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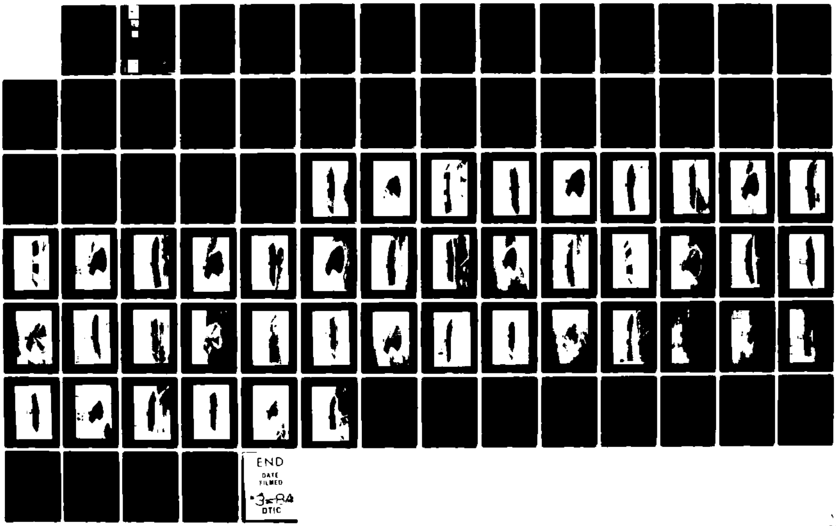

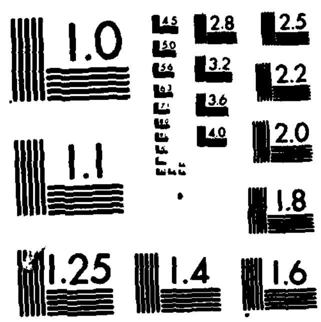

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US Army Corps of Engineers

TECHNICAL REPORT HL-83-18

BREAKWATER STABILITY STUDY, MISSION BAY, CALIFORNIA

Hydraulic Model Investigation

by

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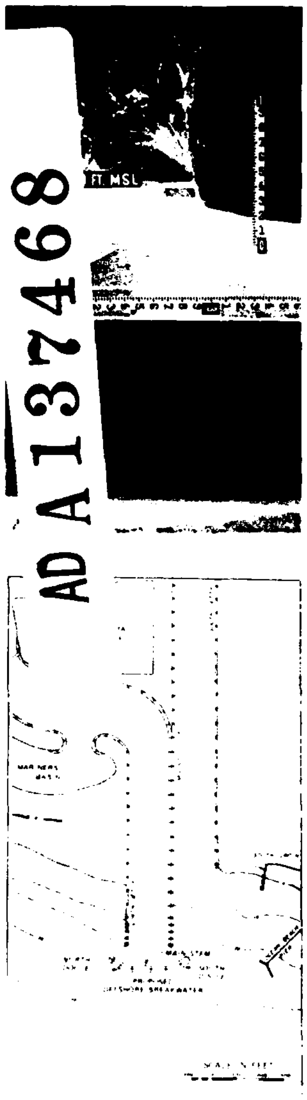

September 1983

Final Report

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HYDRAULICS LABORATORY

Prepared for U. S. Army Engineer District, Los Angeles
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A hydraulic model investigation was conducted using a three-dimensional stability model at an undistorted linear scale of 1:36 (model to prototype). The purpose of the stability tests was to develop a random-placed armor-stone design for a proposed offshore breakwater (to be located seaward of the existing north jetty and middle jetty at Mission Bay, California) that will be stable for nonbreaking wave heights up to and including 16.7 ft at still-water-

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levels of 0.0 and +5.4 ft mllw. Test results indicated that two alternative plans would be adequate for the nonbreaking wave conditions. One plan consisted of two layers of 29,000-lb stone random-placed on 1V-on-2H slopes on the breakwater heads and two layers of 22,700-lb stone random-placed on the 1V-on-2H ocean-side slopes and the 1V-on-1.5H channel-side slopes of the breakwater trunks. The other plan consisted of two layers of 29,000-lb stone random-placed on the 1V-on-2H to 1V-on-1.5H slopes on the breakwater heads, the 1V-on-1.5H ocean-side slopes, and the 1V-on-1.25H channel-side slopes.

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PREFACE

The hydraulic model investigation reported herein was requested by the U. S. Army Engineer District, Los Angeles, in November 1979 and was subsequently authorized by Intra-Army Order Number CIV-81-98 dated 29 June 1981.

The study was conducted by personnel of the Hydraulics Laboratory, U. S. Army Engineer Waterways Experiment Station (WES), during the period February 1982 to February 1983 under the general direction of Mr. H. B. Simmons, Chief of the Hydraulics Laboratory; Dr. R. W. Whalin and Mr. C. E. Chatham, former and acting Chiefs of the Wave Dynamics Division, respectively; and Mr. D. D. Davidson, Chief of the Wave Research Branch. The tests were conducted by Messrs. M. S. Taylor, H. F. Acuff, C. Lewis, and Mrs. B. J. Wright, Civil Engineering Technicians, under the supervision of Mr. D. G. Markle, Project Engineer. This report was prepared by Mr. Markle.

Commander and Director of WES during the conduct of the study and the preparation and publication of this report was COL Tilford C. Creel, 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
acres	4046.856	square metres
feet	0.3048	metres
miles (U. S. statute)	1.609344	kilometres
pounds (force)	4.448222	newtons
pounds (force) per cubic foot	157.087467	newtons per cubic metre

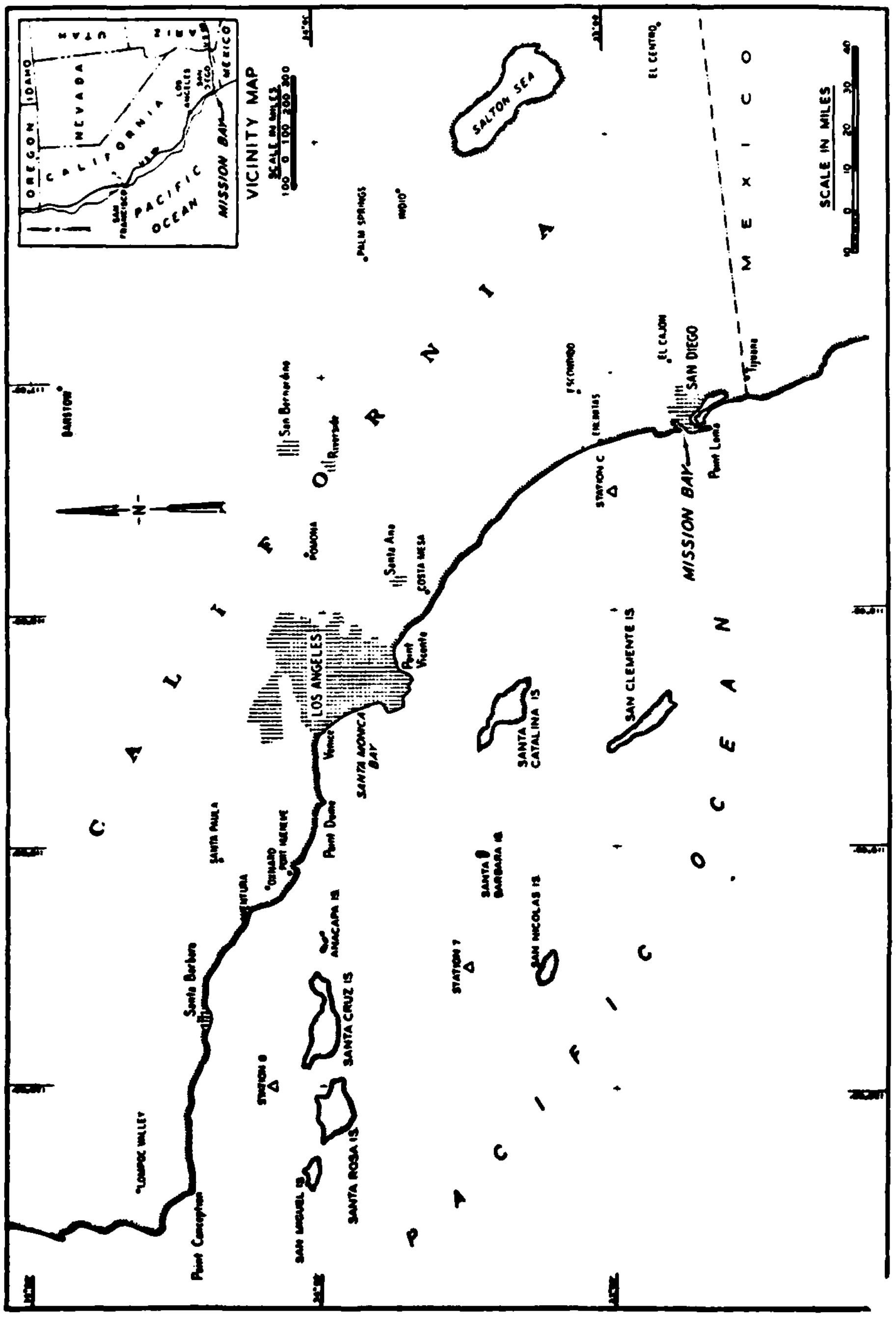

Figure 1. Project location map

BREAKWATER STABILITY STUDY, MISSION BAY, CALIFORNIA

Hydraulic Model Investigation

PART I: INTRODUCTION

The Prototype

1. Mission Bay is a tidal lagoon situated in the city of San Diego in southern California (Figure 1). The bay is separated from the Pacific Ocean by a broad 2-mile*-long sand spit called Mission Beach. The entrance to the bay is protected by two jetties (designated north jetty and middle jetty) that extend approximately 3,800 and 4,600 ft into the bay, respectively. The bay is comprised of several coves and basin and has an effective harbor area of about 2,000 acres of navigable water. The shallow-draft harbor can accommodate about 1,900 small boats consisting entirely of recreational and sport fishing craft.

The Problem

2. Various sea and swell storm conditions produce undesirable wave conditions at the entrance to the harbor and in the basins and coves. These wave conditions make it very hazardous to obtain entrance to the harbor during storms and often cause damage to moored vessels.

Proposed Offshore Breakwater

3. Test results on the three-dimensional (3-D) harbor wave action model of Mission Bay (Curren, in preparation) showed that an offshore breakwater, positioned as shown in Figure 2, would reduce the wave energy in the entrance channel, Mariners Basin, and Quivira Basin to the desired levels. The proposed offshore breakwater would consist of 350-ft-long north and south doglegs connected by a 900-ft-long main stem. The breakwater would have a continuous

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

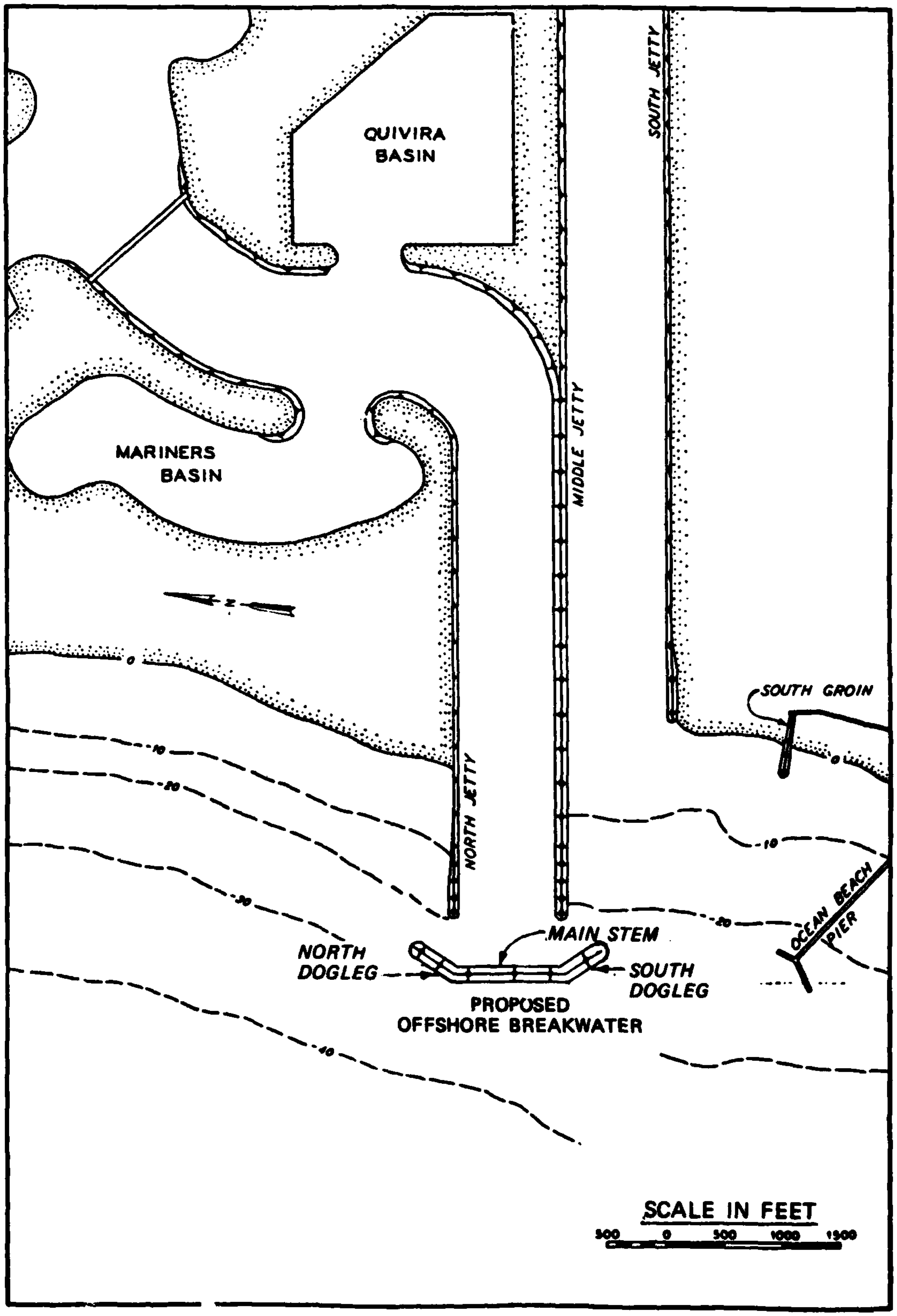

Figure 2. Proposed offshore breakwater

crown elevation of +17.5 ft mllw and head and ocean-side slopes of 1V on 2H and channel-side slopes of 1V on 1.5H.

Purpose of the Model Study

4. The purposes of the wave stability tests reported herein were as follows:

a. Develop a stable random-placed armor-stone design for the heads of the north and south doglegs.

b. Develop a stable random-placed armor-stone design for the ocean-side and channel-side slopes and crown of the north and south doglegs and the main stem.

PART II: THE MODEL

Design of the Model

5. Three-dimensional wave stability tests (for conditions with incident wave crests both parallel and at angles to the longitudinal axes of the breakwater) were conducted at an undistorted linear scale of 1:36, model to prototype. Scale selection was based on the size of model armor stone relative to the size of the prototype armor stone, elimination of stability scale effects (Hudson 1975), prototype wave and still-water level conditions, and capabilities of the available wave flume. Based on Froude's model law (Stevens 1942) and the linear scale of 1:36, the following model to prototype relations were derived. Dimensions are in terms of length (L) and time (T).

Characteristic	Dimension*	Model to Prototype Scale Relation
Length	L	$L_r = 1:36$
Area	L^2	$A_r = L_r^2 = 1:1,296$
Volume	L^3	$V_r = L_r^3 = 1:46,656$
Time	T	$T_r = L_r^{1/2} = 1:6$

6. The specific weight of water used in the model was 62.4 pcf and that of seawater is 64.0 pcf. The specific weight of the model construction material was identical with its prototype counterpart. Based on this information, the following transference equation was used to calculate the respective weights of the model construction material:

$$\frac{\left(W_a\right)_m}{\left(W_a\right)_p} = \frac{\left(\gamma_a\right)_m}{\left(\gamma_a\right)_p}\left(\frac{L_m}{L_p}\right)^3 \left[\frac{\left(S_a\right)_p - 1}{\left(S_a\right)_m - 1}\right]^3 \tag{1}$$

where

subscripts m and p = model and prototype quantities, respectively

W_a = weight of an individual stone, lb

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* For convenience, symbols and unusual abbreviations are listed and defined in the Notation (Appendix B).

γ_a = specific weight of an individual stone, pcf

L_m/L_p = linear scale of model

S_a = specific gravity of an individual stone relative to water in which the breakwater is constructed, i.e., $S_a = \gamma_a/\gamma_w$

γ_w = the specific weight of the water, pcf

7. The layer thicknesses of the various armor-stone and underlayer materials were calculated using the following equation:

$$t = nk_\Delta \left(\frac{W_a}{\gamma_a}\right)^{1/3} \qquad (2)$$

where

t = thickness, ft

n = number of stone layers

k_Δ = layer coefficient (k_Δ = 1.15 for rough quarrystone)

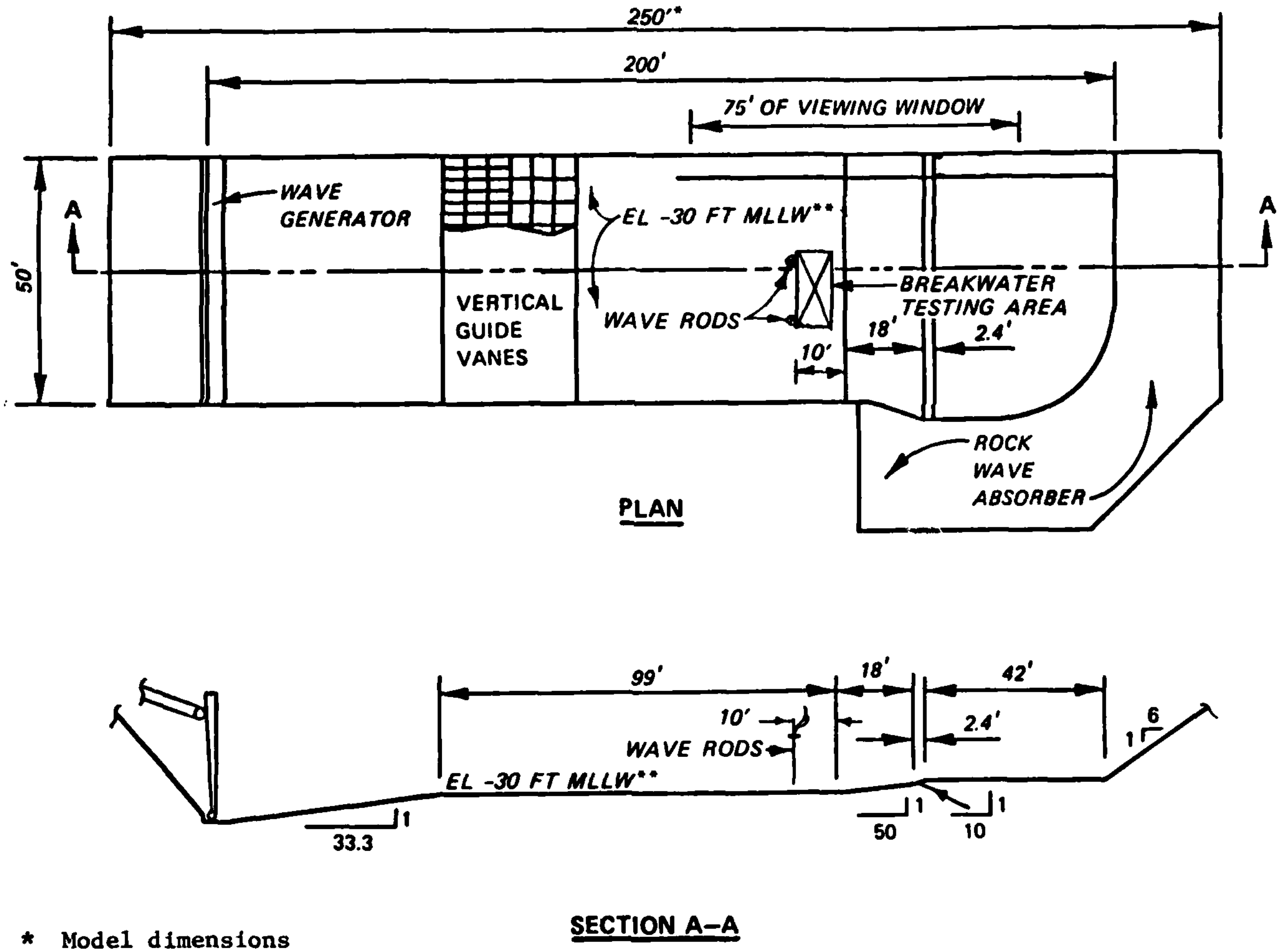
Test Facilities and Equipment

8. All tests were conducted in a portion of an L-shaped wave basin, 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 (Figure 3). The test facility was equipped with a flap-type wave generator, capable of producing monochromatic waves of various periods and heights.

Model Construction and Test Procedures

Modeling local bathymetry

9. The prototype bathymetry seaward of the proposed breakwater location is comprised of slopes ranging from 1V on 100H to 1V on 80H. Elevations along the toe of the breakwater will vary from approximately -24 to -30 ft mllw. Through a review of available data for the Mission Bay area, it was found that the largest waves that have reached the proposed breakwater location were nonbreaking. Therefore the shoaling created by the prototype slopes did not have to be reproduced in the stability model as long as the correct wave heights were reproduced at the toe of the model breakwater. For this reason, a flat bathymetry was reproduced at -30 ft mllw both seaward of and beneath



* Model dimensions
** Prototype elevations

Figure 3. Wave basin geometry and locations of breakwater and wave rods

the model breakwater. Since the tests were to include only nonbreaking wave test conditions, this constant toe elevation did not affect the stability test results.

Selection of test conditions

10. Test conditions were selected considering hindcast data used in both the 3-D harbor wave action model (Curren, in preparation) and research work by Hales (1979). These data were refracted to the wave generator position in the 3-D harbor wave action model and tests were conducted in the 3-D harbor wave action model to determine the largest wave heights that could occur at the proposed breakwater site. The largest significant wave heights recorded at the breakwater location were as follows:

Still-Water Level ft mllw	Wave Period sec	Wave Height ft
0.0	7	9.2
0.0	9	14.9
0.0	11	15.7
+5.4*	7	10.8
+5.4*	9	15.1
+5.4*	11	16.7

* Mean higher water.

At the request of the U. S. Army Engineer District, Los Angeles (SPL), the available wave data published by the State of California in the Coastal Data Information Program reports for the Mission Bay area were reviewed; and no recorded wave data for the Mission Bay area exceeded the significant wave heights listed above. Based on discussions between SPL and the U. S. Army Engineer Waterways Experiment Station (WES), the wave and still-water level (swl) conditions listed above were selected for consideration in the breakwater stability tests.

11. The refraction study, observations, and overhead photographs taken during the conduct of the 3-D harbor wave action model tests revealed that incident waves could approach from any direction between perpendicular to the north dogleg to perpendicular to the south dogleg. For the purpose of the breakwater stability study, two incident wave directions were selected (Plate 1). These are perpendicular to the crown of the dogleg and head (wave direction 1) and perpendicular to the crown of the main stem (wave

direction 2). Thus the incident wave crests from wave direction 1 make an angle of 33 deg with the crown of the main stem and are parallel to the crowns of the head and dogleg, while the incident wave crests from wave direction 2 make an angle of 33 deg to the crowns of the head and dogleg and are parallel to the crown of the main stem.

12. Model observations on the first test sections in the stability model for incident waves from wave directions 1 and 2 revealed that the 9- and 11-sec waves produced the worst nonbreaking wave attack at both water levels. These two wave periods were selected for inclusion in the full-length stability tests for both incident wave directions. These wave and swl conditions were referred to as Hydrograph A (Plate 2 and Table 1).

13. At the request of SPL, the acceptable breakwater design was exposed to longer period waves to check the integrity of the design relative to the more infrequent, high amplitude swell conditions. Two approaches were used to determine these test conditions. The first approach used the prototype data and the method of analysis used to arrive at the test conditions of Hydrograph A. During the prototype wave data analysis and subsequent measurements in the Mission Bay 3-D wave action model, it was found that a 15-sec, 9.5-ft nonbreaking wave was the largest wave with a wave period greater than 11 sec that could occur at the proposed breakwater location. This wave height was arrived at by numerically refracting the largest 15-sec wave height found in the prototype data to the water depth simulated in the wave generator pit of the 3-D wave action model. This wave height then was reproduced in the 3-D wave action model and the corresponding wave height was measured after the wave had shoaled and refracted to the proposed breakwater construction site. The second approach used an analysis of extreme wave heights at the Mission Bay entrance conducted by Seymour (1982). The prototype data used by Seymour were collected during the period 15 May 1979 to 21 June 1982 under the State of California Coastal Data Information Program. Using various statistical schemes, these data were extrapolated to wave heights that had return periods of up to 100 years. The swell-dominated observation (defined by Seymour as wave periods greater than 10 sec) had a wave height of 16.7 ft for a 50-year return period.

14. It was agreed by WES and SPL that the acceptable breakwater design was to be subjected to the swell conditions derived by WES (15-sec, 10-ft waves) and without reconstructing the test section, it would be exposed to the

higher amplitude swell conditions derived by Seymour (15-sec, 16.7-ft waves). These test conditions were referred to as Hydrograph B (Plate 3 and Table 2) and Hydrograph C (Plate 4 and Table 3), respectively.

Flume calibration

15. Before construction of the first breakwater test section, the wave basin was calibrated for the test waves and swl conditions described in paragraphs 10-14. Test waves of the required characteristics for the selected test depths were generated by varying the frequency and amplitude of the wave generator paddle motion. Changes in water-surface elevation with time (wave heights) were measured by electrical wave rods positioned where the sea-side toe of the breakwater would be located (Figure 3) and recorded on chart paper by an electrically operated oscillograph.

Methods of constructing test sections

16. Model breakwater sections were constructed to reproduce, as closely as possible, the results of prototype construction. The bedding layer and core were dumped by bucket or shovel and smoothed to grade with hand trowels. Metal templates, which extended through the breakwater core only, and an engineer's level were used to control the slopes and elevations. The core material was compacted with hand trowels to simulate normal consolidation that will occur in the prototype due to wave attack during the construction season. The underlayer stone was placed and smoothed to grade in the same manner as the core and bedding material, but the underlayer stone was not compacted. The armor-stone cover layers were constructed using random placement. Random placement means that the stones were selected at random from the stockpile and were individually placed, but were laid down in such a manner that no intentional interlocking or special orientation of the armor stone was achieved. The breakwater crown stones were placed in a somewhat random manner, but care was taken to assure that the correct crown elevation was achieved. This required some selective picking of stone shapes and some selective orientation so that stones would fit into the crown geometry. Crown width, average placement density, and thickness of armor layer used in the model followed those recommended in the Shore Protection Manual (CERC 1977) for two layers of rough quarrystone.

17. Since the north and south doglegs are symmetrical to the main stem, it would have been redundant to test both doglegs; therefore only one dogleg

and a portion of the main stem were reproduced in the model. Stability results determined on the model dogleg should be applicable to both prototype doglegs.

Model operation

18. Each of the breakwater plans was constructed in the test flume, before-test photographs were taken, the test flume was flooded to the appropriate depth, and the plan was exposed to the shakedown and test waves. Prototype test time was accumulated in 30-sec (model time) cycles (i.e., the wave generator was started, run for 30 sec, and then stopped). After each 30-sec cycle, sufficient time was provided for the test flume to still out before the next cycle was run. This procedure eliminated contamination of generated waves by rereflected waves from the wave generator. During still time between cycles, detailed model observations of the structure's response to the previous cycle of test waves were recorded by the model operator. These observations included any movement occurring on the structure and a general statement of the condition of the structure at that point in the test. No prototype data were available that indicated durations of various storm conditions. For this reason, each wave condition was run for the prototype duration indicated for the various test hydrographs. If all damage had not stabilized by completion of a test condition, the test was extended until such a time that all damage had stabilized or the amount of damage accrued was deemed unacceptable. At the conclusion of the test, the flume was drained and the after-test conditions of the structure were summarized in the test notes and documented with photographs. Each test plan then was rebuilt and the test was repeated. The purpose of the repeat test was to determine if there were any uncontrolled variations in the model construction that affected the stability of the structure. All of the initial and repeat test results obtained during this study were very similar. For this reason, only one test--either the initial or repeat test--was selected for inclusion in this report. Where the damage levels were slightly different, the test showing the higher damage level was selected for inclusion herein.

