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January 1996

US Army Corps of Engineers
Waterways Experiment Station

Newport North Marina, Yaquina Bay, Oregon, Design for Wave Protection

Coastal Model Investigation

by Robert R. Bottin, Jr., Michael J. Briggs

Prepared for U.S. Army Engineer District, Portland

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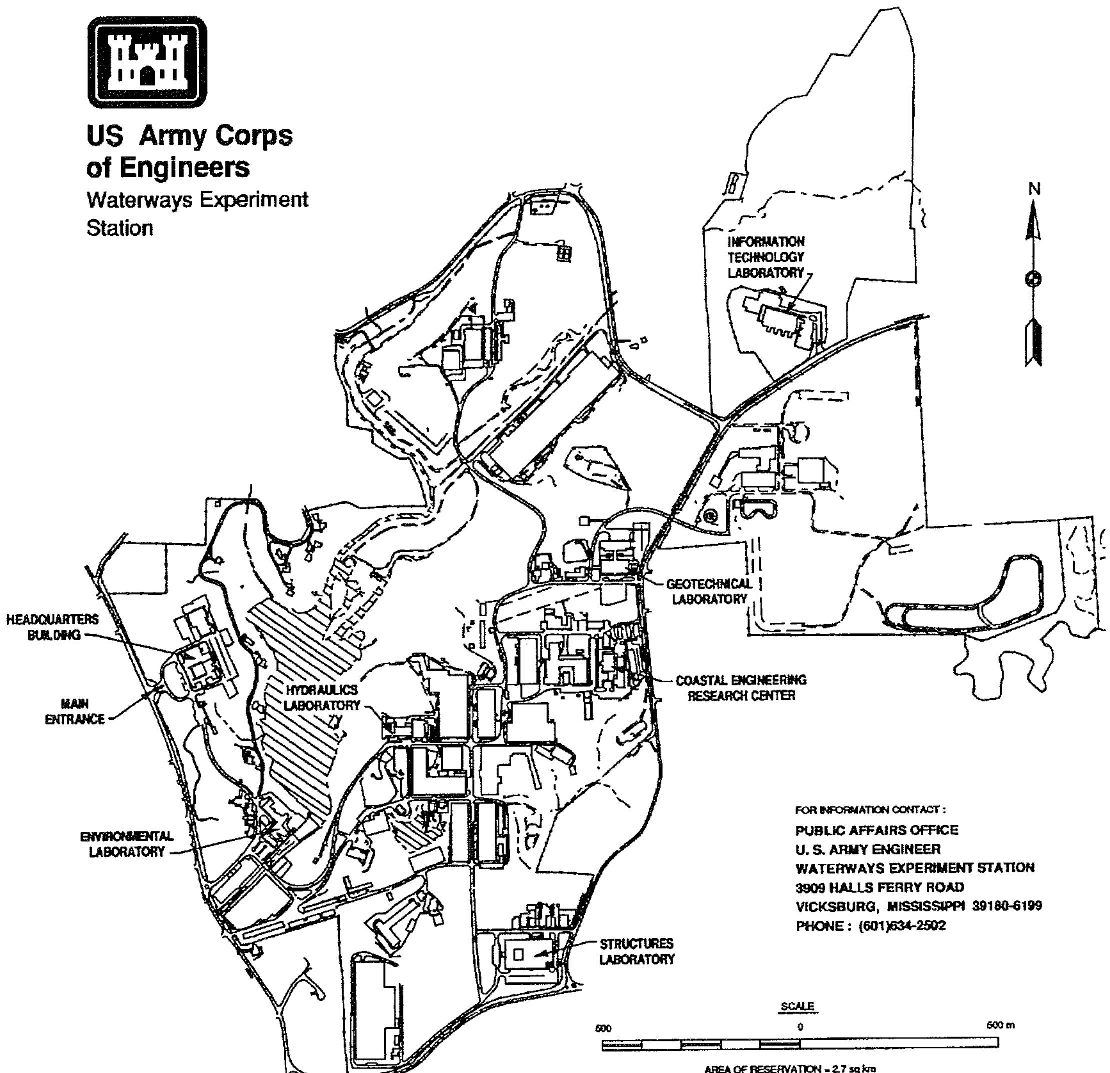

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Contents

Preface

A request for a model investigation to study breakwater modifications at Newport North Marina, Yaquina Bay, OR, was initiated by the U.S. Army Engineer District, Portland (NPP) in a letter to the U.S. Army Engineer Division, North Pacific (NPD). Authorization for the U.S. Army Engineer Waterways Experiment Station (WES), Coastal Engineering Research Center (CERC), to perform the study was subsequently granted by Headquarters, U.S. Army Corps of Engineers. Funds were provided by the NPP on 18 November 1994 and 3 March 1995.

Model tests were conducted at WES during the period May through July 1995 by personnel of the Wave Processes Branch (WPB) of the Wave Dynamics Division (WDD), CERC, under the direction of Dr. James R. Houston and Mr. Charles C. Calhoun, Jr., Director and Assistant Director of CERC, respectively; and under the direct guidance of Messrs. C. E. Chatham, Jr., Chief of WDD; and Dennis G. Markle, Chief of WPB. Model design and construction were supervised by Messrs. Larry A. Barnes, Civil Engineering Technician, and Michael J. Briggs, Research Hydraulic Engineer. Tests were conducted by Messrs. Hugh F. Acuff and Cecil Dorrell, Civil Engineering Technicians, and William G. Henderson, Computer Assistant, under the supervision of Mr. Robert R. Bottin, Jr., Research Physical Scientist. Mr. Henderson performed all data analysis during the investigation. This report was prepared by Messrs. Bottin and Briggs.

Prior to the model investigation, Messrs. Briggs and Barnes met with representatives of NPP and visited the Newport North Marina site. During the course of the study, liaison was maintained by means of conferences, telephone communications, and monthly progress reports. Ms. Heidi Moritz was technical point of contact for NPP. The following personnel visited WES to attend conferences and/or observe model operation during the course of the study.

Mr. Brad Bird	NPD
COL Tim Wood	Commander, NPP
Mr. Bill Branch	NPP
Ms. Heidi Moritz	NPP
Mr. Bud Shoemake	Harbor Master, Newport North Marina

Dr. Robert W. Whalin was Director of WES during model testing and the preparation and publication of this report. COL Bruce K. Howard, EN, was Commander.

Conversion Factors, Non-SI to SI Units of Measurement

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

Multiply	By	To Obtain
cubic feet per second	0.02831685	cubic meters per second
cubic yards	0.7646	cubic meters
degrees (angle)	0.01745329	radians
feet	0.3048	meters
feet per second	0.3048	meters per second
inches	2.54	centimeters
miles (U.S. statute)	1.609347	kilometers
pounds (mass)	0.4536	kilograms
pounds (mass) per cubic foot	16.02	kilograms per cubic meter
square feet	0.09290304	square meters
square miles (U.S. statute)	2.589988	square kilometers
tons (2,000 lb, mass)	907.1848	kilograms

1 Introduction

The Prototype

The Yaquina Bay estuary is located on the Oregon coast about 185 km (115 miles)[1] south of the Washington border (Figure 1). The major tributary to the estuary is the Yaquina River, which drains approximately 650 sq km (250 sq miles) of largely forested area on the west side of the Coast Range. Two rubble-mound jetties have been constructed at the mouth of the Yaquina River. The north jetty is 2,134 m (7,000 ft) long and the south jetty is 2,621 m (8,600 ft) in length. The distance between the jetties is 305 m (1,000 ft) at their outer ends.

Newport North Marina is situated on the north bank of the Yaquina River about 3.2 km (2 miles) upstream from the seaward ends of the Yaquina River jetties (Figure 2). The marina was constructed in 1946 and includes an 808-m (2,650-ft) timber breakwater that protects a small-boat marina from wave action. The crest elevation of the timber structure was constructed to +4.3 m (+14 ft)[2] relative to mean lower low water (mllw). The mooring areas in the marina were dredged to a depth of -3 m (-10 ft). A 1994 aerial photograph of the Newport North Marina is shown in Figure 3.

The Problem

Newport North Marina is experiencing excessive wave energy due to waves from the Pacific Ocean propagating through the west entrance. The majority of the problems are experienced in the western one-third of the marina during winter storms at high tide stages. Waves ranging from 0.9 to 1.2 m (3 to 4 ft) have been observed in the marina during storm wave conditions. In November 1981, a "3-year storm event" destroyed a port dock and caused $720,000 in

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[1] Units of measurement in this report are shown in SI (metric) units, followed by non-SI (British) units in parentheses. In addition, a table of factors for converting non-SI units of measurement used in plates, figures, photos, and tables in this report to SI units is presented on page vi.

[2] All elevations cited herein are in meters (feet) referred to mean lower low water (mllw) unless otherwise noted.

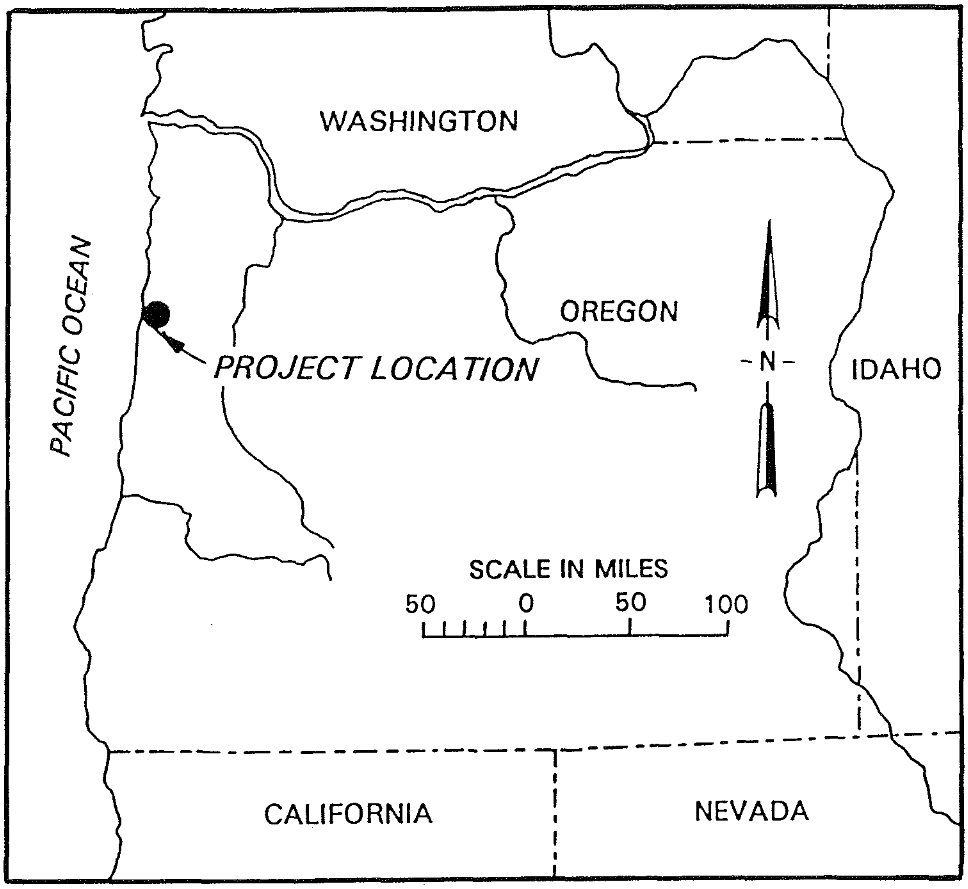


Figure 1. Project location

damages to the marina. Another dock experienced damage to water and electrical lines during January 1990 storms (U.S. Army Engineer District (USAED), Portland 1994). Overtopping of the existing deteriorated timber breakwater may occur as often as four to six times during one winter. Little wave energy appears to come in from the marina's east entrance.

Proposed Improvements

Three design alternatives were originally proposed by the Portland District (NPP) to reduce wave energy from the west by changing the marina entrance opening configuration. Either a timber or rubble-mound extension or detached breakwater concept was envisioned. The three alternatives included:

a. Straight extension of the existing breakwater to the west along the existing alignment.

b. Dogleg extension of the existing breakwater to the northwest.

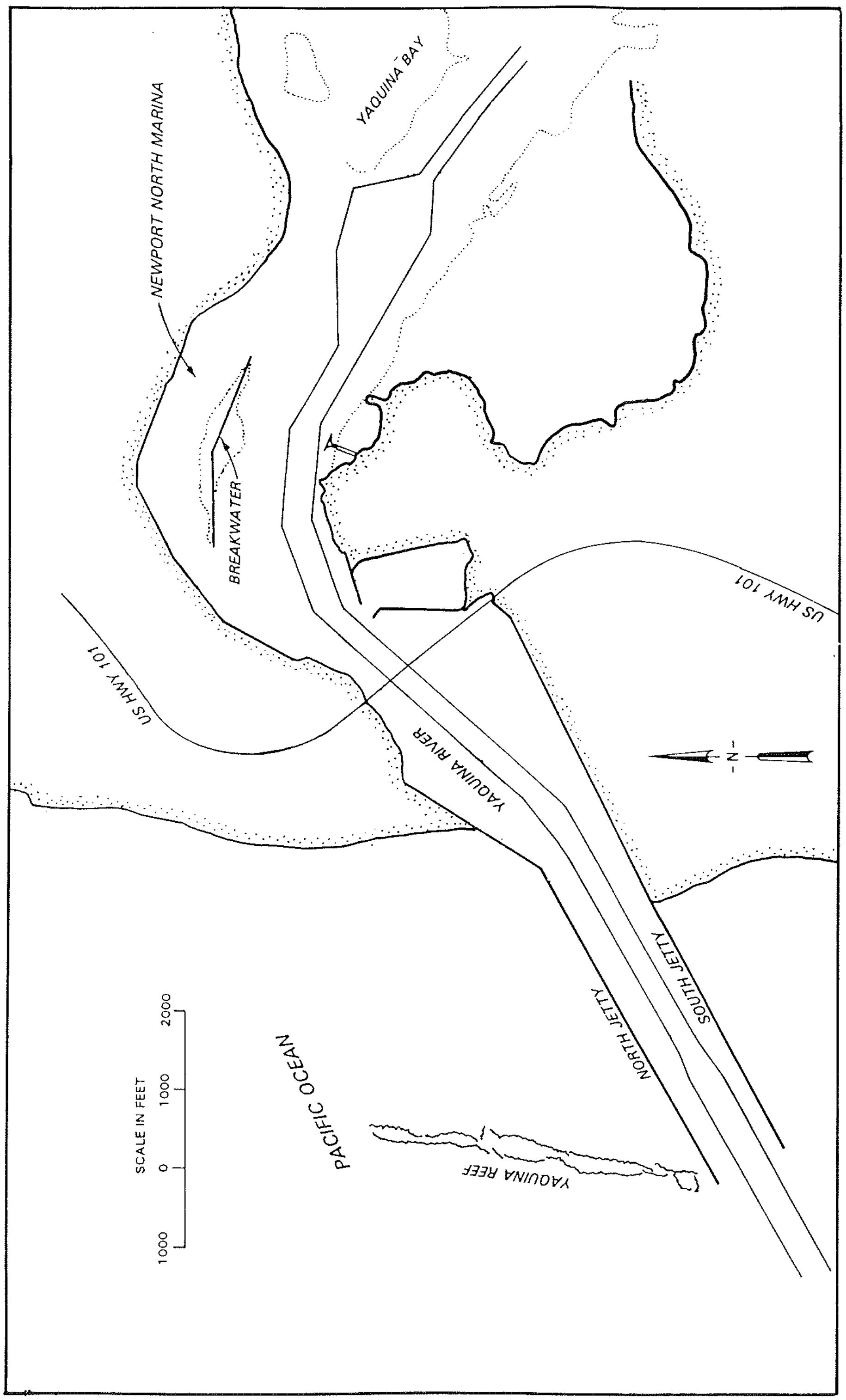


Figure 2. Newport North Marina relative to Yaquina River

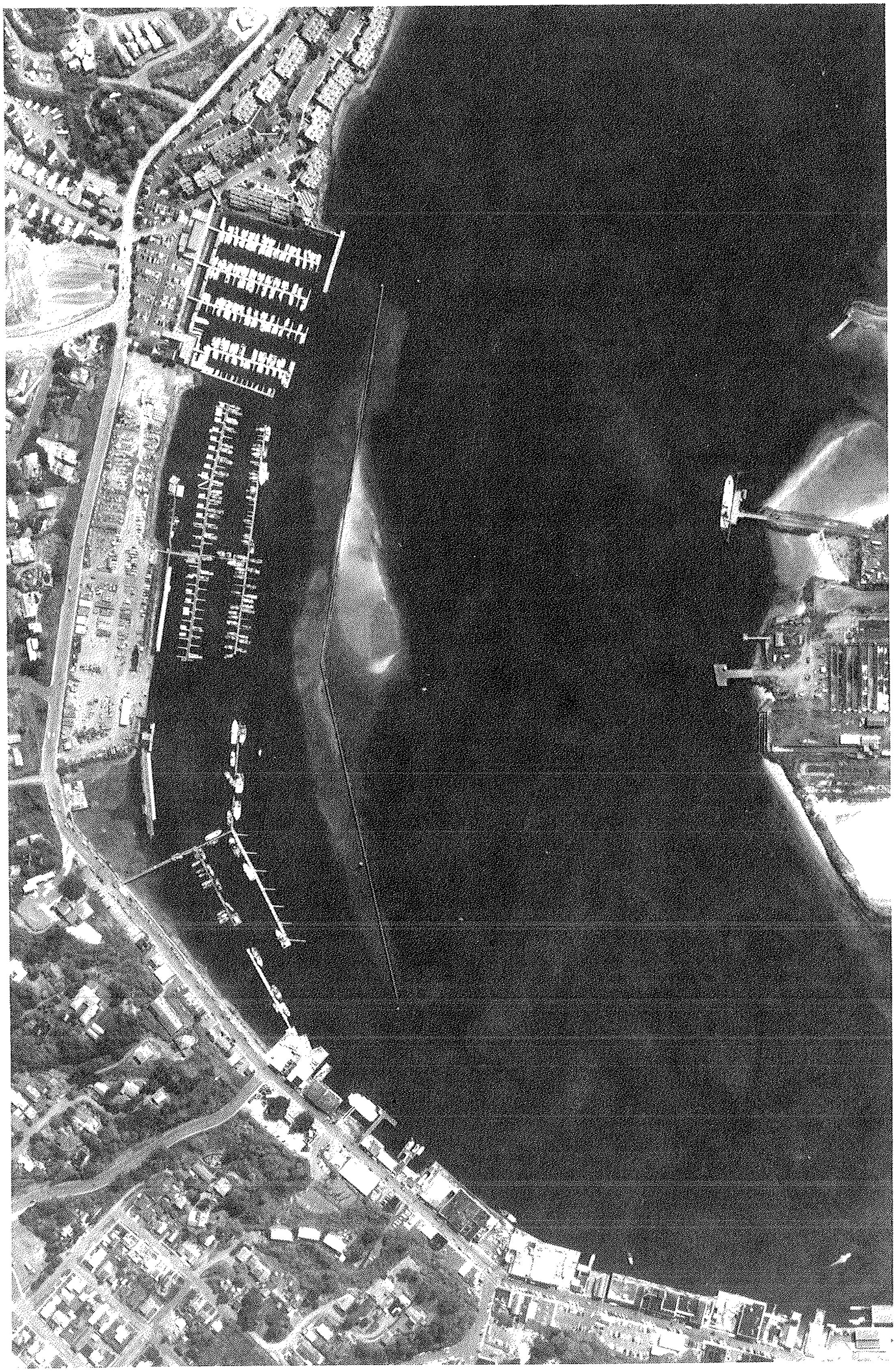


Figure 3. Aerial view of Newport North Marina

c. Detached breakwater positioned southwest of the existing west entrance.

An existing shoal around the timber breakwater retains a relatively stable configuration. In 1946, the authorizing document for the original breakwater referred to the shoal as the "middle ground." Aerial photographs dating back to 1973 indicate that the shoal has not changed significantly in recent history (USAED Portland 1994). Any changes to the marina's west entrance must ensure that sediment deposition will not adversely affect navigation or frequency of dredging. Also, water quality and basin flushing must not be adversely altered by breakwater modifications.

Purpose of the Model Study

At the request of NPP, a physical coastal hydraulic model investigation was initiated by the U.S. Army Engineer Waterways Experiment Station (WES) to:

a. Determine wave and current conditions and sediment patterns at the existing marina for storm waves approaching from the Pacific Ocean through the Yaquina River jettied entrance.

b. Determine if the proposed breakwater improvements would provide adequate wave protection to the mooring areas in the marina without adversely affecting existing facilities, ease of navigation, basin flushing, and deposition of sediment within the marina or in the entrance.

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

d. Determine if design modifications could be made to the proposed plans that could reduce construction costs without sacrificing adequate protection.

Wave-Height Criterion

Completely reliable criteria have not yet been developed for ensuring satisfactory navigation and mooring conditions in small-craft harbors during attack by storm waves. For this study, NPP initially specified that for an improvement plan to be acceptable, maximum significant wave heights were not to exceed 0.3 m (1.0 ft) in the existing marina mooring areas for storm wave conditions. As the study progressed, however, economic analyses of wave protection provided versus construction costs allowed NPP to relax the original criterion.

2 The Model

Design of Model

The Newport North Marina model was constructed to an undistorted linear scale of 1:60, model to prototype. Scale selection was based on the following factors:

a. Depth of water required in the model to prevent excessive bottom friction.

b. Absolute size of model waves.

c. Available shelter dimensions and area required for model construction.

d. Efficiency of model operation.

e. Available wave-generating and wave-measuring equipment.

f. Model construction costs.

A geometrically undistorted model was necessary to ensure accurate reproduction of wave and current patterns. Following selection of the linear scale, the model was designed and operated in accordance with Froude's model law (Stevens et al. 1942). The scale relations used for design and operation of the model were as follows:

Characteristic	Model-Prototype Dimension[1]	Scale Relations
Length	L	$L_r = 1:60$
Area	L^2	$A_r = L_r^2 = 1:3,600$
Volume	L^3	$\forall_r = L_r^3 = 1:216,000$
Time	T	$T_r = L_r^{1/2} = 1:7.75$
Velocity	L/T	$V_r = L_r^{1/2} = 1:7.75$
Discharge	L^3/T	$Q_r = L_r^{5/2} = 1:27,870$

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

The existing absorbers, revetments, groins, etc., in Yaquina River, and the proposed breakwater modifications at Newport North Marina, included the use of rubble-mound structures. Based on experience, 1:60-scale model structures should not create sufficient scale effects to warrant geometric distortion of stone sizes in order to ensure proper transmission and reflection of wave energy. Therefore, rock size selection was based on linear scale relations and a specific weight of 2,723 kg/m^3 (170 lb/ft^3) for the prototype stone.

Ideally, a quantitative, three-dimensional, movable-bed model investigation would best determine the impacts of breakwater modifications with regard to sediment deposition in the vicinity of the marina. 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 analyzed. In view of the complexities involved in conducting movable-bed model studies and due to limited funds and time for the Newport North Marina project, the model was molded in cement mortar (fixed-bed), and a tracer material was obtained to qualitatively determine sediment patterns in the area immediately adjacent to the marina entrance.

The Model and Appurtenances

The model reproduced a portion of the Yaquina River from immediately west of the U.S. Highway 101 bridge upstream and included Newport North Marina on the north bank as well as South Beach Marina on the south bank. Figure 4 shows the approximate model limits relative to the lower reaches of Yaquina River, and Figure 5 depicts detailed features included within the model limits. The total area reproduced in the model was approximately 930 sq m (10,000 sq ft), representing about 3.4 sq km (1.3 sq miles) in the prototype. Vertical control for model construction was based on mean lower low water (mllw), and horizontal control was referenced to a local prototype grid system. A general view of the model is shown in Figure 6.

Model waves were generated by a 12.2-m-long (40-ft-long), unidirectional spectral, electrohydraulic, wave generator with a trapezoidal-shaped plunger. The vertical motion of the plunger was controlled by a computer-generated command signal, and movement of the plunger caused a displacement of water which generated required test waves.

A water circulation system (Figure 5), consisting of a 20.3-cm (8-in.), perforated pipe water intake manifold, a 0.14-cms (5-cfs) pump, and sonic flow transducers with a multiprocessor transmitter, was used in the model to reproduce steady-state tidal flows through the lower reaches of the river. These flows corresponded to maximum flood and ebb tidal discharges measured in the prototype. The magnitudes of model tidal currents were measured by timing the progress of weighted floats over known distances.

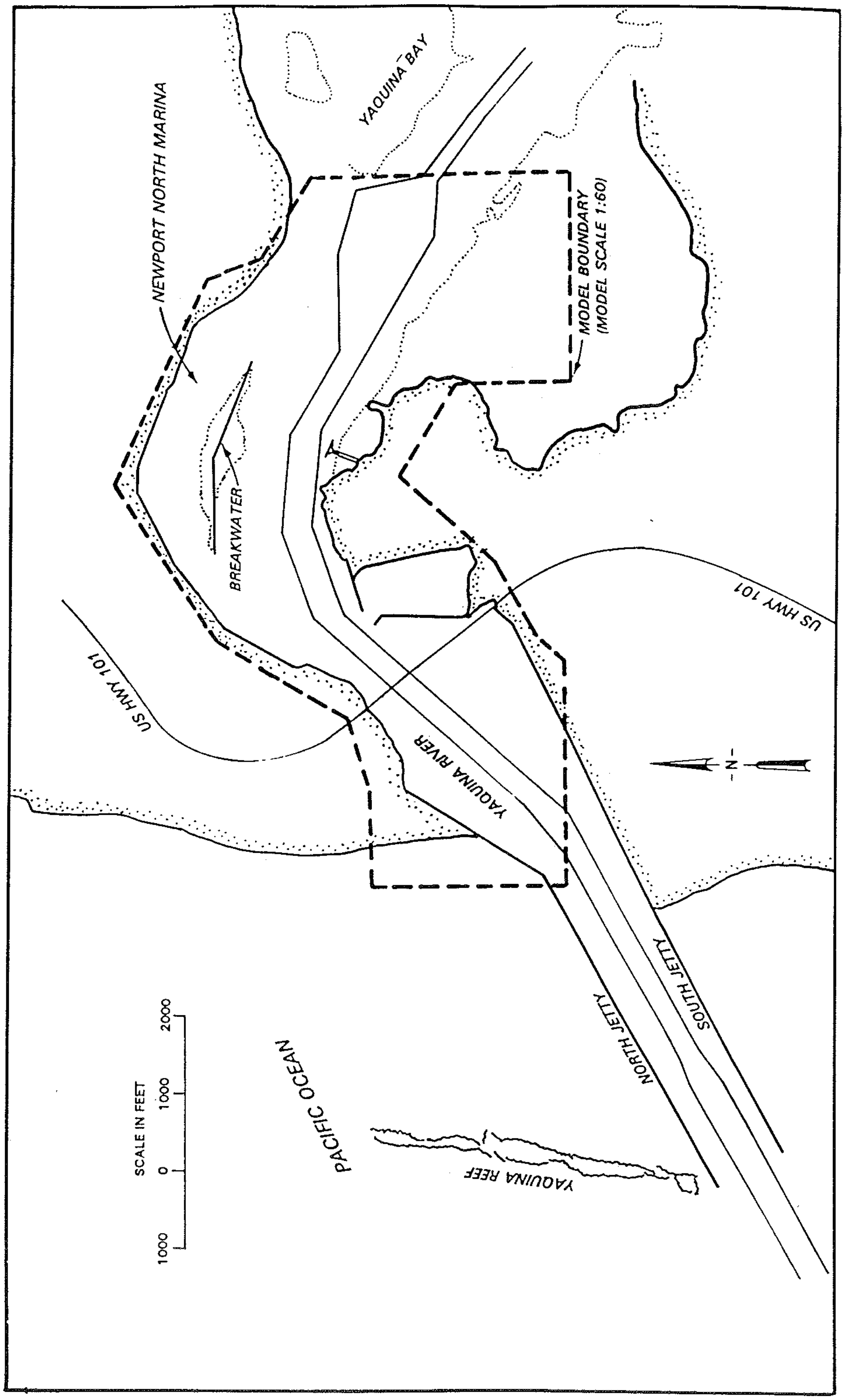


Figure 4. Approximate model limits relative to Yaquina River

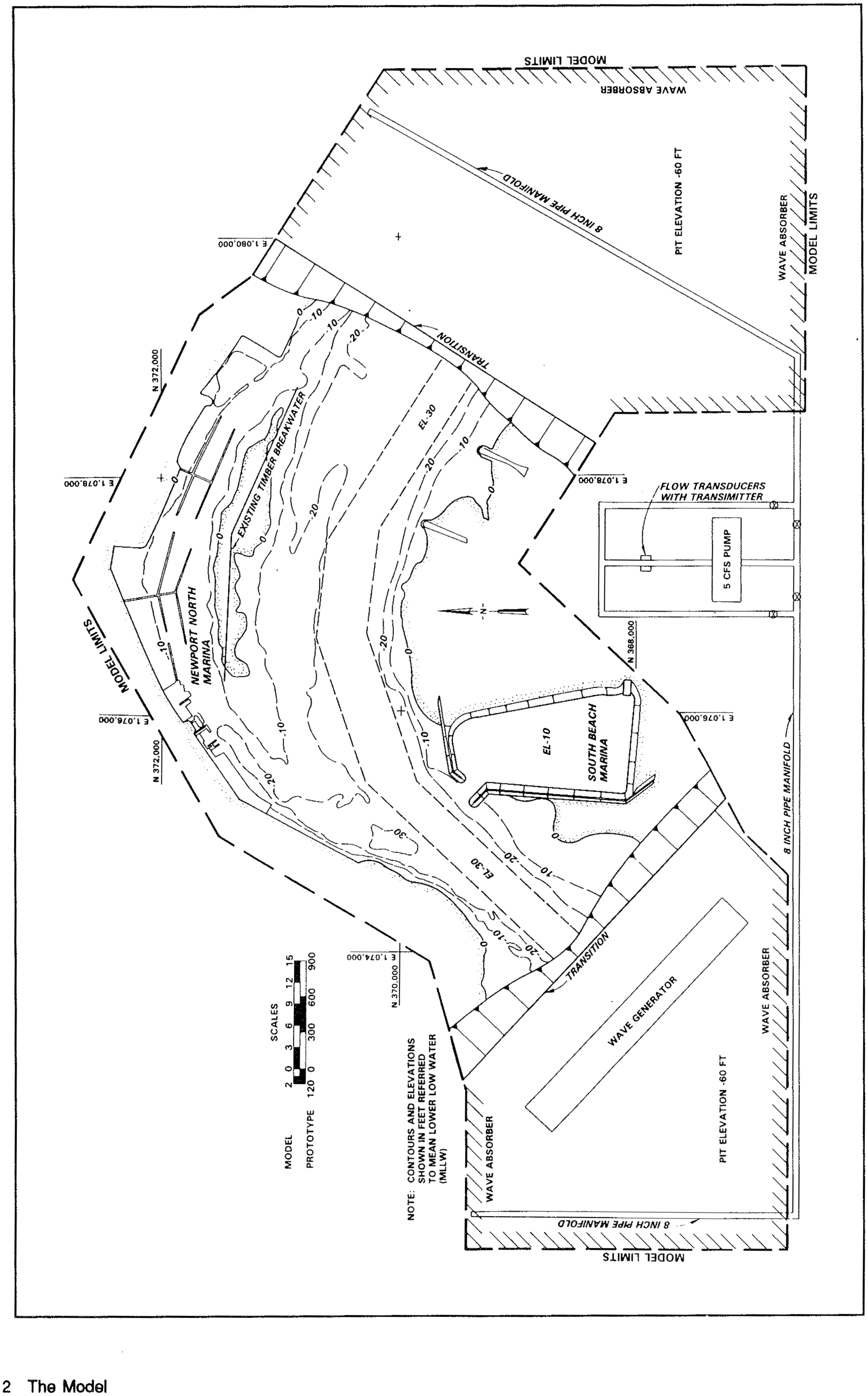


Figure 5. Model layout

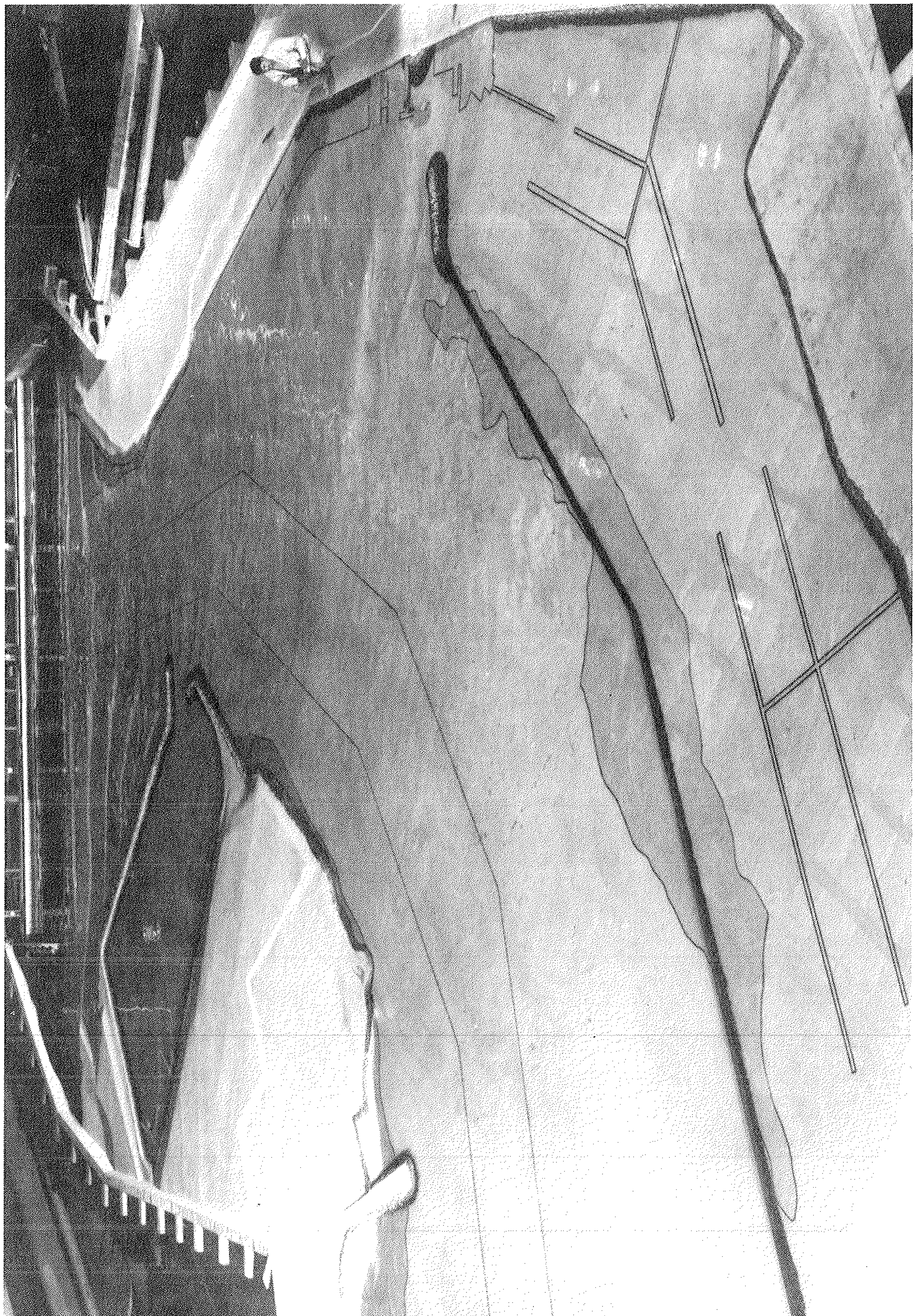


Figure 6. General view of model

An Automated data acquisition and control system, designed and constructed at WES (Figure 7), was used to generate and transmit wave generator control signals, monitor wave generator feedback, and secure and analyze wave data at selected locations in the model. Through the use of a microvax computer, the electrical output of parallel-wire, capacitance-type wave gauges, which varied with the change in water-surface elevation with respect to time, were recorded on magnetic disks. These data were then analyzed to obtain the parametric wave data.

