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July 1998

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Three-Dimensional Breakwater Stability Tests at Vale de Cavaleiros, Cape Verde

by Ernest R. Smith, Jeffrey A. Melby

WES

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Prepared for Joint Venture RRI-BCEOM

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U.S. Army Corps of Engineers
Waterways Experiment Station
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Final report

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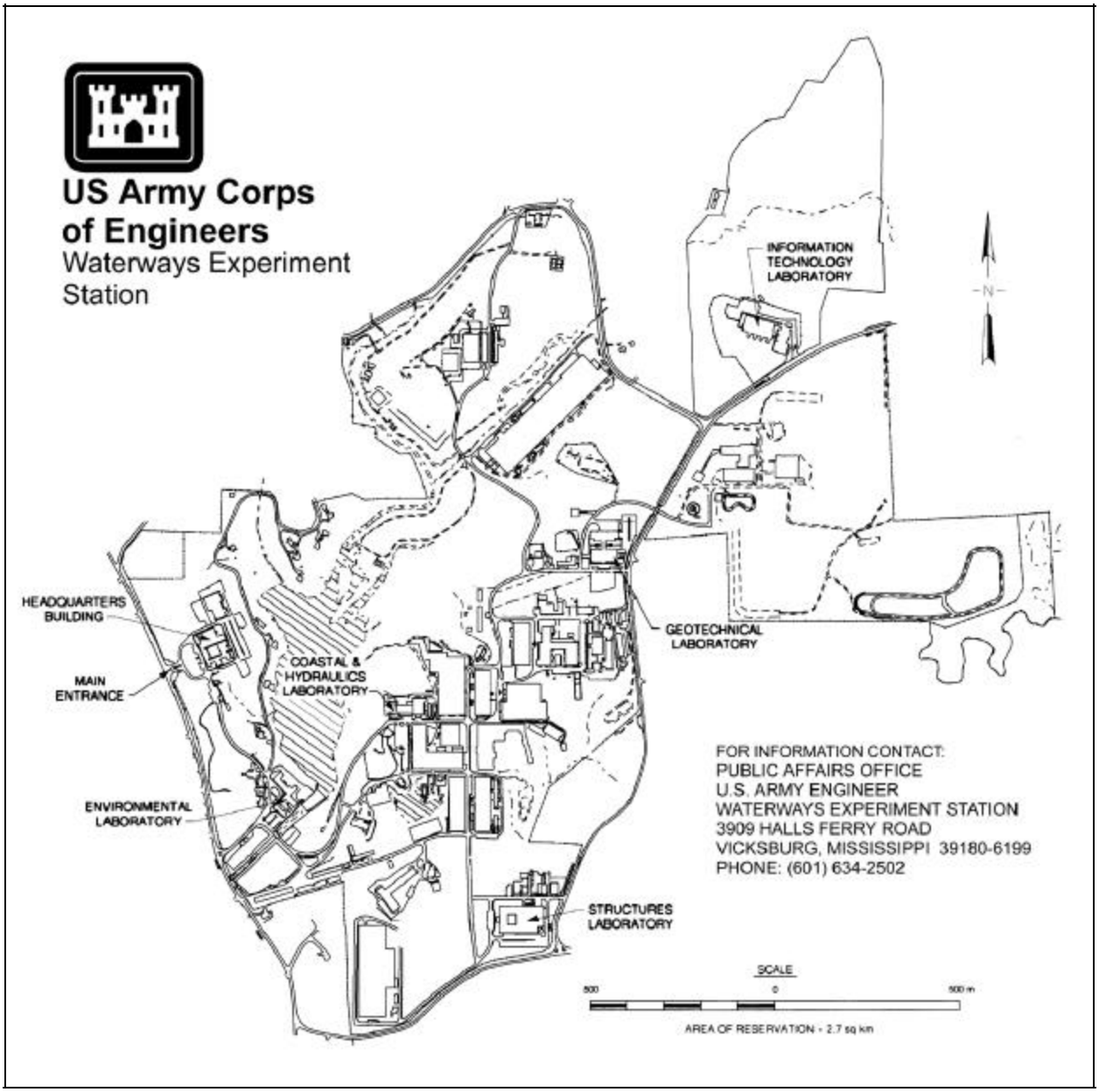

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Preface

The model investigation of the Vale de Cavaleiros breakwater reported herein was requested by Joint Venture Rhein Ruhr Ingenieur-Gesellschaft mbH (RRI) and BCEOM Société Française d'Ingénierie (JV RRI-BCEOM) and was conducted at the Coastal Engineering Research Center (CERC) of the U. S. Army Engineer Waterways Experiment Station (WES). In October 1996, CERC merged with the WES Hydraulics Laboratory to become the Coastal and Hydraulics Laboratory. Authorization for WES to perform the study was granted by Cooperative Research and Development Act (CRDA) No. WES-96-CERC-01 dated 11 January 1996.

The study was conducted at WES during the period January 1996 through May 1996 by personnel of the Coastal Structures Branch (CSB) of the Navigation and Harbors Division (NHD), CHL, under the direction of Dr. James R. Houston and Mr. Charles C. Calhoun, Jr., Director and Assistant Director of CHL, respectively; and the direct guidance of Messrs. C. E. Chatham, Jr., Chief of NHD; and Mr. D. D. Davidson, Chief of CSB. The physical model was designed by Messrs. Ernest R. Smith and Jeffrey A. Melby, Research Hydraulic Engineers, CSB. Experiments were conducted by Messrs. Smith, Willie G. Dubose, Civil Engineering Technician; John H. Williams, Civil Engineering Technician; Johnny Heggins, Civil Engineering Technician; and David Daily, Instrumentation Services Technician, NHD. This report was prepared by Messrs. Smith and Melby.

Liaison was maintained with JV RRI-BCEOM through progress reports and telephone conversations during the course of the investigation. Prior to construction of the breakwater cross section, Mr. A. Merrien, France, visited WES during 20 to 22 March 1996 to discuss proposed test conditions. Messrs. Merrien and L. Fischer, Germany, visited WES on 17 and 18 April 1996 to observe experiments and discuss alternative breakwater plans.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander of WES was COL Robin R. Cababa, EN.

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1 Introduction

Background

Vale de Cavaleiros is a small port on the west coast of the Island of Fogo, Republic of Cape Verde. Maps of the region and harbor are shown in Figures 1 and 2, respectively. The harbor is manmade, being enclosed by breakwaters extending from a natural cape. The harbor provides shelter for local fishing and commercial cargo vessels, and recreational beach use. Commercial cargo vessels are loaded and unloaded using local lighterage.

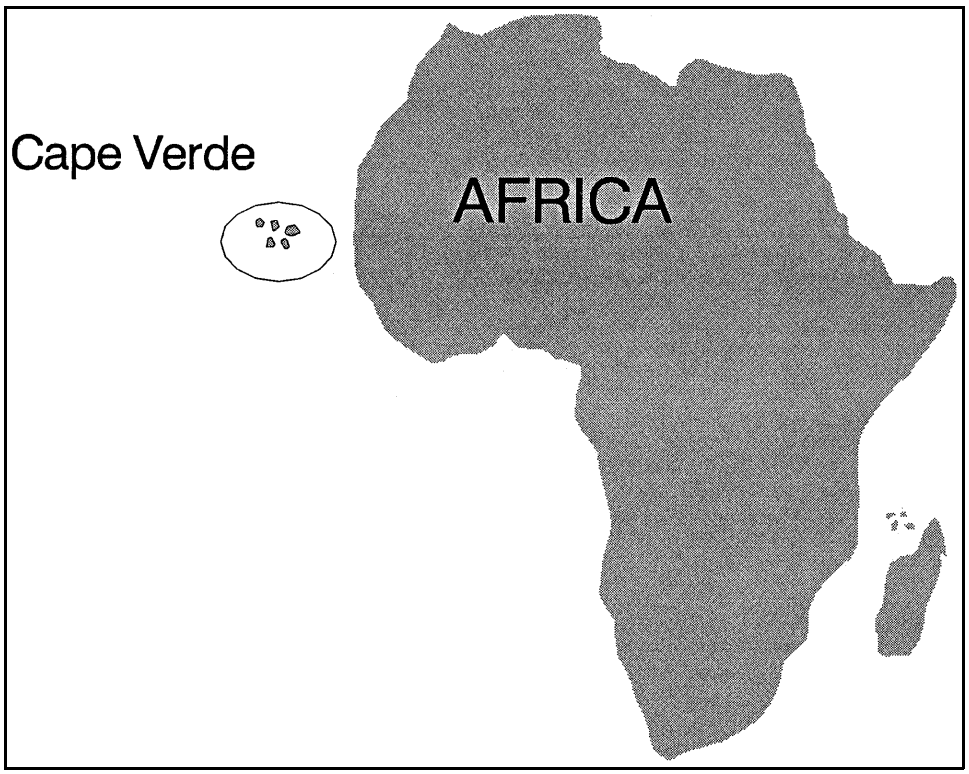

Figure 1. Location of Cape Verde

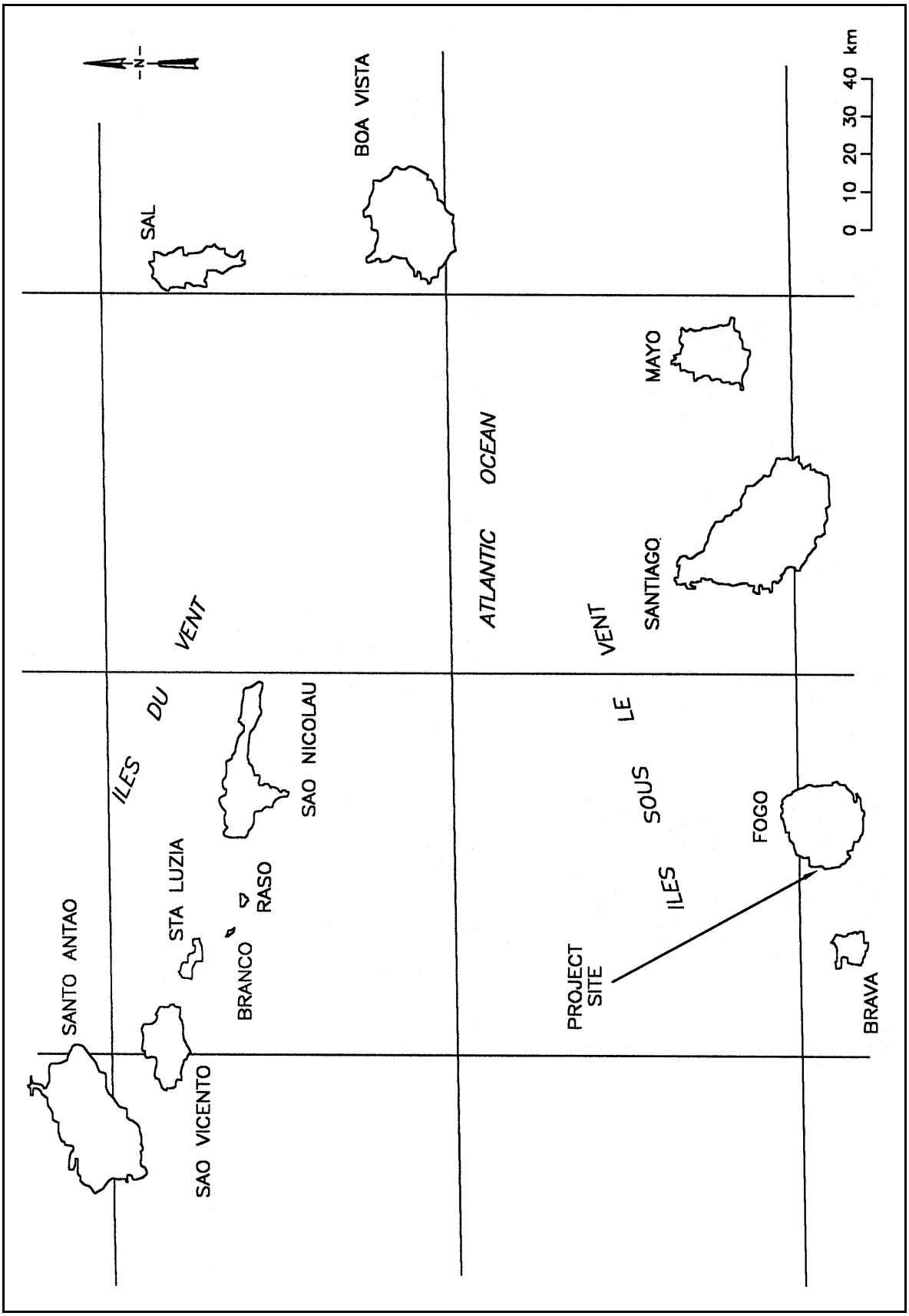

Figure 2. Cape Verde Islands

The site is exposed to Atlantic Ocean storms, with predominant swell from 300 deg north in the winter months. The sponsor-predicted 50-year return period wave height ranged from 6.7 to 7.5 m. A secondary summer storm swell is incident from approximately 230 deg north.

The existing main breakwater is shown in Figures 3 and 4, with the layout view in Figure 4 dividing the breakwater into seven profiles. The porous breakwater consists of a traditional rubble mound fronting a recurved seawall. The crest elevation of the seawall is +8.15 m CD (CD refers to the lowest theoretical chart datum) while the crest elevation of the 6.25-tonne-tetrapod armor layer is approximately +7 m CD. The head of the breakwater was virtually destroyed by storms over 25 years and its remnants are completely submerged. The remaining tetrapod-armored trunk section extends south for approximately 180 m from Profile 1 (Figure 4). The existing tetrapod section extends down to approximately 0 m CD.

The depth along the toe varies from near 2 m along the trunk to 9 m on the head, so the significant wave in the design spectrum is depth limited. The water level can vary up to 2 m due to a combination of both tide and storm surge. The seaward bottom slope is relatively steep, averaging between 1V on 12H to 1V on 15H. The foundation at the site consists of layers of sand over bedrock or old breakwater remnants with sporadic rock outcroppings. Net littoral drift is to the south.

Much of the trunk section shows considerable damage to the existing tetrapod armoring. The main armor of tetrapods was under-designed for the 50-year return period, 7-m significant wave height at the head. Tetrapods along the trunk section north of Profile 4 have remained relatively stable.

Purpose

A breakwater rehabilitation has been proposed for Vale de Cavaleiros. The rehabilitation would include extending the breakwater length, placing all salvageable tetrapods in Profiles 1 through 4, and placing Core-Loc armor units on the remainder of the structure. At the request of Joint Venture Rhein Ruhr Ingenieur-Gesellschaft mbH and BCEOM Société Française d'Ingénierie (JV RRI-BCEOM), a breakwater stability model investigation was initiated by the U.S. Army Engineer Waterways Experiment Station's Coastal Engineering Research Center. The study goal was to evaluate the stability of the Core-Loc armor layer for the proposed rehabilitation of the breakwater.

This report describes the design and facilities used (Chapter 2), and results of the three-dimensional stability study (Chapter 3). The study is summarized in Chapter 4. Appendix A contains photographs of the three-dimensional model and Appendix B includes symbol notation used in the report.

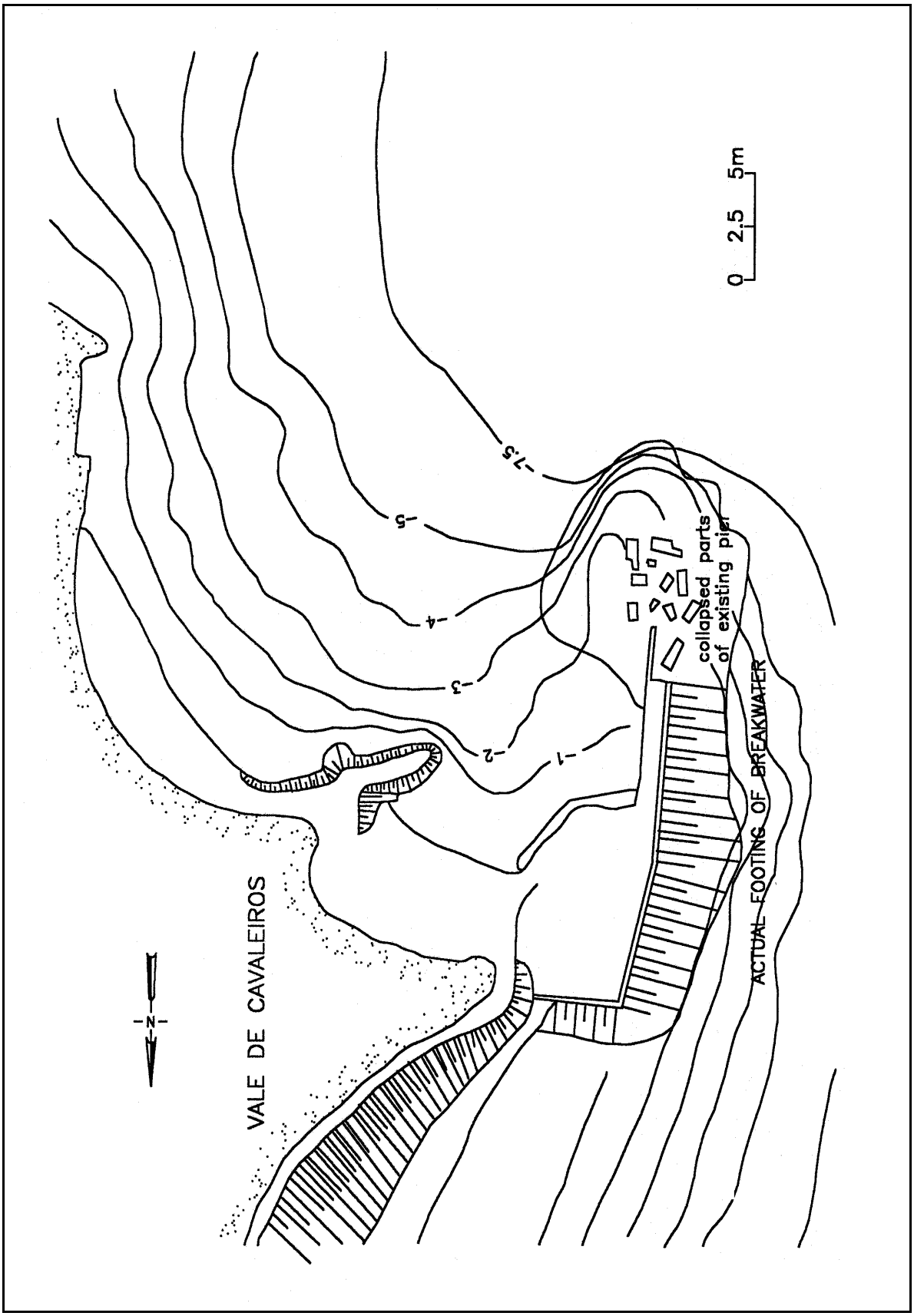

Figure 3. Existing site location map

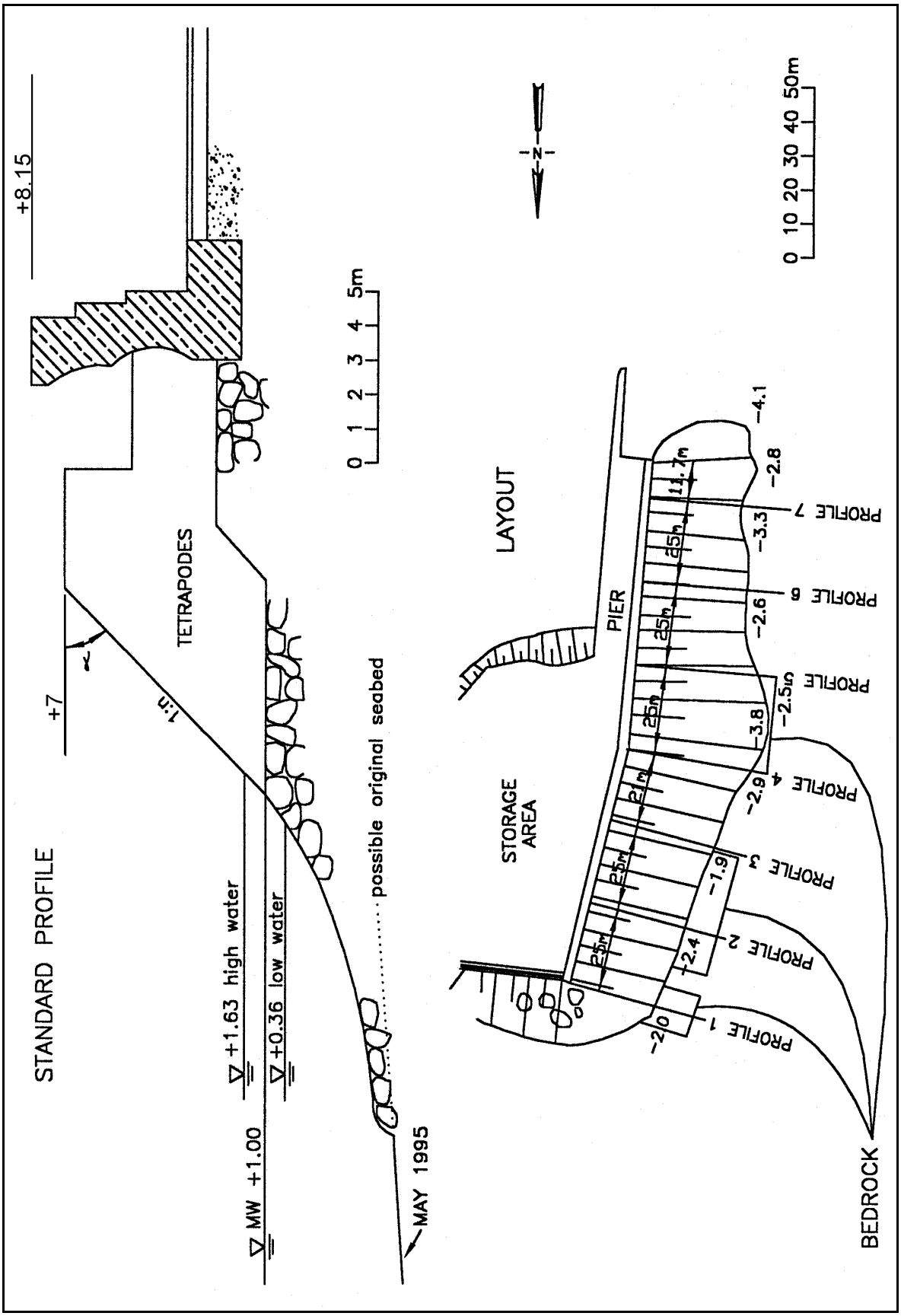

Figure 4. Existing structure

2 The Model

Model Design

Model experiments were conducted at a geometrically undistorted linear scale of 1:48.4, model to prototype. Scale was based on the size availability of model armor units and the capabilities of the available wave generator to produce required wave heights at modeled water depths. Time relations were scaled according to Froude Model Law (Stevens et al. 1942) and model-prototype relations were defined in terms of length *l* and time *t* shown in Table 1.

Table 1
Model-Prototype Scale Relations (1:48.4 scale)

Characteristic	Dimension	Scale Relations Model:Prototype
Length	l	l_r = 1:48.4
Area	l^2	a_r = 1:2342.6
Volume	l^3	v_r = 1:113,380
Time	$l^{1/2}$	t_r = 1:6.96

The specific weights of water and construction materials differed between the model and prototype; therefore, the transference equation of Hudson (1975) was used to determine model material weights:

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

where

m = model quantities

p = prototype quantities

W_a = weight of individual armor or stone

γ_a = specific weight of an individual armor unit or stone

l_m/l_p = linear scale of the model

S_a = specific gravity of an individual armor unit or stone relative to the water in which it is placed; γ_γ/γ_w, in which γ_w is specific weight of water

Material sizes and densities for prototype and model armor layer W_1, under-layer W_2, and core W_3 are listed in Table 2.

