but Kr for trapezoidal structures increases as wave height increases.

BREAKWATER
EXPERIMENTAL
LABORATORY
PERFORMANCE
PERMEABLE

Kono, T.
Tsukayama, S.

Wave transformation on reef and some consideration on its application to field.

Coastal Engineering in Japan. Vol. 23. 1980. pp. 45-57.

Field observations of wave decay across a reef are compared with results obtained in the laboratory. Wave spectra from four field locations, one offshore and three along a profile across the reef, are analyzed and compared to the theories of Longuet-Higgins and Phillips. Representative wave heights at the three reef locations are compared to the incoming wave height in order to assess the decay as the wave progresses across the reef. The distribution of wave period at two reef sites is compared to Bretschneider's theoretical wave period distribution for a fully developed sea. The coefficient of sea bottom friction is compared to theoretical results from laminar boundary layer theory. Data exhibit a considerable amount of scatter and although the authors state that they agree reasonably well with theory, this conclusion is not justified. Results from a 1:65 scale model of the reef are equally unsatisfying. Lab data are obtained for regular waves for a range of wave steepness, H/L, from about 0.01 to 0.09. Transmission coefficients, K, at the three field locations and K at one location on the lab reef are plotted against wave steepness. Steepness of the field waves is calculated using the average, significant, highest one-tenth, and maximum wave heights. Only a few of the lab data points are within the range of steepness of the field data. Because of the amount of scatter in the field data and the paucity of lab points available for comparison, no justifiable conclusions can be drawn with respect to the comparison between the two.

REEFS
EXPERIMENTAL
FIELD
LABORATORY
MODEL
WAVE TRANSFORMATIONS

Lillevang, O.J.

Breakwater subject to heavy overtopping; concept, design, construction and experience.

Proceedings of ASCE Specialty Conference Ports '77. Long Beach, California. March 1977. Vol. 2. pp. 61-93.

The design, scale modeling, construction, service record, and post-construction overhaul of a set of breakwaters protecting the water intakes at the Diablo Canyon nuclear power plant in California are described. Discussions of the stability of rear slope armor on heavily overtopped breakwaters and problems created by pressure build-up under concrete caps are included. Two types of model tests were performed. The first was a 1:100 scale, non-distorted, three-dimensional model designed to defing the wave climate behind the breakwater for several design conditions. The model was tested with regular waves from three defferent directions. Each direction was tested with five wave periods. The second set of tests were conducted in a wave flume at a scale of 1:75 in order to determine stability of the tribar armor units. It was found that the rear slope armor stability is the critical design factor for heavily overtopped structures. The author makes the following general conclusions based on his work with this project:

1) As of 1977, analytical design of backslope stability for overtopped breakwaters was not possible; scale modeling is the best way to ensure backslope stability of heavily overtopped structures;

2) Detailed bathymetric site surveys should be conducted to minimize construction costs and to avoid potential future problems;

3) Pressure release vents should be provided when concrete caps are used on the breakwater crest. Vents help prevent a build-up of pore pressure that could lead to instability; and

4) A single layer of carefully placed tribar provides excellent armoring and uses comparatively less material and individual units. Strict quality control on unit placement must be employed, however, or another type of armoring should be used.

BREAKWATER
EXPERIMENTAL
LABORATORY
MODEL
FIELD MONITORING
WAVE OVERTOPPING

Madsen, O.S. Wave transmission through porous structures.

ASCE Journal of the Waterways, Harbors, and Coastal Engineering Division. Vol. 100, No. WW3. August 1974. pp. 169-188.

An analytical approach to the problem of wave reflection and transmission from permeable rectangular structures is discussed. It is assumed that the incident wave is normal to the structure and may be described by linear wave theory; the flow resistance in the porous medium is linearly proportional to the flow velocity, and the constant of proportionality is a friction factor, f; and incident waves are long relative to the structure width. Sollitt and Cross (1972) present a similar derivation but without the long wave assumption. Their approach requires a lengthy iterative solution for f. By adding the third assumption, however, the series solution is reduced to the leading term, and f can be found explicitly as a function of breakwater geometry, incident wave characteristics, and hydraulic properties of the breakwater expressed in terms of empirical constants, A and B. Reflection and transmission coefficients, Kr and Kt, are found to be functions of a single parameter which is defined in terms of the wave number, structure width, f and porosity. Success of the method for predictive purposes is dependent upon the ability to determine A and B accurately. Empirical relationships that relate A and B to medium porosity and characteristic particle diameter are examined and the computed values are compared to experimentally determined values. The empirical relationships are found to be adequate for preliminary design purposes. Comparison of theory with the experimental data of Sollitt and Cross (1972) shows good agreement for Kt. Agreement with the predicted Kr is not as good; in particular, the measured Kr exhibits less variation with incident wave steepness than predicted by theory.

BREAKWATER
THEORETICAL
PERFORMANCE
PERMEABLE

Madsen, P.A. Wave reflection from a vertical permeable wave absorber.

Coastal Engineering. Vol. 7, No. 4. 1983. pp. 381-396.

A theoretical solution for the reflection of periodic, linear, shallow water waves from a rectangular, porous wave absorber on a horizontal bottom is presented. The wave absorber is considered to be a subaerial, homogeneous rubble structure with an impermeable core. Continuity of pressure and mass are used to match the periodic solutions at the front face of the structure. Using the solution technique of Madsen and White (1976), reflection coefficient is derived as a function of the linearized friction factor, porosity, and wave number times structure width. The friction factor is determined implicitly as a function of wave height and period, porosity, grain diameter, and water depth using Lorentz' principle of equivalent work which states that the rate of energy dissipation is the same for the linearized and non-linearized friction terms. Theoretical results agree in most cases to results from a numerical short wave model. Differences are due to non-linear effects in the theory. Data from vertical-faced wave absorbers with sloped tops on non-horizontal bottoms are also compared with theory. Although the trends are the same, the sloped structures are much more efficient wave absorbers than predicted by the simple theory. Other notable observations include:

1) Long waves are not necessarily more difficult to damp. Rather, degree of absorption depends on the specific combination of controlling factors;

2) High waves are absorbed better by high porosity materials; low waves are absorbed better by low porosity materials; and

3) Because the solution is strongly non-linear, theoretical results are not applicable to a wave spectrum (superposition of regular waves).