Methods of reporting model observations and test results

19. The following list of adjectives, in order of increasing severity, were used for recording model observations of armor unit activity and reporting test results of damage on each test section: (a) slight, (b) minor,

(c) moderate, (d) significant, (e) major, and (f) extensive. Slight and minor were used to describe acceptable activities or results, moderate described borderline acceptability, while significant to extensive described unacceptable conditions of increasing severity. Use of these adjectives allowed some quantification of the severity and/or amount of rocking in place, onslope displacement, and resulting damage accrued by the breakwater's cover-layer stones. By using the descriptive adjectives and the before-and-after test photographs, comparisons can be made between the alternative test plans.

PART III: TESTS AND RESULTS

Stability Tests--Hydrograph A

Plan S1, wave directions 1 and 2

20. Plan S1 (Plate 1 and Photos 1-3) reproduced the head and 180 ft of the dogleg, the 33-deg transition between the main stem and dogleg, and 180 ft of the main stem. The 1V-on-2H slopes on the head were armored with two layers of random-placed, 29,022-lb stone. Two layers of random-placed, 22,690-lb stone covered the 1V-on-2H ocean-side slopes, crown, and down to the 0.0-ft mllw elevation on the channel side of the remainder of the structure. Three layers of random-placed, 11,081-lb stone covered the channel-side slope between the 0.0 and -17.0 ft mllw elevations. The remainder of the channel-side slopes were constructed using 2,270-lb stone. The 18-ft-wide crown had a continuous elevation of +17.5 ft mllw. The 1V-on-2H slope of the head transitioned over a 75-ft length to a 1V-on-1.5H slope on the channel side of the breakwater.

21. Plan S1 accrued slight to minor spot damage during its exposure to the test conditions of Hydrograph A from wave direction 1 (Photos 4-6). Three armor stones were displaced downslope on the breakwater head (minor spot damage) and one armor stone was displaced downslope on the sea side of both the trunk of the dogleg (slight spot damage) and trunk of the main stem (slight spot damage). The armor-stone displacement occurred during Steps 3 and 4 of Hydrograph A. Some minor in-place armor-stone reorientation occurred on the sea-side slopes of the main stem and dogleg trunks. No other armor-stone displacement was observed, and all damage had stabilized well before the conclusion of Hydrograph A.

22. Plan S1 was turned (the wave generator position was fixed in the L-shaped wave basin; therefore model structures were turned to change incident wave directions) in order to check the stability of the breakwater when exposed to the wave and swl conditions of Hydrograph A from wave direction 2. The armor-stone layers were rebuilt (Photos 7-9) and Plan S1 was exposed to the test conditions of Hydrograph A from wave direction 2. Three to four armor stones shifted slightly downslope on the head (slight to minor damage). This was actually more of an in-place consolidation and reorientation of the armor stones as opposed to actual displacement. Five 22,690-lb armor stones

were displaced on the dogleg. Four of these stones migrated part way down the ocean-side slope (slight spot damage), and one stone was washed off the channel side of the dogleg crown (very slight spot damage) and came to rest on the upper part of the channel-side slope. Four 22,690-lb stones were displaced on the 33-deg transition between the dogleg and main stem (three down the ocean-side and one down the channel-side slopes, respectively). Seven and twelve 22,690-lb stones were displaced down the ocean-side and channel-side slopes, respectively, of the main stem (minor to moderate damage). In addition, nine 11,081-lb stones were displaced on the lower channel-side slope of the main stem (minor to moderate damage). All damage had stopped well before the end of the test, and the final condition of the test section is shown in after-test Photos 10-12. During the repeat testing of Plan S1 from wave direction 2, the head accrued a slightly higher degree of damage (six armor stones displaced, slight to moderate damage, Photo 13) while the remainder of the breakwater sustained less damage.

Plan S2, wave directions 1 and 2

23. In an effort to optimize the breakwater's armor-stone design, tests were initiated on Plan S2 (Plate 5 and Photos 14-16) for the wave and swl conditions of Hydrograph A from wave direction 2. The overall size and geometry of Plan S2 were identical with Plan S1. The armor-stone and underlayer stone weights were reduced in Plan S2 to see if smaller armor stone could withstand the test conditions without accruing unacceptable degrees of damage. Two layers of 22,690-lb armor stone were random-placed over the 2,270-lb underlayer stone on the breakwater head. The 22,690-lb, random-placed armor stone and 2,270-lb underlayer stone on Plan S1 were replaced with 18,470-lb and 1,850-lb stone, respectively, in Plan S2. The remainder of the construction material used in Plan S2 was identical with Plan S1. By the conclusion of Hydrograph A, the armor-stone layers of Plan S2 had sustained damage that ranged from slight spot damage to areas of concentrated significant damage (Photos 17-19). Twenty-three armor stones were displaced on the breakwater head. This displacement caused significant damage which resulted in a large area that had one layer of armor-stone protection and several areas of slight to minor spot damage. The dogleg sustained slight spot damage (two and one 18,470-lb stones displaced on the ocean-side and channel-side slopes, respectively). On the 33-deg breakwater transition, seven (moderate damage) and three (slight spot damage) stones were displaced on the ocean-side and

channel-side slopes, respectively. Twenty-five and sixteen 18,470-lb stones were displaced down the ocean-side and channel-side slopes, respectively, of the breakwater's main stem. Between sixteen and twenty 11,081-lb stones migrated downslope on the channel side of the main stem. Displacement of these two armor-stone sizes on the main stem resulted in moderate to significant damage. All damage had stopped before the end of the test, but the structure showed much more damage than would be acceptable.

24. Plan S2 was turned in the test facility in order to check the stability of the breakwater when exposed to the wave and swl conditions of Hydrograph A from wave direction 1. These tests were conducted to assure a complete stability analysis of Plan S2 from both test directions. Plan S2 was rebuilt (Photos 20-22) and the test section was exposed to the wave and swl conditions of Hydrograph A from wave direction 1. By the conclusion of the test, the breakwater head had accrued damage ranging from slight spot damage to significant concentrated damage, and the remainder of the test section had sustained spot damage that ranged from slight to minor (Photos 23-25). Nine armor stones were displaced on the breakwater head. Part of this displacement resulted in an area with only one layer of stone protection (approximately 3 to 4 stones wide), while the remainder of the displacement caused slight spot damage. Eight 18,470-lb armor stones (four on the ocean side and four on the channel side) were displaced downslope on the dogleg (slight to minor spot damage). One 18,470-lb stone was displaced downslope on the channel side of the transition between the main stem and the dogleg. Six 18,470-lb armor stones were displaced downslope on the main stem, resulting in slight spot damage. Three of these stones migrated down the channel side slope, while the other three were displaced on the ocean-side slope. During Steps 3 and 4 of Hydrograph A, approximately ten 11,041-lb armor stones were displaced downslope on the channel side of the test section. All armor-stone displacement stopped prior to the end of the test.

Plans S1 and S2, wave overtopping

25. Observations of wave overtopping during testing of Plans S1 and S2 for the wave and swl conditions of Hydrograph A from wave directions 1 and 2 revealed that Steps 1 and 2 produced slight to minor overtopping while Steps 3 and 4 produced moderate to significant overtopping. As well as producing the higher levels of overtopping, Steps 3 and 4 also caused the majority of the damage sustained by both breakwater plans. Overtopping was more pronounced on

the dogleg during wave direction 1 tests and was more pronounced on the main stem of the breakwater when test waves arrived from wave direction 2.

Stability Tests--Hydrographs B and C

26. With Plan S1 proving to be an adequate design for the wave and swl conditions of Hydrograph A from wave directions 1 and 2 and Plan S2 proving to be an inadequate design for the same test conditions, Plan S1 was reconstructed in the test flume (Photos 26-28) to check its integrity when exposed to the swell conditions previously described in paragraph 14. During previous testing of Plans S1 and S2, it was noted that incident waves from wave direction 2 produced more breakwater damage than incident waves from wave direction 1. For this reason, Plan S1 was reoriented and reconstructed in the test flume for testing with incident waves from wave direction 2. Plan S1 was exposed to the wave and swl conditions of Hydrograph B and the test section sustained no damage (Photos 29-31). The only armor-stone movement observed was some minor in-place rocking of a few stones on the ocean-side slopes of the breakwater. Steps 1 and 2 of Hydrograph B produced slight and minor wave overtopping, respectively, but no armor-stone movement occurred on the channel side of the breakwater. Without rebuilding the test section, Plan S1 was exposed to the wave and swl conditions of Hydrograph C. Step 1 produced minor overtopping and some moderate in-place rocking of a few armor stones on the ocean side of the breakwater, but no armor-stone movement was observed on the channel side of the test section. During Step 2, Plan S1 accrued slight to moderate damage. The moderate to significant overtopping produced by Step 2 displaced thirteen 22,690-lb armor stones (minor to moderate damage) and ten to twelve 11,081-lb armor stones (slight to minor damage) on the channel-side slopes and two 29,022-lb armor stones (slight damage) on the breakwater head. The overtopping wave energy also produced minor to moderate amounts of rocking and in-place reorientation of several armor stones on the channel-side slopes. Except for some minor to moderate in-place rocking of a few armor stones on the ocean-side slopes, no other armor-stone activity was observed. All damage stabilized well before the end of Hydrograph C and the after-test condition of Plan S1 is shown in Photos 32-34.

Transmission Tests

27. During the conduct of the 3-D harbor wave action model tests of Mission Bay (Curren, in preparation), it was assumed that 10-ton armor stone would be needed for stability and that the prototype core and first underlayer stone would be impermeable to wave transmission. Therefore a barrier extending up to +7.5 ft mllw was placed along the longitudinal center line of the breakwater. Results of the 3-D breakwater stability tests reported herein have shown that approximately 11.25-ton armor stone is needed for stability on the trunks of the breakwater (Plan S1), and therefore, if the +17.5 ft mllw crown elevation is maintained the first underlayer will only extend up to +5.6 ft mllw. Also, a barrier was not placed along the center line of the breakwater during testing of the 3-D breakwater stability model. Subsequent to the completion of the stability tests, SPL became concerned about what effect, if any, the assumption that the first underlayer is impermeable and the lowering of the first underlayer stone would have on wave transmission. Thus at the request of SPL, WES conducted comparative wave transmission tests using the 3-D stability breakwater model. A description of these tests and their results are presented in Appendix A.

Stability Tests--Hydrographs A and C

28. Subsequent to the conduct of the test previously described, SPL requested that WES conduct a stability check test of a breakwater plan that was identical in overall length with Plans S1 and S2, but that had 1V-on-1.5H slopes on the ocean side of the trunks and 1V-on-1.25H slopes on the channel side of the trunks. The 1V-on-1.5H slopes transitioned (linearly, over a length of 75 ft) to 1V-on-2H slopes on the ocean side of the heads (Plate 6). The 1V-on-2H slopes continued around to the end of the heads and then spiraled in to form 1V-on-1.5H slopes on the channel side of the heads. These 1V-on-1.5H slopes then transitioned (linearly, over a length of 75 ft) to the 1V-on-1.25H slopes on the channel side of the doglegs. Test results of Plan S1 showed that 22,690-lb stone was needed for stability on the 1V-on-2H ocean-side slopes and upper channel-side slopes of the main stem and dogleg trunks. By steepening these slopes, larger armor stone is needed in order to attain stability equivalent to that of Plan S1. By use of the Hudson equation

$$W = \frac{\gamma_a H^3}{K\left(S_a - 1\right)^3 \cot \alpha} \quad (3)$$

where

W = weight of an individual stone, lb

γ_a = specific weight of an individual stone, pcf

H = design or test wave height, ft

K = stability coefficient

S_a = specific gravity of an individual stone relative to the water in which the breakwater is constructed, i.e., $S_a = \gamma_a/\gamma_w$

γ_w = the specific weight of the water, pcf

α = angle the breakwater slope makes with the horizontal, deg

for $W = 22{,}690$ lb, $\gamma_a = 165$ pcf, $H = 16.7$ ft, $\gamma_w = 64.0$ pcf, and $\cot \alpha = 2.0$, it can be shown that the stability of the trunks of Plan S1 are representative of a stability coefficient of 4.3. Using the proposed steeper slope of 1V on 1.5H and the calculated stability coefficient ($K = 4.3$), it was determined that approximately 30,000-lb armor stone would be needed for stability on the steeper sloped trunks. This was very close to the 29,022-lb armor-stone weight that was needed for stability on the 1V-on-2H sloped heads of Plan S1. Therefore it was decided that the new plan would be tested with two layers of random-placed, 29,022-lb stone on the heads, ocean-side slopes, crown, and down to an elevation of 0.0 ft mllw on the channel side of the breakwater. Random-placed armor stone having individual weights of 14,500 lb would be placed between el 0.0 and -17.0 ft mllw on the channel sides of the main stem and dogleg trunks. The remainder of the channel-side slopes would be constructed using 2,900-lb underlayer material. It was requested by SPL that the steeper sloped plan be tested with two layers of first underlayer material on the ocean-side and channel-side slopes and that the first underlayer crown be reduced to one layer in order to bring the core material up to an elevation of +1.6 ft mllw. This was done in an effort to reduce the amount of wave energy being transmitted through the breakwater.

29. Plan S3 (Plate 6 and Photos 35-37) reproduced, as closely as possible, the conditions described in the preceding paragraph. The model test section reproduced the head, 180 ft of the dogleg, the 33-deg transition between the main stem and dogleg, and 180 ft of the main stem. The test section

accrued damage ranging from slight to moderate during exposure to the wave and swl conditions of Hydrograph A from wave direction 2 (Photos 38-40). Seven (minor to moderate damage) and five (minor damage) 29,022-lb armor stones were displaced downslope on the ocean sides of the main stem and dogleg, respectively. The head sustained very slight spot damage due to the displacement of one armor stone. Ten 29,022-lb armor stones were displaced downslope on the channel side of the dogleg resulting in minor to moderate damage. One of these stones came from the breakwater crown. Four and two 29,022-lb and 14,500-lb stones, respectively, were displaced down the channel-side slope of the 33-deg transition. The channel side of the main stem accrued moderate damage as a result of the significant overtopping produced by Steps 3 and 4 of Hydrograph A. Fifteen and eight 29,022-lb and 14,500-lb stones, respectively, were displaced downslope resulting in one spot lowering in the crown (one armor stone displaced) and two areas on the upper slope with only one layer of armor-stone protection. Without any rebuilding or repair of the armor-stone layers, the test section was exposed to Hydrograph C from wave direction 2 to see if the longer period waves would cause any additional damage. Seven 29,022-lb stones (six on the channel side of the dogleg and one on the ocean side of the head) were displaced downslope during Step 2 of Hydrograph C. One 14,500-lb armor stone was displaced on the main stem during this same time period. This additional displacement increased the damage level on the channel side of the dogleg from minor to moderate damage to moderate damage. Damage levels on the remainder of the structure were considered unchanged by the wave and swl conditions of Hydrograph C. All damage stabilized well before the end of the test. Photos 41-43 show the condition of Plan S3 at the end of the test.

PART IV: DISCUSSION

30. Test results indicated that while both Plans S1 and S3 are considered adequate designs for the selected test conditions, they are not "no damage" designs. Both plans accrued moderate degrees of damage on the channel sides, which will most likely require some maintenance after major storm events; but neither plan showed potential for loss of their functional and structural integrities. Model observations also showed that the channel-side slopes of Plan S3 (1V on 1.25H) are slightly less stable than those of Plan S1 (1V on 1.5H) and in the long term will most likely require more maintenance.

PART V: CONCLUSIONS

31. Based on the tests and results* reported herein, it is concluded that:

a. Plan S1 is an adequate design for the wave and swl conditions of Hydrograph A from wave directions 1 and 2, provided the minor to moderate amounts of damage accrued by the breakwater are acceptable.

b. Plan S1 is a very adequate design for the wave and swl conditions of Hydrograph B from wave direction 2.

c. Plan S1 is an adequate design for the wave and swl conditions of Hydrograph C from wave direction 2, provided the minor to moderate amounts of damage accrued by the breakwater are acceptable.

d. The head of Plan S2 is not an adequate design and the remainder of the breakwater is an adequate design for the wave and swl conditions of Hydrograph A from wave direction 1.

e. Neither the head nor the trunk sections of Plan S2 are an adequate design for the wave and swl conditions of Hydrograph A from wave direction 2.

f. Plan S3 is an adequate design for the wave and swl conditions of Hydrograph A from wave direction 2, provided the minor to moderate amounts of damage accrued by the breakwater are acceptable.

g. Plan S3 is an adequate design for the wave and swl conditions of Hydrograph C from wave direction 2, provided the moderate amounts of damage accrued by the breakwater are acceptable.

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* Test results presented in this report relate to the stability of the breakwater and should be considered with test results by Curren (in preparation) for selection of optimum harbor protection.

PART VI: RECOMMENDATIONS

32. Even though Plans S1 and S3 showed adequate stability when exposed to the selected test conditions, they did sustain moderate amounts of damage. As discussed in paragraph 19, this terminology is used to describe designs that show borderline acceptability. Therefore periodic inspections and maintenance of the structures will be necessary, especially after major storm events. Both structures were deemed adequate based on random armor-stone placement, i.e., each stone was individually placed but laid down in such a manner that no intentional interlocking or special orientation was achieved. If it could be stressed to the contractor to achieve better than random armor-stone placement, especially on the crowns and channel-side slopes, the long-term maintenance costs for this structure should be significantly reduced.

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Table 1

Hydrograph A

Wave Directions 1 and 2

Step	swl, ft mllw	Test Wave Period, sec	Test Wave Height, ft	Prototype Duration, hr	Wave Type
	0.0	9	8.5	0.25	Shakedown
1	0.0	9	14.9	1.0	Nonbreaking
2	0.0	11	15.7	1.0	Nonbreaking
3	+5.4	9	15.1	1.0	Nonbreaking
4	+5.4	11	16.7	1.0	Nonbreaking

Table 2

Hydrograph B

Wave Direction 2

Step	swl, ft mllw	Test Wave Period, sec	Test Wave Height, ft	Prototype Duration, hr	Wave Type
	0.0	9.0	8.5	0.25	Shakedown
1	0.0	15.0	10.0	1.00	Nonbreaking
2	+5.4	15.0	10.0	1.00	Nonbreaking

Table 3

Hydrograph C

Wave Direction 2

Step	swl, ft mllw	Test Wave Period, sec	Test Wave Height, ft	Prototype Duration, hr	Wave Type
1	0.0	15.0	16.7	1.0	Nonbreaking
2	+5.4	15.0	16.7	1.0	Nonbreaking

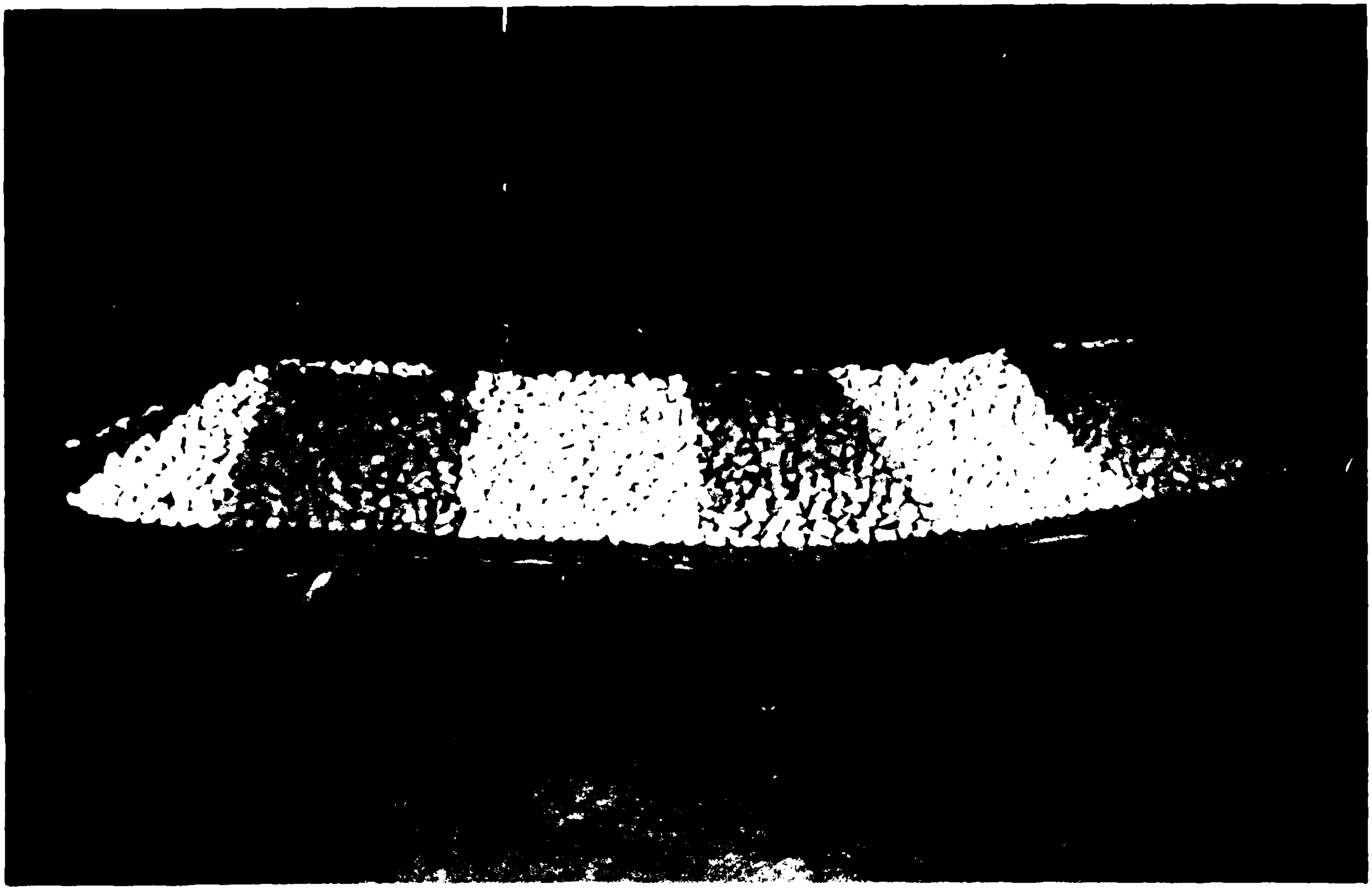


Photo 1. Ocean-side view of Plan S1 before testing Hydrograph A, wave direction 1

Photo 2. End view of Plan S1 before testing Hydrograph A, wave direction 1

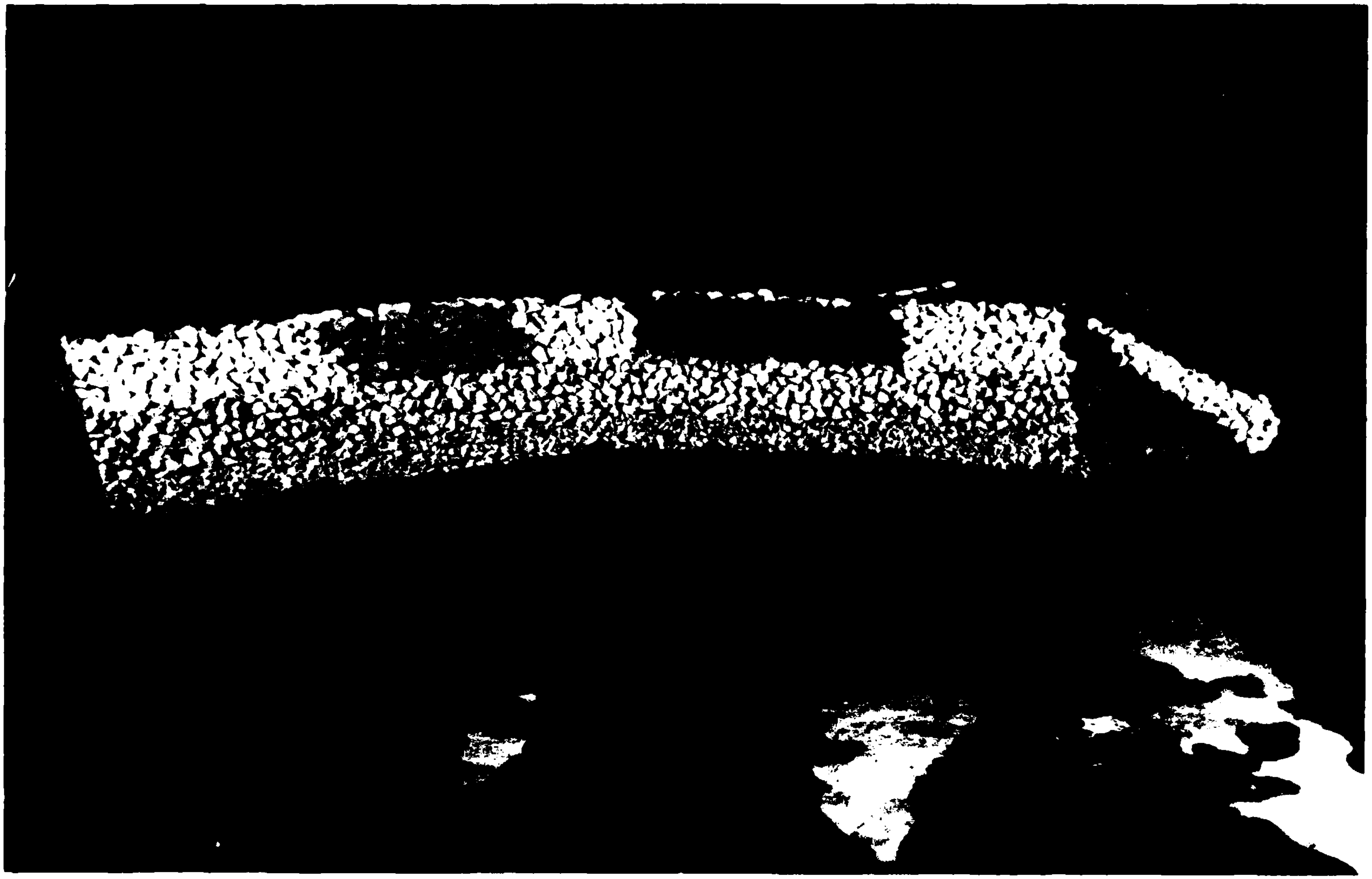


Photo 3. Channel-side view of Plan S1 before testing Hydrograph A, wave direction 1