Table 2
Prototype and Model Material Sizes

Material	Prototype		Model	
	S_a	W_a	S_a	W_a
W_1	2.62	11 tonnes	2.29	145.2 g
W_1	2.62	8 tonnes	2.28	104.3 g
W_2	2.9 - 3.1	750 - 1800 kg	2.65	9.2 - 22.1 g
W_3	2.9 - 3.1	12.5 - 250 kg	2.65	0.15 - 3.1 g

Experiment Facilities and Equipment

Experiments were conducted in a 29.3-m-long, 29.6-m-wide, 1.5-m-deep wave basin. The model was constructed and molded of concrete to represent approximately 825 m of shoreline encompassing the harbor and breakwater location. Contours were molded to -20 m CD, and a 1V on 5H transition slope was molded from the -20-m contour to the model floor elevation of -21.3 m CD. Wave absorber was placed around the perimeter of the basin to minimize the effects of reflection. A photograph of the model is shown in Figure 5.

Waves were generated by a piston-type electronically controlled hydraulic system. Displacement of the wave board was controlled by a command signal transmitted to the board by a DEC Micro VAX II computer, and waves were produced by the periodic displacement of the board. Irregular wave command signals to drive the board were generated to simulate a Texel, Marsen, and Arsloe (TMA) shallow-water spectrum (Hughes 1984) for the design wave periods.

Water surface elevations were recorded by single wire capacitance-type gauges, sampled at 20 Hz. Eight gauges were used for calibration and testing. Three gauges (Gauges 1 through 3) were positioned on the flat portion of the model floor (-21.3 m CD) 3 m from the generator in an array that allowed

Figure 5. Three-dimensional stability model

calculation of incident and reflected wave heights by Goda and Suzuki (1976). The remaining gauges were placed at locations around the breakwater shown in Figure 6. Data obtained from the gauges were analyzed using the Time Series Analysis (TSA) computer program of Long and Ward (1987). Operations performed on wave data from individual gauges were mean down-crossing analysis to obtain significant wave height H_s, maximum and average wave heights, significant and average wave periods, and mean water levels at each gauge. Operations performed on the wave gauge array were unidirectional spectral density incident/reflection analysis to determine peak wave period T_p, and incident and reflected wave heights at the gauge array. Following calibration of the basin, Gauges 4 and 5 were removed and used in locations 9 and 10 during stability tests.

Selected Study Conditions

As indicated in Chapter 1 the most severe wave conditions approached the harbor from 300 deg from north; therefore, all tests were conducted for waves approaching from this direction. Prior to construction of the breakwater, wave absorber was placed over the quay wall to minimize reflection and the basin was calibrated for the design periods from the 300-deg direction. The selected water depth for all experiments was +1.8 m CD, which was based on tide and surge, and the design periods were 13, 16, and 19 sec. The maximum design storm wave height was defined by the sponsor to be as high as 6.7 m at the -21 m CD contour.

Incident significant wave height H_s^i obtained from Gauges 1 through 3 during calibration is plotted versus percent of generator stroke in Figures 7 through 9.

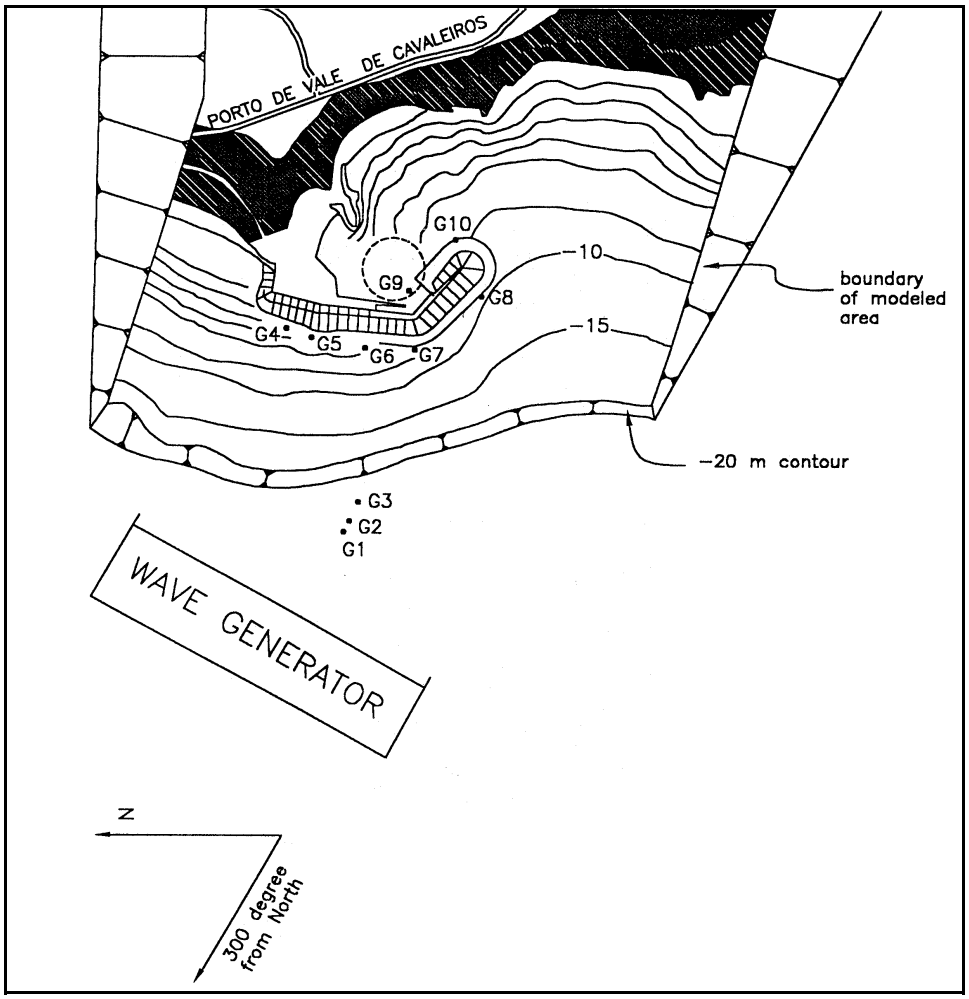

Figure 6. Three-dimensional model boundaries and wave gauge locations

The series of wave conditions selected as design storm conditions for stability experiments are shown in Table 3. The total duration of the storm was approximately 17 hr prototype. Significant wave heights recorded at Gauge locations 4 through 10 during calibration also are shown in Table 3. The basic breakwater configuration remained the same for Plans 1 through 1C, but the head portion of the breakwater was raised from +5 m CD to +8 m CD for Plans 2 through 2D. Therefore, representative wave heights for the two breakwater configurations are given by Plan 1C and Plan 2B in Table 3. For all of the wave events conducted, critical breaking waves were produced at or near some portion of the armor toe.

Experiment Procedures

Photographs were taken before each experiment was initiated without water in the basin. Following before-test photographs, the basin was flooded to +1.8-m CD and the structure was exposed to low-level waves, T_p = 13 sec, H_s' = 3.0 m,

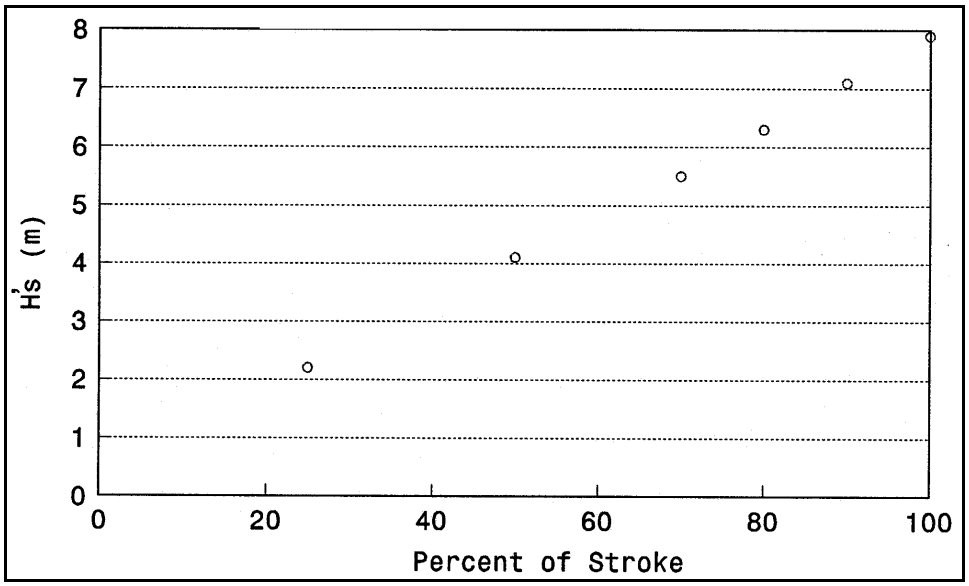

Figure 7. H_s' at -21.3 m CD versus generator stroke, 13-sec waves

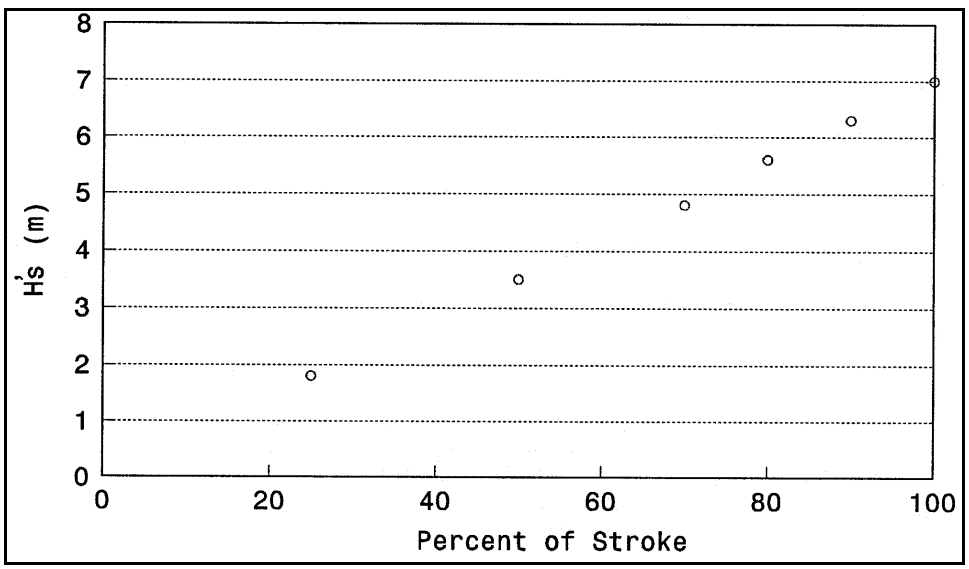

Figure 8. H_s' at -21.3 m CD versus generator stroke, 16-sec waves

wave condition 1 in Table 3. The low-level series allowed settling and nesting of the newly constructed section which would occur under typical daily wave conditions prior to being exposed to a design-level storm, and the small motion of armor units under these conditions would not normally cause breakage of units. The remainder of the wave conditions listed in Table 3 were generated upon completion of the low-level waves beginning with 13-sec, 3.7-m waves and progressing to longer periods of constant height, i.e., 16-sec, 3.7-m and 19-sec, 3.7-m waves. Wave height was increased after all periods of a given height were completed.

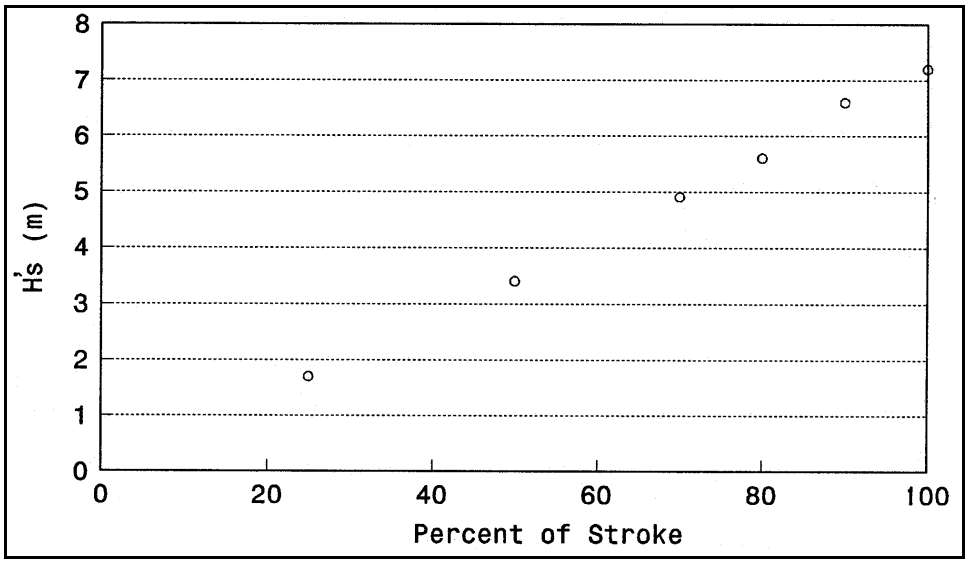

Figure 9. H_s' at -21.3 m CD versus generator stroke, 19-sec waves

Table 3
Stability Study Wave Conditions

Breakwater Plan	H_s', m	H_s at Gauge, m						
		4	5	6	7	8	9	10
T_p = 13 sec								
Plan 1C	3.0			4.3	2.3	1.8	0.8	1.3
Plan 2B	3.0			4.4	2.3	1.9	0.5	1.2
Calibration	3.0	4.0	4.6	3.1	2.1	1.0		
Plan 1C	3.7			4.8	2.8	2.4	0.9	1.6
Plan 2B	3.7			4.8	2.7	2.4	0.6	1.4
Calibration	3.7	4.5	4.9	3.7	2.9	1.2		
Plan 1C	5.2			5.3	4.2	4.2	1.1	2.5
Plan 2B	5.2			5.4	4.2	4.3	0.7	2.0
Calibration	5.2	5.1	5.7	4.3	4.3	2.1		
Plan 1C	6.7			5.4	5.5	5.8	1.2	3.0
Plan 2B	6.7			5.6	5.5	5.8	0.8	2.5
Calibration	6.7	5.2	5.9	4.4	5.7	3.2		
T_p = 16 sec								
Plan 1C	3.7			4.7	3.6	3.6	1.0	2.0
Plan 2B	3.7			4.7	3.6	3.6	0.6	1.6
Calibration	3.7	4.6	5.2	4.0	3.4	1.5		

(Continued)

Table 3 (Concluded)								
Breakwater Plan	H_s', m	H_s at Gauge, m						
		4	5	6	7	8	9	10
T_p = 16 sec								
Plan 1C	5.2			5.3	5.3	5.8	1.2	2.8
Plan 2B	5.2			5.3	5.2	5.7	0.7	2.3
Calibration	5.2	5.2	5.8	4.8	4.6	2.5		
Plan 1C	6.7			5.6	6.5	7.2	1.3	3.2
Plan 2B	6.7			5.7	6.5	7.3	0.8	2.7
Calibration	6.7	5.2	5.9	5.0	5.9	4.0		
T_p = 19 sec								
Plan 1C	3.7			4.6	3.9	4.0	1.0	1.9
Plan 2B	3.7			4.5	3.8	4.0	0.6	1.5
Calibration	3.7	4.3	4.6	5.1	3.2	2.0		
Plan 1C	5.2			5.4	5.9	6.4	1.2	2.7
Plan 2B	5.2			5.4	6.0	6.4	0.8	2.3
Calibration	5.2	5.1	5.6	6.2	4.8	2.8		
Plan 1C	6.7			5.9	7.2	7.8	1.3	3.3
Plan 2B	6.7			6.0	7.2	7.9	0.8	2.8
Calibration	6.7	5.3	6.0	5.4	6.2	4.1		

Response of the structure was recorded during and after each wave condition. Photographs also were taken at seaside locations while the basin was flooded if significant damage to the structure occurred during a wave condition. A detailed inspection of the structure also was performed and effects of the waves on individual units, toe buttress protection, and the general condition of the structure were recorded. The basin was drained, and after-experiment photographs were taken after all waves of the storm series were generated or the structure had suffered significant damage. Before and after photographs are located in Appendix A.

Model Breakwater Construction

Construction of the modeled section simulated prototype construction as closely as possible. The core, bedding, and underlayer of material were dumped by shovel, smoothed to grade, and compacted with hand trowels to simulate consolidation that would have occurred due to wave action.

The various model plans consisted of 8- and/or 11-tonne Core-Locs placed in the armor layer from Profile 4 to the breakwater head, and 6.25-tonne tetrapods

placed between Profiles 1 and 3. Core-Locs were placed according to the method given by Melby and Turk (1995).

The number of Core-Loc units placed on the breakwater, or density of units, was based on the equation:

$$\frac{N}{A} = \phi V^{-\frac{2}{3}} \tag{2}$$

where N is the number of units in a given area, A is the section area, V is the armor unit volume, ϕ is the packing density coefficient which is dependent upon armor layer thickness and armor layer porosity. Armor layer thickness is equal to about 0.92 of the respective Core-Loc leg length and the average armor layer porosity is about 60 percent. Core-Loc is a relatively new armor unit and tests are ongoing to determine optimal and constructable placement density of the units. For this stability study, ϕ was purposely unspecified so that each model armor placement would be a natural coverage and not exceed prototype construction limitations. For the tests conducted, ϕ ranged from 0.55 to 0.63 depending on the base area covered and variance due to the random nature of the packing.

Reporting Model Observations

Visual inspections were made during and after wave action on the structure. Because Core-Locs are placed in one layer, less than 2 percent displacement by unit count was desired.

3 Results

Introduction

Three-dimensional stability experiments were conducted for nine configurations at a model scale of 1:48.4. The configurations consisted of two basic breakwater plans. Plan 1 and Plan 2 differed in that Plan 1 consisted of 8- and 11-tonne Core-Locs and a crest elevation of +5 m CD at the head section, whereas Plan 2 was armored entirely with 11-tonne Core-Locs and had a head section elevation of +8 m CD. Four toe protection schemes were studied using the Plan 1 configuration, and five toe configurations were used with Plan 2.

Plan 1

Plan 1, the initial plan (Figure 10, Photos A1 through A3), consisted of 6.25-tonne tetrapods from the shoreward end of the breakwater to Profile 4; 8-tonne Core-Locs from Profile 4 to a point between Profiles 6 and 7 (a total reach of approximately 92 m); and 11-tonne Core-Locs on the remainder of the seaward trunk, around the head and terminating at the quay wall on the leeward side of the structure. For this plan, packing densities of the 8- and 11-tonne Core-Loc reaches were 0.55 and 0.59, respectively. The crest elevation of the head section was +5 m CD. Plan 1 was subjected to "shakedown" waves (13 sec, 3 m) to settle and nestle the armor units. During the wave series, apron material was displaced between Profiles 4 and 9, and Core-Locs were displaced at the toe near Profile 4 (Photos A4 through A6). The structure was subjected to 3.7-m waves for 13- and 16-sec periods, which caused additional toe unit displacement at Profile 4 and at the elbow near Profile 8.

Plan 1A

Plan 1A was the same as Plan 1 except a toe buttress was constructed of stone of identical weight as the underlayer stone (0.75 to 1.8 tonne) and placed between Profile 4 and the terminus of the quay wall near Profile 10 (Photos A7 through A9). The buttress was placed two stones high and three stones wide

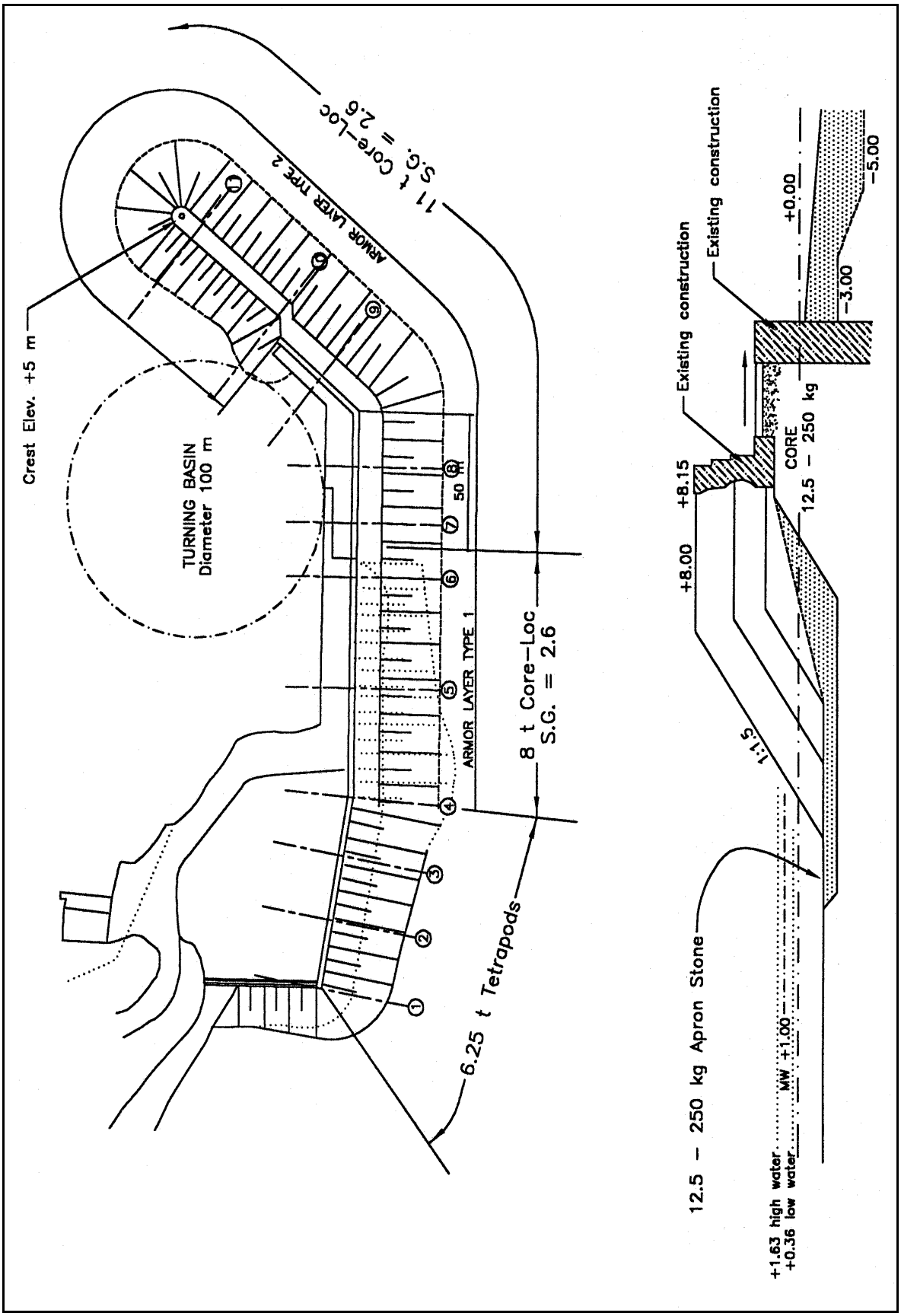

Figure 10. Elements of plan 1

(Figure 11). The 8-tonne Core-Loc armor layer was rebuilt resulting in a packing density of 0.60. The buttress material was displaced between Profiles 4 and 5 and between Profile 8 and the elbow of the structure during 13-sec, 3-m waves. Core-Locs in the vicinity of Profile 4 began to slide seaward, but the structure remained stable. However, the buttress was removed during 13-sec, 3.7-m waves between Profiles 4 and 6 and toe units in this region began to move. Eight-tonne units were displaced between Profiles 4 and 5 during 16-sec, 3.7-m waves and a hole developed at the crest near Profile 4 (Photo A10). The larger units, 11 tonnes, remained stable during Plan 1A tests (Photos A11 and A12). It was noted that the steep approach slope and the shallow depth in this region produced breaking waves that plunged along the toe region. The plunging breakers caused apron material to erode and forced the toe Core-Locs away from the structure.

