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SOUTH JETTY STABILITY STUDY
MASONBORO INLET, NORTH CAROLINA

Hydraulic Model Investigation

by

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Hydraulic model flume tests were conducted to investigate the stability response of a jetty cross section proposed for the South Jetty, Masonboro Inlet, North Carolina. The original design (Plan 1) was not stable, and three additional designs were tested in an effort to find a satisfactory solution. All designs tested consist of one layer of armor stone subjected to breaking wave conditions. None of the designs were completely stable for all the designated storm conditions, and sufficient funds were not available to further the

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20. ABSTRACT (Continued).

investigation. Based on the tests conducted, results show that all the designs were stable for storm conditions at the +8.5 ft mllw swl, but none of the designs were stable for storm conditions at the +12.5 ft mllw swl. Thus, whether the designs tested meet the no-damage criteria depends upon the selection of the design storm condition.

PREFACE

The model investigation reported herein was requested by the U. S. Army Engineer District, Wilmington (SAW), in a telephone communication to the U. S. Army Engineer Waterways Experiment Station (WES) on 7 February 1978. Funding authorization by SAW was granted on IAO No. SAWEN-DC-78-173, dated 8 February 1978 and subsequent change order No. 1, dated 7 March 1978.

Model tests were conducted at WES during February 1978, under the general direction of Mr. H. B. Simmons, Chief of the Hydraulics Laboratory; Dr. R. W. Whalin, Chief, Wave Dynamics Division; and Mr. D. D. Davidson, Chief, Wave Research Branch. Tests were conducted by Messrs. R. D. Carver and D. G. Markle, Research Hydraulics Engineers, and Messrs. C. R. Herrington and C. Lewis, engineering technicians. This report was prepared by Messrs. Carver and Markle.

Liaison was maintained during the course of the investigation by progress reports and telephone conversations.

Director of WES during the conduct of this study and the preparation and publication of this report was COL John L. Cannon, CE. Technical 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
tons (2000 lb mass)	907.1847	kilograms
pounds (mass) per cubic foot	16.01846	kilograms

SOUTH JETTY STABILITY STUDY, MASONBORO INLET, NORTH CAROLINA

Hydraulic Model Investigation

PART I: INTRODUCTION

The Prototype

1. Masonboro Inlet, a natural inlet through the coastal barrier beach of North Carolina, is located 8 miles* southeast of Wilmington, North Carolina (Plate 1). The inlet is at the southern end of Wrightsville Beach, an important resort beach located in New Hanover County.

2. A short chronology of improvements of Masonboro Inlet, as derived from reports of the Wilmington District[1,2] (SAW), is as follows. Improvements for the inlet, authorized in 1949, included two jetties, an ocean entrance channel between the jetties, and navigation channels to the Atlantic Intracoastal Waterway. Initially, the interior channels were dredged in 1957, and in April-June 1959 the ocean navigation channel was dredged. Construction of the north jetty was initiated in August 1965 and completed in June 1966. During design of the south jetty concern arose as to the stability of the proposed structure. As designed, the jetty will be subjected to breaking waves which either strike the structure directly or are tripped by it, causing major turbulence and overtopping. Although model tests conducted at the WES in connection with the OCE Research and Development Program[3,4,5] have provided considerable data for design of rubble mound structures, the majority of the data addresses nonovertopping structures subject to nonbreaking waves. Anticipated wave conditions and the proposed geometry of the Masonboro jetty are similar to conditions presently being tested for the Oregon Inlet Jetties at Oregon Inlet, North Carolina;** however, the

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

** Jetty Stability Study, Oregon Inlet, N. C. (unpublished, study presently being conducted at WES).

Masonboro design deviates significantly from the Oregon Inlet design in that it is proposed to use only one layer of armor rock. Therefore, it was considered necessary to conduct stability tests of the proposed south jetty.

Purpose and Approach

3. The original purpose of the investigation was to determine by two-dimensional (2-D) flume tests the stability response of a single jetty cross section (Plan 1) as proposed by SAW. However, once it became apparent that Plan 1 was not an adequate design, three additional plans were tested in an effort to find a stable design.

PART II: THE MODEL

Design of the Model

4. Tests were conducted at a geometrically undistorted linear scale of 1:33, model to prototype. Scale selection was determined by the absolute size of model breakwater sections necessary to ensure the preclusion of stability scale effects,[6] capabilities of the available wave generator, and depth of water at the top of the breakwater. Based on Froude's model law[7] and the linear scale of 1:33, the following model-prototype relations were derived. Dimensions are in terms of length (L) and time (T).

Characteristics	Dimensions	Model-Prototype Scale Relation
Length	L	$L_r = 1:33$
Area	L^2	$A_r = L_r^2 = 1:1,089$
Volume	L^3	$V_r = L_r^3 = 1:35,937$
Time	T	$T_r = L_r^{1/2} = 1:5.74$

5. The specific weight of water used in the model was assumed to be 62.4 pcf and that of seawater 64.0 pcf, also specific weights of model breakwater construction materials were not the same as their prototype counterparts. These variables were related using the following transference equation:

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

where

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

m, p = subscripts representing model and prototype quantities, respectively

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

L_m/L_p = linear scale of the model

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

Method of Constructing Test Sections

6. All model-breakwater sections were constructed to reproduce as closely as possible the usual methods of constructing prototype breakwaters. The bedding material was smoothed to grade with hand trowels as it was dumped by bucket or shovel into the flume. The core stone was compacted and smoothed to grade with hand trowels as it was dumped by bucket or shovel. This was done in an effort to simulate the natural consolidation that would occur due to wave action during prototype constuction. Once the core material was in place it was sprayed with a low velocity water hose to ensure adequate compaction. Armor stone used in the cover layer was placed in a random manner, i.e., laid down in such a way that no intentional interlocking of the stones was obtained.

Test Facilities and Equipment

7. A concrete wave flume, 6.6 ft wide, 4 ft deep, and 119 ft long was used for all tests. The flume is equipped with a vertical-displacement wave generator capable of producing sinusoidal waves of various periods and heights. Test waves of the required characteristics were generated by varying the frequency and amplitude of the plunger motion. Breakwater sections were installed in the flume about 85 ft from the wave generator. Local prototype bathymetry was molded for a simulated prototype distance of 611 ft seaward of the test sections (Plate 2). Changes in water surface elevation (wave heights) as a function of time were measured by electrical wave height gages and recorded on chart paper by an electrically operated oscillograph. The electrical output of each wave gage was directly proportional to its submergence depth in water.

PART III: TESTS AND RESULTS

Selection of Test Conditions

8. It was desired to design a rock-armored structure using a toe elevation of -6.0 ft mlw which would be stable for storm surges up to +12.5 ft mlw. Initially swls of +1.4, +5.0, +8.5, and +12.5 ft mlw were considered. Observations of Plan 1 under wave attack showed the worst breaking waves (as a function of both period and height) that could be made experimentally to attack the section for the selected conditions were as follows:

SWL ft, mlw	Depth ft, mlw	Wave Period (sec)	Worst Breaking Wave Height, ft
+1.4	7.4	8	5.7
+1.4	7.4	11	6.0
+1.4	7.4	14	6.4
+5.0	11.0	11	8.5
+5.0	11.0	13	8.9
+5.0	11.0	15	9.5
+8.5	14.5	11	12.1
+8.5	14.5	13	12.6
+8.5	14.5	15	13.5
+12.5	18.5	11	15.3
+12.5	18.5	13	15.8
+12.5	18.5	15	16.9

Model observations also showed that for a given swl the longest wave period considered was always the most detrimental to the structural integrity of the section. Therefore, full-length stability tests were conducted using only wave periods of 14 and 15 sec.

Plans Tested and General Results

9. A total of 4 plans were tested. All plans used a toe elevation of -6.0 ft mlw, armor slopes of 1:2 (both sea side and beach side), one layer of armor, and sea-side and beach-side toe widths of 3 and 4 armor

stones, respectively. Details of the plans tested and general results are presented in the following paragraphs.

10. Plan 1 (Plate 3) was constructed to a crown elevation of +8.5 ft mlw. Twelve-ton armor was used both sea side and beach side. Vertical sheet piling, which will be used in the prototype to an elevation of +6.5 ft mlw, was simulated in the model with 18-gage galvanized sheet steel. Plan 1 was initially subjected to 14-sec, 6.4-ft waves at an swl of +1.4 ft mlw for 1 prototype hour. No damage was observed for this wave condition. The swl was then raised to +5.0 ft mlw and the structure was subjected to 1 prototype hour of 15-sec, 9.5-ft waves. Again, no damage was observed. The structure was then subjected to 15-sec, 13.5-ft waves for 1 prototype hr at an swl of +8.5 ft mlw. This condition produced minor damage with 2 sea-side armor stones and 1 crown stone being displaced onto the beach side of the structure. Finally, the swl was raised to +12.5 ft mlw and the structure was subjected to 15-sec, 16.9-ft waves for 2 prototype hours, and the structure experienced significant damage. As illustrated in Photos 1 and 2, the crown elevation was lowered to the extent that the sheet pile wall was exposed in several places. Since damage to the structure had not stabilized, testing was extended for 2 more prototype hours. As shown in Photos 3 and 4, the structure continued to deteriorate. The seaward crown elevation was significantly lowered and the sheet pile wall and the sea-side armor were separated to the extent that a gap was opened between the two which significantly exposed the core stone. Since the structure had deteriorated past the point of acceptablity, the tests were terminated.

11. Based on observations of Plan 1 it was felt that damage would have been more severe if the sheet piling had failed. However, the limited scope of the testing program precluded any attempts to properly simulate the structural characteristics of the sheet piling. It also was felt that neither sufficient data nor methodologies were available to analytically predict when and to what extent the sheet piling might fail. Therefore, even though the prototype designs will use the sheet piling to make the jetty impermeable to sand, subsequent model plans

were tested without sheet piling under the assumption that the stability of the armor stone should not depend upon the structural integrity of the sheet piling.

12. Plan 2 (Plate 4 and Photos 5 and 6) was similar to Plan 1 except the crown elevation was raised to +9.0 ft mlw, the sheet piling was eliminated, and the armor weight was increased to 15 tons. The structure was initially subjected to 15-sec, 13.5-ft waves at an swl of +8.5 ft mlw for 2 prototype hours. Moderate damage was observed; however, the structure's stability response was judged to be marginally acceptable. The swl was then raised to +12.5 ft, and the structure was subjected to 15-sec, 16.9-ft waves for 2 prototype hours. As shown in Photos 7 and 8, this wave condition produced extensive damage. The average crown elevation was lowered 3 to 5 ft. Both the sea-side and beach-side slopes were flattened to approximately 1:3, and the core material was exposed in several places.