BREAKWATER
THEORETICAL
PERFORMANCE
PERMEABLE

Madsen, O.S.
Shusang, P.
Hanson, S.A.

Wave transmission through trapezoidal breakwaters.

Proceedings of the 16th Coastal Engineering Conference. Hamburg, Germany. August/September 1978. pp. 2140-2152.

A proposed improvement to the method of Madsen and White (1975) for prediction of reflection and transmission coefficients for subaerial, trapezoidal, rubble structures is discussed. In particular, improvement to the prediction of reflection coefficient is sought. In the earlier work, the trapezoidal structure is represented by a hydraulically equivalent rectangle, and internal and external dissipation are considered separately. The present paper seeks to solve the governing equations within the "water wedge" region, e.g., along the sloped structure faces, rather than applying the previously used simplification. In this way the two dissipative mechanisms are treated together, not separately. Solution for flow within a porous medium is given by Madsen (1974). Solutions outside the structure region are well-known. In all cases, the friction factors are linearized. Experiments were conducted in a 24 m long wave flume at MIT. Water depth was 30 cm and incident periods were 1.6, 1.8, and 2.0 sec. Foreslopes of 1:1.5, 1:2.0, and 1:2.5 were tested with a constant backslope of 1:1.5. Structure material was homogeneous with an equivalent diameter of 1.5 cm and porosity of 40 percent. Test results show that the new method is only marginally better than the old method in that it accurately predicts decreasing wave reflection for increasing wave steepness for the 1:2.5 slope. There are still large discrepancies, however, between the actual and predicted reflection coefficients. The discrepancy increases as the structure slope decreases. The new method underpredicts reflection for the 1:2.0 and 1:2.5 slopes, while the old method overpredicts.

BREAKWATER
THEORETICAL
EXPERIMENTAL
LABORATORY
PERFORMANCE
PERMEABLE

Madsen, O. S.
White, S. M.

Reflection and transmission characteristics of porous rubble-mound breakwaters.

US Army Corps of Engineers Coastal Engineering Research Center. Miscellaneous Report No. 76-5. March 1976. 138 pp.

An explicit analytical solution for wave reflection and transmission coefficients, Kr and Kt, of rectangular crib structures and a semi-empirical approach to calculating reflection from a rough, impermeable slope are developed and combined to get an approximate solution for Kr and Kt of trapezoidal, permeable, multi-layered structures. Solutions assume than incident waves are normal to the structure, non-breaking, and can be described by linear long wave theory. Also the structures have a natural stone armor layer. The solution for rectangular structures is the same as Madsen (1974), and is the portion of the overall solution that accounts for internal energy dissipation. The second part of the solution accounts for external energy dissipation by regarding the seaward slope of the structure as impermeable. The solution for Kr is obtained analytically. The linearized bottom friction is related to a wave friction factor, fw, and this relationship is applied to experimental results in order to find an empirical expression for fw in terms of water depth, average stone diameter, slope angle, and complex vertical wave amplitude of the wave motion on the slope. The procedure for the approximate solution for the trapezoid is to determine Kr due to the impermeable slope and calculate a new incident wave amplitude using this Kr. In order to be able to apply the solution for the rectangular crib, the trapezoid is reduced to an hydraulically equivalent rectangle. The new wave amplitude is applied to the solution for the rectangle to get the total resulting Kr and Kt. Comparison of data from Sollitt and Cross (1972) over a range of wave steepness (H/L= 0.002-0.03) indicates that theory predicts Kt very well throughout the entire range of H/L. Predictions for Kr are good within the range of H/L less than 0.008.

Madsen, O.S.
White, S.M.

Reflection and transmission characteristics of porous rubble-mound breakwaters (cont).

For higher values of wave steepness, experimental Kr decreases with increasing H/L, but theory predicts that Kr increases as H/L increases.

BREAKWATER
THEORETICAL
PERFORMANCE
PERMEABLE

Mansour, W.O. Development of a methodology for the design, construction and quality assurance of rubble mound breakwaters.

California Sea Grant Report N 22554. 291 pp. + appendices.

The design and construction of rubblemound breakwaters from ancient to present times is reviewed. Emphasis is placed on the breakwater core. The topics discussed include; history of breakwaters; importance of the core and its effect on permeability and energy transmission; foundation conditions and preparation; desirable characteristics of core material; review of construction equipment and methods; effect of construction procedure on stability, cost, and performance; and descriptions of different construction monitoring systems. Information about construction materials, equipment, techniques, and specifications for offshore breakwaters is included in the appendices. In addition, 1:20 scale model tests were conducted in order to determine the most effective and least wasteful way to place the core material. The dumping of core material was simulated in a wave flume 8 ft wide by 5 ft deep by 160 ft long under varying wave and current conditions in depths ranging from one to three feet. The effects of placing wet vs. dry rock, dumping rate, and seafloor characteristics are also examined. The following observations and conclusions are made:
1) Dumped rock forms a circular pattern that is elongated into an ellipse when wave or current are present. Size and shape of the pattern is independent of water depth and seafloor characteristics;
2) In this study there was no difference between wet and dry rock with respect to the pattern formed. However, the wet rock was not saturated, only wet down before dumping. Field observations of pre-saturated rock dumped from a bottom-dump barge indicate that pre-saturation reduces the size of the dumped pattern. Additional studies are recommended.
3) A current tends to flatten the slope of the dumped rock;

Mansour, W. O. Development of a methodology for the design, construction and quality assurance of rubble mound breakwaters (cont).

4) Natural segregation of material occurs with dumping, and the coarse material is deposited on the outside;
5) Very fine material washes out and is dispersed radially; and
6) Slow rather than sudden dumping is preferred because the material does not spread out as much.
It is noted that results from field tests designed to determine the impact of dredged material disposal in the ocean are similar to the laboratory observation. Field experiments dealing with the relative merits of three rock placement methods are discussed briefly.

