CORPS OF ENGINEERS, U. S. ARMY

STABILITY OF RUBBLE-MOUND BREAKWATERS

HYDRAULIC MODEL INVESTIGATION

TECHNICAL MEMORANDUM NO. 2-365

CONDUCTED FOR

BUREAU OF YARDS AND DOCKS
DEPARTMENT OF THE NAVY

BY

WATERWAYS EXPERIMENT STATION

VICKSBURG, MISSISSIPPI

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PREFACE

The Bureau of Yards and Docks, Department of the Navy, requested the Waterways Experiment Station to conduct a hydraulic model study of the stability of rubble-mound breakwaters, in a letter dated 21 August 1942. Authority to perform the model study was granted by the Chief of Engineers, Department of the Army, on 25 August 1942. The investigation was conducted intermittently during the period October 1942 to December 1950. At various times during the course of the investigation the testing program was revised and extended.

Liaison was maintained during the course of the investigation by means of progress reports, interim reports and conferences. Representatives of the Bureau of Yards and Docks, the late Mr. Harris Epstein, Messrs. R. C. Stokes and N. M. Brown, visited the Waterways Experiment Station at various times in connection with the study. Lt. J. H. Vaughan, CEC, USN, Bureau of Yards and Docks liaison officer, was stationed at the Waterways Experiment Station while preparation of this report was in progress. Mr. J. R. Ayers, Head, Waterfront Structures, Engineering Consultants Branch, Bureau of Yards and Docks, also visited the Experiment Station during this time. Both Lt. Vaughan and Mr. Ayers rendered valuable assistance in preparation of the report. Their comments and suggestions on review of the report are gratefully acknowledged.

Waterways Experiment Station engineers actively connected with the model study were Messrs. J. B. Tiffany, E. P. Fortson, Jr., G. B. Fenwick, F. R. Brown, R. Y. Hudson, and R. A. Jackson. Messrs. Hudson and Jackson were in actual charge of the tests and preparation of this report.

CONTENTS

CONTENTS (Continued)

LIST OF SYMBOLS AND DEFINITIONS

A = surface area of rock over which wave force (F_{dy}) acts in equations (3), (6), and (8) of appendix A and in equation (1) of appendix B.

A_i = amplitude of incident wave in equations (7) and (12-15).

A_r = amplitude of reflected waves in equations (8) and (12-15).

A_r = area scale of model, L_r^2.

C = wave celerity, rate of travel of wave crest ($C^2 = gd$).

d = depth of water measured from swl to bottom in equation (6), and in equation (13) of appendix A.

E_r = energy scale of model.

e = height of individual cap rock measured perpendicular to breakwater side slope.

F = a force dimension.

F_{dy} = dynamic force of waves on cap rock in equations (3) and (4) and fig. A1 of appendix A, and in equations (1) and (4) of appendix B.

F_r = resistive force of cap rock in equations (2, 3, 4) of appendix B.

F_r = force scale of model.

f = function of.

g = gravitational acceleration (32.2 ft per sec^2).

H = wave height, vertical distance from crest to trough.

H_f, H_w = wave height in liquid heavier or lighter than pure water, and wave height in pure water, respectively.

H_i, H_r = height of incident and reflected waves, respectively, in equation (16).

H_1, H_2 = maximum and minimum wave heights, respectively, in equation (16).

h = height of breakwater crown above swl in equation (13) of appendix A.

K = coefficient in equation (1), and in equations (1) and (2) of appendix A.

K' = dimensionless coefficient in equations (2) and (5), and in equations (11-13) of appendix A.

$k = \frac{2\pi}{L}$ in equation (10), and a coefficient in equations (7), (8), (12), and (13).

k = a drag coefficient in equations (3), (6), and (9-11) of appendix A, and in equations (1) and (4) of appendix B.

k = ratio of effective to total wave energy in equation (6).

k_1 = dimensionless shape coefficients in equation (7) and in equations (9-11) of appendix A, and in equations (3) and (4) of appendix B.

k_2 = dimensionless shape coefficients in equation (8) and in equations (9-11) of appendix A, and in equations (1) and (4) of appendix B.

L = a length dimension.

L = wave length, horizontal distance between consecutive wave crests in equations (6) and (10), and in equation (13) of appendix A.

L_r = linear scale of model.

$N_x = (\frac{x}{e})$ = ratio of length of individual cap rock measured parallel to the length of breakwater to the height of individual cap rock measured perpendicular to slope of breakwater, equation (6).

$N_z = (\frac{z}{e})$ = ratio of width of individual cap rock measured parallel to slope of breakwater to height of individual cap rock measured perpendicular to slope of breakwater, equation (6).

P = downslope component of submerged rock weight (W') in equation (5) and fig. A1 of appendix A.

P_r = pressure scale of model.

R = coefficient in equation (3).

R = a friction force in equation (4) and fig. A1 of appendix A.

R' = dimensionless coefficient in equations (4-6).

S_f = specific gravity of liquid heavier or lighter than pure water in equations (6) and (7) of appendix B.

S_r = specific weight of cap rock in metric tons per cubic meter in equation (1), and in equation (1) of appendix A.

S_r = specific gravity of cap rock in equation (3).

S_w = specific gravity of pure water in equation (6) of appendix B.

$S_{r(1)}$, $S_{r(2)}$ = specific gravity of individual cap rock, simulated and used, respectively, in equations (6) and (7) of appendix B.

T = a time dimension and wave period, time required for a wave crest to travel one wave length ($T = \frac{L}{C}$) in equation (9).

T_r = time scale of model.

t = a specific time in equations (7), (8), (12) and (13).

ton = 2000 lb.

V = velocity of jet formed by breaking wave.

$\overline{V}_r$ = volume scale of model.

$\overline{V}_r$ = total volume of individual cap rock in equation (2) of appendix B.

V_r = velocity scale of model.

V_s = size of voids between rock in equation (13) of appendix A.

W = weight of individual cap rock in equations (1-4), and in equations (1), (2), (12) and (13) of appendix A.

W' = weight of submerged rock in equations (4-7) and fig. A1 of appendix A.

W_r = weight scale of model.

w = width of breakwater near swl in equation (13) of appendix A.

x = rectangular coordinate in equations (7), (8), (12), and (13), length of individual cap rock measured parallel to the length of the breakwater and a pure number indicating the proportion of total volume ($\overline{V}_r$) submerged in the liquid in equations (2-6) of appendix B.

y = characteristic linear dimension of individual cap rock in equations (7-10) of appendix A, and in equations (1), (3), and (4) of appendix B.

z = position of the cap rock with respect to still-water level in equation (13) of appendix A.

z = width of individual cap rock measured parallel to slope of breakwater.

α = angle of seaside slope measured from the horizontal in equations (1-6), in equations (1), (2), (4), (5), (6), (9), (10), (12), and (13) and fig. A1 of appendix A.

γ_f = specific weight of liquid heavier or lighter than pure water in equations (2) and (4), in equations (3), (6), (7), (9), (10), (12), and (13) of appendix A, and in equations (1-5) of appendix B.

γ_r = specific-weight scale of model.

γ_r = specific weight of individual cap rock in equations (2) and (4), in equations (2), (7), (9), (10), (12), and (13) of appendix A, and in equations (2-4) of appendix B.

$\gamma_{r(1)}$, $\gamma_{r(2)}$ = specific weight of individual cap rock, simulated and used, respectively, in equation (5) of appendix B.

γ_w = specific weight of pure water in equation (2) of appendix A, and equation (5) of appendix B.

$\triangle_s$ = shape factor of individual cap rock in equation (13) of appendix A.

η = resultant surface elevation due to components of surface elevation of incident and reflected waves in equations (11-13).

η_i = component of surface elevation of incident waves in equations (7) and (11).

η_r = component of surface elevation of reflected waves in equations (8) and (11).

η_1 = maximum component of surface elevation due to incident and reflected waves in equation (14).

η_2 = minimum component of surface elevation due to incident and reflected waves in equation (15).

μ = effective coefficient of friction of rock on rock in equations (2-6), and in equations (4), (6), (9), (10), (12), and (13) of appendix A.

ρ = mass density $(\frac{\gamma}{g})$.

$\sigma = \frac{2\pi}{T}$ (equation (9)), and a coefficient in equations (7), (8), (12), and (13).

ϕ = angle of incidence of wave attack in equation (13) of appendix A.

coth = hyperbolic cotangent.

M-K-S = meter-kilogram-second system of units.

SUMMARY

The investigation reported herein was undertaken originally to determine whether the breakwater proposed for construction at Roosevelt Roads, Puerto Rico, would be adequate to withstand the attack of the largest waves occurring at the breakwater site. Shortly after the model investigation was undertaken the Roosevelt Roads breakwater project became of less importance from a military standpoint, and it was decided to discontinue further tests on the original problem. Because of the lack of knowledge concerning the phenomena of waves attacking rubble mounds, the Bureau of Yards and Docks decided to broaden the scope of the investigation to include a study of problems of a general nature. The investigation was conducted in a 5-ft-by-18-ft-by-119-ft concrete flume. Models with linear scales of 1:30, 1:45 and 1:60 were used.

First, it was determined that model-prototype transference equations based upon the Froudian relationships were applicable to all important motion occurrences affecting the stability of rubble-mound breakwaters. Then data were obtained on: the stability of component breakwater materials during various stages of construction; the accuracy of the Iribarren and Epstein-Tyrrell formulas for design of rubble breakwaters; coefficients of reflection of waves from rubble breakwaters; and the effect of angle of incidence of wave attack on the stability of rubble breakwaters.

The most important findings of the investigation concerned the use of Iribarren's formula for design of rubble breakwaters. The coefficient in Iribarren's formula was found to vary appreciably with slope of the

seaside face. The Iribarren formula is believed sufficiently accurate for design of rubble breakwaters if used in conjunction with coefficients such as those developed during the model tests.

STABILITY OF RUBBLE-MOUND BREAKWATERS

Hydraulic Model Investigation

PART I: INTRODUCTION

1. The investigation reported herein was begun as a study of the stability of rubble-mound breakwaters proposed for construction at Roosevelt Roads, Puerto Rico. The original design for these breakwaters specified large proportions of heavy cap stone. However, a survey of the quarry from which construction materials were to be obtained revealed no stone of the size specified. The hydraulic model investigation was then initiated to determine whether the available rock would be adequate in size for the proposed structures.

2. After the model study had been undertaken, the Roosevelt Roads breakwater project became of less importance from a military standpoint. The testing program, however, was continued to provide information on the stability of rubble breakwaters. As the study progressed, the scope of the investigation was broadened to include a more detailed program of tests seeking to verify and supplement the scant existing knowledge of the factors concerned in the stability of breakwaters constructed of quarried rock.

3. The specific purposes of the general investigation, as finally evolved after the several revisions in the testing program, were to establish the reliability of selected transference equations for breakwater stability models; to determine the most efficient design of a composite rubble-mound breakwater of a type similar to that proposed for

the Roosevelt Roads project, including determination of effects of wave-action forces on a rubble breakwater during different stages of construction; to develop criteria for use in designing rubble-mound breakwaters; and to study certain rational formulas for the design of rubble-mound breakwaters. In addition, a few miscellaneous tests relating to breakwater design and wave action on breakwaters were performed.

PART II: TEST APPARATUS AND MODEL-TO-PROTOTYPE SCALE RATIOS

4. The study was conducted in a 5-ft-by-18-ft-by-119-ft concrete flume equipped with a plunger-type wave machine. Fig. 1 is a general view of the flume with a longitudinal dividing wall installed; fig. 2 (page 4) is a close-up of the wave generator. Wave heights were measured by electrical gages designed and constructed especially for this purpose. The gage data were recorded photographically by an oscillograph. These apparatus are described in Waterways Experiment Station Technical Memorandum No. 2-237, "Model Study of Wave and Surge Action, Naval Operating Base, Terminal Island, San Pedro, California," September 1947.

Fig. 1. General view of wave flume

Fig. 2. Wave generator

5. Linear scales of 1:30, 1:45 and 1:60 were used in the model studies. The stability model was designed and constructed, and the tests were performed and test data analyzed in accordance with Froude's model law.* The resulting model-to-prototype ratios are as follows:

Characteristic	Dimension**	Model-prototype Ratio
Length	L	L_r
Area	L^2	$A_r = L_r^2$
Volume	L^3	$\overline{V}_r = L_r^3$
Time	T	$T_r = L_r^{1/2}$
Velocity	L/T	$V_r = L_r^{1/2}$
Specific weight	F/L^3	γ_r
Unit pressure	F/L^2	$P_r = L_r \gamma_r$
Force	F	$F_r = L_r^3 \gamma_r$

* ASCE Manual of Engineering Practice, No. 25, "Hydraulic Models," pp 9 and 43.

** In terms of force, length and time.

Characteristic	Dimension	Model-prototype Ratio
Weight	F	$W_r = L_r^3 \gamma_r$
Energy	FL	$E_r = L_r^4 \gamma_r$

With a specific-weight ratio (γ_r) of 1:1 the numerical values of the above ratios for the three model scales are as follows:

Characteristic	Numerical Ratios (A)	(B)	(C)
Length	1:30	1:45	1:60
Area	$1:(30)^2$	$1:(45)^2$	$1:(60)^2$
Volume	$1:(30)^3$	$1:(45)^3$	$1:(60)^3$
Time	1:5.48	1:6.71	1:7.75
Velocity	1:5.48	1:6.71	1:7.75
Specific weight	1:1	1:1	1:1
Unit pressure	1:30	1:45	1:60
Force	$1:(30)^3$	$1:(45)^3$	$1:(60)^3$
Weight	$1:(30)^3$	$1:(45)^3$	$1:(60)^3$
Energy	$1:(30)^4$	$1:(45)^4$	$1:(60)^4$

PART III: RELIABILITY OF TRANSFER EQUATIONS -- SCALE EFFECTS

6. As stated in part II, the transference equations selected for this study were derived from Froude's model law. These equations are known to apply to gravity-wave phenomena. However, it was not known whether the model-prototype ratios based on Froude's law were applicable directly, without modification, to the quantity and time rate of movement of breakwater material caused by wave action. Therefore, model tests were planned to establish the validity of the Froudian scales with respect to the movement of material from the design section of rubble breakwaters. The model breakwaters tested for determination of the reliability of transference equations were constructed on a sand base, while certain other tests were performed on breakwaters constructed on the concrete floor of the wave flume (see paragraphs 20, and 26-35). These tests, therefore, served a secondary purpose in affording a comparison of the stability of breakwaters founded upon sand and rock.

Theory of Procedure

7. Comparison of results of wave action on a model breakwater with results of similar wave action on its prototype would provide direct confirmation of the applicability of Froudian scale relationships to motion occurrences involved in the displacement of breakwater materials. Unfortunately, the vagaries of weather deny procurement of accurate prototype data necessary to such direct procedure. A practical alternative is the use of models of several scales, the results of tests thereon being confirmed as substantially as possible with such available

prototype data as may admit of limited comparison.

8. In applying the use of several models in the present study, consideration was first given to simply testing the models to the point of stability and comparing the results. However, the difficulty of ascertaining the precise point of stability, in the range where the movement of breakwater materials is quite gradual, made necessary a variation in procedure wherein the time at which stability obtained was ascertained on one model selected as a pilot, after which the other models were operated for equivalent periods of time.

Description of Tests

9. Identical stability tests were performed on three models of the breakwater section previously proposed for the Roosevelt Roads project (plate 1). This particular breakwater section was reproduced because the sizing of model breakwater material is a tedious, expensive, and time-consuming procedure, and accurately sized material was already available for a 1:30-scale breakwater of this type design. The models were constructed to linear scales of 1:30, 1:45 and 1:60, the largest scale representing the capacity of the wave-generating apparatus, and the smallest the limiting accuracy of available wave-measuring equipment and the limiting feasibility of sizing of model rubble. The intermediate scale was a median condition selected to give further substance to the experimental results.

Details of test setup

10. Table 1 shows the graduation of materials contained in the test breakwater, and plate 1 shows the arrangement of these materials in

the complete breakwater section. The class C material used for the 1:30-scale model consisted of a mixture of crushed limestone and sand; the class C material used for the 1:45- and 1:60-scale models consisted of sand with a specific gravity of 2.65. The class A and B materials used for all three models consisted of crushed limestone with specific gravity of 2.70.

11. Tests using each of the three scales selected were conducted upon three breakwater sections:

- a. A partial section having a core of class C material.
- b. A partial section having a core of class C material, overlaid with the protective enrockment of class B material.
- c. The complete section of three materials arranged as shown on plate 1.

Figs. 3-5 show the 1:45-scale test sections as constructed in the wave flume before wave attack.

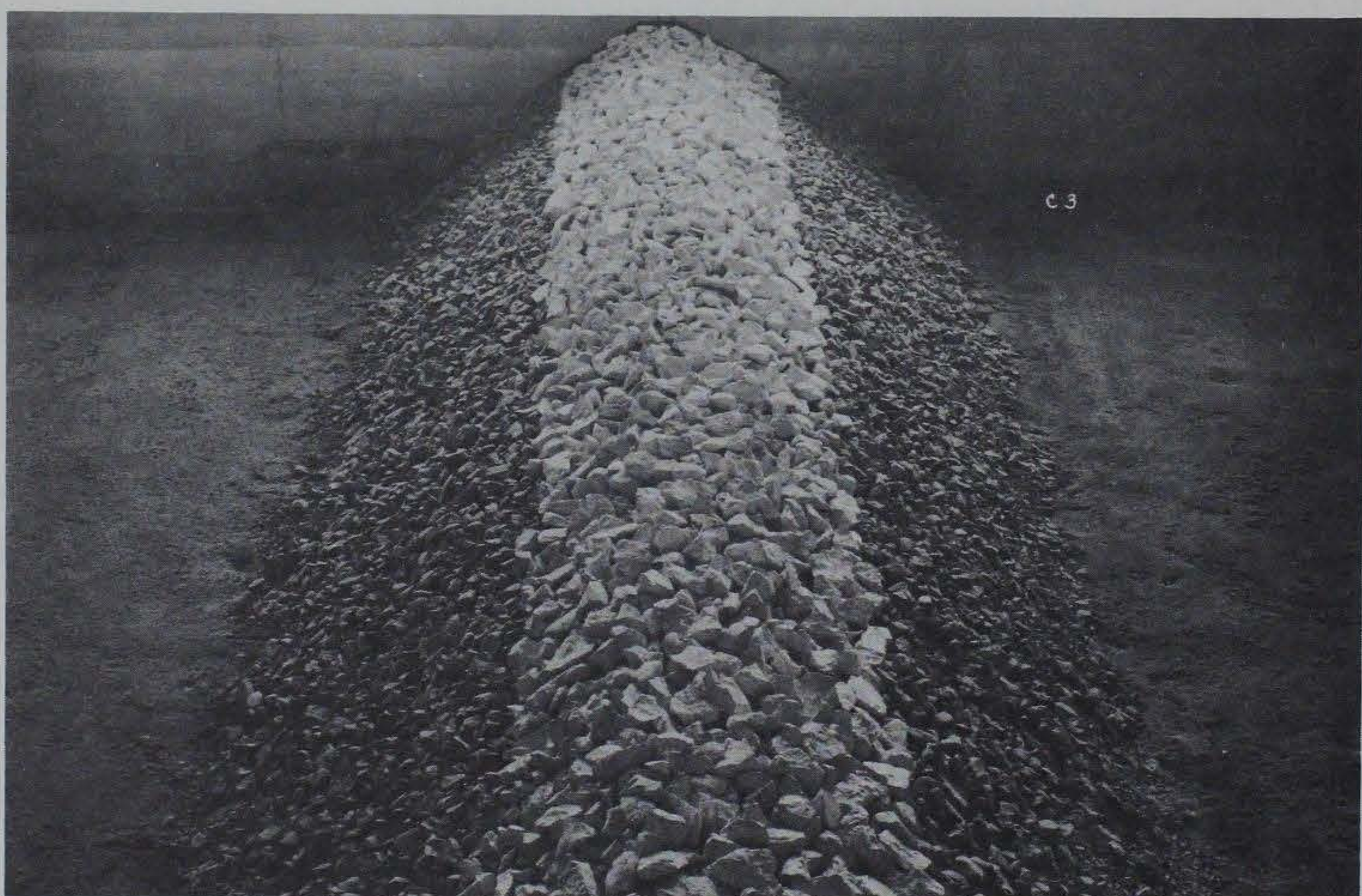

Fig. 3. Complete breakwater section before wave attack, 1:45-scale model

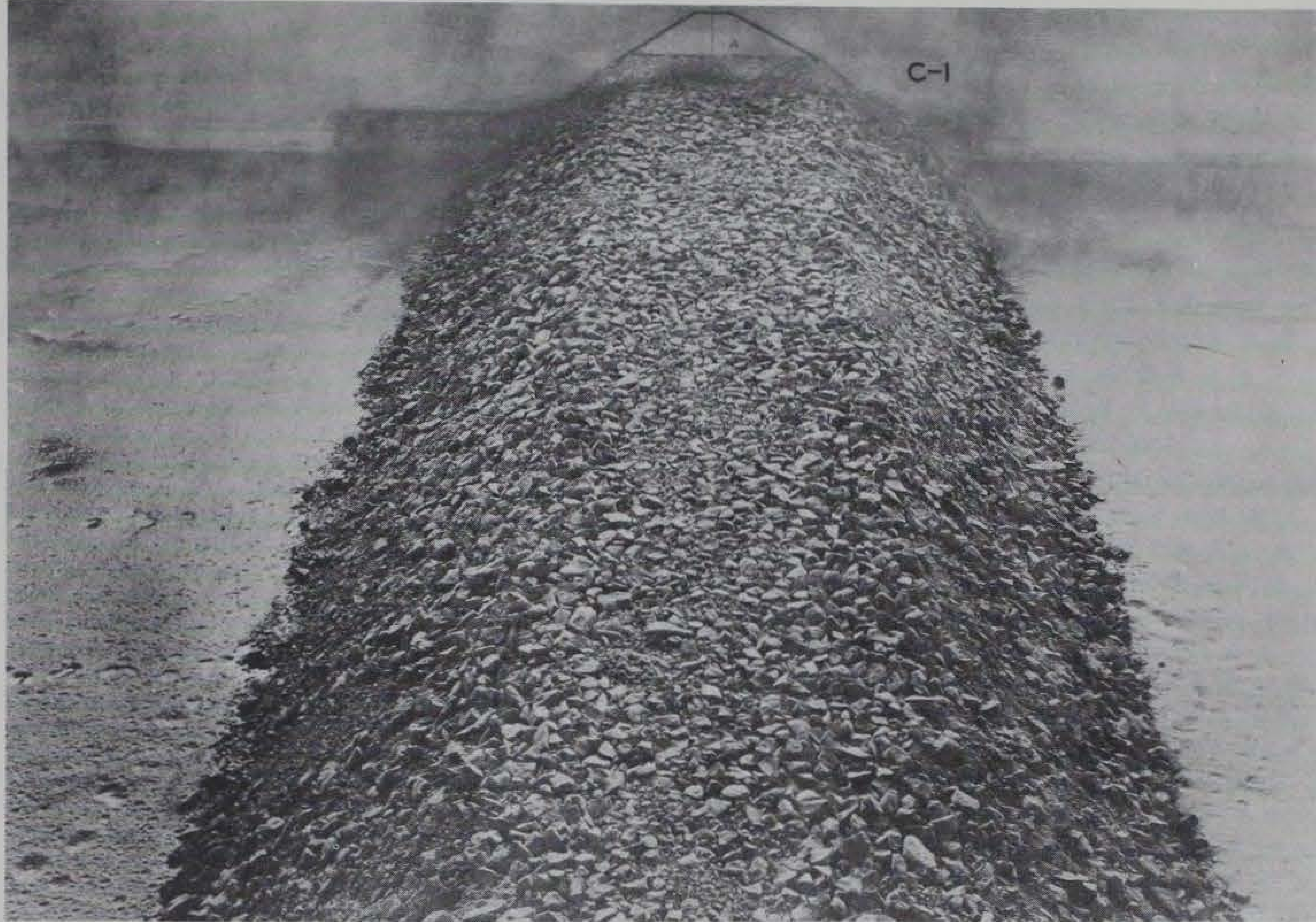

Fig. 4. Class B section before wave attack, 1:45-scale model

Fig. 5. Class C section before wave attack, 1:45-scale model

12. The model breakwater sections reproduced prototype structures situated in water 58 ft deep (prototype). They were placed across the 18-ft-wide flume approximately 75 ft from the wave-generating device, and tested with model waves representing, to the applicable scale, prototype waves 15 ft high by 270 ft long and 21 ft high by 300 ft long.

Testing procedure

13. The 1:30-scale (largest) model was selected as the pilot and tests were conducted as outlined in paragraph 8. Stability tests were performed on this model for each of the breakwater sections described in paragraph 11 using both 15- and 21-ft waves. The breakwater sections were considered stabilized by the forces of wave action when displacement of material ceased and an additional period of wave attack caused no further displacement. Progressive displacement of material was followed throughout the tests by cross-sectioning the model breakwaters at frequent intervals. Cross sections were measured at 0.5-ft (model) stations on the 10-ft portion of the model in the middle of the flume. The extremities of the model adjacent to the walls were excluded because of the unnatural effects of the flume walls on these portions of the breakwater.

14. The time required to stabilize the 1:30-scale breakwater sections was converted by the Froudian time relationship to scales of 1:45 and 1:60, and models of the breakwater sections to these scales were subjected to the attack of equivalent waves for the respective equivalent intervals of time. This procedure furnished comparative stability data on models for three linear scales with the same period of time of prototype wave attack. Comparison of the results obtained in this manner demonstrated the degree of reliability of the transference equations.

The time of model wave attack on the 1:45- and 1:60-scale models was continued beyond the calculated stability time in sufficient instances to verify the actual attainment of a stabilized condition.

Test Results

Time required to stabilize model sections

15. The time intervals required for wave attack to stabilize the model breakwater sections, as experimentally determined for the 1:30-scale model, and as computed for the 1:45-scale and 1:60-scale models, were as follows:

		Stabilization Time in Hours			
		Experimentally Obtained	Computed from 1:30-scale Model by Froudian Relationship		
Section	Wave Ht, Ft	1:30-scale Model	Prototype	1:45-scale Model	1:60-scale Model
Class C	21	7	38-1/4	5-3/4	5
Class C	15	4-3/4	26	3-3/4	3-1/4
Class B	21	4-1/4	23-1/4	3-1/2	3
Class B	15	3	16-1/2	2-1/2	2-1/4
Class A	21	4-3/4	26	4	3-1/4
Class A	15	4	22	3-1/4	2-3/4

Comparison of test results, 1:30-, 1:45-, and 1:60-scale models

16. Cross sections showing the conformation of the test sections of the three different scale models before, at and after stabilization are presented on plates 2-4. These data are considered excellent experimental results, such dissimilarity as exists being ascribed to the human error inherent in repeating an operation involving the placing -- and especially the keying -- of multisized material in a model breakwater.

Had the dissimilarity been due to scale effect, it would have displayed a consistency which is entirely absent.

17. A comparison of corresponding test sections shown on plates 3 and 4 shows that no appreciable erosion of breakwater material in the smaller models occurred after the period of wave attack computed to be required for stabilization. This fact, together with the fact that the three models showed similar results with respect to the amount and disposition of material eroded from the breakwater sections for equivalent prototype time periods of wave attack, proves the applicability of the Froudian relationships to all important motion occurrences within the range of model scales employed in these tests.

Comparison of model results with known breakwater damage

18. An opportunity for prototype confirmation was afforded by data that became available on storm-wave damage to the detached breakwater in San Pedro Bay, California. These prototype data were selected for comparison with model results because of the close similarity in design of the San Pedro breakwater and the typical breakwater used for the model tests. Also, the storm waves that caused the damage to the San Pedro breakwater were near the height of the model test waves, though of greater length.

19. Results of this comparison are shown on plate 5, together with the material specifications for both breakwaters. The disposition of material and final shape of the damaged San Pedro breakwater were very similar to that occurring in the model, although the amount of model breakwater material displaced was greater than that displaced in the prototype occurrence. The model breakwater was subjected to wave attack

until all movement of material ceased, whereas the storm waves that attacked the San Pedro Bay breakwater occurred intermittently and with varying dimensions for a period of about 30 hours. The largest waves occurring during this period were estimated to be about 20 ft by 600 ft, and it is probable that the larger storm waves did not prevail for the length of time required to stabilize the prototype breakwater. The storm waves approached the San Pedro Bay breakwater at an angle of incidence of about 45 degrees. In the model tests the angle of wave approach was normal to the breakwater. However, according to the results of model tests performed as another phase of the rubble-mound stability study (see paragraphs 68-70), wave damage to breakwaters of this type is not a function of angle of incidence. It is believed that the similarity of breakwater damage (prototype and model) as shown on plate 5 can be interpreted as further evidence that test results obtained from correctly designed and constructed rubble-mound breakwater models are reliable.

Effect of sand base under test section

20. The effect of a sand foundation versus a rock foundation on the amount and disposition of breakwater material eroded by wave action is presented on plate 6. These data show that the class C and B materials were spread farther from the toe of the original design section when a concrete base was used. This condition is attributed to the lesser frictional resistance between the breakwater material and the concrete foundation. The type of foundation material used did not affect the results of tests for the complete breakwater section.

PART IV: TESTS OF COMPOSITE BREAKWATER SECTIONS

21. This phase of the breakwater stability investigation was concerned with various aspects of the design and construction of composite rubble-mound breakwaters. The objectives were to obtain general information concerning the stability of partial sections of rubble breakwaters during different stages of construction, and to develop plans for efficient utilization of the component materials of composite rubble-mound-type breakwaters. Only one general type of rubble mound was investigated in this part of the stability study. The scope of this phase of the testing program could not be readily enlarged because of the limited quantity of suitable and properly graded, sized, and shaped rock available for use as model breakwater material. It was decided, therefore, to utilize as much as possible of the model breakwater material previously processed. The considerable time required to perform tests of this nature, and the need for conserving available funds, also limited the scope of these tests.

Description of the Model and Prototype Breakwaters

22. The model breakwaters used for these tests reproduced to a linear scale of 1:30 a rubble-mound structure similar in cross section and material to that proposed for the Roosevelt Roads breakwater project (plate 1). Three different classes of material were used for construction of the model breakwater. The model stones were sized so that the weight of the rock particles conformed to the weight of the prototype rock particles in accordance with a weight scale of $1:30^3$. The prototype

material specifications were reproduced by using the following materials:

a. Class A stone. Class A stone, consisting of molded concrete cubes weighted by iron filings to a specific gravity of 2.65-2.70, was used in a few tests. Other tests were conducted using selected pieces of crushed limestone with a specific gravity of 2.70.

b. Class B stone. Crushed limestone with a specific gravity of 2.70 was used for armor rock to protect the core of the structure. This material was used for construction of seaside and harborside footings and lower slopes up to -10 ft swl.

c. Class C stone. Sand and crushed limestone, with a specific gravity of 2.65 to 2.70, were used for construction of the core. The crown elevation of the core material was -24 ft swl.

23. The Bureau of Yards and Docks furnished the following specifications for the Roosevelt Roads (prototype) breakwater:

a. Class A stone shall be selected from the quarry. No piece used shall weigh less than one ton, and at least 75 per cent by weight shall be in pieces weighing 10 tons or more each.

b. Class B stone shall be quarry run, of which not more than 25 per cent by weight may be in pieces of less than 20 lb, and not less than 40 per cent by weight shall be in pieces of one ton or more each. Spalls smaller than one pound and earthy material aggregating in total not more than 5 per cent of any scowload or carload will be accepted, but such material will be considered a part of the 25 per cent by weight of material which will be accepted in pieces weighing less than 20 lb each.

c. Class C material shall be all residuum from quarry operations, similar material obtained in the vicinity of the quarry, or dredged material.

24. The prototype specifications for the class A, class B, and class C stone were made more definite for model use by subdividing the allowable ranges of weights as follows:

Class A Stone

Per Cent of Total	Prototype Weight
75	10 - 12 tons
20	3 - 9 tons
5	1 - 2 tons

Class B Stone

15	2 - 4 tons
30	1 - 2 tons
15	100 - 1000 lb
10	50 - 100 lb
5	20 - 50 lb
5	10 - 20 lb
10	5 - 10 lb
5	1 - 5 lb
5	less than 1 lb

Class C Material

50	0.50 - 1.00 lb
50	0.25 - 0.50 lb

The model materials were carefully prepared to conform with the above specifications.

Test Procedures

25. The results of tests described in part III of this report show that Froude's model law is applicable to stability tests involving the quantity, rate and disposition of material eroded from small-scale rubble breakwaters. Thus, model results can be interpreted quantitatively in accordance with the scale ratios listed in paragraph 5, for a linear scale of 1:30, provided the basic static and dynamic forces are reproduced accurately. These requirements were satisfied by constructing the model breakwaters geometrically similar in cross section to the prototype; using model materials similar in size, shape, surface roughness and

specific weight to the corresponding prototype materials; and by reproducing model waves in accordance with a linear scale of 1:30 and time and velocity scales of 1:5.48 (paragraph 5). All breakwaters and test sections used in this series of tests were subjected to wave attack until stabilized sections were obtained. For test series 1-4 the model sections were constructed on a concrete base. For the series-5 tests the model sections were founded on a sand base.

Tests and Results

Test series 1, class C material only

26. Description. Four partial sections were tested to determine the stability of class C material (plate 7). These sections represented four stages in the construction of the core of a rubble breakwater. Top elevations of the test sections, referred to swl, were -49 ft, -38 ft, -29 ft, and -24 ft. Each section was molded across the wave tank 75 ft from the wave machine. The breakwater material was placed under water. All breakwater sections tested were situated in water corresponding to a prototype depth of 58 ft (the original breakwater section proposed for the Roosevelt Roads breakwater was to have been tested using a water depth of 58 ft). The class C material sections were tested using the following size waves:

Wave Height, Ft	Wave Length, Ft	Wave Period, Sec
7.5	210	6.6
10.5	210	6.6
15.0	270	7.7
21.0	300	8.4

After the breakwater was stabilized by the attack of 7.5-ft waves, the section was examined to determine the extent of erosion. If no appreciable displacement had occurred, the wave machine was adjusted to generate the next larger size wave and operation was resumed. If the section showed appreciable erosion, it was repaired to conform with the line and grade of the original section before testing was resumed using the next larger size wave.

27. Results. The results of this series of tests are shown on plates 8-11. These data show that:

a. For the test section molded of class C material to a top elevation of -49 ft swl (9-ft fill), no appreciable displacement of material resulted from the four wave heights used (plate 8).

b. For the test section molded of class C material to a top elevation of -38 ft swl (20-ft fill), 7.5-ft and 10.5-ft waves caused no displacement, but the 15.0-ft waves caused a slight displacement and the 21-ft waves caused an appreciable displacement. These data are shown on plate 9.

c. For the test section molded to a top elevation of -29 ft swl (29-ft fill), 7.5-ft waves caused no displacement, 10.5-ft waves caused a slight displacement, and the 15.0-ft and 21.0-ft waves caused appreciable displacement of material (plate 10).

d. For the completed class-C-material section molded to a top elevation of -24 ft swl (34-ft fill), 7.5-ft waves caused no erosion, 10.5-ft waves caused moderate erosion, and 15.0-ft and 21.0-ft waves caused considerable displacement. These data are shown on plate 11.

28. The larger waves displaced rapidly the class C material from the top of the test section at the beginning of each test. However, the rate of displacement decreased noticeably as the material eroded to lower elevations. Decrease in the rate of erosion with time can be explained as follows: (a) the resisting forces of the material increase

as the slope of the section becomes flatter; (b) orbital velocities of the waves, and thus dynamic forces, decrease with distance downward from the water surface; and (c) wave action consolidates the material. The breakwater materials were moved toward both the harborside and seaside of the test section, with movement toward the harborside predominating. Testing times for several of the tests were the same (see table 2) because the results of initial tests showed that, after a short period of operation, the rate of material displacement was nearly constant and continuous cross-sectioning was not necessary. For economy, therefore, the sections were cross-sectioned hourly. As a result of this procedure, different model tests often terminated after the elapse of the same number of hours.

29. Data obtained from tests on partial and completed sections of class C material show the extent to which the class C material can be placed without displacement by wave action. The approximate height at which fills of class C material in water depths of about 58 ft, and without class B enrockment, would be stable during wave attack are shown below:

Wave Height, Ft	Wave Length, Ft	Top Elevation of Stable Section, Ft below swl	Height of Fill, Ft
7-8	210	20	38
10-11	210	30	28
15-16	270	40	18
20-21	300	50	8

Using the above table and the height of waves which can be expected at a breakwater site, an estimate can be made of the safe height to which a class-C-material section may be placed. These results are strictly applicable only for material of 2.65-2.70 specific gravity placed in depths of water such that the $\frac{d}{L}$ ratios are the same as those in the tests.

Test series 2, class C material with class B enrockment

30. Description. Three partial breakwater sections were tested to determine the best method of protecting the class C material during initial stages of construction. The top elevation of these sections, referred to swl, were -38 ft, -29 ft, and -24 ft. Tests on the partial section with a top elevation of -49 ft swl were omitted because test series 1 (plate 8) showed that this section without toe protection would be stable when subjected to the attack of waves as high as 21.0 ft. Three variations of each section were tested as follows: (a) class C material with class B enrockment on the harborside; (b) class C material with class B enrockment on the seaside; and (c) class C material with protective class B enrockment on both the seaside and harborside. The class B enrockment was molded on a slope of 1 on 1-1/4, and for each test its top elevation was the same as that of the class C material. Each test section was subjected in turn to the attack of the four test waves listed in paragraph 26, except that tests using waves 7.5 ft and 10.5 ft in height were omitted in those instances where previous tests had shown that these smaller waves would not cause displacement of breakwater material. Molding of the breakwater sections, operation of the wave machine, and the observation of test results were performed as described in paragraph 26.

31. Results. Results of test series 2 are described in the following subparagraphs.

a. Protection on harborside only. The results of tests with class B material on the harborside are shown on plates 12-14. Comparison of these test results with those of test series 1 shows that no advantage would be obtained by protecting only the harborside. If waves 15.0 ft to 21.0 ft in height occurred during construction, the class C material would be removed from the seaside and deposited on the class B enrockment on the harborside. This overlaying of the class B enrockment would result in an unsatisfactory distribution of materials.

b. Protection on seaside only. The results of these tests (plates 15-17) show that for sections with the lower elevations (-38 ft and -29 ft swl) no advantage would result from protecting only the seaside. For the higher elevation (-24 ft swl) the class B enrockment would provide fair protection to the class C material (plate 17). This method of construction would be practical during relatively calm weather, but if a severe storm occurred, considerable class C material would be displaced landward. This displacement would be undesirable because of the time and expense involved in removal of core material before placement of class B material on the harborside could be continued.

c. Protection on both harborside and seaside slopes. Plates 18-20 show the results of these tests. For the sections with the lower elevations large quantities of class C material were eroded from the unprotected top of the class C section by the larger test waves. The shape of the scoured area was concave and the displaced class C material was deposited on the harborside. As the top elevation of the sections was raised, the width of exposed class C material was decreased and the amount of material displaced was progressively less. As a result of this condition practically no class C material was displaced from the section with a top elevation of -24 ft swl, even by waves 21.0 ft in height. These test results show conclusively that placing the class B enrockment on both harborside and seaside slopes simultaneously with the construction of the class C core is the best construction procedure. During initial construction (plate 18), a severe storm would cause class C material to be displaced beyond the design limits of the class B material. However, this displaced class C material would not interfere with the placement of the class B material -- another advantage of this method of construction. To take full advantage of this method, the rate of placement of class C core material should not greatly exceed that of the class B enrockment.

Test series 3, class B stone

32. Description. Tests similar to those described above were performed to determine the stability of the class B stone in place on top of the class C material section without the class A cap rock. These tests involved only the partial breakwater section shown in fig. 6; test waves 7.5 ft, 10.5 ft, 15.0 ft and 21.0 ft in height were used.

Fig. 6. Test series 3, class B section before wave attack

33. Results. The results of tests on the complete class B section (plate 21) show that 7.5-ft and 10.5-ft waves caused very little displacement of material. These data also show that 15.0-ft and 21.0-ft waves caused considerable displacement of material, with the greatest deposition of material occurring on the harborside. Comparison of figs. 6 and 7 shows the deposition of material caused by 21.0-ft waves. Although

damage from the larger waves was considerable, and repairs would be required before commencing the placing of class A stone, the deposition of the displaced class B stone was such that the slopes were flattened and a more stable breakwater obtained.

