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August 1993

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Waterways Experiment Station

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

by Robert D. Carver, Willie G. Dubose, Brenda J. Wright
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WES

Prepared for U.S. Army Engineer District, Chicago

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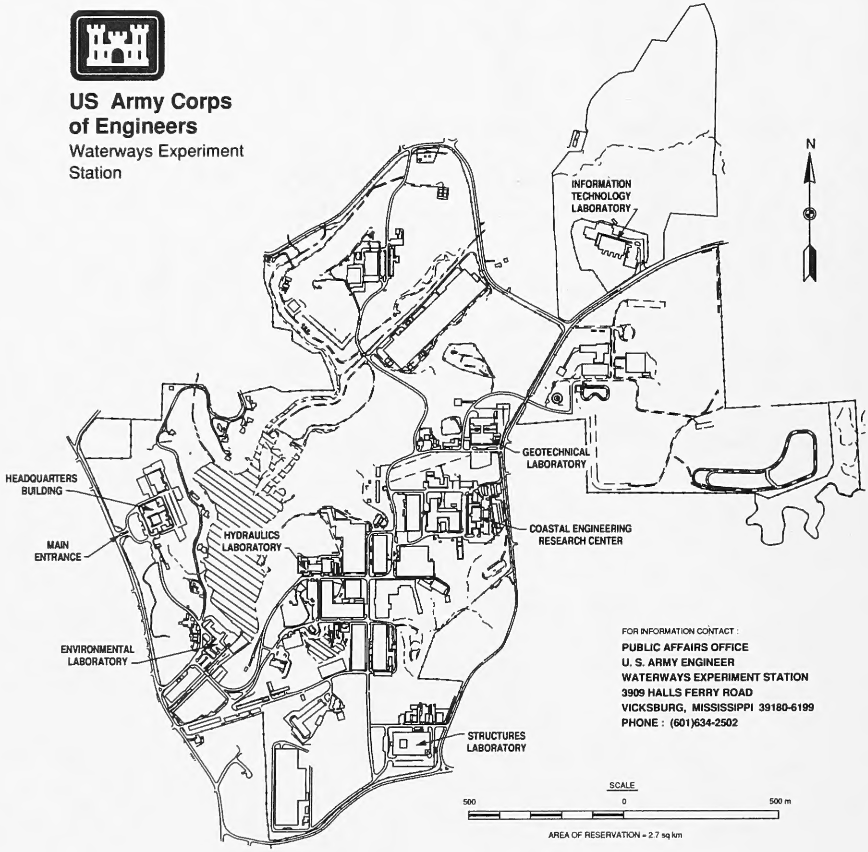

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Contents

Preface

The model investigation described herein was requested by the US Army Engineer District, Chicago (NCC), in a letter to the US Army Engineer Waterways Experiment Station (WES) dated 5 June 1990. Funding authorization was granted by NCC in Intra-Army Order No. NCC-IA-90-27EJ, dated 5 June 1990. Model tests were conducted during the period January-December 1992.

The study was conducted by personnel of the WES 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 (CW-R). Tests were conducted by Mr. W. G. Dubose and Ms. B. J. Wright, WRB, WDD, under the direction of Mr. R. D. Carver, Principal Investigator, WRB, WDD. This report was prepared by Messrs. Carver and Dubose and Ms. Wright.

Ms. Heidi Pfeiffer coordinated testing efforts for NCC. During the course of this study, communication was maintained by monthly progress reports, conferences, 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 units as follows:

Multiply	By	To Obtain
feet	0.3048	metres
miles (U.S. statute)	1.609347	kilometers
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic meters
square feet	0.09290304	square meters
tons (2,000 lb, mass)	907.1847	kilograms

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 industries 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 10-16 and 16-20 tons on the trunk and head, respectively.

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 (an 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.

Purposes of Model Investigation

The purposes of the investigation described herein were as follows:

a. Evaluation of stability/transmission using a 1985 condition survey and February 1987 storm conditions.

b. Evaluation of stability/transmission improvements with 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.

c. Evaluation of stability/transmission improvements with a berm breakwater attached to the lakeside of existing structure.

d. Evaluation of stability/transmission improvements achieved with addition of 18-ton angular stone on the lakeside, harbor side, and/or raising the crest with one layer of 18-ton stone.

e. Evaluation of stability/transmission improvements with existing stone reworked into special placement at crest.

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 segment of a concrete wave flume 11 ft wide and 245 ft long (Figure 2). A 1V on 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 gages 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 gage, 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 gages. Wave signal generation and data acquisition were controlled using a DEC MicroVax I computer. Wave data analyses were accomplished using a DEC VAX 3600.

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, which was oversized to aid in compensating for transmission scale effects, 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 (W/10), which was equal in size to the core (due to core oversizing for transmission effects), then was added by shovel and smoothed to grade by hand or with trowels. Limestone blocks used in the cover layers and sublayer (W/2) 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. It was necessary at the original building and each major rebuilding to readjust the armor blocks in the cover and sublayer to reproduce the desired prototype wave transmission. Once the prototype transmission had been reproduced on the existing structure, it was not rebuilt unless substantial damage was observed or the plan to be tested called for changes that would purposely affect existing wave transmission. If slight damage, i.e., a few randomly displaced armor blocks, did occur to the existing structure during any specified test plan, the displaced armor blocks were replaced back on the existing structure.

Simulation of Existing Structure (Plans 1, 1A, 1A1, 1A2 and 1A3)

Plan 1 (Figure 3) was constructed to a crown elevation of +13 ft lwd and used armor slopes of 1V on 1.7H, both lakeside and harbor-side. The lake-side 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. The distribution of individual stone weights within these mixtures was as follows:

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

Transmission tests of Plans 1, 1A, and 1A1

Initial tests consisted of checking the transmission response of Plan 1 (Photos 1-3). Three prototype spectra were selected for verification of the model breakwater. Characteristics of the chosen prototype spectra were as follows:

T_p, sec	H_i, ft	H_t, ft	C_t
7.1	9.2	1.4	0.15
9.2	6.6	1.1	0.17
11.6	15.6	3.6	0.23

The first attempt to reproduce the desired wave conditions in the model yielded the following results:

T_p, sec	H_i, ft	H_t, ft	C_t
7.0	7.2	0.8	0.11
9.0	6.0	0.9	0.15
11.8	17.0	3.7	0.22

Peak periods of these spectra are in good agreement with the prototype; however, both the incident and transmitted wave heights are low for the 7- and 9-sec periods and most importantly, the amount of energy transmitted through the structure is low for these periods. Therefore, it was decided to attempt to increase the porosity of the structure by rebuilding the armor in a more random manner (Plan 1A shown in Photos 4 and 5), increase the energy levels of the 7- and 9- sec spectra, decrease the energy level of the 11.8-sec spectra, and repeat the tests. Results of the initial tests of Plan 1A were as follows:

T_p, sec	H_i, ft	H_t, ft	C_t
7.0	9.4	1.4	0.15
9.0	6.7	1.2	0.18
11.9	16.0	4.8	0.30

These data show good agreement for the 7- and 9-sec wave periods; however, energy transmission for the 11-sec period is too high. Finally, the structure was rebuilt (Plan 1A1) and retested with the following results:

T_p, sec	H_i, ft	H_t, ft	C_t
7.2	9.8	1.4	0.14
8.9	6.9	1.1	0.16
11.9	15.6	3.7	0.24

Figure 4 presents C_t as a function of wave period for the prototype data, Plan 1, Plan 1A, and Plan 1A1, with Plan 1A1 showing excellent agreement with the prototype.

Stability tests of Plan 1A1

Upon completion of the transmission tests, armor stability was investigated by subjecting Plan 1A1 to progressively larger wave heights and observing the number of armor stones displaced from the structure. Testing with the 7- to 9-sec waves showed these conditions to become steepness limited before heights sufficient to produce damage could be reached; therefore, stability tests were concentrated at the 11-sec period. The first testing of Plan 1A1 produced the following results:

		Cumulative Number of Stones Displaced	
Step	H_{mo}, ft	Lakeside	Harbor Side
1	15.6	0	2
2	17.2	0	2
3	18.6	0	4
4	19.9	0	5
5	21.0	1	8
6	22.4	1	8
7	23.1	3	9

The structure was rebuilt and retested with the following results:

Step	H_{mo}, ft	Cumulative Number of Stones Displaced	
		Lakeside	Harbor Side
1	15.6	1	10
2	17.2	1	18
3	18.6	2	25
4	19.9	3	28
5	21.0	4	30
6	22.4	8	33
7	23.1	8	33

A comparison of the tabulated damages and the after-testing photos (Photos 6 and 7 after the initial test and Photos 8 and 9 after the repeat test) show the structure exhibited a higher level of movement in the repeat test. Results of this type are not necessarily unusual and generally reflect random differences in building that occur from one model structure to another and from one section to another in the prototype breakwater. Model damage patterns appear consistent with the prototype, with most of the damage occurring on the harbor side.

The threshold of instability is generally defined as the point at which 2-3 percent of the armor units are displaced from their original areas. The 3-ft-wide model section required about 450 and 200 armor stones on the lakeside and harbor side, respectively. Thus, displacement of more than 12 lakeside or more than 6 harbor-side armor units could be considered the onset of instability. Following this logic, Plan 1A1 would be considered stable for wave heights through 19.9 ft on the first test and a wave height somewhat less than 15.6 ft on the second test.

Rationale for Plan 1A2

A review of prototype photos revealed that, in some areas of the structure, armor stones were placed more randomly than they were in Plan 1A1. Therefore, it was decided that additional stability tests should be conducted with this more random configuration to quantify its effects. Thus, Plan 1A2 (Photos 10-12) was conceived. Plan 1A2 was identical to Plan 1A1, except that the armor was placed in a totally random manner.

Stability tests of Plan 1A2

Stability test results for Plan 1A2 were as follows:

Step	H_{mo}, ft	Cumulative Number of Stones Displaced	
		Lakeside	Harbor Side
1	15.6	0	0
2	17.2	1	1
3	18.6	3	4
4	19.9	4	7
5	21.0	4	9
6	22.4	4	9
7	23.1	5	11

Following completion of the first test, the structure was rebuilt and retested with the following results:

Step	H_{mo}, ft	Cumulative Number of Stones Displaced	
		Lakeside	Harbor Side
1	15.6	2	0
2	17.2	2	0
3	18.6	2	0
4	19.9	2	0
5	21.0	2	2
6	22.4	2	2
7	23.1	5	5

As evidenced above, initial and repeat test results were similar. Photos 13-15 show the structure after the initial test.

Transmission tests of Plan 1A2

Plan 1A2, with its totally random placement, was more porous than the previous structures. Consequently, one would expect to see an increase in transmitted wave energy. Tests conducted with the same incident conditions used on the previously investigated plans yielded the following results:

T_p, sec	H_i, ft	H_t, ft	C_t
7.0	10.2	1.9	0.19
9.0	6.9	1.5	0.22
11.6	15.6	4.3	0.28

These results are depicted graphically in Figure 5. A direct comparison of transmission coefficients between Plans 1A1 and 1A2 yields the following:

T_p, sec	Plan 1A1 C_t	Plan 1A2 C_t	Percent Increase
7.0	0.14	0.19	36
9.0	0.16	0.22	38
11.6	0.24	0.28	17

This substantial increase in transmission is consistent with the increase in stability.

Rationale for Plan 1A3

Further review of available prototype information (photos, soundings, records) confirmed that Plan 1A2 probably replicated prototype armor placement; however, it was felt that the W/2 and W/3 stone was probably more uniformly placed between -7 ft lwd and its top elevation of +3 ft lwd. Thus, Plan 1A3 (Photo 16), incorporating this change in the W/2 and W/3 placement, was conceived.

Transmission tests of Plan 1A3

Tests conducted with the same incident conditions used on previously investigated plans yielded the following results:

T_p, sec	H_i, ft	H_t, ft	C_t
7.0	10.7	1.6	0.15
9.0	7.1	1.3	0.18
11.6	15.6	3.8	0.24

These results, along with those obtained from previously investigated plans, are presented in Figure 6. As shown therein, Plan 1A3 gives transmission results that are in excellent agreement with the prototype.

Stability tests of Plan 1A3

Stability test results for Plan 1A3 were as follows:

Step	H_{mo}, ft	Cumulative Number of Stones Displaced	
		Lakeside	Harbor Side
1	15.6	1	0
2	17.2	1	1
3	18.6	1	2
4	19.9	1	3
5	21.0	2	5
6	22.4	4	5
7	23.1	5	5

The structure was rebuilt and an abbreviated repeat test was conducted with the following results:

Step	H_{mo}, ft	Cumulative Number of Stones Displaced	
		Lakeside	Harbor Side
1	15.6	1	4
2	18.6	1	7
3	21.0	2	13
4	23.1	2	14

Photos 17-19 show the structure after the repeat stability test.

Summary and Development of Stability Coefficients

As expected, all plans tested reproduced prototype wave energy transmission to some extent. Plans 1A1 and 1A3 most closely reproduced prototype wave energy transmission. Wave heights observed just prior to instability can be used in concert with the Hudson formula to determine corresponding stability coefficients by rearranging the formula as follows:

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

and substituting

γ_a = specific weight of armor unit, 145 pcf
W = weight, 12.4 tons = 24,800 lb

S_a = specific gravity of an individual armor unit relative to the water in which it is placed, $S_a = \gamma_a / \gamma_w$

$\cot \alpha$ = reciprocal of breakwater slope, 1.7

thus obtaining

$$K = 0.001495H^3$$

Stability results for the three plans investigated can be summarized as follows:

Plan	H_{mo}, ft*	Stability Coefficient
1A1	<15.6	<5.7
1A2	18.6	9.6
1A3	15.6	5.7
* Lesser value of initial and repeat tests.		

Development of Improvement Plans

A number of plans to improve stability of the existing structure were considered. Those chosen for model testing included placing a submerged breakwater 100-200 ft lakeward of the existing structure, attaching a berm breakwater to the lakeside of the existing structure, and the addition of 18-ton angular stone to the lakeside and/or harbor-side slope of the structure.

Base Conditions

In order to provide a baseline for comparing various improvement plans, transmitted wave heights were measured for a range of wave conditions on the existing structure. Results of these tests were as follows:

T_p, sec	Incident H_{mo}, ft	Transmitted H_{mo}, ft
swl[1] = 0.0 ft lwd		
7.0	2.3	0.5
7.0	4.0	0.7
7.0	6.7	0.9
7.0	9.7	1.2
7.0	11.7	1.5
7.0	2.9	0.7
(Continued)		
[1] swl = still-water level.		

(Concluded)		
T_p, sec	Incident H_{mo}, ft	Transmitted H_{mo}, ft
swl = 0.0 ft lwd		
9.0	4.7	1.0
9.0	6.8	1.3
9.0	8.3	1.4
9.0	9.4	1.6
9.0	10.7	1.7
11.6	2.0	0.7
11.6	4.2	1.2
11.6	6.6	1.6
11.6	9.1	2.0
11.6	11.5	2.5
11.6	13.9	3.0
11.6	15.9	3.6
swl = +4.0 ft lwd		
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

Figure 7 presents transmitted wave height as a function of incident wave height. As would be expected, these data show larger transmitted wave heights at the higher swl and at the longest wave period.

Reef Breakwaters (Plans 2, 2A, 3, 4, 4A, and 4A1)

This approach would use a reef breakwater of sufficient size to reduce 19-ft incident waves to about 13-ft waves in front of the existing structure. The first reef structure tested, Plan 2 shown in Figure 8, was constructed to an elevation of -20 ft lwd. It used a crown width of 72 ft, a stone weight of 5 tons, and was placed 150 ft lakeward of the existing structure. Testing with various incident conditions produced the following results:

	H_{mo}, ft Measured			
T_p, sec	Incident	Behind Reef	Behind Breakwater	C_t
7.0	2.3	2.1	0.5	0.22
7.0	4.1	3.7	0.7	0.17
7.0	6.1	5.3	0.9	0.15
7.0	7.6	6.6	1.1	0.14
7.0	10.2	8.3	1.4	0.14
7.0	11.7	9.4	1.6	0.14
9.0	2.8	2.7	0.7	0.25
9.0	4.8	4.4	1.0	0.21
9.0	7.0	6.4	1.3	0.19
9.0	8.2	7.3	1.5	0.18
9.0	9.4	8.4	1.7	0.18
11.6	1.8	1.6	0.7	0.39
11.6	3.9	3.4	1.1	0.28
11.6	6.3	5.3	1.6	0.25
11.6	8.6	6.9	2.0	0.23
11.6	13.1	10.0	2.9	0.22
11.6	16.8	11.9	4.1	0.24
11.6	18.6	13.0	4.8	0.26
11.6	19.8	14.0	5.4	0.27

The above data show that the chosen structure was successful in reducing a 19-ft incident wave to a height of slightly more than 13; however, maximum transmitted heights, though somewhat reduced, still exceeded 1 ft at all periods and 3 ft at the 11.6-sec period. Both the reef (Photo 20) and existing breakwater (Photo 21) were completely stable.

The second structure tested (Plan 2A shown in Figure 8) was identical to Plan 2, except the crest width was narrowed to 45 ft. Transmission test results were as follows:

T_p, sec	H_{mo}, ft Measured			C_t
	Incident	Behind Reef	Behind Breakwater	
7.0	2.2	2.1	0.5	0.23
7.0	3.9	3.7	0.7	0.18
7.0	6.7	6.1	0.9	0.13
7.0	9.6	8.7	1.4	0.15
7.0	11.5	10.2	1.7	0.15
9.0	2.8	2.8	0.7	0.25
9.0	4.6	4.6	1.0	0.22
9.0	6.5	6.4	1.3	0.20
9.0	7.9	7.7	1.5	0.19
9.0	9.2	8.9	1.7	0.18
11.6	1.9	1.9	0.7	0.37
11.6	4.9	4.9	1.3	0.27
11.6	6.6	6.6	1.7	0.26
11.6	9.0	9.0	2.2	0.24
11.6	13.7	13.7	3.2	0.23
11.6	17.5	17.0	4.5	0.26
11.6	19.7	18.8	5.3	0.27
11.6	21.7	20.2	6.1	0.28

As shown above, transmission results were similar to Plan 2. The reef was stable (Photo 22); however, several armor stones were displaced from the harbor-side slope of the existing breakwater (Photo 23).

It was decided to test a third submerged structure that would use about the same volume of material as Plan 2 and, thus, would have approximately the same cost as Plan 2. To accomplish this, the crown elevation was raised to -10 ft lwd and the crown width was narrowed to 36 ft (Plan 3, Figure 8, Photo 24). It was hoped that Plan 3 would provide a higher level of protection with about the same amount of material as Plan 2. Transmission test results were as follows:

T_p, sec	H_{mo}, ft Measured			C_t
	Incident	Behind Reef	Behind Breakwater	
7.0	2.3	1.9	0.5	0.22
7.0	4.0	3.5	0.7	0.18
7.0	7.0	5.6	1.0	0.14
7.0	10.0	7.4	1.3	0.13
7.0	12.2	8.5	1.6	0.13
9.0	3.0	2.7	0.7	0.23
9.0	4.9	4.4	1.0	0.20
9.0	7.0	6.0	1.3	0.19
9.0	8.2	6.9	1.5	0.18
9.0	9.3	7.8	1.7	0.18
9.0	10.4	8.4	1.8	0.17
11.6	1.9	1.8	0.7	0.37
11.6	4.1	3.8	1.2	0.29
11.6	6.6	5.7	1.7	0.26
11.6	9.0	7.6	2.2	0.24
11.6	11.4	9.3	2.6	0.23
11.6	13.8	10.7	3.0	0.22
11.6	17.5	12.9	4.2	0.24
11.6	19.4	14.2	5.0	0.26
11.6	20.3	15.0	5.8	0.29

As shown above, the transmission response of Plan 3 is essentially identical to Plan 2. Both the reef (Photo 25) and the existing breakwater (Photo 26) were completely stable.

Since raising the crown elevation did not improve performance, it was decided to test an additional plan with a significantly wider crown. Plan 4 (Figure 8) was constructed to a crown elevation of -20 ft lwd and used a crown width of 150 ft. Transmission test results were as follows:

T_p, sec	H_{mo}, ft Measured			C_t
	Incident	Behind Reef	Behind Breakwater	
7.0	2.2	1.9	0.5	0.23
7.0	4.0	3.3	0.7	0.18
7.0	7.0	5.2	1.0	0.14
7.0	9.8	7.2	1.3	0.13
7.0	12.0	8.4	1.6	0.13
9.0	3.0	2.7	0.7	0.23
9.0	4.9	4.3	1.0	0.20
9.0	6.9	5.8	1.3	0.19
9.0	8.3	6.8	1.5	0.18
9.0	9.5	7.7	1.7	0.18
9.0	10.6	8.5	1.8	0.17
11.6	1.9	1.9	0.7	0.37
11.6	4.1	3.7	1.2	0.29
11.6	6.5	5.7	1.7	0.26
11.6	9.0	7.6	2.1	0.23
11.6	11.3	9.3	2.5	0.22
11.6	13.9	11.1	2.9	0.21
11.6	17.6	13.1	3.9	0.22
11.6	19.7	14.1	4.7	0.24
11.6	21.4	15.0	5.5	0.26

The transmission response of Plan 4 is very similar to Plans 2, 2A, and 3 for the 7- and 9-sec periods. The only noticeable improvement occurs for the larger 11.6-sec waves. Photos 27-29 show that both the reef and the existing breakwater were stable.

Plan 4A (Figure 8) used the same volume of material as Plan 4; however, the crown elevation was raised to the water surface (+4-ft lwd) and the crown width was narrowed to 30 ft. Transmission results were as follows:

T_p, sec	H_{mo}, ft Measured			C_t
	Incident	Behind Reef	Behind Breakwater	
7.0	2.2	0.8	0.4	0.18
7.0	4.0	1.4	0.6	0.15
7.0	6.6	2.4	0.9	0.14
7.0	9.6	3.6	1.2	0.13
7.0	13.8	4.9	1.4	0.10
11.6	1.9	1.2	0.6	0.32
11.6	4.1	2.1	1.1	0.27
11.6	6.5	3.3	1.6	0.25
11.6	8.9	4.6	2.1	0.24
11.6	11.4	6.0	2.5	0.22
11.6	13.6	7.3	3.0	0.22
11.6	17.5	9.4	4.1	0.23
11.6	19.3	10.5	4.9	0.25
11.6	20.9	11.3	5.5	0.26

As shown above, Plan 4A showed slightly improved transmission performance, relative to Plan 4. The existing breakwater again was completely stable (Photos 30 and 31). The reef experienced some damage (Photo 32) with stone from the lakeward edge of the crown being displaced downslope.

Summary of results (Plans 2, 2A, 3, 4, and 4A)

Figures 9, 10, and 11 present transmitted wave height as a function of incident wave height for constant wave period. These data show that all plans produced similar transmission results, with Plans 4 and 4A providing slightly greater protection. Figure 12 presents wave heights behind the reef and these data show that all plans (except Plan 2A) reduce 19-ft incident waves to heights of about 13 ft or less.

All plans except 2A eliminated damage to the existing breakwater. The 5-ton stone was acceptable for all of the improvement structures; however, Plan 4A experienced some damage with stone from the lakeward edge of the crown being displaced downslope.

Plans 2, 3, 4, and 4A varied such parameters as the volume of stone, structure height, and crown width. However, all tests were conducted with a 150-ft spacing between the reef and the existing breakwater; therefore, it was decided to test one additional plan (Plan 4A1) with this spacing reduced to 75 ft. Plan 4A1, shown in Photo 33, was identical to Plan 4A except for the reduced spacing. Transmission test results were as follows:

T_p, sec	H_{mo}, ft Measured			C_t
	Incident	Behind Reef	Behind Breakwater	
7.0	2.3	1.1	0.3	0.13
7.0	4.1	1.6	0.5	0.12
7.0	6.8	3.1	0.8	0.12
7.0	9.5	4.1	1.1	0.12
7.0	11.4	5.0	1.4	0.12
11.6	1.9	1.1	0.5	0.26
11.6	4.8	2.4	1.0	0.21
11.6	6.6	3.5	1.5	0.23
11.6	9.0	4.6	2.0	0.22
11.6	11.4	6.0	2.4	0.21
11.6	13.9	7.1	2.9	0.21
11.6	17.6	9.0	4.1	0.23
11.6	20.0	10.0	5.1	0.26
11.6	21.8	11.0	5.9	0.27

Plan 4A1 generally produced similar but slightly reduced transmitted heights relative to Plan 4A (Figures 13 and 14). Also similar to Plan 4A, the reef experienced some damage with stone from the lakeward edge of the crown being displaced downslope. The existing breakwater was completely stable (Photos 34 and 35).

Attached Berms (Plans 5 and 6)

The first structure tested, Plan 5 (shown in Figure 15 and Photo 36), was constructed to an elevation of -10 ft lwd. It used a crown width of 100 ft and a stone weight of 5 tons. Transmission test results were as follows:

T_p, sec	Incident H_{mo}, ft	Transmitted H_{mo}, ft	C_t
7.0	2.3	0.5	0.22
7.0	4.0	0.6	0.15
7.0	6.7	1.0	0.15
7.0	9.2	1.2	0.13
7.0	11.8	1.5	0.13

(Continued)

(Concluded)			
T_p , sec	Incident H_{mo}, ft	Transmitted H_{mo}, ft	C_t
9.0	2.9	0.6	0.21
9.0	5.0	1.0	0.20
9.0	7.0	1.2	0.17
9.0	9.6	1.6	0.17
9.0	10.8	1.8	0.17
11.6	2.0	0.7	0.35
11.6	4.2	1.2	0.29
11.6	6.6	1.6	0.24
11.6	9.1	2.1	0.23
11.6	11.4	2.7	0.24
11.6	13.7	3.2	0.23
11.6	17.3	4.3	0.25
11.6	19.2	5.0	0.26
11.6	20.4	5.7	0.28

Stability of the existing breakwater was improved relative to base conditions (Photo 37); however, five armor stones were displaced from the harbor side.

A second and final berm was tested. Plan 6 (Figure 16 and Photos 38 and 39) was the same as Plan 5 except the crown elevation was raised to the water level (+4 ft lwd). Test results were as follows:

T_p , sec	Incident H_{mo}, ft	Transmitted H_{mo}, ft	C_t
7.0	2.3	0.4	0.17
7.0	4.1	0.5	0.12
7.0	6.6	0.8	0.12
7.0	10.0	1.1	0.11
7.0	11.7	1.4	0.12
9.0	2.8	0.5	0.18
9.0	4.7	0.8	0.17
9.0	6.8	1.1	0.16
9.0	8.1	1.2	0.15
9.0	9.3	1.4	0.15
9.0	10.4	1.6	0.15
			(Continued)

(Concluded)			
T_p, sec	Incident H_{mo}, ft	Transmitted H_{mo}, ft	C_t
11.6	1.9	0.3	0.16
11.6	4.2	0.9	0.21
11.6	6.7	1.4	0.21
11.6	9.2	1.9	0.21
11.6	11.5	2.4	0.21
11.6	13.8	2.8	0.20
11.6	17.6	4.1	0.23
11.6	19.5	4.9	0.25
11.6	21.1	5.7	0.27

Transmission data from the berm tests are presented in Figures 17, 18, and 19 as a function of incident wave height for constant wave period. These data show that Plan 6 yielded consistently lower transmitted wave heights for all conditions investigated.

Photos 40-42 show the structure after wave attack. Some reshaping of the berm occurred near the water surface as a significant number of the 5-ton stones were moved under wave attack. However, the existing structure was stable, with only one harbor-side stone being displaced. Based on observed movement, 5-ton stone appears to be only minimally adequate for the berm stone if the crown is brought to the water surface.

Overlays (Plans 7, 8, 8A, and 9)

Plan 7 (Figure 20 and Photo 43) consisted of overlaying the lakeside face of the existing breakwater with a protective covering of 18-ton angular stone, placed at a 1V on 3H slope. Test results were as follows:

T_p, sec	Incident H_{mo}, ft	Transmitted H_{mo}, ft	C_t
7.0	2.5	0.5	0.20
7.0	4.3	0.7	0.16
7.0	6.8	1.0	0.15
7.0	10.0	1.2	0.12
7.0	12.0	1.5	0.13
9.0	2.7	0.6	0.22
9.0	4.8	1.0	0.21
9.0	6.9	1.2	0.17
			(Continued)

(Concluded)			
T_p, sec	Incident H_{mo}, ft	Transmitted H_{mo}, ft	C_t
9.0	8.2	1.4	0.17
9.0	9.6	1.6	0.17
9.0	10.8	1.8	0.17
11.6	1.9	0.7	0.37
11.6	4.5	1.1	0.24
11.6	6.9	1.6	0.23
11.6	9.1	2.0	0.22
11.6	11.6	2.4	0.21
11.6	13.8	2.9	0.21
11.6	17.7	3.8	0.21
11.6	19.6	4.5	0.23
11.6	20.8	5.1	0.25

Transmission results are depicted graphically in Figure 21. These data show that the 1-ft transmission criterion is reached for 7-sec, 7-ft; 9-sec, 5-ft; and 11-sec, 4-ft incident waves. The 3-ft transmission criterion is exceeded by 15-ft, 11.6-sec waves.

Photos 44-46 show the structure after wave attack. A few of the 18-ton overlay stones rocked or shifted in the vicinity of the swl as they sought a more stable orientation; however, none were displaced. One harbor-side armor unit was displaced from the existing structure.

