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HYDRAULIC MODEL STUDY OF PROTECTIVE WORKS FOR FLEET BERTHS IN NARRAGANSETT BAY

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INTRODUCTION

THE model study reported on here had as its purpose, determination of the optimum size and location of a breakwater to protect the existing and future berthing facilities of the United States Navy's Atlantic Destroyer Fleet in Coddington Cove, Narragansett Bay, Rhode Island.

The existing and proposed piers in Coddington Cove are intended to be used as berthing facilities for the repair and refitting of the ships and the relief of their crews. Continued maintenance of seaworthiness is currently necessary in order to permit evacuation at the threat of high seas which normally accompany north and northwest winds. Examination of annual average wind records shows winds from the northwest quadrant to exceed 20 knots, 20% of the time.

The objective of the study was to determine the best size and location of a breakwater compatible with:

1. Maximum protection of ships and structures from wind waves.
2. Minimum interruption of the existing harbor "flushing" created by tidal surface currents.
3. Minimum interruption of littoral sand transport.

A detailed description of the entire study may be found in Reference (1).

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The Prototype

Coddington Cove is an embayment on the western coast of Aquidneck Island near the mouth of Narragansett Bay. It can be found at the lower end of the shaded portion of Figure 1.

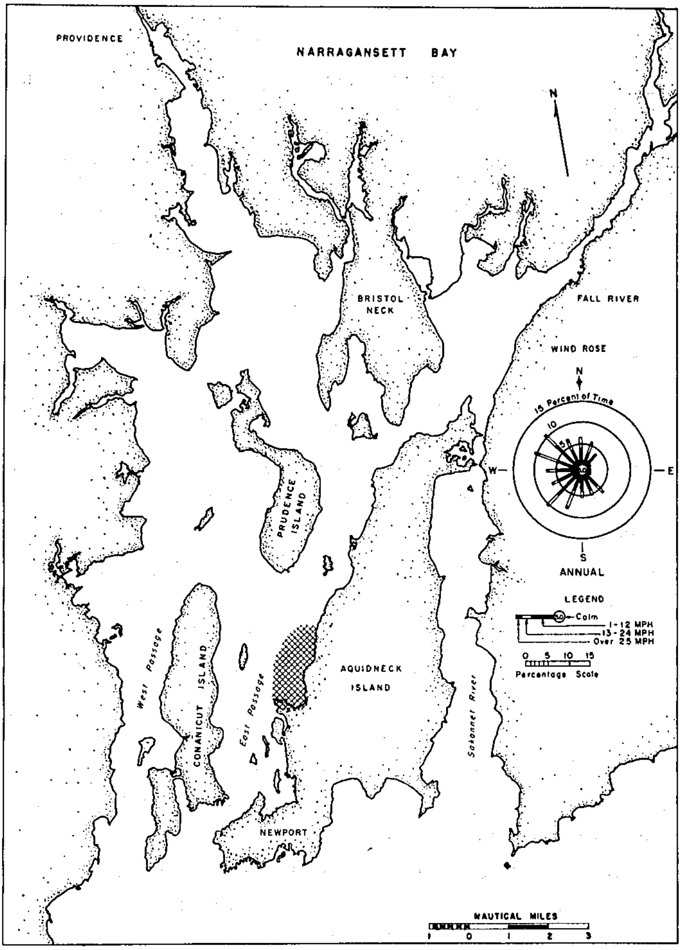

FIG. 1.—LOCATION MAP.

Harbor Structures

During World War II the Coddington Cove area became a part of the expanding U.S. Naval Base at Newport, Rhode Island and various harbor structures were constructed. These structures and existing topography along with proposed improvements are shown in Figure 2.

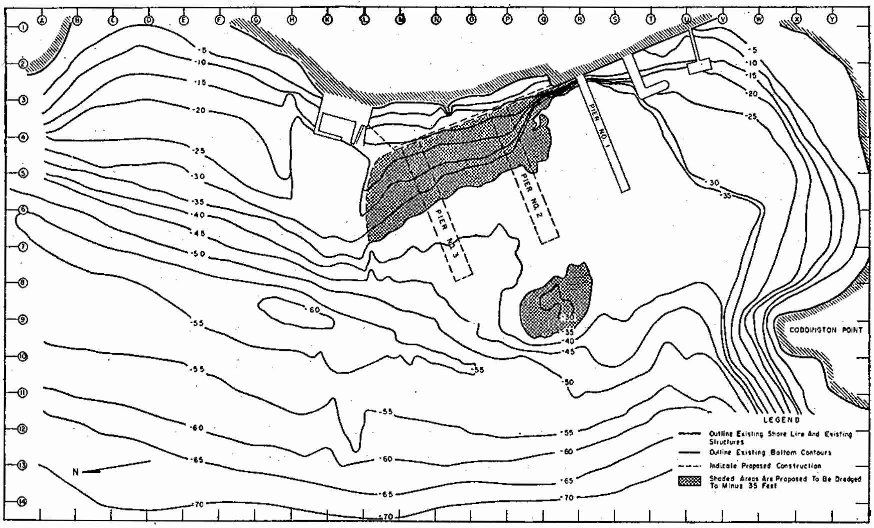

FIG. 2.—DETAILED MAP OF MODELED AREA.

Starting in the bay and working northward along the coast the first structure seen is a dock and transit shed herein called the "south basin." Both arms of this structure are of impermeable construction. Pier No. 1 is 1350 ft. long and 100 ft. wide and rests on 1638, 16 in. monotube piles. Proposed piers 2 and 3 will each be 1525 ft. long and 200 ft. wide resting on 14" cylindrical piles spaced 10 ft. on centers. Both of these piers are designed to be built with sheet pile bulkheads along their northern side to yield protection for ships berthed in their lee until such time as a breakwater can be constructed. The northernmost structure is a dock and ferry slip herein called "north basin" and has a substructure of timber piling. The proposed rubble breakwater has a 10 ft. top width, side slopes of approximately 1¼ to 1 and is to be faced on the weather side with 7 ton stones.

Available Oceanographic Data

1. Detailed records of wind direction, intensity and duration have been obtained by personnel of the U.S. Weather Bureau's Airport Station at Hillsgrove, Rhode Island, since 1949.

2. Tidal current measurements have been made at several points in the Coddington Cove area and are listed by the U.S. Department of Commerce (2).

3. Visual inspection of the Cove and adjacent coast showed an absence of fine sand and comparison of sets of serial photographs indicated no appreciable coastal changes since the beginning of World War II.

4. The only wave data available for Narragansett Bay are those by Hicks (3) which represent a very few observations of height and direction in the East Passage.

Because of the need for information concerning significant wave periods as well as wave heights, a capacitive type prototype wave gage was constructed and installed in 35 feet of water at the end of Pier No. 1. Unfortunately, the model study was completed before the statistical analyses of the resulting measurements were available. Following, therefore, is a description of the techniques used to obtain from the available wind data, estimates of the waves which have occurred in the harbor area and may thus be expected to occur again.

Wave Statistics by Hindcasting Procedures

Hindcasting may be defined as the application of wave forecasting techniques to past meteorological data in order to determine the wave conditions which existed at that time.