13. Plan 3 (Plate 5 and Photos 9 and 10) was constructed to a crown elevation of +7.0 ft mlw and again excluded the sheet pile wall. Eighteen-ton armor was used both sea side and beach side. The bedding material (W_3) was covered with screen in an effort to simulate gabion mats which are to be used during prototype construction. Attack of 15-sec, 13.5-ft waves at an swl of +8.5 ft mlw for 2 prototype hours produced very minor damage to the first row of sea-side toe armor. The swl was then raised to +10.5 ft mlw and the structure was subjected to 15-sec, 15-ft waves for 2 prototype hours (it had been decided between the testings of Plans 2 and 3 to add an swl intermediate to the +8.5-ft and +12.5-ft swls). The +10.5 ft swl produced extensive damage to the sea-side toe. Also, the average crown elevation was lowered 1 to 2 ft and the core stone was exposed on several areas of the crown. Finally the swl was raised to +12.5 ft mlw and the structure was subjected to 15-sec, 16.9-ft waves for 2 prototype hours. As evidenced in Photos 11-13, this wave condition produced extensive damage. Seaward migration of both crown armor and core material resulted in an additional 2 to 3 ft lowering of the crown, and the core was randomly exposed such that it would take about 10 armor stones to cover the exposed areas represented

by the 165-ft length of prototype structure (5-ft length of model structure).

14. Observations of Plan 3 under wave attack indicated that some of the damage to the crown area may have been initiated by sliding of the toe armor. It is felt that the sliding of the toe armor which occurred in the model may not be a realistic simulation of the prototype because sand impoundment and differential settlement of the gabion mats may increase prototype toe stability above that predicted by Plan 3 in the model. It was, therefore, decided to determine the stability response of the 18-ton armor in the absence of toe slippage.

15. Plan 3A (Plate 6) was the same as Plan 3 except small wooden strips were placed along the seaward and beachward toes of the structure to prevent horizontal sliding of the toe armor. The structure was initially subjected to 15-sec, 13.5-ft waves at an swl of +8.5 ft mlw for 2 prototype hours. As shown in Photos 14-16, this wave condition produced minor damage to the crown with 2 armor units being displaced onto the beach side of the structure. The swl was then raised to +10.5 ft mlw and the structure was subjected to 15-sec, 15-ft waves for 2 prototype hours. As can be seen in Photos 17-19, the structure's crown received moderate to extensive damage with the average elevation being lowered 1 to 2 ft. Seven armor units were displaced from the crown and the core stone was exposed in several areas sufficient to require 6 or 7 armor stones to cover the exposure. Finally the swl was raised to +12.5 ft mlw and the structure was subjected to 15-sec, 16.9-ft waves for 2 prototype hours. Referring to Photos 20-22, it can be seen that this condition produced extensive damage to the crown of the structure. The average crown elevation was lowered an additional 1 to 2 ft and exposure of the core stone became more pronounced, requiring up to a total of 10 or 11 armor stones, to cover the exposed areas. Repeat test verified Plan 3A's stability response.

PART IV: CONCLUSIONS

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

- a. None of the plans tested can be considered to be a no-damage design for storm surges above +8.5 ft mlw.
- b. Plan 3A* exhibited the best stability response of all plans tested.
- c. As compared to Plan 3, Plan 3A* showed a greatly improved toe stability and a slightly improved crown stability.

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* Plan 3A was the same as Plan 3; however, the model toe was artificially held in place by a small wooden strip to prevent horizontal sliding of the toe armor which appeared to be unrealistically large in Plan 3. It was assumed that as soon as a toe unit slid off the gabion mat it would settle in the sand. The prototype situation would probably best be represented by a damage intermediate to that indicated for Plan 3 and Plan 3A.

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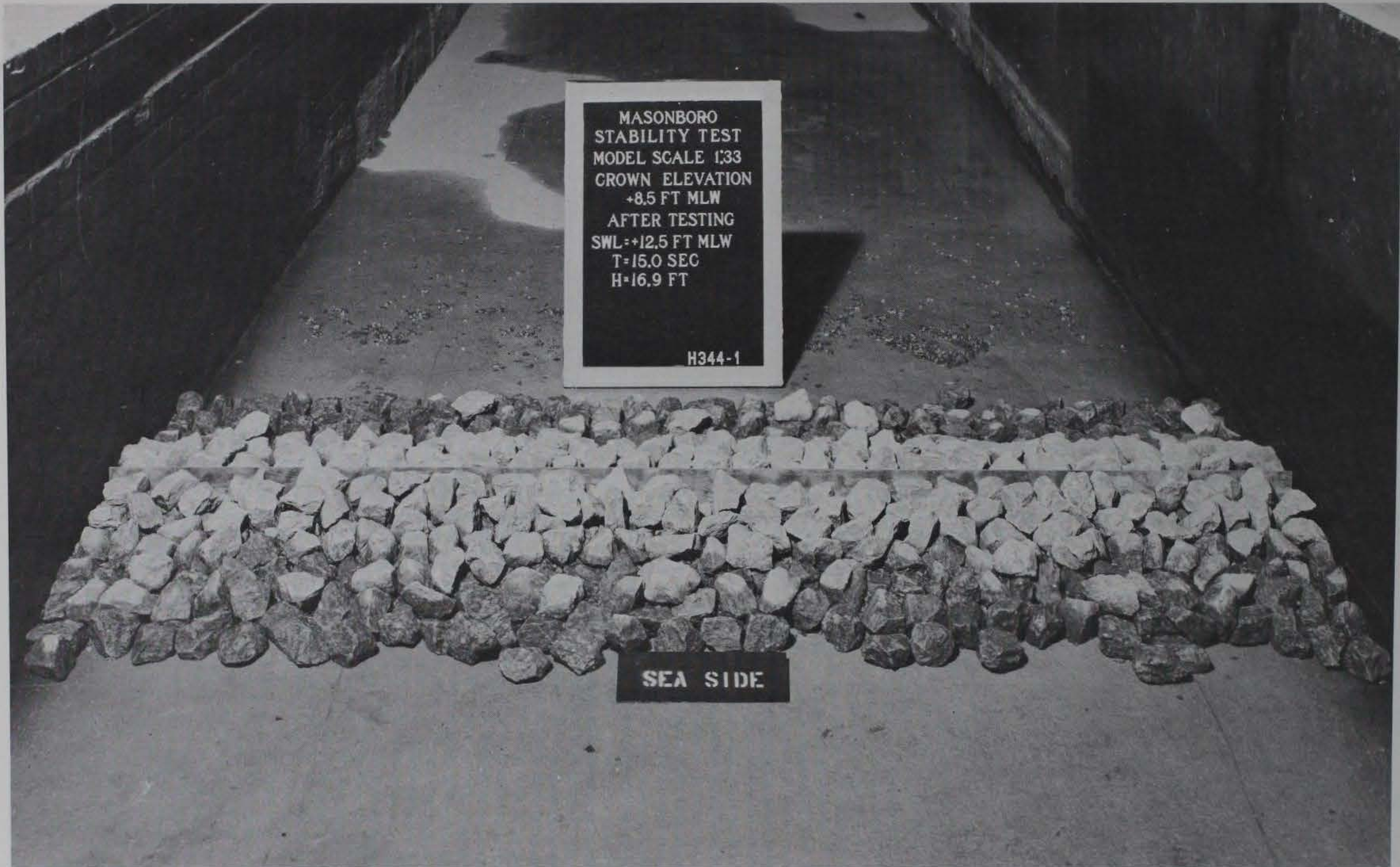


Photo 1. Sea-side view of Plan 1 after attack of 15-sec, 16.9-ft waves at an swl of +12.5 ft mlw for 2 prototype hr

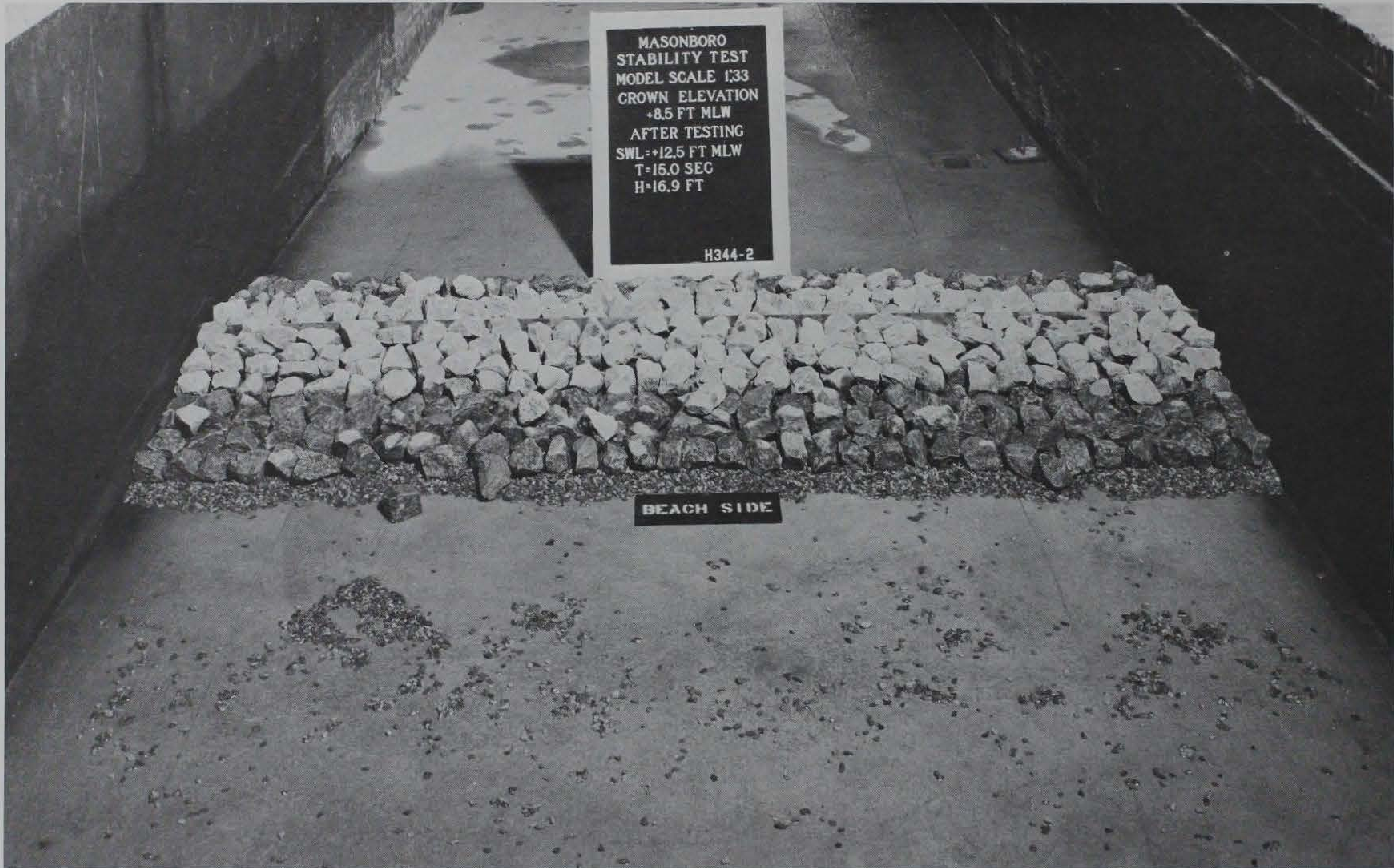


Photo 2. Beach-side view of Plan 1 after attack of 15-sec, 16.9-ft waves at an swl of +12.5 ft mlw for 2 prototype hr