Plan 8 (Figure 22) consisted of adding one layer of 18-ton stone to the crest and two layers of 18-ton stone to the lakeward face of the existing structure. Transmission test results were as follows:

T_p, sec	Incident H_{mo}, ft	Transmitted H_{mo}, ft	C_t
7.0	2.4	0.4	0.18
7.0	4.4	0.7	0.15
7.0	6.7	1.0	0.14
7.0	9.9	1.2	0.12
7.0	11.4	1.5	0.13
9.0	2.8	0.7	0.24
9.0	4.8	1.1	0.22
9.0	6.9	1.3	0.19
			(Continued)

(Concluded)			
T_p, sec	Incident H_{mo}, ft	Transmitted H_{mo}, ft	C_t
9.0	8.1	1.5	0.18
9.0	9.2	1.7	0.18
9.0	10.6	1.9	0.18
11.6	2.0	0.7	0.34
11.6	4.1	1.2	0.28
11.6	6.8	1.7	0.25
11.6	9.4	2.2	0.23
11.6	11.8	2.5	0.22
11.6	14.0	2.9	0.21
11.6	17.7	3.9	0.22
11.6	19.6	4.6	0.24
11.6	21.1	5.3	0.25

Transmission results are depicted graphically in Figure 23. These data show that the 1-ft transmission criteria is reached for 7-sec, 7-ft; 9-sec, 4.5-ft; and 11-sec, 4-ft incident waves. The 3-ft transmission criterion is exceeded by 15-ft, 11.6-sec waves.

The stability response of Plan 8 was marginal. As shown in Photos 47-49, a significant number of armor stones were displaced down the lakeward face from the vicinity of the swl with six stones being removed from the structure. Three harbor-side armor units were displaced from the existing structure and three 18-ton armor stones were displaced from the crest to the harbor side.

Plan 8A (Figure 24 and Photo 50) consisted of adding one layer of 18-ton stone to the crest and two layers of 10-ton stone to the harbor-side face of the structure. Also, a 4-ft-thick layer of 1,000-lb stone was placed beneath the 10-ton, harbor-side stone. Transmission results were as follows:

T_p, sec	Incident H_{mo}, ft	Transmitted H_{mo}, ft	C_t
7.0	2.3	0.3	0.13
7.0	4.2	0.5	0.12
7.0	7.0	0.8	0.11
7.0	9.9	1.0	0.10
7.0	11.6	1.4	0.12
			(Continued)

(Concluded)			
T_p, sec	Incident H_{mo}, ft	Transmitted H_{mo}, ft	C_t
9.0	2.8	0.5	0.18
9.0	4.9	0.8	0.16
9.0	7.0	1.0	0.14
9.0	8.2	1.2	0.15
9.0	9.4	1.3	0.14
9.0	10.7	1.5	0.14
11.6	1.9	0.5	0.26
11.6	5.1	0.9	0.18
11.6	6.8	1.3	0.19
11.6	9.4	1.8	0.19
11.6	11.9	2.1	0.18
11.6	14.1	2.5	0.18
11.6	18.0	3.4	0.19
11.6	20.0	4.1	0.21
11.6	21.9	4.8	0.22

Transmission results, plotted in Figure 25, show that the 1-ft transmission criterion is reached for 7-sec, 10-ft; 9-sec, 7-ft; and 11.6-sec, 5-ft incident waves. The 3-ft transmission criterion is exceeded by 16-ft, 11.6-sec waves.

As shown in Photos 51-53, stability of the structure was marginal. Six blocks were displaced from the unprotected lakeside slope and a significant number of 10-ton overlay stones were displaced from the area between the crest and the water surface. Based on stable response of the 18-ton crest stone, this weight should probably be continued down the harbor-side slope to at least the water surface.

Plan 9 (Figure 26 and Photos 54 and 55) was the same as Plan 8 except the slope of the 18-ton overlay stone was flattened to 1:2.25 in an effort to improve lakeside stability. Transmission test results were as follows:

T_p, sec	Incident H_{mo}, ft	Transmitted H_{mo}, ft	C_t
7.0	2.4	0.4	0.18
7.0	4.3	0.6	0.14
7.0	6.7	0.8	0.12
7.0	9.3	1.1	0.12
7.0	11.7	1.4	0.12
			(Continued)

(Concluded)			
T_p, sec	Incident H_{mo}, ft	Transmitted H_{mo}, ft	C_t
9.0	2.8	0.5	0.18
9.0	4.9	0.9	0.18
9.0	6.8	1.2	0.18
9.0	8.2	1.4	0.17
9.0	9.2	1.6	0.17
9.0	10.6	1.7	0.16
11.6	1.9	0.6	0.32
11.6	4.1	1.1	0.27
11.6	6.7	1.6	0.24
11.6	9.3	2.0	0.22
11.6	11.7	2.4	0.21
11.6	13.9	2.8	0.20
11.6	17.6	3.7	0.21
11.6	19.9	4.4	0.22
11.6	21.5	5.0	0.23

Transmission results are plotted in Figure 27. These data show that the 1-ft transmission criterion is reached for 7-sec, 8-ft; 9-sec, 5-ft; and 11-sec, 4-ft incident waves. The 3-ft transmission criteria is exceeded by 15-ft, 11.6-sec waves. As would be expected, results are very similar to those observed for Plans 7 and 8.

The stability response of Plan 9 was acceptable and intermediate to results achieved for Plans 7 and 8. As shown in Photos 56-58, several armor stones were displaced down the lakeward face from the vicinity of the swl; however, the integrity of the overlay was not jeopardized. Two harbor-side armor stones were displaced from the existing structure and two 18-ton armor stones were displaced from the crest to the harbor side.

Summary of Results (Plans 2-9)

The first 12 improvement plans significantly improved stability of the existing breakwater and reduced transmitted wave heights to some extent. In order to help quantify performance, average transmission coefficients were calculated, with the following results:

Plan	Average C_t for Indicated Wave Period			Average C_t
	7.0 sec	9.0 sec	11.6 sec	All Periods
1A3 (Existing)	0.18	0.21	0.30	0.23
2	0.16	0.20	0.27	0.21
2A	0.17	0.21	0.27	0.22
3	0.16	0.19	0.27	0.21
4	0.16	0.19	0.26	0.20
4A	0.14	--[1]	0.25	0.20
4A1	0.12	--[1]	0.23	0.18
5	0.16	0.18	0.26	0.20
6	0.13	0.16	0.22	0.17
7	0.15	0.19	0.24	0.19
8	0.14	0.20	0.25	0.20
8A	0.12	0.15	0.20	0.16
9	0.13	0.17	0.24	0.18

[1] Not tested.

The above data, graphically presented in Figures 28-31, show that Plans 6 and 8A yielded the lowest transmitted wave heights of all plans investigated. Unfortunately, these wave heights were still larger than desired. Therefore, it was decided to test an additional plan (Plan 10) that would be the same as Plan 6, except the interface between the existing breakwater and the 5-ton berm stone was sealed with a sheet of plastic to simulate an impermeable barrier in the prototype. Plan 6 was selected over Plan 8A because of its better stability.

Stability of the existing structure, quantified as percent damage (number of armor units displaced divided by total numer of armor units in that section) to the lakeside and harbor-side armor, is summarized as follows:

Plan	Percent Damage to Existing Structure	
	Lakeside armor	Harbor-Side Armor
1A3 (Existing)	2.5	5
2	0	0
2A	0	2
3	0	0
4	0	0
4A	0	0

(Continued)

(Concluded)		
	Percent Damage to Existing Structure	
Plan	Lakeside armor	Harbor-Side Armor
4A1	0	0
5	0	2
6	0	1
7	0	1
8	0	2
8A	2	0
9	0	1.5

Lakeside and harbor-side damages also are presented in Figures 32 and 33. These data show that all improvement plans, except Plan 8A, which provided no protection on the lakeside, eliminated lakeside damage. Also, all plans reduced harbor-side damage to an acceptable level, i.e., 2 percent or less.

Impermeable Barrier (Plans 10 and 10A)

Plan 10 (Photo 59), tested at both the 0- and +4-ft swl's, produced the following results:

T_p, sec	Incident H_{mo}, ft	Transmitted H_{mo}, ft	C_t
swl = 0.0-ft lwd			
7.0	2.2	0.2	0.09
7.0	4.2	0.4	0.10
7.0	7.1	0.6	0.08
7.0	9.8	0.9	0.09
7.0	11.7	1.2	0.10
9.0	2.8	0.3	0.11
9.0	4.8	0.5	0.10
9.0	6.9	0.8	0.12
9.0	8.1	0.9	0.11
9.0	9.4	1.1	0.12
9.0	10.7	1.2	0.11
			(Continued)

(Concluded)			
T_p, sec	Incident H_{mo}, ft	Transmitted H_{mo}, ft	C_t
swl = 0.0-ft lwd			
11.6	1.9	0.4	0.21
11.6	4.2	0.7	0.17
11.6	6.7	1.1	0.16
11.6	9.2	1.6	0.17
11.6	11.3	1.9	0.17
11.6	13.9	2.4	0.17
11.6	17.4	3.4	0.20
11.6	19.4	4.1	0.21
11.6	20.5	4.8	0.23
swl = +4.0-ft lwd			
7.0	2.3	0.3	0.13
7.0	4.2	0.4	0.10
7.0	6.9	0.7	0.10
7.0	9.9	1.1	0.11
7.0	11.7	1.4	0.12
9.0	2.9	0.4	0.14
9.0	5.0	0.7	0.14
9.0	6.6	0.9	0.14
9.0	7.7	1.1	0.14
9.0	9.2	1.3	0.14
9.0	10.7	1.4	0.13
11.6	1.9	0.4	0.21
11.6	4.2	0.9	0.21
11.6	6.7	1.3	0.19
11.6	9.0	1.8	0.20
11.6	11.5	2.3	0.20
11.6	13.9	2.9	0.21
11.6	18.0	4.2	0.23
11.6	19.9	4.9	0.25
11.6	21.7	5.7	0.26

Transmission results are plotted for constant wave period and swl in Figures 34 and 35. These data show that the 0-ft swl produced consistently lower transmitted heights with the 1-ft transmission criteron being reached for 7-sec, 10-ft and 9-sec, 9-ft incident waves. The +4-ft swl required 7-sec, 9-ft; 9-sec, 7-ft incident waves to produce 1-ft transmitted waves. The 3-ft transmission criterion is exceeded by 16-ft (0-ft swl) and 14-ft (+4-ft swl), 11.6-sec waves.

The stability response of Plan 10 was acceptable. As shown in Photos 60-62, some of the 5-ton berm stone moved under wave attack with resultant reshaping of the berm at its lakeward edge. The existing structure was reasonably stable, with two harbor-side blocks being displaced downslope.

Plan 10A (Figure 36 and Photo 63) was similar to Plan 10, except the berm was reduced in width from 100 to 50 ft and the impervious plastic sheet was replaced with 1,000-lb filter stone. Testing at the +4-ft swl produced the following results:

T_p, sec	Incident H_{mo}, ft	Transmitted H_{mo}, ft	C_t
7.0	2.3	0.3	0.13
7.0	4.2	0.4	0.10
7.0	6.9	0.7	0.10
7.0	9.9	1.1	0.11
7.0	11.7	1.4	0.12
9.0	2.9	0.4	0.14
9.0	5.0	0.7	0.14
9.0	6.6	0.9	0.14
9.0	7.7	1.1	0.14
9.0	9.2	1.3	0.14
9.0	10.7	1.4	0.13
11.6	1.9	0.4	0.21
11.6	4.2	0.9	0.21
11.6	6.7	1.3	0.19
11.6	9.0	1.8	0.20
11.6	11.5	2.3	0.20
11.6	13.9	2.9	0.21
11.6	18.0	4.2	0.23
11.6	19.9	4.9	0.25
11.6	21.7	5.7	0.26

Transmission results are plotted in Figure 37. These data show that the 1-ft transmission criteria was reached for 7-sec, 9-ft and 9-sec, 8-ft incident waves. The 3-ft transmission criterion is exceeded by 14-ft, 11.6-sec waves.

The stability response of Plan 10A was unacceptable. As shown in Photos 64-67, the single layer of 5-ton berm stone experienced excessive movement under wave attack with resultant exposure and erosion of the 1,000-lb filter stone. The existing structure's stability was little improved relative to the no improvement plan, with six harbor-side blocks being displaced downslope.

Restacking Existing Armor (Plan 11)

Plan 11 (Photos 68-70) consisted of restacking the harbor-side armor blocks in an area bounded by the center line and the -7-ft lwd depth. Armor units were placed as close together as practical with their long axis generally perpendicular to the long axis of the breakwater. Transmission test results were as follows:

T_p, sec	Incident H_{mo}, ft	Transmitted H_{mo}, ft	C_t
7.0	2.3	0.4	0.17
7.0	4.2	0.5	0.12
7.0	6.7	0.8	0.12
7.0	9.7	1.2	0.12
7.0	11.7	1.7	0.15
9.0	2.8	0.6	0.21
9.0	4.7	0.9	0.19
9.0	6.7	1.2	0.18
9.0	8.1	1.5	0.19
9.0	9.4	1.7	0.18
9.0	10.5	1.8	0.17
11.6	1.9	0.6	0.32
11.6	5.0	1.2	0.24
11.6	6.6	1.6	0.24
11.6	9.0	2.2	0.24
11.6	11.2	2.8	0.25
11.6	14.0	3.6	0.26
11.6	17.7	5.3	0.30
11.6	19.7	6.2	0.31
11.6	21.6	7.0	0.32

Transmission results are depicted graphically in Figure 38. These data show that the 1-ft transmission criterion is reached for 7-sec, 8-ft; 9-sec, 5-ft; and 11-sec, 4-ft incident waves. The 3-ft transmission criteria is exceeded by 12-ft, 11.6-sec waves.

The stability response of Plan 11 was marginal. As shown in Photos 71-73, a significant number of armor stones were displaced down the lakeward face from the vicinity of the swl, with three stones being removed from the structure. Five harbor-side armor units were displaced.

Summary of Results (All Improvement Plans)

Average transmission coefficients for Plans 10, 10A, and 11 were as follows:

Plan	Average C_t for Indicated Wave Period			Average C_t
	7.0 sec	9.0 sec	11.6 sec	All Periods
10	0.09[1]	0.11[1]	0.19[1]	0.13[1]
10	0.11	0.14	0.22	0.16
10A	0.12	0.16	0.24	0.17
11	0.14	0.19	0.28	0.20

[1] swl = 0.0 ft lwd.

Figures 39-42 summarize transmission test results for the 15 improvement plans tested. These data show Plans 8A and 10 produced the most improvement in wave transmission (average C_t = 0.16), followed closely by Plans 6 and 4A1 with average C_t's of 0.17 and 0.18, respectively. Also, these data show that in general the submerged reefs (Plans 2-4A1) and restacking of the existing armor (Plan 11) were least effective in reducing wave energy, whereas the toe berms (Plans 5, 6, and 10) and the large-stone overlays (Plans 7, 8, 8A, and 9) were most effective.

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
1 (Existing)	2.5	5
2	0	0
2A	0	2
3	0	0
4	0	0

(Continued)

(Concluded)		
	Percent Damage to Existing Structure	
Plan	Lakeside Armor	Harbor-Side Armor
4A	0	0
4A	0	0
5	0	2
6	0	1
7	0	1
8	0	2
8A	2	0
9	0	1.5
10	0	1.5
10A	0	3.5
11	5	2

Lakeside and harbor-side damages also are presented in Figures 43 and 44. These data show that all improvement plans, except Plans 8A and 11, which provided no protection on the lakeside, eliminated lakeside damage. All plans, except 10A, reduced harbor-side damage to an acceptable level, i.e., 2 percent or less.

4 Conclusions

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

a. The model was able to accurately replicate prototype wave energy transmission, as evidenced in test results of Plan 1A3.

b. Test results for the detached reefs (Plans 2, 2A, 3, 4, 4A, and 4A1) show that all plans except 2A reduce 19-ft incident waves to heights of 13 ft or less and eliminate damage to the existing breakwater. Also, major changes can be made in the geometry and size of the reef with little resultant change in the observed transmission, as shown in Figures 8-11.

c. Plans 5 and 6 showed that a 100-ft-wide attached berm constructed of 5-ton stone would also be successful in protecting the existing structure and reducing wave heights in the harbor.

d. Test results for Plans 7, 8, and 9 show that 18-ton stone would need to be placed on no steeper than a 1V on 2.25H slope to be stable on the lakeside of the breakwater.

e. Plan 8A, the only harbor-side repair option tested, was one of the most successful plans in terms of reducing wave energy; however, stability was marginal.

f. Plan 10 yielded the largest reduction in wave energy transmission in concert with acceptable stability.

g. Plan 10A, a 50-ft-wide attached berm, was the only improvement plan that did not show acceptable stability.

h. Plan 11, restacking of the existing armor, was not effective in significantly reducing wave transmission.

Generally, the 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 in reducing or eliminating damage to the existing breakwater.

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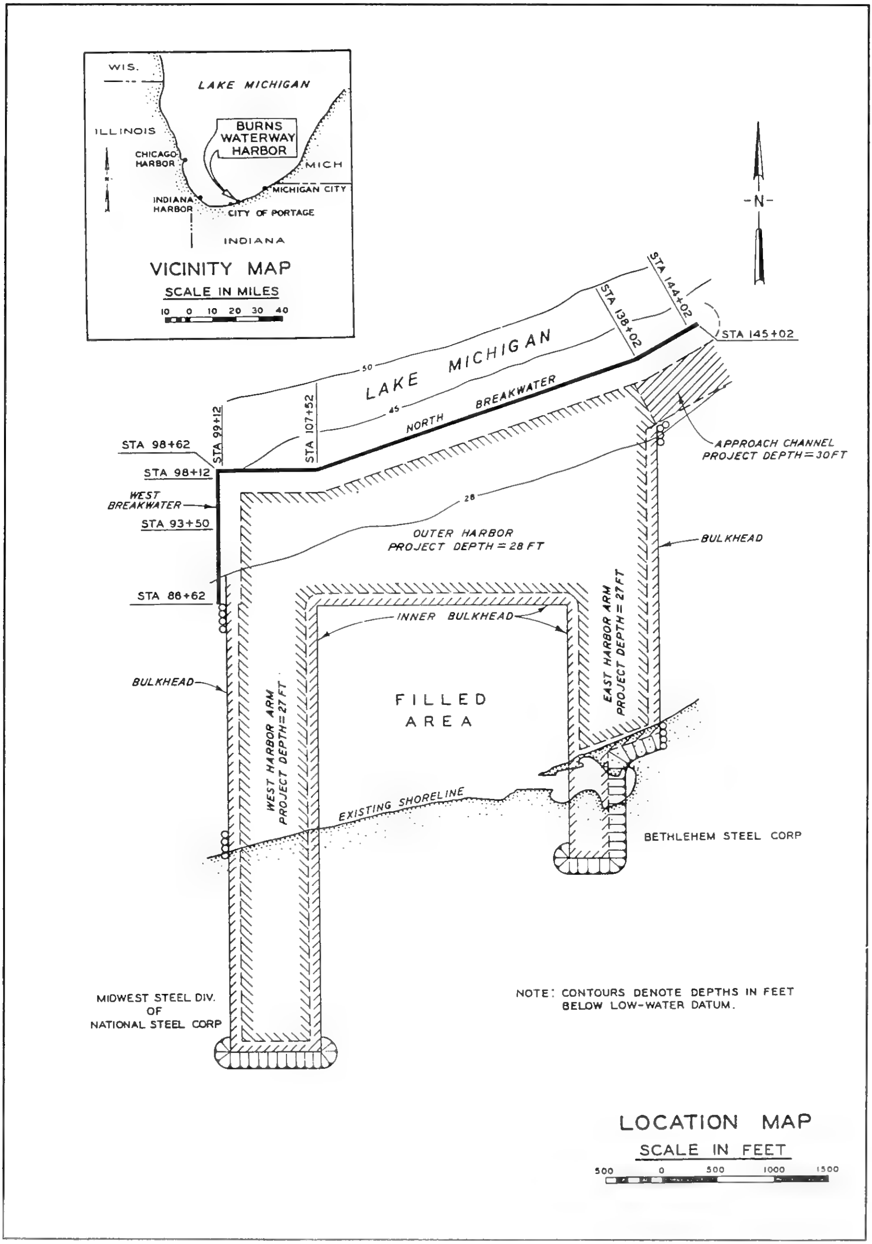