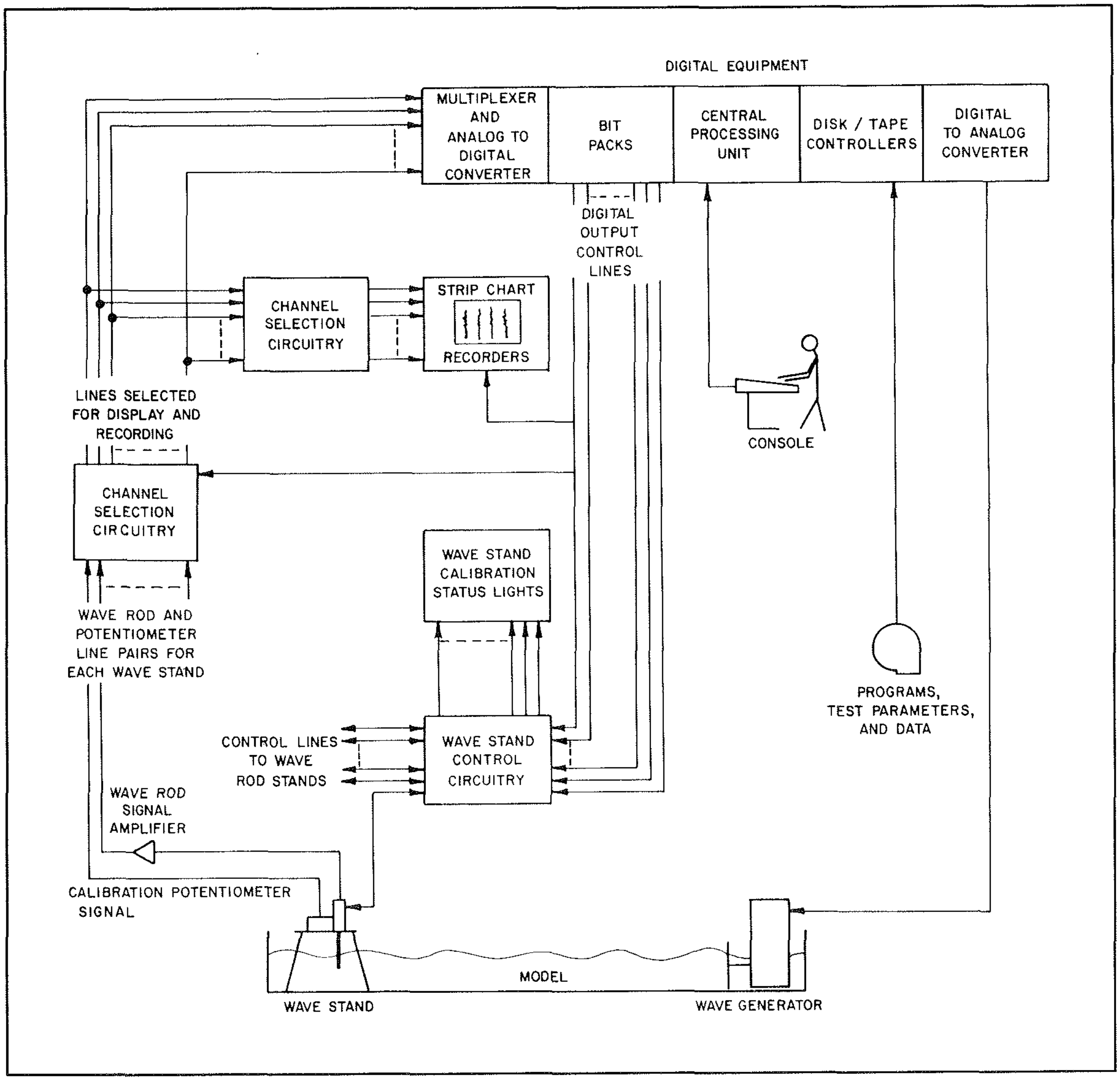


Figure 7. Automated data acquisition and control system

A 0.6-m (2-ft) (horizontal) solid layer of fiber wave absorber was placed at strategic locations along the inside perimeter of the model to dampen 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.

Design of Tracer Material

As discussed previously, a fixed-bed model was constructed and a tracer material designed and prepared to qualitatively determine movement and deposition of sediment in the immediate vicinity of Newport North Marina entrance. Tracer was chosen in accordance with the scaling relations of Noda (1972), which indicate a relation or model law among the four basic scale ratios, i.e., the horizontal scale λ; the vertical scale μ; the sediment size ratio η_D; and the relative specific weight ratio η_γ. These relations were determined experimentally using a wide range of conditions and bottom materials.

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 fixed-bed model of Newport North Marina was undistorted to allow accurate reproduction of short-period wave and current patterns, the following procedure (which has been successfully used and validated for undistorted models) was used to design a tracer material. Using the prototype sand characteristics (median diameter, D_{50} = 0.15 - 0.20 mm, specific gravity = 2.68) from USAED, Portland (1991) 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 were 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} = 0.38 - 0.66 mm) were selected for use as a tracer material throughout the model investigation.

3 Test Conditions and Procedures

Selection of Test Conditions

Still-water level

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

In most cases, it is desirable to select a model swl that closely approximates the higher water stages which normally occur in the prototype for the following reasons:

a. The maximum amount of wave energy reaching a coastal area normally occurs during the higher water phase of the local tidal cycle.

b. Most storms moving onshore are characteristically accompanied by a higher water level due to wind, tide, and storm surge.

c. The selection of a high swl helps minimize model scale effects due to viscous bottom friction.

d. When a high swl is selected, a model investigation tends to yield more conservative results.

Yaquina Bay experiences tides of the mixed semidiurnal type, with two highs and two lows occurring daily. Estuary volume is 55.2 million cu m (72.2 million cu yd) at mean higher high water (mhhw) and 26.3 million cu m (34.4 million cu yd) at mllw (USAED Portland 1994). The surface area of the estuary ranges from 17.1 sq km (6.6 sq miles) at mean high water to 9.1 sq km (3.5 sq miles) at mean low water. Tidal elevations at Newport North Marina typically range from 0 m (0 ft) to +2.4 m (+8.0 ft); however, extremes can range from -0.9 m (-3.0 ft) to +3.5 m (+11.5 ft). In addition, storm surges of 0.6 to 0.9 m (2 to 3 ft) are typical (USAED Portland 1994). Tidal monitoring (Goodwin, Emmett, and Glenne 1970) indicated essentially no time lag of high or low tide and negligible tidal range reduction between the outside of the estuary and a point at the Oregon State University Marine Science Center (located across the river from Newport North Marina).

Swl's of 0.0, +1.5, +2.4, and +3.4 m (0.0, +5.0, +8.0, and +11.0 ft) were selected by NPP for use in testing the Newport model. The 0.0- and +2.4-m (0.0- and +8.0-ft) swl's were representative of mllw and mean higher high water (mhhw), respectively. The +1.5-m (+5.0-ft) swl was representative of the tidal elevation in the river when maximum flood and ebb velocities occur; therefore, tidal flows were superimposed with the +1.5-m (+5.0-ft) swl. The +3.4-m (+11.0-ft) swl represented high tide conditions (mhhw) with a 0.9-m (3.0-ft) storm surge superimposed.

Factors influencing selection of test wave characteristics

In planning the testing program for a model investigation of harbor wave-action problems, it is necessary to select heights, periods, 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 significant 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 distance over water (fetch) which the wind blows. Selection of test wave conditions entails evaluation of such factors as:

a. 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 approach the problem area.

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

c. Alignment, size, and relative geographic position of the navigation structures.

d. Alignments, lengths, and locations of the various reflecting surfaces in the area.

e. Refraction of waves caused by differentials in depth in the area seaward of the site, which may create either a concentration or a diffusion of wave energy.

Wave and storm data

Measured prototype data covering a sufficiently long duration from which to base a comprehensive statistical analysis of wave conditions for the Newport North Marina were not available. Seismometer wave gauge data, however, covering the period 1971 to present were available and utilized for wave height analysis. This instrument was installed at the Oregon State University Marine Science Center on the south bank of the river. Also, during previous studies of the Yaquina North Jetty (Grace and Dubose 1988; Briggs, Grace, and Jensen 1989), statistical wave hindcast estimates over a 20-year period (1956-1975) were obtained at the seaward ends of the jetties. The six most severe storms in this hindcast data set had wave periods of 12.5, 14.3, and 16.7 sec and significant wave heights ranging from 4.6 to 7 m (15 to 23 ft). An additional study of jetty stability (Carver and Briggs 1994) scanned meteorological and buoy records for the worst storms during the 1979-1980 storm season. Wave periods ranging from 10 to 17 sec and significant wave heights ranging from 1.5 to 5.2 m (5 to 17 ft) were identified at the jetty heads.

A study was conducted by NPP to determine wave and storm conditions inside the Yaquina River incident to Newport North Marina. Historical records, observations, and predictions from a numerical model of wave transformation in a channel bounded by rubble-mound breakwaters (Melo and Guza 1991) were used in the conduct of the study. The modified diffraction model reported in Melo and Guza (1991) is based on the linear mild-slope equation and predicts the complex patterns of wave evolution due to dissipation along the jetties and diffraction from the channel interior. The study established wave periods ranging from 12 to 17 sec and significant wave heights ranging from 0.9 to 2.4 m (3 to 8 ft). Data results revealed a 0.9-m (3-ft) wave will be exceeded at least 10 percent of the time during the winter months (October through March). Also, a 1.8-m (6-ft) wave can be expected to occur on an average of at least once a year. These wave periods and heights incident to the marina appear reasonable relative to those predicted at the seaward ends of the North Jetty in the previous studies. Incident wave direction for Newport North Marina is controlled by the orientation of the entrance channel through the Yaquina north and south Jetties.

Selection of test waves

Based on hindcast data and the reconnaissance study performed, NPP selected the test wave characteristics listed in the following tabulation for reproduction in the model investigation. Waves approached the marina from approximately 222 deg (along the longitudinal axis of the river channel). Incident wave characteristics were measured in the model in the river seaward of the marina at the approximate location of the U. S. Highway 101 bridge. Model contours then transformed the wave characteristics as they approached the marina. Based on the hindcast data obtained, all waves were assumed to be swell except for storm wave conditions at the +3.4-m (+11.0-ft) swl.

Selected Test Waves[1]		
Period, sec	**Height, m (ft)**	**swl, m (ft)**
12.5	0.9 (3) 1.8 (6) 2.4 (8)	0.0, +1.5, +2.4 (0, +5, +8) 0.0, +1.5, +2.4 (0, +5, +8) 0.0, +1.5, +2.4, +3.4 (0, +5, +8, +11)
14.3	0.9 (3) 1.8 (6) 2.4 (8)	0.0, +1.5, +2.4 (0, +5, +8) 0.0, +1.5, +2.4 (0, +5, +8) 0.0, +1.5, +2.4, +3.4 (0, +5, +8, +11)
16.7	0.9 (3) 1.8 (6) 2.4 (8)	0.0, +1.5, +2.4 (0, +5, +8) 0.0, +1.5, +2.4 (0, +5, +8) 0.0, +1.5, +2.4, +3.4 (0, +5, +8, +11)

[1] Incident wave conditions generated in the river seaward of the marina and measured at the approximate location of the U.S. Highway 101 bridge.

Unidirectional wave spectra were generated using a depth-limited TMA (Texel-MARSDEN-ARSLOE) spectral form for the selected test waves and throughout the model investigation. Plots of a typical wave spectra are shown in Figure 8. The solid line represents the desired spectra, while the dashed line represents the spectra reproduced in the model at the U.S. Highway 101 bridge location. The second peak in the desired spectra is due to wave breaking along the sides of the river as the waves propagated to the bridge. These were judged to be transitory artifacts in the wave generation area that would not propagate upstream into the marina area. The larger the gamma value, the sharper the peak in the energy distribution curve. Typically, this value varies from 1 to 3.3 for sea (storm) conditions and 7 and higher for swell waves. TMA gamma functions of 2 and 10 were used to determine the spread of the spectra for sea and swell conditions, respectively. A typical wave time series is shown in Figure 9, which depicts water surface elevation η versus time. Selected test waves were defined by significant wave height, the average height of the highest one-third of the waves or H_s. In deep water, H_s is very similar to H_{mo} (energy-based wave) where $H_{mo} = 4\ (E)^{1/2}$, and E equals total energy in the spectra which is obtained by integrating the energy density spectra over the frequency range.

Tidal flows and velocities

Prototype data obtained by Goodwin, Emmett, and Glenne (1969) indicated that maximum flood and ebb tidal velocities were 0.6 mps (1.9 fps) near the Oregon State University Marine Science Center (across the river from Newport North Marina). These values (for both flood and ebb conditions) occurred with a tide level of +1.5 m (+5.0 ft). Flood and ebb tidal discharges were reproduced by the model circulation system and calibrated to simulate steady-state flows with velocities of 0.6 mps (1.9 fps) at this location. These flows were used during model testing with the +1.5-m (+5.0-ft) swl. Velocities were

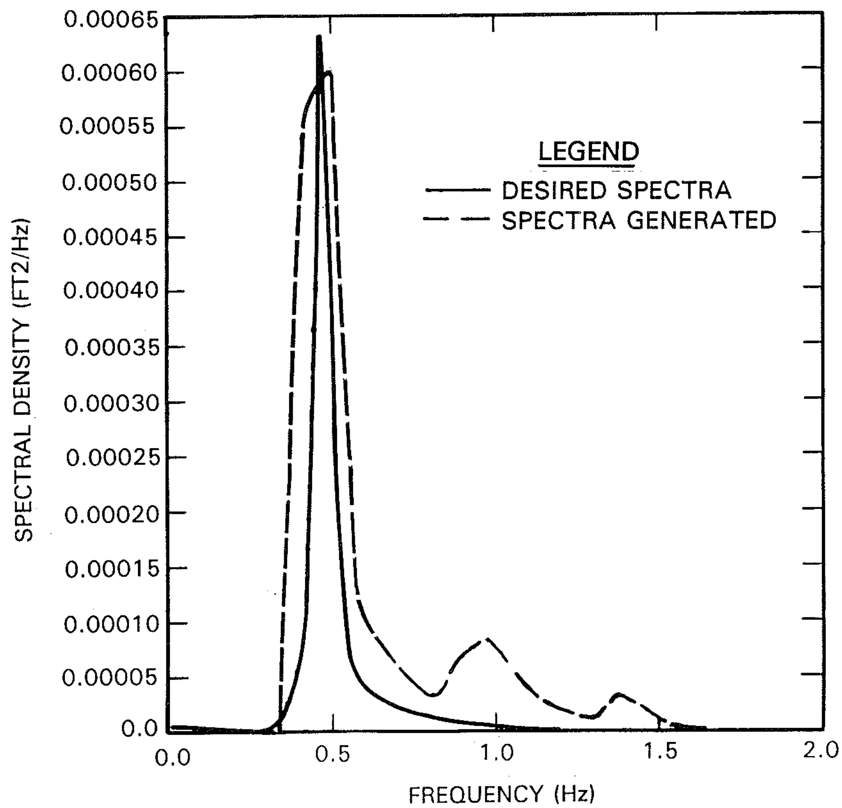


Figure 8. Typical energy density versus frequency plots (model terms) for a wave spectra; 16.7-sec, 3-ft test waves

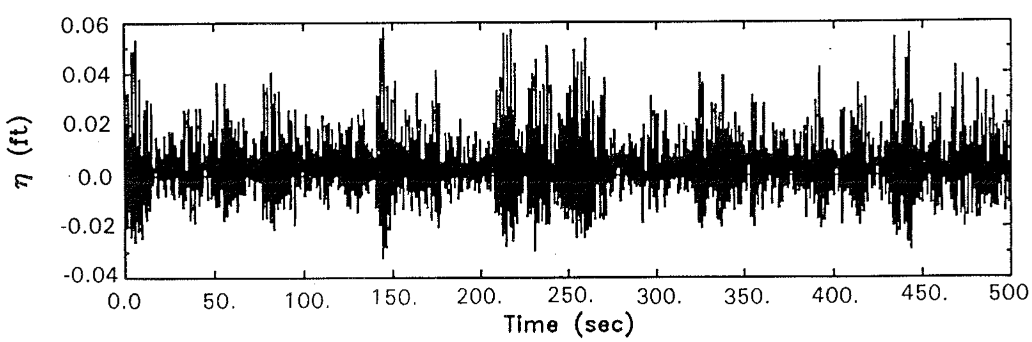


Figure 9. Typical wave train time series, 16.7-sec, 3-ft test waves

measured by timing the progress of a weighted float over a known distance on the model floor.

Analysis of Model Data

Relative merits of the various plans tested were evaluated by:

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

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

c. Comparison of sediment tracer movement and subsequent deposits.

d. Visual observations and wave pattern photographs.

In the wave-height data analysis, the average height of the highest one-third of the waves (H_s), recorded at each gauge location, was computed. All wave heights then were adjusted by application of Keulegan's equation[1] to compensate for excessive model wave height attenuation due to viscous bottom friction. From this equation, reduction of model wave heights (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, and the model data can be corrected and converted to their prototype equivalents.

[1] G. H. Keulegan, 1950, "The Gradual Damping of a Progressive Oscillatory Wave with Distance in a Prismatic Rectangular Channel," unpublished data, National Bureau of Standards, Washington, DC, prepared at request of Director, WES, Vicksburg, MS, by letter of 2 May 1950.

4 Tests and Results

The Tests

Preliminary test series

Initially, wave heights and wave patterns were obtained for existing conditions (Plate 1) and eight test plan variations in the design elements of three basic improvement plan concepts. Basic improvement plans consisted of (a) straight, and (b) angled breakwater extensions, and (c) a detached breakwater protecting the harbor entrance. Brief descriptions of the preliminary test plans are presented in the following subparagraphs; dimensional details are presented in Plates 2-6.

a. Plan 1 (Plate 2) consisted of a 70.1-m-long (230-ft-long) rubble-mound breakwater extension originating at the western end of the existing timber breakwater and extending on the same alignment as the existing structure (273-deg azimuth).

b. Plan 1A (Plate 2) involved the elements of Plan 1 with an 18.3-m-long (60-ft-long) extension of the rubble-mound structure on the same alignment. This resulted in an 88.4-m-long (290-ft-long) breakwater extension.

c. Plan 1B (Plate 3) entailed the elements of Plan 1 with a 30.5-m-long (100-ft-long) extension of the rubble-mound structure in a southwesterly alignment (237-deg azimuth) that paralleled the pier line on the north bank of the river. This resulted in a 100.6-m-long (330-ft-long) breakwater extension.

d. Plan 2 (Plate 4) consisted of a 60.9-m-long (200-ft-long) detached breakwater situated southwesterly of the marina entrance.

e. Plan 2A (Plate 4) included the elements of Plan 2 with a 15.2-m-long (50-ft-long) northwesterly extension of the inner end of the detached breakwater. This resulted in a 76.2-m-long (250-ft-long) structure.

f. Plan 2B (Plate 5) entailed the elements of Plan 2A with a 54.9-m-long (180-ft-long) easterly extension of the outer end of the detached breakwater. This resulted in a 131.1-m-long (430-ft-long) structure.

g. Plan 3 (Plate 6) consisted of a 45.7-m-long (150-ft-long) rubble-mound extension originating at the western end of the existing timber breakwater and extending at an angle in a northwesterly direction (326-deg azimuth).

h. Plan 3A (Plate 6) included the elements of Plan 3 with a 15.2-m-long (50-ft-long) extension of the rubble-mound structure on the same northwesterly alignment. This resulted in a 60.9-m-long (200-ft-long) breakwater extension.

In addition, during the preliminary test series, wave height tests were conducted for approximately 25 expeditiously constructed breakwater plans, which included changing the lengths, locations, and alignments of rubble-mound structures. Also, some plans included short vertical structures (representing timber structures) located along the north bank west of the marina entrance and installed perpendicular to shore. Results of these tests provided insight into expected wave conditions in the marina versus various proposed structure locations, sizes, types, and orientations.

Final test series

After completion of preliminary tests, existing conditions were "modified" and subjected to comprehensive testing. "Modified" existing conditions (Plate 7) included the installation of a revetment along the shoreline in the marina from Port Dock 3 northeasterly for a distance of about 381 m (1,250 ft). This revetment will be constructed in the prototype prior to the completion of any of the breakwater improvement plans currently being studied. Riprap and wooden wave screens also were installed in the model under some of the existing wharves westward of the marina entrance. Tests were conducted for 12 design alternatives of an angled rubble-mound breakwater extension for the final test series. Brief descriptions of the final improvement plans are presented in the following subparagraphs, and dimensional details are shown in Plates 8-10. A typical cross section of the rubble-mound breakwater used for all test plans is shown in Plate 11.

a. Plan 4 (Plate 8) consisted of a 47.2-m-long (155-ft-long) rubble-mound breakwater extension originating at the western end of the existing timber breakwater and extending in a northwesterly alignment (311-deg azimuth). This orientation resulted in a 45.7-m-wide (150-ft-wide) entrance opening.

b. Plan 4A (Plate 8) involved the 47.2-m-long (155-ft-long) rubble-mound breakwater extension of Plan 4 with a 30.5-m-long (100-ft-long) vertical structure installed west of Port Dock 3.

c. Plan 4B (Plate 8) included the 47.2-m-long (155-ft-long) rubble-mound breakwater extension of Plan 4 with a 33.5-m-long (110-ft-long) vertical structure installed along the east side of Port Dock 3.

d. Plan 4C (Plate 8) entailed the 47.2-m-long (155-ft-long) rubble-mound breakwater extension of Plan 4 with a 30.5-m-long (100-ft-long) vertical structure installed west of Port Dock 3 and a 33.5-m-long (110-ft-long) vertical structure installed along the east side of Port Dock 3.

e. Plan 5 (Plate 9) consisted of a 54.9-m-long (180-ft-long) rubble-mound breakwater extension originating at the western end of the existing timber breakwater and extending in a northwesterly alignment (311-deg azimuth). This orientation resulted in a 38.1-m-wide (125-ft-wide) entrance opening.

f. Plan 5A (Plate 9) included the 54.9-m-long (180-ft-long) rubble-mound breakwater extension of Plan 5 with a 30.5-m-long (100-ft-long) vertical structure installed west of Port Dock 3.

g. Plan 5B (Plate 9) involved the 54.9-m-long (180-ft-long) rubble-mound breakwater extension of Plan 5 with a 33.5-m-long (110-ft-long) vertical structure installed along the east side of Port Dock 3.

h. Plan 5C (Plate 9) entailed the 54.9-m-long (180-ft-long) rubble-mound breakwater extension of Plan 5 with a 30.5-m-long (100-ft-long) vertical structure installed west of Port Dock 3 and a 33.5-m-long (110-ft-long) vertical structure installed along the east side of Port Dock 3.

i. Plan 6 (Plate 10) consisted of a 77.7-m-long (255-ft-long) rubble-mound breakwater extension originating at the western end of the existing timber breakwater and extending in a northwesterly alignment (311-deg azimuth) for a distance of 47.2 m (155 ft). The structure then extended southwesterly (237-deg azimuth) and paralleled the pier line on the north bank of the river for a distance of 30.5 m (100 ft). This configuration resulted in a 45.7-m-wide (150-ft-wide) entrance opening.

j. Plan 6A (Plate 10) involved the 77.7-m-long (255-ft-long) rubble-mound breakwater extension of Plan 6 with a 30.5-m-long (100-ft-long) vertical structure installed west of Port Dock 3.

k. Plan 6B (Plate 10) entailed the 77.7-m-long (255-ft-long) rubble-mound breakwater extension of Plan 6 with a 33.5-m-long (110-ft-long) vertical structure installed west of along the east side of Port Dock 3.

l. Plan 6C (Plate 10) included the 77.7-m-long (255-ft-long) rubble-mound breakwater extension of Plan 6 with a 30.5-m-long (100-ft-long) vertical wall installed west of Port Dock 3 and a 33.5-m-long (110-ft-long) vertical structure installed along the east side of Port Dock 3.

Wave height tests and wave patterns

Wave heights and wave patterns were obtained for existing conditions, preliminary plans, "modified" existing conditions, and the final improvement plans for one or more of the test waves listed under "selection of test waves" on page 15. Tests involving certain proposed plans were limited to the most critical swl (i.e., +2.4 or +3.4 m (+8.0 or +11.0 ft)). Existing conditions, "modified" existing conditions, and the selected improvement plan (Plan 5) were tested comprehensively for waves for all swl's. Wave gauge locations are shown in the referenced plates.

Wave-induced current patterns and magnitudes

Wave-induced current patterns and magnitudes were obtained for "modified" existing conditions and the selected improvement plan (Plan 5) for representative test waves with all swl's. These tests were conducted by timing the progress of a dye tracer relative to a known distance on the model surface at selected locations in the vicinity of and throughout the marina.

Sediment tracer tests

Sediment tracer tests were conducted for "modified" existing conditions and the selected improvement plan (Plan 5) using representative test waves for all swl's. Tracer material was introduced into the model along the shoal in the vicinity of the marina west entrance to determine sediment tracer patterns and subsequent deposits.

Test Results

In analyzing test results, the relative merits of various improvement plans were based initially on measured wave heights in the marina mooring areas. Further evaluation was based on wave-induced current patterns and magnitudes and the movement of sediment tracer material and subsequent deposits. Model wave heights (significant wave heights or H_s) were tabulated to show measured values at selected locations. Wave-induced current patterns and magnitudes and sediment tracer patterns and subsequent deposits were shown in photographs. Arrows were superimposed onto these photographs to define direction of movement.

Preliminary test series

Wave heights obtained for existing conditions are presented in Table 1 for test waves for all swl's. For the 0.0-m (0.0-ft) swl, maximum wave heights[1] were 0.49 m (1.6 ft) in the marina mooring area (gauge 8) for 12.5-sec, 2.4-m (8-ft) test waves. With the +1.5-m (+5.0-ft) swl, maximum wave heights were 0.88 and 0.79 m (2.9 and 2.6 ft) in the mooring area (gauge 4) for 14.3-sec, 2.4-m (8-ft) test waves, respectively, for ebb and flood flow conditions. Maximum wave heights were 1.01 m (3.3 ft) in the mooring area (gauge 4) for 12.5-sec, 2.4-m (8-ft) test waves for both the +2.4- and +3.4-m (+8.0- and +11.0-ft) swl's. Typical wave patterns secured for existing conditions are presented in Photo 1.

Results of wave height tests for Plans 1, 1A, and 1B are presented in Table 2 for 2.4 m (8-ft) test waves with the +2.4- and +3.4-m (+8.0- and +11.0-ft) swl's. For the +2.4-m (+8.0-ft) swl, maximum wave heights in the marina mooring areas (gauges 4 and 6 - 10) were 0.46, 0.37, and 0.34 m (1.5, 1.2, and 1.1 ft), respectively, for Plans 1, 1A, and 1B. With the +3.4-m (+11-ft) swl, maximum wave heights were 0.52, 0.43, and 0.40 m (1.7, 1.4, and 1.3 ft) in the mooring areas, respectively, for Plans 1, 1A, and 1B. Typical wave patterns for Plans 1, 1A, and 1B are shown in Photos 2-4.

Wave height test results obtained for Plans 2, 2A, and 2B are presented in Table 3 for 2.4-m (8-ft) test waves for the +2.4-m (+8.0-ft) swl. Only Plan 2 was subjected to tests with the +3.4-m (+11.0-ft) swl, and these results also are presented in Table 3. Maximum wave heights obtained in the marina mooring areas with the +2.4-m (+8.0-ft) swl were 0.79, 0.76, and 0.43 m (2.6, 2.5, and 1.4 ft), respectively, for Plans 2, 2A, and 2B. With the +3.4-m (+11.0-ft) swl, maximum wave heights in the mooring areas were 0.79 m (2.6 ft) for Plan 2. Typical wave patterns obtained for Plans 2, 2A, and 2B are presented in Photos 5-7.

Wave heights obtained for Plans 3 and 3A are presented in Table 4 for 2.4-m (8-ft) test waves with the +2.4-m (+8.0-ft) swl. Maximum wave heights obtained were 0.55 and 0.46 m (1.8 and 1.5 ft) in the marina mooring areas for Plans 3 and 3A, respectively. Typical wave patterns for Plans 3 and 3A are shown in Photos 8 and 9.

At this point in the investigation, preliminary wave height tests were conducted for about 25 expeditiously constructed breakwater plans. Wave height data for 12.5-sec, 2.4-m (8-ft) waves with the +2.4-m (+8.0-ft) swl were recorded to determine relative wave conditions in the marina for various rubble-mound structure locations, lengths, and alignments. Vertical structures (representing solid timber breakwaters) also were installed perpendicular to the shore at various locations around and west of the marina entrance. These data are not reported due to the conceptual and expedited nature of the tests;

[1] Refers to maximum significant wave heights throughout report.

however, this series of tests were very valuable in determining locations and alignments of structures for the final test plan series.

Final test series

Wave heights obtained for "modified" existing conditions are presented in Table 5 for test waves for all swl's. For the 0.0-m (0.0-ft) swl, maximum wave heights were 0.37 m (1.2 ft) in the mooring area (gauge 4) for 12.5- and 14.3-sec, 2.4-m (8-ft) test waves. With the +1.5-m (+5.0-ft) swl, maximum wave heights were 0.88 and 0.76 m (2.9 and 2.5 ft) in the mooring area (gauge 4) for 14.3-sec, 2.4-m (8-ft) test waves, respectively, for ebb and flood flow conditions. Maximum wave heights in the mooring area (gauge 4) were 1.04 m (3.4 ft) for 12.5-sec, 2.4-m (8-ft) test waves with the +2.4-m (+8.0-ft) swl and 0.94 m (3.1 ft) for 12.5-sec, 2.4-m (8-ft) test waves with the +3.4-m (+11.0-ft) swl.

Sheet metal was installed adjacent to the existing timber breakwater with "modified" existing conditions to prevent overtopping of the structure. These tests were conducted to determine what magnitude of wave heights in the marina area can be attributed to wave energy overtopping the existing timber breakwater. Wave height test results for 1.8- and 2.4-m (6- and 8-ft) test waves for the +2.4- and +3.4-m (+8.0- and +11.0-ft) swl's are shown in Table 6. For the +2.4-m (+8.0-ft) swl, maximum wave heights were 0.98 m (3.2 ft) in the marina mooring area (gauge 4); and for the +3.4-m (+11.0-ft) swl, maximum wave heights were 0.91 m (3.0 ft) in the mooring area, both for 12.5-sec, 2.4-m (8-ft) test waves.

Wave-induced current patterns and magnitudes for "modified" existing conditions are shown in Photos 10-18 for representative test waves for all the swl's. In general, current patterns moved through the marina from west to east for all swl's. Currents generally entered through the west entrance and exited through the east entrance; however, there were some areas in the marina in which eddying occurred for some wave and swl conditions. Maximum velocities obtained through the marina were 0.46, 0.61, 0.64, 0.43, and 0.27 mps (1.5, 2.0, 2.1, 1.4, and 0.9 fps), respectively, for the 0.0-, +1.5-(maximum ebb), +1.5-(maximum flood), +2.4-, and +3.4-m (0.0-, +5.0-(maximum ebb), +5.0-(maximum flood), +8.0-, and +11.0-ft) swl's. Representative wave patterns obtained for "modified" existing conditions are also presented in Photos 10-18 for test waves from all swl's.

Typical placement patterns of tracer material prior to each model test are shown in Photo 19. The general movement of tracer material and subsequent deposits in the vicinity of the marina entrance for "modified" existing conditions are shown in Photos 20 - 28. Regardless of the swl, most material placed adjacent to the existing timber breakwater migrated easterly for the larger 2.4-m (8-ft) test waves, while the smaller 0.9-m (3-ft) test waves did not move the sediment significantly. The movement of sediment tracer placed southwesterly of the entrance tended to move toward the south side of the existing breakwater with some moving into the entrance for the 2.4-m (8-ft) test waves. Tracer material tended to penetrate further into the entrance for the 0.0-m (0.0-ft) swl. This material did not move significantly for the smaller 0.9-m (3-ft) test waves.

Results of wave height tests conducted for Plans 4, 4A, 4B, 4C, 5, 5A, 5B, 5C, 6, 6A, 6B, and 6C are presented in Table 7 for 12.5-sec, 2.4-m (8-ft) test waves with the +2.4-m (+8.0-ft) swl. Maximum wave heights obtained in the marina mooring area (gauge 4) were 0.58, 0.49, 0.52, and 0.46 m (1.9, 1.6, 1.7, and 1.5 ft), respectively, for Plans 4, 4A, 4B, and 4C. For Plans 5, 5A, 5B, and 5C, maximum wave heights were 0.49, 0.37, 0.40, and 0.37 m (1.6, 1.2, 1.3, and 1.2 ft), respectively, in the marina mooring area (gauge 4). Maximum wave heights were 0.40, 0.37, 0.40, and 0.30 m (1.3, 1.2, 1.3, and 1.0 ft) in the mooring area (gauge 4) for Plans 6, 6A, 6B, and 6C, respectively. Typical wave patterns for Plans 4, 4A, 4B, 4C, 5, 5A, 5B, 5C, 6, 6A, 6B, and 6C are shown in Photos 29-40.

After a review of the data at this point in the model investigation, NPP requested that additional testing be conducted for Plans 4, 5, and 6. Wave heights obtained for Plan 4 are presented in Table 8 for test waves with the +2.4- and +3.4-m (+8.0- and +11.0-ft) swl's. Maximum wave heights in the mooring area (gauge 4) were 0.58 m (1.9 ft) for 12.5-and 14.3-sec, 2.4-m (8-ft) test waves and 0.52 m (1.7 ft) for 12.5-sec, 2.4-m (8-ft) test waves for the +2.4- and +3.4-m (+8.0- and +11.0-ft) swl's, respectively. Wave height test results obtained for Plan 5 for comprehensive test waves and swl's are presented in Table 9. For the 0.0-m (0.0-ft) swl, maximum wave heights were 0.30 m (1.0 ft) in the mooring area (gauges 9 and 10) for 12.5-sec, 2.4-m (8-ft) test waves. With the +1.5-m (+5.0-ft) swl, maximum wave heights in the mooring area (gauge 4) were 0.43 m (1.4 ft) for ebb flow conditions with 12.5-sec, 2.4-m (8-ft) test waves and 0.34 m (1.1 ft) for flood flow conditions with 14.3-sec, 2.4-m (8-ft) test waves. Maximum wave heights in the mooring area (gauge 4) were 0.49 m (1.6 ft) with the +2.4-m (+8.0-ft) swl, and 0.43 m (1.4 ft) with the +3.4-m (+11.0-ft) swl, both for 12.5-sec, 2.4-m (8-ft) test waves. Wave heights obtained for Plan 6 are presented in Table 10 for test waves with the +2.4- and +3.4-m (+8.0- and +11.0-ft) swl's. Maximum wave heights were 0.40 m (1.3 ft) in the mooring area (gauge 4) for 12.5-sec, 2.4-m (8-ft) test waves for both swl's.

Wave-induced current patterns and magnitudes obtained for Plan 5 are shown in Photos 41-49 for representative test waves for all the swl's. Currents generally moved from west to east through the marina for test waves for all swl's. They entered through the west entrance and flowed out through the east entrance. For some conditions, eddying occurred in some areas in the marina. Maximum velocities obtained in the marina were 0.43, 0.46, 0.58, 0.30, and 0.18 mps (1.4, 1.5, 1.9, 1.0, and 0.6 fps), respectively, for the 0.0-, +1.5-(maximum ebb), +1.5-(maximum flood), +2.4-, and +3.4-m (0.0-, +5.0-(maximum ebb), +5.0-(maximum flood), +8.0-, and +11.0-ft) swl's. Typical wave patterns for Plan 5 also are shown in Photos 41-49.

The general movement of tracer material and subsequent deposits for Plan 5 in the vicinity of the marina entrance are shown in Photos 50-58. For the smaller 0.9-m (3-ft) test waves, tracer material did not move significantly, regardless of the swl tested, and for the larger 2.4-m (8-ft) test waves, material placed adjacent to the existing timber breakwater migrated easterly. Sediment placed southwesterly of the entrance tended to move toward the south side of the existing breakwater with some moving adjacent to the rubble-mound extension for the 2.4-m (8-ft) test waves. Some material moved into the entrance for the 0.0-m (0.0-ft) swl. The smaller 0.9-m (3-ft) test waves resulted in no significant movement of this material.

The rubble-mound breakwater extension was removed, and wave gauges were placed along the center line of the proposed Plan 5 and 6 structures (Plate 12). Wave heights were obtained for 2.4-m (8-ft) test waves for all the swl's to provide design wave information. Results of these tests are presented in Table 11. Maximum wave heights of 1.71 m (5.6 ft) were recorded at gauge 16 for 12.5-sec, 2.4-m (8-ft) incident waves at an swl of +3.4 m (+11.0 ft).

Discussion of Test Results

Preliminary test series

Results of wave height tests for existing conditions revealed rough and turbulent wave conditions in the mooring areas (gauges 4 and 6-10, Plate 1) of the marina. Wave heights in excess of 0.9 m (3 ft) were measured for the +2.4- and +3.4-m (+8.0- and +11.0-ft) swl's. Wave heights ranging from 0.6 to 0.9 m (2 to 3 ft) were common in the marina mooring areas for the +1.5-m (+5.0-ft) swl with both the ebb and flood tidal flows; and wave heights exceeded 0.3 m (1 ft) with the 0.0-m (0.0-ft) swl.