Plan 1B

Plan 1B was identical to Plan 1 except three widths of bundled steel chain were placed along the toe from Profile 4 to the elbow (Figure 12). A single chain was placed at the toe at the elbow to the terminus of the quay wall. Additionally, a single chain was placed around the toe of the tetrapod section. The purpose of the chain was to stabilize the toe to observe the stability of upslope units. Rebuilding of the 8-tonne Core-Loc section resulted in $\phi = 0.60$. The addition of the chain stabilized the Core-Locs at the toe; however, wave energy also displaced the chain and it was necessary to reposition the chain between wave series. The armor layer loosened between Profiles 4 and 5, and apron material was displaced for wave conditions up to 17-sec, 5.2-m, but the section remained stable (Photo A13). During the 17-sec, 5.2-m condition, 11-tonne units at the toe near Profile 10, which was unprotected by the chain, were removed by waves, causing the upper units in this area to be displaced. A hole developed in the armor layer near the crown at Profile 10, which increased in size and migrated “north” along the structure (Photos A14 through A16). Additionally, green water overtopping was observed over the +5-m CD breakwater section for waves 3.7 m in height and greater. The severe breaking condition migrated to deeper water as peak period and/or wave height increased.

Plan 1C

Plan 1C was the same as Plan 1B except three widths of steel chain were included along the toe from Profile 4 to the leeward side of Profile 11 for Plan 1C (Figure 12). In addition, the chain was anchored to the breakwater at approximately 8-m (prototype) intervals using pins inserted under the structure. The breakwater was rebuilt prior to experiments and the resulting packing densities for the 8- and 11-tonne Core-Locs were 0.62 and 0.59, respectively. The structure was stable for waves up to 6.7 m, but it was observed that the armor layer loosened in areas between chain anchors. During 16-sec, 6.7-m waves, one

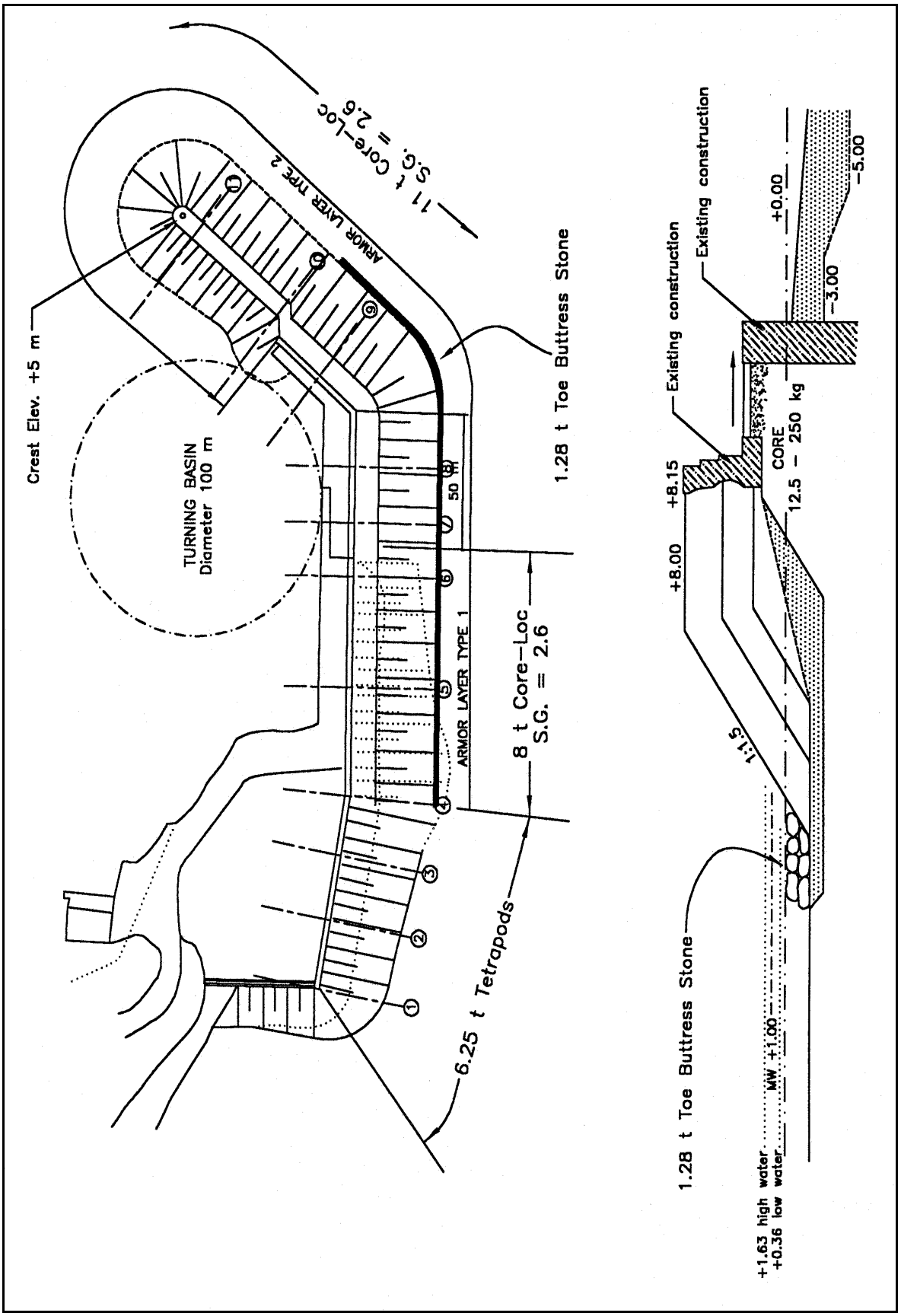

Figure 11. Elements of plan 1A

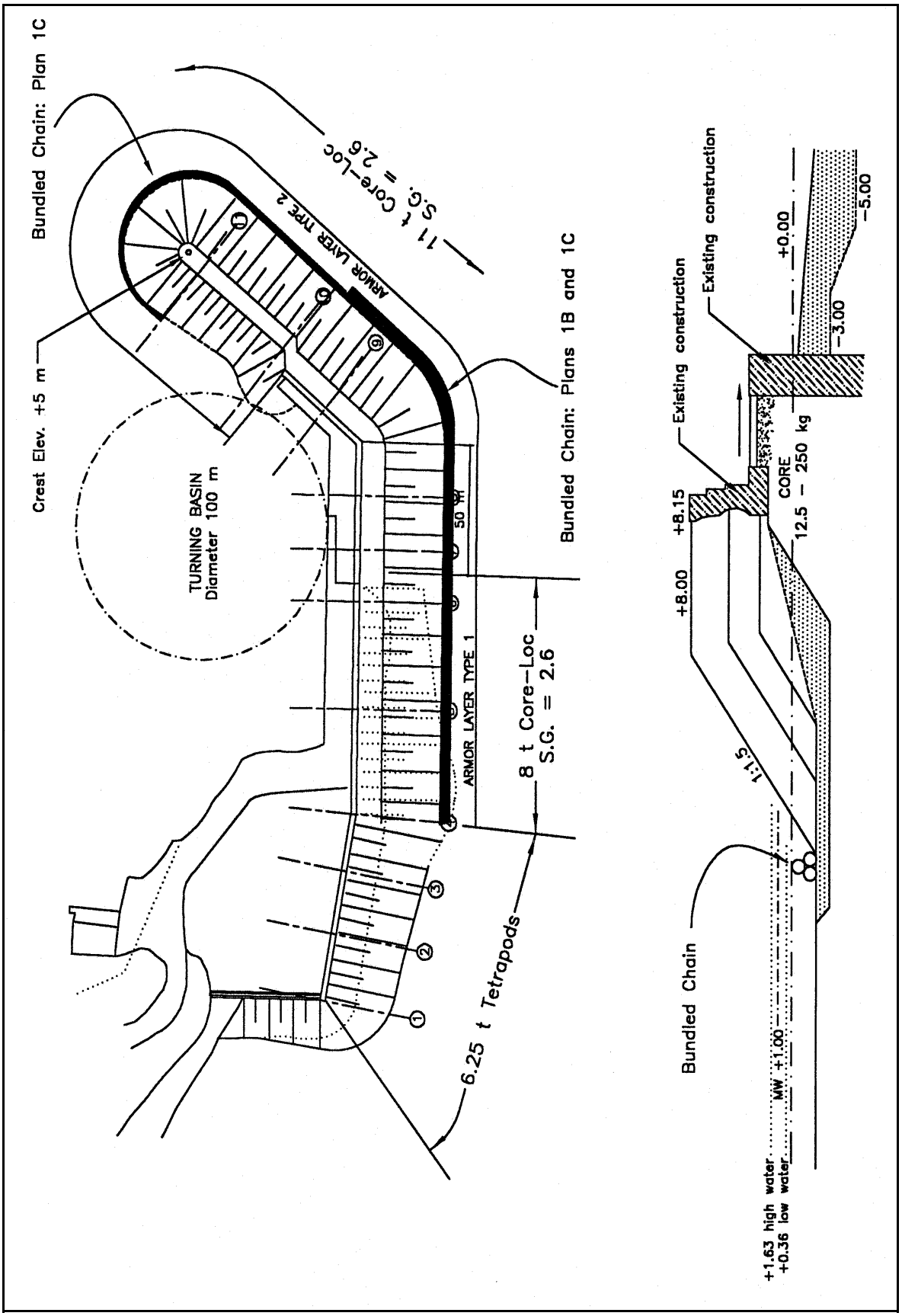

Figure 12. Elements of plans 1B and 1C

11-tonne and three 8-tonne toe units were displaced. The toe failed near Profile 10 during 19-sec, 6.7-m waves at a location where the chain was forced seaward between anchors (Photos A17 through A19).

Plan 1C was rebuilt and repeated with additional pins used to anchor the chain to the structure. Packing densities remained the same as the original Plan 1C. The plan was subjected to the entire storm series and was moderately stable. Three 8-tonne units were displaced between Profiles 4 and 5, and two 11-tonne units were displaced between Profiles 9 and 10 (Photos A20 through A22). All displaced units originated at the toe.

Plan 2

Based on observations of the model and conversations with JV RRI-BCEOM personnel Messrs. A. Merrien and L. Fischer, Plan 2 consisted of raising the +5-m head portion of the breakwater to bring the entire structure to +8 m to reduce overtopping. The 8-tonne units were replaced with 11-tonne Core-Locs and the toe was reinforced with additional 11-tonne units placed in a single row (Figure 13). The packing density of the 11-tonne Core-Locs was 0.62. Toe protection units were placed 90 deg to the toe units of the structure in a manner in which adjacent units interlocked (Photos A23 through A25). The 11-tonne Core-Locs used for toe protection were displaced during 13- and 16-sec, 3.7-m waves, and the armor layer failed due to toe instability (Photos A26 and A27).

Plan 2A

Plan 2A was the same as Plan 2 except the single row of 11-tonne Core-Locs was replaced with a wood board anchored to the floor around the entire Core-Loc armor layer to act as an immovable toe restraint. The board simulated a toe trench 1 m deep and included a 45-deg bevel adjacent to the toe unit (Figure 14). The plan consisted entirely of 11-tonne Core-Locs ($\phi = 0.61$) beginning at Profile 4 with a crest elevation of +8 m CD throughout the structure (Photos A28 through A30). One toe unit was displaced during 16-sec, 3.7-m waves near Profile 5, but the structure was stable for waves up to 16-sec, 5.2-m waves. During the 16-sec, 5.2-m condition, one toe unit was displaced near Profile 6, one toe unit was displaced from Profile 9, and three toe units were displaced near Profile 10. No further displacement of Core-Locs was observed for 19-sec, 5.2-m or 13-sec, 6.7-m waves; however, underlayer stone was observed to be displaced between Profiles 4 and 6 in areas of toe displacement. Additionally, the armor layer began to loosen in this area. During 16-sec, 6.7-m waves, 16 additional toe units were displaced, producing noticeable damage to the structure. Most of the armor was displaced off the structure at the elbow between Profiles 8 and 9 during 19-sec, 6.7-m waves (Photos A31 through A33).

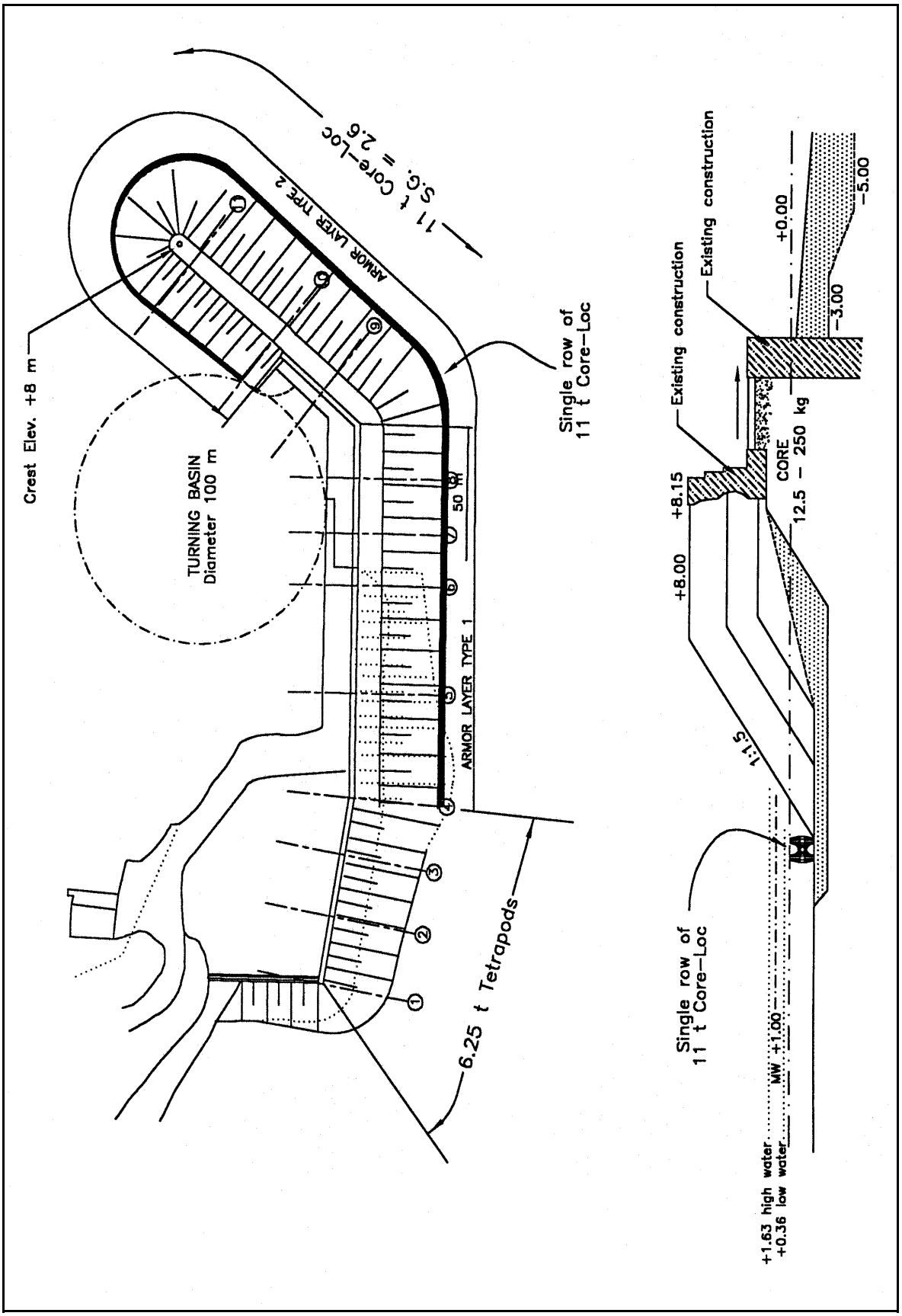

Figure 13. Elements of plan 2

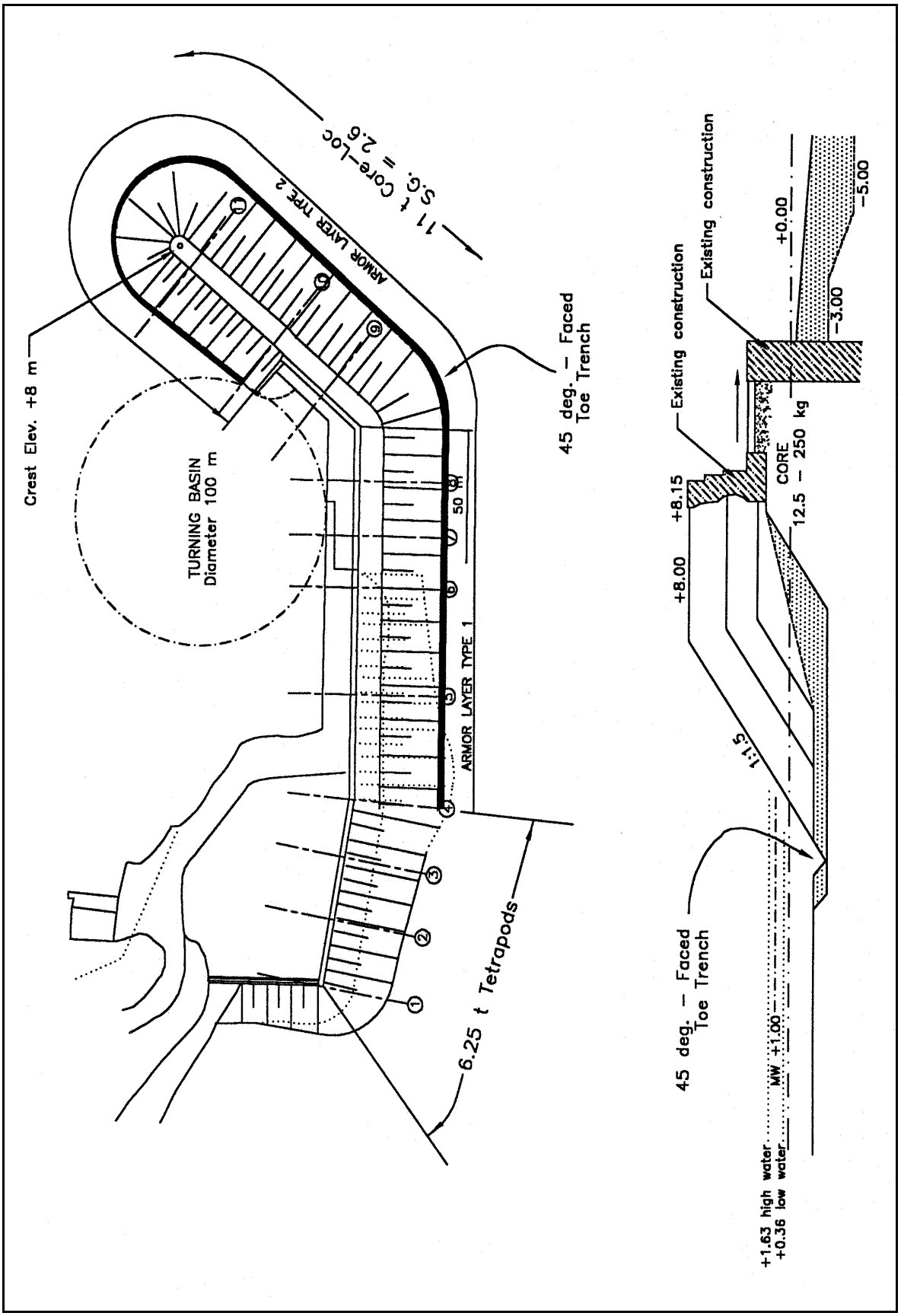

Figure 14. Elements of plan 2A

Plan 2B

Plan 2B was identical to plan 2A except the toe reinforcement was constructed using a board anchored to the floor to represent a toe trench 1.5 m high with vertical sides (Figure 15). The breakwater armor was rebuilt and the packing density for the 11-tonne Core-Locs was 0.63. To expedite the study, all 3.7-m waves and the 13-sec, 5.2-m condition were omitted for this test series. From previous experiments with plans which included a restrained toe, the omitted wave conditions caused no or only minor damage to the breakwater. No units were displaced during this series (Photos A34 through A36). Plan 2B was rebuilt with the same packing density and the experiment was repeated. One Core-Loc was displaced off the head midway through 19-sec, 6.7-m waves, but the structure remained stable throughout the rest of the wave condition.

Plan 2C

The breakwater was rebuilt entirely of 11-tonne Core-Locs (ϕ=0.63), but the board used to simulate a toe trench was removed between Profile 4 to a location 70 m from the elbow (Figure 16, Photos A37 through A39). Model concrete blocks were placed in this area to simulate 6.1-m-long, 2.5-m-wide, 1.2-m-high cargo containers filled with concrete. The containers had an approximate prototype weight of 42.4 tonnes using a concrete specific gravity of 2.3 in the prototype. The containers were placed 1 m apart along the toe and 11-tonne Core-Locs were placed against the containers.

The structure was subjected to all waves listed in Table 3. The containers began to displace during 13-sec, 3.0-m waves and movement of containers increased as wave height increased. However, the containers provided some sheltering and prevented unraveling of the Core-Loc toe up to 3.7-m waves for all three periods. Waves higher than 3.7 m for all periods displaced the containers out of the section and moved them southward along the toe and around the elbow to the head (Photos A40 through A42). After the containers were displaced from the original section, toe units in the area were displaced and upslope units slipped, causing significant exposure of the underlayer stone between Profiles 4 and 5 (Photo A40).

