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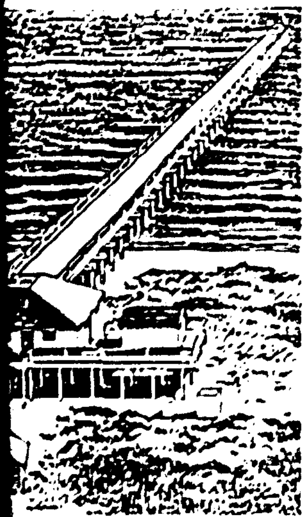

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VENTURA HARBOR, CALIFORNIA, DESIGN FOR WAVE AND SHOALING PROTECTION

Coastal Model Investigation

by

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A 1:75 scale, three-dimensional hydraulic model was used to investigate the design of proposed harbor structures and channel modifications at Ventura Harbor, California, with respect to wave and shoaling conditions in the harbor entrance. The model reproduced approximately 9,400 ft of the California shoreline and included portions of the existing harbor and offshore bathymetry in the Pacific Ocean to a depth of -40 ft mean lower low water (mllw). Improvement plans consisted of a seaward extension of the detached breakwater, the installation of spur groins on the north jetty, construction of a new groin south of the south jetty, and modifications to the entrance channel. An 80-ft-long unidirectional, spectral wave generator, an automated data acquisition system, and a crushed coal tracer material were used in model operation. It was concluded from test results that:

a. Existing conditions are characterized by excessive wave conditions in the harbor entrance. Waves heights up to 7 ft will occur in the entrance for

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9-ft incident wave conditions. During severe storm periods wave heights in excess of 11 ft will occur in the entrance.

b. For existing conditions, sediment from the north shoreline will move southerly where some will deposit in the deposition basin and some will move around the head of the north jetty and deposit in the harbor entrance channel.

c. For existing conditions, sediment from the south shoreline will move northerly along the shoreline and south jetty and result in deposits in the harbor entrance channel.

d. The proposed 300-ft-long detached breakwater extension of Plan 1 will result in wave heights in the entrance within the desired wave height criterion. Incident waves of 9 ft seaward of the harbor will be reduced by approximately 2 ft (from 7 to 5 ft in the entrance), and severe storm waves will be reduced by approximately 3 ft (from 11 to 8 ft in the entrance).

e. Of the spur groin plans tested on the north jetty (Plans 2 through 8 and 11), the 325-ft-long groin of Plan 7 and the 300-ft-long groin of Plan 11 appeared to be optimum with respect to reducing shoaling in the entrance channel. Relative to other spur groin plans tested, Plan 7 and Plan 11 groins will result in the minimum entrance shoaling for sediment approaching from the north.

f. Installation of the south groin (Plan 9) will reduce shoaling in the harbor entrance channel. Most material moving northerly around the head of the groin for southerly test waves will be transported back to the south when predominant westerly wave conditions occur.

PREFACE

A request for a model investigation of wave and shoaling conditions at Ventura Harbor, California, was initiated by the US Army Engineer District, Los Angeles (SPL), in a letter to the US Army Engineer Division, South Pacific. Authorization for the US Army Engineer Waterways Experiment Station (WES) to perform the study was subsequently granted by Headquarters, US Army Corps of Engineers. Funds for model testing were authorized by SPL on 15 November 1989 and 10 January 1990.

Model tests were conducted at WES during the period June through October 1990 by personnel of the Wave Processes Branch (WPB) of the Wave Dynamics Division (WDD), Coastal Engineering Research Center (CERC), WES, under the general supervision of Dr. James R. Houston and Mr. Charles C. Calhoun, Jr., Chief and Assistant Chief of CERC, respectively; and under the direct guidance of Messrs. C. E. Chatham, Jr., Chief of WDD, and Dennis G. Markle, Chief of WPB. Tests were conducted by Messrs. Larry R. Tolliver and Hugh F. Acuff, Civil Engineering Technicians, and Mr. William G. Henderson, Computer Assistant, under the supervision of Mr. Robert R. Bottin, Jr., Project Manager, WDD. This report was prepared by Mr. Bottin, typed by Ms. Debbie S. Fulcher, WPB, and edited by Ms. Lee T. Byrne, Information Technology Laboratory, WES. Prior to the model investigation, Messrs. Bottin and Markle met with representatives of SPL and visited Ventura Harbor to inspect the prototype site and attend a general design conference. During the course of the investigation, liaison was maintained by means of conferences, telephone communications, and monthly progress reports. Mr. Chris Andrassy, SPL, visited WES and was present during most model testing. Other visitors to WES, who observed model operation and/or participated in conferences, during the course of the study were:

Mr. Art Shak	US Army Engineer District, Los Angeles
Mr. Richard Parsons	General Manager, Ventura Port District
Dr. Rich Kent	Consultant to Ventura Port District
Mr. Rick Raives	City of Ventura, Senior Engineer
Mr. Jon Moore	Consultant to City of Ventura
Dr. Scott Jenkins	Surfrider Foundation
Mr. Bill Crew	Ventura Port Commissioner

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

CONTENTS

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

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

Multiply	By	To Obtain
acres	4046.873	square metres
cubic yards	0.7646	cubic metres
degrees (angle)	0.01745329	radians
feet	0.3048	metres
miles (US statute)	1.609347	kilometres
pounds (mass)	0.4535924	kilograms
square feet	0.09290304	square metres
square miles (US statute)	2.589998	square kilometres
tons (2,000 lb mass)	907.1847	kilograms

VENTURA HARBOR, CALIFORNIA, DESIGN FOR WAVE AND SHOALING PROTECTION

Coastal Model Investigation

PART I: INTRODUCTION

The Prototype

1. Ventura Harbor is located on the California coast approximately 55 miles* northwest of Los Angeles (Figure 1). It lies within the city limits of San Buena Ventura, Ventura County. The harbor is entirely man-made and consists of three mooring basins and extensive land area totaling 275 acres. Of this amount, about 125 acres are usable water area. The harbor setting provides a water-oriented commercial and recreational focal point within the county. The facilities also attract frequent transient users from adjacent counties and tourist-related traffic from more distant locations.

2. The water area includes commercial, recreational, and sport fishing boat activity. The harbor is located close to excellent sport and commercial fishing areas. The harbor is also near offshore oil reserves in Santa Barbara Channel and serves as home port to an oil platform support fleet. The combined onshore and water-related businesses attract over 2 million visitors annually. The transient and regular traffic to and from the harbor results in healthy commerce within the facility and supports other services and industry within the city of San Buena Ventura and surrounding communities.

3. There are 1,600 berthing facilities in the harbor that are available to commercial vessels and the general public. Ventura Keys, a privately constructed residential marina adjoining the harbor, has approximately 300 private boat slips. Harbor facilities include three boat repair facilities, two fuel docks, three cranes for off-loading commercial fishing vessels, three wholesale fish buying stations, a public launch ramp facility, a dry boat storage, and numerous retail businesses and hotels. The estimated tonnage of cargo handled at Ventura Harbor in 1986 was 25,150 tons (US Army Engineer District (USAED), Los Angeles 1988a).

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

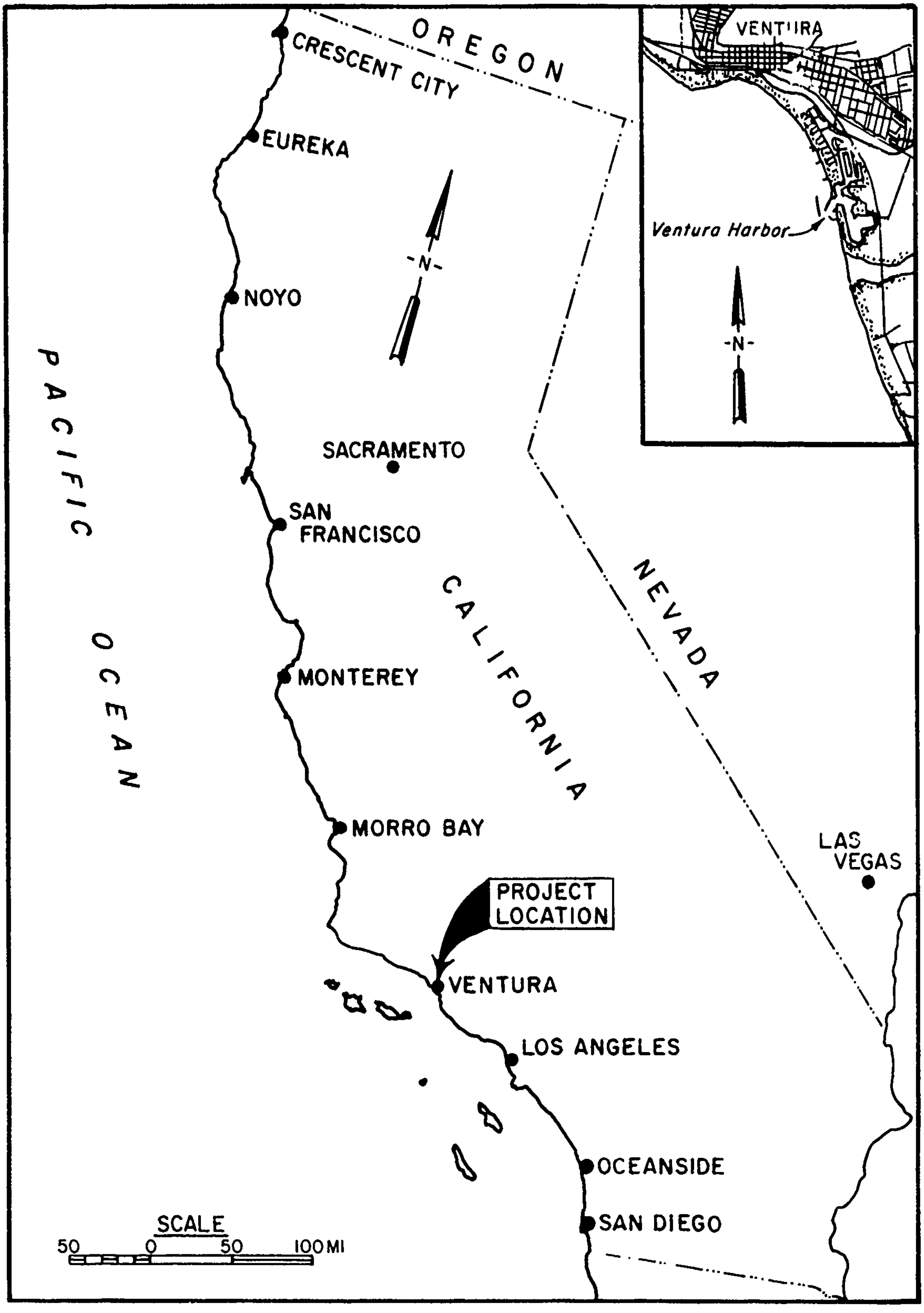

Figure 1. Project location

4. Ventura Harbor was originally developed by the Ventura Port District in 1963; however, in 1968 the Federal government adopted responsibility for maintenance of the project. The harbor has undergone modifications, improvements, and repairs since initial construction (USAED, Los Angeles 1988a; Bottin 1988) and currently consists of a 1,500-ft-long offshore breakwater (elevation (el) +14 ft*); north and south jetties with 1,250- and 1,070-ft lengths, respectively (el vary from +15 to +20 ft); and a 250-ft-long middle jetty (el +8 to +10 ft). Also included in the project are a 1,750-ft-long, 300-ft-wide entrance channel (el -20 ft); sand traps (el vary from -25 to -40 ft) located in the lee of the offshore breakwater; and three mooring basins. An aerial photograph of the harbor is shown in Figure 2.

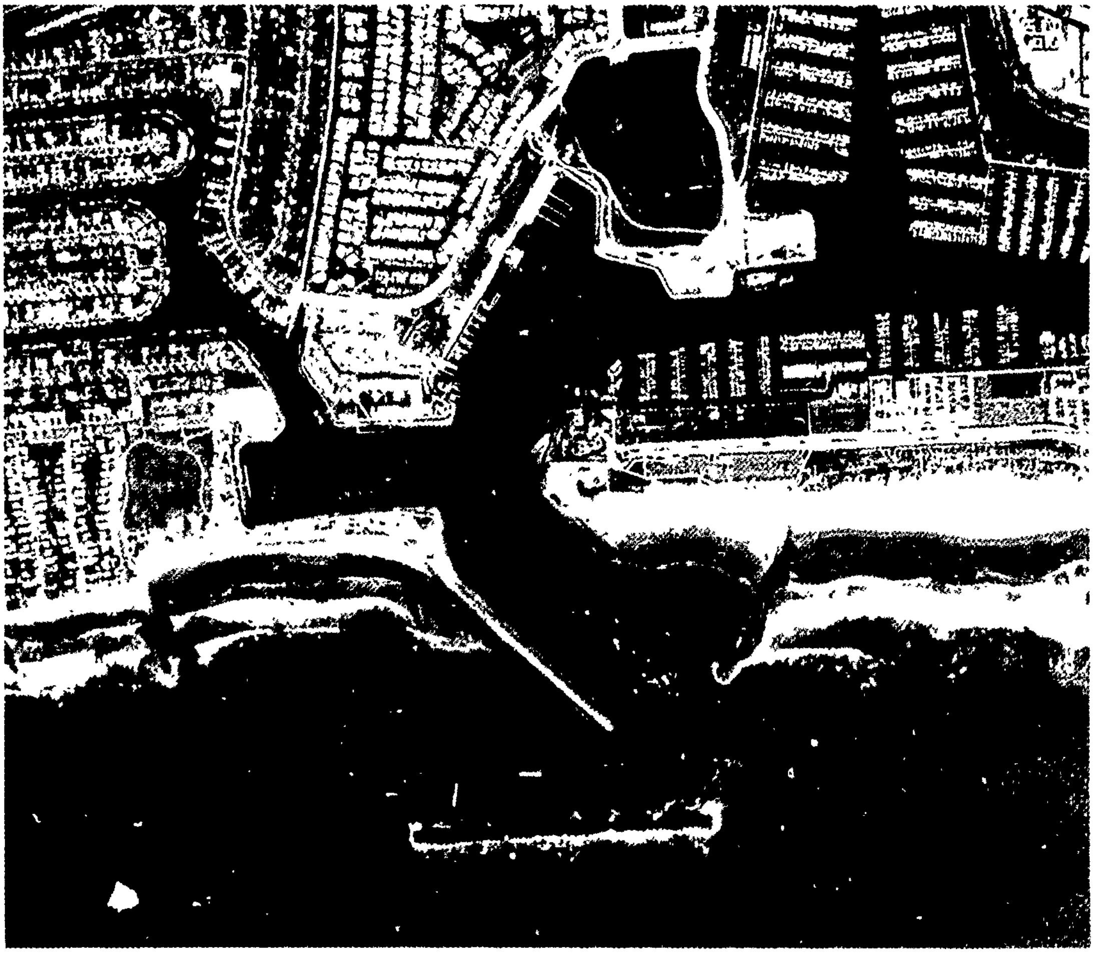

Figure 2. Aerial view of Ventura Harbor

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* All elevations cited herein are in feet referred to mean lower low water (mllw).

The Problem

5. The orientation of the harbor and shoreline is such that waves originating from the west cause sediment to move predominantly in the downcoast (southerly) direction. During most of the year, sand from upcoast beaches and the Ventura River (3 miles upcoast of the harbor) migrates along the beaches into the sand traps and entrance channel. In the summer months, primarily when southern swells are present, a considerable volume of sediment from downcoast sources is deposited in the entrance channel. Historical records indicate that a yearly average of 640,000 cu yd of sediment enters the sand trap areas and entrance channel (USAED, Los Angeles 1988a). Of this volume, about 540,000 cu yd moves downcoast (southerly), with approximately 400,000 cu yd accumulating in the sand trap in the lee of the detached breakwater and about 140,000 cu yd moving around the north jetty into the entrance channel. It is estimated that annually 100,000 cu yd of material migrates upcoast around the head of the south jetty and into the entrance channel.

6. Shoaling of the entrance results in frequent maintenance dredging. The accumulation of sediment in the entrance also creates hazardous navigation conditions, due to breaking waves and shallow depths, for both large- and small-craft vessels. The conditions repeatedly result in capsized vessels, personal injuries, groundings, and loss of revenue from commercial ventures that rely on the harbor for their livelihood. Since 1982, over 60 capsized or damaged vessels and 11 injuries have occurred (USAED, Los Angeles 1988a). Average annual damages are estimated to be $47,000, and average annual losses of income are about $500,000. These values result from hazardous wave conditions in the harbor entrance that occur approximately 110 days per year and prevent vessels from leaving or entering the harbor (USAED, Los Angeles 1988a).

Purpose of Model Study

7. At the request of the USAED, Los Angeles (SPL), a coastal hydraulic model investigation was initiated by the Coastal Engineering Research Center (CERC), US Army Engineer Waterways Experiment Station (WES), to:

a. Study wave and shoaling conditions for the existing harbor configuration.

b. Determine if proposed improvements would provide adequate wave and shoaling protection in the harbor entrance.

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

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

Wave-Height Criteria

8. Completely reliable criteria have not yet been developed for ensuring satisfactory navigation conditions in small-craft harbors during attack by storm waves. For this study, however, SPL specified that for an improvement plan to be acceptable, maximum significant wave heights were not to exceed the criteria established in their Feasibility Report (USAED, Los Angeles 1988a). In general, for an improvement plan to be acceptable, maximum significant wave heights in the entrance were not to exceed a range of 4 to 5 ft for incident wave heights ranging from 8 to 10 ft seaward of the existing structures. These values are based on the percentage of vessels that do not leave the harbor because of hazardous waves in the entrance for various incident conditions.

PART II: THE MODEL

Design of Model

9. The Ventura Harbor model (Figure 3) was constructed to an undistorted linear scale of 1:75, model to prototype.

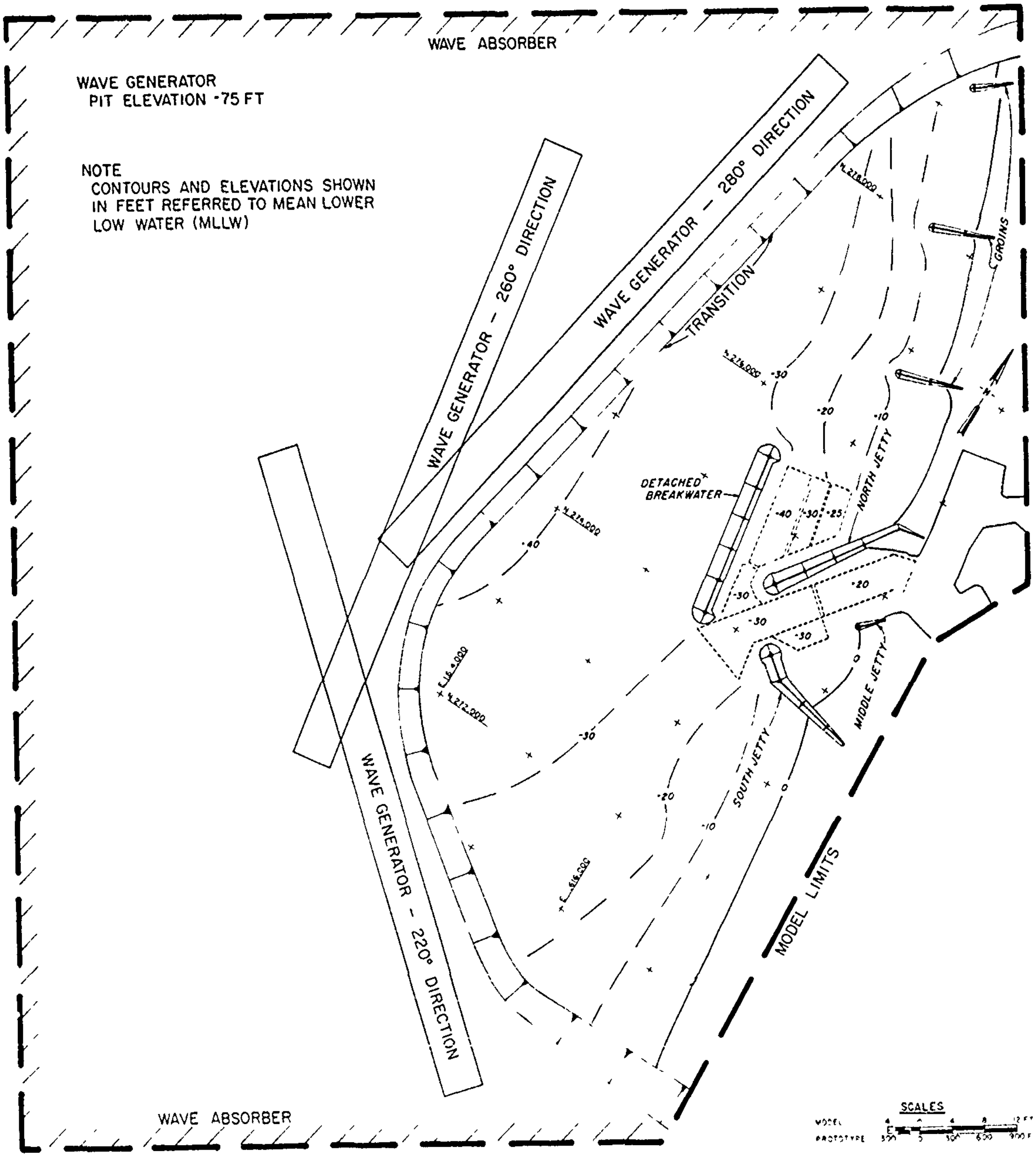

Figure 3. Model layout

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	Dimension	Model-Prototype Scale Relations
Length	L	$L_r = 1:75$
Area	L^2	$A_r = L_r^2 = 1:5,625$
Volume	L^3	$V_r = L_r^3 = 1:421,875$
Time	T	$T_r = L_r^{1/2} = 1:8.66$
Velocity	L/T	$V_r = L_r^{1/2} = 1:8.66$

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

10. The existing breakwaters and revetments at Ventura Harbor, as well as proposed improvements, are rubble-mound structures. Experience and experimental research have shown that considerable wave energy passes through the interstices of this type structure; thus, the transmission and absorption of wave energy became a matter of concern in design of the 1:75-scale model. In small-scale hydraulic models, rubble-mound structures reflect relatively more and absorb or dissipate relatively less wave energy than geometrically similar prototype structures (Le Méhauté 1965). Also, the transmission of wave energy through a rubble-mound structure is relatively less for the small-scale model than for the prototype. Consequently, some adjustment in small scale model rubble-mound structures is needed to ensure satisfactory reproduction of wave-reflection and wave-transmission characteristics. In past investigations (Dai and Jackson 1966, Brasfeild and Ball 1967) at WES,

this adjustment was made by determining the wave-energy transmission characteristics of the proposed structure in a two-dimensional model using a scale large enough to ensure negligible scale effects. A section then was developed for the small-scale, three-dimensional model that would provide essentially the same relative transmission of wave energy. Therefore, from previous findings for structures and wave conditions similar to those at Ventura Harbor, it was determined that a close approximation of the correct wave-energy transmission characteristics could be obtained by increasing the size of the rock used in the 1:75-scale model to approximately 1.5 times that required for geometric similarity. Accordingly, in constructing the rubble-mound structures in the Ventura Harbor model, the rock sizes were computed linearly by scale and then multiplied by 1.5 to determine the actual sizes to be used in the model.