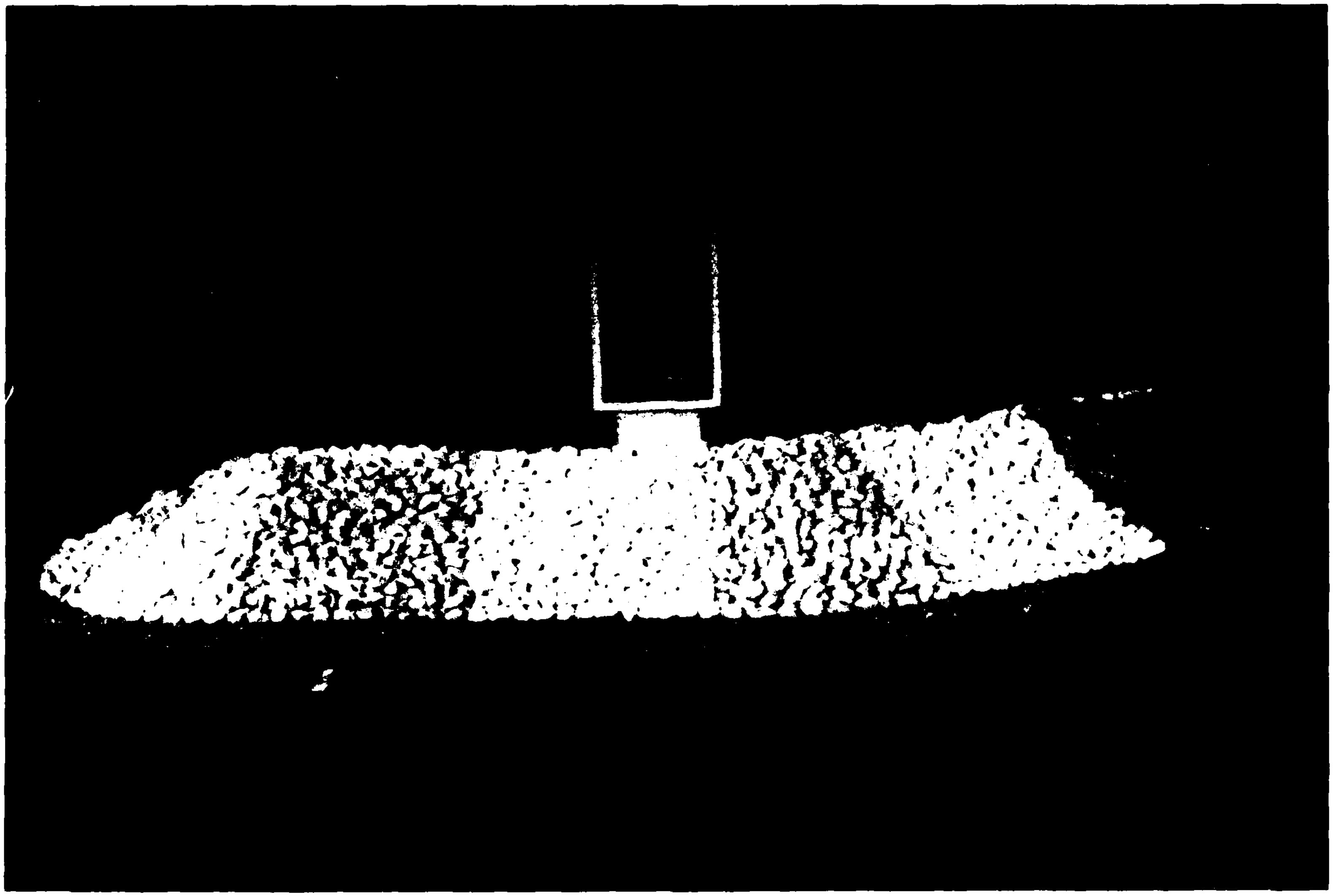


Photo 4. Ocean-side view of Plan S1 after testing Hydrograph A, wave direction 1

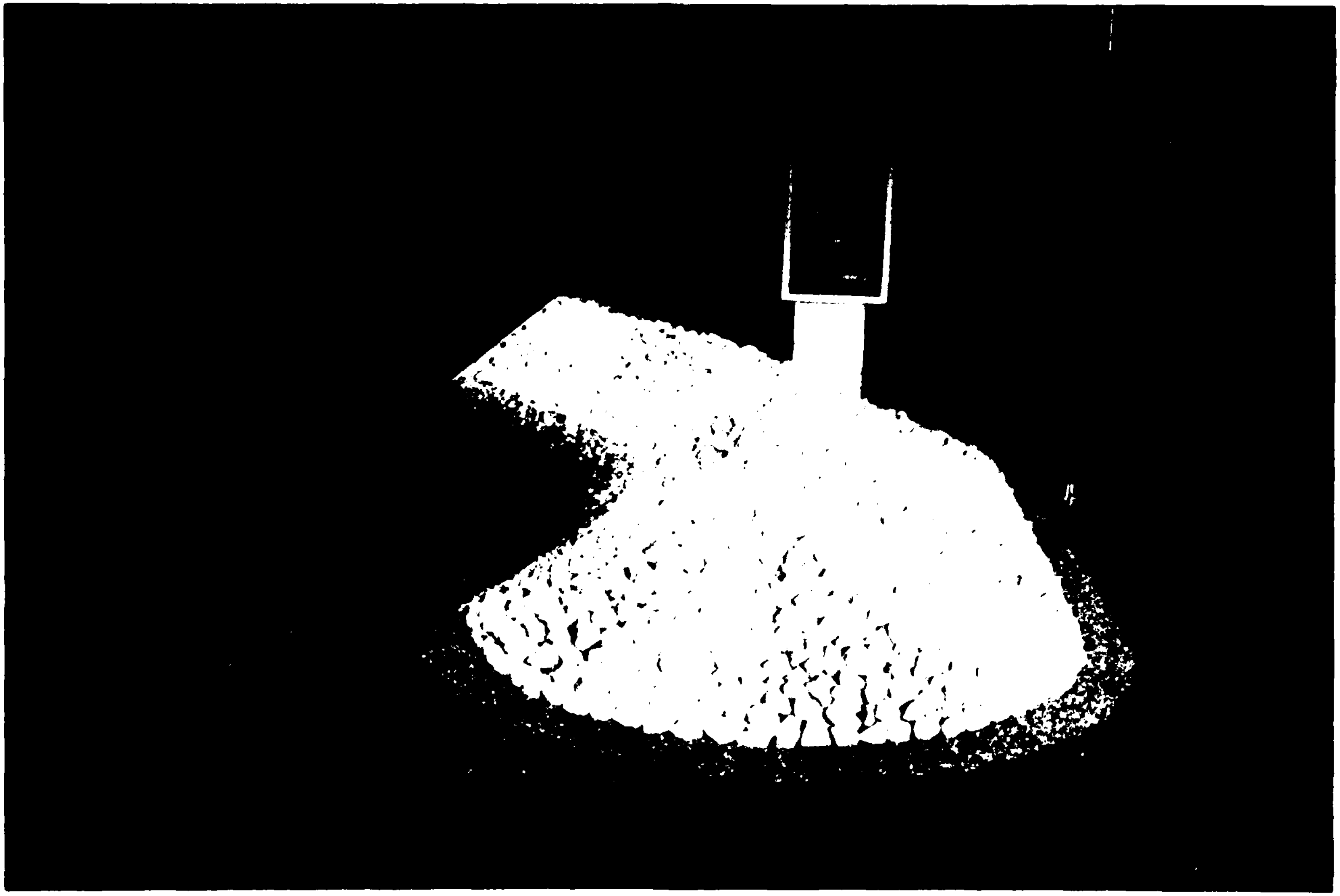


Photo 5. End view of Plan S1 after testing Hydrograph A, wave direction 1

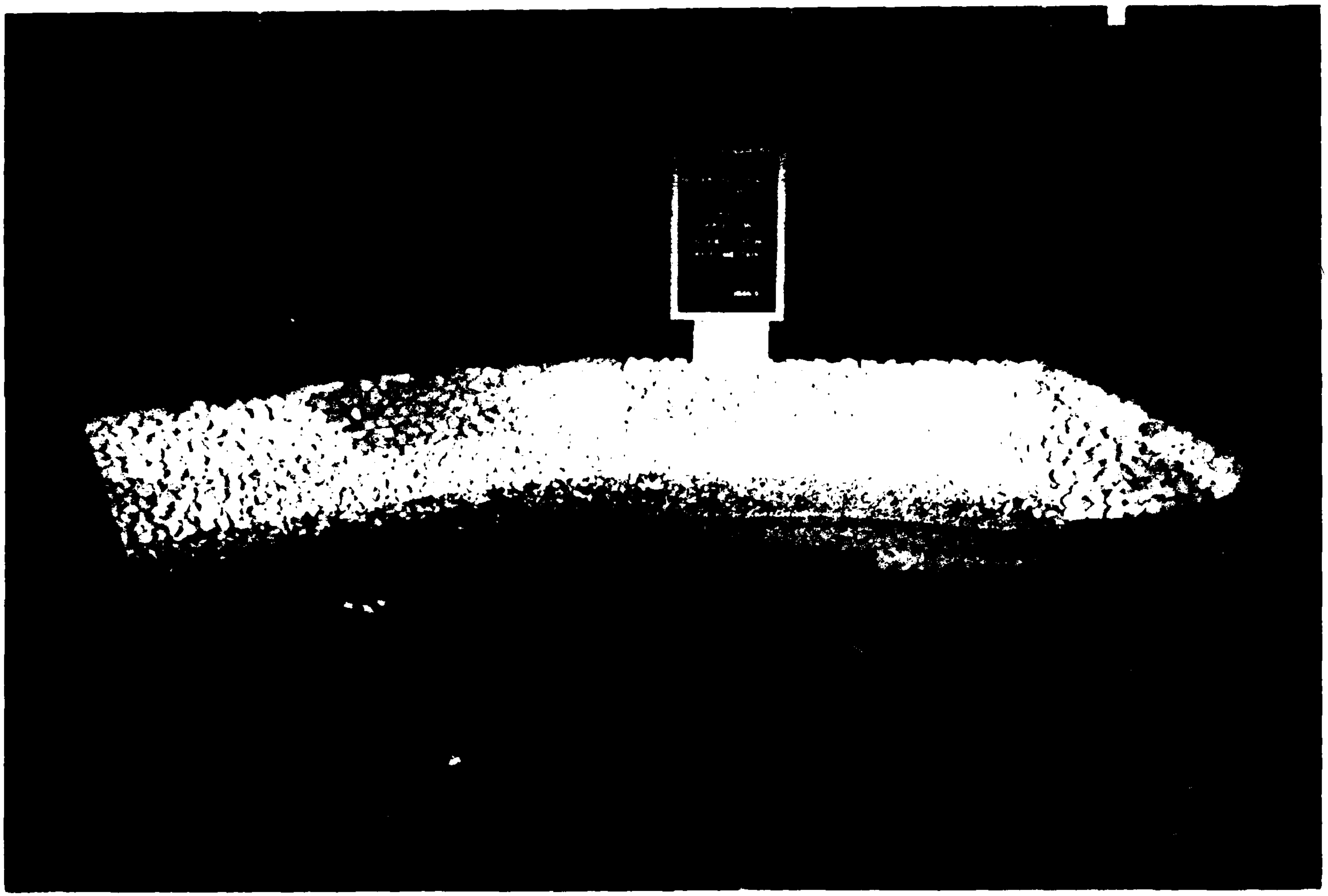


Photo 6. Channel-side view of Plan S1 after testing Hydrograph A, wave direction 1

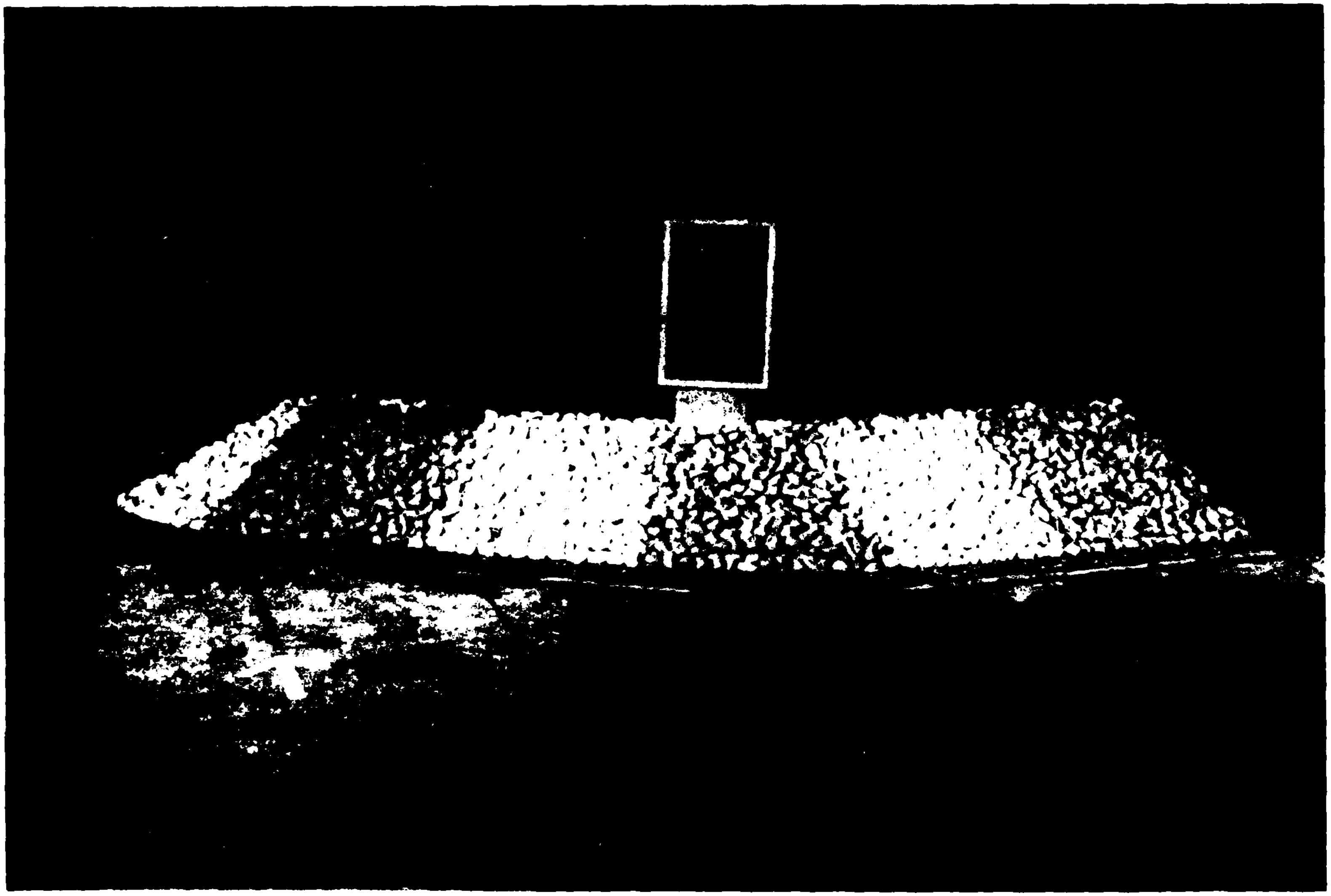


Photo 7. Ocean-side view of Plan S1 before testing Hydrograph A, wave direction 2

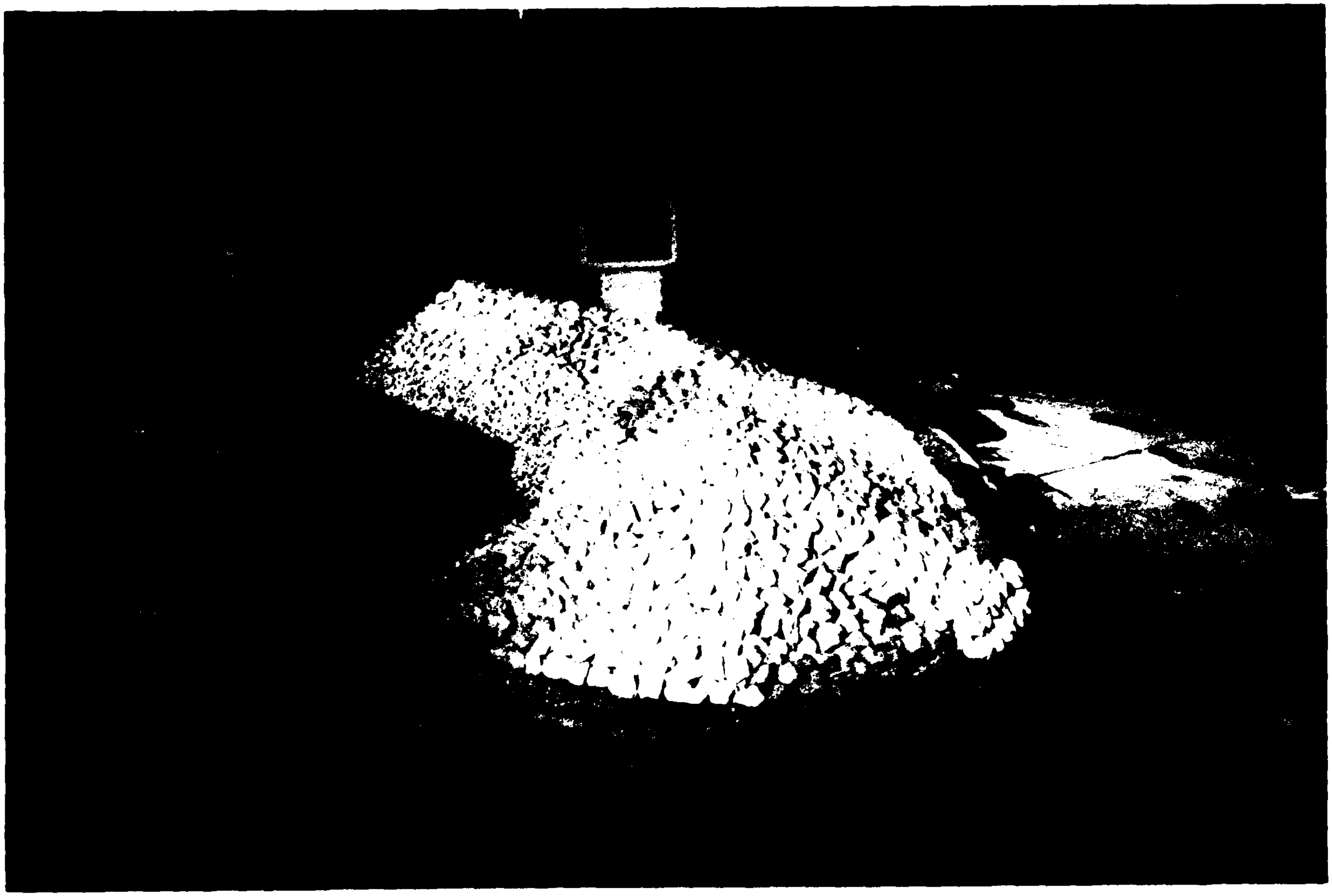


Photo 8. End view of Plan S1 before testing Hydrograph A, wave direction 2

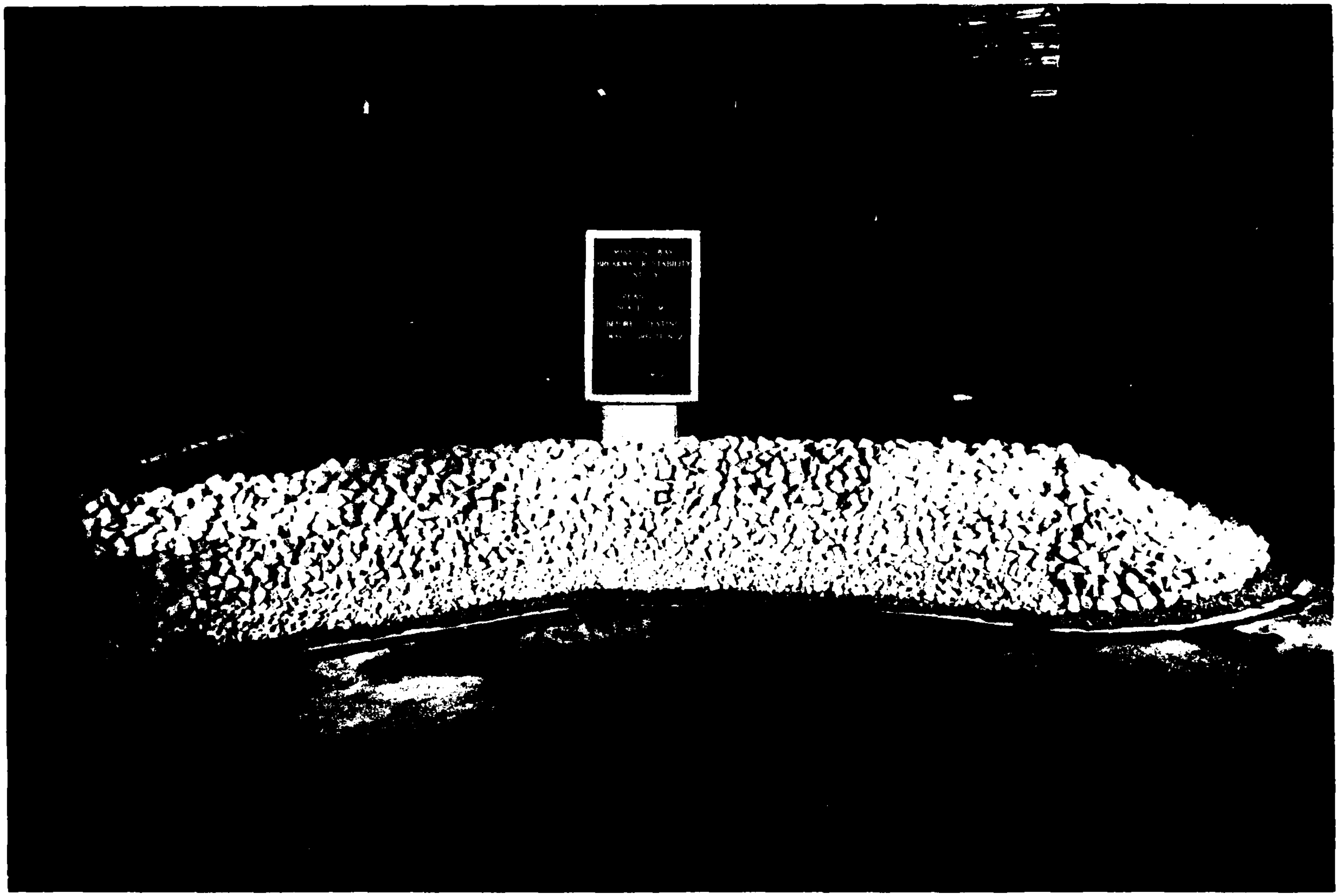


Photo 9. Channel-side view of Plan S1 before testing Hydrograph A, wave direction 2

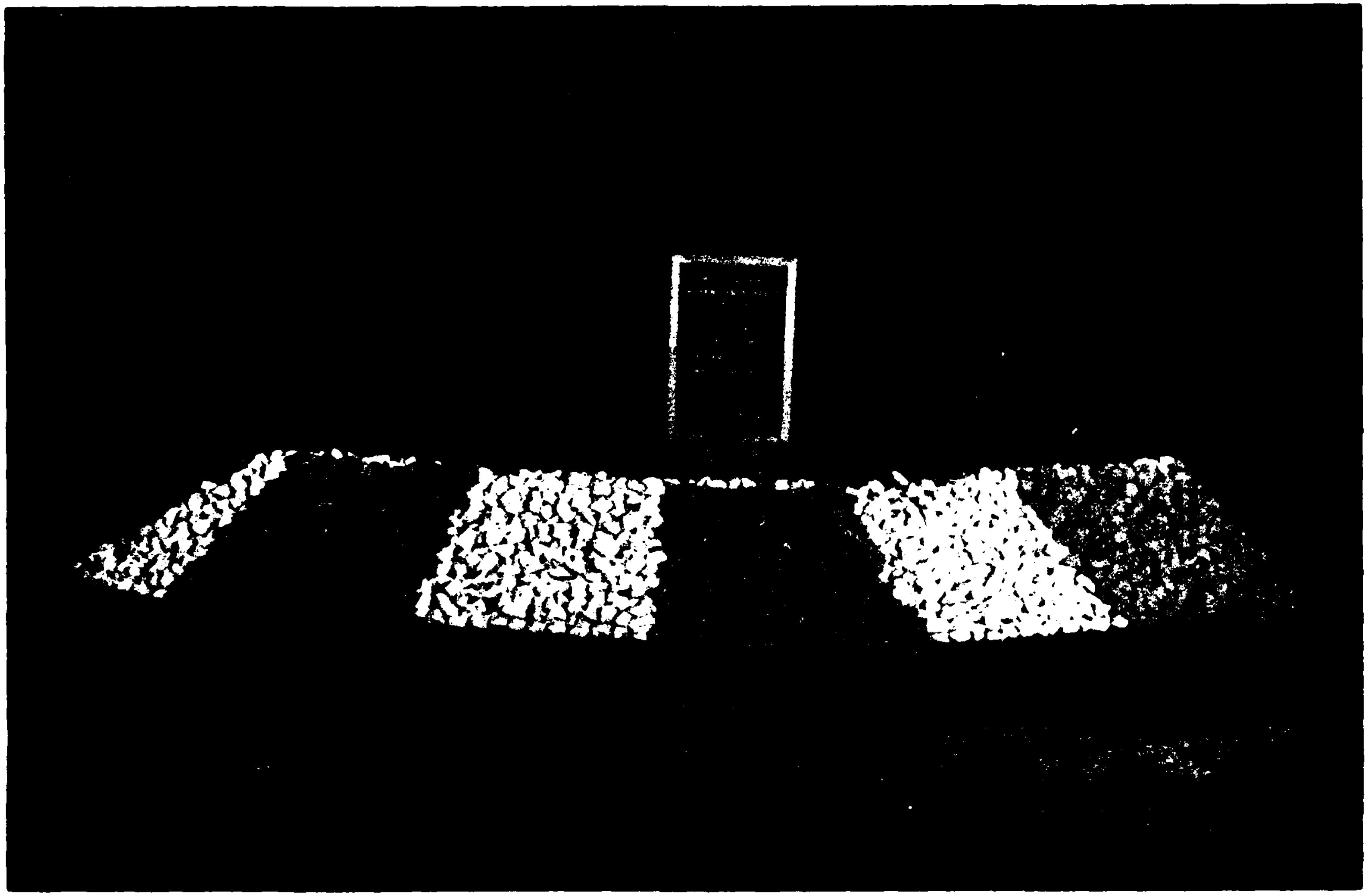


Photo 10. Ocean-side view of Plan S1 after testing Hydrograph A, wave direction 2

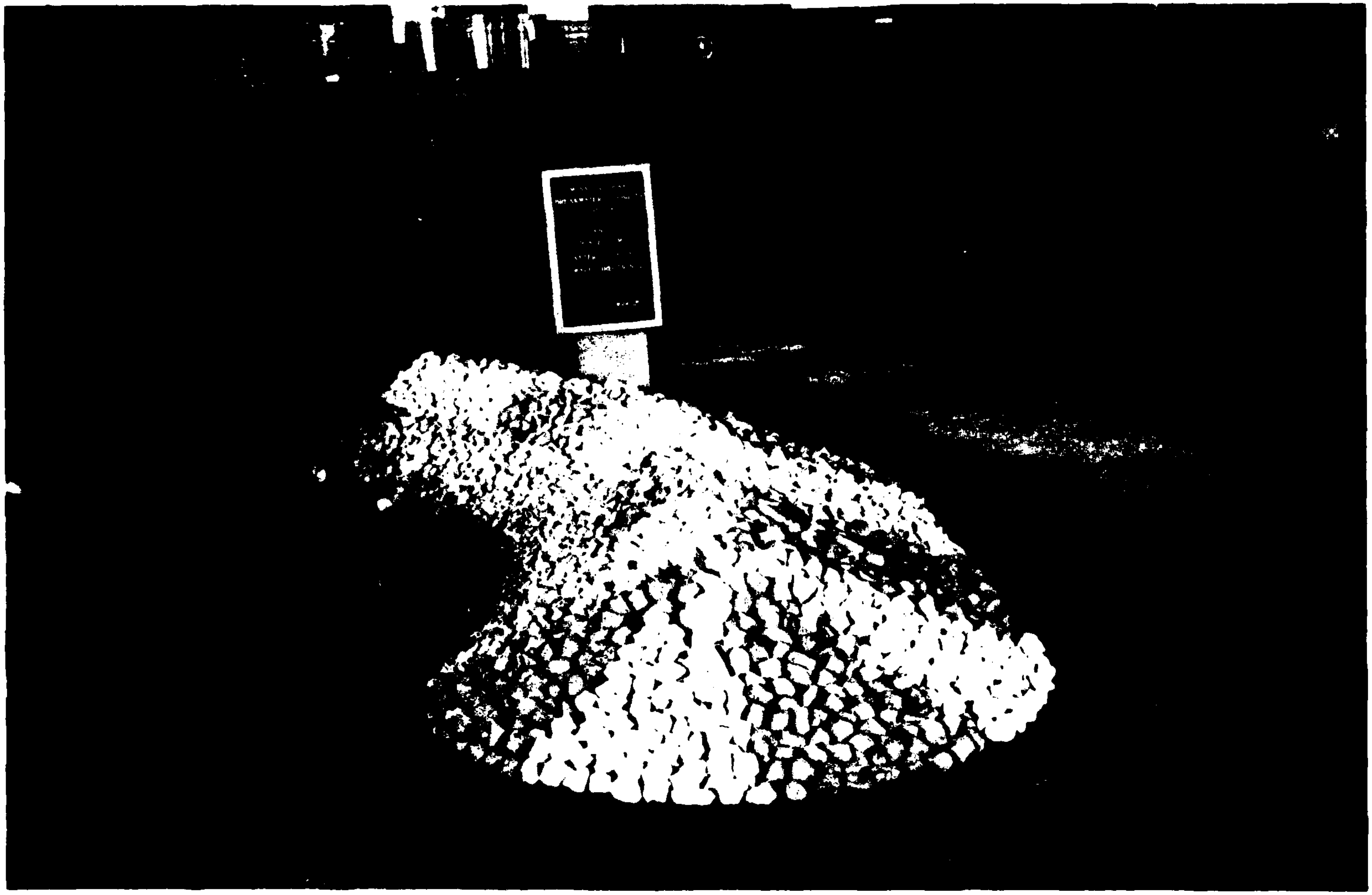


Photo 11. End view of Plan S1 after testing Hydrograph A, wave direction 2

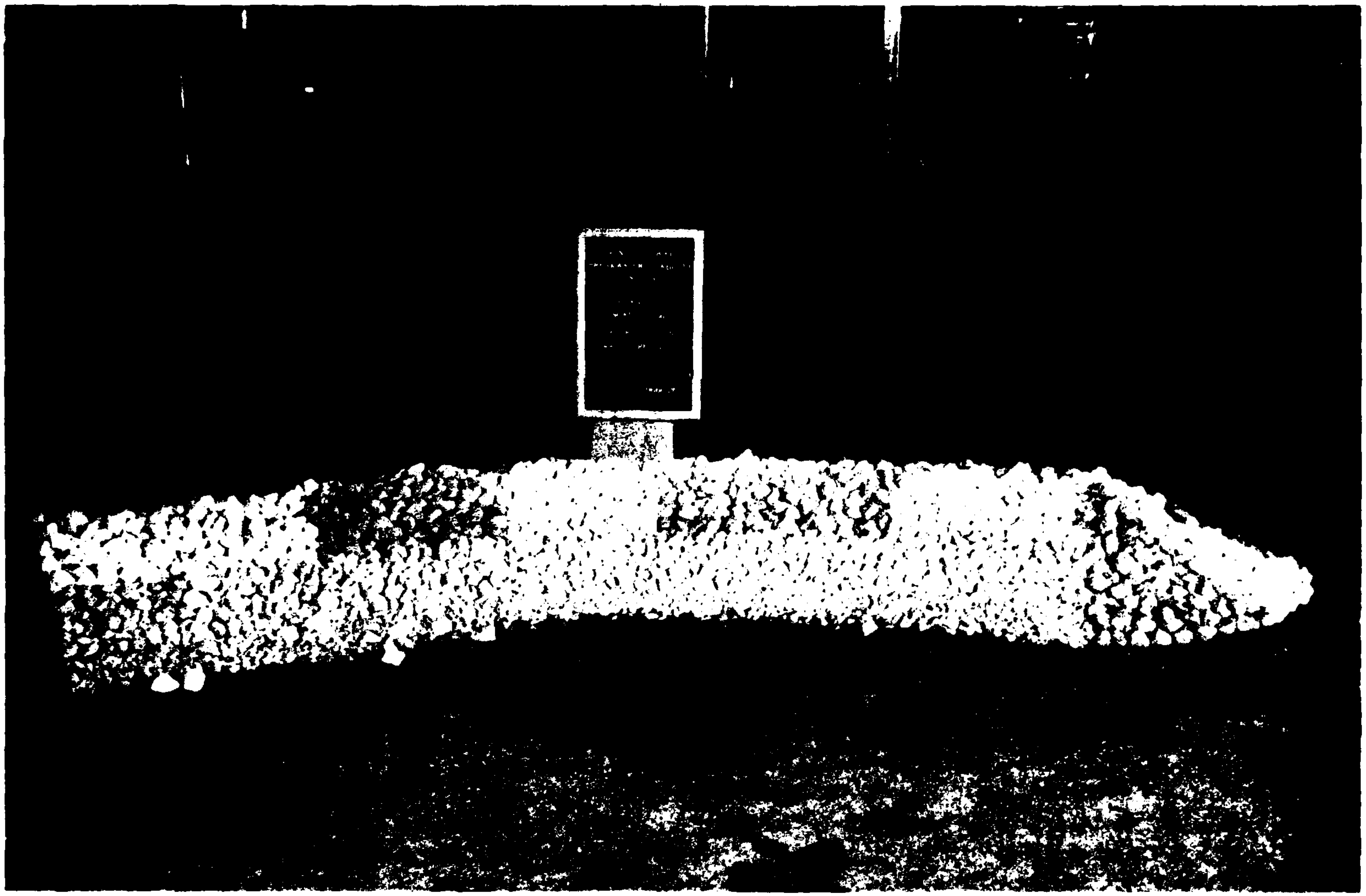


Photo 12. Channel-side view of Plan S1 after testing Hydrograph A, wave direction 2