Figure 1. Location and vicinity map

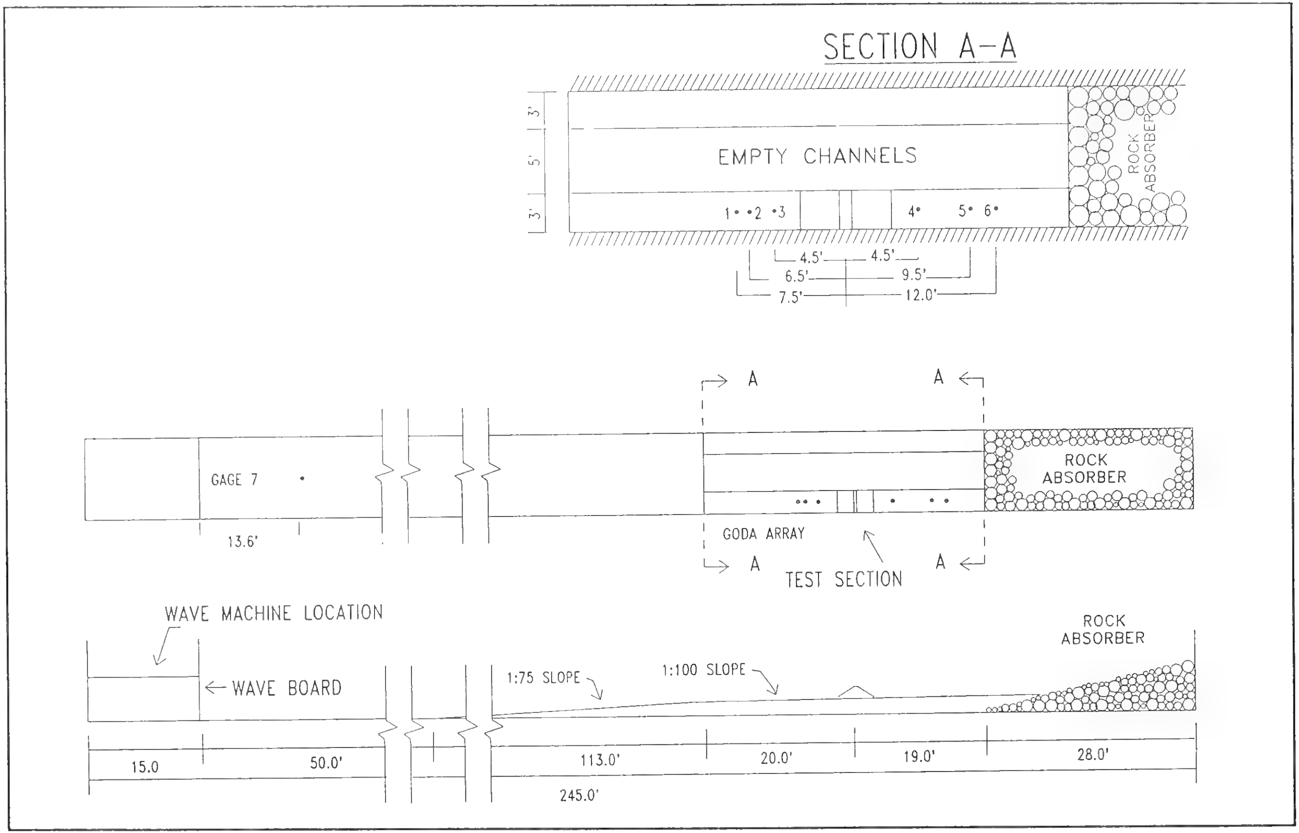

Figure 2. Wave tank cross section

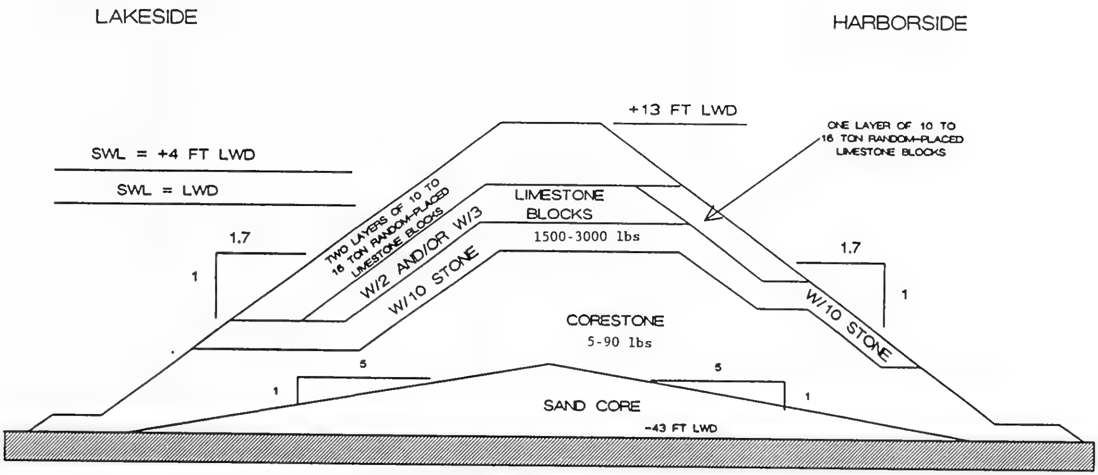

Figure 3. Plans 1, 1A, 1A1, 1A2 and 1A3

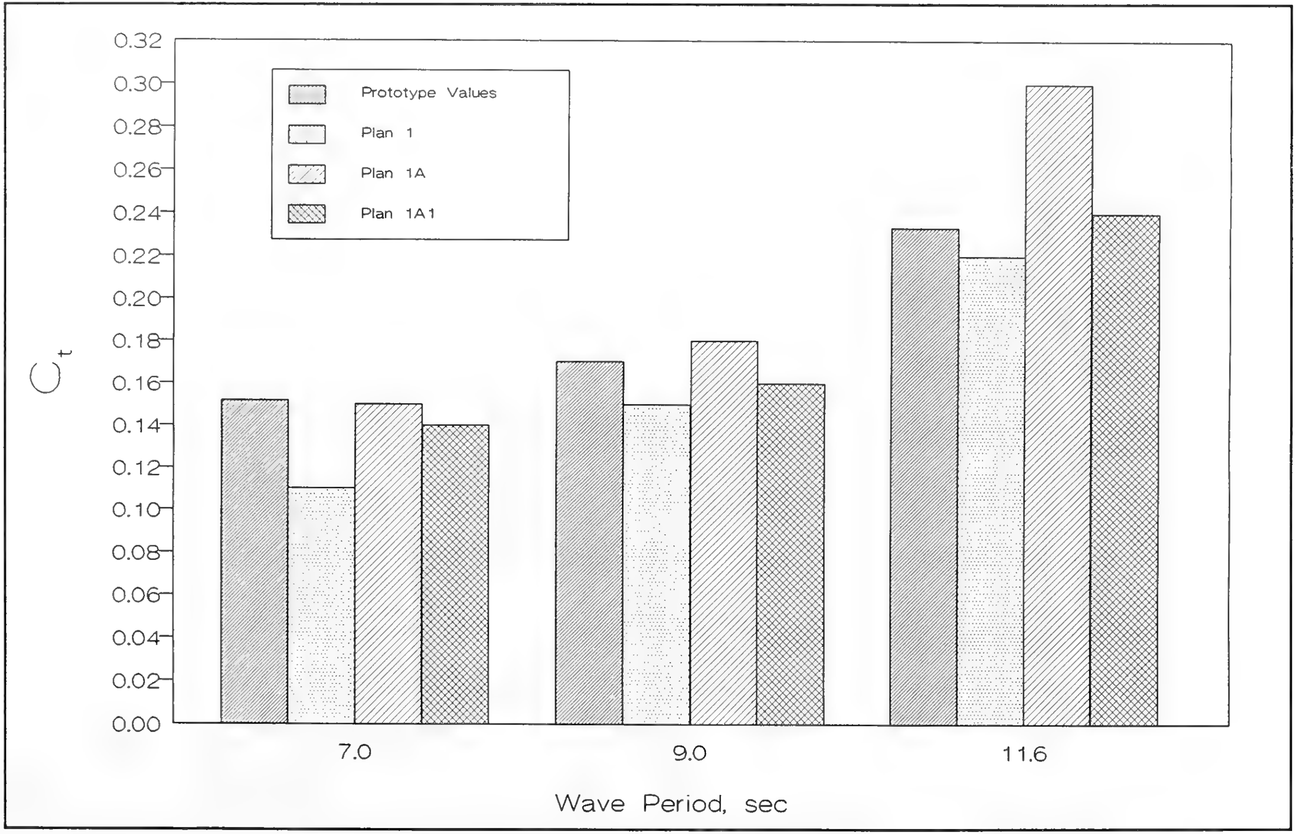

Figure 4. Transmission response of Plans 1, 1A, and 1A1

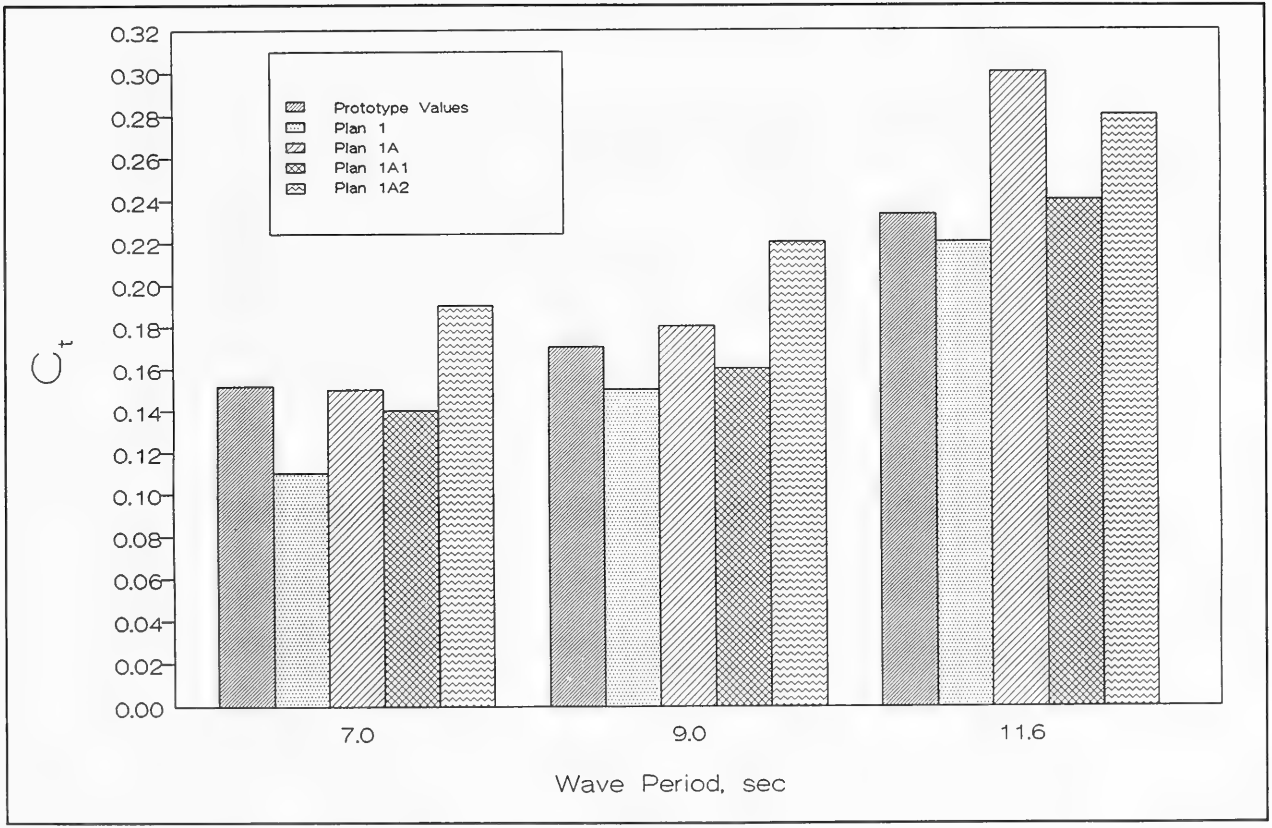

Figure 5. Transmission response of Plans 1, 1A, 1A1, and 1A2

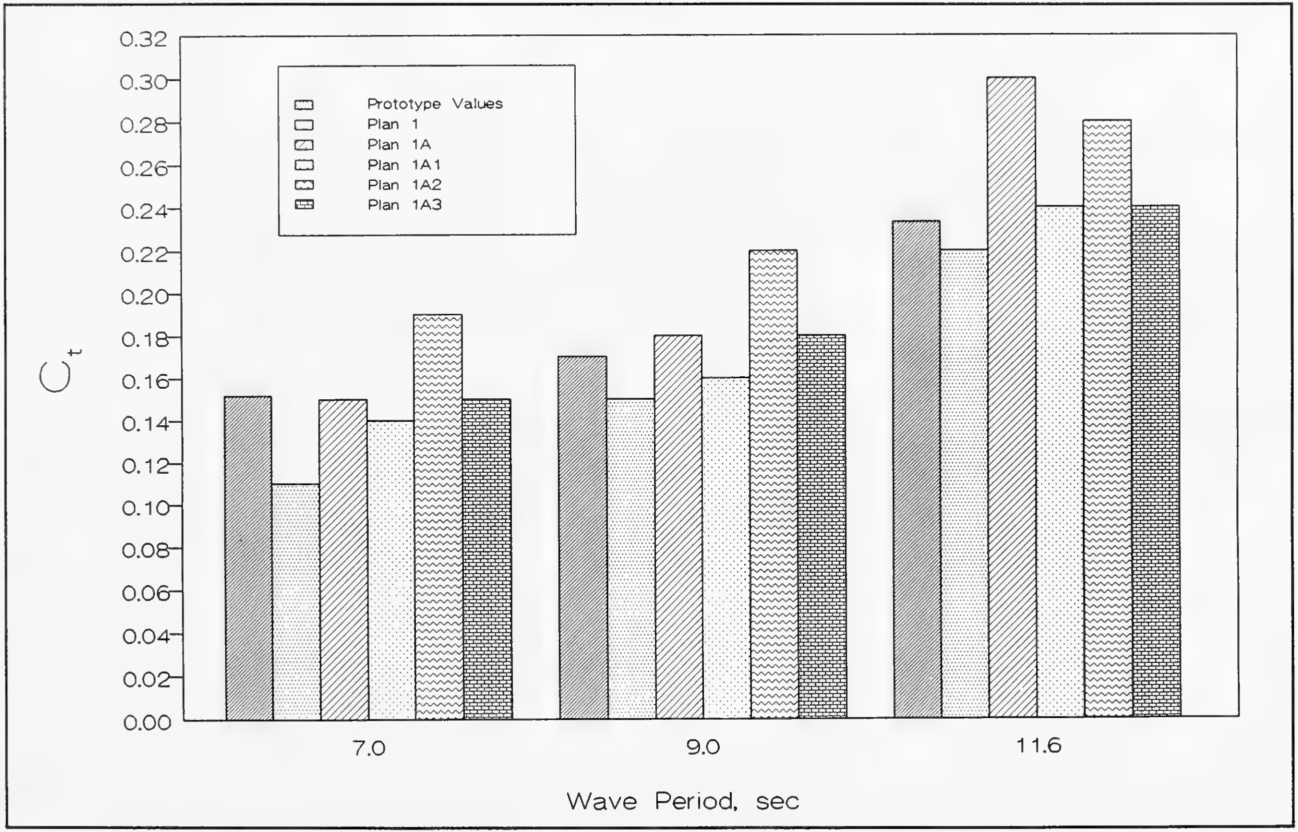

Figure 6. Transmission response of Plans 1, 1A, 1A1, 1A2, and 1A3

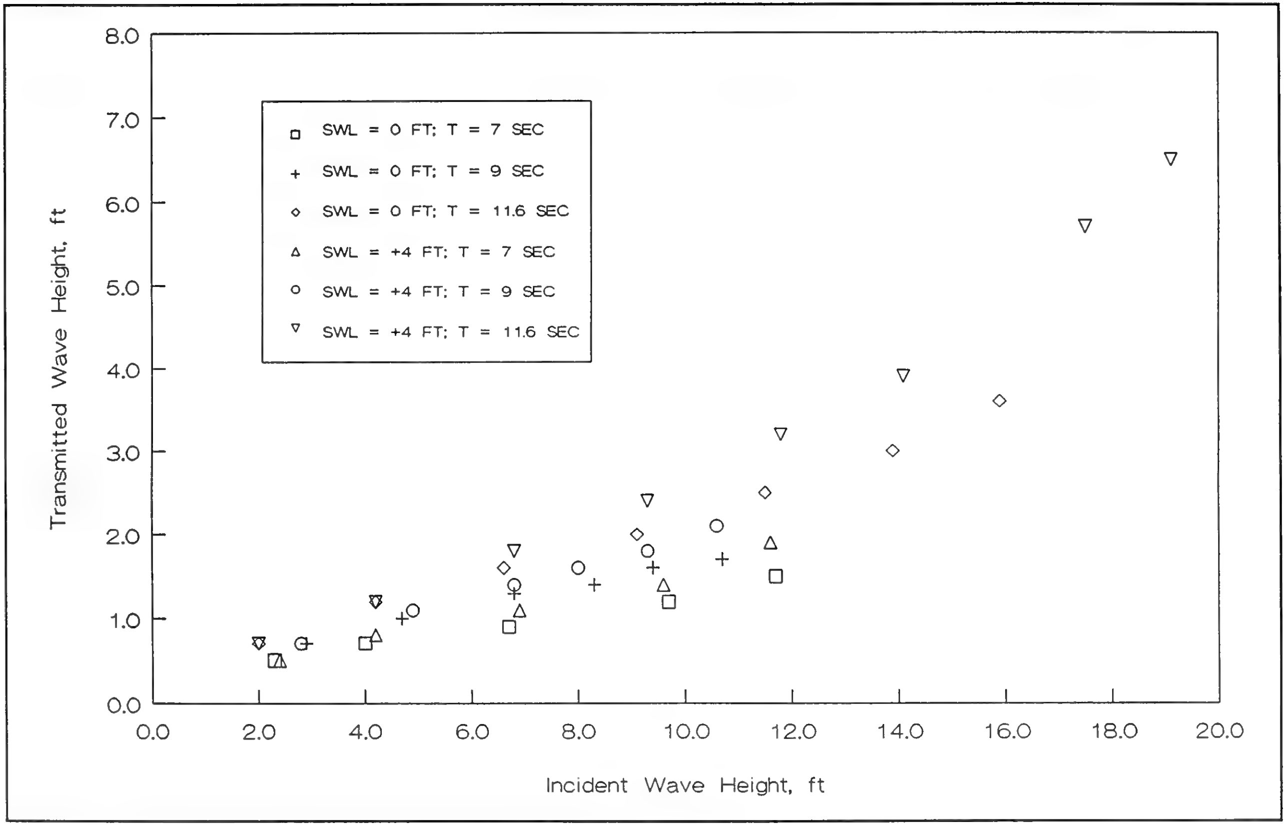

Figure 7. Transmission versus incident wave heights for existing conditions

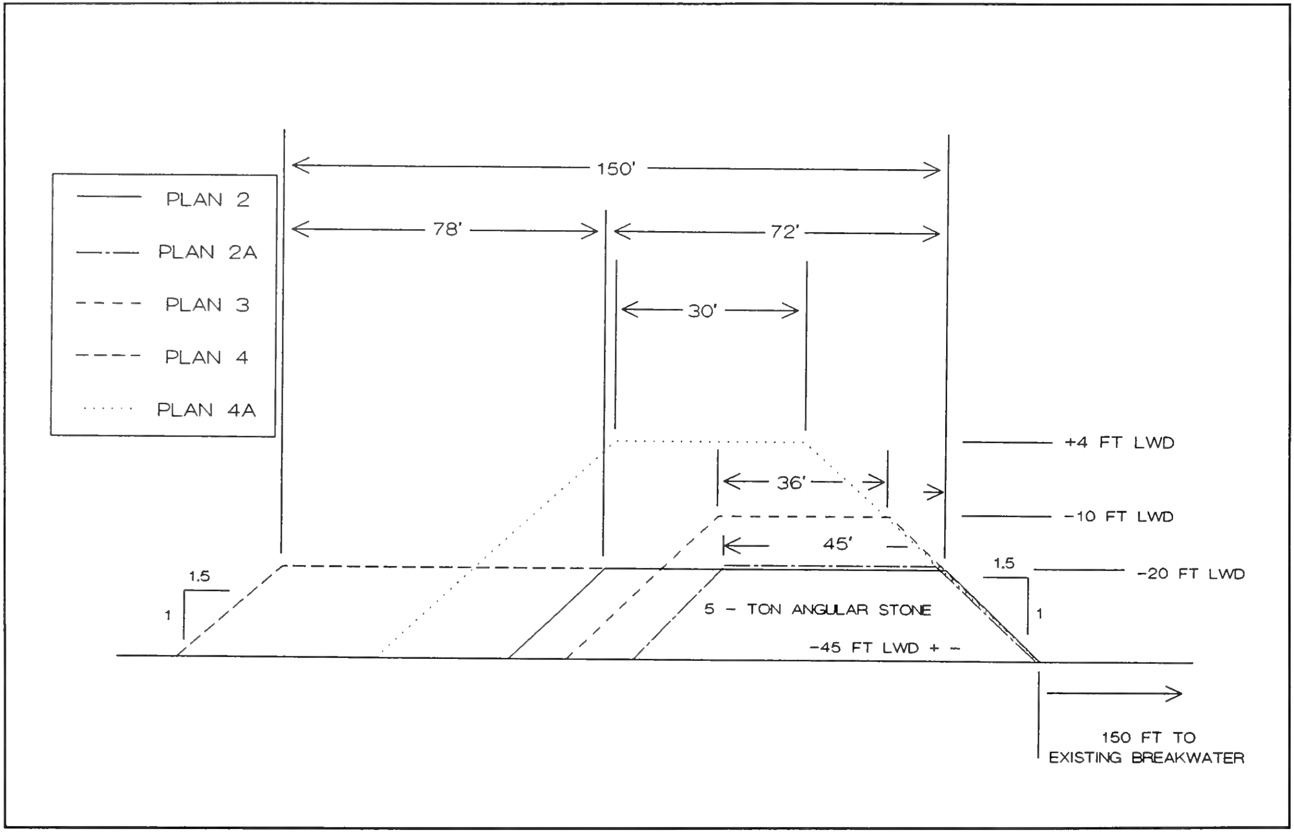

Figure 8. Plans 2, 2A, 3, 4, and 4A

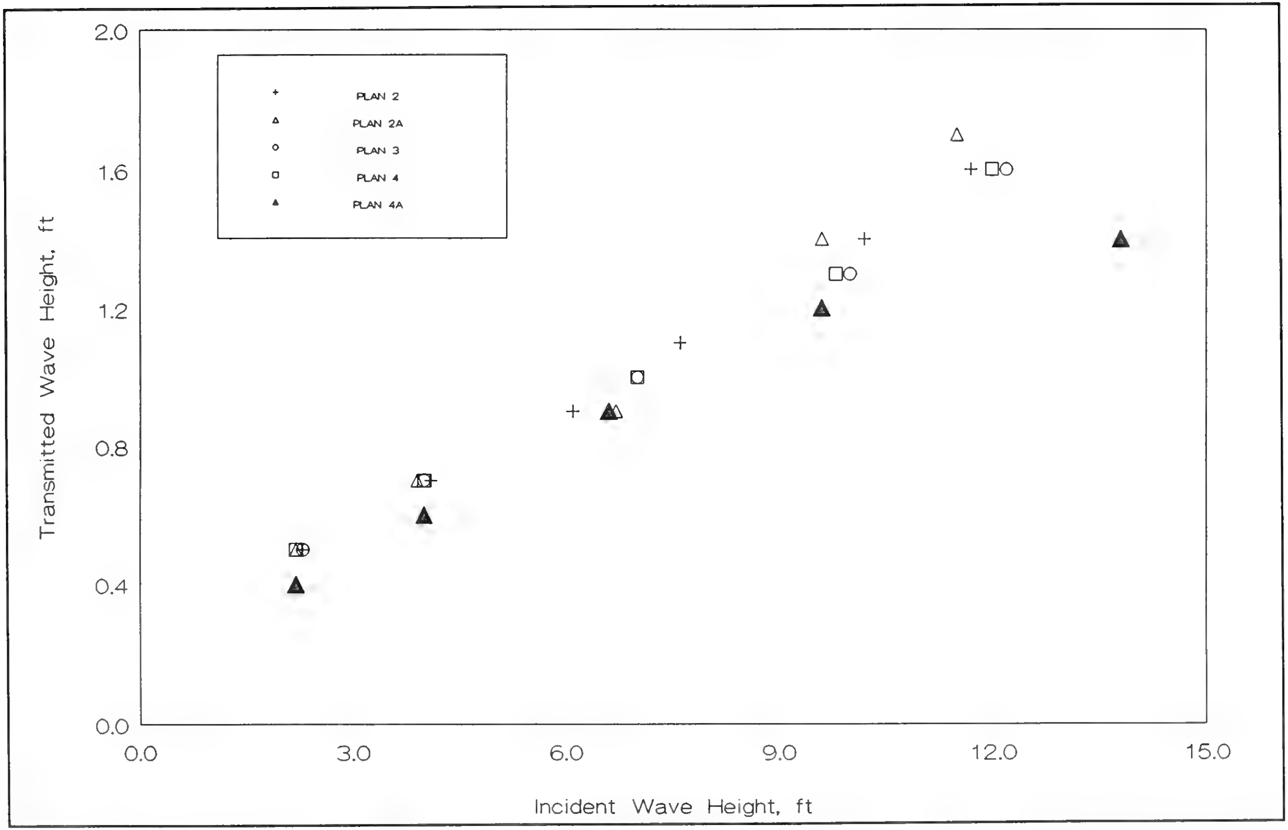

Figure 9. Transmission response of Plans 2, 2A, 3, 4, and 4A; 7-sec wave period

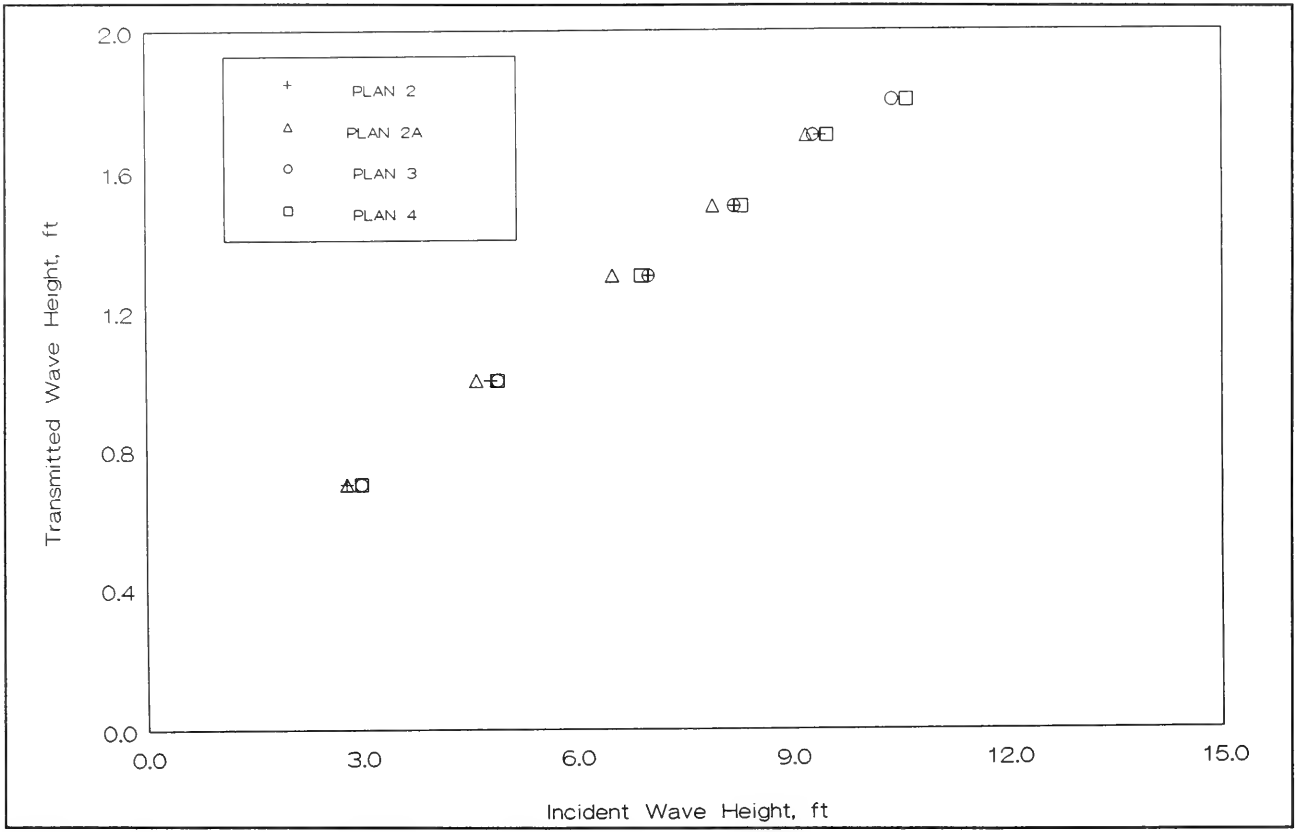

Figure 10. Transmission response of Plans 2, 2A, 3, and 4; 9-sec wave period

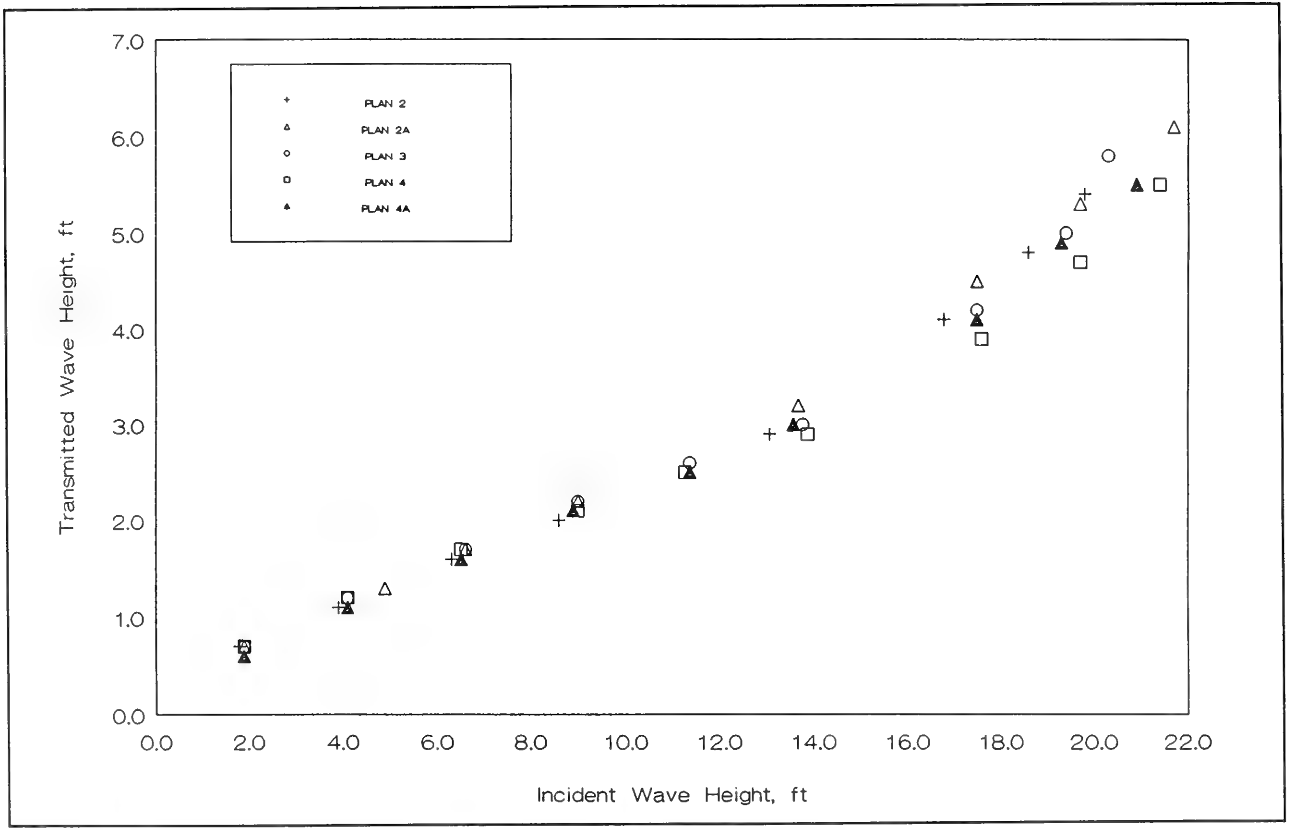

Figure 11. Transmission response of Plans 2, 2A, 3, 4, and 4A; 11.6-sec wave period

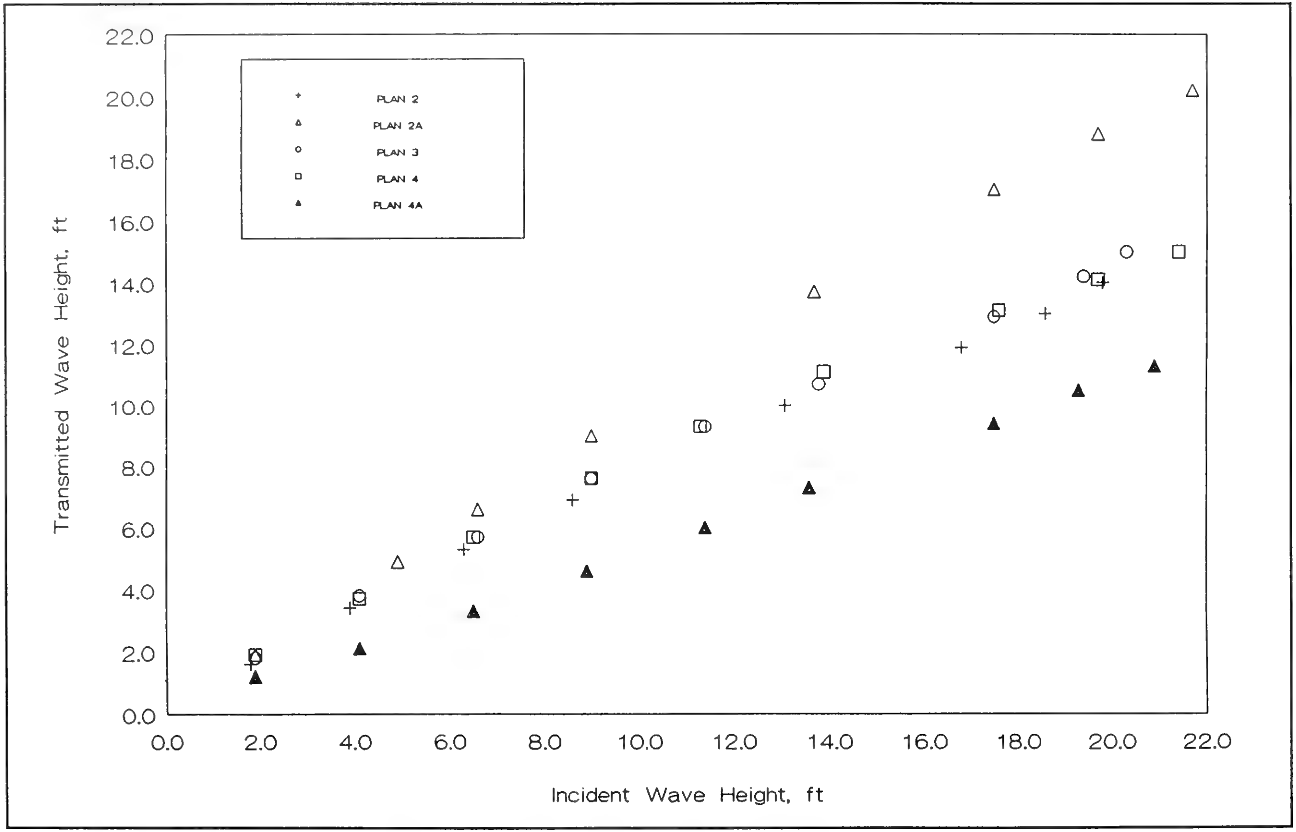

Figure 12. Transmissed wave heights recorded shoreward of reef; 11.6-sec wave period