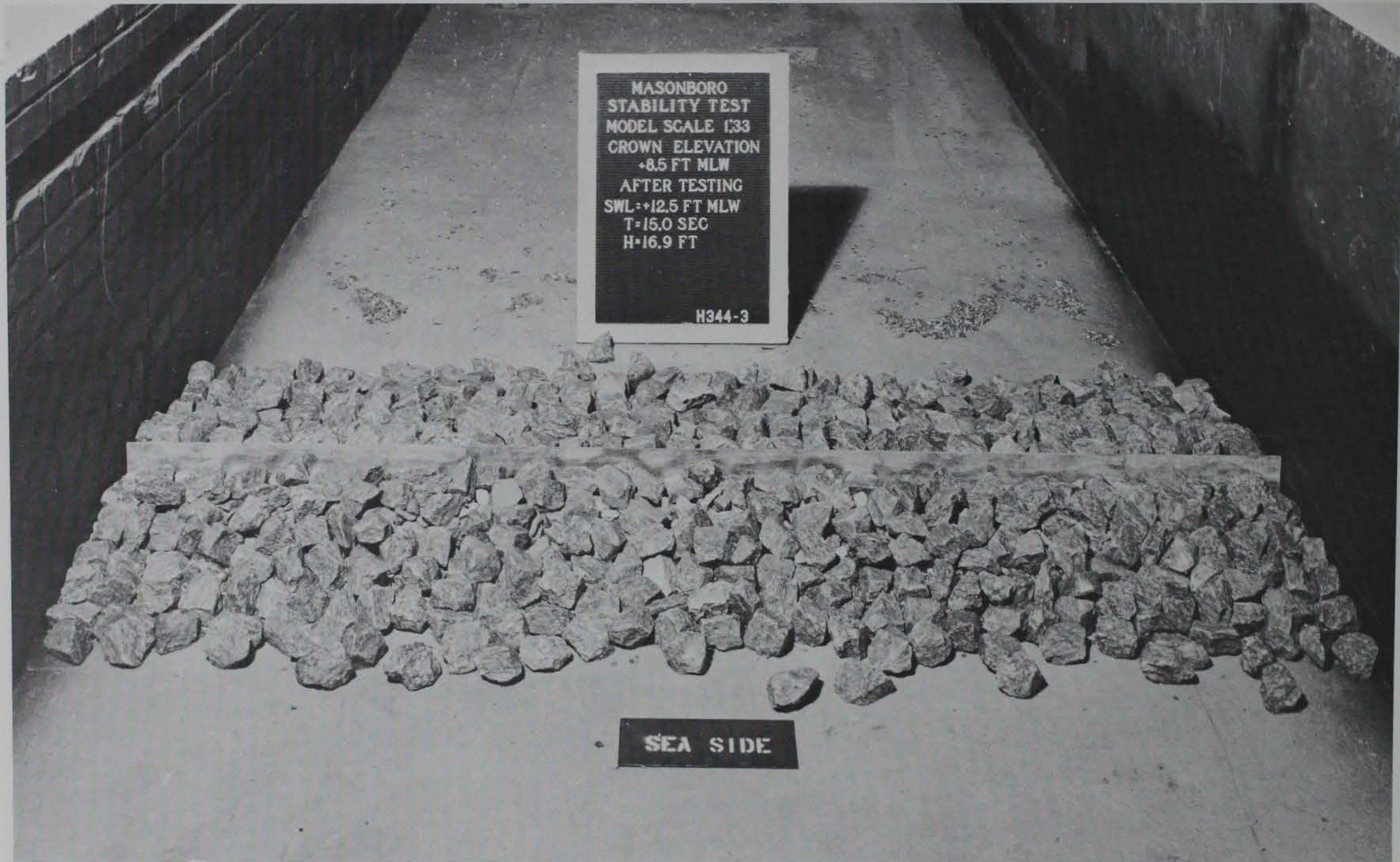


Photo 3. Sea-side view of Plan 1 after attack of 15-sec, 16.9-ft waves at an swl of +12.5 ft mlw for 4 prototype hr

Photo 4. Beach-side view of Plan 1 after attack of 15-sec, 16.9-ft waves at an swl of +12.5 ft mlw for 4 prototype hr

Photo 5. Sea-side view of Plan 2 before wave attack

Photo 6. Beach-side view of Plan 2 before wave attack

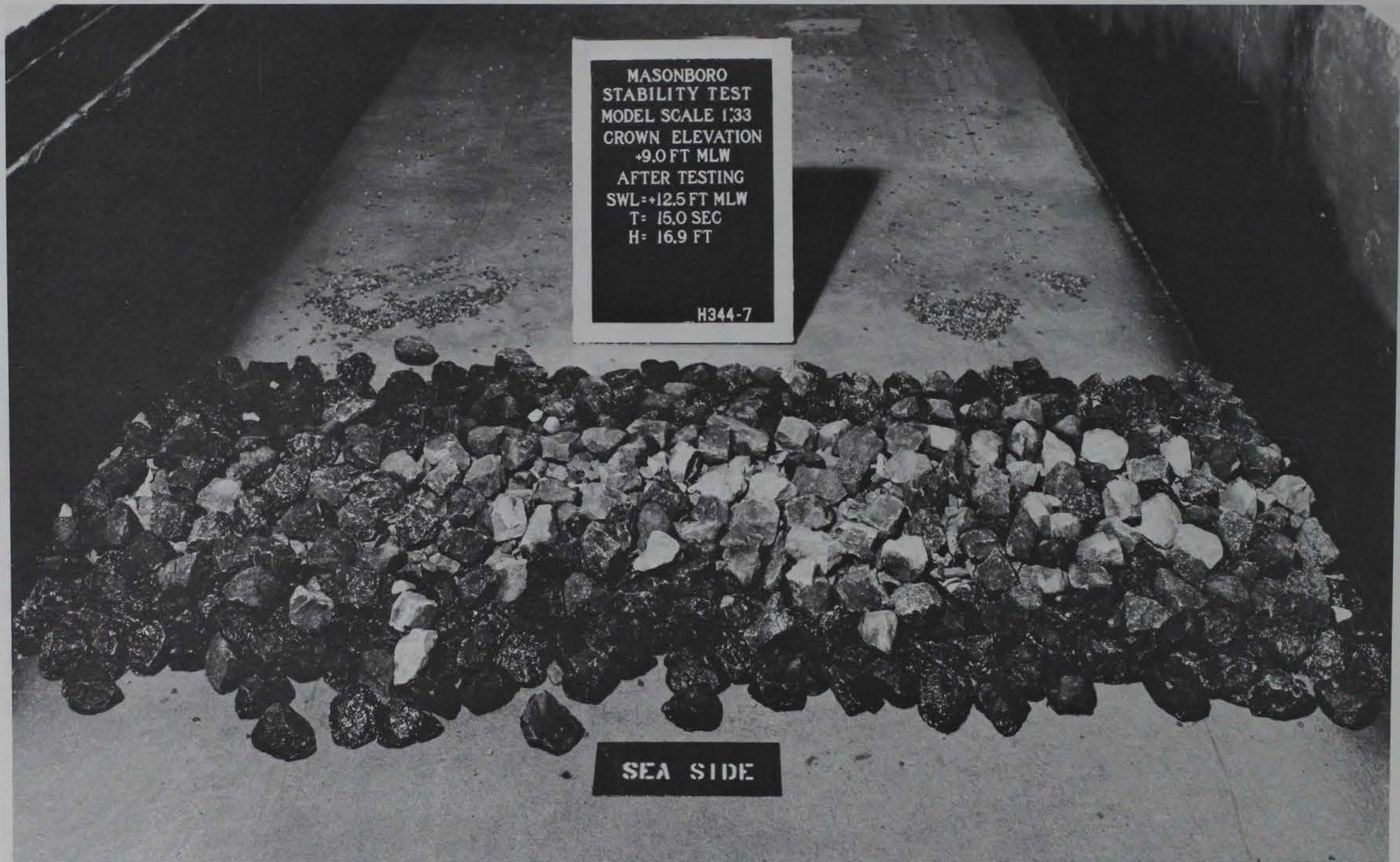


Photo 7. Sea-side view of Plan 2 after attack of 15-sec, 16.9-ft waves at an swl of +12.5 ft mlw for 2 prototype hr