11. The seaward portions of the north and south jetties at Ventura Harbor have been capped with tribar armor units. Small-scale tribars were not available at CERC for the model tests. Small-scale tetrapods, however, were available, and it was felt that tetrapods would more closely approximate the reflection and transmission coefficients of tribars, as opposed to stone. Therefore, small-scale tetrapods were placed as the primary armor on these portions of the north and south jetties for all model tests.

The Model and Appurtenances

12. The model reproduced about 9,400 ft of the California shoreline and included the harbor entrance and underwater topography in the Pacific Ocean to an offshore depth of -40 ft with a sloping transition to the wave generator pit el of -75 ft. The total area reproduced in the model was approximately 14,900 sq ft, representing about 3.0 square miles in the prototype. A general view of the model is shown in Figure 4. Vertical control for model construction was based on mllw. Horizontal control was referenced to a local prototype grid system.

13. Model waves were generated by an 80-ft-long, unidirectional spectral, electrohydraulic wave generator with a trapezoidal-shaped, vertical-motion plunger. The vertical motion of the plunger was controlled by a computer-generated command signal, and the movement of the plunger caused a displacement of water that generated the required test waves. The wave

Figure 4. General view of model

generator was mounted on retractable casters, which enabled it to be positioned to generate waves from required directions.

14. An Automated Data Acquisition and Control System, designed and constructed at WES (Figure 5), was used to generate and transmit control signals, monitor wave generator feedback, and secure and analyze wave-height data at selected locations in the model. Through the use of a Microvax computer, the electrical output of parallel-wire, capacitance-type wave gages, which varied with the change in water-surface elevation with respect to time, was recorded on magnetic disks. These data were then analyzed to obtain the wave-height data.

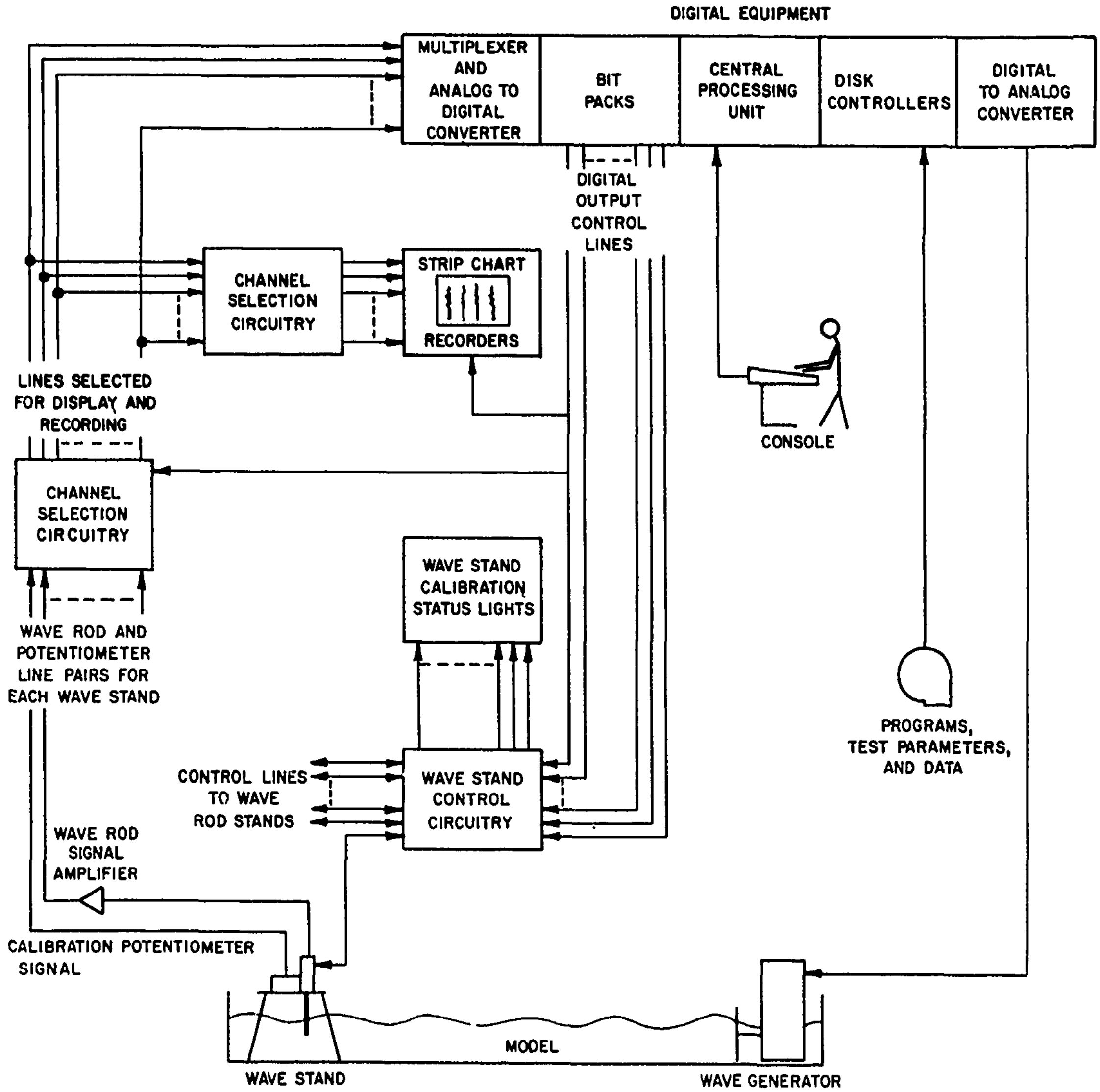

Figure 5. Automated Data Acquisition and Control System

15. A 2-ft (horizontal) solid layer of fiber wave absorber was placed around 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.

Selection of Tracer Material

16. A fixed-bed model molded in cement mortar was constructed, and a tracer material selected to qualitatively determine the movement and deposition of sediment in the vicinity of the harbor. The 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, n_D ; and the relative specific weight ratio, n_γ' . These relations were determined experimentally using a wide range of wave conditions and bottom materials and are valid mainly for the breaker zone.

17. Noda's scaling relations indicate that movable-bed models with scales in the vicinity of 1:75 (model to prototype) should be distorted (i.e., they should have different horizontal and vertical scales). Since the fixed-bed model of Ventura Harbor was undistorted to allow accurate reproduction of short-period wave and current patterns. the following procedure was used to select a tracer material. Using the prototype sand characteristics (median diameter, D_{50} = 0.17 mm, specific gravity = 2.65) and assuming the horizontal scale to be in similitude (i.e. 1:75), the median diameter for a given specific gravity of tracer material and the vertical scale were computed. The vertical scale was then assumed to be in similitude, and the tracer median diameter and horizontal scale were computed. This computation 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} = 0.44 mm) were selected for use as a tracer material throughout the model investigation.

PART III: TEST CONDITIONS AND PROCEDURES

Selection of Test Conditions

Still-water level

18. Still-water levels (swl's) for harbor wave action models are selected so that the various wave-induced phenomena 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.

19. In most cases, it is desirable to select a model swl that closely approximates the higher water stages that 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 shoreward mass transport.

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.

20. Ventura Harbor experiences two high and two low tides daily, typical of the Pacific coast of North America. These tides are of diurnal inequality. The range between mean lower low water and mean higher high water is 5.4 ft. Storm surge along the entire coast of southern California is relatively small (generally less than 1 ft) when compared with tidal fluctuations (USAED, Los Angeles 1988a).

21. Based on a review of 63 years of tide data from a gage located in Los Angeles Harbor, the annual return interval water level at the site is +7.0 ft (USAED, Los Angeles 1988b). An swl of +7.0 ft, therefore, was selected by SPL for use during model testing of wave heights in the entrance. In addition, an swl of +3.0 ft was determined to be more representative of average conditions at the site and was selected by SPL for use during shoaling tests.

Factors influencing selection of test wave characteristics

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

- a. The fetch and decay distances (the latter being the distance over which waves travel after leaving the generating area) for various directions from which waves can approach the problem area.
- b. The frequency of occurrence and duration of storm winds from the different directions.
- c. The alignment, size, and relative geographic position of the navigation entrance to the harbor.
- d. The alignments, lengths, and locations of the various reflecting surfaces inside the harbor.
- e. The refraction of waves caused by differentials in depth in the area seaward of the harbor, which may create either a concentration or a diffusion of wave energy at the harbor site.

Wave refraction

23. When wind waves move into water of gradually decreasing depth, transformations take place in all wave characteristics except wave period (to the first order of approximation). The most important transformations with respect to the selection of test wave characteristics are the changes in wave height and direction of travel due to the phenomenon referred to as wave refraction. The change in wave height and direction may be determined by using the numerical Regional Coastal Processes Wave Transformation Model (RCPWAVE) developed by Ebersole (1985). This model predicts the transformation of monochromatic waves over complex bathymetry and includes refractive and diffractive effects. Diffraction becomes increasingly important in regions with complex bathymetry. Finite difference

approximations are used to solve the governing equations, and the solution is obtained for a finite number of grid cells comprising the domain of interest. Much of the early work in this area during the 1950s was based on wave ray methods and manual construction of refraction diagrams using linear, gravity wave theory. During the 1960s and early 1970s, the linear wave refraction problem was solved more efficiently through the use of digital computers. All of these methods, however, addressed the refraction problem only.

24. The solution technique employed by RCPWAVE is a finite difference approach; thus, the wave climate in terms of wave height, H, wave period, T, and wave direction of approach, θ, is available at a large number of computational points throughout the region of interest, and not just along wave rays. Computationally, the model is very efficient for modeling large areas of coastline subjected to widely varying wave conditions and, therefore, is an extremely useful tool in the solution of many types of coastal engineering problems.

25. When the refraction coefficient (K_r) is determined, it is multiplied by the shoaling coefficient (K_s) and gives a conversion factor for transfer of deepwater wave heights to shallow-water values. The shoaling coefficient, a function of wavelength and water depth, can be obtained from the Shore Protection Manual (1984). For this study, extensive wave refraction/diffraction/shoaling analyses were conducted for the Ventura Harbor site by SPL. The analyses indicated that the harbor is located within a convergence zone that amplifies westerly sea and swell. Waves that approach Ventura from the southerly sectors, however, are significantly lower in height in the vicinity of the harbor.

Prototype wave data and selection of test waves

26. Deepwater storm waves predominantly approach the outer continental shelf of the southern California coast from the northwest; however, storm waves generated by distant Southern Hemisphere disturbances occasionally approach from the westerly and southerly quadrants. Ventura Harbor is partially sheltered from waves by the adjacent shoreline and offshore channel islands. Due to its orientation on the southern California coast, the harbor is exposed to large waves propagating from storms on the Pacific Ocean which travel shoreward through three corridors bounded by azimuths (a) 168 through 186 deg. (b) 214 through 220 deg, and (c) 262 through 280 deg (Figure 6).

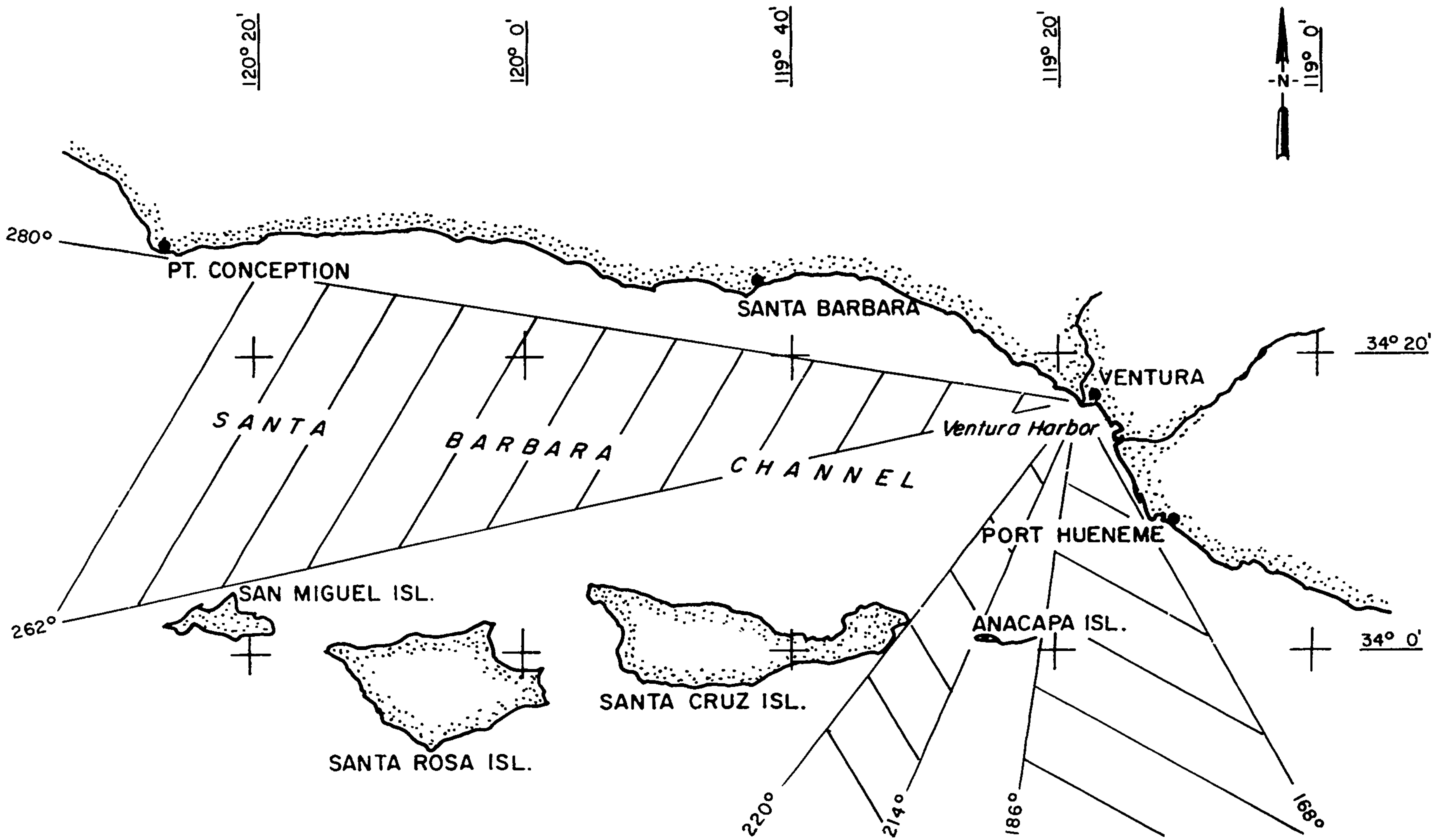

Figure 6. Ventura Harbor storm wave exposure windows

27. Waves generated by local winds may arrive from a wider spread of directions, but wind waves from approximately due west are dominant throughout the year, except in December, January, and February, when waves from the south are equally common. Waves approaching Ventura Harbor average about 3 ft in height, but can be as high as 22 ft (USAED, Los Angeles 1988a). A large shoal, at a depth of approximately 400 ft, lies offshore about 20 miles west of the harbor. This shoal causes some deepwater waves to refract and converge prior to reaching the harbor.

28. Measured prototype wave data covering a sufficiently long duration from which to base a comprehensive statistical analysis of wave conditions for the Ventura Harbor area were not available. However, statistical wave hindcast estimates, as generated by meteorological phenomenon, have been developed (Corson et al. 1987; National Marine Consultants 1960; and Kent 1988a*, 1988b**). Some of these wave estimates are seaward of the channel islands, and their approach would be blocked by the islands. This blocking action is dependent on water depth and wave period, with longer period waves requiring deeper water for passage than short-period waves. Wave statistics prepared for Pacific Weather Analysis for a site about 12 miles west-southwest of Ventura Harbor (Kent 1988a*) and for Ventura Port District at a site about 10 miles south of the harbor (Kent 1988b**) were used for this study. Both of these sites were in the lee of the offshore islands.

29. The analysis indicated that waves approach Ventura Harbor from the 262- to 280-deg sector over 94 percent of the time in an average year. The total annualized frequency of occurrence from the other approach directions is less than 3 percent. The remaining 3 percent applies to waves arriving at the referenced site from sectors to which Ventura Harbor is not exposed. In addition, data from the southern California Bight hindcast study were used in determining an average annual predominant wave climate for the model testing program. Through application of refraction and shoaling coefficients, wave characteristics were transformed to shallow-water values in the vicinity of the harbor entrance. Most waves approach the harbor entrance from 260 to

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* Richard E. Kent, 1988a, "Wave Statistics for a Site in Eastern Santa Barbara Channel," unpublished data prepared for Pacific Weather Analysis, Bellingham, WA.

** Richard E. Kent, 1988b, "Wave Climate at Ventura Harbor, Ventura, California," unpublished data prepared for Ventura Port District, Bellingham, WA.

275 deg. Based on these results, SPL selected wave characteristics seaward of the entrance with periods ranging from 8 to 16 sec and heights ranging from 6 to 15 ft for model testing of navigation conditions in the harbor entrance.

30. The analysis revealed that the approach to the harbor of refracted waves, representing Southern Hemisphere swell, is approximately 220 deg, and SPL selected 17-sec, 4-ft test waves as representative of these conditions. Another angle of approach for waves from the Southern Hemisphere is about 205 deg as determined subsequent to the initiation of model testing. The direction is more representative of waves approaching from the southern sector and tests; therefore, those from the 205-deg direction were also used as representative of Southern Hemisphere swell.

31. For shoaling tests at the northern portion of the harbor, SPL selected test waves from 280 deg. After reviewing initial results, the 280-deg direction was chosen for shoaling tests because of qualitative effects on the resultant shoaling patterns. The 280-deg direction created the most severe longshore currents that were the force which moved the sediment tracer material. It moved sediment similar to the 270-deg direction, but in less time. After observing several wave conditions in the model, 11-sec, 12-ft waves were selected for use during tracer tests for waves from 280 deg. Wave-height tests were not conducted from this direction.

32. In summary, characteristics of test waves used in the model are shown in the following tabulation:

	Selected Test Waves		
Direction, deg	Period, sec	Height, ft*	Swl(s), ft, mllw
280	11	12	+3.0, +7.0
270	8	6	+7.0
	10	6	+7.0
		9	+7.0
		12	+7.0
	13	6	+7.0
		9	+7.0
		12	+7.0
		15	+7.0
	16	6	+7.0
		9	+7.0
		12	+7.0
		15	+7.0

(Continued)

* All selected test wave heights reported herein are defined seaward of the harbor entrance at approximately an el of -32 ft.

(Concluded)

	Selected Test Waves		
Direction, deg	Period, sec	Height, ft*	Swl(s), ft, mllw
260	8	6	+7.0
	10	6	+7.0
		9	+7.0
		12	+7.0
	13	6	+7.0
		9	+7.0
		12	+7.0
		15	+7.0
	16	6	+7.0
		9	+7.0
		12	+7.0
		15	+7.0
220	17	4	+3.0, +7.0
205	17	4	+3.0

33. Unidirectional wave spectra for the selected test waves were generated (based on JONSWAP parameters) and used throughout the model investigation. Plots of typical wave spectra are shown in Figure 7. The dashed line represents the desired spectra, whereas the solid line represents the spectra generated by the wave machine. A typical wave train is also shown in Figure 8, which depicts water-surface elevation (η) versus time. The selected test waves were significant wave heights, the average height of the highest one-third of the waves or H_s. Usually H_s is equivalent in deep water to H_{mo} (energy-based wave).