Wave heights obtained for the originally proposed design alternatives (Plans 1-3, Plates 2-6) indicated that none of the test plans met the established 0.3-m (1-ft) wave height criterion in the mooring areas of the marina. A comparison of the straight breakwater extension concept (Plan 1 series) versus the angled breakwater extension concept (Plan 3 series) revealed that the angled structure provided similar wave protection in the mooring areas with less breakwater length. For example, the 70.1-m-long (230-ft-long) extension of Plan 1 resulted in 0.46-m (1.5-ft) wave heights in the mooring area for 2.4-m (8-ft) test waves with the +2.4-m (+8.0-ft) swl versus 0.46-m (1.5-ft) wave heights for the 60.9-m-long (200-ft-long) extension of Plan 3A for the same test conditions. The detached breakwater concept (Plan 2 series) provided the least wave protection to the mooring area relative to structure length versus the breakwater extensions. The 76.2-m-long (250-ft-long) detached breakwater of Plan 2A resulted in 0.76-m (2.5-ft) wave heights in the mooring area for 2.4-m (8-ft) test waves with the +2.4-m (+8.0-ft) swl.

Wave height tests for the expeditiously constructed breakwater alternatives allowed for quick comparisons of wave data for various rubble-mound and solid vertical structure locations, lengths, and alignments. The alignment of the rubble-mound breakwater extension and locations of solid vertical structures installed from the shore within the pier line were selected for the final test series. The rubble-mound extension was aligned parallel to incoming wave crests, thus providing maximum wave protection with minimum structure length. Viable locations of the solid vertical structures adjacent to existing wharves were coordinated with NPP and the Newport North Marina harbor master during this phase of testing.

Final test series

Results of wave height tests for "modified" existing conditions (with the revetment and timber wave screens installed in the marina) revealed rough and turbulent wave conditions in the mooring areas of the marina. Maximum wave heights were in excess of 0.9 m (3 ft) for storm waves with the +2.4- and +3.4-m (+8.0- and +11.0-ft) swl's, similar to the initial tests for existing conditions. It was noted, however, that wave conditions, in general, slightly improved due to the installation of the planned revetment inside the marina. The revetment is scheduled to be constructed in the fall of 1995. The number of instances in which the larger wave heights occurred for the various swl's was reduced (when compared to initial existing conditions), particularly in the eastern portion of the marina.

The tests conducted for "modified" existing conditions, where sheet metal was installed adjacent to the existing timber breakwater to prevent overtopping, revealed wave heights in excess of 0.9 m (3 ft) in the marina mooring areas. With no overtopping, wave heights in the marina increased slightly at some locations and decreased slightly at some locations, depending on the test waves and swl's tested. The maximum change in wave height in the mooring areas was 0.12 m (0.4 ft) at one location for one test condition. Most of the changes, however, were on the order of 0.03 to 0.06 m (0.1 to 0.2 ft). These results indicate that wave overtopping of the existing timber structure is not a significant problem with respect to excessive wave conditions in the marina.

Wave-height tests conducted for the final test series for Plans 4-4C, 5-5C, and 6-6C indicated that only Plan 6C (77.7-m-long (255-ft-long) rubble-mound breakwater extension with a cumulative 67.1-m (220-ft) length of vertical structure installed at Port Dock 3) met the established 0.3-m (1-ft) wave height criterion in the marina mooring areas. The solid vertical walls installed in the vicinity of Port Dock 3 for all the test plans were effective in reducing wave heights in the marina mooring areas; however, NPP was reluctant to include these structures in the final plan until a detailed design analysis could be conducted. An assessment of economic benefits at this point in the investigation allowed NPP to relax the original 0.3-m (1-ft) wave height criterion somewhat. Maximum wave heights were obtained at Port Dock 3 in the model study; however, fewer vessels are moored at this dock than at other areas in the marina. Considering wave protection provided the marina versus estimated construction costs, Plan 5 (54.9-m-long (180-ft-long) rubble-mound extension) was selected by NPP as the most cost-effective plan and was therefore subjected to comprehensive testing.

Wave-induced current patterns and magnitudes obtained for "modified" existing conditions and Plan 5 revealed similar circulation patterns throughout the marina. Currents tended to enter through the west entrance, flow easterly, and exit through the east entrance. Current magnitudes measured in the marina indicated that the Plan 5 breakwater extension will result in slightly decreased velocities as opposed to "modified" existing conditions; however, no stagnant areas were observed. It was noted during testing that tidal ebb and flood flows tended to be concentrated in the deeper river channel south of the marina. These flows had little effect on marina circulation when compared to wave-induced circulation. Based on the test results, construction of the Plan 5 breakwater extension will have minimal impact on current patterns and magnitudes in the marina.

A comparison of sediment tracer patterns and subsequent deposits for "modified" existing conditions and Plan 5 indicated that sediment placed adjacent to the existing timber breakwater migrated easterly for each condition tested. Sediment placed southwesterly of the entrance moved to the south of the existing timber breakwater and into the entrance for "modified" existing conditions. Tests involving sediment tracer movement in the model are valid assuming there is bed-load sediment available to be moved in the area. The existing shoal in the prototype has been relatively stable for years (USAED Portland 1994). Since shoaling of the entrance has not occurred often in the prototype, there is probably minimal loose bed-load sediment in this vicinity to be moved. For Plan 5, material southwesterly of the entrance moved south of the existing timber breakwater and adjacent to the proposed breakwater extension. If material were available for movement in this area, the Plan 5 breakwater extension would probably improve sedimentation conditions since material did not enter the entrance to the degree that it did for "modified" existing conditions.

Wave height data along the wharves and docks west of the marina entrance (gauges 1 and 2) were compared to data for "modified" existing conditions and Plan 5 to determine if they were impacted by the installation of the proposed breakwater extension. Considering all test conditions, maximum wave heights obtained were 0.73 and 0.76 m (2.4 and 2.5 ft) at gauge 1 and 0.61 and 0.58 m (2.0 and 1.9 ft) at gauge 2 for "modified" existing conditions and Plan 5, respectively. Wave heights at these locations increased slightly for some wave conditions and decreased slightly for others with Plan 5 installed. An average of the changes in wave conditions, considering all test conditions, revealed changes of less than 0.03 m (0.1 ft) at these locations; therefore, it was determined that the Plan 5 breakwater extension should have no adverse impacts on wave conditions to the existing wharves/docks west of the marina entrance.

5 Conclusions

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

a. Existing conditions are characterized by rough and turbulent wave conditions during periods of storm wave attack. Wave heights in excess of 0.9 m (3 ft) occurred in the marina mooring areas.

b. Preliminary tests for the three originally proposed design alternatives (Plans 1-3, Plates 2-6) indicated that none of the test plans would meet the original 0.3-m (1-ft) criterion in the marina mooring area.

c. Of the three originally proposed design alternatives, preliminary tests indicated that the angled rubble-mound breakwater extension concept (Plan 3 series) was most effective considering wave protection provided in the mooring area versus structure length. The detached breakwater concept (Plan 2 series) proved to be the least effective.

d. Preliminary testing of the expeditiously constructed breakwater plans proved valuable in the selection of the structure alignments, lengths, and locations used for the final test series.

e. Test results for "modified" existing conditions (revetment and timber wave screen installed) revealed rough and turbulent wave conditions in the marina with wave heights in excess of 0.9 m (3 ft) during storm wave conditions. Generally, however, the revetment slightly improved overall wave conditions in the marina.

f. Tests conducted in the model, in which overtopping of the existing timber breakwater was prevented, revealed that wave overtopping is not a significant problem with respect to excessive wave conditions in Newport North Marina.

g. Results of wave-height tests for the 12 final test plans revealed that only Plan 6C (77.7-m-long (255-ft-long) rubble-mound breakwater extension and cumulative 67.1-m (220-ft) length of vertical structures, Plate 10) met the originally established 0.3-m (1-ft) wave height criterion in the marina mooring areas.

h. After an assessment of economic benefits, Plan 5 (54.9-m-long (180-ft-long) rubble-mound breakwater extension, Plate 9) was selected as the most cost-effective plan considering wave protection provided the marina mooring areas versus construction costs.

i. Construction of the Plan 5 rubble-mound breakwater extension will have minimal impact on circulation patterns and magnitudes in the marina.

j. Construction of the Plan 5 rubble-mound breakwater extension will have no adverse impacts on sedimentation at the marina entrance.

k. Construction of the Plan 5 rubble-mound breakwater extension will have no adverse impacts on wave conditions along the existing docks and wharves west of the existing entrance.

References

Bottin, R. R., Jr., and Chatham, C. E., Jr. (1975). "Design for wave protection, flood control, and prevention of shoaling, Cattaraugus Creek Harbor, New York; hydraulic model investigation," Technical Report H-75-18, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Briggs, M. J., Grace, P. J., and Jensen, R. E. (1989). "Directional spectral wave transformation in the nearshore region; Report 1, Directional spectral performance characteristics," Technical Report CERC-89-14, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Carver, R. D., and Briggs, M. J. (1994). "Stability study of 1978 jetty rehabilitation, Yaquina Bay, Oregon, in response to 1979-1980 storm season waves," Technical Report CERC-94-15, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Giles, M. L., and Chatham, C. E., Jr. (1974). "Remedial plans for prevention of harbor shoaling, Port Orford, Oregon; hydraulic model investigation," Technical Report H-74-4, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Goodwin, C. R., Emmett, E. W., and Glenne, B. (1970). "Tidal study of three Oregon estuaries," Bulletin No. 45, Civil Engineering Department, Oregon State University, Corvallis.

Grace, P. J., and Dubose, W. G. (1988). "Jetty rehabilitation stability study, Yaquina Bay, Oregon," Technical Report CERC-88-14, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Melo, E., and Guza, R. T. (1991). "Wave propagation in jettied entrance channels; I: Models," *Journal of Waterways, Port, Coastal, and Ocean Engineering*, American Society of Civil Engineers 117(5), New York.

Noda, E. K. (1972). "Equilibrium beach profile scale-model relationship," *Journal Waterways, Harbors, and Coastal Engineering Division*, American Society of Civil Engineers 98(WW4), 511-528.

Stevens, J. C., et al. (1942). "Hydraulic Models," *Manuals of Engineering Practice No. 25*, American Society of Civil Engineers, New York.

U.S. Army Engineer District, Portland. (1991). "Characterization of sediments at Yaquina Bay and Harbor; Final report," Portland, OR.

__________. (1994). "Newport North Marina Section 107 study, Newport, Oregon," Portland, OR.

Table 1
Wave Heights for Existing Conditions

Test Wave		Wave Heights, ft, at Indicated Gauge Location												
Period sec	Height ft	Gauge 1	Gauge 2	Gauge 3	Gauge 4	Gauge 5	Gauge 6	Gauge 7	Gauge 8	Gauge 9	Gauge 10	Gauge 11	Gauge 12	Gauge 13
swl = 0.0 ft														
12.5	3	0.3	0.5	0.4	0.4	0.3	0.3	0.2	0.3	0.1	0.1	1.0	0.1	0.1
	6	0.8	1.2	1.4	0.9	0.8	0.6	0.6	0.7	0.4	0.6	3.0	0.6	0.3
	8	1.2	2.9	1.9	1.3	1.4	0.9	1.3	1.6	0.9	1.1	4.4	1.0	0.4
14.3	3	0.2	0.4	0.4	0.4	0.2	0.2	0.1	0.3	0.1	0.1	0.8	0.1	0.1
	6	0.7	1.1	1.2	0.8	0.8	0.6	0.5	0.6	0.3	0.3	2.3	0.4	0.1
	8	0.9	1.7	1.7	1.2	1.1	0.8	0.9	1.0	0.6	0.7	3.0	0.6	0.2
16.7	3	0.3	0.5	0.6	0.5	0.4	0.3	0.2	0.3	0.1	0.1	0.9	0.1	0.1
	6	0.7	1.1	1.3	0.9	0.8	0.6	0.4	0.6	0.3	0.4	2.2	0.5	0.1
	8	0.8	1.2	1.6	1.1	1.1	0.6	0.4	0.6	0.2	0.1	3.4	0.6	0.1
swl = +5.0 ft (maximum ebb)														
12.5	3	0.4	0.7	0.6	0.5	0.7	0.6	0.5	0.4	0.3	0.3	1.5	0.8	0.5
	6	1.1	1.8	2.4	2.2	2.0	1.8	1.5	1.4	0.9	0.8	3.8	1.5	1.1
	8	1.3	3.2	2.8	2.6	2.3	2.2	2.0	1.9	1.2	1.0	4.3	1.5	1.2
14.3	3	0.5	0.7	0.6	0.7	0.7	0.6	0.5	0.5	0.4	0.3	1.2	0.6	0.4
	6	1.1	2.5	2.2	2.4	1.9	1.9	1.7	1.5	1.0	0.8	3.4	1.4	1.0
	8	1.4	5.0	2.8	2.9	2.8	2.3	2.5	2.1	1.3	1.1	4.6	1.7	1.2

(Sheet 1 of 3)

Table 1 (Continued)

Test Wave		Wave Heights, ft, at Indicated Gauge Location												
Period sec	Height ft	Gauge 1	Gauge 2	Gauge 3	Gauge 4	Gauge 5	Gauge 6	Gauge 7	Gauge 8	Gauge 9	Gauge 10	Gauge 11	Gauge 12	Gauge 13
swl = +5.0 ft (maximum ebb) (Concluded)														
16.7	3	0.7	0.8	0.8	0.8	0.8	0.8	0.6	0.5	0.5	0.4	1.4	0.6	0.5
	6	1.7	3.2	2.1	1.9	2.0	1.8	1.8	1.5	1.0	0.9	3.4	1.3	1.0
	8	1.8	5.4	2.5	2.5	2.5	2.2	2.4	2.2	1.4	1.2	4.4	1.6	1.2
swl = +5.0 ft (maximum flood)														
12.5	3	0.8	0.6	0.8	0.5	0.4	0.4	0.3	0.3	0.2	0.2	0.9	0.3	0.1
	6	1.4	1.3	2.1	1.4	1.3	1.3	1.1	0.8	0.7	0.5	1.8	0.6	0.3
	8	1.6	1.5	2.4	2.0	2.0	1.8	1.4	1.0	0.8	0.7	2.4	0.8	0.4
14.3	3	0.8	0.7	0.9	0.5	0.5	0.5	0.4	0.4	0.4	0.3	0.7	0.2	0.1
	6	1.4	1.1	2.3	1.6	1.4	1.5	1.2	1.1	0.9	0.7	1.6	0.7	0.3
	8	1.9	2.1	3.1	2.6	2.1	2.2	1.8	1.4	1.1	0.9	2.6	1.1	0.5
16.7	3	0.7	0.6	0.8	0.5	0.5	0.5	0.4	0.4	0.3	0.2	0.7	0.2	0.1
	6	1.5	1.2	1.6	1.1	1.2	1.1	0.9	0.7	0.6	0.6	1.8	0.7	0.3
	8	1.7	1.4	2.1	1.7	1.4	1.6	1.3	0.9	0.8	0.7	2.3	0.9	0.4

(Sheet 2 of 3)

Table 1 (Concluded)														
Test Wave		**Wave Heights, ft, at Indicated Gauge Location**												
Period sec	**Height ft**	**Gauge 1**	**Gauge 2**	**Gauge 3**	**Gauge 4**	**Gauge 5**	**Gauge 6**	**Gauge 7**	**Gauge 8**	**Gauge 9**	**Gauge 10**	**Gauge 11**	**Gauge 12**	**Gauge 13**
swl = +8.0 ft														
12.5	3	0.4	0.5	0.7	0.7	0.8	0.6	0.5	0.5	0.4	0.6	0.9	0.4	0.3
	6	1.0	1.1	2.1	2.3	2.1	1.7	1.7	1.3	1.1	1.3	2.3	1.0	0.7
	8	1.4	1.5	3.2	3.3	2.7	2.5	2.3	1.8	1.5	1.6	3.5	1.5	1.0
14.3	3	0.5	0.6	0.7	0.8	0.8	0.8	0.6	0.6	0.5	0.5	1.0	0.4	0.2
	6	1.0	1.2	1.7	1.8	1.7	1.8	1.4	1.3	1.0	1.1	2.0	0.9	0.6
	8	1.4	1.9	2.7	2.6	2.3	2.5	2.0	1.8	1.4	1.4	2.8	1.3	0.9
16.7	3	0.6	0.8	0.8	0.7	0.7	0.8	0.6	0.6	0.4	0.5	1.0	0.4	0.3
	6	1.3	1.5	1.7	1.7	1.5	1.7	1.4	1.1	0.9	1.1	2.5	1.1	0.8
	8	1.6	2.2	2.4	2.4	2.1	2.4	2.0	1.6	1.3	1.4	3.6	1.5	1.0
swl = +11.0 ft														
12.5	8	1.5	1.6	2.9	3.3	3.2	3.0	2.2	2.2	1.6	1.7	3.4	1.2	1.2
14.3	8	1.5	1.9	2.7	2.9	3.1	2.9	2.3	2.2	1.5	1.7	3.4	1.2	1.1
16.7	8.	1.4	1.6	2.4	2.4	2.4	2.4	2.1	1.9	1.4	1.6	3.7	1.1	1.0

(Sheet 3 of 3)

Table 2
Wave Heights for Plans 1-1B

Test Wave		Wave Height, ft, at Indicated Gauge Location												
Period sec	Height ft	Gauge 1	Gauge 2	Gauge 3	Gauge 4	Gauge 5	Gauge 6	Gauge 7	Gauge 8	Gauge 9	Gauge 10	Gauge 11	Gauge 12	Gauge 13
Plan 1														
swl = +8.0 ft														
12.5	8	1.5	1.8	2.8	1.5	1.6	1.3	1.3	1.1	0.8	1.1	3.8	1.6	1.1
14.3	8.	1.5	2.0	2.4	1.3	1.3	1.2	1.0	1.1	0.7	0.9	2.8	1.2	0.9
16.7	8	1.5	2.1	2.1	1.3	1.2	1.1	1.1	1.0	0.7	0.8	3.4	1.4	1.0
swl = +11.0 ft														
12.5	8	1.7	1.6	2.7	1.6	1.7	1.7	1.2	1.2	1.0	1.1	3.7	1.4	1.2
14.3	8	1.5	1.9	2.4	1.4	1.6	1.5	1.2	1.3	0.9	1.0	3.6	1.3	1.2
16.7	8	1.4	1.5	2.1	1.4	1.3	1.2	1.1	1.1	0.9	1.0	3.8	1.3	1.1
Plan 1A														
swl = +8.0 ft														
12.5	8	1.4	1.6	2.0	1.1	1.3	1.0	1.0	1.0	0.7	0.8	3.6	1.6	1.1
14.3	8	1.7	2.3	2.1	1.2	1.3	1.0	1.0	1.1	0.7	0.8	2.9	1.4	1.1
16.7	8	1.5	1.8	1.6	1.0	1.0	0.8	0.9	0.9	0.6	0.8	3.5	1.4	1.0

(Continued)

Table 2 (Concluded)

Test Wave		Wave Height, ft, at Indicated Gauge Location												
Period sec	Height ft	Gauge 1	Gauge 2	Gauge 3	Gauge 4	Gauge 5	Gauge 6	Gauge 7	Gauge 8	Gauge 9	Gauge 10	Gauge 11	Gauge 12	Gauge 13
swl = +11.0 ft														
12.5	8	1.6	1.5	2.1	1.4	1.6	1.4	1.0	1.2	0.9	1.0	3.7	1.4	1.3
14.3	8	1.3	1.3	1.7	1.1	1.1	1.0	0.9	0.9	0.7	0.8	2.7	1.0	0.8
16.7	8	1.4	1.5	1.7	1.3	1.2	1.1	1.1	1.1	0.8	1.0	3.7	1.3	1.1
Plan 1B														
swl = +8.0 ft														
12.5	8	1.5	1.7	2.0	1.1	1.3	1.0	1.0	1.0	0.7	0.8	3.5	1.5	1.0
14.3	8.	1.6	2.0	1.9	1.1	1.2	0.9	0.9	1.0	0.7	0.7	3.0	1.2	1.0
16.7	8	1.5	2.0	1.6	1.1	1.0	0.9	1.0	0.9	0.6	0.8	3.4	1.3	1.0
swl =+11.0 ft														
12.5	8	1.6	1.5	2.0	1.3	1.4	1.2	0.9	1.1	0.8	0.9	3.7	1.4	1.2
14.3	8	1.6	1.8	2.0	1.2	1.4	1.3	1.1	1.2	0.8	0.9	3.4	1.3	1.2
16.7	8	1.4	1.6	1.7	1.2	1.2	1.1	1.1	1.1	0.9	1.0	3.7	1.2	1.1

Table 3
Wave Height for Plans 2-2B

Test Wave		Wave Height, ft, at Indicated Gauge Location												
Period sec	Height ft	Gauge 1	Gauge 2	Gauge 3	Gauge 4	Gauge 5	Gauge 6	Gauge 7	Gauge 8	Gauge 9	Gauge 10	Gauge 11	Gauge 12	Gauge 13
Plan 2														
swl = +8.0 ft														
12.5	8	1.5	1.5	2.0	2.6	1.6	2.0	1.7	1.5	1.2	1.4	3.8	1.5	0.9
14.3	8	1.7	1.8	1.8	2.3	1.5	2.1	1.6	1.5	1.2	1.3	2.9	1.3	0.9
16.7	8	1.5	1.8	1.9	1.7	1.4	1.8	1.5	1.3	1.0	1.1	3.4	1.4	0.9
swl = +11.0 ft														
12.5	8	1.5	1.3	1.8	2.6	2.0	2.3	1.6	1.6	1.2	1.4	3.7	1.3	1.1
14.3	8	1.7	1.6	1.9	2.3	2.0	2.3	1.8	1.7	1.2	1.4	3.6	1.4	1.0
16.7	8	1.4	1.5	1.8	1.9	1.6	1.7	1.6	1.5	1.1	1.4	3.8	1.2	0.9
Plan 2A														
swl = +8.0 ft														
12.5	8	1.4	1.3	1.8	2.5	1.5	1.9	1.6	1.3	1.1	1.3	3.6	1.4	0.8
14.3	8	1.6	1.4	1.8	2.2	1.2	2.0	1.4	1.3	1.1	1.2	3.0	1.2	0.7
16.7	8	1.5	1.5	1.7	1.6	1.1	1.6	1.3	1.1	0.8	1.0	3.5	1.3	0.7
Plan 2B														
swl = +8.0 ft														
12.5	8	1.5	1.4	1.2	1.3	0.9	1.1	0.8	0.9	0.6	0.8	4.0	1.6	0.8
14.3	8	1.7	1.5	1.0	1.4	0.9	1.1	0.7	0.8	0.6	0.7	3.0	1.4	0.8
16.7	8	1.7	1.6	1.1	1.1	0.9	1.0	0.8	0.8	0.6	0.8	3.5	1.4	0.8

Table 4
Wave Heights for Plans 3-3A

Test Wave		Wave Height, ft, at Indicated Gauge Location												
Period sec	Height ft	Gauge 1	Gauge 2	Gauge 3	Gauge 4	Gauge 5	Gauge 6	Gauge 7	Gauge 8	Gauge 9	Gauge 10	Gauge 11	Gauge 12	Gauge 13
Plan 3														
swl = +8.0 ft														
12.5	8	1.3	1.5	3.5	1.8	1.3	1.6	1.3	1.0	0.8	1.1	3.8	1.4	0.8
14.3	8	1.5	1.6	2.8	1.5	1.0	1.4	1.0	0.9	0.7	0.9	3.0	1.3	0.8
16.7	8	1.5	1.8	2.3	1.4	1.0	1.2	1.1	0.9	0.7	0.8	3.5	1.4	0.9
Plan 3A														
swl = +8.0 ft														
12.5	8	1.5	1.3	3.4	1.5	1.2	1.3	1.1	0.8	0.7	0.9	3.8	1.4	0.9
14.3	8	1.7	1.5	2.5	1.3	0.9	1.0	0.8	0.8	0.5	0.8	2.7	1.3	1.0
16.7	8	1.6	1.5	2.5	1.2	0.9	1.0	0.9	0.8	0.6	0.7	3.6	1.4	0.9

Table 5
Wave Heights for "Modified" Existing Conditions

Test Wave		Wave Height, ft, at Indicated Gauge Location												
Period sec	Height ft	Gauge 1	Gauge 2	Gauge 3	Gauge 4	Gauge 5	Gauge 6	Gauge 7	Gauge 8	Gauge 9	Gauge 10	Gauge 11	Gauge 12	Gauge 13
swl = 0.0 ft														
12.5	3	0.4	0.2	0.4	0.3	0.1	0.1	0.1	0.1	0.1	0.1	1.0	0.1	0.1
	6	0.9	1.0	1.1	0.8	0.5	0.5	0.4	0.6	0.7	0.8	3.0	0.6	0.1
	8	1.5	1.6	1.6	1.2	1.0	0.9	0.7	0.8	1.0	1.1	4.4	1.0	0.3
14.3	3	0.3	0.2	0.5	0.3	0.2	0.2	0.1	0.1	0.1	0.1	0.8	0.1	0.1
	6	0.9	0.8	1.1	0.8	0.6	0.5	0.4	0.3	0.2	0.4	2.4	0.4	0.1
	8	1.1	1.2	1.4	1.2	0.8	0.6	0.6	0.4	0.7	0.6	3.3	0.7	0.2
16.7	3	0.4	0.3	0.6	0.5	0.3	0.3	0.1	0.1	0.1	0.1	0.9	0.2	0.1
	6	1.0	0.9	1.1	0.9	0.5	0.5	0.3	0.2	0.3	0.4	2.0	0.4	0.1
	8	1.3	1.3	1.4	1.0	0.7	0.6	0.4	0.3	0.4	0.5	2.8	0.6	0.2
swl = +5.0 ft (maximum ebb)														
12.5	3	0.3	0.3	0.6	0.4	0.4	0.2	0.2	0.1	0.1	0.2	1.2	0.7	0.2
	6	1.1	1.4	2.5	2.1	1.6	1.2	0.8	0.5	0.4	0.6	3.8	1.1	0.5
	8	1.2	1.7	2.9	2.6	1.7	1.4	1.0	0.7	0.6	0.8	4.8	1.2	0.7

(Sheet 1 of 4)

Table 5 (Continued)

Test Wave		Wave Height, ft, at Indicated Gauge Location												
Period sec	Height ft	Gauge 1	Gauge 2	Gauge 3	Gauge 4	Gauge 5	Gauge 6	Gauge 7	Gauge 8	Gauge 9	Gauge 10	Gauge 11	Gauge 12	Gauge 13
swl = +5.0 ft (Maximum ebb) (Concluded)														
14.3	3	0.3	0.3	0.7	0.7	0.5	0.4	0.4	0.1	0.1	0.3	1.0	0.6	0.2
	6	1.1	1.2	2.6	2.2	1.4	1.3	1.0	0.6	0.5	0.8	3.2	1.1	0.5
	8	1.3	1.5	3.2	2.9	1.7	1.5	1.3	0.8	0.7	1.1	4.6	1.4	0.7
16.7	3	0.6	0.4	0.8	0.6	0.6	0.6	0.5	0.3	0.3	0.5	1.2	0.6	0.3
	6	1.5	1.1	2.4	1.9	1.3	1.2	0.9	0.6	0.5	0.8	3.2	1.1	0.6
	8	1.6	1.4	2.7	2.4	1.4	1.3	1.1	0.7	0.6	0.9	4.1	1.3	0.7
swl = +5.0 ft (maximum flood)														
12.5	3	0.6	0.7	0.6	0.5	0.3	0.3	0.2	0.2	0.1	0.1	0.5	0.2	0.1
	6	1.4	1.5	2.2	1.5	1.2	0.9	0.7	0.6	0.4	0.5	1.6	0.6	0.2
	8	1.7	1.6	2.7	2.3	1.7	1.3	1.0	0.6	0.5	0.6	2.4	0.8	0.4
14.3	3	0.7	0.7	1.0	0.5	0.5	0.4	0.3	0.3	0.1	0.3	0.5	0.2	0.1
	6	1.4	1.5	2.4	1.7	1.2	1.1	0.8	0.7	0.4	0.6	1.4	0.6	0.2
	8	2.4	2.0	3.0	2.5	1.6	1.4	1.1	0.7	0.6	0.8	2.6	1.0	0.4

(Sheet 2 of 4)

Table 5 (Continued)

Test Wave		Wave Height, ft, at Indicated Gauge Location												
Period sec	Height ft	Gauge 1	Gauge 2	Gauge 3	Gauge 4	Gauge 5	Gauge 6	Gauge 7	Gauge 8	Gauge 9	Gauge 10	Gauge 11	Gauge 12	Gauge 13
swl = +0.50 ft (Maximum flood) (Concluded)														
16.7	3	0.6	0.5	1.0	0.4	0.5	0.3	0.3	0.2	0.1	0.3	0.4	0.2	0.1
	6	1.4	1.1	2.0	1.1	1.1	0.9	0.7	0.6	0.5	0.8	1.5	0.5	0.2
	8	1.7	1.5	2.4	1.7	1.2	1.2	0.9	0.7	0.6	0.9	2.0	0.8	0.3
swl = +8.0 ft														
12.5	3	0.3	0.4	1.0	0.7	0.7	0.6	0.3	0.2	0.2	0.2	1.0	0.2	0.2
	6	1.2	0.9	2.6	2.2	1.8	1.5	1.1	1.0	0.9	0.8	2.3	1.0	0.7
	8	1.5	1.5	3.9	3.4	2.5	2.2	1.6	1.4	1.3	1.3	3.7	1.4	0.9
14.3	3	0.4	0.3	1.2	0.8	0.7	0.7	0.5	0.6	0.3	0.7	1.1	0.3	0.2
	6	1.1	0.8	2.5	1.9	1.6	1.3	1.0	1.1	0.8	1.0	2.1	0.8	0.5
	8	1.6	1.3	3.4	3.1	2.0	1.8	1.5	1.3	1.1	1.3	2.9	1.3	0.8
16.7	3	0.4	0.3	1.0	0.5	0.4	0.6	0.4	0.3	0.1	0.4	0.9	0.2	0.2
	6	1.0	0.9	2.0	1.3	1.2	1.2	0.9	0.8	0.6	1.0	2.3	0.9	0.6
	8	1.4	1.2	2.9	2.2	1.7	1.7	1.3	1.0	0.9	1.2	3.3	1.2	0.8

(Sheet 3 of 4)

Table 5 (Concluded)

Test Wave		Wave Height, ft, at Indicated Gauge Location												
Period sec	Height ft	Gauge 1	Gauge 2	Gauge 3	Gauge 4	Gauge 5	Gauge 6	Gauge 7	Gauge 8	Gauge 9	Gauge 10	Gauge 11	Gauge 12	Gauge 13
swl = +11.0 ft														
12.5	8	1.6	1.5	4.2	3.1	2.9	2.4	1.7	1.7	1.3	1.6	3.6	1.2	1.0
14.3	8.	1.4	1.5	3.7	2.6	2.7	2.3	1.7	1.6	1.2	1.9	3.4	1.1	1.0
16.7	8	1.4	1.5	3.2	2.1	2.2	1.8	1.4	1.4	1.1	1.8	3.5	0.9	0.9

(Sheet 4 of 4)

Table 6
Wave Heights for "Modified" Existing Conditions With No Over-Topping of Existing Structure

Test Wave		Wave Height, ft, at Indicated Gague Location												
Period sec	Height ft	Gauge 1	Gauge 2	Gauge 3	Gauge 4	Gauge 5	Gauge 6	Gauge 7	Gauge 8	Gauge 9	Gauge 10	Gauge 11	Gauge 12	Gauge 13
swl = +8.0 ft														
12.5	6	1.1	1.0	2.7	2.3	1.9	1.5	1.2	1.0	1.0	0.9	2.2	1.1	0.6
	8	1.6	1.4	3.6	3.2	2.3	1.9	1.5	1.3	1.3	1.2	3.5	1.4	0.8
14.3	6	1.0	0.9	2.6	2.0	1.6	1.3	1.2	1.1	0.9	1.1	2.1	0.9	0.5
	8	1.6	1.3	3.7	3.0	2.0	1.7	1.5	1.3	1.1	1.4	2.8	1.2	0.7
16.7	6	1.1	1.0	2.0	1.4	1.2	1.2	1.1	0.8	0.7	1.1	2.2	0.9	0.6
	8	1.4	1.4	2.9	2.3	1.7	1.7	1.4	1.1	1.0	1.3	3.3	1.2	0.8
swl = +11.0 ft														
12.5	8	1.6	1.6	4.0	3.0	2.4	2.0	1.6	1.5	1.3	1.5	3.7	1.2	0.9
14.3	8	1.4	1.6	3.7	2.7	2.4	2.1	1.7	1.6	1.2	1.8	3.4	1.1	1.0
16.7	8	1.4	1.5	3.1	2.2	2.0	1.7	1.4	1.4	1.0	1.7	3.7	1.0	0.8

Table 7
Wave Heights for Plans 4-6C for 12.5-sec, 8-ft Test Waves: swl = +8.0 ft

	Wave Height, ft, at Indicated Gauge Location												
Plan	Gauge 1	Gauge 2	Gauge 3	Gauge 4	Gauge 5	Gauge 6	Gauge 7	Gauge 8	Gauge 9	Gauge 10	Gauge 11	Gauge 12	Gauge 13
4	1.6	1.6	3.2	1.9	1.3	1.5	1.1	1.0	0.8	1.0	3.8	1.6	0.8
4A	1.8	1.7	3.1	1.6	1.2	1.3	0.9	0.8	0.7	0.9	3.7	1.6	1.0
4B	1.7	1.7	3.3	1.7	1.2	1.3	0.9	0.9	0.8	0.9	3.6	1.7	0.9
4C	1.7	1.6	3.0	1.5	1.2	1.1	0.9	0.8	0.7	0.9	3.6	1.7	0.9
5	1.7	1.7	3.1	1.6	1.2	1.4	1.0	0.9	0.8	0.9	3.7	1.7	0.9
5A	1.6	1.6	3.0	1.2	1.0	1.1	0.8	0.7	0.6	0.7	3.8	1.4	0.8
5B	1.6	1.5	2.9	1.3	1.0	1.0	0.7	0.7	0.5	0.8	3.8	1.5	0.9
5C	1.8	1.6	3.1	1.2	1.2	1.0	0.8	0.7	0.7	0.8	3.8	1.7	1.0
6	1.5	1.4	2.2	1.3	1.0	1.1	0.8	0.8	0.7	0.9	3.6	1.5	0.9
6A	1.7	1.6	2.5	1.2	1.0	1.1	0.8	0.8	0.7	0.8	3.7	1.6	0.9
6B	1.5	1.5	2.4	1.3	1.0	1.0	0.7	0.7	0.6	0.8	3.7	1.4	0.9
6C	1.6	1.5	2.4	1.0	1.1	1.0	0.7	0.8	0.7	0.8	3.9	1.7	0.9

Table 8
Wave Heights for Plan 4

Test Wave		Wave Height, ft, at Indicated Gauge Location												
Period sec	Height ft	Gauge 1	Gauge 2	Gauge 3	Gauge 4	Gauge 5	Gauge 6	Gauge 7	Gauge 8	Gauge 9	Gauge 10	Gauge 11	Gauge 12	Gauge 13
swl = +8.0 ft														
12.5	3	0.4	0.5	0.5	0.3	0.2	0.2	0.2	0.2	0.1	0.1	0.9	0.2	0.2
	6	1.1	1.1	1.7	1.1	0.7	0.9	0.7	0.7	0.5	0.6	2.3	0.8	0.5
	8	1.6	1.6	3.2	1.9	1.3	1.5	1.1	1.0	0.8	1.0	3.8	1.6	0.8
14.3	3	0.5	0.4	0.7	0.3	0.2	0.3	0.3	0.2	0.2	0.3	1.0	0.4	0.2
	6	1.1	0.9	1.8	1.1	0.6	0.7	0.7	0.6	0.4	0.7	2.1	1.0	0.6
	8	1.5	1.5	2.8	1.9	1.0	1.1	1.0	0.9	0.7	0.9	3.3	1.4	0.8
16.7	3	0.4	0.4	0.5	0.3	0.1	0.2	0.2	0.2	0.1	0.2	0.9	0.2	0.2
	6	1.1	1.0	1.3	0.9	0.5	0.6	0.6	0.5	0.4	0.6	2.4	0.9	0.6
	8	1.5	1.4	2.0	1.4	0.8	0.9	0.8	0.7	0.6	0.8	3.6	1.2	0.8
swl = +11.0 ft														
12.5	8	1.7	1.7	2.9	1.7	1.4	1.3	1.0	1.0	0.9	1.0	3.6	1.6	0.9
14.3	8	1.5	1.7	2.6	1.6	1.5	1.3	1.0	0.9	0.8	1.0	3.7	1.5	0.9
16.7	8	1.4	1.5	2.0	1.2	1.1	0.9	0.7	0.8	0.7	0.9	3.6	1.2	0.8

