TECHNICAL REPORT HL-80-10

OCEANSIDE HARBOR AND BEACH, CALIFORNIA DESIGN OF STRUCTURES FOR HARBOR IMPROVEMENT AND BEACH EROSION CONTROL

Hydraulic Model Investigation

by

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June 1980

Final Report

Approved For Public Release; Distribution Unlimited

Prepared for U. S. Army Engineer District, Los Angeles
Los Angeles, Calif. 90053

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE *(When Data Entered)*

REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
1. REPORT NUMBER Technical Report HL-80-10	2. GOVT ACCESSION NO.	3. RECIPIENT'S CATALOG NUMBER
4. TITLE *(and Subtitle)* OCEANSIDE HARBOR AND BEACH, CALIFORNIA, DESIGN OF STRUCTURES FOR HARBOR IMPROVEMENT AND BEACH EROSION CONTROL; Hydraulic Model Investigation		5. TYPE OF REPORT & PERIOD COVERED Final report 6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) Charles R. Curren Claude E. Chatham, Jr.		8. CONTRACT OR GRANT NUMBER(s)
9. PERFORMING ORGANIZATION NAME AND ADDRESS U. S. Army Engineer Waterways Experiment Station Hydraulics Laboratory P. O. Box 631, Vicksburg, Miss. 39180		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
11. CONTROLLING OFFICE NAME AND ADDRESS U. S. Army Engineer District, Los Angeles P. O. Box 2711 Los Angeles, Calif. 90053		12. REPORT DATE June 1980 13. NUMBER OF PAGES 339
14. MONITORING AGENCY NAME & ADDRESS *(if different from Controlling Office)*		15. SECURITY CLASS. *(of this report)* Unclassified 15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT *(of this Report)*

Approved for public release; distribution unlimited.

17. DISTRIBUTION STATEMENT *(of the abstract entered in Block 20, if different from Report)*

18. SUPPLEMENTARY NOTES

19. KEY WORDS *(Continue on reverse side if necessary and identify by block number)*

Beach erosion	Hydraulic models
Breakwaters	Jetties
Erosion control	Oceanside, Calif.--Harbor
Groins	Shoaling
Harbor structures	Water wave action on maritime structures
Harbors	

20. ABSTRACT *(Continue on reverse side if necessary and identify by block number)*

A 1:100 scale (undistorted) hydraulic model, reproducing Oceanside Harbor, approximately 5.7 miles of shoreline, and sufficient offshore area to permit generation of the required test waves was used to investigate the arrangement and design of proposed structures for (a) improving navigation and mooring and prevention of shoaling of Oceanside Harbor and (b) prevention of beach erosion. The proposed structures for the harbor consisted of (a) offshore breakwaters, (b) jetty extensions, (c) groins, and (d) interior breakwaters. The proposed structures for mitigation of beach erosion consisted of various groin and offshore breakwater configurations. A 190-ft-long wave generator, crushed coal tracer material, and an automated data acquisition and control system (ADACS) were used during model operation. For the harbor, it was concluded from model test results that:

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20. ABSTRACT (Continued).

a. Existing conditions are characterized by strong longshore currents for moderate to large wave conditions with considerable longshore movement of material into and past the harbor entrance. The resultant shoal at the harbor entrance created hazardous entrance wave conditions due to breaking waves. Also, wave heights in certain areas of the harbor are excessive. Significant quantities of material pass into the harbor through the voids of the north jetty.

b. The original improvement plan for prevention of harbor shoaling for Oceanside (i.e., the offshore breakwater plan of Plan 1) was ineffective in trapping material outside the harbor entrance and, in fact, contributed to the shoaling problem by trapping material in the entrance channel.

c. The original improvement plan for wave protection to the harbor expansion (i.e., the inner breakwater of Plan 25) was ineffective in reducing wave heights to the level desired.

d. Of the plans tested, Plans 4, 23, 24, 61, and 62 provided adequate prevention of harbor shoaling for all wave conditions tested. Plans 38C, 44, and 45 provided adequate prevention of harbor shoaling for waves from northerly directions and, if combined with the south jetty extension of Plan 21, would prevent shoaling for all wave conditions tested. Plan 4 required the greatest volume of rock. Plans 23 and 24 required less rock but would require bypassing large volumes of sand. Plans 38C, 44, and 45 (in conjunction with the south jetty extension of Plan 21) would require bypassing smaller volumes of sand due to the natural containment of sand by the groin. Plans 61 and 62 required the least volume of rock (no south jetty extension required) and would utilize the natural containment of sand by the groin.

e. Many of the plans tested provided marginally adequate wave protection for the harbor expansion. However, considering all aspects of this expansion (entrance conditions, wave heights in berthing areas, harbor usage, cost and methods of construction, etc.), Plan 60 is considered the optimum based on results of the model tests (i.e. without regard to other factors).

For the beach, it was concluded from model test results that:

a. Existing conditions are characterized by strong longshore currents for moderate to large wave conditions with considerable longshore transport.

b. The original improvement plan for Oceanside Beach (i.e. Plan 6, the 5-groin plan) was not effective in retaining the beach fill material and preventing erosion of the shoreline because of the spacing.

c. Plans 9, 9B, 12, 14A, and 17 are all viable alternatives. Plan 12 protected the longest section of beach (from sta 47+00 to 175+00) and, due to the number of groins, used the largest volume of rock. Plans 9, 14A, and 17 were effective in protecting the beach fill between sta 105+00 and 155+00. These plans were effective in containing the beach fill and movement of tracer within the study area. The transitional groins of Plan 9B effectively minimized effects of the groin field on adjacent beaches. Such transitional groins could also be used effectively with Plans 14A and 17.

PREFACE

A request for a model investigation of Oceanside Harbor and Beach, California, was initiated by the District Engineer, U. S. Army Engineer District, Los Angeles (SPL), in a letter to the Division Engineer, U. S. Army Engineer Division, South Pacific (SPD), and subsequent authorization was granted by the Office, Chief of Engineers, U. S. Army. Initial funds were authorized by SPL on 7 October 1977, with subsequent installments authorized through 25 June 1979.

The model study was conducted at the U. S. Army Engineer Waterways Experiment Station (WES) during the period November 1977 through July 1979 under the direction of Mr. H. B. Simmons, Chief of the Hydraulics Laboratory, and Dr. R. W. Whalin, Chief of the Wave Dynamics Division. Tests were conducted by Mr. C. R. Curren, Project Engineer, with the assistance of Messrs. H. Acuff, civil engineering technician, R. E. Ankeny, computer technician, and P. M. Kransnoff, engineering student trainee, under the supervision of Mr. C. E. Chatham, Chief of the Wave Processes Branch. This report was prepared by Messrs. Curren and Chatham. During the course of the investigation, liaison was maintained with SPL by means of conferences, telephone communications and monthly progress reports. Messrs. Chatham and Curren and Dr. Whalin visited Oceanside to confer with representatives of the City and SPL and to inspect the prototype site.

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

Congressman Robert E. Badham	U. S. House of Representatives
Mr. Howard Seelye	Administrative Assistant to Congressman Badham
BG Hugh Robinson	Los Angeles District Engineer
Dr. Richard Seymour	California Department of Boating and Waterways
Mr. John Habel	California Department of Boating and Waterways
Mr. Orville Magoon	SPD
Mr. Ted Albrecht	SPD

Mr. Thurman Wathen	SPD
Mr. Charlie Fisher	SPL
Mr. Win Collins	SPL
Mr. Dan Muslin	SPL
Mr. Claude Wong	SPL
Mayor Paul Graham	City of Oceanside
Mr. Tom Missett	President, Oceanside Chamber of Commerce
Mr. Pat O'Day	Oceanside Chamber of Commerce
Mr. John Casey	Oceanside City Council
Mr. Tom Gorman	Los Angeles Times
Mr. David Gould	Oceanside Blade Tribune

COL John L. Cannon, CE, and COL Nelson P. Conover, CE, were Commanders and Directors of WES during the conduct of this investigation and the preparation and publication of this report. Mr. F. R. Brown was Technical Director.

CONTENTS

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
cubic yards	0.7645549	cubic metres
feet	0.3048	metres
feet per second	0.3048	metres per second
miles (U. S. statute)	1.609344	kilometres
pounds (mass)	0.4535924	kilograms
square feet	0.09290304	square metres
square miles (U. S. statute)	2.589988	square kilometres
tons (2000 lb, mass)	907.1847	kilograms

OCEANSIDE HARBOR AND BEACH, CALIFORNIA
DESIGN OF STRUCTURES FOR HARBOR IMPROVEMENT
AND BEACH EROSION CONTROL

Hydraulic Model Investigation

PART I: INTRODUCTION

The Prototype

1. Oceanside Harbor and Beach are located on the Pacific Ocean approximately 80 miles* southeast of Los Angeles and 30 miles northwest of San Diego (Figure 1). The harbor complex includes the Del Mar Boat Basin (also known as the Camp Pendleton Harbor) and the Oceanside Small Craft Harbor (Figure 2). While the Camp Pendleton Harbor is used entirely for military purposes, Oceanside Harbor and Beach are used primarily for recreation. The harbors are protected by a 4350-ft-long north jetty and a 1330-ft-long south jetty. A 920-ft-long south groin is located at the mouth of the San Luis Rey River approximately 3900 ft north of the Oceanside fishing pier. The sea floor is characterized by gently sloping contours that bend around the harbor and increase somewhat in slope south of the fishing pier.

The Problem

2. Since the construction of the Del Mar Boat Basin in 1943, persistent and devastating erosion of the beaches south of the harbor complex has occurred with an accompanying accretion of sand in the harbor and entrance channel. In 1958, the problem was further aggravated by the extension of the north breakwater to its present length and configuration in an attempt to reduce shoaling of the harbor entrance. Taking

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* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 4.

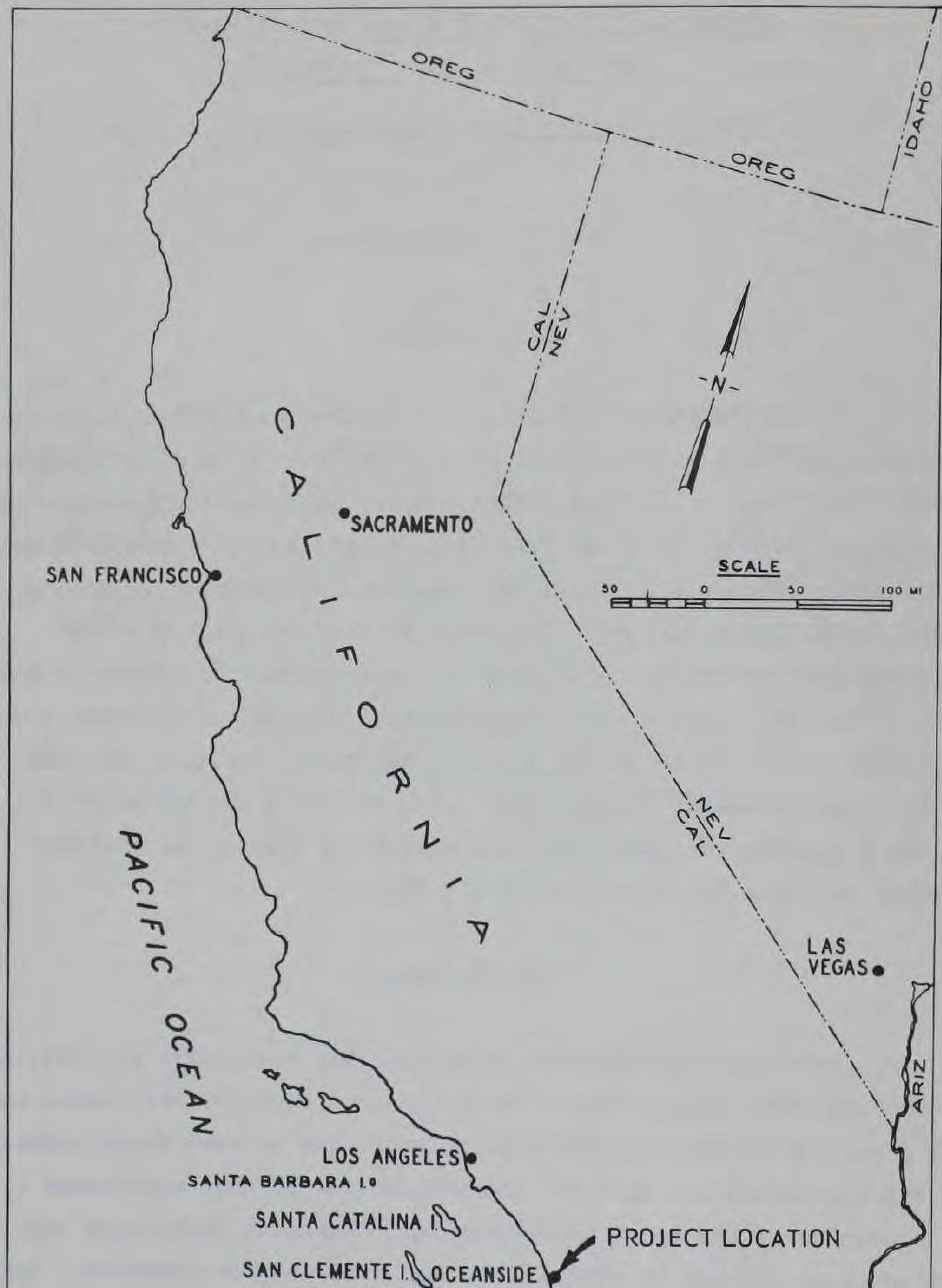

Figure 1. Project location

Figure 2. Aerial view of Oceanside Harbor

advantage of a readily available source of sand for a beach replenishment project, the City of Oceanside planned the development of a small-craft recreational harbor concurrently with dredging operations, and construction of Oceanside Harbor was completed in 1963. The jetties at Camp Pendleton Harbor, which were constructed as a wartime measure without provisions for possible adverse effects to the adjoining shores, are assumed to be partially responsible for the erosion problem at Oceanside and this severe erosion is illustrated in Figure 3.

Proposed Improvements

3. Improvements for Oceanside Harbor and Beach, proposed by the U. S. Army Engineer District, Los Angeles (SPL), and shown in Figure 4, were separated into two sections as follows:

   a. The harbor. The original proposal for the prevention of harbor shoaling included a 1400-ft-long offshore breakwater and a 735-ft-long extension of the south jetty designed to trap sediments in a predetermined location outside the entrance channel. Also proposed was a plan for expanding the present harbor facilities by converting the turning basin into an inner mooring basin. A 2200-ft-long inner breakwater was included to provide wave protection to the mooring basin, if needed.

   b. The beach. There were two proposed solutions for preventing beach erosion. One involved a 4900-ft-long offshore breakwater of alternating high and low crown elevations while the other used a series of five 800-ft-long groins.

Purposes of the Model Study

4. Purposes of the model study were to:

   a. Determine the mechanisms by which sand is entering the harbor and being lost from the existing beach.

   b. Study shoaling and wave conditions with the proposed improvement plans installed in the model.

Figure 3. Typical beach erosion at Oceanside

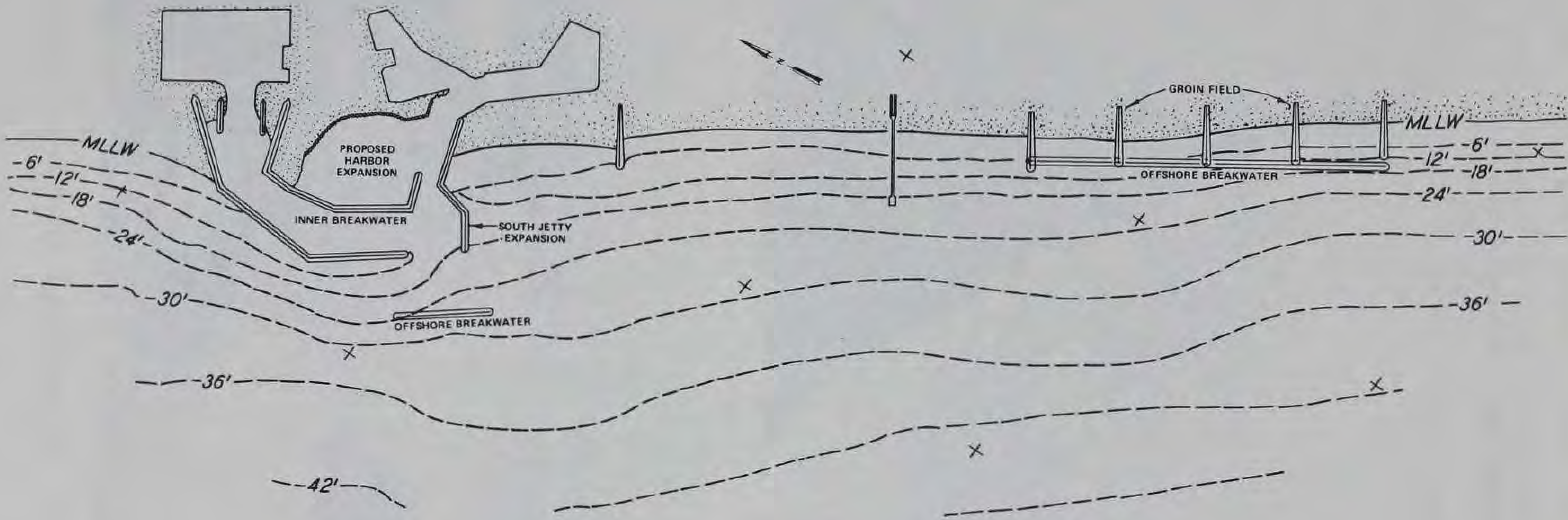

Figure 4. Proposed improvements for Oceanside Harbor and Beach

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

d. Determine whether suitable design modifications of the proposed plans could be made that would reduce construction costs significantly and still provide adequate protection.

Wave-Height Criteria

5. Completely reliable criteria have not yet been developed for ensuring satisfactory navigation and berthing in small-craft harbors during attack by waves. However, for the study reported herein, SPL specified that for an improvement plan to be acceptable, maximum wave heights in the harbor should not exceed 1.5 ft in berthing areas of the harbor expansion (inner basin) and 4.0 ft in the expansion entrance.

Design of the Model

6. The Oceanside model (Figure 5) was constructed to a linear scale of 1:100, model to prototype. Scale selection was based on such factors as:

- a. Depth of water required in the model to prevent excessive bottom friction effects.
- b. Absolute size of the model waves.
- c. Available shelter dimensions and area required for model construction.
- d. Efficiency of model operation.
- e. Capabilities of available wave-generating and wave-measuring equipment.
- f. Model construction cost.

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

Characteristic	Dimension*	Model:Prototype Scale Relations
Length	L**	$L_r = 1:100$
Area	L^2	$A_r = L_r^2 = 1:10,000$
Volume	L^3	$V_r = L_r^3 = 1:1,000,000$
Time	T	$T_r = L_r^{1/2} = 1:10$
Velocity	L/T	$V_r = L_r^{1/2} = 1:10$

* Dimensions are in terms of length and time.
** For convenience, symbols and unusual abbreviations are listed and defined in the Notation (Appendix B).

7. Ideally, a quantitative, three-dimensional, movable-bed model

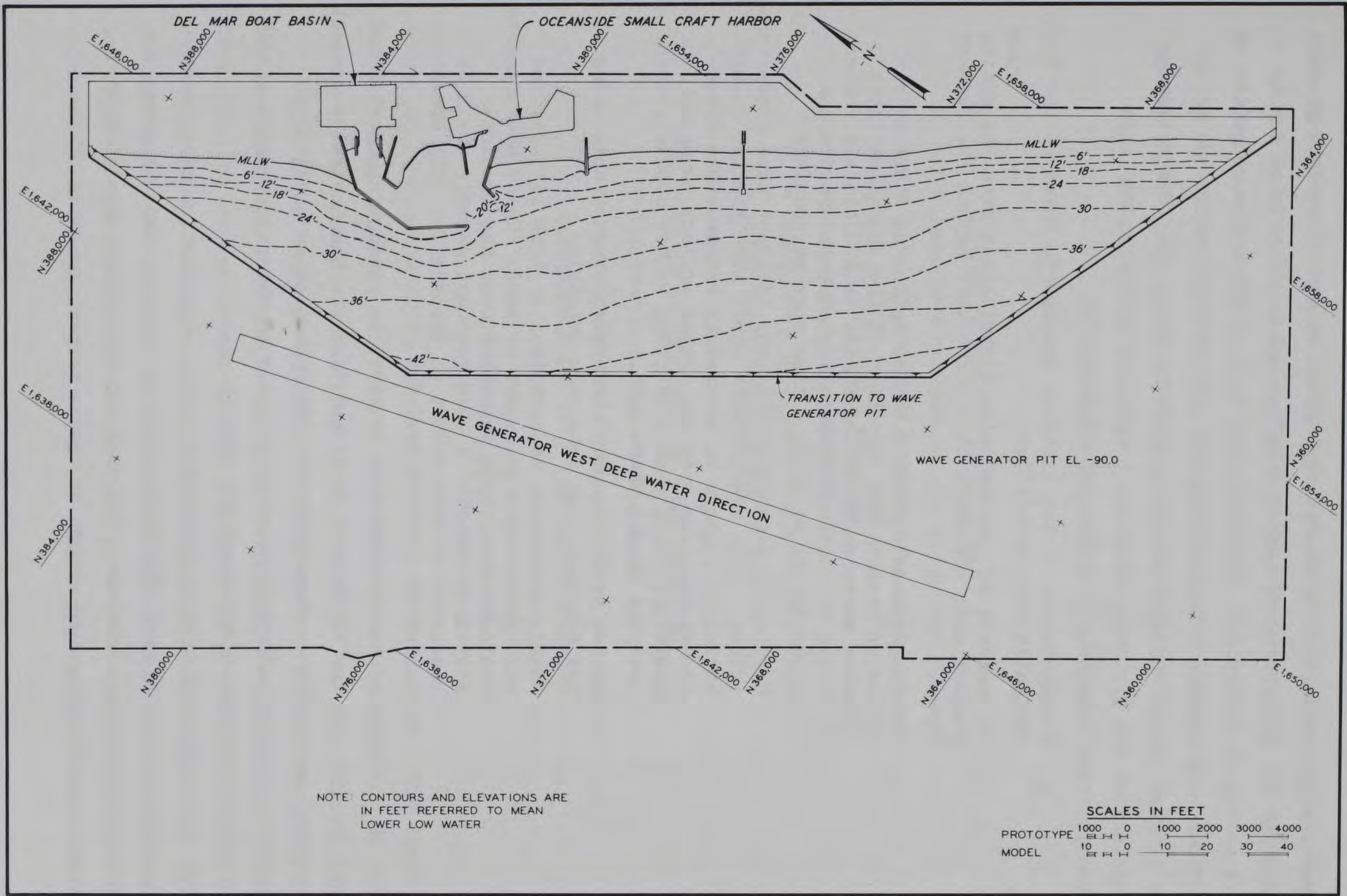

Figure 5. Model layout

investigation would best determine the effectiveness of various project plans for the prevention of harbor shoaling and beach erosion due to wave action at Oceanside. 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. The following computations and prototype data are considered essential for such investigations (Chatham, Davidson, and Whalin 1973):

a. A computation of the littoral transport, based on the best available wave statistics.

b. An analysis of the sand-size distribution over the entire project area (offshore to a point well beyond the breaker zone).

c. Simultaneous measurements of the following items over a period of erosion and accretion of the shoreline (this duration measurement period should be judiciously chosen to obtain the maximum probability of both erosion and accretion during as short a time span as possible):

   (1) Continuous measurements of the incident wave characteristics. Such measurements would mean placing enough redundant sensors to accurately estimate the directional spectrum over the entire project area and, in addition, would mean conducting rather sophisticated analyses of all these data.

   (2) Bottom profiling over the entire project area using the shortest time intervals possible.

   (3) Nearly continuous measurements of both littoral and onshore-offshore transport of sand. These measurements would be especially important over the erosion-accretion period. A wave forecast service would be essential to this effort to prepare for full operation during the erosion period.

8. In view of the complexities involved in conducting movable-bed model studies and due to limited funds and time for the Oceanside project, the model was molded in cement mortar (fixed bed) at an undistorted scale of 1:100 and a tracer material was used to determine qualitatively the degree of sediment movement for various plans.

9. Model limits were selected to allow reproduction of as much shoreline on each side of the study areas as possible while keeping construction costs at a minimum. The main considerations were that the updrift shoreline be long enough to ensure proper formation of the

longshore currents before reaching the study area and that the downdrift shoreline be of sufficient length to prevent deflection of the longshore currents back into the study area.

10. For development of a longshore current along a straight beach, Eagleson (1965) gave the following relation:

$$u_L^2\ (x) = A - \left[A - u_L^2\ (o)\right] e^{-Bx}$$

where

$u_L\ (x)$ = longshore current at distance x

$u_L\ (o)$ = longshore current at the origin $x = 0$

$A = 3/8\ gH_b^2 n_b/d_b \sin\alpha \sin\theta_b \sin 2\theta_b/f$

$B = 2/5 \left[f/(d_b \cos\alpha \sin\theta_b)\right]$

H_b = wave height at breaking

$n_b = 1/2 \left(1 + 2\ k_b\ d_b/\sin b\ 2\ k_b\ d_b\right)$

d_b = water depth at breaking

α = beach slope

θ_b = angle of incident wave at breaking

b = the width of the breaker zone

k_b = coefficient of breaking

f = Darcy-Weisbach friction factor suggested to be:

$$f = \left[2 \log_{10}\left(\frac{d_b}{k_*}\right) + 1.74\right]^{-2}$$

where k_* is the absolute roughness of the beach surface. The longshore current velocity u_L tends to a constant velocity $A^{1/2}$ as the value of Bx grows. For $Bx = 2$, $u_L = 0.92\ A^{1/2}$.

11. Since the contours at Oceanside are relatively straight in the breaker zone, Eagleson's equation was applied for a range of incident waves to determine approximate distances for formation of longshore currents. In all cases, these distances were less than 3000 ft. The shoreline distances of 6400 ft and 5200 ft reproduced to the north and south of the study areas, respectively, and the generator length extending 5800 ft to the north and 4800 ft to the south of the study area

therefore were considerably greater than those distances required for proper formation of longshore currents.

12. In an effort to minimize any possible effects of model boundaries on longshore currents, a recirculation channel was designed and incorporated in the model. Preliminary model tests were conducted to evaluate the effectiveness of the recirculation channel and it was found that a minor part of the longshore current returned to the updrift boundary through the recirculation channel. The major part of the longshore current was deflected seaward at the downdrift boundary and moved behind the wave generator to the updrift boundary forming a somewhat natural recirculation system. No model boundary effects on the longshore current were noted in the study area for at least 1 hr (model time). The longshore currents reached equilibrium very rapidly (within about 30 sec model time), and all model data were taken within the first 30 min as a precaution against possible model boundary effects.

13. Based on the principles of hydraulic similitude, the model correctly reproduced:

- a. Wave refraction.
- b. Wave shoaling.
- c. Wave diffraction.
- d. Wave breaking.
- e. Nearshore circulation cells (rip, feeder, and eddy currents).
- f. Longshore currents generated by breaking waves (within the area covered by the wave generator).
- g. Qualitative sediment transport in the breaker zone.

14. The model did not reproduce longshore currents and sediment transport at the boundaries. Some of the problems associated with reproducing longshore currents and/or sediment transport at the model boundaries were:

- a. Lack of prototype data.
- b. Complexity of nearshore circulation cells and longshore current patterns.
  - (1) Longshore currents are interrupted by rip currents and eddies.

(2) Wave refraction causes areas of energy convergence and divergence.

(3) Wave refraction changes the breaking wave angle.

c. Operational problems.

(1) Lack of information about the proper location and design of circulation systems at model boundaries, current distribution, and friction effects.

(2) Boundary conditions change for each test wave and direction (this would require, in addition, that extensive prototype data be acquired to properly attempt to reproduce this boundary condition). Effects of lateral model boundaries were negligible within the model test area and were minimized by the procedure discussed in paragraph 12.

15. The proposed improvement plans for Oceanside included the use of rubble-mound breakwaters, groins, and jetties. Experience and experimental research have shown that considerable wave energy passes through the interstices of this type of structure; thus, the transmission and absorption of wave energy became a matter of concern during design of the 1:100-scale model. In small-scale models, rubble-mound structures reflect relatively more and absorb or dissipate relatively less wave energy than geometrically similar prototype structures (Le Mehaute 1965). Also, the transmission of wave energy through the structure is relatively less for the small-scale model than for the prototype. Consequently, some adjustment in small-scale rubble-mound structures is needed to ensure satisfactory reproduction of wave-reflection and wave-transmission characteristics. In past investigations at U. S. Army Engineer Waterways Experiment Station (WES) (Dai and Jackson 1966, Ball and Brasfeild 1967), 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. Therefore, based on previous findings for structures and wave conditions similar to those at Oceanside, 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 1:100-scale model to approximately 2.0 times that required for geometric similarity. Accordingly, in constructing the rubble-mound structures

in the Oceanside model, rock sizes were computed linearly by scale, then multiplied by 2.0 to determine the actual sizes to be used in the model.

The Model and Appurtenances

16. The model reproduced 5.7 miles of shoreline and underwater contours to offshore depths ranging from -42 ft to -48 ft, with a sloping transition to the wave generator pit elevation of -90 ft. The total model area of 40,800 sq ft represented about 14.6 square miles in the prototype. A general view of the model is shown in Figure 6. Vertical control for model construction was based on the mean lower low water (mllw) elevation* of 0.0 ft. Horizontal control was based on a local prototype grid system.

17. Model waves were generated by a 190-ft-long wave generator with a trapezoidal-shaped, vertical-motion plunger. The vertical motion of the plunger caused a periodic displacement of water incident to this motion. The length of stroke and period of the vertical motion were variable over the range necessary to generate waves with the required characteristics. In addition, the wave generator was mounted on retractable casters which enabled it to be positioned to generate waves from the required directions.

18. An Automated Data Acquisition and Control System (ADACS), designed and constructed at WES (Figure 7) was used to secure wave-height data at selected locations in the model. Basically, through the use of a minicomputer, ADACS recorded onto magnetic tape the electrical output of parallel-wire, resistance-type sensors. These sensors measured the change in water-surface elevation with respect to time. The magnetic tape output of ADACS then was analyzed to obtain the wave-height data.

19. A 2-ft (horizontal) solid layer of fiber wave absorber was placed around the inside perimeter of the model to damp any wave energy

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

Figure 6. General view of model

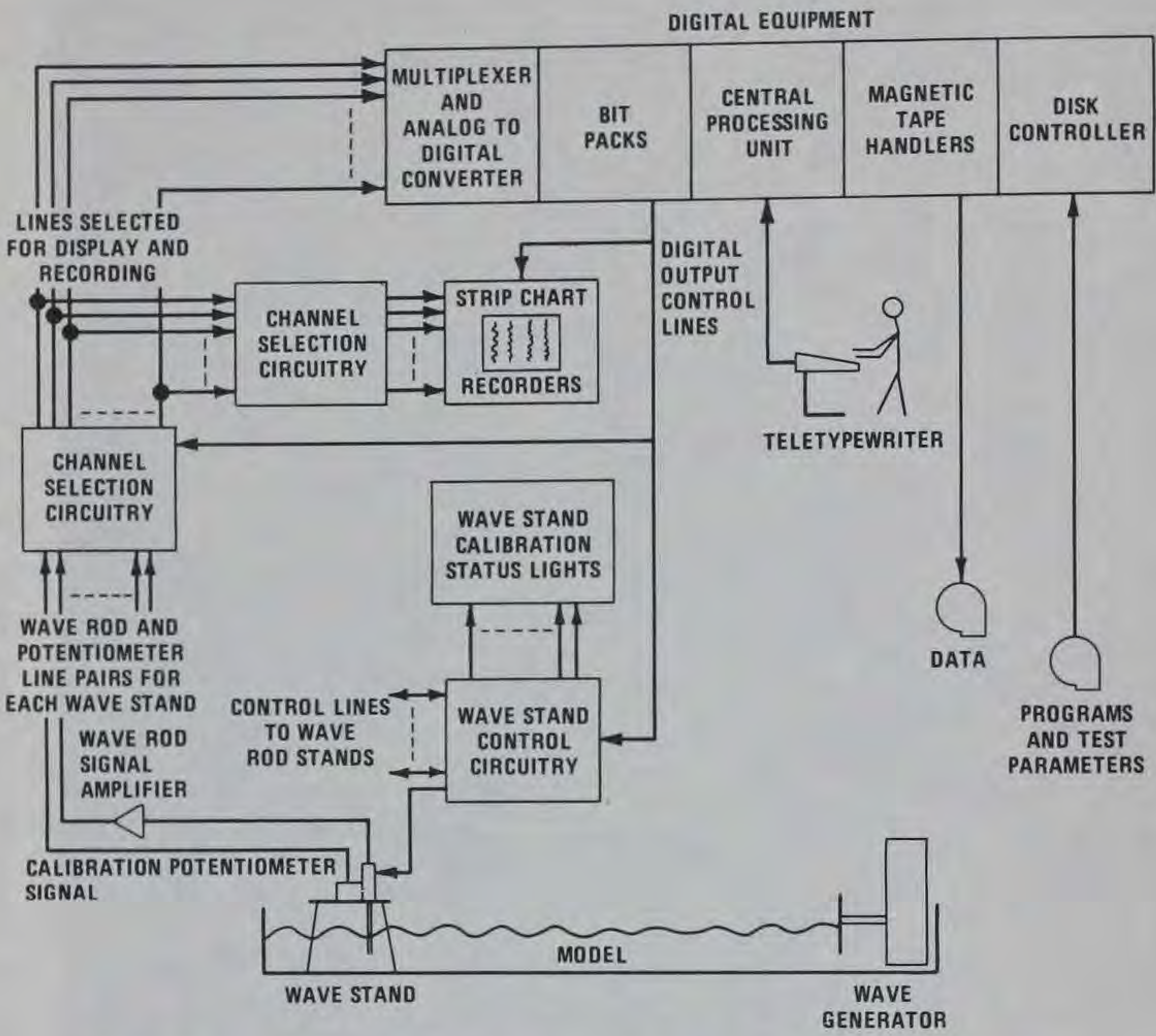

Figure 7. Automated Data Acquisition and Control System

that might otherwise be reflected from the model walls. In addition, guide vanes were placed along the sides of the wave generator to ensure proper formation of the wave train incident to the model contours.

Selection of Tracer Material

20. As previously mentioned in paragraph 8, a fixed-bed model was constructed and a tracer material selected to determine qualitatively the degree of sediment transport and extent of erosion and accretion for various improvement plans. As in previous WES investigations (Giles and Chatham 1974, Bottin and Chatham 1975, Curren and Chatham 1977,

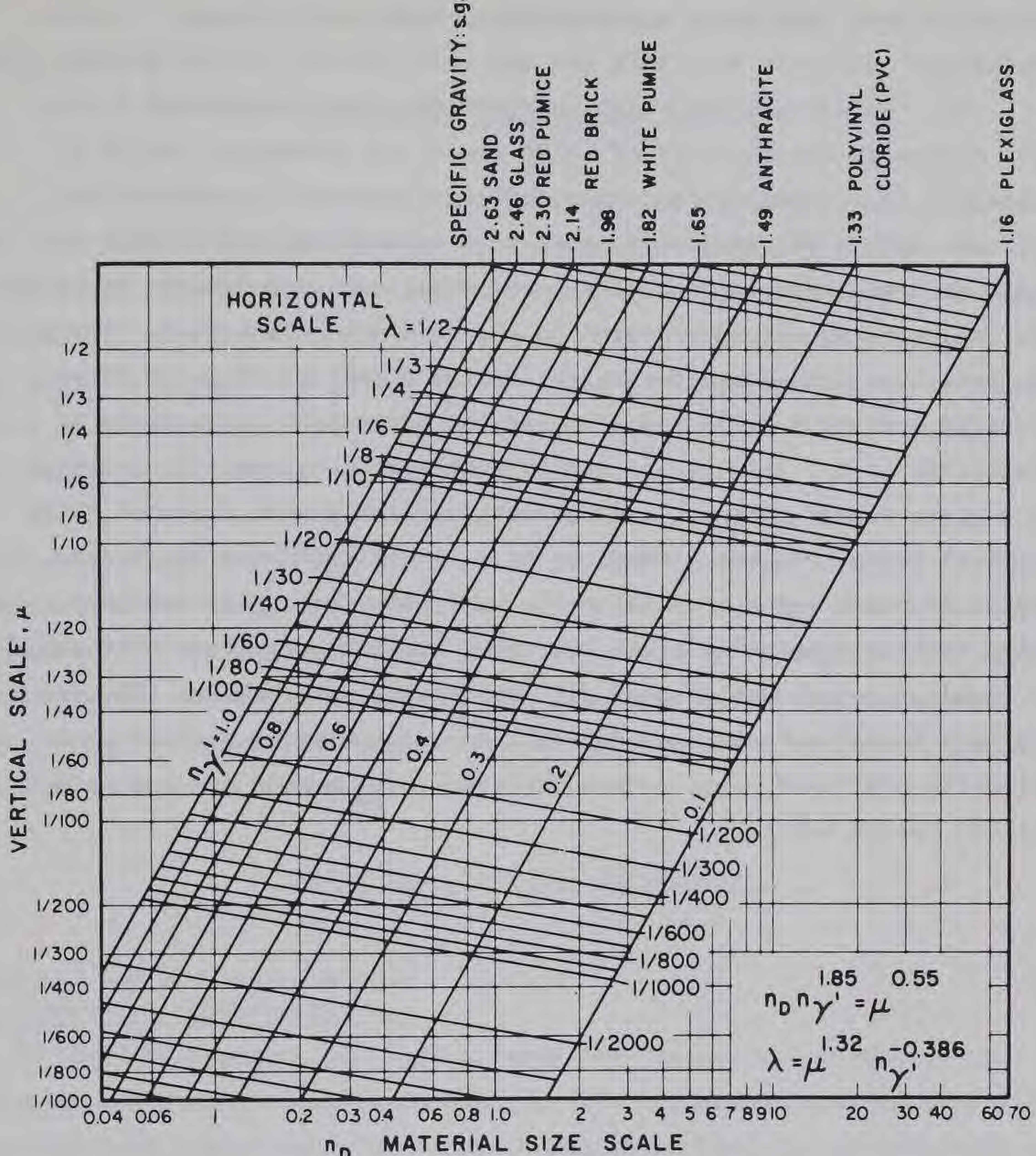

Figure 8. Graphical representation of model law (Noda 1971)

Bottin 1977, Curren and Chatham 1979) the tracer material was chosen in accordance with the scaling relations of Noda (1971), 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 (Figure 8). These relations were determined experimentally using a wide range of wave conditions and beach materials and are valid mainly for the breaker zone.

21. Noda's scaling relations indicate that movable-bed models with scales in the vicinity of 1:100 (model to prototype) should be distorted (i.e., they should have different horizontal and vertical scales). Since the fixed-bed model of Oceanside was undistorted to allow accurate reproduction of sea and swell and wave-induced currents, 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., 1:100), the median diameter for a specific gravity of a given tracer material and the vertical scale were computed. The vertical scale then was assumed to be in similitude, and the tracer median diameter and horizontal scale were computed. This resulted in a range of tracer material sizes for given specific gravities that could be used. A search was made of all movable-bed materials at WES, preliminary model tests were conducted, and a quantity of crushed coal (specific gravity = 1.30, median diameter, $D_{50} = 0.38$ mm) was selected for the tracer tests.

PART III: TEST CONDITIONS AND PROCEDURES

Selection of Still-Water Levels

22. Still-water levels (swl) 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 as they approach the study area, overtopping of structures by waves, position and strength of longshore currents, reflection of wave energy from structures, and transmission of wave energy through porous structures.

23. From U. S. Coast and Geodetic Survey (now National Ocean Survey) records (1950-1961), the mllw level at Oceanside is 0.0 ft, and the mean higher high water (mhhw) level is +5.4 ft. The mhhw stage was considered to be representative of water levels to be expected during a severe storm and a swl of +5.4 ft was selected for use in the model. The mllw level also was selected for use in the model to determine if the relative effectiveness of various plans was sensitive to the swl.

Wave Dimensions and Directions

Factors influencing selection of test-wave characteristics

24. In planning the test program for a model investigation of wave-action problems, it is necessary to select dimensions and directions for the test waves that will afford a realistic test for the proposed improvement plans and allow an accurate evaluation of the elements of the various proposals. Surface wind waves are generated by the interactions between tangential stresses of wind flowing over water, resonance between the water surface and atmospheric turbulence, and interactions between individual wave components. The height and period of the maximum wave that can be generated by a given storm depend on the wind speed, the length of time that a wind of a given speed continues to blow (duration), and the water distance (fetch) over 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 the various directions from which waves can attack the problem area.
- b. Frequency of occurrence and duration of storm winds from the different directions.
- c. Alignment and relative geographic position of the study area.
- d. Alignments, lengths, and locations of various structures in the study area.
- e. Refractions of waves caused by differentials in depths in the area seaward of the study area, which may cause either a convergence or a divergence of wave energy.

Wave refraction

25. 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 selection of test-wave characteristics are the changes in wave height and direction of travel due to the phenomenon referred to as wave refraction. Changes in wave height and direction can be determined by plotting refraction diagrams and calculating refraction coefficients. These diagrams are constructed by plotting the position of wave orthogonals (lines drawn perpendicular to wave crests) from deep water into shallow water. If it is assumed that the waves do not break and that there is no lateral flow (diffraction) of energy along the wave crest, the ratio between the wave height in deep water (H_o) and the wave height in shallow water (H) will be inversely proportional to the square root of the ratio of the corresponding orthogonal spacings (b_o and b) or $H/H_o = K(b_o/b)^{1/2}$. The quantity $(b_o/b)^{1/2}$ is the refraction coefficient; K is the shoaling coefficient. Thus, the refraction coefficient multiplied by the shoaling coefficient gives a conversion factor for transfer of deepwater wave heights to shallow-water values. The shoaling coefficient, which is a function of

wavelength and water depths, can be obtained from the Shore Protection Manual (CERC 1977).

26. Wave-refraction diagrams from a previous investigation conducted at WES (Hales 1978), supplemented by additional refraction diagrams where needed, were used for deepwater wave directions ranging from 165° to 330° and wave periods from 2 to 18 sec. These diagrams represented the propagation of wave fronts from deep water to shallow water (to the point of breaking). By positioning the wave generator to correspond with the wave front at -90 ft (the elevation of the wave-generator pit), the refracted wave from the deepwater direction was accurately reproduced.

Prototype wave data and selection of test waves

27. Estimated durations and magnitudes of deepwater waves approaching Oceanside, California, were obtained from a wave hindcast prepared in 1977 for the Department of Navigation and Ocean Development (DNOD) (Meteorology International, Inc. 1977) for waves from the south, southwest, and west. Locally generated waves from the northwest were calculated using forecasting techniques from the Shore Protection Manual (CERC 1977), appropriate fetch lengths, and wind speeds and durations taken from the 1977 DNOD report. These data are summarized in Table 1. Using refraction coefficients from the refraction analysis discussed in paragraph 26, and shoaling coefficients for the water depths at the model wave generator, the deepwater data in Table 1 were converted to shallow-water values and are summarized in Table 2. Test waves used in the model were selected from Table 2 as shown in the following tabulation.

Selected Test Waves and Directions

Deepwater Wave Direction	Selected Shallow-Water Wave Test Direction	Selected Test Wave Period sec	Height ft
Northwest (315°)	274°	7	5
			10
West (270°)	253°	7	4
			10

(Continued)

Selected Test Waves and Directions

Deepwater Wave Direction	Selected Shallow-Water Wave Test Direction	Selected Test Wave Period sec	Selected Test Wave Height ft
West (270°) (Cont)		9	4
			12
		11	4
			10
		14	6
		17	6
Southwest (225°)	224°	7	4
			10
		9	4
			16
		11	8
		14	6
		17	10
		19	6
South (180°)	198°	7	4
			10
		9	4
			10
		14	6
		17	6
		19	4

Selection of test procedures

28. During the conduct of this model study, three different types of data were obtained: wave heights, wave-induced current patterns and magnitudes, and sediment tracer patterns. The procedures used for securing these data are as follows:

- a. Wave-height tests. These data were obtained by placing parallel-wire resistance-type wave sensors at strategic locations in the area of interest. As each wave test was run, voltage differentials across the parallel wires of each gage were measured by ADACS and translated into wave-height data.
- b. Wave-induced current patterns and magnitudes. These data were determined by timing the progress of a dye tracer relative to a known distance on the model surface. These model times were then converted to prototype velocities and superimposed on wave pattern photographs.
- c. Sediment tracer tests. For the harbor, these data were obtained by continuously feeding fixed-bed tracer material

into the breaker zone at a point outside the study area and recording the movement by means of photographs and model data sheets. The tracer material was removed before each test except where otherwise noted. For the beach, movable-bed tracer tests were run for the major improvement plans using both continuous feeding and nonfeeding of material. In general, the movable-bed tracer material was not remolded until after a series of waves from one direction (see paragraph 191) was tested. This was done to give a better indication of beach stability and substantially reduce model testing time. For the movable-bed tracer tests, measurements were taken of the shoreline configuration after each test and plotted on drawings of each plan.

Analysis of Model Data

29. The relative merits of the various plans tested were evaluated using (a) comparison of wave heights at selected locations in the study area, (b) comparison of current patterns and magnitudes, (c) comparison of tracer patterns, (d) comparison of resultant tracer shorelines, and (e) visual observations and photographs. In the wave-height data analysis, the average of the highest one third of the waves (significant wave height) at each gage location was selected. By using Keulegan's equation (Keulegan 1950) the reduction of wave heights in the model due to bottom friction was calculated as a function of water depth, width of wave front, wave period, water viscosity, and distance of wave travel and appropriate corrections were made at each gage location.

30. Since the primary purposes of this study were to develop plans to prevent (a) loss of beach material from the study area and (b) accumulation of sand in the harbor entrance, movement of tracer material was a prime concern. Since these tests are of a qualitative nature and no estimates of quantities transported are possible, any appreciable movement of tracer material into the harbor entrance was considered a problem. Only those plans with very little or no movement of tracer material into the entrance were considered as viable alternatives. Likewise for the beach, any appreciable shoreline erosion and loss of tracer material from the study area was considered a problem;

and only those plans with minimal shoreline erosion and loss of tracer material were considered as viable alternatives.

PART IV: HARBOR TESTS AND RESULTS

Description of Tests

Existing conditions

31. Prior to tests of various improvement plans, comprehensive tests were performed for existing conditions (Plate 1). Wave-height data were obtained for various stations along the proposed offshore breakwater center line, within the entrance channel, and within the small-boat basin for the test conditions listed in paragraph 27. Wave-induced current patterns and current magnitudes and tracer patterns also were secured for representative waves from the four selected test directions.

Harbor improvement plans

32. Wave-height, current pattern and magnitude, and/or tracer tests were conducted for 88 plan variations. These variations consisted of changes in the lengths and alignments of the breakwater structures and jetty extensions, changes in the north jetty cross section, the addition of another small-craft basin, and the construction of sand traps. Photographs of wave patterns and/or tracer patterns were obtained for all major improvement plans. Brief descriptions of the harbor improvement plans are presented below; dimensional details are presented in Plates 2-51.

> Plan 1 (Plate 2) consisted of a 1400-ft-long rubble-mound breakwater with a crown elevation of +14.0 ft positioned 800 ft seaward of the harbor entrance. Also, the south jetty was extended a total of 735 ft (171 ft along the alignment of the existing jetty with a 564-ft-long seaward leg) providing an entrance channel width of 800 ft.
>
> Plan 1A (Plate 2) consisted of the elements of Plan 1 with an 800-ft extension added to the northern end of the offshore breakwater.
>
> Plan 1B (Plate 2) entailed the elements of Plan 1A with a 400-ft extension added to the northern end of the offshore breakwater.
>
> Plan 1C (Plate 2) consisted of the elements of Plan 1B with 100 ft of the breakwater extension removed.

Plan 1D (Plate 2) consisted of the elements of Plan 1C with an additional 100 ft of breakwater extension removed.

Plan 2 (Plate 3) entailed the elements of Plan 1A but with the offshore breakwater moved 300 ft shoreward.

Plan 2A (Plate 3) was the same as Plan 2 with a 100-ft-long rubble-mound jetty spur placed 300 ft from the end of the north jetty.

Plan 2B (Plate 3) was the same as Plan 2A with the jetty spur relocated 600 ft to the north.

Plan 3 (Plate 4) consisted of the elements of Plan 2B with the entire 735-ft south jetty extension realigned with the existing south jetty.

Plan 4 (Plate 5) consisted of the elements of Plan 2B with a 500-ft dogleg attached to the end of the proposed south jetty extension allowing a 400-ft-wide harbor entrance.

Plan 5 (Plate 6) entailed the elements of Plan 2B with a 1000-ft south jetty extension constructed in a straight line from the existing south jetty to a point 400 ft shoreward of the south end of the offshore breakwater.

Plan 19 (Plate 7) provided an inner mooring basin with depths of 10 ft and enclosed by a 2200-ft-long rubble-mound breakwater tied into the south jetty of the Del Mar Boat Basin. Crown elevations of the inner breakwater were +12 ft for the first 1100 ft and +16 ft for the final 1100 ft. Also, north and south jetty extensions of 600 ft and 700 ft, respectively, were installed.

Plan 20 (Plate 8) entailed the elements of Plan 19 with the south jetty extension straightened and extended to a total length of 950 ft.

Plan 21 (Plate 9) included the elements of Plan 20 with a 300-ft-long dogleg installed at the end of the south jetty extension, bringing the total extension length to 1250 ft.

Plan 21A (Plate 9) consisted of the elements of Plan 21 with 100 ft added to the north jetty extension.

Plan 21B (Plate 9) entailed the elements of Plan 21 with the inner breakwater extended 200 ft.

Plan 21C (Plate 9) entailed the elements of Plan 21 with the inner breakwater extended 300 ft.

Plan 21D (Plate 9) involved the elements of Plan 21 with the inner breakwater extended 400 ft.

Plan 21E (Plate 10) consisted of the elements of Plan 21 with a 400-ft-long inner jetty (crown elevation +16 ft) connected to the shore and extended toward the inner breakwater head.

Plan 21F (Plate 9) involved the elements of Plan 21D with the crown elevation of the final 1500 ft of the inner breakwater lowered to +14 ft.

Plan 22 (Plate 11) consisted of the elements of Plan 21A with the final 100 ft of the north jetty extension angled seaward.

Plan 22A (Plate 11) entailed the elements of Plan 22 with the north jetty extension lengthened an additional 300 ft.

Plan 23 (Plate 12) consisted of the elements of Plan 21 with the north jetty extension removed. Also, a 1200-ft-long offshore breakwater (crown el +14 ft) was installed north of the harbor with a sand deposition basin (el -30 ft) located in the lee of the breakwater.

Plan 24 (Plate 13) entailed the elements of Plan 23 with the offshore breakwater and deposition basin relocated seaward of the middle leg of the north jetty.

Plan 25 (Plate 14) consisted of the elements of Plan 21 with the north and south jetty extensions removed.

Plan 25A (Plate 14) involved the elements of Plan 25 with the inner breakwater lengthened 400 ft and the crown elevation of the final 1500 ft of the inner breakwater lowered to +14 ft.

Plan 26 (Plate 15) entailed the elements of Plan 25A with the north jetty made impervious and its crown elevation raised from +14 ft to +22 ft.

Plan 26A (Plate 15) consisted of the elements of Plan 26 with 200 ft of the shoreward terminus of the inner breakwater removed.

Plan 27 (Plate 16) involved the elements of Plan 26 with the crown elevation of the final 1500 ft of the inner breakwater raised to +16 ft.

Plan 27A (Plate 16) involved the elements of Plan 27 with 200 ft removed from the shoreward terminus of the inner breakwater.

Plan 27B (Plate 16) entailed the elements of Plan 27 with 400 ft removed from the shoreward terminus of the inner breakwater.

Plan 27C (Plate 16) consisted of the elements of Plan 27 with 600 ft removed from the shoreward terminus of the inner breakwater.

Plan 28 (Plate 17) consisted of the elements of Plan 27B with the addition of the north and south jetty extensions of Plan 21.

Plan 29 (Plate 18) consisted of the elements of Plan 28 with the north jetty extension and inner breakwater removed, the 1250-ft-long south jetty extension replaced with an 1100-ft-long extension (700-ft-long seaward extension with a 400-ft-long southerly dogleg), and a 1400-ft-long offshore breakwater (crown el +14 ft) constructed 500 ft seaward of and parallel to the north jetty.

Plan 30 (Plate 19) entailed the elements of Plan 29 with the crown elevation of the offshore breakwater raised to +22 ft.

Plan 30A (Plate 19) involved the elements of Plan 30 with the offshore breakwater extended 400 ft to the south.

Plan 31 (Plate 20) entailed the elements of Plan 30 with the offshore breakwater made impervious.

Plan 31A (Plate 20) consisted of the elements of Plan 31 with the offshore breakwater lengthened 200 ft to the south.

Plan 31B (Plate 20) involved the elements of Plan 31 with the offshore breakwater extended 300 ft to the south.

Plan 31C (Plate 20) entailed the elements of Plan 31 with the offshore breakwater extended 400 ft to the south.

Plan 31D (Plate 20) involved the elements of Plan 31C with the southernmost 200 ft of the offshore breakwater angled 30 deg shoreward.

Plan 31E (Plate 20) consisted of the elements of Plan 31D with 200 ft added to the north end of the offshore breakwater.

Plan 31F (Plate 20) involved the elements of Plan 31A with the offshore breakwater extended 200 ft to the north and to the south.

Plan 32 (Plate 21) consisted of the elements of existing conditions with a 1000-ft-long offshore breakwater (crown el +14 ft) constructed parallel to the last leg of the existing north jetty and connected to the last bend in the north jetty with a 350-ft-long breakwater (crown el +14 ft).

Plan 33 (Plate 22) involved the elements of Plan 32 with the 350-ft-long connecting breakwater removed.

Plan 34 (Plate 23) consisted of the elements of Plan 32 with the 350-ft-long connecting breakwater replaced with a low-sill structure (crown el 0.0).

Plan 35 (Plate 24) involved the elements of Plan 34 with the addition of a 500-ft-long groin (crown el +14 ft) placed perpendicular to the offshore breakwater and connected to the first bend of the north jetty.

Plan 35A (Plate 24) entailed the elements of Plan 35 with the groin shortened to 400 ft.

Plan 36 (Plate 25) consisted of the elements of Plan 35A with the offshore breakwater shortened 200 ft on the north end and the groin relocated 200 ft to the south.

Plan 37 (Plate 26) involved the elements of Plan 36 with the 350-ft-long low-sill connecting structure and the 400-ft-long groin removed and a 650-ft-long groin (the outer 250 ft of which was curved to the south) located in the same position as in Plan 35.

Plan 38 (Plate 27) entailed the elements of Plan 37 with the 250-ft-long curved section of the groin removed.

Plan 38A (Plate 27) involved the elements of Plan 38 with the addition of the 350-ft-long low-sill structure connecting the offshore breakwater with the north jetty.

Plan 38B (Plate 27) entailed the elements of Plan 38 with the groin lengthened 150 ft.

Plan 38C (Plate 27) consisted of the elements of Plan 38 with the groin extended 250 ft.

Plan 38D (Plate 27) involved the elements of Plan 38 with the groin extended 350 ft.

Plan 39 (Plate 28) consisted of the elements of Plan 38 with a 350-ft-long structure (curved to the north) added to the groin.

Plan 39A (Plate 28) involved the elements of Plan 39 with the addition of the 350-ft-long low-sill structure connecting the offshore breakwater with the north jetty.

Plan 40 (Plate 29) consisted of the elements of Plan 34 with an 800-ft-long groin (crown el +14 ft) positioned 500 ft north of the groin of Plan 39.

Plan 40A (Plate 29) involved the elements of Plan 40 with the groin lengthened 100 ft.

Plan 40B (Plate 29) involved the elements of Plan 40 with the groin lengthened 600 ft.

Plan 41 (Plate 30) consisted of the elements of Plan 40A with the outer breakwater and low-sill connecting structure removed and the groin extended and curved 600 ft to the north.

Plan 42 (Plate 31) consisted of the elements of Plan 38B with the offshore breakwater removed and the groin extended and curved 400 ft to the north.

Plan 43 (Plate 32) involved the elements of Plan 42 with the groin angled 30 deg to the north.

Plan 43A (Plate 32) entailed the elements of Plan 43 with the groin lengthened 200 ft.

Plan 44 (Plate 33) consisted of the elements of Plan 38C with the addition of a 30-ft deep deposition basin in the lee of the offshore breakwater.

Plan 45 (Plate 34) involved the elements of Plan 44 with the 800-ft-long offshore breakwater removed.

Plan 46 (Plate 35) consisted of an inner basin (depths of 10 ft) enclosed by a 2050-ft-long rubble-mound breakwater tied into the south jetty of the Del Mar Boat Basin. Crown elevations of this inner breakwater were +12 ft for the first 1250 ft and +16 ft for the final 800 ft. A 300-ft-long inner jetty (crown el +10 ft) was connected to the shore and extended toward the inner breakwater head. Also, north and south jetty extensions of 400 ft and 1250 ft, respectively, were installed.

Plan 47 (Plate 36) entailed the elements of Plan 46 with the addition of a 250-ft-long stub groin tied into the south jetty.

Plan 47A (Plate 36) involved the elements of Plan 47 with the stub groin lengthened to 300 ft.

Plan 47B (Plate 36) entailed the elements of Plan 47 with the stub groin shortened to 200 ft.

Plan 48 (Plate 37) entailed the elements of Plan 47 with the inner jetty shortened to 250 ft.

Plan 48A (Plate 37) involved the elements of Plan 47 with the inner jetty shortened to 200 ft.

Plan 48B (Plate 37) entailed the elements of Plan 47 with the entire inner jetty removed.

Plan 49 (Plate 38) involved the elements of Plan 48B with the north jetty extension removed.

Plan 50 (Plate 39) involved the elements of Plan 49 with the north jetty extended 300 ft along the alignment of the existing jetty.

Plan 50A (Plate 39) entailed the elements of Plan 50 with the north jetty extension shortened to 200 ft.

Plan 51 (Plate 40) entailed the elements of Plan 50A with the 250-ft-long stub groin removed.

Plan 52 (Plate 41) involved the elements of Plan 51 with the addition of the 300-ft-long inner jetty.

Plan 53 (Plate 42) entailed the elements of Plan 52 with the north jetty extension removed.

Plan 54 (Plate 43) entailed the elements of Plan 53 with the addition of a 200-ft-long dogleg to the inner jetty head.

Plan 55 (Plate 44) involved the elements of Plan 54 with the inner basin entrance sealed. This was done to determine the amount of wave energy entering the inner basin through the inner breakwater.

Plan 56 (Plate 45) entailed the elements of Plan 53 with the final 1300 ft of the inner breakwater sealed.

Plan 57 (Plate 46) entailed the elements of Plan 56 with the entire inner breakwater sealed.

Plan 58 (Plate 47) involved the elements of Plan 50 with the addition of a 300-ft-long inner jetty.

Plan 59 (Plate 48) involved the elements of Plan 58 with the final 1300 ft of the inner breakwater sealed.

Plan 60 (Plate 49) entailed the elements of Plan 58 with the core elevation of the final 800 ft of the inner breakwater raised from 0.0 ft to +4.0 ft.

Plan 61 (Plate 50) consisted of the elements of Plan 60 with the 1250-ft-long south jetty extension removed and two circular deposition basins with a depth of 30 ft and a radius of 500 ft installed at the head of the existing south jetty and at the head of a 550-ft-long groin extending seaward from the first bend in the north jetty.

Plan 62 (Plate 51) involved the elements of Plan 61 with the south deposition basin relocated at the head of the 250-ft-long stub groin.

Typical sections of the various structures described above are shown in Appendix A.

Harbor wave-height tests

33. Wave-height tests for existing conditions and various improvement plans were conducted using test waves from one or more of the test directions listed in paragraph 27. As an expedient, tests involving certain proposed improvement plans were limited to one or two critical directions of approach. Following existing conditions, wave-height tests were temporarily discontinued as an expedient for the development of additional improvement plans. It became apparent that the most sensitive and critical tests performed for evaluation of plans designed to prevent harbor shoaling were the tracer tests. If no material (or a relatively negligible amount) entered the harbor, then the plan was considered potentially acceptable. After the development of a promising plan, wave-height tests then were conducted to determine if the plan created any adverse harbor wave conditions. In the development of plans designed to provide wave protection to the proposed harbor expansion, wave-height tests were of primary importance. The wave-gage locations for existing conditions and each improvement plan are shown in the referenced plates.

Harbor current pattern and magnitude tests

34. Wave-induced current patterns and magnitudes were determined at selected locations by timing the progress of a dye tracer relative to a known distance on the model surface. These tests were conducted

for existing conditions and various improvement plans using the same test directions and test waves as for the wave-height tests.

Harbor tracer tests

35. Tracer tests were conducted for existing conditions and various improvement plans using the same test directions and test waves as for the wave-height tests. During each test, tracer material was fed into the updrift breaker zone to determine the effectiveness of the individual plans in preventing tracer material from entering the harbor.

Test Results

36. In evaluating test results, the relative merits of each plan were based primarily on an analysis of wave heights, the movement of tracer material and subsequent deposits, and current pattern and magnitudes. From this evaluation, the best improvement plans were selected.

Existing conditions

37. Wave heights for existing conditions were measured at 15 gage locations along the center line of the proposed offshore breakwater (without the breakwater in place), in the harbor entrance, and inside the harbor. These data are presented in Tables 3-6. The maximum wave height recorded along the breakwater center line (gages 1-3) was 23.0 ft, and wave heights exceeded 20 ft four times. The maximum wave height recorded in the entrance channel (gages 4-6) was 20.2 ft, and wave heights exceeded 16 ft four times.

38. Current patterns and magnitudes secured for existing conditions revealed that for waves from the northwest deepwater direction (Photo 1), strong longshore currents flowed along the north jetty, across the harbor entrance, and past the end of the south groin. For waves from the west deepwater direction (Photo 2), a convergence of longshore currents created rip currents at the shoreward terminus of the north jetty. Strong longshore currents were observed moving across the harbor entrance and past the south groin. For waves from the southwest deepwater direction (Photo 3), northerly longshore currents formed

along the inner end of the north jetty. Currents in the vicinity of the harbor entrance were generally confused with rip currents and eddies forming to the south. In general, currents for waves from the south deepwater direction (Photo 4) were characterized by strong longshore currents moving past the end of the south groin, across the harbor entrance, and along the north jetty. In general, for all waves, currents were stronger for tests conducted at mllw than at mhhw. Considerable overtopping of the north jetty was observed at mhhw. Wave-generated currents inside the harbor were very slow.

39. Tracer tests for waves from the northwest deepwater direction (Photos 5 and 6) showed a southerly movement of tracer material along the north beach, along the north jetty, and past the harbor entrance. Much of this material remained in the entrance channel. For waves from the west deepwater direction (Photos 7 and 8), tracer material moving alongshore was caught in rip currents at the shoreward terminus of the north jetty. Tracer material moving past the end of the north jetty was carried into and/or past the entrance channel. For waves from the southwest deepwater direction (Photo 9), a northerly movement of tracer material along the shoreward section of the north jetty was observed. At the harbor entrance, there was some movement of tracer material directly into the harbor. Tracer tests for waves from the south deepwater direction (Photo 10) showed substantial movement of tracer material past the end of the south jetty and some movement into the entrance channel. In some instances, tracer material moved past the channel to end up on the seaward side of the north jetty. For all waves, movement of tracer material was generally greater at mllw than at mhhw. A substantial amount of tracer material was carried over and through the north jetty for waves at mhhw. No overtopping was observed at mllw.

Harbor improvement plans

40. As an expedient in developing an effective sand trap in the overlapping area between the existing north jetty and the proposed offshore breakwater, Plan 1 and subsequent variations first were tested using waves from the west deepwater direction. When a plan was found that effectively trapped tracer material outside the harbor entrance,

it then was tested for waves from the south deepwater direction to determine its effectiveness in trapping tracer material moving northward.

41. Tracer tests from the west for Plan 1 (1400-ft-long breakwater) showed that tracer material which had previously bypassed the harbor entrance for existing conditions now became entrapped in the entrance. Also, significant amounts of tracer material were observed passing over and through the north jetty.

42. Tracer tests for Plan 1A (2200-ft-long breakwater) showed a reduction in tracer movement. However, the amount of tracer material entering the harbor still was significant.

43. Tracer tests showed that Plan 1B (2600-ft-long breakwater) essentially eliminated tracer movement into the harbor entrance. However, due to the large volume of rock required for construction (Table 7), this structure was not considered economically feasible.

44. In an effort to determine the minimum length of structure required to prevent shoaling of the harbor entrance, the 2600-ft-long structure was shortened to 2500 ft (Plan 1C). Tests results showed that this structure effectively prevented harbor entrance shoaling for all waves.

45. The structure then was shortened to 2400 ft (Plan 1D) and test results showed that this structure effectively prevented harbor entrance shoaling with the exception of the 11-sec, 10-ft wave from the west at mllw for which Plan 1D was considered marginally adequate. Considering that this wave condition occurs an average of only 1 hour per year, however, the optimum length for a breakwater structure at this location appeared to be 2400 ft.

46. Tracer tests for Plan 2 showed that moving the breakwater 300 ft closer to the harbor had little effect on the performance of the structure. The plan was still considered marginally adequate for the 11-sec, 10-ft wave at mllw. However, the volume of rock required for construction was significantly reduced.

47. Tracer tests for Plan 2A revealed that a 100-ft-long stub breakwater tied into the north jetty 300 ft from the end increased current velocities moving between the breakwater and, therefore, increased

tracer movement into the harbor entrance.

48. Tracer tests for Plan 2B, which involved relocating the 100-ft-long stub breakwater 850 ft from the end of the north jetty, showed that currents caused by waves diffracting around the end of the breakwater were intercepted and forced into deeper water. This caused the tracer material to deposit more readily in the trap area for all waves tested.

49. Tracer tests then were conducted for Plan 2B using waves from the south deepwater direction. For waves at mhhw, tracer material collected in eddies south of the tip of the proposed south jetty extension. However, for waves at mllw, tracer material moved past the end of the proposed south jetty extension and deposited in the entrance channel.

50. In an attempt to intercept the northerly longshore current and force it to eddy, the south jetty extension was aligned with the existing dogleg (Plan 3). Tracer tests for Plan 3 revealed that the longshore current was redirected across the harbor entrance, resulting in large tracer deposits.

51. Plan 4 involved a 500-ft-long dogleg installed at the end of the proposed south jetty extension. Tracer tests showed that the longshore currents were forced back toward the south, thus creating an eddy in which tracer material was deposited. Exceptions were for the 7-sec, 10-ft and 9-sec, 10-ft waves at mllw. For these waves, some tracer material moved past the 500-ft-long extension but did not shoal the entrance. In all cases, the 400-ft-minimum channel width specified by the SPL was maintained. All subsequent waves brought the tracer material back into the eddy and not into the harbor.

52. In an effort to reduce the cost of the south jetty extension, a straight extension (Plan 5) terminating at the same location as Plan 4 was tested. However, tracer tests showed shoaling of the harbor entrance due to the reduced eddying effect.

53. Comprehensive tests then were run on the selected best plan (Plan 4). Wave-height measurements for Plan 4, presented in Tables 8-11, showed a marked reduction in wave energy entering the harbor. In determining wave-height reductions, values recorded at specific gage

locations (Plate 5) were averaged for all waves tested and compared with those for existing conditions. Wave-height reductions for Plan 4 averaged 73 percent in the harbor entrance (gages 4-6) and 60 percent in the turning basin (gages 9 and 10). Maximum wave heights for Plan 4 were reduced by 52 percent in the harbor entrance (gage 4) and 38 percent in the turning basin (gage 10).

54. Current patterns and magnitudes for Plan 4 for waves from the northwest deepwater direction (Photo 11) exhibited moderate longshore currents along the north jetty, which moved into the lee of the offshore breakwater and exited between the offshore breakwater and south jetty extension. For waves from the west deepwater direction (Photo 12), a rip current formed at the shoreward terminus of the north jetty as for existing conditions. However, the 2200-ft-long offshore breakwater and 100-ft-long stub intercepted the southerly flowing longshore currents with the stub forcing them into deeper water. For waves from the southwest deepwater direction (Photo 13), northerly longshore currents formed along the north jetty. Currents in the vicinity of the harbor entrance were generally slow and confused, with eddies occurring to the south of the south jetty. For waves from the south deepwater direction, the strong longshore current moving past the harbor entrance for existing conditions was forced to eddy due to the addition of the 400-ft-long dogleg to the proposed south jetty extension. Exceptions were for the 7-sec, 10-ft and 9-sec, 10-ft waves at mllw (Photo 14), which were characterized by strong rip currents moving southerly past the dogleg. In general, for all waves, currents were stronger for tests conducted at mllw than at mhhw. Currents inside the harbor remained very low.

55. Tracer tests for Plan 4 for waves from the northwest deepwater direction (Photo 15) showed southerly movement of tracer material along the north jetty and into the overlapping area between the breakwater and jetty, where the tracer material was deposited. A small amount of tracer material migrated to the north part of the entrance channel for the 7-sec, 10-ft wave at mllw. However, due to the limited duration of this wave, this was not considered serious. Tracer tests for waves from the west deepwater direction (Photo 16) showed a deposit

at the middle of the north jetty and in the overlapping area between the breakwater and jetty. No tracer material moved into the entrance channel. For waves from the southwest and south deepwater directions (Photos 17 and 18), tracer material moved northerly past the south groin and into an eddy between the south jetty and the south groin. For most waves, no tracer material moved past the end of the 400-ft-long dogleg extension. Exceptions were for the 7-sec, 10-ft and 9-sec, 10-ft waves from the south at mllw. For these waves, some tracer material moved southerly past the end of the 400-ft-long extension but did not shoal the entrance. Subsequent smaller test waves brought this tracer material back into the eddy and not into the harbor. In all cases, the 400-ft-minimum channel width specified by SPL was maintained.

56. Tracer tests for Plan 19 (inner breakwater and extended north and south jetties) for waves from the south deepwater direction showed, in general, the formation of an eddy adjacent to the 700-ft-long south jetty extension. The south jetty extension worked well for all waves at mhhw. However, at mllw, the surf zone moved seaward; and for the larger waves, the strong longshore current at the initial breaking point was beyond the reach of the south jetty extension. This resulted in large deposits of tracer material in the harbor entrance (Photo 19).

57. In an effort to intercept this longshore current and force it to eddy, the south jetty was extended a total of 950 ft with no dogleg (Plan 20). Test results showed improved shoaling conditions with tracer material being deflected seaward for the 9-sec, 10-ft wave (worst wave condition from previous tests) at mllw (Photo 20). However, the formation of a shoal in this area may be undesirable.

58. In an attempt to force the currents observed along the Plan 20 south jetty to eddy, a 300-ft-long dogleg was added to the south jetty extension of Plan 20 (Plan 21), thereby bringing the total extension length to 1250 ft. In general, the dogleg forced the longshore current to eddy and deposit tracer material south of the harbor (Photo 21). For the 9-sec, 10-ft wave from the south at mllw (Photo 22), a small amount of tracer material was forced seaward past the end of the south jetty extension. The harbor entrance remained relatively unobstructed,

however; and subsequent smaller waves carried this material back shoreward and not into the harbor entrance.

59. Tracer tests of the north jetty extension of Plan 21 for waves from the northwest deepwater direction (Photo 23) showed that tracer material moved along the north jetty and around the end of the north jetty extension where it deposited in the entrance channel. Also, significant quantities of tracer material passed over and through the extension. (See paragraph 66 for a discussion of wave tests of Plan 21.)

60. Tracer tests of Plan 21A for waves from the northwest deepwater direction showed that lengthening the north jetty extension by 100 ft merely reduced the rate of tracer material (Photo 24) with some tracer material still passing around and through the extension.

61. The 100-ft addition to the proposed north jetty extension was repositioned (Plan 22) in an effort to deflect currents away from the harbor entrance. Test results (Photo 25) showed little change when compared with Plan 21A.

62. The extension of Plan 22 was lengthened an additional 300 ft (total extension length of 1000 ft) and designated as Plan 22A. Results of tracer tests showed decreased entrance shoaling conditions for waves at mhhw. However, for large waves at mllw (Photo 26), longshore currents were considerably stronger and movement of tracer material around the end of the extension was considerably greater. Due to the apparent excessive length required for a north jetty extension to be effective in this configuration, it was decided to abandon these tests and attempt to trap the tracer material in a more convenient location.

63. Tracer tests for Plan 23 (a 1200-ft-long offshore breakwater constructed north of the north jetty, protecting a 30-ft-deep deposition basin, and north jetty extension removed) for waves from the northwest deepwater direction (Photos 27 and 28) showed that tracer material (fed into the breaker zone north of the basin) moved to the south and into the sheltered deposition basin, where the longshore currents slowed in the deeper water and the tracer material was deposited. A small amount of fine tracer material passed through the trap for a limited number of conditions, but this should pose no problem. Tracer tests for waves

from the west deepwater direction (Photo 29) showed a decreased rate of coal transport. No coal tracer bypassed the sand trap.

64. Plan 24 consisted of a 1200-ft-long offshore breakwater constructed seaward of the middle leg of the north jetty, protecting a 30-ft-deep deposition basin. The deposition basin of Plan 23 was filled with coal tracer to restore natural depths in this area to prevent any undue variations in longshore currents. Test results for waves from the west deepwater direction (Photo 30) showed little movement of coal tracer out of the sand trap. However, due to the convergence of longshore currents from the north and the currents flowing north along the jetty, a rip current was formed which carried some coal tracer seaward where it migrated south outside the sand trap. Test results for waves from the northwest deepwater direction (Photo 31) showed no loss of coal tracer from or around the sand trap.

65. Wave-height tests were conducted for Plan 25 using waves from the northwest, west, southwest, and south deepwater directions. Representative wave pattern photographs are shown in Photos 32-35. Wave-height data, presented in Table 12, showed an increasing amount of wave energy entering the inner basin as the wave direction changed from northwest to south. Wave heights exceeded the criteria (1.5 ft in berthing areas and 4.0 ft in the entrance) frequently, particularly for the longer period (14 sec and 17 sec) waves. Waves from the southwest and south test directions were observed moving unobstructed between the jetties and into the entrance channel. Waves diffracting around the end of the inner breakwater accounted for most of the wave energy in the inner basin.

66. In an attempt to lessen wave heights in the entrance channel and inner basin, the north and south jetty extensions were added (Plan 21) and tests were conducted using waves from the south and southwest. Tracer tests for this plan were described in paragraphs 58 and 59. Wave-height data for Plan 21 (Table 13) showed, in general, a slight decrease in wave energy when compared with Plan 25. There were, however, some increases in wave heights for the 17-sec, 10-ft wave from the southwest. Indications are that the jetty extensions (particularly the

one on the south) may be funneling some wave energy into the harbor.

67. In an effort to reduce harbor wave heights for this condition, the 2200-ft-long inner breakwater was lengthened 200, 300, and 400 ft (Plans 21B, 21C, and 21D, respectively). Also tested was Plan 21 with a 400-ft-long shore-connected structure opposite the end of the inner breakwater (Plan 21E). As an expedient, only five wave gages in the inner basin and entrance were monitored for these tests. Results of wave-height tests for Plans 21B-21E (Table 13) showed Plan 21D to be the best of these plans with respect to entrance and inner harbor wave conditions for the 17-sec, 10-ft wave from the southwest. This plan then was tested for all waves from this direction using all 14 gages. Test results (Table 13) showed that maximum wave heights (1.8 and 1.7 ft) in the inner basin exceeded the desired criterion (1.5 ft) slightly, but this may not be a serious problem due to the infrequency of the waves producing these heights. Wave heights in the entrance were as high as 6.2 ft.

68. Results of wave-height tests for Plan 21F using waves from the southwest and west deepwater directions (Table 13) showed slightly increased wave heights in the inner basin and entrance channel. A typical wave pattern photograph for this plan is shown in Photo 36.

69. Results of wave-height tests for Plan 25A (Plan 21F without the jetty extensions) using waves from the west and southwest deepwater directions (Table 14) showed excessive wave heights at gage 9 for the 14- and 17-sec waves. A typical wave pattern photograph for Plan 25A is shown in Photo 37.

70. Wave-height test results for Plan 26 for waves from the southwest deepwater direction (Table 15) indicated that raising the crown elevation of the north jetty to +22 ft and making it impervious did not reduce wave heights to an acceptable level. Most of the wave energy entering the inner basin passed over and through the inner breakwater. A typical wave pattern photograph is shown in Photo 38.

71. Results of wave-height tests for Plan 26A (200 ft of shoreward terminus of inner breakwater removed) for waves from the southwest deepwater direction (Table 15) revealed a slight increase in wave heights in the inner basin (when compared with Plan 26) with most of the wave

energy still passing over and through the inner breakwater.

72. Wave-height test results for Plan 27 (raising inner breakwater crown elevation to +16 ft) (Table 16) indicated that for waves from the southwest, wave heights in the inner basin were reduced to a marginally acceptable level (still slightly exceed the 1.5-ft criterion but only for the infrequent 14- and 17-sec waves).

73. In an attempt to determine the minimum amount of structure required for inner basin wave protection, lengths of 200, 400, and 600 ft were removed from the shoreward terminus of the inner breakwater (Plans 27A, 27B, and 27C, respectively) and tests were conducted for the 17-sec, 10-ft wave. Optimum wave-height conditions for these plans occurred for Plan 27B (Table 16). Wave-height tests for Plan 27B using all waves from this direction (Table 16) showed acceptable wave heights for all gages in the inner basin except for the 14- and 17-sec waves. Wave conditions in the entrance, however, continued to substantially exceed the 4.0-ft maximum criterion. A typical wave pattern photograph is shown in Photo 39.

74. Wave-height test results for Plan 28 for waves from the southwest deepwater direction (Table 17) showed a significant reduction in entrance wave conditions when compared with previous plans; however, the 4.0-ft criterion was still exceeded. A typical wave pattern photograph of Plan 28 is shown in Photo 40.

75. Wave-heights for Plan 29 (offshore breakwater and no inner breakwater) using 14- and 17-sec waves from the southwest deepwater direction (Table 17) showed improved entrance conditions but excessive wave heights in the inner basin. Substantial wave energy was observed passing over and through the offshore breakwater as illustrated in Photo 41.

76. Wave-height test results for Plan 30 (Table 17) showed improved but still excessive wave heights in the inner basin. Raising the crown elevation of the offshore breakwater eliminated overtopping; however, transmission of wave energy through the structure and diffraction of wave energy around the south end of the structure remained significant.

77. Wave-height test results for Plan 30A (Table 17) showed decreased yet still excessive wave heights in the inner basin.

78. Wave-height test results for Plan 31A (impervious offshore breakwater) for waves from the southwest deepwater direction (Table 18) showed excessive wave heights for the 14-sec wave.

79. Wave-height test results for Plan 31A (200-ft extension of offshore breakwater) for waves from the southwest deepwater direction (Table 18) showed a reduction of wave heights to a more acceptable level; however, wave heights for waves from the south deepwater direction (Table 18) were excessive in the inner basin.

80. Wave-height test results for Plan 31B (300-ft extension of offshore breakwater) using waves from the south deepwater direction (Table 18) showed increased wave heights when compared with Plan 31A.

81. Wave-height test results for Plan 31C (400-ft extension of offshore breakwater) for waves from the south test direction (Table 18) showed increased wave heights when compared with Plan 31B.

82. Results of wave-height tests for Plan 31D (200-ft dogleg extension of offshore breakwater) using waves from the south (Table 18) revealed increased wave heights when compared with Plan 31C.

83. At this point, transmission of wave energy through the breakwater was reevaluated and a careful check revealed that the sheet metal placed in the breakwater to make it impervious was not making a complete seal. After the structure had been rebuilt, making it completely impervious, a check test was run for Plan 31D using the same 17-sec, 6-ft wave from the south. Results (Table 18) indicated a significant decrease in inner basin wave heights.

84. Wave-height test results for Plan 31E (200-ft southern dogleg extension and 200-ft northern extension to offshore breakwater) showed a small decrease in inner basin wave heights (Table 18).

85. Plan 31A then was retested to determine the effect of resealing the breakwater. The result (Table 18) was a lowering of wave heights to a marginally acceptable level.

86. Wave-height test results for Plan 31F (400-ft southern extension and 200-ft northern extension to offshore breakwater) for waves from

the south and west test directions (Table 18) showed acceptable wave heights for both directions, except for the 9-sec, 12-ft wave from the west which was marginally acceptable. A typical wave pattern photograph for Plan 31F is shown in Photo 42.

87. As an expedient, tracer tests of the following sand trap plans were conducted using waves from the northwest deepwater direction only. Tracer tests for Plan 32 (1350-ft breakwater attached to north jetty) showed the formation of a strong eddy at the sand trap entrance. Longshore currents were forced back to the north by the offshore breakwater. As the currents slowed upon entering deeper water, the waves pushed the tracer material shoreward as illustrated in Photo 43.

88. Tracer test results for Plan 33 (350-ft gap between north jetty and 1000 ft breakwater), shown in Photo 44, indicated that tracer material moved farther into the trap area because most of the longshore currents were allowed to escape through the trap without being forced to double back to the north, thereby reducing the magnitude of the corresponding eddy.

89. Tracer test results for Plan 34 (low-sill connecting structure in 350-ft gap) showed that part of the longshore current dissipated over and through the low sill while the remainder moved north along the inside of the breakwater to form an eddy (Photo 45). This configuration effectively trapped the tracer material in the center of the trap. No tracer material was observed bypassing the trap.

90. Results of tracer tests for Plan 35 (500-ft groin added north of trap) showed some loss of tracer material seaward past the end of the breakwater due to the seaward deflection of longshore currents by the 500-ft-long groin.

91. Tracer tests for Plan 35A showed that shortening the groin to 400 ft reduced the seaward deflection of longshore currents. Loss of tracer material seaward of the breakwater was reduced and movement of tracer into the trap was improved.

92. Tracer test results for Plan 36 (breakwater shortened 200 ft and groin moved 200 ft south) showed the formation of an eddy in the trap entrance. However, some tracer material was still observed moving seaward of the breakwater.

93. Tracer test results for Plan 37 (low-sill connecting structure and 400-ft-long groin removed; added 650-ft curved groin) showed an increased rate of accumulation of tracer material in the trap area. The curved section of the groin tended to direct some longshore current into the trap.

94. Results of tracer tests for Plan 38 (removed 250-ft curved section of groin) indicated a strengthening of currents inside the trap area, due to increased wave energy entering the trap, which moved some of the finer particles through the 350-ft-wide gap.

95. Tracer tests for Plan 38A (added low-sill connecting structure) showed that the low sill prevented loss of coal tracer through the 350-ft-wide gap. However, this low sill caused some backup of current in the trap which resulted in some fines being lost seaward of the breakwater.

96. Tracer tests for Plan 38B (lengthened groin 150 ft) showed that due to the increased depth of water at the groin head and the reduced amount of wave energy entering the trap, the tracer material moved more slowly around the groin and into the trap. There was no loss of coal tracer through the 350-ft-wide gap and very little loss of tracer material past the end of the breakwater (only a portion of the tracer material which was very fine and easily held in suspension).

97. Tracer tests for Plan 38C (lengthened groin another 100 ft) showed reduced movement of tracer material past the groin head. A small eddy was observed to the north of the groin near its outer end which was the result of currents created by waves breaking at the groin head opposing the longshore currents. Virtually no tracer material moved seaward of the breakwater.

98. Tracer tests for Plan 38D (lengthened groin another 100 ft) showed a significant loss of tracer seaward of the breakwater. When the groin was extended past the breaker zone, the deflected longshore currents became stronger than the wave forces acting on the tracer.

