Technical Report CERC-95-12
August 1995

US Army Corps of Engineers
Waterways Experiment Station

Reef Breakwater Wave-Attenuation and Stability Tests, Burns Waterway Harbor, Indiana

by Robert D. Carver, Brenda J. Wright

Prepared for U.S. Army Engineer District, Chicago

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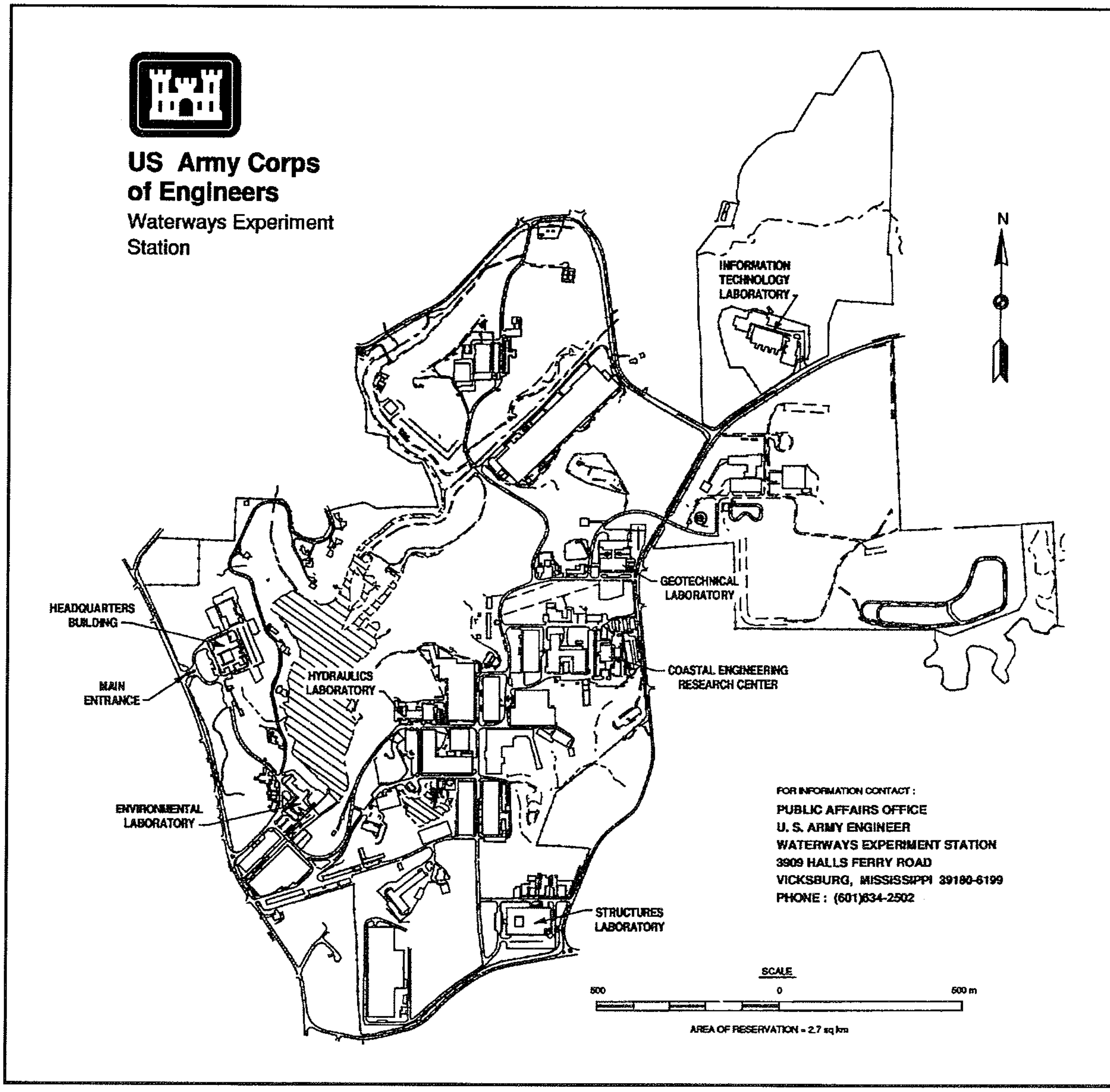

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Contents

Preface

The model investigation described herein was requested by the U.S. Army Engineer District, Chicago (NCC), in a letter to the U.S. Army Engineer Waterways Experiment Station (WES) dated 9 September 1993. Funding authorization was granted by NCC in Intra-Army Order No. NCC-IA-93-54, dated 15 September 1993. Model tests were conducted during the period September 1993 through March 1994.

The study was conducted by personnel of the Coastal Engineering Research Center (CERC) under the general direction of Dr. James R. Houston, Director, CERC, and Mr. Charles C. Calhoun, Jr., Assistant Director, CERC. Direct guidance was provided by Messrs. C. E. Chatham, Chief, Wave Dynamics Division (WDD), and D. Donald Davidson, Chief, Wave Research Branch (WRB). Tests were conducted by Mrs. Brenda J. Wright, Engineering Technician, and Mr. Charles Kappler, Jr., contract student, under the direction of Mr. R. D. Carver, Principal Investigator. This report was prepared by Mr. Carver and Mrs. Wright.

Ms. Anne Smith and Mr. Erik Matthews coordinated testing efforts for NCC. During the course of this study, communication was maintained by progress reports, telephone calls, and FAXES.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Bruce K. Howard, EN.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
feet	0.3048	meters
miles (U.S. nautical)	1.852	kilometers
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic meter
tons		tonnes

1 Introduction

The Prototype

Burns Waterway Harbor is a man-made harbor located on the southern tip of Lake Michigan, about 9 miles[1] east of Gary Harbor and 14 miles west of Michigan City Harbor. Burns Harbor was primarily constructed to facilitate shipping materials to and from steel industry in northern Indiana. The Burns Harbor structures include a 4,600-ft-long rubble-mound breakwater with an east-west alignment positioned at the north side of the harbor, a 1,200-ft-long rubble-mound breakwater with a north-south alignment located at the west side of the harbor, and a steel sheet-pile cell structure (Figure 1).

The rubble-mound structures use a multi-layered random placement design with a toe elevation of about -43 ft low water datum (lwd) and a crest elevation of +13 ft lwd. Armor stones, cut from Indiana Bedford Limestone, weigh from 10 to 15 tons on the trunk and from 15 to 20 tons on head.

Since completion of construction in 1969, two problem areas have arisen. Maintenance of the design crest elevation and structure cross section has required the addition of large amounts of stone (average of 7,640 tons per year for the first 19 years of operation). Also, unacceptably large wave conditions within the harbor (recorded data show transmission coefficients as high as 25 percent) have led to cases of extensive damage to harbor facilities and moored vessels.

Background

Extensive model tests were conducted by Carver, Dubose, and Wright (1993) to evaluate various plans of improvement that included:

a. A submerged breakwater placed 75 to 200 ft lakeward of the existing breakwater.

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[1] A table of factors for converting non-SI units of measurement to SI units is presented on page v.

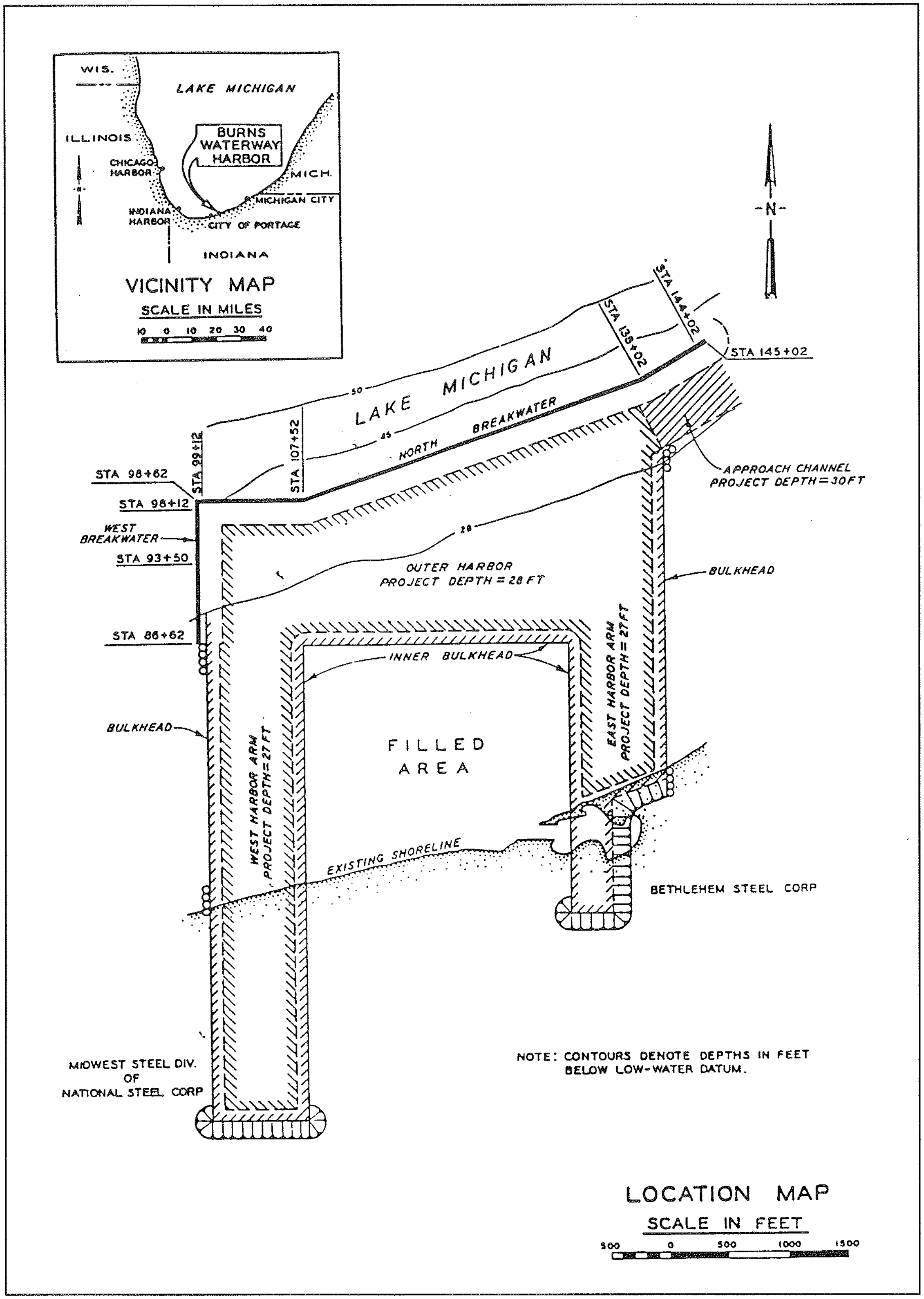

Figure 1. Location and vicinity map

b. A berm breakwater attached to the lake side of the existing structure.

c. Addition of 18-ton angular stone on the lake side and/or raising the crest with one layer of 18-ton stone.

d. Reworking existing stone into special placement at the crest.

The study concluded that submerged reefs and restacking of the existing armor were the least effective approaches to reducing wave transmission, whereas the toe berms and the large-stone overlays were the most effective. However, the submerged reefs proved to be the most effective means of reducing or eliminating damage to the existing breakwater.

After analyzing alternate designs using the results of Carver, Dubose, and Wright (1993) and based on economic considerations, the recommended plan was determined to be the segmented reef breakwater. Although the general design was determined in the 1993 study, another two-dimensional physical model study was deemed necessary to optimize the reef breakwater cross section for performance and cost.

Objective of Model Investigation

The objective of the present investigation was to conduct sufficient tests such that an optimum submerged reef could be assured. Specifically, it was desired to quantify performance (stability/transmission response) in terms of:

a. Structure height and width.

b. Location relative to existing breakwater.

c. Stone size and gradation.

2 The Model

Model-Prototype Scale Relationships

Tests were conducted at a geometrically undistorted scale of 1:36, model to prototype. Scale selection was based on the sizes of model armor available compared with the estimated size of prototype armor required for stability, minimization of wave transmission scale effects, preclusion of stability scale effects (Hudson 1975), and capabilities of the available wave tank. Based on Froude's model law (Stevens 1942) and the linear scale of 1:36, the following model-prototype relations were derived. Dimensions are in terms of length (*L*) and time (*T*).

Characteristic	Dimension	Model-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	$T_r = L_r^{1/2}$ = 1:6.0

The specific weight of water used in model tests was assumed to be the same as the prototype and equal to 62.4 pcf. Also, specific weights of model breakwater construction materials were the same as their prototype counterparts. Thus, the weight ratio of individual stones was the same as the volume ratio, i.e., 1:46,656.

In a hydraulic model investigation of this type, gravitational forces predominate (Froudian model law), except when energy transmission through the breakwater is considered (Keulegan 1973; Le Mehaute 1965). If the core material was geometrically scaled according to Froudian model relationships, internal Reynolds numbers would be too low and too much energy would be dissipated. Therefore, for all plans tested, the core stone and W/10 stone were geometrically oversized to aid in reproducing wave energy transmission.

Test Equipment and Facilities

All tests were conducted in a 3-ft-wide portion of a concrete wave flume 11 ft wide and 245 ft long (Figure 2). A 1V:100H slope, representative of the existing prototype lake bottom, was molded lakeward of the test section. Irregular waves were generated by a hydraulically actuated piston-type wave machine.

Wave data were collected on electrical capacitance wave gauges which were calibrated daily with a computer-controlled procedure incorporating a least square fit of measurements at 11 steps. This averaging technique, using 21 voltage samples per gauge, minimizes the effects of slack in the gear drives and hysteresis in the sensors. Typical calibration errors are less than 1 percent of full scale for the capacitance wave gauges. Wave signal generation and data acquisition were controlled using a DEC MicroVax I computer. Wave data analysis was accomplished using a DEC VAX 3600.

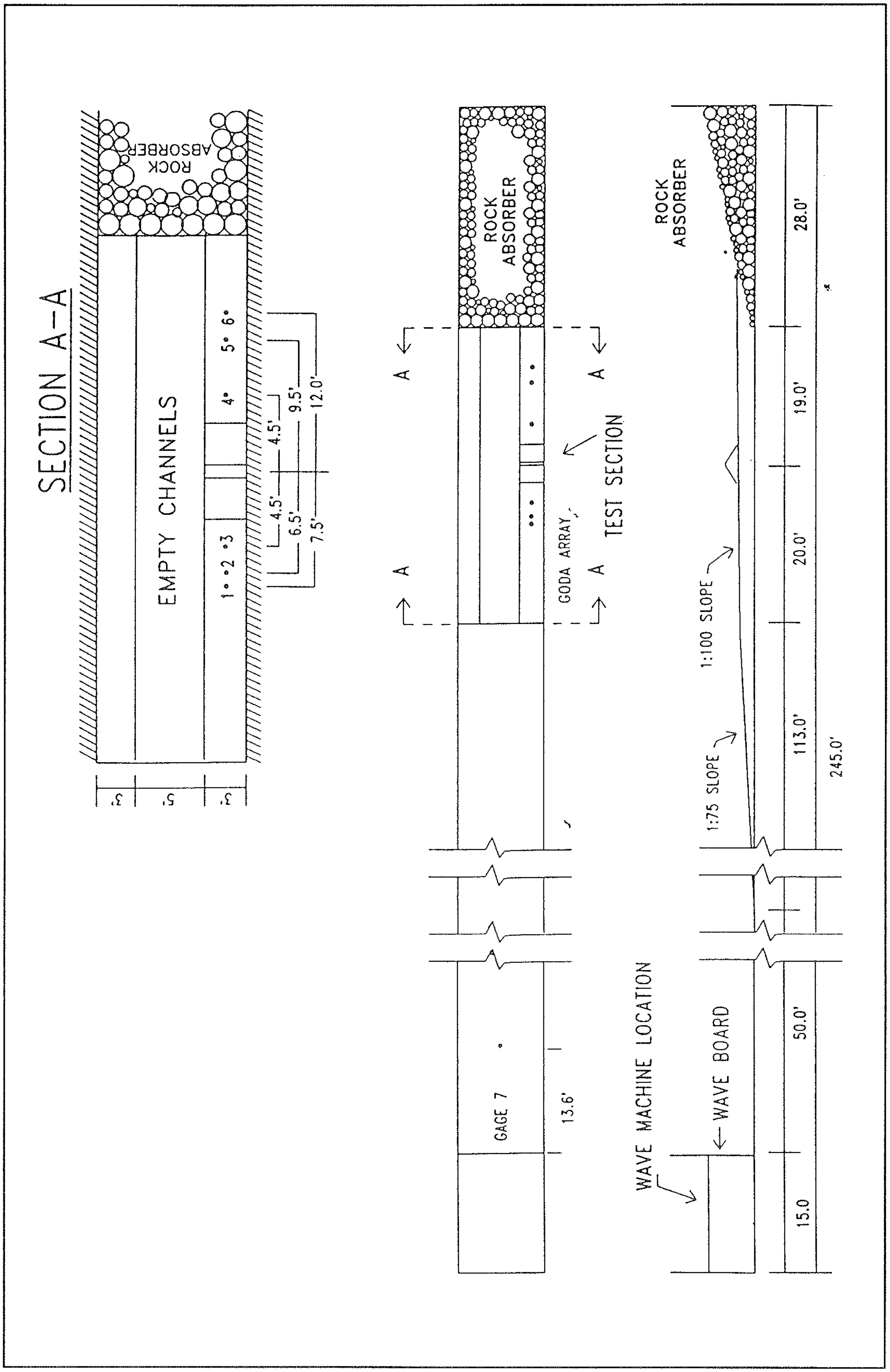

Figure 2. Wave tank cross section

3 Tests and Results

Method of Constructing Test Sections

All experimental breakwater sections were constructed to reproduce as closely as possible results of the usual methods of constructing full-scale breakwaters. The core material was dampened as it was dumped by bucket or shovel into the flume and was compacted with hand trowels to simulate natural consolidation resulting from wave action during construction of the prototype structure. Once the core material was in place, it was sprayed with a low-velocity water hose to ensure adequate compaction of the material. The underlayer stone was then added by shovel and smoothed to grade by hand or with trowels. Armor units used in the cover layers were placed in a random manner corresponding to work performed by a general coastal contractor; i.e., they were individually placed but were laid down without special orientation or fitting. After each test, the armor units were removed from the breakwater, all of the underlayer stones were replaced to the grade of the original test section, and the armor was replaced.