BREAKWATER
EXPERIMENTAL
LABORATORY
CONSTRUCTION

MONITORING
CORE MATERIALS

Markle, D.G. Kahului breakwater stability study, Kahului, Maui, Hawaii.

U.S. Army Corps of Engineers Waterways Experiment Station. Tech. Rpt. HL-82-14. July 1982. 29 pp. + figures.

A hydraulic model investigation was conducted at geometrically undistorted, linear scales of 1:33, 1:36, and 1:40, model to prototype, to evaluate the stability against wave attack of proposed rehabilitation designs for two areas, each on the harbor sides of the east and west breakwaters at Kahului Harbor, Maui, Hawaii. A proposed rehabilitation design for the sea-side slope of the west breakwater at sta 18+50 and the existing sea-side slope protection on the west breakwater at sta 21+25 also were evaluated for stability against wave attack. Where the proposed designs failed, additional tests of alternative plans were conducted until stable design sections were found. All plans were tested for the worst breaking wave conditions that could be produced for the selected wave periods, water depths, and bathymetry seaward of the test sections. Full length stability tests were conducted with prototype periods of 16- and 18-sec. Six adjectives were used to describe the degree of structural damage. Slight and minor; moderate; and significant, major, and extensive indicated acceptable, borderline acceptable, and unacceptable conditions, respectively. In addition, photographs were used to document pre- and post-test conditions. The existing 19-ton tribars on the sea-side slope of the west breakwater at sta 21+25 proved to be stable, and six plans (three dolos and three tribar) were found acceptable for the proposed harbor-side slope rehabilitation. With the addition of a concrete rib cap on the crwn of the west breakwater at sta 18+50, 11-ton and 5-ton tribars were found to be stable on the sea- and harbor-side slopes, respectively. For the east breakwater at sta 26+10, 9-ton tribars showed very good stability when placed on a 1V-on-2H harbor-side slope. A concrete rib cap was added to stabilize the crown and

Markle, D.G. Kahului breakwater stability study. (continued)

upper sea-side slope of the east breakwater at sta 23+35 and 9-ton tribars placed on a 1V-on-2H slope provided stable protection for the harbor-side slope. The stabilities of all plans found acceptable are dependent upon trenching and/or special placements of the toe armor units, as described for each alternative design. Model observations also indicated that the harbor-side armor unit protection should not extend above the breakwater crown any more than absolutely necessary. Tests indicated that it was preferable to leave a small gap between the concrete rib cap and the upper harbor-side armor protection than to fit a unit in this area if a large portion of the unit had to project above the crown of the structure.

BREAKWATER
EXPERIMENTAL
LABORATORY
MODEL

STABILITY

Markle, D.G. Breakwater stability study, Mission Bay, California.

U.S. Army Corps of Engineers Waterways Experiment Station. Tech. Rpt. HL-83-13. September 1983. 26 pp. + figures.

A hydraulic model investigation was conducted using a three-dimensional stability model at an undistorted linear scale of 1:36 (model to prototype). The purpose of the stability tests was to develop a random-placed armor-stone design for a proposed offshore breakwater (to be located seaward of the existing north jetty and middle jetty at Mission Bay, California) that will be stable for non-breaking wave heights up to and including 16.7 ft at still-water levels of 0.0 and +5.4 ft mllw, with structure crest elevation at +17.5 ft mllw. Two incident wave directions were tested using 9-, 11-, and 15-sec wave periods. The 15-sec waves were included to simulate high amplitude swell conditions. Wave steepness (H/T^2) varied from 0.0444 to 0.1864. Six adjectives were used to describe the degree of structural damage. Slight and minor; moderate; and significant, major, and extensive indicated acceptable, borderline acceptable, and unacceptable conditions, respectively. In addition, photographs were used to document pre- and post-test conditions. Test results indicated that two alternative plans would be adequate for the non-breaking wave conditions. One plan consisted of two layers of 29,000-lb stone random-placed on 1V-on-2H slopes on the breakwater heads and two layers of 22,700-lb stone random-placed and the 1V-on-2H ocean-side slopes and the 1V-on-1.5H channel-side slopes of the breakwater trunks. The other plan consisted of two layers of 29,000-lb stone random placed on the 1V-on-2H to 1V-on-1.5H slopes on the breakwater heads, the 1V-on-1.5H ocean-side slopes, and the 1V-on-1.25H channel-side slopes. Both plans sustained moderate damage, so periodic inspections and maintenance will be necessary. Better than random armor unit placement should also help to reduce maintenance costs.

BREAKWATER
EXPERIMENTAL
LABORATORY
MODEL
STABILITY

Markle, D.G.
Carver, R.D.

Breakwater stability study, Imperial Beach, California.

U.S. Army Corps of Engineers Waterways Experiment Station. Tech. Rpt. H-77-22. December 1977. 24 pp. + figures.

A hydraulic model investigation was conducted at a geometrically undistorted scale of 1:16, model to prototype, to design stable rubble-mound breakwater sections to protect a beach nourishment area at Imperial Beach, California. Both the -5.0 ft mllw contour (shallow water location) and -10.0 ft mllw contour (deeper water location) were given as proposed construction sites. Full length stability tests were conducted using 14-sec waves with wave heights of 4.5 and 8.5 ft at the -5.0 ft depth and 8.3 and 12.1 ft at the -10.0 ft depth. These heights represent the maximum breaking wave height for still water levels of 0.0 and +5.4 ft. Stability is assessed qualitatively based on the degree of movement of armorstone. Pre- and post-test conditions were photographed. Twenty-one plans were tested, resulting in two adquate breakwater designs for each of the two proposed sites. A constant high-sill structure with crest elevation +5.0 ft mllw, using 3-ton graded armor stone, and an alternating high- and low-sill structure (+10.2 and +5.0 ft mllw), using 5- and 0.5-ton armor stone, were stable for the design conditions at the shallow water location. Two alternating high- and low-sill structures proved adequate for the design condition at the deeper water location. One used 5- and 3-ton graded armor stone on the breakwater trunks (0.0 ft mllw) and 7-ton capstone on the ends (-5.0 ft mllw) of the breakwater system on 1V-on-3H side slopes while the other design used trunk sections (-5.0 ft mllw) of 7- and 5_ton graded armor stone on 1V-on-2H side slopes and 7-ton capstone on the head section (0.0 ft mllw) with 1V-on-3H side slopes.