A very brief description of the two most commonly used techniques, their differences and a definition of terms is in order.

Figure 3 depicts the variation of water surface elevation with time at a given point for two different conditions. Figure 3a shows the pure or "classical" wave form which repeats itself exactly every T seconds and is governed, for small H, by the familiar equation:

$$C = \frac{L}{T} = \left[\frac{qL}{2\pi} \tanh \frac{2\pi d}{L} \right]^{1/2}$$

in which:

$C =$ wave celerity in ft/sec
$T =$ wave period in sec.
$L =$ wave length in ft.
$g =$ gravitational constant, 32.2 ft/sec^2
$d =$ stillwater depth

For deep water ($d > L/2$) this wave can be described completely by the height, H, and either the length or period.

The wind never generates such well defined wave trains with uniform heights, periods, or wave lengths. The energy supplied by the wind to the waves creates surface disturbances of widely varying period, length, height, phase (and also direction) with each individual

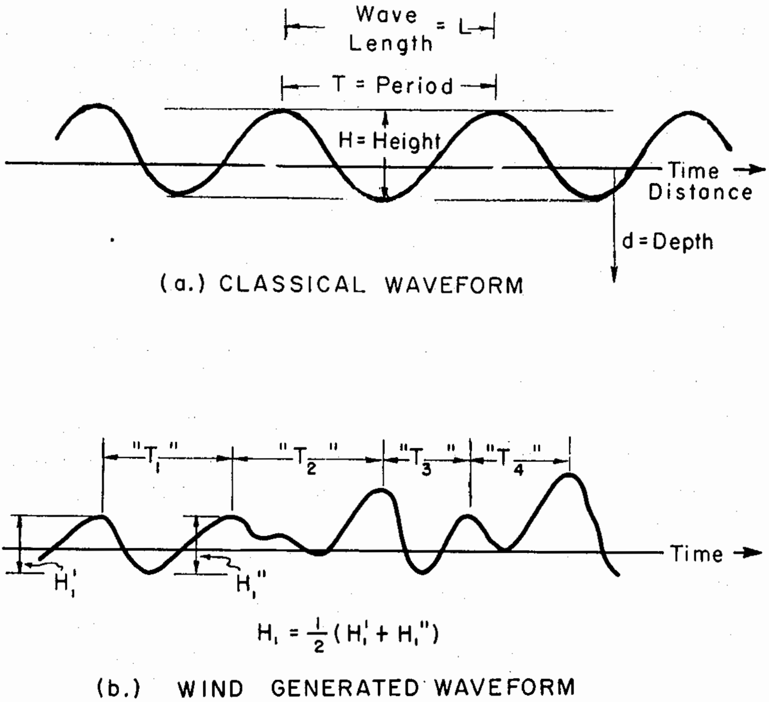

FIG. 3.—COMPARISON OF CLASSICAL AND WIND GENERATED WAVE FORMS.

disturbance following the classical laws. The composite of all these various periodic components is known as the "sea" (Fig. 3b) or the "spectrum" of ocean waves.

It is difficult to describe this wind generated wave pattern by means of only two parameters such as the "wave height" or "wave period" of some fictitious wave of classical form.

Two approaches to deep-water forecasting have grown out of an attempt to supply a simple yet adequate description of the true sea surface. Both methods begin with the equations of motion describing the transfer of energy between wind and water and both use the same data on observed wave characteristics to bridge the gap between theory and a workable forecasting technique.

The Sverdrup-Munk method (4, 5), as modified by Bretschneider (6) describes the sea in terms of a "significant" wave which is determined once the following data are available:

a. The fetch or distance over which the generating wind blows.
b. The wind direction, speed and duration within the fetch.
c. Wind conditions which may exist between the fetch and the target.
d. The decay distance from the fetch to the target.

The "significant" wave is defined by:

a. "Significant" wave height: the average height of the highest one-third of all observed waves larger than some arbitrary minimum.
b. "Significant" wave period: the average period of the one-third highest waves.

The Pierson-Neumann-James approach (7) utilizes statistics to forecast the distribution of the square of the wave height as a function of the period of the classical components making up the sea. This distribution is called the energy spectrum and its time derivative is the power spectrum. This technique also provides the two parameter description of the former method. However, it utilizes the average of all waves greater than the chosen minimum to define a representative mean height and mean "period." Component or true periods are denoted by T and significant or mean "periods" by "T."

These techniques were developed for generation in deep water and since the average depth of Narragansett Bay to the northwest from the Cove is approximately 30 feet, all generated waves having periods of 3.5 sec. or more will be shallow water waves ($d < L/2$).

Bretschneider (8) has studied shallow water generation and has found from theoretical considerations that both the significant "period" and height of the shallow water wave will be less than those of that wave which is generated by the same wind over the same fetch in deep water.

Using both laboratory and field data, however, Sibul (9) found the significant shallow water wave height to be slightly larger than the deep water value for high winds and the depth to have only a slight decreasing effect on the significant wave "period" for the range of fetch, wind speed and depth of interest in this study.

It was thus decided to use the accepted deep water hindcasting technique for prediction of significant wave height and "period."

The state of growth of waves due to wind of a given intensity has been stated to be a function of both fetch and duration. As a wind begins to blow with a given intensity across a body of water, the wave height at any given distance from the upwind shore will increase with time and reach an essentially steady state at a time denoted as the minimum wind duration for that intensity and fetch. If one then examines the steady state conditions as successively greater fetches, one finds the spectrum to be correspondingly broader. There is a certain minimum fetch and a minimum duration for this wind speed for which all possible wave components in the spectrum are present with their maximum energy. In this state, the sea is called "fully arisen."

Application

The annual wind rose for Airport Station, Hillsgrove, R. I., is presented in Fig. 1 as compiled by Fay, Spofford and Thorndike, Inc. Two directions are seen to be important, northwest and southwest.

Examining the location map the following maximum fetches are obtained:

TABLE I

Wind Fetches at Coddington Cove

Direction	Fetch to end of Pier No. 1 (Nautical Miles)
N 8° E	10.1
N 22° W	10.1
S 42° W	3

Examination of the wave data for the dates on which damage occurred (10) yields:

TABLE II

Wind Conditions Leading to Reported Damage

Date	Intensity (Knots)	Duration (hrs.)	Direction
Nov. 20, 1955	25	18	NW
Mar. 16-17, 1956	35-40	12	NNW

It is apparent from the above figures that the north and northwest directions are critical. Islands shelter the Cove from Westerly winds and the sinuosities of the East Passage apparently filter all but the shortest components of a S.W. ocean wave.

A fetch of 10 nautical miles was thus chosen for the study of expected waves.

