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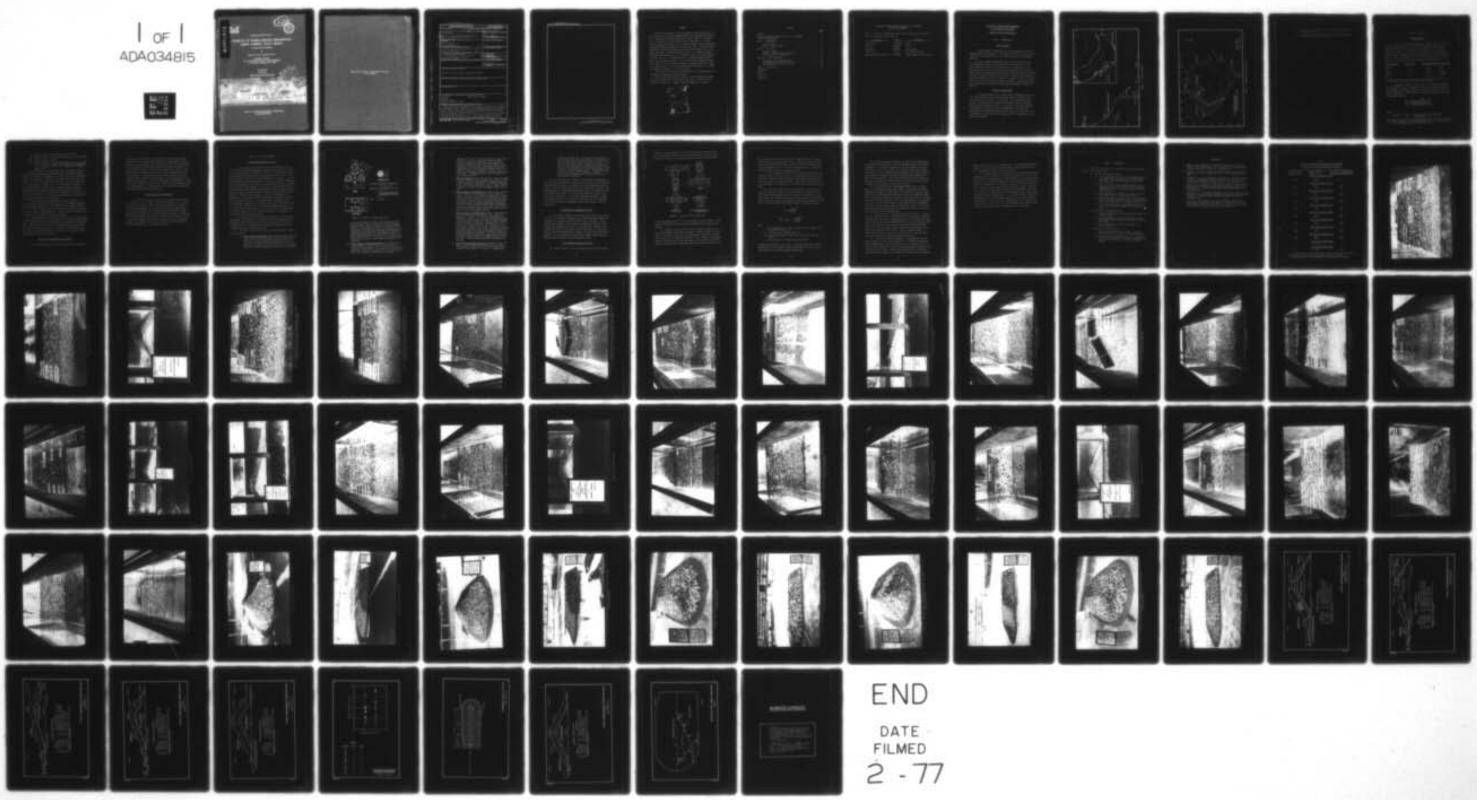

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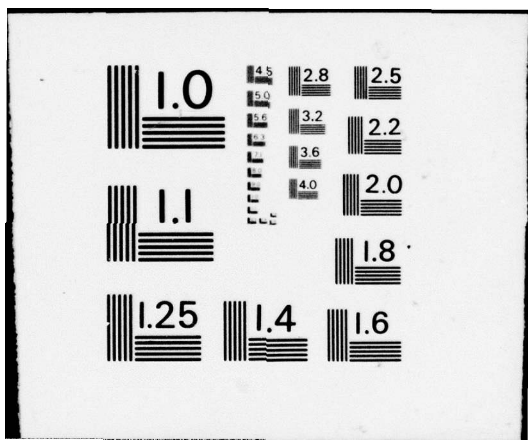
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TECHNICAL REPORT H-76-20

STABILITY OF RUBBLE-MOUND BREAKWATER JUBAIL HARBOR, SAUDI ARABIA

Hydraulic Model Investigation

by

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December 1976

Final Report

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Prepared for U. S. Army Engineer Division, Mediterranean
Livorno, Italy 09019

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An undistorted-scale hydraulic model study was conducted to investigate the adequacy of seven breakwater cross sections considered for use at Jubail Harbor, Saudi Arabia. Tests conducted in evaluating the various plans consisted of (a) two-dimensional stability tests (Plans 1, 2, 2A, 3, 4, and 5), (b) two-dimensional transmission tests (Plans 1, 2, 3, and 5), and (c) three-dimensional stability tests (Plan 6). Test results indicate that for the selected design conditions 5.0-metric-ton dolos armor will be of adequate size for use on all portions of the Jubail breakwater system.

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PREFACE

The model investigation reported herein was requested by the U. S. Army Engineer Division, Mediterranean (MDD) in a conference held at the U. S. Army Engineer Waterways Experiment Station (WES) on 13 December 1974. Authorization by the Office, Chief of Engineers, U. S. Army, had been granted along with that of the harbor model on 24 October 1973. Funding authorization by MDD was on 27 January 1975. Model tests were conducted at WES intermittently during the period January 1975 to February 1976 under the general direction of Mr. H. B. Simmons, Chief of the Hydraulics Laboratory, Dr. R. W. Whalin, Chief of the Wave Dynamics Division, and Mr. D. D. Davidson, Chief of the Wave Research Branch. Tests were conducted by Messrs. R. D. Carver, research hydraulic engineer, and A. W. Garcia, project engineer, assisted by Messrs. C. Lewis, W. G. Dubose, and R. W. Williams, engineering technicians. This report was prepared by Messrs. Carver and Davidson.

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

Directors of WES during the conduct of this study and the preparation and publication of this report were COL G. H. Hilt, CE, and COL John L. Cannon, CE. Technical Director was Mr. F. R. Brown.

CONTENTS

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

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

Multiply	By	To Obtain
centimetres	0.03937007	inches
metres	3.280839	feet
kilograms	2.204622	pounds (mass)
metric tons	1.10231	tons (2000 lb, mass)
grams per cubic centimetre	0.0361273	pounds (mass) per cubic inch

STABILITY OF RUBBLE-MOUND BREAKWATER
JUBAIL HARBOR, SAUDI ARABIA

Hydraulic Model Investigation

PART I: INTRODUCTION

The Prototype

1. Jubail Harbor is proposed to be located immediately south of Jubail, Saudi Arabia (Figure 1). The town of Jubail lies on the western coastline of the Arabian Gulf in the eastern province of the Arabian Peninsula.

2. Jubail Harbor, being developed as part of the Royal Saudi Naval Expansion Program, will provide a base of operations for the Saudi Naval East Flotilla Force operating in the Arabian Gulf. The base will provide facilities for berthing, maintaining, and repairing naval ships. Additionally, provisions for berthing and fueling visiting naval and commercial ships will be provided. The berthing, maintenance, and repair facilities will be located on an offshore island; the island will be connected to shore by a landfill causeway. The proposed harbor will be protected by a system of rubble-mound breakwaters as shown in Figure 2.

Purpose of Model Study

3. During the design of the Jubail Harbor breakwater system, the U. S. Army Engineer Division, Mediterranean (MDD), considered using either natural rock, tribar, or dolos armor. Consideration was also given to the feasibility of using a cast-in-place concrete crownwall atop the rock and tribar sections. The purpose of the model study was to investigate through the use of two-dimensional (2-D) and three-dimensional (3-D) breakwater-stability tests and 2-D transmission tests the adequacy of the various plans proposed by MDD. Specifically, it was desired to determine:

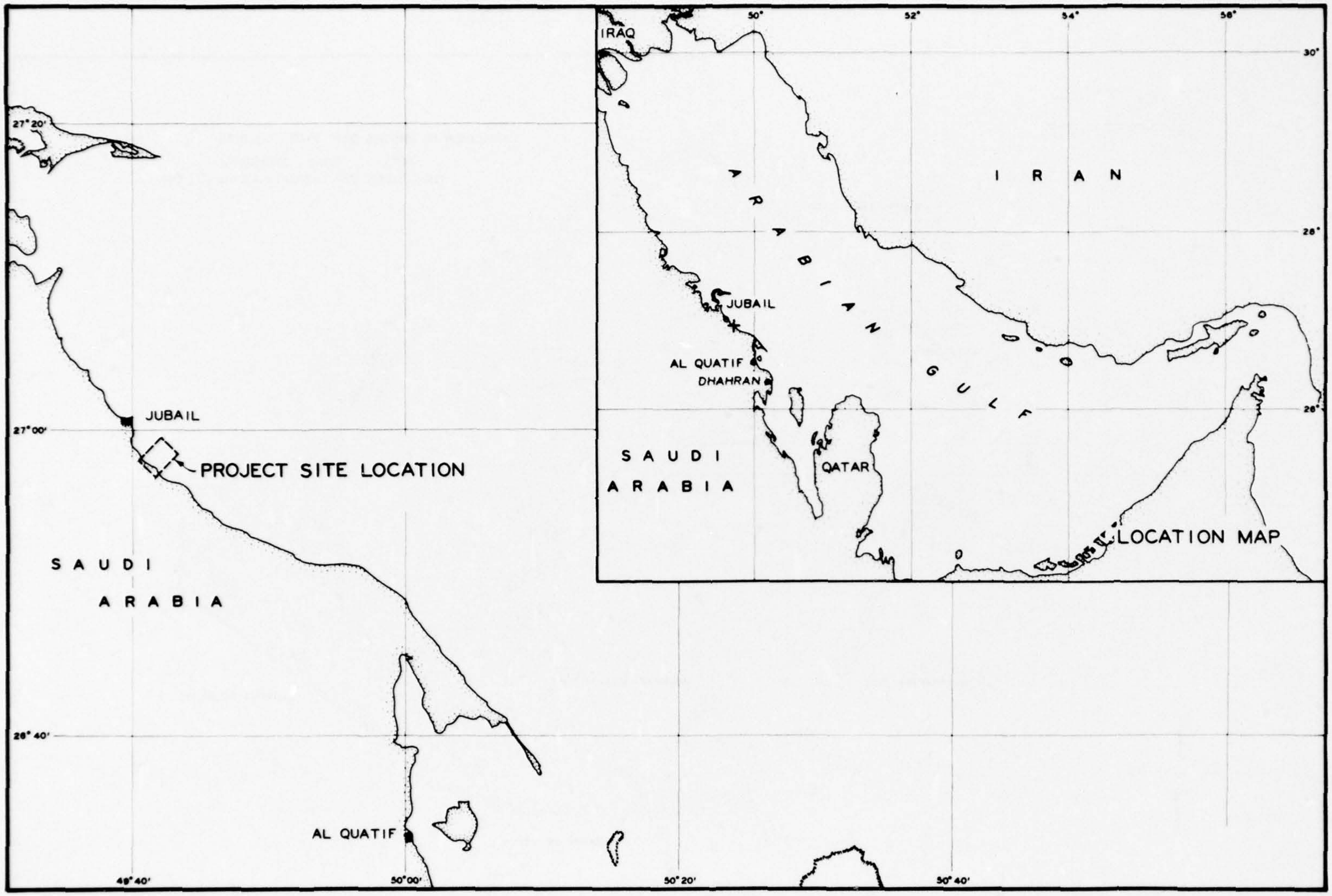

Figure 1. Project location

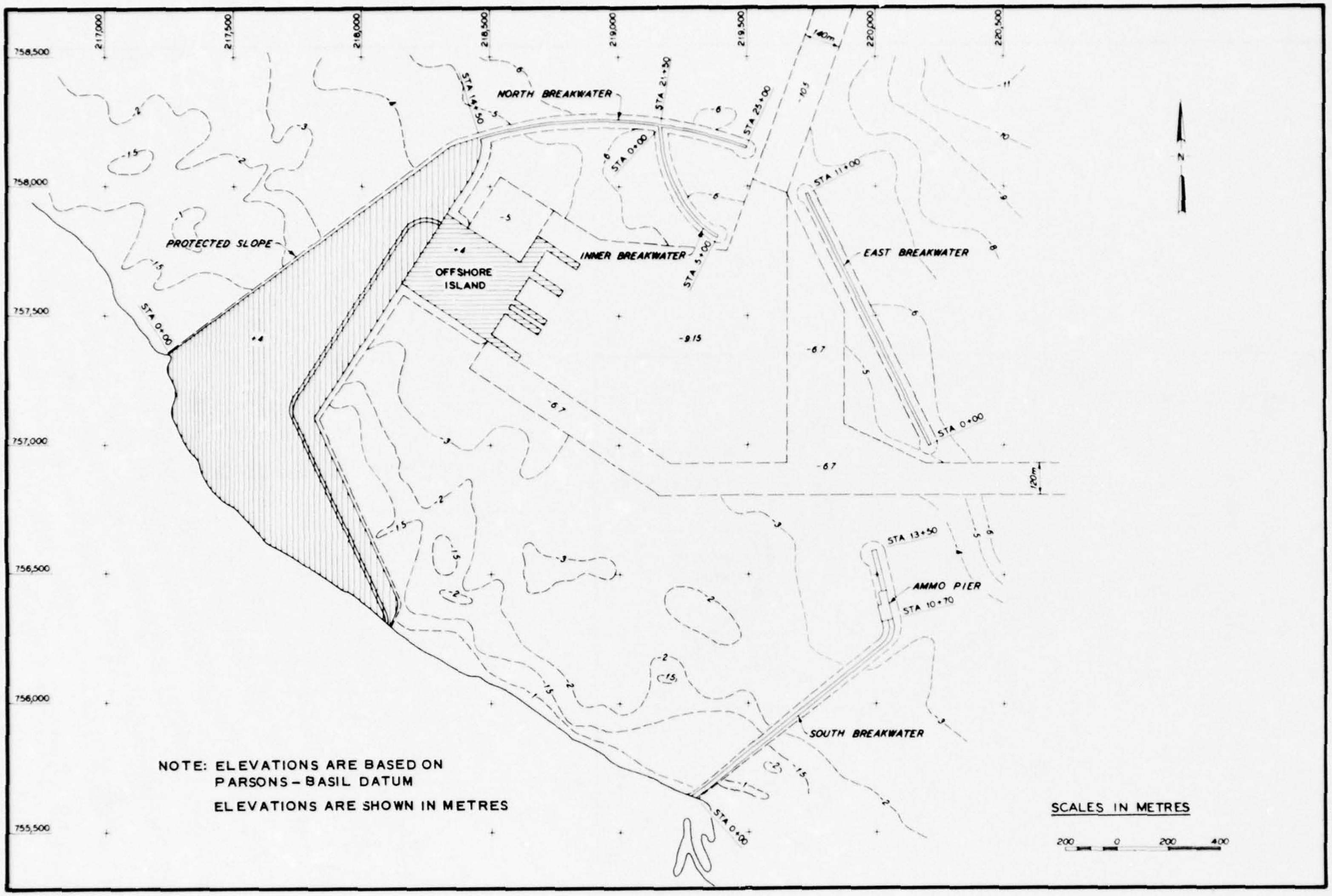

Figure 2. Breakwater configuration

a. The stability of the various sections when attacked by selected test waves.

b. The magnitude of transmitted wave heights associated with selected incident wave conditions.

PART II: THE MODEL

Design of Model

4. Tests were conducted at undistorted linear scales of 1:18 and 1:28, model to prototype. Scale selection was based on the size of model units available compared with the estimated size of prototype armor units required for stability, the preclusion of stability scale effects,[1] and the capabilities of the available wave tank and wave generator. Based on Froudian model law[2] and the linear scales of 1:18 and 1:28, the following model-to-prototype relations were derived:

Characteristics	Dimensions*	Model:Prototype Scale Relations	
Length	L	1:18	1:28
Area	L^2	1:324	1:784
Volume	L^3	1:5832	1:21,952
Time	T	1:4.24	1:5.29

* Dimensions are in terms of length (L) and time (T).

5. The specific weight of water used in the model was assumed to be 1.0 g/cm^3** and that of seawater, 1.025 g/cm^3. Specific weights of model breakwater construction materials were not the same as their prototype counterparts. These variables were related using the following transference equation:

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

where

subscripts m and p = model and prototype quantities, respectively

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

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

γ_r = specific weight of an individual armor unit or rock, g/cm^3

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, g/cm^3.

6. In a hydraulic model investigation of this nature, gravitational forces predominate (Froudian model law[1]), except when energy transmission through the breakwater is considered.[3] Energy transmission through the breakwater is dependent on viscous forces and, hence, dependent on the Reynolds number ($\mathfrak{R}_e = VL/\nu$) where V is the velocity, L is a characteristic length, and ν is the kinematic viscosity. If the core material were geometrically or weight scaled according to Froudian model relationships, internal Reynolds numbers would be too low and insufficient wave energy would be transmitted. Therefore, for all plans considered, the core material was geometrically oversized to ensure proper transmission of wave energy.

7. For those plans which employed a concrete crownwall, the model crownwall sections reproduced both prototype geometry and weight to ensure dynamic similarity. The 170-metric-ton prototype sections (10 × 4.5 × 3.0 m) were reproduced by 7.75 kg (1:28 scale) and 29.18 kg (1:18 scale) model crownwall sections. The model sections were made of concrete and cast separately from the structures.

8. The maximum waves selected for testing (4.1 m to 4.9 m) could not always be generated in the test flume for the correctly scaled water depth. Therefore, for the 2-D stability tests conducted at the deeper water location and for all of the 3-D stability tests, the test structures and their related still-water levels (swl's) were raised until there was sufficient water depth in the flume to allow waves of specified heights to reach the structures.

Method of Constructing Test Sections

9. All model breakwater sections were constructed to reproduce

as closely as possible the usual methods of constructing prototype breakwaters. The 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 structure. Once the core material was in place, it was sprayed with a low-velocity water hose to ensure complete saturation of the material. The underlayer stone was then added by shovel and smoothed to grade by hand or with trowels. No excessive pressure or compaction was applied to any of the underlayer stone placements. Armor units (rock, tribars, and dolosse) used in the cover layers were placed in a random manner, i.e., laid down in such a way that no intentional interlocking of the units was obtained.

Test Facilities and Equipment

10. The 3-D tests were conducted in an L-shaped concrete flume 76 m long, 15 m and 24 m wide at the top and bottom of the L, respectively, and 1.4 m deep. The 2-D tests were conducted in a flume 1.5 m wide and approximately 31 m long located within the L-shaped flume. The flumes were equipped with a paddle-type wave generator capable of producing sinusoidal waves of various periods and heights. Changes in water-surface elevation 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.

PART III: TESTS AND RESULTS

Two-Dimensional Stability Tests

11. The adequacy of six potential plans of improvement for the Jubail Harbor rubble-mound breakwater system was investigated by means of 2-D stability tests. Stability of Plans 1 and 5 was checked at both a shallow-water and deeper water location whereas Plans 2, 2A, 3, and 4 were checked for stability at only the deeper water location. It was desired by MDD that the various plans be stable for a 4.1-m breaking wave in the shallow-water location and a 4.1-m nonbreaking wave in the deeper water location at wave periods of 7 and 9 sec. These conditions were selected because they represent design conditions for the maximum breaking wave height at the shallow-water location and the significant wave height at the deeper water location. The swl's designated for the shallow-water location were 0.00 m Parsons-Basil Datum (PBD) for Plan 1 and +2.35 m PBD for Plan 5. At the deeper water location, a swl of +2.35 m PBD was designated for all plans except Plan 5 which used a swl of +1.50 m PBD. Selected test sections which remained stable for the 4.1-m waves in the deeper water location were tested with a higher wave to indicate safety factors in the various designs. A 4.9-m wave was arbitrarily selected for these tests.

12. It was determined during testing of Plan 1 that at both the shallow-water and deeper water locations the 9-sec wave period was more detrimental to the structural integrity of the test section than the 7-sec wave period. Consequently, all subsequent stability tests were conducted with only the 9-sec wave period.

13. The plans tested, specific test conditions, and test results were as follows:

a. Plan 1 (Plate 1 and Photos 1-3) used a crown elevation of +5.75 m PBD, toe elevations of -6.00 m PBD (shallow-water location) and -9.00 m PBD (deeper water location), and slopes of 1:1.5 both seaside and beachside. The structure used 5.75-metric-ton tribar armor (Figure 3) on the seaside face, 0.95-metric-ton rock armor on the beachside face, and a concrete crownwall as described in paragraph 7.