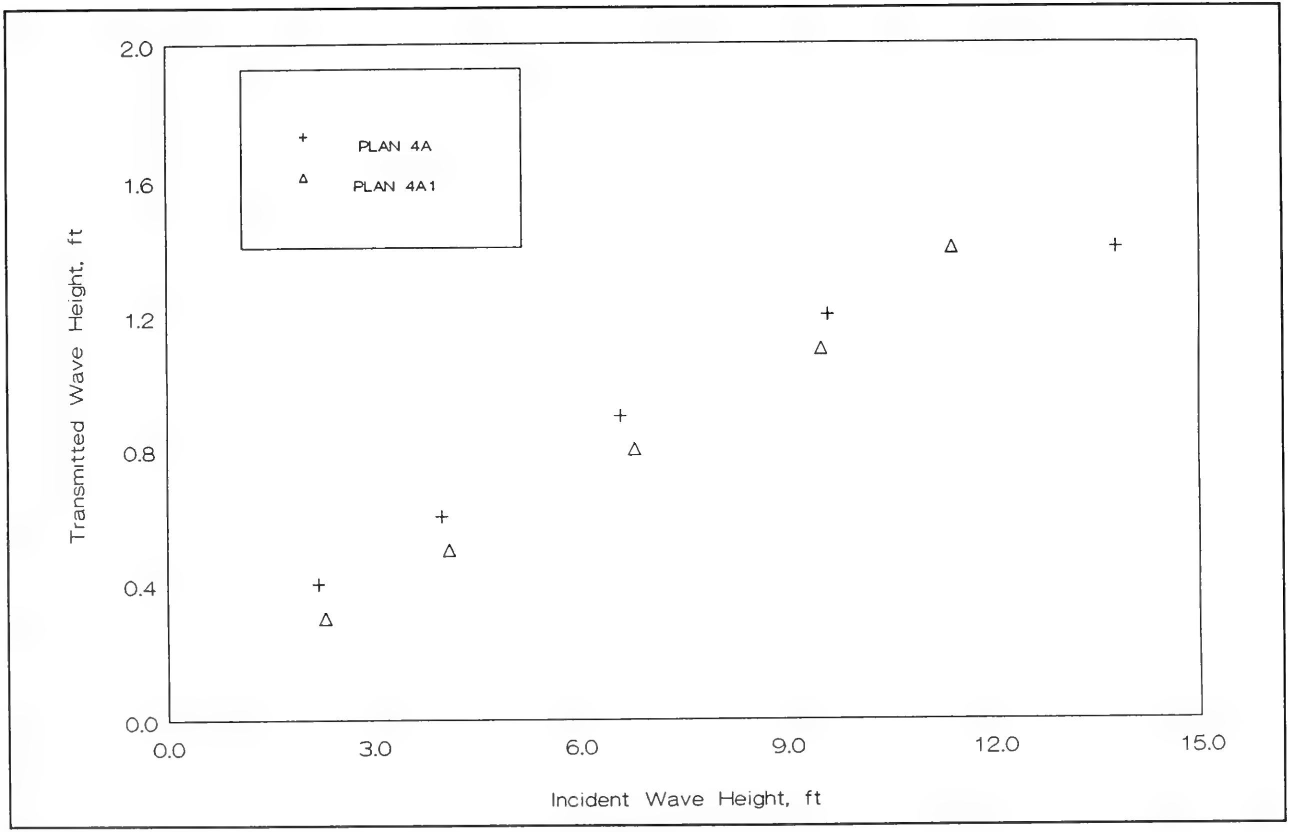

Figure 13. Comparison of transmission test results; 7-sec wave period

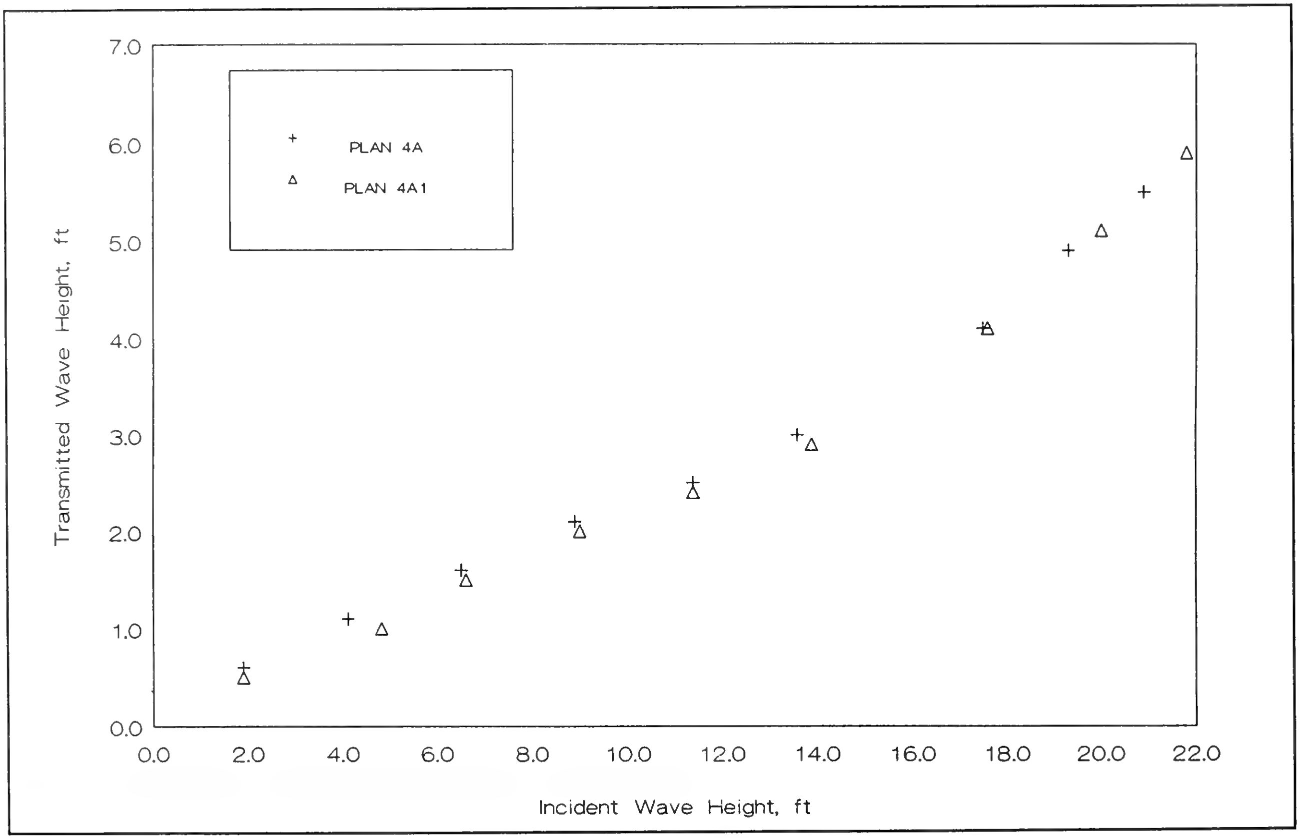

Figure 14. Comparison of transmission test results; 11.6-sec wave period

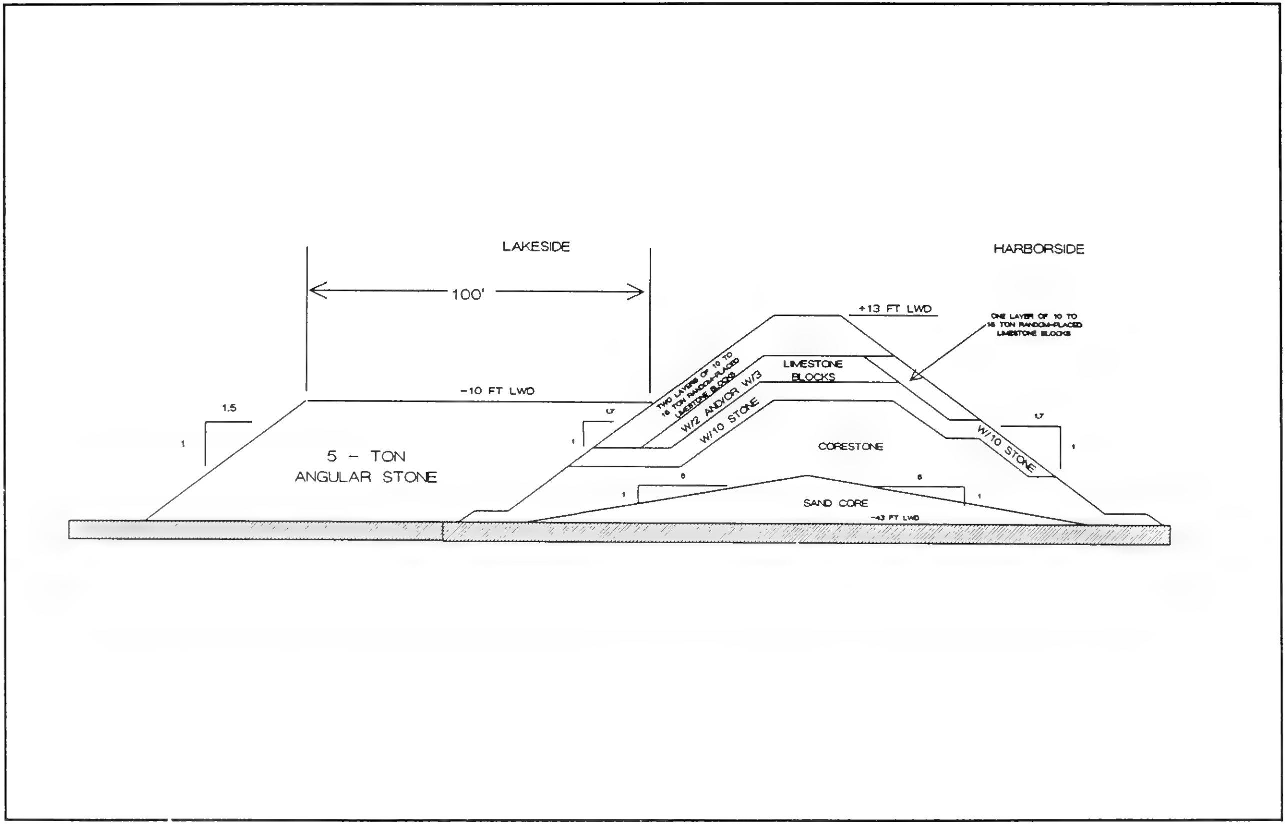

Figure 15. Plan 5

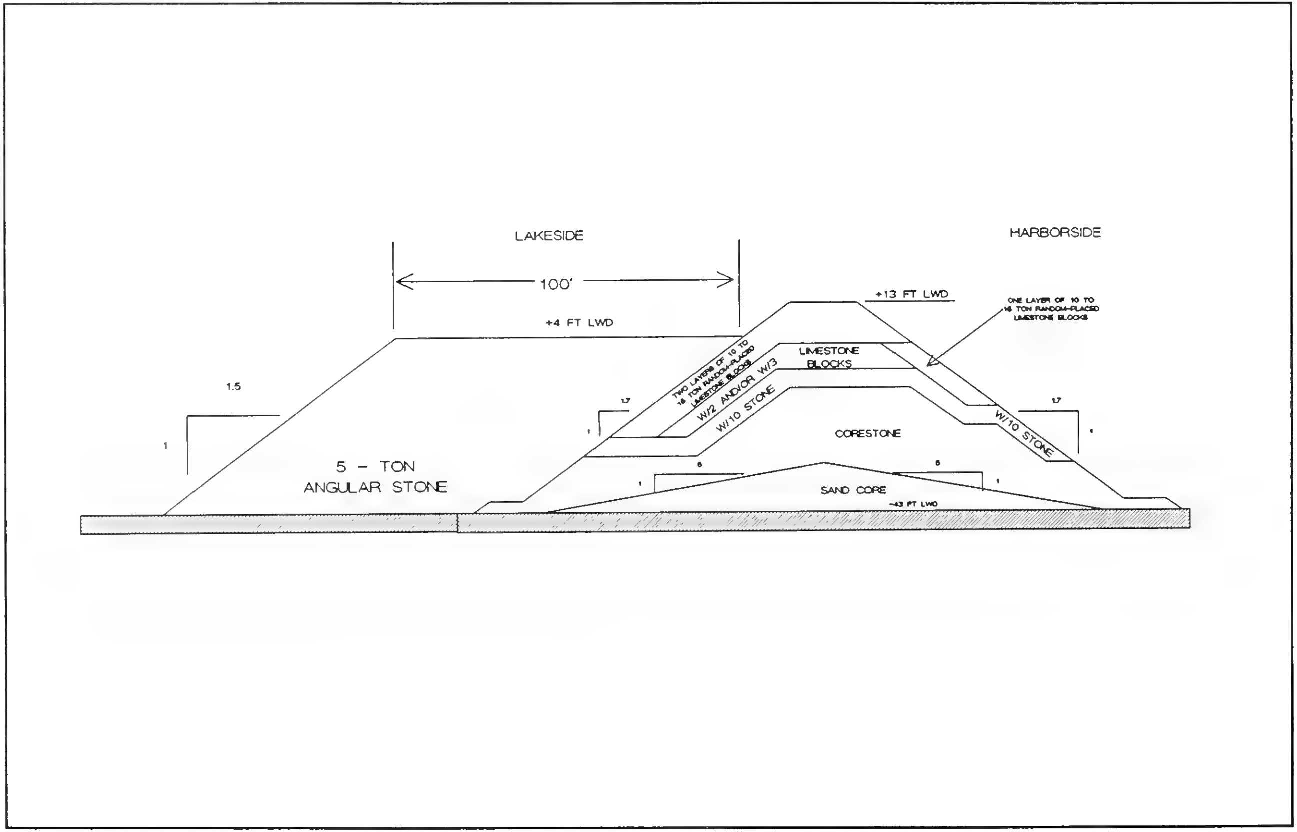

Figure 16. Plan 6

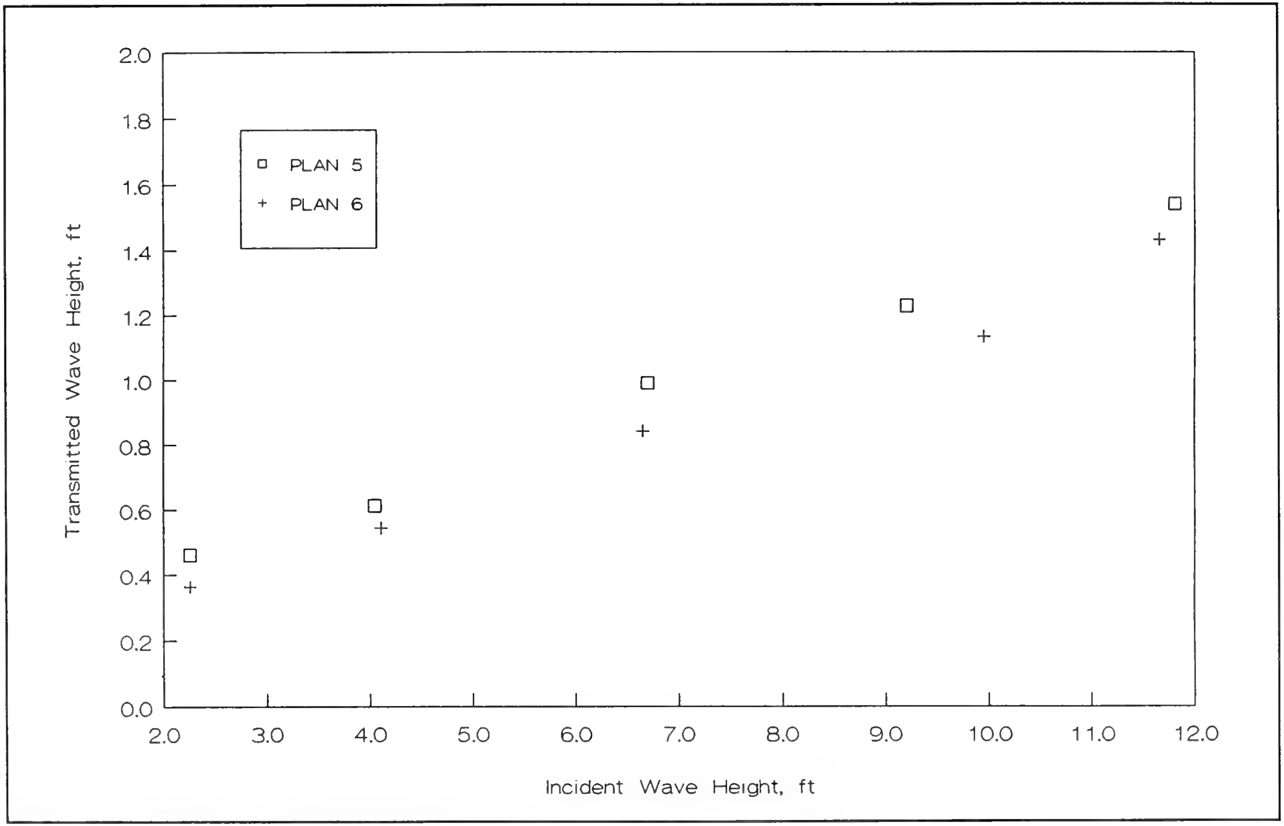

Figure 17. Comparison of transmission test results; Plans 5 and 6; 7-sec wave period

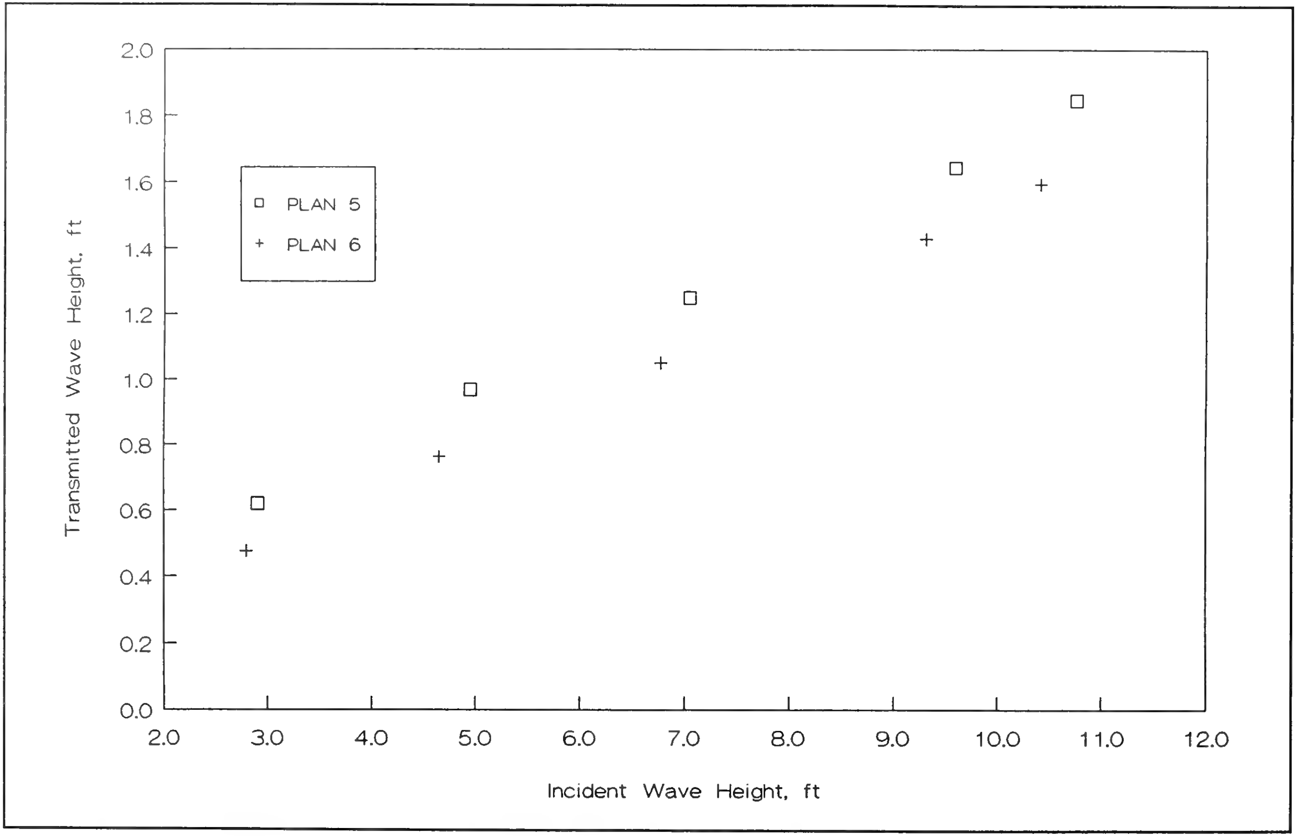

Figure 18. Comparison of transmission test results; Plans 5 and 6; 9-sec wave period

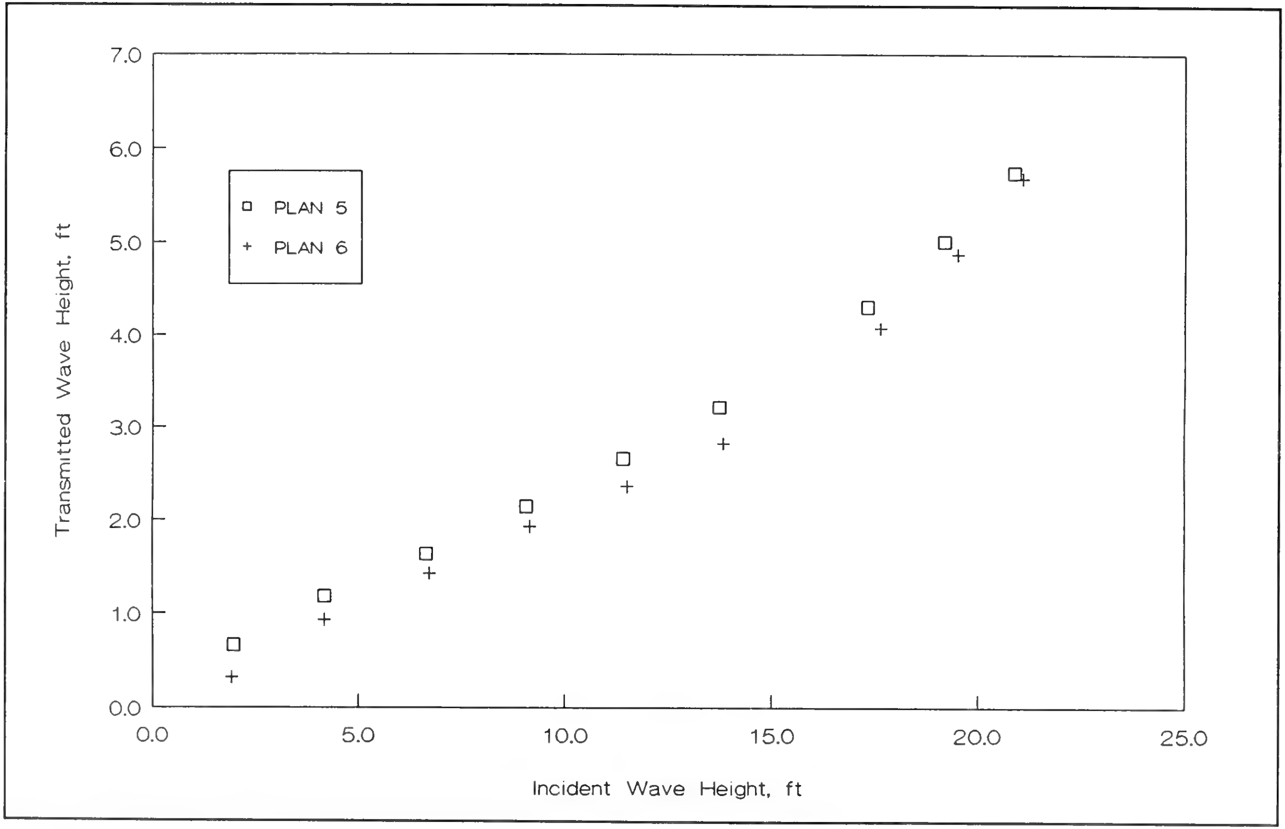

Figure 19. Comparison of transmission test results; Plans 5 and 6; 11.6-sec wave period