Plan 2D

Plan 2D was identical to Plan 2C except the cargo containers were placed end to end along the Core-Loc toe (Figure 16, Photos A43 and A44). The armor layer was rebuilt with a packing density of 0.62. Results were similar to experiments with Plan 2C; the containers began to displace during 13-sec, 3.0-m waves and movement continued with higher waves. Eventually, all containers were displaced from their original position and moved southward along the breakwater. Without toe protection, Core-Loc toe units were displaced and upslope

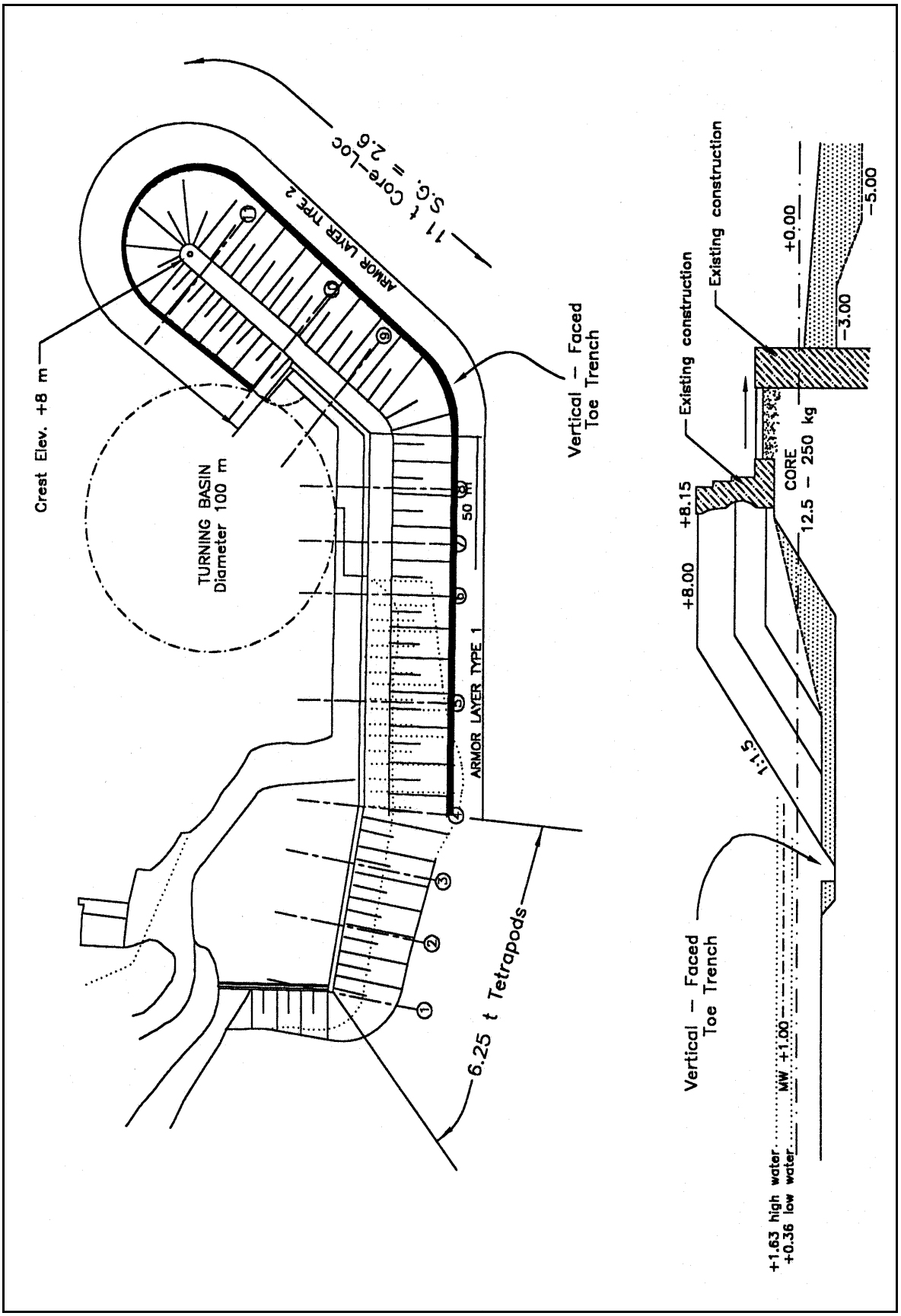

Figure 15. Elements of plan 2B

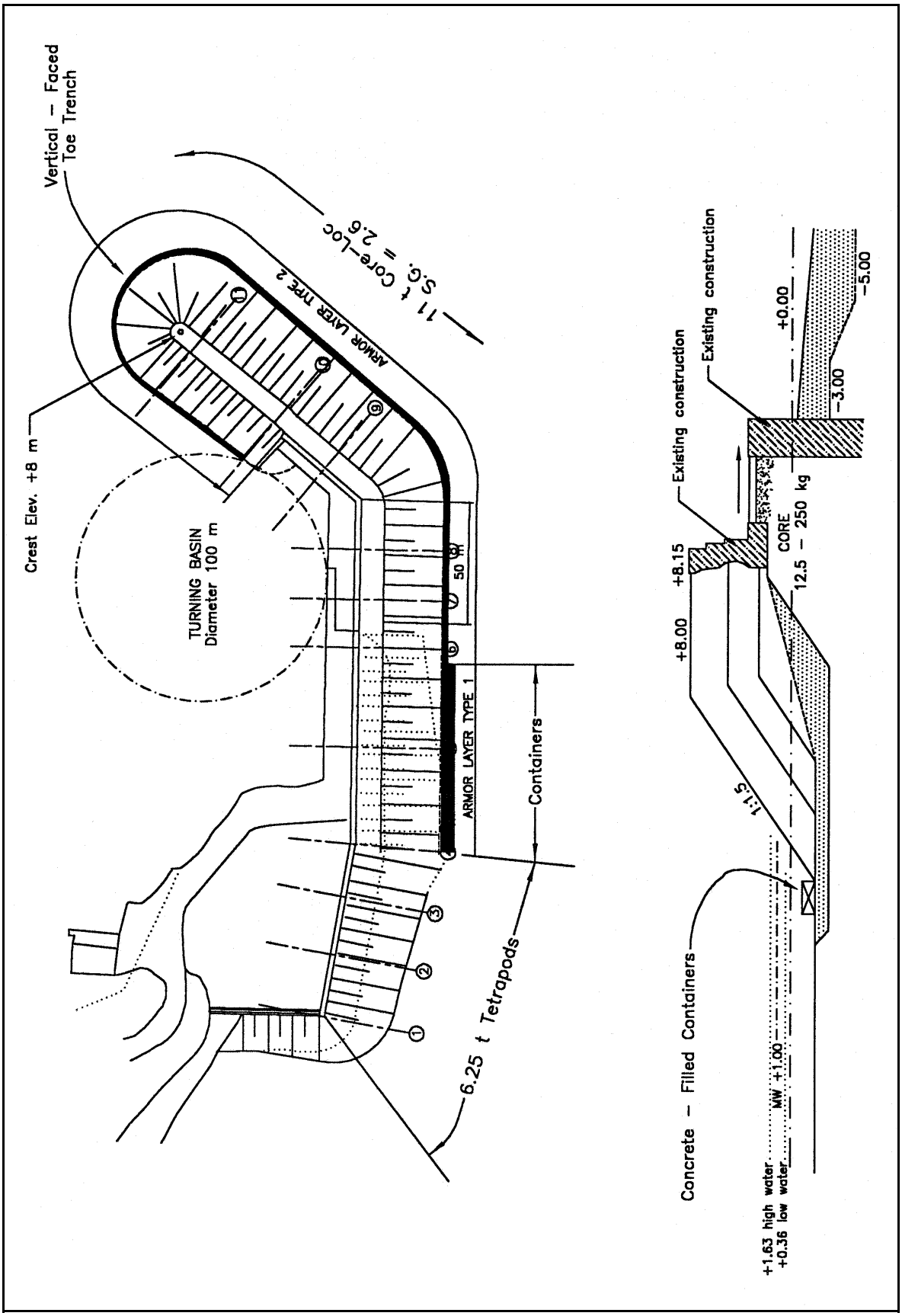

Figure 16. Elements of plans 2C and 2D

units settled causing exposure of underlayer stone between Profiles 4 and 5 (Photos A45 and A46).

Overtopping

Observations during the experiments showed that overtopping was essentially the same for Plans 1 through 1C and for Plans 2 through 2D, because only the toe stability configuration differed between plans. All Plan 1 series experiments were conducted with 8- and 11-tonne Core-Locs, and the head portion of the breakwater was +5 m CD. The Plan 2 series of experiments consisted of all 11-tonne Core-Locs and the head portion of the breakwater was +8 m CD.

Overtopping was classified as minor, moderate, or major. Minor overtopping was defined for the present study as occasional or no overtopping. Moderate overtopping was defined as regular overtopping with occasional green water. Conditions that produced frequent overtopping and green water were classified as major.

The classification of overtopping (minor, moderate, or major) is plotted versus wave condition in Figures 17 through 19. The wave conditions in the figures are labeled from left to right by the sequence in which they were generated during the experiment, i.e., 13-sec, 3.0-m waves were generated first and 19-sec, 6.7-m waves were generated last. Figure 17 shows the amount of overtopping that occurred at the trunk for all plans. Major overtopping occurred at the trunk for 16-sec, 6.7-m waves, but was minor or moderate for all other conditions. Figures 18 and 19 show overtopping classifications for Plan 1 and Plan 2 series experiments at the head, respectively. The figures illustrate the reduction of overtopping by raising the crest elevation to +8 m CD (Plan 2 series) from +5 m CD (Plan 1 series).

Summary

Stability plans tested are summarized in Table 4. Based on experiments conducted on the three-dimensional model of Vale de Cavaleiros breakwater, 11-tonne Core-Locs are stable on the structure if the toe is stable. It was necessary in the model to simulate a trench having a near vertical face and a depth of 1.5 m. No reduction in stability was observed by raising the +5-m CD portion of the breakwater to +8 m CD.

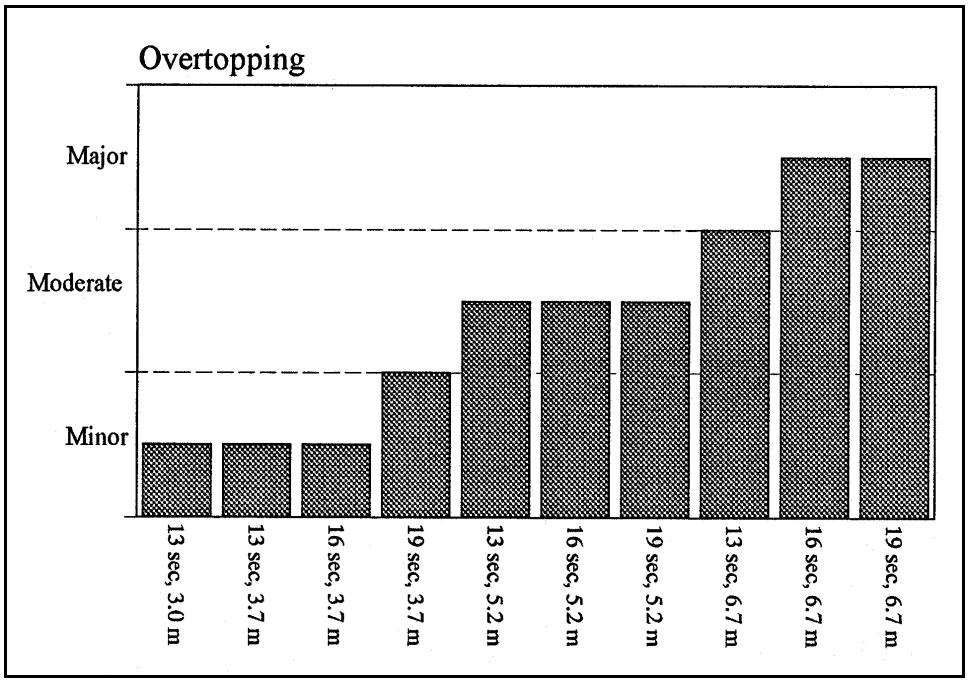

Figure 17. Wave overtopping at trunk, all plans

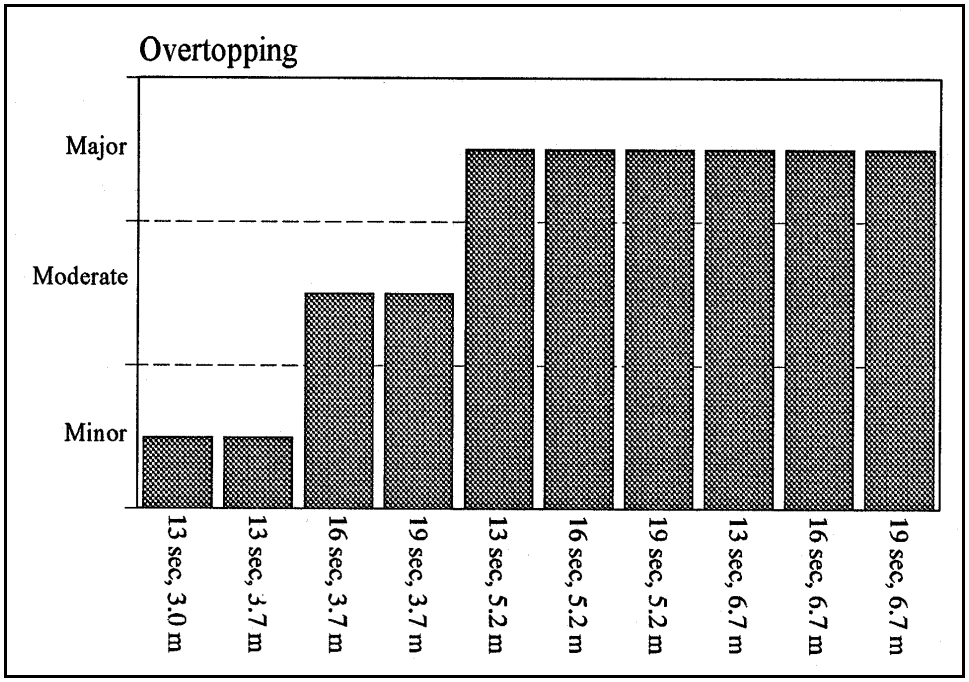

Figure 18. Wave overtopping at breakwater head, plan 1 series waves

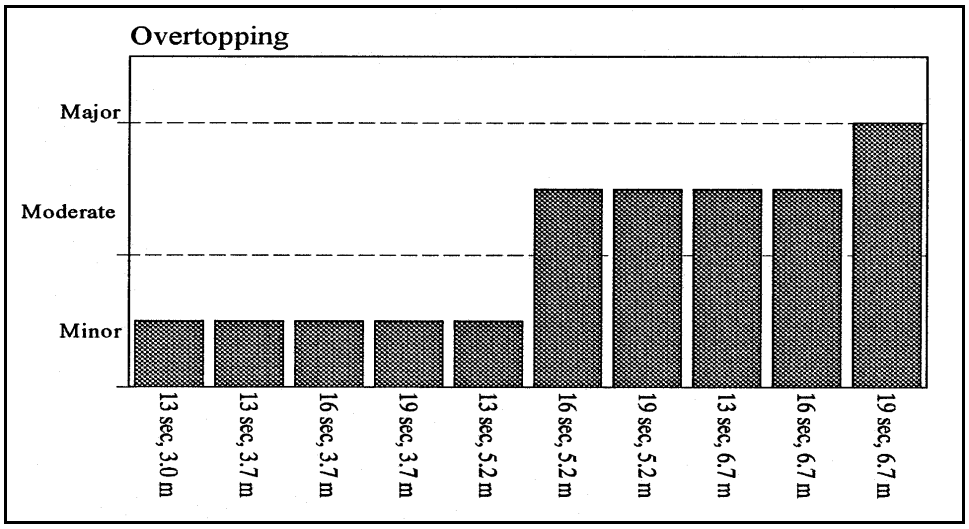

Figure 19. Wave overtopping at breakwater head, plan 2 series waves

Table 4
Summary of Stability Experiments

Plan	Main Features	Figure	Photos
1	Initial Plan Profiles 1 - 4: 6.25-tonne Tetrapods Profiles 4 - 6/7: 8-tonne Core-Locs Profiles 6/7 to head: 11-tonne Core-Locs Crest: +5 m CD south of elbow Packing density (ϕ): 8-tonne Core-Locs - 0.55 11-tonne Core-Locs - 0.59	10	A1 through A6
1A	Same as Plan 1 except: Profiles 4 through 10: Toe Protection: buttress of 0.75- to 1.8-tonne stone ϕ = 0.60 (8-tonne Core-Locs)	11	A7 through A12
1B	Same as Plan 1 except: Toe Protection: chain (Profile 4 to elbow)	12	A13 through A16
1C	Same as Plan 1B except: Toe Protection: anchored chain (Profiles 4 to 11) ϕ = 0.62 (8-tonne Core-Locs)	12	A17 through A22
2	Same as Plan 1C except: Crest elevation raised from +5 m CD to + 8 m CD 11-tonne Core-Locs used on entire structure Toe Protection: Single row of 11-tonne Core-Locs ϕ = 0.62	13	A23 through A27
2A	Same as Plan 2 except: Toe Protection: Toe trench with 45-deg bevel ϕ = 0.61	14	A28 through A33
2B	Same as Plan 2A except: Toe Protection: Toe trench with vertical face ϕ = 0.63	15	A34 through A36
2C	Same as Plan 2A except: Toe protection: Concrete-filled containers spaced 1 m apart (Profile 4 to 70 m north of elbow)	16	A37 through A42
2D	Same as Plan 2C except: Toe protection: Containers spaced end to end ϕ = 0.62	16	A43 through A46

4 Risk Considerations

Wave Conditions

The sponsor-predicted return period of the higher waves generated on the model was 50 years, i.e., the storm event is predicted to occur only once every 50 years. Experiments conducted on the model showed that the design waves were depth-limited and broke seaward of the structure. Therefore, it was concluded that waves exceeding the design condition would also break seaward of the structure and would not increase damage.

The most severe storms approach the site from the northwest and stability experiments were performed only from this direction. However, the structure also is subjected to southern swell; therefore, the entire structure, including the head section, should use 11-tonne armor units and the same toe protection scheme as the breakwater trunk.

Armor Unit Selection

The Core-Loc, a recently developed armor unit (Melby and Turk 1995), was selected for use in the armor layer for the rehabilitation. Construction of the Port Saint Francis breakwater in South Africa and experiments by Smith and Hennington (1995), and Smith (1996) have shown the Core-Loc is reliable and an improvement to the Accropode, which is an armor unit that has been used extensively and successfully for 20 years worldwide. Features of the Core-Loc included improved stability to the Accropode by increasing the porosity of the armor layer; no tendency for units to rock on slope; reserve stability for wave conditions exceeding the design event; hydraulic stability when placed as a repair with other armor shapes (in Vale de Cavaleiros, some tetrapod units will remain in the northern part of the jetty); and low internal stresses. The Core-Loc is presently the most efficient armor unit for rubble-mound breakwaters, because of these properties

Displaced armor units were counted after each wave series to assess the reliability of breakwater protection. Several units were displaced during the initial plans, but only one Core-Loc was displaced over the entire breakwater for

the final plan, Plan 2B, which included a 1.5-m-high toe trench. One displaced unit is low and does not indicate any endangerment to the structure.

5 Summary

A three-dimensional physical model study was conducted to test stability of the proposed breakwater rehabilitation at Vale de Cavaleiros. The direction of storm waves was 300 deg from the north. The series of waves generated on the model was equivalent to a 17-hr storm prototype. The storms initiated with moderate waves of 3 m and were incrementally increased in height up to a depth-limited height of 6.7 m.

Results

Results of the model study indicated:

a. The armor units selected for the original design were stable if the toe was stable. Plan 1C, which included 8- and 11-tonne Core-Locs, and bundled steel chain to anchor the breakwater toe, was stable during original and repeat tests. However, it was noted that the 8-tonne units rocked in place during tests and were considered moderately stable. Subsequent tests included 11-tonne Core-Locs on the entire structure, but a constructable prototype toe anchor was still required.

b. Plans 2A and 2B included a board anchored to the model floor at the base of the breakwater to stabilize the toe. Analogous results would be expected if a toe trench was used to fix the toe. The board used in Plan 2A was 1 m deep and included a 45-deg angle adjacent to the toe units. The sloped trench was stable for waves up to 16 sec, 5.2 m but significant damage occurred between Profiles 8 and 9 for 16-sec, 6.7-m waves. The trench simulated in Plan 2B was 1.5 m high and had a vertical face. Plan 2B was stable for original and repeat tests; one unit was displaced during the repeat tests, but the structure remained stable.

c. Different toe reinforcement schemes such as a stone buttress of 0.75- to 1.8-tonne stone (Plan 1A), 11-tonne Core-Locs (Plan 2) placed at the toe, and concrete-filled cargo containers (Plans 2C and 2D) placed at the toe were tested, but were unsuccessful in stabilizing the toe.

Results from the three-dimensional stability tests indicated the most stable plan was Plan 2B, which consisted entirely of 11-tonne Core-Locs, a constant crest elevation of +8 m CD, and a vertical-face toe trench 1.5 m high.

Toe Trench Construction

For the conditions tested in the model, the breakwater was not damaged if a stable toe trench, 1.5 m deep and near vertical, was installed. The model tests were conducted on a fixed bottom, which in nature would be analogous to a smooth rocky bottom in the prototype. An example of a prototype toe trench constructed in a rocky bottom is shown in Figure 20. The seaward face of the trench should be as near vertical as possible.

No model tests were conducted on stability with a movable bed; therefore, it was not possible to quantify the effects of a sandy bottom on the stability of the toe trench because the model floor was fixed. For structures placed in shallow water the *Shore Protection Manual* (1984) recommends a toe protection scheme similar to Figure 21, in which a wide trench is constructed and replaced with armor. The *Shore Protection Manual* suggests constructing the trench horizontally 2 times the water depth or 2 to 3 times the design wave height for the most severe scour.

