

CHAPTER 71

EXPERIMENTAL STUDIES ON A FIXED PERFORATED BREAKWATER

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ABSTRACT

Model tests on a perforated breakwater system were carried out to evaluate effectiveness in different states of sea. Force measurements were made in three long-crested irregular wave systems and, by spectral analysis, the full-scale statistical characteristics of force were obtained. The effect of perforating the back wall was studied to determine the extent of further reduction of force on the structure. An interior perforated wall was also added for force reduction but was not as effective as the perforated back wall. The system was mounted on piles and imbedded in sand for a quasi-quantitative study of scouring in regular waves. In this case, the elevation of the breakwater walls above the sandy bottom was varied. Both the perforations and the elevation contributed to significant reduction in observed scour. The end result of these experiments is to provide information on the kind of improved performance that might be expected as each variation is introduced which, in turn provides input to design for particular applications.

INTRODUCTION

OCEANICS, Inc. has been engaged in development and evaluation of a design concept for a portable breakwater that comprises a perforated front wall with a solid back wall. As a floating system, the chief virtue that is claimed for the perforated breakwater is that it affects a considerable reduction in the height of the incoming waves without paying a penalty in excessive forces on the mooring lines. When the perforated breakwater is fixed to the bottom, it is purported to dissipate sufficient wave energy to cause it to be structurally sound for long periods of time. Furthermore, it is believed that the front wall porosity is an important factor in maintaining stability of bottom sediment such that scouring is minimized and the probability of structural damage and overturning is greatly reduced.

The first phase in the evaluation of the perforated breakwater was aimed at determining whether it was generally more satisfactory than a caisson of the same dimensions for fixed and/or floating operation. That is, do the perforations result in less force on the structure than a solid wall, when the structure is fixed to the bottom? And, when floating, does the perforated breakwater reduce the waves more and still experience appreciably less force? If the perforated breakwater is superior, what is its optimum geometry?

In general, the Phase I study reported by Marks (1966) revealed the following:

1. The fixed perforated breakwater experiences less force overall than the solid breakwater. Greater effectiveness was usually found at lower wave periods except for the very important upward vertical force where the solid breakwater experienced forces greater than 11 times that of the perforated breakwater, at the design wave (13 seconds, 15 feet).
2. The breakwater geometry specifying: 4-foot diameter holes, 4-foot wall thickness, and 40 feet between front and back wall was found to be most effective, as predicted by theory.
3. For the floating case, the perforated breakwater experienced less force in the 4 mooring lines. Again, the degree varied being just slightly less in one instance and one-tenth of the force in the mooring line of the solid breakwater in another. At the design wave, the mooring lines in the perforated breakwater experienced less force by about a factor of 2.
4. The motion of the breakwaters as measured by horizontal and vertical accelerations showed no clear superiority and this was reflected in wave reduction behind the breakwaters.
5. The excessive vertical forces on the face of the solid fixed breakwater caused a layer of sand 3 inches high and one foot in extent to be completely cleared away from the foot of the breakwater. The perforated breakwater produced little or no scouring.

On the basis of the results achieved in the first phase of the study, it was considered to be more practical to pursue the development of the more promising fixed perforated breakwater. Thus, the essence of the work reported here deals with examination of certain variations in breakwater design that are aimed at further reduction of force on the system (fixed to the bottom) without vitiating its wave reduction prowess. The force on the basic structure is developed theoretically and then measurements at model scale are described. Additional measurements were made with such variations as perforating the back wall and

inserting a perforated interior wall. The effect of scour at the base was also examined and is reported here. There is evidence that the perforated breakwater is admirably suited for amphibious operations.

PHYSICAL CHARACTERISTICS OF BREAKWATER

The basic purpose of any breakwater system is to present an obstacle to the oncoming waves that will cause the wave height (hence energy) to be substantially reduced on the shoreward side, without compromising the functional efficiency of the breakwater system during the required time of operation. The perforated breakwater has been specifically designed for such a mission. The original concept of a perforated breakwater was developed by Jarlan (1965).

The dynamic processes that result from the incidence of waves on the perforated breakwater can best be visualized by considering that as the wave impinges on the porous front wall, part of its energy is reflected and the remainder passes through the perforations. The potential energy in the wave is converted to kinetic energy in the form of a jet, upon passage through the perforation, which then tends to be partially dissipated by viscosity in the channel and partially by turbulence in the fluid chamber behind the perforated wall. As the water in the fluid chamber flows back out of the holes, it encounters the next oncoming wave and partial energy destruction is accomplished even before that wave reaches the breakwater. If the walls were not perforated (e.g. a caisson), total reflection would occur on the face of the wall with resultant high impact forces and scouring at the base, if it is fixed to the bottom. If the breakwater were floating and anchored, part of the incident wave force would be transmitted to the mooring cables and part would be directed to oscillating the breakwater thus inducing it to make waves on the shoreward side. In the case of the perforated breakwater, that part of the incident wave energy which is dissipated internally in the form of heat and eddies is not available for such deleterious activity.

To understand more fully what happens when a wave is incident upon a breakwater, consider the following development. The forces exerted by wind-induced gravity waves on a vertical obstacle extending from above the surface to the sea bottom, at a given depth d , are:

1. The weight of the obstacle,
2. The hydrostatic under pressure,
3. The hydrostatic pressures exerted on both vertical sides (horizontal pressures),

4. The dynamic uplift pressure.

Assume that the breakwater is a caisson-like structure of concrete block resting on a stone foundation placed at the bottom and is subject to oscillatory forces. Although the base of the structure may be in contact with the foundation over about 50% of its surface, it is generally assumed that an apparent weight must be accounted for and that the hydrostatic pressure exerted on the base of the breakwater affects its entire area.