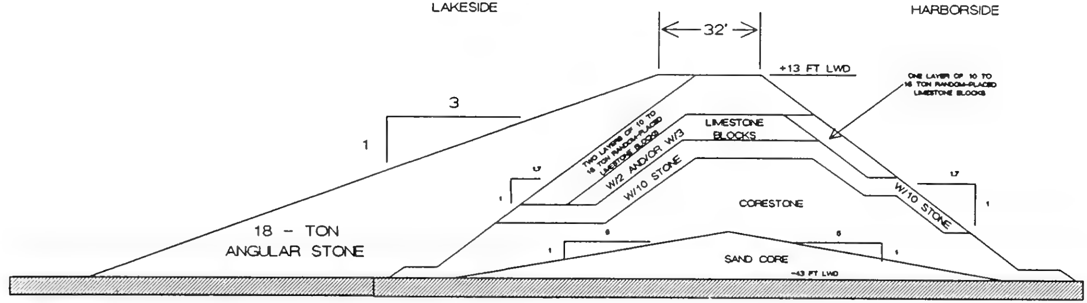

Figure 20. Plan 7

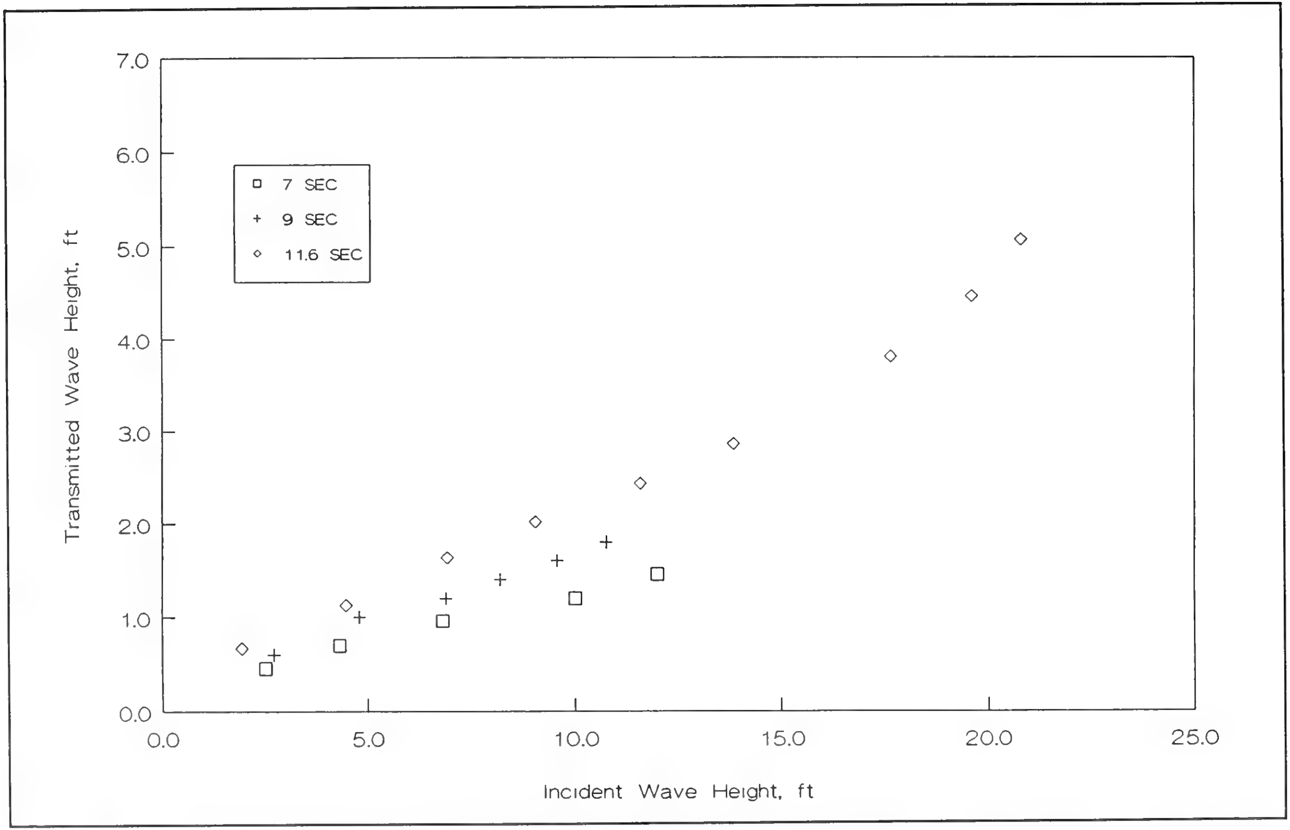

Figure 21. Transmission test results for Plan 7

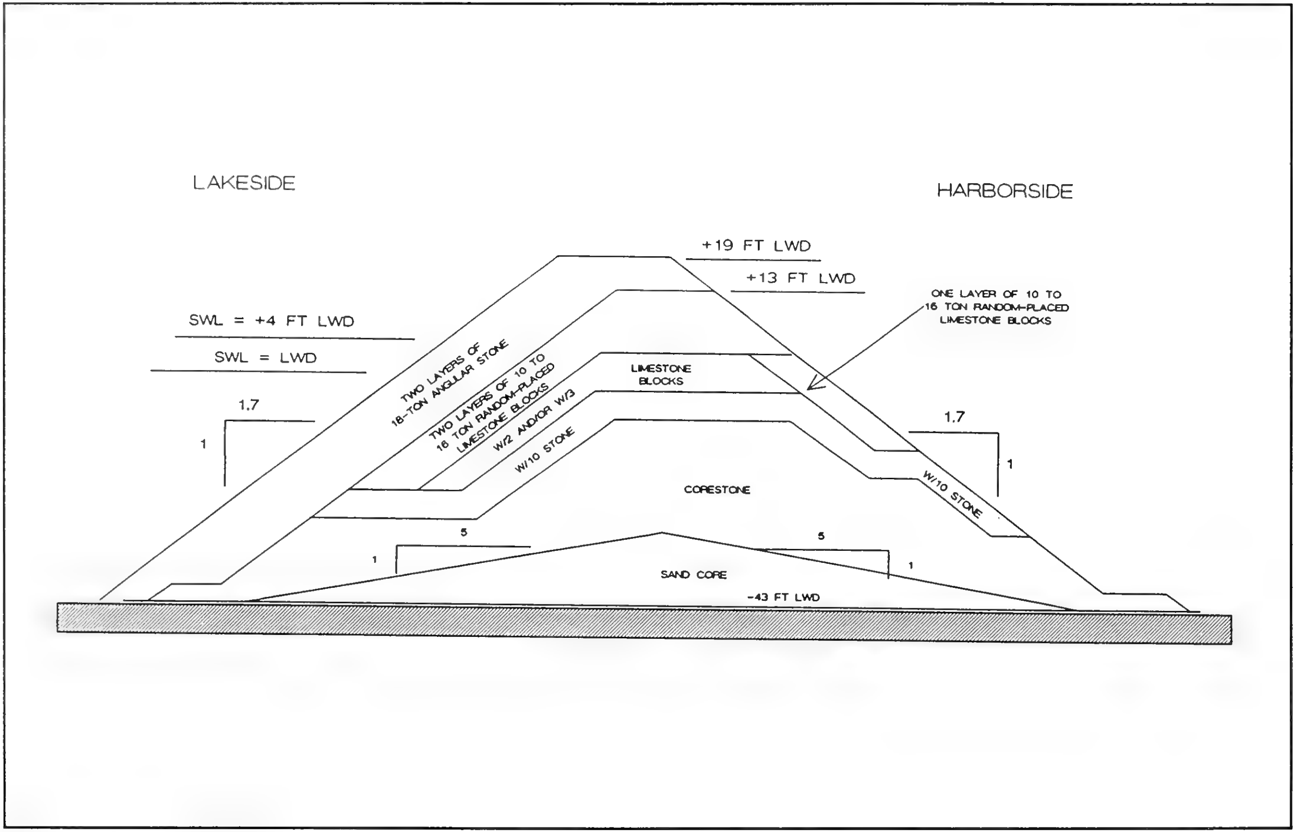

Figure 22. Plan 7

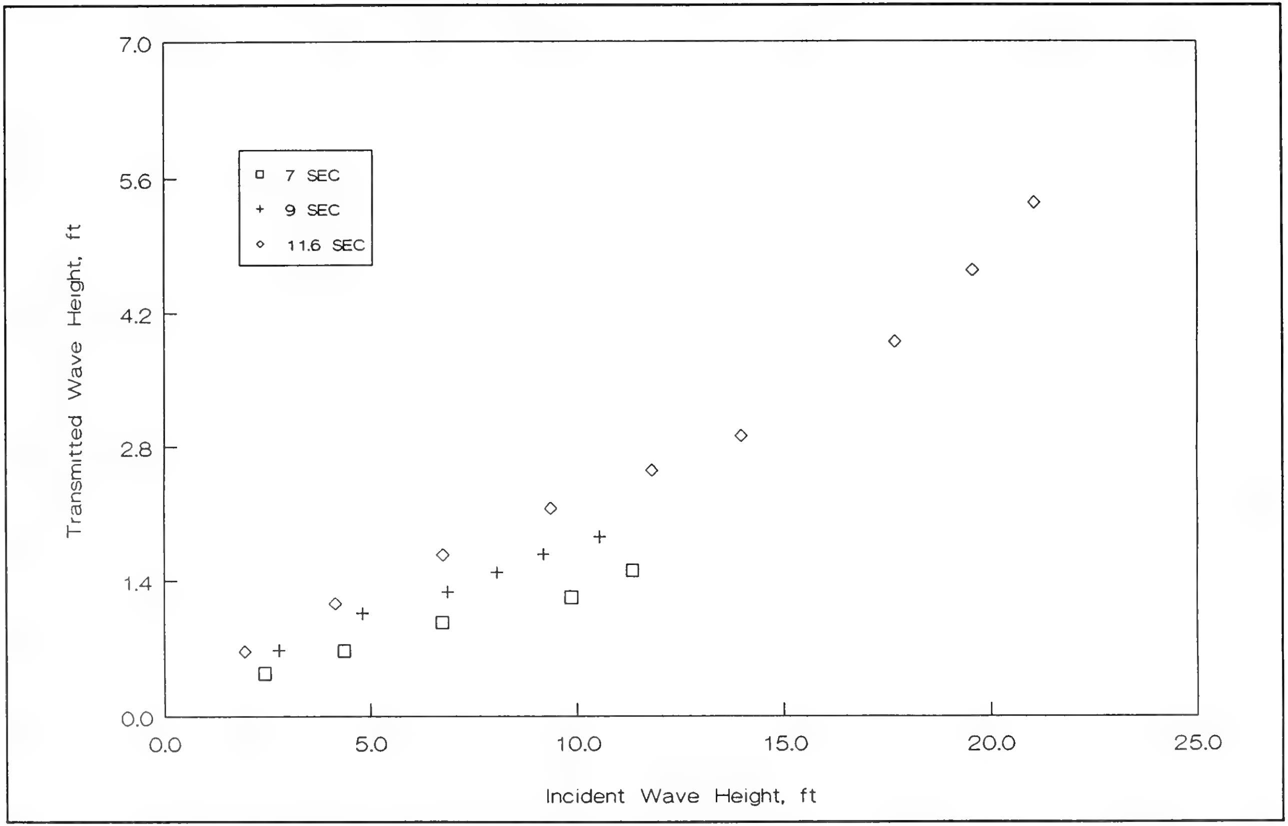

Figure 23. Transmission test results for Plan 8

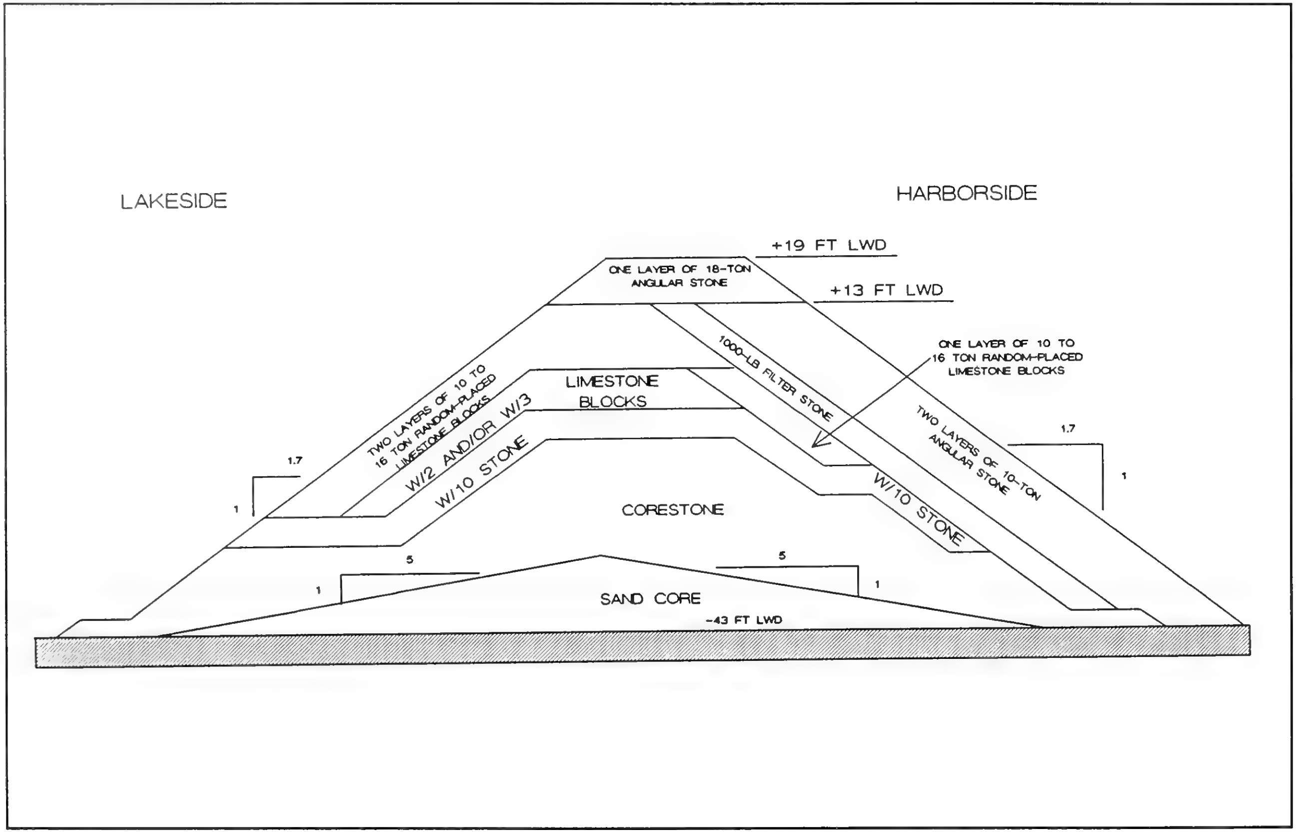

Figure 24. Plan 8A

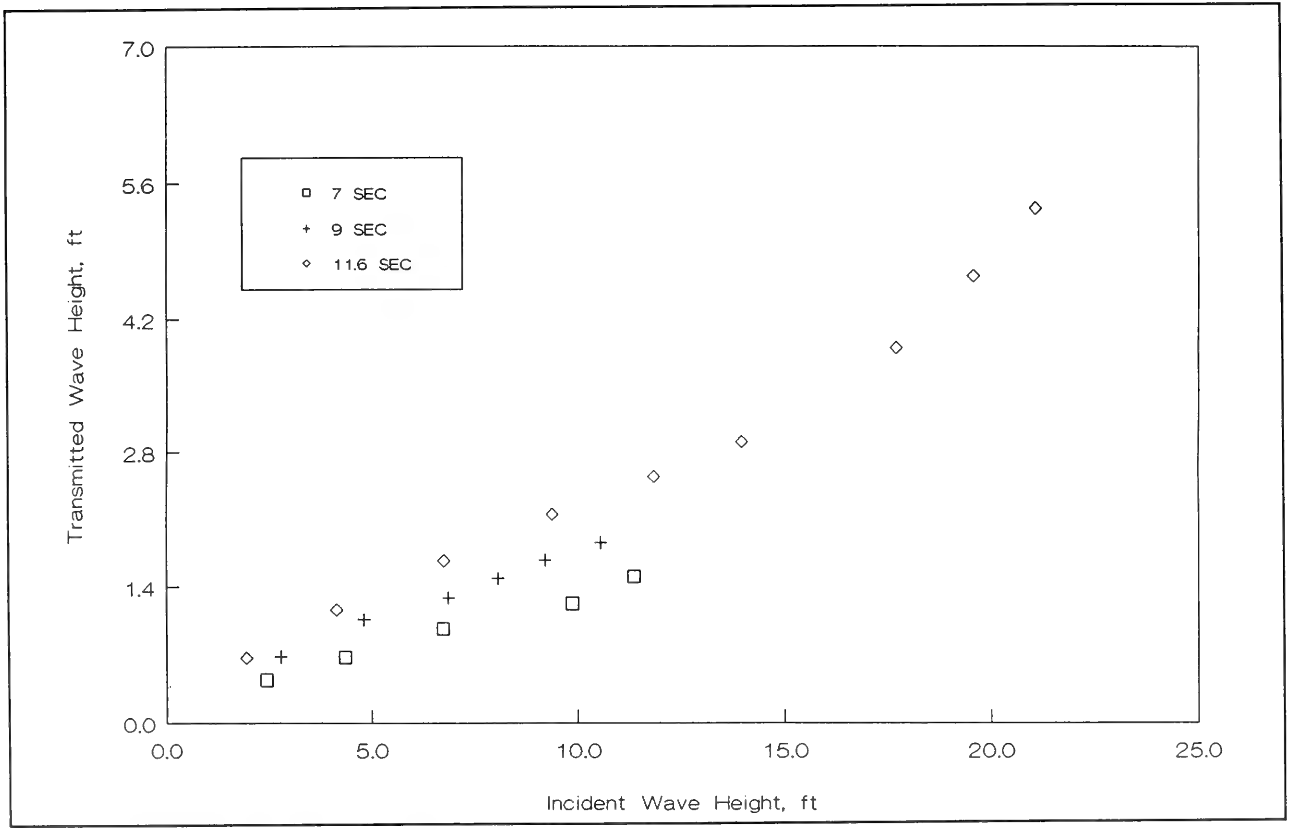

Figure 25. Transmission test results for Plan 8A

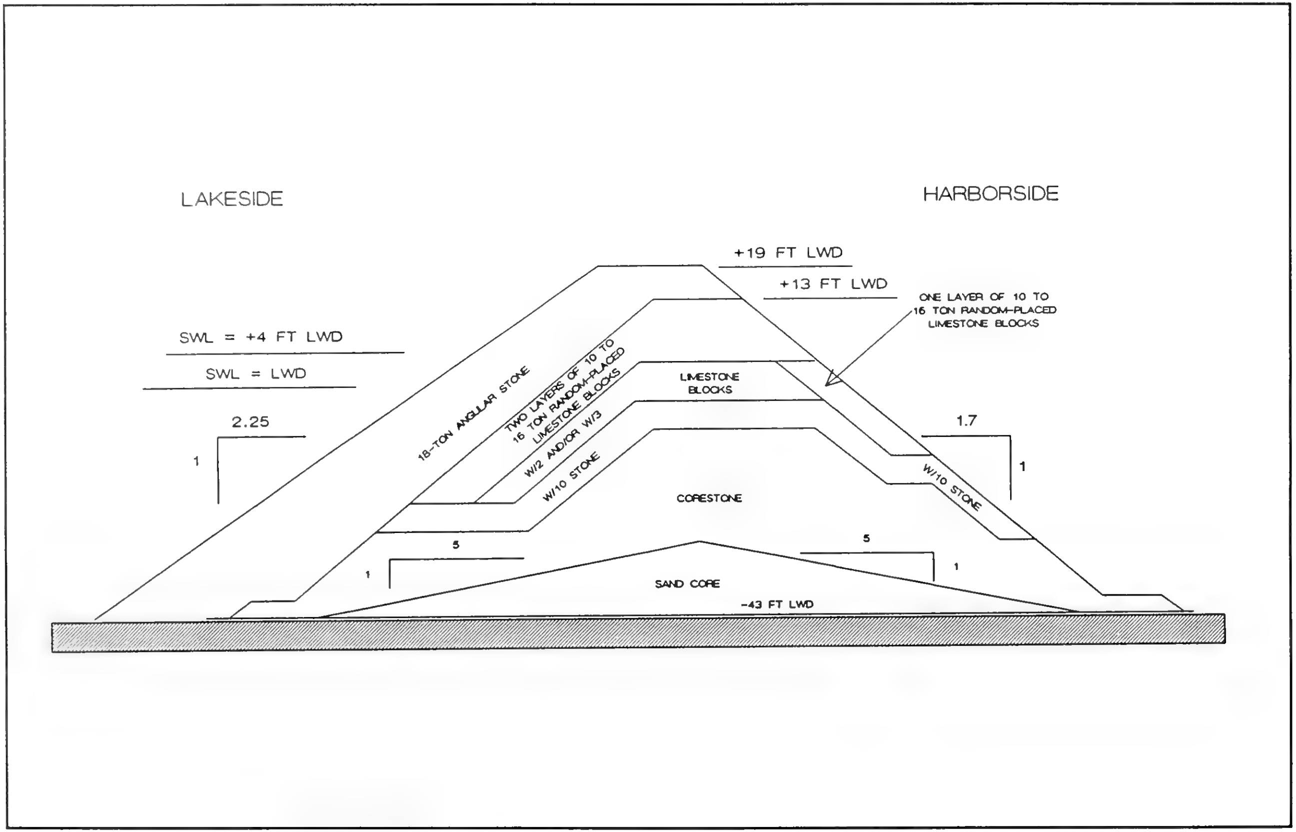

Figure 26. Plan 9

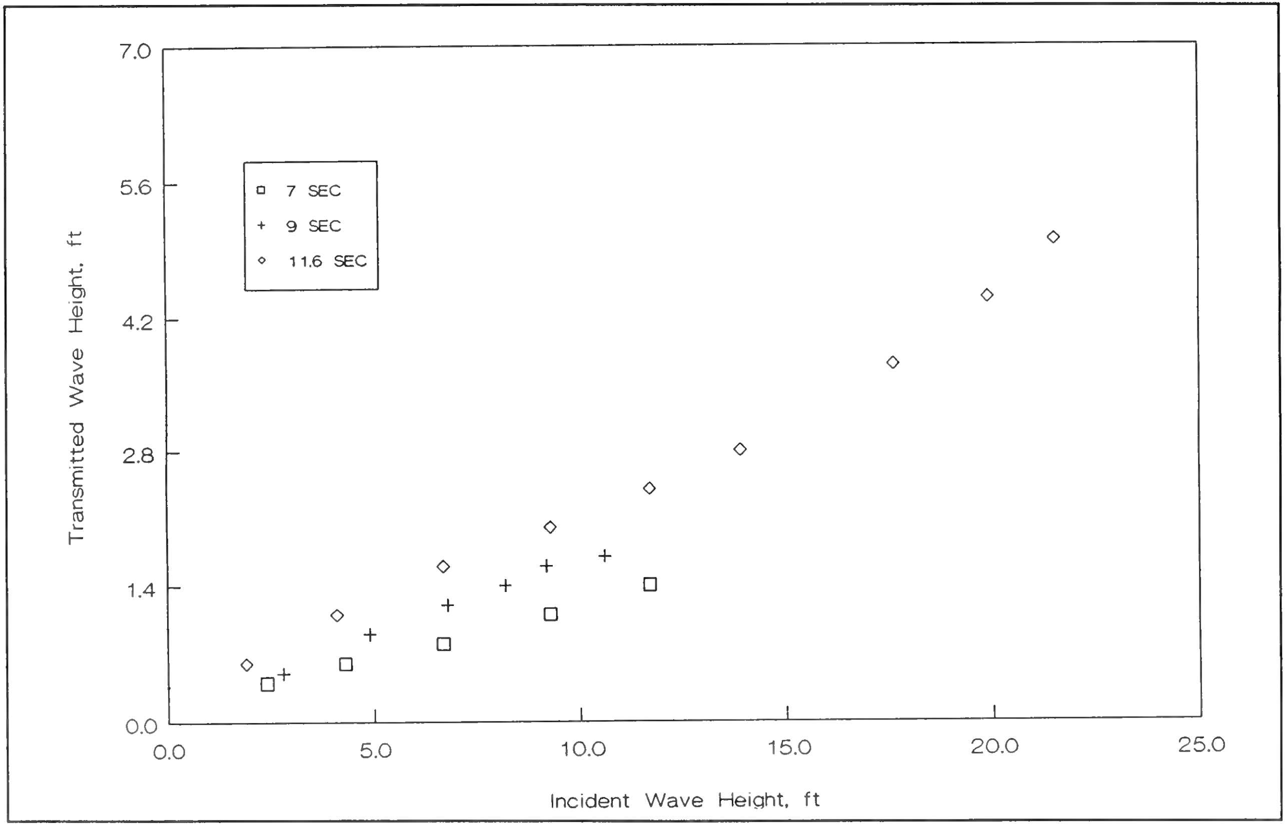

Figure 27. Transmission test results for Plan 9

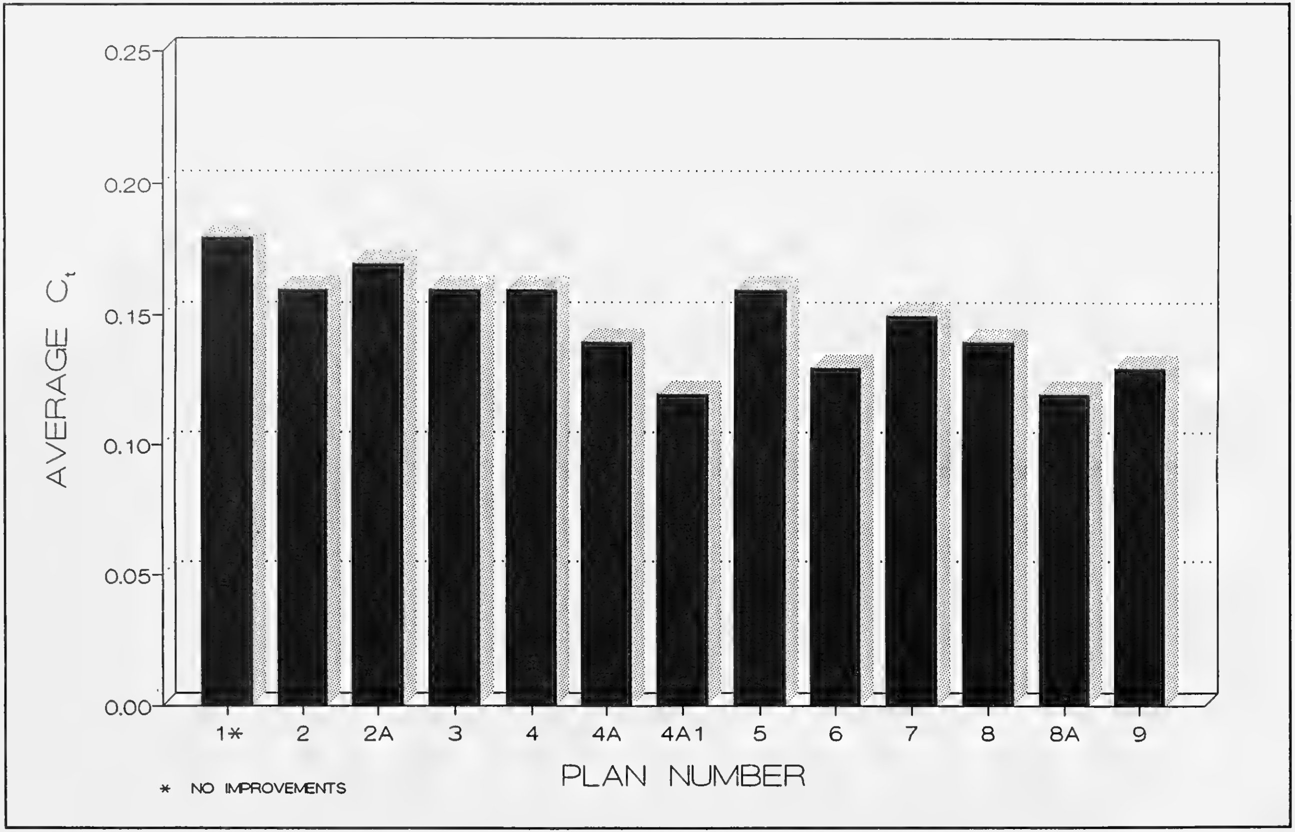

Figure 28. Average values of C_t; 7-sec wave period

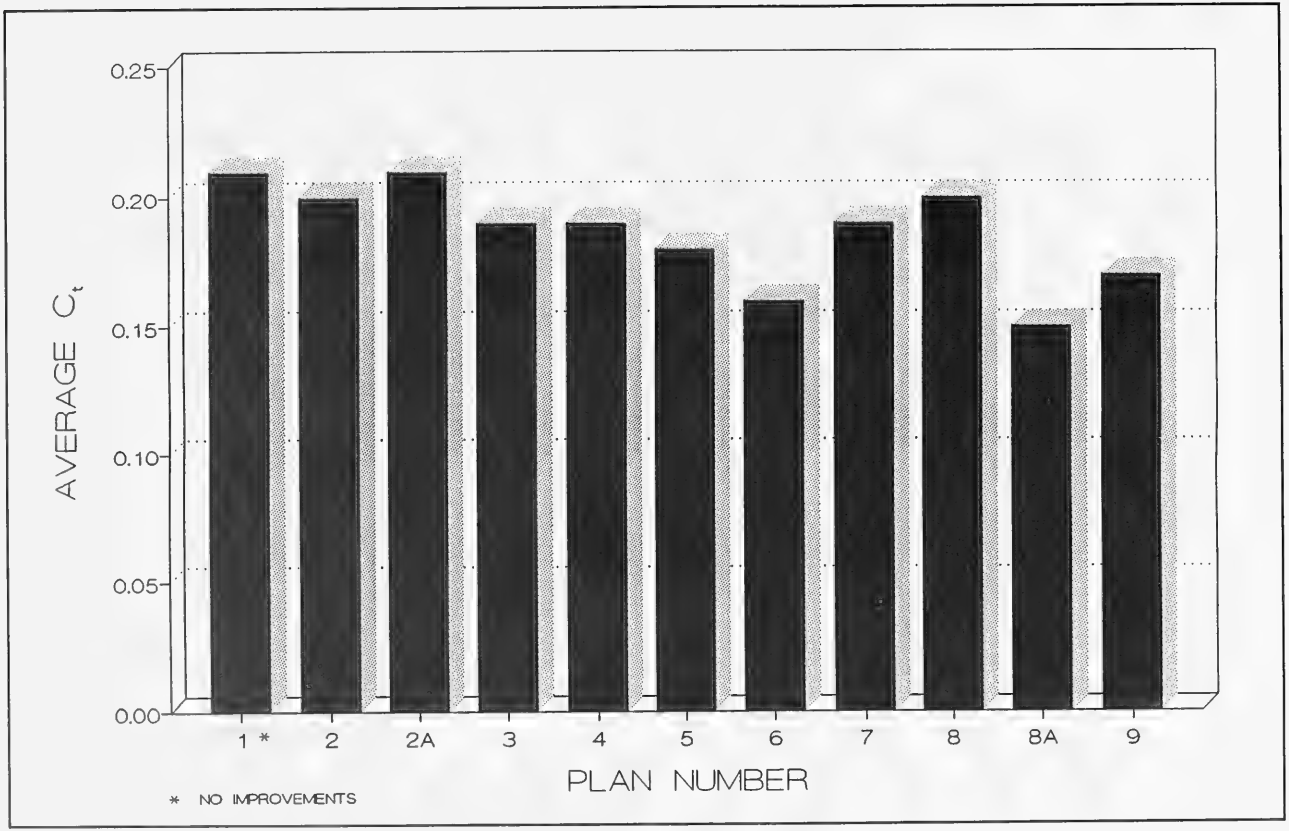

Figure 29. Average values of C_t; 9-sec wave period

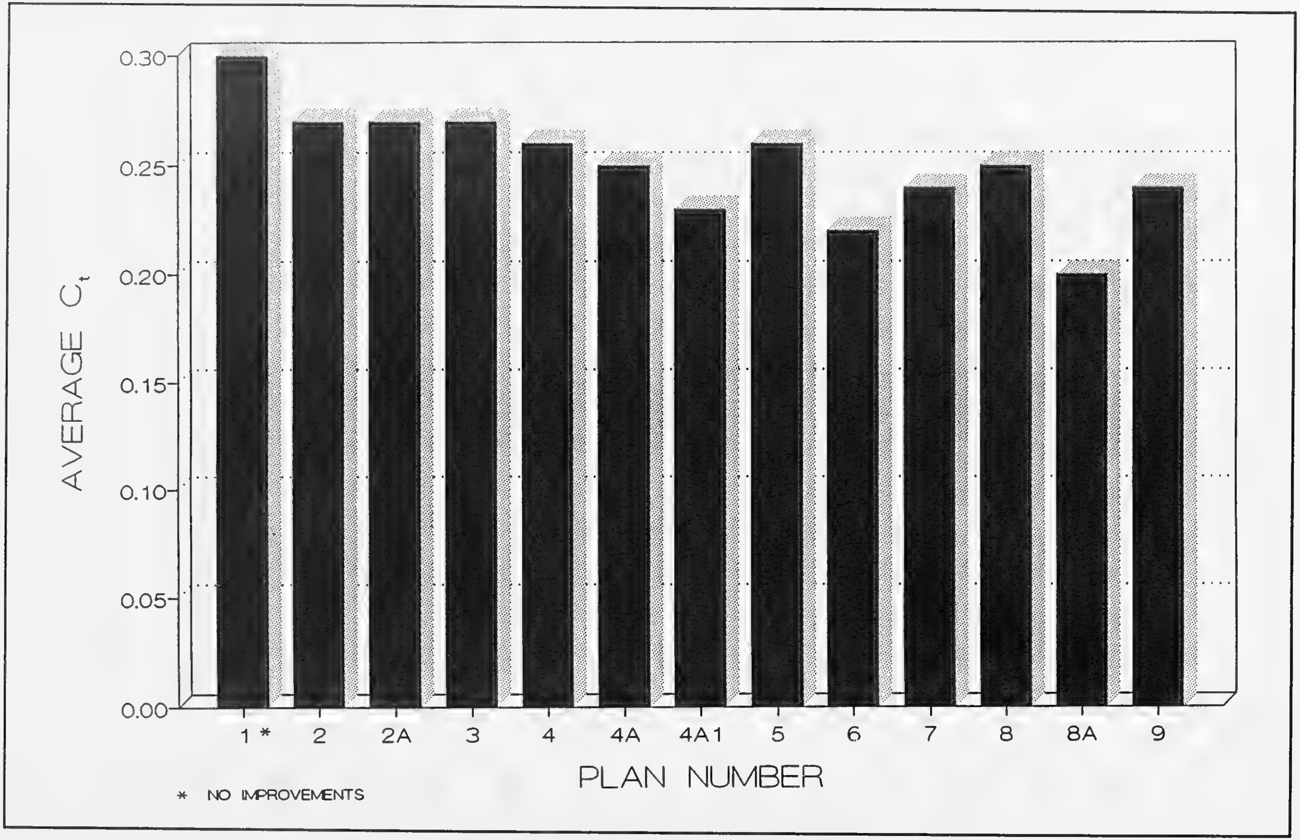

Figure 30. Average values of C_t; 11.6-sec wave period

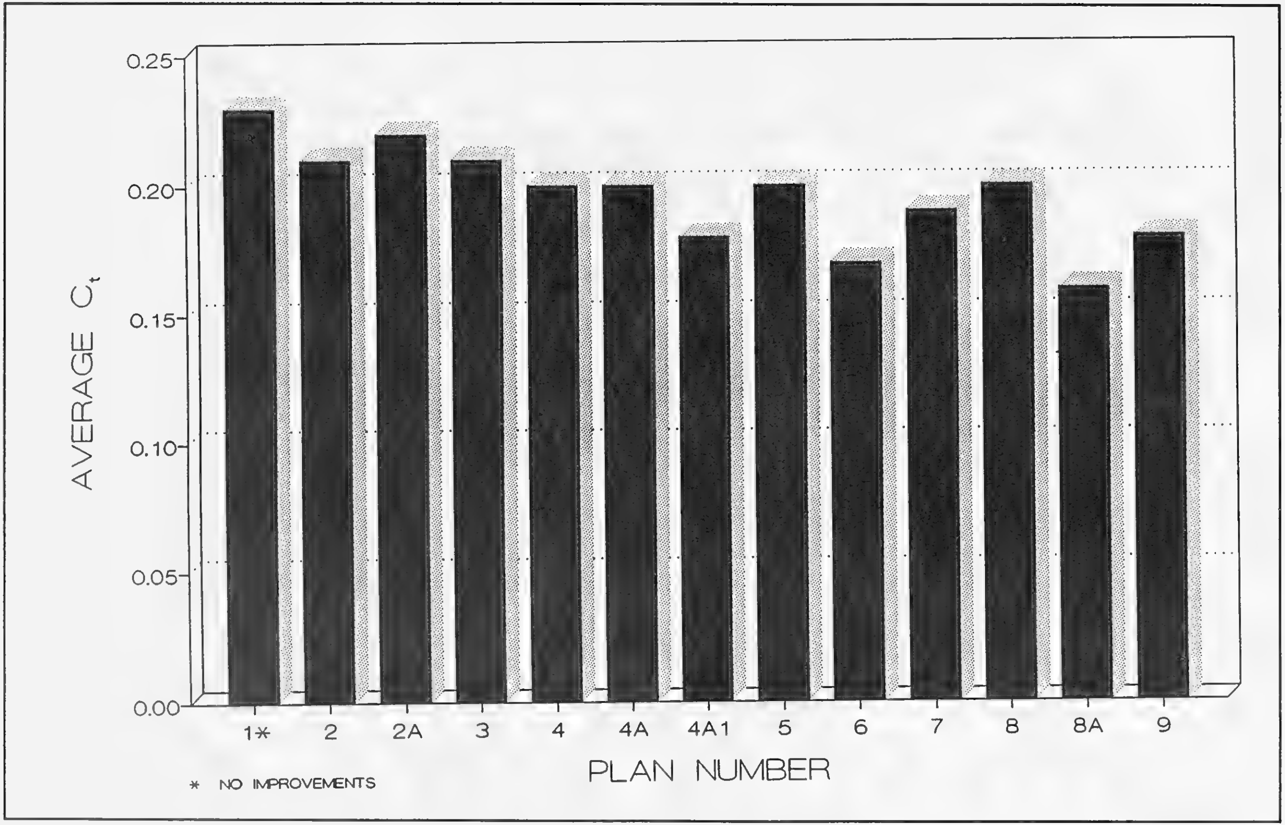

Figure 31. Average values of C_t; all wave periods

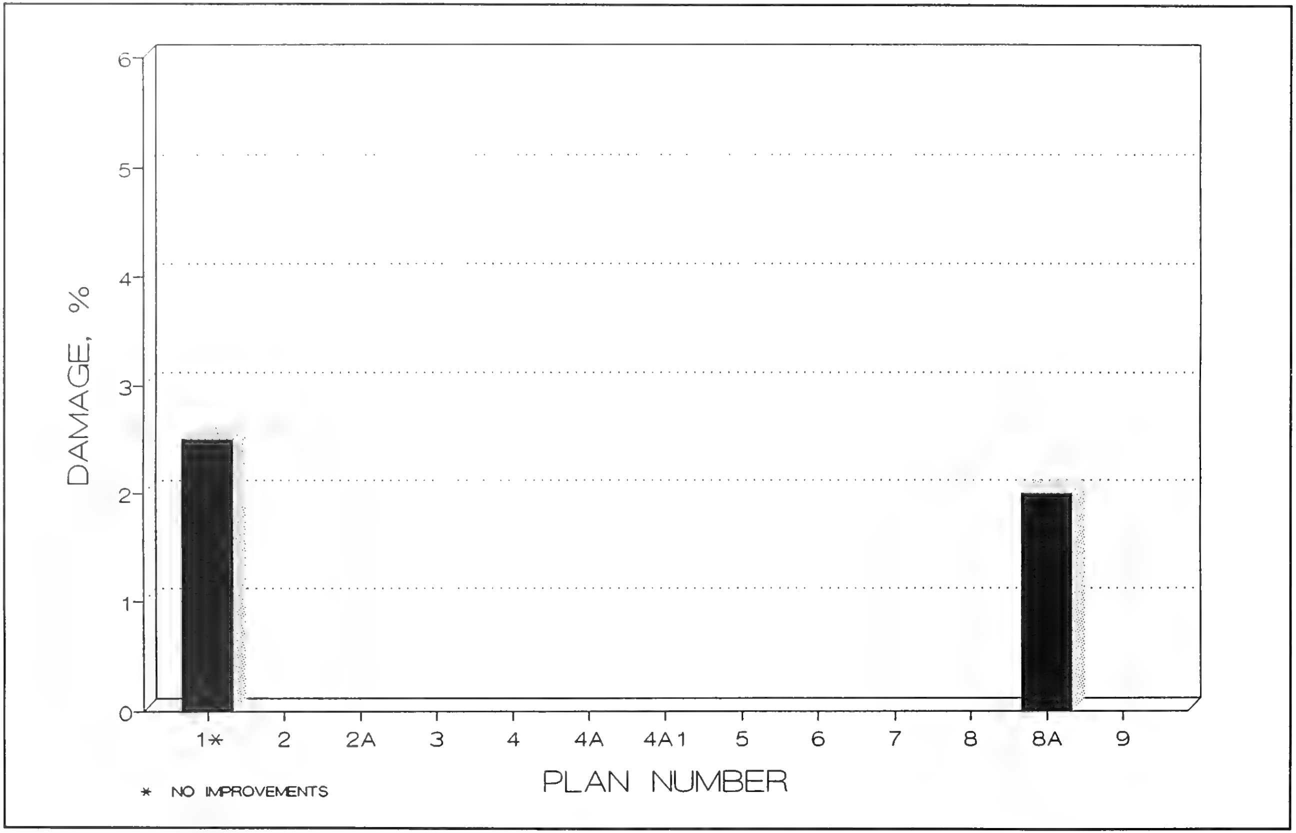

Figure 32. Percent damage incurred by lake-side armor

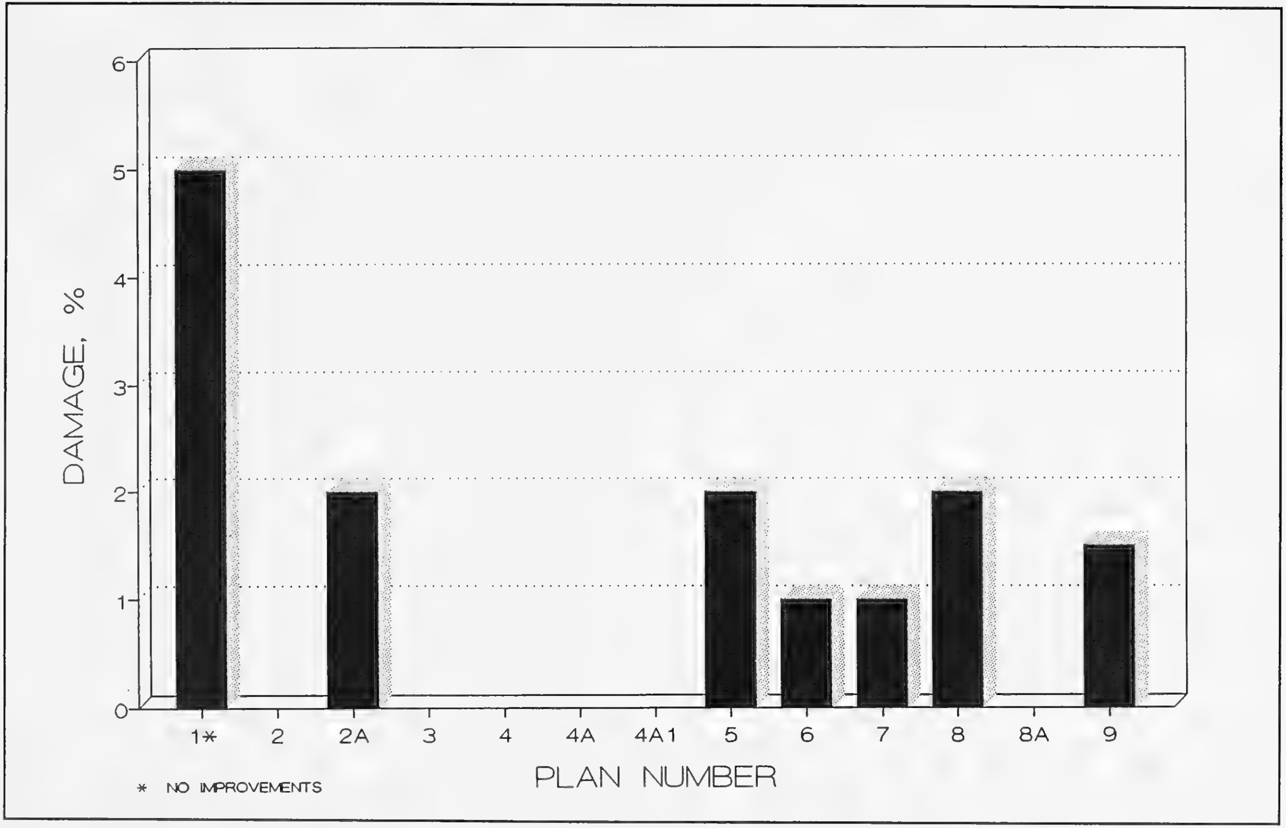

Figure 33. Percent damage incurred by harbor-side armor

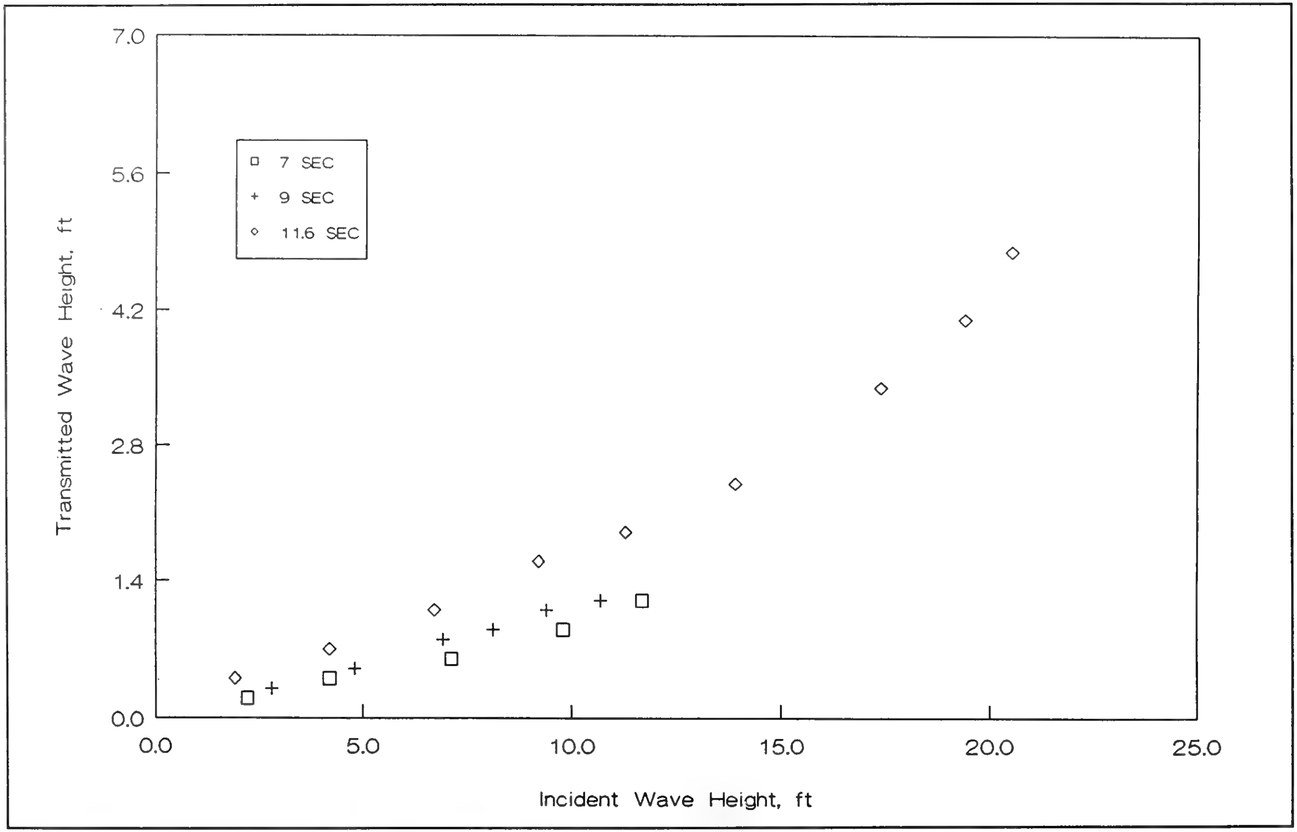

Figure 34. Transmission test results for Plan 10; 0.0-ft swl

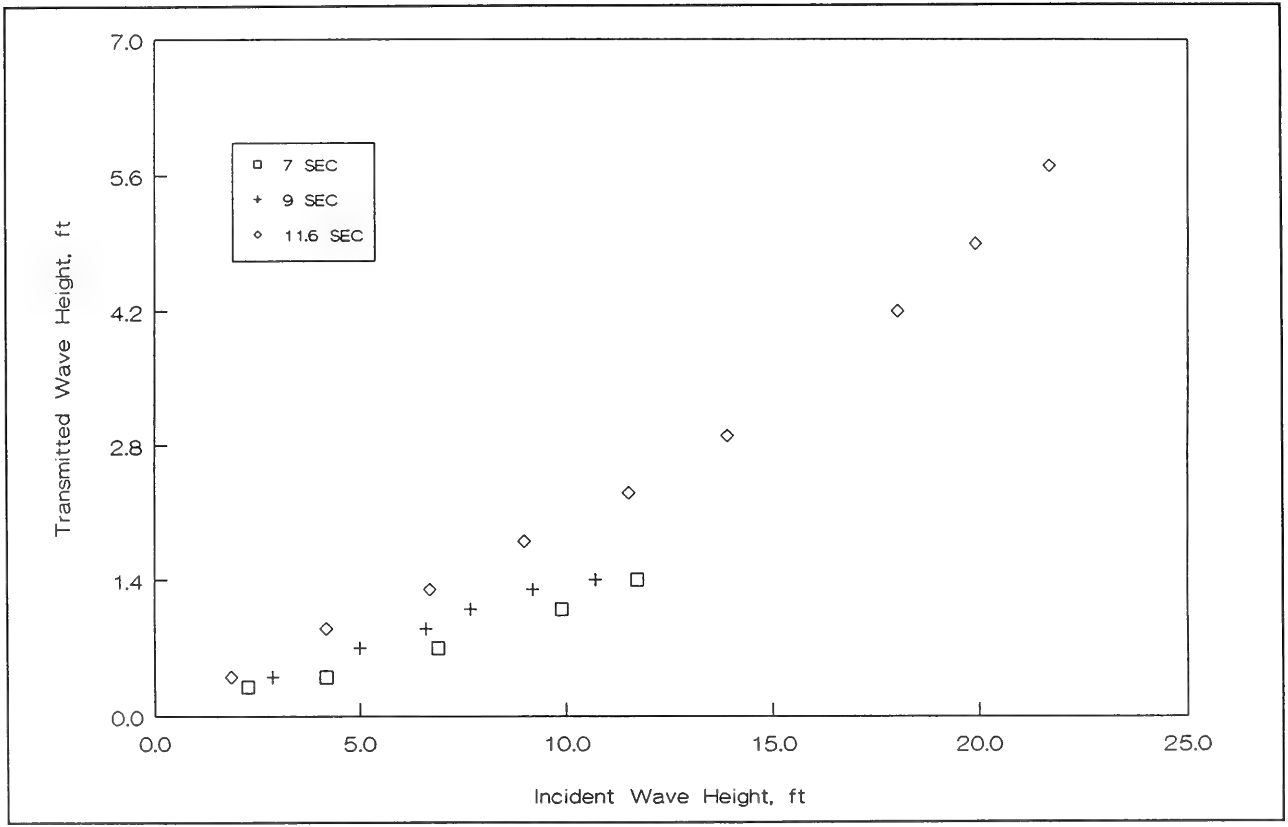

Figure 35. Transmission test results for Plan 10; +4.0-ft swl

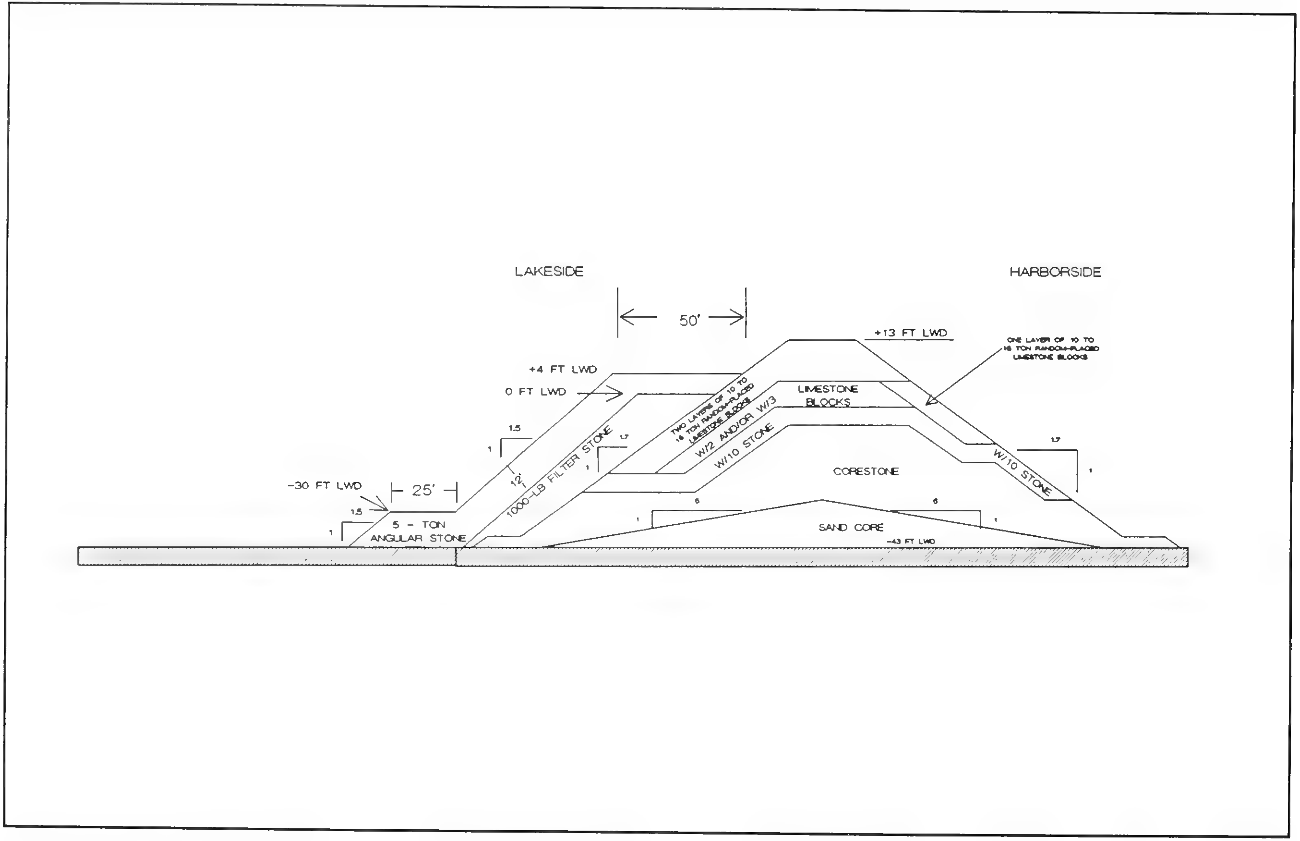

Figure 36. Plan 10A

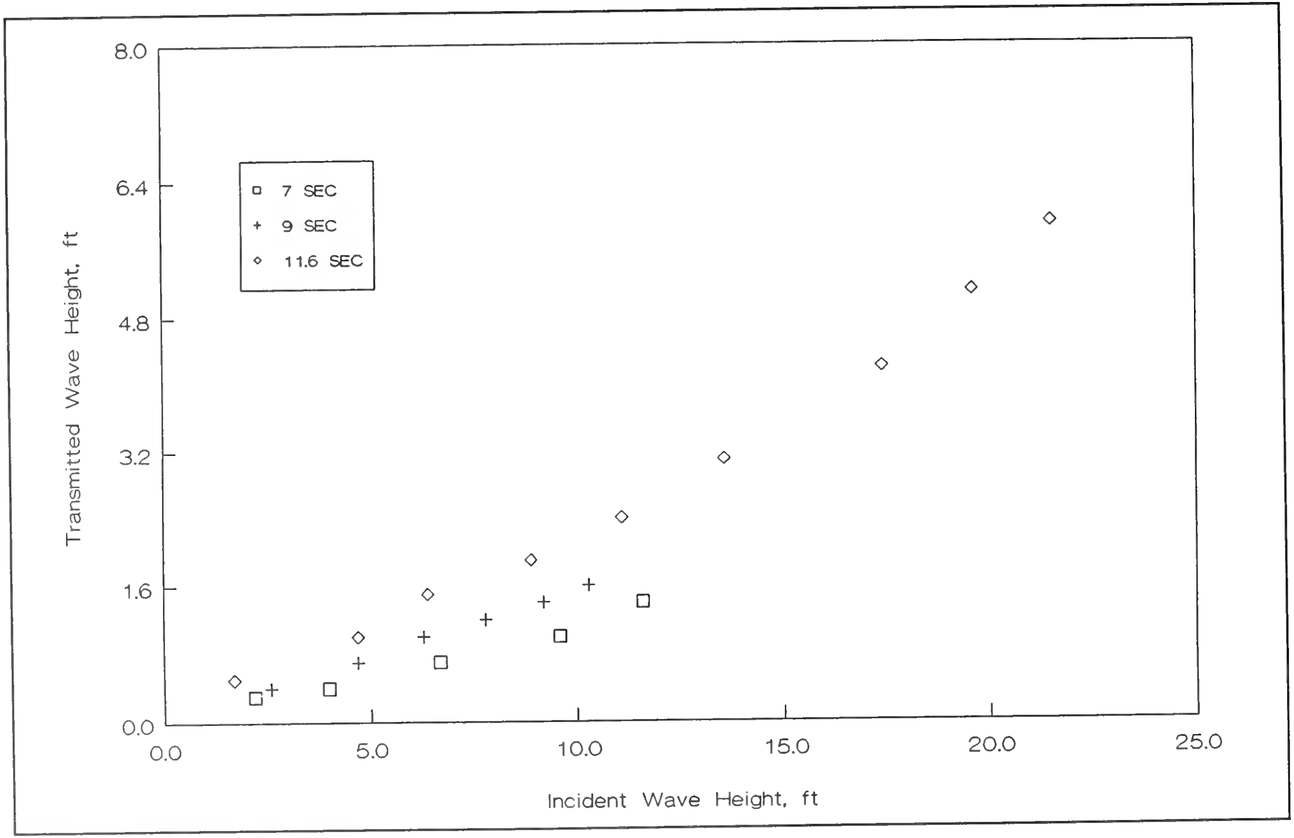

Figure 37. Transmission test results for Plan 10A

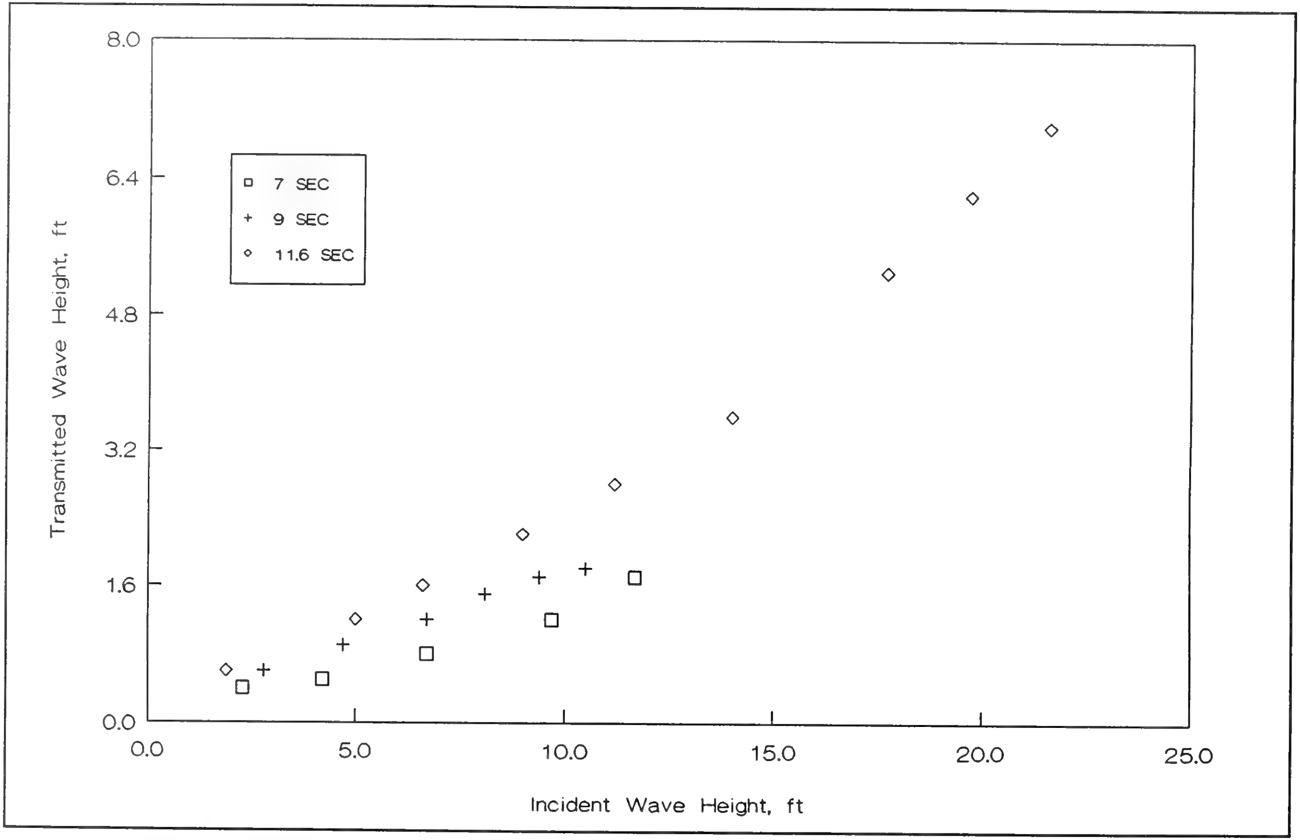

Figure 38. Transmission test results for Plan 11

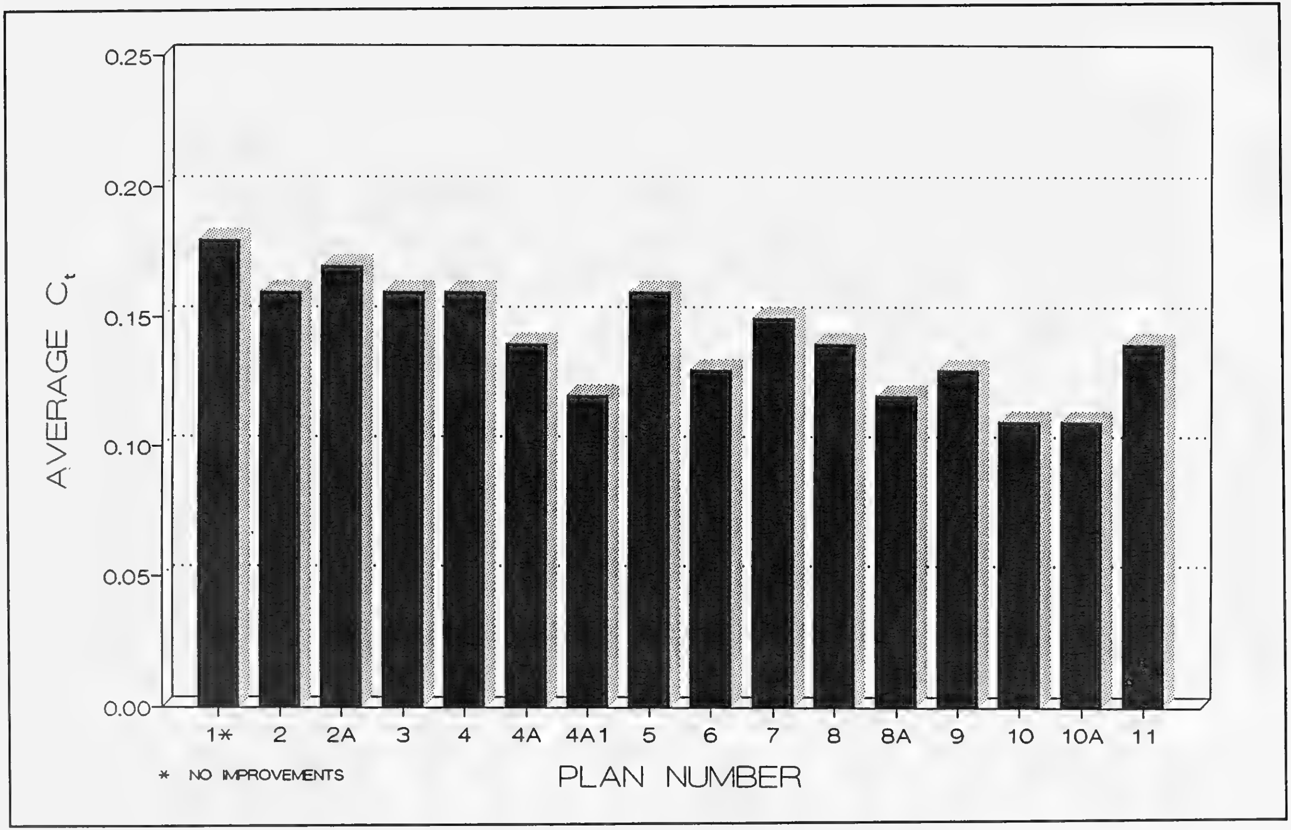

Figure 39. Average values of C_t for all plans; 7-sec wave period

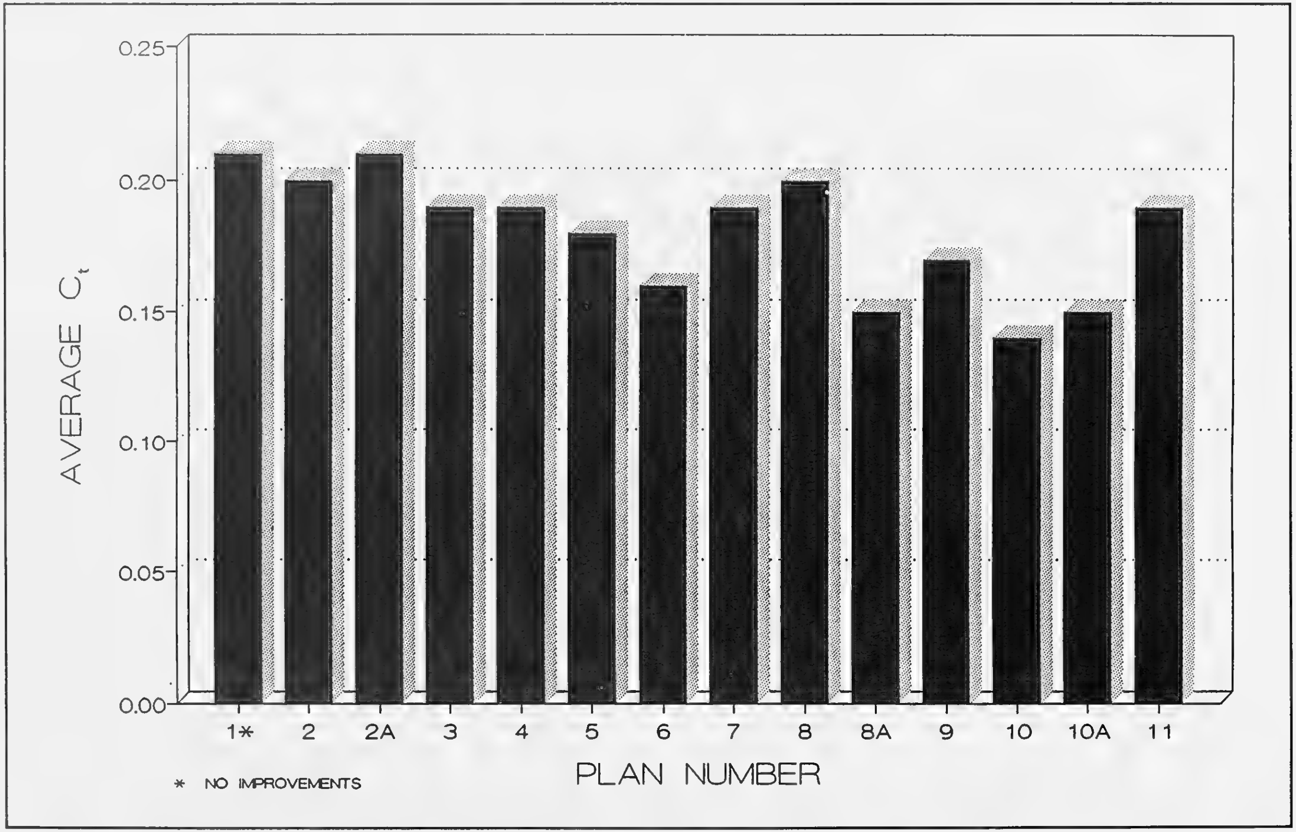

Figure 40. Average values of C_t for all plans; 9-sec period

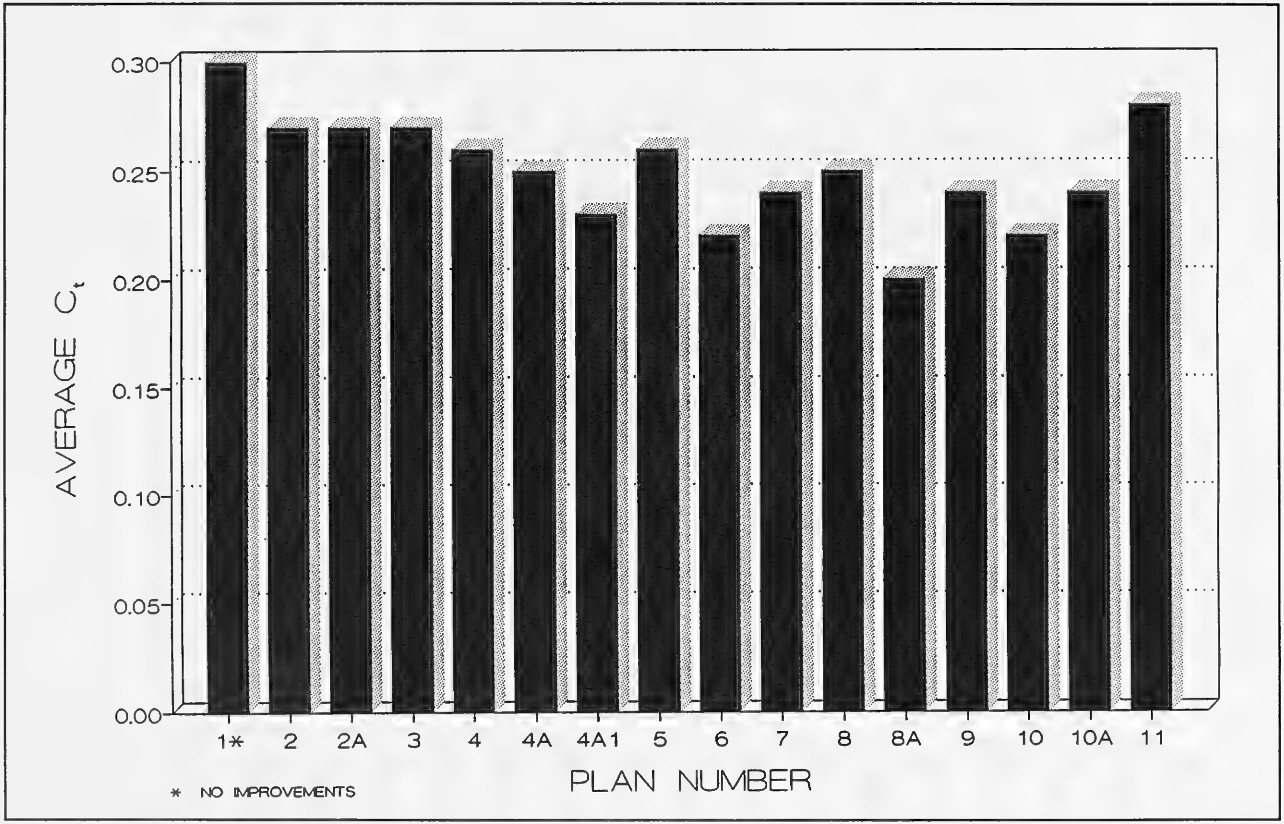

Figure 41. Average values of C_t for all plans; 11.6-sec wave period

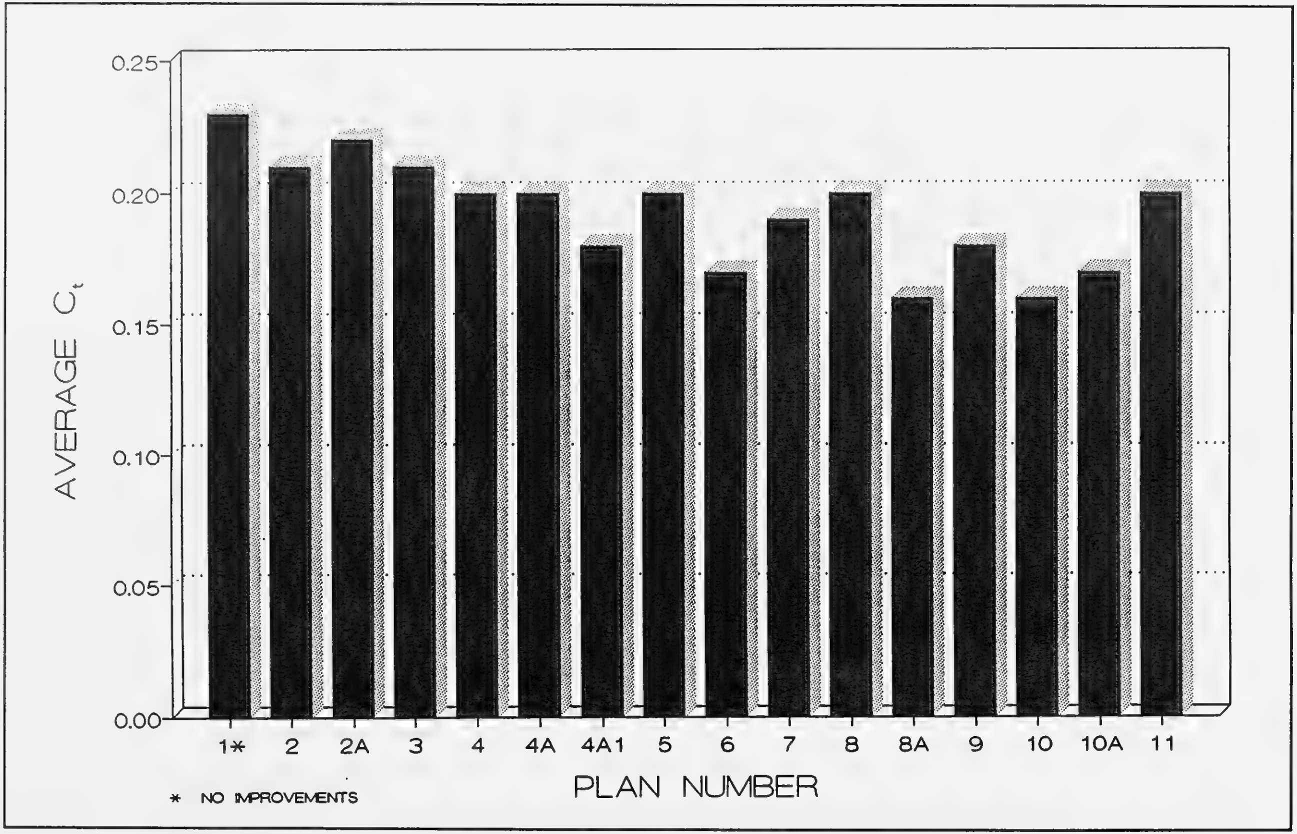