Table 9
Wave Heights for Plan 5

Test Wave		Wave Height, ft, at Indicated Gauge Location												
Period sec	Height ft	Gauge 1	Gauge 2	Gauge 3	Gauge 4	Gauge 5	Gauge 6	Gauge 7	Gauge 8	Gauge 9	Gauge 10	Gauge 11	Gauge 12	Gauge 13
swl = 0.0 ft														
12.5	3	0.3	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.9	0.1	0.1
	6	0.9	1.0	1.0	0.5	0.3	0.3	0.3	0.4	0.5	0.6	2.8	0.7	0.2
	8	1.5	1.6	1.6	0.9	0.6	0.7	0.7	0.6	1.0	1.0	4.6	1.0	0.3
14.3	3	0.2	0.1	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.7	0.1	0.1
	6	0.9	0.7	0.9	0.4	0.2	0.2	0.2	0.3	0.3	0.3	2.1	0.4	0.1
	8	1.1	1.2	1.2	0.6	0.4	0.4	0.4	0.4	0.4	0.5	3.2	0.7	0.2
16.7	3	0.3	0.2	0.3	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.8	0.1	0.1
	6	1.1	1.0	1.0	0.4	0.2	0.2	0.2	0.2	0.2	0.3	2.3	0.5	0.1
	8	1.4	1.4	1.5	0.6	0.4	0.5	0.4	0.3	0.4	0.4	2.9	0.7	0.2
swl = +5.0 ft (maximum ebb)														
12.5	3	0.5	0.3	0.3	0.1	0.1	0.1	0.1	0.1	0.1	0.1	1.4	0.6	0.3
	6	1.2	1.4	1.8	1.0	0.7	0.6	0.4	0.3	0.3	0.4	3.6	1.3	0.7
	8	1.4	1.8	2.5	1.4	0.9	1.0	0.7	0.5	0.6	0.8	4.6	1.4	0.9
14.3	3	0.5	0.2	0.4	0.1	0.1	0.2	0.1	0.1	0.1	0.2	1.1	0.5	0.3
	6	1.3	1.0	2.0	0.8	0.7	0.6	0.4	0.3	0.3	0.5	3.4	1.3	0.7
	8	1.6	1.7	2.5	1.3	0.8	0.8	0.5	0.3	0.5	0.6	4.6	1.4	0.9

(Sheet 1 of 3)

Table 9 (Continued)

Test Wave		Wave Height, ft, at Indicated Gauge Location												
Period sec	Height ft	Gauge 1	Gauge 2	Gauge 3	Gauge 4	Gauge 5	Gauge 6	Gauge 7	Gauge 8	Gauge 9	Gauge 10	Gauge 11	Gauge 12	Gauge 13
swl = +5.0 ft (Maximum ebb)														
16.7	3	0.8	0.3	0.5	0.2	0.1	0.2	0.1	0.1	0.1	0.2	1.3	0.5	0.3
	6	1.7	1.2	1.9	0.9	0.6	0.6	0.4	0.3	0.3	0.5	3.5	1.1	0.7
	8	1.9	1.5	2.1	1.0	0.7	0.7	0.5	0.3	0.4	0.5	4.2	1.3	0.7
swl= +5.0 ft (maximum flood)														
12.5	3	0.8	0.6	0.5	0.3	0.1	0.2	0.2	0.2	0.1	0.1	0.7	0.2	0.1
	6	1.5	1.6	1.6	0.7	0.6	0.7	0.4	0.4	0.3	0.4	1.8	0.6	0.2
	8	1.9	1.8	2.0	1.0	0.8	0.7	0.5	0.4	0.3	0.5	2.7	0.9	0.4
14.3	3	0.8	0.7	0.9	0.3	0.2	0.4	0.2	0.3	0.1	0.2	0.5	0.2	0.1
	6	1.5	1.4	1.8	0.7	0.5	0.6	0.4	0.4	0.2	0.4	1.7	0.7	0.2
	8	2.5	1.9	2.3	1.1	0.9	0.8	0.5	0.5	0.3	0.5	2.8	1.2	0.4
16.7	3	0.8	0.5	0.9	0.3	0.2	0.3	0.2	0.2	0.1	0.3	0.5	0.2	0.1
	6	1.5	1.2	1.6	0.6	0.4	0.5	0.4	0.3	0.2	0.4	1.7	0.6	0.2
	8	1.7	1.5	1.7	0.7	0.5	0.6	0.4	0.3	0.3	0.5	2.4	0.9	0.3

(Sheet 2 of 3)

Table 9 (Concluded)

Test Wave		Wave Height, ft, at Indicated Gauge Location												
Period sec	Height ft	Gauge 1	Gauge 2	Gauge 3	Gauge 4	Gauge 5	Gauge 6	Gauge 7	Gauge 8	Gauge 9	Gauge 10	Gauge 11	Gauge 12	Gauge 13
swl = +8.0 ft														
12.5	3	0.4	0.4	0.7	0.3	0.1	0.3	0.2	0.2	0.1	0.1	0.9	0.4	0.1
	6	1.2	1.1	2.0	1.0	0.6	0.8	0.6	0.6	0.4	0.5	2.3	1.1	0.6
	8	1.7	1.7	3.1	1.6	1.2	1.4	1.0	0.9	0.8	0.9	3.7	1.7	0.9
14.3	3	0.5	0.4	0.8	0.3	0.1	0.3	0.2	0.2	0.1	0.3	1.0	0.5	0.1
	6	1.1	0.9	1.8	0.8	0.5	0.7	0.5	0.5	0.3	0.6	2.1	1.1	0.5
	8	1.6	1.4	2.6	1.3	0.8	0.9	0.6	0.7	0.5	0.8	2.9	1.5	0.8
16.7	3	0.4	0.3	0.6	0.2	0.1	0.2	0.1	0.1	0.1	0.2	0.9	0.4	0.1
	6	1.0	0.9	1.3	0.7	0.4	0.5	0.4	0.4	0.3	0.6	2.2	1.2	0.5
	8	1.4	1.2	1.9	1.1	0.7	0.8	0.5	0.5	0.5	0.7	3.2	1.4	0.7
swl = +11.0 ft														
12.5	8	1.5	1.7	2.9	1.4	1.3	1.1	0.9	0.7	0.7	0.8	3.5	1.3	1.0
14.3	8	1.4	1.6	2.5	1.2	1.3	1.0	0.8	0.6	0.7	0.9	3.4	1.3	0.9
16.7	8	1.3	1.5	2.1	1.1	1.0	0.9	0.7	0.6	0.6	0.8	3.5	1.1	0.9

(Sheet 3 of 3)

Table 10
Wave Height for Plan 6

Test Wave		Wave Height, ft, at Indicated Gauge Location												
Period sec	Height ft	Gauge 1	Gauge 2	Gauge 3	Gauge 4	Gauge 5	Gauge 6	Gauge 7	Gauge 8	Gauge 9	Gauge 10	Gauge 11	Gauge 12	Gauge 13
swl = +8.0 ft														
12.5	3	0.4	0.3	0.4	0.2	0.2	0.3	0.1	0.1	0.1	0.1	0.9	0.4	0.1
	6	1.1	0.9	1.5	0.9	0.7	0.8	0.5	0.5	0.4	0.5	2.4	1.0	0.5
	8	1.5	1.4	2.2	1.3	1.0	1.1	0.8	0.8	0.7	0.9	3.6	1.5	0.9
14.3	3	0.5	0.4	0.6	0.3	0.2	0.3	0.2	0.2	0.1	0.2	1.0	0.5	0.1
	6	1.1	0.9	1.4	0.7	0.5	0.6	0.4	0.5	0.3	0.6	2.1	1.1	0.4
	8	1.6	1.4	2.1	1.1	0.8	0.8	0.6	0.7	0.5	0.9	2.7	1.4	0.7
16.7	3	0.4	0.3	0.6	0.2	0.1	0.3	0.1	0.1	0.1	0.2	0.9	0.4	0.1
	6	1.1	1.0	1.2	0.6	0.5	0.6	0.4	0.5	0.3	0.6	2.2	1.1	0.5
	8	1.3	1.2	1.7	0.9	0.8	0.8	0.5	0.6	0.5	0.8	3.3	1.4	0.7
swl = +11.0 ft														
12.5	8	1.5	1.7	2.3	1.3	1.4	1.2	0.9	0.8	0.8	0.9	3.5	1.3	1.0
14.3	8	1.3	1.6	2.2	1.2	1.2	1.1	0.9	0.7	0.7	0.9	3.2	1.2	0.9
16.7	8	1.3	1.5	1.8	1.0	1.0	0.9	0.7	0.6	0.7	0.9	3.7	1.1	0.8

Table 11
Wave Heights Obtained Along Centerline of Proposed Plans 5 and 6 Breakwaters

Test Waves		Wave Height, ft, at Indicated Gauge Location		
Period, Sec	Height, ft	Gauge 14	Gauge 15	Gauge 16
swl = 0.0 ft				
12.5	8.0	1.8	1.6	1.9
14.3	8.0	1.5	1.3	1.7
16.7	8.0	1.7	1.5	2.0
swl = +5.0 ft (maximum ebb)				
12.5	8.0	3.3	3.2	3.9
14.3	8.0	3.4	3.0	4.8
16.7	8.0	3.0	2.9	4.4
swl = +5.0 ft (maximum flood)				
12.5	8.0	3.1	2.8	4.2
14.3	8.0	3.3	3.1	3.9
16.7	8.0	2.5	2.5	3.0
swl = +8.0 ft				
12.5	8.0	4.2	4.0	5.4
14.3	8.0	3.7	3.6	5.2
16.7	8.0	3.1	3.0	4.2
swl = +11.0 ft				
12.5	8.0	4.3	4.3	5.6
14.3	8.0	3.8	4.0	5.3
16.7	8.0	3.1	3.3	4.3

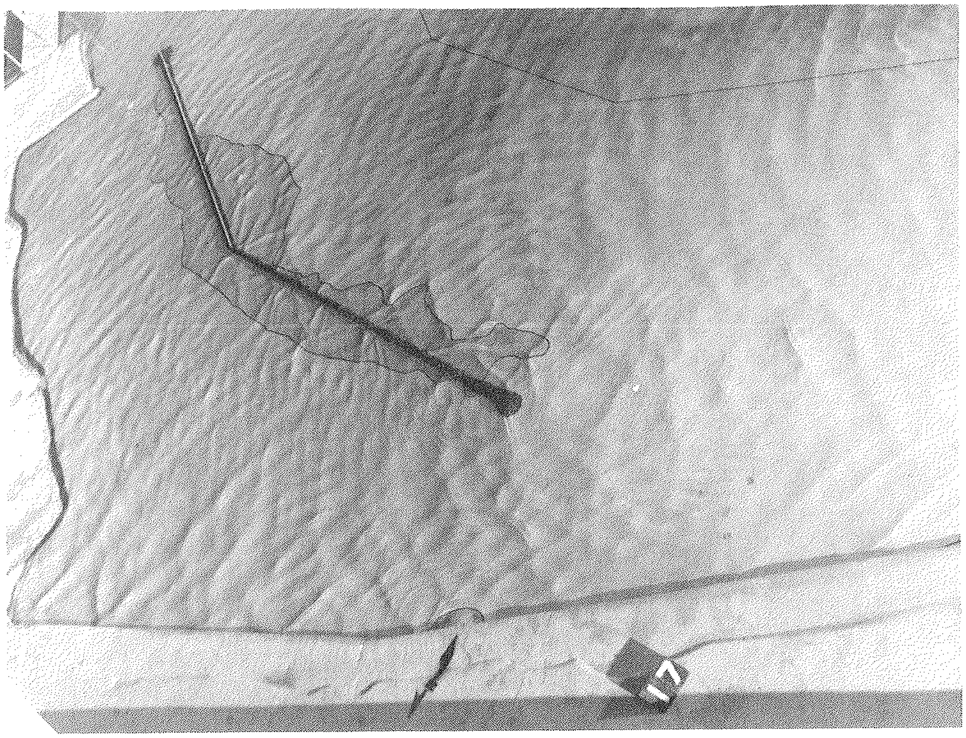


Photo 1. Typical wave patterns for existing conditions; preliminary tests; 12.5-sec, 8-ft test waves; swl = +8.0 ft

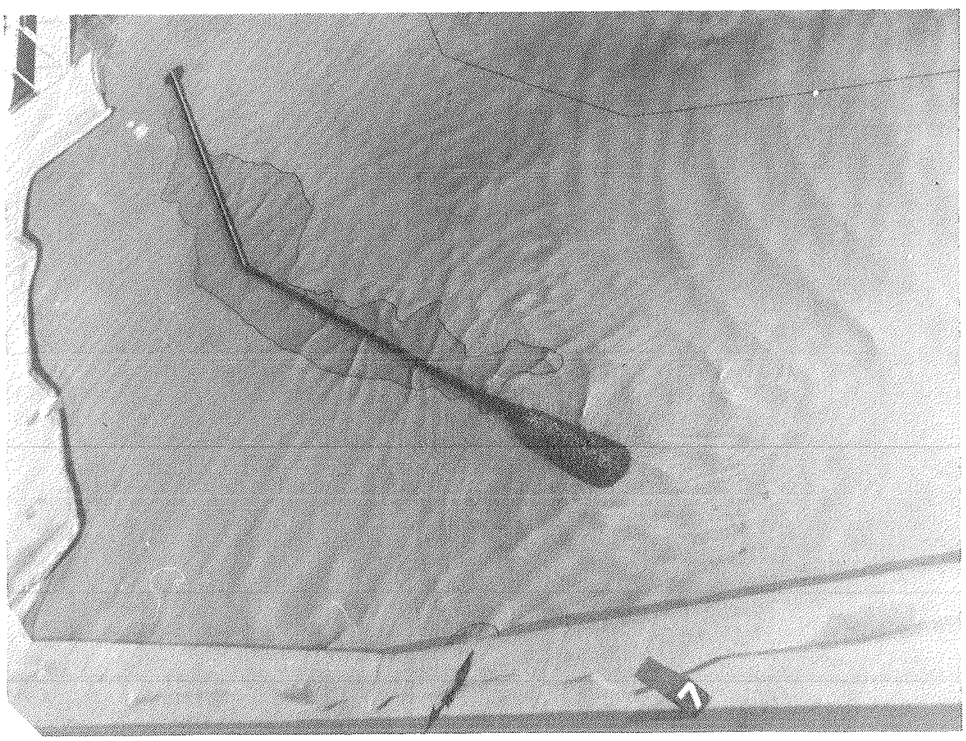


Photo 2. Typical wave patterns for Plan 1; preliminary tests; 12.5-sec, 8-ft test waves; swl = +8.0 ft

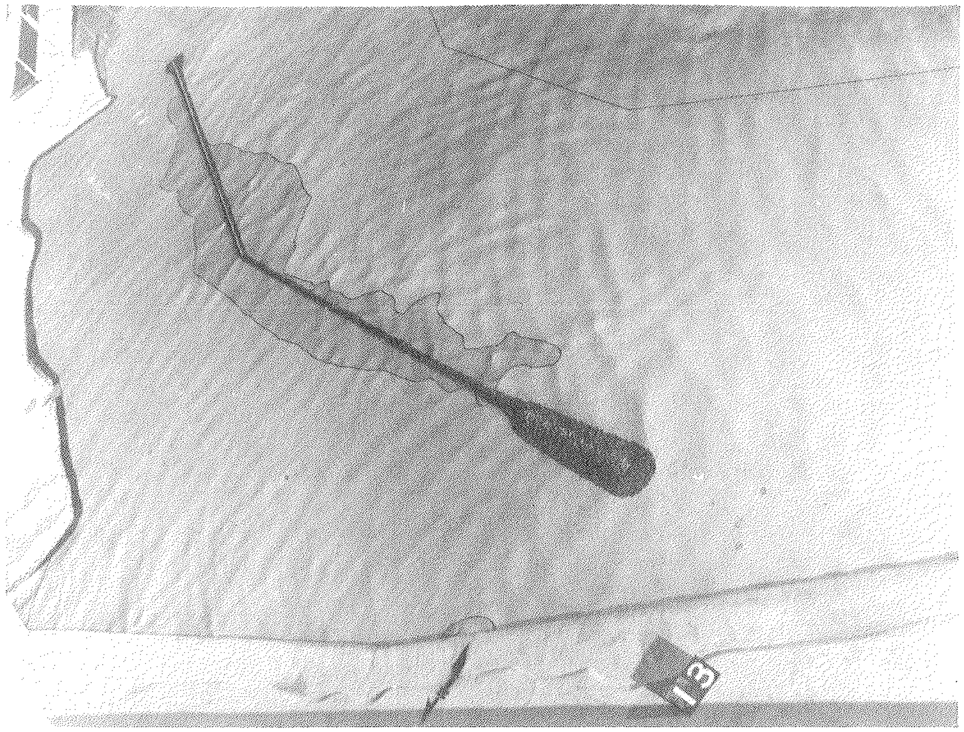


Photo 3. Typical wave patterns for Plan 1A; preliminary tests; 12.5-sec, 8-ft test waves; swl = +8.0 ft

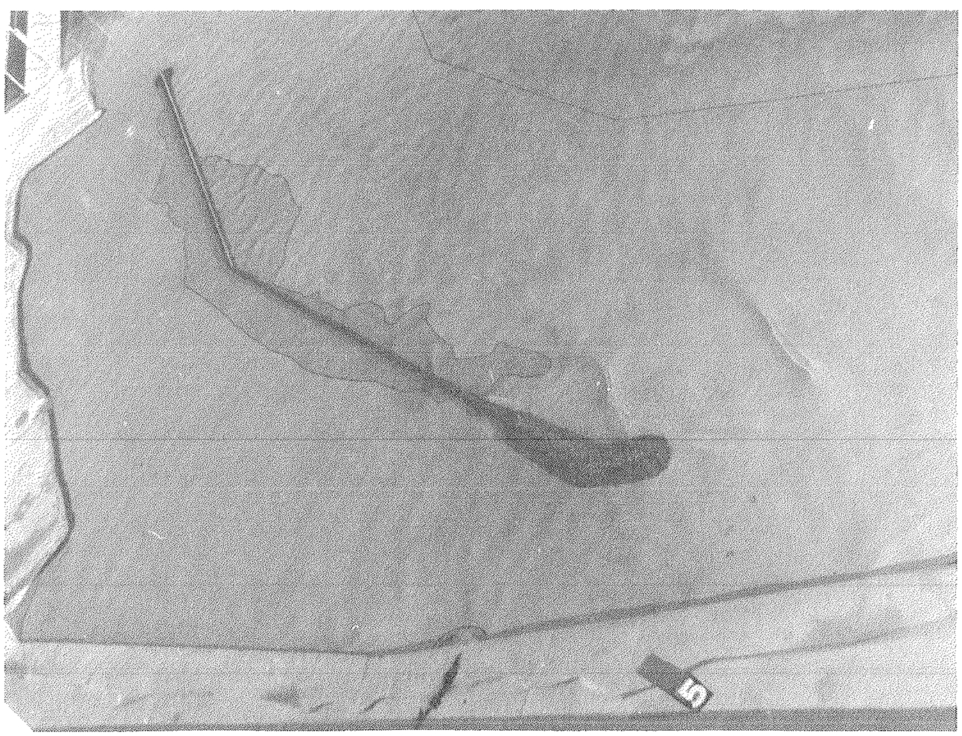


Photo 4. Typical wave patterns for Plan 1B; preliminary tests; 12.5-sec, 8-ft test waves; swl = +8.0 ft

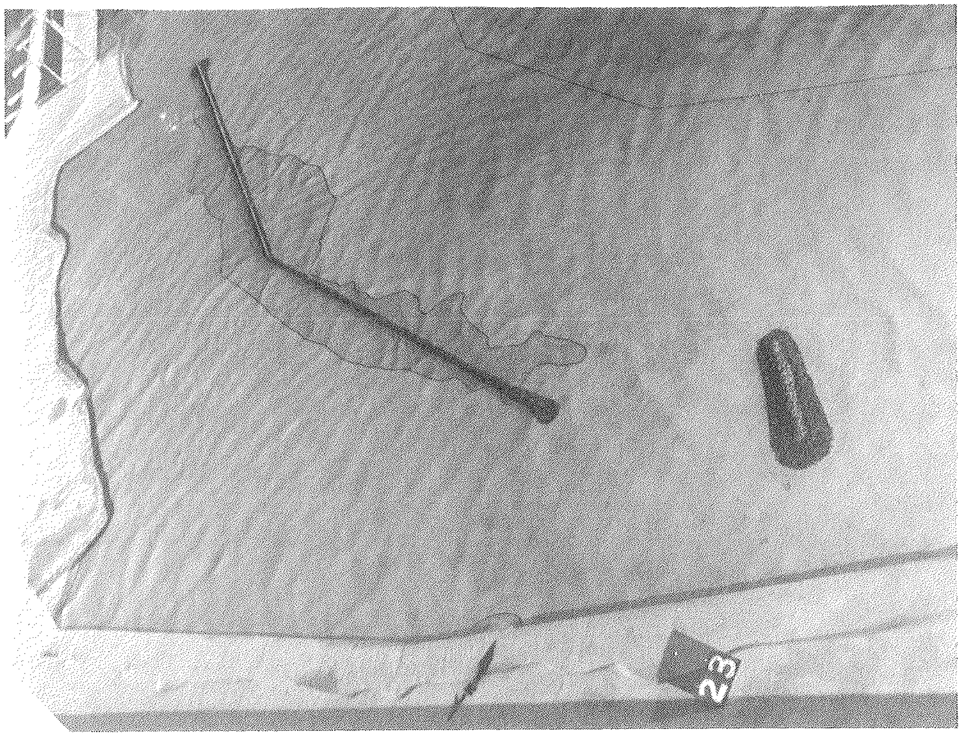


Photo 5. Typical wave patterns for Plan 2; preliminary tests; 12.5-sec, 8-ft test waves; swl = +8.0 ft

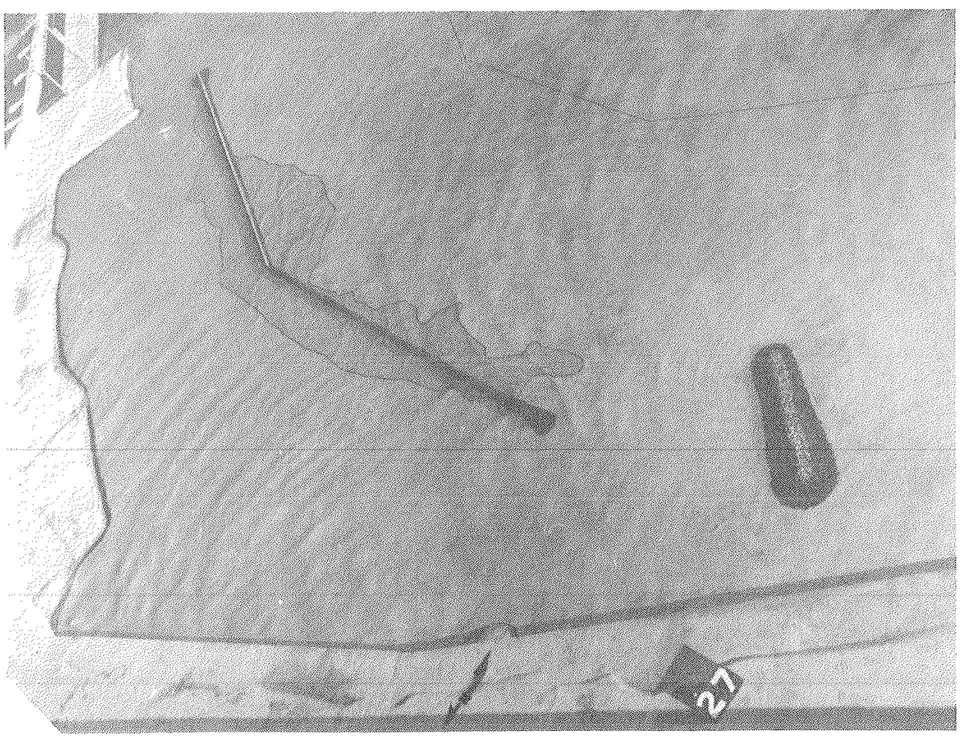


Photo 6. Typical wave patterns for Plan 2A; preliminary tests; 12.5-sec, 8-ft test waves; swl = +8.0 ft

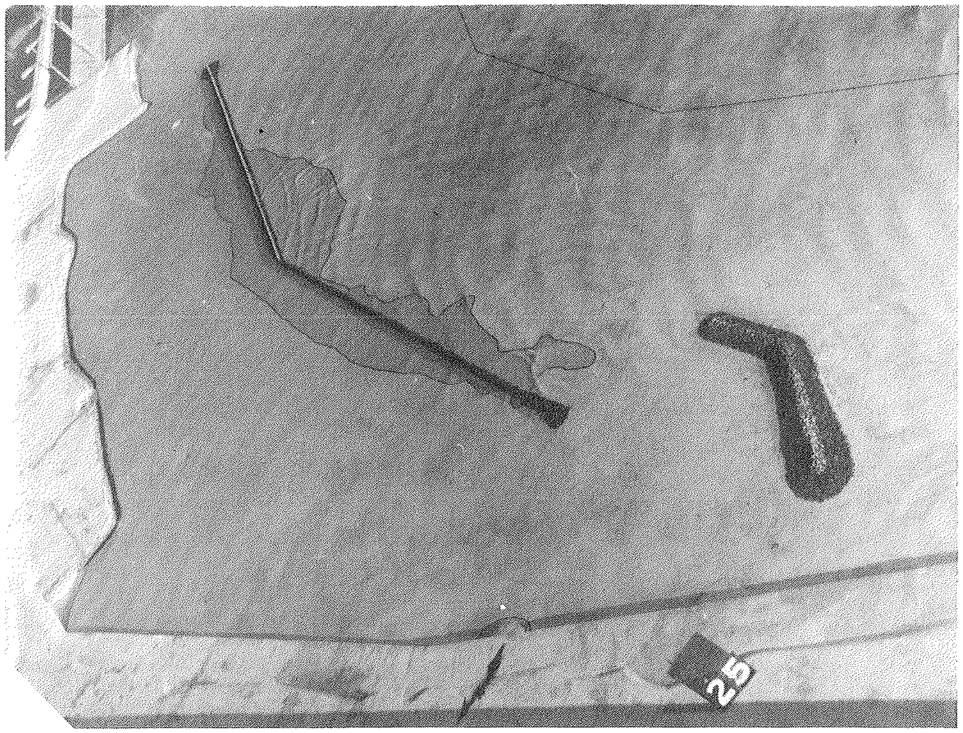


Photo 7. Typical wave patterns for Plan 2B; preliminary tests; 12.5-sec, 8-ft test waves; swl = +8.0 ft

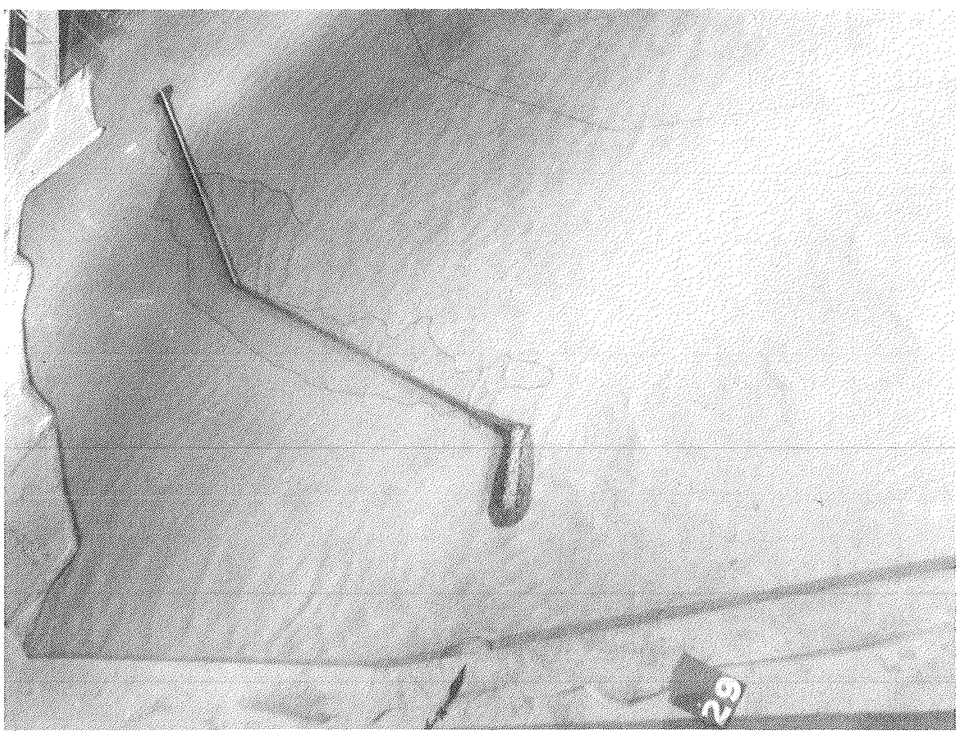


Photo 8. Typical wave patterns for Plan 3, preliminary tests; 12.5-sec, 8-ft test waves; swl = +8.0 ft

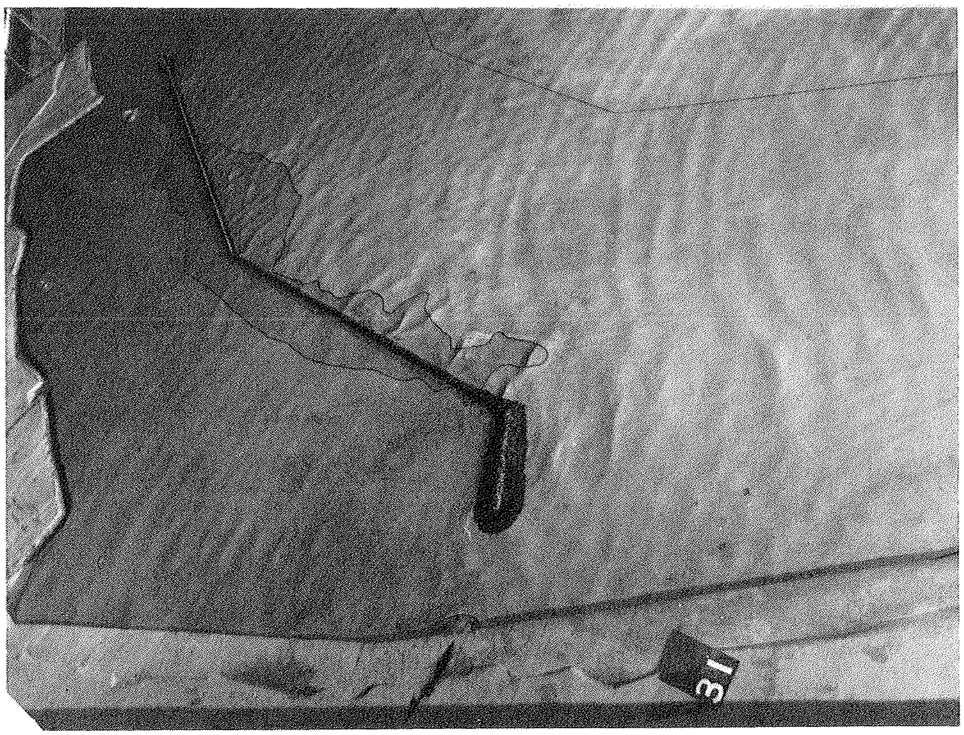


Photo 9. Typical wave patterns for Plan 3A, preliminary tests; 12.5-sec, 8-ft test waves; swl = +8.0 ft

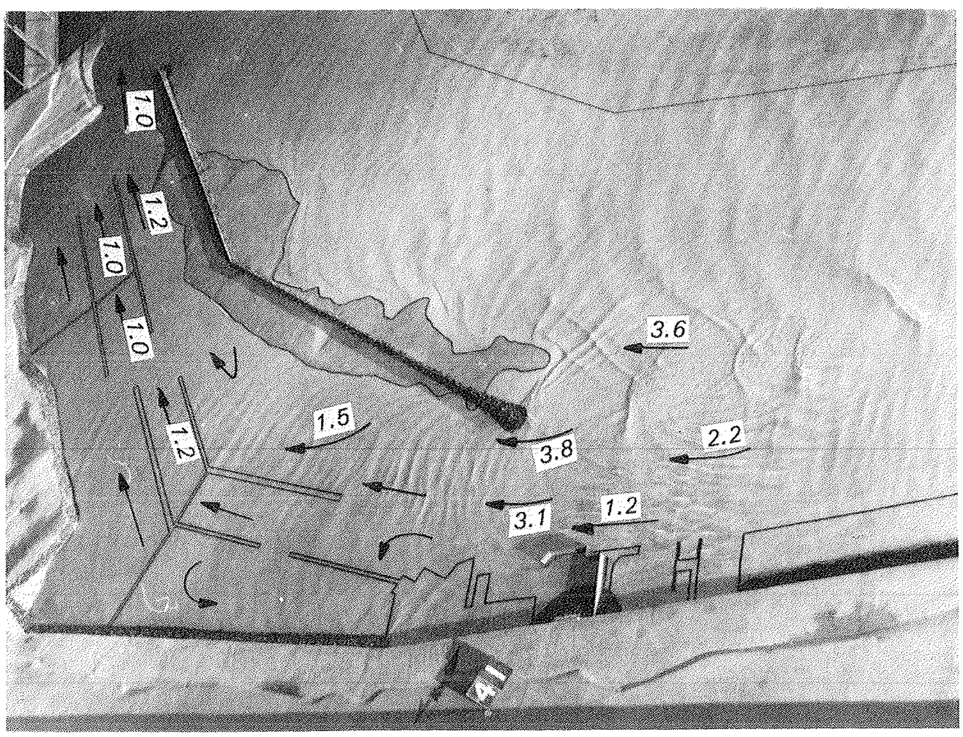


Photo 10. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for "modified" existing conditions; 12.5-sec, 8-ft test waves; swl = 0.0 ft

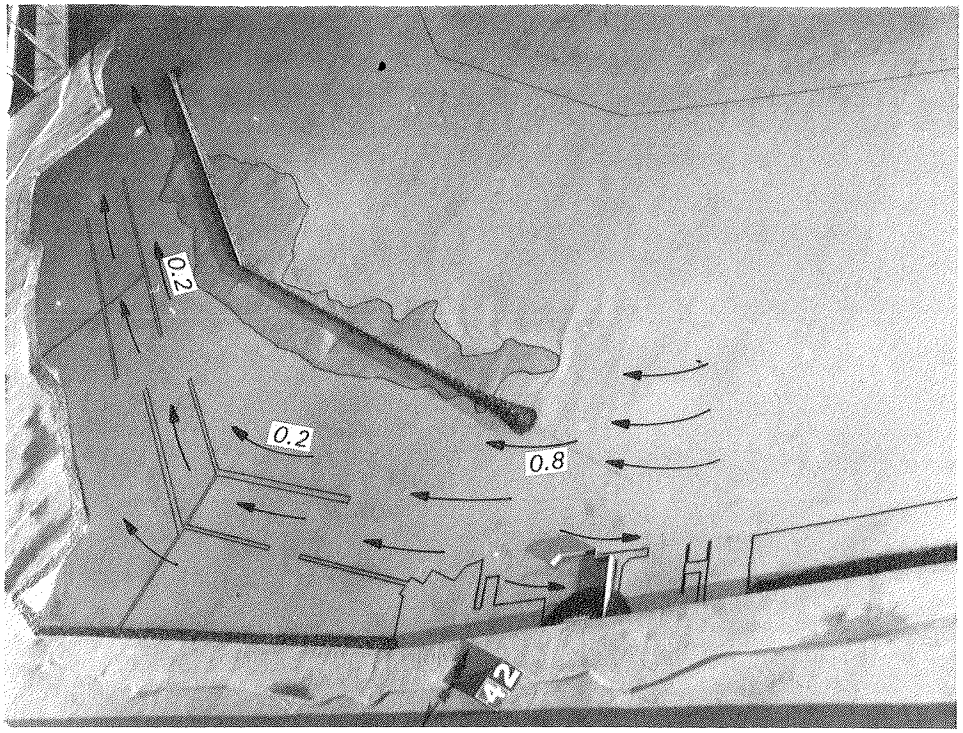


Photo 11. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for "modified" existing conditions; 14.3-sec, 3-ft test waves; swl = 0.0 ft

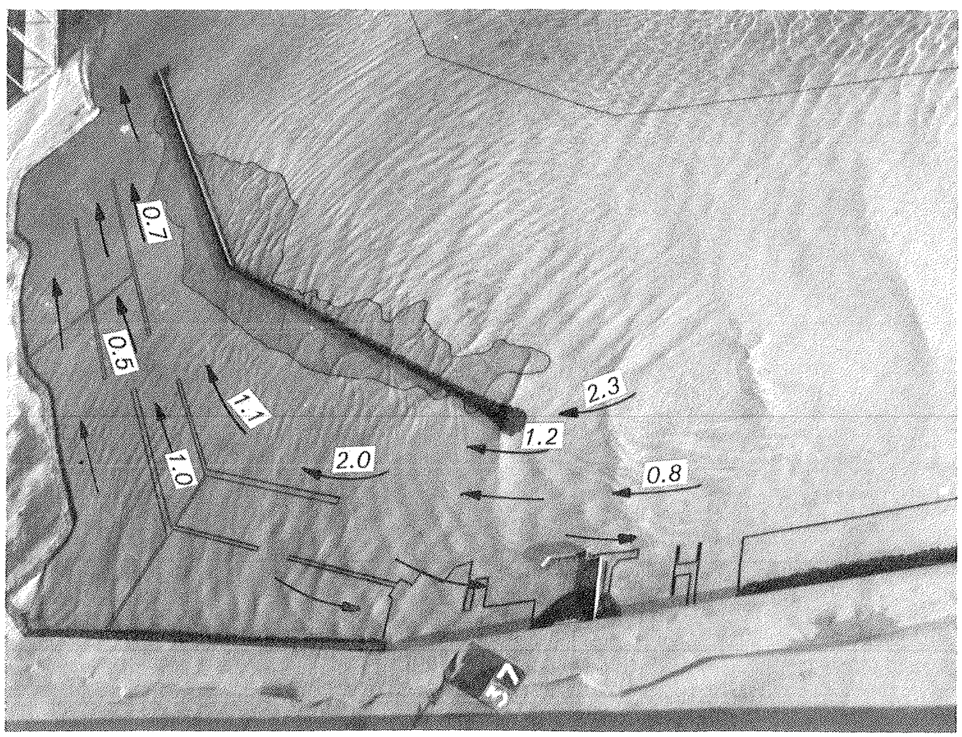


Photo 12. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for "modified" existing conditions; 12.5-sec, 8-ft test waves; swl = +5.0 ft (maximum ebb)