Since the surface of the clapotis (standing wave due to reflection from vertical wall) is alternately above and below still water, the accompanying pressure changes due to this vertical motion will increase or decrease the hydrostatic pressure by an amount

$$P = g\rho_w \frac{H}{\cosh \frac{2\pi d}{L}} \quad (1)$$

where ρ_w = specific gravity of water (slugs), H = amplitude of the clapotis, and L = wave length at depth d (in feet).

As the wave crest reaches the wall (Figure 1), the pressure at the toe on the sea side is greater than on the shore side by an amount equivalent to AB . Since the pressure is transmitted under the structure at approximately the velocity of sound, an uplift pressure will develop under the breakwater according to a triangular distribution. It is assumed that the head losses in the stone mound on which the breakwater is resting are linear.

A similar phenomenon applies in the opposite direction when the trough of the wave is present on the sea side. It must be mentioned that these uplift pressures act in the same direction as the forces exerted on the vertical wall, thereby increasing the overturning moment. It is possible to calculate the moment generated by the forces involved and from this to deduce the stabilizing moment due to weight. With a triangular distribution of the uplift force the resultant cuts the base at the edge of the middle third.

To determine the resistance against sliding, it is necessary to multiply the effective downward force on the structure by a coefficient of static friction. This result is then divided by the horizontal wave pressure, assuming a friction coefficient of about 0.5. The factor of safety should not be less than 2.

The foregoing is a brief review of the procedure generally followed when designing an ordinary vertical wall breakwater. The resultant of the forces per unit length of wall (R) and the moment (M) about the base are given by the formulae

$$R = \frac{1}{2} (d+H+h_o) (\rho d+P) - \rho \frac{d^2}{2} \quad (2)$$

$$M = \frac{1}{6} (d+H+h_o) (\rho d+P) - \rho \frac{d^3}{6} \quad (3)$$

where P = pressure due to wave, $\rho = 62.5 \text{ lb.ft.}^{-3}$, h_o is the wave set-up (calculated by the relationship $h_o = \frac{\pi H^2}{L} \coth \frac{2\pi d}{L}$, where L = wave length). The existence of this set-up results from the fact that a pile-up of water, due to orbital motion and mass transport, occurs at the wall, raising the mean sea level by an amount h_o which is a function of $\frac{d}{L}$ and $\frac{H^2}{L}$; h_o defines the mean level of the clapotis.

In the case of the perforated breakwater, two phenomena modify the situation encountered in the case of a plain-wall breakwater. Firstly, the porosity of the system reduces the reflection, thereby diminishing the amplitude of the clapotis which then becomes a partial clapotis. Secondly, the dissipation of the energy is such that about 65% of the incoming wave energy is dissipated through jet diffusion in the back chamber. The energy transferred to the system is thus about 35% of the total energy. Assuming this energy to correspond to a given wave height, the following relationship holds.

$$\left(\frac{H_2}{H_1} \right)^2 = 0.35 \quad (4)$$

Hence, $H_2 \approx 0.6H_1$ (equivalent wave height of the non-dissipated energy). For the design wave, $H_1 = 15$ feet and $H_2 = 9$ feet.

On the other hand, one may assume a reflection coefficient of about 30% yielding for the amplitude of the clapotis a value $H = 9$ feet. The total height of the partial clapotis is then 18 feet while the full clapotis would have a height of 30 feet. Under these conditions, when $H = 15$ feet, $T = 13$ seconds, $d = (40 + 10)$ feet (height of the breakwater 50 feet) a value of the resultant of $R \approx 25,000$ lb. is obtained by substitution in Equation (4). In the case of the perforated breakwater, $h_o \approx 1$ ft. for the considered initial conditions corresponding to a partial clapotis of 18 feet. The moment is then from Equation (3) $M \approx 589,000$ ft.-lb. If for stability against overturning, it is required that the resultant of wave pressure and weight must fall within the middle third of the base, then assuming uplift (triangular distribution) the width of the caisson is found to be about 46 feet and the weight per linear foot is 49,000 lb.

From these simplified calculations, it is seen that the design wave can produce very large forces which are diminished considerably by the perforations. The width of the breakwater is commensurate with the design width required for maximum efficiency and the calculated weight is reduced by supporting the breakwater on piles driven into the bottom. The experiments described here are aimed at further reducing the force on the breakwater by inducing additional energy dissipation through a perforated back wall and through insertion of an inner perforated wall.

The efficiency of energy dissipation, in the breakwater principle proposed here, depends on the geometry of the system which in turn is determined by the nature of the design wave conditions. From the laws of fluid motion in the chamber, the following design criteria are obtained:

1. Ratio of chamber width to wave length.
2. Ratio of wall thickness (channel length) to hole diameter.
3. Ratio of perforated to unperforated areas (solidity ratio).

The theoretical development leading to the establishment of the above criteria, for particular wave inputs, was presented by Jarlan (1965) and Jarlan and Marks (1965).

MODEL EXPERIMENTS

The model experiments were two-dimensional and were carried out in the ship model towing tank (100 ft. x 10 ft. x 5-feet deep) at Webb Institute of Naval Architecture. As in the previous tests reported by Marks (1966) the tank was modified to simulate shallow water by installation of a flume with sloping beach 1:16 (Figure 2). The models were mounted in the shallow end of the flume so that at a scale of 1:45, the models are essentially in 45 feet of water.

The tests were conducted in long-crested irregular waves to simulate, as near as possible, actual conditions. There were three sets of waves employed corresponding to low, moderate and high waves. The wave spectra are shown in Figure 3 and the statistical characteristics for each are given in Table I.

Table I. Statistical Characteristics of Irregular Waves Used in Model Tests.

Wave Spectrum	\bar{H} (ft.)	\bar{H} (1/3)	\bar{H} (1/10)
Low (I)	4.5	7.1	9.0
Moderate (II)	6.9	11.1	14.1
High (III)	8.6	13.8	17.5

In Table I, \bar{H} is the average wave height (in feet); $\bar{H} (1/3)$ is the average of the one-third highest waves (significant height) and $\bar{H} (1/10)$ is the average of the one-tenth highest waves. All of the numbers in the table correspond to full-scale conditions.