Analysis of Model Data

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

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

b. Comparison of sediment tracer movement and subsequent deposits.

c. 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 gage location, was computed. All wave heights then were adjusted to compensate for excessive model wave-height attenuation due to viscous bottom friction by application of Keulegan's

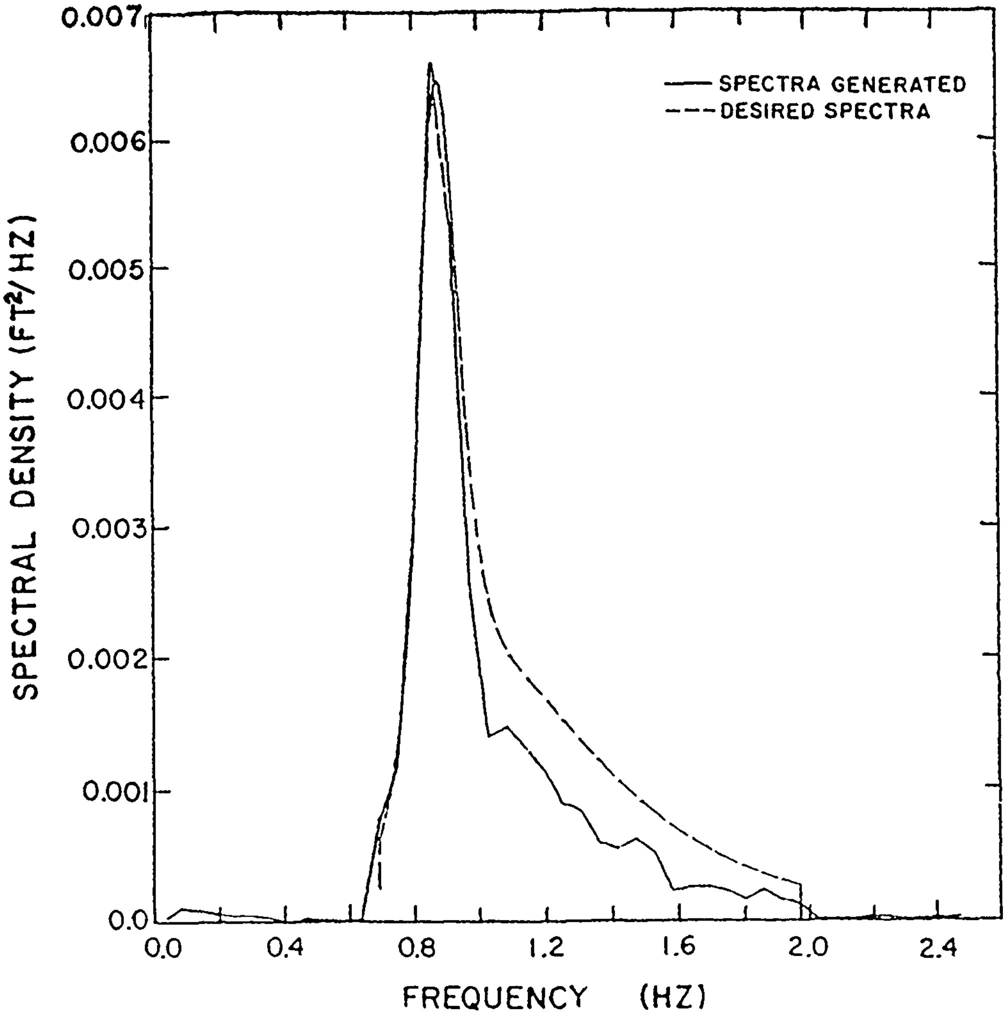

Figure 7. Typical energy density versus frequency plot for wave spectra; 11-sec, 12-ft test waves

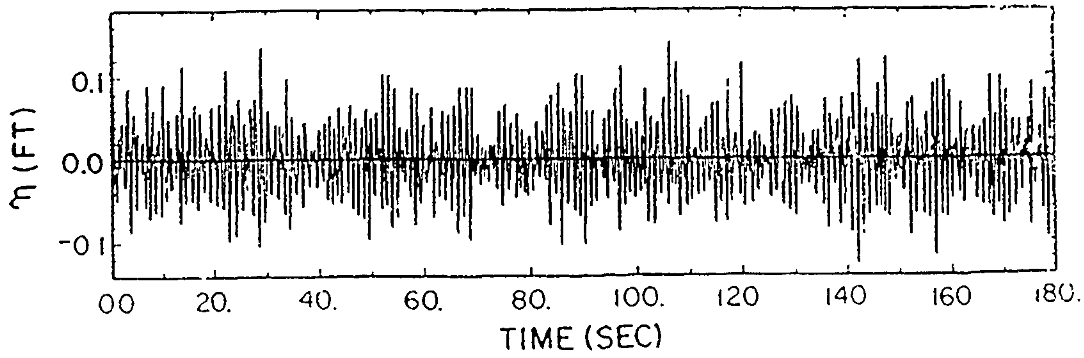

Figure 8. Typical wave train; 11-sec, 12-ft test waves

equation*. From this equation, reduction of wave heights in the model (relative to the prototype) can be calculated as a function of water depth, width of wave front, wave period, water viscosity, and distance of wave travel.

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* 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.

PART IV: TESTS AND RESULTS

The Tests

Existing conditions

35. Prior to testing the various improvement plans, tests were conducted for existing conditions (Plate 1) to establish a base from which to evaluate the effectiveness of the plans. Wave-height data were secured at various locations throughout the harbor entrance for the selected test waves from 270, 260, and 220 deg. In addition, sediment tracer patterns and wave pattern photographs were obtained for representative test waves from the various directions.

Improvement plans

36. The originally proposed improvement plan consisted of an extension of the existing detached breakwater, a new south beach groin, a spur groin extending from the existing north jetty, and additional channel dredging. Wave heights, sediment tracer patterns, and/or wave patterns were secured for 11 test plan configurations. Variations consisted of changes in the length, alignment, and location of the proposed north spur groin. Brief descriptions of the improvement plans are presented in the following subparagraphs; dimensional details are presented in Plates 2-12. Typical breakwater and groin sections are shown in Plate 13. Various structure lengths referred to in the following subparagraphs indicate lengths at the crest.

- a. Plan 1 (Plate 2) consisted of a 300-ft-long seaward extension of the existing detached breakwater. The breakwater extension originated at the southern end of the detached structure and extended southwesterly parallel to the entrance channel. In addition, the entrance channel was enlarged and deepened to an el of -40 ft to increase its capacity for holding sediment.
- b. Plan 2 (Plate 3) entailed the elements of Plan 1 with a 250-ft-long spur groin attached to the north jetty. The spur originated at the head of the jetty and extended northwesterly perpendicular to the axis of the jetty.
- c. Plan 3 (Plate 4) involved the elements of Plan 1 with a 250-ft-long spur groin attached to the north jetty. The spur originated at the head of the jetty and extended westerly at an angle of 45 deg to the axis of the jetty. The alignment of the spur was perpendicular to the detached breakwater.
- d. Plan 4 (Plate 5) included the elements of Plan 1 with a 250-ft-long spur groin attached to the north jetty. The spur originated at a point 450 ft shoreward of the head of the jetty and extended northwesterly perpendicular to the axis of the jetty.

e. Plan 5 (Plate 6) consisted of the element of Plan 1 with a 300-ft-long spur groin attached to the north jetty. The spur originated at a point 825 ft shoreward of the head of the jetty and extended northwesterly at an angle of 75 deg to the axis of the jetty.

f. Plan 6 (Plate 7) included the elements of Plan 1 with a 300-ft-long spur groin attached to the north jetty. The spur originated at a point 1,110 ft shoreward of the head of the jetty and extended northwesterly at an angle of 75 deg to the axis of the jetty.

g. Plan 7 (Plate 8) involved the elements of Plan 1 with a 325-ft-long spur groin attached to the north jetty. The spur originated at the head of the jetty and extended northwesterly perpendicular to the axis of the jetty.

h. Plan 8 (Plate 9) entailed the elements of Plan 1 with two spur groins attached to the north jetty. The outer spur groin was 200 ft in length and originated at the head of the jetty, and the inner spur was 125 ft in length and originated at a point 720 ft shoreward of the head of the jetty. Both spur groins extended northwesterly perpendicular to the axis of the jetty.

i. Plan 9 (Plate 10) consisted of the elements of Plan 7 with a south groin installed. The new groin was located approximately 1,000 ft south of the existing south jetty and was 650 ft in length.

j. Plan 10 (Plate 11) entailed the 300-ft-long detached breakwater extension of Plan 1, the 325-ft-long north spur groin of Plan 7, and the 650-ft-long south groin of Plan 9. The depths in the entrance channel were raised to an el of -20 ft except in areas where the natural contours would result in greater depths at the entrance.

k. Plan 11 (Plate 12) involved the 300-ft-long detached breakwater extension of Plan 1 and the 650-ft-long south groin of Plan 9. In addition, a 300-ft-long spur groin was attached to the north jetty and extended northwesterly from the head at an angle of 65 deg to the axis of the jetty. The spacing between the detached breakwater and the new spur groin allowed for a 100-ft opening at an el of -15 ft.

Wave-height tests and wave patterns

37. Wave heights and patterns were obtained for three of the improvement plans for representative waves from the selected test directions. Most tests were conducted for incident waves from 260 and 270 deg. Wave gage locations for the plans tested (Plans 1, 9, and 10) are shown in Plates 2, 10, and 11.

Sediment tracer tests

38. Sediment tracer tests were conducted for most of the improvement plans. Tracer material was introduced into the model north of the existing

groin just north of the harbor and south of the proposed south groin to represent sediment from the north and south shorelines, respectively. During testing, a predetermined amount of sediment tracer material was fed into the model for a given time duration for each plan so that test results would have a common base for comparison. Tests involving the proposed spur groin on the north jetty were conducted for test waves from 280 deg, and tests involving the proposed south groin were conducted for test waves from 220 and/or 205 deg.

Test Results

39. In evaluating tests results, the relative merits of the various plans were based on an analysis of measured wave heights in the harbor entrance, the movement of tracer material and subsequent deposits, and visual observations. Model wave heights (significant wave height or $H_{1/3}$) were tabulated to show measured values at selected locations. The general movement of tracer material and subsequent deposits were shown in photographs. Arrows were superimposed onto photographs to define sediment movement patterns.

Existing conditions

40. Results of wave-height tests conducted for the existing harbor configuration with postdredge (unshoaled) entrance conditions are presented in Table 1. Maximum significant wave heights in the entrance were 10.7 and 11.1 ft for test waves from 270 and 260 deg, respectively. For 9-ft incident wave conditions seaward of the harbor, maximum significant wave heights were 6.5 and 6.4 ft in the entrance for test waves from 270 and 260 deg, respectively. Test waves from 220 deg yielded maximum significant wave heights of 3.8 ft in the entrance. Typical wave patterns for existing conditions (without shoaled entrance) are shown in Photos 1-6.

41. Wave-height test results obtained for existing conditions with a typical shoaled entrance condition are presented in Table 2. The bathymetry of the shoal in the entrance was provided by SPL and constructed in the model with sand and/or gravel material. Maximum significant wave heights were 11.2 and 11.3 ft in the entrance for test waves from 270 and 260 deg, respectively. For 9-ft incident waves, maximum significant wave heights in the entrance were 6.7 and 7.0 ft, for test waves from 270 and 260 deg, respectively. Typical wave patterns for existing conditions with the shoaled entrance are shown in Photos 7-10.

42. The general movement of tracer material and subsequent deposits for existing conditions with an unshoaled entrance are shown in Photos 11 and 12. For test waves from 280 deg, tracer material migrated southerly around the head of the existing groin north of the harbor. Some moved in the swash zone along the shore, and some moved in the breaker zone offshore. Sediment along the shoreline migrated along the north jetty, and sediment in the breaker zone moved into the deposition basin. Some material settled in the basin, and some moved around the head of the north jetty and deposited in the entrance channel. For test waves from 220 deg, sediment tracer material moved northerly along the shoreline in the swash and breaker zones. Some material deposited along the south shoreline and jetty, and some (moving in the breaker zone) migrated around the head of the south jetty and deposited in the entrance channel.

Improvement plans

43. Results of wave-height tests conducted with Plan 1 installed in the model are presented in Table 3. Maximum significant wave heights in the entrance were 8.2 and 7.7 ft for test waves from 270 and 260 deg, respectively. For 9-ft incident wave conditions, maximum significant wave heights in the entrance were 5.0 and 4.7 ft for test waves from 270 and 260 deg, respectively. Typical wave patterns obtained for Plan 1 are shown in Photos 13-16.

44. The general movement and subsequent deposition of tracer material for the north jetty spur groins of Plans 2-8 are shown in Photos 17-23, respectively. Sediment tracer initially moved southerly toward the north jetty, as it did for existing conditions. For all plans, sediment migrated around the heads of the proposed spur groins and eventually into the harbor entrance channel. All these spur groin configurations, with the exception of Plan 7, resulted in significant deposits in the harbor entrance. Relative comparisons of the sediment tracer deposits indicated that the Plan 7 spur groin resulted in less material penetrating into the harbor entrance. The longer Plan 7 structure created a narrower opening between it and the detached breakwater, and a strong clockwise eddy also formed in the deposition basin, which tended to divert tracer material back to the north. The structure tended to break up the southerly longshore current that existed between the jetty and the offshore breakwater.

45. The general movement of tracer material and subsequent deposits with Plan 9 installed for test waves from 220 deg are shown in Photo 24. Sediment tracer material moved northerly in the swash and breaker zones.

Sediment accumulated on the shoreline adjacent to the proposed groin, and material in the breaker zone moved past the head of the groin. Some deposited offshore, and some migrated around the groin to the north. No material entered the harbor entrance for these conditions. This shoal formation was subjected to 9-sec, 4.5-ft test waves from 270 deg (day-to-day wave conditions), and the material in the offshore bar moved shoreward with most migrating back to the south. Some of the material that had been deposited between the proposed groin and the existing south jetty moved shoreward, and some moved offshore adjacent to the south jetty. The movement of material and subsequent deposits are shown in Photo 25. No sediment tracer material entered the harbor entrance channel. This condition then was subjected to a storm wave condition (11-sec, 12-ft test waves) from 270 deg. The material seaward of the head of the proposed south groin moved in a southerly direction, and the material between the groin and south jetty moved northerly toward the jetty. A counterclockwise eddy moved some material into the entrance channel, as shown in Photo 26. Tracer tests were conducted for the Plan 9 groin for test waves from 205 deg, also. The general movement of tracer and subsequent deposits are shown in Photo 27. Material moved northerly with deposits around the head of the groin similar to the patterns observed for test waves from 220 deg.

46. Wave-height data secured for Plans 9 and 10 are presented in Table 4. Test waves from 220 deg for Plan 9 resulted in maximum significant wave heights of 3.4 ft in the harbor entrance. For Plan 10, maximum significant wave heights in the entrance were 8.5 and 7.9 ft for test waves from 270 and 260 deg, respectively. For 9-ft incident wave conditions, maximum significant wave heights were 5.1 ft in the entrance for test waves from both 270 and 260 deg. Typical wave patterns obtained for Plan 10 are shown in Photos 28-31.

47. After exposure to test waves from 280 deg, the general movement and deposition of tracer material for the spur groin of Plan 11 are shown in Photo 32. Tracer material moved through the deposition basin and along the north jetty. The spur deflected longshore currents in a clockwise eddy in the deposition basin, and most sediment tracer material settled in the basin. Some material did move around the head of the spur groin and into the entrance channel; however, a comparison of the results with other plans tested (with the exception of Plan 7) indicated significant improvement with minimal shoaling deposits.

48. The deposition basin was partially filled, based on bathymetry data furnished by SPL (Plate 12), and tracer tests were repeated for the Plan 11 spur groin. A view of the partially filled basin, prior to testing, is shown in Photo 33, and results of sediment tracer tests are shown in Photo 34. Material moved into the deposition basin similar to previous test, but the spur groin and fill condition prevented all but a minute amount of tracer from moving around the head of the groin toward the entrance channel.

Discussion of test results

49. Wave heights obtained in the entrance for existing conditions indicated maximum significant wave heights ranging from 5.7 to 6.5 ft for postdredge conditions and from 6.0 to 7.0 ft for shoaled conditions with 9-ft incident wave conditions. Also, during periods of severe storm wave attack, wave conditions in excess of 11 ft occurred in the entrance for existing conditions. The purpose of breakwater improvements at the Ventura Harbor site was directed towards reducing adverse wave conditions in the entrance channel that discourage harbor egress and render inbound transit dangerous. The structure was designed and oriented to be least disruptive to navigation, alleviate inbound small craft exposure to broaching conditions, and reduce prevailing westerly sea and swell heights between the two existing jetties. It was determined that wave heights of 4 to 5 ft in the entrance for incident wave conditions of 8 to 10 ft would provide required benefits with respect to economic considerations (USAED, Los Angeles 1988a).

50. With the 300-ft-long breakwater extension in place and the entrance channel dredged to the proposed depths (Plan 1), maximum significant wave heights obtained in the entrance ranged from 3.9 to 5.0 ft for 9-ft incident wave conditions. These values fell within the established wave-height criterion. When the harbor entrance was shoaled to -20 ft (Plan 10), maximum significant wave heights in the entrance ranged from 4.1 to 5.1 ft, which exceeded the criterion by only 0.1 ft (acceptable to SPL). During severe storm periods, significant wave heights for the proposed improvement plan were in excess of 8 ft in the entrance for Plan 1; however, this condition was a 3-ft reduction when compared with existing conditions. Based on test results, the 300-ft-long extension appeared to be optimal with respect to wave protection provided in the entrance and costs (i.e., decreasing the structure length would increase wave heights in the entrance, and increasing the length of the structure would increase costs).

51. Sediment tracer test results for existing conditions with test waves from 280 deg indicated that sediment moved from the north shoreline into the deposition basin and alongside the north jetty. Some deposited in the deposition basin, and some moved southerly between the north jetty head and the detached breakwater and resulted in heavy deposits in the harbor entrance.

52. The purpose of north jetty improvement plans was to prevent as much littoral sand as possible from bypassing the north jetty, which in turn would reduce the frequency of dredging. The intent of the spurs was to dissipate the wave-induced circulation cell between the jetty and the detached breakwater and to deflect sand into the deposition basin. In addition, the natural formation of a sand plug was desirable.

53. The orientations of the 250-ft-long spur groins at the jetty head (Plans 2 and 3) resulted in sediment moving around the head of the spur groins and into the entrance channel for test waves from 280 deg. The relocation of the spur groins along the trunk of the jetty (Plans 4-6) and/or the placement of two spur groins (Plan 8) did not significantly reduce sediment deposits in the entrance channel of the harbor. Additional spur groin configurations expeditiously tested in the model revealed that the opening between the spur groin and the existing detached breakwater should be reduced in width and/or moved northerly. This modification would allow sediment moving southerly between the structures, due to longshore currents, to settle before it migrated to the entrance channel.

54. The 325-ft-long spur groin of Plan 7 was originally selected as the optimum plan based on relative comparisons of sediment tracer deposits in the entrance channel. At this point in the investigation, it was determined that the minimum navigation width between the spur groin and the detached breakwater (for access of the dredge) was 100 ft at an el of -15 ft. This criterion allowed the spur groin to be relocated and reduced in length by 25 ft (Plan 11). Test results for Plan 11 revealed only minimal shoaling of the entrance (similar to Plan 7). The Plan 7 and Plan 11 spur groins created clockwise eddies over the deposition basin in which most of the sediment deposited. Some sediment did penetrate the opening between the spur groins and detached breakwater, but most appeared to settle before reaching the entrance channel. Based on relative comparisons of shoaling patterns for all plans, Plans 7 and 11 significantly reduced shoaling in the harbor entrance channel.

55. With the partially filled deposition basin of Plan 11, tracer test results indicated that practically no sediment should enter the harbor entrance channel. These tests indicate that the tendency for the formation of a sand plug may exist when the deposition basin becomes partially filled.

56. Sediment tracer test results for existing conditions with test waves from 220 deg revealed that some sediment would move northerly along the shoreline toward the south jetty, and some would move northerly alongshore in the breaker zone and eventually into the entrance channel.

57. The purpose of the proposed south groin was to impound sand at a more distant location from the entrance channel. Its design function was to encourage temporary storage of sand transported during periods of upcoast reversal. Impounded material then would be transported southerly during prevailing westerly sea and swell.

58. Tracer tests with the proposed south groin installed in the model (Plan 9) indicated that the longshore currents would be broken up for waves from the southerly directions. Some sediment accumulated against the groin, and some moved around its head and deposited in a bar formation. The more predominant westerly waves then moved most of the material back to the south, as desired by the structure design. Some material did move between the proposed groin and existing south jetty for smaller wave conditions from the west. Storm waves from the west formed an eddy with rip currents adjacent to the south jetty and resulted in some material moving into the entrance channel. The results indicated that the south groin should not prevent shoaling in the entrance for all conditions, but based on relative tests, will minimize entrance channel shoaling.

PART V: CONCLUSIONS

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

a. Existing conditions are characterized by excessive wave conditions in the harbor entrance. Significant wave heights up to 7 ft will occur in the entrance for 9-ft incident wave conditions. During periods of severe storm wave attack, significant wave heights in excess of 11 ft will occur in the entrance.

b. For existing conditions, sediment from the north shoreline will move southerly, where some will deposit in the deposition basin and some will move around the head of the north jetty and result in deposits in the harbor entrance channel.

c. For existing conditions, sediment from the south shoreline will move northerly along the shoreline and south jetty and result in deposits in the harbor entrance channel.

d. The 300-ft-long detached breakwater extension of Plan 1 will result in wave heights in the entrance within the established wave-height criterion. Incident waves of 9 ft seaward of the harbor will be reduced by about 2 ft (from 7 to 5 ft in the entrance), and severe storm waves will be reduced by about 3 ft (from 11 to 8 ft in the entrance).

e. Of the spur groin plans tested on the north jetty (Plans 2 through 8 and 11), the 325-ft-long groin of Plan 7 and the 300-ft-long groin of Plan 11 appeared to be optimum with respect to shoaling in the entrance channel. Relative to other spur groin plans tested, the Plan 7 and Plan 11 groins will result in minimal shoaling of the entrance for sediment approaching the harbor from the north.

f. Installation of the south groin (Plan 9) will reduce shoaling in the harbor entrance channel. Most material moving northerly around the head of the groin for southerly test waves will be transported back to the south during times of the more predominant westerly wave conditions.