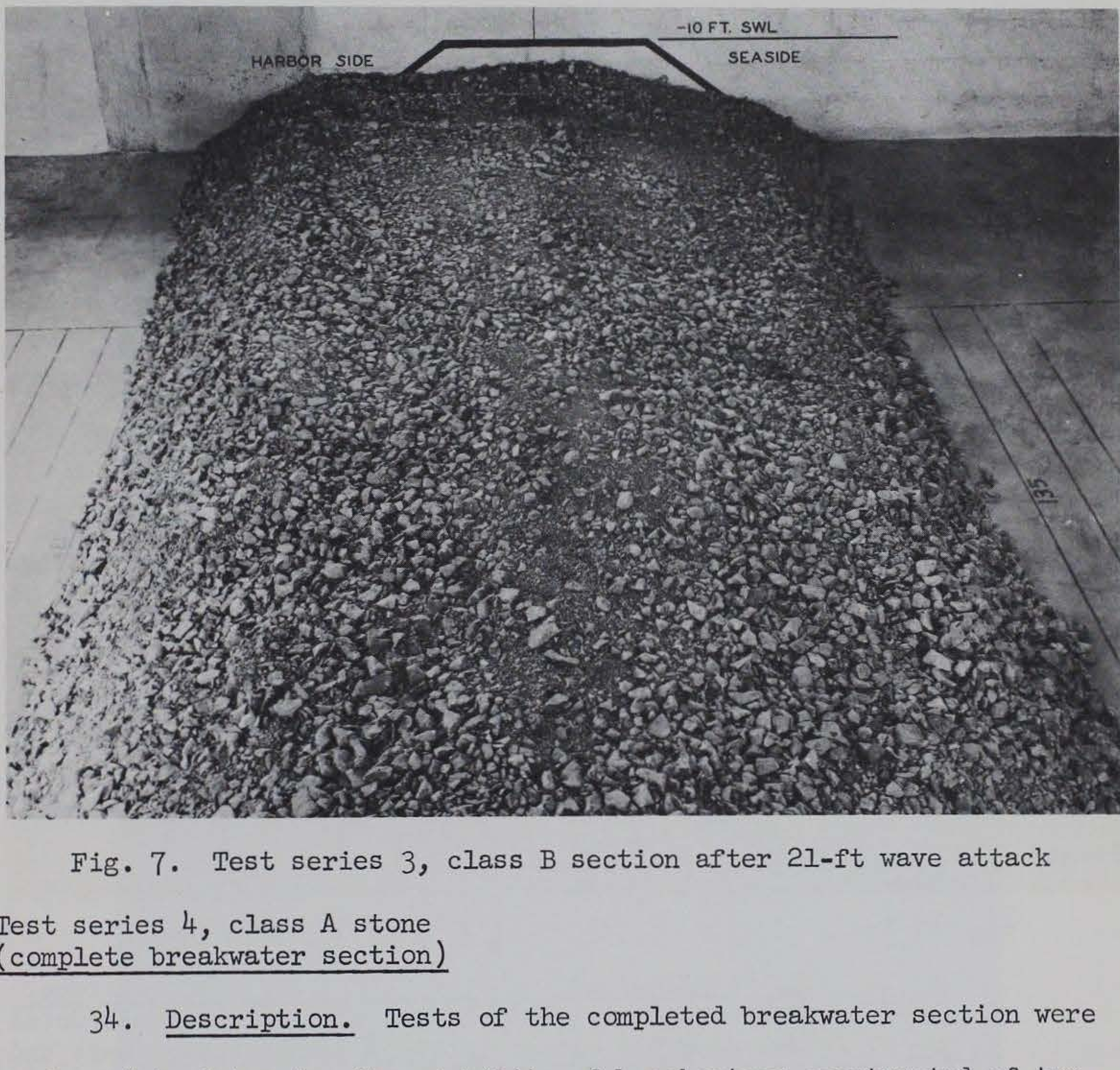

Fig. 7. Test series 3, class B section after 21-ft wave attack

Test series 4, class A stone
(complete breakwater section)

34. Description. Tests of the completed breakwater section were performed to determine the stability of breakwaters constructed of two kinds of cap rock (class A stone): (1) random-shaped rubble, and (2) molded concrete cubes. The model breakwaters tested in this series were constructed in the following manner: (a) the class C and class B materials were placed in the manner previously described to conform with the

shape of the selected prototype breakwater, and class A cap stones, which were small concrete blocks weighted with iron filings, were placed by hand (fig. 8); and (b) the class A and class B materials were molded to the design sections and class A stones, which consisted of graded and

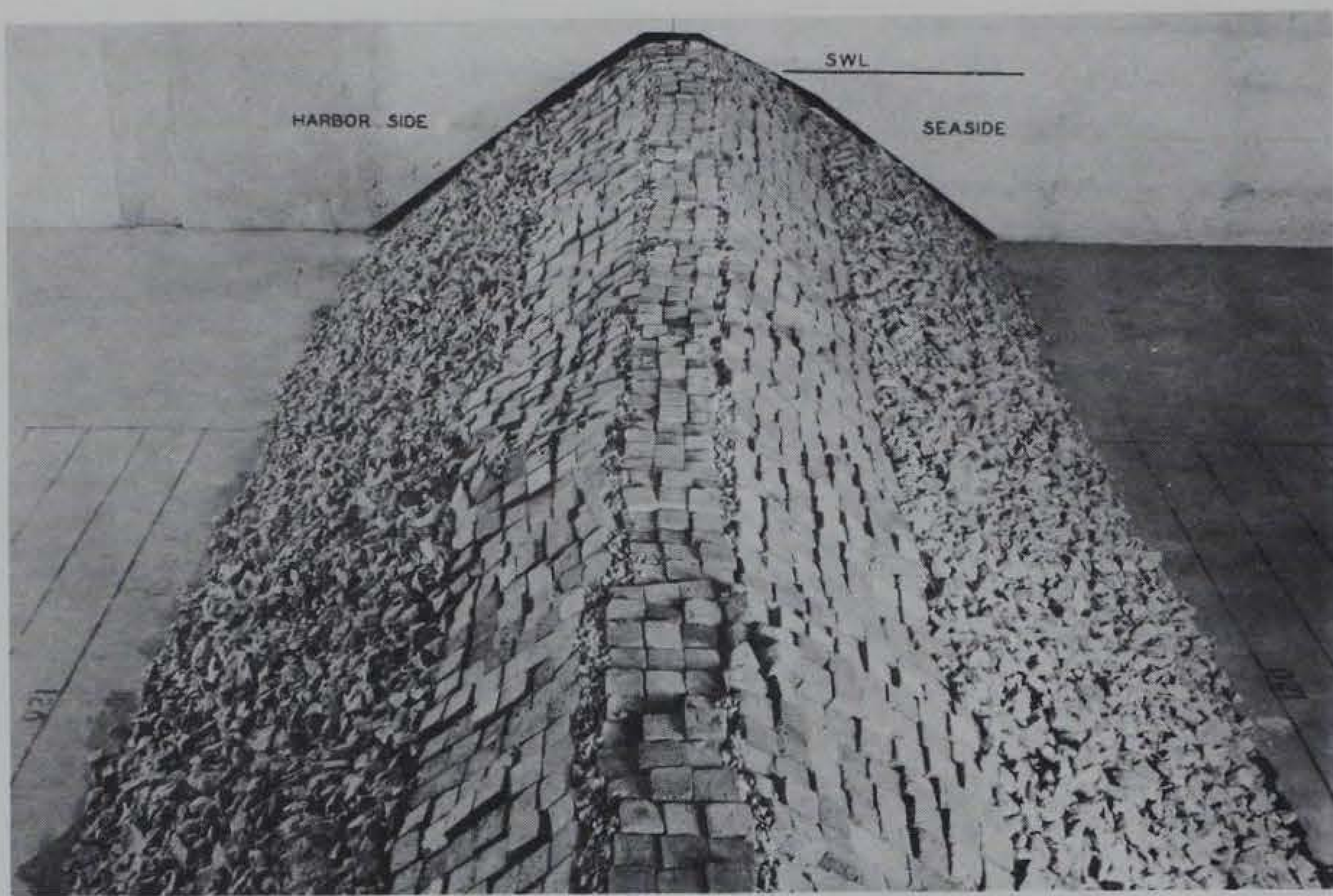

Fig. 8. Test series 4, complete breakwater section before wave attack, class A stone of molded concrete blocks

sized limestone, were hand-placed as before (fig. 11, page 26). The two types of class A sections were tested using waves 7.5 ft, 10.5 ft, 15.0 ft and 21.0 ft in height.

35. Results. Results of test series 4 are as follows:

a. Sections using molded concrete blocks for class A stone. The results of tests conducted on breakwaters constructed of molded concrete blocks (plate 22) show that 7.5-ft and 10.5-ft waves caused no displacement of material from the breakwater crown. Some slight movement of the class A cap stones from the seaside slope was observed during tests using 7.5-ft and 10.5-ft waves. This movement consisted of the shifting and nesting of the blocks according to the wave forces acting upon them. This action of the blocks occurred during a very short period of model operation, after which the breakwater reached a state of stability. The results of tests with 15.0-ft and 21.0-ft waves showed considerable displacement of class A stone, together with some slight displacement of class B enrockment. Figs. 9 and 10 illustrate the displacement caused by these waves. Observations of wave action on the test sections revealed that the greatest damage to the breakwater occurred during the first few minutes of wave

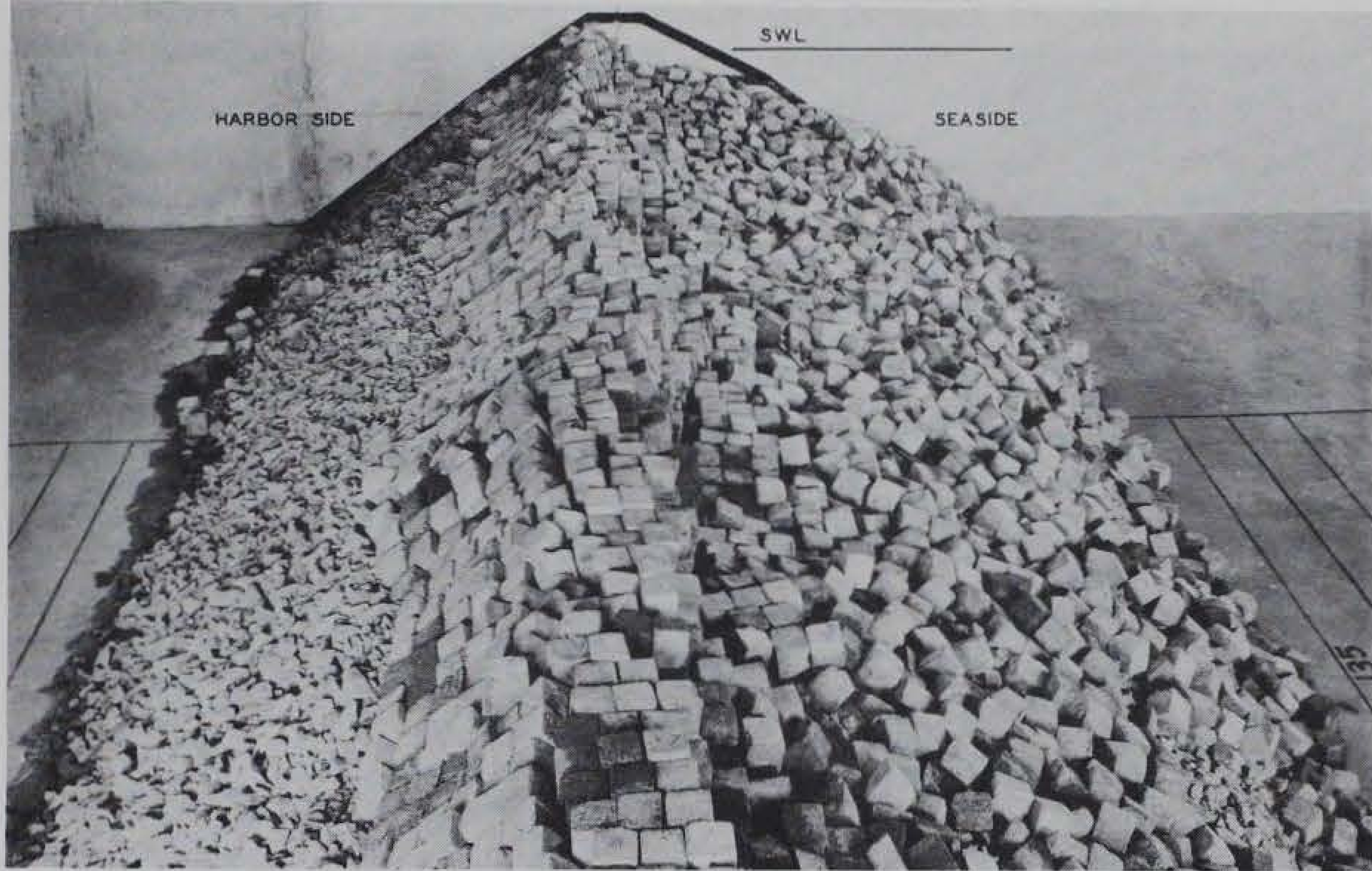

Fig. 9. Test series 4, complete breakwater section after 15-ft wave attack, class A stone of molded concrete blocks

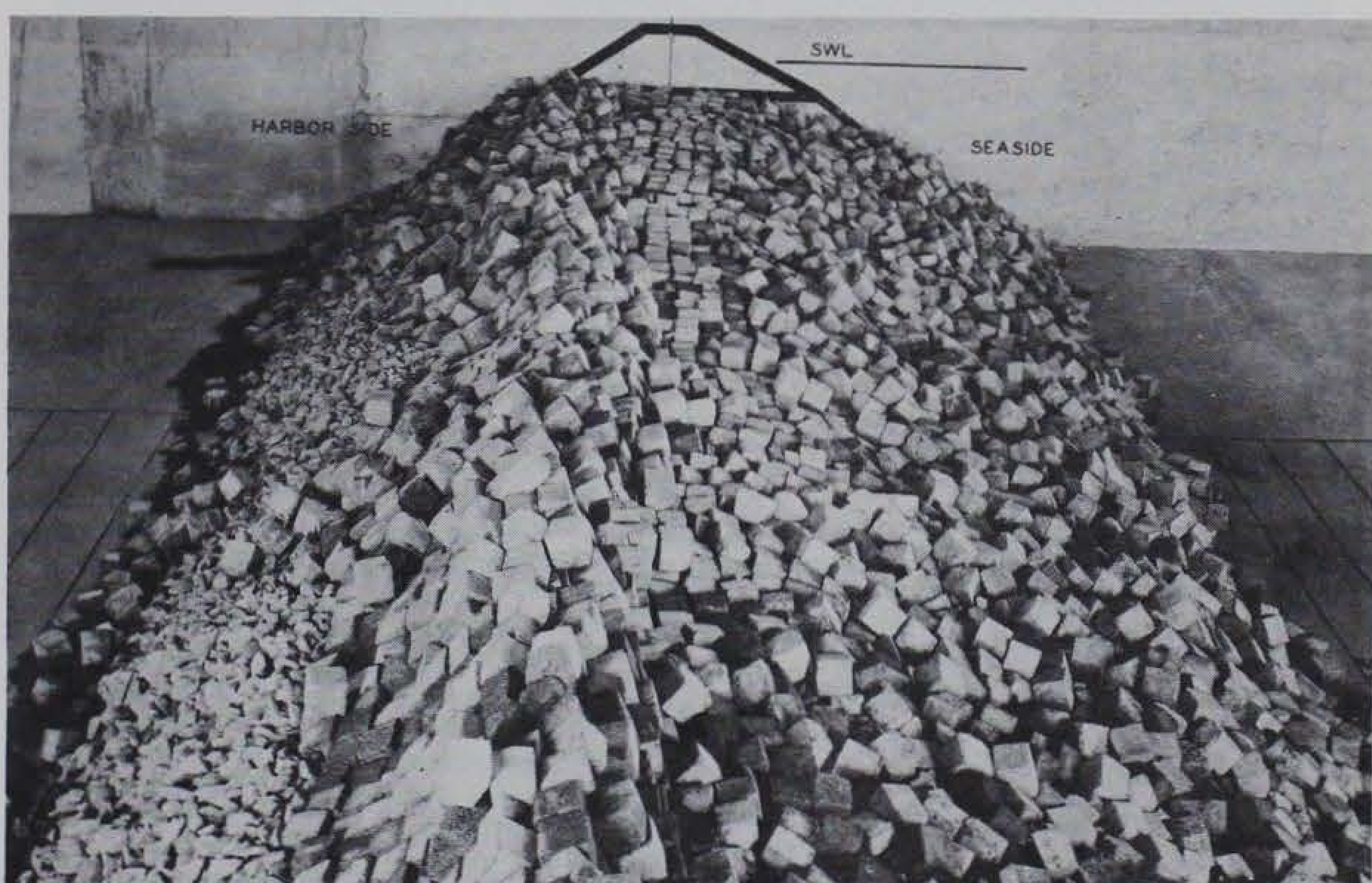

Fig. 10. Test series 4, complete breakwater section after 21-ft wave attack, class A stone of molded concrete blocks

attack. After the test sections were stabilized, the slope on the seaside was about 1 on 2-1/2. The slope on the harborside was not changed. The results of these tests indicate that, for locations that are subject to severe storms, the seaside slope of the breakwater should be somewhat flatter than that tested in the model, the class A stone should be extended to a lower elevation, and a better method of placing the cap rock should be devised to insure that the bottom or key stones would be more secure. Also, raising the crown of the breakwater to prevent excessive overtopping should be given consideration.

Fig. 11. Test series 4, complete breakwater section before wave attack, class A stone of crushed limestone

b. Sections using crushed limestone for class A stone. Tests using 7.5-ft waves were omitted because previous tests indicated that waves of this height would not damage the section. The data on plate 23 show that waves 10.5 ft high caused very little displacement of cap rock. Waves 15.0 ft and 21.0 ft high caused considerable damage (see figs. 12 and 13). In general, test data show little difference in the final stability of breakwaters constructed of crushed limestone and molded concrete blocks (compare figs. 9 and 10 with figs. 12 and 13). The principal difference between the results of these tests was the time rate of displacement. For tests on concrete blocks most of the displacement was caused by the first two or three waves. For crushed limestone cap rock the time rate of displacement was less than that observed for concrete blocks. However, after stability was obtained, results for both types of cap stone were about the same.

For the conditions tested, the difference between the interlocking characteristics of the two types of materials does not appear to affect the breakwater slope at the time of stability. The test data indicate that the breakwater originally proposed for Roosevelt Roads would not have been stable under attack of the larger (20-25 ft) waves which may occur at the proposed breakwater site.

Fig. 12. Test series 4, complete breakwater section after 15-ft wave attack, class A stone of crushed limestone

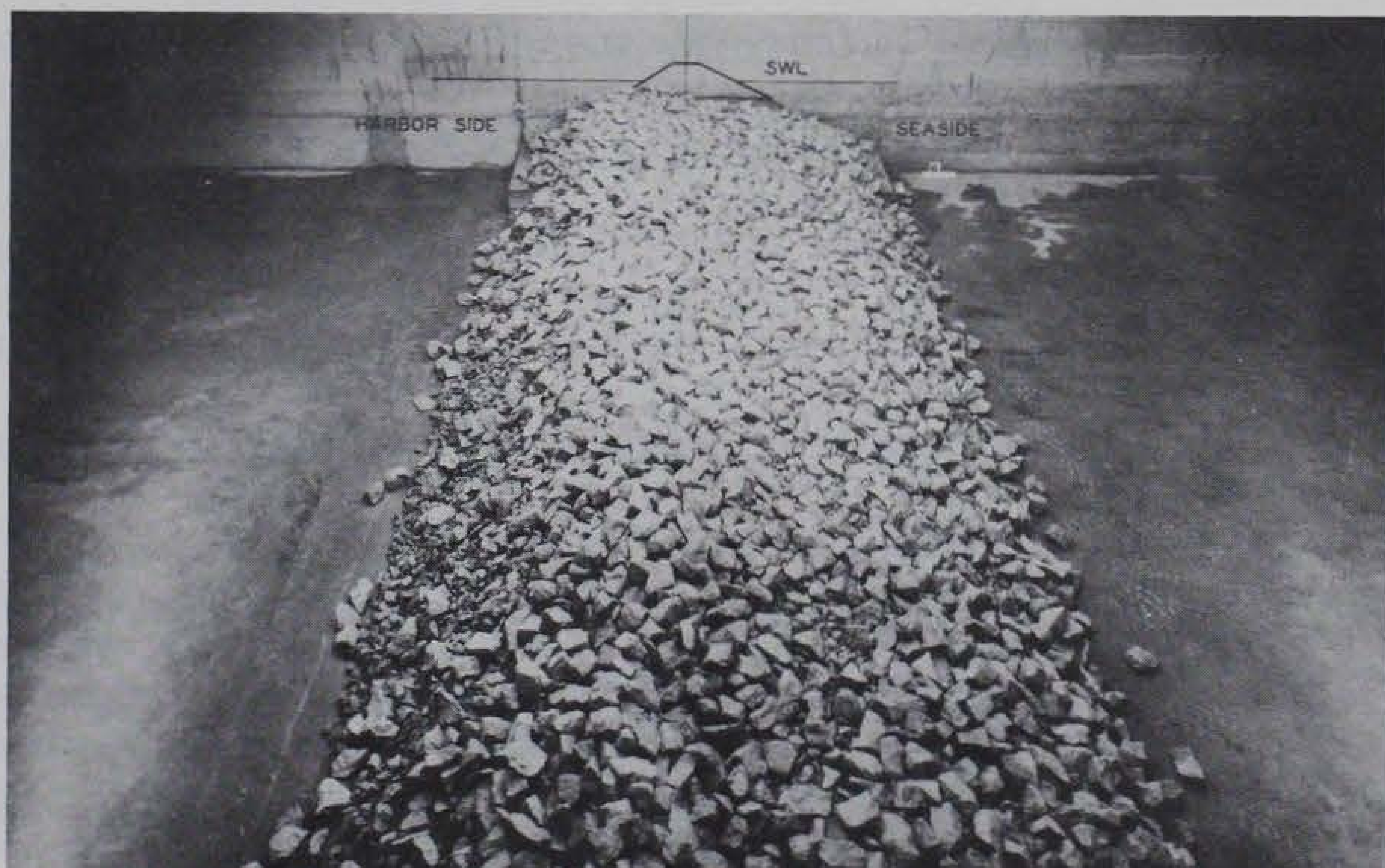

Fig. 13. Test series 4, complete breakwater section after 21-ft wave attack, class A stone of crushed limestone

Test series 5, miscellaneous design problems

36. Description. This series of tests, involving miscellaneous design and development problems of composite rubble-mound breakwaters, was performed to provide information concerning: (a) the minimum quantities of class B material required to protect the class C material during breakwater construction (designs 1, 2, and 3); (b) the most advantageous proportions and placement of class A and class B materials (designs 4 and 5); (c) a breakwater section that would be more stable than the original section when subjected to the attack of 21.0-ft by 300-ft waves in 58 ft of water (designs 6 and 7); (d) the effects of different crown widths when the top elevation of the original breakwater section was raised from 10 ft to 15 ft above still-water level (designs 8, 9, and 10); (e) the effects of varying the crest elevation on the stability of the best model-developed breakwater section (design 11); and (f) the effects on stability of using heavier class A cap rock (design 12). Tests of this series were conducted using 21.0-ft by 300-ft waves, unless otherwise noted. The test sections were subjected to wave attack until stability obtained. The intervals of time required for stabilization of designs involving class B sections and complete sections were 4-1/4 and 4-3/4 hours (model time), respectively. The elements of the series 5 breakwater test sections are shown on plates 24 and 25.

a. Designs 1, 2 and 3. The crown shape and elevation of design 1 (plate 24) were established from the results of tests previously conducted on a class B breakwater section. Design 2 was established by reducing the thickness of the class B enrockment (measured at the center line of section) to about half that used for the original design (plate 1). Design 3 represents a breakwater section constructed in lifts in such a manner as to utilize a maximum amount of class C material and a minimum amount of class B material.

b. Designs 4 and 5. The top elevations and slopes of these test sections (plate 24), together with the boundaries of the various classes of materials were established from the results of tests shown on plate 22. Stability tests on designs 4 and 5 show the effect of flatter seaside slopes on the breakwater.

c. Designs 6 and 7. Designs 6 and 7 (plates 24 and 25) were developed from the results of tests on the original breakwater section. These test results indicated the need for flatter seaside slopes. Design 6 provides the flatter side slope. Design 7 is a rebuilt design 6 section with material previously eroded by wave action left in place on the seaside of the structure.

d. Designs 8, 9 and 10. The elements of these designs are shown on plate 25. The crown widths of the breakwater test sections were 10 ft, 20 ft, and 30 ft. The side slopes were the same as those used for the original breakwater design.

e. Design 11. Design 11 (plate 25) was developed from the results of tests on design 7 (plate 27). Design 7 was selected because it appeared to be the most stable breakwater section developed from the results of the series 5 tests. The water level was raised or lowered as required to obtain effective top elevations of 0.0 ft, 5.0 ft, 10.0 ft and 15.0 ft above still-water level.

f. Design 12. Design 12 (plate 25) consisted of the original breakwater section using class A cap rock which conformed to the following specifications.

Per Cent of Total	Prototype Weight
75	20 - 24 tons
20	6 - 18 tons
5	2 - 4 tons

The original specifications for the class A cap rock are tabulated in paragraph 24.

37. Results. Results of tests of composite rubble-mound breakwaters are discussed in the following subparagraphs:

a. Designs 1, 2 and 3. The results of stability tests on designs 1 and 2 (plate 26) show that, for the test conditions used, the amount of class B material was sufficient to provide adequate protection for the class C material during breakwater construction. Stability tests on design 3 (plate 26) indicate that this design would not provide adequate protection for the class C material. Results of tests on all three designs show that for 21.0-ft waves the top elevation of the class B material should not be raised above about -15 ft swl. A blanket of class B material about 8 ft thick (measured at the center line of section) would be sufficient to protect the class C material from erosion provided the top of the class B material was not raised above about -15 ft swl. Comparison of the results of designs 1 and 2 with design 3 shows that the depth of the top of the class B material below swl was more critical than the thickness of the blanket of this material.

b. Designs 4 and 5. The results of stability tests on designs 4 and 5 (plate 26) show that flattening the side slopes on the seaward side of the breakwater increases the stability of the structure. However, despite the flatter side slopes, which involve the use of considerably more material, neither of these designs was stable under the attack of 21.0-ft waves. Comparison of the stabilized sections of these two designs shows that design 5 was the better design. Because the side slopes of design 5 were flatter than those of design 4, it is evident that flattening the seaside slopes increases the stability of a rubble-mound breakwater.

c. Designs 6 and 7. The results of tests on designs 6 and 7 show the effects on stability of flattening the seaside slopes of the original design. As shown on plate 26, the top elevation of the stabilized design 6 section was about the same as before wave attack. Since the crown elevation and width were not changed by wave action, the effectiveness of design 6 for harbor protection would be unimpaired by the attack of 21.0-ft waves. Design 7 represents a rebuilt design 6 section with the eroded material left in place on the seaside toe of the structure. Comparison of the stabilized sections of designs 6 and 7 (plates 26 and 27) shows that the stability of a breakwater would be improved measurably by rebuilding a section that had been previously stabilized by wave action.

d. Designs 8, 9 and 10. The stabilized sections of designs 8, 9 and 10 are shown on plate 27. These data indicate that, with a crown elevation of +15 ft swl, decreasing the crown width of the structure results in more erosion on the seaside of the breakwater with a corresponding decrease in stability. The shapes of the seasides of these breakwaters were somewhat similar, the principal variation being the difference in elevation of crown.

e. Design 11. The results of tests conducted on design 11 (plates 27 and 28) show that minimum erosion occurs for the breakwater section with a top elevation of +10 ft swl. This particular section was eroded in such a manner that its crown elevation was unchanged. Test data show that, for the test conditions used, design 11, with a crown elevation of +10 ft swl, would require a minimum of repairs to maintain that crown elevation. The elevation of the crown of the design 11 breakwater tested in 53 ft of water was reduced from 15 ft to about 12 ft above still-water level by the attack of 21.0-ft by 300-ft waves. Considerable erosion also occurred on the seaside of this test section. Stability tests on design 11 with a crown elevation of 0.0 ft swl, in 68 ft of water, showed little or no erosion of the crown or of the seaside slope.

f. Design 12. Comparison of results of stability tests on design 12 with test results on a similar type breakwater constructed of lighter weight material (compare plates 23 and 28) indicates that the use of heavier cap rock causes more erosion from the design section. These results are contrary to what would be expected from rational analysis. Check tests would have been conducted except that similar tests were then being planned to determine the reliability of Iribarren's formula for the design of rubble-mound breakwaters. The results of the latter tests (see part V, paragraphs 41-53) showed that the stability of rubble breakwaters increases when the weight of cap rock is increased. The test results on the design 12 breakwater section cannot, therefore, be explained satisfactorily.

Summary of results (test series 1-5)

38. Class C (core) material placed without class B cap stone on the side slopes was eroded considerably by wave action. The amount of erosion increased as the wave dimensions and height of fill increased. Erosion of core material was reduced appreciably by the placing of class B stone on both side slopes concurrently with the dumping of core material. The covering layer of class B stone on top of the core-material section was eroded by the larger waves used in the tests. This indicates

that class A cap rock should be placed in prototype construction as soon as possible after the dumping of class B stone, if construction is in progress during storm seasons.

39. Both types of class A cap rock used in these tests were displaced extensively by the larger waves. There was no appreciable difference in the amount of displacement which occurred for the rubble and the molded concrete blocks. The stability of each type cap rock was improved by rebuilding to the original section after damage occurred. Cap rock should extend below swl a sufficient distance to prevent erosion at the toe of the cap-rock slope.

40. The results of tests in which the crown width was varied showed that erosion of the crown by overtopping is increased when the crown width is reduced. Erosion of the crown and seaside slope occurred for the larger waves, and was maximum for the higher and more narrow crowns.

PART V: ACCURACY OF FORMULAS FOR DESIGN OF RUBBLE-MOUND BREAKWATERS

Introductory Remarks

41. The ultimate objective of a comprehensive investigation of rubble-mound breakwaters should be to provide design engineers with sufficient information for the solution of all important problems encountered in the design and construction of adequate and economical structures. A large number of variables are involved in the phenomenon of waves attacking a mound of rubble, and the questions that arise in the selection and design of rubble breakwaters are numerous and of considerable difficulty. Several specific problems of breakwater design were studied in those phases of the investigation described in this part of the report. Yet it was realized that maximum value would not be obtained from the investigation unless the test results could be generalized as much as possible. A testing program that would include all conditions and variables involved in rubble breakwater design was considered impracticable. It was decided, therefore, that tests would be performed on a simple idealized section of rubble breakwater in such manner that the accuracy of the best existing design formulas could be determined. The design of rubble breakwaters involves primarily the selection of cap rock of adequate size and specific weight to withstand the forces imposed by waves of different dimensions on rubble mounds of different side slopes. Other variables, which may or may not be important, include the angle of incidence of the attacking waves, depth of water, wave period or wave length, shape of rock, porosity of or size of voids in the rubble mound,

width of structure near swl, and height of crown above swl. The best known and most widely used design formula was developed in 1938 by Ramon Iribarren Cavanilles. A similar formula* was developed by Messrs. Harris Epstein and F. C. Tyrrell in 1949. Tests were performed to determine the reliability of these formulas. Also, some progress was made in establishing coefficients for the Iribarren formula which should improve its accuracy and range of application.

Investigation of Iribarren's Formula

42. The original Iribarren formula is:

$$W = \frac{K\, H^3\, S_r}{(\cos\alpha - \sin\alpha)^3\,(S_r - 1)^3} \qquad (1)$$

In equation (1), W is the weight of individual cap rock in kilograms; K is a coefficient with a value of 15 for natural rock and 19 for concrete blocks; H is wave height in meters; S_r is the specific weight of cap rock in metric tons per cubic meter; and α is the angle of the seaside slope, measured from the horizontal. A discussion of this formula, and a detailed explanation of the more general formula and its derivation, are presented in appendix A of this report. The more complete and general form of the Iribarren formula was shown to be

$$W = \frac{K'\, \gamma_f^{\,3}\, \gamma_r\, \mu^3\, H^3}{(\mu\cos\alpha - \sin\alpha)^3\,(\gamma_r - \gamma_f)^3} \qquad (2)$$

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* "Design of Rubble-mound Breakwaters," by Harris Epstein and F. C. Tyrrell, Section 2, Communication 4, XVIIth International Congress of Navigation, Lisbon, 1949.

In equation (2), W is the weight of individual cap rock; K' is an undetermined dimensionless coefficient; γ_f is the specific weight of the liquid in which the cap rock is submerged; γ_r is the specific weight of the cap rock; μ is the effective coefficient of friction, rock on rock; H is the height of wave at the position of the breakwater before the structure was constructed; and α is the angle of the seaside slope, measured from the horizontal. Equation (2) is dimensionally homogeneous and any consistent system of units can be used. Hereinafter, the American system of units is used, in which length is in feet, time is in seconds, force is in pounds and mass is in slugs. If all the assumptions used in the derivation of the above equations were completely valid, the so-called constant K of equation (1) would indeed be constant, and K' of equation (2) also would be constant. The tests were performed in such manner that variations of K' with variations of other important parameters that influence the stability of cap rock could be determined.

Testing procedure

43. The tests were conducted on a 1:45-scale model of idealized rubble-mound breakwater sections. A prototype water depth of 90 ft was simulated in these tests in order that the range of wave dimensions generated by the wave machine could be increased. Plate 29 and fig. 14 (page 36) show elements of typical test sections. Most of the tests were conducted with the 18-ft-wide flume divided longitudinally, resulting in a test section 8.5 ft long. However, some of the tests were conducted using a test section only 5 ft in length. These lengths are model dimensions.

44. The model breakwater sections shown on plate 29 were constructed of cap rock, armor rock and core material. In each test, cap

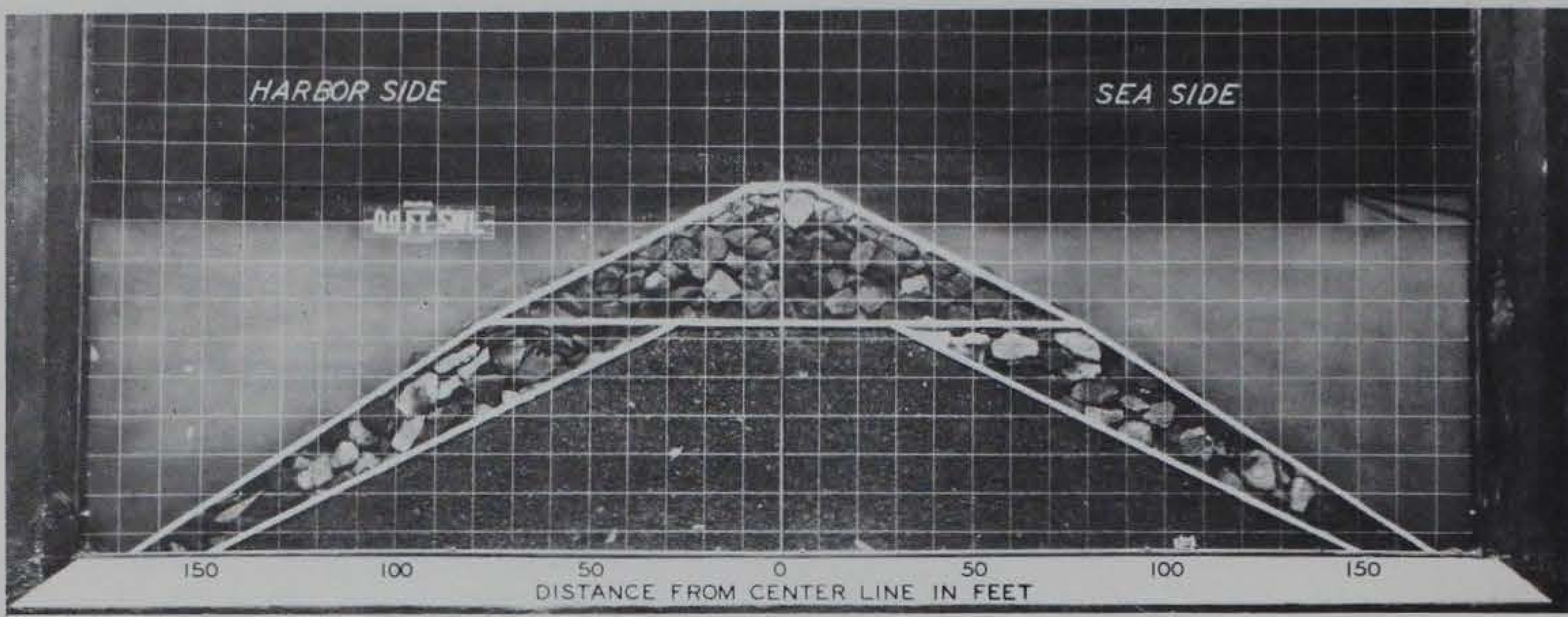

Fig. 14. Investigation of Iribarren's formula, elements of typical test section

rock of the same weight, specific weight and, as nearly as possible, the same shape, was used above an elevation of -25 ft swl (figs. 15-17). Armor rock was used on the side slopes below elevation -25 ft swl. All armor rock had the same specific weight as that of the cap rock and was of approximately the same weight, but no effort was made to shape the armor rock as was done in the case of the cap rock. Armor rock was graded by use of sieves, whereas each individual cap rock was sized by hand and weighed on torsion balances. The cap and armor rock varied in weight according to the different tests performed. However, the same core material was used in all tests. This material was crushed rock simulating, to a numerical ratio of $1:45^3$, prototype rock weighing from 5 lb to 1000 lb.

45. The model breakwaters were constructed on a sand base 90 ft (model) from the wave generator. The core material was molded with no water in the flume. Some consolidation of the core was accomplished by wetting during the molding operation. Armor rock was also placed in a dry flume. After the water, or other testing liquid, had been brought to

Fig. 15. Investigation of Iribarren's formula, 4.6-ton, 2.8-specific-gravity cap rock

Fig. 16. Investigation of Iribarren's formula, 13.5-ton, 2.8-specific-gravity cap rock

Fig. 17. Investigation of Iribarren's formula, 27.4-ton, 2.8-specific-gravity cap rock

the proper elevation (zero swl), cap rock from elevation -25 ft to zero ft swl was dumped from a bucket at the water surface. Cap rock above swl was placed by hand. This method of constructing the model breakwaters was adopted to simulate as nearly as possible a prototype method consisting of pushing stone from barges or gondolas and placing above-water stone by crane sling. After each test the cap rock above -25 ft swl was removed and reconstructed and the armor rock was repaired to the line and grade of the original test section.

46. The design wave for a particular breakwater test was determined experimentally by subjecting the model structure to wave attack until a wave height was found which was slightly less than that required to move cap rock from the breakwater slope. This wave was called the design wave for the no-damage criterion. For a few tests another design wave was determined which would damage the breakwater slightly but would not cause sufficient damage to decrease appreciably the efficiency or stability of the structure. This wave was called the design wave for the slight-damage criterion. Plate 30 illustrates the criteria for selection of these two types of design waves.

47. Accurate determination of the height of test waves was complicated by the existence of waves of abnormal height in the train of waves between the wave machine and test section. These larger waves were caused by starting and stopping the wave machine. It was thought that the larger waves at the start and end of each wave train influenced the stability of cap rock, and the selection of design waves. Therefore it was decided to consider these waves as an integral part of the wave train. The selected design waves were defined statistically as the average height of the one-third highest waves of the wave train. Complete wave trains were recorded by use of the wave gages and oscillograph. Waves thus defined are called "significant waves" by Sverdrup and Munk*.

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* "Wind, Sea and Swell: Theory of Relations for Forecasting," by H. U. Sverdrup and W. H. Munk. Hydrographic Office, U. S. Navy Department, Washington, D. C., March 1947.

48. It had been observed from previous tests on rubble-mound breakwaters that the displacement of rock from the face slope occurred over a considerable period of time when waves were of sufficient height to cause damage to the test section. In view of this fact use of the maximum testing time possible was considered desirable. A test period of two hours model time was selected; however, to establish that this was sufficient time to obtain a stable section, a test was made in which the testing time was five hours. The results of this test showed that two hours of wave attack was sufficient; therefore each test section was subjected to this test period. In this series of tests, identical repeat tests were performed to obtain information as to the over-all accuracy of the testing procedure. The results of these tests, shown on plate 31, indicate only small differences between the initial and check-test results.

Description of tests performed

49. Stability tests were performed with waves approaching perpendicular to the breakwater alignment (wave crests parallel to the breakwater) for the following range of conditions of waves and breakwater characteristics (prototype values):

Characteristic	Range of Test Conditions
Wave height (H)	2 to 31 ft
Water depth (d)	One depth only, 90 ft
Wave length (L)	245 - 565 ft
Wave period (T)	7 - 12 sec
Specific weight cap rock (γ_r) actual*	175 lb/cu ft
Specific weight cap rock (γ_r) simulated*	137 - 175 lb/cu ft
Weight of individual cap rock* (W)	4.6 - 36.4 ton
Specific weight of testing liquid* (γ_f)	62.4 - 79.3 lb/cu ft
(Continued)	

* All tests were performed using the same cap rock of specific gravity 2.8. However, other weight and specific-weight rock were simulated by the use of higher-density liquid (see appendix B).

Characteristic	Range of Test Conditions
Breakwater slope	1 on 1 to 1 on 3
Breakwater test section	(see plate 29)
Breakwater crown elevation referred to swl	(see note)* ft

* The crown elevations were always of sufficient height to prevent overtopping of the structure.

Figs. 18 and 19 show the elements of two model test sections. Other

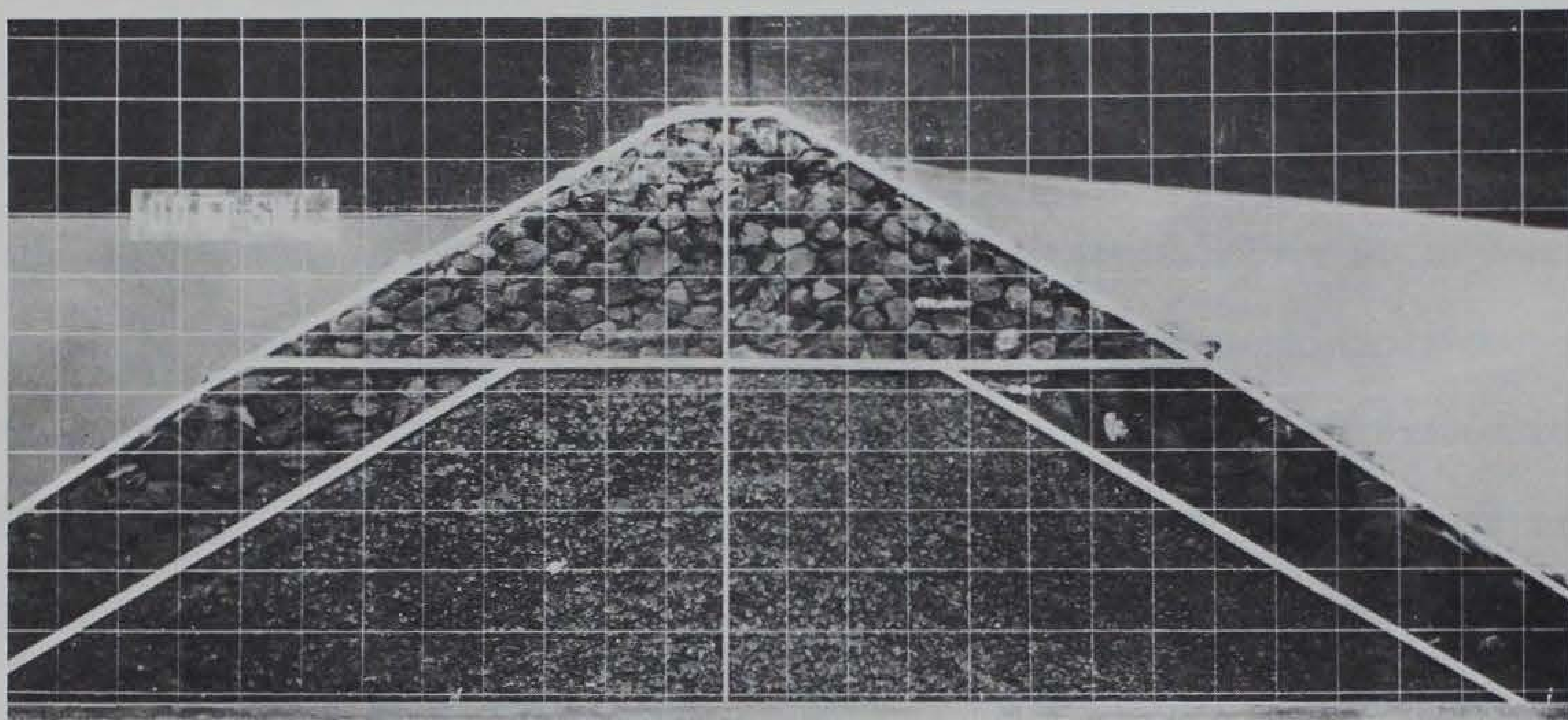

Fig. 18. Investigation of Iribarren's formula, 10-sec waves 11 ft high attacking a rubble breakwater constructed of 4.6-ton, 2.8-specific-gravity, basalt cap rock placed on 1 on 1-2/3 slope

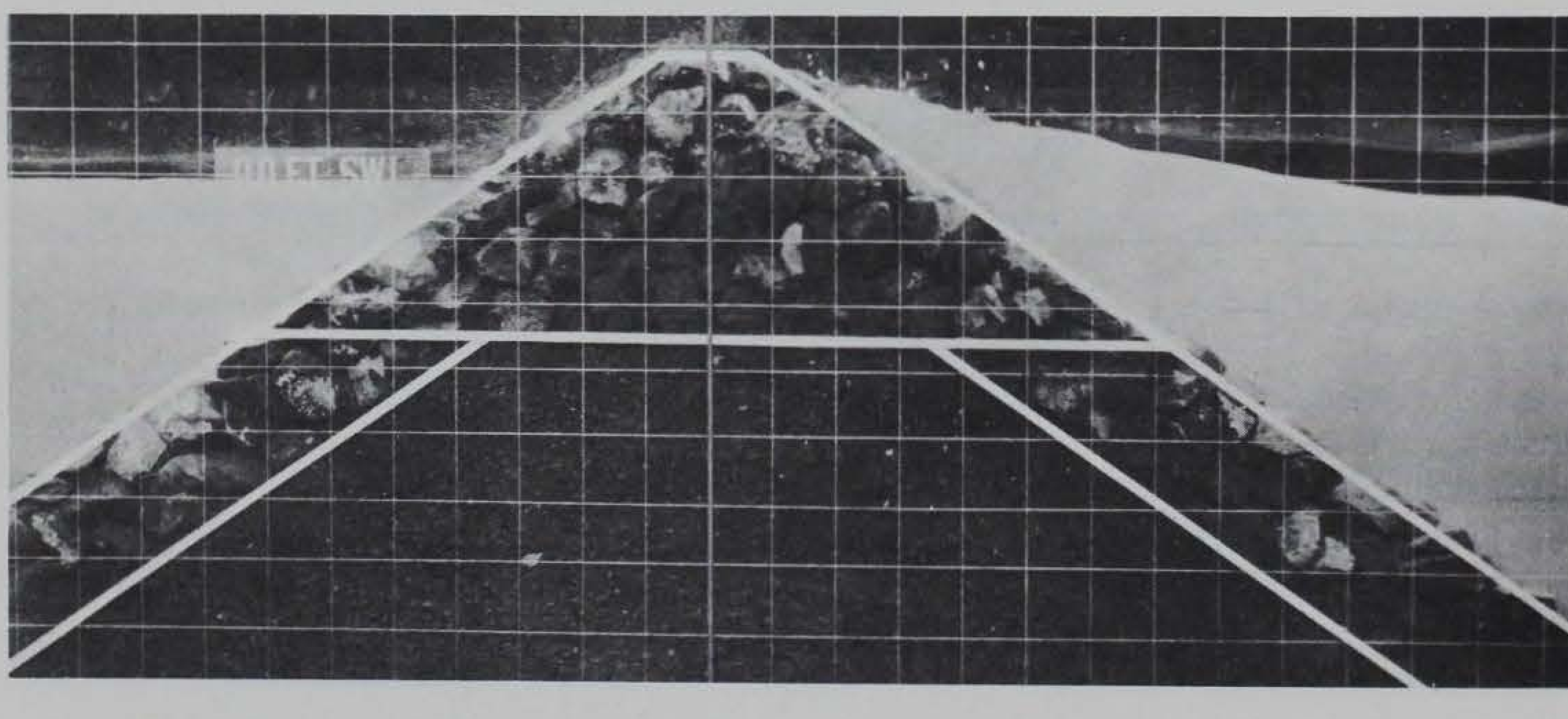

Fig. 19. Investigation of Iribarren's formula, 10-sec waves 17 ft high attacking a rubble breakwater constructed of 27.4-ton, 2.8-specific-gravity, basalt cap rock placed on a 1 on 1-1/2 slope

tests were performed in which the angle of incidence of the attacking waves was varied from zero to 90°. These were special tests to determine the effects of a particular variable and are presented in part VI of this report. Also, several tests were made in which the crown elevation of the breakwater was held constant at +10 ft swl. In these tests, the results of which are presented in part VI, design waves for both the no-damage and slight-damage criteria were determined. The basic assumptions from which Iribarren's formula was derived do not include overtopping of the structure (see appendix A). The accuracy of the formula, therefore, should not be judged by the results of such tests.