Simulation of Existing Structure (Plan 1)

Plan 1 (Figure 3) was constructed to a crown elevation of +13 ft lwd and used armor slopes of 1V:1.7H both lakeside and harbor side. The lakeside slope (above -27 ft lwd) and crest were armored with two layers of 10- to 16-ton limestone blocks whereas the harbor-side slope used one layer of 10- to 16-ton blocks between +3 and -13 ft lwd. A graded mixture of limestone blocks was used to form the armor layer and underlayer. Distribution of individual stone weights within these mixtures was as follows:

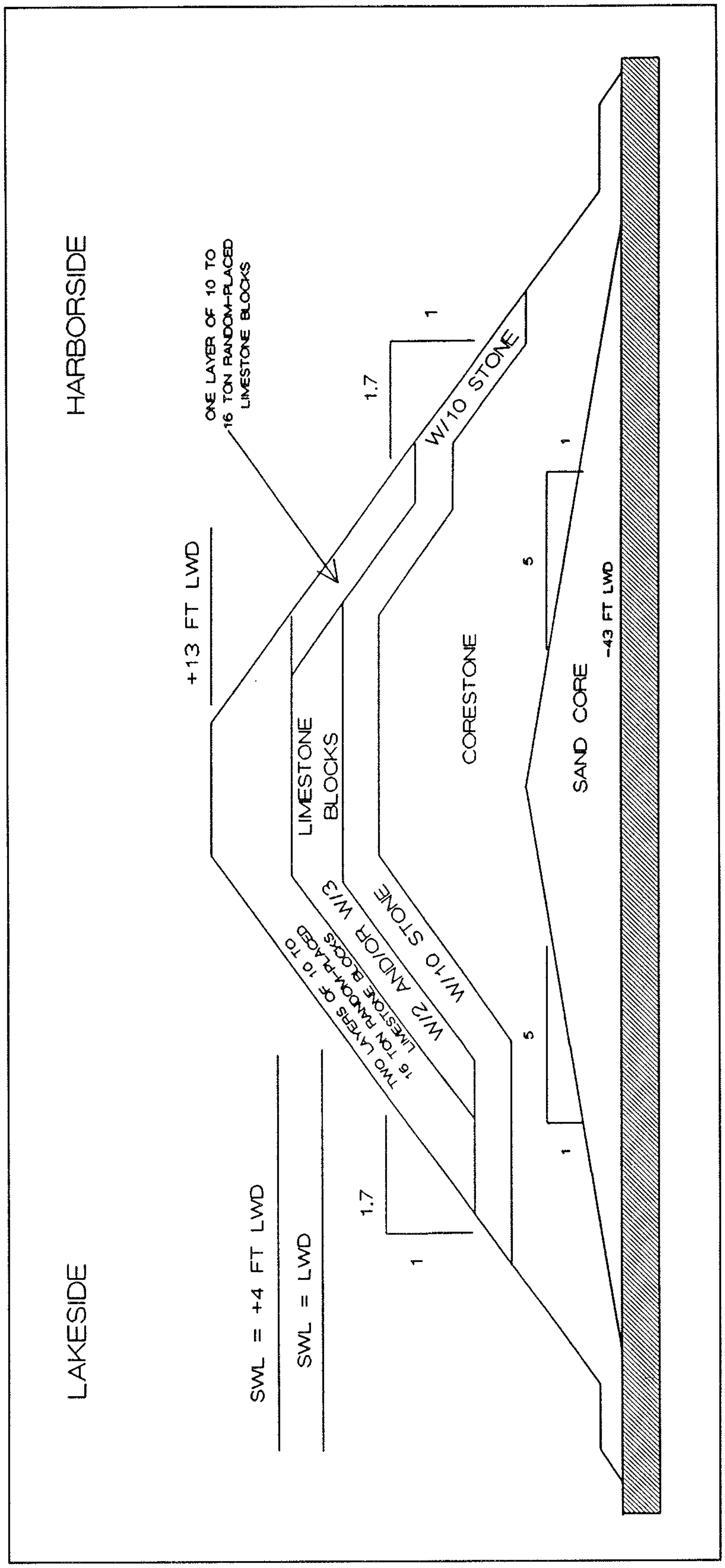

Figure 3. Existing breakwater

Type of Stone	Weight, tons	Percent by Weight
W	10	30
	12	30
	14	30
	16	10
W/2	5	33
	8	33
	10	34
W/3	3	25
	5	25
	8	25
	10	25

Measurement of incident and transmitted wave heights by Carver, Dubose, and Wright (1993) for existing conditions produced the following at the +4-ft still-water level (swl):

T_p, sec	Incident H_{mo}, ft	Transmitted H_{mo}, ft
7.0	2.4	0.5
7.0	4.2	0.8
7.0	6.9	1.1
7.0	9.6	1.4
7.0	11.6	1.9
9.0	2.8	0.7
9.0	4.9	1.1
9.0	6.8	1.4
9.0	8.0	1.6
9.0	9.3	1.8
9.0	10.6	2.1
11.6	2.0	0.7
11.6	4.2	1.2
11.6	6.8	1.8
11.6	9.3	2.4
11.6	11.8	3.2
11.6	14.1	3.9
11.6	17.5	5.7
11.6	19.1	6.5
T_p = wave period of peak energy density of spectrum, sec H_{mo} = zero-moment wave height, ft		

Figures 4 and 5 present transmitted wave height as a function of incident wave height for the 1993 and present investigations. These data show that the

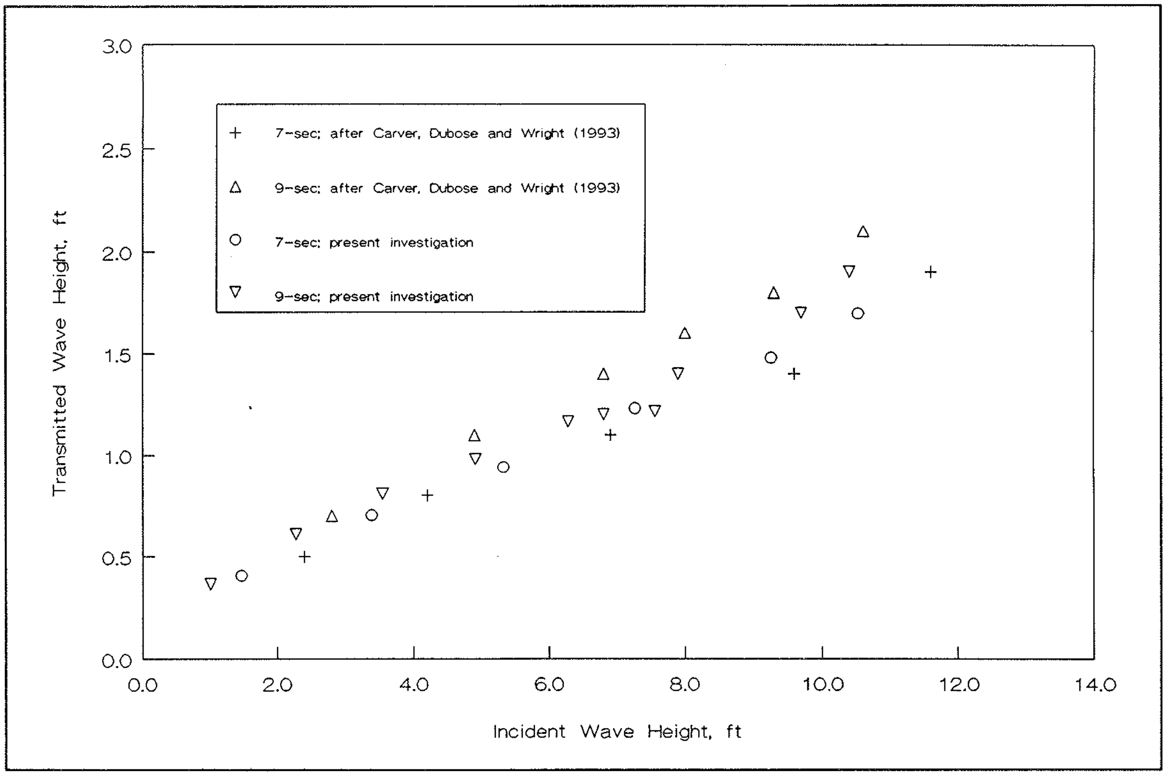

Figure 4. Comparison of 7-sec and 9-sec transmission test results

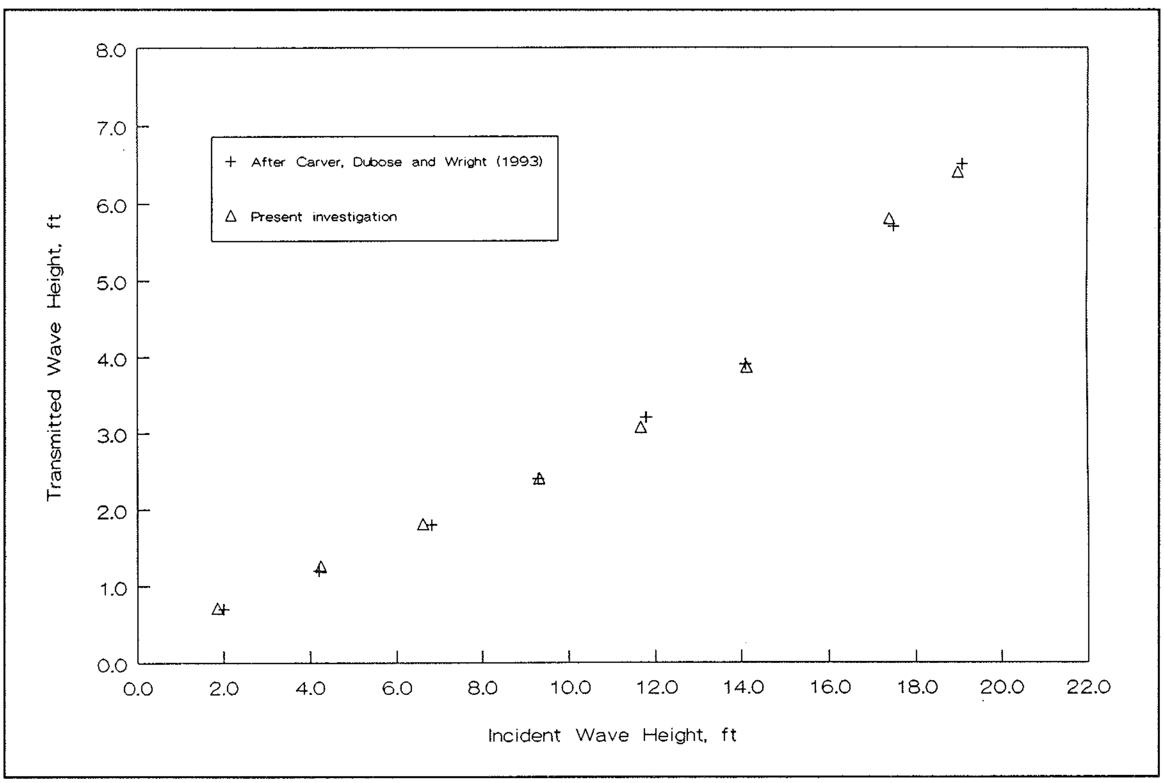

Figure 5. Comparison of 11.6-sec transmission test results

existing breakwater as built in the present investigation properly replicates wave energy transmission.

Development of Plans

The first structure tested, Plan R1 shown in Figure 6 and Photo 1, was constructed to an elevation of -20 ft lwd. It used a crown width of 60 ft, an armor stone weight of 5 tons, and was placed 75 ft lakeward of the existing structure. The 75-ft spacing was chosen based on results of the previous investigation (Carver, Dubose, and Wright 1993) in which two identical structures (Plans 4A and 4A1) showed improved performance with a 75-ft spacing versus a 150-ft spacing. Transmission test results were as follows:

	H_{mo}, ft Measured			
T_p, sec	Incident	Behind Reef	Behind Breakwater	C_t
7.0	2.5	2.2	0.5	0.20
7.0	4.4	3.9	0.8	0.18
7.0	7.3	6.5	1.1	0.15
7.0	9.9	8.3	1.4	0.14
7.0	12.3	9.8	1.7	0.14
9.0	2.9	2.7	0.6	0.21
9.0	4.8	4.6	0.9	0.19
9.0	6.9	6.4	1.2	0.17
9.0	8.3	7.3	1.4	0.17
9.0	9.6	8.8	1.5	0.16
9.0	11.0	10.1	1.7	0.15
11.6	2.1	2.0	0.7	0.33
11.6	4.6	4.2	1.2	0.26
11.6	7.3	6.6	1.6	0.22
11.6	10.0	9.0	2.2	0.22
11.6	12.7	11.4	2.8	0.22
11.6	15.1	13.0	3.5	0.23
11.6	18.5	15.1	4.7	0.26
11.6	21.9	17.1	5.8	0.25

H_t = transmitted wave height, ft
H_i = incident wave height, ft
C_t = transmission coefficient (H_t/H_i)

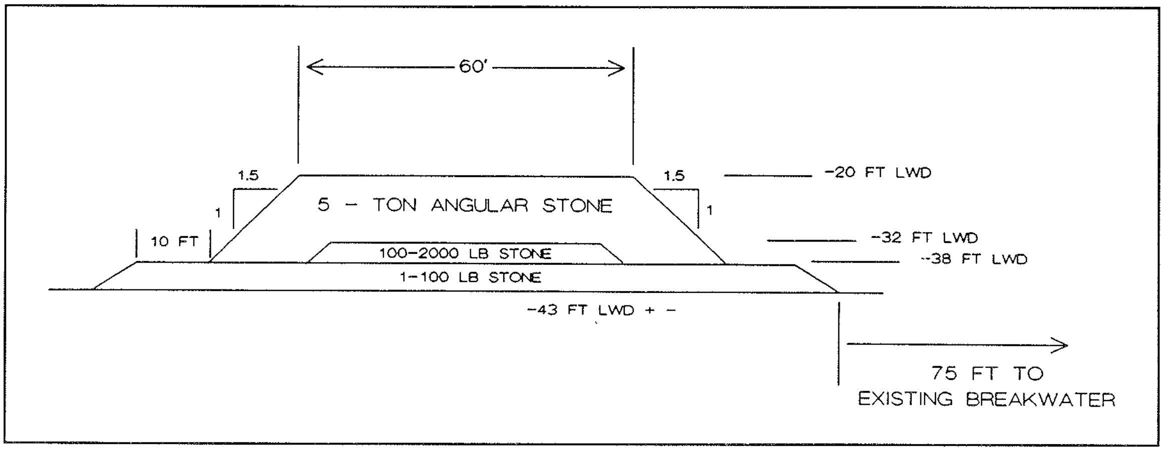

Figure 6. Elements of Plan R1

As shown in Figure 7, Plan R1 was successful in reducing 7- and 9-sec, 5-ft incident waves to heights of about 1 ft behind the breakwater. Also, as desired, transmitted wave heights of about 3 ft were observed for 11.6-sec, 13-ft incident waves. The reef was stable (Photo 2) and stability of the existing breakwater was significantly improved (Photo 3) relative to base conditions; however, a few armor stones were displaced.

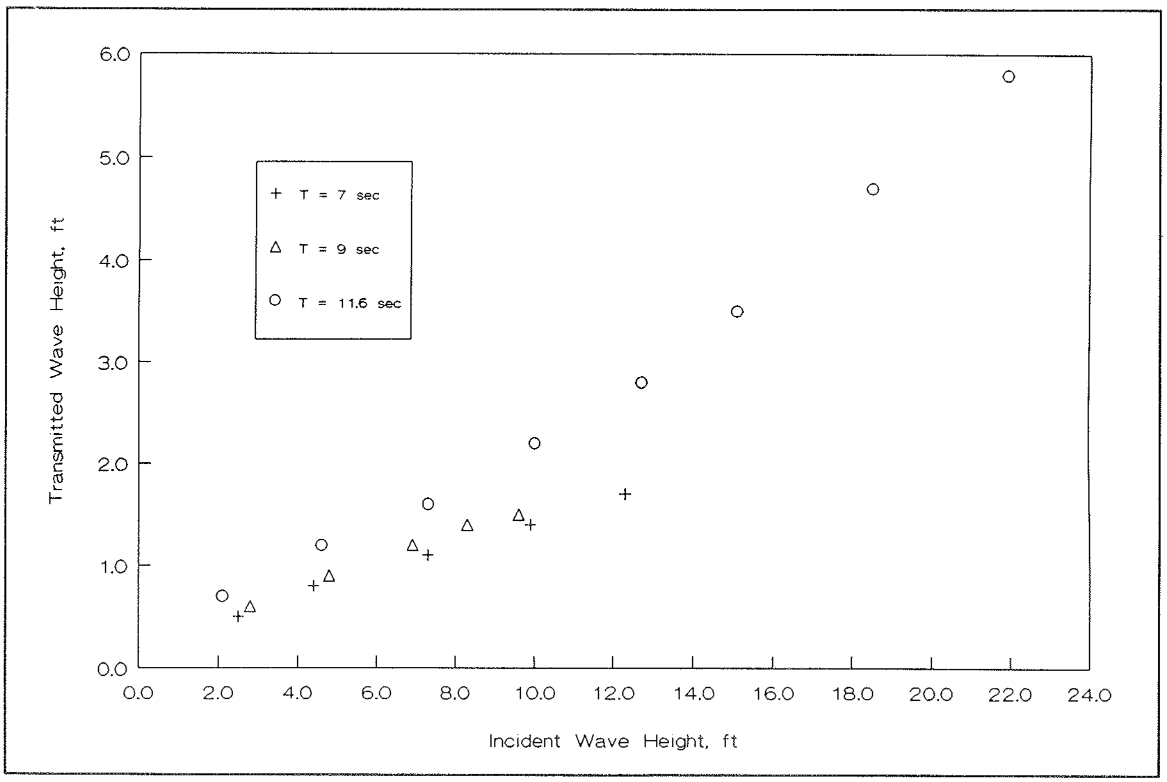

Figure 7. Transmission test results for Plan R1

The second section tested was similar to Plan R1 except the crown elevation was raised to -10 ft lwd. As shown in Figure 8 and Photo 4, Plan R2 was created by projecting the 1V:1.5H armor slopes of Plan R1 10 ft higher, thus yielding a 30-ft crown width. Test results for Plan R2 were as follows:

		H_{mo}, ft Measured		
T_p, sec	Incident	Behind Reef	Behind Breakwater	C_t
7.0	2.5	2.2	0.5	0.20
7.0	4.4	3.7	0.8	0.18
7.0	8.0	6.7	1.1	0.14
7.0	10.5	8.0	1.4	0.13
7.0	13.2	9.3	1.8	0.14
9.0	3.0	2.6	0.6	0.20
9.0	5.1	4.4	1.0	0.20
9.0	7.3	6.4	1.1	0.15
9.0	8.8	7.6	1.3	0.15
9.0	10.1	8.4	1.5	0.15
9.0	11.4	9.1	1.6	0.14
11.6	2.1	1.8	0.6	0.29
11.6	4.7	4.0	1.1	0.23
11.6	7.3	6.3	1.6	0.22
11.6	10.2	8.5	2.1	0.21
11.6	12.8	10.1	2.6	0.20
11.6	15.4	11.6	3.1	0.20
11.6	19.9	13.8	4.4	0.22
11.6	22.5	15.1	5.3	0.24

As shown above and in Figure 9, Plan R2 yielded results similar to Plan R1 for the lower wave heights and improved performance for the larger wave heights. The reef was stable (Photo 5) and stability of the existing breakwater was acceptable (Photo 6) with a few armor stones displaced from the harbor side.

The third structure, Plan R2A shown in Figure 10 and Photo 7, was identical to Plan R2 except the 5-ton stone between -10 and -20 ft lwd was replaced with smaller 3- to 5-ton material. It was assumed that if the smaller stone proved to be stable in this region it would be stable at any greater depth. Photo 8 shows the structure before wave attack. Plan R2A was tested

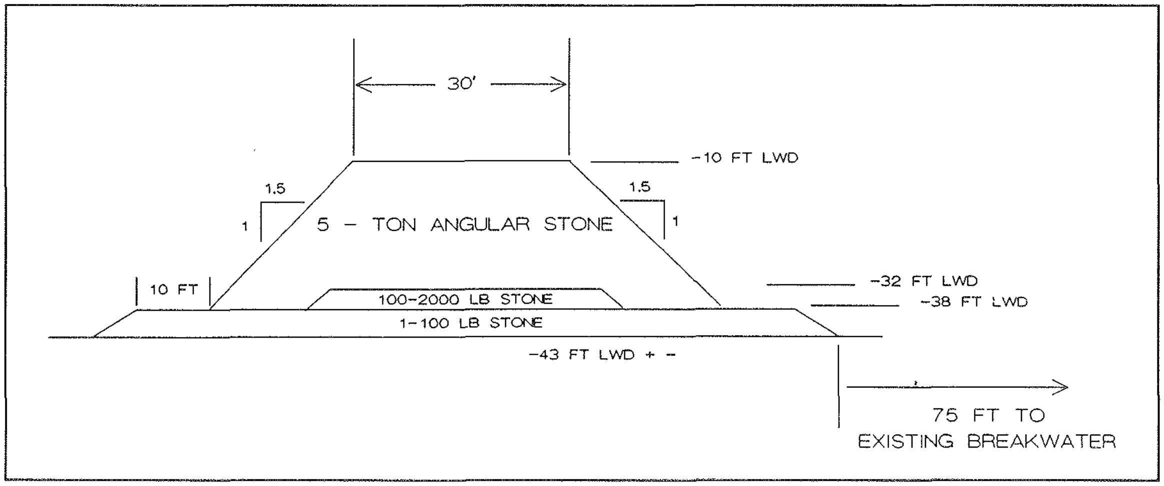

Figure 8. Elements of Plan R2

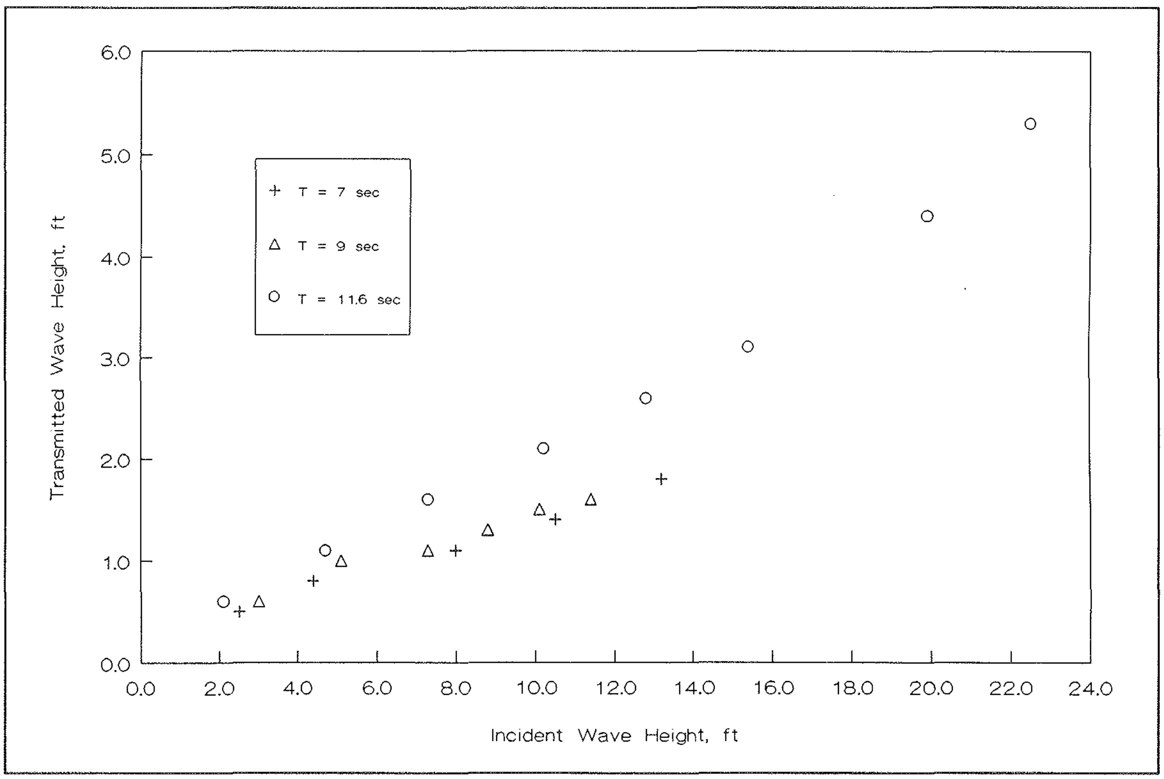

Figure 9. Transmission test results for Plan R2

primarily for stability; therefore, only incident wave conditions above 10 ft were considered. Transmitted wave heights, measured incidental to the stability tests, were as follows:

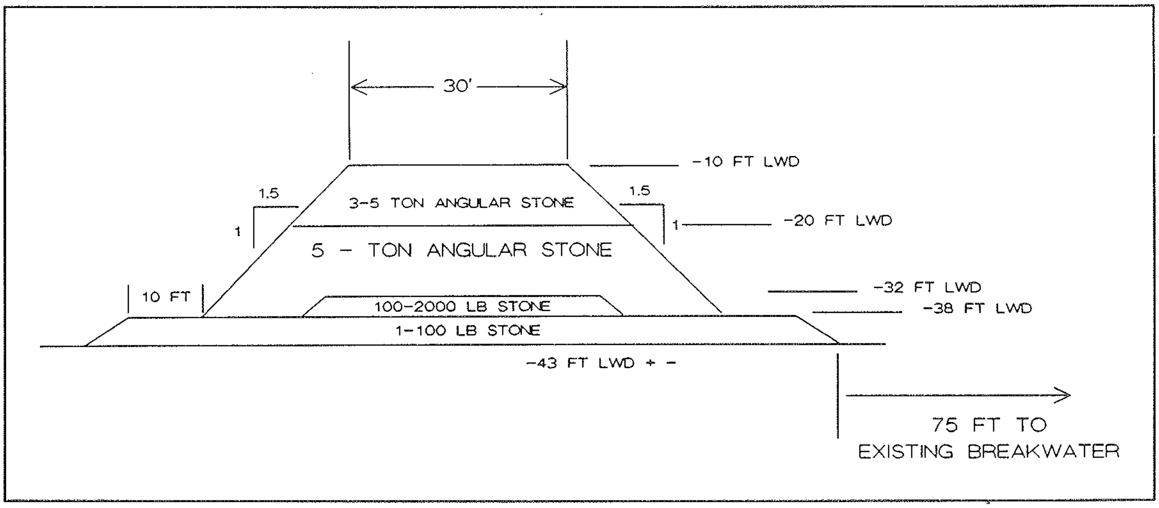

Figure 10. Elements of Plan R2A

T_p, sec	Incident	H_{mo}, ft Measured		C_t
		Behind Reef	Behind Breakwater	
7.0	10.2	7.8	1.4	0.14
7.0	13.3	9.3	1.8	0.14
9.0	10.1	8.2	1.5	0.15
9.0	11.5	9.1	1.6	0.14
11.6	12.9	10.1	2.5	0.19
11.6	15.3	11.4	3.1	0.20
11.6	19.6	13.8	4.5	0.23
11.6	22.3	14.9	5.3	0.24

As expected and shown in Figure 11, Plan R2A produced transmission results almost identical to Plan R2. Stability of the reef was considered marginal (Photo 8) with about 5 percent of the 3- to 5-ton stone volume being displaced down the lakeward face.