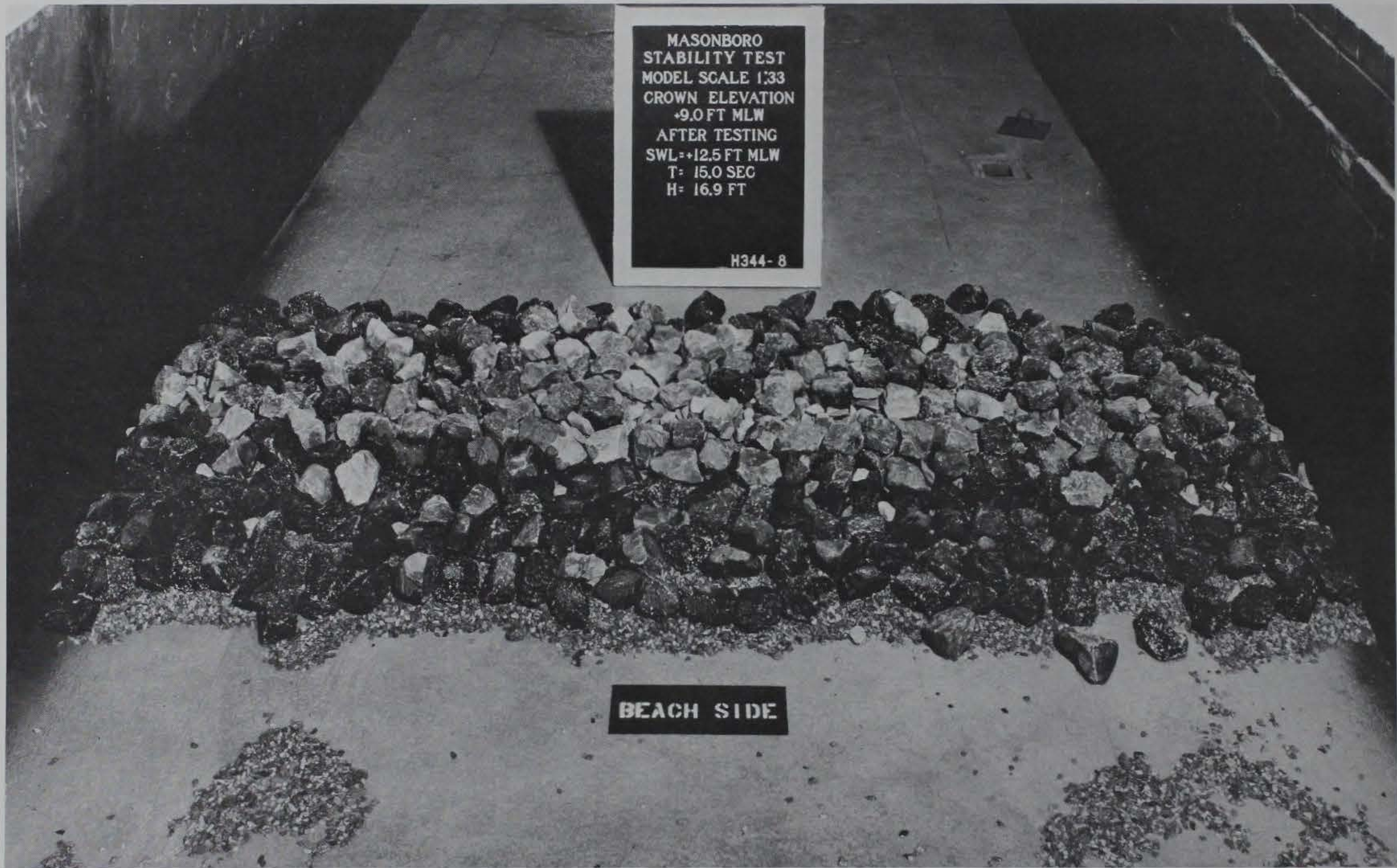


Photo 8. Beach-side view of Plan 2 after attack of 15-sec, 16.9-ft waves at an swl of +12.5 ft mlw for 2 prototype hr

Photo 9. Sea-side view of Plan 3 before wave attack

Photo 10. Beach-side view of Plan 3 before wave attack

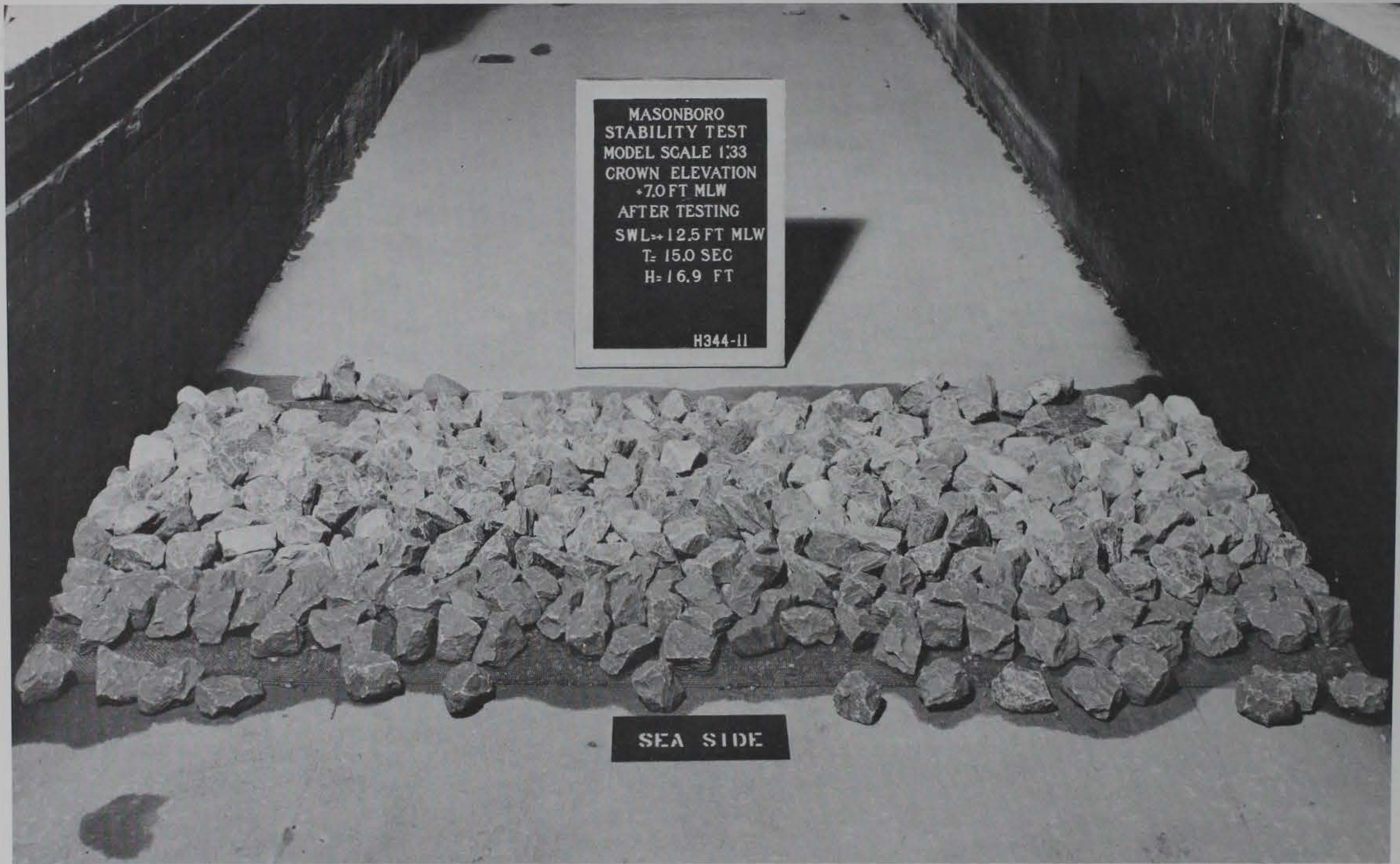


Photo 11. Sea-side view of Plan 3 after attack of 15-sec, 16.9-ft waves at an swl of +12.5 ft mlw for 2 prototype hr

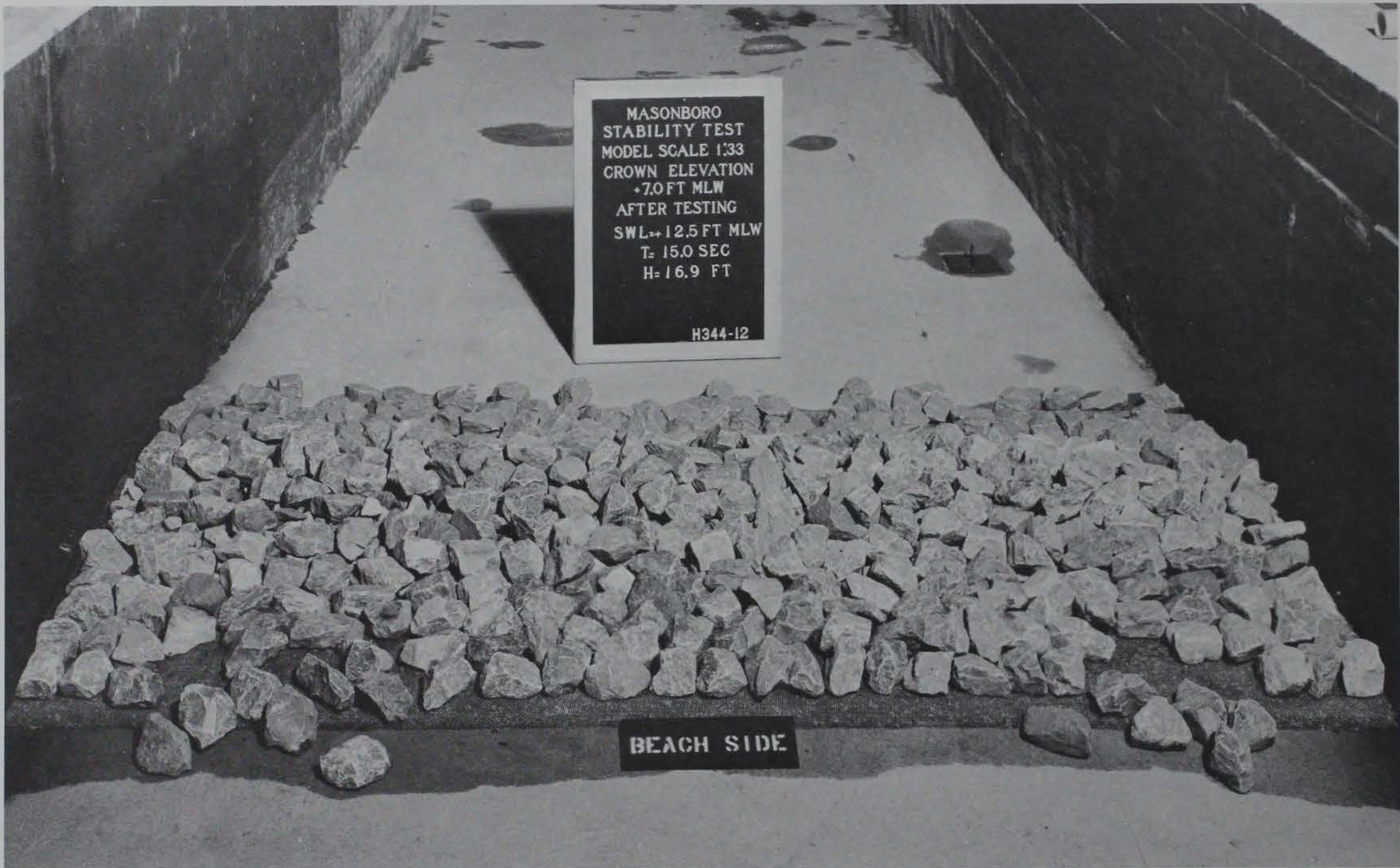


Photo 12. Beach-side view of Plan 3 after attack of 15-sec, 16.9-ft waves at an swl of +12.5 ft mlw for 2 prototype hr