Results of tests

50. The results of tests to determine the reliability of Iribarren's formula are shown on the following page in tabular form and by plates 32 and 33 where the selected design wave for each test condition is compared with that calculated by the Iribarren formula. Also the results are shown by plates 34 and 35 where the calculated values of K', using model test data, are plotted against dimensionless parameters found to be important with respect to the stability of cap rock. These latter data are considered most significant in that they show that K' is not constant but varies with breakwater slope (α) and $\frac{d}{L}$ and $\frac{H}{L}$ ratios. For all the test data tabulated on the following page crown elevations were sufficient to prevent overtopping, the water depth was 90 ft, and the design waves were selected on the basis of the no-damage criterion:

Test Data*: Investigation of Iribarren's Formula

Side Slope	Crown Elev, Ft, swl	Wave Period, Sec	Design-wave Heights, Ft Model Data	Design-wave Heights, Ft Calculated**
		4.6-ton, 2.8-specific-gravity Cap Rock		
1 on 1-1/4	+10	10	9	5
1 on 1-1/2	+14	10	11	8
1 on 1-2/3	+18	10	11	9
		13.5-ton, 2.8-specific-gravity Cap Rock		
1 on 1	+10	7.0	10	2
1 on 1	+10	7.0	8#	2
1 on 1	+10	10.0	7	2
1 on 1	+10	12.0	6	2
1 on 1-1/4	+10	7.0	11	8
1 on 1-1/4	+10	10.0	13	8
1 on 1-1/4	+13	10.0	11#	8
1 on 1-1/2	+13	7.0	14	13
1 on 1-1/2	+13	10.0	13	13
1 on 1-1/2	+13	12.0	12	13
1 on 1-2/3	+15	9.3	15	15
1 on 1-2/3	+15	10.0	13	15
1 on 1-2/3	+15	10.0	16#	15
1 on 1-5/6	+20	10.0	18	17
1 on 2	+20	7.0	19	19
1 on 2	+20	10.0	18	19
1 on 2	+20	12.0	17	19
1 on 2-1/2	+22	8.0	21	23
1 on 2-1/2	+22	10.0	20	23
1 on 2-1/2	+22	12.0	19	23
1 on 3	+25	8.0	21	26
1 on 3	+25	10.0	20	26
1 on 3	+25	12.0	19	26
		27.4-ton, 2.8-specific-gravity Cap Rock		
1 on 1-1/4	+15	10.0	15	13
1 on 1-1/2	+18	10.0	17	18
1 on 1-2/3	+20	10.0	19	21
		12.5-ton, 2.6-specific-gravity Cap Rock##		
1 on 1-1/2	+13	10.0	11	11
1 on 2	+20	10.0	15	17
1 on 2-1/2	+22	10.0	18	21

* Test data are converted to equivalent prototype units.

** Calculated using Iribarren's formula.

\# Check tests.

\## Simulated by using 13.5-ton, 2.8-specific-gravity cap rock and liquids heavier than pure water (specific gravity greater than one) (appendix B).

Test Data*: Investigation of Iribarren's Formula (Continued)

Side Slope	Crown Elev, Ft, swl	Wave Period, Sec	Design-wave Heights, Ft Model Data	Calculated**
	11.5-ton, 2.4-specific-gravity Cap Rock##			
1 on 1-1/2	+12	10.0	10	10
1 on 2	+18	10.0	14	15
1 on 2-1/2	+18	10.0	16	18
	10.6-ton, 2.2-specific-gravity Cap Rock##			
1 on 1-1/2	+12	10.0	8	8
1 on 2	+16	10.0	11	13
1 on 2-1/2	+16	10.0	13	15

* Test data are converted to equivalent prototype units.
** Calculated using Iribarren's formula.
Simulated by using 13.5-ton, 2.8-specific-gravity cap rock and liquids heavier than pure water (specific gravity greater than one) (appendix B).

51. On plates 32 and 33 results of tests are compared with the answers obtained by substitution in the Iribarren formula. On these plates curves of design-wave height plotted against breakwater slope were prepared by use of the Iribarren formula, and the design-wave heights determined by the model tests were plotted on the curves. In this manner direct comparison of the formula with model test data is obtained. According to the model test results, as shown by these plates, the formula gives answers which: (a) are conservative, resulting in over-designed and, therefore, uneconomical structures, for breakwater slopes between 1 on 1 and 1 on 1-1/4; (b) are in excellent agreement with test results for breakwater slopes between 1 on 1-3/8 and 1 on 2-1/4; and (c) result in unsafe designs for breakwaters with slopes between 1 on 2-3/8 and 1 on 3. From an over-all point of view the results of tests to determine the reliability of Iribarren's formula are very encouraging. They indicate

that the Iribarren formula has possibilities of becoming a dependable tool of the design engineer. However, it was proved conclusively that the formula cannot be considered adequate unless used in conjunction with curves of corrective coefficients.

52. The type of coefficient curves required is shown by plate 34. These data show that the coefficient K' is not constant for a given type of rock, but varies considerably with breakwater slope and to a lesser extent with wave steepness and relative depth ($\frac{H}{L}$ and $\frac{d}{L}$ ratios). The product of these two dimensionless ratios ($\frac{Hd}{L^2}$) was used in order to present the data in the most condensed form possible. A similar set of curves is obtained by using only the $\frac{d}{L}$ ratio as the abscissa (plate 35). It was not convenient to obtain sufficient test data to allow plots of K' vs $\frac{H}{L}$, because the value of H cannot be predetermined, but in each case must be determined experimentally by trial and error. The good correlation of test data obtained by plotting the results in the form used indicates that the parameters chosen are closely interrelated and are important variables which should be considered in the design of rubble breakwaters.

53. Other variables, outlined in appendix A, could not be investigated because of limitations of time and funds. Performance of tests to determine the importance of variables not investigated in this project (see paragraph 41), and extension of the range of variables studied in this project, are considered highly desirable. For instance it would be beneficial to determine values of K' for structures with slopes flatter than 1 on 3 and for larger waves of greater length and period in depths of water less than 90 ft.

Investigation of the Epstein-Tyrrell Formula

54. The Epstein-Tyrrell formula* for design of rubble-mound breakwaters situated in water (sp gr = 1) is

$$W = \frac{R\ S_r H^3}{(\mu - \tan\alpha)^3\ (S_r - 1)^3} \qquad (3)$$

where R is an undetermined dimensional coefficient and W , S_r , H , μ and α are, respectively, weight of individual cap rock, specific gravity of cap rock, design-wave height, coefficient of friction of rock on rock, and angle of breakwater slope. If the necessary changes are made so that the formula will be applicable for breakwaters situated in any liquid (sp wt = γ_f), and if the coefficient R is made dimensionless ($R' = \frac{R}{\gamma_f}$), equation (3) becomes, by proper substitution and rearranging of terms,

$$W = \frac{R'\ \gamma_f^{\,3}\ \gamma_r\ H^3 \cos^3\alpha}{(\mu\cos\alpha - \sin\alpha)^3\ (\gamma_r - \gamma_f)^3} \qquad (4)$$

where γ_r is the specific weight of cap rock and the other terms are as previously defined. Equation (4) is dimensionally homogeneous and any consistent system of units can be used.

55. Derivation of the Epstein-Tyrrell formula was based on a force diagram similar to that used by Iribarren (fig. A1, page A3), except that a tangential force was added which was attributed to the movement of water along the slope. The vertical component of the dynamic pressure on the rock was assumed proportional to the vertical component of the orbital velocity of wave motion, and the horizontal component of

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* Epstein and Tyrrell, loc. cit.

dynamic pressure was assumed proportional to the product of the wave celerity and the horizontal component of the orbital velocity. Also, it was assumed that the waves at the breakwater do not break. Although a rational interpretation of all the assumptions upon which the Epstein-Tyrrell formula is based is difficult, it is interesting to note that the formula is the same as that of Iribarren if

$$R' = \frac{K' \mu^3}{\cos^3 \alpha} \tag{5}$$

(compare equations (2) and (4)).

56. Unlike the Iribarren formula, in which the coefficient K' is required to be determined empirically, Messrs. Epstein and Tyrrell derived an equation for the coefficient R in terms of α, μ, H, d, L and three additional coefficients defined as functions of the size and shape rock. This equation, made dimensionally homogeneous by substitution of $R' = \frac{R}{\gamma_f}$, is

$$R' = \frac{k^3 N_x}{8 N_z^2} \left[(1 + N_z \mu \tan \alpha)^3 + (N_z \mu - \tan \alpha)^3 \left(1 + \frac{\pi H \coth \frac{2\pi d}{L}}{2L} \right)^3 \right] \tag{6}$$

in which the various new terms are defined as follows:

k = ratio of effective to total wave energy ($k \leqq 1$)

$N_x = \frac{x}{e}$

$N_z = \frac{z}{e}$

x = length of individual cap rock measured parallel to the length of breakwater

z = width of individual cap rock measured parallel to slope of breakwater

e = height of individual cap rock measured perpendicular to slope of breakwater

In the American system of units, x, z and e are in feet. The coefficient k of equation (6), like the coefficient K' of Iribarren's formula, must be evaluated experimentally or by direct observation of wave action on prototype structures. The coefficients N_x and N_z can be determined by direct measurement if the cap rocks are regular in shape. In most instances both N_x and N_z can be assumed to be unity with sufficient accuracy. Model tests to determine the accuracy and reliability of the Epstein-Tyrrell formula were performed under such conditions that $N_x = N_z = 1$, and in such manner that the coefficient k could be evaluated.

Description of tests

57. A few tests were conducted in which design-wave heights were determined for a breakwater section with cap rock of 36.4-ton, 2.8-specific-gravity, concrete cubes (figs. 20-22). The effective coefficient

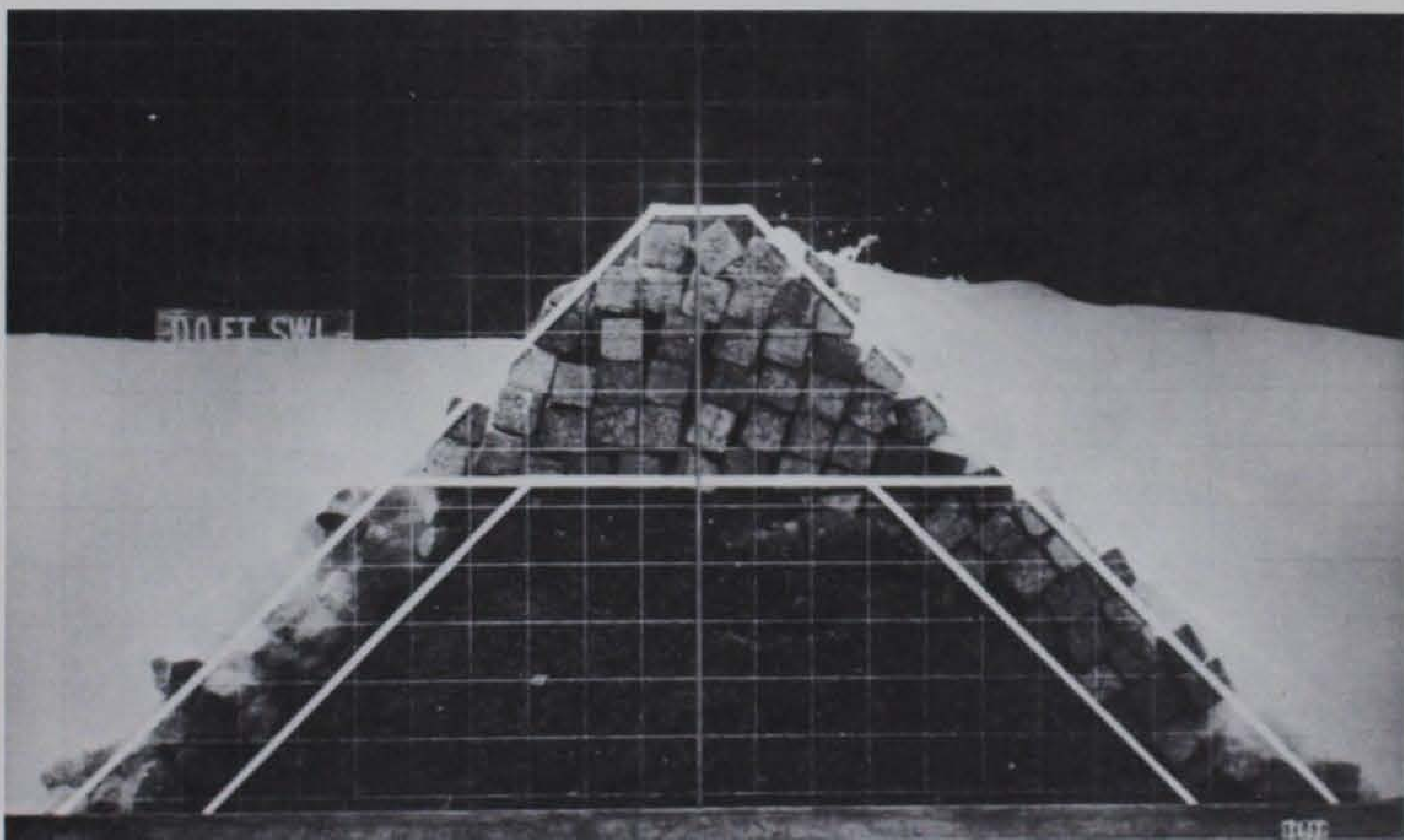

Fig. 20. Investigation of the Epstein-Tyrrell formula, 10-sec waves, 19 ft high, attacking a rubble breakwater constructed of 36.4-ton, 2.8-specific-gravity, concrete-block cap rock placed on a 1-on-1 slope

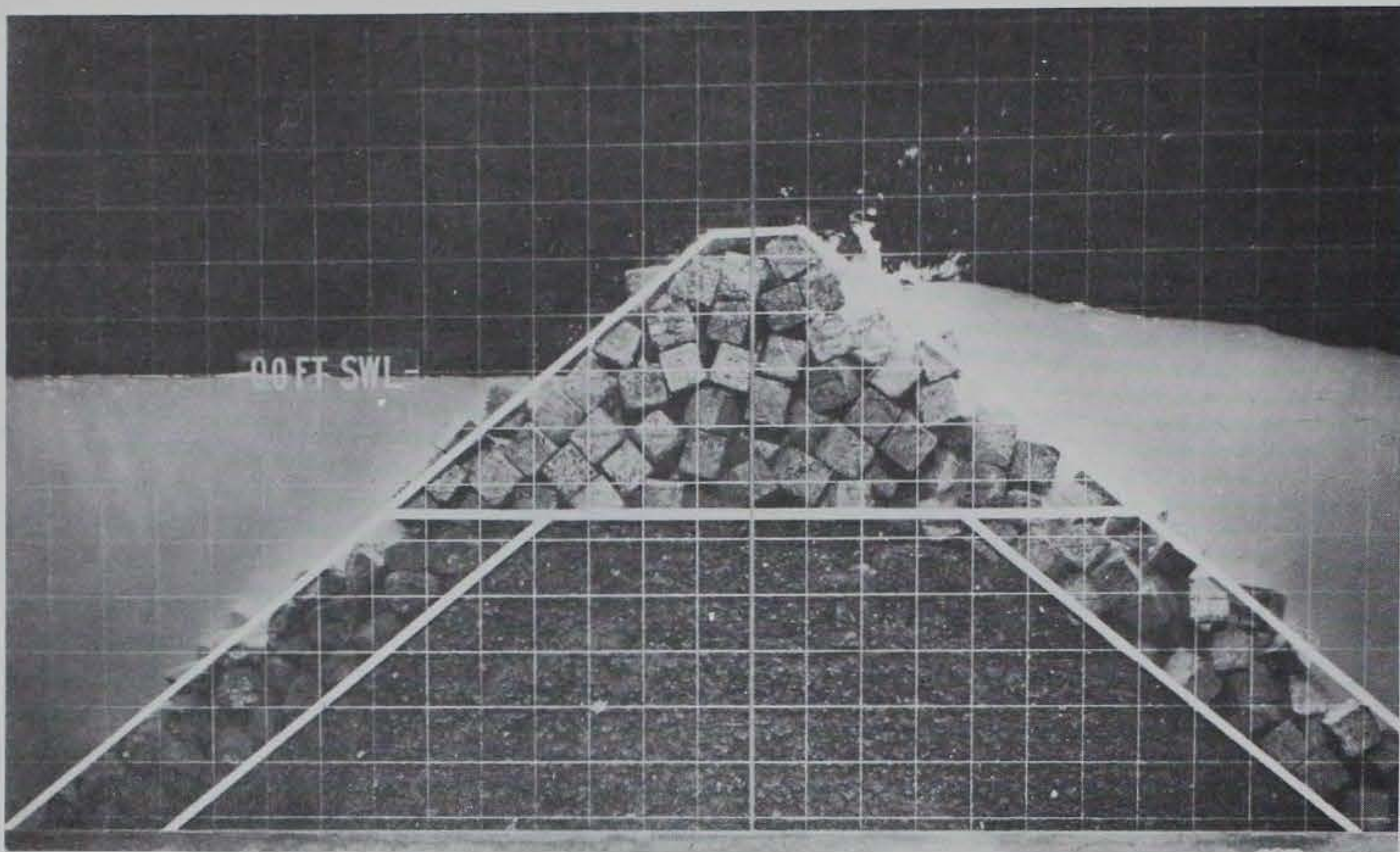

Fig. 21. Investigation of the Epstein-Tyrrell formula, 10-sec waves, 21 ft high, attacking a rubble breakwater constructed of 36.4-ton, 2.8-specific-gravity, concrete-block cap rock placed on a 1-on-1-1/4 slope

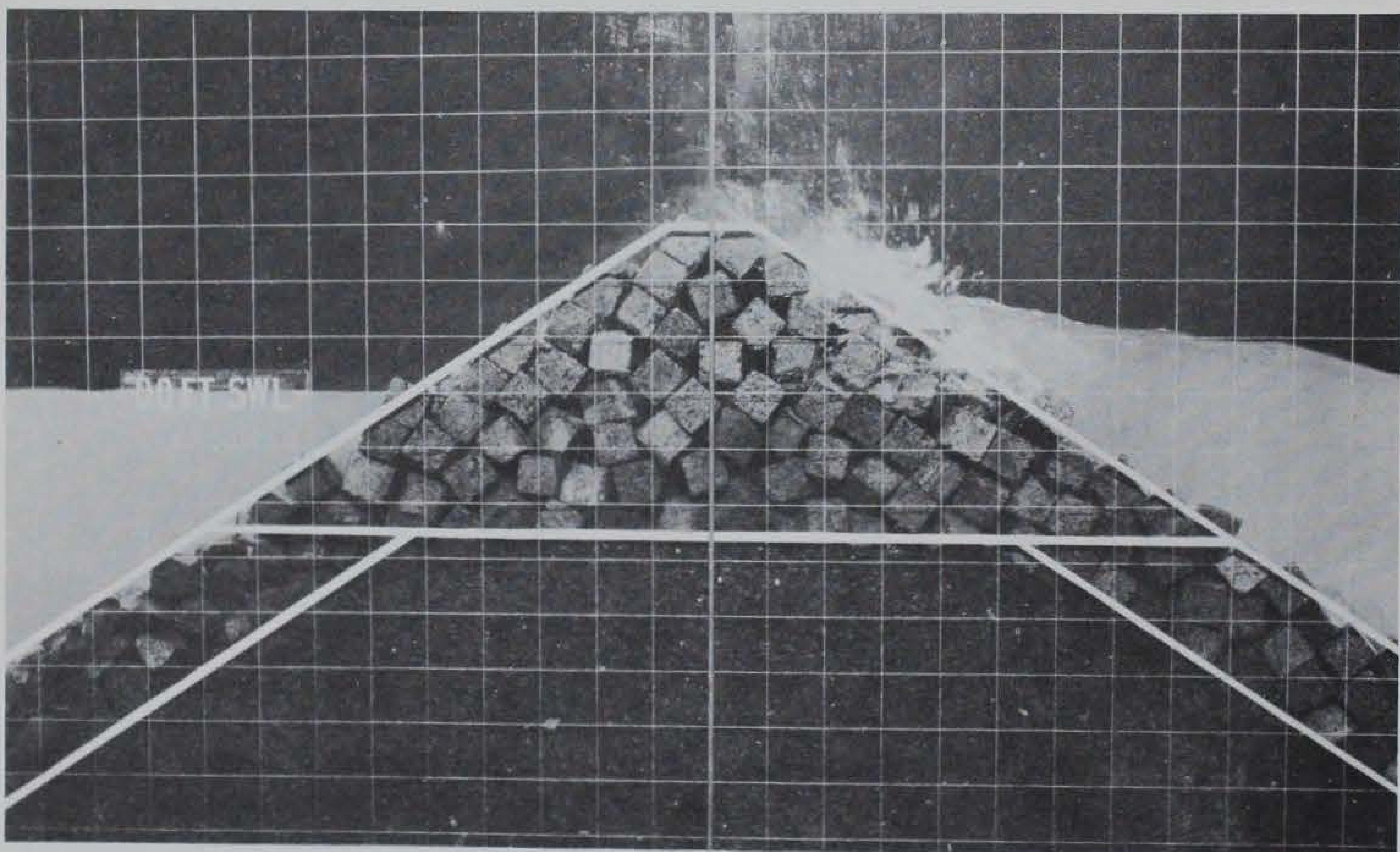

Fig. 22. Investigation of the Epstein-Tyrrell formula, 10-sec waves, 26 ft high, attacking a rubble breakwater constructed of 36.4-ton, 2.8-specific-gravity, concrete-block cap rock placed on a 1-on-1-1/2 slope

of friction of these cubes was found to average 1.34. These tests were conducted in the same manner as those performed to determine the accuracy of Iribarren's formula (paragraphs 43-48).

58. Other tests were conducted in which design wave heights were determined for a solitary (sp gr = 2.25) brick and a solitary concrete cube (sp gr = 2.72) placed at different elevations (referred to swl) on a sloping plane. A water depth of 90 ft, 10-second-period waves and breakwater side slopes of 1 on 1-2/3, 1 on 2, and 1 on 2-1/2 were used. The crown elevations of the test sections were sufficient to prevent overtopping. The sloping surface on which the idealized cap rock was tested was placed on a wooden frame about three feet (model) in length. This apparatus was situated in the center of the breakwater test section. The remaining portion of the test section, on adjoining sides of the framework, was constructed of rubble. The sloping face of the impermeable test section was covered with a strip of rubber 1/8 inch in thickness. Guide vanes were placed between the test section and the adjoining rubble sections. Design-wave heights were determined with different sides of the brick turned toward the incident waves. Three positions of the prisms (zero ft, -15 ft and -30 ft swl)* were used.

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* The Epstein-Tyrrell formula, as presented in equations (3), (4) and (6) was said to be applicable for cap rock situated within the zone that extends from a distance H above swl to H/2 below swl. However, Messrs. Epstein and Tyrrell explain a method of generalizing the equation in such a way that the required size of cap rock, as a function of depth below swl, can be calculated. This formula is somewhat complicated, requires considerable work in its substitution, and is not presented in this report (see page 91 of Section 2, Communication 4, XVIIth International Navigation Congress, Lisbon, 1949).

Results of tests

59. 36.4-ton, 2.8-specific-gravity concrete cubes. Design-wave heights were determined for three breakwater sections constructed of concrete cubes, and the resulting values of k in the formula for R' (equation (6)) were calculated. These data are tabulated below:

Side Slope	Crown Elev, Ft, swl	Wave Period, Sec	Design-wave Ht, Test*	k (equation (6))
1 on 1	22	10.0	19	0.21
1 on 1-1/4	24	10.0	21	0.34
1 on 1-1/2	30	10.0	26	0.35

60. Tests of solitary cube and brick. The results of tests in which design-wave heights were determined for a concrete cube and a brick are presented in table 3. In these tests the wave length was 440 ft, the wave period was 10 sec and the water depth was 90 ft. Other pertinent data are listed in table 3.

61. The average value of k from the data presented above is 0.30. The values of k in table 3 vary from 0.26 to 3.36. By definition, the maximum possible value of k is unity. Therefore, no logical conclusions can be formulated from the results of these tests. It is evident that either the tests were not performed with sufficient accuracy, or the Epstein-Tyrrell equation for R' is inadequate. Because of the fact that the testing technique used for these tests was generally

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* The corresponding design-wave heights calculated from Iribarren's formula are 9 ft, 16 ft and 21 ft, respectively. This indicates that breakwaters with steep slopes constructed of concrete cubes would be considerably overdesigned if Iribarren's formula and his recommended K' for concrete blocks (0.019) were used. Three tests are not sufficient for definite conclusions, and further tests using concrete cubes for cap rock would be desirable.

the same as that used for studying the accuracy of the Iribarren formula, and in view of the excellent K' curves obtained from those tests, it is believed that the Epstein-Tyrrell formula for R' is not adequate and should not be used in the design of rubble-mound breakwaters.

62. An independent check on this conclusion can be obtained by utilizing the data from the investigation of Iribarren's formula, combined with analytical reasoning. As pointed out in paragraph 55, the equations of Iribarren and Epstein-Tyrrell are similar and, numerically,

$$R' = \frac{K' \mu^3}{\cos^3 \alpha} \qquad (5)$$

In the tests of Iribarren's formula, values of K' were determined experimentally for known values of μ and α. Therefore with these data and equation (5), values of R' can be calculated for rubble mounds constructed of rock similar to that used in the Iribarren tests. Also, if average values of k, N_x and N_z are used for the particular rock used in the Iribarren tests, corresponding values of R' can be calculated from the Epstein-Tyrrell equation for R' (equation (6)). Comparison of these two sets of R' values should show whether the Epstein-Tyrrell formula for R' is reliable.

63. The curves of R' vs $\frac{Hd}{L^2}$ and α shown on plate 36 are plots of equations (4) and (5), using values of K' from the curves of plate 34 with corresponding value of μ and α. The curves of R' vs $\frac{Hd}{L^2}$ and α shown on plate 37 are plots of equation (6), using an average value of 0.6*

* The value of 0.6 for k was obtained from table 3. By definition $k \leq 1.0$. Therefore, all values > 1.0 were discarded and the remaining values averaged. In this manner k = 0.6 was obtained.

for k and $N_x = N_z = 1$*. It can be seen that the curves on plate 36 show no similarity to the curves of plate 37. If the Epstein-Tyrrell formula for R' (equation (6)) were completely rational, the two sets of curves would not only be similar but would be practically identical. The evidence indicates, therefore, that the Epstein-Tyrrell formula for R' is not reliable, and should not be used for design of rubble breakwaters. However, the general Epstein-Tyrrell formula (equation (4)) can be made as useful and reliable as the Iribarren formula if it is used in conjunction with curves of R' similar to those on plate 36. The experimental work involved in the accumulation of data necessary for preparation of the curves on plate 36 is the same as that needed for preparation of K' curves for the Iribarren formula (plate 34). In fact, the same data can be used for preparation of both the K' and R' coefficient curves.

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* $N_x = N_z = 1$ corresponds to cap rock in the form of cubes or spheres. Also, for the rock used in the tests of Iribarren's formula, the average values of N_x and N_z were approximately unity.

PART VI: MISCELLANEOUS TESTS

64. A few tests were performed to obtain information of limited scope concerning certain phases of breakwater design and the action of waves on these structures under particular conditions. Data were obtained concerning: (a) the magnitude of waves reflected from rubble breakwaters as a function of breakwater slope (α), weight rock (W), and the ratio $\frac{Hd}{L^2}$; (b) the effects of angle of incidence of wave attack on the stability of rubble breakwaters and on the selection of design-wave heights; (c) the effects on design-wave heights of allowing slight damage to the breakwater; (d) the efficacy of a rubble mound capped with molded concrete cubes, placed in such manner as to obtain a plane surface; and (e) the effects of compressed air on wave heights.

Magnitude of Waves Reflected from Rubble-mound Breakwaters

65. Tests to determine the reliability of the Iribarren and Epstein-Tyrrell formulas provided an opportunity to obtain data concerning the reflection of waves from sloping rubble-mound breakwaters. The literature contains no quantitative information, either experimental or theoretical, pertaining to the reflection of surface (wind) waves (progressive oscillatory waves) from rubble mounds. The reason for this lack of information became apparent when a method was sought by which the magnitude of reflected waves could be determined. When oscillatory waves are reflected from a vertical wall they are equal in height to the incident waves, and a standing-wave system is developed in which the amplitude at loop points is twice the amplitude of incident waves. However, when

only a portion of the wave energy is reflected, as in the case of most rubble breakwaters, the standing-wave system is more complex because the amplitude of the reflected waves is smaller than that of the incident waves.

66. The method used to determine the height of waves reflected from rubble-mound breakwaters was suggested by Dean M. P. O'Brien, University of California, Berkeley, California. This method, which was later developed mathematically by Dr. G. H. Keulegan*, is explained as follows: A wave system seaward of a breakwater can be considered to consist of an incident wave moving in a positive direction along the x axis

$$\eta_i = A_i \left(\cos kx - \sigma t\right) , \tag{7}$$

and a reflected wave moving in the opposite direction

$$\eta_r = A_r \left(\cos kx + \sigma t\right) . \tag{8}$$

η_i and η_r are the components of the surface elevation due to each wave; A_i and A_r are the amplitudes of the incident and reflected waves; and σ and k are related to the wave period (T) and the wave length (L) in the manner

$$\sigma = \frac{2\pi}{T} \tag{9}$$

and

$$k = \frac{2\pi}{L} . \tag{10}$$

Based upon the principle of superposition, which applies strictly only

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* "A Method of Determining the Form of Oscillatory Waves Reflected From a Breakwater," by G. H. Keulegan, National Bureau of Standards, Washington, D. C., July 31, 1950.

to waves of sinusoidal form (or in practice, waves of small height relative to length), the resultant displacement is

$$\eta = \eta_i + \eta_r , \tag{11}$$

or,

$$\eta = A_i \, (\cos kx - \sigma t) + A_r \, (\cos kx + \sigma t) . \tag{12}$$

Introducing the expressions for the cosine of the sum and difference of two angles and combining terms:

$$\eta = (A_i + A_r) \cos kx \cos \sigma t + (A_i - A_r) \sin kx \sin \sigma t . \tag{13}$$

There are two positions within a distance of $\frac{L}{4}$ from the reflecting surface where variations of η with respect to x vanish. At one of these positions η is a maximum. This position is at $x = 0$. At the other position, $x = \frac{L}{4}$, η is a minimum. Denoting the positions of maximum and minimum fluctuation as η_1 and η_2, respectively, and substituting $x = 0$ and $t = 0$ in equation (13), it is found that

$$\eta_1 = A_i + A_r . \tag{14}$$

In like manner, by substituting $x = \frac{L}{4}$ and $t = \frac{T}{4}$ in equation (13),

$$\eta_2 = A_i - A_r . \tag{15}$$

Therefore, the coefficient of reflection for waves of small height is

$$\frac{H_r}{H_i} = \frac{H_1 - H_2}{H_1 + H_2} \tag{16}$$

where H_1 and H_2 are the wave heights measured at $x = 0$ and $x = \frac{L}{4}$, respectively.

67. The numerical magnitude of the coefficient of reflection ($\frac{H_r}{H_i}$) was determined for most of the breakwaters tested in connection with the study of Iribarren's formula. These data are presented in table 4 and plate 38. It can be seen from the curves of plate 38 that the coefficient of reflection varies considerably with both the slope (α) of the breakwater and the $\frac{Hd}{L^2}$ ratio. The scattering of points in the median range of slopes (1 on 1-1/2 to 1 on 2) is believed to be due in most part to experimental error and the effects of different size voids in the rubble mounds, corresponding to the different weight rock used. All coefficients of reflection were obtained for rubble breakwaters with no overtopping. Although these data should be useful for purposes of preliminary design, further tests are needed to isolate the effects of rock size and to obtain similar curves with different degrees of overtopping. Also, the actual coefficients of reflection of rubble-mound breakwaters are believed to be considerably greater than these results indicate. The theory upon which equation (16) was based assumes a pure sine wave. For standing waves generated by pure sine waves the amplitude at node points is zero and is twice the amplitude of the incident wave at loop points. However, the test waves used were elliptically trochoidal and the vertical motion at the node points was not zero. The calculated value of the reflection coefficient for a vertical-wall breakwater, using model-test measurements and equation (16), was found to be as low as 0.6-0.7. If the wave form is sinusoidal the coefficient ($\frac{H_r}{H_i}$) should be unity for a vertical-wall breakwater. Therefore, the data presented are believed to be in error as much as 30-40 per cent. The average error is probably about 25 per cent.

Effects of Angle of Incidence on the Stability of Rubble-mound Breakwaters

68. All tests discussed in other parts of this report were performed with the direction of approach perpendicular to the alignment of the breakwater (angle of incidence = 90°). The effects of varying the direction of approach on the stability of rubble-mound cap rock were studied in special tests in which angles of incidence of 60, 30, and zero degrees were used. In these tests breakwaters with side slopes of 1 on 1-1/4 and 1 on 2-1/2, constructed of 13.5-ton, 2.8-specific-gravity cap rock, were investigated. Prototype waves 10 seconds in period were simulated. The results of these tests were then compared with similar tests already conducted in which the angle of incidence was 90°.

69. The tests in which angles of incidence of 30° and 60° were used were performed by placing the breakwater section across the flume at the proper angle with approaching waves. A short wall, shaped to conform with the cross-sectional shape of the end of the breakwater, was used to support the end of the breakwater. A spending beach was placed along the flume wall opposite the test section to absorb the reflected waves (fig. 23, page 58). Tests in which the angle of incidence was zero were conducted with the model breakwater placed in the flume with the longitudinal center line of the breakwater coincident with the longitudinal center line of the flume. The shape of the seaward end (end toward wave machine) was a frustrum of a right circular cone, the slope of which was the same as that of the longitudinal breakwater section (fig. 24, page 58).

70. The results of these tests, compared with the results of tests

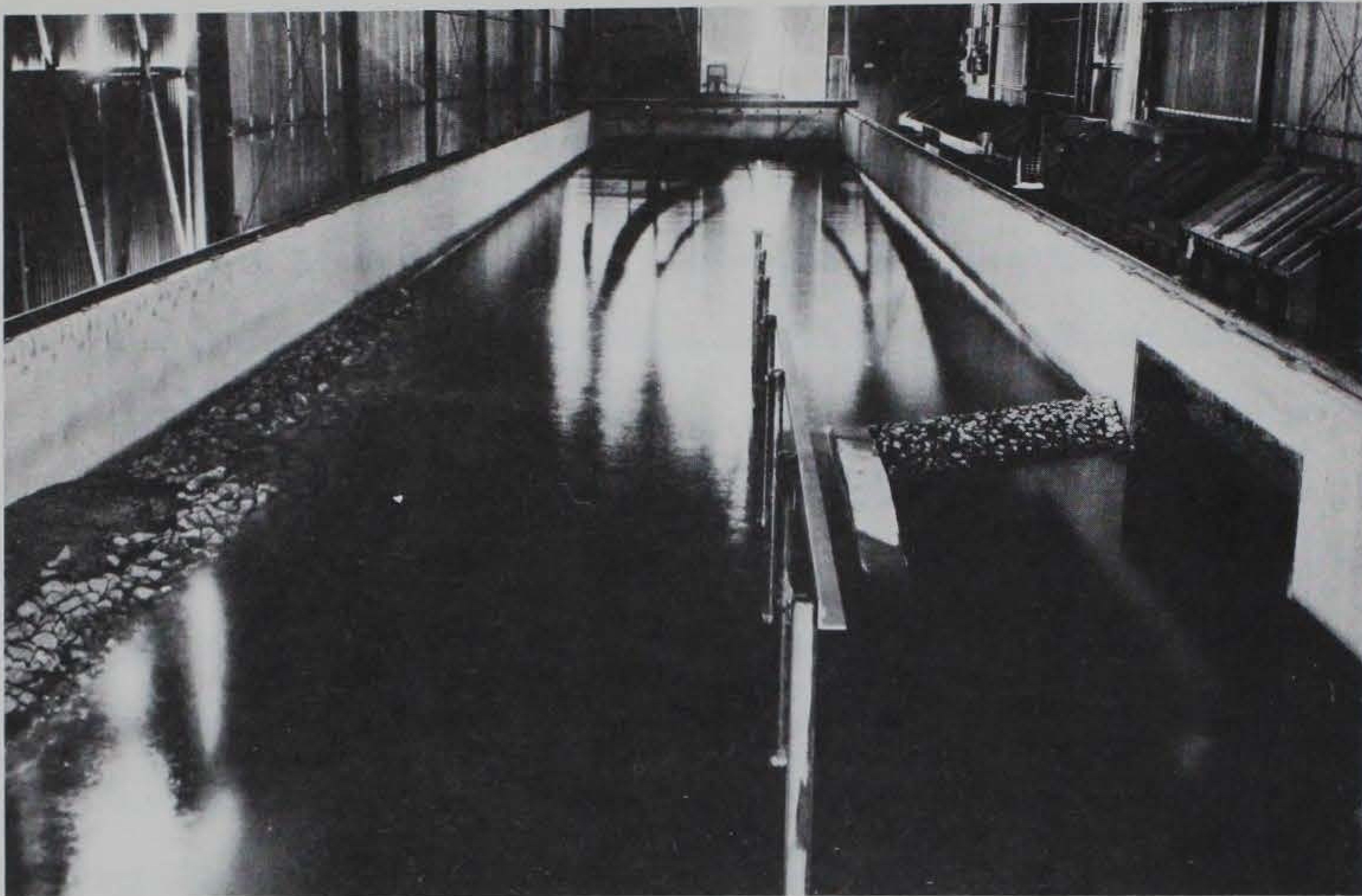


Fig. 23. Effects of angle of incidence, wave tank prepared for testing with waves at angles of 30 and 60 degrees

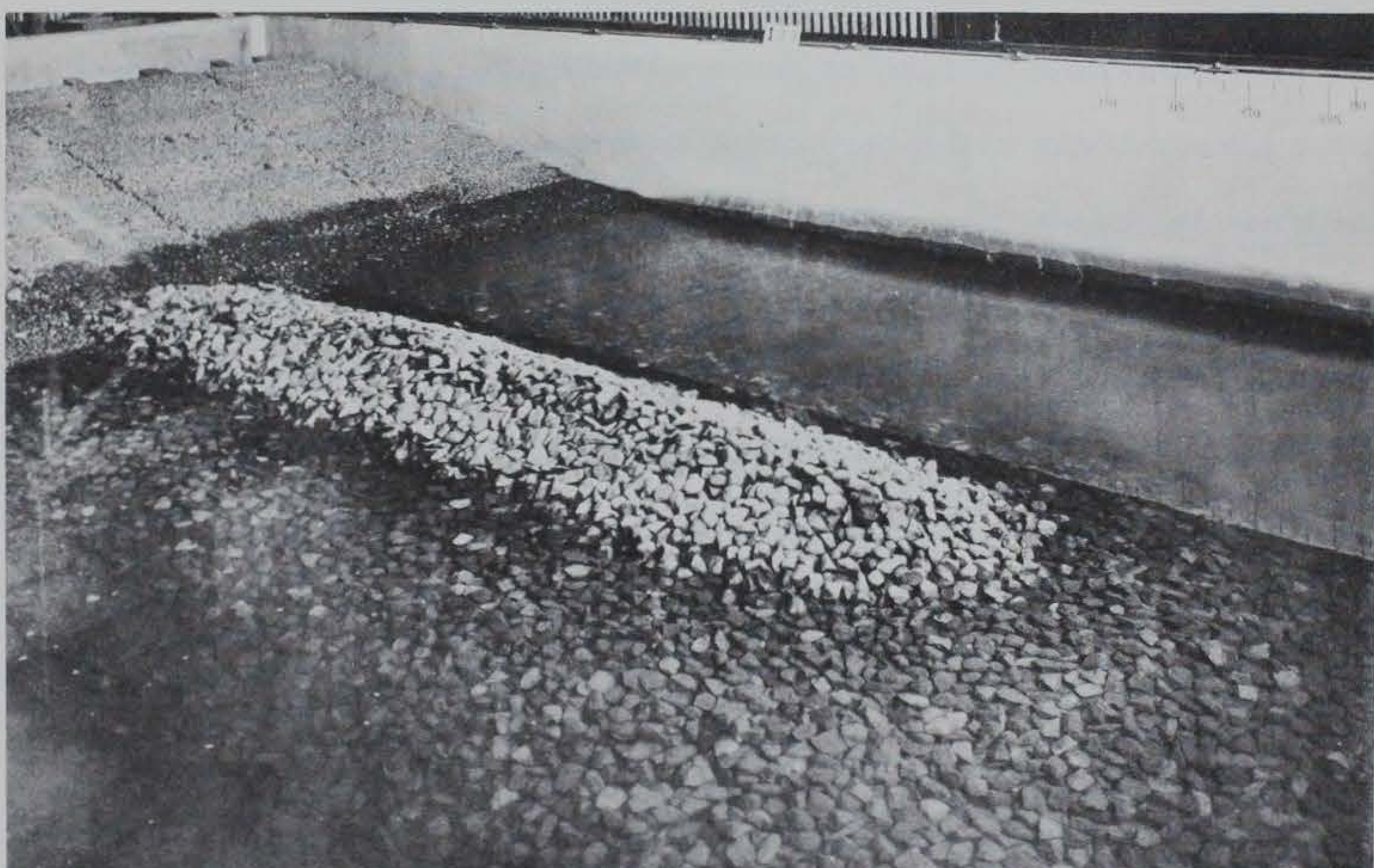


Fig. 24. Effects of angle of incidence, test section with angle of incidence of zero degrees

in which the angle of incidence was 90°, are summarized in the following table:

Side Slope	Angle of Incidence, Deg	Portion of Breakwater	Design-wave Height, Ft Model Data	Design-wave Height, Ft Calculated*
1 on 1-1/4	zero	Conical end	13	8
1 on 1-1/4	zero	Seaward side	15	8
1 on 1-1/4	30	Seaward side	11	8
1 on 1-1/4	60	Seaward side	13	8
1 on 1-1/4	90	Seaward side	13	8
1 on 2-1/2	zero	Conical end	18	23
1 on 2-1/2	zero	Seaward side	20	23
1 on 2-1/2	30	Seaward side	18	23
1 on 2-1/2	60	Seaward side	20	23
1 on 2-1/2	90	Seaward side	20	23

* Calculated using Iribarren's formula.

These data show that the stability of rubble breakwaters is not affected appreciably by variations in the angle of wave approach. The results indicate that the worst condition for the steeper slope (1 on 1-1/2) was the angle of 30 degrees. However, the tests were not sufficiently extensive to warrant attaching special significance to this isolated result.