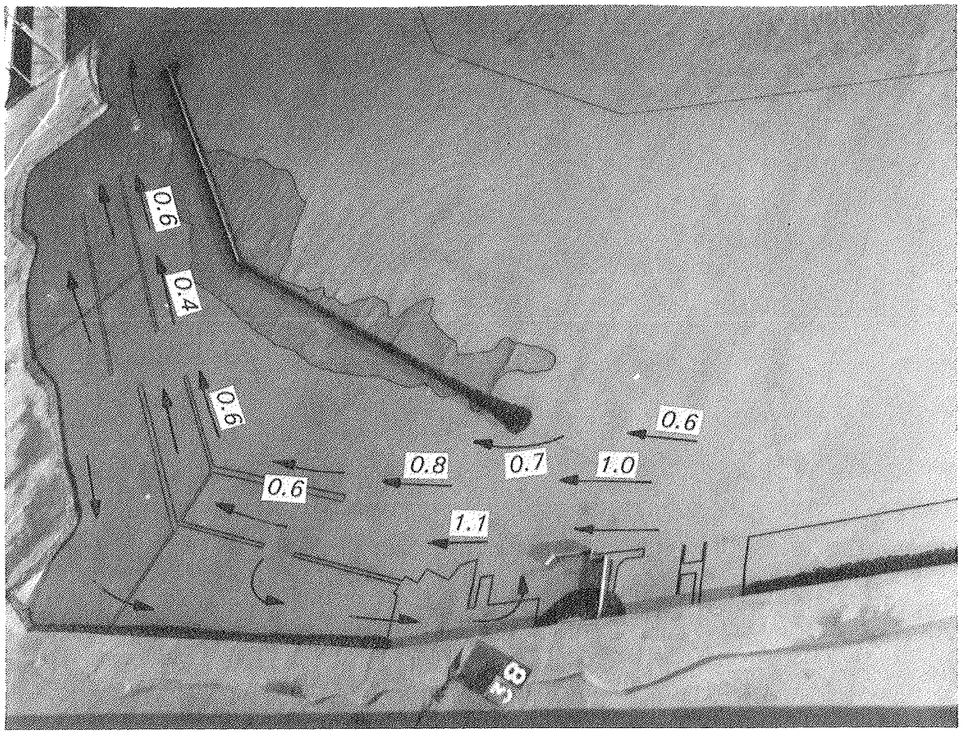


Photo 13. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for "modified" existing conditions; 14.3-sec, 3-ft test waves; swl = +5.0 ft (maximum ebb)

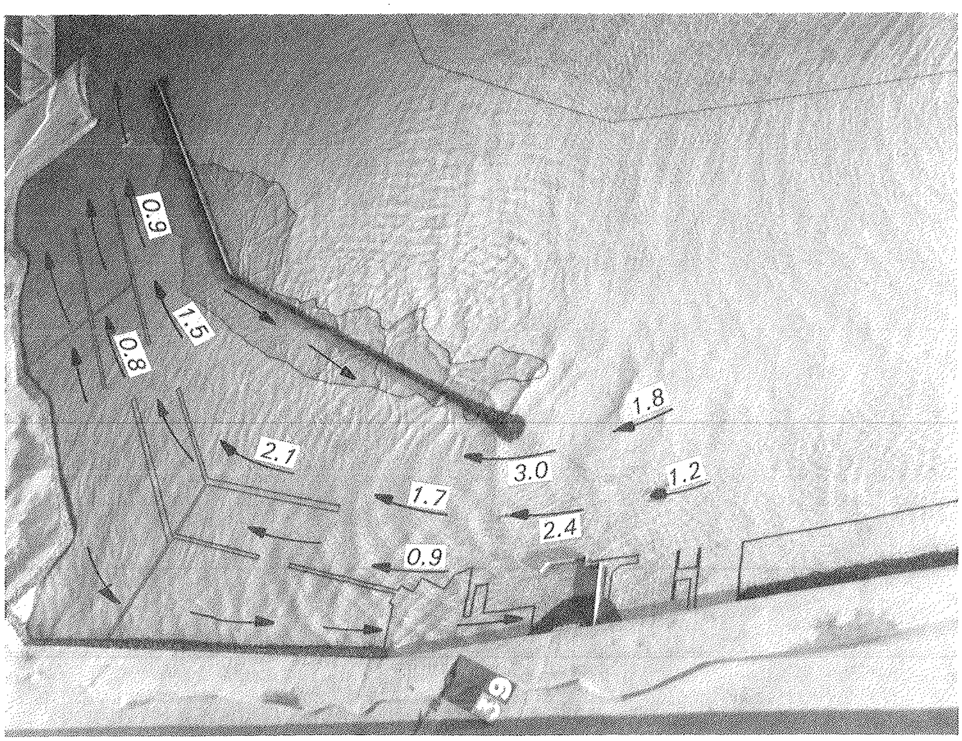


Photo 14. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for "modified" existing conditions; 12.5-sec, 8-ft test waves; swl = +5.0 ft (maximum flood)

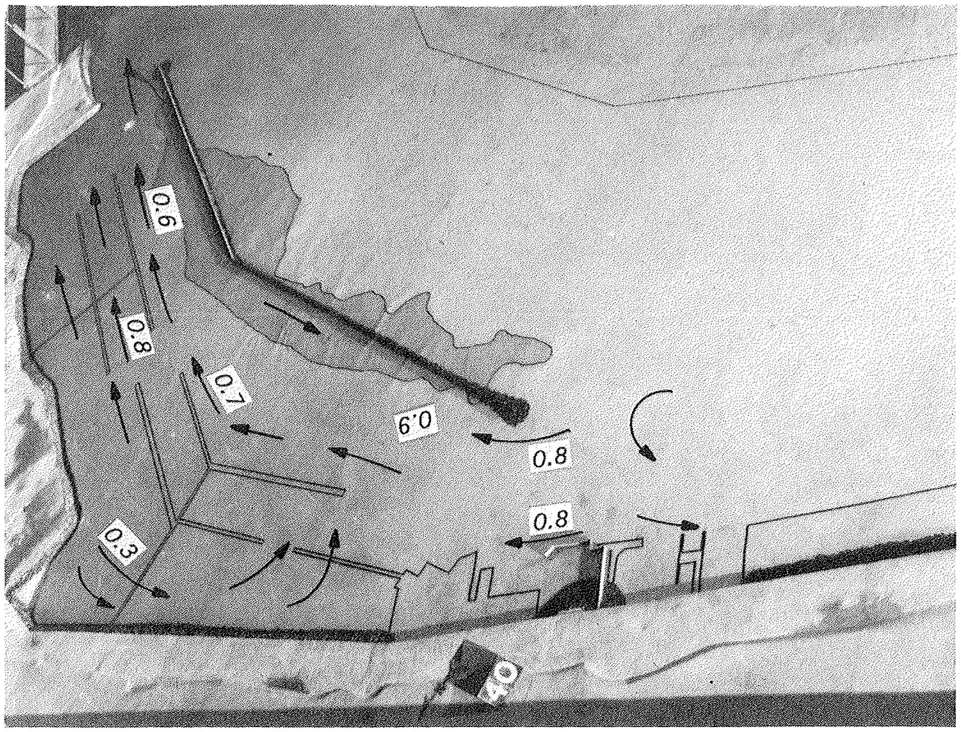


Photo 15. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for "modified" existing conditions; 14.3-sec, 3-ft test waves; swl = +5.0 ft (maximum flood)

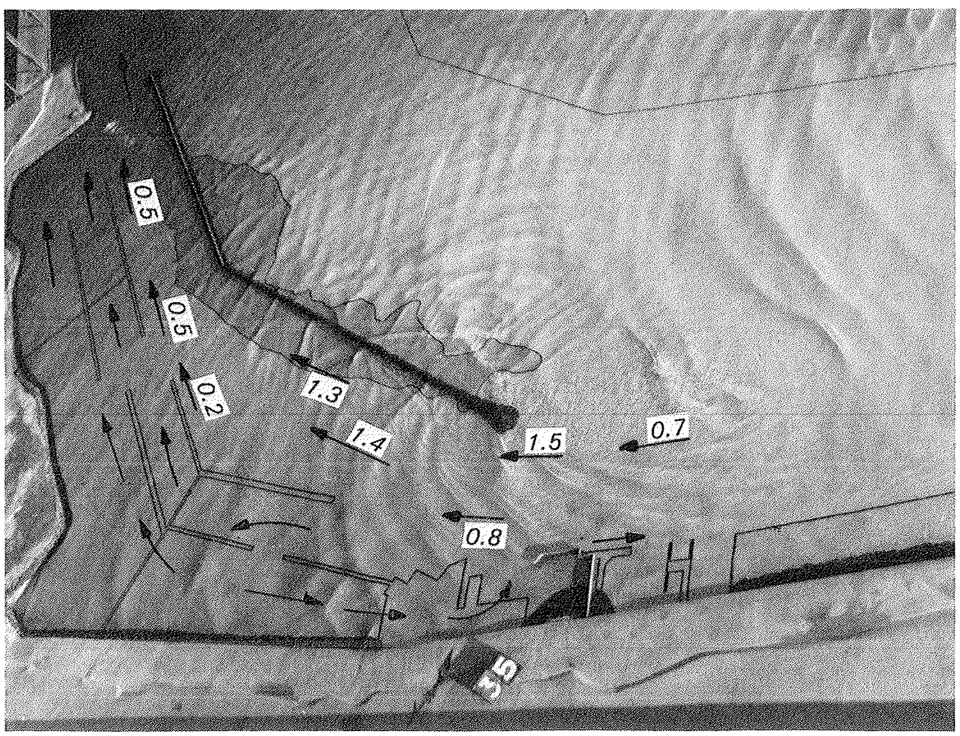


Photo 16. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for "modified" existing conditions; 12.5-sec, 8-ft test waves; swl = +8.0 ft

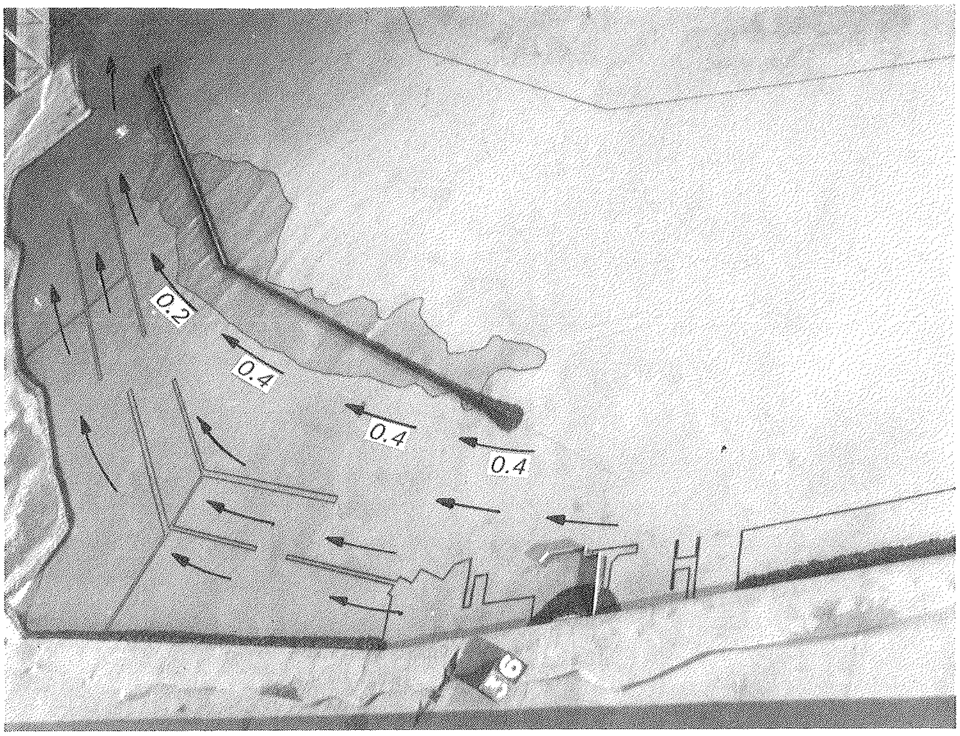


Photo 17. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for "modified" existing conditions; 14.3-ft, 3-ft test waves; swl = +8.0 ft

Photo 18. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for "modified" existing conditions; 12.5-sec, 8-ft test waves; swl = +11.0 ft

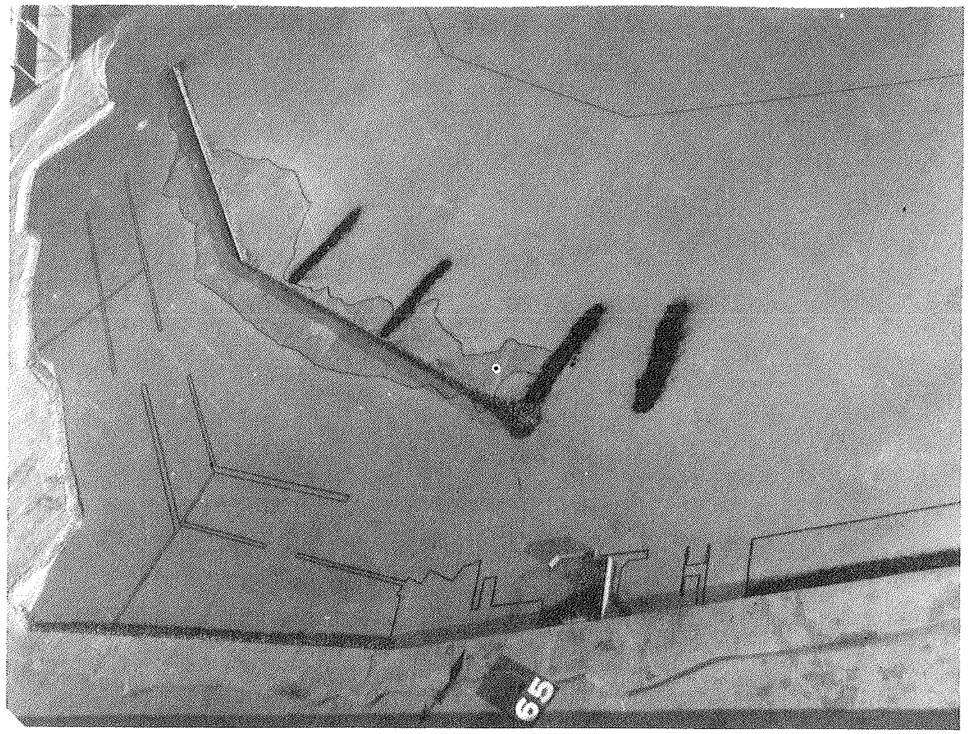


Photo 19. Placement of tracer material prior to model testing

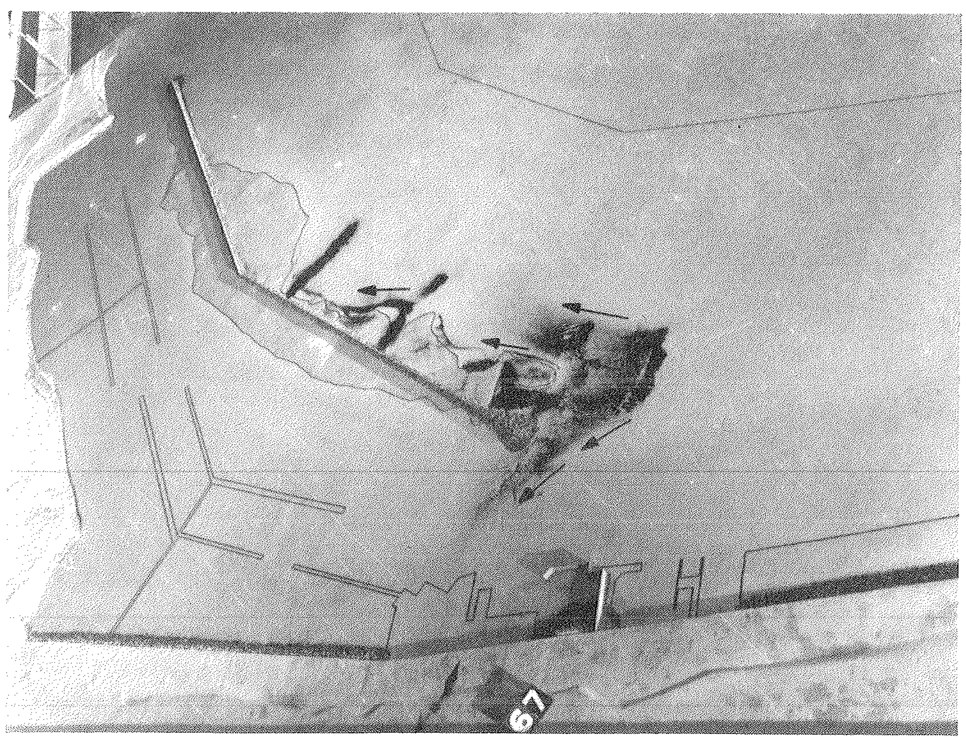


Photo 20. General movement of tracer material and subsequent deposits for "modified" existing conditions for 12.5-sec, 8-ft test waves; swl = 0.0 ft

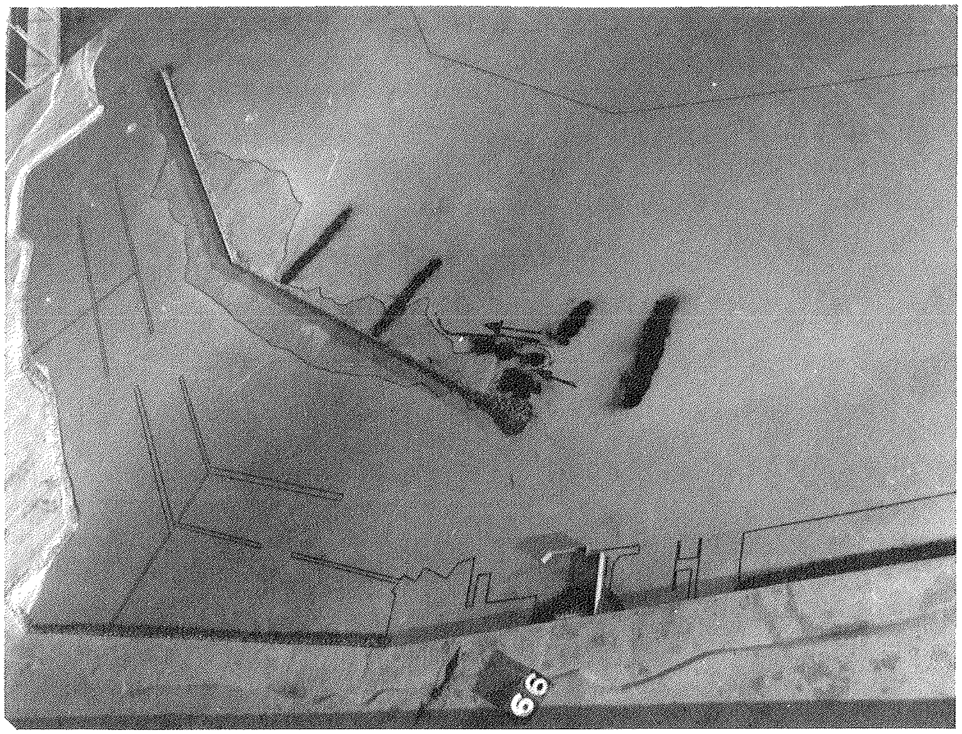


Photo 21. General movement of tracer material and subsequent deposits for "modified" existing conditions for 14.3-sec, 3-ft test waves; swl = 0.0 ft

Photo 22. General movement of tracer material and subsequent deposits for "modified" existing conditions for 12.5-sec, 8-ft test waves; swl = +5.0 ft (maximum ebb)

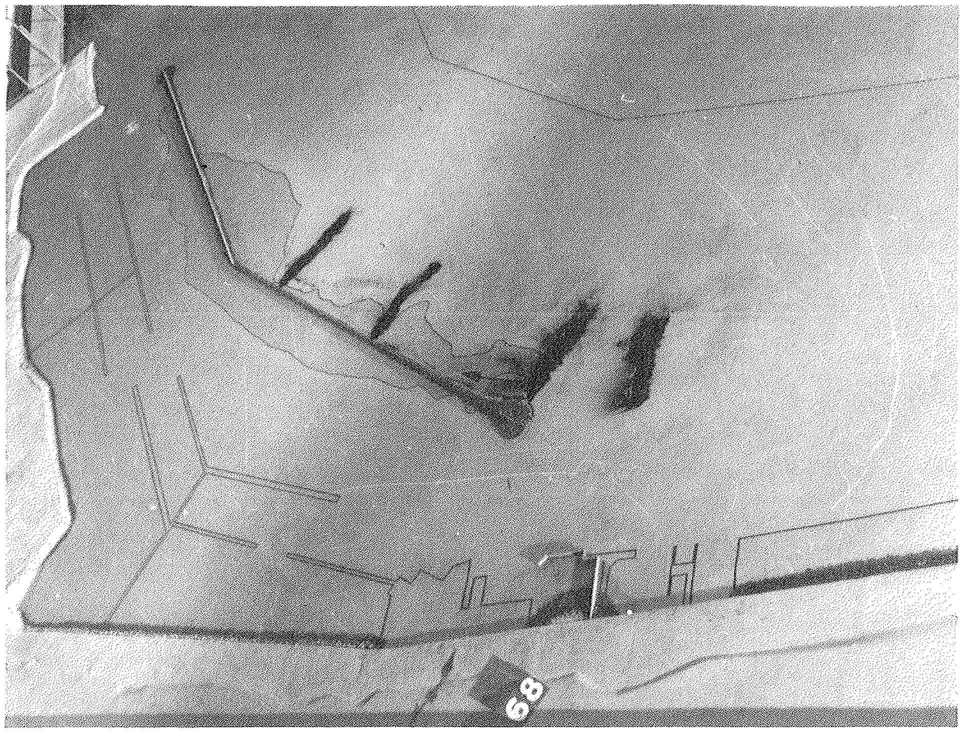


Photo 23. General movement of tracer material and subsequent deposits for "modified" existing conditions for 14.3-sec, 3-ft test waves; swl = +5.0 ft (maximum ebb)

Photo 24. General movement of tracer material and subsequent deposits for "modified" existing conditions for 12.5-sec, 8-ft test waves; swl = +5.0 ft (maximum flood)

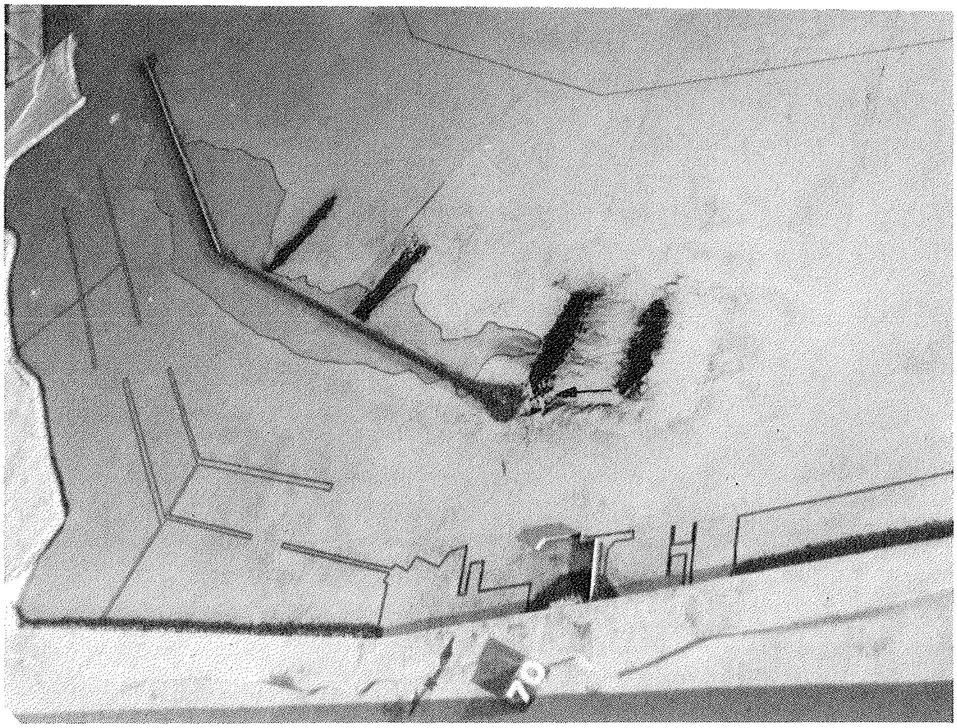


Photo 25. General movement of tracer material and subsequent deposits for "modified" existing conditions for 14.3-sec, 3-ft test waves; swl = +5.0 ft (maximum flood)

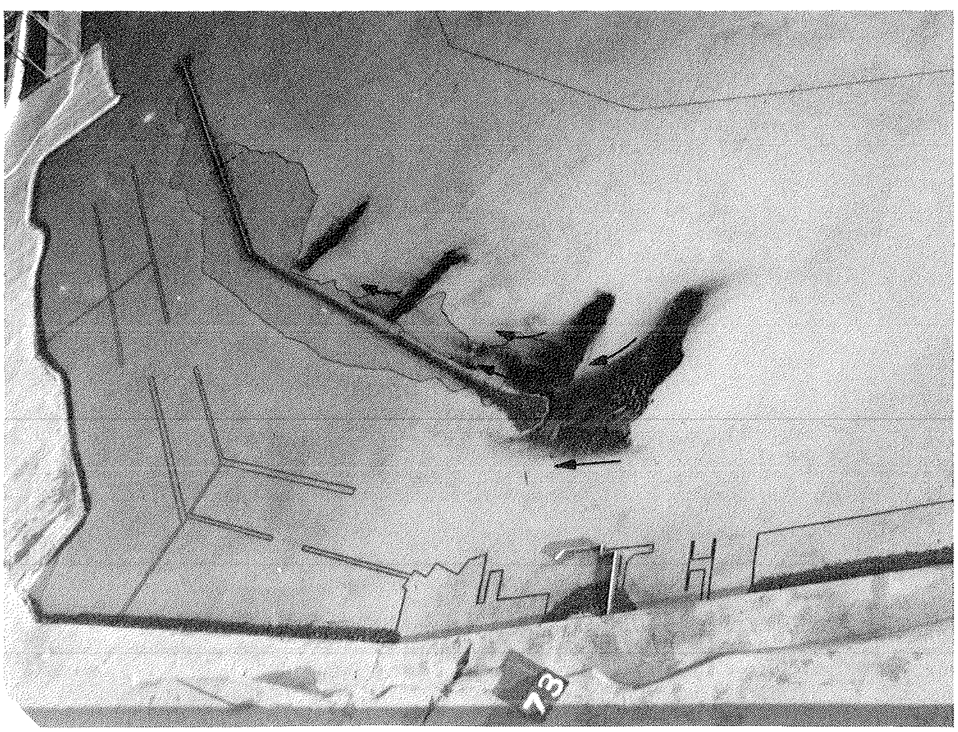


Photo 26. General movement of tracer material and subsequent deposits for "modified" existing conditions for 12.5-sec, 8-ft test waves; swl = +8.0 ft

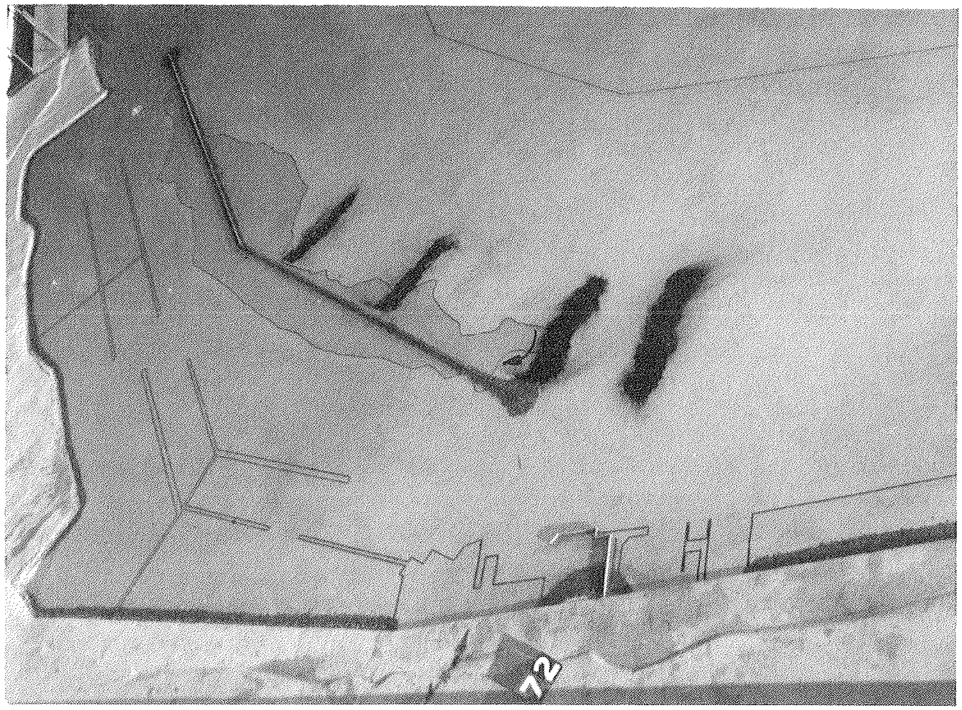


Photo 27. General movement of tracer material and subsequent deposits for "modified" existing conditions for 14.3-sec, 3-ft test waves; swl = +8.0 ft

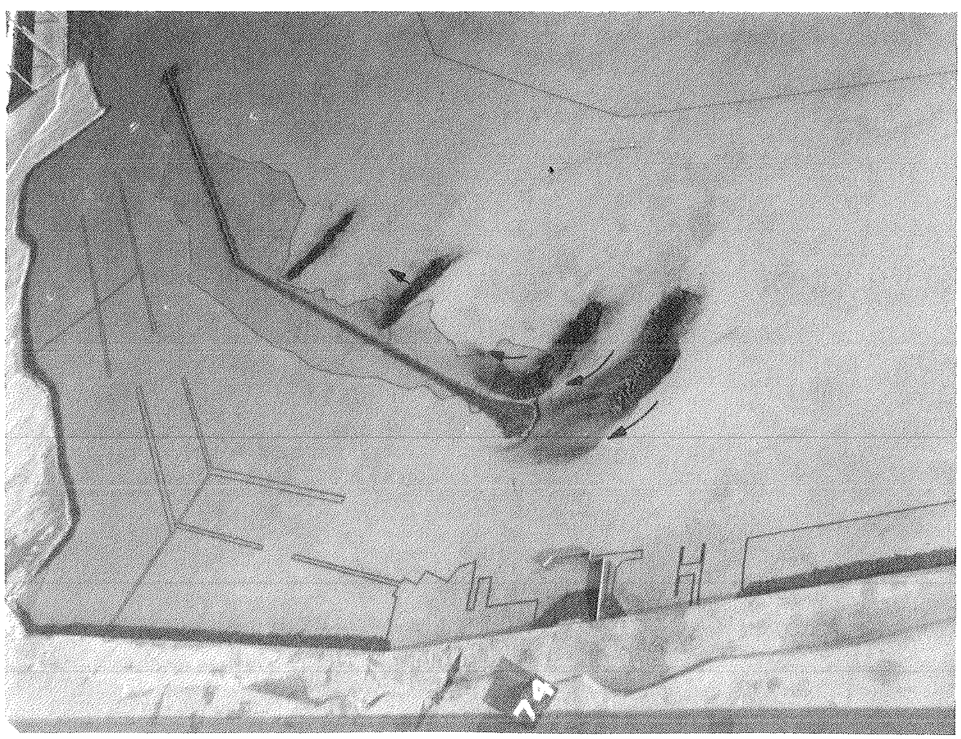


Photo 28. General movement of tracer material and subsequent deposits for "modified" existing conditions for 12.5-sec, 8-ft test waves; swl = +11.0 ft

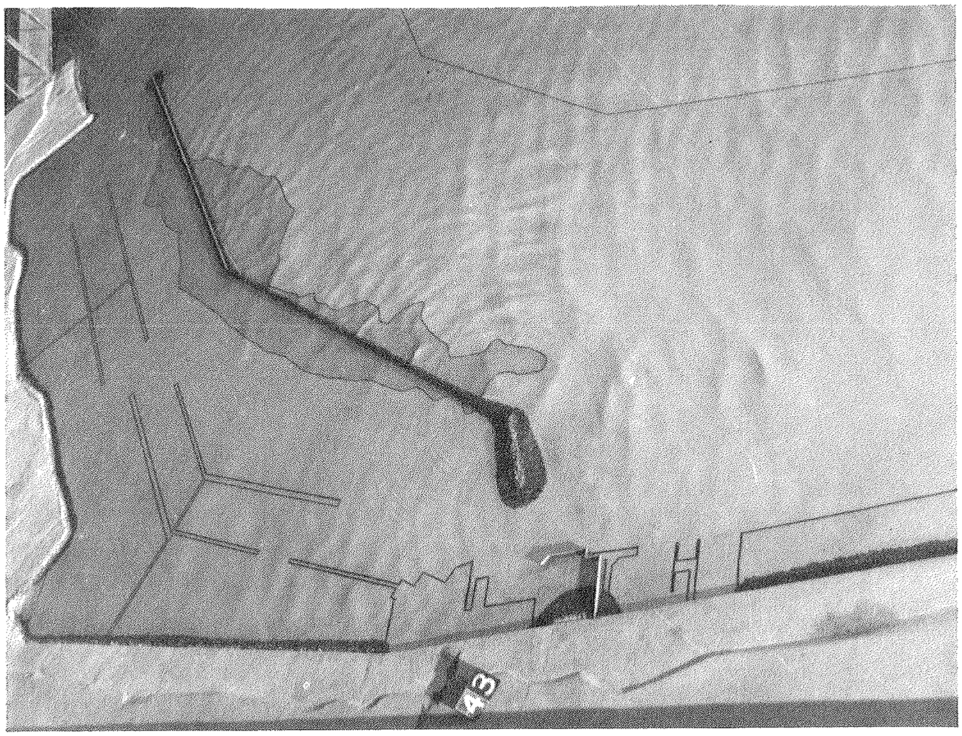


Photo 29. Typical wave patterns for Plan 4; 12.5-sec, 8-ft test waves; swl = +8.0 ft

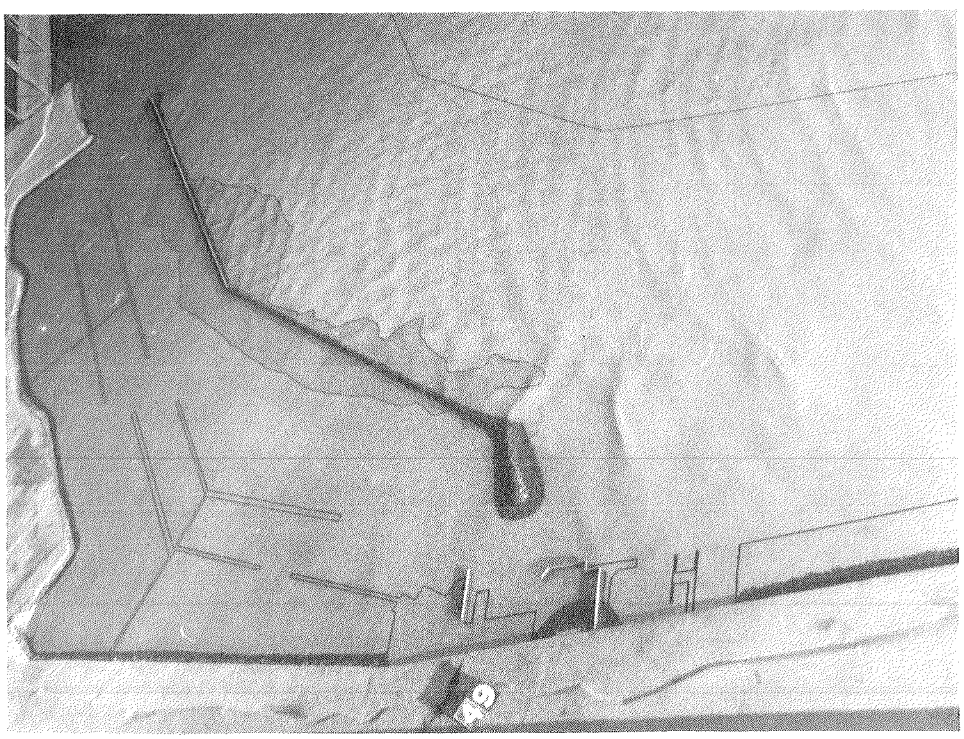


Photo 30. Typical wave patterns for Plan 4A; 12.5-sec, 8-ft test waves; swl = +8.0 ft

