CHAPTER ONE HUNDRED SEVENTY EIGHT

Reef Type Breakwaters

John P. Ahrens, Aff. M. ASCE*

Introduction

Reef type breakwaters refers to a low-crested rubble mound break-water without the traditional multilayer cross section. This type of breakwater is little more than a homogenous pile of stones with indivi-dual stone weights sufficient to resist wave attack.

In recent years a number of low-crested breakwaters has been built or considered for use at a variety of locations. Most of these struc-tures are intended to protect a beach or reduce the cost of beach main-tenance. Other applications include protecting the water intakes for power plants, the entrance channel for small boat harbors, and providing an alternative to revetment for stabilizing an eroding shore line. In situations where only partial attenuation of the waves on the leeside of the structure is required, or possibly even advantageous, a low-crested rubble mound breakwater is a logical selection. Since the cost of a rubble mound increases rapidly with the height of the crest, the economic advantage of a low-crested structure over a traditional breakwater that is infrequently overtopped is obvious. Because the reef type breakwater represents the ultimate in design simplicity it could be the optimum structure for many situations. Unfortunately, the performance of low-crested rubble mound structures, and particularly a reef type breakwater, is not well documented or understood.

Background and Objectives

A number of papers have noted that the armor on the landside slope of a low-crested breakwater is more likely to be displaced by heavy over-topping than the armor on the seaward face, Lording and Scott (1971), Raichlen (1972), and Lillevang (1977). Raichlen discusses the character-istics of the overtopping over the crest and the inherent complexity of the problem. Walker, et al. (1975) give a carefully reasoned discussion of the many factors influencing the stability of heavily overtopped rub-ble mound breakwaters. Walker, et al. also show a figure which suggests what armor weight might be required for stability on the beachside of a low-crested breakwater. Unfortunately, the data scatter shown in the figure undermines confidence in the suggested armor weights.

In Australia the breakwater at Rosslyn Bay was damaged severely during cyclone "David" in 1976, Bremner, et al. (1980). The crest height of the structure was reduced as much as four meters but still functioned effectively as a submerged breakwater for over two years until it was

*Oceanographer, Coastal Engineering Research Center, U. S. Army Engineer Waterways Experiment Station, P. O. Box 631, Vicksburg, MS 39180

repaired. Based on the surprisingly good performance of the damaged
Rosslyn Bay breakwater and the findings from model tests, a low-crested
design was chosen for the breakwater at Townsville Harbor, Australia.
This breakwater is unusual because it was built entirely of stone in the
3- to 5-ton range, Bremner et al. Reef breakwaters, as described in this
paper, are very similar to the Townsville breakwater except a wider gra-
dation of stone was used in the model breakwater tests discussed herein.

    Seelig (1979) conducted an extensive series of model tests to deter-
mine the wave transmission and reflection characteristics of low-crested
breakwaters, including submerged structures. From these tests Seelig
concluded that the component of transmission due to wave overtopping was
very strongly dependent on the relative freeboard, i.e. the freeboard
divided by the incident significant wave height. Recent work by Allsop
(1983) with multilayered, low-crested breakwaters showed that the wave
transmission was strongly dependent on a dimensionless freeboard param-
eter which included the zero-crossing period of the irregular wave condi-
tions. Allsop did not find much wave period dependency in his evaluation
of breakwater stability but indicates that since the wave transmission
(which is largely due to overtopping) is dependent on the period, then
possibly the stability of the backface would also be a function of wave
period.

    A study currently being conducted at the Coastal Engineering
Research Center (CERC) is intended to document the performance of low-
crested breakwaters. This paper discusses laboratory model tests of reef
type breakwaters and provides information on their stability to wave
attack and the wave transmission and reflection characteristics of the
structures.

Techniques Used

    To date 205 two-dimensional laboratory tests of reef breakwaters
have been completed. These tests were conducted in a 61-centimeter-wide
channel within CERC's 1.2- by 4.6- by 42.7-meter wave tank (see Figure 1
for a plan view of the tank and test setup).