Figure 42. Average values of C_t for all plans

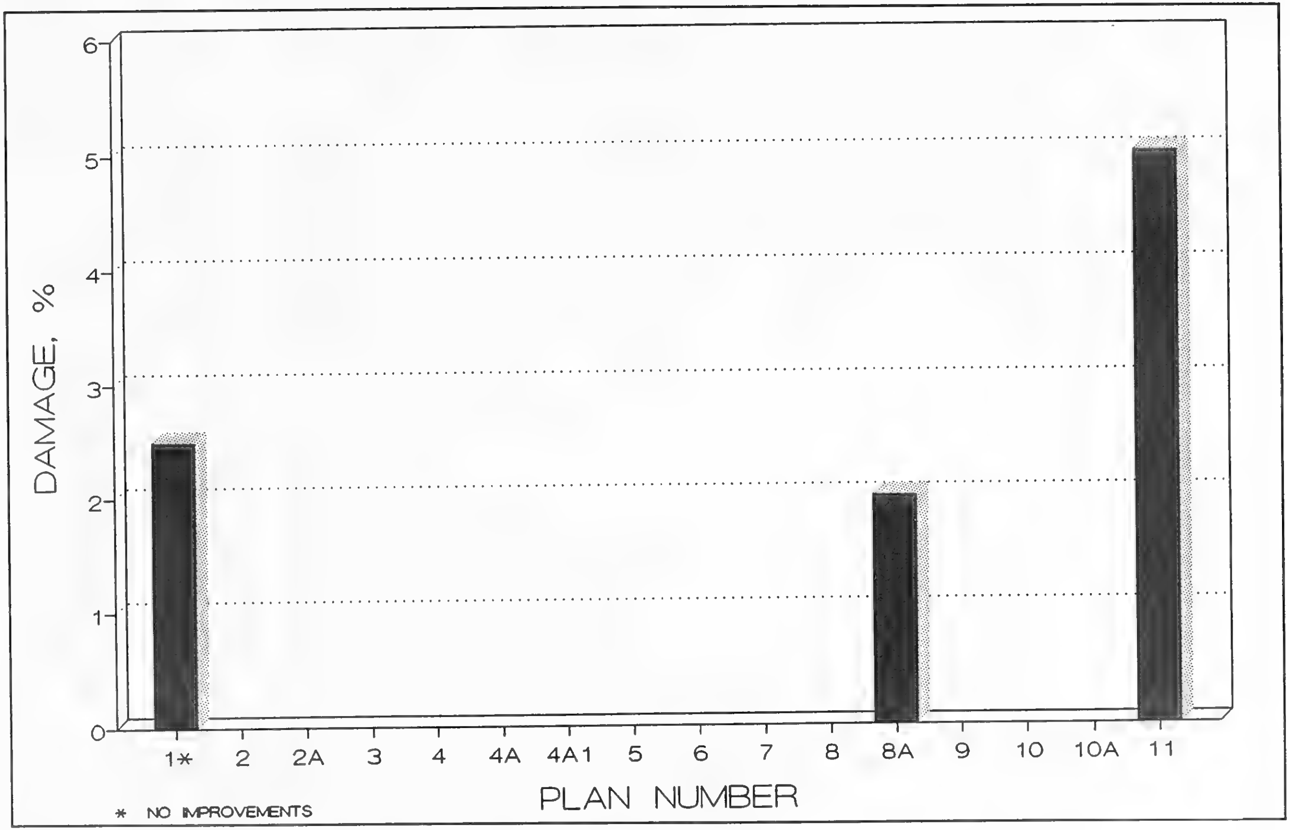

Figure 43. Percent damage incurred by lakeside armor; all plans

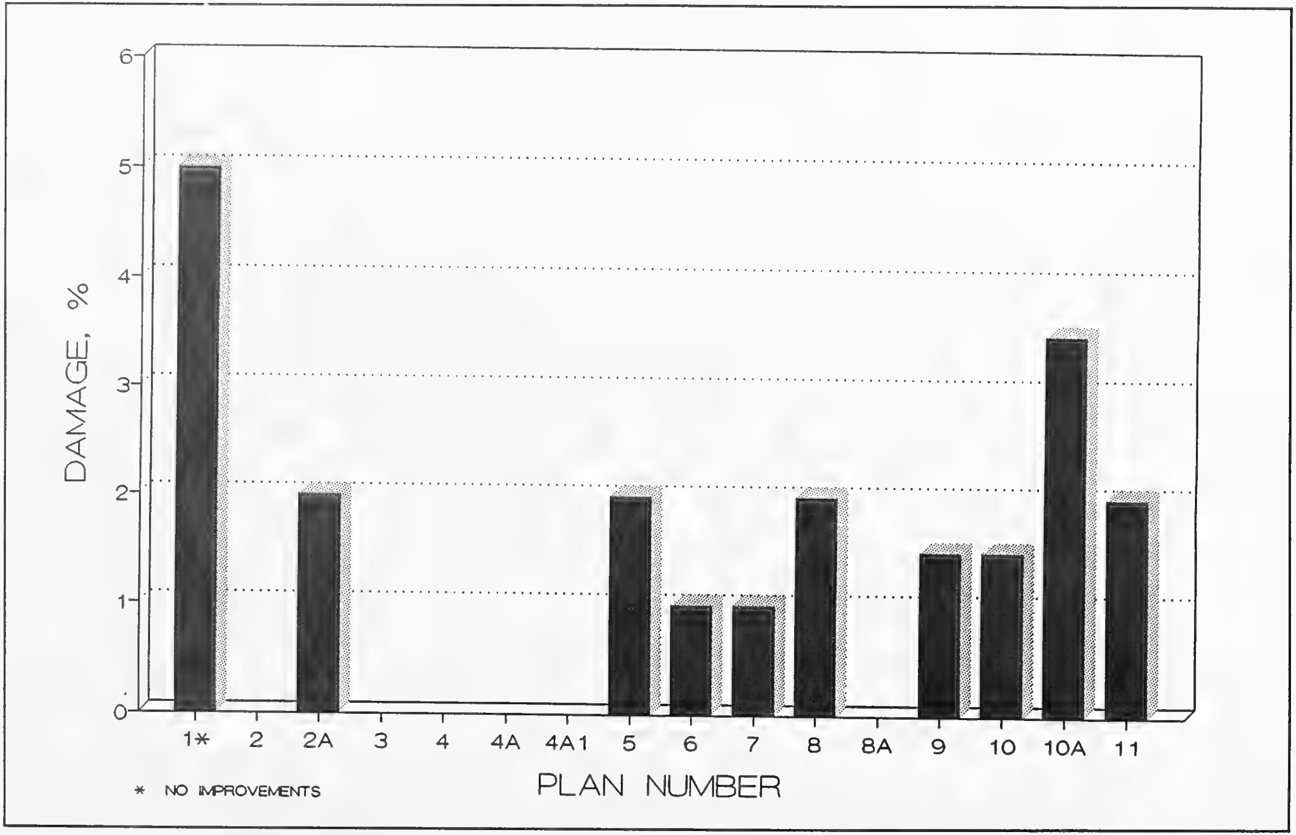

Figure 44. Percent damage incurred by harbor-side armor; all plans

Photo 1. End view of Plan 1 before wave attack

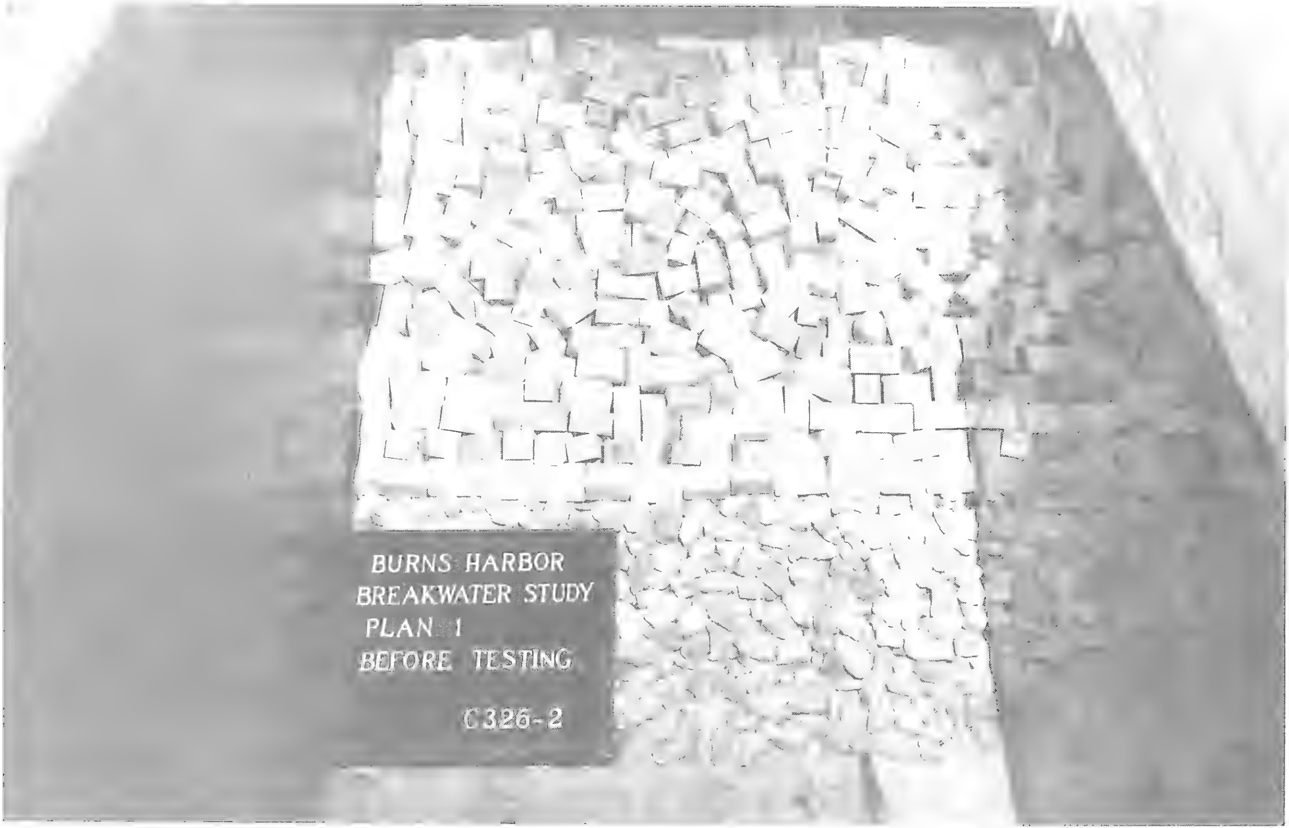

Photo 2. Lakeside view of Plan 1 before wave attack

Photo 3. Harbor-side view of Plan 1 before wave attack

Photo 4. End view of Plan 1A before wave attack

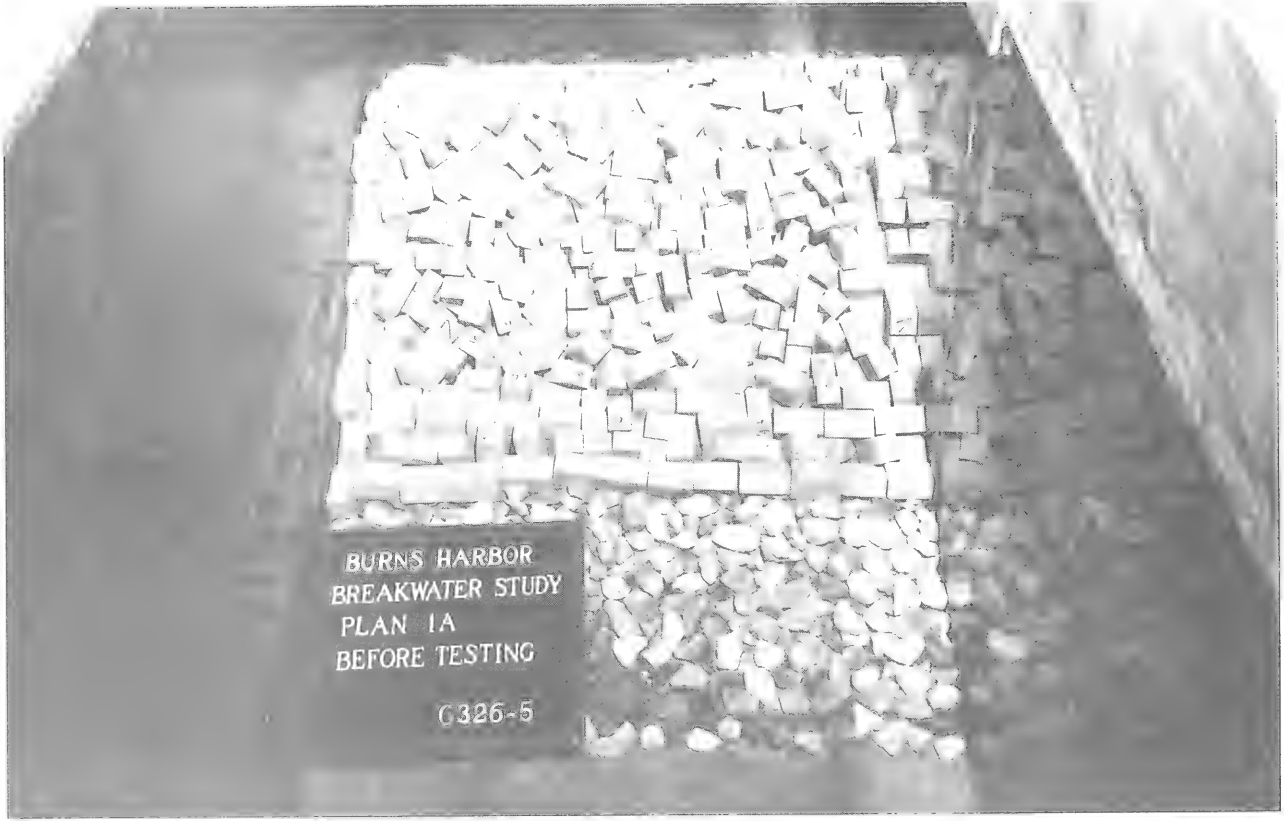

Photo 5. Lakeside view of Plan 1A before wave attack

Photo 6. Lakeside view of Plan 1A1 after the initial stability test

Photo 7. Harbor-side view of Plan 1A1 after the initial stability test

Photo 8. Lakeside view of Plan 1A1 after the repeat stability test

Photo 9. Harbor-side view of Plan 1A1 after the repeat stability test

Photo 10. End view of Plan 1A2 before wave attack

Photo 11. Lakeside view of Plan 1A2 before wave attack

Photo 12. Harbor-side view of Plan 1A2 before wave attack

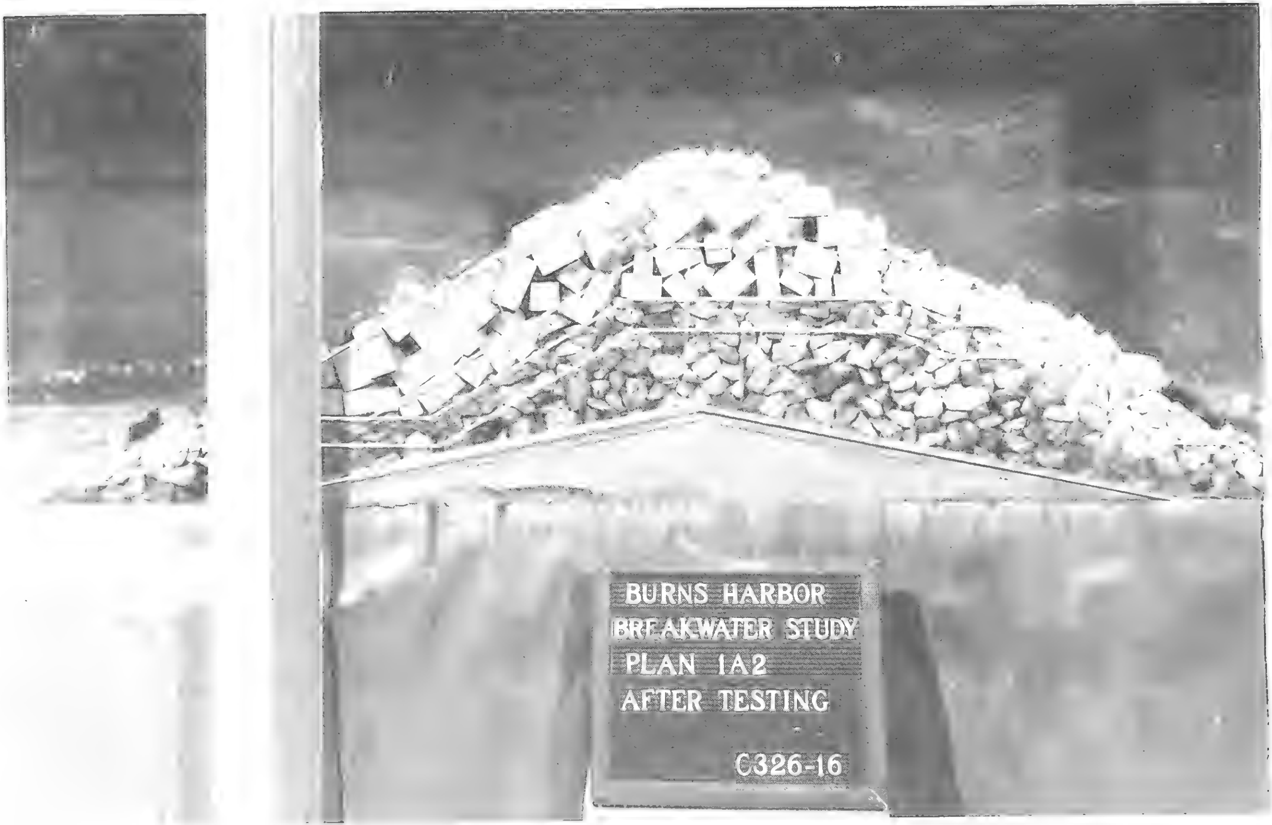

Photo 13. End view of Plan 1A2 after wave attack

Photo 14. Lakeside view of Plan 1A2 after wave attack

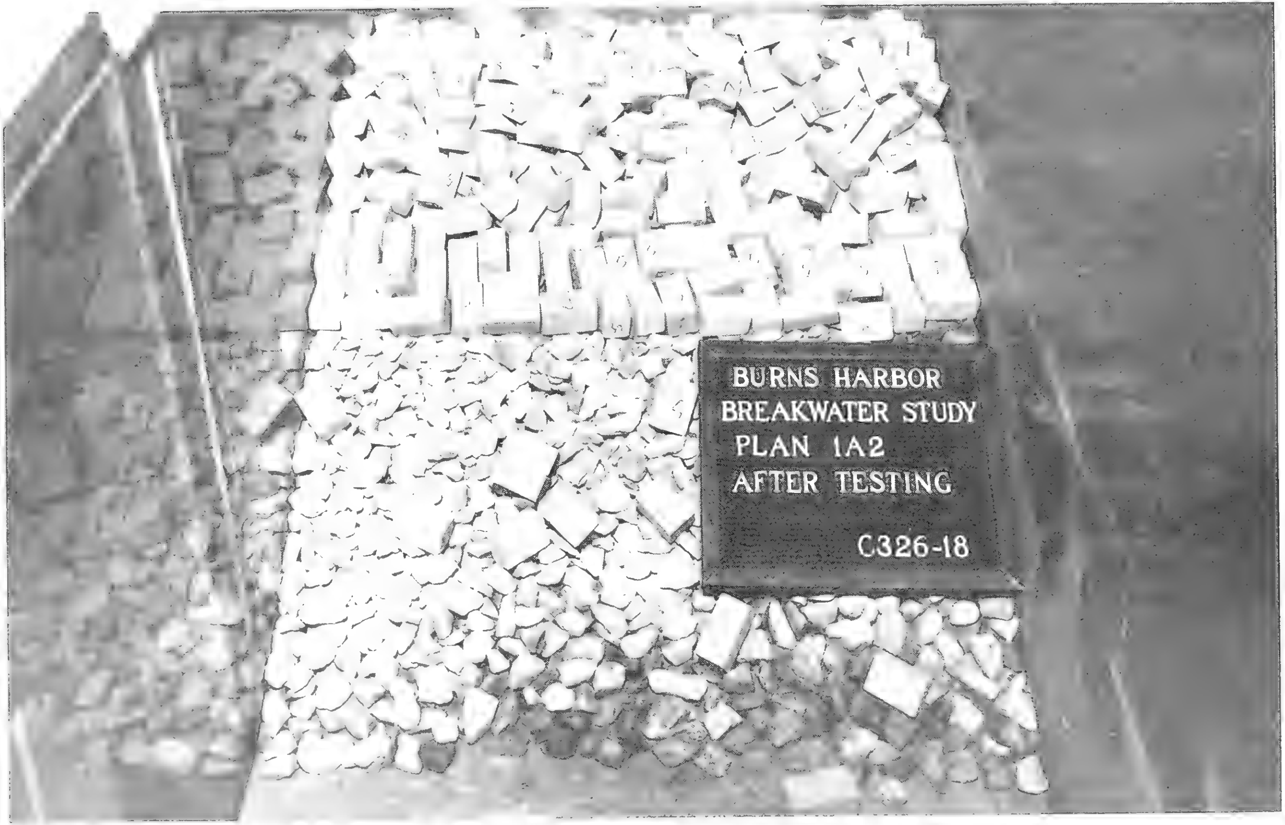

Photo 15. Harbor-side view of Plan 1A2 after wave attack

Photo 16. End view of Plan 1A3 before wave attack

Photo 17. End view of Plan 1A3 after wave attack

Photo 18. Lakeside view of Plan 1A3 after wave attack

Photo 19. Harbor-side view of Plan 1A3 after wave attack

Photo 20. End view of Plan 2 (reef) after wave attack

Photo 21. End view of Plan 2 (existing structure) after wave attack

Photo 22. End view of Plan 2A (reef) after wave attack

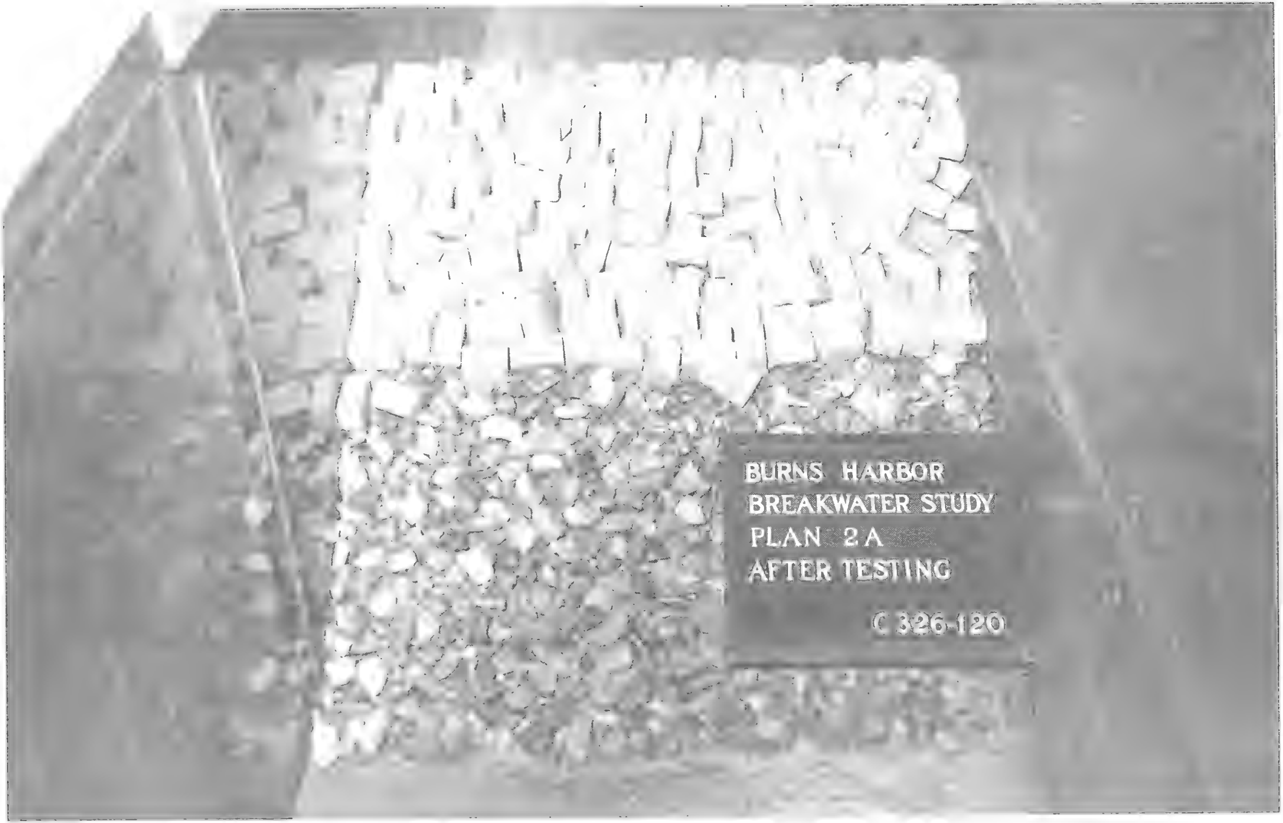

Photo 23. Harbor-side view of Plan 2A (existing structure) after wave attack

Photo 24. End view of Plan 3 before wave attack

Photo 25. End view of Plan 3 (reef) after wave attack

Photo 26. End view of Plan 3 (existing structure) after wave attack