For each of the breakwater variations tested, the force on the structure (vertical and horizontal) was measured as well as the irregular wave pattern. The breakwater models were mounted so that the back wall was rigidly fixed to a force transmitting bar that extended across the flume. The front wall was fastened to the back wall by 6 rods and both walls were free of the bottom and sides (by very small clearances) and extended above the design height so that all of the force on the structure would be communicated to the bar without loss. The force transmitting bar passed through the steel side walls of the flume and was fixed rigidly at both ends to the force-measuring strain-gage systems. The strain gages (horizontal and vertical) were mounted rigidly to the steel side walls. Thus, the deflection of the bar relative to the rigid steel sidewalls is a measure of the force exerted on the breakwater by the waves.

DISCUSSION OF RESULTS

The model tests produced a number of records of vertical and horizontal force. These records were converted to appropriate force spectra (Figures 4 and 5) and from these outputs total force calculations were made.

Forces on Prototype Breakwater

The first set of force measurements was made on the basic perforated breakwater. The results are shown in Table II where \bar{F} is the average force and the other symbols correspond to the definitions given in Table I.

Table II. Statistics of Total Force (pounds per foot) on Perforated Breakwater.

Wave Spectrum	\bar{F}	$\bar{F} (1/3)$	$\bar{F} (1/10)$
I	200	320	407
II	240	384	288
III	282	450	572

Table III shows the same force statistics when the breakwater is not perforated.

Table III. Statistics of Total Force (pounds per foot) of Solid-Wall Breakwater.

Wave Spectrum	\bar{F}	$\bar{F} (1/3)$	$\bar{F} (1/10)$
I	444	710	903
II	510	814	1035
III	604	965	1230

It is seen by comparison of Table II with Table III that the perforations reduce substantially the force on the breakwater. In this case, the reduction is more than 50% which is in line with theoretical prediction.

Perforated Interior Wall

A perforated wall was placed halfway between the front and back wall in an effort to induce further internal energy dissipation. The results are given in Table IV.

Table IV. Statistics of Total Force (pounds per foot) of Perforated Breakwater with 10% Perforated Interior Wall.

Wave Spectrum	\bar{F}	$\bar{F} (1/3)$	$\bar{F} (1/10)$
I	425	680	842
II	497	795	1020
III	575	920	1170

Comparison of the statistics in Table IV with those in Tables II and III indicate that the interior perforated wall is relatively ineffective in dissipating wave energy. Moreover, it appears to substantially decrease the effectiveness of the outer perforated wall. There is no evidence that optimizing the geometry of the interior wall will improve performance materially.

Perforated Back Wall

A perforated back wall was installed to further reduce wave force. However, in this case, it was necessary to observe wave formation on the shoreward side of the breakwater and to ascertain that the outpouring did not exceed 10 feet which is assumed to be the maximum stand-off distance for a ship, when the breakwater is used as a pier. It was impossible to make precise measurements, so visual observations had to suffice. Tables V and VI show the results of a perforated back wall. In the case shown in Table V the back wall was 20% perforated; regular and irregular waves were used.

Table V. Observed Outpouring from 20% Perforated Back Wall.

<u>Regular Waves</u>		
Period (sec.)	Height (ft.)	Distance (ft.)
7	10.2	1.87
10	11.2	0.94
13	4.3	0.94
13	10.2	1.87
16	7.0	1.87

Irregular Waves

	Average Distance	Maximum Distance
Low	1.98	5.63
Moderate	2.25	6.75
High	2.82	6.75

It appears that the stand-off distance of 10 feet is relatively safe under the conditions tested here. Furthermore, for the design wave period (10 seconds) at an incident wave height of 10.2 feet, the waves on the shoreward side of the breakwater were only about 1/5 the height on the seaward side.

A second experiment was carried out by inserting an interior wall with 10% perforations. The results are shown in Table VI.

Table VI. Observed Outpouring from 20% Perforated Back Wall with 10% Perforated Interior Wall.

Regular Waves

Period (sec.)	Height (ft.)	Distance (ft.)
7	10.2	0.47
10	11.2	1.87
13	4.3	0.94
13	10.2	3.75
16	7.0	1.98

Irregular Waves

	Average Distance	Maximum Distance
Low	0.94	2.34
Moderate	0.94	2.34
High	1.98	3.75

Here the stand-off distance is in no danger of being violated and the waves on the shoreward side are again only about 1/5 the height of those on the seaward side. The interior wall appears to have an overall beneficial effect for this application.

Scour at Base of Structure

Modeling erosion problems is quite complex if precise quantitative measurements are required. In this case, however, it was desired first to establish that perforations reduce scour and then to determine generally how the scour was effected by varying the distance between the bottom of the breakwater and the sand bottom in which it was installed.

Figure 6 shows a schematic drawing of the general scour experiments. A sloping bed of fine sand (dashed lines) was spread under the model which was supported on piles resting on "bedrock". The incident waves were regular with periods of 7, 10, 13 and 16 seconds. The wavemaker was stopped, after the sand deformation appeared to reach a state of equilibrium. Measurements were made of prominent mound locations and depths, as indicated in Figure 6, and from these measurements, the profile of deformation was reconstructed, for each case. In addition, notes were taken on the condition at the base of the piles. Figures 7-9 show some of the results of the scour experiments.

Figure 7 deals with a caisson-type breakwater (solid front and back wall) in which the bottom of the breakwater is on the sand bed. It is seen that the solid-wall breakwater resting on the sand bed is very vulnerable to erosion; the effect increases with wave period, as expected. At 13 and 16 seconds, the erosion under the walls is down to bedrock and would probably have extended deeper if there was more room. When the breakwater is raised off the sand bed (not shown) the effect of scouring diminishes, but is still somewhat pronounced even when the breakwater is 13 feet from the bottom. Equally significant is the observation that the base of the piles was considerably eroded even when the distance to the bottom was a maximum.

