CHAPTER 158

OVERTOPPING OF RUBBLE-MOUND BREAKWATERS BY IRREGULAR WAVES

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ABSTRACT

This paper describes the results of an experimental study on the effect of waves on rubble-mound breakwaters, wave transmission subsequent to wave overtopping, the stability of the three sides subjected to wave action and the effect of the breakwaters on waves. Two different rubble-mound breakwaters were tested, i. e. one with a rigid impermeable crest and the other with a flexible permeable crest. Tests were performed with both regular and irregular wave train systems. To obtain the simulated irregular wave trains, four theoretical spectra were chosen: Neumann, Bretschneider, Moskowitz, and Scott. Results obtained from tests with irregular wave trains were compared to those obtained from tests with regular wave trains. It was found that more information was obtained on the behaviour of the structure when it was submitted to the attack of irregular waves than when submitted to regular waves, and that the use of irregular wave trains gave more interesting results.

1. INTRODUCTION

In some cases, it is not necessary to protect completely the lee side of a breakwater and some overtopping is allowed. An example of this is the use of an overtopped breakwaters for the partial protection of the Pilgrim Nuclear Power Station[6]. Some research works have been made on the overtopping effect, of which the following ones have been consulted: Cross and Sollitt[3], Hiroyoshi

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and Tsugio[5], Lording and Scott[6], Ouellet[9], Shiraishi, Numata and Endo[13], Tsurata and Goda[15], and the U.S. Corps of Engineers[16]. Most of the experimental works have been done under regular wave conditions.

It was then intended to establish an experimental programme to study the effect of irregular wave overtopping on structures as well as the effect of structures of waves. Although the stability of the structures and the wave overtopping are part of the study, the wave transmission subsequent to wave overtopping was mainly concerned.

2. THEORETICAL CONSIDERATIONS

Transmission Coefficient - The transmission of regular wave trains past a structure may be described in terms of the transmission coefficient, K_t, which is defined as the ratio of the height of the wave transmitted past the stucture to the height of the wave incident on the structure

$$K_t = \frac{H_t}{H_i} \qquad (1)$$

Transfer Function - When a structure is submitted to the attack of irregular trains, the characteristics are best described in terms of its transfer function[1]. If x(t) and y(t) are, respectively, the incident and transmitted wave profiles, and X(f) and Y(f) are, respectively, the Fourier transforms of x(t) and y(t), then the transfer function may be defined as:

$$H(f) = \frac{Y(f)}{X(f)} \qquad (2)$$

where X(f) and Y(f) are respectively the Fourier transform of x(t) and y(t):

$$\left.\begin{aligned} X(f) &= \int_{-\infty}^{\infty} x(t)\ e^{-i2\pi ft}\ dt \\ Y(f) &= \int_{-\infty}^{\infty} y(t)\ e^{-i2\pi ft}\ dt \end{aligned}\right\} \qquad (3)$$

in which f = the frequency, in Hertz. In fact, we know that the transfer function is the Fourier transform of the system to a unit impulse.

In practice, however, the following relation is used to determine the transfer function:

$$H(f) = \frac{S_{yx}(f)}{S_{xx}(f)} = \frac{X^*(f)\ Y(f)}{X^*(f)\ X(f)} \tag{4}$$

in which $S_{yx}(f) = Y(f)\ X^*(f)$ represents the cross power spectral density function of y(t) with respect to x(t), $S_{xx}(f) = X(f)\ X^*(f)$ represents the auto power spectral density function of x(t), and $X^*(f)$ represents the complex conjugate of X(f). This procedure makes it possible to retain the phase information without the need of having to synchronize the two signals.

Coherence Function - The preceding equations are based upon linear systems. Even if, in practice, linear systems do rarely occur, the use of the constant parameter linear system is often justified. In order to judge the applicability of the linear model it is possible to estimate the accuracy of the transfer function by means of the coherence function

$$\gamma^2_{xy}(f) = \frac{|S_{xy}|\ /S_{xx}{}^2}{S_{yy}/S_{xx}} = \frac{S_{xy}\ S^*_{xy}}{S_{yy}\ S_{xx}} = \frac{|S_{xy}|^2}{S_{yy}\ S_{xx}} \tag{5}$$

which represents the ratio of the magnitudes of the response function obtained, respectively from the cross-spectral density function and from the input and output density spectra. The coherency for an ideal system with no noise in measured input and output is unity. The presence of noise or the nonlinearity of the system reduces the coherence function. The coherence function is then an indicator of the system behavior in showing those parts of the output not correlated with the input over the frequency range.

3. TEST PROCEDURE

Wave Flume - A sketch of the wave tank layout is shown in Fig. 1. The channel is 111 ft (36 m) long, 6 ft (1.86 m) wide, and 4 ft (1.3 m) deep. The distance between the wave paddle and the model breakwater (center of crest) is about 65 ft (21 m).

The wave generator consists of a paddle operated by a hydraulic piston, the movements of which are controlled by an electric signal from a programming device capable of generating regular and irregular wave trains. A wave filter which absorbs, to some extent, waves reflected by the model is in front of the wave generator.

The flume was divided into two sections, i.e., the front one of 3.5 ft (1.1 m) and the back one of 2.5 ft (0.76 m).The front section was used as the test section whereas the back section was used to measure the incident wave characteristics. This arrangement made possible the measurement of the incident, reflected and transmitted wave profiles.

Wave heights and periods were measured with resistance type wave gages. A first gage, located in the smaller section of the flume where reflection is almost eliminated by a wave absorber placed at the end of the flume, was used to measure the incident wave characteristics. A second gage, placed behind the structure in the larger section of the flume measured the transmitted wave characteristics, while a third movable gage, placed in front of the structure, measured the incident and reflected wave characteristics. Measurements taken on the third gage, which also included the envelope of the incident and reflected waves, were only used to perform some verifications in cases of experiments with regular waves.