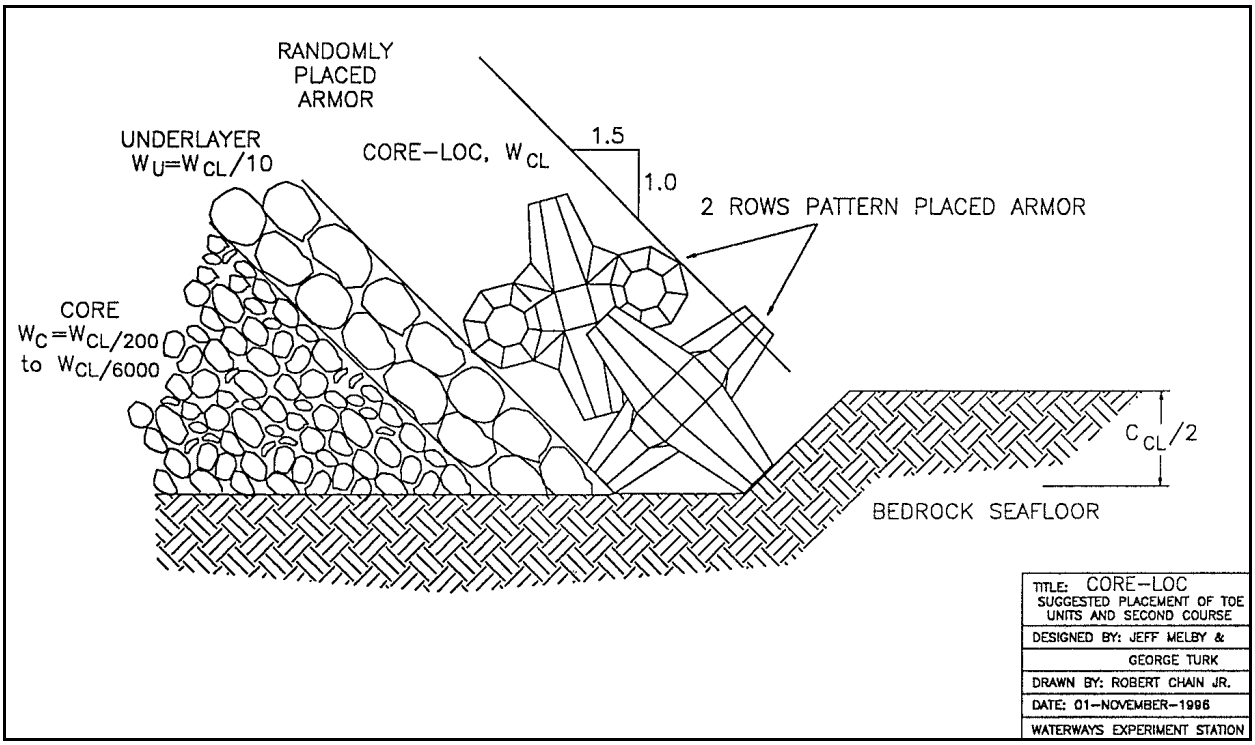

Figure 20. Example of toe trench constructed on rocky bottom

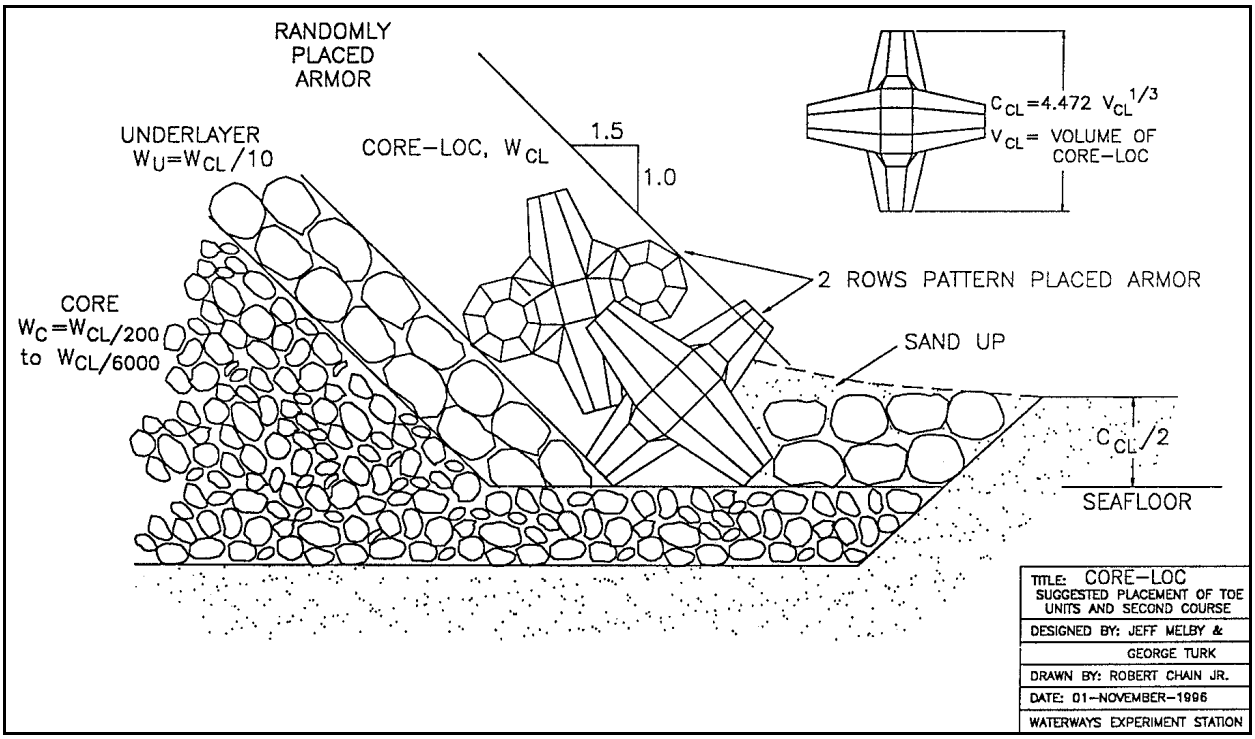

Figure 21. Example of toe trench constructed on sandy bottom

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Appendix A
Photographs

Photo A1. Sea-side view of plan 1 before testing

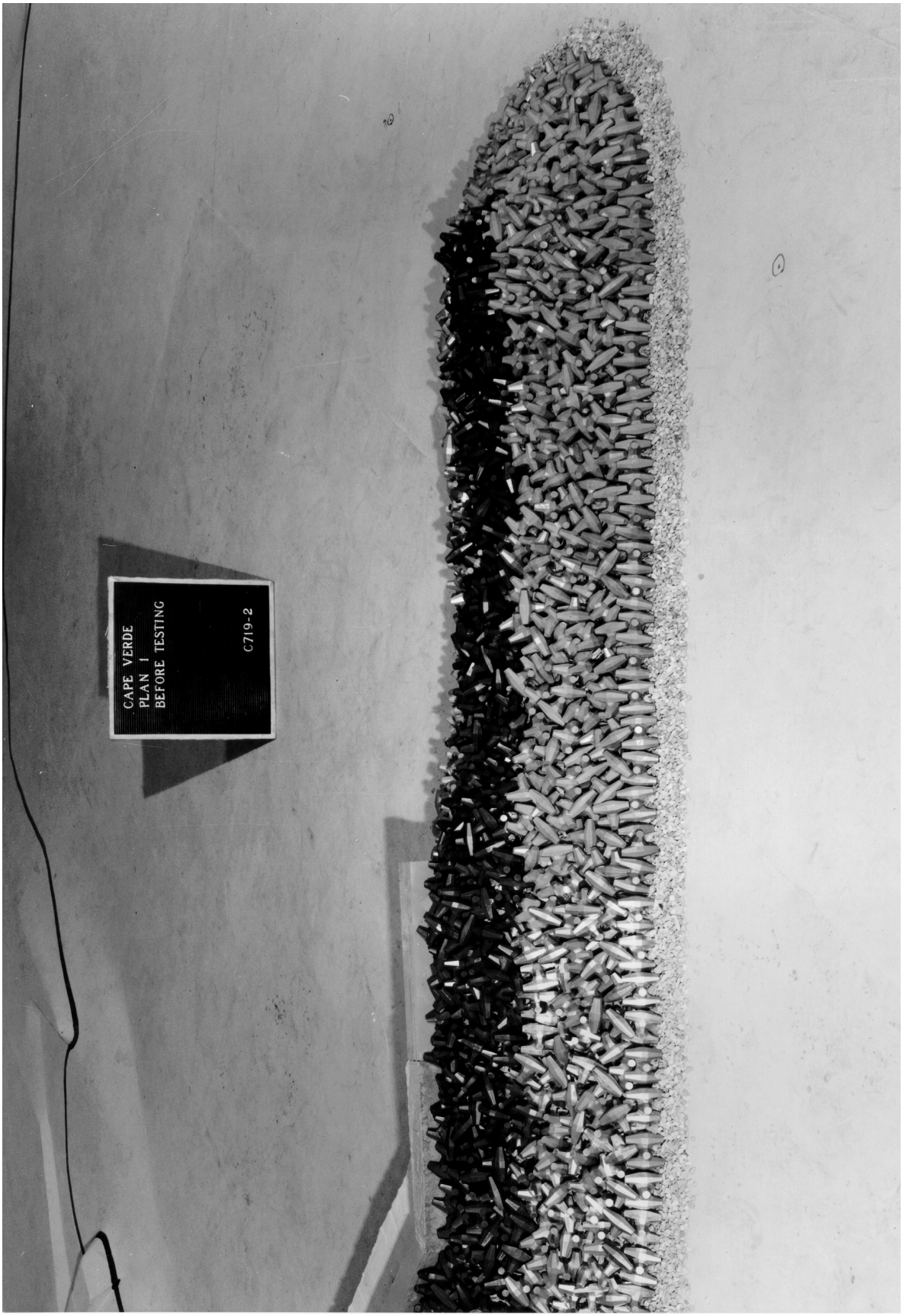

Photo A2. Sea-side view of plan 1 breakwater head before testing

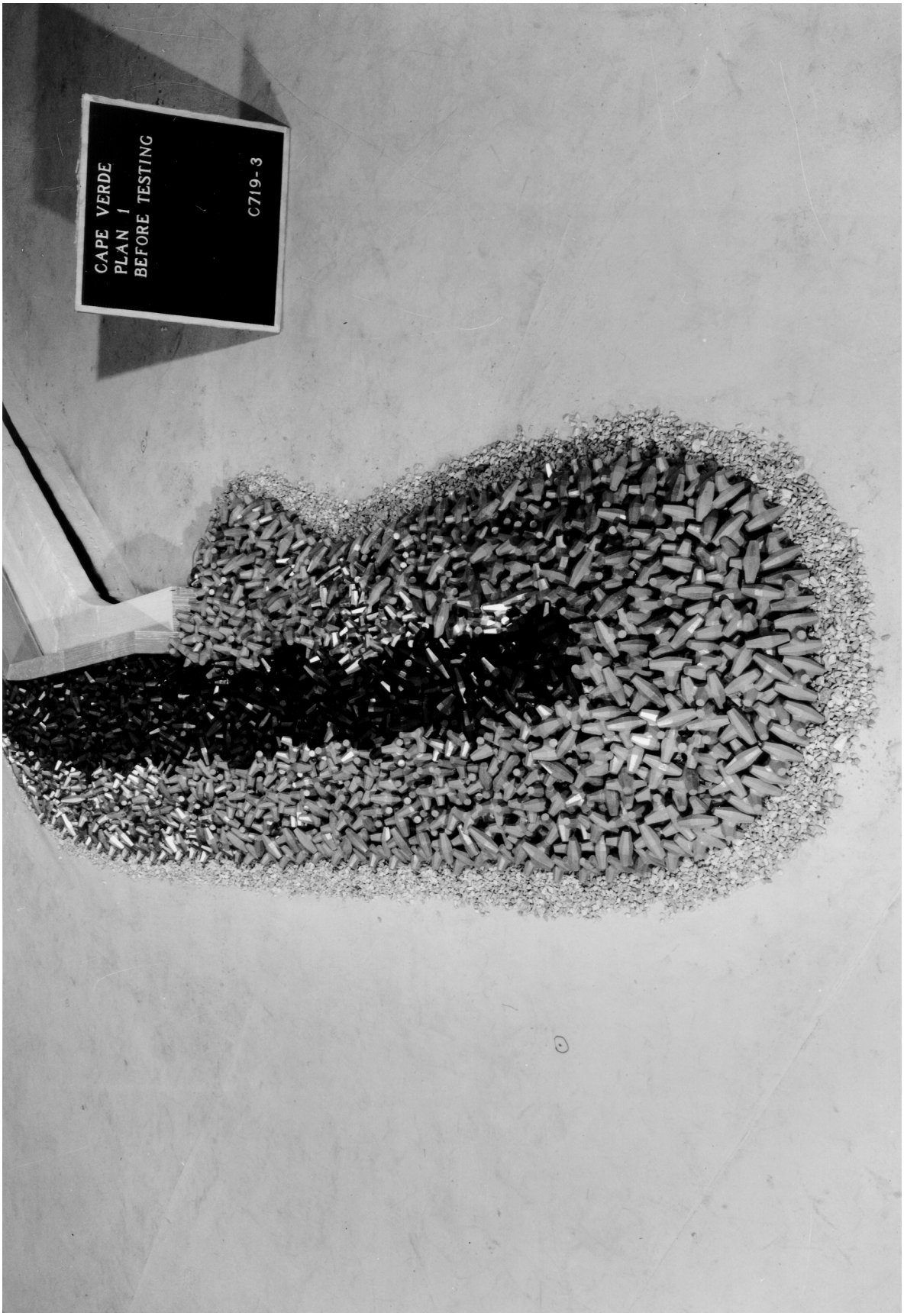

Photo A3. View of plan 1 breakwater head before testing

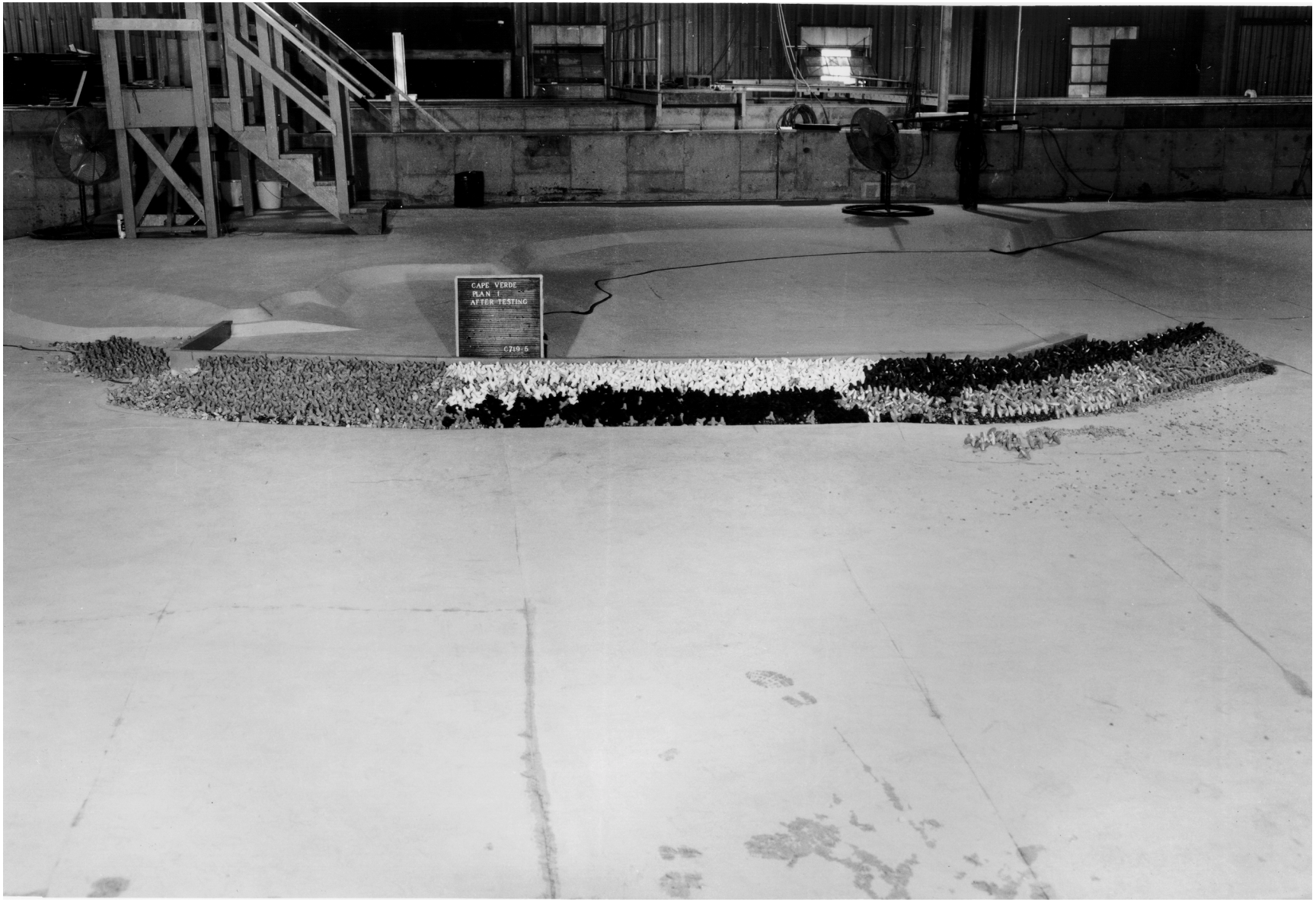

Photo A4. Seaside view of plan 1 after testing with 13-sec, 3-m waves

Photo A5. Sea-side view of plan 1 breakwater head after testing with 13-sec, 3-m waves

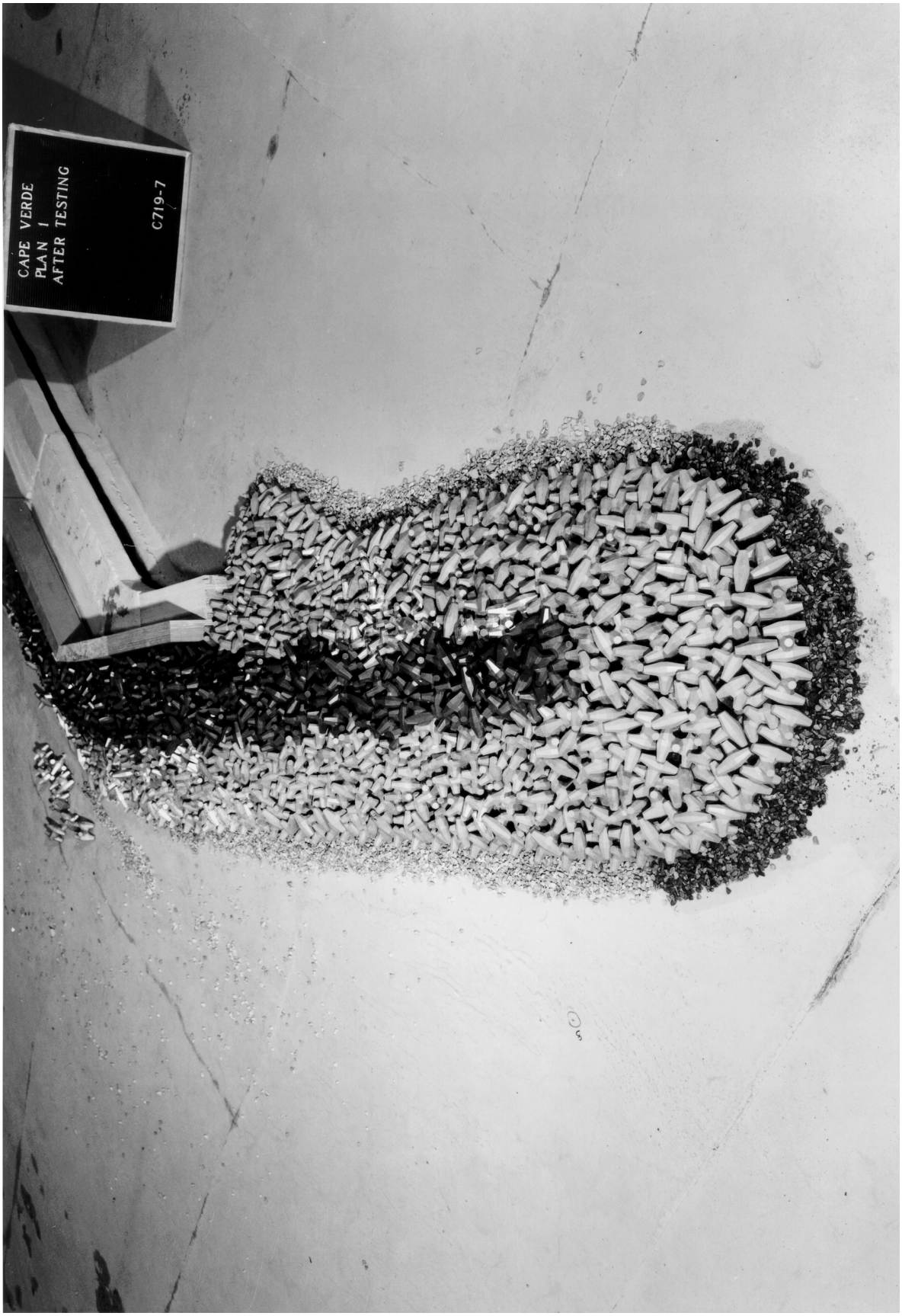

Photo A6. View of plan 1 breakwater head after testing with 13-sec, 3-m waves

Photo A7. Sea-side view of plan 1A before testing

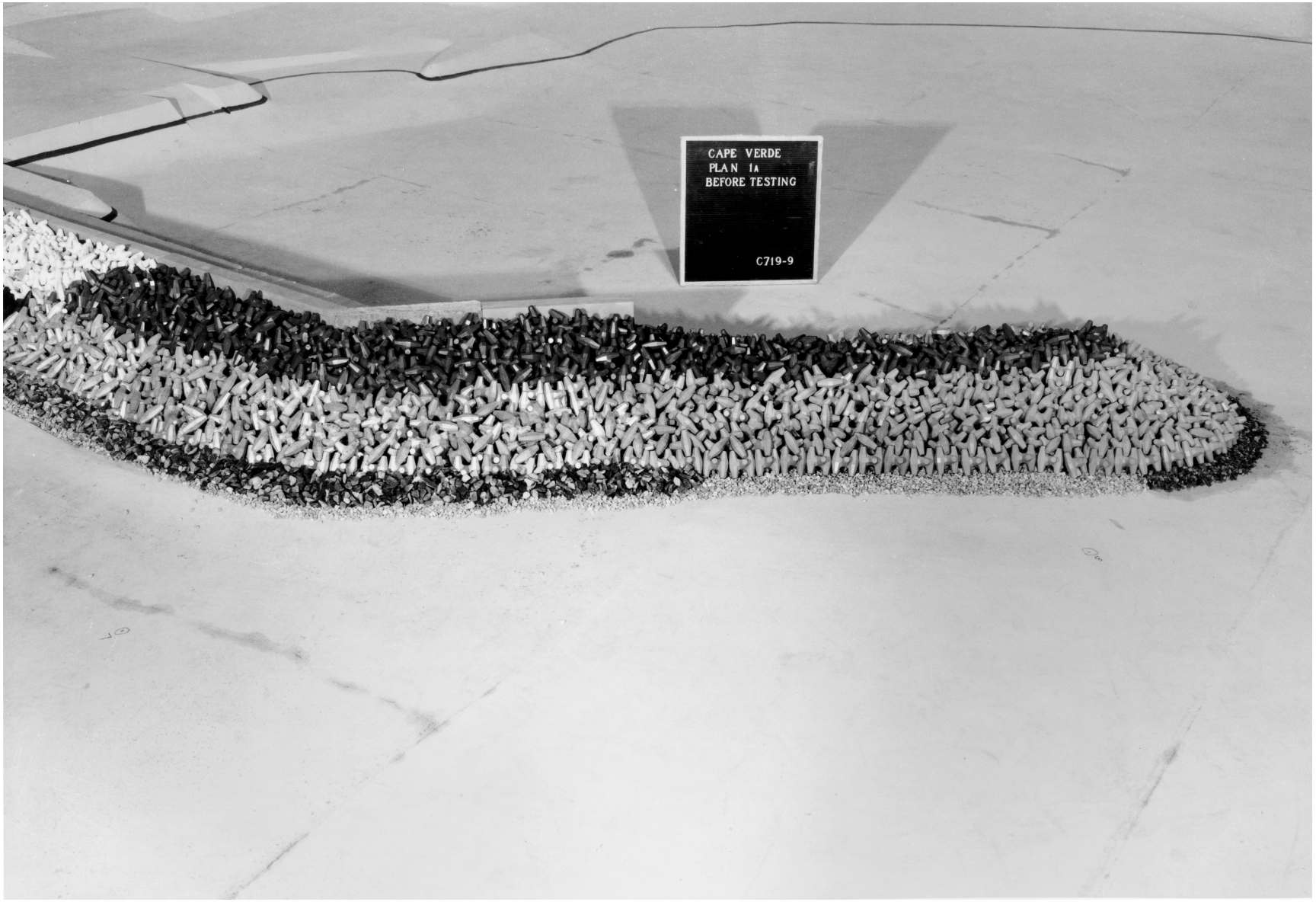

Photo A8. Sea-side view of plan 1A breakwater head before testing

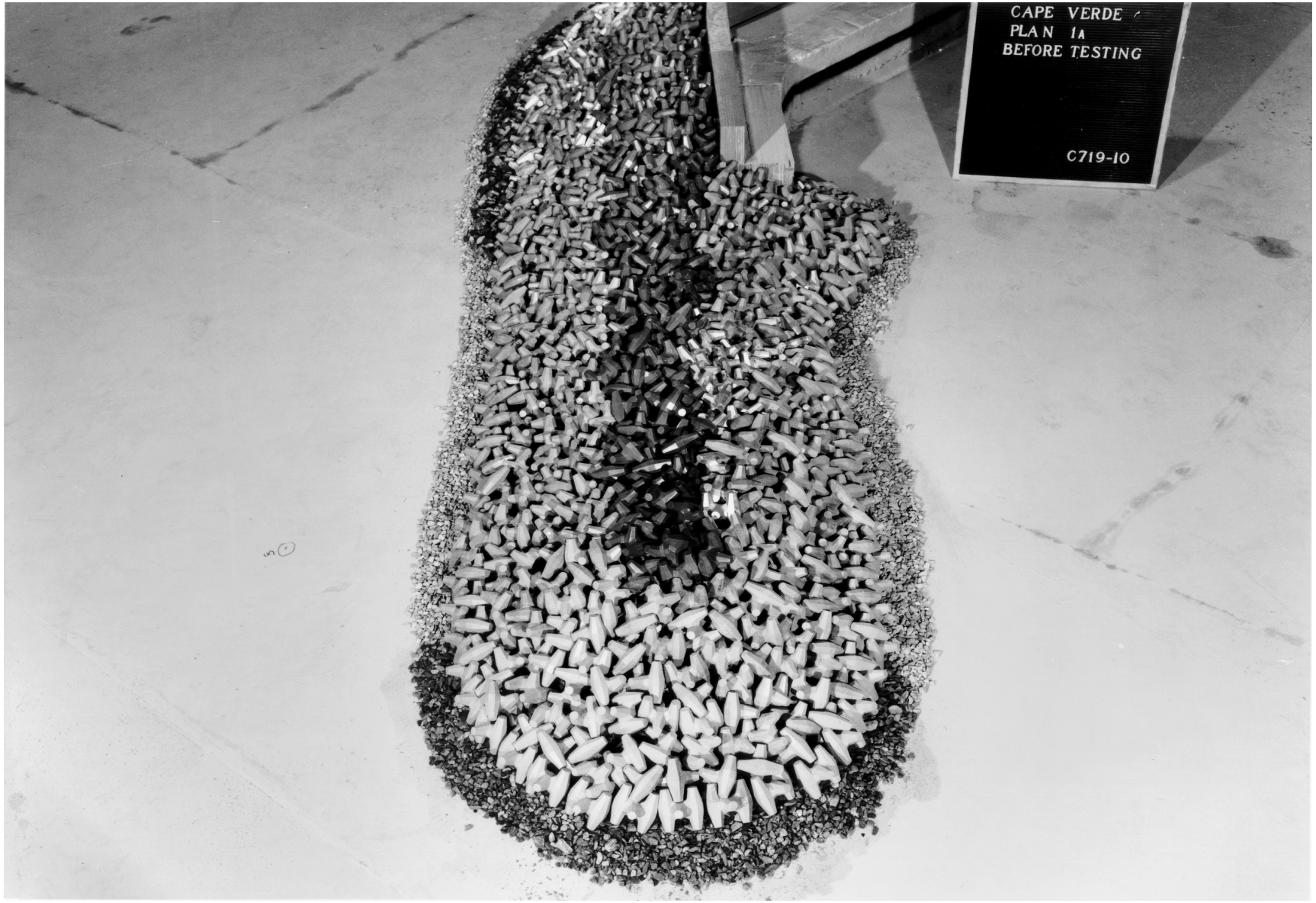

Photo A9. View of plan 1A breakwater head before testing with 16-sec, 3.7-m waves