Plan R3, shown in Figure 12 and Photo 9, was the same as Plan R1, except the crown width was increased to 75 ft and the 5-ton armor was replaced with 3- to 5-ton material. Transmission test results were as follows:

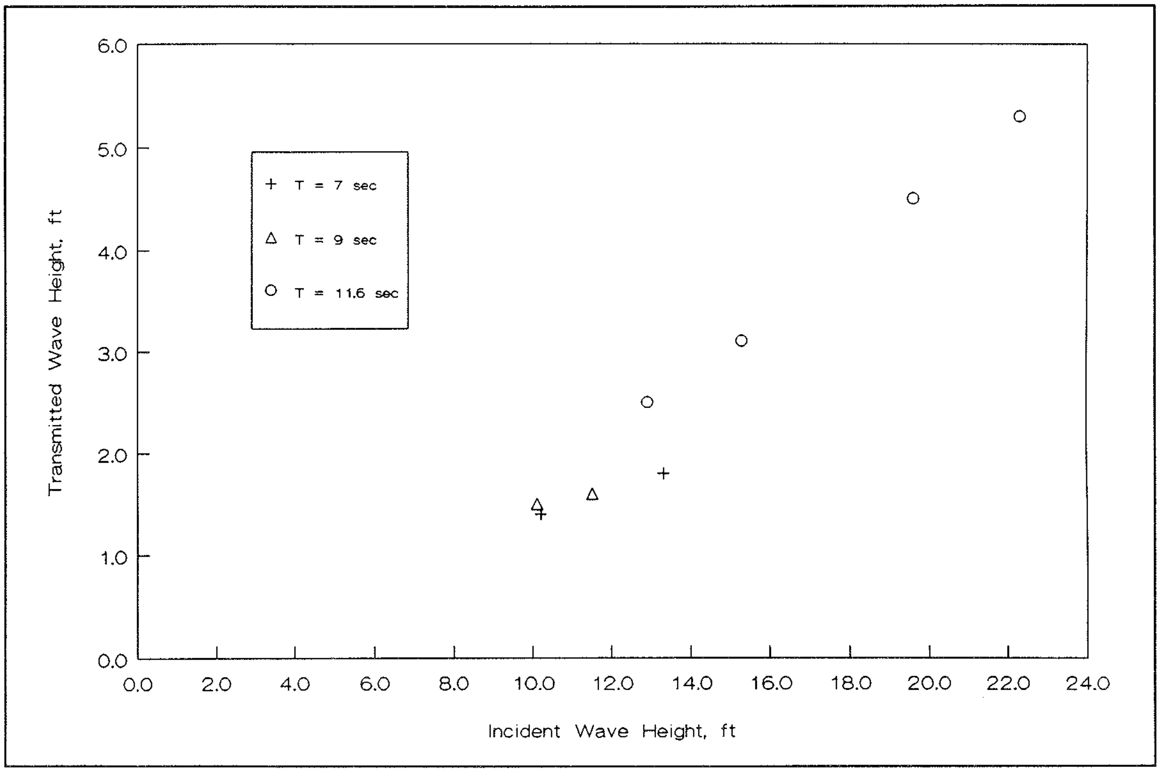

Figure 11. Transmission test results for Plan R2A

T_p, sec	Incident	H_{mo}, ft Measured Behind Reef	H_{mo}, ft Measured Behind Breakwater	C_t
7.0	2.3	2.0	0.5	0.22
7.0	4.1	3.7	0.7	0.17
7.0	6.5	5.7	1.0	0.15
7.0	10.2	8.5	1.4	0.14
7.0	12.9	10.2	1.8	0.14
9.0	2.7	2.6	0.6	0.22
9.0	4.8	4.5	0.9	0.19
9.0	7.0	6.4	1.1	0.16
9.0	8.2	7.6	1.3	0.16
9.0	9.8	8.9	1.6	0.16
9.0	11.2	9.8	1.7	0.15
11.6	2.1	1.9	0.7	0.33
11.6	4.6	4.1	1.2	0.26
11.6	7.3	6.5	1.5	0.21
11.6	9.9	8.8	2.1	0.21
11.6	12.5	10.8	2.6	0.21
11.6	14.8	12.3	3.3	0.22
11.6	19.6	15.1	4.6	0.23
11.6	21.5	16.1	5.7	0.27

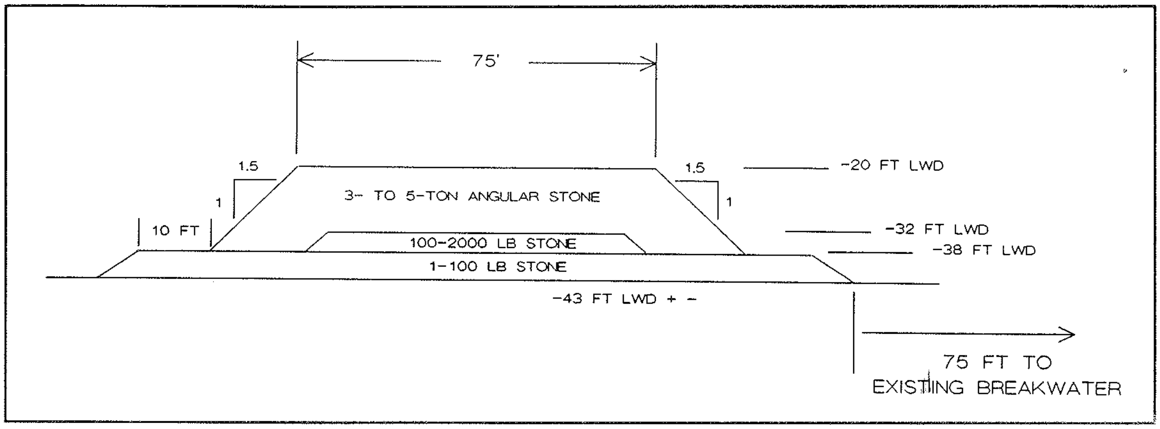

Figure 12. Elements of Plan R3

As shown in the previous table and in Figure 13, Plan R3 produced results very similar to Plan R2 and slightly better than Plan R1. The reef was stable (Photos 10 and 11) with 1 to 2 percent of the 3- to 5-ton stone displaced. Similar to previous plans, stability of the existing breakwater was significantly improved relative to base conditions.

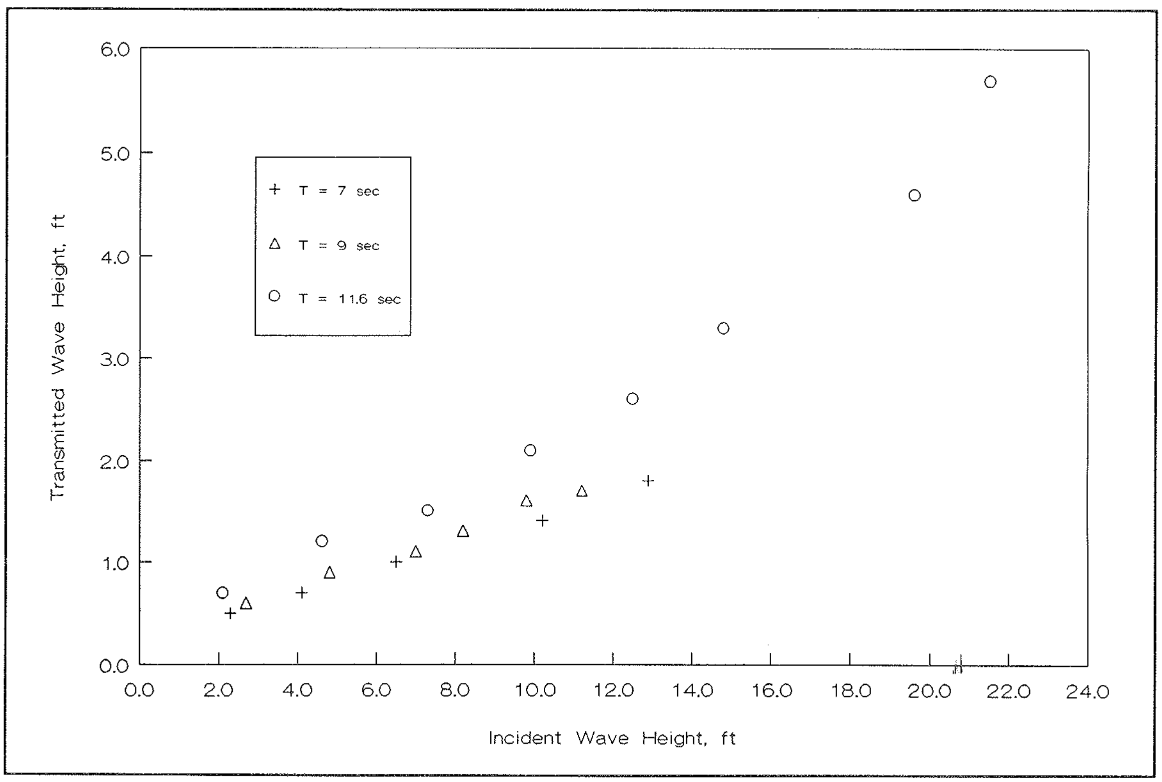

Figure 13. Transmission test results for Plan R3

Plan R4 (Figure 14) was the same as Plan R3, except the 3- to 5-ton stone was replaced with a 1- to 5-ton mixture in an effort to reduce costs and better use all of the quarry yield. Results of the transmission tests were as follows:

		H_{mo}, ft Measured		
T_p, sec	Incident	Behind Reef	Behind Breakwater	C_t
7.0	2.3	1.9	0.5	0.22
7.0	4.1	3.5	0.7	0.17
7.0	7.1	6.2	1.0	0.14
7.0	10.3	8.5	1.5	0.15
7.0	12.9	10.2	1.8	0.14
9.0	2.8	2.6	0.6	0.21
9.0	4.9	4.6	0.9	0.18
9.0	6.8	6.3	1.1	0.16
9.0	8.2	7.6	1.4	0.17
9.0	9.7	8.8	1.6	0.16
9.0	11.0	9.8	1.7	0.15
11.6	2.0	1.8	0.6	0.30
11.6	4.5	4.0	1.1	0.24
11.6	7.2	6.4	1.6	0.22
11.6	9.8	8.7	2.1	0.21
11.6	12.5	10.8	2.6	0.21
11.6	14.9	12.3	3.4	0.23
11.6	19.4	15.1	4.7	0.24
11.6	21.9	16.5	5.6	0.26

As expected and shown in Figure 15, Plan R4 produced transmission results very similar to Plan R3. The 1- to 5-ton stone used to armor the reef showed more movement than the 3- to 5-ton stone used in Plan R3 (Photos 12 and 13); however, this movement amounted to only about 3 to 4 percent of the original stone volume and was considered acceptable. One lakeside and three harbor-side armor units were displaced from the existing breakwater (Photo 14).

Plan R5 (Figure 14) was the same as Plan R4, except the toe-to-toe spacing from the existing breakwater was increased from 75 to 100 ft. Comparison of test results for plans tested herein which used a 75-ft toe-to-toe spacing with

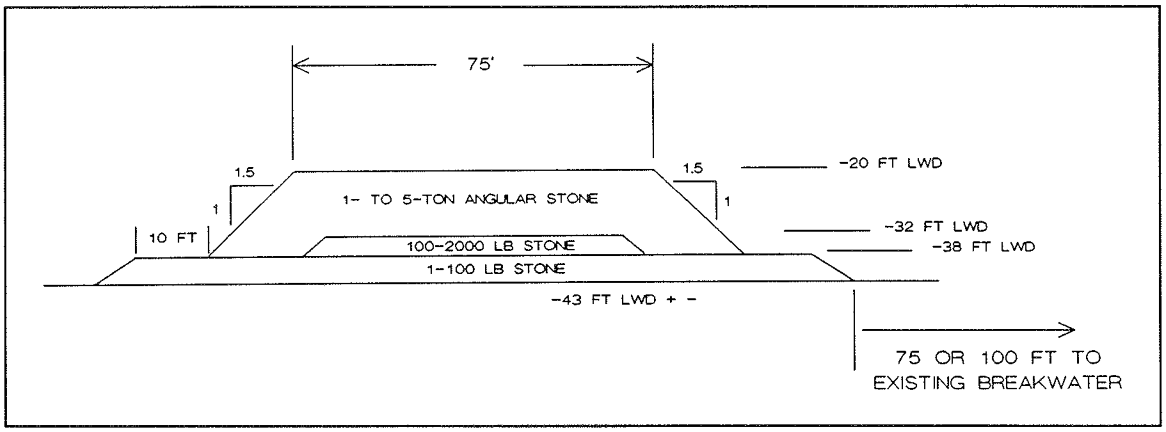

Figure 14. Elements of Plans R4 and R5

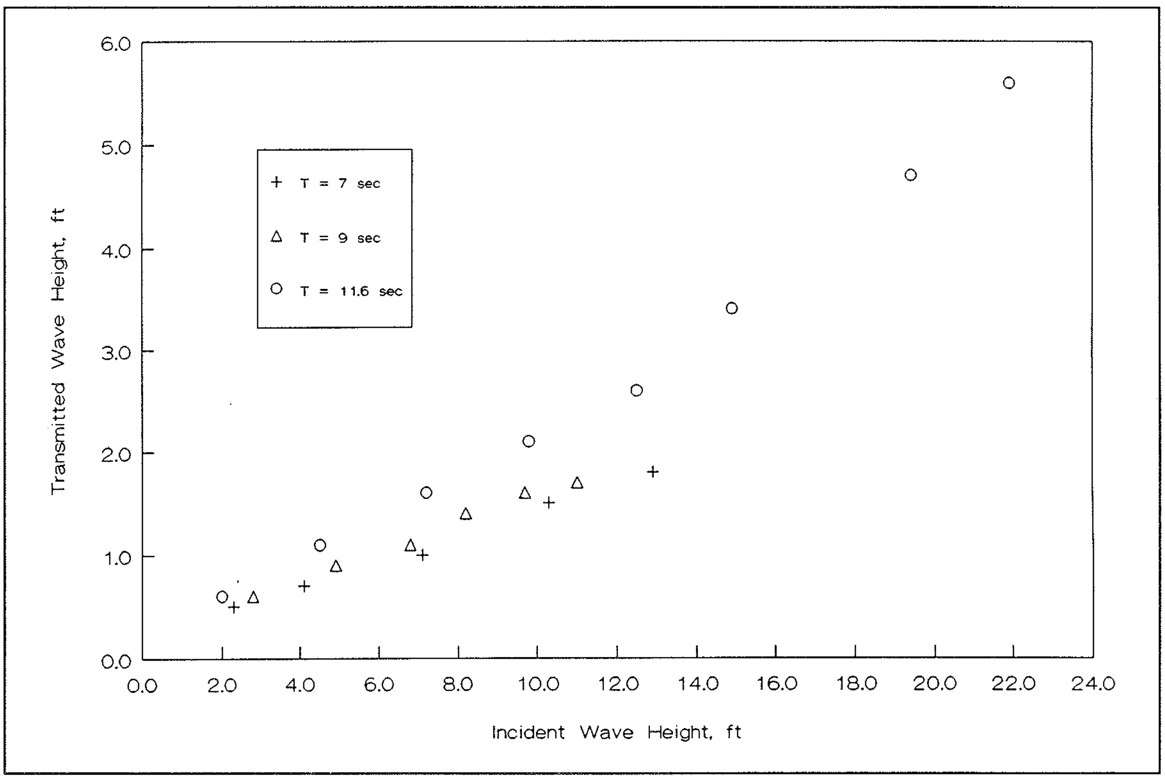

Figure 15. Transmission test results for Plan R4

results obtained by Carver, Dubose, and Wright (1993) using a 150-ft spacing showed that the 75-ft spacing generally gave better transmission results but not as much improvement in stability of the existing breakwater. Therefore, it was decided to test an intermediate spacing in an effort to determine if stability of the existing breakwater could be improved without a significant increase in transmission. Results of the transmission tests were as follows:

T_p, sec	Incident	H_{mo}, ft Measured		C_t
		Behind Reef	Behind Breakwater	
7.0	2.4	2.2	0.5	0.21
7.0	4.4	3.9	0.8	0.18
7.0	7.4	6.4	1.1	0.15
7.0	11.0	8.9	1.6	0.15
7.0	13.0	9.9	1.9	0.15
9.0	2.7	2.6	0.6	0.22
9.0	4.8	4.4	1.0	0.21
9.0	6.9	6.3	1.2	0.17
9.0	8.3	7.5	1.4	0.17
9.0	9.8	8.6	1.5	0.15
9.0	11.2	9.6	1.7	0.15
11.6	2.1	2.0	0.7	0.33
11.6	4.5	4.2	1.2	0.27
11.6	7.2	6.7	1.6	0.22
11.6	10.0	9.1	2.1	0.21
11.6	12.6	11.3	2.6	0.21
11.6	15.1	13.0	3.4	0.23
11.6	19.3	15.7	4.7	0.24
11.6	21.7	17.0	5.6	0.26

As shown above and in Figure 16, Plan R5 produced slightly larger transmitted wave heights than were observed for Plan R4. Stability of the 1- to 5-ton stone used to armor the reef was very similar to Plan R4 and again considered acceptable with about 3 to 4 percent of the original stone volume being displaced (Photo 15). The existing breakwater did not experience any armor displacement (Photo 16).