Wave data were analysed using a Fourier Analyzer model HP-5451A, a product of Hewlett Packard Company[4]. Fig. 2 shows an overview of the analyzer system

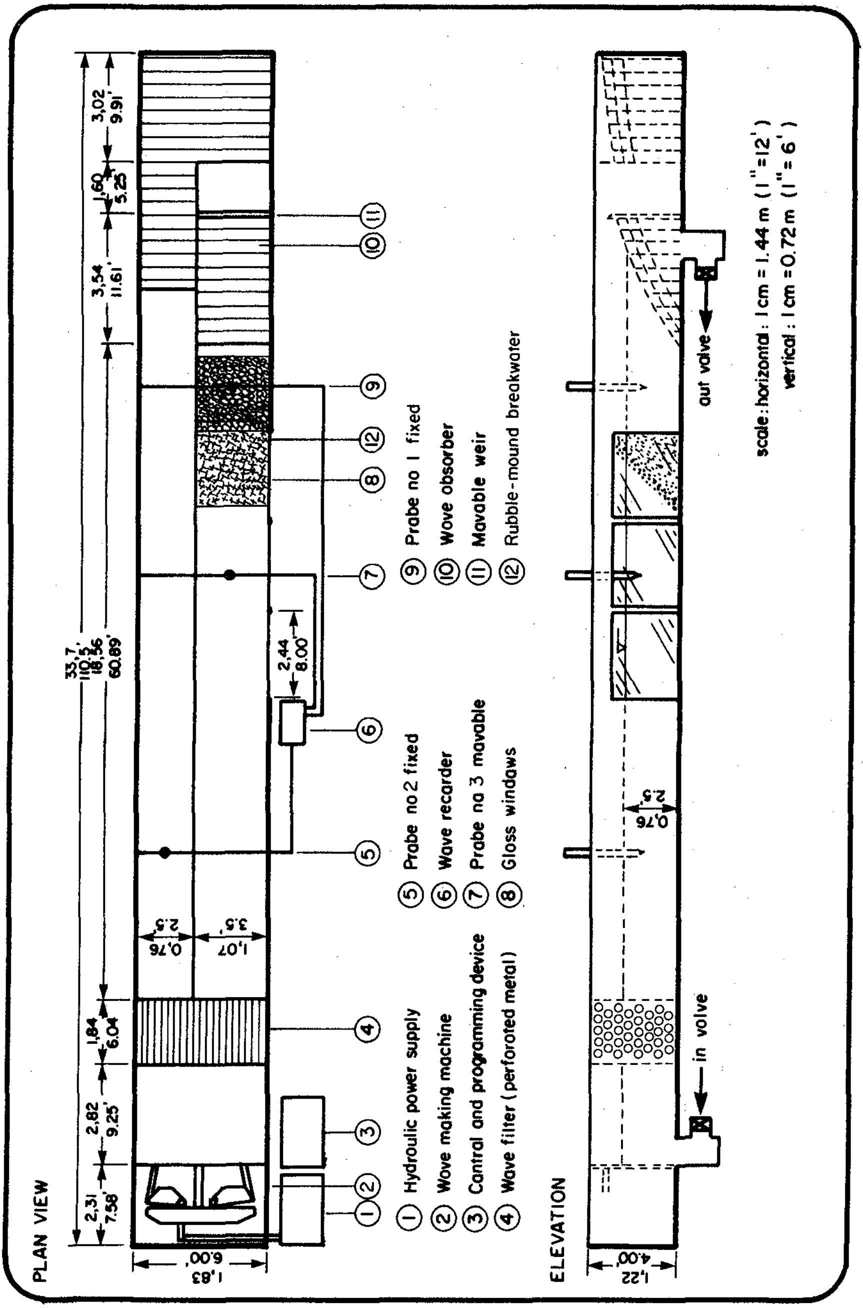

Fig. 1 —SKETCH OF WAVE TANK LAYOUT.

and details of its operation are available in user manuals furnished with the instruments.

Fig.2 — FOURIER ANALYSER SYSTEM.

Description of Models - A simplified sketch of the cross-section of the two models studied is shown in Fig. 3. The first model consists of a rubble-mound breakwater formed by two layers of dolos armor units seaside, two layers of rubble on the lee side, and a thick concrete cap crest. The second structure is similar to the first differing only at the crest; the crest of the latter is formed by two layers of dolos armor units. Because both models were placed behind observation windows, the observation of certain characteristics, such as over-topping height, was made possible.

Test Conditions - In order to verify the models and to make possible further comparisons, both models were first tested under regular wave attack. All tests were carried out at four different water depths varying both the wave height and