When the front wall was perforated (Figure 8), the situation was altered significantly. That is, the front wall sustained very little erosion, while the back wall suffered considerably. Again, the scouring increased with period, but in this case there was no significant difference.

In the final test, both front and back wall were perforated (Figure 9), but only the case of the breakwater resting on the sand bed was treated. As expected, the erosion effects were least, of all the cases tested, and at no time was "bedrock" reached for any wave condition. It is likely that raising the breakwater off the sand bed would have produced still better results as was evidenced in the prior cases.

CONCLUSIONS AND RECOMMENDATIONS

As a result of the experiments reported here and the supporting analytical work, the following conclusions are drawn:

1. There is a substantial force reduction achieved by perforating the front wall of a vertical-wall breakwater. For design purposes, it is likely that a force of 1000 lb./ft. may be adopted with relative safety under most environmental conditions (Tables II and III).

2. The use of an interior perforated wall shows little evidence of being effective in reducing overall force on the structure and is not recommended for such an application.
3. The perforated back wall serves a useful function. Although force measurements were not made, it is obvious that relief of pressure on the back wall must be beneficial. It is important to note that the perforated back wall does not interfere with using the breakwater as a pier. Nor does it permit waves of any appreciable height to form on the shoreward side.
4. Scouring at the base of the breakwater can be a severe problem. The perforations on front and back wall have an astonishing effect on stabilizing sediment transport and reducing erosion, even at the base of piles. It is likely that a relatively small distance between the bottom of the breakwater and the sea-bed will be even more useful in preventing erosion.

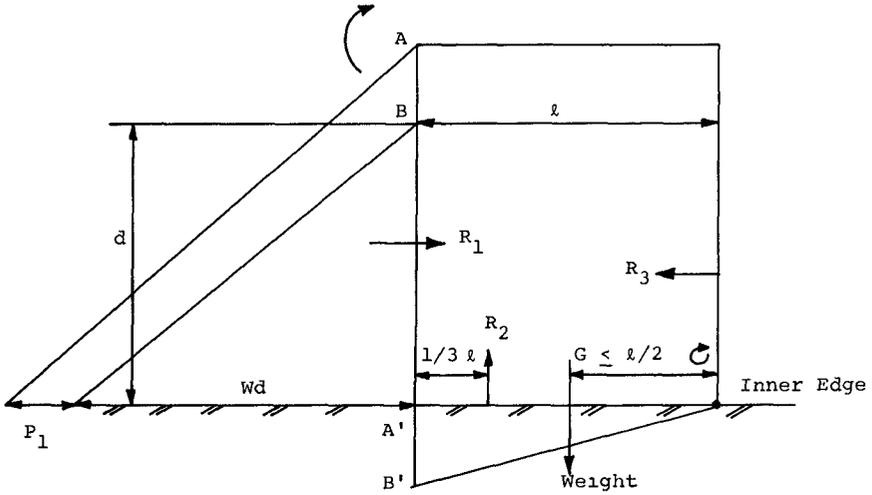
There is now adequate evidence that the perforated breakwater can perform a service that is sorely needed. If further evidence is required, it is recommended that a 1/4 scale model be erected and installed at a suitable locale (Chesapeake Bay, Long Island Sound) where it can be instrumented and observed for one winter season. Adequate data could thereby be collected to justify construction of needed systems.

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R_1 is the resultant of the horizontal pressure (hydrostatic and dynamic)

R_2 is the uplift pressure

R_3 is the resultant of the initial pressure (hydrostatic)

Figure 1 Wave forces on vertical-wall structure.

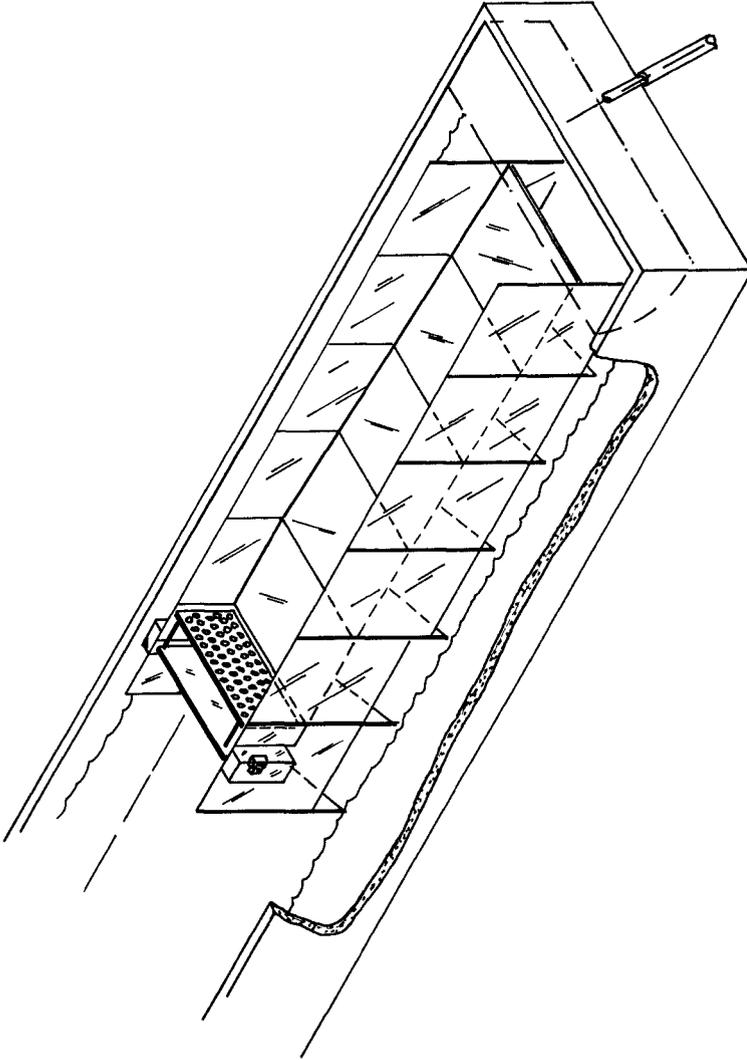


Figure 2 Sketch of OCEANICS shallow-water flume in wave tank.