In Fig. 4 the relationship given by Sverdrup and Munk (5) defining the minimum wind duration necessary for steady state conditions of wave growth is plotted as a function of wind intensity for a fetch of

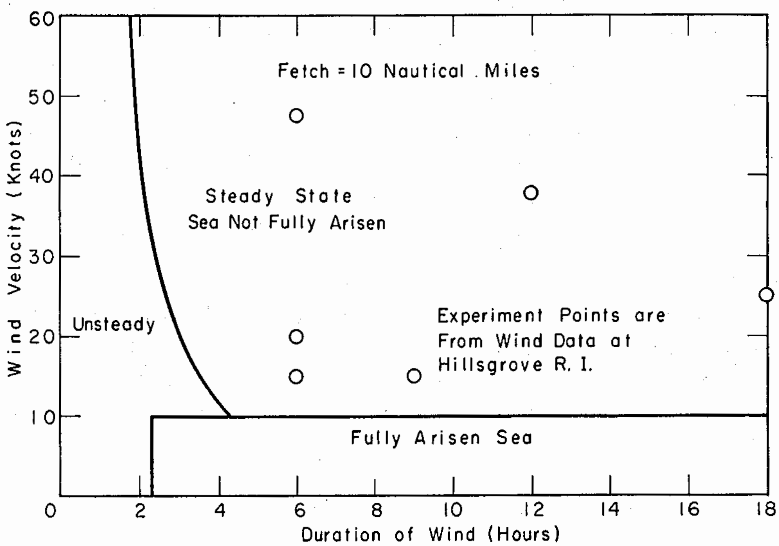

FIG. 4.—WIND SPEED VS. MINIMUM DURATION FOR STEADY STATE DEVELOPMENT.

10 nautical miles. The relationship of Pierson, Neumann and James (7) defining the maximum wind intensity and minimum duration allowable for a fully arisen sea to be obtained at the end of the fetch is also shown.

Some of the wind data including the two storms for which damage was reported (10) are plotted on Fig. 4, and clearly indicate the fetch and not the duration to limit the growth of the sea. For the winds of interest therefore, the state of the sea will be steady but not fully arisen.

Using the maximum fetch of 10 n.m. wave heights and periods were hindcast by both techniques described above. The resulting characteristics are shown in Fig. 5 as a function of wind intensity. Shown on the plot are:

(a) Significant height and "period" by Sverdrup-Munk-Bretschneider (6).
(b) Average height and "period" by Pierson-Neumann-James (7).
(c) True period, T_{max}, at which the maximum spectral energy is concentrated, by Pierson-Neumann-James (7).

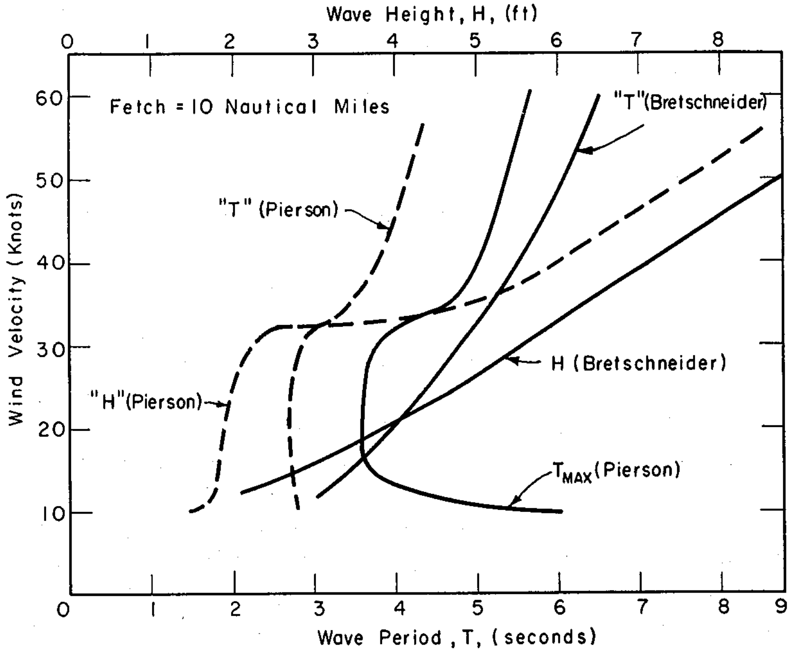

FIG. 5.—HINDCASTS OF WAVE HEIGHT AND PERIOD.

Since the experimental equipment to be used can generate only pure waves it is desired to model, for each wind speed, a wave with the period, T_{max}, at which most of the spectral energy is concentrated, and having the significant height.

Because of its broader background of successful usage the Sverdrup-Munk-Bretschneider technique was selected as the predictor. It was recognized that T_{max} will probably be larger than the given deep water "T" and also that the actual shallow water "T" will be somewhat less than that shown in Fig. 5. With these compensating factors in mind the apparent "periods" of Bretschneider were chosen as most nearly representing the component wave at which most of the spectral energy is concentrated.

Two basic waves were chosen as representative of moderate and severe storm wind conditions:

TABLE III

Prototype Characteristics of Chosen Representative Waves

Wave No.	Wind Velocity (knots)	Wave Period (sec.)	Wave Height (ft.)
1	30	5.0	5.5
2	60	6.5	10.0

The Model

Scale Selection

Model size requirements and laboratory space restrictions dictated a horizontal scale ratio:

$$L_r = \frac{L_{model}}{L_{prototype}} = 1/400 \qquad [1]$$

Since the mean depth in the vicinity of the piers is only 40 feet an undistorted model would thus have only 0.1 ft. of water in the anchorage. This is undesirably small for several reasons. In order to appreciate these reasons the basis for similitude is briefly reviewed.

The basic requirement for a satisfactory model is that those of the basic forces (gravity, viscosity, surface tension, elasticity) which are important in determining the prototype behavior be important to the same relative degree in the model. It is practically impossible to satisfy this requirement exactly in any model study. A compromise solution is to reproduce to proper scale the one (or possibly two) basic forces causing the primary prototype phenomena. Similarity is thus sacrificed to a more or less predictable degree for those secondary phenomena dependent upon other forces.

For the model study under consideration, the primary phenomena, waves and tidal surface currents are both gravity phenomena. The "Froude Law"[1] will therefore govern the model-prototype relationships yielding:

[1] By "Froude Law" is meant the obtainment of dynamic similarity where gravity forces predominate through equality of Froude Numbers, F, at geometrically similar points in model and prototype. $F = \frac{V}{\sqrt{gy}}$ where V is the local fluid velocity and y is a characteristic length (often the local depth).

$$\text{Velocity ratio} = V_r = \frac{V_{model}}{V_{prototype}} = L_r^{1/2} \qquad [2]$$

$$\text{Time ratio} = T_r = L_r V_r^{-1} = L_r^{1/2} \qquad [3]$$

Because of the presence of a free water surface, reduction of scale to model size may make surface tension forces important whereas their role in the prototype is infinitesimal.

To a vertical scale of 1:400 a 4 foot prototype wave would be only 0.1 in. high which is of the order of the meniscus at the measuring element. Accuracy of vertical measurement would thus be lost.

Dissipation of the energy of these small waves at the beaches, in the breakwater and among the pilings would be controlled by the large surface tension forces.

Similarity with respect to tidal currents is dependent upon the effects of viscosity being small in the model; i.e., model Reynolds numbers[2] must remain large. Reduction of velocities by the Froude Law produces model Reynolds numbers considerably less than prototype values.