Slight-damage vs No-damage Criterion in Selection of Design-wave Heights

71. Because of the assumptions upon which the Iribarren formula is based relative to the movement of cap rock from the face slope, the stability tests involved determination of the maximum size wave which would not displace cap rock from the face slope. The values of K' were calculated from the Iribarren formula using the design-wave heights determined on the basis of the no-damage criterion. It was noticed during the tests, however, that the selected design-wave heights could be increased considerably if slight damage to the face slope, but not sufficient to

decrease the efficacy of the structure in providing protection from wave action, were allowed. Descriptions of both the no-damage and slight-damage criteria are shown on plate 30.

72. Increases in design-wave height resulting from data based on the slight-damage criterion, compared with corresponding data using the no-damage criterion, are shown below.

Side Slope	Design-wave Height, Ft No-damage Criterion	Design-wave Height, Ft Slight-damage Criterion
1 on 1	10	16
1 on 1-1/4	11	16
1 on 1-1/2	13	22
1 on 2	17	23
1 on 3	24	28

The average increase in design-wave height is 6 ft. Cross sections on plate 39 show the slight amount of damage to a rubble breakwater resulting from the attack of waves whose heights were 5 ft greater than the selected design-wave height. If the criterion for slight damage were relaxed still further -- and considerably more damage could be tolerated in most instances -- the design-wave heights could be increased correspondingly. Fig. 25 shows the attack of and the damage due to a 28-ft wave, 10 ft

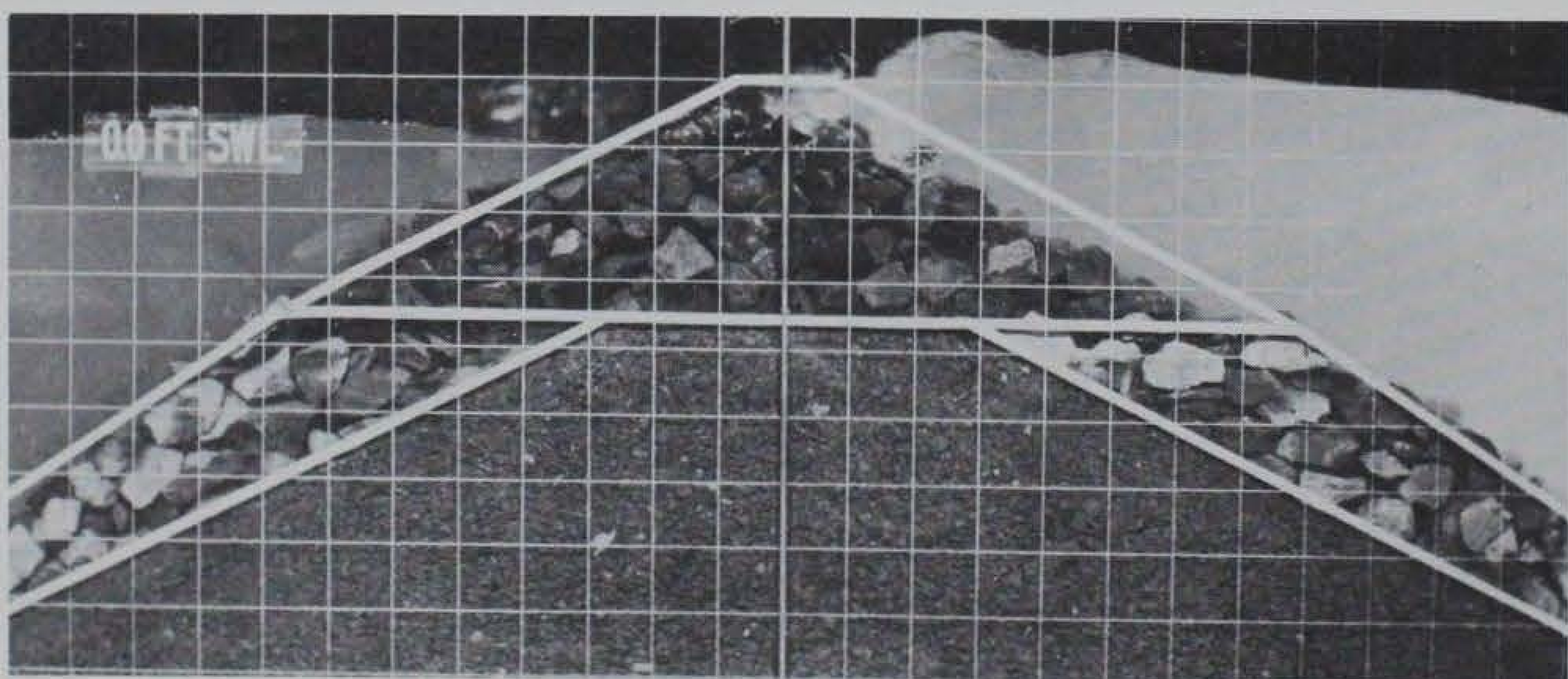


Fig. 25. Selection of design waves. Damage due to the attack of waves 10 ft greater than the selected design-wave height

greater than the selected design wave. The wave in this figure is 10 sec in period. The breakwater was constructed of 13.5-ton, 2.8-specific-gravity cap rock placed on a 1-on-2 slope.

Stability of Masonry-type Breakwaters

73. Sloping-face breakwaters made of concrete blocks, using masonry-type construction, could become feasible economically when quarry rock of suitable size, specific weight and durability are not available within reasonable shipping distance, and if depths of water are not prohibitive. Therefore, tests using molded concrete cubes of different weight, and placed on several different side slopes, were performed to determine the stability of this type structure compared with ordinary rubble-mound breakwaters. However, because of shortage of funds and the necessity of using the wave-flume facilities for other tests, it was not possible to complete the tests required to establish adequate design criteria for masonry-type breakwaters.

74. One test was performed on the breakwater section shown by fig. 26 (page 62). This section was constructed of 36.4-ton, 2.8-specific-gravity concrete cubes. The side slope was 1 on 1, the crown elevation was +10 ft swl, and the water depth was 90 ft. Test results showed that the largest wave which could be generated with the available wave-machine assembly (H_{max} = 30 ft, T = 10 sec) would not damage the breakwater. Fig. 26 shows this wave at the crest position on the breakwater. The results of this test demonstrated that the masonry-type breakwater could not be tested with the available equipment unless cap rock of smaller size and lower specific gravity were used. The results also indicated

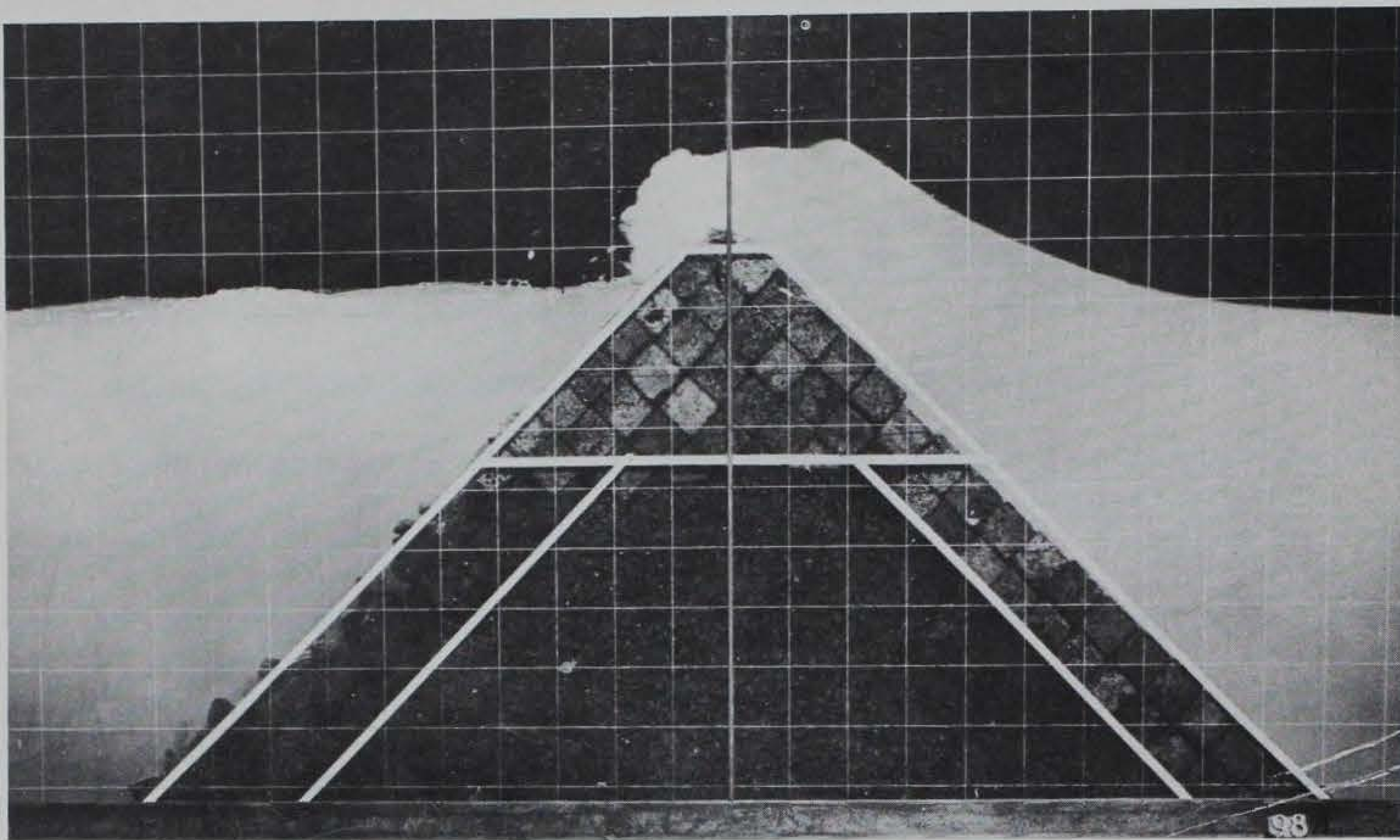


Fig. 26. Stability of masonry-type breakwaters, 10-sec waves, 30 ft high, attacking a masonry-type breakwater constructed of 36.4-ton concrete cubes, specific weight 2.8, placed on 1-on-1 slope

that Iribarren's formula cannot be used for designing masonry-type breakwaters.

75. The design-wave height, calculated by the Iribarren formula for a rubble-mound breakwater constructed of the same weight and size cap rock, and placed on a slope of 1 on 1, is only 8 ft in height. The ability of the sloping masonry-type breakwater to resist such large waves indicates that the manner by which waves dislocate cap rock from the face slope of these structures is not the same as that by which waves remove rock from the face slope of rubble-mound breakwaters. It is believed that the design of cap rock for breakwaters of the type under consideration could be based on the assumption that the critical forces involved are those due to a hydrostatic head on the lowest rock immediately above the trough positions of the attacking waves. However, the maximum

lifting force due to hydrostatic pressures would be a function of the wave height, wave period, and the rate of percolation through the interstices. Therefore, additional tests would be required to evaluate these functions.

Reduction of Wave Heights by Compressed Air

76. Accurate determination of K' in Iribarren's formula depended to a large extent upon the accuracy with which design waves could be selected and measured. Determination of the height of design waves was complicated by the larger-than-average waves resulting from the starting and stopping of the wave-machine plunger. Air bubbles were introduced into the water in the flume in an attempt to reduce the larger-size waves so that the waves attacking the breakwater test sections would be of more uniform size.

77. The blanket of air bubbles was 60 ft from the wave machine and 30 ft from the test section. The air bubbles were generated by an apparatus consisting of a system of 3/4-in. pipes placed across the wave-flume floor on 12-in. centers (see fig. 1, page 3). Compressed air, at 50-lb-per-sq-in. pressure, was forced through small holes in the pipe at a rate of 100 cu ft per minute. The holes were 0.0432 in. in diameter and spaced on 1/2-in. centers.

78. Tests were performed to determine the efficacy of the air-bubble blanket in the reduction of waves as they traveled through the area of air bubbles. It was found that this method of reducing the size of waves is not very effective. The test results showed, however, that the amount of reduction in wave heights, effected by what might be called

the compressed-air breakwater, is a function of the $\frac{d}{L}$ ratio of the waves. The $\frac{d}{L}$ ratio of the waves tested varied from 0.25 to 0.37. The compressed-air breakwater reduced wave heights as much as 25 per cent when the $\frac{d}{L}$ ratio was 0.37. No appreciable reduction was effected when the $\frac{d}{L}$ ratio was 0.25. It was noticed that the reduction of wave heights tended to increase when the $\frac{H}{L}$ ratio was increased; however, the range of wave heights used in these tests was not sufficient to warrant quantitative conclusions concerning the effects of this parameter on the reduction of wave heights by the compressed-air breakwater.

PART VII: CONCLUSIONS

79. The principal conclusions derived from the results of tests conducted to investigate the stability of rubble-mound breakwaters are as follows:

a. Model-prototype transference equations based upon the Froudian relationships are applicable to all important motion occurrences affecting the stability of rubble-mound breakwaters.

b. Hydraulic models can be relied upon to furnish accurate information concerning the effects of wave action on rubble breakwaters.

c. Protective cap rock should be placed on both the seaside and harborside slopes simultaneously with the dumping of light-weight core material to prevent the washing out of the core material by large waves during the construction period.

d. The stability of rubble breakwaters is improved if the structures are adequately repaired after damage from wave attack.

e. The Iribarren formula for design of rubble breakwaters can be made sufficiently accurate by using the formula in conjunction with curves of dimensionless coefficients such as those developed during this investigation. The coefficient curves ($K' = f(\alpha, d/L, H/L, \text{etc.})$) should be extended to cover a greater range of slopes, depths of water, wave periods, wave heights, and size and shape of rock. Additional tests are especially needed to determine coefficients for breakwaters of slopes flatter than 1 on 3 and d/L ratios less than 0.15.

f. The Epstein-Tyrrell formula for design of rubble breakwaters is essentially the same as that of Iribarren, except that Epstein and Tyrrell developed an additional formula for the coefficient. However, this latter formula was determined by model tests to be inadequate. Therefore, the Epstein-Tyrrell method of designing rubble breakwaters is not considered an improvement over the Iribarren method.

g. The reflection coefficients of rubble breakwaters determined during the tests performed in this investigation are not believed to be quantitatively accurate. However, the data show that the reflection and absorption characteristics of rubble breakwaters vary with the breakwater side slope, wave dimensions and depth of water.

h. The stability of rubble breakwaters is not affected appreciably by variations in the incident angle of wave attack.

i. If slight damage of a rubble breakwater can be allowed without reducing appreciably the effectiveness of the structure in performing its assigned function, the design-wave height can be increased as much as 6 ft in many instances.

j. Masonry-type breakwaters constructed of cut stone or molded block can be made much more resistant to storm-wave action than ordinary dumped rubble mounds. However, the cost of construction would be increased considerably and the stability of this type structure is endangered by settlement of the base and the consequent dislodging of key rock.

k. Compressed-air breakwaters were not effective in the reduction of wave heights within the range of wave dimensions used ($d/L = 0.25 - 0.37$).

TABLES

TABLE I

BREAKWATER MATERIALS USED IN SCALE EFFECT TESTS

CLASS MATERIAL	PROTOTYPE SPECIFICATIONS		MATERIAL USED FOR BREAKWATERS (LIMESTONE AND SAND SIZED AS SHOWN)											
			1:30 SCALE MODEL				1:45 SCALE MODEL				1:60 SCALE MODEL			
	PER CENT	WEIGHT OF INDIVIDUAL PIECES	WEIGHT (M) RANGE IN LBS.	DIA. OF EQUIV. SPHERE	SIEVE SIZE USED		WEIGHT (M) RANGE IN LBS.	DIA. OF EQUIV. SPHERE	SIEVE SIZE USED		WEIGHT (M) RANGE IN LBS.	DIA. OF EQUIV. SPHERE	SIEVE SIZE USED	
					PASSED	RETAINED			PASSED	RETAINED			PASSED	RETAINED
CLASS A	75	10 - 12 tons	0.741 0.888	2.439 " 2.590 "	Weighed	Weighed	0.219 0.263	1.623 " 1.725 "	Weighed	Weighed	0.0926 0.1111	1.22 " 1.295 "	1.50"	1.05"
	20	3 - 9 tons	0.222 0.667	1.632 " 2.355 "	Weighed	Weighed	0.0658 0.1975	1.089 " 1.570 "	1.50"	1.05"	0.0278 0.0833	0.816 " 1.178 "	1.05"	0.742"
	5	1 - 2 tons	0.0741 0.148	1.130 " 1.424 "	Weighed	Weighed	0.0219 0.0439	0.754 " 0.952 "	1.05"	0.742"	0.00926 0.0185	0.565 " 0.712 "	0.742"	0.525"
CLASS B	15	2 - 4 tons	0.148 0.296	1.424 " 1.795 "	2.00"	1.05"	0.0439 0.0878	0.952 " 1.193 "	1.50"	0.742"	0.0185 0.0370	0.712 " 0.898 "	1.05"	0.525"
	25	1 - 2 tons	0.0741 0.148	1.130 " 1.424 "	1.50"	1.05"	0.0219 0.0439	0.754 " 0.952 "	1.05"	0.742"	0.00926 0.0185	0.565 " 0.712 "	0.742"	0.525"
	5	1000 - 2000 lbs	0.0370 0.0741	0.898 " 1.130 "	1.50"	0.742"	0.0110 0.0219	0.598 " 0.754 "	0.742"	0.525"	0.00463 0.00926	0.449 " 0.565 "	0.525"	0.371"
	15	100 - 1000 lbs	0.00370 0.0370	0.416 " 0.898 "	1.05"	0.371"	0.00110 0.0110	0.278 " 0.598 "	0.525"	0.263"	0.000463 0.00463	0.208 " 0.449 "	0.525"	0.185"
	10	50 - 100 lbs	0.00185 0.00370	0.332 " 0.416 "	0.525"	0.263"	0.000548 0.00110	0.222 " 0.278 "	0.371"	0.185"	0.000231 0.000463	0.166 " 0.208 "	0.263"	0.131"
	5	20 - 50 lbs	0.000741 0.00185	0.244 " 0.332 "	0.371"	0.185"	0.000219 0.000548	0.163 " 0.222 "	0.263"	0.131"	0.0000926 0.000231	0.122 " 0.166 "	0.185"	0.093"
	5	10 - 20 lbs	0.000370 0.000741	0.193 " 0.244 "	0.263"	0.185"	0.000110 0.000219	0.129 " 0.163 "	0.185"	0.093"	0.0000463 0.0000926	0.0965" 0.122 "	0.131"	0.093"
	5	5 - 10 lbs	0.000185 0.000370	0.154 " 0.193 "	0.263"	0.131"	0.0000548 0.000110	0.103 " 0.129 "	0.131"	0.093"	0.0000231 0.0000463	0.0770" 0.0965"	0.131"	0.065"
	10	1 - 5 lbs	0.0000370 0.000185	0.0898" 0.154 "	0.131"	0.065"	0.0000110 0.0000548	0.0598" 0.103 "	0.131"	0.046"	0.00000463 0.0000231	0.0449" 0.0770 "	0.093"	0.0328
	5	Less than 1 lb	Less than 0.0000370	Less than 0.0898"	0.093"	0.065"	Less than 0.0000110	Less than 0.0598 "	0.065"	0.046"	Less than 0.00000463	Less than 0.0449 "	0.065"	0.0328"
CLASS C	20	.75 - 1.00 lbs	0.0000278 0.0000370	0.0816" 0.0898"	0.093"	0.065"	0.00000823 0.0000110	0.0545 " 0.0598 "	0.065"	0.046"	0.00000347 0.00000463	0.0408 " 0.0449 "	0.046"	0.0328"
	30	.50 - .75 lbs	0.0000185 0.0000278	0.0712" 0.0816"	0.093"	0.065"	0.00000548 0.00000823	0.0475 " 0.0545 "	0.065"	0.046"	0.00000231 0.00000347	0.0356 " 0.0408 "	0.046"	0.0328"
	30	.25 - .50 lbs	0.00000926 0.0000185	0.0565" 0.0712"	0.093"	0.046"	0.00000274 0.00000548	0.0377 " 0.0475 "	0.046"	0.0328"	0.00000116 0.00000231	0.0283 " 0.0356 "	0.046"	0.0232"
	20	Less than .25 lb	Less than 0.00000926	Less than 0.0565"	0.065"	0.046"	Less than 0.00000274	Less than 0.0377 "	0.0328"	0.0232"	Less than 0.00000116	Less than 0.0283 "	0.0328"	0.0232"

Table 2

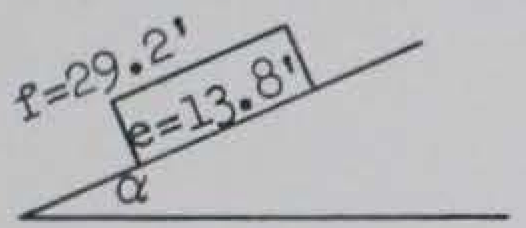
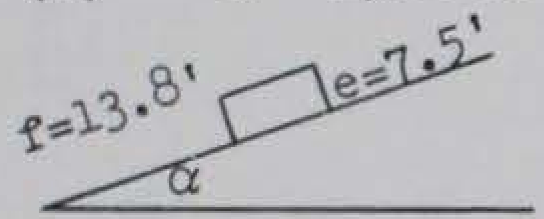
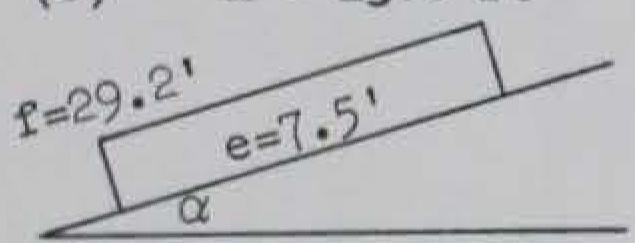
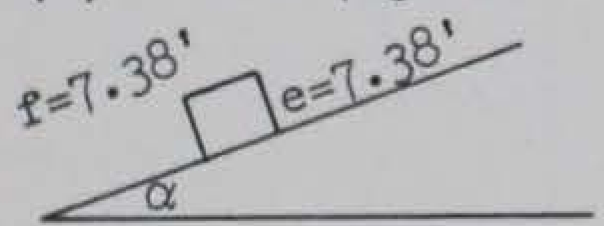
DURATION OF WAVE ATTACK

Test Series 1-4

Test Series	Type of Section	Top Elevation in Ft swl	Wave Height, Ft	Duration of Wave Attack, Model Hours
1	Class C stone	-49	7.5	2.5
		-49	10.5	5.5
		-49	15.0	5.5
		-49	21.0	5.5
	Class C stone	-38	7.5	5.5
		-38	10.5	5.5
		-38	15.0	2.0
		-38	21.0	2.0
	Class C stone	-29	7.5	2.5
		-29	10.5	2.5
		-29	15.0	4.7
		-29	21.0	5.0
	Class C stone	-24	7.5	3.5
		-24	10.5	3.3
		-24	15.0	4.7
		-24	21.0	6.0
2	Class C section with B toe on harborside	-38	7.5	No Test
		-38	10.5	No Test
		-38	15.0	4.0
		-38	21.0	4.0
	Class C section with B toe on harborside	-29	7.5	4.0
		-29	10.5	4.0
		-29	15.0	4.0
		-29	21.0	4.0
	Class C section with B toe on harborside	-24	7.5	4.0
		-24	10.5	4.0
		-24	15.0	4.0
		-24	21.0	4.0
	Class C section with B toe on seaside	-38	7.5	No Test
		-38	10.5	No Test
		-38	15.0	4.0
		-38	21.0	4.0
	Class C section with B toe on seaside	-29	7.5	4.0
		-29	10.5	4.0
		-29	15.0	4.0
		-29	21.0	4.0
	Class C section with B toe on seaside	-24	7.5	4.0
		-24	10.5	4.0
		-24	15.0	4.0
		-24	21.0	4.0
	Class C section with B toe on harborside and seaside	-38	7.5	3.1
		-38	10.5	4.0
		-38	15.0	4.33
		-38	21.0	4.00
	Class C section with B toe on harborside and seaside	-29	7.5	3.2
		-29	10.5	4.0
		-29	15.0	4.0
		-29	21.0	4.0
	Class C section with B toe on harborside and seaside	-24	7.5	4.0
		-24	10.5	4.0
		-24	15.0	4.0
		-24	21.0	4.0
3	Class B section complete	-10	7.5	3.0
		-10	10.5	3.5
		-10	15.0	4.16
		-10	21.0	4.25
4	Class A section complete, molded concrete blocks	+10	7.5	4.33
		+10	10.5	5.16
		+10	15.0	5.70
		+10	21.0	4.70
	Class A section complete, crushed limestone	+10	7.5	No Test
		+10	10.5	3.0
		+10	15.0	4.0
		+10	21.0	4.75

Table 3

SELECTION OF DESIGN-WAVE HEIGHTS FOR A SOLITARY BRICK AND A SOLITARY CONCRETE CUBE

		Side Slope 1 on 1-2/3			Side Slope 1 on 2			Side Slope 1 on 2-1/2		
Aspect of Test Block on Slope	Center of Gravity of Test Block, Ft, swl	Design-wave Height, H, Ft	Coef of Static Friction, μ	k	Design-wave Height, H, Ft	Coef of Static Friction, μ	k	Design-wave Height, H, Ft	Coef of Static Friction, μ	k
					239.6-ton, 2.25-specific-gravity Brick					
(a) x = 7.5 ft	0	9	0.78	0.73	10	0.77	1.04	20	0.64	0.54
	-15	8	0.73	0.74	17	0.77	0.74	17	0.66	0.85
	-30	10	0.69	0.50	33	0.75	0.42	27	0.70	0.69
(b) x = 29.2 ft	0	3	0.85	1.46	2	0.76	2.58	3	0.70	2.23
	-15	3-1/2	0.88	1.68	5	0.72	1.10	4	0.78	2.42
	-30	6	0.79	1.79	11	0.77	0.70	18	0.72	0.56
(c) x = 13.8 ft	0	9	0.87	0.61	9	0.75	0.67	15	0.75	0.59
	-15	11	0.86	0.57	15	0.74	0.48	15	0.74	0.71
	-30	20	0.84	0.38	18	0.79	0.57	27	0.69	0.44
					36.4-ton, 2.72-specific-gravity Concrete Cube					
(d) x = 7.38 ft	0	1-1/2	0.64	0.50	1-1/2	0.77	3.36	1-1/2	0.59	2.65
	-15	2	0.69	0.98	1	0.68	3.10	2-1/2	0.61	1.74
	-30	7	0.67	0.26	9	0.74	0.71	18	0.65	0.40

Note: The symbol x denotes the test block dimension parallel to the longitudinal axis of the model test section.

Table 4

REFLECTION OF WAVES FROM RUBBLE-MOUND BREAKWATERS

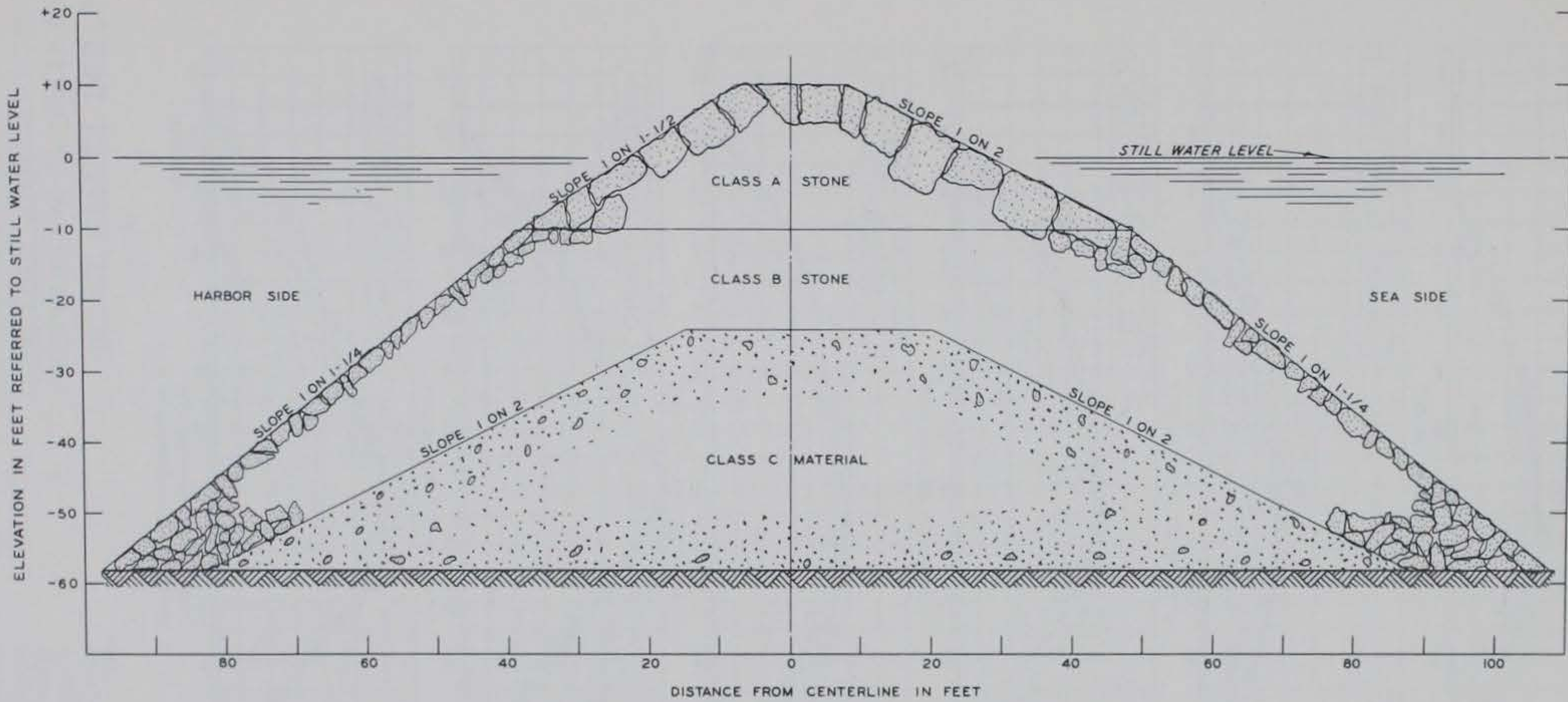
Side Slope	Crown Elevation, Ft, swl	Incident Waves Height, Ft	Incident Waves Period, Sec	Incident Waves Length, Ft	$\frac{H_r}{H_i}$	$\frac{Hd}{L^2}$
		4.6-ton Cap Rock (Basalt)				
1 on 1-1/4	+10	9	10	440	0.28	0.0042
1 on 1-1/4	+10	11	10	440	0.28	0.0051
1 on 1-1/2	+14	11	10	440	0.22	0.0051
1 on 1-1/2	+14	14	10	440	0.24	0.0065
1 on 1-2/3	+18	11	10	440	0.25	0.0051
1 on 1-2/3	+18	12	10	440	0.22	0.0056
1 on 1-2/3	+18	16	10	440	0.25	0.0075
		13.5-ton Cap Rock (Basalt)				
1 on 1	+10	7	7	245	0.30	0.0105
1 on 1	+10	7	10	440	0.33	0.0033
1 on 1	+10	9	10	440	0.39	0.0042
1 on 1	+10	6	12	565	0.37	0.0017
1 on 1	+10	8	12	565	0.39	0.0023
1 on 1-1/4	+13	11	10	440	0.35	0.0051
1 on 1-1/4	+13	12	10	440	0.35	0.0056
1 on 1-1/2	+13	12	7	245	0.18	0.0180
1 on 1-1/2	+13	15	7	245	0.20	0.0225
1 on 1-1/2	+13	12	10	440	0.23	0.0056
1 on 1-1/2	+13	14	10	440	0.25	0.0065
1 on 1-1/2	+13	12	12	565	0.32	0.0034
1 on 1-1/2	+13	13	12	565	0.24	0.0037
1 on 1-2/3	+15	13	10	440	0.20	0.0061
1 on 1-2/3	+15	15	10	440	0.19	0.0069
1 on 1-2/3	+15	17	10	440	0.24	0.0079
1 on 2	+20	15	7	245	0.14	0.0225
1 on 2	+20	18	7	245	0.13	0.0270
1 on 2	+20	17	10	440	0.25	0.0079
1 on 2	+20	18	10	440	0.21	0.0084
1 on 2	+20	19	12	565	0.19	0.0054

(Continued)

Table 4 (Continued)

Side Slope	Crown Elevation, Ft, swl	Incident Waves Height, Ft	Period, Sec	Length, Ft	$\frac{H_r}{H_i}$	$\frac{Hd}{L^2}$
1 on 2-1/2	+22	21	8	311	0.10	0.0196
1 on 2-1/2	+22	18	10	440	0.12	0.0084
1 on 2-1/2	+22	22	10	440	0.12	0.0102
1 on 2-1/2	+22	18	12	565	0.17	0.0051
1 on 2-1/2	+22	20	12	565	0.22	0.0057
1 on 3	+25	20	8	311	0.04	0.0186
1 on 3	+25	21	8	311	0.06	0.0196
1 on 3	+25	18	10	440	0.06	0.0084
1 on 3	+25	22	10	440	0.08	0.0102
1 on 3	+25	18	12	565	0.16	0.0051
1 on 3	+25	20	12	565	0.14	0.0057
		27.4-ton Cap Rock (Basalt)				
1 on 1-1/4	+15	14	10	440	0.19	0.0065
1 on 1-1/4	+15	16	10	440	0.21	0.0075
1 on 1-1/2	+18	16	10	440	0.22	0.0075
1 on 1-1/2	+18	18	10	440	0.24	0.0084
1 on 1-2/3	+20	20	10	440	0.22	0.0093
1 on 1-2/3	+20	24	10	440	0.24	0.0111
		36.4-ton Cap Rock (Concrete Cubes)				
1 on 1	+22	16	10	440	0.28	0.0075
1 on 1	+22	18	10	440	0.23	0.0084
1 on 1-1/4	+24	20	10	440	0.24	0.0093
1 on 1-1/4	+24	24	10	440	0.24	0.0111
1 on 1-1/2	+30	25	10	440	0.22	0.0116
1 on 1-1/2	+30	26	10	440	0.22	0.0121

PLATES



CROSS SECTION AT 58 FEET WATER DEPTH

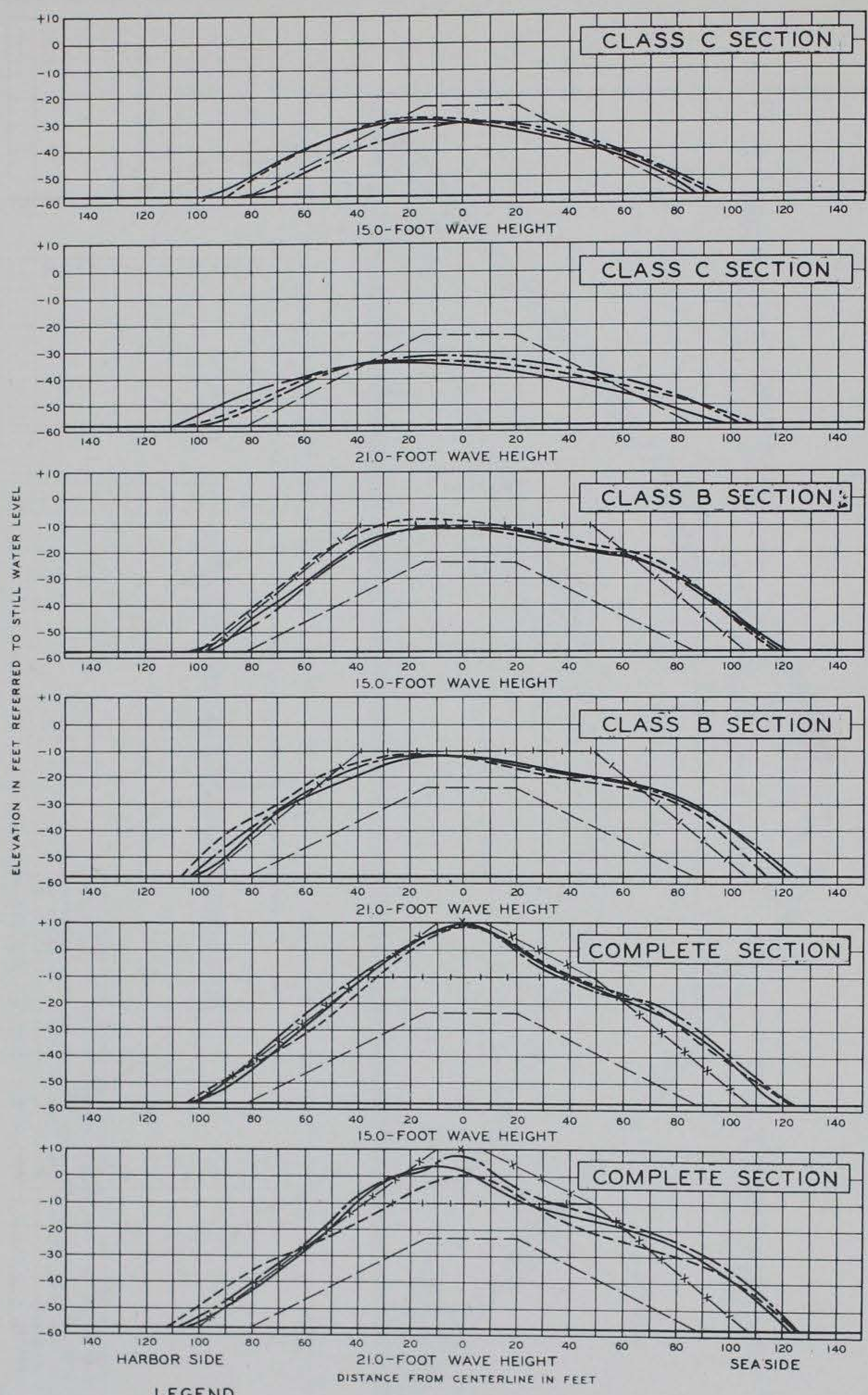
NOTE: CLASS A STONE IS SELECTED FROM QUARRY. NO PIECES LESS THAN 1 TON AND AT LEAST 75 PER CENT (BY WEIGHT) WEIGHING 10 TONS OR MORE EACH.

CLASS B STONE IS QUARRY RUN. NOT MORE THAN 25 PER CENT BY WEIGHT IN PIECES LESS THAN 20 LB, AND NOT LESS THAN 40 PER CENT IN PIECES OF 1 TON OR MORE EACH.

CLASS C MATERIAL IS A RESIDUUM FROM QUARRY OPERATIONS OR A DREDGED MATERIAL.