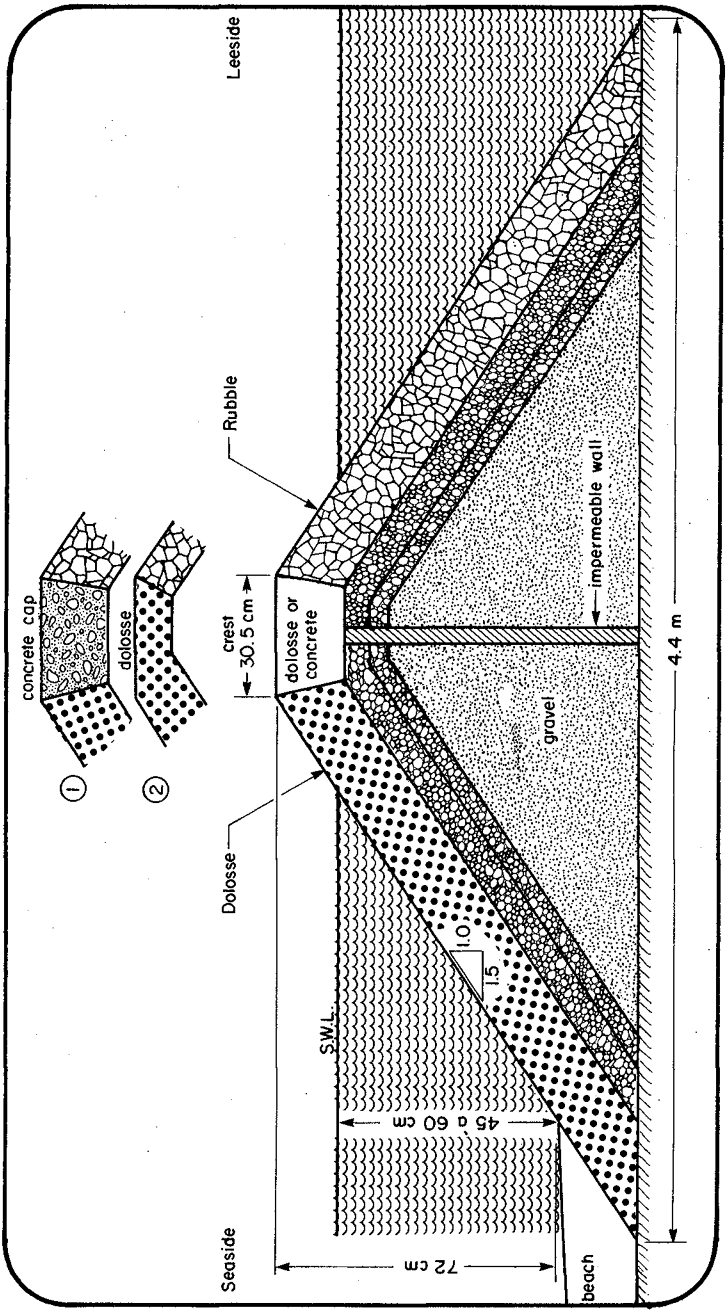

Fig. 3——SKETCH OF RUBBLE-MOUND BREAKWATER.

the frequency. Typical results of these tests are shown in Fig. 4 and 5, respectively for the concrete cap breakwater and the breakwater covered with dolosse. The transmission coefficient, expressed as a percentage, is plotted versus the frequency. The transmission coefficient was also represented as a function of the relative depth, but for the present paper only the frequency is used.

The two models were also submitted to the attack of irregular wave trains which were produced by adjusting the variable amplitudes and phases of the 20 different harmonic components of the programming device. As in the case of regular wave tests, the water depth varied from 45 cm to 60 cm. In order to make even further comparison possible, four different types of wave spectra were simulated on the model on a scale of 1:16. These consist of Neumann[8], Bretschneider[2], Moskowitz[7,11] and Scott[12] which are shown in Fig. 6, with the corresponding wind velocities used. The related equations are the same as those in previous studies[9,10], from which simulated spectra were obtained (Fig. 7).

4. TESTS RESULTS AND ANALYSIS

Fig. 8 and 9 show typical examples of recorded surface profiles of incident and transmitted waves, and the results of the spectral analysis of the above signals in the case of a simulated Neumann spectrum. Fig. 8 corresponds to the concrete cap breakwater with the depth h = 60 cm and the simulated significant wave height H_s = 4.25 m (13 ft). Fig. 9, on the other hand, corresponds to the other structure for the same values of h and H_s.

By means of spectral analysis[1,14], the following results were obtained and plotted for all tests involving irregular wave trains: the incident wave spectrum, S_I, from which the significant wave height may be obtained; the transmitted wave spectrum, S_T, from which the transmitted significant wave height

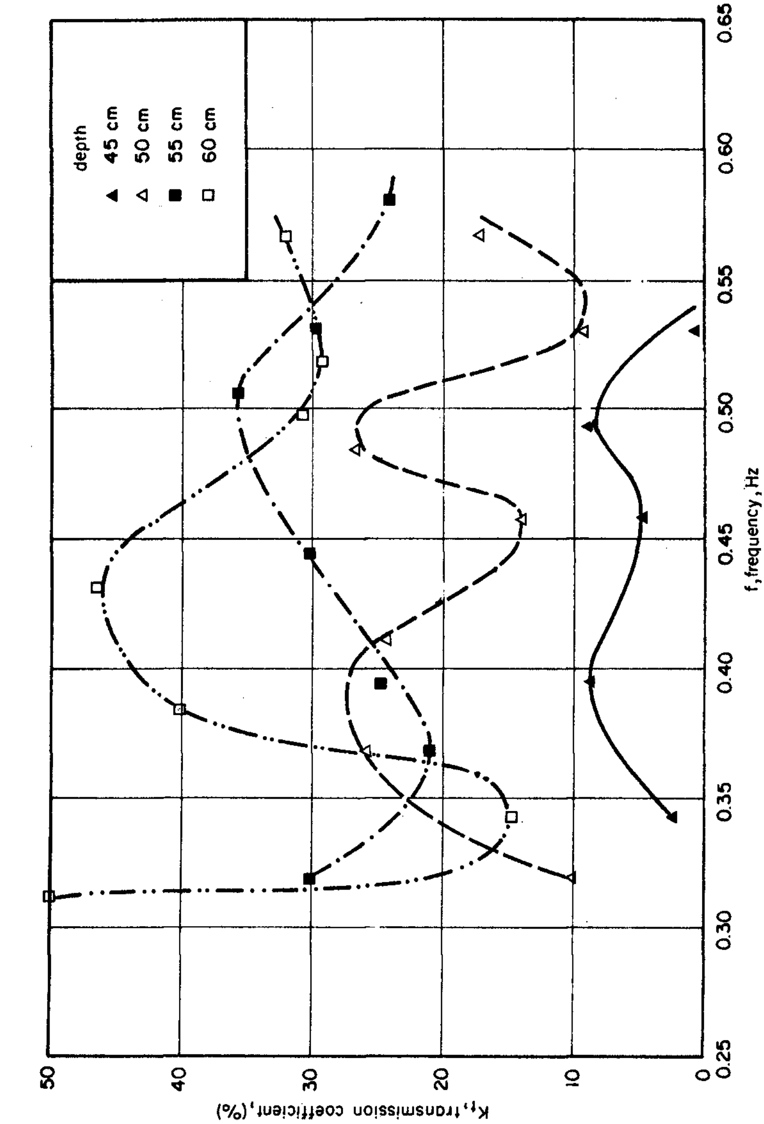

Fig. 4—TRANSMISSION COEFFICIENT AS A FUNCTION OF FREQUENCY, (REGULAR WAVES).
(amplitude: SKT 7, crest: concrete)