A solution to these problems, but one which introduces new inaccuracies of its own, is to give the horizontal and vertical scales separate consideration and construct a distorted model. Such distortion must not, however, for reasons of its own, destroy the similarity between model and prototype.

Examining Equation [1], the familiar relationship governing the celerity of small amplitude gravity waves, we find that the celerity is a function of both depth and wave length (i.e. a vertical and a horizontal dimension) for $0.5 > d/L > .025$. The significant prototype range of the latter parameter is $.55 > d/L > .19$ for the two waves chosen thus we must write:

$$C_r^2 = \left[\frac{C_{model}}{C_{prototype}}\right]^2 = \frac{L_m}{L_p} \frac{\tanh \dfrac{2\pi d_m}{L_m}}{\tanh \dfrac{2\pi d_p}{L_p}} \qquad [4]$$

Distortion of a model concerned with shoaling waves is treacherous in this range of d/L since it will delay the effect of the bottom until

[2] Reynolds numbers, R, should be equal at geometrically similar points in model and prototype to insure dynamic similarity if viscous forces are important. $R = \frac{Vy}{v}$ where v is the kinematic fluid viscosity and the characteristic local length is the depth in this case.

the wave is closer to shore and will thereby modify the refraction pattern.

The effect of scale distortion on the refraction pattern was investigated theoretically as described below. A distortion of approximately 3 to 1 was selected as producing tolerable departures from the true refraction pattern and yet yielding measurable values of wave height.

The maximum variance of the celerity ratio at the edge of the model from its deep water value,

$$C_r^2 = L_r, \qquad [5]$$

may be seen by substituting into Equation [4] conditions accompanying the 60 knot prototype wave in 70 feet of water. This yields

$$C_r^2 = 1.03\ L_r. \qquad [6]$$

For the purposes of this study, therefore, the celerity ratio is assumed given by Equation [5].

A summary of the transfer relationships used in this study follows:

TABLE IV

Model-Prototype Transfer Relationships

Quantity	Relationship	Magnitude
Horizontal Scale	L_r	1:400
Vertical Scale	Y_r	1:120
Wave Period Ratio	$L_r^{1/2}$	1:20
Tidal Current Velocity Ratio	$Y_r^{1/2}$	1:11

Reliability

As was mentioned above, the effect of vertical distortion is to keep the model wave a deep-water and thus unrefracted wave farther inshore than in the prototype or in an undistorted model.

The effect of the distortion on the refraction pattern is cumulative thus refraction diagrams were plotted for the longest wave in order to evaluate the magnitude of the error introduced. In Fig. 6 two sets of refraction lines (wave crests) are shown; one is for normal depths as given by the contours of Fig. 2; the other is for depths 3.33 times the normal. Comparison of the results shows how the model refraction (solid lines) differs from prototype refraction (dashed lines). Serious discrepancies appear only very near the shore for the important case of northwesterly waves.

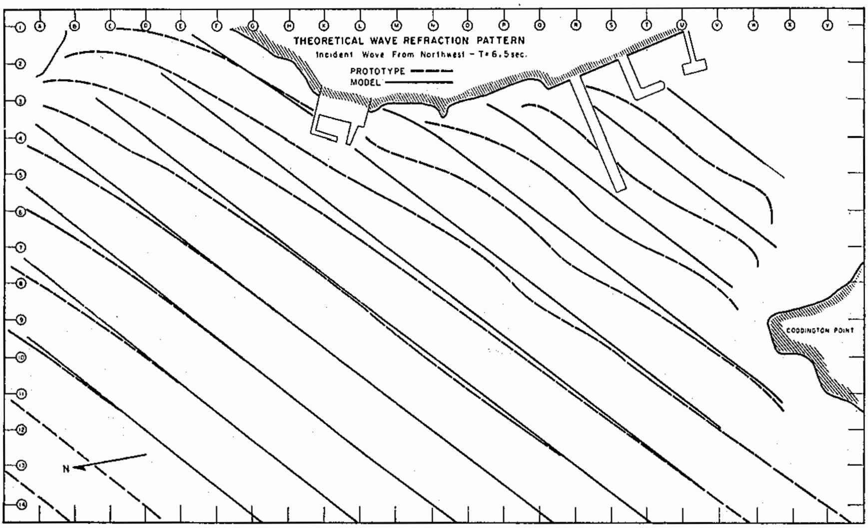

FIG. 6.—WAVE REFRACTION DIAGRAM — EFFECT OF MODEL DISTORTION.

Further detailed considerations of reliability are given in Ref. (1); however, a summary of the additional important conclusions follows:

1. Model wave lengths may be up to 16% too long in the zone of interest.
2. Periods at which abnormal wave heights are observed as a result of local resonance may be in error by as much as 8% for prototype periods up to 7 seconds.
3. Model wave heights tend to be from 10 to 15% too low but the error in estimation of significant height may be ± 15%.
4. Surface current patterns should be accurate in so far as the patterns are independent of current strength.

EXPERIMENTAL EQUIPMENT AND PROCEDURES

The experimental model basin consists of a rectangular area 50 feet long and 28 feet wide enclosed by a cement block wall 17 inches high. Waterproofing of the basin was achieved through a .040 inch coating of vinyl plastic applied to floor and walls.

The basin is filled and emptied from below through a 5″ line cut into the basin floor.

Circulation of water within the basin for tidal current simulation is provided by a small portable pump.

Model topography was laid out on female templates spaced with concrete blocks and was brought to grade by carefully trowelled cement.

All three model piers and the structure of the north basin were constructed of clear plastic using 0.035 inch diam. copper wires to simulate piling. The south basin was constructed of cement.

The model breakwater was built in six inch sections in order to facilitate revisions in its location. That portion below Elev. —10 ft.

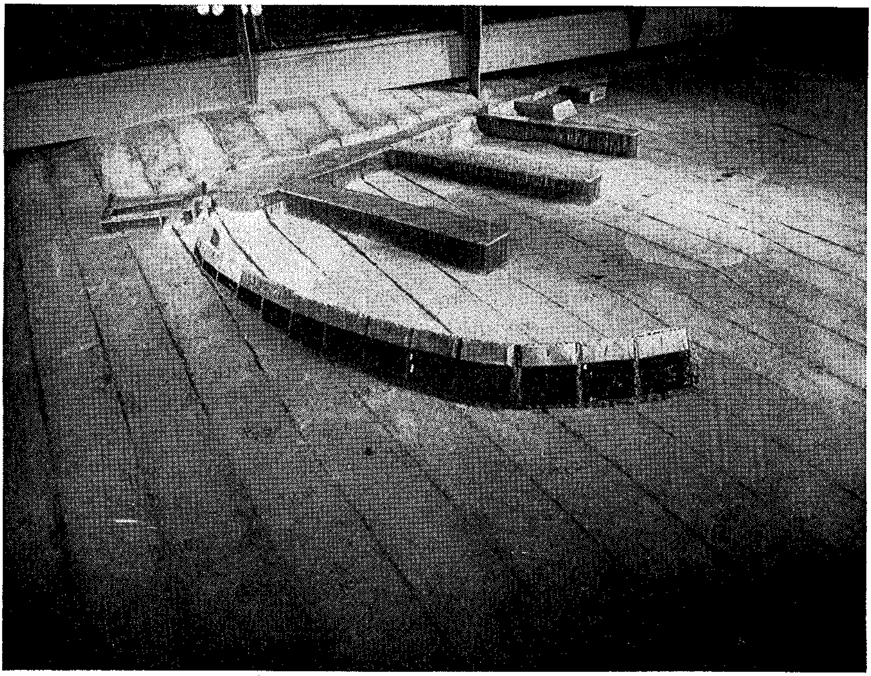

FIG. 7.—MODEL HARBOR STRUCTURES.