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Table 1

Wave Heights for Existing Conditions

Without Shoaled Entrance*

Test Wave		Height, ft, Gage Number						
Period sec	Height ft	1	2	3	4	5	6	7
				270 deg				
8	6	6.3	4.0	4.0	2.1	1.6	4.7	7.2
10	6	6.3	4 0	3.8	2.8	1.3	5.0	8.3
	9	9.3	5.7	5.4	3.8	1.7	5.9	10.1
	12	12.1	7.1	6.7	4.5	2.5	5.9	10.0
13	6	6.2	4.4	3.9	3.3	1.2	5.2	9.9
	9	9.1	6.4	5.8	4.5	1.9	6.6	10.2
	12	12.0	7.8	7.1	5.2	2.3	6.9	10.1
	15	15.3	10.1	8.9	6.4	3.7	7.9	10.3
16	6	6.0	4.5	4.2	3.3	1.2	5.4	9.9
	9	9.1	6.5	6.0	4.3	2.1	6.8	10.8
	12	12.2	8.7	8.0	5.5	3.2	7.7	10.9
	15	15.4	10.7	10.6	6.7	4.1	8.6	11.2
				260 deg				
8	6	6.4	4.0	4.1	3.1	1.1	5.2	7.9
10	6	6.2	4.2	3.7	3.5	1.2	5.3	8.9
	9	9.1	5.9	5.4	4.5	1.8	6.2	9.5
	12	11.7	7.1	6.6	5.3	2.7	6.8	10.3
13	6	6.4	4.2	4.1	3.6	1.3	5.6	9.7
	9	9.2	6.3	6.3	4.8	1.9	6.4	10.7
	12	12.2	7.7	7.8	5.5	2.8	7.2	11.4
	15	14.9	9.4	9.3	6.7	3.3	7.9	11.3
16	6	6.5	4.5	4.3	3.7	1.4	5.8	9.7
	9	9.2	6.4	6.1	4.8	2.1	6.9	10.6
	12	12.1	8.7	7.8	6.3	3.0	7.5	11.4
	15	15.2	11.1	10.4	7.5	4.0	9.2	11.9
				220 deg				
17	4	4.2	3.3	3.8	2.3	1.4	4.8	6.1

* swl = +7.0 ft

Table 2

Wave Heights for Existing Conditions

With Shoaled Entrance*

Test Wave Period sec	Test Wave Height ft	Height, ft, Gage Number 1	2	3	4	5	6	7
				270 deg				
8	6	6.3	3.9	3.8	2.0	0.8	4.7	7.2
10	6	6.4	3.8	4.2	2.5	1.0	4.7	8.8
	9	9.4	5.6	6.0	3.4	1.4	6.3	9.5
	12	12.4	6.9	7.9	4.1	1.9	6.7	9.6
13	6	6.2	4.2	4.5	2.8	1.0	4.7	10.2
	9	9.1	6.3	6.6	3.9	1.7	6.9	10.8
	12	12.4	8.0	8.6	5.1	2.4	7.6	10.3
	15	15.7	10.2	10.2	6.4	3.3	8.3	10.7
16	6	6.2	4.4	4.5	2.9	1.2	5.4	10.3
	9	9.2	6.5	6.7	4.3	1.9	7.5	11.0
	12	12.4	8.4	8.7	5.5	2.9	8.1	11.1
	15	15.1	10.2	11.2	6.5	3.9	8.8	11.4
				260 deg				
8	6	6.4	4.0	4.2	1.9	1.1	5.1	7.9
10	6	6.2	4.0	4.4	2.3	1.2	5.3	8.8
	9	9.0	5.6	6.3	3.0	1.8	6.3	9.9
	12	12.0	7.1	8.2	3.9	2.3	6.9	10.5
13	6	6.3	4.0	4.8	2.5	1.3	4.7	9.5
	9	9.1	5.9	7.0	3.6	1.9	6.8	10.3
	12	12.2	7.3	8.9	4.4	2.6	7.6	10.9
	15	15.1	8.9	10.8	5.2	3.3	8.6	11.3
16	6	6.5	4.5	4.9	3.0	1.6	4.8	9.8
	9	9.1	6.5	6.7	4.4	2.2	7.0	10.9
	12	12.0	8.5	9.0	5.8	3.1	8.3	11.3
	15	15.2	10.7	11.3	7.0	4.2	8.4	11.6

* swl = +7.0 ft

Table 3

Wave Heights for Plan 1*

Test Wave Period sec	Test Wave Height ft	Height, ft, Gage Number 1	2	3	4	5	6	7
				270 deg				
8	6	6.1	2.4	2.7	0.9	1.1	3.0	6.9
10	6	6.3	2.8	2.9	1.1	1.0	3.4	9.0
	9	9.4	3.7	3.9	1.5	1.4	5.6	10.2
	12	12.3	4.4	4.8	2.2	2.1	6.0	10.2
13	6	5.6	2.9	3.2	1.5	1.0	3.2	10.3
	9	8.2	4.2	4.6	2.1	1.6	6.0	11.2
	12	11.0	5.0	5.3	2.9	2.5	6.2	11.0
	15	13.5	6.3	7.1	4.1	3.7	7.4	11.0
16	6	5.9	3.5	3.5	1.7	0.9	3.5	10.4
	9	8.9	5.0	5.0	2.6	1.7	6.5	11.1
	12	11.4	6.1	6.4	3.5	2.6	6.9	11.0
	15	13.7	7.5	8.2	4.7	3.7	7.6	10.9
				260 deg				
8	6	6.2	2.6	3.1	1.5	1.2	3.4	7.8
10	6	6.0	2.8	3.1	1.7	1.0	3.8	8.3
	9	8.8	3.8	4.2	2.1	1.5	5.5	9.0
	12	11.6	4.7	4.9	2.6	2.2	5.8	10.1
13	6	5.5	3.1	3.3	2.0	0.8	3.2	8.9
	9	7.9	4.2	4.6	2.6	1.4	5.5	10.3
	12	10.6	5.1	6.1	3.1	2.1	6.2	10.8
	15	13.0	5.8	7.1	3.7	2.9	6.6	10.9
16	6	5.7	3.4	3.3	2.1	1.0	3.1	9.6
	9	8.0	4.7	4.5	2.9	1.5	5.6	10.9
	12	11.0	6.1	5.9	3.7	2.3	6.5	11.1
	15	13.4	7.7	7.6	4.6	3.3	7.4	11.4

* swl = +7.0 ft

Table 4

Wave Heights for Plans 9 and 10*

Test Wave Period sec	Test Wave Height ft	Height, ft, Gage Number 1	2	3	4	5	6	7
		Plan 9, 220 deg						
17	4	4.2	3.0	3.4	2.0	1.3	4.6	6.3
		Plan 10, 270 deg						
8	6	5.9	2.5	2.8	0.9	0.9	4.0	6.8
10	6	6.0	2.5	2.8	0.9	0.9	4.2	7.8
	9	9.0	3.9	4.1	1.3	2.0	5.5	10.0
	12	12.0	5.0	4.9	2.0	2.6	5.9	9.8
13	6	6.0	3.0	3.0	1.2	1.2	3.6	8.9
	9	8.7	4.3	3.8	2.0	2.2	6.2	10.7
	12	11.2	5.3	5.9	2.6	3.0	7.2	10.9
	15	13.8	6.4	7.2	3.9	4.0	7.8	10.8
16	6	6.0	3.5	3.8	1.7	1.4	4.5	9.8
	9	8.8	4.9	5.1	2.5	2.0	6.0	11.0
	12	11.7	6.2	6.7	3.4	3.1	7.6	11.0
	15	14.8	7.8	8.5	4.9	4.1	8.2	11.1
		Plan 10, 260 deg						
8	6	6.2	2.5	2.9	1.6	1.2	4.3	7.5
10	6	6.1	2.9	3.2	1.9	1.5	4.4	8.2
	9	8.9	3.9	4.2	2.4	2.2	5.4	9.2
	12	11.9	5.0	5.3	2.9	2.8	6.0	9.7
13	6	5.7	3.3	3.6	1.9	1.4	4.2	8.9
	9	8.1	4.7	5.1	2.4	1.8	6.1	9.8
	12	10.3	5.4	6.2	3.5	2.4	6.7	10.4
	15	13.2	6.6	7.3	3.9	3.2	7.2	10.6
16	6	5.8	3.3	3.5	2.4	1.2	3.9	9.2
	9	8.2	4.8	4.8	3.5	1.8	6.1	10.3
	12	11.0	6.5	6.4	3.9	2.6	7.4	10.6
	15	13.6	7.6	7.9	4.5	3.2	8.2	11.1

* swl = +7.0 ft

Photo 1. Typical wave patterns for existing conditions (without shoaled entrance); 10-sec, 12-ft test waves from 270 deg

Photo 2. Typical wave patterns for existing conditions (without shoaled entrance); 13-sec, 9-ft test waves from 270 deg

Photo 3. Typical wave patterns for existing conditions (without shoaled entrance); 10-sec, 12-ft test waves from 260 deg

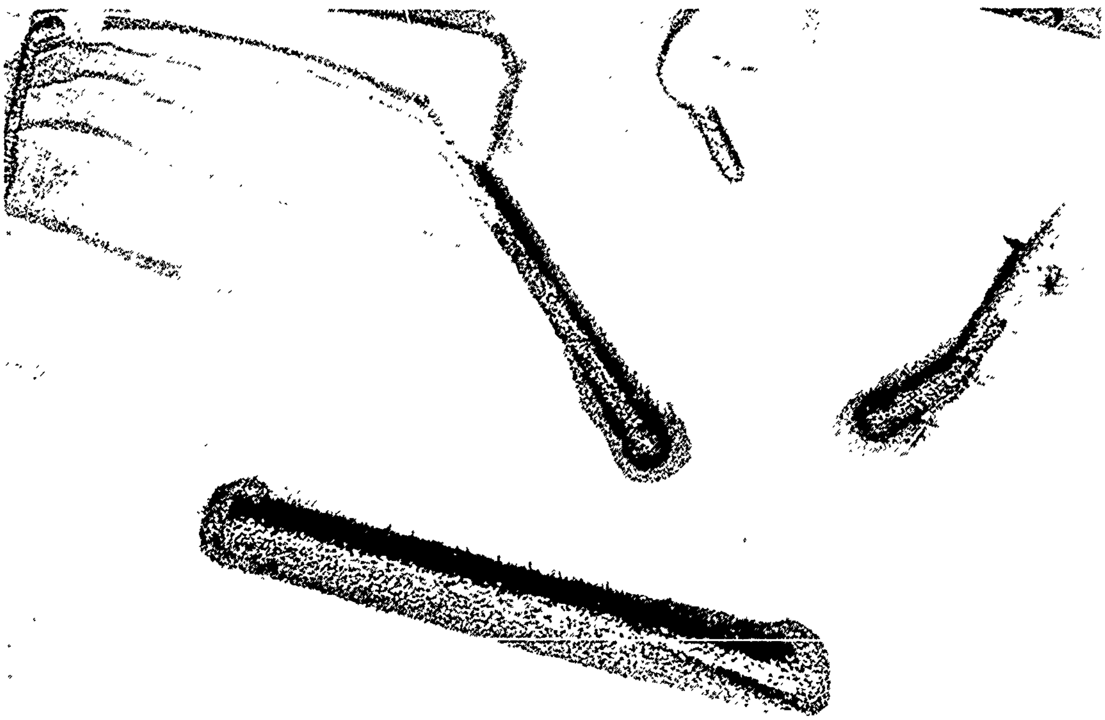


Photo 4. Typical wave patterns for existing conditions (without shoaled entrance); 13-sec, 9-ft test waves from 260 deg

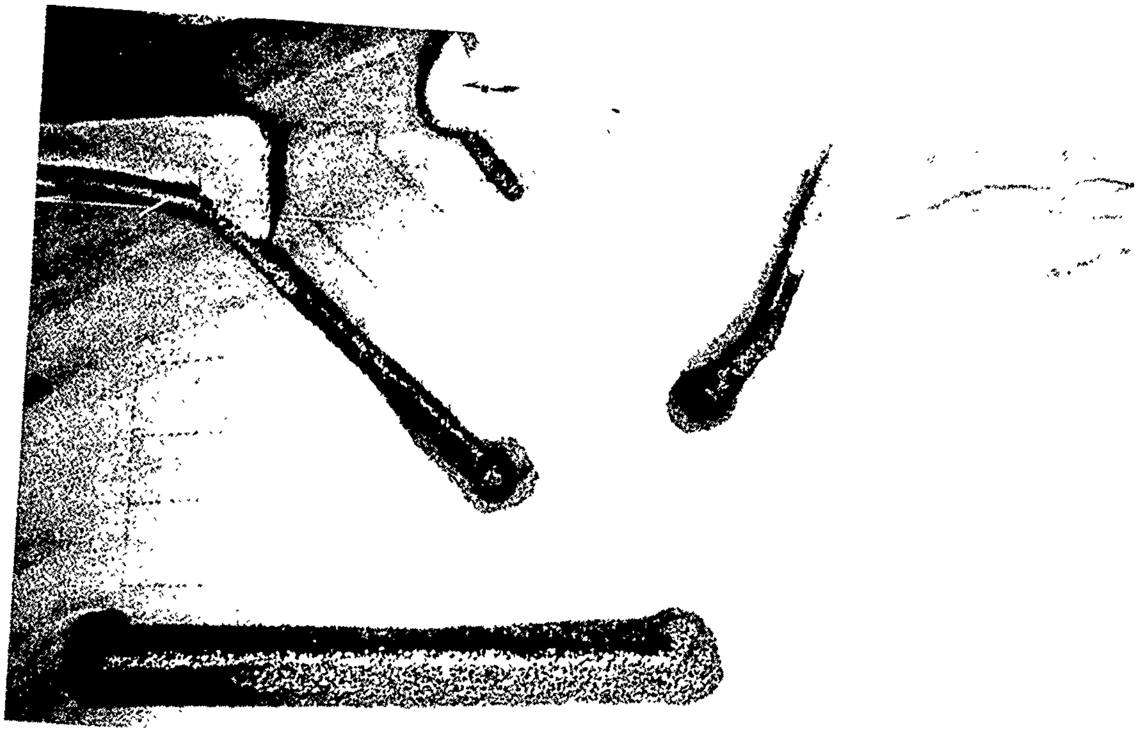


Photo 5. Typical wave patterns for existing conditions (without shoaled entrance); 17-sec, 4-ft test waves from 220 deg

Photo 6. Typical wave patterns for existing conditions (without shoaled entrance); 11-sec, 12-ft test waves from 280 deg

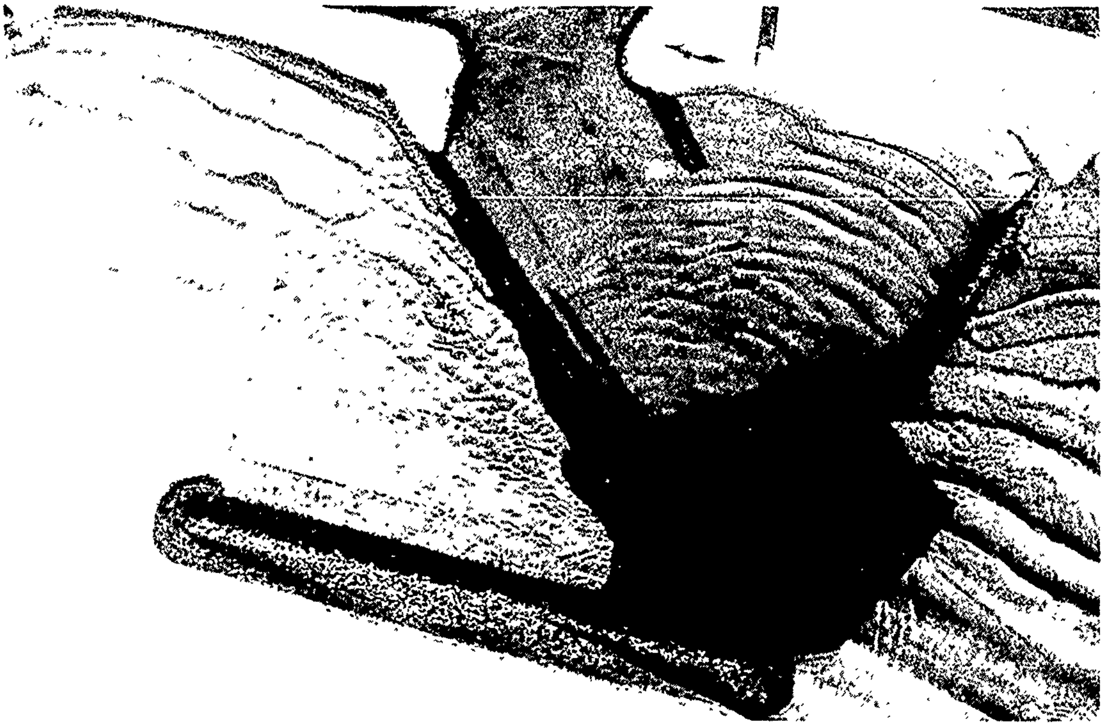


Photo 7. Typical wave patterns for existing conditions (with shoaled entrance); 10-sec, 12-ft test waves from 270 deg

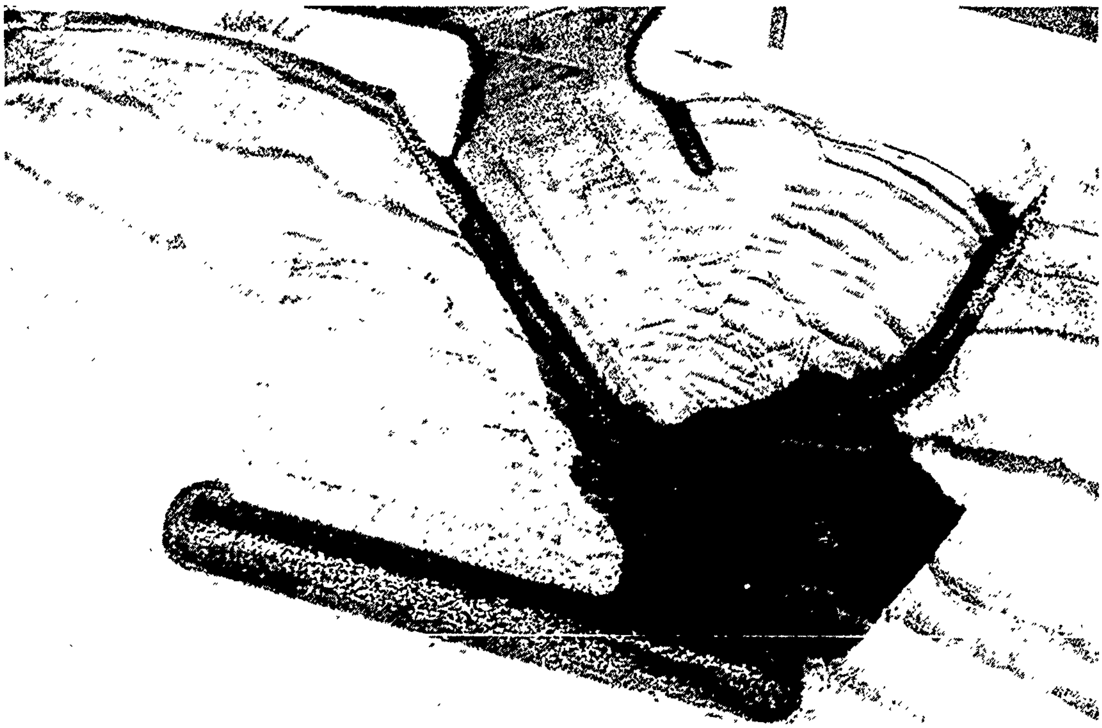


Photo 8. Typical wave patterns for existing conditions (with shoaled entrance); 13-sec, 9-ft test waves from 270 deg

Photo 9. Typical wave patterns for existing conditions (with shoaled entrance); 10-sec, 12-ft test waves from 260 deg

Photo 10. Typical wave patterns for existing conditions (with shoaled entrance); 13-sec, 9-ft test waves from 260 deg

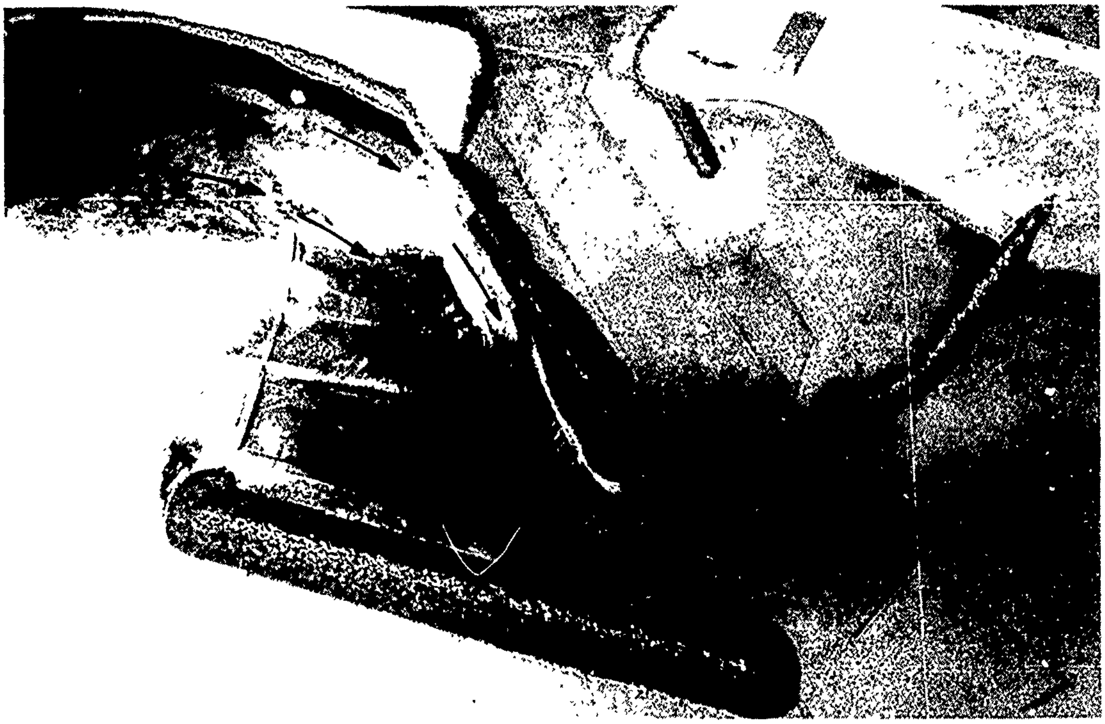


Photo 11. General movement of tracer material and subsequent deposits for existing conditions for test waves from 280 deg

Photo 12. General movement of tracer material and subsequent deposits for existing conditions for test waves from 220 deg

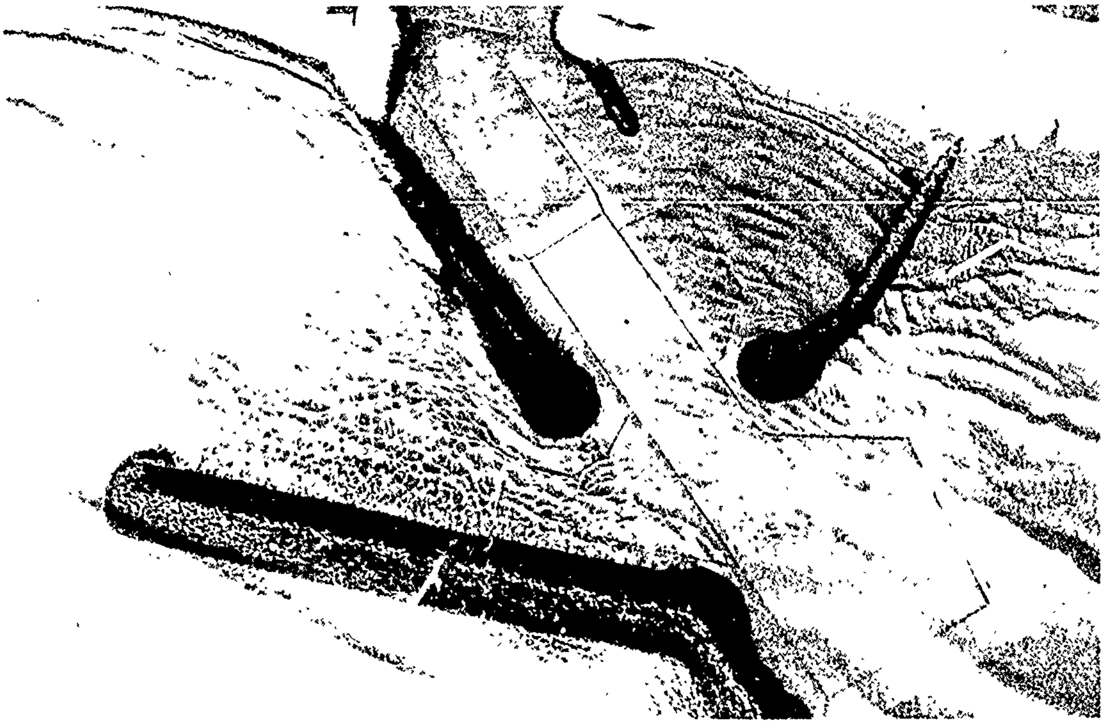


Photo 13. Typical wave patterns for Plan 1; 10-sec, 12-ft test waves from 270 deg