Photo A10. Sea-side view of plan 1A after testing

Photo A11. Sea-side view of plan 1A breakwater head before testing with 16-sec, 3.7-m waves

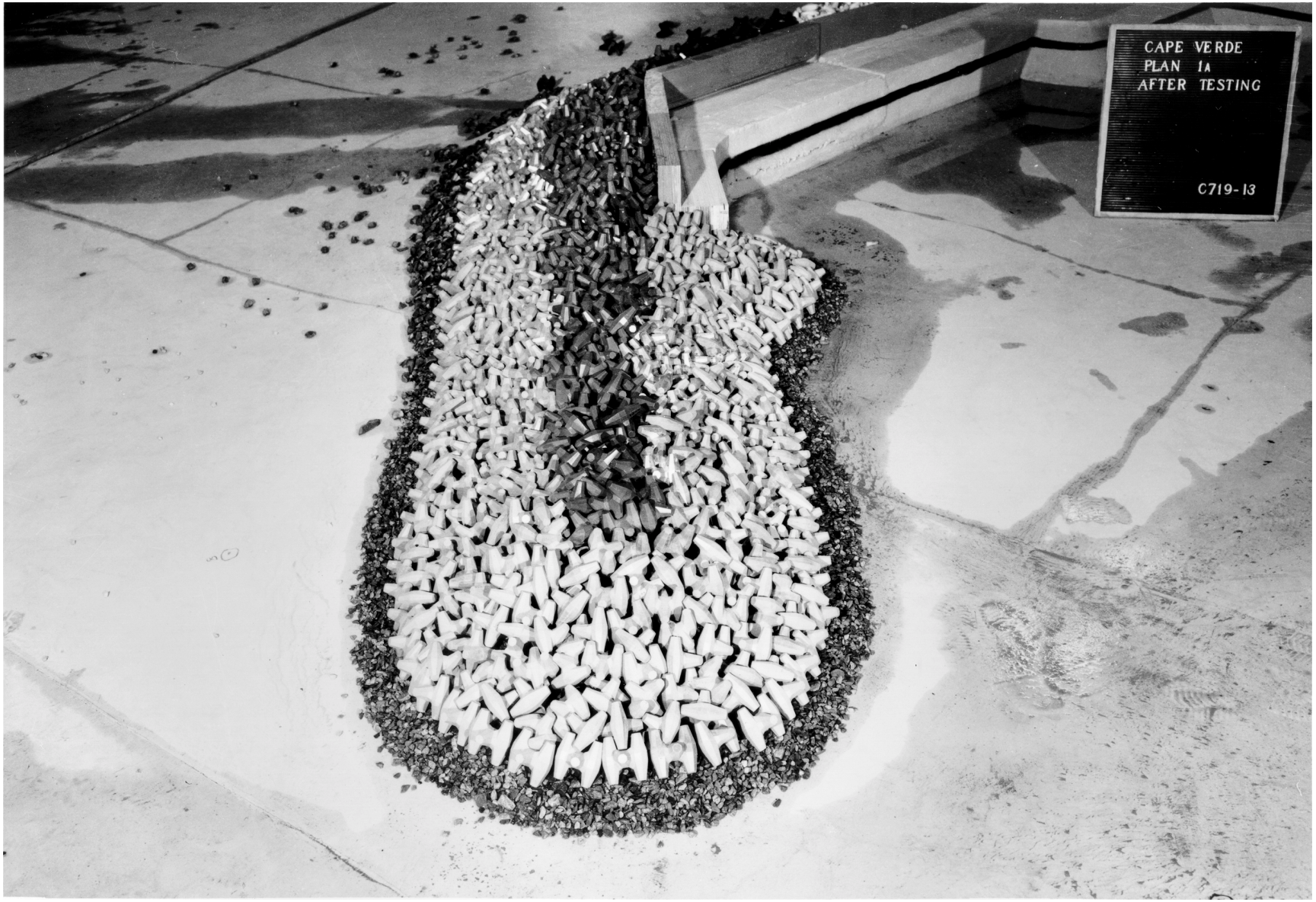

Photo A12. View of plan 1A breakwater head before testing with 16-sec, 3.7-m waves

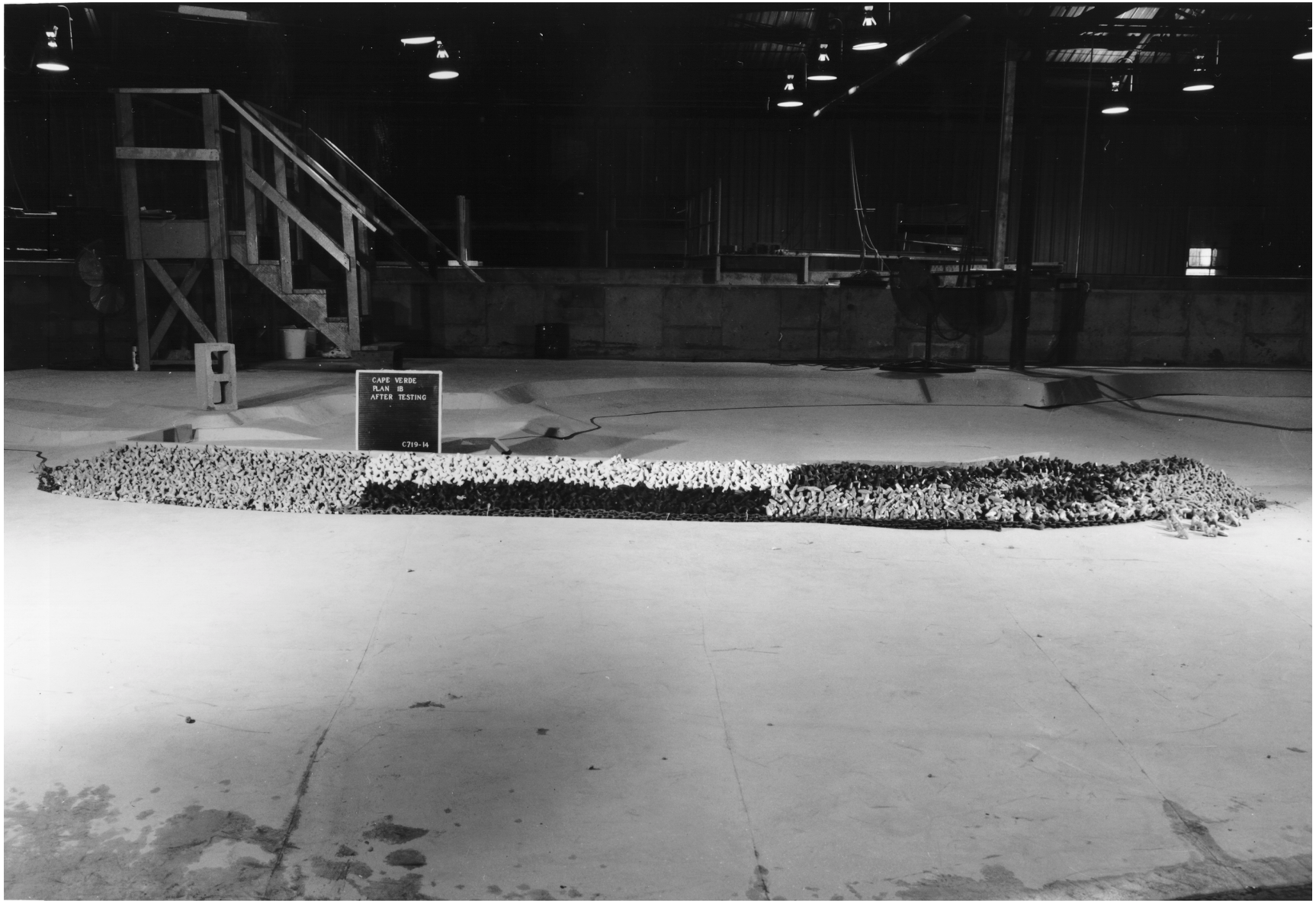

Photo A13. Sea-side view of plan 1B after testing with 17-sec, 5.2-m waves

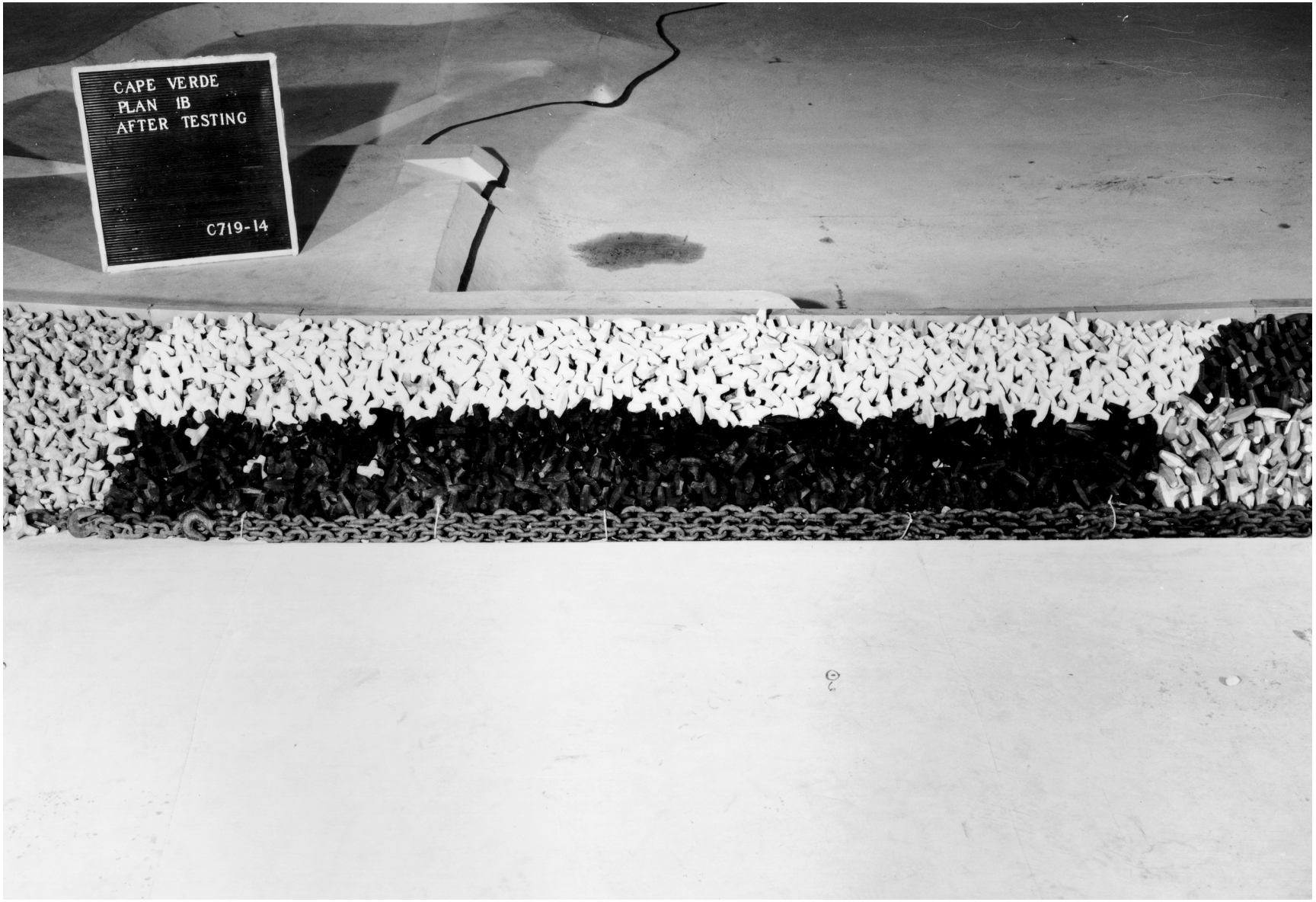

Photo A14. Sea-side view of plan 1B, profiles 4 through 7, after testing with 17-sec, 5.2-m waves

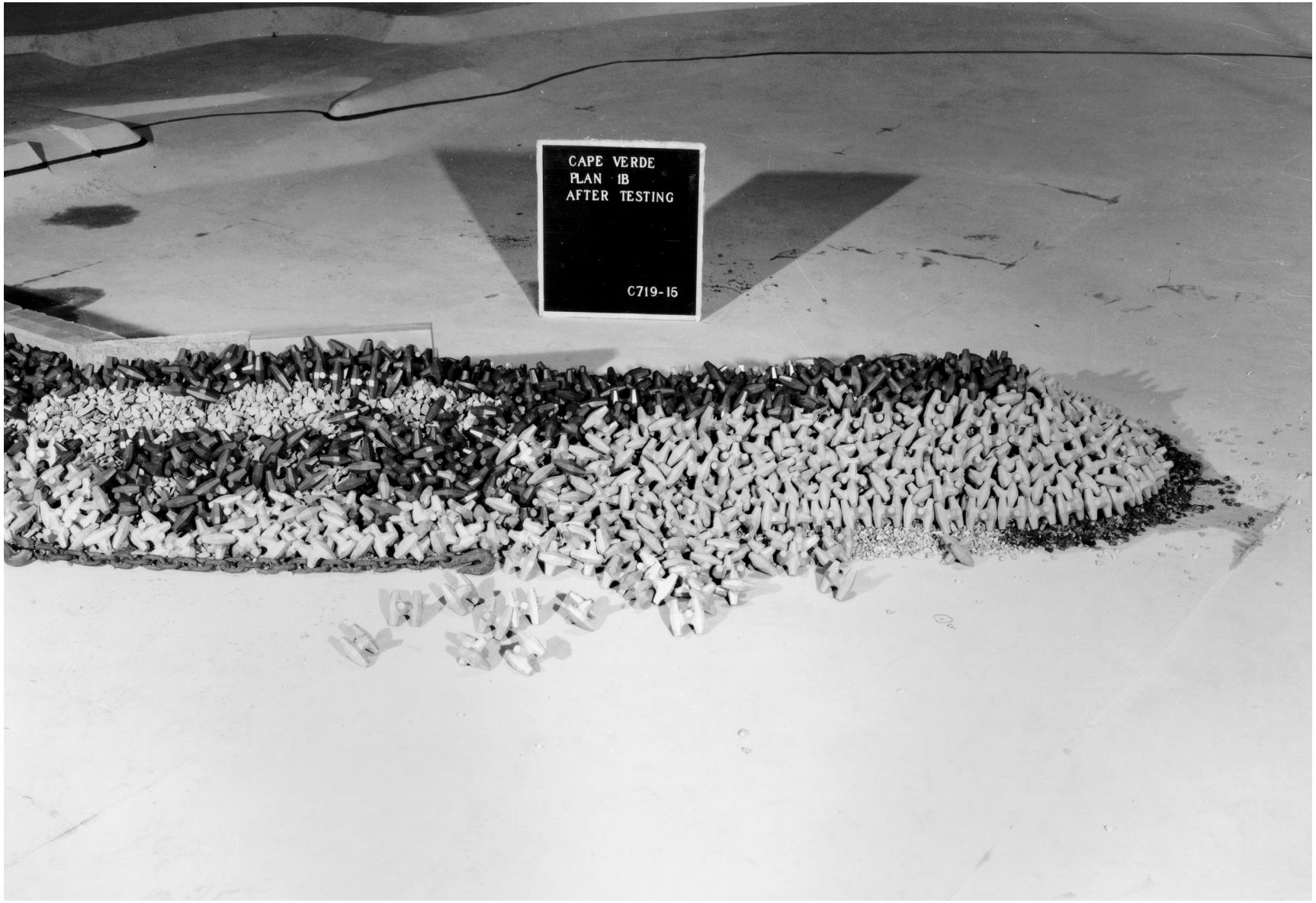

Photo A15. Sea-side view of plan 1B breakwater head before testing with 17-sec, 5.2-m waves

Photo A16. View of plan 1B breakwater head before testing with 17-sec, 5.2-m waves

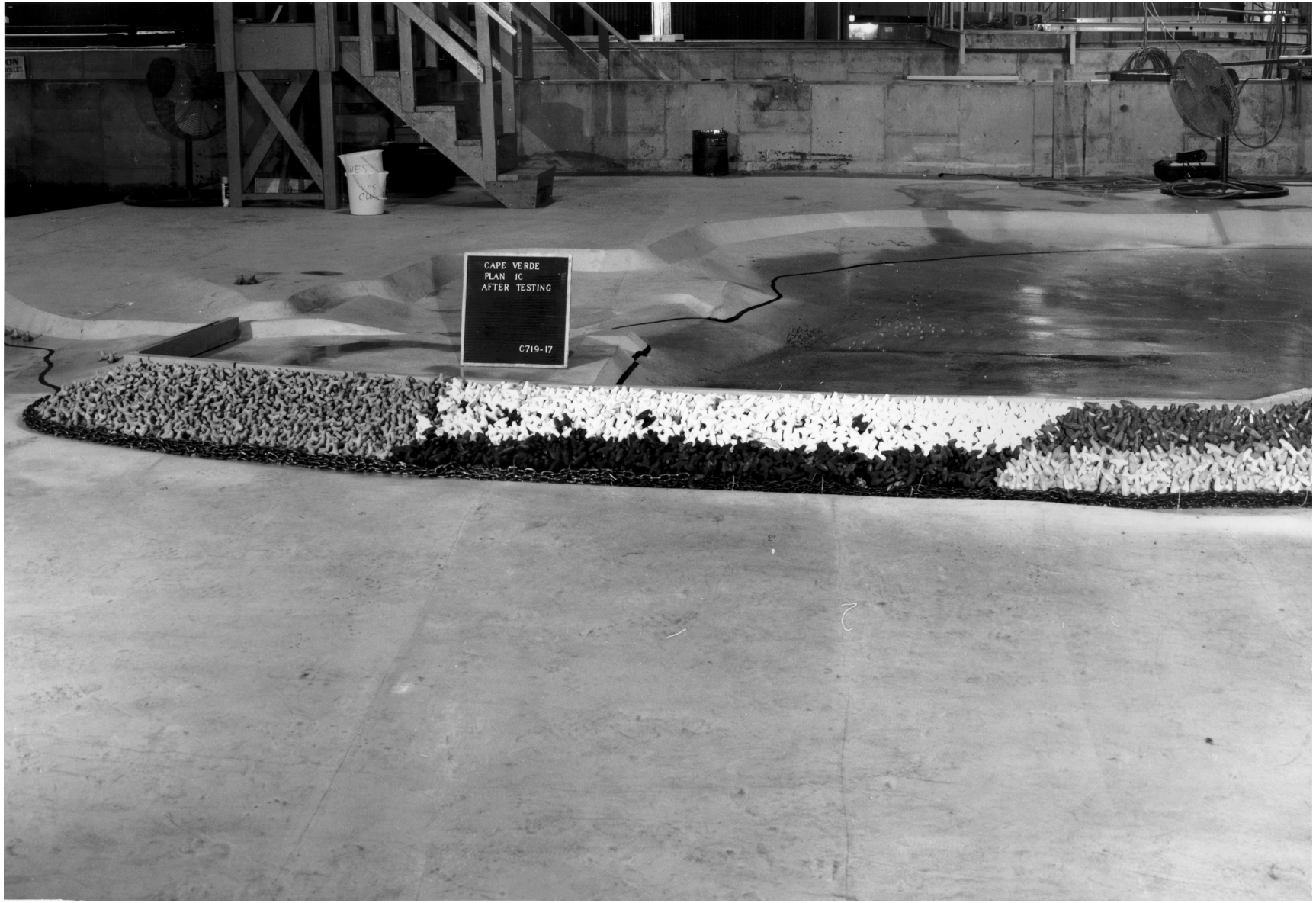

Photo A17. Sea-side view of plan 1C after testing with 19-sec, 6.7-m waves

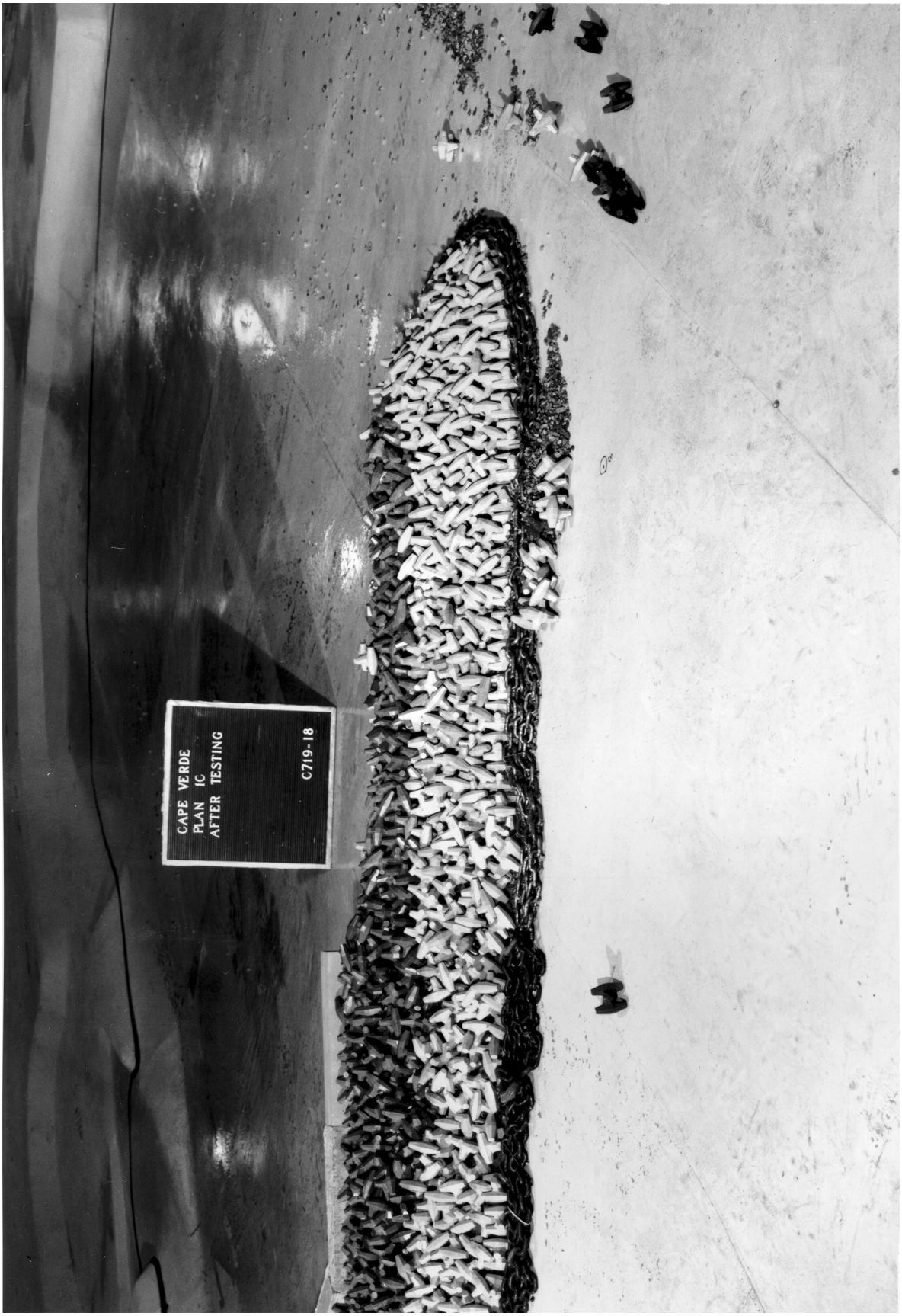

Photo A18. Sea-side view of plan 1C breakwater head before testing with 19-sec, 6.7-m waves