was made impermeable and was cast out of plaster of paris. From Elev. —10 to +10 ft. the sections consisted of wire baskets filled with 0.2 in. diam. gravel which corresponds to the 5 ton prototype stone reduced by the average scale ratio. In order to prevent the exaggerated damping accompanying the shoaling of the model waves at the very low Reynolds numbers of an undistorted model it was decided to distort the breakwater slopes as well.

Model harbor structures are shown in Fig. 7.

Waves were generated by means of a movable generator of the vertical plunger variety (see Fig. 8) having continuously variable speed and stroke.

Determination of wave characteristics was accomplished using variable capacitance wave probes as the active arm of wheatstone bridge circuits, the outputs of which were fed to a multiple channel recording oscillograph.

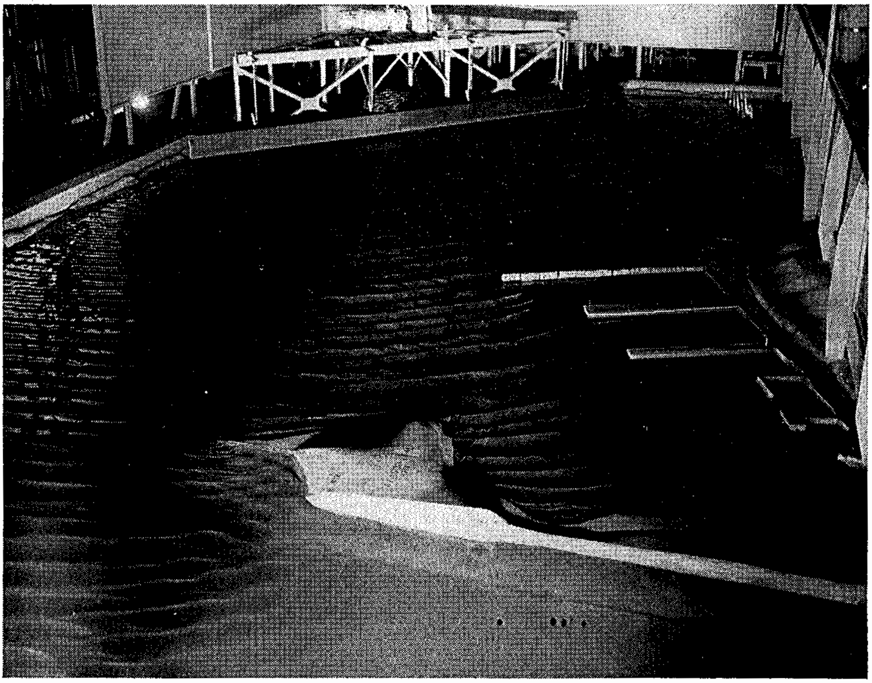

FIG. 8.—VIEW OF WAVE GENERATOR AND MODEL BASIN.

As shown in Fig. 9, the model was gridded at 1 foot intervals to provide reference points for wave height measurement. The wave generator was adjusted until the desired wave was obtained at the reference station, C-14.

The original and proposed structural and hydrographic configurations were investigated with respect to waves and tidal currents for the various breakwater locations shown in Fig. 9 and in the presence and absence of a solid bulkhead on the north side of Pier No. 2.

In order to determine the tendency of any harbor configuration toward resonant response to wave excitation, wave height measurements were taken at several selected spots (see Fig. 9) in the model while the period of the incident wave was varied in finite intervals over a wide range.

With the exception of tests concerning overtopping of the breakwater, all experiments were performed at MLW. Absence of a suitable

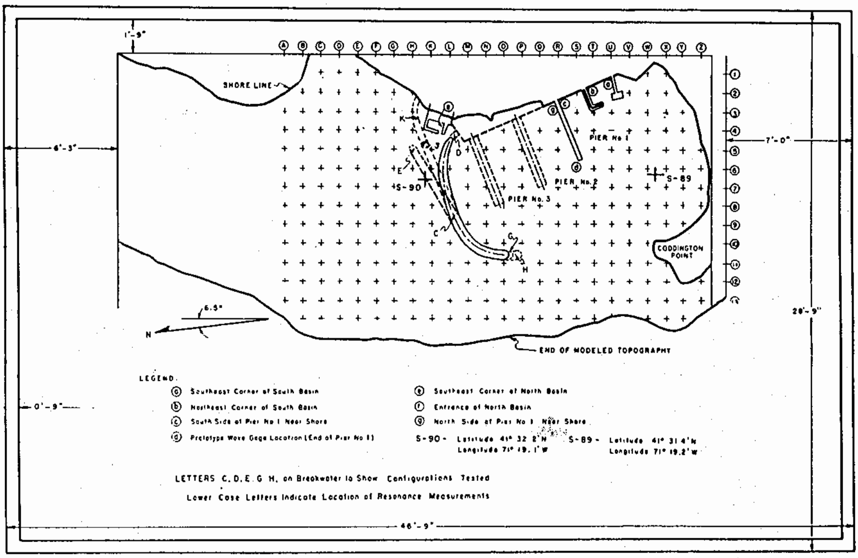

FIG. 9.—MODEL CONFIGURATIONS TESTED.

tide generating mechanism precluded generation of the time varying tidal current, however, two U.S.C. and G.S. current stations lay within the area of modeled topography and as a substitute a steady ebb or flood current was provided by a pump and movable walls were positioned in order to simultaneously bring the mean current magnitude and direction to the proper value at these points. Current measurements were made by observing the motion of floating cork wafers.

SUMMARY OF RESULTS

In the limited space available here it is not possible to present more than a brief summary of the results obtained.

Tidal Surface Currents

In Fig. 10a the ebb flow pattern of mean tidal surface currents is presented for the unimproved configuration. The main observations to be made are the presence of a large, single-celled circulation system in the Cove and the absence of observable motion through and south of Pier No. 1.

In Fig. 10b the effect of a typical breakwater configuration is shown. The pattern is now double-celled and much weaker while the area between piers is essentially stagnant with or without the Pier 2 bulkhead. Similar results were obtained for all other breakwater configurations tested.

It became apparent that the presence of the body of an effective breakwater, no matter what the disposition of its ends, destroys any slight cleansing current. It thus appears that if wave motion is to be reduced, a pollution problem exists which must be solved separately.