99. Tracer tests for Plan 39 (total groin length of 750 ft, including 350-ft curve) showed that the longshore currents bypassed the groin and entered the trap area. Also, the curved section of the groin

funneled wave energy into the trap which caused some loss of tracer material through the 350-ft-wide gap.

100. Tracer tests for Plan 39A showed that the addition of the 350-ft-long low-sill connecting structure prevented tracer material from leaving the trap area but did not generate a large-scale eddy.

101. Tracer tests for Plan 40 (1000-ft offshore breakwater; low-sill connecting structure; 800-ft groin moved 500 ft north) showed movement of tracer material past the groin head and into an eddy in the lee of the groin. Tracer material also was carried into the trap area and formed another eddy. No tracer material was lost from the trap and very little was lost seaward.

102. Results of tracer tests for Plan 40A (groin lengthened 100 ft) were the same as those for Plan 40.

103. Tracer tests for Plan 40B (groin lengthened another 500 ft) showed a significant amount of tracer passing seaward.

104. Tracer tests for Plan 41 (removed offshore breakwater and low-sill connecting structure; groin length 1500 ft, including 600-ft curve to the north) showed that the curved portion of the groin forced the longshore current into an oblong eddy with no tracer movement past the groin.

105. Results of tracer tests for Plan 42 (800-ft groin with outer 400 ft curved to the north) showed a loss of tracer material seaward past the groin head.

106. Tracer tests for Plan 43 (groin angled 30 deg to the north) showed improved yet still undesirable seaward movement of tracer material.

107. Tracer tests for Plan 43A (groin lengthened 200 ft) showed the development of an oblong eddy with no tracer movement past the groin.

108. At this point, a comparison was made of all the north sand trap plans tested and the best plan was selected using the following criteria.

- a. Must effectively prevent tracer from entering the harbor entrance.
- b. Should build as large a fillet as possible shoreward of

the sand trap for maximum natural sand storage for waves from the northwest.

c. The sand trap must effectively contain the excess material once the fillet has reached its maximum volume.

d. Should allow material accumulated in the fillet to return to the north for subsequent waves from southerly directions.

Plan 38C (Plate 27) appeared to offer the most effective and economical solution and then was tested for 7-sec, 4- and 10-ft waves from the northwest at mllw and mhhw with tracer material continuously being fed into the breaker zone. The 10-ft and 4-ft waves were run for a total of 6 hr (model time) and 2 hr (model time), respectively, at each water level. To prevent scale effects from model circulation, each continuous test run was limited to 30 min (model time). Observations showed that for the 10-ft wave, tracer material moved along the breaker zone and out to the groin head where the breaking waves pushed it shoreward along the groin. In this way, the fillet began building from the groin toward the north. The fillet was allowed to build to a maximum volume and then spill into the sand trap. Photographs taken with and without waves (the shoreline marked with string for the latter) are shown in Photos 46 and 47. Plots of the original and final shorelines are presented in Figure 9.

109. Without disturbing the tracer fillet, the wave generator then was moved to the southwest test direction and Plan 38C was tested using 9-sec, 16-ft, 11-sec, 8-ft, and 17-sec, 10-ft waves at both mhhw and mllw to determine the effectiveness of this plan in allowing the tracer material accumulated in the fillet to return to the north. Each wave was run a total of 2 hr (model time) in the following sequence:

Period sec	Height ft	swl
11	8	mllw
9	16	mllw
17	10	mllw
11	8	mhhw
9	16	mhhw
17	10	mhhw

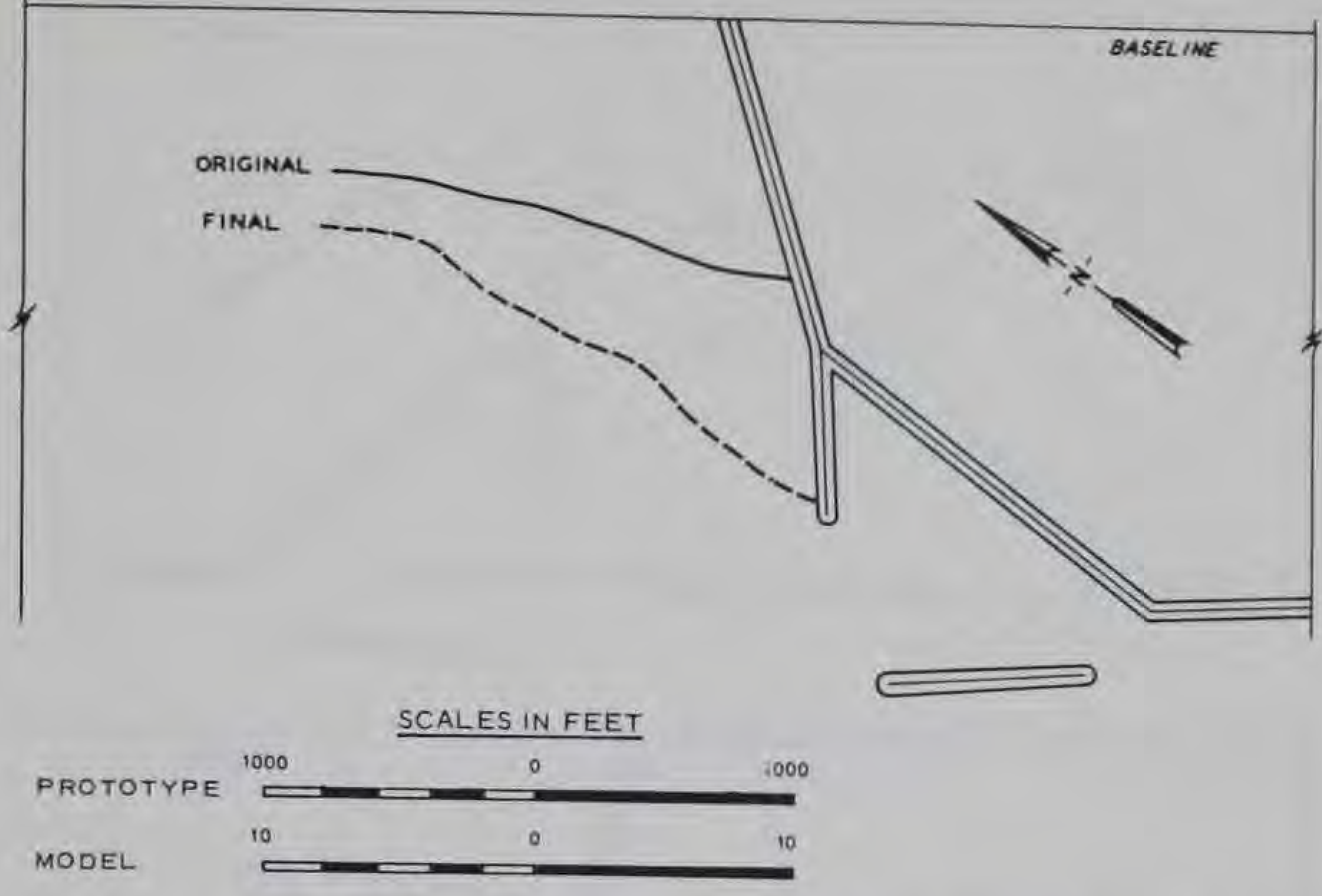

Figure 9. Shoreline plots for Plan 38C after 7-sec, 10-ft waves from northwest at mllw

The tests were run consecutively with no reshaping of tracer material between tests. Figures 10 and 11 show the resultant shoreline for each test versus the original mllw shoreline. It was observed that all but a small fillet of tracer material next to the groin was eventually displaced to the north. Also, for all waves (particularly the 9-sec, 16-ft wave), much of the tracer material that had deposited in the sand trap was pushed around the end of the groin and to the north (Photos 48 and 49). It should be pointed out that the tracer material observed in the harbor and entrance channel in Photos 46-49 resulted from washing out of tracer material that had been trapped in the voids of the north jetty from previous tests. Only a very small percentage of the tracer material, consisting of the very smallest fines (i.e. dust) easily held in suspension, managed to bypass the north trap.

110. A 30-ft-deep deposition basin was installed in the trap area of Plan 38C (Plan 44), the fillet of maximum volume for the Plan 38C groin was reconstructed with tracer material, and the plan was tested using 7-sec, 10-ft waves from the northwest at mllw and mhhw. Test results (Photos 50 and 51) showed that tracer material moved around the

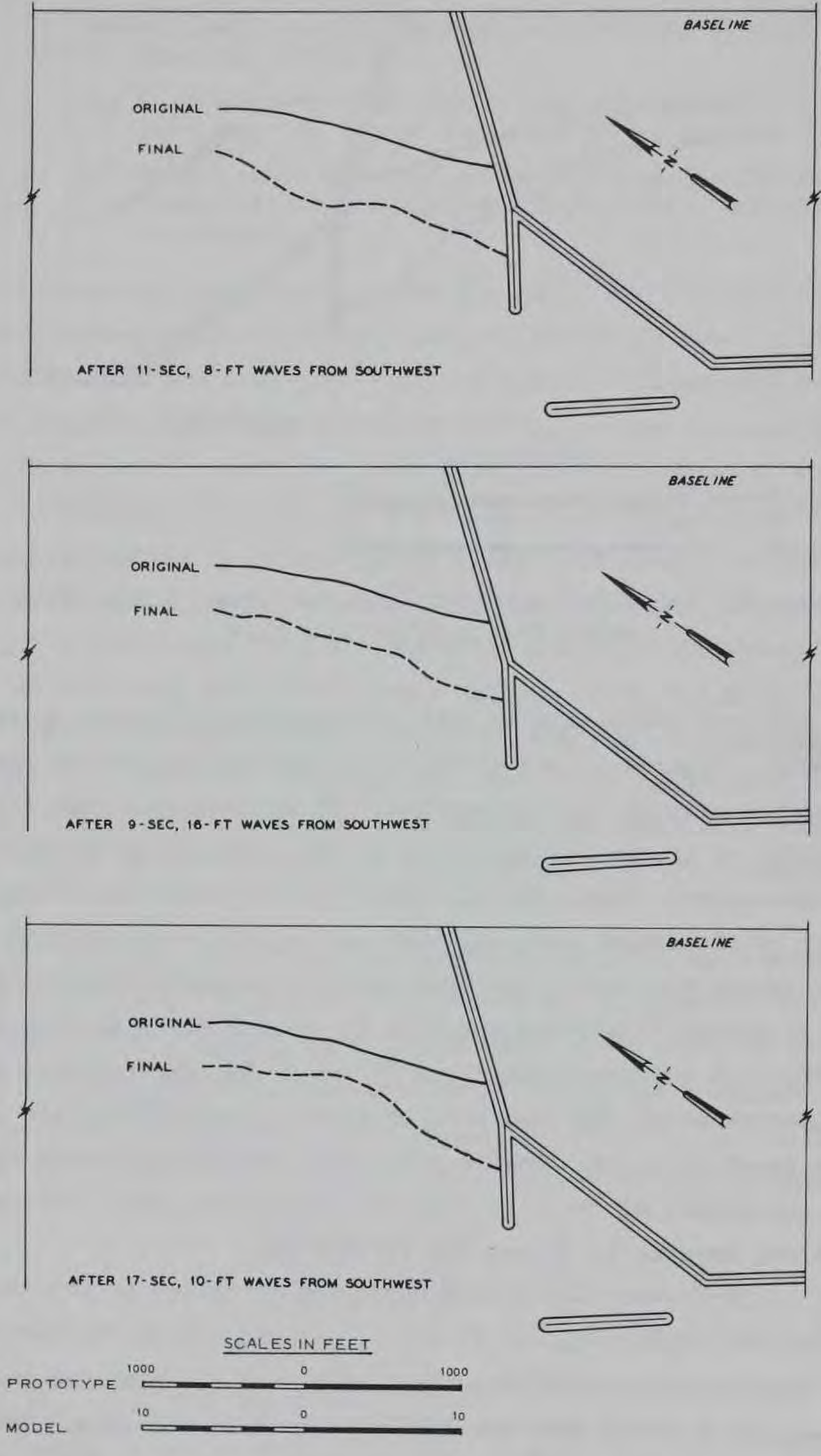

Figure 10. Shoreline plots for Plan 38C, waves from southwest at mllw

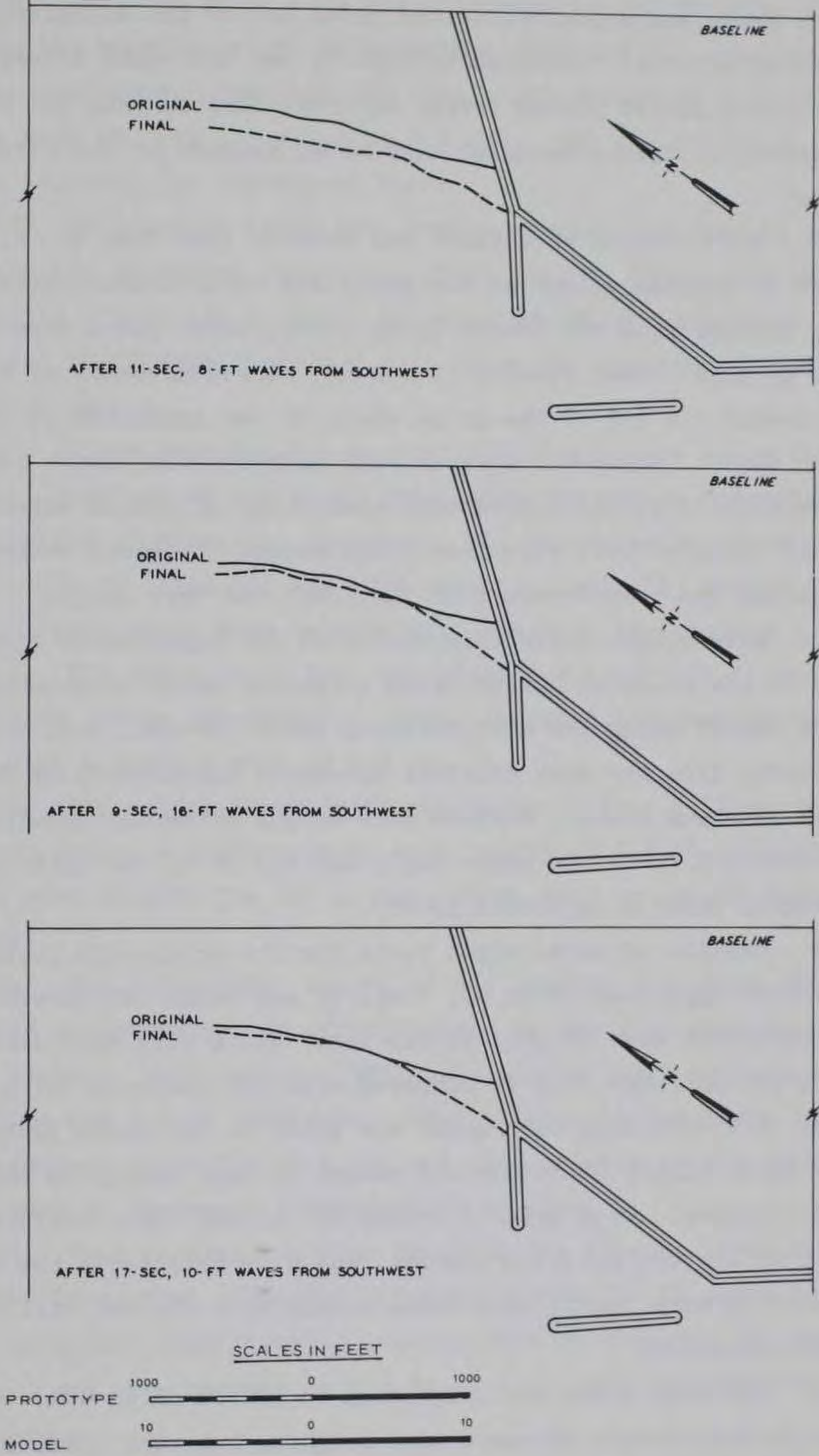

Figure 11. Shoreline plots for Plan 38C, waves from southwest at mhhw

end of the groin and deposited in the north end of the deposition basin. As the tracer material continued to deposit and the water became shallower, wave forces pushed tracer material farther into the basin. A small amount of tracer material also moved seaward of the offshore breakwater.

111. The offshore breakwater was removed from Plan 44 (Plan 45), the fillet of maximum volume at the groin was reconstructed with tracer material, and the plan was tested using 7-sec, 10-ft waves from the northwest at mllw. Test results (Photo 52) showed movement of tracer material around the end of the groin where it was deposited in the deposition basin. Because the waves were unobstructed, those breaking on the north jetty forced substantial quantities of tracer material through the voids of this structure. The amount of tracer material which bypassed the trap (seaward of the trap) was very small.

112. Wave-height tests were conducted for improvement plans for expansion of the existing harbor using a revised inner breakwater. The plans were tested using the most critical waves (14-sec, 6-ft and/or 17-sec, 10-ft) from the most critical direction (southwest) as determined from previous tests. Maximum wave-height criteria, provided by SPL, remained 1.5 ft in the inner basin and 4.0 ft in the inner entrance channel (gage 4 in Plates 35-49).

113. Results of wave-height tests for the originally proposed revised harbor expansion (Plan 46, Plate 35 and Table 19) showed slightly excessive wave heights in the inner basin with very large wave heights in the entrance (7.9 ft compared with the criterion of 4.0 ft).

114. A 250-ft-long stub groin was added to the south jetty (Plan 47) in an effort to reduce the amount of wave energy in the inner basin and entrance. Wave heights (Table 19) in the inner basin were reduced to within the 1.5-ft criterion, and wave heights in the entrance were reduced to more nearly acceptable levels (5.1 ft) but still exceeded the criterion.

115. The stub groin was lengthened to 300 ft (Plan 47A) in an effort to further reduce entrance wave heights, but test results (Table 19) showed little change over the preceding test.

116. The stub groin then was shortened to 200 ft (Plan 47B) to determine the minimum length of structure required for acceptable harbor wave conditions. Test results (Table 19) showed an increase in entrance wave heights (from 5.1 to 5.9 ft). The 250-ft length therefore was selected for subsequent tests.

117. Wave-height tests then were conducted for Plan 47 using all waves from the southwest direction. Test results (Table 19) showed wave heights within the criteria selected by SPL for all waves except the 17-sec, 10-ft wave where a 5.1-ft value was recorded in the entrance at gage 4.

118. Since wave heights in the inner basin were below the 1.5-ft criterion, sections of the inner jetty were removed to determine the minimum length of structure required. The structure was shortened from 300 ft to 250 ft (Plan 48) and test results (Table 19) showed an increase in inner basin wave heights with one gage registering 1.7 ft.

119. The inner jetty then was shortened to 200 ft (Plan 48A), and test results (Table 19) showed a slight increase in inner basin wave heights with a maximum wave of 1.8 ft.

120. The entire inner jetty was removed (Plan 48B) and results (Table 19) showed a maximum basin wave height of 1.8 ft. However, the entrance wave heights for the 17-sec, 10-ft wave increased significantly to 6.3 ft.

121. The north jetty extension was removed from Plan 48B (Plan 49) and tested to determine its effectiveness in reducing wave energy entering the harbor. Test results (Table 19) showed inner basin wave heights to be below the 1.5-ft criterion. Wave heights in the inner basin entrance remained excessive (6.4 ft) and wave heights in the harbor entrance (gage 1) increased substantially to 16.8 ft.

122. Test results for Plan 50 using the 17-sec, 10-ft wave from southwest showed that extending the north jetty 300 ft along its present alignment significantly reduced wave heights for all gages (Table 19) and wave heights in the inner basin and entrance were within the criteria.

123. In an effort to reduce the amount of structure required for

construction, the north jetty extension was shortened to 200 ft (Plan 50A) and tested. Results (Table 19) showed an increase in wave heights in the inner basin and entrance. Inner basin wave heights remained within the 1.5-ft criterion; however, gage 4 exceeded the 4-ft criterion with a height of 5.2 ft.

124. The 250-ft-long stub groin was removed from Plan 50A (Plan 51) and test results (Table 19) showed a substantial increase in wave heights in the inner basin and entrance. Both the 1.5-ft and 4.0-ft criteria were exceeded.

125. The 300-ft-long inner jetty was added to Plan 51 (Plan 52) in an effort to reduce inner harbor wave heights. Results (Table 19) indicated a very slight reduction but both criteria still were exceeded.

126. The 200-ft-long north jetty extension of Plan 52 was removed (Plan 53) and tests showed considerable wave energy in the inner basin and entrance (Table 19) with a 2.2-ft wave recorded at gage 8.

127. A 200-ft-long dogleg was installed at the end of the inner jetty (Plan 54) in an effort to reduce the amount of wave energy diffracting around the end of the inner breakwater. Results (Table 19) showed a slight decrease in wave heights in the inner basin, but the wave height at gage 8 still exceeded the 1.5-ft criterion.

128. The inner basin entrance was completely sealed (Plan 55) in an effort to determine the amount of energy entering the basin through the inner breakwater. Test results (Table 19) showed that wave heights were lessened at gage 8 and increased at gages 6, 7, and 9 which indicated that excessive wave energy was being transmitted through the inner breakwater.

129. The inner basin entrance was reopened, the 200-ft-long inner jetty dogleg was removed, and the final 1300-ft leg of the inner breakwater was sealed (Plan 56). Test results (Table 19) showed a reduction of wave heights in the inner basin to within the criterion.

130. The entire inner breakwater then was sealed (Plan 57) and test results (Table 19) showed an increase in inner basin wave heights exceeding the 1.5-ft criterion. This increase was unexpected, and the cause appeared to be reduced interference of diffracted and transmitted

waves and wave energy being reflected from the vertical wall of the sealed section.

131. Plan 50 appeared to be the best of the revised expansion plans tested and therefore was selected for further testing. In order to obtain a more complete set of wave-height data in the inner basin, gages 1-3 were moved to locations in the inner basin as shown in Plate 52. Plan 50 then was tested using all waves from the south, southwest, and west deepwater directions (Table 19). Wave heights in the entrance exceeded 4 ft three times and wave heights in the inner basin exceeded 1.5 ft three times.

132. The 300-ft-long inner jetty was added to Plan 50 (Plan 58) and tested for the waves that exceeded the 1.5-ft criterion. Inner basin wave heights (Table 19) were reduced to within the 1.5-ft criterion except for the 17-sec, 6-ft wave where wave heights at gages 3 and 8 actually increased. This wave was tested several times to verify this fact.

133. The final 1300 ft of the inner breakwater was resealed (Plan 59) and tested for the 17-sec, 6-ft wave. The result (Table 19) was a substantial reduction of inner basin wave heights to a level well below the 1.5-ft maximum criterion.

134. In an effort to reduce construction costs yet still provide adequate inner basin wave protection, the elevation of the core of the final 800 ft of the inner breakwater of Plan 58 was raised to an elevation of +4.0 ft (Plan 60) and tested for the 17-sec, 6-ft wave. The resultant inner basin wave heights (Table 19) were within the 1.5-ft criterion.

135. A portion of this study was devoted to developing effective and economical ways to prevent harbor shoaling by sand bypassing and/or backpassing. Two plans (Plans 61 and 62) were designed and tested for use with the Eductor Jet Pump System developed and tested by WES. Since a fixed jet pump installation operates within a 500-ft radius, a circular deposition basin of the same radius was constructed on both sides of the harbor entrance.

136. Tracer tests were conducted for Plan 61 (Plate 50) using

9-sec, 4- and 10-ft waves from the south at mhhw and mllw. For the small waves, tracer movement was generally onshore. For the 10-ft wave at mhhw (Photo 53), tracer material moved into the south trap with no bypassing. For the 10-ft wave at mllw (Photo 54), however, the surf zone migrated seaward causing most tracer material to deposit at the seaward edge of the trap with some material bypassing the trap. No tracer material entered the harbor, however.

137. Tracer tests were conducted for Plan 62 (Plate 51) using the same test conditions as for Plan 61, and results (Photos 55 and 56) were generally the same. Since the south trap for this plan did not extend as far seaward as Plan 61, more tracer material was able to bypass the trap but no tracer material entered the harbor.

138. Tracer tests then were conducted for Plan 62 using 7-sec, 5- and 10-ft waves from the northwest at mhhw and mllw. Results showed that for the 5-ft waves, movement of tracer material was generally onshore. For the 10-ft waves (Photos 57 and 58), tracer material moved into the north side of the north deposition basin. No tracer material bypassed the trap.

Discussion of test results

139. Test results obtained for existing conditions (with moderate to large incident waves) revealed rough and turbulent wave conditions in the entrance channel due to (a) waves breaking on the shoal across the harbor entrance, (b) waves diffracting around the jetties, and (c) waves overtopping the north jetty. Contributing to these hazardous entrance conditions were strong longshore currents (as high as 10 fps) flowing across the harbor entrance, especially for waves from the south and northwest. Tracer tests indicated that the model accurately reproduced the general sediment patterns observed in the prototype (as evidenced by visual observations and aerial photographs).

140. A comparison of Plans 1, 1A, 1B, 1C, and 1D with existing conditions indicated that Plan 1 was totally ineffective in trapping tracer material outside the harbor entrance and, in fact, contributed to shoaling of the harbor entrance. As the length of the offshore breakwater increased, its effectiveness in the prevention of harbor

shoaling also increased. However, due to the substantially larger volume of rock required for this configuration to be effective (Table 7), these plans were not considered desirable from an economic standpoint.

141. In an effort to reduce the volume of rock required and still prevent shoaling, the offshore breakwater was moved 300 ft shoreward (Plans 2, 2A, and 2B). For waves from the northwest, Plan 2B was effective in preventing shoaling of the harbor entrance while Plans 2 and 2A were only marginally effective. However, Plan 2B was ineffective in preventing shoaling for waves from the south.

142. The south jetty extension was lengthened (Plan 3) to intercept the northerly longshore current, but the current was redirected across the entrance resulting in large tracer deposits. It was obvious that a jetty extension with a southerly seaward leg (similar to Plan 4) was necessary to make the longshore current eddy.

143. A comparison of Plan 4 with existing conditions showed that tracer material which had previously moved south along the north jetty for waves from the northwest and west directions now settled in the trap area between the north jetty and offshore breakwater. Tracer material which had previously moved north past the end of the south jetty and into the harbor for waves from the south and southwest directions was now forced to eddy south of the harbor entrance. Also, wave heights and current magnitudes in the entrance and turning basin were substantially reduced. On the basis of these results, Plan 4 may be considered a viable solution from a functional standpoint but will require a large volume of rock for construction.

144. Plan 5 was ineffective in preventing shoaling of the harbor for waves from the south test direction. As in Plan 3, the longshore currents were redirected across the harbor entrance, resulting in severe harbor shoaling.

145. The south jetty extension of Plan 19 was effective in creating an eddy south of the structure for waves at mhhw. However, at mllw the surf zone moved seaward past the reach of the structure and caused severe shoaling.

146. Plan 20 created a shoal outside the harbor entrance

which may be hazardous to navigation.

147. The south jetty extension of Plan 21 was effective in preventing harbor shoaling for all waves at mhhw and mllw. Some tracer material moved south past the jetty head for the 9-sec, 10-ft wave at mllw but did not shoal the entrance. All subsequent smaller waves moved this material back shoreward.

148. Results of tracer tests on variations of a north jetty extension (Plans 21, 21A, 22, and 22A) revealed that attempting to trap tracer material with this type of structure would require the structure to be excessively long.

149. Plan 23 utilized a 1200-ft-long offshore breakwater (located north of the harbor) to reduce wave action and slow down longshore currents enough to allow tracer material to settle into the 30-ft-deep deposition basin situated in the lee of the structure. Wave forces and currents set up by waves diffracting around the downdrift end of the breakwater assisted in slowing down the longshore currents and preventing tracer material from leaving the trap area. For waves from the west, the convergence of longshore currents from the north and the currents flowing north along the jetty appeared to aid in trapping tracer material. For Plan 24, however, these currents carried some material seaward where it migrated south, bypassing the trap. Also, Plan 23 trapped tracer material moving south before it reached the harbor; whereas for Plan 24, tracer material moved alongside the middle leg of the north jetty before entering the trap where some of this material was forced over and through the voids of the north jetty.

150. A comparison of wave heights for harbor expansion plans with and without north and south jetty extensions (Plans 21 and 25) showed that the addition of the extensions reduced inner basin wave heights slightly. However, there were some increases in wave heights for the 17-sec, 10-ft wave from the southwest indicating that the extensions could be funneling some wave energy into the harbor for the longer periods from this direction.

151. A comparison of plans with various lengths of structure added to the inner breakwater (Plans 21-21D) showed Plan 21D to be the

optimum plan for the 17-sec, 10-ft wave from the southwest. For all waves from this direction, maximum wave heights in the inner basin exceeded the desired criterion of 1.5 ft slightly (1.8 ft). Wave heights in the entrance exceeded the desired criterion of 4.0 ft significantly (6.2 ft).

152. Plan 21E indicated that moving the entrance to the inner basin 400 ft toward the harbor entrance significantly increased inner basin wave heights.

153. Decreasing the elevation of the final 1500 ft of the inner breakwater (Plan 21F) resulted in increased (but possibly acceptable) wave heights in the inner basin.

154. Removing the north and south jetty extensions (Plan 25) resulted in excessive wave heights at gage 9 in the inner basin for the 14- and 17-sec waves.

155. Raising the crown elevation of the north jetty increased wave heights in the inner basin. This was probably due to reduced interference of waves previously passing over and through the north jetty with waves traveling directly into the harbor entrance.

156. A comparison was made of wave heights for plans with various lengths removed from the shoreward terminus of the inner breakwater, with the final 1500 ft of the inner breakwater returned to a +16 ft elevation (Plans 27-27C). Test results showed Plan 27B (400 ft of structure removed) to be the optimum plan with acceptable basin wave heights for all except the 14- and 17-sec waves. Wave heights in the entrance, however, continued to substantially exceed the 4.0-ft maximum criterion (maximum height was 10.0 ft). The addition of the north and south jetty extensions to Plan 27B (Plan 28) reduced entrance wave heights significantly (to 5.0-ft maximum) but they still exceeded the 4.0-ft maximum criterion.

157. The offshore breakwater of Plan 29 reduced entrance wave heights to within the desired 4.0-ft criterion; however, the inner basin wave heights were now excessive (as high as 4.1 ft).

158. A comparison was made of various lengths of an offshore breakwater raised to crown el +22 ft (Plans 30 and 30A). Results showed

improved but still excessive wave heights in the inner basin. It was observed that for these plans, a substantial portion of wave energy entering the harbor was the result of transmission through the breakwater.

159. An examination of data for Plans 31-31F (various lengths and orientations of a sealed offshore breakwater) showed acceptable wave heights in the entrance for all waves but wave heights in the berthing area were in excess of 2 ft for the larger waves. Considering the infrequency of these waves, some of these plans may be considered marginally acceptable from a functional standpoint. However, these plans require a substantially larger quantity of rock when compared with some of the other plans tested.

160. The north sand trap plans (Plans 32-43A) were tested first using waves from the northwest deepwater direction as an expedient in developing an optimum plan. When a promising plan was found, it then was tested using waves from both the northwest and southwest test directions. A comparison of Plans 32-34 showed Plan 34 to be the most effective in containing the tracer material. The low-sill structure allowed some longshore current to pass through which drew tracer material farther into the trap but did not allow the material to exit.

161. Plans 35-40B involved variations in the lengths and configurations of the offshore breakwater, low sill, and groin. It was apparent that as more longshore current was allowed to enter the trap area from the north and exit to the south, the greater was the tendency for tracer material to enter the trap area and not be lost seaward. If the currents entering the trap were large and those exiting the trap were small, a backup resulted which forced longshore currents, and therefore tracer material, seaward. If currents entering the trap were large and those exiting the trap also were large, excessive loss of tracer material through the 350-ft-long gap could occur. So, in effect, these plans were attempts to find the correct configuration which not only prevented loss of tracer material seaward but also satisfied those factors listed in paragraph 108. From initial tests, Plan 38C appeared to offer the most effective and economical solution. This plan retained the tracer

material north of the trap until a fillet of maximum volume was achieved at which time the tracer material spilled past the groin and into the trap. There was no significant loss of tracer material either seaward of the trap or south through a 350-ft-long gap.

162. Attempts were made to create an effective sand trap using only a groin. Plans 41 and 43A showed that the groin effectively prevented tracer material from moving southward by forcing the wave-induced longshore currents to eddy. However, in order to induce the longshore currents to eddy, the groin was necessarily curved to the north. This would, in effect, provide some wave protection to any accumulated fillet and would hamper natural transport of sand from the fillet to the north for subsequent waves from southerly directions. Also, after a fillet of maximum volume accumulated, any additional sand would bypass the groin and migrate toward the harbor entrance.

163. A comparison of Plans 38C, 44, and 45 indicated that Plans 44 and 45 would trap a larger amount of material moving around the groin during a storm than would Plan 38C due to the additional storage of the 30-ft-deep dredged basin. While Plan 44 uses a greater volume of rock than Plan 45, indications are that due to the large waves in this area the amount of material which could be retained in the deposition basin of Plan 45 would be substantially reduced. Wave protection for a dredge or sand bypassing plant also would be a concern. It should be pointed out that all plans involving a sand trap alongside the north jetty had substantial movement of tracer material through the voids of the north jetty, and this structure probably should be sealed.

164. A comparison of Plans 46-47B indicated that Plan 47 was the optimum with wave heights within the criteria requested by SPL for all waves except the 17-sec, 10-ft wave from the southwest for which one gage in the entrance registered 5.1 ft. Since this wave occurs infrequently (less than 1 hr per year), this may be acceptable.

165. A comparison of Plans 48-48B showed that only Plan 48 reduced wave heights in the inner basin to within the desired criterion. However, increases in inner basin wave heights for Plans 48A and 48B were small. Entrance wave heights for Plan 48B for the 17-sec, 6-ft

wave significantly exceeded the 4.0-ft criterion.

166. When the north jetty extension of Plan 48B was removed (Plan 49), inner basin wave heights decreased. This may be due to increased amounts of wave energy transmitted through the inner breakwater interfering with waves diffracting around the end of the inner breakwater into the inner basin. Wave heights in the inner basin entrance remained about the same; however, wave heights in the harbor entrance increased substantially.

167. A comparison of Plans 50 and 50A indicated that both plans reduced inner basin and entrance wave heights to within the desired criteria except for Plan 50A where gage 4 exceeded the 4.0-ft criterion with a height of 5.2 ft.

168. Wave-height tests for Plans 51-54 showed that all of these plans exceeded the desired criteria.

169. Tests of Plans 55 and 56 indicated that excessive wave energy was being transmitted through the inner breakwater.

170. Sealing the entire inner breakwater (Plan 57) caused increased inner basin wave heights by reducing interference of waves traveling through the entrance with waves previously transmitted through the unsealed portion of the inner breakwater. Wave energy reflecting off the vertical wall of the sealed section also contributed to this increase.

171. A comparison of Plans 46-57 indicated that Plan 50 appeared to be the optimum plan with respect to inner basin and entrance wave conditions and length of structure required. Tests for this plan using waves from the south, southwest, and west deepwater directions showed that wave heights in the entrance exceeded 4 ft three times and wave heights in the inner basin exceeded 1.5 ft three times. Plan 58 involved adding an inner jetty to Plan 50, and it was tested using those waves that exceeded the 1.5-ft criterion for Plan 50. Inner basin wave heights were reduced to within the 1.5-ft criterion except for the 17-sec, 6-ft wave from the south. Plan 59 was tested for this wave and the result was a substantial reduction of inner basin wave heights to a level well below the 1.5-ft criterion.

172. A comparison of Plans 59 and 60 showed that each plan reduced wave heights to within the 1.5-ft criterion. However, Plan 60 should be the more economical from a construction standpoint.

173. An examination of tracer test results for Plans 61 and 62 shows that the south trap of Plan 61 (500-ft radius from end of south jetty) captured more northerly moving material at mllw than did the south trap of Plan 62 (500-ft radius from end of stub groin). The north trap (same for both Plans 61 and 62) was effective in trapping southerly moving material for all wave conditions and water levels.

PART V: BEACH TESTS AND RESULTS

Description of Tests

Existing conditions

174. Prior to tests of various improvement plans, comprehensive tests were performed for the existing beach (Plate 53). Wave-height data were obtained for various stations along the initially proposed breakwater center line and groin heads for the conditions listed in paragraph 27. Wave-induced current patterns and magnitudes and tracer patterns also were secured for representative waves from the four selected test directions.

Beach protection plans

175. Current pattern and magnitude and/or tracer tests were conducted for 16 plan variations. These variations consisted in changes in the lengths, elevation, spacing, and location of the breakwater and groin structures. Photographs of tracer patterns were obtained for all major improvement plans. Brief descriptions of the beach protection plans are presented in the following subparagraphs;

- a. Plan 6 (Plate 54) consisted of five 800-ft-long groins spaced 1250 ft apart beginning at sta 105+00 and ending at sta 155+00.
- b. Plan 7 (Plate 55) involved the elements of Plan 6 with each groin extended 200 ft.
- c. Plan 8 (Plate 56) involved the elements of Plan 6 with the addition of 200-ft-long "T-heads" to each groin.
- d. Plan 9 (Plate 57) consisted of six 800-ft-long groins spaced 1000 ft apart beginning at sta 105+00 and ending at sta 155+00.
- e. Plan 9A (Plate 57) involved the elements of Plan 9 with the addition of a 400-ft-long groin and a 200-ft-long groin 800 ft (sta 163+00) and 1400 ft (sta 169+00), respectively, south of the southernmost groin (sta 155+00).
- f. Plan 9B (Plate 57) involved the elements of Plan 9A with the 200-ft-long groin relocated 200 ft to the north (sta 167+00).
- g. Plan 10 (Plate 58) entailed the elements of Plan 9 with the addition of 250-ft-long T-heads to each groin.

h. Plan 11 (Plate 59) consisted of ten 800-ft-long groins spaced to divide the beach as follows: three cells 1767 ft long from sta 47+00 to 100+00, two cells 1250 ft long from sta 100+00 to 125+00, and five cells 1000 ft long from sta 125+00 to 175+00.

i. Plan 12 (Plate 60) consisted of ten 800-ft-long groins spaced to divide the beach as follows: two cells 1767 ft long from sta 47+00 to 82+34, three cells 1422 ft long from sta 82+34 to 125+00, and five cells 1000 ft long from sta 125+00 to 175+00.

j. Plan 13 (Plate 61) consisted of a 4900-ft-long offshore breakwater made up of 700-ft-long sections alternating between crown elevations of 0.0 ft and -5.0 ft. The breakwater was situated 800 ft from and parallel to the baseline at sta 120+00 and extended from sta 105+50 to 154+50.

k. Plan 14 (Plate 62) entailed the elements of Plan 13 with the addition of a groin at each end of the breakwater. The crown elevation of these groins was +10 ft from the shoreward terminus to within 565 ft from the breakwater at which point it followed a constant slope to the breakwater elevation of 0.0 ft.

l. Plan 14A (Plate 62) involved the elements of Plan 14 with the +10 ft el of the groins lengthened 325 ft seaward.

m. Plan 15 (Plate 63) involved the elements of Plan 14 with the -5.0 ft el low-sill sections of the offshore breakwater removed.

n. Plan 16 (Plate 64) involved a proposed beach-fill plan between sta 105+00 and 155+00 with no protective structures.

o. Plan 17 (Plate 65) involved the elements of Plan 15 with the crown elevation of the offshore breakwater raised to +5 ft and the +10 crown elevation section of the groins lengthened an additional 325 ft seaward where it sloped to +5 ft at the breakwater.

p. Plan 18 (Plate 66) involved the elements of Plan 9 with each groin shortened 100 ft.

176. Typical sections of the various structures described above are shown in Appendix A.

Beach wave-height tests

177. Wave-height tests for the existing beach were conducted using test waves from the south and west deepwater directions. Wave-gage locations for existing conditions are shown in Plate 53. Following tests of

existing conditions, wave-height tests were discontinued as an expedient for the development of improvement plans. It became apparent that the most sensitive and critical tests performed for evaluation of plans designed to prevent beach erosion were the tracer tests. If no tracer material (or a relatively negligible amount) left the study area, the plan was considered functionally acceptable.

Beach current pattern
and magnitude tests

178. Wave-induced current patterns and magnitudes were determined at selected locations by timing the progress of a dye tracer relative to a known distance on the model surface. The tests were conducted for existing conditions using the same test waves and directions as for the wave-height tests.

Beach tracer tests

179. Tracer tests were conducted for existing conditions and the various improvement plans using test waves from one or more of the directions listed in paragraph 27. One of three different types of tracer tests were used;

- a. Fixed bed - tracer material was placed on the fixed-bed model surface at selected locations and/or fed into the longshore current to determine the mechanisms of littoral movement in the study area.
- b. Semimovable bed - tracer material was placed in a layer representing beach fill on the model surface to determine areas of accretion and erosion. The extent of erosion was limited by the fixed model surface.
- c. Movable-bed section - the fixed-bed contours between sta 105+00 and 155+00 were removed to a point well beyond the breaker zone and remolded entirely with crushed coal tracer. This type of test proved the most reliable in determining areas of accretion and erosion and was used for all the major beach protection plans.

Test Results

180. In evaluating test results, the relative merits of each plan were based primarily on an analysis of the movement of tracer material. From this evaluation, the best improvement plans were selected.

Existing conditions

181. Wave heights for existing conditions were measured at 10 gage locations along the center line of the initially proposed breakwater and at the ends of the initially proposed groins. These data are presented in Table 20. The maximum wave height at the groin heads was 20.0 ft at gage 1 for a 11-sec, 10-ft wave from the west deepwater direction at mhhw. This value is somewhat larger than the maximum breaking wave height to be expected at this depth contour from the generally accepted criterion of $H_B = 0.78 \times \text{depth}$. This increase is due to a rise in water level due to shoreward mass transport and the fact that the wave was peaking and breaking directly on the gage. Since none of the other test waves or gages showed heights this great, a more reasonable value (say 16 to 17 ft) could be used for the design wave in this area. The maximum wave height anywhere along the breakwater center line was 17.0 ft at gage 7 for a 9-sec, 12-ft wave from the west deepwater direction at mhhw.

182. Current patterns and magnitudes for waves from the west deepwater direction revealed, in general, a strong southerly longshore current as high as 6.7 fps. Current patterns and magnitudes for waves from the south deepwater direction showed strong northerly longshore currents, often as high as 10.0 fps.

183. Tracer tests showed an onshore movement of coal tracer for the small, low-steepness waves with longshore transport at the shoreline. For the high-steepness waves (Photos 59 and 60), coal tracer moved seaward into the initial breaker zone forming a bar which migrated north or south depending on wave direction. These waves then re-formed and broke the second time near the shoreline, resulting in a second zone of longshore transport.

Beach improvement plans

184. Results of semimovable-bed tracer tests for Plan 6 showed that for all waves from the south deepwater direction, a rip current formed along each groin. For small waves the rip currents were slow, causing little or no loss of tracer from the cells; for the large waves (Photo 61) rip currents were much stronger, causing substantial loss of

coal. Tracer material in the breaker zone tended to bypass the groins; and some tracer material moved out of the study area while some replenished adjacent cells. Tracer material shoreward of the breaker zone moved alongshore to form a fillet at each groin, extending the shoreline width by 100 to 150 ft in some cases. However, the beach at the center of the two southernmost cells eroded approximately 100 to 150 ft.

185. In an effort to reduce the amount of tracer leaving the study area and reduce erosion in the southern cells, the groins were lengthened to 1000 ft (Plan 7). Results of the semimovable tracer tests, for waves from the south (Photo 62), were similar to those of Plan 6. The lengthened groins tended to slow the transport of coal from the groin field; however, erosion of the shoreline in the two southern cells remained about the same.

186. In an effort to force the strong rip currents alongside each groin to eddy, 200-ft-long T-heads were installed at the seaward ends of the 800-ft-long groins (Plan 8). Results of semimovable-bed tracer tests for waves from the south (Photo 63) indicated that this plan did not protect the beach fill adequately since a substantial amount of tracer material still was lost from the study area and erosion of the shoreline remained about the same.

187. Results of semimovable-bed tracer tests for Plan 9 for waves from the south test direction (Photo 64) showed a significant reduction in the amount of shoreline erosion observed in the southernmost cells for previous plans. Due to the decreased longshore groin spacings (1000 ft instead of 1250 ft), the rip currents at each groin were reduced, carrying less coal tracer from the study area. Waves at mhhw broke closer to shore, causing the strongest rip currents and the most shoreline erosion. Waves at mllw often broke seaward of the groin field, causing the strongest longshore transport of coal outside the system. Results indicated that the spacings of the groins might possibly be varied (i.e., wider spacing to the north and closer spacing to the south).

188. In an effort to further reduce the loss of coal tracer from the groin field, 250-ft-long T-heads were installed at the end of the

Plan 9 groins (Plan 10). Results of the semimovable-bed tracer tests for waves from the south deepwater direction (Photo 65) showed little additional benefit.

189. While the primary area designated for beach protection structures was between sta 105+00 and 155+00, it was considered advisable to use the model (for a limited number of tests) to provide guidance on groin spacing required to the north and south of this area for possible future reference.

190. Semimovable-bed tracer tests for Plan 11 indicated that this plan provided adequate protection to the beach fill along the entire reach of beach, except for minor erosion between sta 82+34 and 100+00. Spacings in this area were changed so that the three cells between sta 82+34 and 125+00 were 1422 ft long (Plan 12). Results of semimovable-bed tracer tests using waves from the south deepwater direction (Photos 66 and 67) showed some longshore transport of tracer material outside the groin field. However, most of the coal tracer moving northward became trapped in the northernmost cells with some tracer material entering the harbor. Shoreline erosion was limited mainly to the southernmost cells where approximately 50 to 75 ft of shoreline at the middle of the cells was eroded. Some of this was brought back by small waves. The coal fill then was left in this configuration and tested using waves from the west deepwater direction. Test results (Photo 68) showed a southerly movement of tracer material outside of the groin field; however, the shoreline remained essentially unchanged. The coal then was reshaped to the specified initial beach slopes and tests again were conducted from the west deepwater direction. More longshore movement of coal tracer was observed outside the groin field (Photo 69), but shoreline erosion generally was less than for tests conducted using waves from the south. The semimovable-bed tracer tests indicated that after an initial loss of tracer material from the toe of the proposed beach fill, the Plan 12 groin spacing would provide adequate protection to the fill along the entire section of beach studied. It should be kept in mind, however, that vertical erosion and beach steepness are limited in this type of test.

191. In an effort to provide more reliable data on loss of beach fill, the fixed-bed contours between sta 105+00 and 155+00 were removed and remolded with crushed coal tracer. Test waves were allowed to run until the shoreline stabilized. Test wave durations for the following plans were as follows.

Test Wave Period sec	Test Wave Height ft	Duration Model hr	Duration Prototype hr
West Deepwater Direction, mhhw			
7	4	0.25	2.5
7	10	.50	5.0
11	4	.25	2.5
11	10	.50	5.0
17	6	.50	5.0
West Deepwater Direction, mllw			
11	4	0.25	2.5
11	10	.50	5.0
South Deepwater Direction, mhhw			
7	4	0.25	2.5
7	10	.50	5.0
9	4	.25	2.5
9	10	.50	5.0
17	6	.50	5.0
South Deepwater Direction, mllw			
9	4	0.25	2.5
9	10	.50	5.0

To prevent model circulation effects, each continuous test run was limited to 15 min. Measurements were taken of the shoreline configuration after each test and plotted on drawings of each plan.

192. Results of movable-bed tracer tests for Plan 9 using waves from the west deepwater direction are presented in Photos 70 and 71 and Plate 67. The +5.4 line on the shoreline plots represents the initial waterline of mhhw. In general, small waves accreted the shoreline with more accretion near the groins. The larger waves moved

material from the middle of each cell to the downcoast groin, building a fillet. Waves diffracting around the end of each groin caused some buildup of tracer material in the lee of the groins. The amount of tracer material moving downcoast out of the groin field was small. For this condition, little or no erosion of the shoreline was observed.

193. Plan 9 then was tested using waves from the south deepwater direction. Test results (Photo 72, Plate 68) indicated that waves from this direction caused considerably more movement of tracer material than did waves from the west. Again, the small waves accreted the shoreline in each cell while the large waves moved coal to each downcoast groin, building a sizable fillet. Also, waves diffracting around the ends of the groins built a small fillet in the lee of the structures. Waves overtopping the groin at sta 155+00 caused some erosion of the coal adjacent to the groin. The building of a fillet on the south side of the groin (as will probably be the case in the prototype) should eliminate this problem. Moderate shoreline erosion was observed in the middle of the cells for the larger waves, but most of this material simply migrated to the downcoast groins and was retained in the cells. Results indicated the formation of an offshore bar inside each cell for the large waves. Subsequently, when a small wave was run, this bar migrated shoreward, eventually merging with the shoreline. There was little movement of tracer material from one cell to another, and loss of coal from the groin field remained small. In viewing the tracer photographs for the movable-bed series of tests, it should be noted that some of the contours reproduced in movable-bed (coal) material were seaward of the ends of the groins (Photo 70). The seaward or longshore movement of tracer material seen on the after-testing photographs (which may appear misleading) is usually this seaward material and does not necessarily represent erosion of beach fill placed between the groins.

194. The coal fill then was reshaped and Plan 9 tested with coal gradually introduced into the breaker zone south of sta 155+00 to determine the effect of the groin field on material moving in the littoral zone. Test results (Photo 73, Plate 69) showed that the small waves had insufficient energy to carry the tracer material past the

southernmost groin and a fillet was formed. The large waves, however, continually moved coal past the end of the southernmost groin where it accumulated in the cells. Some of the coal was carried over the southernmost groin, decreasing the amount of erosion adjacent to the groin experienced in the preceding test. The southernmost cell accreted coal until full, and then material began moving into the next cell. It was apparent that if the test was run long enough with a large wave and material being fed in at the south end, the entire groin field could have been completely filled and material would move along the shore north of the groin field.

195. The coal fill again was reshaped and a 9-sec, 10-ft storm wave was tested first to determine its effect on the initial beach fill. Results (Plate 70) showed moderate erosion in the middle of the cells. Again offshore bars were formed inside the cells at the initial breakpoint of the incoming waves. Coal tracer then was fed into the system at the south end, drastically increasing the size of the fillets. A small wave then was run which moved tracer material shoreward and effectively filled in the areas of erosion.

196. Since the 1000-ft spacings of the Plan 9 groins appeared to offer adequate protection to the proposed beach fill and trapped significant amounts of littoral material, it was decided to test a wider spacing in an effort to reduce costs. Plan 6 (1250-ft groin spacings) was reinstalled in the model, and movable-bed tracer tests were conducted using waves from the south deepwater direction. Results (Photo 74, Plate 71) showed increased longshore and rip current magnitudes in each cell, causing increased coal movement. The shoreline in the middle of the cells eroded more rapidly (more than 100 ft for the 9-sec, 10-ft wave at mhhw). Movement of coal from one cell to another and eventually out of the groin field was significantly increased when compared with Plan 9. Plan 6 was considered marginally effective in protecting the placed beach fill; but when material was fed from the updrift beaches, enough material was trapped to be a satisfactory alternative. The availability of sufficient quantities of updrift material in the prototype may be questionable however.

197. Results of movable-bed tests for Plan 13 under a "no-feed" condition (i.e., no tracer material being fed into the littoral currents on the updrift side of the study area) are presented in Photo 75 and Plate 72. In general, small waves accreted the shoreline with the largest accretions occurring in the lee of the higher breakwater (0.0 crown elevation) sections. The larger waves moved material to the north. Due to the steep angle at which the waves approached the shoreline at the southern end of the study area, a portion of the fill material was unprotected; and severe erosion in this area was observed. The movement of material from the unprotected area rapidly built a partial tombolo in the lee of the southernmost high breakwater section. Longshore transport of material past the tombolo was reduced due to deflected longshore currents and reduced incident wave angle. Tracer material continued to move northward at a decreasing rate which caused some shoreline accretion at the northern end of the beach fill. Substantial amounts of tracer material moved north out of the protected area.

198. Test results for Plan 13 for a "feed" condition (i.e., material being fed into the littoral currents in the updrift side of the study area) are presented in Photo 76 and Plate 73. Introducing coal into the breaker zone did not prevent the erosion of the shoreline at the southern end of the study area. For the large waves, most of the coal tracer introduced into the system on the updrift side was caught in a rip current formed at the southern end of the study area and was carried outside and north along the breakwaters. Subsequently, some of this material was pushed across the low breakwater (-5.0 crown elevation) sections and into the protected area, which smoothed the shoreline somewhat.

199. In an effort to reduce the amount of erosion experienced in the southern end of the study area and the amount of coal lost to the north, 800-ft-long groins (+10 ft crown elevation for 235 ft, sloping to 0.0-ft crown elevation at the groin head) were installed at each end of the breakwater (Plan 14). Test results for a no-feed condition (Photo 77, Plate 74) showed greatly reduced erosion of the shoreline adjacent to the southern groin and reduced size of the resulting tombolo

behind the first breakwater section. Movement of tracer material over and through the north groin remained significant. Moderate erosion of the southern shoreline and accretion of the northern shoreline was observed, illustrating a general shift of the beach fill to the north.

200. Test results for Plan 14 for a feed condition (Photo 78, Plate 75) were generally the same as for a no-feed condition. Most of the fed material moved north along the outside of the breakwaters and out of the study area. The installation of the two groins forced water, pumped into the system by the breaking waves, to exit over the low breakwater sections. This prevented tracer material from entering the protected area.

201. In an effort to reduce the volume of rock required for construction, the low breakwater sections were removed (Plan 15). Test results for a no-feed condition (Photo 79, Plate 76) showed marked accretions and erosions in the shadows of the breakwater sections and gaps, respectively. Substantial shoreline accretion was observed at the north end of the study area; however, significant quantities of tracer material passed over and through the north groin.

202. Test results for Plan 15 for a feed condition (Photo 80, Plate 77 were generally the same as for the no-feed condition. The fed material moved north along the outside of the breakwaters as in the previous plan. However, some tracer material did enter the protected area for the 17-sec, 6-ft wave (Photo 81).

203. Movable-bed tests for a beach-fill plan (Plan 16) with no protective structures were conducted using waves from the west and south deepwater directions. Observations from model tests indicated that an extended test wave duration (relative to that for previous plans) was necessary since the coal fill did not stabilize. Test wave durations for the following plans were as follows:

Test Wave		Duration	
Period sec	Height ft	Model hr	Prototype hr
West Deepwater Direction, mhhw			
7	4	0.25	2.5
7	10	1.00	10.0
11	4	.25	2.5
11	10	2.00	20.0
17	6	.50	5.0
West Deepwater Direction, mllw			
11	4	0.25	2.5
11	10	1.00	10.0
South Deepwater Direction, mhhw			
7	4	0.25	2.5
7	10	1.00	10.0
9	4	.25	2.5
9	10	2.00	20.0
17	6	.50	5.0
South Deepwater Direction, mllw			
9	4	0.25	2.5
9	10	1.00	10.0

204. Results of movable-bed tests for Plan 16 from the south deepwater direction (tested in the order shown in the above tabulation) showed extreme erosion of the updrift shoreline for the large waves (Photo 82, Plate 78). Longshore currents moving northward were deflected seaward by the beach fill to form a spit. This rapidly eroding area became a source of material for the downcoast beach which did not suffer significant erosion until the upcoast supply had been exhausted. As attack by the large waves continued, the coal spit migrated northward leaving the southern section of beach completely devoid of coal fill. It was apparent that if these waves were allowed to continue, eventually all fill material would be lost from the study area. The coal then was reshaped and tested with a 9-sec, 10-ft wave first to determine the effect a large storm wave would have on the initial beach fill. The result (Photo 83, Plate 78) was an increased erosion rate (when compared

with the original test wave sequence).

205. Without reshaping the coal, the wave generator was moved to the west deepwater direction to determine whether or not the displaced coal would return to the eroded beach. Results (Photo 84, Plate 79) showed that some material did reenter the study area and began filling the "hole" created by waves from the south. However, most of the material remained on the shoreline north of the study area. Even as the eroded beach between sta 135+00 and 155+00 began accreting material, the north end of the study area began eroding. The coal then was reshaped and a storm wave (11-sec, 10-ft) from the west was tested to determine its effect on the initial beach fill (Photo 85, Plate 79). Again, this resulted in extreme erosion of the updrift shoreline. The test was stopped after 6 hr, but again it was apparent that all fill would be eroded if the test continued.

206. Plan 14 then was reinstalled in the model (Photo 86) and tested using waves from the west deepwater direction for the extended durations listed in paragraph 203. Results (Photo 87, Plate 80) showed significant shoreline erosion adjacent to the north groin due to waves overtopping the structure. The shoreline adjacent to the south groin continued to accrete until reaching a point where the groin became submerged, after which the material moved over the groin and out of the study area.

207. In an effort to reduce shoreline erosion adjacent to the north groin and loss of material past the south groin, the +10 ft elevation of both groins was extended 325 ft, after which it sloped down to an elevation of 0.0 ft at the breakwater (Plan 14A, Photo 88). Test results (Photo 89, Plate 81) showed an improvement in these two areas.

208. Test results for Plan 17 are presented in Photos 90-96 and Plate 82 and show a relatively stable and accreted shoreline, except for moderate erosion of the shoreline adjacent to the north groin due to waves overtopping this structure.

209. In an effort to reduce the volume of rock required for the six-groin plan (Plan 9), a length of 100 ft was removed from the end of

each groin (Plan 18, Photo 97). Movable-bed test results using waves from the west deepwater direction (Photos 98-104, Plate 83) showed, in general, accretion for the small waves and erosion for the large waves. The most erosion was observed between sta 135+00 and 155+00. A significant amount of material was lost from the study area.

210. The coal then was reshaped and Plan 18 was tested using waves from the south deepwater direction. Test results (Photos 105-111, Plate 84) showed shoreline erosion in most cells with the 9-sec, 10-ft wave proving to be the worst condition. A substantial quantity of tracer material was lost from the study area.

211. Plan 9 was retested for the extended durations listed in paragraph 203 and results (Photos 112-119, Plate 85) when compared with those of Plan 18 showed substantial reductions in shoreline erosion for all wave conditions. Tracer material lost from the study area also was greatly reduced. Test results indicated that the shoreline for the shorter groins receded approximately the length of groin reduction (i.e. 100 ft).

212. In an effort to reduce the quantity of sand required for the beach fill, revised beach slopes of 1:10 above mllw and 1:15 below mllw were tested with the Plan 9 groins installed in the model for the worst wave condition (9-sec, 10-ft wave from south at mhhw). Because of a camera malfunction, a photograph of the result is not available; however, a plot of the resultant shoreline is shown in Plate 86. When this plot is compared with the shoreline plot of the same plan with the initial beach slope, it can be seen that the shape of the shoreline changed very little. It also was observed that less tracer material left the study area since the toe of the new slope was closer to shore and more protected by the groins.

213. The wave generator was moved to a position (representing waves from S55°W) that produced an incident wave front parallel to the beach. The coal was reshaped (Photo 120) to represent the proposed beach fill (1:20 slope) and tested to determine the degree of onshore/offshore tracer movement (including movement due to rip currents). Test results (Photo 121, Plate 86) showed weak rip currents that formed along

each side of each groin. These currents were generally too weak, however, to carry much tracer past the groin heads. The shoreline accreted in each cell, and there was virtually no loss of coal from the study area.

214. Movable-bed tracer tests were conducted at the southern end of the Plan 9 groin field in an attempt to determine the impact of the groins on the adjacent south beach. In order to conduct movable-bed tests of this area, within the limits of the existing movable-bed model area and wave generator coverage, the groin field was effectively moved 3000 ft to the north. To expedite testing, a rather severe wave (9 sec, 10 ft) was tested from the northwest direction. Test results (Photo 122, Plate 87 showed considerable shoreline erosion south of the last groin (receding to the concrete behind the coal fill).

215. In an effort to reduce the severity of erosion experienced in this area, two groins, 400 ft long and 200 ft long, were installed 800 ft (sta 163+00) and 1400 ft (sta 169+00), respectively, south of the southernmost groin (sta 155+00) of Plan 9 (Plan 9A). Results showed decreased erosion between sta 155+00 and 163+00; however, the shoreline between the two transitional groins continued to erode excessively (Photo 123, Plate 87).

216. In an attempt to reduce the erosion between the two transitional groins, the 200-ft-long groin was repositioned 200 ft to the north (sta 167+00, Plan 9B). Results (Photo 124, Plate 87) showed decreased shoreline erosion in this area.

Discussion of test results

217. A comparison of Plans 6-8 indicated that the groin spacings (1250 ft) were too great. For the large storm waves, the amount of energy entering each cell was enough to cause substantial longshore and subsequent rip currents at each groin. These currents carried coal tracer past the ends of the groins where it moved alongshore and eventually out of the study area. Lengthening the groins (Plan 7) or installing T-heads on the groins (Plan 8) did not adequately reduce or deflect these currents. Because of the amount of tracer material that left the groin field and the amount of erosion observed in the southernmost cells, these plans were not considered as adequate beach protection.

218. Test results for Plans 9 and 10 showed that a groin field with 1000-ft spacing effectively prevented erosion of the shoreline and loss of beach fill from the study area. Plan 10, however, provided little benefit over Plan 9 with a significant increase in structure required. Plan 9 was considered the most viable beach protection scheme.

219. Tests of Plans 11 and 12 were run to provide information on the groin spacings required for adequate protection of the beach between sta 47+00 and 175+00. Test results showed that the 1767-ft spacings between sta 82+34 and 100+00 probably were too wide, allowing excessive wave energy to enter the cells. When the spacings between sta 82+34 and 125+00 were changed to 1422 ft (Plan 12), results showed reduced erosion in this area.

220. A comparison between Plan 12 and existing conditions showed that the groins had little effect on the movement of the offshore bar as illustrated in Photos 60 and 67. The groins effectively prevented tracer material along the shoreline from migrating along the coast. Conditions at the harbor remained the same.

221. Movable-bed tests of Plan 13 indicated that due to erosion of the shoreline on the updrift side of the protected area and loss of coal from the downdrift side, this offshore breakwater plan was not effective in providing adequate protection to the beach fill.

222. A comparison of Plans 13-15 (offshore breakwater schemes) showed that Plans 14 and 14A seem to be the most viable alternatives. Observations showed that coal moved along the shoreline replacing the coal which had left immediately before, resulting in a fairly stable shoreline (as long as the updrift supply is constant). As the supply ceases, slight erosion of the upcoast shoreline begins, supplying coal to the downcoast shoreline. Raising the crown elevations of the groins to prevent overtopping (Plan 14A) was beneficial in reducing the amount of coal leaving the study area. Some shifting of beach fill from one end of the system to the other (depending on wave direction) was observed, but this should not present a serious problem.

223. Movable-bed tests of Plan 16 (unprotected beach-fill plan) showed that any placed beach fill (without protective structures or

periodic replacement) in this area would suffer serious erosion.

224. Plan 17 (+5.0 ft crown elevation of breakwaters and groin heads) prevented more wave energy from entering the system than did Plan 15. This resulted in decreased longshore and rip currents and decreased shoreline erosion and coal loss. This plan appears to be a viable alternative.

225. Plan 18 showed that shortening the groins of Plan 9 decreased the beach width by approximately the amount that the groins were shortened (i.e. 100 ft). The groin test results indicate that for the same wave conditions, the width of the resultant beach is related to groin length and spacing. Shortening the groins decreases the beach width; increasing the groin spacings also decreases the beach width.

226. While the results of the tests described in paragraphs 214-216 were very interesting, caution must be exercised in the interpretation of the results. As mentioned in paragraph 214, the entire groin field was moved 3000 ft to the north which may affect the mechanics of littoral transport in the area; the 9-sec, 10-ft wave from the northwest, used to increase erosion rates and decrease model testing time, is somewhat more severe than waves normally occurring from this direction; and finally, the extent of shoreline erosion was limited by the width of the movable bed. As can be seen in Plate 87, the eroded areas that appear as a straight line are those areas in which the shoreward limit of the movable bed has been reached. However, a qualitative comparison of the results of Plans 9, 9A, and 9B reveals that Plan 9B offers the smoothest transition from groin field to existing beach. While some shoreline erosion probably will occur south of the last groin regardless of groin location (requiring periodic nourishment using material bypassed or dredged from the harbor area), Plan 9B should help minimize this erosion.

227. For all the plans tested, erosion occurred primarily for large or storm-wave conditions. Under more normal wave conditions, the shoreline accreted.

228. After completion of model testing, it was decided by SPL to extend the groin field to sta 218+00 on the south (six more groins).

The length and spacing of these new groins could not be tested in the model due to the limited extent of model contour and wave generator coverage in this area. However, considering that underwater contours and incident waves in the extended area are similar to those in the reach between about sta 140+00 and 175+00, it may be inferred that the same groin spacing and length developed in the model for sta 140+00 to 175+00 could be used in the extended area. This same reasoning also applies for the transitional groins.

PART VI: CONCLUSIONS

The Harbor

229. Based on results from the three-dimensional model investigation reported herein, it is concluded that:

- a. Existing conditions are characterized by strong longshore currents for moderate to large wave conditions with considerable longshore movement of material into and past the harbor entrance. The resultant shoal at the harbor entrance created hazardous entrance wave conditions due to breaking waves. Also, wave heights in certain areas of the harbor are excessive. Significant quantities of material pass into the harbor through the voids of the north jetty.
- b. The original improvement plan for prevention of harbor shoaling for Oceanside (i.e., the offshore breakwater plan of Plan 1) was ineffective in trapping material outside the harbor entrance and, in fact, contributed to the shoaling problem by trapping tracer material in the entrance channel.
- c. The original improvement plan for wave protection to the harbor expansion (i.e., the inner breakwater of Plan 25) was ineffective in reducing wave heights to the level desired.
- d. Of the plans tested, Plans 4, 23, 24, 61, and 62 provided adequate prevention of harbor shoaling for all wave conditions tested. Plans 38C, 44, and 45 provided adequate prevention of harbor shoaling for waves from northerly directions and, if combined with the south jetty extension of Plan 21, would prevent shoaling for all wave conditions tested. Plan 4 required the greatest volume of rock. Plans 23 and 24 required less rock but would require bypassing large volumes of sand. Plans 38C, 44, and 45 (in conjunction with the south jetty extension of Plan 21) would require bypassing smaller volumes of sand due to the natural containment of sand by the groin. Plans 61 and 62 required the least volume of rock (no south jetty extension required) and would utilize the natural containment of sand by the groin.
- e. Many of the plans tested provided marginally adequate wave protection for the harbor expansion. However, considering all aspects of this expansion (entrance conditions, wave heights in berthing areas, harbor usage, cost and methods of construction, etc.), Plan 60 is

considered the optimum based on results of the model tests (i.e., without regard to other factors).

The Beach

230. Based on results from the three-dimensional model investigation reported herein, it is concluded that:

a. Existing conditions are characterized by strong longshore currents for moderate to large wave conditions with considerable longshore transport.

b. The original improvement plan for Oceanside Beach (i.e. Plan 6, the 5-groin plan) was not effective in retaining the beach-fill material and preventing erosion of the shoreline because of the spacing.

c. Plans 9, 9B, 12, 14A, and 17 are all viable alternatives for either groin or offshore breakwater configurations at Oceanside. Plan 12 protected the longest section of beach (from sta 47+00 to 175+00) and, due to the number of groins, used the largest volume of rock. Plans 9, 14A, and 17 all were effective in protecting the beach fill between sta 105+00 and 155+00. These plans were effective in containing the beach fill and minimizing movement of tracer within the study area. The transitional groins of Plan 9B effectively minimized effects of the groin field on adjacent beaches. Such transitional groins could also be used effectively with offshore breakwater Plans 14A and 17.

PART VII: SUMMARY

231. Based on the assumption that construction of the harbor at Oceanside has contributed in some degree to beach erosion to the south of the harbor, the Federal Government has placed all material dredged from the harbor on these southern beaches. In the past, however, placement of beach fill has been somewhat less than ideal (i.e., harbor dredging has normally been accomplished every 18 to 24 months and considerable erosion has occurred in the interim; placement of dredged material has sometimes occurred at inopportune times such as during storms, during the least optimal season of the year, when predominant littoral drift was not in the desired direction, etc.; and sometimes the locations where material has been placed were not selected on the basis of maximizing the nourishment potential of dredged material discharged). The most satisfactory means of absolving the Federal Government of any responsibility for beach erosion at Oceanside would be to restore the conditions that occurred prior to construction of the harbor. The most obvious way to do this would be to remove the harbor; however, such an alternative appears at present to be infeasible. Consequently, in seeking the optimum improvement plan for Oceanside, SPL has tried to develop one that provides for continuous bypassing and/or backpassing of the harbor, allowing placement of material when and where needed. The necessary elements of such a plan would include (a) trap areas to the north and south of the harbor (may include structural modifications and/or dredged deposition basins) where material could be picked up and bypassed and/or backpassed (these traps should also aid in minimizing harbor shoaling); (b) a flexible distribution system from the traps to required discharge locations; (c) a dredge (either conventional or jet pump type system) for moving material; and (d) a schedule or plan of dredging and discharging which is flexible enough to accommodate widely varying conditions.

232. If a satisfactory bypassing/backpassing system can be implemented at Oceanside, then the Federal Government would come as close as possible to returning the beaches at Oceanside to their natural state.

This would not guarantee, however, that beach erosion would not continue to occur at Oceanside. It would only eliminate that erosion caused by the harbor. It is suspected (but not within the purview of this investigation) that if the harbor had never been constructed, some erosion would be occurring at Oceanside due to a decreased supply of sediment to the coast. If this is the case, then auxiliary measures (structures, beach fill from external sources, etc.) may be required to supplement the bypassing/backpassing system.

233. All improvement plans tested in the hydraulic model were aimed at meeting the requirements discussed above, along with the additional requirement of harbor expansion and improvement of wave conditions in the existing harbor areas. Both structural (breakwaters, jetty extensions, and/or groins) and nonstructural (dredged deposition basins) trap areas were developed to the north of the harbor where material moving from north to south could be picked up and bypassed to the southern beaches or backpassed to the northern beaches if necessary. Based on the assumption that the net littoral drift is from north to south, this would be the primary trap area for nourishing the eroding southern beaches. Both structural and nonstructural trap areas also were developed to the south of the harbor. It is anticipated that this would be an auxiliary trap area primarily for backpassing material to the southern beaches. Structural modifications for harbor expansion and improvement (inner breakwaters, offshore breakwaters, north and south jetty extensions, and stub groins) also were tested to determine their effects on harbor shoaling and beach erosion. The plans considered as viable alternatives for harbor development, when installed in conjunction with an acceptable bypassing/backpassing system, should not create additional beach erosion problems at Oceanside.

234. Assuming that beach erosion would continue to occur at Oceanside following implementation of a bypassing/backpassing scheme which would eliminate erosion caused by the harbor, then installation of one of the viable offshore breakwater or groin plans developed in the model should provide protection to that section of beach protected by the breakwaters or groins. Progressively shorter groins at each end of

a breakwater or groin system should help to smooth the transition from the breakwater or groin protection section of beach to the existing beach and tend to minimize the impact of erosion at the ends. Since the breakwater or groin plans would be filled from external sources, there should be no significant entrapment of material currently in the littoral system by the individual breakwater or groin cells. The entire breakwater or groin system will act as a small headland, however; and material can be trapped between the harbor and breakwater or groin system. For this reason, any bypassing/backpassing system for the harbor must have the capability to discharge material south of the breakwater or groin system so that the downdrift (south) beaches will not be adversely impacted. This would dictate that the last structure in the selected system be located somewhat northerly of the southerly limit of the authorized project.

235. While it is not within the purview of this investigation to make definite recommendations regarding the type of dredging system to be used for Oceanside, it is felt that the jet pump system offers many advantages (particularly those of significantly less structural requirements and almost continuous operation as opposed to periodic placement of material at yearly or longer intervals), and this system should be given strong consideration. Subsequent to the present investigation, SPL has initiated detailed investigations of this concept.

236. In summary, it is felt that sufficient alternative plans have been developed, as a result of the design efforts of SPL and the model tests reported herein, to enable the selection of a satisfactory solution to the problems of harbor shoaling, harbor expansion and improvement, and beach erosion. It is essential to use a flexible quasi-continuous bypassing/backpassing system in conjunction with any structural modifications intended to mitigate the beach erosion problem and/or in conjunction with the harbor improvement plan.

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TABLE 1

ESTIMATED DURATION OF DEEPWATER WAVES APPROACHING OCEANSIDE, CALIFORNIA, FROM VARIOUS DIRECTIONS

WAVE HEIGHT* FT	DURATION (HR/YR) PER WAVE PERIOD,* SEC									
	2-4	4-6	6-8	8-10	10-12	12-14	14-16	16-18	≥18	TOTAL
					NORTHWEST (300°-330°)					
0-2	780									780
2-4		261								261
4-6		89								89
6-8			33							33
8-10			5							5
TOTAL	780	350	38							1168
					WEST (245°-285°)					
0-2		265	649	281	166	1111	597	148		3217
2-4		550	290	18	25	62	54	20		1019
4-6		34	272	18		T**				324
6-8			41	32	1					74
8-10			1	5	1					7
10-12				T						
TOTAL		849	1253	354	193	1173	651	168		4641
					SOUTHWEST (205°-245°)					
0-2		17	17	4	2	1202	798	611	331	2982
2-4		65	39	6	3	511	518	197	265	1604
4-6		14	28	2	T			13		57
6-8		1	22	T	T			1		24
8-10			4	6						10
10-12				T						
12-14				T						
14-16				T						
TOTAL		97	110	18	5	1713	1316	822	596	4677
					SOUTH (165°-205°)					
0-2		6	3	T	T	1420	1352	519	101	3401
2-4		22	6	1	T	11	9	2		51
4-6		4	7	T						11
6-8		T	6	T						6
8-10			T	2						2
TOTAL		32	22	3		1431	1361	521	101	3471

* WAVE-HEIGHT AND WAVE-PERIOD GROUPINGS INCLUDE THE LOWER BUT NOT THE UPPER VALUES.

** T - TRACE DURATIONS LESS THAN 1 HR/YR.

TABLE 2

ESTIMATED DURATION OF SHALLOW-WATER WAVES APPROACHING OCEANSIDE, CALIFORNIA, FROM VARIOUS DIRECTIONS

WAVE HEIGHT* FT	DURATION (HR/YR) PER WAVE PERIOD,* SEC 2-4	4-6	6-8	8-10	10-12	12-14	14-16	16-18	≥18	TOTAL
					NORTHWEST (300°-330°)					
0-2	693	29								722
2-4	87	252								339
4-6		69	7							76
6-8			27							27
8-10			3							3
TOTAL	780	350	37							1167
					WEST (245°-285°)					
0-2		295	665	282	166	1111	568	128		3215
2-4		525	317	19	25	59	75	35		1055
4-6		28	238	22	T**	3	7	5		303
6-8			32	28	1					61
8-10			T	4	1					5
10-12				T						
TOTAL		848	1252	355	193	1173	650	168		4639
					SOUTHWEST (205°-245°)					
0-2		20	19	4	2	1202	760	555	288	2850
2-4		63	40	6	2	511	530	217	248	1617
4-6		12	30	2	T		26	46	60	176
6-8		T	18	1	T			4		23
8-10			3	4				T		7
10-12				T						
12-14				T						
14-16				T						
TOTAL		95	110	17	4	1713	1316	822	596	4673
					SOUTH (165°-205°)					
0-2		8	4	1	T	1353	1229	452	84	3131
2-4		20	6	1	T	78	130	69	17	321
4-6		3	7	T		T	T	1		11
6-8		T	5	T						5
8-10			T	2						2
TOTAL		31	22	4		1431	1359	522	101	3470

* WAVE-HEIGHT AND WAVE-PERIOD GROUPINGS INCLUDE THE LOWER BUT NOT THE UPPER VALUES.
** T - TRACE DURATIONS LESS THAN 1 HR/YR.

