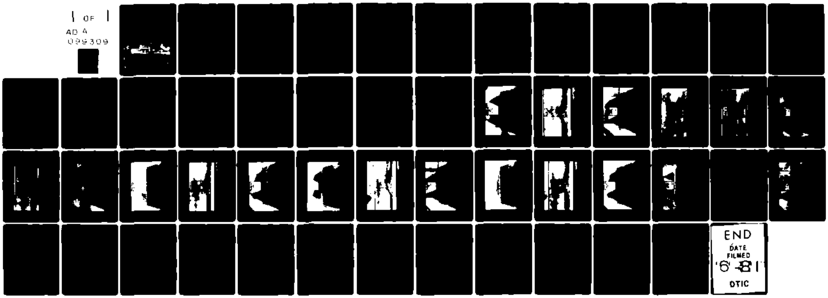
AD-A099 309 ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG--ETC F/G 13/2
RUBBLE-MOUND BREAKWATER STABILITY AND WAVE-ATTENUATION TESTS, P--ETC(U)
APR 81 R D CARVER, D G MARKLE
UNCLASSIFIED WES/TR/HL-81-5 NL



12

TECHNICAL REPORT HL-81-5

RUBBLE-MOUND BREAKWATER STABILITY AND WAVE-ATTENUATION TESTS PORT ONTARIO HARBOR, NEW YORK

Hydraulic Model Investigation

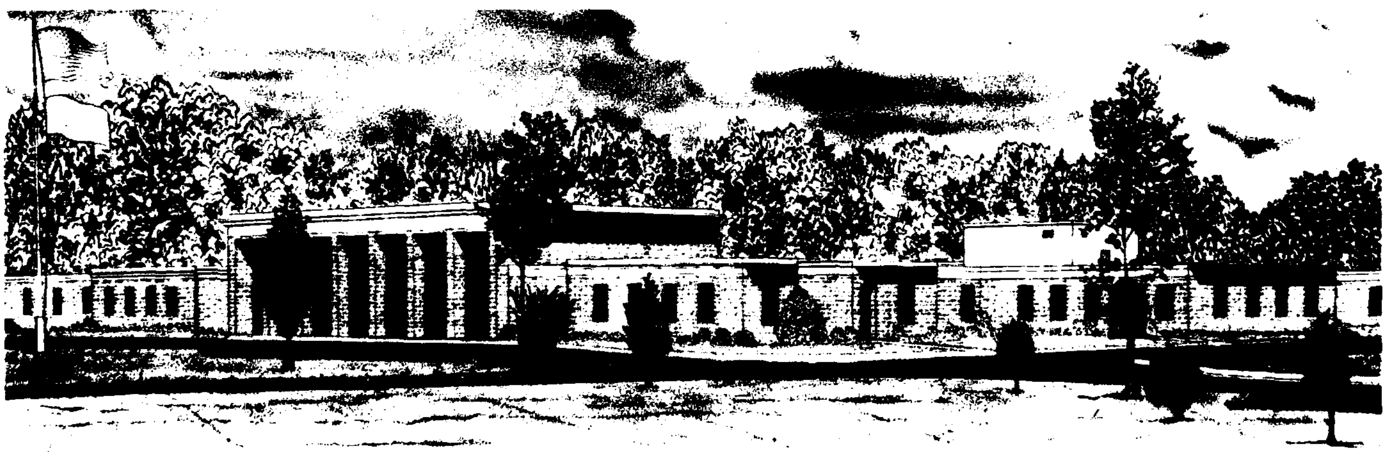
by

Robert D. Carver, Dennis G. Markle

Hydraulics Laboratory
U. S. Army Engineer Waterways Experiment Station
P. O. Box 631, Vicksburg, Miss. 39180

April 1981
Final Report

Approved For Public Release; Distribution Unlimited



Prepared for U. S. Army Engineer District, Buffalo
Buffalo, New York 14207

/TR/HL-81-

Unclassified
SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE

READ INSTRUCTIONS BEFORE COMPLETING FORM

1. REPORT NUMBER
Technical Report HL-81-5

2. GOVT ACCESSION NO.
AD-A099309

3. RECIPIENT'S CATALOG NUMBER

4. TITLE (and Subtitle)
RUBBLE-MOUND BREAKWATER STABILITY AND WAVE-ATTENUATION TESTS, PORT ONTARIO HARBOR, NEW YORK, Hydraulic Model Investigation.

5. TYPE OF REPORT & PERIOD COVERED
Final report

6. PERFORMING ORG. REPORT NUMBER

7. AUTHOR(s)
Robert D./Carver
Dennis G./Markle

8. CONTRACT OR GRANT NUMBER(s)

9. PERFORMING ORGANIZATION NAME AND ADDRESS
U. S. Army Engineer Waterways Experiment Station
Hydraulics Laboratory
P. O. Box 631, Vicksburg, Miss. 39180

10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS

11. CONTROLLING OFFICE NAME AND ADDRESS
U. S. Army Engineer District, Buffalo
1776 Niagara Street
Buffalo, N. Y. 14207

12. REPORT DATE
Apr[redacted]81

13. NUMBER OF PAGES
47

14. MONITORING AGENCY NAME & ADDRESS (if different from Controlling Office)
(12) 52

15. SECURITY CLASS. (of this report)
Unclassified

15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

Approved for public release; distribution unlimited.

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)
Hydraulic models
Port Ontario Harbor
Rubble-mound breakwaters
Water wave attentuation

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

An undistorted-scale hydraulic model study was conducted to investigate the armor stability and wave-transmission design of three breakwater cross sections for Port Ontario Harbor, New York. Plan 1 was constructed to a crown elevation of +10 ft lwd and used armor slopes of 1V on 2H and 1V on 1.5H lake-side and harbor side, respectively. A crown width of 16 ft, equivalent to three armor-stone diameters, was used and the slopes and crown were armored

(Continued)

DD FORM 1 JAN 73 1473 EDITION OF 1 NOV 65 IS OBSOLETE

Unclassified
SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

411384

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE(When Data Entered)

20. ABSTRACT (Continued).

with two thicknesses of 7.8-ton stone. Plan 1A was the same as Plan 1 except that the crown elevation was lowered to +9 ft lwd. Plan 2 was similar to Plan 1 except that the armor weight was reduced to 5.3 tons and the crown width was narrowed to 14 ft. Based on results of model tests, it was concluded that Plans 1 and 2 meet the designated wave-transmission criteria of significant transmitted wave height $\leq$ 3.0 ft and are stable designs for the maximum breaking wave heights that can be produced in the model for 7- to 11-sec waves at swl's of +1.0 and +4.6 ft lwd. Plan 1 exhibited the best stability response of all three plans investigated. Maximum significant transmitted wave heights were 2.5, 3.0 and 2.4 ft for Plans 1, 1A, and 2, respectively.

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE(When Data Entered)

PREFACE

The model investigation reported herein was requested by the U. S. Army Engineer District, Buffalo (NCB), in a letter to the U. S. Army Engineer Waterways Experiment Station (WES) dated 2 May 1980. Funding authorization was granted by NCB on Intra-Army Order No. NCB-IA-80-58JD, dated 27 May 1980.

Model tests were conducted at WES during the period July 1980 to September 1980, under the general direction of Mr. H. B. Simmons, Chief of the Hydraulics Laboratory, and Dr. R. W. Whalin, Chief of the Wave Dynamics Division. Tests were conducted by Messrs. R. D. Carver and D. G. Markle, Research Hydraulic Engineers; Mr. M. S. Taylor, Engineering Technician; and Mrs. B. J. Wright, Engineering Aid. Execution of this study and preparation of this report was performed by Messrs. Carver and Markle under the supervision of Mr. D. D. Davidson, Chief of the Wave Research Branch.

Liaison between NCB and WES was maintained during the course of the investigation by telephone communication and progress reports.

Commander and Director of WES during the conduct of the study and the preparation and publication of this report was COL Nelson P. Conover, CE. Techincal Director was Mr. F. R. Brown.

CONTENTS

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
feet	0.3048	metres
miles (U. S. statute)	1.609344	kilometres
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
tons (2000 lb mass)	907.1847	kilograms

RUBBLE-MOUND BREAKWATER STABILITY AND WAVE-ATTENUATION TESTS, PORT ONTARIO HARBOR, NEW YORK

Hydraulic Model Investigation

PART I: INTRODUCTION

Background

1. Port Ontario Harbor, New York, is situated at the mouth of the Salmon River, about 20 miles* south of the entrance to Henderson Bay, on the eastern shore of Lake Ontario (Plate 1). The area tributary to Port Ontario Harbor is principally recreational and agricultural with the village of Port Ontario, 1 mile upstream, catering to summer vacationers. At present, a constantly shifting sand and cobble bar, caused mainly by littoral drift due to wave action, poses numerous navigation problems.

2. A plan of improvement for Port Ontario Harbor based on hydraulic model tests,** fiscal considerations, and local interests has been formulated by the U. S. Army Engineer District, Buffalo (NCB). Proposed improvements include dredging an entrance channel which will be protected by a rubble-mound breakwater. Design constraints dictate that the crest elevation of the breakwater not exceed +10 ft low water datum (lwd); therefore, it is anticipated that major wave overtopping will occur and may cause instability of the breakwater's back slope. Design criteria require that the significant transmitted waves do not exceed 3.0 ft.

Purpose of Model Study

3. The original purpose of the model study was to experimentally

---

* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 3.

** R. R. Bottin, Jr. 1977 (Nov). "Port Ontario Harbor, New York, Design for Wave Protection and Prevention of Shoaling; Hydraulic Model Investigation," Technical Report H-77-20, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

investigate the armor stability and wave transmission properties of a breakwater section proposed by NCB for use at Port Ontario Harbor. Plan 1 used 7.8-ton armor stone, a crown width of 16 ft, and armor slopes of 1V on 2H and 1V on 1.5H lakeside and harbor side, respectively. Later, following completion of tests for Plan 1, it was decided to investigate two alternate plans in an attempt to reduce construction costs for the breakwater.

PART II: THE MODEL

Design of Model

4. Tests were conducted at an undistorted linear scale of 1:28, model to prototype. Scale selection was based on the size of model armor units available compared with the estimated size of prototype armor units required for stability, the elimination of stability scale effects,* and capabilities of the available wave tank. Based on Froude's model law** and the linear scale of 1:28, the following model-prototype relations were derived. Dimensions are in terms of length (L) and time (T).

Characteristics	Dimensions	Model:Prototype Scale Relations
Length	L	$L_r = 1:28$
Area	L^2	$A_r = L_r^2 = 1:784$
Volume	L^3	$V_r = L_r^3 = 1:21,952$
Time	T	$T_r = L_r^{1/2} = 1:5.29$

5. The specific weight of water used in model tests was assumed to be the same as the prototype and equal to 62.4 pcf. However, specific weights of model breakwater construction materials were not the same as their prototype counterparts. These variables were related using the following transference equation:

$$\frac{(W_r)_m}{(W_r)_p} = \frac{(\gamma_r)_m}{(\gamma_r)_p}\left(\frac{L_m}{L_p}\right)^3\left[\frac{(S_r)_p - 1}{(S_r)_m - 1}\right]^3$$

---

* R. Y. Hudson. 1975 (Jun). "Reliability of Rubble-Mound Breakwater Stability Models," Miscellaneous Paper H-75-5, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

** J. C. Stevens et al. 1942. "Hydraulic Models," Manuals on Engineering Practice No. 25, American Society of Civil Engineers, New York, N. Y.

where subscripts m and p represent model and prototype quantities, respectively, and

W_r = weight of an individual armor unit or stone, lb

γ_r = specific weight of an individual armor unit or stone, pcf

L_m/L_p = linear scale of the model

S_r = specific gravity of an individual armor unit or stone relative to the water in which the breakwater is constructed, i.e., $S_r = \gamma_r/\gamma_w$ where γ_w = the specific weight of water, pcf

Test Equipment

6. Tests were conducted in a portion of an L-shaped concrete flume (100 ft long, 5 ft wide, and 3 ft deep) which has overall dimensions of 250 ft long, 50 and 80 ft wide at the top and bottom of the L , respectively, and 4.5 ft deep. The flume layout is shown in Plate 2. A 44-ft (model) length of 1V-on-50H slope, representative of the existing prototype sea bottom, was molded and test sections were installed 20 ft beachward of the slope's toe. The test facility is equipped with a flap-type wave generator, capable of producing sinusoidal waves of various periods and heights. Tests waves of the required characteristics were generated by varying the frequency and amplitude of the plunger motion. Changes in water-surface elevation, as a function of time, were measured by electrical wave-height gages in the vicinity of where the toe of the test section was to be placed and recorded on chart paper by an electrically operated oscillograph. Wave-generator calibration, without the test section in place, simulated existing conditions.

PART III: TESTS AND RESULTS

Method of Constructing Test Sections

7. Model breakwater sections were constructed to reproduce as closely as possible results of the usual methods of constructing prototype structures. Core material, dampened as it was dumped by bucket or shovel into the flume, was compacted with hand trowels to simulate natural consolidation resulting from wave action during construction of the prototype breakwater. Once the core material was in place, it was sprayed with a low-velocity water hose to ensure adequate compaction of the material. Underlayer stone then was added by shovel and smoothed to grade by hand or with trowels but was not packed in place. Armor units used in the cover layer were placed by hand in a random manner (i.e., laid down in such a way that no intentional interlocking of the units was obtained). Model elevations were controlled with an engineer's level to a tolerance of ±0.005 ft.

Description of Plan 1

8. Plan 1 (Plate 3 and Photos 1-3) was constructed to a crown elevation of +10 ft lwd and used armor slopes of 1V on 2H and 1V on 1.5H lakeside and harbor side, respectively. A crown width of 16 ft, equivalent to three armor-stone diameters, was used; and the slopes and crown were armored with two thicknesses of 7.8-ton stone. A 10-ft-wide berm of 1560-lb underlayer stone was placed at the toe of both the lakeside and harbor-side slopes.