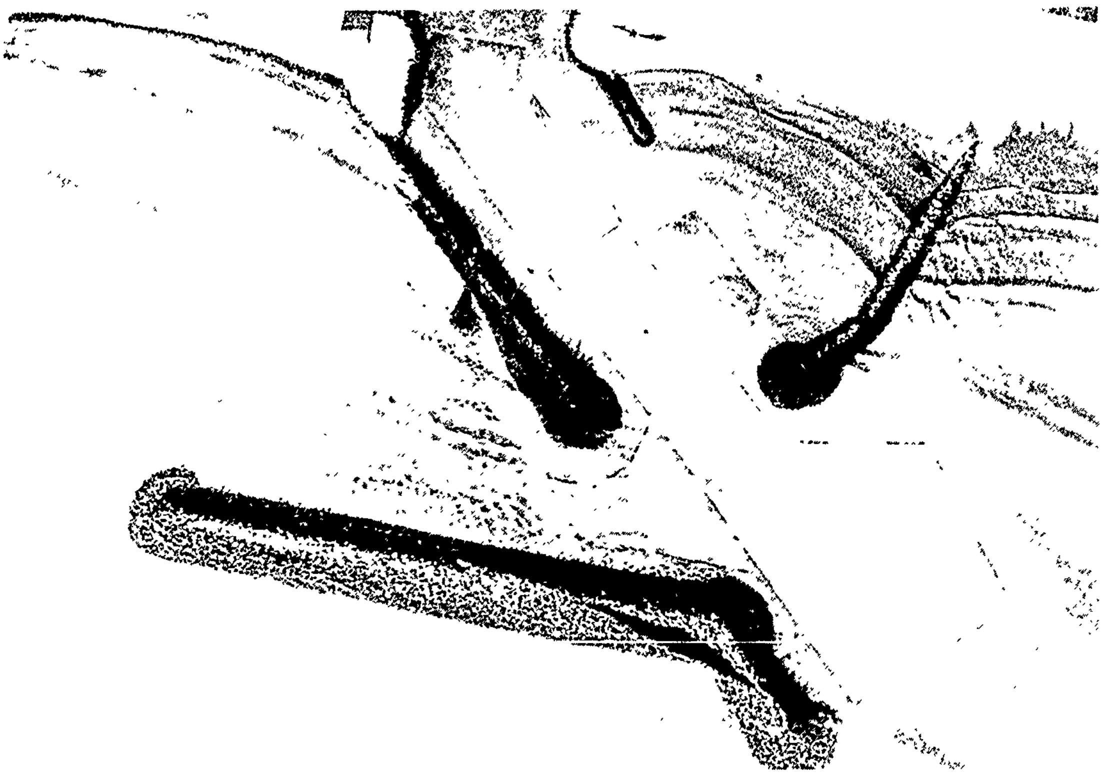


Photo 14. Typical wave patterns for Plan 1; 13-sec, 9-ft test waves from 270 deg

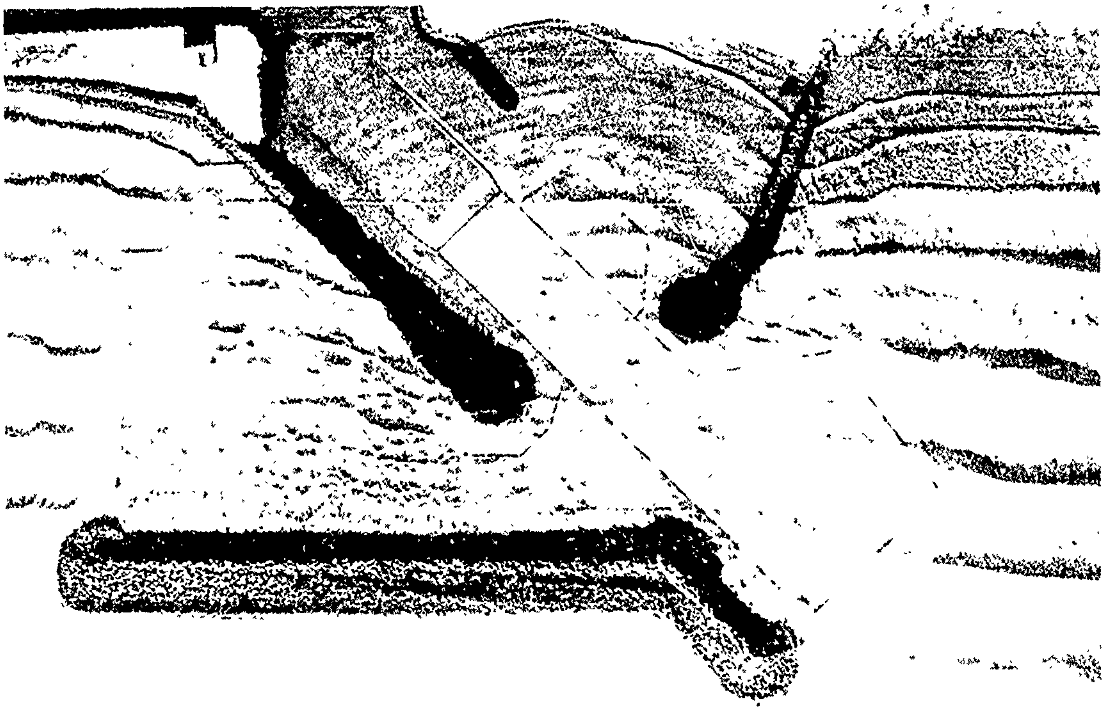


Photo 15. Typical wave patterns for Plan 1; 10-sec, 12-ft test waves from 260 deg

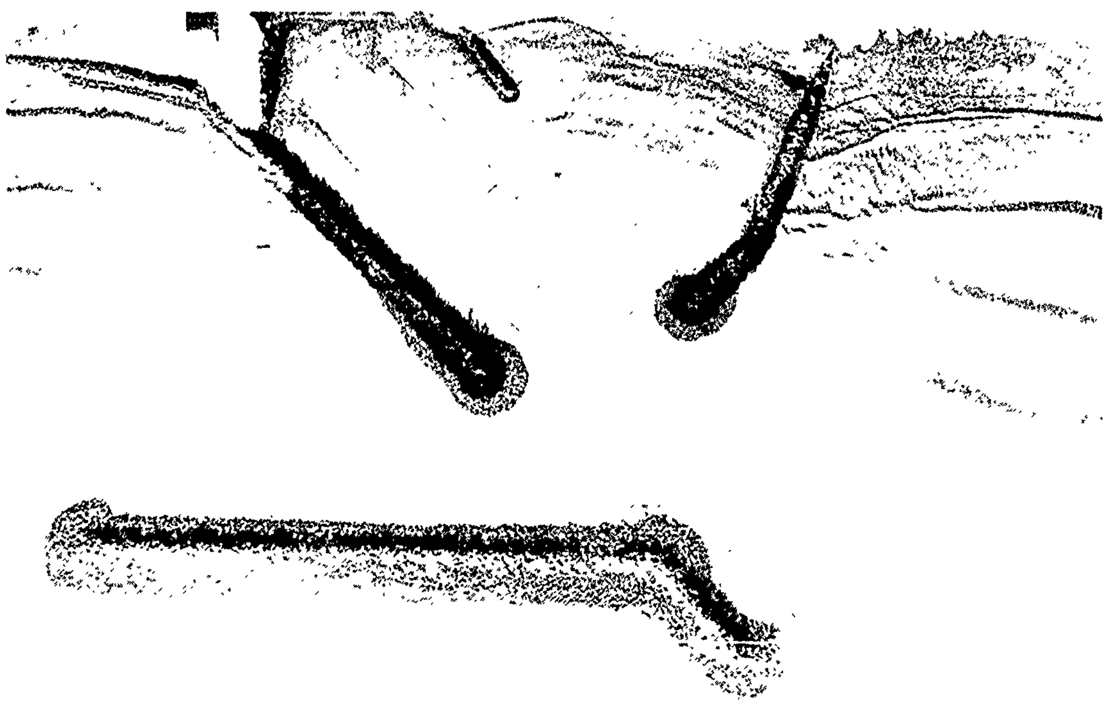


Photo 16. Typical wave patterns for Plan 1; 13-sec, 9-ft test waves from 260 deg

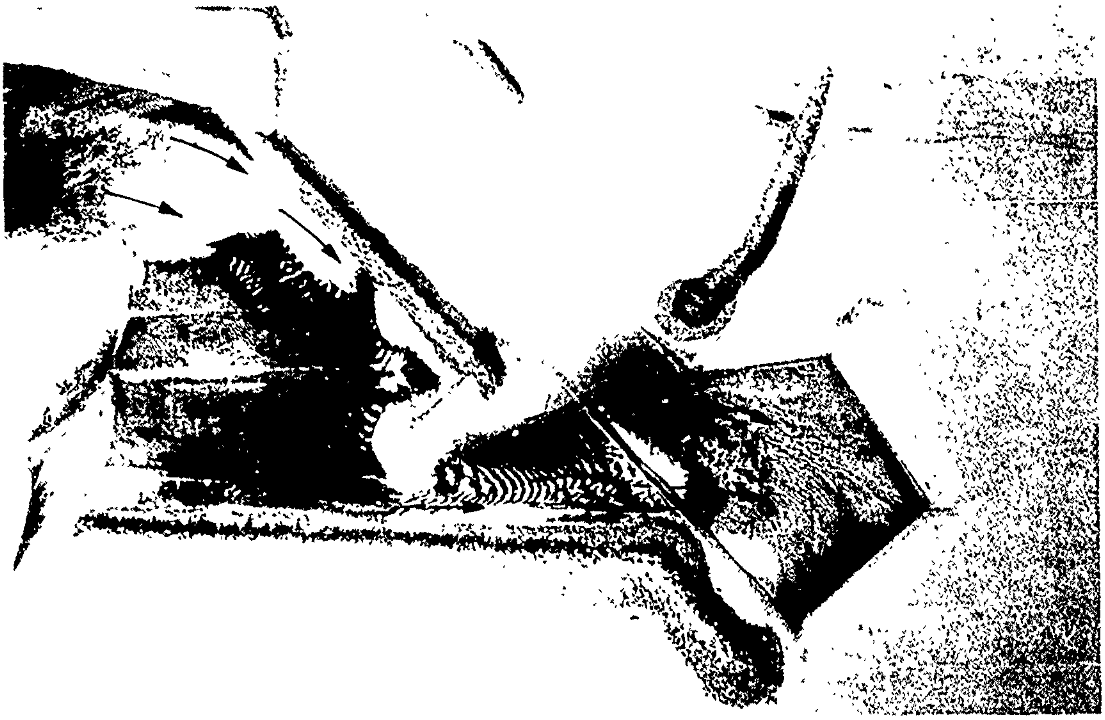


Photo 17. General movement of tracer material and subsequent deposits for Plan 2 for test waves from 280 deg

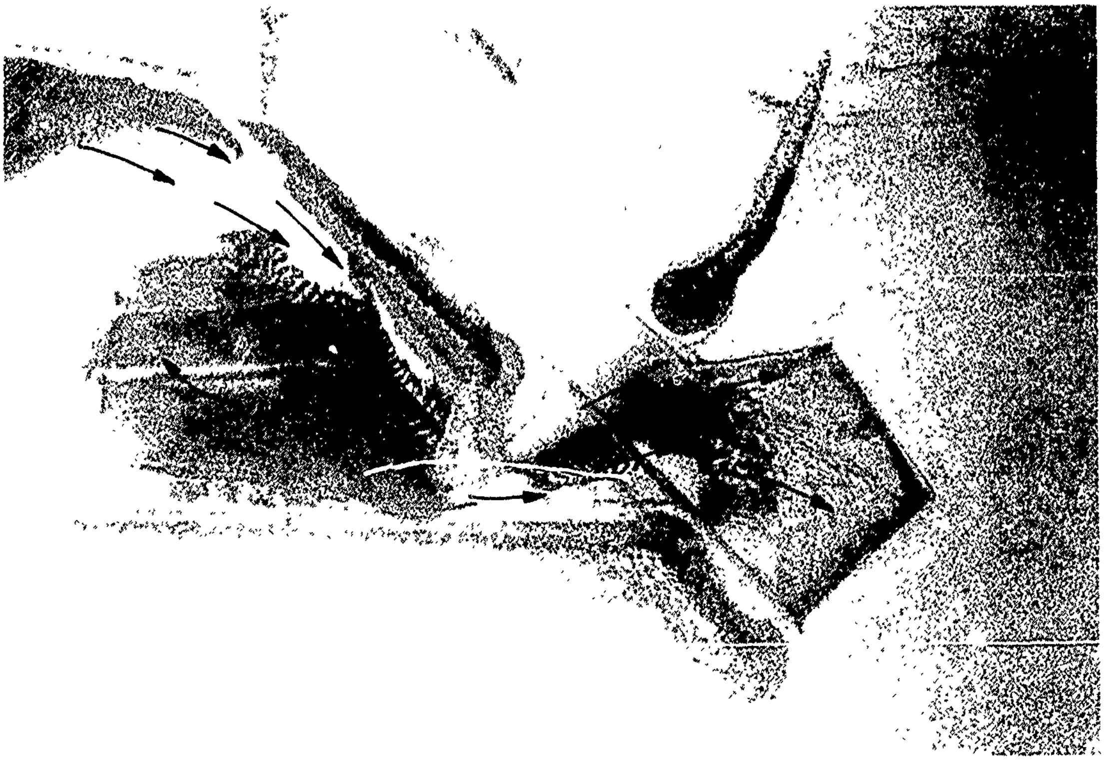


Photo 18. General movement of tracer material and subsequent deposits for Plan 3 for test waves from 280 deg

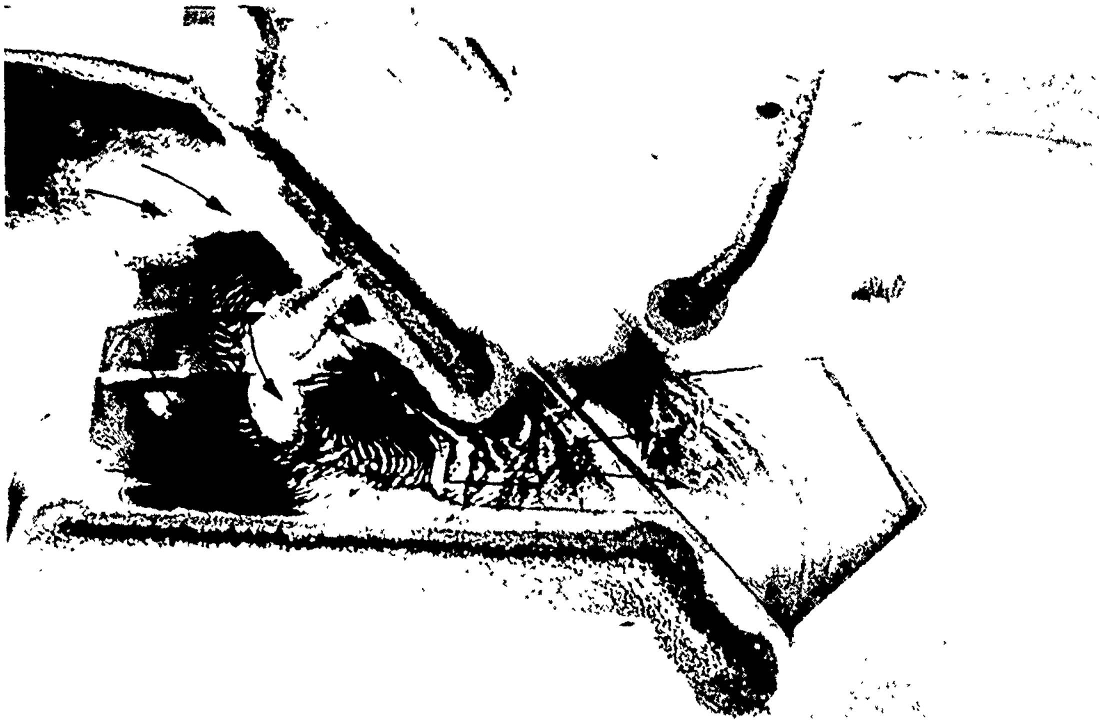


Photo 19. General movement of tracer material and subsequent deposits for Plan 4 for test waves from 280 deg

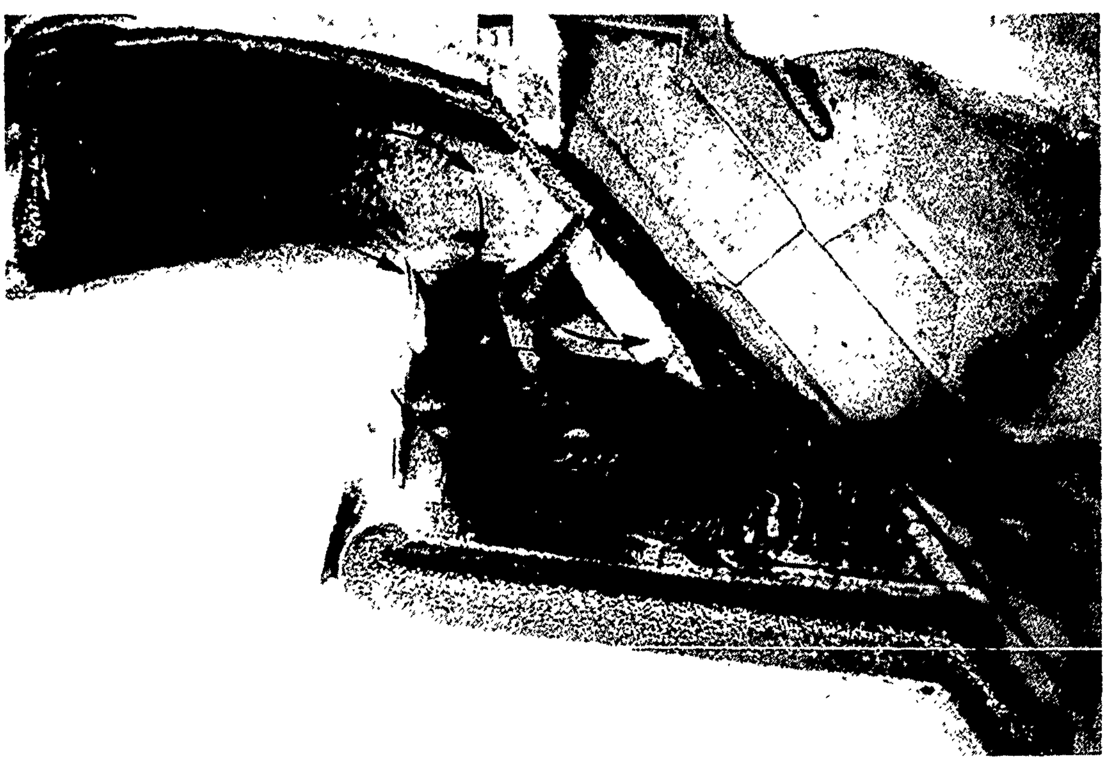


Photo 20. General movement of tracer material and subsequent deposits for Plan 5 for test waves from 280 deg

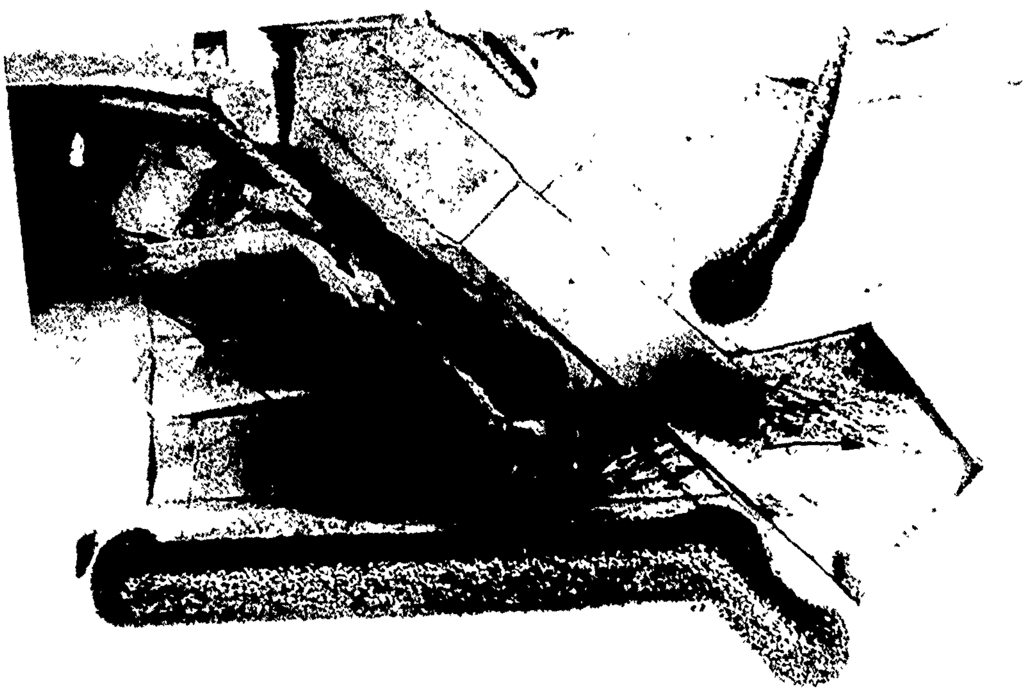


Photo 21. General movement of tracer material and subsequent deposits for Plan 6 for test waves from 280 deg