BREAKWATER
EXPERIMENTAL
LABORATORY
MODEL

STABILITY

Markle, D.G.
Carver, R.D.

Unique design of semi-submerged offshore structures.

Coastal Structures '79. Alexandria, Virginia. March 1979. pp. 209-229.

See Markle, D.G. and R.D. Carver, 1977. Breakwater stability study, Imperial Beach, California.

BREAKWATER
EXPERIMENTAL
LABORATORY
MODEL

STABILITY

Markle, D.G.
Herrington, C.R.

Nawiliwili breakwater stability, Nawiliwili Harbor, Kauai, Hawaii.

U.S. Army Corps of Engineers Waterways Experiment Station. Tech. Rpt. HL-83-21. September 1983. 24 pp. + figures.

A hydraulic model investigation was conducted at geometrically undistorted, linear scales of 1:31 and 1:25, model to prototype, to evaluate the stability against wave attack of three areas of the existing breakwater at Nawiliwili Harbor, Kauai, Hawaii. Where the existing crown and/or harbor-side slope proved to be unstable, additional tests were conducted to check the stability of rehabilitation designs proposed by the Pacific Ocean Division. All existing and proposed rehabilitation designs were tested for the worst breaking wave conditions that could be produced for the selected wave periods, water depths, and bathymetry seaward of the test sections. Tests were conducted using 12-, 14- and 16-sec waves at a still water level of +4.0 ft mllw. Structure crests varied from +10.0 ft to +16.0 ft mllw. Six adjectives were used to characterize the degree of structural damage. Acceptable, borderline acceptable, and unacceptable conditions were indicated by the words slight and minor; moderate; and significant, major and extensive, respectively. In addition, photographs were used to document pre- and post- test conditions. The existing, 22,000-lb dolosse on the sea-side slope at sta 19+50 proved to be unstable for the wave conditions of Hydrograph I (maximum wave height of 24.5 ft) while the remainder of the breakwater cross-section (concrete crown cap and 16,000- to 20,000-lb stone on harbor-side slope) proved to be stable for the selected test condition. The stability of the existing breakwater cross-section at sta 14+00 proved to be dependent upon how tight a keyed and fitted construction of the 16,000- to 20,000-lb armor stone actually exists on the crown and harbor-side slope. Model test results using Hydrograph II (maximum wave height of 22.5 ft) indicated that if a tight, keyed and

Markle, D.G.
Herrington, C.R.

Nawiliwili breakwater stability study. (continued)

fitted construction exists, the breakwater at sta 14+00 is an adequate design, but if this tight construction does not exist, the crown and harbor-side slope could sustain severe damage and this damage could result in the undermining and displacement of existing dolosse (22,000-lb) along the breakwater crown. A concrete rib cap and one layer of uniformly placed, 13,000-lb tribars proved to be adequate rehabilitation designs for the crown and harbor-side slope, respectively, at sta 14+00. The existing 14,000- to 24,000-lb random-placed armor stone on the sea-side slope, and 16,000- to 20,000-lb keyed and fitted armor stone on the crown and harbor-side slope at sta 10+00 proved to be adequate designs for the selected test conditions of Hydrograph III (maximum wave height of 11.6 ft).

BREAKWATER
EXPERIMENTAL
LABORATORY
MODEL

STABILITY

Morison, J.R. Model study of wave action on underwater barriers.

Institute of Engineering Research, University of California, Berkeley.
Report No. HE 116-304.

Laboratory experiments on transmission of regular waves across impermeable, rectangular, submerged structures on horizontal and 1:20 sloped bottoms are discussed. Test variables were wave height, H; wave period (wavelength), T (L); water depth, d; structure width, W; and structure height, B. Data analysis is made in terms of the non-dimensional parameters H/L, W/H, d/H, d/L, and d/B. For the horizontal bottom, the experimental ranges of these parameters were H/L= 0.0195-0.0928; W/H= 0.84-6.45; d/H= 2.19-6.58; d/L= 0.096-0.388; and d/B= 0.9-2.3. For the sloping bottom, the test ranges were H/L= 0.070-0.12; W/H= 0.92-3.33; d/H=1.50-2.81; d/L= 0.139-0.208; and d/B= 1.0-2.2. For the latter type of tests, the barrier was tested at different distances offshore of the breaker plunge point. The parameter W/L is not examined for either type of test, but d/B vs. the transmission coefficient, Kt, is plotted for different values of W/H, holding H/L, d/H and d/L, constant, so the influence of W/L can be deduced. Other graphical presentations include, d/B vs. Kt for different values of H/L and d/L, holding W/H and d/H constant; d/B vs. Kt for different distances to the plunge point, where the distance is given in one-quarter multiples of L; and Kt vs. H/L for different values of d/H. Not all of the experimental data are presented, but from the data shown certain trends are obvious:
1) Kt increases as d/B increases;
2) Kt decreases as W/H and W/L increase, if everything else remains constant; and
3) Long waves over a low barrier approach 100 percent transmission.

BREAKWATER
SUBMERGED
EXPERIMENTAL
LABORATORY

IMPERMEABLE

Nagai, K. Diffraction of the irregular sea due to breakwater.

Coastal Engineering in Japan. Vol. 15. 1972. pp. 59-67.

Diffraction of a fully developed sea specified by the Pierson-Moskowitz spectrum and the SWOP directional function is examined with respect to semi-infinite breakwaters and breakwaters with gaps. Linear superposition of the diffraction caused by the different monochromatic spectral components is applied in order to obtain the diffraction diagram for the directional sea. Theoretical results are presented without comparison to laboratory or field data. Results for the case where the dominant wave direction is normally incident are presented for both structures; results for 45 degree incident waves are given for the gap breakwater. In general, the model predicts substantially higher diffraction coefficients for the directional, irregular spectrum than for the monochromatic wave field and their distributions are quite different. The percent difference between the two fields increases as distance from the breakwater tip increases. The change in significant wave period behind the structures is at most 15 percent, and the effect of water depth on the diffraction coefficient is only slight. The author also examines the sensitivity of his results to the number of frequency and directional components, thus allowing for substantial reductions is computation time by the judicious choice of number of frequency and directional components.