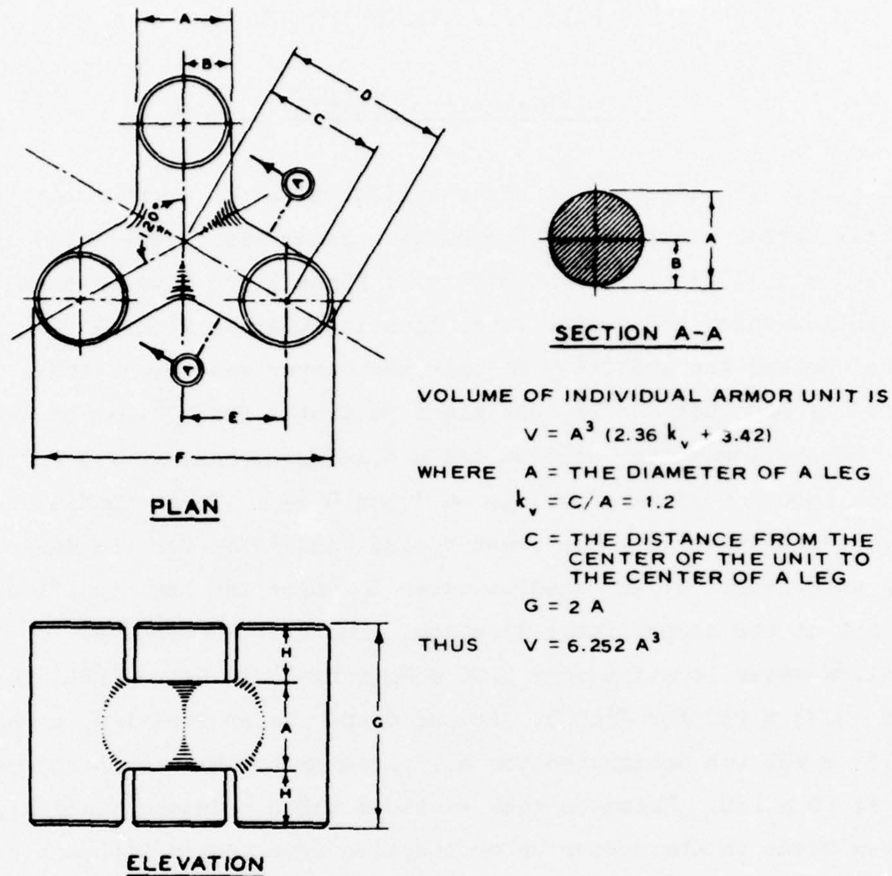

Figure 3. Details of tribar armor unit

At the shallow-water location, the structure was completely stable. Photos 4 and 5 show the test section after attack of 9-sec, 4.1-m waves in the shallow-water location. Under attack of 9-sec, 4.1-m waves in the deeper water location, the tribar armor experienced minor displacement, the beachside rock experienced moderate downslope displacement, and the crownwall failed. The deeper water after-testing condition of the structure is shown in Photos 6 and 7. Toe stone in the seaward apron was stable for both the shallow-water and deeper water locations.

b. Plan 2 (Plate 2 and Photos 8 and 9) was constructed to a crown elevation of +5.75 m PBD, a toe elevation of -9.00 m PBD, and slopes of 1:3.5 and 1:1.5 seaside and beachside, respectively. The section used 3.5-metric-ton rock armor (seaside), 0.95-metric-ton rock armor (beachside), and a concrete crown wall as described in paragraph 7. Photo 10 shows the structure under attack by 9-sec, 4.1-m waves. This wave condition caused moderate downslope displacement of both seaside and beachside armor and a slight beachward movement of the crownwall. Attack by 9-sec, 4.9-m waves caused severe displacement of both seaside and beachside armor and a complete failure of the crownwall. Photos 11 and 12 illustrate the damage incurred during attack of the 9-sec, 4.9-m waves. No damage occurred to the toe stone for either wave condition.

c. Plan 2A (Plate 2) was the same as Plan 2 except one layer of first underlayer rock was bonded to the bottom of the crownwall to more closely simulate casting-in-place prototype construction techniques. The stability response of the structure was the same as Plan 2. Photos 13 and 14 show the structure after testing with 9-sec, 4.9-m waves.

d. Plan 3 (Plate 3 and Photos 15-17) used a crown elevation of +5.00 m PBD, a toe elevation of -9.00 m PBD, and slopes of 1:3.5 and 1:1.5 seaside and beachside, respectively. The armor consisted of 3.5-metric-ton rock on the seaside slope, across the crown, and down the beachside slope. Both the seaside and beachside slopes experienced moderate downslope displacement under attack of 9-sec, 4.1-m waves and extensive downslope displacement when attacked by 9-sec, 4.9-m waves. Photos 18 and 19 show the structure after testing with 9-sec, 4.9-m waves. No toe stone damage occurred.

e. Plan 4 (Plate 4 and Photos 20-22) used a crown elevation of +5.00 m PBD, a toe elevation of -9.00 m PBD, and slopes of 1:2.25 and 1:1.5 seaside and beachside, respectively. Plans 2, 2A, and 3 had reproduced prototype armor-rock specific weights of only 2.24 g/cm^3; however, it was later learned that the prototype specific weight could be as high as 2.64 g/cm^3. Therefore, the 4.5-metric-ton rock armor used on Plan 4 reproduced a prototype specific weight of 2.64 g/cm^3. Subjection to 9-sec, 4.1-m waves produced extensive downslope displacement of the seaside armor with one layer of material being completely displaced in some areas. Both the beachside and crown armor experienced moderate displacement; however, the crown was not breached. No damage occurred to the toe stone. Photos 23 and 24 show the structure after testing.

f. Plan 5 (Plate 5 and Photos 25-27) employed a crown elevation of +5.00 m PBD, toe elevation of -6.00 m PBD (shallow-water location) and -9.00 m PBD (deeper water location), and slopes of 1:2.0 and 1:1.5 seaside and beachside, respectively. The structure used 6.5-metric-ton rock armor with specific weight of 2.64 g/cm^3, which basically proved to be stable for 9-sec, 4.1-m waves in both the shallow-water and deeper water locations. There was very minor displacement of armor on the beachside slope at the deeper water location and only minor rocking of armor on the seaside slope at the shallow-water location. No toe stone damage occurred. The after-testing stability condition of the structure is shown in Photos 28 and 29 (shallow-water location) and 30 and 31 (deeper water location).

14. In conducting the stability tests described in paragraph 13, test sections were subjected to wave attack in 30-sec intervals between which the wave generator was stopped and the wave energy was allowed to dissipate. This procedure was necessary to prevent the structures from being subjected to an undefined wave system created by reflections from the wave-generator paddle. Structures were subjected to wave attack until they stabilized, i.e., until all significant movement of breakwater material abated or until failure occurred. Also, all stability test results presented in paragraph 13 were verified by at least one repeat test.

Two-Dimensional Transmission Tests

15. Tests were conducted in the deeper water location on Plans 1, 2, 3, and 5 to determine the magnitude of transmitted wave heights associated with a selected range of incident wave conditions. It was desired that transmitted wave heights not exceed 60.0 and 91.0 cm for incident wave heights of 3.1 and 4.0 m, respectively. Results of the transmission tests are presented in Table 1 and Plate 6. These data show all observed transmitted wave heights to be well below the desired maximum. For a given plan and incident wave height, transmitted wave heights always increased with increasing wave period.

Three-Dimensional Stability Tests

16. Based on results of the 2-D stability tests and anticipated

problems in obtaining adequate sizes and quantities of prototype armor rock, it was deemed advisable to consider alternate armor unit shapes. Dolos armor (Figure 4) has exhibited excellent stability characteristics

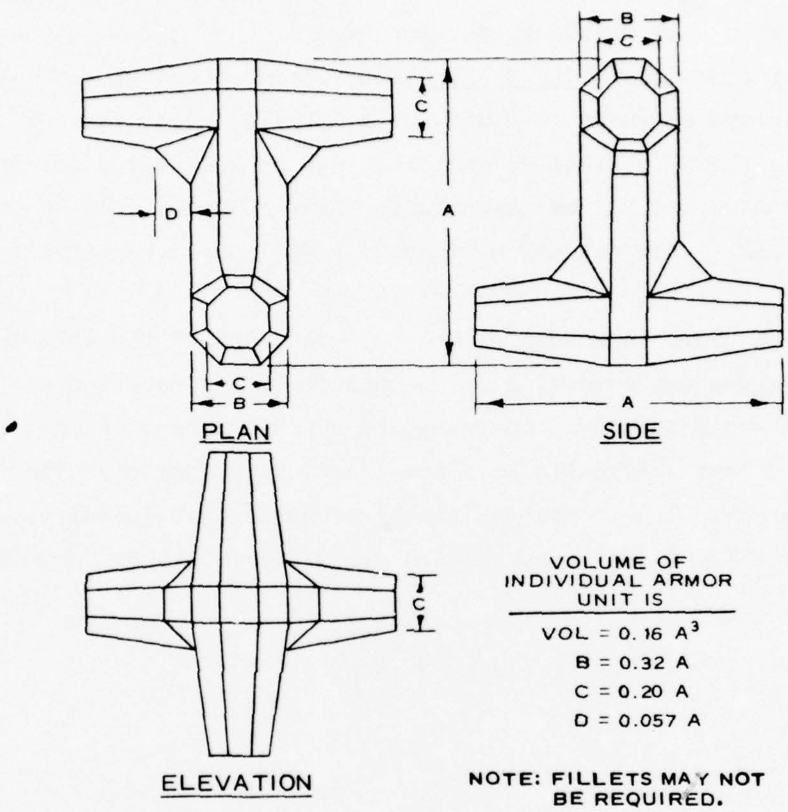

Figure 4. Details of dolos armor unit

in previous model studies.[4,5] It was therefore decided by MDD to investigate the feasibility of armoring the breakwater with 5.0-metric-ton dolosse.

17. Because storm waves produce more detrimental stability effects on breakwater heads than on breakwater trunks, an armor size which proves to be stable on the head can reasonably be assumed to be stable on the trunk for the same wave conditions. Also, because all breakwater heads in the system are of the same shape, it was possible to investigate the stability of all heads by modeling only one head section. The

head section chosen was the one that will experience the most severe wave conditions in the prototype. Thus, it was agreed that the armor protection affirmed by the selected head test would be sufficient for all the breakwater heads in the system and would provide conservative protection when used on the breakwater trunks.

18. The 3-D stability model (Plan 6) reproduced 90 m of the prototype structure as shown in Plate 7. A detailed cross section of the structure is shown in Plate 8; Photos 32 and 33 show the model breakwater as constructed in the wave flume. Plan 6 used a crown elevation of +5.00 m PBD, a toe elevation of -9.00 m PBD, and 5.0-metric-ton dolos armor.

19. As shown in Photos 32 and 33, placement of the dolosse in the cover layers was random, i.e., no particular orientation was followed which would purposely increase the natural interlocking of the units beyond that obtainable by a construction contractor. The thickness of the cover layers and the number of armor units required per unit area of cover layer can be calculated using the following formulas:

$$t = nk_\Delta \left(\frac{W_r}{\gamma_r}\right)^{1/3}$$

$$\frac{N_r}{A} = nk_\Delta \, (1 - P) \left(\frac{\gamma_r}{W_r}\right)^{2/3}$$

where

t = the thickness of n layers of armor units of weight, W_r, and specific weight, γ_r

k_Δ = a characteristic shape coefficient

N_r = the required number of armor units for a given surface area, A

P = the porosity of the cover layers in percent.

Presently used values of k_Δ and P, which were determined experimentally by a limited number of small-scale armor unit tests prior to this study, are 1.0 and 63 percent, respectively.

20. Specific prototype conditions for which it was desired that the proposed test section be checked for stability were a flat bottom topography of -6 m PBD and subjection of the test section to a 9-sec, 4.1-m wave at a swl of 0.0 m PBD. To satisfy the selected prototype conditions, it is not always possible to generate in the test facility the required wave height in the correctly scaled water depth (6 m). Thus, it was necessary to elevate the test section and its related swl until there was sufficient water depth (9 m) in the facility to allow the specified wave heights to reach the structure.

21. Based upon review of photographs taken of the 3-D harbor model tests conducted by WES during 1974-75 to determine the optimum breakwater alignment for Jubail Harbor,[6] two angles of the most severe wave attack on the breakwater heads were chosen. The angles of attack which the wave front makes with respect to the center line of the breakwater were 22° (wave direction 1) and 36° (wave direction 2), as shown in Plate 9. Of all possible angles of wave attack for all breakwater heads in the Jubail system, these two angles were chosen to be the wave conditions which should have the most detrimental effect on the structural integrity of the breakwater head.

22. Plan 6 proved to be completely stable for 9-sec, 4.1-m waves from both wave directions. Initially, Plan 6 was subjected to attack from wave direction 1 with no damage resulting. The structure was then rebuilt and the tests were repeated with the results verifying the first tests. Photos 34 and 35 show the structure after testing from wave direction 1. The structure was then rotated to wave direction 2, rebuilt, and subjected to wave attack. Again no damage occurred. A repeat test verified the results of the first test. Photos 36 and 37 show the structure after testing from wave direction 2.

23. It was also deemed advantageous to determine what the stability response of the structure would be if it should be attacked by waves somewhat larger than the design waves. To accomplish this, the structure was subjected to a 9-sec, 4.7-m breaking wave from each of the selected wave directions. Stability tests from both wave directions (again verified by repeat tests) showed the structure to be completely

stable for the 9-sec, 4.7-m breaking waves. The after-testing stability condition of the structure is shown in Photos 38 and 39 (wave direction 1) and Photos 40 and 41 (wave direction 2).

24. In conducting the 3-D stability tests, the structure was subjected to attack of the 9-sec, 4.1-m and 4.7-m waves for 2 hr and 1 hr (prototype time), respectively. Normal procedure, as discussed in paragraph 14, is to subject the structure to wave attack until stability is achieved or failure occurs. However, the stability response of Plan 6 was so favorable that no armor units even rocked in place. Therefore, the structure could have been considered to be stabilized after only one 30-sec cycle of wave attack and the cumulative time of wave attack became an academic consideration, i.e., the after-testing condition of the structure could reasonably be expected to be the same whether it was subjected to 20 min or 20 hr of attack by either of the selected-wave conditions. The 2- and 1-hr time increments were therefore arbitrarily selected and were not significant to the test results.

PART IV: CONCLUSIONS

25. Based on the results of the hydraulic model study reported herein, it is concluded that:

<u>a</u>. For the 2-D stability tests:

(1) Plan 1 was stable for 9-sec, 4.1-m breaking waves at the shallow-water location; but it was not stable for 9-sec, 4.1-m nonbreaking waves at the deeper water location.

(2) Plans 2, 2A, and 3 were marginally stable at the deeper water location for 9-sec, 4.1-m nonbreaking waves but failed to withstand the 9-sec, 4.9-m nonbreaking waves.

(3) Plan 4 was generally unstable for 9-sec, 4.1-m nonbreaking waves at the deeper water location.

(4) Plan 5 was stable for both the 9-sec, 4.1-m breaking waves at the shallow-water location and the 9-sec, 4.1-m nonbreaking waves at the deeper water location.

(5) Plan 5 exhibited the best stability characteristics of all plans considered during the 2-D stability tests.

<u>b</u>. For the 2-D transmission tests:

(1) Observed transmitted wave heights were all below the desired maximum.

(2) For a given plan and incident wave height, transmitted wave heights always increased with increasing wave period.

(3) Plan 2 transmitted the least wave energy of all plans considered.

<u>c</u>. For the 3-D stability tests:

(1) Plan 6 was stable for both the 9-sec, 4.1-m nonbreaking waves and the 9-sec, 4.7-m breaking waves.

(2) For the selected design conditions, the 5.0-metric-ton dolosse tested in Plan 6 will be of adequate size for use on all portions of the Jubail breakwater system.

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Table 1

Values of Incident and Transmitted Wave Heights

Plans 1, 2, 3, and 5; Deeper Water Location

Incident Wave Height,* H_i , m	Transmitted Wave Height, H_t , cm	Location of Transmitted Wave-Height Measurements**
	Plan 1, 7-sec Wave Period	
3.1	7.9	L
4.0	10.9	L
	Plan 1, 9-sec Wave Period	
3.1	16.7	L/2
4.0	23.0	L/2
	Plan 1, 11-sec Wave Period	
3.1	20.6	L/2
4.0	34.5	L/2
	Plan 2, 7-sec Wave Period	
3.1	6.3	L/2
4.0	6.7	L/2
	Plan 2, 9-sec Wave Period	
3.1	10.6	L/2
4.0	13.0	L/2
	Plan 2, 11-sec Wave Period	
3.1	16.0	L/2
4.0	25.4	L/2
	Plan 3, 7-sec Wave Period	
3.1	6.8	L/2
4.0	7.0	L/2
	Plan 3, 9-sec Wave Period	
3.1	11.6	L/2
4.0	18.4	L/2
	Plan 3, 11-sec Wave Period	
3.1	19.4	L/2
4.0	35.7	L/2
	Plan 5, 7-sec Wave Period	
3.1	21.1	L
	Plan 5, 9-sec Wave Period	
3.1	31.8	L/2

* Measured at toe of structure without structure in place.