Photo 27. End view of Plan 4 after wave attack

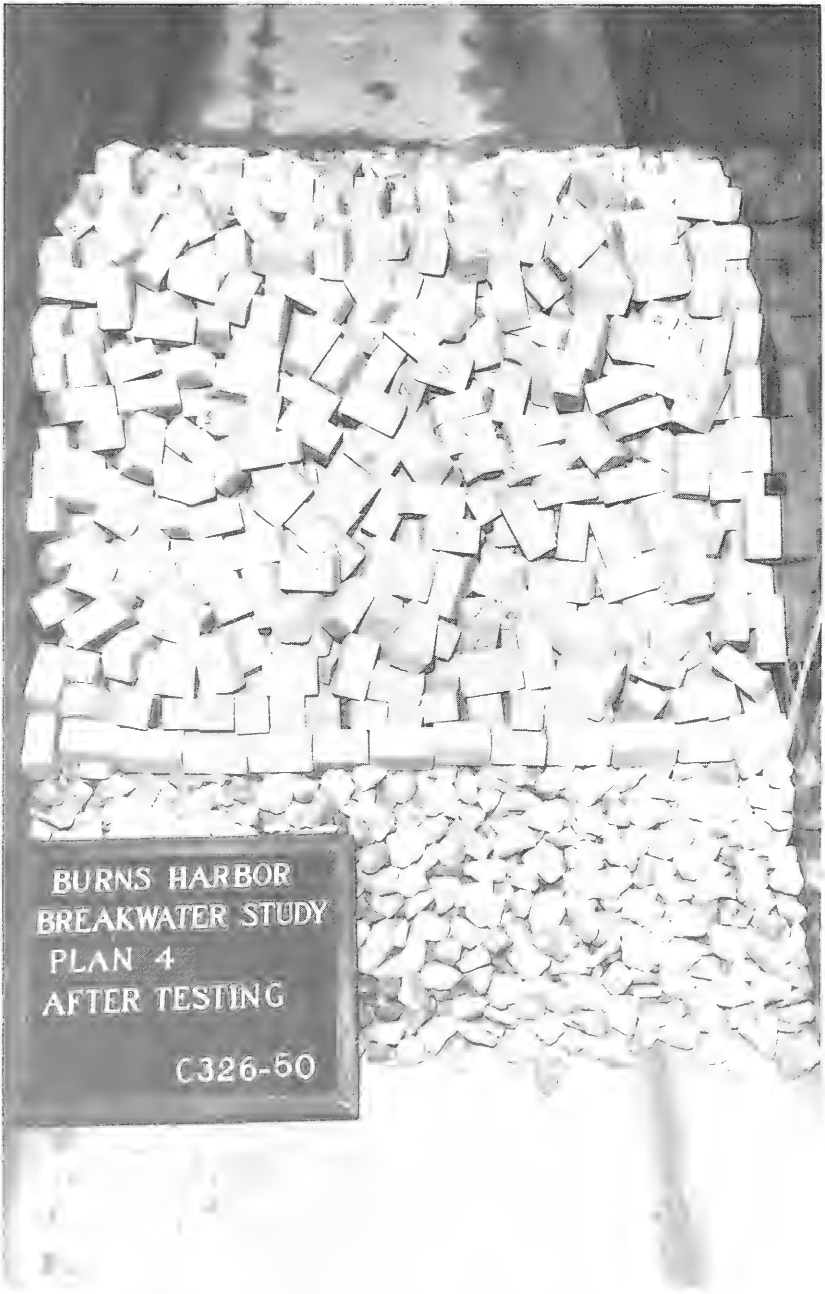

Photo 28. Lakeside view of Plan 4 (existing structure) after wave attack

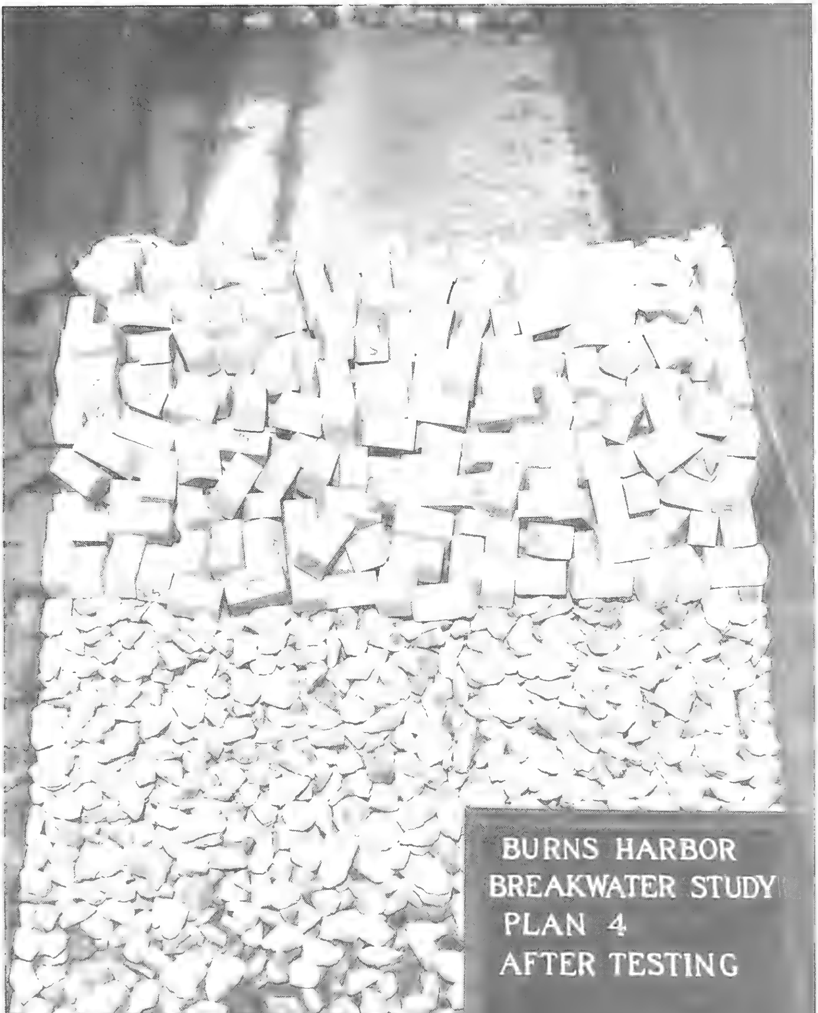

Photo 29. Harbor-side view of Plan 4 (existing structure) after wave attach

Photo 30. Lakeside view of Plan 4A (existing structure) after wave attack

Photo 31. Harbor-side view of Plan 4A (existing structure) after wave attack

Photo 32. End view of Plan 4A after wave attack

Photo 33. Lakeside view of Plan 4A1 before wave attack

Photo 34. Lakeside view of Plan 4A1 after wave attack

Photo 35. Harbor-side view of Plan 4A1 after wave attack

Photo 36. End view of Plan 5 before wave attack

Photo 37. Harbor-side view of Plan 5 after wave attack

Photo 38. End view of Plan 6 before wave attack

Photo 39. Lakeside view of Plan 6 before wave attack

Photo 40. End view of Plan 6 after wave attack

Photo 41. Lakeside view of Plan 6 after wave attack

Photo 42. Harbor-side view of Plan 6 after wave attack

Photo 43. End view of Plan 7 before wave attack

Photo 44. End view of Plan 7 after wave attack

Photo 45. Lakeside view of Plan 7 after wave attack

Photo 46. Harbor-side view of Plan 7 after wave attack

Photo 47. End view of Plan 8 after wave attack

Photo 48. Lakeside view of Plan 8 after wave attack

Photo 49. Harbor-side view of Plan 8 after wave attack

Photo 50. End view of Plan 8A before wave attack

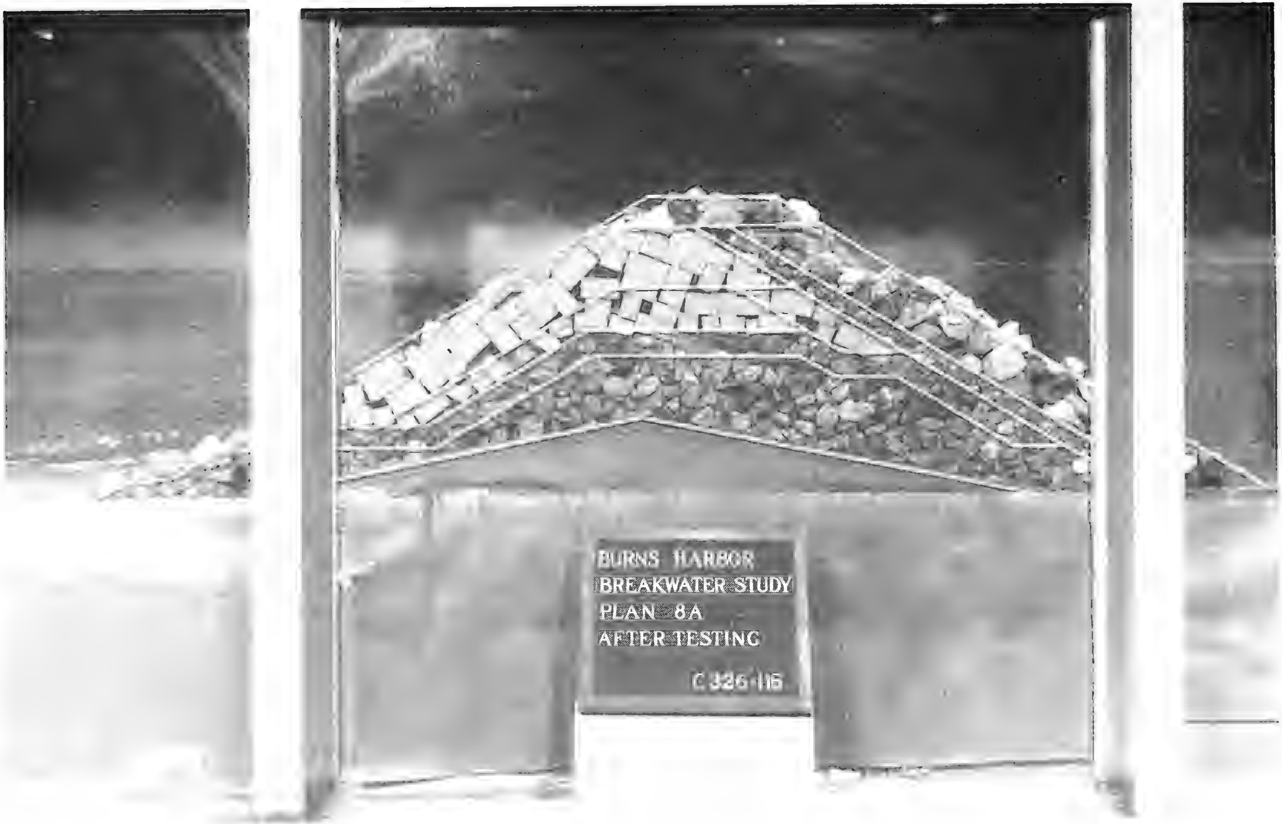

Photo 51. End view of Plan 8A after wave attack

Photo 52. Lakeside view of Plan 8A after wave attack

Photo 53. Harbor-side view of Plan 8A after wave attack

Photo 54. End view of Plan 9 before wave attack

Photo 55. Lakeside view of Plan 9 before wave attack

Photo 56. End view of Plan 9 after wave attack

Photo 57. Lakeside view of Plan 9 after wave attack

Photo 58. Harbor-side view of Plan 9 after wave attack

Photo 59. End view of Plan 10 before wave attack

Photo 60. End view of Plan 10 after wave attack