Plan R6 was the same as Plan R4, except the thickness of the 1- to 5-ton stone layer was reduced from 12 to 9 ft, as shown in Figure 17 and Photo 17. This 25-percent reduction in the 1- to 5-ton layer and corresponding increase in 100- to 2,000-lb layer was investigated in an effort to further reduce costs. Plan R6 was tested at swl's of 0.0, +4.0, and +6.0 ft lwd with the following results:

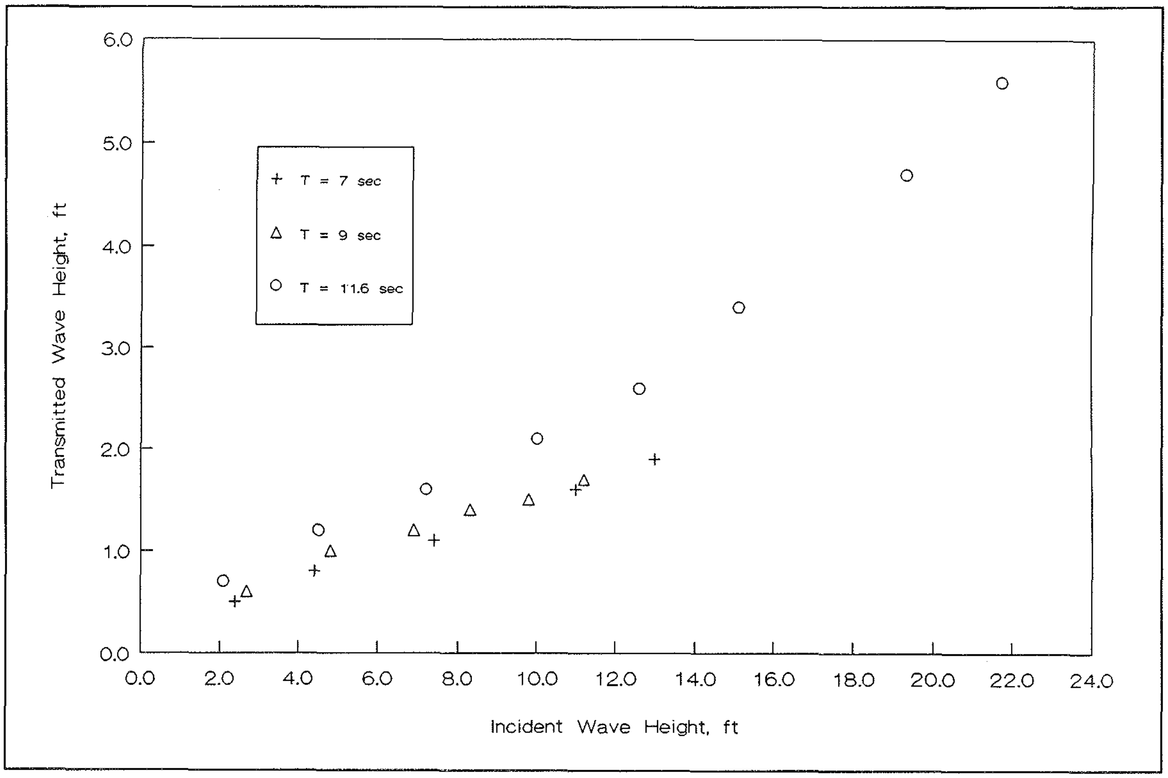

Figure 16. Transmission test results for Plan R5

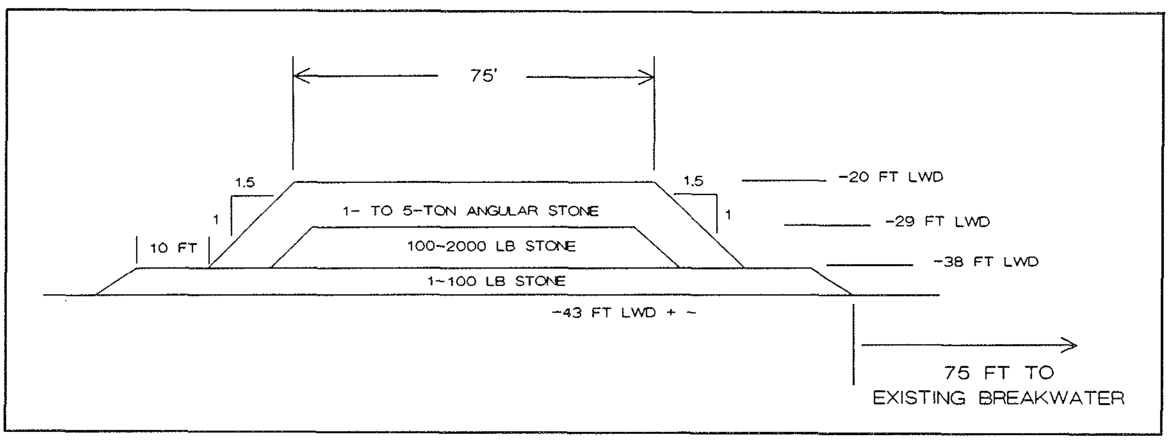

Figure 17. Elements of Plan R6

T_p, sec	Incident	H_{mo}, ft Measured		C_t
		Behind Reef	Behind Breakwater	
		swl = 0.0 ft lwd		
7.0	2.2	1.9	0.4	0.18
7.0	4.2	3.7	0.6	0.14
7.0	7.2	6.2	0.9	0.13
7.0	11.0	8.6	1.3	0.12
7.0	13.2	9.8	1.5	0.11
9.0	2.8	2.6	0.5	0.18
9.0	4.8	4.5	0.8	0.17
9.0	7.3	6.7	1.0	0.14
9.0	8.7	7.9	1.2	0.14
9.0	9.9	8.9	1.3	0.13
9.0	11.4	9.9	1.4	0.12
11.6	2.0	1.8	0.6	0.30
11.6	4.5	4.0	0.9	0.20
11.6	7.2	6.4	1.4	0.19
11.6	9.8	8.7	1.8	0.18
11.6	12.5	10.5	2.3	0.18
11.6	15.1	12.1	2.7	0.18
11.6	19.3	14.3	3.6	0.19
11.6	21.5	15.4	4.2	0.20
		swl = +4.0 ft lwd		
7.0	2.3	2.0	0.5	0.22
7.0	4.2	3.6	0.8	0.19
7.0	6.9	6.1	1.1	0.16
7.0	9.9	8.3	1.5	0.15
7.0	12.5	9.6	1.8	0.14
9.0	2.8	2.5	0.6	0.21
9.0	4.7	4.4	0.9	0.19
9.0	6.9	6.4	1.1	0.16
9.0	8.1	7.5	1.4	0.17
9.0	9.5	8.5	1.5	0.16
9.0	10.9	9.7	1.7	0.16
11.6	2.0	1.8	0.6	0.30
11.6	4.5	4.1	1.1	0.24
11.6	7.2	6.5	1.6	0.22
11.6	9.9	8.8	2.1	0.21
11.6	12.6	10.9	2.7	0.21
11.6	15.0	12.5	3.3	0.22
11.6	19.2	15.1	4.7	0.24
11.6	21.7	16.2	5.3	0.24
				(Continued)

		H_{mo}, ft Measured		
T_p, sec	Incident	Behind Reef	Behind Breakwater	C_t
swl = +6.0 ft lwd				
7.0	2.3	2.1	0.6	0.26
7.0	4.2	3.7	0.9	0.21
7.0	7.2	6.4	1.3	0.18
7.0	10.1	8.6	1.8	0.18
7.0	13.0	10.4	2.2	0.17
9.0	2.9	2.7	0.8	0.28
9.0	5.0	4.6	1.1	0.22
9.0	7.2	6.7	1.4	0.19
9.0	9.1	8.4	1.7	0.19
9.0	10.4	9.4	2.0	0.19
9.0	11.6	10.3	2.3	0.20
11.6	2.1	1.9	0.8	0.38
11.6	4.6	4.2	1.3	0.28
11.6	7.4	6.7	1.8	0.24
11.6	10.2	9.1	2.4	0.24
11.6	12.8	11.2	3.1	0.24
11.6	15.4	13.1	3.8	0.25
11.6	20.0	15.8	5.5	0.28
11.6	22.5	17.2	6.3	0.28

Plan R6 produced transmission results very similar to Plans R3 and R4 at the +4-ft swl. As expected and shown in Figures 18-20, consistently lower and higher transmitted heights were observed at the 0- and +6-ft swl's. Stability of the 1- to 5-ton stone used to armor the reef was very similar to Plans R4 and R5 and again considered acceptable with 4 to 5 percent of the original stone volume being displaced (Photos 17 and 18). One harbor-side armor unit was displaced from the existing breakwater (Photos 19 and 20).

Summary of Results

The seven improvement plans significantly improved stability of the existing breakwater and reduced transmitted wave heights to some extent. In order to help quantify performance, transmission coefficients were calculated with the following results:

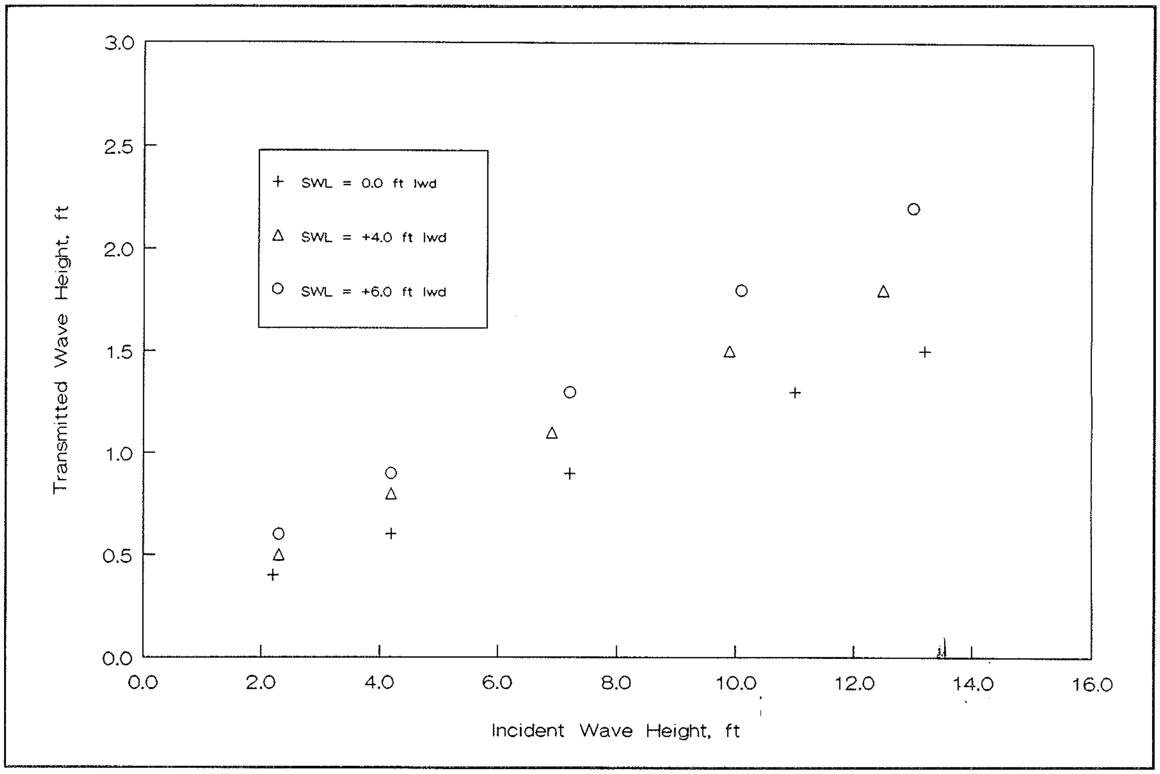

Figure 18. Transmission test results for Plan R6; 7-sec wave period

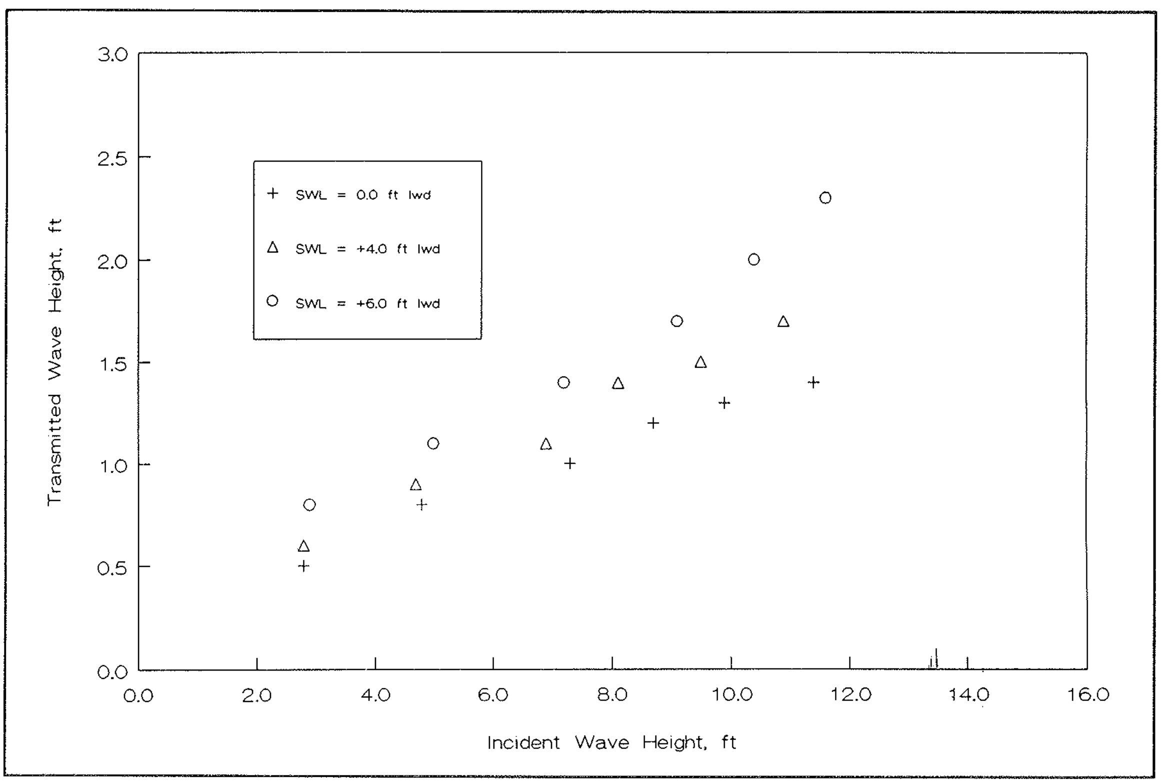

Figure 19. Transmission test results for Plan R6; 9-sec wave period

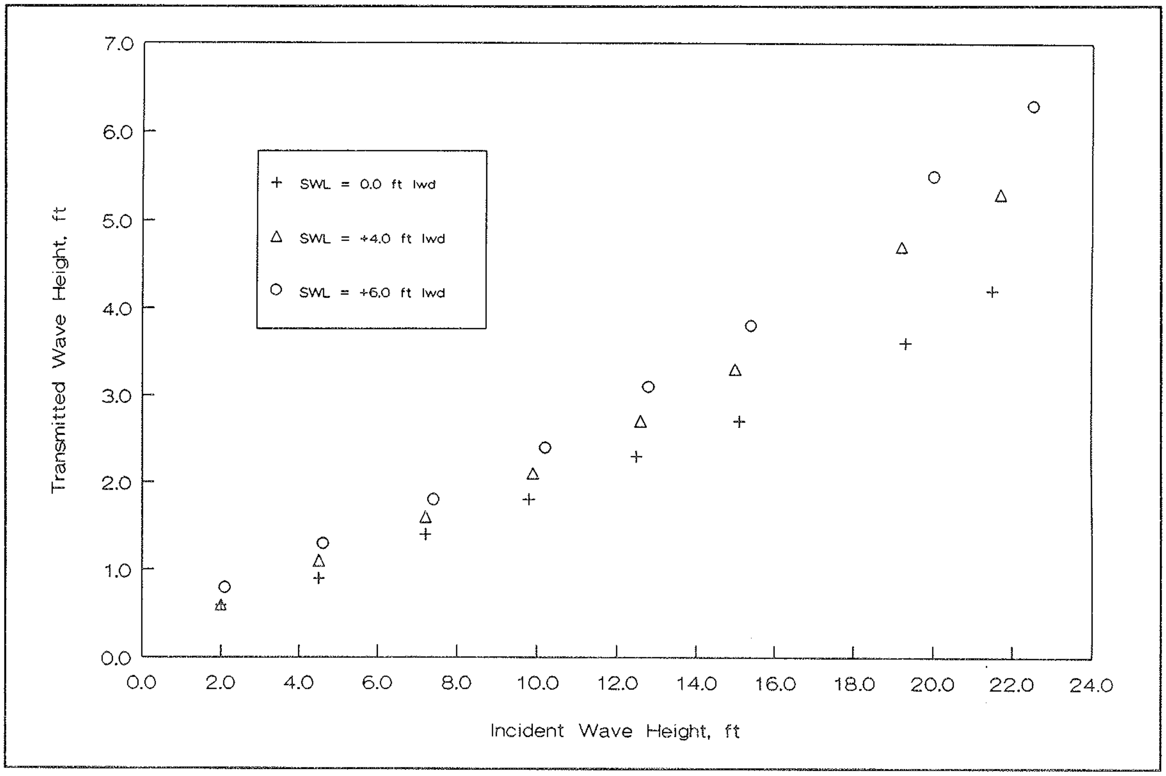

Figure 20. Transmission test results for Plan R6; 11.6-sec wave period

Plan	Average C_t for Indicated Wave Period			Average C_t All Periods
	7.0 sec	9.0 sec	11.6 sec	
No Improvement	0.18	0.21	0.30	0.23
R1	0.16	0.18	0.25	0.20
R2	0.16	0.17	0.23	0.19
R2A	1	1	1	1
R3	0.16	0.17	0.24	0.19
R4	0.16	0.17	0.24	0.19
R5	0.17	0.18	0.25	0.20
R6[2]	0.14	0.15	0.20	0.16
R6	0.17	0.17	0.24	0.19
R6[3]	0.20	0.21	0.27	0.23

[1] Transmission tests too limited to establish averages.
[2] swl = 0.0-ft lwd
[3] swl = +6.0-ft lwd

The data in the previous table, graphically presented in Figures 21-24, show that Plans R2, R3, R4, and R6 yielded similar results. Plan R5, the same as Plan R4 except the toe-to-toe spacing from the existing breakwater was increased from 75 to 100 ft, produced slightly larger transmitted wave heights than were observed for Plan R4.

Stability of the existing structure, quantified as percent damage to the lakeside and harbor-side armor, is summarized as follows:

Plan	Percent Damage to Existing Structure	
	Lakeside Armor	Harbor-side Armor
No Improvement	2.5	5.0
R1	0.5	1.0
R2	0.0	0.5
R2A	0.0	1.5
R3	1.0	1.0
R4	0.5	1.0
R5	0.0	0.0
R6	0.0	0.5

Lakeside and harbor-side damages are also presented in Figures 25 and 26. These data show that all improvement plans reduced damages to an acceptable level, i.e., 2 percent or less by number of the primary armor stone placed.

Discussion

During the present investigation, wave heights of about 15 ft or less were observed behind the reefs for 11.6-sec, 19-ft incident waves, thus eliminating most damage to the existing breakwater. At the onset of this study, it was desired to reduce 11.6-sec, 19-ft waves to heights of about 13 ft. However, during the course of the study, it became apparent that the desired stability and transmission responses could be achieved with up to 11.6-sec, 15-ft waves behind the submerged reefs. Actually, stability tests were conducted with maximum wave heights of about 22 ft incident on the reefs and only very minor damage was observed for any plan. Thus, it was decided to relax the 11.6-sec, 13-ft criterion to 11.6-sec, 15-ft maximum waves.

Stability observations reported herein are consistent with the earlier work of Jackson (1967) and the recent investigation of Carver, Dubose, and Wright (1993). In all cases, the existing breakwater was found to be stable for incident wave heights of at least 15 ft.

