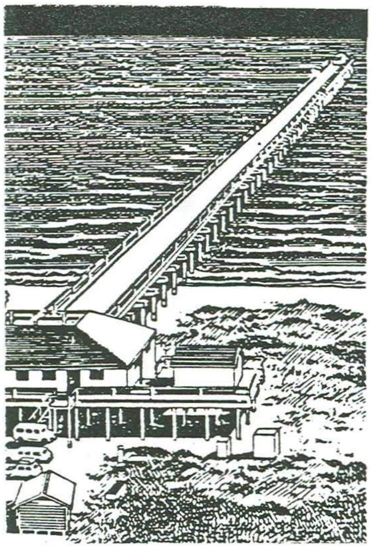
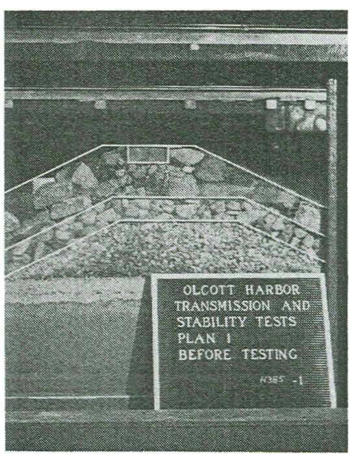
TECHNICAL REPORT CERC-91-5

RUBBLE-MOUND BREAKWATER WAVE-ATTENUATION AND STABILITY TESTS, OLCOTT HARBOR, NEW YORK

Coastal Model Investigation





by

Robert D. Carver

Coastal Engineering Research Center

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers 3909
Halls Ferry Road, Vicksburg, Mississippi 39180-6199

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13. ABSTRACT *(Maximum 200 words)*

A 1 :20-scale experimental model investigation was conducted to investigate the wave transmission response of a breakwater proposed for Olcott Harbor, New York. A check of the structure's stability showed the proposed section to be conservatively stable. Therefore, an alternate plan also was investigated in an attempt to reduce construction costs for the breakwater.

Based on model tests results, it. was concluded that:

a. Both plans tested are stable designs for the maximum wave heights that can be expected to occur (6- to lO-sec waves at still-water levels of +4.3 and +5.1 ft low,.water datum).

b. Maximum transmitted wave heights were 0.9 and 1.5 ft for Plans 1 and lA, respectively.

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Contents

Preface

The model investigation described herein was requested by the US Army Engineer District, Buffalo (NCB), in a letter to the US Army En;. gineer Waterways Experiment Station (WES) dated 5 June 1990. Funding authorization was granted by NCB in Intra-Army Order No. NCB-IA-9027EJ, dated 5 June 1990.

The study was conducted by personnel of the Coastal Engineering Research Center (CERC), WES, under the general direc"tion of Dr. James R. Houston, Chief, CERC, and Mr. Charles C. Calhoun, Jr., Assistant Chief, CERC. Direct guidance was provided by Messrs. C. E. Chatham, Chief, Wave Dynamics Division (WDD), and D. Donald Davidson, Chief, Wave Research Branch (WRB), WDD. Tests were conducted by Ms. Brenda J. Wright and Messrs. Willie G. Dubose and C. Ray Herrington, Engineering Technicians, under the direction of Mr. Robert D. Carver, Principal Investigator. This report was prepared by Mr. Carver.

COL Larry B. Fulton, EN, was the Commander and Director of WES during report publication. Dr. Robert W. Whalin was Technical Director.

Conversion Factors, Non-51 to 51 Units of Measurement

Non-51 units of measurement used in this report can be converted to 51 units as follows:

Multiply	By	To Obtain
cubic feet	0.02831685	cubic metres
degrees (angle)	0.01745329	radians
feet	0.3048	metres
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
square feet	0.09290304	square metres
tons (2,000 pounds, mass)	907.1847	kilograms

1 Introduction

Prototype

Olcott Harbor, New York, is situated at the mouth of Eighteen Mile Creek on the southern shore of Lake Ontario (Figures 1 and 2). Construction of an 850-ft[1]-long east pier and an 873-ft-Iong west pier was completed in 1918. The piers were originally of stone-filled timber crib construction with timber decks. In 1930, both piers were capped with stone and concrete. Repairs were made to the east pier in 1949 by driving rows of sheetpiling on each side of the pier, filling the voids with granular fill, and capping the structure with concrete. A similar repair procedure was performed on the west pier in 1963.

Presently, the entrance channel to the harbor area inside the mouth of the creek is safe only during calm weather. Proposed channel improvements will provide an urgently needed all weather entrance channel and additional berthing area for local craft. A feasibility study was prepared by the US Army Engineer District, Buffalo (NCB), and recommended construction of breakwater, jetty, and channel improvements.

Bottin and Acuff[2] conducted a three-dimensional physical model study to develop the optimum plan for harbor improvements to meet small boat harbor wave height criteria. Improvements were designed to protect against waves entering through the new proposed harbor entrance and from waves overtopping the breakwater sections.

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

2 Bottin, R. R., and Acuff, H. F. 1990. Olcott Harbor, New York, design for harbor improvements. Technical Report CERC-90-1. Vicksburg, MS: US Army Engineer Waterways Experiment Station.

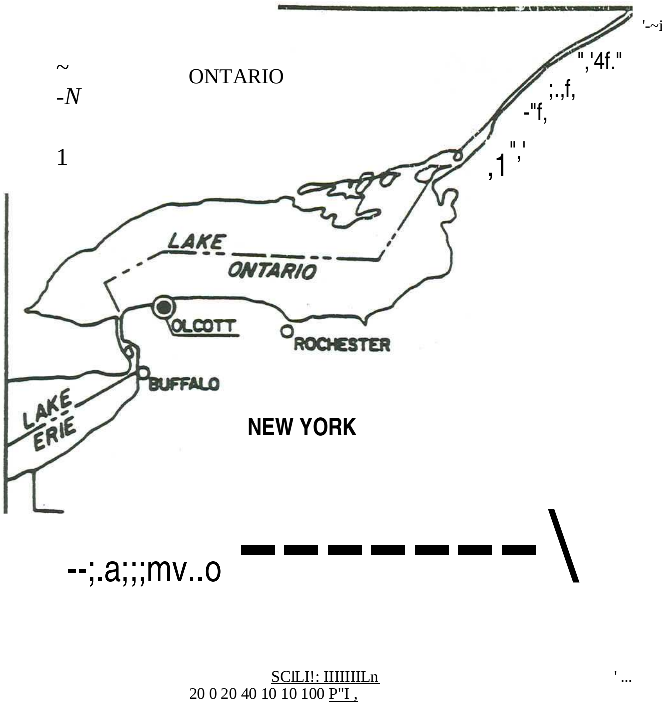


Figure 1. Project location

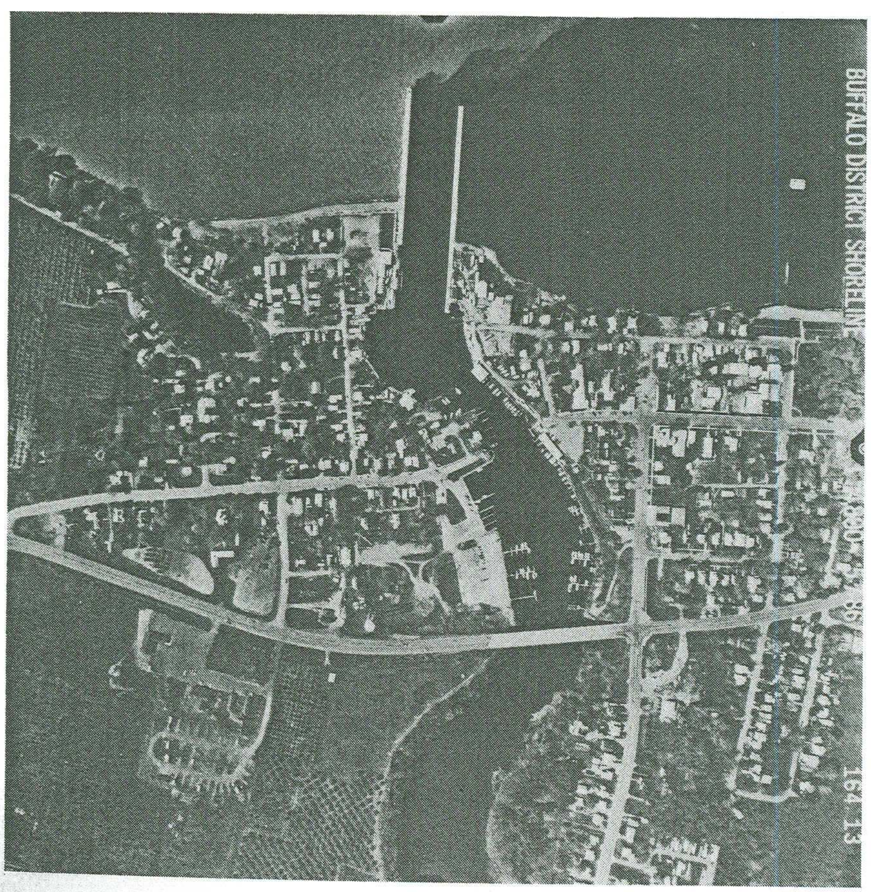


Figure 2. Aerial view of harbor

Purpose of Model Investigation

. The. initial objective of this study was to investigate the wave transmission response of the proposed breakwater. A secondary benefit of tests conducted herein, a check of the structure's stability, showed the proposed section to be conservatively stable. Therefore, an alternate plan also was investigated in an attempt to reduce construction costs for the breakwater.

2 The Model

Model-Prototype Scale Relationships

Tests were conducted at a geometrically undistorted scale of 1 :20, 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, elimination of wave transmission scale effects, preclusion of stability scale effects,[1] and capabilities of the available wave tank. Based on Froude's modellaw[2] and the linear scale of 1:20, the following model-prototype relations were derived. Dimensions are in terms of length (L)[3] and time (T).

Characteristic	Dimension	Model-Prototype Scale Relation
Length	L	L_r= 1:20
Area	L^2	=L_r=1:400
Volume	L^3	=L_r=1:8000
Time	T	T=L_r=1:4.47

where

r = ratio of model quantities to prototype quantities

A = area, ft 2

V = volume, ft 3

[1] Hudson, R. Y. 1975 (Jun). Reliability of rubble-mound breakwater stability models. Miscellaneous Paper H-75-5. Vicksburg, MS: US Army Engineer Waterways Experiment Station.

[2] Stevens, J. C. 1942. Hydraulic Models. Manuals of Engineering Practice No. 25. New York: American Society of Civil Engineers.

[3] For convenience, symbols and abbreviations are listed in the Notation (Appendix A).

The specific weight of water used in model tests was assumed to be the same as the prototype and equal to 62.4 pcf. However, specific weights of model breakwater construction materials were not the same as their prototype counterparts. These variables were related using the following transference equation:

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

where

W_a = weight of individual armor unit, lb

a = armor stone

m = model quantities

p = prototype quantities

Y_a = specific weight of armor unit, pcf

S_a = specific weight of individual armor unit relative to water in which breakwater is constructed

Test Equipment and **Facilities**

All tests were conducted in a concrete wave flume 3 ft wide and 150 ft long (Figure 3). 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. The test section was installed approximately 84.3 ft from the wave board.

Wave data were collected on electrical 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.

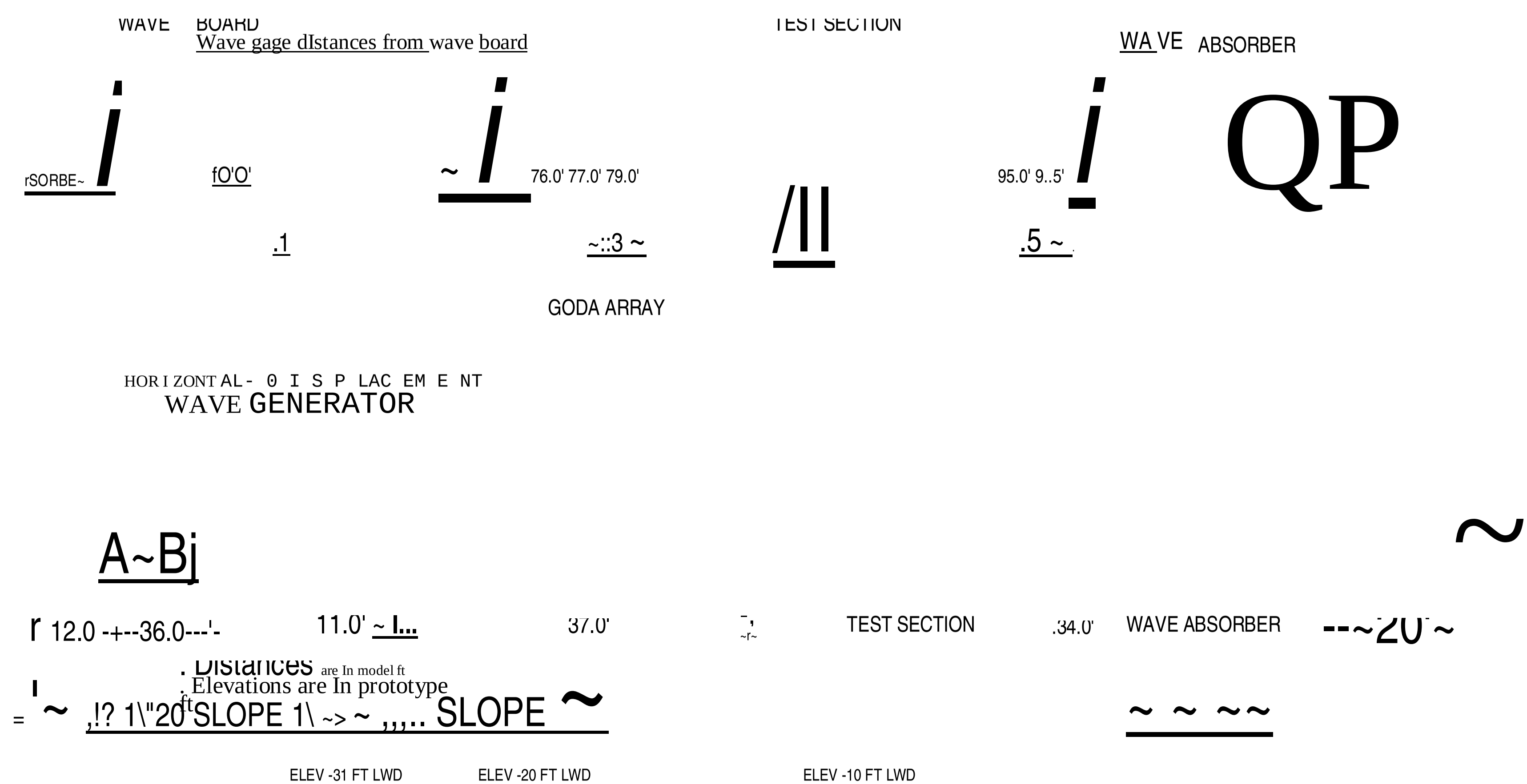


Figure 3. Wave flume cross section

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 then was 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; Le., 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.

Description of Plan 1

Plan 1 (Figure 4 and Photos 1 and 2) was constructed to a crown elevation of + 14 ft low-water datum (lwd) and used armor slopes of 1 V on 2H both lakeside and harbor side. A crown width of 16.2 ft, equivalent to two armor-stone diameters plus a 7-ft-wide walkway, was used. The lakeside slope was armored with two layers of 4- to II-ton stone, whereas the harbor-side slope used only one layer of 4- to 11-ton stone. In an effort to preclude toe slippage, the first row of armor stone at the toe of each slope used the largest size stone that was available in the specified armor stone range.

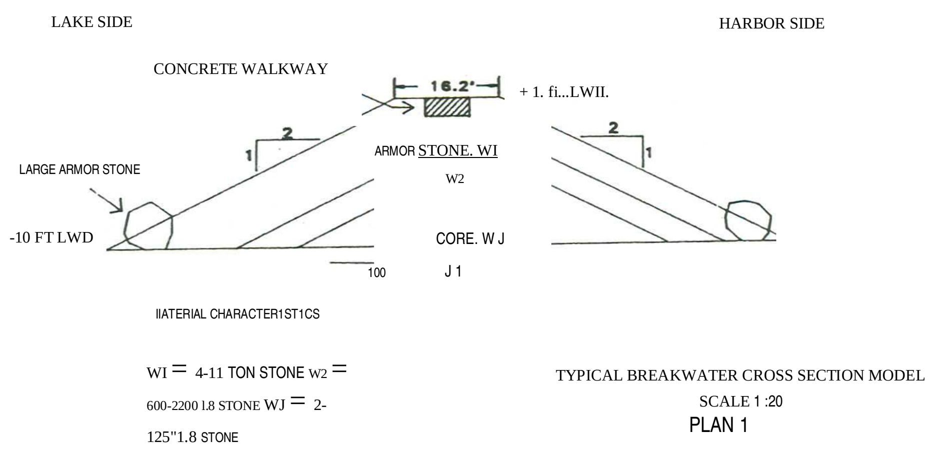


Figure 4. Cross section of Plan 1

Selection of Test Conditions

Based on siting of the breakwater in shallow water, tests were conducted with a Texel, Marsen, Arsloe (TMA) spectrum using peak wave periods (T) of 6, 7, 8, 9, and 10 sec. The wave basin was calibrated for wave heights (Hmo values) of 3 to 12 ft measured in front of the wave generator and in front of the structure. Transmitted wave heights were measured 100 and 150 ft shoreward of the breakwater. Goda and Suzuki's[1] method was used to resolve the incident and reflected spectra.

Test Results of Plan 1

Wave-attenuation test results are presented in Tables 1 and 2. Transmission coefficients (H/Hi) are based on incident wave heights measured at the wave generator because these wave heights relate to the percent time of occurrence wave tables used in the harbor model.[2] In general, the data show that (a) there is little difference between transmitted wave heights measured at 100 to 150 ft shoreward of the structure and (b) if

[1] Goda, Y., and Suzuki, Y. 1976. Estimation of incident and reflected waves in random wave experiments. In *Proceedings, 15th international conference on coastal engine"ering.* Honolulu, Hawaii.

[2] Bottin and Acuff, op. cit.

the incident wave height is held constant and the wave period is increased, transmitted wave heights increase. Maximum transmitted wave heights of 0.9 ft were observed at the 9- and 10-sec wave periods.

Observations of incident wave forms, made during the wave attenuation tests, showed that the most severe wave conditions which experimentally could be made to attack the section for the selected conditions occurred at the lO-sec peak period with maximum wave height of about 11 ft. Therefore, it was decided the stability response of the proposed section could be adequately evaluated by subjecting the structure to the following storm-surge hydrograph:

Step	Swl ft,lwd	Wave Period Tp. see	Wave Height Hmo, ft	Prototype Duration, hr
1	+4.3	10	11.1	4
2	+5.1	10	11.2	4
3	+4.3	10	11.1	4
Note: Swl = still-water level.				

As evidenced in Photos 3 and 4, Plan 1 exhibited an excellent stability response. Minor rocking of a few armor stones was observed; however, none were displaced.

Rationale and Description of Plan 1 A

Based on the excellent stability response of Plan 1, it was decided to investigate alternative schemes that might reduce the structure's cost without significantly affecting its functional performance. Some of the factors that govern material volumes and costs are eleva~ion and width of the crown, type and weight of armor, and slope on which the armor is placed. Based on discussions between NCB and US Army Engineer Waterways Experiment Station, it was decided that, in this particular study, the greatest cost savings with the least probable impact on functionality could probably be achieved by lowering the crown elevation.

Plan lA was the same as Plan 1 except a toe elevation of -11.5 ft lwd and crown elevation of +12.5 ft 1wd were used. This simulation was achieved by simply increasing the water depth 1.5 ft and assuming the new depth also represented an swl of +5.1 ft lwd. This approach reduced the freeboard by 1.5 ft and effectively achieved the same results (relative to transmission) as would have been achieved by lowering the model structure 1.5 ft.

Test Results of Plan 1A

Wave attenuation test results are presented in Table 3. These data show the same general trends as those observed with Plan 1. As would be expected with the reduced crown elevation, Plan IA showed increased wave transmission. A maximum transmitted wave of 1.5 ft was observed for the IO-sec wave period. Figure 5 shows average wave transmission coefficient for the 150-ftspacing versus peak wave period for Plans 1 and lA.

Plan lA was stable. Minor rocking of a few armor units was observed; however, none were displaced, and the integrity of the section was not jeopardized. Photos 5 and 6 show the structure at the conclusion of testing.

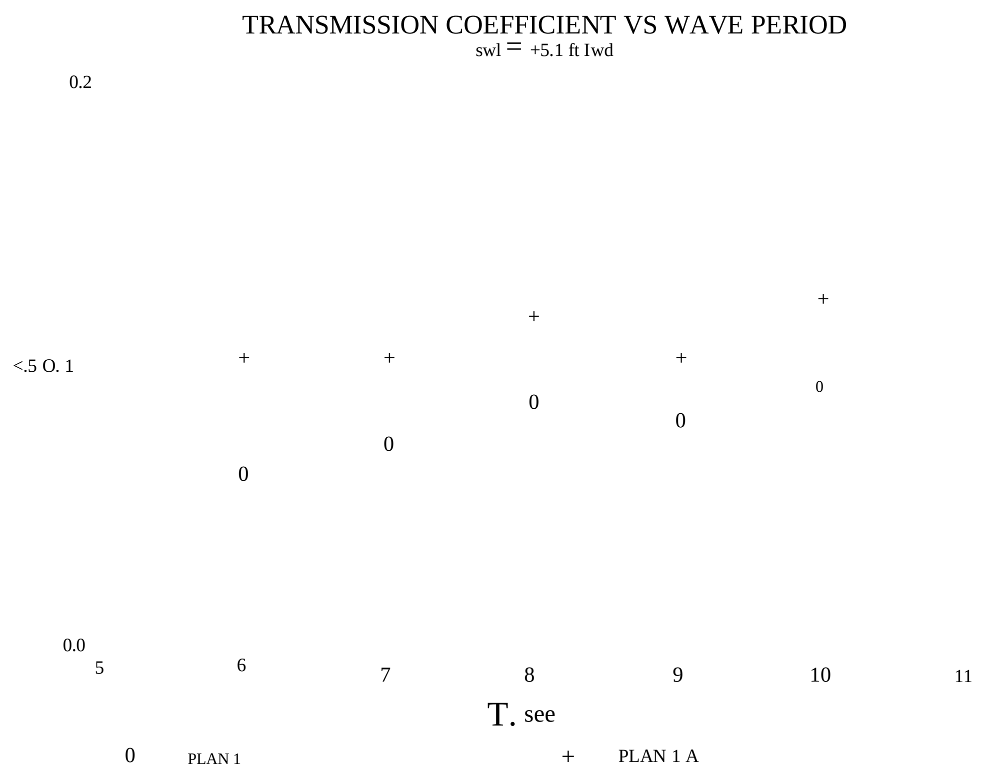


Figure 5. Coefficient of transmission (Ct) versus wave period (T p)

4 Conclusions

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

a. Plans 1 and 1A are stable designs for the maximum wave heights that can be expected to occur (6- to lO-sec waves at swl's of +4.3 and +5.1 ft lwd.)

b. Maximum transmitted wave heights were 0.9 and 1.5 ftfor Plans 1 and lA, respectively.

"

Table 1

Incident and Transmitted Wave Heights: Plan 1, swl = +4.3 ft Iwd

Wave Period sec	Incident Wave Height ft1	Incident Wave Height ft2	Transmitted Wave Height; ft 100 ft shoreward	Ct3	Transmitted Wave Height, ft 150 ft shoreward	Ct3
6.0	2.7	3.1	0.2	0.06	0.2	0.06
6.0	4.1	4.6	0.3	0.07	0.3	0.07
6.0	5.2	6.1	0.4	0.07	0.4	0.07
6.0	6.2	7.4	0.4	0.05	0.4	0.05
				-4 Ct = 0.06		Ct = 0.06
7.0	3.5	3.9	0.3	0.08	0.3	0.08
7.0	2.5	5.8	0.4	0.07	0.4	0.07
7.0	6.6	7.6	0.5	0.07	0.5	0.07
7.0	7.3	9.1	0.6	0.07	0.5	0.05
				Ct = 0.07		Ct = 0.07
8.0	3.6	3.9	0.4	0.10	0.4	0.10
8.0	5.4	5.9	0.5	0.08	0.5	0.08
8.0	6.8	7.7	0.5	0.06	0.6	0.08
8.0	7.8	9.4	0.6	0.06	0.7	0.07
				Ct = 0.08		Ct = 0.08
9.0	4.4	4.8	0.4	0.08	0.5	0.10
9.0	6.5	7.1	0.6	0.08	0.6	0.08
9.0	7.7	9.1	0.7	0.08	0.7	0.08
9.0	8.3	11.0	0.7	0.06	0.7	0.06
				Ct = 0.08		Ct = 0.08
10.0	4.5	4.6	0.5	0.11	0.5	0.11
10.0	6.5	7.0	0.6	0.09	0.6	0.09
10.0	7.9	9.4	0.7	0.07	0.7	0.07
10.0	8.3	11.1	0.8	0.07	0.8	0.07
				Ct = 0.09		Ct = 0.09

1 Measured at Goda array in front of structure.

2 Measured at wave generator.

3 Transmission coefficient (H/Hi) based on incident wave heights measured at the wave generator.

4Ct = average Ct.

I

i

I

Table 2
Incident and Transmitted Wave Heights: Plan 1, swl = +5.1 ft lwd

Wave Period see	Incident Wave Height ft1	Incident Wave Height ff	Transmitted Wave Height, ft 100 ft shoreward	13	Transmitted Wave Height, ft 150 ft shoreward	3
6.0	12.8	3.1	0.2	0.06	0.2	0.06
6.0	14.1	4.6	0.3	0.07	0.3	0.07
6.0	15.3	I 6.0	0.4	0.07	0.4	0.07
6.0	16.3	7.3	0.5	0.07	0.5	0.07
				-4 Ct = 0.07		Ct = 0.07
7.0	3.6	3.9	0.3	0.08	0.3	0.08
7.0	5.4	5.8	0.5	0.09	0.5	0.09
7.0	6.8	7.6	0.6	0.08	0.6	0.08
7.0	7.8	9.1	0.6	0.07	0.6	0.07
			I	Ct = 0.08		Ct = 0.08
8.0	3.6	3.8	10.4	I 0.11	0.4	I 0.11
8.0	5.6	5.8	0.5	0.09	0.5	0.09
8.0	7.0	7.8	0.7	0.09	0.7	0.09
8.0	7.9	9.3	0.8	0.09	0.7	0.08
				Ct = 0.09		Ct = 0.09
9.0	4.5	4.7	0.5	0.11	0.5	0.11
9.0	6.6	7.1	0.7	0.10	0.6	0.08
9.0	8.0	9.1	0.8	0.09	0.8	0.09
9.0	8.7	11.0	0.9	0.08	0.9	0.08
				Ct = 0.09		Ct = 0.09
10.0	4.7	4.8	0.6	0.13	0.5	0.10
10.0	6.8	7.1	0.7	0.10	.0.7	0.10
10.0	8.2	9.5	0.9	0.09	0.9	0.09
10.0	8.6	11.2	0.9	0.08	0.9	0.08
				Ct =0.10		Ct = 0.09

Measured at Goda array in front of structure.
Measured at wave generator.

- l"l)smission coefficient (H/Hj) based on incident wave heights measured at the wave generator.

Ct= average Ct.

Table 3

Incident and Transmitted Wave Heights: Plan 1 A, swl = +5.1 It Iwd

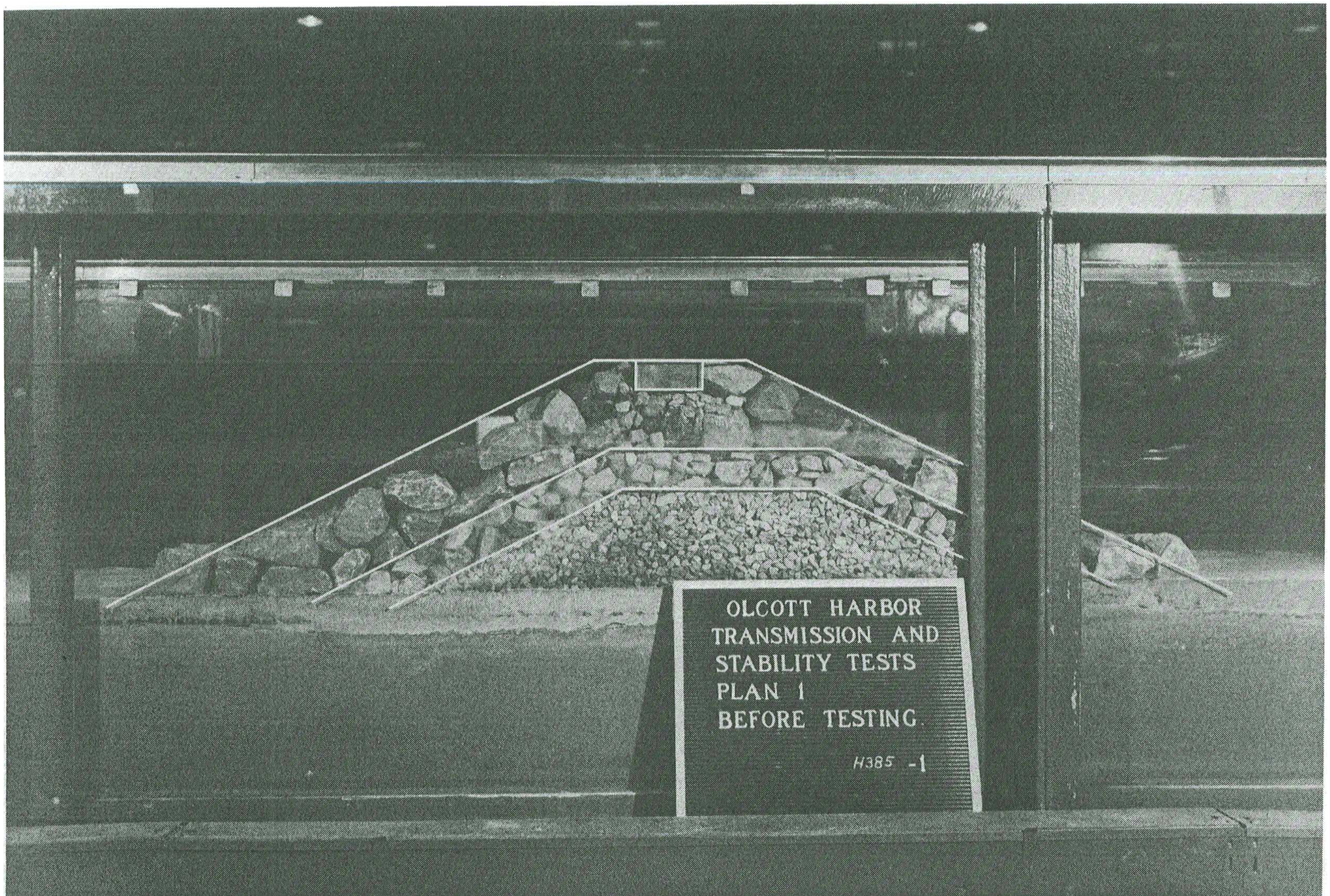
Wave Period' sec	Incident Wave Height ft1	Incident Wave Height W	Transmitted Wave Height, ft 100 ft shoreward	Cl3	Transmitted Wave Height, ft 150 ft shoreward	Cl3
6.0	2.8	3.3	0.4	0.12	0.4	0.12
6.0	4.2	4.8	0.5	0.10	0.5	0.10
6.0	5.5	6.2	0.6	0.10	0.6	0.10
6.0	6.6	7.5	0.7	0.09	0.7	0.09
				-4 C, = 0.10		C, = 0.10
7.0	3.6	4.3	0.6	0.14	0.6	0.14
7.0	5.3	6.4	0.8	0.13	0.'1	0.11
7.0	6.7	8.4	0.9	0.11	0.8	0.10
7.0	7.7	10.0	1.0	0.10	0.9	0.09
				C, = 0.12		C, = 0.11
8.0	3.8	4.1	0.6	0.15	0.6	0.15
8.0	5.7	6.2	0.8	0.13	0.8	0.13
8.0	7.3	8.1	0.9	0.11	0.9	0.11
8.0	8.3	10.0	1.1	0.11	1.0	0.10
				C, = 0.12		C, = 0.12
9.0	4.4	5.1	0.8	0.16	0.7	0.14
9.0	6.5	7.5	1.0	0.13	0.9	0.12
9.0	8.0	9.8	1.2	0.12	1.1	0.11
9.0	8.9	12.0	1.3	0.11	1.2	0.10
				C, = 0.13		C, = 0.12
10.0	4.6	5.2	0.8	0.15	0.7	0.13
10.0	6.7	7.9	1.1	0.14	1.0	0.13
10.0	8.2	10.3	1.3	0.13	1.3	0.13 ,
10.0	9.1	12.3	1.5	0.12	1.4	0.11 4
				C, = 0.14		Ct = 0.13

1 Measured at Goda array in front of structure.

2 Measured at wave genefator.

3 Transmission coefficient (H/Hj) based on incident wave heights measured at the wave generator.

4- C, = average C,.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TRANSMISSION AND
STABILITY TESTS
PLAN 1
BEFORE TESTING.
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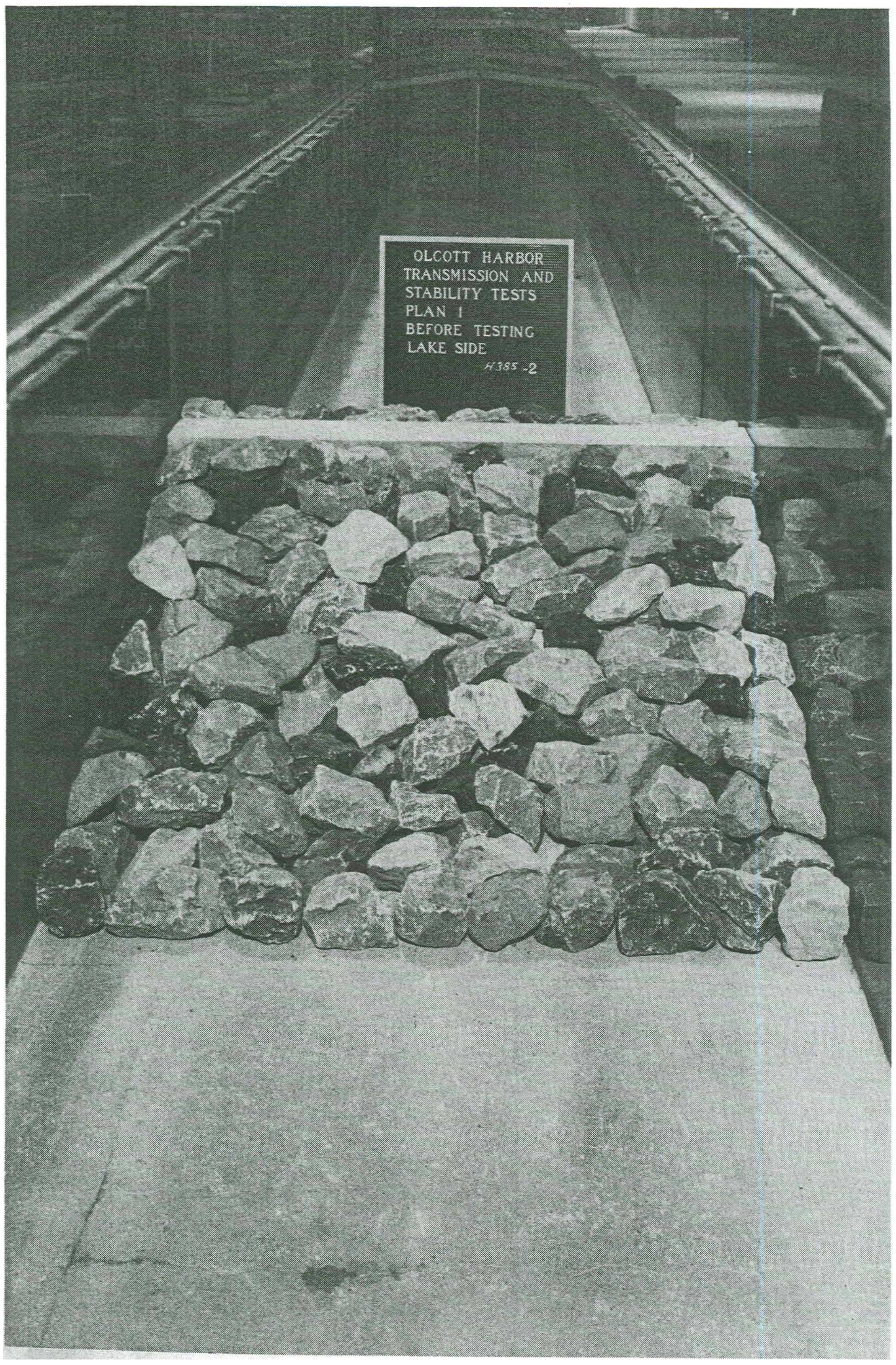


PhC)to 2. Lakeside view of Plan 1 before wave attack

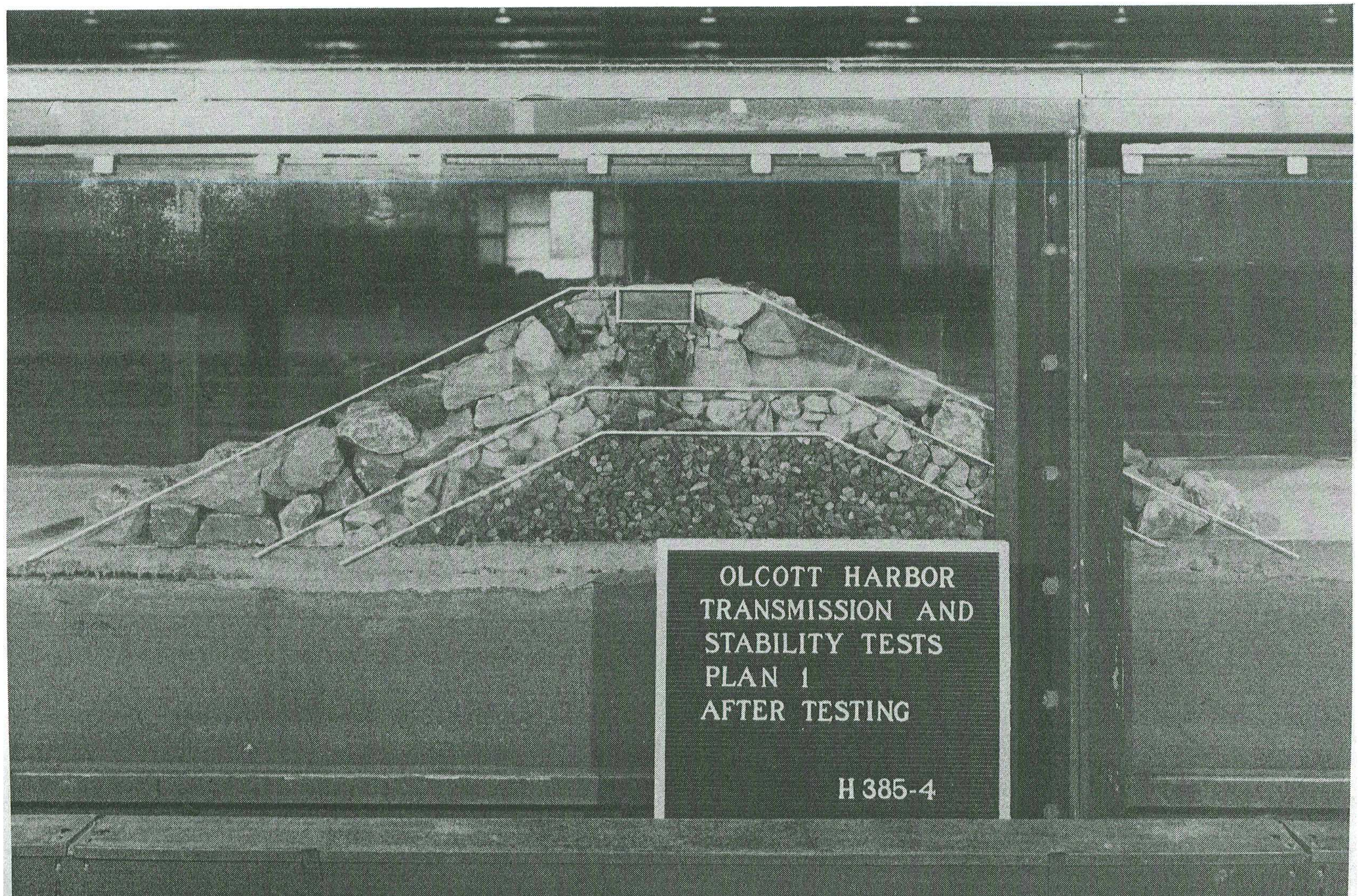


Photo 3. Side view of Plan 1 after wave attack

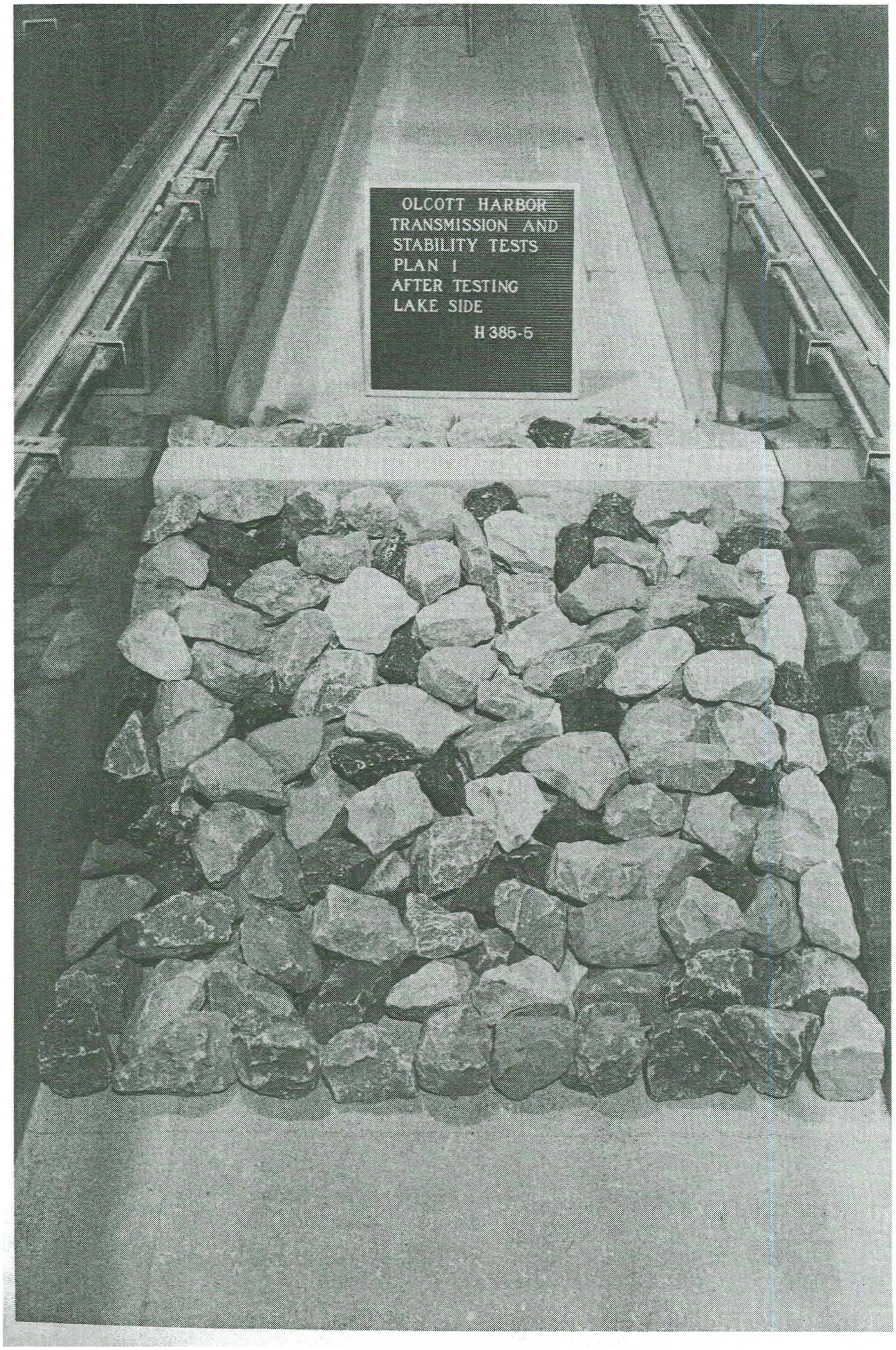
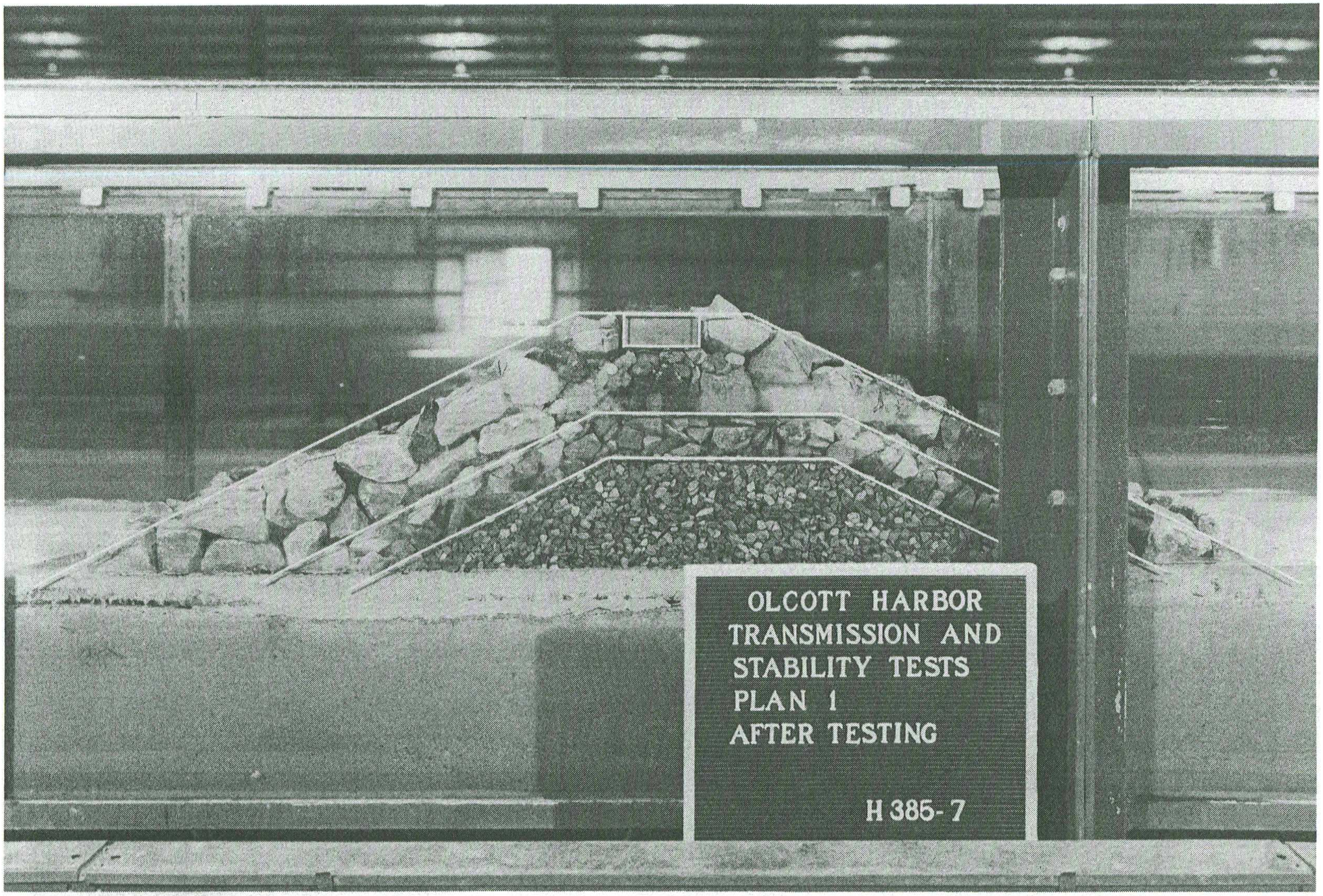


PHoto 4.l,..akeside view of Plan 1 after wave attack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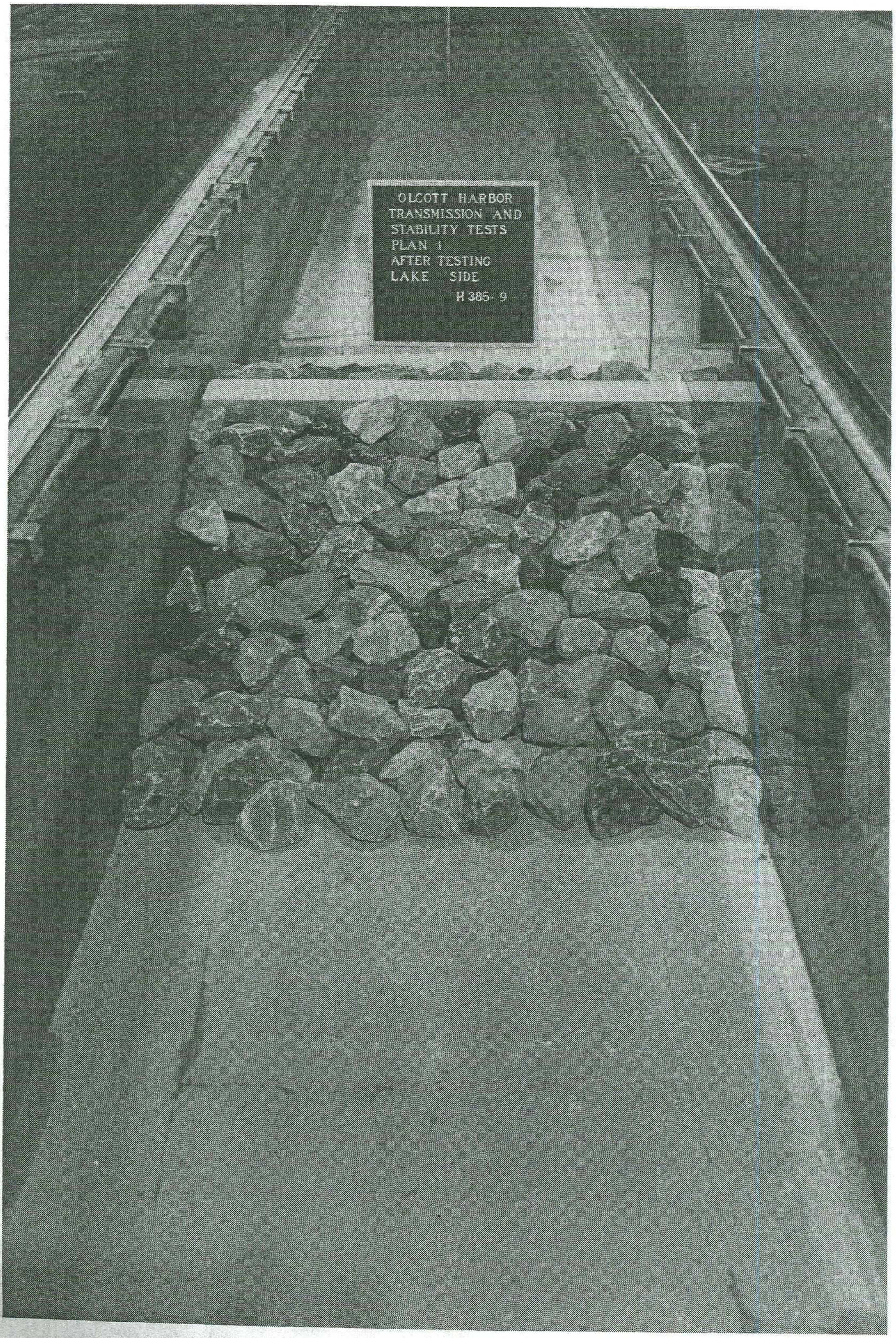


Photo 6. Lake-side view of Plan 1A after wave attack

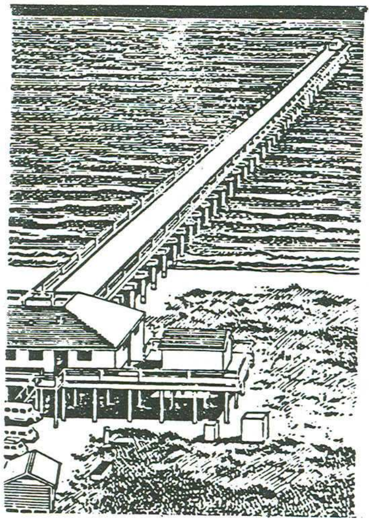
Appendix A
Notation

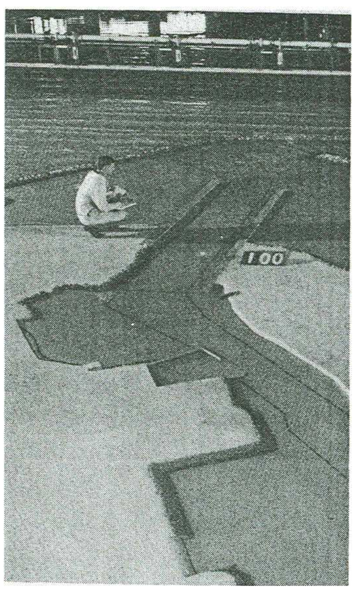
A	Area, £12
C_t	Transmission coefficient (H_t/H_i)
C_t	Average C_t
H_1	Incident wave height
H_t	Transmitted wave height
H_{mo}	Zero-moment wave height, ft
L	Length, linear scale, ft
S_a	Specific weight of an individual armor relative to the water in which the breakwater is constructed, Le., $S_a = Y_a/Y_w$
T	Time
T_p	Wave period of peak energy density of spectrum, sec
V	Volume, £13
W_a	Weight of individual armor, lb
Y_a	Specific weight of armor unit, pcf

Subscripts

a	Refers to armor stone
m	Refers to model quantities
p	Refers prototype quantities
r	Refers to ratio of model quantities to prototype quantities
w	Refers to water

US Army Corps of Engineers





TECHNICAL REPORT CERC-90-1

OLCOTT HARBOR, NEW YORK, DESIGN FOR HARBOR IMPROVEMENTS

Coastal Model Investigation

by

Robert R. Bottin, Jr., Hugh F. Acuff
Coastal Engineering Research Center

DEPARTMENT OF THE ARMY Waterways Experiment Station, Corps of Engineers 3909 Halls Ferry Road, Vicksburg, Mississippi 39180-6199

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19. ABSTRACT (Continue on reverse if necesury and identify by block number)

A 1:60-scale (undistorted) hydraulic model of Olcott Harbor, New York, was used to investigate wave, current, creek flow conditions, and sediment patterns for the existing harbor configuration and various improvement plans. The model reproduced approximately 3,300 and 3,600 ft of the New York shoreline on the east and west sides of the harbor, respectively, about 3,000 ft of the lower reaches of Eighteenmile Creek, and sufficient offshore bathymetry in Lake Ontario to permit generation of the required test waves. Proposed improvements consisted f the installation of rubble-mound breakwaters and channel dredging. An 80-ft-long unidirectional, spectral wave generator, an automated data acquisition and control system, and a crushed coal tracer material were utilized in model operation. It was concluded from test results that:

i!. Existing conditions are characterized by rough and turbulent wave conditions during periods of storm-wave attack. Wave heights up to 6.5 ft will occur in the existing entrance during boating season.

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19. ABSTRACT (Continued).

Q. The first basic harbor configuration (with the proposed mooring area east *of* the existing entrance, Plan 1 *of* 23 test plan variations) resulted in wave heights well within the established criteria (3.0 ft. in the proposed entrance and 1.0 ft in the proposed mooring area) for boating season wave conditions.

£. The following modifications may be made to the detached breakwaters *of* the first harbor configuration and still. achieve acceptable boating seaso~.wave conditions.

(1) .The east and west detachep breakwaters may be reduced in elevation from +16.2 and +15.3 ft, respectively, to el +14.5 ft.

(2) The length *of* the east breakwater may be reduced by 125 ft (removal from the shoreward end *of* the structure).

(3) The length *of* the west breakwater may be reduced by 350 ft (removal *of* 50 ft from the 1akeward end and 300 ft from the shoreward end *of* the structure).

g. Based on test results, the detached east and west breakwaters *of* the second basic harbor configuration were reduced to el +14.5 ft and the east breakwater length was reduced by 125 ft (paragraphs cl and c2). In addition, 50 ft may be removed from the shoreward end *of* the west breakwater (Plan 19) and acceptable wave conditions during boating season will be achieved for the second harbor configuration (mooring areas east and west *of* the existing entrance).

~. The openings between the attached and detached east and west breakwaters *of* the second basic harbor configuration will provide wave-induced current flow through the harbor and should enhance circulation.

f. The construction *of* the proposed harbor plan will have minimal impact on water surface elevations and creek current velocities in the lower reaches *of* Eighteenmile Creek.

g. The opening between the attached and detached west breakwaters (Plan 19) may result in minor shoaling in the mooring area in the western portion *of* the harbor for test waves from 313 and 334 deg, provided a sediment source is available. The installation *of* a sill between the structures (Plan 21), an extension *of* the attached breakwater (Plan 22), or a spur on the attached structure) Plan 23) will alleviate this shoaling.

h. Sediment placed between the existing groins east *of* the harbor for Plan 19 move easterly and westerly between the structures, but will remain relatively stable and not move from one cell to another. Accumulations may occur on the western sides *of* each cell, however, due to the predominance *of* the wave directions attacking the groin field.