TABLE 3

WAVE HEIGHTS FOR EXISTING CONDITIONS

(TEST WAVES FROM NW)

Test Wave Direction	Test Wave Period, sec	Test Wave Height, ft	Wave Height, ft: Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8
			SWL = +5.4 FT							
NW	7.0	5.0	4.7	6.2	4.5	5.4	6.1	6.0	0.2	0.8
		10.0	13.5	11.6	10.5	11.0	11.0	12.0	0.3	1.1
			SWL = 0.0 FT							
		5.0	4.4	5.4	4.3	4.9	5.4	4.7	<0.1	0.6
		10.0	11.6	11.9	10.7	14.5	6.3	6.8	0.2	0.4

Test Wave Direction	Test Wave Period, sec	Test Wave Height, ft	Wave Height, ft: Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 14	Gage 15
			SWL = +5.4 FT						
NW	7.0	5.0	1.0	1.4	0.9	0.2	0.9	0.5	0.3
		10.0	1.5	2.8	1.3	0.3	1.1	0.7	0.4
			SWL = 0.0 FT						
		5.0	0.4	1.1	0.9	<0.1	0.6	0.1	0.2
		10.0	1.1	2.4	1.5	0.1	0.9	0.3	0.2

TABLE 4

WAVE HEIGHTS FOR EXISTING CONDITIONS

(TEST WAVES FROM WEST)

TEST WAVE			WAVE HEIGHT, FT							
DIRECTION	PERIOD SEC	HEIGHT FT	GAGE 1	GAGE 2	GAGE 3	GAGE 4	GAGE 5	GAGE 6	GAGE 7	GAGE 8
					SWL = +5.4 FT					
W	7.0	4.0	3.3	4.5	3.6	4.0	3.9	4.7	0.2	0.9
		10.0	13.9	10.9	11.4	12.7	13.3	14.3	0.2	1.8
	9.0	4.0	6.8	4.7	4.4	4.8	5.8	5.1	0.4	2.3
		12.0	15.1	14.0	16.7	16.8	10.5	11.0	0.3	2.4
	11.0	4.0	4.4	5.1	5.4	6.0	8.5	7.2	0.4	2.7
		10.0	19.3	15.4	15.2	16.8	13.1	14.0	0.5	2.7
	14.0	6.0	14.1	9.3	8.0	8.6	13.2	11.9	0.5	3.1
	17.0	6.0	10.9	9.6	9.6	8.3	12.6	13.4	0.6	2.2
					SWL = 0.0 FT					
	7.0	4.0	6.1	5.0	4.3	4.9	4.3	6.0	<0.1	0.5
		10.0	13.0	10.2	10.7	11.9	6.8	8.2	0.1	0.4
	9.0	4.0	4.5	5.2	4.5	5.7	6.6	5.2	<0.1	0.5
		12.0	19.2	13.6	16.3	13.0	7.8	9.8	0.1	0.8
	11.0	4.0	8.8	5.5	5.3	5.1	8.1	7.9	<0.1	0.5
		10.0	19.1	14.0	16.0	15.4	8.2	9.3	0.2	0.7
	14.0	6.0	10.2	8.8	8.8	10.7	9.7	8.9	0.2	1.0
	17.0	6.0	11.0	10.4	10.8	10.9	8.5	9.9	0.2	0.8

(CONTINUED)

TABLE 4 (CONCLUDED)

Test Wave Direction	Test Wave Period, Sec	Test Wave Height, Ft	Wave Height, Ft: Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 14	Gage 15
				SWL = +5.4 FT					
W	7.0	4.0	0.7	2.6	0.9	0.1	1.1	0.5	0.3
		10.0	0.8	3.6	1.8	0.3	1.8	0.6	0.6
	9.0	4.0	0.6	5.5	1.1	0.1	0.2	0.1	0.1
		12.0	1.5	6.8	2.1	0.2	0.4	0.3	0.3
	11.0	4.0	0.8	4.5	0.9	0.2	0.3	0.2	0.1
		10.0	2.8	5.2	2.6	0.2	0.7	0.6	0.4
	14.0	6.0	2.3	9.0	1.4	0.2	1.3	0.8	0.6
	17.0	6.0	3.1	5.7	1.5	0.3	1.2	1.8	0.4
				SWL = 0.0 FT					
	7.0	4.0	1.0	1.9	1.5	0.2	1.3	0.3	0.6
		10.0	0.8	3.1	1.6	0.1	1.5	0.4	0.5
	9.0	4.0	1.1	3.3	2.0	0.2	0.5	0.6	0.4
		12.0	0.8	4.0	1.8	0.1	0.5	0.6	0.3
	11.0	4.0	1.7	3.1	1.2	<0.1	0.7	0.3	0.1
		10.0	1.3	3.7	1.3	0.1	0.7	0.4	0.2
	14.0	6.0	1.7	4.3	1.6	0.2	0.8	0.3	0.3
	17.0	6.0	2.7	3.8	1.6	0.2	0.9	0.5	0.3

TABLE 5

WAVE HEIGHTS FOR EXISTING CONDITIONS

(TEST WAVES FROM SW)

TEST WAVE			WAVE HEIGHT, FT							
DIRECTION	PERIOD SEC	HEIGHT FT	GAGE 1	GAGE 2	GAGE 3	GAGE 4	GAGE 5	GAGE 6	GAGE 7	GAGE 8
			SWL = +5.4 FT							
SW	7.0	4.0	4.2	3.4	4.2	3.3	4.6	3.7	0.4	0.7
		10.0	12.2	10.6	10.7	12.7	11.4	14.6	0.5	1.8
	9.0	4.0	5.4	4.2	5.8	4.5	6.3	3.8	0.6	1.2
		16.0	19.9	22.6	23.0	14.8	11.3	13.8	0.6	3.3
	11.0	8.0	11.2	9.3	8.4	10.4	12.0	13.8	0.5	4.2
	14.0	6.0	9.5	7.9	8.5	10.0	11.8	9.8	0.5	3.1
	17.0	10.0	16.7	12.6	14.9	20.2	12.4	13.2	0.6	4.0
	19.0	6.0	8.9	8.0	10.0	9.5	13.5	12.8	0.9	4.3
			SWL = 0.0 FT							
	7.0	4.0	5.2	4.7	4.6	4.9	6.0	4.7	0.2	0.4
		10.0	11.4	10.1	10.4	14.3	11.0	11.3	0.2	0.9
	9.0	4.0	5.8	4.6	5.2	4.8	8.0	4.4	0.1	0.6
		16.0	17.5	22.9	12.8	16.0	11.8	8.4	0.2	1.6
	11.0	8.0	14.6	11.5	13.8	13.3	9.3	10.9	0.3	1.9
	14.0	6.0	10.3	8.1	8.4	9.7	13.5	15.6	0.3	1.3
	17.0	10.0	20.7	16.3	18.1	15.4	9.4	10.5	0.3	1.5
	19.0	6.0	11.4	9.0	7.2	14.0	10.8	12.0	0.2	1.3

(CONTINUED)

TABLE 5 (CONCLUDED)

TEST WAVE			WAVE HEIGHT, FT						
DIRECTION	PERIOD SEC	HEIGHT FT	GAGE 9	GAGE 10	GAGE 11	GAGE 12	GAGE 13	GAGE 14	GAGE 15
					SWL = +5.4 FT				
SW	7.0	4.0	0.4	2.8	0.4	<0.1	0.4	0.2	0.1
		10.0	2.5	4.9	3.6	0.8	3.8	1.7	1.1
	9.0	4.0	2.9	3.3	0.7	<0.1	0.1	0.1	0.1
		16.0	4.3	7.4	2.1	0.3	0.5	0.3	0.3
	11.0	8.0	5.8	9.2	2.3	0.1	0.5	0.4	0.3
	14.0	6.0	3.3	8.5	1.8	0.3	1.1	0.8	0.7
	17.0	10.0	5.1	5.6	2.6	0.4	1.8	2.8	1.0
	19.0	6.0	4.0	8.7	2.1	0.4	1.3	0.7	1.2
					SWL = 0.0 FT				
	7.0	4.0	1.0	3.5	0.4	<0.1	0.2	<0.1	0.1
		10.0	2.4	5.1	2.6	0.2	1.9	0.7	0.5
	9.0	4.0	1.3	5.1	1.7	0.1	0.3	0.5	0.3
		16.0	3.3	7.4	2.2	0.2	0.7	0.8	0.4
	11.0	8.0	2.9	5.6	1.3	0.1	0.6	0.3	0.3
	14.0	6.0	2.3	6.5	1.0	0.1	0.6	0.3	0.3
	17.0	10.0	3.8	5.6	2.1	0.3	1.0	1.0	0.5
	19.0	6.0	3.1	7.2	1.9	0.2	0.9	1.1	0.6

TABLE 6

WAVE HEIGHTS FOR EXISTING CONDITIONS

(TEST WAVES FROM SOUTH)

TEST WAVE			WAVE HEIGHT, FT							
DIRECTION	PERIOD SEC	HEIGHT FT	GAGE 1	GAGE 2	GAGE 3	GAGE 4	GAGE 5	GAGE 6	GAGE 7	GAGE 8
					SWL = +5.4 FT					
S	7.0	4.0	3.7	3.5	3.4	4.2	4.0	3.8	0.1	1.2
		10.0	10.6	9.3	11.1	11.3	15.7	10.8	0.2	2.3
	9.0	4.0	4.5	4.1	3.3	6.1	5.5	5.5	0.3	0.8
		10.0	11.9	11.0	10.2	13.0	18.0	13.5	0.5	2.4
	14.0	6.0	8.5	6.8	10.7	10.2	11.7	13.0	0.6	2.0
	17.0	6.0	8.5	9.2	8.4	10.6	14.3	13.0	0.7	3.9
	19.0	4.0	3.7	4.5	4.4	6.1	7.0	7.8	0.9	1.7
					SWL = 0.0 FT					
	7.0	4.0	4.3	4.0	4.2	5.4	5.2	5.5	0.2	1.1
		10.0	10.8	9.2	10.6	10.6	12.4	8.5	0.1	1.7
	9.0	4.0	5.0	4.1	4.2	5.9	6.2	6.2	<0.1	1.1
		10.0	12.0	11.2	10.4	14.4	13.1	8.2	0.2	2.2
	14.0	6.0	9.2	7.5	7.8	13.7	11.6	10.3	0.1	0.9
	17.0	6.0	8.8	8.0	7.2	11.4	14.5	12.2	0.2	1.9
	19.0	4.0	3.8	4.6	4.0	5.9	8.1	6.9	0.1	1.2

(CONTINUED)

TABLE 6 (CONCLUDED)

TEST WAVE			WAVE HEIGHT, FT						
DIRECTION	PERIOD SEC	HEIGHT FT	GAGE 9	GAGE 10	GAGE 11	GAGE 12	GAGE 13	GAGE 14	GAGE 15
			SWL = +5.4 FT						
S	7.0	4.0	2.7	1.5	0.4	<0.1	0.5	0.2	0.1
		10.0	5.0	9.3	1.1	0.2	1.3	0.7	0.5
	9.0	4.0	3.2	2.4	0.4	<0.1	<0.1	<0.1	<0.1
		10.0	7.5	9.4	2.2	0.2	0.4	0.4	0.1
	14.0	6.0	5.4	7.6	1.3	0.2	1.0	0.5	0.8
	17.0	6.0	5.9	7.3	2.2	0.3	1.6	2.4	0.8
	19.0	4.0	3.9	4.1	1.1	0.3	0.6	0.3	0.7
			SWL = 0.0 FT						
	7.0	4.0	2.5	2.0	0.7	<0.1	0.5	<0.1	0.2
		10.0	4.7	5.7	1.9	0.1	1.4	0.3	0.3
	9.0	4.0	1.6	1.8	0.8	<0.1	0.2	0.2	0.1
		10.0	4.1	4.9	2.1	0.2	0.5	0.4	0.2
	14.0	6.0	4.1	7.8	1.2	0.1	0.7	0.2	0.2
	17.0	6.0	3.8	7.8	2.3	0.2	1.3	1.1	0.5
	19.0	4.0	3.2	2.7	0.8	0.1	0.2	0.4	0.2

TABLE 7

COMPARISON OF ROCK VOLUMES FOR VARIOUS TEST PLANS

PLAN NUMBER	VOLUME × $10^3 YD^3$	PLAN NUMBER	VOLUME × $10^3 YD^3$
	HARBOR PLANS		
1	181.7	31D	425.8
1A	259.1	31E	448.3
1B	296.9	31F	449.5
1C	287.5	32	102.9
1D	278.0	33	84.4
2	226.9	34	89.7
2A	231.5	35	109.6
2B	231.5	35A	103.3
3	231.5	36	71.0
4	261.8	37	84.7
5	262.7	38	74.3
19	185.5	38A	79.6
20	207.8	38B	81.7
21	233.1	38C	87.0
21A	252.7	38D	92.7
21B	243.7	39	92.7
21C	254.3	39A	98.0
21D	264.9	40	116.9
21E	264.9	40A	112.6
21F	253.5	40B	158.4
22	252.8	41	71.3
22A	285.0	42	45.9
23	291.3	43	45.9
24	313.4	44	87.0
25	100.4	45	26.4
25A	111.5	46	220.8
26	239.4	47	237.7
26A	230.9	47A	241.1
27	249.5	47B	234.3
27A	241.9	48	236.0
27B	234.2	48A	234.3
27C	226.6	48B	227.5
28	367.0	49	198.8
29	319.6	50	216.8
30	373.6	50A	210.8
30A	427.0	51	193.9
31	373.6	52	204.1
31A	399.2	53	192.1
31B	412.6	54	198.9
31C	427.0	55	198.9

(CONTINUED)

TABLE 7 (CONCLUDED)

PLAN NUMBER	VOLUME × $10^3 YD^3$	PLAN NUMBER	VOLUME × $10^3 YD^3$
HARBOR PLANS (CONTINUED)			
56	192.1	60	227.1
57	192.1	61	125.4
58	227.1	62	125.4
59	227.1		
BEACH PLANS			
6	85.6	12	171.1
7	125.5	13	447.1
8	119.6	14	63.6
9	102.7	14A	71.9
9A	109.4	15	32.5
9B	109.4	17	54.5
10	153.8	18	84.6
11	171.1		

TABLE 8

WAVE HEIGHTS FOR PLAN 4

(TEST WAVES FROM NW)

Test Wave			Wave Height, ft							
Direction	Period sec	Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8
			SWL = +5.4 FT							
NW	7.0	5.0	0.7	1.0	2.0	1.1	0.8	0.9	0.2	0.3
		10.0	4.6	2.4	3.5	3.8	3.2	2.2	0.2	1.8
			SWL = 0.0 FT							
		5.0	1.4	0.9	0.8	1.1	1.2	0.8	<0.1	0.5
		10.0	1.9	1.7	2.1	1.8	2.9	1.6	0.1	0.4

Test Wave			Wave Height, ft						
Direction	Period sec	Height ft	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 14	Gage 15
			SWL = +5.4 FT						
NW	7.0	5.0	0.8	0.9	0.2	<0.1	0.2	0.1	<0.1
		10.0	0.9	1.8	0.6	<0.1	0.5	0.2	0.1
			SWL = 0.0 FT						
		5.0	0.2	0.2	0.3	<0.1	0.3	<0.1	<0.1
		10.0	0.3	0.4	0.3	<0.1	0.4	0.2	0.1

TABLE 9

WAVE HEIGHTS FOR PLAN 4

(TEST WAVES FROM WEST)

Test Wave Direction	Period, Sec	Height, Ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8
				SWL = +5.4 FT						
W	7.0	10.0	3.1	2.4	3.0	2.7	2.3	1.9	0.3	1.2
	9.0	12.0	5.5	4.7	5.3	5.0	6.4	6.6	0.3	1.7
	11.0	10.0	7.3	6.1	5.9	7.4	8.6	6.0	0.5	4.2
	17.0	6.0	4.2	4.2	4.4	3.2	4.6	3.3	0.6	2.8
				SWL = 0.0 FT						
	7.0	10.0	1.7	1.7	2.2	0.7	1.5	1.6	<0.1	0.3
	9.0	12.0	3.7	3.0	2.4	2.8	3.5	3.3	0.1	0.6
	11.0	10.0	6.6	4.2	3.9	3.1	2.9	3.2	0.1	0.7
	17.0	6.0	4.7	2.4	3.4	2.2	3.7	2.8	0.2	0.7

Test Wave Direction	Period, Sec	Height, Ft	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 14	Gage 15
				SWL = +5.4 FT					
W	7.0	10.0	1.0	2.1	0.5	0.1	0.5	0.4	0.2
	9.0	12.0	1.2	4.7	1.8	0.2	0.5	0.4	0.3
	11.0	10.0	2.6	5.1	1.7	0.2	0.4	0.5	0.3
	17.0	6.0	1.6	3.0	1.0	0.2	0.4	1.0	0.5
				SWL = 0.0 FT					
	7.0	10.0	0.2	0.8	0.5	<0.1	0.3	<0.1	<0.1
	9.0	12.0	0.7	1.4	0.6	0.1	0.2	0.2	0.2
	11.0	10.0	0.6	1.1	0.5	0.1	0.2	0.2	0.1
	17.0	6.0	1.1	1.4	0.5	<0.1	0.3	0.3	0.2

TABLE 10

WAVE HEIGHTS FOR PLAN 4

(TEST WAVES FROM SW)

TEST WAVE DIRECTION	PERIOD SEC	HEIGHT FT	GAGE 1	GAGE 2	GAGE 3	GAGE 4	GAGE 5	GAGE 6	GAGE 7	GAGE 8
					SWL = +5.4 FT					
SW	7.0	10.0	4.5	1.8	3.3	1.4	2.4	2.2	0.4	1.0
	9.0	6.0	7.4	12.1	10.2	7.8	7.6	9.0	0.5	2.5
	11.0	8.0	5.4	6.3	4.4	5.1	4.3	3.1	0.7	2.9
	17.0	10.0	6.1	8.3	10.4	7.3	6.0	6.8	0.8	3.0
					SWL = 0.0 FT					
	7.0	10.0	1.9	1.5	2.4	1.1	0.8	1.7	0.2	0.2
	9.0	6.0	3.6	3.0	5.6	4.7	2.5	3.0	0.2	0.7
	11.0	8.0	2.5	2.1	3.1	2.9	3.3	2.4	0.2	0.4
	17.0	10.0	7.9	4.8	6.4	3.2	4.4	4.8	0.2	0.8

TEST WAVE DIRECTION	PERIOD SEC	HEIGHT FT	GAGE 9	GAGE 10	GAGE 11	GAGE 12	GAGE 13	GAGE 14	GAGE 15
					SWL = +5.4 FT				
SW	7.0	10.0	1.0	1.3	0.3	<0.1	0.4	0.2	0.1
	9.0	6.0	2.5	5.8	1.6	0.2	0.5	0.4	0.3
	11.0	8.0	2.3	3.3	0.8	<0.1	0.2	0.3	0.3
	17.0	10.0	3.7	3.0	1.4	0.3	1.0	2.0	0.7
					SWL = 0.0 FT				
	7.0	10.0	0.5	0.7	0.1	<0.1	<0.1	<0.1	<0.1
	9.0	6.0	1.2	1.7	0.8	0.1	0.4	0.3	0.2
	11.0	8.0	1.0	1.7	0.4	<0.1	0.1	0.1	0.1
	17.0	10.0	2.3	1.9	0.5	0.2	0.4	0.3	0.2

TABLE 11

WAVE HEIGHTS FOR PLAN 4

(TEST WAVES FROM SOUTH)

TEST WAVE			WAVE HEIGHT, FT							
DIRECTION	PERIOD SEC	HEIGHT FT	GAGE 1	GAGE 2	GAGE 3	GAGE 4	GAGE 5	GAGE 6	GAGE 7	GAGE 8
				SWL = +5.4 FT						
S	7.0	10.0	4.0	3.2	3.5	2.8	2.6	1.8	0.1	1.1
	9.0	10.0	4.7	4.8	6.6	2.7	4.5	4.8	0.5	1.6
	17.0	6.0	4.8	2.2	5.5	1.9	3.8	3.9	0.8	2.4
				SWL = 0.0 FT						
	7.0	10.0	1.7	0.8	2.9	1.4	1.4	0.9	<0.1	0.3
	9.0	10.0	2.3	2.0	3.7	2.3	1.5	1.6	0.1	0.2
	17.0	6.0	1.8	2.0	4.0	1.8	3.0	2.1	0.2	0.6

TEST WAVE			WAVE HEIGHT, FT						
DIRECTION	PERIOD SEC	HEIGHT FT	GAGE 9	GAGE 10	GAGE 11	GAGE 12	GAGE 13	GAGE 14	GAGE 15
				SWL = +5.4 FT					
S	7.0	10.0	1.5	1.1	0.3	<0.1	0.2	0.2	0.1
	9.0	10.0	2.7	2.1	0.8	0.2	0.2	0.3	0.2
	17.0	6.0	2.9	1.4	0.3	<0.1	0.3	0.6	0.2
				SWL = 0.0 FT					
	7.0	10.0	0.4	0.3	0.1	<0.1	<0.1	<0.1	<0.1
	9.0	10.0	0.6	0.6	0.2	<0.1	<0.1	<0.1	<0.1
	17.0	6.0	1.1	0.5	<0.1	<0.1	<0.1	<0.1	<0.1

TABLE 12

WAVE HEIGHTS FOR PLAN 25

IN THE HARBOR

SWL = +5.4 FT

TEST WAVE			WAVE HEIGHT, FT													
DIRECTION	PERIOD SEC	HEIGHT FT	GAGE 1	GAGE 2	GAGE 3	GAGE 4	GAGE 5	GAGE 6	GAGE 7	GAGE 8	GAGE 9	GAGE 10	GAGE 11	GAGE 12	GAGE 13	GAGE 14
NW	7.0	5.0	5.4	1.3	1.8	2.3	0.6	0.2	<0.1	0.3	0.4	0.6	<0.1	0.8	0.3	0.1
		10.0	9.4	2.4	2.5	5.9	1.8	0.5	0.2	0.4	0.6	1.3	0.1	1.5	0.5	0.2
W		4.0	6.1	1.0	1.0	3.9	1.2	0.1	0.1	0.4	0.4	0.9	0.2	1.4	0.4	0.3
		10.0	16.2	2.0	2.6	5.8	2.0	0.2	0.3	0.5	1.2	1.8	0.2	2.0	0.8	0.2
	9.0	12.0	11.8	1.7	2.3	4.6	2.9	0.4	0.5	0.8	0.4	2.0	0.2	0.3	0.3	0.2
	11.0	10.0	8.4	3.3	2.6	6.3	3.2	0.9	0.9	0.7	1.0	3.1	0.2	1.2	1.2	0.7
	14.0	6.0	14.6	3.3	3.6	4.1	4.1	0.8	1.3	1.0	2.5	1.4	0.2	0.4	0.4	0.5
	17.0	6.0	13.7	3.1	2.8	4.8	1.6	0.6	1.2	0.7	1.9	1.5	0.2	0.8	1.0	0.9
SW	7.0	4.0	3.2	1.3	2.4	2.7	0.8	0.4	0.2	0.3	0.7	0.8	<0.1	1.0	0.4	0.1
		10.0	9.5	2.9	2.7	9.0	3.0	0.4	0.3	0.7	1.2	2.0	0.3	2.7	1.3	0.3
	9.0	16.0	12.3	4.1	2.5	6.4	3.0	0.9	1.2	0.9	1.2	2.1	0.1	0.4	0.4	0.2
	11.0	8.0	9.0	5.4	3.5	5.1	2.8	0.7	0.9	0.9	1.4	2.8	0.4	0.6	0.7	0.5
	14.0	6.0	7.7	6.0	2.9	4.9	3.7	0.9	1.2	1.3	2.4	1.2	0.2	0.7	0.4	0.2
	17.0	1.2	12.5	4.7	2.7	6.9	2.7	1.8	2.0	1.2	1.4	2.2	0.3	1.1	1.5	1.2
S	7.0	4.0	4.1	3.0	1.4	4.1	1.3	0.3	0.2	0.4	0.9	1.3	0.2	1.8	0.6	0.2
		10.0	10.4	2.7	2.7	9.8	3.7	0.7	0.6	1.1	2.3	2.3	0.4	2.6	2.1	0.4
	9.0	4.0	3.3	3.3	0.6	2.1	0.7	0.3	0.4	0.2	1.2	0.5	<0.1	0.2	0.2	0.2
		10.0	10.0	4.2	3.8	7.7	3.2	0.9	0.7	1.9	1.6	2.0	0.2	0.6	0.5	0.4
	14.0	6.0	8.5	6.0	4.3	4.5	6.3	1.2	2.4	1.9	3.1	1.7	0.2	0.9	0.5	0.6
	17.0	6.0	6.1	5.1	4.2	5.0	3.2	1.6	3.1	2.3	2.5	1.8	0.2	0.9	1.5	0.8

TABLE 13

WAVE HEIGHTS FOR PLANS 21-21F

IN THE HARBOR

SWL = +5.4 FT

TEST WAVE			WAVE HEIGHT, FT						
DIRECTION	PERIOD SEC	HEIGHT FT	GAGE 1	GAGE 2	GAGE 3	GAGE 4	GAGE 5	GAGE 6	GAGE 7
			PLAN 21						
S	7.0	4.0	2.8	1.5	0.5	2.6	0.9	0.4	0.2
		10.0	6.0	2.8	1.7	5.7	2.2	0.6	0.4
	9.0	4.0	1.1	2.3	1.2	1.4	0.5	0.4	0.3
		10.0	7.0	4.0	2.7	6.9	2.7	0.6	0.7
	14.0	6.0	5.5	4.3	2.4	3.0	4.6	0.6	2.2
SW	7.0	4.0	0.6	1.8	1.5	1.1	0.4	0.3	0.1
		10.0	3.3	1.4	1.9	2.5	1.0	0.2	0.2
	9.0	16.0	13.1	4.1	4.1	6.5	2.5	1.1	0.7
	11.0	8.0	7.5	3.5	2.4	2.8	2.1	0.6	0.3
	14.0	6.0	8.3	2.8	3.5	3.0	3.3	0.4	2.0
	17.0	1.2	11.2	7.3	3.3	5.9	3.7	0.8	3.2
			PLAN 21B						
		10.0	4.7	0.9	1.9	1.2	2.2	<0.1	<0.1
			PLAN 21C						
		10.0	4.7	0.9	1.7	0.9	1.9	<0.1	<0.1
			PLAN 21D						
	7.0	4.0	0.7	0.8	2.5	1.2	0.4	0.2	0.1
		10.0	4.2	2.0	1.7	2.3	0.4	0.3	0.2
	9.0	16.0	9.4	3.3	3.2	5.6	1.0	0.4	0.2
	11.0	8.0	7.7	4.0	2.7	2.6	1.1	0.4	0.3
	14.0	6.0	7.9	3.3	3.9	2.8	1.3	0.4	1.2
	17.0	10.0	8.8	7.8	3.8	6.2	2.9	1.0	1.8
		10.0	5.1	1.0	3.1	2.4	3.7	<0.1	<0.1
			PLAN 21E						
	7.0	4.0	0.6	1.5	1.8	1.2	0.3	0.3	0.2
			PLAN 21F						
		10.0	4.2	1.8	2.1	3.4	0.7	0.2	0.2
	9.0	16.0	10.6	3.0	3.9	6.3	0.8	1.1	0.6
	11.0	8.0	6.5	4.9	3.3	2.3	0.9	0.4	0.3
	14.0	6.0	8.9	4.0	4.3	3.7	2.1	1.0	1.8
	17.0	10.0	8.9	7.6	3.9	7.3	3.7	1.7	2.1
W	7.0	4.0	1.1	0.5	1.5	1.0	0.3	0.4	<0.1
		10.0	2.1	2.2	2.8	0.7	0.2	0.2	0.2
	9.0	12.0	5.1	2.3	2.3	3.0	0.5	0.6	0.3
	11.0	10.0	2.7	3.8	2.7	2.0	0.7	0.8	0.4
	14.0	6.0	5.8	3.9	3.0	2.6	0.8	0.6	0.7
	17.0	6.0	7.3	4.9	4.2	2.2	1.4	1.0	0.4

(CONTINUED)

TABLE 13 (CONCLUDED)

TEST WAVE			WAVE HEIGHT, FT						
DIRECTION	PERIOD SEC	HEIGHT FT	GAGE 8	GAGE 9	GAGE 10	GAGE 11	GAGE 12	GAGE 13	GAGE 14
			PLAN 21						
S	7.0	4.0	0.3	0.5	0.9	0.1	1.2	0.4	0.1
		10.0	0.4	1.6	1.3	0.1	1.6	0.5	0.2
	9.0	4.0	0.4	0.4	0.2	<0.1	<0.1	<0.1	<0.1
		10.0	1.9	1.6	1.3	0.1	0.4	0.5	0.3
	14.0	6.0	1.2	2.9	0.9	0.1	0.3	0.2	0.3
SW	7.0	4.0	<0.1	0.2	0.3	<0.1	0.3	0.2	<0.1
		10.0	0.4	0.7	0.9	<0.1	1.1	0.4	0.2
	9.0	16.0	0.9	1.5	1.4	0.1	0.3	0.3	0.3
	11.0	8.0	0.6	0.9	1.9	0.2	0.4	0.4	0.2
	14.0	6.0	1.0	2.2	0.8	0.1	0.4	0.2	0.3
	17.0	1.2	1.6	3.3	2.3	0.4	1.1	1.8	0.9
			PLAN 21B						
		10.0	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
			PLAN 21C						
		10.0	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
			PLAN 21D						
	7.0	4.0	0.1	0.2	0.3	<0.1	0.7	0.3	0.1
		10.0	0.3	0.5	0.5	<0.1	1.0	0.5	0.3
	9.0	16.0	0.7	1.1	1.1	0.2	0.6	0.6	0.6
	11.0	8.0	0.3	0.2	1.1	0.2	0.4	0.4	0.3
	14.0	6.0	0.4	1.0	0.7	0.2	0.5	0.4	0.2
	17.0	10.0	0.8	1.7	2.1	0.3	1.2	1.4	0.9
		10.0	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
			PLAN 21E						
	7.0	4.0	<0.1	0.4	0.4	<0.1	0.8	0.4	0.1
			PLAN 21F						
		10.0	0.3	0.6	0.9	0.1	1.5	0.7	0.2
	9.0	16.0	1.2	1.6	1.3	0.3	0.6	0.8	0.3
	11.0	8.0	0.3	0.3	1.0	0.3	0.4	0.4	0.3
	14.0	6.0	0.4	1.3	1.0	0.2	0.8	0.4	0.4
	17.0	10.0	1.4	2.0	2.2	0.3	1.4	1.2	1.0
W	7.0	4.0	0.1	0.3	0.4	<0.1	0.5	0.2	0.1
		10.0	0.2	0.2	0.2	<0.1	0.2	0.1	<0.1
	9.0	12.0	0.4	0.6	0.8	0.1	0.3	0.3	0.2
	11.0	10.0	0.3	0.4	0.7	0.1	0.2	0.1	0.2
	14.0	6.0	0.6	1.3	1.1	0.2	0.6	0.4	0.5
	17.0	6.0	1.0	0.4	0.7	0.1	0.4	0.8	0.3

TABLE 14

WAVE HEIGHTS FOR PLAN 25A

IN THE HARBOR

SWL = +5.4 FT

Test Wave Direction	Test Wave Period SEC	Test Wave Height FT	Wave Height, FT: Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 14
W	7.0	4.0	4.6	1.1	1.4	4.3	0.8	0.1	0.1	0.2	0.3	1.5	0.2	2.1	1.1	0.4
		10.0	12.1	1.8	1.6	6.1	1.2	0.1	0.2	0.2	0.2	1.9	0.4	2.7	1.3	0.8
	9.0	12.0	8.2	1.2	1.7	5.1	1.1	0.4	0.4	0.5	0.7	2.2	0.4	0.5	0.5	0.3
	11.0	10.0	9.3	2.3	2.8	4.8	2.0	0.5	0.5	0.6	0.6	3.3	0.3	1.1	1.2	0.6
	14.0	6.0	12.9	2.6	2.0	4.8	2.2	0.8	1.0	1.1	2.4	2.0	0.3	1.2	0.7	0.6
	17.0	6.0	14.0	2.5	2.7	5.9	1.8	1.0	1.2	1.2	2.4	2.2	0.3	1.4	2.0	0.9
SW	7.0	4.0	3.7	1.4	1.7	2.3	0.4	0.3	<0.1	0.1	0.6	0.6	<0.1	1.1	0.3	0.2
		10.0	10.0	2.8	2.5	10.8	2.7	0.4	0.2	0.3	0.5	3.3	0.5	4.4	1.5	1.1
	9.0	16.0	11.1	3.9	3.4	7.1	1.3	1.2	1.0	0.8	1.7	2.0	0.3	0.8	0.6	0.4
	11.0	8.0	7.3	5.3	3.4	5.1	2.4	0.6	0.4	0.8	0.4	3.0	0.3	1.1	1.2	0.5
	14.0	6.0	9.7	6.1	2.9	5.3	2.8	1.1	1.0	0.7	2.6	2.3	0.4	2.1	1.3	0.7
	17.0	10.0	7.8	5.8	3.0	4.7	1.7	1.0	1.0	0.7	1.8	1.9	0.3	1.4	1.9	0.6

TABLE 15

WAVE HEIGHTS FOR PLANS 26 AND 26A

IN THE HARBOR

SWL = +5.4 FT

Test Wave Direction	Test Wave Period sec	Test Wave Height ft	Wave Height, ft Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 14
								PLAN 26								
SW	7.0	4.0	4.3	1.1	0.7	3.5	0.9	0.3	<0.1	0.1	0.6	1.2	0.2	1.7	0.7	0.3
		10.0	9.8	2.0	1.1	13.2	3.2	1.1	0.3	0.4	0.7	4.1	0.8	5.5	2.7	1.1
	9.0	16.0	11.0	2.8	2.3	8.3	2.1	1.4	0.6	0.9	1.7	2.1	0.4	1.2	1.0	0.7
	11.0	8.0	8.8	2.5	1.7	4.7	2.5	0.5	0.5	0.5	0.6	1.8	0.3	0.6	0.7	0.6
	14.0	6.0	11.2	7.5	2.3	5.6	2.8	1.5	1.0	0.9	2.8	2.6	0.4	1.8	1.3	0.8
	17.0	10.0	22.7	3.8	1.9	10.5	5.9	1.8	2.0	1.2	2.6	4.2	0.6	1.8	2.6	1.3
								PLAN 26A								
	7.0	4.0	3.5	1.4	0.6	3.0	0.6	0.3	<0.1	0.2	0.6	0.9	0.1	1.3	0.4	0.2
		10.0	9.2	1.6	0.8	9.0	1.7	0.6	0.2	0.3	0.5	2.8	0.4	3.5	1.8	1.2
	9.0	16.0	13.5	2.4	1.7	6.9	2.0	0.8	0.7	0.7	1.9	2.1	0.4	0.9	0.6	0.6
	11.0	8.0	8.7	2.2	1.2	4.7	2.8	0.5	0.6	0.6	0.6	2.0	0.3	0.6	0.6	0.4
	14.0	6.0	9.0	6.2	1.5	6.4	2.3	1.0	1.4	0.8	2.5	2.4	0.4	2.1	1.1	0.8
	17.0	10.0	18.0	3.5	2.3	10.4	5.6	2.4	2.6	2.0	2.4	4.0	0.5	2.0	2.0	1.4

TABLE 16

WAVE HEIGHTS FOR PLANS 27-27C

IN THE HARBOR

SWL = +5.4 FT

Test Wave Direction	Test Wave Period SEC	Test Wave Height FT	Wave Height, FT Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 14
							PLAN 27									
SW	7.0	4.0	3.4	1.3	0.5	3.0	0.5	0.3	<0.1	0.3	0.4	0.9	0.1	1.4	0.4	0.2
		10.0	10.0	1.2	0.5	8.1	2.0	0.3	0.3	0.3	0.4	2.9	0.5	3.3	2.3	1.1
	9.0	16.0	14.2	3.3	1.4	6.5	1.6	0.8	0.5	0.6	1.3	2.1	0.3	0.9	0.8	0.6
	11.0	8.0	10.9	2.8	1.5	8.9	3.4	0.4	0.5	0.5	0.7	3.7	0.4	1.1	0.7	0.7
	14.0	6.0	10.0	6.1	1.3	5.3	2.6	0.9	0.9	0.6	2.2	2.6	0.4	1.9	1.1	0.8
	17.0	10.0	20.2	3.8	1.8	9.7	5.9	1.5	1.6	1.2	2.0	4.5	0.7	1.9	2.7	1.5
							PLAN 27A									
		10.0	21.6	3.0	2.2	10.5	6.5	1.8	2.0	1.4	1.7	5.0	0.7	2.0	2.6	1.5
							PLAN 27B									
	7.0	4.0	3.5	0.4	0.2	3.5	0.8	0.2	0.2	0.2	0.5	1.3	0.1	1.6	0.8	0.2
		10.0	10.4	1.7	0.6	10.0	2.1	0.6	0.3	0.4	0.7	3.7	0.7	4.6	1.8	0.8
	9.0	16.0	14.5	3.7	2.3	9.1	1.3	1.0	0.6	0.8	1.5	2.0	0.3	0.8	0.7	0.4
	11.0	8.0	9.5	2.4	1.6	5.7	2.4	0.5	0.3	0.9	0.6	1.9	0.3	0.6	0.6	0.5
	14.0	6.0	9.1	6.5	1.5	5.3	1.9	0.7	1.3	1.2	2.1	2.3	0.3	1.9	1.0	0.7
	17.0	10.0	22.3	3.7	1.6	9.1	5.3	1.8	2.0	0.8	1.5	4.0	0.8	2.0	2.8	1.4
							PLAN 27C									
		10.0	21.4	3.4	2.0	10.1	6.4	2.0	2.9	1.6	1.7	4.6	0.7	2.0	2.6	1.3

TABLE 17

WAVE HEIGHTS FOR PLANS 28,29,30 AND 30A

IN THE HARBOR

SWL = +5.4 FT

TEST WAVE			WAVE HEIGHT, FT													
DIRECTION	PERIOD SEC	HEIGHT FT	GAGE 1	GAGE 2	GAGE 3	GAGE 4	GAGE 5	GAGE 6	GAGE 7	GAGE 8	GAGE 9	GAGE 10	GAGE 11	GAGE 12	GAGE 13	GAGE 14
							PLAN 28									
SW	7.0	4.0	0.6	2.0	1.1	1.2	0.2	<0.1	0.2	0.1	0.3	0.4	<0.1	0.5	0.3	0.1
		10.0	4.1	1.1	0.8	2.9	0.4	0.3	0.2	0.2	0.3	0.8	0.2	1.2	0.4	0.4
	9.0	16.0	12.3	3.5	1.7	5.0	1.2	0.5	0.4	0.7	0.9	1.3	0.2	0.6	0.6	0.5
	11.0	8.0	7.0	2.7	1.3	1.9	1.2	0.3	0.5	0.4	0.4	0.7	0.2	0.3	0.2	0.2
	14.0	6.0	7.3	1.6	1.2	4.4	2.3	1.2	1.4	0.4	1.1	1.7	0.3	1.0	0.6	0.6
	17.0	10.0	1.4	0.9	0.8	1.1	0.8	0.7	0.6	0.6	0.6	0.8	0.6	0.8	0.9	0.6
							PLAN 29									
	14.0	6.0	4.2	1.3	0.9	2.3	1.7	3.6	0.8	2.7	3.4	0.4	<0.1	0.2	0.2	0.1
	17.0	10.0	7.7	2.1	1.1	3.1	2.2	4.1	3.5	2.0	3.9	1.0	0.1	0.5	1.0	0.5
							PLAN 30									
	14.0	6.0	2.7	1.0	0.7	1.7	1.0	3.2	0.8	2.6	2.6	0.3	<0.1	0.1	0.2	<0.1
	17.0	10.0	6.3	1.1	1.2	2.7	1.9	2.9	2.4	2.9	3.4	0.6	0.2	0.3	0.8	0.3
							PLAN 30A									
		10.0	5.5	0.8	0.8	1.9	1.5	2.4	1.9	2.3	2.8	0.6	0.1	0.3	0.7	0.3

TABLE 18

WAVE HEIGHTS FOR PLANS 31-31D IN THE HARBOR, SWL = +5.4 FT

TEST WAVE			WAVE HEIGHT, FT													
DIRECTION	PERIOD SEC	HEIGHT FT	GAGE 1	GAGE 2	GAGE 3	GAGE 4	GAGE 5	GAGE 6	GAGE 7	GAGE 8	GAGE 9	GAGE 10	GAGE 11	GAGE 12	GAGE 13	GAGE 14
PLAN 31 BREAKWATER PARTIALLY SEALED																
SW	11.0	8.0	2.9	0.8	0.9	1.0	0.5	0.9	0.8	1.1	1.4	0.5	<0.1	<0.1	<0.1	<0.1
	14.0	6.0	2.4	1.2	0.7	1.3	0.6	3.0	1.0	2.2	1.9	0.3	<0.1	0.1	0.1	0.1
	17.0	10.0	3.1	1.2	0.7	1.5	1.1	1.6	1.1	1.7	1.6	0.4	0.1	0.2	0.4	0.2
PLAN 31A BREAKWATER PARTIALLY SEALED																
	7.0	4.0	0.4	0.2	<0.1	0.1	<0.1	0.5	0.1	0.3	0.5	<0.1	<0.1	<0.1	<0.1	<0.1
		10.0	0.4	0.3	0.2	0.4	0.5	0.7	0.2	0.3	0.8	0.2	<0.1	0.2	0.2	<0.1
	9.0	16.0	3.9	1.1	0.5	2.1	1.1	1.5	0.9	1.1	1.2	0.4	<0.1	0.1	0.1	0.1
	17.0	10.0	3.7	0.8	0.9	1.4	0.9	1.6	1.1	2.0	1.7	0.5	0.2	0.2	0.5	0.2
	14.0	6.0	2.2	0.9	0.5	1.1	0.8	2.1	0.9	1.3	1.1	0.2	<0.1	0.1	0.1	0.1
	11.0	8.0	1.9	0.8	0.5	1.0	0.4	1.6	1.1	1.1	1.1	0.2	<0.1	<0.1	<0.1	<0.1
S	7.0	4.0	0.8	<0.1	<0.1	0.3	0.2	0.4	0.1	0.2	0.4	0.1	<0.1	0.1	<0.1	<0.1
		10.0	1.2	0.5	0.2	0.4	0.5	0.9	0.5	0.4	0.7	0.2	<0.1	0.2	0.1	<0.1
	9.0	4.0	0.7	0.5	0.2	0.4	0.5	1.0	0.4	0.7	0.5	0.3	<0.1	<0.1	<0.1	<0.1
		10.0	2.3	0.7	0.4	1.4	0.4	2.5	1.3	2.1	1.1	0.3	<0.1	<0.1	<0.1	<0.1
	14.0	6.0	1.0	1.2	0.5	0.8	0.6	2.1	1.0	2.0	1.5	0.3	<0.1	0.1	0.1	0.1
	17.0	6.0	2.0	1.8	0.6	1.2	0.7	0.9	1.8	2.4	2.2	0.5	0.1	0.3	0.4	0.2
PLAN 31B BREAKWATER PARTIALLY SEALED																
	9.0	10.0	1.7	0.6	0.3	1.3	0.5	1.5	1.2	1.6	0.5	0.2	<0.1	<0.1	<0.1	<0.1
	14.0	6.0	1.0	0.9	0.8	0.8	0.6	1.6	1.0	2.2	1.9	0.6	0.1	0.4	0.3	0.2
	17.0	6.0	2.2	1.4	0.4	1.5	0.9	1.0	1.9	2.7	2.0	0.4	0.2	0.3	0.2	0.1
PLAN 31C BREAKWATER PARTIALLY SEALED																
		6.0	2.3	1.3	0.4	1.3	1.0	1.1	1.9	2.8	2.2	0.4	0.2	0.2	0.2	0.1
PLAN 31D BREAKWATER PARTIALLY SEALED																
		6.0	2.4	1.4	0.4	1.2	1.0	0.9	1.9	2.4	2.3	0.5	0.2	0.2	0.3	0.2

(CONTINUED)

TABLE 18 (CONCLUDED)

Test Wave Direction	Test Wave Period sec	Test Wave Height ft	Wave Height, ft Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 14
			PLAN 31D BREAKWATER COMPLETELY SEALED													
S	17.0	6.0	1.7	1.0	0.4	1.2	0.8	0.6	1.5	2.1	1.8	0.5	0.1	0.2	0.4	0.2
			PLAN 31E BREAKWATER COMPLETELY SEALED													
		6.0	1.9	0.9	0.5	0.8	0.7	0.8	0.8	2.1	1.3	0.5	0.1	0.3	0.5	0.2
			PLAN 31A BREAKWATER COMPLETELY SEALED													
	7.0	4.0	0.3	<0.1	<0.1	0.3	0.1	0.5	0.3	0.4	0.3	0.1	<0.1	0.1	<0.1	<0.1
		10.0	1.1	0.5	0.2	0.6	0.3	0.6	0.5	0.4	0.5	0.2	<0.1	0.2	<0.1	<0.1
	9.0	4.0	0.5	0.3	0.2	0.3	0.6	0.7	0.3	0.8	0.7	0.3	<0.1	<0.1	0.1	<0.1
		10.0	2.3	0.7	0.6	1.4	0.5	2.3	1.2	1.7	0.6	0.3	<0.1	<0.1	0.2	0.1
	14.0	6.0	1.6	1.1	0.6	0.6	0.5	1.8	1.3	1.8	0.8	0.4	<0.1	0.2	0.2	<0.1
	17.0	6.0	1.6	1.3	0.3	1.3	0.6	0.6	1.5	2.1	1.7	0.5	0.1	0.3	0.4	0.1
			PLAN 31F BREAKWATER COMPLETELY SEALED													
	7.0	4.0	0.6	0.3	0.2	0.5	<0.1	0.2	0.3	0.3	0.5	0.2	<0.1	0.2	0.1	<0.1
		10.0	0.7	0.5	0.2	0.6	0.3	0.9	0.9	0.5	0.7	0.1	<0.1	0.1	<0.1	<0.1
	9.0	4.0	1.1	0.3	0.2	0.3	0.7	0.7	0.4	0.6	0.3	0.3	<0.1	0.1	0.2	0.1
		10.0	1.8	0.7	0.4	1.5	0.7	0.9	0.7	1.0	0.7	0.2	<0.1	<0.1	0.1	<0.1
	14.0	6.0	1.9	0.5	0.7	0.9	0.4	1.7	1.0	1.8	0.8	0.4	0.1	0.2	0.2	0.1
	17.0	6.0	2.0	1.1	0.6	0.9	0.6	0.9	1.1	2.3	1.4	0.5	<0.1	0.3	0.5	0.1
W	7.0	4.0	0.7	0.1	0.2	0.3	0.1	0.1	0.2	0.1	0.2	<0.1	<0.1	0.1	<0.1	<0.1
		10.0	1.4	0.3	0.3	0.4	0.2	0.4	0.2	0.2	0.5	<0.1	<0.1	0.1	0.1	<0.1
	9.0	12.0	2.3	0.7	0.3	1.8	1.1	1.7	0.5	1.6	2.0	0.6	<0.1	0.1	0.2	0.2
	11.0	10.0	2.7	0.7	0.6	1.8	1.3	1.2	0.8	0.5	0.8	0.4	<0.1	0.1	0.1	<0.1
	14.0	6.0	2.9	1.2	0.9	1.0	0.9	1.1	1.4	0.9	1.5	0.4	<0.1	0.3	0.2	0.1
	17.0	6.0	1.6	0.7	0.4	0.8	0.6	0.9	0.9	0.4	1.2	0.3	<0.1	0.3	0.6	0.2

TABLE 19

WAVE HEIGHTS FOR PLANS 46-60

IN THE HARBOR

SWL = +5.4 FT

TEST WAVE			WAVE HEIGHT, FT													
DIRECTION	PERIOD SEC	HEIGHT FT	GAGE 1	GAGE 2	GAGE 3	GAGE 4	GAGE 5	GAGE 6	GAGE 7	GAGE 8	GAGE 9	GAGE 10	GAGE 11	GAGE 12	GAGE 13	GAGE 14
							PLAN 46									
SW	14.0	6.0	7.6	5.5	3.9	5.0	1.8	0.5	0.7	0.6	1.1	1.4	0.1	0.5	0.5	0.2
	17.0	10.0	10.6	5.6	4.3	7.9	2.7	1.3	0.8	1.6	1.2	1.5	0.3	1.0	1.4	0.4
							PLAN 47									
		10.0	10.4	4.8	4.5	5.1	2.0	1.2	0.8	1.3	0.8	1.2	0.2	0.8	1.1	0.4
							PLAN 47A									
		10.0	11.4	4.5	4.5	5.2	1.7	1.2	0.7	1.2	0.9	1.2	0.2	0.7	0.9	0.4
							PLAN 47B									
		10.0	10.9	4.8	4.1	5.9	1.9	1.2	0.7	1.3	0.9	1.2	0.2	0.8	1.2	0.4
							PLAN 47									
	7.0	4.0	4.3	2.2	1.9	0.9	0.6	0.5	0.2	0.2	0.2	0.1	<0.1	0.1	0.1	<0.1
		10.0	7.8	2.6	2.0	2.3	1.2	0.5	0.2	0.3	0.4	0.5	0.1	0.4	0.4	0.1
	9.0	16.0	10.2	4.1	2.9	3.2	1.3	0.8	0.5	0.6	0.7	1.0	0.2	0.2	0.3	0.2
	11.0	8.0	5.4	2.7	4.7	2.4	1.3	0.6	0.5	0.5	0.9	0.6	0.1	0.2	0.1	0.1
	14.0	6.0	7.4	6.1	3.8	3.2	1.1	0.5	0.6	0.7	0.8	0.5	0.1	0.3	0.2	0.2
	17.0	10.0	10.4	4.8	4.5	5.1	2.0	1.2	0.8	1.3	0.8	1.2	0.2	0.8	1.1	0.4

(CONTINUED)

TABLE 19 (CONTINUED)

TEST WAVE			WAVE HEIGHT, FT													
DIRECTION	PERIOD SEC	HEIGHT FT	GAGE 1	GAGE 2	GAGE 3	GAGE 4	GAGE 5	GAGE 6	GAGE 7	GAGE 8	GAGE 9	GAGE 10	GAGE 11	GAGE 12	GAGE 13	GAGE 14
							PLAN 48									
SW	14.0	6.0	9.2	4.9	3.8	3.6	1.7	0.6	1.0	0.6	1.0	0.7	0.1	0.3	0.3	0.2
	17.0	10.0	11.1	6.0	6.3	4.7	2.2	1.5	1.7	1.5	1.2	1.3	0.3	0.7	1.1	0.6
							PLAN 48A									
	14.0	6.0	9.7	4.7	3.7	3.4	1.6	0.7	1.0	0.6	0.9	0.6	0.1	0.4	0.3	0.2
	17.0	10.0	11.8	5.6	6.3	4.9	2.3	1.5	1.8	1.5	1.3	1.2	0.3	0.7	1.1	0.6
							PLAN 48B									
	14.0	6.0	8.7	6.3	3.4	3.5	1.4	0.6	0.9	0.8	0.8	0.8	0.1	0.3	0.3	0.2
	17.0	10.0	9.8	6.0	5.0	6.3	2.8	1.3	1.4	1.8	1.2	1.4	0.3	1.0	1.4	0.6
							PLAN 49									
		10.0	16.8	6.6	4.7	6.4	2.4	1.2	1.3	1.3	1.2	1.9	0.4	1.2	1.6	0.6
							PLAN 50									
		10.0	12.0	6.0	4.1	4.0	1.7	1.0	1.1	0.9	0.9	1.1	0.2	0.8	1.0	0.4
							PLAN 50A									
		10.0	14.5	5.9	4.7	5.2	2.1	1.3	1.2	1.2	1.0	1.3	0.3	1.0	1.2	0.5

(CONTINUED)

TABLE 19 (CONTINUED)

TEST WAVE			WAVE HEIGHT, FT													
DIRECTION	PERIOD SEC	HEIGHT FT	GAGE 1	GAGE 2	GAGE 3	GAGE 4	GAGE 5	GAGE 6	GAGE 7	GAGE 8	GAGE 9	GAGE 10	GAGE 11	GAGE 12	GAGE 13	GAGE 14
							PLAN 51									
SW	14.0	6.0	13.9	5.3	3.4	5.7	2.0	0.7	1.2	1.3	1.1	1.1	0.2	0.6	0.6	0.3
	17.0	10.0	18.9	5.7	4.3	8.5	3.9	1.1	0.8	1.6	1.2	1.6	0.4	1.1	1.5	0.6
							PLAN 52									
	14.0	6.0	12.9	5.2	3.1	6.3	2.2	0.7	1.1	0.9	1.0	1.4	0.2	0.6	0.6	0.3
	17.0	10.0	14.1	4.2	4.1	8.1	3.3	1.0	0.9	1.7	1.0	1.9	0.5	1.1	1.6	0.7
							PLAN 53									
		10.0	13.0	4.1	4.3	11.0	4.3	1.0	0.8	2.2	1.0	2.7	0.5	1.5	1.9	0.8
							PLAN 54									
		10.0	14.9	4.6	4.0	9.6	2.8	1.2	0.7	2.1	0.6	2.3	0.6	1.4	1.7	0.8
							PLAN 55									
		10.0	16.4	5.6	4.6	10.8	1.5	1.6	1.4	1.2	0.7	3.1	0.6	1.6	2.1	1.3
							PLAN 56									
		10.0	17.6	5.5	4.4	10.4	4.2	1.5	0.8	1.0	0.9	2.9	0.6	1.6	2.3	1.2
							PLAN 57									
		10.0	17.4	4.8	4.5	11.2	4.0	2.3	1.5	1.6	1.0	3.0	0.6	1.8	2.2	1.2

(CONTINUED)

TABLE 19 (CONTINUED)

TEST WAVE			WAVE HEIGHT, FT													
DIRECTION	PERIOD SEC	HEIGHT FT	GAGE 1	GAGE 2	GAGE 3	GAGE 4	GAGE 5	GAGE 6	GAGE 7	GAGE 8	GAGE 9	GAGE 10	GAGE 11	GAGE 12	GAGE 13	GAGE 14
							PLAN 50*									
SW	7.0	4.0	0.2	0.2	0.4	1.0	0.6	0.5	0.1	0.3	0.2	0.3	<0.1	0.3	0.1	<0.1
		10.0	0.8	0.2	0.3	2.3	1.1	0.3	0.1	0.3	0.4	0.6	0.1	0.7	0.4	0.3
	9.0	16.0	0.6	0.5	0.5	2.7	1.1	0.9	0.6	0.5	1.0	0.8	0.2	0.3	0.3	0.2
	11.0	8.0	0.6	0.9	0.6	2.0	0.8	0.7	0.8	0.7	1.3	0.6	0.1	0.2	0.2	0.1
	14.0	6.0	1.1	0.6	1.1	3.1	1.4	0.9	1.2	0.8	0.8	0.6	0.1	0.4	0.3	0.2
	17.0	10.0	1.4	1.5	1.7	4.8	1.8	1.1	1.4	1.2	1.0	1.0	0.3	0.9	1.1	0.5
W	7.0	4.0	0.3	0.1	0.1	1.3	0.7	0.2	<0.1	0.2	0.2	0.3	<0.1	0.5	0.2	0.2
		10.0	0.8	0.1	0.4	1.7	1.1	0.6	0.1	0.2	0.6	0.5	<0.1	0.7	0.3	0.2
	9.0	12.0	0.3	0.3	0.3	3.0	0.9	0.6	0.3	0.2	0.7	0.7	<0.1	0.2	<0.1	<0.1
	11.0	10.0	0.8	0.4	0.5	3.0	1.0	1.0	0.9	0.7	1.8	0.9	0.1	0.2	0.2	0.1
	14.0	6.0	0.9	0.5	0.7	2.6	1.1	0.7	0.5	0.6	1.1	0.7	0.1	0.4	0.2	0.2
	17.0	6.0	0.7	0.6	1.1	2.6	1.1	0.8	0.7	1.2	0.8	0.9	0.2	0.5	0.7	0.2
S	7.0	4.0	0.5	0.3	0.2	1.6	0.8	0.5	0.2	0.3	0.3	0.4	<0.1	0.5	0.2	0.1
		10.0	1.3	0.4	0.7	5.1	1.9	0.6	0.3	0.8	0.9	1.0	0.2	1.1	0.7	0.3
	9.0	4.0	0.4	0.4	0.7	1.5	0.5	0.6	0.3	0.7	0.6	0.4	<0.1	0.1	0.1	0.1
		10.0	1.5	0.7	0.7	4.7	1.5	0.9	0.6	0.8	1.8	1.0	0.1	0.2	0.1	0.2
	14.0	6.0	0.7	0.6	0.6	2.8	0.5	0.7	0.4	0.9	1.0	0.4	0.1	0.2	0.1	0.1
	17.0	6.0	0.7	1.2	1.5	3.2	1.6	0.9	0.7	2.1	0.5	0.7	0.2	0.5	1.0	0.3

(CONTINUED)

* GAGES 1-3 MOVED INSIDE INNER BREAKWATER AS SHOWN IN PLATE 52.

TABLE 19 (CONCLUDED)

TEST WAVE			WAVE HEIGHT, FT													
DIRECTION	PERIOD SEC	HEIGHT FT	GAGE 1	GAGE 2	GAGE 3	GAGE 4	GAGE 5	GAGE 6	GAGE 7	GAGE 8	GAGE 9	GAGE 10	GAGE 11	GAGE 12	GAGE 13	GAGE 14
							PLAN 58*									
W	11.0	10.0	0.7	0.4	0.6	3.7	1.3	1.0	0.8	0.7	1.5	1.1	0.1	0.2	0.2	0.1
S	9.0	10.0	1.0	0.7	0.8	4.8	1.8	0.7	0.7	1.0	1.0	0.9	0.1	0.2	0.4	0.2
	17.0	6.0	1.0	1.4	1.7	3.9	1.9	1.0	0.8	2.5	0.5	1.1	0.2	0.8	1.0	0.3
		6.0	1.0	1.3	1.6	3.9	1.9	1.0	0.8	2.4	0.6	1.0	0.3	0.8	1.0	0.3
		6.0	1.0	1.5	1.8	3.5	1.7	1.1	0.6	2.3	0.6	0.9	0.2	0.6	1.1	0.3
							PLAN 59*									
		6.0	0.7	0.6	0.6	3.8	1.9	0.5	0.4	0.4	0.4	0.9	0.2	0.6	1.0	0.3
							PLAN 60*									
		6.0	1.2	0.5	0.7	4.3	1.8	0.5	0.5	1.2	0.6	0.9	0.2	0.7	1.1	0.3

* GAGES 1-3 MOVED INSIDE INNER BREAKWATER AS SHOWN IN PLATE 52.

TABLE 20

WAVE HEIGHTS FOR EXISTING CONDITIONS

AT THE BEACH

SWL = +5.4 FT

TEST WAVE			WAVE HEIGHT, FT									
DIRECTION	PERIOD SEC	HEIGHT FT	GAGE 1	GAGE 2	GAGE 3	GAGE 4	GAGE 5	GAGE 6	GAGE 7	GAGE 8	GAGE 9	GAGE 10
W	7.0	4.0	4.9	4.4	3.9	4.7	4.2	4.3	4.1	3.8	3.2	4.2
		10.0	13.3	12.0	11.4	12.2	10.9	10.9	12.8	9.5	13.0	11.4
	9.0	4.0	7.7	5.9	5.3	7.0	7.3	4.5	5.5	4.2	6.4	5.8
		12.0	15.2	15.6	10.9	9.2	8.7	14.5	17.0	7.6	11.4	11.3
	11.0	4.0	7.2	5.7	6.8	7.3	7.5	3.5	7.2	6.6	6.3	6.5
		10.0	20.0	16.8	14.6	10.8	9.7	11.6	16.6	9.5	11.2	13.0
	14.0	6.0	12.6	13.7	12.1	12.2	14.2	5.6	14.4	14.8	12.8	12.7
	17.0	6.0	11.5	13.0	11.6	11.8	13.4	3.7	12.6	12.4	14.3	10.7
S	7.0	4.0	3.5	3.8	3.8	3.7	4.3	3.6	3.7	3.6	3.7	2.9
		10.0	12.1	12.4	12.4	12.0	9.9	12.6	11.4	10.1	10.7	11.1
	9.0	4.0	4.8	5.6	5.6	4.8	6.6	6.2	5.3	6.2	5.2	4.4
		10.0	13.2	15.6	12.1	11.2	10.2	14.9	14.1	9.3	11.2	14.7
	14.0	6.0	9.4	10.2	13.5	15.3	13.7	11.8	9.5	14.2	15.1	10.5
	17.0	6.0	7.1	10.4	11.4	13.0	12.4	10.8	8.9	11.5	14.3	13.3
	19.0	6.0	9.0	7.9	13.1	12.3	12.8	9.6	10.8	12.9	15.6	9.0

Photo 1. Typical wave and current patterns and current magnitudes (prototype feet per second) for existing conditions; 7-sec, 10-ft waves from northwest at mllw

Photo 2. Typical wave and current patterns and current magnitudes (prototype feet per second) for existing conditions; 7-sec, 10-ft waves from west at mhhw

Photo 3. Typical wave and current patterns and current magnitudes (prototype feet per second) for existing conditions; 11-sec, 8-ft waves from southwest at mllw

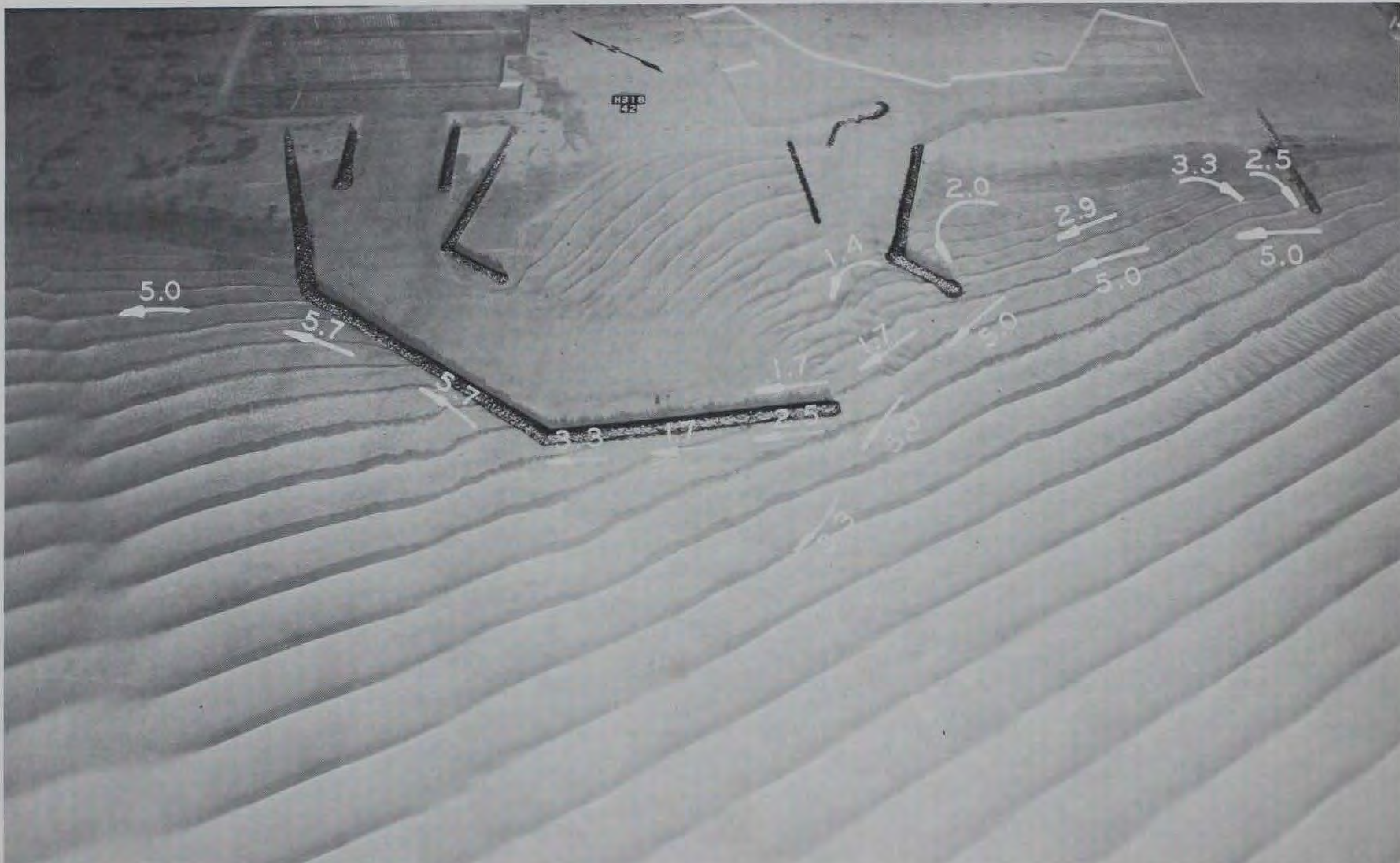


Photo 4. Typical wave and current patterns and current magnitudes (prototype feet per second) for existing conditions; 9-sec, 10-ft waves from south at mllw

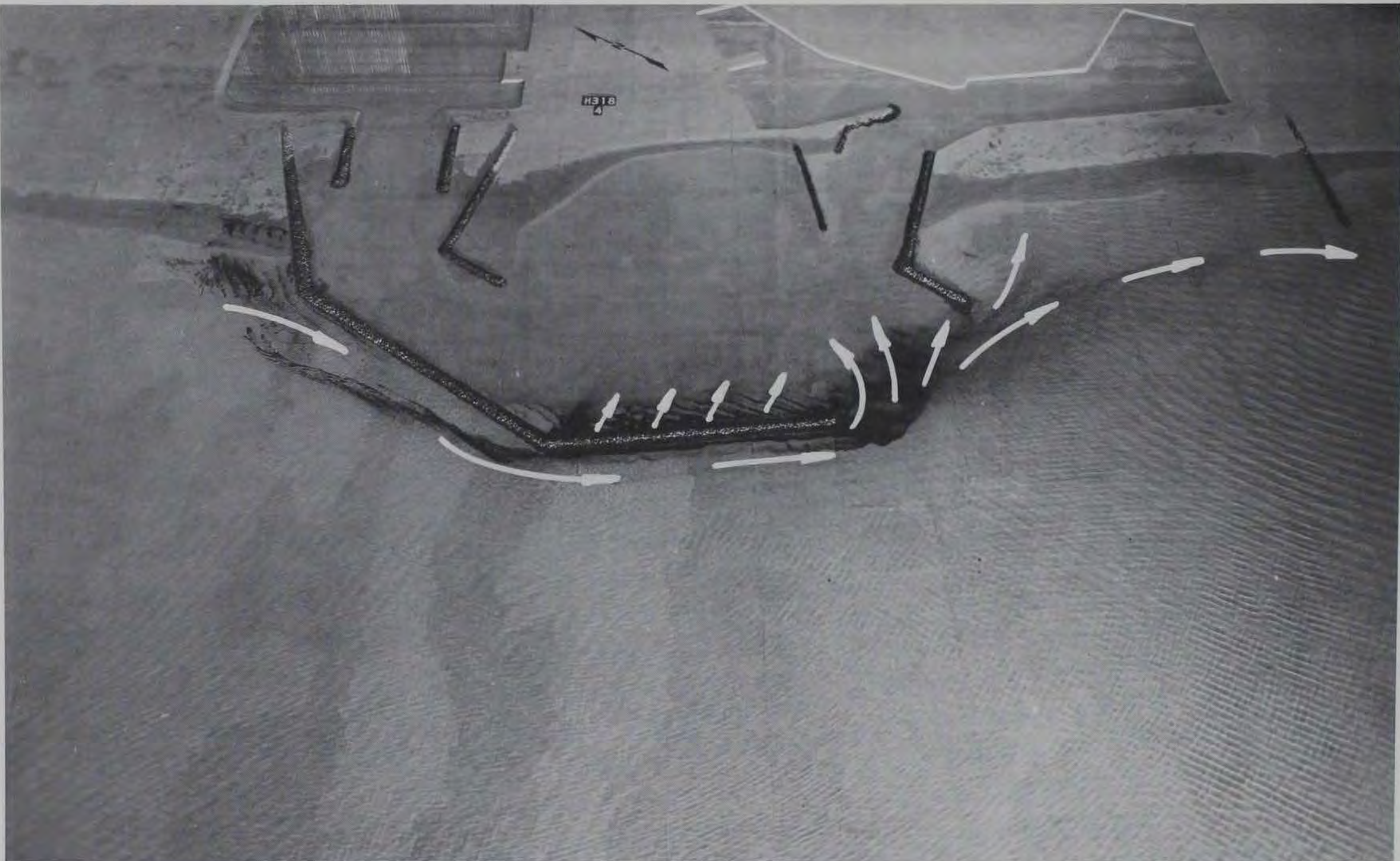


Photo 5. Typical tracer movement for existing conditions resulting from 7-sec, 10-ft waves from northwest at mhhw

Photo 6. Typical tracer movement for existing conditions resulting from 7-sec, 10-ft waves from northwest at mllw

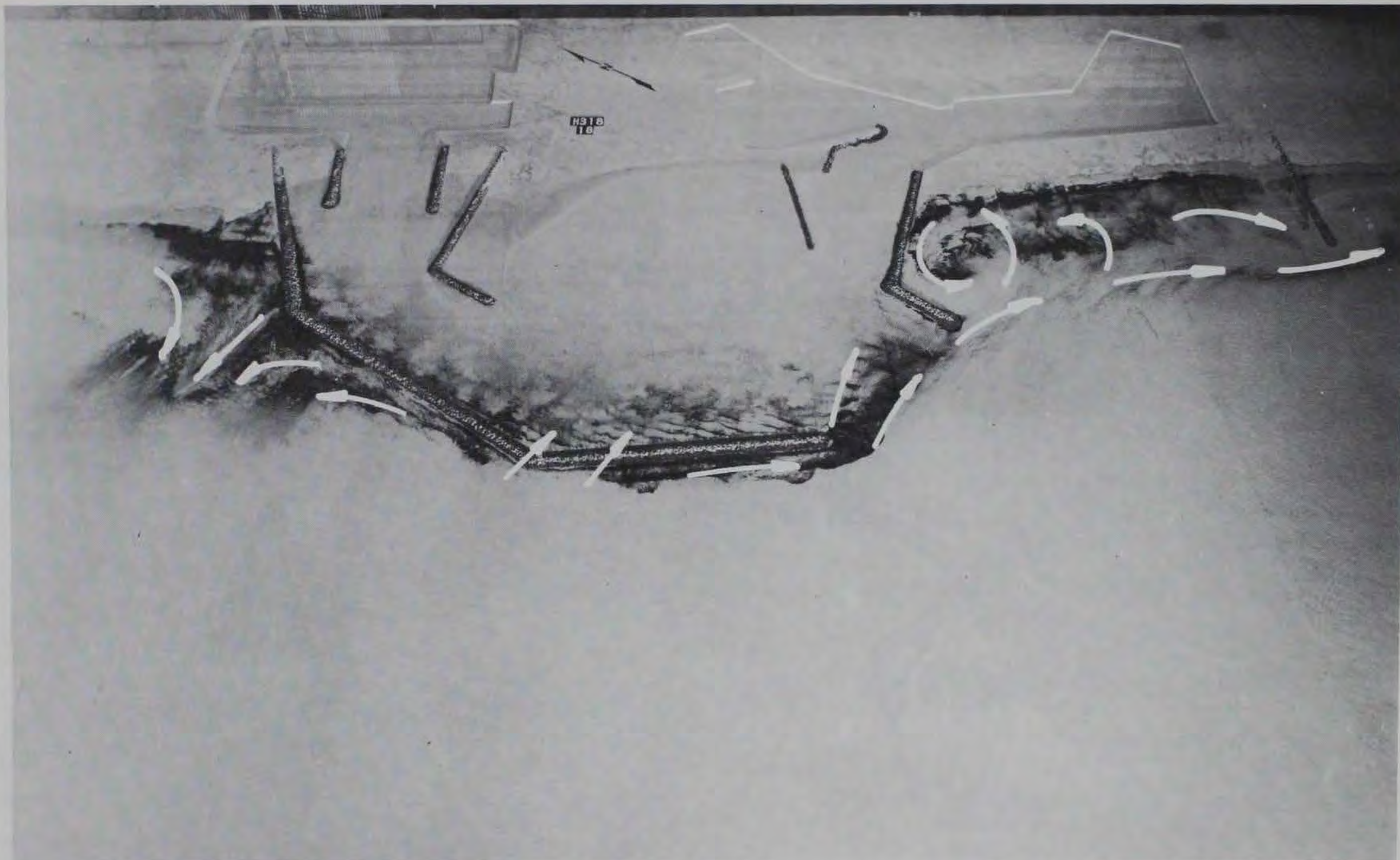


Photo 7. Typical tracer movement for existing conditions resulting from 7-sec, 10-ft waves from west at mhhw

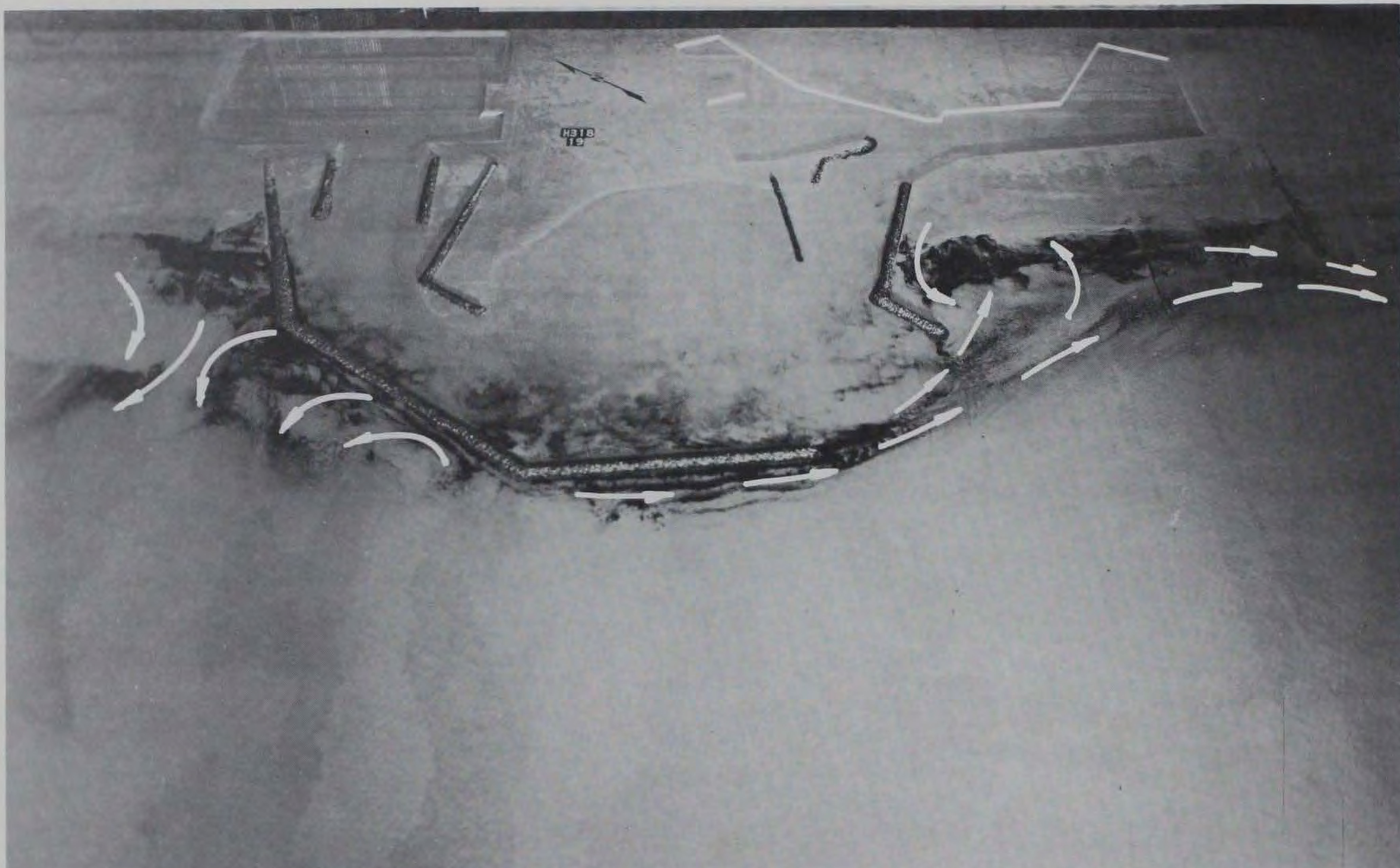


Photo 8. Typical tracer movement for existing conditions resulting from 7-sec, 10-ft waves from west at mllw

Photo 9. Typical tracer movement for existing conditions resulting from 11-sec, 8-ft waves from southwest at mllw

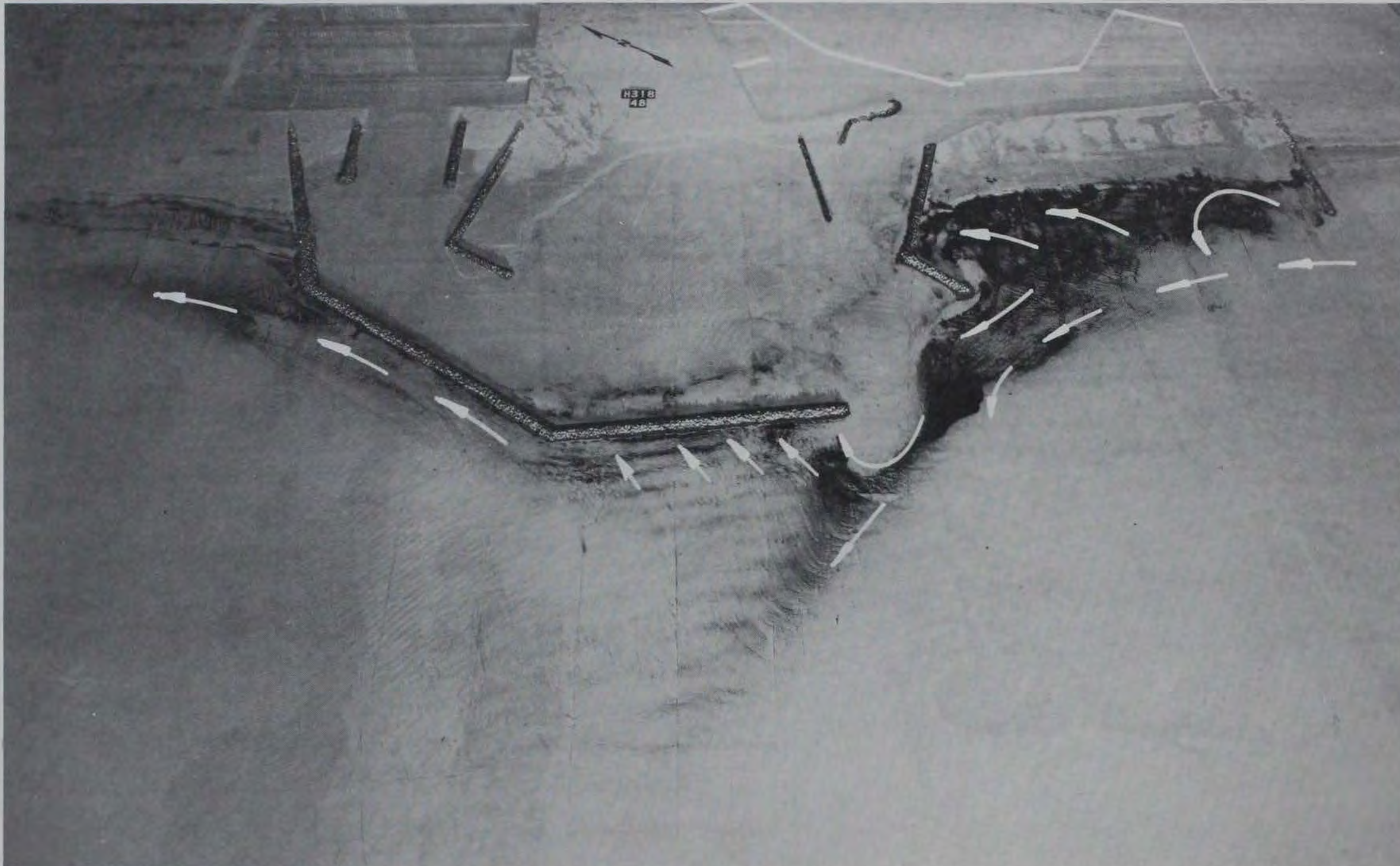


Photo 10. Typical tracer movement for existing conditions resulting from 9-sec, 10-ft waves from south at mllw

Photo 11. Typical wave and current patterns and current magnitudes (prototype feet per second) for Plan 4; 7-sec, 10-ft waves from northwest at mllw

Photo 12. Typical wave and current patterns and current magnitudes (prototype feet per second) for Plan 4; 11-sec, 10-ft waves from west at mllw

Photo 13. Typical wave and current patterns and current magnitudes (prototype feet per second) for Plan 4; 9-sec, 16-ft waves from southwest at mllw

Photo 14. Typical wave and current patterns and current magnitudes (prototype feet per second) for Plan 4; 9-sec, 10-ft waves from south at mllw

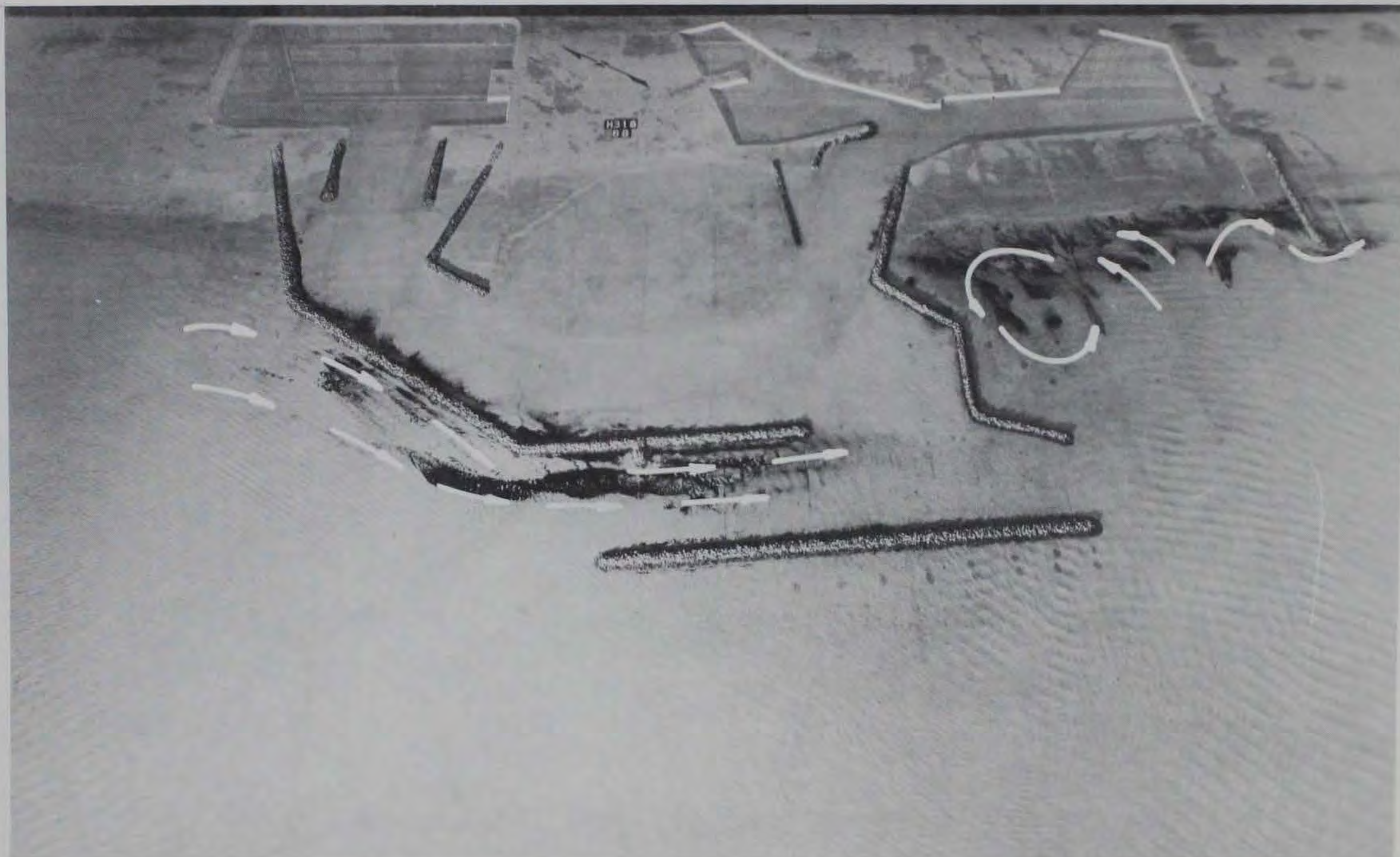


Photo 15. Typical tracer movement for Plan 4 resulting from 7-sec, 10-ft waves from northwest at mllw

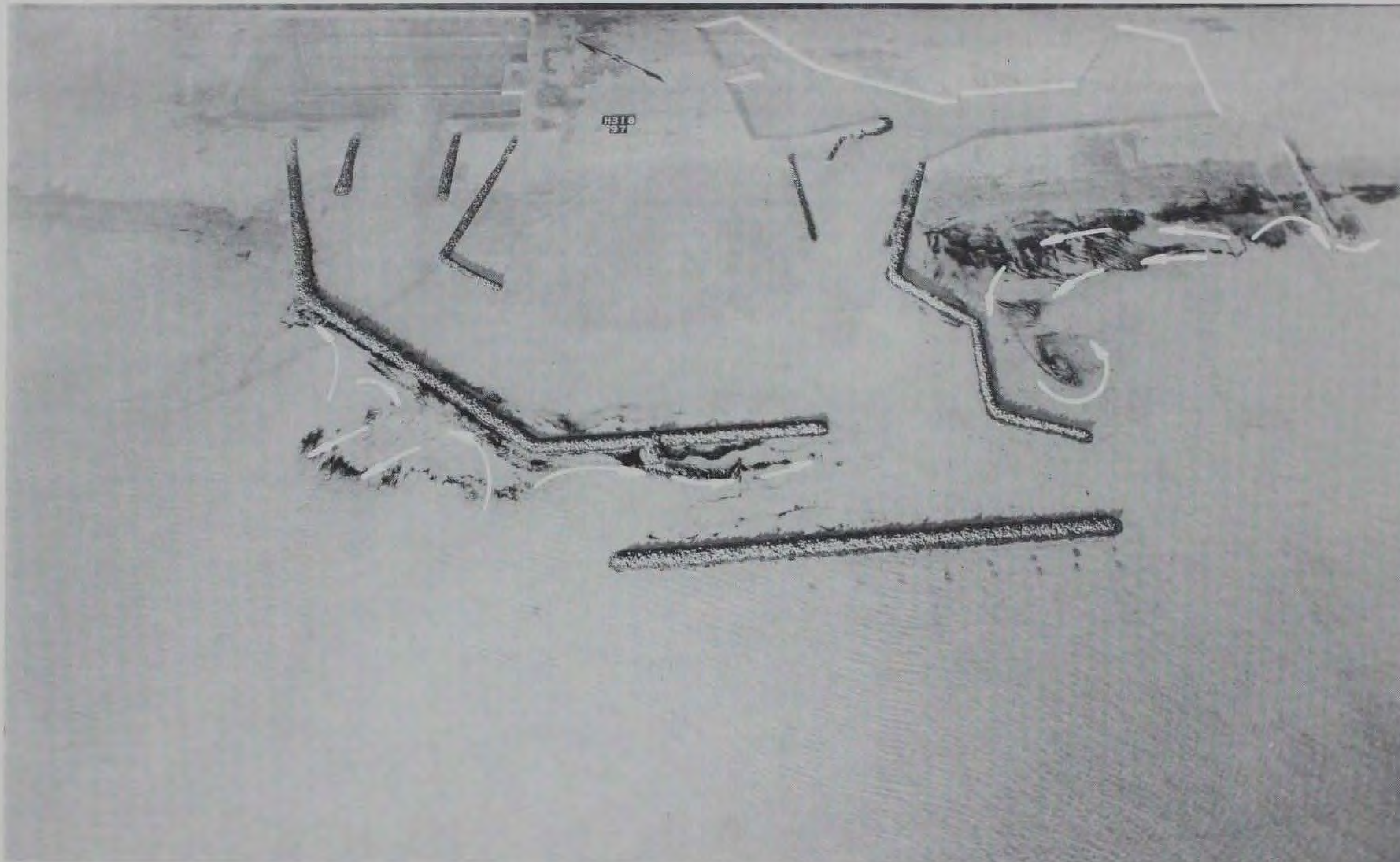


Photo 16. Typical tracer movement for Plan 4 resulting from 11-sec, 10-ft waves from west at mllw

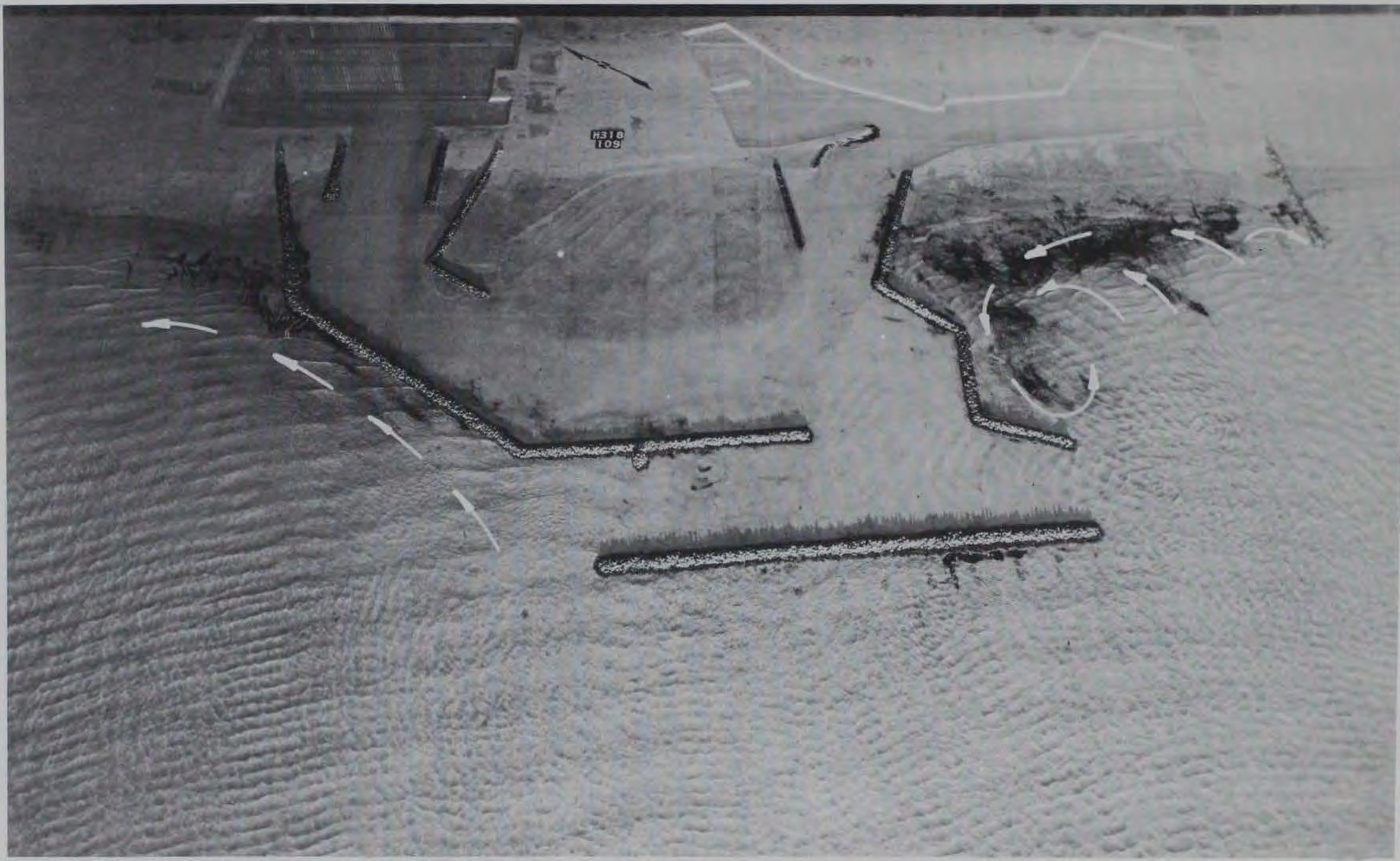


Photo 17. Typical tracer movement for Plan 4 resulting from 9-sec, 16-ft waves from southwest at mllw