Selection of Test Conditions

9. Based on anticipated prototype wave conditions,* it was

* D. T. Resio and C. L. Vincent. 1976 (Mar). "Design Wave Information for the Great Lakes; Lake Ontario," Technical Report H-76-1, Report 2, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

decided that the stability tests should consider 7-, 9-, and 11-sec waves at still water levels (swl's) of +1.0 and +4.6 ft lwd. Model observations indicated that for the selected wave periods and swl's, the corresponding maximum breaking wave was always more severe than any lesser wave height. Observations of incident wave forms at the structure showed the most severe breaking wave conditions that experimentally could be made to attack the section for the selected conditions were as follows:

Swl ft lwd	Wave Period sec	Most Severe Breaking Wave Height, ft
+1.0	7.0	6.1
+1.0	9.0	6.8
+1.0	11.0	7.0
+4.6	7.0	7.9
+4.6	9.0	9.3
+4.6	11.0	9.8

It was decided that for the range of wave conditions considered, the stability response of the proposed test sections could be adequately evaluated by subjecting the structure to the following storm-surge hydrograph:

		Test Wave (Prototype Dimensions)			
Step	Swl ft lwd	Period, sec	Height, ft	Prototype Duration, hr	Wave Type
	+1.0	7.0	3.0	0.33	Shakedown
1	+1.0	7.0	6.1	0.50	Severe breaking
2	+1.0	9.0	6.8	0.50	Severe breaking
3	+1.0	11.0	7.0	1.00	Severe breaking
4	+4.6	7.0	7.9	0.50	Severe breaking
5	+4.6	9.0	9.3	0.50	Severe breaking
6	+4.6	11.0	9.8	1.00	Severe breaking
7	+1.0	7.0	6.1	0.50	Severe breaking
8	+1.0	9.0	6.8	0.50	Severe breaking
9	+1.0	11.0	7.0	1.00	Severe breaking

10. Wave-attenuating capabilities of the proposed breackwater sections also were investigated. In these tests, transmitted wave heights were measured at distances of one-half wavelength (L/2) and one wavelength (L) behind the center line of the breakwater for 7-, 9-, and 11-sec waves at an swl of +4.6 ft lwd. Incident wave heights ranged from 5 ft up to the maximum depth-limited waves that could reach the

structure for the selected wave periods and swl.

Test Results of Plan 1

11. Plan 1 exhibited an excellent stability response. Moderate wave overtopping, present during steps 4, 5, and 6 of the test hydrograph, caused minor rocking of a few 7.8-ton, harbor-side armor units; however, no displacement damage occurred. The 1560-lb, toe-protection stone proved to be stable. Photo 4 shows an 11-sec, 9.8-ft wave (step 6) impinging on the breakwater; and Photo 5 shows the wave overtopping the structure. Photos 6-8 show the structure after exposure to the hydrograph. Stability test results of Plan 1 were verified by a complete reconstruction and retesting.

12. Wave-attenuation test results are presented in Table 1 and Plates 4 and 5. The values presented therein are the measured significant wave heights. Detailed analysis of the wave records showed that for a given test condition, the maximum transmitted wave height never exceeded the significant transmitted wave height by more than 10 percent. In general, the data show that if the incident wave height were held constant and the wave period were increased, transmitted heights generally increased; and transmitted wave heights measured at L/2 were slightly larger than those observed at a distance L behind the breakwater. High-frequency disturbances (which quickly dissipated), produced by incident waves overtopping and transmitting through the porous armor stone, probably accounted for the slightly larger transmitted heights observed one-half wavelength behind the breakwater. Typical transmitted wave forms are illustrated in Plate 6.

Rationale and Description of Plans 1A and 2

13. Based on the excellent stability response of Plan 1, it was decided to investigate alternative schemes that might substantially reduce the structure's cost without significantly affecting its functional performance. Some of the factors that govern material volumes and costs

are elevation and width of the crown, type and weight of armor, and slope on which the armor is placed. Based on discussions between NCB and U. S. Army Engineer Waterways Experiment Station, it was decided that in this particular study the greatest cost savings with the least probable impact on functionality could probably be achieved by either lowering the crown elevation or reducing the armor weight. Thus, two additional plans (Plans 1A and 2) were selected for testing. Plan 1A (Plate 3 and Photos 9-11) was the same as Plan 1 except that the crown elevation was lowered to +9 ft lwd. Plan 2 (Plate 7 and Photos 12-14) was similar to Plan 1 except that the armor weight was reduced to 5.3 tons and the crown width was narrowed to 14 ft. Also, the underlayer and toe-protection stone weight was proportionately reduced to 1060 lb.

Test Results of Plans 1A and 2

14. Plan 1A demonstrated a good stability response. Relative to Plan 1, increased wave overtopping was observed during steps 4, 5, and 6 of the test hydrograph. Attack of 11-sec, 9.8-ft waves (step 6) displaced one harbor-side armor unit downslope; however, this displacement had no effect on the overall stability of the breakwater. Photos 15-17 show the after-testing condition of the structure.

15. Plan 2 also demonstrated a good stability response. No armor movement was detected for any of the low-water conditions (hydrograph steps 1, 2, 3, 7, 8, and 9); however, rocking of a few 5.3-ton armor units was observed during hydrograph steps 4, 5, and 6. Attack of 11-sec, 9.8-ft waves (step 6) displaced two lakeside armor units onto the 1060-lb toe-protection stone. This displacement had no impact on the overall integrity of the breakwater, and the final stabilized condition was deemed to be completely acceptable. Photos 18-20 show the breakwater after completion of the hydrograph.

16. The 1560-lb and 1060-lb toe-protection stone used in Plans 1A and 2, respectively, proved to be stable. Stability test results of both Plans 1A and 2 were verified by complete reconstructions and retestings.

17. Wave-attenuation test results (significant transmitted wave

heights) are presented in Tables 2 and 3 and Plates 8-11. These data show the same general trends as those observed with Plan 1. If the incident wave height were held constant and the wave period increased, transmitted heights increased; and transmitted wave heights measured at L/2 were slightly larger than those observed at a distance L behind the breakwater. For a given test condition, the maximum transmitted wave height never exceeded the significant transmitted wave height by more than 10 percent. Relative to Plan 1, Plan 1A showed slightly increased wave transmission and Plan 2 showed slightly decreased wave transmission. These results seem very reasonable since Plan 1A was the same as Plan 1 except that the crown elevation was lowered 1 ft, and Plan 2 was similar to Plan 1 except that the reduced armor weight and slightly higher core tended to decrease the permeability of the breakwater.

PART IV: CONCLUSIONS

18. Based on assumptions, tests, and results reported herein, it is concluded that:

a. Plans 1, 1A, and 2 are stable designs for the maximum breaking wave heights that can be expected to occur for 7- to 11-sec waves at swl's of +1.0 and +4.6 ft lwd.

b. Plan 1 exhibited the best stability response of all plans investigated.

c. Maximum significant transmitted wave heights were 2.5, 3.0, and 2.4 ft for Plans 1, 1A, and 2, respectively.

Table 1

Values of Incident and Significant Transmitted Wave Heights for Plan 1; swl = +4.6 ft lwd

Incident Wave Height,* H_i, ft	Significant Transmitted Wave Height, H_t, ft Measured at** L/2	L	Incident Wave Form
	7-sec Wave Period		
5.0	1.3	1.2	Nonbreaking
6.0	1.6	1.4	Nonbreaking
7.0	1.7	1.5	Nonbreaking
7.9	1.9	1.7	Breaking
	9-sec Wave Period		
5.0	1.3	1.2	Nonbreaking
6.0	1.8	1.4	Nonbreaking
7.0	1.9	1.6	Nonbreaking
8.0	2.1	1.9	Nonbreaking
9.0	2.3	2.0	Nonbreaking
9.3	2.3	2.0	Breaking
	11-sec Wave Period		
5.0	1.5	1.4	Nonbreaking
6.0	1.8	1.7	Nonbreaking
7.0	2.0	1.9	Nonbreaking
8.0	2.2	2.1	Nonbreaking
9.0	2.4	2.2	Nonbreaking
9.8	2.5	2.3	Breaking

* Measured at toe of structure without structure in place.

** Measured distance in wavelengths from center line of structure.

Table 2

Values of Incident and Significant Transmitted Wave Heights for Plan 1A; swl = +4.6 ft lwd

Incident Wave Height,* H_i, ft	Significant Transmitted Wave Height, H_t, ft Measured at** L/2	L	Incident Wave Form
	7-sec Wave Period		
5.0	1.6	1.5	Nonbreaking
6.0	1.9	1.7	Nonbreaking
7.0	2.1	2.0	Nonbreaking
7.9	2.3	2.2	Breaking
	9-sec Wave Period		
5.0	1.7	1.6	Nonbreaking
6.0	2.0	1.8	Nonbreaking
7.0	2.2	2.0	Nonbreaking
8.0	2.3	2.2	Nonbreaking
9.0	2.5	2.3	Nonbreaking
9.3	2.6	2.3	Breaking
	11-sec Wave Period		
5.0	1.8	1.8	Nonbreaking
6.0	2.1	2.0	Nonbreaking
7.0	2.3	2.1	Nonbreaking
8.0	2.6	2.3	Nonbreaking
9.0	2.8	2.5	Nonbreaking
9.8	3.0	2.8	Breaking

* Measured at toe of structure without structure in place.

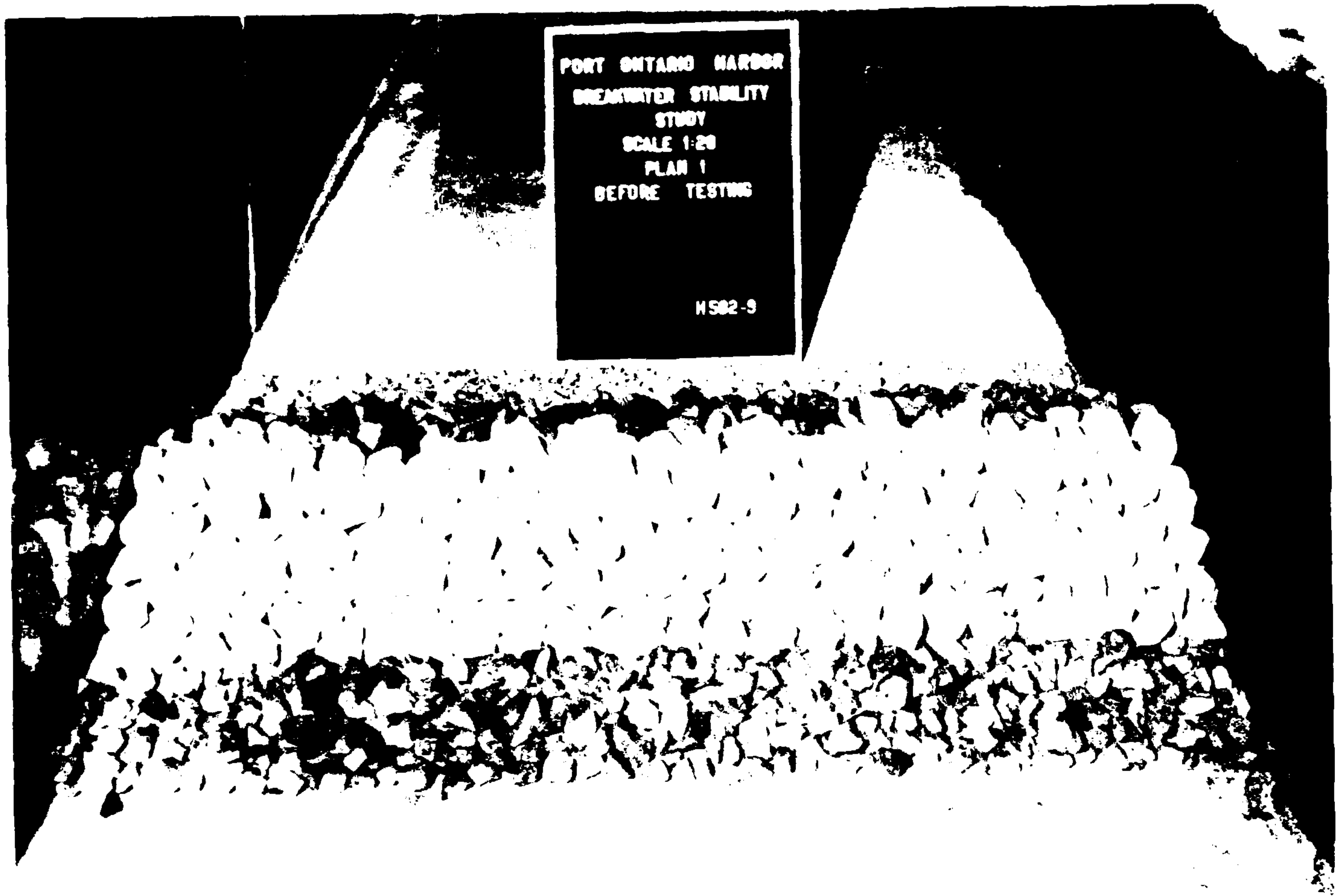
** Measured distance in wavelengths from center line of structure.

Table 3

Values of Incident and Significant Transmitted Wave Heights for Plan 2; swl = +4.6 ft lwd

Incident Wave Height,* H_i, ft	Significant Transmitted Wave Height, H_t, ft — Measured at** L/2	Significant Transmitted Wave Height, H_t, ft — Measured at** L	Incident Wave Form
	7-sec Wave Period		
5.0	1.1	1.0	Nonbreaking
6.0	1.4	1.2	Nonbreaking
7.0	1.5	1.4	Nonbreaking
7.9	1.6	1.5	Breaking
	9-sec Wave Period		
5.0	1.2	1.1	Nonbreaking
6.0	1.5	1.3	Nonbreaking
7.0	1.8	1.5	Nonbreaking
8.0	2.0	1.7	Nonbreaking
9.0	2.1	2.0	Nonbreaking
9.3	2.2	2.0	Breaking
	11-sec Wave Period		
5.0	1.4	1.4	Nonbreaking
6.0	1.7	1.7	Nonbreaking
7.0	1.9	1.9	Nonbreaking
8.0	2.1	2.0	Nonbreaking
9.0	2.2	2.2	Nonbreaking
9.8	2.4	2.4	Breaking

* Measured at toe of structure without structure in place.
** Measured distance in wavelengths from center line of structure.