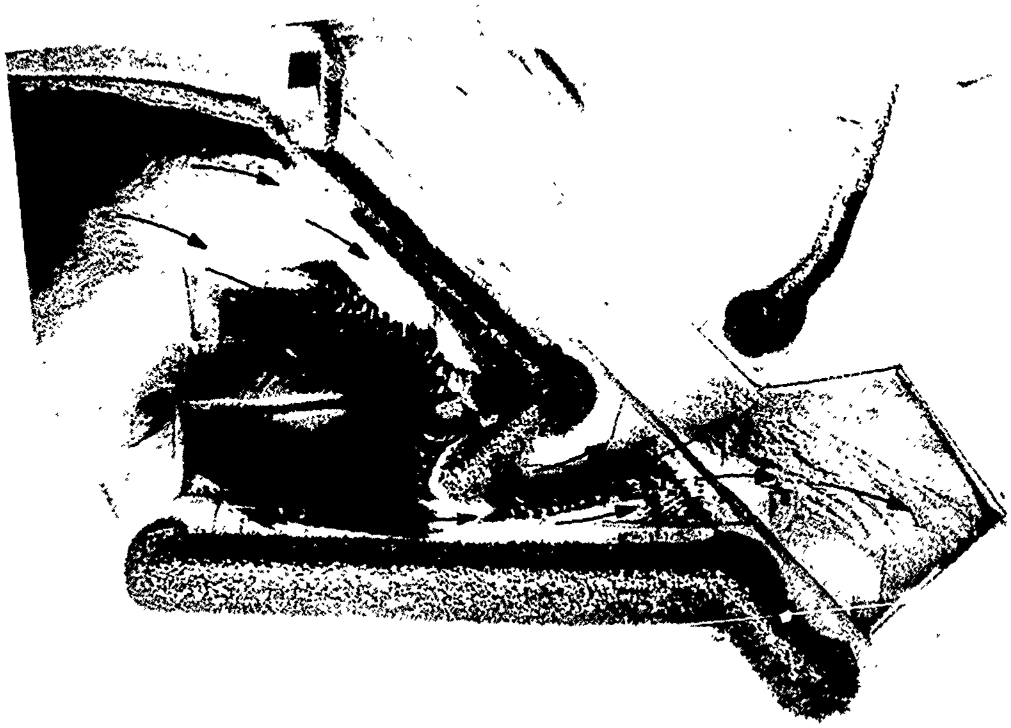


Photo 22. General movement of tracer material and subsequent deposits for Plan 7 for test waves from 280 deg

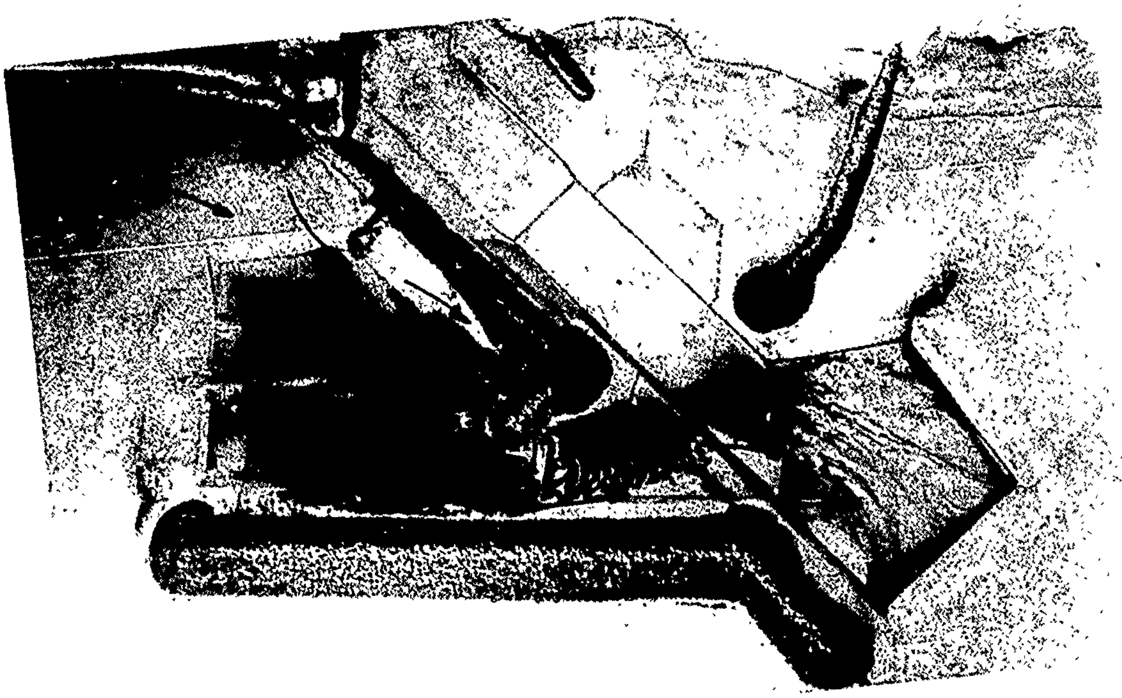


Photo 23. General movement of tracer material and subsequent deposits for Plan 8 for test waves from 280 deg

Photo 24. General movement of tracer material and subsequent deposits for Plan 9 for test waves from 220 deg

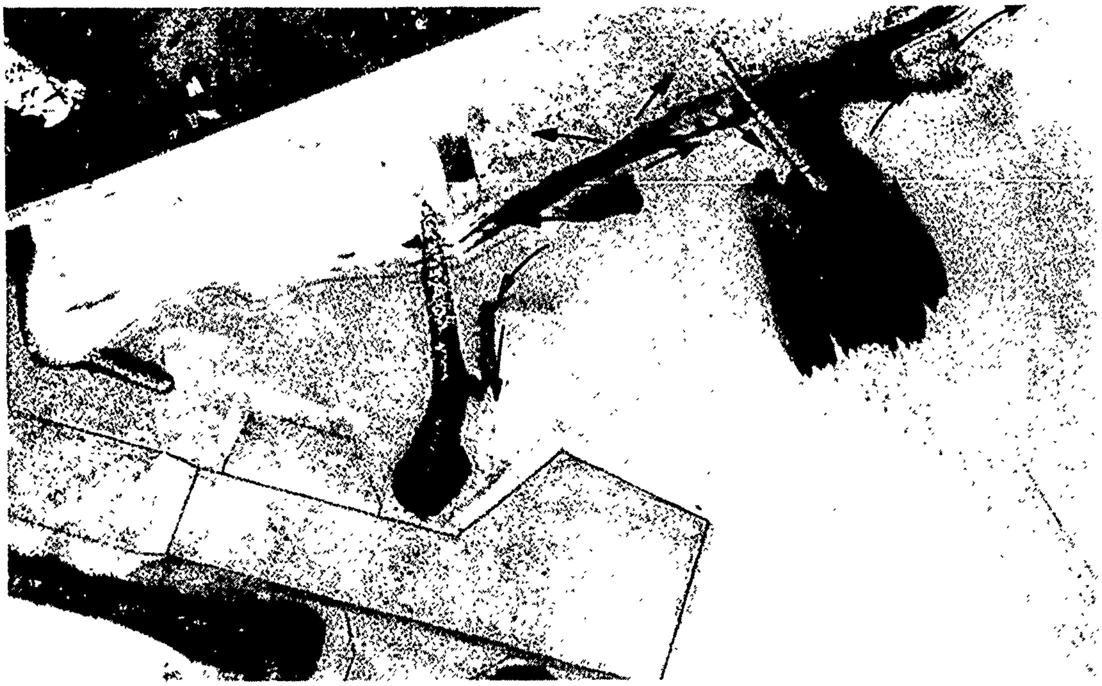


Photo 25. General movement of tracer material and resulting deposits obtained for Plan 9 after attack by 17-sec, 4-ft test waves from 220 deg and 9-sec, 4.5-ft test waves from 270 deg

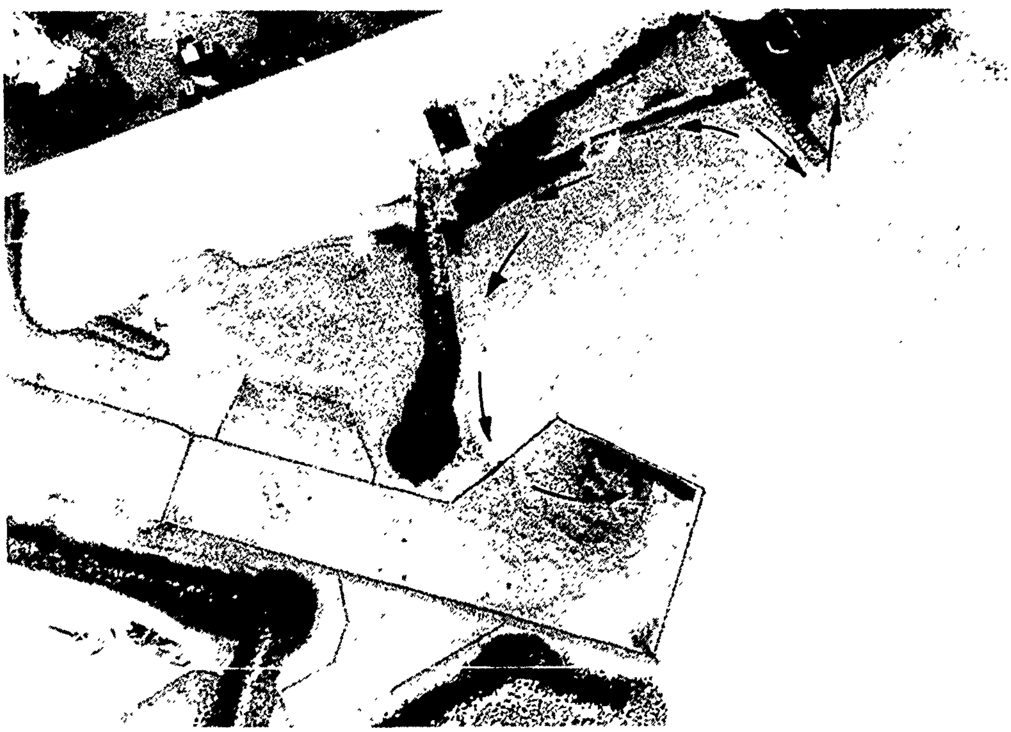


Photo 26. General movement of tracer material and resulting deposits obtained for Plan 9 after attack by 17-sec, 4-ft test waves from 220 deg, 9-sec, 4.5-ft test waves from 270 deg, and 11-sec, 12-ft test waves from 270 deg

Photo 27. General movement of tracer material and subsequent deposits for Plan 10 for test waves from 205 deg

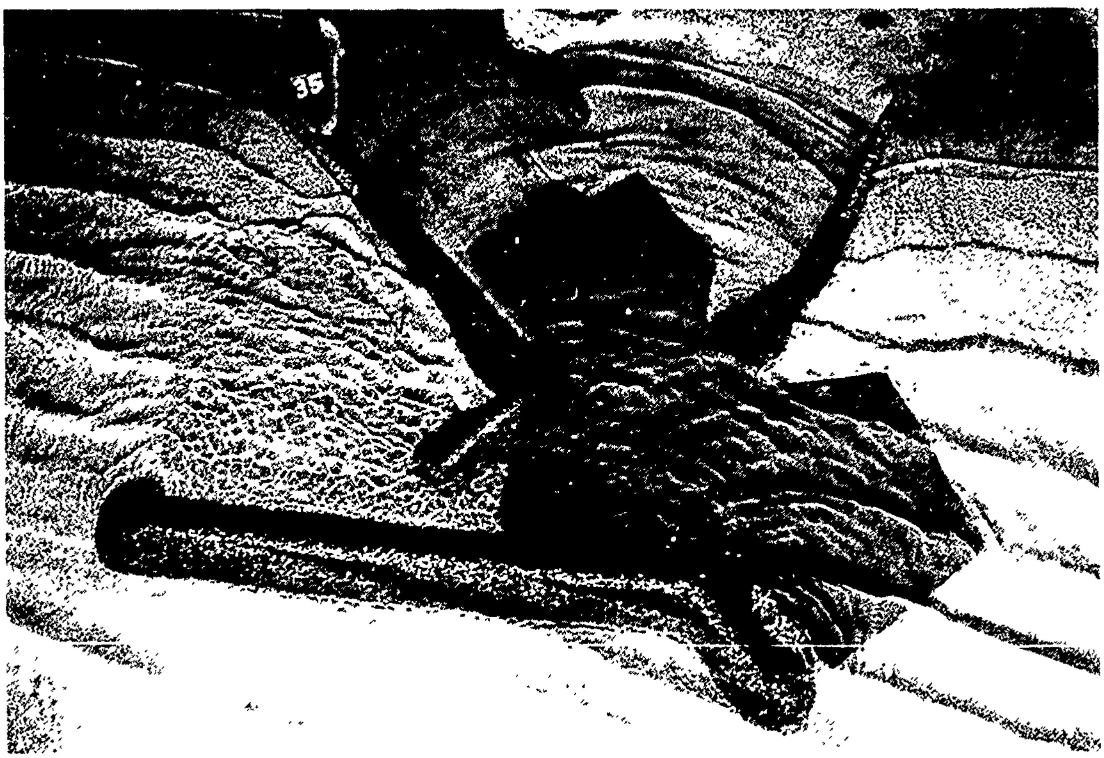


Photo 28. Typical wave patterns for Plan 10; 10-sec, 12-ft test waves from 270 deg

Photo 29. Typical wave patterns for Plan 10; 13-sec, 9-ft test waves from 270 deg

Photo 30. Typical wave patterns for Plan 10; 10-sec, 12-ft test waves from 260 deg

Photo 31. Typical wave patterns for Plan 10; 13-sec, 9-ft test waves from 260 deg

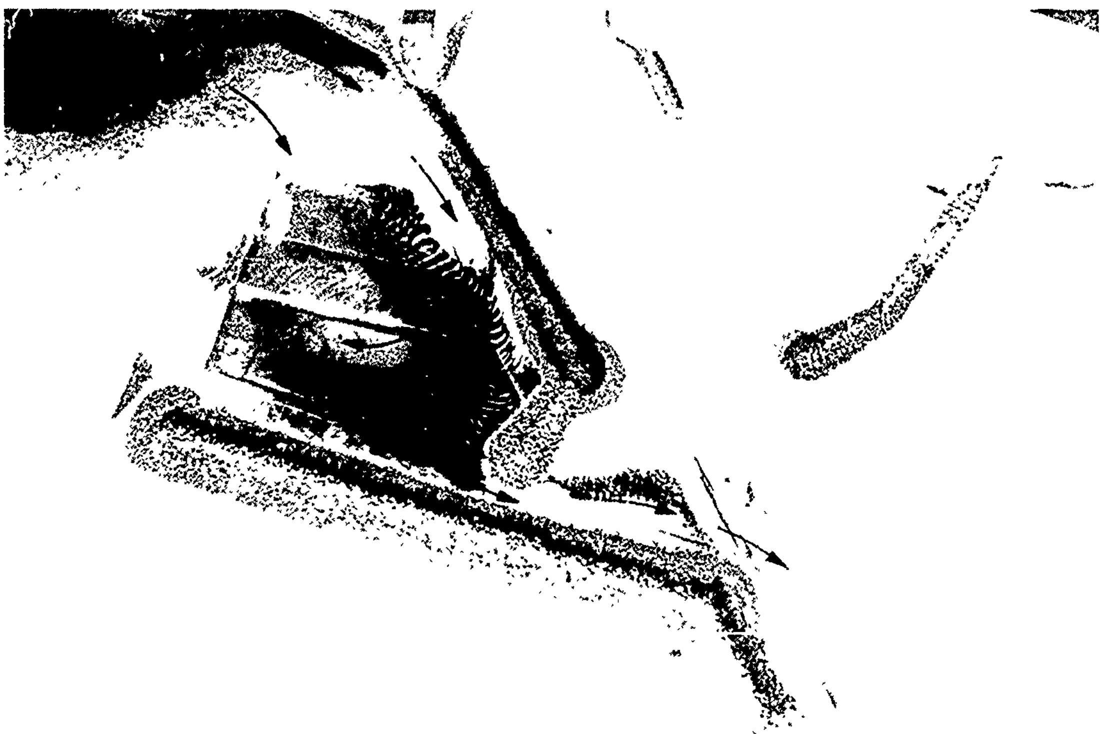
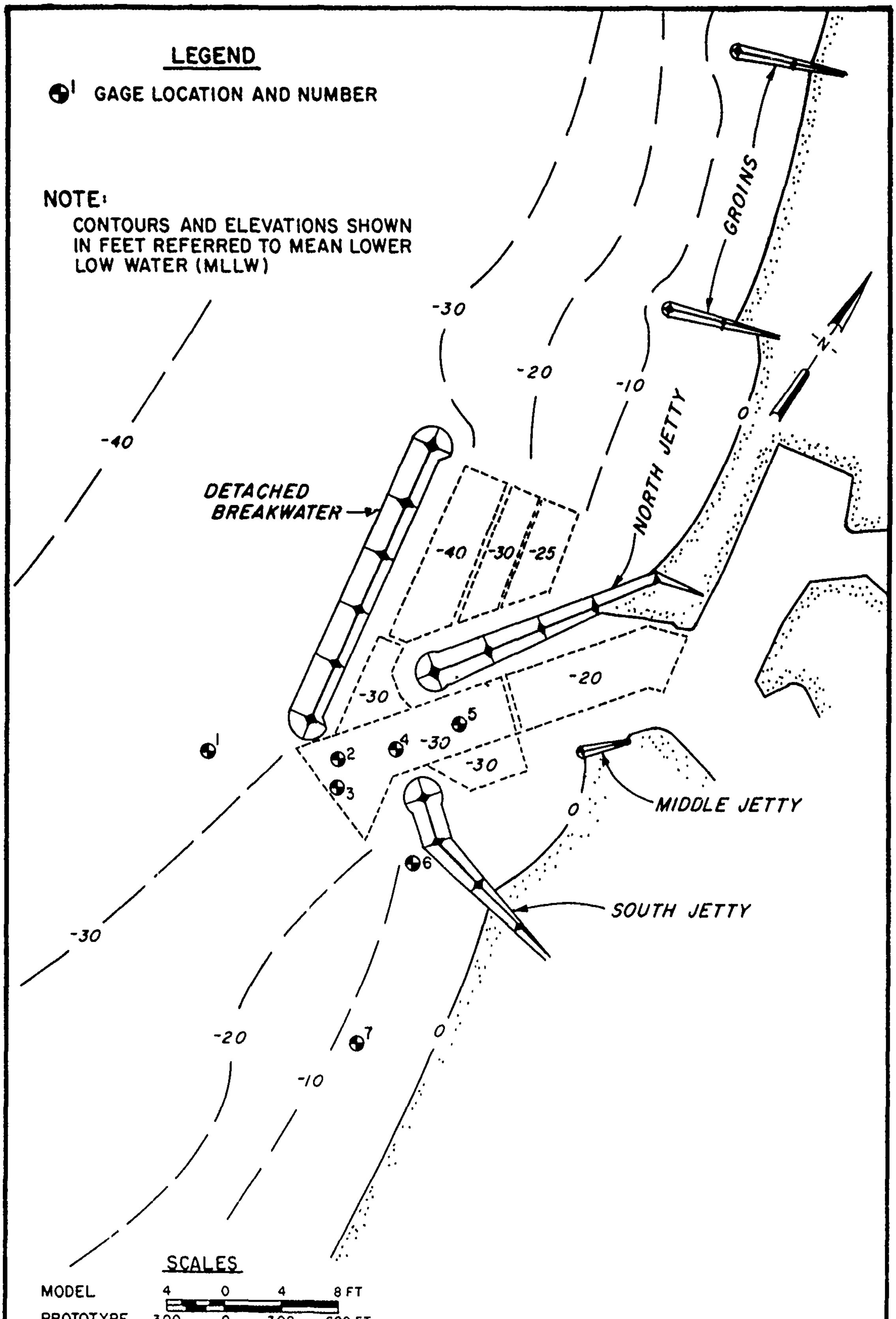


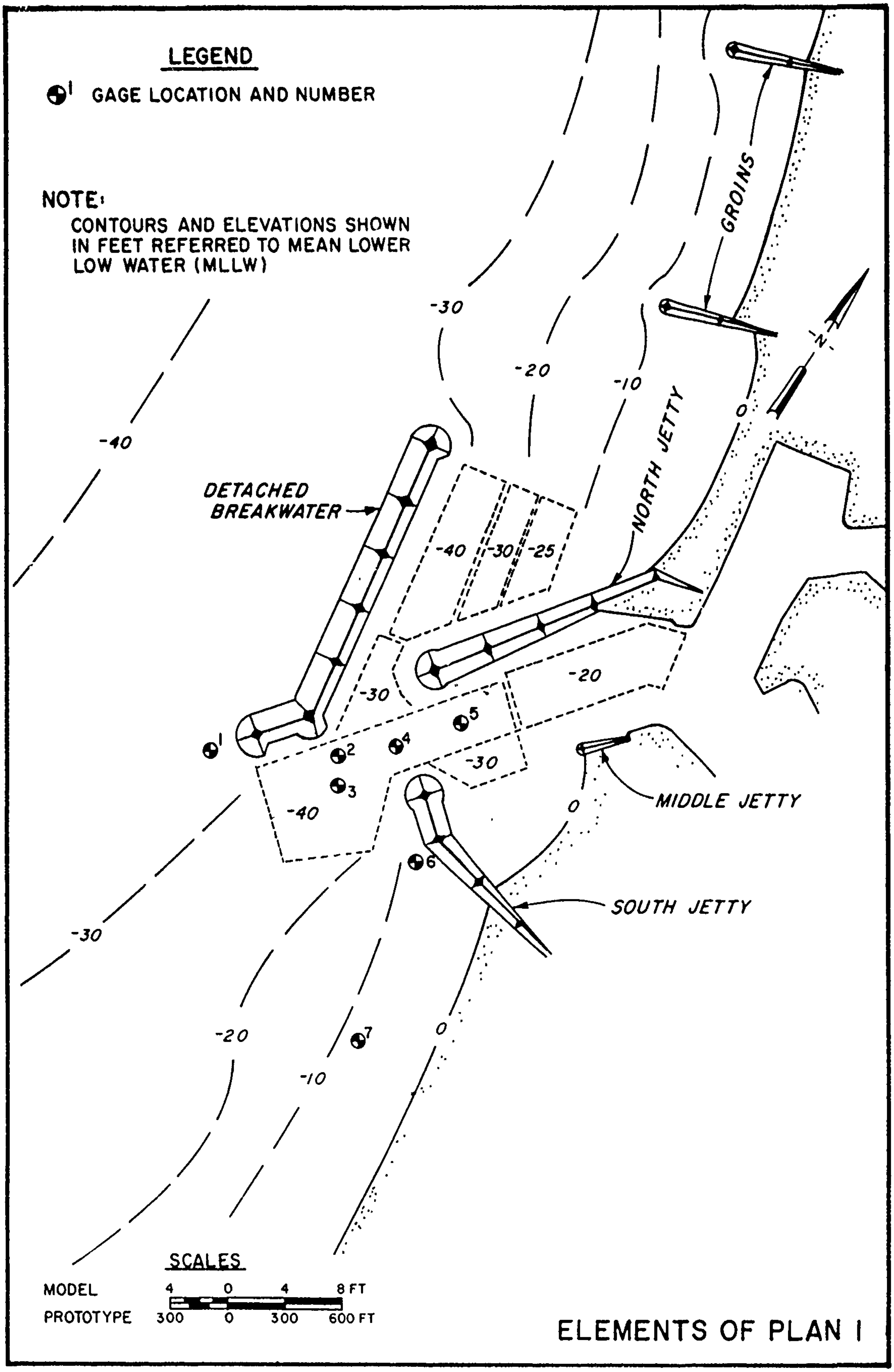
Photo 32. General movement of tracer material and subsequent deposits for Plan 11 for test waves from 280 deg