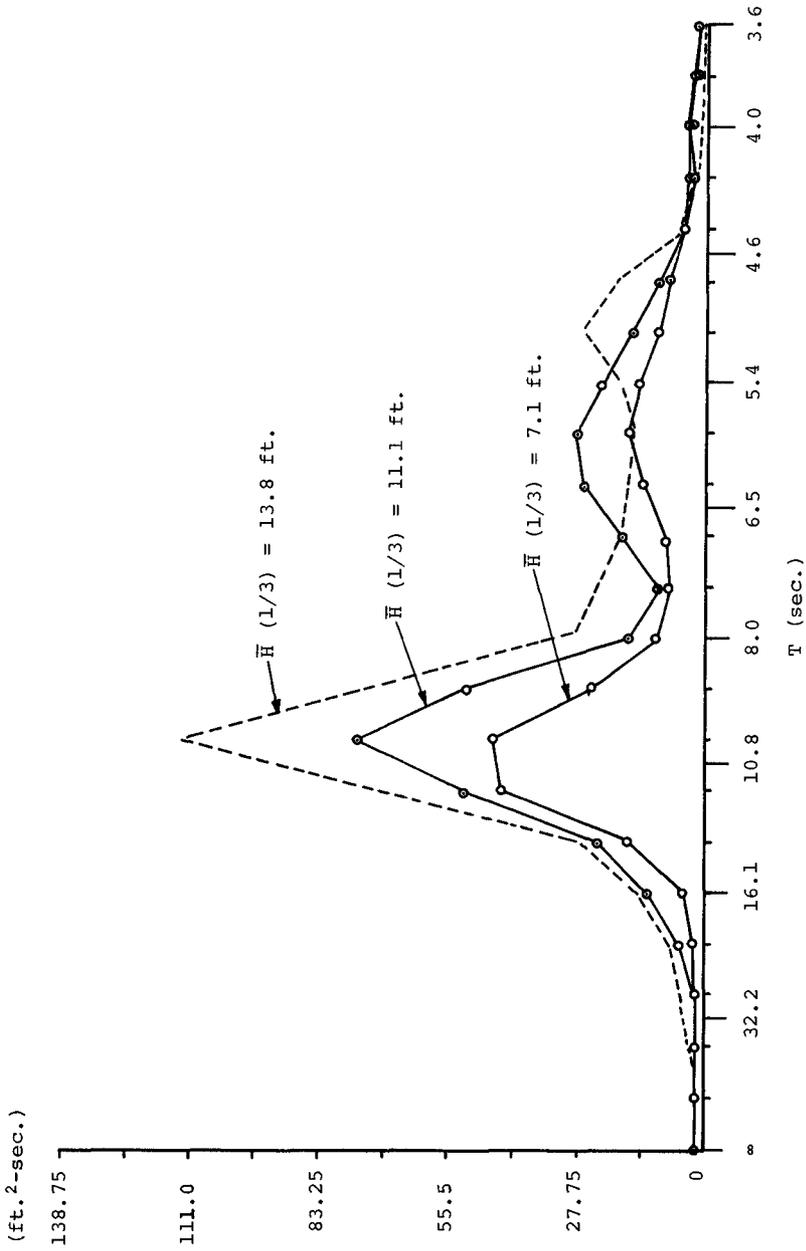


Figure 3 Samples of wave spectra used in model tests.