    All tests were conducted with irregular waves. The spectra used
have wave periods of peak energy density, T_p , ranging from about 1.45
to 3.60 seconds. The signals to control the wave blade were stored on
magnetic tape and were transferred to the wave generator through a data
acquisition computer system (DAS). For this study there were four files
on the tape which would produce a distinct spectrum for each file.
Table 1 gives the nominal period of peak energy density for each file.

Tape File	Approximate T_p (sec)
1	1.45
2	2.25
3	2.86
4	3.60

Table 1. Period of peak energy density for each file.

SCALE

0   1   2   3   4   5 (m)

o DENOTES WAVE GAGE LOCATION

WALL OF WAVE TANK

TO WAVE
GENERATOR,
19 m

1 ON 15

GRAVEL WAVE ABSORBER
BEACH

GRAVEL WAVE ABSORBER BEACH

61 cm

SHOALING SLOPE   oo   o

PONDING RELIEF CHANNEL

WAVE
ABSORBER
MATERIAL

TRAINING
WALLS

REEF
BREAKWATER

PONDING RELIEF CHANNEL

GRAVEL WAVE ABSORBER BEACH

GRAVEL WAVE
ABSORBER BEACH

AUXILIARY CHANNEL

GRAVEL WAVE ABSORBER BEACH

WALL OF WAVE TANK

PLAN VIEW

Figure 1.   Plan view of wave tank and test setup.

TYPICAL DAMAGE PROFILE
TEST 37, H_s = 13.76 cm, T_p = 3.00 SEC

LANDSIDE   DAMAGE AREA   SWL   SEASIDE   ELEVATION ABOVE PLATFORM, CM

INITIAL PROFILE

h_c h'_c ACCRETION AREA   DAMAGE PROFILE d_s

ACCRETION AREA

CONCRETE PLATFORM

120   110   100   90   80   70   60   50   40   30   20   10   0

DISTANCE ALONG CHANNEL, CM
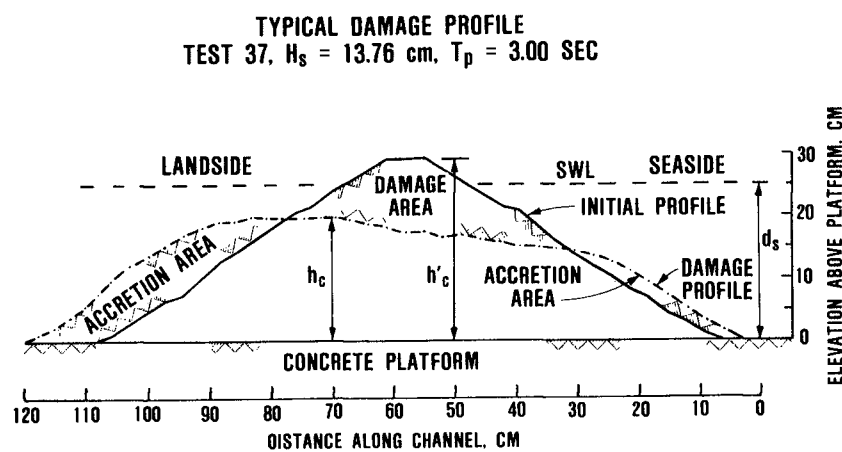
Figure 2.   Cross sectional view of the initial
            and damaged breakwater profile.

If there were no attenuation of the signal to the wave generator the
files used were intended to produce a saturated spectrum at all
frequencies above the frequency of peak energy density for the water
depth at the wave blade. For frequencies lower than that of the peak,
the energy density falls off rapidly. This procedure produced a spectrum
of the Kitaigorodshii type as described by Vincent (1981). The amplitude
of the signal to the wave generator could be attenuated by a 10-turn
potentiometer in a voltage divider network which allowed control of the
wave heights generated. In addition, the waves were generated in a water
depth 25 centimeters greater than at the breakwater and shoaled to the
water depth of the structure over a 1 on 15 slope (see Figure 1). This
setup ensures that very severe conditions can be developed at the struc-
ture site. Incident zero-moment wave heights ranged from about 1 to 18
centimeters.