[illegible] Plan 1 before wave attack

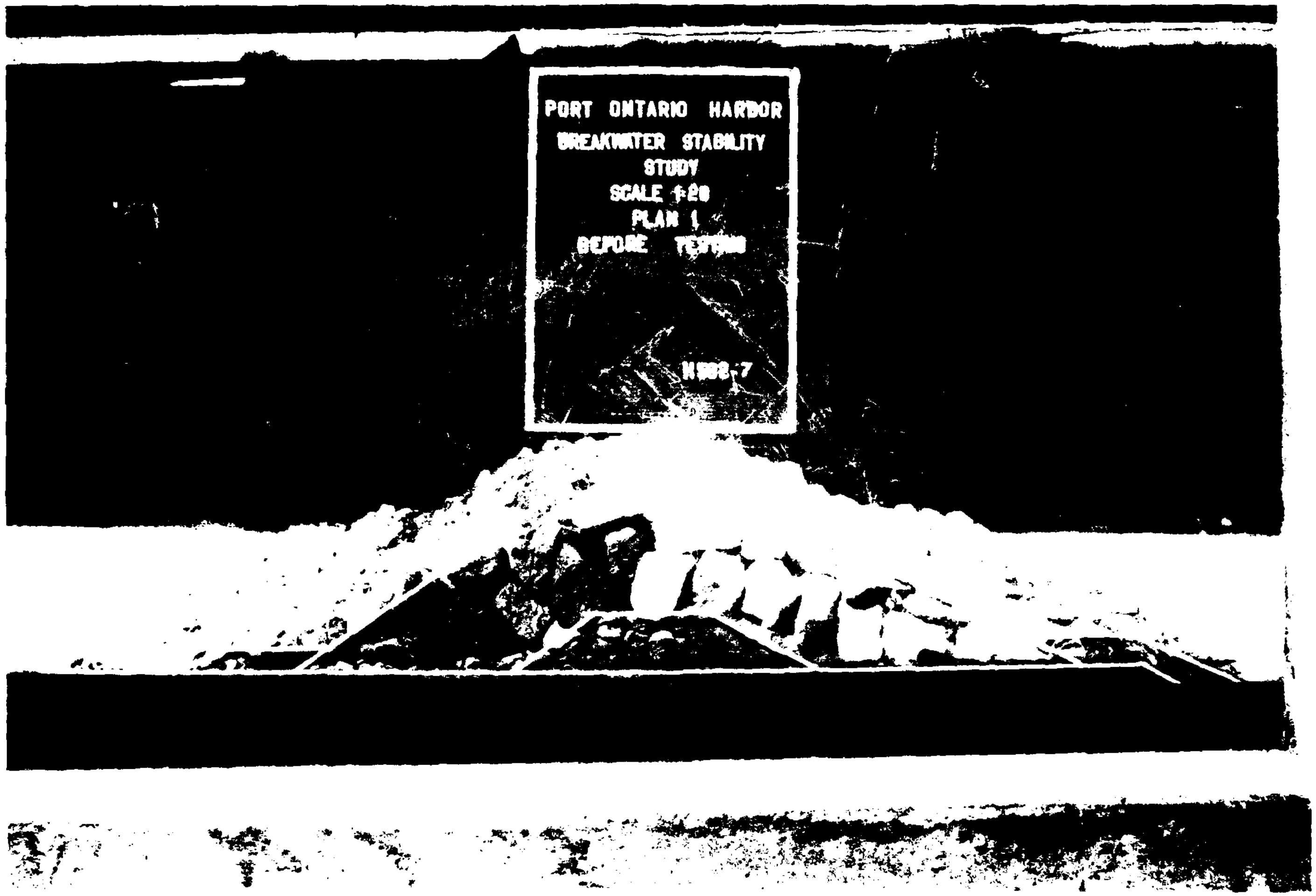


Photo 2. Side view of Plan 1 before wave attack

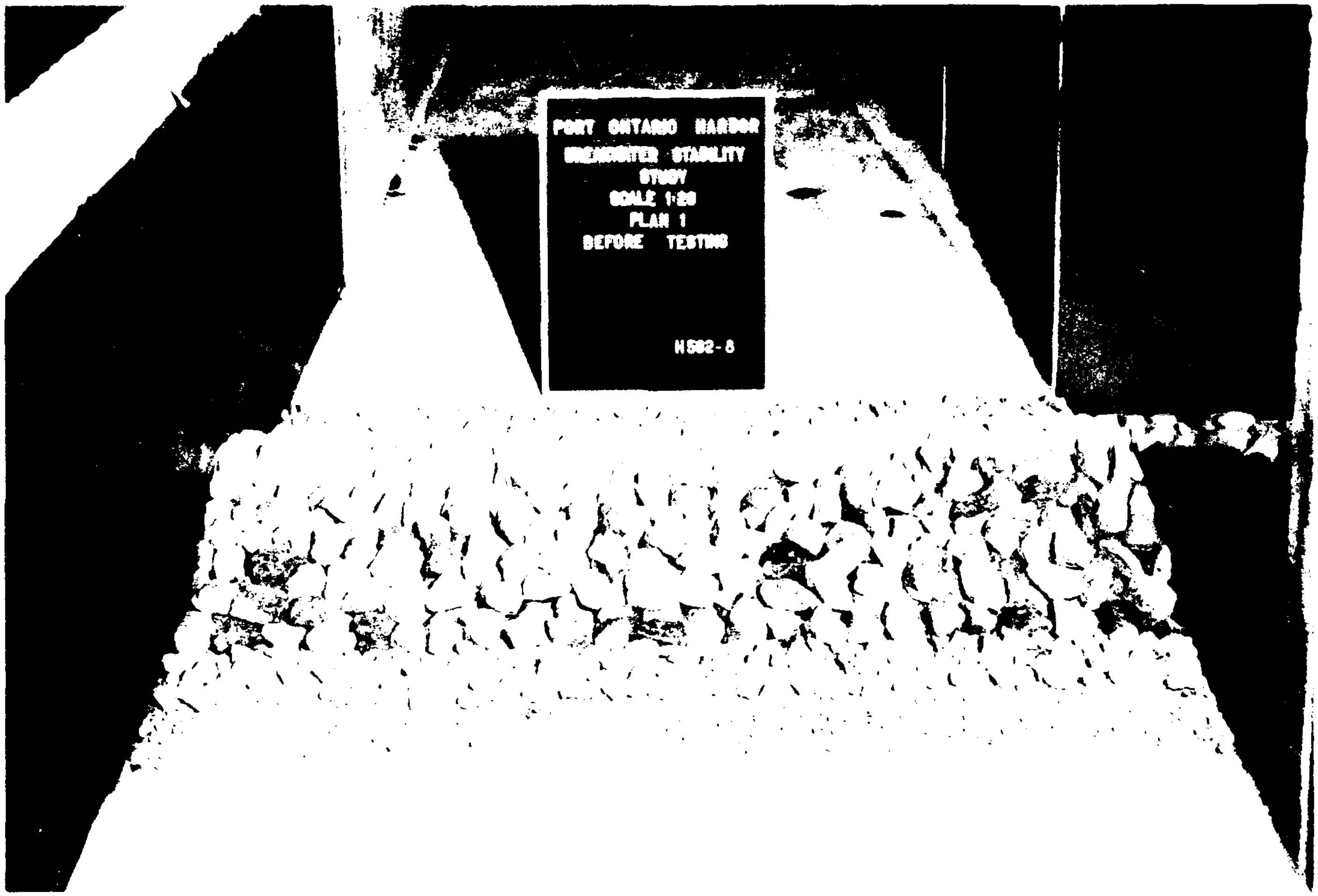


Photo 3. Harbor-side view of Plan 1 before wave attack

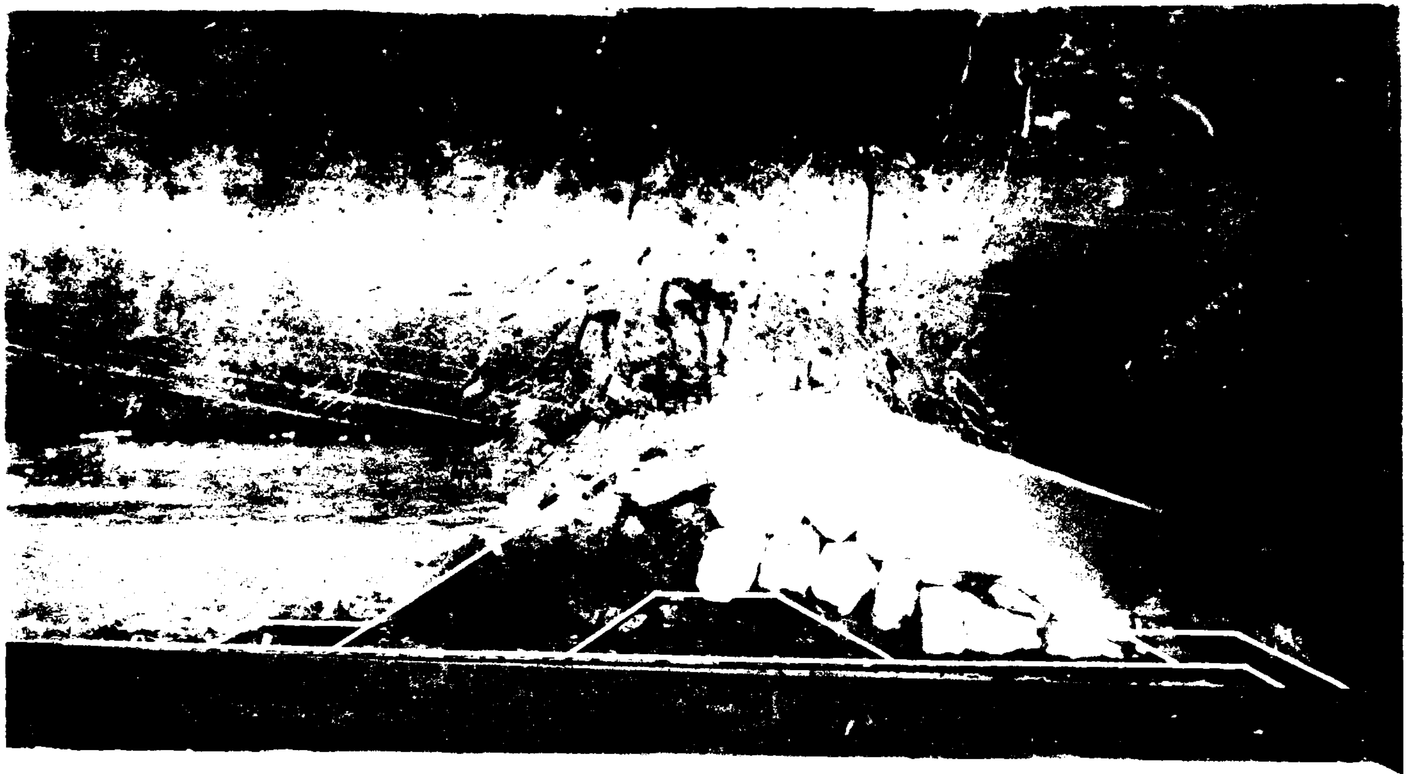


Photo 4. Side view of an 11-sec, 9.8-ft wave impinging on Plan 1

Photo 5. Side view of Plan 1 being overtopped by an 11-sec, 9.8-ft wave