None of the mean current magnitudes observed were large enough to cause any navigational difficulties. However instantaneous magnitudes may, particularly near the tip of the breakwater. It must be remembered that wind and wave induced currents will also be present and may, in fact, control the net surface current pattern. Determination of their magnitude was not within the scope of this study. However, their effect on the pollution question is apparent. The prevailing winds and thus the waves are from the northwest. Thus all surface refuse will be piled up in the stagnant area south of Pier 1. The negative conclusions from tidal current studies conducted in the absence of wave motion are therefore conservative from the pollution standpoint.

Wave Height

All data presented in this section will be for the wave resulting from the 60 knot northwest wind (herein called "Northwest No. 2").

A map of the water surface in the unimproved case is shown in Fig. 11a.

For comparison Fig. 11b shows the effect of the breakwater for an improved configuration. As might be expected the waves diffract around the outer end of the breakwater and decrease in amplitude as their energy spreads out over a longer crest. The breakwater length can thus be seen to be critical in determining the degree of protection afforded the outer berths of piers 1 and 2. Wave heights outside the breakwater are increased in the presence of the breakwater due to wave reflection. This tendency is somewhat exaggerated in the model due to the distorted slopes.

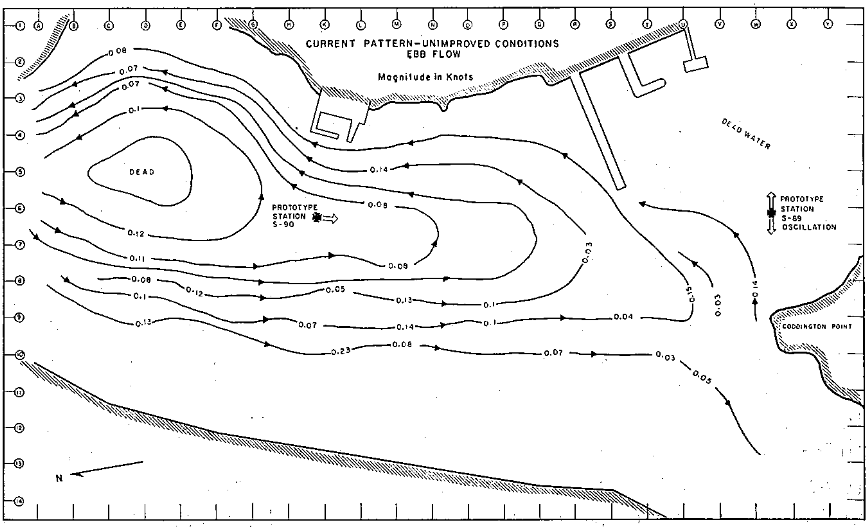

FIG. 10a.—MEAN TIDAL SURFACE CURRENTS, EBB FLOW, UNIMPROVED.

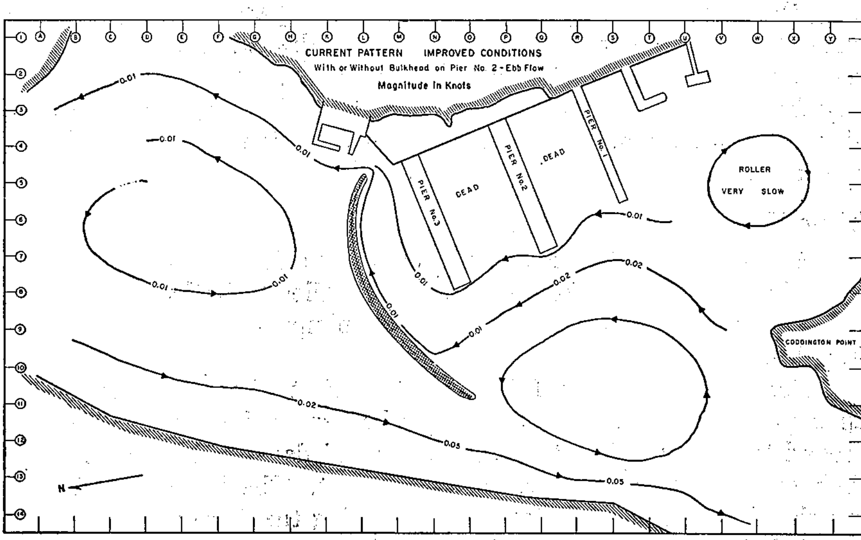

FIG. 10b.—MEAN TIDAL SURFACE CURRENTS, EBB FLOW, IMPROVED CONFIGURATION.

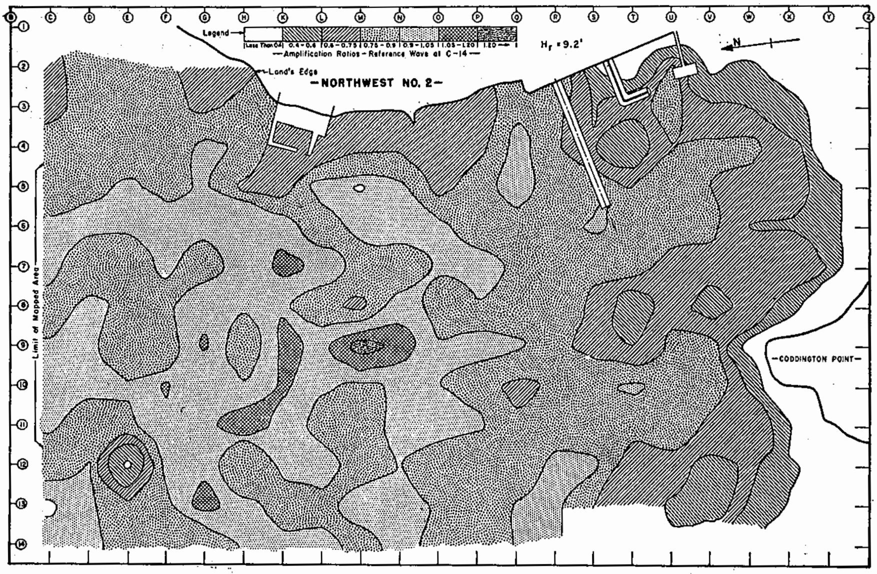

FIG. 11a.—WATER SURFACE MAP, UNIMPROVED CONFIGURATION.

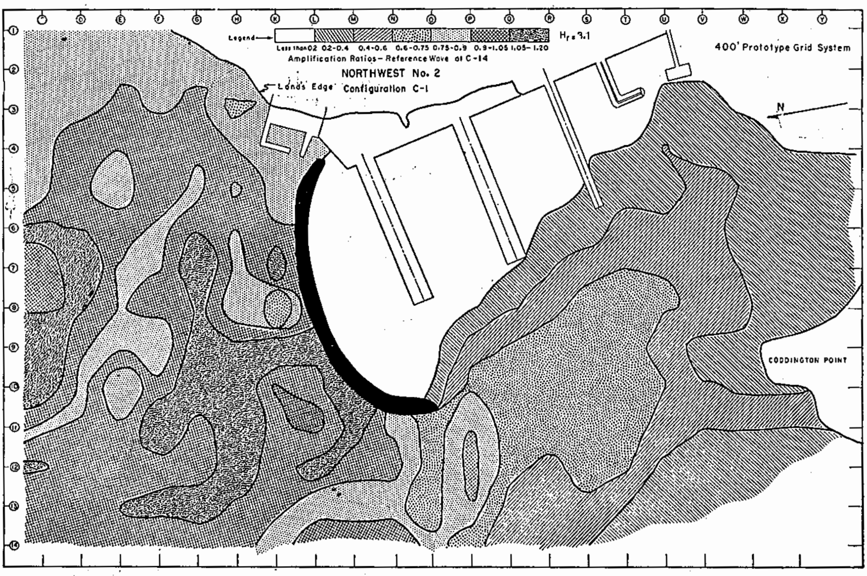

FIG. 11b.—WATER SURFACE MAP, IMPROVED CONFIGURATION.