CHARACTERISTICS OF PROTOTYPE BREAKWATER

PLATE 1



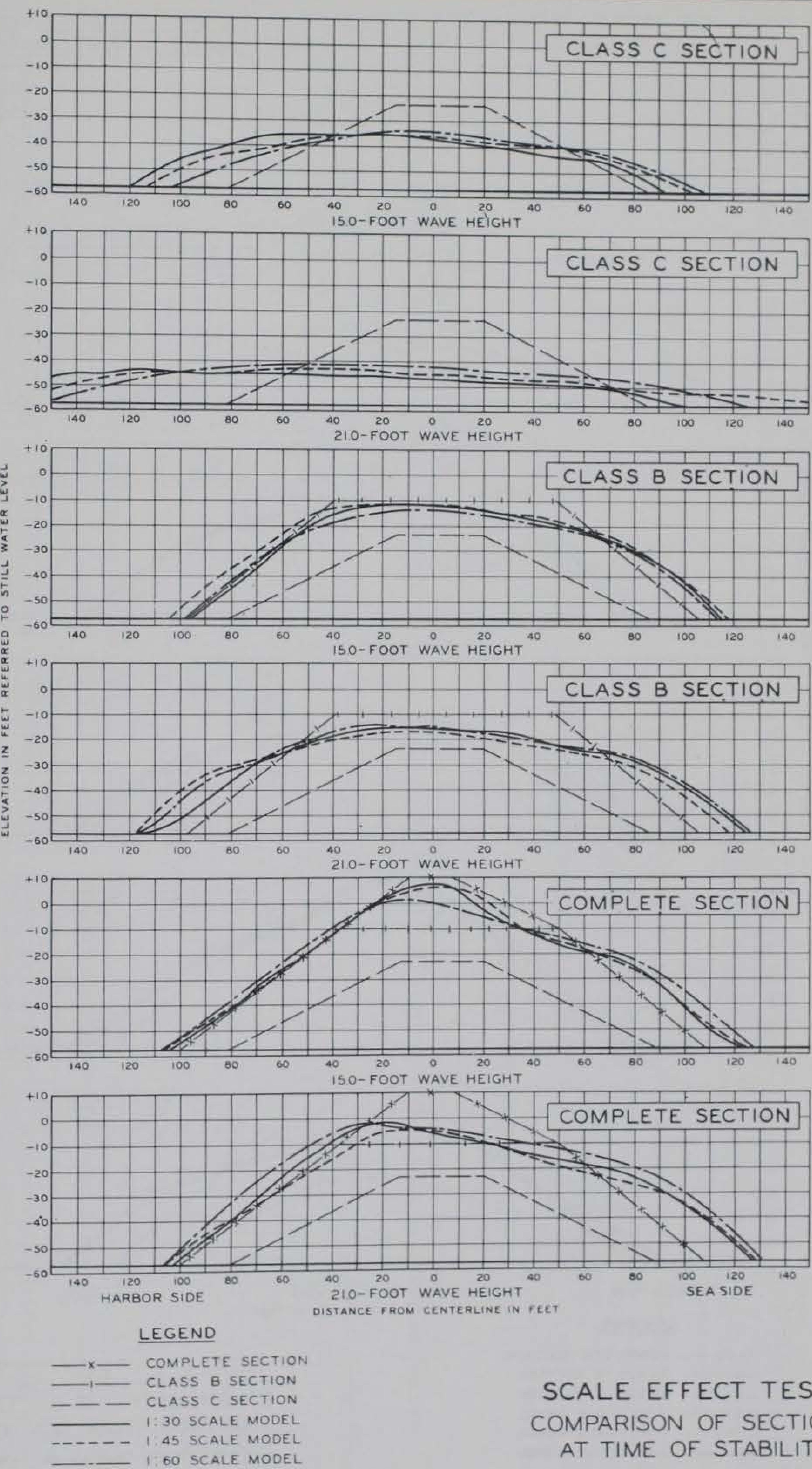
LEGEND

- —x— COMPLETE SECTION
- —I— CLASS B SECTION
- — — — CLASS C SECTION
- ——— 1:30 SCALE MODEL
- - - - - 1:45 SCALE MODEL
- —·— 1:60 SCALE MODEL

SCALE EFFECT TESTS

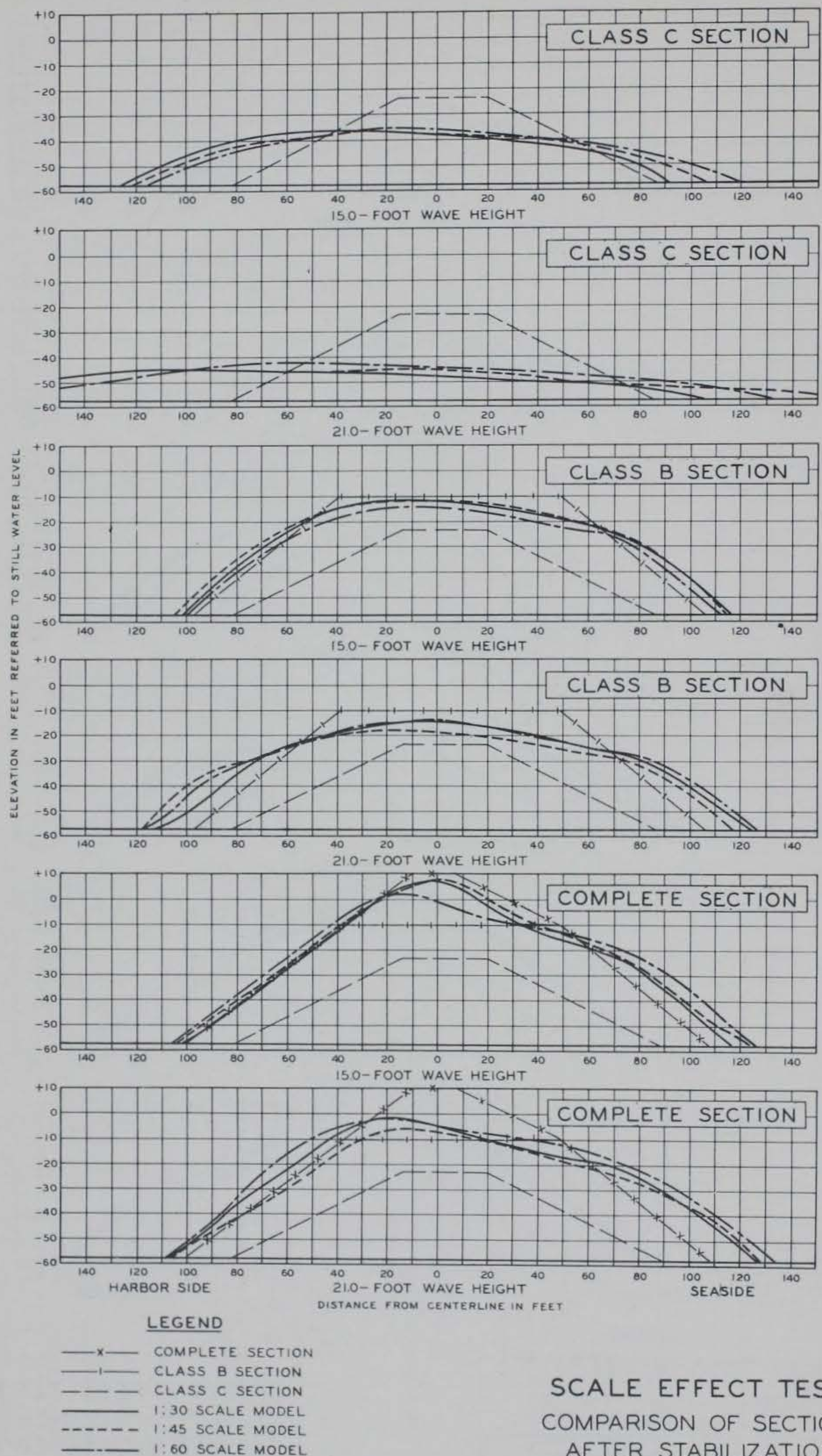
COMPARISON OF SECTIONS BEFORE STABILIZATION

(PROTOTYPE TIME-3 1/4 HRS)



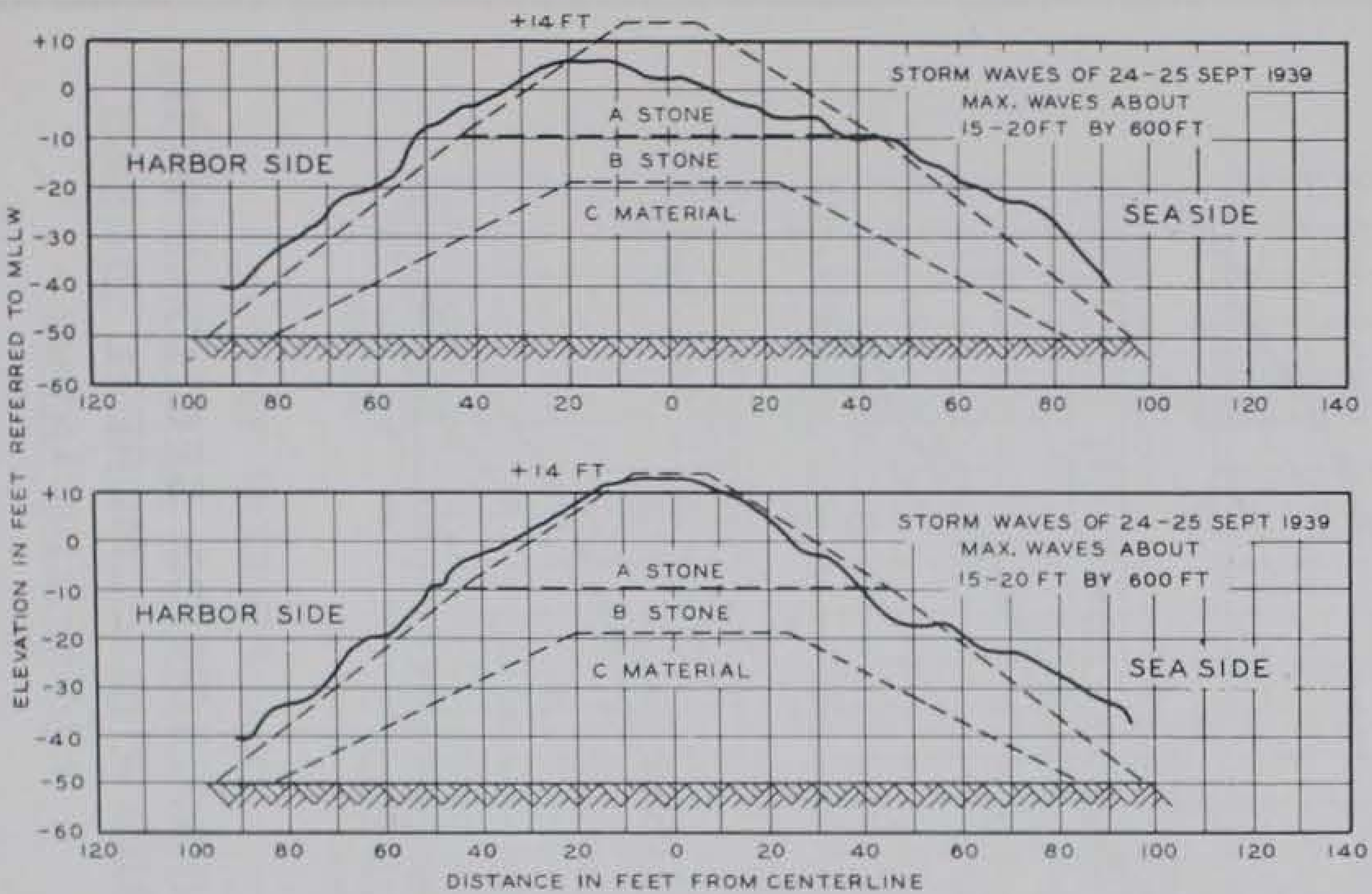
SCALE EFFECT TESTS

COMPARISON OF SECTIONS AT TIME OF STABILITY

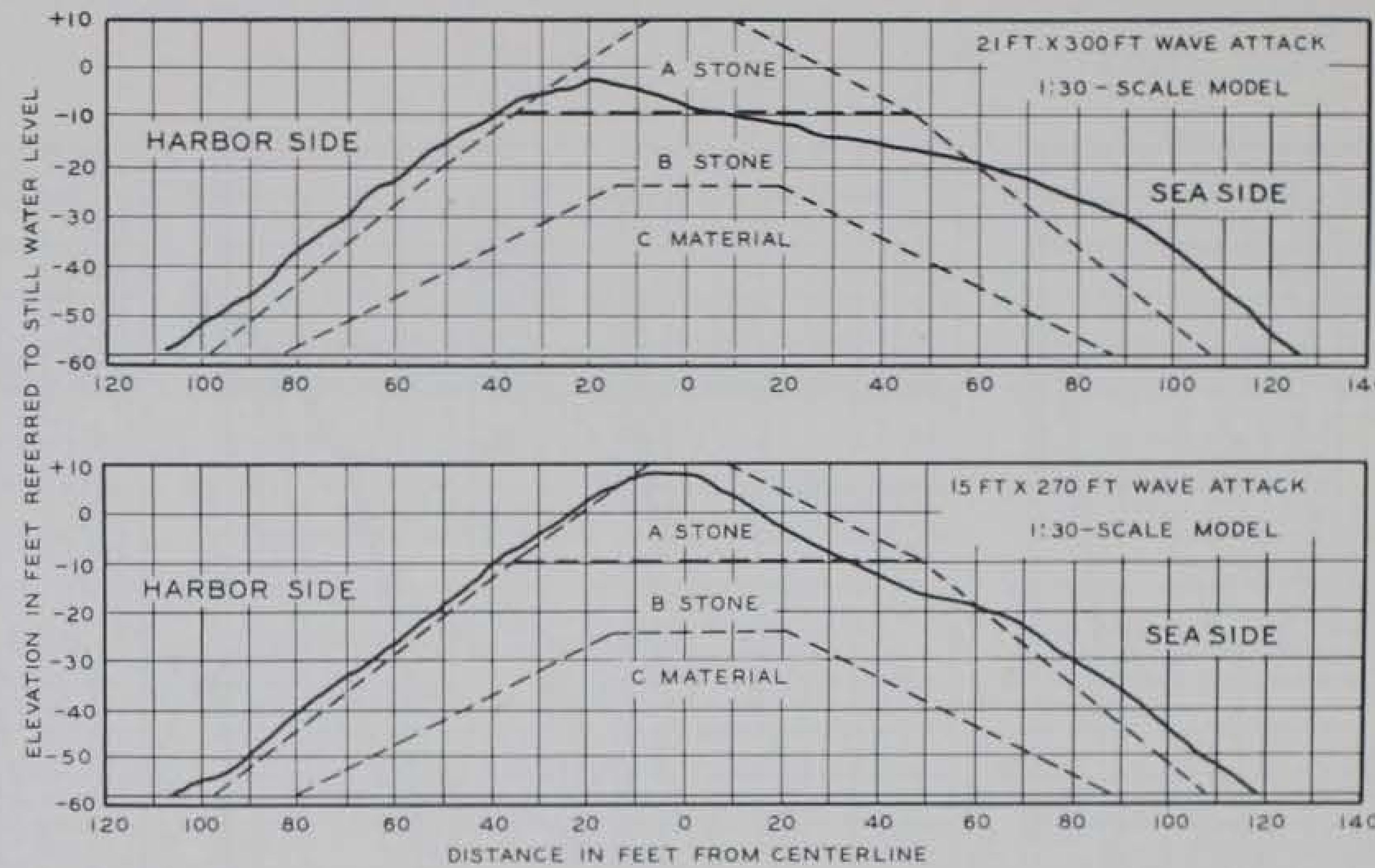


SCALE EFFECT TESTS

COMPARISON OF SECTIONS AFTER STABILIZATION



MAXIMUM AND MINIMUM STORM DAMAGE TO DETACHED BREAKWATER, SAN PEDRO BAY, CALIFORNIA



DAMAGE TO MODEL BREAKWATER WITH WAVE ATTACK UNTIL MOVEMENT CEASED

SPECIFICATIONS OF BREAKWATER MATERIALS

SAN PEDRO BREAKWATER

CLASS A STONE. QUARRY RUN, WEIGHING NOT LESS THAN 160 LBS. PER CU. FT. (SOLID DRY), IN PIECES OF NOT LESS THAN ONE TON, OF WHICH NOT LESS THAN 50 PER CENT BY WEIGHT TO BE IN PIECES OF TEN TONS OR MORE EACH.

CLASS B STONE. QUARRY RUN, WEIGHING NOT LESS THAN 120 LBS. PER. CU. FT. (SOLID DRY), OF WHICH NOT MORE THAN 25 PER CENT BY WEIGHT TO BE IN PIECES OF LESS THAN 20 LBS. EACH, AND AT LEAST 40 PER CENT BY WEIGHT TO BE IN PIECES OF ONE TON OR MORE EACH.

CLASS C MATERIAL. A MOUND OF DREDGED SAND SURMOUNTED BY A MOUND OF DREDGED CLAY.

MODEL BREAKWATER

CLASS A STONE. SELECTED STONE FROM QUARRY (STONE FROM QUARRY UNDER CONSIDERATION WEIGHS 165-170 LBS. PER CU. FT.). NO PIECE TO WEIGH LESS THAN ONE TON, AND AT LEAST 75 PER CENT BY WEIGHT TO BE IN PIECES WEIGHING TEN TONS OR MORE EACH.

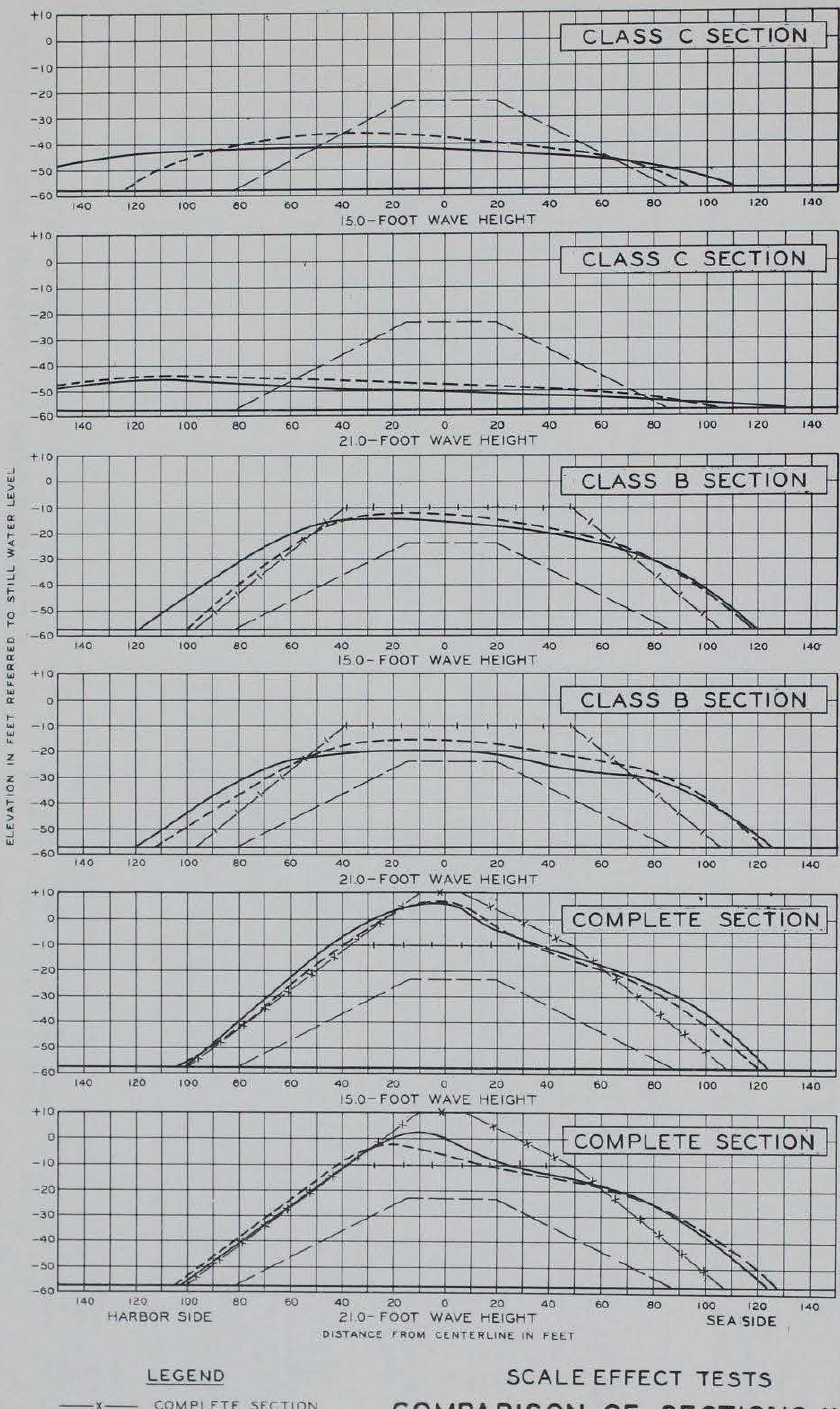
CLASS B STONE. QUARRY RUN, OF WHICH NOT MORE THAN 25 PER CENT BY WEIGHT TO BE IN PIECES OF LESS THAN 20 LBS., AND NOT LESS THAN 40 PER CENT BY WEIGHT TO BE IN PIECES OF ONE TON OR MORE EACH.

CLASS C MATERIAL. ALL RESIDUAL MATERIAL FROM QUARRY OPERATIONS, SIMILAR MATERIAL OBTAINED IN THE VICINITY OF THE QUARRY, OR DREDGED MATERIAL.

SCALE EFFECT TESTS

COMPARISON OF KNOWN BREAKWATER DAMAGE WITH RESULTS OF MODEL STABILITY TESTS

PLATE 5



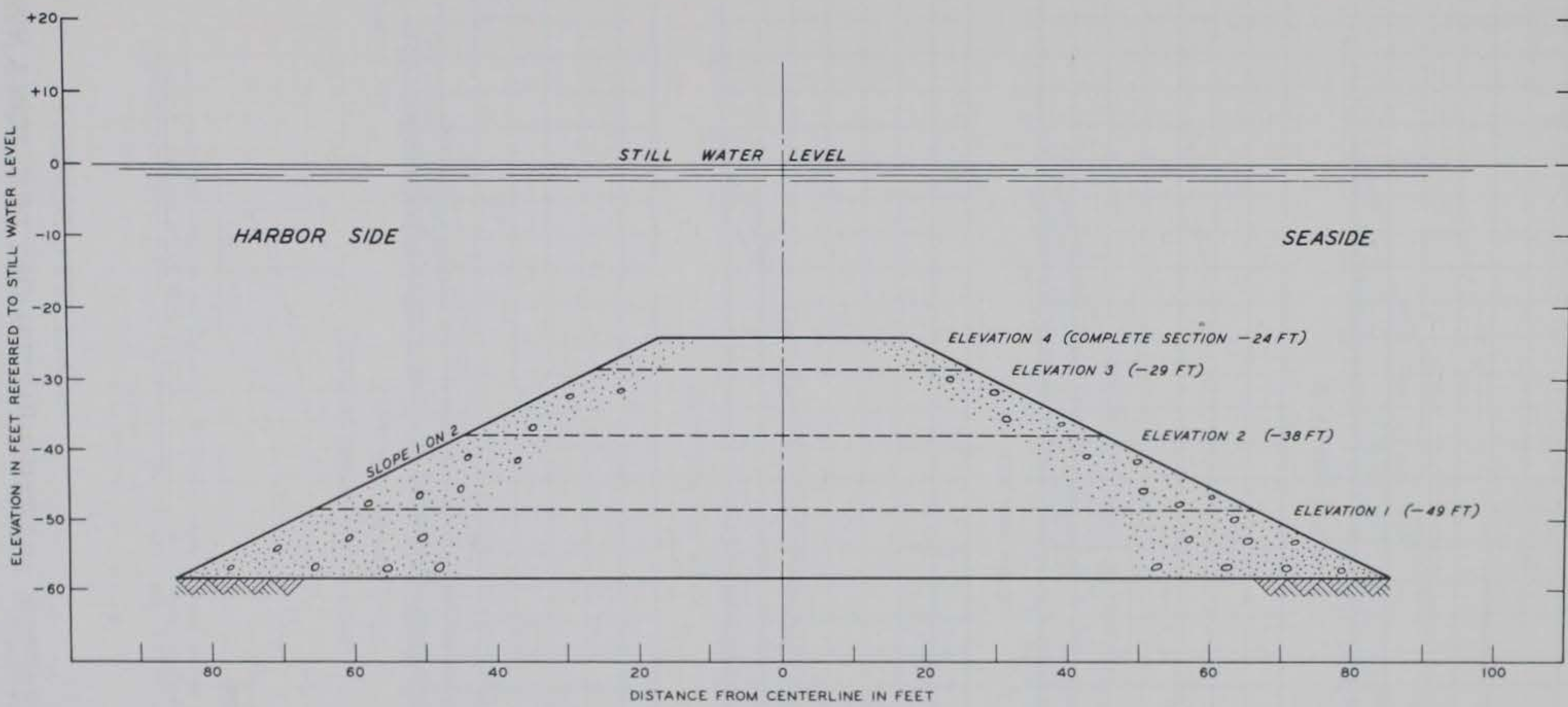
LEGEND

—x— COMPLETE SECTION
—I— CLASS B SECTION
— — — CLASS C SECTION
——— CONCRETE BASE
- - - - - SAND BASE

SCALE EFFECT TESTS

COMPARISON OF SECTIONS WITH SAND AND CONCRETE BASES

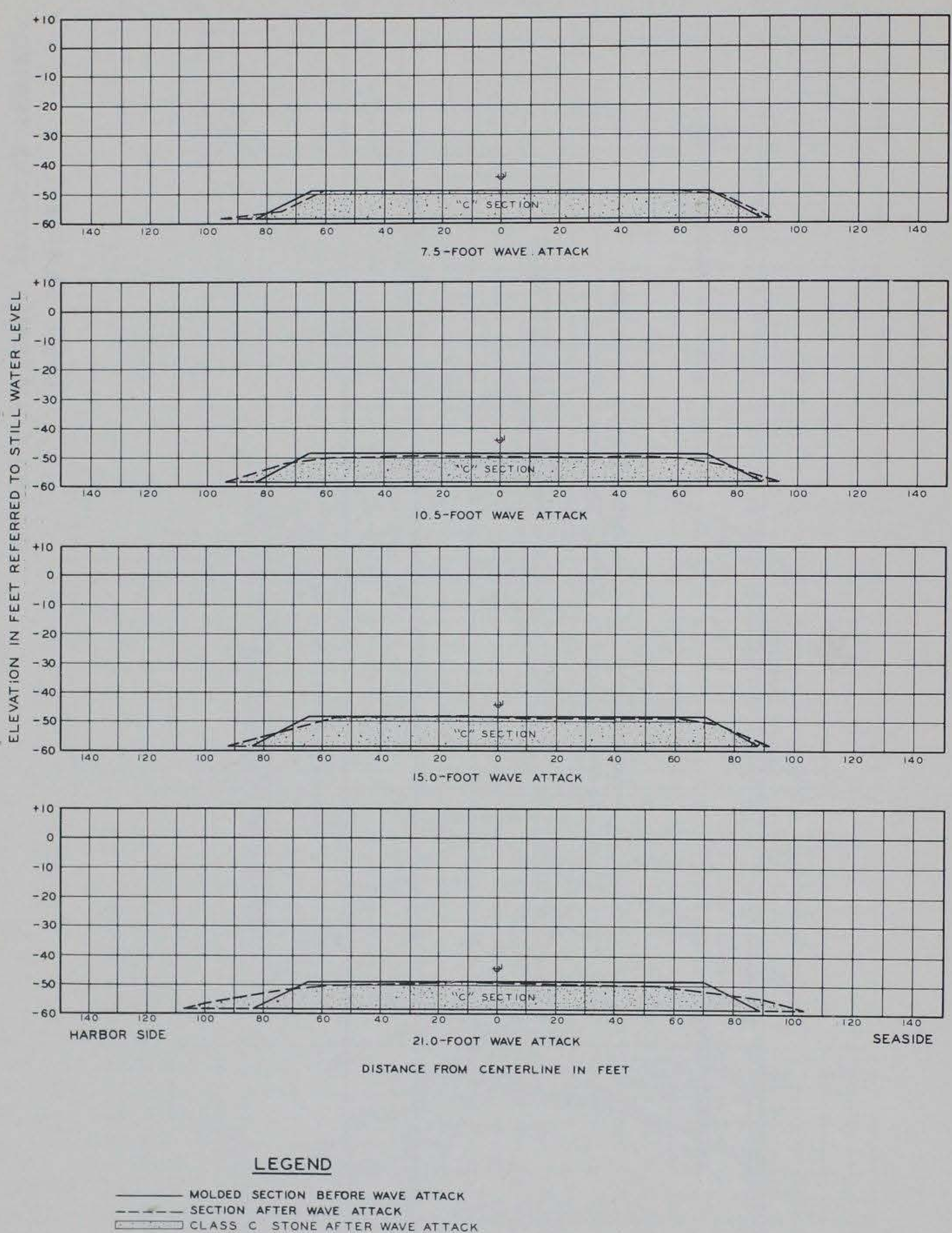
1:30 SCALE MODEL



CROSS SECTION AT 58 FEET WATER DEPTH

TEST SERIES I

CLASS C MATERIAL SECTIONS TESTED

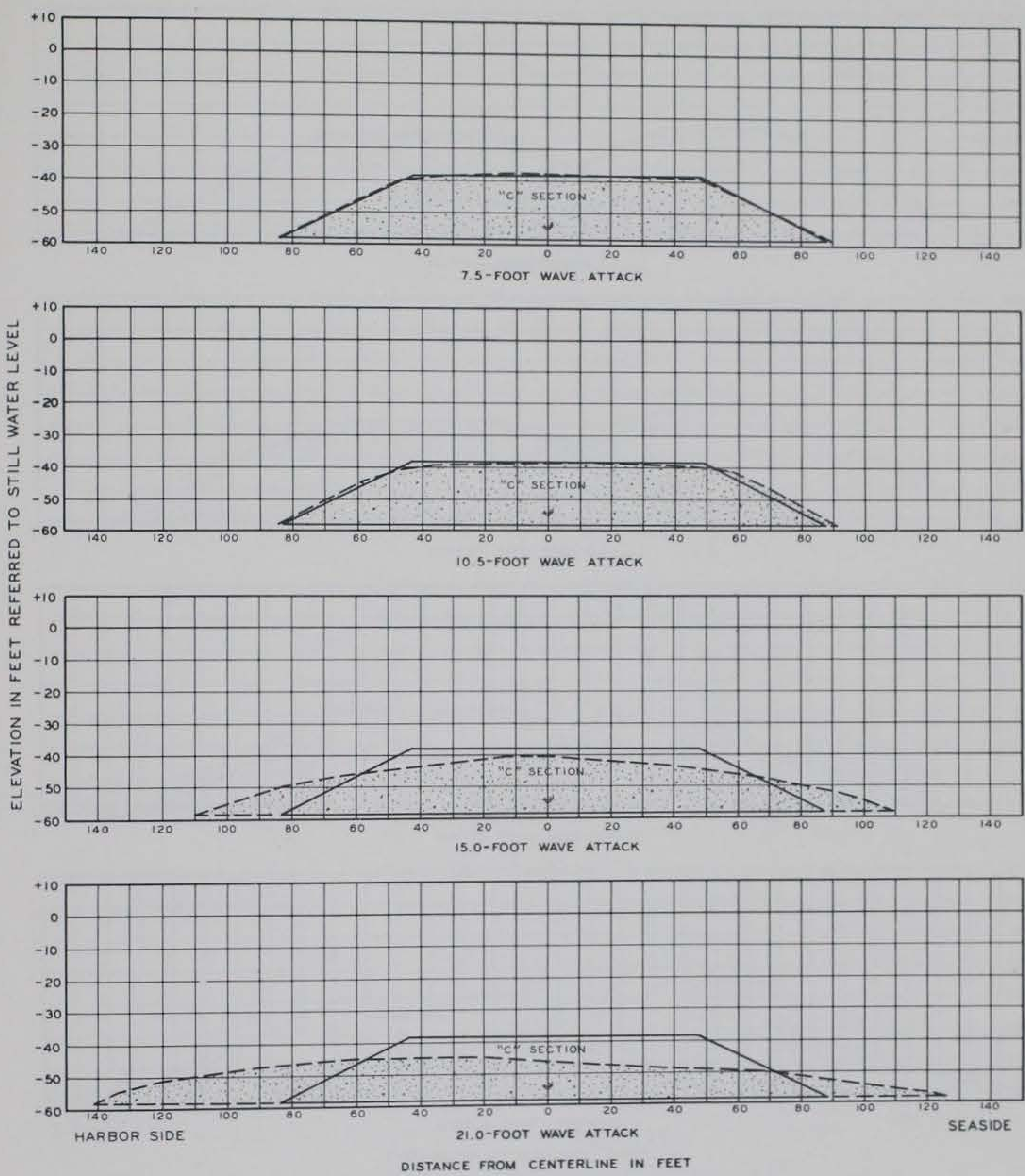


NOTE: SCALE RATIO = 1 TO 30 MODEL-TO-PROTOTYPE
ALL SECTIONS SUBJECTED TO WAVE ATTACK
UNTILL DISPLACEMENT OF BREAKWATER
MATERIAL CEASED
DEPTH OF WATER = 58 FEET

TEST SERIES I

DISPLACEMENT OF BREAKWATER MATERIAL BY WAVE ACTION

CLASS C MATERIAL
TOP ELEVATION-49 FT SWL



LEGEND

———— MOLDED SECTION BEFORE WAVE ATTACK
– – – – – SECTION AFTER WAVE ATTACK
CLASS C STONE AFTER WAVE ATTACK

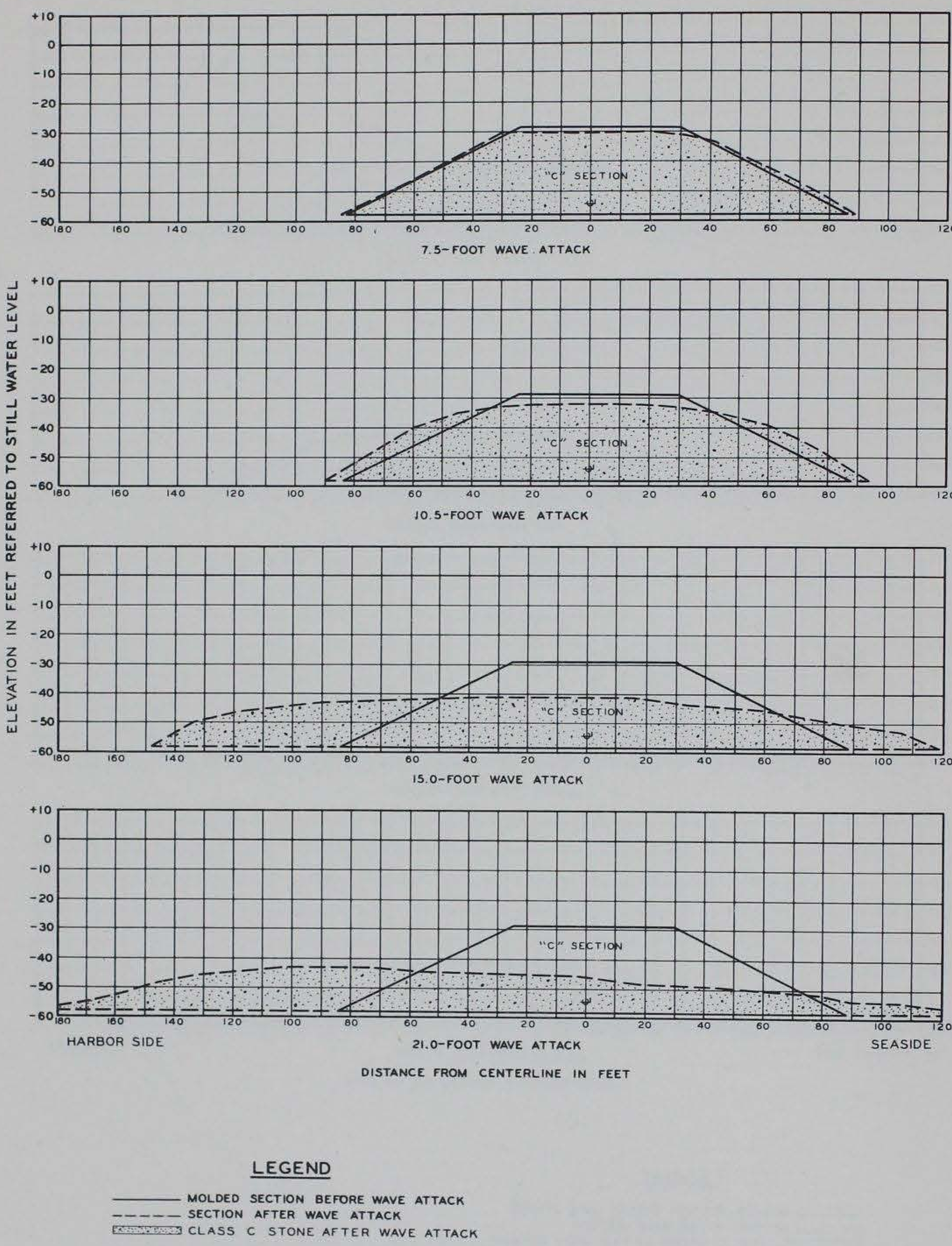
NOTE: SCALE RATIO = 1 TO 30 MODEL-TO-PROTOTYPE
ALL SECTIONS SUBJECTED TO WAVE ATTACK
UNTILL DISPLACEMENT OF BREAKWATER
MATERIAL CEASED
DEPTH OF WATER = 58 FEET

TEST SERIES I

DISPLACEMENT OF BREAKWATER MATERIAL BY WAVE ACTION

CLASS C MATERIAL

TOP ELEVATION -38 FT SWL



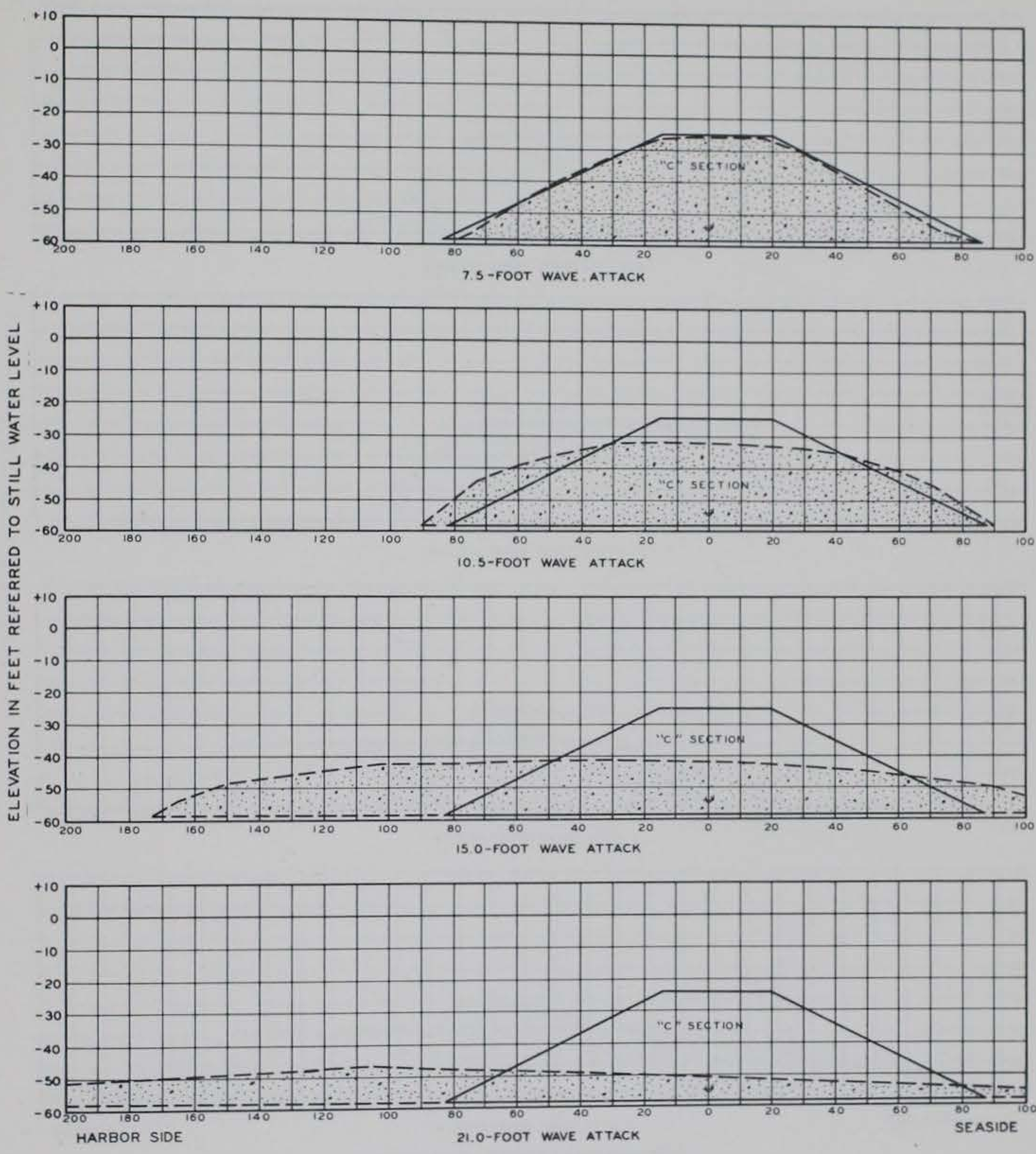
NOTE: SCALE RATIO = 1 TO 30 MODEL-TO-PROTOTYPE
ALL SECTIONS SUBJECTED TO WAVE ATTACK
UNTILL DISPLACEMENT OF BREAKWATER
MATERIAL CEASED
DEPTH OF WATER = 58 FEET

TEST SERIES I

DISPLACEMENT OF BREAKWATER MATERIAL BY WAVE ACTION

CLASS C MATERIAL

TOP ELEVATION -29 FT SWL



LEGEND

MOLDED SECTION BEFORE WAVE ATTACK
SECTION AFTER WAVE ATTACK
CLASS C STONE AFTER WAVE ATTACK

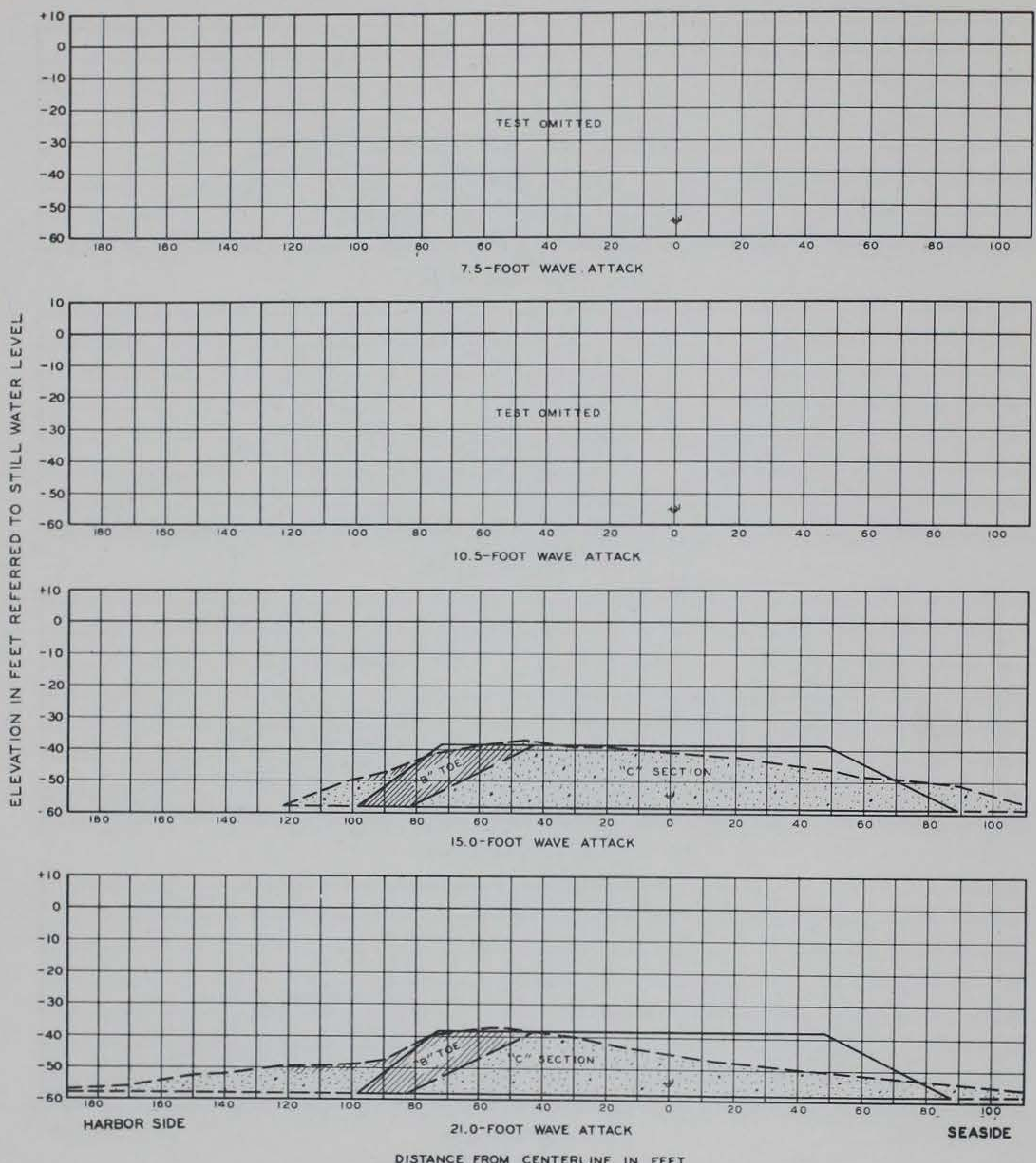
NOTE: SCALE RATIO = 1 TO 30 MODEL-TO-PROTOTYPE
ALL SECTIONS SUBJECTED TO WAVE ATTACK UNTILL DISPLACEMENT OF BREAKWATER MATERIAL CEASED
DEPTH OF WATER = 58 FEET

TEST SERIES I

DISPLACEMENT OF BREAKWATER MATERIAL BY WAVE ACTION

CLASS C MATERIAL

TOP ELEVATION -24 FT SWL



LEGEND

MOLDED SECTION BEFORE WAVE ATTACK
SECTION AFTER WAVE ATTACK
CLASS B STONE AFTER WAVE ATTACK
CLASS C STONE AFTER WAVE ATTACK

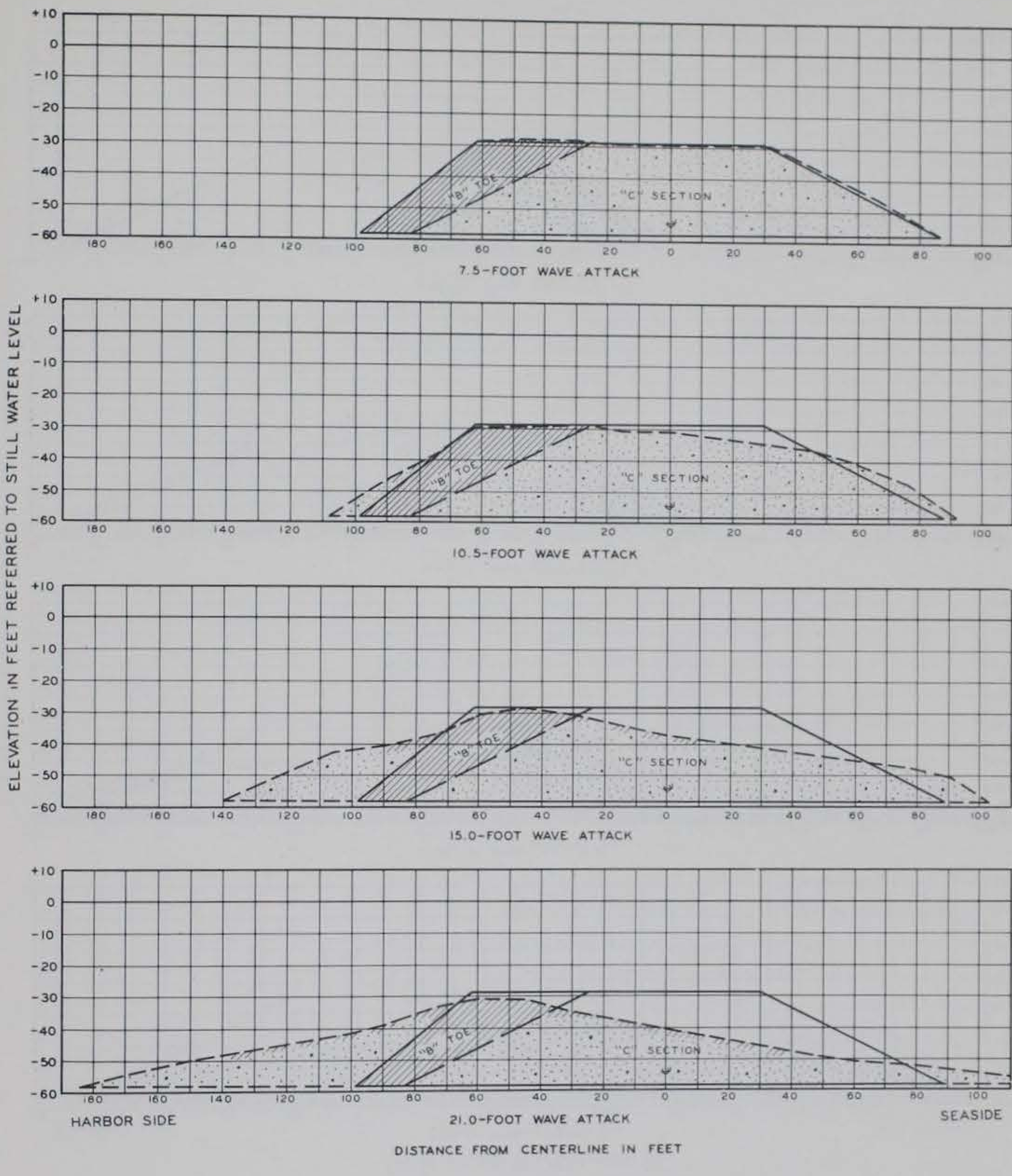
NOTE: SCALE RATIO = 1 TO 30 MODEL-TO-PROTOTYPE
ALL SECTIONS SUBJECTED TO WAVE ATTACK
UNTILL DISPLACEMENT OF BREAKWATER
MATERIAL CEASED
DEPTH OF WATER = 58 FEET