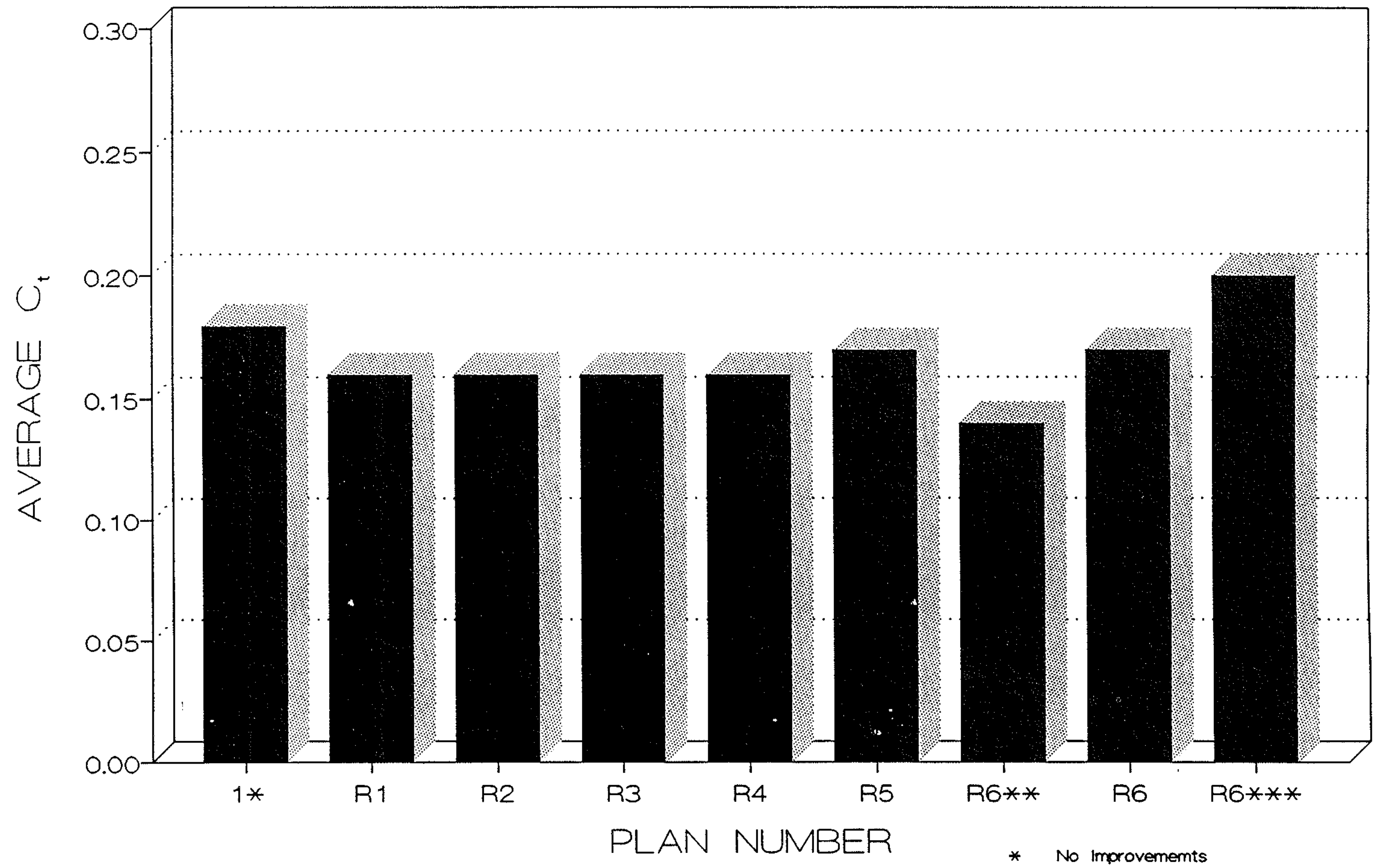

Figure 21. Average values of C_t ; 7-sec wave period

Figure 21. Average values of C_t; 7-sec wave period

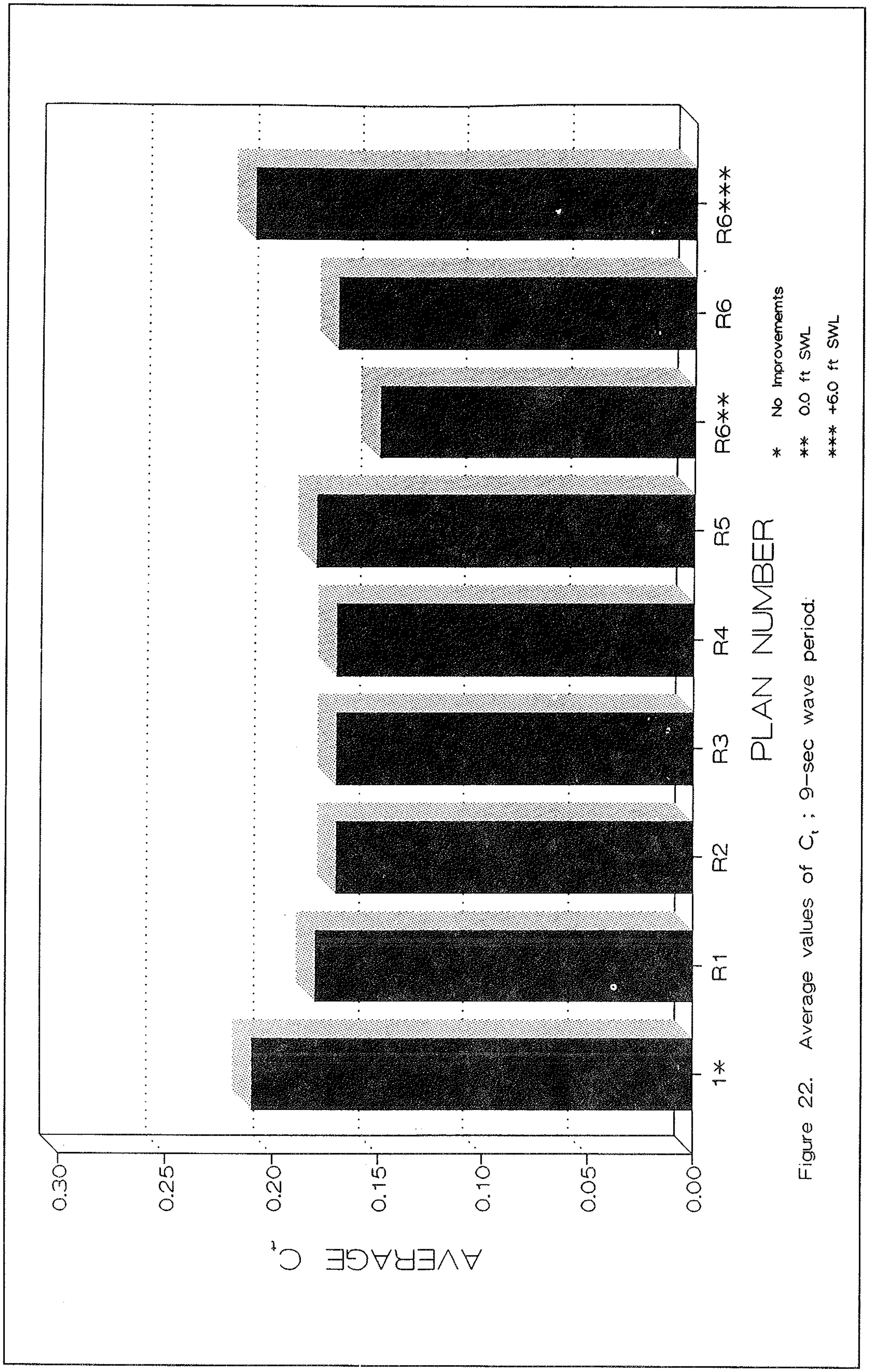

Figure 22. Average values of C_t ; 9-sec wave period.

Figure 22. Average values of C_t; 9-sec wave period

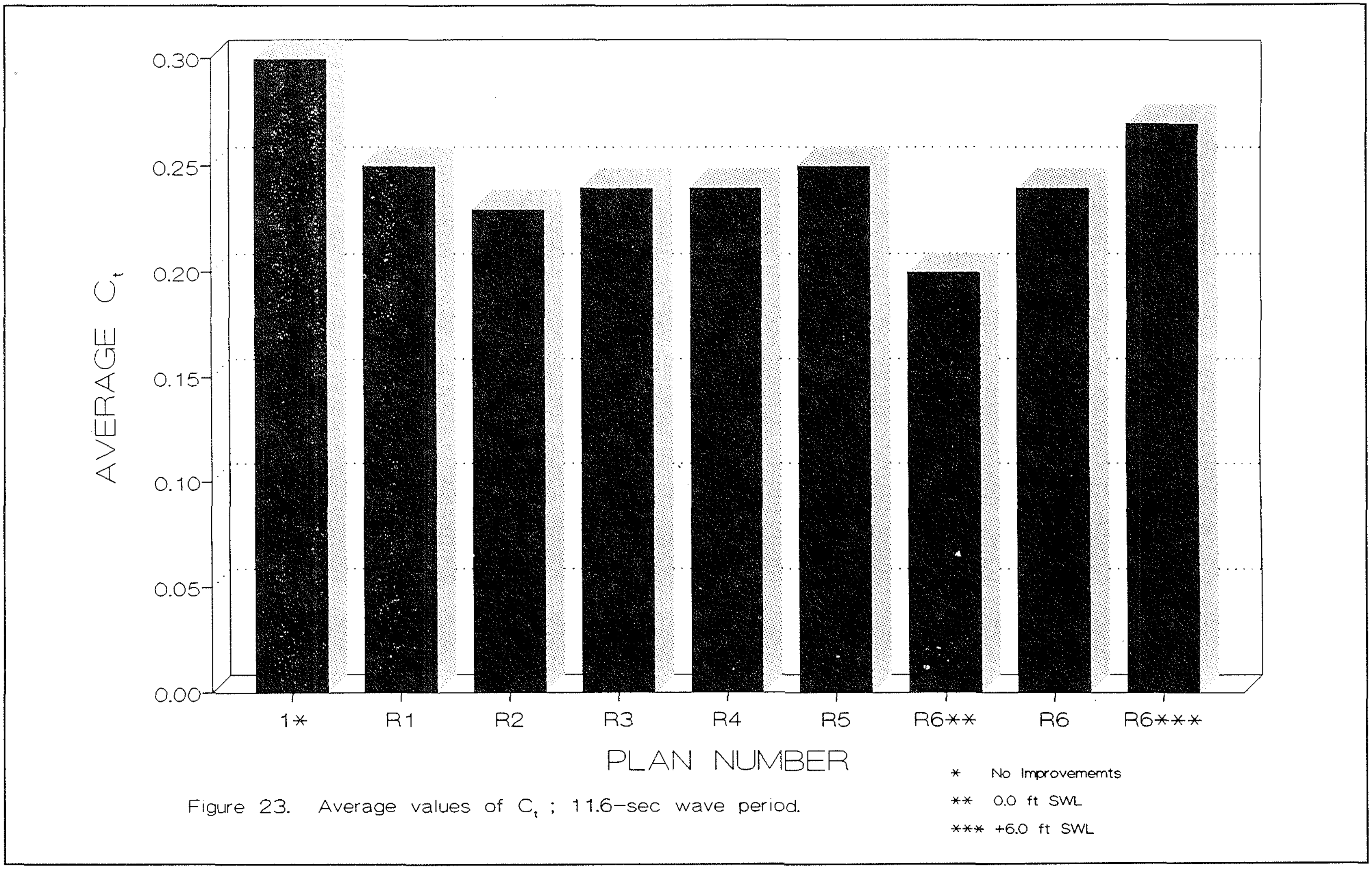

Figure 23. Average values of C_t; 11.6-sec wave period

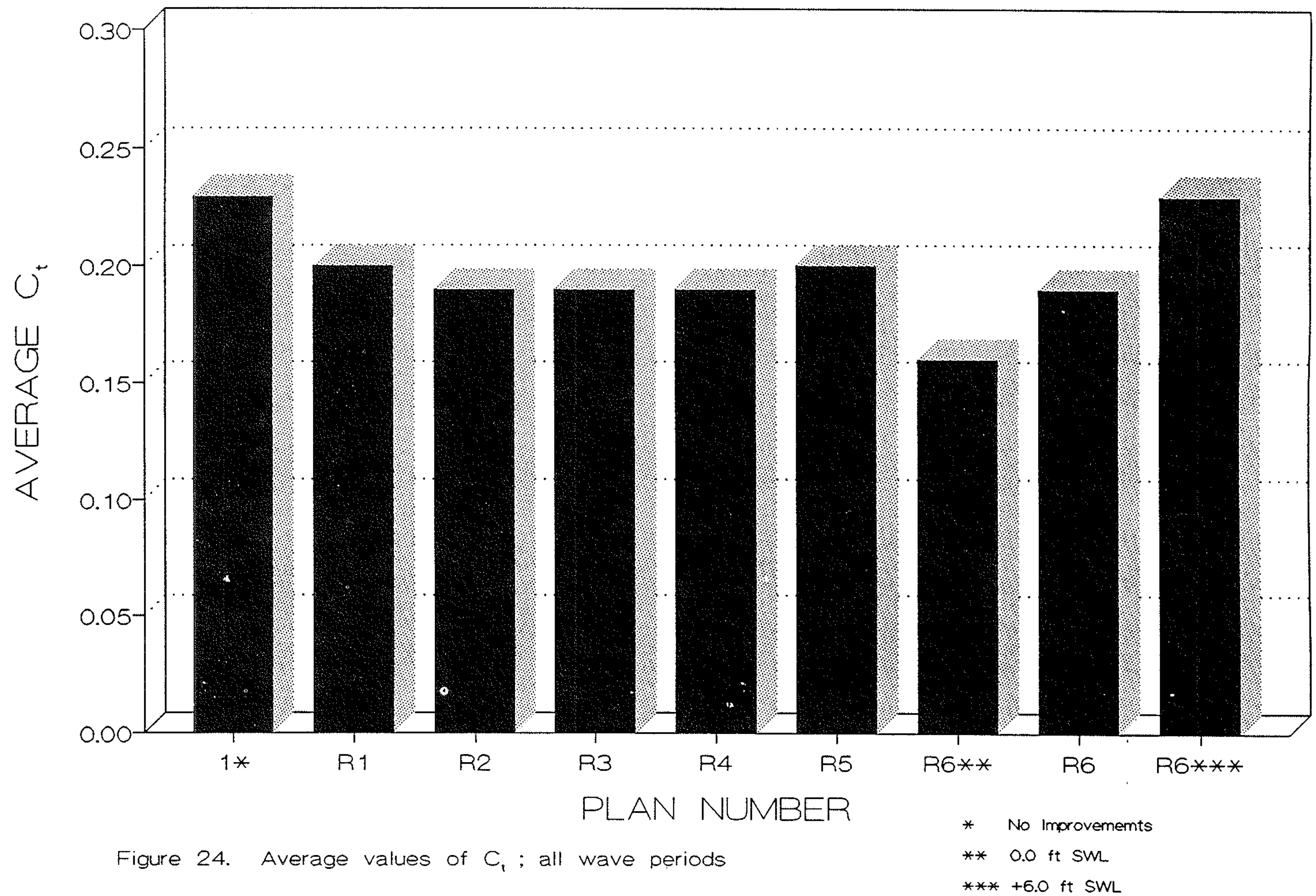

Figure 24. Average values of C_t; all wave periods

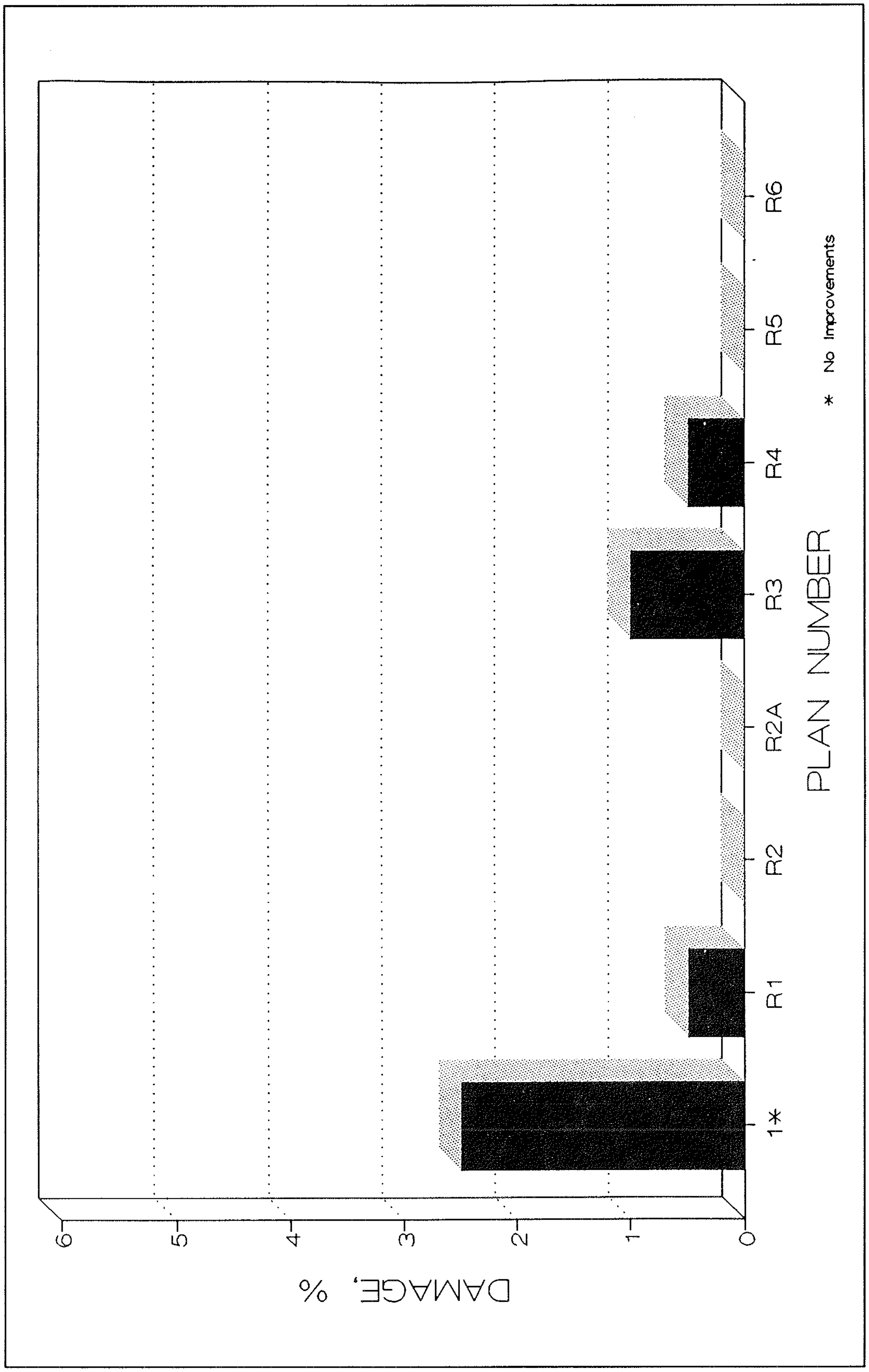

Figure 25. Percent damage incurred by lakeside armor

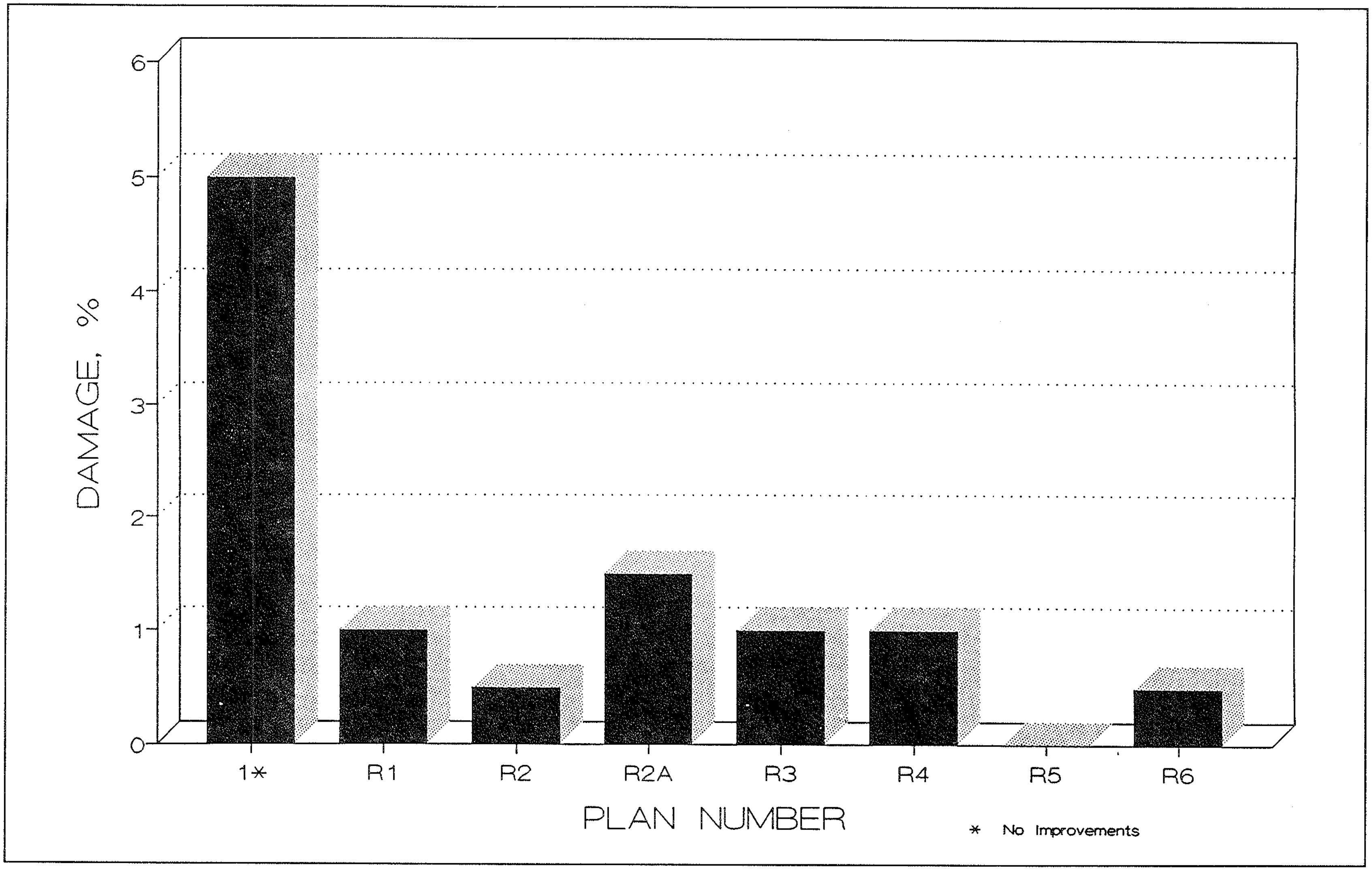

Figure 26. Percent damage incurred by harbor-side armor

It should be noted that the actual section recommended for construction will differ slightly from the plans tested herein. This section (Figure 27) will incorporate a 3-ft-thick sand blanket that was not represented in the model tests. Also, the 1- to 5-ton armor, which proved to be stable in the model, is a minimum size and, depending on final quarry yield, a larger armor stone weight range (2- to 6-ton, 3- to 7-ton, and etc.) could be used.