In Fig. 12 the effects of the sheet pile bulkhead along the north side of Pier 2 are shown clearly. The bulkhead side of the pier is rendered untenable due to reflections but conditions at berths in the lee of this bulkhead are considerably improved. Also discovered but not shown is the fact that enough of the wave energy reaching the bulkhead even in the presence of a breakwater is reflected northward to double the heights experienced at Pier No. 3.

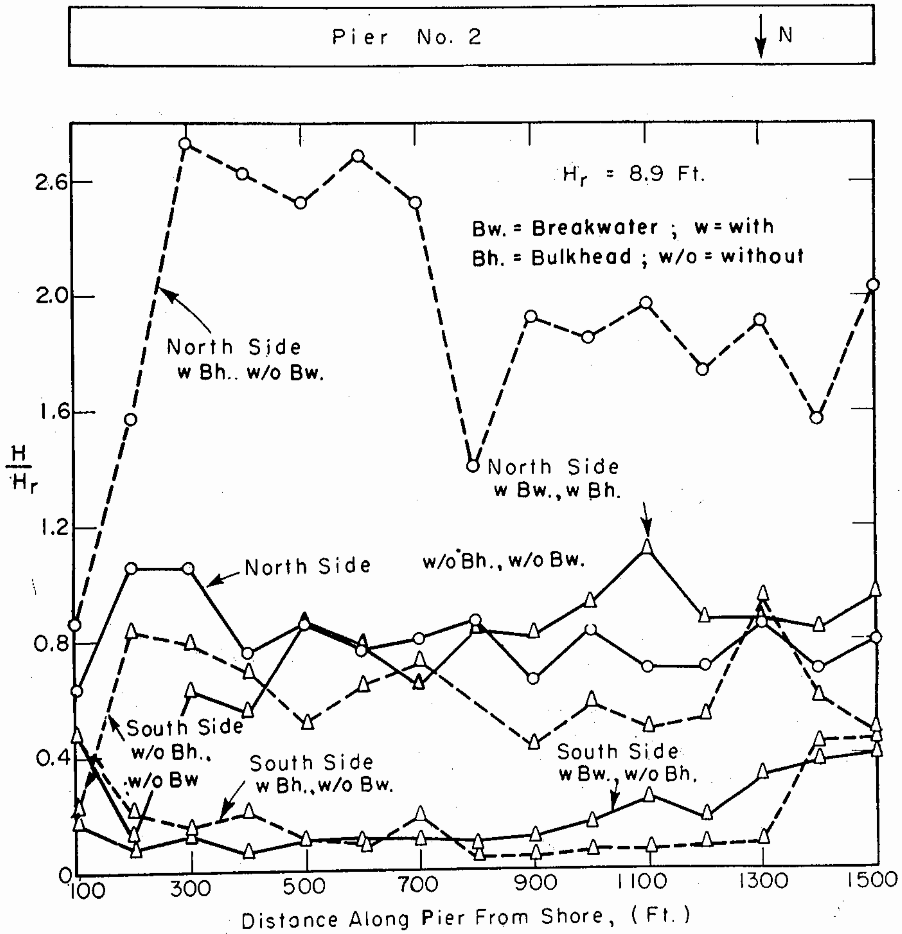

FIG. 12.—SOME EFFECTS OF BREAKWATER EXTENSION ON WAVE HEIGHT IN THE OUTBOARD BERTH OF PIER NO. 1.

Also shown in Fig. 12 is the improvement in wave height along Pier No. 2 due to the presence of the breakwater in Configuration D. Similar results are obtained for the other two piers with the exception that the decrease in protection toward the outboard end of the pier is greater for Pier 1 and less for Pier 3.

In order to decrease wave heights in the outboard berths of Piers 1 and 2 the breakwater was extended into deeper water. Figure 13

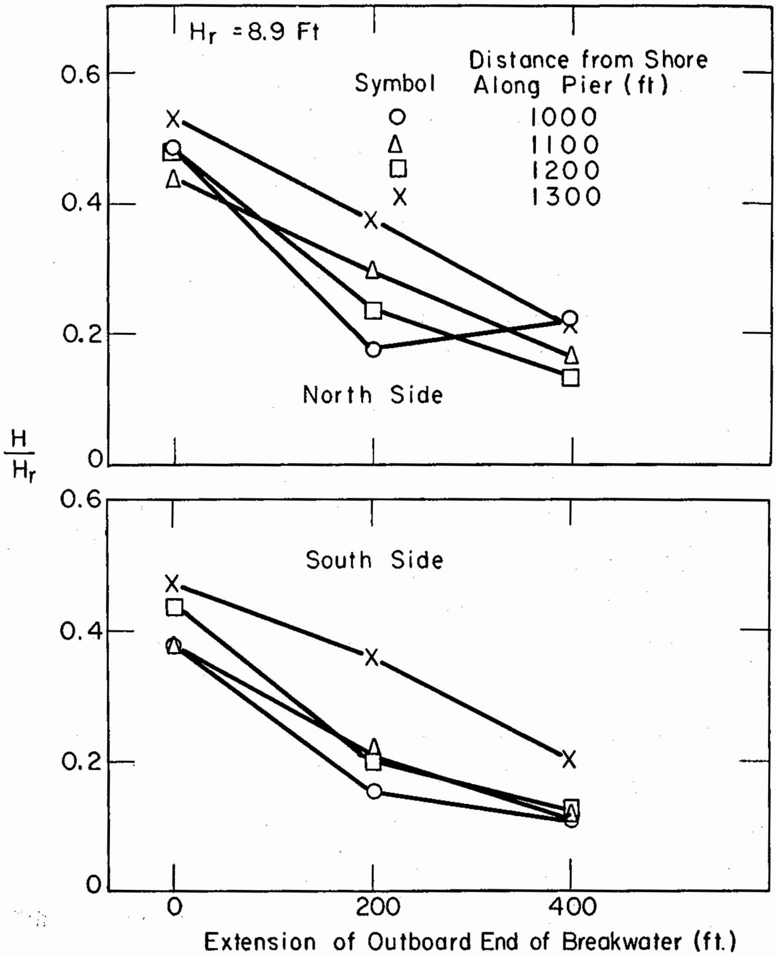

FIG. 13.—EFFECT OF BREAKWATER EXTENSION ON WAVE HEIGHT IN THE OUTBOARD BERTH OF PIER NO. 1.

shows the resulting continual reduction in wave height obtained at various points along Pier 1. Similar results were obtained for Pier 2.