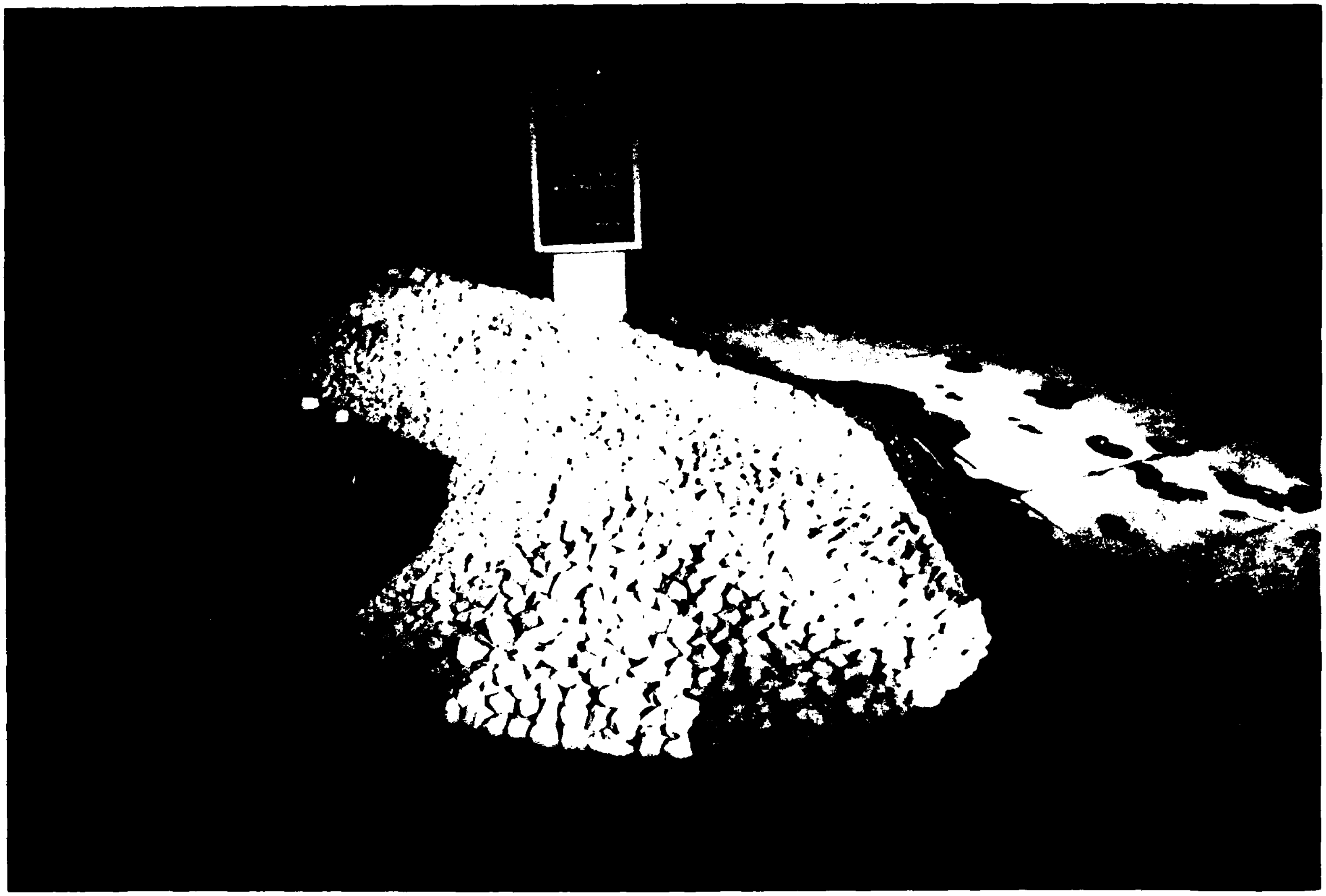


Photo 13. End view of Plan S1 after testing Hydrograph A, wave direction 2, repeat test

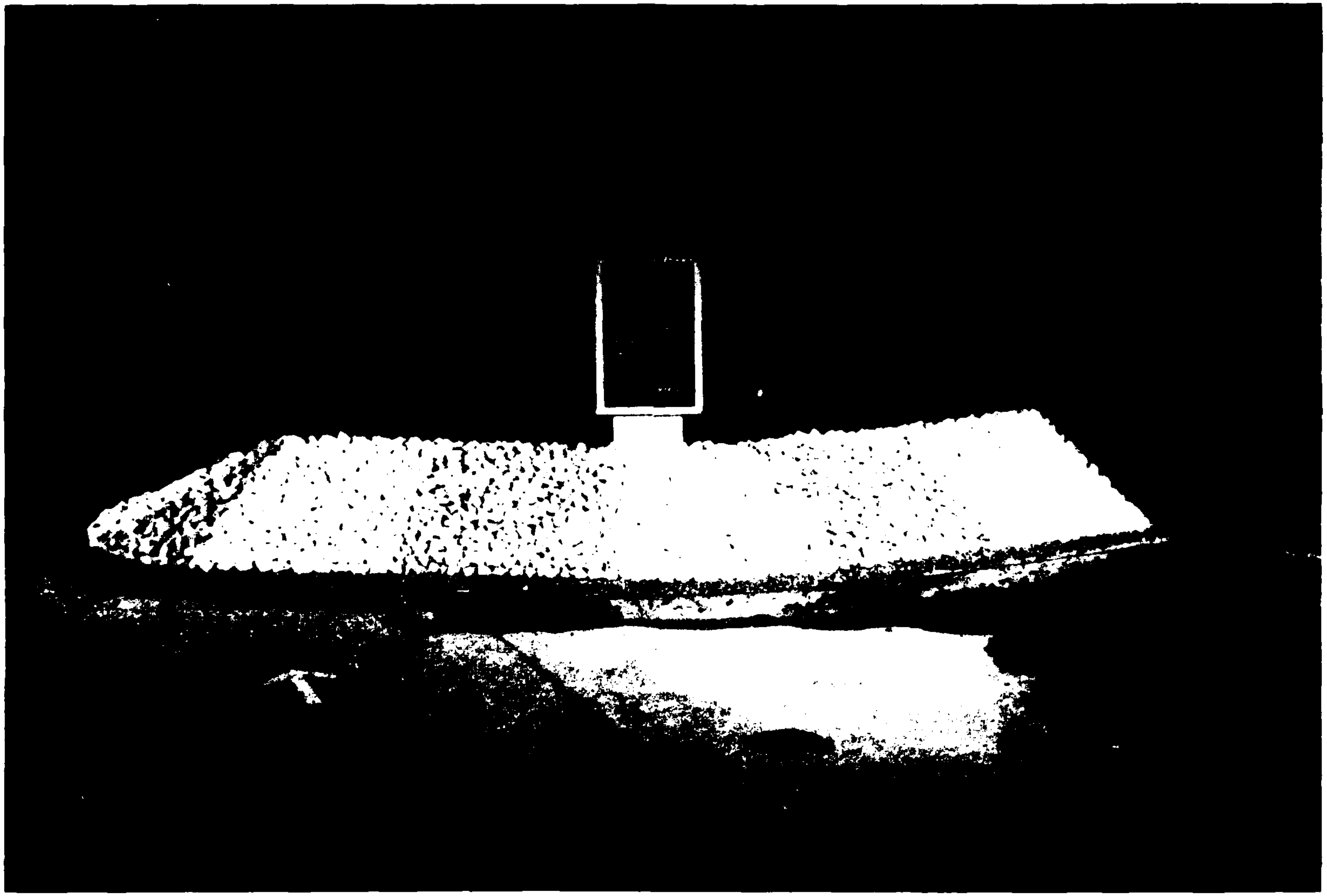


Photo 14. Ocean-side view of Plan S2 before testing Hydrograph A, wave direction 2

Photo 15. End view of Plan S2 before testing Hydrograph A, wave direction 2

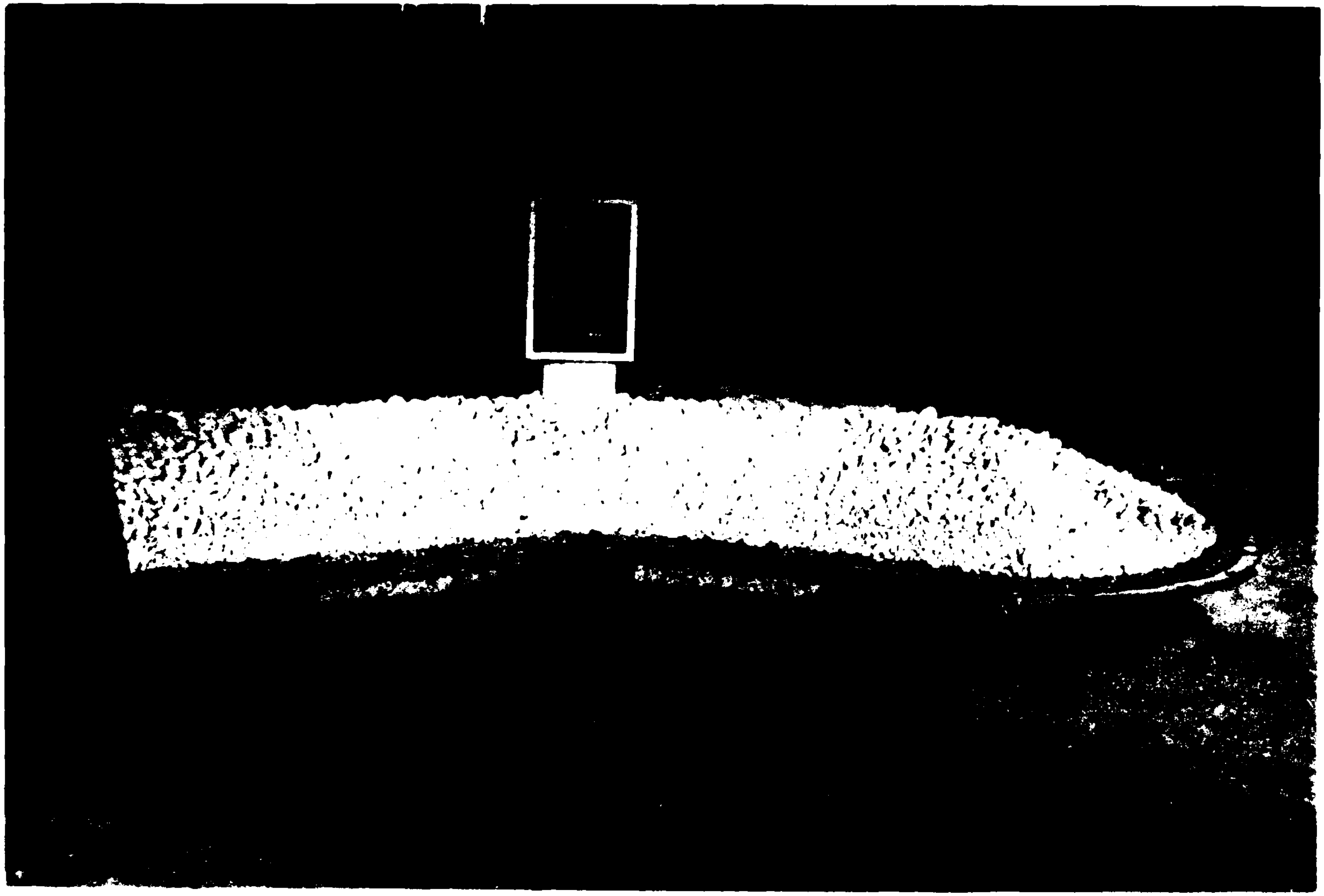


Photo 16. Channel-side view of Plan S2 before testing Hydrograph A, wave direction 2

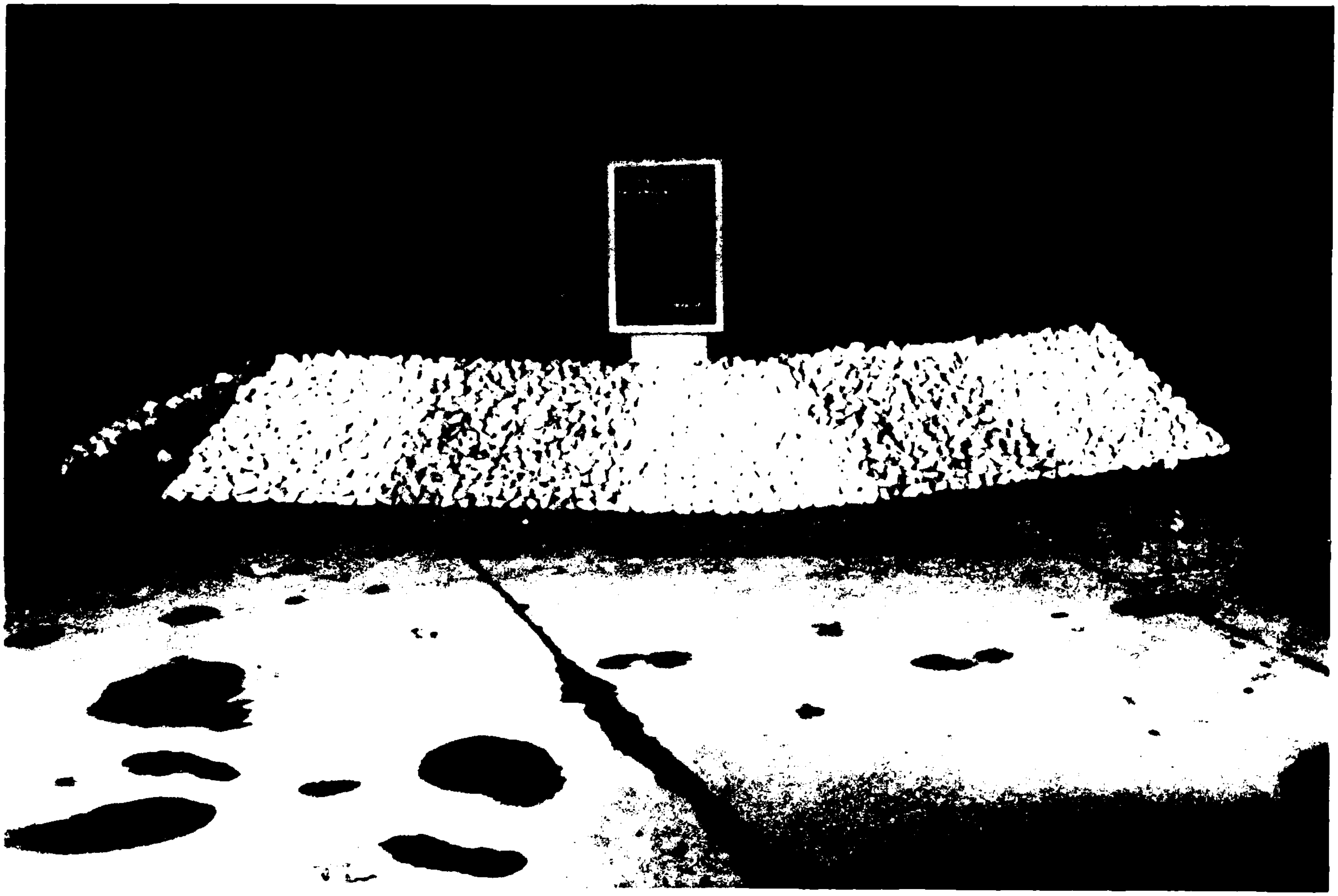


Photo 17. Ocean-side view of Plan S2 after testing Hydrograph A, wave direction 2

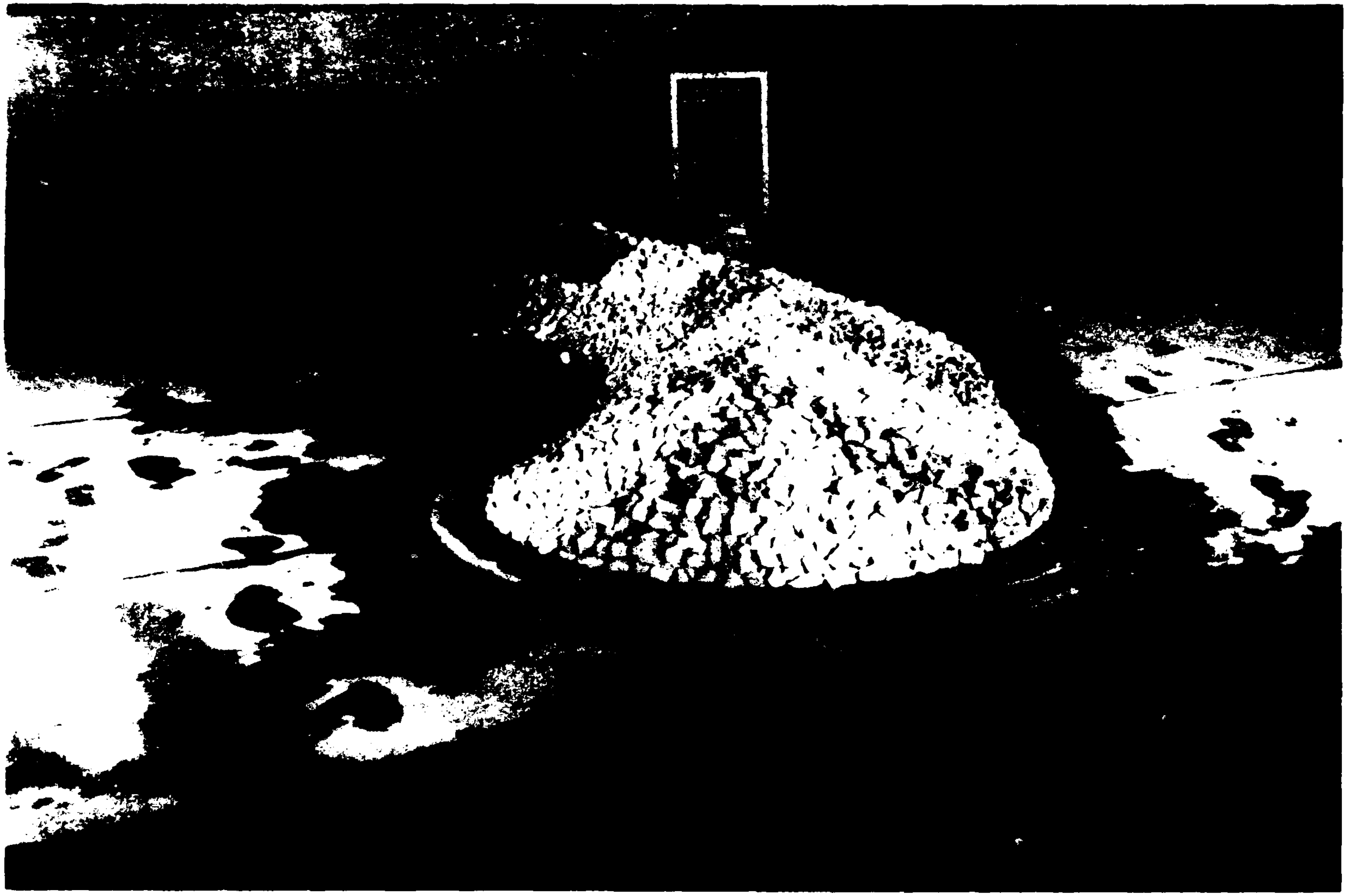


Photo 18. End view of Plan S2 after testing Hydrograph A, wave direction 2

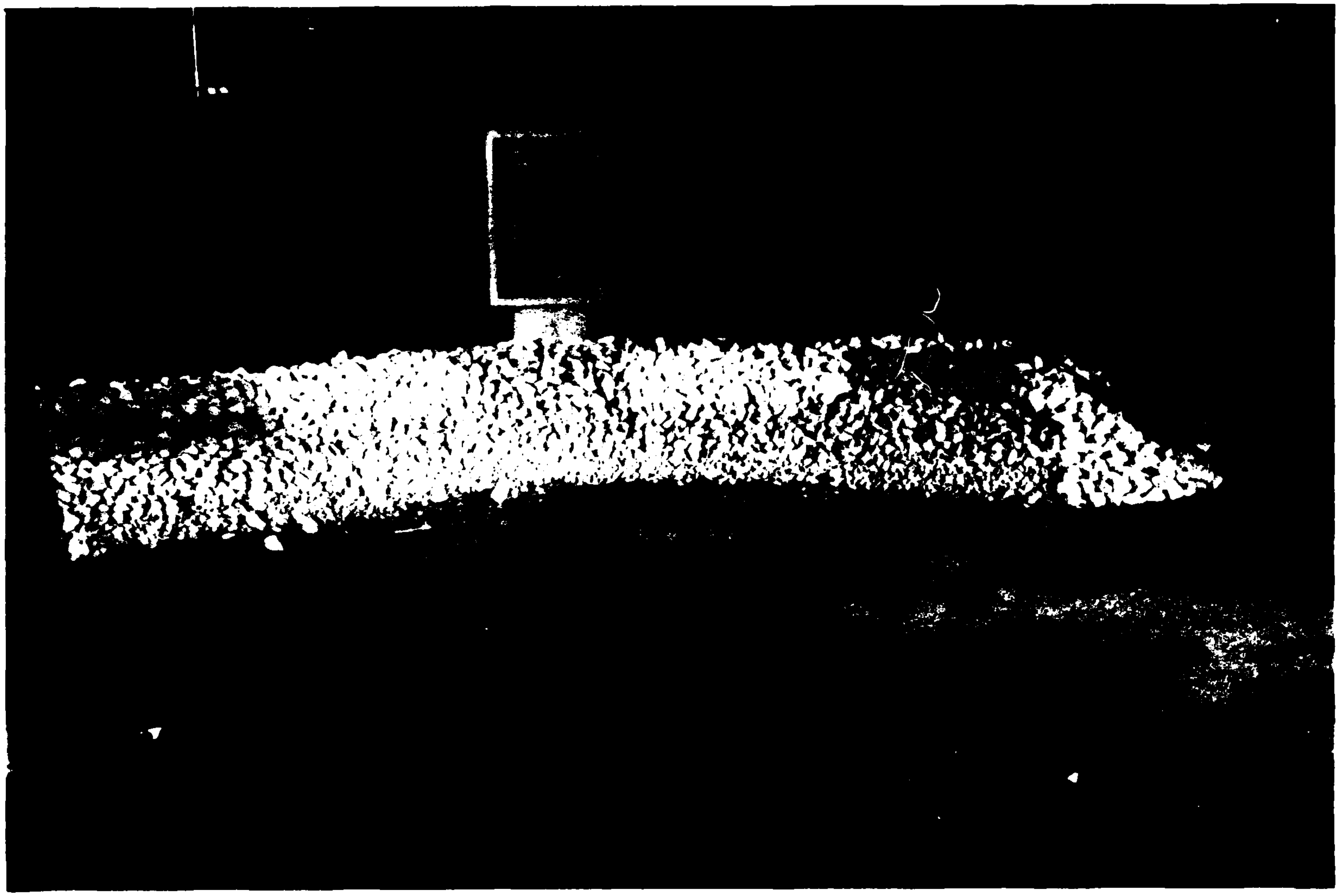


Photo 19. Channel-side view of Plan S2 after testing Hydrograph A, wave direction 2

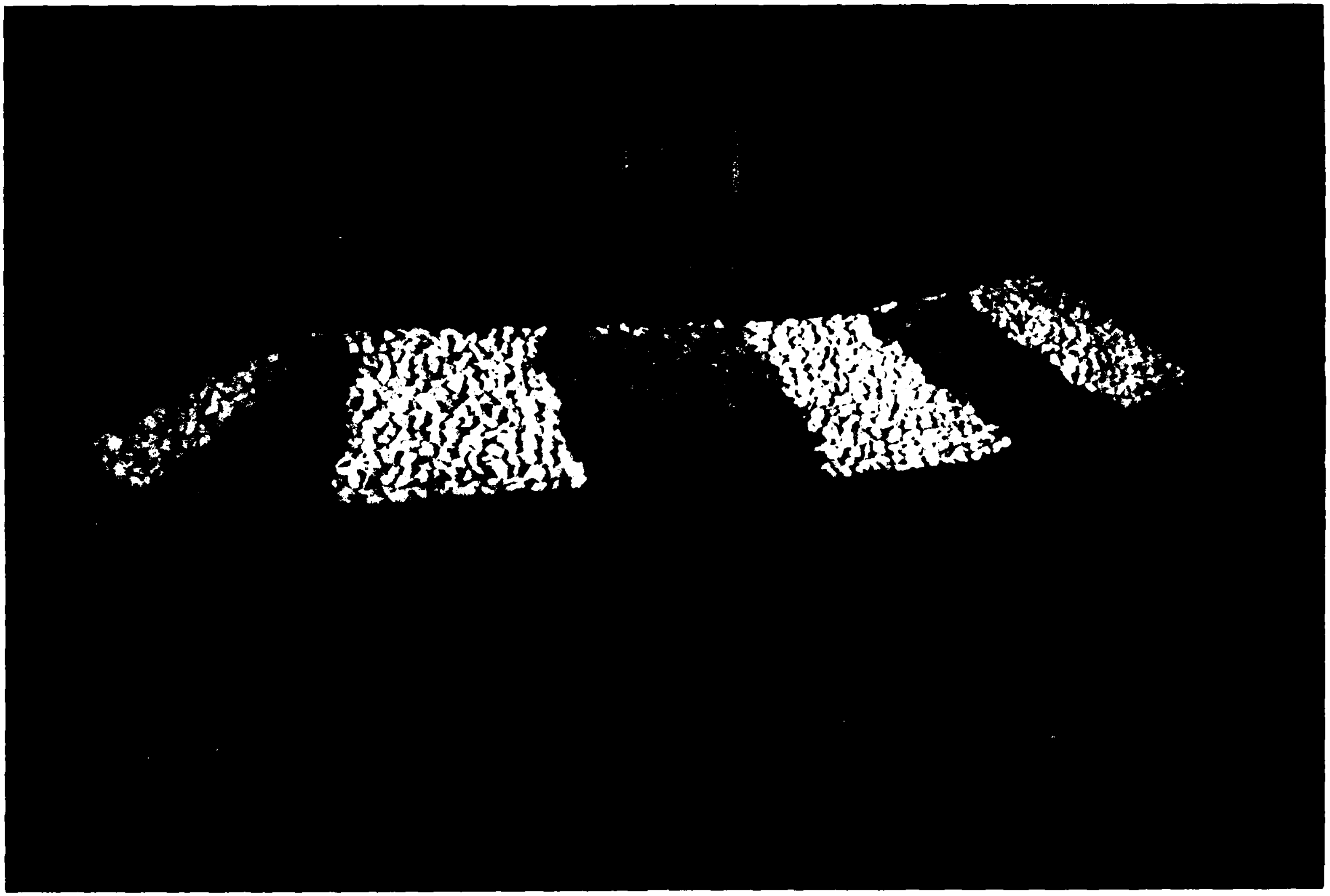


Photo 20. Ocean-side view of Plan S2 before testing Hydrograph A, Wave direction 1