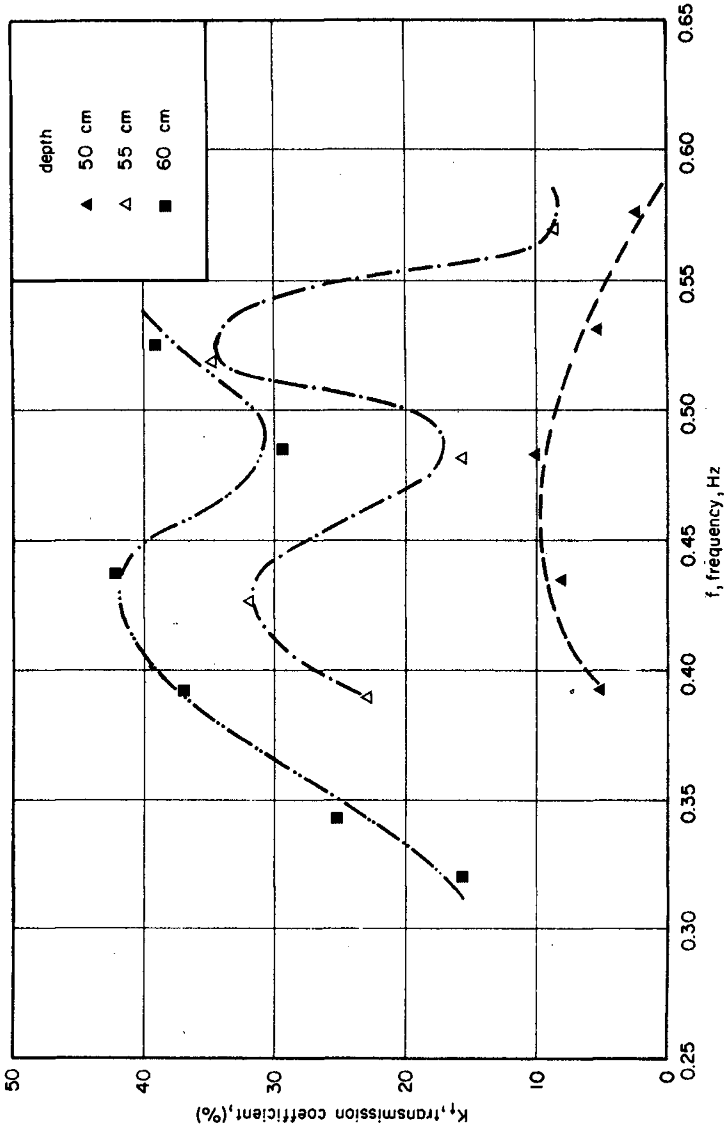

Fig. 5—TRANSMISSION COEFFICIENT AS A FUNCTION OF FREQUENCY, (REGULAR WAVES).
(amplitude: SKT 7, crest: dolosse)