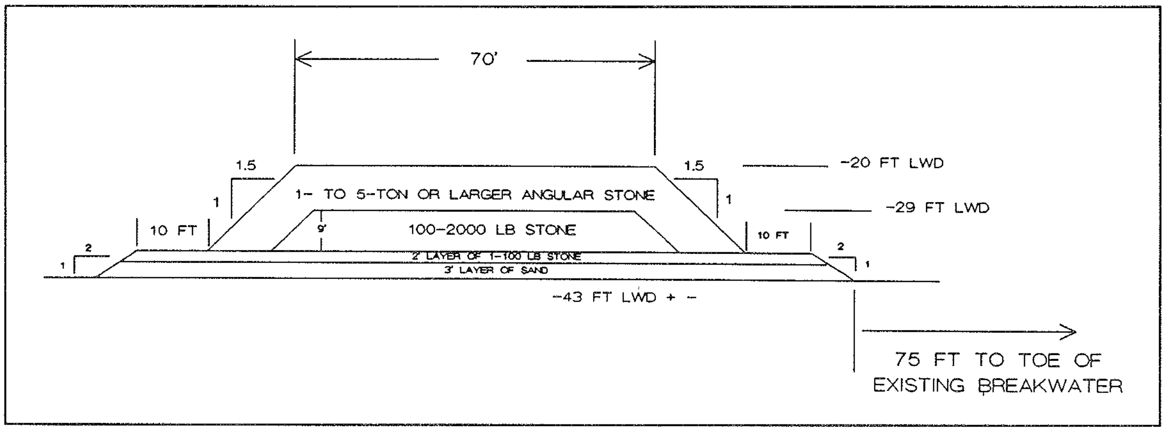

Figure 27. Recommended reef breakwater cross section

4 Conclusions

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

a. The model was able to accurately replicate prototype wave energy transmission, as evidenced in Figures 4 and 5.

b. Test results for the various improvement plans show that all structures tested were successful in reducing 7- and 9-sec, 5-ft incident waves to heights of 1 ft or less behind the existing breakwater. Also, as desired, wave heights of about 15 ft or less were observed behind the reef for 11.6-sec, 19-ft incident waves, thus eliminating most damage to the existing breakwater.

c. The 5-ton armor stone used for Plans R1 and R2 was completely stable at crest elevations of -10 and -20 ft lwd.

d. The 3- to 5-ton stone used on Plan R2A was considered marginal when extended to the -10-ft lwd crest; however, this same stone mix proved acceptable when used on Plan R3 at a -20-ft lwd crest elevation.

e. Stability of the 1- to 5-ton stone used on Plans R4 and R5 was very similar and considered to be acceptable with about 3 to 4 percent of the original stone volume being displaced.

f. Increasing the toe-to-toe spacing of the reef from the existing breakwater from 75 to 100 ft slightly improved stability of the existing structure and slightly increased transmission.

g. The objective of this study, as stated in Chapter 1, was met.

h. Plan R6 appears to yield the largest reduction in wave energy transmission in concert with acceptable stability and minimal cost.

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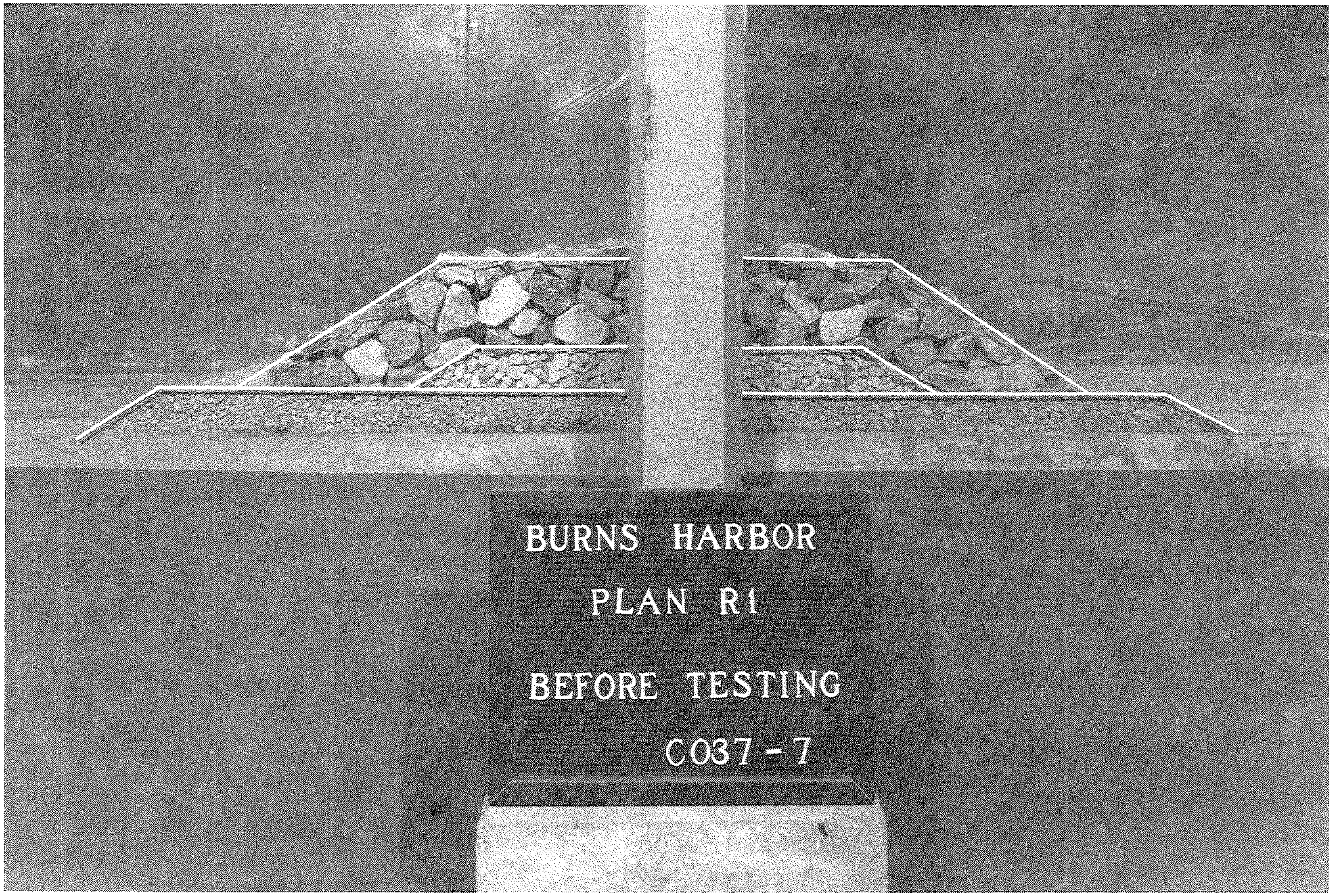