Photo 21. End view of Plan S2 before testing Hydrograph A, wave direction 1

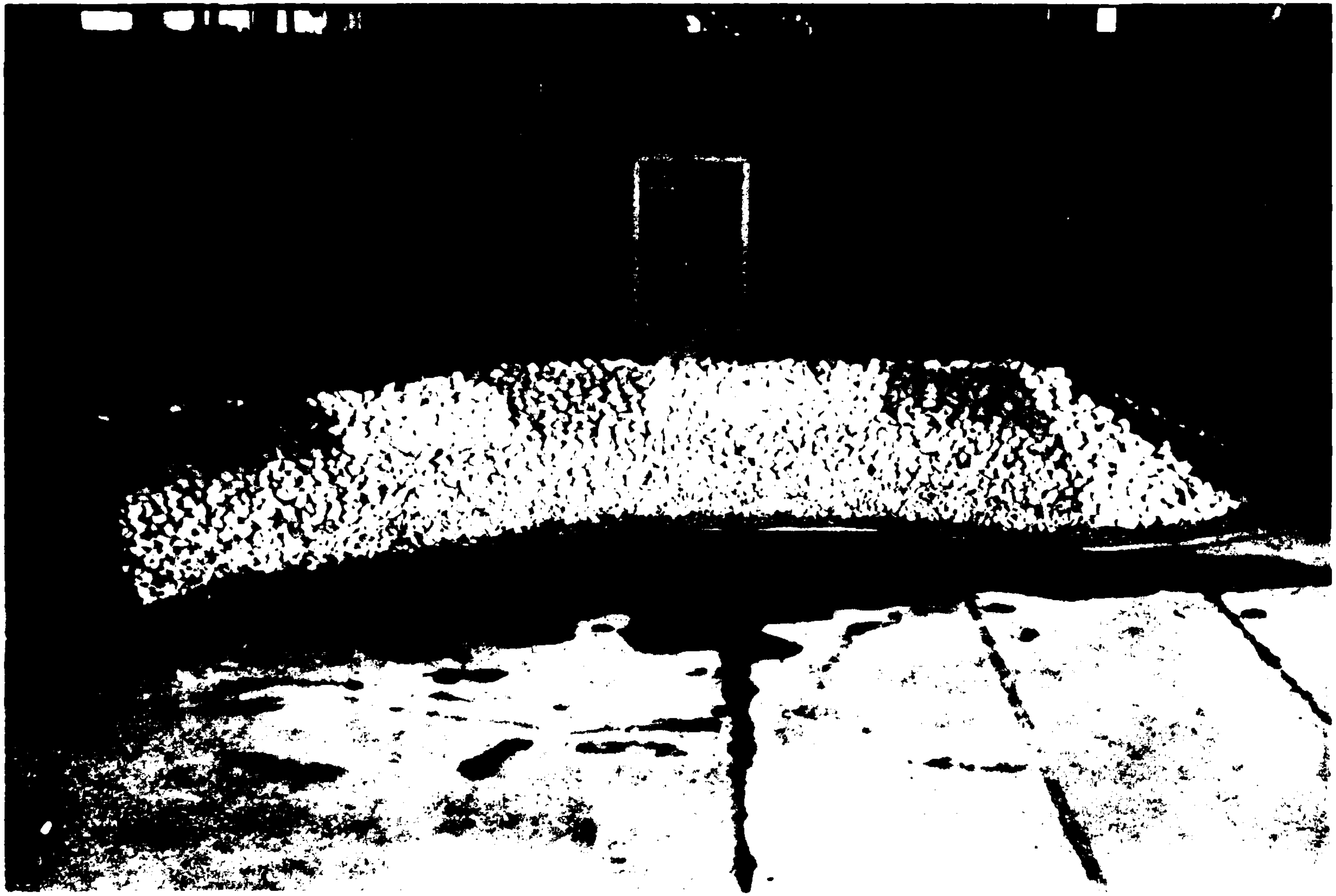


Photo 22. Channel-side view of Plan S2 before testing Hydrograph A, wave direction 1

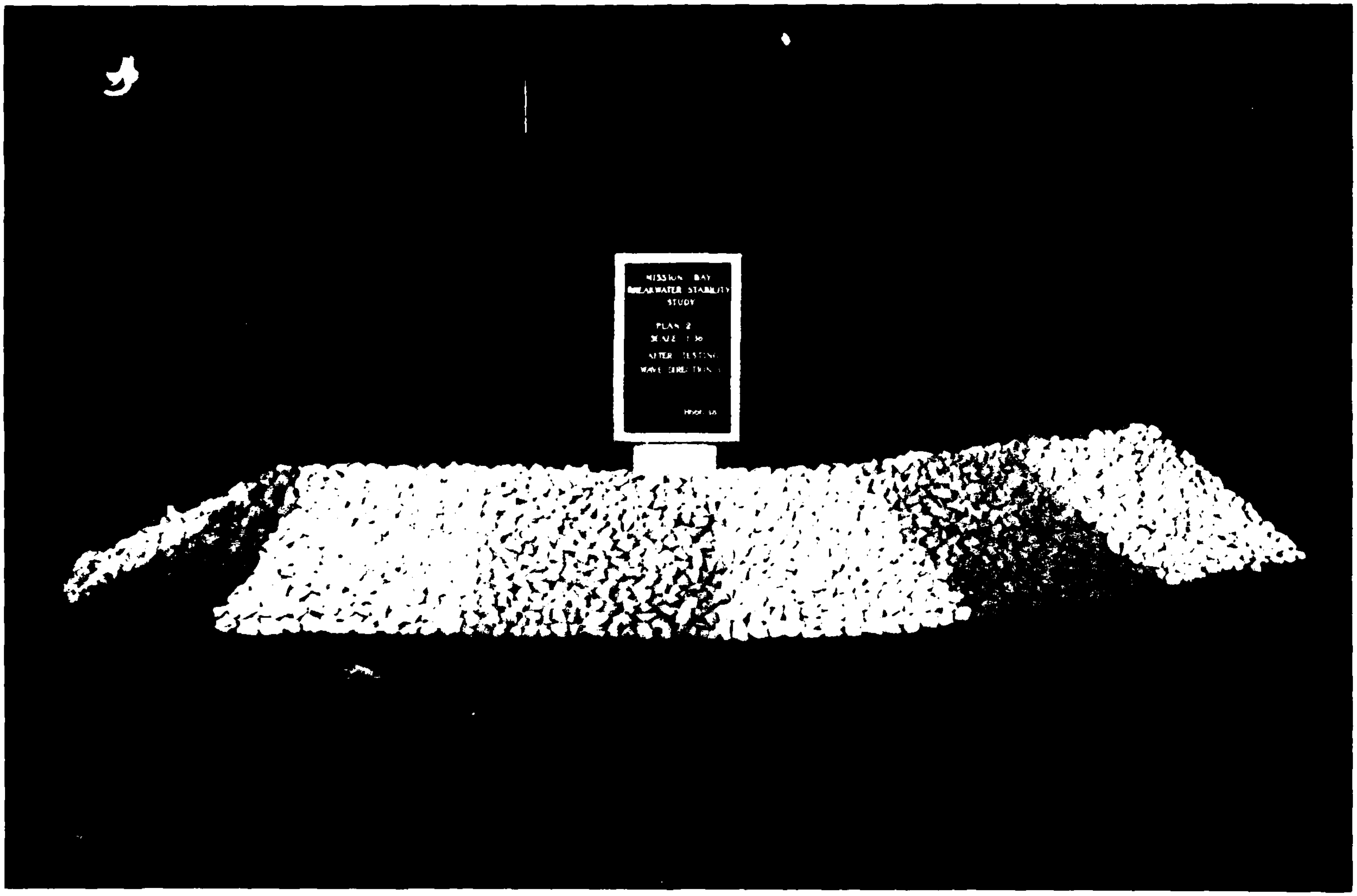


Photo 23. Ocean-side view of Plan S2 after testing Hydrograph A, wave direction 1

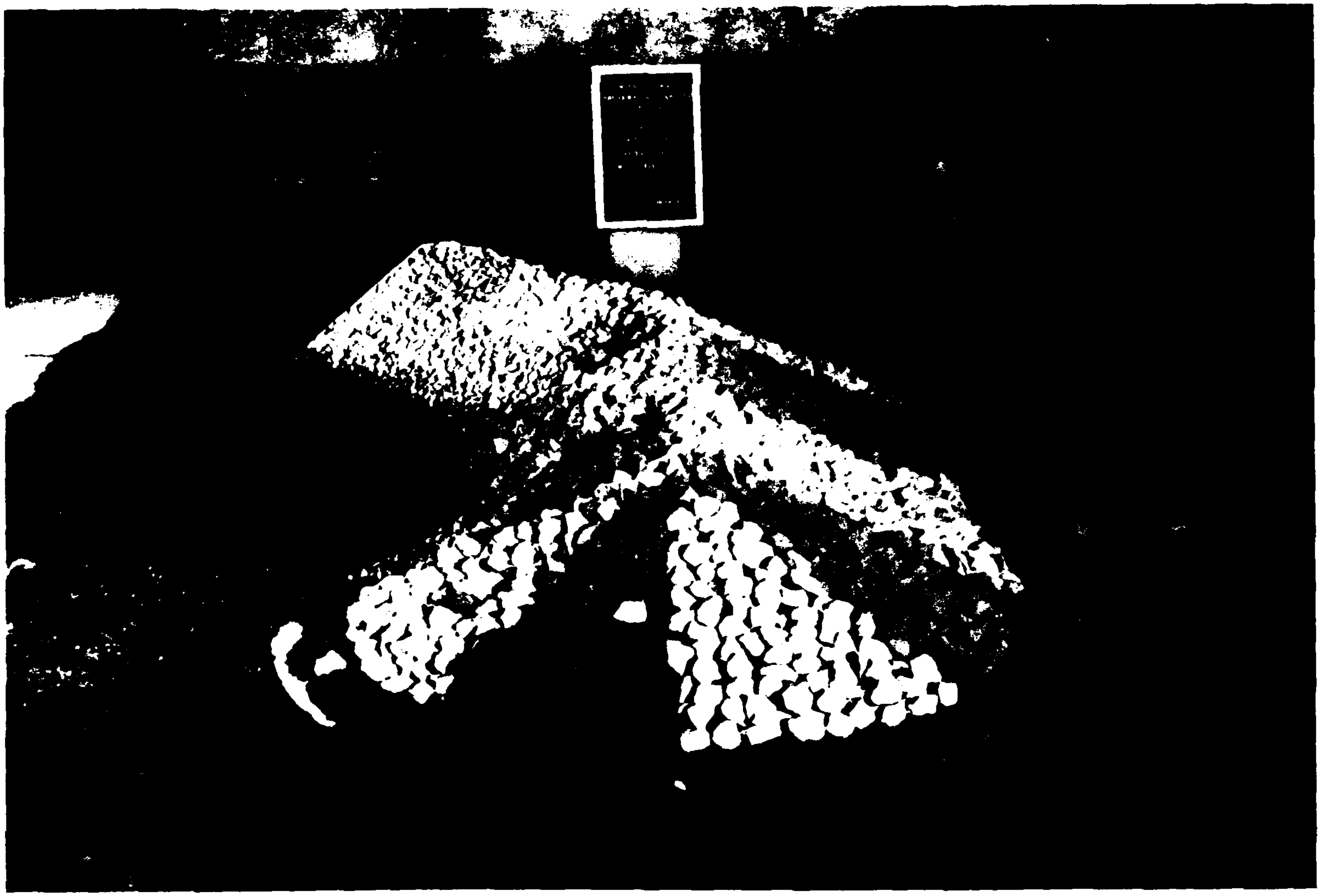


Photo 24. End view of Plan S2 after testing Hydrograph A, wave direction 1

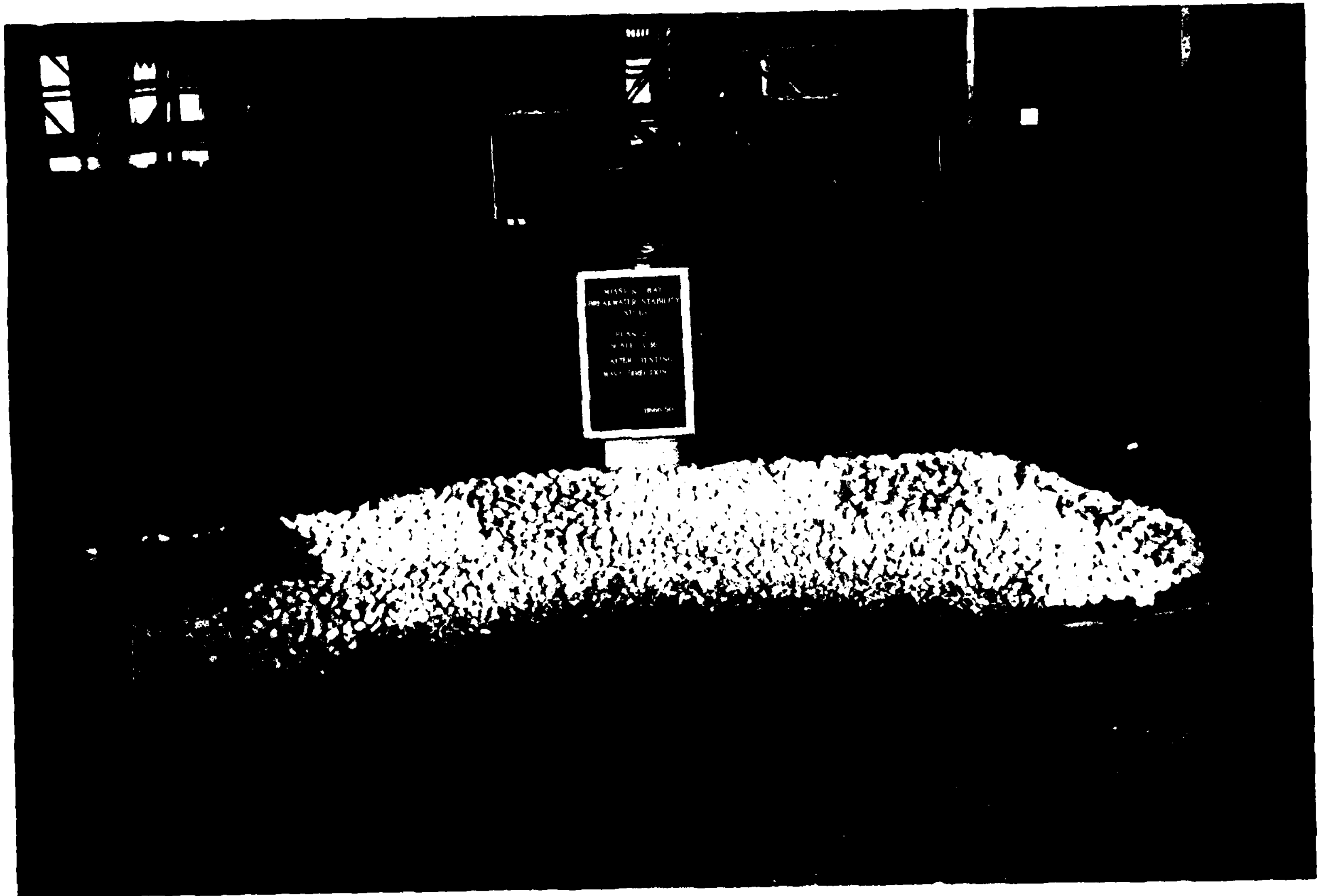


Photo 25. Channel-side view of Plan S2 after testing Hydrograph A, wave direction 1

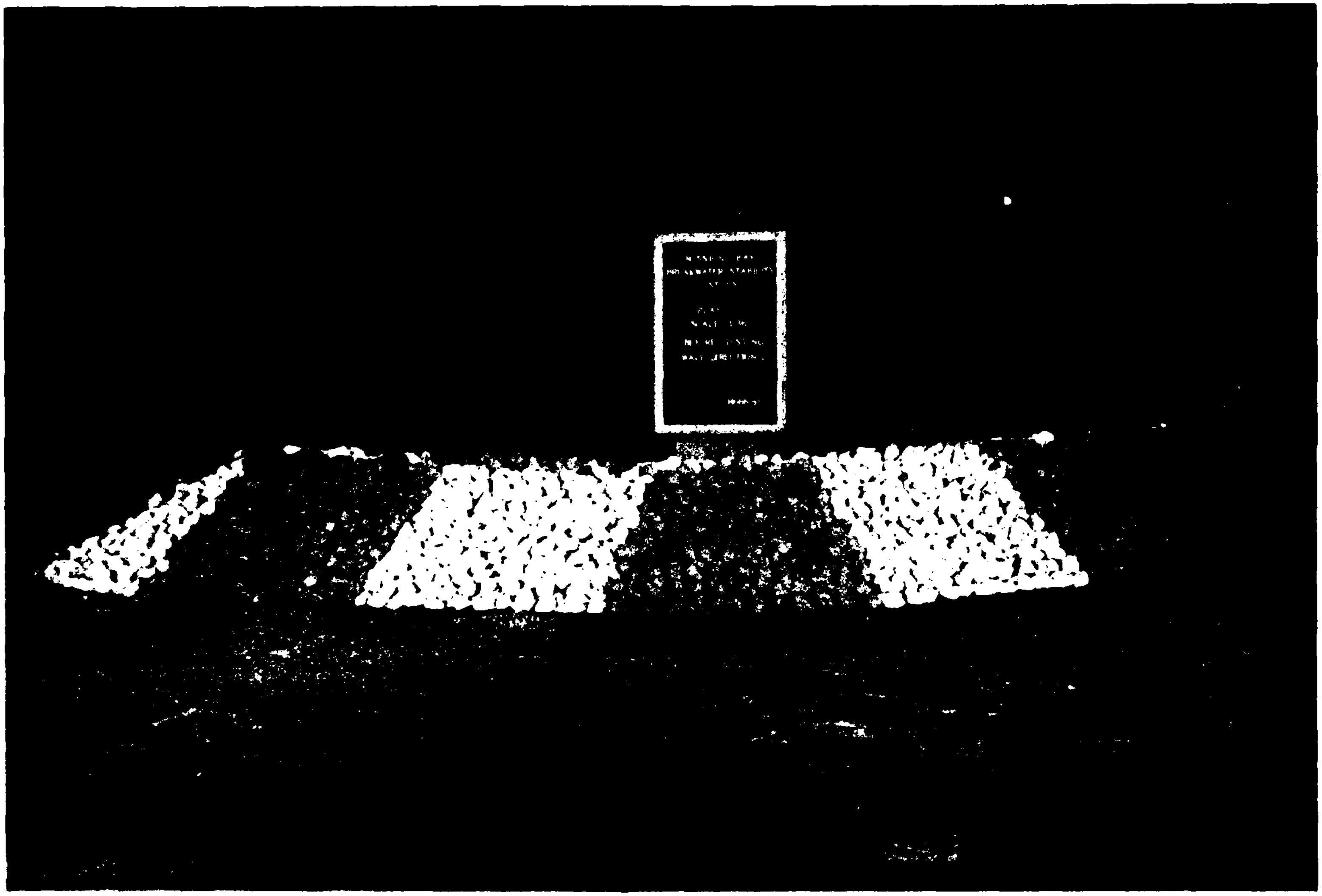


Photo 26. Ocean-side view of Plan S1 before testing Hydrograph B, wave direction 2

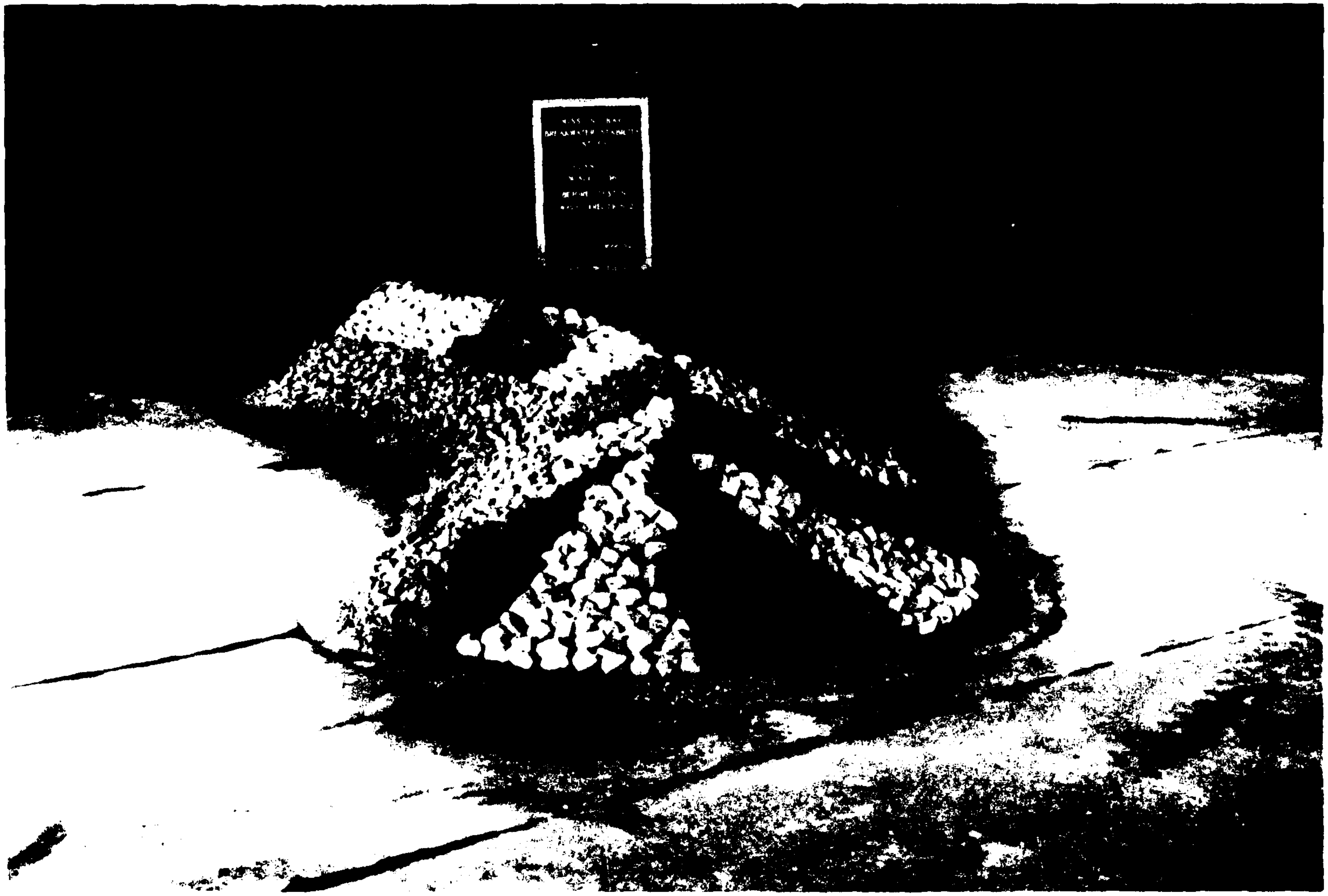


Photo 27. End view of Plan S1 before testing Hydrograph B, wave direction 2

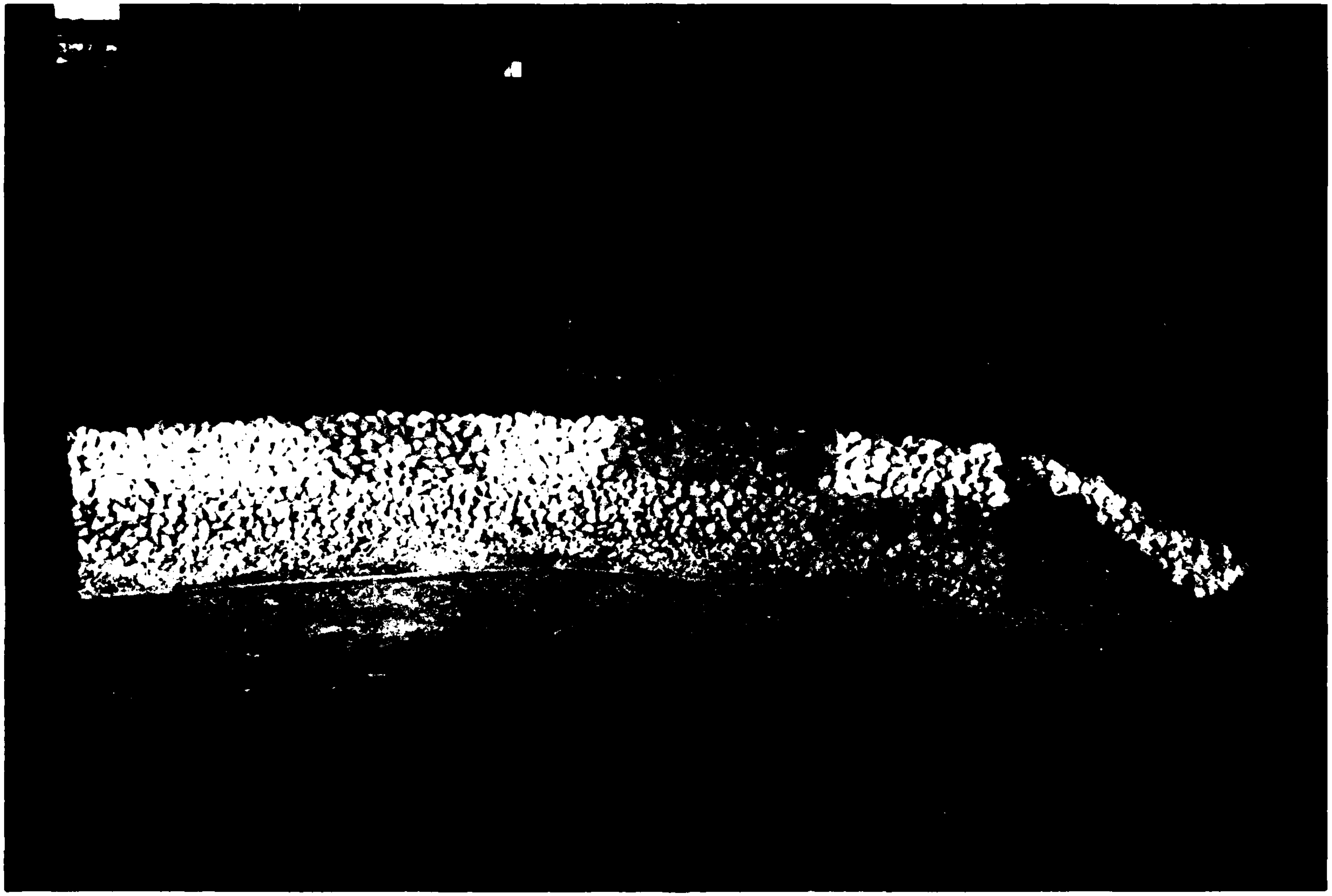


Photo 28. Channel-side view of Plan S1 before testing Hydrograph B, wave direction 2