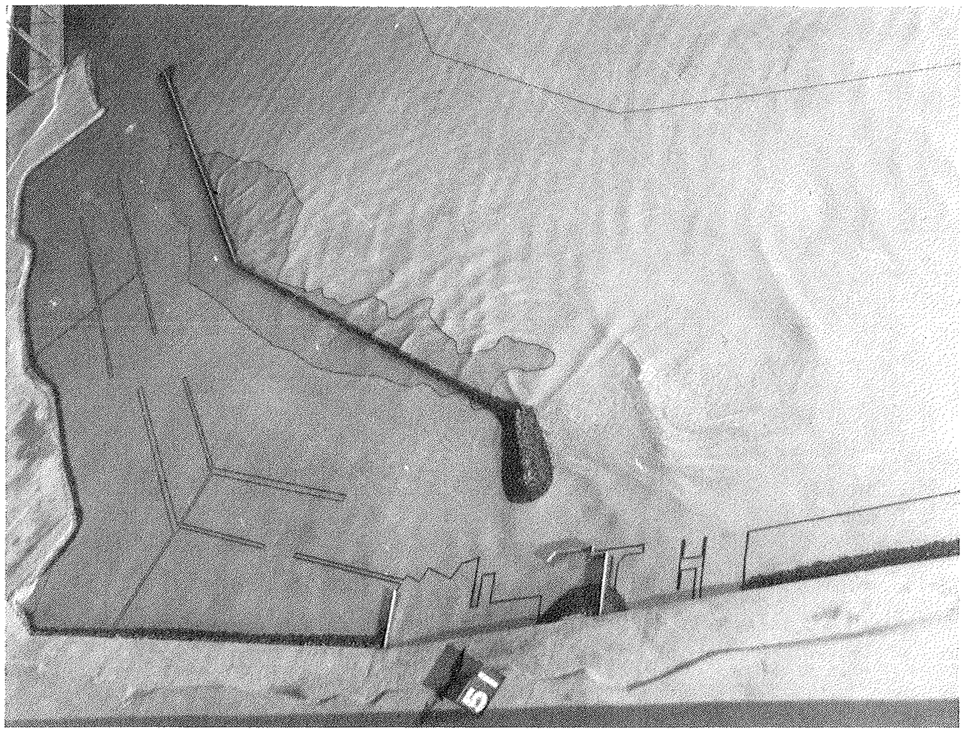


Photo 31. Typical wave patterns for Plan 4B; 12.5-sec, 8-ft test waves; swl = +8.0 ft

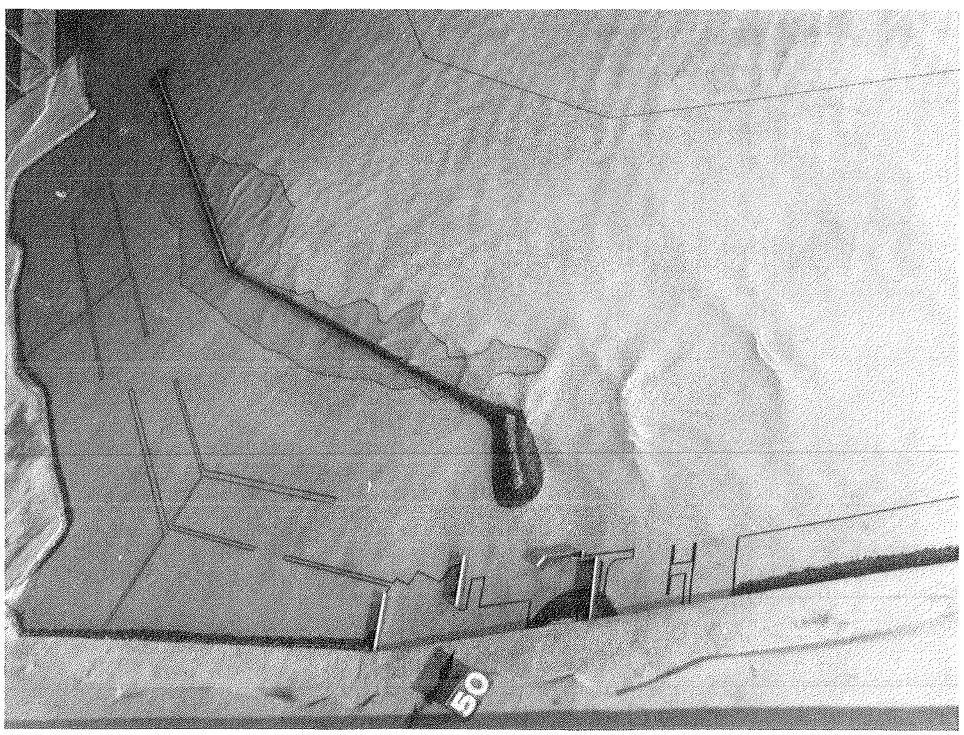


Photo 32. Typical wave patterns for Plan 4C; 12.5-sec, 8-ft test waves; swl = + 8.0 ft

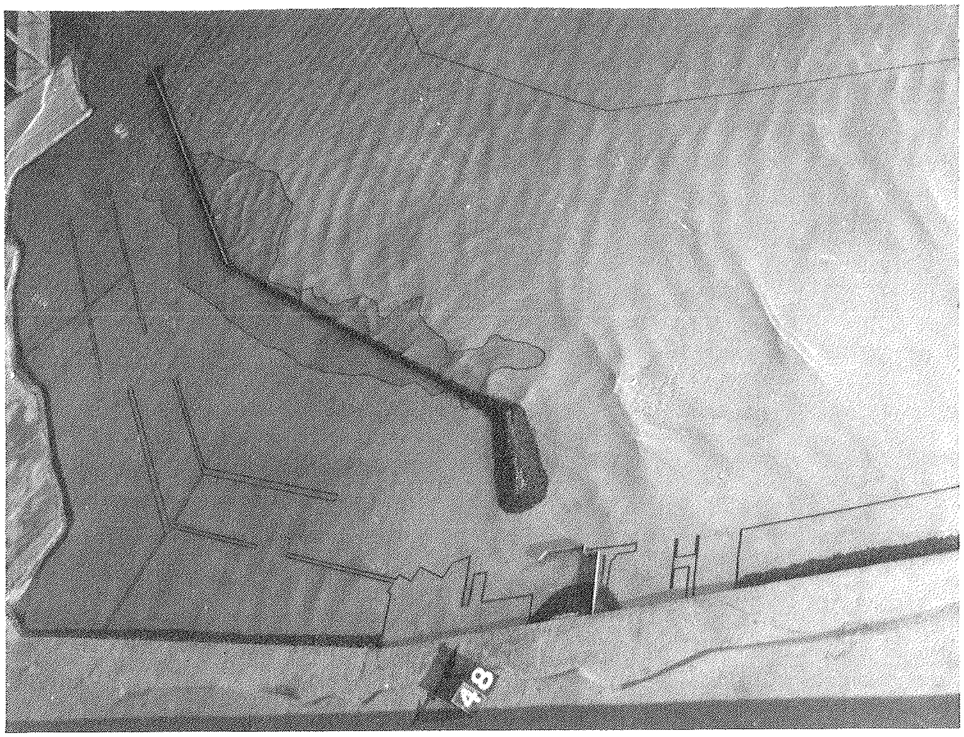


Photo 33. Typical wave patterns for Plan 5; 12.5-sec, 8-ft test waves; swl = +8.0 ft

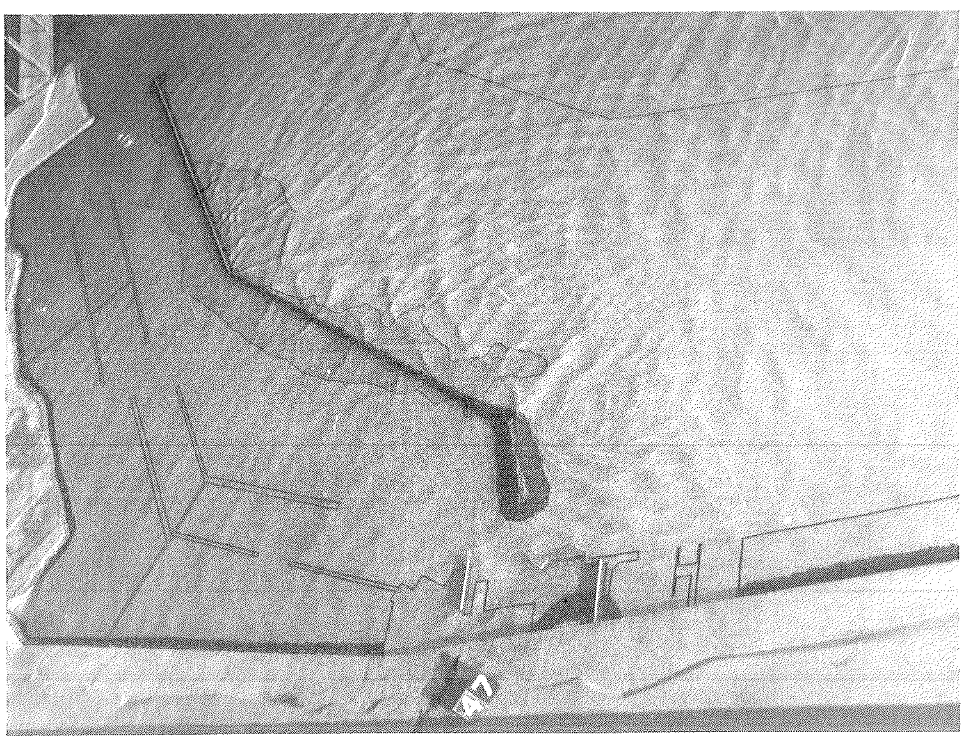


Photo 34. Typical wave patterns for Plan 5A; 12.5-sec, 8-ft test waves; swl = +8.0 ft

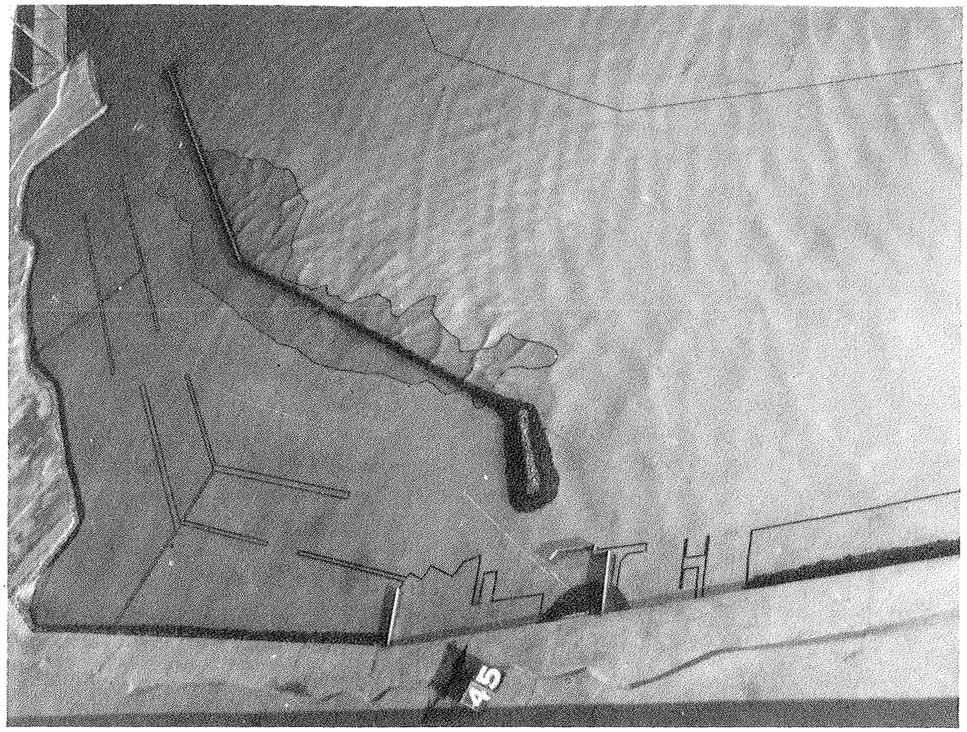


Photo 35. Typical wave patterns for Plan 5B; 12.5-sec, 8-ft test waves; swl = +8.0 ft

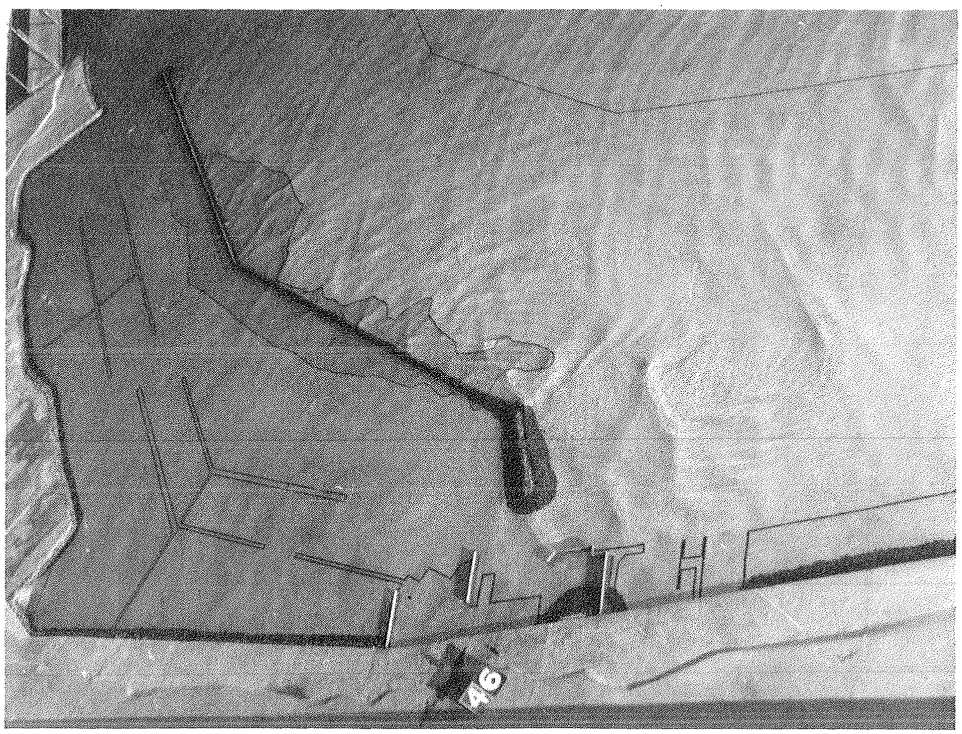


Photo 36. Typical wave patterns for Plan 5C; 12.5-sec, 8-ft test waves; swl = +8.0 ft

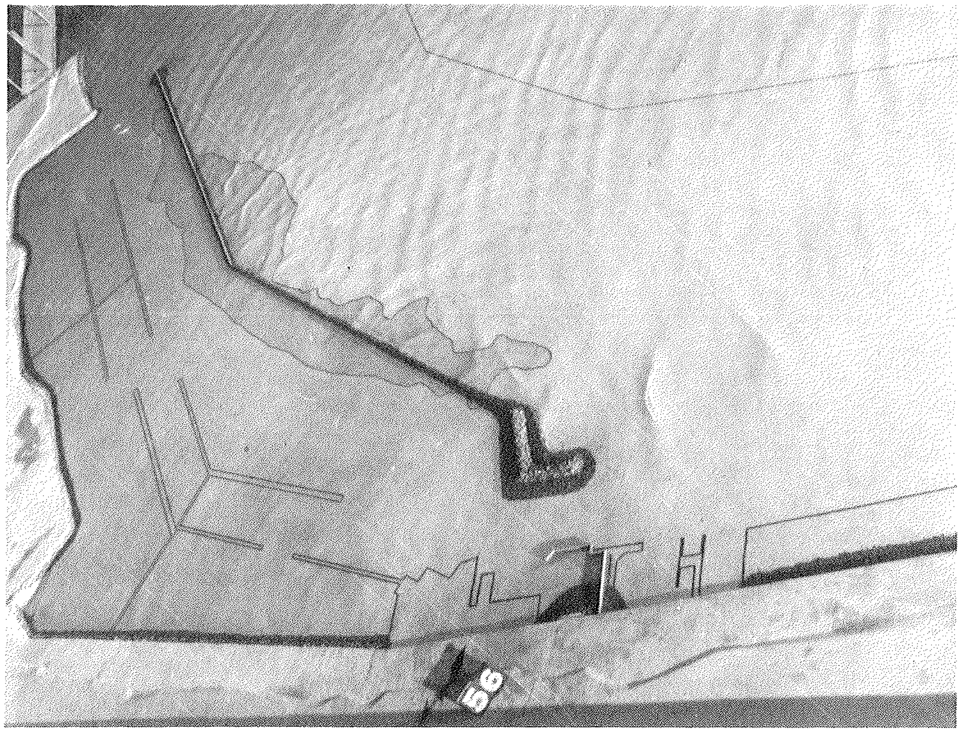


Photo 37. Typical wave patterns for Plan 6; 12.5-sec, 8-ft test waves; swl = +8.0 ft

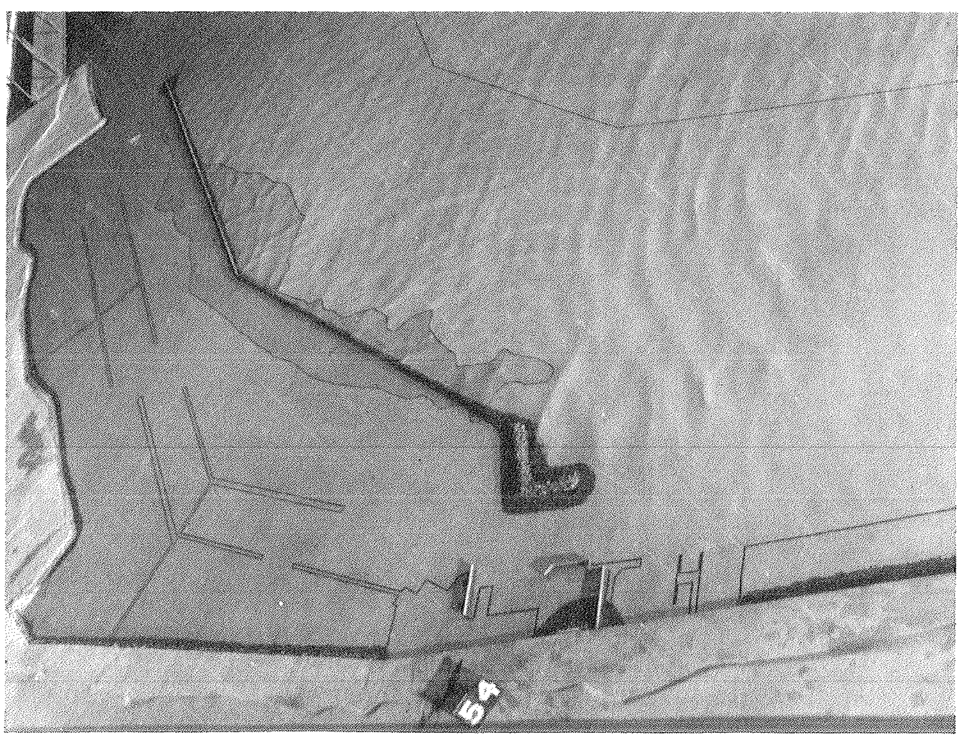


Photo 38. Typical wave patterns for Plan 6A; 12.5-sec, 8-ft test waves; swl = +8.0 ft

Photo 39. Typical wave patterns for Plan 6B; 12.5-sec, 8-ft test waves; swl = +8.0 ft

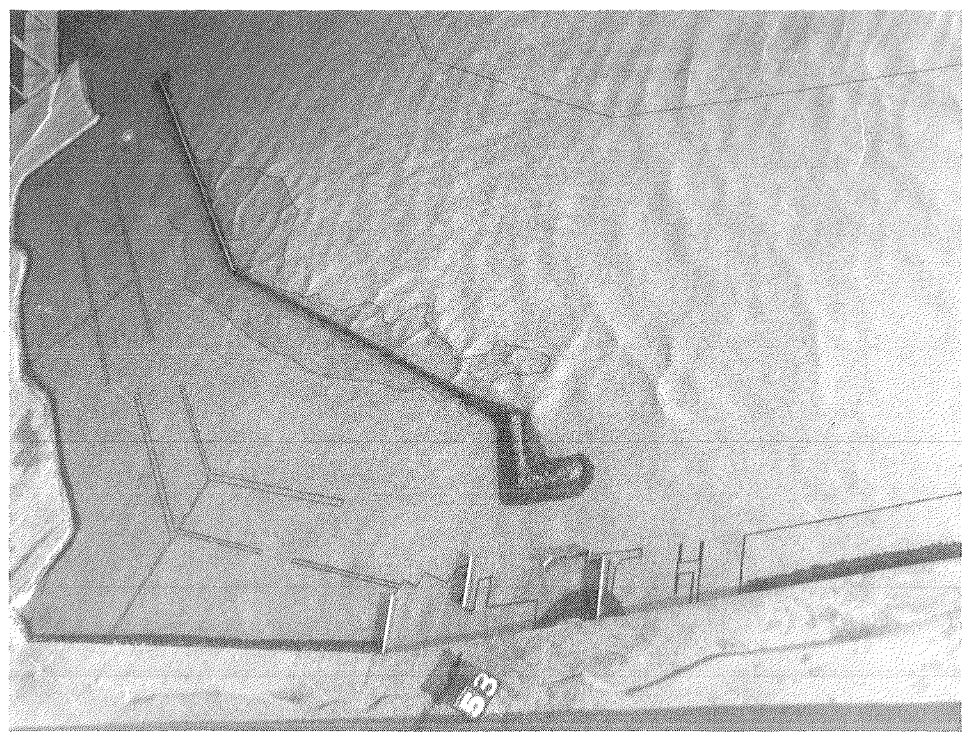


Photo 40. Typical wave patterns for Plan 6C; 12.5-sec, 8-ft test waves; swl = +8.0 ft

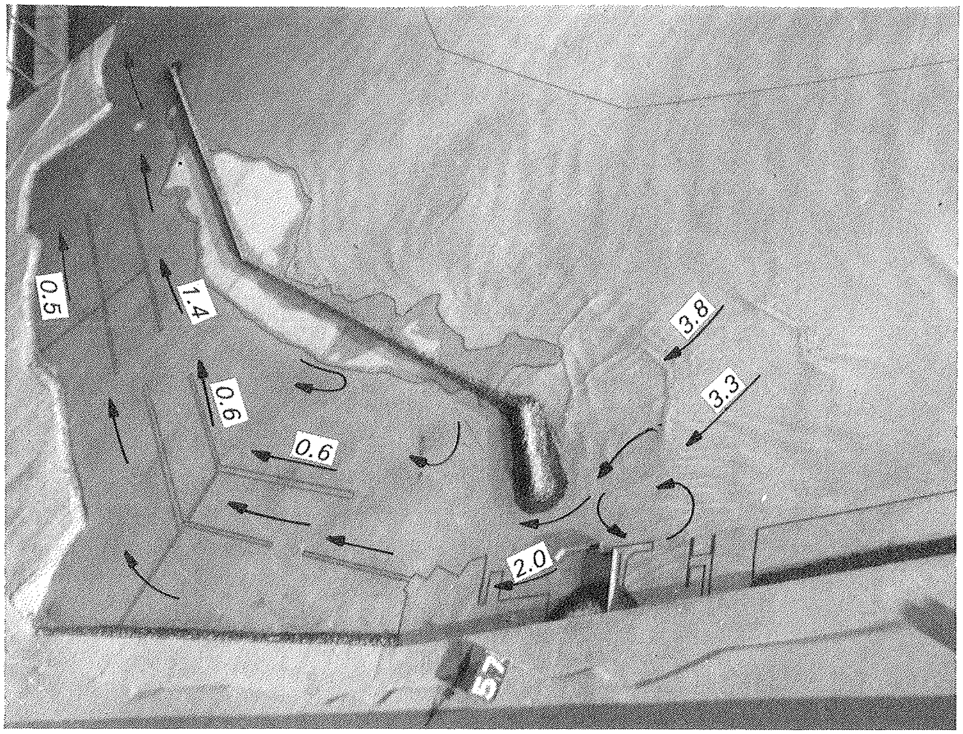


Photo 41. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for Plan 5; 12.5-sec, 8-ft test waves; swl = 0.0 ft

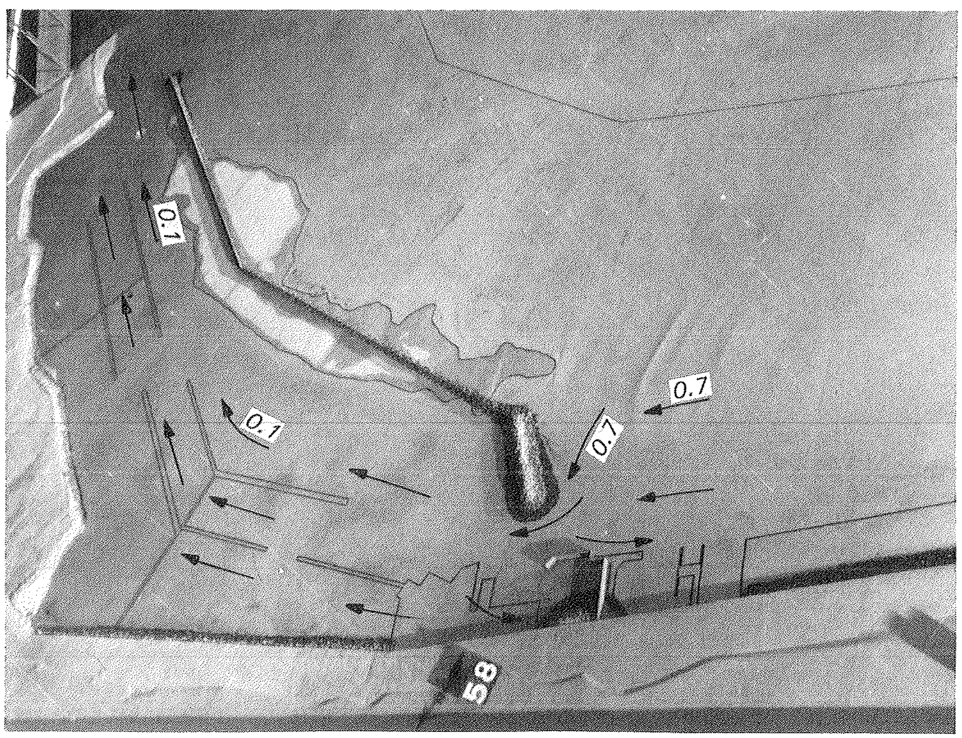


Photo 42. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for Plan 5; 14.3-sec, 3-ft test waves; swl = 0.0 ft

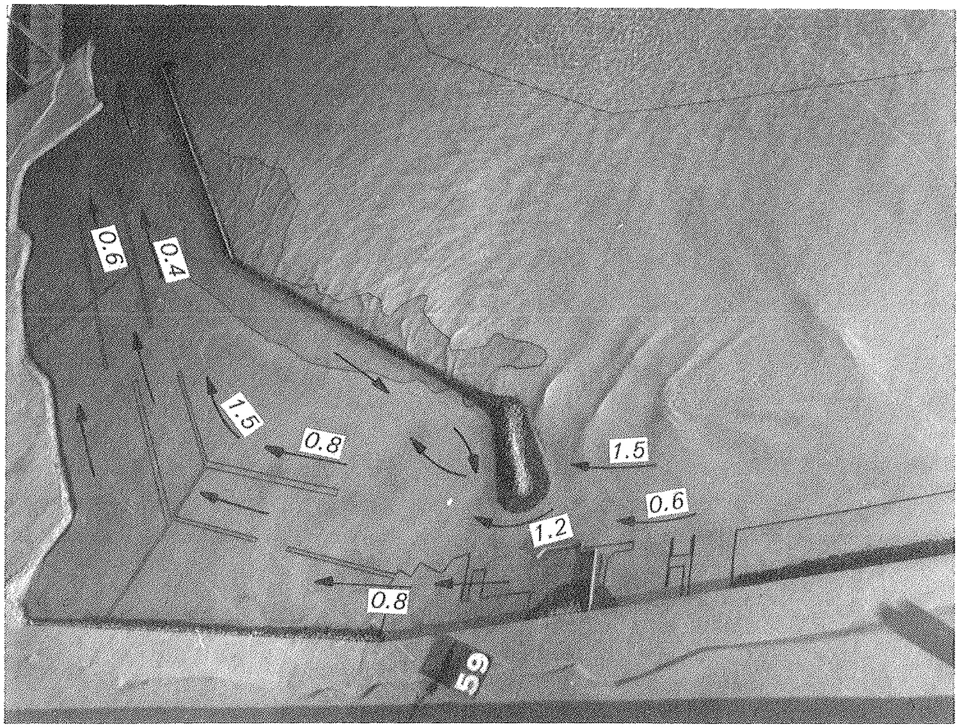


Photo 43. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for Plan 5; 12.5-sec, 8-ft test waves; swl = +5.0 ft (maximum ebb)

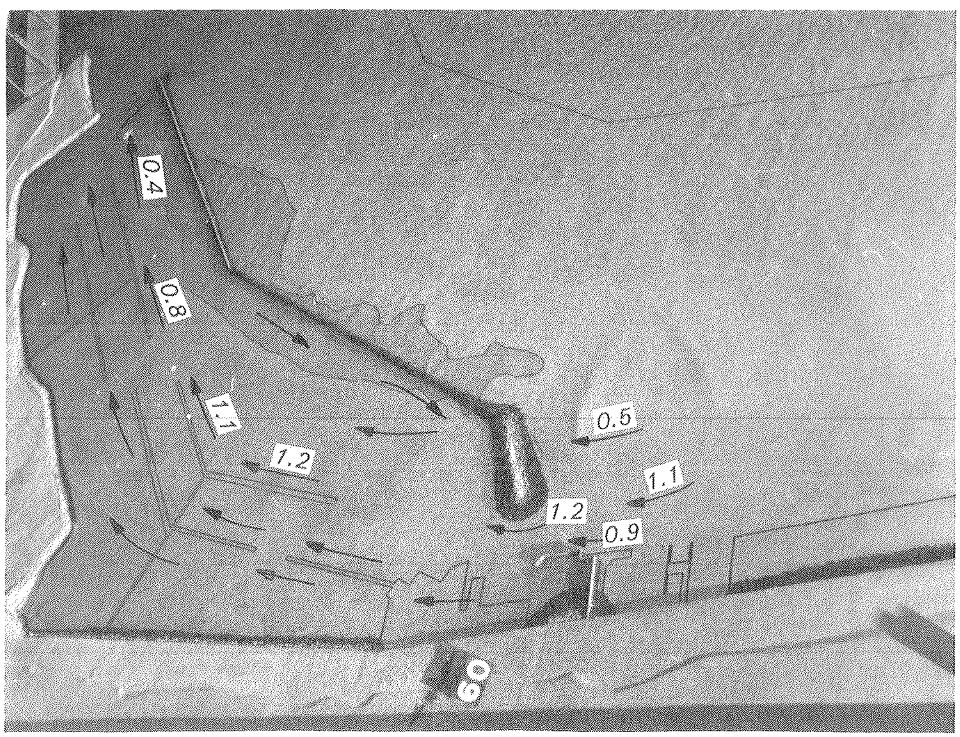


Photo 44. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for Plan 5; 14.3-sec, 3-ft test waves; swl = +5.0 ft (maximum ebb)

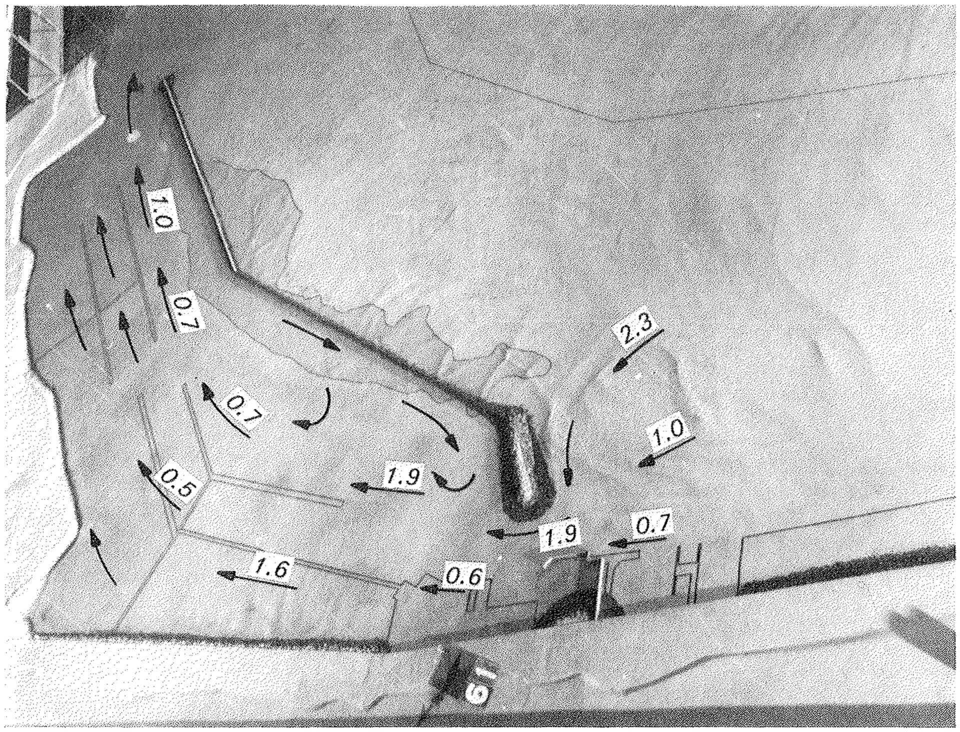


Photo 45. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for Plan 5; 12.5-sec, 8-ft test waves; swl = +5.0 ft (maximum flood)

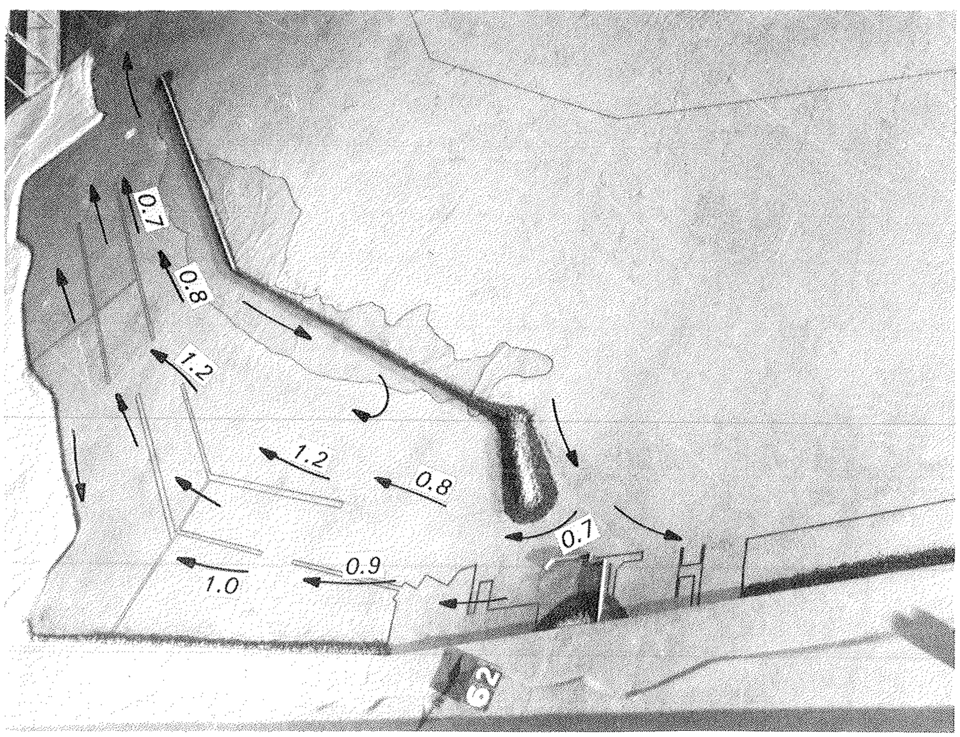


Photo 46. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for Plan 5; 14.3-sec, 3-ft test waves; swl = +5.0 ft (maximum flood)