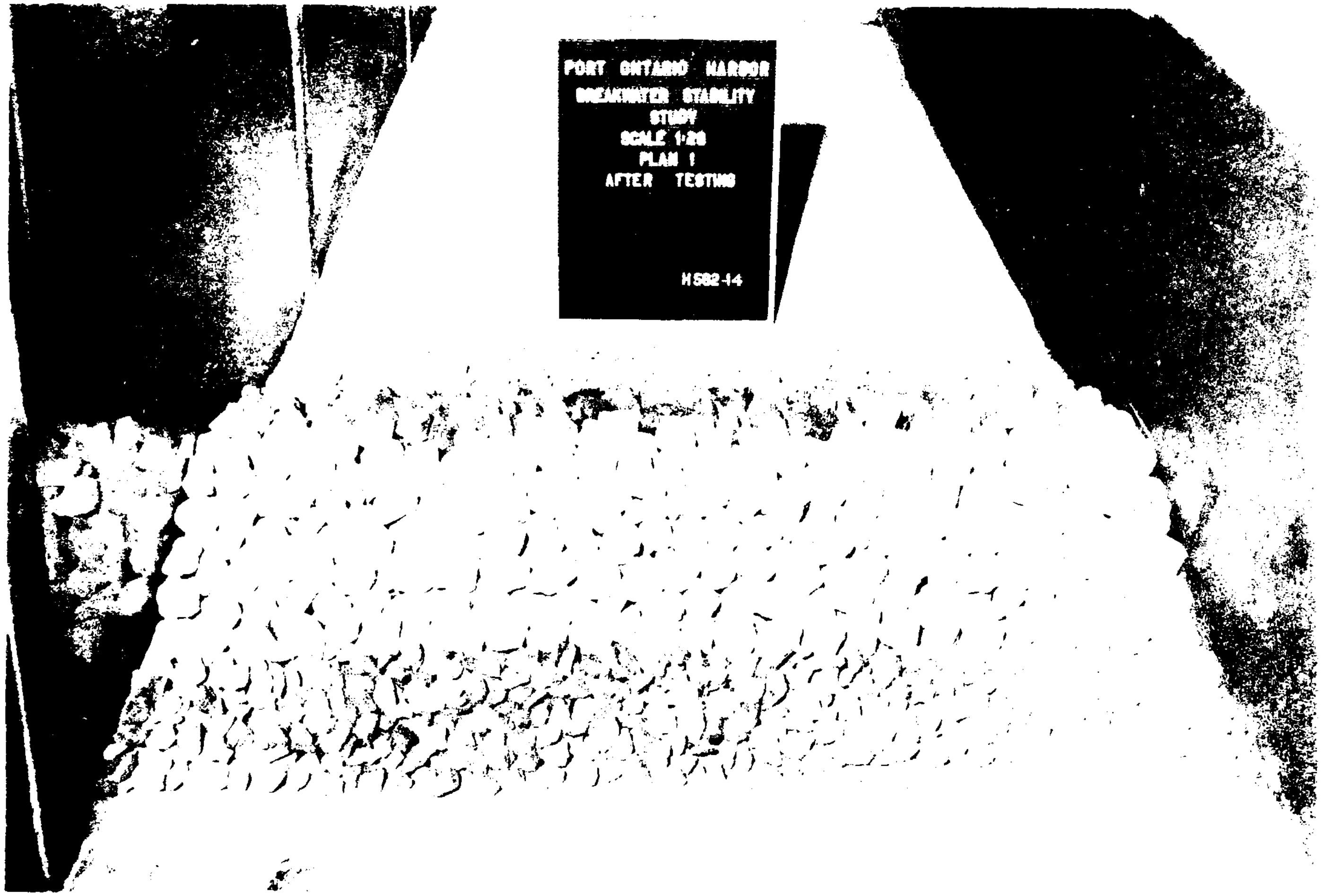


Photo 6. Lakeside view of Plan 1 after completion of storm-surge hydrograph

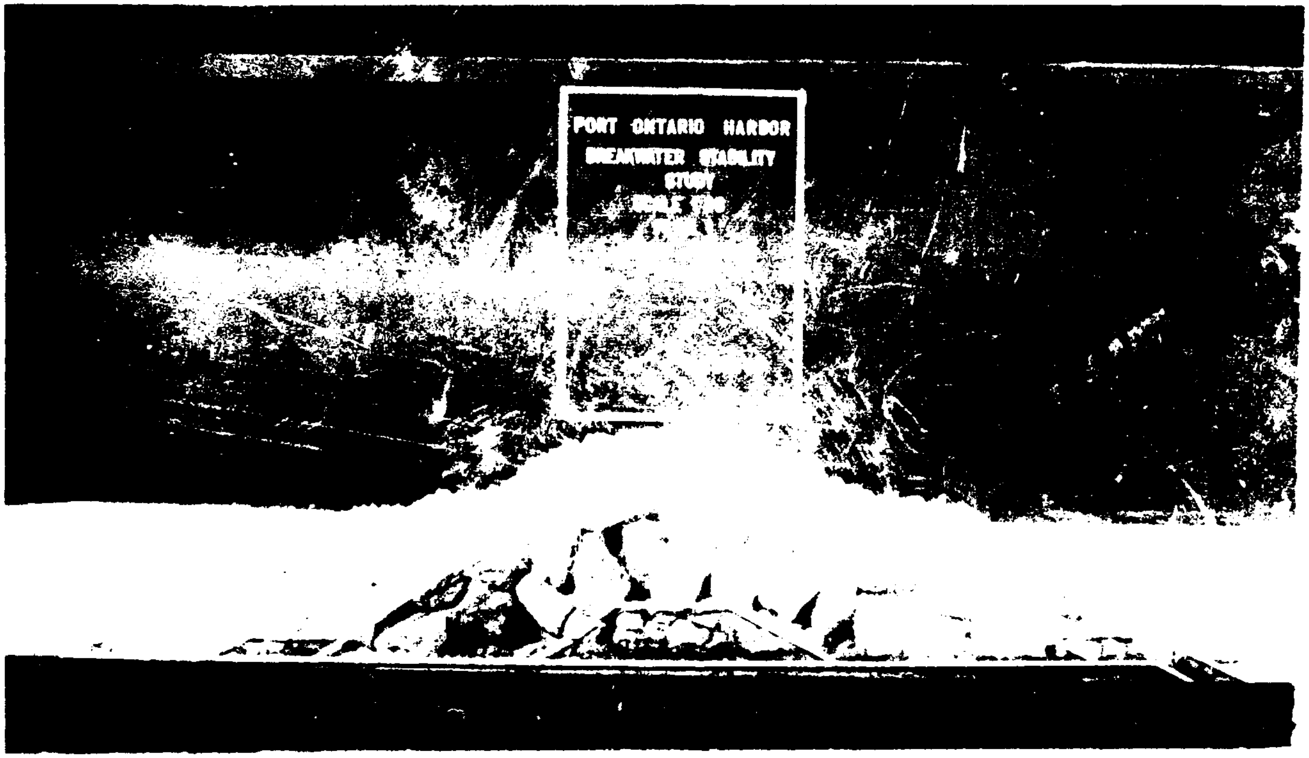


Photo 7. Side view of Plan 1 after completion of storm-surge hydrograph

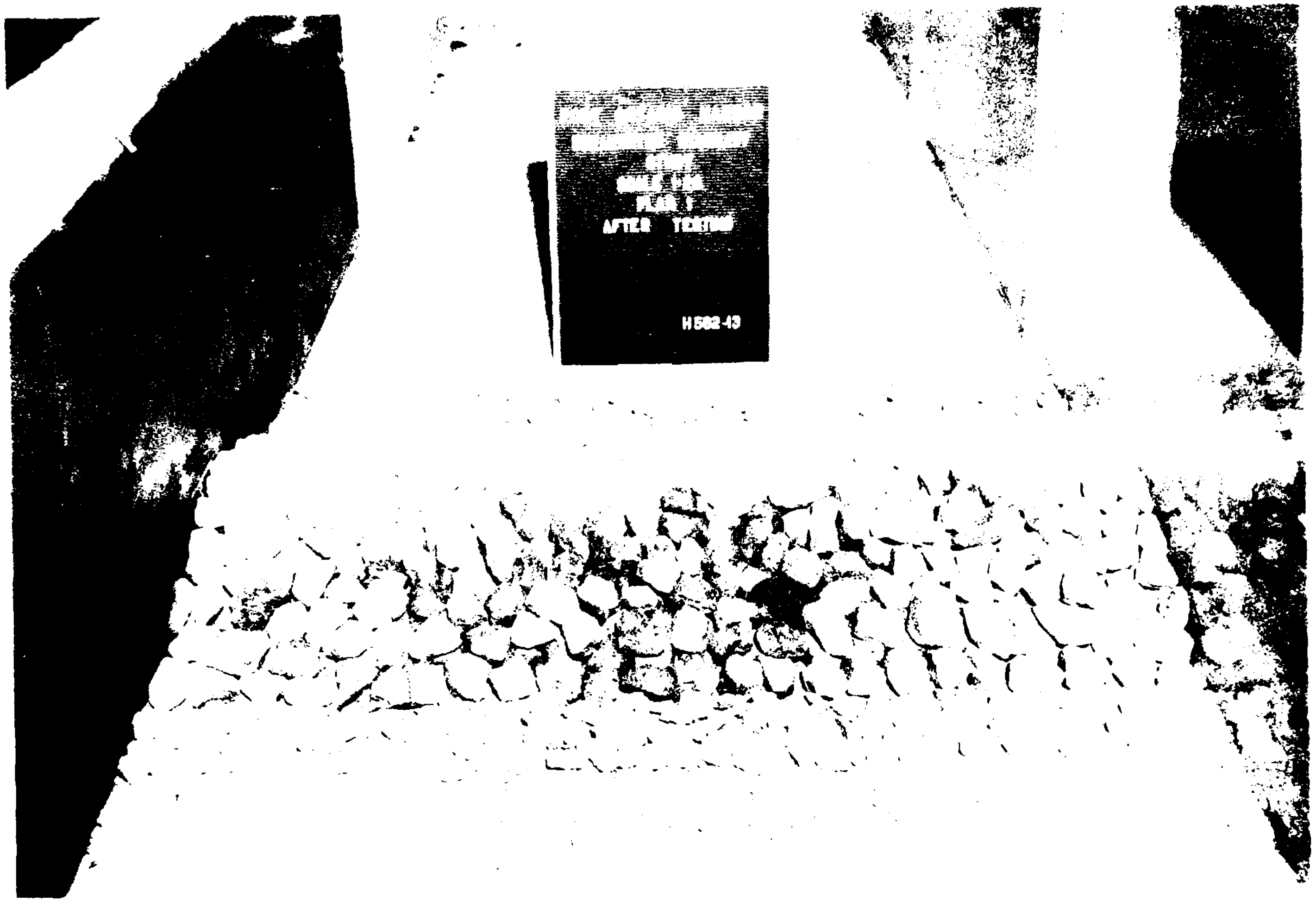


Photo 8. Harbor-side view of Plan 1 after completion of storm-surge hydrograph

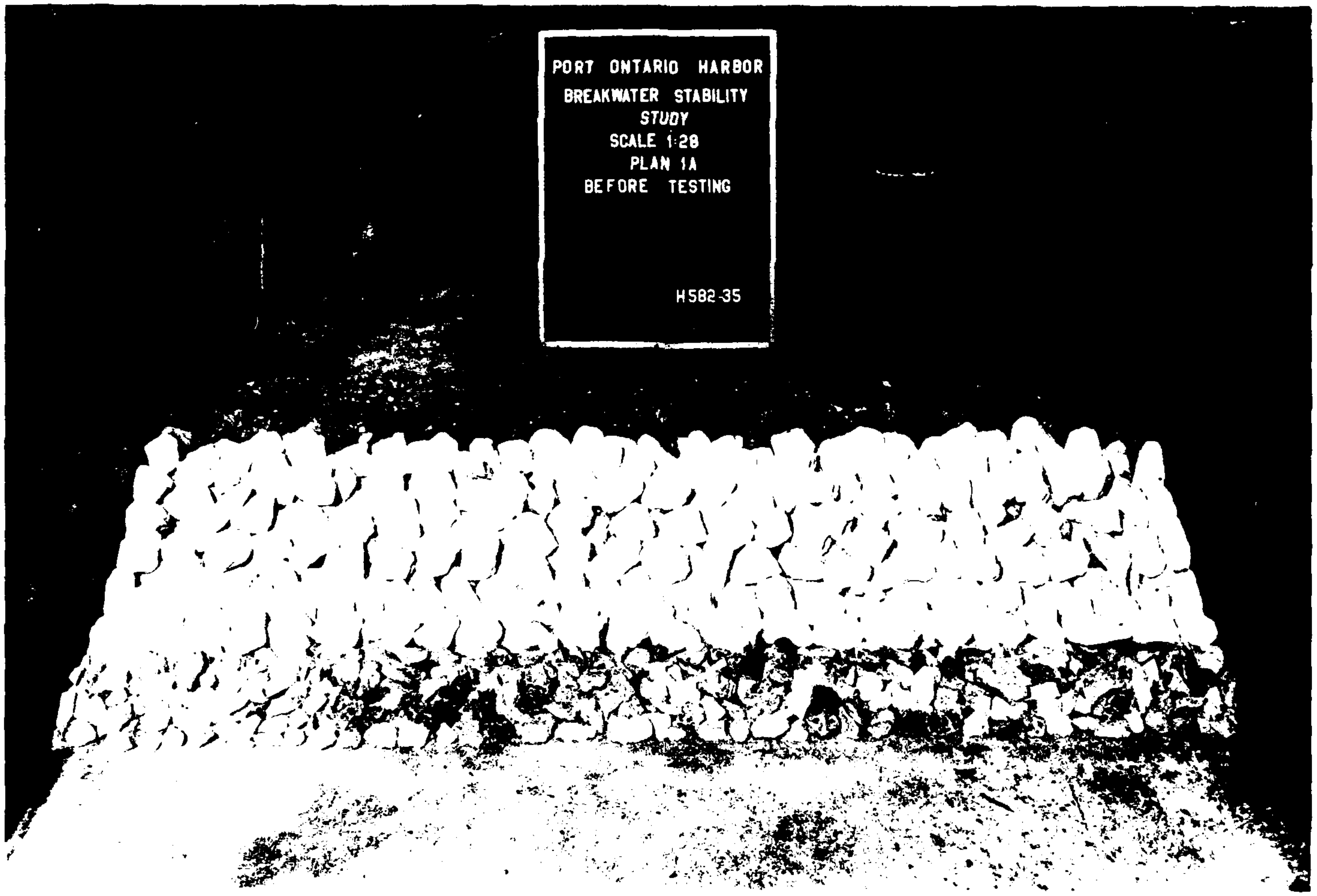


Photo 9. Lakeside view of Plan 1A before wave attack