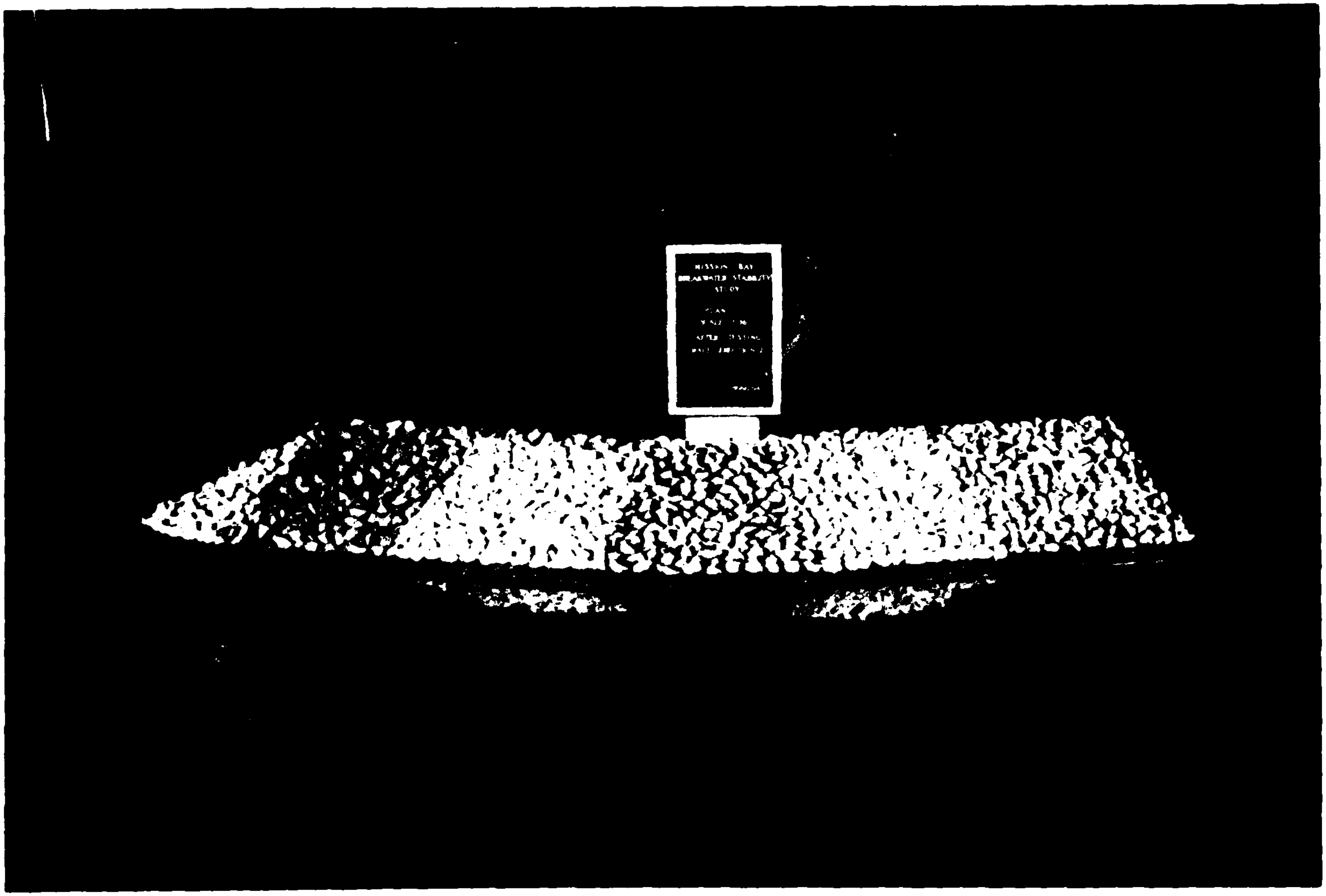


Photo 29. Ocean-side view of Plan S1 after testing Hydrograph B, wave direction 2

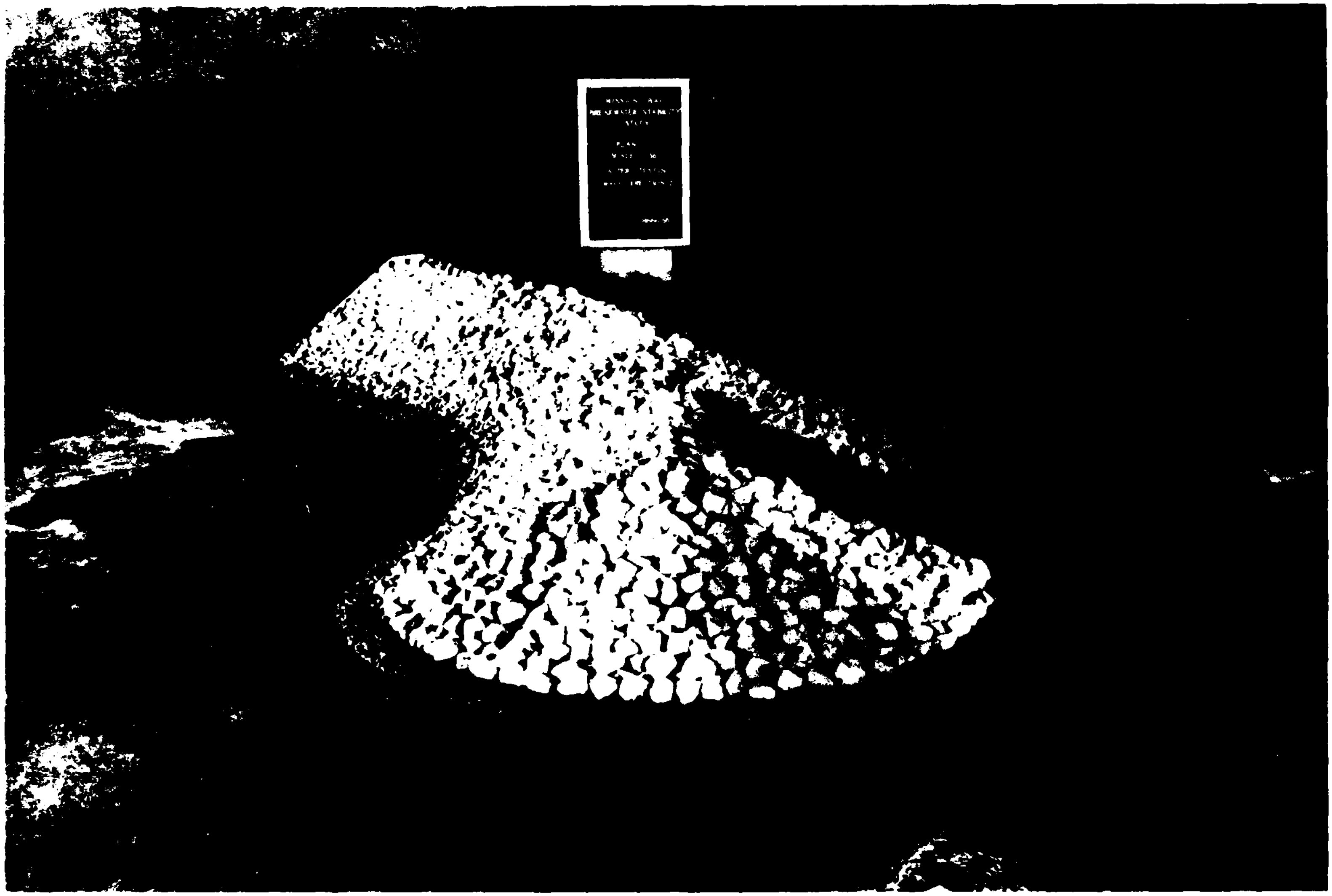


Photo 30. End view of Plan S1 after testing Hydrograph B, wave direction 2

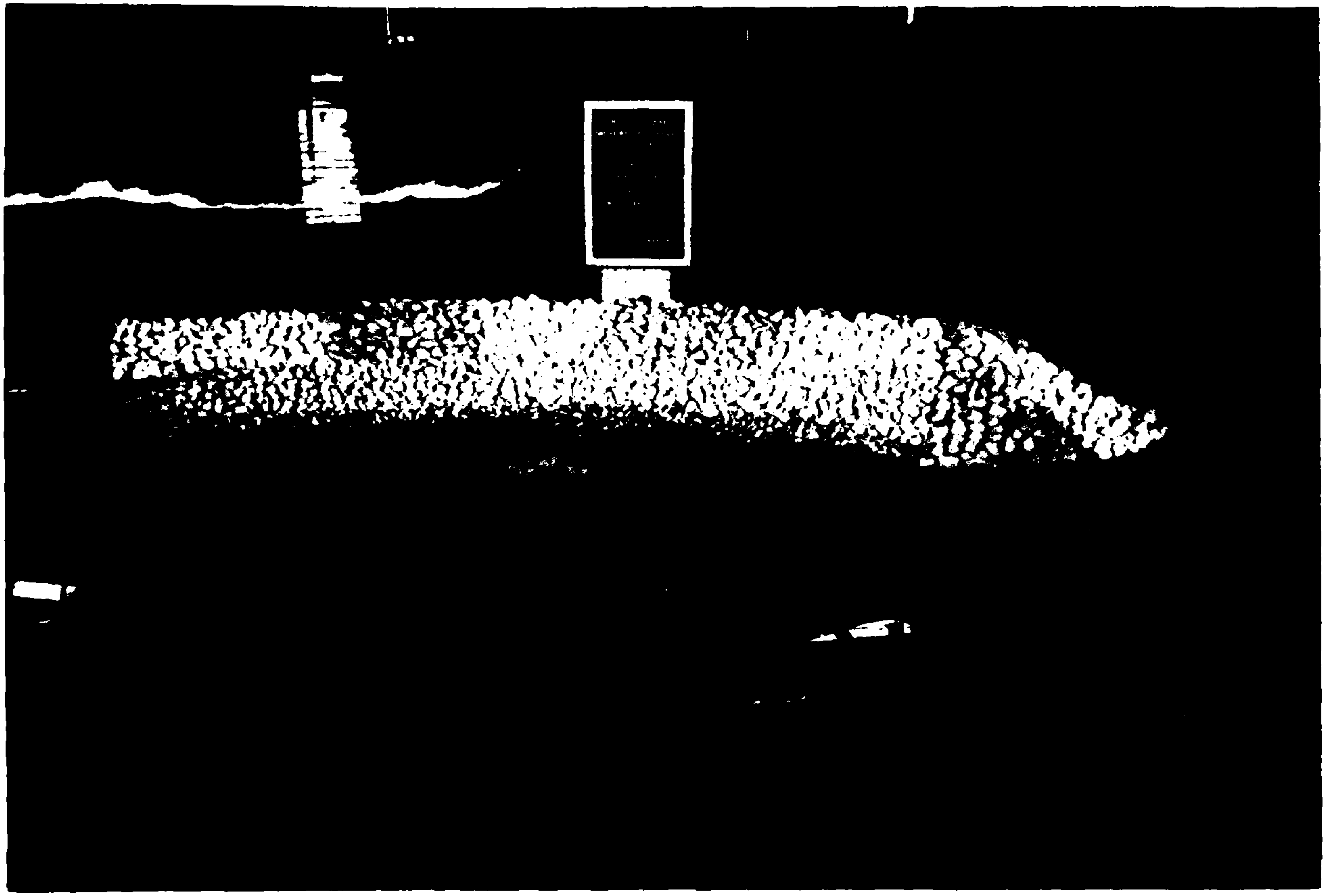


Photo 31. Channel-side view of Plan S1 after testing Hydrograph B, wave direction 2

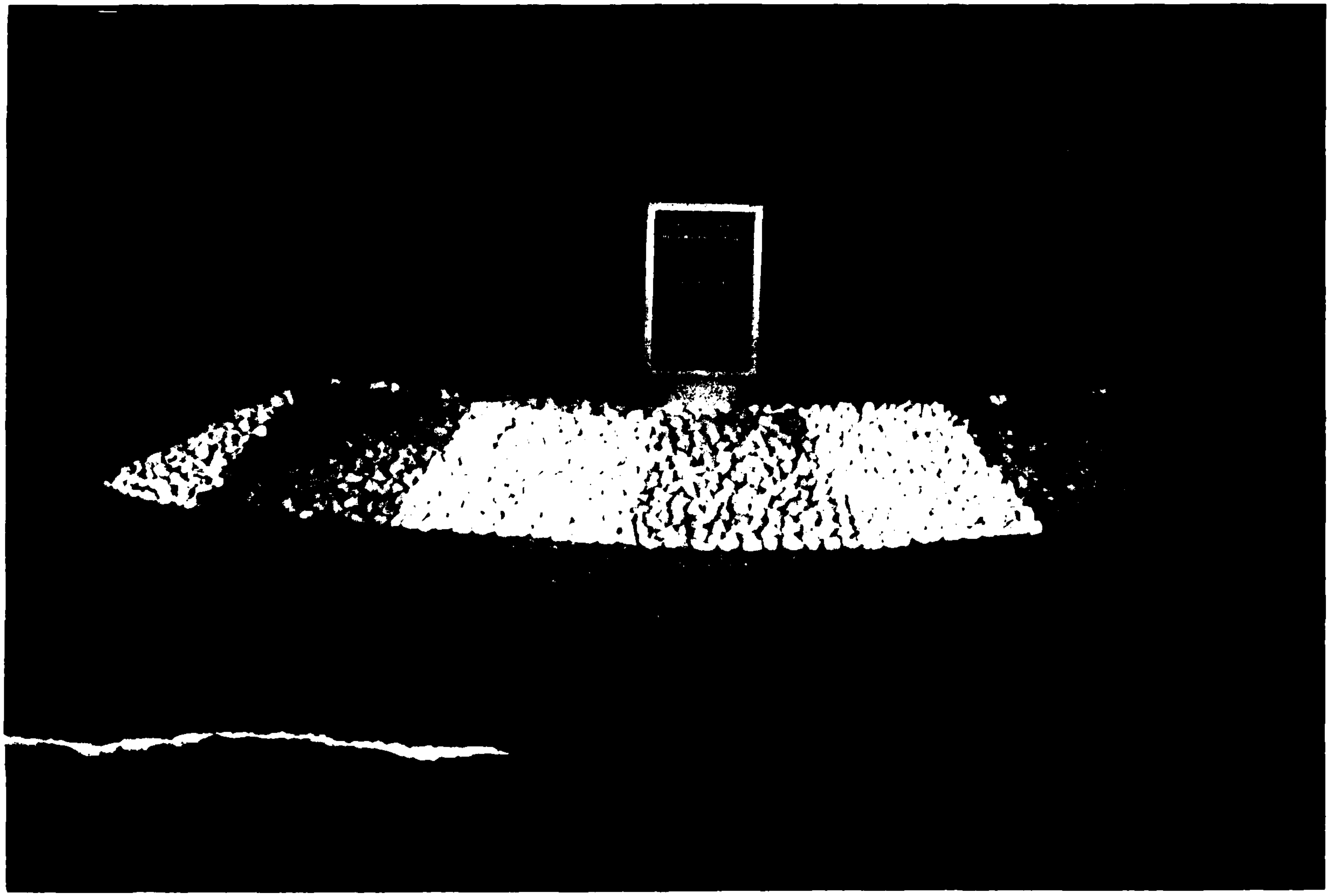


Photo 32. Ocean-side view of Plan S1 after testing Hydrographs B and C, wave direction 2

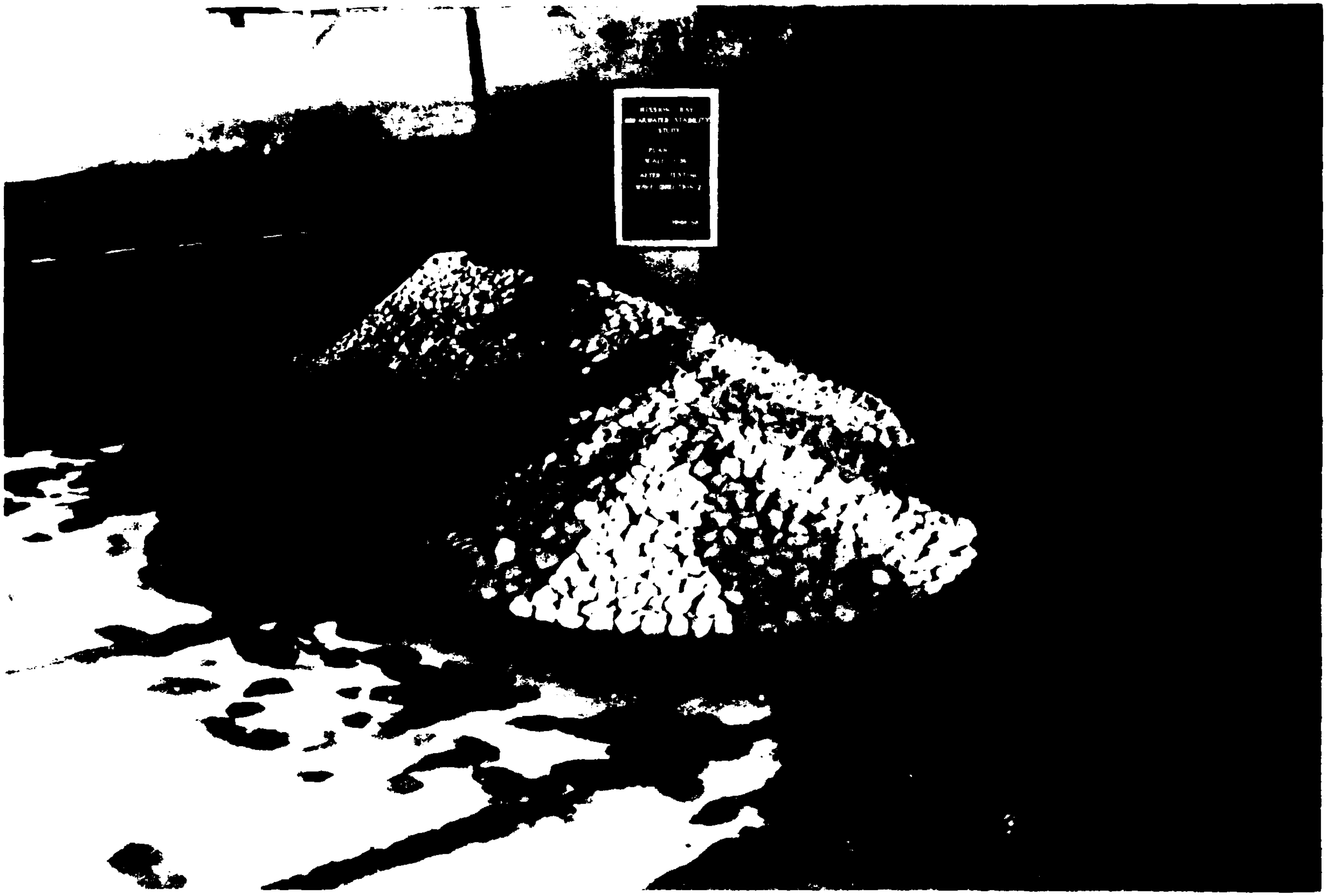


Photo 33. End view of Plan S1 after testing Hydrographs B and C, wave direction 2

Photo 34. Channel-side view of Plan S1 after testing Hydrographs B and C, wave direction 2

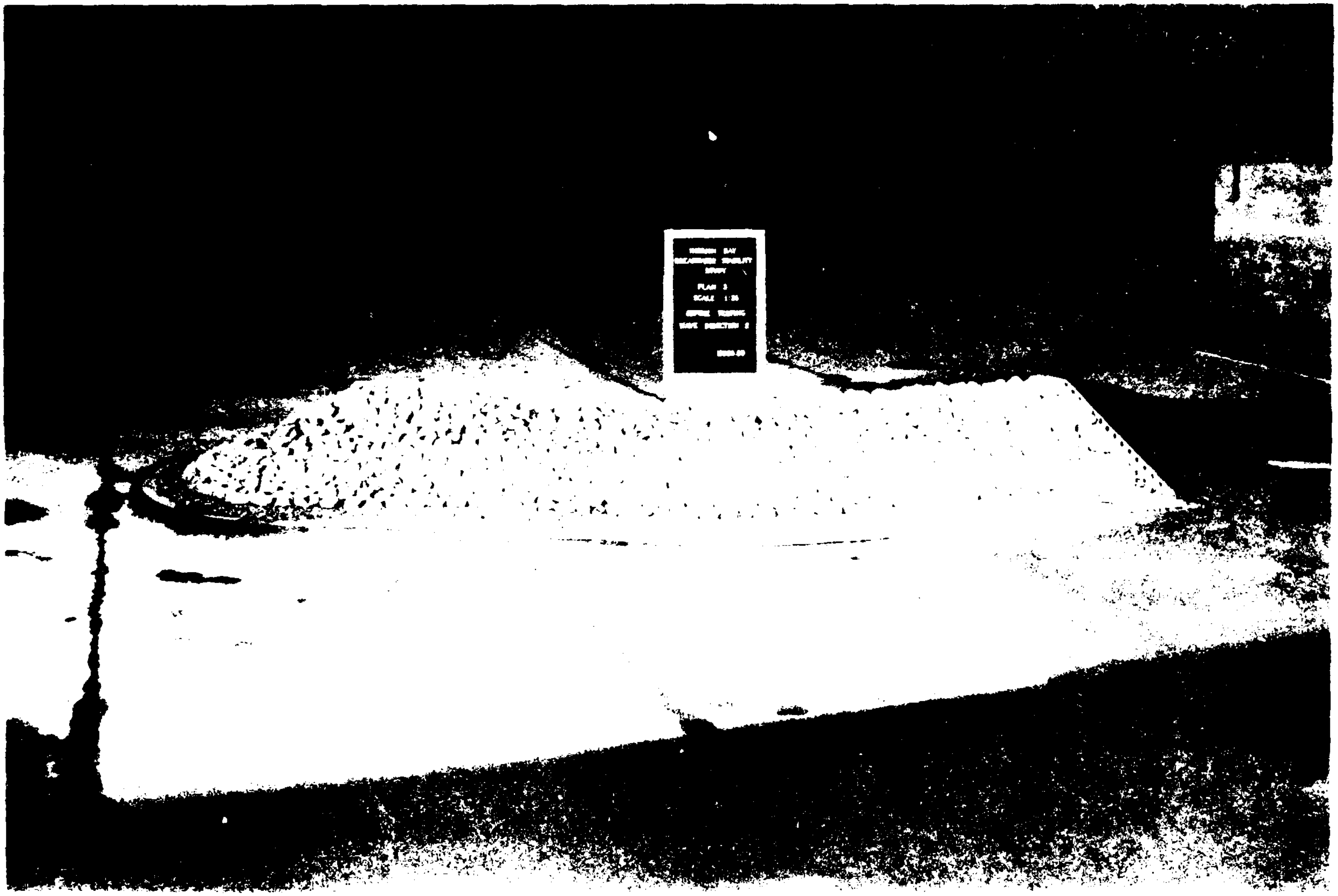


Photo 35. Ocean-side view of Plan S3 before testing Hydrograph A, wave direction 2

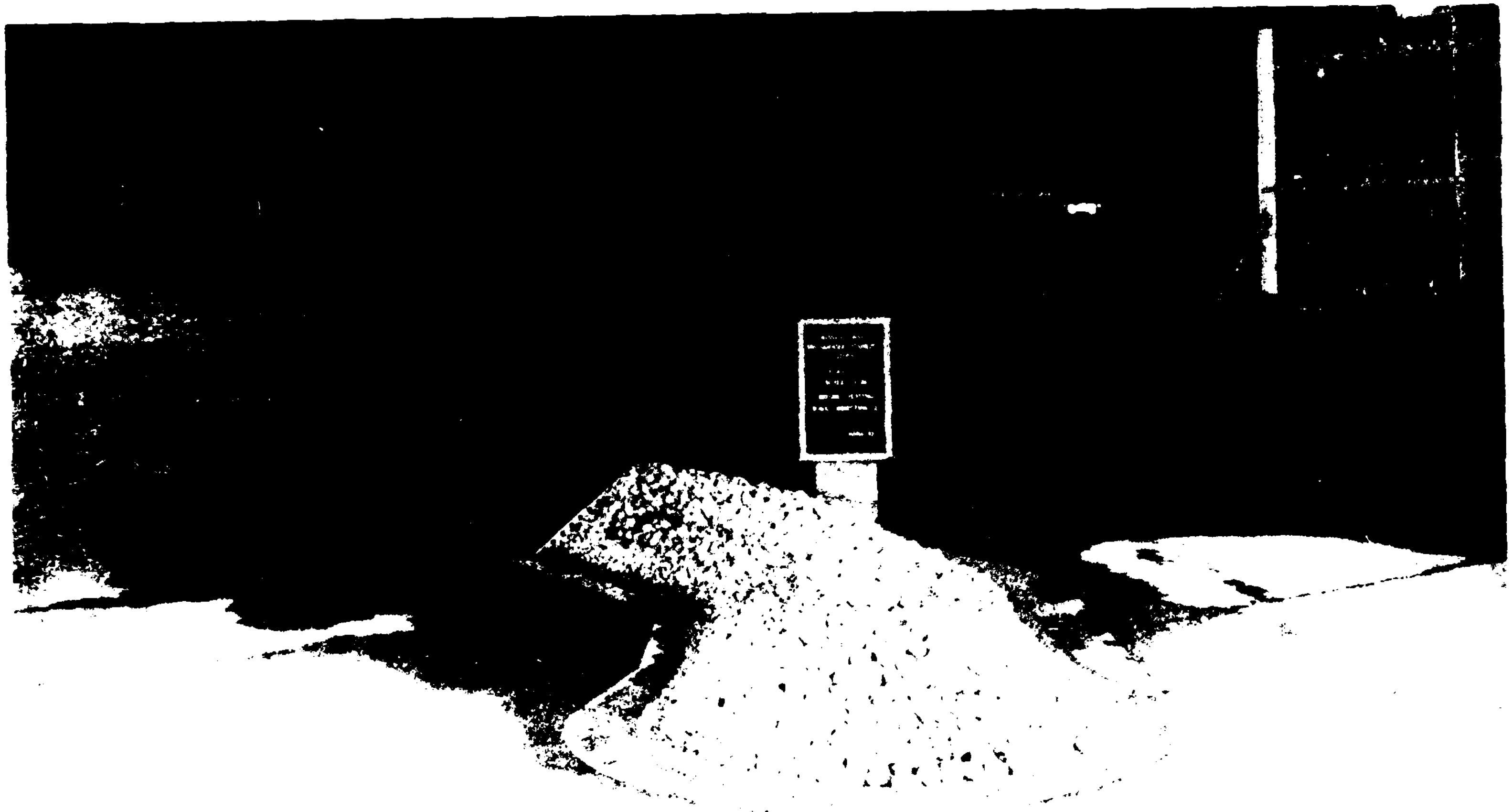


Photo 36. End view of Plan S3 before testing Hydrograph A, wave direction 2

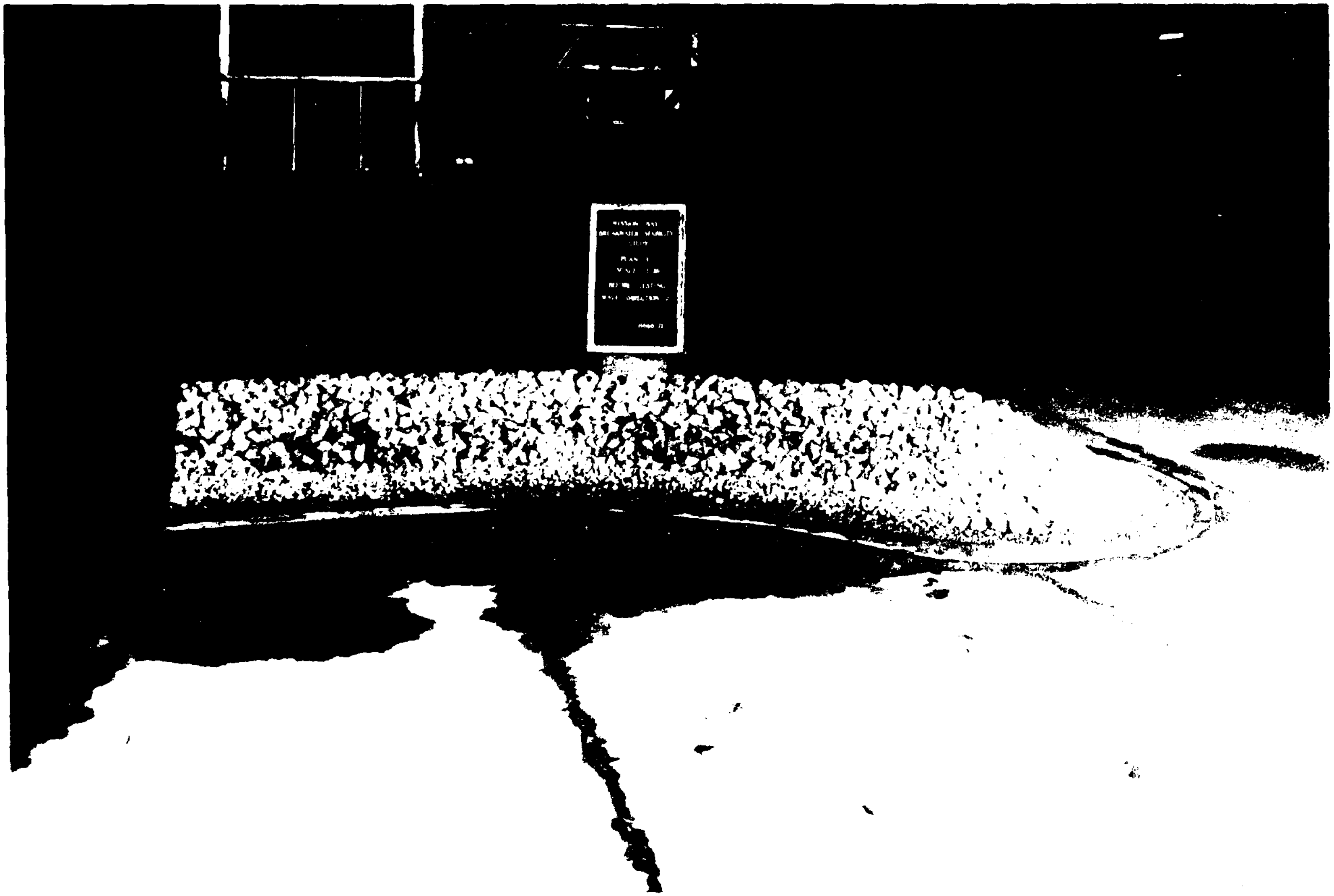


Photo 37. Channel-side view of Plan S3 before testing Hydrograph A, wave direction 2