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

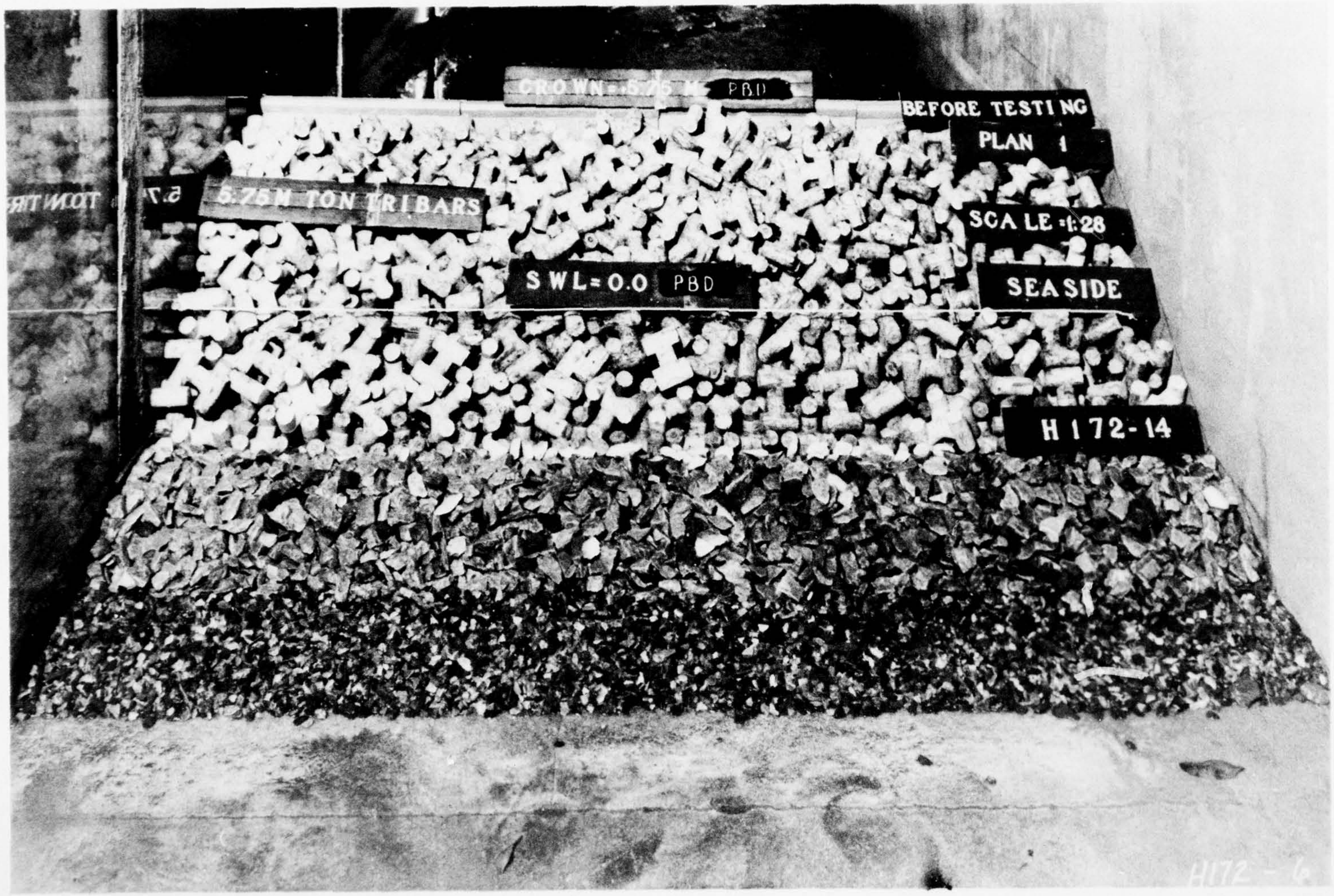


Photo 1. Seaside view of Plan 1

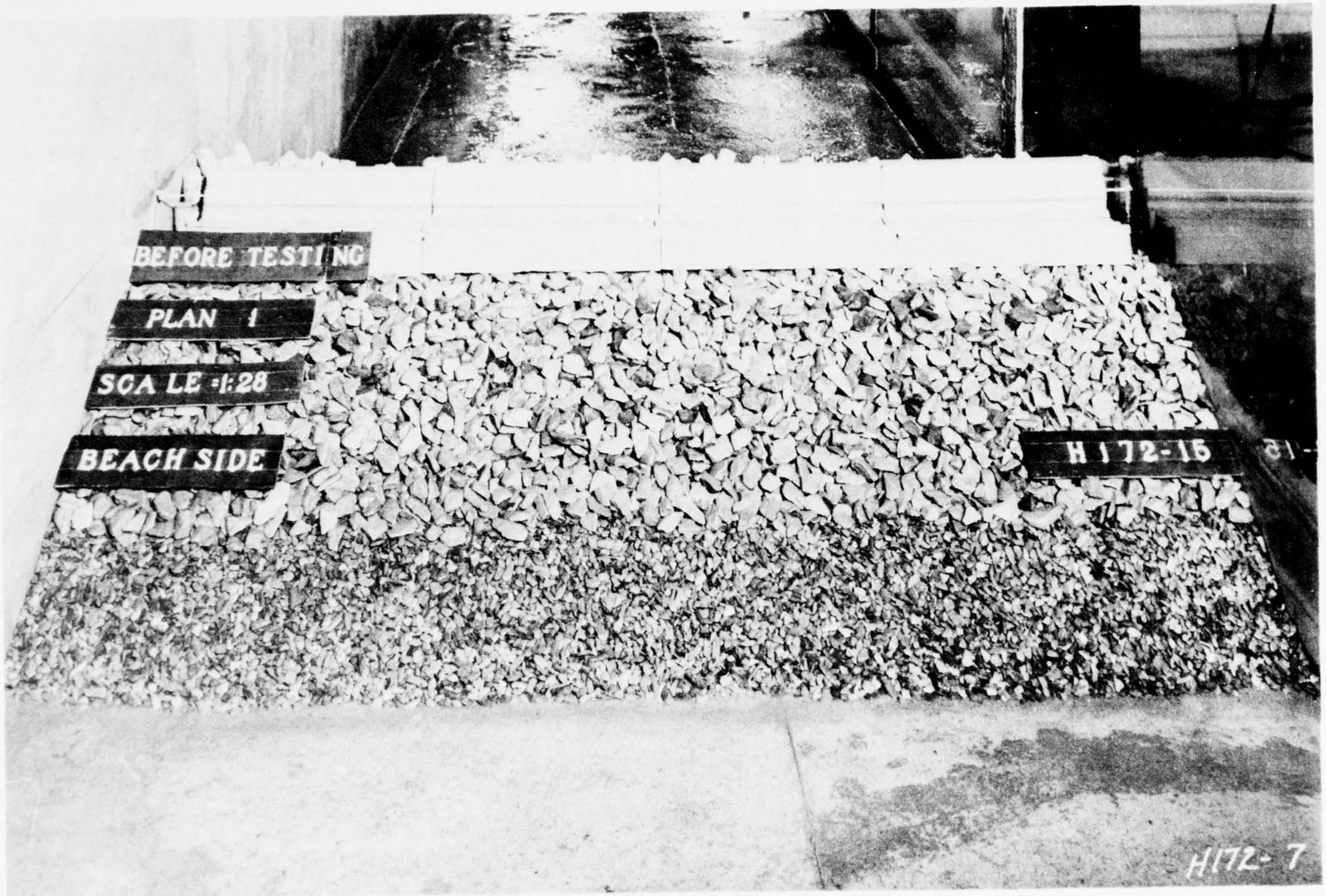


Photo 2. Beachside view of Plan 1

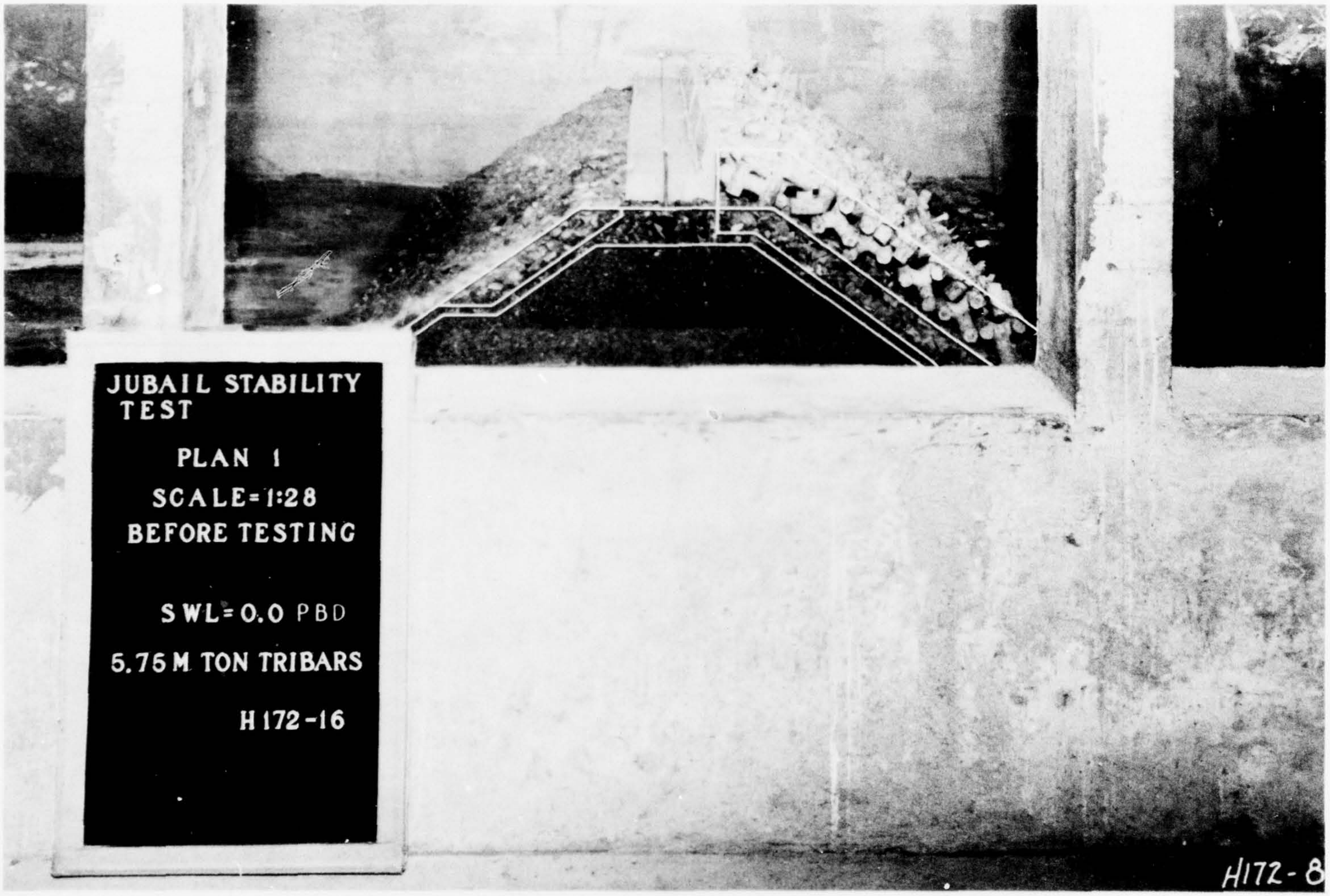


Photo 3. End view of Plan 1

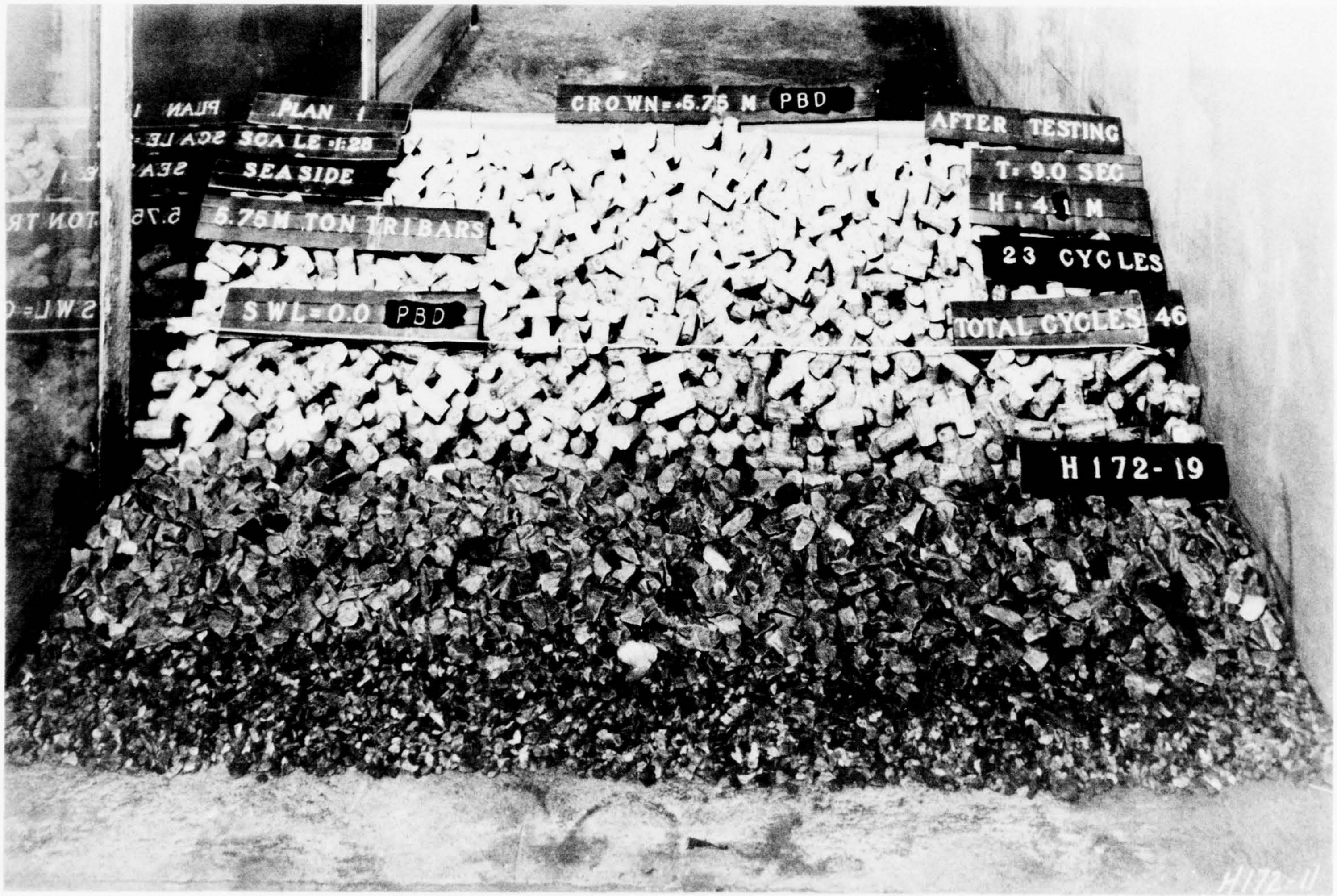


Photo 4. Seaside view of Plan 1 in the shallow-water location after attack of 9-sec, 4.1-m waves at a swl of 0.0-m PBD

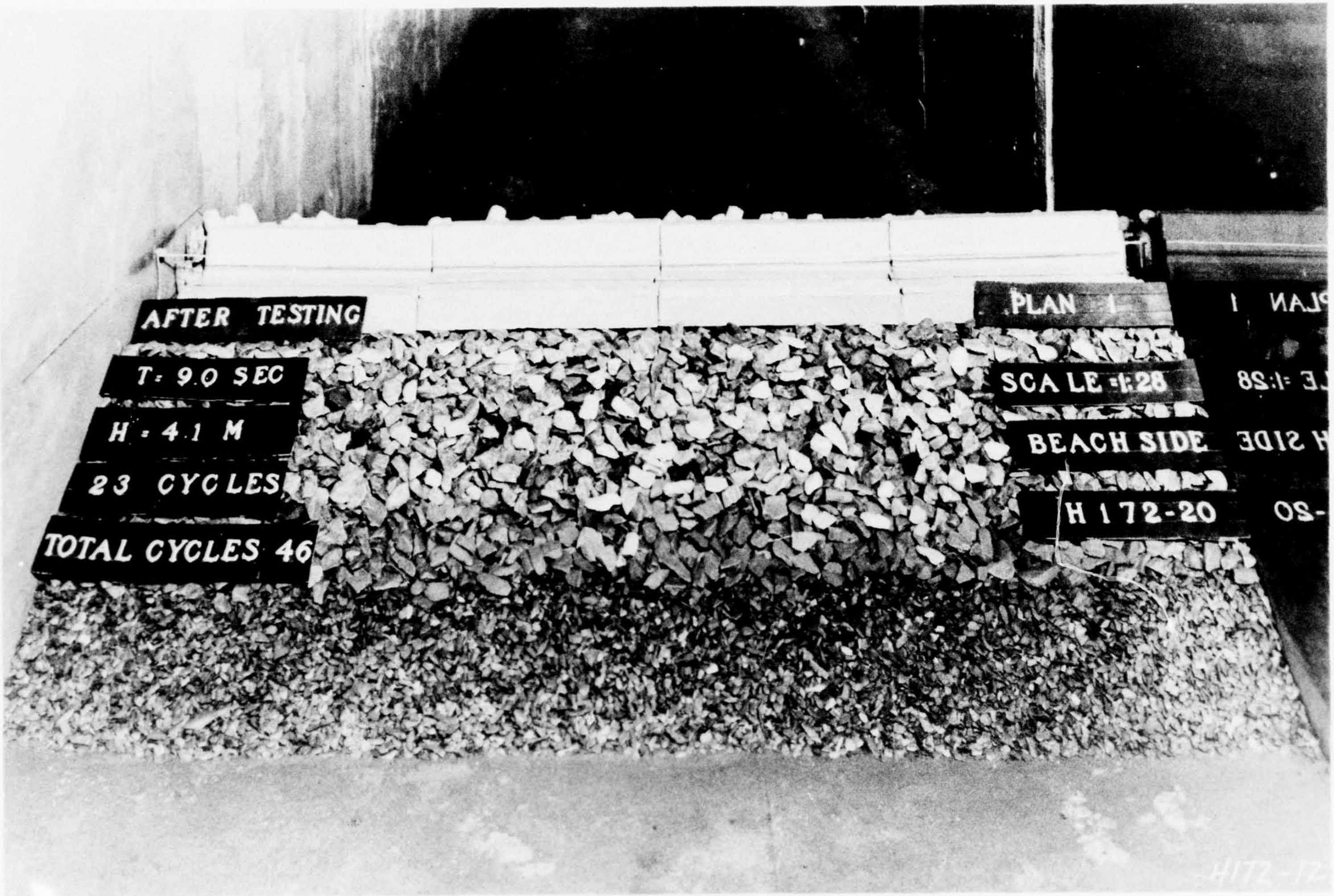


Photo 5. Beachside view of Plan 1 in the shallow-water location after attack of 9-sec, 4.1-m waves at a swl of 0.0-m PBD

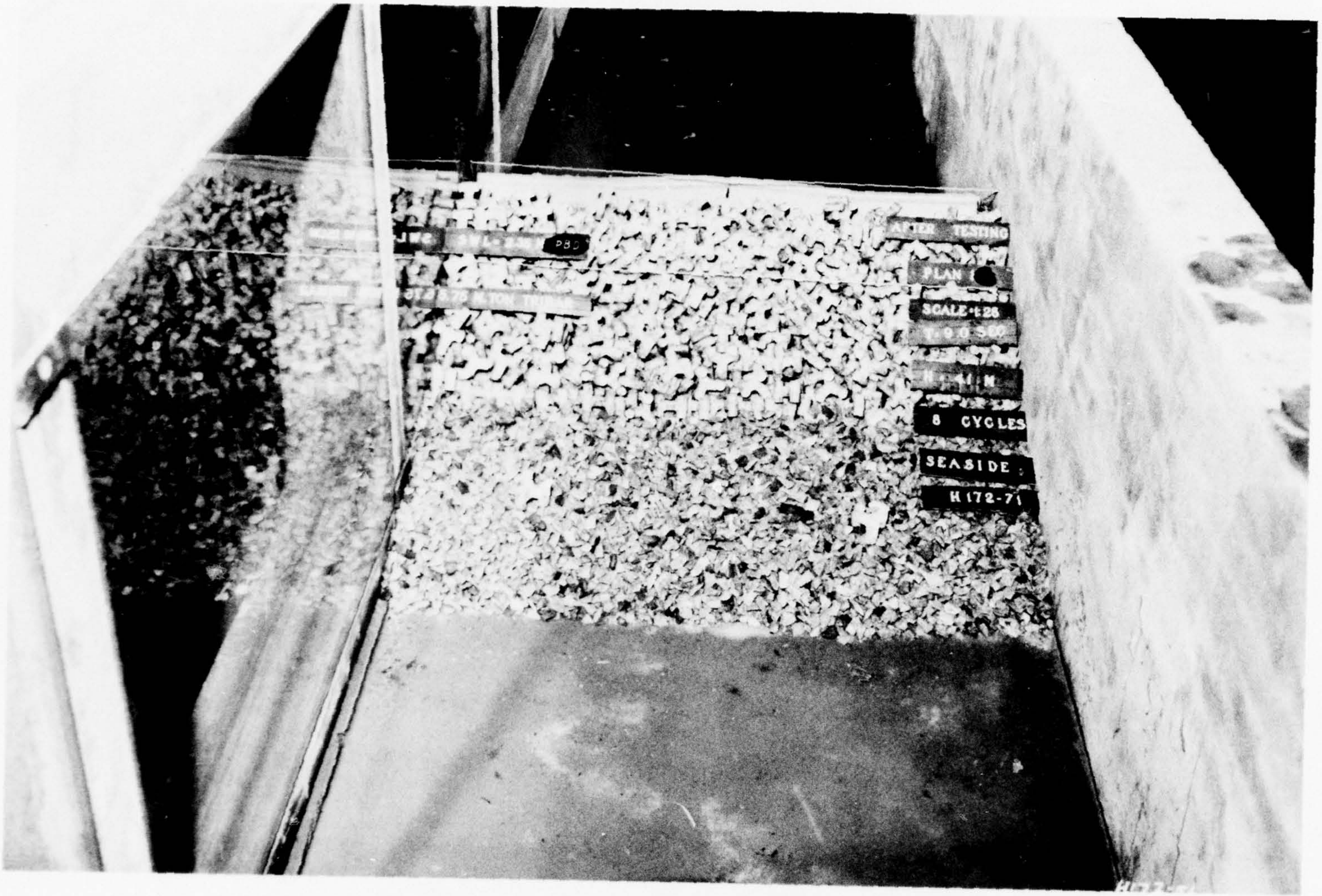


Photo 6. Seaside view of Plan 1 in the deeper water location after attack of 9-sec, 4.1-m waves at a swl of +2.35-m PBD

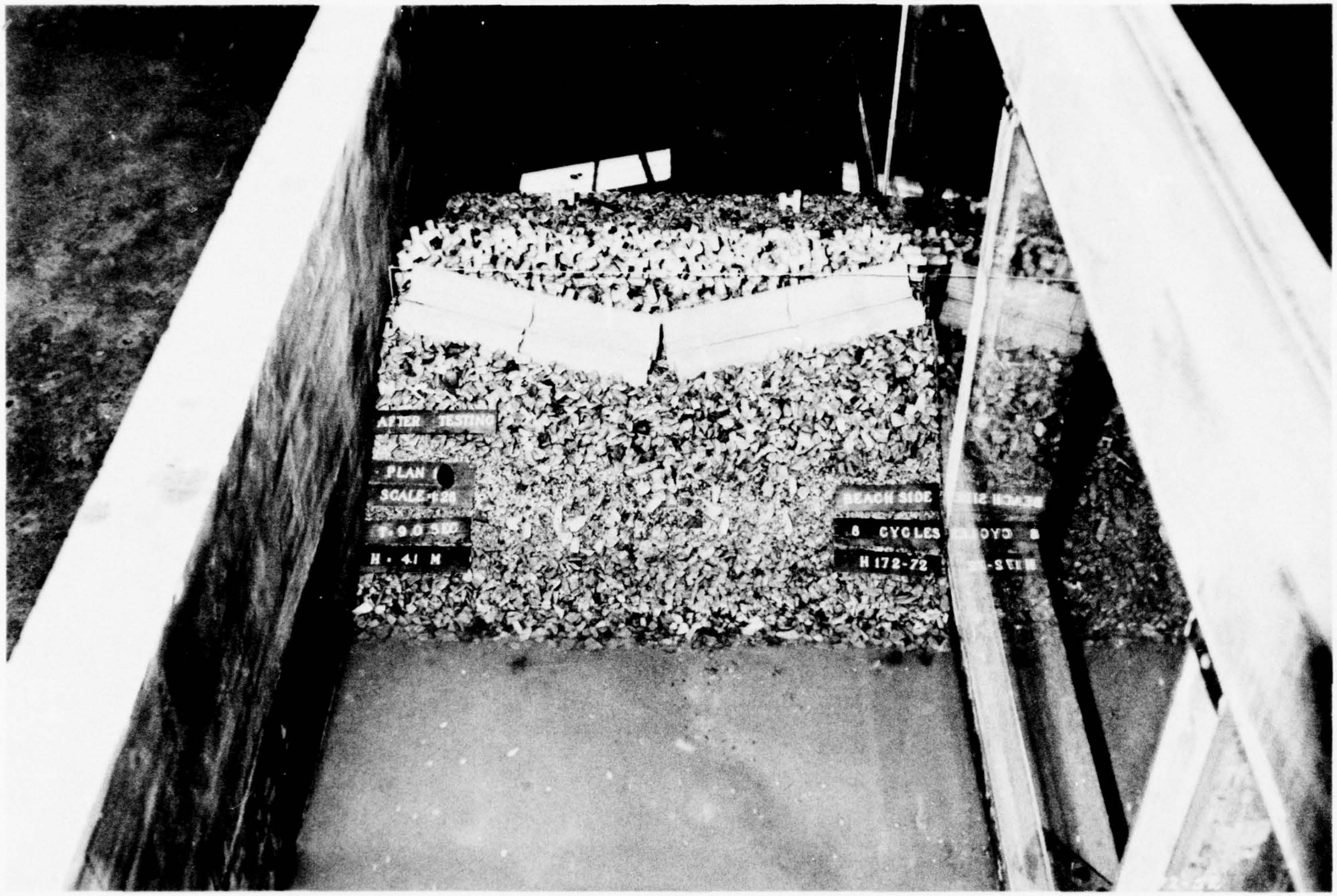


Photo 7. Beachside view of Plan 1 in the deeper water location after attack of 9-sec, 4.1-m waves at a swl of +2.35-m PBD