Photo 33. Partially filled deposition basin for Plan 11 prior to testing

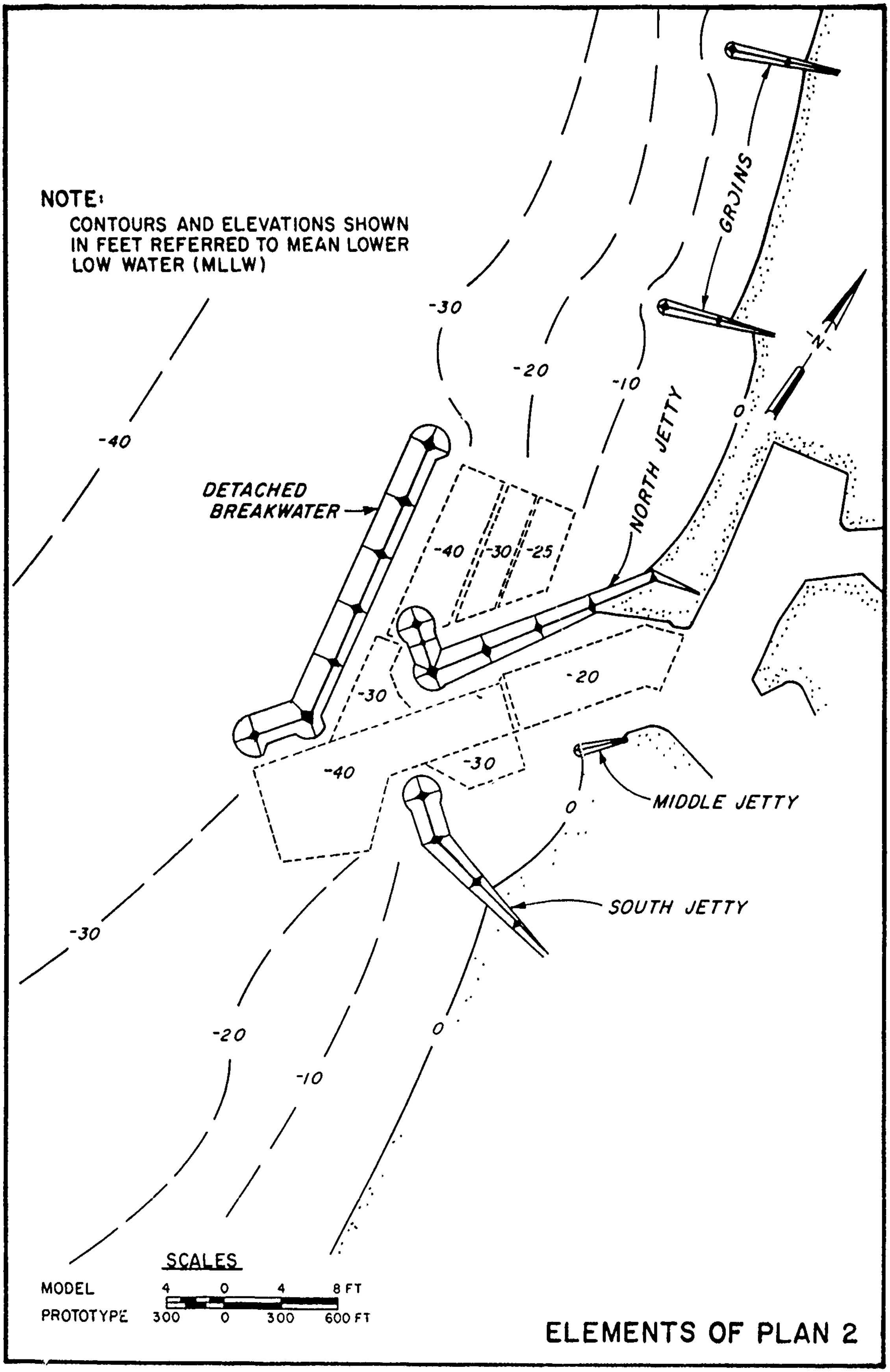
Photo 34. General movement of tracer material and subsequent deposits for Plan 11 for test waves from 280 deg with partially filled deposition basin