TEST SERIES 2

DISPLACEMENT OF BREAKWATER MATERIAL BY WAVE ACTION

CLASS C MATERIAL; B-MATERIAL TOE ON HARBOR SIDE

TOP ELEVATION -38 FT SWL



LEGEND

MOLDED SECTION BEFORE WAVE ATTACK
SECTION AFTER WAVE ATTACK
CLASS B STONE AFTER WAVE ATTACK
CLASS C STONE AFTER WAVE ATTACK

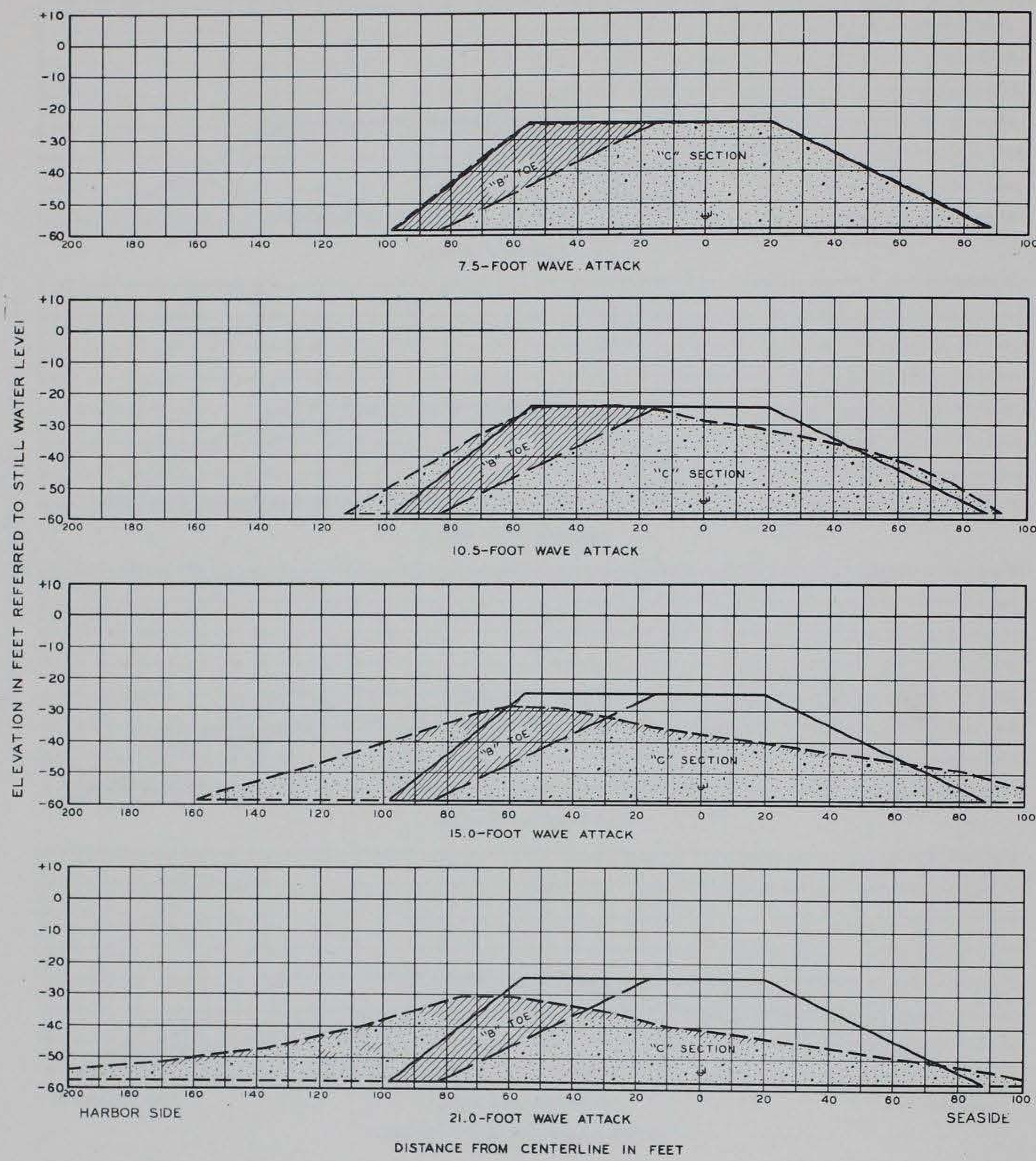
NOTE: SCALE RATIO = 1 TO 30 MODEL-TO-PROTOTYPE
ALL SECTIONS SUBJECTED TO WAVE ATTACK
UNTILL DISPLACEMENT OF BREAKWATER
MATERIAL CEASED
DEPTH OF WATER = 58 FEET

TEST SERIES 2

DISPLACEMENT OF BREAKWATER MATERIAL BY WAVE ACTION

CLASS C MATERIAL; B-MATERIAL TOE ON HARBOR SIDE

TOP ELEVATION -29 FT SWL



LEGEND

- ——— MOLDED SECTION BEFORE WAVE ATTACK
- – – – – SECTION AFTER WAVE ATTACK
- CLASS B STONE AFTER WAVE ATTACK
- CLASS C STONE AFTER WAVE ATTACK

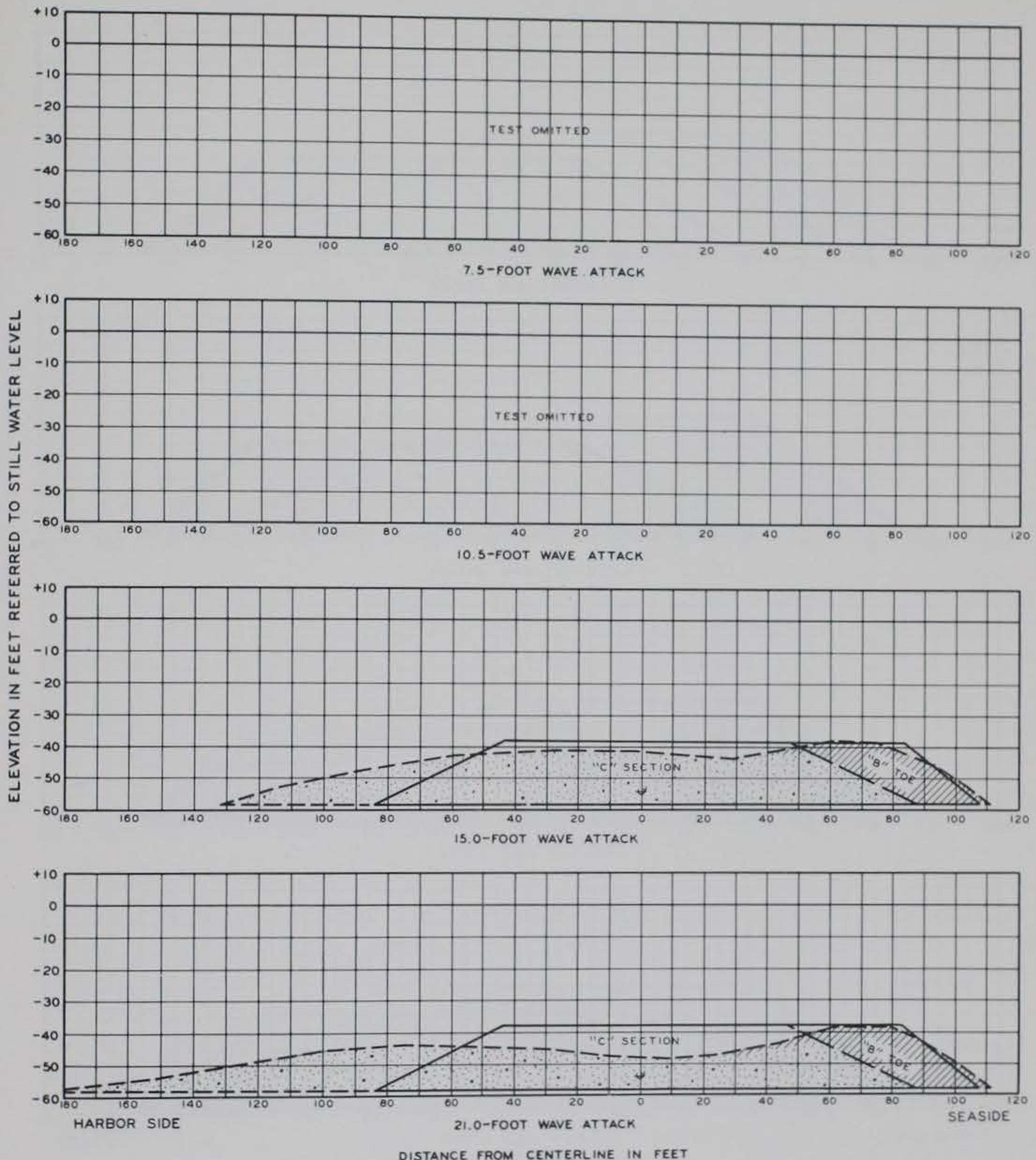
NOTE: SCALE RATIO = 1 TO 30 MODEL-TO-PROTOTYPE
ALL SECTIONS SUBJECTED TO WAVE ATTACK UNTILL DISPLACEMENT OF BREAKWATER MATERIAL CEASED
DEPTH OF WATER = 58 FEET

TEST SERIES 2

DISPLACEMENT OF BREAKWATER MATERIAL BY WAVE ACTION

CLASS C MATERIAL; B-MATERIAL TOE ON HARBOR SIDE

TOP ELEVATION -24 FT SWL



LEGEND

———— MOLDED SECTION BEFORE WAVE ATTACK
– – – – – SECTION AFTER WAVE ATTACK
CLASS B STONE AFTER WAVE ATTACK
CLASS C STONE AFTER WAVE ATTACK

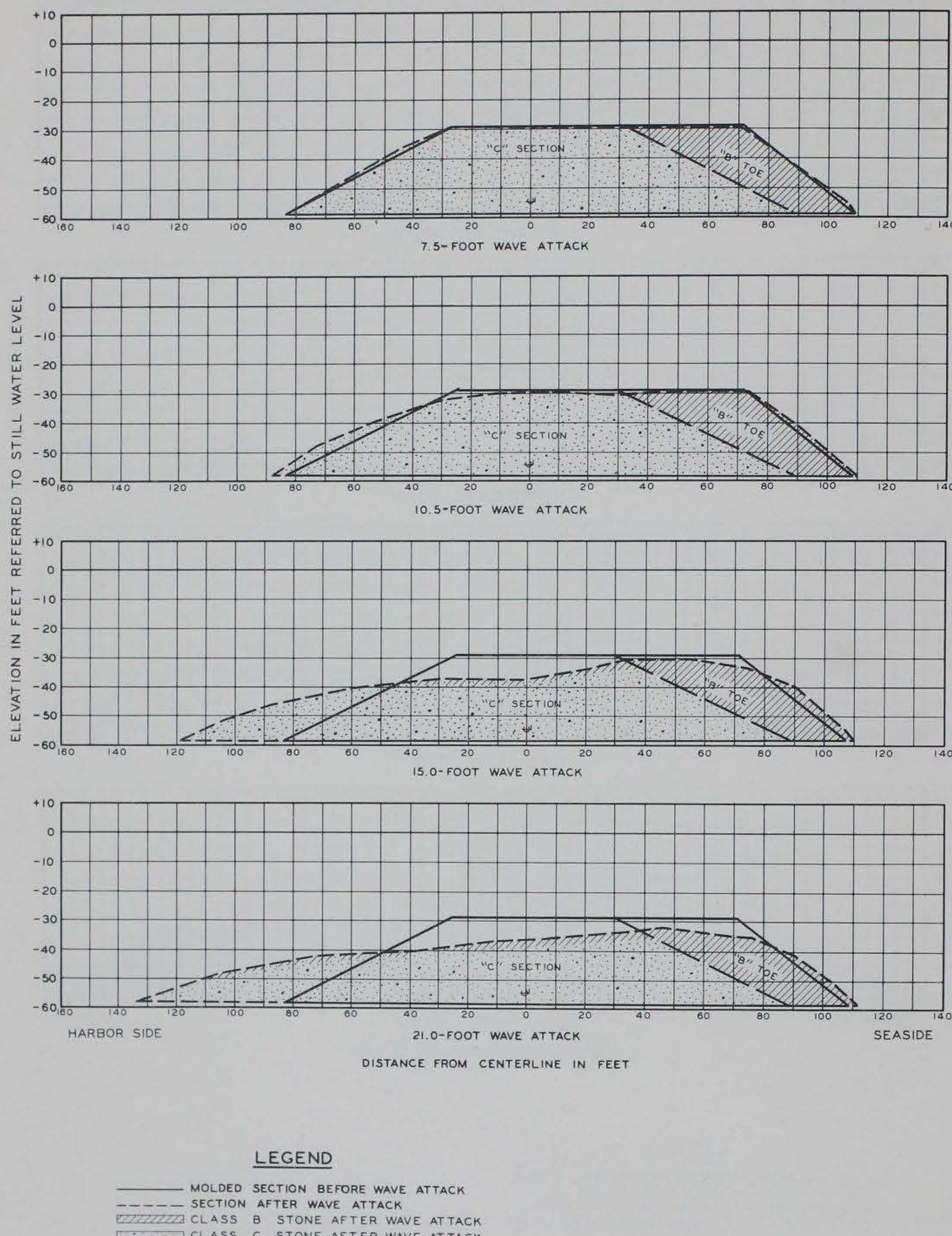
NOTE: SCALE RATIO = 1 TO 30 MODEL-TO-PROTOTYPE
ALL SECTIONS SUBJECTED TO WAVE ATTACK
UNTILL DISPLACEMENT OF BREAKWATER
MATERIAL CEASED
DEPTH OF WATER = 58 FEET

TEST SERIES 2

DISPLACEMENT OF BREAKWATER MATERIAL BY WAVE ACTION

CLASS C MATERIAL; B-MATERIAL TOE ON SEASIDE

TOP ELEVATION -38 FT SWL



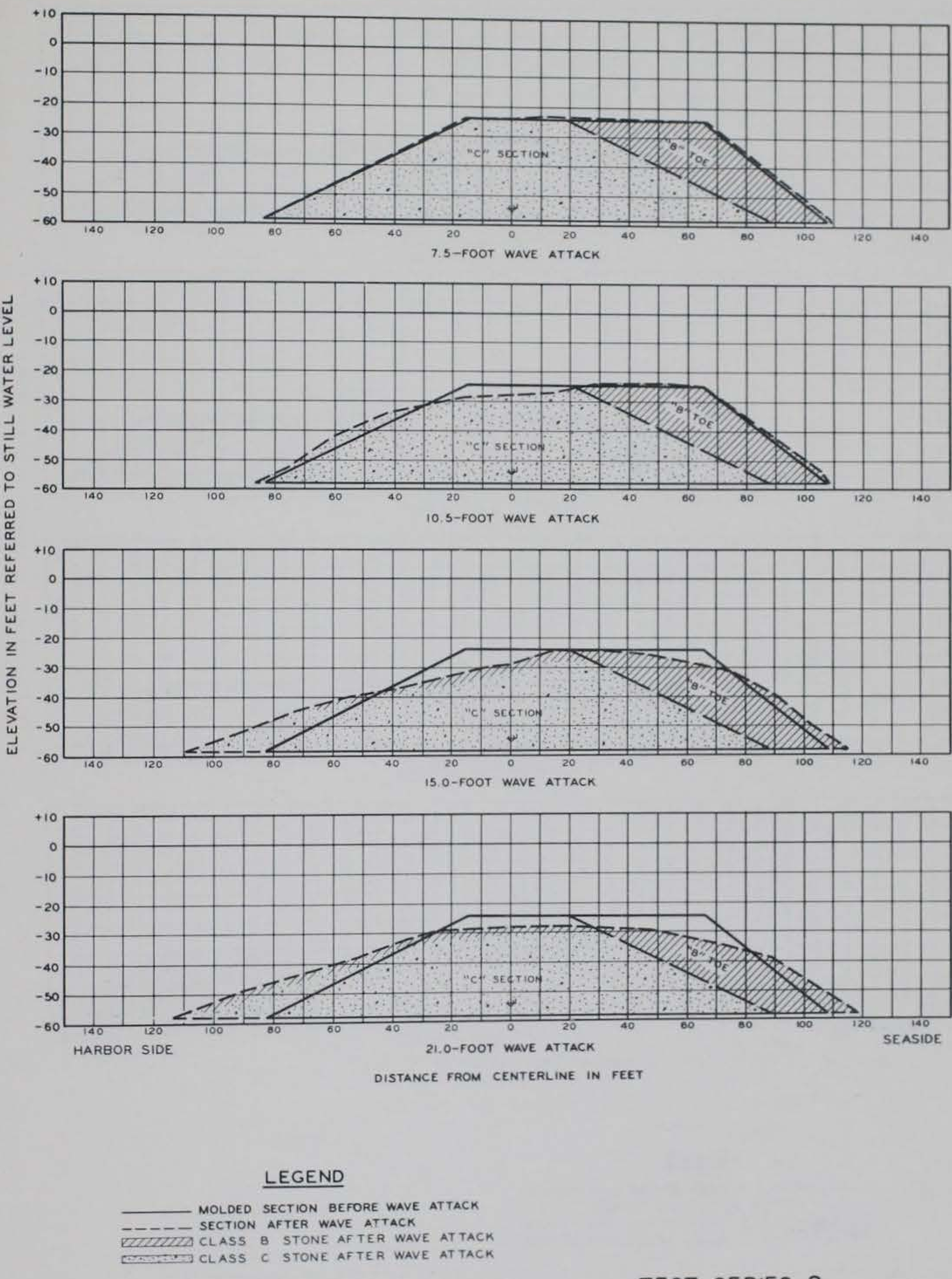
NOTE: SCALE RATIO = 1 TO 30 MODEL-TO-PROTOTYPE
ALL SECTIONS SUBJECTED TO WAVE ATTACK
UNTILL DISPLACEMENT OF BREAKWATER
MATERIAL CEASED
DEPTH OF WATER = 58 FEET

TEST SERIES 2

DISPLACEMENT OF BREAKWATER MATERIAL BY WAVE ACTION

CLASS C MATERIAL; B-MATERIAL TOE ON SEASIDE

TOP ELEVATION -29 FT SWL



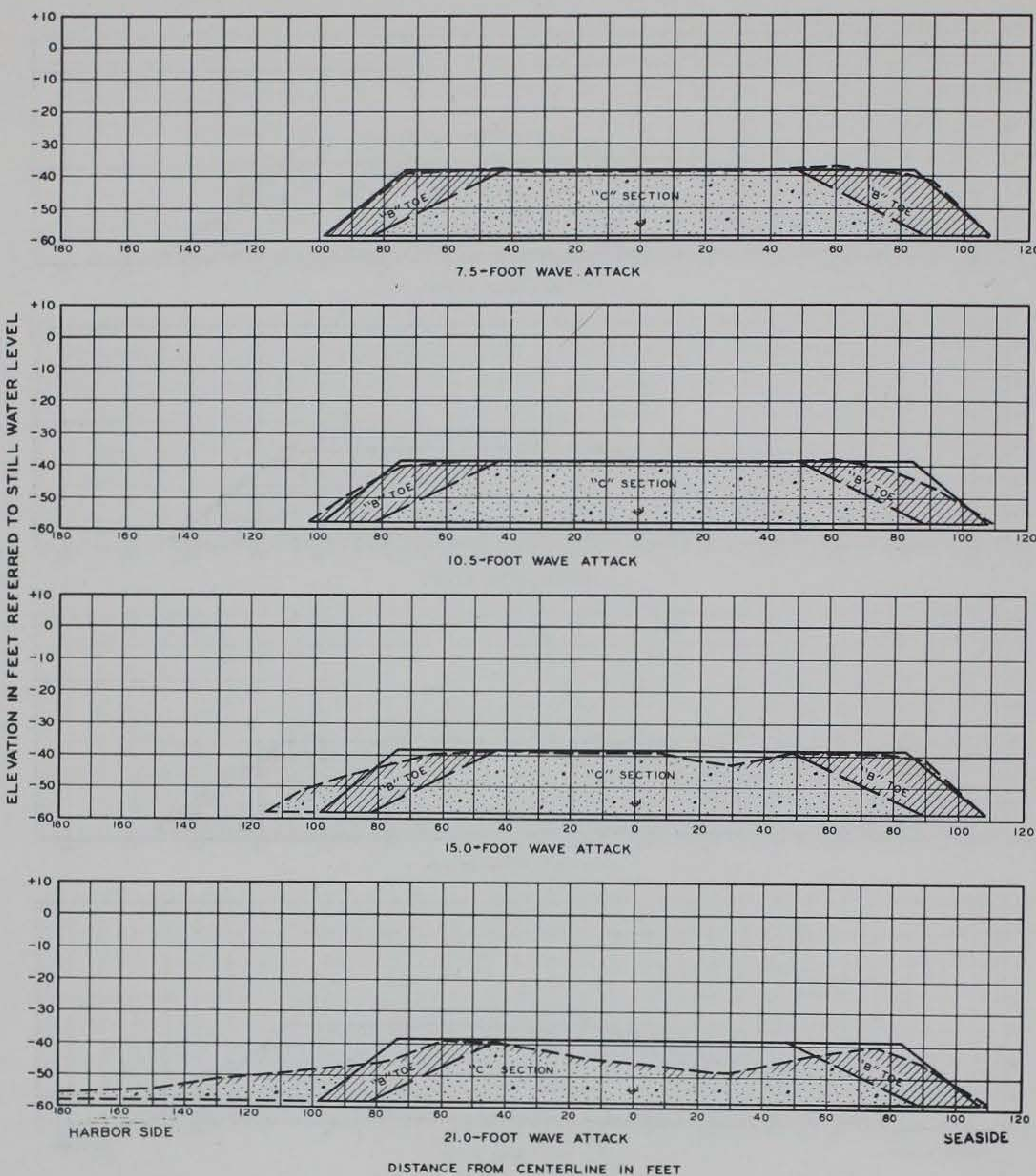
NOTE: SCALE RATIO = 1 TO 30 MODEL-TO-PROTOTYPE
ALL SECTIONS SUBJECTED TO WAVE ATTACK
UNTILL DISPLACEMENT OF BREAKWATER
MATERIAL CEASED
DEPTH OF WATER = 58 FEET

TEST SERIES 2

DISPLACEMENT OF BREAKWATER MATERIAL BY WAVE ACTION

CLASS C MATERIAL; B-MATERIAL TOE ON SEASIDE

TOP ELEVATION -24 FT SWL



LEGEND

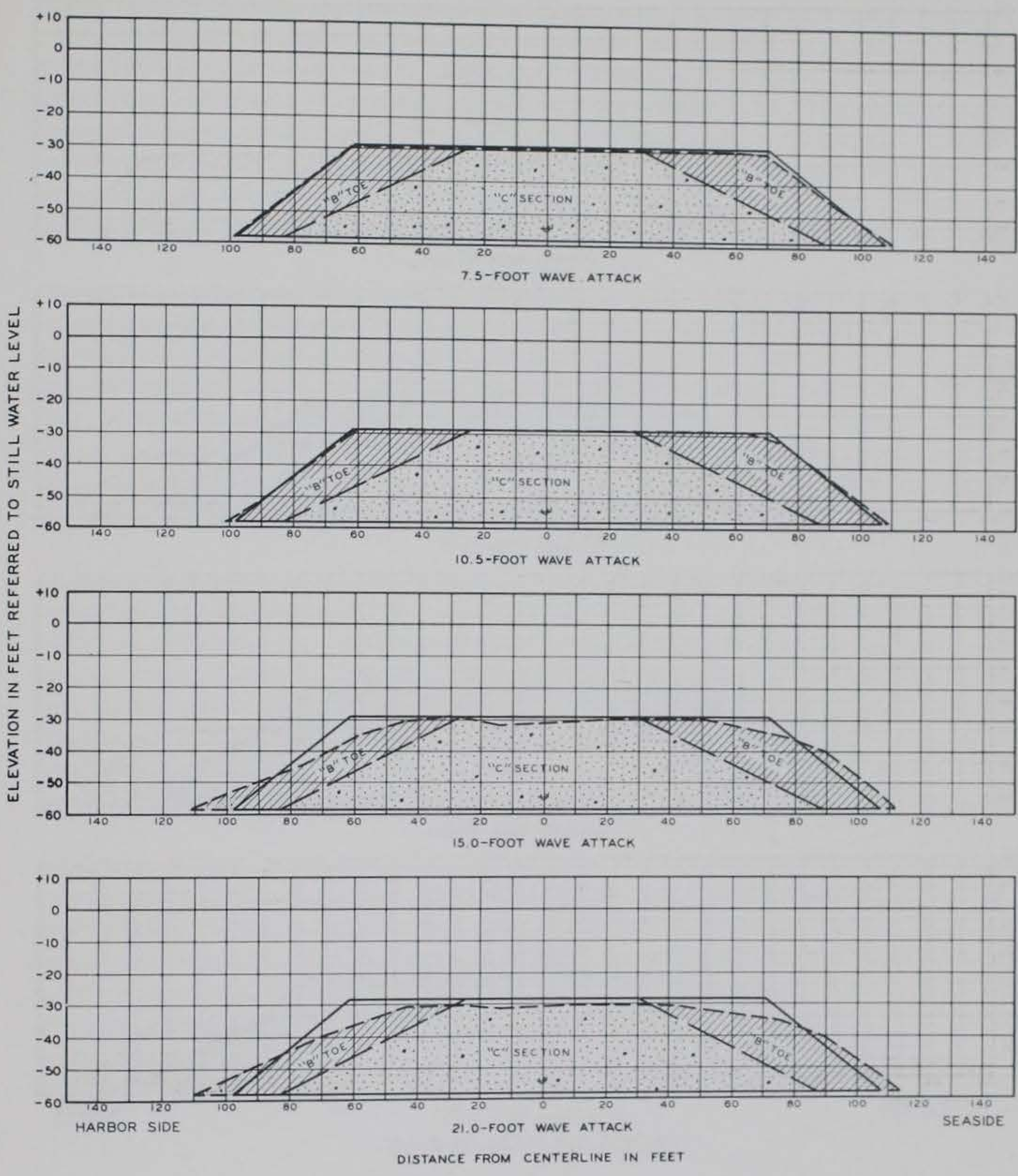
- MOLDED SECTION BEFORE WAVE ATTACK
- SECTION AFTER WAVE ATTACK
- CLASS B STONE AFTER WAVE ATTACK
- CLASS C STONE AFTER WAVE ATTACK

NOTE: SCALE RATIO = 1 TO 30 MODEL-TO-PROTOTYPE
ALL SECTIONS SUBJECTED TO WAVE ATTACK
UNTILL DISPLACEMENT OF BREAKWATER
MATERIAL CEASED
DEPTH OF WATER = 58 FEET

TEST SERIES 2

DISPLACEMENT OF BREAKWATER MATERIAL BY WAVE ACTION

CLASS C MATERIAL; B-MATERIAL
TOE ON BOTH HARBOR AND SEASIDE
TOP ELEVATION -38 FT SWL



LEGEND

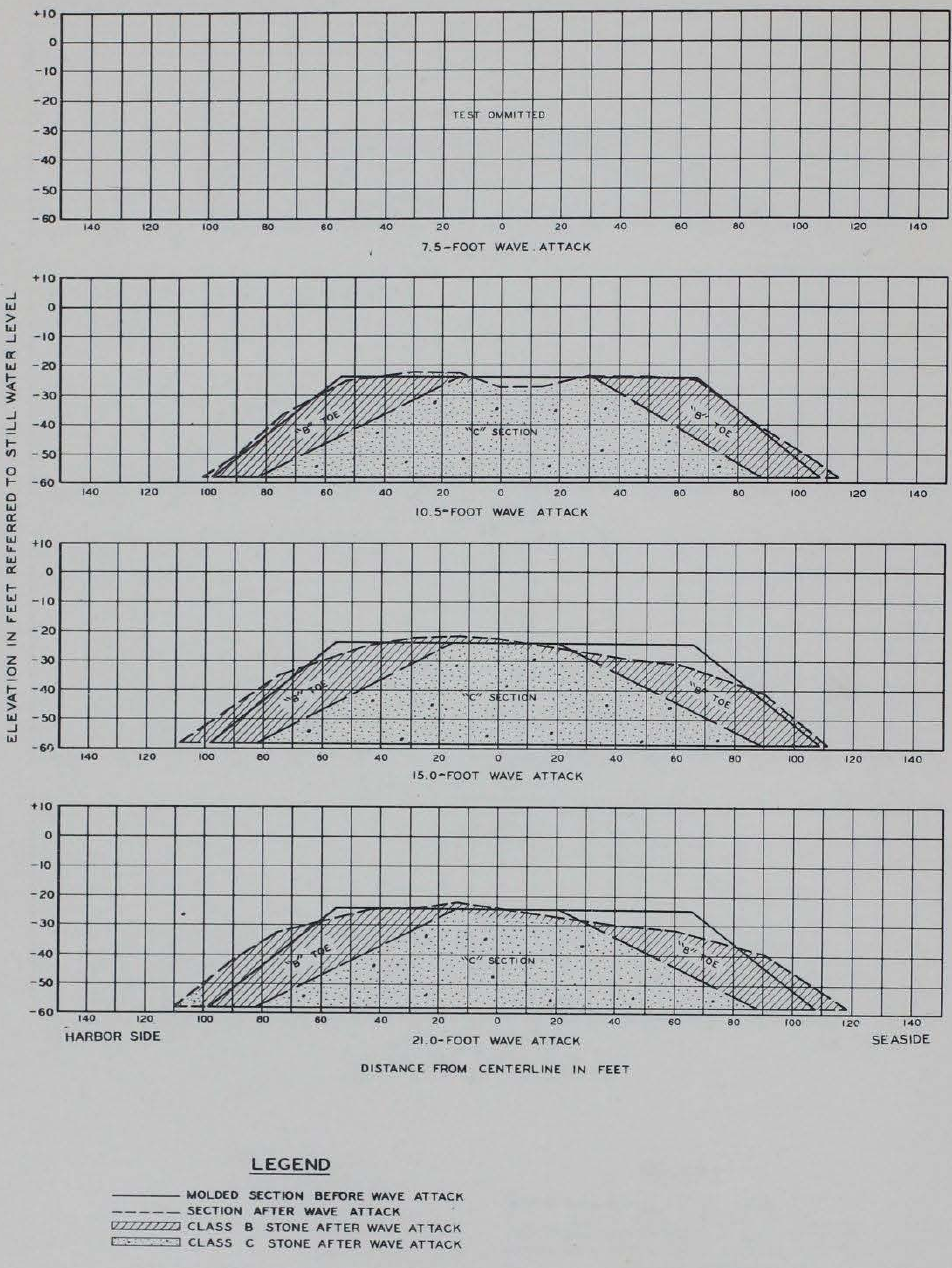
- ———— MOLDED SECTION BEFORE WAVE ATTACK
- – – – – SECTION AFTER WAVE ATTACK
- CLASS B STONE AFTER WAVE ATTACK
- CLASS C STONE AFTER WAVE ATTACK

NOTE: SCALE RATIO = 1 TO 30 MODEL-TO-PROTOTYPE
ALL SECTIONS SUBJECTED TO WAVE ATTACK UNTILL DISPLACEMENT OF BREAKWATER MATERIAL CEASED
DEPTH OF WATER = 58 FEET

TEST SERIES 2

DISPLACEMENT OF BREAKWATER MATERIAL BY WAVE ACTION

CLASS C MATERIAL; B-MATERIAL TOE ON BOTH HARBOR AND SEASIDE TOP ELEVATION -29 FT SWL

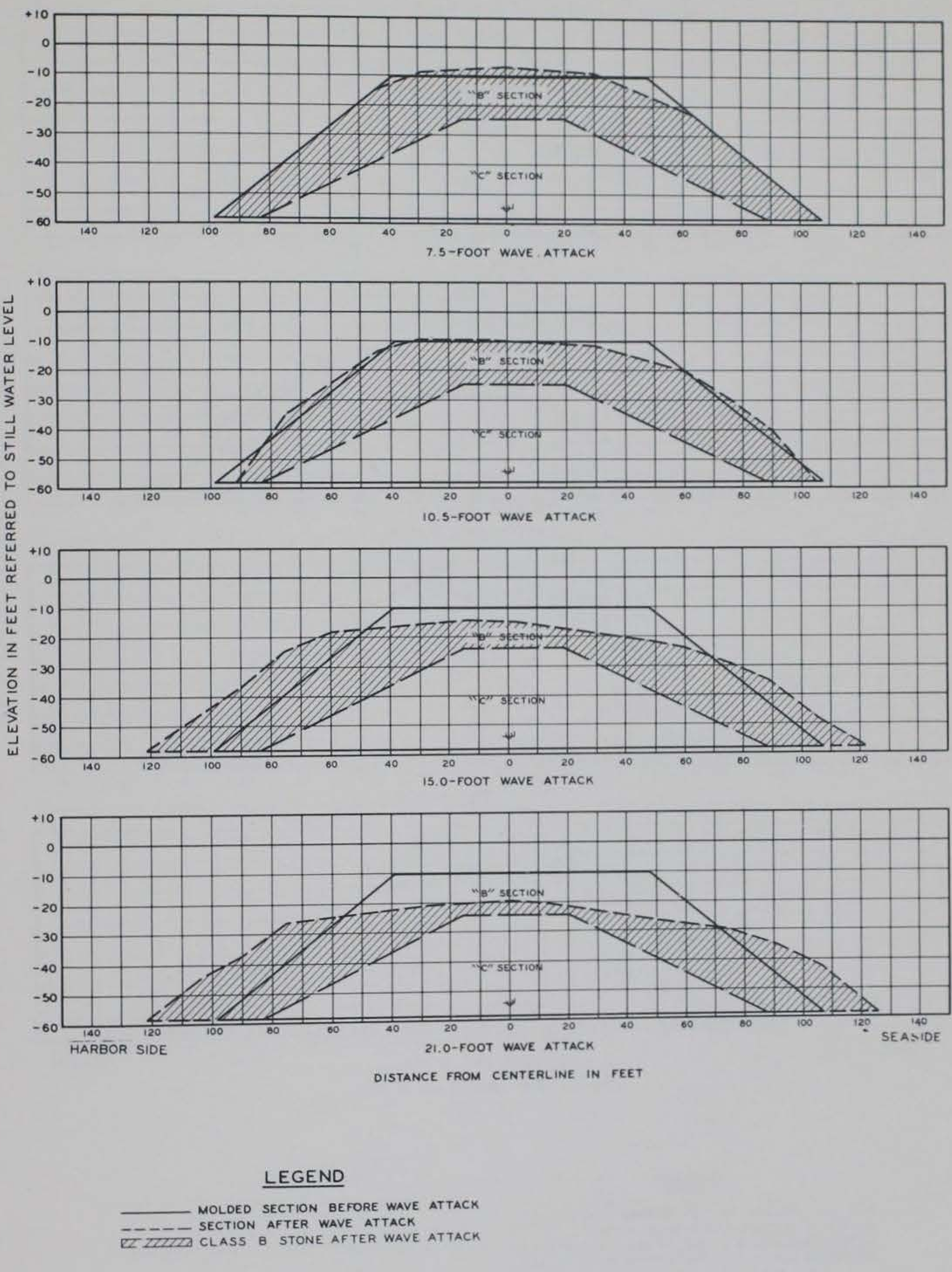


NOTE: SCALE RATIO = 1 TO 30 MODEL-TO-PROTOTYPE
ALL SECTIONS SUBJECTED TO WAVE ATTACK UNTILL DISPLACEMENT OF BREAKWATER MATERIAL CEASED
DEPTH OF WATER = 58 FEET

TEST SERIES 2

DISPLACEMENT OF BREAKWATER MATERIAL BY WAVE ACTION

CLASS C MATERIAL; B-MATERIAL TOE ON BOTH HARBOR AND SEASIDE
TOP ELEVATION -24 FT SWL

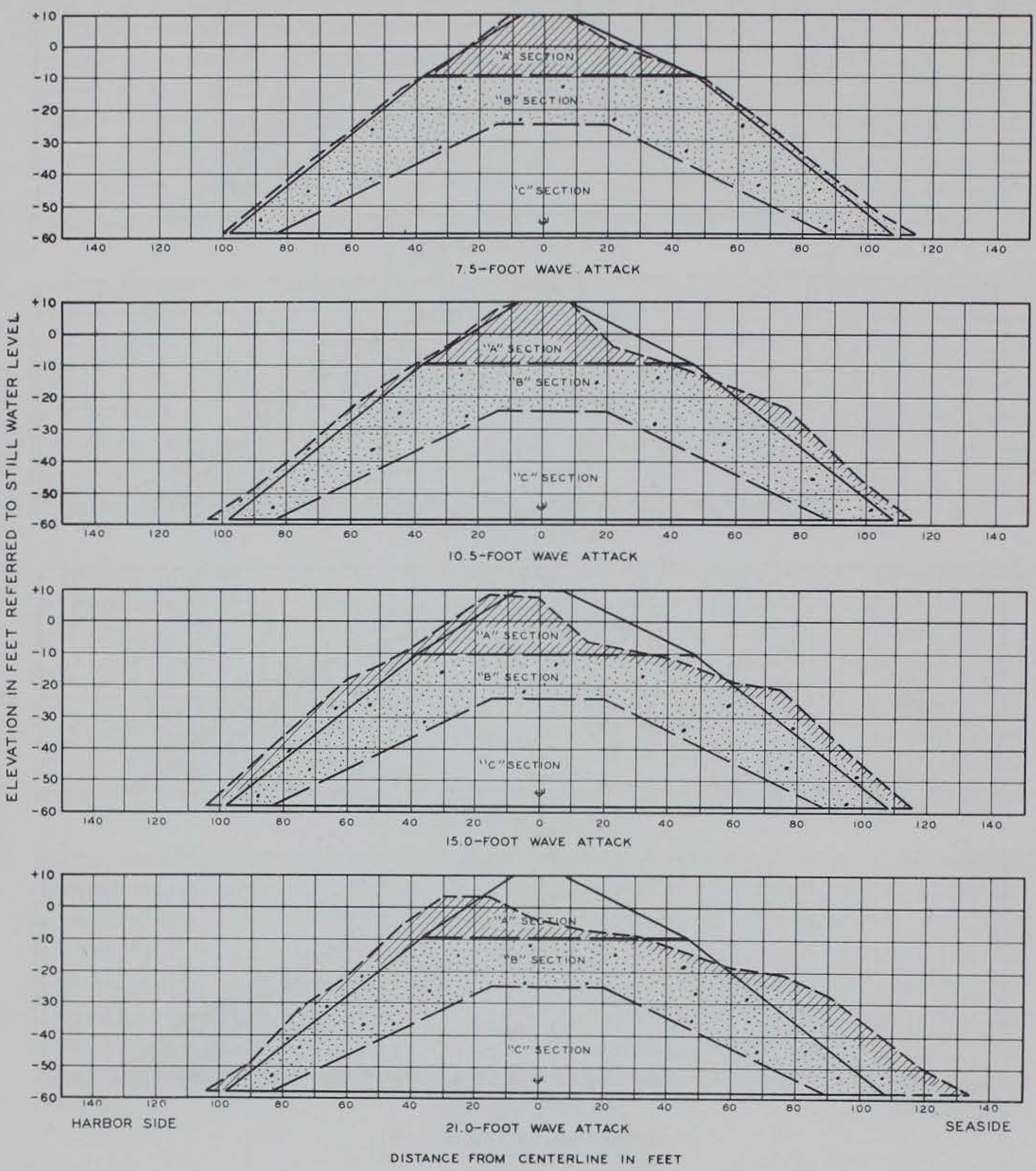


NOTE: SCALE RATIO = 1 TO 30 MODEL-TO-PROTOTYPE
ALL SECTIONS SUBJECTED TO WAVE ATTACK
UNTILL DISPLACEMENT OF BREAKWATER
MATERIAL CEASED
DEPTH OF WATER = 58 FEET

TEST SERIES 3

DISPLACEMENT OF BREAKWATER MATERIAL BY WAVE ACTION

CLASS B MATERIAL SECTION
TOP ELEVATION -10 FT SWL



LEGEND

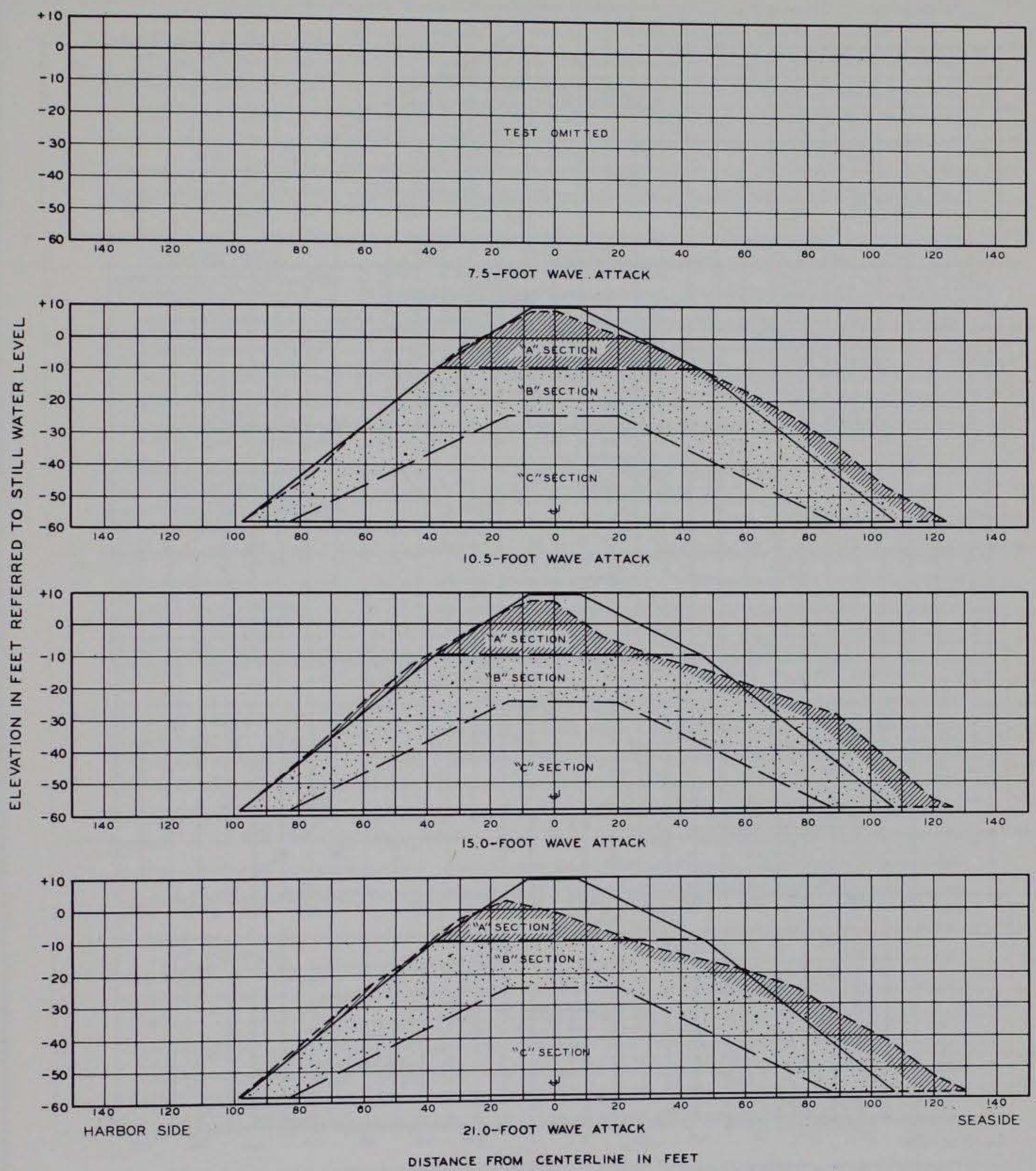
- MOLDED SECTION BEFORE WAVE ATTACK
- SECTION AFTER WAVE ATTACK
- CLASS B STONE AFTER WAVE ATTACK
- CLASS A STONE AFTER WAVE ATTACK