Photo 13. Overhead view of Plan 3 after attack of 15-sec, 16.9-ft waves at an swl of +12.5 ft mlw for 2 prototype hr

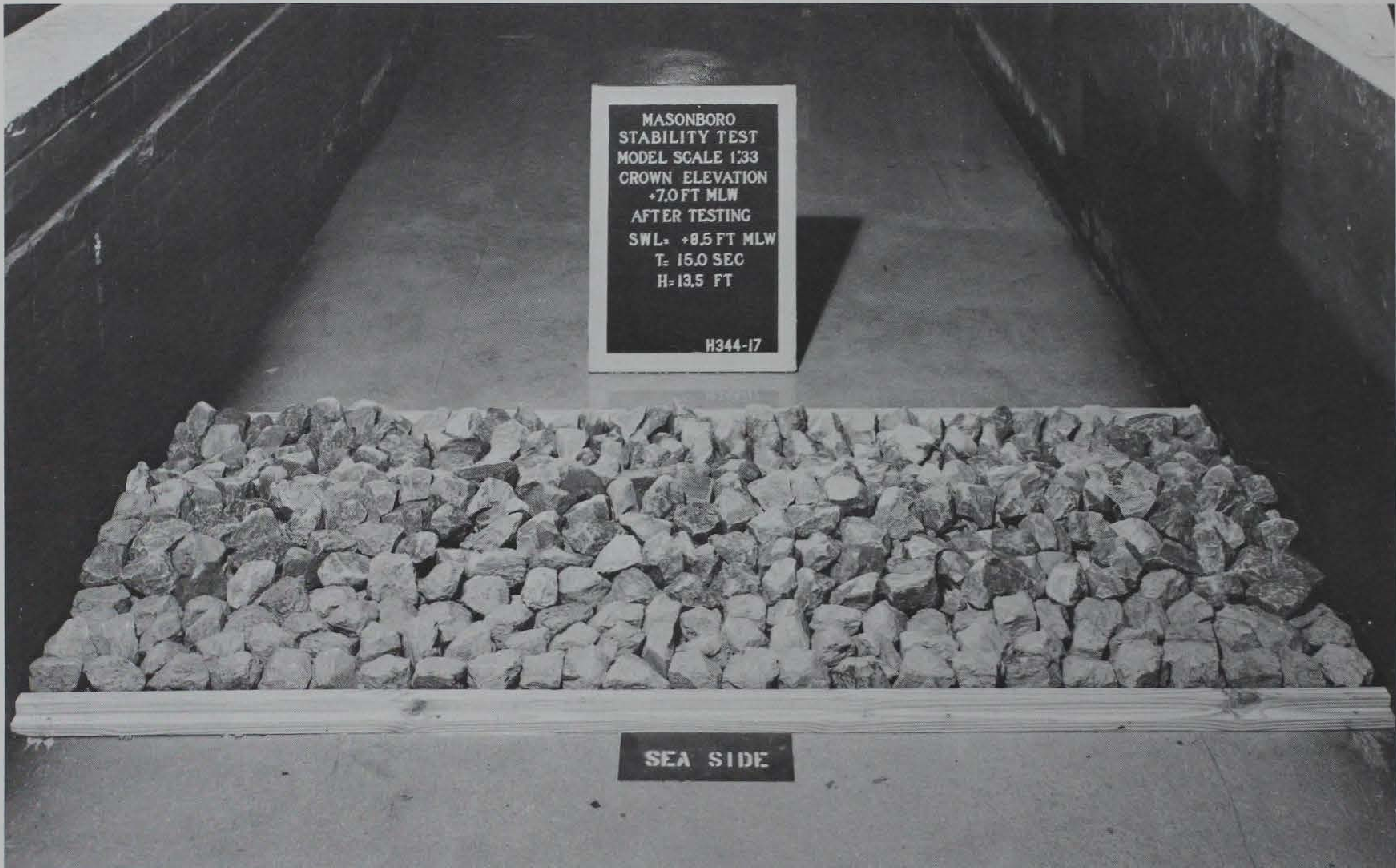


Photo 14. Sea-side view of Plan 3A after attack of 15-sec, 13.5-ft waves at an swl of +8.5 ft mlw for 2 prototype hr

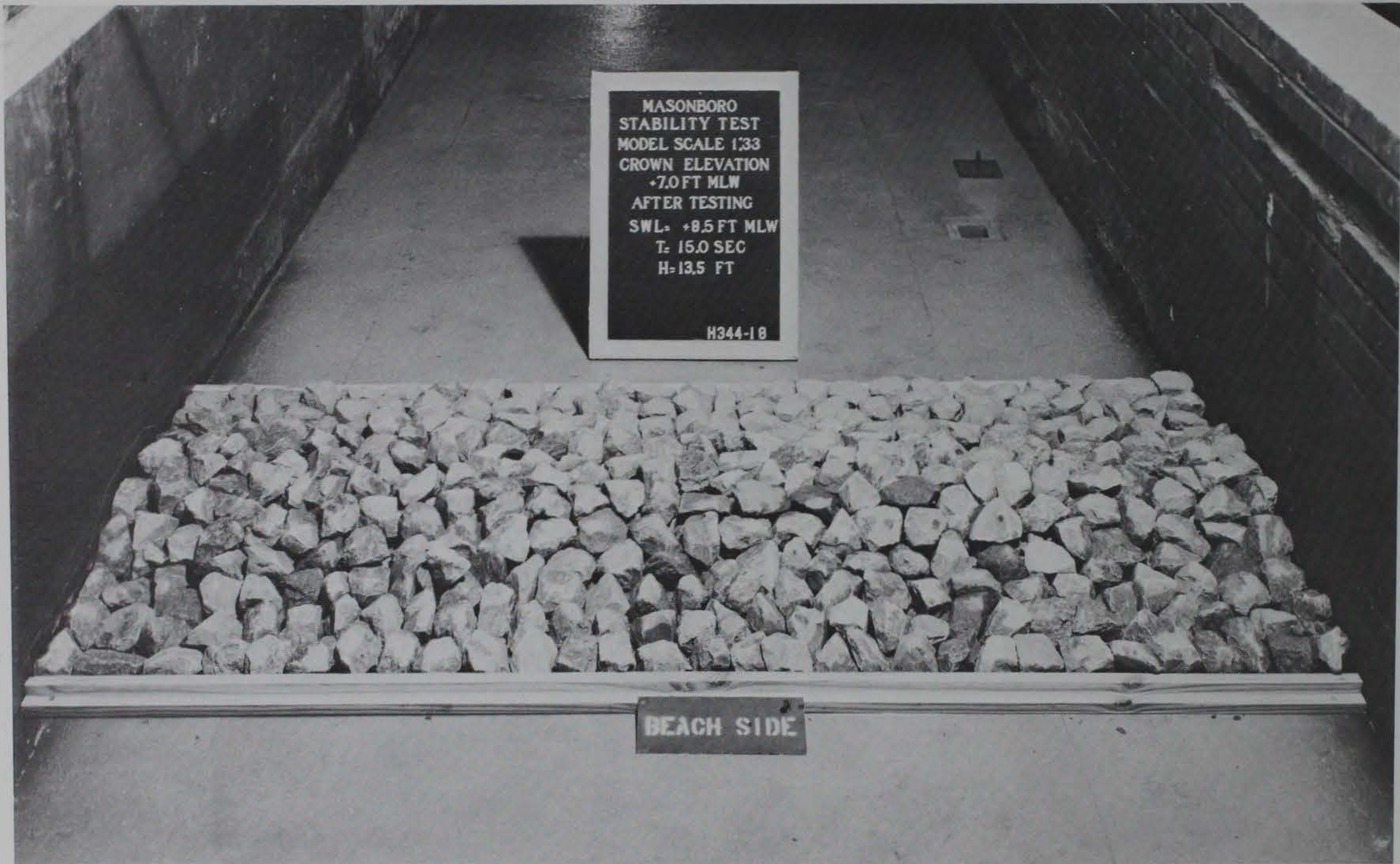


Photo 15. Beach-side view of Plan 3A after attack of 15-sec, 13.5-ft waves at an swl of +8.5 ft mlw for 2 prototype hr