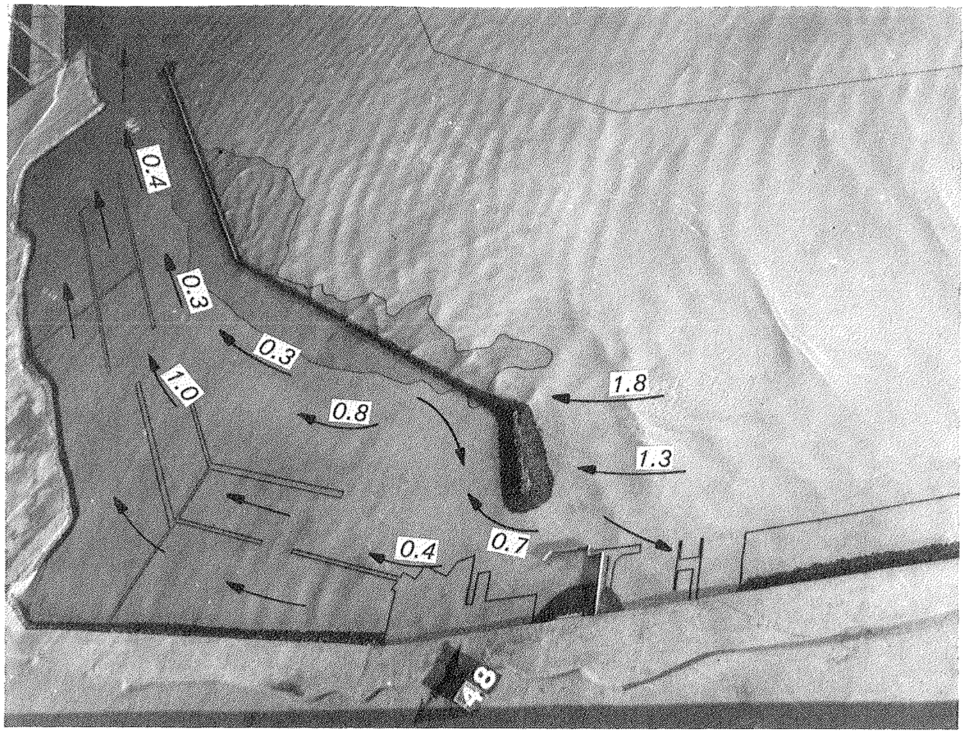


Photo 47. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for Plan 5; 12.5-sec, 8-ft test waves; swl = +8.0 ft

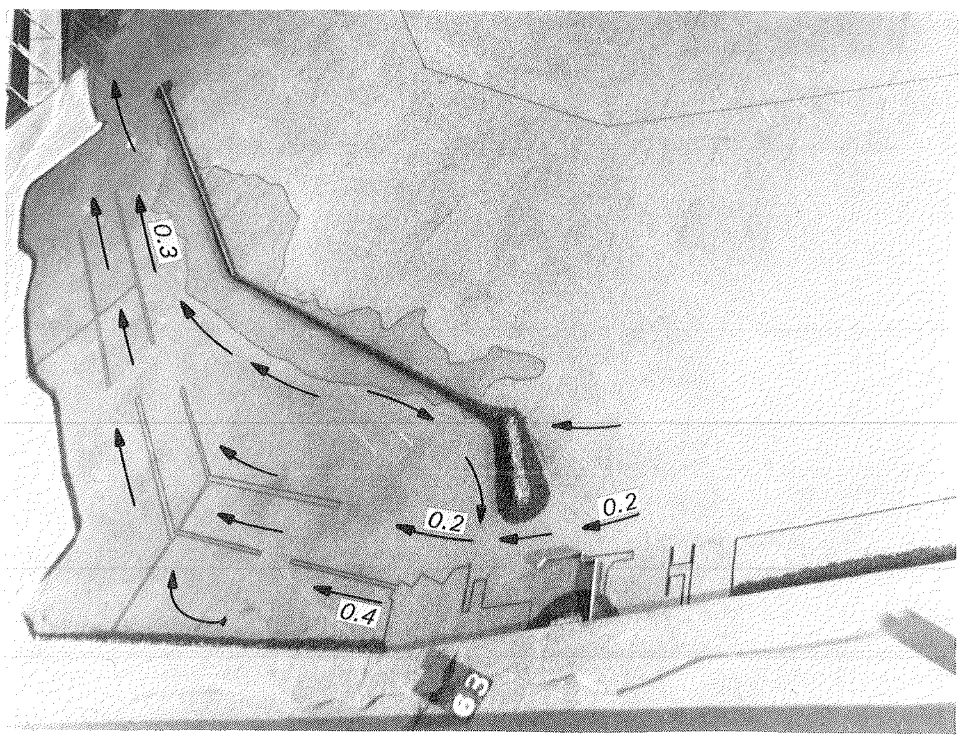


Photo 48. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for Plan 5; 14.3-sec, 3-ft test waves; swl = +8.0 ft

Photo 49. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for Plan 5; 12.5-sec, 8-ft test waves; swl = +11.0 ft

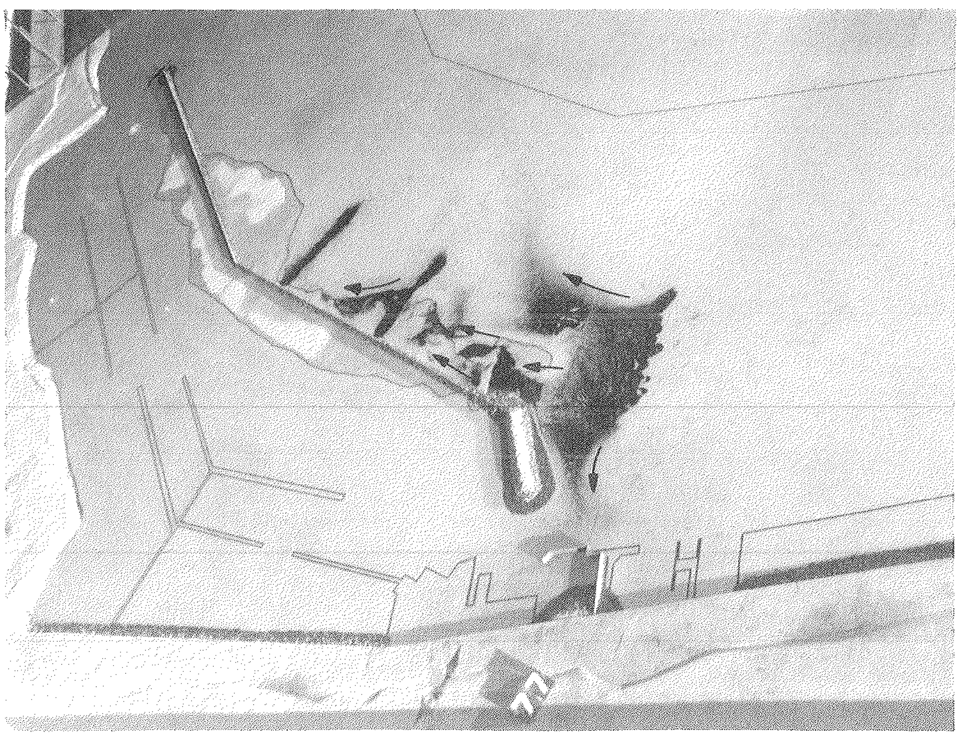


Photo 50. General movement of tracer material and subsequent deposits for Plan 5 for 12.5-sec, 8-ft test waves; swl = 0.0 ft

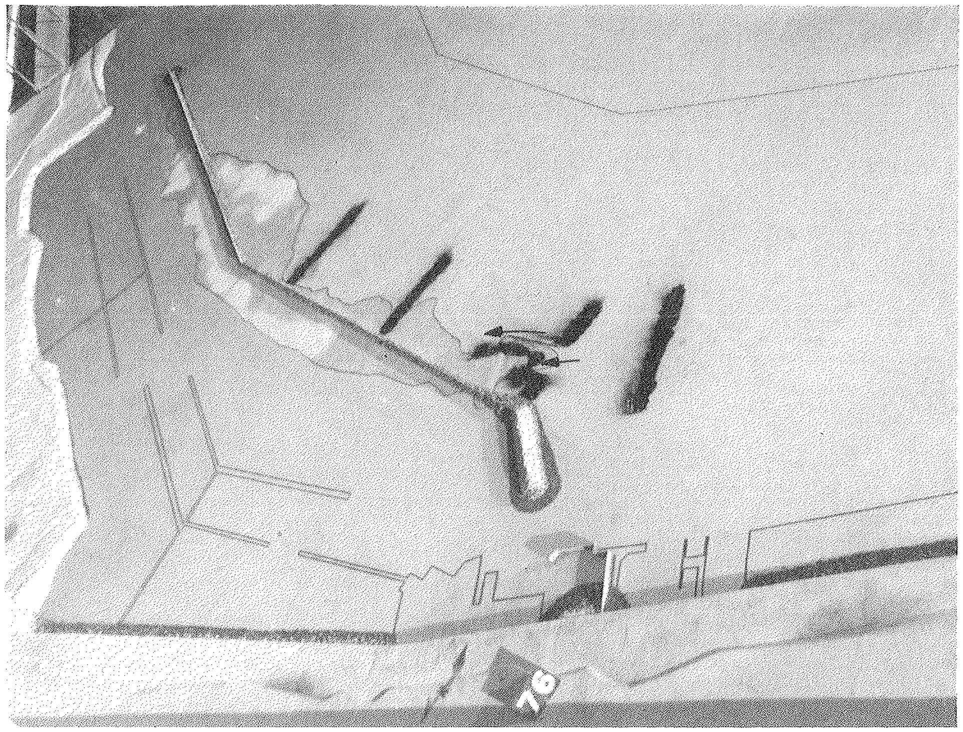


Photo 51. General movement of tracer material and subsequent deposits for Plan 5 for 14.3-sec, 3-ft test waves; swl = 0.0 ft

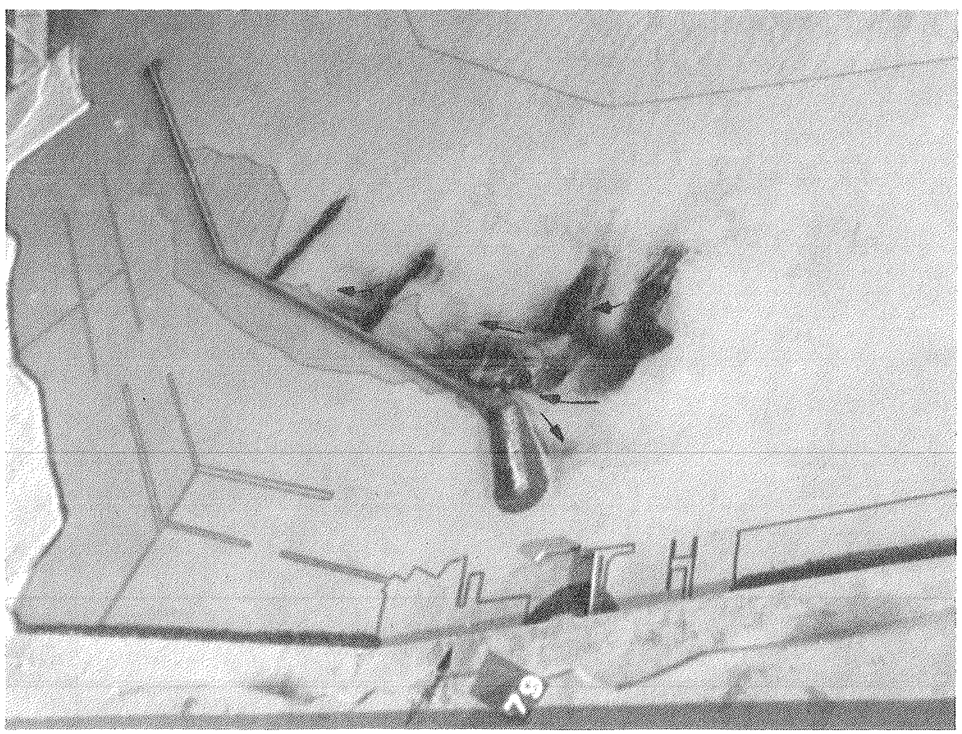


Photo 52. General movement of tracer material and subsequent deposits for Plan 5 for 12.5-sec, 8-ft test waves; swl = +5.0 ft (maximum ebb)

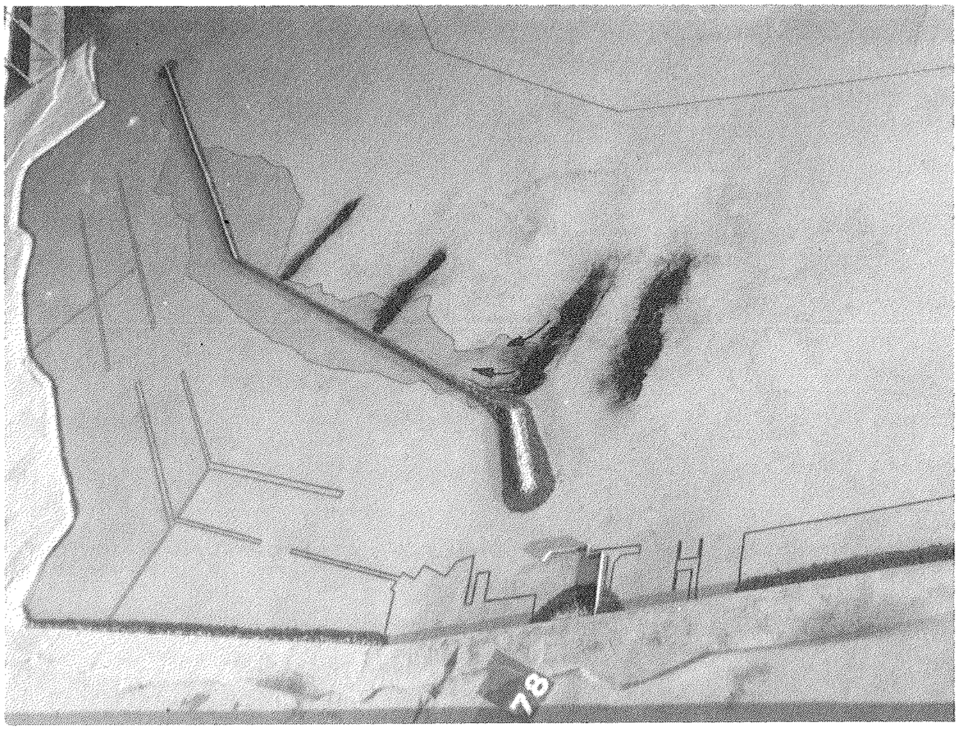


Photo 53. General movement of tracer material and subsequent deposits for Plan 5 for 14.3-sec, 3-ft test waves; swl = +5.0 ft (maximum ebb)

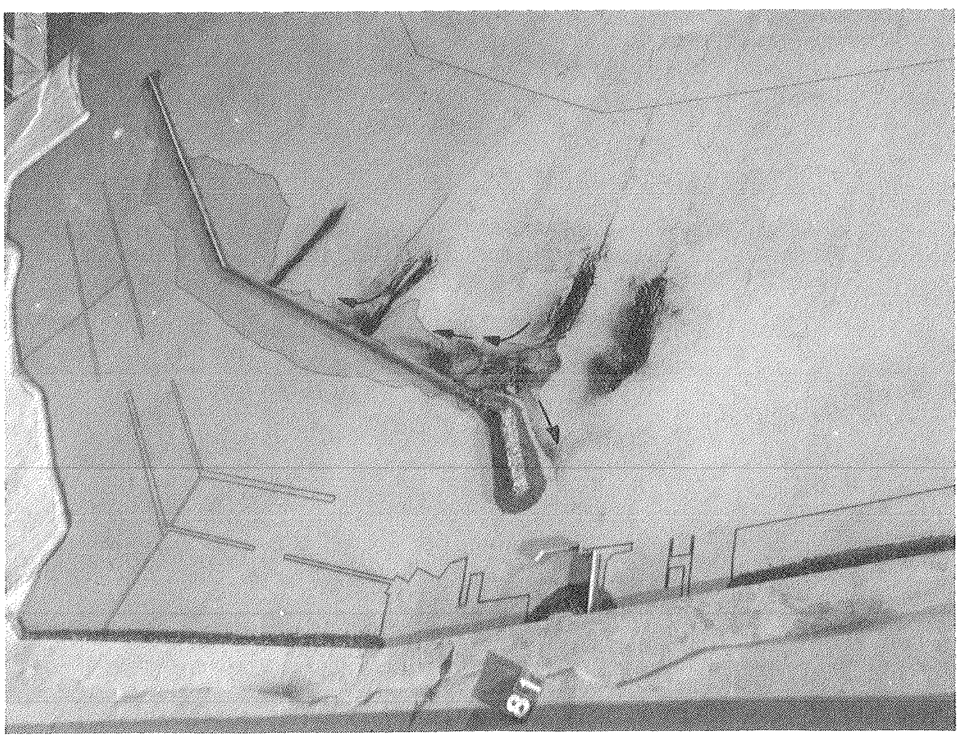


Photo 54. General movement of tracer material and subsequent deposits for Plan 5 for 12.5-sec, 8-ft test waves; swl = +5.0 ft (maximum flood)

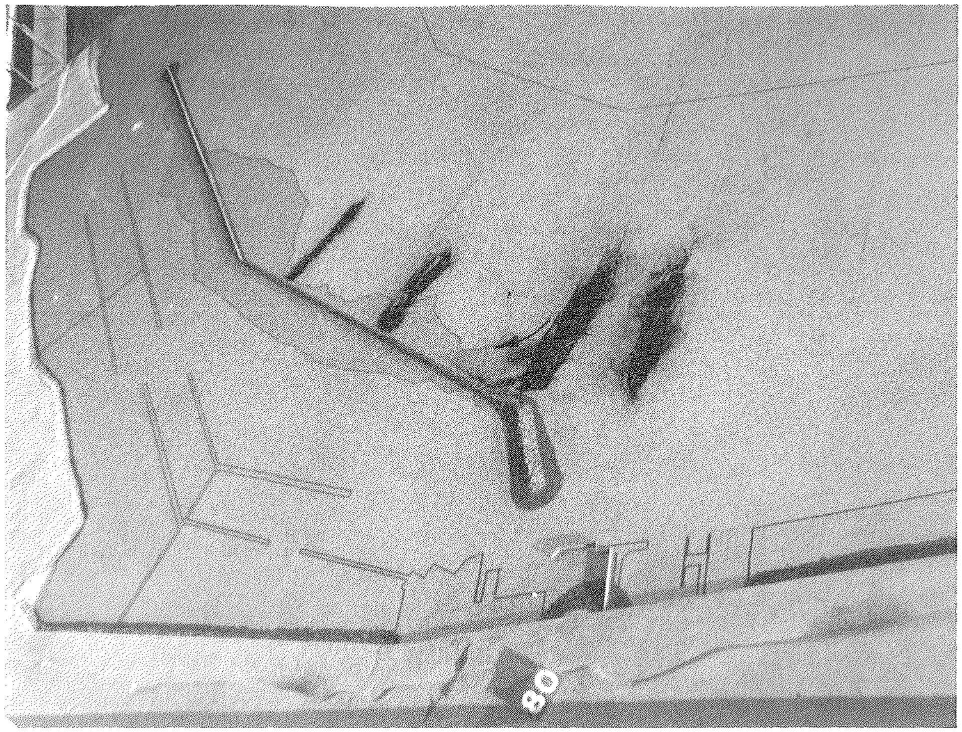


Photo 55. General movement of tracer material and subsequent deposits for Plan 5 for 14.3-sec, 3-ft test waves; swl = +5.0 ft (maximum flood)

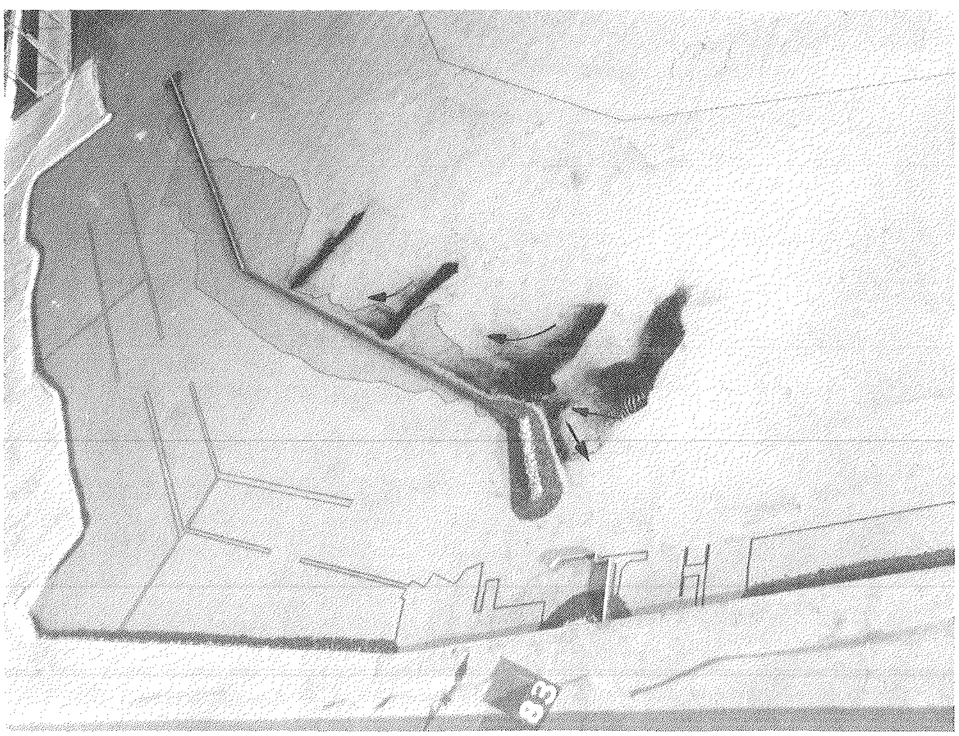


Photo 56. General movement of tracer material and subsequent deposits for Plan 5 for 12.5-sec, 8-ft test waves; swl = +8.0 ft

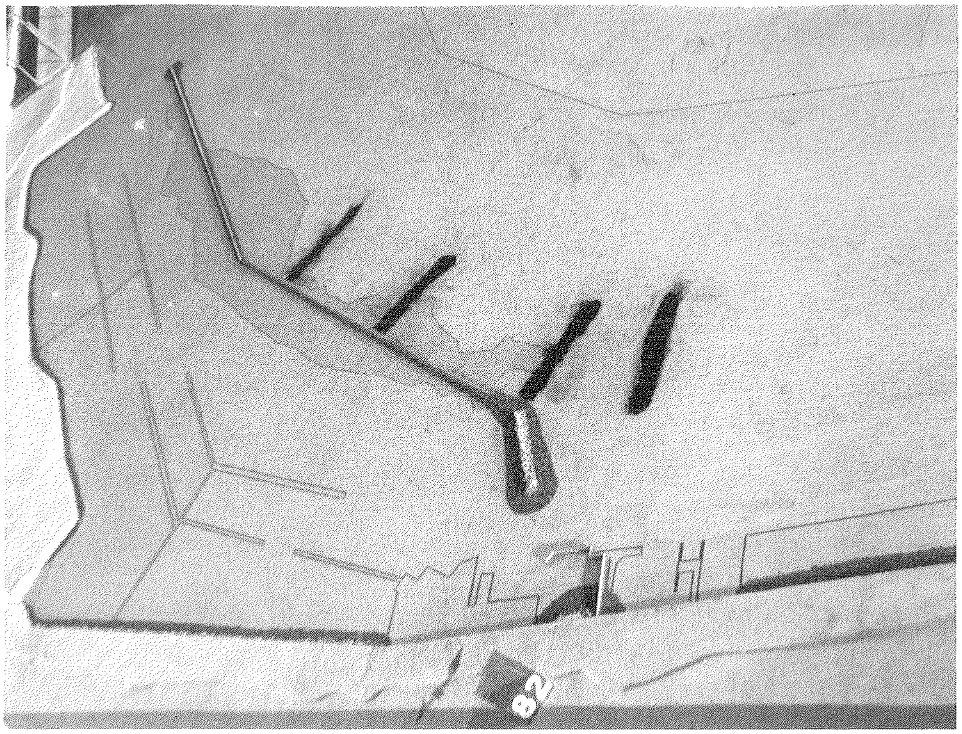


Photo 57. General movement of tracer material and subsequent deposits for Plan 5 for 14.3-sec, 3-ft test waves; swl = +8.0 ft

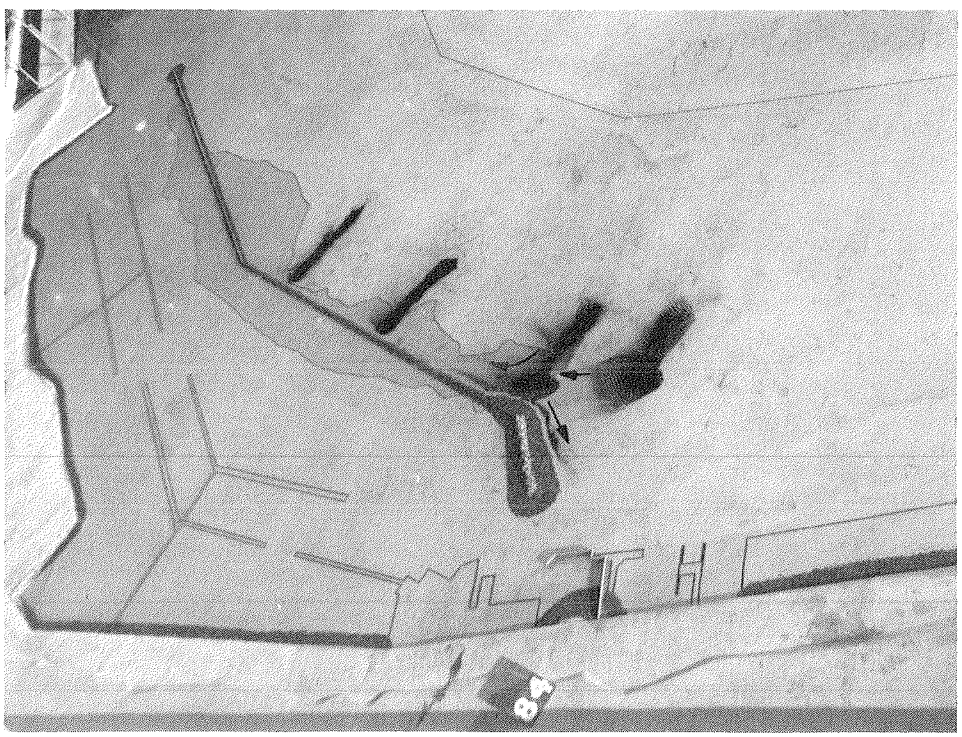
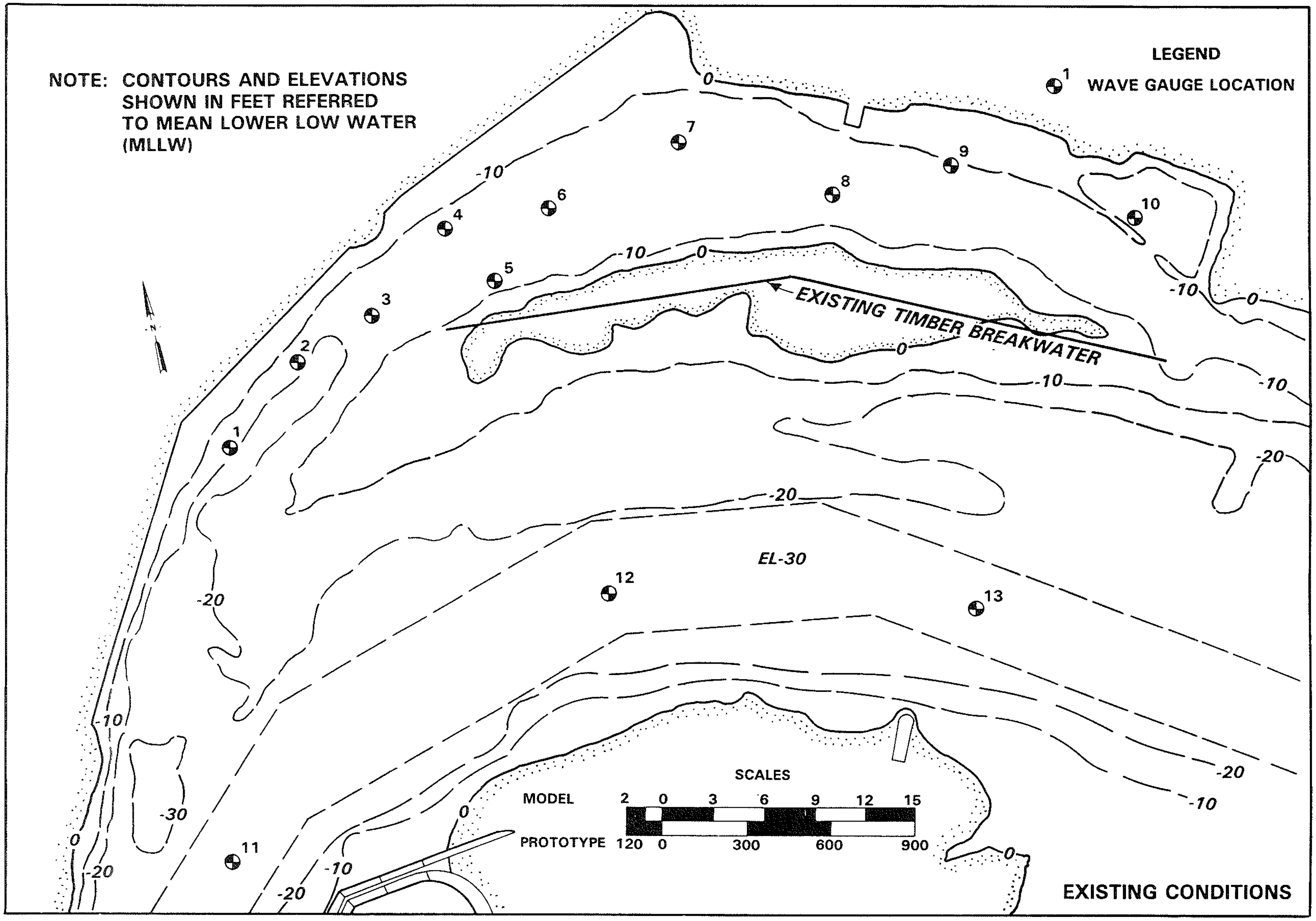
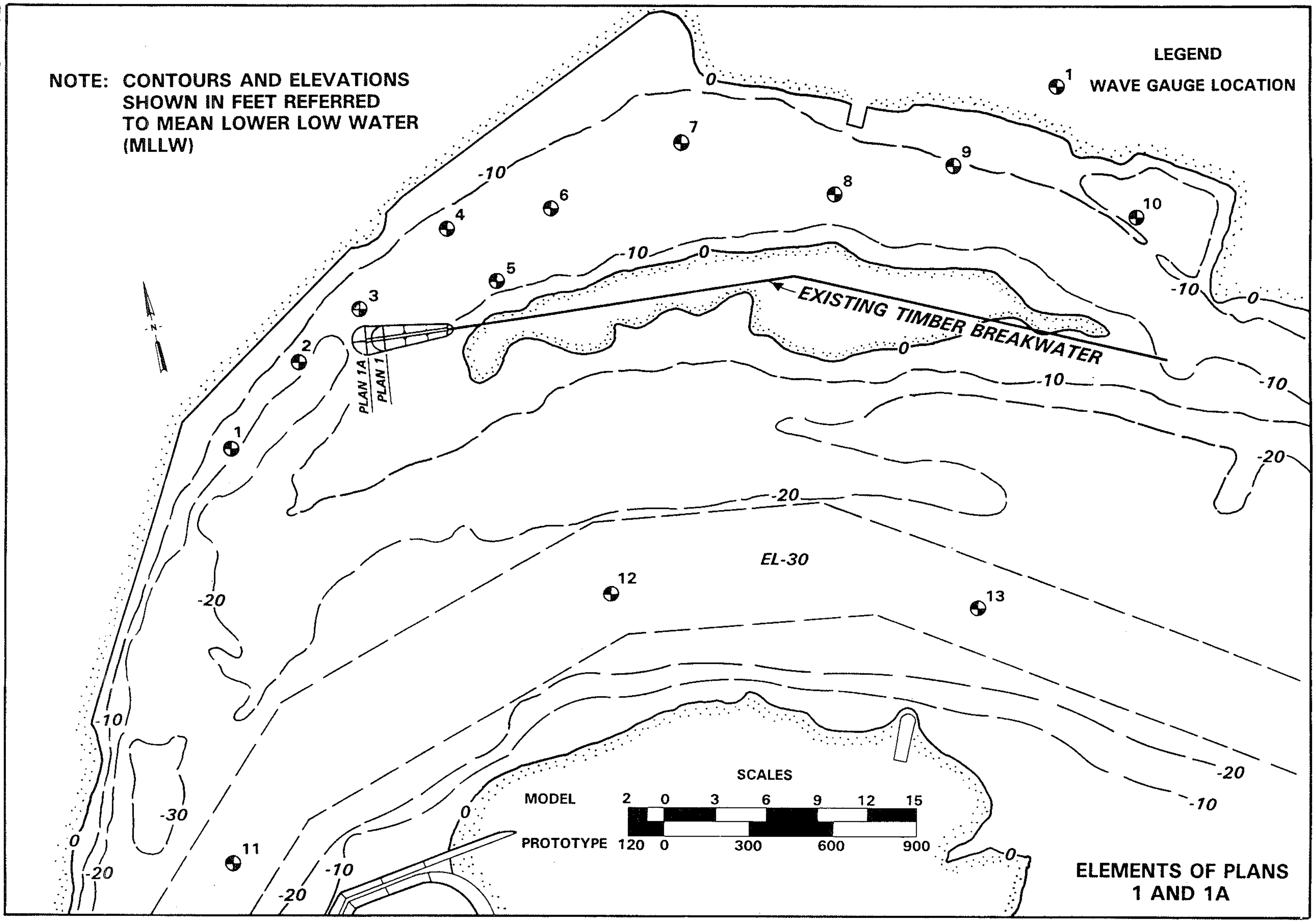


Photo 58. General movement of tracer material and subsequent deposits for Plan 5 for 12.5-sec, 8-ft test waves; swl = +11.0 ft

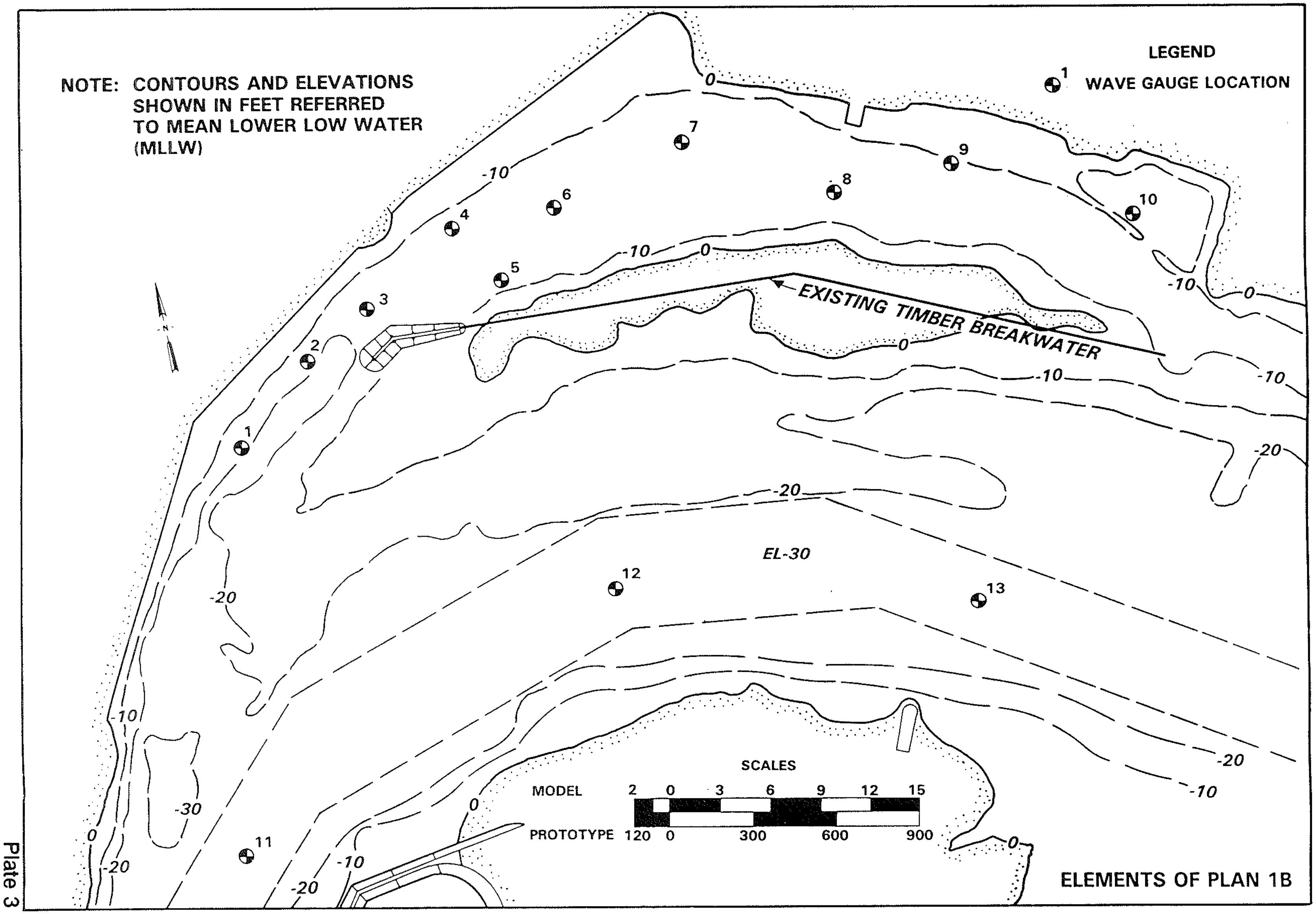


NOTE: CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO MEAN LOWER LOW WATER (MLLW)
LEGEND
1 WAVE GAUGE LOCATION
EXISTING TIMBER BREAKWATER
EL-30
0
-10
-20
-30
1
2
3
4
5
6
7
8
9
10
11
12
13
SCALES
MODEL 2 0 3 6 9 12 15
PROTOTYPE 120 0 300 600 900
EXISTING CONDITIONS