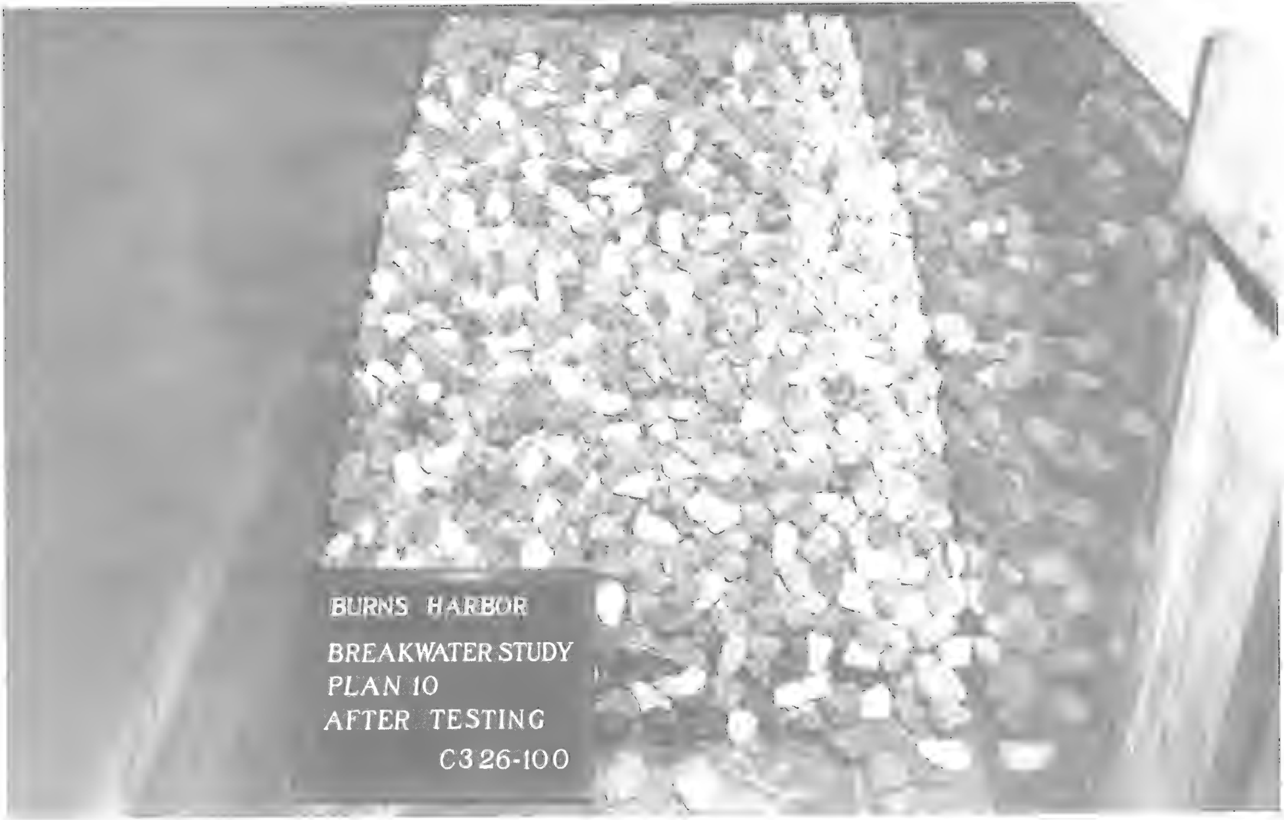

Photo 61. Lakeside view of Plan 10 after wave attack

Photo 62. Harbor-side view of Plan 10 after wave attack

Photo 63. End view of Plan 10A before wave attack

Photo 64. End view of Plan 10A after wave attack

Photo 65. Lakeside view of Plan 10A after wave attack

Photo 66. Close-up view of damage incurred by 5-ton stone in the vicinity of the swl

Photo 67. Harbor-side view of Plan 10A after wave attack

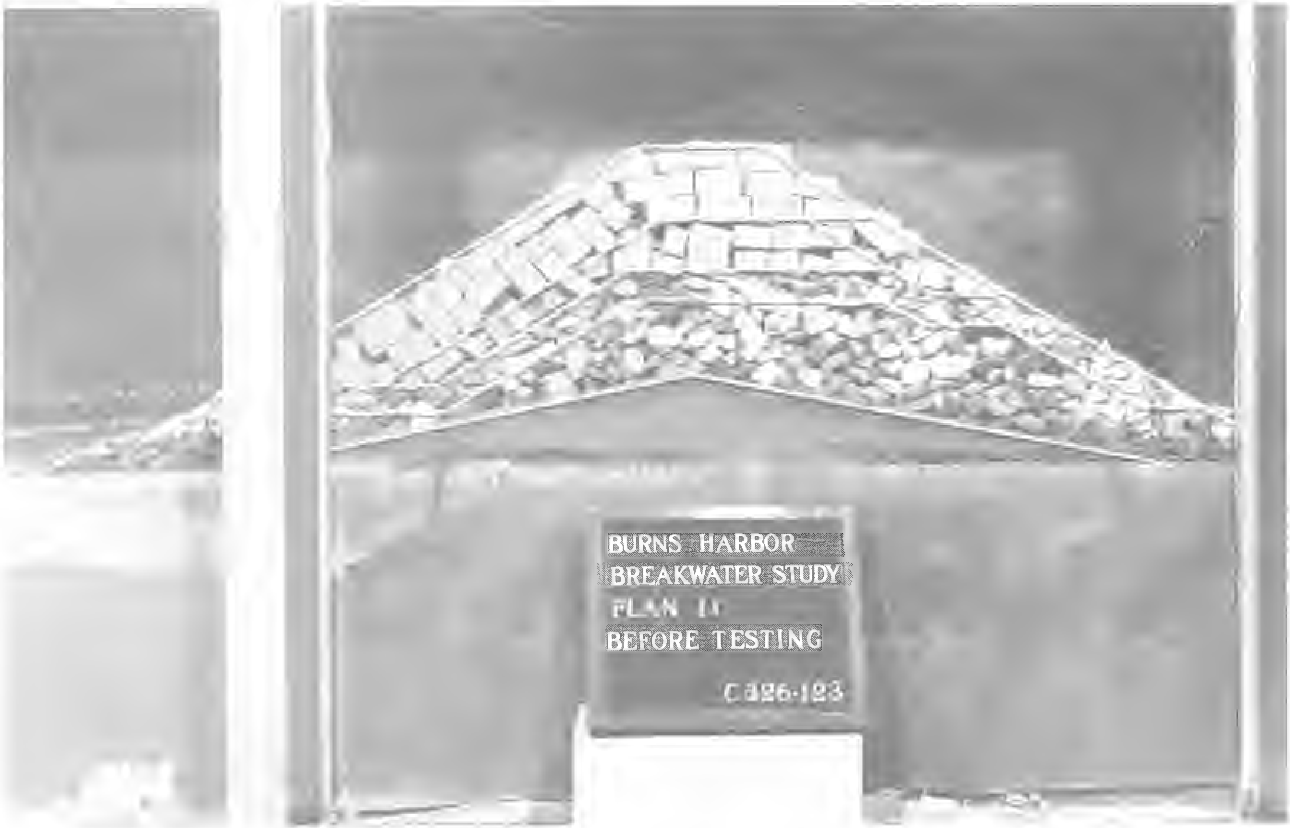

Photo 68. End view of Plan 11 before wave attack

Photo 69. Lakeside view of Plan 11 before wave attack

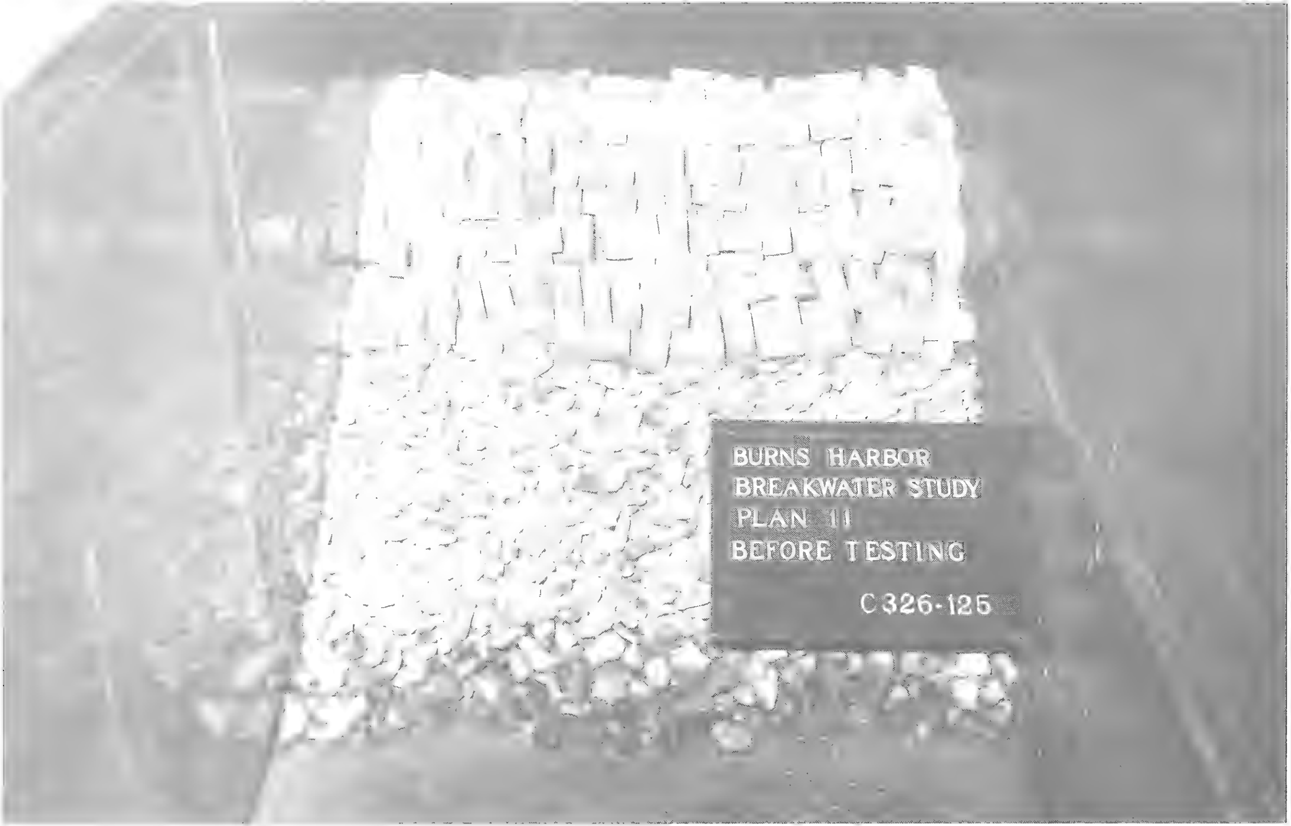

Photo 70. Harbor-side view of Plan 11 before wave attack

Photo 71. End view of Plan 11 after wave attack

Photo 72. Lakeside view of Plan 11 after wave attack

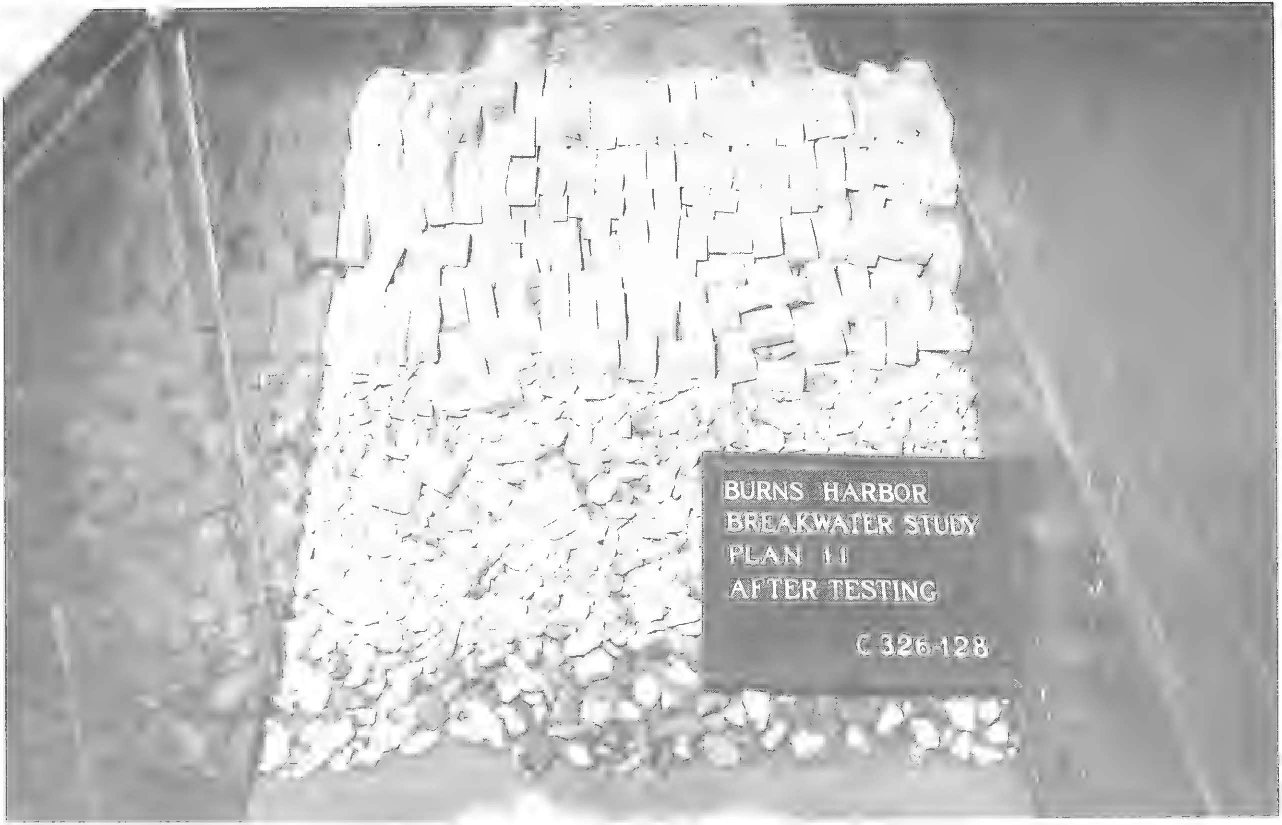

Photo 73. Harbor-side view of Plan 11 after wave attack

Appendix A Notation

H_{mo}	Zero-moment wave height, ft
T_p	Wave period of peak energy density of spectrum, sec
W	Weight, lb
$\cot \alpha$	Reciprocal of breakwater slope
γ_a	Specific weight of armor unit, pcf
γ_w	Specific weight of water, pcf
S_a	Specific gravity of an individual armor unit relative to the water in which it is placed, $S_a = \gamma_a/\gamma_w$
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)
K	Stability coefficient

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13. ABSTRACT *(Maximum 200 words)*

A two-dimensional model study of the damaged Burns Waterway Harbor breakwater was conducted. The 1:36-scale undistorted flume tests were used to evaluate various repair options that included placing a submerged breakwater lakeward of the existing breakwater, attaching a berm breakwater to the lakeside of the structure, the addition of an 18-ton angular stone overlay, and reworking the existing stone into special placement at the crest.

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

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