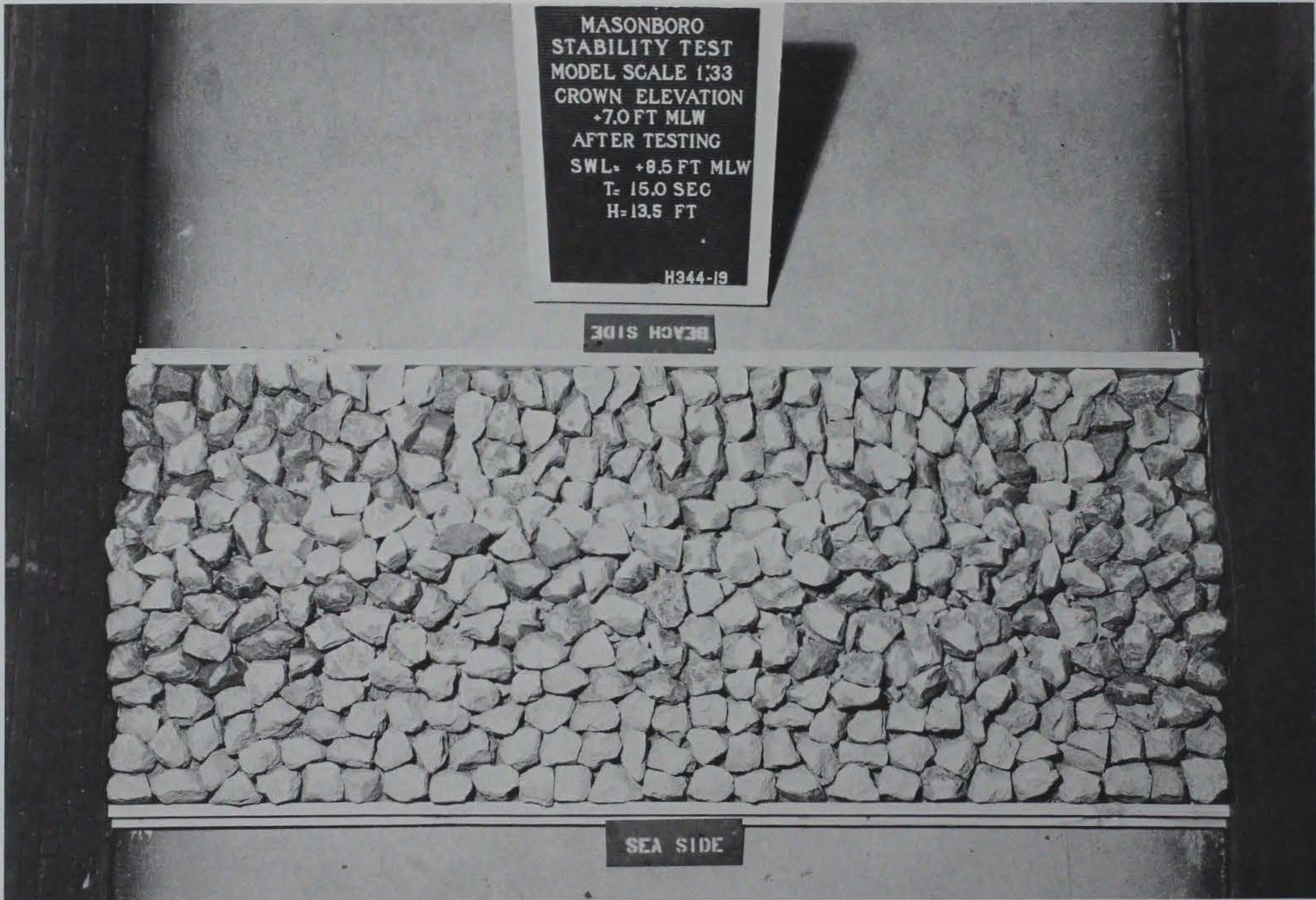


Photo 16. Overhead view of Plan 3A after attack of 15-sec, 13.5-ft waves at an swl of +8.5 ft mlw for 2 prototype hr

Photo 17. Sea-side view of Plan 3A after attack of 15-sec, 15-ft waves at an swl of +10.5 ft mlw for 2 prototype hr

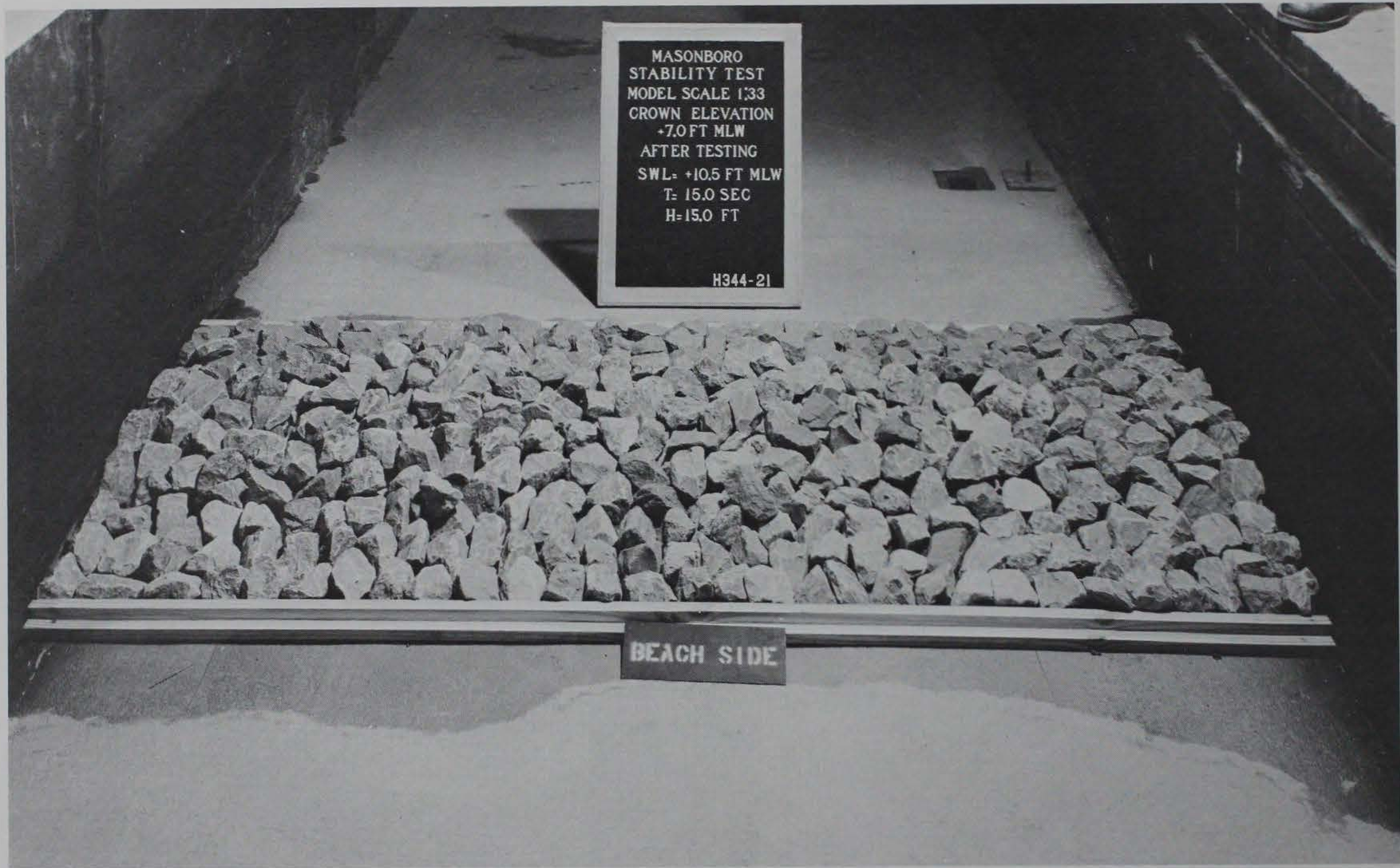


Photo 18. Beach-side view of Plan 3A after attack of 15-sec, 15-ft waves at an swl of +10.5 ft mlw for 2 prototype hr

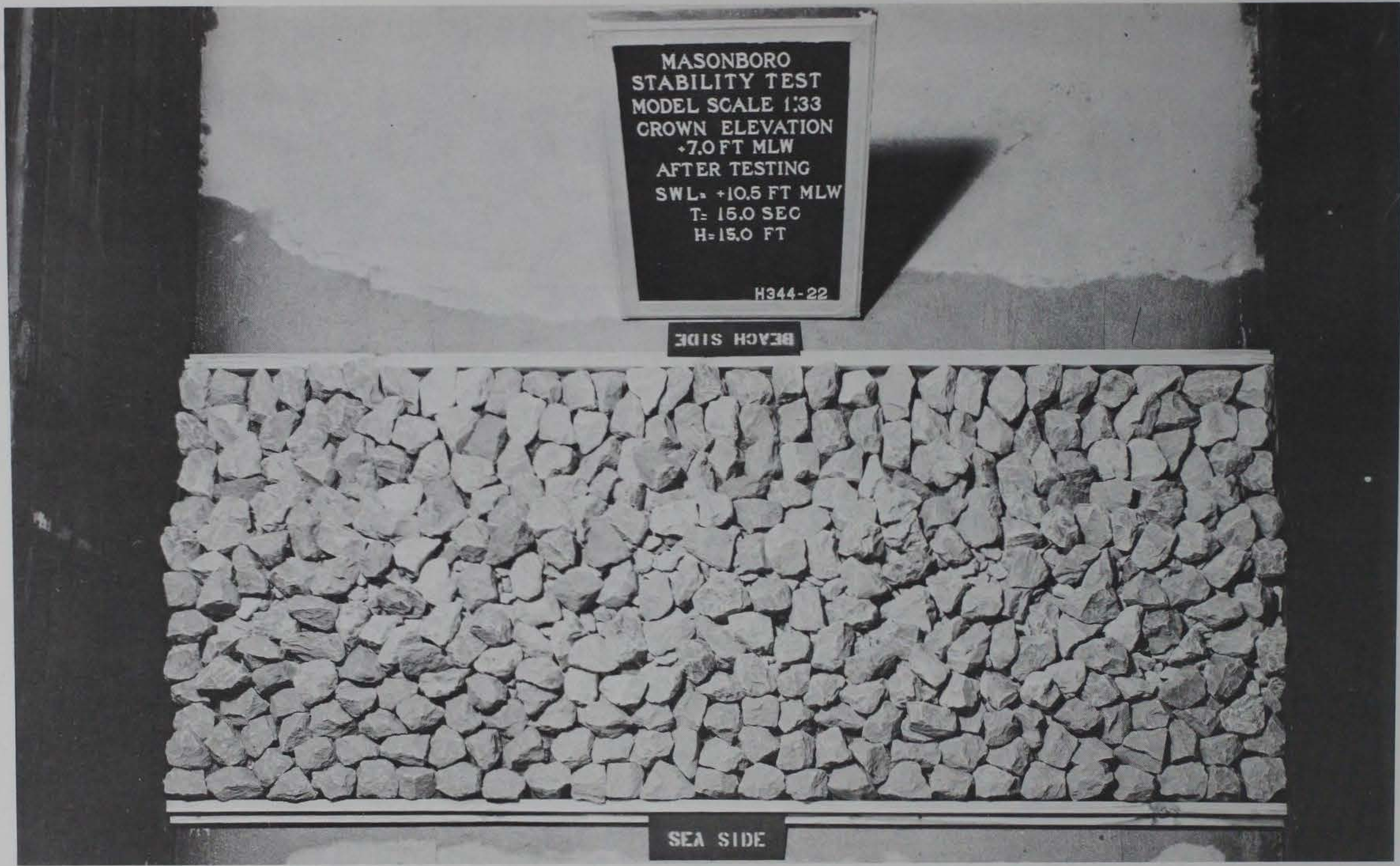


Photo 19. Overhead view of Plan 3A after attack of 15-sec, 15-ft waves at an swl of +10.5 ft mlw for 2 prototype hr