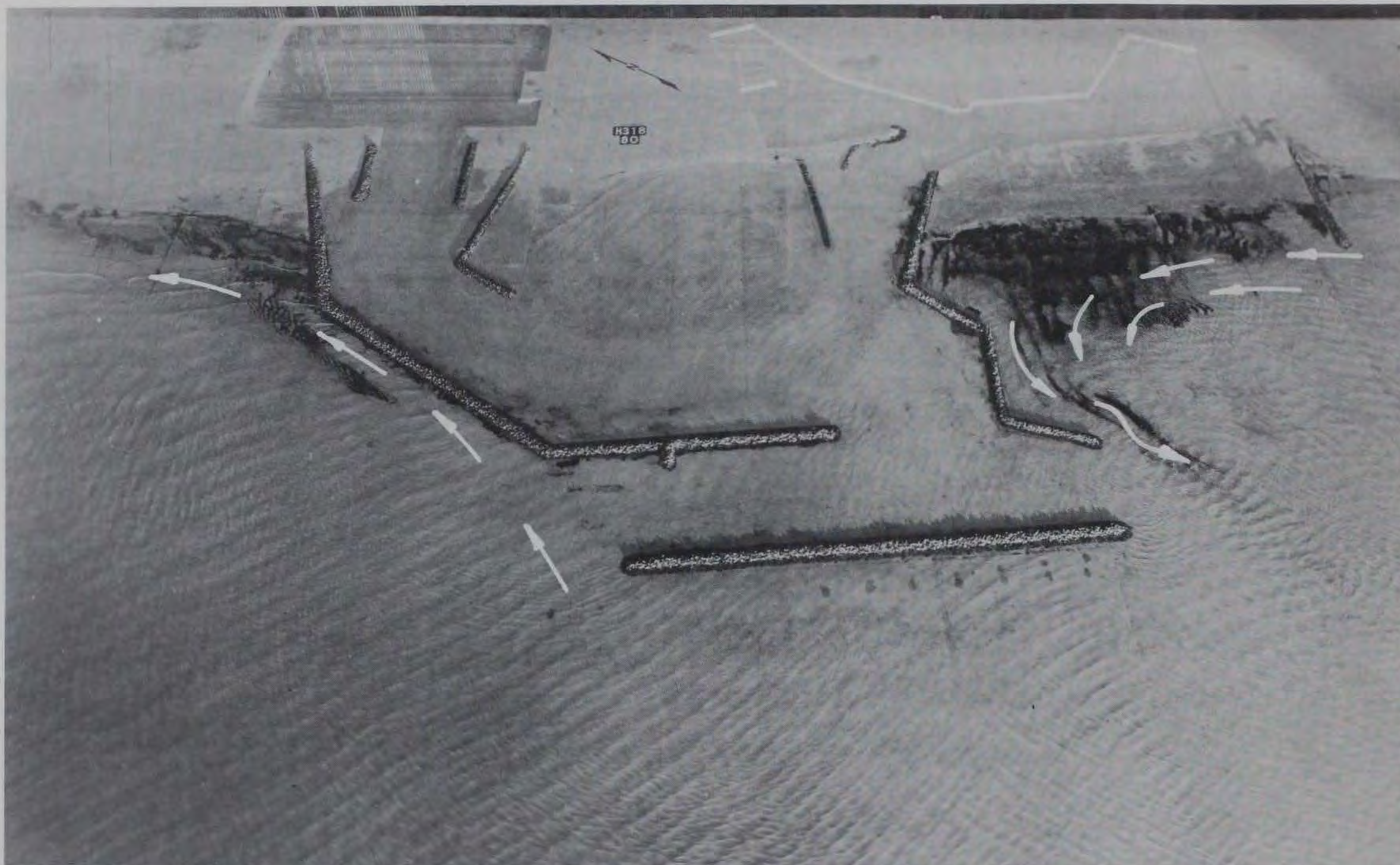


Photo 18. Typical tracer movement for Plan 4 resulting from 9-sec, 10-ft waves from south at mllw

Photo 19. Typical tracer movement for Plan 19 resulting from 9-sec, 10-ft waves from south at mllw

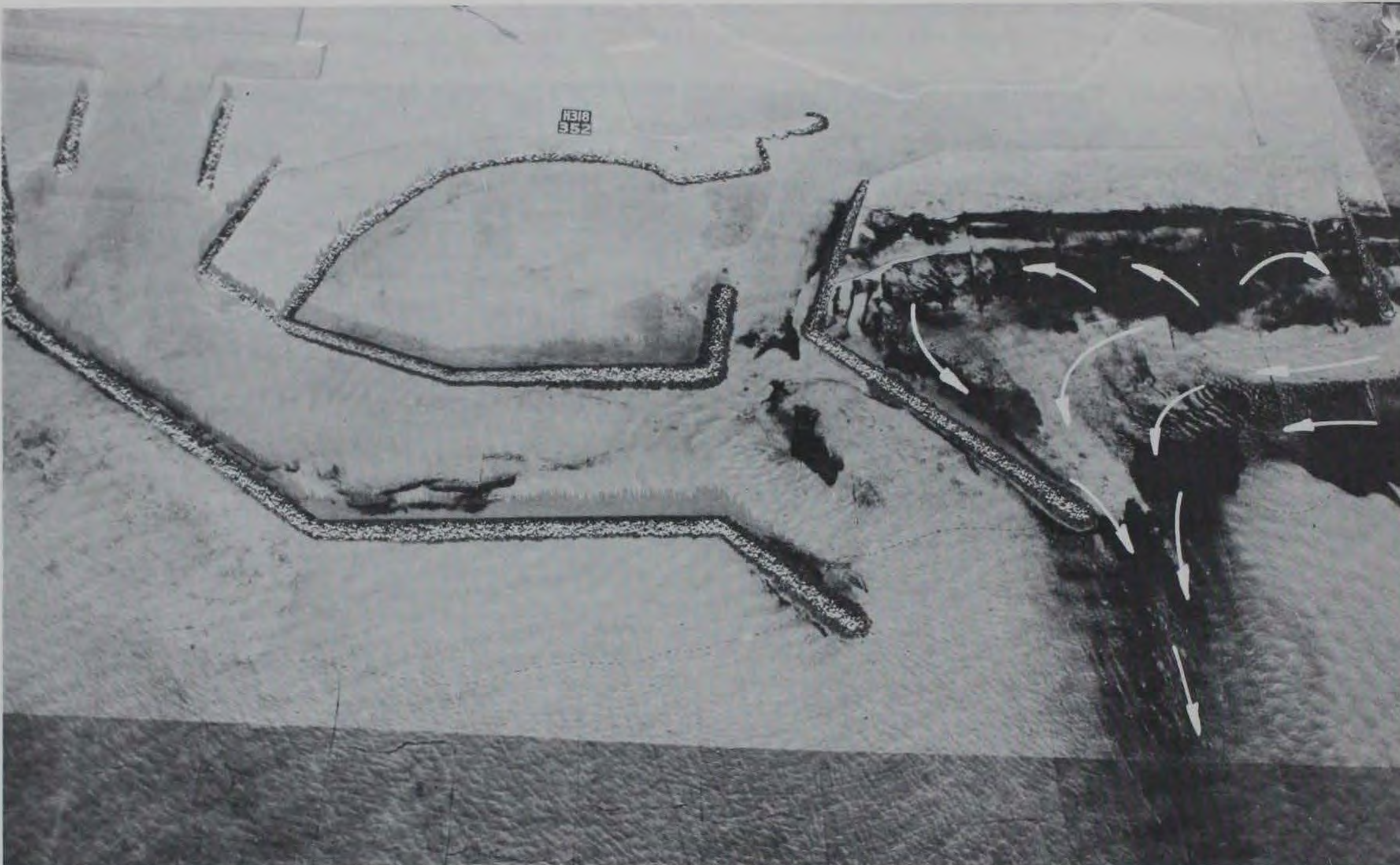


Photo 20. Typical tracer movement for Plan 20 resulting from 9-sec, 10-ft waves from south at mllw

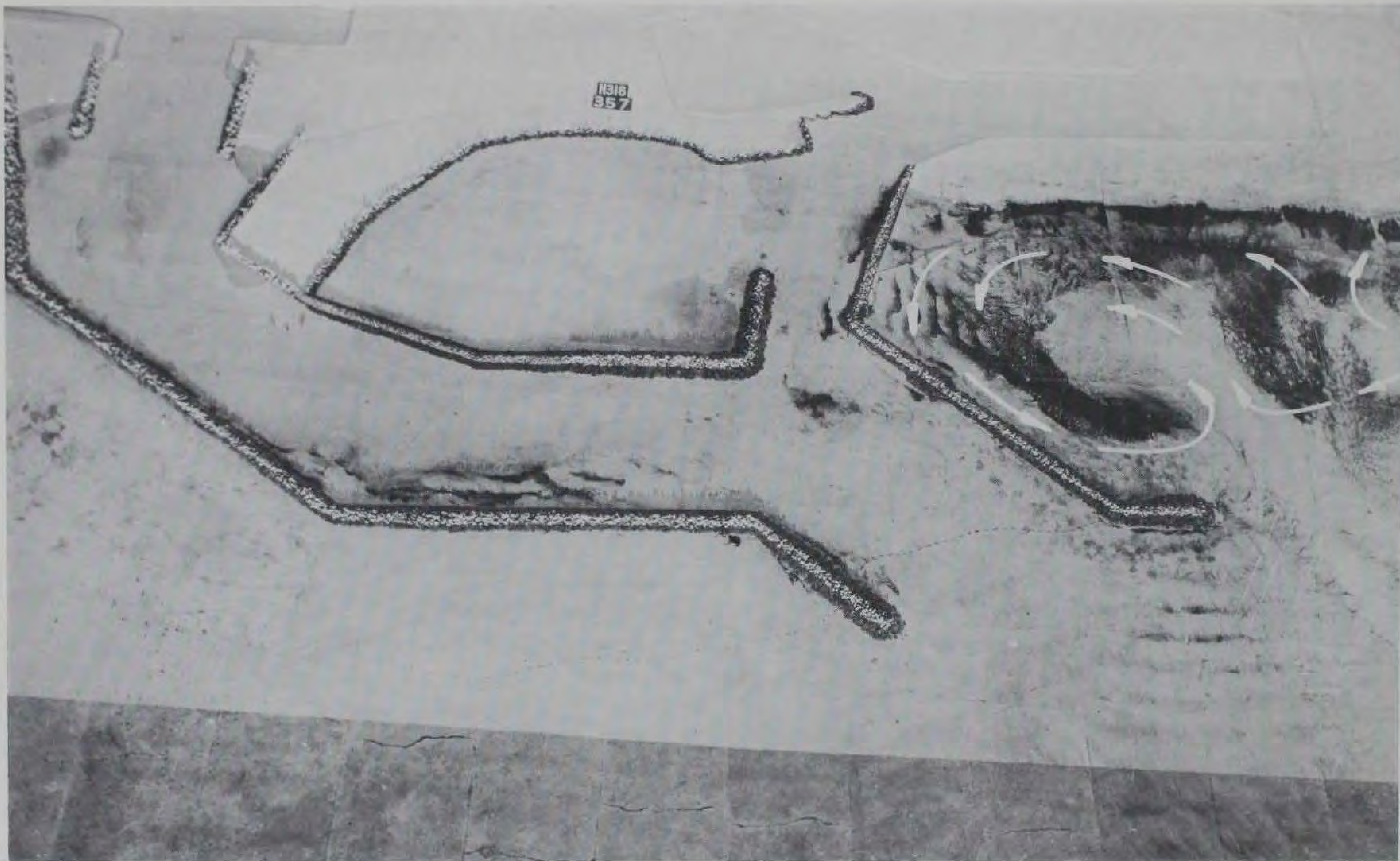


Photo 21. Typical tracer movement for Plan 21 resulting from 9-sec, 10-ft waves from south at mhhw

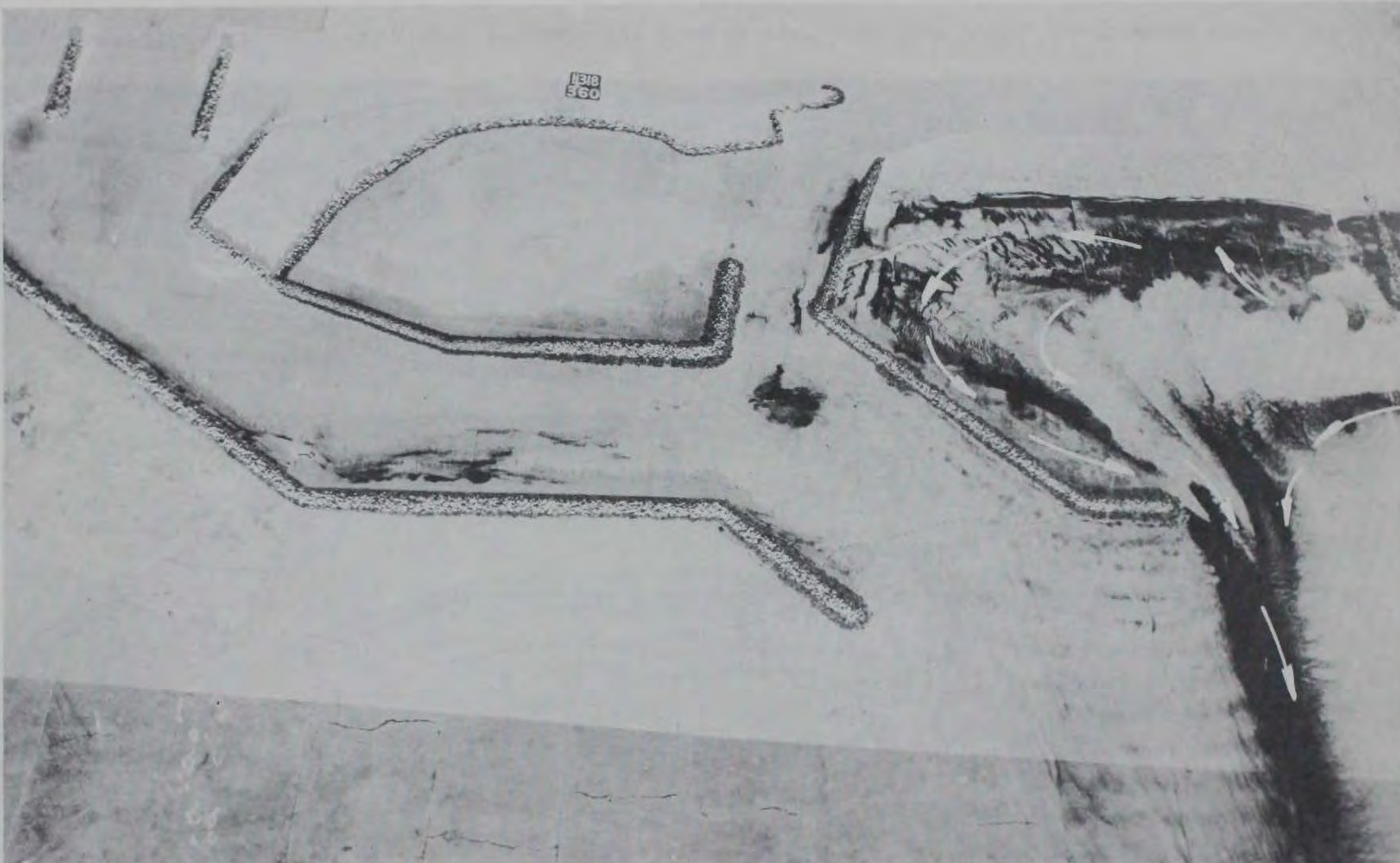


Photo 22. Typical tracer movement for Plan 21 resulting from 9-sec, 10-ft waves from south at mllw

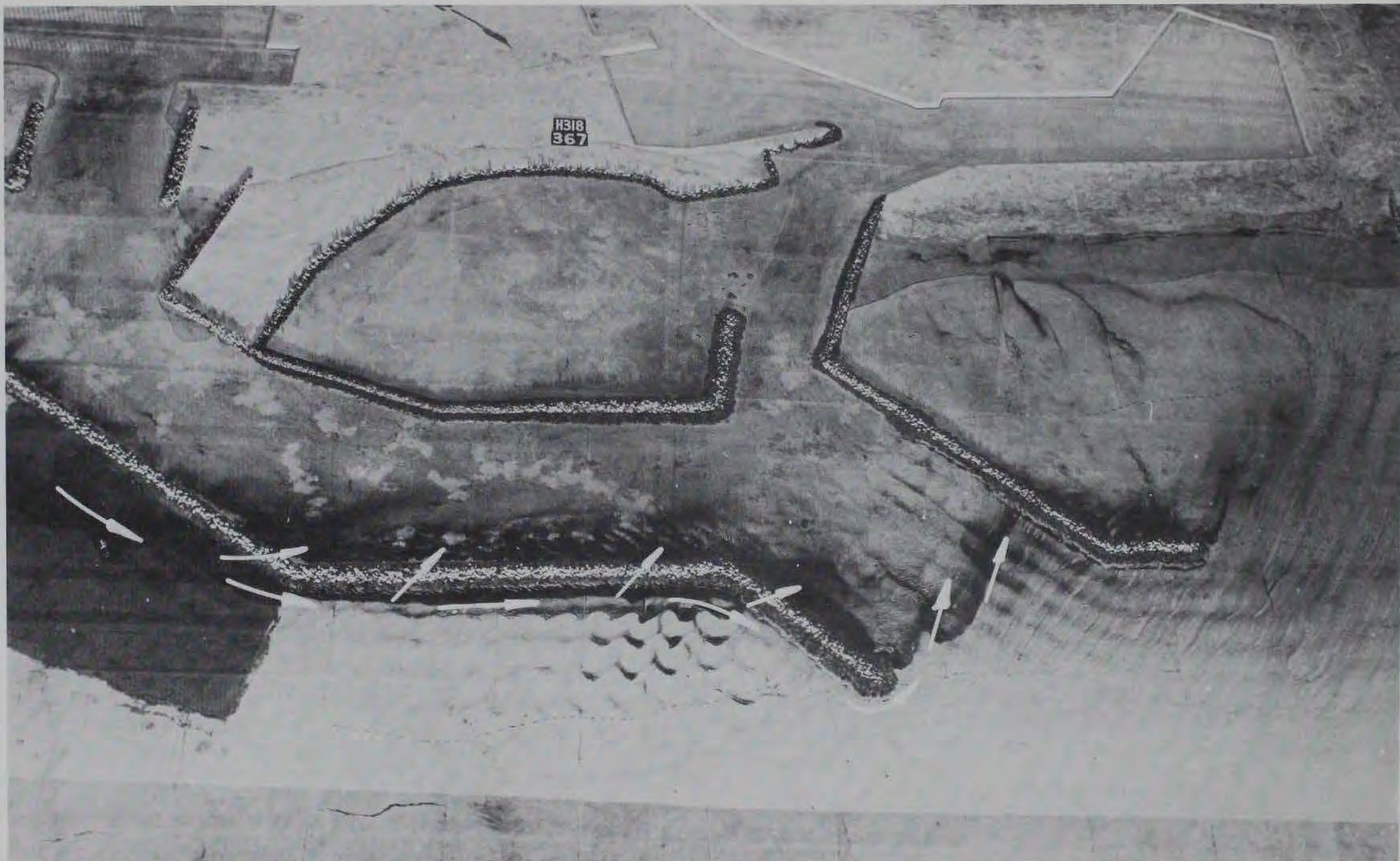


Photo 23. Typical tracer movement for Plan 21 resulting from 7-sec, 10-ft waves from northwest at mhhw

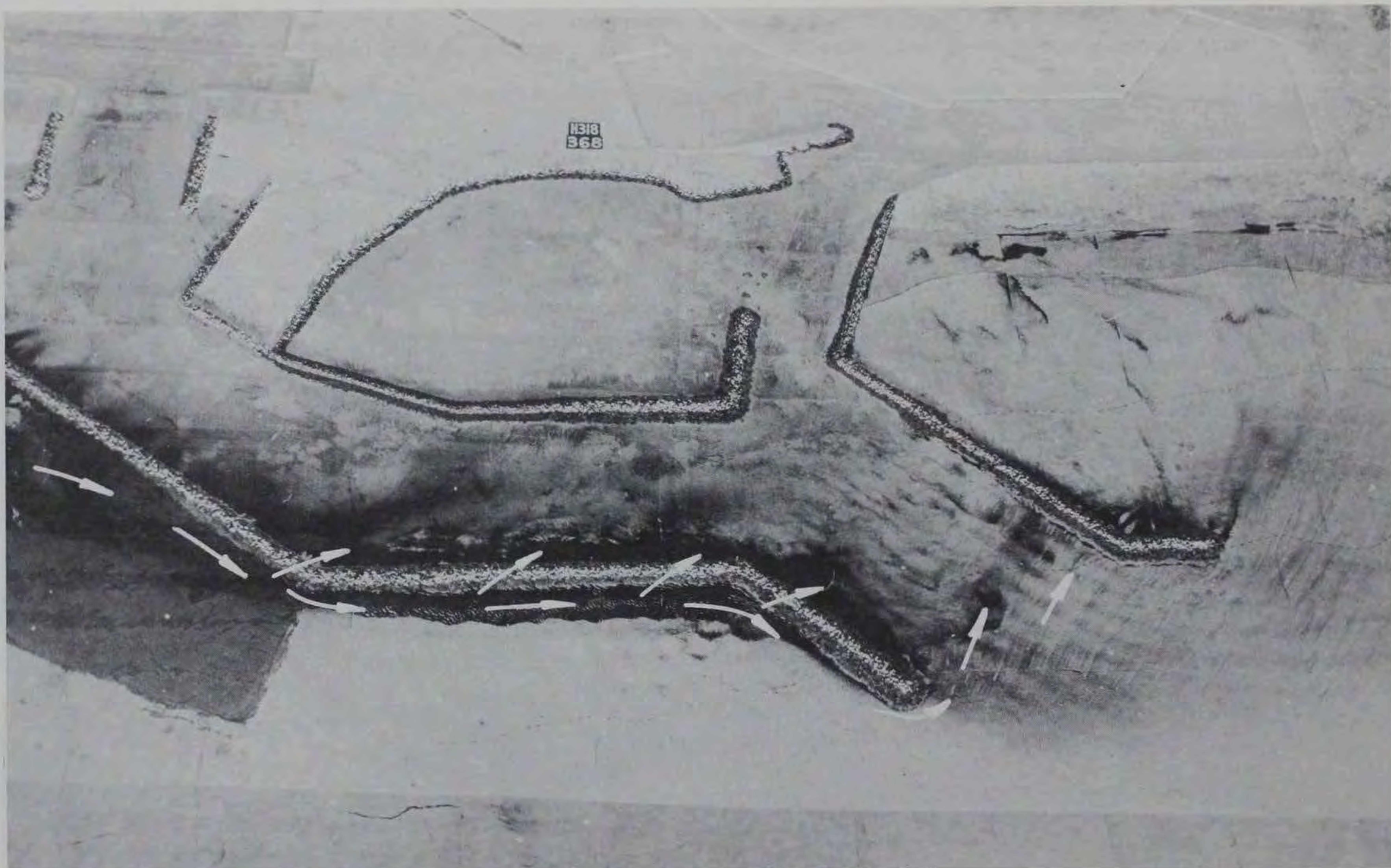


Photo 24. Typical tracer movement for Plan 21A resulting from 7-sec, 10-ft waves from northwest at mhhw

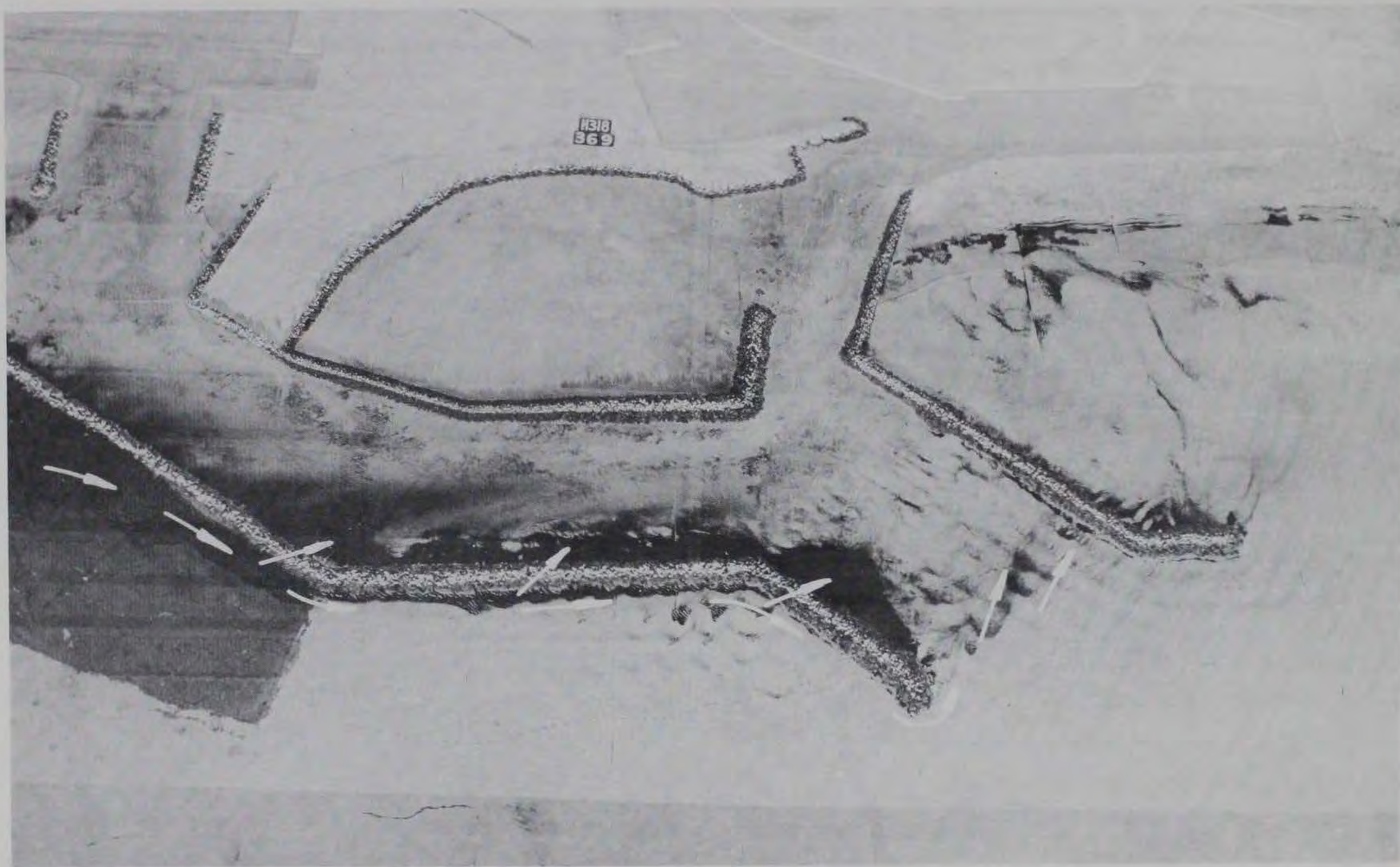


Photo 25. Typical tracer movement for Plan 22 resulting from 7-sec, 10-ft waves from northwest at mhhw

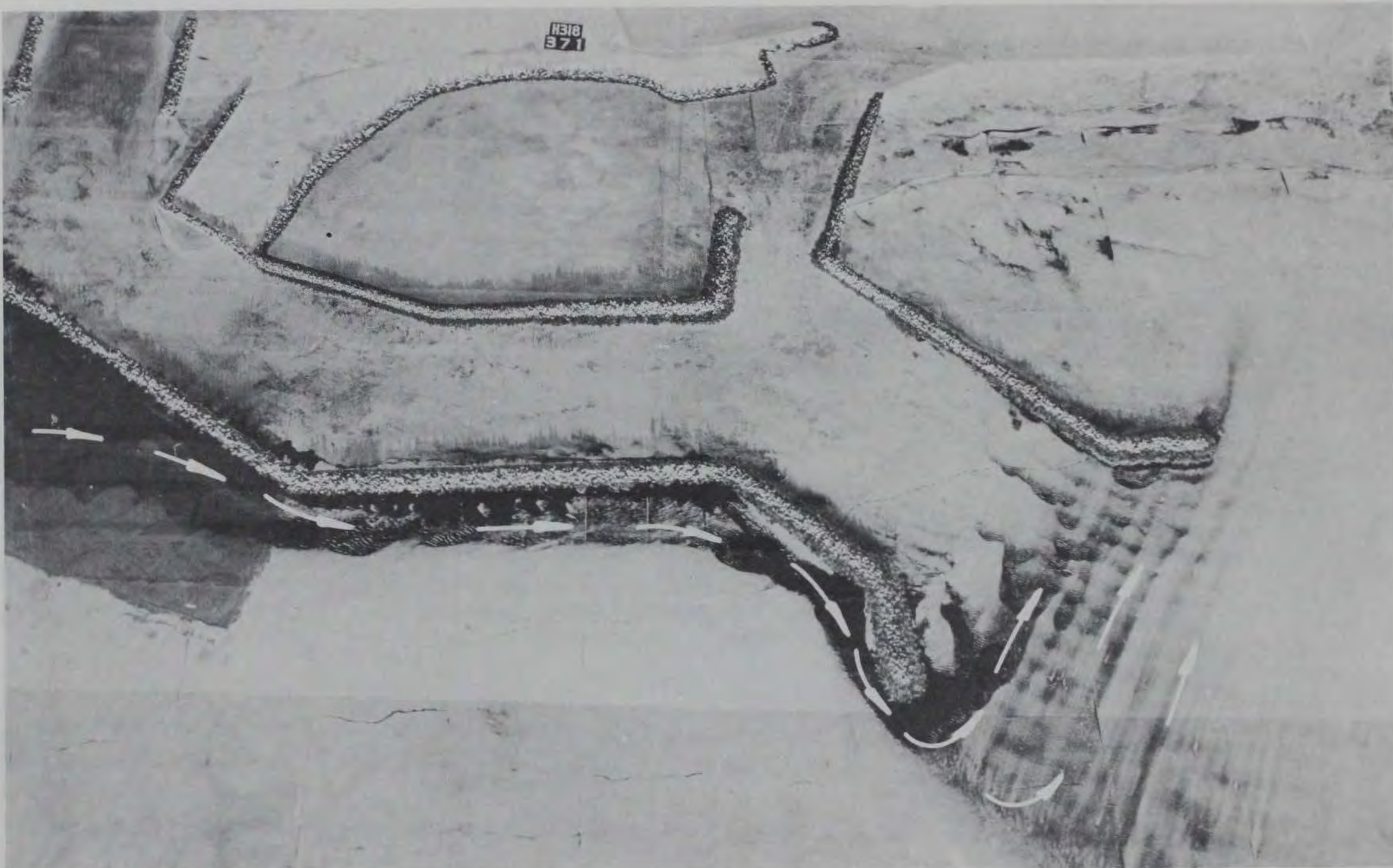


Photo 26. Typical tracer movement for Plan 22A resulting from 7-sec, 10-ft waves from northwest at mllw

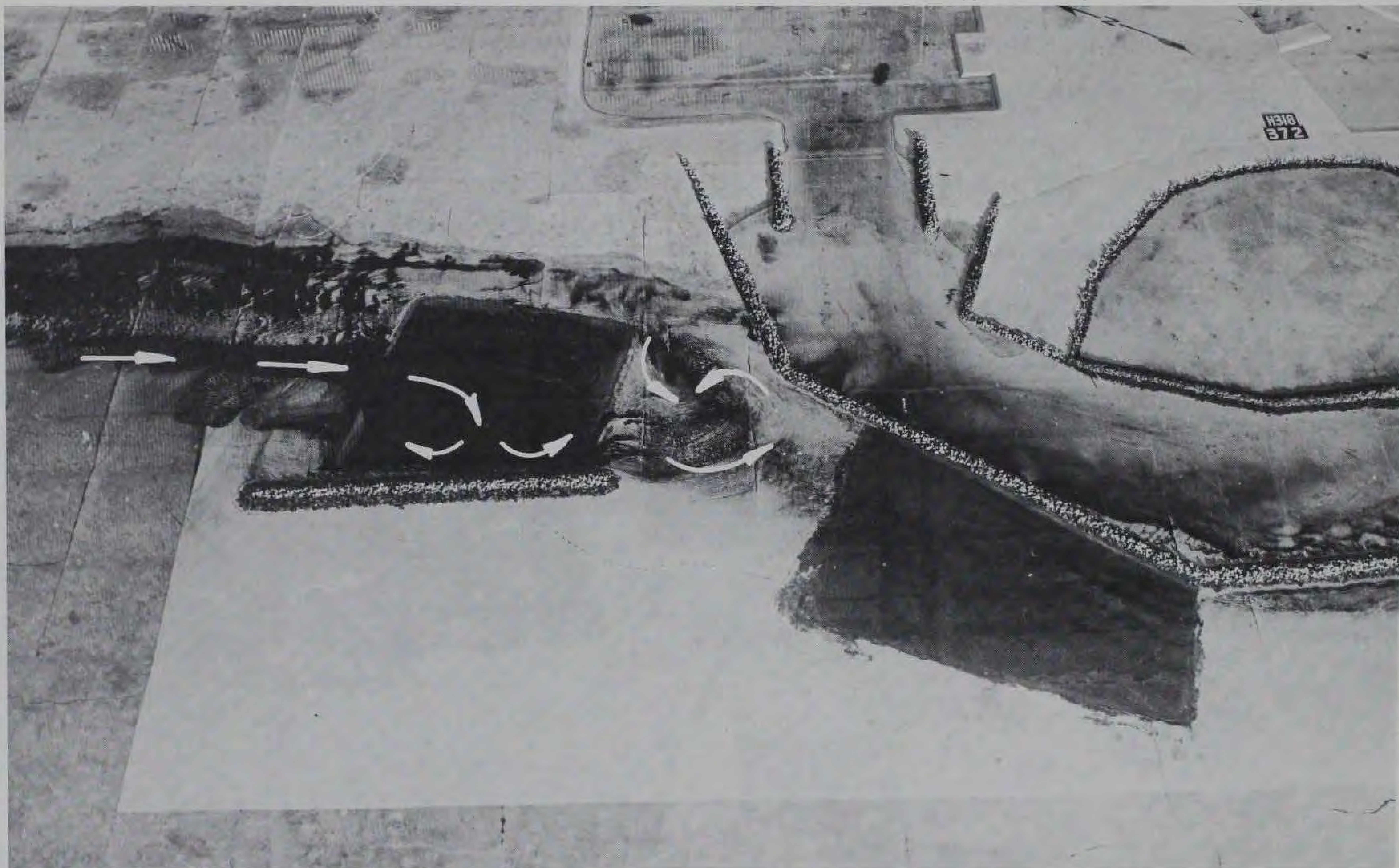


Photo 27. Typical tracer movement for Plan 23 resulting from 7-sec, 10-ft waves from northwest at mllw

Photo 28. Typical tracer movement for Plan 23 resulting from 7-sec, 10-ft waves from northwest at mhhw

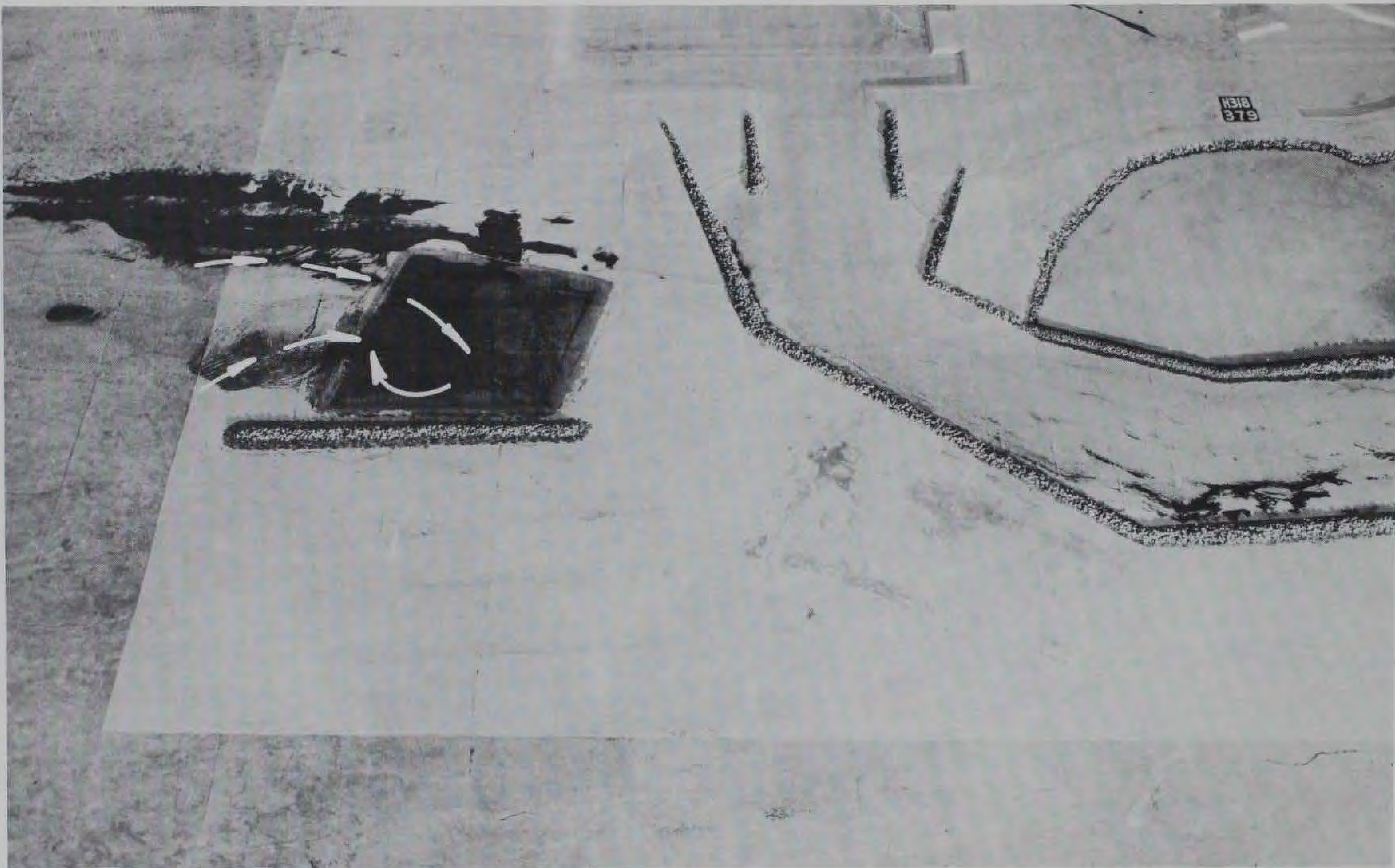


Photo 29. Typical tracer movement for Plan 23 resulting from 17-sec, 6-ft waves from west at mllw

Photo 30. Typical tracer movement for Plan 24 resulting from 17-sec, 6-ft waves from west at mllw

Photo 31. Typical tracer movement for Plan 24 resulting from 7-sec, 10-ft waves from northwest at mllw

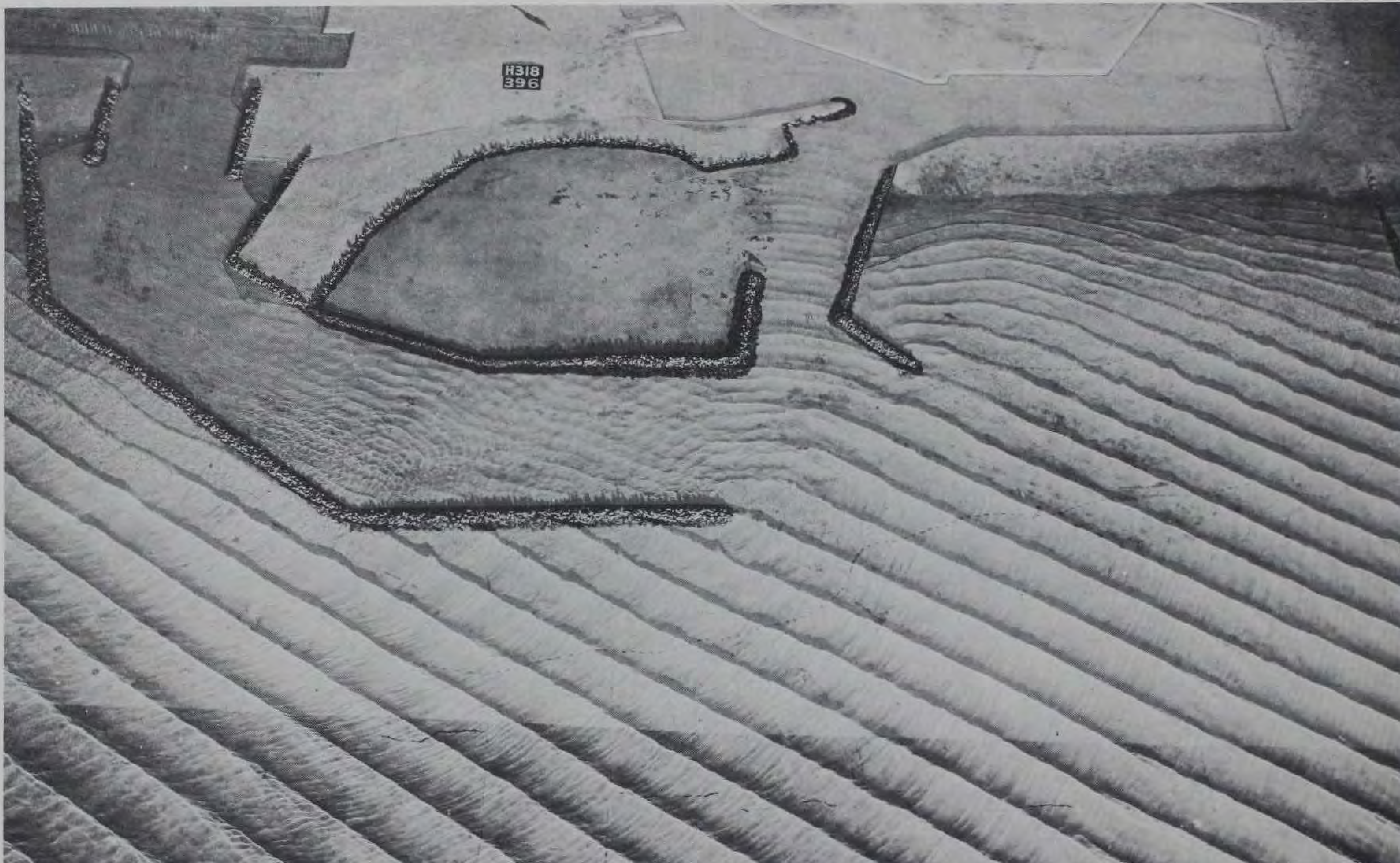


Photo 32. Typical wave patterns for Plan 25; 7-sec, 10-ft waves from northwest at mhhw

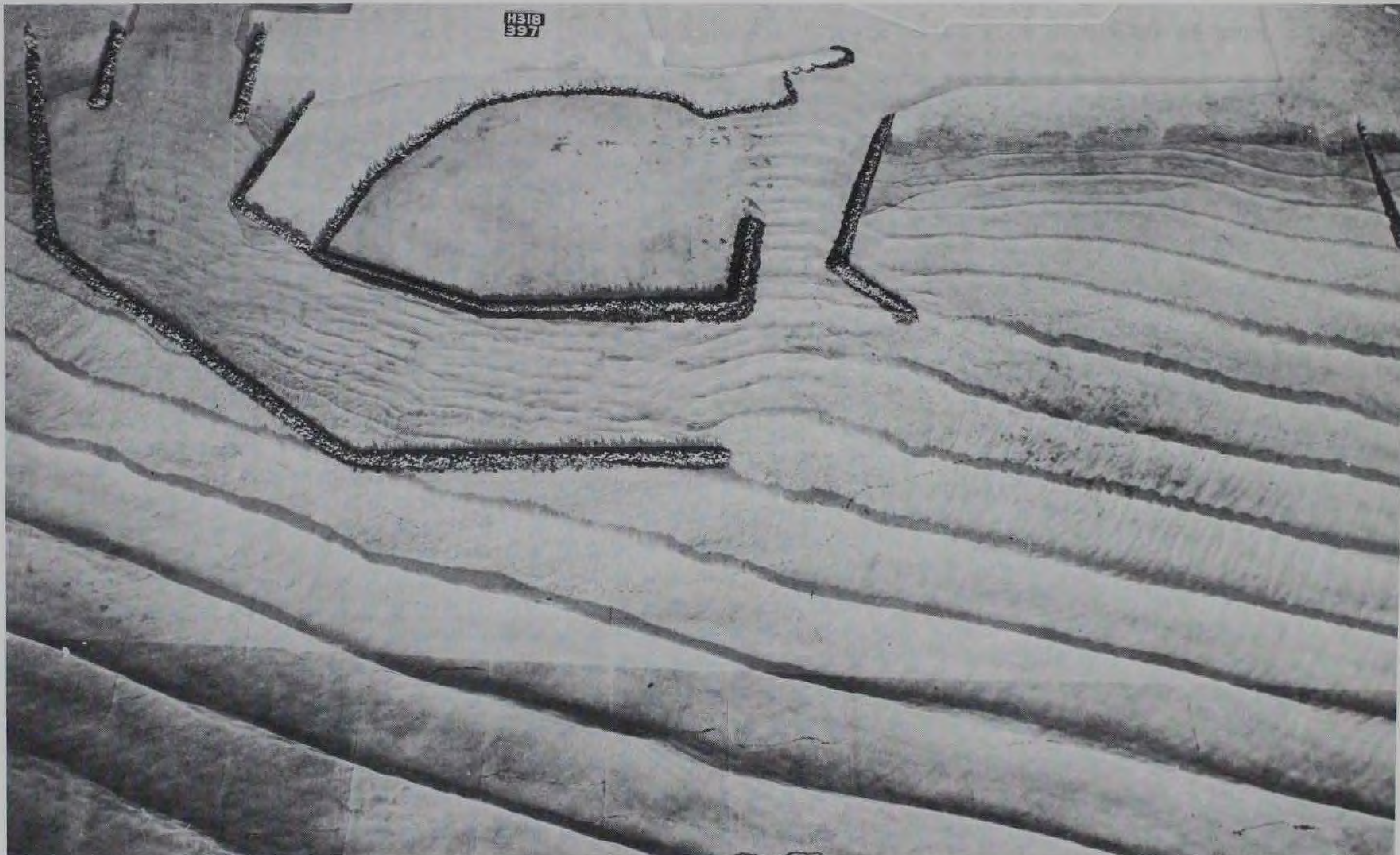


Photo 33. Typical wave patterns for Plan 25; 11-sec, 10-ft waves from west at mhhw

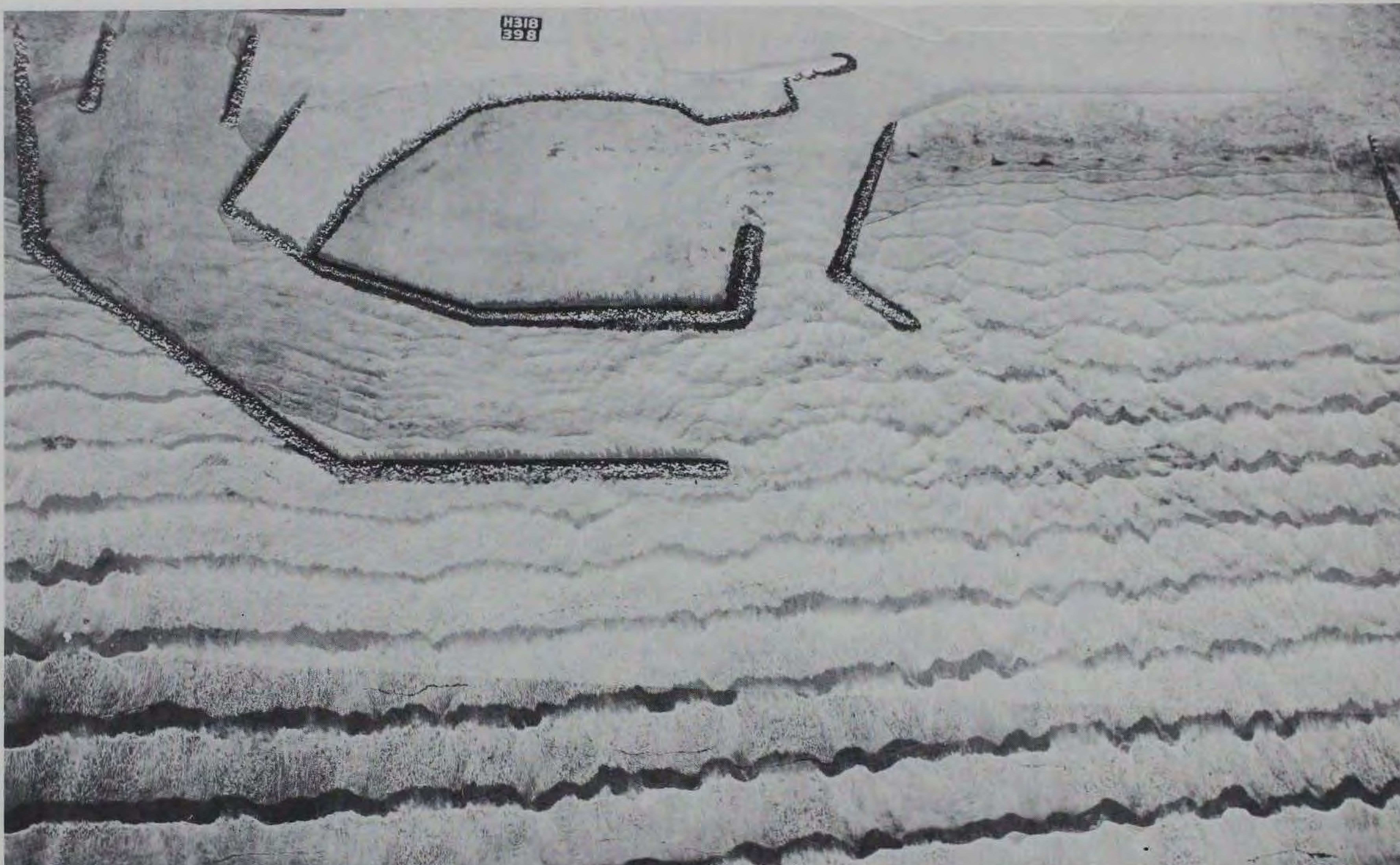


Photo 34. Typical wave patterns for Plan 25; 9-sec, 10-ft waves from southwest at mhhw

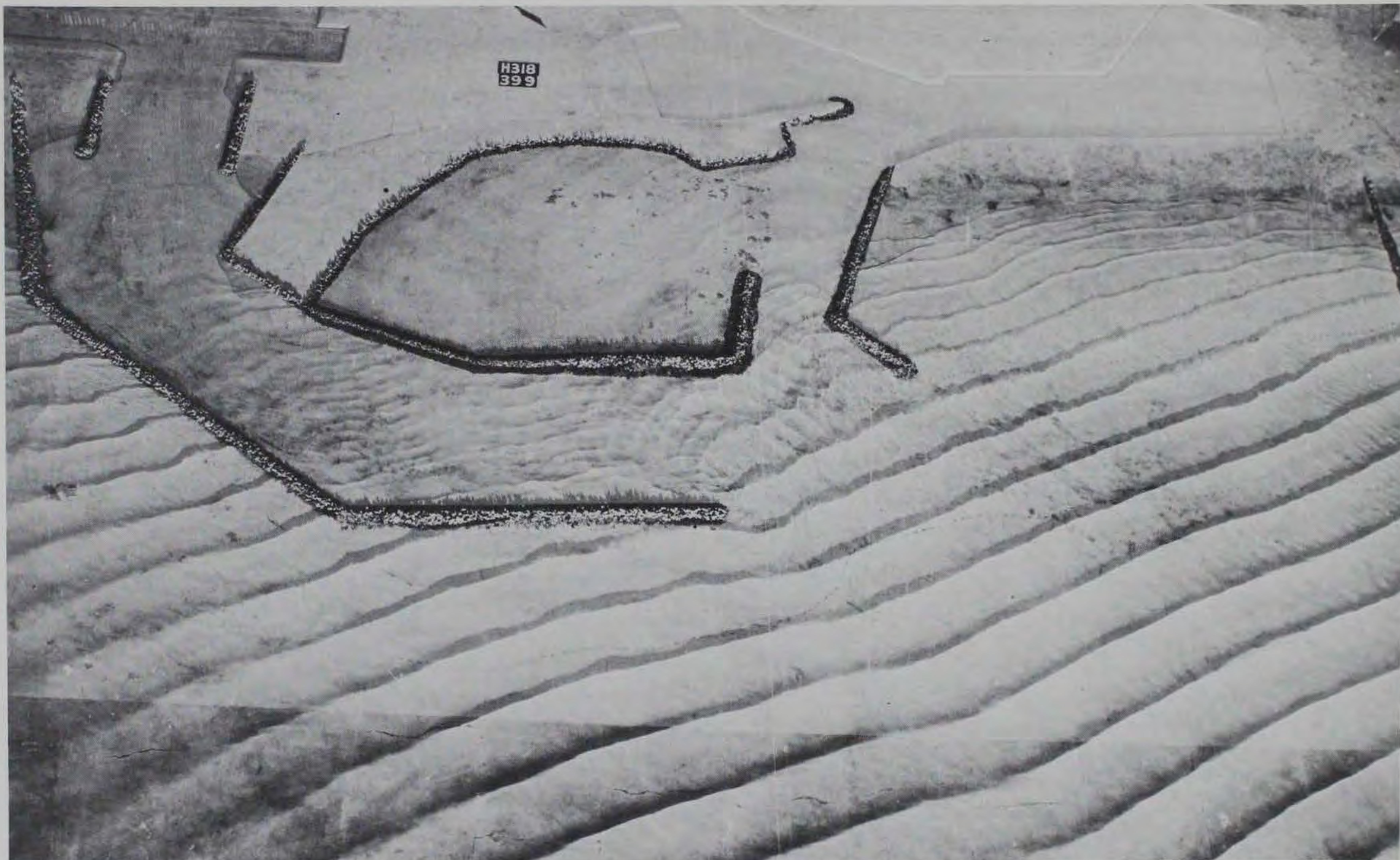


Photo 35. Typical wave patterns for Plan 25; 9-sec, 10-ft waves from south at mhhw

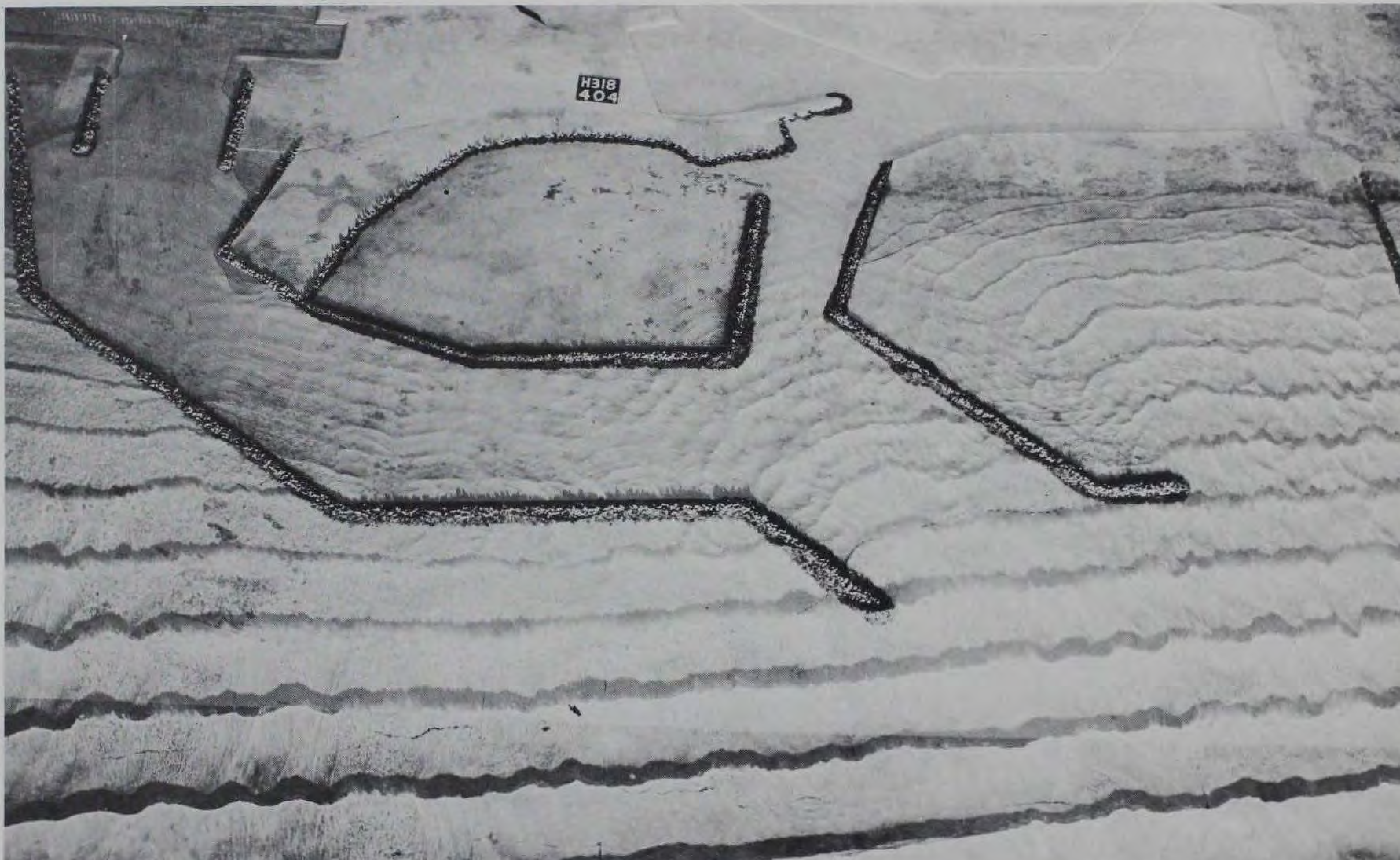


Photo 36. Typical wave patterns for Plan 21F; 9-sec, 16-ft waves from southwest at mhhw

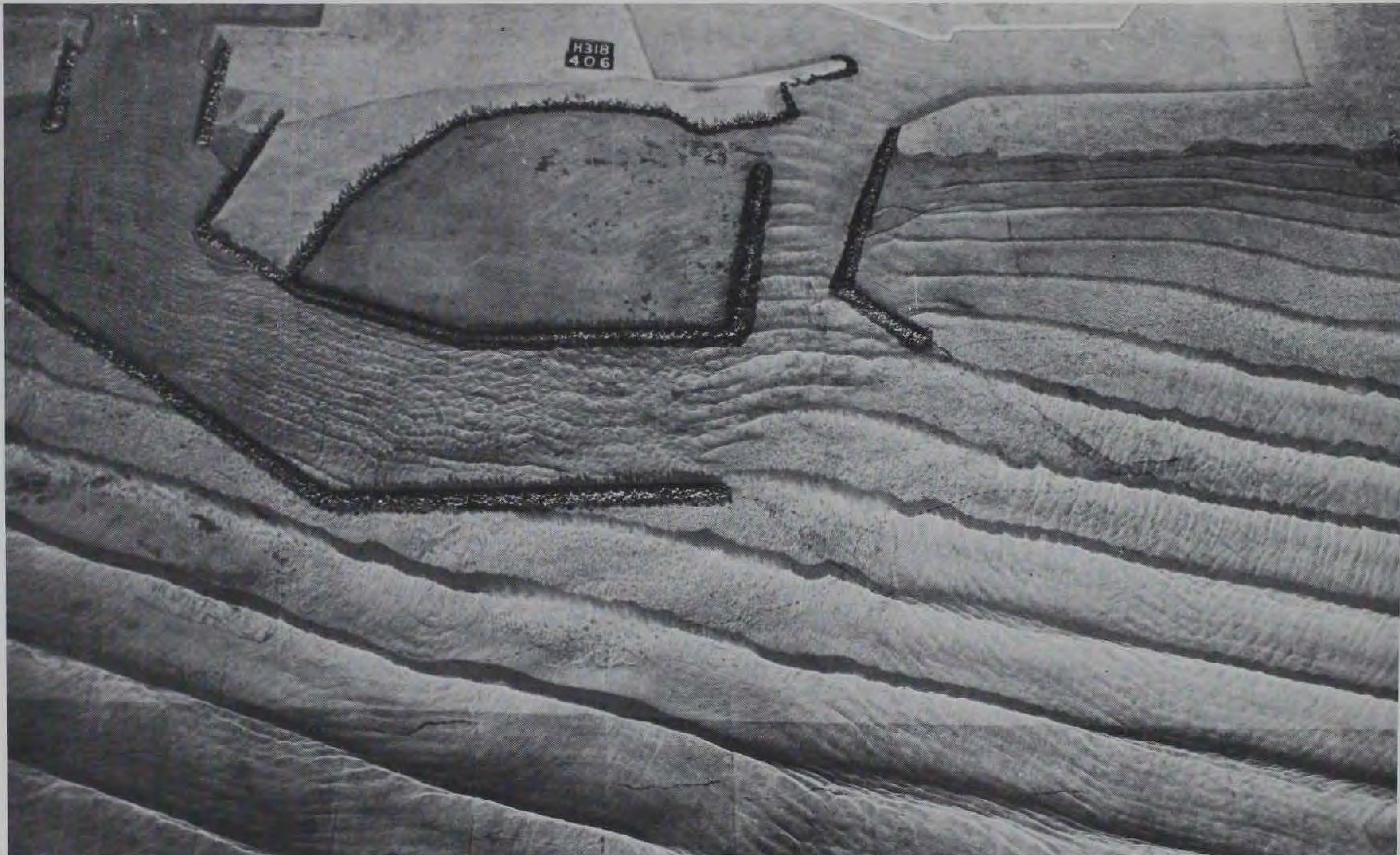


Photo 37. Typical wave patterns for Plan 25A; 11-sec, 10-ft waves from west at mhhw

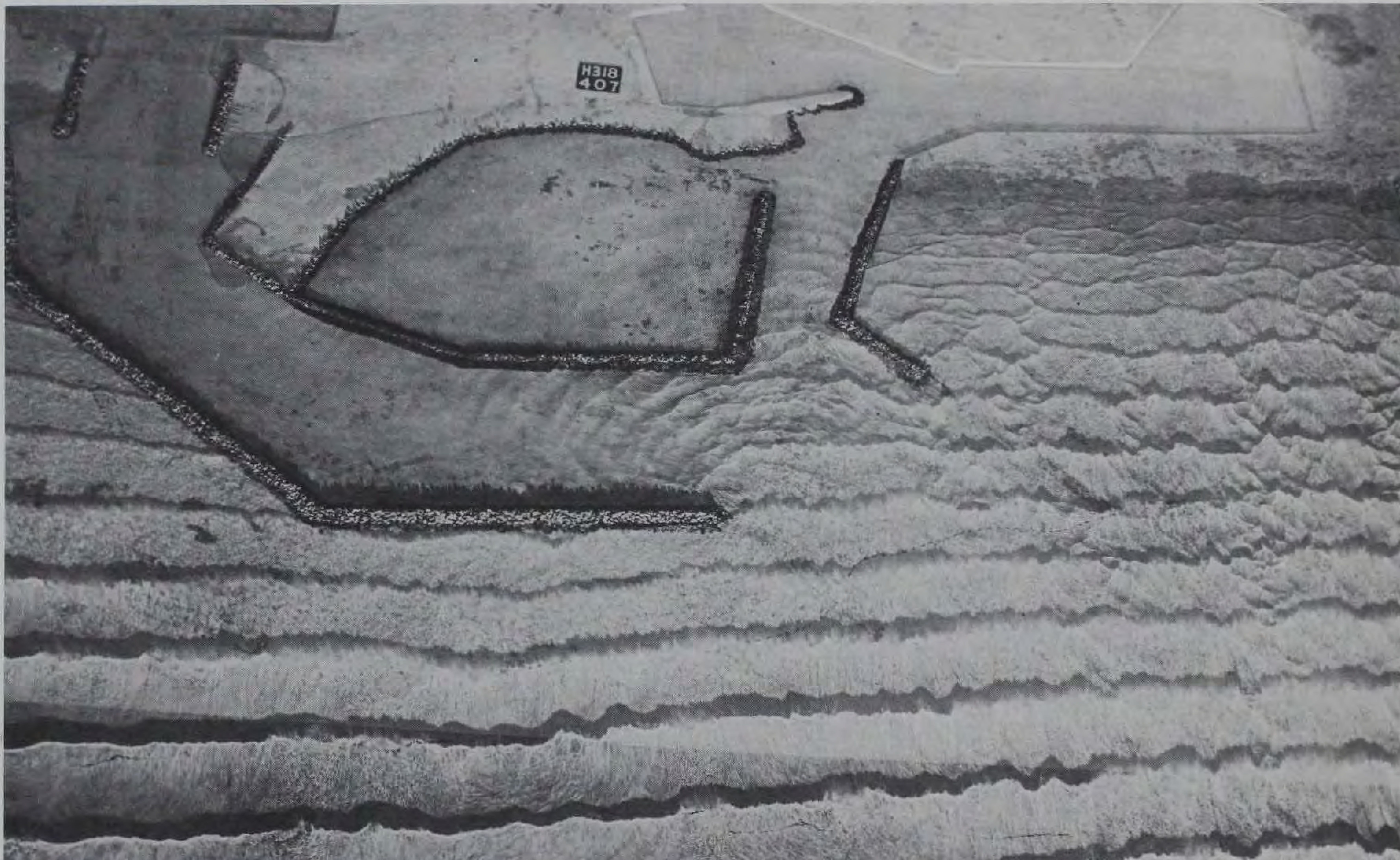


Photo 38. Typical wave patterns for Plan 26; 9-sec, 16-ft waves from southwest at mhhw

Photo 39. Typical wave patterns for Plan 27B; 17-sec, 10-ft waves from southwest at mhhw

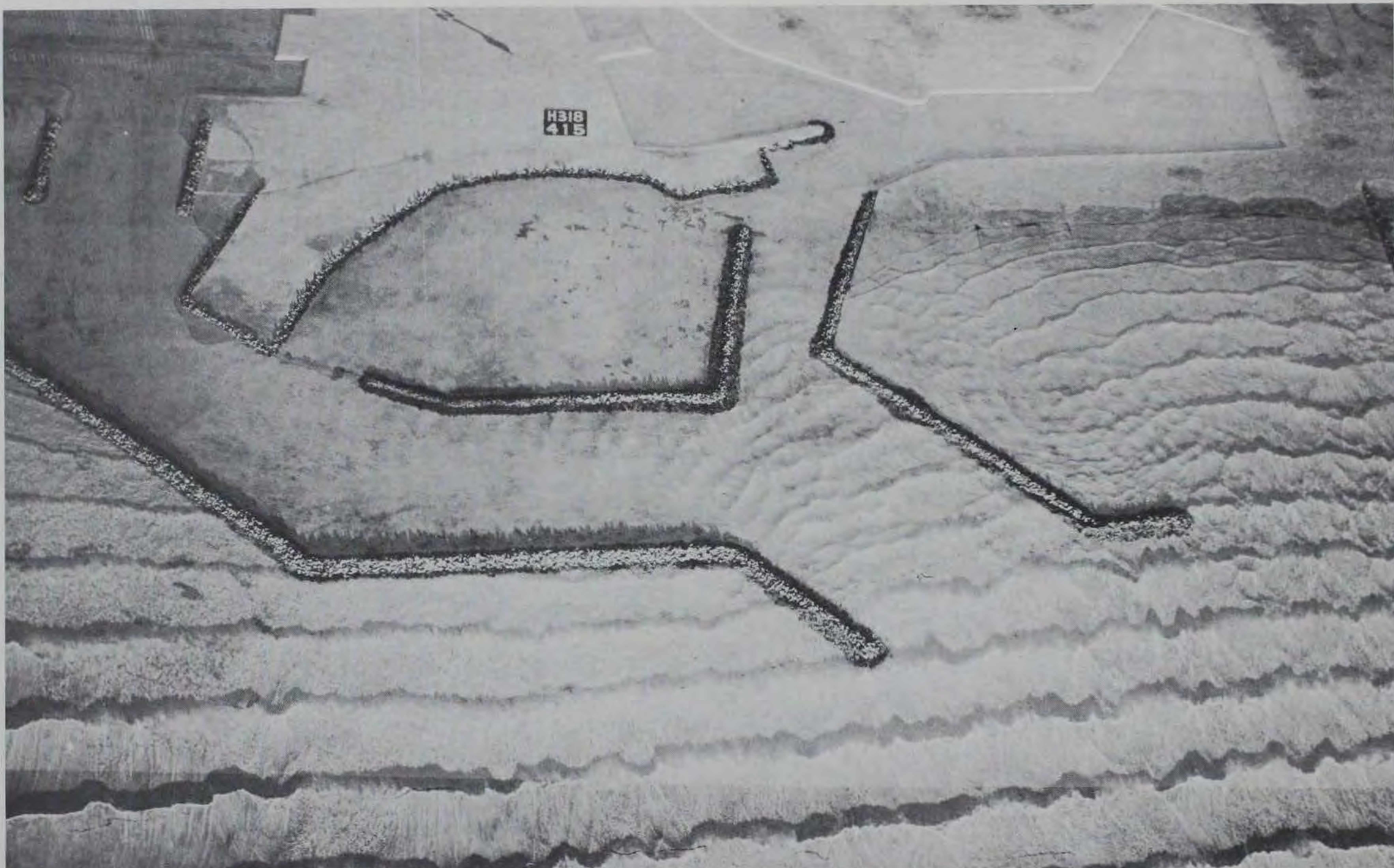


Photo 40. Typical wave patterns for Plan 28; 9-sec, 16-ft waves from southwest at mhhw

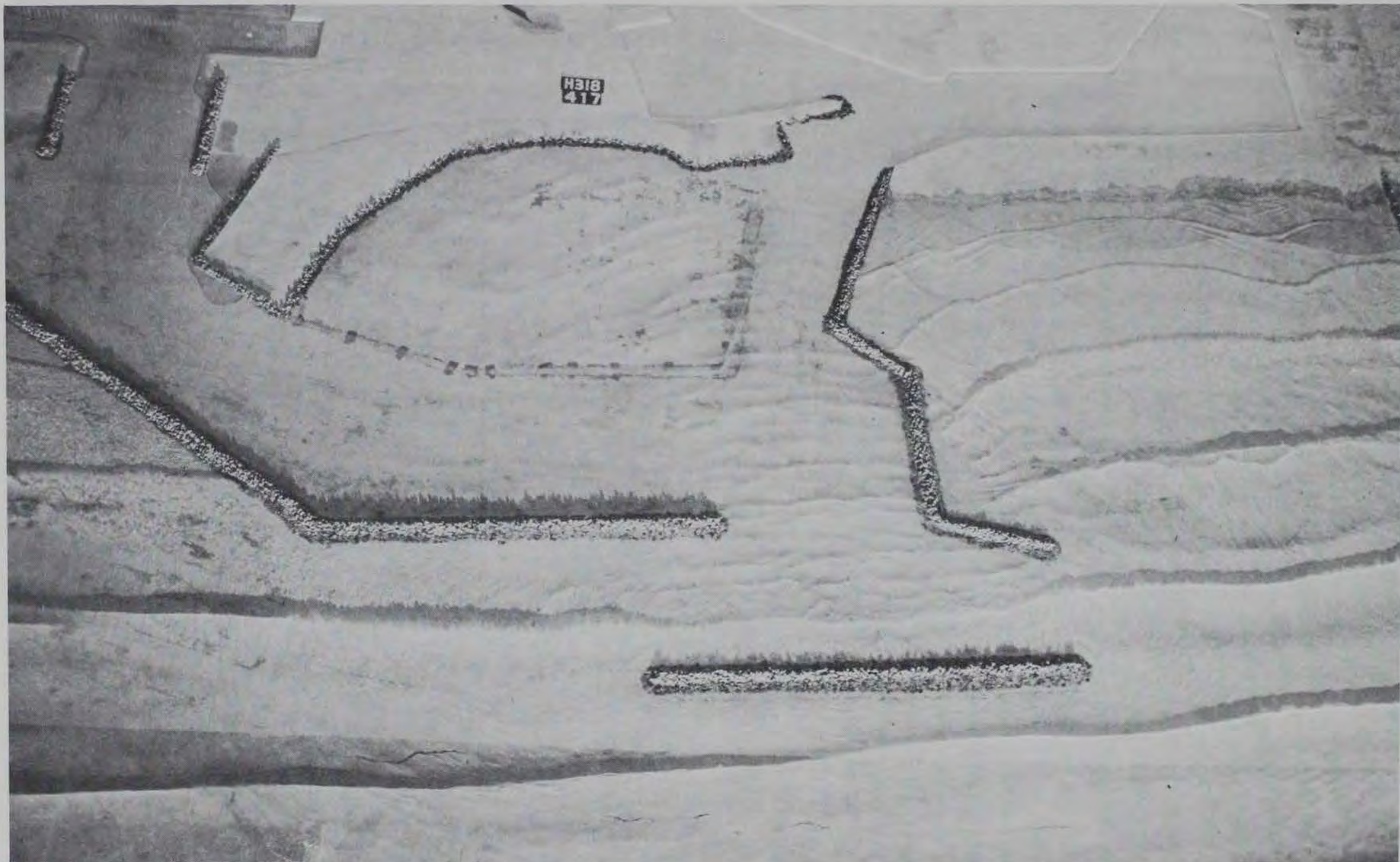


Photo 41. Typical wave patterns for Plan 29; 17-sec, 10-ft waves from southwest at mhhw

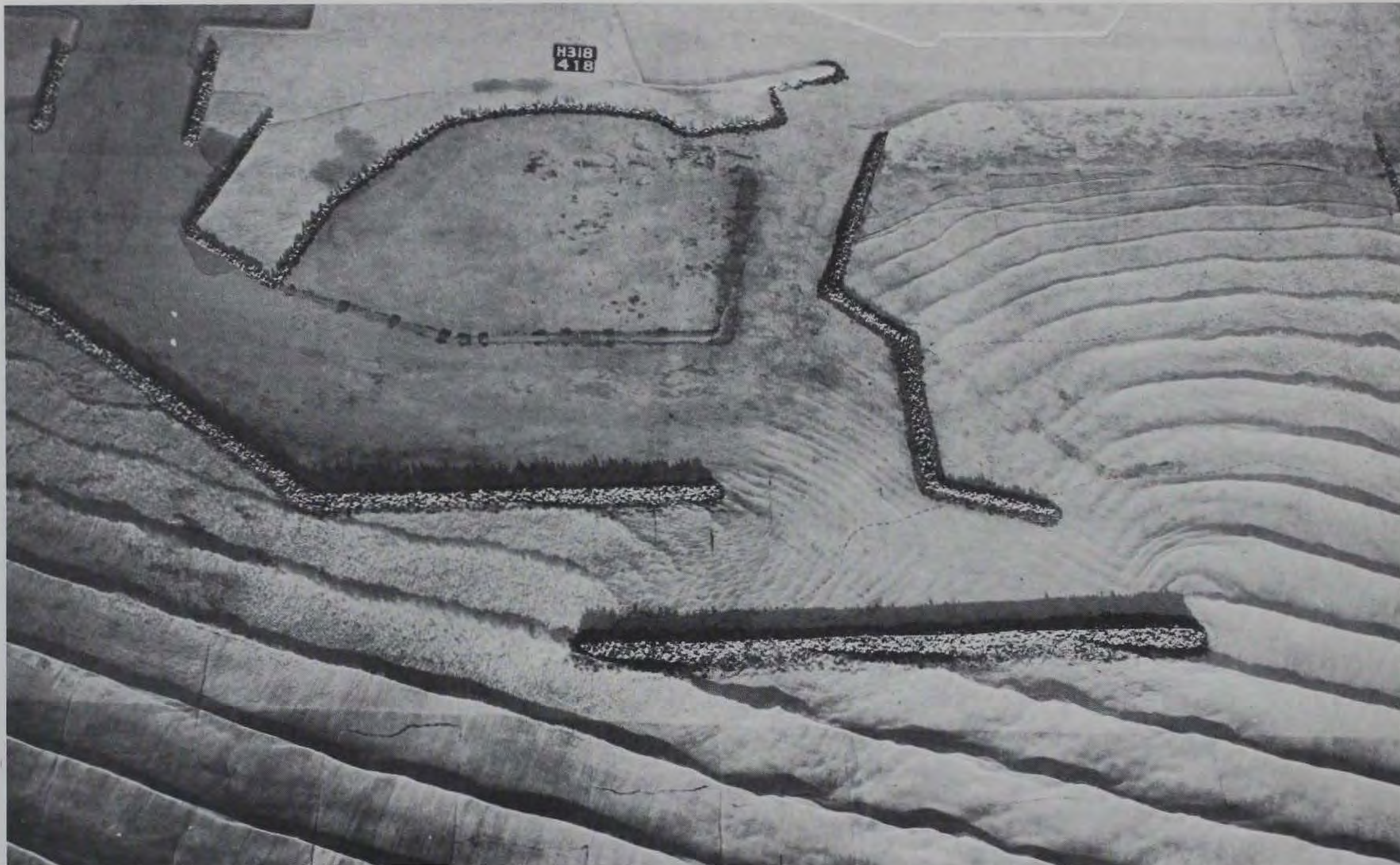


Photo 42. Typical wave patterns for Plan 31F; 9-sec, 12-ft waves from west at mhhw

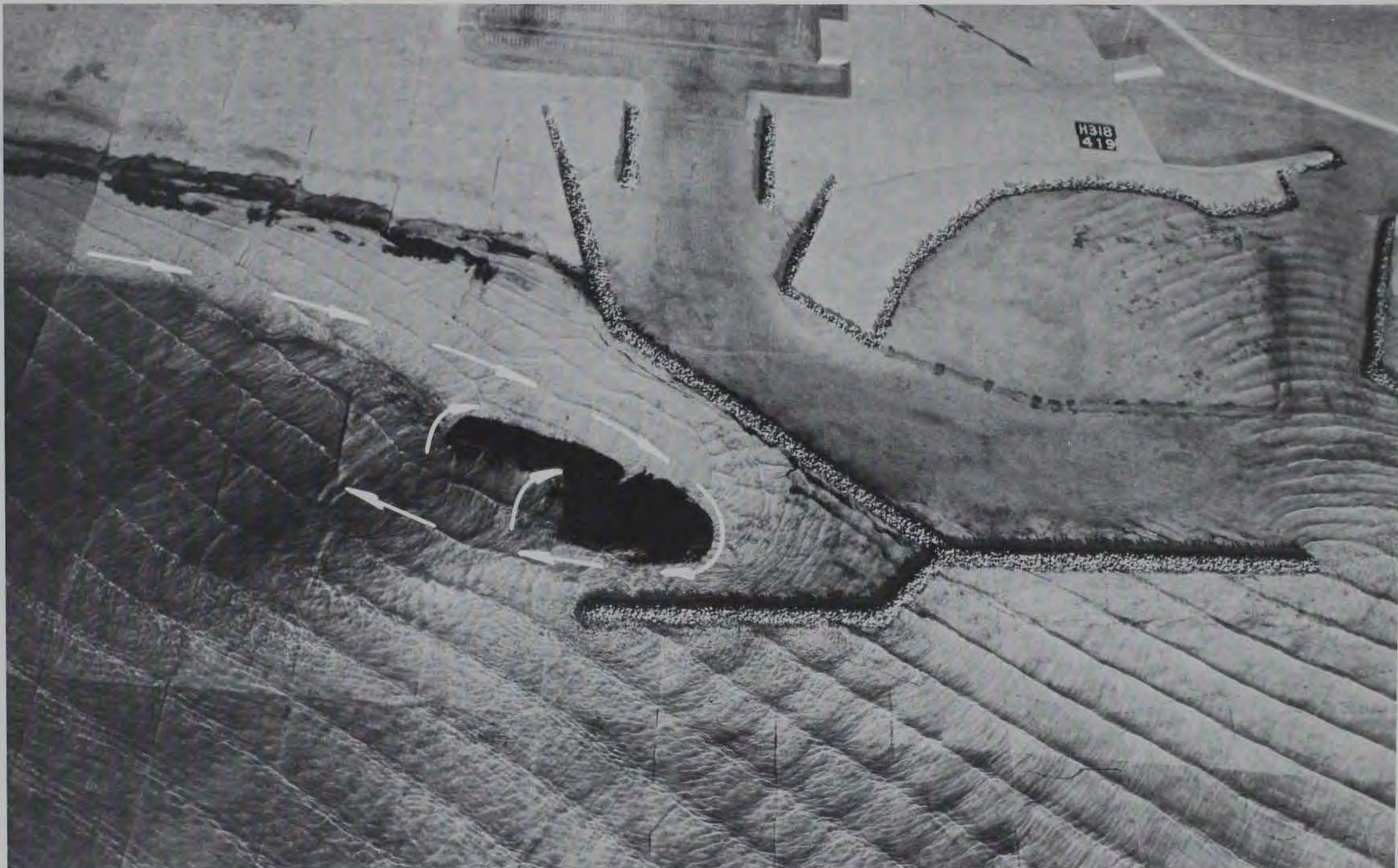


Photo 43. Typical tracer movement for Plan 32 resulting from 7-sec, 10-ft waves from northwest at mllw