     Three parallel wire resistance-type wave gages were used in front of
the breakwater to resolve the incident and reflected wave spectrum using
the method of Goda and Suzuki (1976). Two wave gages were behind the
structure to measure the transmitted wave height. The location of the
gages is shown in Figure 1. During data collection the gages were sam-
pled at a rate of 16 times per second for 256 seconds by the same DAS
which controlled the wave generator motion.

     Two distinct types of model tests were conducted during this study.
They will be referred to as "stability" tests and "previous damage"
tests. For a stability test the following test sequence was used:

          1. Rebuild the breakwater from the previously damaged condition.
          2. Survey the breakwater to document its initial condition.
          3. Calibrate the wave gages.
          4. Select the wave file and signal attenuation setting.
          5. Start the wave generator and run waves.
          6. Collect wave data (several or more times).
          7. Stop the wave generator.
          8. Survey the breakwater to document its final condition.

The duration of wave action lasted from 1-1/2 hours for a test using the
File 1 spectrum to 3-1/2 hours for a File 4 spectrum. Generally, the
technicians observing the tests thought that most of the stone movement
occurred during the first 10 or 15 minutes of wave generation, so the
final survey is regarded as an equilibrium profile for the structure.
In rebuilding the breakwater the technicians rarely touched the stone
but merely pushed it around by foot until the shape conformed to the
desired initial profile. This procedure was a conscious effort to avoid
overly careful placement of the stone. Outlines of the desired initial
profile were fixed to the walls of the testing channel and a moveable
template was used to ensure that the initial profile was reasonably close
to the desired profile. Initial configuration of the breakwater for a
stability test is a narrow, trapezoidal shape with the seaward and land-
ward slopes of 1 on 1-1/2 (see Figure 2). Figure 2 also shows a typical
profile after moderately severe wave attack during a stability test.
Wave transmission and reflection also were measured during a stability
test.

Tests referred to as previous damage were conducted to answer the question of how the breakwater would perform for typical wave conditions after it had been damaged by very severe wave conditions. For previous damage tests there was very little readjustment of the damage profile from test to test since the breakwater is not rebuilt at the end of a test. There really was no stability information obtained from these tests and the duration of wave action is only half an hour, but wave transmission and reflection were measured. Previous damage tests had the following sequence of events:

1. Final survey of the breakwater for the last test which becomes the initial survey for this test.
2. Calibrate wave gages.
3. Select the wave file and signal attenuation setting.
4. Start generator and run waves for half an hour.
5. Collect wave data (two or three times).
6. Stop the wave generator.
7. Survey breakwater as noted above in Step 1.

All 205 of the completed tests of this study logically can be divided into 10 subsets or test series. Because of the test plan stability test series have odd numbers and previous damage test series have even numbers. Table 2 gives the basic information about each subset.

Subset No.	No. of Tests	Water Depth (cm)	Crest Height as Built (cm)	Median Stone Weight (gr).	Area of Breakwater Cross Section (cm^2)
1	27	25	25	17	1170
2	3	25	NA	17	1170
3	29	25	30	17	1560
4	12	25	NA	17	1560
5	41	25	35	17	2190
6	11	25	NA	17	2190
7	38	25	32	71	1900
8	26	25	NA	71	1900
9	13	30	32	71	1900
10	5	30	NA	71	1900

Table 2. Basic data for each subset.

Two different sizes of stone were used during this study. For subsets 1 through 6 an angular quartzite with a median weight of 17 grams was used and for subsets 7 through 10 a blocky to angular diorite was used. Table 3 summarizes the information about the stone used in this study.