Photo 8. Seaside view of Plan 2

Photo 9. Beachside view of Plan 2

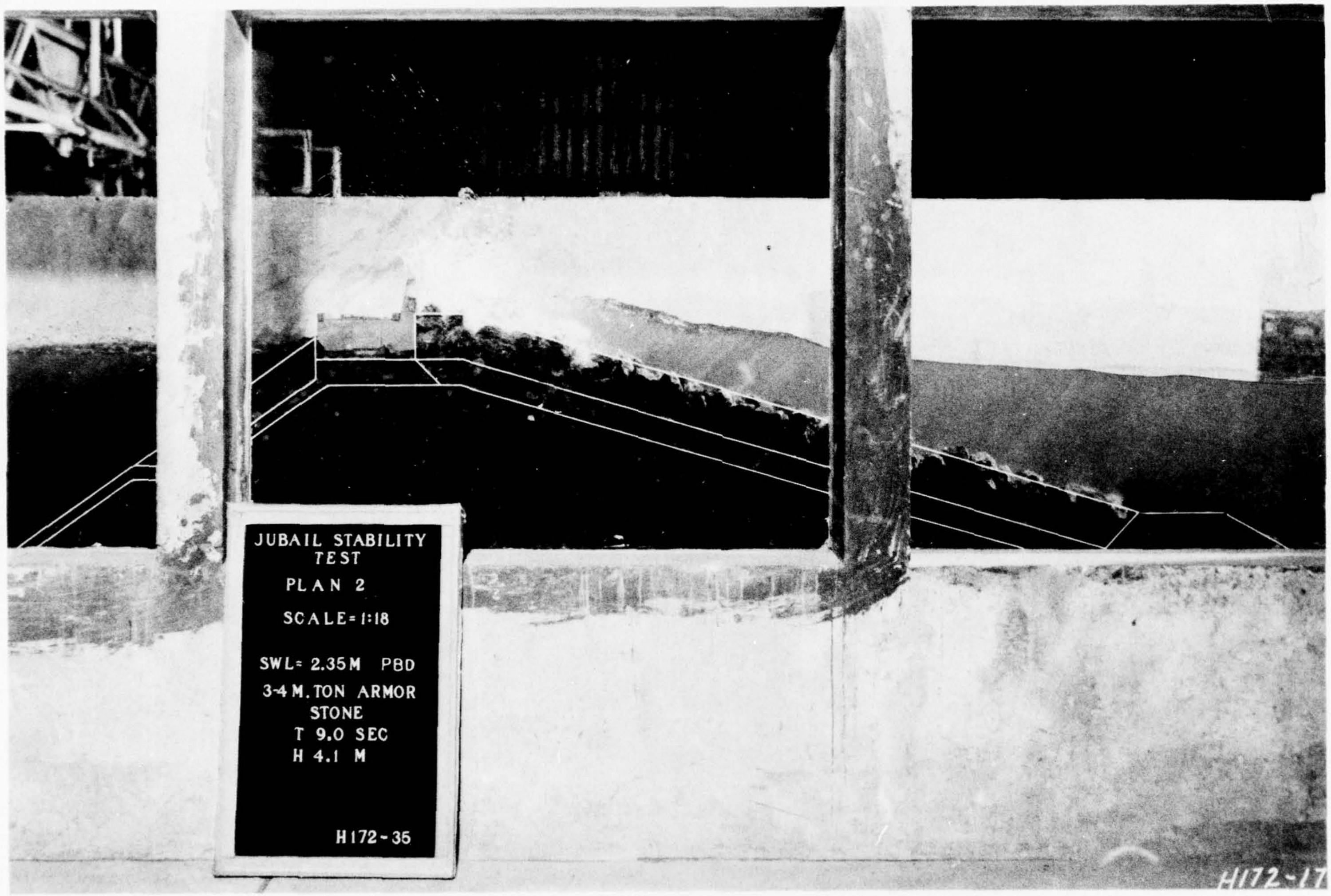


Photo 10. End view of Plan 2 under attack by 9-sec, 4.1-m waves at a swl of +2.35-m PBD

Photo 11. Seaside view of Plan 2 after attack of 9-sec, 4.9-m waves at a swl of +2.35-m PBD

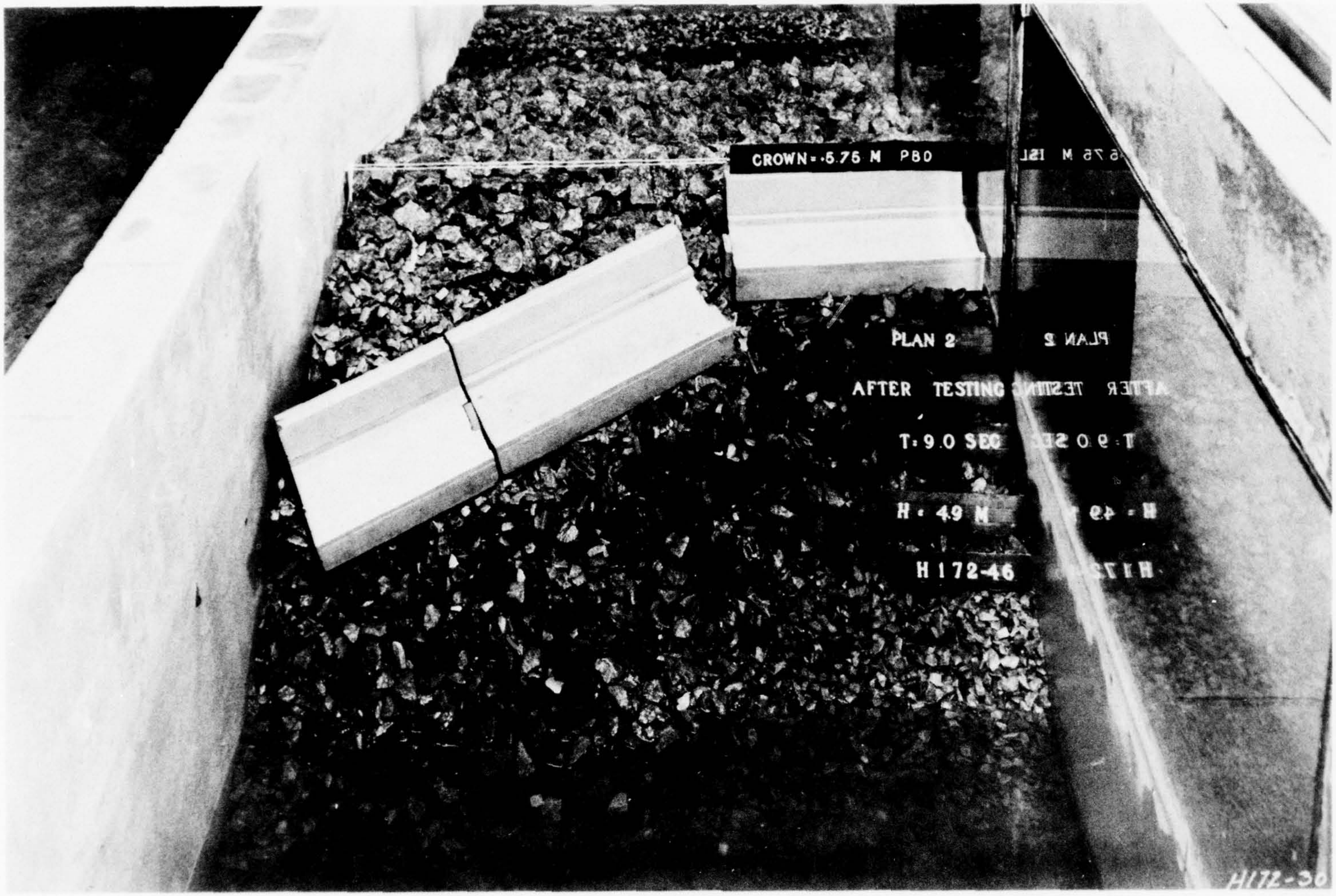


Photo 12. Beachside view of Plan 2 after attack of 9-sec, 4.9-m waves at a swl of +2.35-m PBD

Photo 13. Seaside view of Plan 2A after attack of 9-sec, 4.9-m waves at a swl of +2.35-m PBD

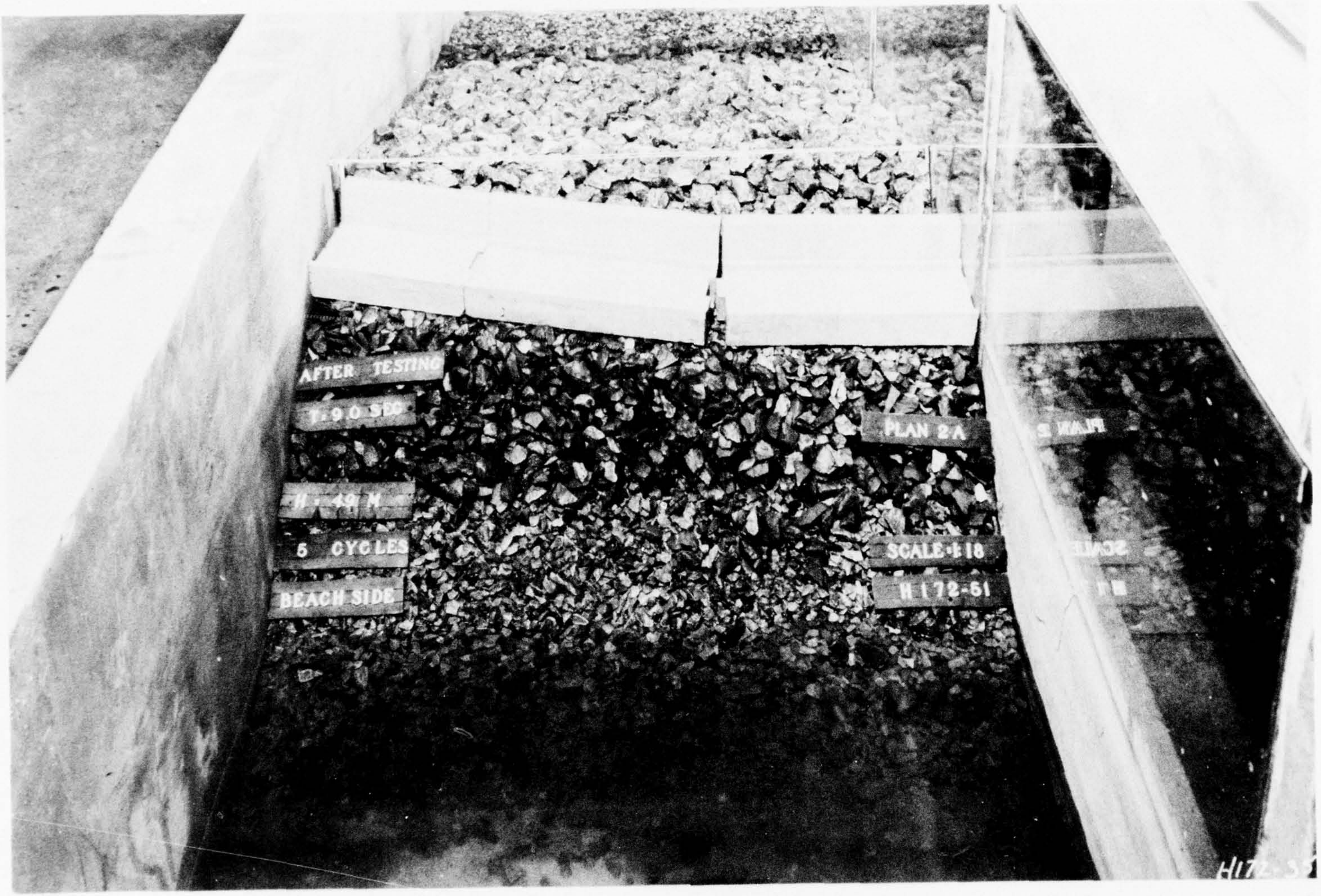


Photo 14. Beachside view of Plan 2A after attack of 9-sec, 4.9-m waves at a swl of +2.35-m PBD

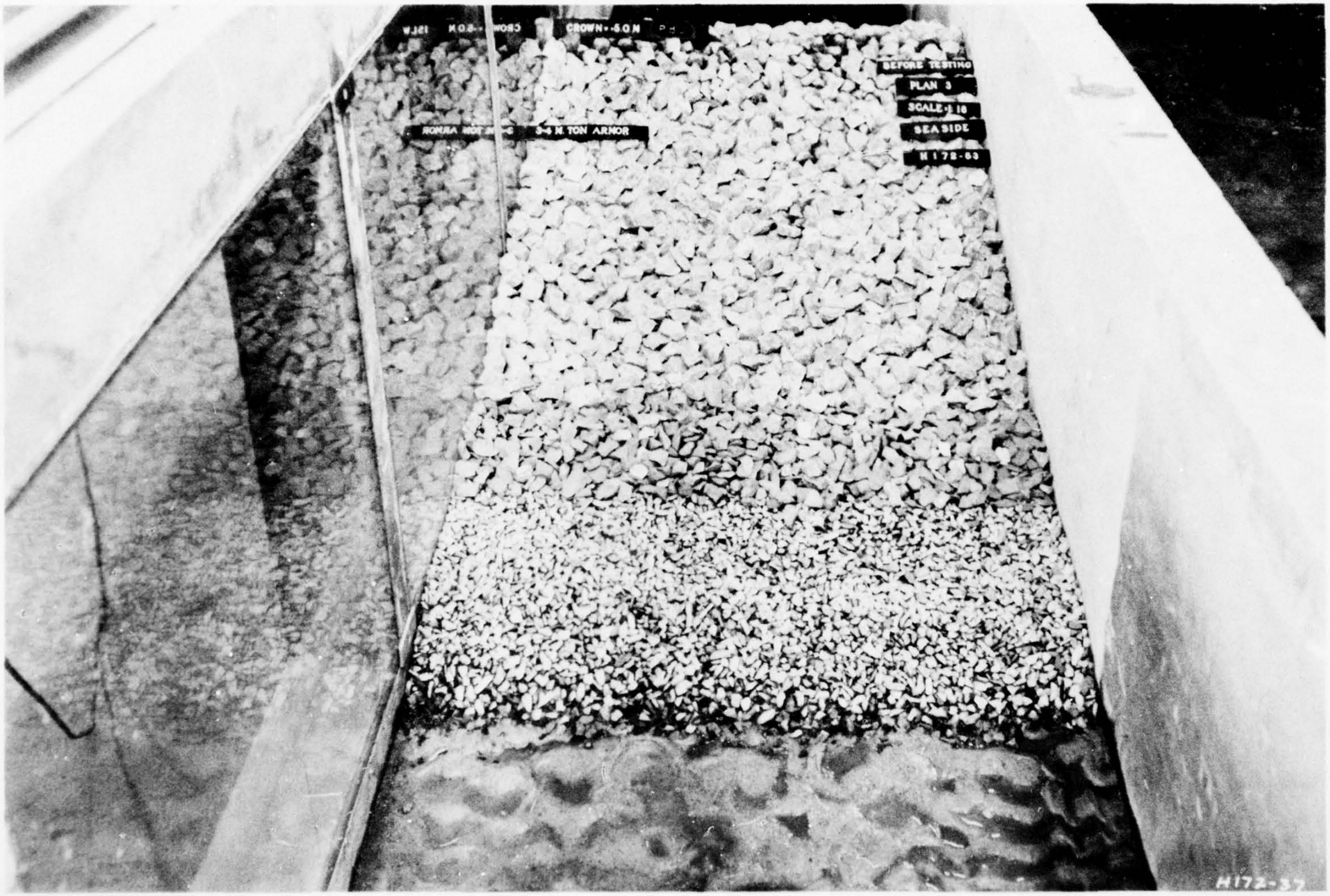


Photo 15. Seaside view of Plan 3

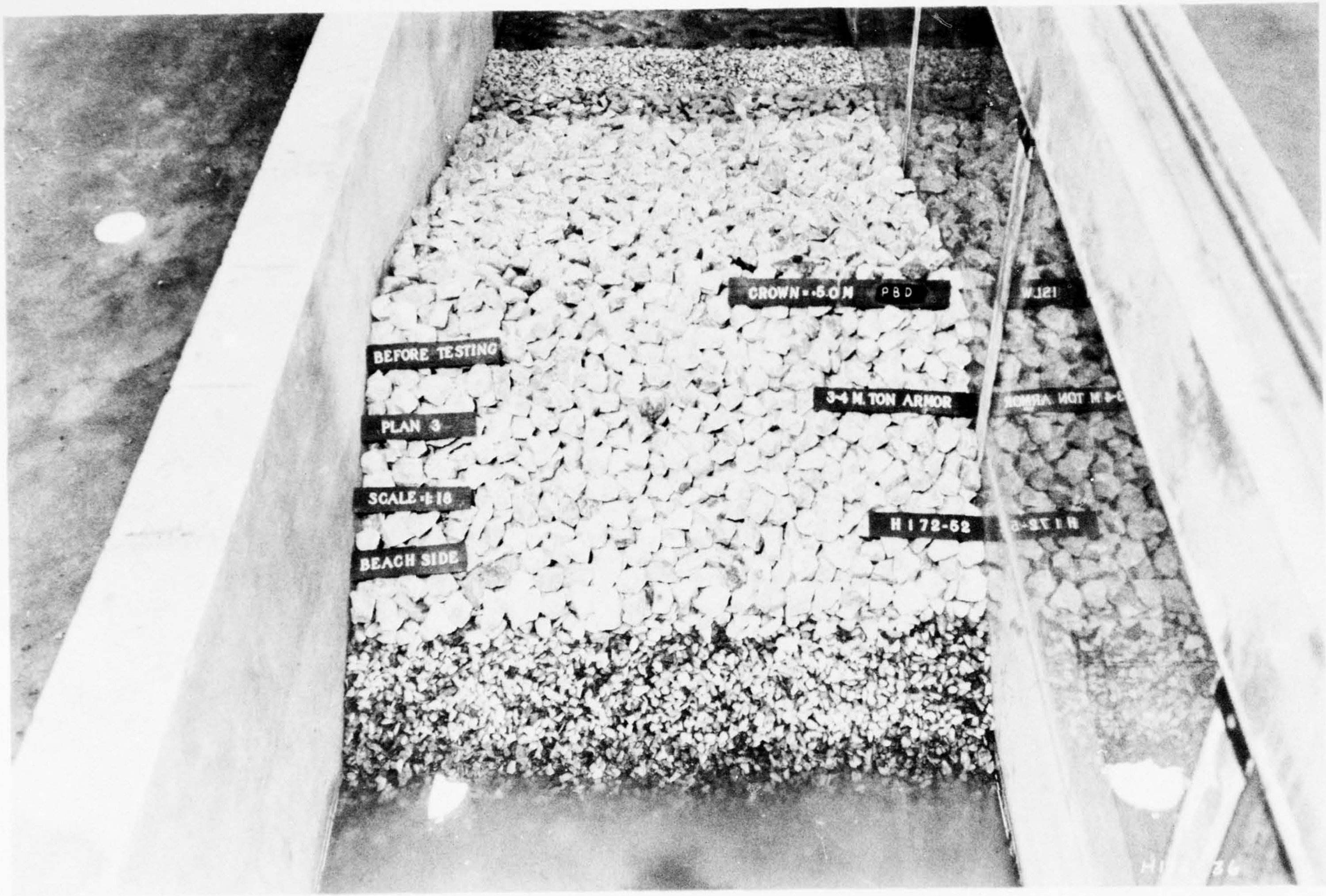


Photo 16. Beachside view of Plan 3

Photo 17. End view of Plan 3

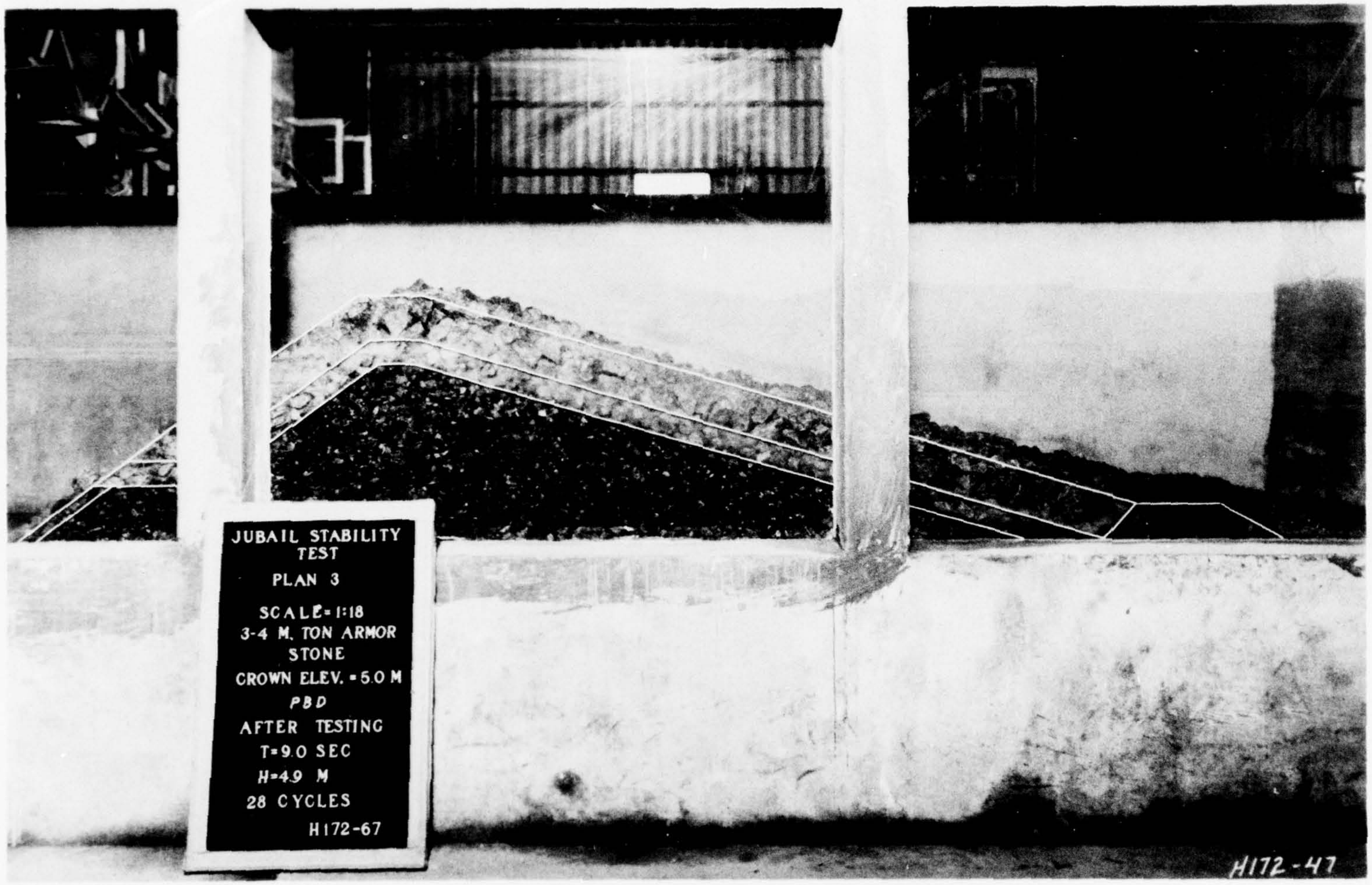


Photo 18. End view of Plan 3 after attack of 9-sec, 4.9-m waves at a swl of +2.35-m PBD