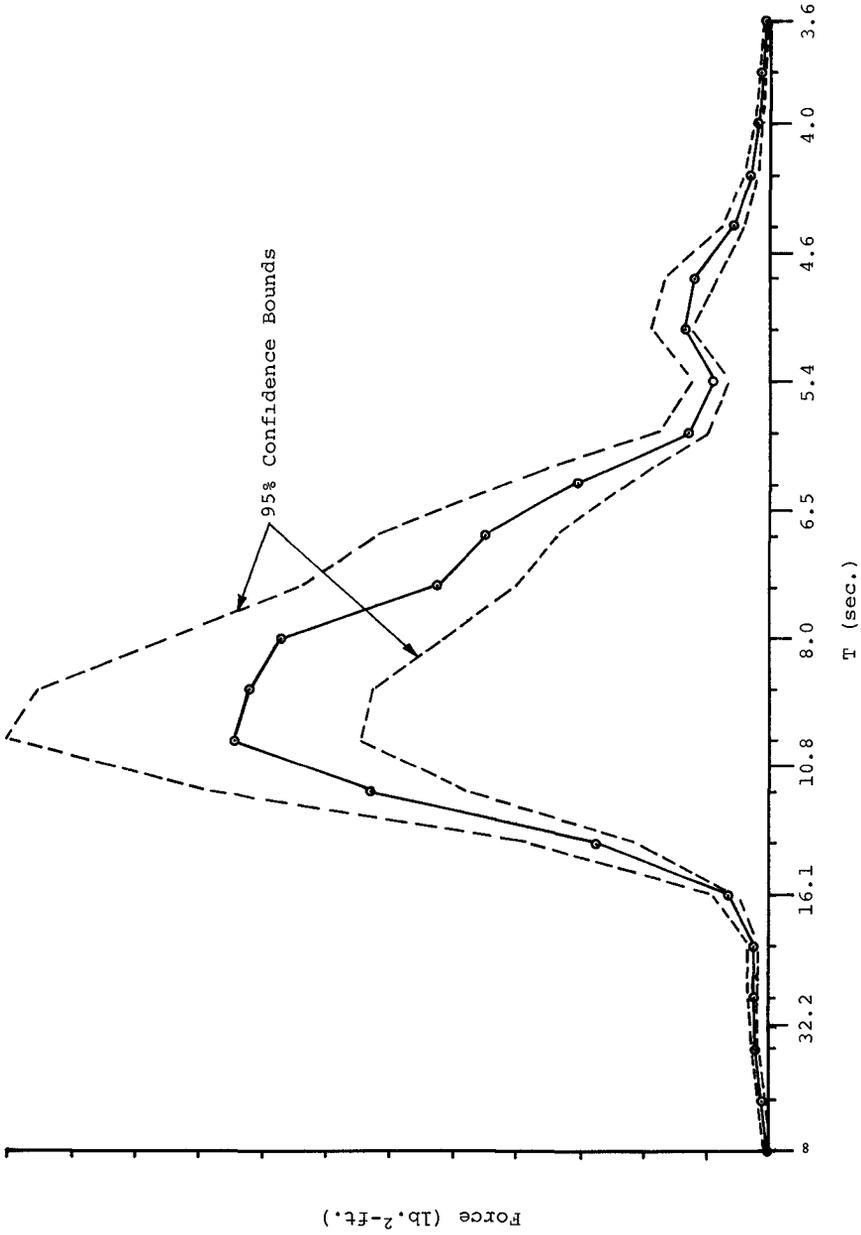


Figure 4 Sample of horizontal force spectrum.

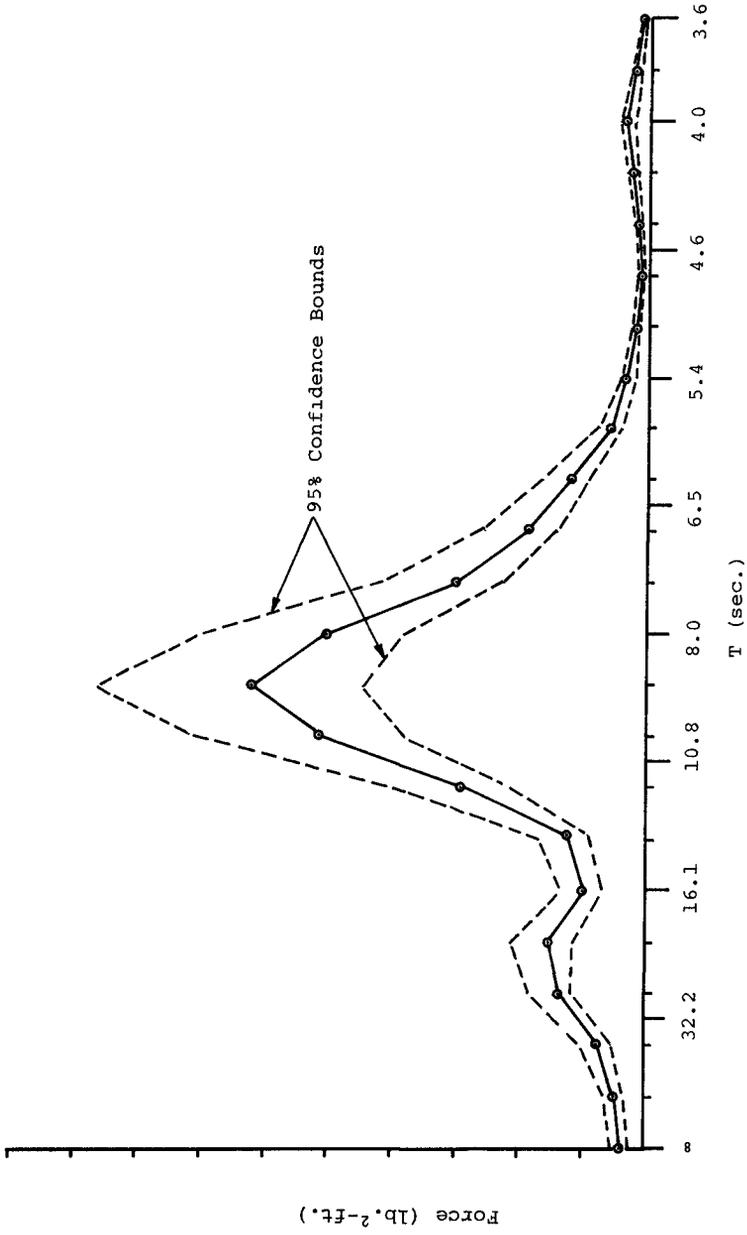


Figure 5 Sample of vertical force spectrum.