Characteristic	Quartzite	Stone Type Diorite
Median weight (gr)	17.0	71.0
Density	2.63	2.83
W_{98}/W_2	3.9	10

Table 3. Stone and gradation characteristics.

   Table 4 provides a summary of the definitions for variables used in this study.

Symbol, Definition (units)

H_s , incident zero-moment wave height (cm)

H_T , zero-moment transmitted wave height (cm)

H_c , zero-moment wave height at transmitted gage locations with no breakwater in channel (cm)

T_p , wave period of peak energy density of spectrum (sec)

d_s , water depth at breakwater (cm)

L_p , Airy wave length calculated using T_p and d_s (cm)

W_{50} , median stone weight, subscript indicates percent of total weight of gradation contributed by stones of lesser weight (gr)

w_r , density of stone

w_w , density of water, tests conducted in fresh water w_w = 1.0

h_c , crest height of breakwater as built (cm)

h_c , crest height of breakwater after wave attack (cm)

A_t , cross sectional area of breakwater (cm^2)

A_D , area of damage to breakwater (cm^2)

K_R , reflection coefficient of breakwater as defined and calculated by method of Goda and Suzuki (1976)

Results

Stability. To prevent confusion it should be mentioned that the stability will be quantified by the damage or lack of damage during a test. For reef breakwaters, stability logically can be viewed from two perspectives. One perspective is volumetric damage. This type of damage is related to the number of stones displaced from their original location and possibly includes where the stones were deposited. Information regarding volumetric damage is important when considering maintenance requirements for the structure. The second aspect of stability is the reduction in the crest height of the breakwater due to wave action. Since the performance of a reef breakwater will be judged largely by its wave transmission characteristics, this aspect of stability is quite important because wave transmission is very sensitive to the crest height

of the structure relative to the water depth. Two dimensionless varia-
bles are used to define these two aspects of the stability of the break-
water; they are the relative crest height, h_c/d_s, and the dimensionless
damage, D' , which is defined as

$$D' = \frac{A_d}{(W_{50}/w_r)^{2/3}}$$

The relative crest height is the ratio of the equilibrium or stable crest
height to the water depth at the site for the given wave conditions and
D' is a measure of the number of stones removed from the damaged area.
Experience indicates that damage to a rubble mound breakwater will be
strongly dependent on Hudson's stability number N_s , Hudson (1959). The
stability number is defined for irregular waves as

$$N_s = \frac{H_s}{(W_{50}/w_r)^{1/3}\left(\frac{w_r}{w_w} - 1\right)}$$

In Figures 3a and 3b h_c/d_s and D' are plotted versus N_s for data of
subset 1. These data trends generally are what would be expected: as
the severity of wave attack increases, i.e. increasing N_s , the volume
of stone displaced increases at an ever-increasing rate and the height of
the breakwater decreases gradually. On the whole, the stability number
does a fairly good job of explaining the damage to the structure but
careful inspection of the data points in Figures 3a and 3b suggests that
there is a wave period effect. The effect indicated is that wave spectra
with large T_p's do more damage than spectra with small T_p's , other
factors being equal. This finding is consistent with the findings of
Gravesen, et al. (1980) who have conducted an extensive study of the
stability of rubble mound breakwaters to irregular wave attack. The
breakwaters studies by Gravesen, et al. are considerably different from
reef breakwaters in that they are high multilayered structures with
large concrete caps designed so the structure is rarely overtopped.
Gravesen's, et al. work suggested that a modified stability number which
took into account the period of peak energy density of the spectrum might
describe damage to the breakwaters better than the traditional stability
number. Following Gravesen, et al. the spectral stability number , N_s^*
is defined

$$N_s^* = \frac{(H_s^2 L_p)^{1/3}}{(W_{50}/w_r)^{1/3}\left(\frac{w_r}{w_w} - 1\right)} \qquad \text{Eq. 1}$$