Photo 10. Side view of Plan 1A before wave attack

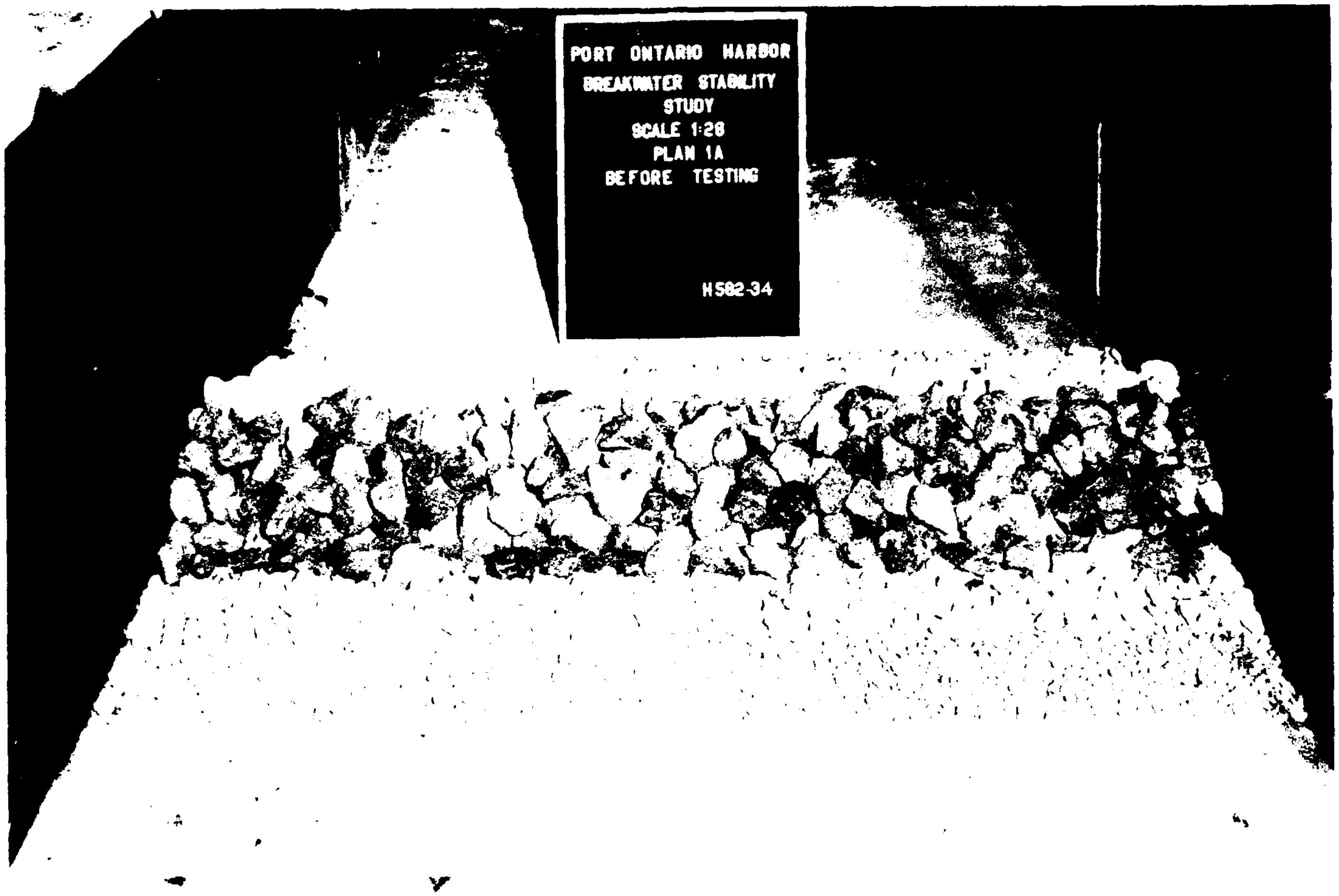


Photo 11. Harbor-side view of Plan 1A before wave attack

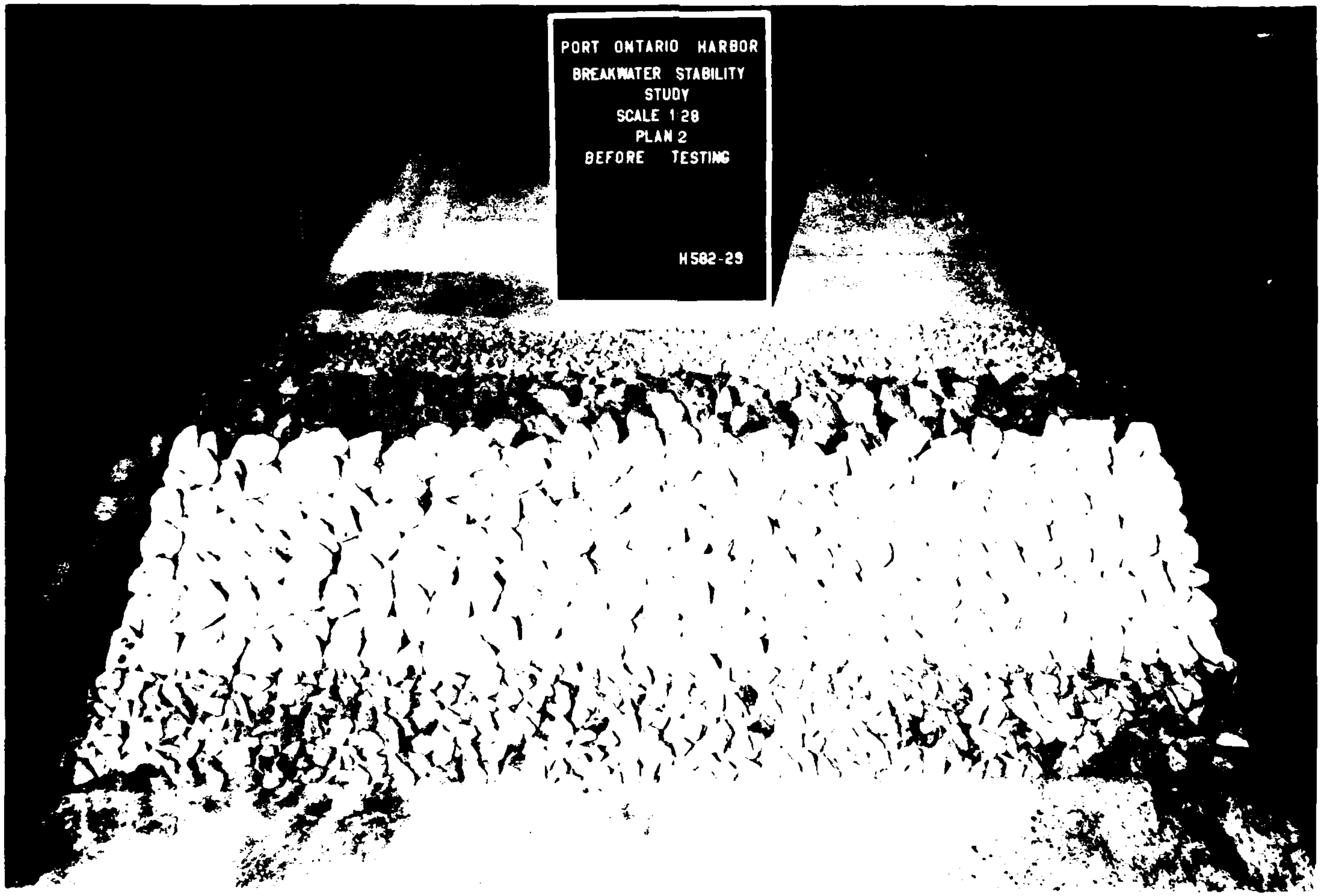


Photo 12. Lakeside view of Plan 2 before wave attack

Photo 16. Side view of Plan 2 before wave attack

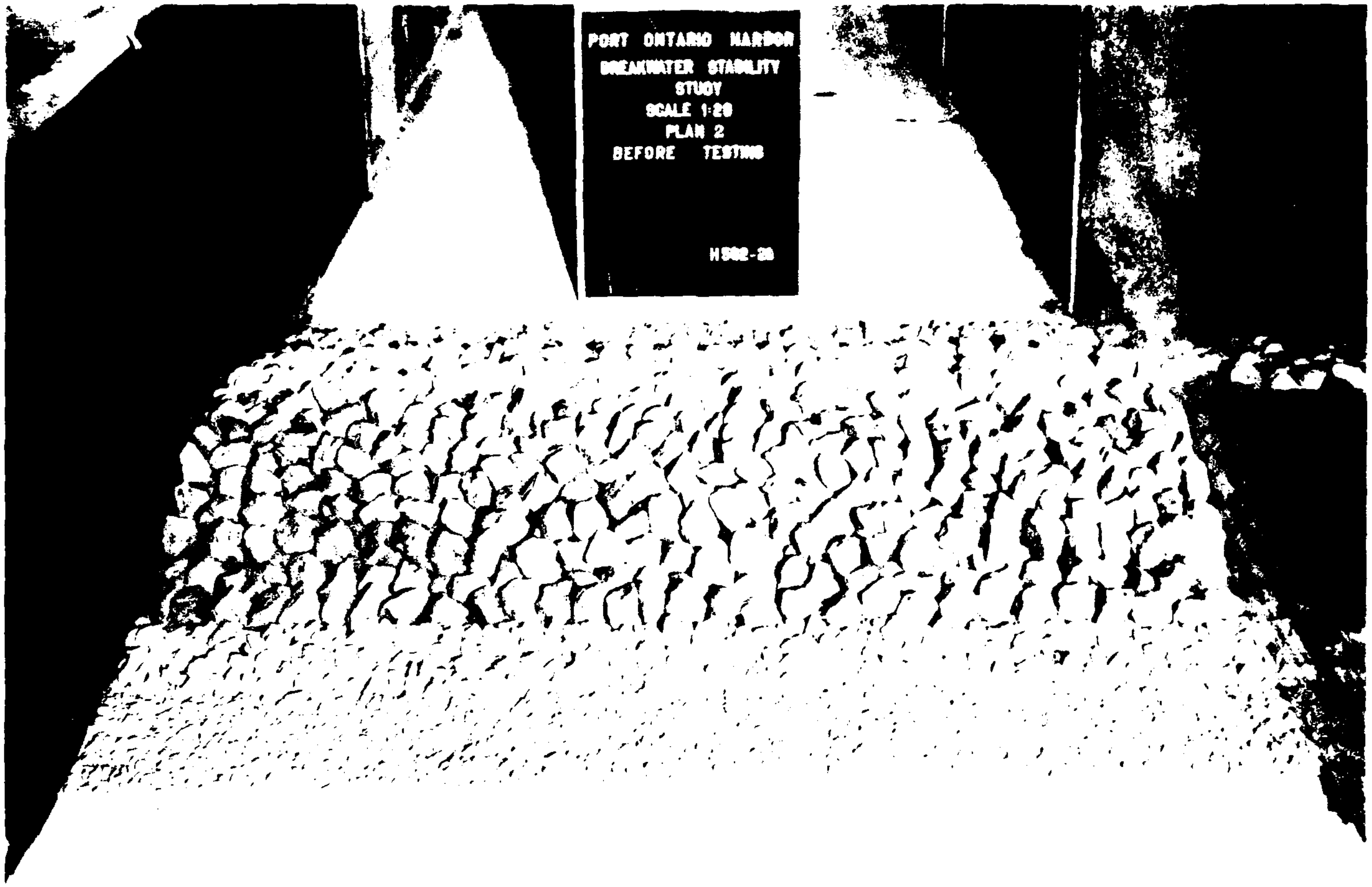
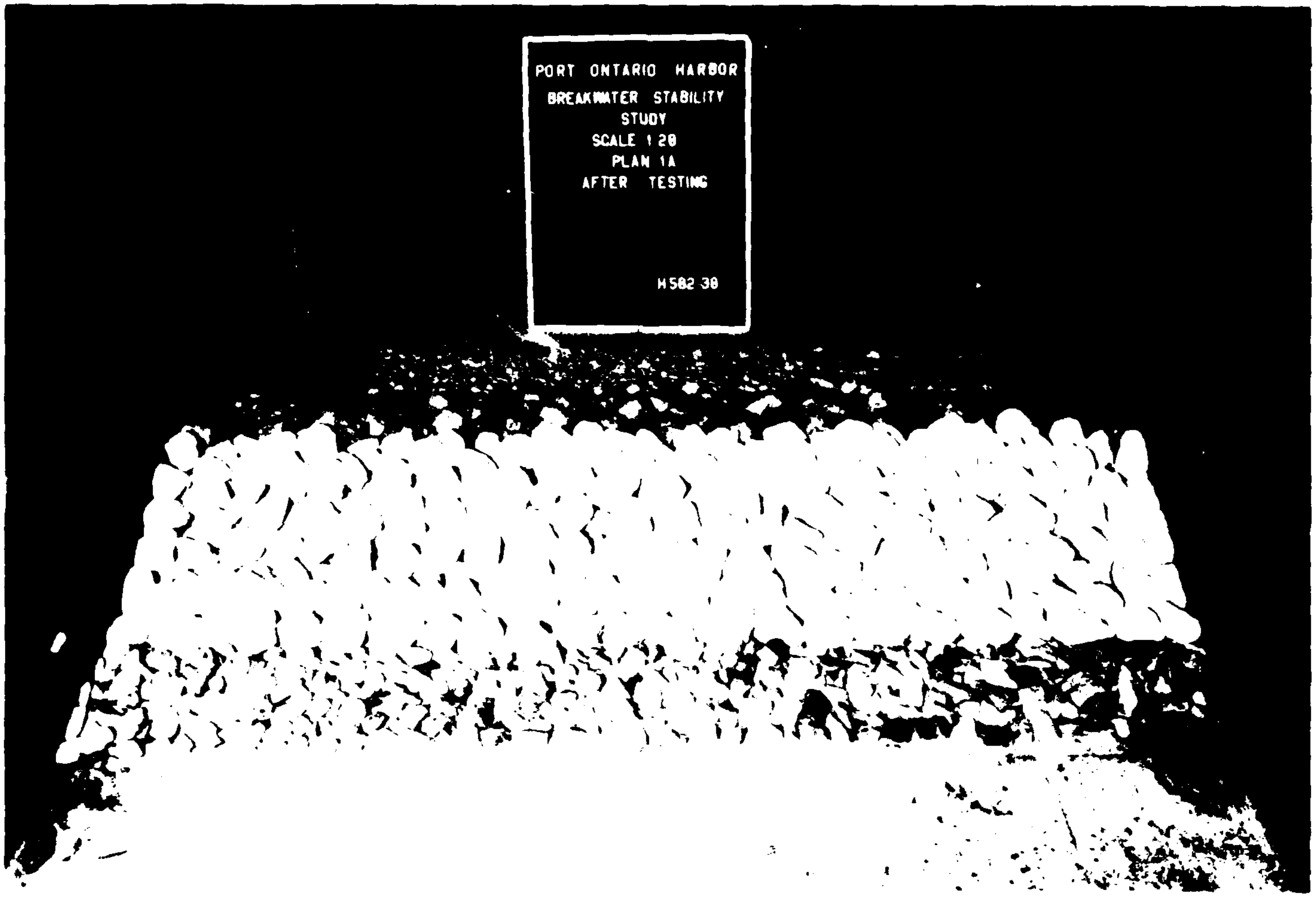
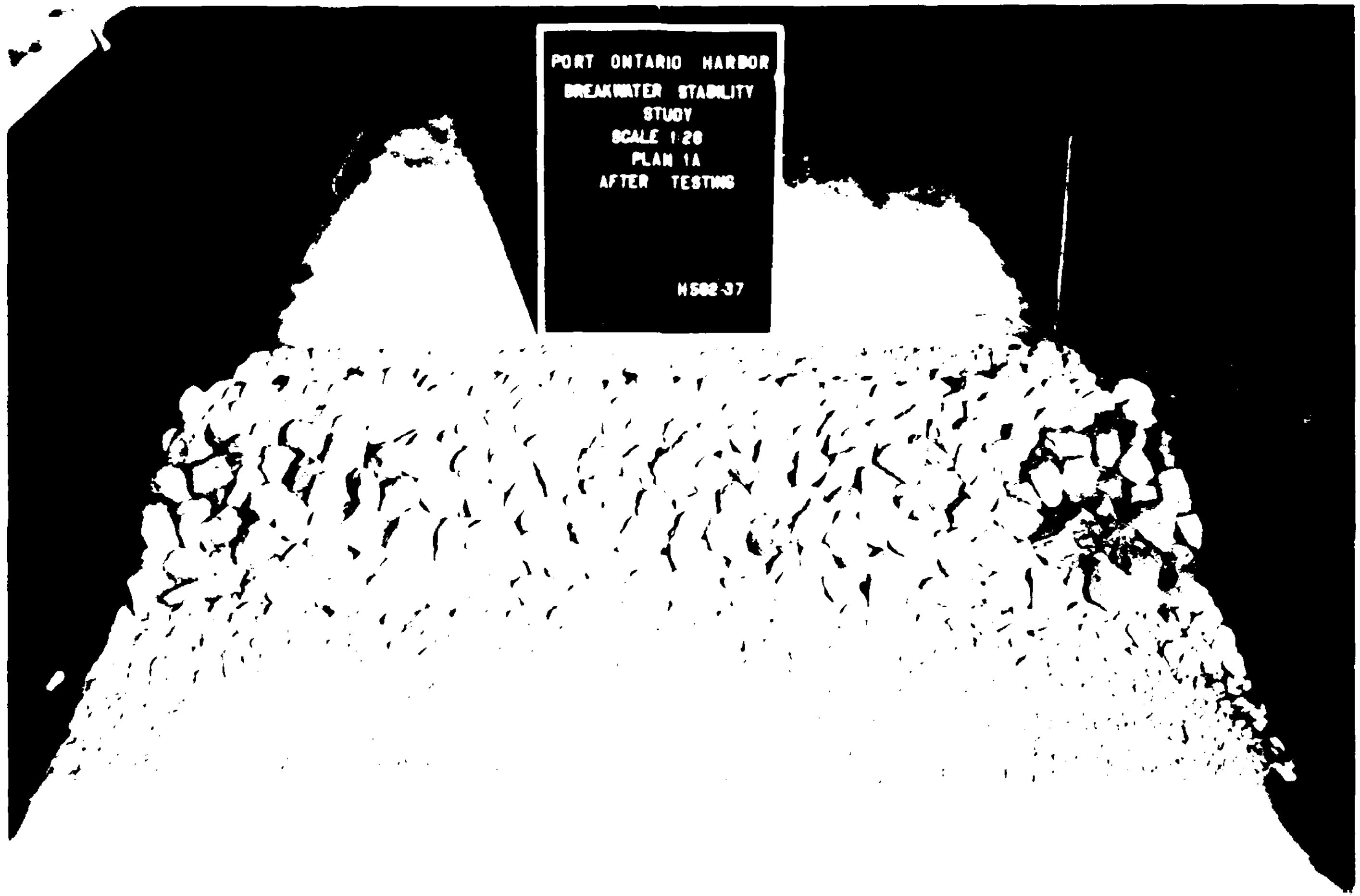