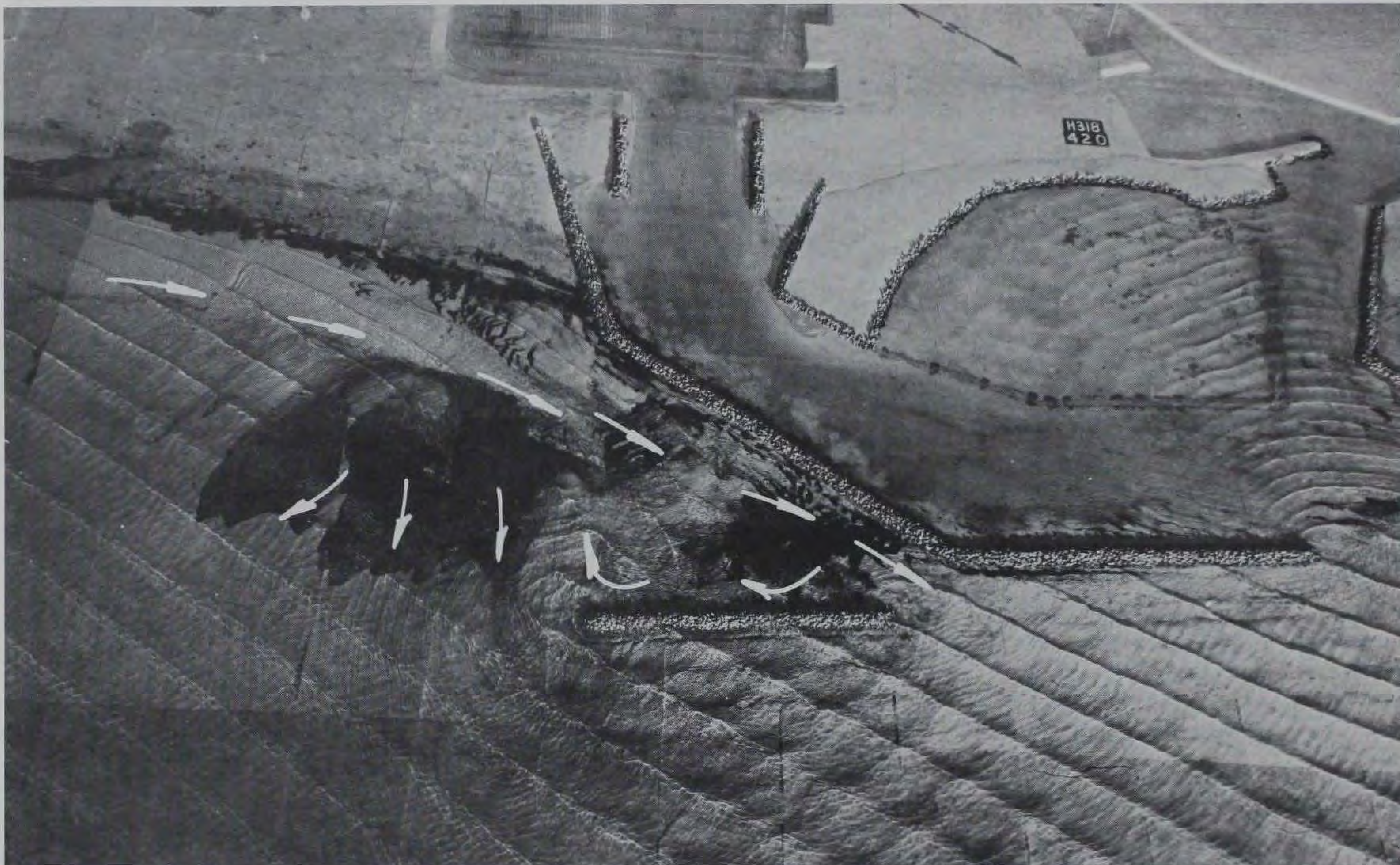


Photo 44. Typical tracer movement for Plan 33 resulting from 7-sec, 10-ft waves from northwest at mllw

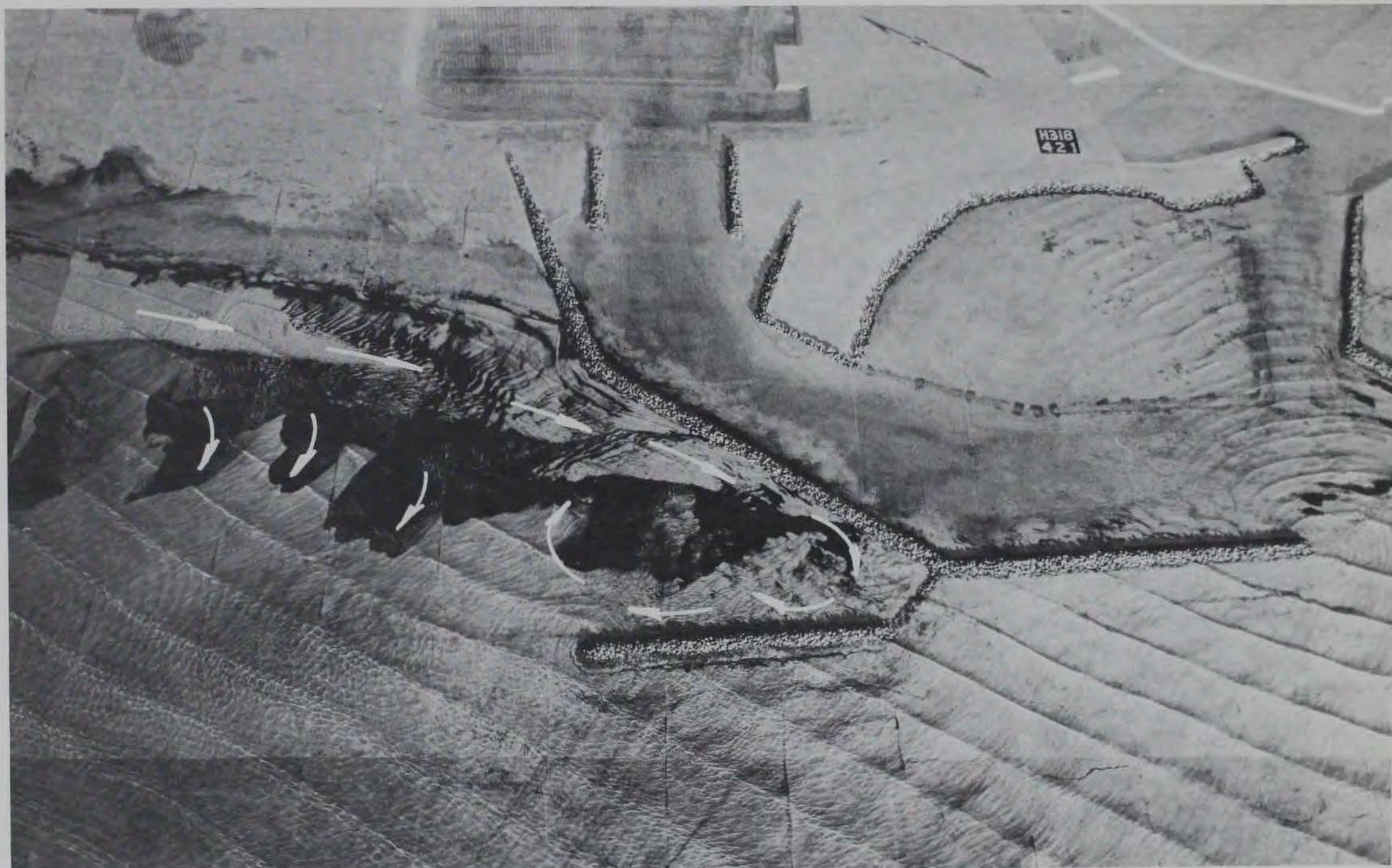


Photo 45. Typical tracer movement for Plan 34 resulting from 7-sec, 10-ft waves from northwest at mllw

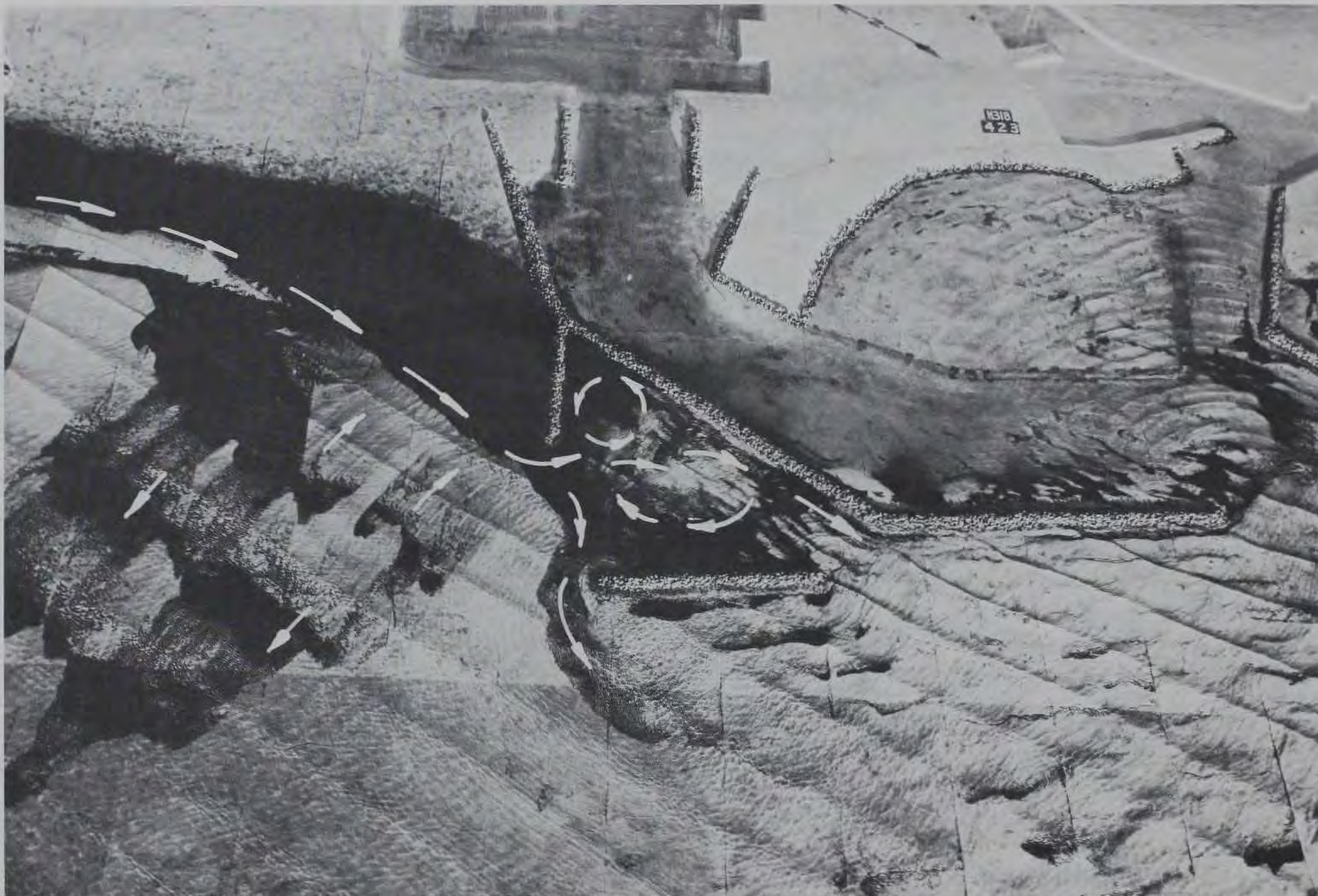


Photo 46. Typical tracer movement for Plan 38C resulting from 7-sec, 10-ft waves from northwest at mllw (with waves)

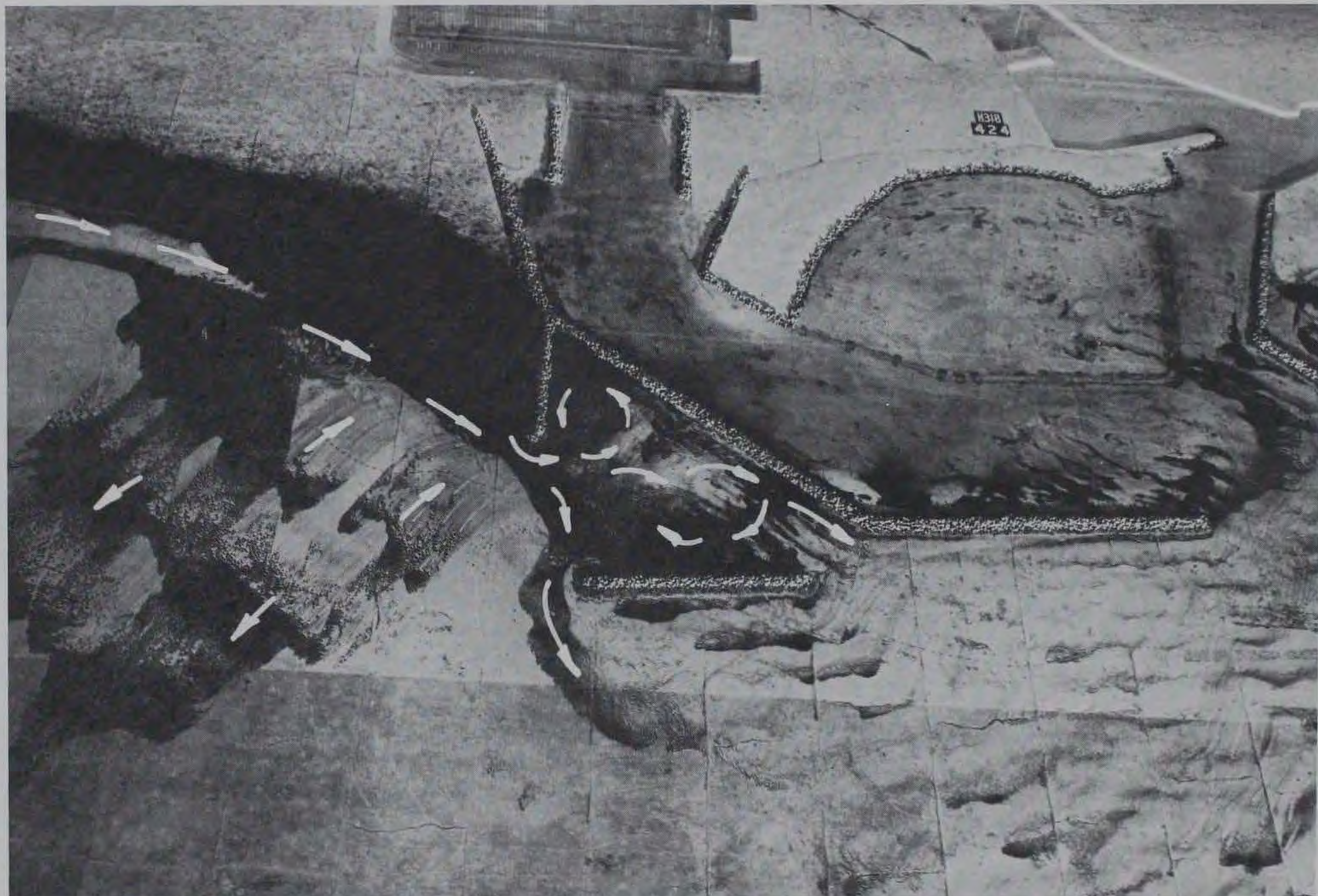


Photo 47. Typical tracer movement for Plan 38C resulting from 7-sec, 10-ft waves from northwest at mllw (without waves)

Photo 48. Typical tracer movement for Plan 38C resulting from 17-sec, 10-ft waves from southwest at mhhw (with waves)

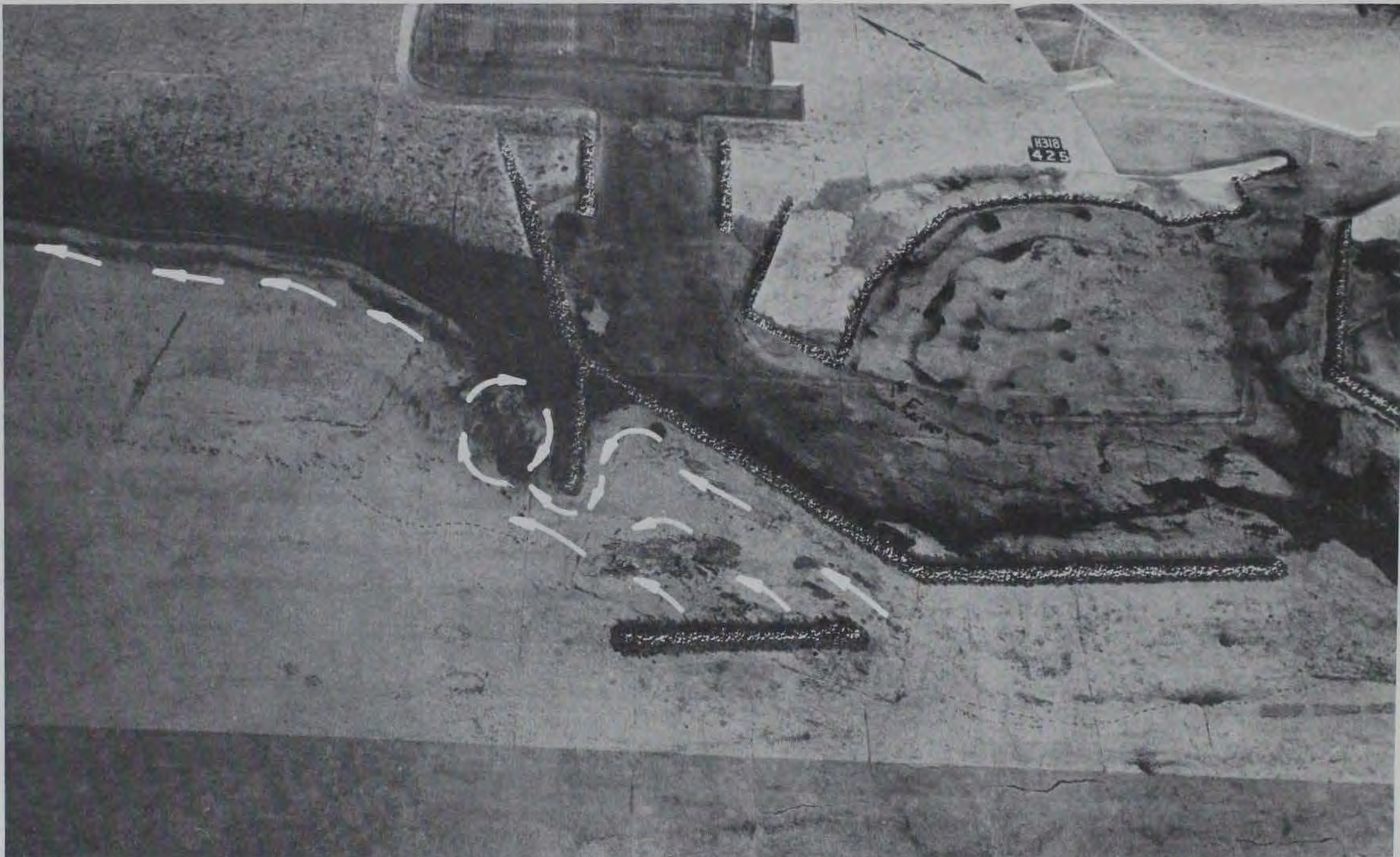


Photo 49. Typical tracer movement for Plan 38C resulting from 17-sec, 10-ft waves from southwest at mhhw (without waves)

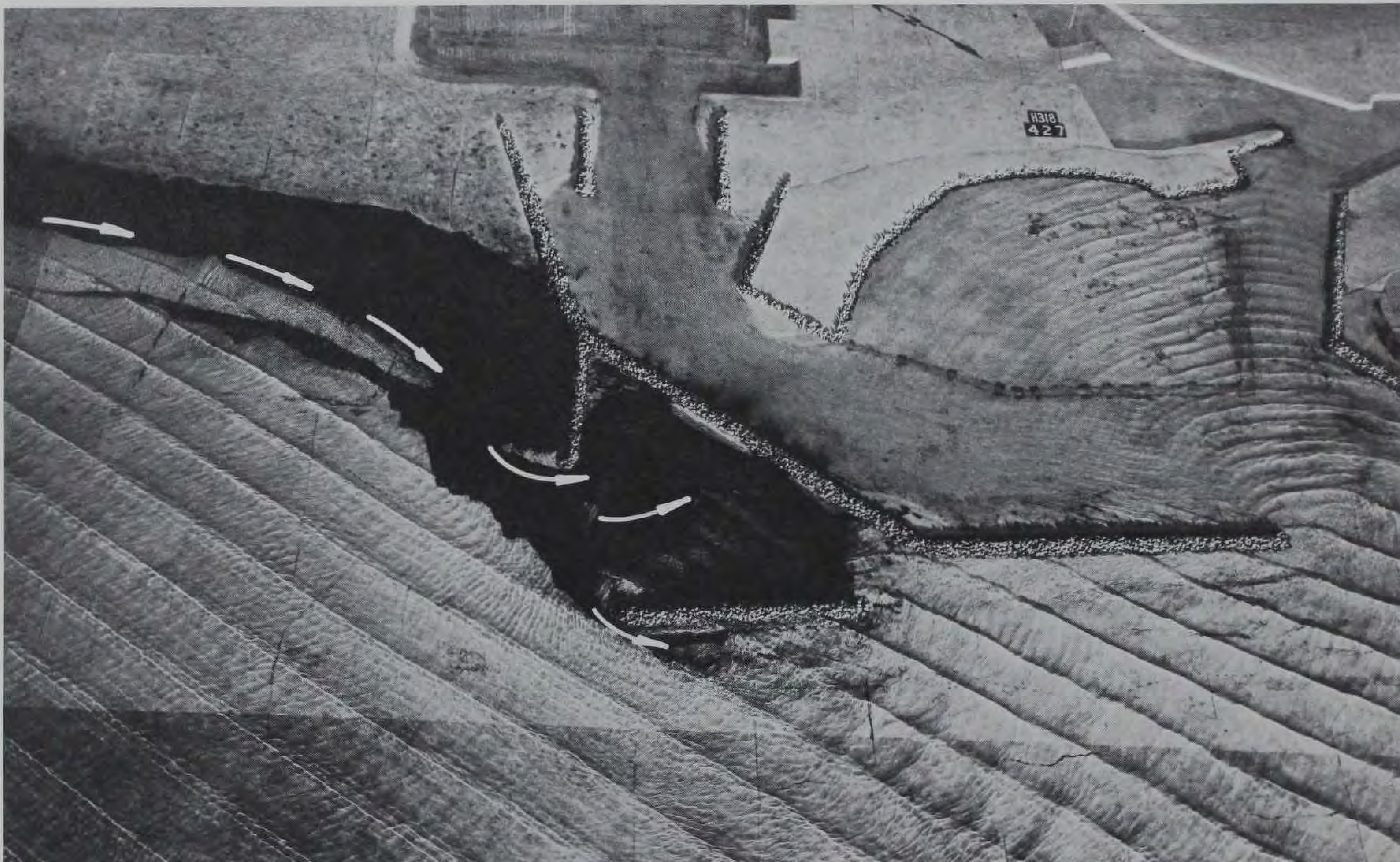


Photo 50. Typical tracer movement for Plan 44 resulting from 7-sec, 10-ft waves from northwest at mllw

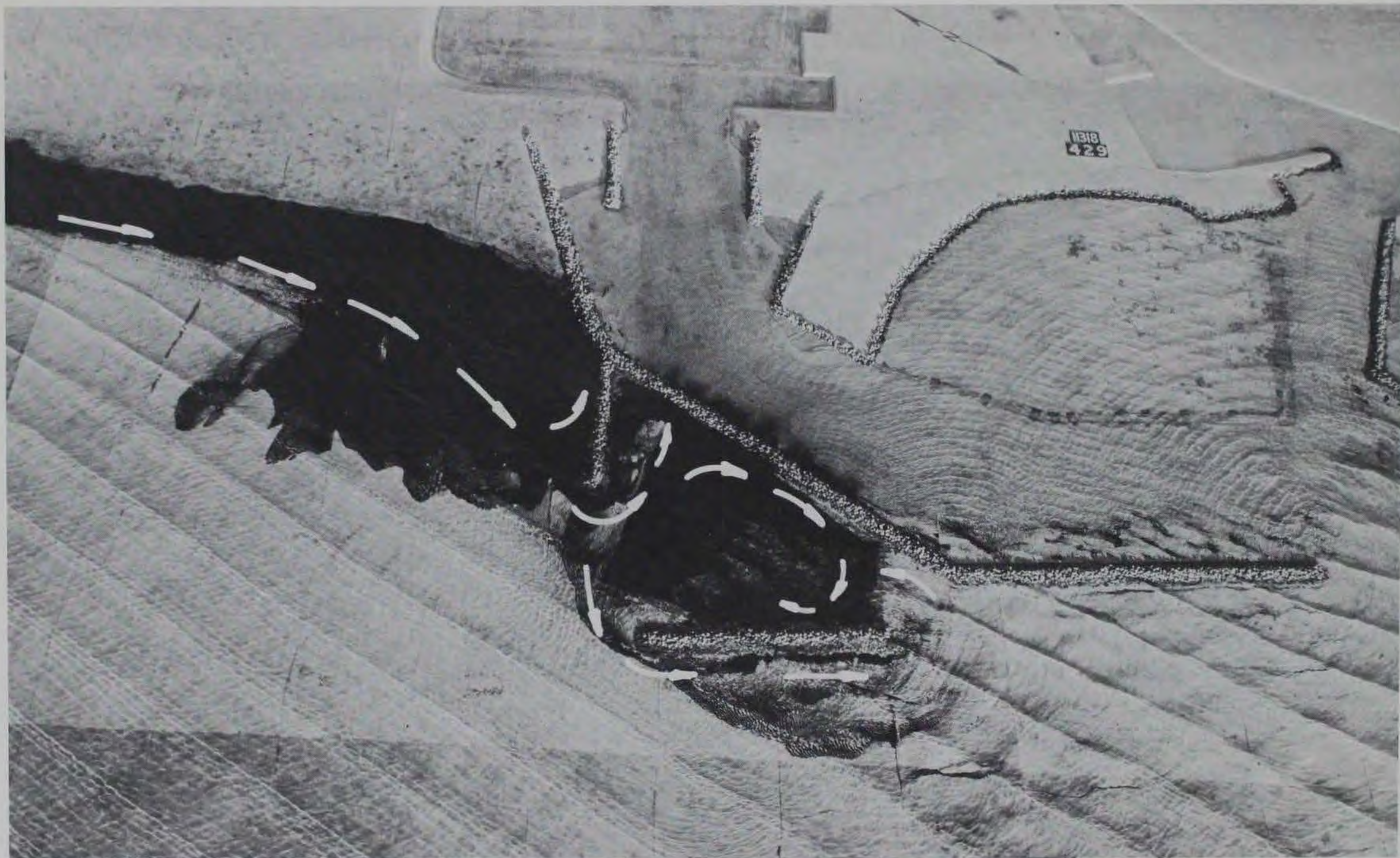


Photo 51. Typical tracer movement for Plan 44 resulting from 7-sec, 10-ft waves from northwest at mhhw

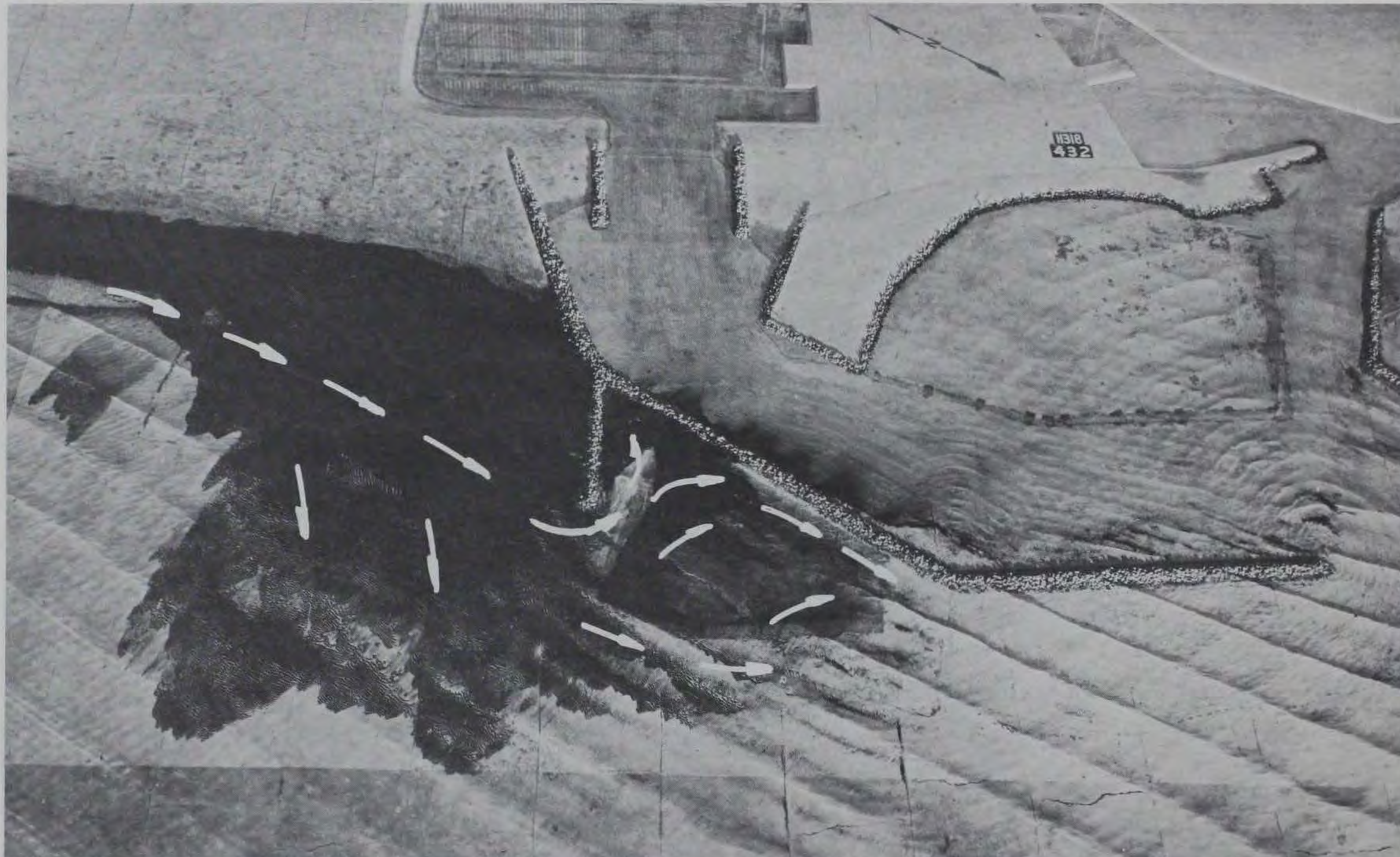


Photo 52. Typical tracer movement for Plan 45 resulting from 7-sec, 10-ft waves from northwest at mllw

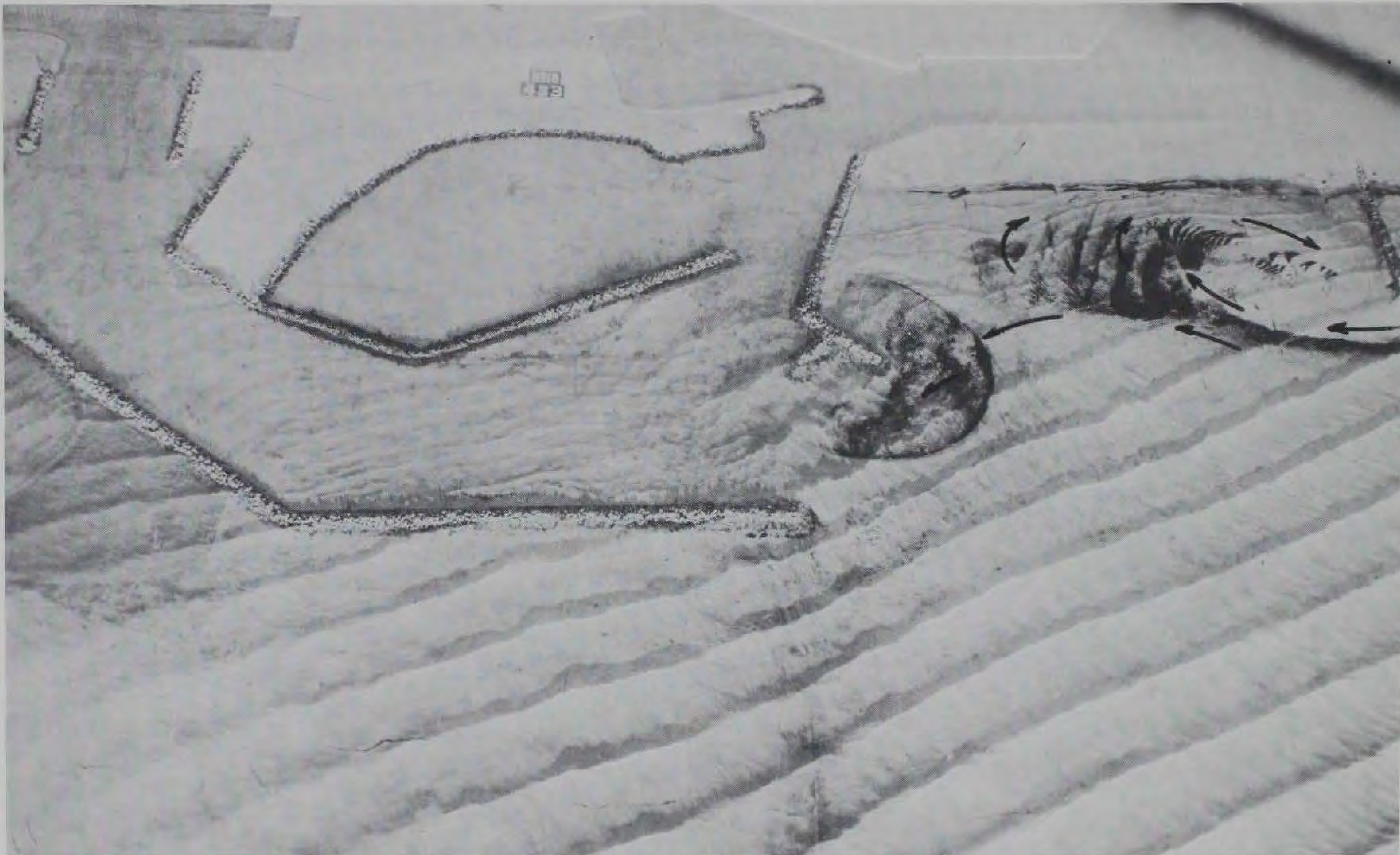


Photo 53. Typical tracer movement for Plan 61 resulting from 9-sec, 10-ft waves from south at mhhw

Photo 54. Typical tracer movement for Plan 61 resulting from 9-sec, 10-ft waves from south at mllw

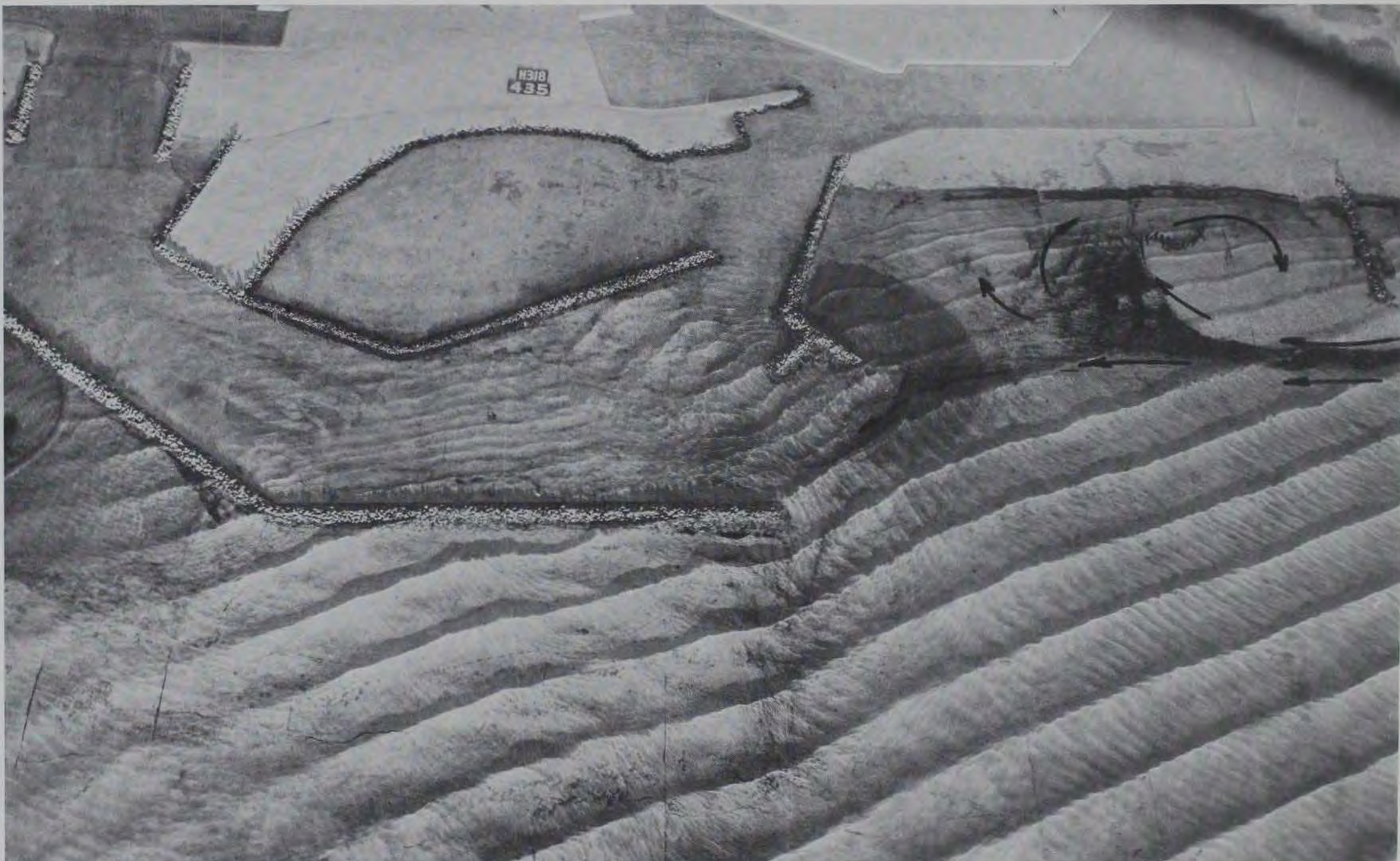


Photo 55. Typical tracer movement for Plan 62 resulting from 9-sec, 10-ft waves from south at mhhw

Photo 56. Typical tracer movement for Plan 62 resulting from 9-sec, 10-ft waves from south at mllw

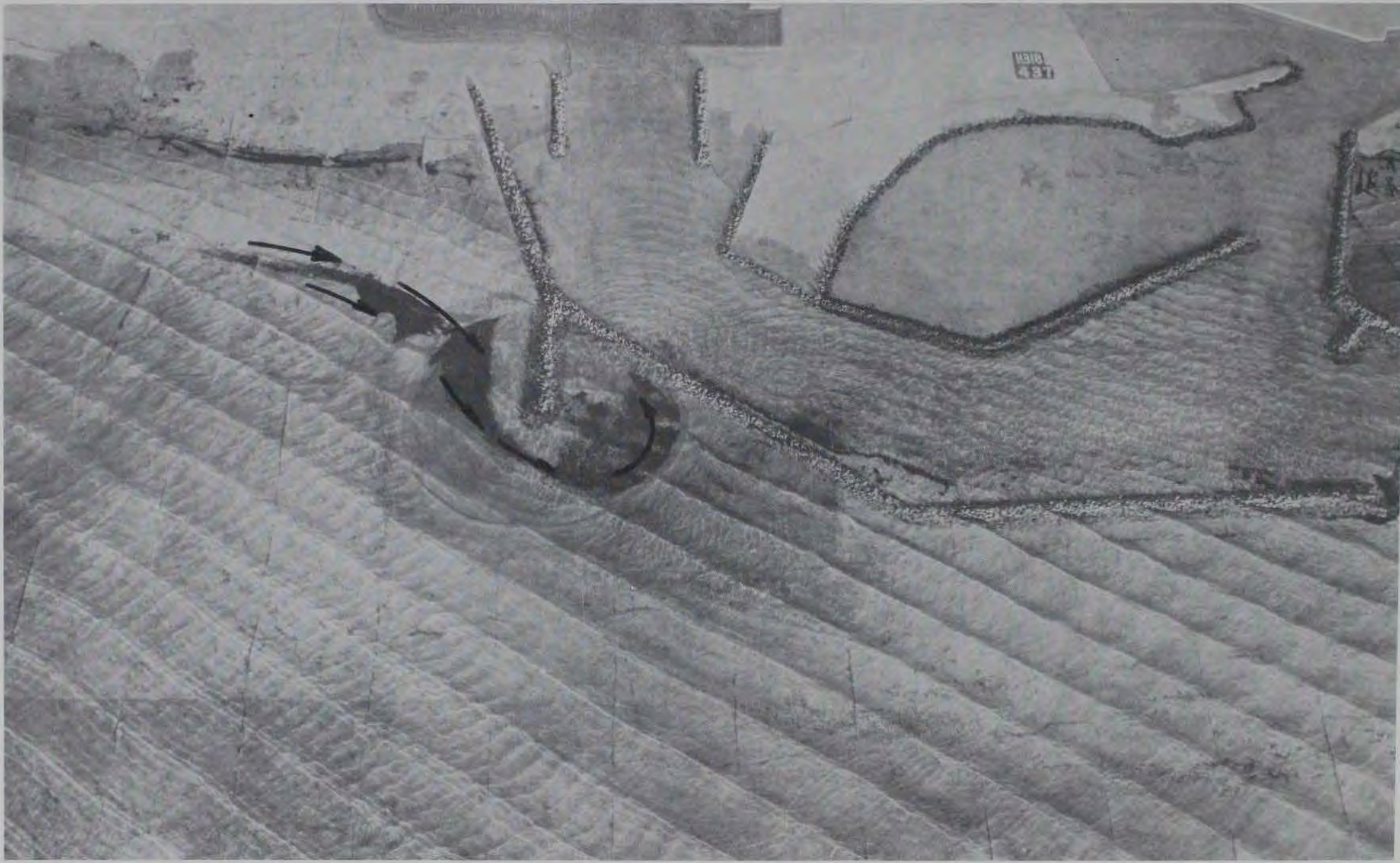


Photo 57. Typical tracer movement for Plan 62 resulting from 7-sec, 10-ft waves from northwest at mhhw

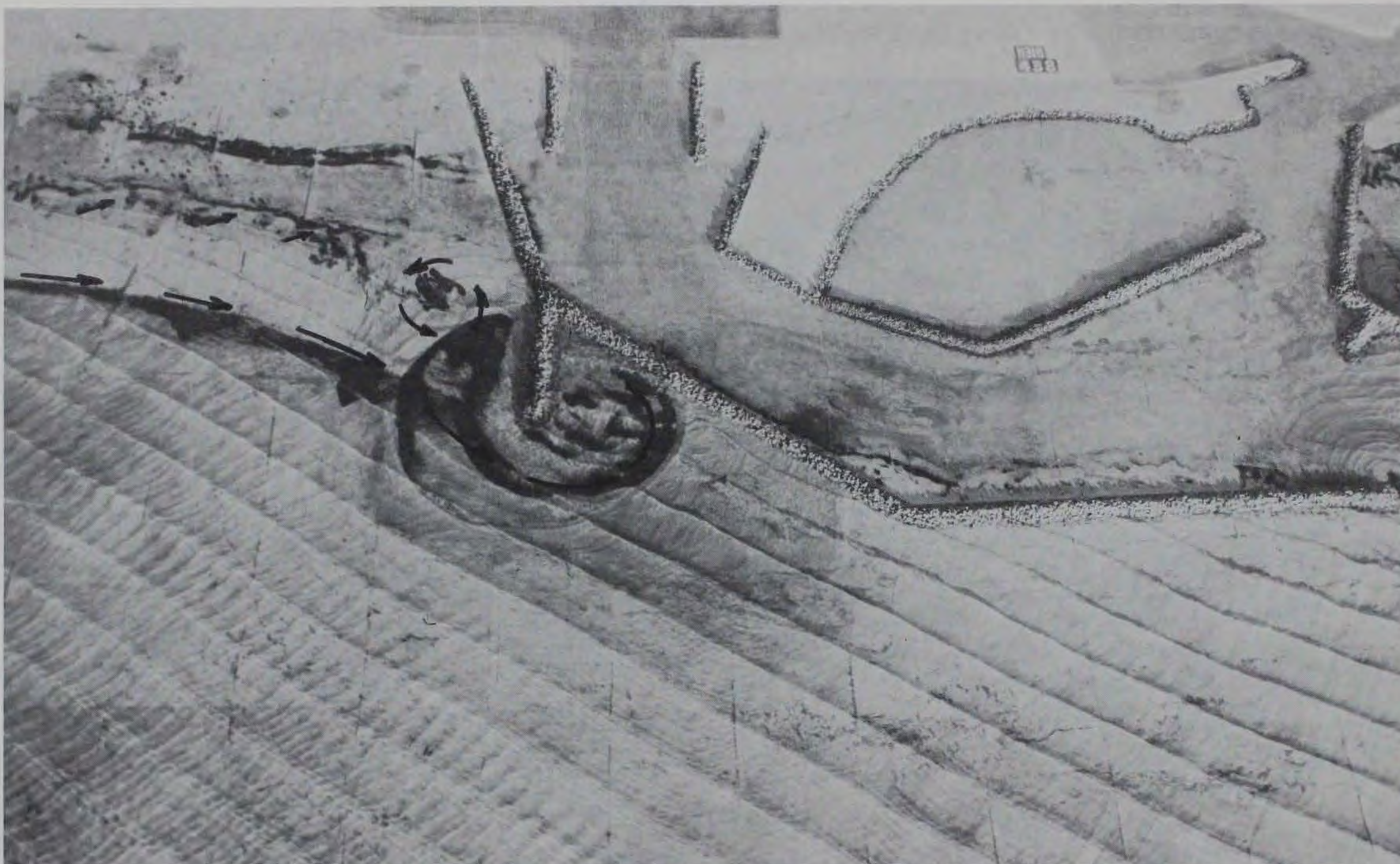


Photo 58. Typical tracer movement for Plan 62 resulting from 7-sec, 10-ft waves from northwest at mllw

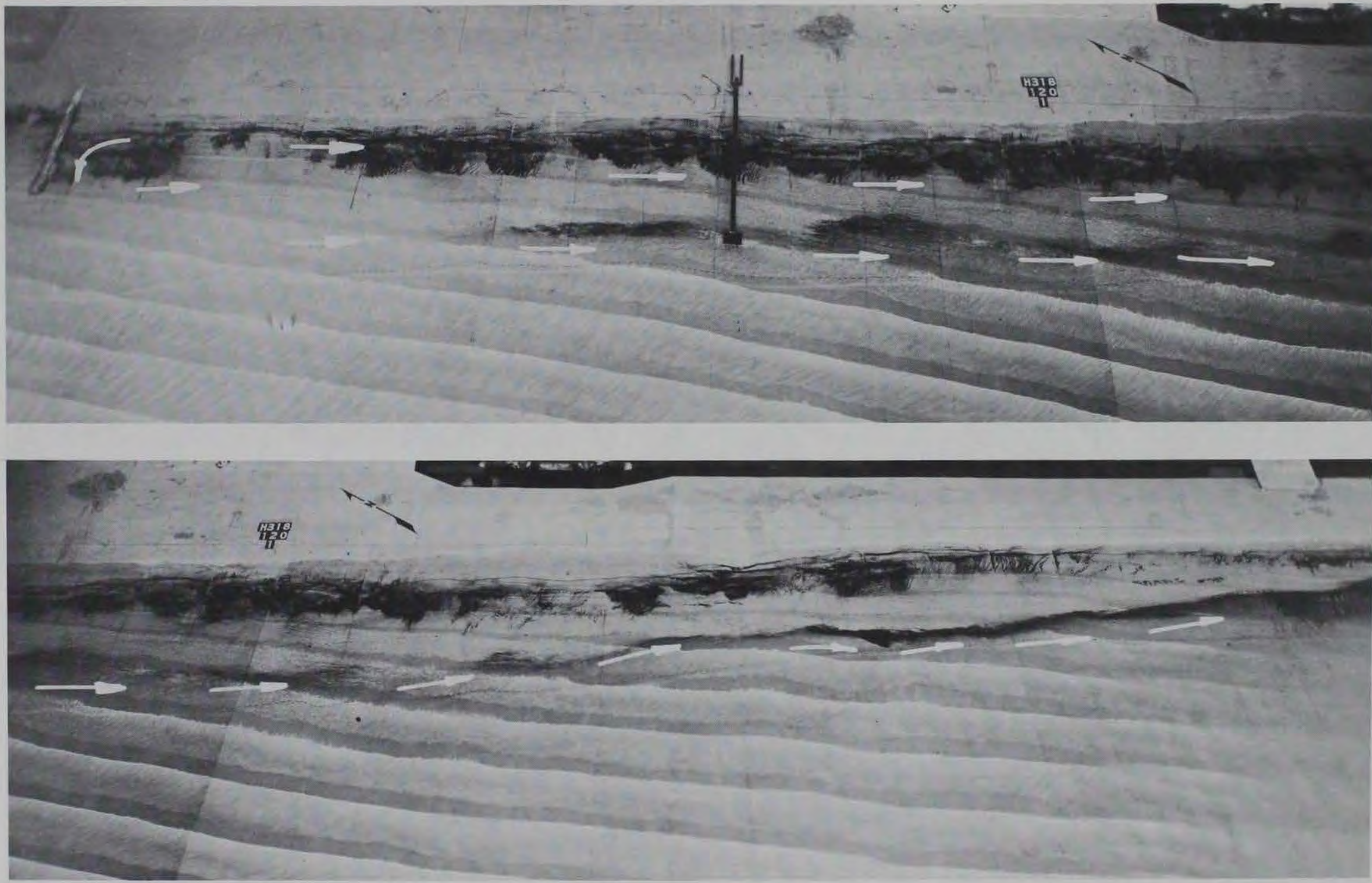


Photo 59. Typical tracer movement for existing conditions resulting from 11-sec, 10-ft waves from west at mllw

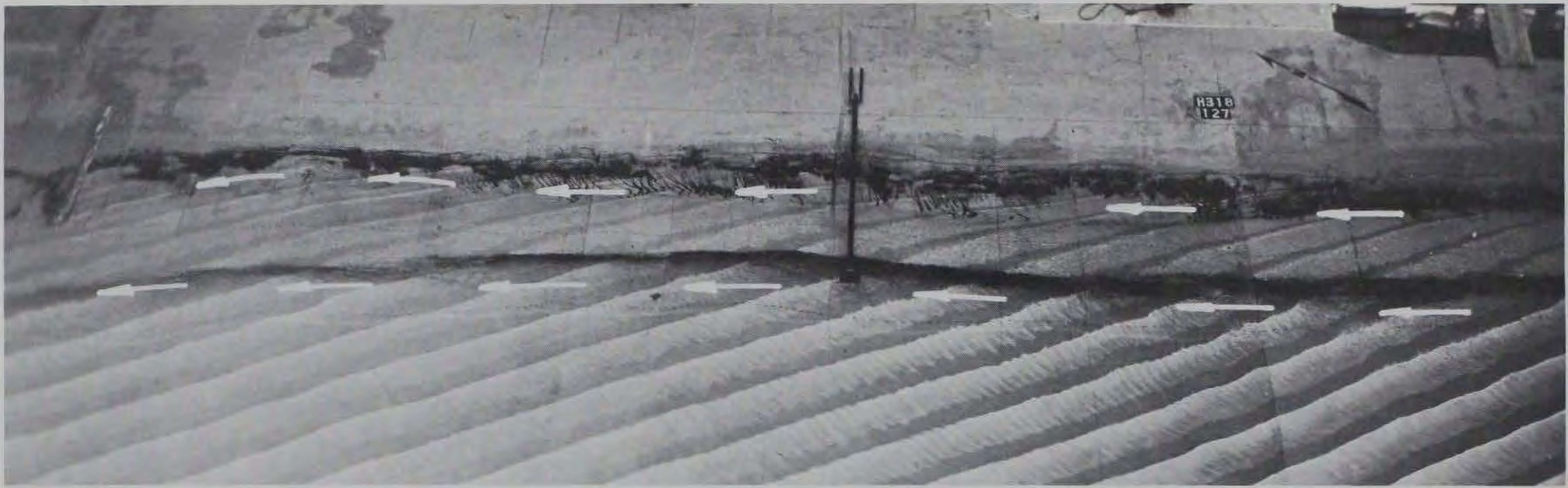


Photo 60. Typical tracer movements for existing conditions resulting from 9-sec, 10-ft waves from south at mllw

Photo 61. Semimovable-bed tracer deposits for Plan 6 resulting from 9-sec, 10-ft waves from south at mllw

Photo 62. Semimovable-bed tracer deposits for Plan 7 resulting from 9-sec, 10-ft waves from south at mllw

Photo 63. Semimovable-bed tracer deposits for Plan 8 resulting from 9-sec, 10-ft waves from south at mllw

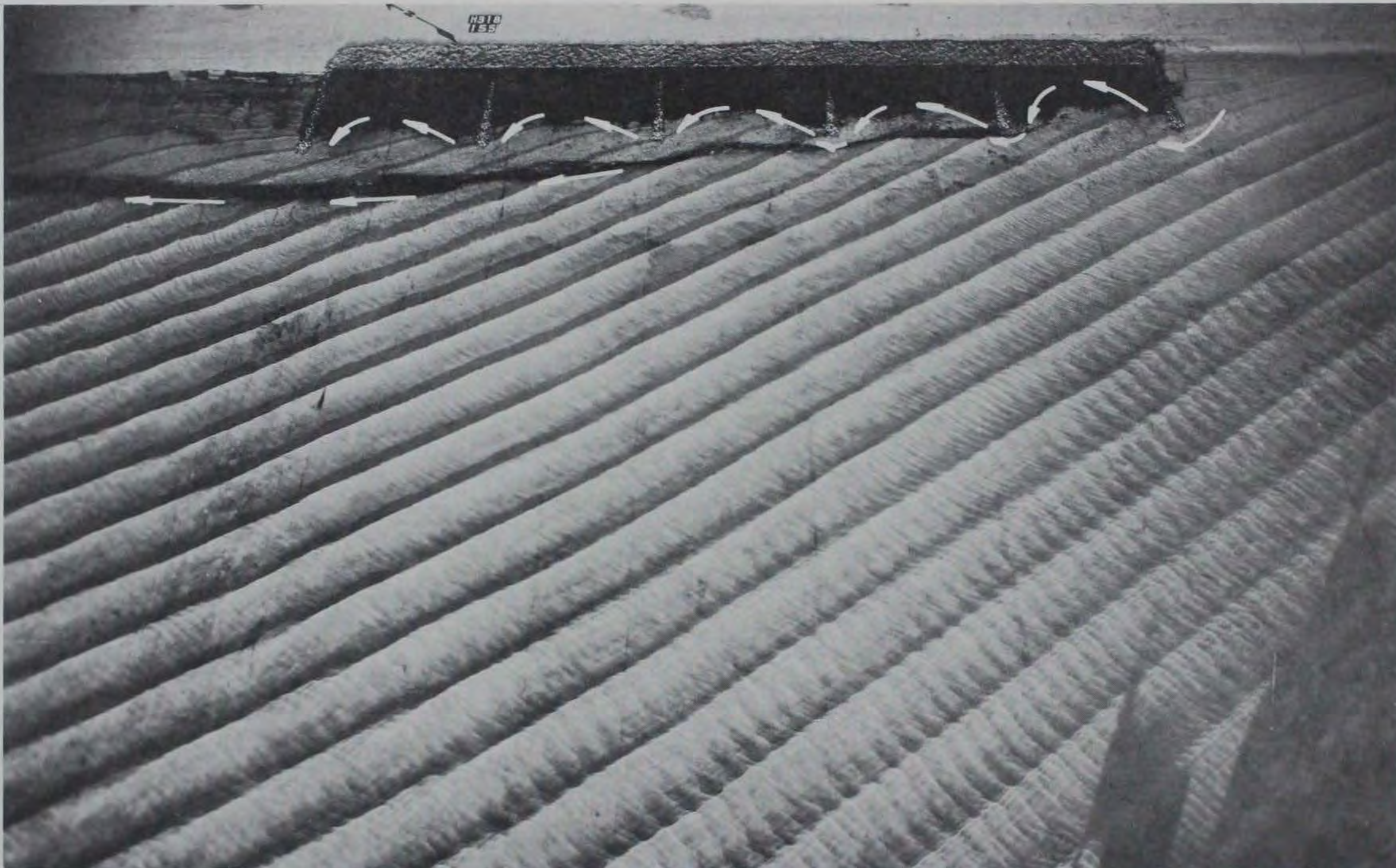


Photo 64. Semimovable-bed tracer deposits for Plan 9 resulting from 9-sec, 10-ft waves from south at mllw

Photo 65. Semimovable-bed tracer deposits for Plan 10 resulting from 9-sec, 10-ft waves from south at mllw

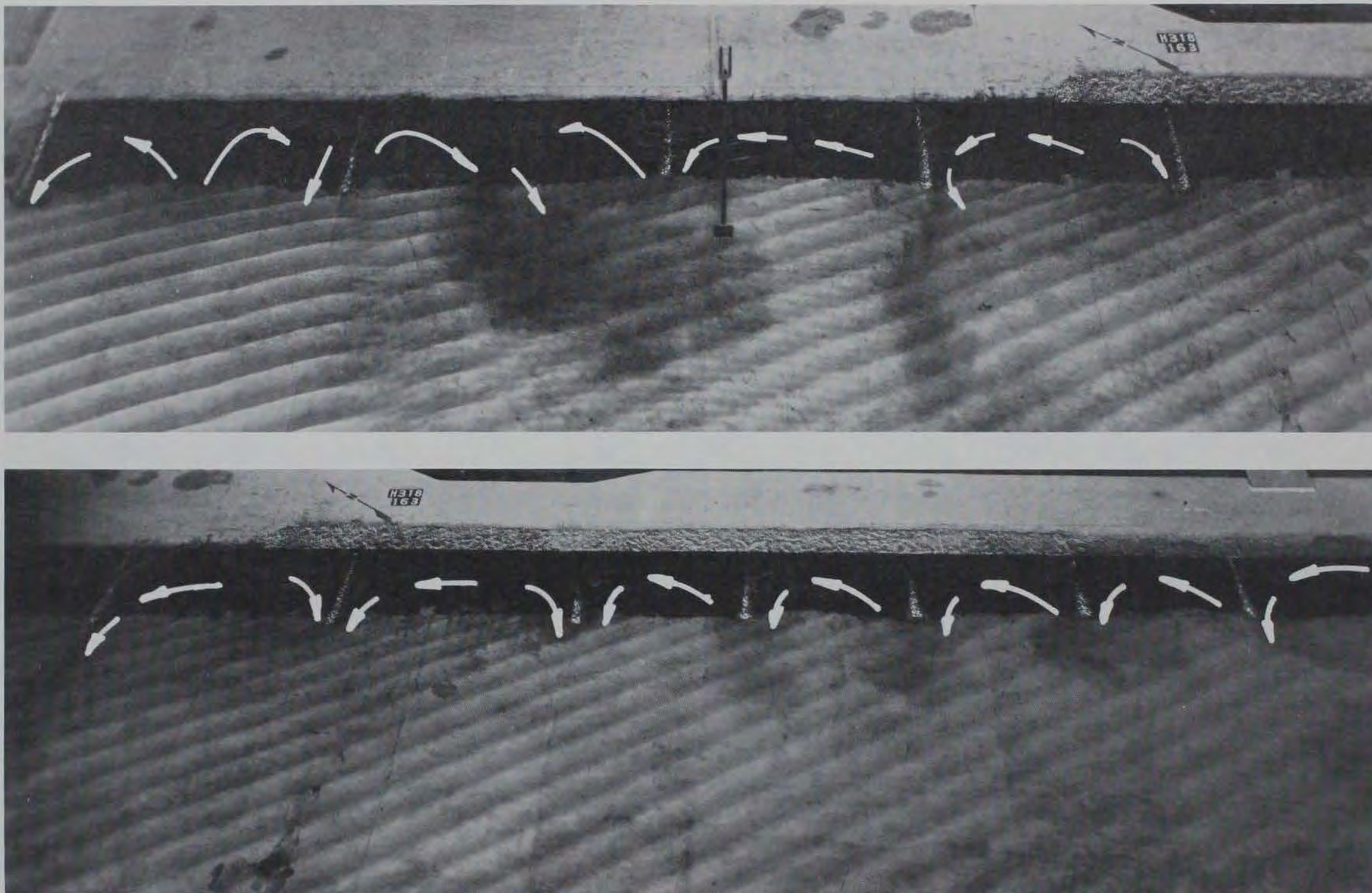


Photo 66. Semimovable-bed tracer deposits for Plan 12 resulting from 7-sec, 4-ft waves from south at mhhw

Photo 67. Semimovable-bed tracer deposits for Plan 12 resulting from 9-sec, 10-ft waves from south at mllw

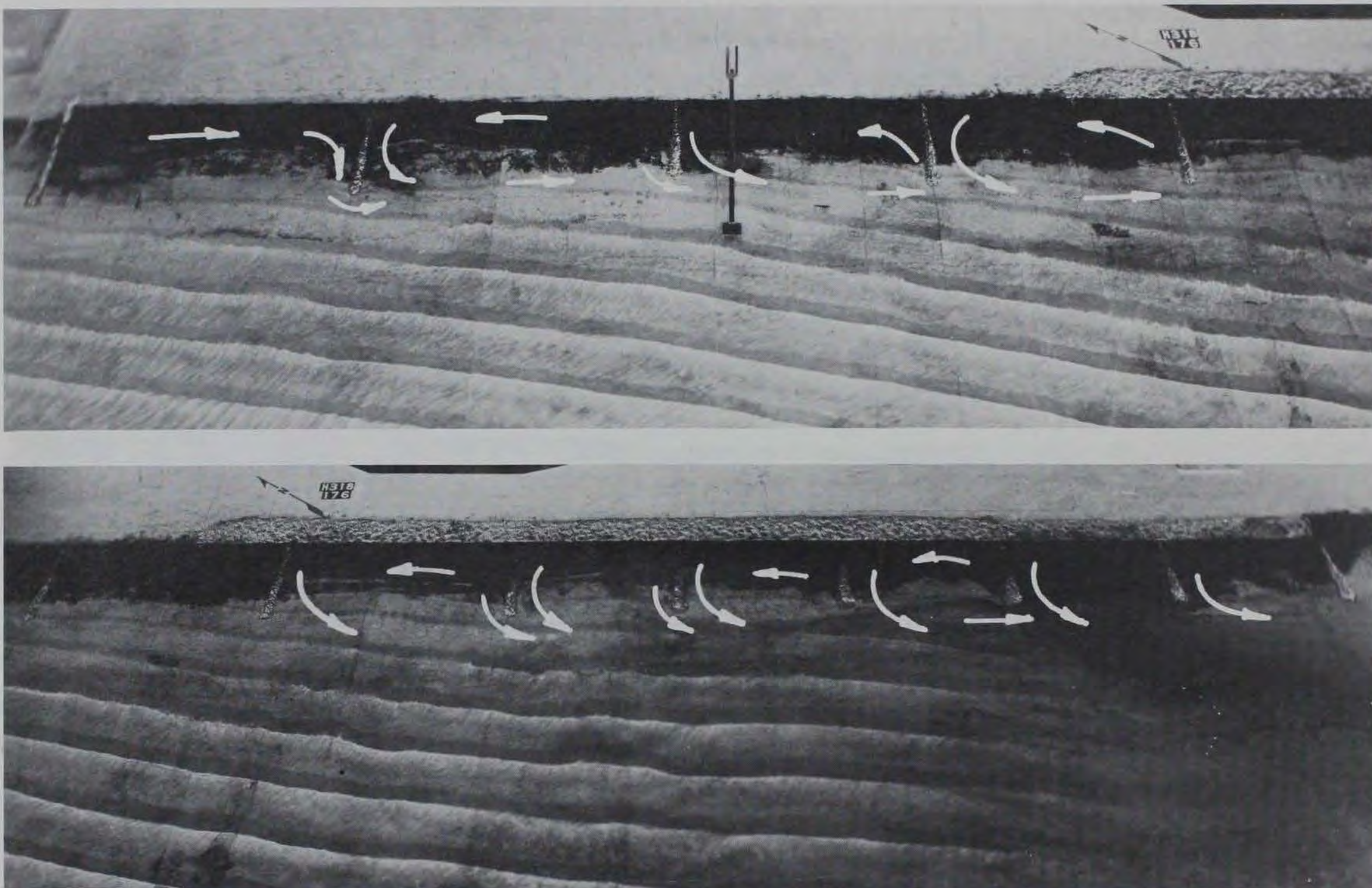


Photo 68. Semimovable-bed tracer deposits for Plan 12 resulting from 11-sec, 10-ft waves from west at mllw (tested with final beach of previous test)

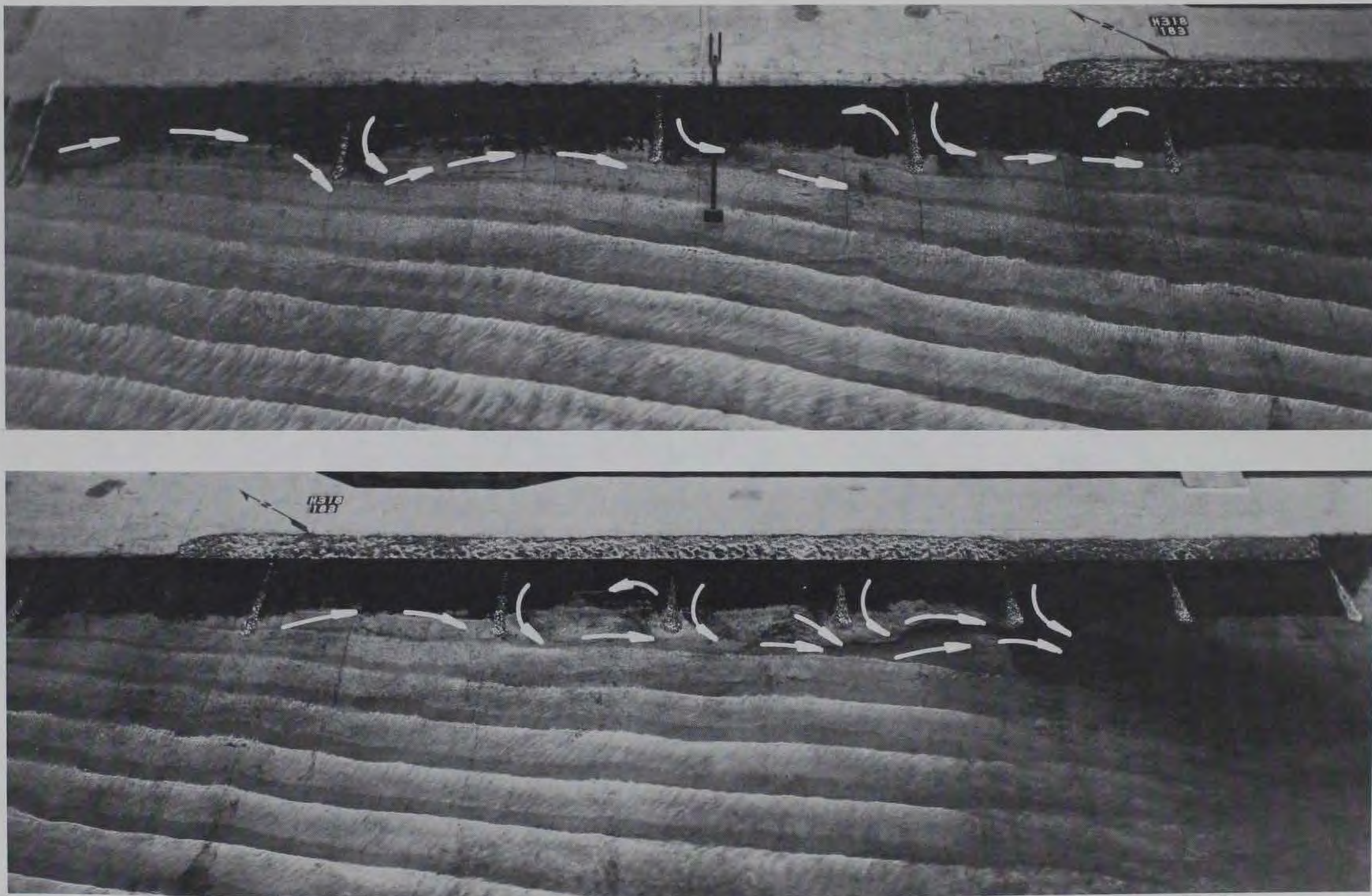


Photo 69. Semimovable-bed tracer deposits for Plan 12 resulting from 11-sec, 10-ft waves from west at mllw (with initial beach fill)

Photo 70. Placement of movable-bed tracer for Plan 9 before testing

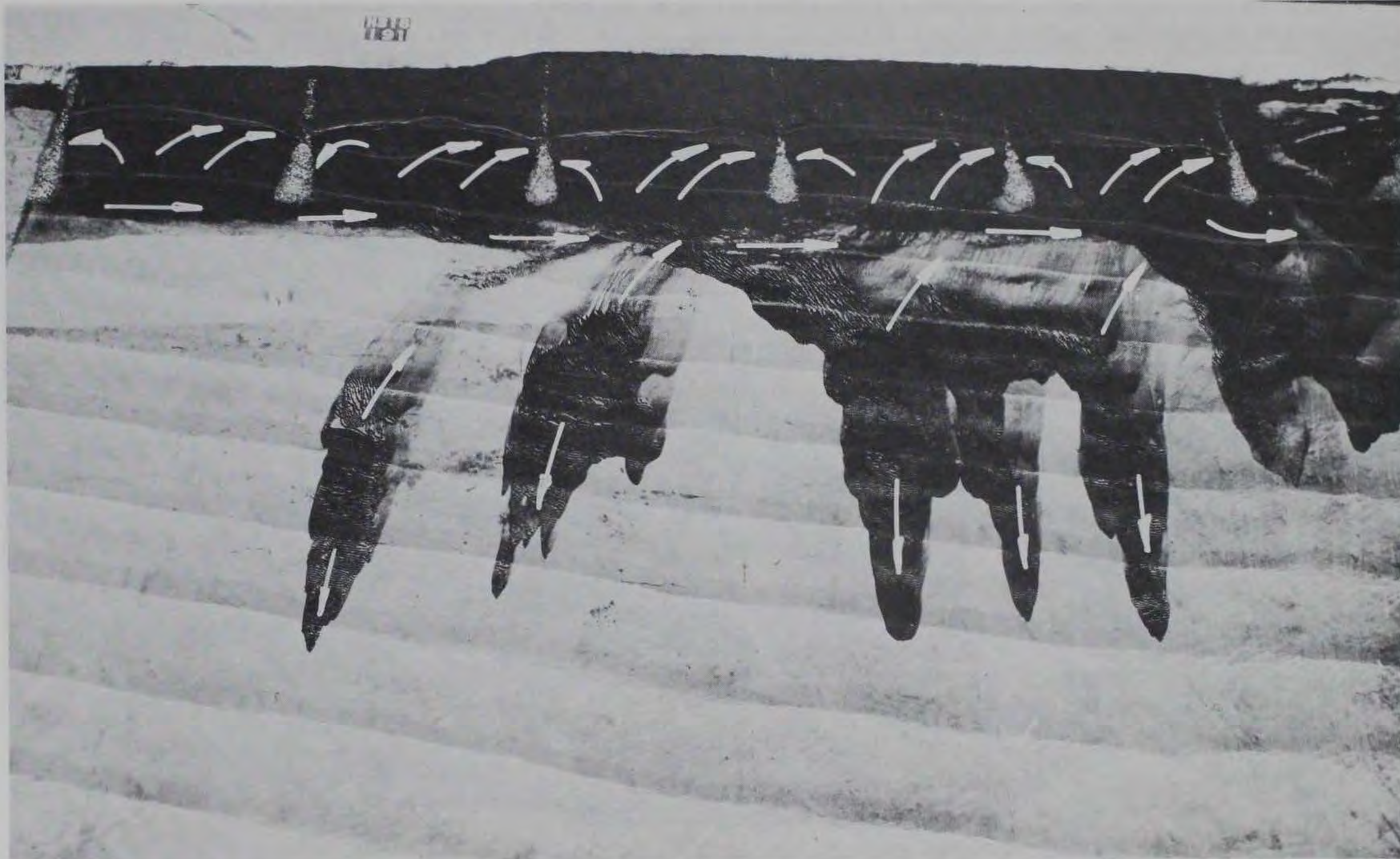


Photo 71. Movable-bed tracer deposits for Plan 9 resulting from 11-sec, 10-ft waves from west at mllw

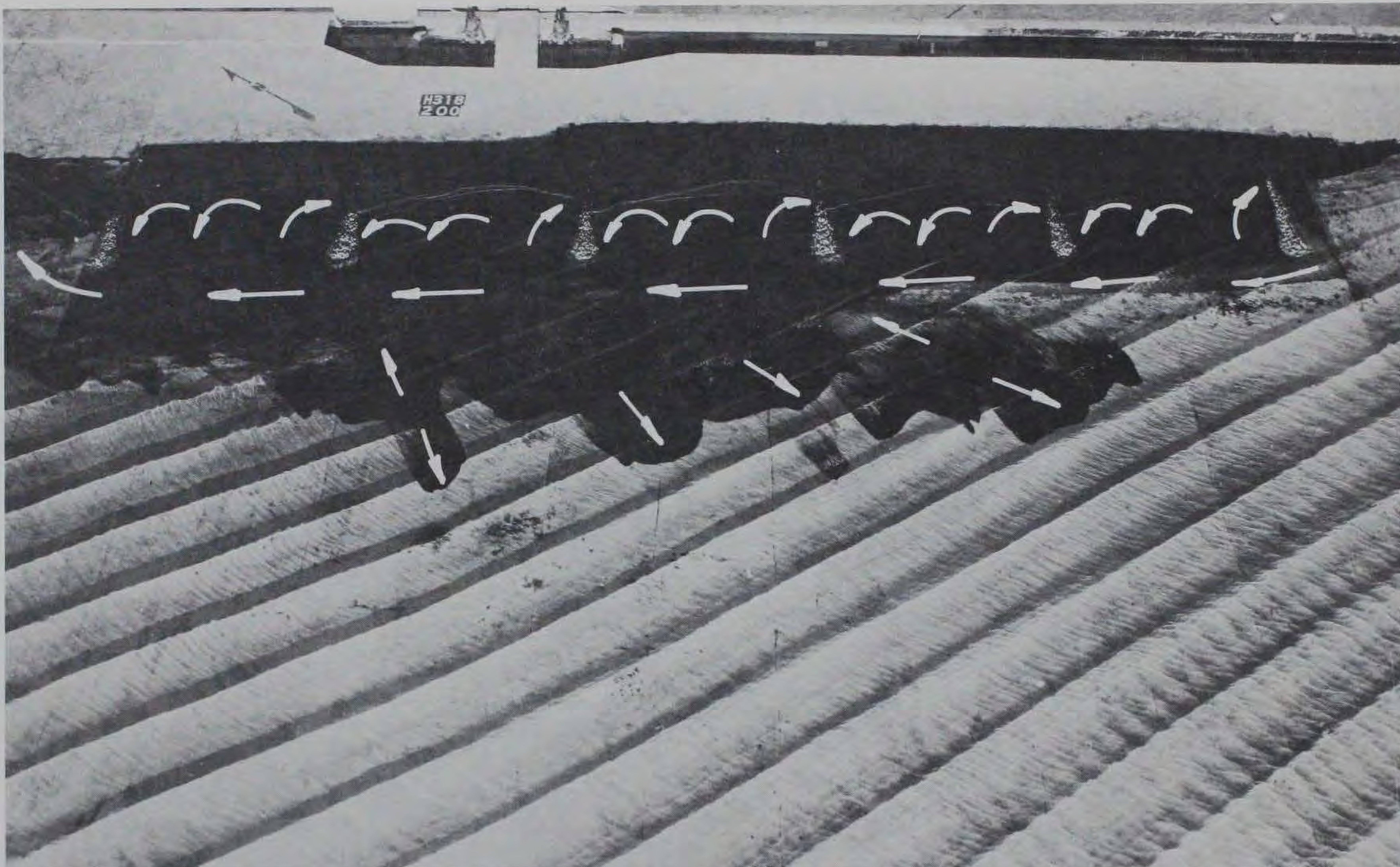


Photo 72. Movable-bed tracer deposits for Plan 9 resulting from 9-sec, 10-ft waves from south at mllw (without coal feed)

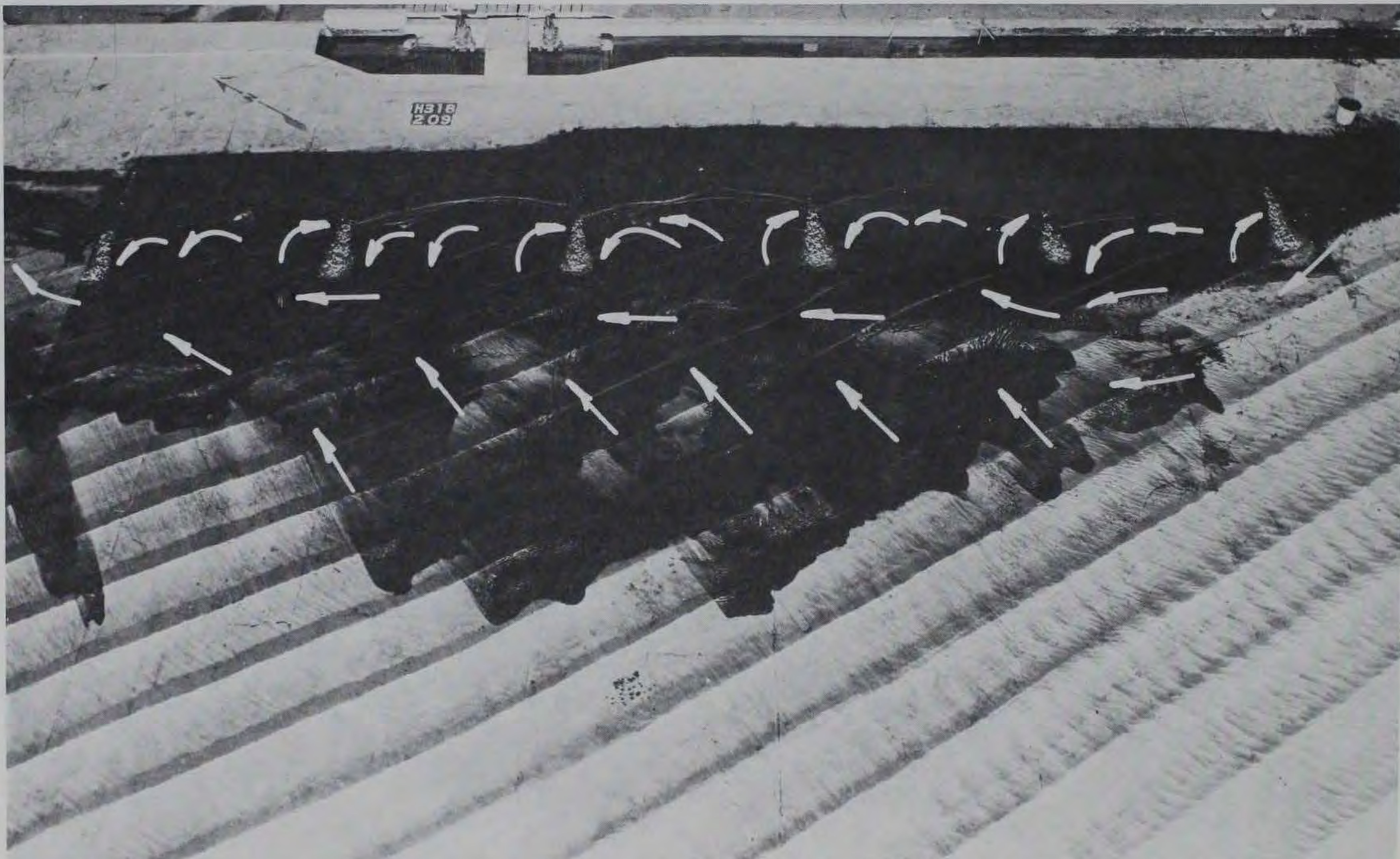


Photo 73. Movable-bed tracer deposits for Plan 9 resulting from 9-sec, 10-ft waves from south at mllw (with coal feed)

Photo 74. Movable-bed tracer deposits for Plan 6 resulting from 9-sec, 10-ft waves from south at mllw (with coal feed)

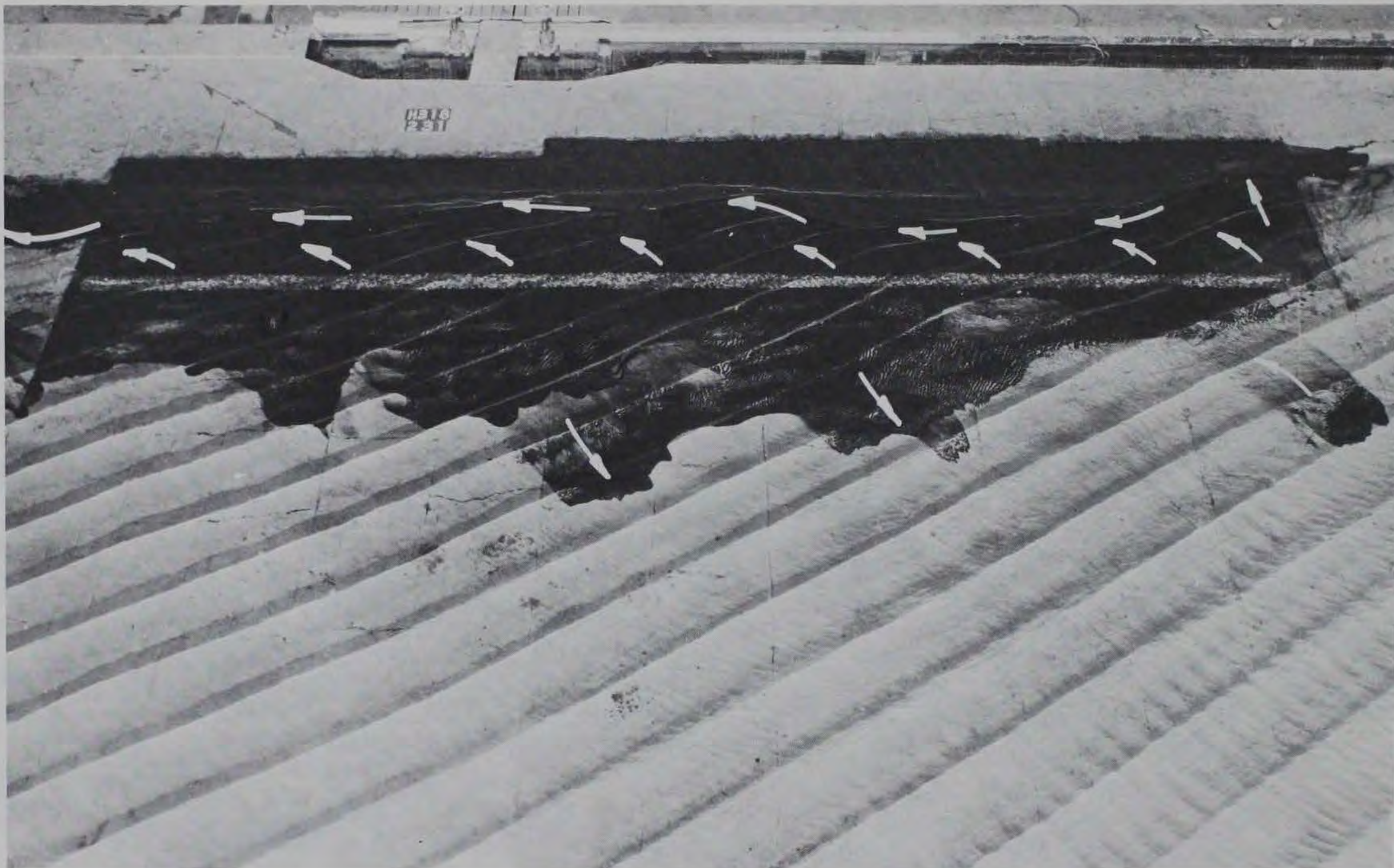


Photo 75. Movable-bed tracer deposits for Plan 13 resulting from 9-sec, 10-ft waves from south at mllw (without coal feed)