BREAKWATER
THEORETICAL
PERFORMANCE
THREE-DIMENSIONAL

Nakamura, M.
Shiraishi, H.
Sasaki, Y.

Wave damping effect of submerged dike.

Proceedings of the 10th Coastal Engineering Conference. Tokyo, Japan. September 1966. pp. 254-267.

Laboratory experiments designed to investigate wave transmission across a submerged, rectangular, impermeable dike on a horizontal bottom are discussed. Model test variable include water depth (d= 40-70 cm), incident wave height (H= 3-25 cm), incident wave period (T= 1.00-2.50 sec), structure height (D= 40-70 cm), and structure width (W= 0-4.0 cm). Both breaking and non-breaking waves are considered. Incident wave characteristics are converted to deep water conditions using small amplitude wave theory. Measured parameters include transmitted wave height, length, and period, and for the case of breaking waves, the distance from the inshore edge of the breakwater to the point where the wave reforms. Wave breaking is related to the dimensionless variables d/W, Ho/Lo, and d/Lo. Design curves for predicting wave transmission are provided for both breaking and non-breaking conditions.

BREAKWATER
SUBMERGED
EXPERIMENTAL
LABORATORY

PERFORMANCE
IMPERMEABLE

Numata, A.

Laboratory formulation for transmission and reflection at permeable breakwaters of artificial blocks.

Coastal Engineering in Japan. Vol. 19. 1976. pp. 47-58.

Dimensionless empirical formulas of wave transmission through and reflection from permeable breakwaters are presented. The relatonships account for the effects of incident wave characteristics, structure width, and rock size. Reflection coefficients also include the effect of water depth. Data are obtained from laboratory tests by the author on rectangular and trapezoidal breakwaters constructed of artificial blocks, and tests of other investigators on rubblemound rectangular structures. The fomulas obtained from the author's experiments give good agreement where the ratio of water depth to wavelength is less than 0.25, and there is no overtopping. It was also observed in the author's tests that the scale effect is on the same order as the scatter of the experimental data.

BREAKWATER
EXPERIMENTAL
LABORATORY
PERFORMANCE
PERMEABLE

Sawaragi, T.
Iwata, K.

Effect of structural shape on wave run-up and wave damping.

Coastal Engineering in Japan. Vol. 13. 1970. pp. 55-74.

Experiments to study wave runup and damping of regular waves due to 1) single large piles (islands) of different shapes and 2) triangular and rectangular pile groups composed of smaller circular piles are discussed. Theoretical expressions for reflection, transmission and energy loss due to single and double rows of piles are derived and compared to experimental results. Theory is based on small amplitude, shallow water wave theory (kh <1). Triangular, square, and circular island shapes were tested. Circle diameter and side measurement of the triangle and square were 60 cm. Tests were conducted in a 16.8 m wide by 0.45 m deep by 19.2 m long tank. Islands were fronted by a 1:40 slope and water depths ranged from 3.85 to 7.95 cm. Incident wave steepness, H/L, ranged from 0.030 to 0.088. Island orientation with respect to the incident waves had a large effect on the maximum runup. In general, more streamlined configurations produced less runup. A triangle pointed into the flow gave the smallest values of maximum runup. Pile group performance for different wave conditions was evaluated with respect to pile diameter, pile spacing, and number of piles. Triangular groups were tested at orientations pointing into and away from the flow. Rectangles were situated with a flat side facing the flow. Tests were conducted in a 70 cm wide by 95 cm deep by 30 m long wave flume, and piles were fronted by a 1:7 slope. Triangular pile groups were tested in water depths of 6 to 9 cm, and with a wave height and period of 5.8 cm and 0.65 sec, respectively. Pile diameters of 1.80 and 4.25 cm were used, and the ratio of gap width to diameter, b/D, varied from 2.52 to 15.66. Rectangles were tested in 15 cm of water with constant incident wave height of 4.0 cm and two wave periods, 0.65 and 1.30 sec. Pile diameter was 4.25 cm and b/D ranged from 0.65

Sawaragi, T.
Iwata, K.

Effect of structural shape on wave run-up and wave damping (cont).

to 3.24. In general, it was observed that as the number of rows increased so did energy loss and maximum runup. Within the range of b/D tested, theory and experiment agreed fairly well. Unfortunately, however, the range of b/D in which the reflection and transmission coefficients are predicted to change most rapidly (b/D < 0.65) was not tested.

PILES-GROUPS,SINGLE
THEORETICAL
EXPERIMENTAL
PERFORMANCE

WAVE RUNUP

Sawaragi, T.
Iwata, K.

On wave deformation due to permeable structures.

Coastal Engineering in Japan. Vol. 16. 1973. pp. 107-122.

Analytically derived expressions for reflection and transmission due to any number of rows of permeable walls are compared to experimental results. Types of structures considered include one to five rows of cylindrical piles and a permeable quay consisting of one perforated wall a distance, x, in front of an impermeable wall. Derivation of an expression for water surface elevation at the impermeable wall is also given. The derivations assume that small amplitude shallow water wave theory is valid and that the walls are inflexible. Theory predicts that the minimum reflection and transmission coefficients, and hence maximum energy dissipation, occur when the spacing between the walls is (2n+1)/4 times the wavelength, L, and (n= 0, 1, 2 ...). Conversely, dissipation is minimum when x equals even half-multiples of L. Piles were tested for wave steepness, H/L, ranging from 0.008 to 0.06 and x/L from 0.116 to 1.00. The ratios of pile diameter and water depth to waveheight, and the spacing between individual piles were also varied. Agreement of the theory with measured reflection coefficient, Kr, was generally better than with measured transmission coefficient, Kt, particularly as the number of rows increased. In the case of the quay, Kr and Kt are functions of porosity and friction and discharge coefficients of the wall in addition to the incident wave conditions. It should be noted that the permeable wall consisted of a series of horizontal layers separated by a gap of width D. Test variables included approach slope (0 and 1:15); wave steepness (H/L=0.01 to 0.04); ratio of structure width to gap width (b/D=0.366 to 1.36); ratio of water depth to wavelength (h/L=0.0261 to 0.2838); ratio of water depth to incident wave height (h/H=0.5 to 10.25); ratio of gap width to wave height (D/H= 0.26

Sawaragi, T.
Iwata, K.