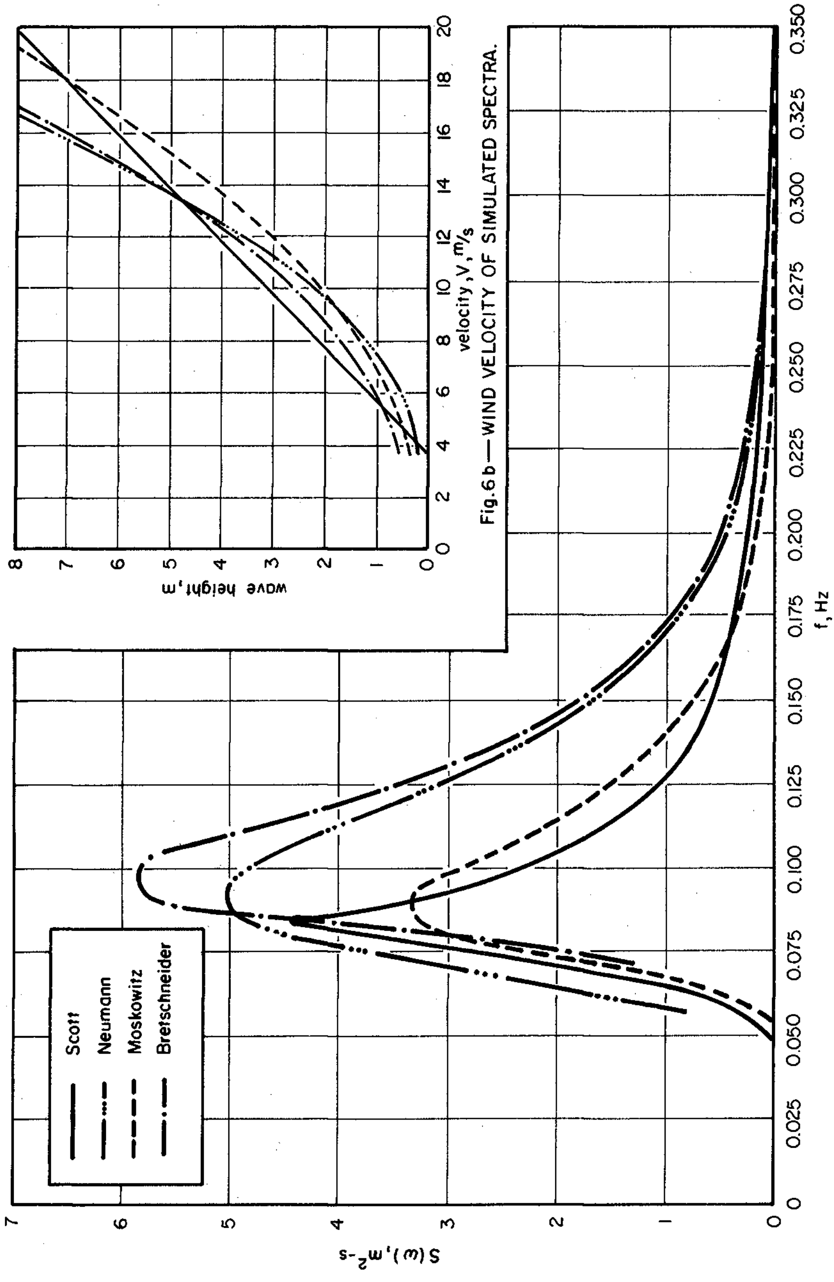

Fig.6b — WIND VELOCITY OF SIMULATED SPECTRA.

Fig.6a — THEORETICAL WAVE SPECTRA.

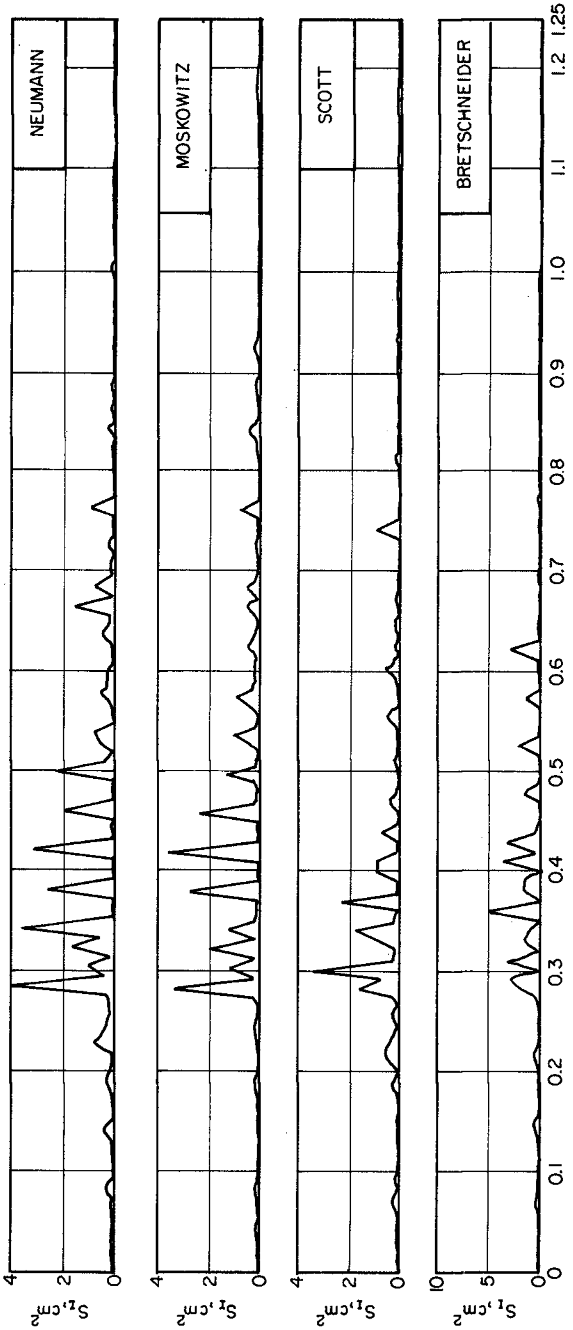

Fig. 7—SIMULATED WAVE SPECTRA.