Photo 19. Beachside view of Plan 3 after attack of 9-sec, 4.9-m waves at a swl of +2.35-m PBD

Photo 20. Seaside view of Plan 4

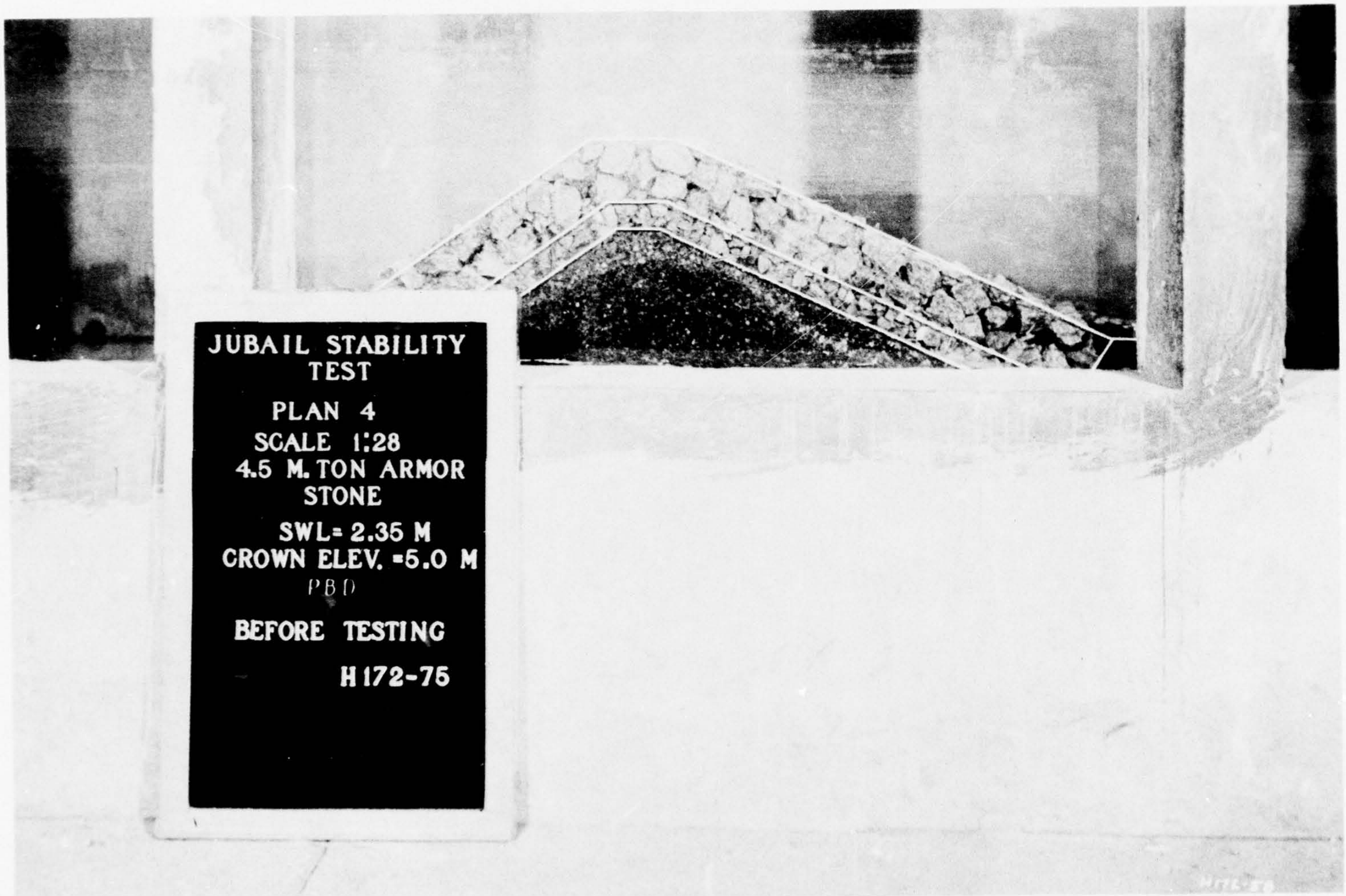


Photo 21. End view of Plan 4

Photo 22. Beachside view of Plan 4

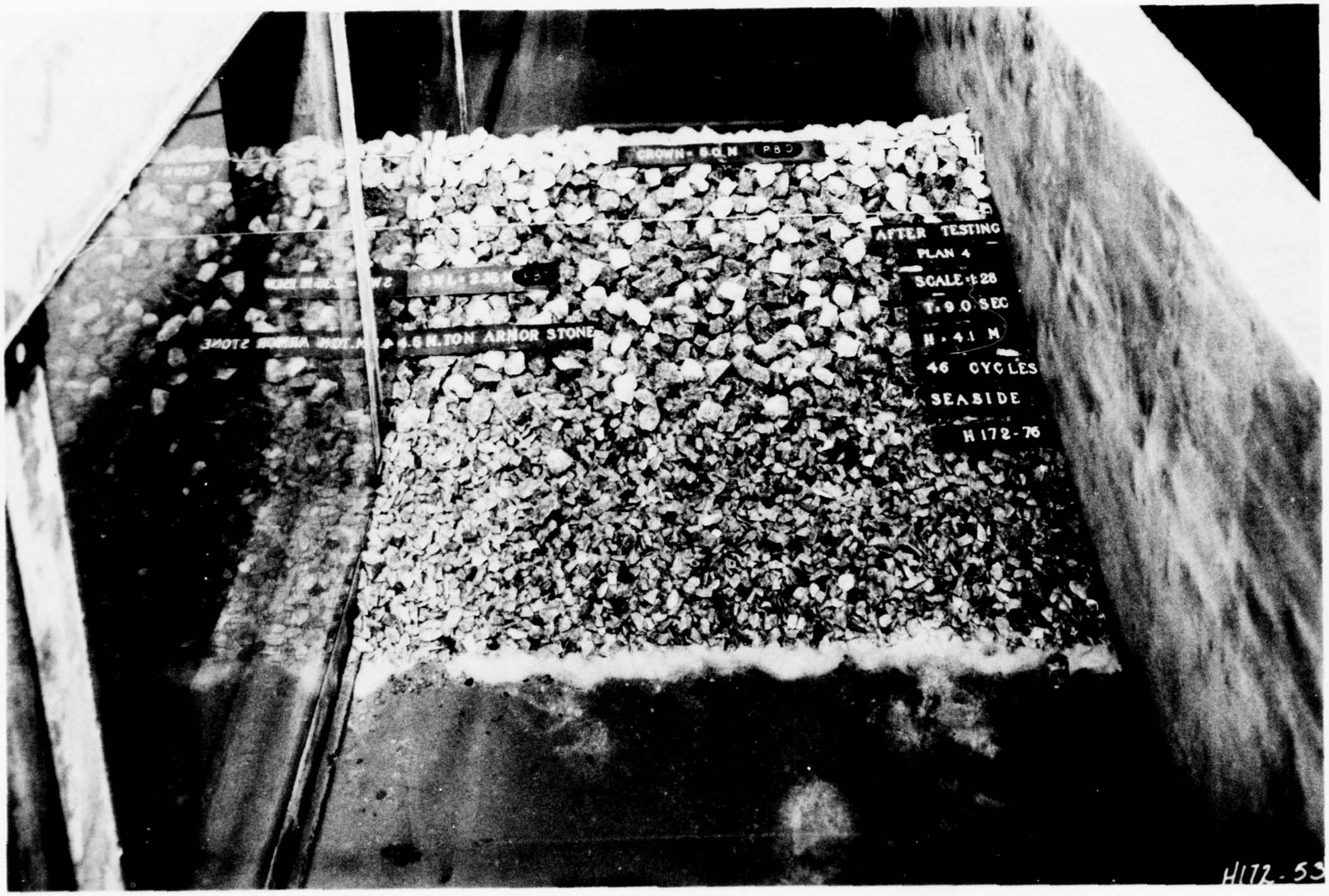


Photo 23. Seaside view of Plan 4 after attack of 9-sec, 4.1-m waves at a swl of +2.35-m PBD

Photo 24. Beachside view of Plan 4 after attack of 9-sec, 4.1-m waves at a swl of +2.35-m PBD

Photo 25. Seaside view of Plan 5

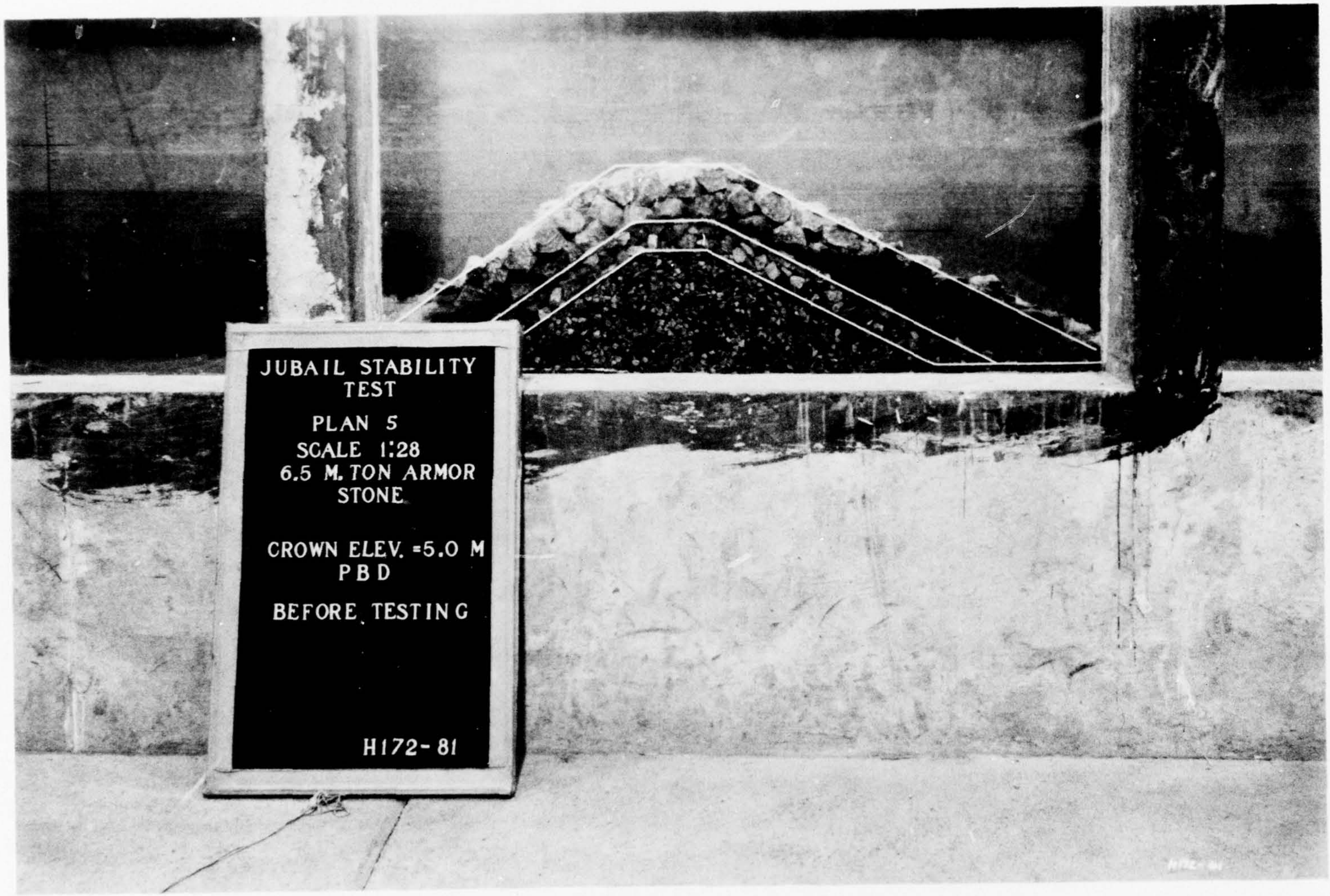


Photo 26. End view of Plan 5

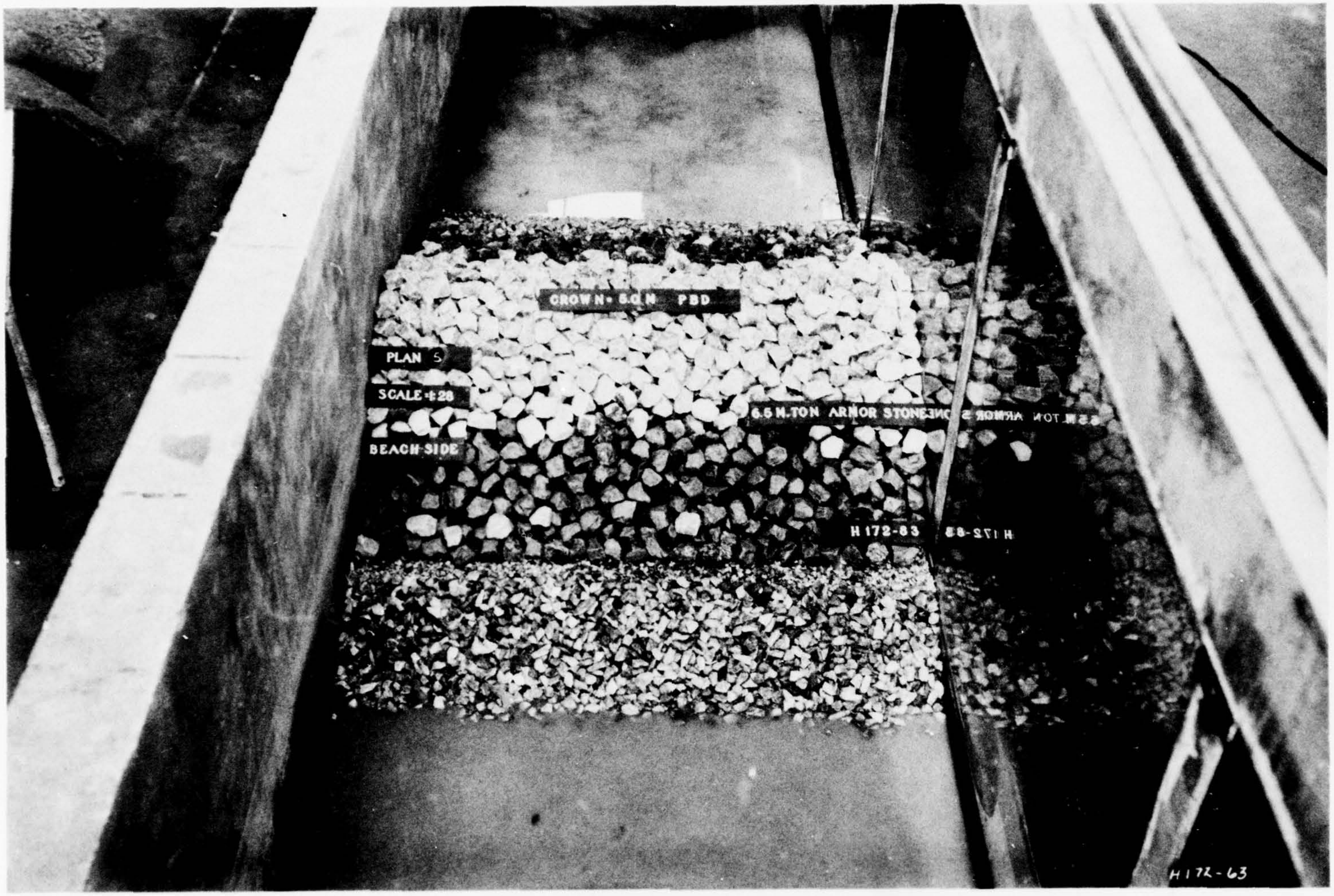


Photo 27. Beachside view of Plan 5

Photo 28. Seaside view of Plan 5 in the shallow-water location after attack of 9-sec, 4.1-m waves at a swl of +2.35-m PBD

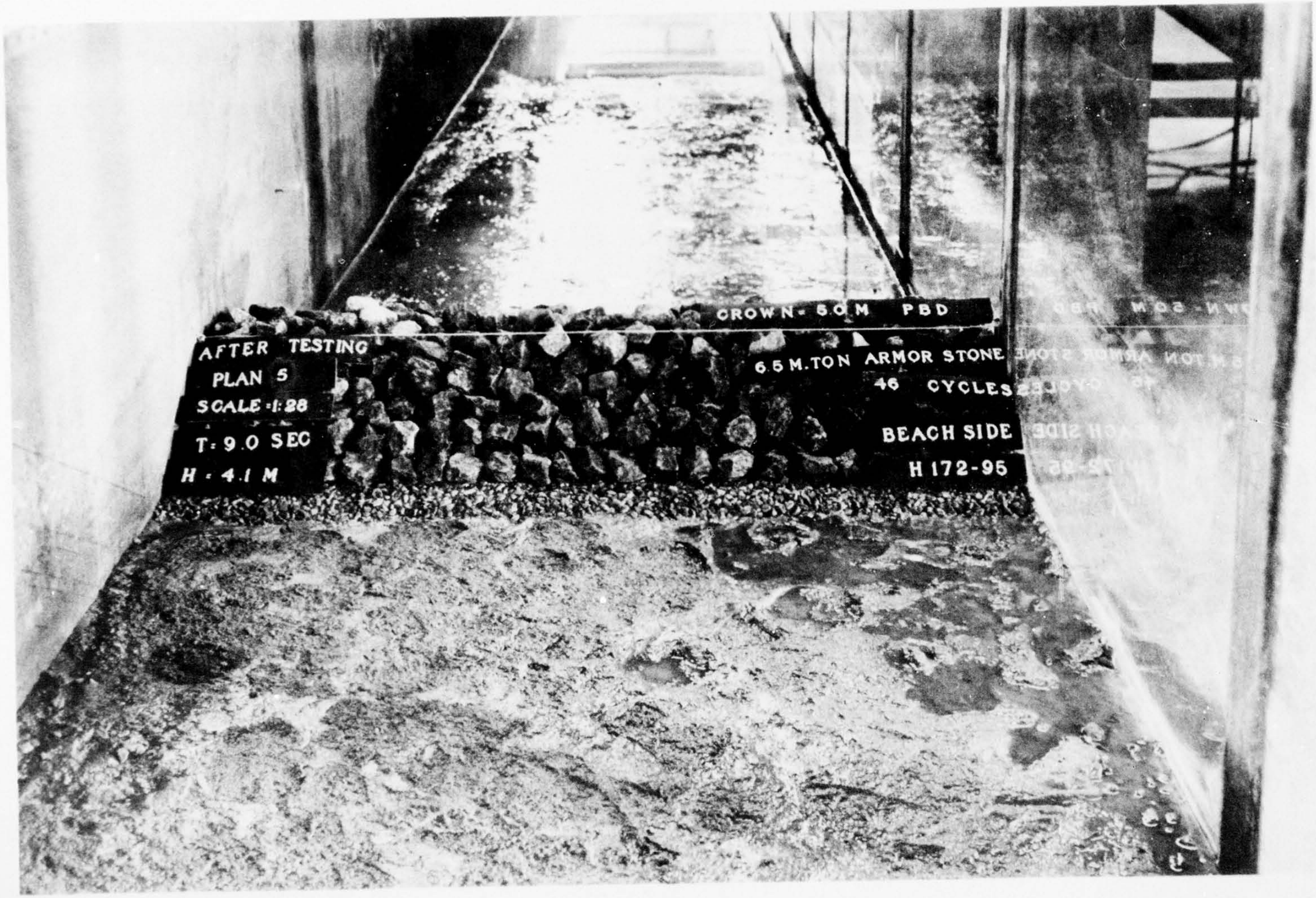


Photo 29. Beachside view of Plan 5 in the shallow-water location after attack of 9-sec, 4.1-m waves at a swl of +2.35-m PBD