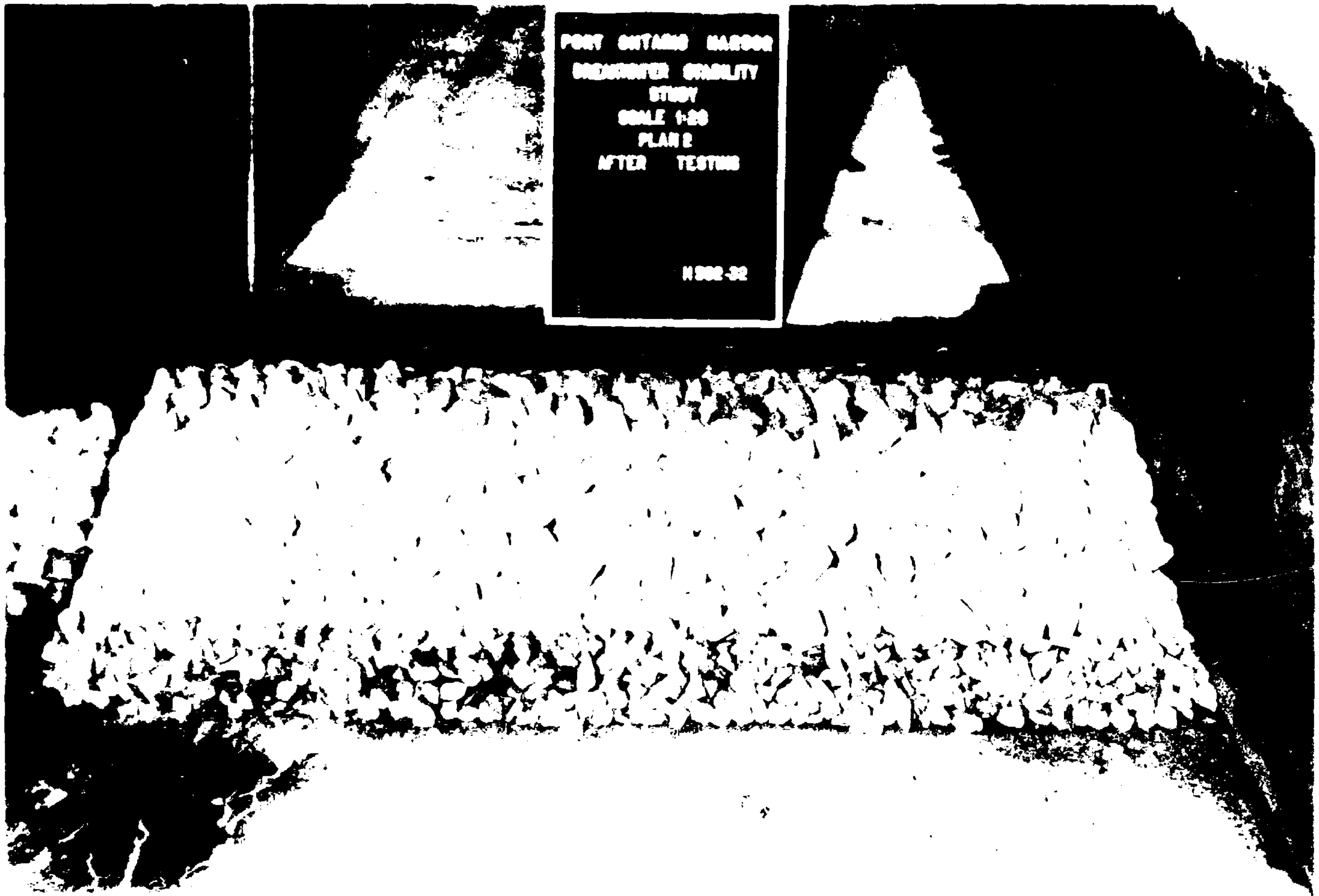
Photo 14. Harbor-side view of Plan 2 before wave attack


PORT ONTARIO HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:20
PLAN 1A
AFTER TESTING
H582 38

[illegible] after completion of storm-surge hydrograph



... storm-surge hydrograph


PORT ONTARIO HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:28
PLAN 2
AFTER TESTING
H 302-32

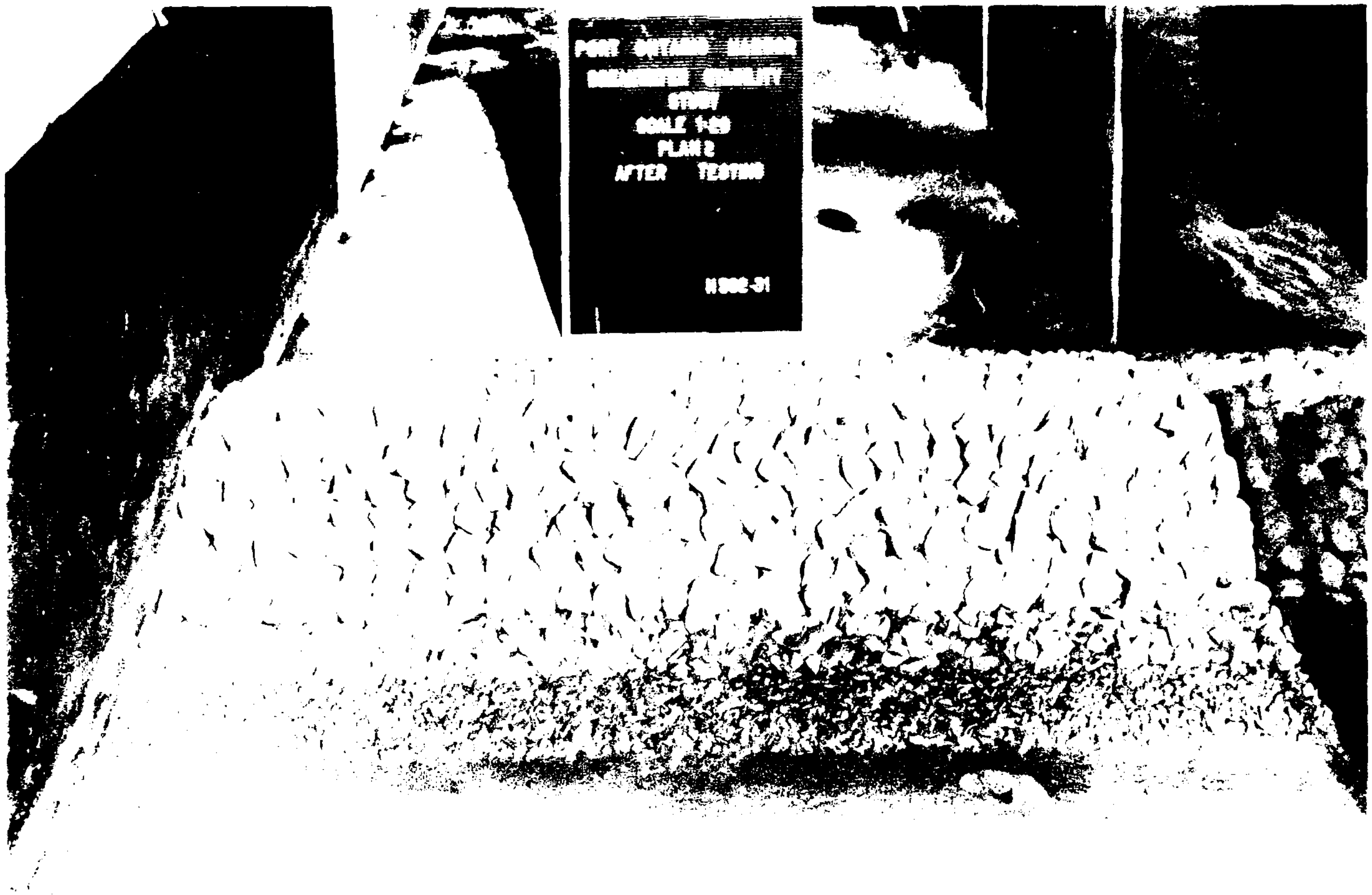
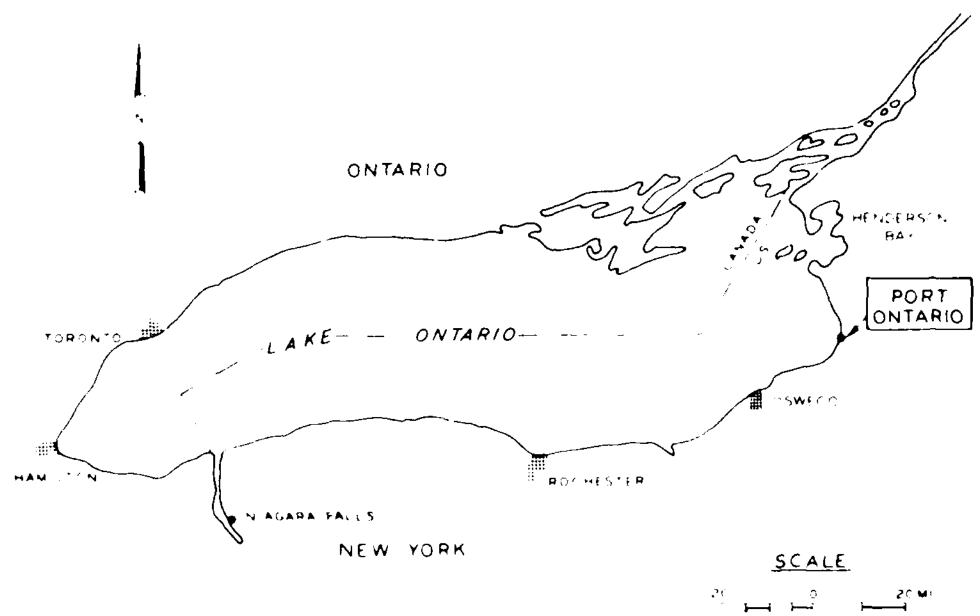
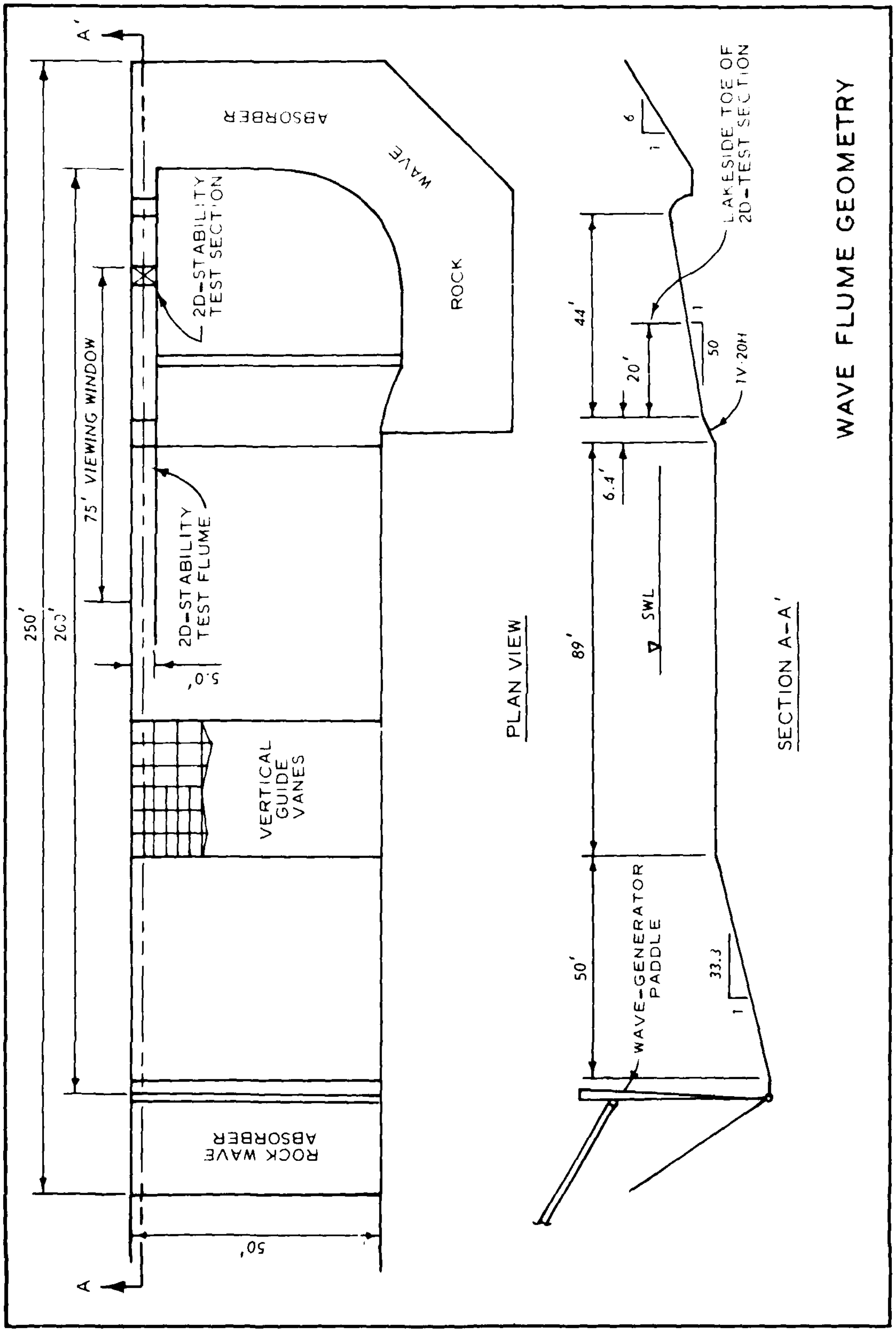