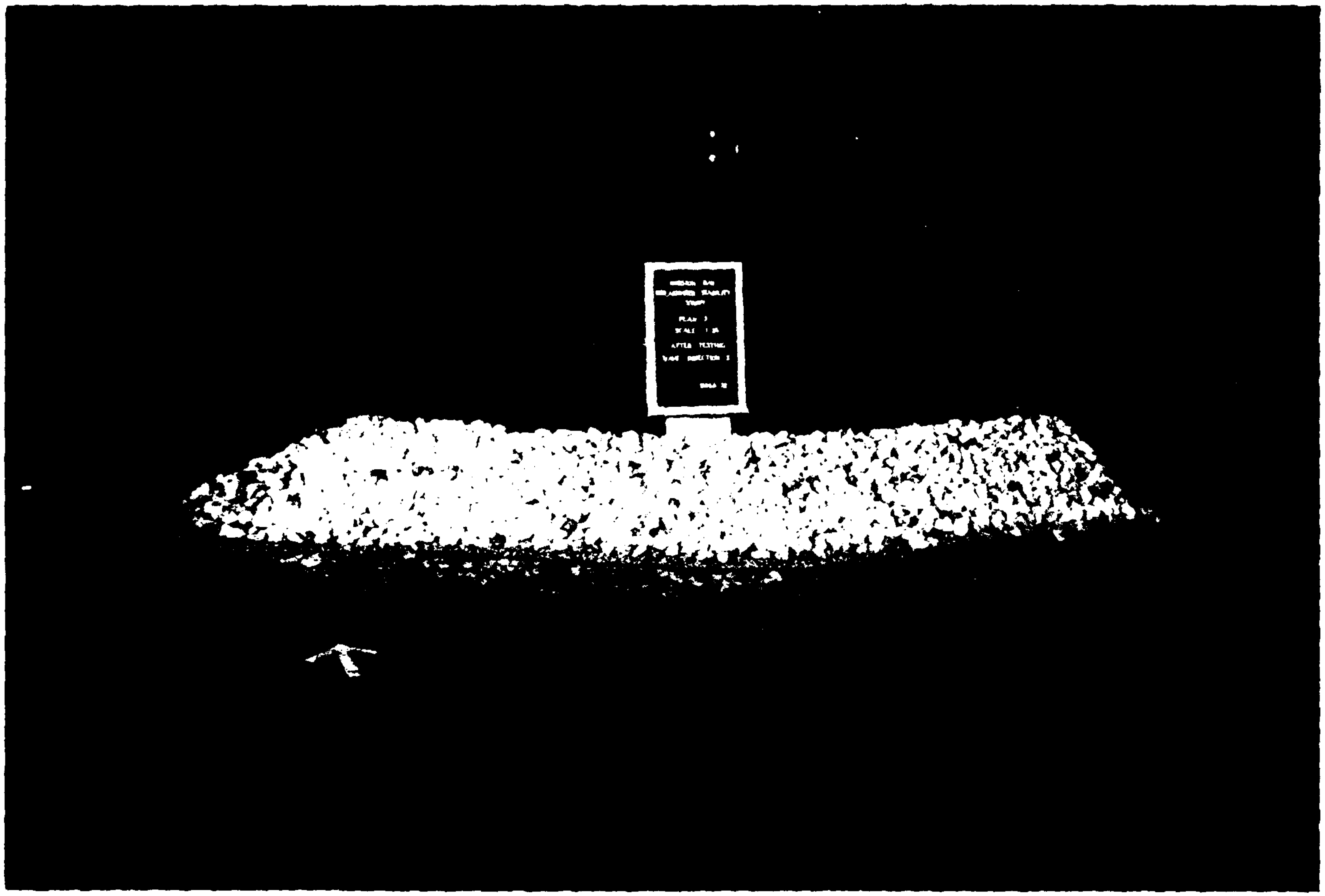


Photo 38. Ocean-side view of Plan S3 after testing *Hydrograph A*, wave direction 2

Photo 39. End view of Plan S3 after testing Hydrograph A, wave direction 2

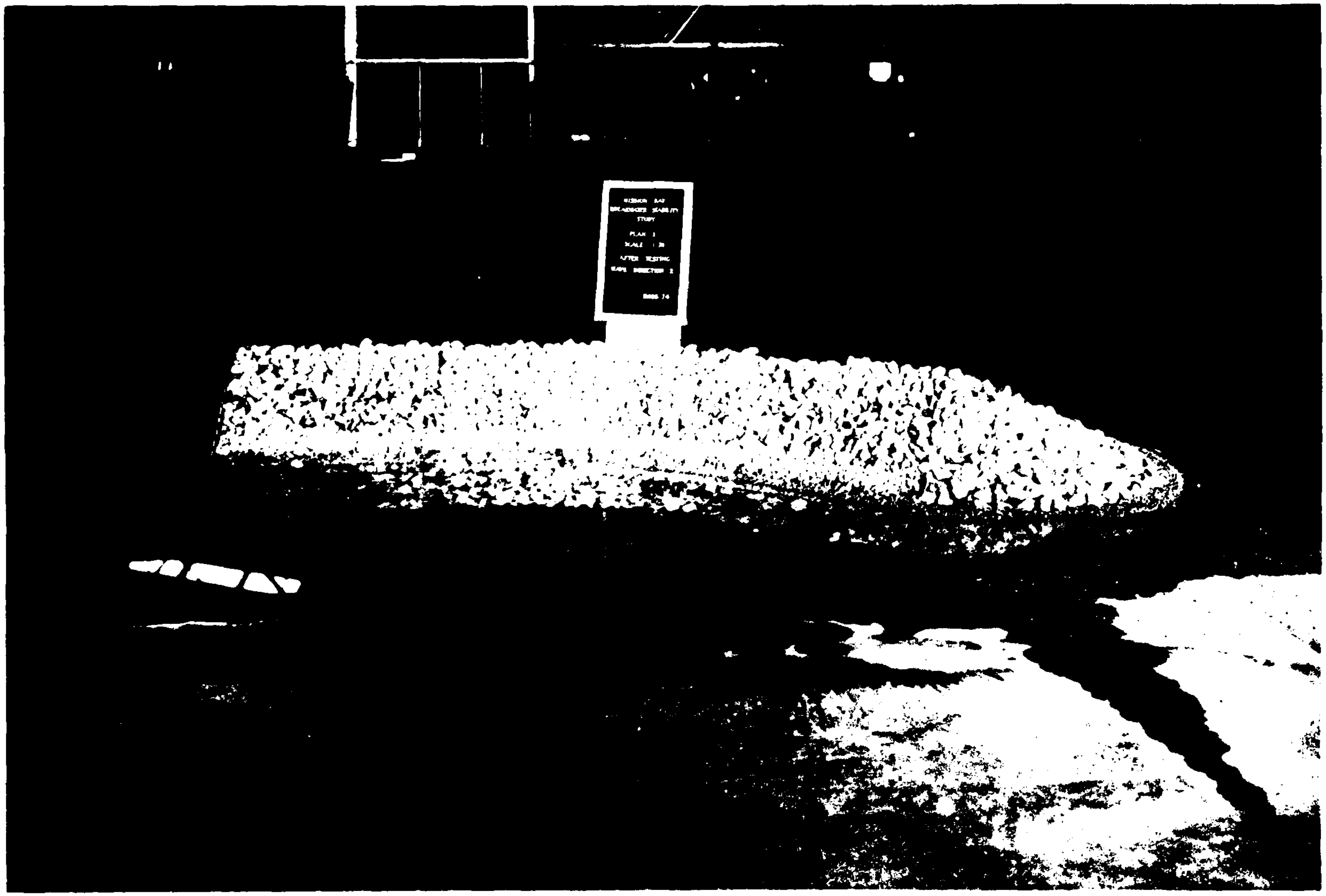


Photo 40. Channel-side view of Plan S3 after testing Hydrograph A, wave direction 2

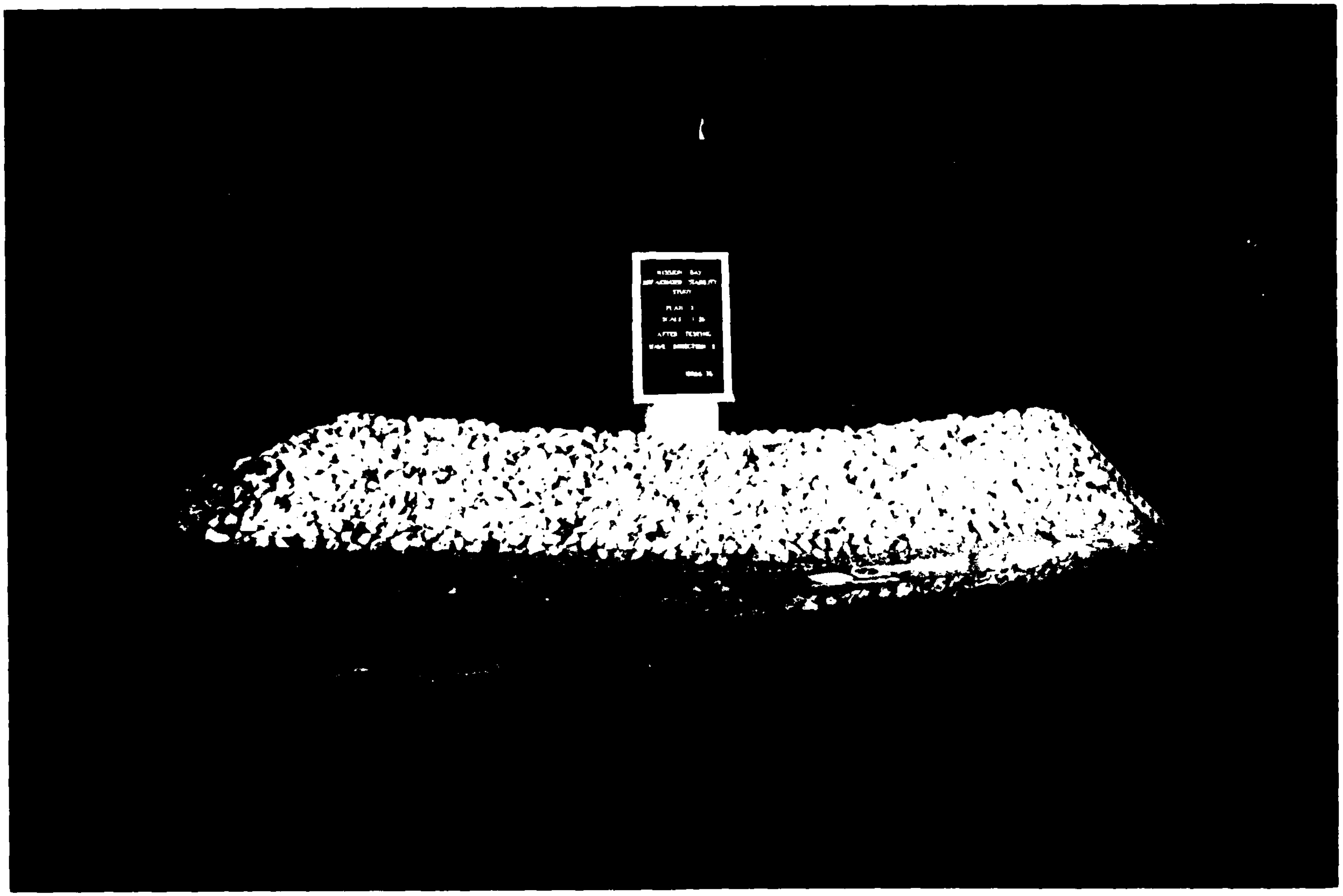


Photo 41. Ocean-side view of Plan S3 after testing Hydrographs A and C, wave direction 2

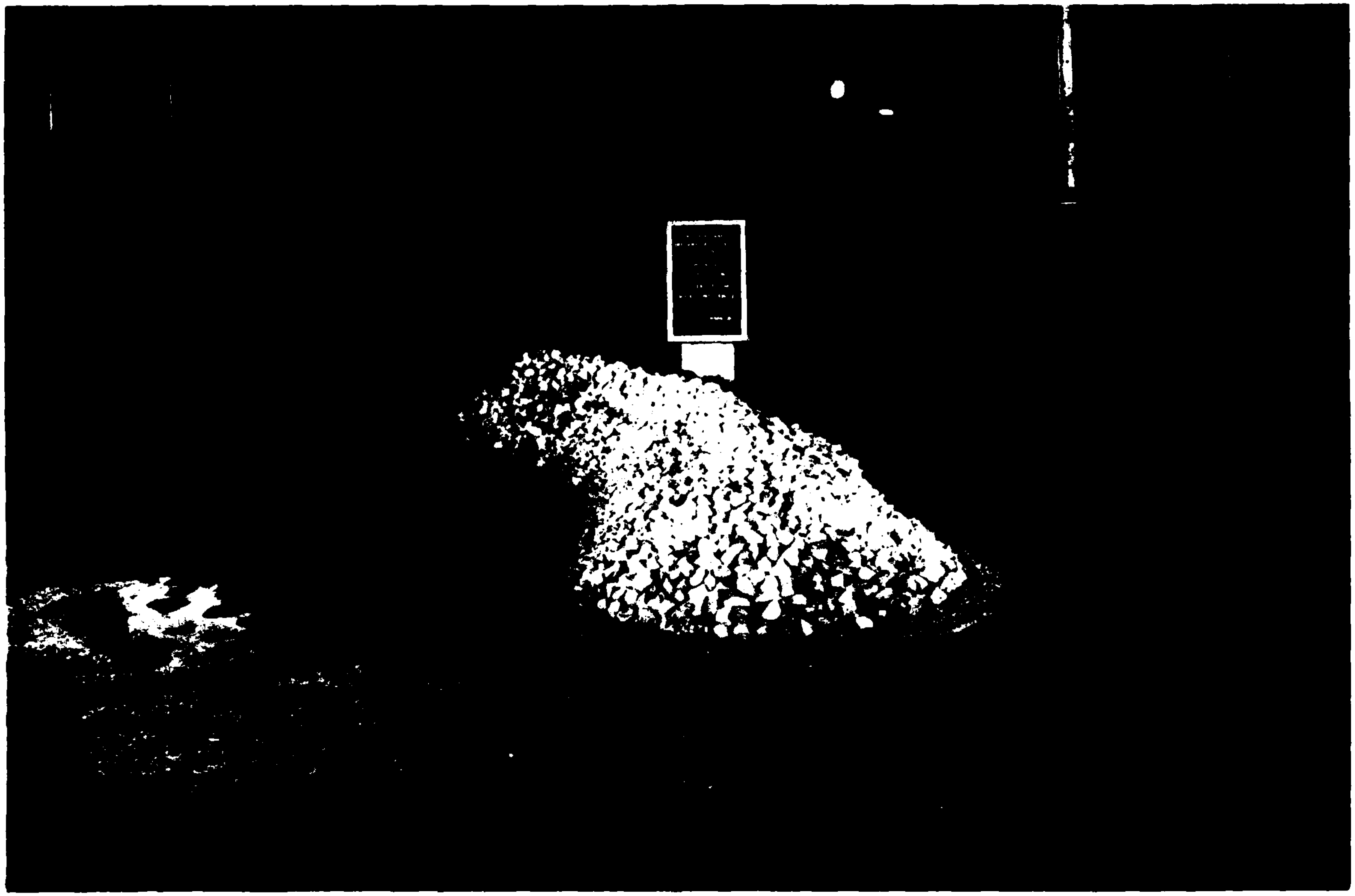


Photo 42. End view of Plan S3 after testing Hydrographs A and C, wave direction 2

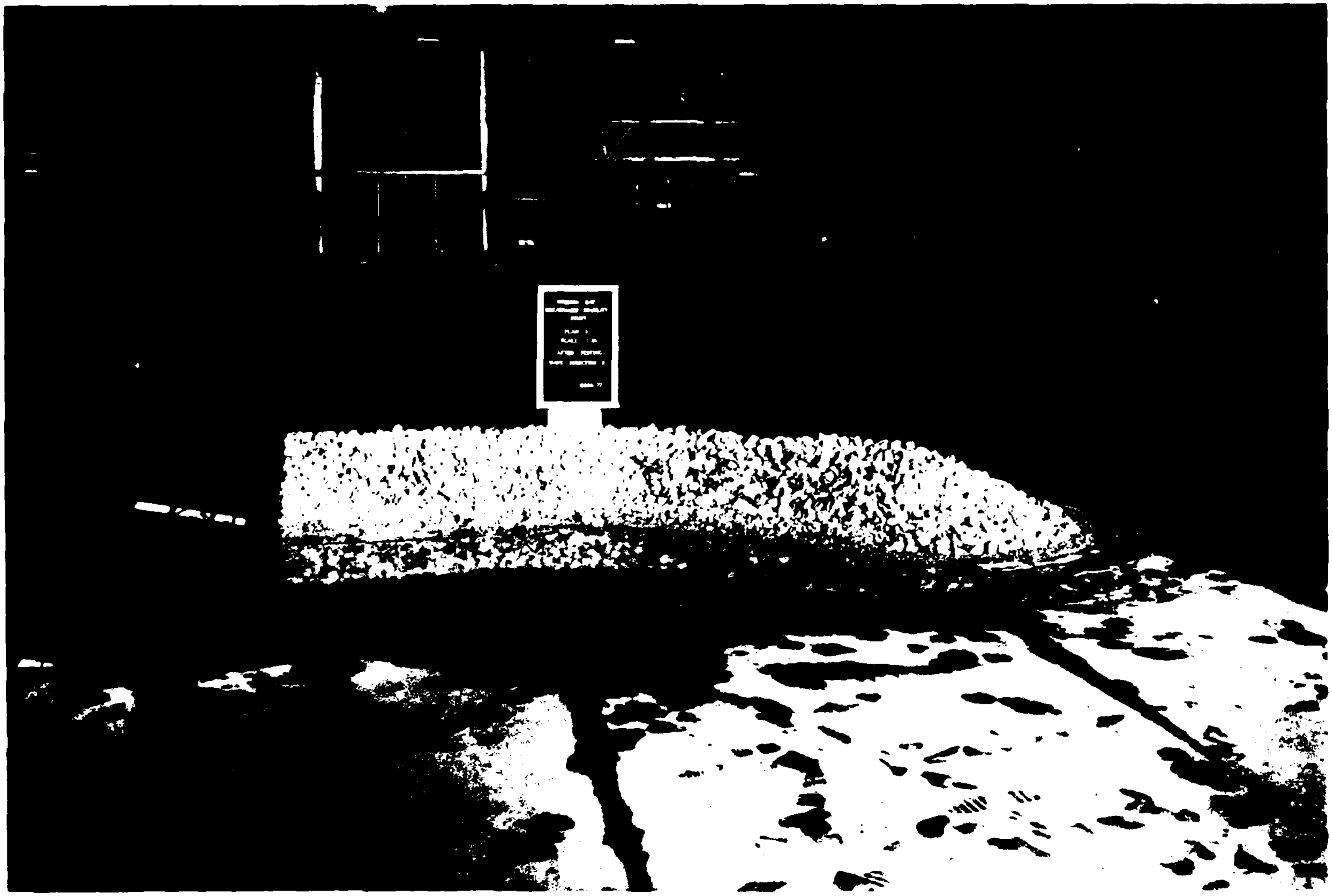
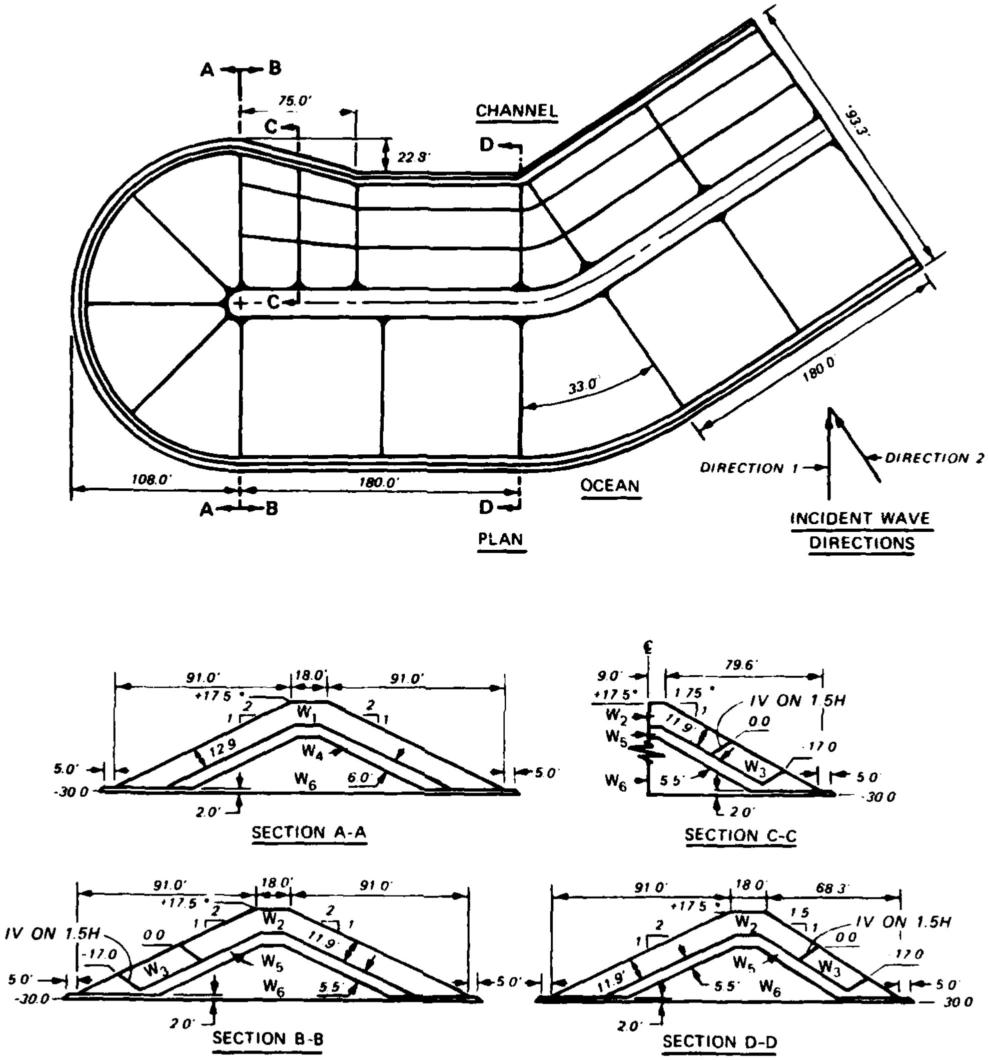


Photo 43. Channel-side view of Plan S3 after testing Hydrographs A and C, wave direction 2



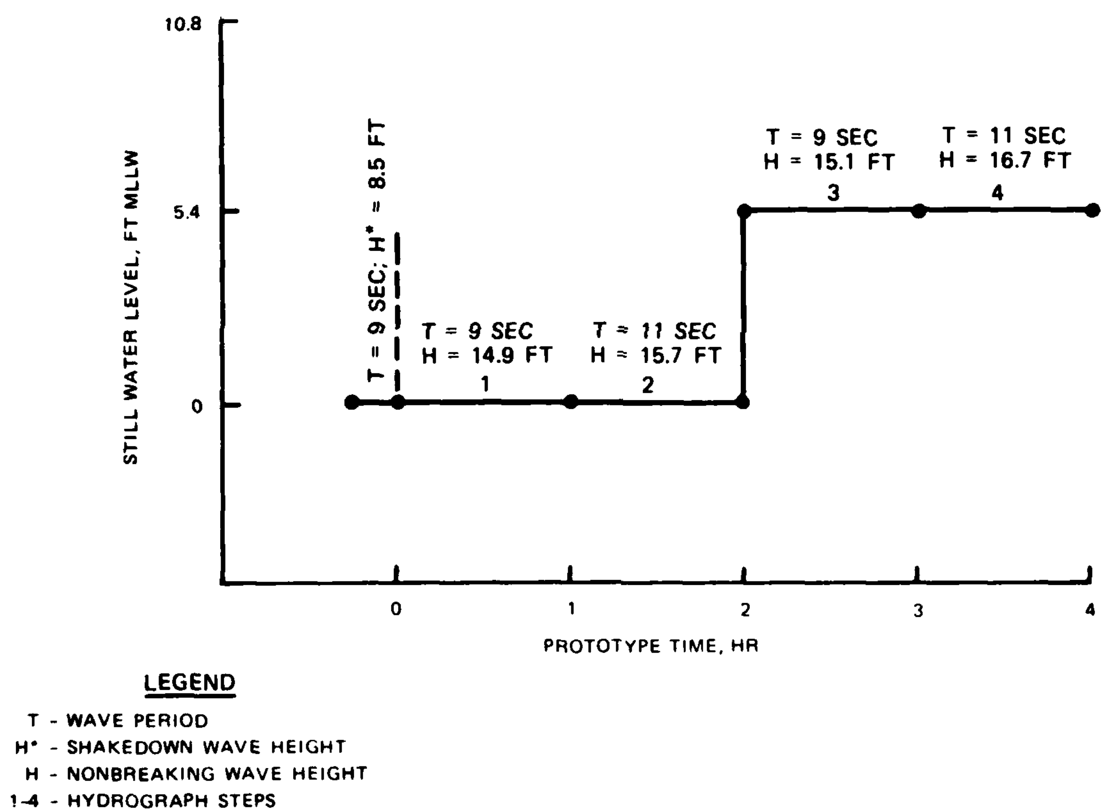
MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
** W_1 · 0.55-LB ROCK @ 165 PCF	W_1 · 29,022-LB ROCK @ 165 PCF
** W_2 · 0.43-LB ROCK @ 165 PCF	W_2 22,690-LB ROCK @ 165 PCF
** W_3 · 0.21-LB ROCK @ 165 PCF	W_3 · 11,081-LB ROCK @ 165 PCF
W_4 · 0.055-LB ROCK @ 165 PCF	W_4 2,900-LB ROCK @ 165 PCF
W_5 · 0.043-LB ROCK @ 165 PCF	W_5 · 2,270-LB ROCK @ 165 PCF
$W_6 \leq$ 0.003-LB ROCK @ 165 PCF	W_6 · 145-LB TO 4-LB ROCK @ 165 PCF