PREFACE

A request for a model investigation of Olcott Harbor, New York, was initiated by the US Army Engineer District, Buffalo (NCB) , in a letter to the US Army Engineer Division, North Central, dated 11 July 1988. Authorization for the US Army Engineer Waterways Experiment Station (WES) to perform the study was subsequently granted by Headquarters, US Army Corps of Engineers (HQUSACE). Funds were authorized by NCB on 1 August 1988 and 13 April 1989.

Model testing was conducted at WES during the period February-July 1989 by personnel of the Wave Processes Branch (WPB), Wave Dynamics Division (WDD), Coastal Engineering Research Center (CERC), under the direction of Dr. James R. Houston, Chief, CERC; Mr. C. C. Calhoun, Jr., Assistant Chief, CERC; Mr. C. E. Chatham, Jr., Chief, WDD; and Mr. D. G. Outlaw, Chief, WPB. The tests were conducted by Messrs. H. F. Acuff, Civil Engineering Technician, under the supervision of Mr. R. R. Bottin, Jr., Project Manager, WPB. This report was prepared by Messrs. Bottin and Acuff.

Prior to the model investigation, Messrs. Outlaw, Acuff, and Bottin met with representatives of NCB and visited Olcott Harbor to inspect the prototype site. During the course of the investigation, liaison was maintained by means of conferences, telephone communications, and monthly progress reports.

The following personnel visited WES to observe model operation and participate in conferences during the course of the study.

Mr. Glenn Drummond	Headquarters, US Army Corps of Engineers US
Mr. Charlie Johnson	Army Engineer Division, North Central
Mr. Ken Hallock	US Army Engineer District, Buffalo
Mr. Denton Clark	US Army Engineer District, Buffalo US
Mr. Wiener Cadet	Army Engineer District, Buffalo US Army
Mr. Pete Crawford	Engineer District, Buffalo US Army
Mr. Thomas Bender	Engineer District, Buffalo New York
The Honorable John Connolly Mr.	State Senate
Ivan Vamos	New York Parks and Recreation
Mr. Ted Belling	Niagara County Planning Wendel
Mr. Tony McKenna	Engineers
Mr. James Kramer, Sr.	Town of Newfane, New York Town of
Mr. Timothy Horanburg	Newfane, New York

COL Larry B. Fulton, EN, is Commander and Director of WES. Dr. Robert W. Whalin is Technical Director.

CONTENTS

CONVERSION FACTORS, NON-SI TO SI
(METRIC) UNITS OF MEASUREMENT

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

Multiuly	~	To Obtain
acres	4,046.856	square metres
cubic feet	0.02831685	cubic metres
degrees (angle)	0.01745329	radians
feet	0.3048	metres
inches	25.4	mil1imetres
miles (US statute)	1. 609347	kilometres
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
square feet	0.09290304	square metres
square miles (US statute)	2.589988	square kilometres

OLCOTT HARBOR, NEW YORK, DESIGN FOR

HARBOR IMPROVEMENTS

Coastal Model Investigation

PART I: INTRODUCTION

The prototyoe

1. Olcott Harbor is located on the southern shore of Lake Ontario (Figure 1) at the mouth of Eighteenmi1e Creek. It is a small hamlet in Niagara County in the town of Newfane, NY, situated about 18 mi1es* east of the mouth of Niagara River. Eighteenmi1e Creek is about 14 miles long and drains an area of approximately 85 square miles. An active power dam, located about 2 miles upstream, regulates to some degree the flow conditions in the lower reaches of the creek. The dam also traps sediments, and, therefore, sedimentation in the stream below the dam is relatively low in comparison to other harbors maintained by the Corps of Engineers at the mouth of rivers and creeks (US Army Engineer District (USAED), Buffalo, 1978).

2. The existing Federal project for Olcott Harbor was authorized by the River and Harbor Act of 1913 and provides for parallel jetties at the creek mouth located 200 ft apart (Figure 2). The east and west jetties are 850 and 873 ft long with crest elevations (e1)** of 6 and 7 ft, respectively. They are concrete capped, vertical, steel sheet-pile structures. The project also includes a 12-ft-deep, 140-ft-wide entrance channel extending lakeward from the shoreward ends of the jetties to the -12 ft contour in Lake Ontario. A case history of the jetty structures at Olcott Harbor may be obtained from (Bottin 1988).

3. Olcott Harbor has been fully developed with boat docks and facilities on both banks of the creek. The harbor has a mooring capacity of 134 vessels and can accommodate boats ranging up to 68 ft in length. Major economic activity in Olcott is centered in commercial business enterprises,

A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 3.

All elevations (el) cited herein are in feet referred to low water datum (LWD). Low water datum on Lake Ontario is 242.8 ft above International Great Lakes Datum (IGLD) of 1955.

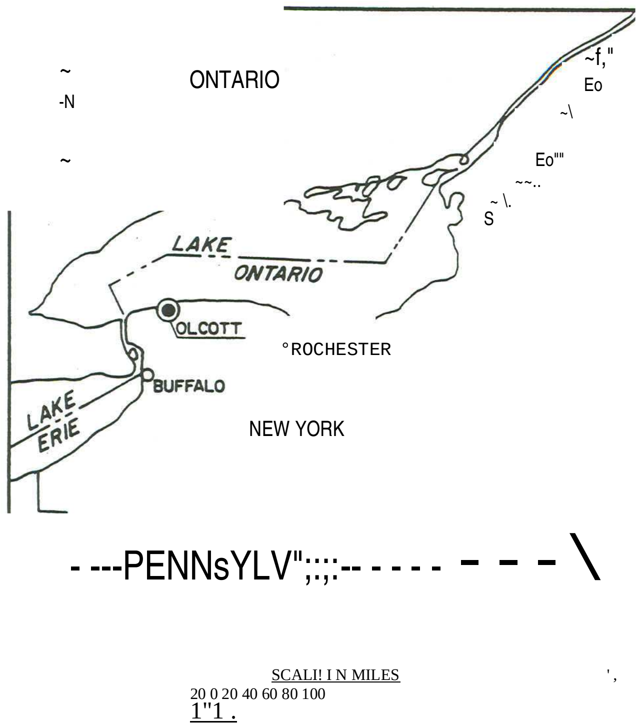


Figure 1. Project location

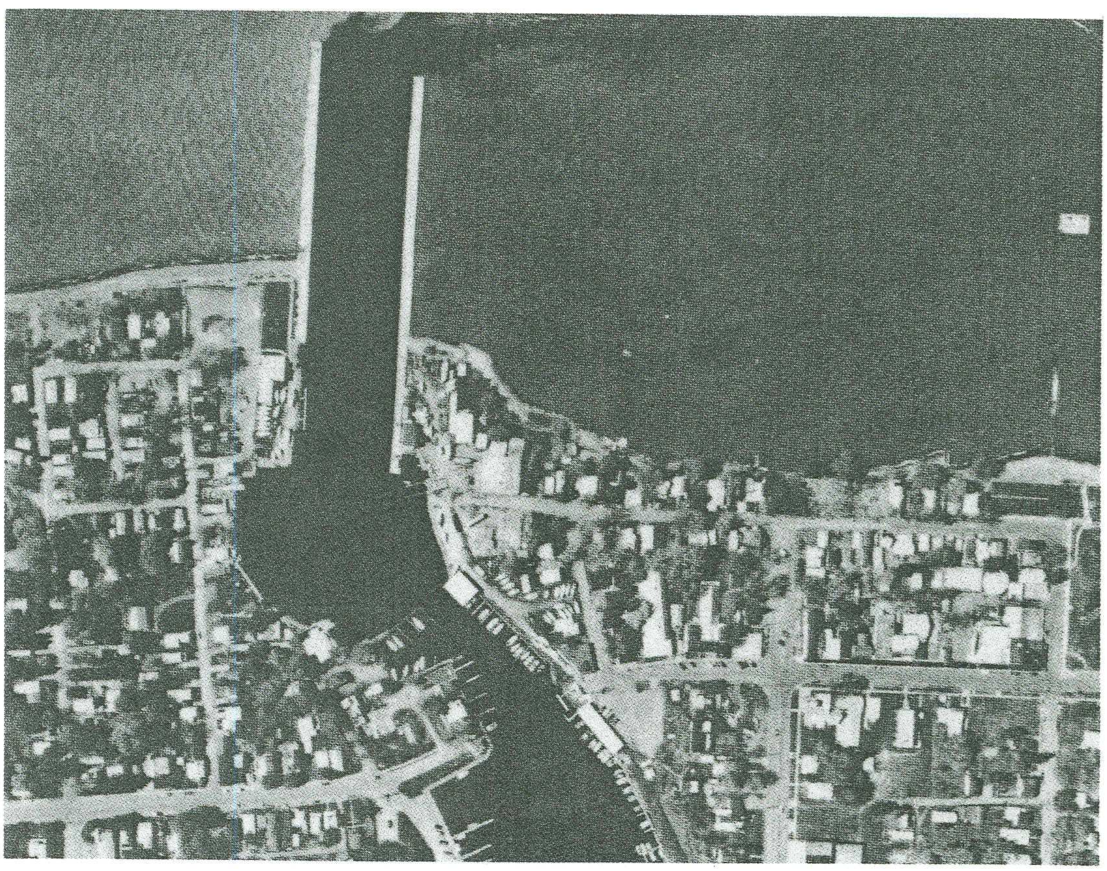


Figure 2. Aerial view of harbor

especially marine-related businesses. Krull Park, a 329-acre county park, is situated about 1,300 ft east of the harbor entrance. It provides recreational facilities for swimming and picnicking, and has six ball fields, a field house, wading pool, and parking area.

The Problem

4. During storms with winds from the northerly quadrant, waves entering between the jetties are reflected back into the entrance channel. This situation combined with waves overtopping the jetties, results in extremely rough conditions in the harbor entrance. Local residents report that boating in the entrance is frequently more difficult than in the open lake. This situation is particularly dangerous for strangers seeking refuge during storm wave conditions. Also, due to a crowded harbor, visiting craft have difficulty in finding mooring space.

5. The harbor is exposed to northerly storms and waves entering between the jetties, causing vessels to break loose from their moorings, and resulting in damages to themselves and other boats against which they strike. Harbor facilities also have been damaged. Damages from individual storms have reached over $20,000 (USAED, Buffalo, 1978).

6. Submerged remains of a bridge pier, midstream harbor, restricts free and easy navigation upstream. A shallow, poorly defined, irregular, natural channel with navigable widths limited to 10 ft in places also causes navigational difficulties to boat owners in the area. The development of additional berthing facilities on the creek banks upstream is restricted due to these navigational hazards. A regional analysis of boating needs on Lake Ontario and in Niagara County indicates an immediate need for more than 500 additional berths for permanently based vessels at Olcott Harbor and a demand for 300 additional moorings by 1996.

7. In summary, improvements are needed at Olcott Harbor to provide safe entrance channel conditions and protected mooring facilities during attack by storm waves. Harbor modifications would also provide a harbor-of-refuge for small boats caught in the open lake during storms and alleviate crowded conditions by providing additional berths to accommodate the high and growing demand for such facilities in the area.

Purpose of Model Study

8. At the request of the US Army Engineer District, Buffalo (NCB), a hydraulic model study was conducted by the US Army Engineer Waterways Experiment Station's (USAEWES) Coastal Engineering Research Center (CERC) to:

g. Study wave, current, creek flow, and shoaling conditions for the existing harbor configuration.

h. Determine if the proposed improvements would provide adequate wave, current, creek flow, and shoaling conditions in the harbor.

£. Develop remedial plans for the alleviation of undesirable conditions as found necessary.

g. Determine if suitable design modifications to the proposed plans could be made to significantly reduce construction costs without sacrificing adequate protection.

Wave Height Criteria

9. Completely reliable criteria have not yet been developed for ensuring satisfactory navigation and mooring conditions in small-craft harbors during attack by waves. For this study, however, NCB specified that for any of the various improvement plans to be acceptable, maximum wave heights were not to exceed 3 ft in the proposed entrance, or I ft in the proposed mooring areas for wave conditions occurring during the boating season (spring, summer, and fall).

PART II: THE MODEL

Design of Model

10. 'The Olcott Harbor model (Figure 3) was constructed to an undistorted linear scale of 1:60, model to prototype. Scale selection was based on such factors as:

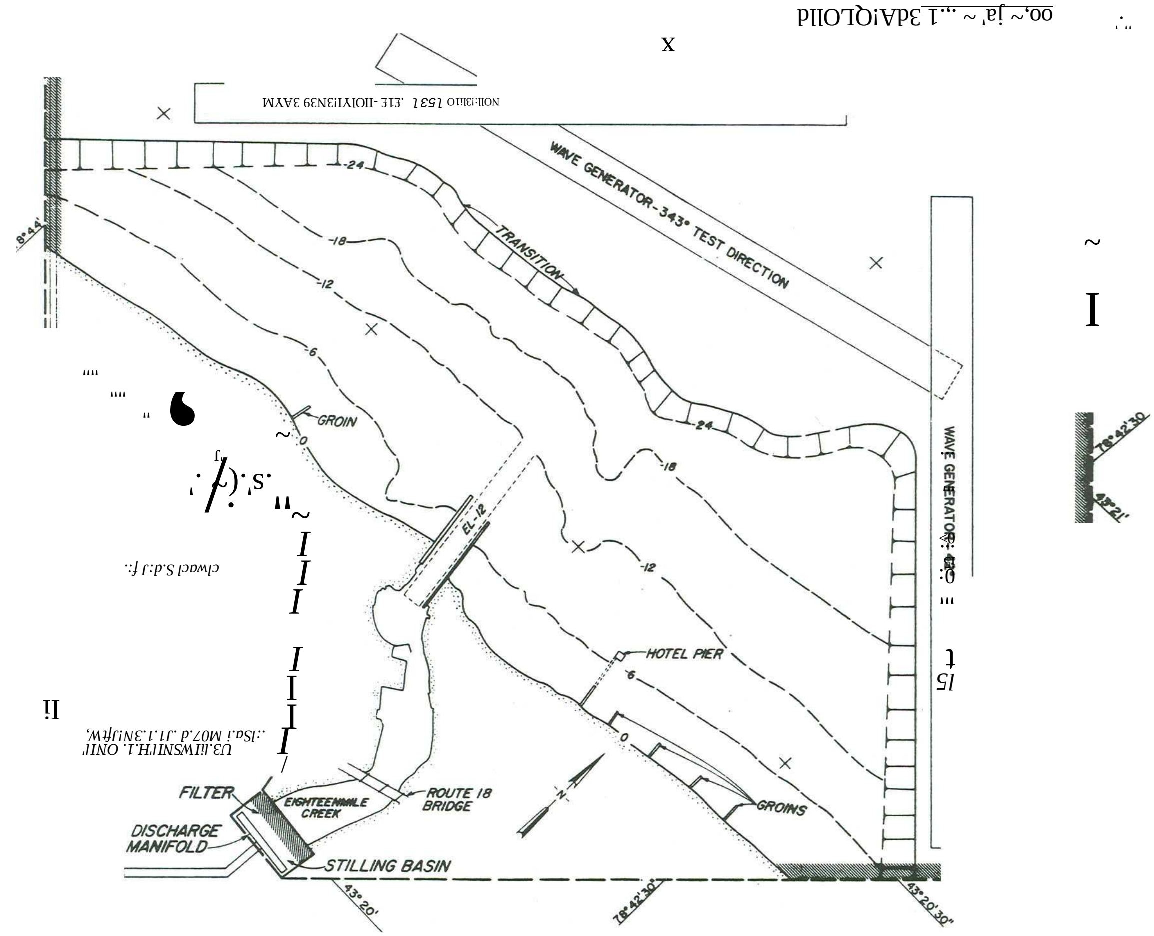
- g. Depth of water required in the model to prevent excessive bottom friction.
- Q Absolute size of model waves.
- £. Available shelter dimensions and area required for model construction.
- g. Efficiency of model operation.
- ~. Available wave generating and wave measuring equipment.
- f. Model construction costs.

A geometrically undistorted model was necessary to ensure accurate reproduction of short-period wave and current patterns. Following selection of the linear scale, the model was designed and operated in accordance with Froude's model law (Stevens ~942). The scale relations used for design and operation of the model were as follows:

Characteristic	Dimension*	Scale Relations Model: PrototyPe
Length	L	L_r = 1: 60
Area	L^2	$A_r = L_r^2$ = 1:3,600
Volume	L^3	$V_r = L_r^3$ = 1:216,000
Time	T	$T_r = L_r$ = 1:7.75
Velocity	L/T	$V_r = L_r$ = 1:7.75
Roughness (Manning's coefficient, n)	$L^{1/6}$	= $L_r^{1/6}$ = 1: 1. 979
Discharge	L^3/T	$Q_r = L_r^{5/2}$ = 1:27,885

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

11. Proposed improvement plans for Olcott Harbor included the use of rubble-mound breakwaters. Based on past experience, l:60-scale model structures should not create sufficient scale effects to warrant geometric


STILLING BASIN
DISCHARGE MANIFOLD
FILTER
EIGHTEENMILE CREEK
ROUTE 18 BRIDGE
GROINS
HOTEL PIER
EL-12
GROIN
TRANSITION
WAVE GENERATOR-343° TEST DIRECTION
WAVE GENERATOR
43°20'
78°42'30"
43°20'30"
78°42'30
43°21'
78°44'
-6
-12
-18
-24
0

distortion of stone sizes to ensure proper transmission and reflection of wave energy. Therefore, rock size selection was based on linear scale relations and a specific weight of 155 lb/ft3 for the prototype stone.

12. The values of Manning's roughness coefficient (n) ~sed in the design of the improved creek channel were calculated from water surface pro files of known discharges in the prototype. From these computations and experience, an n value of 0.030 was selected for use in the main creek chan nel. In addition, based on experience, n values of 0.060, in areas where existing depths were greater than 1 ft, and 0.080, in areas where existing depths were less than 1 ft, were selected for use in the creek. Therefore, based on previous WES investigations (Miller and Peterson 1953; and Cox 1973), the various model areas in Eighteenmile Creek were given finishes that would represent prototype n values of 0.030, 0.060, and 0.080.

13. Ideally, a quantitative, three-dimensional, movable-bed model investigation would best determine the impacts of the proposed structures with regard to the deposition of sediment in the vicinity of the harbor. However, this type of model investigation is difficult. and expensive to conduct, and each area in which such an investigation is contemplated must be carefully .ana1yzed. In view of the complexities involved in conducting movable-bed model studies and due to limited funds and time for the Olcott Harbor project, the model was molded in cement mortar (fixed-bed) at an undistorted scale of 1:60. For these reasons, a tracer material was obtained to qualitatively determine sediment patterns in the vicinity of the harbor for existing conditions and the most promising improvement plans.

Model and Appurtenances

14. The model reproduced approximately 7,000 ft of the New York shore line and included the existing harbor entrance and the lower 3,000 ft of Eighteenmile Creek. Underwater bathymetry also were reproduced in Lake Ontario to an offshore depth of -24 ft with a sloping transition to the wave generator pit elevation of -60 ft. The total area reproduced in the model was approximately 13,930 sq ft, representing about 1.8 square miles in the proto type. A general view of the model is shown in Figure 4. Vertical control for model construction was based on low water datum (LWD) , el 242.8 ft above mean water level at Father Point, Quebec (IGLD of 1955).

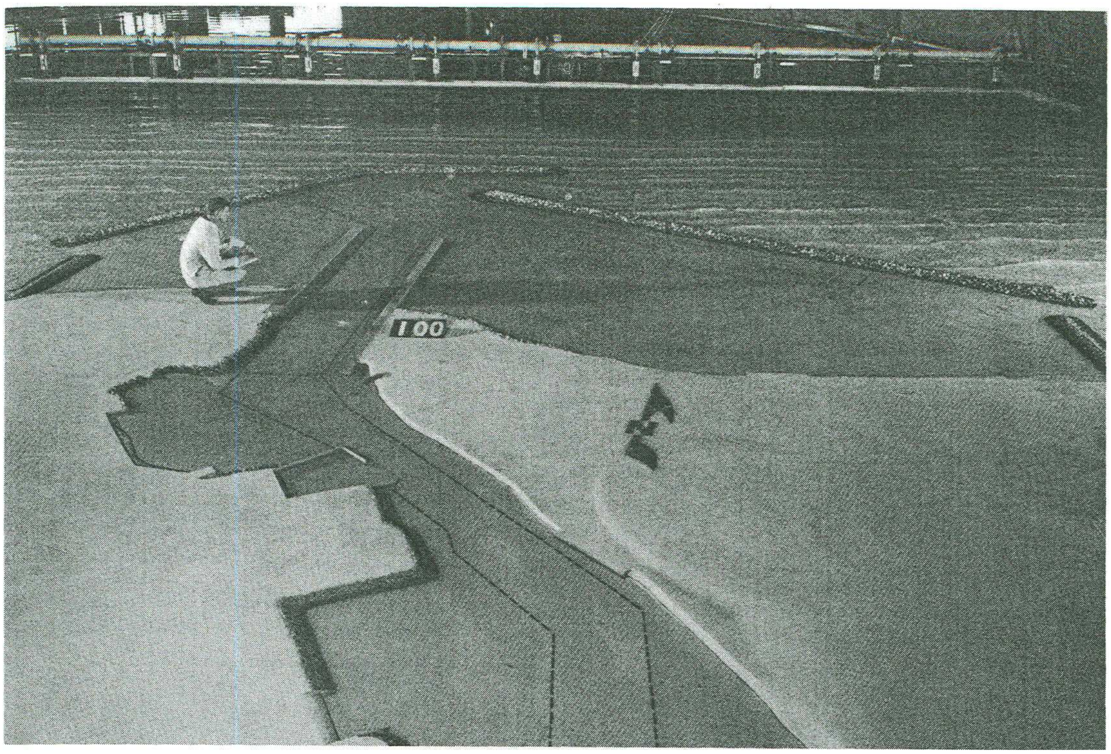


Figure 4. General view of model

Horizontal control was referenced to a local prototype grid system.

15. Model waves were generated by an 80-ft-long, unidirectional spectral wave generator with a trapezoidal-shaped, vertical-motion plunger. The electrohydraulic wave generator utilized a hydraulic power supply. The vertical motion of the plunger was controlled by a computer-generated command signal, and the movement of the plunger caused a periodic displacement of water that generated the required test waves. The wave generator also was mounted on retractable casters that enabled it to be positioned to generate waves from the required directions.

16. A water circulation system (Figure 3), consisting of a 6-in, perforated-pipe water-intake manifold, a 3-cfs pump, and a magnetic flow tube and transmitter, was used in the model to reproduce steady-state flows through the creek channel and harbor area that corresponded to selected prototype creek discharges. The magnitude of river currents were measured by timing the progress of weighted floats over known distances.

17. An Automated Data Acquisition and Control System (ADACS), designed and constructed at WES (Figure 5), was used to generate and transmit control

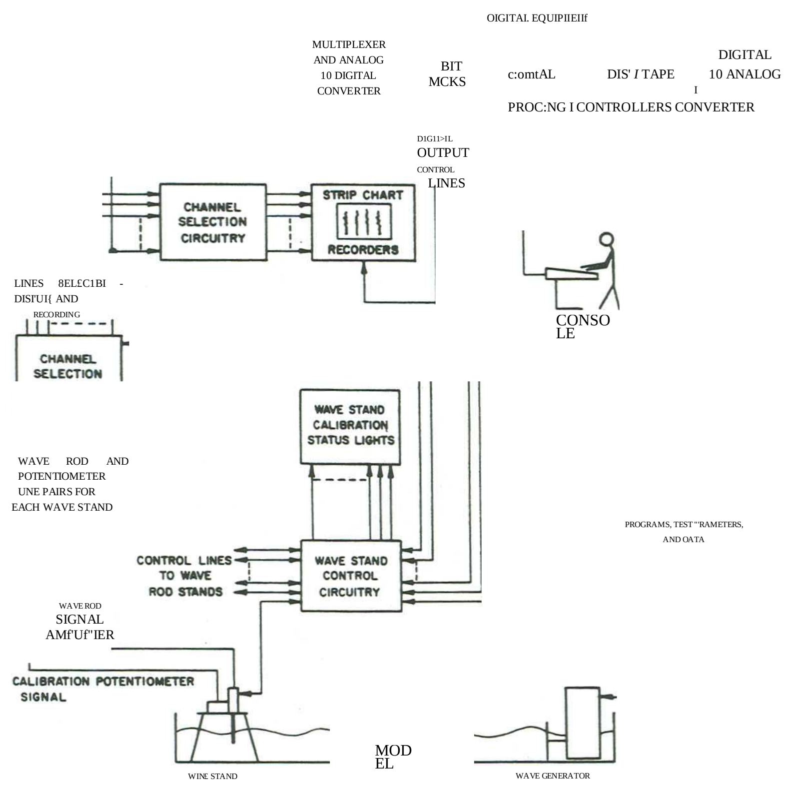


Figure 5. Automated Data Acquisition and Control System (ADACS)

signals, monitor wave generator feedback, and secure and analyze wave height data at selected locations in the model. Basically, through the use of a MICROVAX computer, ADACS recorded onto magnetic discs the electical output of parallel-wire, resistance-type wave gages that measured the change in water surface elevation with respect to time. The magnetic disc output of ADACS was then analyzed to obtain the wave height data.

18. A 2-ft (horizontal) solid layer of fiber wave absorber was placed around the inside perimeter of the model to dampen any wave energy that might otherwise be reflected from the model walls. In addition, guide vanes were placed along the wave generator sides in the flat pit area to ensure proper formation of the wave train incident to the model contours.

Selection of Tracer Material

19. As discussed in paragraph 13, a fixed-bed model was constructed and a tracer material selected to qualitatively determine the deposition of sediment in the vicinity of the harbor. The tracer was chosen in accordance with

the scaling relations of Noda (1972), indicating a relation or model law among the four basic scale ratios, i.e. the horizontal scale, A ; the vertical scale, ~ ; the sediment size ratio, ~D ; and the relative specific weight ratio, ~7 (Figure 6). These relations were determined experimentally using a wide range of wave conditions and bottom materials and are valid mainly for the breaker zone.

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"0 MATERIAL SIZE SCALE

Figure 6. Graphical representation of model law (Noda 1972)

20. Noda's scaling relations indicate that movable-bed models with scales in the vicinity of 1:60 (model to prototype) should be distorted (i.e., they should have different horizontal and vertical scales). Since the fixedbed model of Olcott Harbor was undistorted to allow accurate reproduction of short-period wave and current patterns, the following procedure was used to select a tracer material. Using the prototype sand characteristics (median diameter, Dso = 0.25 Mm, specific gravity = 2.65) and assuming the horizontal scale to be in similitude (i.e. 1:60), the median diameter for a given

vertical scale was then assumed to be in similitude and the tracer median diameter and horizontal scale was computed. This resulted in a range of tracer sizes for given specific gravities that could be used. Although several types of movable-bed tracer materials were available at WES, previous investigations (Giles and Chatham 1974, Bottin and Chatham 1975) indicated that crushed coal tracer more nearly represented the movement of prototype sand. Therefore, quantities of crushed coal (specific gravity = 1.30; median diameter, D_{50} = 0.72 mm) were selected for use as a tracer material throughout the model investigation.

PART III: TEST CONDITIONS AND PROCEDURES

Selection of Test Conditions

Still-water level

21. Still-water levels (swl) for harbor wave action models are selected so that the various wave-induced phenomena that are dependent on water depths are accurately reproduced in the model. These phenomena include the refraction of waves in the project area, the overtopping of harbor structures by the waves, the reflection of wave energy from various structures, and the transmission of wave energy through porous structures.

22. Water levels on the Great Lakes fluctuate from year to year and from month to month. Also, at any given location, the water level can vary from day to day and from hour to hour. Continuous records of the levels of the Great Lakes, tabulated since 1860, indicate that the usual pattern of seasonal variations of water levels consists of highs in the summer and lows in the late winter. For Lake Ontario, the higher levels usually occur in June and the lower levels in January. During the p~riod of record (1860-1952) the average level of Lake Ontario was +2.0 ft (Saville 1953). The highest I-month average level of +4.97 ft occurred in May 1870, and the lowest I-month average level of -1.32 ft occurred in November 1934. The seasonal variation in the mean monthly level of Lake Ontario usually ranges between 1 and 2 ft, with an average variation of 1.8 ft.

23. Seasonal and longer variations in the levels of the Great Lakes are caused by fluctuations in precipitation and other factors that affect the actual quantities of water in the lakes. Wind tides and seiches are relatively short-period fluctuations caused by the tractive force of wind blowing over the water surface and differential barometric pressures and are superimposed on the longer-period variations in lake level. Large short-period rises in local water level are associated with the most severe storms, generally occurring in the winter when lake levels are usually low; therefore, the probability that a high lake level and a large wind tide or seiche will occur simultaneously is relatively small.

24. Still-water levels of +2.8 and +4.0 ft were selected by NCB for use during model testing. The lower value (+2.8 ft) was used in conjunction with test waves that occur during the fall and winter seasons, and the higher value

(+4.0 ft) was used with test waves that occur during the spring and summer seasons. The design lake levels selected are equivalent to the 10-year frequency annual mean lake level for the particular season plus a short-period peak rise having a I-year recurrence interval. The +2.8- and +4.0-ft swl's also were used with flood flows through Eighteenmile Creek while obtaining water surface elevations and creek current magnitudes.

Factors influencing selection of test wave characteristics

25. In planning the testing program for a model investigation of harbor wave-action problems, it is necessary to select dimensions and directions for the test waves that will allow a realistic test of proposed improvement plans and an accurate evaluation of the elements of the various proposals. Surface wind waves are generated primarily by the interactions between tangential stresses of wind flowing over water, resonance between the water surface and atmospheric turbulence, and interactions between individual wave components. The height and period of the maximum wave that can be generated by a given storm depend on the wind speed, the length of time that wind of a given speed continues to blow, and the water distance (fetch) over which the wind blows. Selection of test wave conditions entails evaluation of such factors as:

a. The fetch and decay distances (the latter being the distance over which waves travel after leaving the generating area) for various directions from which waves can attack the problem area.

b. The frequency of occurrence and duration of storm winds from the different directions.

c. The alignment, size, and relative geographic position of the navigation entrance to the harbor.

d. The alignments, lengths, and locations of various reflecting surfaces inside the harbor.

e. The refraction of waves caused by differentials in depth in the area lakeward of the harbor, which may create either a concentration or a diffusion of wave energy at the harbor site.

Wave refraction

26. When wind waves move into water of gradually decreasing depth, transformations take place in all wave characteristics except wave period (to the first order of approximation). The most important transformations with respect to the selection of test wave characteristics are the changes in wave

height and direction of travel due to the phenomenon referred to as wave refraction.

27. When the refraction coefficient Kr is determined, it is multiplied by the shoaling coefficient Ks and gives a conversion factor for 'transfer of deepwater wave heights to shallow-water values. The shoaling coefficient, a function of wave length and water depth, can be obtained from the <u>Shore Protection Manual</u> (SPM) (USAEWES 1984). For this study, refractive diffractive coefficients based on the Regional Coastal Processes Wave Transformation Model (RCPWAVE) were prepared by NCB personnel and furnished to CERC.

28. Using the RCPWAVE transformation model (Ebersole 1985) and methods in the SPM, refraction and shoaling coefficients and shallow-water directions were obtained at Olcott for various wave periods from five deepwater wave directions (300 deg clockwise through 60 deg) and are presented in Table 1. Shallow-water wave directions and refraction coefficients represent an average of the values at approximately the location of the wave generator in the model. Shoaling coefficients were computed for a 60-ft water depth (plus the appropriate lake level) corresponding to the simulated depth at the model wave generator. The wave height adjustment factor, Kr x Ks, can be applied to any deepwater wave height to obtain the corresponding shallow-water value. Based on the refracted directions secured at the approximate locations of the wave generator in the model for each wave period, the following test directions (deepwater direction and corresponding shallow-water direction) were selected for use during model testing:

Deepwater Direction Bearing	Azimuth	Selected Shallow-Water Test Direction Bearing	Azimuth
N600 W,	3000	N47° W,	3130
N300 W,	3300	N26° W,	3340
North,	3600	N17° W,	3430
N300 E,	300	N24° E,	240
N600 E,	600	N42° E,	420

Prototype wave data and selection of test waves

29. Measured prototype wave data on which a comprehensive statistical analysis of wave conditions could be based were unavailable for the Olcott Harbor area. However, statistical deepwater wave hindcast data representative

of this area were obtained from Resio and Vincent (1976), shoreline grid point 3. This reference covers deepwater waves approaching from three angular sectors at the site (Figure 7). Table 2 lists by season and approach angle the 5-, 10-, 20-, 50-, and 100-year deepwater significant wave heights offshore at Olcott. Table 3 shows significant wave periods by angle class and wave height. The wave characteristics used during model testing were 20-year seasonal deepwater values converted to shallow-water values at the location of the wave generator through the use of refraction and shoaling coefficients shown in Table 1. These values were selected from Tables 2 and 3 and converted to shallow-water values by application of refraction and shoaling coefficients as shown in the following tabulation:

Deepwater Azimuth	Shallow-water Azimuth	Wave Period sec	Deepwater Wave Height ft	Shallow-water Wave Height ft	Season(s)*	swl
3000	3130	6.4	6.9	6.3	Sp,Su	+4.0
		7.2	9.2	7.6	F	+2.8
		7.4	9.8	8.0	W	+2.8
3300	3340	6.4	6.9	6.5	Sp,Su	+4.0
		7.2	9.2	8.4	F	+2.8
		7.4	9.8	8.8	W	+2.8
3600	3430	5.7	5.9	5.8	Sp	+4.0
		5.8	6.2	6.1	Su	+4.0
		7.0	10.8	9.9	F	+2.8
		7.4	12.1	11.0	W	+2.8
300	240	5.7	4.9	4.7	Sp	+4.0
		6.4	7.5	6.9	Su	+4.0
		6.0	5.9	5.5	F	+2.8
		6.9	8.9	7.9	W	+2.8
600	420	5.7	4.9	4.0	Sp	+4.0
		6.4	7.5	5.8	Su	+4.0
		6.0	5.9	4.7	F	+2.8
		6.9	8.9	6.4	W	+2.8

* Sp-Spring, Su-Summer, F-Fall, W-Winter

30. Unidirectional wave spectra for the selected test waves listed (based on JONSWAP parameters) were generated and used throughout the model investigation. Plots of typical wave spectra are shown in Figure 8. The dashed line represents the desired spectra while the solid line represents the spectra generated by the wave generator. A typical wave train time history also is shown in Figure 9.

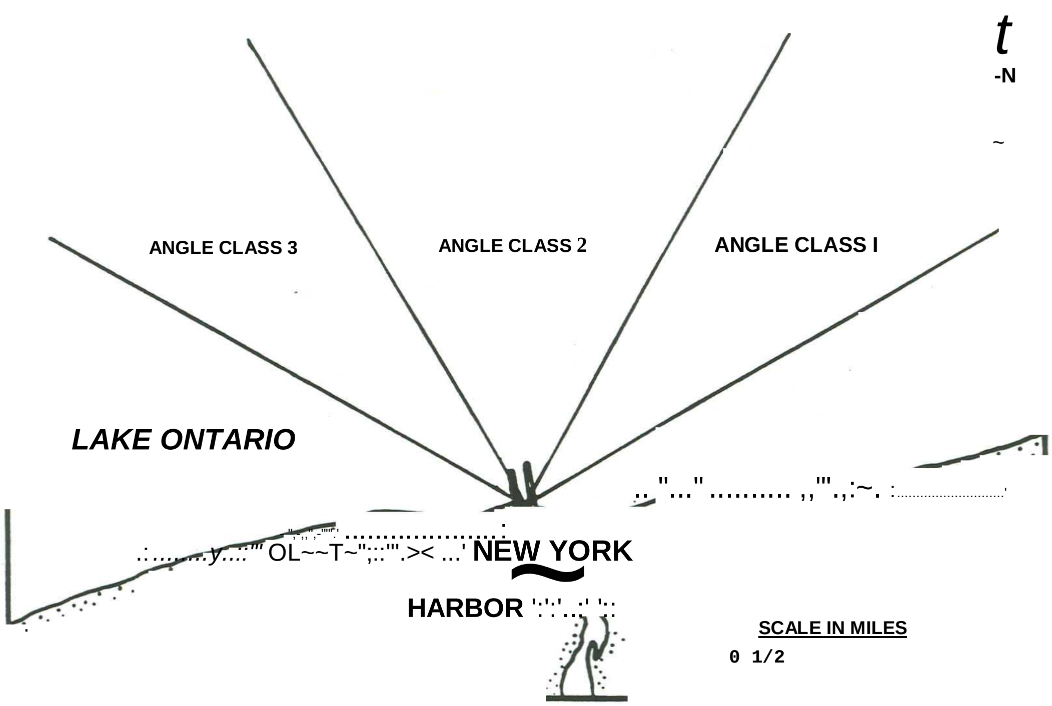

Figure 7. Wave hindcast angle classes

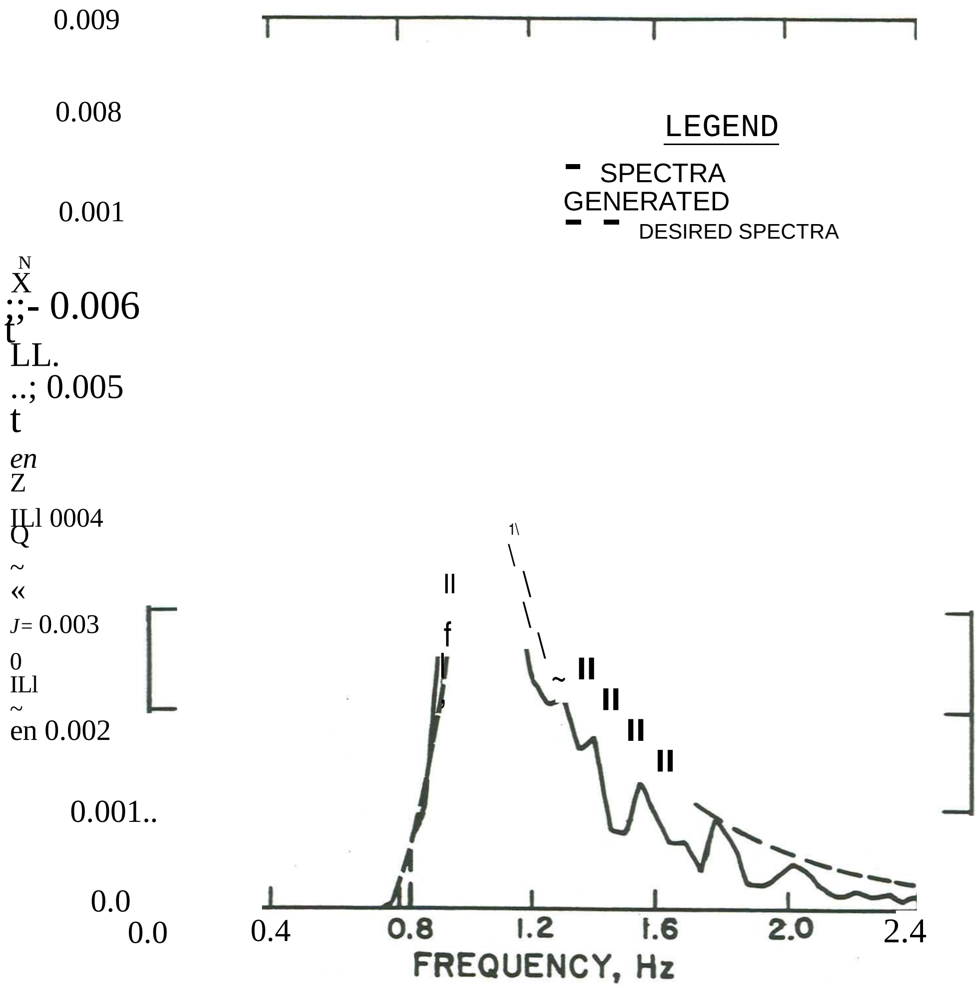

Figure 8. Typical wave-spectra plot, 7.4-sec, ll-ft test waves

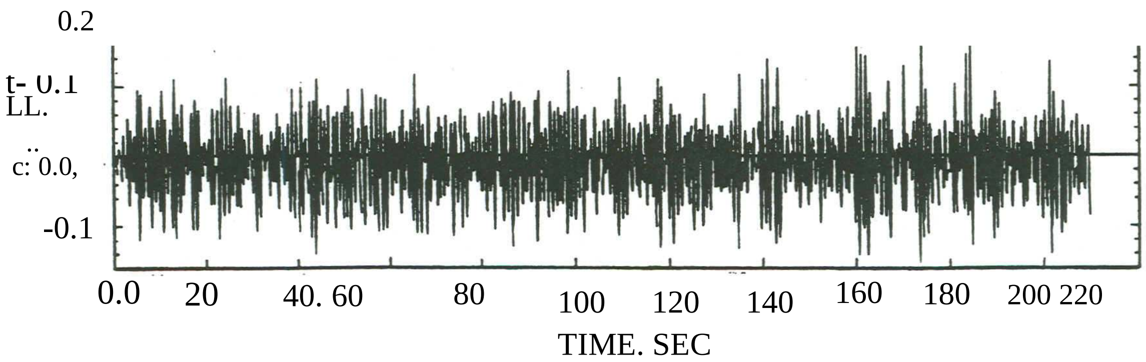

Figure 9. Typical wave train time history, 7.4-sec, ll-ft test waves

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Creek discharges

31. There are no continuous recording gages or crest-stage gages on ~ighteenmile Creek and, hence, no US Geological Survey records of peak discharges or mean daily discharges for the stream. In addition, few stream flow records are available from the Burt Dam located 2 miles upstream. Therefore, ~ba~ed on hydrologic records of other western New York streams, NCB personnel estimated discharge-frequency relationships and average seasonal discharges at ~HOlcott (USAED, Buffalo, 1988).

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N~ 32. Discharge-frequency relationships for Eighteenmile Creek are shown:

Return Interval years	Expected Discharge cfs
2	1,500
	2,300
5	2,900
10	3,700
25	4,400
50	5,100
100	

'Average seasonal discharges are as follows:

Season	Discharge cfs
Spring	180
Summer	80
Fall	150
Winter	110

~ ~ Discharges shown were used during model testing with wave conditions and swl's corresponding to the season tested (i.e. when fall waves and swl's were tested in the model, the fall discharge (150 cfs) was generated in Eighteenmile Creek). In addition, discharges up to 5,100 cfs (IOO-year discharge) were tested to determine current velocities and elevations in the creek.

Analysis of Model Data

33. Relative merits of the various plans tested were evaluated by:

~. Comparison of wave heights at selected locations in the model.

h. Comparison of wave-induced current patterns and magnitudes. Comparison

£. of sediment tracer movement and subsequent deposits.

Q. Comparison of water surface elevations and creek current velocities.

~. Visual observations and wave pattern photographs.

In the wave height data analysis, the average height of the highest one-third of the waves recorded at each gage location was computed. All wave heights then were adjusted to compensate for excessive model wave height attenuation due to viscous bottom friction by application of Keulegan's equation (Keulegan 1950). From this equation, reduction of wave heights in the model (relative to the prototype) can be calculated as a function of water depth, width of wave front, wave period, water viscosity, and distance of wave travel. Wave induced current magnitudes were obtained by timing the progress of an injected dye tracer relative to a thin graduated scale placed on the model floor.

PART IV: TESTS AND RESULTS

The Tests

Existing conditions

34. Prior to testing of the various improvement plans, comprehensive tests were conducted for existing conditions (Plate 1). Wave height data were obtained in the harbor and along the center line of the proposed breakwaters (for design wave information) for the selected test waves and directions listed in paragraph 29. Sediment tracer patterns, wave-induced current patterns and magnitudes, and wave pattern photographs also were .secured for representative test waves from the five test directions. In addition, water surface elevations and creek current velocities were obtained for various discharges for existing conditions.

Im~rovement plans

35. Wave height tests were conducted for 23 test plan variations for two basic harbor configurations. One configuration provided a mooring area to the east of the existing entrance, and one provided mooring areas on both the east and west sides of the existing entrance. Variations consisted of changes in the lengths and crest elevations of the various proposed breakwaters and/or the installation of a breakwater spur or sill. Wave pattern photos, wave-induced current patterns and magnitudes, sediment tracer patterns, creek current velocities, and water surface elevations were obtained for some of the improvement plans. Brief descriptions of the test plans are presented in the following subparagraphs; dimensional details are presented in Plates 2 through 7.

~. Plan 1 (Plate 2) consisted of a detached 1,110-ft-long dogleg west breakwqter, a 1,650-ft-long detached east breakwater, a 340-ft-long east spur breakwater, and channel dredging. The west breakwater had a crest elevation of +15.3 ft, and the east breakwater's crest elevation was +16.2 ft. Both structures had side slopes of 1V:1.5H and 1V:2H on the trunk and head sections, respectively. The spur breakwater had a crest elevation of +12.7 ft with side slopes of 1V:1.5H. A 150-ft width between the crests of the spur breakwater and the east structure was provided for circulation. A 75-ft-wide, 12-ft-deep irregular shaped entrance channel from deep water in Lake Ontario to the existing project channel between the piers also was included. In addition, a 100-ft-wide, 9-ft-deep access channel was dredged on the harbor side and parallel to the east breakwater; and a 9-ft-deep channel in Eighteenmile Creek extended upstream from the present project to the Route 18 bridge, a distance of about 1,500 ft.

Q. Plan 2 (Plate 3) included the elements of Plan 1 with 100 leg of ft of the shoreward the west breakwater removed. This l,OlO-ft-Iong resulted in a structure.

£. Plan 3 (Plate 3) entailed the elements of Plan 1 with 200 leg of ft of the shoreward the west breakwater removed. This. 910-ft-long resulted in a structure.

4. Plan 4 (Plate 3) involved the elements of Plan 1 with 300 ft of the shoreward leg of the west breakwater removed. This resulted in an 8l0-ft-long structure.

g. Plan 5 (Plate 3) entailed the elements of Plan 1 with 400 ft of the shoreward leg of the west breakwater removed. This resulted in a 710-ft-long structure.

f. Plan 6 (Plate 3) included the elements of Plan 1 with 500 ft of the shoreward leg of the west breakwater removed. This resulted in a 6l0-ft-long structure.

g. Plan 7 (Plate 3) involved the elements of Plan 1 with 350 ft of the shoreward leg of the west breakwater removed. This resulted in a 760-ft-Iong structure.

h. Plan 8 (Plate 4) consisted of the elements of Plan 1 and the 760-ft-long west breakwater of Plan 7, but the crest elevations of the east and west detached breakwaters were reduced to +14.5 ft.

1. Plan 9 (Plate 4) entailed the elements of Plan 8 with 50 ft of structure length added to the shoreward leg of the west breakwater. This resulted in an 8l0-ft-long structure.

i. Plan 10 (Plate 4) included the elements of Plan 9 with 100 ft of the shoreward end of the east breakwater remo~ed. This resulted in a l,550-ft-long structure.

k. Plan 11 (Plate 4) involved the elements of Plan 9 with 200 ft of the shoreward end of the east breakwater removed. This resulted in a l,450-ft-long structure.

1. Plan 12 (Plate 4) consisted of the elements of Plan 9 with 150 ft. of the shoreward end of the east breakwater removed. This resulted in a 'l,500-ft-long structure.

m. Plan 13 (Plate 4) included the elements of Plan 9 with 125 ft of the shoreward end of the east breakwater removed. This resulted in a l,525-ft-long structure.

n. Plan 14 (Plate 5) included the elements of Plan 13 with 150 ft of the lakeward leg of the west breakwater removed. This resulted in a 660-ft-long west breakwater.

Q. Plan 15 (Plate 5) involved the elements of Plan 13 with 100 ft of the lakeward leg of the west breakwater removed. .This resulted in a 7l0-ft-long detached west breakwater.

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R. Plan 16 (Plate 5) entailed the elements of Plan 13 with 50 ,ft of the lakeward leg of the west breakwater removed. This resulted in a 760-ft-long detached west breakwater. The east breakwater remained 1,525 ft in length.

g. Plan 17 (Plate 6) consisted of a detached 1,579-ft-long dogleg west breakwater, a 270-ft-long west spur breakwater, a 1,525-ft-long detached east breakwater, a 340-ft-long east spur breakwater, and channel dredging. The detached breakwaters had crest elevations of +14.5 ft and side slopes of lv:l.5h and lv:2h on the trunk and head sections, respectively. The spur breakwaters had crest elevations of +12.7 ft with side slopes of lv:l.5h. A 150-ft-wide, 12-ft-deep, irregular shaped entrance from deep water in Lake Ontario to the existing project channel between the jetties also was included. In addition, 75-ft-wide, 9-ft-deep access channels paralleled the harbor sides of the detached breakwaters; and a 9-ft-deep channel in Eighteenmile Creek extended upstream from th~ present project to the Route 18 bridge, a distance of approximately 1,500 ft.

~, Plan 18 (Plate 6) entailed the elements of Plan 17, but 100 ft of the shoreward leg of the west breakwater was removed. This resulted in a l',425-ft-long structure.

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.§.. Plan 19 (Plate 6) involved the elements of Plan 17, but 50 ft of the shoreward leg of the west breakwater was removed. This resulted in a 1,475-ft-long structure.

~. Plan 20 (Plate 6) included the elements of Plan 17, but a 100ft extension of the shoreward leg of the west breakwater was installed. This resulted in a 1,625-ft-long structure.

y. Plan 21 (Plate 7) consisted of the elements of Plan 19 with a stone sill connecting the attached and detached west breakwaters. The sill was 20 ft in width and had an elevation of -3 ft.

y. Plan 22 (Plate 7) entailed the elements of Plan 19 with a 50-ft-long lakeward extension of the attached west breakwater. The extension was angled toward the shoreward head of the detached west breakwater.

~. Plan 23 (Plate 7) involved the elements of Plan 17, but a 70ft-long spur was installed on the lakeward side of the attached west breakwater. The spur originated approximately 90 ft shoreward of the head of the attached breakwater.

Wave hei&ht tests and wave patterns

36. Wave heights and wave patterns for the various improvement plans were obtained for test waves from one or more of the directions listed in paragraph 29. Tests involving certain proposed improvement plans were limited to the most critical direction of wave approach. The more promising improvement plans were tested comprehensively for waves from all test directions.

Wave-gage locations for each improvement plan are shown in the referenced plates.

Wave-induced current pattern and magnitude tests

37. Wave-induced current patterns and magnitudes were determined at selected locations by timing the progress of an injected dye tracer relative to a graduated scale placed on the model floor. These tests were conducted for the most promising improvement plan (Plan 19) for representative test waves from the various test directions.

Sediment tracer tests

38. Sediment tracer tests were limited to the most promising improvement plans (Plans 16 and 19) as determined by results of wave height testing. Tracer material was introduced into the model east and west of the harbor entrance structures to represent sediment from those shorelines, respectively. In addition, tracer material was introduced between the groins east of the harbor entrance to determine its movement and deposition for various test waves from the five selected directions.

Creek current velocity and water surface elevation tests

39. Creek current velocity measurements and water-surface elevations for the most promising plan of improvement (Plan 19) were secured at various locations in the lower reaches of the creek for discharges of 1,500, 3,700 and 5,100 cfs using the +2.8- and +4.0-ft swl's. Stations, originating at the -12 ft contour in Lake Ontario and extending upstream in the creek, were located along the center line of the maintained channel and/or the center line of the proposed channel extension.

Test Results

40. In evaluating test results, the relative merits of various plans were based initially on an analysis of measured wave heights in the proposed entrance and harbor mooring areas. Further evaluation was based on the movement of tracer material and subsequent deposits, wave-induced current patterns and magnitudes, water surface elevations and/or river current velocities, and visual observations. Model wave heights (significant wave height or Hl/3)' water surface elevations, and river current velocities were tabulated to show

measured values at selected locations. Wave-induced current patterns and magnitudes were superimposed on wave pattern photographs for the corresponding plan and wave condition tested. The general movement of tracer material and subsequent deposits also were shown in photographs. Arrows were superimposed onto these photographs to depict sediment movement.

Existing conditions

41. Results of wave height tests conducted for existing conditions are presented in Table 4. Maximum wave heights obtained for boating season wave conditions (spring, summer, fall) were 6.5 ft in the existing entrance (Gage 8) for 7.2-sec, 8.4-ft test waves from 334 deg and 7.0-sec, 9.9-ft test waves from 343 deg; 5.8 ft between the existing jetties (Gage 9) for 7.2-sec, 8.4-ft test waves from 334 deg; 4.0 ft at the upstream limit of the existing channel (Gage 10) for 6-sec, 4.7-ft test waves from 42 deg; and 1.3 ft in the first bend in the creek (Gage 11) for 5.8-sec, 6.l-ft test waves from 343 deg. Considering all test waves, maximum wave heights were 9.9 ft along the center line of the proposed west breakwater (Gage 2) and 8.0 ft along the center line of the proposed east breakwater (Gage 4) both for 7.4-sec, ll-ft, test waves from 343 deg. Typical wave patterns for existing conditions are shown in Photos 1 through 10.

42. Wave-induced current patterns and magnitudes obtained for existing conditions for representative test waves and directions also are shown in Photos 1 through 10. Maximum velocities secured at various locations were as follows:

Location	Maximum Velocity. fps	Test Wave(s) Period	Test Wave(s) Height ft	Direction(s) deg	swl JL
Shoreline east of harbor entrance	2.4	7.4	11.0	343	+2.8
Area east of east jetty	3.2	6.9	6.4	42	+2.8
Area lakeward of entrance	4.8	7.4	8.8	334	+2.8
Area between jetties	0.7	7.4	8.8	334	+2.8
		7.4	11.0	343	+2.8
Area west of west jetty	2.2 3.5	6.9	6.4	42	+2.8
Shoreline west of harbor entrance		7.4	11.0	343	+2.8
		6.4	6.3	313	+4.0

In general, currents along shore moved from west to east for test waves from 313, 334, and 343 deg; and from east to west for test waves from 24 and 42 deg. Eddies occurred generally on both sides of the structures for most test waves. Both clockwise and counterclockwise eddies were observed depending on direction of wave approach.

43. The placement of tracer material in the model prior to testing is shown in Photos 11 and 12. The general movement of tracer material and subsequent deposits on each side of the harbor for existing conditions are shown in Photos 13 through 17. The tracer initially placed in the model was first subjected to test waves with the +2.8-ft swl, and then progressively, test waves for the +4.0-ft swl for each direction. For test waves from 313 deg, sediment west of the entrance migrated easterly adjacent to the west jetty; and material east of the entrance moved easterly to and over the remnants of the existing hotel pier and deposited along the shoreline and adjacent to the western most groin. Test waves from 334 deg resulted in tracer material moving toward the shore and easterly on the east side of the entrance, while the tracer on the west side of the entrance moved shoreward and slightly westerly due to wave and current patterns in the vicinity. Tracer tests for the 343-deg direction resulted in material on the west of the entrance moving shoreward and westerly; while sediment on the east of the entrance moved toward the shoreline and deposited. For test waves from 24 and 42 deg, tracer material on both sides of the harbor moved in a westerly direction.

44. The general movement of tracer material and deposits in the groin field east of the harbor entrance are shown in Photos 18 through 22. Sediment between the various groins moved shoreward and then either easterly or westerly depending on the incident wave direction. For all directions, the tracer material remained between the groins in which it was originally placed. It did not move around the heads of the groins nor was it washed over the groins from one cell to another.

45. Results of water surface elevation and depth-averaged creek current velocity measurements for existing conditions are shown in Table 5 for the +2.8- and +4.0-ft swl's. For the +2.8-ft swl, the maximum rise in water surface elevation in the creek ranged from 0.12 ft for the 1,500-cfs discharge to 0.24 ft for the 5,100-cfs discharge; and maximum velocities in the. creek ranged from 1.2 to 3.5 fps for the 1,500- and 5,100-cfs discharges, respectively. With the +4.0-ft swl, maximum rises in water surface elevation ranged

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from 0.12 to 0.18 ft; and maximum velocities ranged from 1.0 to 3.2 fps for the 1,500- and 5,100-cfs discharges, respectively.

Improvement vlans

46. P~an 1. Results of wave height tests conducted for Plan 1 are presented in Table 6 for test waves from the five test directions. For waves occurring during boating season (spring, summer, fall), maximum wave heights were 2.6 ft in the proposed entrance (Gage 1); 0.5 ft in the proposed access channel (Gages 3 and 4); 0.5 ft in the proposed mooring area (Gages 5 and 6); 0.6 ft in the existing entrance (Gage 8); and 0.6 ft at the upstream limit of the existing channel (Gage 10). Typical wave patterns for Plan 1 are shown in Photo 23.

47. Plans 2 through 9. Wave height data obtained for Plans 2 through 9 for test waves from 313 deg are presented in Table 7. For boating season wave conditions, maximum wave heights were 0.6, 0.8, 0.9, 1.5, 2.2, 1.1, 1.3, and 0.9 ft in the existing jettied entrance (Gage 8) and 0.4, 0.5, 0.5, 1.1, 1.5, 0.9, 1.1, and 0.9 ft in the proposed mooring area (Gages 5, 6! and 6A) for. Plans 2 through 9, respectively. Plans 2 through 4, 7, and 9 met the established 1.0-ft wave height criterion in the mooring area. Typical wave patterns for Plans 2 through 7 are shown in Photos 24 -through 29 for test waves from 313 deg.