Photo 76. Movable-bed tracer deposits for Plan 13 resulting from 9-sec, 10-ft waves from south at mllw (with coal feed)

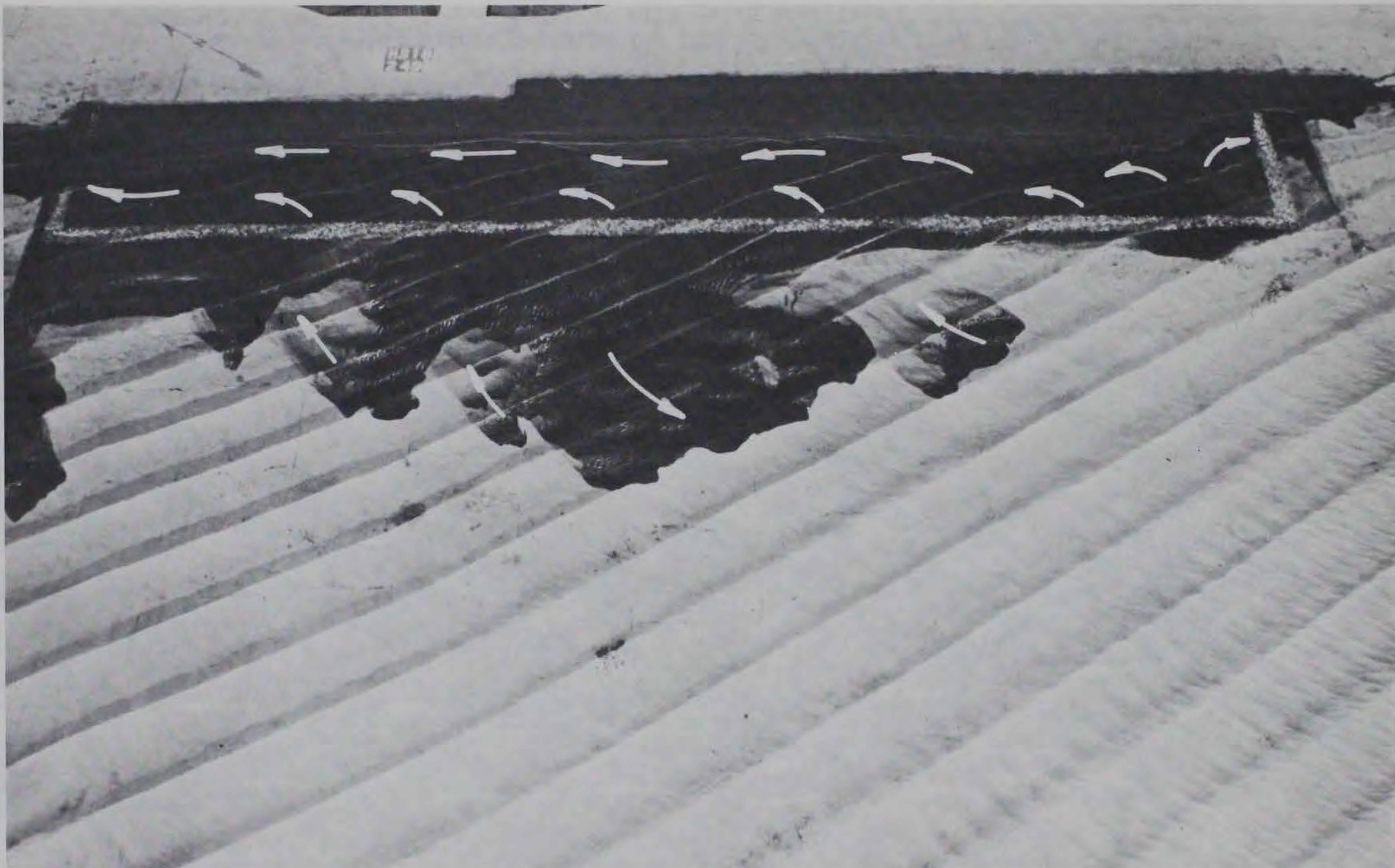


Photo 77. Movable-bed tracer deposits for Plan 14 resulting from 9-sec, 10-ft waves from south at mllw (without coal feed)

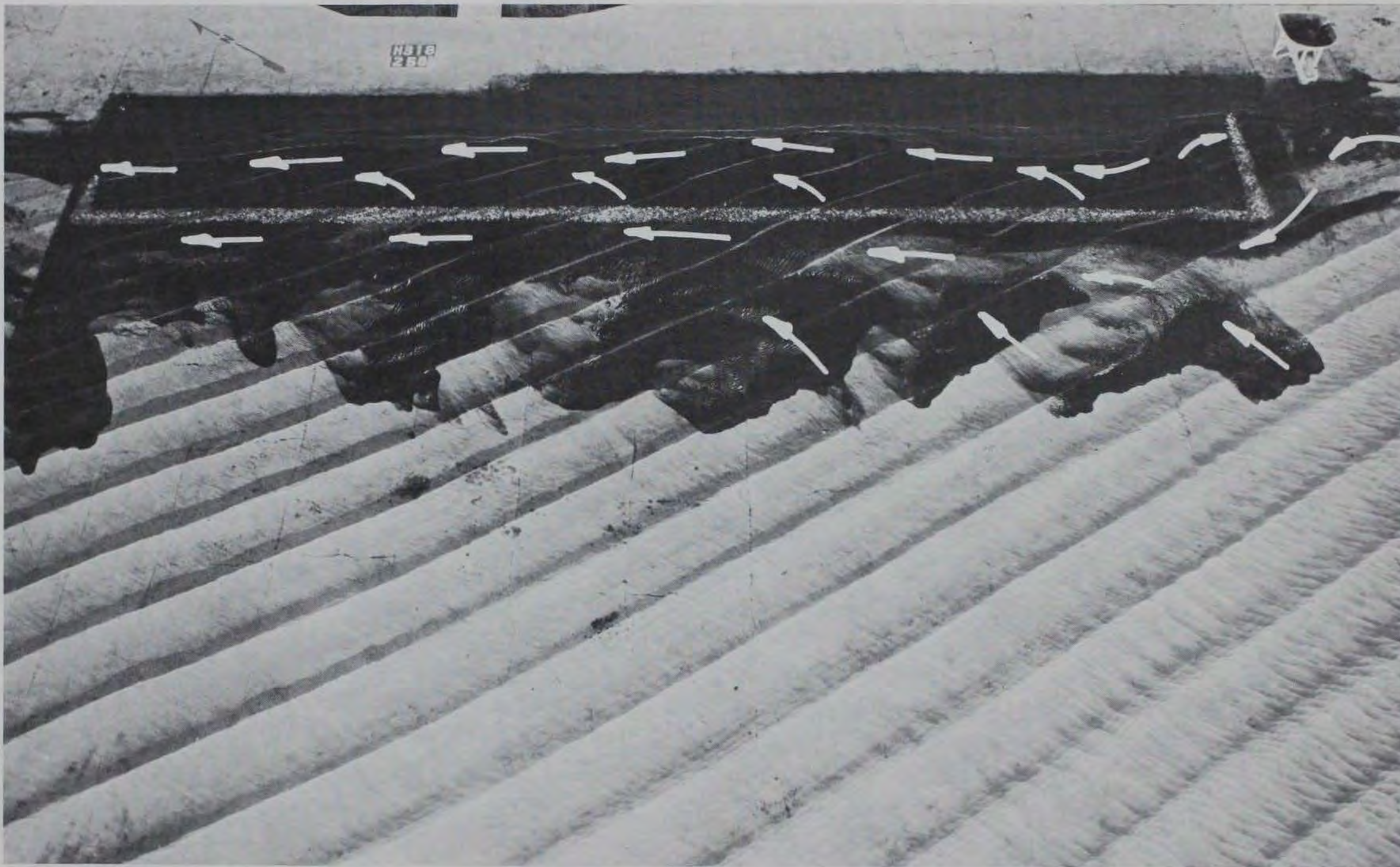


Photo 78. Movable-bed tracer deposits for Plan 14 resulting from 9-sec, 10-ft waves from south at mllw (with coal feed)

Photo 79. Movable-bed tracer deposits for Plan 15 resulting from 9-sec, 10-ft waves from south at mllw (without coal feed)

Photo 80. Movable-bed tracer deposits for Plan 15 resulting from 9-sec, 10-ft waves from south at mllw (with coal feed)

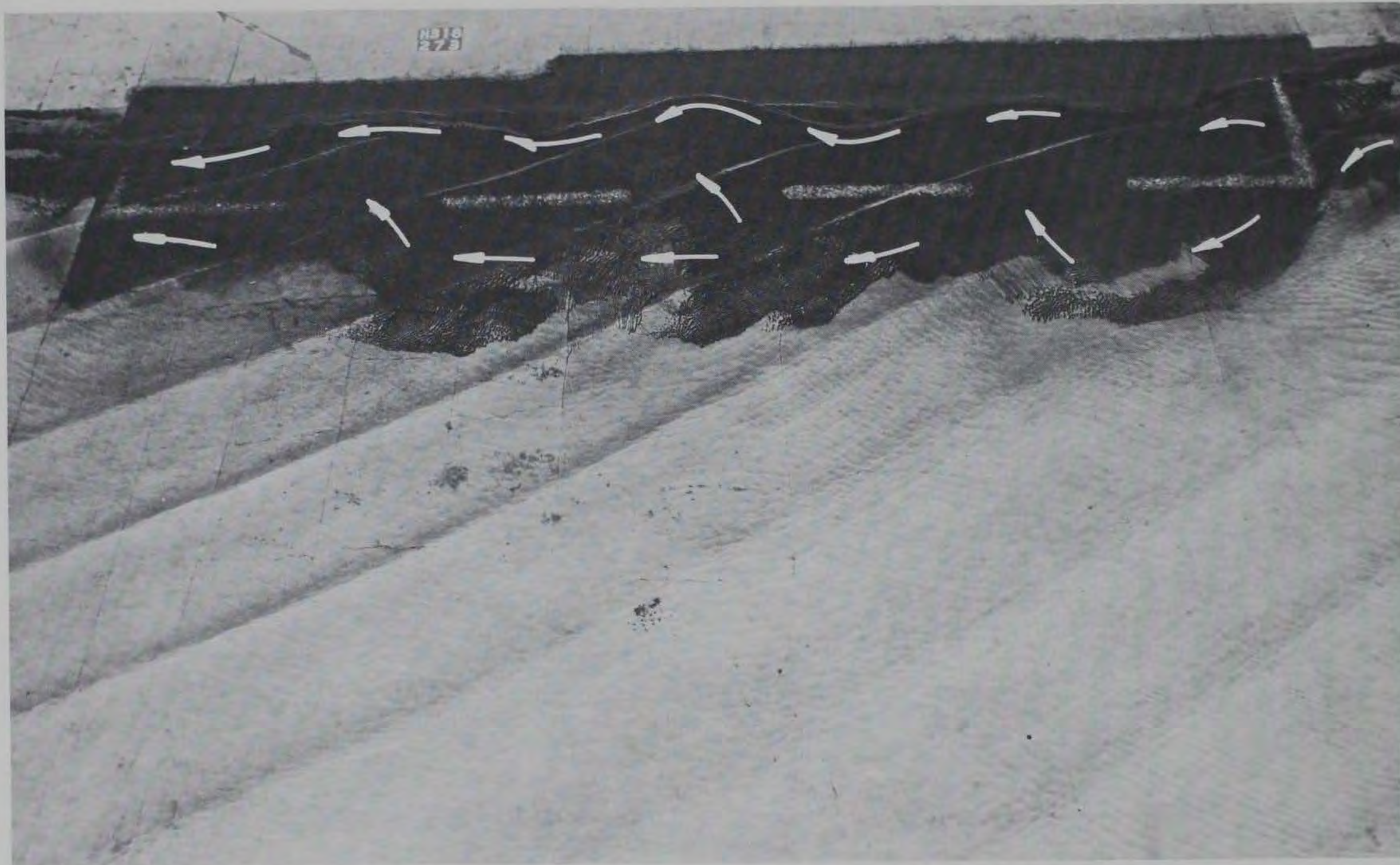


Photo 81. Movable-bed tracer deposits for Plan 15 resulting from 17-sec, 6-ft waves from south at mhhw (with coal feed)

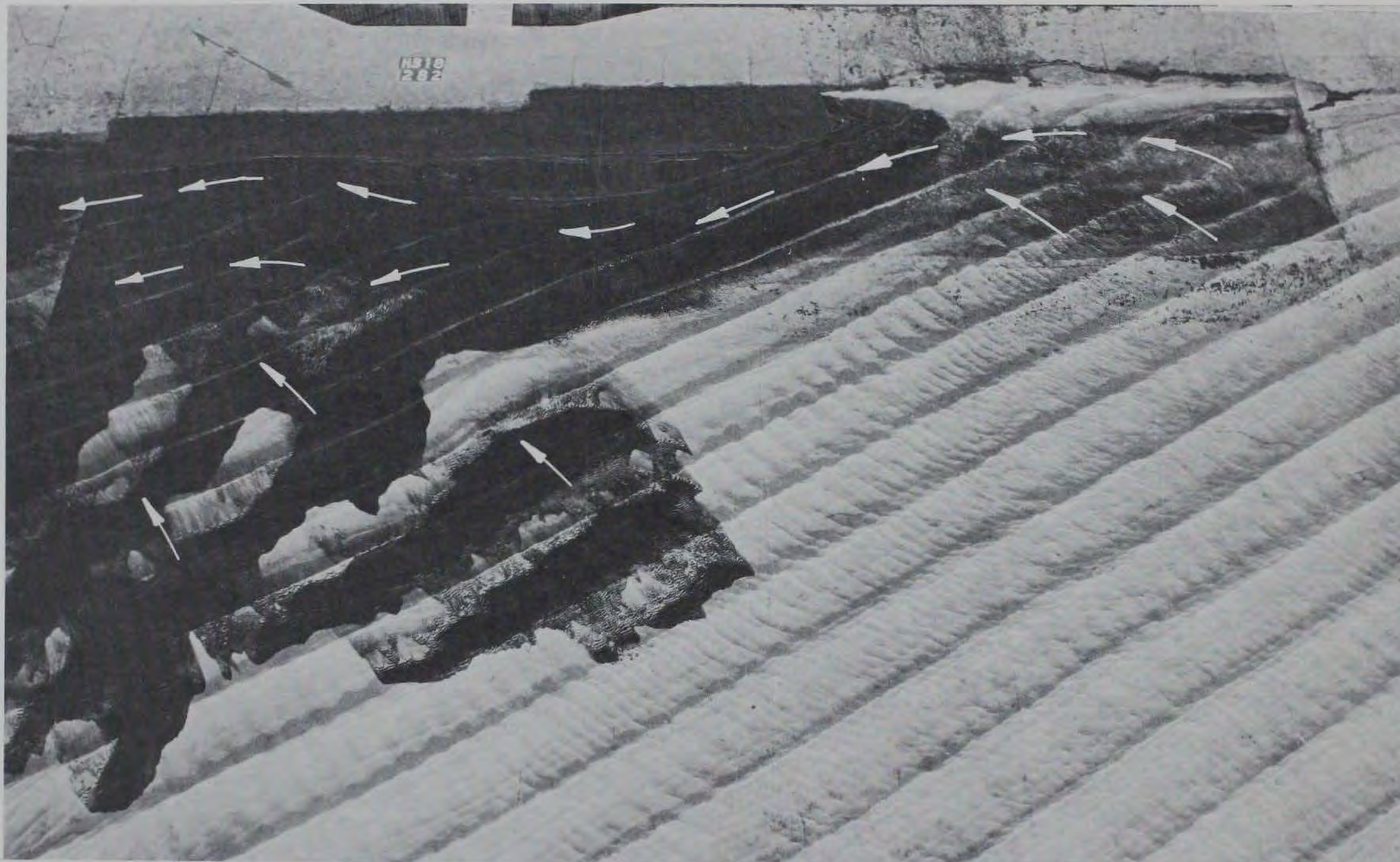


Photo 82. Movable-bed tracer deposits for Plan 16 resulting from 9-sec, 10-ft waves from south at mhhw (after 6 hr)

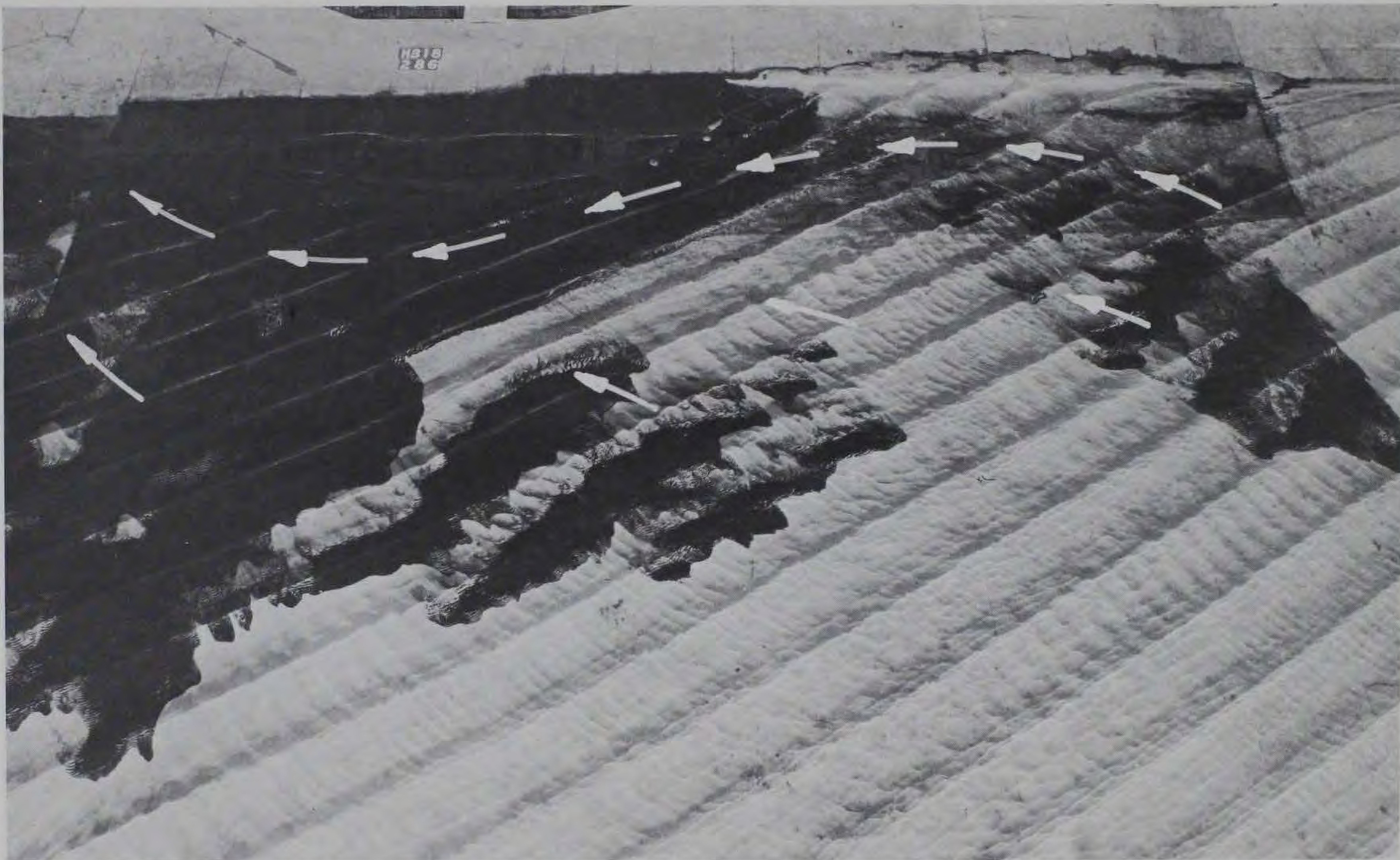


Photo 83. Movable-bed tracer deposits for Plan 16 resulting from 9-sec, 10-ft waves from south at mhhw (tested for 6 hr with initial beach fill)

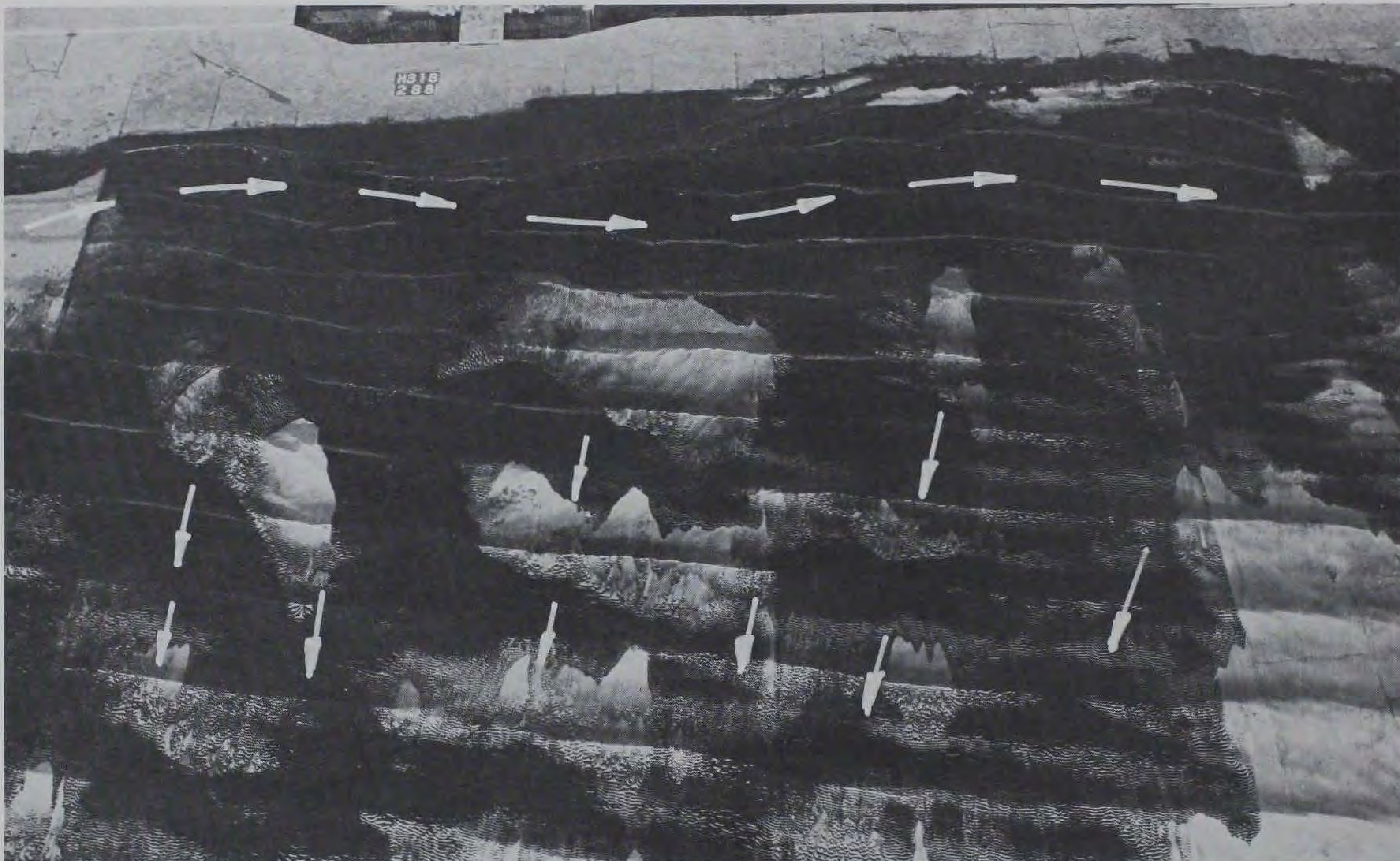


Photo 84. Movable-bed tracer deposits for Plan 16 resulting from 11-sec, 10-ft waves from west at mhhw (tested for 6 hr with final beach of previous test)

Photo 85. Movable-bed tracer deposits for Plan 16 resulting from 11-sec, 10-ft waves from west at mhhw (tested for 6 hr with initial beach fill)

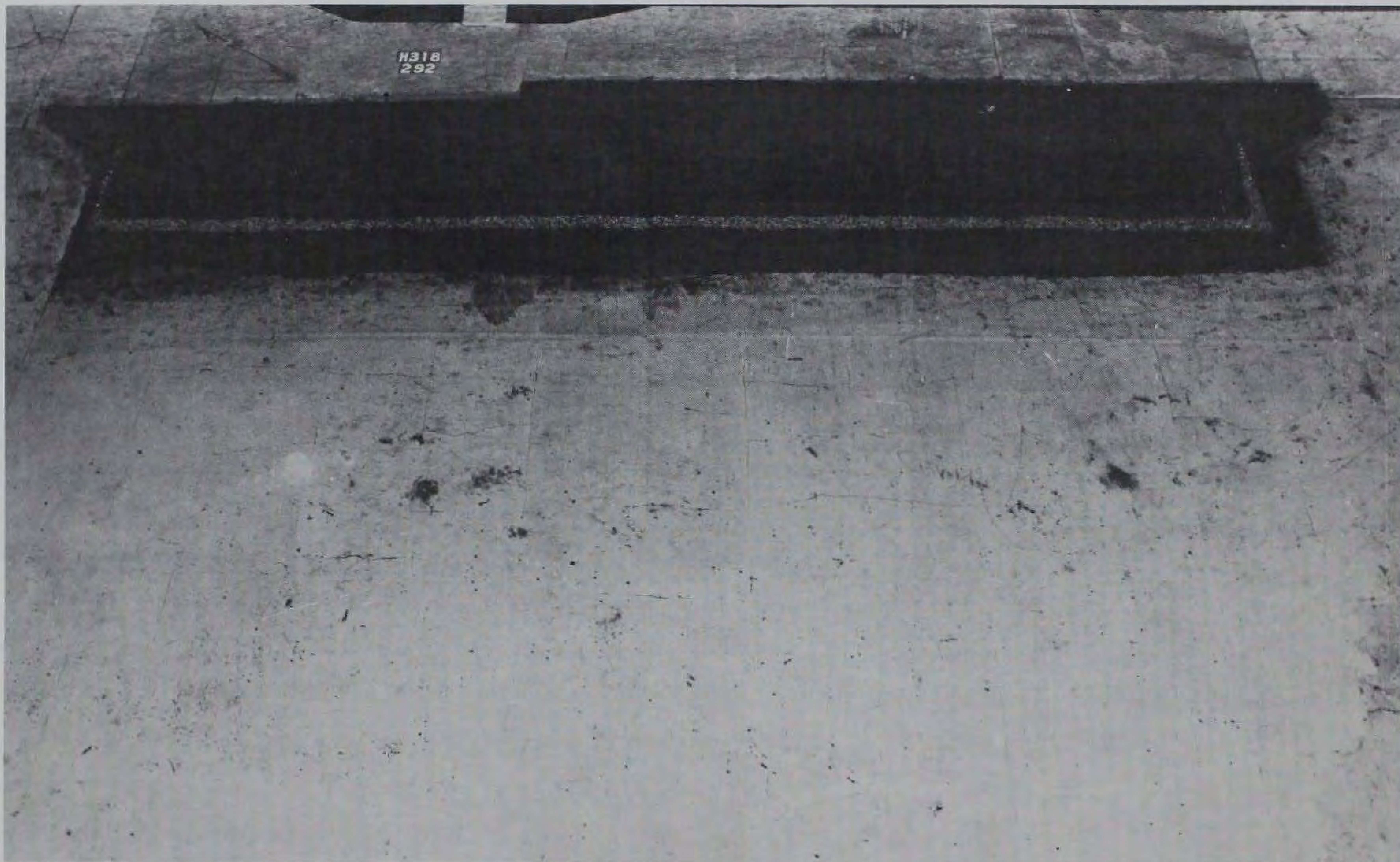


Photo 86. Placement of movable-bed tracer for Plan 14 before testing

Photo 87. Movable-bed tracer deposits for Plan 14 resulting from 11-sec, 10-ft waves from west at mllw (extended test wave duration)

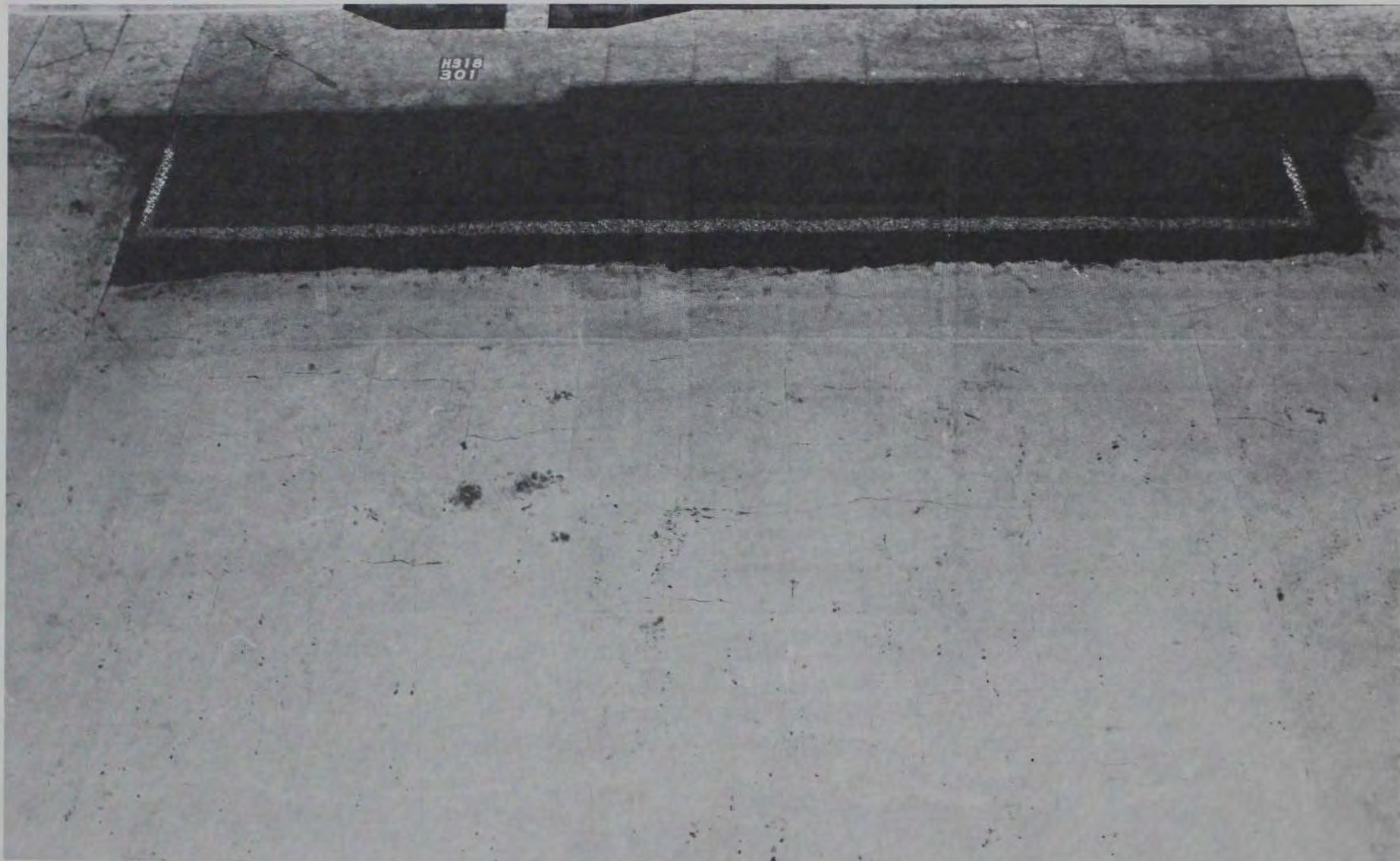


Photo 88. Placement of movable-bed tracer for Plan 14A before testing

Photo 89. Movable-bed tracer deposits for Plan 14A resulting from 11-sec, 10-ft waves from west at mllw (extended test wave duration)

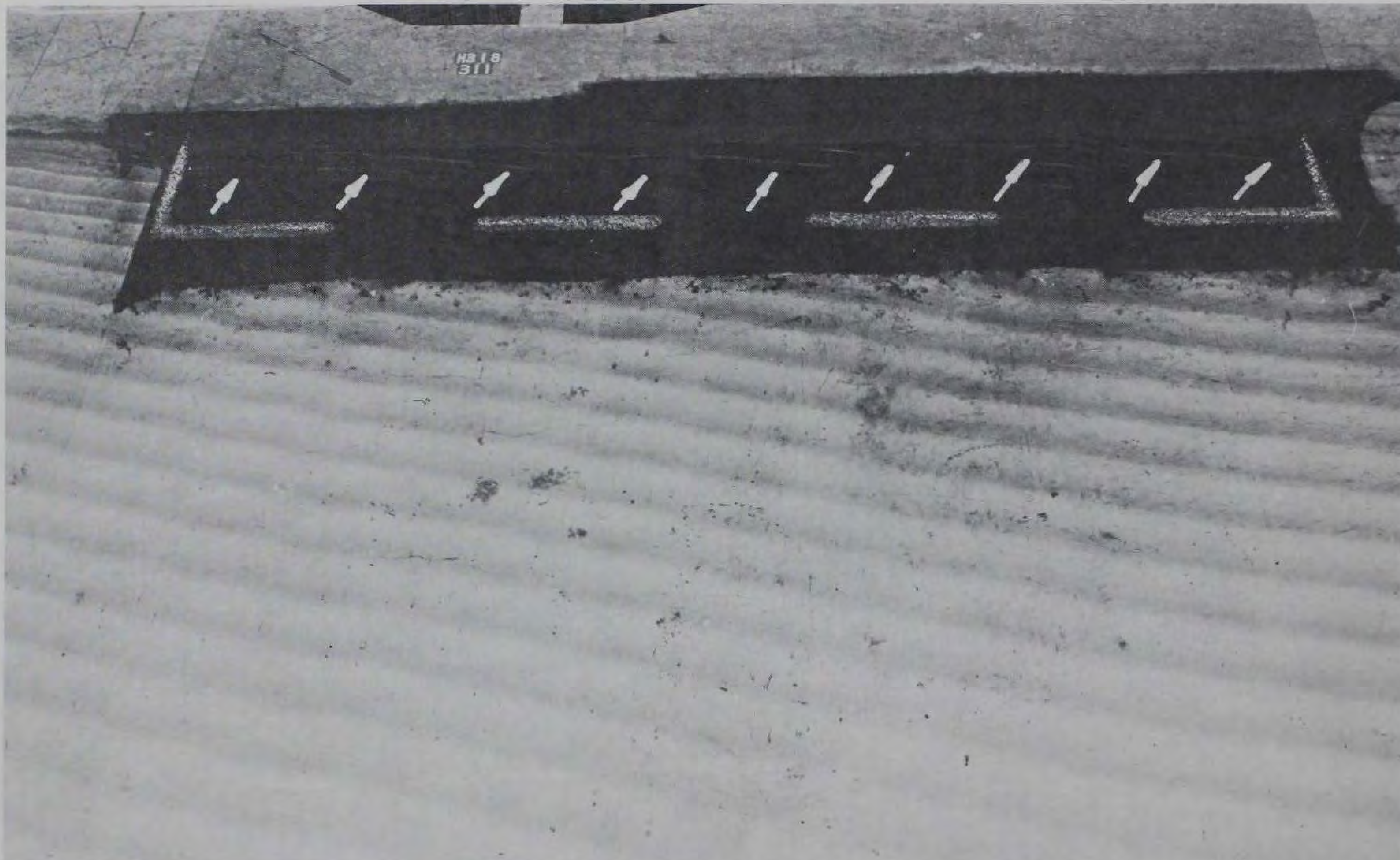


Photo 90. Movable-bed tracer deposits for Plan 17 resulting from 7-sec, 4-ft waves from west at mhhw (extended test wave duration)

Photo 91. Movable-bed tracer deposits for Plan 17 resulting from 7-sec, 10-ft waves from west at mhhw (extended test wave duration)

Photo 92. Movable-bed tracer deposits for Plan 17 resulting from 11-sec, 4-ft waves from west at mhhw (extended test wave duration)

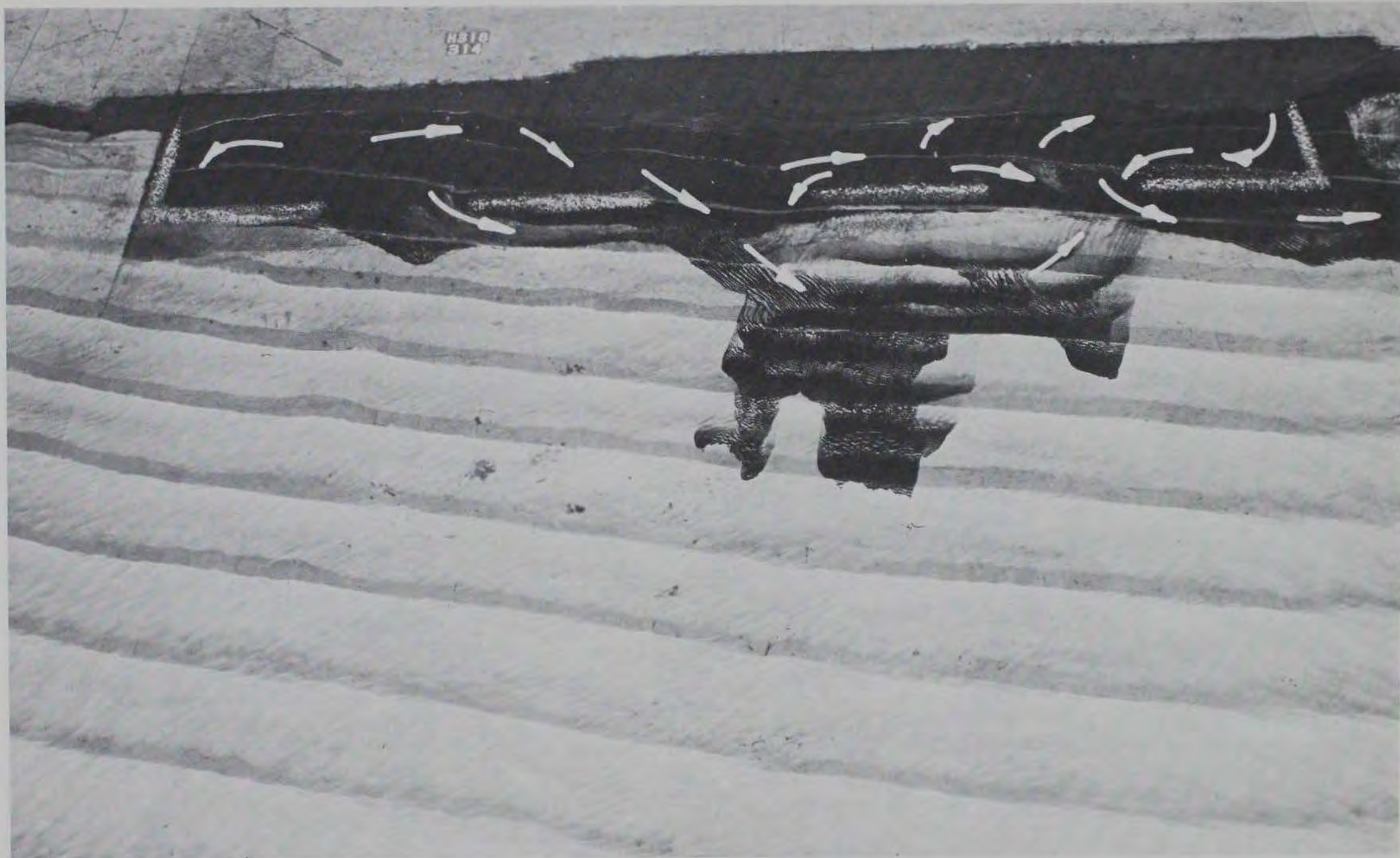


Photo 93. Movable-bed tracer deposits for Plan 17 resulting from 11-sec, 10-ft waves from west at mhhw (extended test wave duration)

Photo 94. Movable-bed tracer deposits for Plan 17 resulting from 17-sec, 6-ft waves from west at mhhw (extended test wave duration)

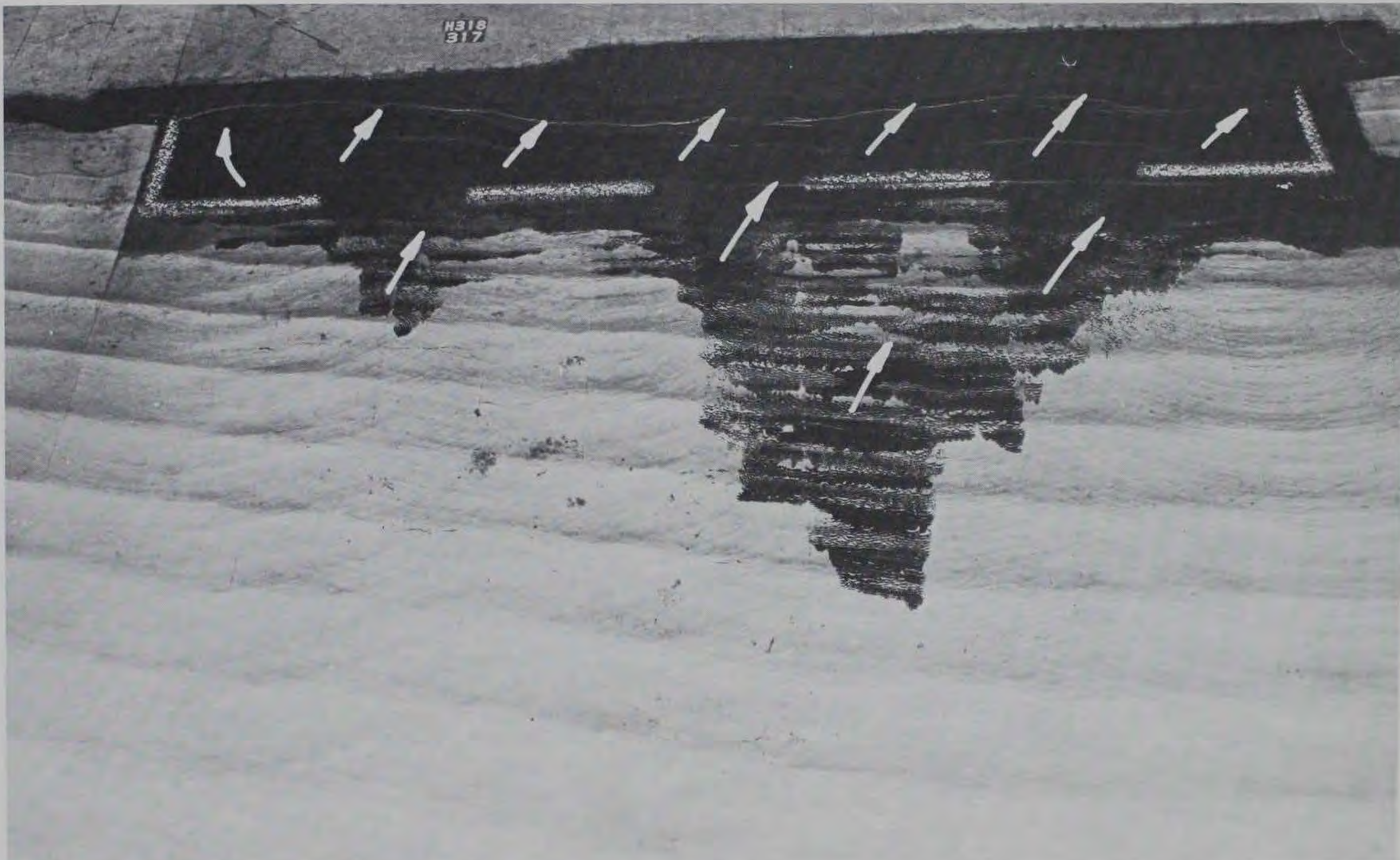


Photo 95. Movable-bed tracer deposits for Plan 17 resulting from 11-sec, 4-ft waves from west at mllw (extended test wave duration)

Photo 96. Movable-bed tracer deposits for Plan 17 resulting from 11-sec, 10-ft waves from west at mllw (extended test wave duration)

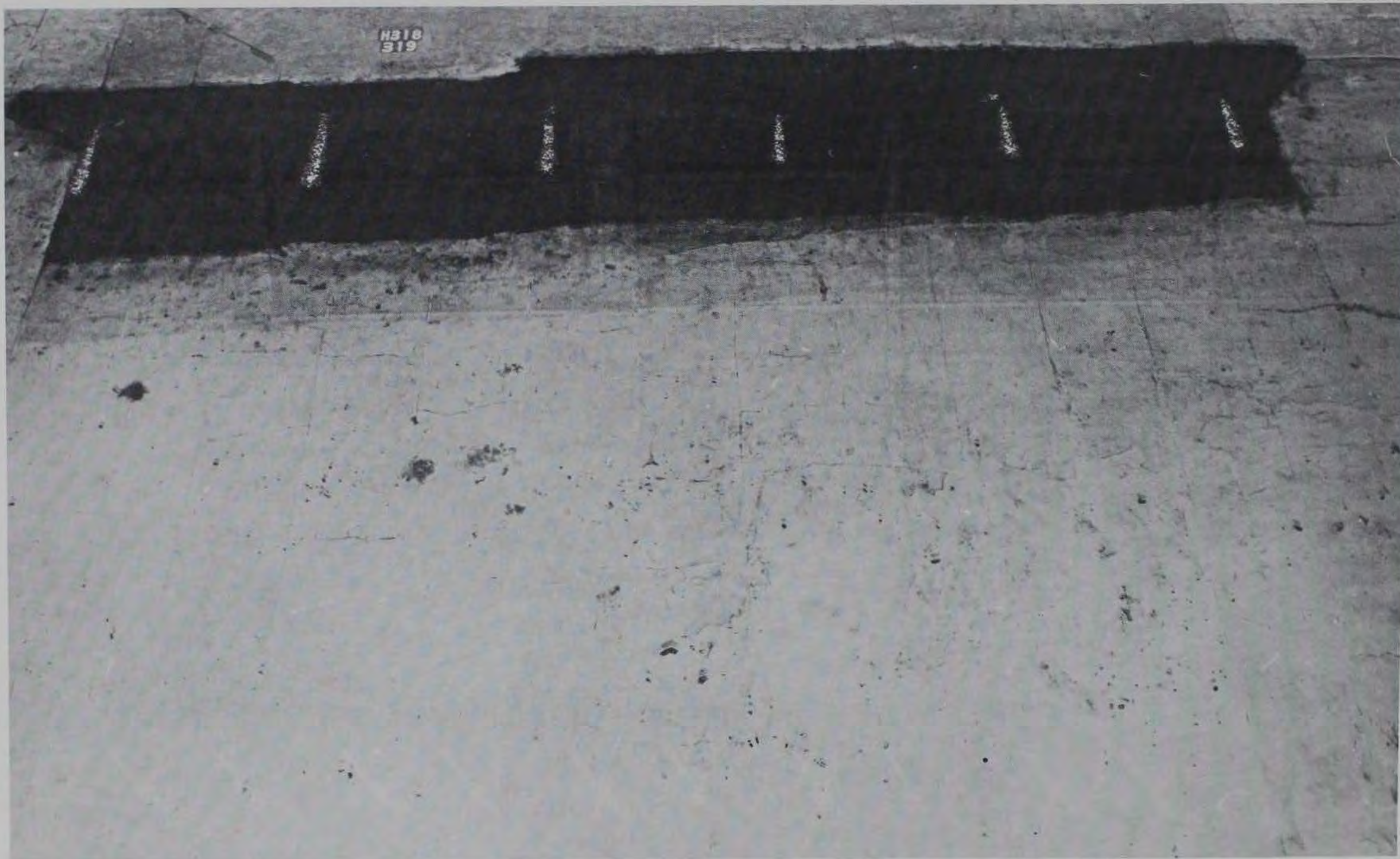


Photo 97. Placement of movable-bed tracer for Plan 18 before testing

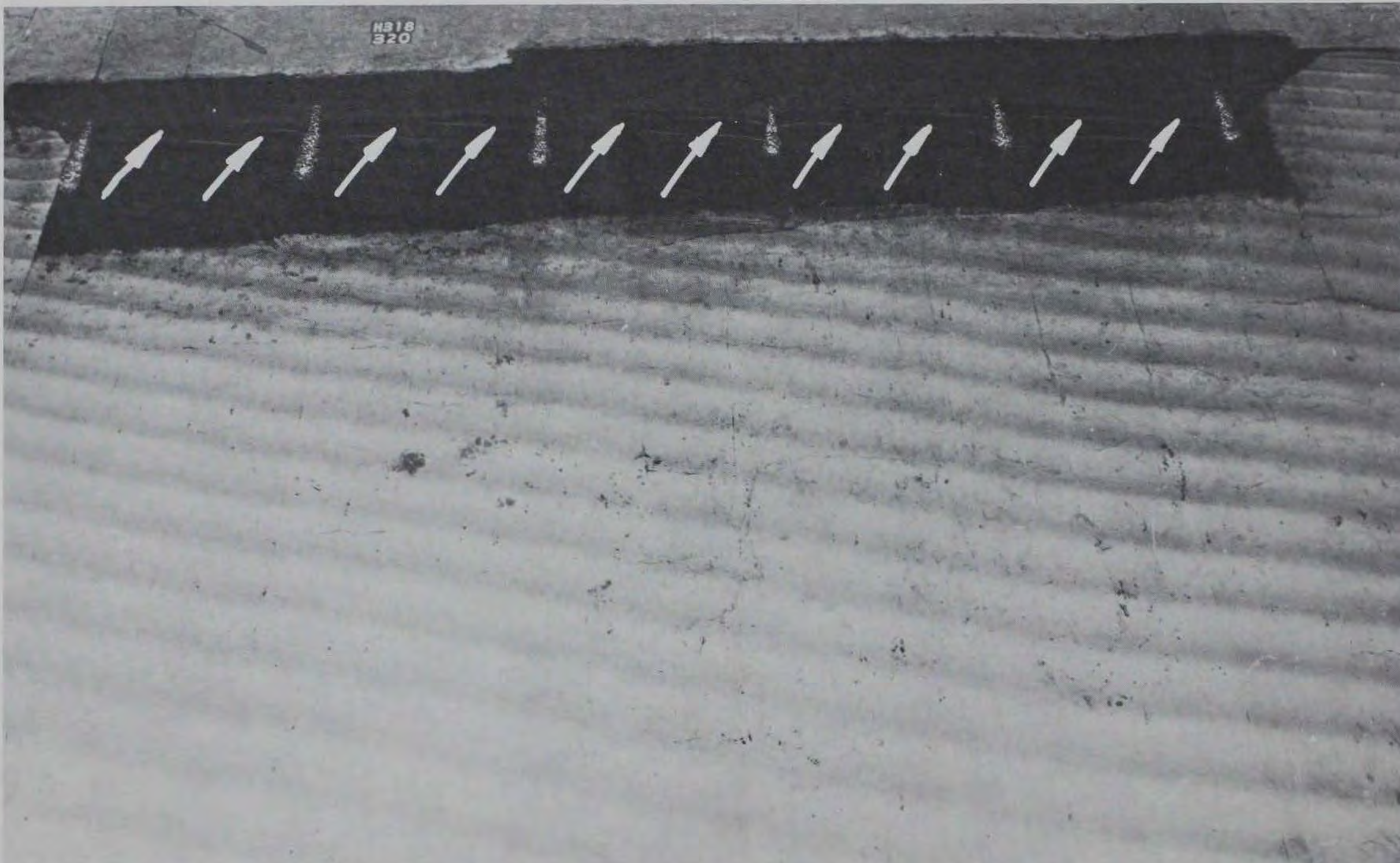


Photo 98. Movable-bed tracer deposits for Plan 18 resulting from 7-sec, 4-ft waves from west at mhhw (extended test wave duration)

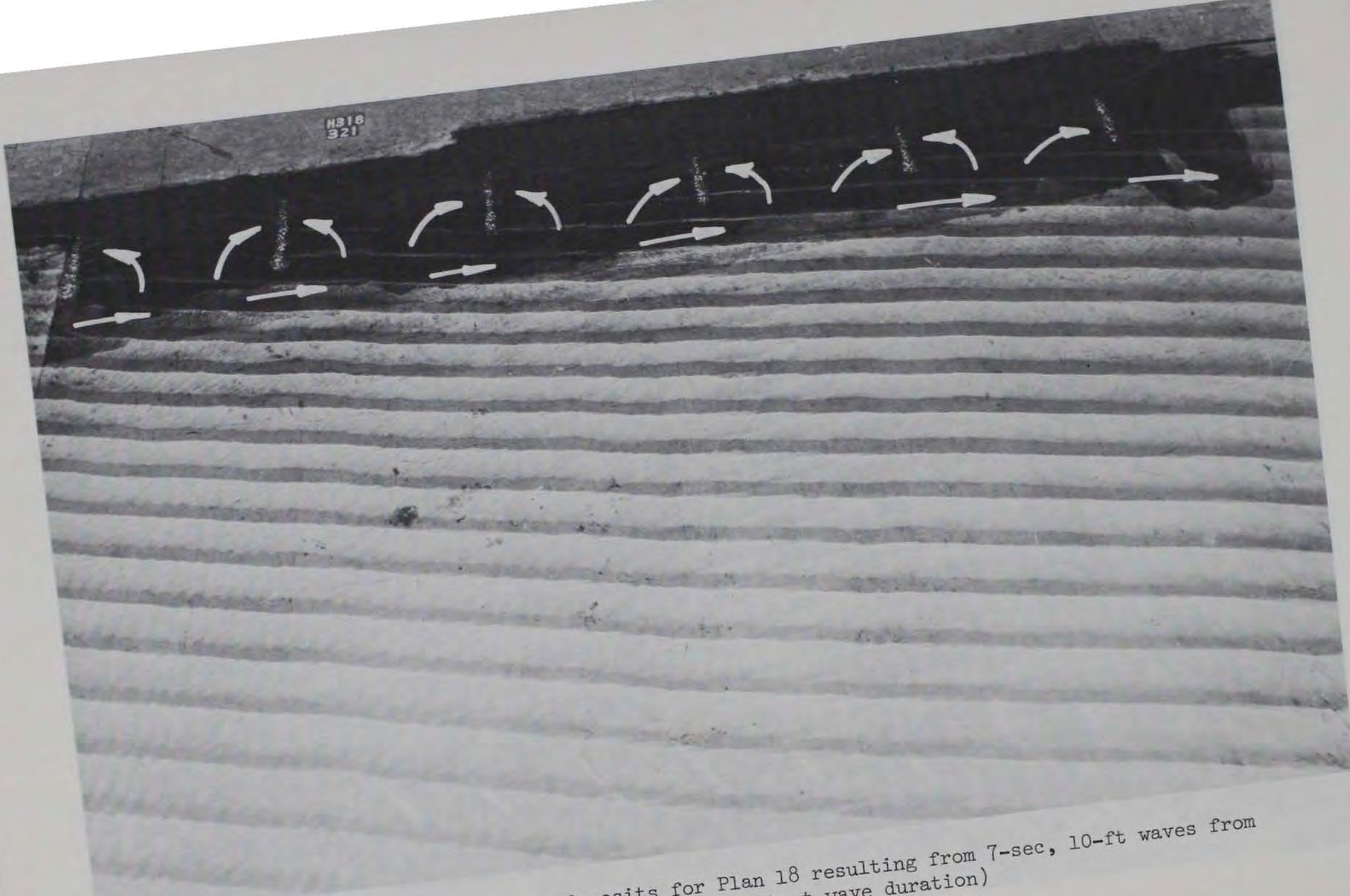


Photo 99. Movable-bed tracer deposits for Plan 18 resulting from 7-sec, 10-ft waves from west at mhhw (extended test wave duration)

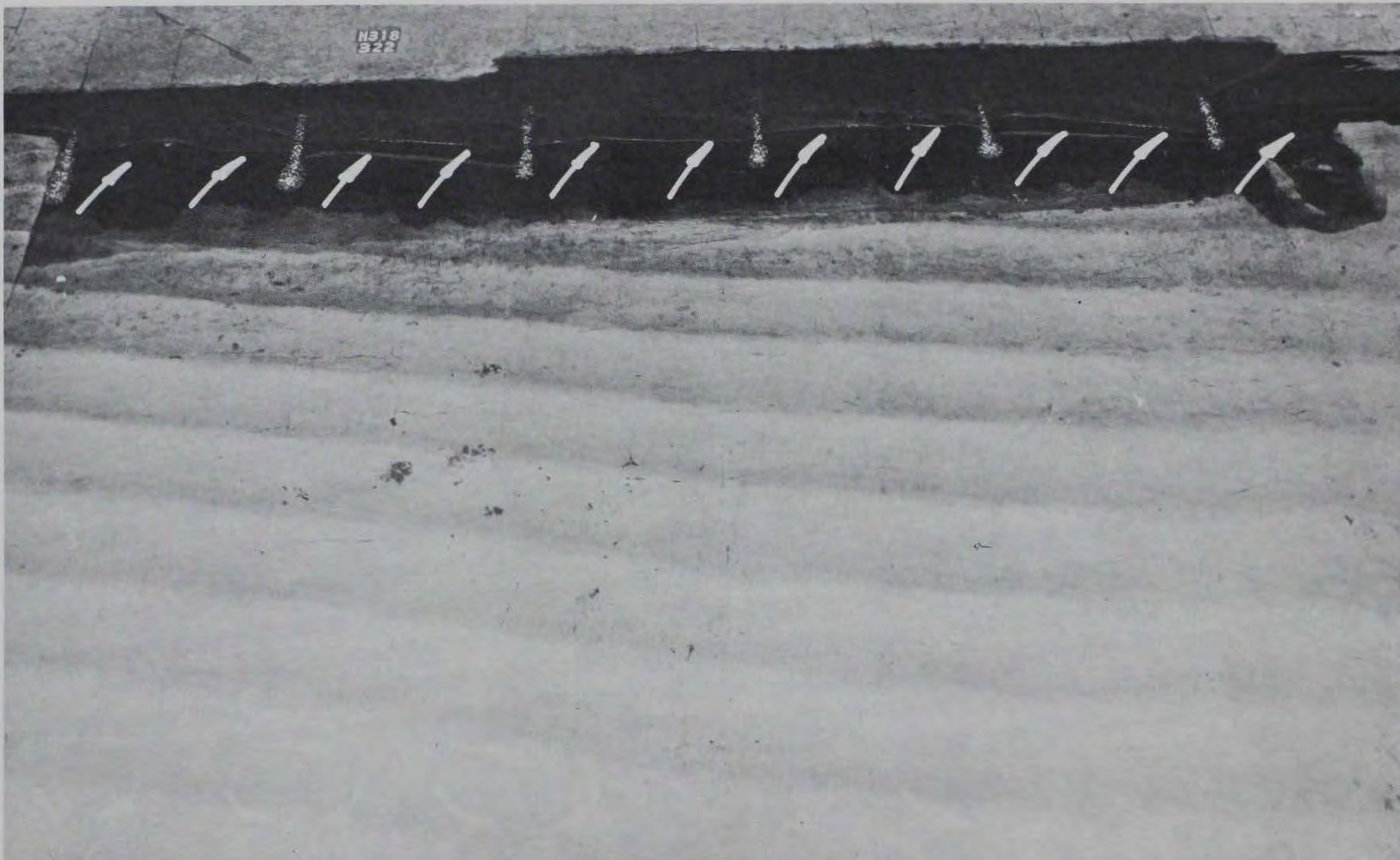


Photo 100. Movable-bed tracer deposits for Plan 18 resulting from 11-sec, 4-ft waves from west at mhhw (extended test wave duration)

Photo 101. Movable-bed tracer deposits for Plan 18 resulting from 11-sec, 10-ft waves from west at mhhw (extended test wave duration)

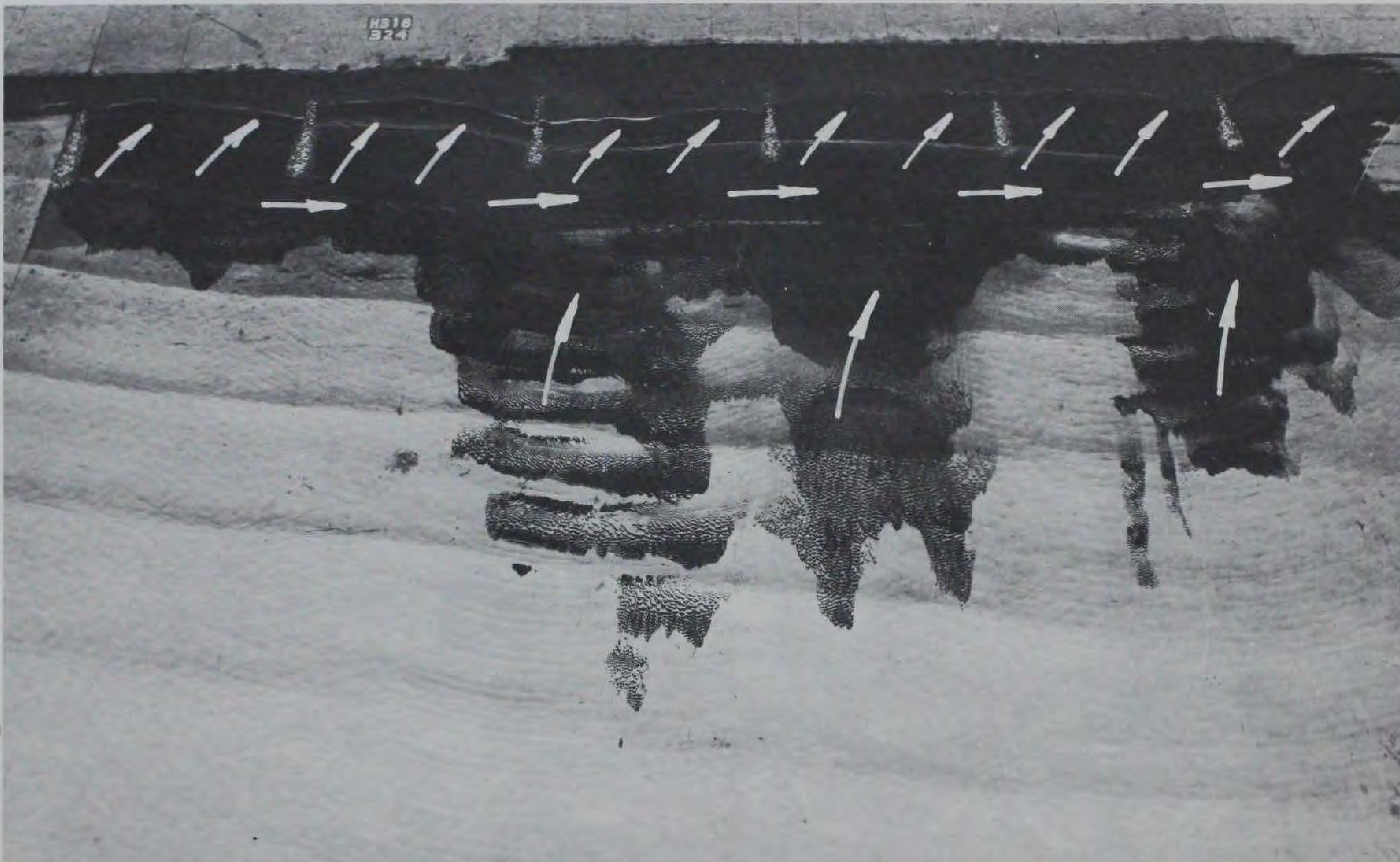


Photo 102. Movable-bed tracer deposits for Plan 18 resulting from 17-sec, 6-ft waves from west at mhhw (extended test wave duration)

Photo 103. Movable-bed tracer deposits for Plan 18 resulting from 11-sec, 4-ft waves from west at mllw (extended test wave duration)

Photo 104. Movable-bed tracer deposits for Plan 18 resulting from 11-sec, 10-ft waves from west at mllw (extended test wave duration)

Photo 105. Movable-bed tracer deposits for Plan 18 resulting from 7-sec, 4-ft waves from south at mhhw (extended test wave duration)

Photo 106. Movable-bed tracer deposits for Plan 18 resulting from 7-sec, 10-ft waves from south at mhhw (extended test wave duration)

Photo 107. Movable-bed tracer deposits for Plan 18 resulting from 9-sec, 4-ft waves from south at mhhw (extended test wave duration)

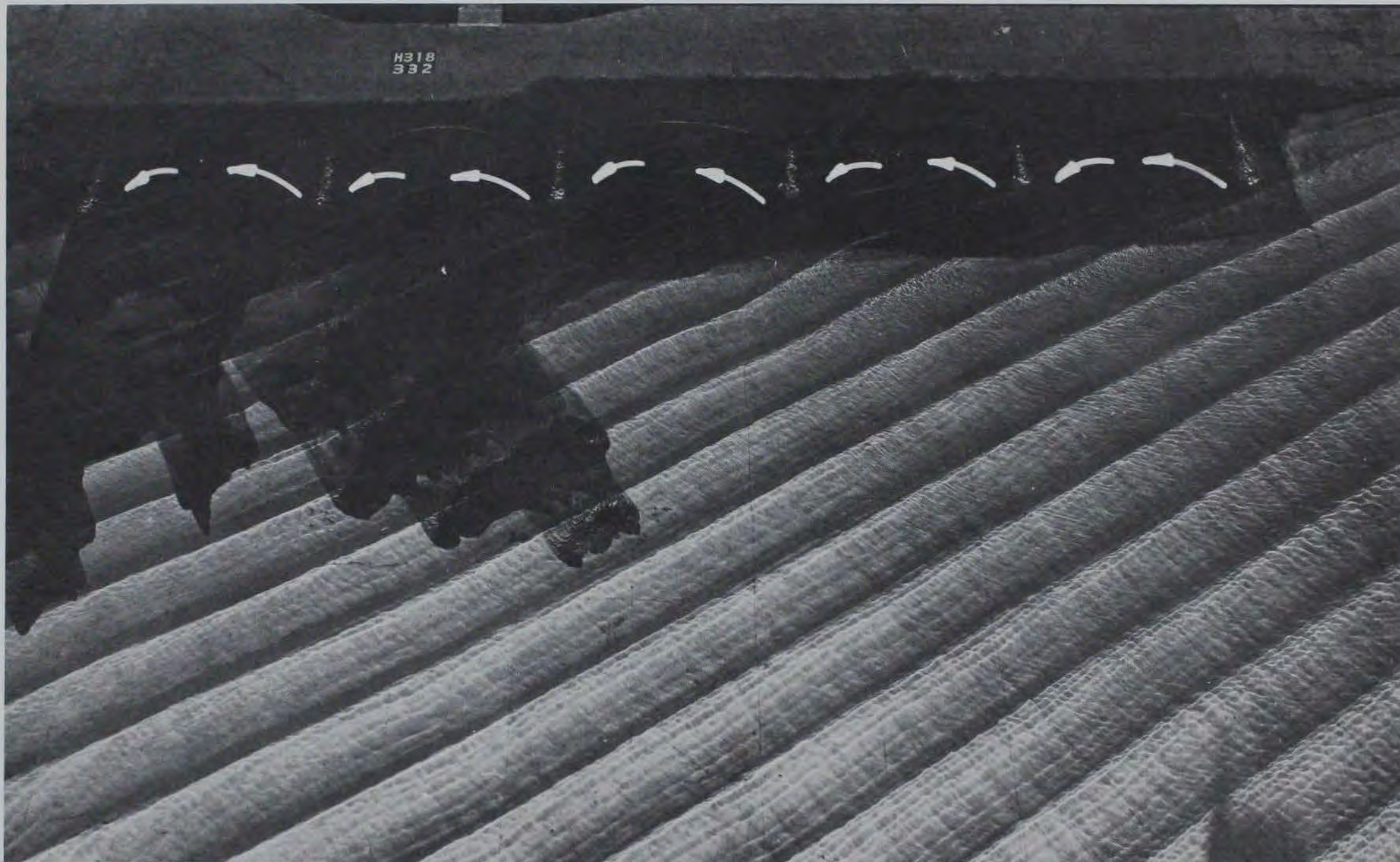


Photo 108. Movable-bed tracer deposits for Plan 18 resulting from 9-sec, 10-ft waves from south at mhhw (extended test wave duration)

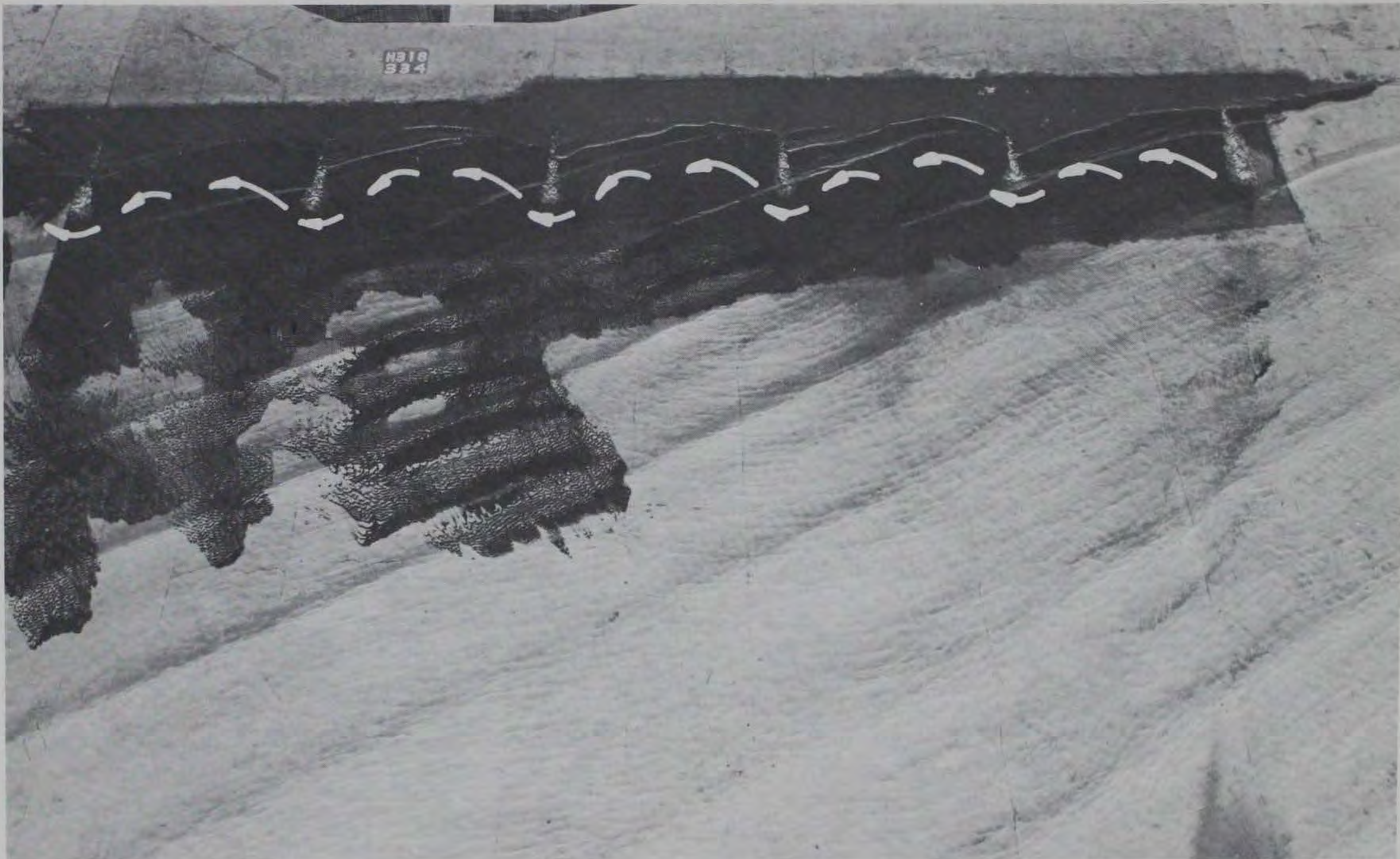


Photo 109. Movable-bed tracer deposits for Plan 18 resulting from 17-sec, 6-ft waves from south at mhhw (extended test wave duration)

Photo 110. Movable-bed tracer deposits for Plan 18 resulting from 9-sec, 4-ft waves from south at mllw (extended test wave duration)

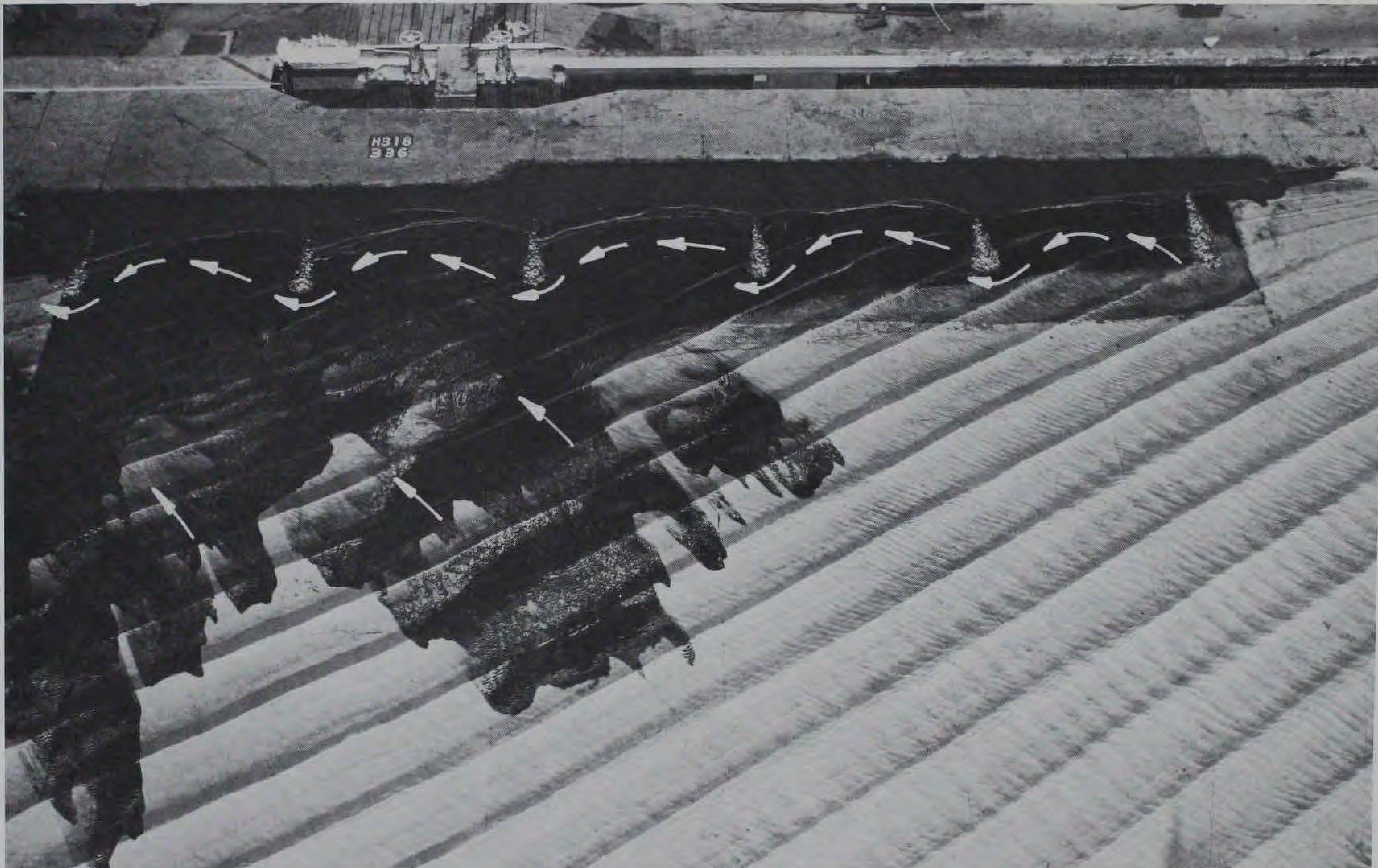


Photo 111. Movable-bed tracer deposits for Plan 18 resulting from 9-sec, 10-ft waves from south at mllw (extended test wave duration)

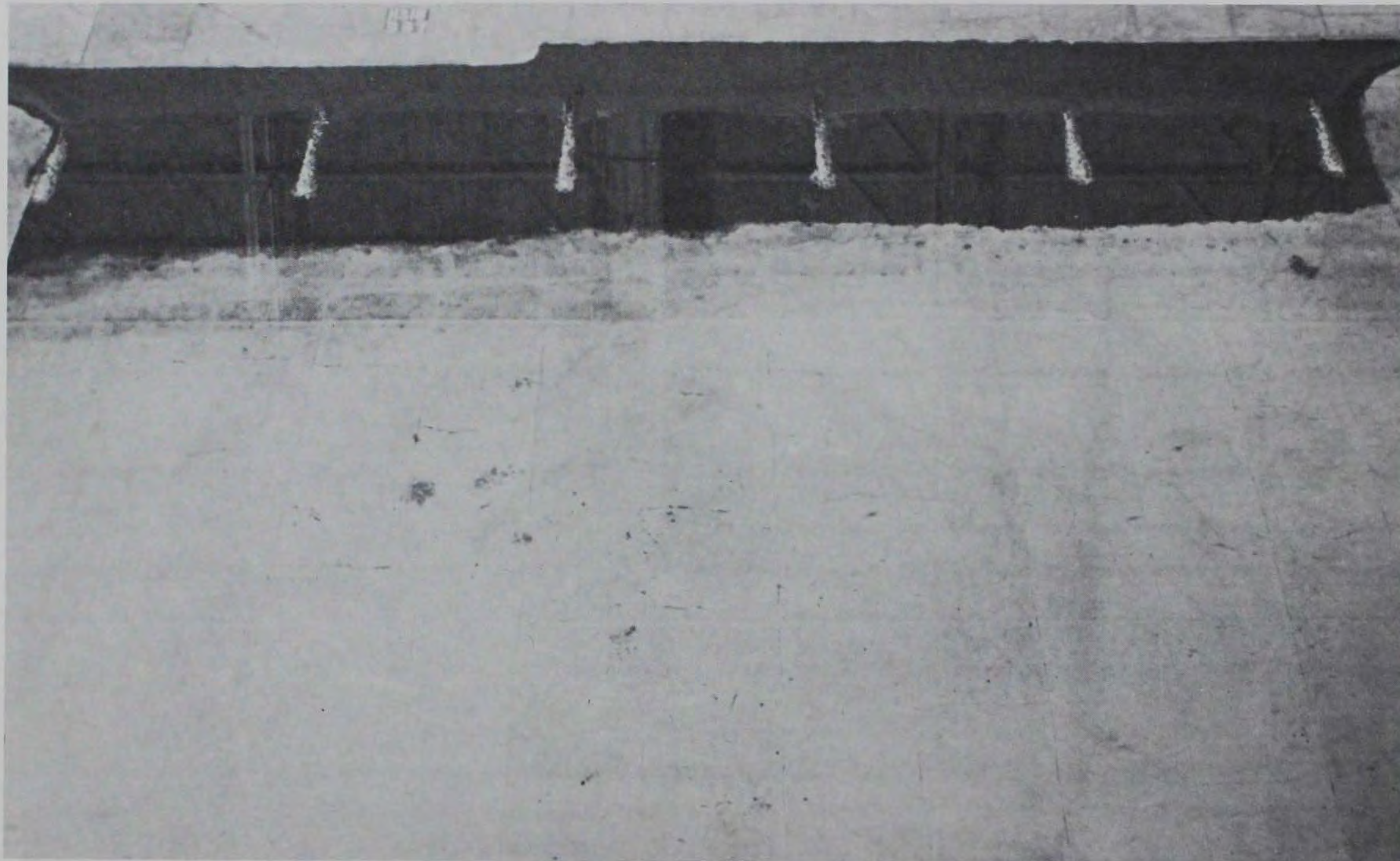


Photo 112. Placement of movable-bed tracer for Plan 9 before testing

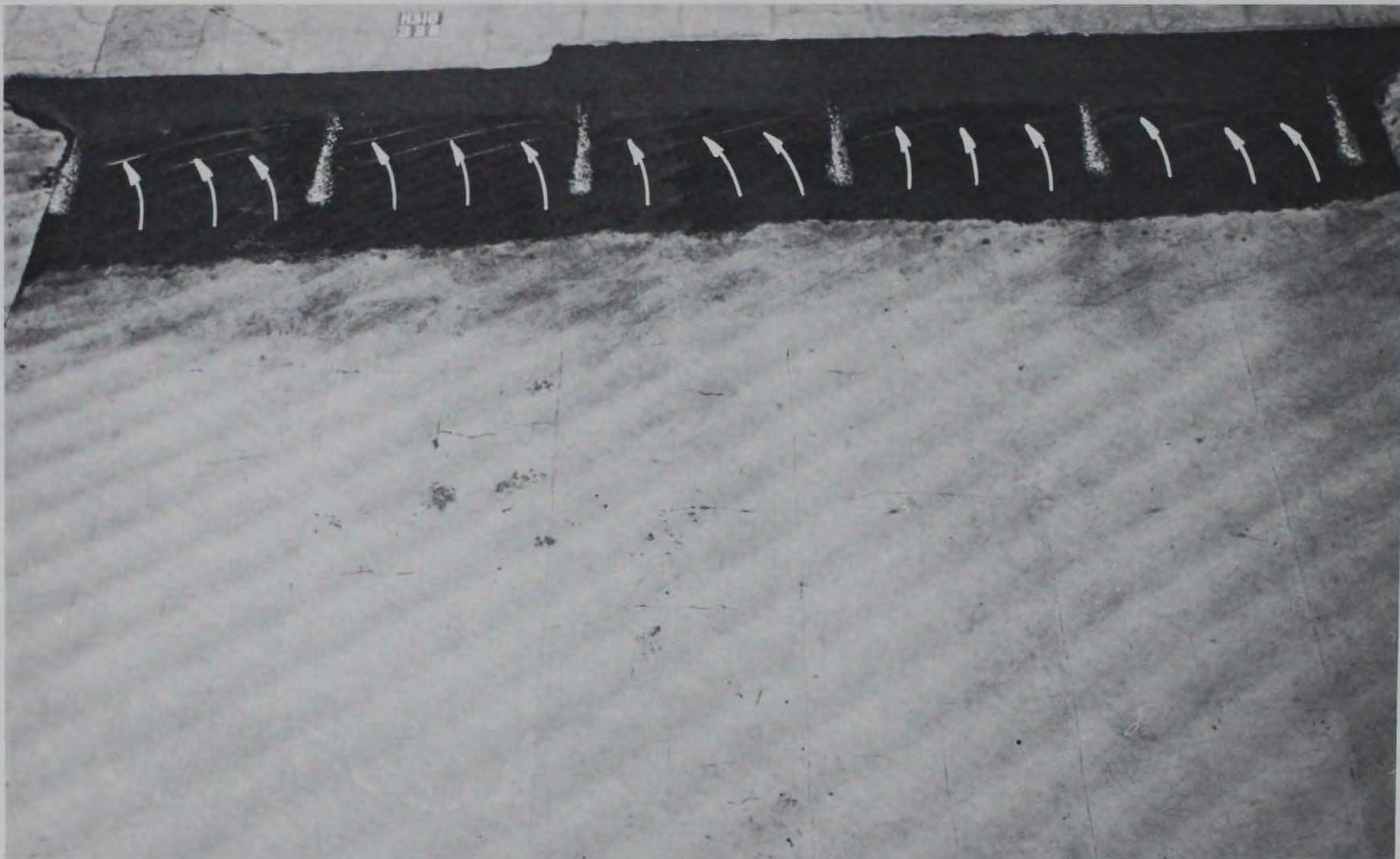


Photo 113. Movable-bed tracer deposits for Plan 9 resulting from 7-sec, 4-ft waves from south at mhhw (extended test wave duration)