* ELEVATIONS IN FEET REFERRED TO MEAN LOWER LOW WATER
** RANDOM-PLACED ARMOR STONE

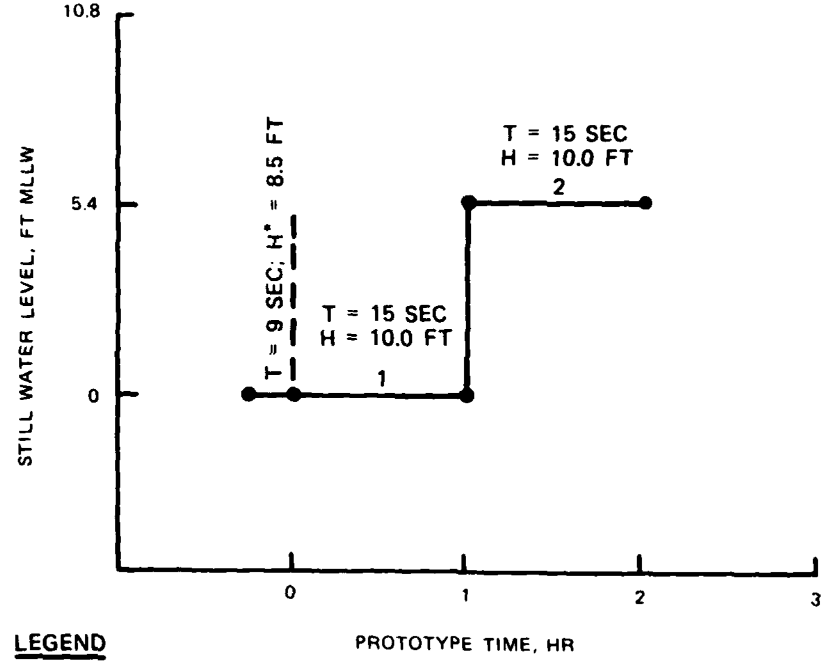
PLAN S1

PLATE 1



HYDROGRAPH A
WAVE DIRECTIONS 1 AND 2

PLATE 2



LEGEND

T - WAVE PERIOD
H* - SHAKEDOWN WAVE HEIGHT
H - NONBREAKING WAVE HEIGHT
1-2 - HYDROGRAPH STEPS

HYDROGRAPH B
WAVE DIRECTION 2

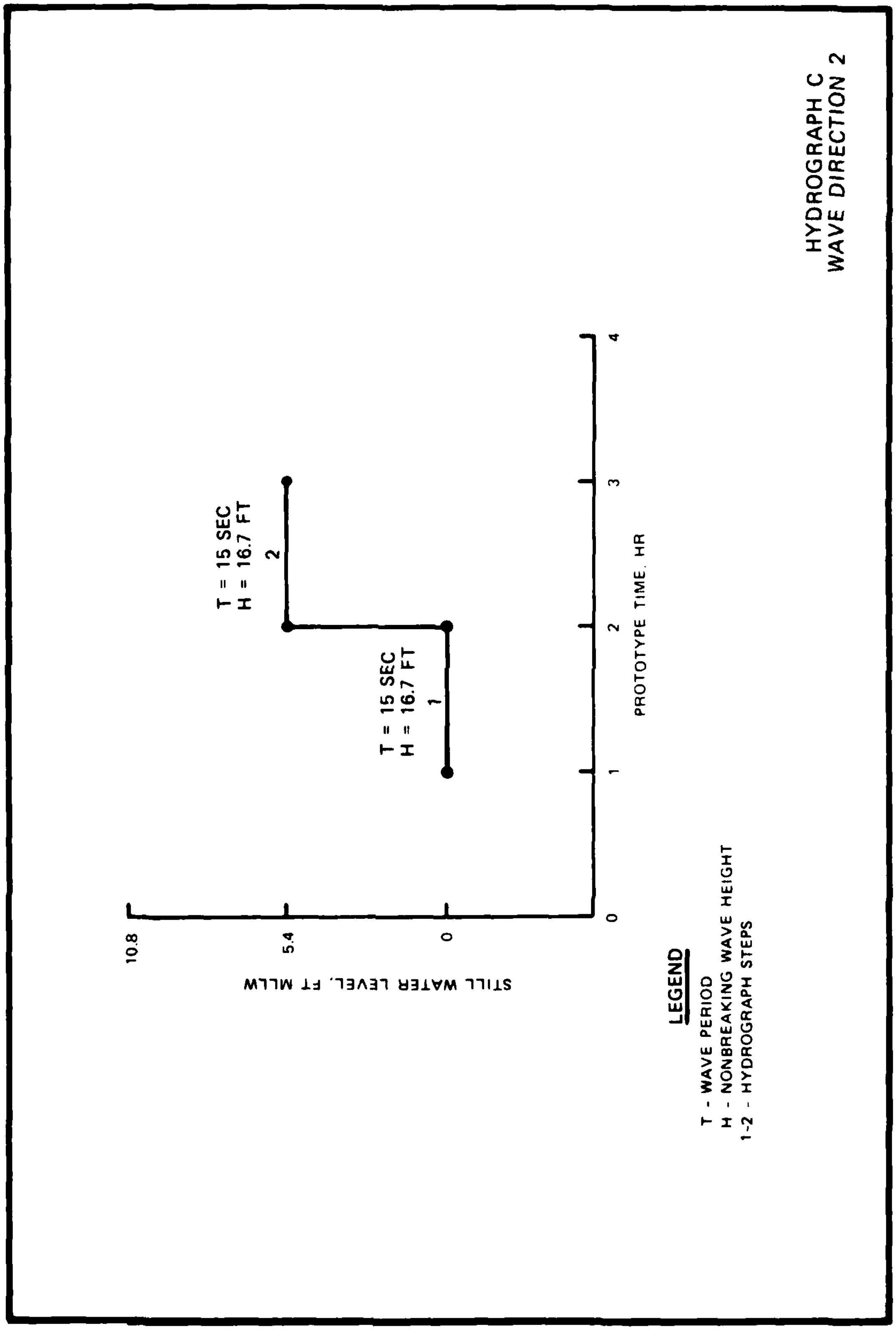
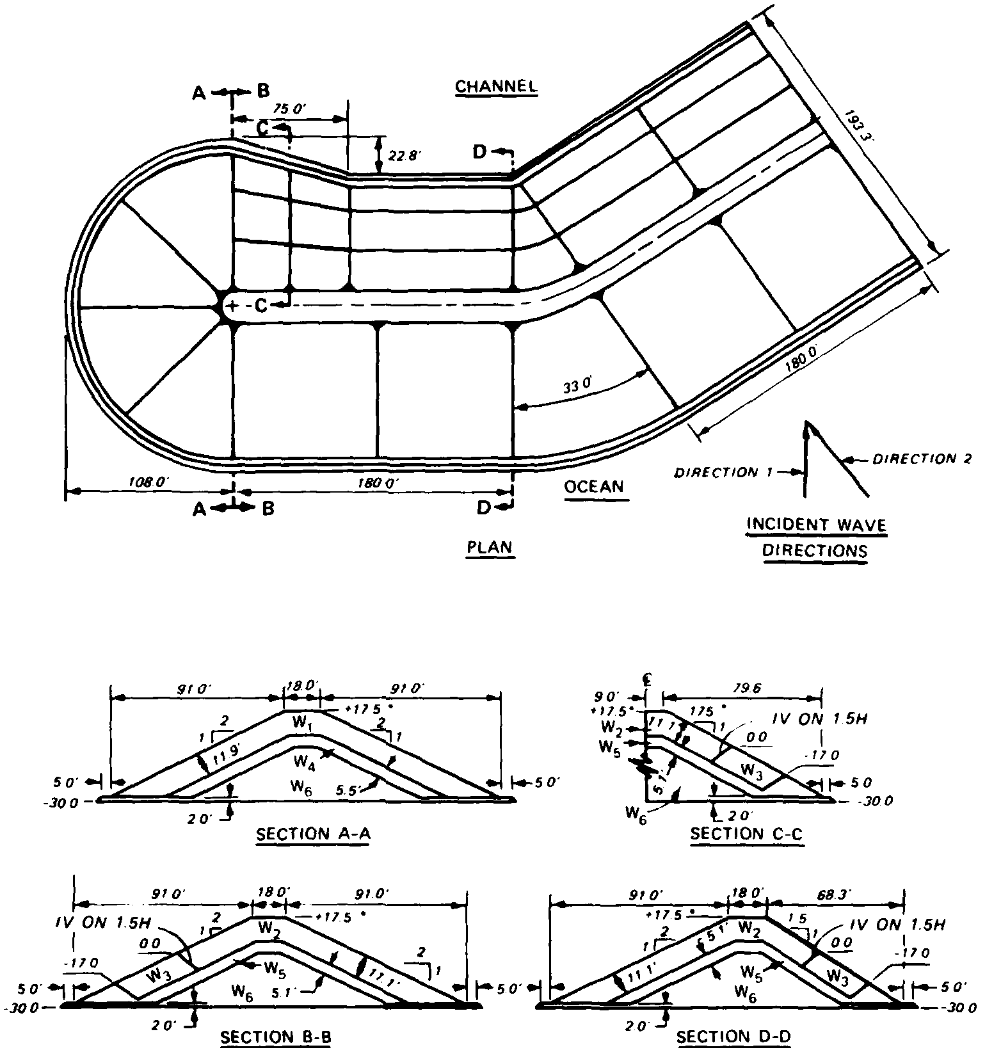

HYDROGRAPH C
WAVE DIRECTION 2
T = 15 SEC
H = 16.7 FT
1
2
STILL WATER LEVEL, FT MLLW
10.8
5.4
0
PROTOTYPE TIME, HR
0
1
2
3
4
LEGEND
T - WAVE PERIOD
H - NONBREAKING WAVE HEIGHT
1-2 - HYDROGRAPH STEPS

PLATE 4

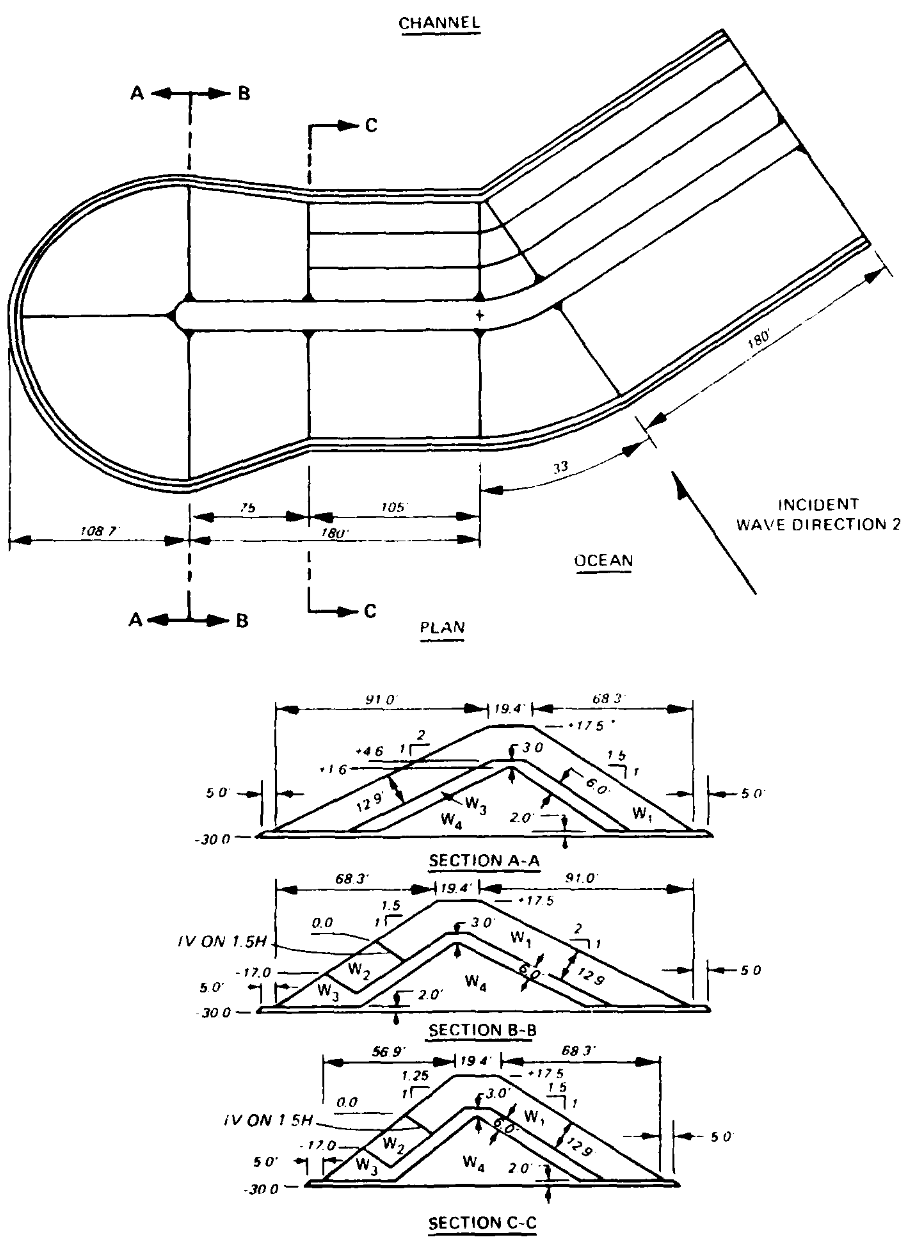


MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
** W_1 0.43-LB ROCK @ 165 PCF	W_1 22,690-LB ROCK @ 165 PCF
** W_2 0.35-LB ROCK @ 165 PCF	W_2 18,470-LB ROCK @ 165 PCF
** W_3 0.21-LB ROCK @ 165 PCF	W_3 11,081-LB ROCK @ 165 PCF
W_4 0.043-LB ROCK @ 165 PCF	W_4 2,270-LB ROCK @ 165 PCF
W_5 0.035-LB ROCK @ 165 PCF	W_5 1,850-LB ROCK @ 165 PCF
$W_6 \leq$ 0.003-LB ROCK @ 165 PCF	W_6 145-LB TO 4-LB ROCK @ 165 PCF

* ELEVATIONS IN FEET REFERRED TO MEAN LOWER LOW WATER
** RANDOM-PLACED ARMOR STONE

PLAN S2



MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
** W_1 0 55-LB ROCK @ 165 PCF	W_1 = 29,022-LB ROCK @ 165 PCF
** W_2 0 28-LB ROCK @ 165 PCF	W_2 = 14,500-LB ROCK @ 165 PCF
W_3 = 0 055-LB ROCK @ 165 PCF	W_3 = 2,900-LB ROCK @ 165 PCF
$W_4 \leq$ 0 003-LB ROCK @ 165 PCF	W_4 145-LB TO 4-LB ROCK @ 165 PCF

* ELEVATIONS IN FEET REFERRED TO MEAN LOWER LOW WATER
** RANDOM-PLACED ARMOR STONE

PLAN S3

APPENDIX A: WAVE TRANSMISSION TESTS

1. Wave transmission tests were conducted using Plan S1 of the 1:36-scale breakwater stability model, both with and without the impermeable barrier in place (Plate A1). The breakwater was subjected to incident wave heights (H_I) of 6 to 16.7 ft from wave direction 2 for wave periods of 7, 9, 11, and 15 sec at an swl of +5.4 ft mllw (Table A1). Transmitted wave heights (H_T) which were a combination of energy transmitted through the voids of the breakwater stone, diffracted around the north end of the structure, and overtopping the crown were measured 250 ft shoreward of the center line of the breakwater crown (Plate A1). From these data, transmission coefficients ($C_T = H_T/H_I$) were calculated in order to compare the levels of transmitted wave energy that occur in the lee of the breakwater both with and without the impermeable barrier. These data are shown in Table A1. As stated in paragraph 17 of the main text, only a portion (180 ft) of the 350-ft prototype dogleg was reproduced for the stability study. For this reason, it was expected that the transmitted wave heights measured in the 3-D stability model would be somewhat higher than those measured for the same incident wave conditions in the 3-D harbor wave action model. These higher transmitted wave heights are the result of larger amounts of wave energy being diffracted around the shorter dogleg on the 3-D stability model. Therefore the absolute value of the transmission coefficients are not valid, but the differences between the transmission coefficients measured with and without the barrier in place are valid indications of the effect of the barrier on transmitted wave energy. The transmission coefficients measured for a given wave period were averaged ($\overline{C_{TP}}$) and are plotted against wave period in Plate A2. The transmission coefficients measured for identical incident wave heights were averaged ($\overline{C_{TH}}$) and are plotted against wave height in Plate A3. These averaged data show 13 to 29 percent increases in the transmission coefficients when the barrier is not in place. These percentages correspond to average increases in transmitted wave heights that range from 0.24 ft for 6-ft incident wave heights to 1.2 ft for 16.7-ft incident wave heights. For the unaveraged data, the maximum increases in transmitted wave heights are 0.36 ft for 6.0-ft incident wave heights and 1.3 ft for 16.7-ft incident wave heights.

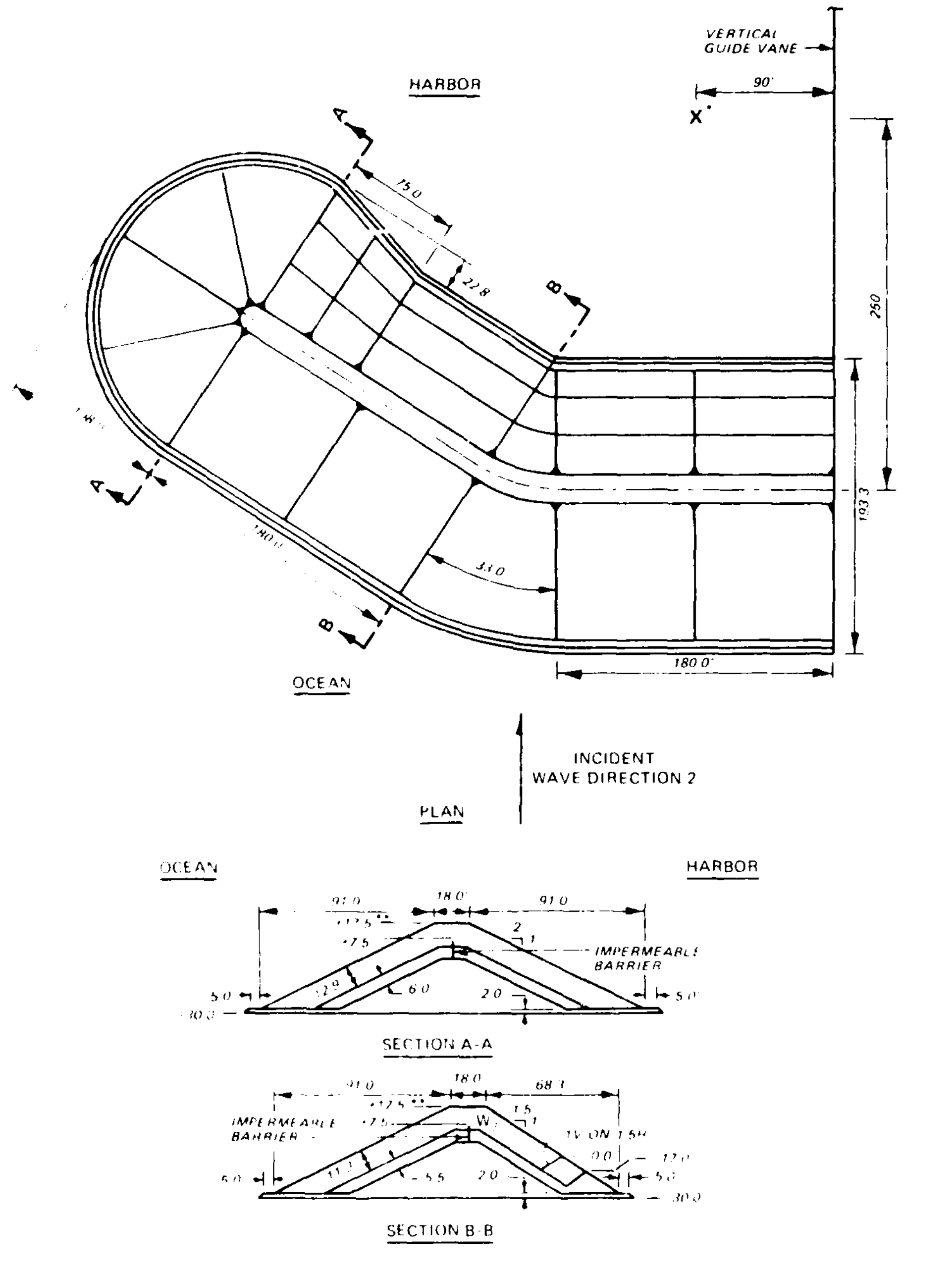
Table A1

Incident Wave Conditions and Transmission Coefficients for Plan S1

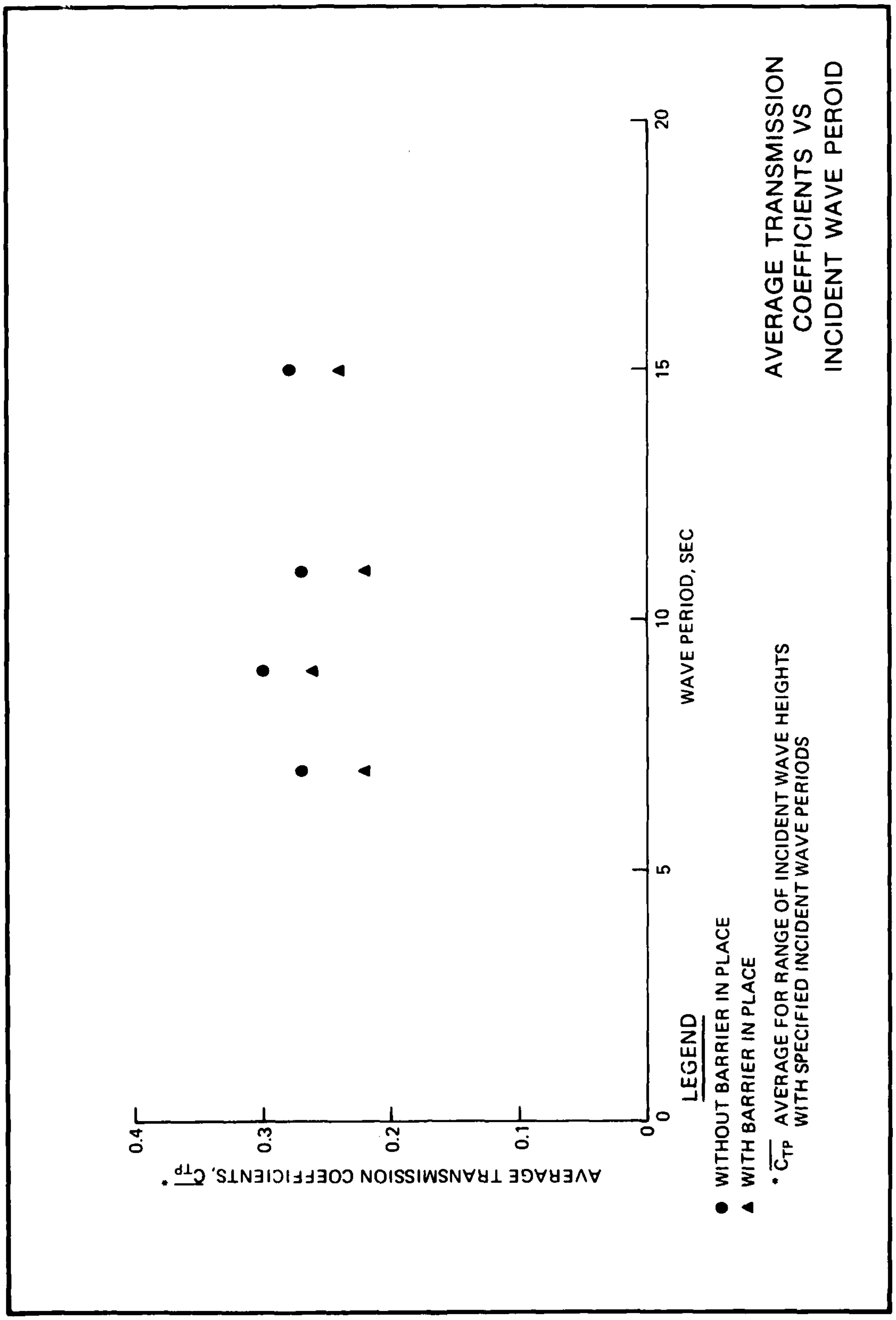
Both with and Without the Impermeable Barrier in Place

swl = +5.4 ft mllw

Incident Wave Conditions		Transmission Coefficients	
Period, sec	Height, ft	Without Barrier	With Barrier
7	6.0	0.31	0.25
7	8.0	0.27	0.24
7	10.8	0.23	0.18
9	6.0	0.30	0.27
9	9.0	0.29	0.24
9	15.1	0.31	0.26
11	6.0	0.27	0.23
11	10.0	0.25	0.21
11	16.7	0.30	0.22
15	6.0	0.27	0.25
15	10.0	0.26	0.23
15	16.7	0.32	0.25



WAVE TRANSMISSION
PLAN S1
WITH AND WITHOUT
IMPERMEABLE BARRIER



AVERAGE TRANSMISSION COEFFICIENTS VS INCIDENT WAVE PEROID

PLATE A2

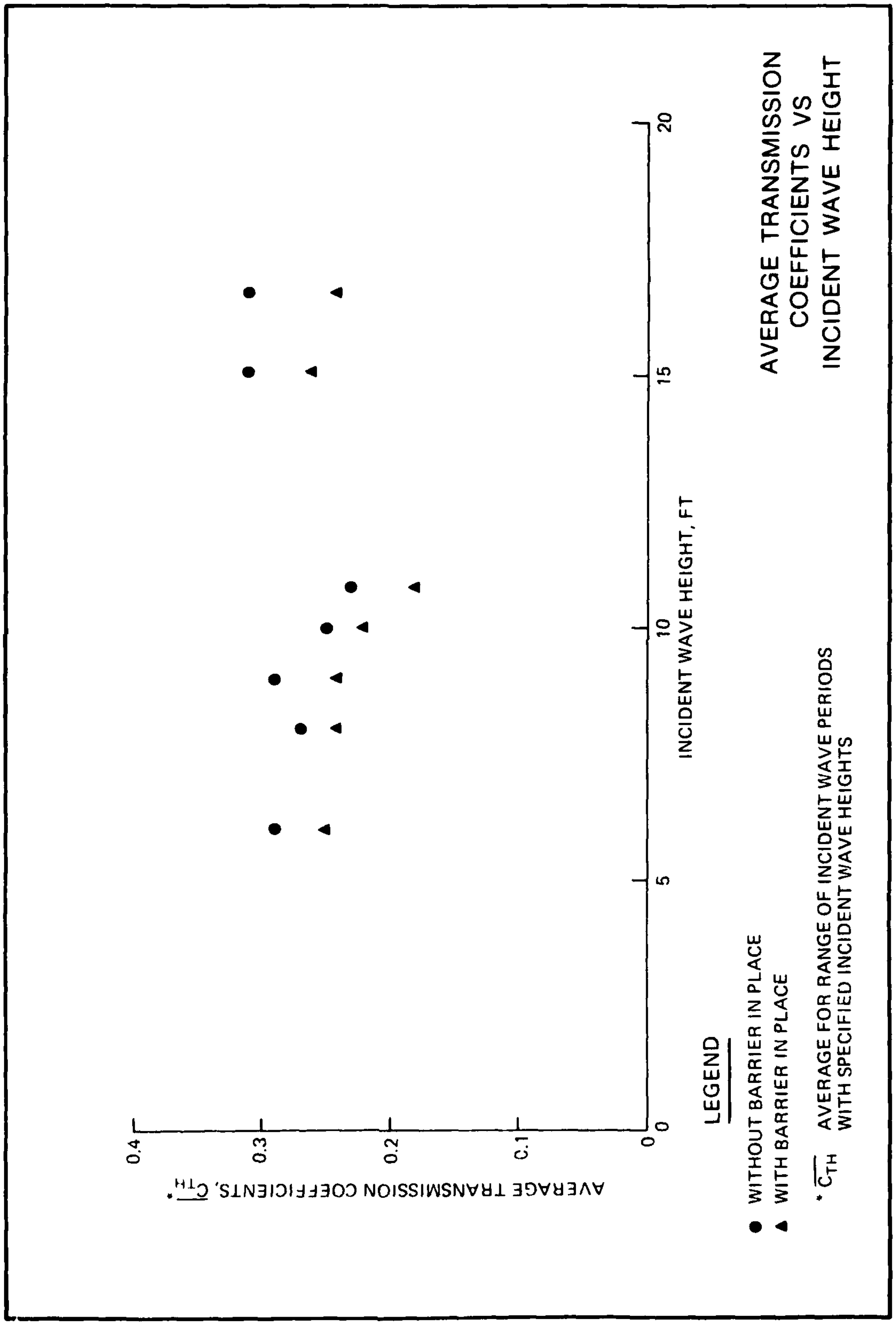

AVERAGE TRANSMISSION COEFFICIENTS VS INCIDENT WAVE HEIGHT
AVERAGE TRANSMISSION COEFFICIENTS, $\overline{C_{TH}}$*
INCIDENT WAVE HEIGHT, FT
0
0.1
0.2
0.3
0.4
5
10
15
20
LEGEND
● WITHOUT BARRIER IN PLACE
▲ WITH BARRIER IN PLACE
* $\overline{C_{TH}}$ AVERAGE FOR RANGE OF INCIDENT WAVE PERIODS WITH SPECIFIED INCIDENT WAVE HEIGHTS

PLATE A3

APPENDIX B: NOTATION

A	Area, ft^2
C_T	Transmission coefficient
$\overline{C_{TH}}$	Averaged transmission coefficient for given wave height
$\overline{C_{TP}}$	Averaged transmission coefficient for given wave period
H	Wave height, ft
H_I	Incident wave height
H_T	Transmitted wave height
k_Δ	Armor-stone layer thickness coefficient
K	Stability coefficient
L	Length, linear scale, ft
mllw	Mean lower low water
n	Number of stone layers
swl	Still-water level
S	Specific gravity
t	Thickness of stone layer, or layers, ft
T	Time, wave period, sec
V	Volume, ft^3
W	Weight, lb
α	Angle breakwater slope makes with the horizontal, deg
γ	Specific weight

Subscripts

a	Refers to stone
m	Refers to model quantities
p	Refers to prototype quantities
r	Refers to ratio of model quantities to prototype quantities (i.e., r = m/p)
w	Refers to water
1-6	Refers to different ston^ sizes

FILME

3-8