48. Plans 9 through l3~ Wave height test results for Plans 9 through 13 are presented in Table 8 for test waves from 42 deg. Maximum wave heights, for boating season storm conditions were 0.6, 0.9, 1.6, 1.1, and 1.0 ft in the proposed mooring area (Gages 5, SA, and 6) for Plans 9 through 13, respectively; and maximum wave heights in the existing jettied entrance (Gage 8) were 0.6 ft for all these plans (9 through 13) for boating season waves. Plans 9, 10, and 13 met the criterion in the mooring area. Wave patterns for Plans 9 through 13 for test waves from 42 deg are shown in Photos 30 through 34.

49. Plans 13 through 16. Wave heights measured for Plans 13 through 16 for test waves from 343 deg are presented in Table 9. For boating season storm waves, maximum wave heights were 2.6, 4.2, 3.6, and 3.0 ft in the proposed entrance (Gage 1); 0.8, 1.0, 0.9, and 0.9 ft in the proposed mooring area (Gages 5, 6, and 6A), and 1.0, 1.3, 1.4, and 1.1 ft in the existing entrance (Gage 8). All these test plans met the established 1.0-ft criterion in the proposed mooring area, however, only Plans 13 and 16 met the 3.0-ft

criterion in the proposed entrance. Typical wave patterns for Plans 13 through 16 are shown in Photos 35 through 38 for test waves from 343 deg.

50. Results of wave height tests for Plan 16 for test waves from 313, 334, 24, and 42 deg are presented in Table 10. For waves occurring during boating season, maximum wave heights were 3.0 ft in the proposed entrance for 6.4-sec, 5.8-ft test waves from 42 deg; 1.0 ft in the proposed mooring area for 7.2-sec, 7.6-ft test waves from 313 deg, 6.0-sec, 4.7-ft and 6.4-sec, 5.8-ft test waves from 42 deg; and 1.0 ft in the existing entrance for 7.2-see, 7.6-ft test waves from 313 deg. The wave height criteria were met by Plan 16 for test waves from the four test directions. Typical wave patterns for Plan 16 are shown in Photos 39 through 42 for the 313-, 334-, 24- and 42-deg directions, respectively.

51. The general movement of tracer material and subsequent deposits on the west side of the harbor configuration with Plan 16 installed are shown in Photo 43 for test waves from 313 deg. Sediment moved easterly along the shoreline and deposited along the shoreline adjacent to the existing west jetty. Material did not move around the jetty head and deposit in the navigation channel.

52. Plans 17 through 19. Wave height test results for Plans 17 through 19 are presented in Table 11 for test waves from 313 deg. For boating season conditions, maximum wave heights were 0.7, 1.2, and 0.9 ft in the proposed mooring area west of the existing entrance (Gage 15) for Plans 17 through 19, respectively. Plans 17 and 19 met the established criteria in the mooring area. Typical wave patterns obtained for Plans 17 through 19 are shown in Photos 44 through 46 for test waves from 313 deg.

53. Wave height data obtained for Plan 19 for test waves from 343 deg are presented in Table 12. Maximum wave heights in the mooring area west of the entrance (Gage 14) were 1.1 ft for boating season conditions. This plan resulted in waves that exceeded the criterion by only 0.1 ft at this location. Maximum wave heights in the proposed entrance (Gage 1) were 2.8 ft, and maximum wave heights in the mooring area east of the existing entrance (Gages 5 and 6A) and in the existing entrance (Gage 8) were 0.6 ft for test waves from 343 deg during boating season conditions. Wave patterns for the test waves that generated these maximum conditions for Plan 19 are presented in Photo 47.

54. Results of wave height tests for Plan 19 for comprehensive test waves from 313, 334, 24, and 42 deg are presented in Table 13. For waves

occurring during boating season, maximum wave heights were 2.9 ft in the proposed entrance (Gage 1) for 6-sec, 4.7-ft test waves from 42 degrees; 0.7 ft in the existing jettied entrance (Gage 8) for 6.4-sec, 5.8-ft test waves from 42 deg; 1.0 ft in the proposed mooring area east of the existing entrance (Gage SA) for 6.4-sec, 5.8-ft test waves from 42 deg.and 6.4 see, 6.9-ft test waves from 24 deg; and 1.0 ft in the proposed mooring area west of the existing entrance (Gages 14 and 15) for 7.2-sec, 8.4-ft test waves from 334 deg, 6.4-sec, 6.9-ft test waves from 24 deg, and 6.4-sec, 5.8-ft test waves from 42 deg. The wave height criteria were met for Plan 19 from test waves from the four directions. Typical wave patterns for Plan 19 for the 5 test directions (313, 334, 343, 24, and 42 deg) are shown in Photos 48 through 57.

55. Wave-induced current patterns and magnitudes obtained for Plan 19 for representative test waves and directions are shown also in Photos 48 through 57. Maximum velocities secured at various locations were as follows:

Location	Maximum Velocity. f?s	Test Wave(s) Period see	Height ft	Direction(s) deg	
Opening between east breakwater	4.0	7.4	8.0	313	+2.8 ft
Area along 1akeward side of detached east breakwater	4.3	7.4	8.0	313	+2.8 ft
Area shoreward of detached east breakwater	1.2	7.4	11.0	343	+2.8 ft
Area between outer heads of detached breakwaters	1.9	6.9 7.4	7.9 11.0	24 343	+2.8 ft +2.8 ft
Area shoreward of detached west breakwater	4.3	7.4	8.0	313	+2.8 ft
Area along 1akeward side of detached west breakwater	4.3	7.4	8.0	313	+2.8 ft
Opening between west breakwaters	5.5	7.4	8.8	334	+2.8 ft
Shoreline west of break waters	3.2	7.4	8.0	313	+2.8 ft

In general, currents moved west to east lakeward of the detached breakwaters for test waves from 313, 334, and 343 deg; and from east to west for test waves from 24 and 42 deg. Both clockwise and counterclockwise eddies were obtained in the mooring areas inside the harbor. The openings between the detached and shore-connected breakwaters resulted in current flow through the harbor and should enhance harbor circulation.

56. Results of water surface elevation and depth-averaged creek current velocity measurements for Plan 19 are presented in Table 14 for the +2.8-ft and +4.0-ft swl's. For the +2.8-ft swl, the maximum rise in water surface elevation in the creek ranged from 0.06 ft for the l,500-cfs discharge to 0.18 ft for the 5,lOO-cfs discharge; and maximum velocities in the creek ranged from 1.5 fps to 3.9 fps for the 1,500- and 5,100-cfs discharges, respectively. With the +4.0-ft swl, maximum rises in water surface elevation ranged from 0.06 to 0.12 ft; and maximum velocities ranged from 1.2 to 3.2 fps for the 1,500- and 5,lOO-cfs discharges, respectively.

57. The general movement of tracer material and subsequent deposits on each side of the harbor (including the groin field east of the harbor) for Plan 19 are shown in Photos 58 through 63. For test waves from 313, 334, and 343 deg, sediment along the shoreline west of the harbor migrated easterly adjacent to the attached west breakwater. Waves from 313 and 334 deg, with the +4.0-ft swl, resulted in fine particles of sediment entering the harbor through the opening between the breakwaters. For test waves from 343, 24, and 42 deg, sediment on the east side of the harbor and between the groins east of the harbor moved shoreward and deposited along the shoreline between the groins and between the westernmost groin and the attached east breakwater.

The tracer material remained between the groins in the cell in which it was placed and did not move around the groin heads nor did it wash over the groins from one cell to another.

58. <u>Plans 20 through 23.</u> The general movement of tracer material and subsequent deposits on the west side of the harbor for Plans 17 and 20 through 23 are shown in Photos 64 through 68 for test waves from 313 deg. Shoreward extensions of the offshore west breakwater (Plans 17 and 20) resulted in fine particles of tracer material entering the harbor through the opening between the breakwaters, similar to Plan 19, for test waves with the +4.0-ft swl. The installation of a stone sill between the heads of the attached and detached west breakwaters (Plan 21) prevented the movement of sediment through the

opening between the structures for both swl's, although a slight bUiidup of material occurred adjacent to the sill for the +4.0-ft swl. Yiththe detached west breakwater extension (Plan 22), very little fine material moved between the opening of the breakwaters for the +4.0-ft swl. This material deposited inside the harbor but did not migrate into the mooring areas ~r the access channel. The installation of the spur on the detached west breakwater (Plan 23) also resulted in a very slight amount of material through the opening between the breakwater with no deposits in the mooring area or access channel. Maximum wave-induced current velocities through the opening between the west breakwaters were checked, and were slightly larger for Plans 21 through 23 than for Plan 19 for test waves from 313 deg. Therefore, the installation of any of these plans should not reduce or inhibit circulation within the harbor. The movement of tracer material and sub~equent deposits on the west side of the harbor for Plans 21 through 23 are shown in Photos 69 through 71 for test waves from 334 deg. For these tests, sediment moved easterly and adjacent to the attached west breakwater, but did not move through the opening between the structures for either the +2.8 or +4.0-ft swl's.

Discussion of test results

59. Results of wave height tests for existing conditions indicated rough and turbulent wave conditions in the entrance. Wave heights up to 6.5 ft were measured in the entrance for boating season conditions, and heights up to 4.0 ft were obtained at the upstream limit of the dredged channel. Visual observations also revealed very confused wave patterns between the jetties due to reflections from the vertical wall structures.

60. Tracer test results for existing conditions indicated that sediment will move easterly or westerly along the shorelines on each side of the harbor entrance depending on the direction of wave approach. Tracer material did not deposit in the jettied entrance for any of the wave conditions tested. Sediment tracer results also indicated that material placed between the groins east of the harbor would remain stable between the groins, however, it may move easterly or westerly depending on direction of wave approach.

61. Water surface elevation tests for existing conditions revealed that the maximum rise in water surface elevation would be only 0.24 ft for the 100-year creek discharge (5,100 cfs), and that maximum current velocities in the lower reaches of the creek would be 3.5 fps for this extreme event.

62. Plan 1. Wave height tests for the originally proposed improvement plan with the proposed mooring area east of the existing creek mouth (Plan 1) indicated that wave heights were well within the established l.O-ft criterion in the proposed mooring area for storm conditions occurring during boating season. Wave heights did not exceed 0.5 ft in the proposed access channel and mooring areas, or 0.6 ft in the existing jettied entrance. These tests indicated that the breakwaters could possibly be lowered and/or reduced in length and still meet the specified criterion.

63. Plans 2 through 8. Results of wave height tests for Plans 2 through 7 for test waves from 313 deg revealed that 350 ft (Plan 7) could be removed from the shoreward leg of the west breakwater, and the 1.0-ft criterion in the mooring area would be met. Maximum wave heights at the Gage 6A location in the mooring area would be 0.9 ft, and 1.1 ft was obtained in the existing jettied entrance. Lowering the crest elevations of the detached breakwaters to +14.5 ft with the removal of 350 ft of the west breakwater (Plan 8) increased wave heights by 0.2 ft in the mooring area and existing entrance to 1.1 and 1.3 ft, respectively; however, removal of 300 ft, as opposed to 350 ft of the shoreward leg, (Plan 9), will result in maximum wave heights of 0.9 ft in both the proposed mooring area and the existing entrance.

64. Plans 9 throu£h 13. Wave height test results for Plans 9 through 13 for test waves from 42 deg indicated that 125 ft of breakwater length could be removed from the shoreward end of the east breakwater (Plan 13) without exceeding the 1.0-ft criterion in the proposed mooring area for waves occurring during boating season. Maximum wave heights in the existing jettied entrance would be only 0.6 ft for waves from this direction with Plan 13 installed.

65. Plans 14 through 16. Wave heights obtained for Plans 13 through 16 for test waves from 343 deg revealed that 150 ft of breakwater length could be removed from the head of the west breakwater (Plan 14) without exceeding the 1.0-ft wave height criterion in the proposed mooring area; however, the 3.0-ft criterion in the proposed entrance was exceeded by 1.2 ft for this test plan for boating season wave conditions. To meet the criterion in the entrance, only 50 ft of the lakeward end of the west breakwater could be removed (Plan 16). This plan also would result in maximum wave heights of 0.9 ft in the proposed mooring area and 1.1 ft in the existing jettied entrance for boating season waves from 343 deg.

66. Test results for Plan 16 for the 313-, 334-, 24-, and 42-deg directions indicated the plan would meet the established 3.0- and 1.0-ft criteria in the proposed entrance and mooring area, respectively, for waves occurring during boating season. Plan 16 was determined to be the optimum plan tested considering wave protection and costs for the first harbor configuration (proposed mooring area east of the existing entrance).

67. Tracer tests conducted on the west side of the Plan 16 harbor configuration indicated that sediment deposits would not occur in the navigation channel. Sediment moved to the existing jetty and deposited adjacent to it, but did not move to its seaward end toward the navigation channel.

68. Based on test results of the first basic harbor configuration, the detached breakwaters were modified prior to installation of the second basic harbor configuration. The east and west detached breakwaters were reduced in elevation from +16.2 and +15.3 ft, respectively, to el +14.5 ft. The east breakwater was also reduced in length by 125 ft (removal from its shoreward end).

69. Plans 17 through 19. Wave heights obtained for Plans 17 through 19 for test waves from 313 deg indicated that the shoreward end of the west breakwater could be reduced by 50 ft (Plan 19) and the 1.0-ft wave height criterion would be met in the mooring area west of the existing entrance for boating season wave conditions. Removal of 100 ft (Plan 18) would result in wave heights of 1.2 ft in the mooring area.

70. Wave heights obtained for Plan 19 for test waves from 343 deg (the most critical direction based on previous tests) indicated that wave heights in the mooring area west of the existing entrance would exceed the criterion at one location by 0.1 ft for boating season conditions. Maximum wave heights in the mooring area east of the existing entrance and in the existing entrance were only 0.6 ft for this wave condition, and maximum wave heights in the new proposed entrance were 2.8 ft (within the established 3.0-ft criterion at this location). NCB indicated that this plan would be acceptable provided the criterion in the west mooring area was not exceeded by boating season waves from the other directions.

71. Test results for Plan 19 for waves from the 313-, 334-, 24-, and 42-deg directions revealed the plan would meet the specified 3.0- and 1.0-ft criteria in the proposed entrance and mooring areas, respectively, for waves occurring during boating season. Considering wave protection and costs,

Plan 19 was selected as the optimum plan tested for the second basic harbor configuration (proposed mooring areas east and west of the existing entrance).

72. Current patterns and magnitudes obtained for Plan 19 indicated that the openings between the attached and detached east and west breakwaters provided circulation within the proposed harbor. Wave-induced currents moved in and/or out of the harbor through the openings and created eddies and current flow throughout the basins. As a result of these openings, harbor circulation should be enhanced. Modifications to the opening between the west breakwaters (tested to prevent sediment from moving into the harbor) did not interfere with harbor circulation in the western portion of the harbor as indicated by the test results. Larger openings between the structures (that may increase circulation) could not be made due to increased wave activity in the harbor during attack by storm waves.

73. Water surface elevation and creek current velocity tests for Plan 19 revealed a maximum rise in water surface elevation of 0.18 ft for the 100-year discharge and maximum velocities in the lower reaches of the creek of 3.9 fps. When compared with existing conditions, these results indicated that the proposed harbor plan would have minimal impact on water surface elevations and velocities through the lower reaches of the creek.

74. Plans 20 through 23. Results of tracer tests for the optimum breakwater configuration (with regard to wave heights) for the second basic harbor configuration (Plan 19) revealed that minor shoaling may occur in the mooring area of the western portion of the harbor for waves from 313 and 334 deg provided a source of sediment is available. Shoreward extensions of the detached breakwater (Plans 17 and 20) resulted in similar results. Test results indicated that a sill between the west breakwaters (Plan 21) would prevent sediment from entering the harbor. If a large source of sediment is available west of the harbor, however, it is possible that a buildup of material would occur adjacent to the sill that would eventually penetrate through the voids of the stone or over the structure. It appeared from current patterns in model tests, however, that deposits would not occur in the mooring area or access channel. An extension of the attached west breakwater (Plan 22) or the installation of a spur on the attached structure (Plan 23) resulted in an accumulation of sediment in the vicinity of the head of the attached breakwater. Very fine particles of material may move through the opening but will not deposit in the mooring area or access channel.

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75. An existing rubble groin is located on the shoreline west of the proposed harbor complex. Tracer tests conducted to determine its effectiveness in trapping sediment from the west for test waves from 313 deg are shown in Photo 72.. Tracer material penetrated through the groin and migrated around its head moving in an easterly direction toward the harbor. These tests indicate that if a source of sediment is located westward of the harbor, the existing groin will not prevent it from moving toward the harbor for test waves from 313 deg.

76. Test results revealed that sediment tracer material placed between the groins east of the harbor would remain between the structures for various test wave conditions for Plan 19. It may move easterly or westerly between the groins in which it is placed but will not move from one cell to another. Since the harbor breakwater structures protect the groin field from wave conditions from. the westerly directions, sediment in the groin fields will be exposed predominantly to waves from the north and east and will likely accumulate on the west side of each cell.

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PART V: CONCLUSIONS

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

~. Existing conditions are characterized by rough and turbulent wave conditions during periods of storm wave attack. Wave heights up to 6.5 ft can occur in the existing entrance during boating season.

Q. The first basic harbor configuration (with the proposed mooring area east of the existing entrance, Plan 1) resulted in wave heights well within the established criteria (3.0 ft in the proposed entrance and 1.0 ft in the proposed mooring area) for boating season wave conditions.

£. The following modifications may be made to the detached breakwaters of the first harbor configuration and acceptable boating season wave conditions will be achieved.

(1) The east and west detached breakwaters may be reduced in elevation from +16.2 and +15.3 ft, respectively, to elevation +14.5 ft.

(2) The length of the east breakwater may be reduced by 125 ft (removal from the shoreward end of the structure).

(3) The length of the west breakwater may be reduced by 350 (removal of 50 ft from the lakeward end and 300 ft from the shoreward end of the structure).

Q. Based on test results, the detached east and west breakwaters of the second basic harbor configuration were reduced to elevations of +14.5 ft and the east breakwater length was reduced by 125 ft (conclusions in paragraphs cl and c2). In addition, SO ft may be removed from the shoreward end of the west breakwater (Plan 19), and acceptable wave conditions during boating season will be achieved for the second harbor configuration (mooring areas east and west of the existing entrance).

~. The openings between the attached and detached east and west breakwaters of the second basic harbor configuration will provide wave-induced current flow through the harbor and should enhance circulation. In the prototype, circulation should be further enhanced by wind driven currents.

f. The construction of the proposed harbor plan will have minimal impact on water surface elevations and creek current velocities in the lower reaches of Eighteenmile Creek.

g. The opening between the attached and detached west breakwaters (Plan 19) may result in minor shoaling in the mooring area in the western portion of the harbor for test waves from 313 and 334 deg, provided a sediment source is available. The installation of a sill between the structures (Plan 21), an extension of the attached breakwater (Plan 22), or a spur on the attached structure (Plan 23) will alleviate this shoaling.

h. Sediment placed between the existing groins east of the harbor for Plan 19 moves easterly and westerly between the structures but will remain relatively stable and not move from one cell to another. Accumulations may occur on the western sides of each cell, however, due to the predominance of the wave directions attacking the groin field.

1. Two-dimensional flume tests can provide additional information concerning structural stability and wave transmission characteristics of the breakwaters.

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Table 1

Summary of Refraction and Shoaling' Analysis

for Olcott Harbor. New York

Deepwater Direction deg	Wave Period sec	Sha11ow-Water* Azimuth deg	Coefficient		Wave Height Adjustment Factor
			Refraction*	Shoa1ing**	
300	6.4	310.7	0.965	0.951	0.918
	7.2	314.5	0.891	0.928	0.827
	7.4	315.1	0.888	0.924	0.821
330	6.4	333.0	0.993	0.951	0.944
	7.2	334.4	0.978	0.928	0.908
	7.4	335.0	0.975	0.924	0.901
360	5.7	348.6	1.010	0.973	0.983
	5.8	348.0	1.007	0.970	0.977
	7.0	338.8	0.987	0.932	0.920
	7.4	336.6	0.983	0.924	0.908
30	5.7	25.6	0.983	0.973	0.956
	6.4	23.5	0.962	0.951	0.915
	6.0	24.3	0.972	0.961	0.934
	6.9	21.6	0.944	0.935	0.883
60	5.7	46.4	0.845	0.973	0.822
	6.4	41. 8	0.806	0.951	0.767
	6.0	43.4	0.822	0.961	0.790
	6.9	37.9	0.773	0.935	0.723

* At approximate locations of wave generator in model.

** At 60-ft pit elevation depth with 2.6- to 4.0-ft storm conditions superimposed based on season of occurrence.

Table 2

Wave Heights for All Approach Angles and Seasons

Recurrence Interval, year	Wave Height, ft Angle Class 1	Angle Class 2	Angle Class 3
	Winter		
5	6.6	8.9	9.2
10	7.5	9.8	9.5
20	8.9	12.1	9.8
50	9.2	13.1	10.5
100	9.8	14.4	13.1
	Spring		
5	3.9	4.9	5.6
10	4.3	5.6	6.2
20	4.9	5.9	6.9
50	5.6	7.9	8.5
100	6.6	8.5	9.2
	Summer		
5	3.6	4.9	4.9
10	5.2	5.6	5.2
20	7.5	6.2	6.9
50	8.9	7.2	7.9
100	10.5	7.5	8.2.
	Fall		
5	4.9	9.8	8.5
10	5.6	10.2	8.9
20	5.9	10.8	9.2
50	7.2	12.5	9.8
100	8.2	12.8	10.8

Table 3

Significant Wave Periods by Angle Class and Wave Height

Wave Height ft	Significant Period. sec Angle Class 1	Angle Class 2	Angle Class 3
1	2.2	2.1	2.3
2	3.5	3.3	3.6
3	4.4	4.2	4.5
4	5.1	4.9	5.2
5	5.7	5.4	5.8
6	6.0	5.7	6.1
7	6.3	6.0	6.4
8	6.6	6.2	6.8
9	6.9	6.5	7.1
10	7.3	6.8	7.4
11	7.6	7.1	7.7
12	7.9	7.4	8.0
13	8.2	7.6	8.4
14	8.5	7.9	8.7
15	8.8	8.2	9.0
16	9.1	8.5	9.3
17	9.4	8.8	9.6
18	9.7	9.0	10.0
19	10.0	9.3	10.3
20	10.3	9.6	10.6
21	10.7	9.9	10.9
22	11.0	10.2	11.2
23	11.3	10.4	11.6
24	11.6	10.7	11.9
25	11.9	11.0	12.2

Table 4

Wave Heights for Existing Conditions

Direction	Test Wave Period	Test Wave Height	Wave Height. ft Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage
deg	sec	ft	--L	-L	-2..	-L	-L	-L	-L	-L..- L.			- 1L	J. L	JL
						swl - +2.8 ft									
313	7.2	7.6	6.4	6.4	7.3	6.4	6.5	6.9	5.0	6.1	5.7	3.7	1.1	0.5	0.4
	7.4	8.0	6.7	7.0	7.7	6.5	6.4	6.7	5.0	6.6	5.9	3.7	1.2	0.5	0.4
						swl - +4.0 ft									
	6.4	6.3	4.9	5.3	5.4	5.8	4.2	5.9	4.2	4.9	4.1	3.0	1.2	0.6	0.4
						swl - +2.8 ft									
334	7.2	8.4	5.6	7.4	7.9	7.6	5.7	6.8	4.8	6.5	5.8	3.4	1.2	0.4	0.4
	7.4	8.8	6.1	7.7	8.3	7.8	6.1	7.4	4.6	6.6	6.0	3.7	1.4	0.5	0.5
						swl - +4.0 ft									
	6.4	6.5	4.9	5.3	6.2	6.0	3.5	5.9	4.4	4.9	4.8	3.6	1.1	0.6	0.4
						swl - +2.8 ft									
343	7.0	9.9	4.3	9.6	7.8	7.6	.6.3	5.6	4.2	6.5	5.2	3.3	1.2	0.4	0.4
	7.4	11.0	4.5	9.9	8.6	8.0	7.2	5.8	4.4	6.7	5.1	3.3	1.3	0.5	0.4
						swl - +4.0 ft									
	5.7	5.8	4.3	5.7	5.9	5.7	3.3	4.7	3.7	5.4	5.0	3.5	1.1	0.4	0.4
	5.8	6.1	4.6	6.8	6.2	6.1	3.5	4.8	3.7	5.5	4.9	3.4	1.3	0.5	0.4
						swl - +2.8 ft									
24	6.0	5.5	5.1	6.5	3.9	3.6	4.7	5.9	4.8	4.0	3.2	2.6	0.8	0.2	0.3
	6.9	7.9	6.5	8.6	5.9	5.5	7.0	6.8	5.1	5.4	5.5	3.2	1.5	0.6	0.5
						swl - +4.0 ft									
	[illegible]	4.	[illegible]	[illegible]	4.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

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Table 4 (~onc1uded)

Direction deg	Test Wave Period sec	Test Wave Height ft	Wave Height. ft Gage -L	Gage --L	Gage -L	Gage --L	Gage --L	Gage -L	Gage -*L*	Gage ---L	Gage --L	Gage - 1Q...	Gage ...1L	Gage -*1L*	Gage ..l.L
42															
					sw1 = +2.8" ft										
	6.0	4.7	3.1	4.7	3.4	3.9	4.8	5.5	4.6	4.9	3.4	4.0	1.1	0.3	0.3
	6.9	6.4	4.4	6.1	4.1	4.9	5.4	6.0	4.5	6.5	5.7	4.4	1.3	0.5	0.5
					sw1 = +4.0 ft										
	5.7	4.0	1.9	2.7	2.2	2.8	3.0	3.3	3.1	2.9	3.0	2.9	1.0	0.3	0.3
	6.4	5.8	2.8	3.9	2.8	3.5	4.2	5.0	4.0	4.0	3.8	3.9	1.1	0.4	0.3

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Table 6

Wave Heights for Plan 1

Direction	Test Wave		Wave Height, ft												
	Period	Height	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage
deg	sec	ft	--L	--L.	--L	--!L	-L	--L	-L	-1L	---2-	.J&..	.J.L	J.L	-1.L
						swl - +2.8 ft									
313	7.2	7.6	1.7	0.8	0.4	0.3	0.4	0.3	0.8	0.5	0.5	0.5	0.5	0.4	0.5
	7.4	8.0	1.6	0.8	0.3	0.3	0.4	0.3	0.8	0.5	0.5	0.5	0.4	0.3	0.4
						swl - +4.0 ft									
	6.4	6.3	0.9	0.6	0.4	0.3	0.3	0.4	0.7	0.4	0.3	0.4	0.3	0.3	0.3
						swl - +2.8 ft									
334	7.2	8.4	2.5	1.0	0.4	0.3	0.4	0.4	0.9	0.6	0.5	0.5	0.3	0.2	0.3
	7.4	8.8	2.4	1.1	0.4	0.4	0.4	0.4	0.9	0.5	0.4	0.5	0.3	0.2	0.2
						swl - +4.0 ft									
	6.4	6.5	1.9	0.8	0.3	0.3	0.3	0.3	0.7	0.5	0.3	0.6	0.2	0.1	0.\3
						swl - +2.8 ft									
343	7.0	9.9	2.6	1.1	0.4	0.5	0.5	0.5	0.9	0.5	0.4	0.6	0.4	0.2	0.3
	7.4	11.0	2.6	1.1	0.4	0.5	0.5	0.5	0.9	0.5	0.4	0.6	0.6	0.2	0.3
						swl - +4.0 ft									
	5.7	5.8	1.6	0.7	0.3	0.2	0.2	0.2	0.5	0.4	0.3	0.4	0.2	0.1	0.1
	5.8	6.1	1.8	0.8	0.3	0.3	0.3	0.2	0.7	0.3	0.3	0.4	0.2	0.1	0.2
						swl - +2.8 ft									
24	6.0	5.5	1.6	0.5	0.3	0.3	0.2	0.3	0.7	0.3	0.3	0.3	0.2	0.2	0.1
	6.9	7.9	3.0	1.2	0.5	0.4	0.4	0.4	0.9	0.6	0.5	0.6	0.3	0.3	0.4
						swl - +4.0 ft									
	5.7	4.7	1.5	0.6	0.3	0.3	0.2	0.3	0.6	0.3	0.3	0.3	0.2	0.2	0.2
	6.4	6.9	1.7	0.7	0.4	0.3	0.3	0.5	0.7	0.4	0.3	0.4	0.3	0.3	0.3

Table 6 (Concluded)

Direction	Test Wave		Wave Height, ft												
	Period	Height	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage
deg	sec	ft		-L	- 2...	--- iL	-L	--L	-l	--- L	--L	.J.Q. ...	J.L	-1L	...1L
					sw1 - +2.8 ft										
42	6.0	4.7	2.5	1.3	0.3	0.3	0.3	0.4	0.8	0.4	0.4	0.4	0.2	0.2	0.2
	6.9	6.4	3.3	1.7	0.4	0.4	0.4	0.4	1.0	0.6	0.5	0.7	0.3	0.2	0.2
					sw1 - +4.0 ft										
	5.7	4.0	1.8	1.0	0.3	0.3	0.3	0.3	0.6	0.4	0.4	0.4	0.2	0.1	0.1
	6.4	5.8	2.4	1.5	0.4	0.4	0.3	0.5	0.8	0.5	0.5	0.5	0.2	0.1	0.2

Table 7

Wave Heights for Plans 2 through 9 for Test Waves

from 313 Deg

Plan	Test Wave		Wave Height. ft													
Plan	Period	Height	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage
No.	sec	ft	--L	--L	---L	-L	---L	-L	..M. ..	-L.	-L	-L	- 19-	-1L	..J L	..ll ..
						swl ... +2.8 ft										
2	7.2	7.6	1.4	0.8	0.4	0.3	0.4	0.3	- -	0.9	0.6	0.6	0.5	0.5	0.3	0.4
	7.4	8.0	1.4	0.8	0.4	0.4	0.4	0.3	- -	0.9	0.6	0.5	0.5	0.4	0.3	0.5
						swl ... +4.0 ft										
	6.4	6.3	1.0	0.6	0.4	0.4	0.4	0.4	- -.	0.7	0.5	0.4	0.4	0.4	0.2	0.4
						swl ... +2.8 ft										
3	7.2	7.6	1.5	0.9	0.5	0.4	0.5	0.4	- -	1.1	0.8	0.5	0.6	0.5	0.3	0.3
	7.4	8.0	1.5	0.8	0.5	0.4	0.5	0.4	--	1.1	0.8	0.5	0.7	0.6	0.3	0.4
						sw1 ... +4.0 ft										
	6.4	6.3	1.0	0.6	0.4	0.4	0.4	0.4	- -	0.8	0.5	0.4	0.5	0.4	0.2	0.2
						sw1 ... +2.8 ft										
4	7.2	7.6	1.5	1.0	0.5	0.4	0.4	0.5	- -	1.6	0.9	0.6	0.8	0.6	0.3	0.4
	7.4	8.0	1.5	1.0	0.6	0.5	0.5	0.4	- -	1.6	0.9	0.6	0.9	0.6	0.3	0.5
						sw1 .. +4.0 ft										
	6.4	6.3	1.1	0.7	0.4	0.4	0.3	0.3	- -	1.1	0.6	0.3	0.6	0.4	0.2	0.3
						sw1 .. +2.8 ft										
5	7.2	7.6	1.6	1.3	0.7	0.6	0.6	0.7	1.1	2.7	1.5	1.0	1.4	0.7	0.3	0.5
	7.4	8.0	1.5	1.1	0.6	0.5	0.5	0.7	- -	2.8	1.4	0.8	1.2	0.7	0.3	0.5
						sw1 .. +4.0 ft										
	6.4	6.3	1.1	0.8	0.5	0.4	0.4	0.6	- -	1.4	0.8	0.5	0.8	0.5	0.2	0.3

(Continued)

Table 7 (Concluded)

Plan	Test Wave		Wave HeiJl:ht. ft													
	Period	Height	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage
N	sec	ft	-L	---1....	----L	-L	--L	-L	--2A....		-- -L	---2....	.JJL	lL	...ll...	-1.L
						swl - +2.8 ft										
6	7.2	7.6	1.7	1.3	0.8	0.7	0.6	1.1	1.5	3.7	2.2	1.4	1.9	0.8	0.4	0.5
	7.4	8.0	1.6	1.3	0.8	0.7	0.7	1.0	--	3.8	2.2	1.3	2.0	0.9	0.4	0.5
						swl - +4.0 ft										
	6.4	6.3	1.0	0.9	0.6	0.6	0.5	0.7	--	2.6	1.3	0.8	1.2	0.5	0.3	0.5
						swl - +2.8 ft										
7	7.2	7.6	1.5	1.1	0.6	0.5	0.5	--	0.9	2.0	1.1	0.7	1.0	0.6	0.3	0.4
8	7.2	7.6	1.4	1.2	0.8	0.6	0.6	--	1.1	2.2	1.3	0.8	1.2	0.6	0.3	0.4
9	7.2	7.6	1.4	1.1	0.7	0.6	0.5	0.6	0.9	1.7	0.9	0.6	0.9	0.5	0.3	0.4
	7.4	8.0	1.5	1.1	0.8	0.6	0.6	0.6	--	1.8	1.1	0.7	1.1	0.6	0.3	0.3
						sw1 - +4.0 ft										
	6.4	6.3	1.0	0.9	0.5	0.5	0.4	0.5	--	1.2	0.7	0.5	0.7	0.5	0.2	0.4

Table 8

Wave HeiEhts for Plans 9 through 13 for Test Waves

from 42 DeE

Plan	Test Wave Period	Test Wave Height	Wave Height, ft Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage
N2...-	sec	ft	-L	-L	--L	--- L	-L	....2L	--2-	L	-L	-2...	-1JL	...1L	-1L	..1L
						swl .. +2.8 ft										
9	6.0	4.7	2.4	1.2	0.3	0.3	0.3	- -	0.3	0.7	0.5	0.4	0.4	0.2	0.1	0.1
	6.9	6.4	3.2	1.6	0.4	0.4	0.4	- -	0.4	1.0	0.6	0.4	0.6	0.2	0.1	0.3
						swl .. +4.0 ft										
9	5.7	4.0	1.7	1.0	0.4	0.3	0.3	- -	0.4	0.6	0.5	0.4	0.5	0.2	0.1	0.1
	6.4	5.8	2.6	1.5	0.5	0.5	0.4	- -	0.6	1.0	0.6	0.5	0.6	0.3	0.1	0.2
10	6.4	5.8	2.5	1.5	0.5	0.5	- -	0.9	0.5	1.0	0.6	0.5	0.6	0.3	0.2	0.3
11	6.4	5.8	2.5	1.5	0.5	0.6	- -	1.6	0.6	0.9	0.6	0.5	0.5	0.2	0.2	0.4
12	6.4	5.8	2.5	1.5	0.5	0.5	- -	1.1	0.5	0.9	0.6	0.5	0.5	0.2'	0.2	0.4
13	6.4	5.8	2.6	1.5	0.5	0.5	0.5	1.0	0.6	1.0	0.6	0.5	0.6	0.3	0.2	0.3
	5.7	4.0	1.8	1.1	0.4	0.3	- -	0.5	0.4	0.7	0.5	0.4	0.5	0.2	0.1	0.1
						sw1 .. +2.8 ft										
13	6.0	4.7	2.5	1.4	0.4	0.3	- -	0.9	0.3	0.8	0.5	0.4	0.4	0.2	0.2	0.1
	6.9	6.4	3.2	1.7	0.5	0.5	- -	1.2	0.' 5	1.0	0.7	0.5	0.7	0.2	0.2	0.2

Table 9

Wave Heights for Plans 13 through 16 for Test Waves

from 343 Deg

Plan	Test Wave Period	Test Wave Height	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage
			Wave Height, ft													
No.	sec	ft	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
						swl - +2.8 ft										
13	7.0	9.9	2.6	1.3	0.6	0.6	0.6	0.7	0.7	1.5	1.0	0.7	1.0	0.7	0.3	0.5
	7.4	11.0	2.6	1.4	0.7	0.6	0.7	0.8	0.8	1.5	1.0	0.7	1.1	0.9	0.4	0.6
						swl - +4.0 ft										
13	5.7	5.8	1.7	0.9	0.3	0.3	0.3	0.5	0.3	0.9	0.6	0.4	0.6	0.3	0.1	0.1
	5.8	6.1	1.9	1.0	0.3	0.3	0.4	0.5	0.4	1.0	0.7	0.4	0.6	0.3	0.1	0.2
						swl - +2.8 ft										
14	7.0	9.9	4.2	2.2	0.8	0.6	0.7	- -	1.0	2.0	1.3	1.1	1.5	0.7	0.3	0.5
15	7.0	9.9	3.6	2.1	0.7	0.7	0.7	- -	0.9	1.8	1.4	0.9	1.3	0.7	0.3	0.5
16	7.0	9.9	3.0	1.5	0.7	0.6	0.7	0.9	0.8	1.7	1.1	0.8	1.0	0.7	0.3	0.5
	7.4	11.0	2.9	1.5	0.7	0.6	0.6	0.9	- -	1.6	1.0	0.7	1.1	0.8	0.4	0.5
						swl - +4.0 ft										
16	5.7	5.8	2.0	0.9	0.3	0.3	0.3	0.4	- -	0.9	0.5	0.4	0.5	0.3	0.1	0.1
	5.8	6.1	2.1	0.9	0.3	0.3	0.4	0.5	- -	0.9	0.6	0.4	0.5	0.2	0.1	0.2

Table 10

Wave Heights for Plan 16 for Test Waves from 313, 334, 24, and 42 Deg

Direction deg	Test Wave Period sec	Test Wave Height ft	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage
						swl = +2.8 ft											
313	7.2	7.6	1.7	1.1	0.6	0.6	0.5	--	--	1.0	1.6	1.0	0.6	0.9	0.5	0.2	0.3
	7.4	8.0	1.8	1.1	0.7	0.5	0.6	--	--	1.0	1.7	1.1	0.6	1.0	0.6	0.2	0.2
						swl = +4.0 ft											
	6.4	6.3	1.4	0.9	0.5	0.4	0.5	--	--	0.6	1.2	0.7	0.4	0.7	0.3	0.1	0.2
						swl = +2.8 ft											
334	7.2	8.4	2.5	1.5	0.6	0.4	0.5	--	--	0.7	1.3	0.8	0.6	0.8	0.6	0.2	0.2
	7.4	8.8	2.6	1.5	0.6	0.5	0.5	--	--	0.8	1.5	0.9	0.6	0.8	0.5	0.2	0.2
						swl = +4.0 ft											
	6.4	6.5	2.2	1.1	0.5	0.3	0.4	--	--	0.6	1.0	0.6	0.5	0.6	0.3	0.2	0.3
						swl = +2.8 ft											
24	6.0	5.5	2.0	1.1	0.3	0.3	--	0.6	0.5	--	0.8	0.5	0.4	0.4	0.1	0.1	0.1
	6.9	7.9	3.1	2.0	0.5	0.4	--	0.8	0.8	--	1.1	0.8	0.6	0.8	0.3	0.2	0.3
						swl = +4.0 ft											
	5.7	4.7	1.8	1.1	0.3	0.4	--	0.5	0.6	--	0.9	0.5	0.5	0.4	0.2	0.1	0.1
42	6.4	6.9	2.1	1.2	0.4	0.5	--	0,8	0.8	--	0.9	0.5	0.4	0.5	0.2	0.1	0.1
						swl = +2.8 ft											
	6.0	4.7	2.7	1.6	0.4	0.4	--	1.0	0.5	--	1.0	0.6	0.5	0.5	0.2	0.1	0.1
	6.9	6.4	3.6	2.1	0.5	0.5	--	1.3	0.7	--	1.2	0.7	0.6	0.7	0.3	0.2	0.3
						swl = +4.0 ft											
	5.7	4.0	2.5	1.7	0.4	0.3	--	0.5	0.5	--	0.9	0.6	0.5	0.6	0.2	0.1	0.1
	6.4	5.8	3.0	1.9	0.5	0.6	--	1.0	0.7	--	1.1	0.7	0.6	0.7	0.3	0.2	0.2

Table 11

Wave Heights for Plans 17 through 19 for Test Waves

from 313 Deg. swl - +2.8 ft

Test Wave period sec	Test Wave Height ft	Gage --L	Gage -L	Gage -L	Gage ---iL	Gage ----L	Gage -M-	Gage --L	Gage -L	Gage --...L	Gage [illegible]	Gage -1.L	Gage -1L	Gage ...1L
Plan 17														
7.2	7.5	1.6	0.9	0.4	0.3	0.4	0.4	0.8	0.5	0.3	0.5	0.3	0.7	0.7
Plan 18														
7.2	7.5	1.6	0.9	0.4	0.3	0.4	0.4	0.8	0.5	0.3	0.4	0.3	0.8	1.2
Plan 19														
7.2	7.5	1.6	0.9	0.4	0.3	0.4	0.4	0.7	0.5	0.4	0.4	0.3	0.8	0.9

Table 12

Wave Heights for Plan 19 for Test Waves

from 343 Deg

Test Wave Period sec	Test Wave Height ft	Gage --L	Gage -L	Gage -L	Gage ---iL	Gage ----L	Gage -M-	Gage --L	Gage -L	Gage --...L	Gage .J..Q.. ..	Gage -1.L	Gage -1L	Gage ...1 L
swl - +2.8 ft														
7.0	9.9	2.8	1.6	0.6	0.6	0.6	0.6	1.0	0.6	0.5	0.7	0.6	1.1	1.0
7.4	11.0	3.0	1.7	0.7	0.6	0.7	0.7	1.1	0.6	0.5	0.7	0.7	1.1	1.0
swl - +4.0 ft														
5.7	.5.8	1.8	0.7	0.4	0.3	0.4	0.4	0.6	0.5	0.5	0.5	0.3	0.7	0.6
5.8	6.1	1.9	0.7	0.4	0.4	0.4	0.3	0.6	0.5	0.4	0.5	0.2	0.7	0.7

Table 13

Wave Heights for Plan 19 for Test Waves
from 313. 334. 24. and 42 Deg

Direction deg	Test Wave Period	Test Wave Height ft	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage	Gage
			swl - +2.8 ft														
313	7.2	7.6	1.6	0.9	0.4	0.3	0.4	--	--	0.4	0.7	0.5	0.4	0.4	0.3	0.8	0.9
	7.4	8.0	1.9	1.1	0.5	0.4	0.5	--	--	0.4	0.9	0.6	0.4	0.5	0.3	0.8	1.0
			swl - +4.0 ft														
	6.4	6.3	1.1	0.7	0.4	0.3	0.3	--	--	0.3	0.5	0.4	0.3	0.4	0.2	0.6	0.8
			swl - +2.8 ft														
334	7.2	8.4	2.5	1.2	0.8	0.4	0.5	--	--	0.4	0.8	0.6	0.5	0.7	0.4	1.0	1.0
	7.4	8.8	2.5	1.3	0.7	0.4	0.5	--	--	0.4	0.9	0.6	0.5	0.7	0.4	1.1	1.0
			swl - +4.0 ft														
	6.4	6.5	1.9	0.9	0.6	0.4	0.4	--	--	0.4	0.6	0.6	0.4	0.7	0.3	0.8	0.8
			swl - +2.8 ft														
24	6.0	5.5	1.7	1.0	0.3	0.4	--	0.8	0.5	--	0.7	0.4	0.3	0J4	0.2	0.7	0.6
	6.9	7.9	;302	1.6	0.6	0.6	--	1.1	0.6	--	1.2	0.8	0.6	0.8	0.5	1.1	0.8
			swl - +4.0 ft														
	5.7	4.7	1.8	1.1	0.4	0.4	--	0.6	0.5	--	0.7	0.5	0.4	0.4	0.2	0.8	0.7
	6.4	6.9	2.0	1.3	0.5	0.6	--	1.0	0.7	--	0.8	0.5	0.4	0.5	0.2	1.0	0.8
			swl - +2.8 ft														
42	6.0	4.7	2.9	1.9	0.4	0.4	--	0.9	0.5	--	1.0	0.6	0.5	0.6	0.2	0.9	0.6
	6.9	6.4	3.7	2.4	0.5	0.6	--	1.3	0.7	--	1.2	0.8	0.6	0.9	0.4	1.2	1.0
			swl - +4.0 ft														
	5.7	4.0	2.0	1.6	0.6	0.4	--	0.7	0.6	--	1.0	0.6	0.5	0.6	0.2	0.8	0.7
	6.4	5.8	3.0	2.2	0.8	0.7	--	1.0	0.8	--	1.1	0.7	0.6	0.7	0.3	1.0	0.9

Table 14

Water Surface Elevations (el) and Creek Current Velocities for Plan 19

Station	+2.8-ft swl = 245.6 IGLD Water Surface el. ft	+2.8-ft swl = 245.6 IGLD Creek Current velocity. fps	+4.0-ft swl + 246.8 IGLD Water Surface el. ft	+4.0-ft swl + 246.8 IGLD Creek Current velocity. fps
		1.500-cfs Discharge		
2900	245.66	0.9	246.86	0.8
2300	245.66	0.6	246.86	0.7
1800	245.60	1.5	246.80	1.2
1300	245.60	1.2	246.80	0.9
600	245.60	0.7	246.80	0.7
0	245.60	0.4	246.80	0.6
		3.700-cfs Discharge		
2900	245.72	1.7	246.92	1.3
2300	245.72	1.6	246.92	1.1
1800	245.60	3.0	246.86	2.6
1300	245.60	2.7	246.80	1.9
600	245.60	1.2	246.80	1.1
0	245.60	1.1	246.80	0.6
		5.100-cfs Discharge		
2900	245.78	2.0	246.92	1.6
2300	245.78	1.5	246.92	1.3
1800	245. 72	3.9	246.86	3.2
1300	245.66	3.2	246.80	2.5
600	245.60	1.3	245.80	1.1
0	245.60	1.1	246.80	0.8

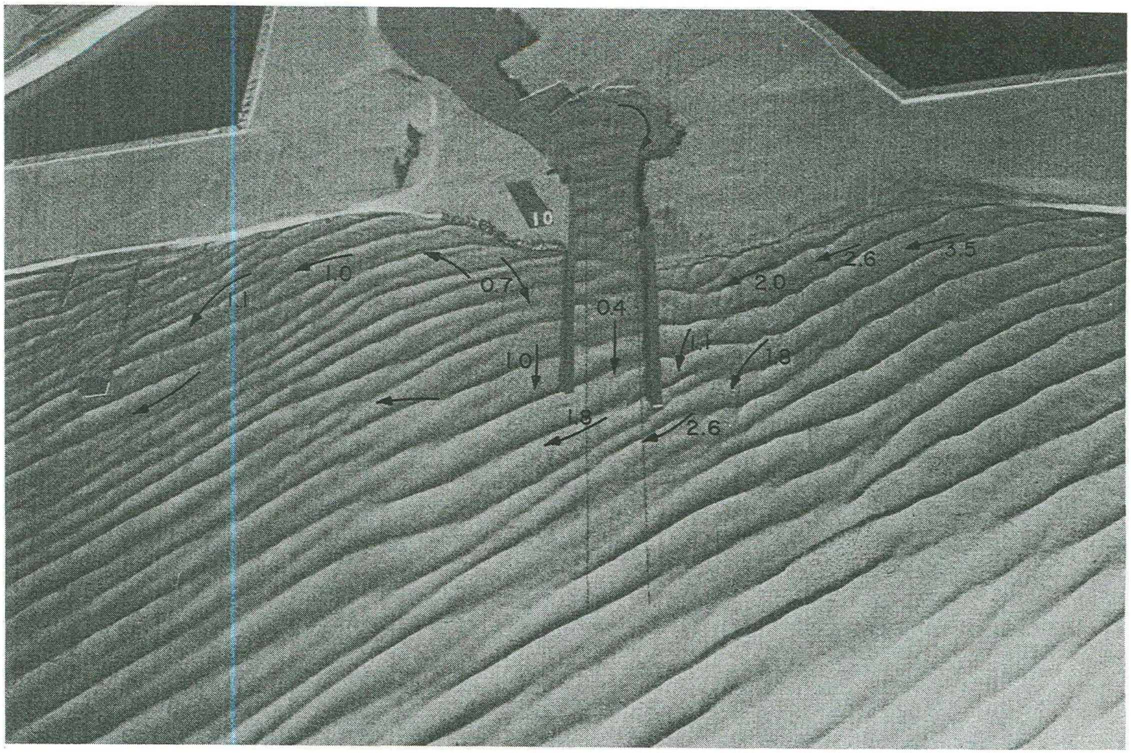

Photo 1. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for existing conditions; 6.4-sec, 6.3-ft waves approaching from 313 deg; +4.0-ft swl

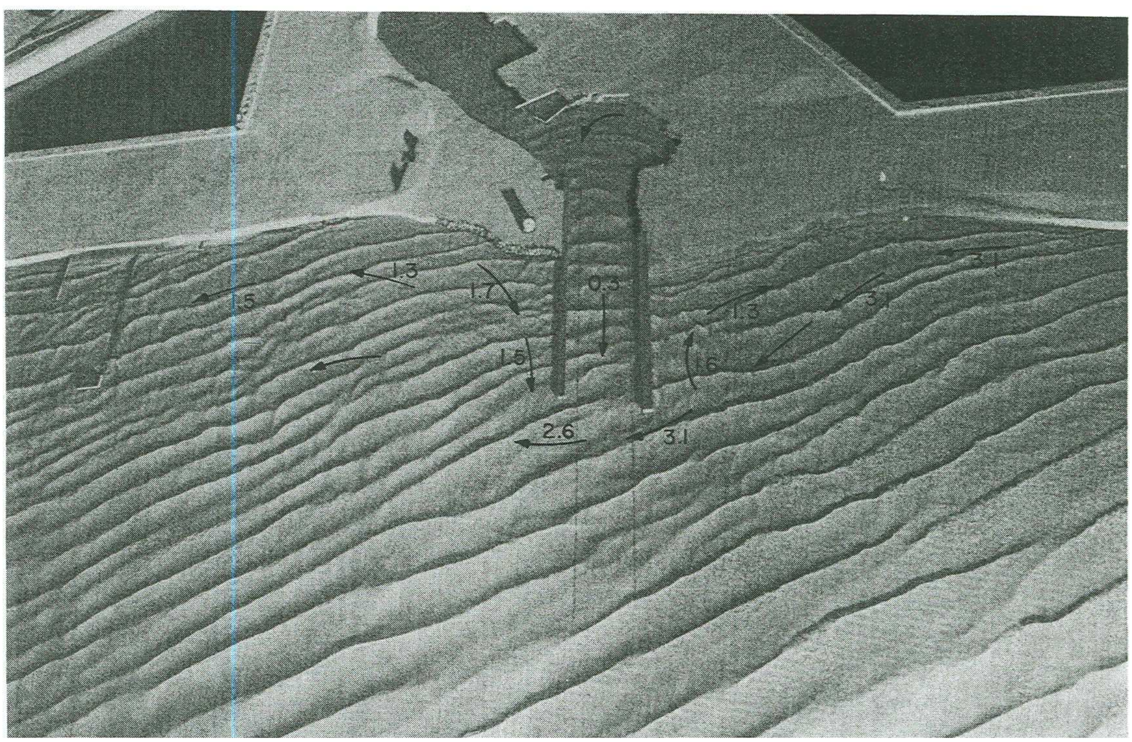

Photo 2. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for existing conditions; 7.4-sec, 8.0-ft waves approaching from 313 deg; +2.8-ft swl

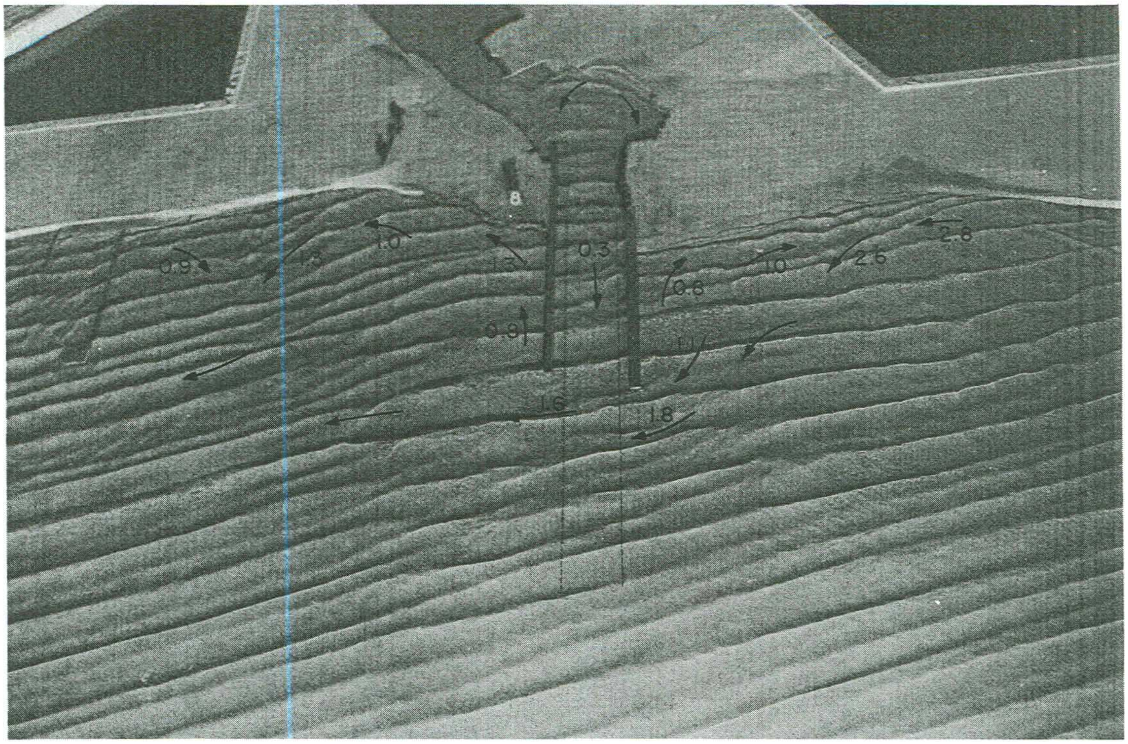

Photo 3. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for existing conditions; 6.4-sec, 6.S-ft waves approaching from 334 deg; +4.0-ft swl

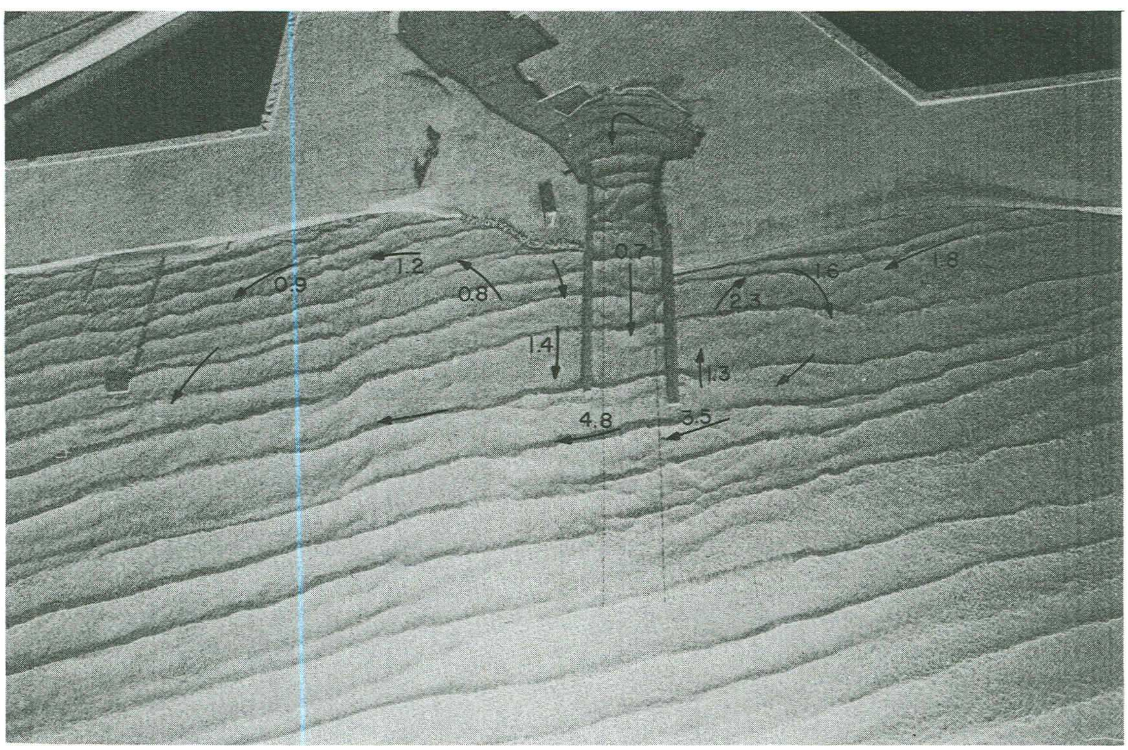

Photo 4. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for existing conditions; 7.4-sec, 8.8-ft waves approaching from 334 deg; +2.8-ft swl