When the data shown in Figures 3a and 3b are replotted versus N_s^* in
Figures 4a and 4b there is a considerable reduction in the data scatter.
When the other stability data subsets are compared as in Figures 3 and 4
it is found that using N_s^* reduces the scatter in all subsets, eg. see
Figure 5 which shows the data for Subset 5.
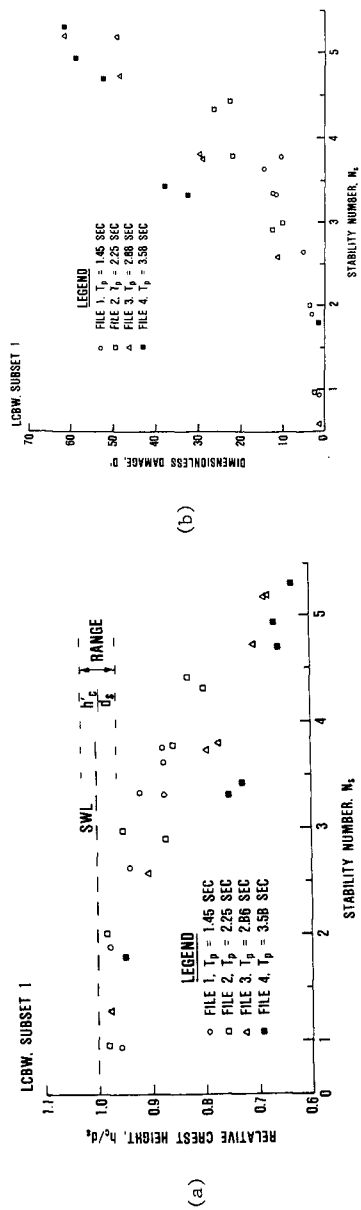
Figure 3. Damage response of breakwater as a function of stability number for data subset 1.
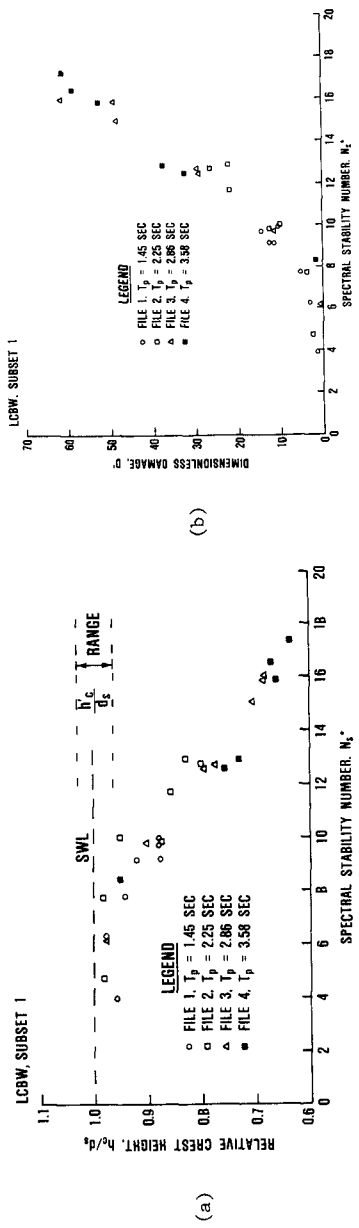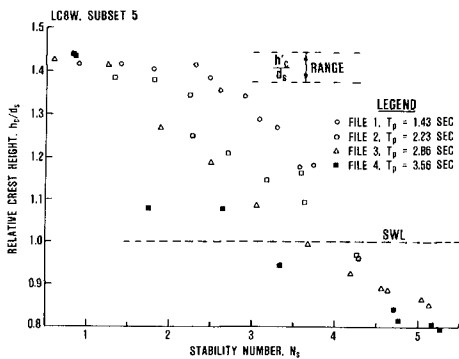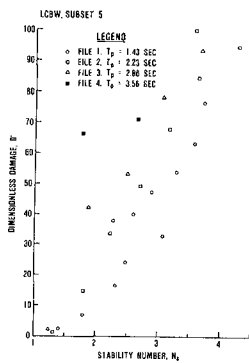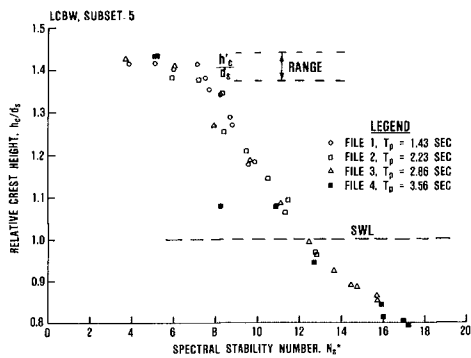
Figure 4. Damage response of breakwater as a function of the spectral stability number for data subset 1.
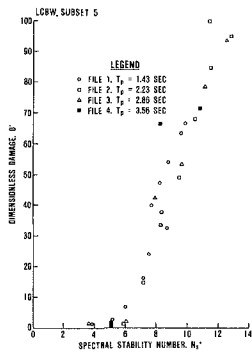
(a)