Photo 1. End view of Plan R1 before wave attack

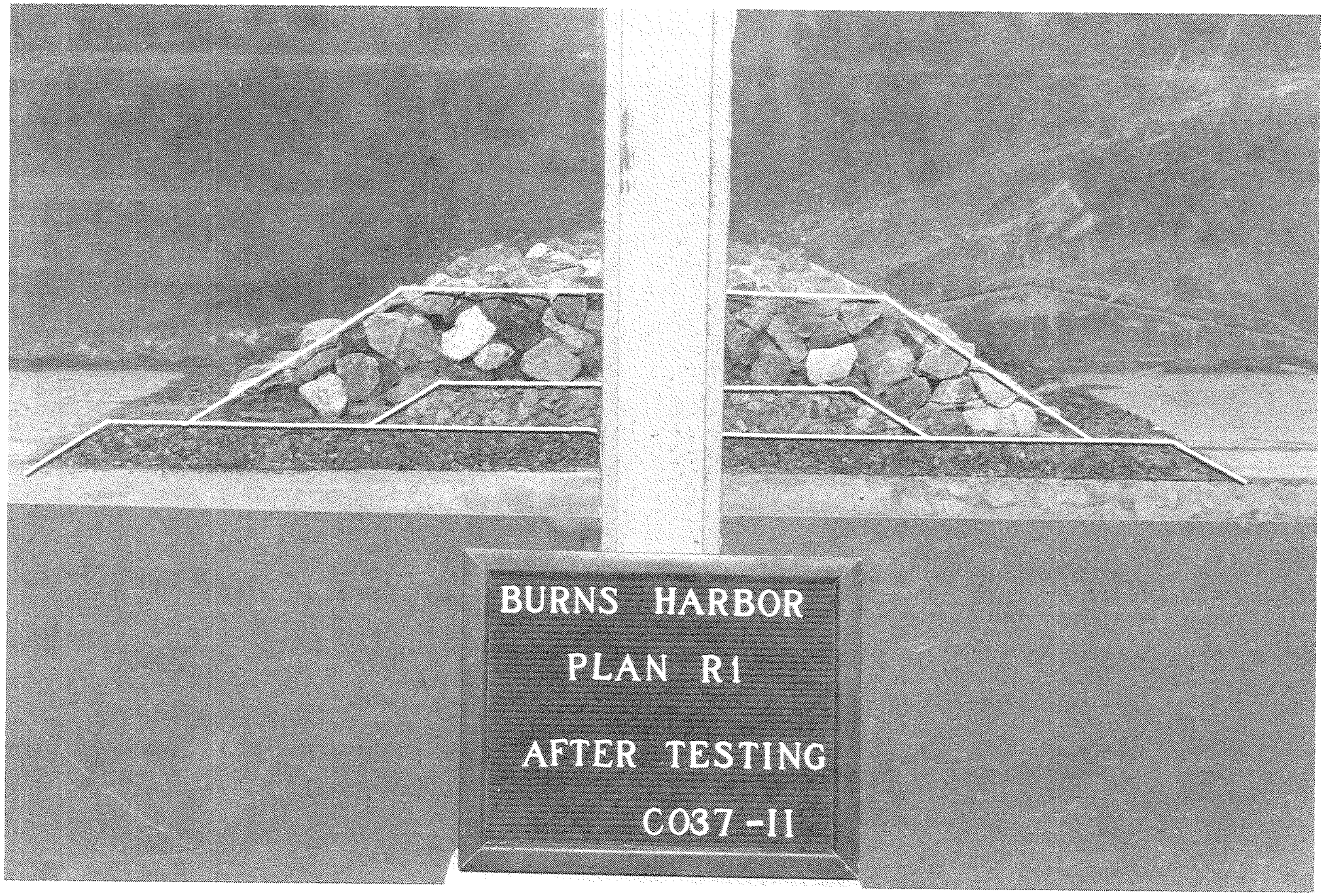

Photo 2. End view of Plan R1 after wave attack

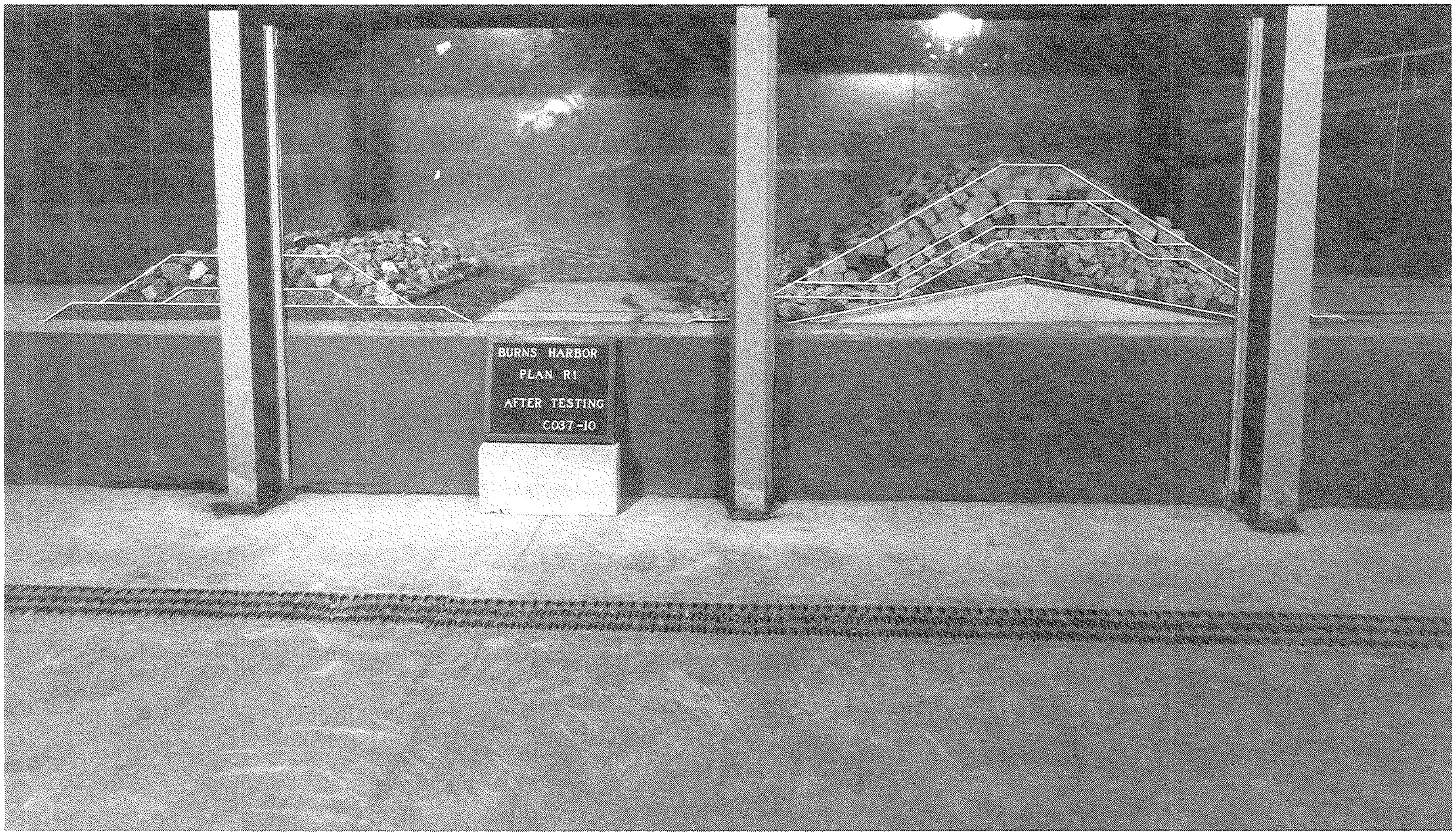

Photo 3. End view of Plan R1 (reef and existing breakwater) after wave attack

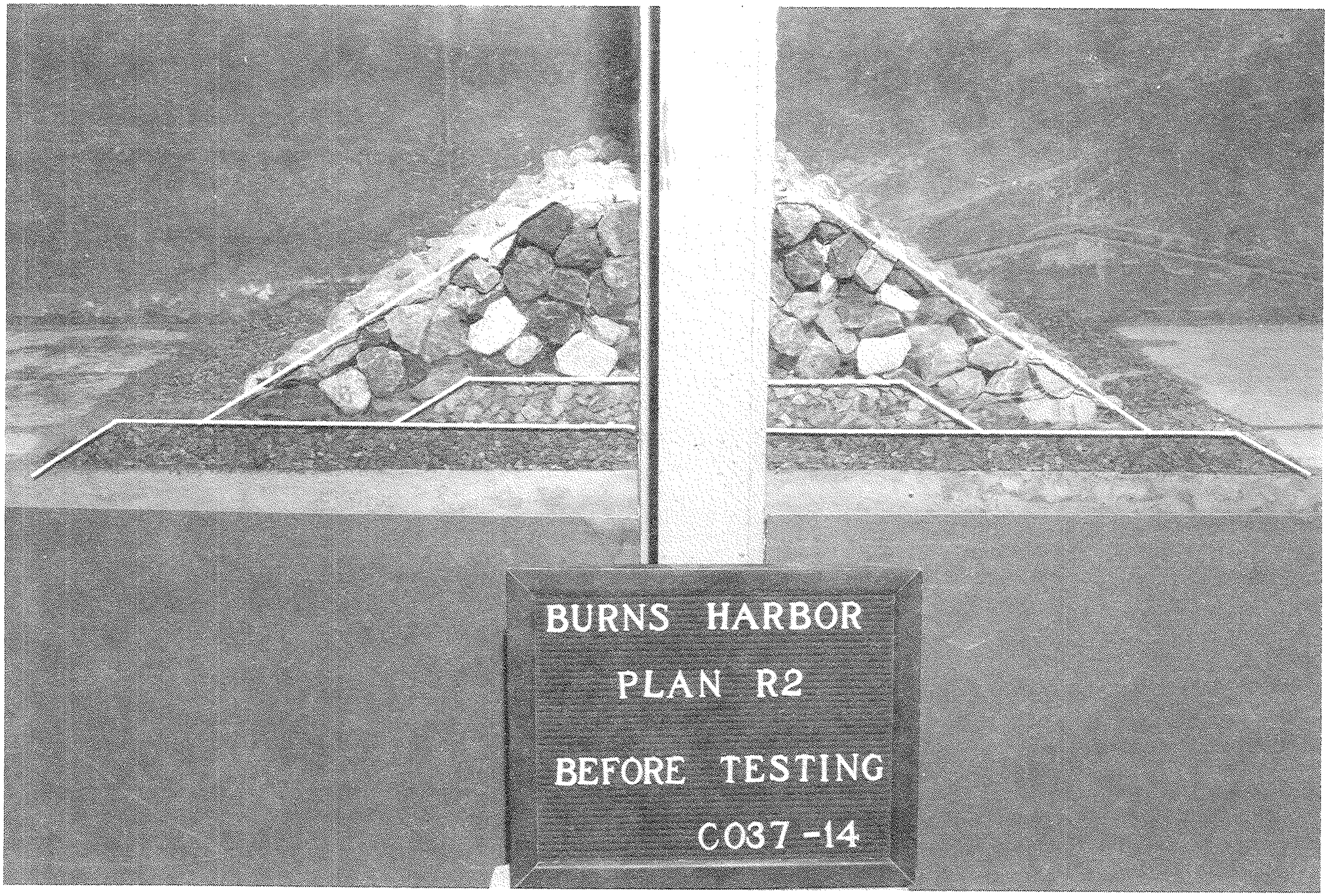

Photo 4. End view of Plan R2 before wave attack

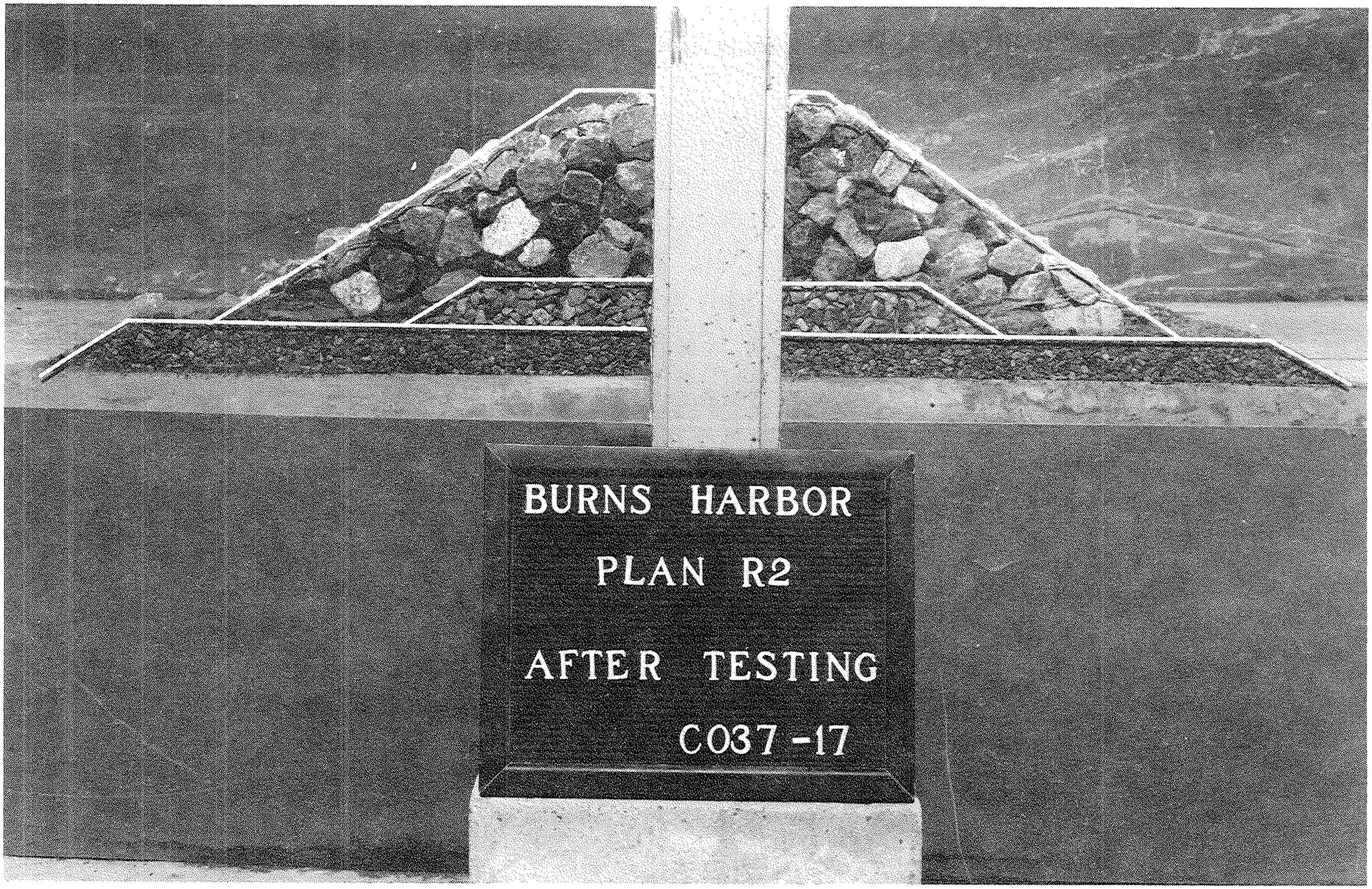

Photo 5. End view of Plan R2 after wave attack

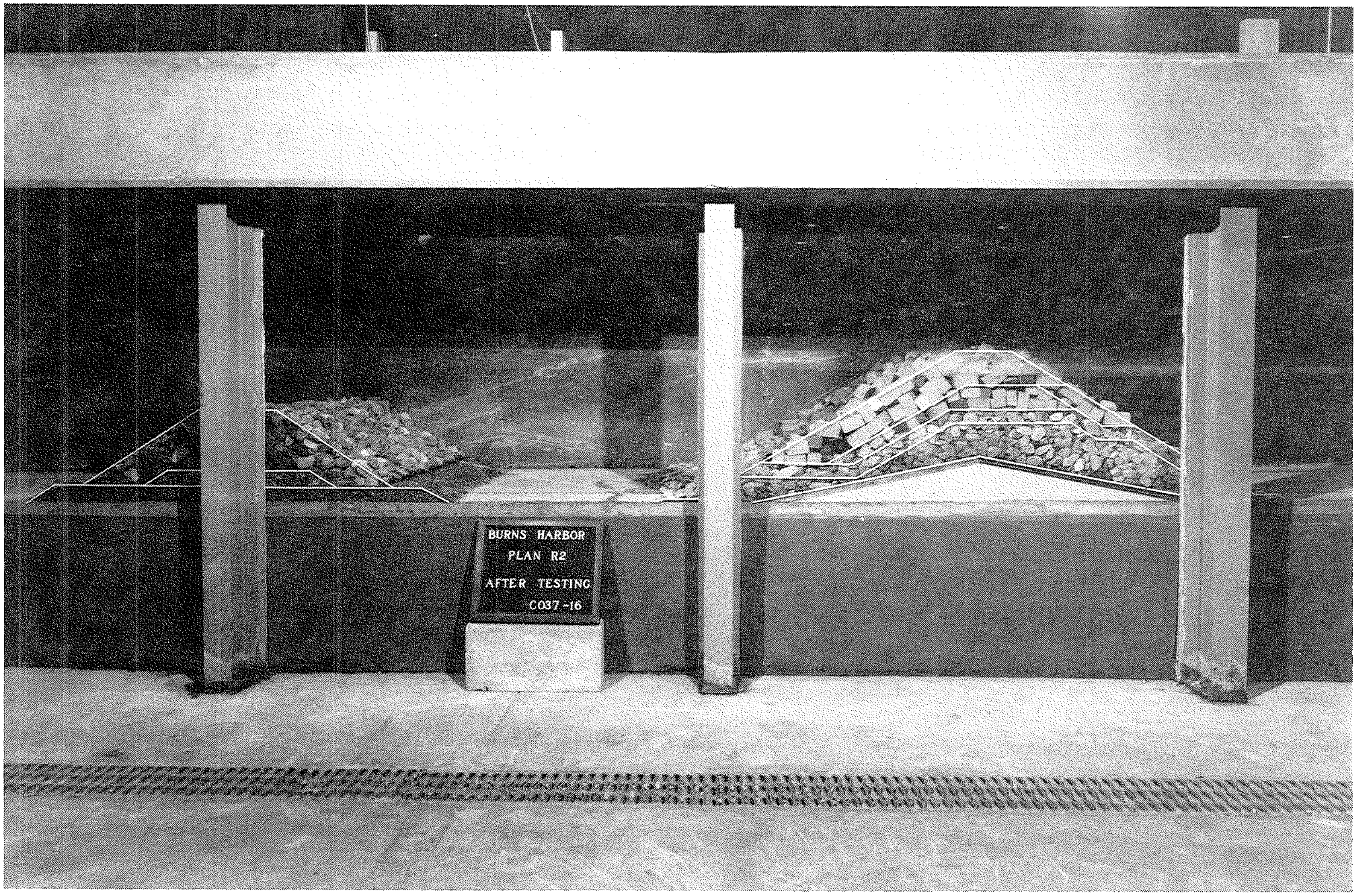

Photo 6. End view of Plan R2 (reef and existing breakwater) after wave attack

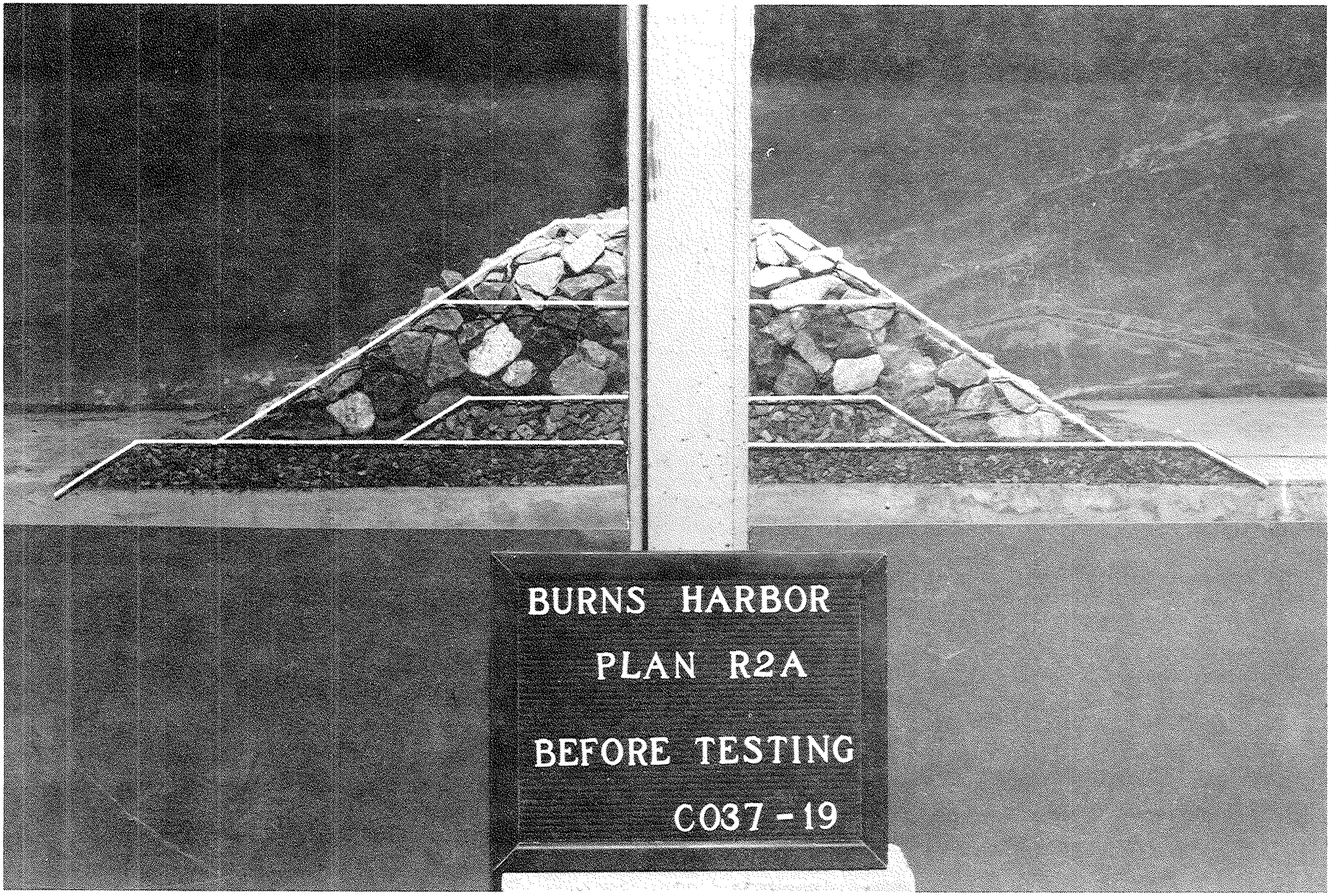

Photo 7. End view of Plan R2A before wave attack

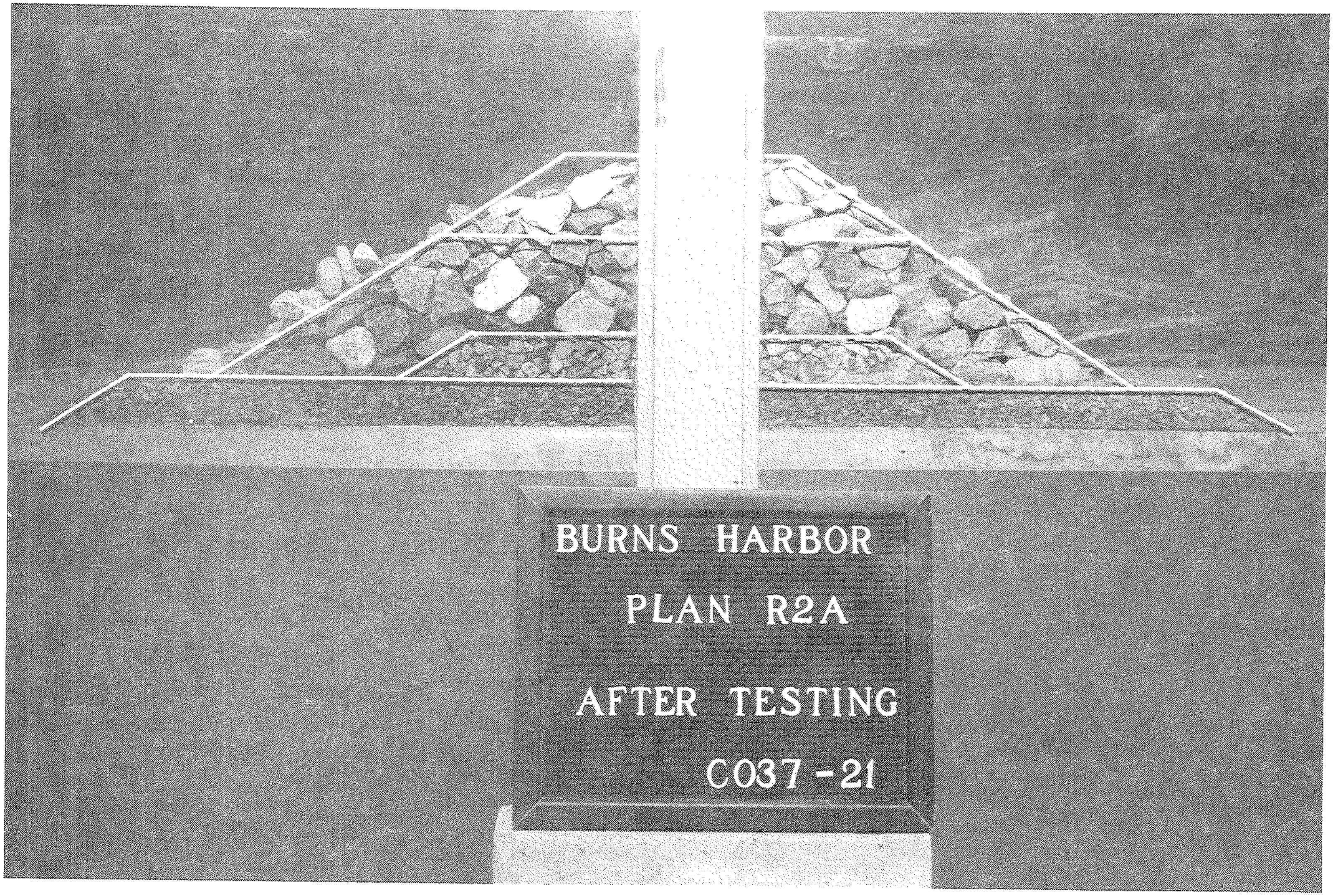

Photo 8. End view of Plan R2A after wave attack

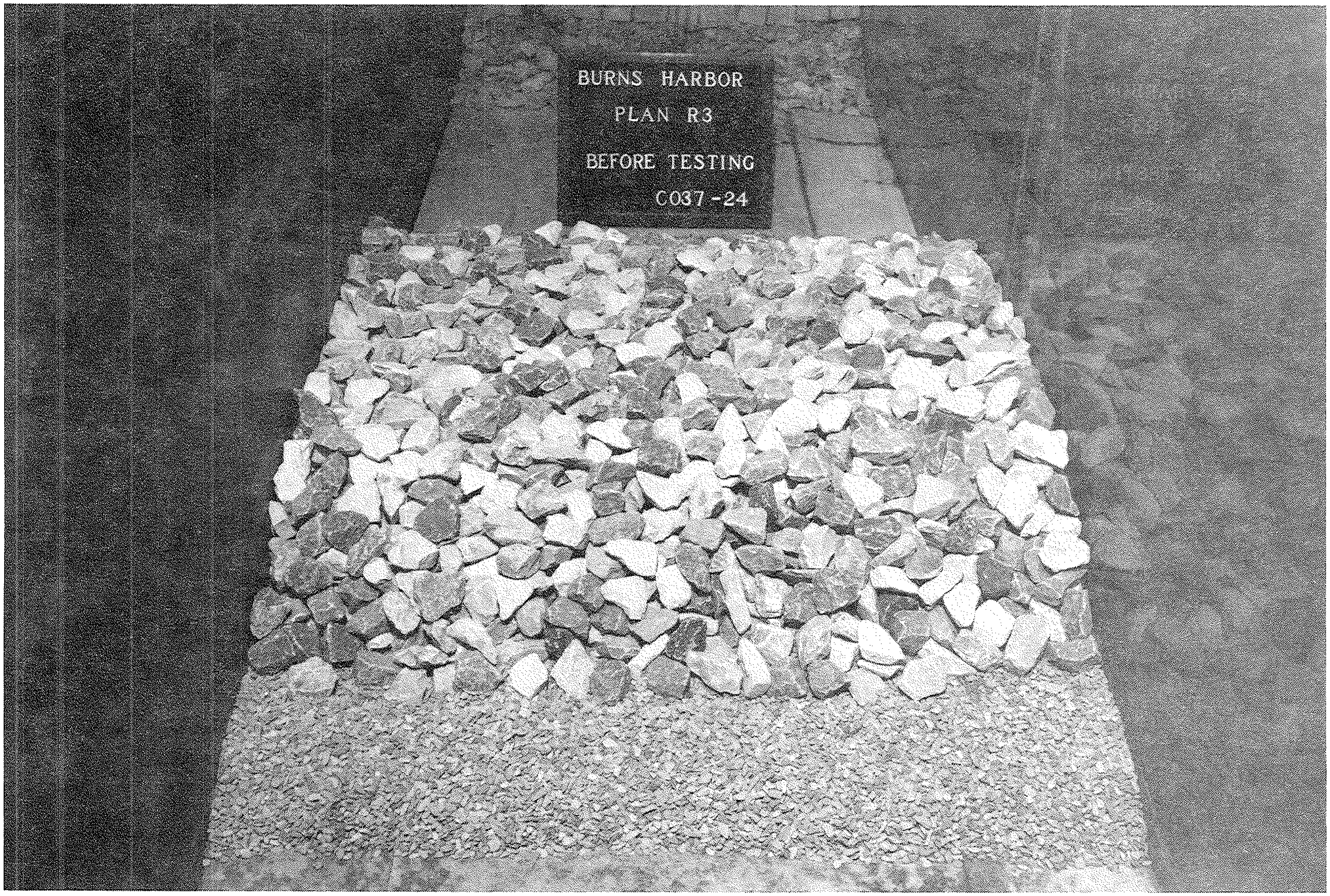

Photo 9. Lakeside view of Plan R3 before wave attack

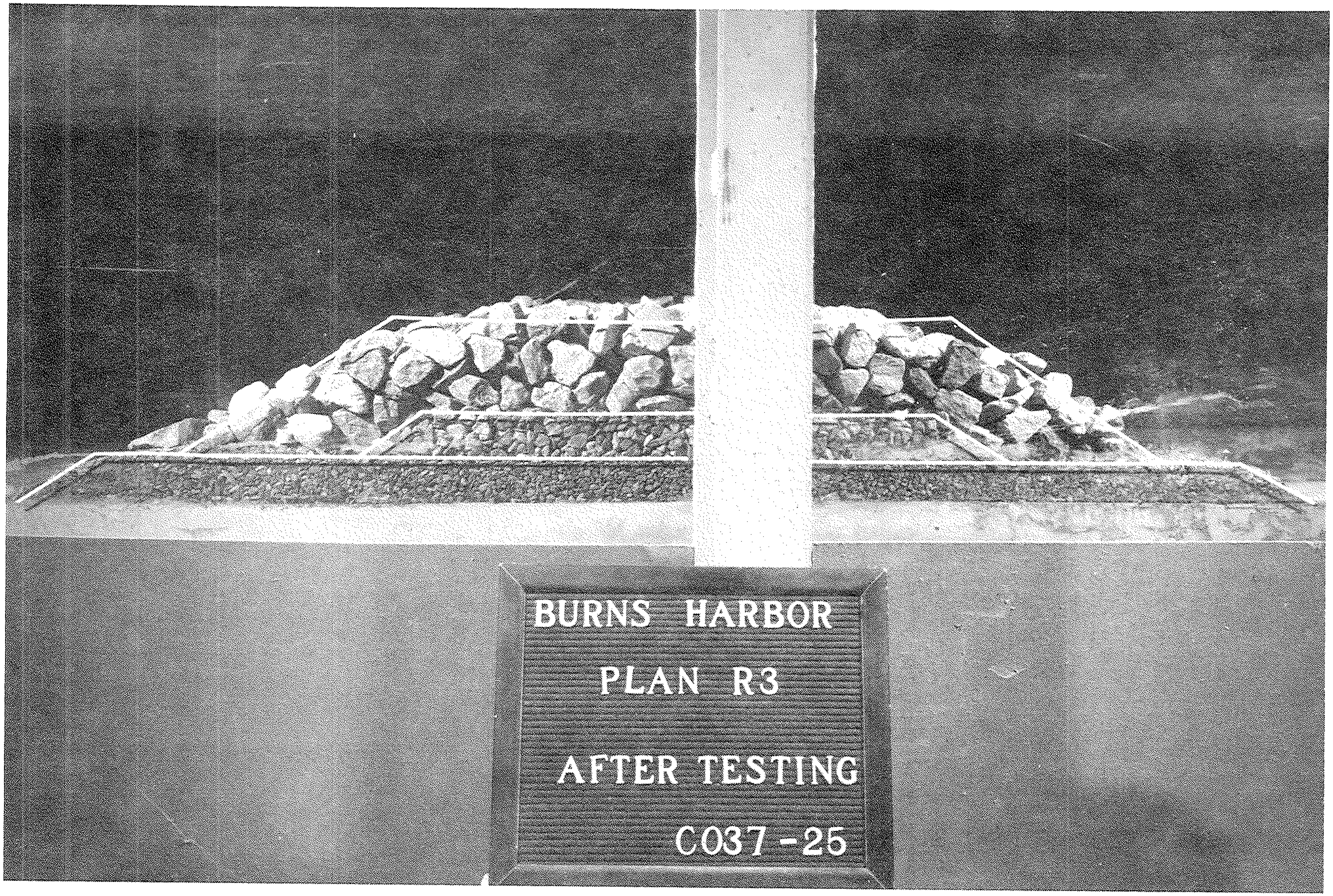

Photo 10. End view of Plan R3 after wave attack

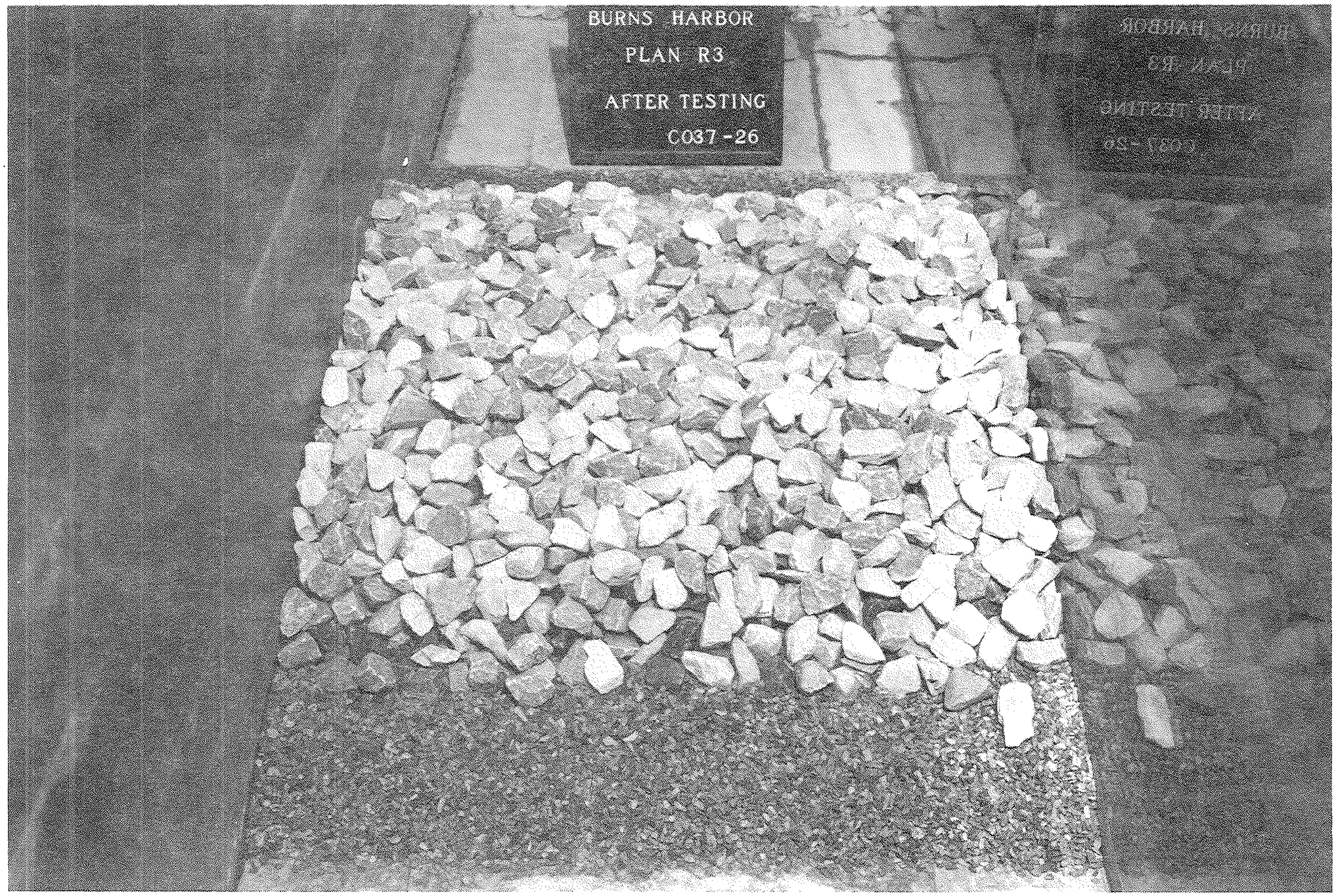

Photo 11. Lakeside view of Plan R3 after wave attack

Photo 12. Lakeside view of Plan R4 (existing breakwater in background) after wave attack