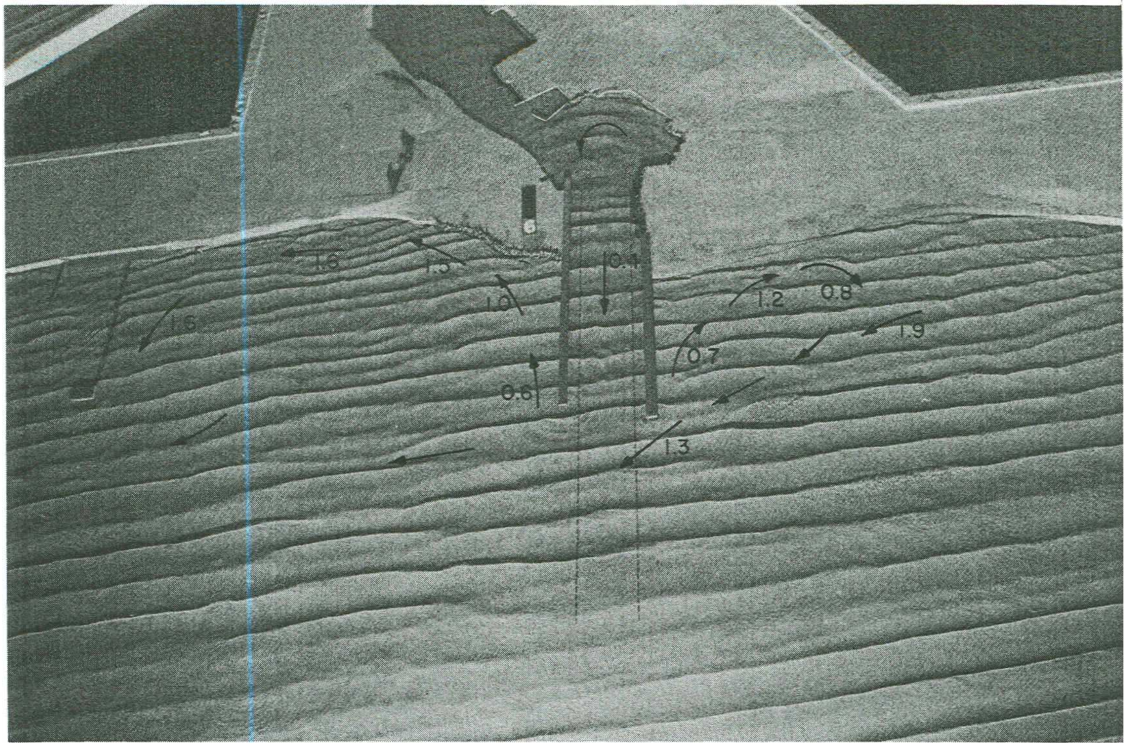

Photo 5. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for existing conditions; S.7-sec, S.8-ft waves approaching from 343 deg; +4.0-ft swl

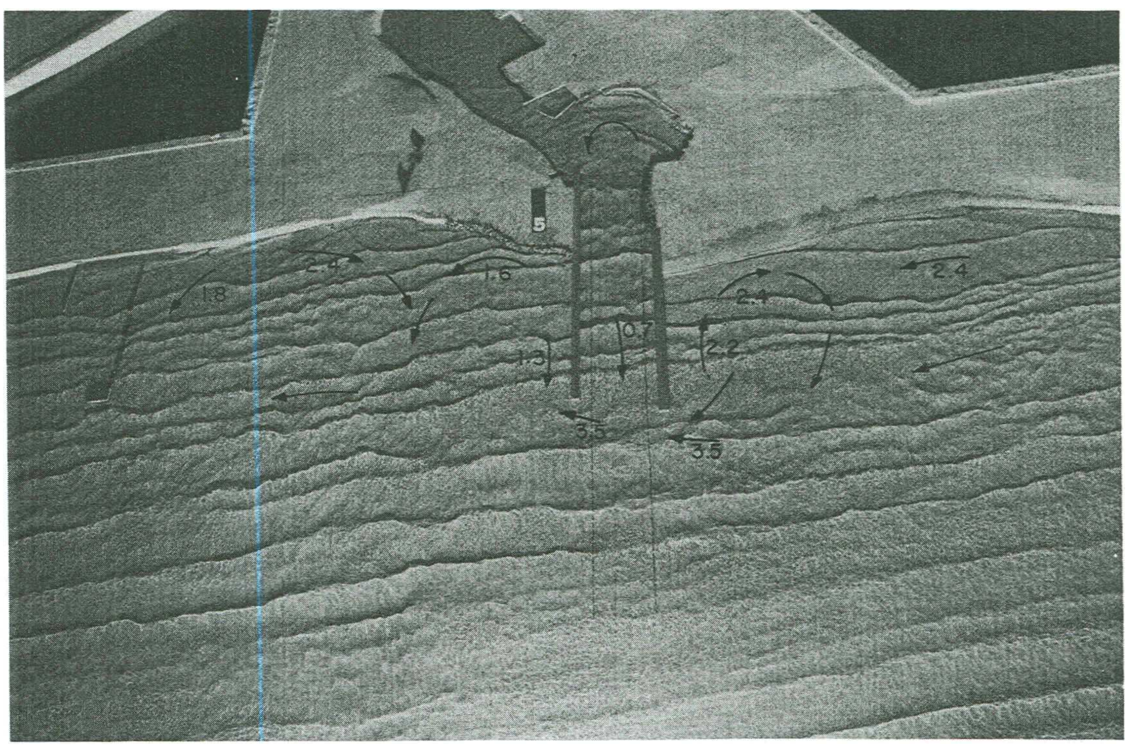

Photo 6. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for existing conditions; 7.4-sec, ll-ft waves approaching from 343 deg; +2.8-ft swl

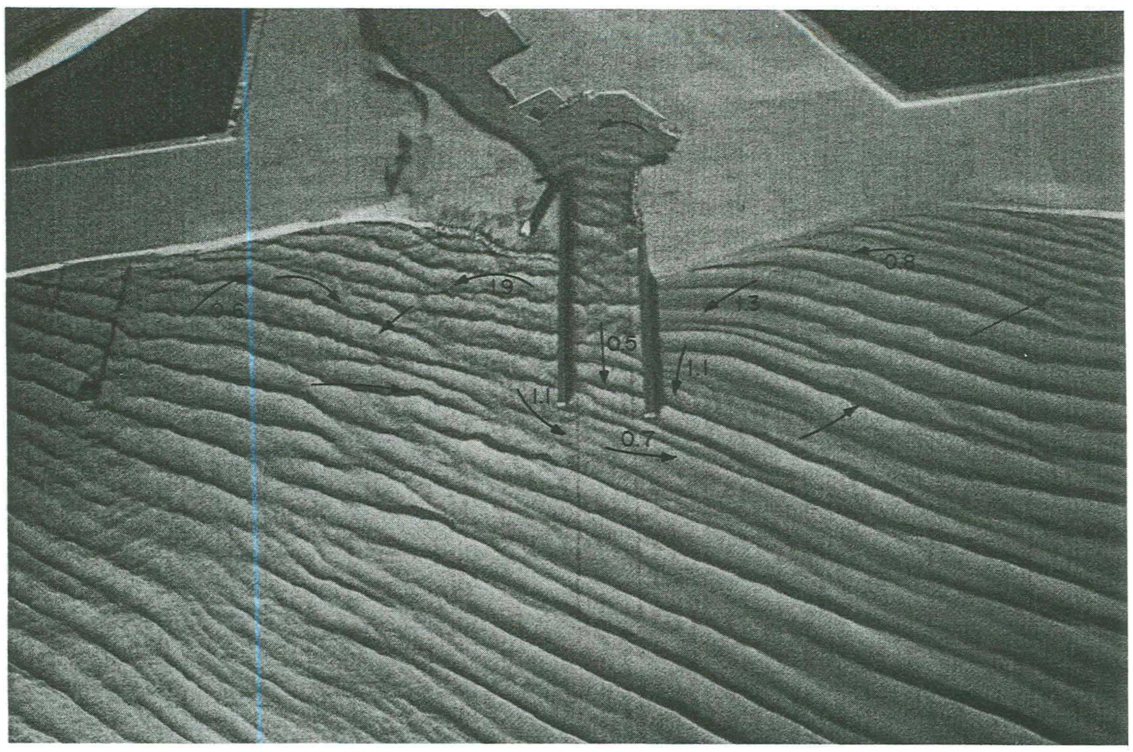

Photo 7. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for existing conditions; S.7-sec, 4.7-ft waves approaching from 24 deg; +4.0-ft swl

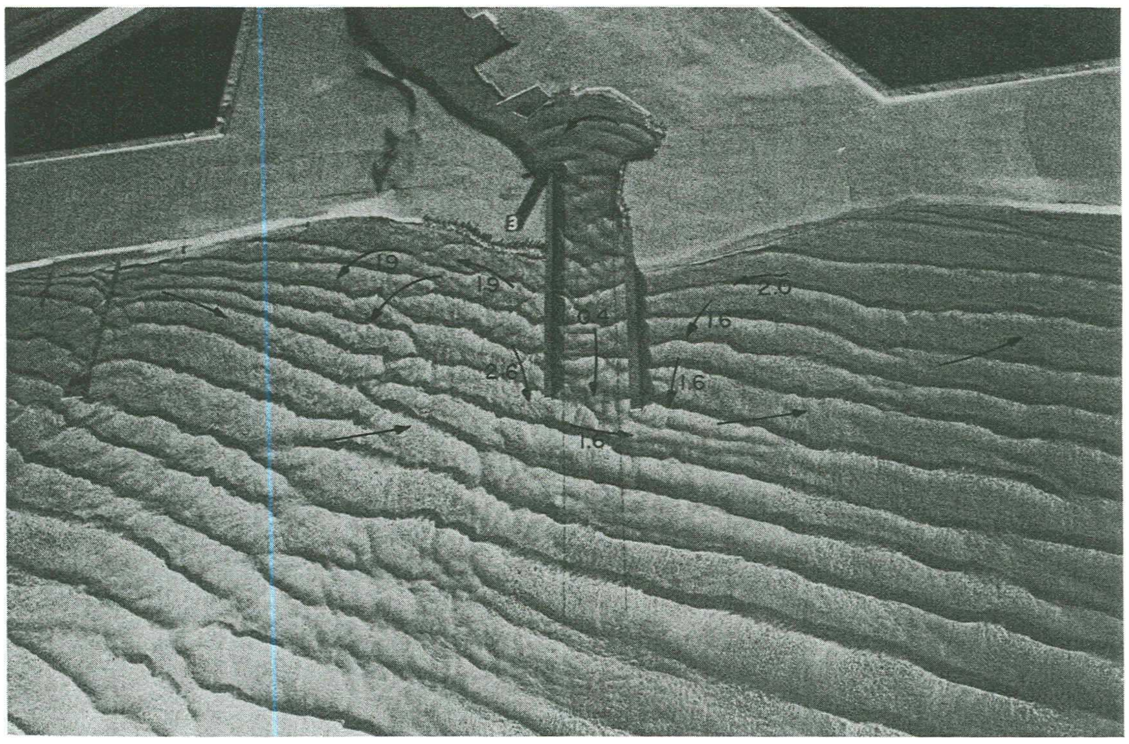

Photo 8. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for existing conditions; 6.9-sec, 7.9-ft waves approaching from 24 deg; +2.8-ft swl

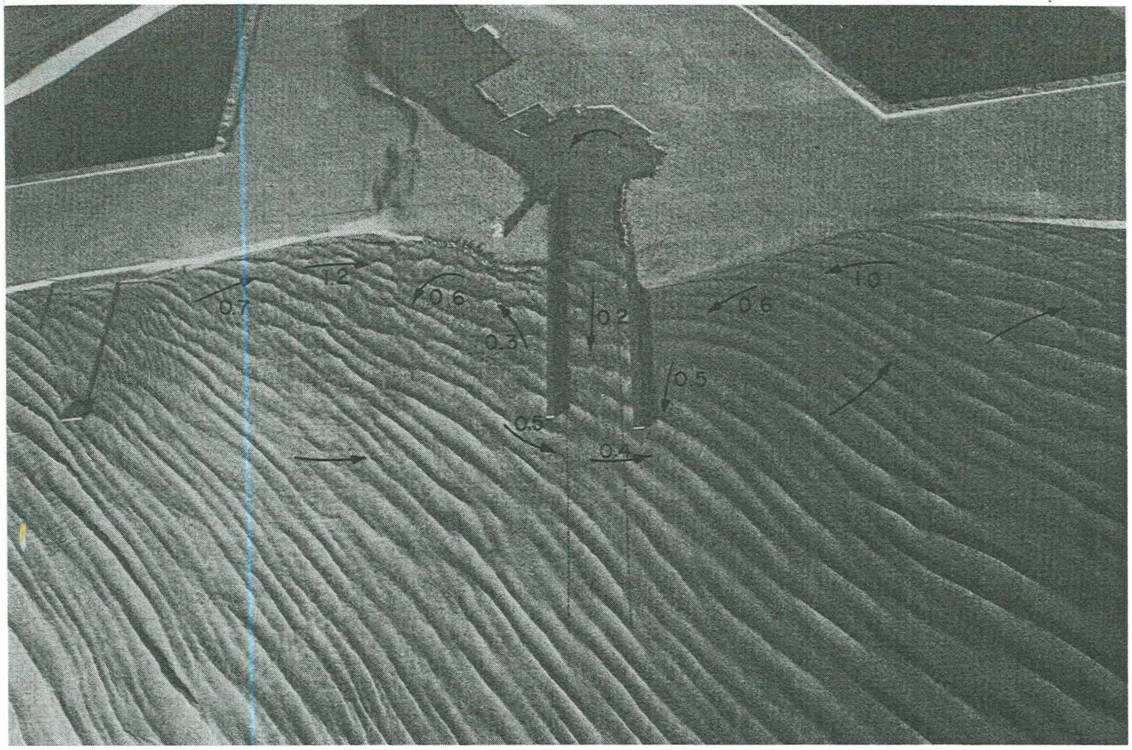

Photo 9. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for existing conditions; S.7-sec,
- 4.0-ft waves approaching from 42 deg; +4.0-ft swl

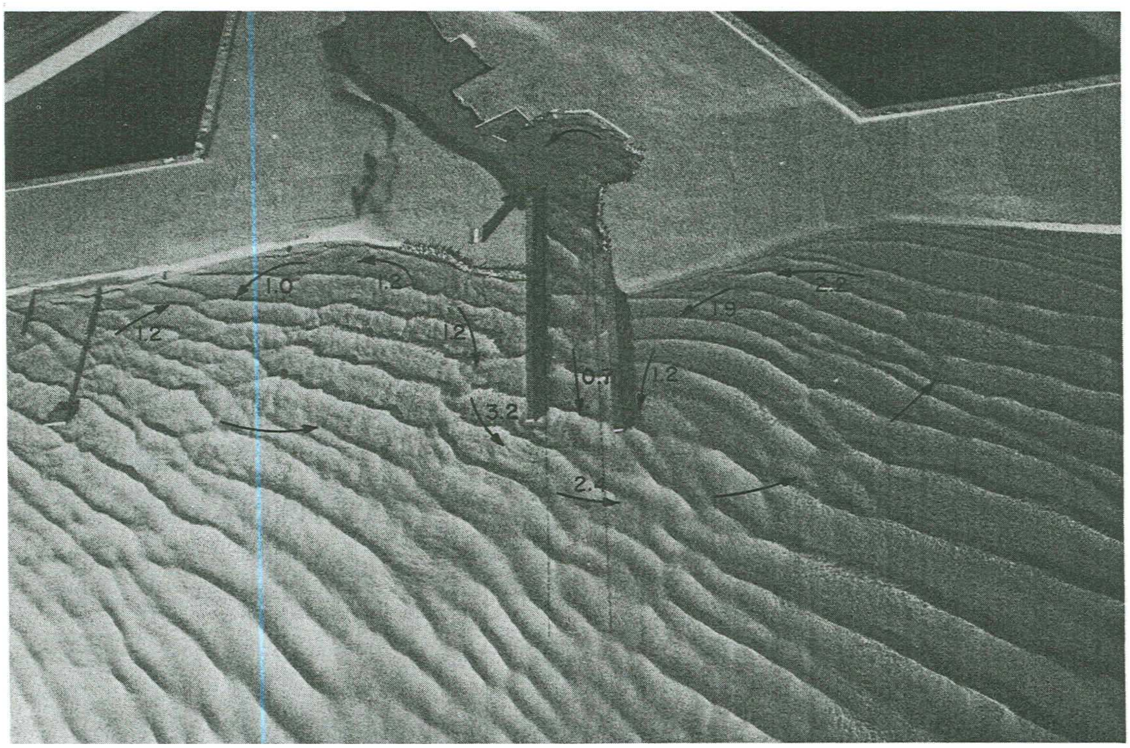

Photo 10. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for existing conditions; 6.9-sec,
6.4-ft waves approaching from 42 deg; +2.8-ft swl

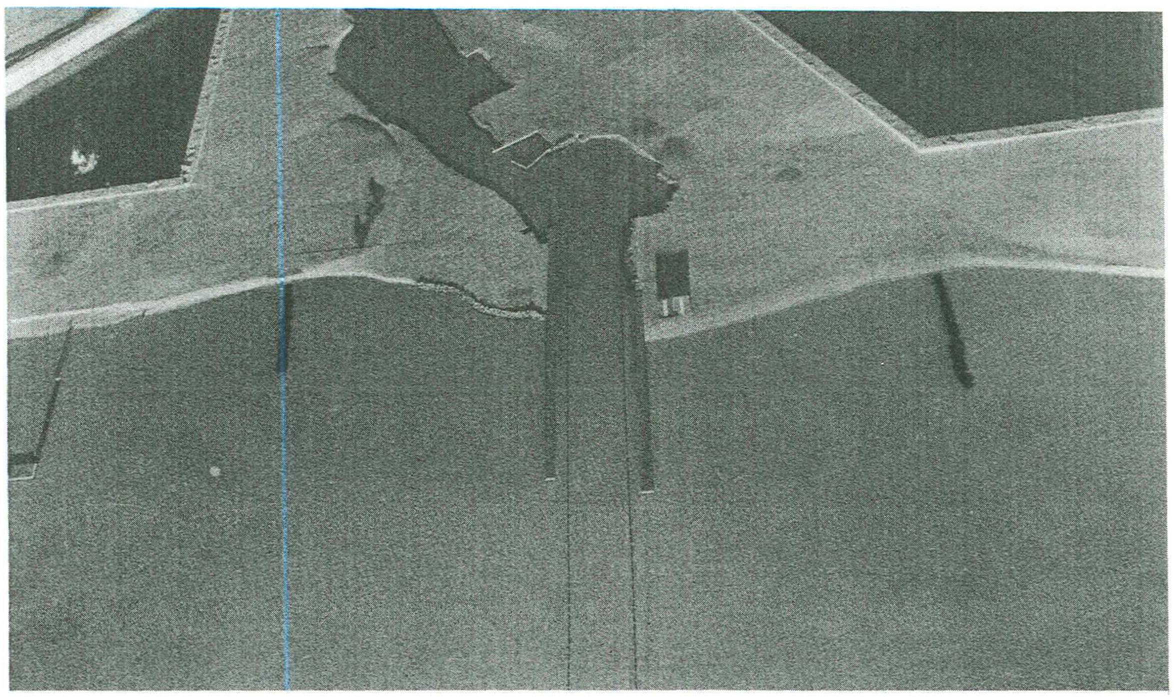

Photo 11. Placement of tracer east and west of the entrance prior to testing of existing conditions

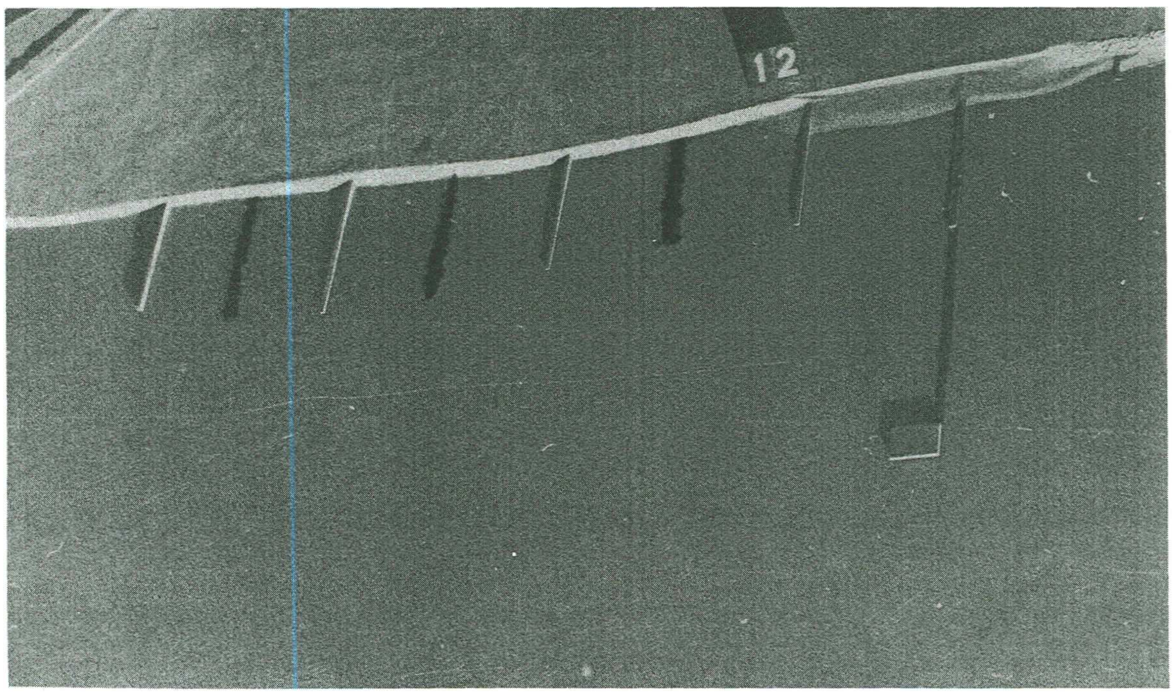

Photo 12. Placement of tracer in the groin field east of the entrance prior to testing of existing conditons

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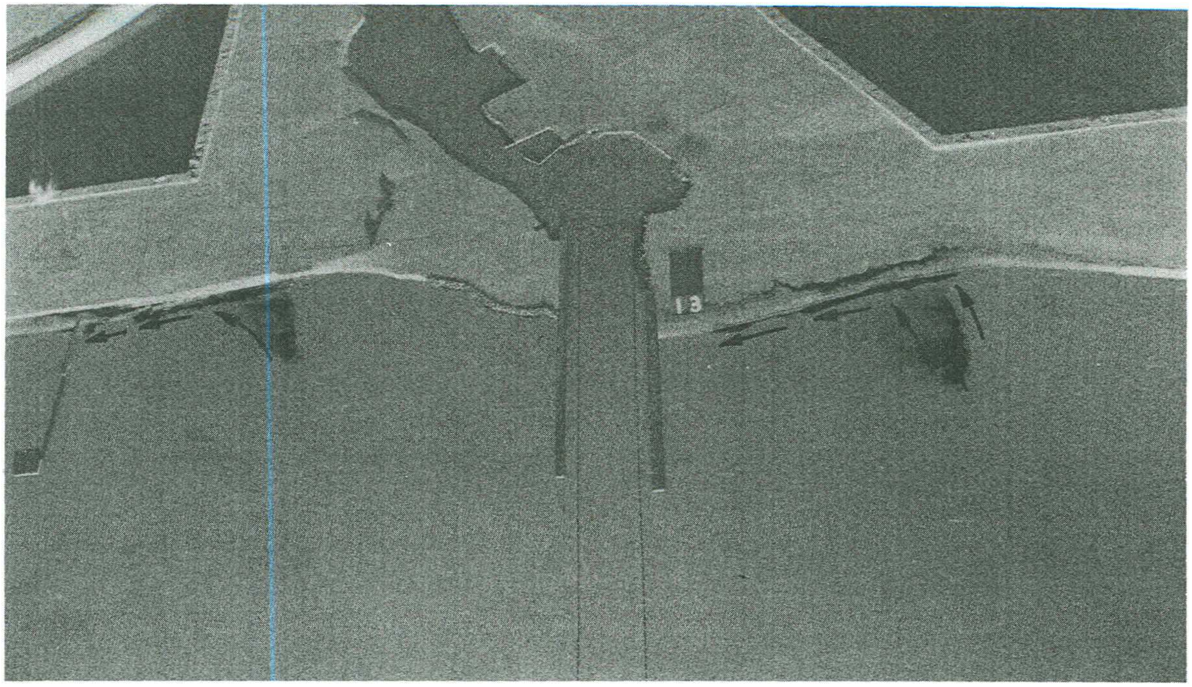
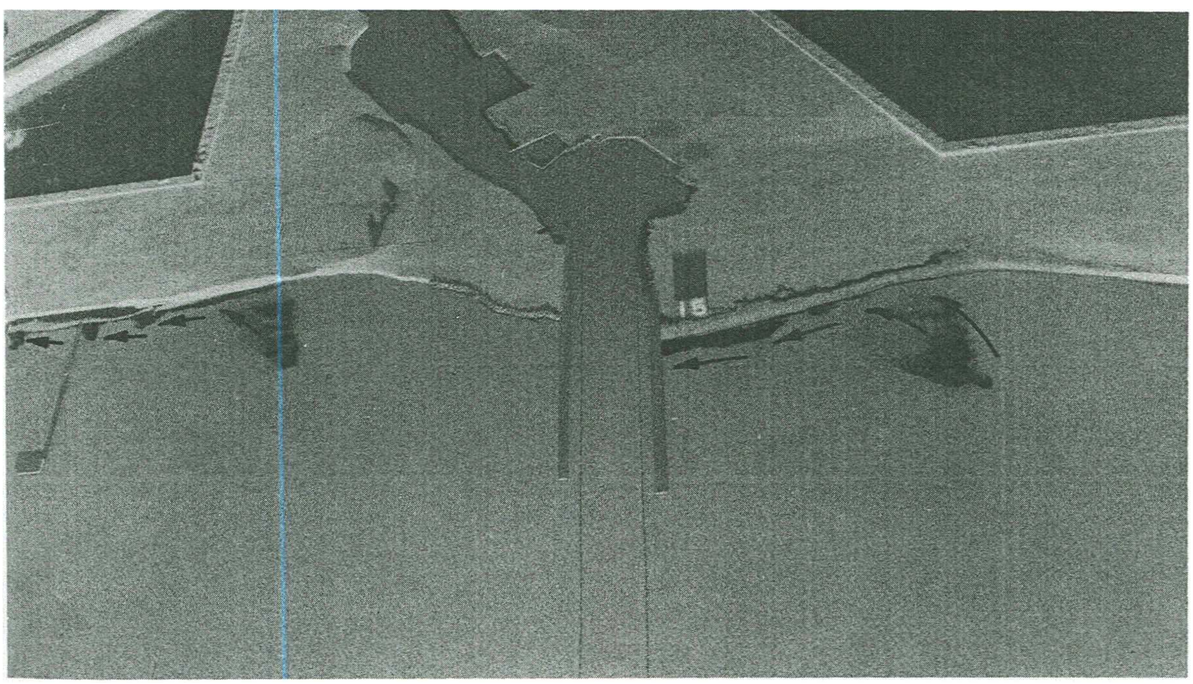
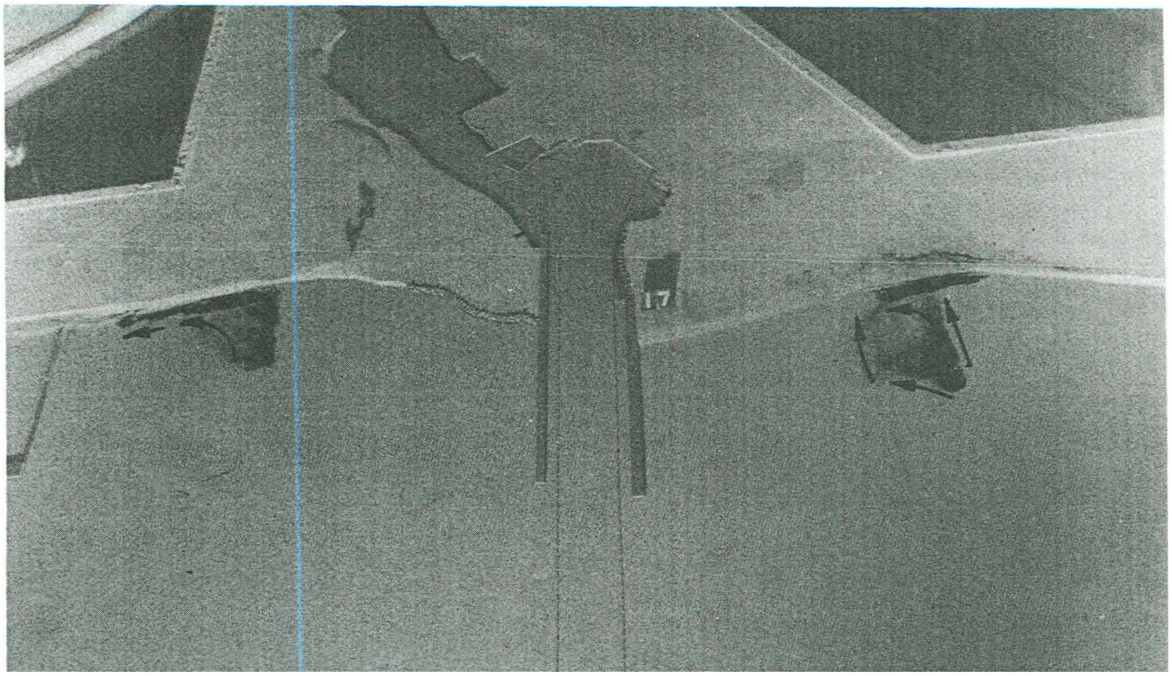


a. 7.4~sec, 8-ft waves; +2.8-ft swl

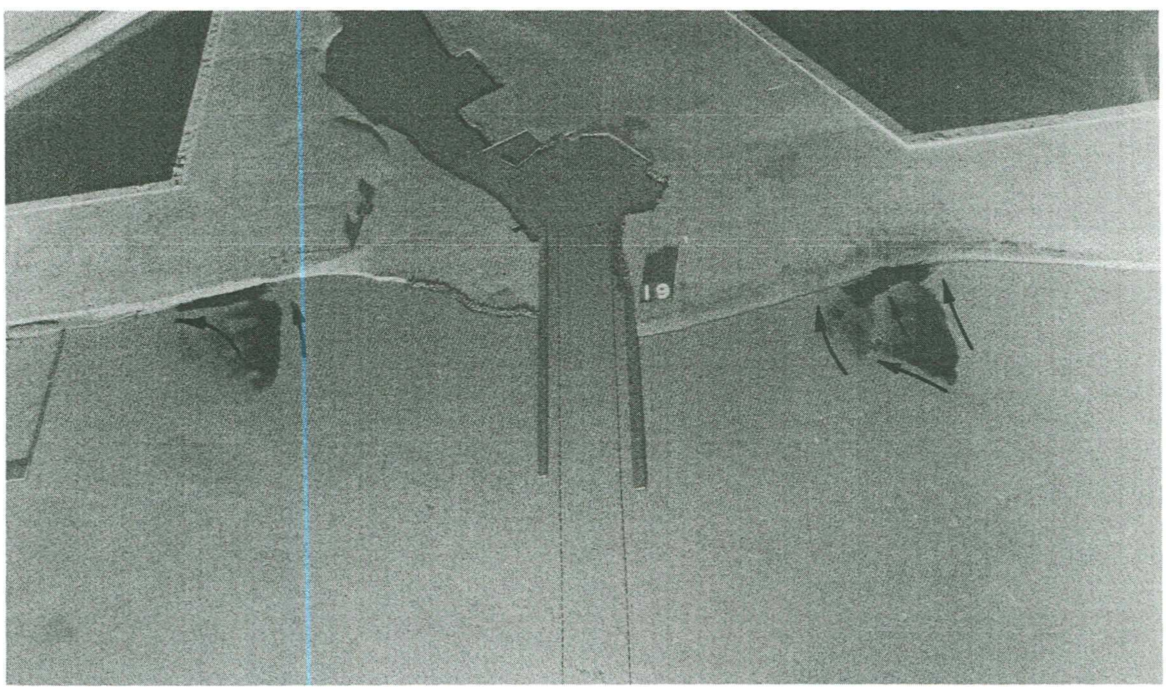


b. 6.4-sec, 6.3-ft waves; +4.0-ft swl

Photo 13. General movement of tracer material and subsequent deposits on each side of the entrance for test waves from 313 deg for existing conditions

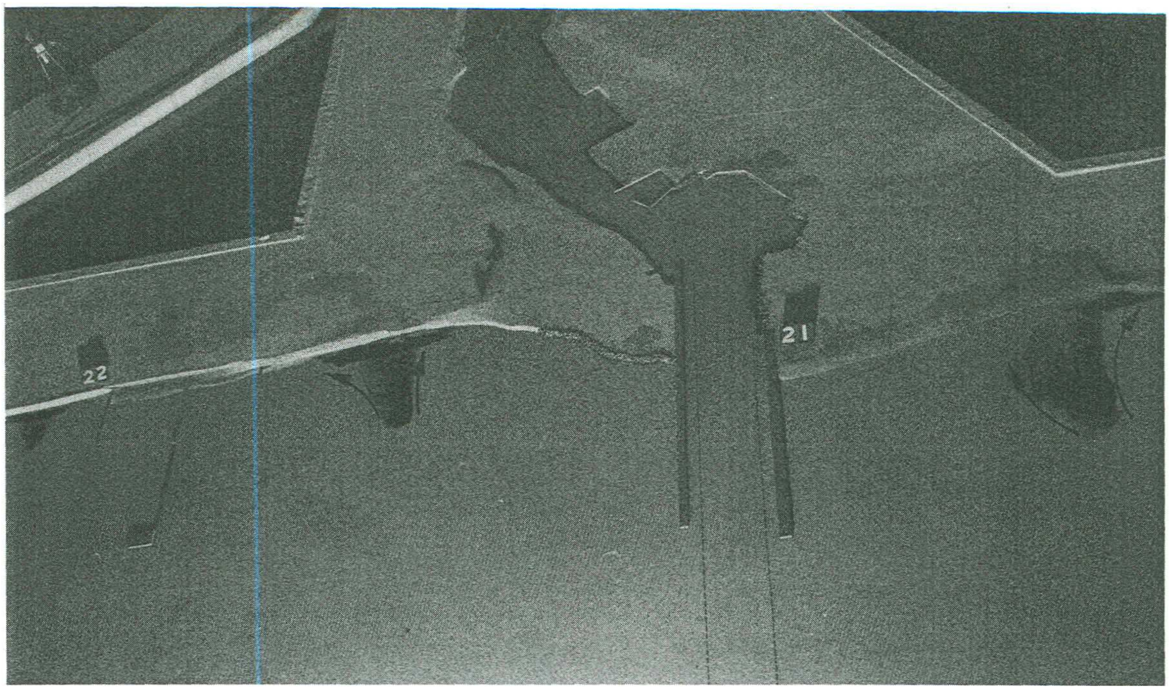


a. 7.4-sec, 8.8-ft waves; +2.8-ft swl

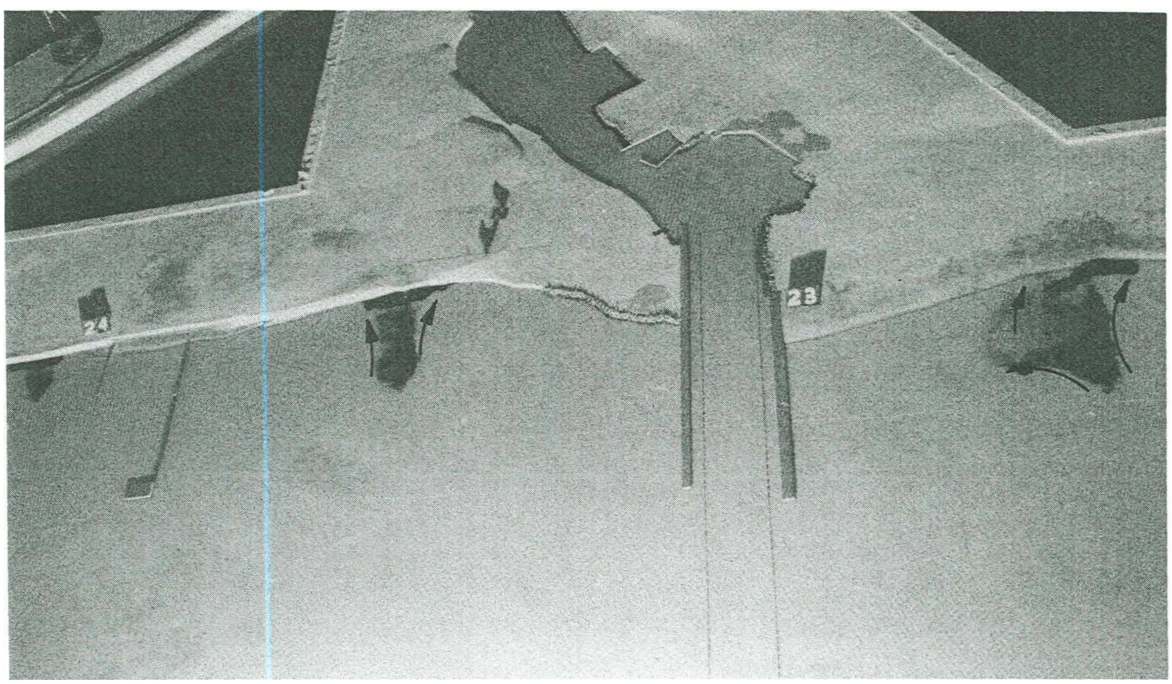


b. 6.4-sec, 6.5-ft waves; +4.0-ft swl

Photo 14. General movement of tracer material and subsequent deposits on each side of the entrance for test waves from 334 deg for existing conditions

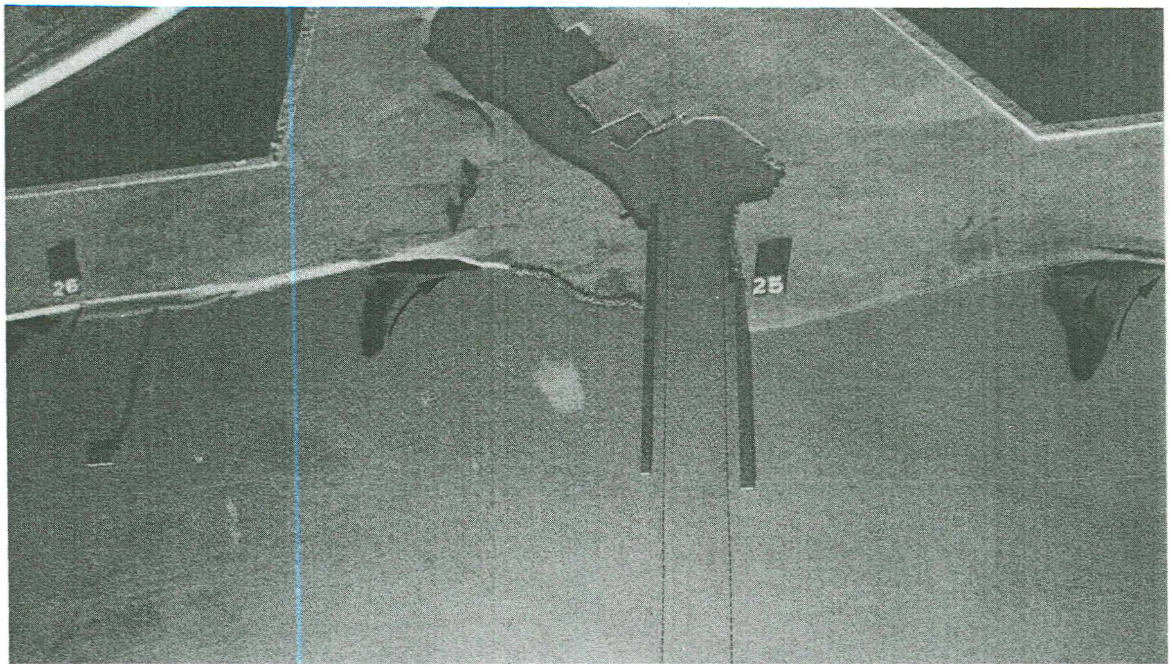


a. 7.4-sec, II.O-ft waves; +2.8-ft swl

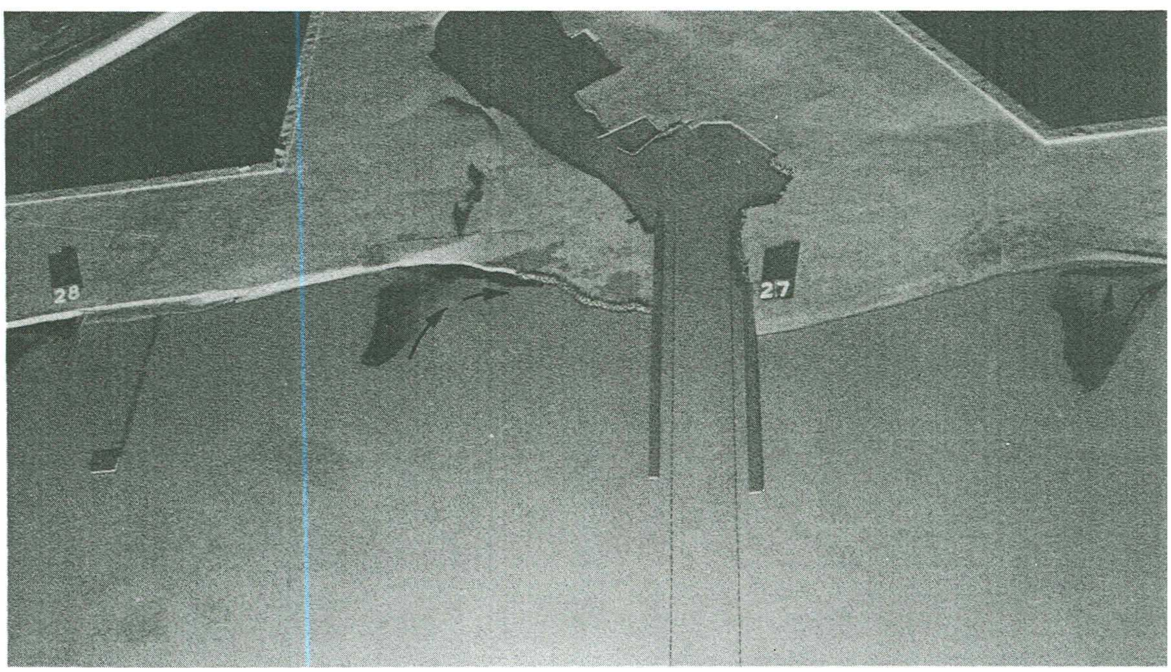


b. s.7-sec, s.8-ft waves; +4.0-ft swl

Photo 15. General movement of tracer material and subsequent deposits on each side of the entrance for test waves from 343 deg for existing conditions

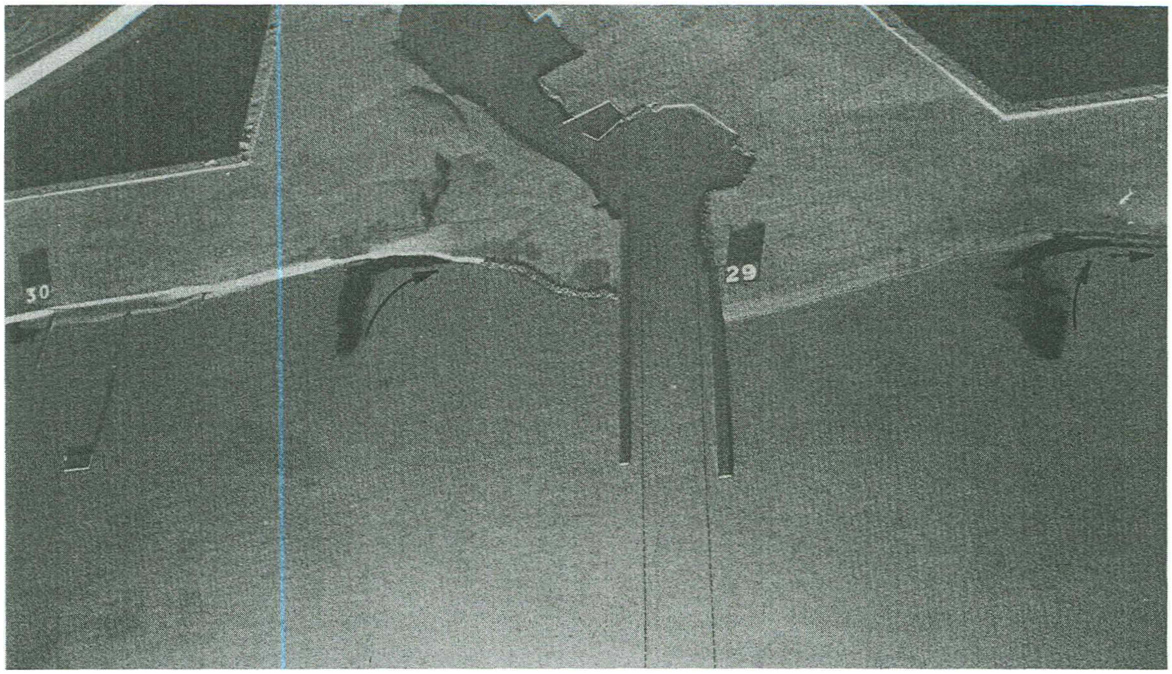


a. 6.9-sec, 7.9-ft waves; +2.8-ft swl



b. S.7-sec, 4.7-ft waves; +4.0-ft swl

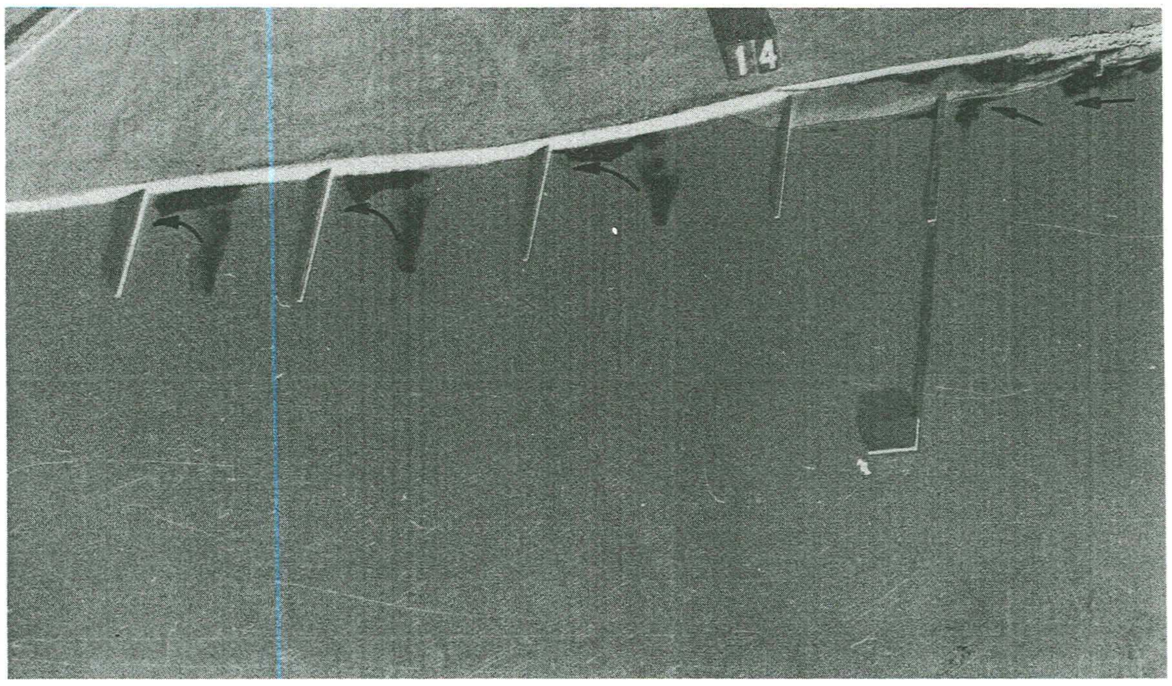
Photo 16. General movement of tracer material and ,subsequent deposits on each side of the entrance for test waves from 24 deg for existing conditions



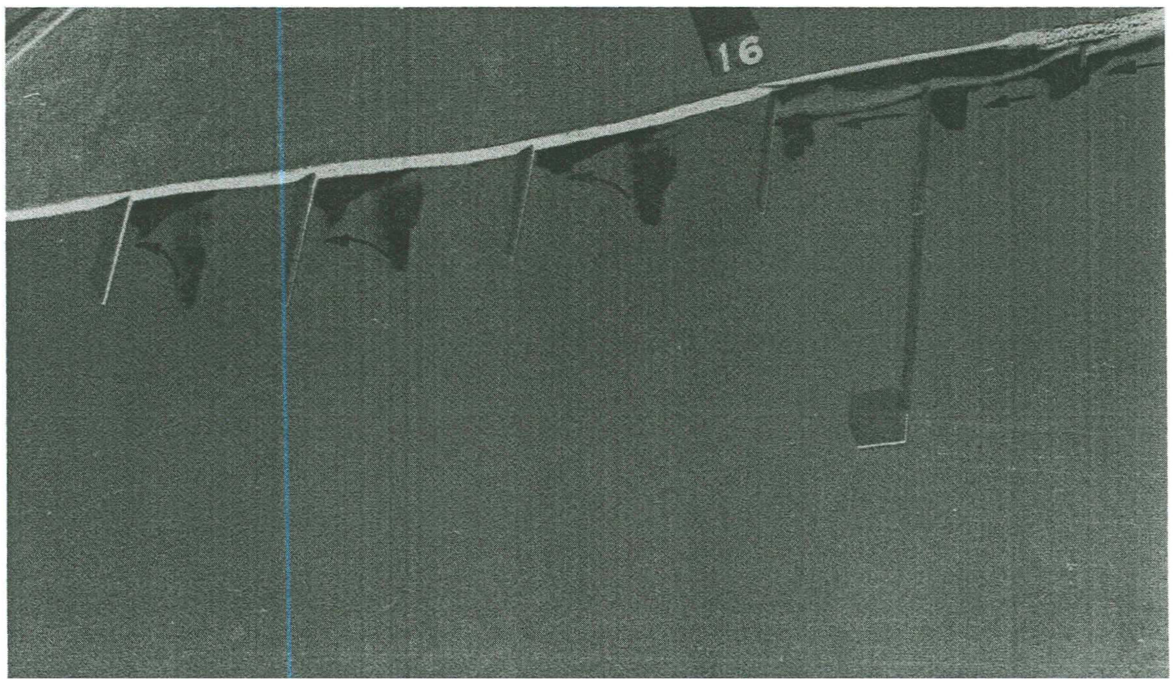
a. 6.9-sec, 6.4-ft waves; +2.8-ft swl

b. S.7-sec, 4.0-ft waves; +4.0-ft swl

Photo 17. General movement of tracer material and subsequent deposits on each side of the entrance for test waves from 42 deg for existing conditions

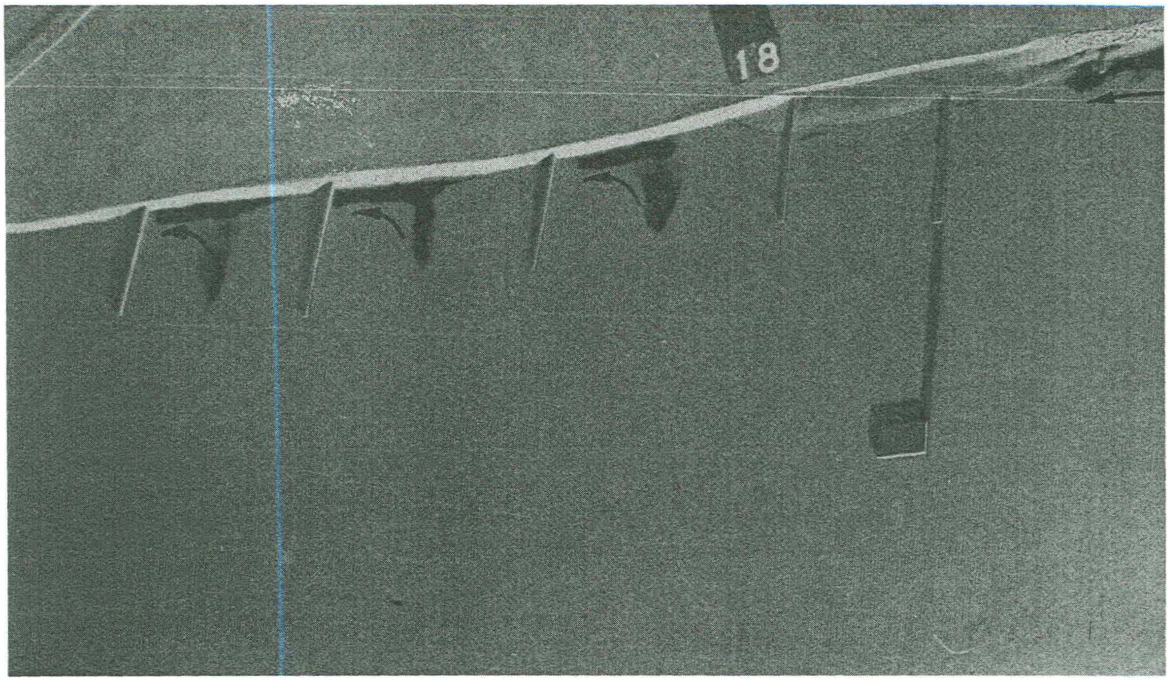


a. 7.4-sec, 8-ft waves; +2.8-ft swl

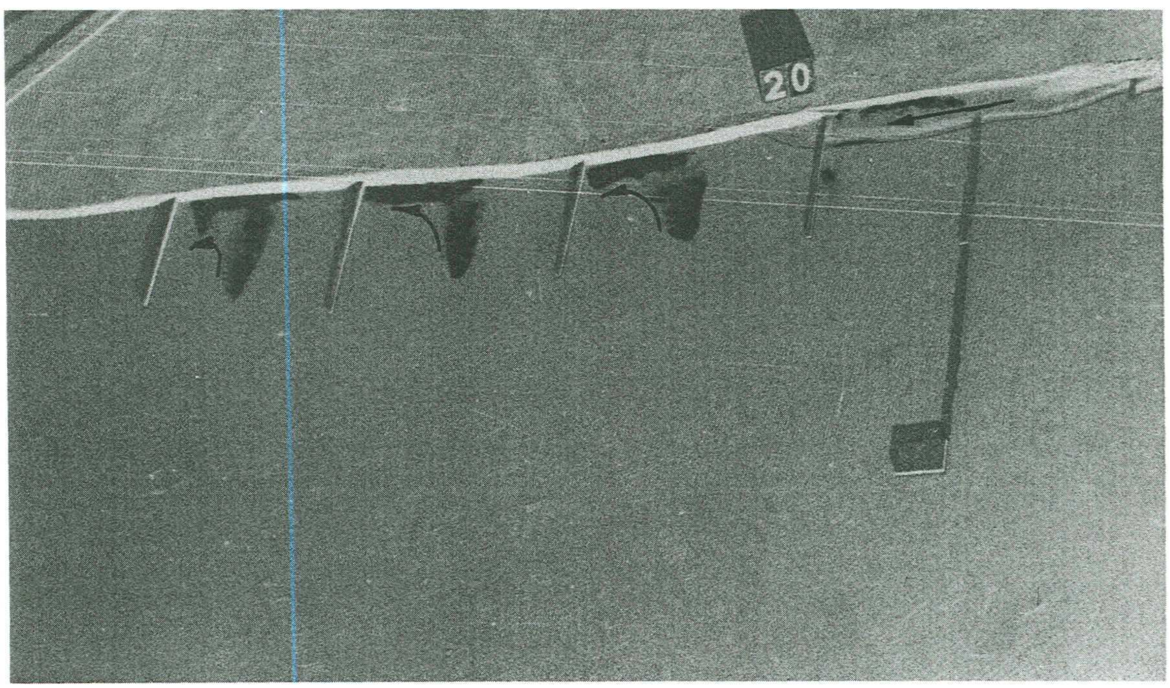


b. 6.4-sec, 6.3-ft waves; +4.0-ft swl

Photo 18. General movement of tracer material and subsequent deposits in the groin field east of the harbor for test waves from 313 deg for existing conditions