On the wave deformation due to permeable structures (cont).

to 1.18); relative spacing (x/L= 0.044 to 0.750); and porosity (m= 10 to 45 percent). Results indicate that the relationship between test variables, empirical coefficients, and Kr and Kt is complex. The proposed theory predicts the trend of Kr, Kt, and wave amplitude at the wall fairly well, but actual values in some cases may vary slightly, particularly at the maximums and minimums. Improved knowledge of the empirical coefficients is desirable.

BREAKWATER
QUAY
THEORETICAL
EXPERIMENTAL

PERFORMANCE
PERMEABLE

Sawaragi, T.
Iwata, K.

A study on the wave absorbing effect of a vertical permeable breakwater quay for irregular waves.

Transactions of the Japan Society of Civil Engineers. Vol. 9. 1977. pp. 147-149.

A theoretical approach to determining the reflection and transmission characteristics of a permeable vertical quay located a distance, l, in front of an impermeable vertical wall is presented. The irregular wave train is expressed as a sum of monochromatic, cosinusoidal waves, and each elementary wave is assumed to reflect independently between the permeable quay and the impermeable wall. Analytical expressions are given for the wave profile in front of the breakwater and the water surface elevation at the impermeable wall in terms of the reflection and transmission coefficients of the permeable wall; phase lags of the reflected and transmitted waves at the permeable wall with respect to the incident wave; wave number; and wave frequency. Experiments were conducted with one-, two-, and three-component waves and irregular waves. Experimental and theoretical results for the maximum wave height distribution in front of the quay compare reasonably well for three monochromatic waves and a component wave consisting of the sum of these three waves. Additional data analyses show the theoretical and actual wave height ratios, e.g.; ratios of maximum wave height between the walls and at the impermeable wall to the maximum incident wave height; for monochromatic, three-component, and irregular waves. These data do not seem to agree very well with theory except in very limiting cases. In addition, there are not enough data points to adequately define the theoretical curve, so it is difficult to draw any conclusions with respect to this portion of the analysis.

BREAKWATER
THEORETICAL
EXPERIMENTAL
PERFORMANCE
PERMEABLE

Sawaragi, T.
Iwata, K.

Wave attenuation of a vertical breakwater with two air chambers.

Coastal Engineering in Japan. Vol. 21. 1978. pp. 63-74.

The wave attenuating capabilities of a structure composed of two permeable walls in front of an impermeable wall are discussed. Reflection coefficients and water surface fluctuations at the impermeable wall are derived theoretically by the linear superposition of reflected waves. Analysis assumes that the vertical walls are stationary; linear superposition is valid; the incident wave reflects and infinite number of times between the first and third walls; and the same reflection and transmission coefficients and phase lag attributed to each wall are the same at every point in the analysis for a given incident wave. Numerical results showing the influence of the individual reflection coefficients of the permeable walls; the distance between the walls relative to the wavelength; and the phase change caused by transmission through the permeable wall on the overall reflection and water surface at the wall are presented. Interaction between these factors is extremely complicated. A comparison to the wave attenuation by a one chamber system shows that the two chamber system is a much better wave attenuator. Optimum combinations of governing factors are calculated for both systems. Performance trends predicted by theory were well-substantiated by experimental results.

QUAY
THEORETICAL
EXPERIMENTAL
LABORATORY
PERFORMANCE
PERMEABLE
IMPERMEABLE

Seelig, W.N. Effect of breakwaters on waves: laboratory tests of wave transmission by overtopping.

Proceedings of Coastal Structures '79. Alexandria, Virginia. March 1979. pp. 941-961.

Both regular and irregular waves were used to investigate wave transmisison by overtopping of a smooth, impermeable, trapezoidal breakwater 75 cm high with a 1 on 1.5 slope. For regular waves, depths tested ranged from 45 to 90 cm with relative depths (d/gT^2) of 0.0065 ,0.016, and 0.05; wave steepness (H/gT^2) from 0.0001 to 0.01; and wave periods from 1.05 to 3.42 sec. Irregular wave tests were conducted at depths of 60 and 75 cm, with relative depths from 0.0005 to 0.04; wave steepness from 0.0007 to 0.008; and spectral peakedness from 2.2 to 5.4. Relative depth and wave steepness were calculated using the period of peak energy in the spectrum. The method of Goda was used to resolve incident and reflected waves. Test results are used to show that incident wave height and breakwater freeboard are the most important parameters controlling the amount of wave transmission. Other spectral factors have minor influence on wave transmission, and the breakwater produced only small changes in the spectral shape, even when the coefficient of transmission is low.

BREAKWATER
SUBMERGED/SUBAERIAL
EXPERIMENTAL
LABORATORY
PERFORMANCE
IMPERMEABLE

Seelig, W.N. Two-dimensional tests of wave transmission and reflection characteristics of laboratory breakwaters.

U.S. Army Corps of Engineers Coastal Engineering Research Center. Technical Report 80-1. 1980. 187 pp.

Monochromatic and irregular wave transmission and reflection measurements were made for various subaerial and submerged trapezoidal breakwater cross sections. These two-dimensional laboratory tests included smooth impermeable breakwaters, rubble-mound breakwaters, and breakwaters armored with dolos units. Wave transmission by overtopping was found to be related to breakwater freeboard, wave runup, and breakwater crest width; a method of estimating transmission by overtopping coefficients is presented. The Madsen and White (1976) numerical procedure was found to be an important tool for predicting the amount of transmission through permeable breakwaters. Suggested procedures for estimating transmission coefficients have been incorporated into the computer programs OVER and MADSEN (included as appendices) and these programs may be used to predict wave transmission coefficients for nonbreaking, breaking, monochromatic, and irregular wave conditions.