Photo A19. View of plan 1C breakwater head after testing with 19-sec, 6.7-m waves

Photo A20. Sea-side view of plan 1C after repeat testing with total storm sequence

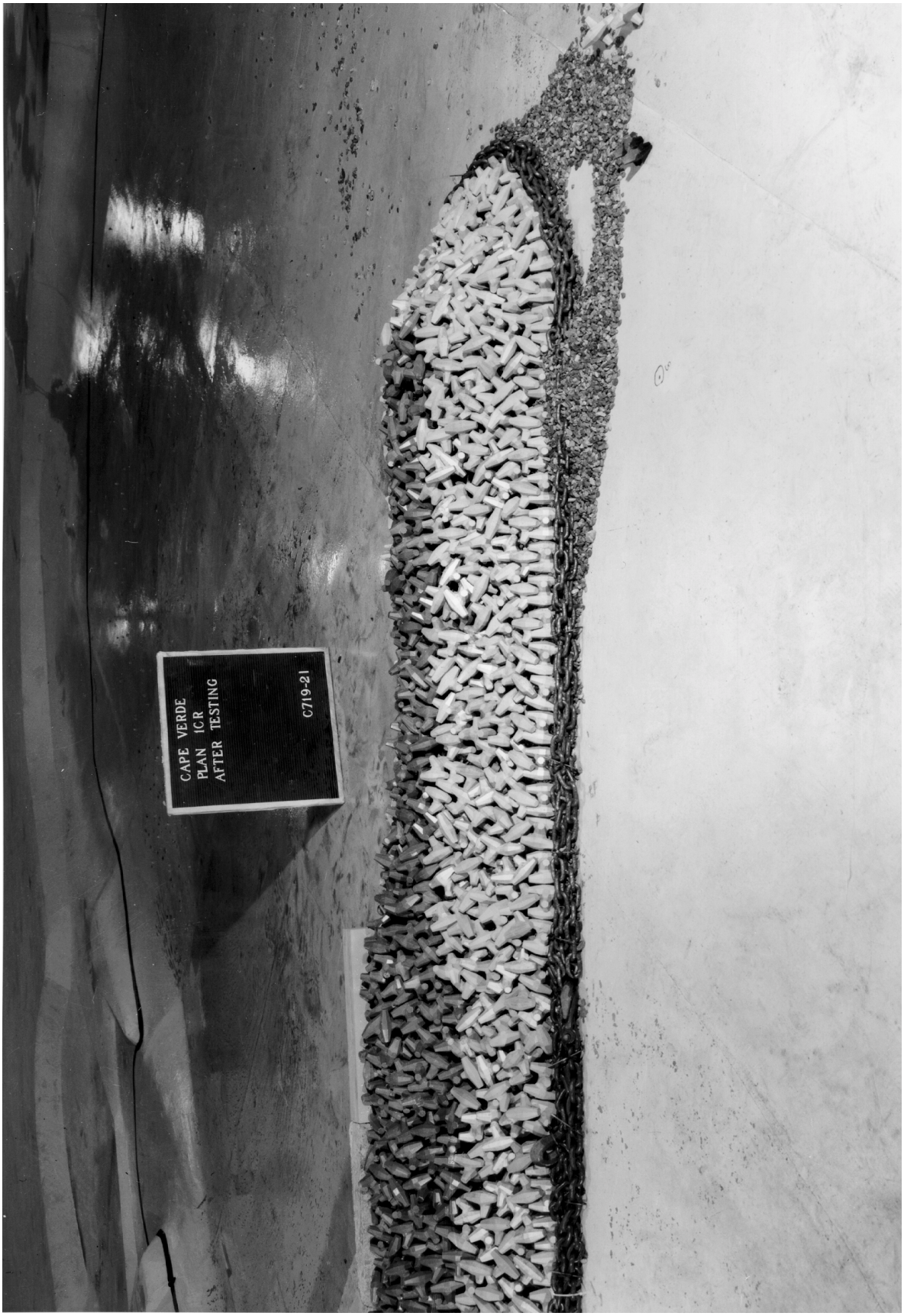

Photo A21. Sea-side view of plan 1C breakwater head after repeat testing with total storm sequence

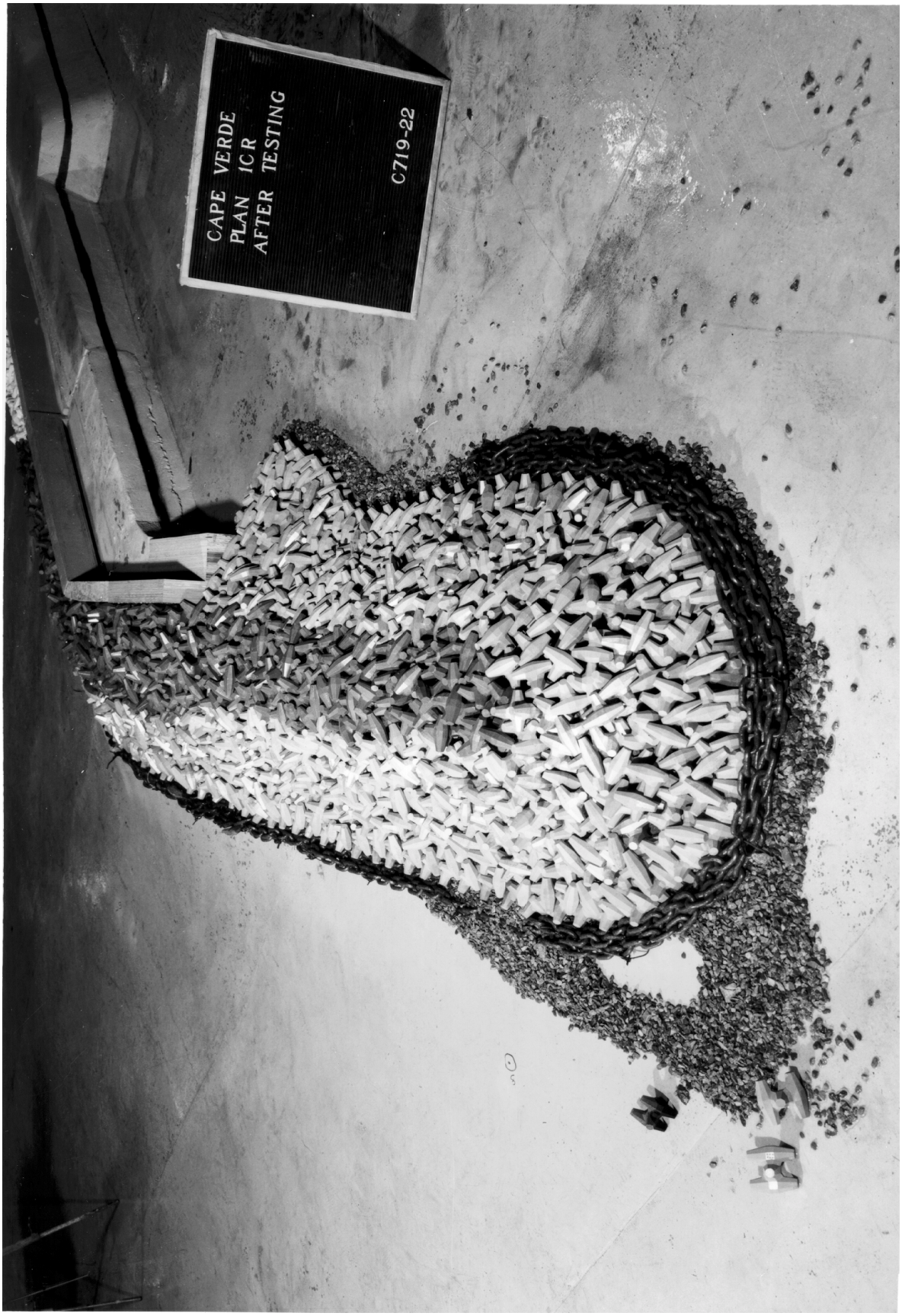

Photo A22. View of plan 1C breakwater head after repeat testing with total storm sequence

Photo A23. Sea-side view of plan 2 breakwater trunk before testing

Photo A24. Sea-side view of plan 2 breakwater head before testing

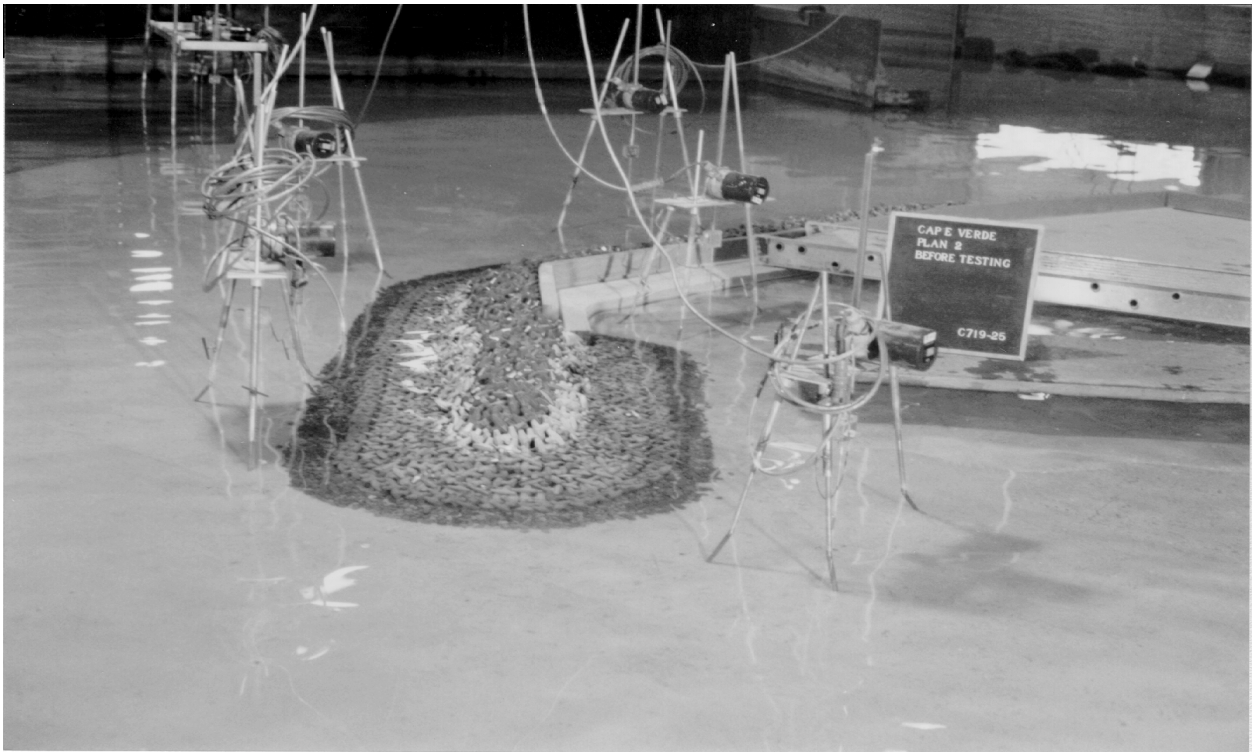

Photo A25. View of plan 2 breakwater head before testing

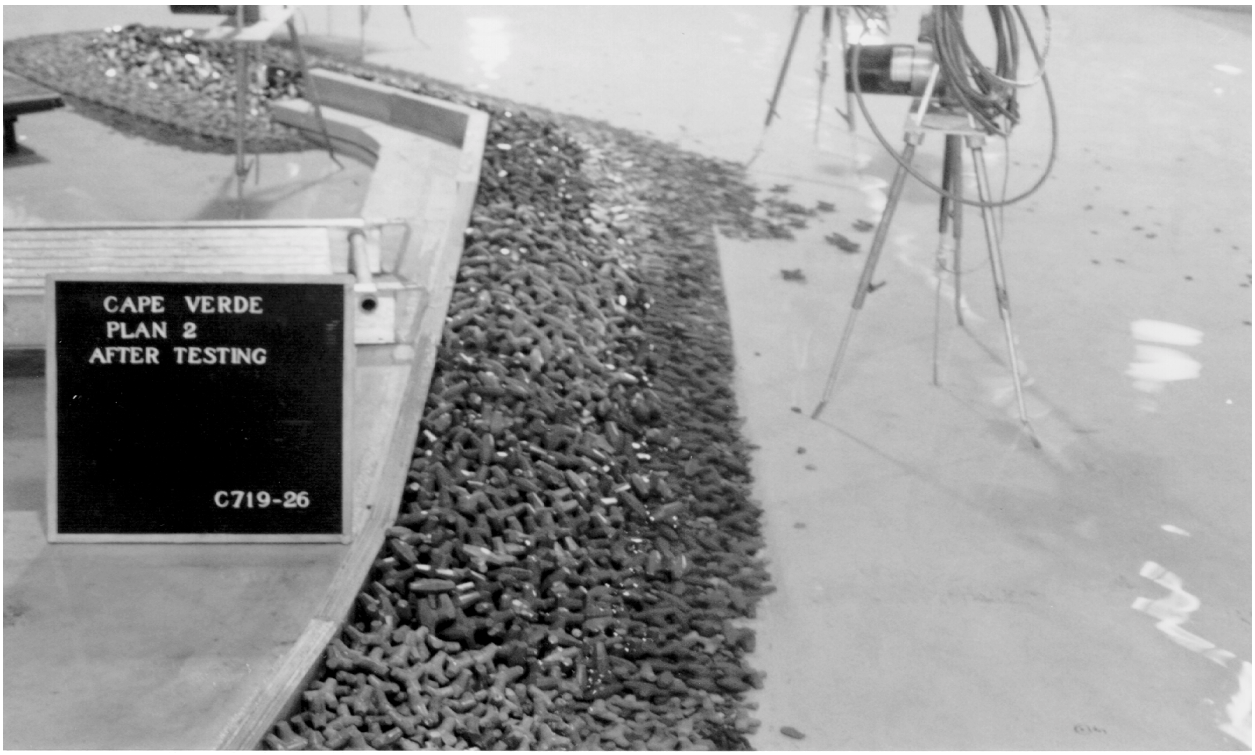

Photo A26. View from north of plan 2 breakwater trunk after testing with 13- and 16-sec, 3.7-m waves

Photo A27. View of plan 2 breakwater trunk, profiles 6 to 8, after testing with 15-sec and 16-sec waves

Photo A28. Sea-side view of plan 2A breakwater trunk before testing

Photo A29. Sea-side view of plan 2A breakwater head before testing

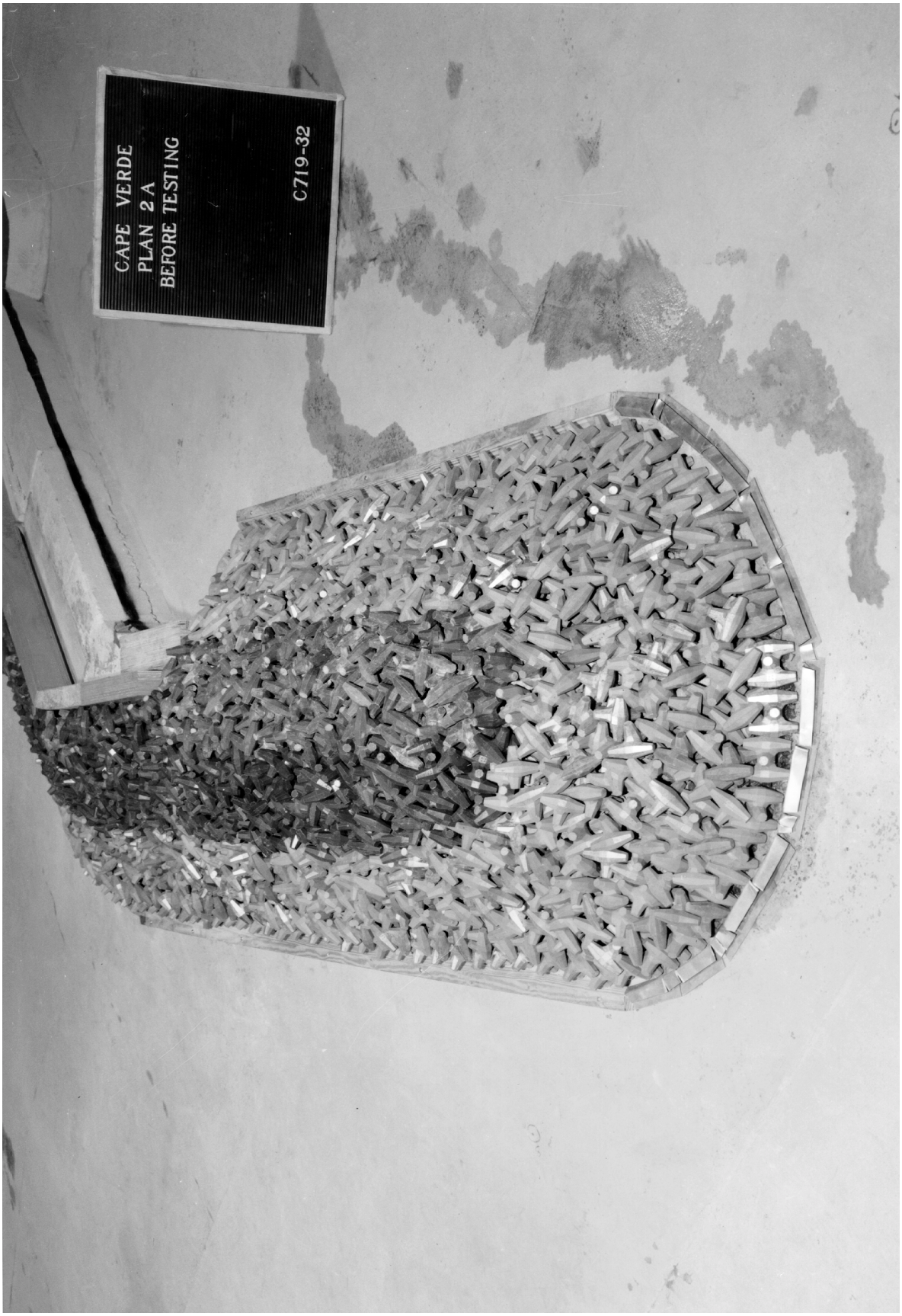

Photo A30. View of plan 2A breakwater head before testing

Photo A31. Sea-side view of plan 2A breakwater trunk after testing with total storm sequence

Photo A32. Sea-side view of plan 2A breakwater head after testing with total storm sequence

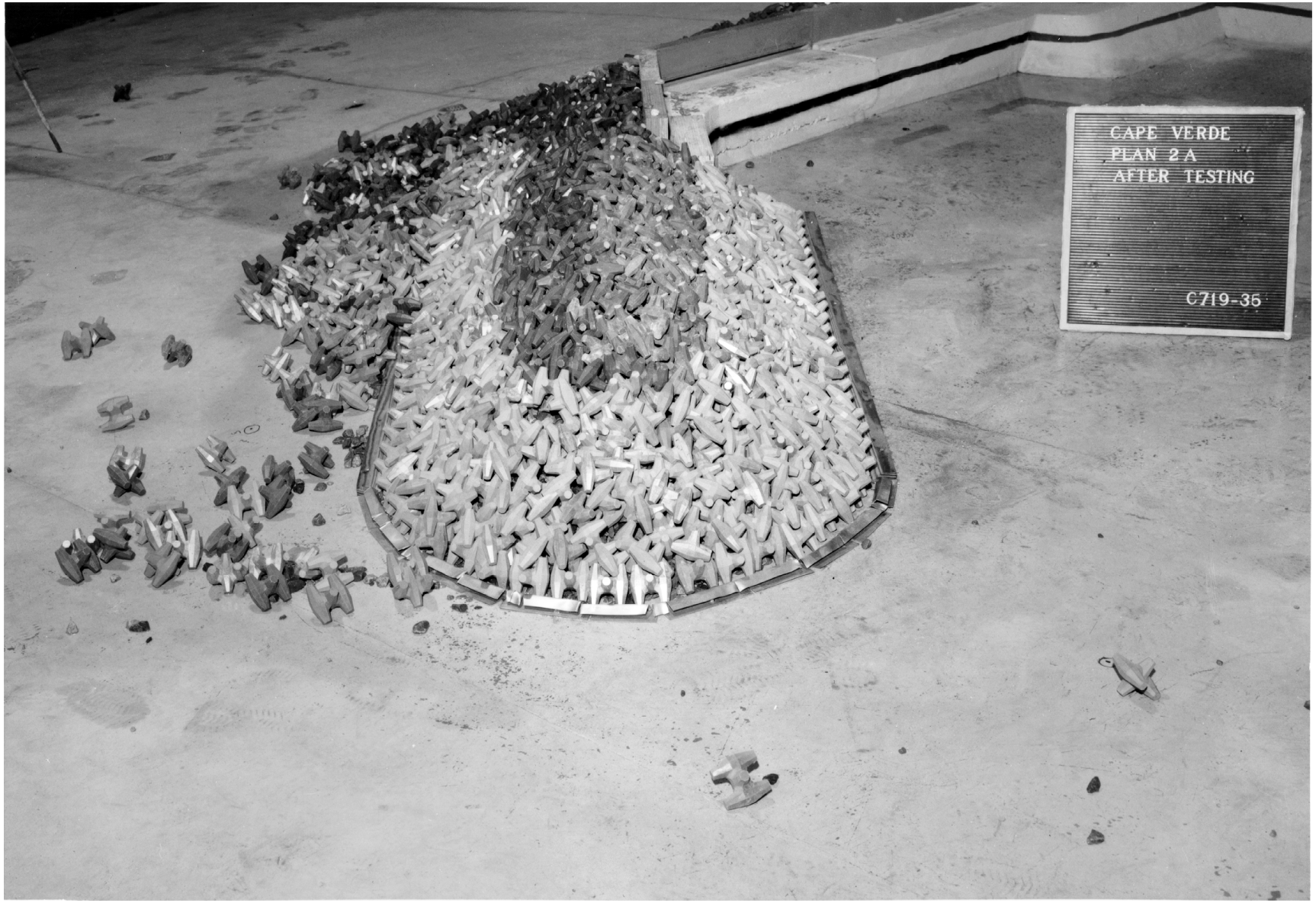

Photo A33. View of plan 2A breakwater head after testing with total storm sequence

Photo A34. Sea-side view of plan 2B breakwater trunk after testing with total storm sequence

Photo A35. Sea-side view of plan 2B breakwater after testing with total storm sequence