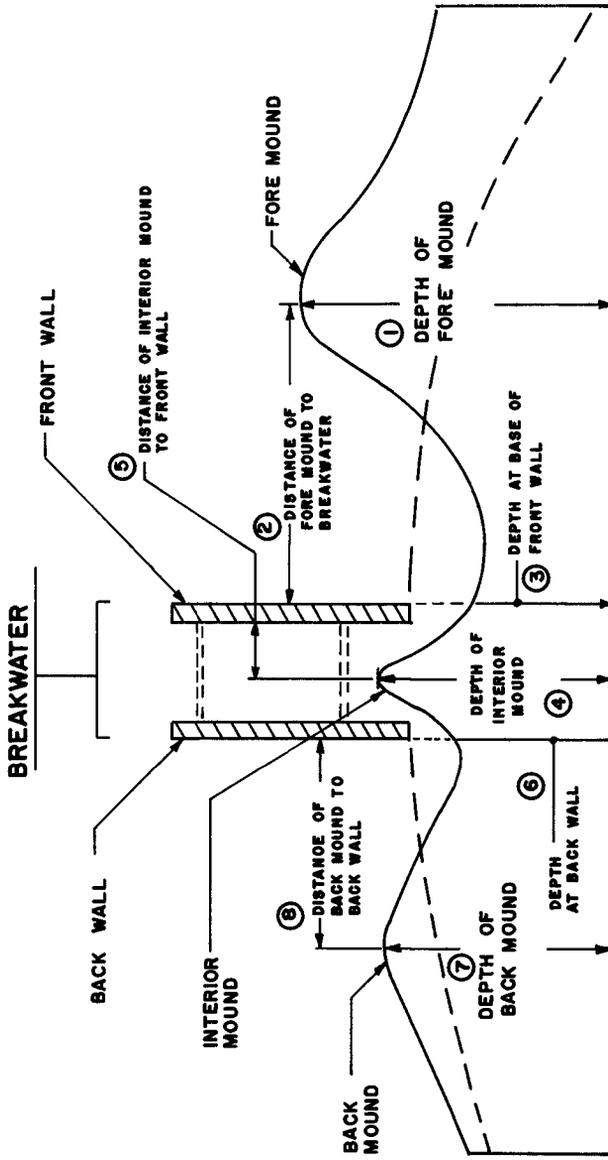


Figure 6 Schematic drawing of scour experiments.

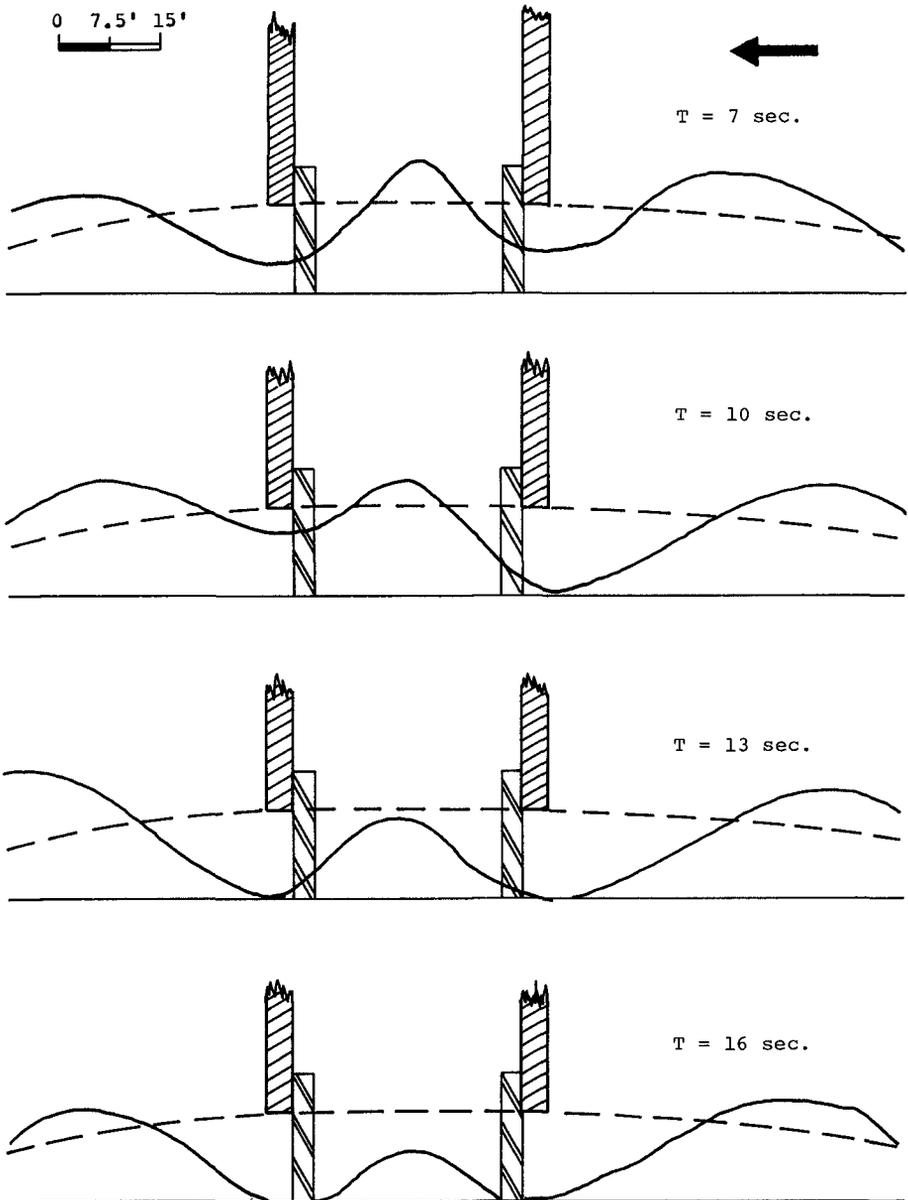


Figure 7 Scour at base of solid-wall breakwater resting initially on sand bed.