BREAKWATER
EXPERIMENTAL
LABORATORY
PERFORMANCE
PERMEABLE

Shiraishi, N.
Numata, A.
Hase, N.

The effect and damage of submerged breakwater in Niigata Coast.

Coastal Engineering in Japan. Vol. 3. 1960. pp. 89-99.

The performance of a low-crested breakwater located 500 m offshore of Niigata Coast in Japan is discussed with respect to wave transmission, bottom profile changes, shoreline recession rates, and stability. Prior to construction of the breakwater the coastline had been eroding at an average rate of 6 m per year for the past 50 yr. Beach protection was initiated in 1945 following an event that resulted in severe damage to the coast. Protection along an experimental section of coast consisted of a combination of groins and a breakwater. Construction of the entire 2.3 km breakwater began in 1952 and was completed in 1955. Along its length, the breakwater is connected to the shore by a system of groins so that longshore sand transport is restricted. Pertinent observations about breakwater performance include:
1) Wave height attenuation by the breakwater ranged from 30 to 70 percent;
2) Transmission decreased with increasing wave steepness;
3) Average depth behind the structure decreased in the two years following construction, and the shoreline advanced;
4) Shoreline recession rates increased along adjacent shorelines; and
5) Porous construction with a deep pile foundation showed the least amount of slumpling (most stable).

BREAKWATER
EXPERIMENTAL
FIELD
PERFORMANCE
STABILITY
MONITORING

Sollitt, C. K.
Cross, R. H.

Wave reflection and transmission at permeable breakwaters.

Massachusetts Institute of Technology,
Department of Civil Engineering.
Report No. 147. March 1972. 235 pp.

An analytical solution to the redistribution of wave energy at permeable, non-overtopped breakwaters is presented. Particular consideration is given to energy transmission through the interstices of structures commonly regarded as impervious. Three breakwater configurations are examined: rectangular with homogeneous rock fill; trapezoidal with layered fill; and vertical pile arrays placed in symmetric patterns. The theoretical development begins with the unsteady equations of motion for flow in the voids of a porous medium. The equations are linearized using a technique which approximates the known turbulent damping condition inside the structure. This gives a potential flow problem that is satisfied by an eigen series solution. Incident waves are assumed to be monochromatic and to approach normal to the structure. Linear wave theory is applied outside the structure. The internal, reflected, and transmitted wave amplitudes are determined by matching the general solutions at the sea-breakwater interfaces and requiring continuity of pressure and horizontal mass flux. Energy dissipation due to wave breaking on the sloped face of the trapezoid is accounted for with a semi-empirical formula that was adapted from Miche's work. Application of the theory to a trapezoid is simplified by reducing the trapezoid to an equivalent rectangle, where the submerged volume of the trapezoid and the rectangle are equal. Laboratory tests were conducted to check the validity of the analytical models. All three types of structures were tested, and results are favorable for all of them. It is suggested, however, that the general validity of the breaking wave analysis for the trapezoidal structure needs to be established. In general, it was found that the transmission coefficient, Kt, decreases as structure

Sollitt, C. K.
Cross, R. H.

Wave reflection and transmission at permeable breakwaters (cont).

porosity and permeability decrease, and wave steepness and structure width increase. Exactly opposite trends are observed for the reflection coefficient, Kr. Correlation between experimental and theoretical results is best when the incident wave height is greater than the median stone diameter. Also, theory predicts Kt better then Kr, apparently because of the sensitivity of Kr to surface effects.

BREAKWATER
THEORETICAL
PERFORMANCE
PERMEABLE

Sollitt, C.K.
Cross, R.H.

Wave reflection and transmission at permeable breakwaters.

U.S. Army Corps of Engineers Coastal Engineering Research Center. Technical Paper 76-8. 1976. 172 pp.

See Sollitt and Cross (1972)
The objective of this study was to develop a theory to predict wave energy transmission through the interstices of structures. The results are intended for use by coastal engineers to compare the effectiveness of alternative breakwater configurations, independent of repetitive experimental programs.
Three breakwater configurations are considered: 1) crib-style breakwaters with vertical walls and homogeneous fill; 2) conventional trapezoidal-shaped breakwaters with layered fill; and 3) pile-array breakwaters composed of vertical piles placed in symmetric patterns. The two-dimensional problem is studied. Waves are assumed to arrive at normal incidence.
The theoretical development begins with the unsteady equations of motion for flow in the voids of an arbitrary porous structure. The equations are linearized using a technique which approximates the known turbulent damping condition inside the structure. This yields a potential flow problem satisfied by an eigen-series solution. Linear wave theory is assumed to apply outside the structure and the excitation is provided by a monochromatic incident wave. Continuity of pressure and horizontal mass flux is required at the sea-breakwater interface. Inclusion of sloping-face structures necessitates and estimation of the breaking losses incurred on the windward slope. A semi-empirical method is used to approximate the effect of wave breaking.
An experimental program is conducted to verify the analytical models. Theory and experiment yield the following general conclusions: 1) the transmission coefficient decreases with decreasing wavelength, breakwater porosity and permeability, and increasing wave-height and breakwater width; 2) the reflection coefficient decreases with increasing

Sollitt, C.K.
Cross, R.H.

Wave reflection and transmission at permeable breakwaters. (continued)

wavelength, breakwater porosity and permeability, and decreasing breakwater width. Application of the theory is limited to wave heights which exceed the medium grain diameter. Experimental results correlate better with the theoretical transmission coefficient than with the reflection coefficient. This seems to be due to the sensitivity of the reflection coefficient to surface effects. The theory provides useful design estimates for all three breakwater configurations and a full range of wavelengths. The proposed wave-breaking calculation gives favorable results for the sloping-face structure tested in this study. However, further comparison is needed to establish the general validity of the wave-breaking analysis.

BREAKWATER
THEORETICAL
PERFORMANCE
PERMEABLE

University of
Florida Department
of Coastal and
Oceanographic
Engineering

Coastal engineering evaluation of planned offshore breakwater at Broward County Beach.