NOTE: SCALE RATIO = 1 TO 30 MODEL-TO-PROTOTYPE
ALL SECTIONS SUBJECTED TO WAVE ATTACK UNTILL DISPLACEMENT OF BREAKWATER MATERIAL CEASED
DEPTH OF WATER = 58 FEET

TEST SERIES 4

DISPLACEMENT OF BREAKWATER MATERIAL BY WAVE ACTION

COMPLETE SECTION; CLASS A STONE OF MOLDED CONCRETE BLOCKS
TOP ELEVATION +10 FT SWL



LEGEND

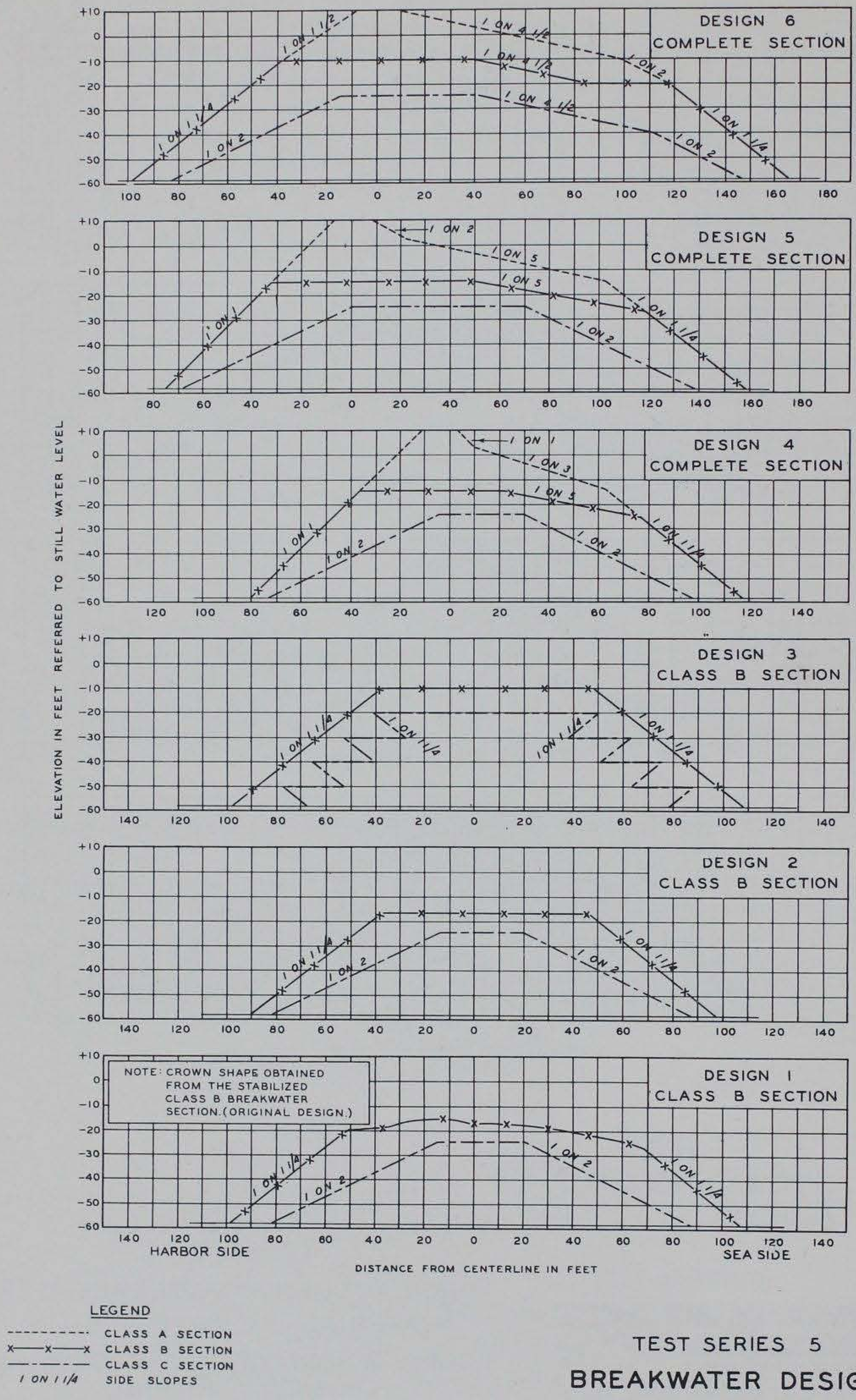
- ———— MOLDED SECTION BEFORE WAVE ATTACK
- - - - - - SECTION AFTER WAVE ATTACK
- CLASS B STONE AFTER WAVE ATTACK
- CLASS A STONE AFTER WAVE ATTACK

NOTE: SCALE RATIO = 1 TO 30 MODEL-TO-PROTOTYPE
ALL SECTIONS SUBJECTED TO WAVE ATTACK
UNTILL DISPLACEMENT OF BREAKWATER
MATERIAL CEASED
DEPTH OF WATER = 58 FEET

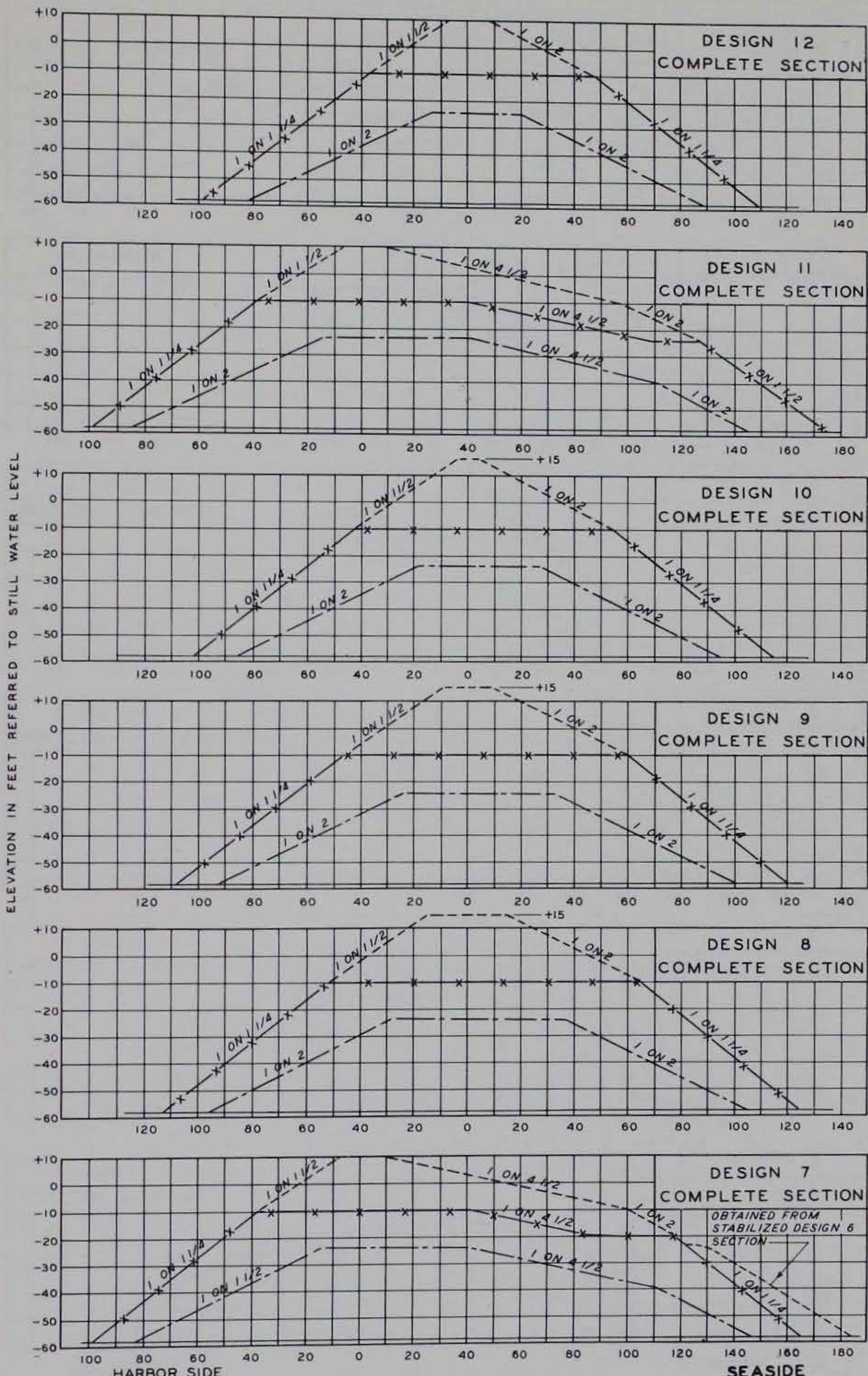
TEST SERIES 4

DISPLACEMENT OF BREAKWATER MATERIAL BY WAVE ACTION

COMPLETE SECTION; CLASS A STONE OF
CRUSHED LIMESTONE
TOP ELEVATION +10 FT SWL



TEST SERIES 5
BREAKWATER DESIGNS
1, 2, 3, 4, 5 AND 6



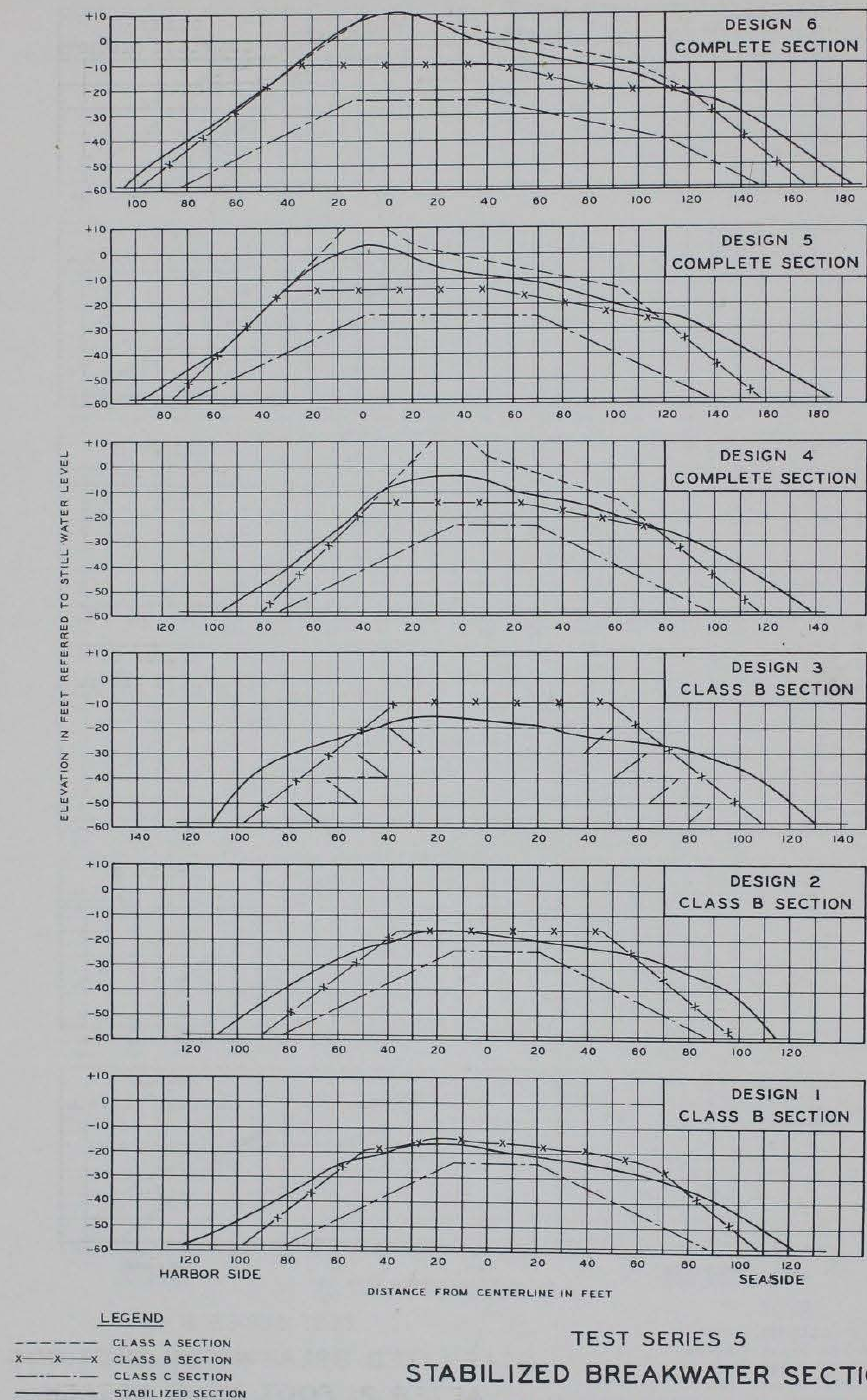
LEGEND

- - - - - - - -	CLASS A SECTION
x——x——x	CLASS B SECTION
— — - —	CLASS C SECTION
1 ON 1 1/4	SIDE SLOPES

NOTE: STABILITY TESTS ON BREAKWATER DESIGN 11 WERE CONDUCTED USING WATER DEPTHS OF 53, 58, 63 AND 68 FT.

TEST SERIES 5

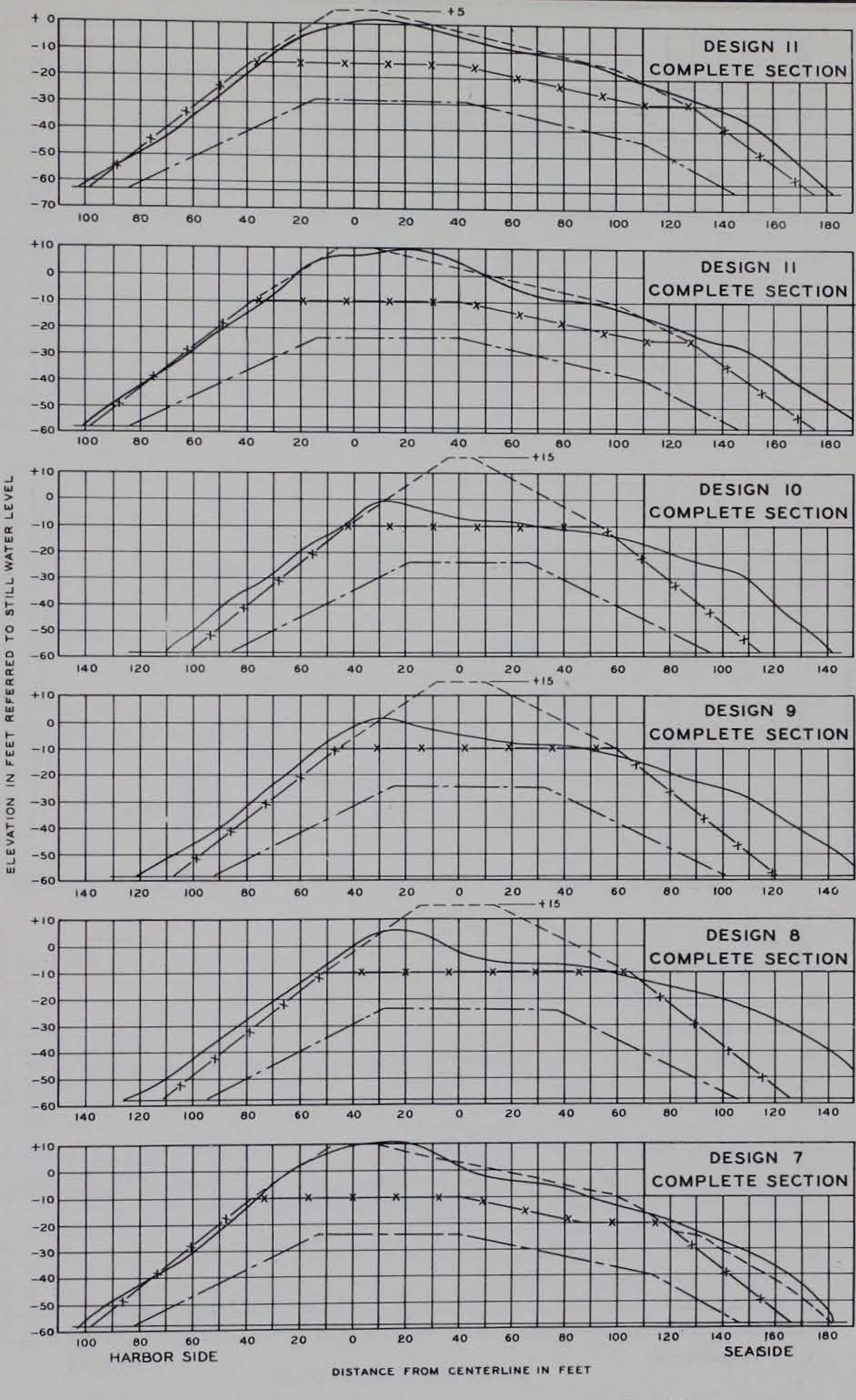
BREAKWATER DESIGNS 7, 8, 9, 10, 11 AND 12



TEST SERIES 5

STABILIZED BREAKWATER SECTIONS
AFTER 21 FOOT WAVE ATTACK

DESIGNS I-6

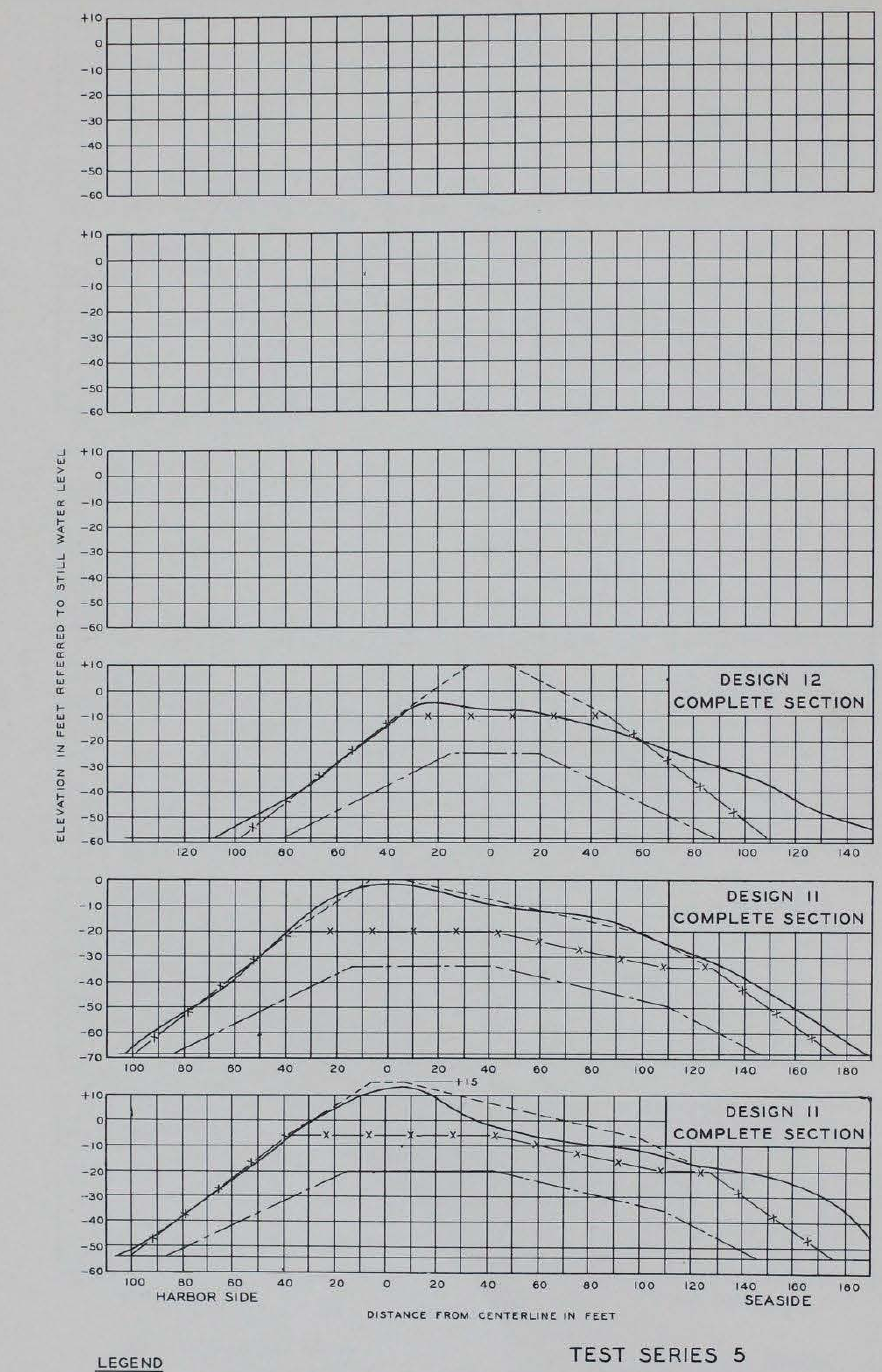


LEGEND

- - - - - - - - CLASS A SECTION
x——x——x CLASS B SECTION
— - — - — CLASS C SECTION
———— STABILIZED SECTION

TEST SERIES 5

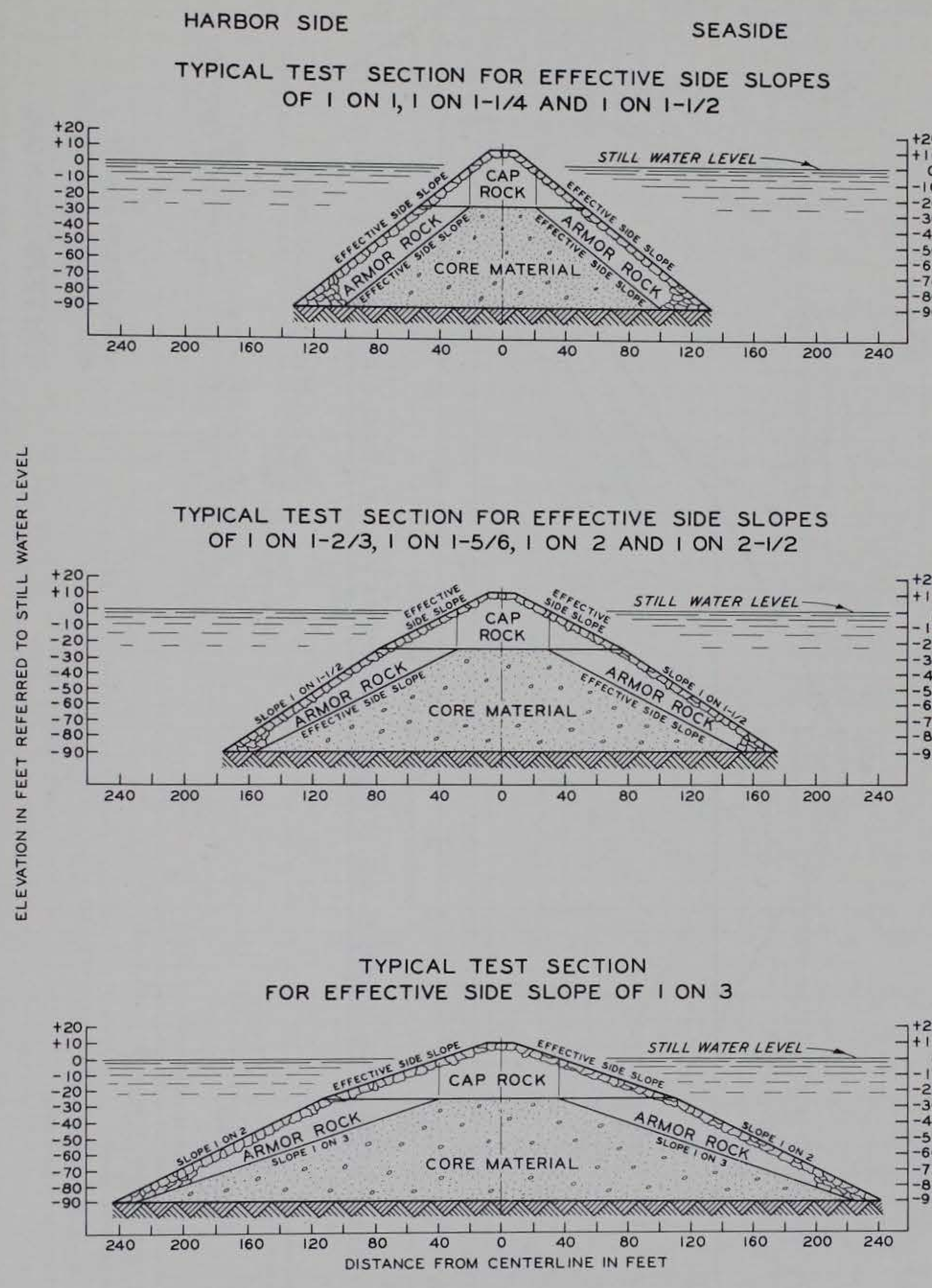
STABILIZED BREAKWATER SECTIONS
AFTER 21 FOOT WAVE ATTACK
DESIGNS 7-11



TEST SERIES 5

STABILIZED BREAKWATER SECTIONS AFTER 21 FOOT WAVE ATTACK

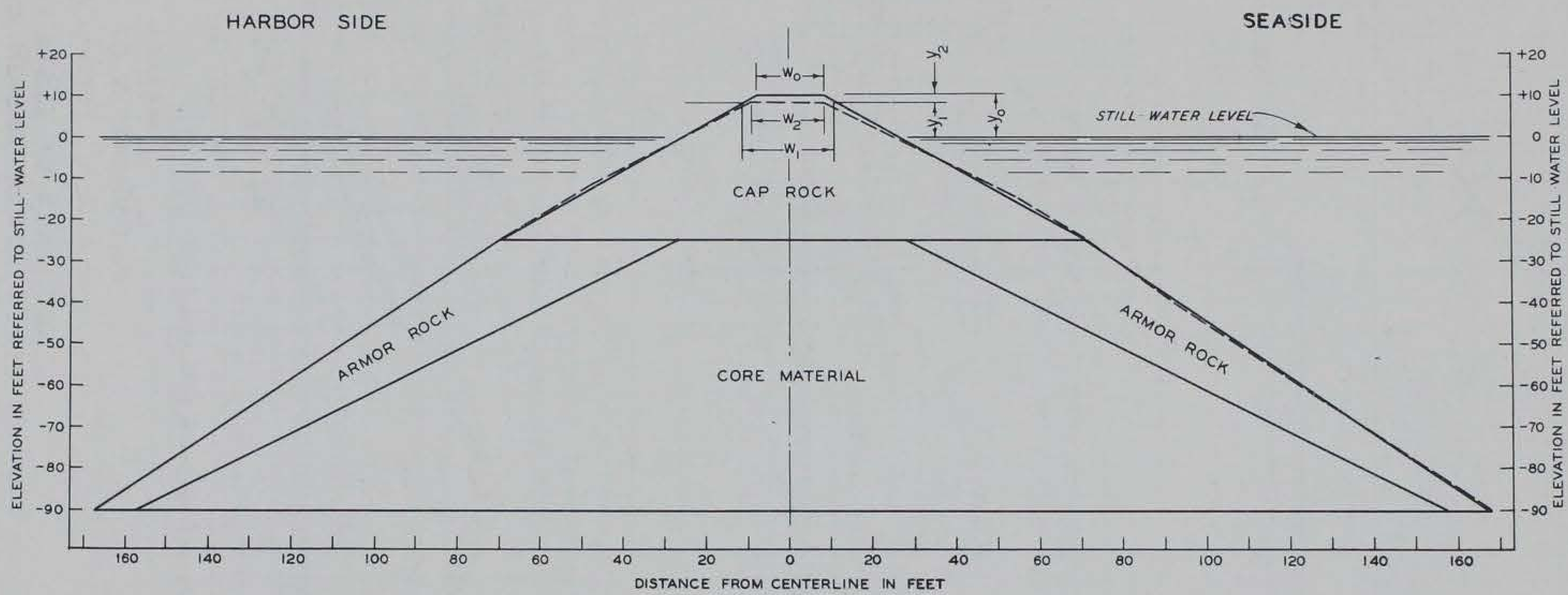
DESIGNS 11 AND 12



CROSS SECTIONS AT 90 FEET WATER DEPTH

INVESTIGATION OF
IRIBARREN'S FORMULA

ELEMENTS OF TYPICAL TEST SECTIONS



ILLUSTRATIVE CROSS SECTIONS AT 90 FEET WATER DEPTH

LEGEND

- ——— MOLDED SECTION BEFORE WAVE ATTACK.
- – – – STABILIZED SECTION AFTER WAVE ATTACK.

NOMENCLATURE

SYMBOL	DEFINITION	UNIT
y_0	HEIGHT OF CROWN OF MOLDED BREAKWATER ABOVE STILL WATER LEVEL.	FT
y_1	HEIGHT OF CROWN OF STABILIZED BREAKWATER ABOVE STILL-WATER LEVEL.	FT
y_2	DIFFERENCE BETWEEN HEIGHT OF CROWN OF MOLDED BREAKWATER AND HEIGHT OF CROWN OF STABILIZED BREAKWATER.	FT
W_0	WIDTH OF CROWN OF MOLDED BREAKWATER.	FT
W_1	WIDTH OF TRANSVERSE SECTION OF MOLDED BREAKWATER MEASURED ALONG PROJECTED CROWN LINE OF STABILIZED BREAKWATER.	FT
W_2	WIDTH OF CROWN OF STABILIZED BREAKWATER.	FT

CRITERION FOR DESIGN WAVES – NO DAMAGE

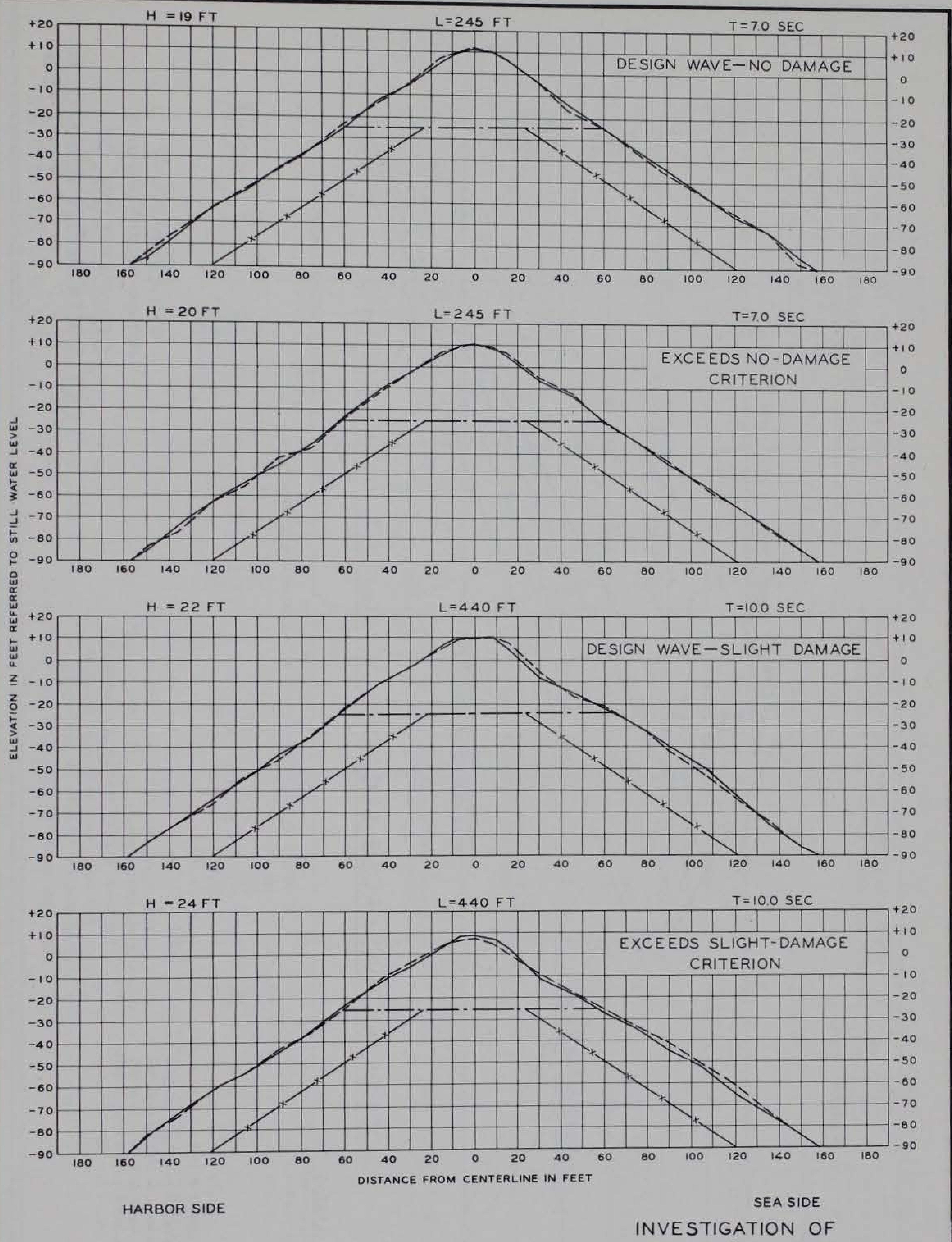
THE MAXIMUM WAVES INCAPABLE OF MOVING CAP ROCK FROM CROWN AND SLOPE OF BREAKWATER.

CRITERION FOR DESIGN WAVES – SLIGHT DAMAGE

THE MAXIMUM WAVES INCAPABLE OF CAUSING DAMAGE TO BREAKWATER SECTION GREATER THAN THAT MEASURED BY EQUATIONS $y_2 = 0.15\,y_0$ AND $W_2 = 0.75\,W_1$.

INVESTIGATION OF
IRIBARREN'S FORMULA

**CRITERION FOR
SELECTION OF DESIGN WAVES**



LEGEND

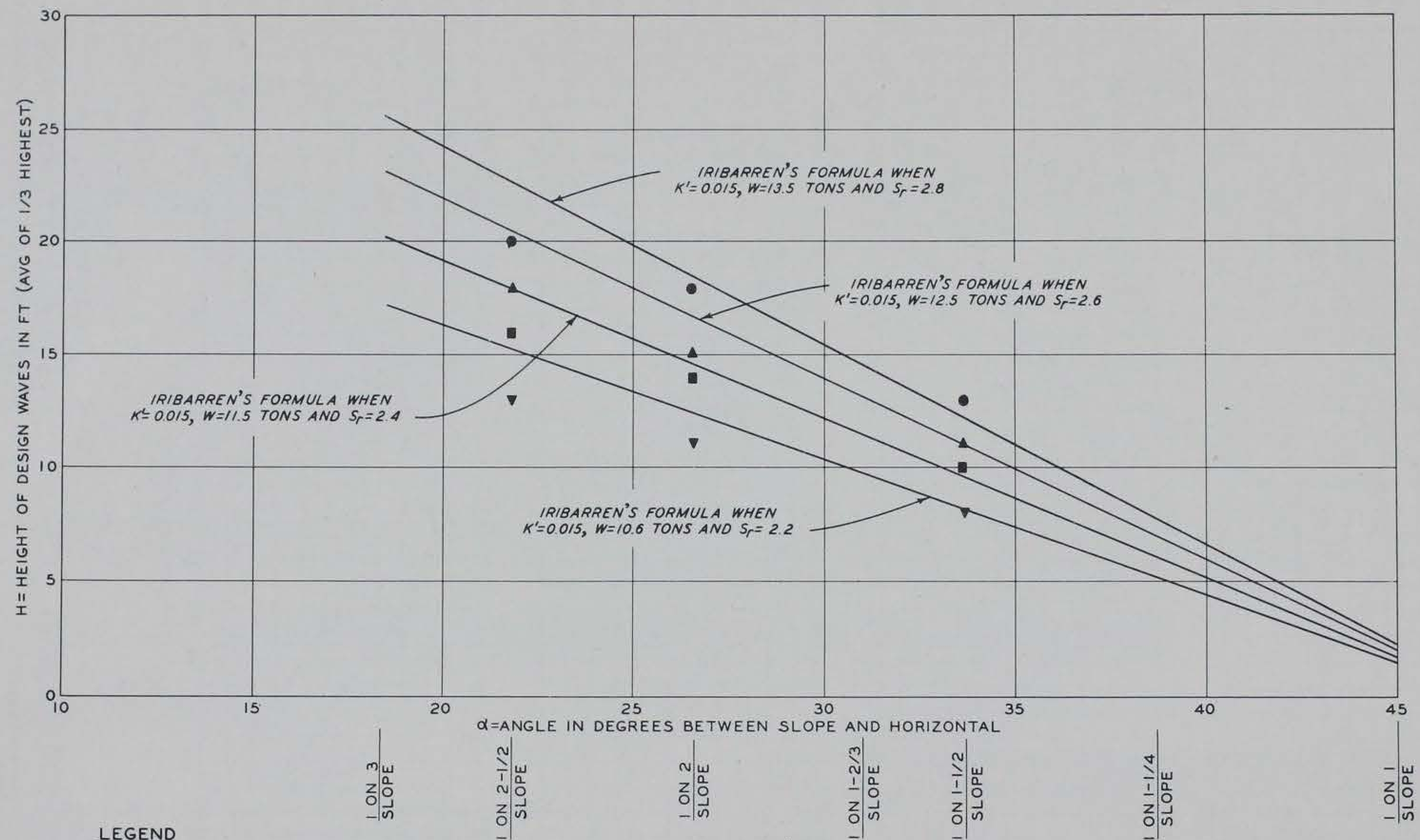
- ——— SECTION AFTER WAVE ATTACK—INITIAL TESTS
- — — — SECTION AFTER WAVE ATTACK—CHECK TESTS
- —·—·— BASE OF CAP ROCK
- —x—x— BOUNDARY BETWEEN INTERIOR CORE AND EXTERIOR ARMOR ROCK

NOTE: H = AVERAGE HEIGHT OF $\frac{1}{3}$ HIGHEST WAVES

INVESTIGATION OF IRIBARREN'S FORMULA

ACCURACY OF TESTS FOR SELECTION OF DESIGN WAVES

WEIGHT OF CAP ROCK=13.5 TONS
EFFECTIVE SIDE SLOPES=1 ON 1-$\frac{1}{2}$
SPECIFIC GRAVITY OF ROCK = 2.8
DEPTH OF WATER=90 FT



LEGEND

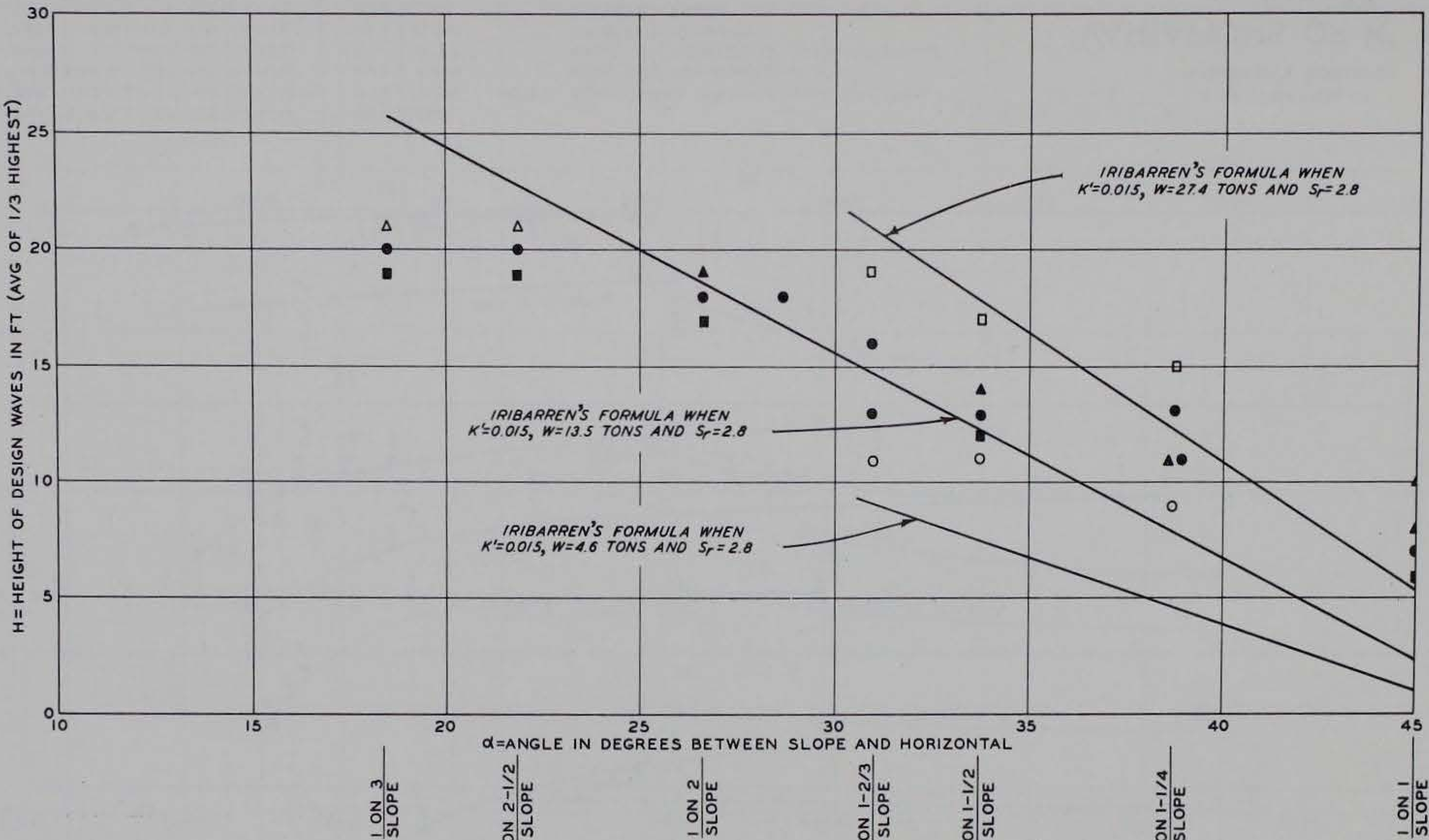
- ● W=13.5 TONS, S_r = 2.8 AND T= 10.0 SEC
- ▲ W=12.5 TONS, S_r = 2.6 AND T= 10.0 SEC
- ■ W=11.5 TONS, S_r = 2.4 AND T= 10.0 SEC
- ▼ W=10.6 TONS, S_r = 2.2 AND T= 10.0 SEC

NOTE: DESIGN WAVES ARE FOR NO-DAMAGE CRITERION
ANGLE OF INCIDENCE OF WAVE ATTACK=90°
EFFECTIVE WEIGHT AND SPECIFIC GRAVITY OF CAP ROCK WAS VARIED BY CHANGING THE SPECIFIC GRAVITY OF TEST FLUID,
SIZE AND SHAPE OF CAP ROCK WAS CONSTANT