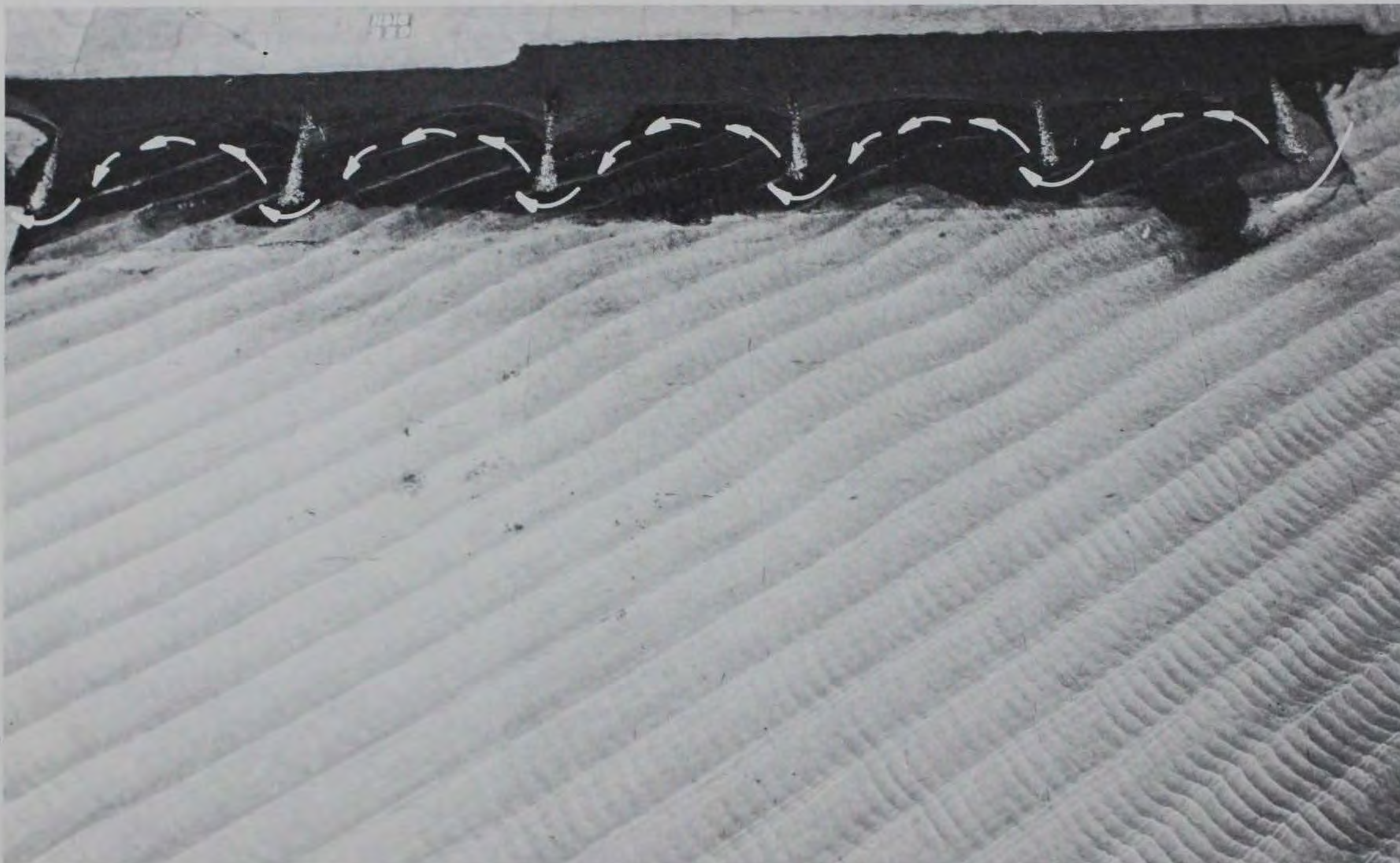


Photo 114. Movable-bed tracer deposits for Plan 9 resulting from 7-sec, 10-ft waves from south at mhhw (extended test wave duration)

Photo 115. Movable-bed tracer deposits for Plan 9 resulting from 9-sec, 4-ft waves from south at mhhw (extended test wave duration)

Photo 116. Movable-bed tracer deposits for Plan 9 resulting from 9-sec, 10-ft waves from south at mhhw (extended test wave duration)

Photo 117. Movable-bed tracer deposits for Plan 9 resulting from 17-sec, 6-ft waves from south at mhhw (extended test wave duration)

Photo 118. Movable-bed tracer deposits for Plan 9 resulting from 9-sec, 4-ft waves from south at mllw (extended test wave duration)

Photo 119. Movable-bed tracer deposits for Plan 9 resulting from 9-sec, 10-ft waves from south at mllw (extended test wave duration)

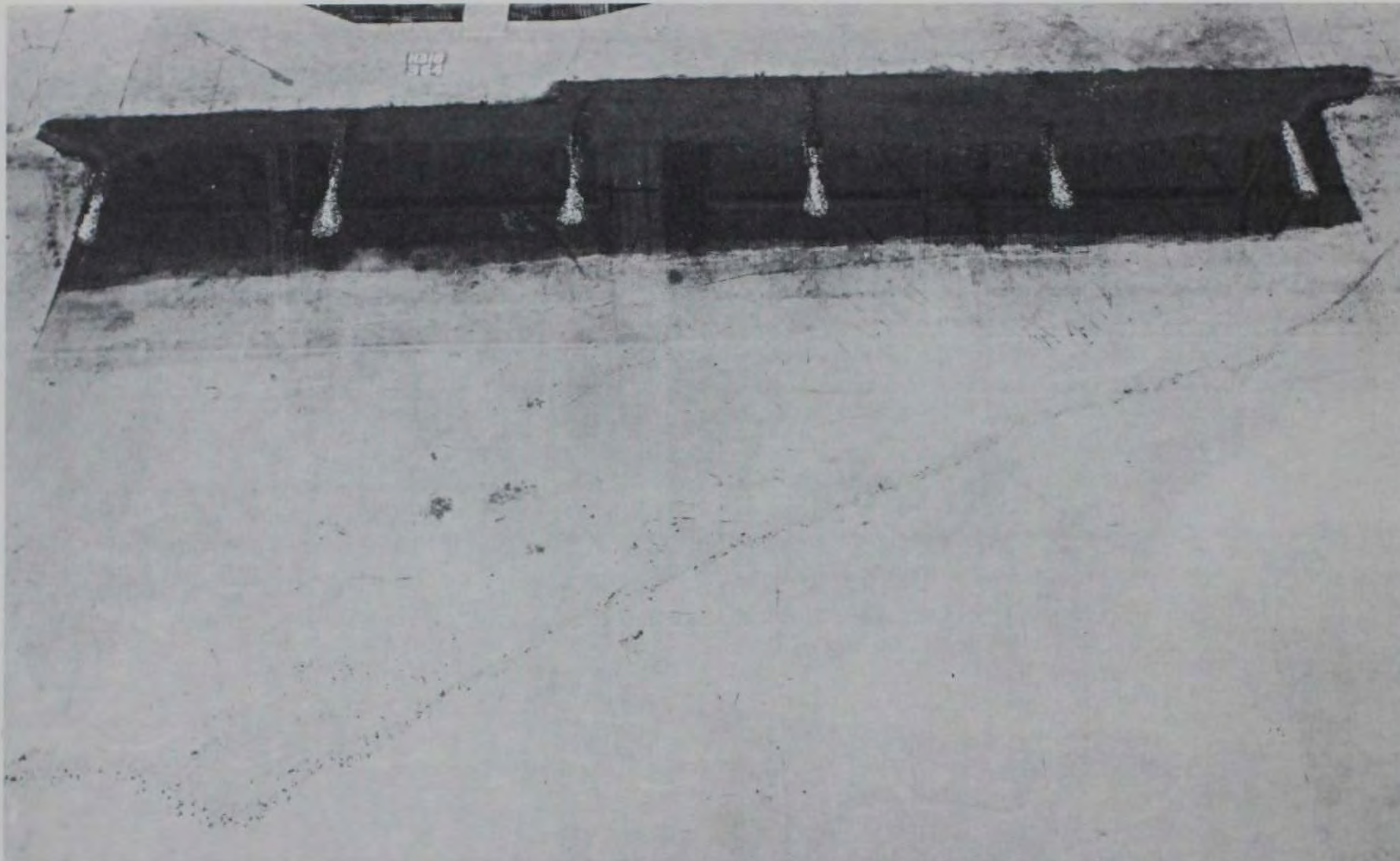


Photo 120. Placement of movable-bed tracer for Plan 9 (with revised beach slope) before testing

Photo 121. Movable-bed tracer deposits for Plan 9 (with revised beach slope) resulting from 9-sec, 10-ft waves from S55°W at mhhw

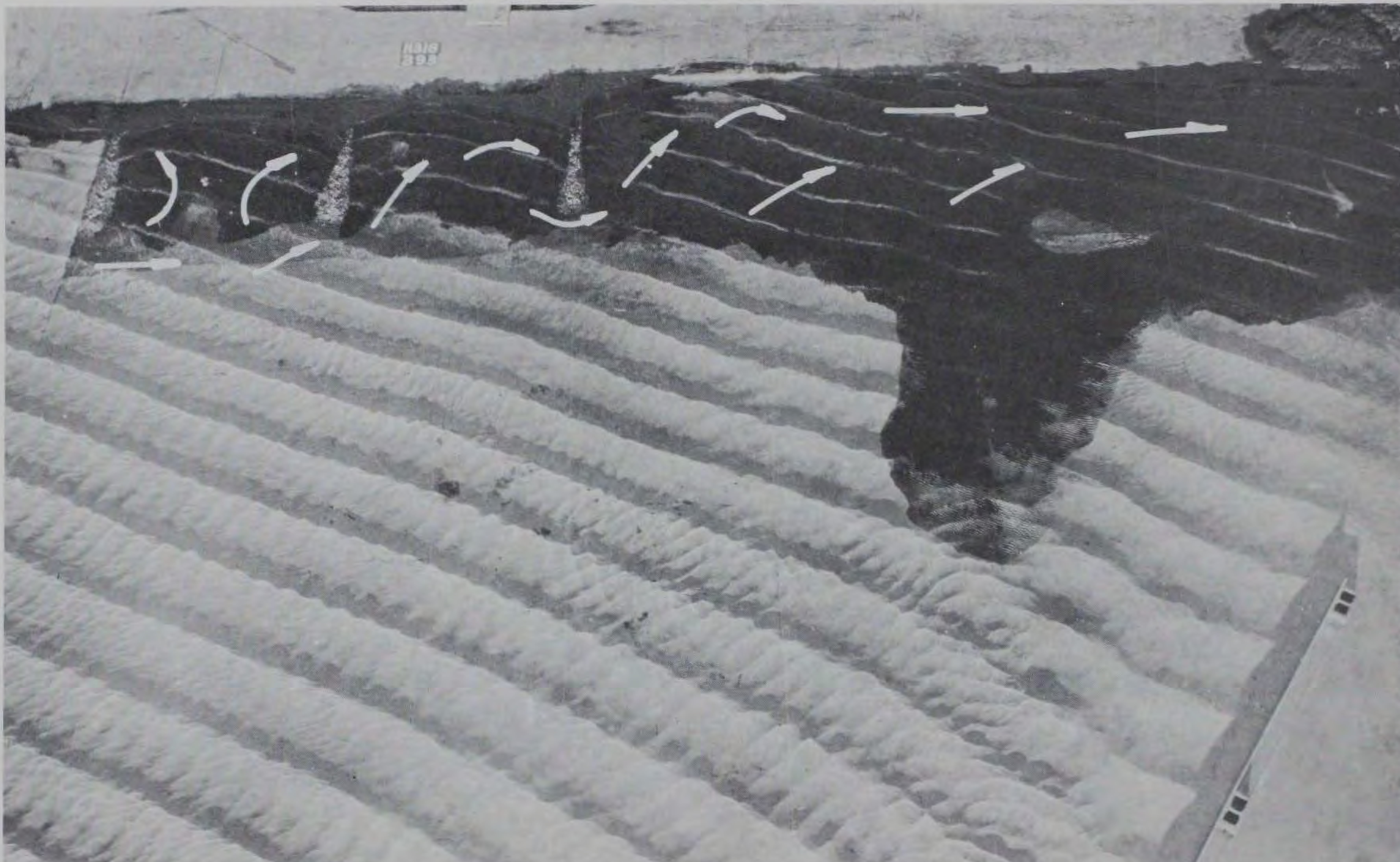
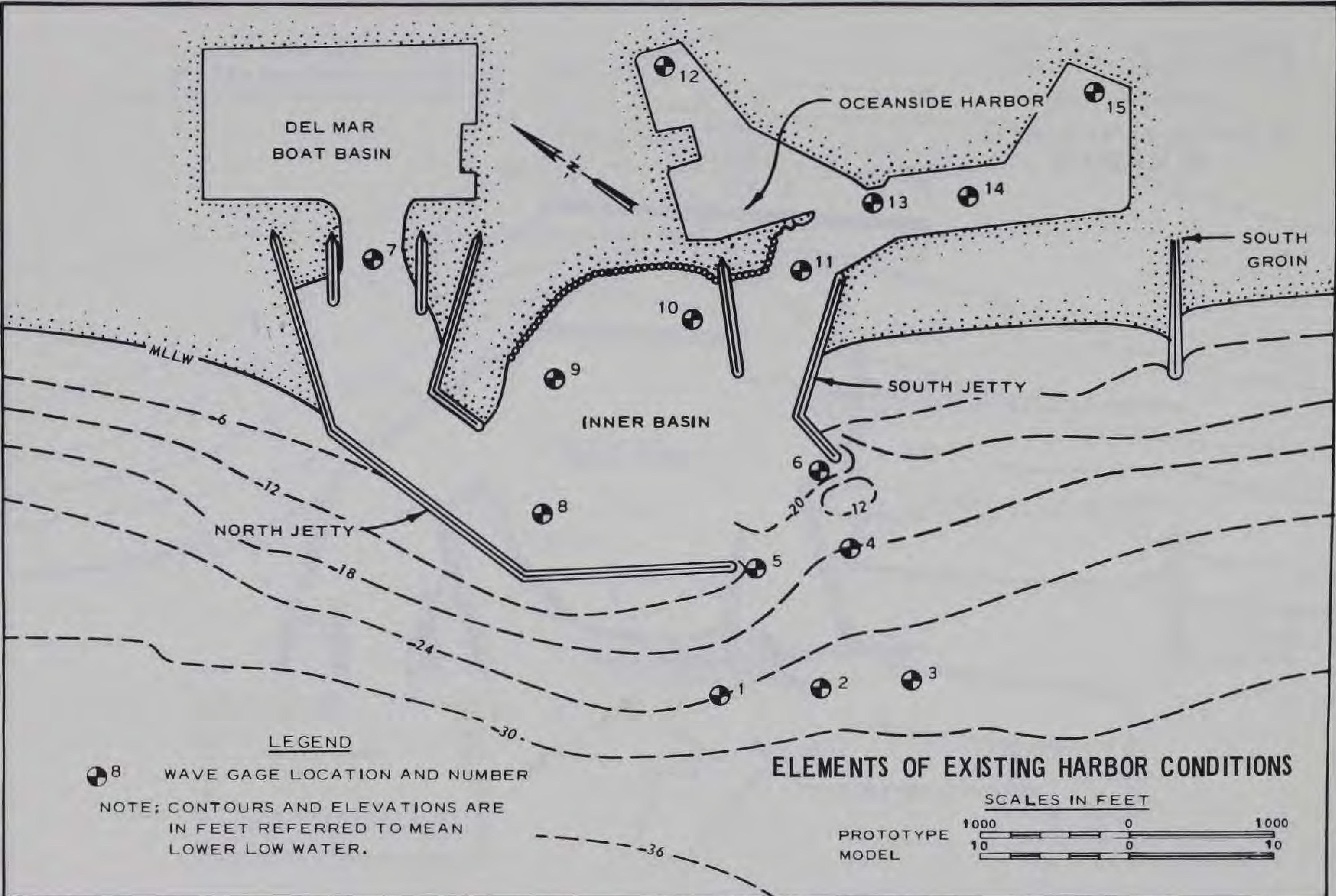


Photo 122. Movable-bed tracer deposits for Plan 9 (and adjacent south beach) resulting from 9-sec, 10-ft waves from northwest at mhhw (after 4 hr)

Photo 123. Movable-bed tracer deposits for Plan 9A (and adjacent south beach) resulting from 9-sec, 10-ft waves from northwest (after 4 hr)

Photo 124. Movable-bed tracer deposits for Plan 9B (and adjacent south beach) resulting from 9-sec, 10-ft waves from northwest at mhhw (after 4 hr)



LEGEND

8 WAVE GAGE LOCATION AND NUMBER

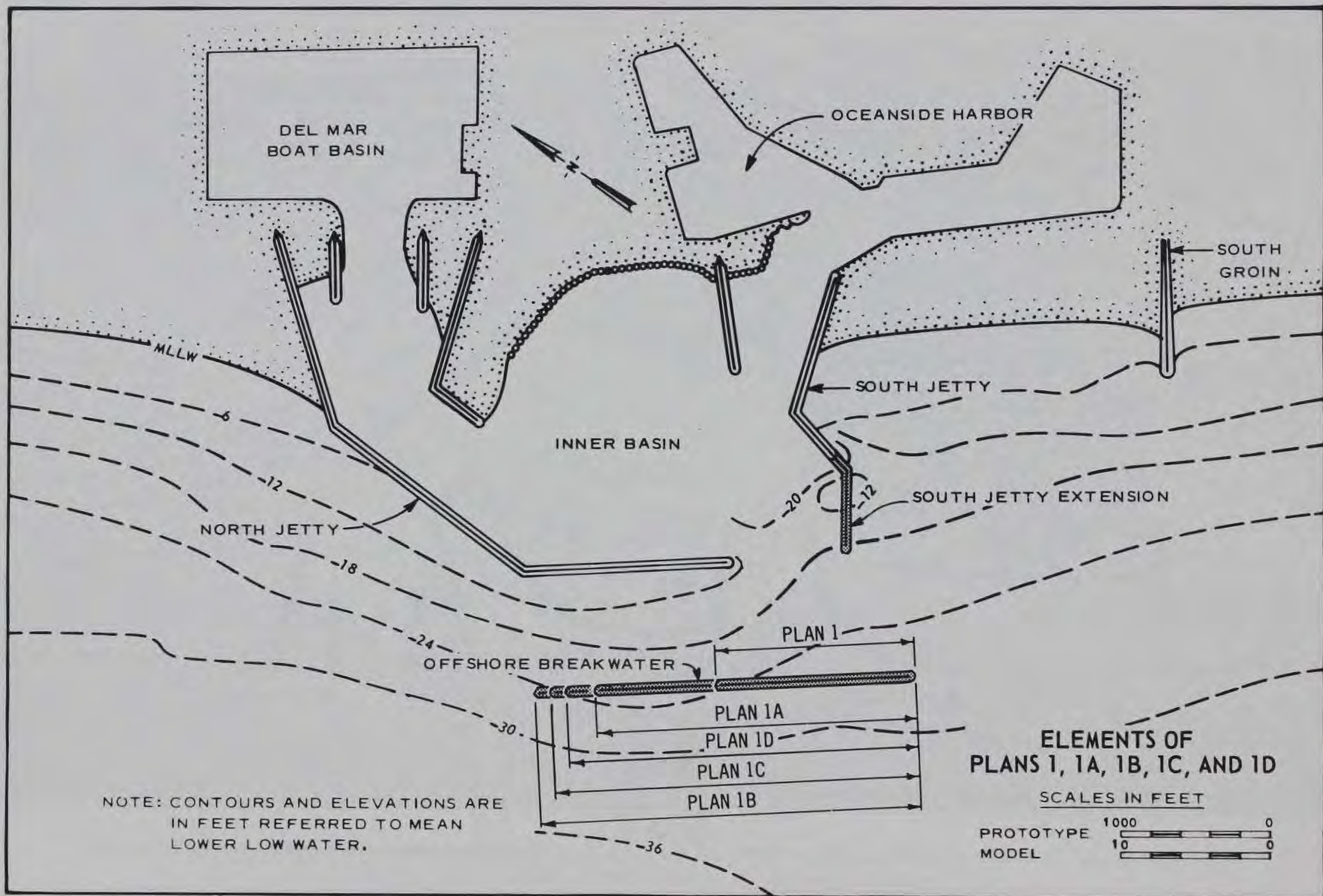
NOTE: CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER.

ELEMENTS OF EXISTING HARBOR CONDITIONS

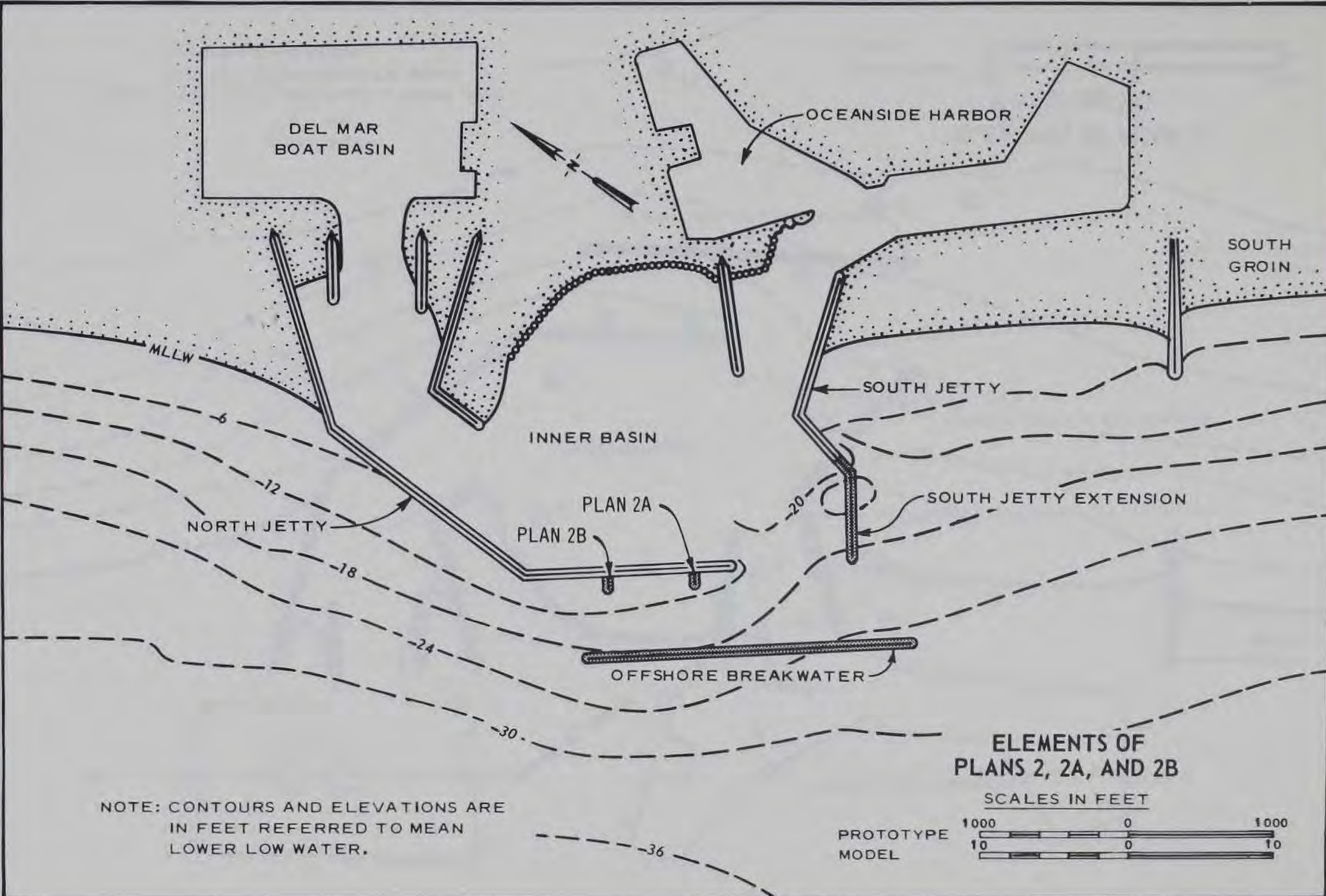
SCALES IN FEET

PROTOTYPE 1000 0 1000

MODEL 10 0 10

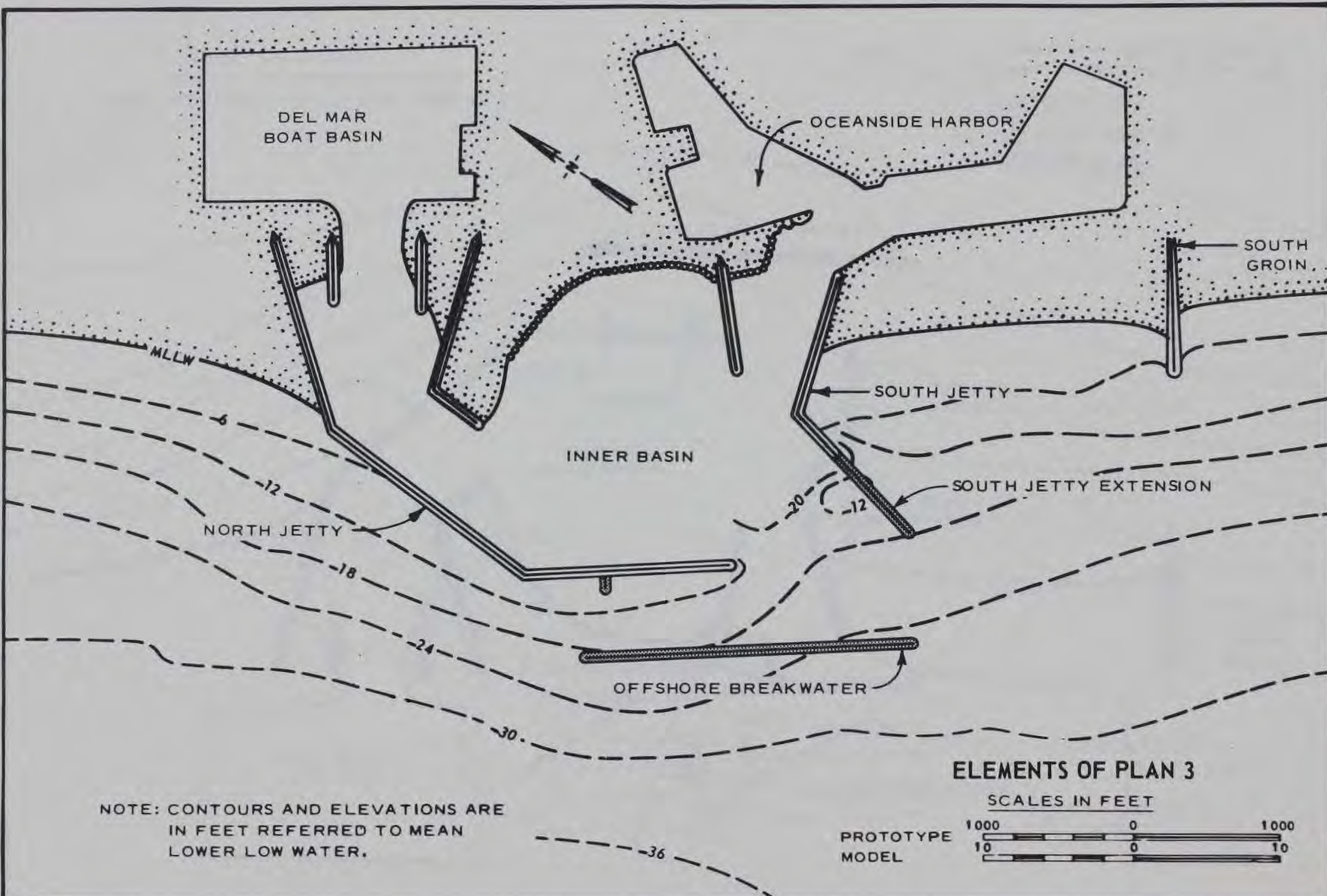


ELEMENTS OF
PLANS 1, 1A, 1B, 1C, AND 1D



NOTE: CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER.

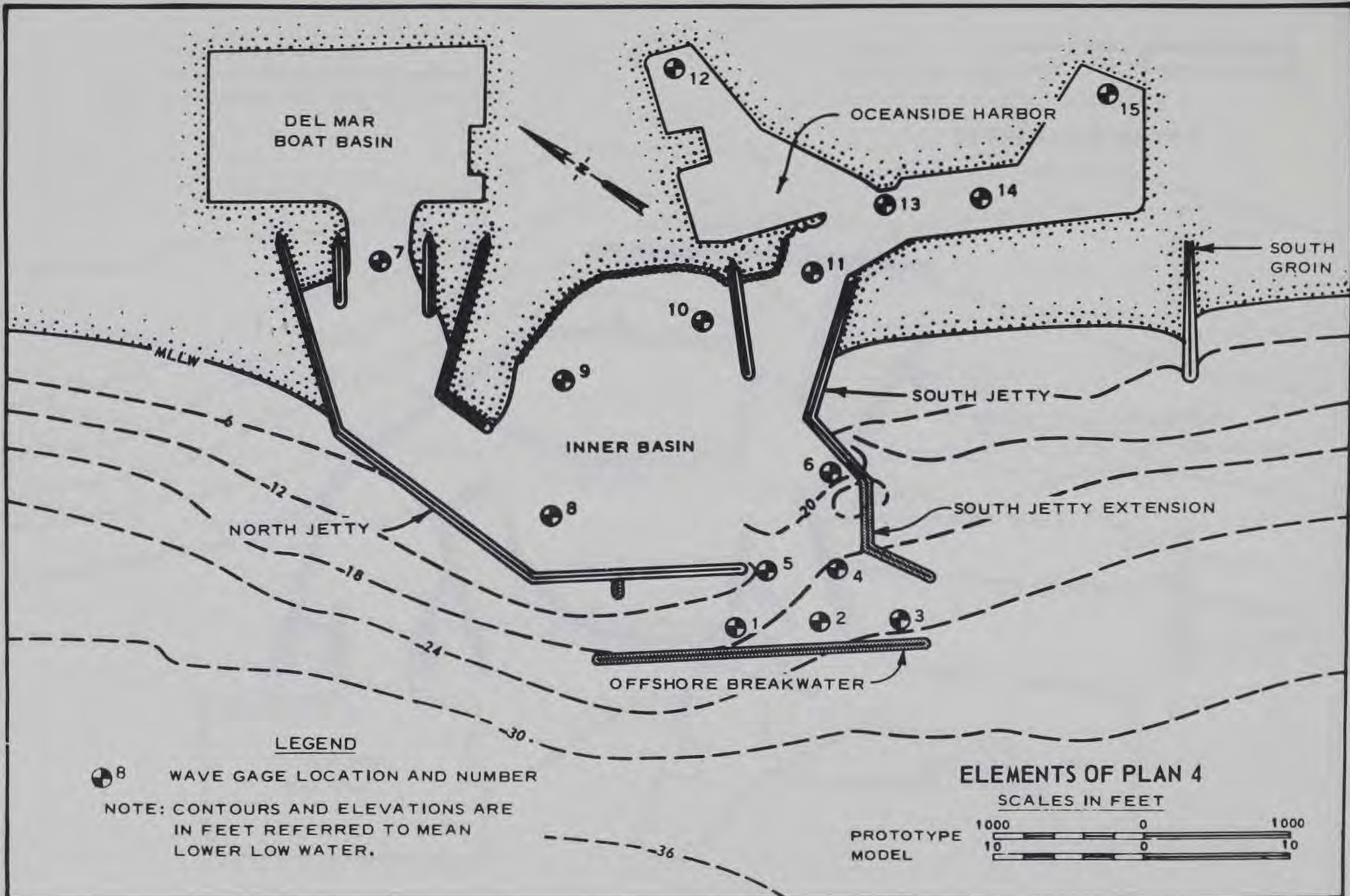
ELEMENTS OF PLANS 2, 2A, AND 2B



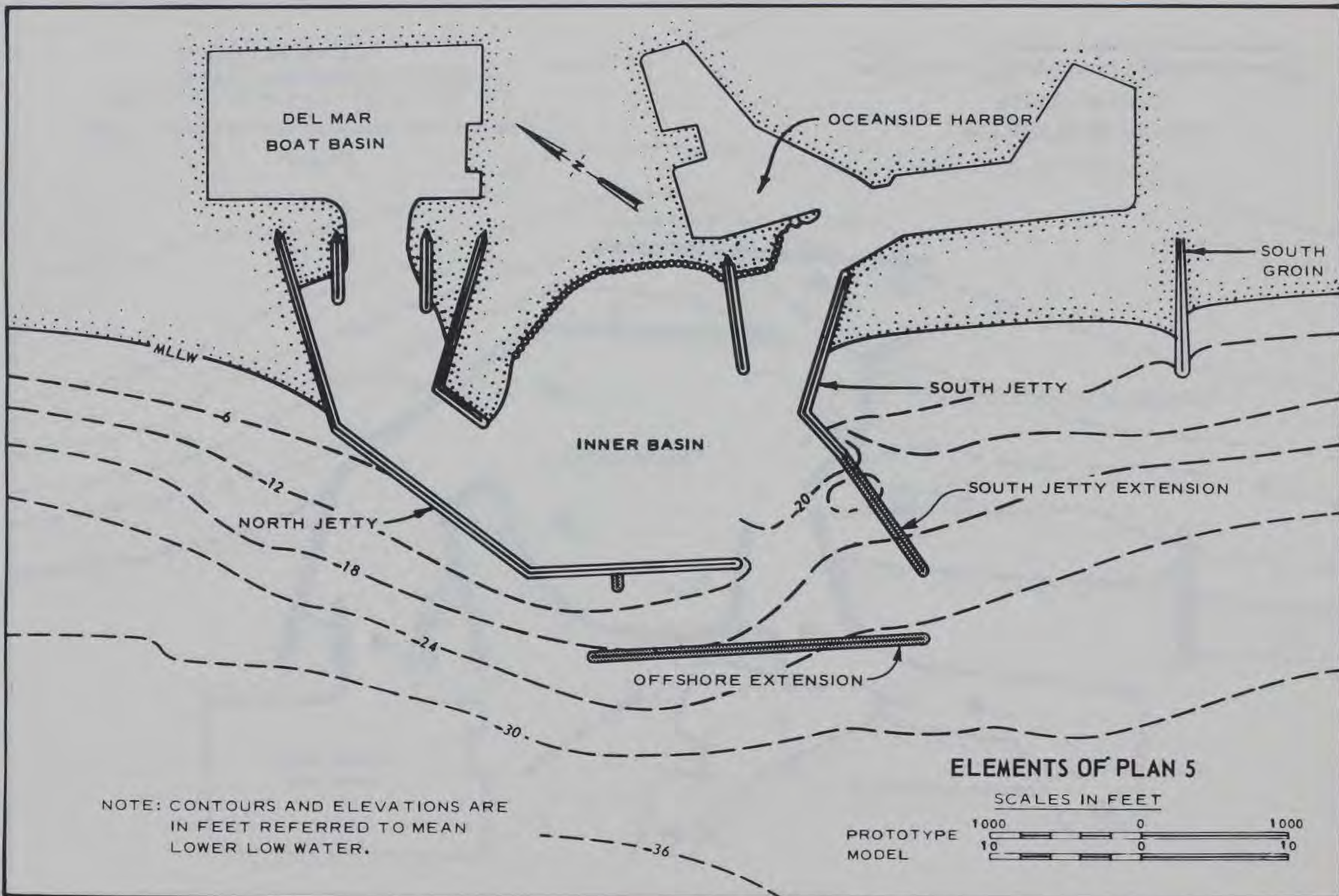
NOTE: CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER.

ELEMENTS OF PLAN 3

SCALES IN FEET

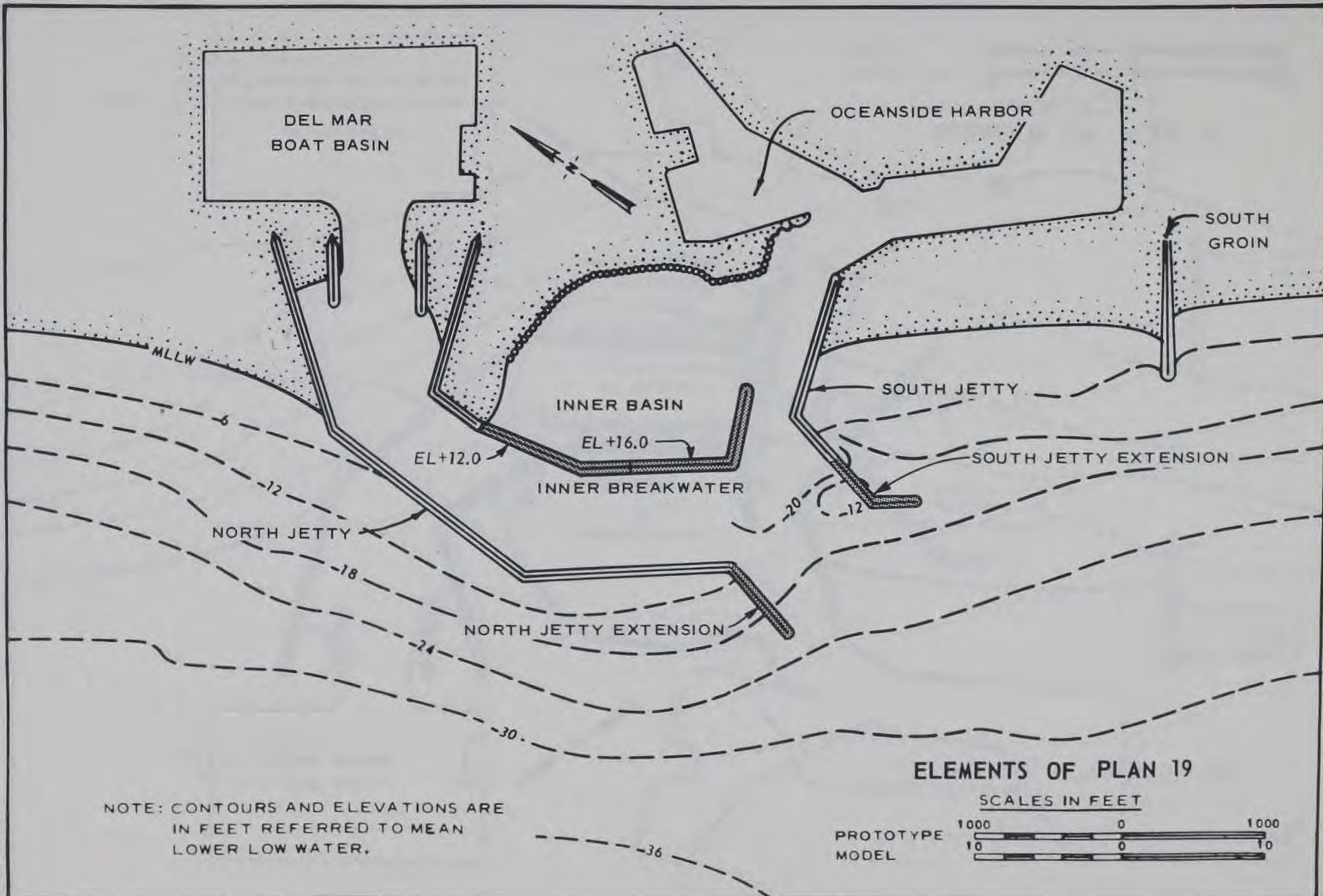


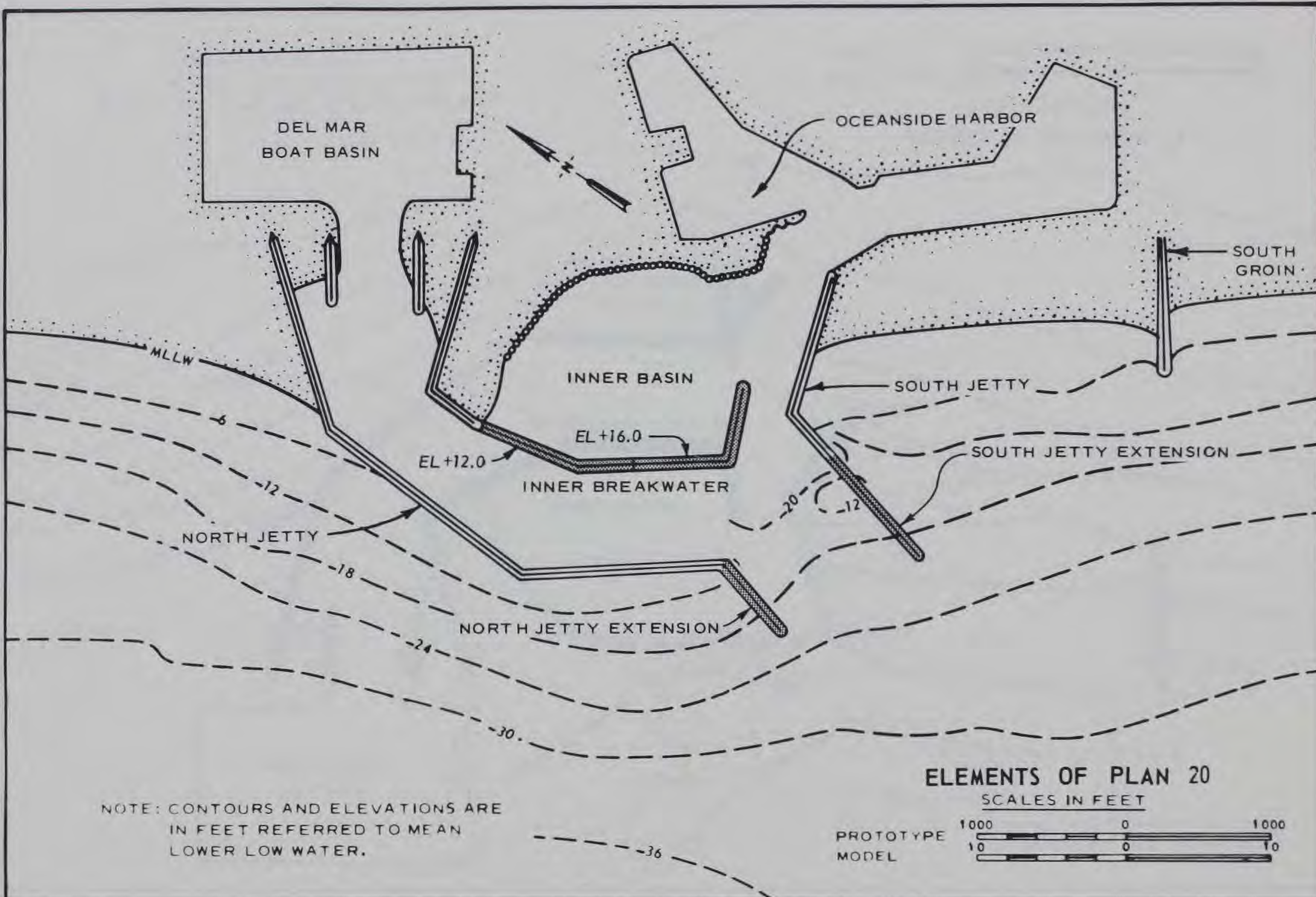
ELEMENTS OF PLAN 4

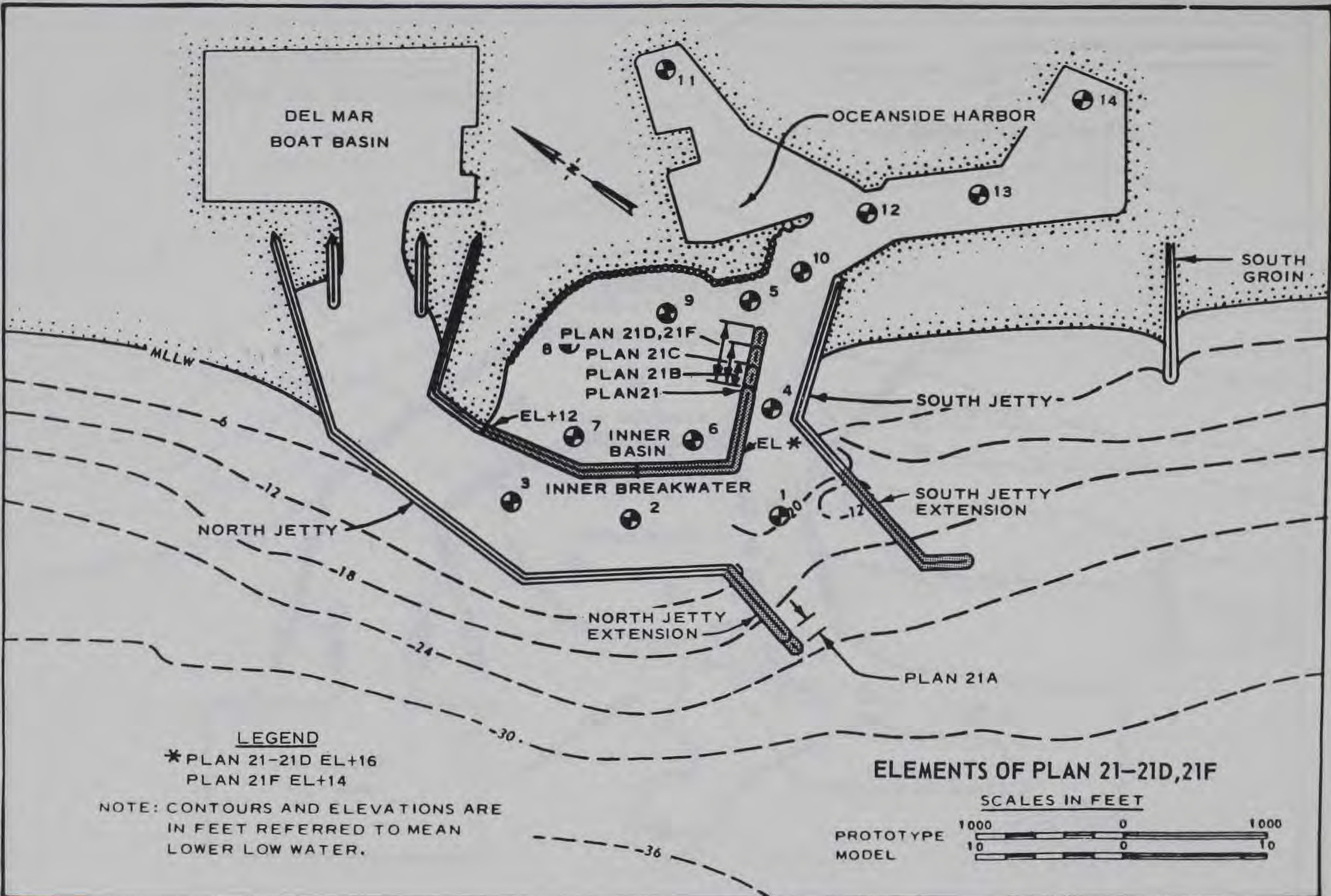


ELEMENTS OF PLAN 5

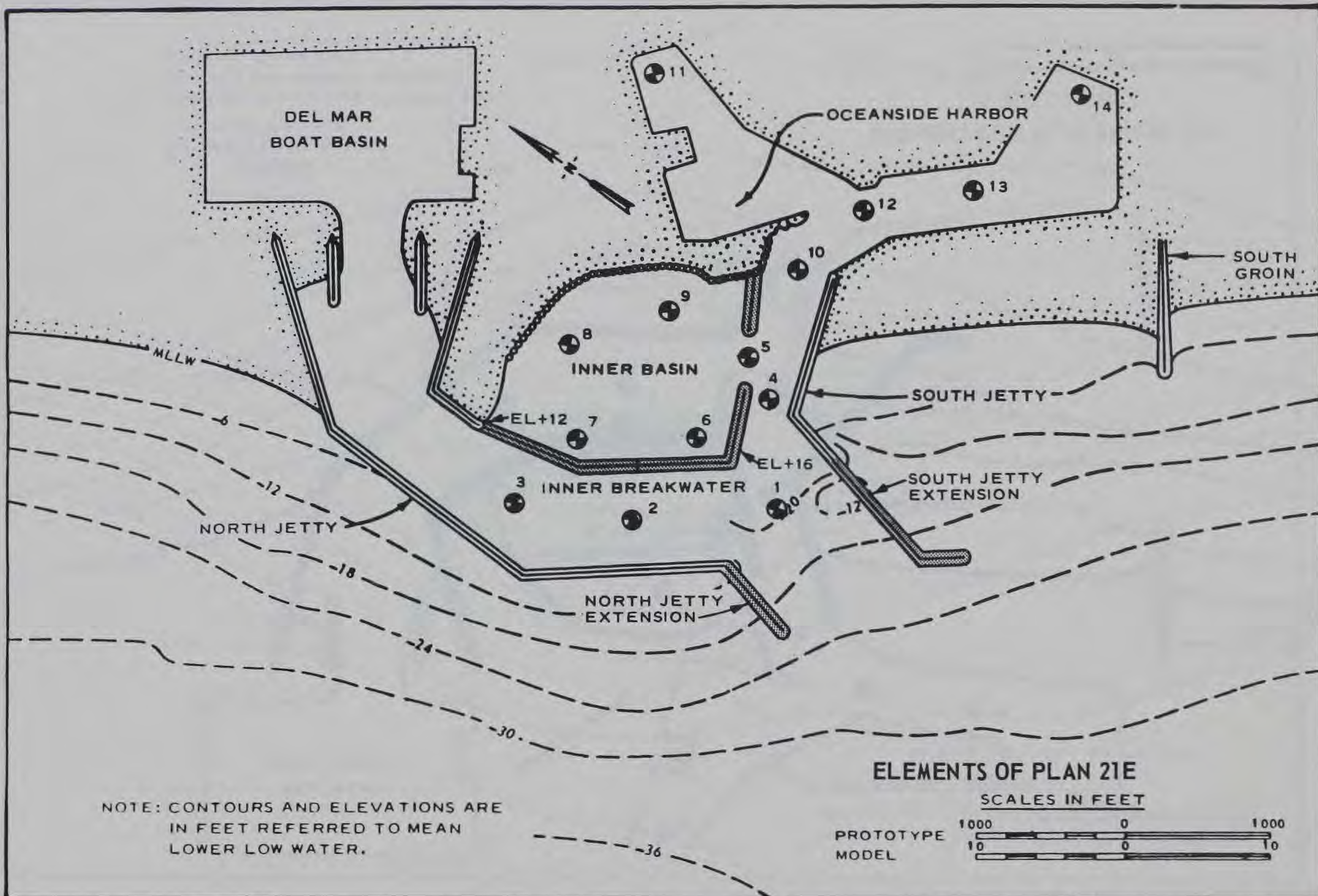
NOTE: CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER.


DEL MAR
BOAT BASIN
OCEANSIDE HARBOR
SOUTH
GROIN
MLLW
INNER BASIN
SOUTH JETTY
EL +16.0
EL+12.0
INNER BREAKWATER
SOUTH JETTY EXTENSION
-6
-12
-20
-12
NORTH JETTY
-18
NORTH JETTY EXTENSION
-24
-30
-36
ELEMENTS OF PLAN 19
SCALES IN FEET
PROTOTYPE
MODEL
1000
0
1000
10
0
10
NOTE: CONTOURS AND ELEVATIONS ARE
IN FEET REFERRED TO MEAN
LOWER LOW WATER.


DEL MAR
BOAT BASIN
N
OCEANSIDE HARBOR
SOUTH
GROIN
MLLW
INNER BASIN
SOUTH JETTY
EL +16.0
EL +12.0
INNER BREAKWATER
SOUTH JETTY EXTENSION
-6
-12
-18
-20
-12
-24
-30
-36
NORTH JETTY
NORTH JETTY EXTENSION
ELEMENTS OF PLAN 20
SCALES IN FEET
PROTOTYPE
MODEL
1000
0
1000
10
0
10
NOTE: CONTOURS AND ELEVATIONS ARE
IN FEET REFERRED TO MEAN
LOWER LOW WATER.



ELEMENTS OF PLAN 21-21D,21F



ELEMENTS OF PLAN 21E

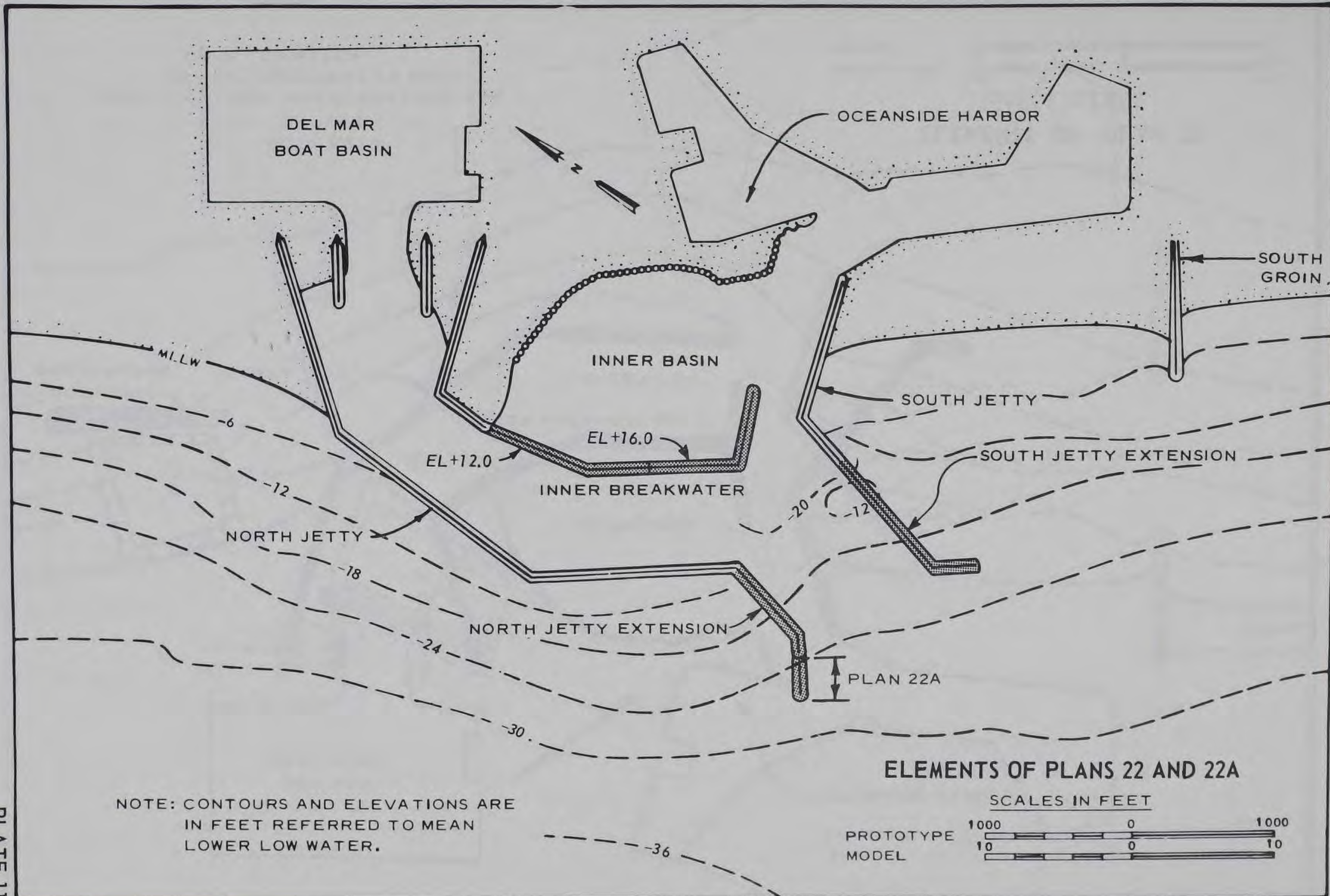
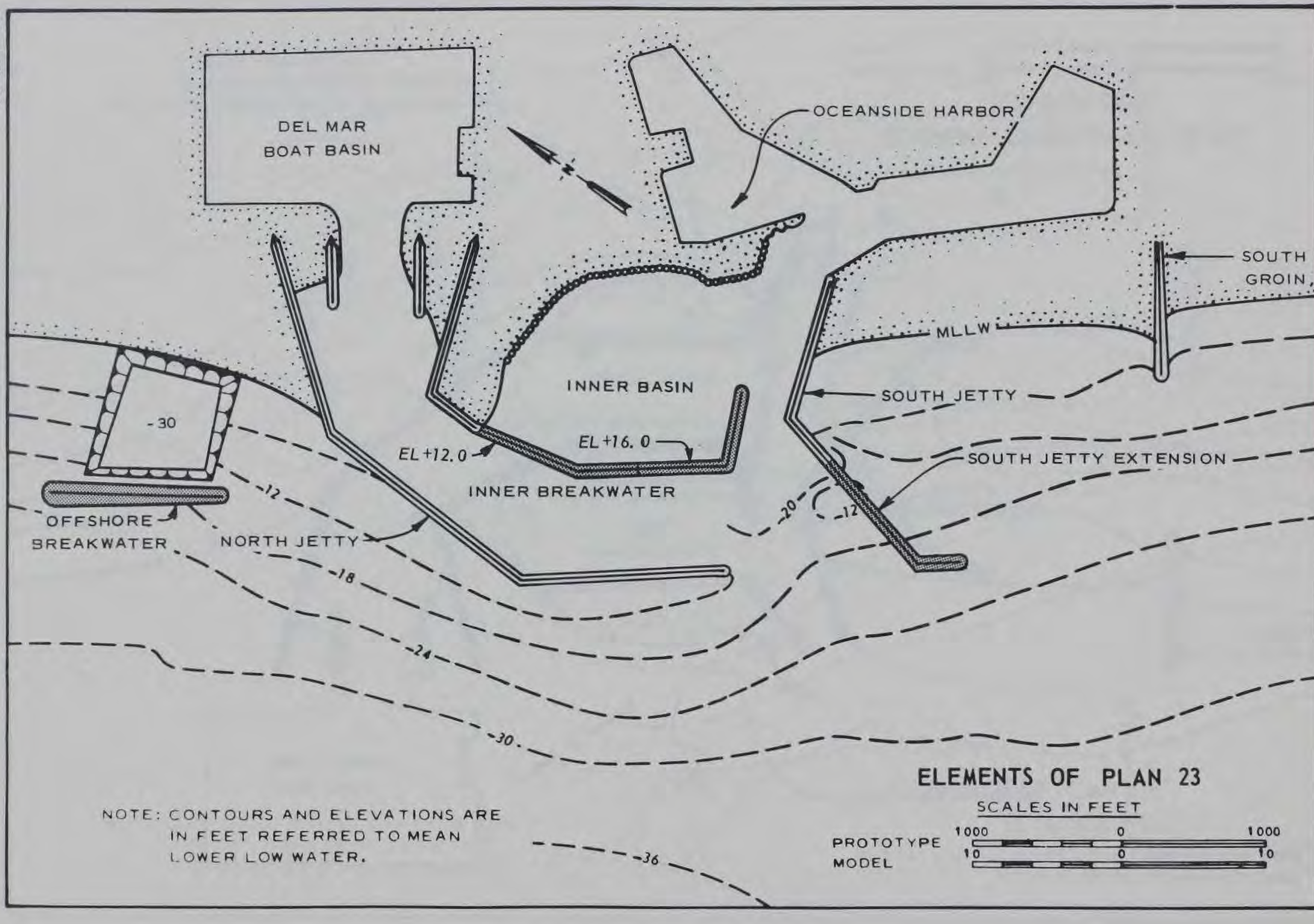
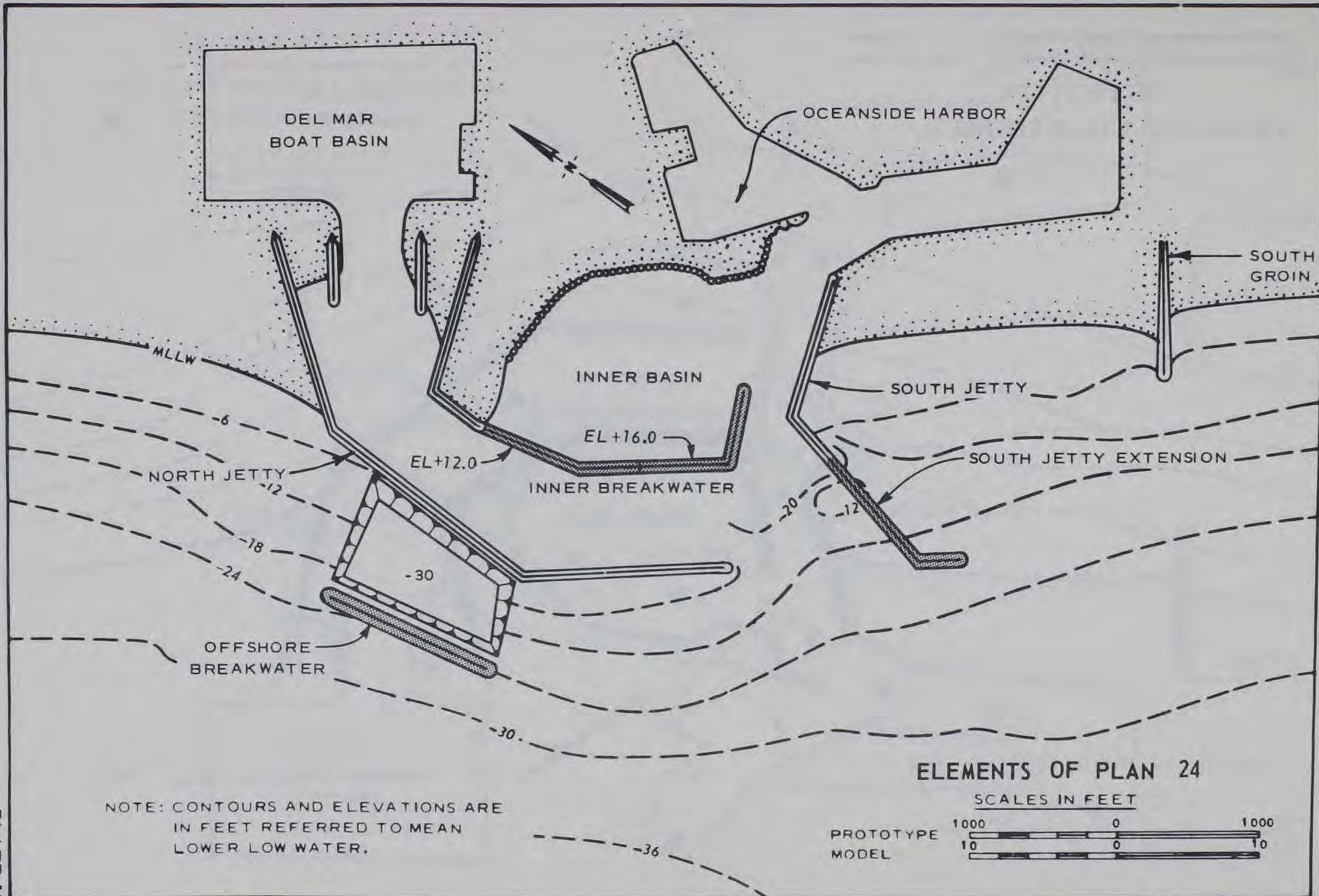

DEL MAR
BOAT BASIN
N
OCEANSIDE HARBOR
SOUTH
GROIN
MLLW
INNER BASIN
SOUTH JETTY
-6
EL+16.0
EL+12.0
SOUTH JETTY EXTENSION
-12
INNER BREAKWATER
-20
-12
NORTH JETTY
-18
NORTH JETTY EXTENSION
-24
PLAN 22A
-30
-36
ELEMENTS OF PLANS 22 AND 22A
SCALES IN FEET
PROTOTYPE
1000 0 1000
MODEL
10 0 10
NOTE: CONTOURS AND ELEVATIONS ARE
IN FEET REFERRED TO MEAN
LOWER LOW WATER.

PLATE 11

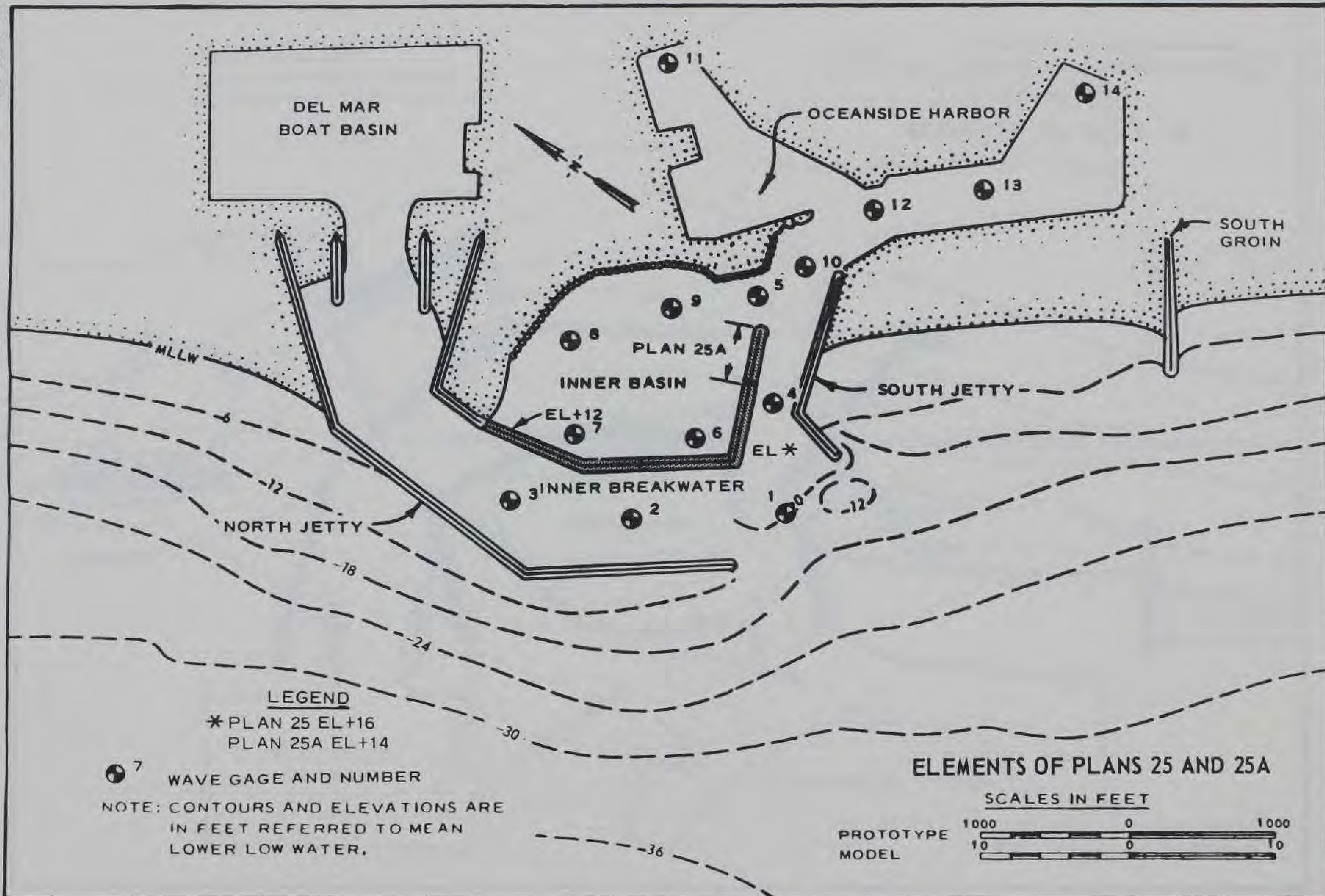


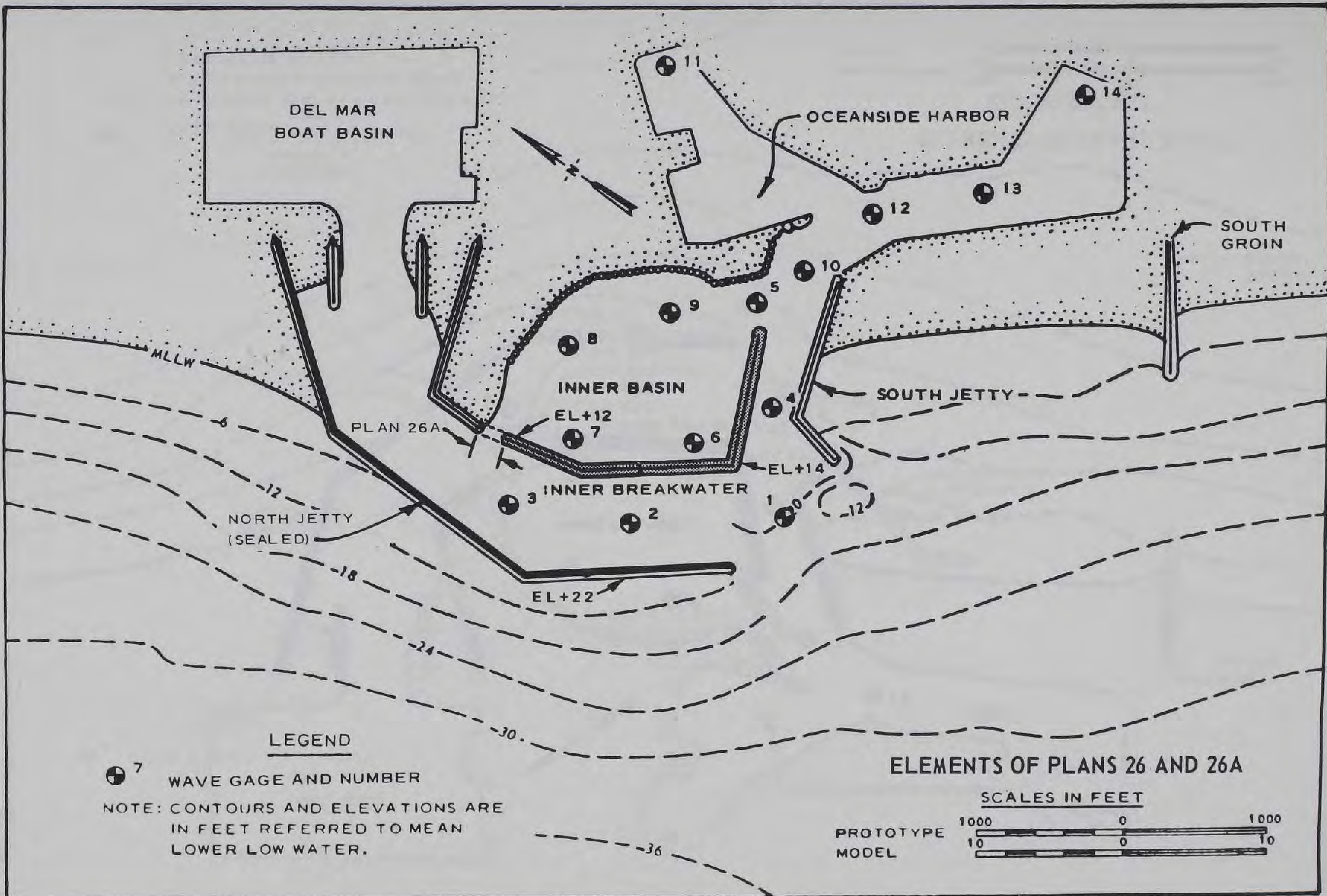
ELEMENTS OF PLAN 23



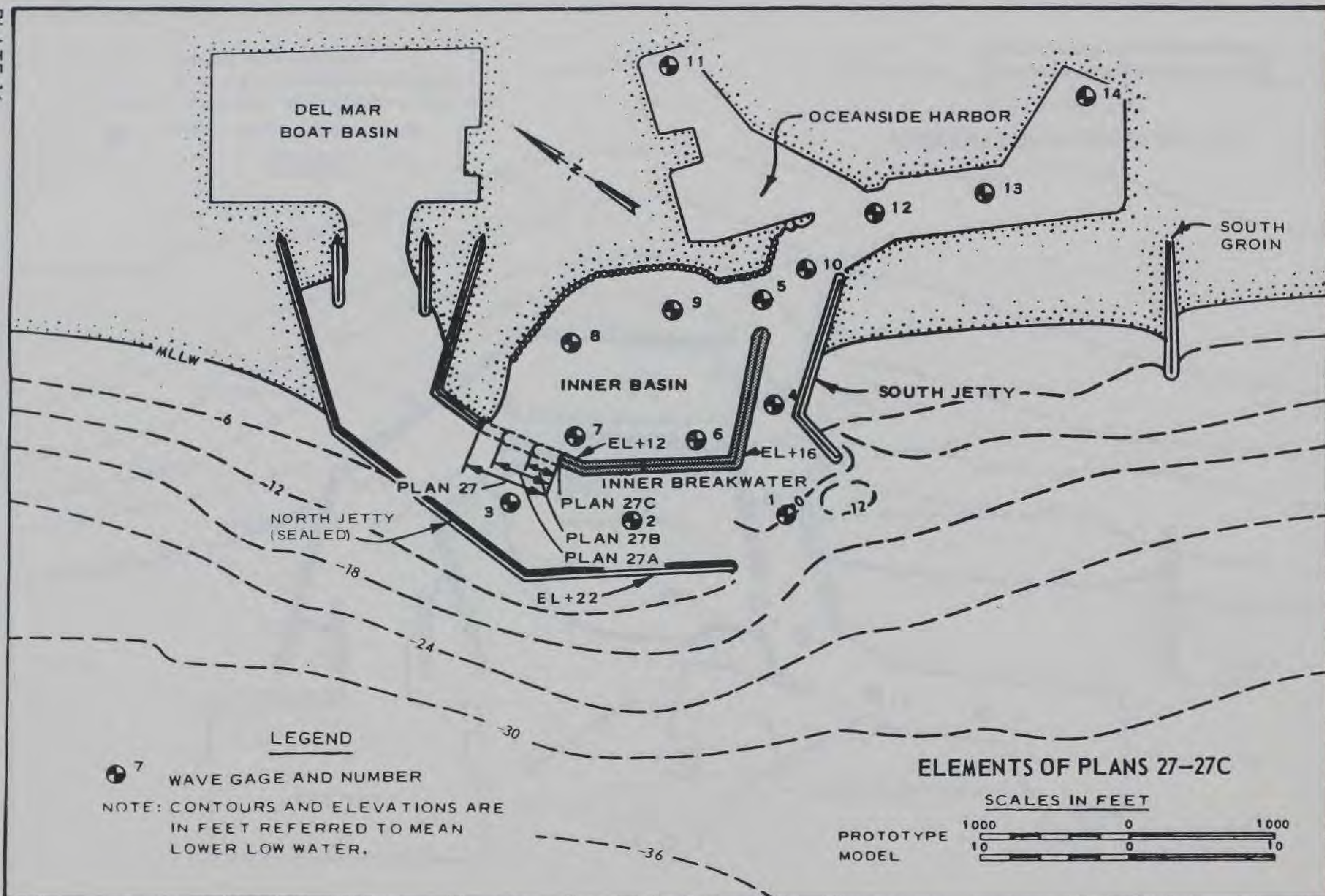
ELEMENTS OF PLAN 24

NOTE: CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER.