University of Florida Department of Coastal and Oceanographic Engineering. Report submitted to the Board of County Commissioners, Broward County, Fort Lauderdale, Florida. 1971. 16 pp. + appendices.

Model tests to determine the quantity of littoral drift for various crest elevations of an offshore breakwater at the Broward County Beach were conducted. Wave heights on the lee side of the breakwater were measured and by theoretical and empirical methods, the capacity of sand transport for each breakwater height was determined. Four specific cases were tested in detail, including sand transport capacity without a breakwater, ie, existing conditions, and sand transport with breakwater crest elevations or +2, -2, and 0 ft above mean sea level. The range of wave heights and water levels in the model tests covered the corresponding parameters in nature. The model test results indicated a sand transport capacity of 148,000 cubic yards per year to the south without the breakwater. The Corps of Engineers estimates the annual drift for this region to be about 153,000 cy and Per Bruun estimates 150,000 cy. The estimated transport rates with a breakwater in place are 62,000; 74,000; and 80,000 cy/yr for crest elevations of +2, 0, and -2 ft above mean sea level, respectively. The results of the tests when plotted on semi-log paper approximate a straight line, therefore reliable interpolations can be made to obtain the quantity of littoral drift for any breakwater height within the limits of the tests.

BREAKWATER
EXPERIMENTAL

SAND TRANSPORT

LABORATORY
MODEL

Vincent, C.L. A method for estimating depth-limited wave energy.

U.S. Army Corps of Engineers Coastal Engineering Research Center. Technical Aid 81-16. 1981. 22 pp.

A procedure for estimating the upper limit of wind wave energy in finite water depth is given. The depth-controlled wave height, H, is calculated as four times the square root of the area under the energy density spectrum. The form of the spectrum is a function of depth, frequency, and a spectral constant, A, that is determined from the wind speed and fetch length. A simplified method for finding H in the shallow water limit is given, including example problems, where the lower cutoff frequency is calculated as 0.9 times the peak frequency and A is estimated from Hasselman et al. (1973). In this limit, H is proportional to the square root of depth. The method is recommended for storm seas (sharply peaked spectra) and not for swell. See the abstract for Vincent (1982) for a more complete description of theoretical background.

SPECTRAL ENERGY
SHALLOW WATER
THEORETICAL

Vincent, C.L. Depth-limited significant wave height: A spectral approach.

U.S. Army Corps of Engineers Coastal Engineering Research Center. Technical Report 82-3. 23 pp.

A method for predicting the upper limit on wind wave energy in finite water depths is presented. The theoretical background and supporting field data are given. Phillips (1958) found that the upper bound on energy density in deep water is proportional to f^(-5). Based on Phillips' work, Kitaigorodskii, Krasitskii, and Zaslavskii (1975) derived a depth-dependent form for the energy density spectrum. In the shallow water limit, the bound on energy density is proportional to f^(-3) and linearly proportional to depth. The proportionality constant is not considered to vary as a function of wave conditions. Data presented in this report suggest that the proportionality function is not a universal constant, but varies with the wind speed and fetch length. The evidence presented also suggests that in very shallow water other mechanisms such as refraction, bottom friction, and strong breaking dominate the spectral shape near the peak. The equation for energy density is, however, conservative in these cases. In a manner analagous to deep water wave spectra, the depth-limited wave height is estimated to be four times the square root of the area under the energy density spectrum. It is also shown that in shallow water, the depth-limited significant wave height is proportional to the square root of depth. In contrast, the monochromatic depth-limited wave height varies linearly with depth.

SPECTRAL ENERGY
THEORETICAL
SHALLOW WATER

Wada, A. On the disturbed waves in a bay sheltered by breakwater.

Coastal Engineering in Japan. Vol. 7. 1964. pp. 31-44.

An analytical solution to the problem of wave sheltering effects of a semi-infinite barrier parallel to an infinite coastline for very long period waves is presented. The boundary value problem is expressed as Wiener-Hopf integral equations and solved using the method of D.S. Jones. Constant depth in the basin and small amplitude, long wave, linear wave theory are assumed. Long period solitary waves are also discussed. Sinusoidal wave calculations are made for angles of incidence t, between 0 and 150 degrees, where a direction of wave propagation at zero degrees opposes the end of the semi-infinite barrier. Wavelengths, L, of 6, 10, 20, 30, 40, and 45 km were used. The separation distance between the two barriers, b, and incident wave height remained constant at 3 km and 1 m, respectively. Theoretical results show that wave amplification may occur behind the structure, particularly for low angles of incidence. Transmission ratio, K, exceeded 1.0 for all conditions of t < 40 deg. K was always greater than 1.0 for the three longest waves. In general the barrier is expected to have a sheltering effect for b/L < 0.15 and normally incident waves. No experimental verification of results was made.

BREAKWATER
THEORETICAL
PERFORMANCE
OBLIQUE WAVES
LONG WAVES

Wiegel, R.L. Transmission of waves past a rigid vertical thin barrier.

ASCE Journal of the Waterways and Harbors Division. Vol. 86, No. WW1. March 1960. pp. 1-12.

A theoretical solution for the transmission of energy across a thin, vertical barrier extending from above the water level to some distance below the surface is developed based on small amplitude, linear wave theory and considerations of wave power transmission. It is assumed that all wave transmission is due to power transmission through the gap under the barrier, e.g., total reflection occurs at the barrier face. Results from both the power transmission theory and Ursell's deep water theory are compared to laboratory data. Experiments were conducted in a 1 ft wide by 3 ft deep by 106 ft long wave tank; in water depths of 1.53 and 2.01 ft; for wave steepness, H/L, ranging from 0.0055 to 0.082; and gap widths of about 5 to 95 percent of still water depth. Experimental results are consistent with theory in that transmission decreases as wave steepness increases. For deep water conditions, Ursell's theory predicts transmission better for small gaps, while the power transmission theory works better for large gaps. In general, the theory presented is adequate for limited engineering purposes, but improvements are desirable.

BREAKWATER
VERTICAL, THIN
THEORETICAL
PERFORMANCE
IMPERMEABLE