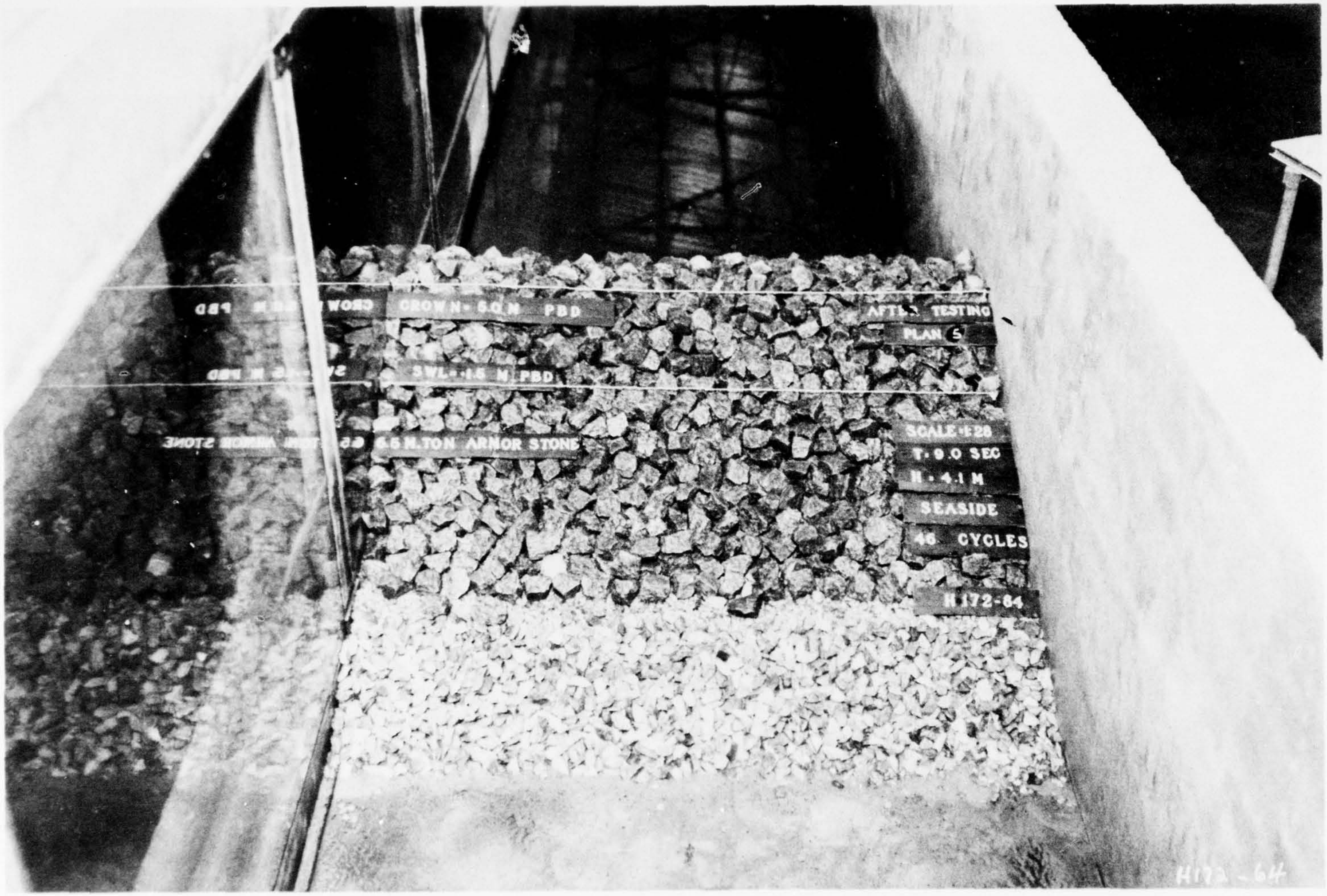


Photo 30. Seaside view of Plan 5 in the deeper water location after attack of 9-sec, 4.1-m waves at a swl of +1.50-m PBD

Photo 31. Beachside view of Plan 5 in the deeper water location after attack of 9-sec, 4.1-m waves at a swl of +1.50-m PBD

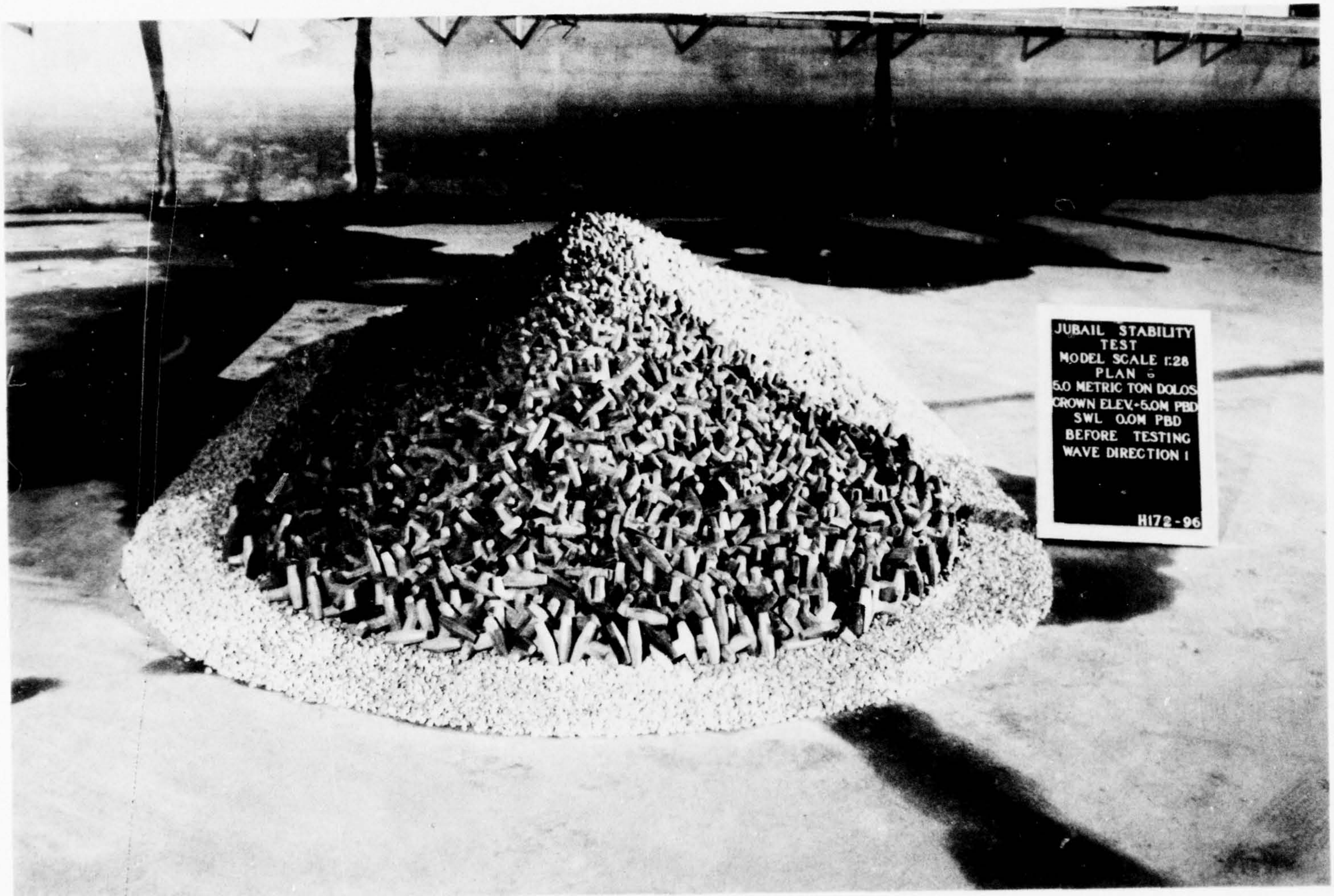


Photo 32. End view of Plan 6

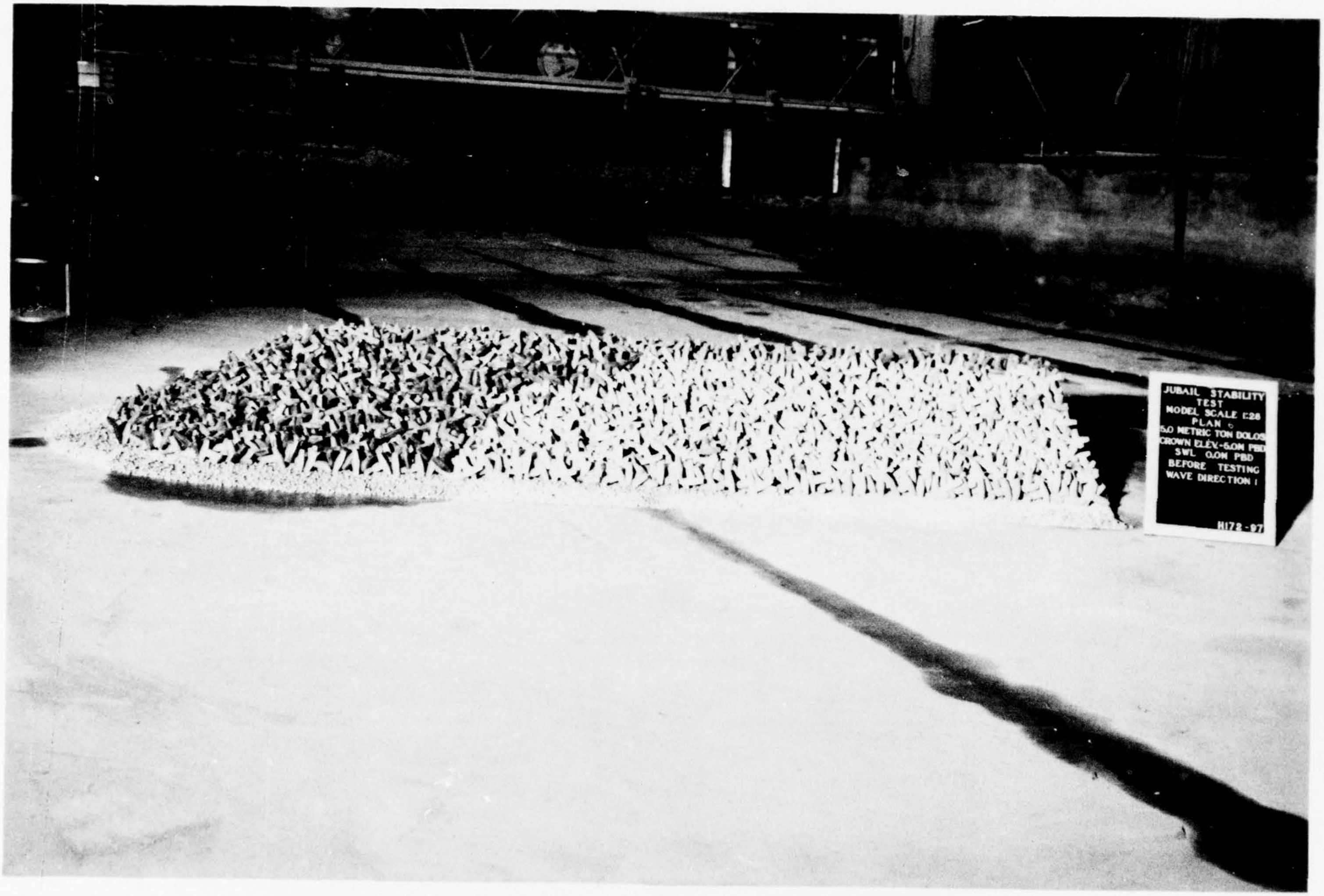


Photo 33. Seaside view of Plan 6

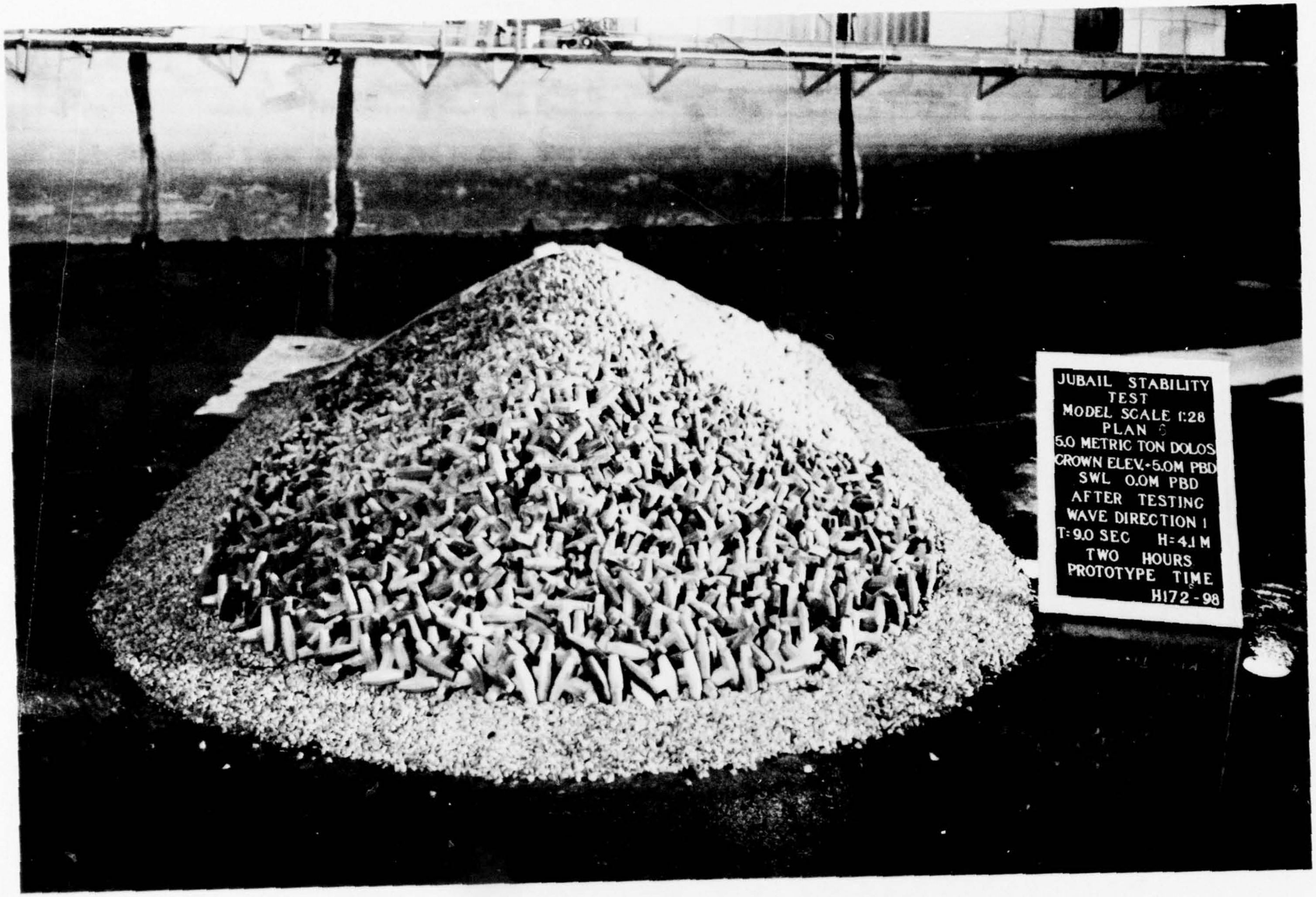


Photo 34. End view of Plan 6 after attack of 9-sec, 4.1-m waves from wave direction 1

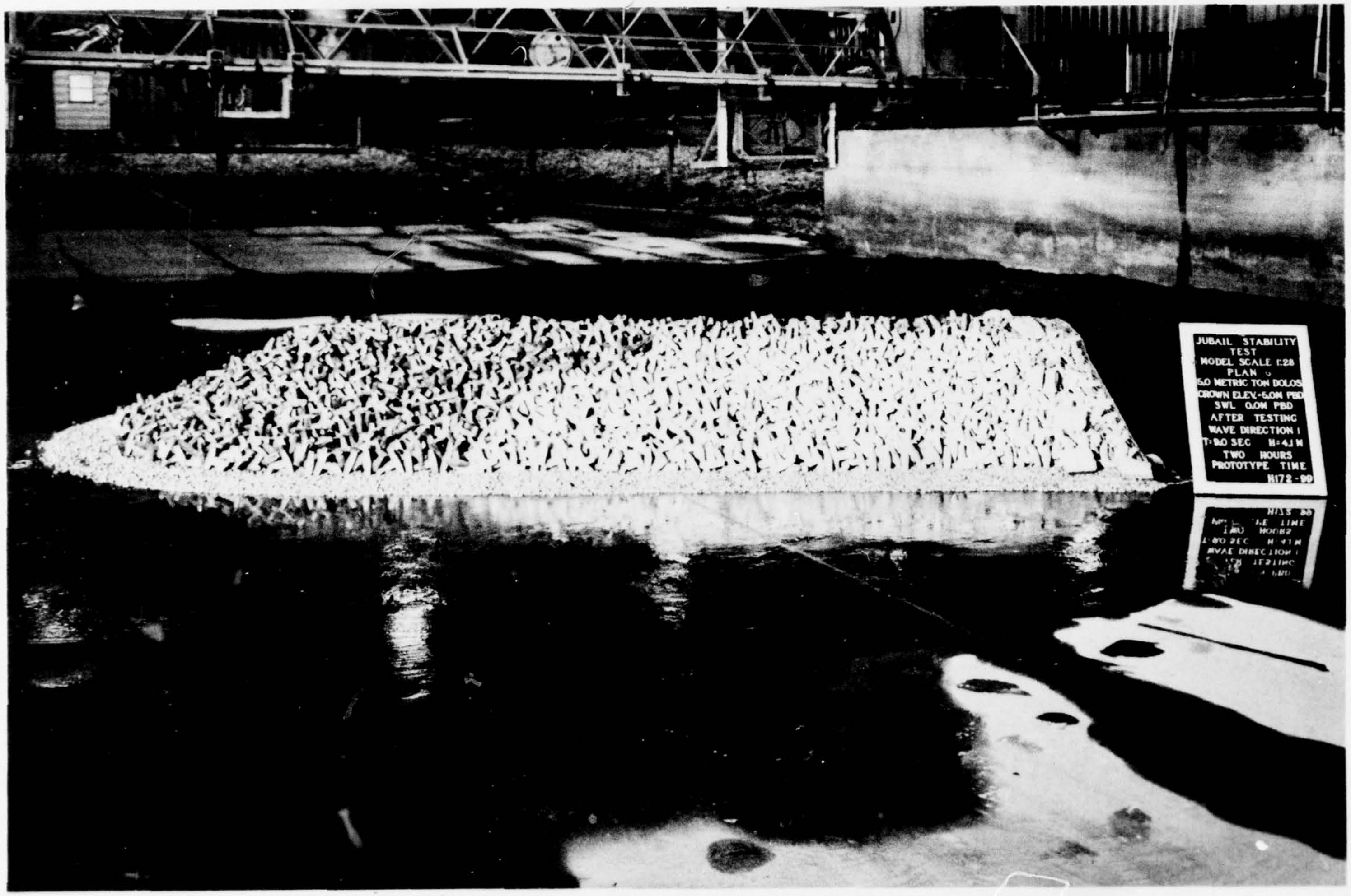


Photo 35. Seaside view of Plan 6 after attack of 9-sec, 4.1-m waves from wave direction 1

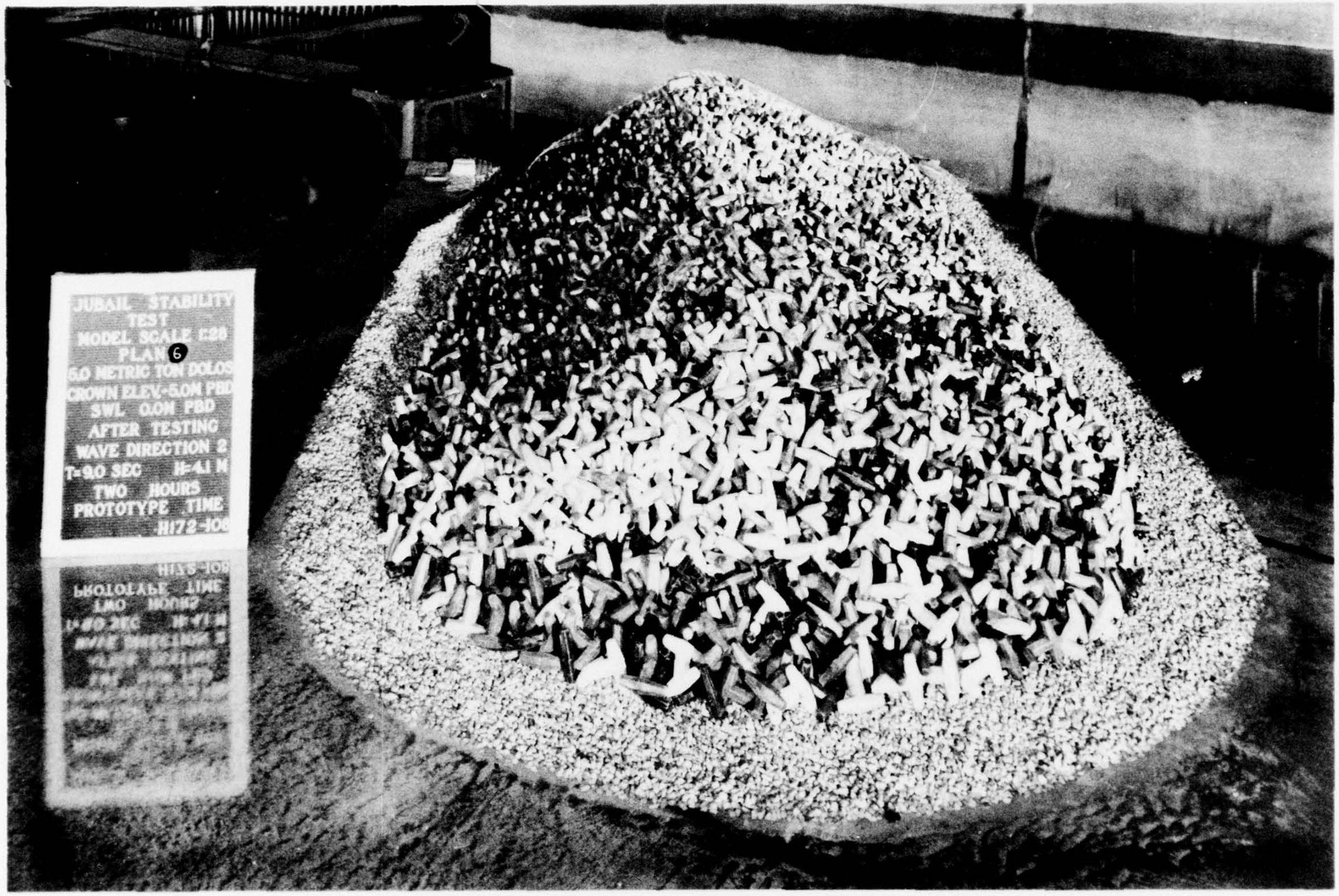


Photo 36. End view of Plan 6 after attack of 9-sec, 4.1-m waves from wave direction 2