(b)

(c)

(d)

Figure 5. Damage response of breakwater for data
subset 5.

Although the reason is not entirely clear, it appears that $H_s^2 L_p$ is a better measure of wave severity than H_s^3, at least as far as reef breakwaters are concerned. Part of the reason may be due to relating damage to the zero-moment wave height rather than a larger wave in the height distribution. It is convenient to relate damage to the zero-moment wave height because of the statistical stability of the parameter and because of its relation to the area under the spectrum; however, it is commonly observed that the larger waves in the distribution move most of the stone. In addition, for a given water depth, the maximum stable wave height increases with period; therefore, for a fixed water depth and zero-moment wave height it would be the file with the longer period of peak energy density which would have the larger waves striking the breakwater. Another factor which might cause longer waves to be more damaging could be called the runup factor. Generally, on rough porous slopes, longer period waves have higher runup than shorter waves if other conditions are equal (see Figure 12 in Seelig 1980). A longer period and higher runup mean a greater return flow which can cause damage on a high structure and a greater volume and celerity of overtopping flow to cause damage on a low-crested breakwater.

Somewhat surprisingly, the classic picture of a large wave curling over and breaking on the structure was rarely observed during this study. Possibly because of the porous nature of the breakwater, waves appeared to be partly absorbed into the structure before they could break on it. Usually it was the drag forces caused by heavy overtopping flow which moved most of the stones rather than wave impact.

Since the relative height of the breakwater was so strongly dependent on N_s^* for all the stability subsets it is instructive to look at all five trends together in one Figure. Figure 6 shows these trends which indicate that there is little or no degradation of the breakwater for $N_s^* \leq 6$, but for the higher structures there is quite noticeable degradation for $N_s^* \geq 8$. Figure 6 also shows that the damage rate is reduced after the crest has been battered down to about the still water level by wave attack. Once the structure is submerged the overlying water provides considerable protection from further damage. This tendency has been observed in the field for a number of damaged breakwaters, Wiegel (1982).

Wave Transmission

For these tests the wave tranmission coefficient K_T is defined as:

$$K_T = \frac{H_t}{H_c}$$

Although this is not the most commonly used definition of K_T it has some advangates over the traditional definition which is given by the ration of H_T to H_s. The definition given above can be stated as
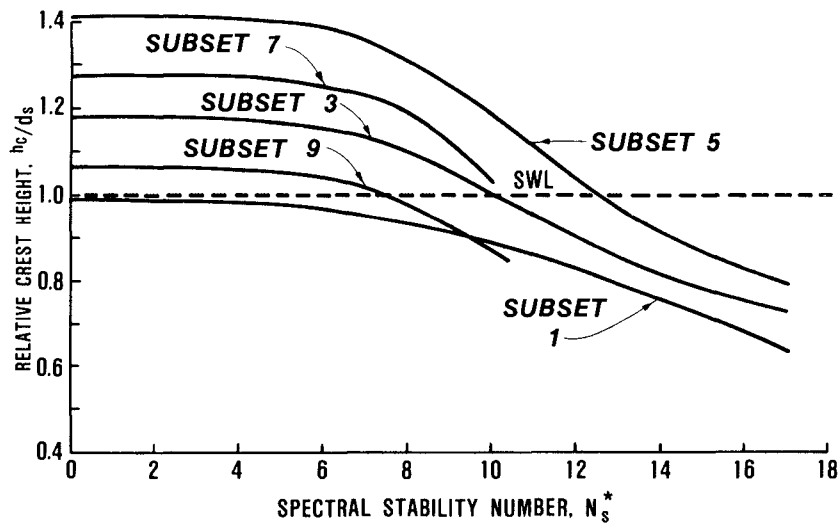
Figure 6. Damage trends as a function of the spectral stability number for data subsets 1, 3, 5, 7, and 9.
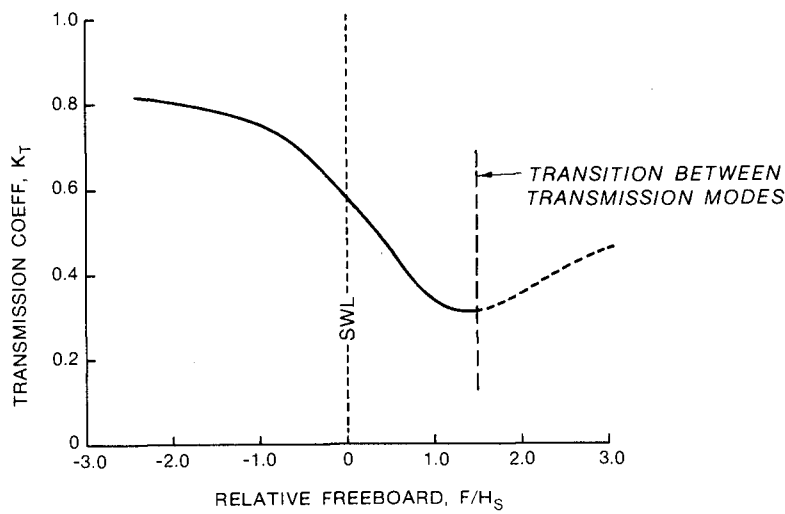
Figure 7. Wave transmission as a function of the relative freeboard.

the ratio of the transmitted wave height to the wave height which would
be observed at the same location without the breakwater in the channel.
This definition eliminates wave energy losses occurring between the in-
cident and transmitted gages in the absence of a breakwater in the test-
ing channel. These losses were observed to be considerable for the most
severe wave conditions during calibration of the channel. In effect K_T
measures the attenuation of wave energy due strictly to the presence of
the breakwater and eliminates additional energy losses due to natural
wave breaking processes occurring between the incident and transmitted
wave gages. Because of this definition K_T should be somewhat conser-
vative.

In an extensive study of wave transmission, Seelig (1979) concluded
that the relative freeboard F/H_s was the most important variable in
explaining the transmission characteristics of submerged and overtopped
breakwaters, where $F = h_c - d_s$. Figure 7 shows a trend curve for
K_T as a function of F/H_s based on results from this study. The curve
shown in Figure 7 is schematic in nature because it tries to follow the
trend of the data from all 10 subsets. There are some small inconsis-
tencies introduced by using this approach because the larger structures
will attenuate the waves better than the smaller structures for the
same relative freeboard, i.e. the broader the crest of the breakwater,
the greater the reduction in the transmitted wave height. Another pro-
blem is that for relatively high breakwaters transmission is no longer
dominated by wave overtopping but by energy transmission through the
structure which is a function of, among other things, wave steepness,
H_s/L_p. It is this changing role in the dependence of K_T on H_s
which causes the paradoxical trend in the data which is suggested by the
dashed curve in Figure 7. The transition in modes of transmission
occurs at approximately $F/H = 1.5$ for reef breakwaters. In Figure 7 the
dashed curve is not intended to suggest that for a fixed incident height
the transmitted height will increase if the freeboard is increased but
rather that with a fixed freeboard the transmission coefficient will
increase if the incident wave height is reduced. Despite its limitations
Figure 7 provides a reasonably good idea about the performance of reef
breakwaters over a wide range of conditions. The problem is a typical
one: the phenomena of interest is not a function of one variable over
the entire range of interest.