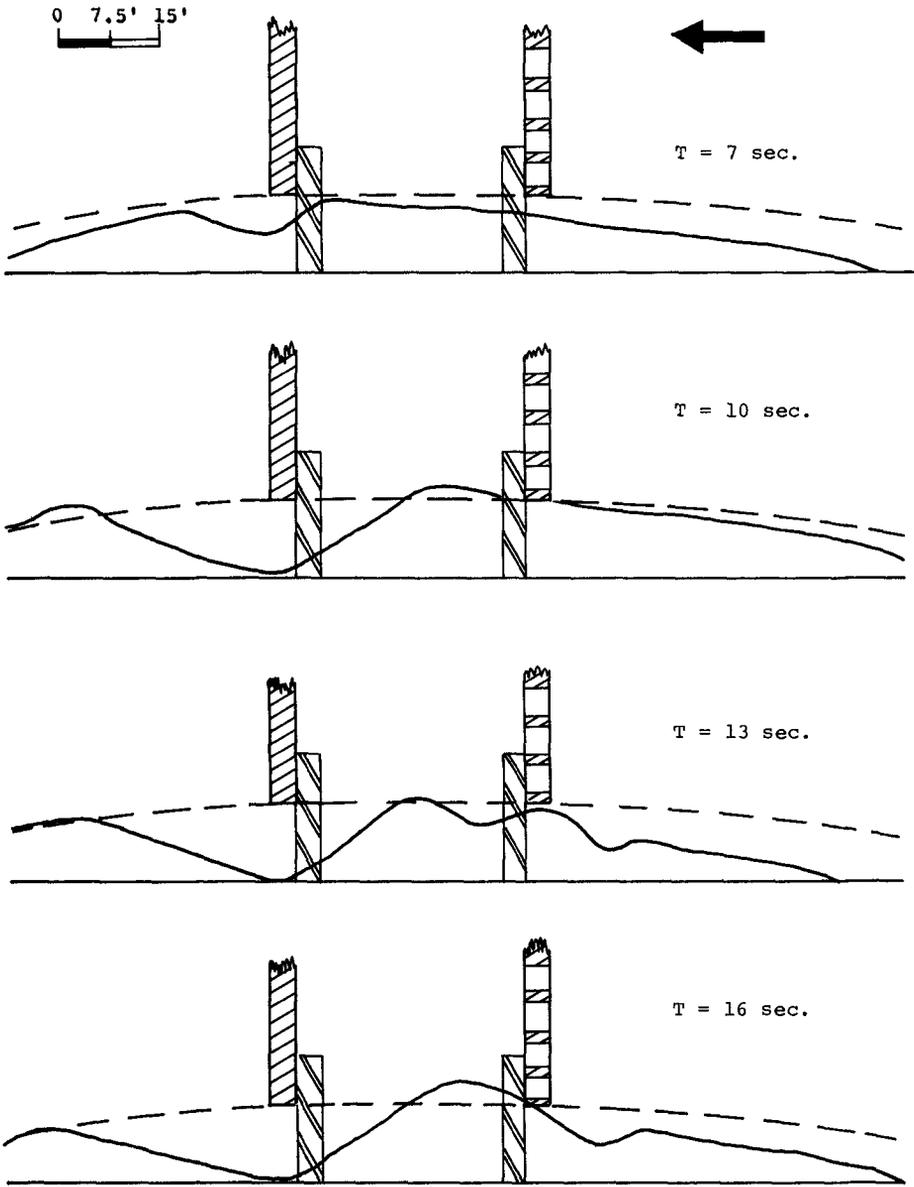


Figure 8 Scour at base of perforated breakwater resting initially on sand bed.

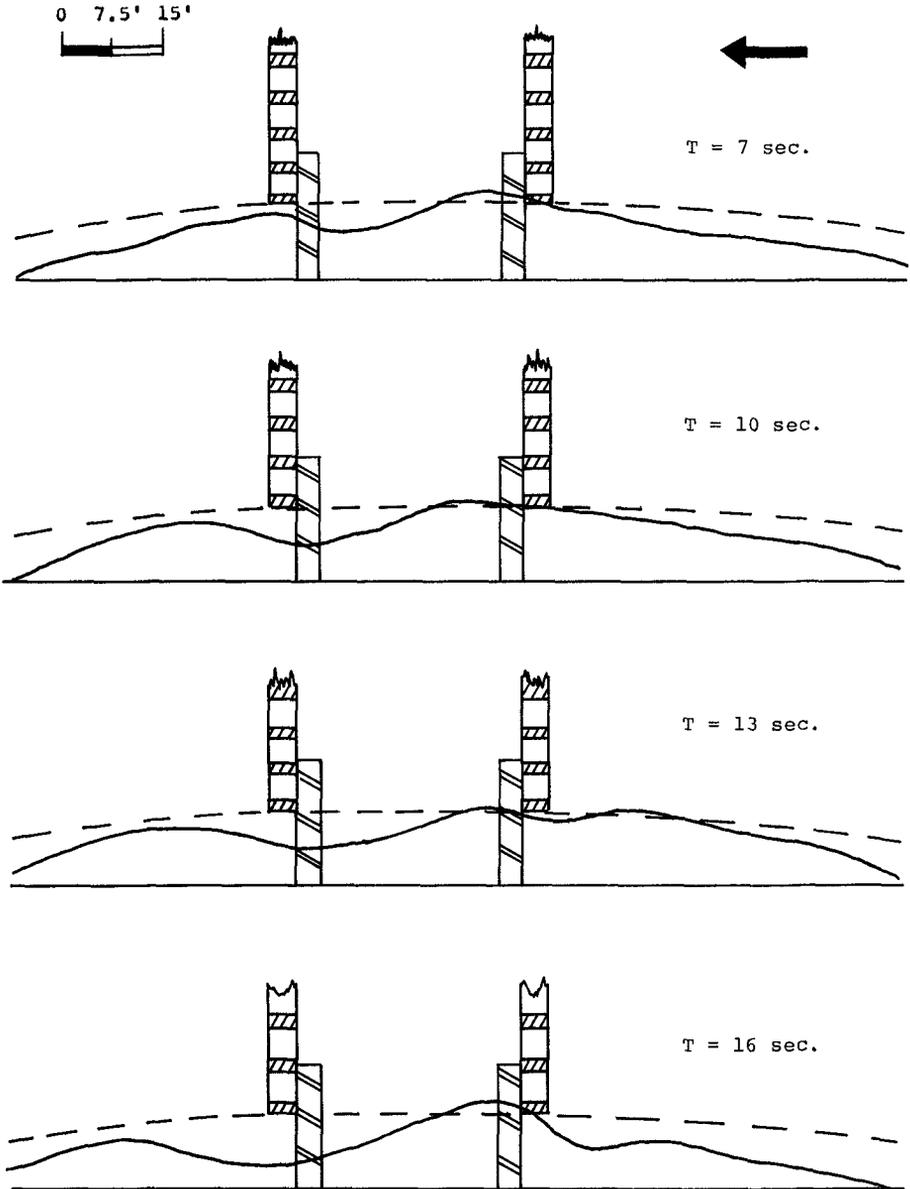


Figure 9 Scour at base of breakwater with perforated front and back walls resting on sand bed.