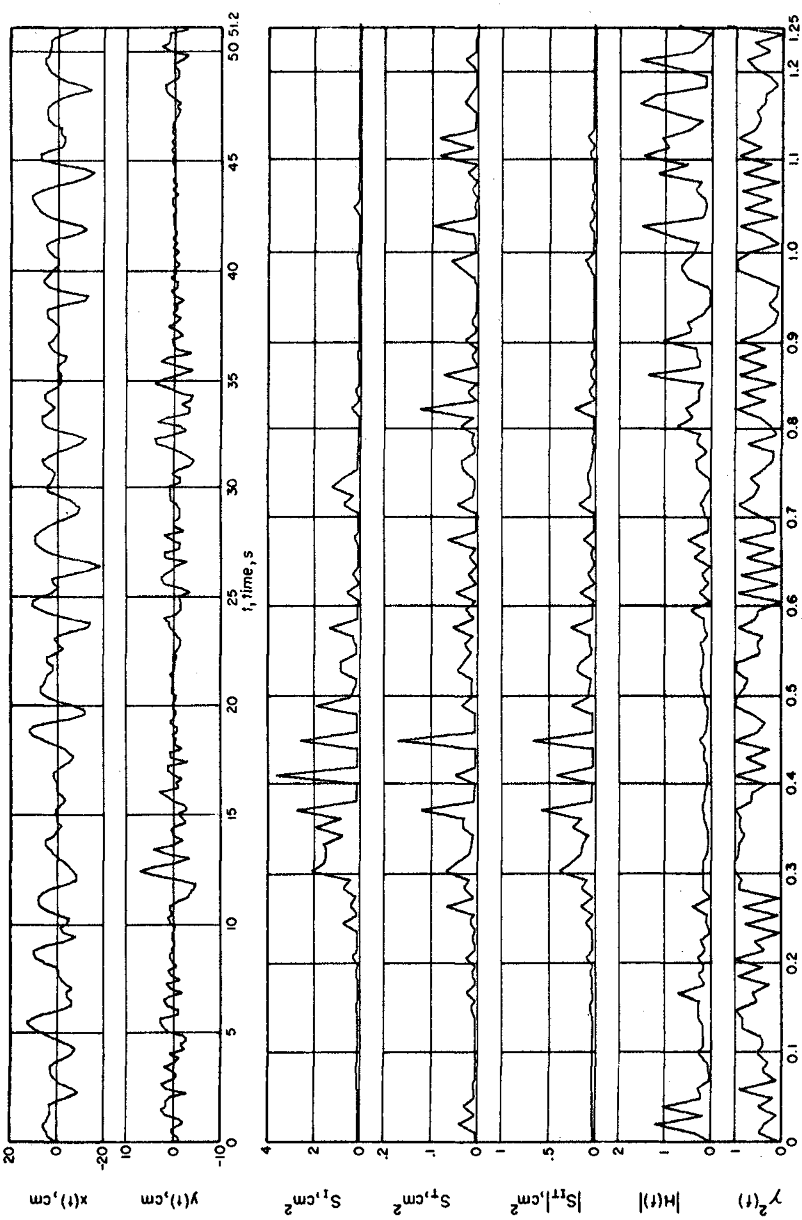

Fig. 8—IRREGULAR WAVES: NEUMANN SPECTRUM.
N-13-60-CB

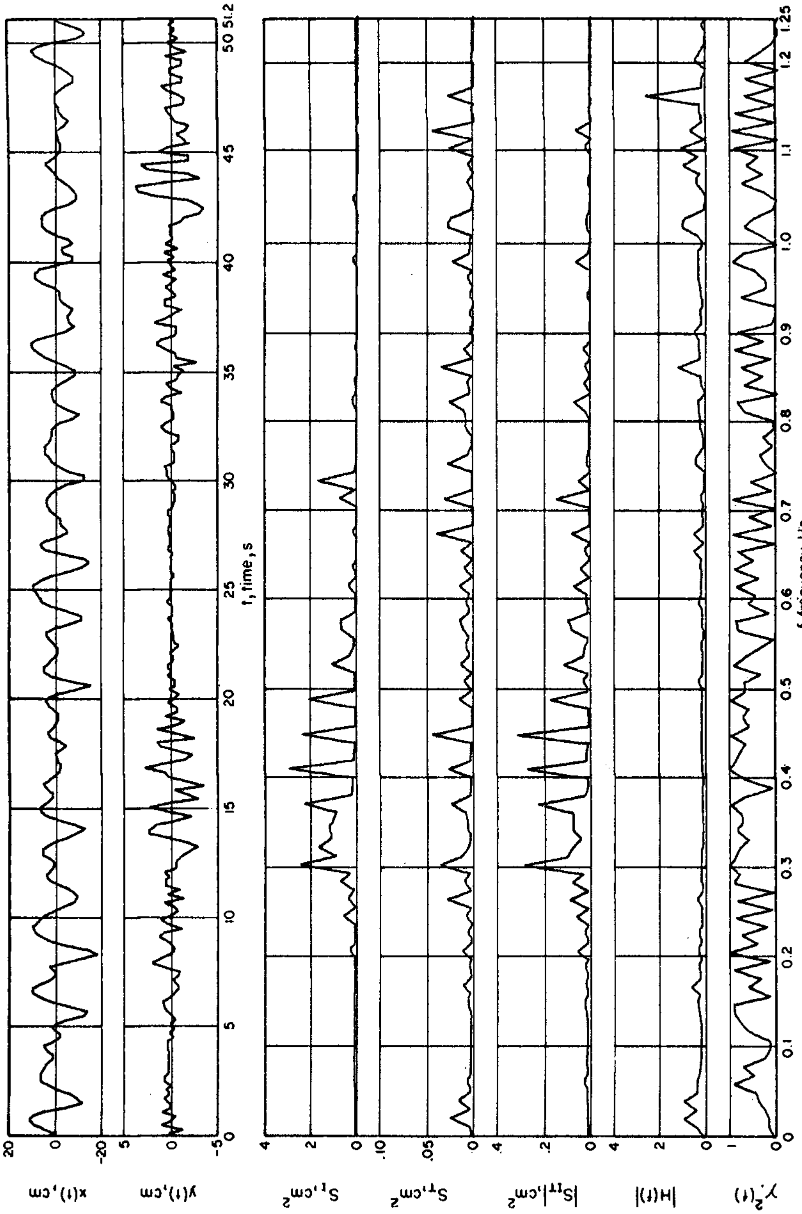

Fig. 9 — IRREGULAR WAVES: NEUMANN SPECTRUM.
N-13-60-D

may be obtained; the magnitude of the cross power spectrum, $|S_{IT}|$; the magnitude of the transfer function, $|H(f)|$; and the coherence function $\gamma^2(f)$. Both structures were submitted to the same tests under similar conditions using all four simulated wave spectra. The results were analyzed individually and then compared to each other as well as to regular wave tests. Under the test conditions examined, the following results were found to be the most interesting:

1° Single test analysis

a) The incident wave energy, S_I, is much greater than the transmitted wave energy, S_T: consequently the incident significant wave height is also greater than the transmitted significant wave height.

b) The magnitude of the transfer function, $|H(f)|$, is weak, showing there is very little energy transmitted over the structure.

c) At frequencies containing energy the coherence function γ^2 generally varies from 0.8 to 1.0, meaning the output signal is affected very little by the presence of noise or the nonlinearity of the signal.