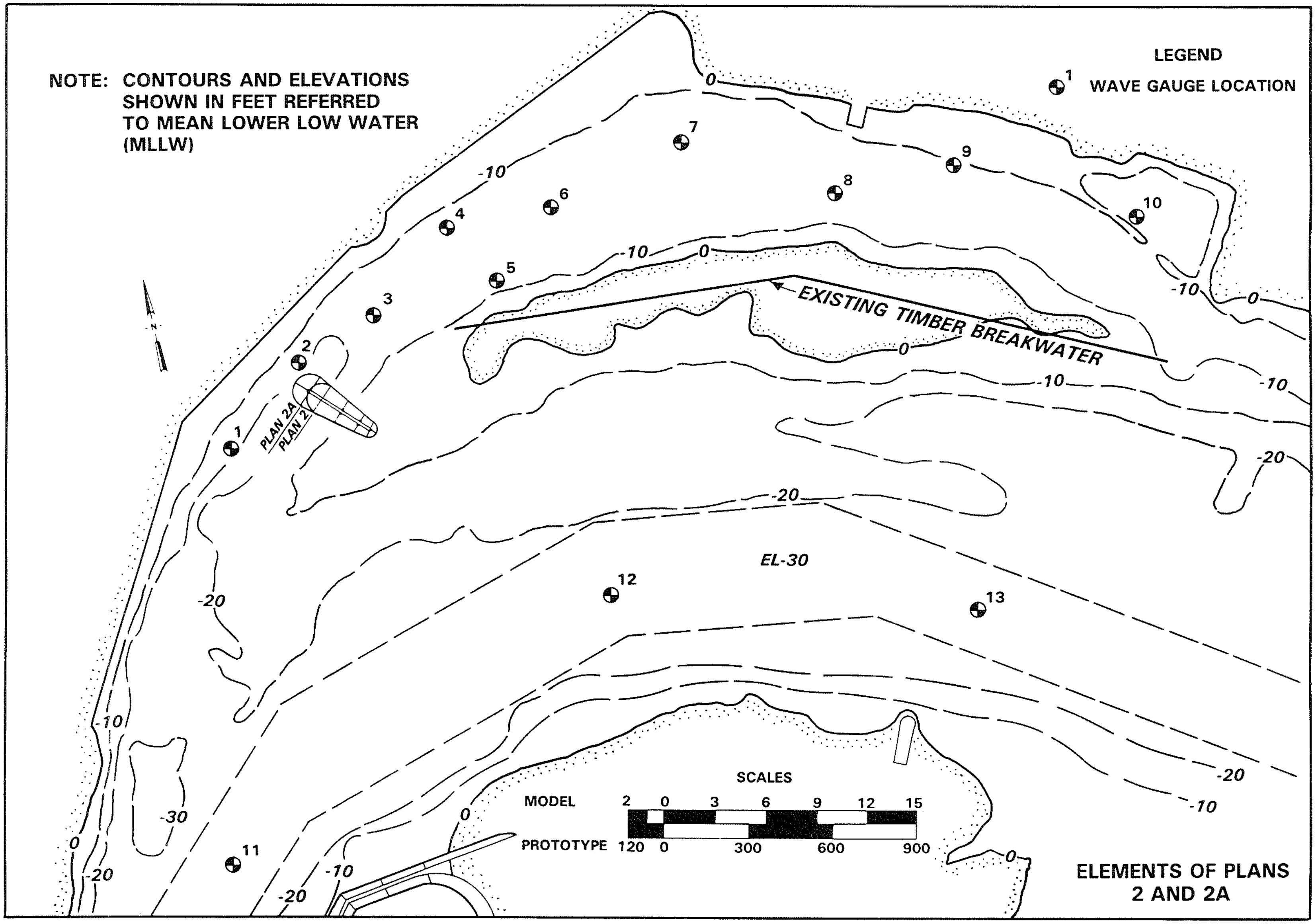
Plate 2

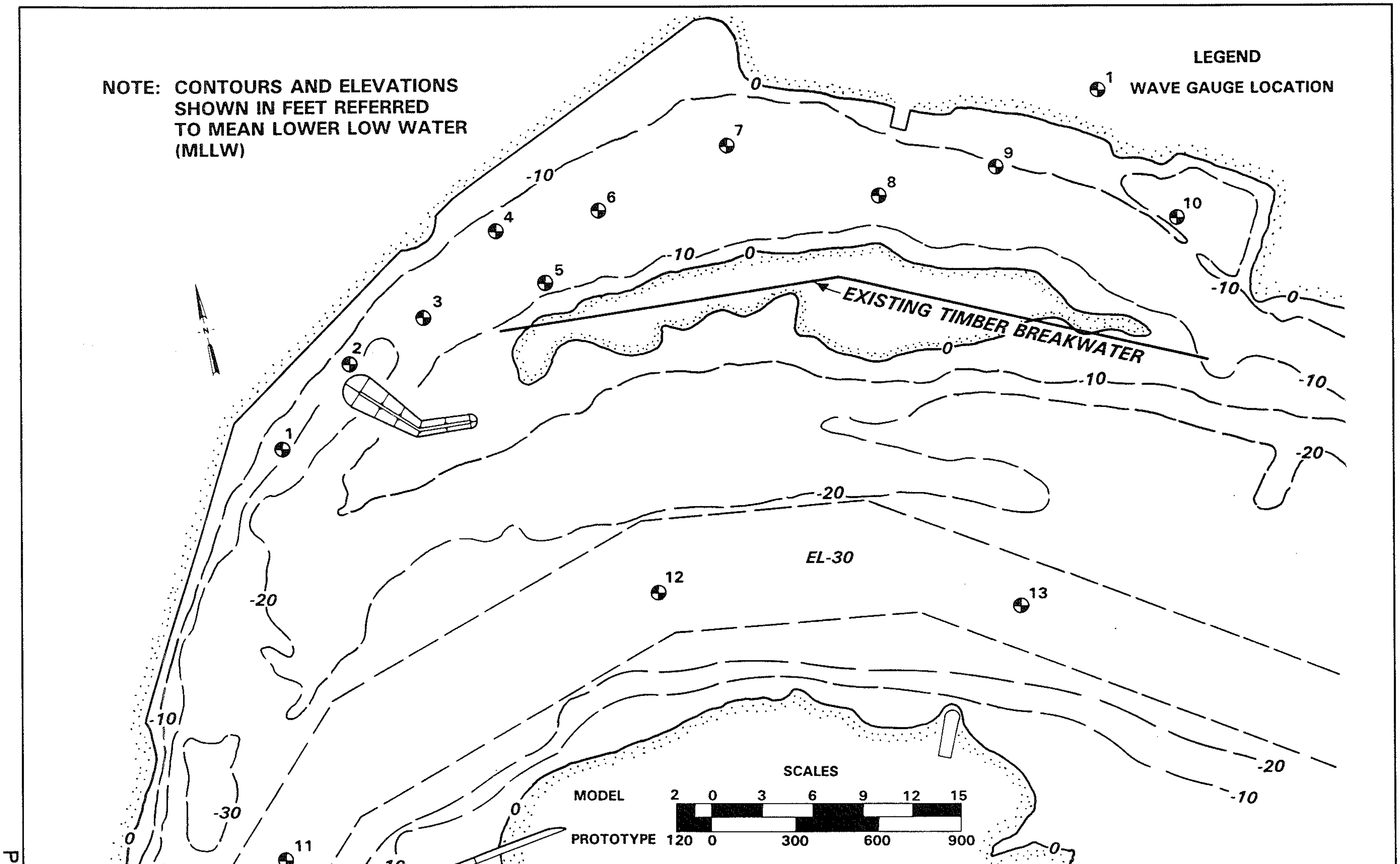


ELEMENTS OF PLANS 1 AND 1A

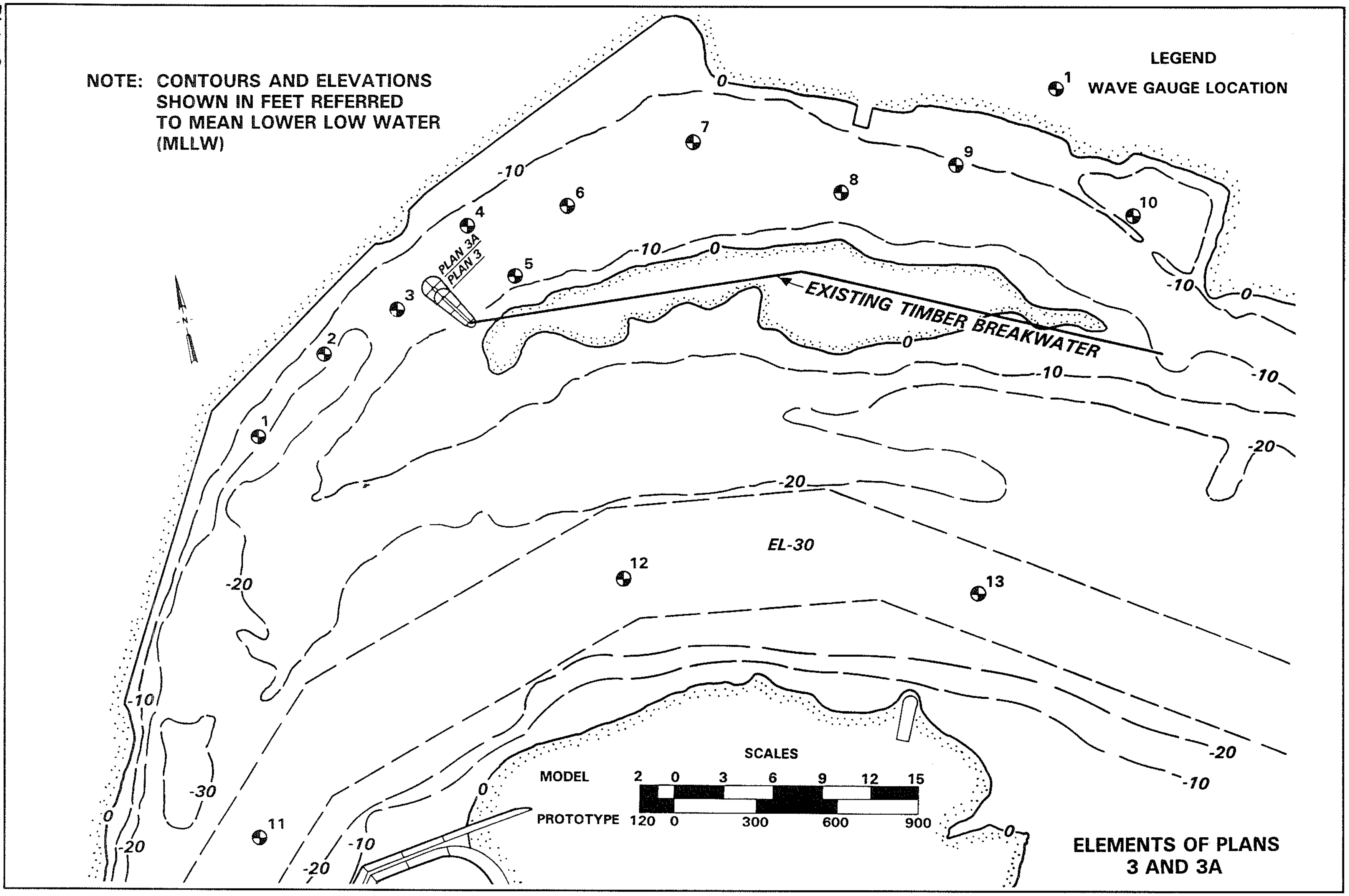


ELEMENTS OF PLAN 1B

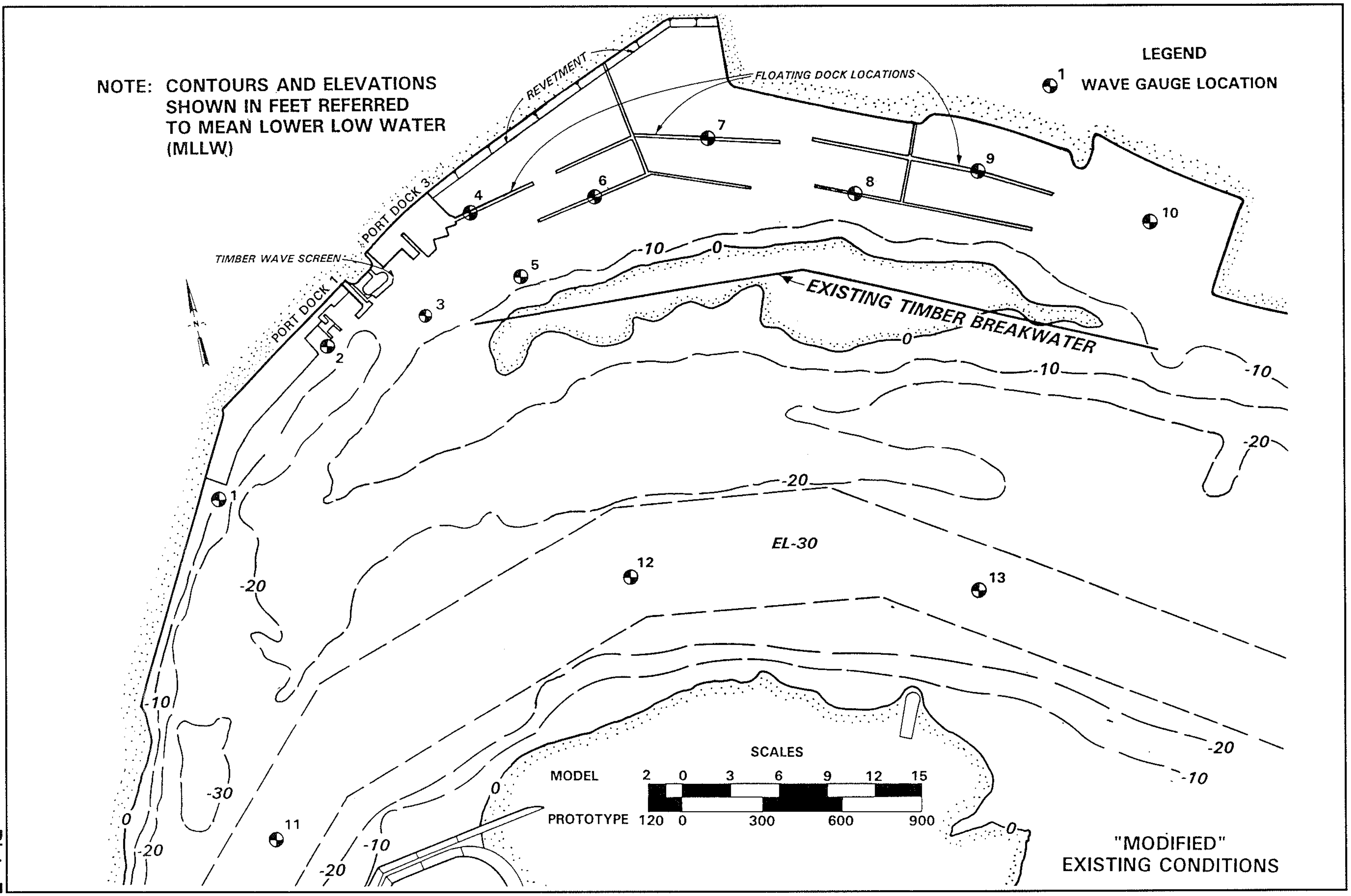

NOTE: CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO MEAN LOWER LOW WATER (MLLW)
LEGEND
1 WAVE GAUGE LOCATION
EXISTING TIMBER BREAKWATER
PLAN 2A
PLAN 2
EL-30
0
-10
-20
-30
1
2
3
4
5
6
7
8
9
10
11
12
13
SCALES
MODEL 2 0 3 6 9 12 15
PROTOTYPE 120 0 300 600 900
ELEMENTS OF PLANS 2 AND 2A



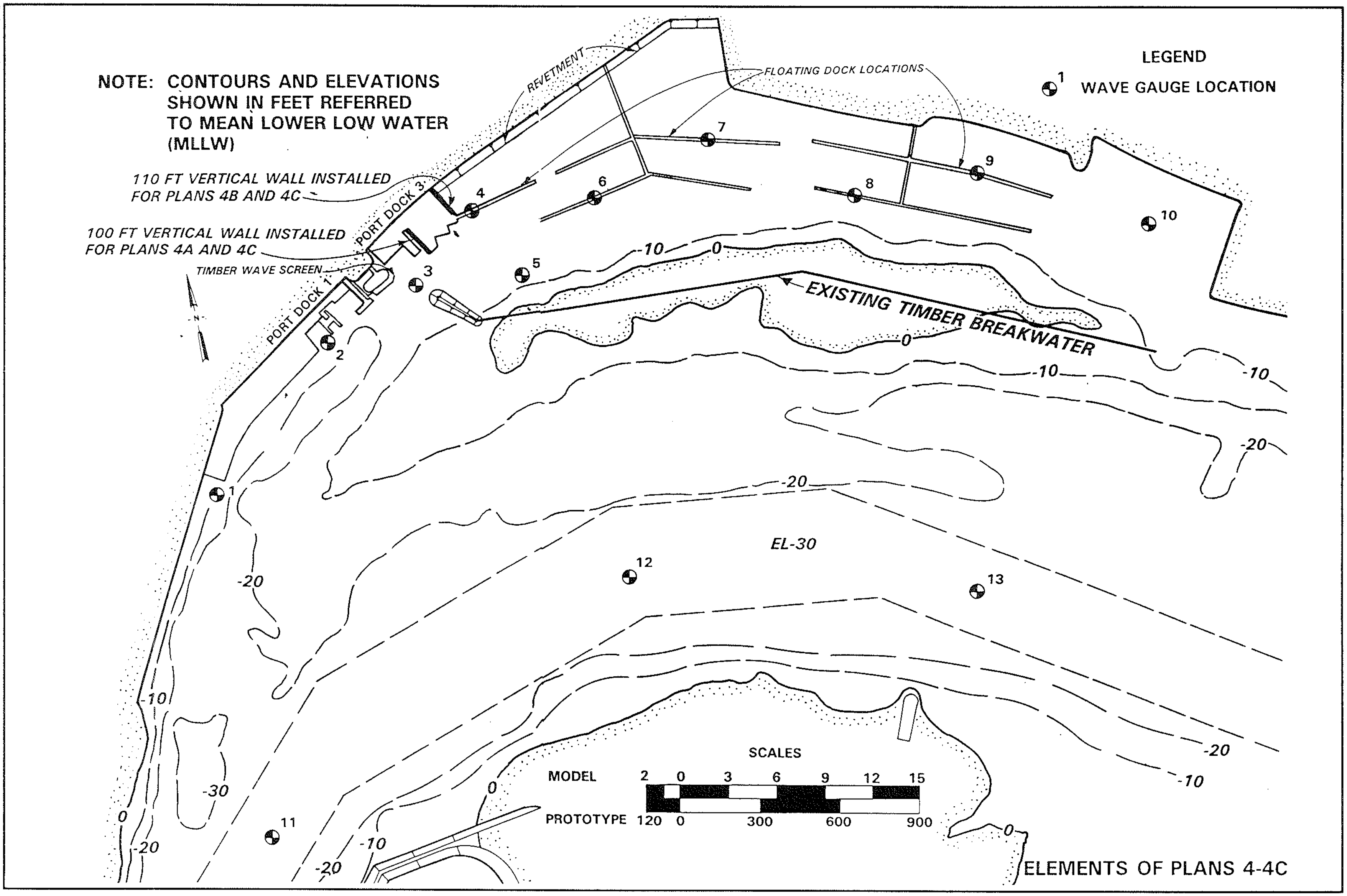
ELEMENTS OF PLAN 2B



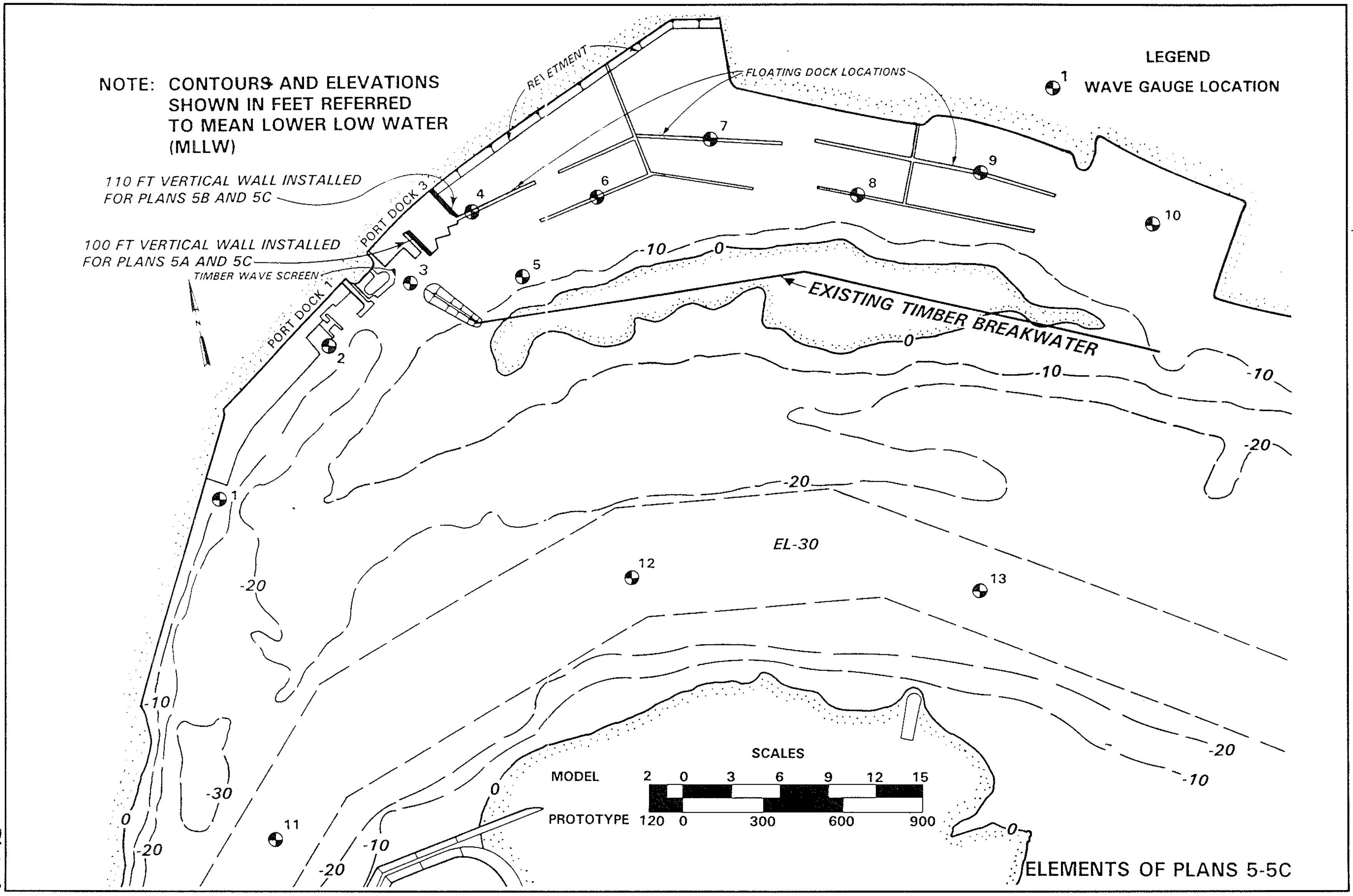
ELEMENTS OF PLANS
3 AND 3A



"MODIFIED"
EXISTING CONDITIONS

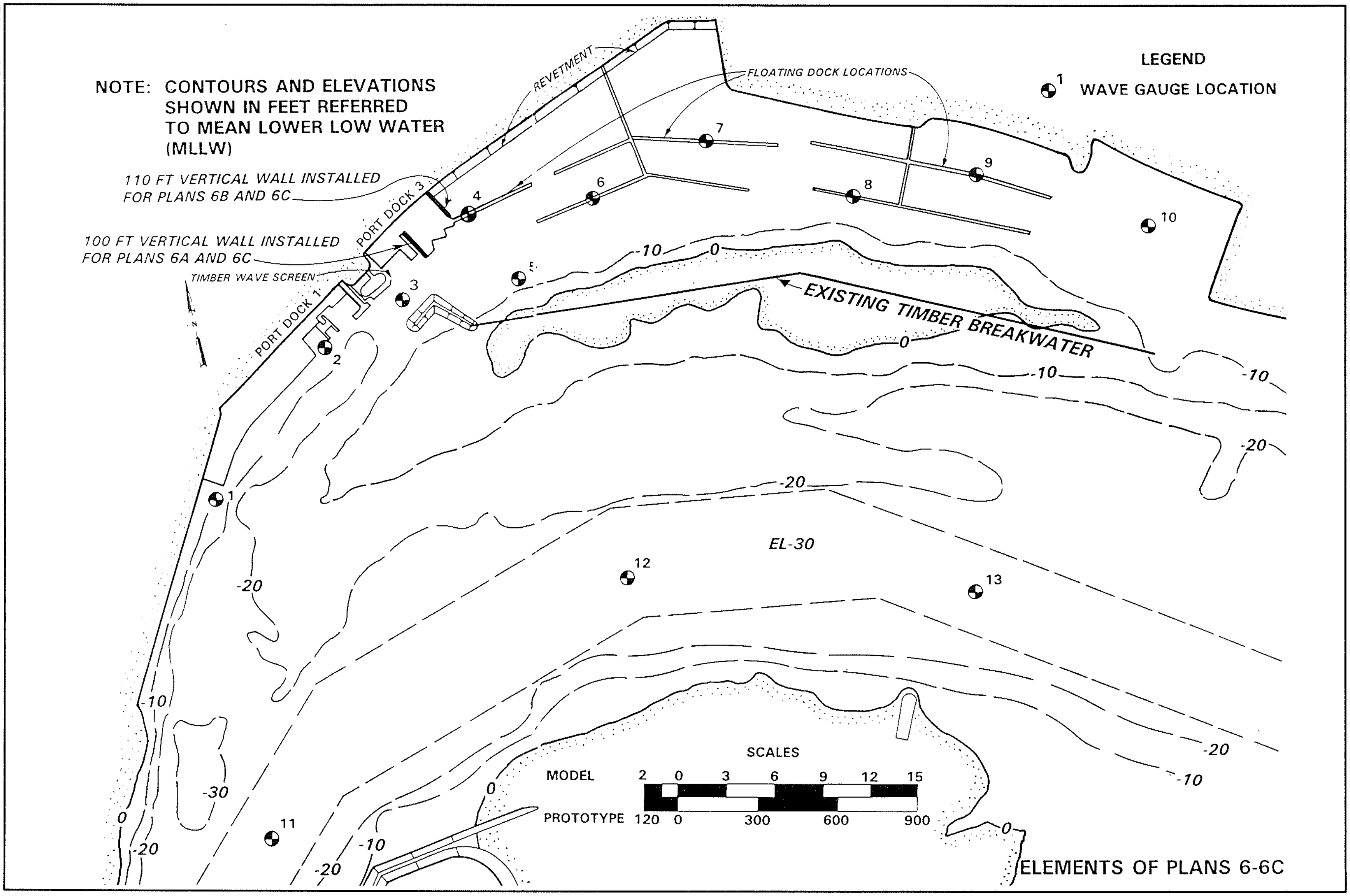


ELEMENTS OF PLANS 4-4C

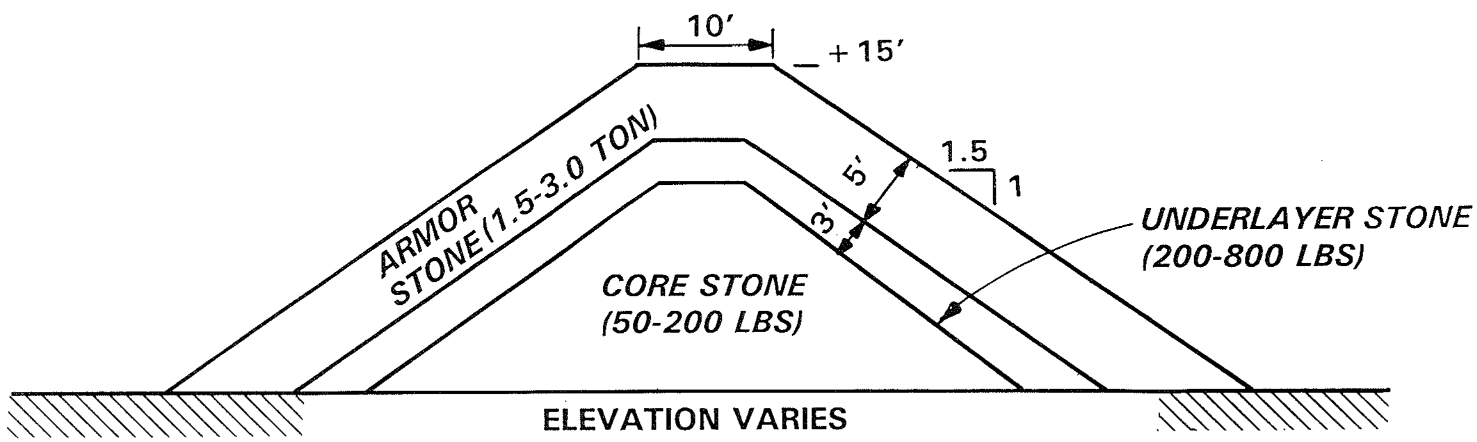


ELEMENTS OF PLANS 5-5C

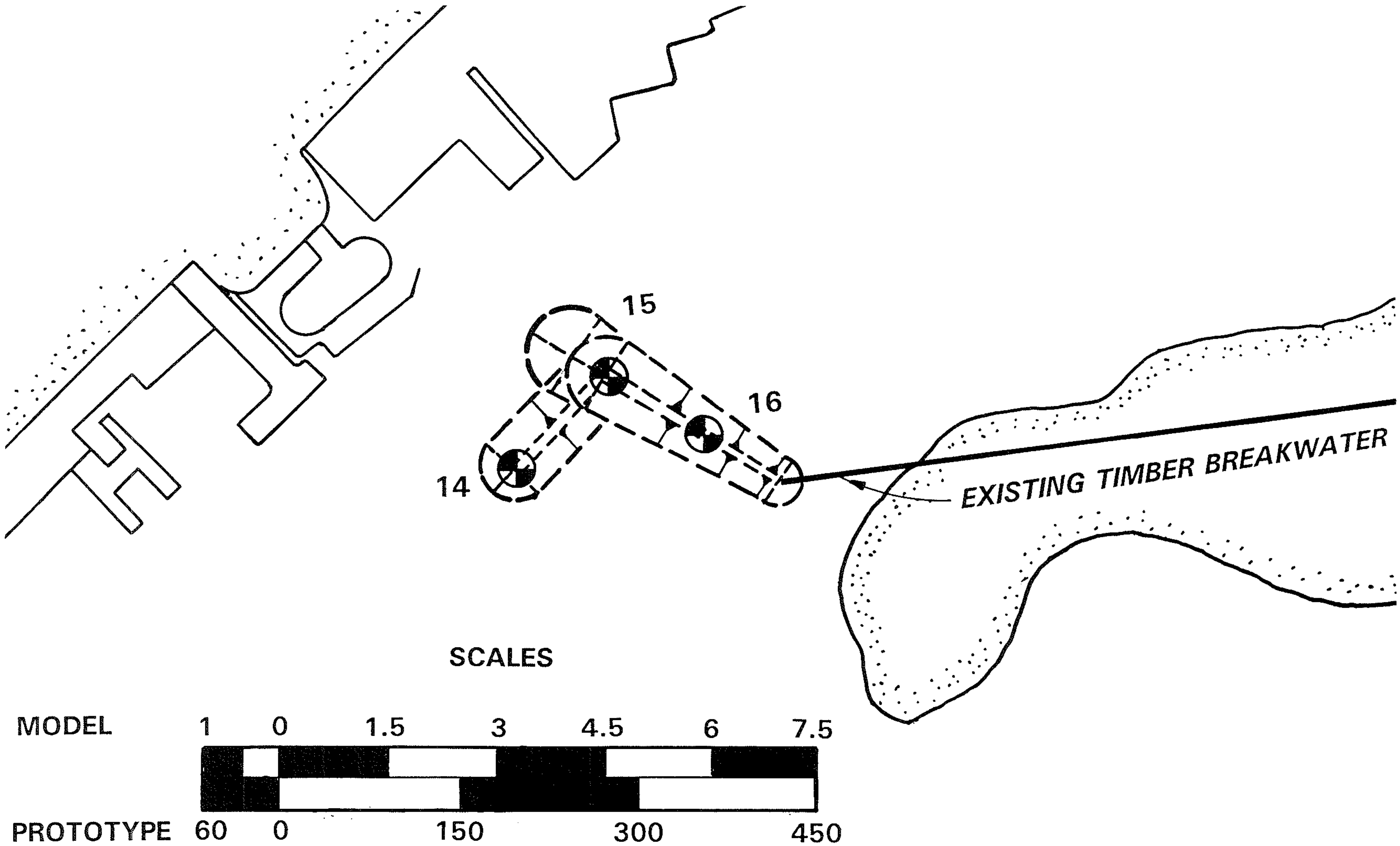
Plate 9



ELEMENTS OF PLANS 6-6C



RUBBLE-MOUND BREAKWATER EXTENSION CROSS-SECTION



DESIGN WAVE
GAUGE LOCATIONS

REPORT DOCUMENTATION PAGE

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13. ABSTRACT ***(Maximum 200 words)***

A 1:60 scale (undistorted) three-dimensional hydraulic model was used to investigate the design of proposed breakwater modifications at Newport North Marina, Yaquina Bay, OR, with respect to wave and current conditions in the harbor and sediment patterns at the site. The model reproduced the existing marina and a portion of the Yaquina River from west of the U.S. Highway 101 bridge upstream. Proposed improvements consisted of breakwater modifications at the marina entrance. A 12.2-m-long (40-ft-long) unidirectional, spectral wave generator, a water circulation system, an automated data acquisition and control system, and a crushed coal tracer material were used in model operation. Test results led to the following conclusions:

a. Existing conditions are characterized by rough and turbulent wave conditions during periods of storm wave attack. Wave heights in excess of 0.9 m (3 ft) occurred in the marina mooring areas.

b. Preliminary tests for the three originally proposed design alternatives (Plans 1-3) indicated that none of the test plans would meet the original 0.3-m (1-ft) criterion in the marina mooring area.

(Continued)

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c. Of the three originally proposed design alternatives, preliminary tests indicated that the angled rubble-mound breakwater extension concept (Plan 3 series) was most effective considering wave protection provided in the mooring area versus structure length. The detached breakwater concept (Plan 2 series) proved to be the least effective.

d. Preliminary testing of the expeditiously constructed breakwater plans proved valuable in the selection of the structure alignments and locations used for the final test series.

e. Test results for "modified" existing conditions (revetment, etc., installed) revealed rough and turbulent wave conditions in the marina with wave heights in excess of 0.9 m (3 ft) during storm wave conditions. Generally, however, the revetment slightly improved overall wave conditions in the marina.

f. Tests conducted in the model, in which overtopping of the existing timber breakwater was prevented, revealed that wave overtopping is not a significant problem with respect to excessive wave conditions in newport North Marina.

g. Results of wave height tests for the final 12 test plans revealed that only Plan 6C (77.7-m-long (255-ft-long) rubble-mound breakwater extension and cumulative 67.1-m (220-ft) length of vertical structures) met the originally established 0.3-m (1-ft) wave height criterion in the marina mooring areas.

h. After an assessment of economic benefits, Plan 5 (54.9-m-long (180-ft-long) rubble-mound breakwater extension) was selected as the most cost-effective plan considering wave protection provided the marina mooring areas versus construction costs.

i. Construction of the Plan 5 rubble-mound breakwater extension will have minimal impact on circulation patterns and magnitudes in the marina.

j. Construction of the Plan 5 rubble-mound breakwater extension will have no adverse impacts on sedimentation in the marina entrance.

k. Construction of the Plan 5 rubble-mound breakwater extension will have no adverse impacts on wave conditions along the existing docks and wharves west of the existing entrance.