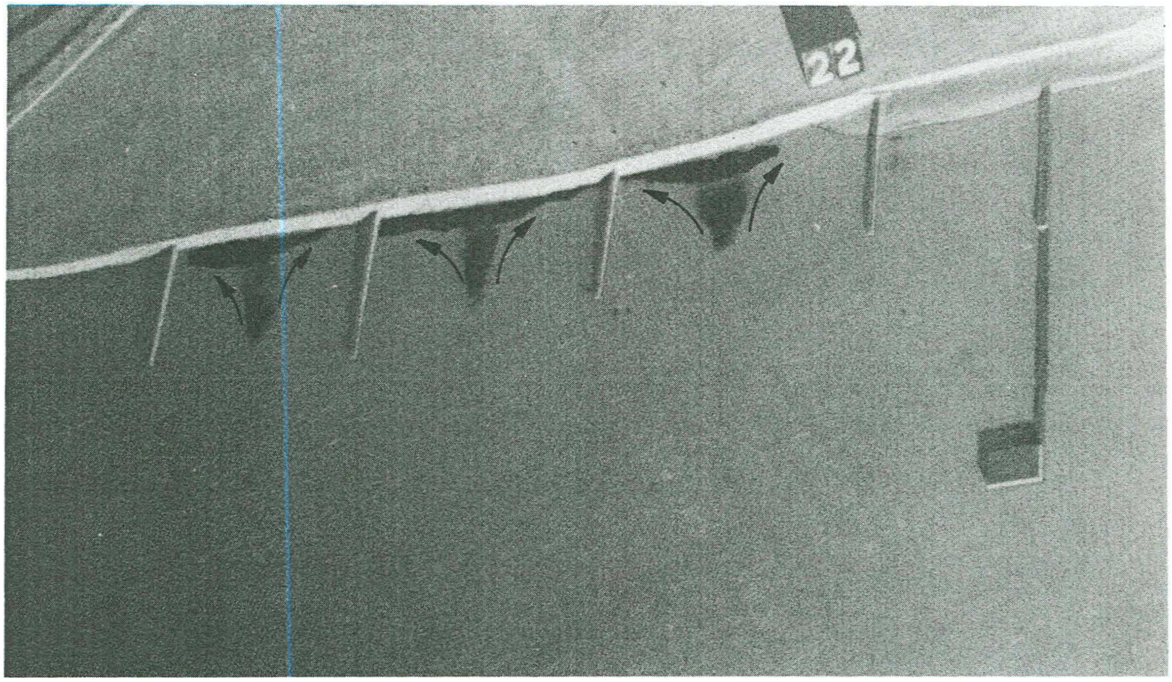


a. 7.4-sec, 8.8-ft waves; +2.8-ft swl

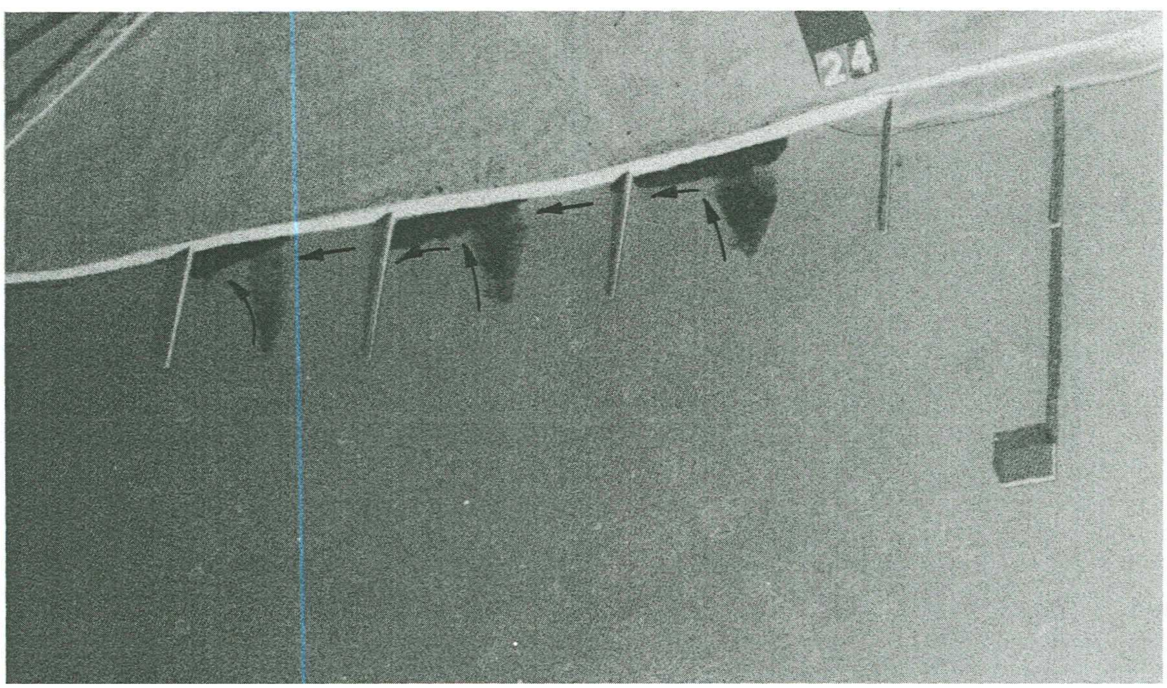


b. 6.4-sec, 6.S-ft waves; +4.0-ft swl

Photo 19. General movement of tracer material and subsequent deposits in the groin field east of the harbor for test waves from 334 deg for existing conditions

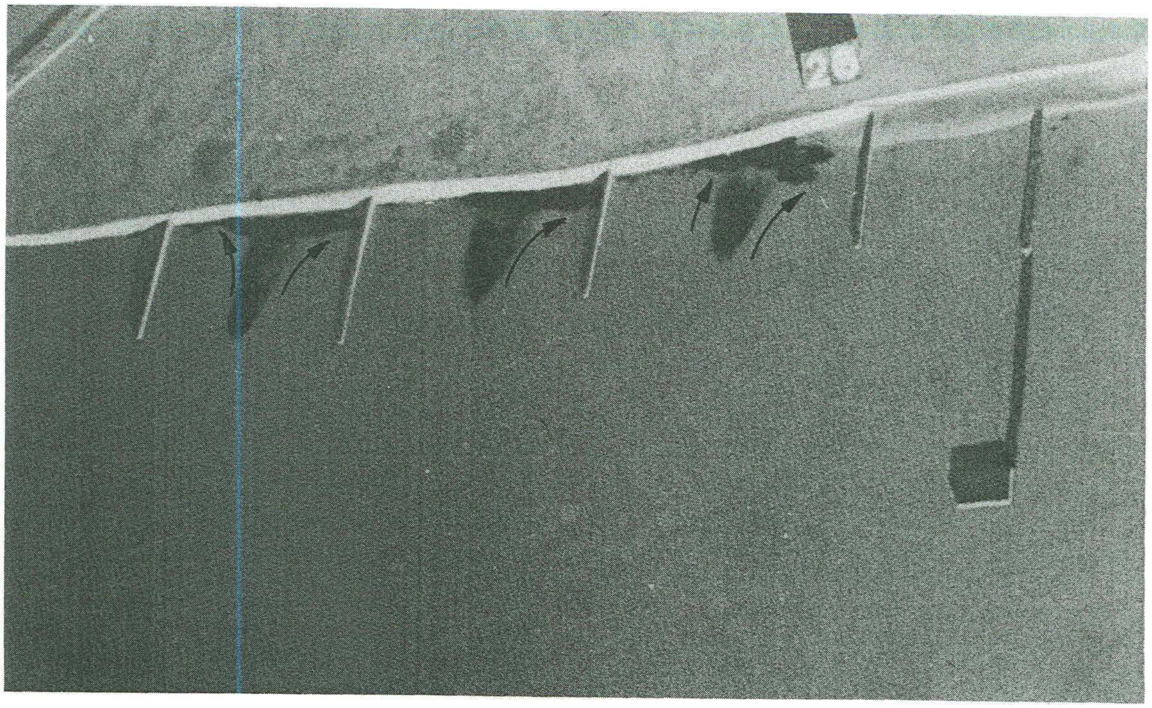


a. 7.4-sec, ll-ft waves; +2.8-ft swl

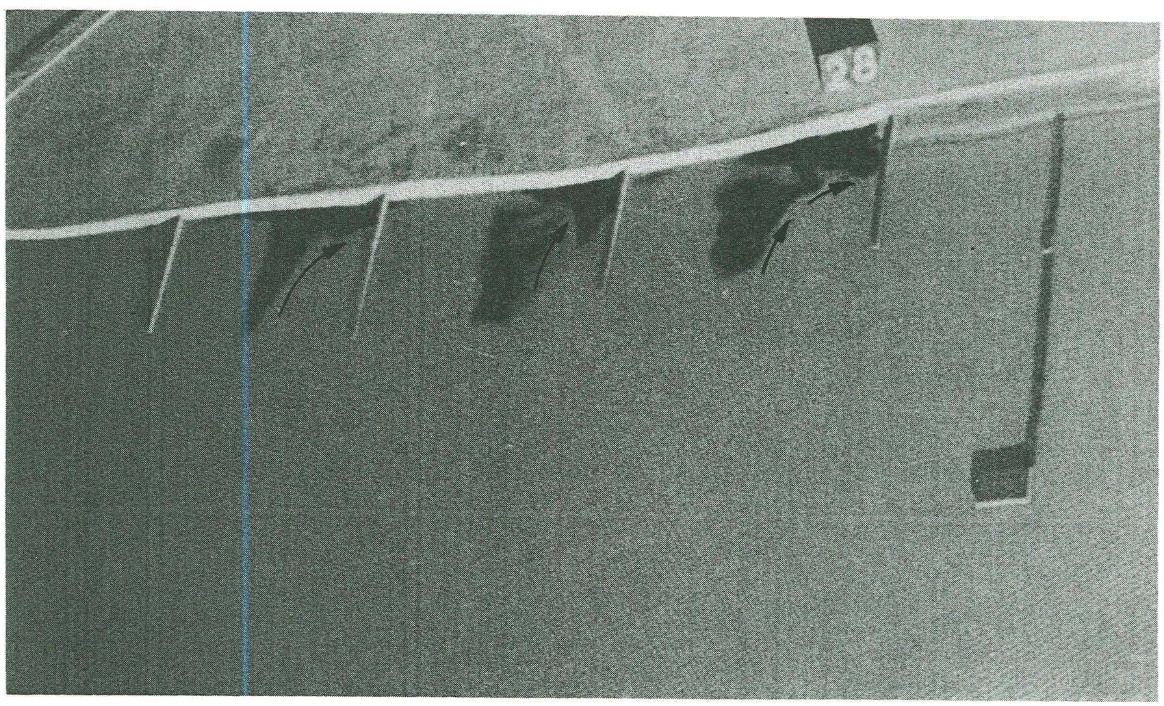


b. s.7-sec, s.8-ft waves; +4.0-ft swl

Photo 20. General movement of tracer material and subsequent deposits in the groin field east of the harbor for test waves from 343 deg for existing conditions

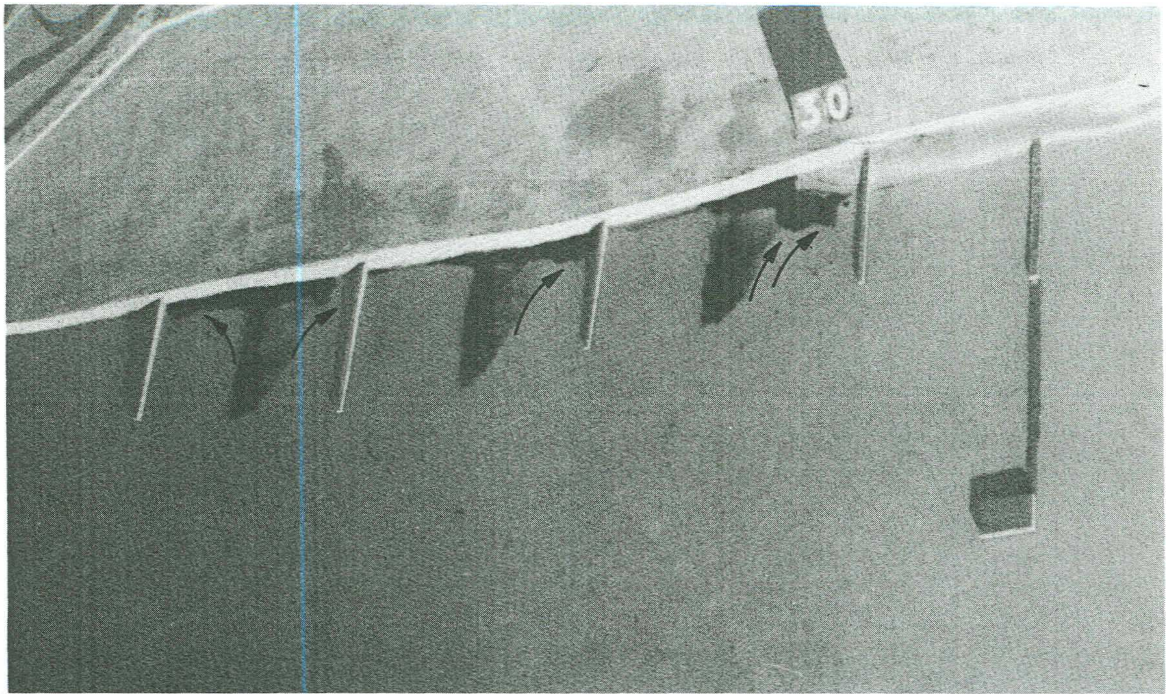


a. 6.9-sec, 7.9-ft waves; +2.8-ft swl

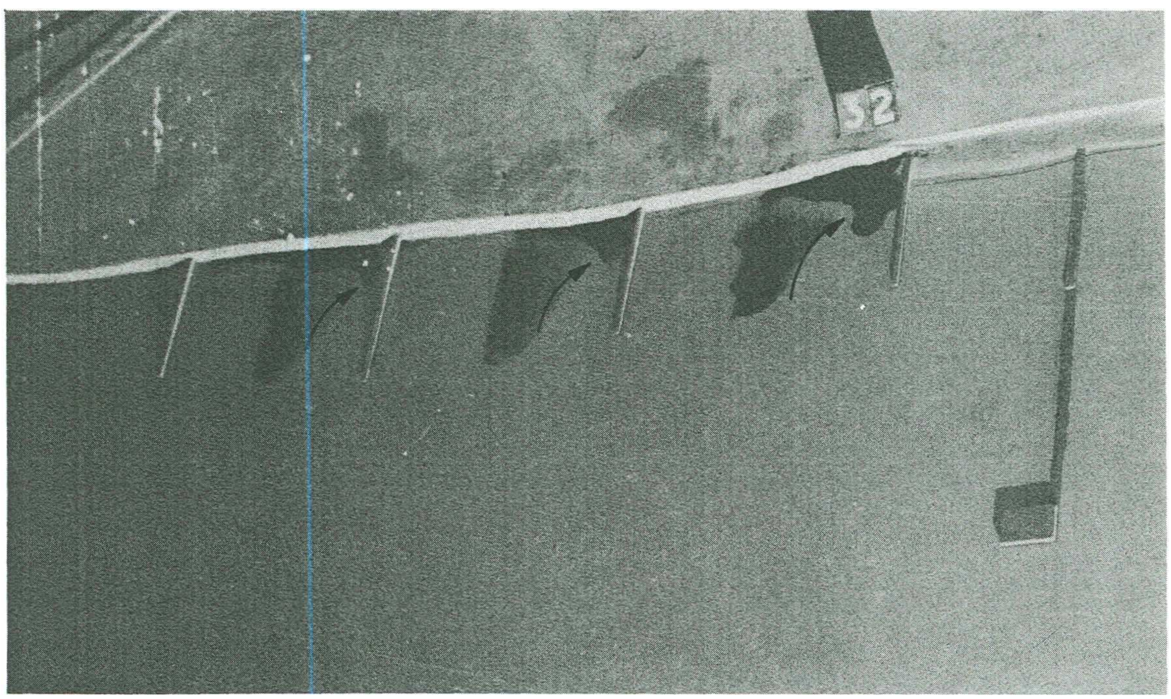


b. S.7-sec, 4.7-ft waves; +4.0-ft swl

Photo 21. General movement of tracer material and subsequent deposits in the groin field east of the harbor for test waves from 24 deg for existing conditions



a. 6.9-sec, 6.4-ft waves; +2.8-ft swl



b. 5.7-sec, 4.0-ft waves; +4.0-ft swl

Photo 22. General movement of tracer material and subsequent deposits in the groin field east of the harbor for test waves from 42 deg for existing conditions

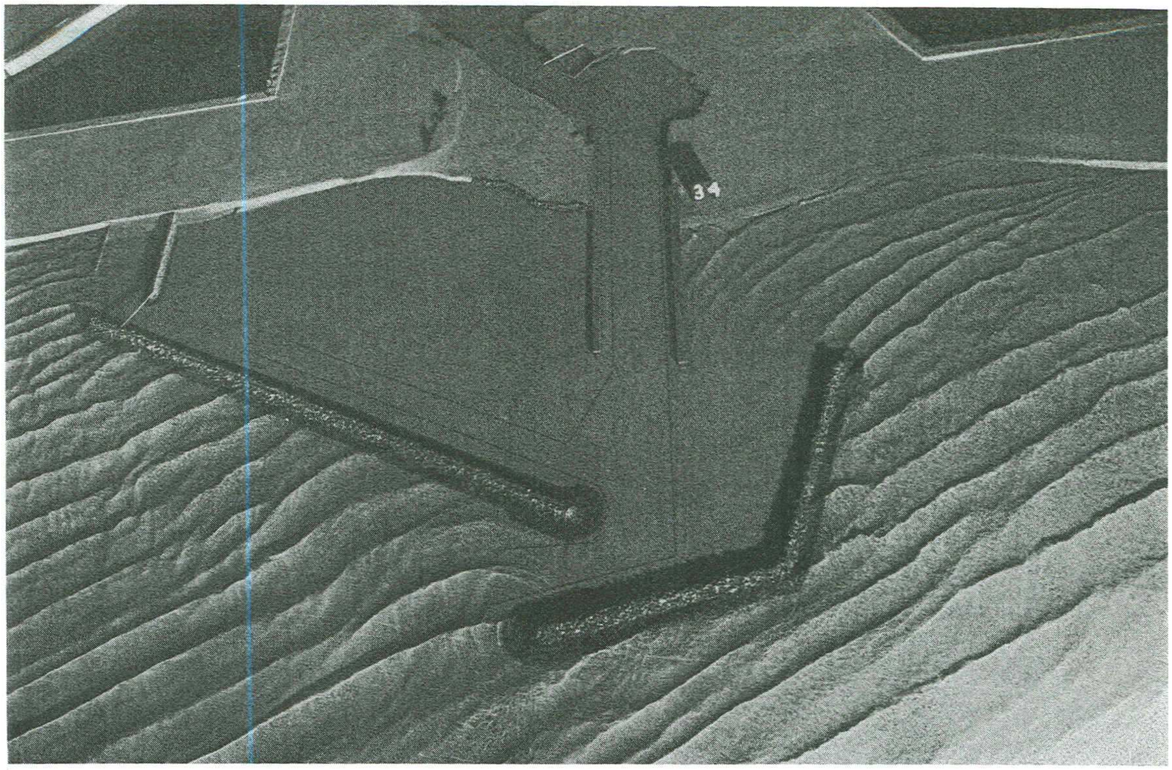


Photo 23. Typical wave patterns for Plan 1; 7.2-sec, 7.6-ft waves approaching from 313 deg; +2.8-ft swl

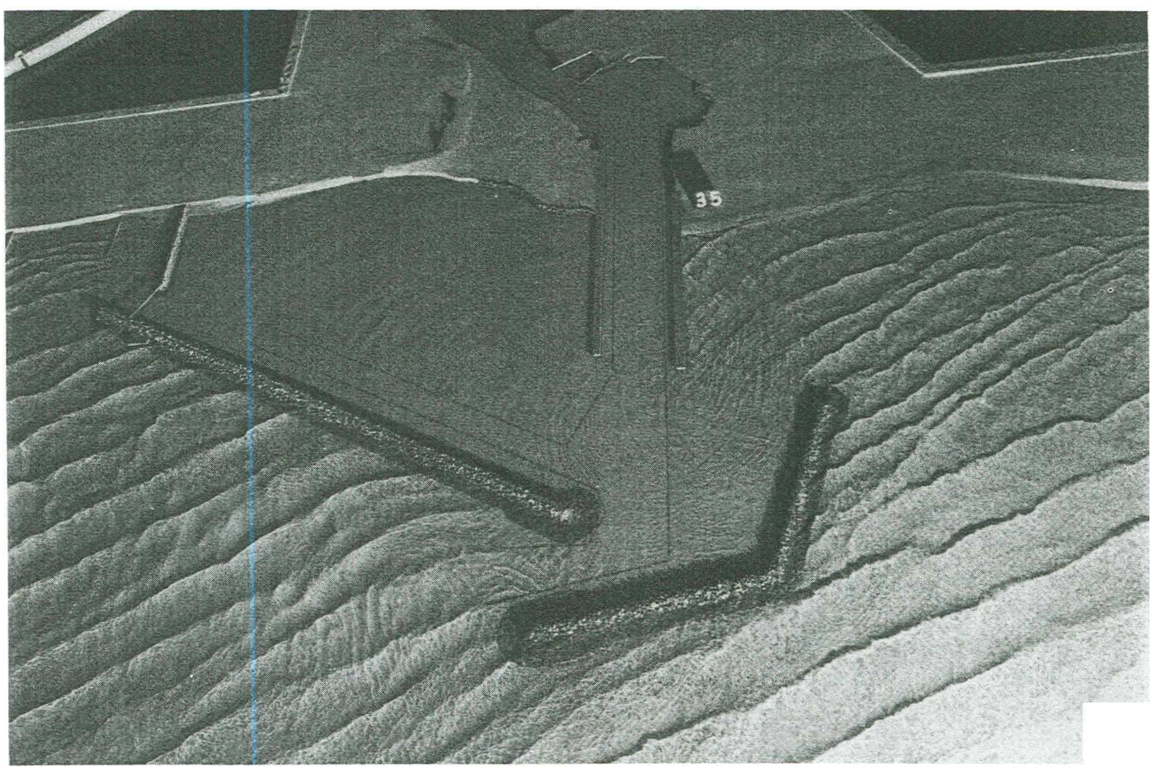


Photo 24. Typical wave patterns for Plan 2; 7.2-sec, 7.6-ft waves approaching from 313 deg; +2.8-ft swl

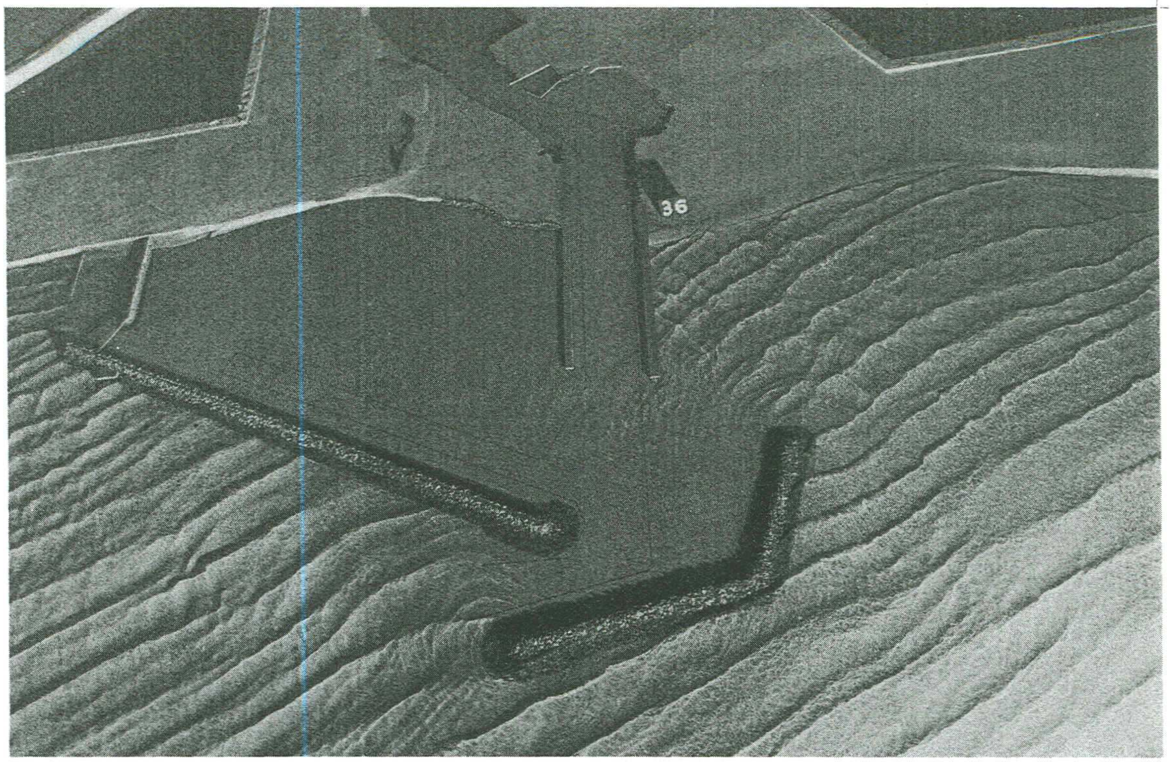


Photo 25. Typical wave patterns for Plan 3; 7.2-sec, 7.6-ft waves approaching from 313 deg; +2. 8-ft swl

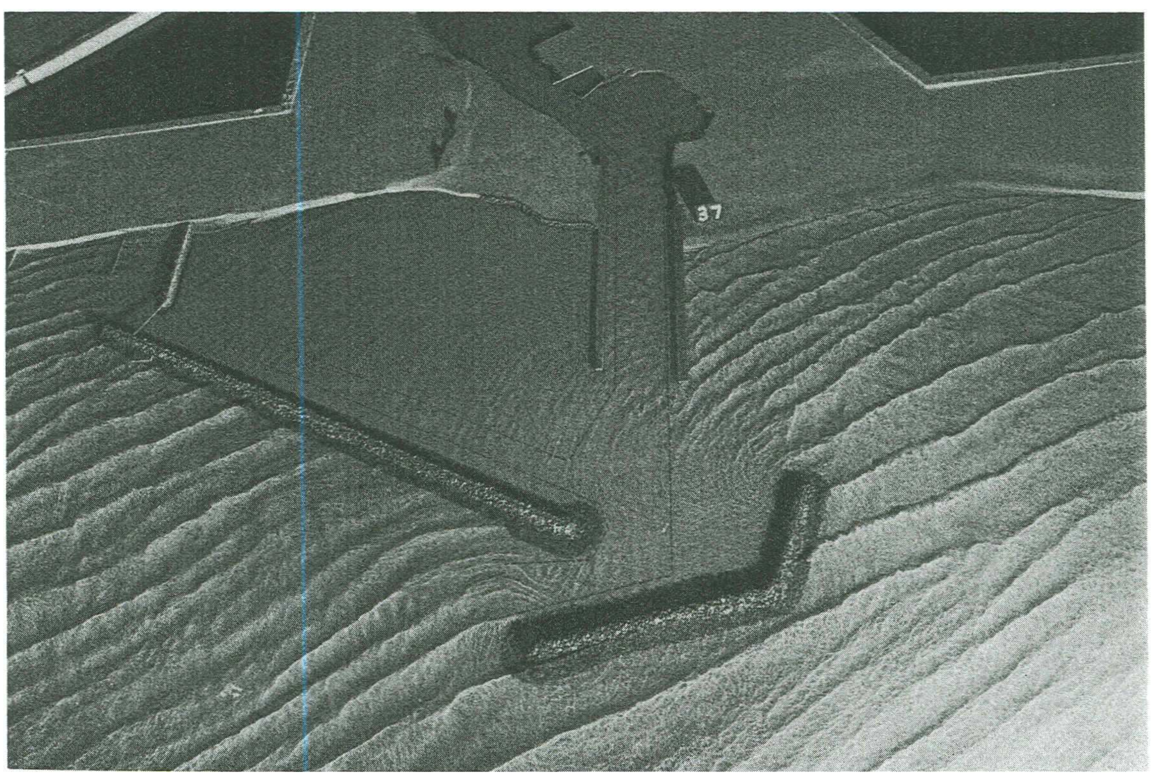


Photo 26. Typical wave patterns for Plan 4; 7.2-sec, 7.6-ft waves approaching from 313 deg; +2.8-ft swl

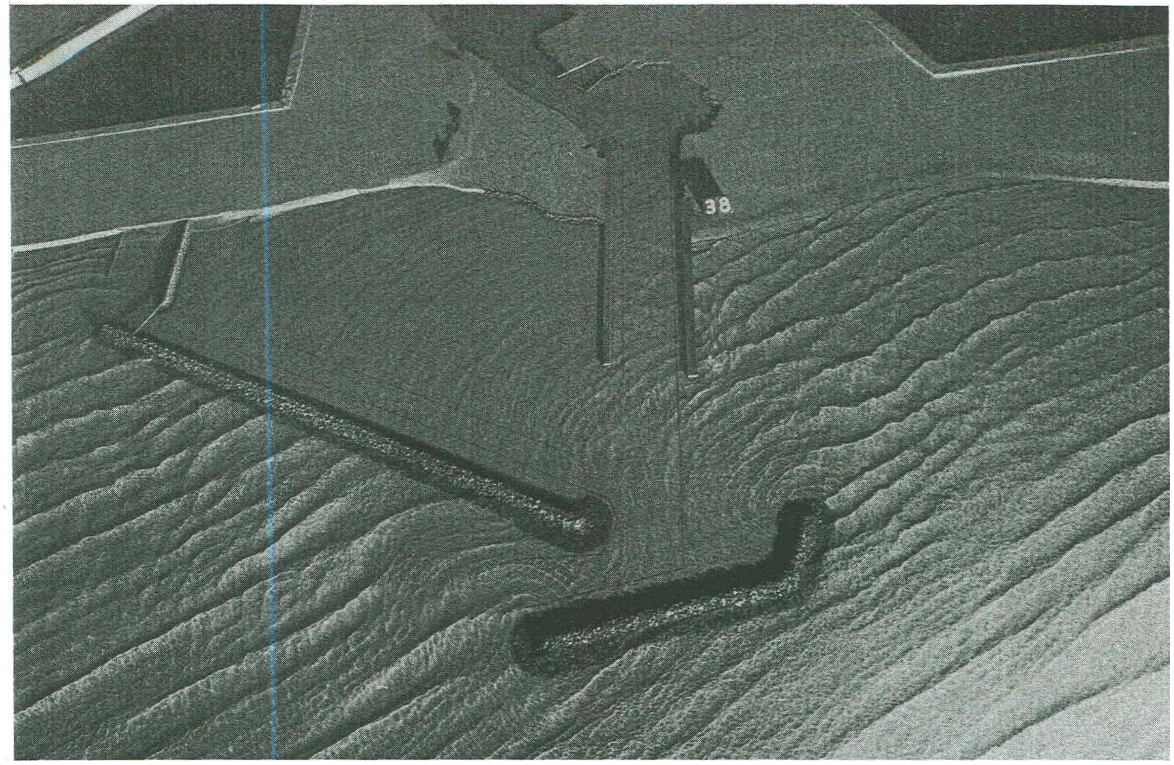


Photo 27. Typical wave patterns for Plan 5; 7.2-sec, 7.6-ft waves approaching from 313 deg; +2.8-ft swl

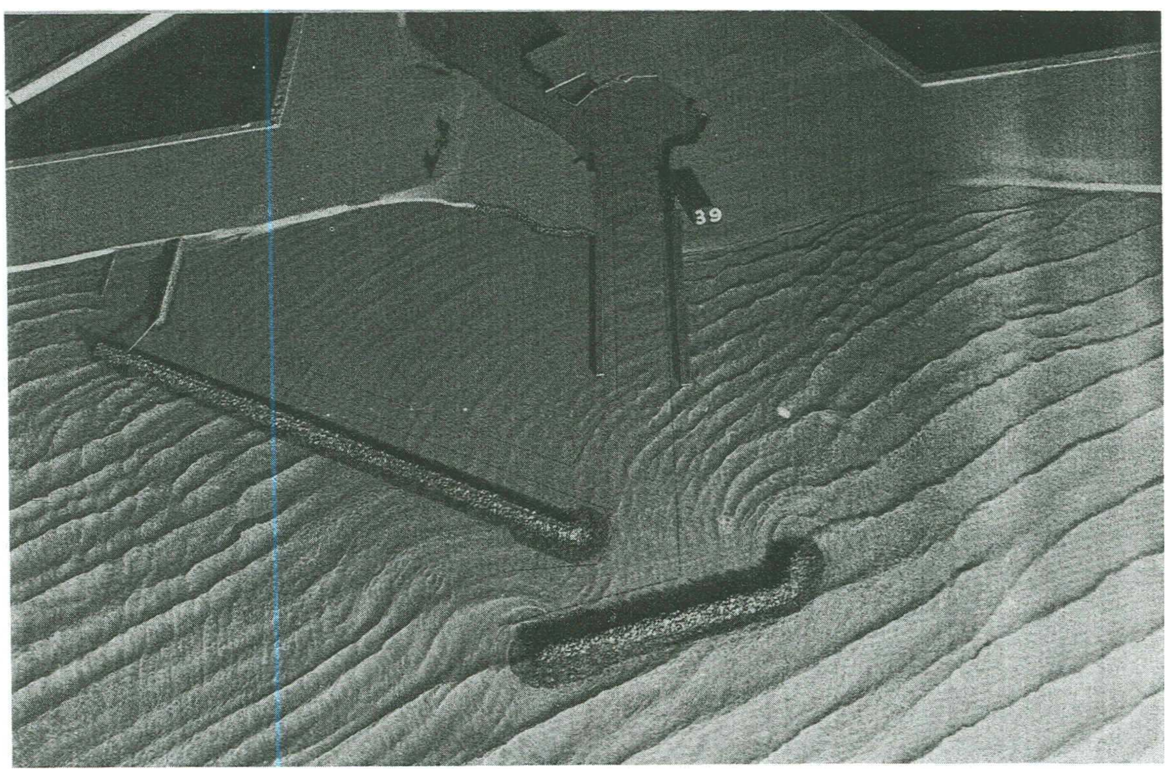


Photo 28. Typical wave patterns for Plan 6; 7.2-sec, 7.6-ft waves approaching from 313 deg; +2.8-ft swl

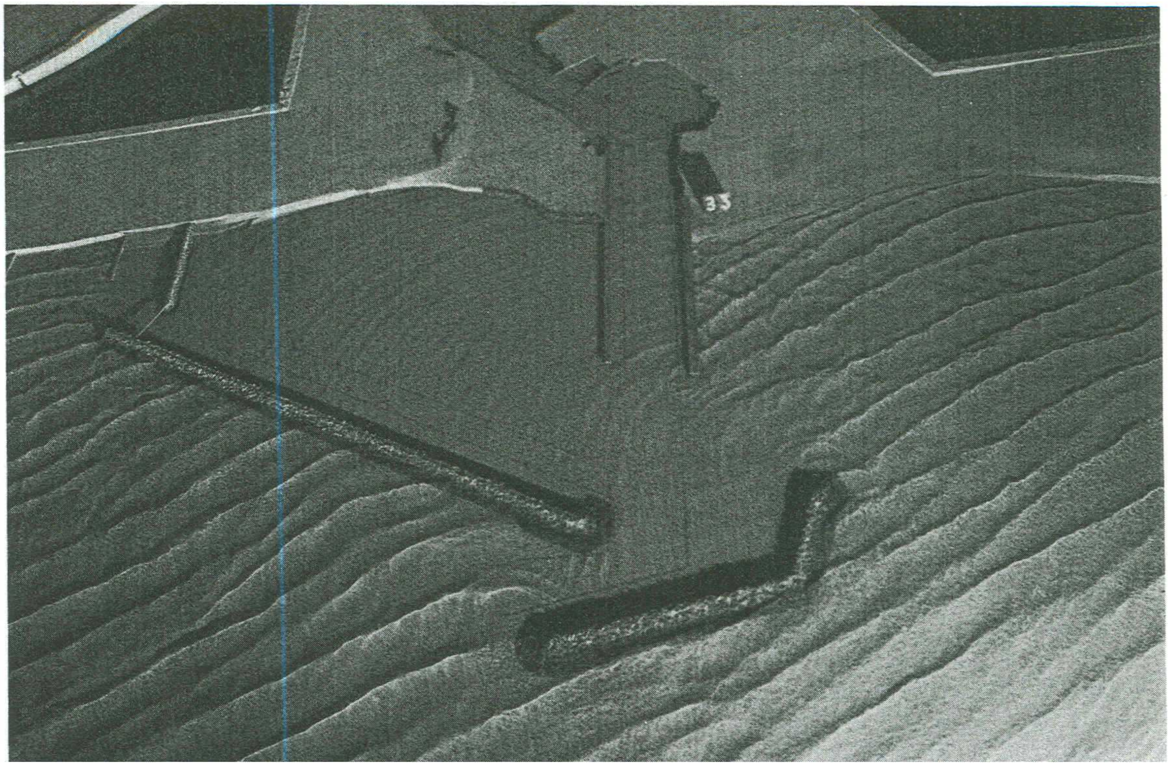
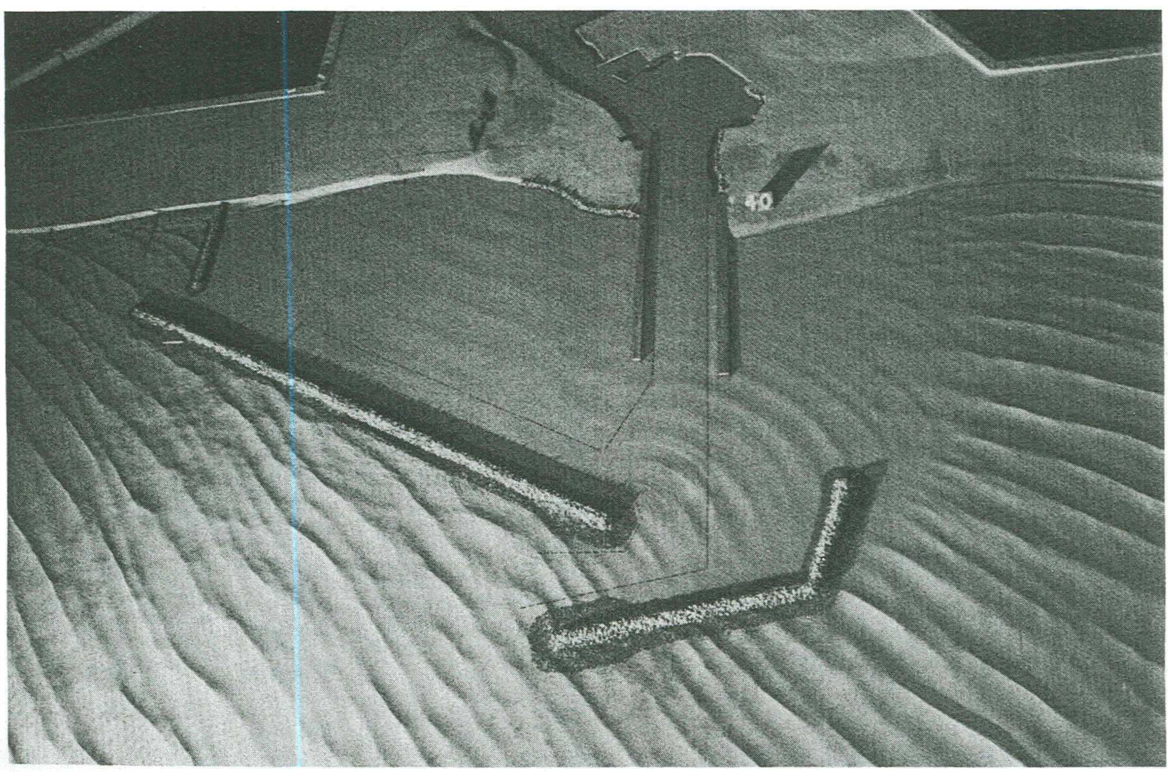


Photo 29. Typical wave patterns for Plan 7; 7.2-sec, 7.6-ft waves approaching from 313 deg; +2.8-ft swl



Photo' 30. Typical wave patterns for Plan 9; 6.4-sec, S.8-ft waves approaching from 42 deg; +4.0-ft swl

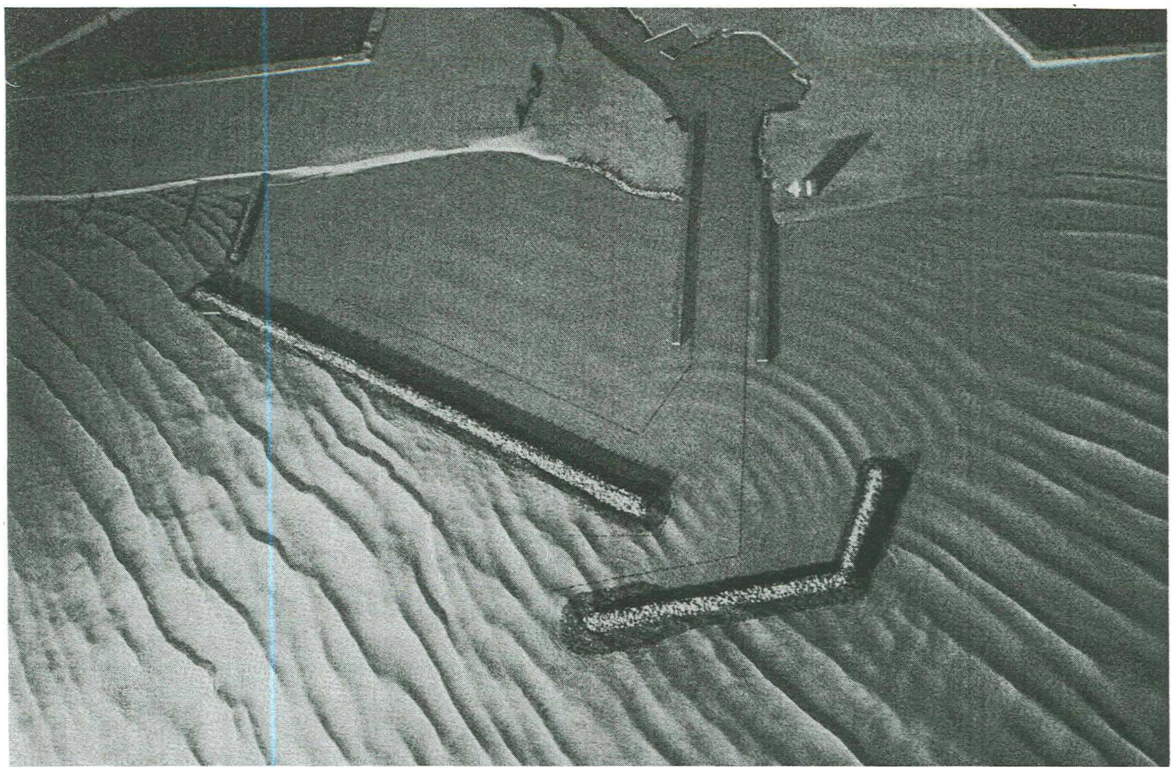


Photo 31. Typical wave patterns for Plan 10;,6.4-sec, 5.B-ft waves approaching from 42 deg; +4.0-ft swl

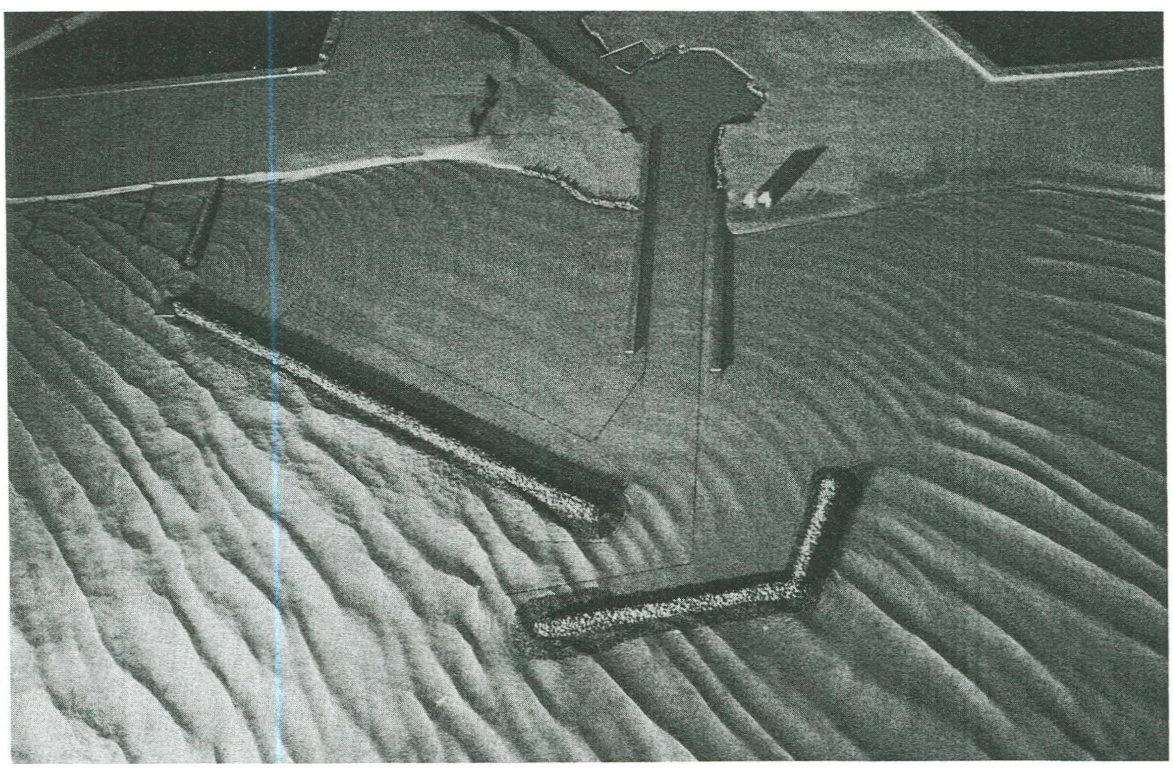


Photo 32. Typical wave patterns for Plan 11; 6.4-sec, 5.B-ft waves approaching from 42 deg; +4.0-ft swl

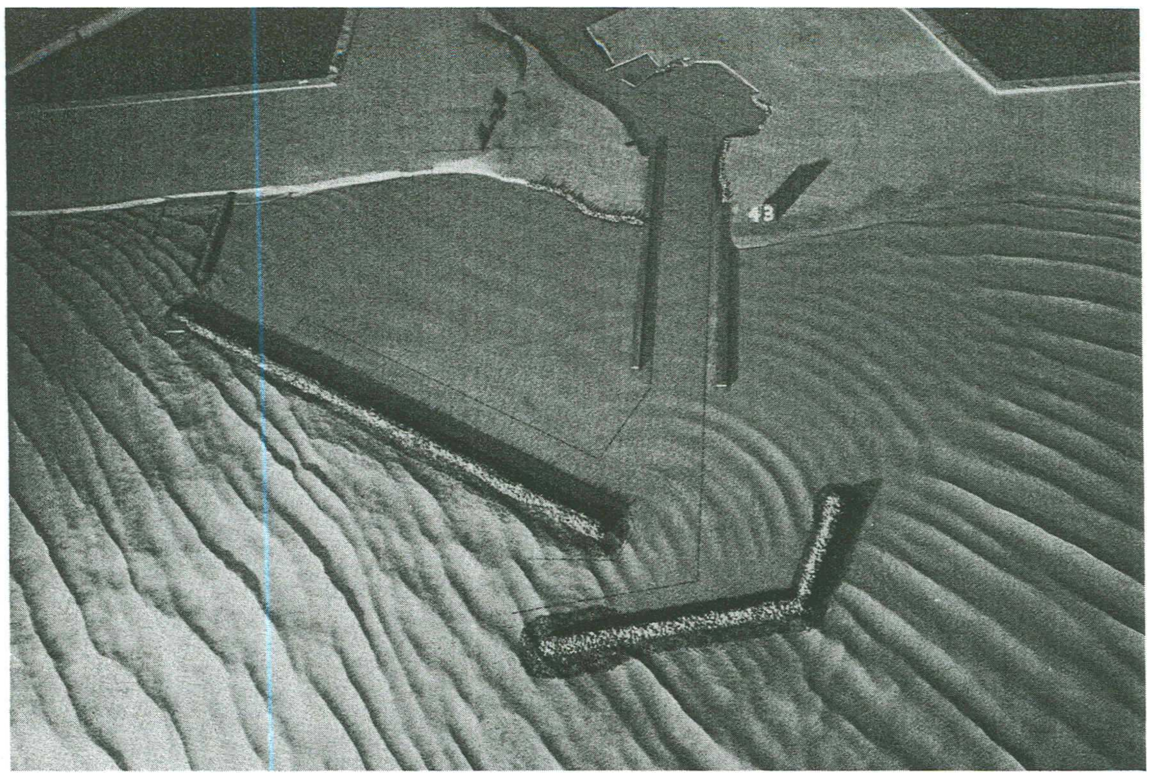


Photo 33. Typical wave patterns for Plan 12; 6.4-sec, 5.B-ft waves approaching from 42 deg; +4.0-ft swl

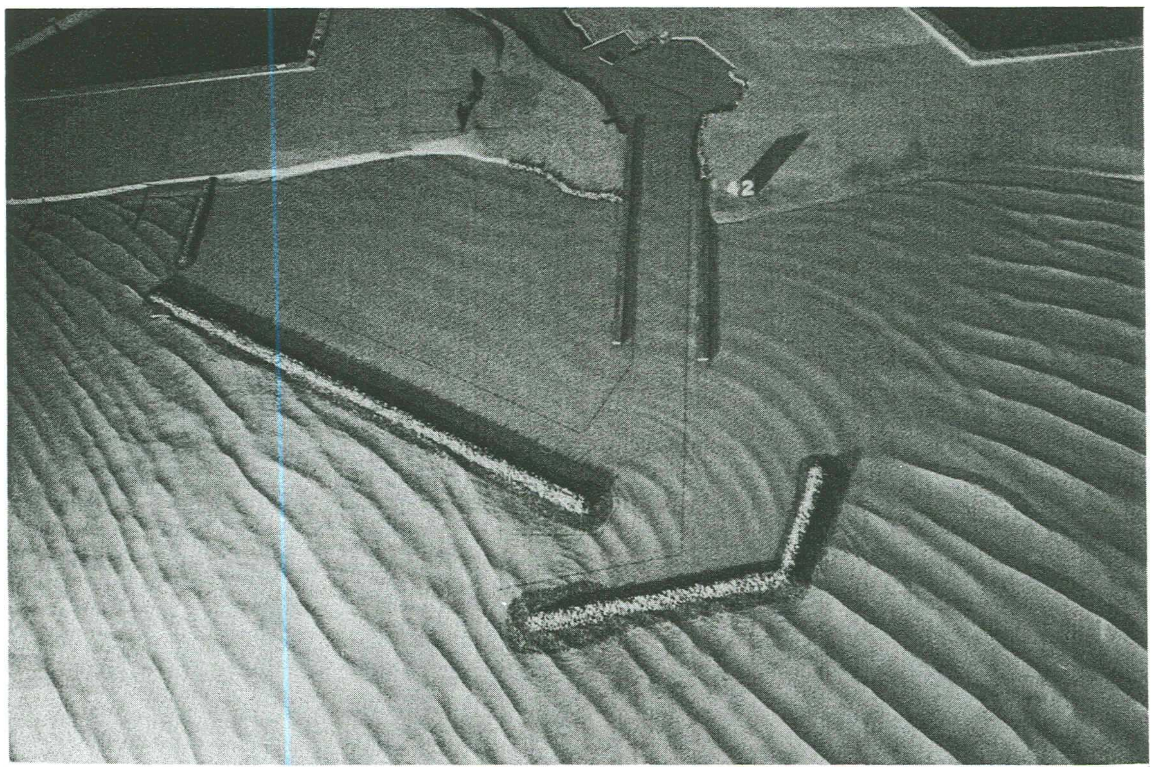


Photo 34. Typical wave patterns for Plan 13; 6.4-sec, 5.B-ft waves approaching from 42 deg; +4.0-ft swl

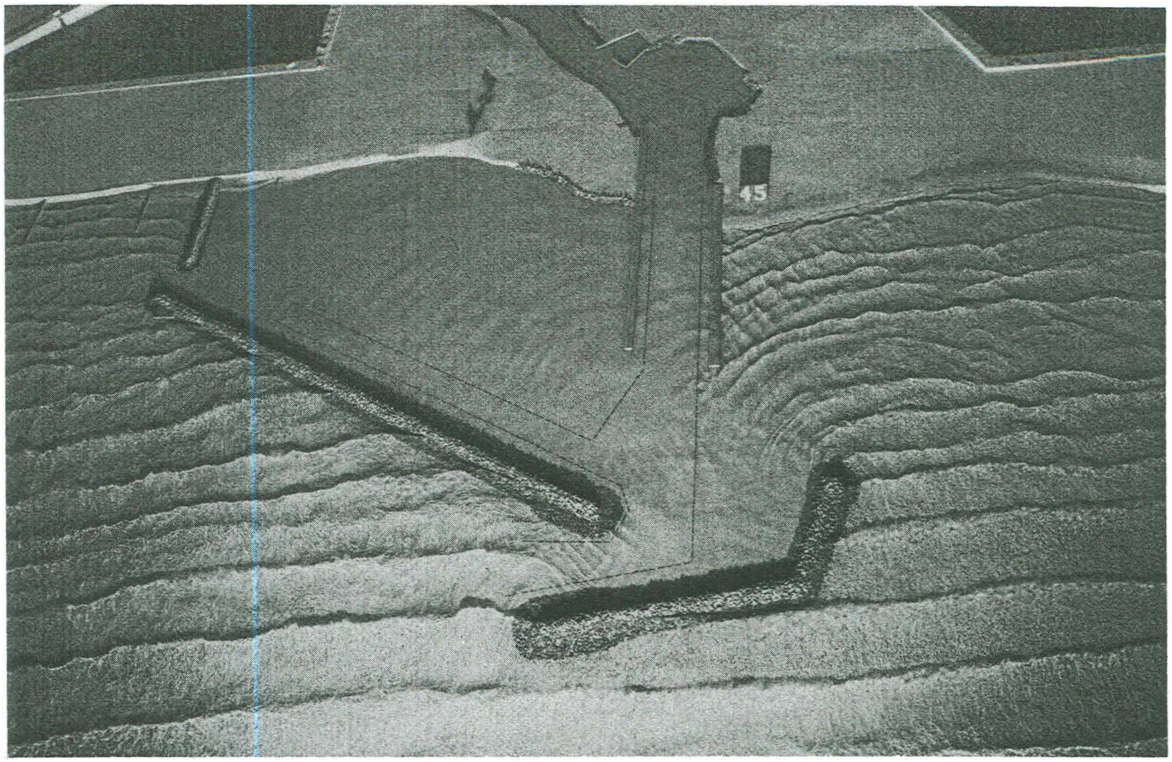


Photo 35. Typical wave patterns for Plan 13; 7-sec, 9.9-ft waves approaching from 343 deg; +2.8-ft swl

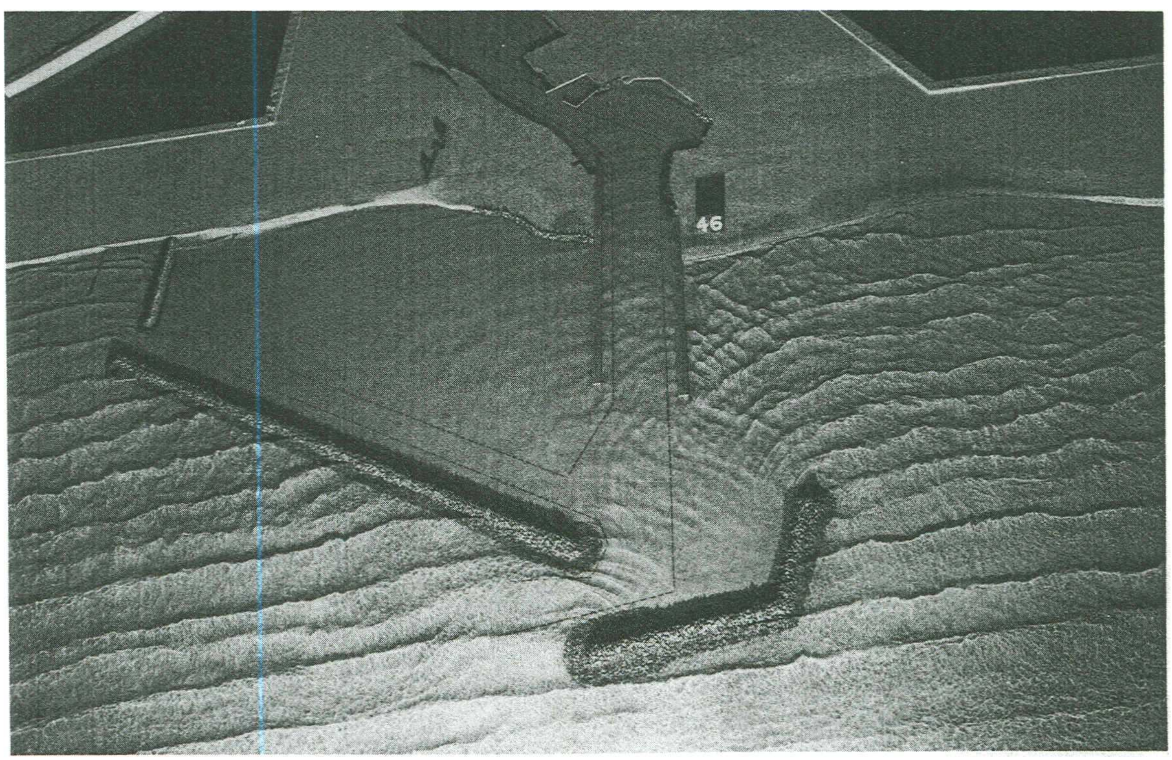


Photo 36. Typical wave patterns for Plan 14; 7-sec, 9.9-ft waves approaching from 343 deg; +2.8-ft swl