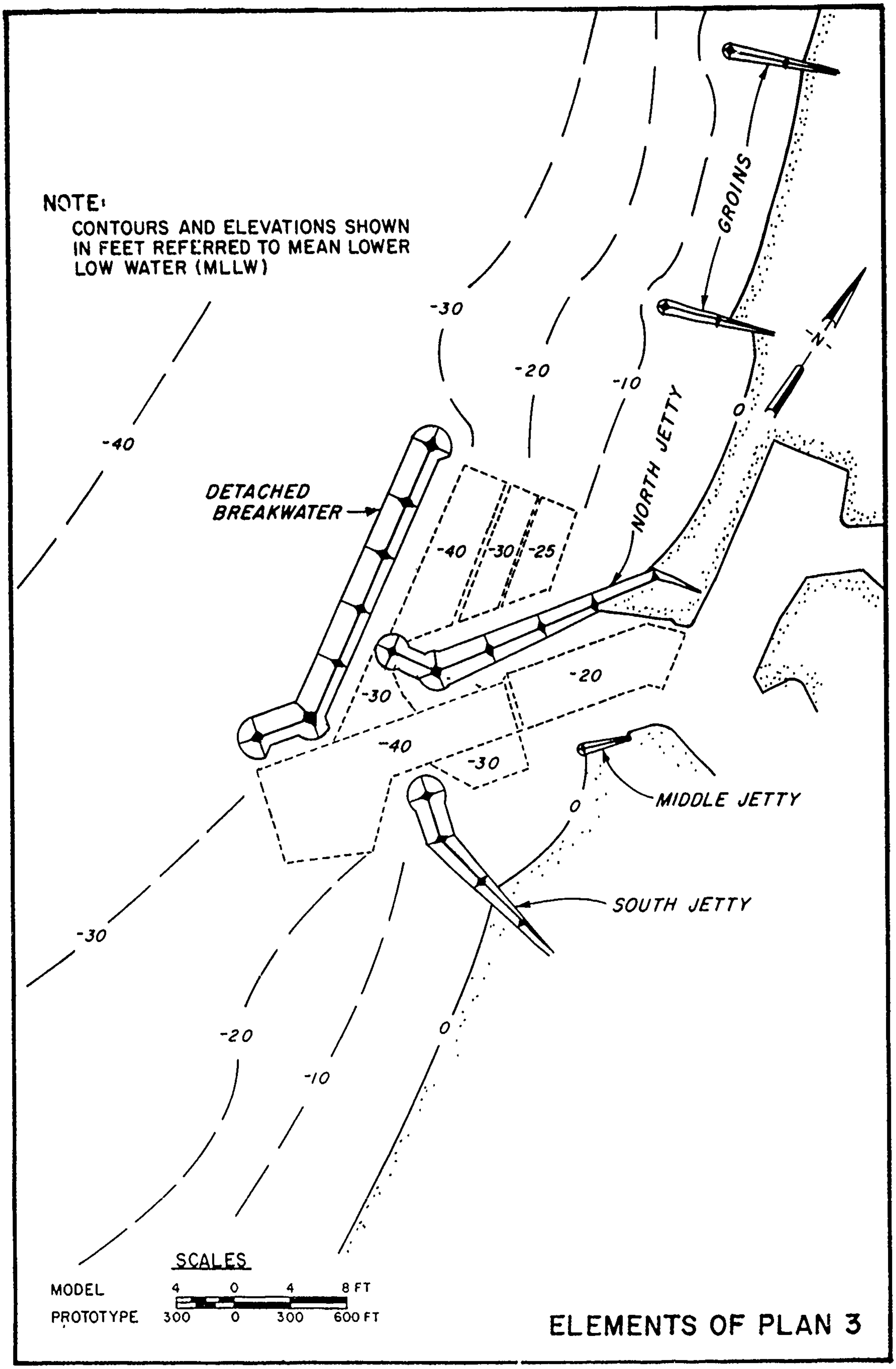
EXISTING CONDITIONS



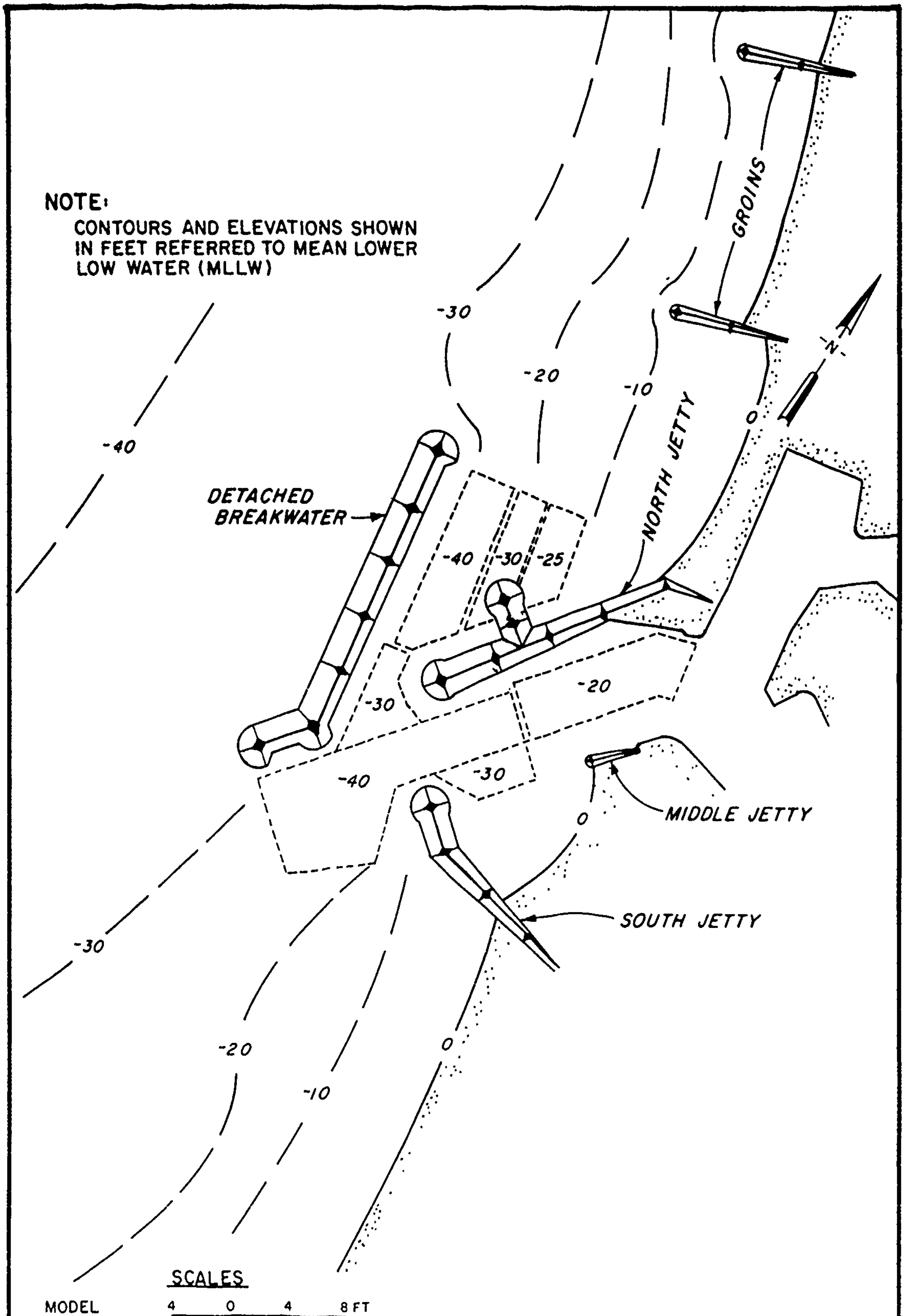
ELEMENTS OF PLAN I



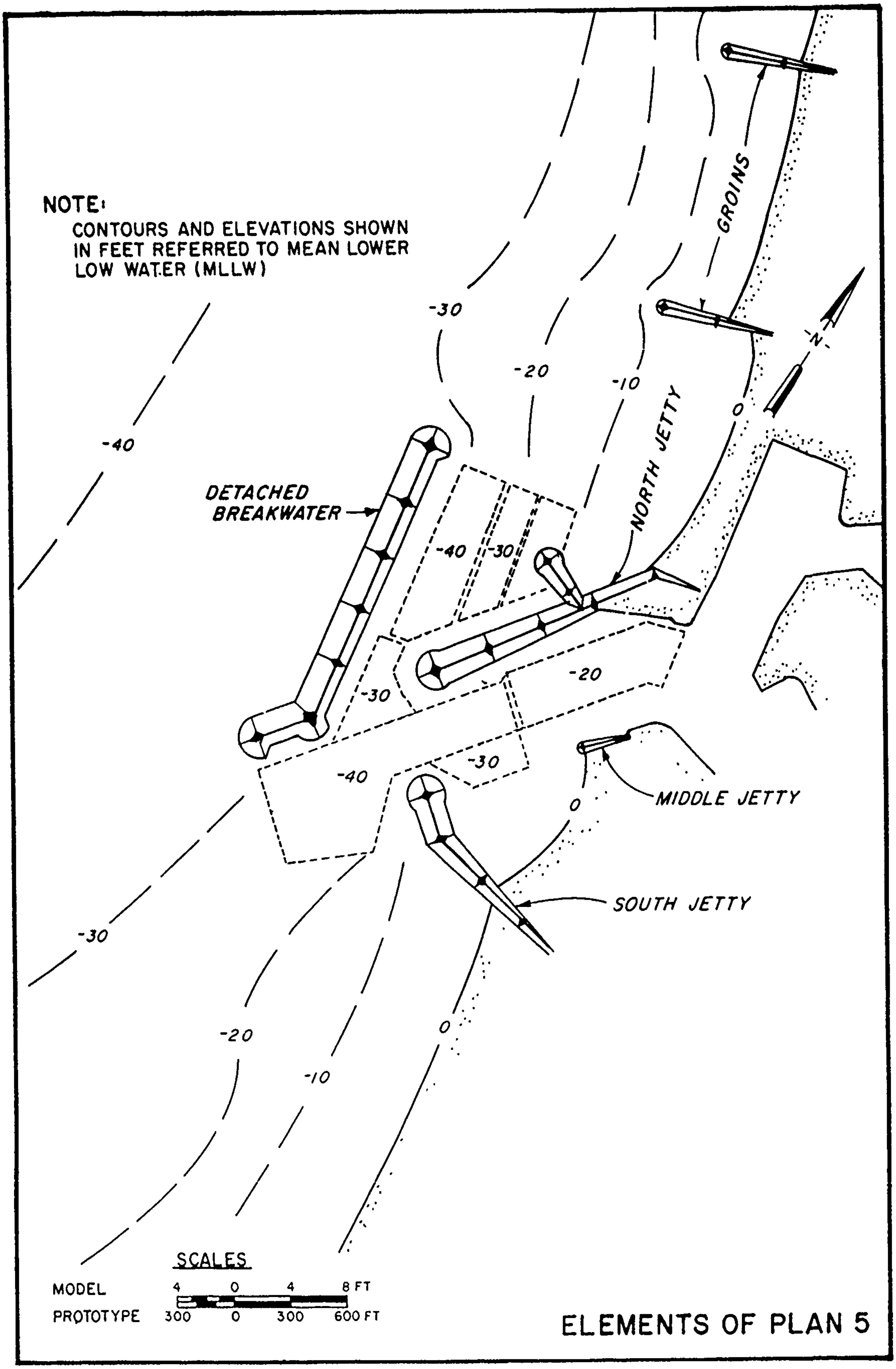
ELEMENTS OF PLAN 2



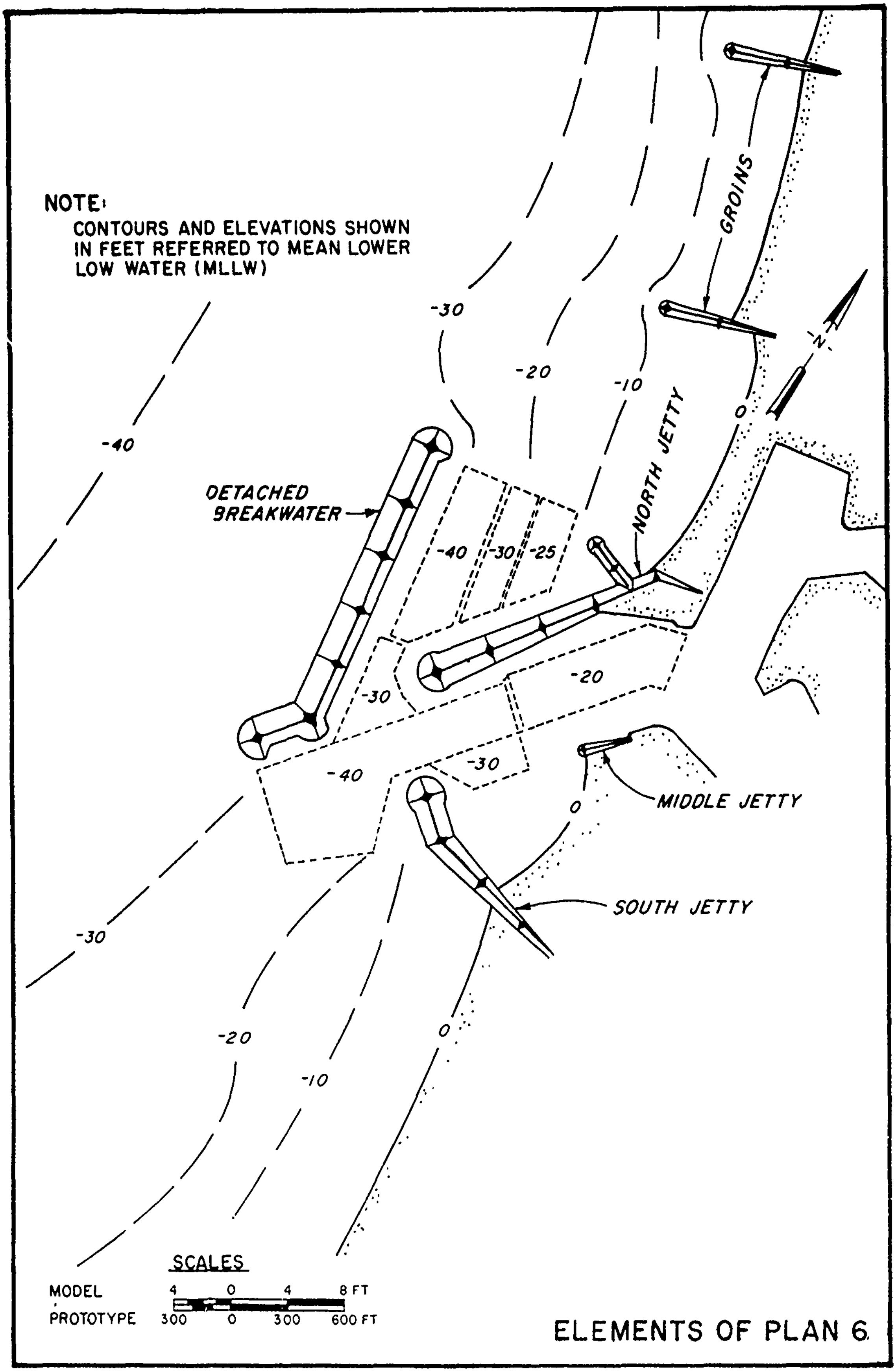
ELEMENTS OF PLAN 3



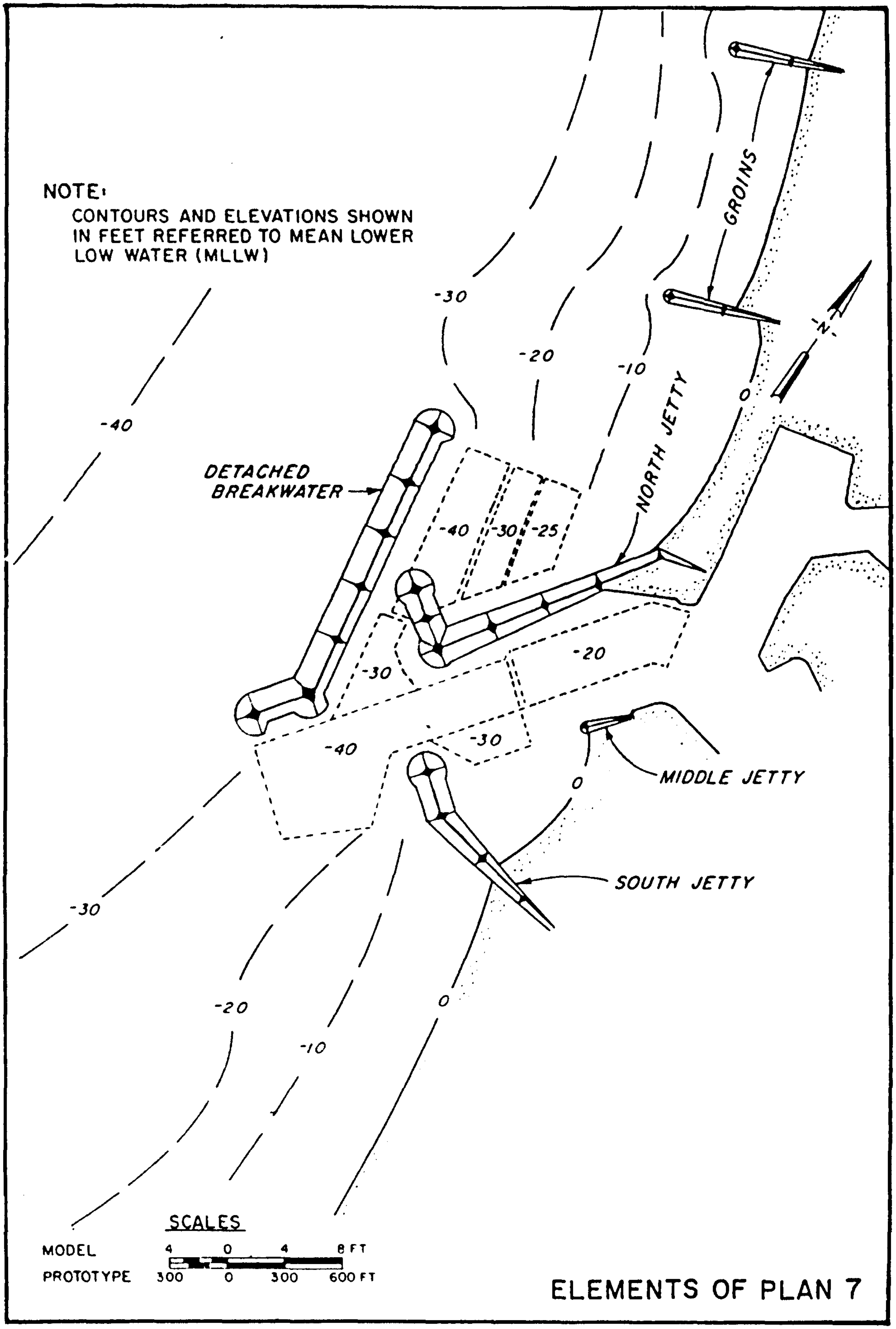
ELEMENTS OF PLAN 4

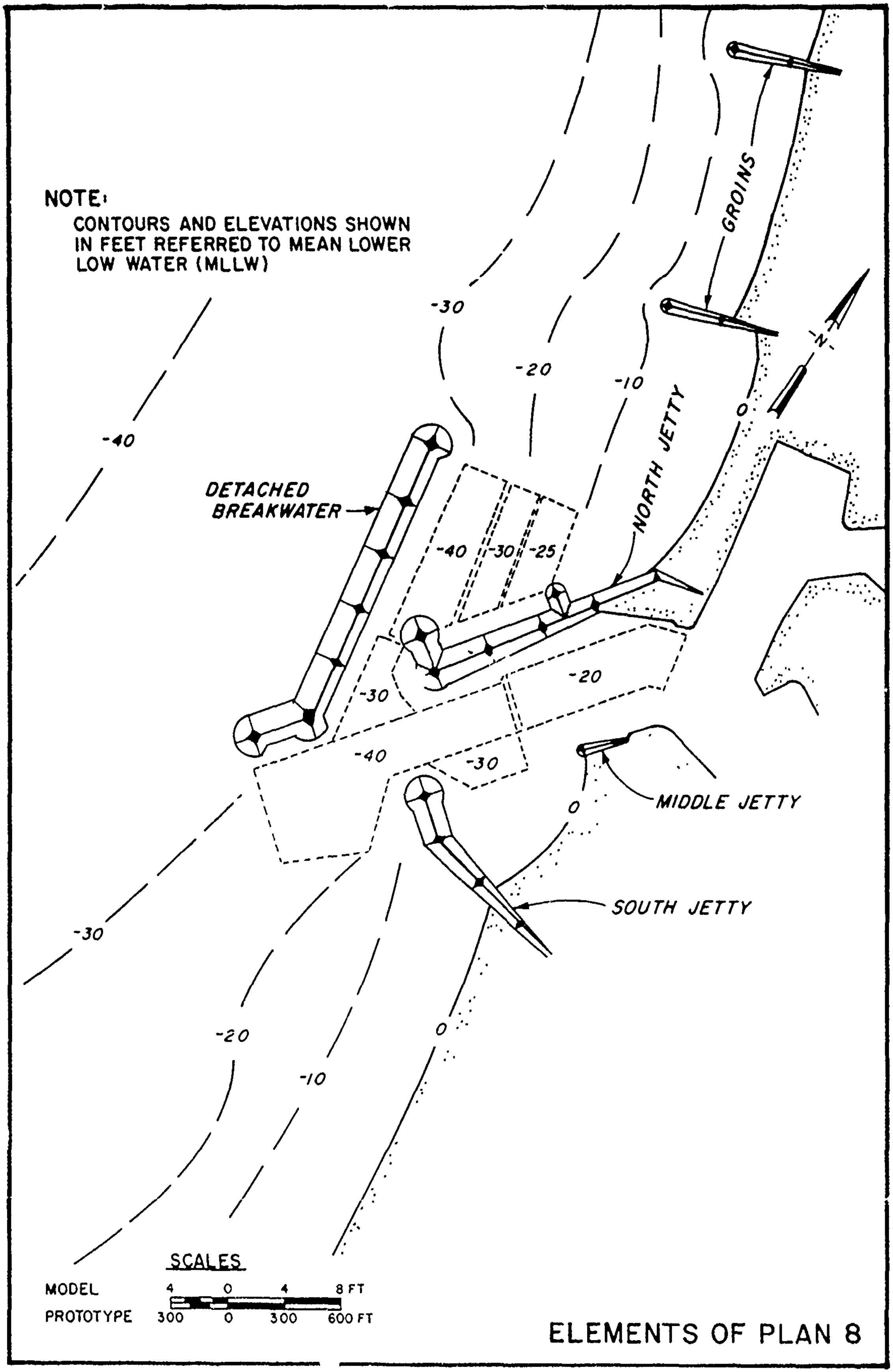


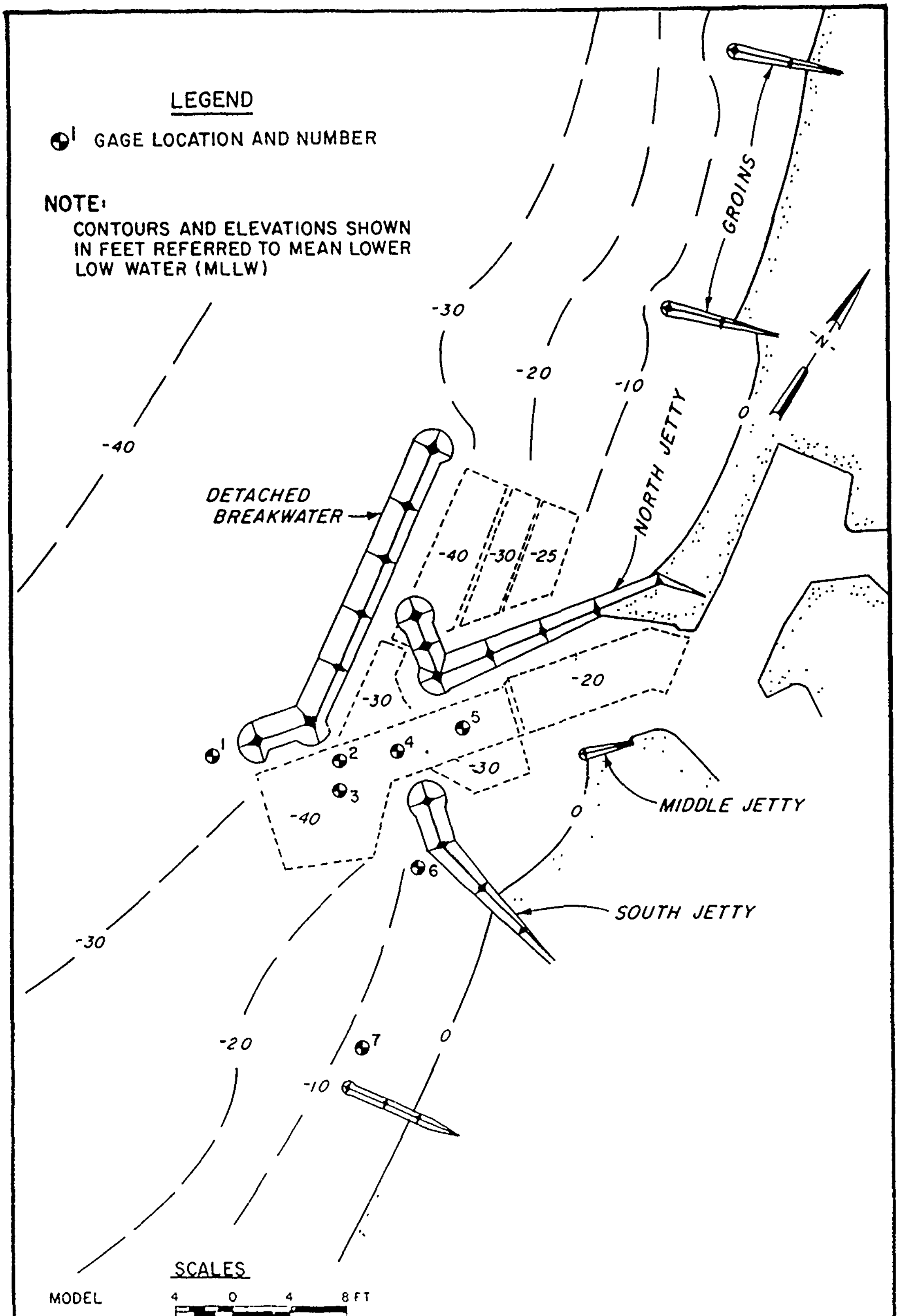
ELEMENTS OF PLAN 5



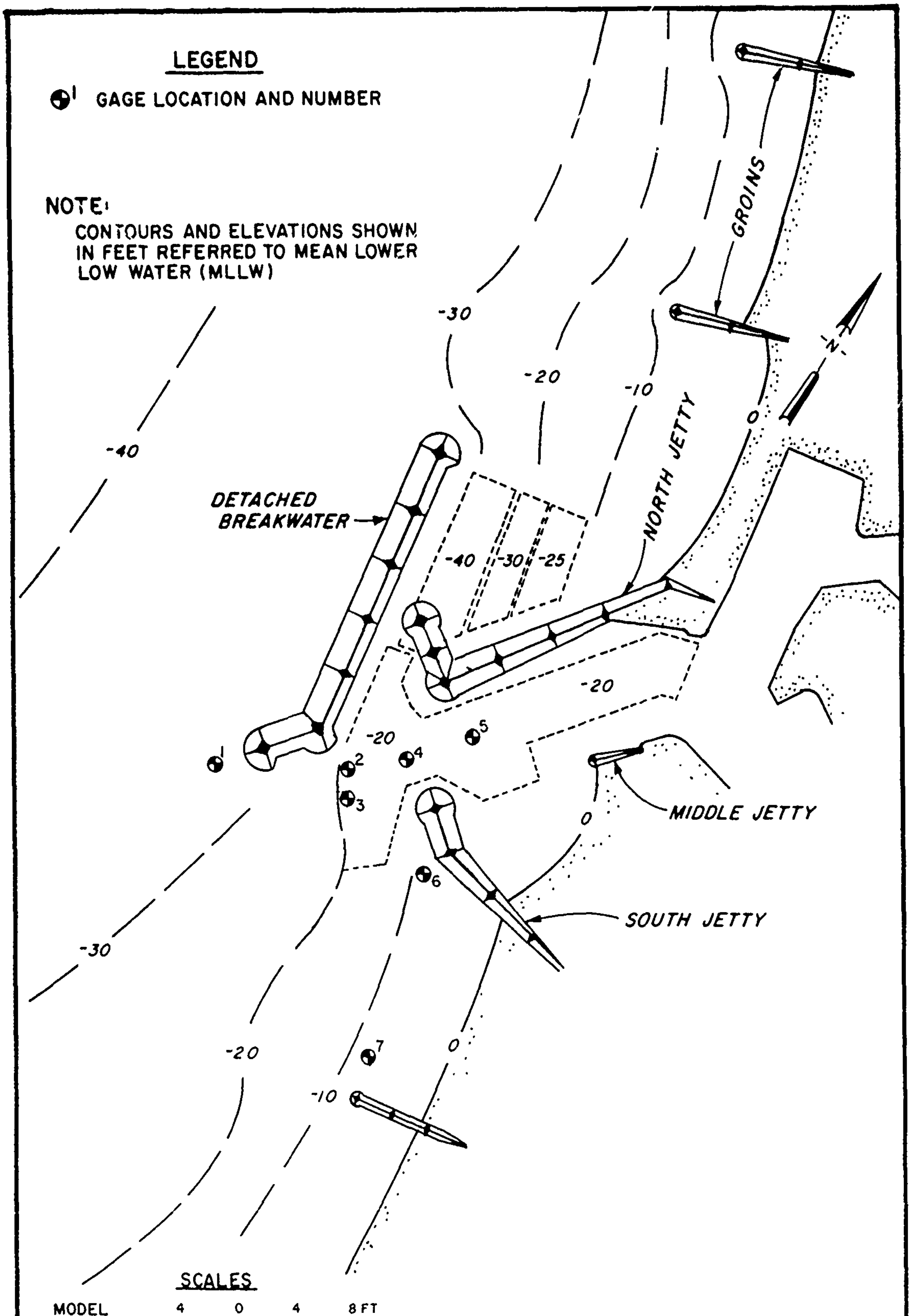
ELEMENTS OF PLAN 6


NOTE:
CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO MEAN LOWER LOW WATER (MLLW)
GROINS
N
-30
-20
-10
0
-40
NORTH JETTY
DETACHED BREAKWATER
-40
-30
-25
-20
-30
-40
-30
MIDDLE JETTY
0
SOUTH JETTY
-30
-20
-10
0
SCALES
MODEL 4 0 4 8 FT
PROTOTYPE 300 0 300 600 FT
ELEMENTS OF PLAN 7

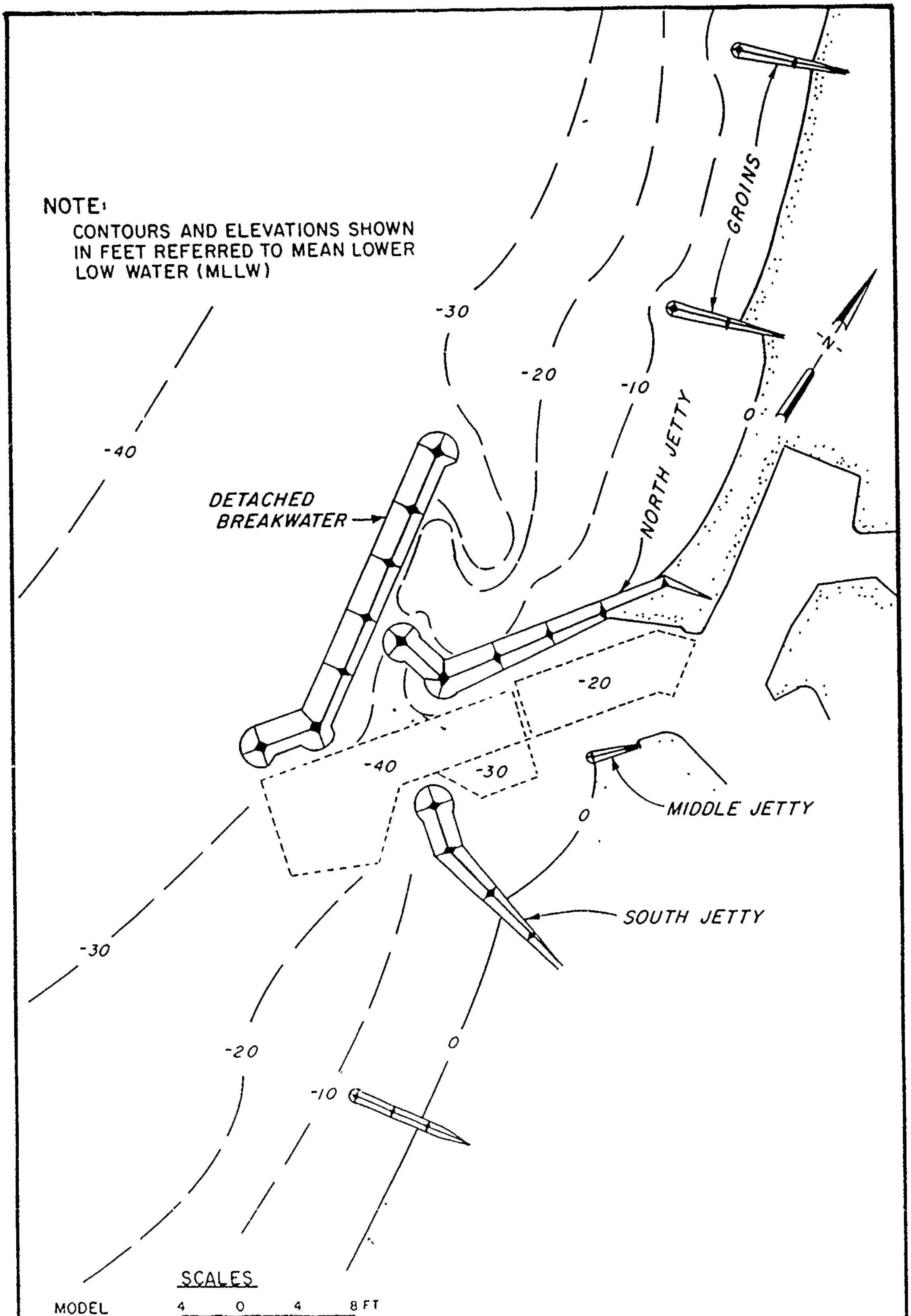

NOTE:
CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO MEAN LOWER LOW WATER (MLLW)
GROINS
N
-30
-20
-10
0
-40
NORTH JETTY
DETACHED BREAKWATER
-40
-30
-25
-30
-20
-40
-30
MIDDLE JETTY
0
SOUTH JETTY
-30
-20
-10
0
SCALES
MODEL 4 0 4 8 FT
PROTOTYPE 300 0 300 600 FT
ELEMENTS OF PLAN 8



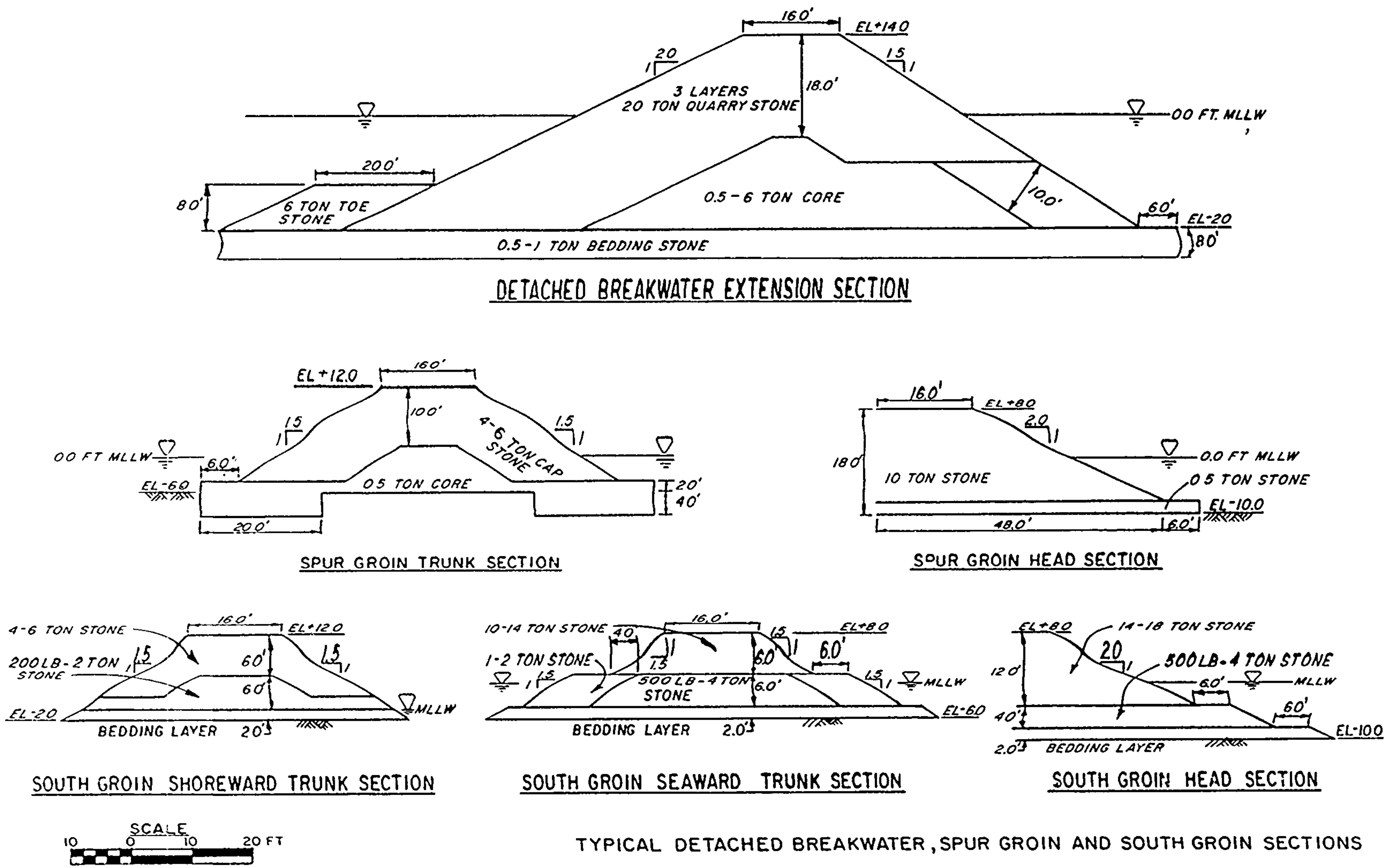
ELEMENTS OF PLAN 9



ELEMENTS OF PLAN 10



ELEMENTS OF PLAN II



TYPICAL DETACHED BREAKWATER, SPUR GROIN AND SOUTH GROIN SECTIONS