It was determined from personnel of the U. S. Navy that a pierside wave height of 2 to 4 feet was the maximum that could be withstood without evacuation of ships. On the basis of reducing all pierside wave heights to a maximum of 3 feet for a 60 knot wind wave the optimum configuration tested was "K" of Fig. 9 with a 200 foot extension on the outboard end.

A fairly detailed study of the frequency response of the various geometric elements indicated serious resonance problems only in the north and south basins at wave periods less than 10 seconds. In a non fully developed sea the period at which the most energy is concentrated (T_{max}) is essentially the maximum period in the spectrum. It is not likely therefore that in this case any significant wind wave energy exists at periods greater than 10 seconds.

Since the mean depth of water along the proposed breakwater centerline is about 40 feet for configuration K the possibility of an economy through lowering the breakwater height was studied.

The only adverse hydraulic feature of overtopping of the breakwater by waves will arise when enough energy passes the breakwater to cause objectionable ship motions at the piers.

For a given incident wave height as the breakwater height is reduced the energy passing over the breakwater will pass through a current producing stage and into a wave producing stage. It is doubtful that the currents so produced will be objectionable with respect to either ship motion or navigation.

Since the worst condition will arise at high tide, these tests were run with a higher still-water level than the previous tests. The U.S.C. and G.S. (12) gives the average spring tide range to be 4.4 feet at Newport.

In addition, however, the wind tide should be included. Assuming the bay (along the northwest fetch from Coddington Cove) to be a bay of reasonably uniform depth and width, the wind tide as given by Sibul (11) is approximately 0.75 feet for the 60 knot wind.

Thus for these tests the still-water level was held at +5.15 ft. and wave heights were measured along the north side of Pier No. 3.

To note the approximate effect of reducing the breakwater height by 4 feet the same test was performed with the still-water level at +9.15 ft.

In Figure 14, some of the results of the overtopping studies are presented. The wave heights along the north side of Pier No. 3 can be seen to increase as the breakwater freeboard is decreased. When interpreting these results it must be remembered that the model breakwater is distorted so that its slopes are three times as steep as the prototype. This will increase the percentage of wave energy reflected but should also decrease the percent dissipated and also increase slightly the percentage transmitted. It is believed that the results are slightly conservative. It can be said with assurance that the breakwater height cannot be reduced to Elev. +6 without making the north side of Pier 3 untenable for 60 knot wind-waves.

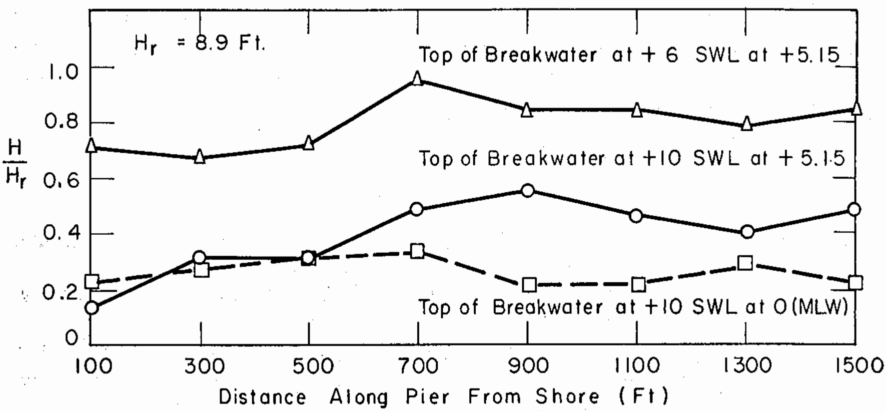

FIG. 14.—EFFECT OF BREAKWATER HEIGHT ON WAVE HEIGHT ALONG PIER NO. 3—CONFIGURATION D.

CONCLUSIONS

A. The optimum breakwater location tested appears to be configuration K (Figure 9) with the addition of a 200 foot extension on the outboard end.

B. Presence of a solid bulkhead on the north side of Pier 2 protects only the inboard two thirds of Pier 1 and the south side of Pier 2.

C. Presence of a solid bulkhead on the north side of Pier 2 is detrimental to use of the north side of Pier 2 in all waves and increases wave heights along Pier 3.

D. The top of the breakwater cannot be lowered to Elev. +6 without making Pier 3 untenable in a 60 knot northwest wind.

E. Presence of the body of an effective breakwater so disrupts tidal surface currents as to present a serious pollution problem requiring independent solution.

F. Littoral sand transport is not significant and need not be a consideration in location of the inboard end of the breakwater.

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References

1. Eagleson, P. S., Marlow, T. A., Luke, C. T. and Turpin, F. J., "Hydraulic Model Study of Protective Works for Fleet Berthing Facilities in Coddington Cove, Narragansett Bay, R. I.," *M.I.T. Hydrodynamics Laboratory, Technical Report No. 25,* June, 1957.
2. Haight, F. J., "Currents in Narragansett Bay, Buzzards Bay, and Nantucket and Vineyard Sounds," *U.S. Dept. of Commerce, Coast and Geodetic Survey, Special Publication No. 208.*
3. Hicks, S. D., Frazier, D. E., and Taylor, Anne P., "Wind Wave Characteristics of Rhode Island Waters," *Hurricane Protection Project, Narragansett Marine Laboratory, Interim Report No. 3, University of Rhode Island,* March, 1956.
4. Sverdrup, H. V., and Munk, W. H., "Wind, Sea, and Swell: Theory of Relations for Forecasting," *U.S. Navy Dept., Hydrographic Office Publication No. 601,* 1947.
5. "Techniques for Forecasting Wind, Waves and Swell," *U.S. Navy Hydrographic Office Publication No. 604,* 1951.
6. Bretschneider, C. L., "Revised Wave Forecasting Relationships," *Proc. Second Conference on Coastal Engineering,* Houston, Texas, 1951.
7. Pierson, W. J., Jr., Neumann, G. and James, R. W., "Practical Methods for Observing and Forecasting Ocean Waves by Means of Wave Spectra and Statistics," *U.S. Navy Hydrographic Office Publication No. 603,* 1955.
8. Bretschneider, C. L., "Generation of Wind Waves over a Shallow Bottom," *Beach Erosion Board Technical Memorandum No. 51,* October, 1954.

9. Sibul, O., "Laboratory Study of the Generation of Wind Waves in Shallow Water," *Beach Erosion Board Technical Memorandum No. 72,* March, 1955.

10. Fay, Spofford and Thorndike, Inc., "Advance Planning FY 1958, Breakwater for Berthing Facilities, IND-N-770," *Fay, Spofford and Thorndike, Inc.,* Boston, Mass., August, 1956.

11. Sibul, O., "Laboratory Study of Wind Tides in Shallow Water," *Beach Erosion Board Technical Memorandum No. 61,* August, 1955.

12. "Tide Tables, East Coast, North and South American," *U.S. Department of Commerce, Coast and Geodetic Survey.*