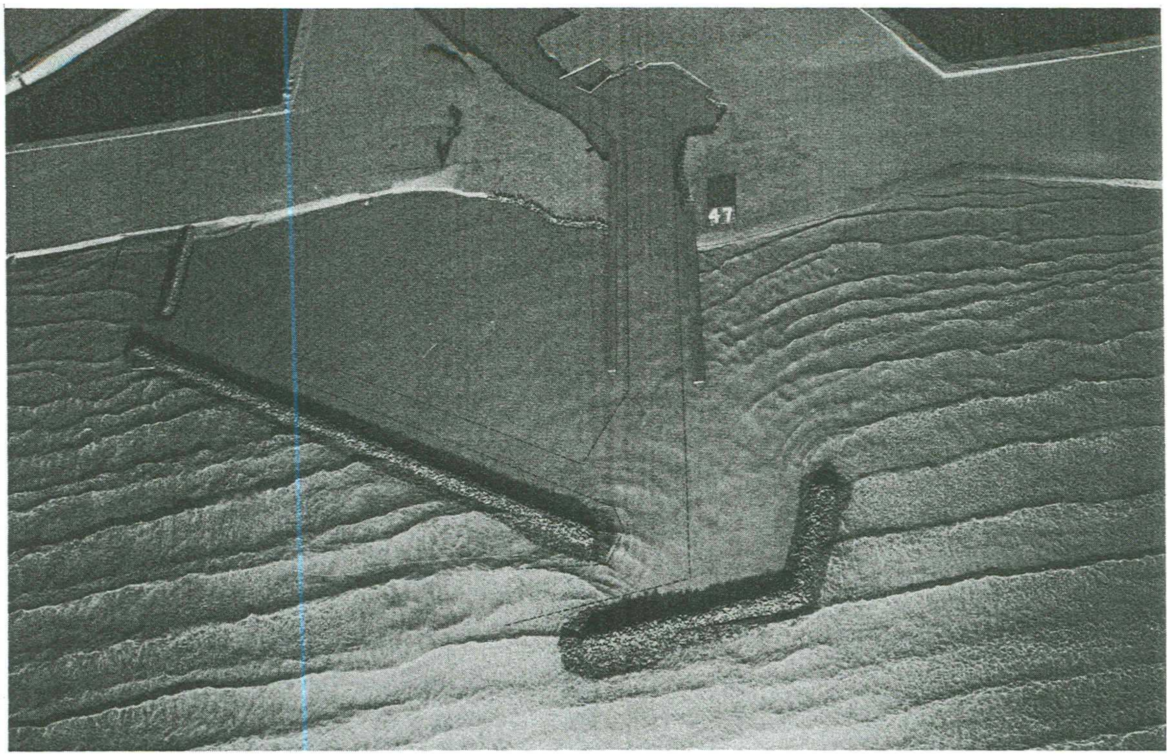
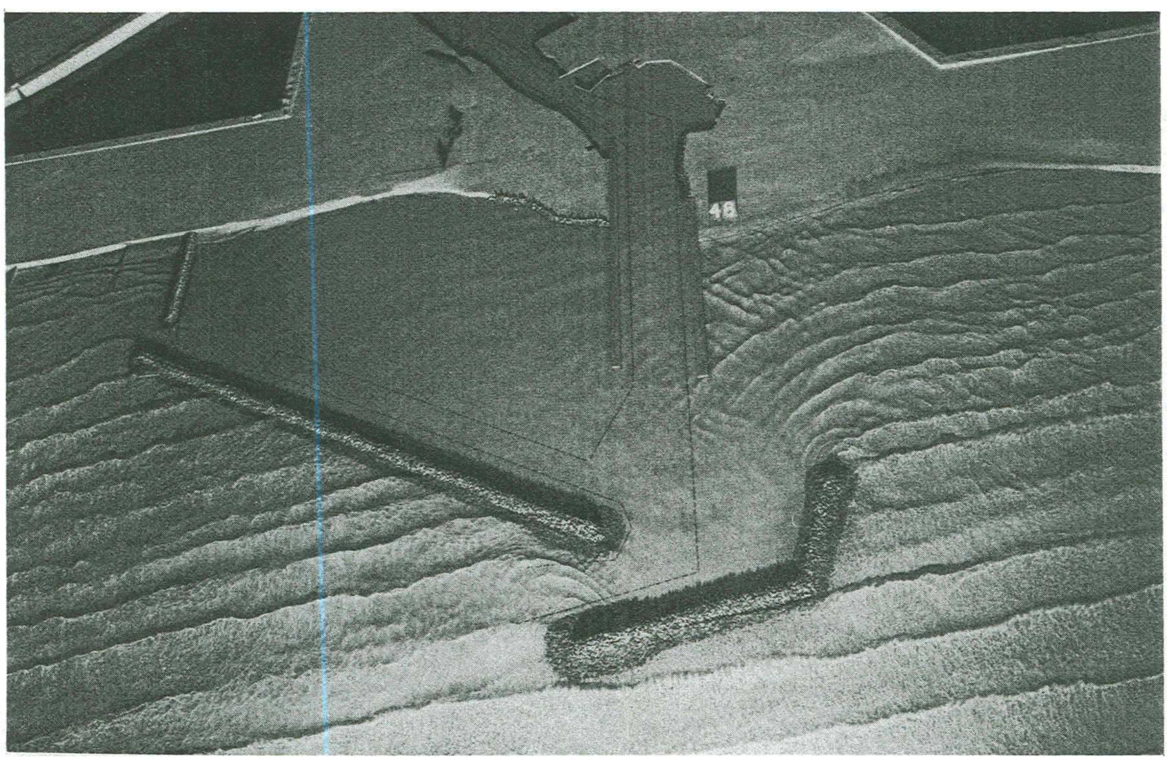


Photo 37. Typical wave patterns for Plan 15; 7-sec, 9.9-ft waves approaching from 343 deg; +2.8-ft swl



P~lOto 38. Typical wave patterns for Plan 16; 7-sec, 9.9-ft waves approaching from 343 deg; +2.8-ft swl

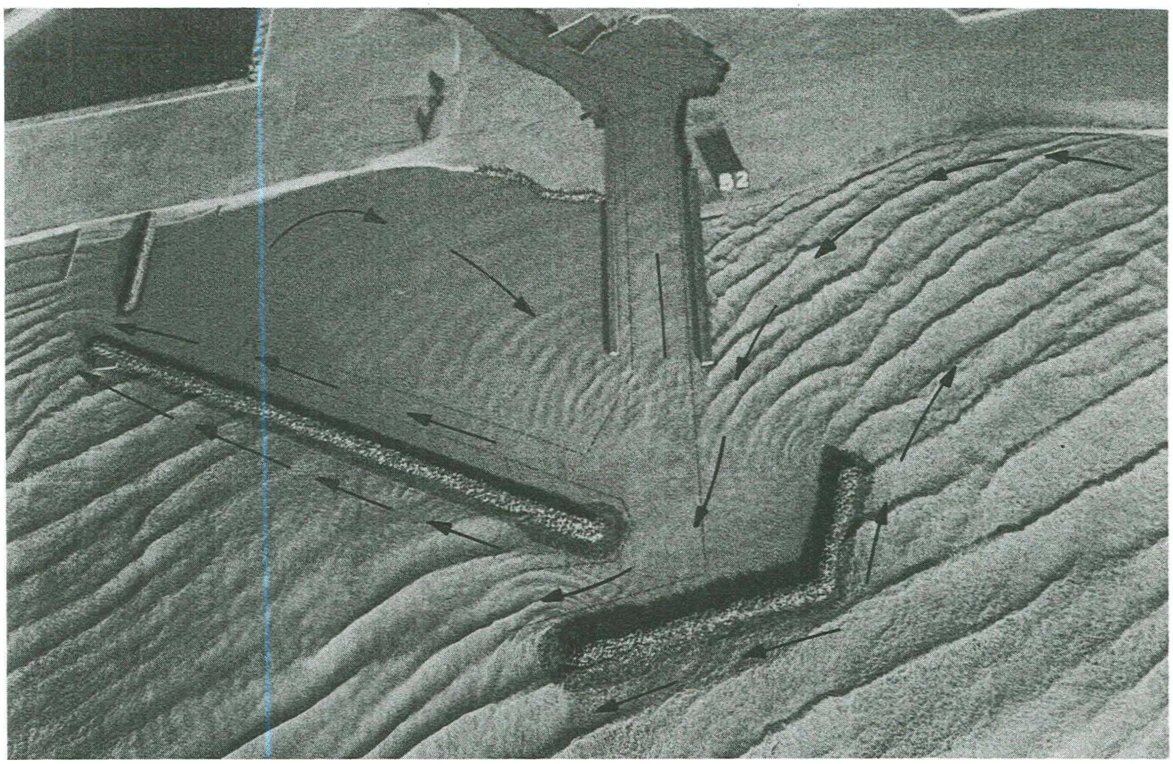


Photo 39. Typical wave and current patterns for Plan 16; 7.2-sec, 7.6-ft waves approaching from 313 deg; +2.8-ft swl

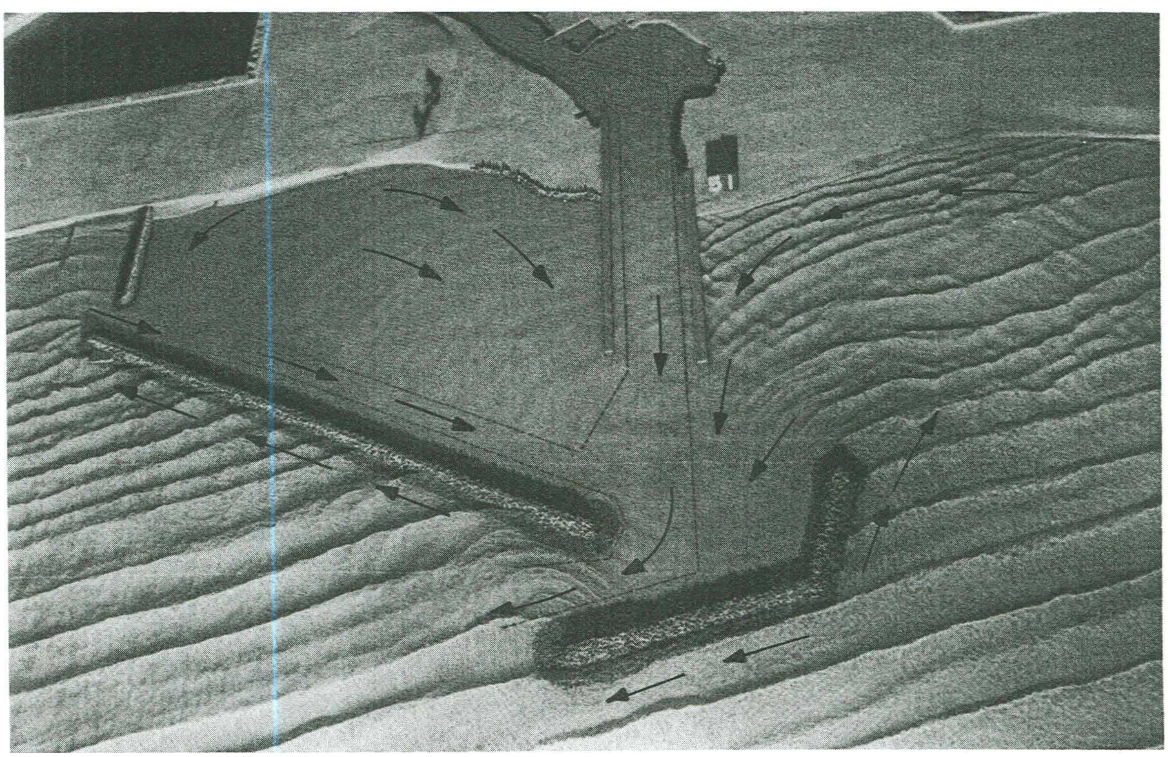


Photo 40. Typical wave and current patterns for Plan 16; 7.2-sec, 8.4-ft waves approaching from 334 deg; +2.8-ft swl

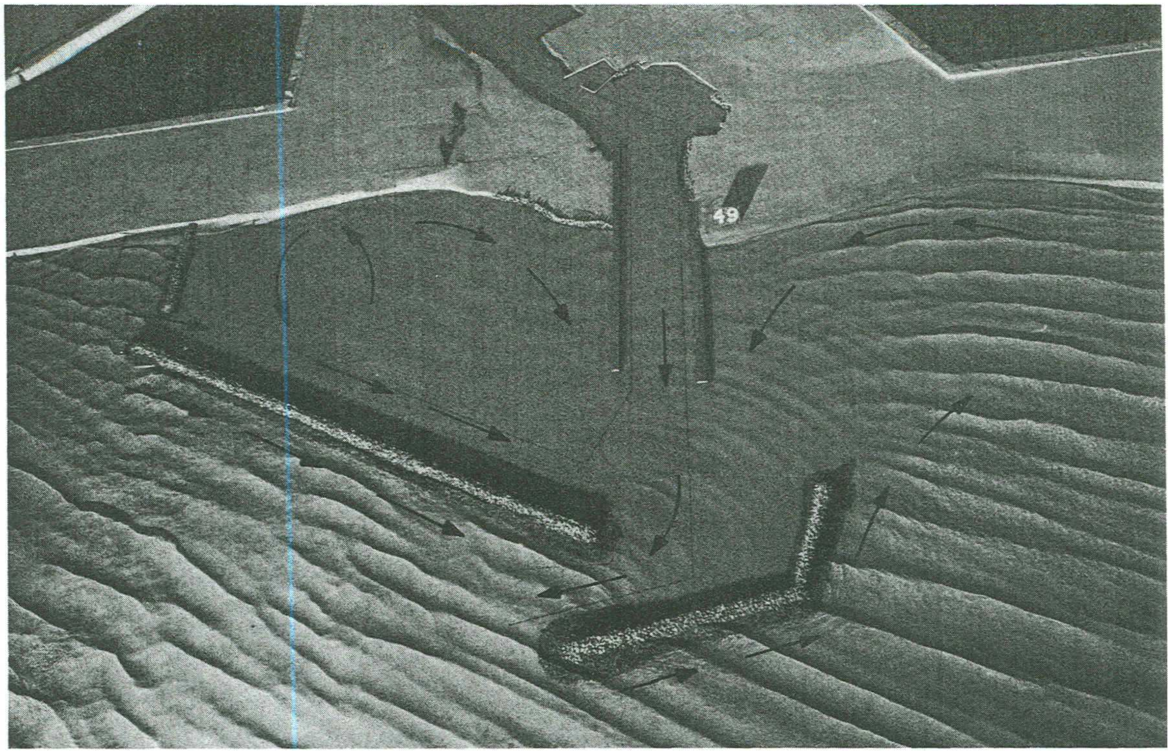


Photo 41. Typical wave and current patterns for Plan 16; 6.4-sec, 6.9-ft waves approaching from 24 deg; +4.0-ft swl

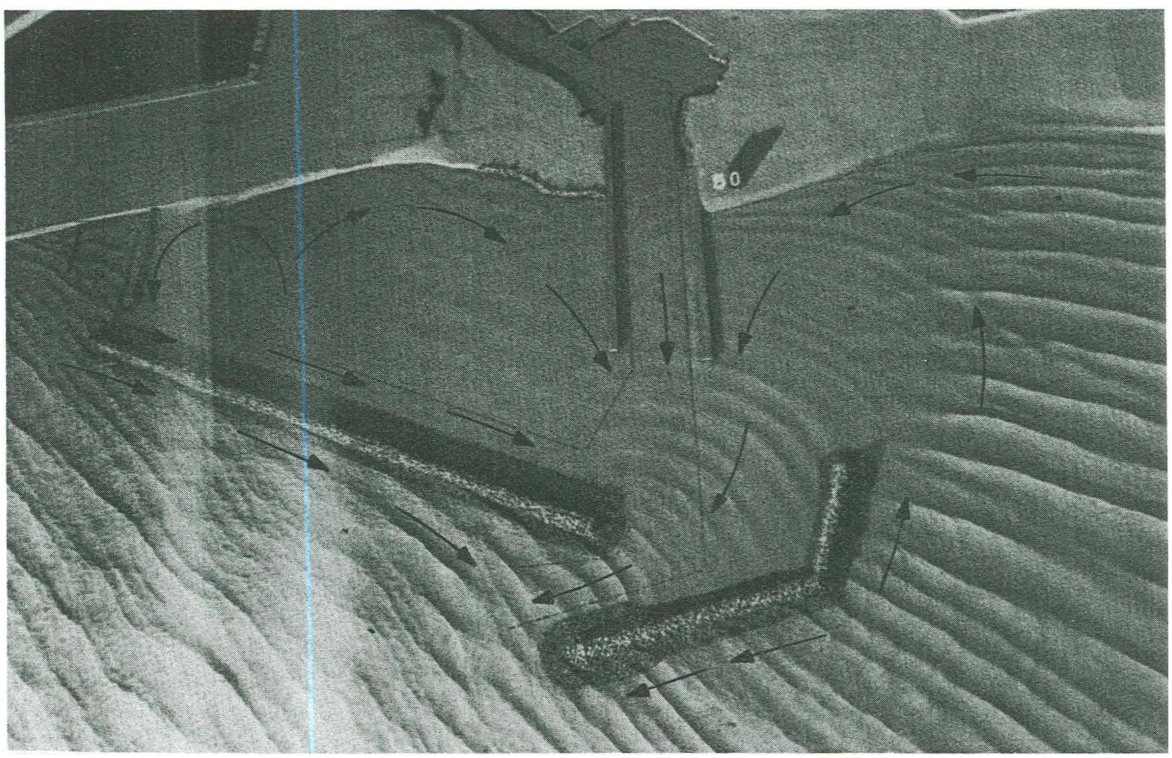
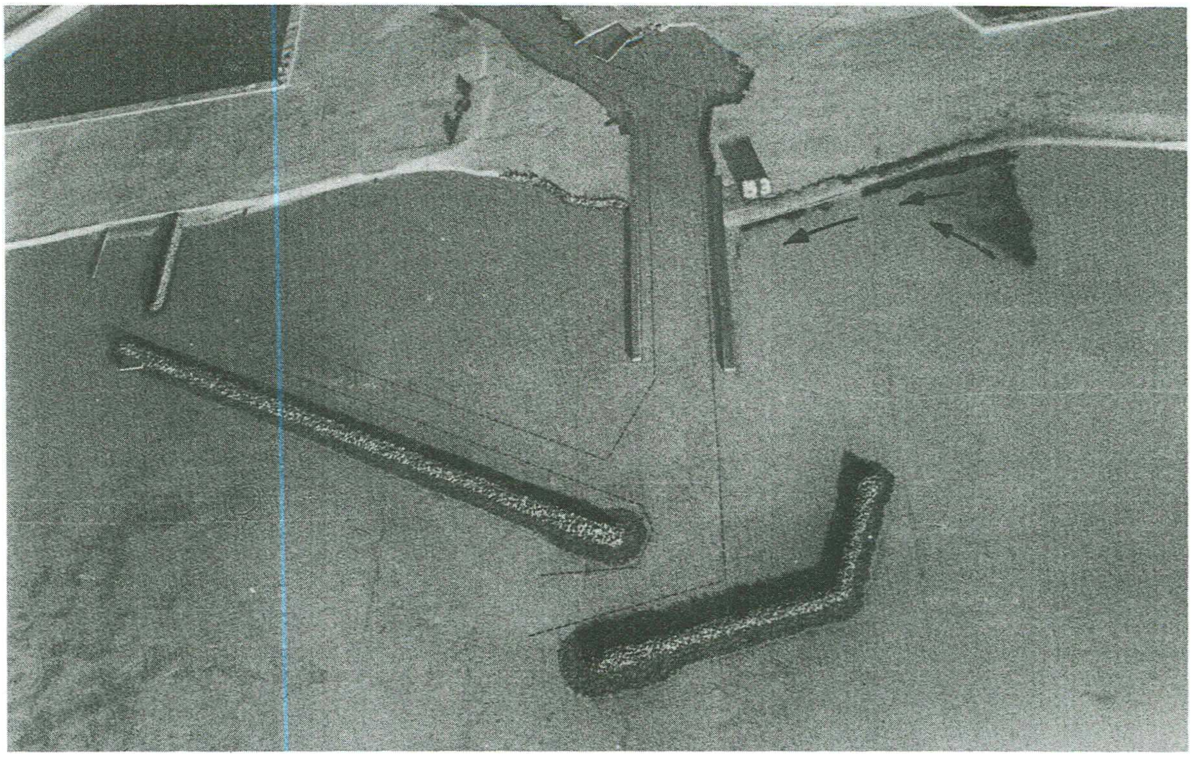
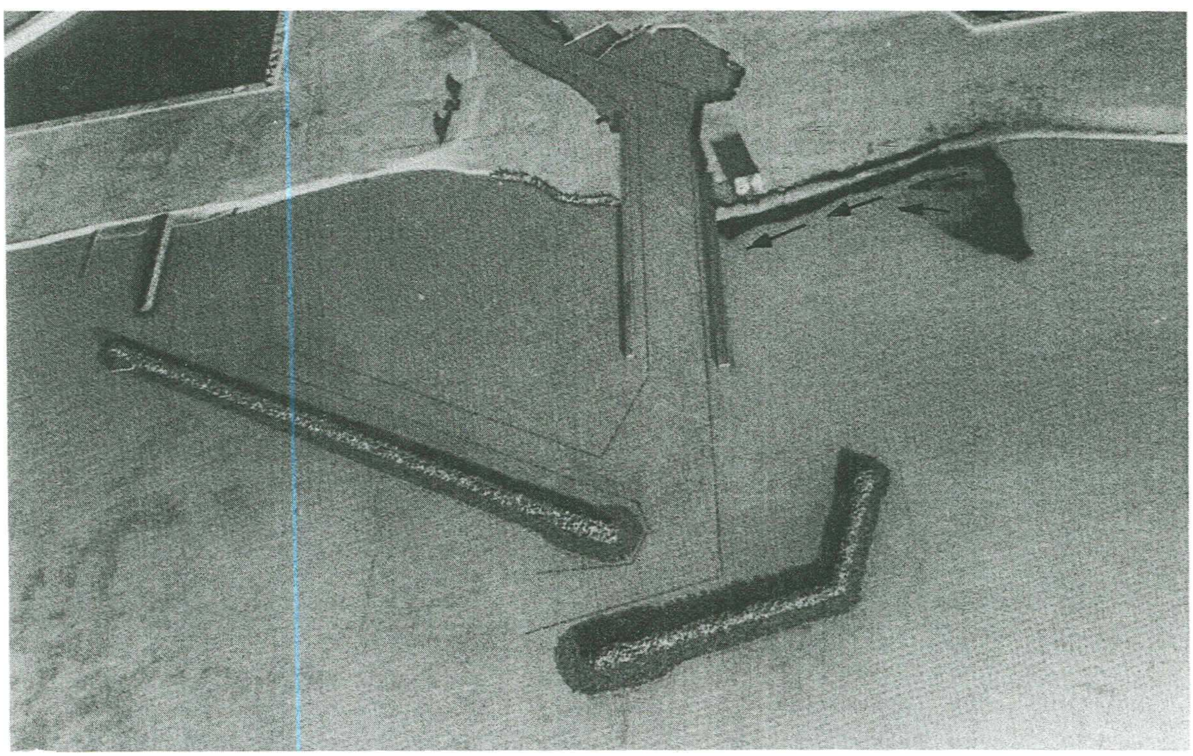


Photo 42. Typical wave and current patterns for Plan 16; 6.4-sec, S.B-ft waves approaching from 42 deg; +4.0-ft swl



a. 7.4-sec, 8-ft waves; +2.8-ft swl.



b. 6.4-sec, 6.3-ft waves; +4.0-ft swl

Photo 43. General movement of tracer material and subsequent deposits on the west side of the harbor for test waves from 313 deg for Plan 16

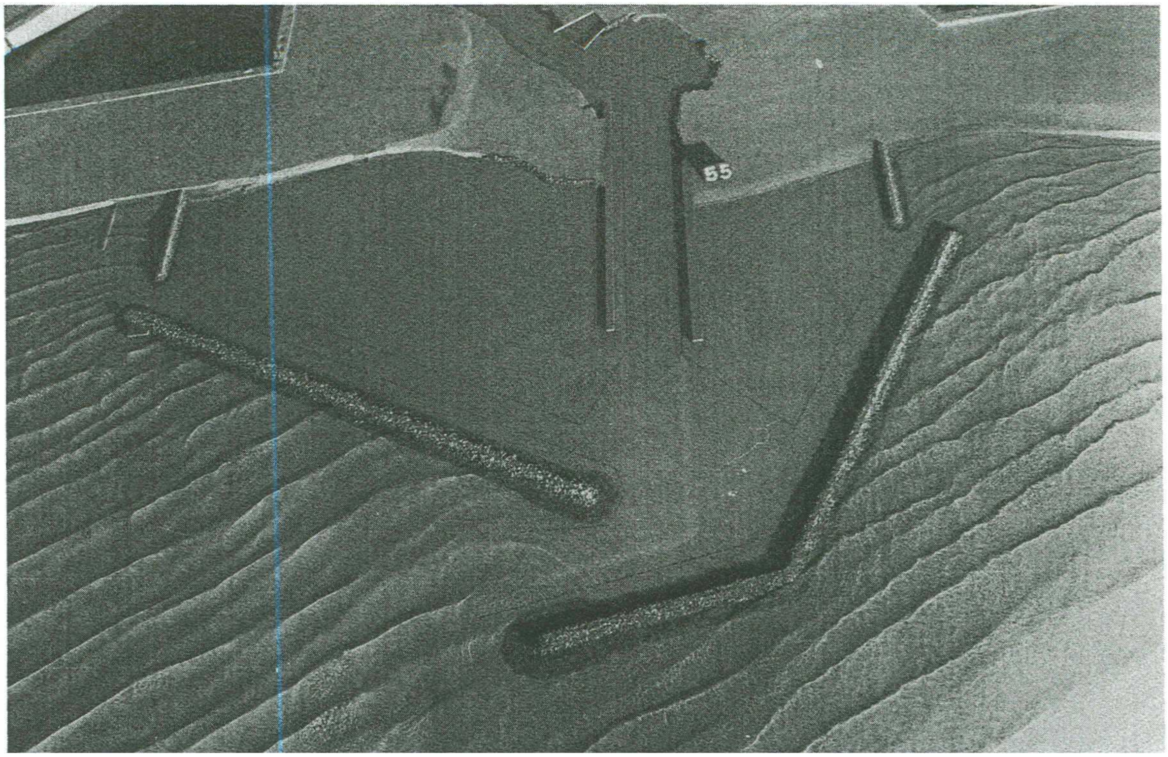


Photo 44. Typical wave patterns for Plan 17; 7.2-sec, 7.6-ft waves approaching from 313 deg; +2.8-ft swl

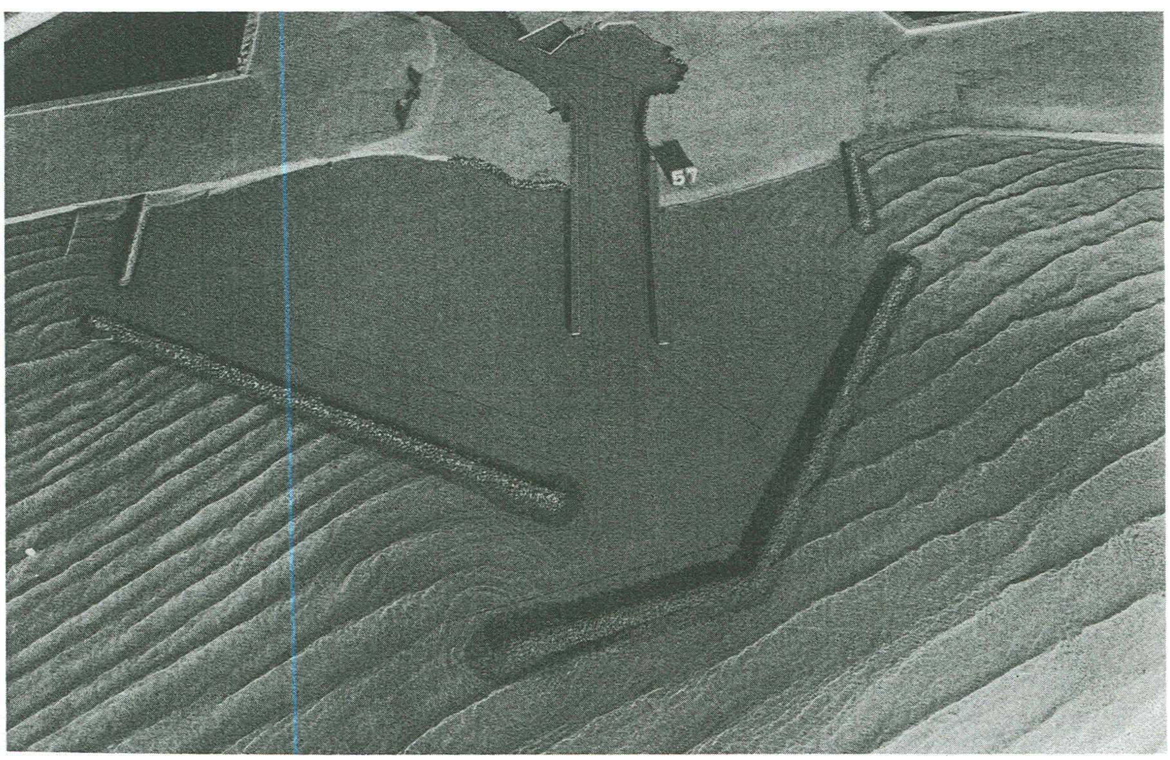


Photo 45. Typical wave patterns for Plan 18; 7.2-sec, 7.6-ft waves approaching from 313 deg; +2.8-ft swl

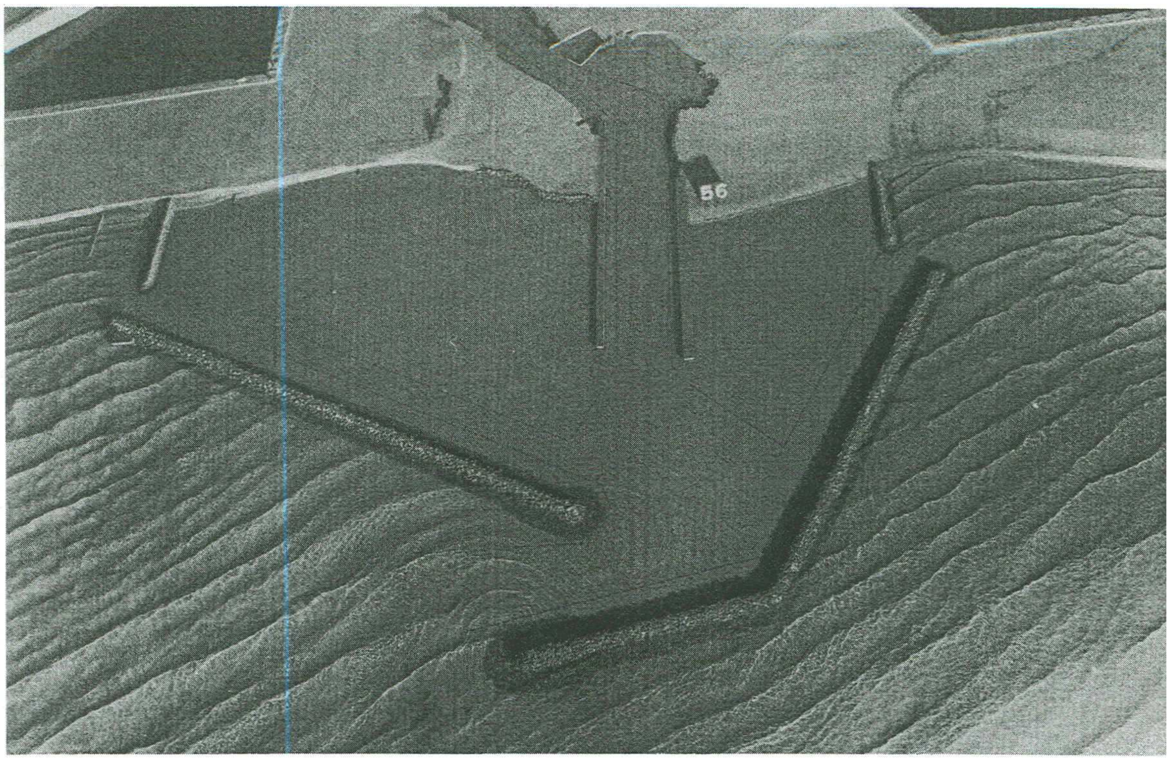


Photo 46. Typical wave patterns for Plan 19; 7.2-sec, 7.6-ft waves approaching from 313 deg; +2.8..ft swl

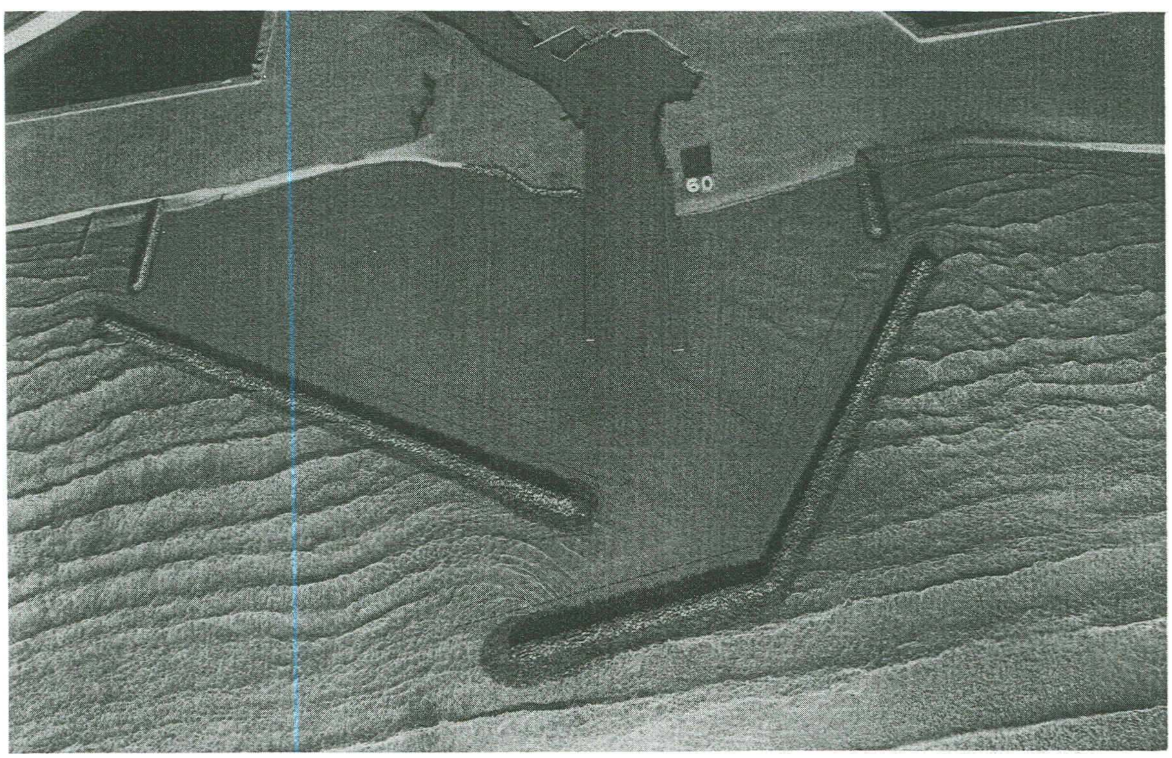


Photo 47. Typical wave patterns for Plan 19; 7-sec, 9.9-ft waves approaching from 343 deg; +2.8-ft swl

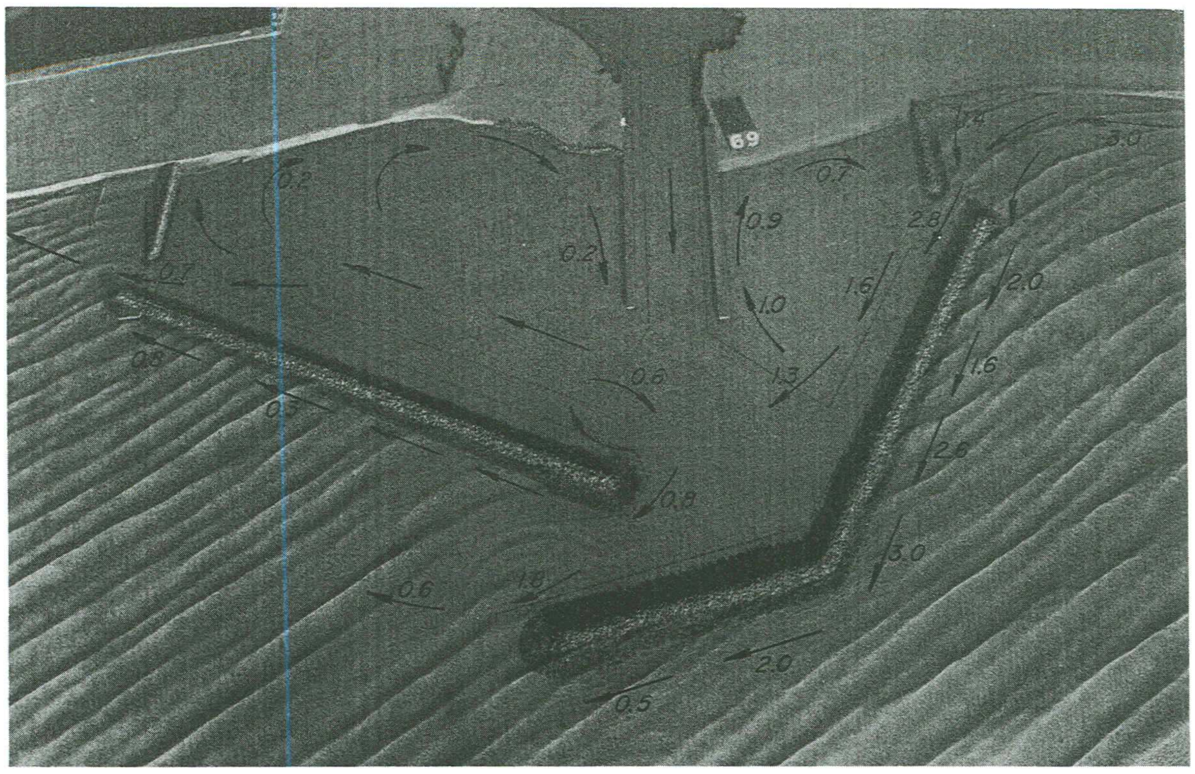


Photo 48. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for Plan 19; 6.4-sec, 6.3-ft waves approaching from 313 deg; +4.0-ft swl

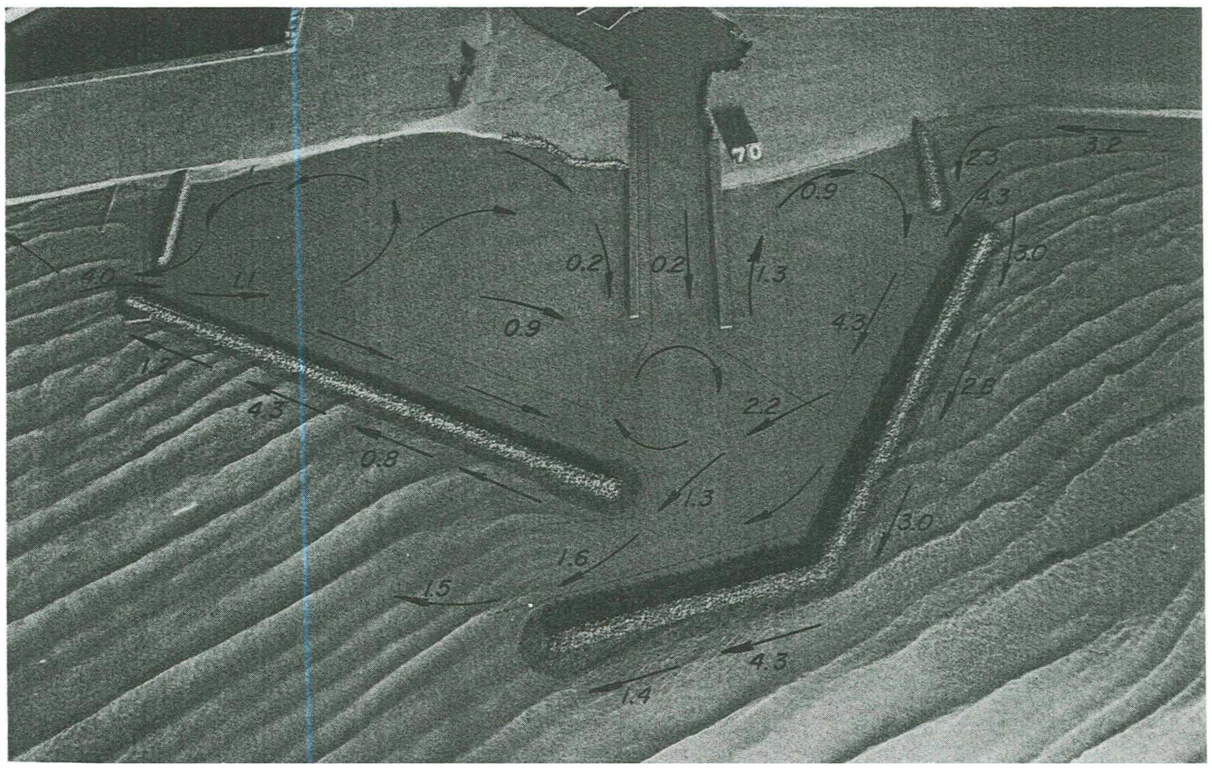


Photo 49. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for Plan 19; 7.4-sec, 8-ft waves approaching from 313 deg; +2.8-ft swl

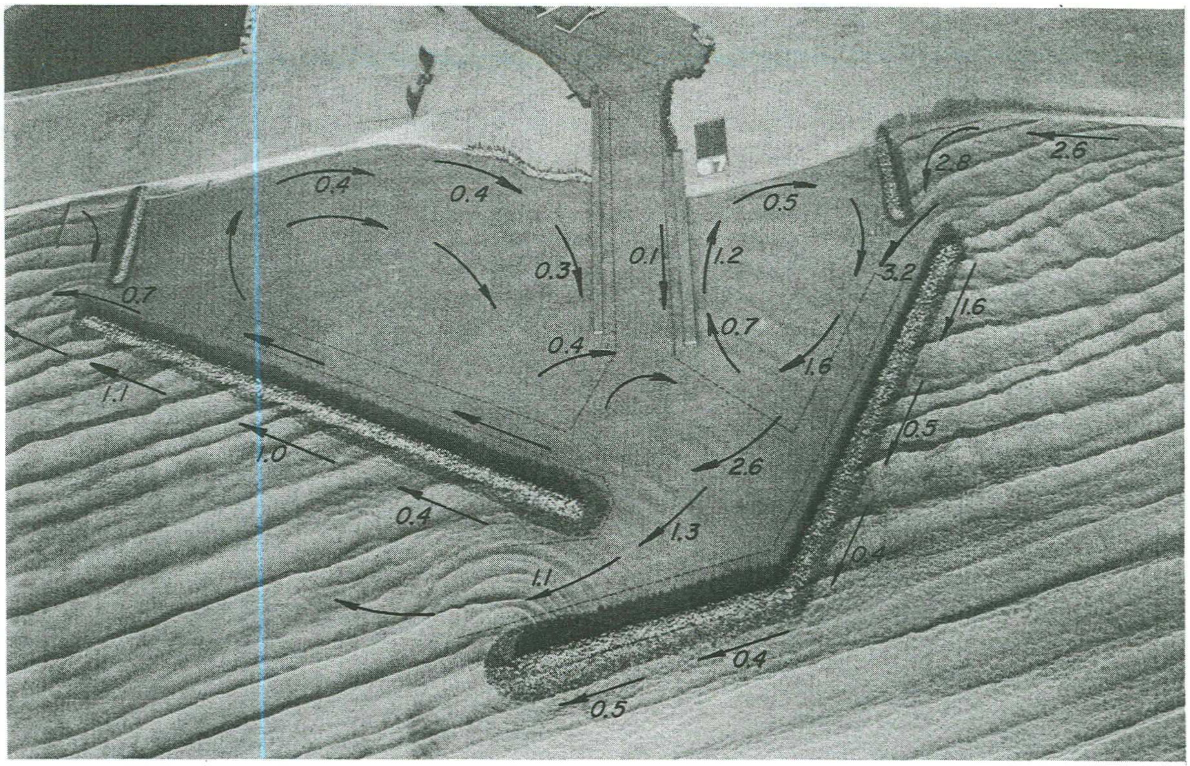


Photo 50. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for Plan 19; 6.4-sec, 6.5-ft waves approaching from 334 deg; +4.0-ft swl

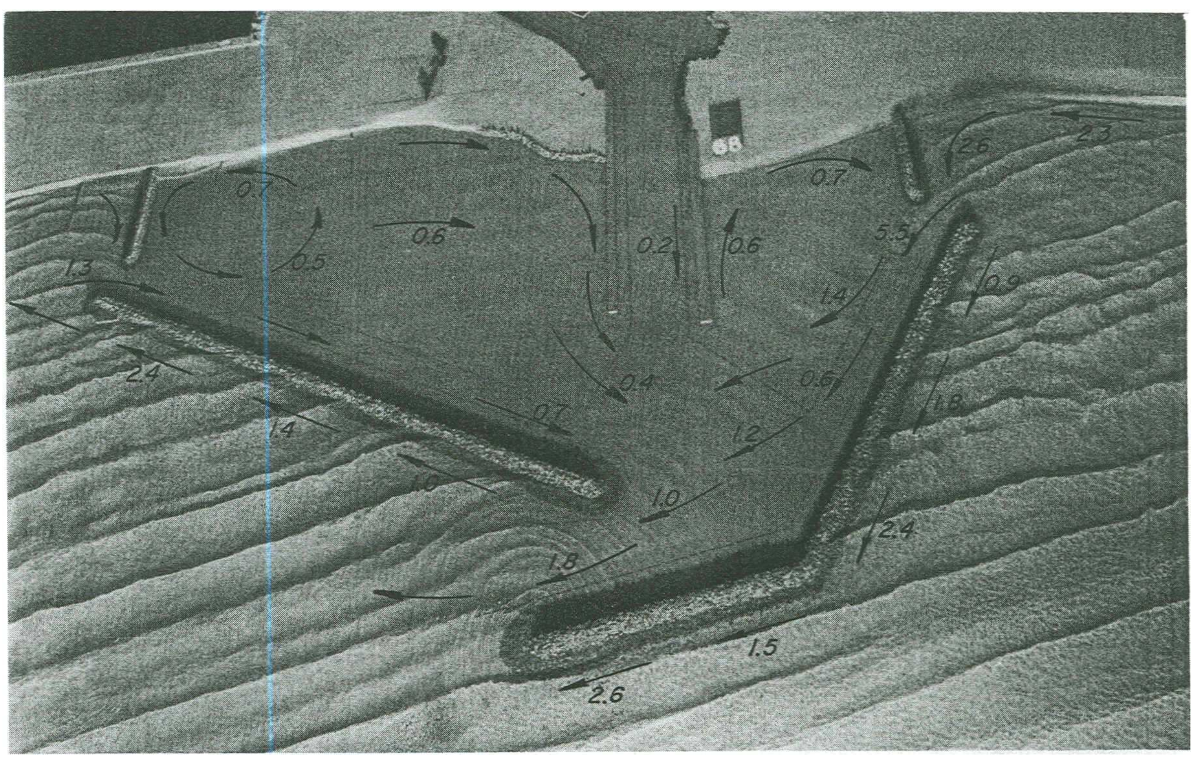


Photo 51. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for Plan 19; 7.4-sec, 8.8-ft waves approaching from 334 deg; +2.8-ft swl

Photo 52. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for Plan 19; 5.7-sec, 5.8-ft waves approaching from 343 deg; +4.0-ft swl

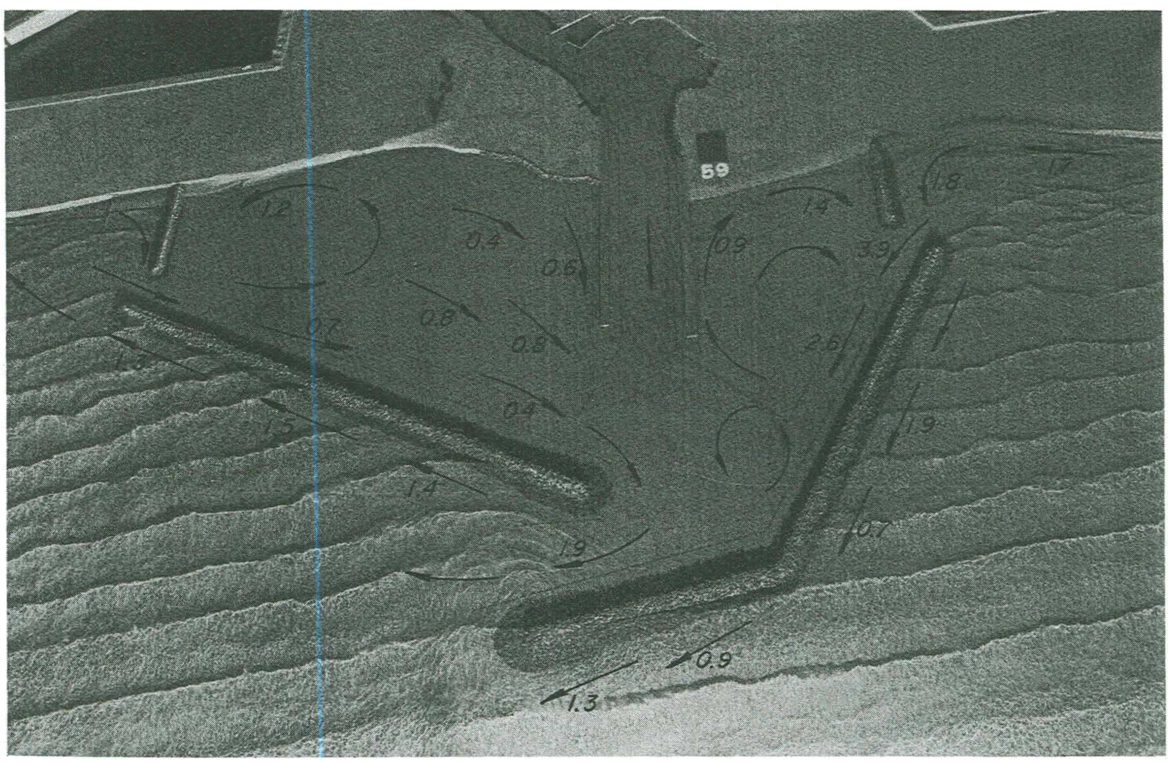


Photo 53. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for Plan 19; 7.4-sec, 11-ft waves approaching from 343 deg; +2.8-ft swl

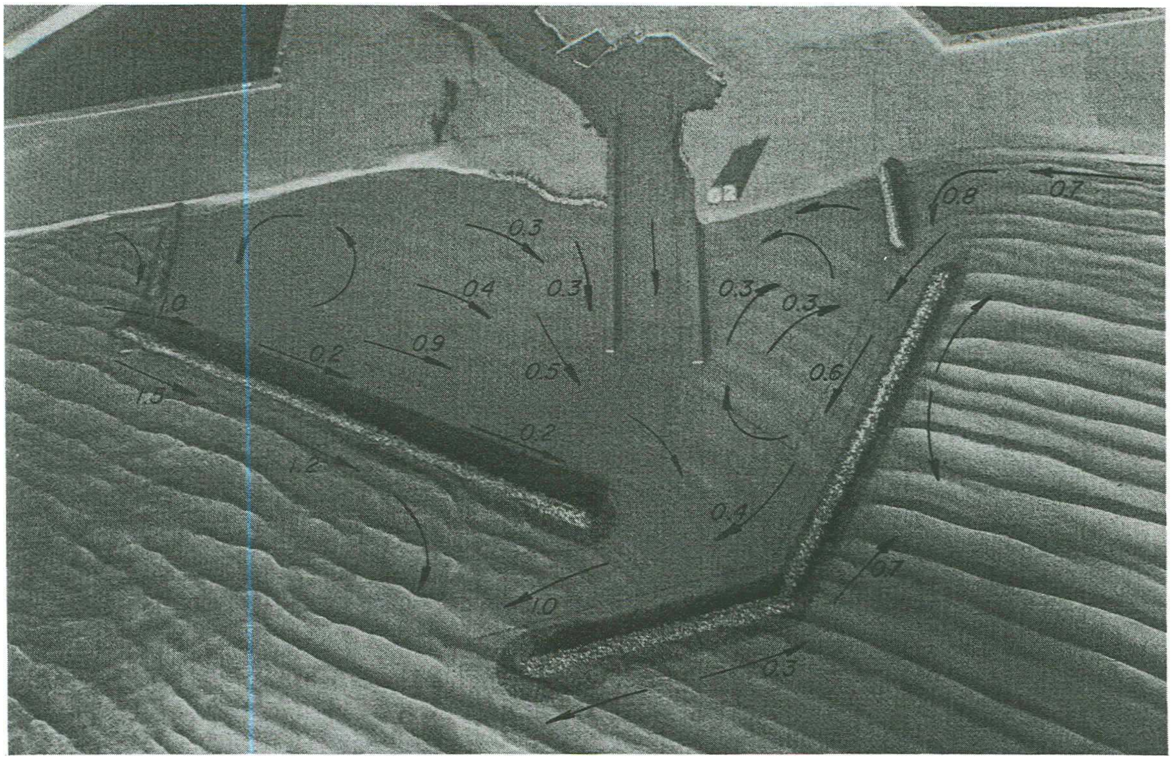


Photo 54. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for Plan 19; 5.7-sec, 4.7-ft waves approaching from 24 deg; +4.0-ft swl

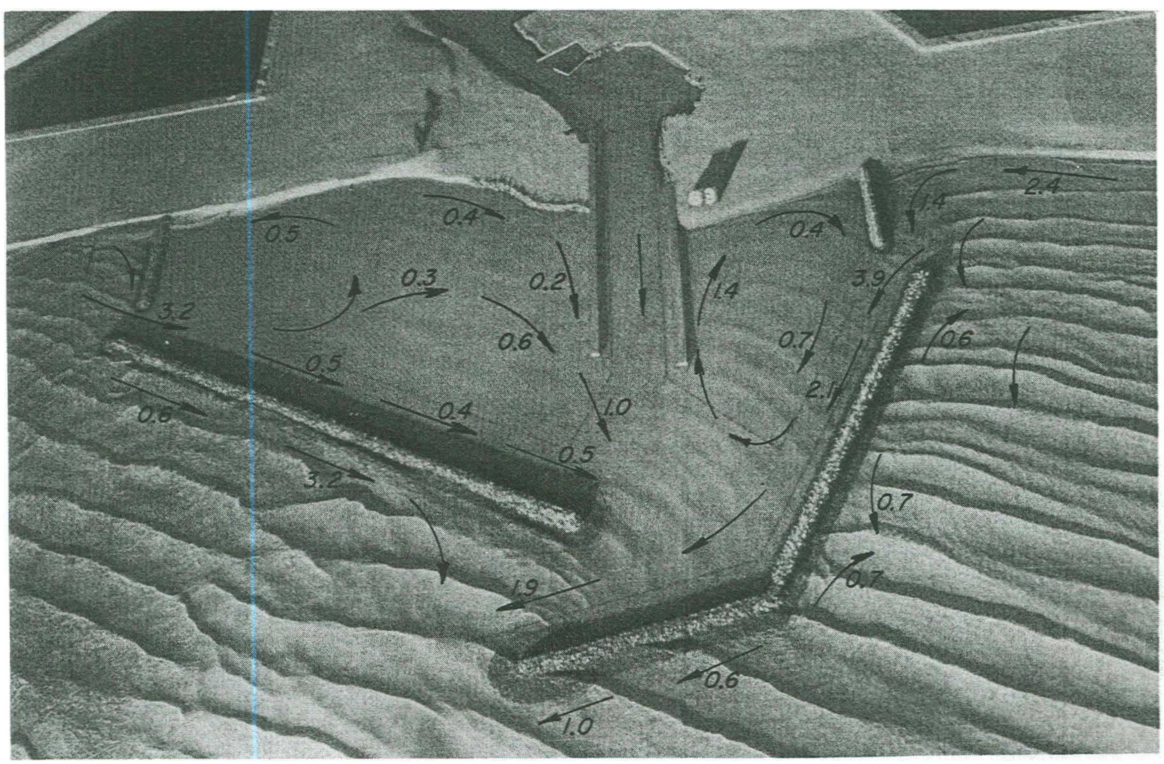


Photo 55. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for Plan 19; 6.9-sec, 7.9-ft waves approaching from 24 deg; +2.8-ft swl