Photo 20. Sea-side view of Plan 3A after attack of 15-sec, 16.9-ft waves at an swl of +12.5 ft mlw for 2 prototype hr

Photo 21. Beach-side view of Plan 3A after attack of 15-sec, 16.9-ft waves at an swl of +12.5 ft mlw for 2 prototype hr

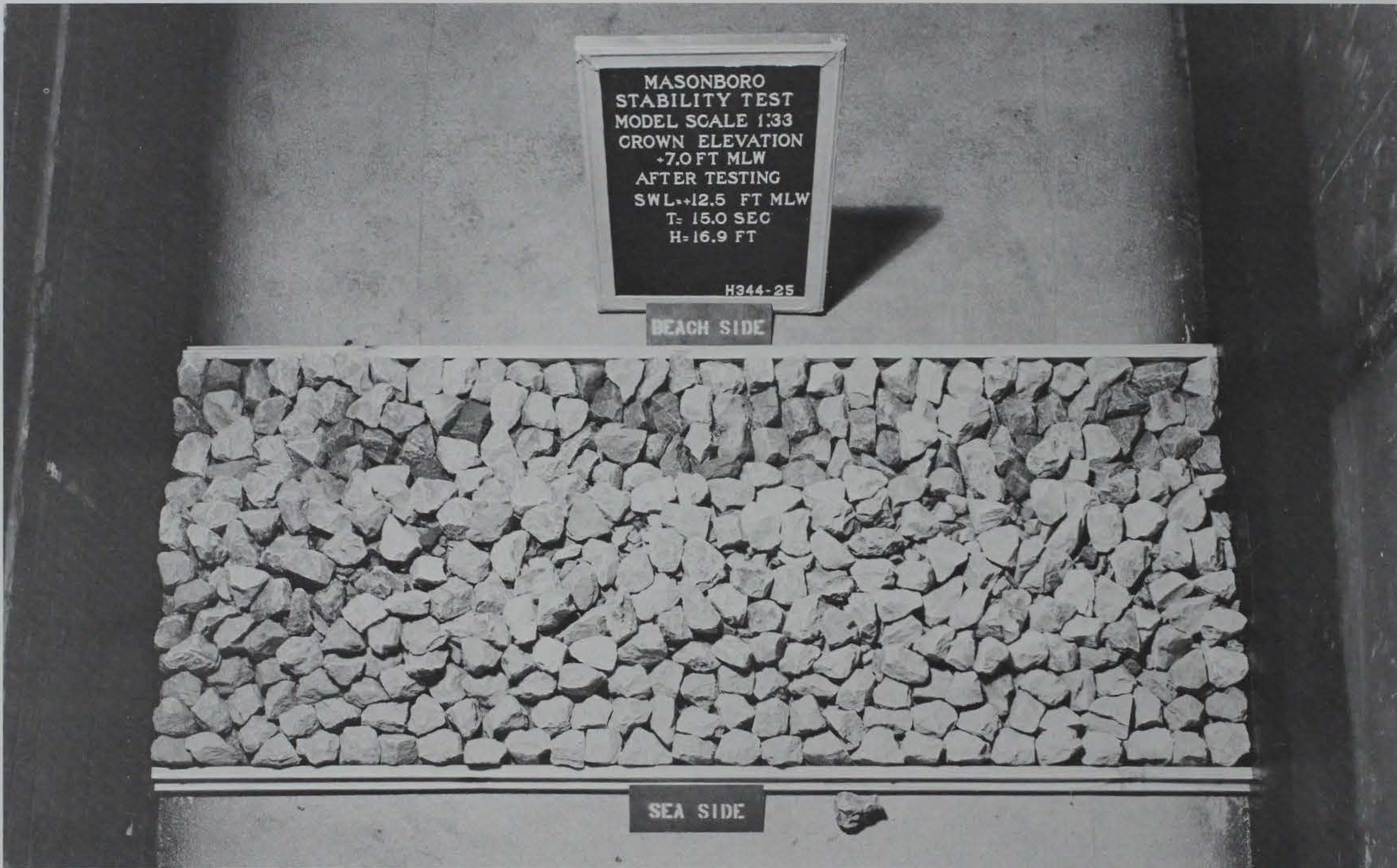
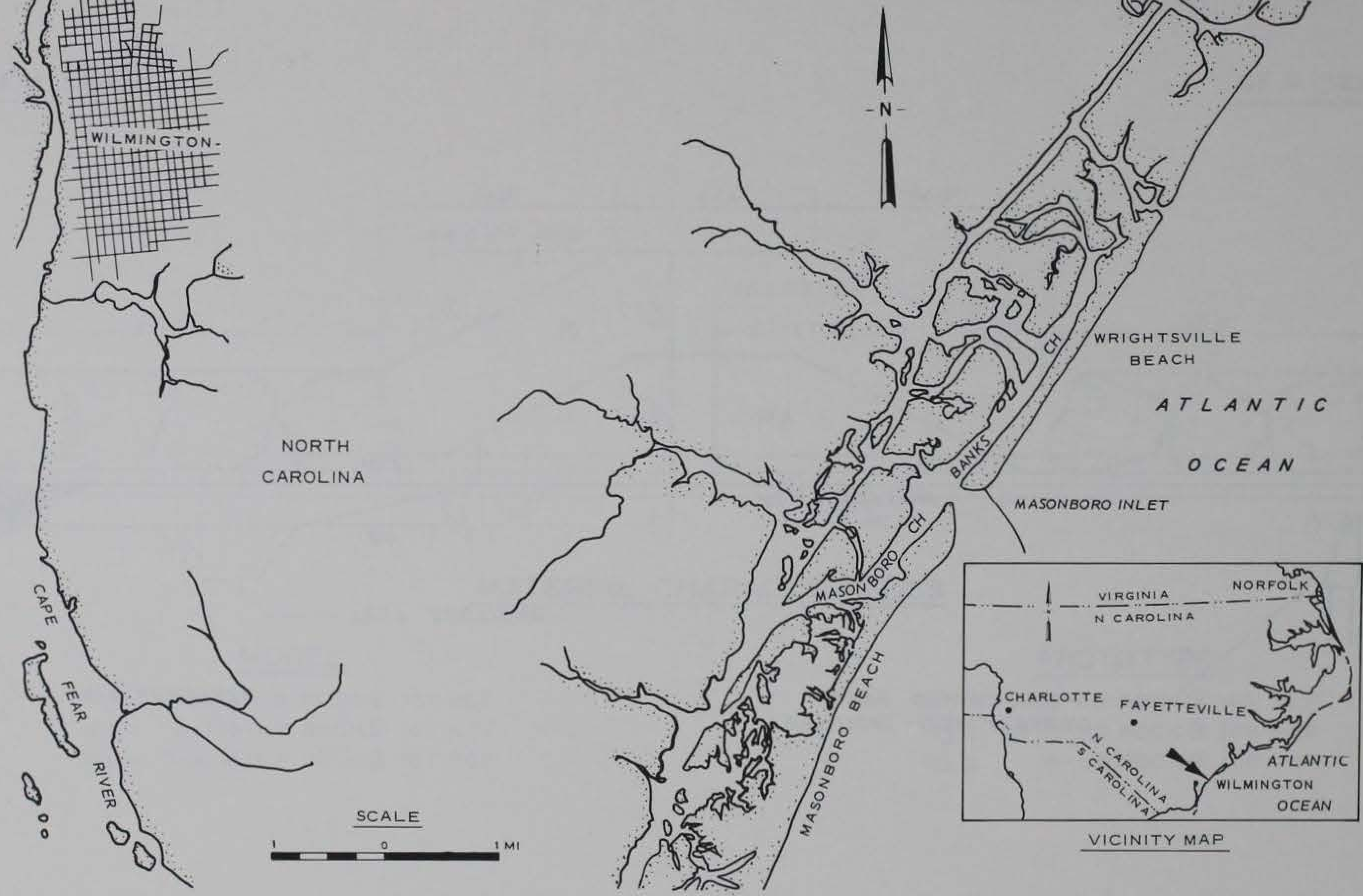
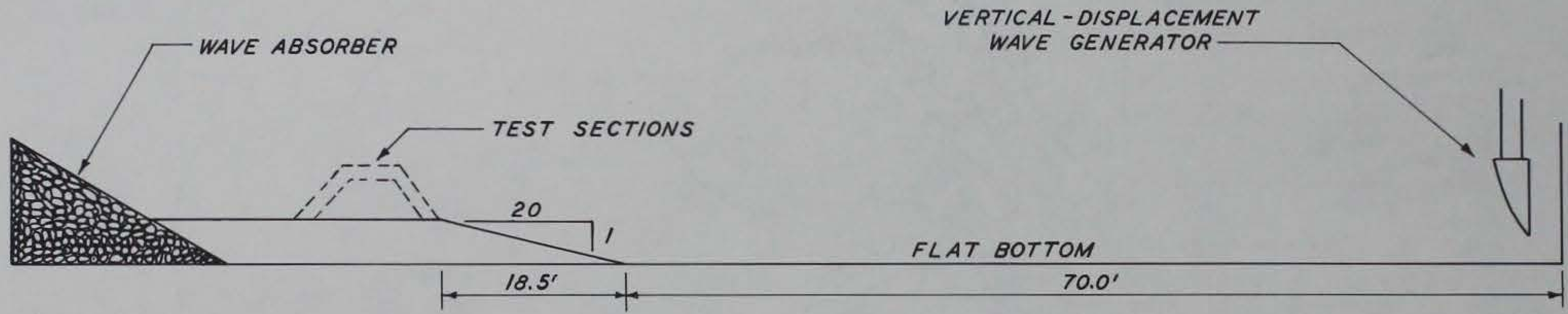


Photo 22. Overhead view of Plan 3A after attack of 15-sec, 16.9-ft waves at an swl of +12.5 ft mlw for 2 prototype hr