2° Comparison between spectra

a) When the incident energy spectra were reverted to significant wave heights it was found that only the Neumann and Bretschneider spectra gave values corresponding to the simulated wave height; Moskowitz and Scott with smaller energies gave wave heights about 1 m smaller than the simulated wave height.

b) For equivalent values of incident energy disregarding the actual wave height as compared to the simulated height, it was found that the transmitted energy varies very little between different spectra.

3° Comparison between structures

a) The concrete cap breakwater is much more stable than the dolos crest breakwater, yet the former allows more overtopping than the latter.

b) Damage tends to occur at almost the same overtopping wave heights for both structures.

4^{o} Comparison with regular wave tests

a) Observations allowed to establish that 3 or 5 out of every 20 waves would overtop the structure with irregular waves whereas all 20 would overtop the structure with regular waves.

b) Damage occurs at a faster rate with regular waves than with irregular waves.

c) The ratio $H_t/H_i(K_t)$ was as high as 50% whereas the ratio H_{ts}/H_{is} only reached 28%.

Figs. 10a and 10b show the relationship between significant wave height and overtopping wave height for all four spectra respectively for the concrete and dolos crest breakwater in 60 cm depth. The same relationship was also found in the case of regular wave trains as shown in Fig. 11a, for the concrete cap and in Fig. 11b for the dolos crest breakwater. Other depth curves have been added on these figures but without the measured points in order not to overcharge the figures. The dispersion of the points is approximately the same for the ones given.

The slope of the curves are approximately the same for the different water depths the difference being only in the ordinate at the origin. Furthermore, visual observations allowed to establish at which overtopping height the breakwater was damaged or destroyed completely. These observations were made for all three exposed faces. Having established that an incident wave height and an equivalent significant wave height have almost identical overtopping heights,

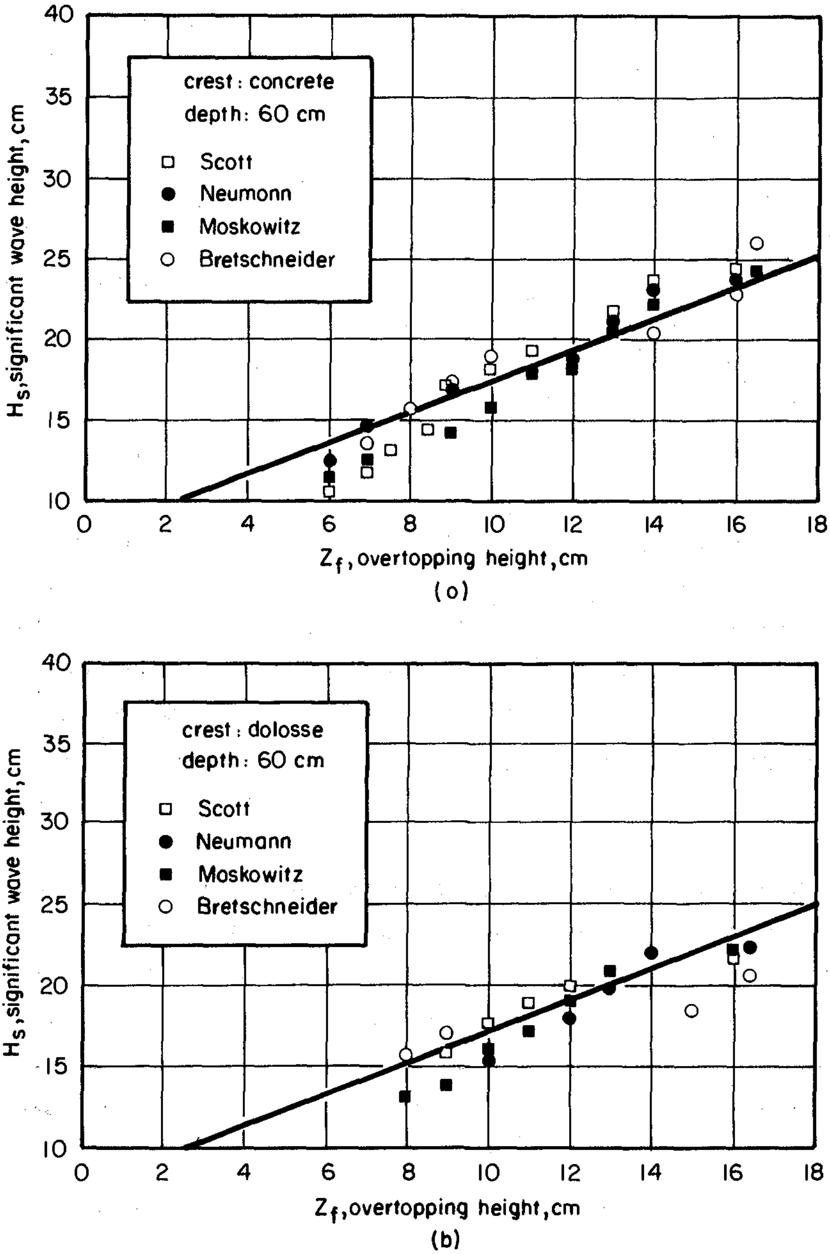

Fig.10 — SIGNIFICANT WAVE HEIGHT VS OVERTOPPING HEIGHT, IRREGULAR WAVES.