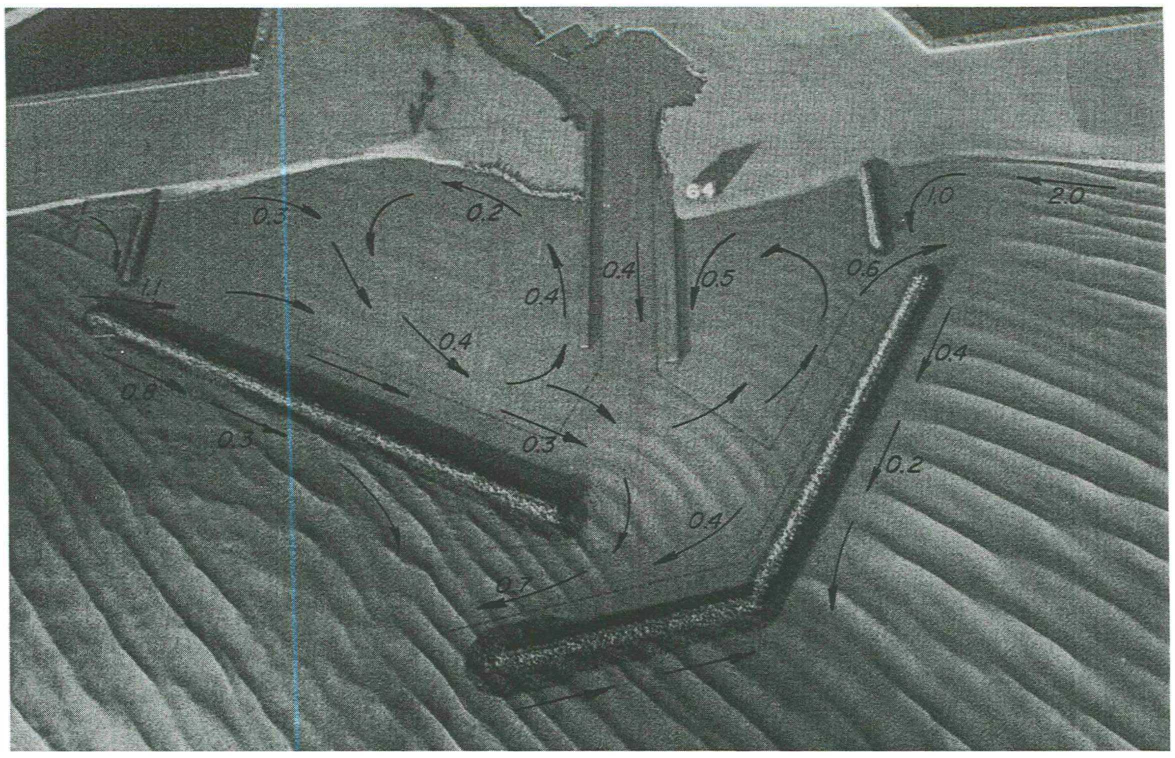


Photo 56. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for Plan 19; 5.7-sec, 4.0-ft waves approaching from 42 deg; +4.0-ft swl

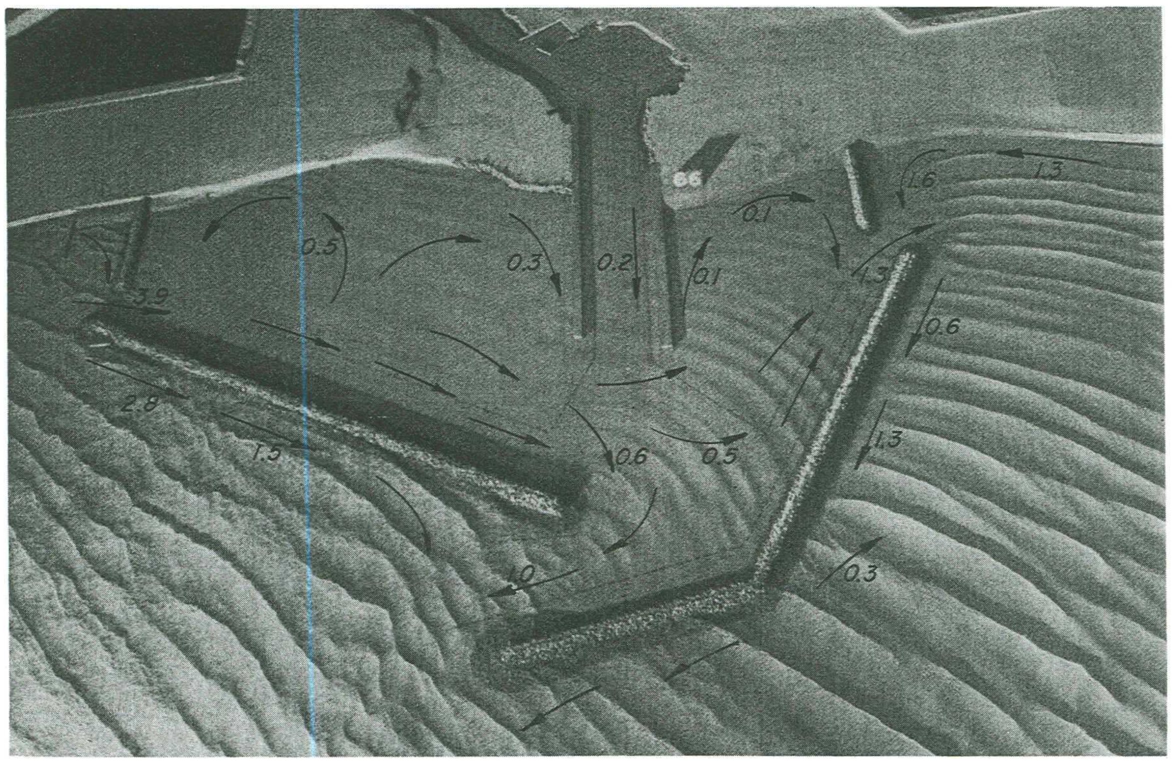
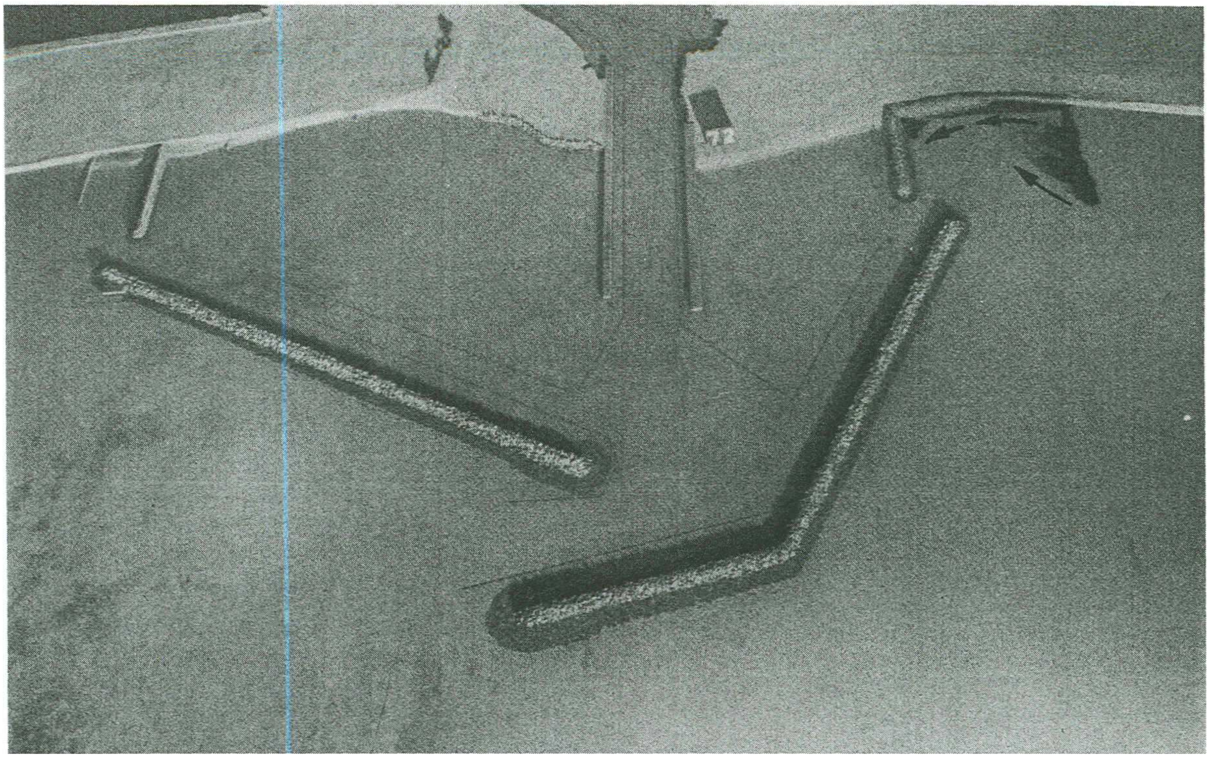
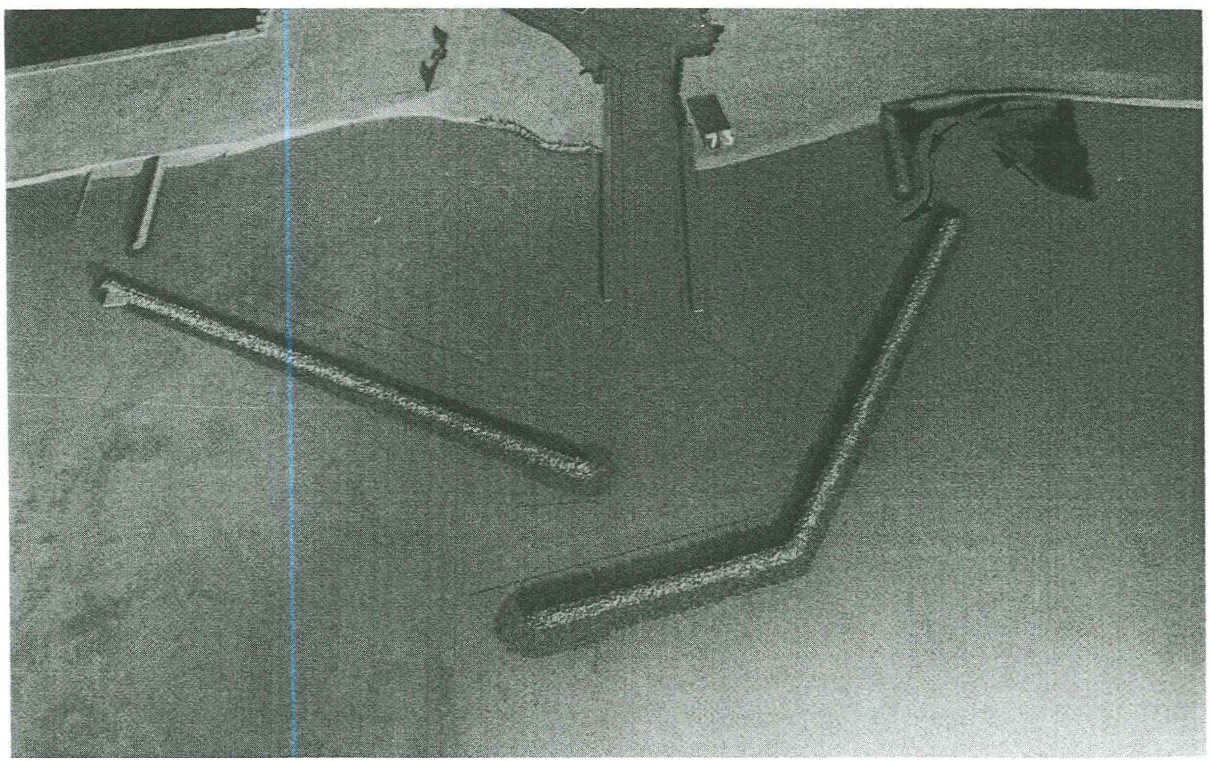


Photo 57. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for Plan 19; 6.9-sec, 6.4-ft waves approaching from 42 deg; +2.8-ft swl

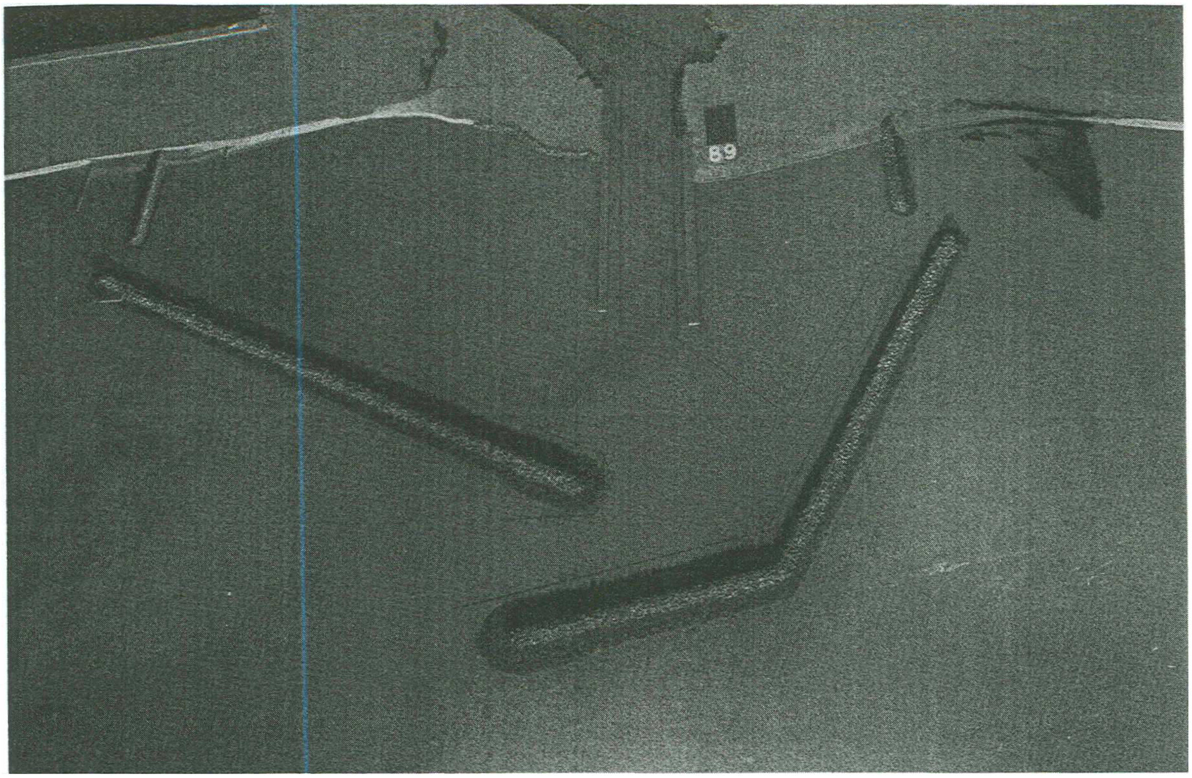


a. 7 .4-sec, 8-ft waves; +2. 8-ft sw1.

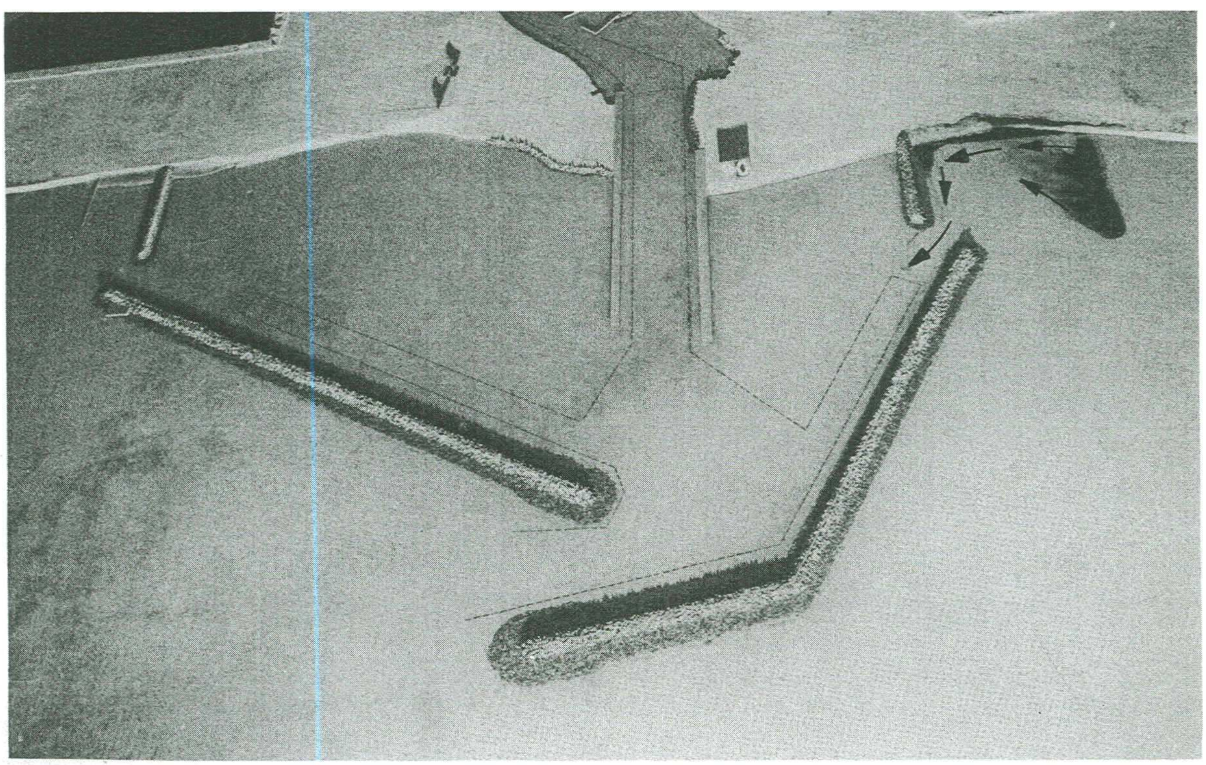


b. 6.4-sec, 6.3-ft waves; +4.0-ft swl

Photo 58. General movement of tracer material and subsequent deposits west of the harbor for test waves from 313 deg for Plan 19

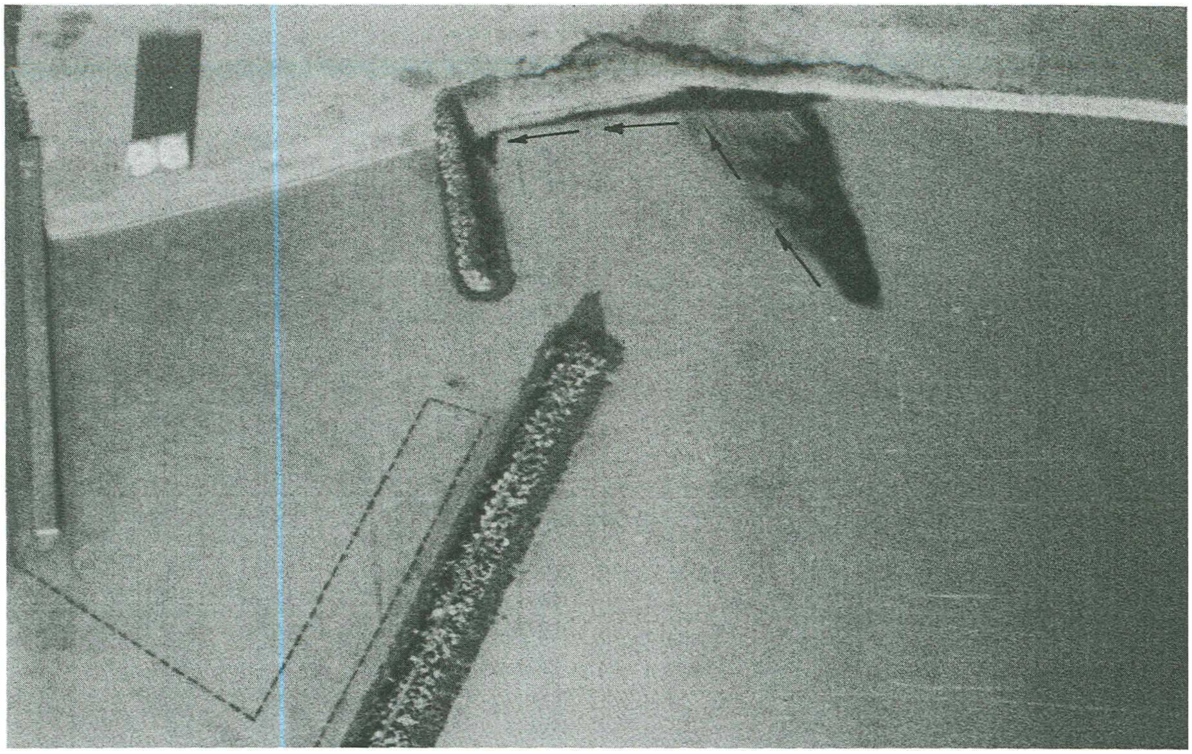


a. 7.4-sec, 8.8-ft waves; +2.8-ft swl

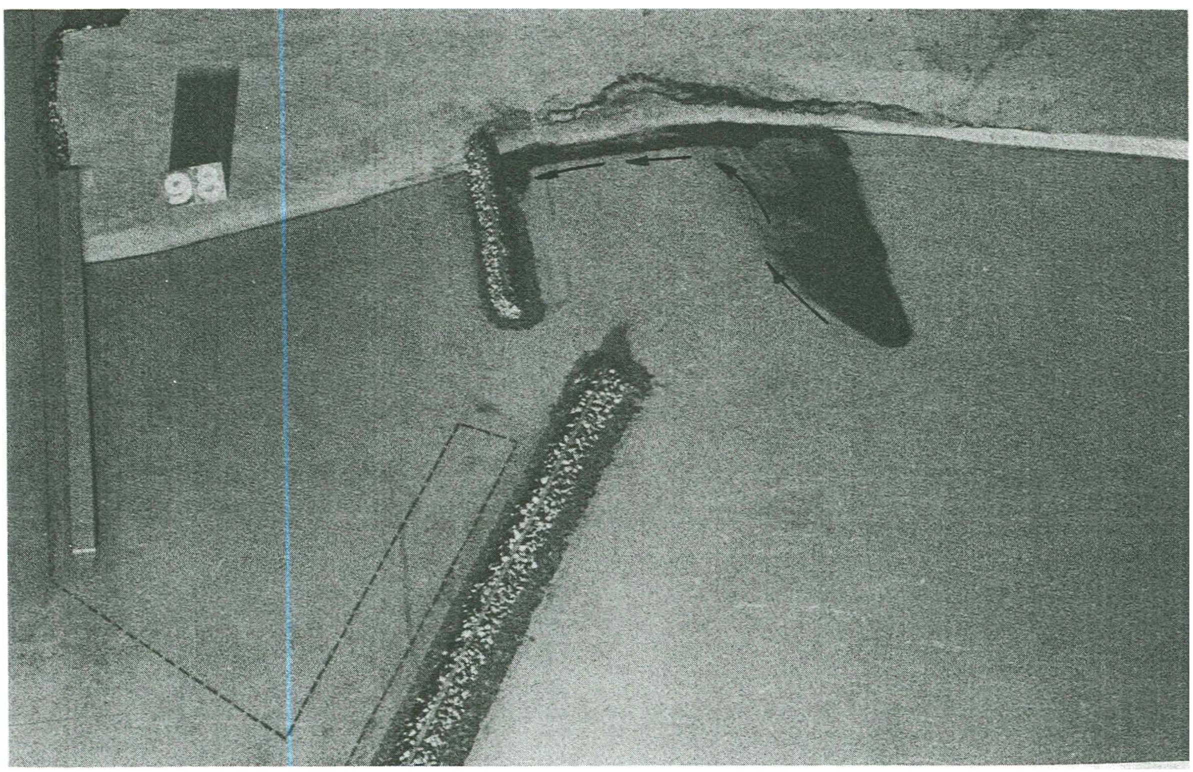


b. 6.4-sec, 6.S-ft waves; +4.0-ft swl

P~oto ~9. General movement of trace~ material and subsequent deposits west of the harbor for test waves from 334 deg for Plan 19

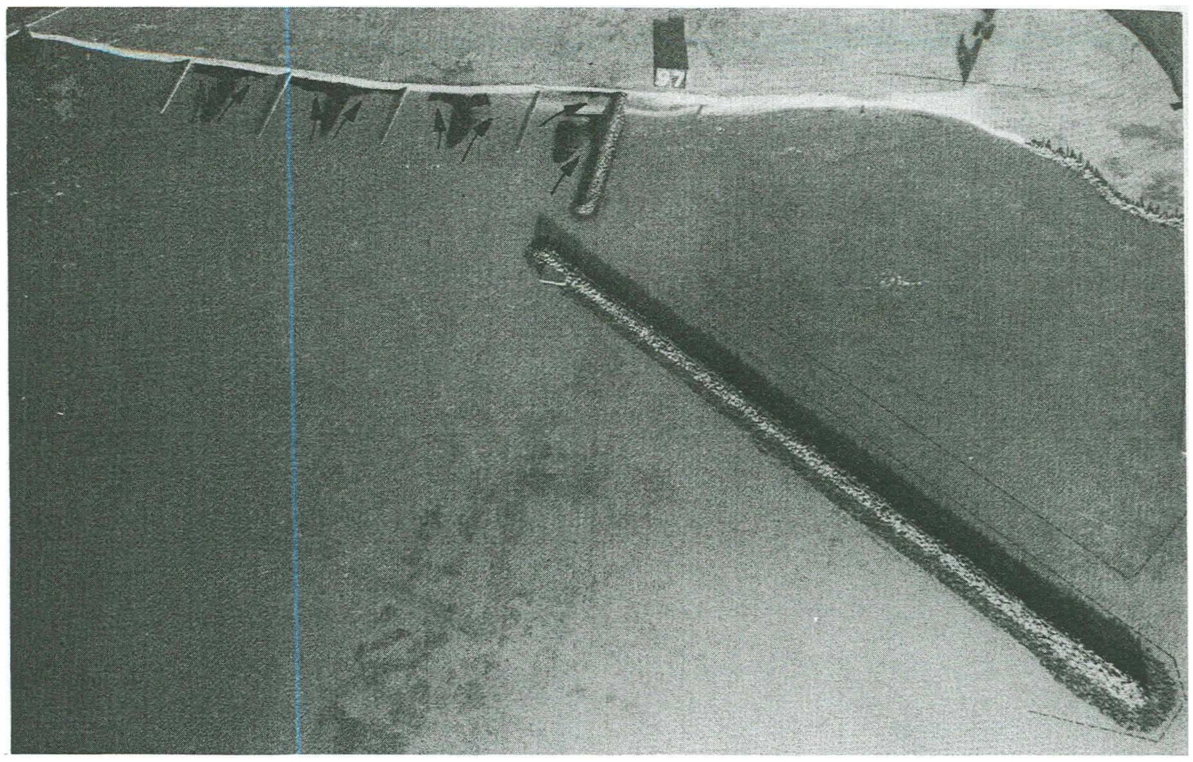


a. 7.4-sec, ll-ft waves; +2.8-ft swl

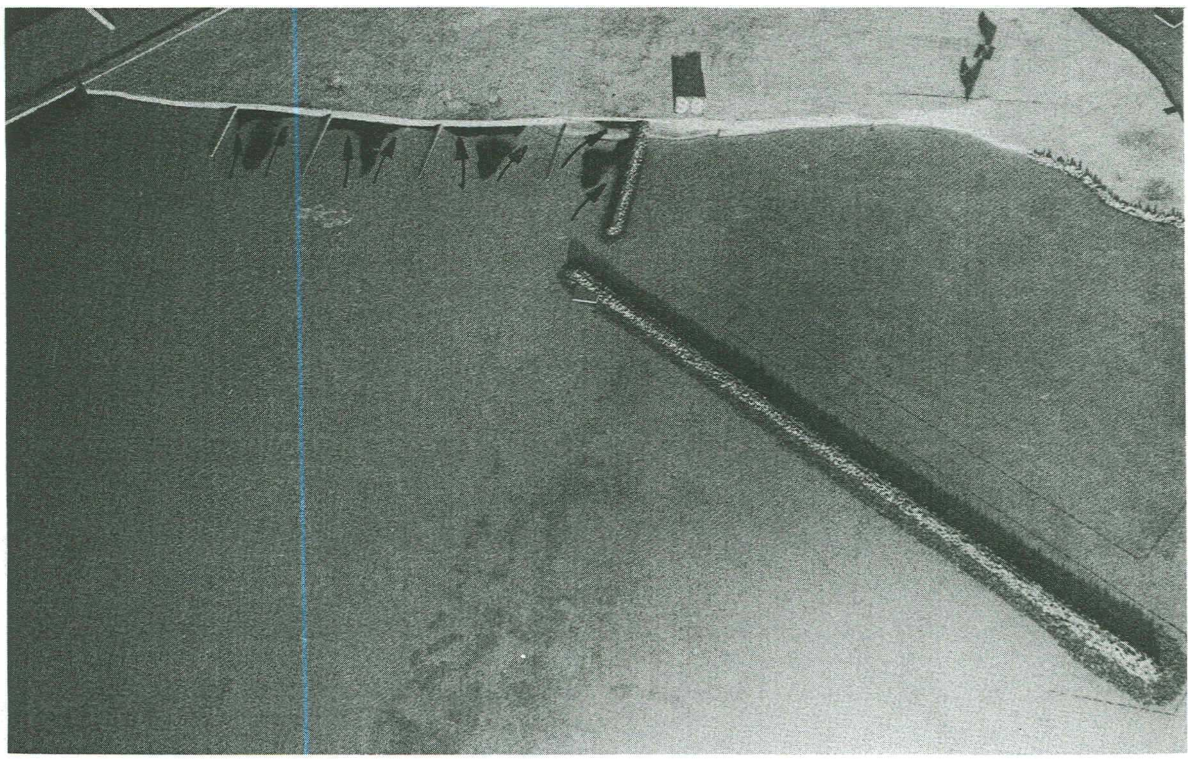


b. S.7-sec, S.B-ft waves; +4.0-ft swl

Photo 60. General movement *of* tracer material and subsequent deposits west *of* the harbor for test waves from 343 deg for Plan 19

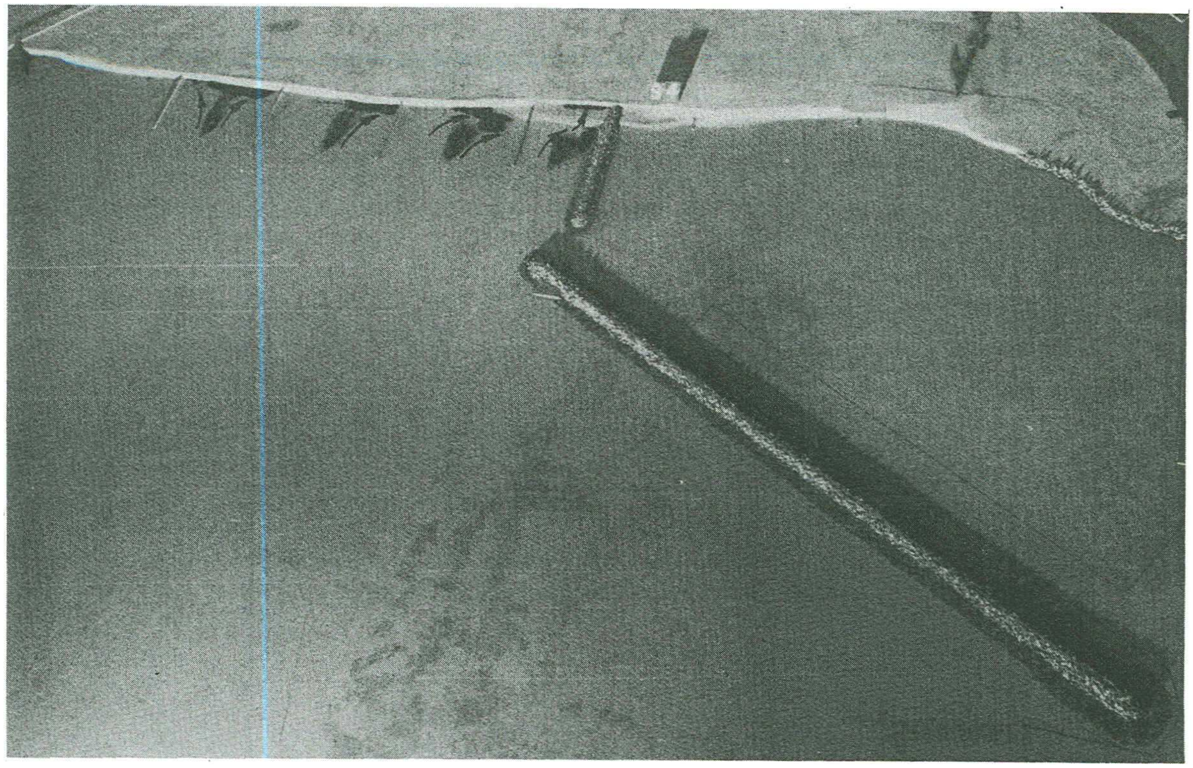


a. 7.4-sec, ll-ft waves; +2.8-ft swl

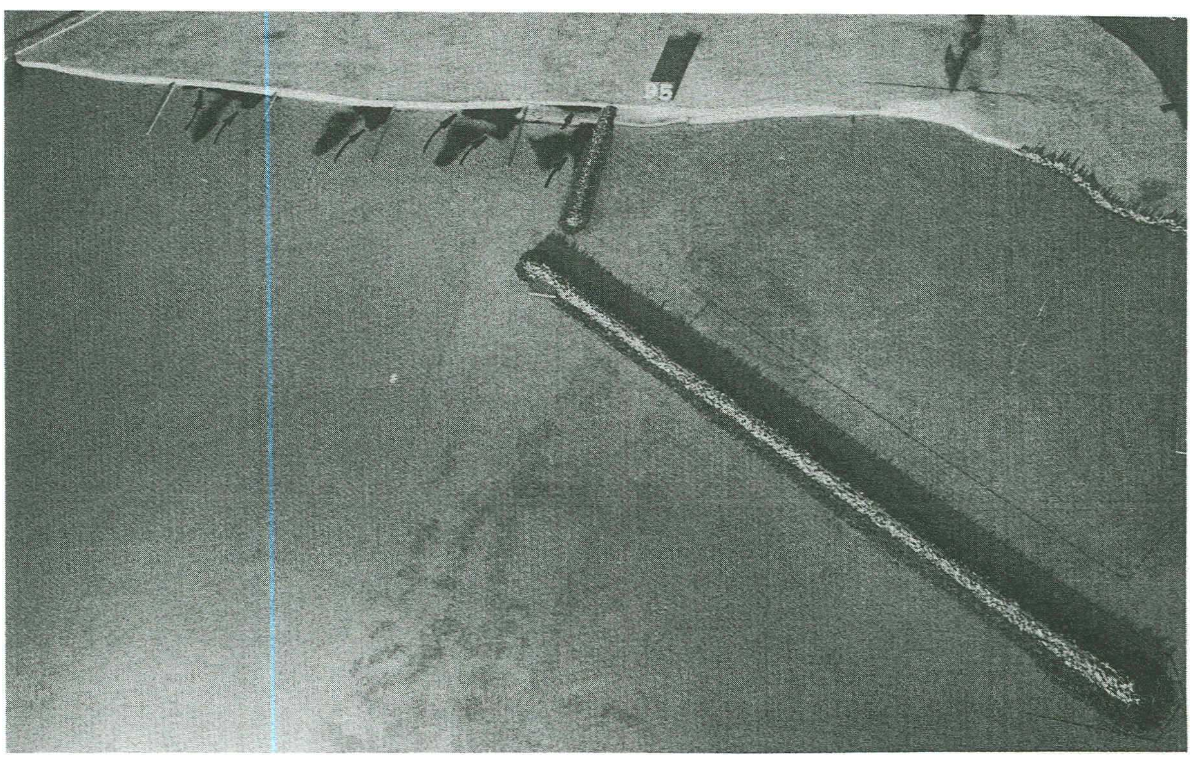


b. s.7-sec, s.8-ft waves; +4.0-ft swl

Photo 61. General movement of tracer material and subsequent deposits east of the harbor for test waves from 343 deg for Plan 19

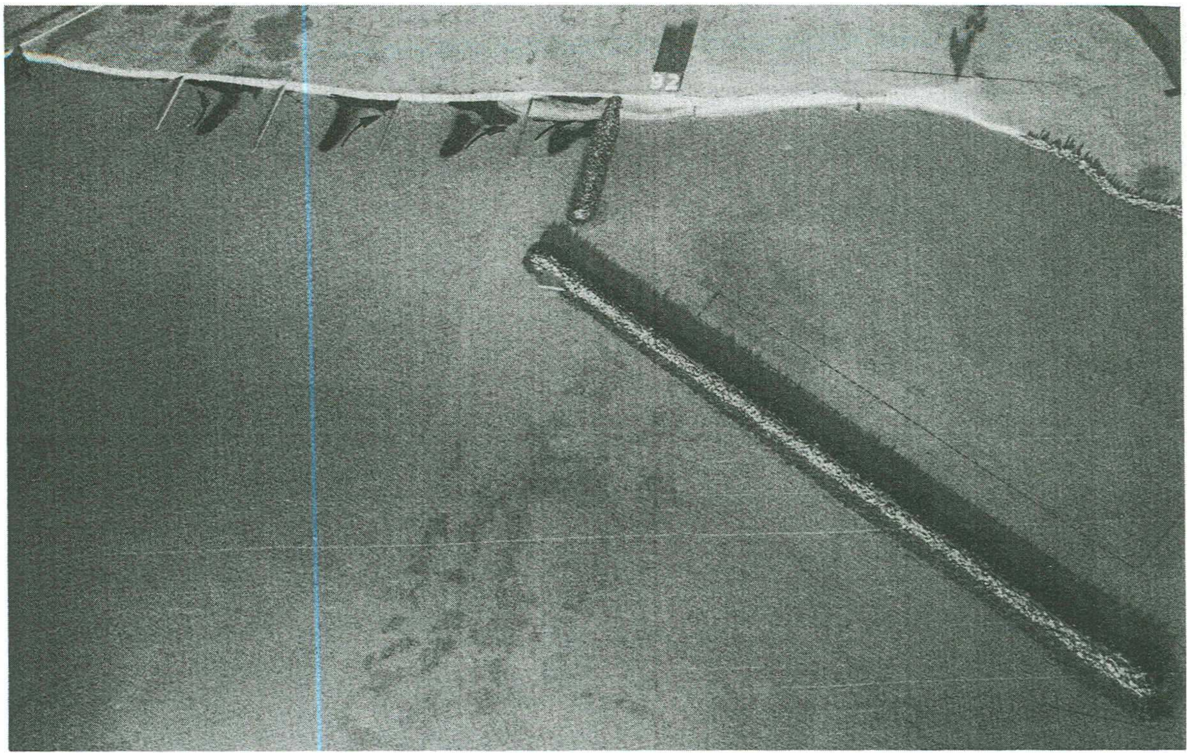


a. 6.9-sec, 7.9-ft waves; +2.8-ft swl

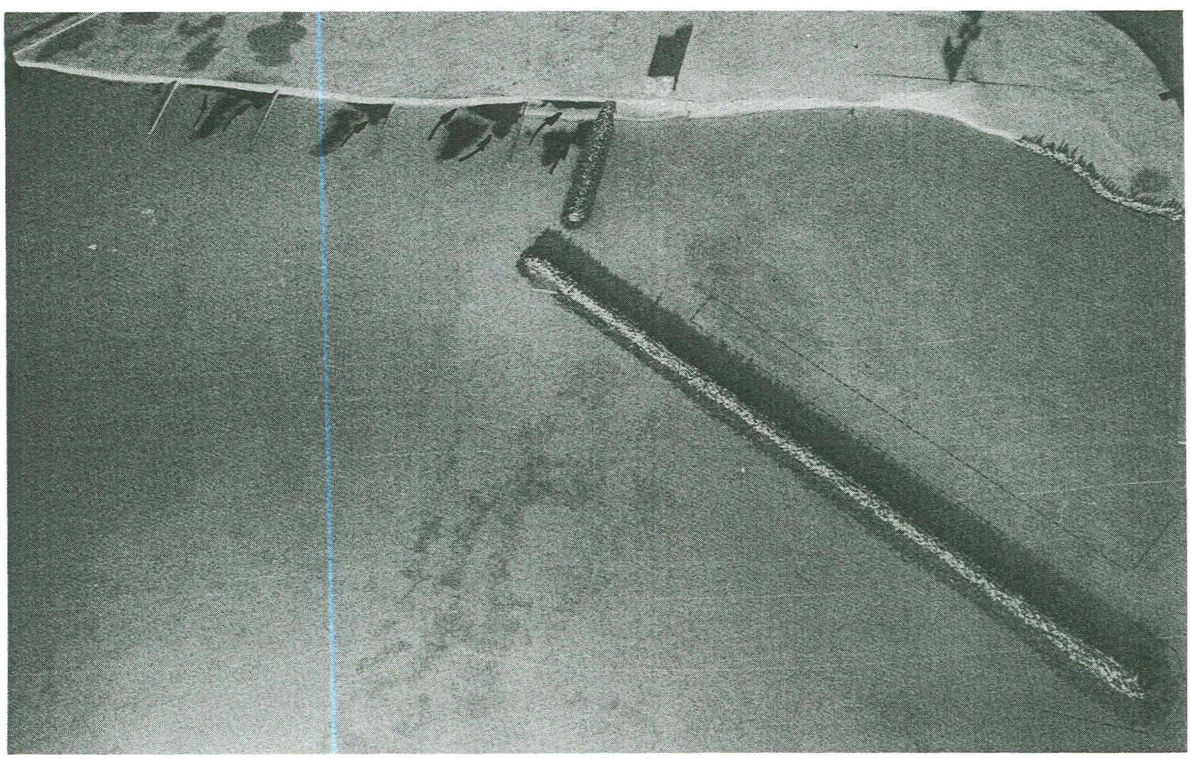


b. S.7-sec, 4.7-ft waves; +4.0-ft swl

Photo 62. General movement of tracer material and subsequent deposits east of the harbor for test waves from 24 deg for Plan 19

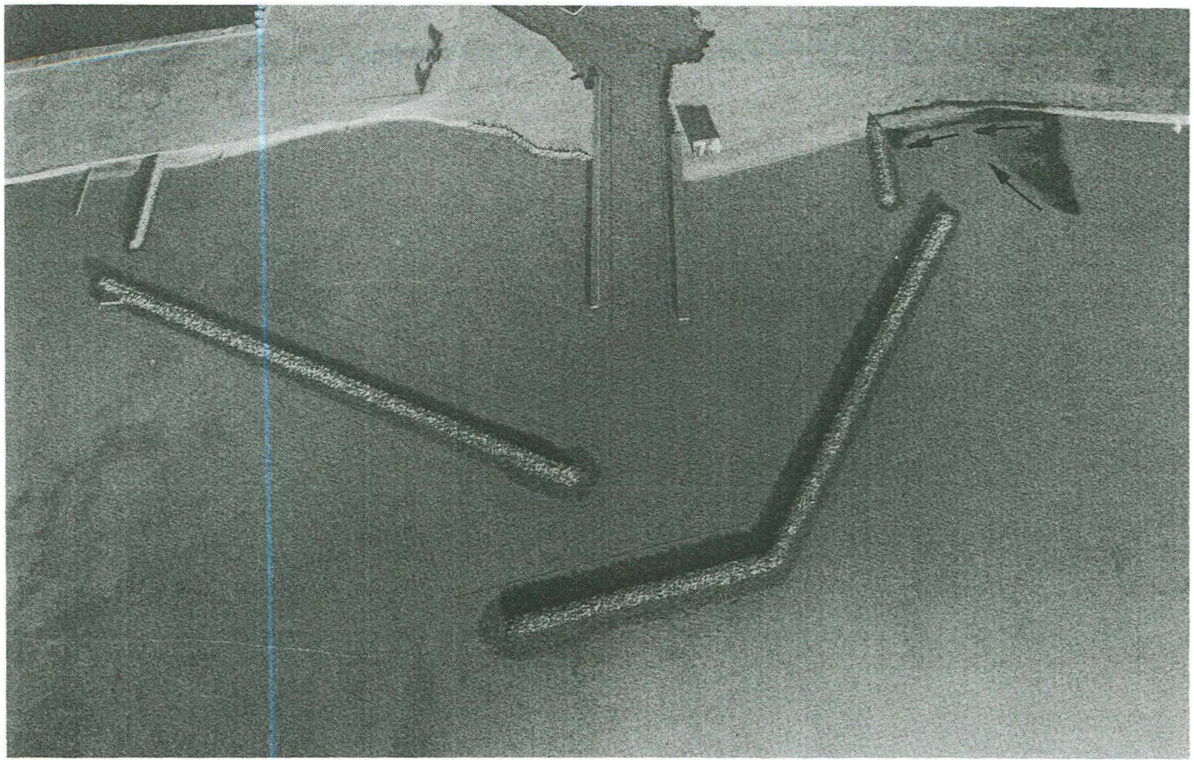


a. 6.9-sec, 1.9-ft waves; +2.8-ft swl

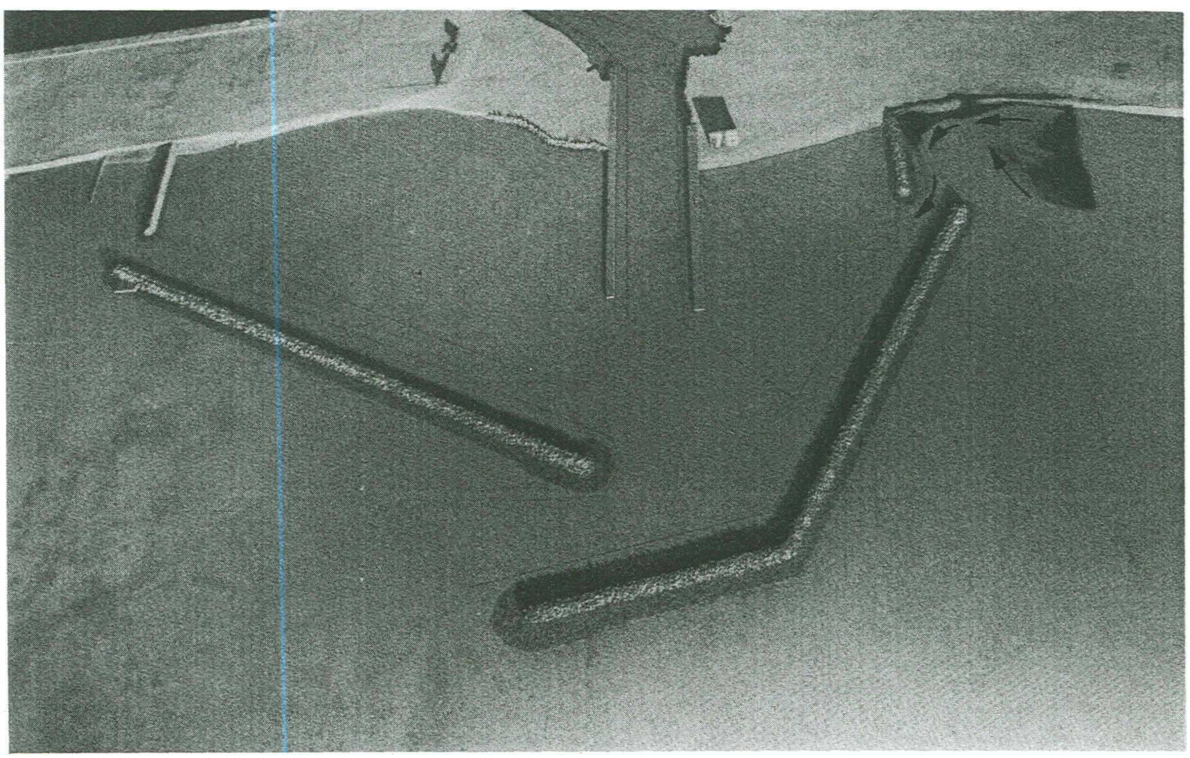


b. 5.7-sec, 4-ft waves; +4.0-ft swl

Photo 63. General movement of tracer material and subsequent deposits east of the harbor for test waves from 42 deg for Plan 19

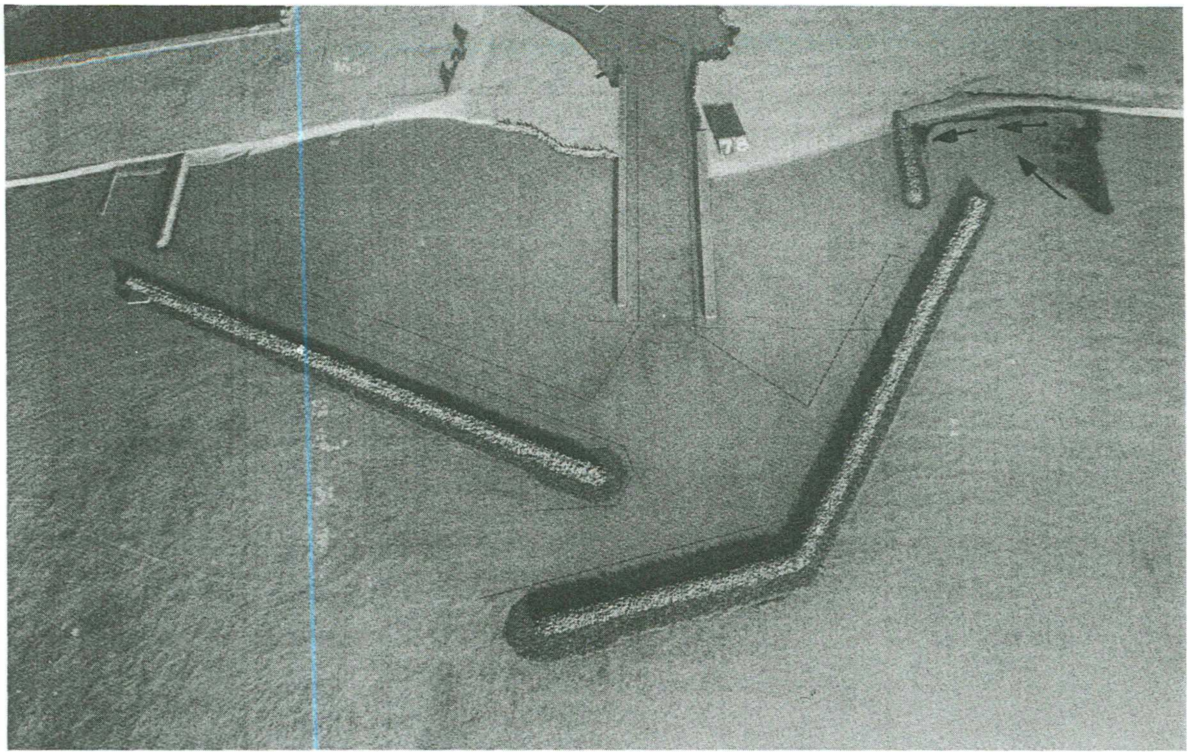


a. 7.4-sec, 8-ft waves; +2.8-ft swl

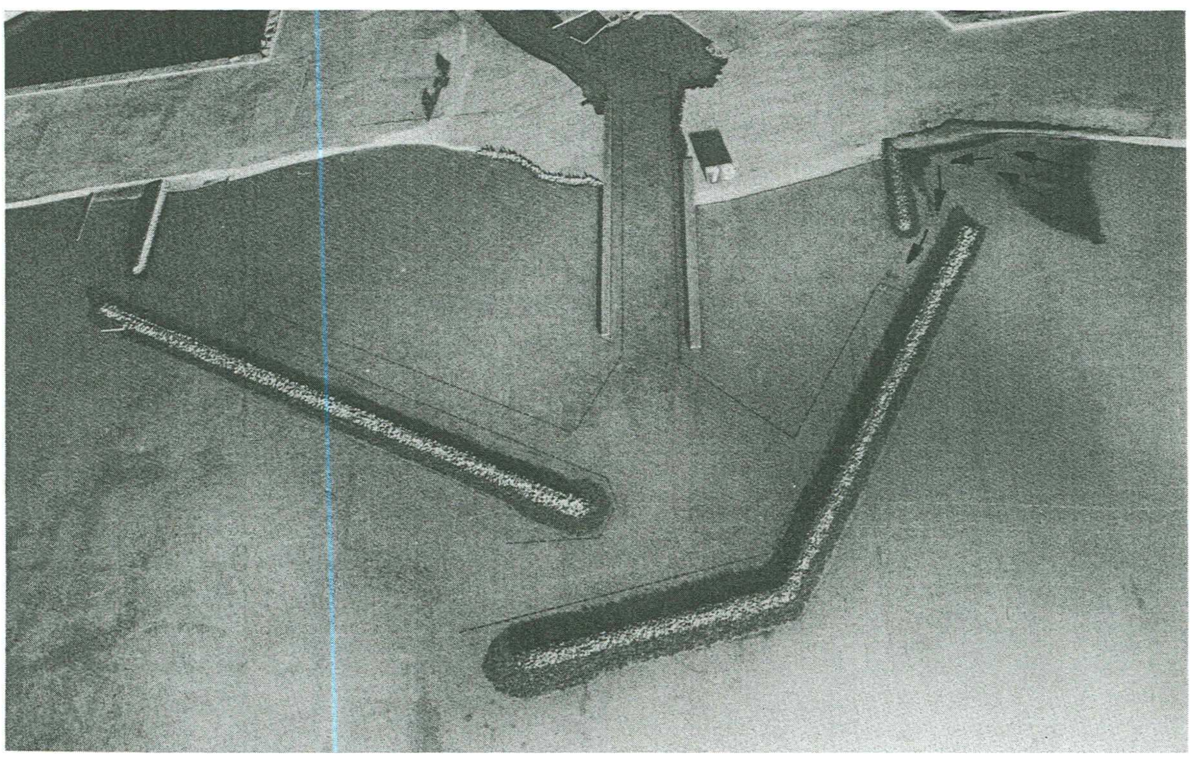


b. 6.4-sec, 6.3-ft waves; +4.0-ft swl

Photo 64. General movement of tracer material and subsequent deposits west of the harbor for test waves from 313 deg for Plan 17

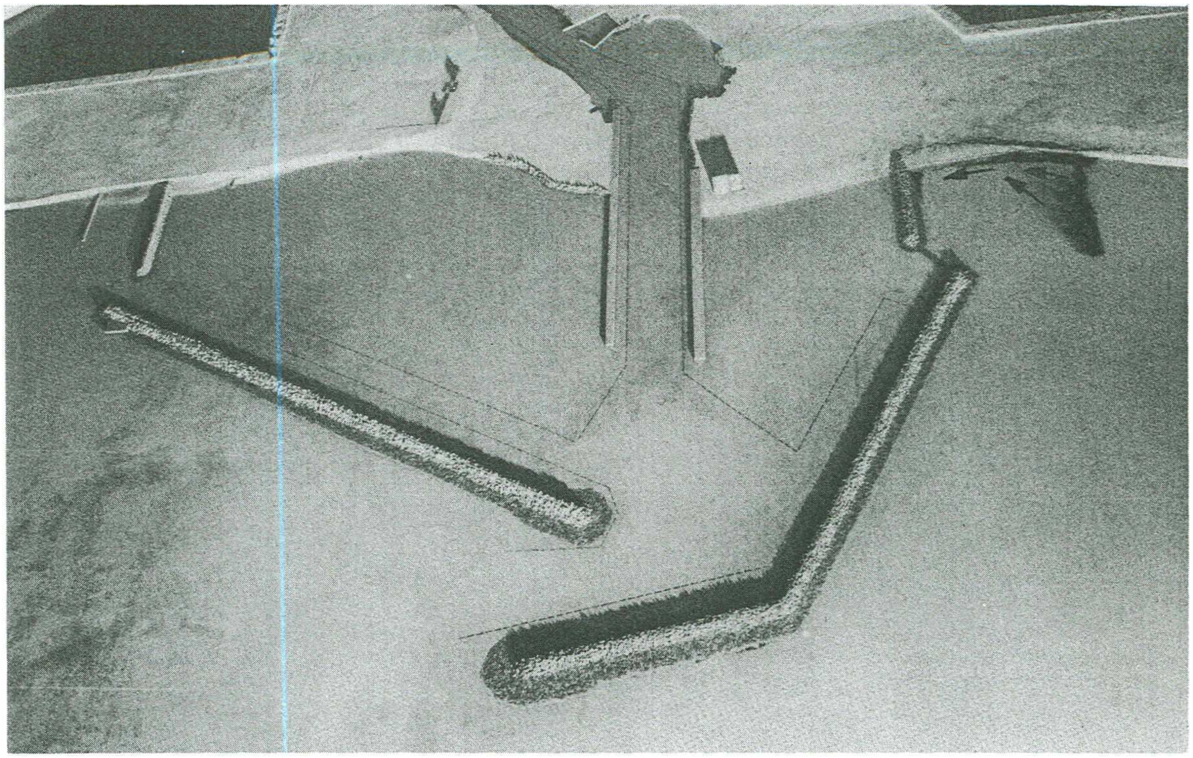


a. 7.4-sec, 8-ft waves; +2.8-ft swl

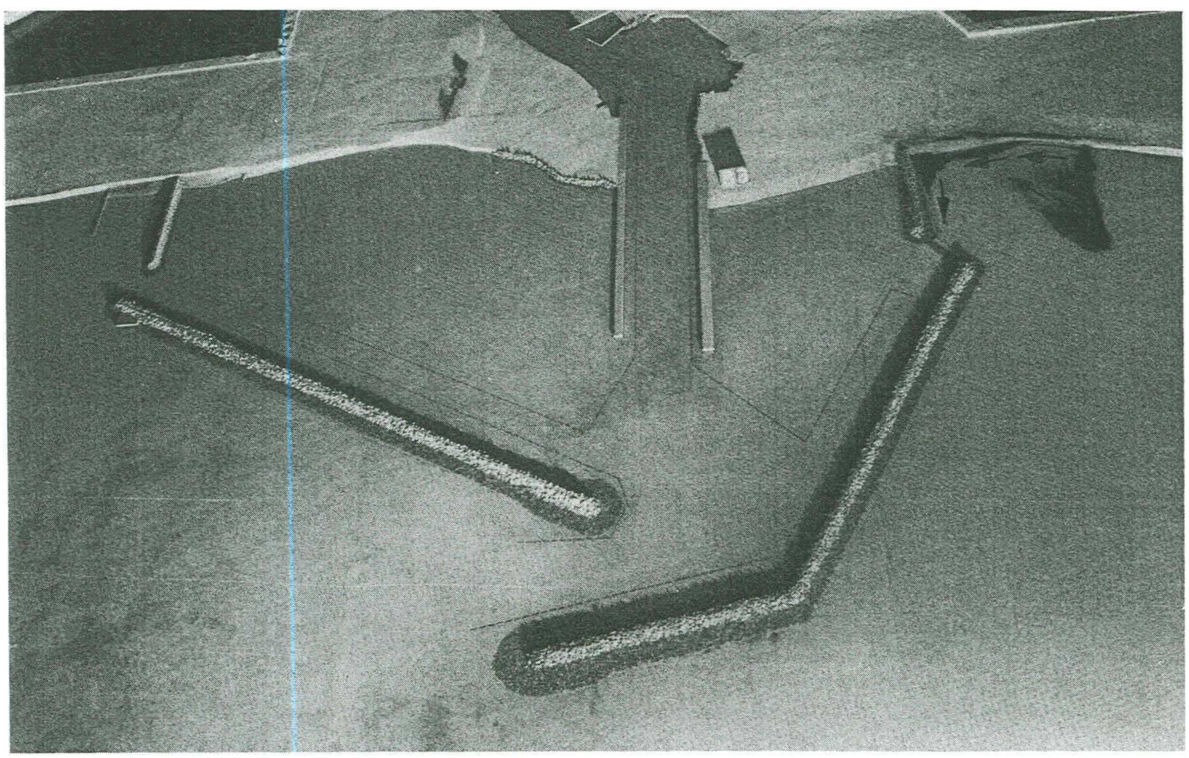


b. 6.4-sec, 6.3-ft waves; +4.0-ft swl

Photo 65. General movement of tracer material and subsequent deposits west of the harbor for test waves from 313 deg for Plan 20