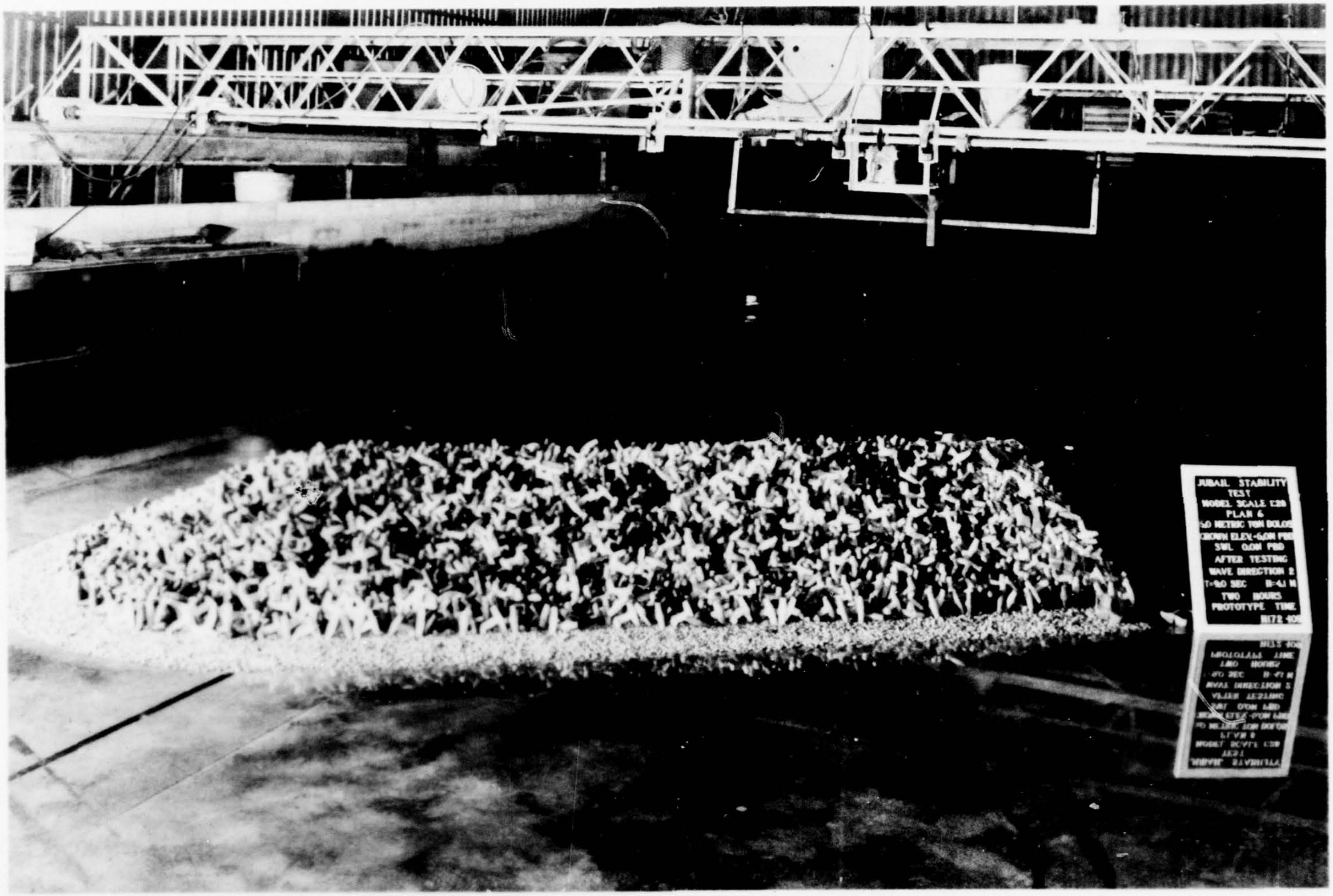


Photo 37. Seaside view of Plan 6 after attack of 9-sec, 4.1-m waves from wave direction 2

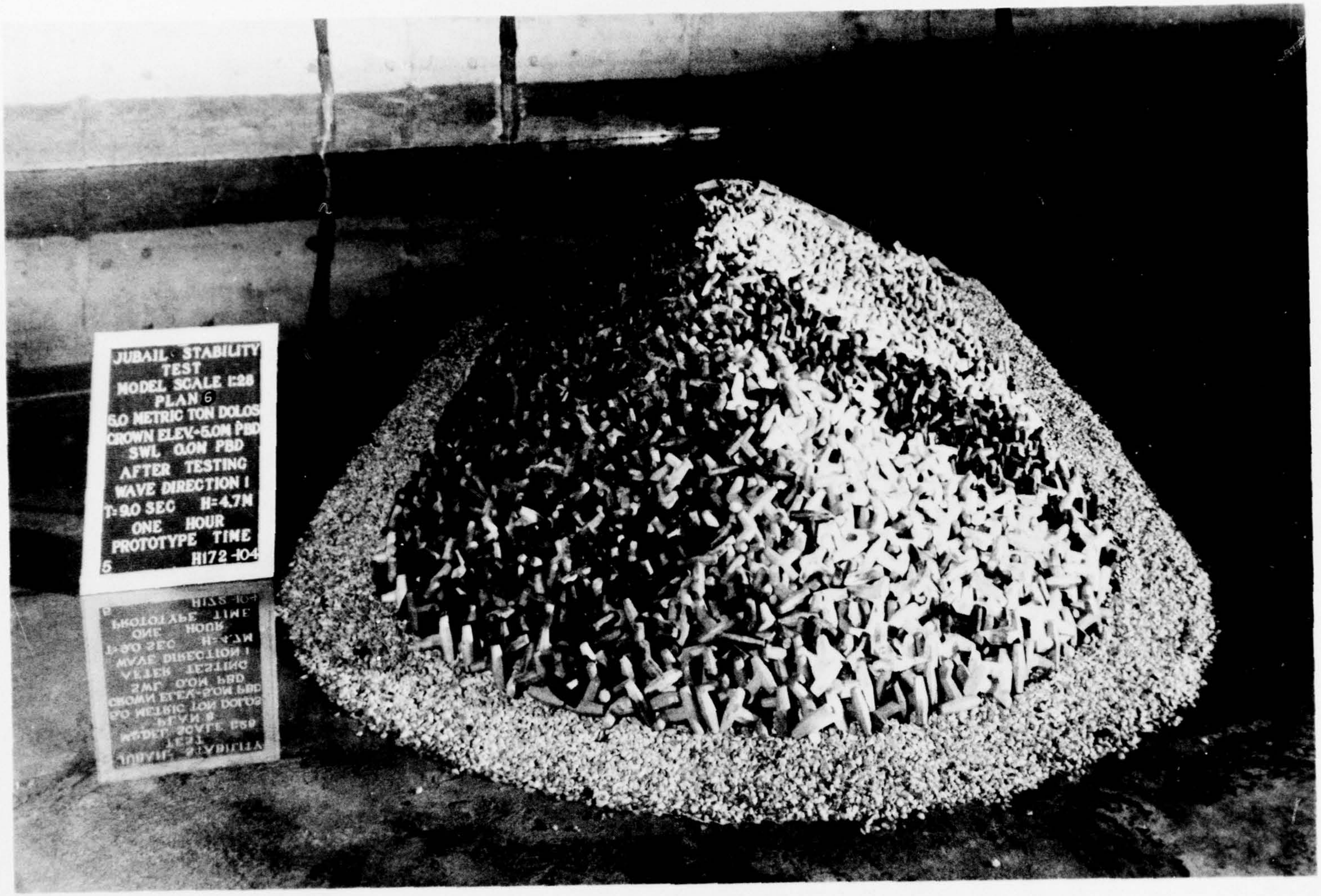


Photo 38. End view of Plan 6 after attack of 9-sec, 4.7-m waves from wave direction 1

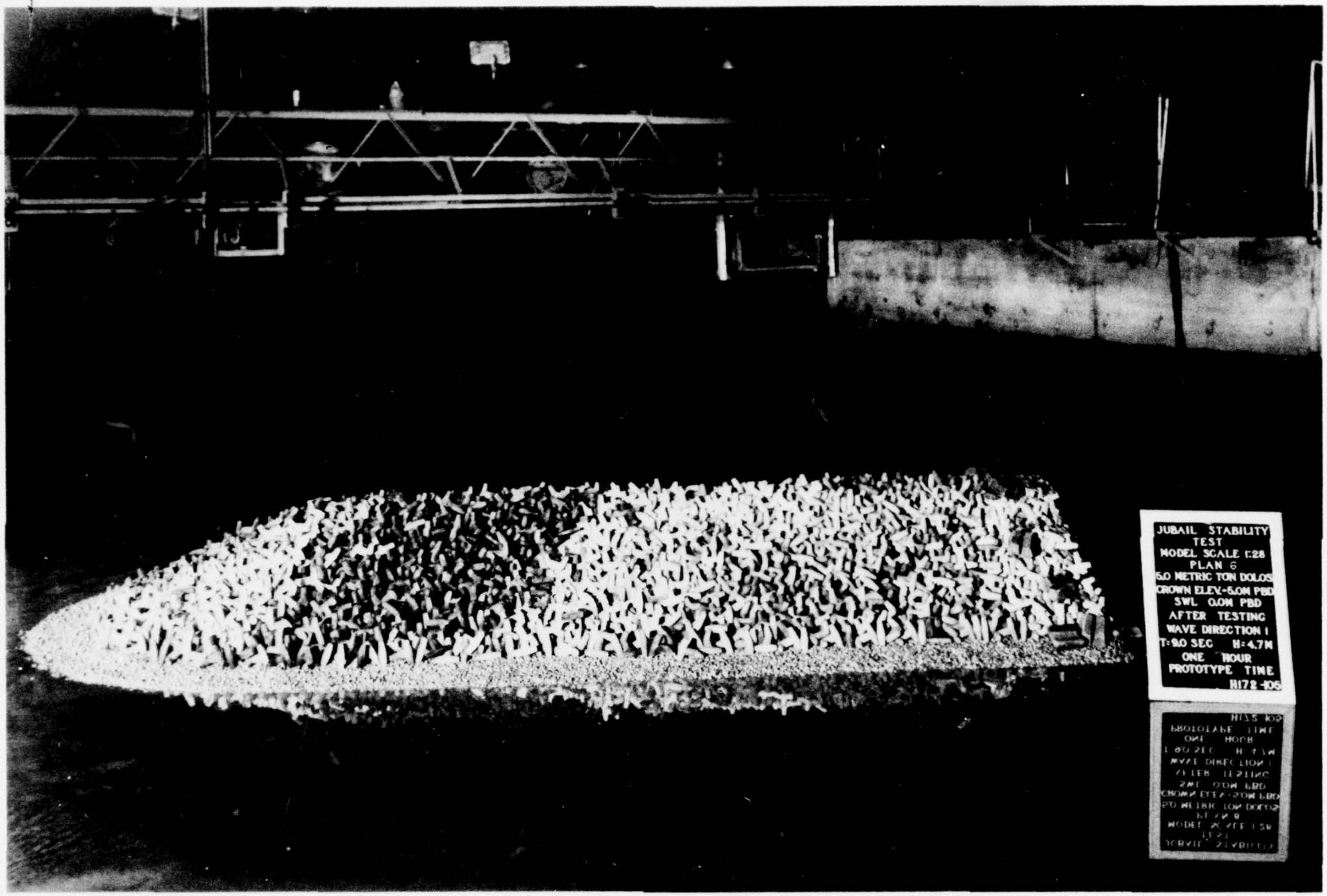


Photo 39. Seaside view of Plan 6 after attack of 9-sec, 4.7-m waves from wave direction 1

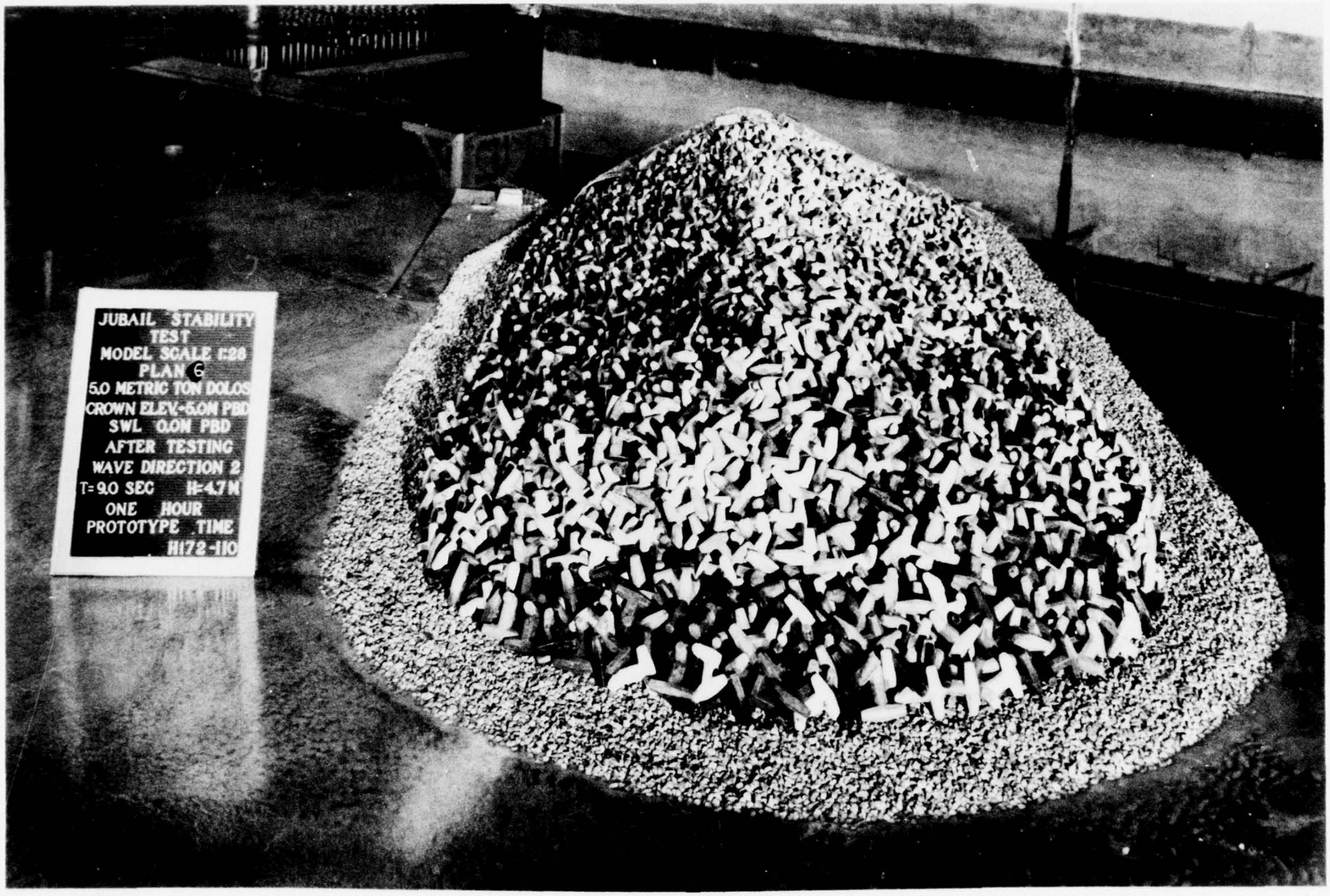


Photo 40. End view of Plan 6 after attack of 9-sec, 4.7-m waves from wave direction 2

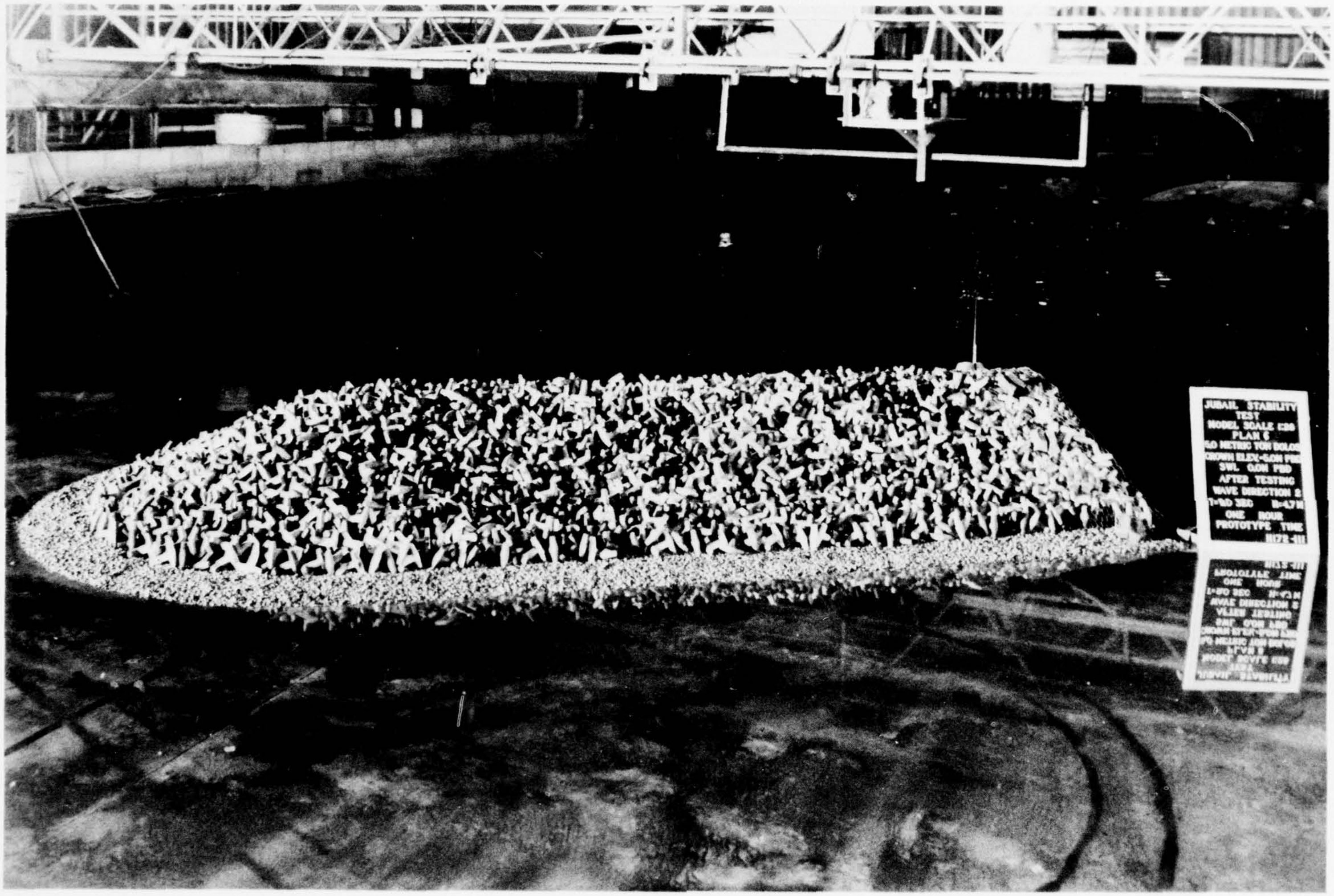
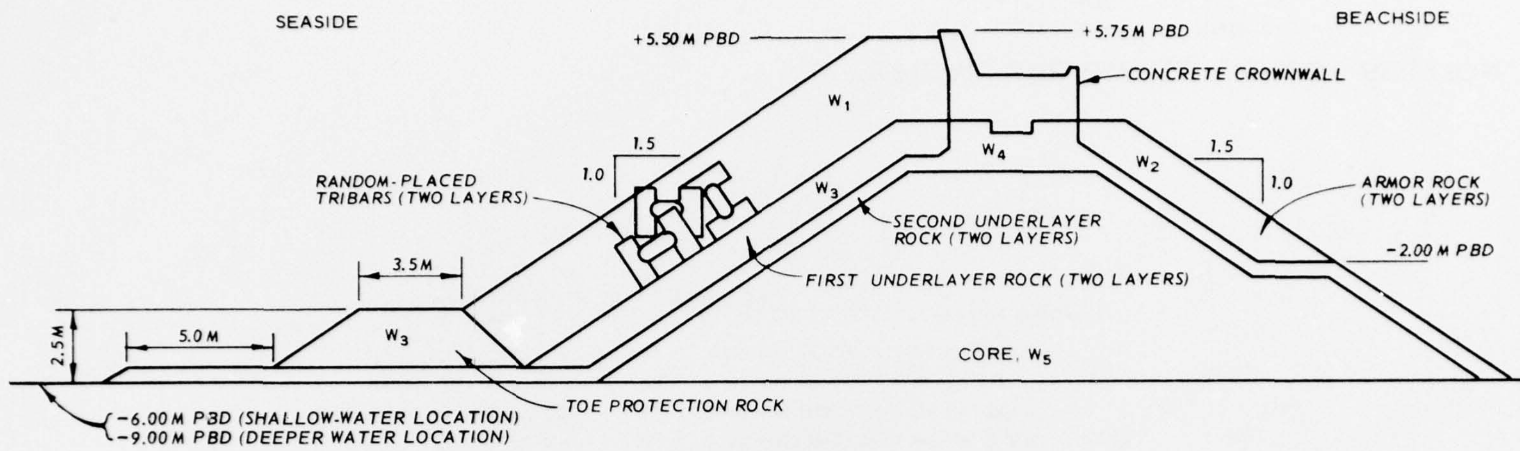


Photo 41. Seaside view of Plan 6 after attack of 9-sec, 4.7-m waves from wave direction 2