Photo 20. Harbor-side view of Plan 2 after completion of storm-surge hydrograph



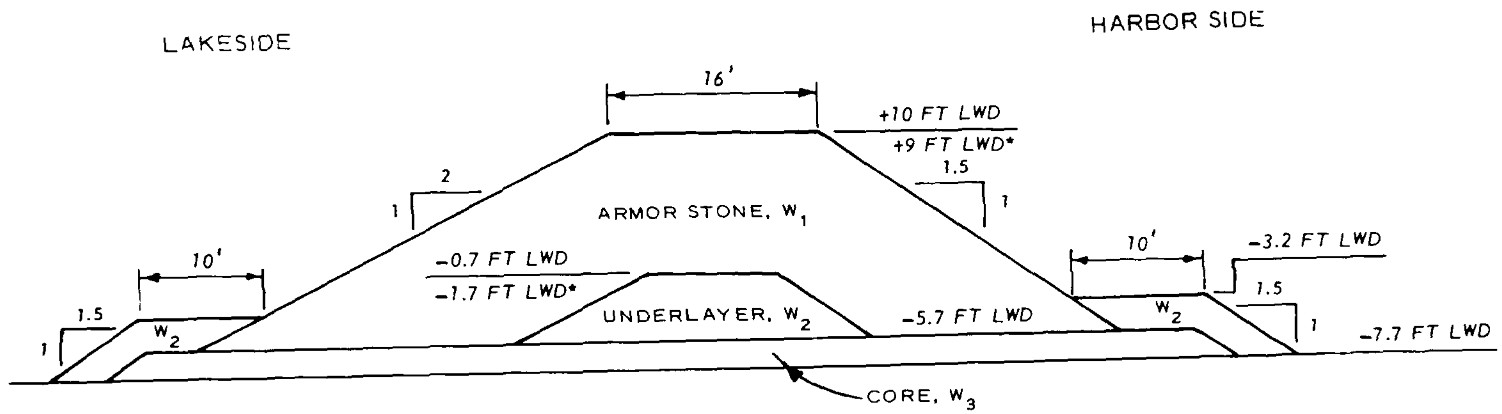
PROJECT LOCATION

PLATE 1



WAVE FLUME GEOMETRY

PLATE 2



MATERIAL CHARACTERISTICS

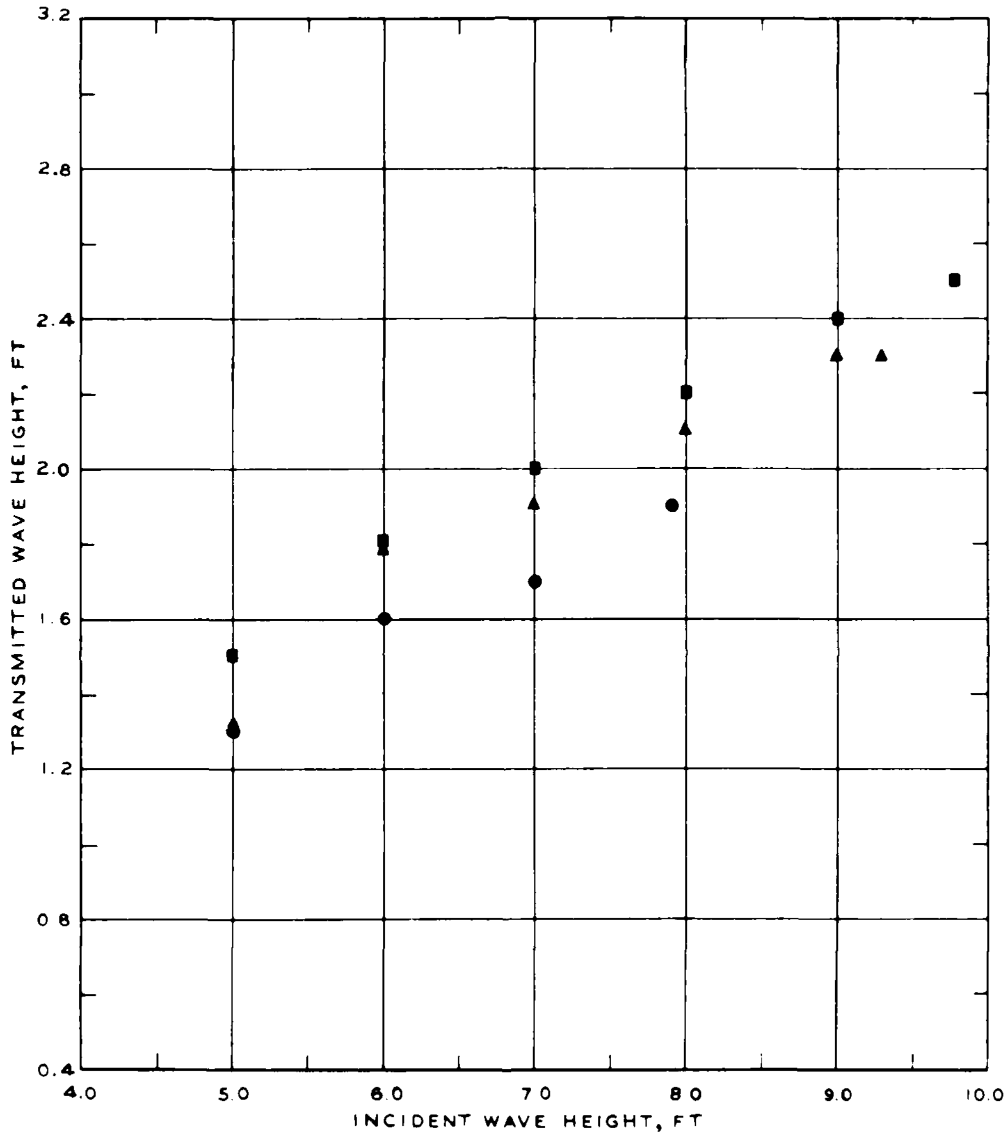
MODEL	PROTOTYPE
W_1 = 0.55-LB STONE @ 165 PCF	W_1 = 7.8-TON STONE @ 155 PCF
W_2 = 0.056-LB STONE @ 165 PCF	W_2 = 1560-LB STONE @ 155 PCF
W_3 = 0.003-LB STONE @ 165 PCF	W_3 = 80-LB STONE @ 155 PCF

* DENOTES PLAN 1A

TYPICAL BREAKWATER CROSS SECTION

MODEL SCALE 1:28

PLANS 1 AND 1A



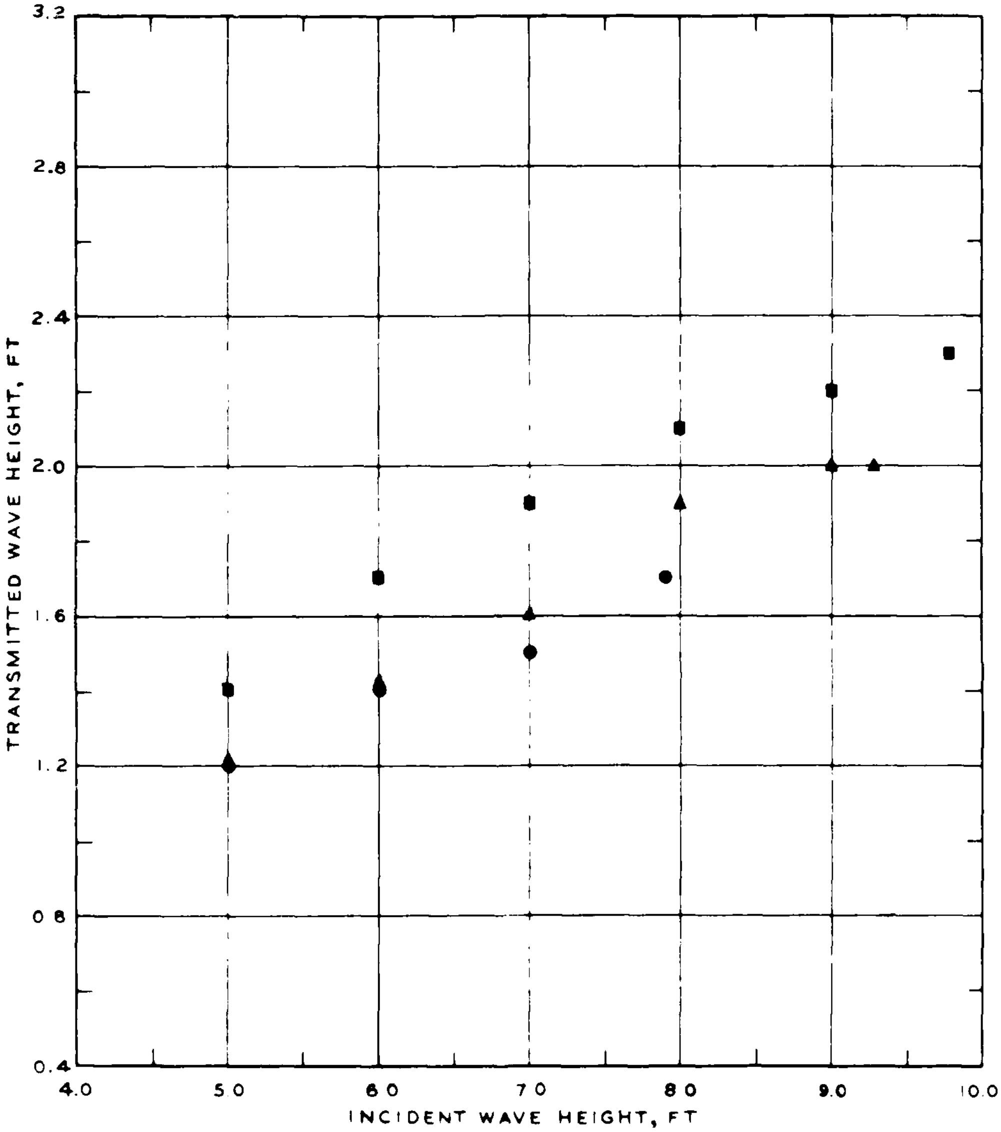
LEGEND

SYMBOL	WAVE PERIOD, SEC
●	7.0
▲	9.0
■	11.0

NOTE: L/2 IS THE DISTANCE IN WAVELENGTHS BEHIND CENTER LINE OF THE BREAKWATER AT WHICH TRANSMITTED WAVE HEIGHTS ARE MEASURED

TRANSMITTED WAVE HEIGHT
VERSUS
INCIDENT WAVE HEIGHT

PLAN 1; L/2



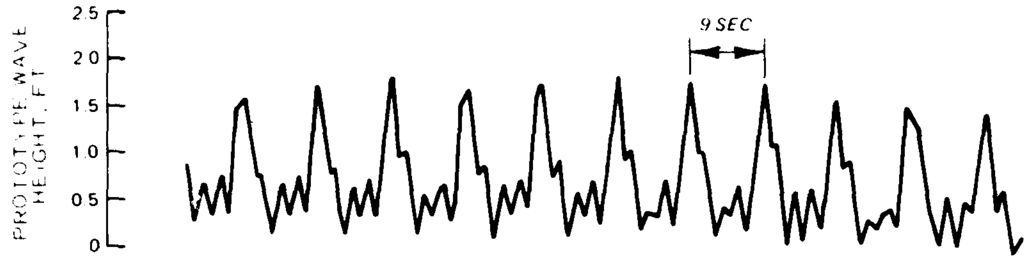
LEGEND

SYMBOL	WAVE PERIOD, SEC
●	7.0
▲	9.0
■	11.0

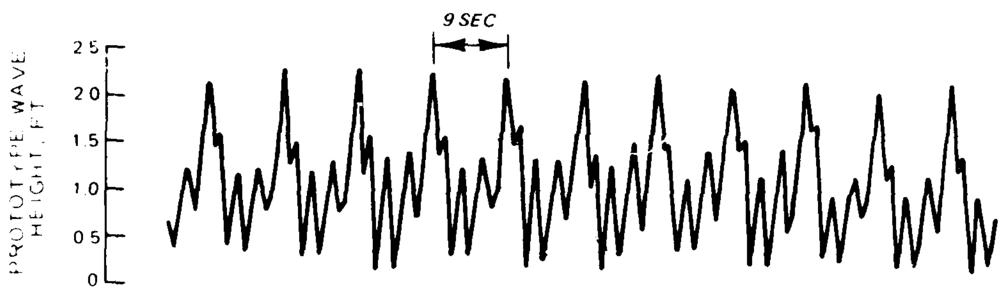
NOTE: L IS THE DISTANCE IN WAVE-LENGTHS BEHIND CENTER LINE OF THE BREAKWATER AT WHICH TRANSMITTED WAVE HEIGHTS ARE MEASURED

TRANSMITTED WAVE HEIGHT
VERSUS
INCIDENT WAVE HEIGHT

PLAN 1; L

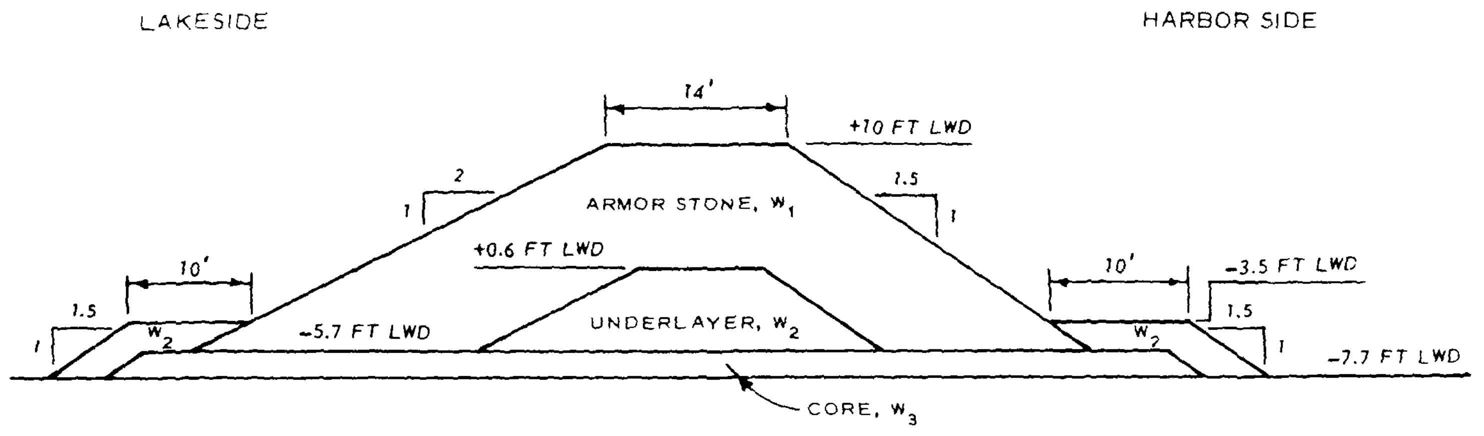


WAVE GAGE POSITIONED
1 WAVELENGTH BEHIND BREAKWATER



WAVE GAGE POSITIONED
1/2 WAVELENGTH BEHIND BREAKWATER

PLAN 1
TYPICAL TRANSMITTED WAVE RECORD
T = 9 SEC; H_i = 7 FT

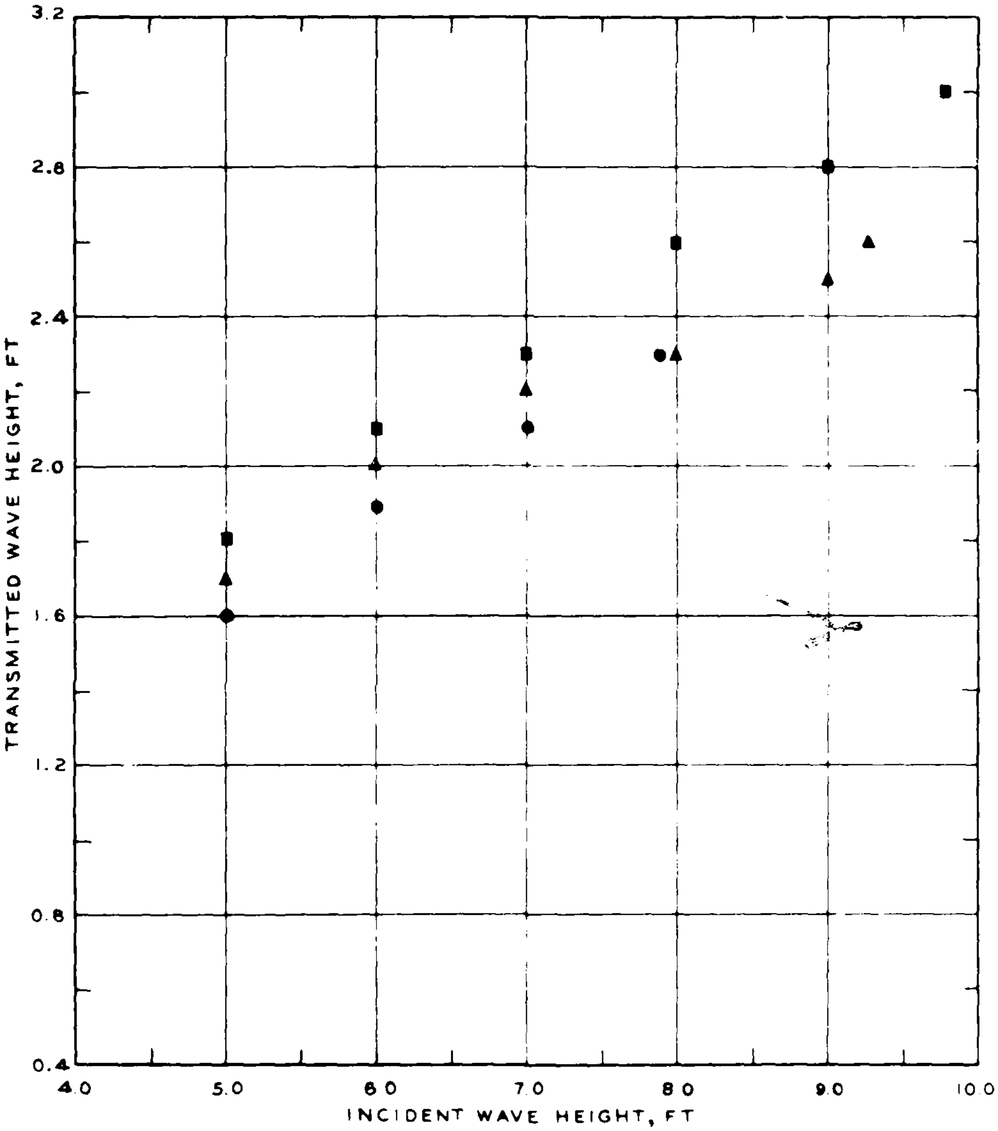


MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
W_1 = 0.38-LB STONE @ 165 PCF	W_1 = 5.3-TON STONE @ 155 PCF
W_2 = 0.038-LB STONE @ 165 PCF	W_2 = 1060-LB STONE @ 155 PCF
W_3 = 0.003-LB STONE @ 165 PCF	W_3 = 80-LB STONE @ 155 PCF