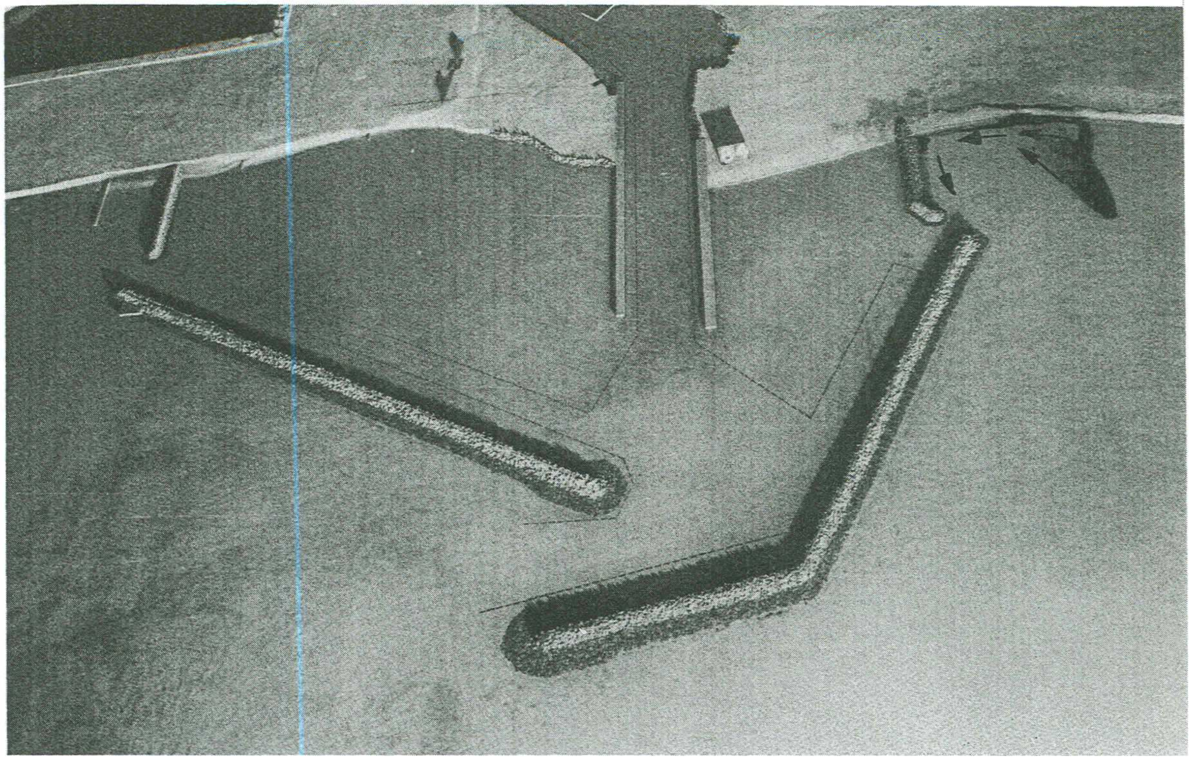


a. 7.4-sec, 8-ft waves; +2.8-ft swl

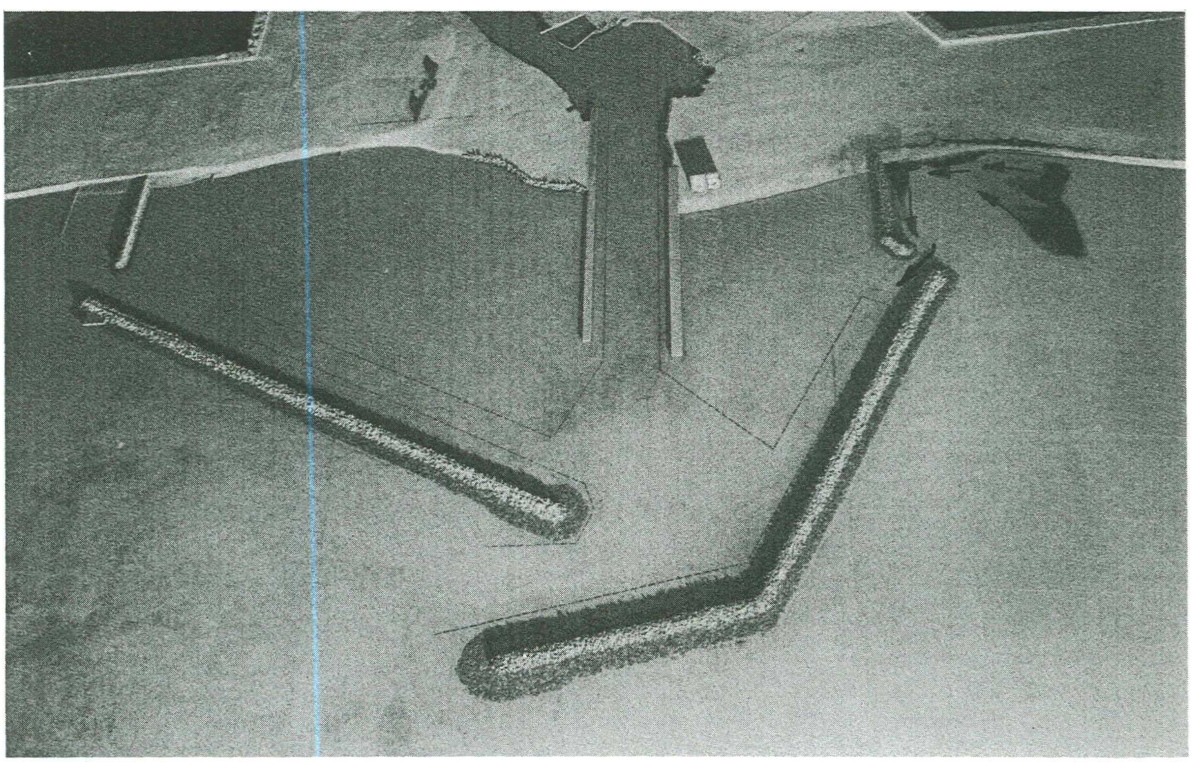


b. 6.4-sec, 6.3-ft waves; +4.0-ft swl

Photo 66. General movement of tracer material and subsequent deposits west of the harbor for test waves from 313 deg for Plan 21

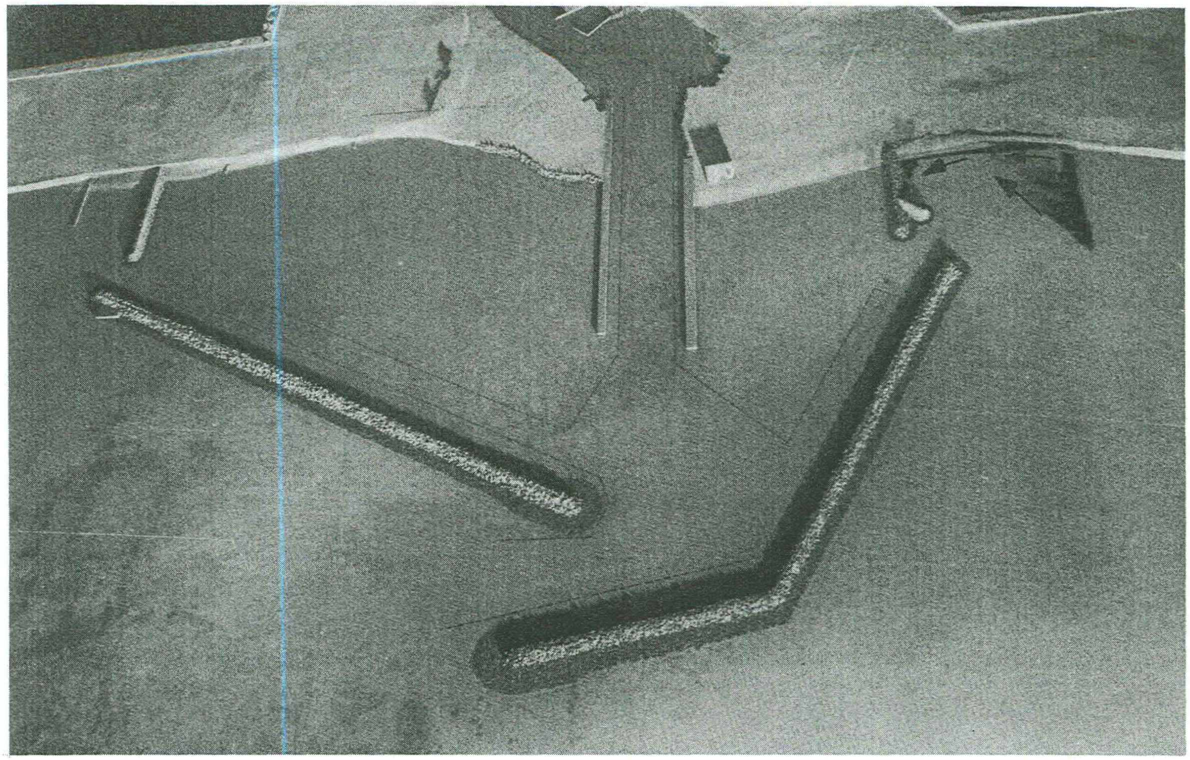


a. 7.4-sec, 8-ft waves; +2.8-ft swl

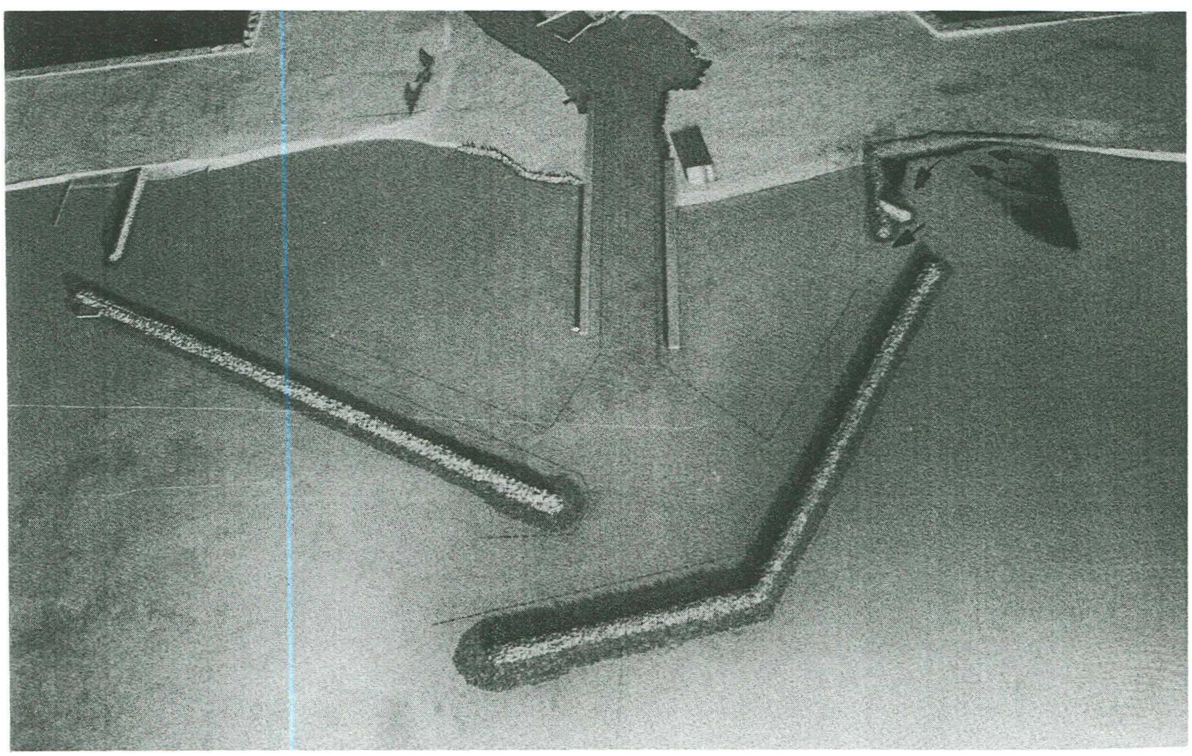


b. 6.4-sec, 6.3-ft waves; +4.0-ft swl

Photo 67. General movement of tracer material and subsequent deposits west of the harbor for test waves from 313 deg for Plan 22

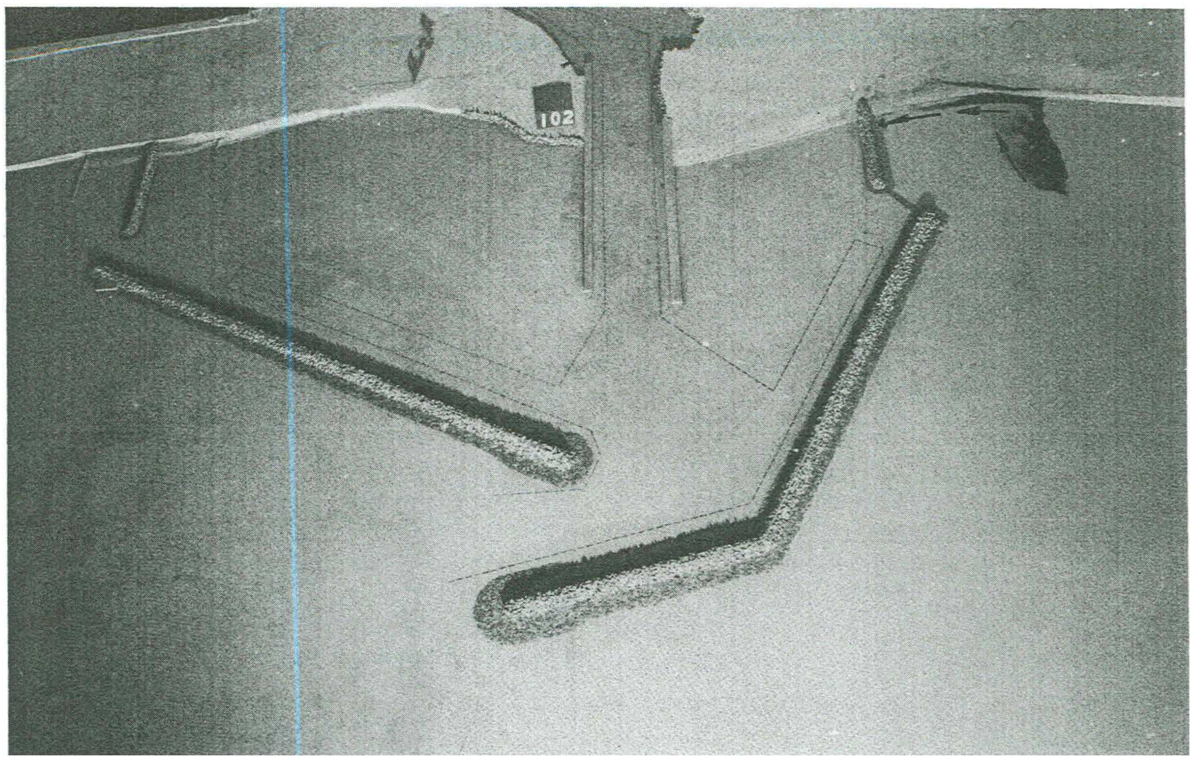


a. 7.4-sec, 8-ft waves; +2.8-ft swl

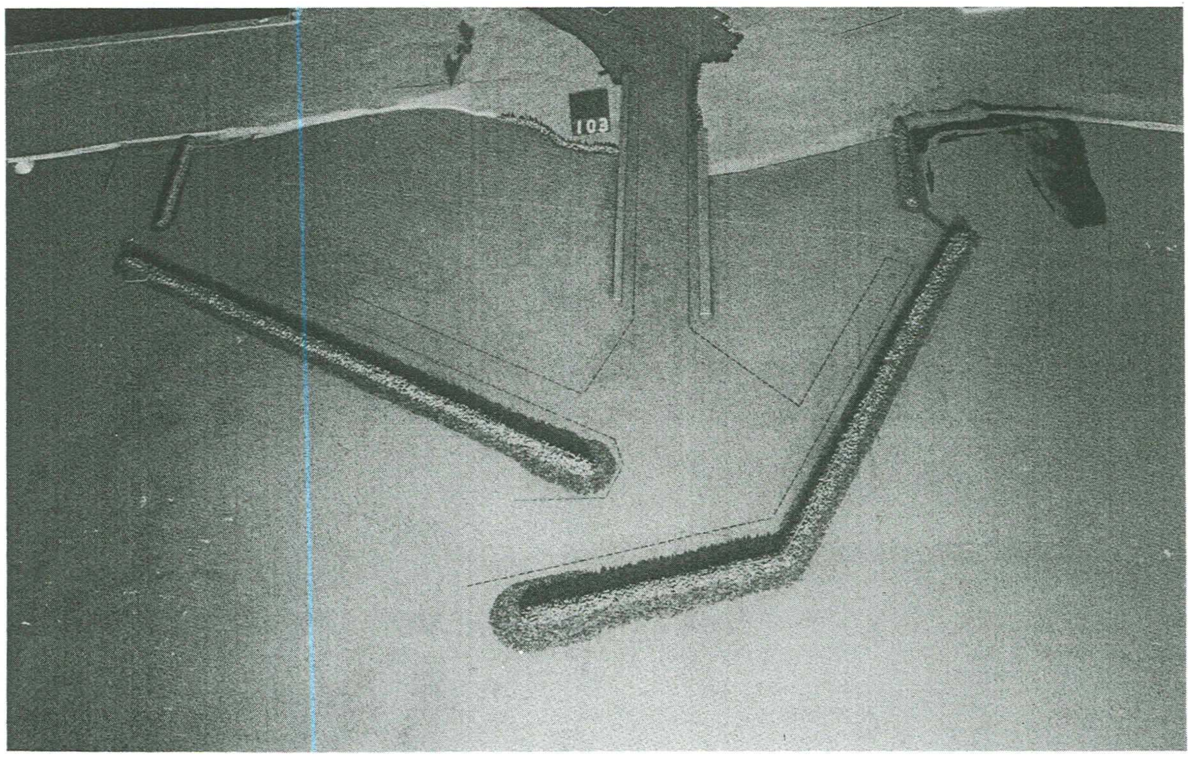


b. 6.4-sec, 6.3-ft waves; +4.0-ft swl

Photo 68. General movement of tracer material and subsequent deposits west of the harbor for test waves from 313 deg for Plan 23

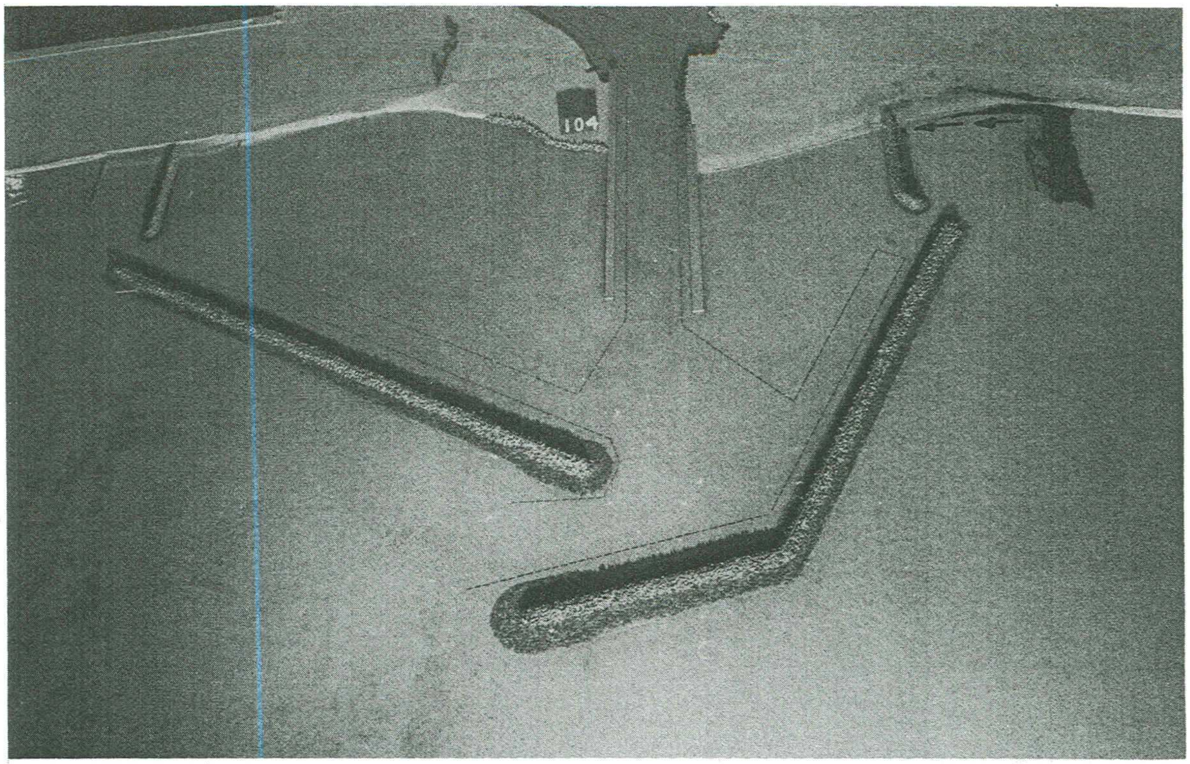


a. 7.4-sec, 8.8-ft waves; +2.8-ft swl

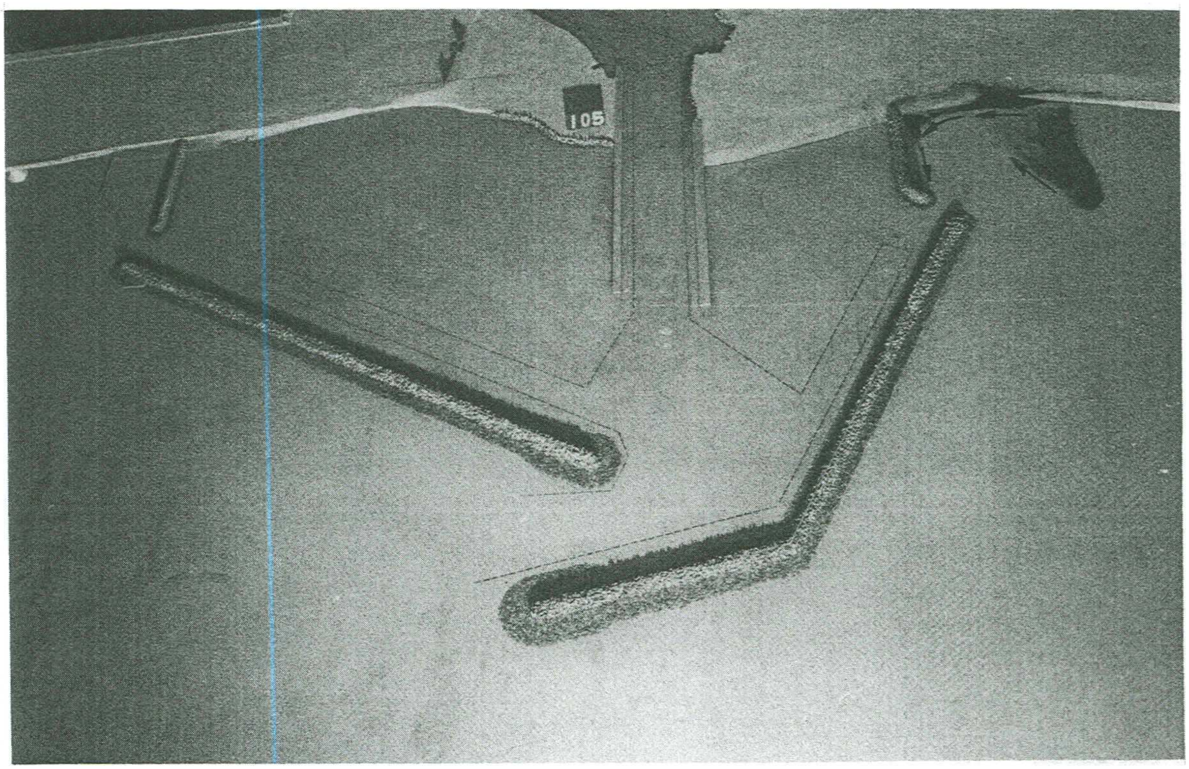


b. 6.4-sec, 6.S-ft waves; +4.0-ft swl

Photo 69. General movement of tracer material and subsequent deposits west of the harbor for test waves from 334 deg for Plan 21

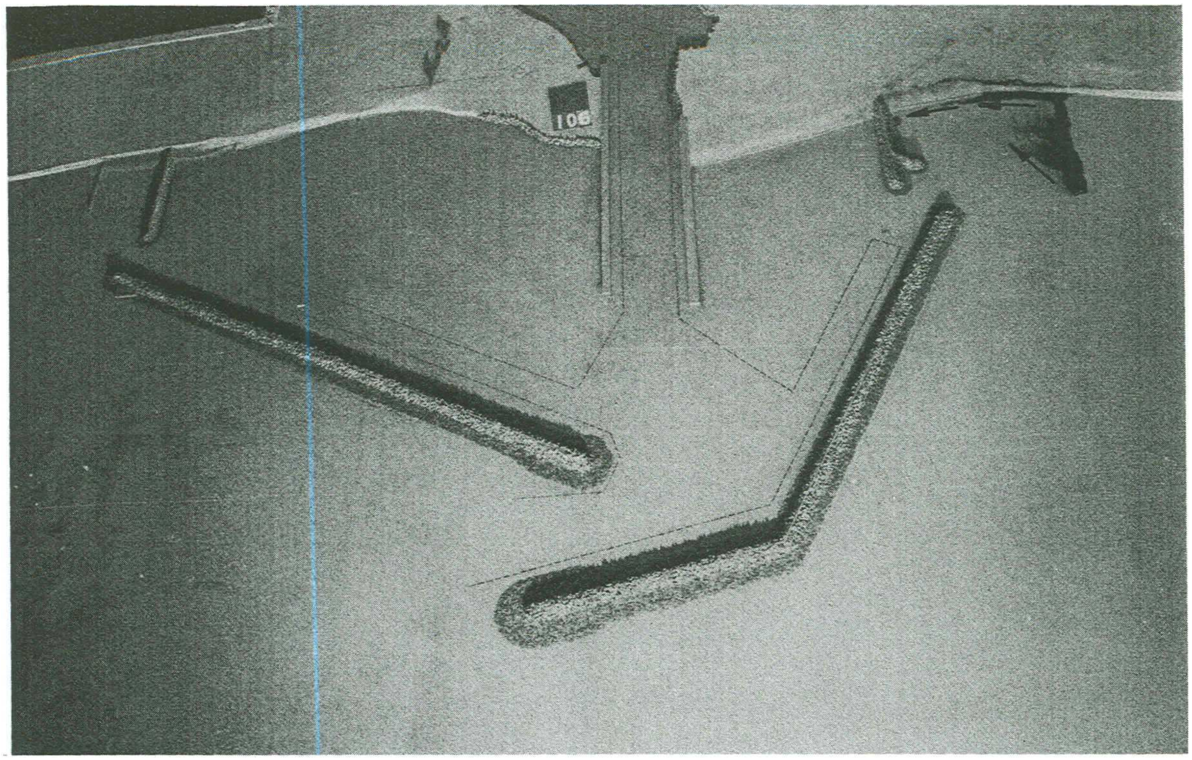


a. 7.4-sec. 8.8-ft waves; +2.8-ft swl

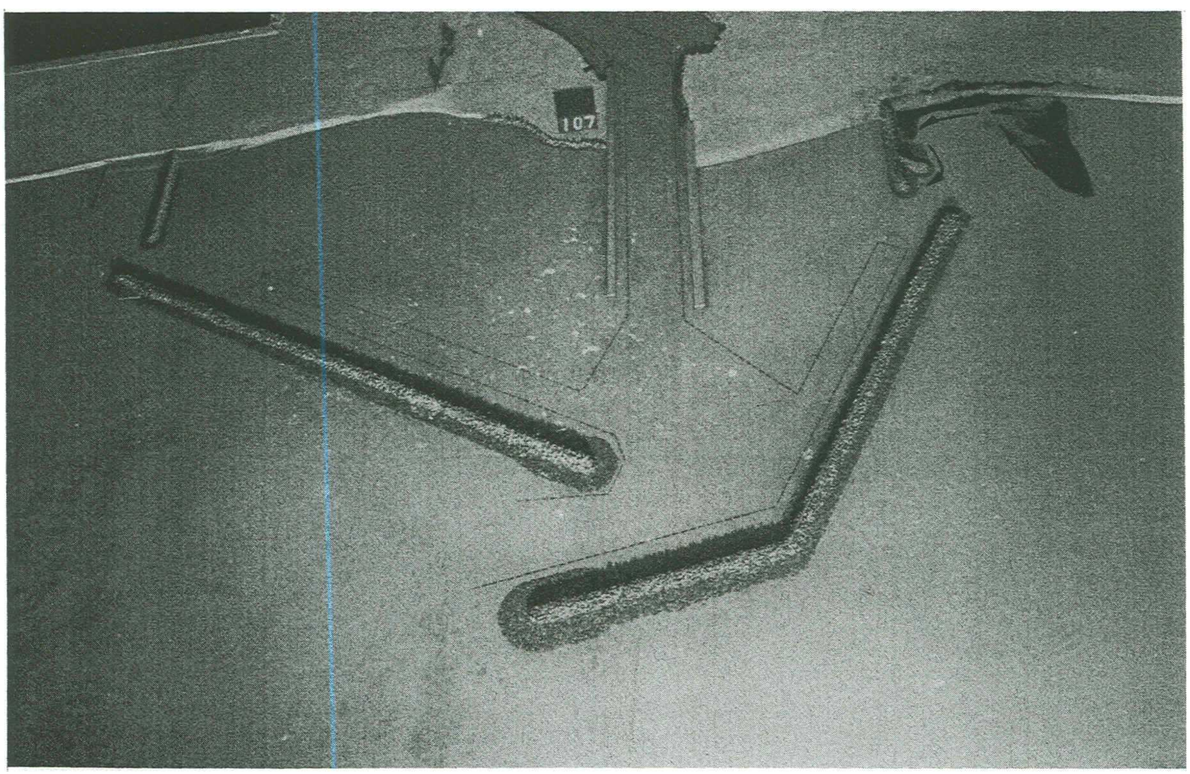


b. 6.4-sec, 6.S-ft waves; +4.0-ft swl

Photo 70. General movement of tracer material and subsequent deposits west of the harbor for test waves from 334 deg for Plan 22

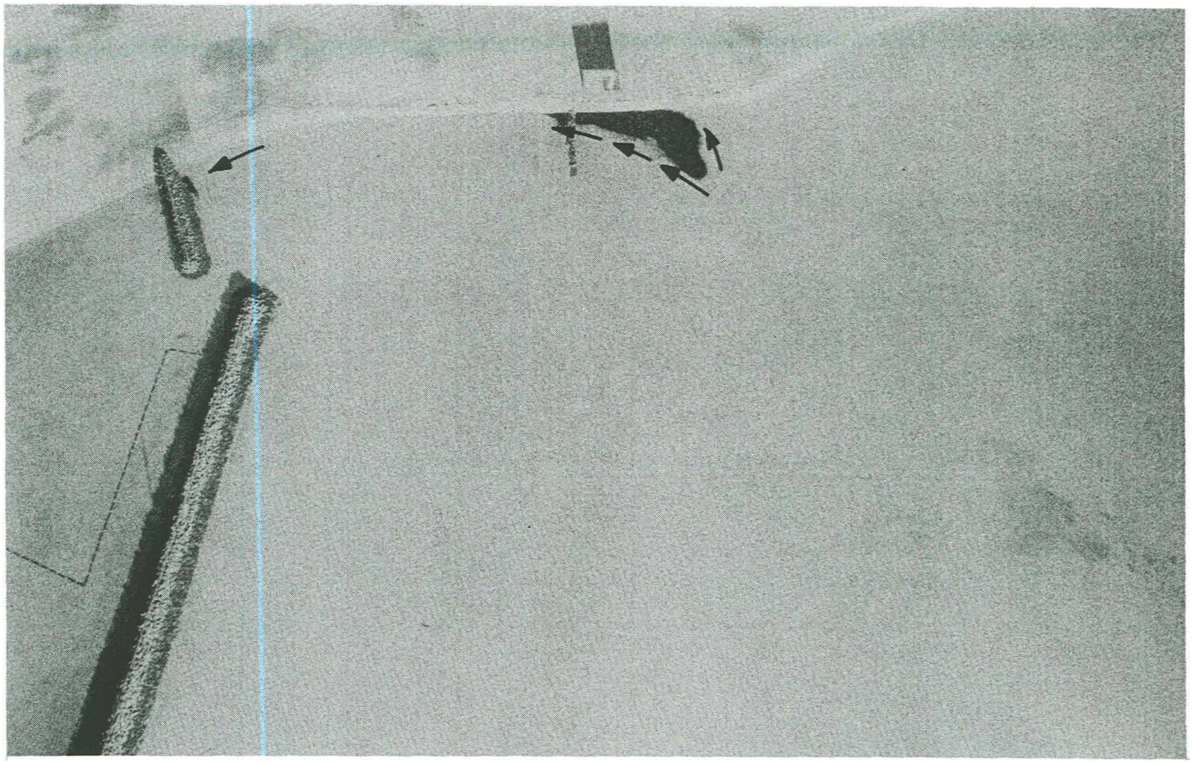


a. 7.4-sec, 8.8-ft waves; +2.8-ft swl

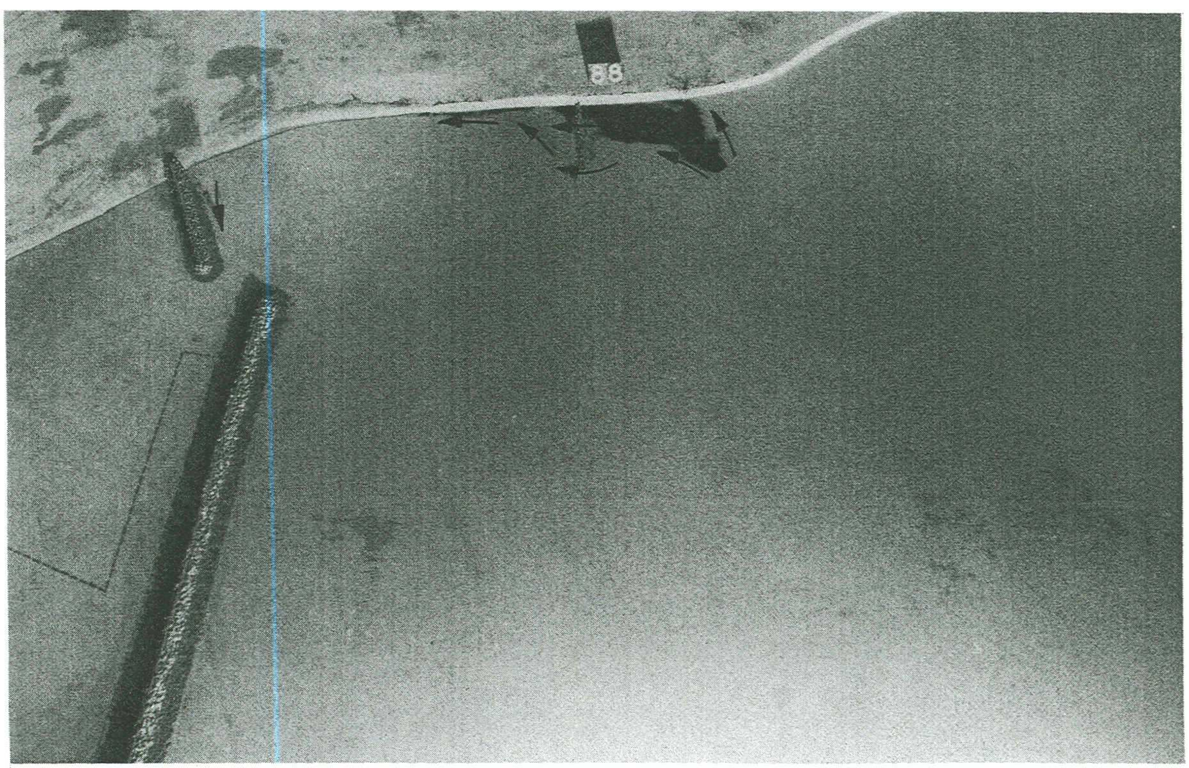


b. 6.4-sec, 6.S-ft waves; +4.0-ft swl

Photo 71. General movement of tracer material and subsequent deposits west of the harbor for test waves from 334 deg for Plan 23



a. 7.4-sec, 8-ft waves; +2.8-ft swl



b. 6.4-sec, 6.3-ft waves; +4.0-ft swl

Photo 72. General movement of tracer material and subsequent deposits around the existing groin west of the harbor for test waves from 313 deg

NOTE. CONTOURS AND ELEVATIONS ARE SHOWN IN FEET REFERRED TO LOW WATER DATUM (LWD)

LEGEND

WAVE GAGE LOCATION

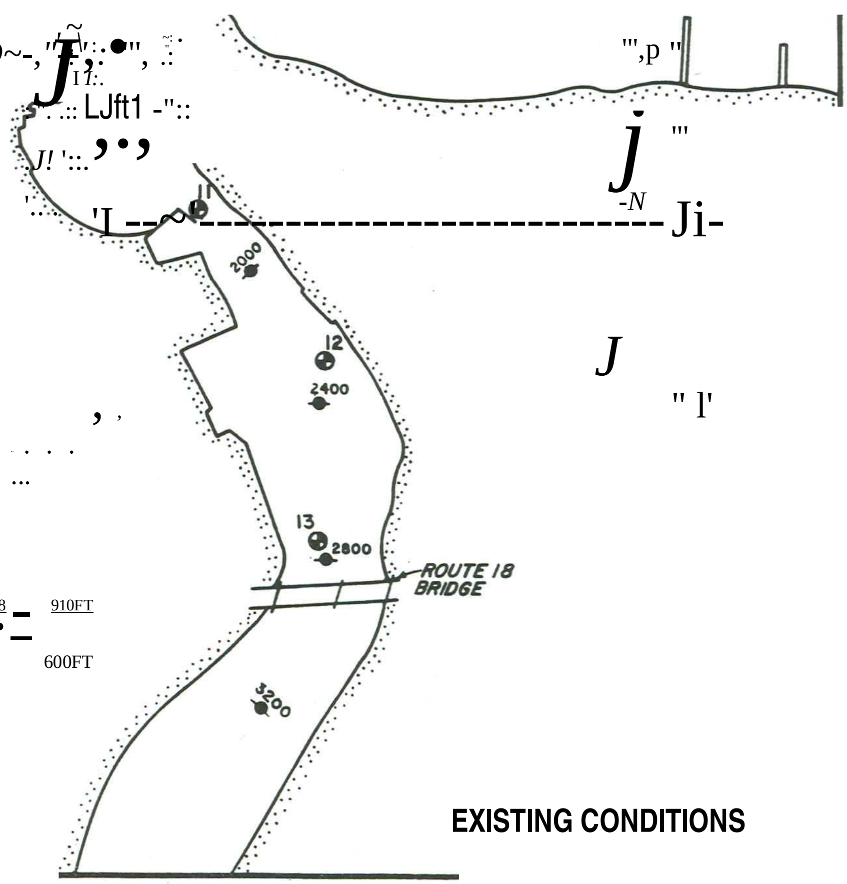
DISTANCE (FT) UPSTREAM OF -12 FT CONTOUR

-18

CENTER OF WATERS

PROPOSED

400



MODEL 0 1 2 3 4 5 6 7 8 9 10 FT

PROTOTYPE 0 200 400 600 FT

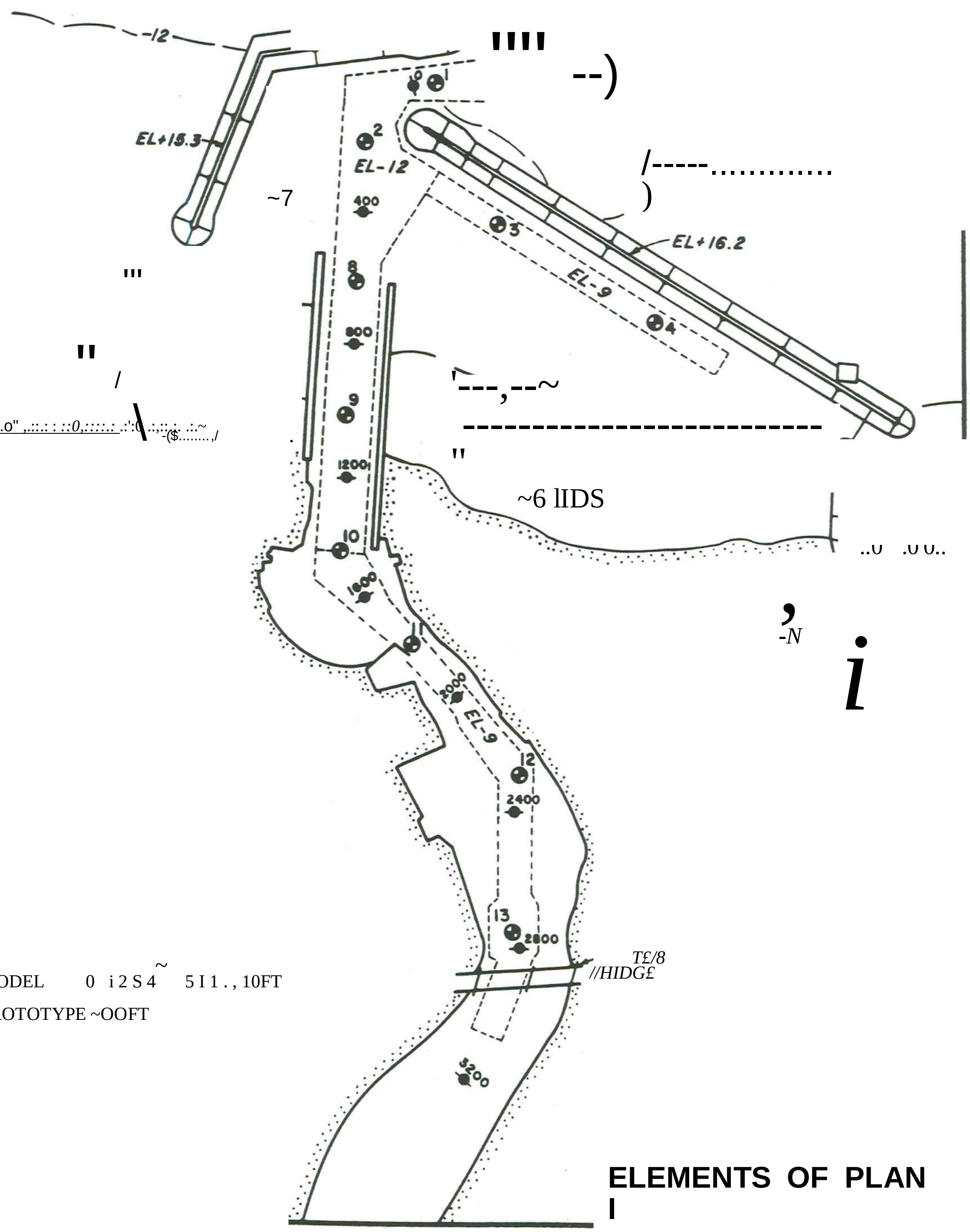
EXISTING CONDITIONS

NOTE' CONTOURS AND ELEVATIONS ARE SHOWN IN
FEET REFERRED TO LOW WATER DATUM (LWD)

LEGEND

qj WAVE"GAGELOCATION

.. DISTANCE (FT) UPSTREAM OF
-12 FT CONTOUR



MODEL 0 i 2 S 4 5 I 1 . , 10FT
PROTOTYPE ~OOFT

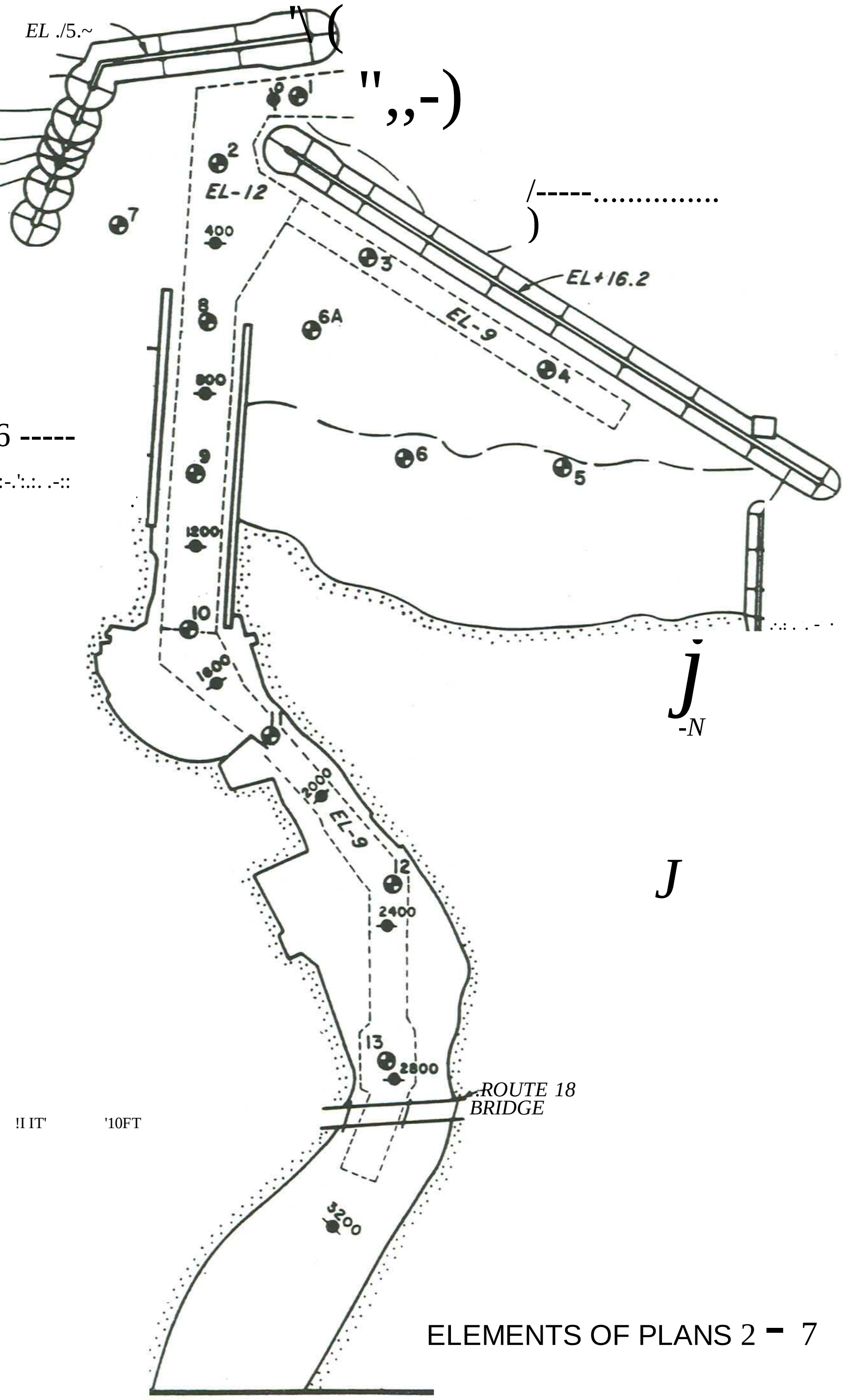
ELEMENTS OF PLAN I

NOTE. CONTOURS AND ELEVATIONS ARE SHOWN IN FEET REFERRED TO LOW WATER DATUM (LWD)

LEGEND

WAVE GAGE LOCATION

DISTANCE (FT) UPSTREAM OF -12 FT CONTOUR



MODEL 0 1 2 3 4 5 FT

PROTOTYPE 0 400 FT

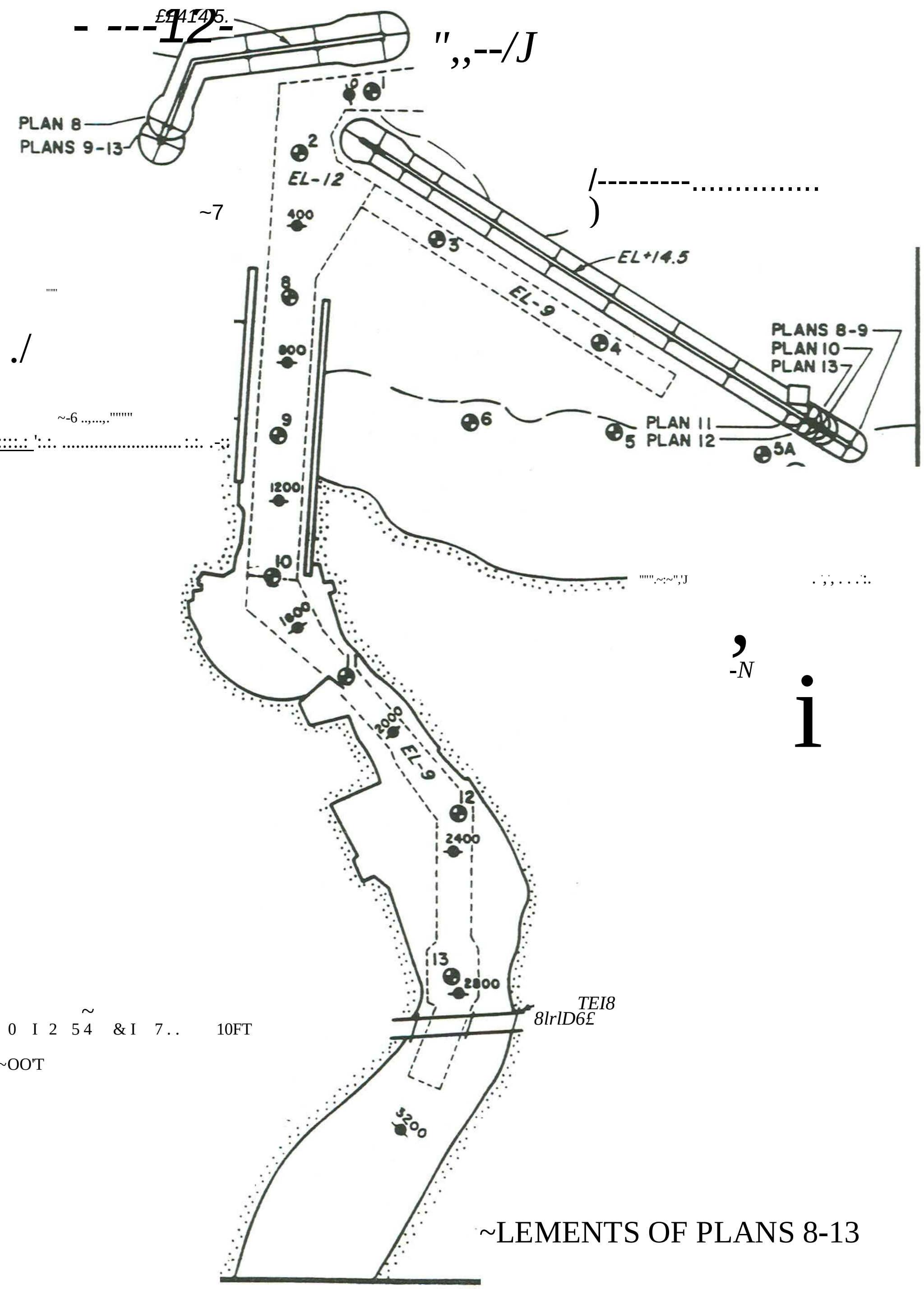
ELEMENTS OF PLANS 2 - 7

NOTE. CONTOURS AND ELEV~IOHS ARE SHOWN IN
FEET REFERRED TO LOW WATER DATUM ILWD)

LEGEND

~ WAVEGAGELOCATION

.. DISTANCE 1FT) UPSTREAM
OF -12 FT CONTOUR



MODEL 0 I 2 5 4 & I 7 . . 10FT

PROTOTYPE ~OO'T

~LEMENTS OF PLANS 8-13

PLATE 4

NOTES:

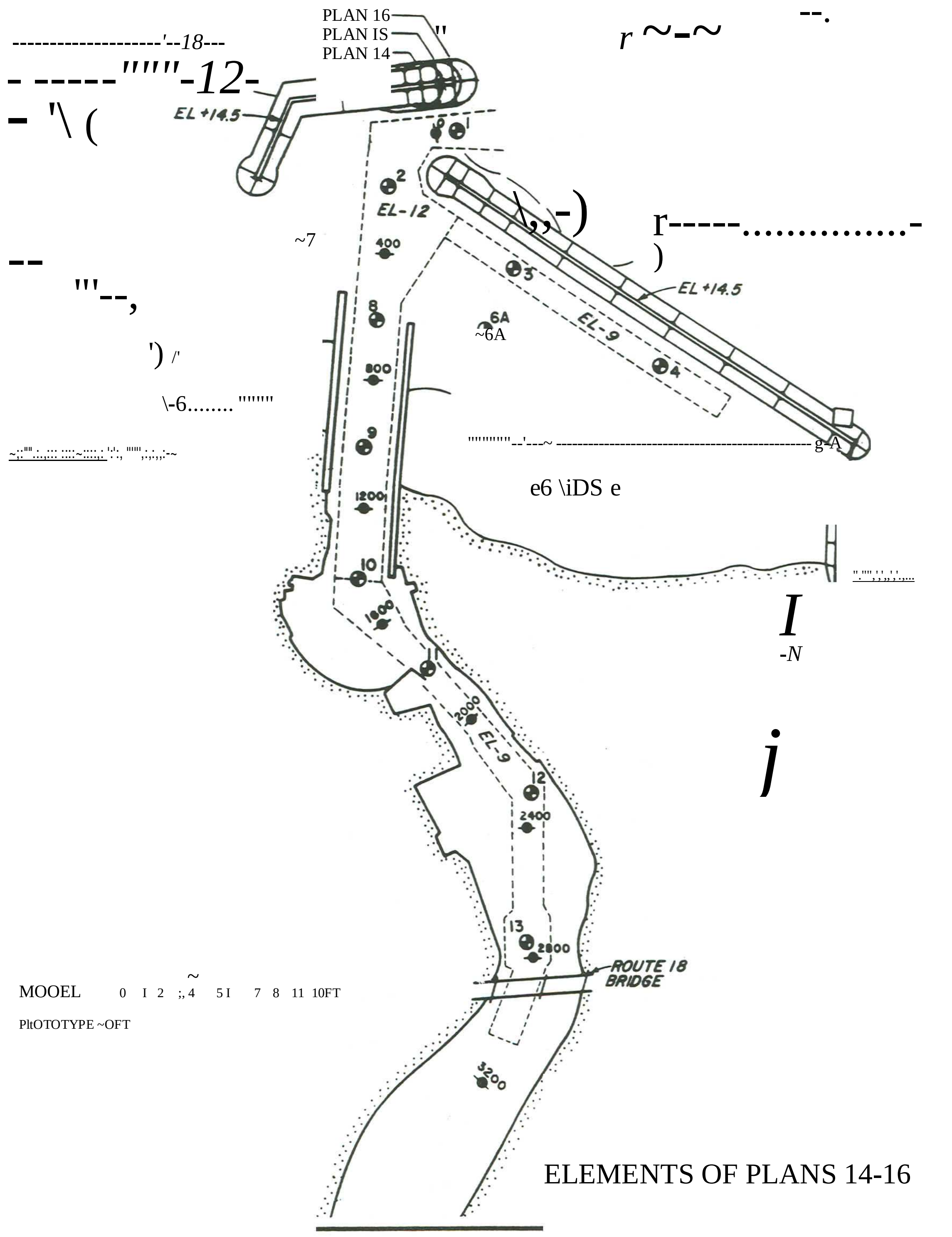
I. CONTOURS AND ELEVATIONS ARE SHOWN IN FEET REFERRED TO LOW WATER DATUM (LWD)

Z, GAGE I WAS CENTERED BETWEEN BREAKWATER HEADS FOR EACH PLAN

LEGEND

~I WAVE GAGE LOCATION

.. DISTANCE 1FT) UPSTREAM OF -12 FT CONTOUR



ELEMENTS OF PLANS 14-16