Photo 13. End view of Plan R4 after wave attack

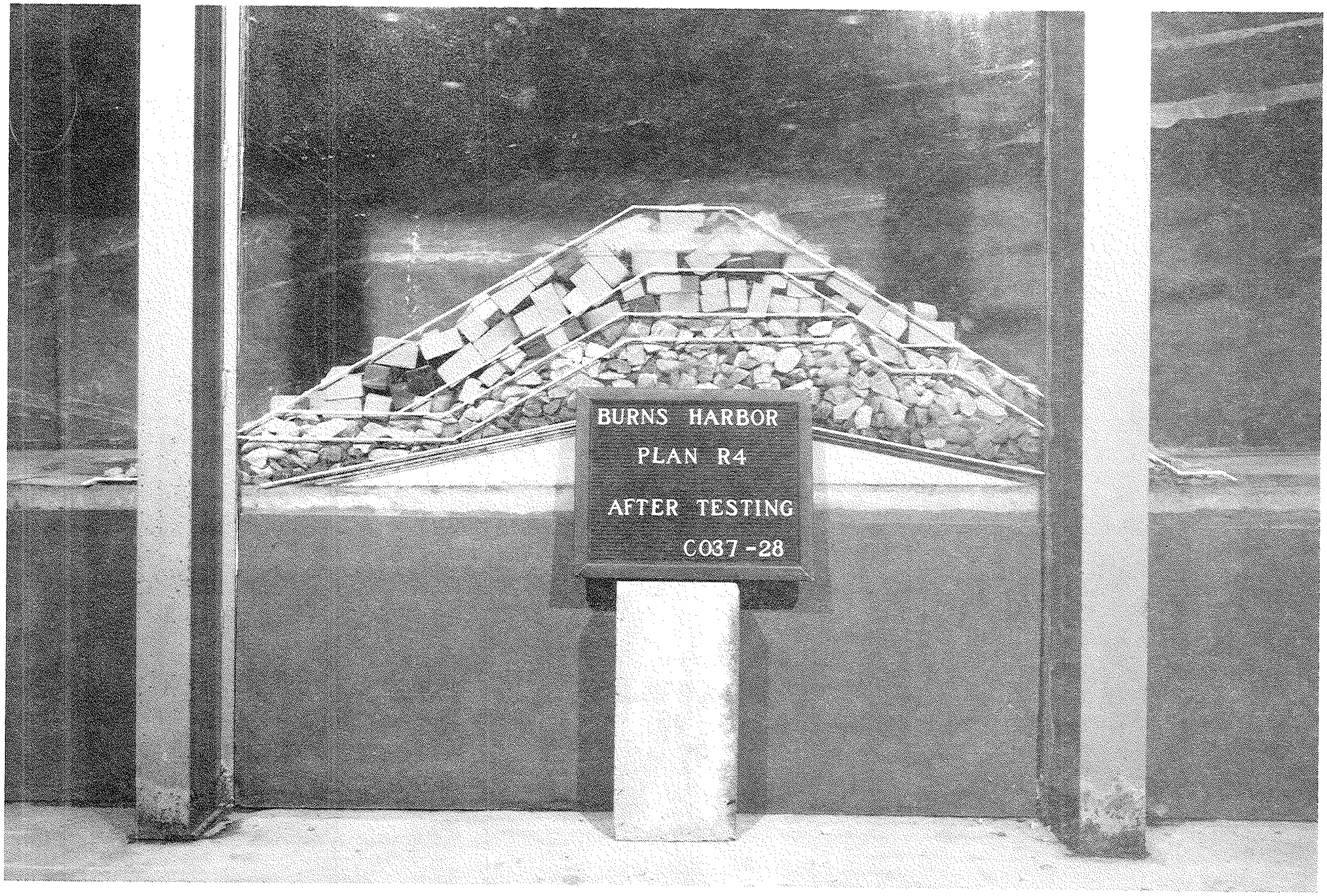

Photo 14. End view of existing breakwater as protected by Plan R4

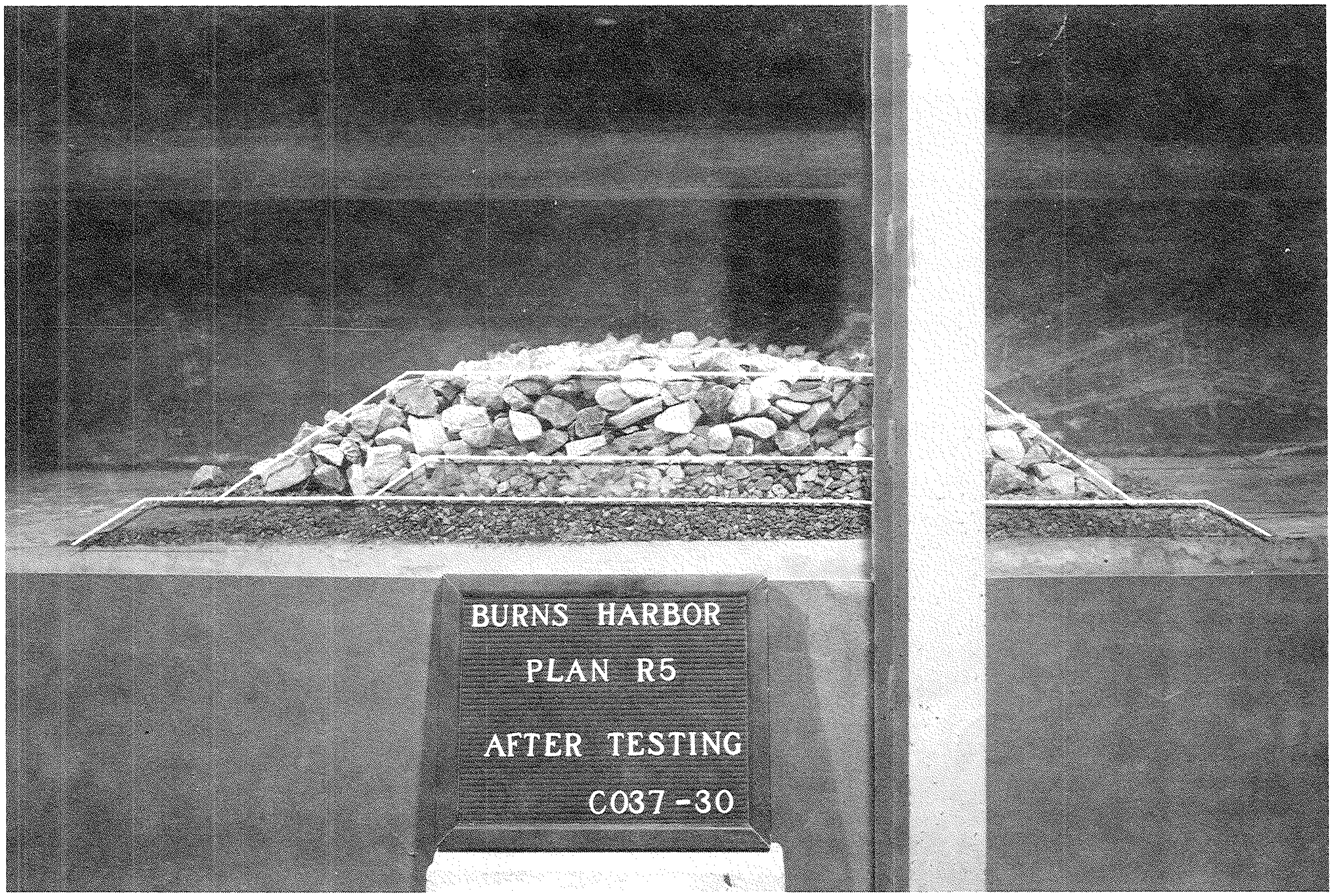

Photo 15. End view of Plan R5 after wave attack

Photo 16. Lakeside view of Plan R5 after wave attack

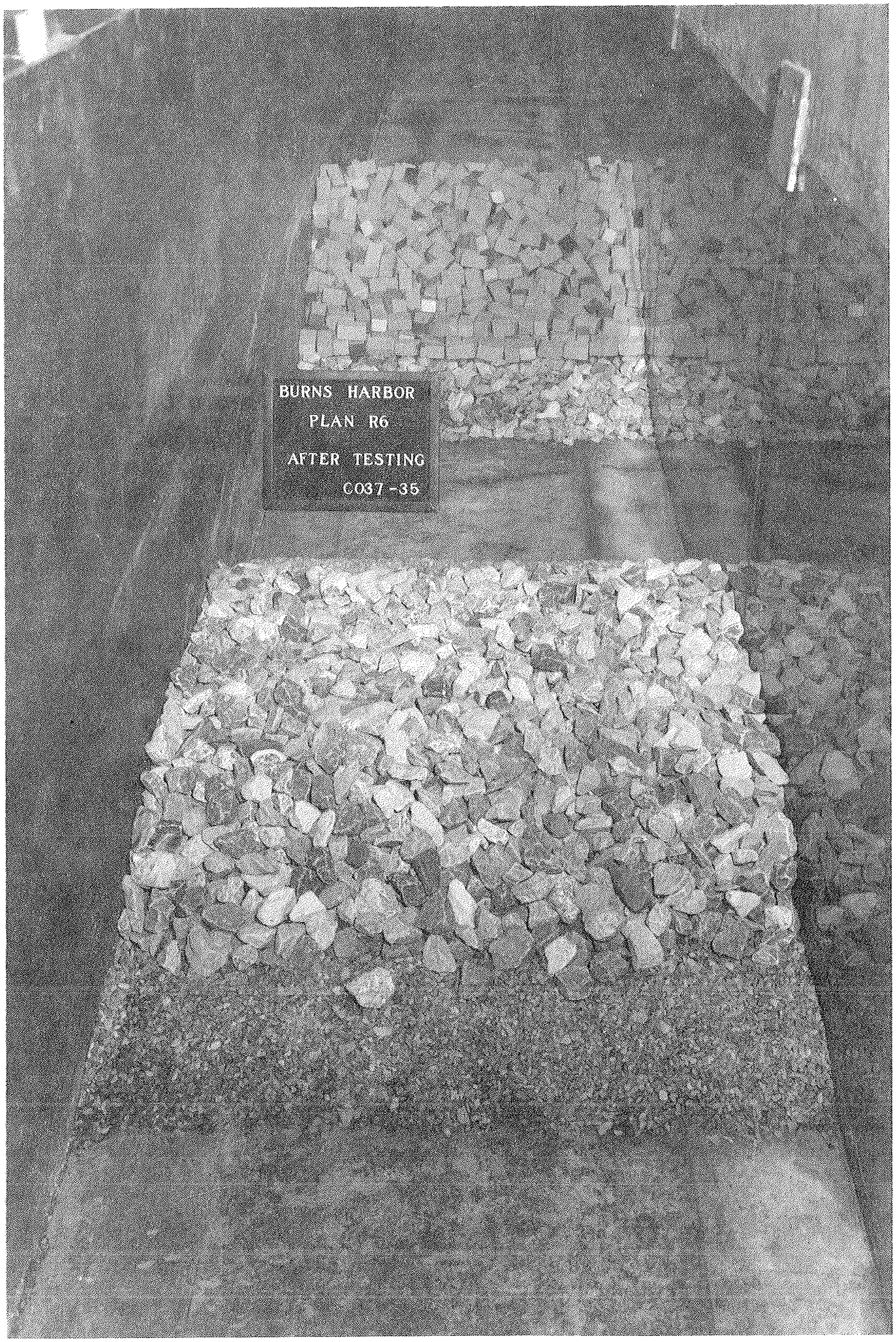

Photo 17. Lakeside view of Plan R6 (existing breakwater in background) after wave attack

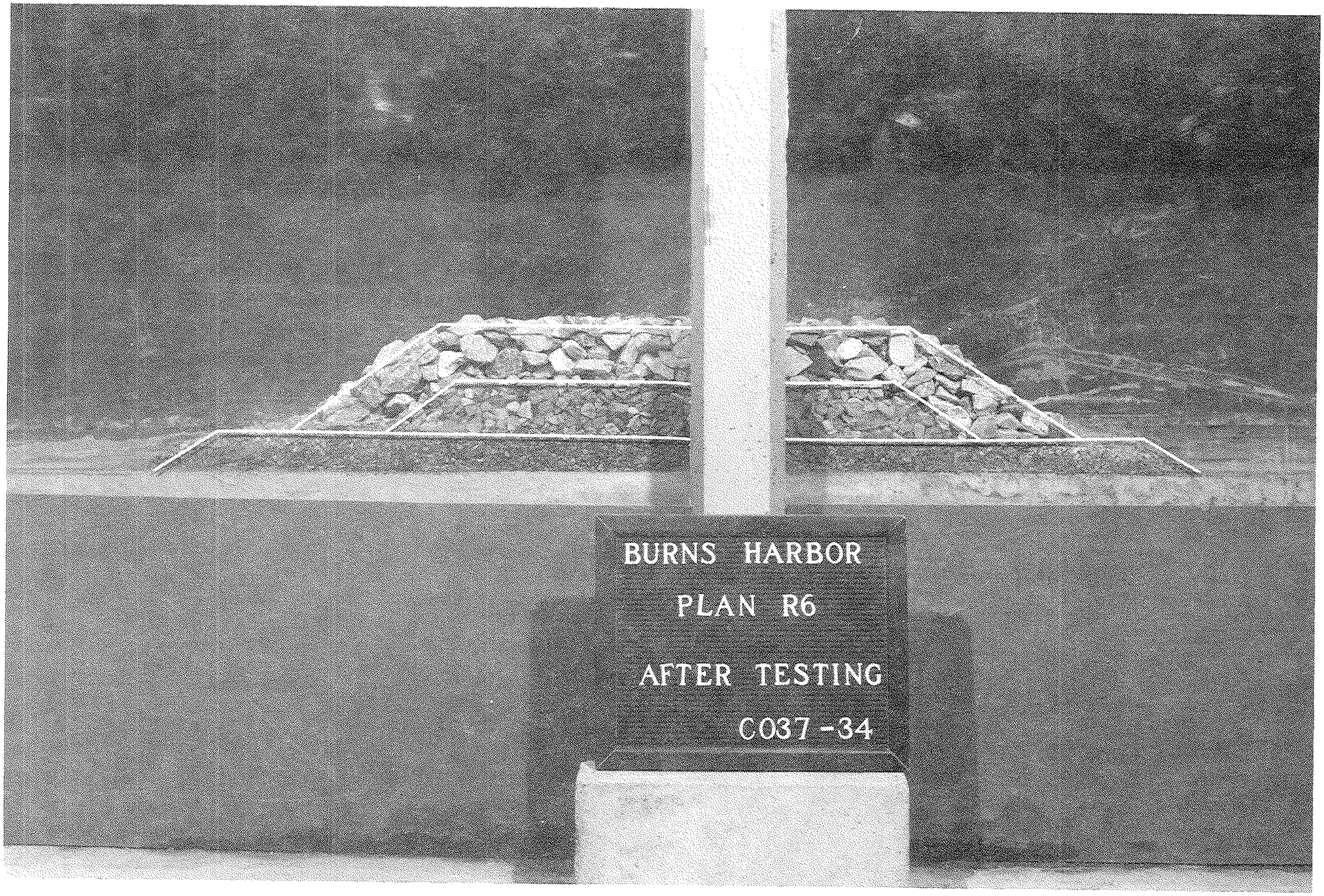

Photo 18. End view of Plan R6 after wave attack

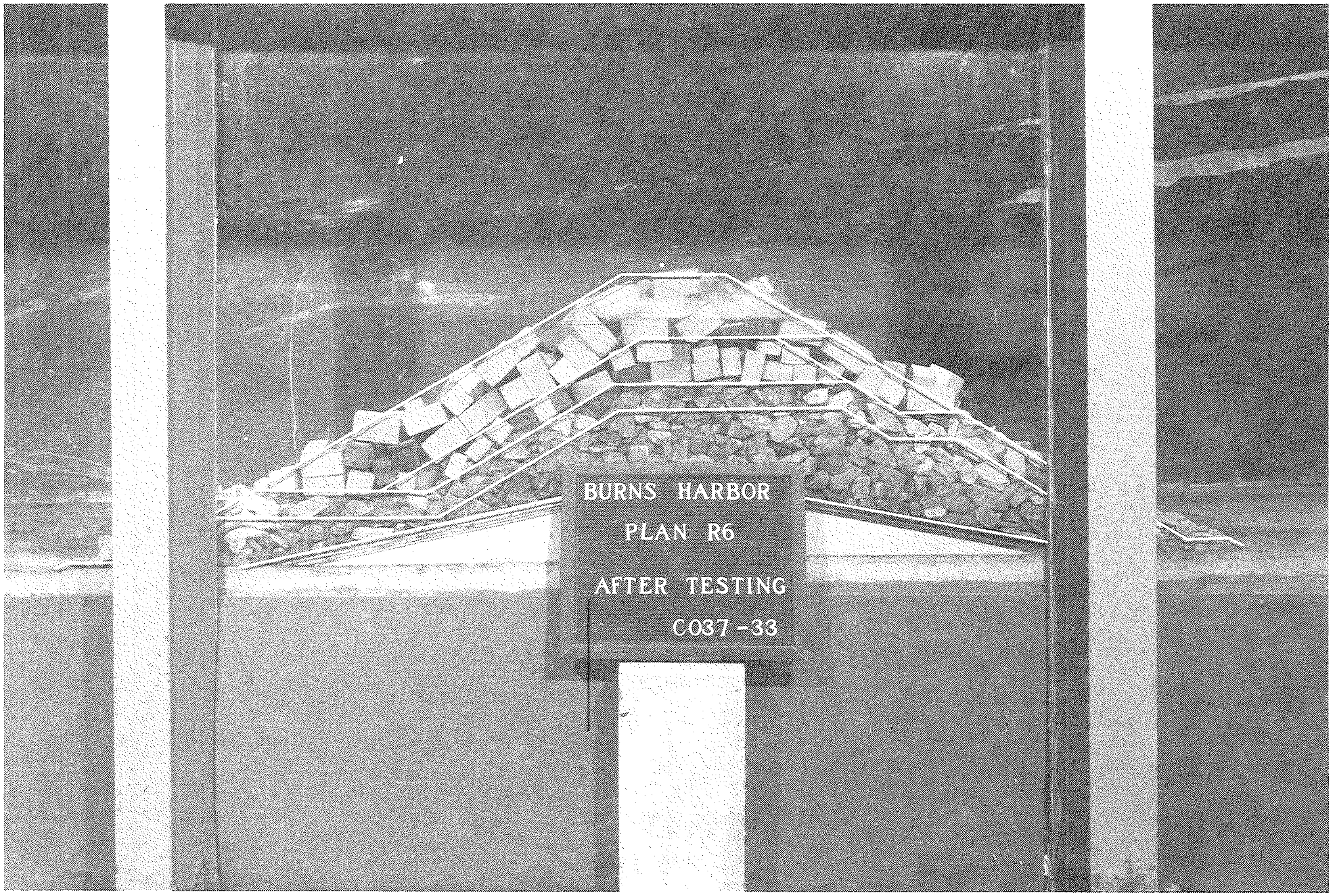

Photo 19. End view of existing breakwater as protected by Plan R6

Photo 20. Harbor-side view of existing breakwater as protected by Plan R6

Appendix A Notation

H_{mo}	Zero-moment wave height, ft
T_p	Wave period of peak energy density of spectrum, sec
L	Length
T	Time
L^2	Area
L^3	Volume
H_i	Incident wave height, ft
H_t	Transmitted wave height, ft
C_t	Transmission coefficient (H_t/H_i)

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13. ABSTRACT *(Maximum 200 words)*

A two-dimensional model study of a proposed reef breakwater for protection of the existing, damage-prone, rubble-mound breakwater at Burns Waterway Harbor was conducted. Sufficient 1:36-scale undistorted flume tests were conducted such that an optimum submerged reef could be selected. Specifically, it was desired to quantify performance (stability/transmission response) in terms of structure height and width, location relative to the existing breakwater, and stone size and gradation.

Seven improvement plans were considered. All significantly improved stability of the existing breakwater and reduced transmitted wave heights to some extent. Test results for the various improvement plans showed that all structures tested were successful in reducing 7- and 9-sec, 5-ft incident waves to heights of 1 ft or less behind the existing breakwater. Also, as desired, wave heights of about 15 ft or less were observed behind the reef for 11.6-sec, 19-ft incident waves, thus eliminating most damage to the existing breakwater. Five-ton armor stone was completely stable at crest elevations of -10 and -20 ft low-water datum. Increasing the toe-to-toe spacing of the reef from the existing breakwater from 75 to 100 ft slightly improved stability of the existing structure and slightly increased transmission.

14. SUBJECT TERMS

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