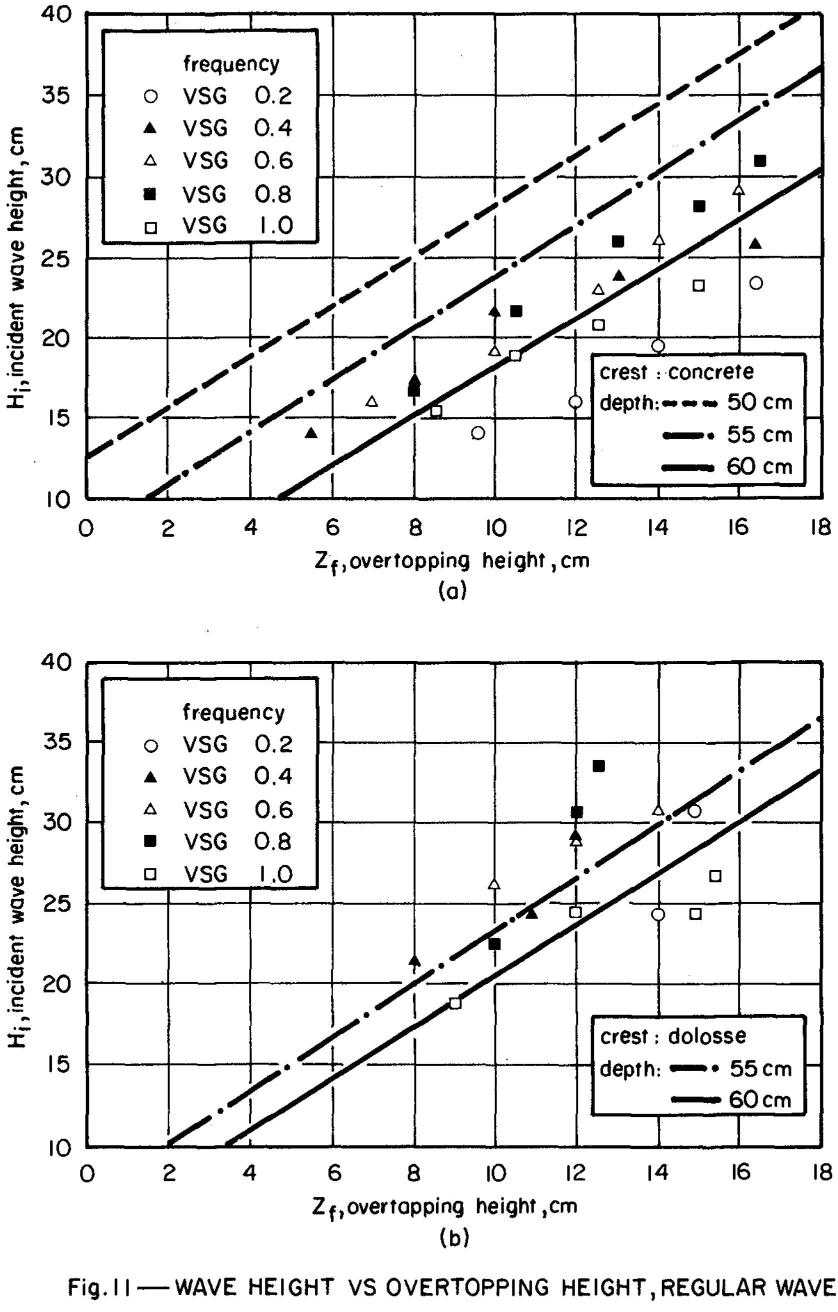

Fig. 11 — WAVE HEIGHT VS OVERTOPPING HEIGHT, REGULAR WAVES.

it would be possible to predict at which significant wave height the structure will be damaged, using only rerular wave trains, the main difference being the quantity of damage.

A comparison between the results obtained for the concrete cap breakwater and the dolos protected breakwater shows that on one hand the former is much more stable than the latter, whereas, on the other hand, the concrete cap structure allows more energy to be transmitted than the dolosse protected structure. These differences are primarily due to the difference in the shape of their respective crests.

In order to estimate the relative efficiency of either structure, the ratio of the transmitted significant wave height to the incident significant wave height has been calculated. The characteristic wave height,

$$H_{m_o} = 4\sigma = \sqrt{\int_0^\infty S(f)\ df} \qquad (6)$$

σ^2 being the variance of the process, has been used to represent the significant wave height from which the ratio of the transmitted wave height to the incident wave height has been determined. From the results it has been found that, for the concrete cap breakwater, the maximum values of this ratio vary between 0.06 [h = 45 cm ; H = 4.25 m Bretschneider spectrum] and 0.28 [h = 60 cm ; H = 4.25 m ; Neumann spectrum] while the corresponding values for the dolos crested breakwater vary from 0.04 [h = 45 cm ; H = 4.25 m ; Bretschneider spectrum] to 0.19 [h = 60 cm ; H = 4.25 m; Neumann spectrum]. It should be remembered, however, that this parameter does not take into account the frequency which is an important factor in such a study.

5. CONCLUSIONS

The described tests have shown that the response of a structure gives more information when submitted to the attack of irregular waves than regular waves. However, the zone of damage caused by given overtopping wave heights is the same regardless of the type of waves used, which implies the predictability of damages. Furthermore, such parameters as the quantity of damage, the rate of wave overtopping, and the total energy of overtopping, and the total energy of overtopping waves are always of greater magnitude when the structure is submitted to regular wave trains.

All the tests with regular waves have been carried out varying the frequency by a small amount at each step. It was noticed that, for one reason or another, the transmission coefficient curves show an oscillating tendency as a function of frequency. Furthermore, the same tests pointed out that the incident wave height curves versus frequency are also oscillating. However, the fact that the incident wave heights may oscillate does not explain entirely the transmission coefficient oscillation; this is subject to further research.

The magnitude of the transfer function shows how the two breakwaters examined are efficient when submitted to irregular wave tests. From the results obtained it can be seen that these breakwaters will, beyond the slightest doubt, prevent or reduce to a minimum energy transmission from the seaside to the leeside of the structure. Further, to support this affirmation, it has already been seen that the maximum ratio of transmitted significant wave height to incident significant wave height was 0.28.

Results have also shown that the structures are quite stable up to 12 cm (1.9 m prototype) of wave overtopping, major damage begins to occur between 12 and 14 cm of wave overtopping depending on the structure, and total destruction occurs for overtopping heights beyond 16 cm (2.6 m prototype).

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