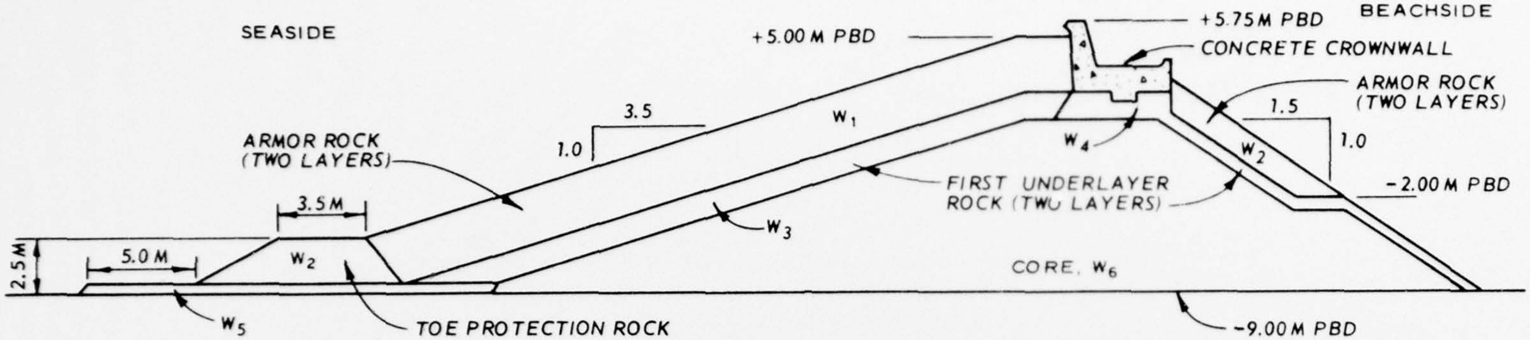
MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
W_1 = 286.0-G TRIBARS	W_1 = 5.75-METRIC-TON TRIBARS @ 2.40 G/CM^3
W_2 = 33.6-G ROCK	W_2 = 0.95-METRIC-TON ROCK @ 2.60 G/CM^3
W_3 = 17.7-G ROCK	W_3 = 0.50-METRIC-TON ROCK @ 2.60 G/CM^3
W_4 = 1.3-G ROCK	W_4 = 35.00-KG ROCK @ 2.60 G/CM^3
W_5 = 1.4-G ROCK*	W_5 = 1.50-KG ROCK @ 2.60 G/CM^3

* GEOMETRICALLY OVERSIZED ROCK USED TO OBTAIN PROPER TRANSMISSION OF WAVE ENERGY

TYPICAL BREAKWATER CROSS SECTION
PLAN 1
MODEL SCALE 1:28

PLATE 1



MATERIAL CHARACTERISTICS

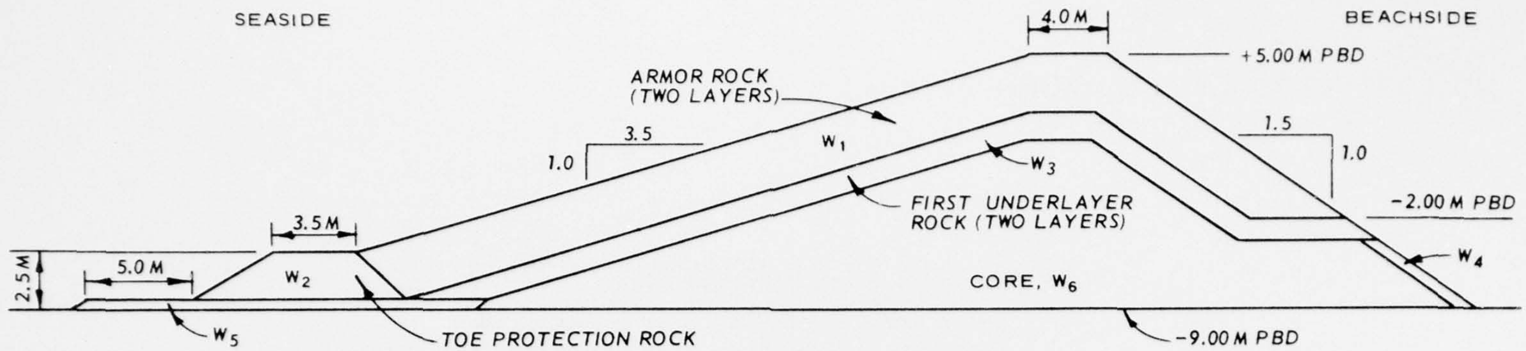
MODEL	PROTOTYPE
W_1 = 249.7-G ROCK	W_1 = 3.50-METRIC-TON ROCK @ 2.24 G/CM^3
W_2 = 67.8-G ROCK	W_2 = 0.95-METRIC-TON ROCK @ 2.24 G/CM^3
W_3 = 25.0-G ROCK	W_3 = 0.35-METRIC-TON ROCK @ 2.24 G/CM^3
W_4 = 5.4-G ROCK	W_4 = 75.00-KG ROCK @ 2.24 G/CM^3
W_5 = 0.1-G ROCK	W_5 = 1.50-KG ROCK @ 2.24 G/CM^3
W_6 = 1.2-G ROCK*	W_6 = 1.50-KG ROCK @ 2.24 G/CM^3

* GEOMETRICALLY OVERSIZED ROCK USED TO OBTAIN PROPER TRANSMISSION OF WAVE ENERGY

TYPICAL BREAKWATER CROSS SECTION

PLANS 2 AND 2A

MODEL SCALE 1:18



MATERIAL CHARACTERISTICS

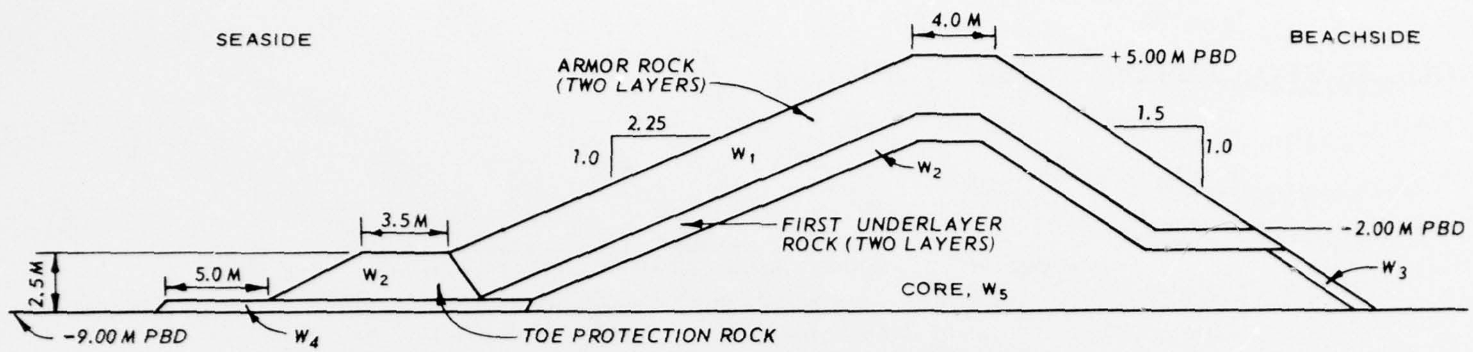
MODEL	PROTOTYPE
W_1 = 249.7-G ROCK	W_1 = 3.50-METRIC-TON ROCK @ 2.24 G/CM^3
W_2 = 67.8-G ROCK	W_2 = 0.95-METRIC-TON ROCK @ 2.24 G/CM^3
W_3 = 25.0-G ROCK	W_3 = 0.35-METRIC-TON ROCK @ 2.24 G/CM^3
W_4 = 5.4-G ROCK	W_4 = 75.00-KG ROCK @ 2.24 G/CM^3
W_5 = 0.1-G ROCK	W_5 = 1.50-KG ROCK @ 2.24 G/CM^3
W_6 = 1.2-G ROCK*	W_6 = 1.50-KG ROCK @ 2.24 G/CM^3

* GEOMETRICALLY OVERSIZED ROCK USED TO OBTAIN PROPER TRANSMISSION OF WAVE ENERGY

TYPICAL BREAKWATER CROSS SECTION

PLAN 3

MODEL SCALE 1:18



MATERIAL CHARACTERISTICS

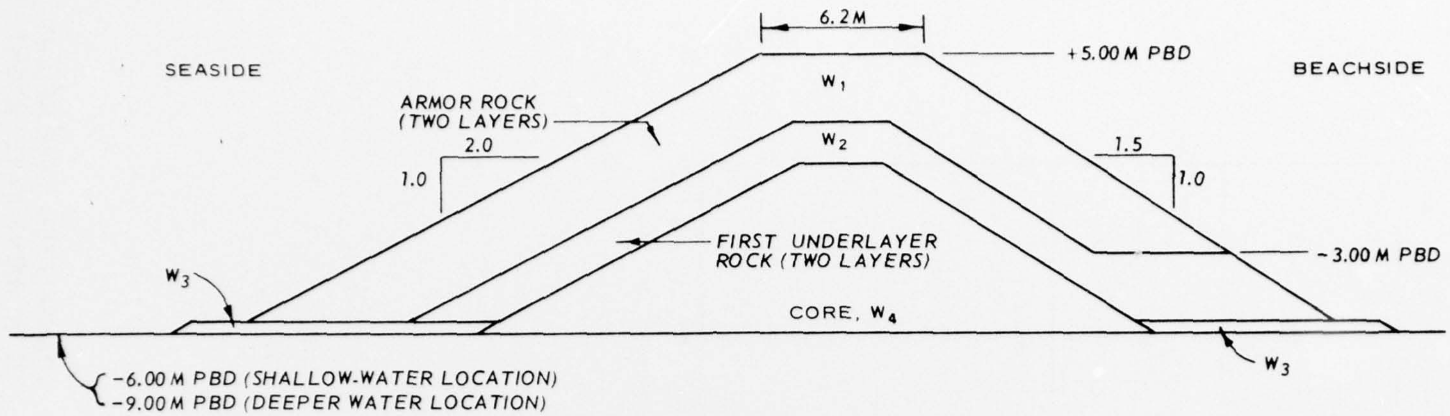
MODEL	PROTOTYPE
W_1 = 172.50-G ROCK	W_1 = 4.5-METRIC-TON ROCK @ 2.64 G/CM^3
W_2 = 9.50-G ROCK	W_2 = 0.5-METRIC-TON ROCK @ 2.24 G/CM^3
W_3 = 0.70-G ROCK	W_3 = 35.0-KG ROCK @ 2.24 G/CM^3
W_4 = 0.03-G ROCK	W_4 = 1.5-KG ROCK @ 2.24 G/CM^3
W_5 = 1.40-G ROCK*	W_5 = 1.5-KG ROCK @ 2.24 G/CM^3

* GEOMETRICALLY OVERSIZED ROCK USED TO OBTAIN PROPER TRANSMISSION OF WAVE ENERGY

TYPICAL BREAKWATER CROSS SECTION

PLAN 4

MODEL SCALE 1:28



MATERIAL CHARACTERISTICS

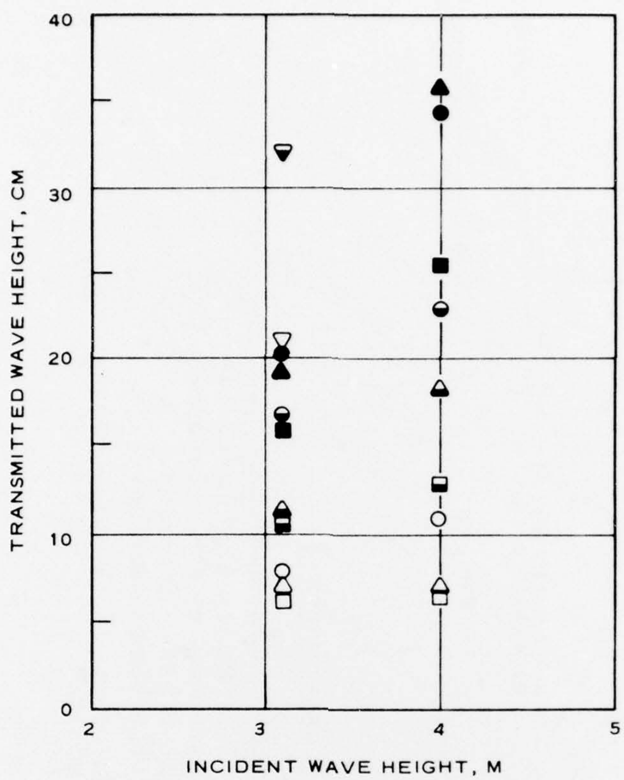
MODEL	PROTOTYPE
W_1 = 249.7-G ROCK	W_1 = 6.5-METRIC-TON ROCK @ 2.64 G/CM^3
W_2 = 23.2-G ROCK	W_2 = 0.6-METRIC-TON ROCK @ 2.64 G/CM^3
W_3 = 0.1-G ROCK	W_3 = 2.0-KG ROCK @ 2.64 G/CM^3
W_4 = 2.0-G ROCK*	W_4 = 2.0-KG ROCK @ 2.64 G/CM^3

* GEOMETRICALLY OVERSIZED ROCK USED TO OBTAIN PROPER TRANSMISSION OF WAVE ENERGY

TYPICAL BREAKWATER CROSS SECTION

PLAN 5

MODEL SCALE 1:28

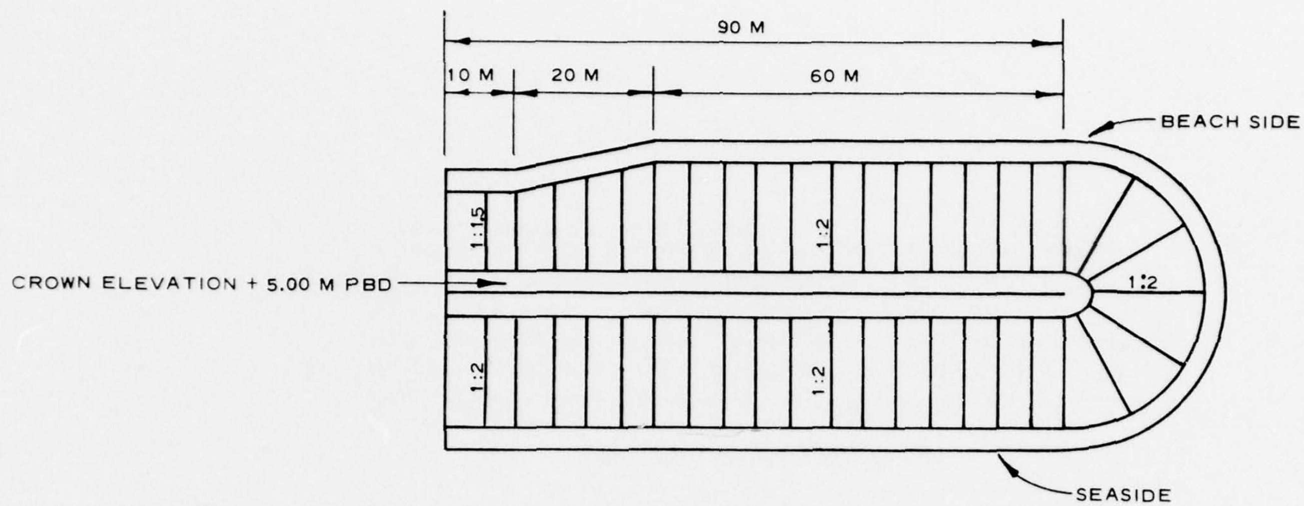


LEGEND

SYMBOL	PLAN	T, SEC
○	1	7
◒	1	9
●	1	11
□	2	7
⬓	2	9
■	2	11
△	3	7
◭	3	9
▲	3	11
▽	5	7
▼	5	9

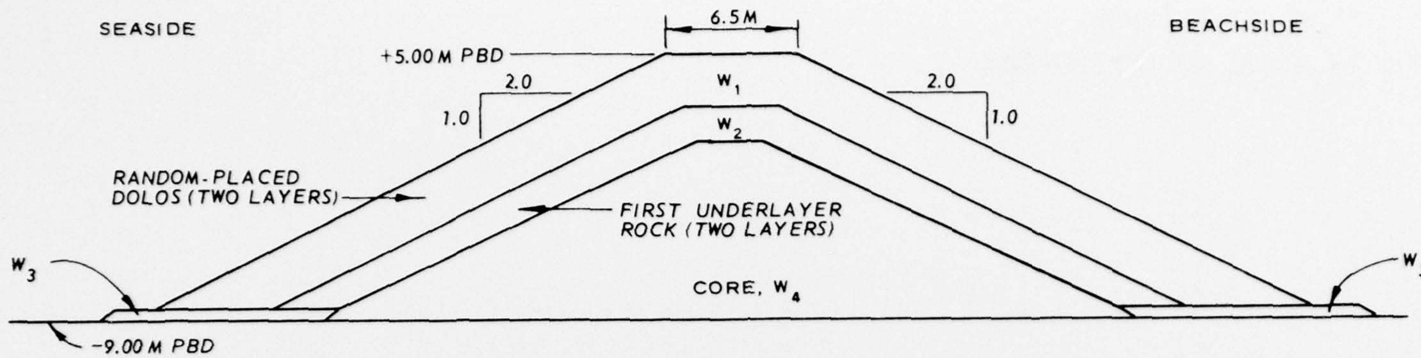
TRANSMITTED WAVE HEIGHT VS INCIDENT WAVE HEIGHT

PLANS 1, 2, 3, AND 5



OVERHEAD VIEW OF PLAN 6

MODEL SCALE 1:28



MATERIAL CHARACTERISTICS

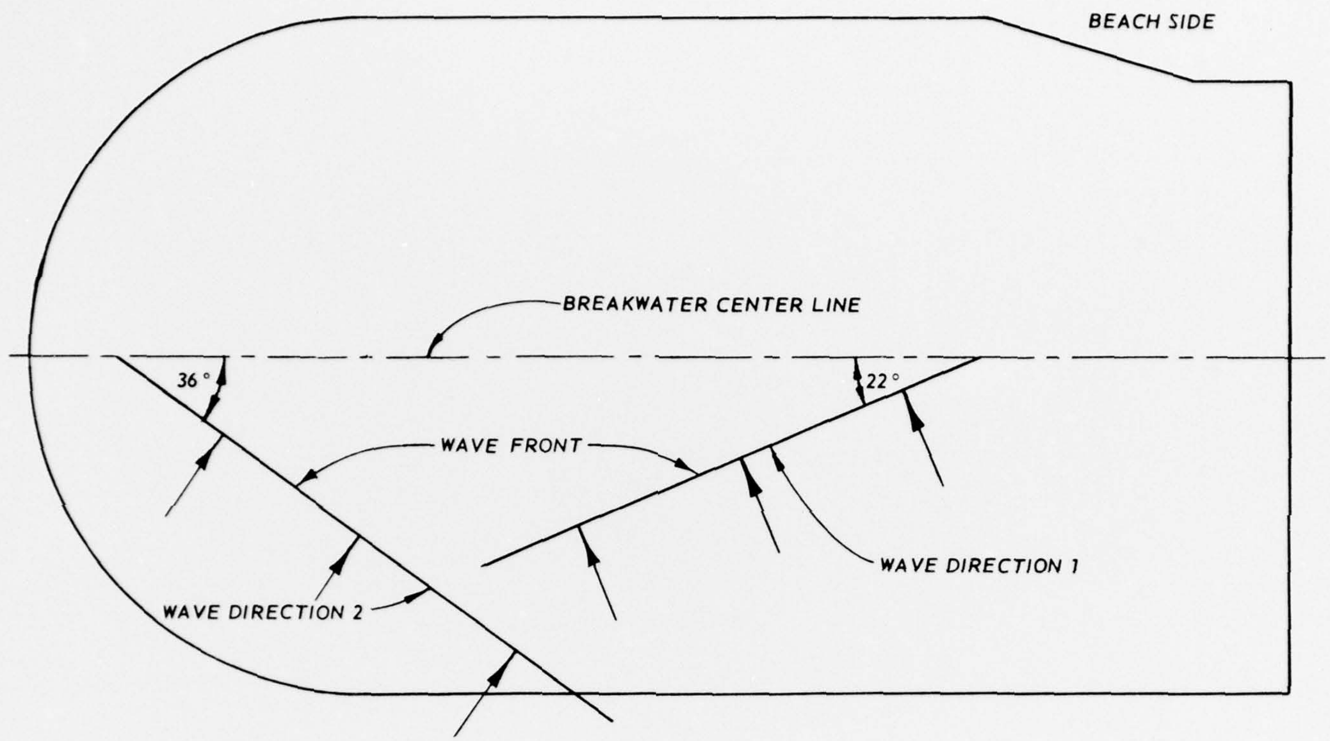
MODEL	PROTOTYPE
W_1 = 267.0-G DOLOS	W_1 = 5.0-METRIC-TON DOLOS @ 2.40 G/CM^3
W_2 = 40.4-G ROCK	W_2 = 1.0-METRIC-TON ROCK @ 2.64 G/CM^3
W_3 = 1.5-G ROCK	W_3 = 37.0-KG ROCK @ 2.64 G/CM^3
W_4 = 3.3-G ROCK*	W_4 = 37.0-KG ROCK @ 2.64 G/CM^3

* GEOMETRICALLY OVERSIZED ROCK USED TO OBTAIN PROPER TRANSMISSION OF WAVE ENERGY.

TYPICAL BREAKWATER CROSS SECTION

PLAN 6

MODEL SCALE 1:28



SELECTED TEST WAVE DIRECTIONS

PLAN 6

In accordance with ER 70-2-3, paragraph 6c(1)(b), dated 15 February 1973, a facsimile catalog card in Library of Congress format is reproduced below.

Carver, Robert D

Stability of rubble-mound breakwater, Jubail Harbor, Saudi Arabia; hydraulic model investigation, by Robert D. Carver [and] D. Donald Davidson. Vicksburg, U. S. Army Engineer Waterways Experiment Station, 1976.

1 v. (various pagings) illus. 27 cm. (U. S. Waterways Experiment Station. Technical report H-76-20)

Prepared for U. S. Army Engineer Division, Mediterranean, Livorno, Italy.

Includes bibliography.

1. Breakwaters. 2. Dolosse. 3. Hydraulic models. 4. Jubail Harbor, Saudi Arabia. 5. Rubble mound breakwaters. I. Davidson, D. Donald, joint author. II. U. S. Army Engineer Division, Mediterranean. (Series: U. S. Waterways Experiment Station, Vicksburg, Miss. Technical report H-76-20)

TA7.W34 no.H-76-20