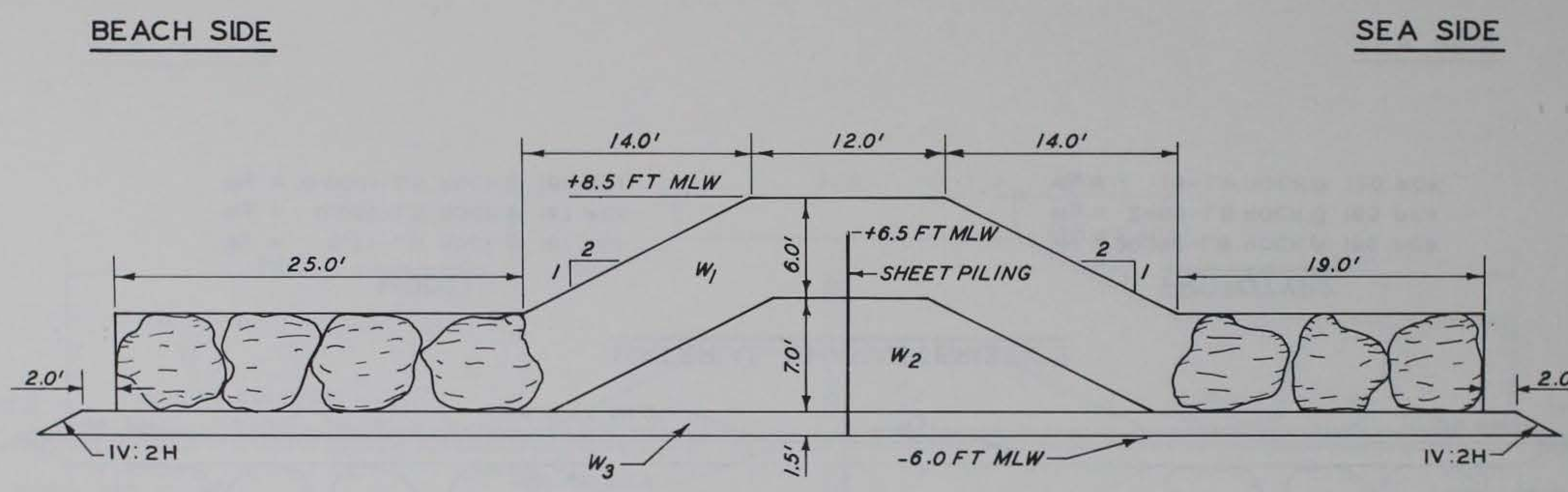


LOCATION MAP

PLATE 1



WAVE FLUME CROSS SECTION

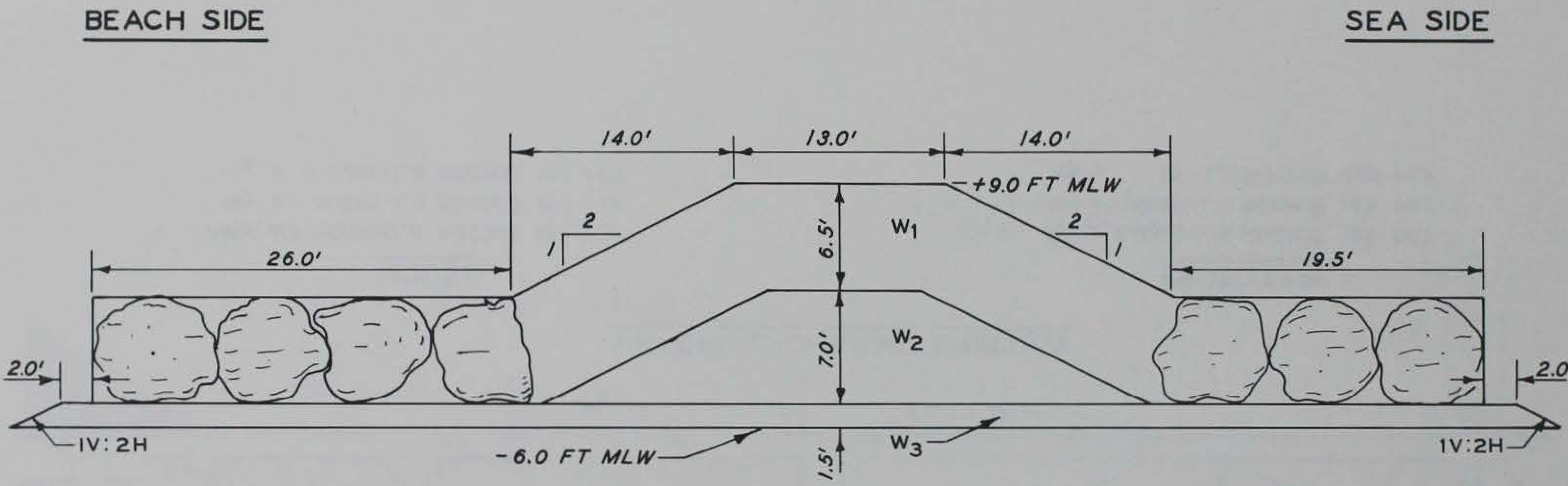


MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
W_1 = 0.55-LB ROCK @ 167 PCF	W_1 = 24000-LB ROCK @ 165 PCF
W_2 = 0.056-LB ROCK @ 167 PCF	W_2 = 2400-LB ROCK @ 165 PCF
W_3 = 0.0001-LB ROCK @ 167 PCF	W_3 = 18-LB ROCK @ 120 PCF

PLAN 1
TYPICAL BREAKWATER CROSS SECTION

PLATE 4



MATERIAL CHARACTERISTICS

MODEL

W_1 = 0.71-LB ROCK @ 167 PCF
W_2 = 0.056-LB ROCK @ 167 PCF
W_3 = 0.0001-LB ROCK @ 167 PCF

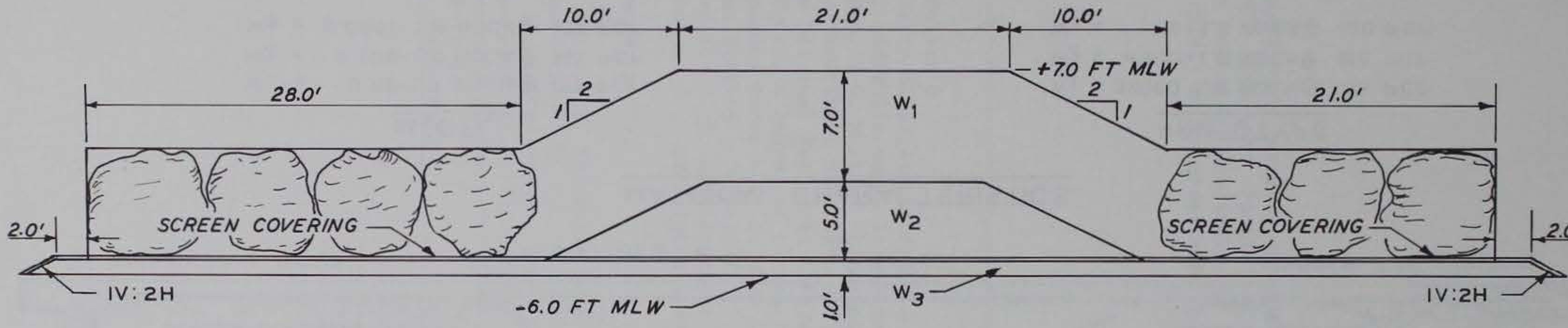
PROTOTYPE

W_1 = 30000-LB ROCK @ 165 PCF
W_2 = 2400-LB ROCK @ 165 PCF
W_3 = 18-LB ROCK @ 120 PCF

PLAN 2
TYPICAL BREAKWATER CROSS SECTION

BEACH SIDE

SEA SIDE



MATERIAL CHARACTERISTICS

MODEL

W_1 = 0.86-LB ROCK @ 167 PCF
W_2 = 0.108-LB ROCK @ 167 PCF
W_3 = 0.0001-LB ROCK @ 167 PCF

PROTOTYPE

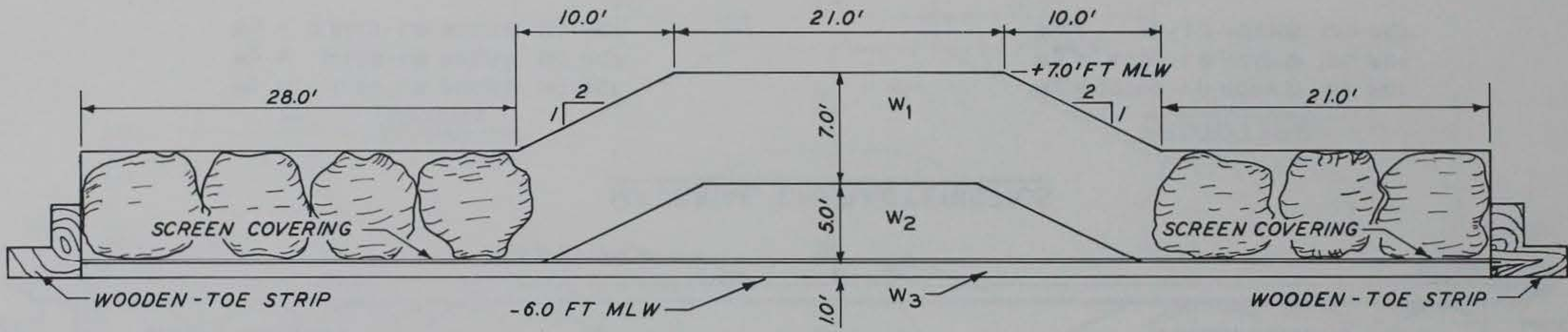
W_1 = 36000-LB ROCK @ 165 PCF
W_2 = 4600-LB ROCK @ 165 PCF
W_3 = 18-LB ROCK @ 120 PCF

PLAN 3

TYPICAL BREAKWATER CROSS SECTION

BEACH SIDE

SEA SIDE



MATERIAL CHARACTERISTICS

MODEL

W_1 = 0.86-LB ROCK @ 167 PCF
W_2 = 0.108-LB ROCK @ 167 PCF
W_3 = 0.0001-LB ROCK @ 167 PCF

PROTOTYPE

W_1 = 36000-LB ROCK @ 165 PCF
W_2 = 4600-LB ROCK @ 165 PCF
W_3 = 18-LB ROCK @ 120 PCF

PLAN 3A

TYPICAL BREAKWATER CROSS SECTION