TYPICAL BREAKWATER CROSS SECTION
MODEL SCALE 1 : 28

PLAN 2



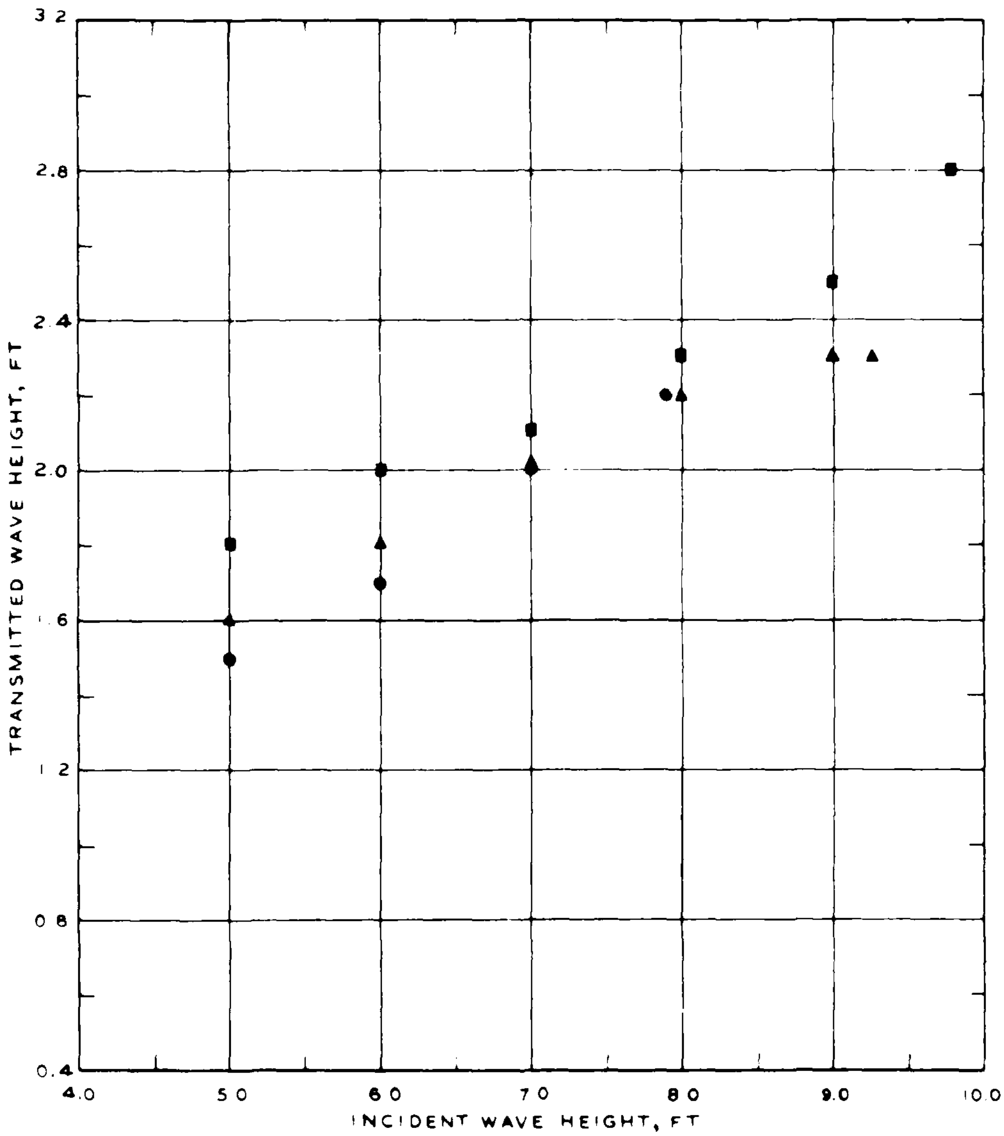
LEGEND

SYMBOL	WAVE PERIOD, SEC
●	7.0
▲	9.0
■	11.0

NOTE: L/2 IS THE DISTANCE IN WAVE-LENGTHS BEHIND CENTER LINE OF THE BREAKWATER AT WHICH TRANSMITTED WAVE HEIGHTS ARE MEASURED.

TRANSMITTED WAVE HEIGHT
VERSUS
INCIDENT WAVE HEIGHT

PLAN 1A; L/2



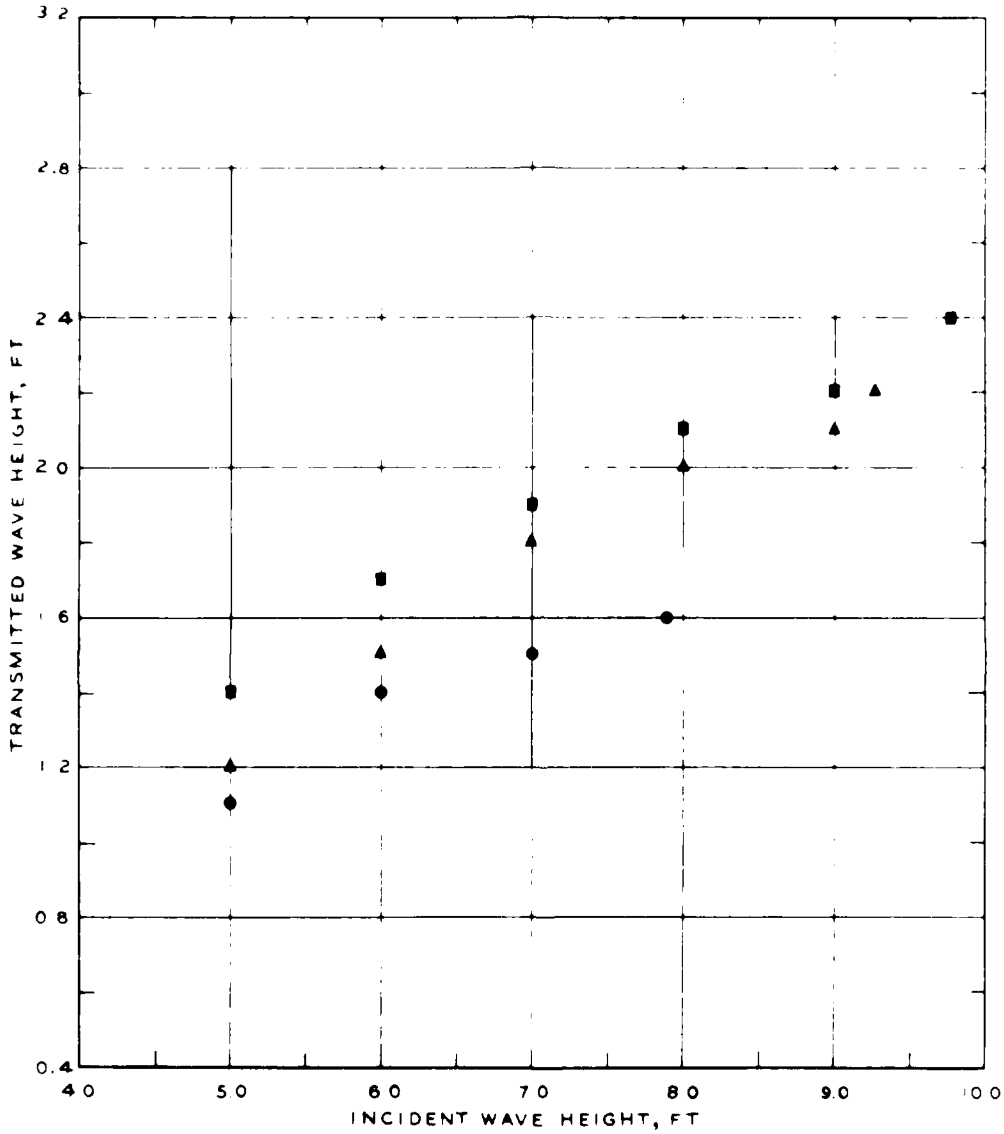
LEGEND

SYMBOL	WAVE PERIOD, SEC
●	7.0
▲	9.0
■	11.0

NOTE: L IS THE DISTANCE IN WAVE-LENGTHS BEHIND CENTER LINE OF THE BREAKWATER AT WHICH TRANSMITTED WAVE HEIGHTS ARE MEASURED

TRANSMITTED WAVE HEIGHT
VERSUS
INCIDENT WAVE HEIGHT

PLAN 1A; L



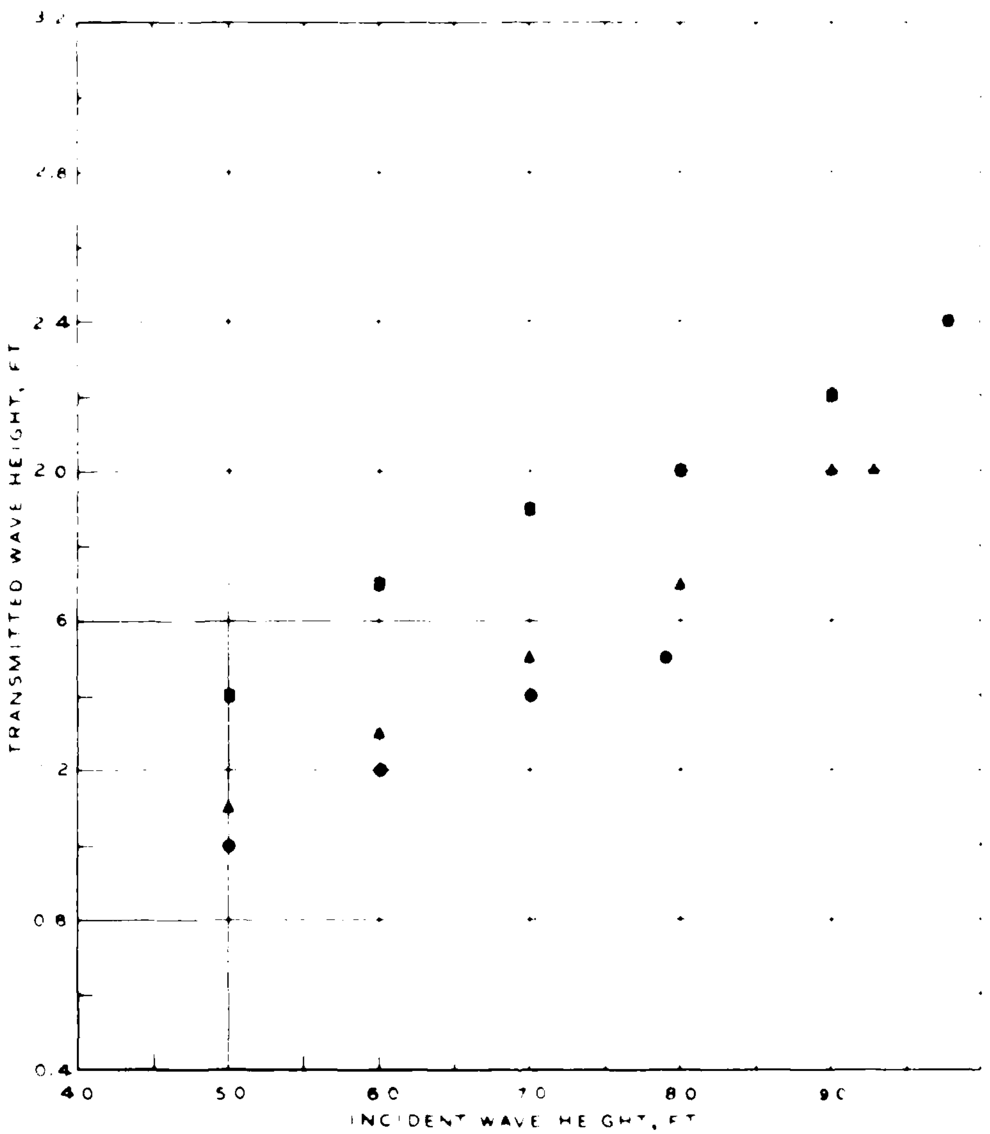
LEGEND

SYMBOL	WAVE PERIOD, SEC
●	7.0
▲	9.0
■	11.0

NOTE: L/2 IS THE DISTANCE IN WAVE-LENGTHS BEHIND CENTER LINE OF THE BREAKWATER AT WHICH TRANSMITTED WAVE HEIGHTS ARE MEASURED.

TRANSMITTED WAVE HEIGHT
VERSUS
INCIDENT WAVE HEIGHT

PLAN 2; L/2



LEGEND

SYMBOL	WAVE PERIOD, SEC
●	7.0
▲	9.0
■	11.0

NOTE: L IS THE DISTANCE IN WAVELENGTHS BEHIND CENTER LINE OF THE BREAKWATER AT WHICH TRANSMITTED WAVE HEIGHTS ARE MEASURED

TRANSMITTED WAVE HEIGHT
VERSUS
INCIDENT WAVE HEIGHT

PLAN 2, L

[illegible] letter from [illegible] AFN ASI dated [illegible] cards for laboratory [illegible] facsimile catalog [illegible] MARC format is reproduced [illegible]

[illegible] antenna [illegible] Army [illegible] antenna [illegible] experiment [illegible]

[illegible]

[illegible]

ATE

LMED

8