Analysis of the wave transmission data is continuing. It appears
that F/H_s and H_s/L_p provide a reasonably good way to parameterize
wave tranmission due to overtopping and through the breakwater, respec-
tively. However, the work of Allsop (1983) suggests that there should
be some influence of wave period in the portion of wave transmission due
to overtopping.

Wave Reflection and Energy Dissipation

Wave reflection from the breakwater is more strongly dependent on
wave period or wave length than is the wave transmission. Reflection,
however, also is strongly dependent on F/H_s which provides an oppor-
tunity to plot wave reflection data along with wave transmission data.

A figure showing both wave reflection and transmission can be used to estimate the amount of wave energy dissipated by the structure. Generally, the more wave energy dissipated the more effectively the breakwater is functioning since both reflection and high levels of transmission are undesirable. The following equation represents the energy balance in the vicinity of the reef breakwater.

$$K_R^2 + K_R^2 + dissipation = 1.0 \qquad\qquad Eq. 2$$

where K_R is the energy-weighted reflection coefficient given by Goda and Suzuki (1976) as

$$K_R = \sqrt{E_R/E_I}$$

where E_R and E_I are the reflected and incident wave energy, respectively, and "dissipation" is the fractional part of the wave energy remaining after the transmitted and reflected energy are subtracted from the total incident wave energy. Since the water depth is the same on both sides of the breakwater and the period of peak energy density is usually about the same on both sides of the breakwater, Equation 2 should be approximately correct. Figure 8 shows the results of using Equation 2 for the data collected during this study. In Figure 8 energy is plotted versus the relative freeboard with the transmitted energy to be read against the scale on the left side of the figure and the reflected energy to be read against the scale on the right side of the figure. Wave energy dissipated by the structure is the energy remaining in the central part of the figure after the reflected and transmitted energy are subtracted from the total incident wave energy. Figure 8 is schematic in nature because there are variables affecting reflection and transmission which cannot be taken into account by the figure, but it illustrates in a reasonably accurate way the most important function of the breakwater, i.e. to dissipate wave energy.

Conclusions

The severity of irregular wave attack is measured better by the parameter $H_s^2 L_p$ than it is using H_s^3, at least for reef breakwaters. If a stability number similar to Hudson's (1959) is defined using $H_s^2 L_p$, a variable is formed which is a very useful measure of the cause of damage to the breakwater for irregular wave conditions. This variable is referred to as the spectral stability number, N_s^*, and is defined by Equation 1. All of the test series used to evaluate stability indicate there is a threshold for stone movement around $N_s^* = 7$. When $N_s^* \lesssim 6$ there is little or no stone movement and when $N_s^* \gtrsim 8$ there is noticeable stone movement and damage to the breakwater.

Wave transmission is caused by wave overtopping and transmission through the breakwater which for a given structure are primarily functions of $(h_c - d_s)/H_s$ and H_s/L_p, respectively.

Wave reflection from the breakwater was measured during these tests. By subtracting the reflected and transmitted energy from the total incident energy an estimate of the wave energy dissipation by the breakwater as a function of $(h_c - d_s)/H_s$ was obtained (see Figure 8).

Acknowledgements

Figure 8. Wave energy allocation in the vicinity
           of a reef breakwater.

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