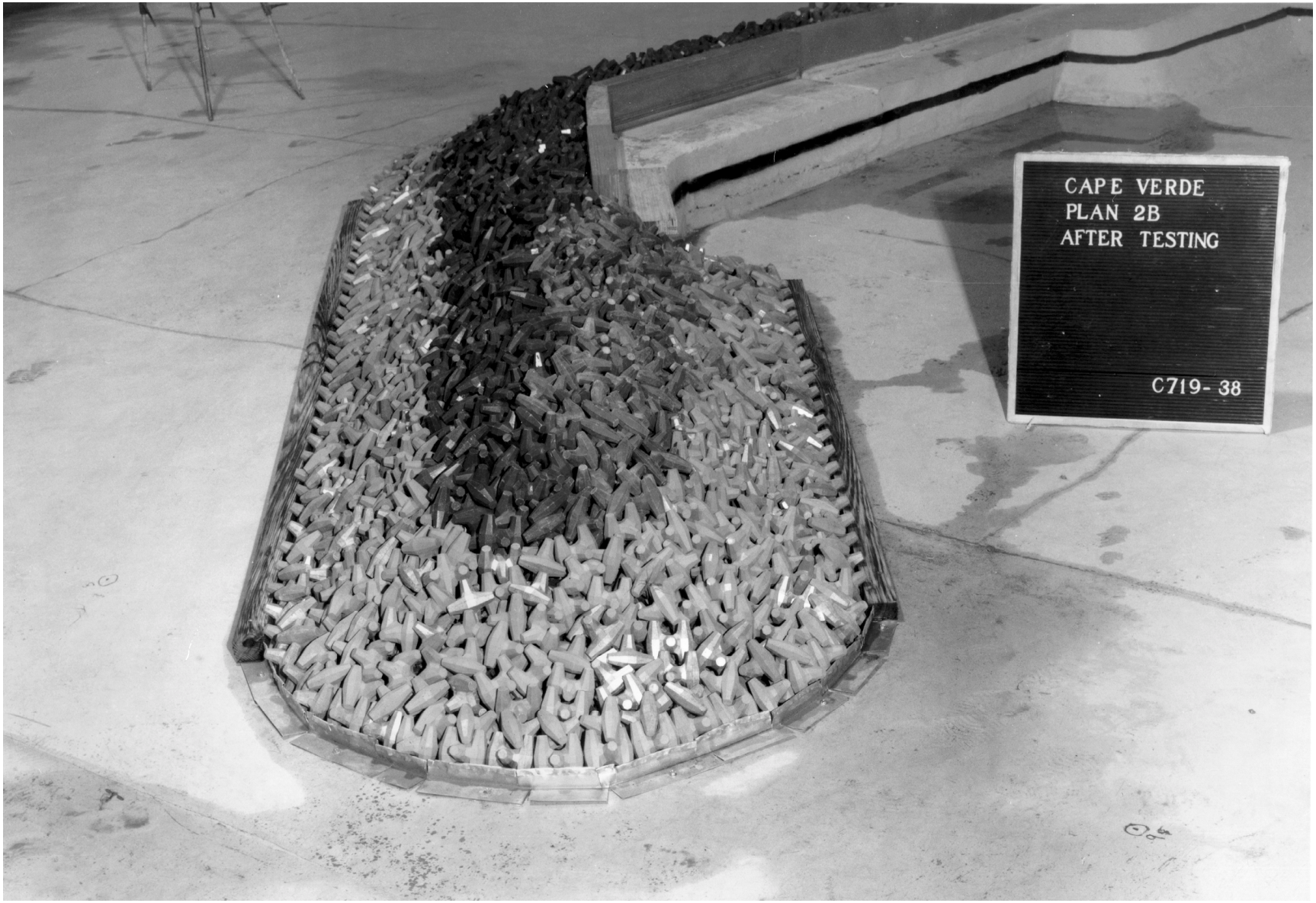

Photo A36. View of plan 2B breakwater head after testing with total storm sequence

Photo A37. Sea-side view of plan 2C breakwater trunk before testing

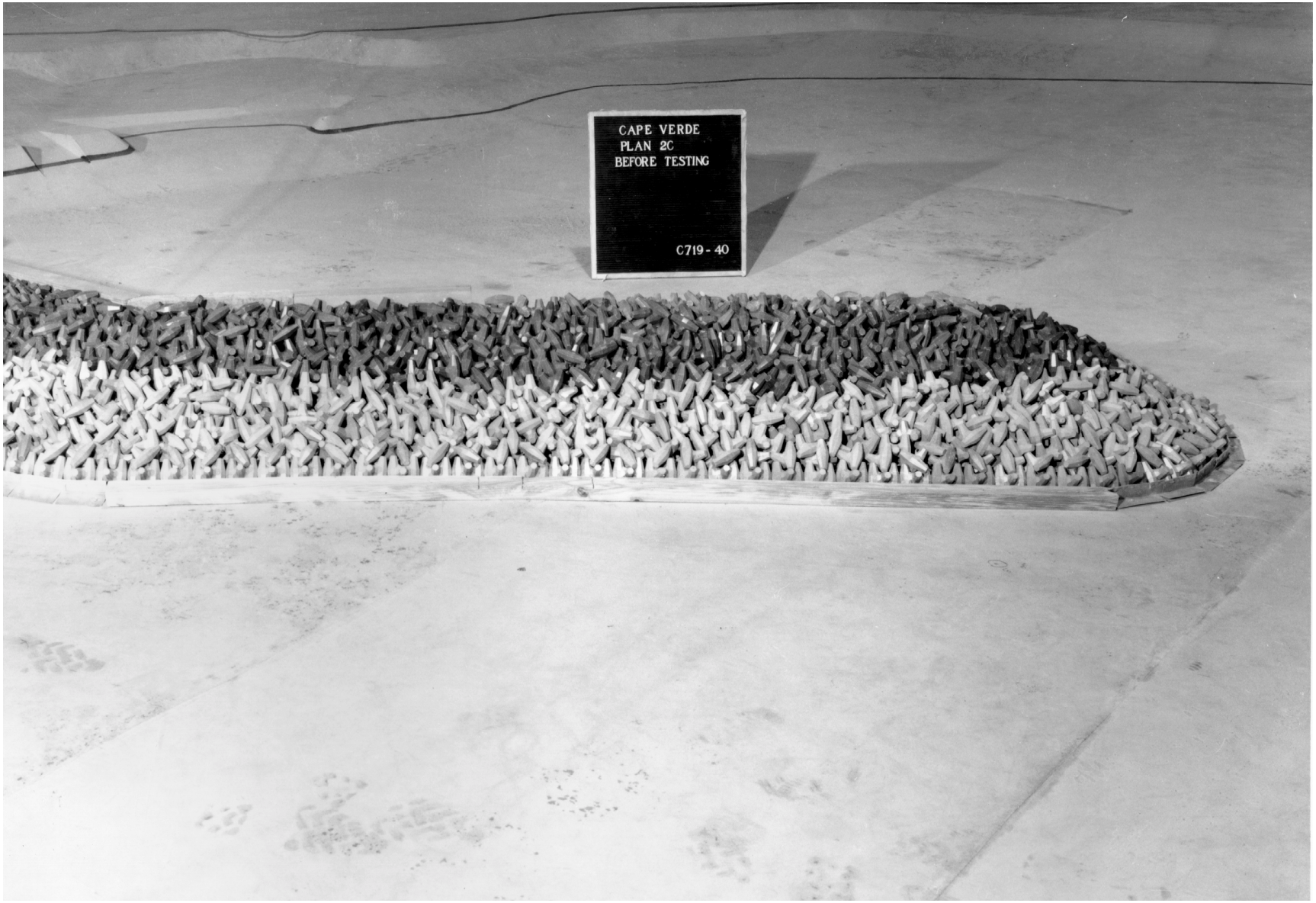

Photo A38. Sea-side view of plan 2C breakwater head before testing

Photo A39. View of plan 2C breakwater head before testing

Photo A40. Sea-side view of plan 2C breakwater trunk after testing with waves above the 3.7-m height

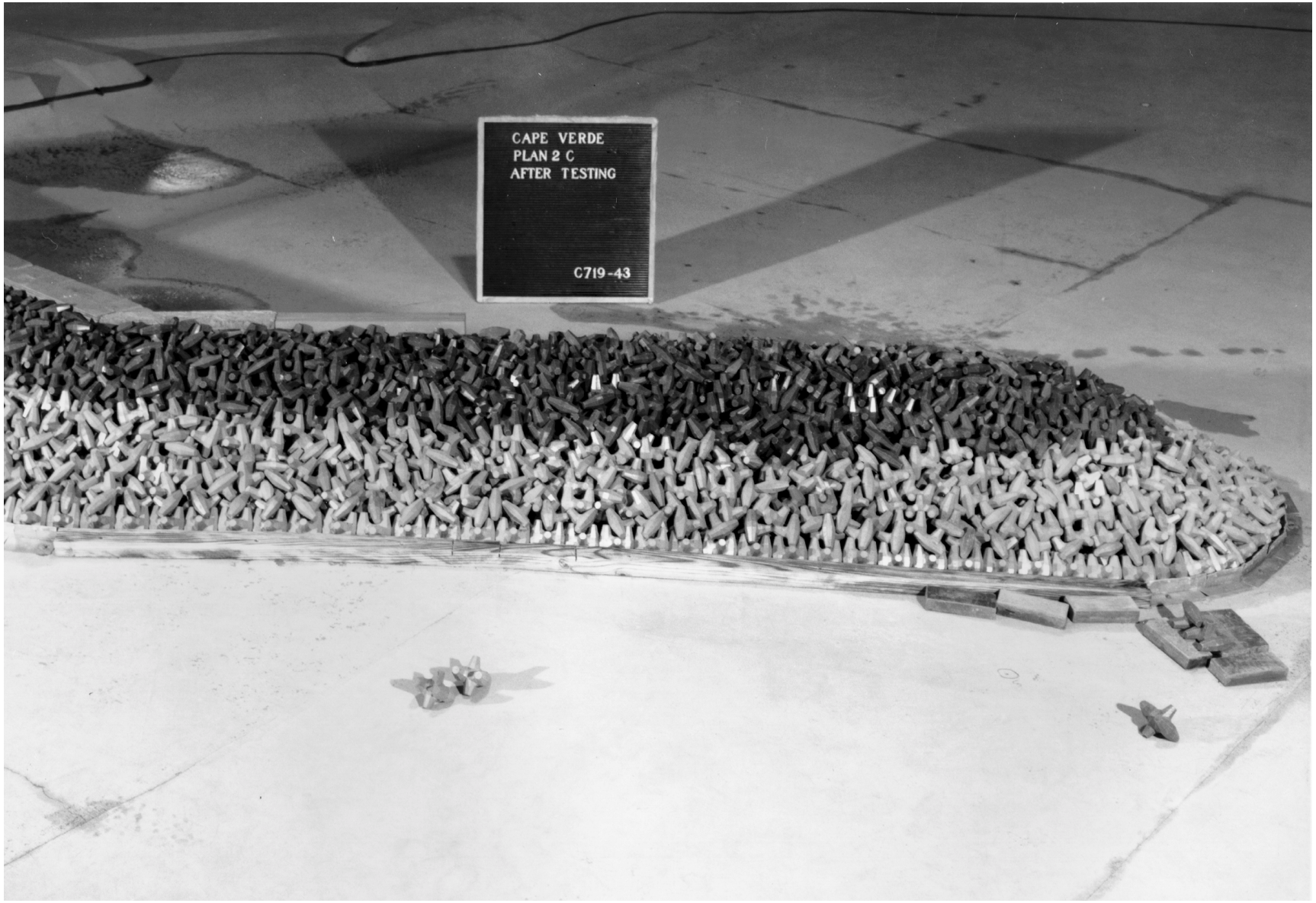

Photo A41. Sea-side view of plan 2C breakwater head after testing with waves above the 3.7-m height

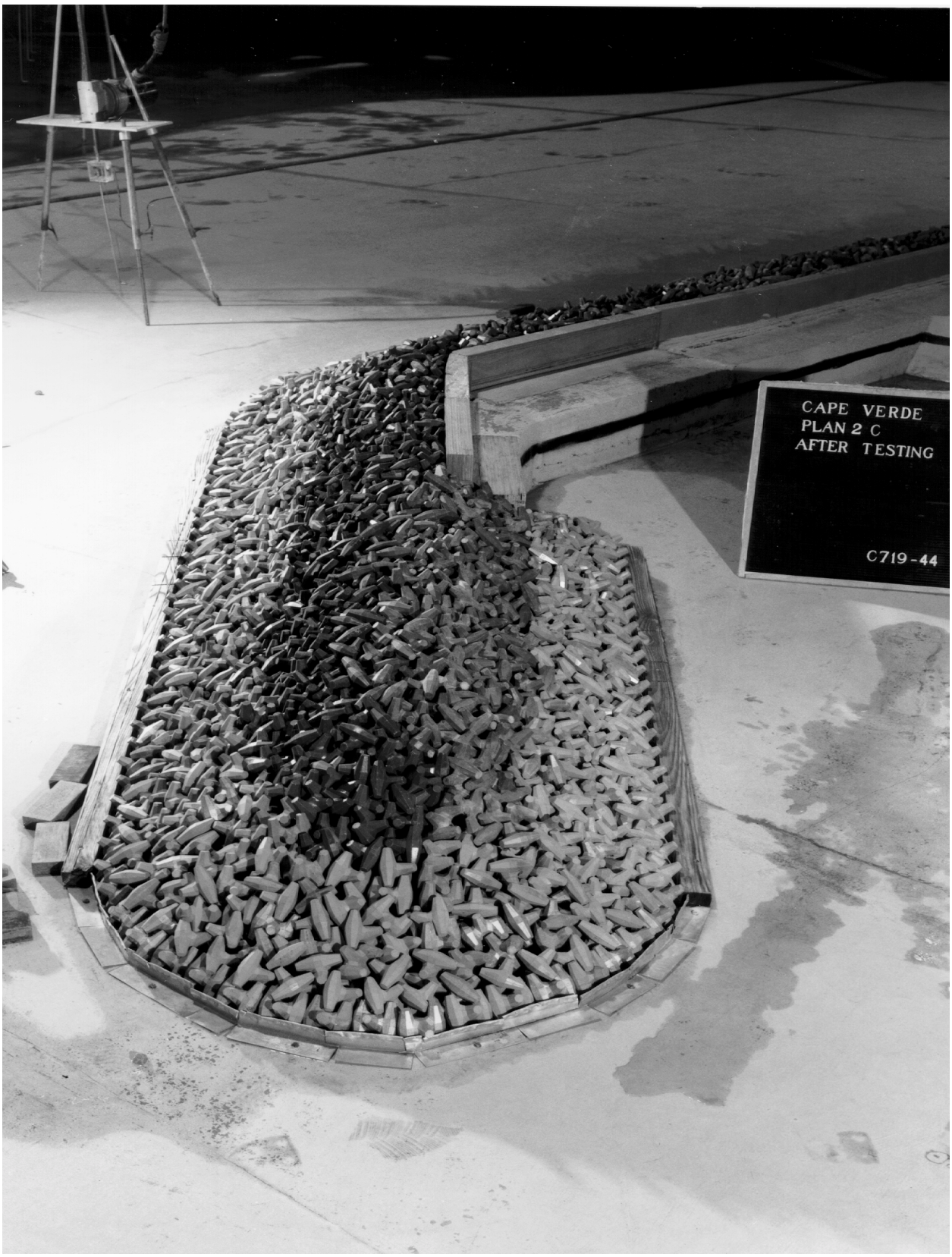

Photo A42. View of plan 2C breakwater head after testing with waves above the 3.7-m height

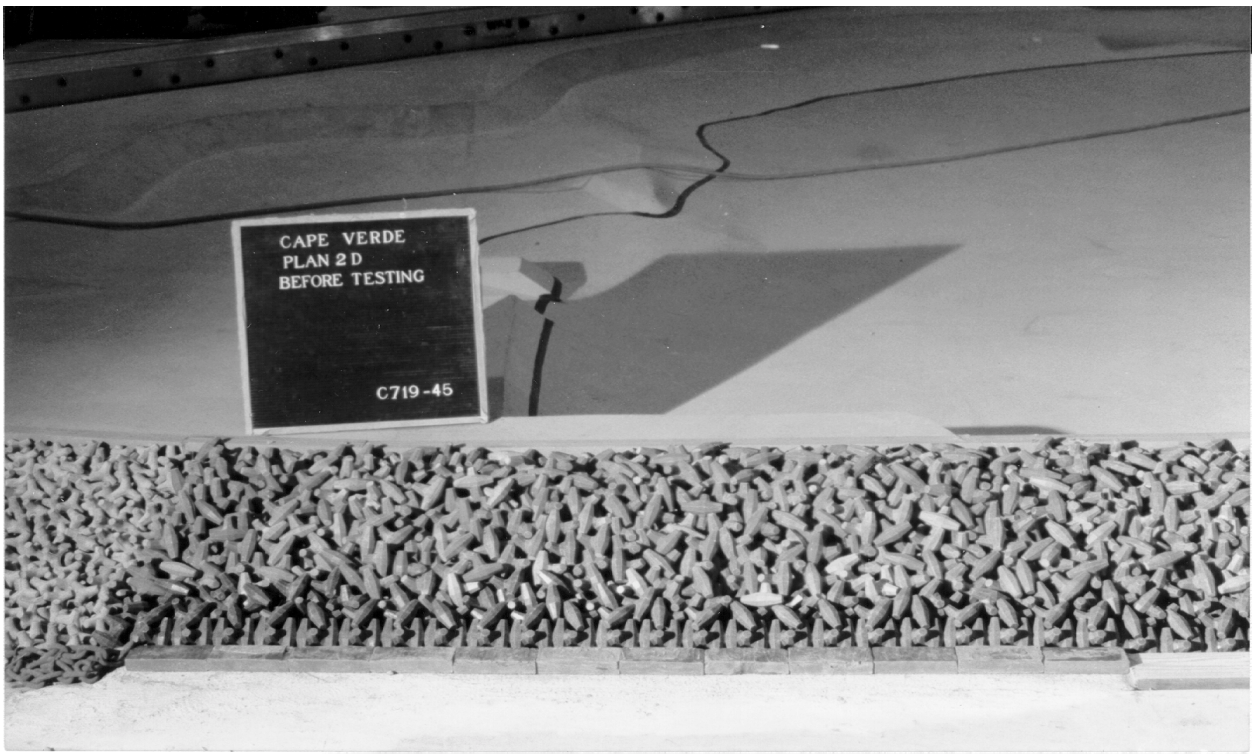

Photo A43. Sea-side view of plan 2D, profiles 4 through 6, before testing

Photo A44. View from north of plan 2D breakwater trunk before testing

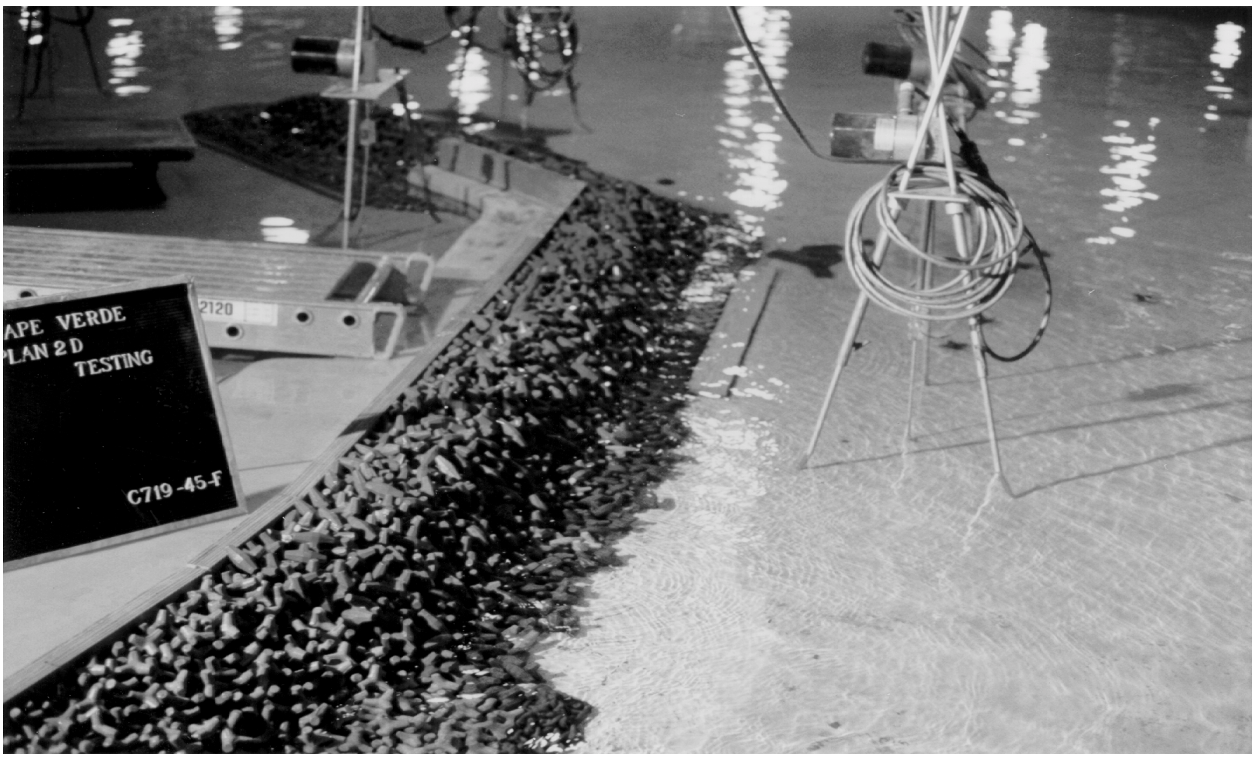

Photo A45. View from north of plan 2D breakwater trunk after testing with total sequence

Photo A46. Sea-side view of plan 2D breakwater head after testing with total storm sequence

Appendix B
Notation

a	Area scale
A	Section area
H_s'	Incident significant wave height at 21.3-m mllw depth
H_{mo}	Zero-moment wave height
H_s	Significant wave height
l	Length scale
m	Model quantity
N	Number of units in a given area
p	Prototype quantity
r	Subscript denoting model to prototype
S_a	Specific gravity of an individual armor unit relative to the water in which it is placed, $S_a = \gamma_a / \gamma_w$
S_r	Density of model and prototype materials
t	Time scale
T_p	Peak wave period
v	Volume scale
V	Armor unit volume
W_1	Weight of an individual unit in armor layer

W_2	Weight of stone in first underlayer
W_3	Weight of stone in core
W_a	Weight of an individual armor unit
ϕ	Packing density
γ_a	Specific weight of an individual armor unit
γ_w	Specific weight of water

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13.ABSTRACT *(Maximum 200 words)*

Lock and Dam 3 is located on the Red River, in a cutoff channel between 1967 river miles 140 and 142, and about 53 channel miles above John H. Overton Lock and Dam. The lock and dam will be the third lock in a series of five locks and dams designed to furnish the required maximum lift of 141 ft to provide year-round navigation on the Old and Red River Waterway from the Mississippi River to Shreveport, LA, a distance of 236 miles. The principal structures are an 84-ft-wide by 685-ft-long lock, a spillway containing six 60-ft-wide by 42-ft-high tainter gates, and a 315-ft-long fixed-crest weir adjacent to the gated spillway. The dam provides a navigation pool that extends upstream to Lock and Dam 4. The dam is operated to maintain a navigation pool of el 95.0 at the dam (all elevations (el) cited herein are in feet referred to the National Geodetic Vertical Datum). A fixed-bed model reproduced about 3.5 miles of the Red River and adjacent overbank from about 13,500 ft upstream to about 4,800 ft downstream of the dam to an undistorted scale of 1:100.

Since Lock and Dam 3 was to be constructed in an excavated channel bypassing the natural river channel, it was important that the alignment of the channel and the arrangement of the lock and dam be satisfactory for navigation. The model investigation was concerned with evaluation of navigation conditions for proposed lock designs and development of modifications required to ensure satisfactory navigation conditions. The study identified any needed modifications to the

(Continued)

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Hydraulic models	Navigation conditions
Lock and Dam 3	Red River

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navigation channel alignment, guard wall lengths, or remedial structures. Results of the investigation revealed that a system of structures was required to eliminate adverse current patterns and establish satisfactory navigation conditions for tows entering and leaving the upper lock approach. A ported upper guard wall was required to reduce outdraft near the upstream end of the guard wall, and a system of closure blocks placed in the three ports closest to the lock reduced the forces acting on the tow in the upper lock approach. A 286-ft-wide navigation channel, aligned to eliminate any impact on the transmission tower along the right bank downstream of the dam, provided satisfactory navigation conditions for tows entering and leaving the lower lock approach.

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