INVESTIGATION OF
IRIBARREN'S FORMULA

COMPARISON OF
TEST DATA WITH FORMULA

S_r = 2.2, 2.4, 2.6 AND 2.8



LEGEND

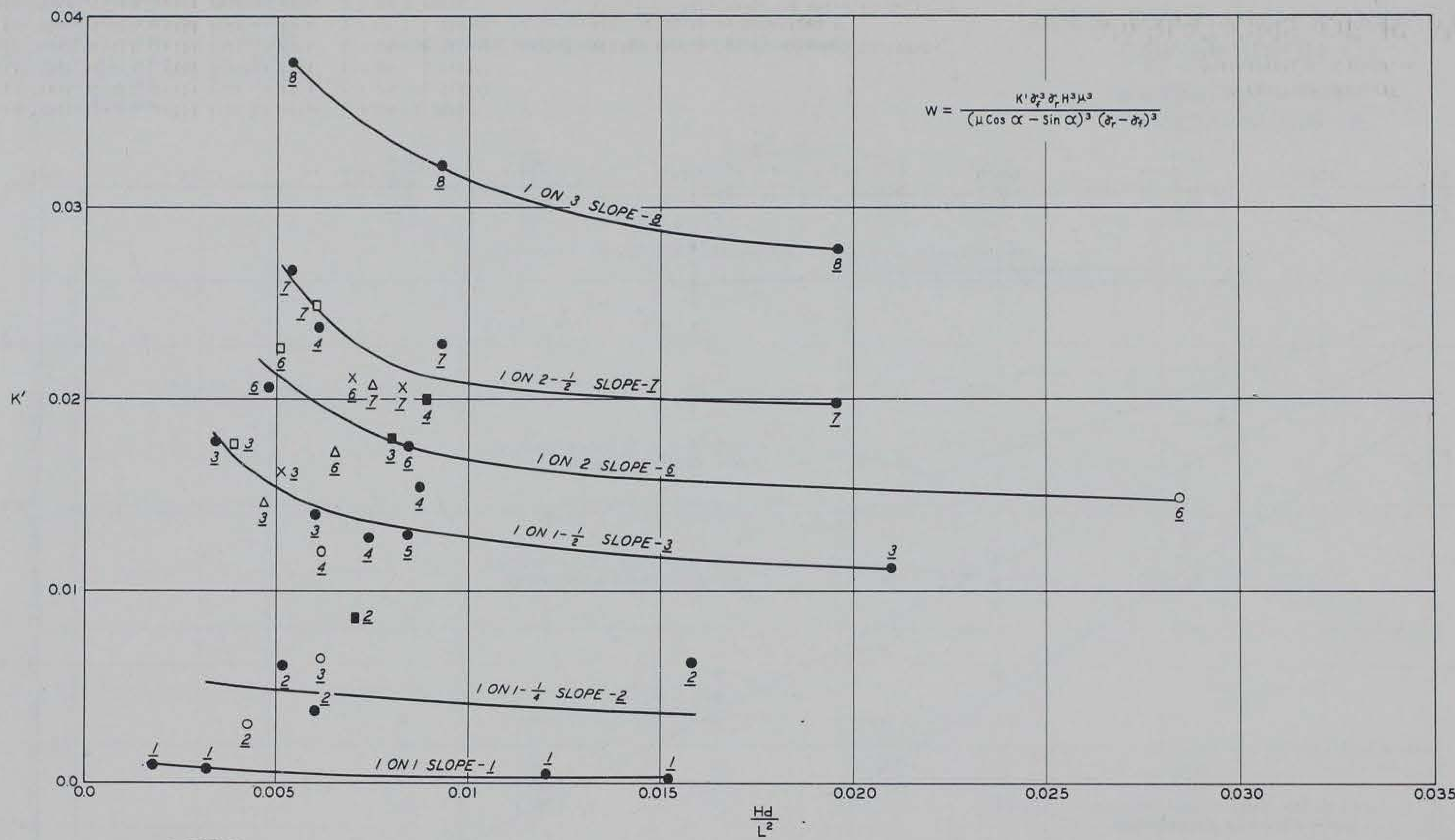
- ○ W = 4.6 TONS, S_r = 2.8 AND T = 10.0 SEC
- ▲ W = 13.5 TONS, S_r = 2.8 AND T = 7.0 SEC
- △ W = 13.5 TONS, S_r = 2.8 AND T = 8.0 SEC
- ● W = 13.5 TONS, S_r = 2.8 AND T = 10.0 SEC
- ■ W = 13.5 TONS, S_r = 2.8 AND T = 12.0 SEC
- □ W = 27.4 TONS, S_r = 2.8 AND T = 10.0 SEC

NOTE: DESIGN WAVES ARE FOR NO-DAMAGE CRITERION
ANGLE OF INCIDENCE OF WAVE ATTACK = 90°

INVESTIGATION OF
IRIBARREN'S FORMULA

COMPARISON OF TEST DATA WITH FORMULA

$S_r = 2.8$



LEGEND

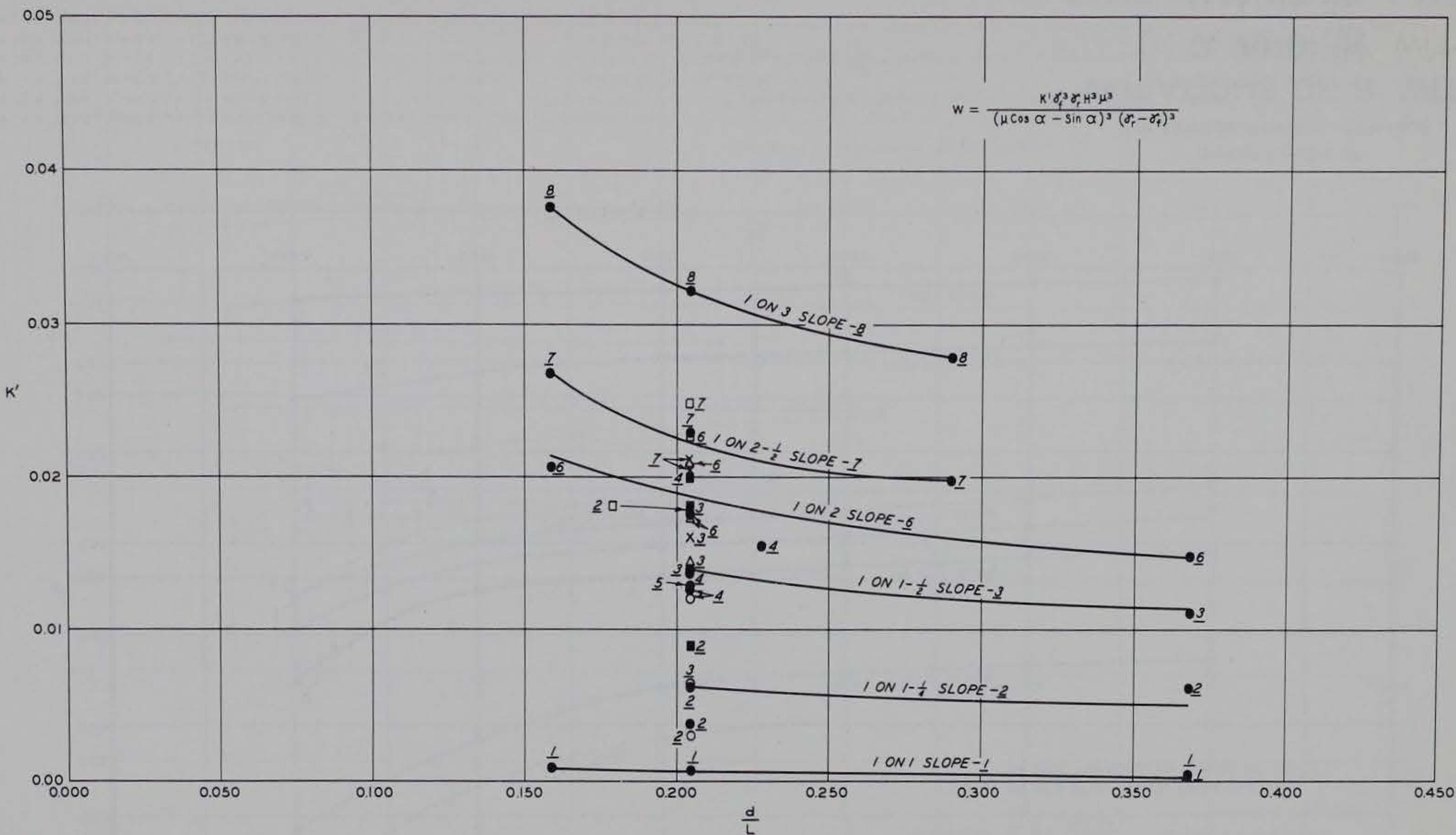
O W= 4.6 TONS, S_r=2.8, μ=1.06 AND ν_r = 41 %
□ W= 10.6 TONS, S_r=2.2, μ=1.09 AND ν_r = 43 %
Δ W= 11.5 TONS, S_r=2.4, μ=1.09 AND ν_r = 43 %
X W= 12.5 TONS, S_r=2.6, μ=1.09 AND ν_r = 43 %
● W= 13.5 TONS, S_r=2.8, μ=1.09 AND ν_r = 43 %
■ W= 27.4 TONS, S_r=2.8, μ=1.18 AND ν_r = 44 %

1 1 ON 1 SLOPE
2 1 ON 1-1/4 SLOPE
3 1 ON 1-1/2 SLOPE
4 1 ON 1-2/3 SLOPE
5 1 ON 1-5/6 SLOPE
6 1 ON 2 SLOPE
7 1 ON 2-1/2 SLOPE
8 1 ON 3 SLOPE

NOTES: DESIGN WAVES FOR NO-DAMAGE CRITERION WERE USED FOR COMPUTING K'
ANGLE OF INCIDENCE OF WAVE ATTACK=90°
d = DEPTH OF WATER
L = WAVE LENGTH

INVESTIGATION OF
IRIBARREN'S FORMULA

VARIATIONS OF K' WITH α AND $\frac{Hd}{L^2}$



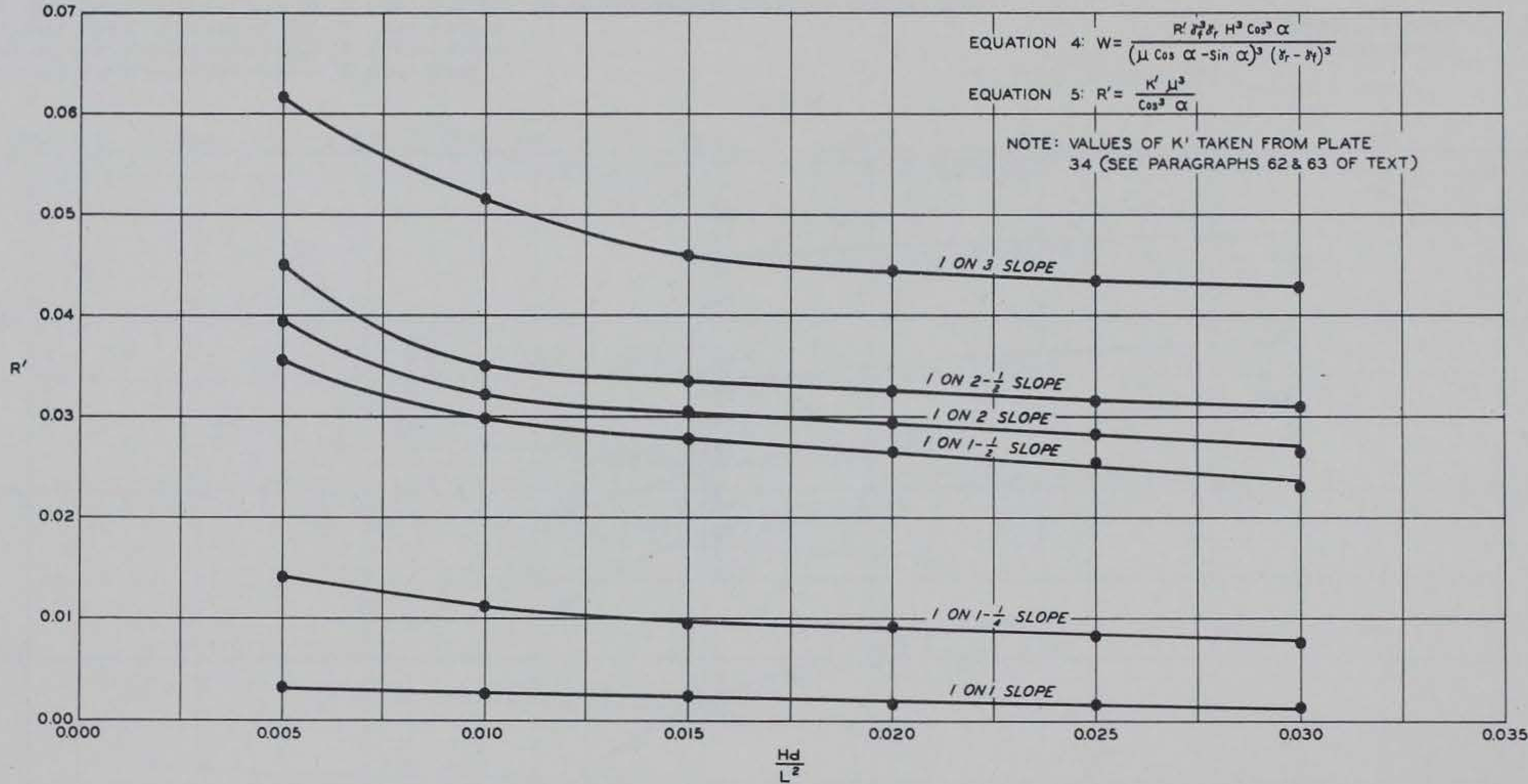
○ W = 4.6 TONS, S_r=2.8, μ=1.06 AND ν_r = 41 %
□ W = 10.6 TONS, S_r=2.2, μ=1.09 AND ν_r = 43 %
△ W = 11.5 TONS, S_r=2.4, μ=1.09 AND ν_r = 43 %
X W = 12.5 TONS, S_r=2.6, μ=1.09 AND ν_r = 43 %
● W = 13.5 TONS, S_r=2.8, μ=1.09 AND ν_r = 43 %
■ W = 27.4 TONS, S_r=2.8, μ=1.18 AND ν_r = 44 %

1 1 ON 1 SLOPE
2 1 ON 1-$\frac{1}{4}$ SLOPE
3 1 ON 1-$\frac{1}{2}$ SLOPE
4 1 ON 1-$\frac{2}{3}$ SLOPE
5 1 ON 1-$\frac{5}{6}$ SLOPE
6 1 ON 2 SLOPE
7 1 ON 2-$\frac{1}{2}$ SLOPE
8 1 ON 3 SLOPE

NOTES: DESIGN WAVES FOR NO-DAMAGE CRITERION WERE USED FOR COMPUTING K'
ANGLE OF INCIDENCE OF WAVE ATTACK=90°
d = DEPTH OF WATER
L = WAVE LENGTH

INVESTIGATION OF
IRIBARREN'S FORMULA

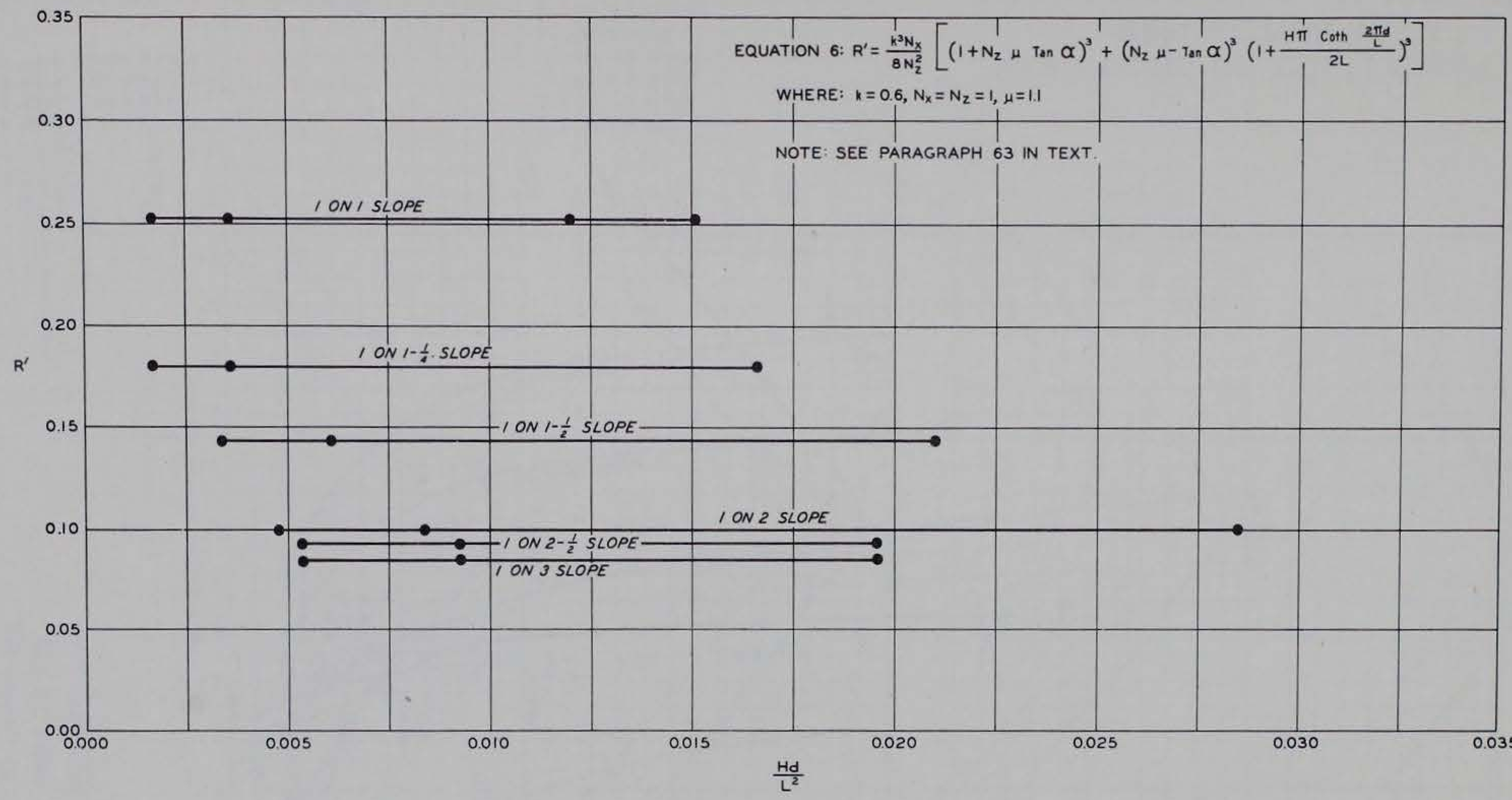
VARIATIONS OF K' WITH α AND $\frac{d}{L}$



INVESTIGATION OF
THE EPSTEIN-TYRRELL FORMULA

VARIATIONS OF R' WITH α AND $\frac{Hd}{L^2}$

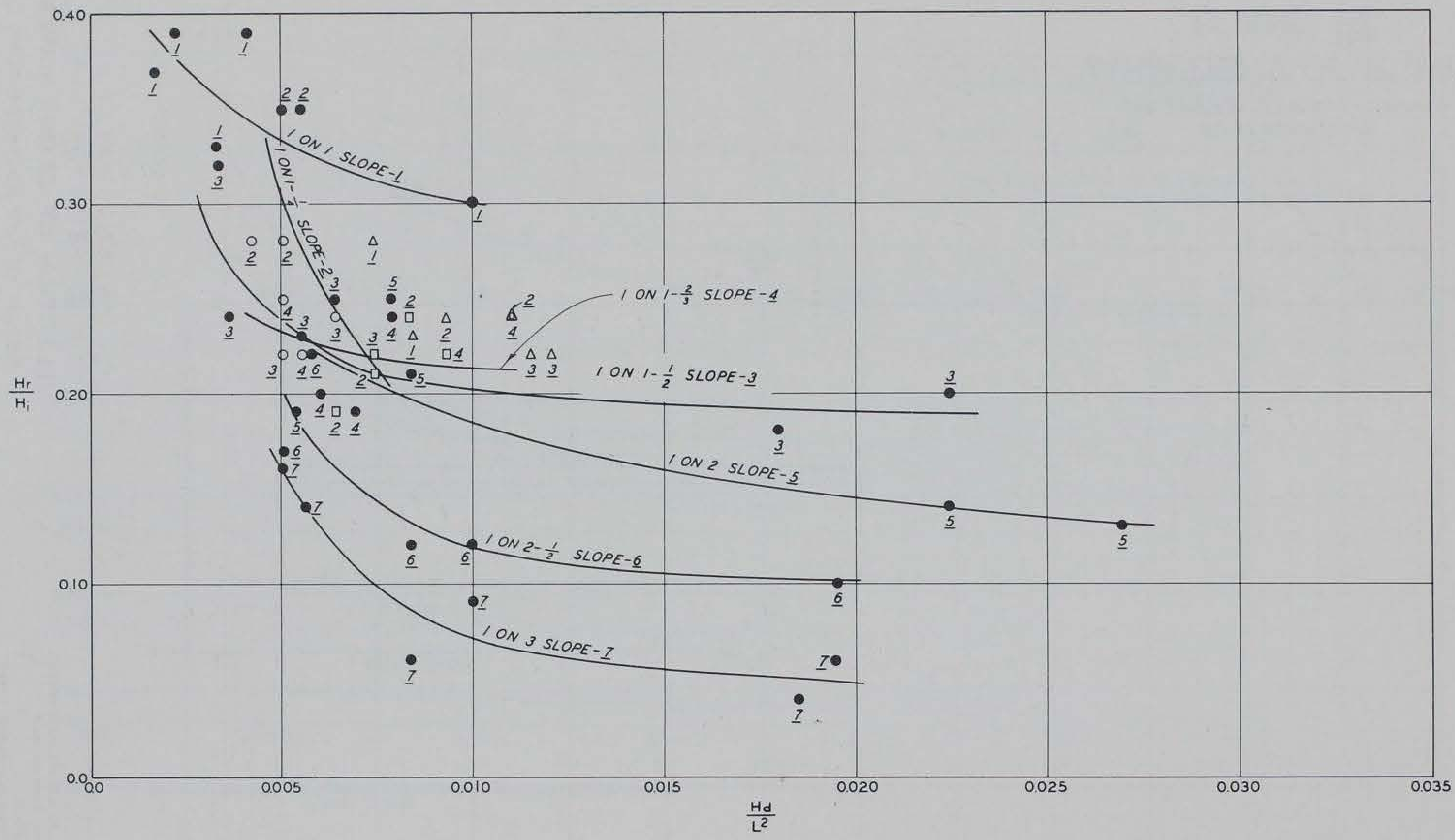
BASED ON EQUATIONS 4 AND 5



INVESTIGATION OF
THE EPSTEIN-TYRRELL FORMULA

VARIATIONS OF R′ WITH
α AND $\frac{Hd}{L^2}$

BASED ON EQUATION 6

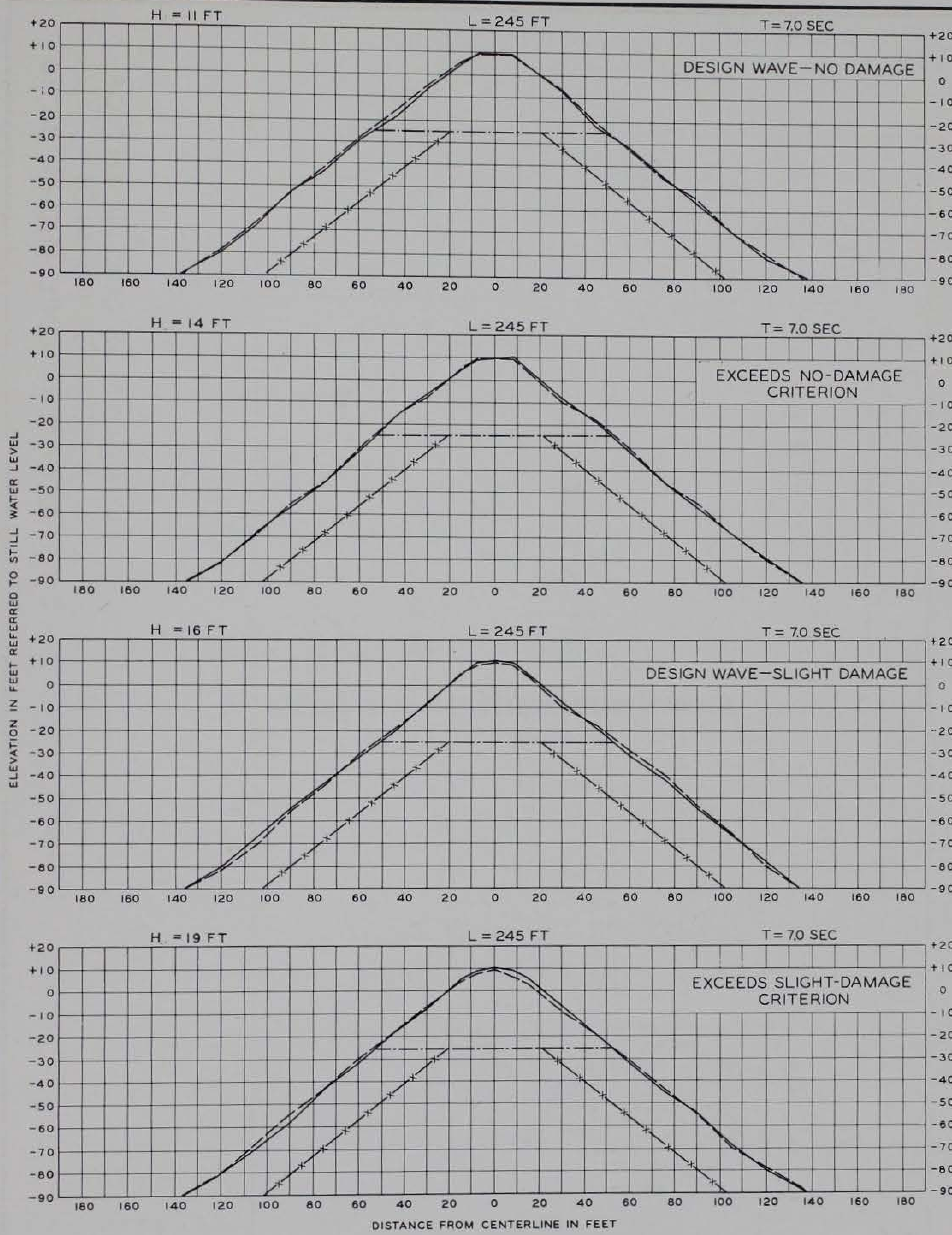


○ W= 4.6 TONS, S_r=2.8 μ= 1.06
● W=13.5 TONS, S_r=2.8 μ= 1.09
□ W=27.4 TONS, S_r=2.8 μ= 1.18
△ W=36.4 TONS, S_r=2.8 μ= 1.35

NOTE: ANGLE OF INCIDENCE OF WAVE ATTACK=90°

1 1 ON 1 SLOPE
2 1 ON 1-$\frac{1}{4}$ SLOPE
3 1 ON 1-$\frac{1}{2}$ SLOPE
4 1 ON 1-$\frac{2}{3}$ SLOPE
5 1 ON 2 SLOPE
6 1 ON 2-$\frac{1}{2}$ SLOPE
7 1 ON 3 SLOPE

VARIATIONS OF COEFFICIENT OF REFLECTION WITH BREAKWATER SLOPE AND $\frac{H_d}{L^2}$ RATIO



LEGEND

————	MOLDED SECTION BEFORE WAVE ATTACK
— — — —	SECTION AFTER WAVE ATTACK
—·—·—	BASE OF CAP ROCK
—x—x—	BOUNDARY BETWEEN INTERIOR CORE AND EXTERIOR ARMOR ROCK

NOTE: H = AVERAGE HEIGHT OF $\frac{1}{3}$ HIGHEST WAVES

INVESTIGATION OF IRIBARREN'S FORMULA

SELECTION OF DESIGN WAVES

WEIGHT OF CAP ROCK= 13.5 TONS

EFFECTIVE SIDE SLOPES 1 ON 1- $\frac{1}{4}$

SPECIFIC GRAVITY OF ROCK = 2.8

DEPTH OF WATER=90 FT

APPENDIX A: THE IRIBARREN FORMULA

APPENDIX A: THE IRIBARREN FORMULA

Discussion of Original Formula

1. The formula proposed by Iribarren* for use in selecting the size and weight cap rock and seaside slope of rubble breakwaters is:

$$W = \frac{KH^3 S_r}{(\cos \alpha - \sin \alpha)^3 (S_r - 1)^3} \tag{1}$$

where W is the weight of individual cap rock in kilograms; $K = 15$ and 19 for breakwaters constructed of natural rock fill and artificial blocks, respectively; H is height of wave, crest to trough, which breaks on the structure, in meters; S_r is the specific weight of cap rock in metric tons per cubic meter; and α is the angle, measured from horizontal, of the seaside slope. This formula represents a simplification of the basic equations, is not dimensionally homogeneous, and the coefficient K is not dimensionless. Although W is defined in terms of kilograms and S_r in metric tons per cubic meter, this does not affect the numerical results of the formula provided the metric-kilogram-second force system of units is used and the liquid in which the rock is placed is pure water (sp gr = 1). The M-K-S force system is used extensively in engineering practice in continental Europe. Some confusion with respect to units could have been avoided for American engineers if the original formula had been expressed

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* "Una formula para el calculo de los diques de escollera," by Ramon Iribarren Cavanilles, Revista de Obras Publicas, Madrid, Spain, July 1938. (Or see "A Formula for the Calculation of Rock-Fill Dykes," by Ramon Iribarren Cavanilles, translated by D. Heinrick, University of California, Dept. of Engineering TR-He-116-295, Berkeley, California, August 1948.

as follows:

$$W = \frac{KH^3 \gamma_r}{(\cos \alpha - \sin \alpha)^3 (\gamma_r - \gamma_w)^3} \tag{2}$$

where γ_r and γ_w are the specific weights of rock and pure water, respectively, in kilograms per cubic meter. The terms W and H are in kilograms and meters, respectively, as in equation (1). However, in equation (2) the values of K become 15×10^6 and 19×10^6, instead of simply 15 or 19 as before. It is believed, therefore, that the original formula (equation (1)) was expressed by Iribarren in such metric units as would result in convenient magnitudes of the coefficient K . The coefficient of friction between rock was assumed to be unity, and the values of K were determined by Iribarren from observation of existing full-scale breakwaters that had been subjected to storm waves.

2. Although equation (2) avoids the confusion of units inherent in equation (1), the coefficient K is not dimensionless, and neither equation (1) nor equation (2) is readily susceptible of general verification by small-scale tests. It was necessary, therefore, to rederive the Iribarren formula to obtain the more complete and general form of the equation.

Derivation of Iribarren's Basic Equation

3. Derivation of the original Iribarren formula was founded upon the assumption that dynamic forces tending to displace rock from the breakwater slope are proportional to wave height, area of rock over which the forces act, and specific weight of the liquid; or

$$F_{dy} = k\ A\ \gamma_f\ H \tag{3}$$

where k is the coefficient of drag for an individual rock. Also, the analysis is based on an assumed force diagram as shown below:

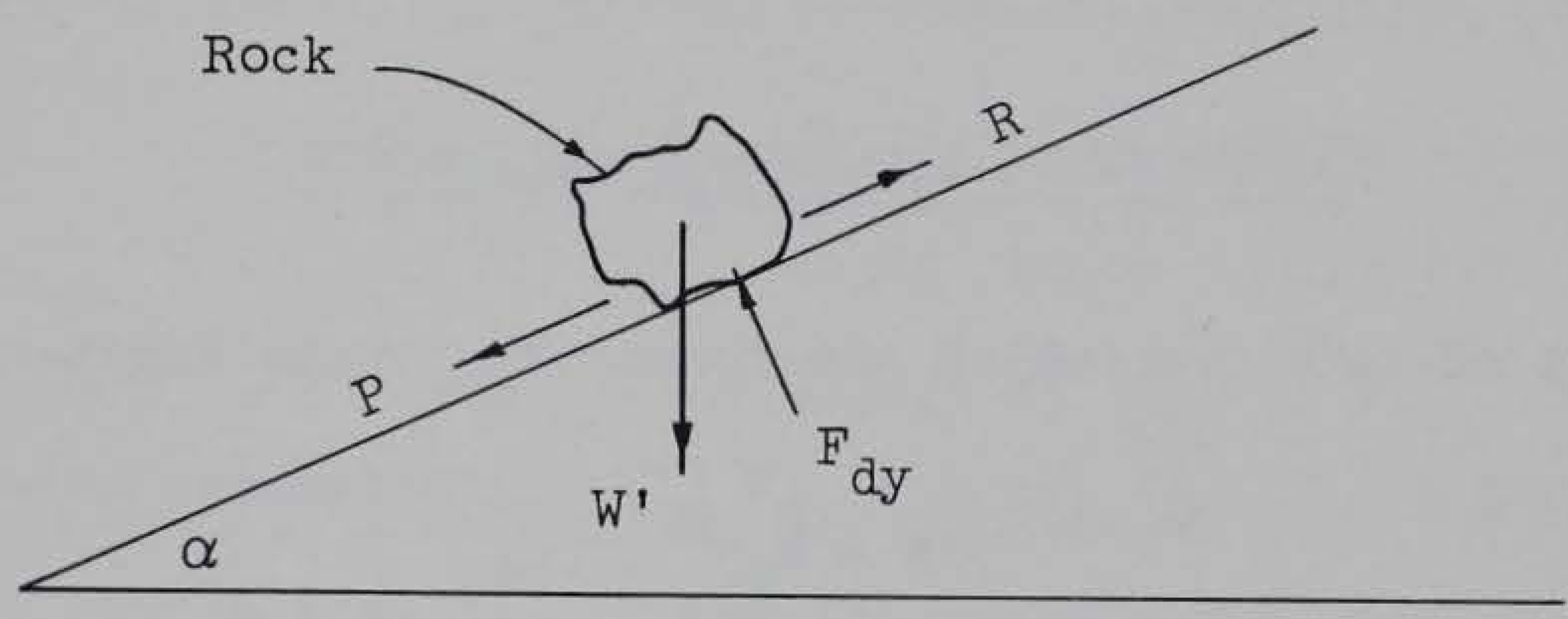

Fig. A1. Definition sketch

For equilibrium, the friction force (R) must be equal to or greater than the downslope component (P) of the submerged rock weight (W'). The dynamic force (F_{dy}) was assumed to act upward, perpendicular to the breakwater slope. This latter assumption is based on the premises that: (a) the waves break on the seaside slope and direct jets of water downward perpendicular to the slope, and (b) at the beginning and end of the splashes the jets create forces opposite in direction to the flow of water in the jets. The friction force tending to keep the rock from sliding down the slope is

$$R = \mu(W' \cos \alpha - F_{dy}) \tag{4}$$

where μ is the effective coefficient of friction, rock on rock. The downslope component of the submerged rock weight is

$$P = W' \sin \alpha \tag{5}$$

The equilibrium equation for the tangential forces, and the equation which expresses the principal thoughts of Iribarren, is

$$W' \sin \alpha = \mu(W' \cos \alpha - k\, A\, \gamma_f\, H) \; . \tag{6}$$

The submerged weight of the rock can be expressed as

$$W' = k_1\, y^3\, (\gamma_r - \gamma_f) \tag{7}$$

and the area of rock over which the force F_{dy} acts can be expressed as

$$A = k_2 y^2 \tag{8}$$

where k_1 and k_2 are dimensionless coefficients which depend upon the shape of the rock, and y is a characteristic linear dimension of the rock. The values of W' and A from equations (7) and (8) are substituted in equation (6), obtaining the relationship

$$k_1\, y^3\, (\gamma_r - \gamma_f) \sin \alpha = \mu \left[k_1\, y^3\, (\gamma_r - \gamma_f) \cos \alpha - k\, k_2 y^2\, \gamma_f\, H) \right] . \tag{9}$$

Solving equation (9) for y

$$y = \frac{k\, k_2}{k_1} \quad \frac{\gamma_f\, H\, \mu}{(\mu \cos \alpha - \sin \alpha)\, (\gamma_r - \gamma_f)} \; . \tag{10}$$

Recalling that $W = k_1 y^3 \gamma_r$, and combining k , k_1 and k_2 ,

$$K' = \frac{k^3 {k_2}^3}{{k_1}^2} \; , \tag{11}$$

then by substitution,

$$W = \frac{K'\, {\gamma_f}^3\, \gamma_r H^3\, \mu^3}{(\mu \cos \alpha - \sin \alpha)^3\, (\gamma_r - \gamma_f)^3} \; . \tag{12}$$

Equation (12) is the revised and more general form of the original Iribarren formula. K' is dimensionless. The equation is dimensionally homogeneous and any consistent system of units can be used. This formula was used as a basis for correlating the results of the present investigation. The accuracy of the formula was tested by determining the values of H experimentally for known values of W, γ_r, γ_f, μ, and α.

Discussion of the General Formula

4. The equation for dynamic force (equation (3)) was formulated on the following assumptions: (1) $F_{dy} = k\ A\gamma_f \frac{2V^2}{2g}$ ($F = k\ A\ \rho \frac{V^2}{2}$ with $\rho g = \gamma$ gives $F_{dy} = k\ A\gamma_f \frac{V^2}{2g}$, and Iribarren argues that the force is twice this amount at the beginning and end of the splashes); (2) the velocity (V) of the jet is equal to the celerity (C) of the wave as it breaks; (3) $C^2 = gd$; and (4) the waves break on the slope in a depth equal to H, so that, by substitution, $V^2 = gH$.

5. Thus it can be seen that the dimensionless coefficient K' has incorporated in its characteristics the effects of any errors in the above assumptions as well as containing the drag and shape coefficients k, k_1 and k_2. The stability of a rock on the face slope of a rubble breakwater should be a function of the following variables: weight of individual cap rock (W), wave height (H), wave length (L), depth of water (d), specific weight of liquid in which the structure is situated (γ_f), specific weight of the cap rock (γ_r), position of the cap rock with respect to swl (z), angle of the breakwater slope measured from horizontal (α), angle of incidence of wave attack (ϕ), height of breakwater above swl (h), width of breakwater near swl (w), coefficient of friction of rock on rock including the effects of angularity (μ), shape factor of rock as it affects the drag coefficient ($\triangle_s$), and porosity or size of

voids between rock (V_s). In algebraic notation an equation of equilibrium for a cap rock on the seaside slope of a rubble mound subjected to wave action will contain expressions involving functions of the above variables. Thus,

$$K' = f\,(W,\ H,\ L,\ d,\ \gamma_f,\ \gamma_r,\ z,\ \alpha,\ \phi,\ h,\ w,\ \mu,\ \triangle_s\,,\ V_s)\,. \qquad (13)$$

The general form of Iribarren's formula (equation (12)) includes the following of the above variables: W, H, γ_f, γ_r, α and μ. If the basic assumptions of Iribarren were entirely correct, and the terms in the above expression (equation (13)) that are not included in Iribarren's formula have no appreciable effect on the stability of cap rock, it should be possible to show by small-scale tests that the coefficient K' is constant. Also, the numerical values of the constant should check very closely the values proposed by Iribarren (from dimensional considerations, and comparison of equations (1), (2) and (12) of this appendix, it can be shown that the values of K' corresponding to the original values of $K = 15$ and $K = 19$ are $K' = 0.015$ and 0.019, respectively) if they were originally chosen with sufficient accuracy. If the additional variables listed, other than those included in the Iribarren formula, do affect the stability of cap rock, or if the derived formula is not sufficiently adequate to describe accurately the phenomenon under consideration, it should be possible to prove by small-scale tests that K' is not constant but varies with some of the variables included in Iribarren's formula, as well as some of the variables listed in the above expression not included in Iribarren's formula. The tests reported in part V of this report were performed to determine the effects of several of the above listed variables on the stability of cap rock and on the values of K' in the general form of Iribarren's formula.

APPENDIX B: EFFECTS OF SPECIFIC WEIGHT ON STABILITY OF CAP ROCK

APPENDIX B: EFFECTS OF SPECIFIC WEIGHT ON STABILITY OF CAP ROCK

1. In the investigation to check the accuracy of Iribarren's formula it was desired to perform tests using rock of different specific weights varying over the range generally encountered in actual breakwater construction. The rock available for construction of a large proportion of rubble breakwaters ranges in specific weight from about 125 to 190 lb per cu ft. The only rock available for the small-scale tests weighed 175 lb per cu ft. Rock of other specific weights could have been obtained for use in the tests but the procuring of additional rock would have been costly and the work involved in hand-sizing individual cap rock would have been a tedious, time-consuming and expensive process. It was decided, therefore, to perform the tests by varying the specific weight of liquid in the wave tank instead of using rock of different specific weights. This necessitated the determination of the specific weight of liquid required to simulate the results of tests made with cap rock of different specific weights.

2. In many respects the problem is similar to that of determining the correct scales for a model in which the specific weight scale of the liquid must be other than 1:1. The problem posed is that of determining the specific weights of liquid that should be used in combination with rock of a given constant specific weight to insure that stability test results are the same as those that would be obtained if rock of different specific weights, and with water as the liquid, were used.

3. An undistorted model is required to maintain dynamic similarity with respect to the forces acting on the cap rock as a result of wave

action. Thus, the $\frac{H}{L}$ and $\frac{d}{L}$ ratios must remain constant for the different tests. If the tests are performed using only one wave period and one depth of water, the $\frac{d}{L}$ ratios will be constant. The $\frac{H}{L}$ ratio can then be made constant if $H_w = H_f$; where H_w is the design wave height for tests if water were used as the liquid, and H_f is the corresponding design wave height for identical test conditions except that a liquid of specific weight γ_f is used. In the tests contemplated the same cap rock would be used in each case. If the pertinent assumptions made by Iribarren are retained, the force tending to displace cap rock is

$$F_{dy} = k\ A\ \gamma_f\ H = k\ k_2 y^2\ \gamma_f\ H \tag{1}$$

with the same notation as that of appendix A. The force tending to resist displacement of cap rock is

$$F_r = \overline{V}_r\ \gamma_r - \overline{V}_r\ x\ \gamma_f \tag{2}$$

where $\overline{V}_r$ is the total volume of individual cap rock and x is a number, equal to or less than one, indicating the proportion of total volume of rock submerged in the liquid (Iribarren assumed that $x = 1$).

4. Equation (2) can be written as follows (see appendix A):

$$F_r = k_1 y^3\ (\gamma_r - \gamma_f\ x)\ . \tag{3}$$

The stability of each cap rock depends on the ratio of wave forces to resisting forces. Therefore, so long as the ratio $\frac{F_{dy}}{F_r}$ remains constant the stability test results will be the same. Thus, the desired basic equation is

$$\frac{F_{dy}}{F_r} = \frac{k\ k_2 y^2\ \gamma_f\ H}{k_1 y^3\ (\gamma_r - \gamma_f\ x)} = \text{constant}\ . \tag{4}$$

If the tests are performed using the same cap rock in each case, and with

$H_w = H_f$, then $(k)_w = (k)_f$, $(k_1)_w = (k_1)_f$, $(k_2)_w = (k_2)_f$, $y_w = y_f$ and $x_w = x_f$. The required relation, therefore, is

$$\frac{\gamma_w}{\gamma_{r(1)} - \gamma_w x} = \frac{\gamma_f}{\gamma_{r(2)} - \gamma_f x} \tag{5}$$

which reduces to

$$\frac{S_w}{S_{r(1)} - S_w x} = \frac{S_f}{S_{r(2)} - S_f x} \tag{6}$$

$S_{r(1)}$ is the specific gravity of the cap rock simulated, and $S_{r(2)}$ is the specific gravity of the cap rock used in the tests. For fresh water the specific gravity (S_w) is unity. Substituting $S_w = 1$, and solving for S_f

$$S_f = \frac{S_{r(2)}}{S_{r(1)}} \tag{7}$$

5. Note that the per cent submergence of the cap rock (x) does not affect the specific gravity of liquid required to simulate test results for rocks of different specific gravity. The validity of Iribarren's assumption relative to the submergence of cap rock was not studied in this investigation. However, cap rock not completely submerged is more stable than that submerged 100 per cent $(x = 1)$. Therefore, insofar as this particular assumption is concerned, Iribarren's formula is conservative.

6. It was desired to perform tests using 13.5-ton, 2.8-specific-gravity cap rock to simulate tests of apparent specific gravities of 2.2, 2.4 and 2.6. The different liquid specific gravities required to simulate these conditions were calculated from equation (7), and liquids with specific gravities of 1.27, 1.17 and 1.08 were used to simulate tests of 2.2, 2.4 and 2.6 specific-gravity cap rock, respectively. The

corresponding apparent weights of cap rock simulated were, to the nearest 0.2 ton, 10.6 tons, 11.5 tons, and 12.5 tons. The specific gravity of the model liquid was varied by the addition of appropriate amounts of calcium chloride to the water. The following approximate amounts of pellet calcium chloride were required to obtain liquid of the desired specific gravities:

Liquid Specific Gravity	Calcium Chloride Added in Lb per Cu Ft of Solution
1.00	0.0
1.08	9.0
1.17	19.0
1.27	31.0