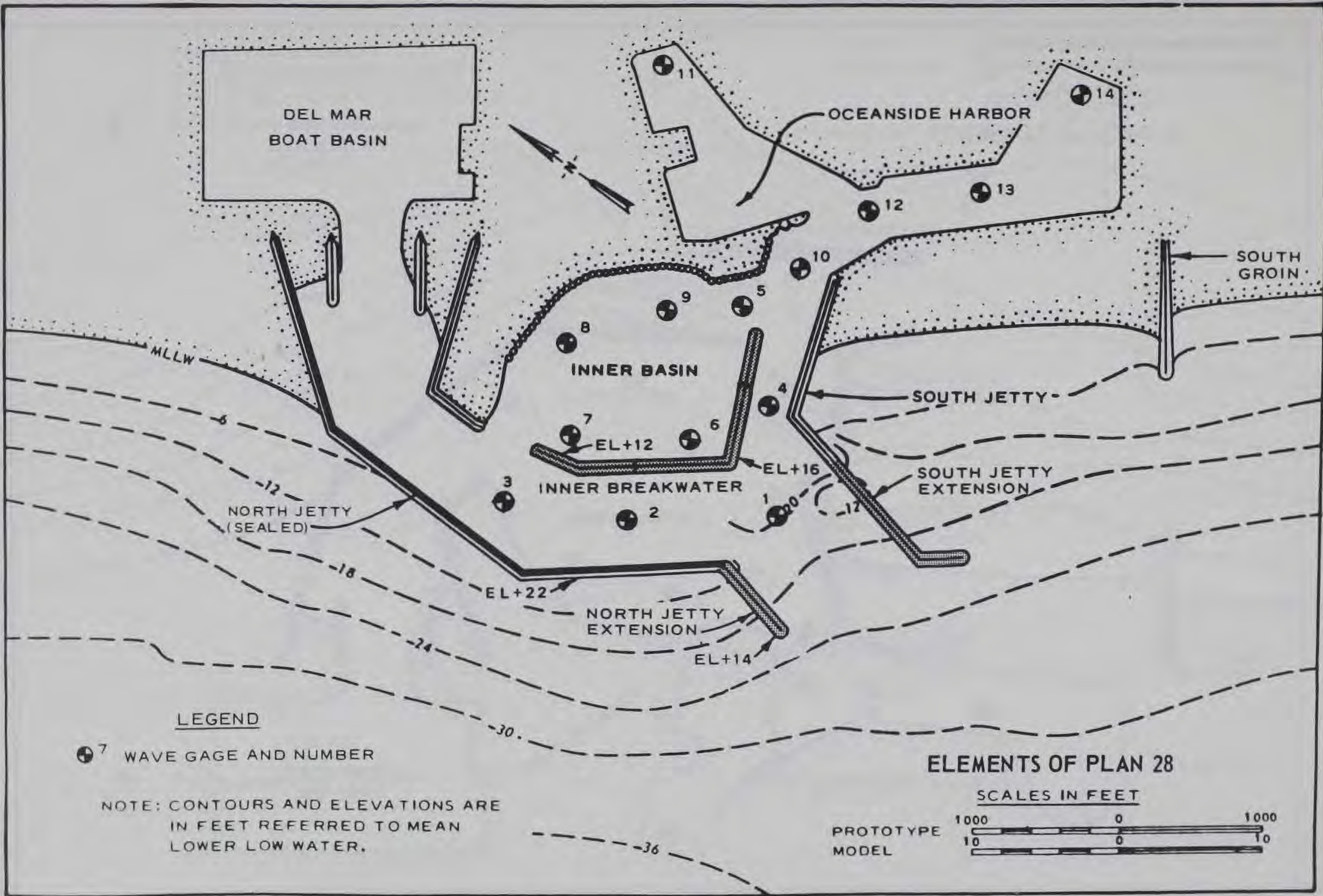

DEL MAR
BOAT BASIN
OCEANSIDE HARBOR
SOUTH
GROIN
MLLW
PLAN 25A
INNER BASIN
SOUTH JETTY
EL+12
EL *
INNER BREAKWATER
NORTH JETTY
-6
-12
-18
-24
-30
-36
LEGEND
* PLAN 25 EL+16
PLAN 25A EL+14
WAVE GAGE AND NUMBER
NOTE: CONTOURS AND ELEVATIONS ARE
IN FEET REFERRED TO MEAN
LOWER LOW WATER.
ELEMENTS OF PLANS 25 AND 25A
SCALES IN FEET
PROTOTYPE
MODEL
1000
0
1000
10
0
10



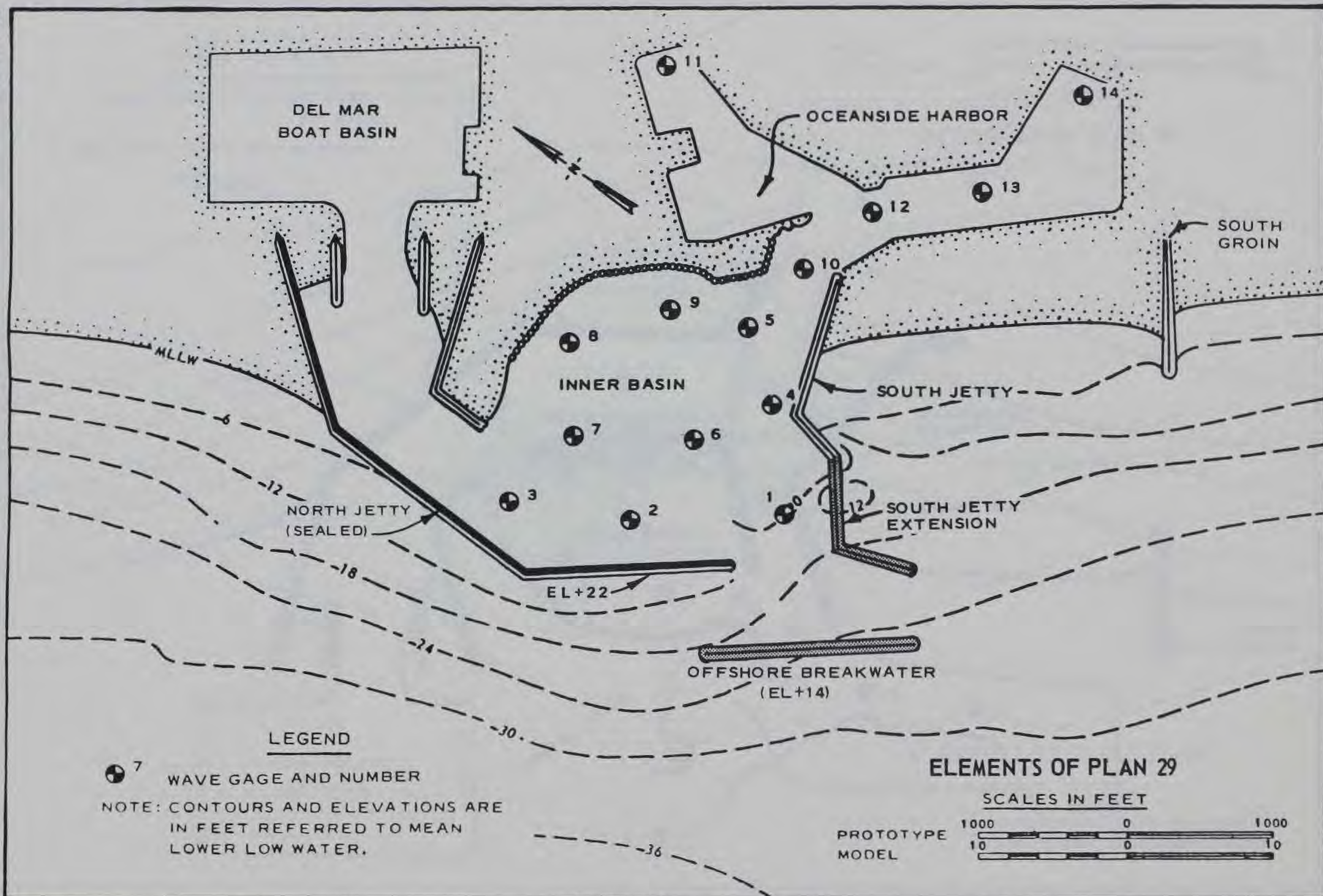
ELEMENTS OF PLANS 26 AND 26A



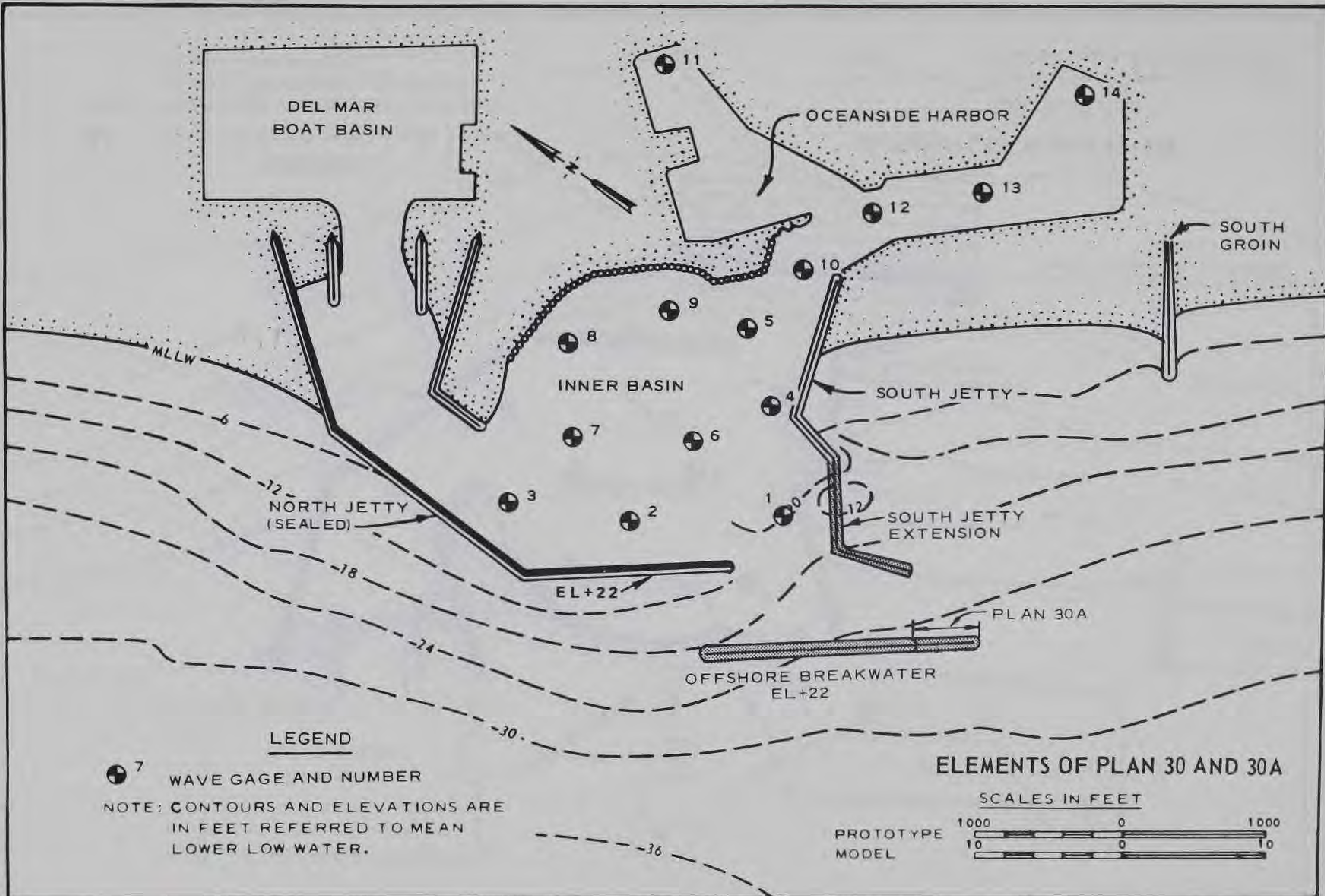
ELEMENTS OF PLANS 27–27C



ELEMENTS OF PLAN 28

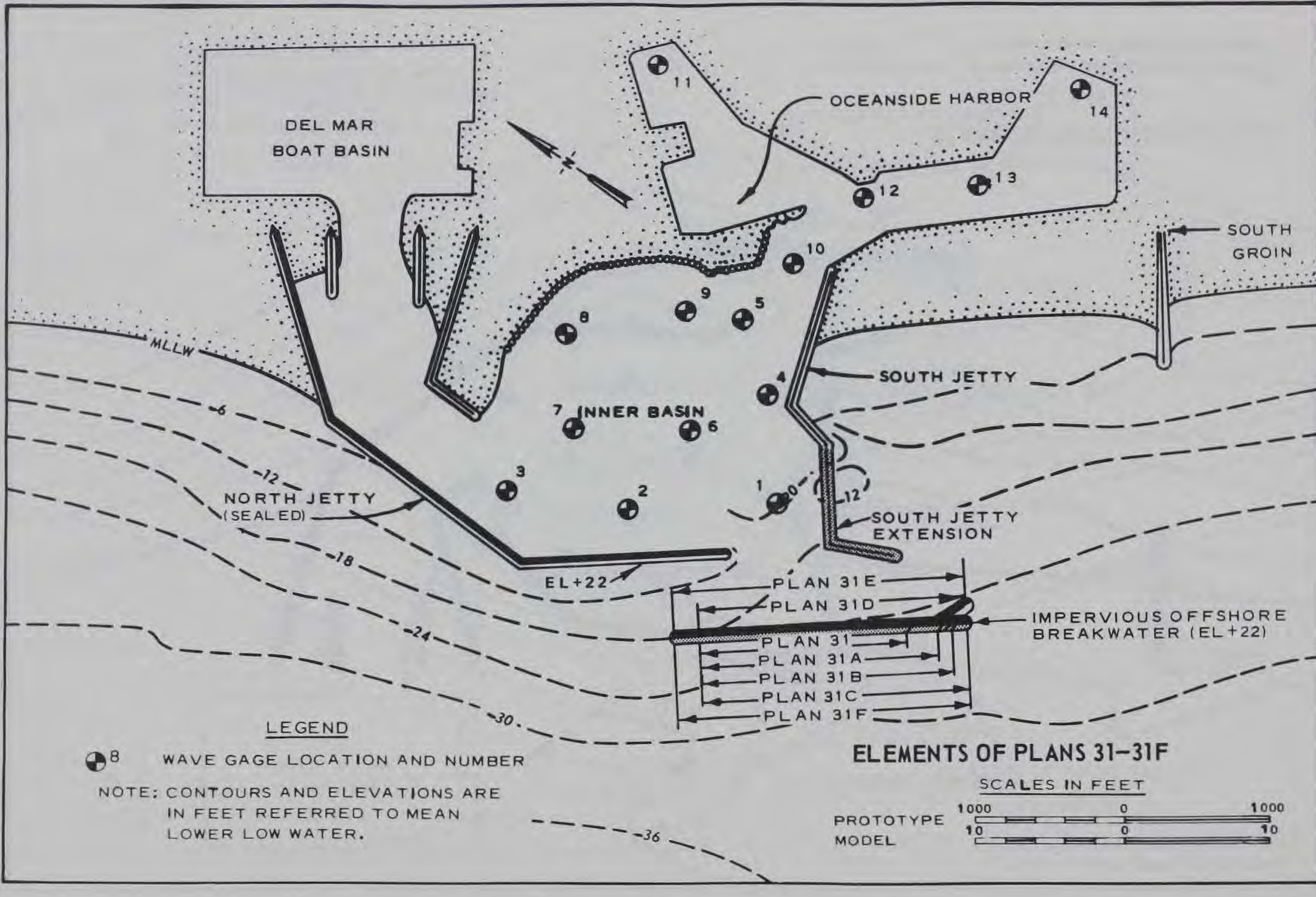


ELEMENTS OF PLAN 29

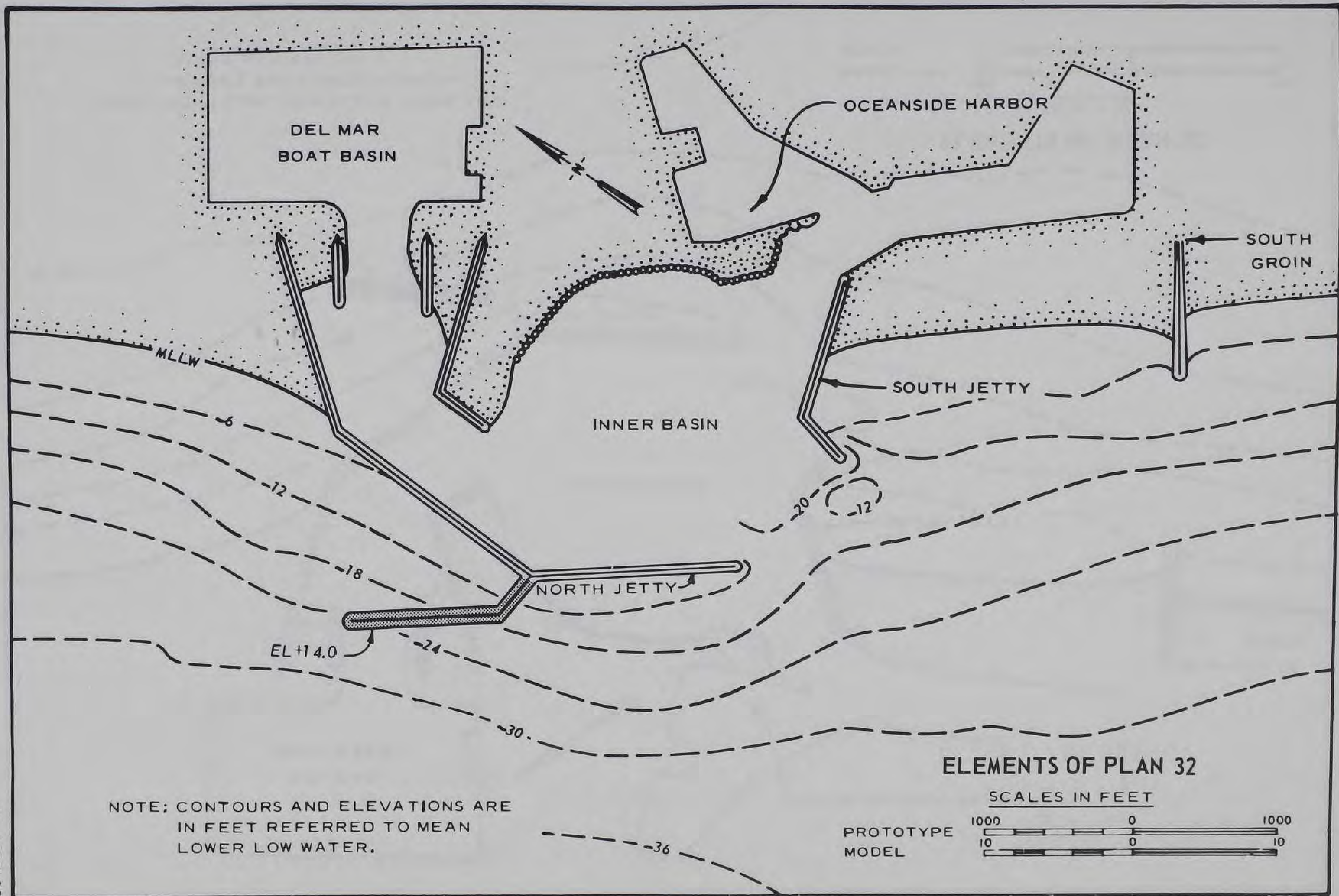


ELEMENTS OF PLAN 30 AND 30A

PLATE 19

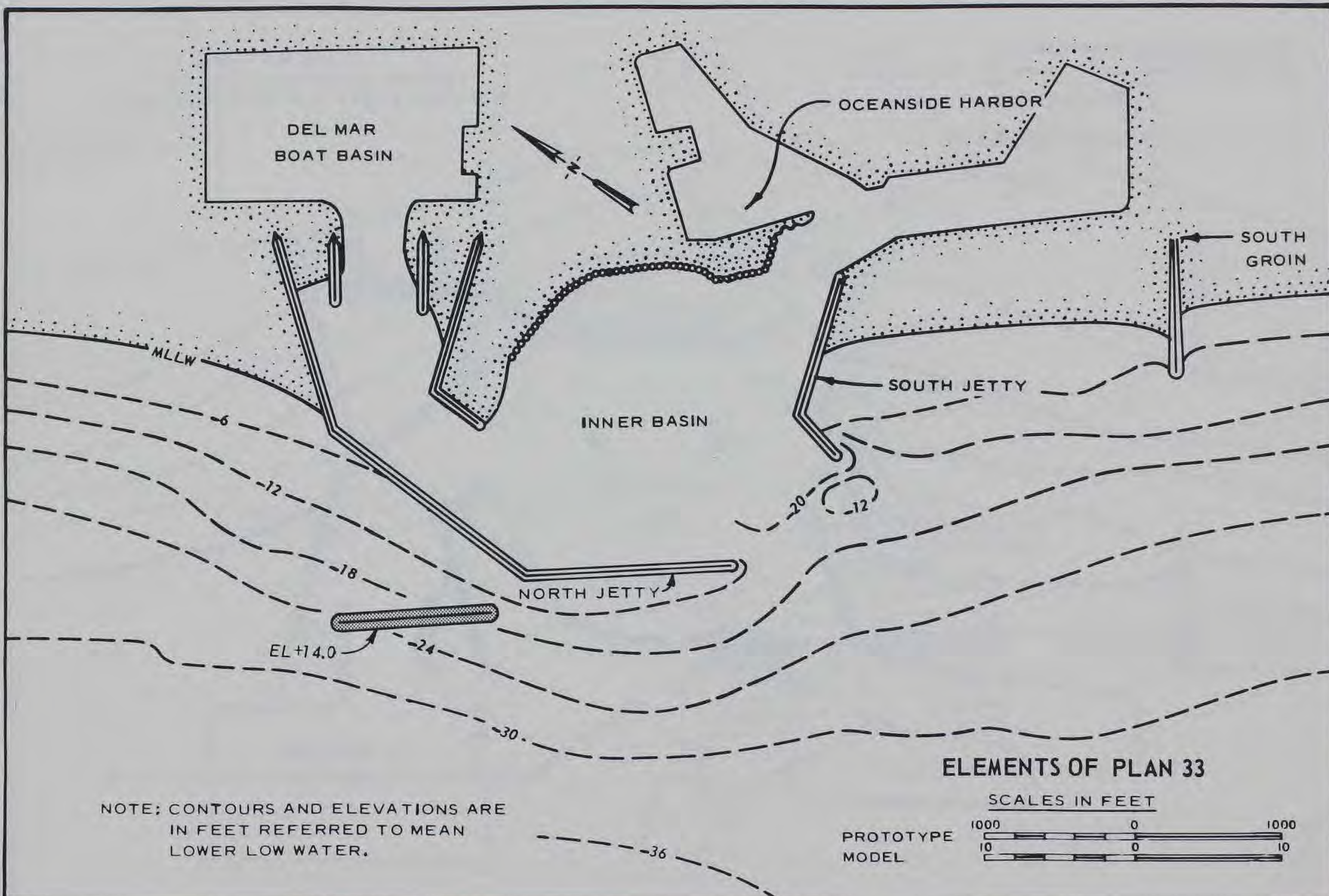


ELEMENTS OF PLANS 31–31F



ELEMENTS OF PLAN 32

NOTE: CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER.



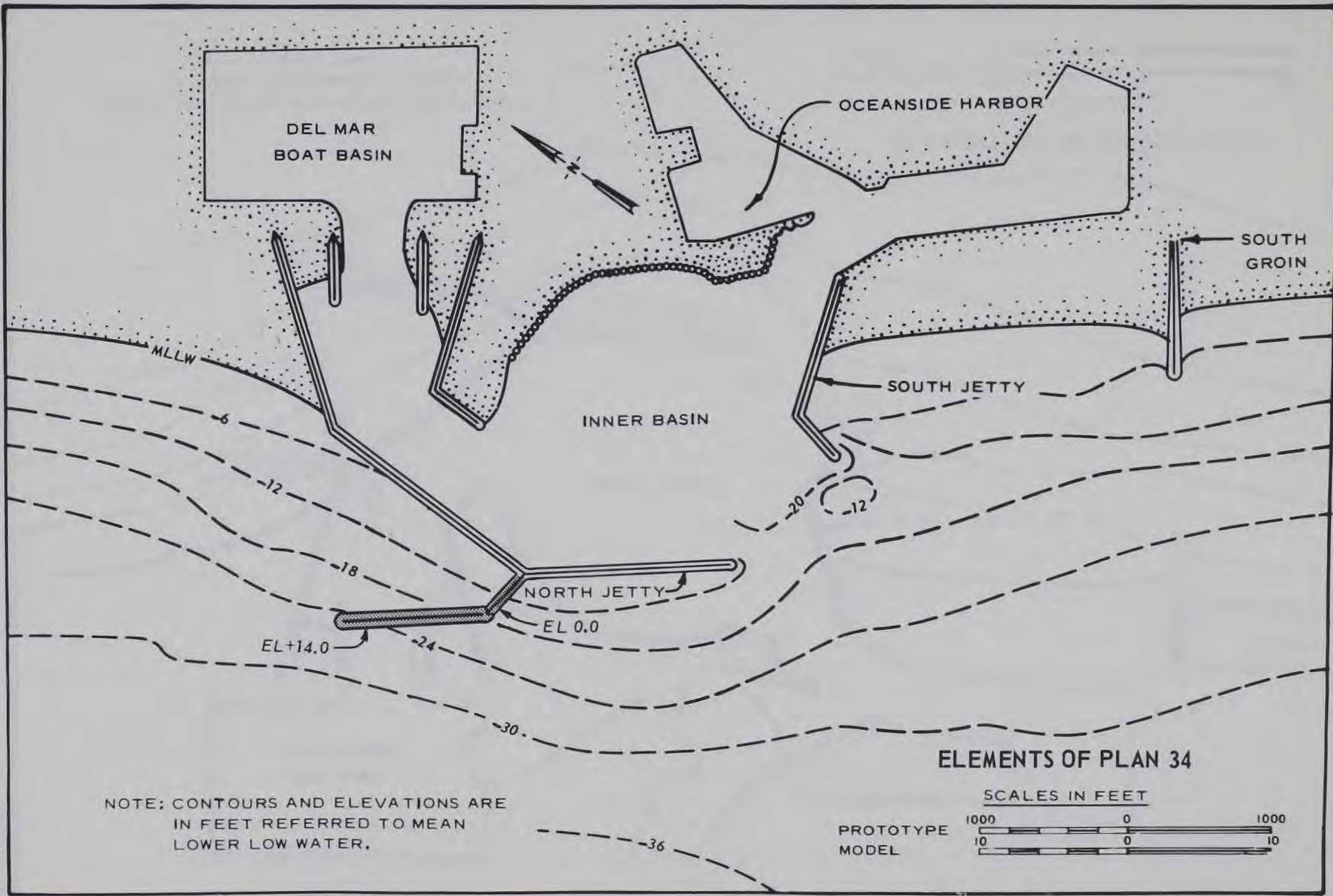
ELEMENTS OF PLAN 33

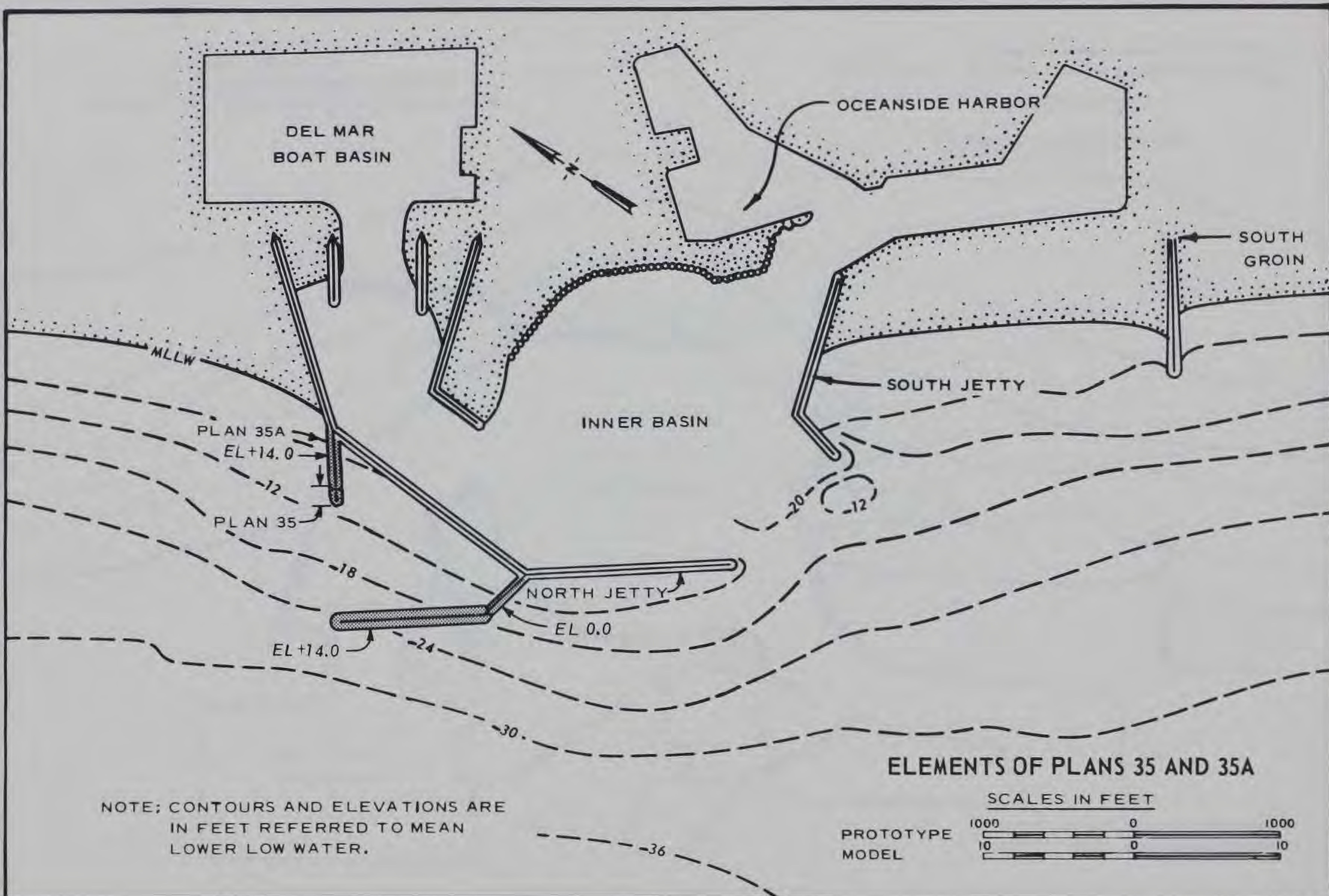
SCALES IN FEET

PROTOTYPE 1000 0 1000

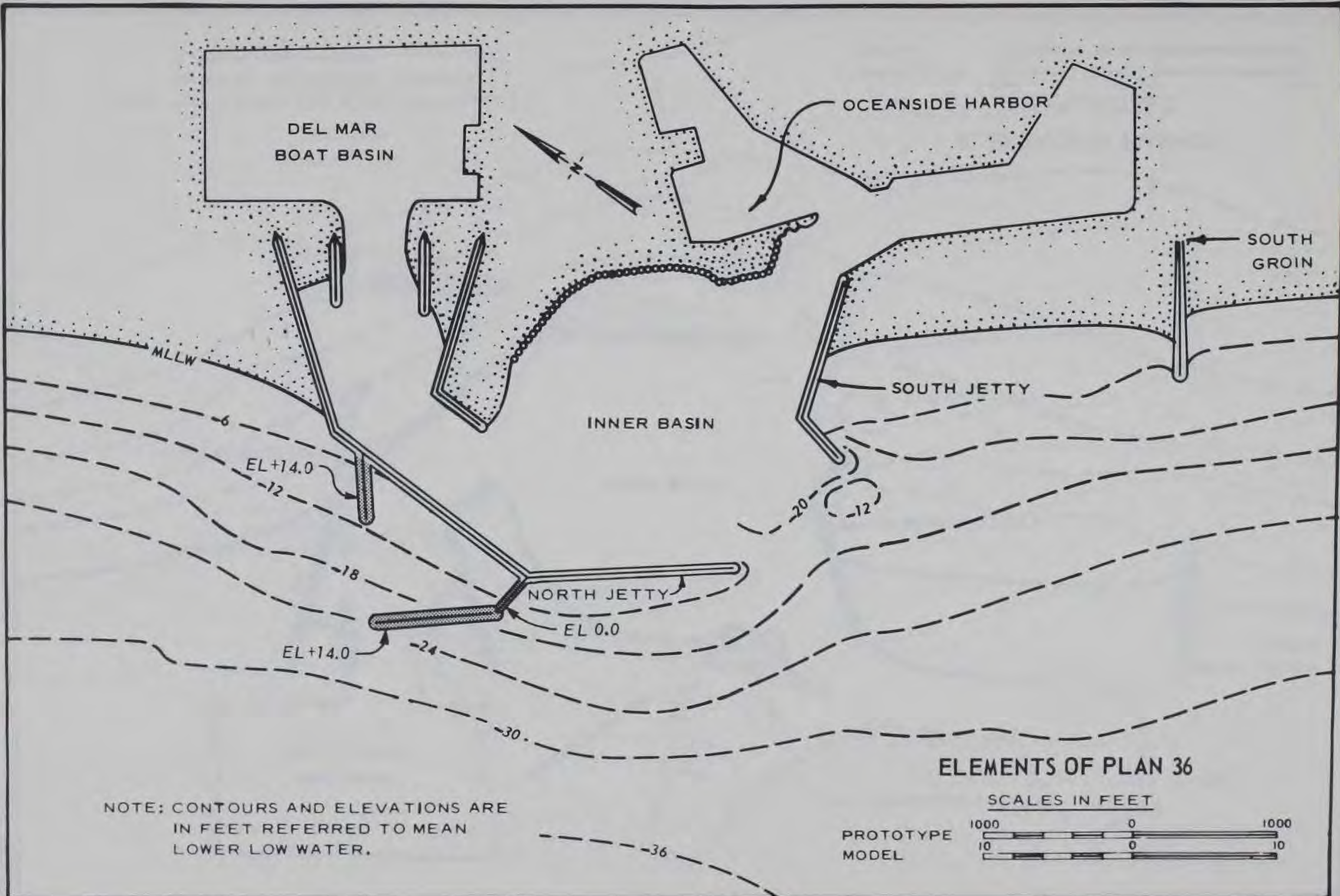
MODEL 10 0 10

NOTE: CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER.


DEL MAR
BOAT BASIN
OCEANSIDE HARBOR
SOUTH
GROIN
MLLW
SOUTH JETTY
INNER BASIN
-6
-12
-18
-20
-12
-24
-30
-36
NORTH JETTY
EL 0.0
EL+14.0
ELEMENTS OF PLAN 34
NOTE: CONTOURS AND ELEVATIONS ARE
IN FEET REFERRED TO MEAN
LOWER LOW WATER.
SCALES IN FEET
PROTOTYPE
MODEL
1000
0
1000
10
0
10

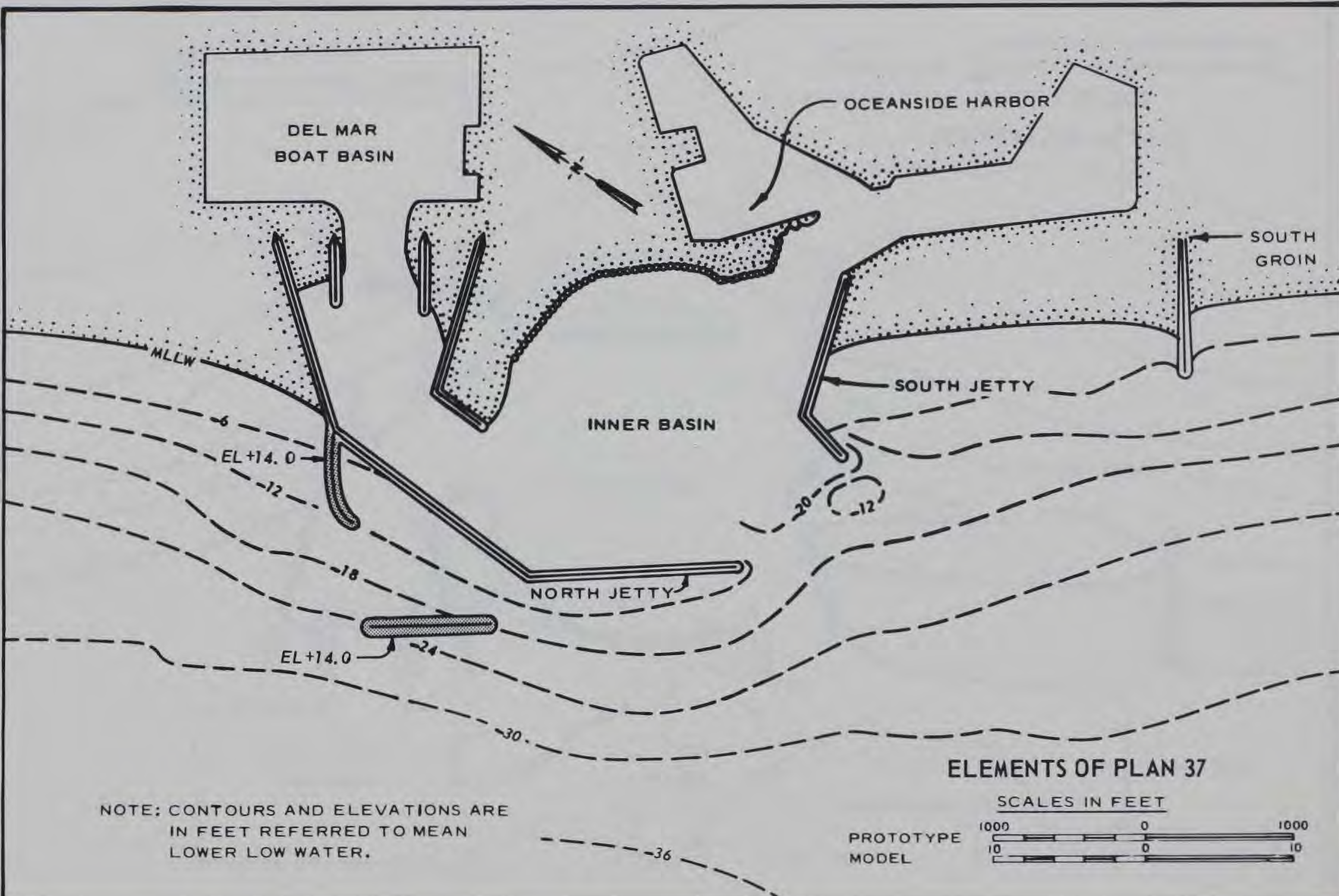


ELEMENTS OF PLANS 35 AND 35A



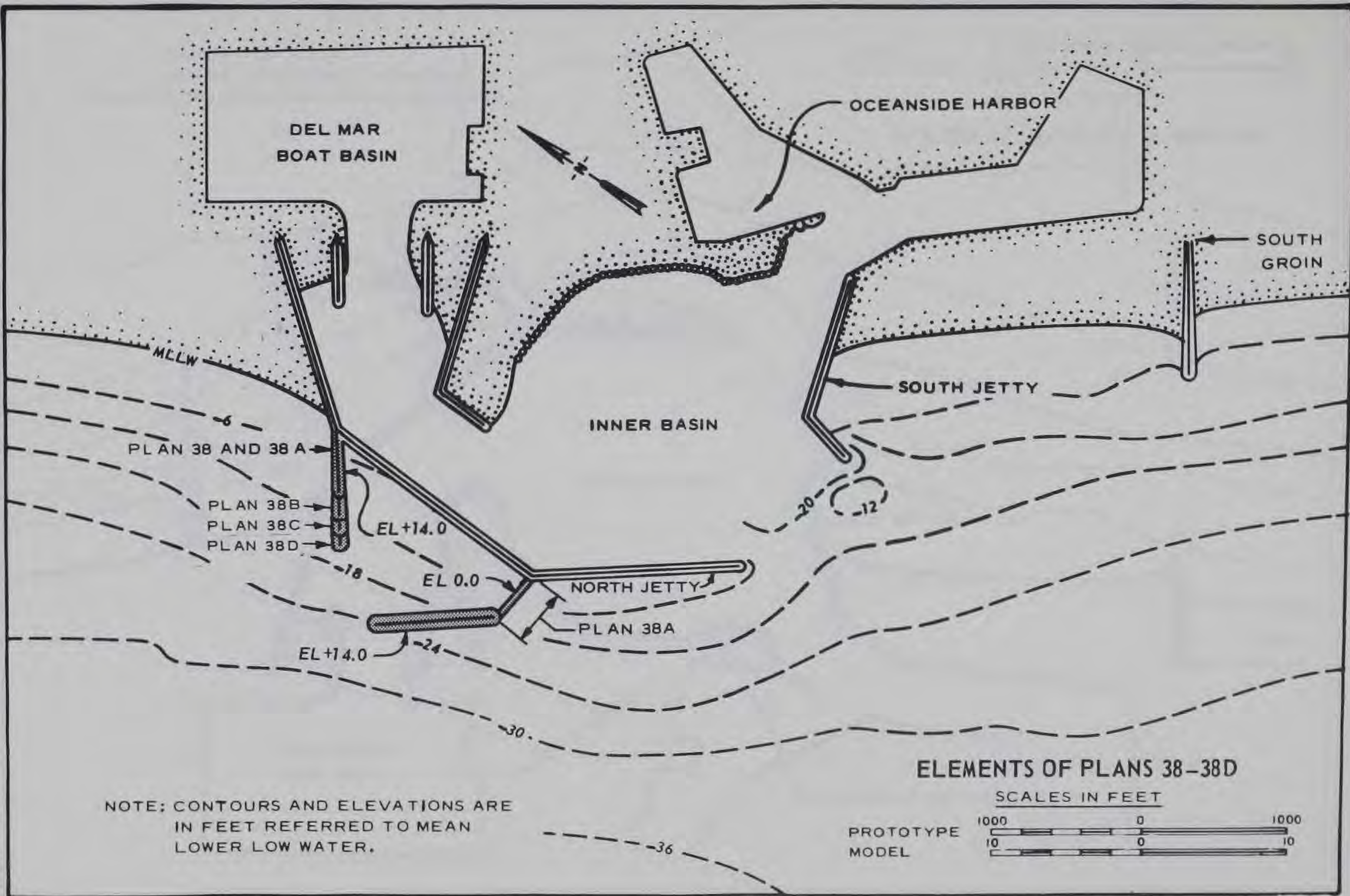
ELEMENTS OF PLAN 36

NOTE: CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER.



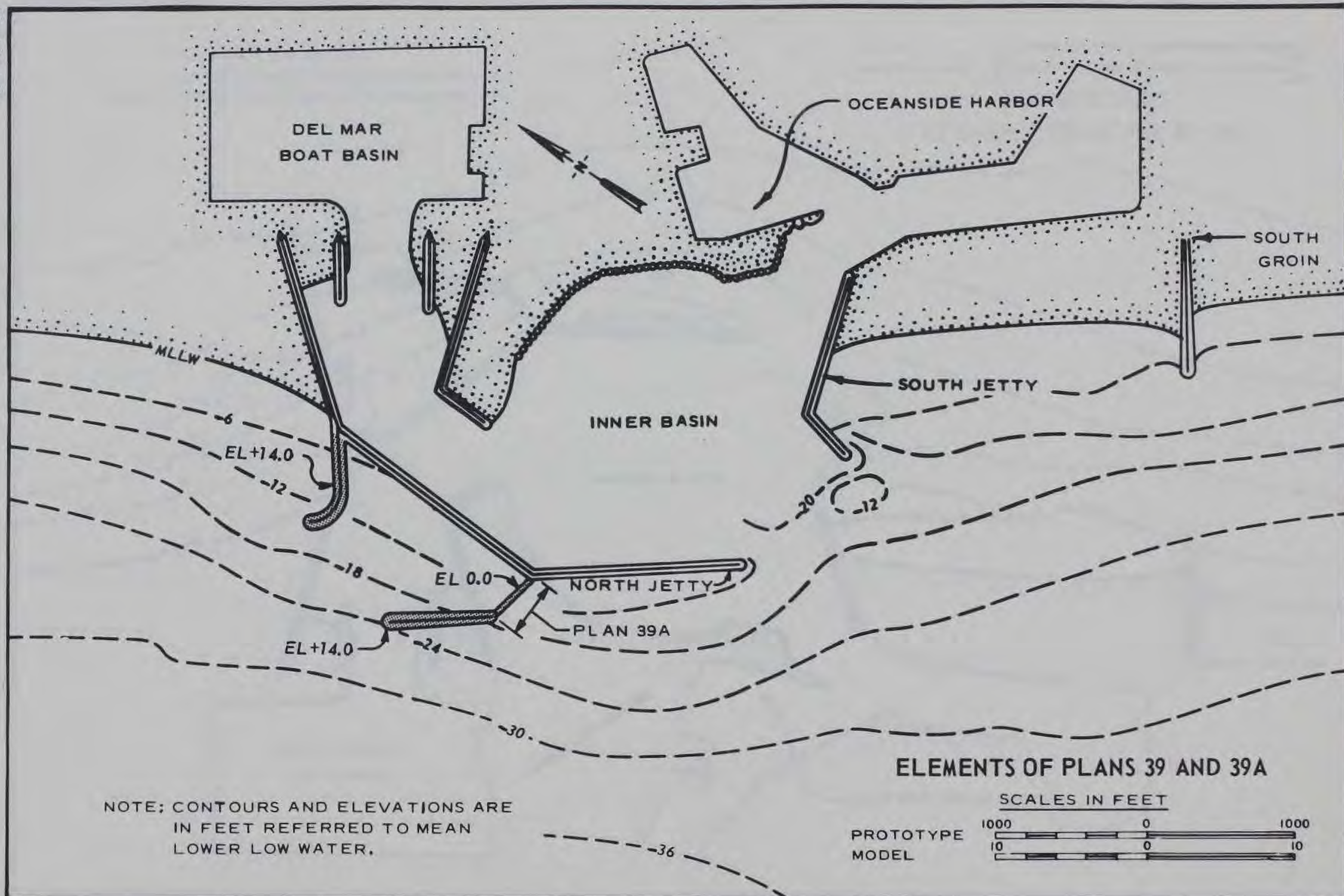
ELEMENTS OF PLAN 37

NOTE: CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER.



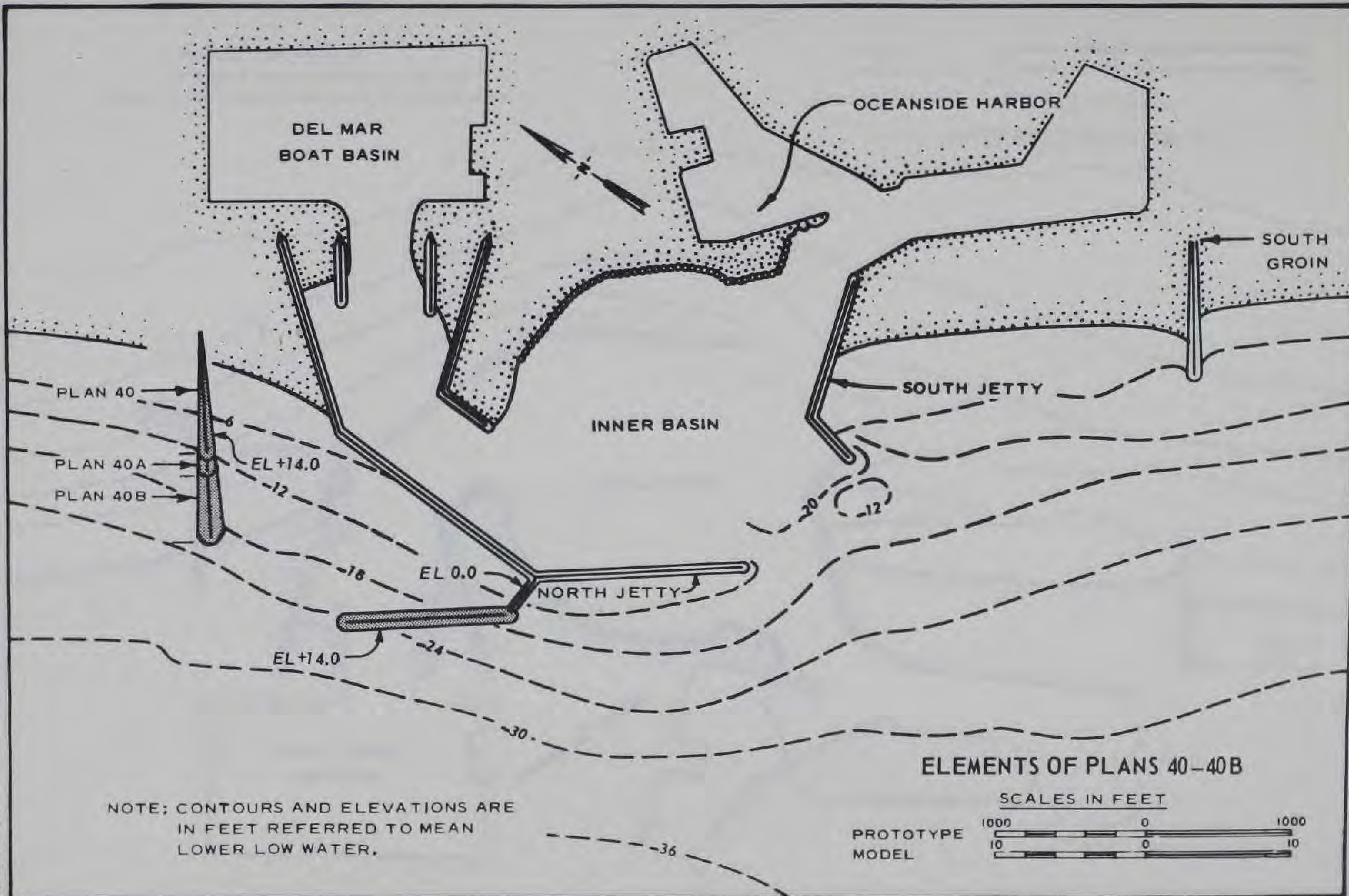
ELEMENTS OF PLANS 38–38D

PLATE 27



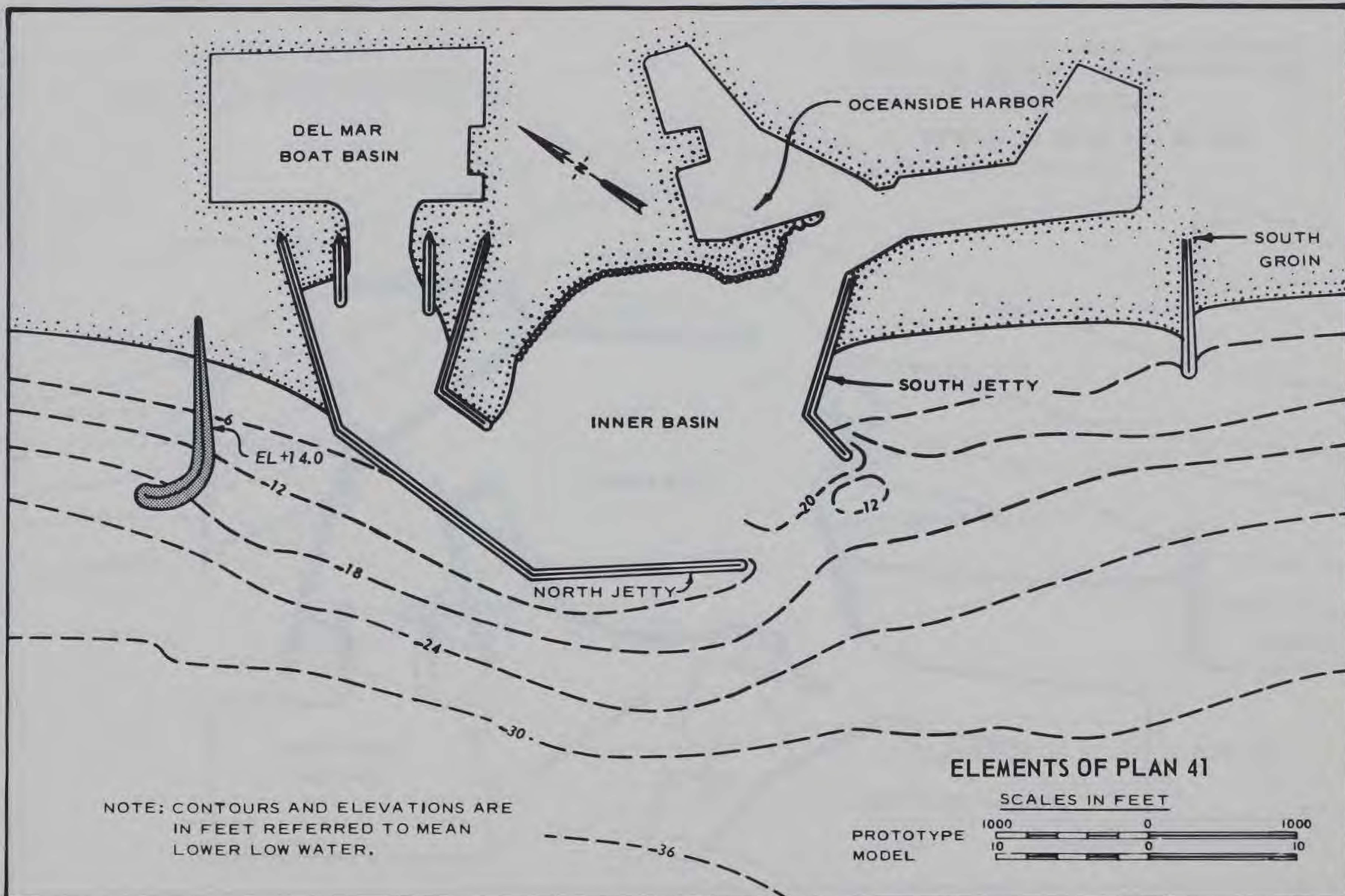
ELEMENTS OF PLANS 39 AND 39A

NOTE: CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER.

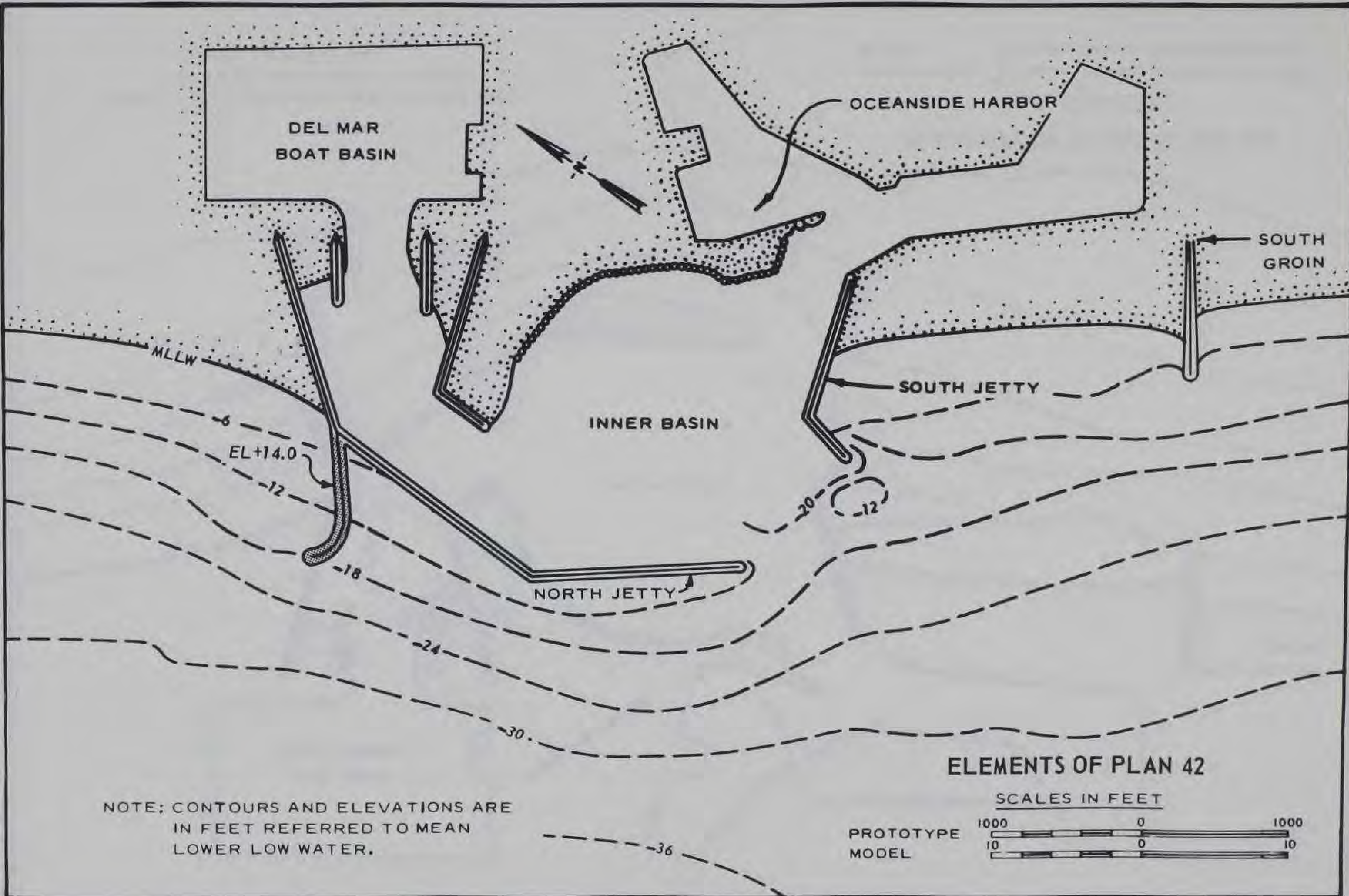


ELEMENTS OF PLANS 40–40B

NOTE: CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER.

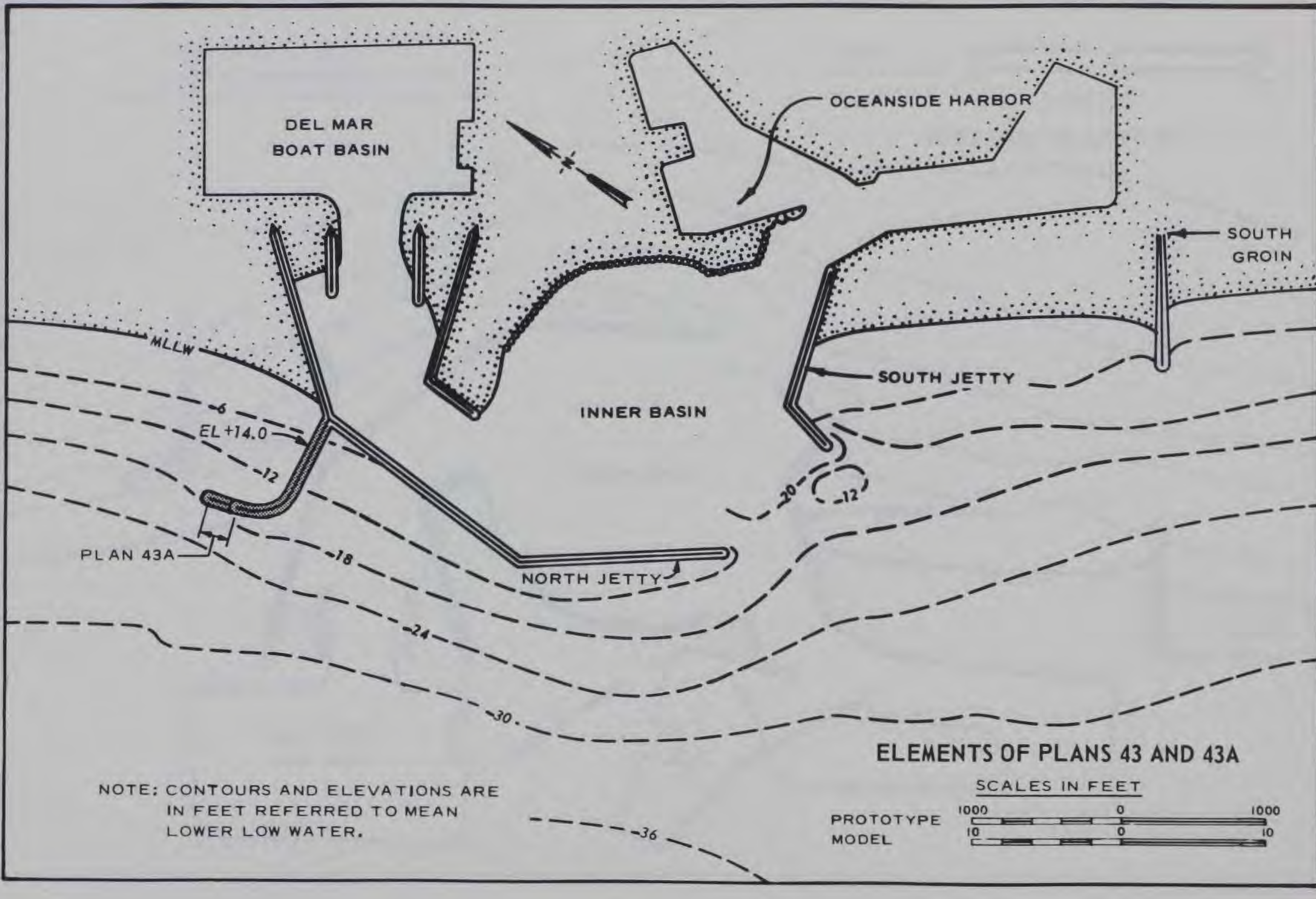


ELEMENTS OF PLAN 41



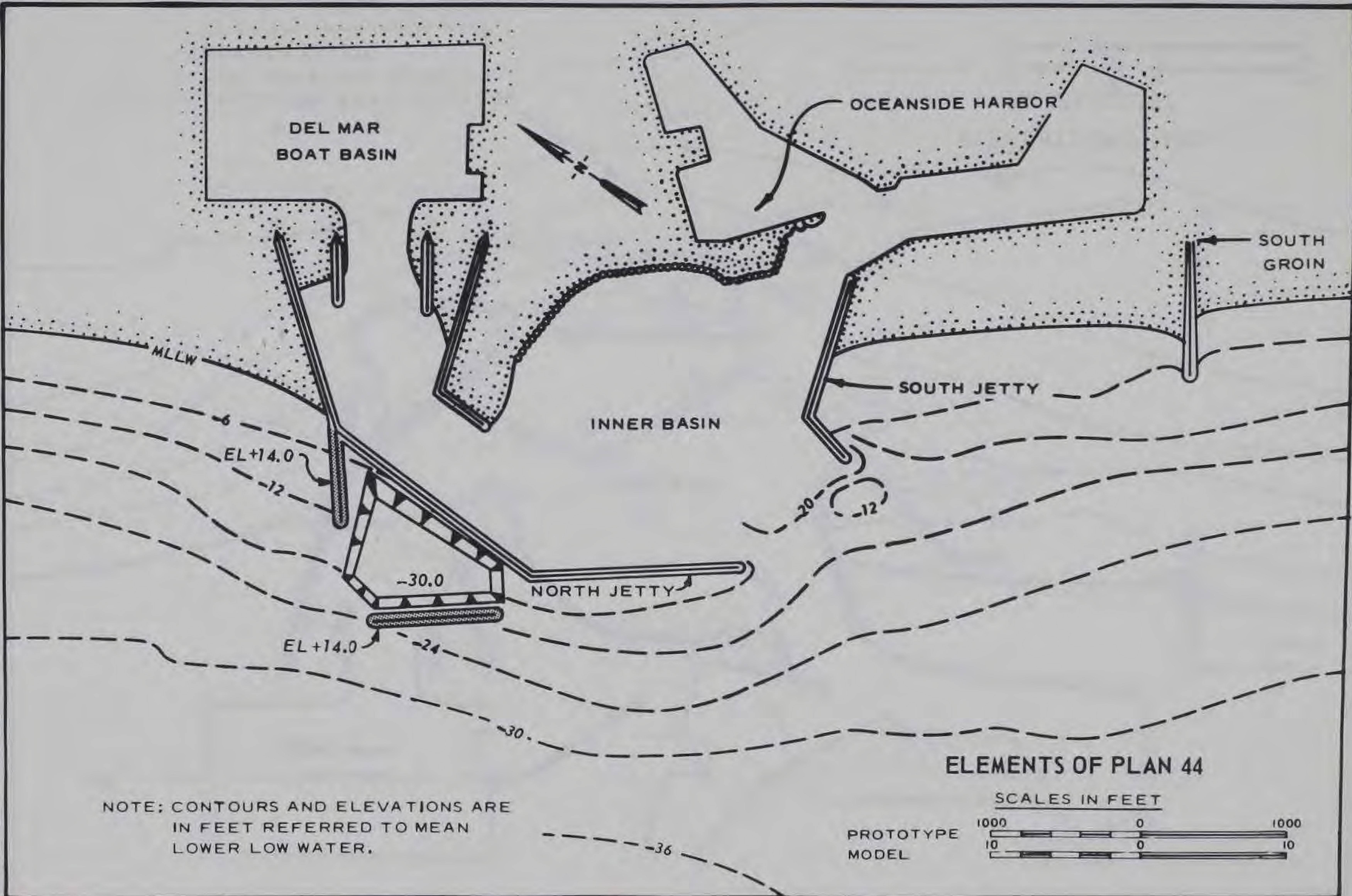
ELEMENTS OF PLAN 42

NOTE: CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER.



ELEMENTS OF PLANS 43 AND 43A

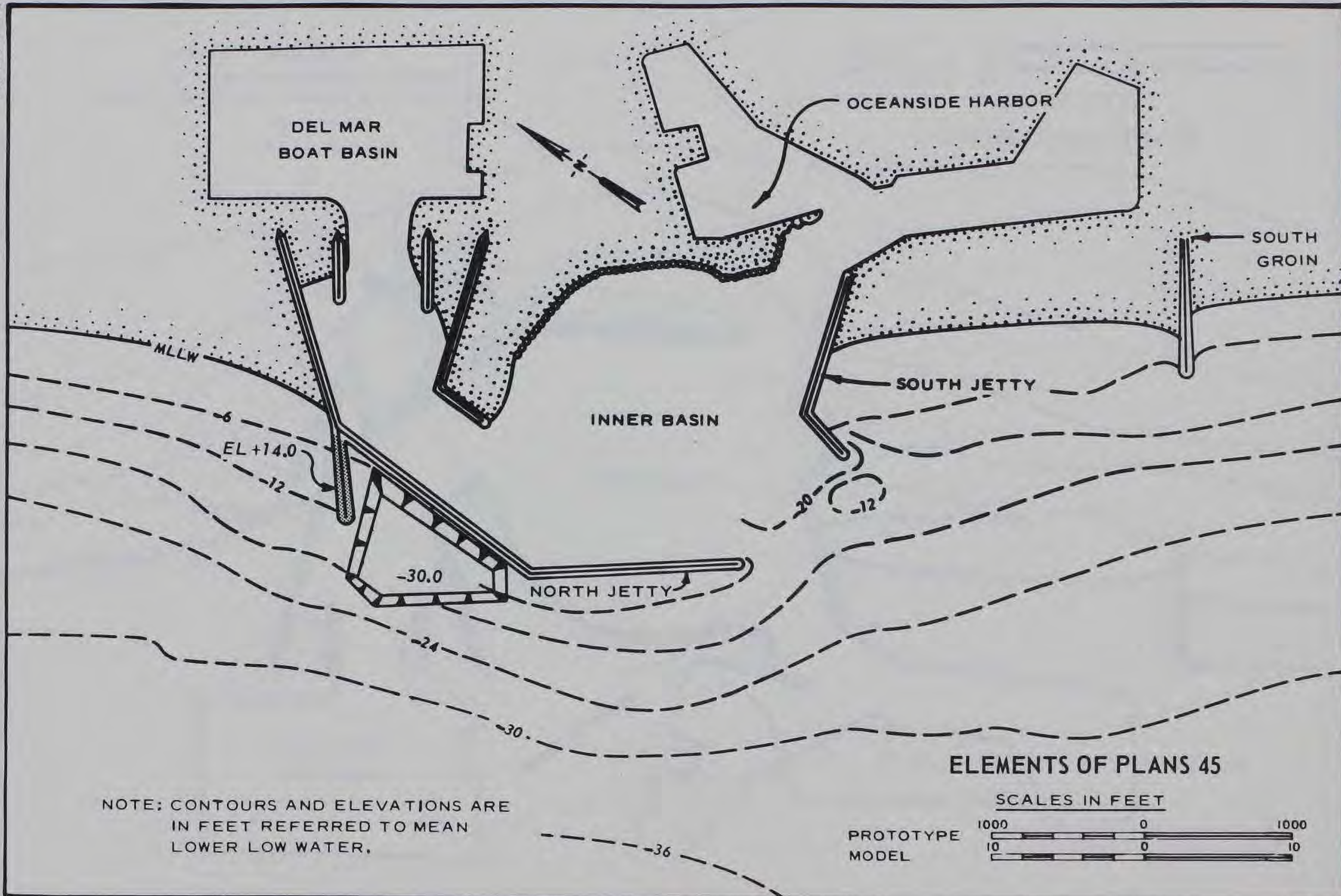
NOTE: CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER.



ELEMENTS OF PLAN 44

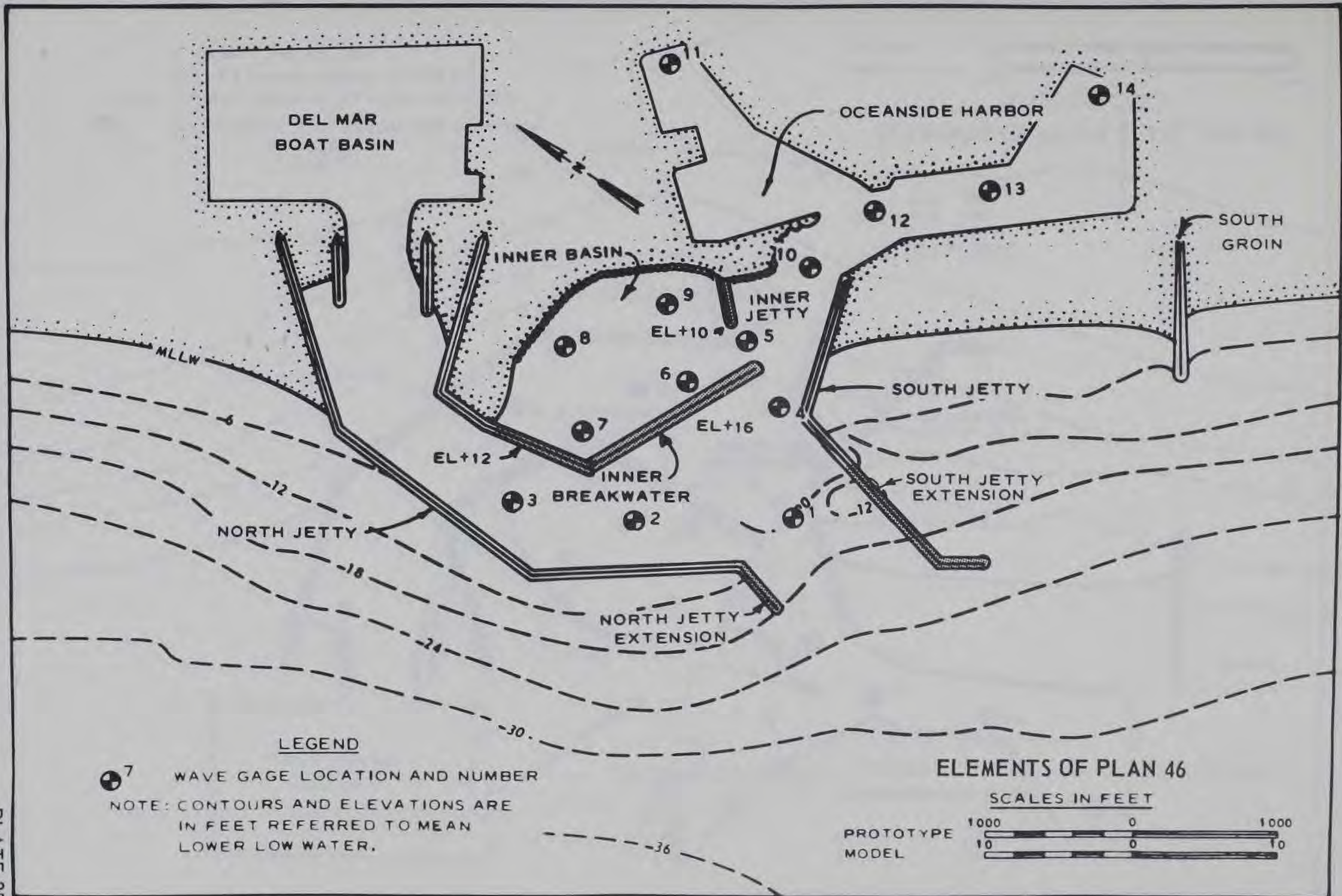
NOTE: CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER.

PLATE 34

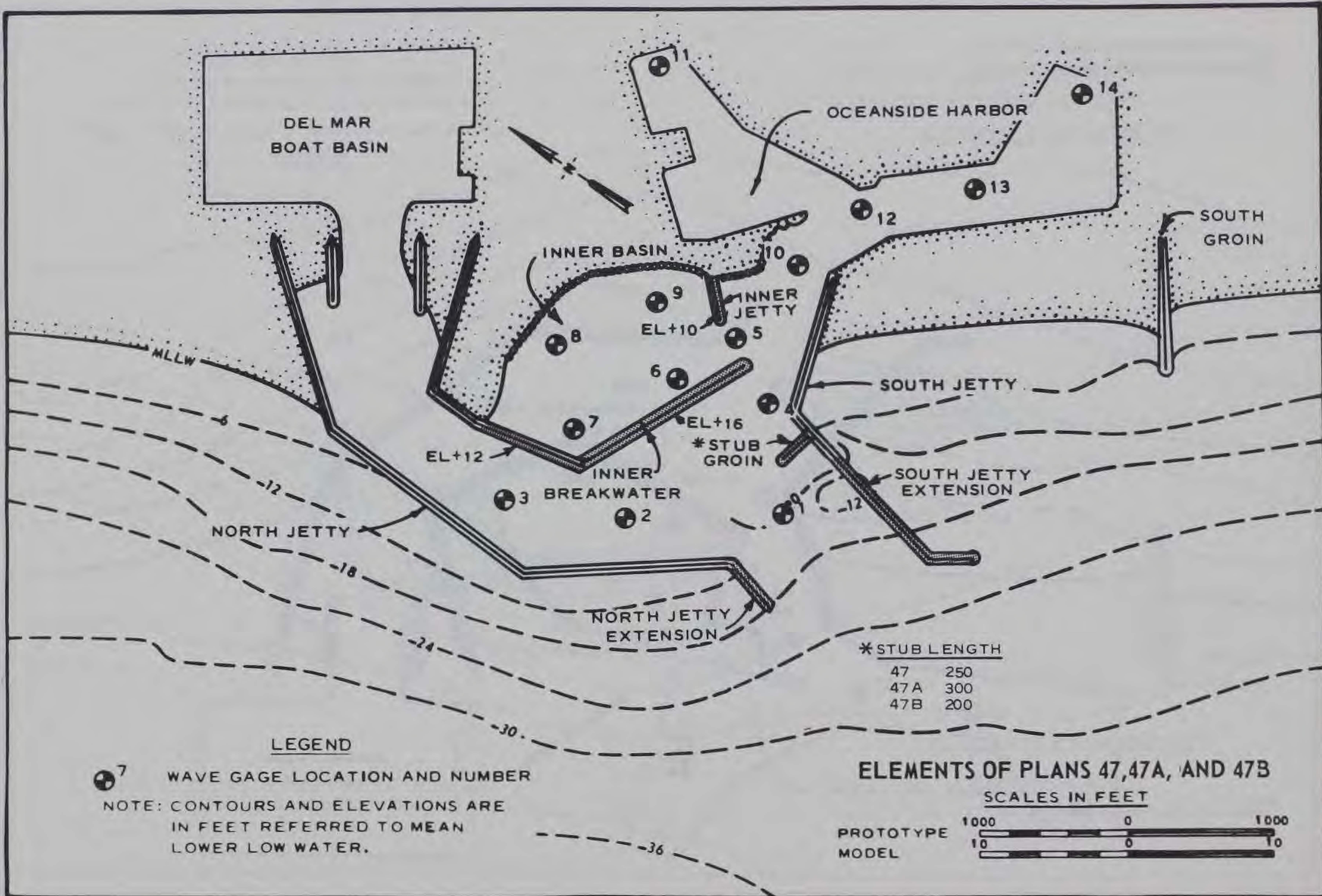


ELEMENTS OF PLANS 45

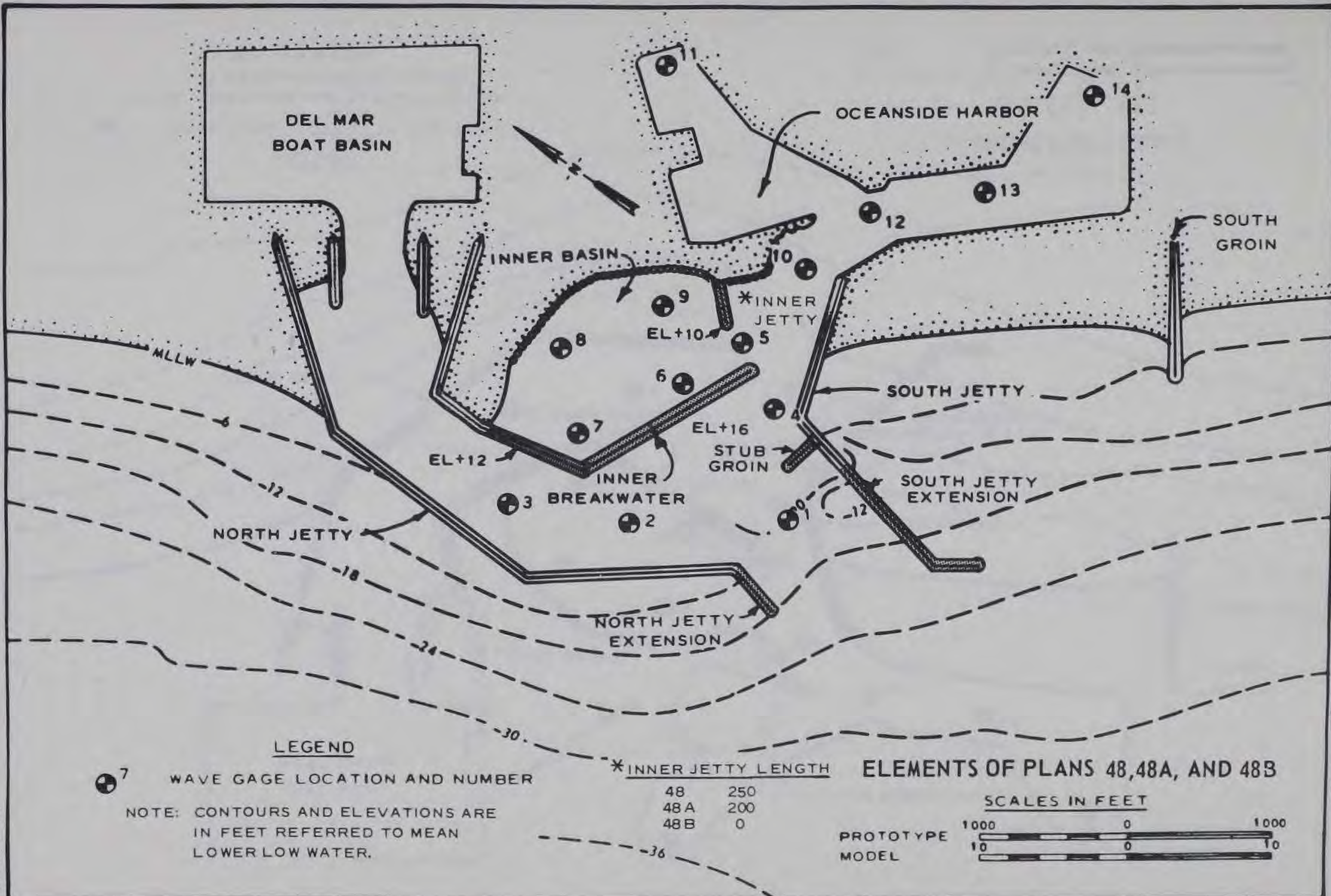
NOTE: CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER.



DEL MAR
BOAT BASIN
N
OCEANSIDE HARBOR
11
14
13
12
SOUTH
GROIN
INNER BASIN
10
9
INNER
JETTY
EL+10
5
8
MLLW
6
SOUTH JETTY
4
7
EL+16
-6
EL+12
INNER
BREAKWATER
SOUTH JETTY
EXTENSION
-12
3
2
1
NORTH JETTY
-18
NORTH JETTY
EXTENSION
-24
-30
LEGEND
7
WAVE GAGE LOCATION AND NUMBER
NOTE: CONTOURS AND ELEVATIONS ARE
IN FEET REFERRED TO MEAN
LOWER LOW WATER.
-36
ELEMENTS OF PLAN 46
SCALES IN FEET
PROTOTYPE
MODEL
1000
0
1000
10
0
10

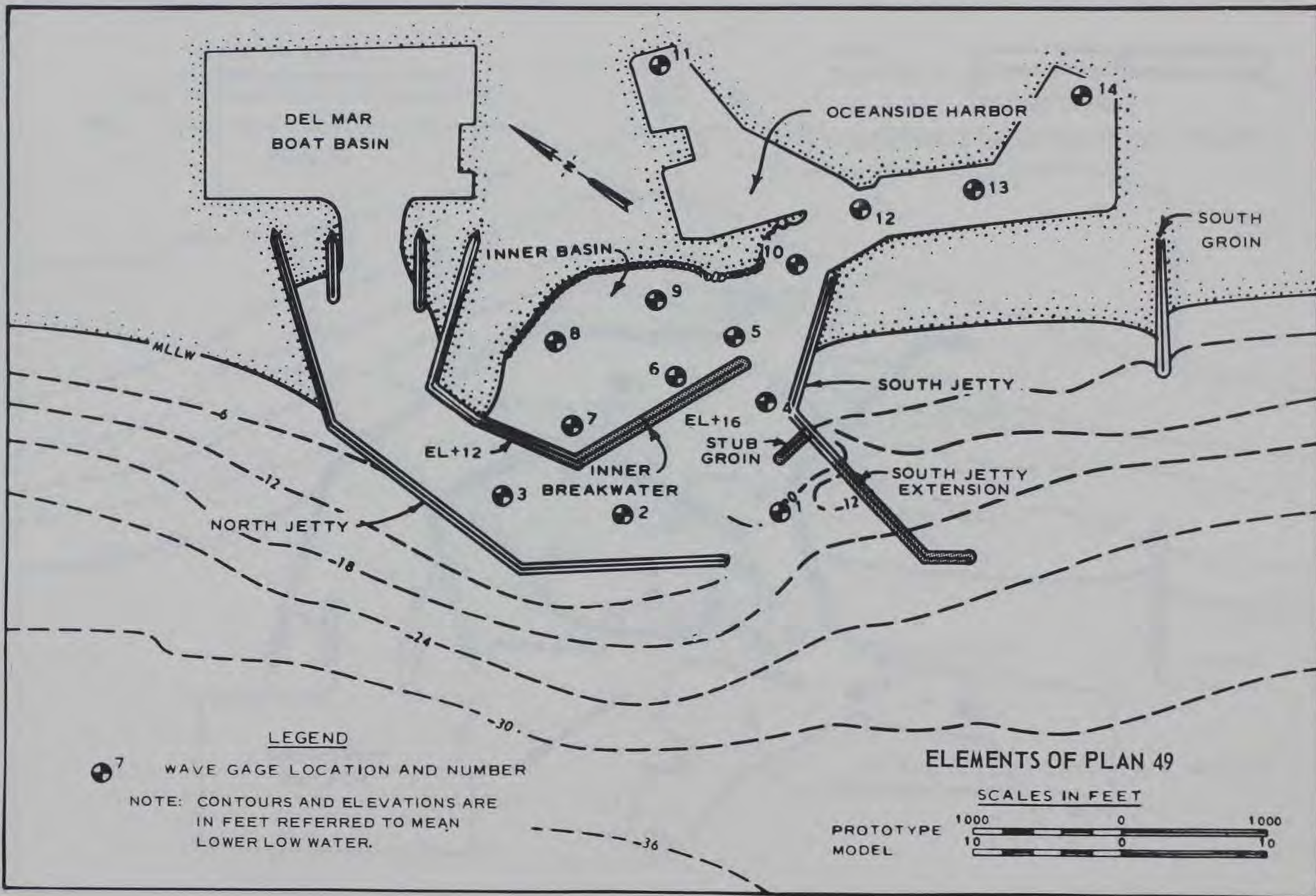


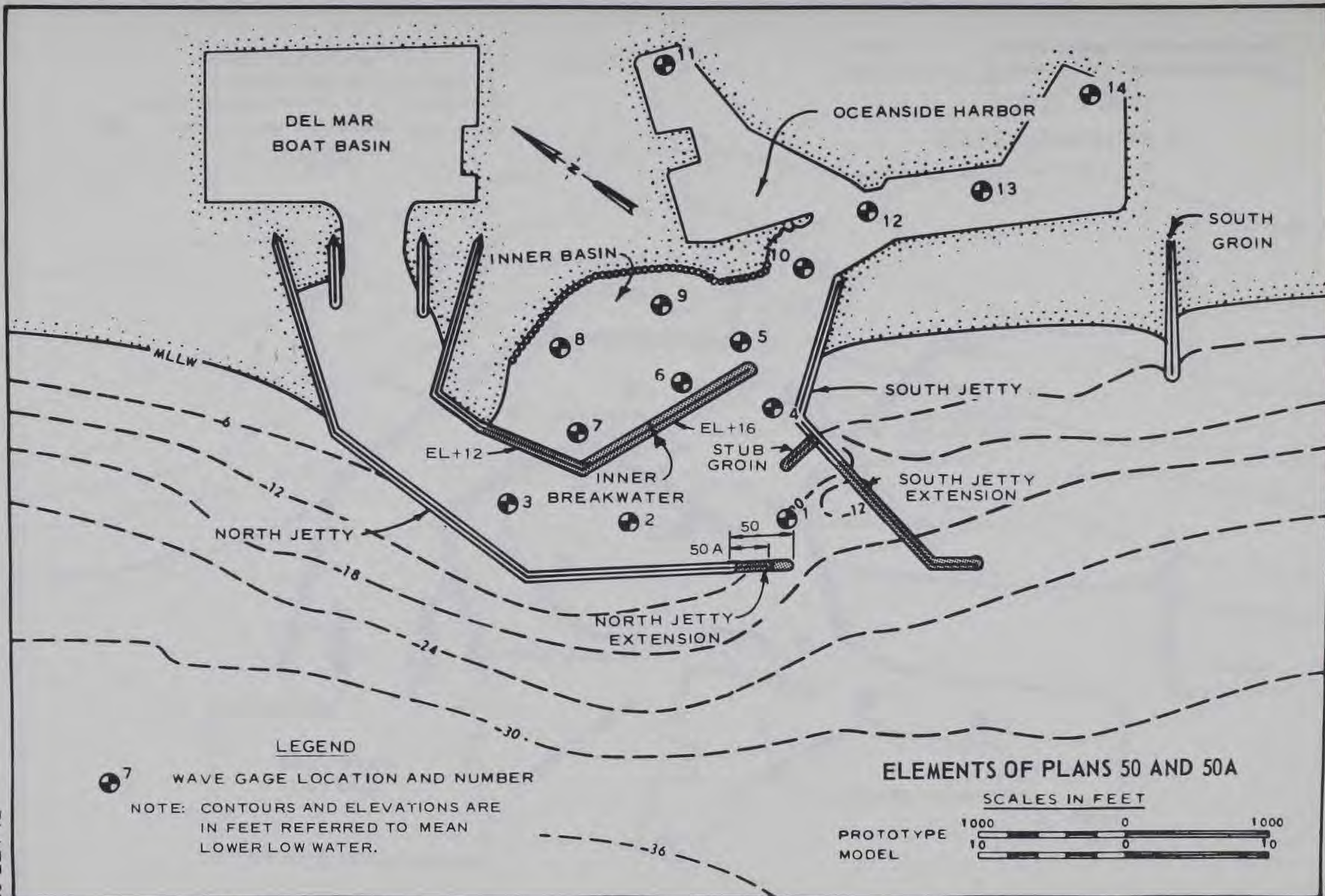
ELEMENTS OF PLANS 47,47A, AND 47B



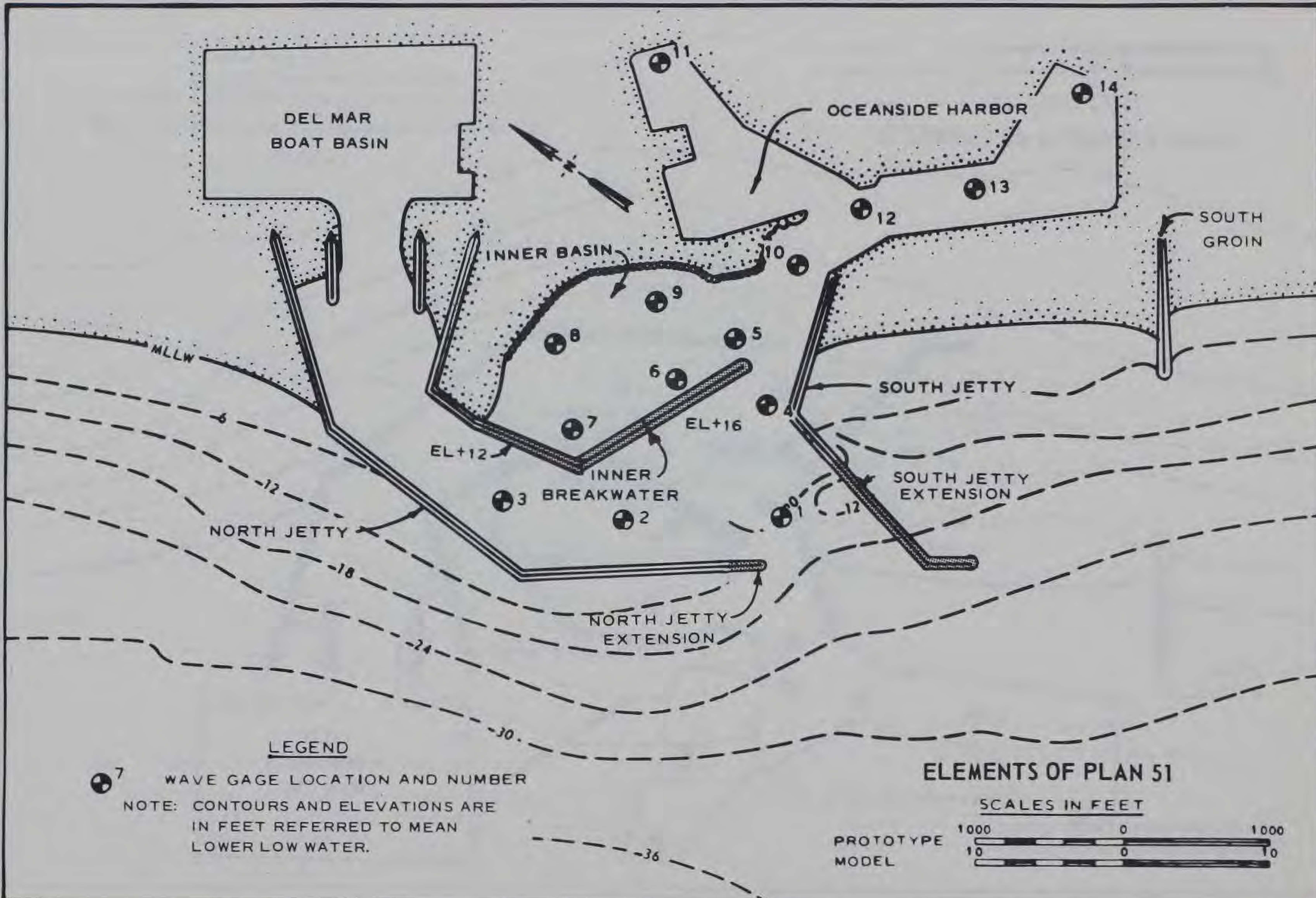
ELEMENTS OF PLANS 48,48A, AND 48B

PLATE 37

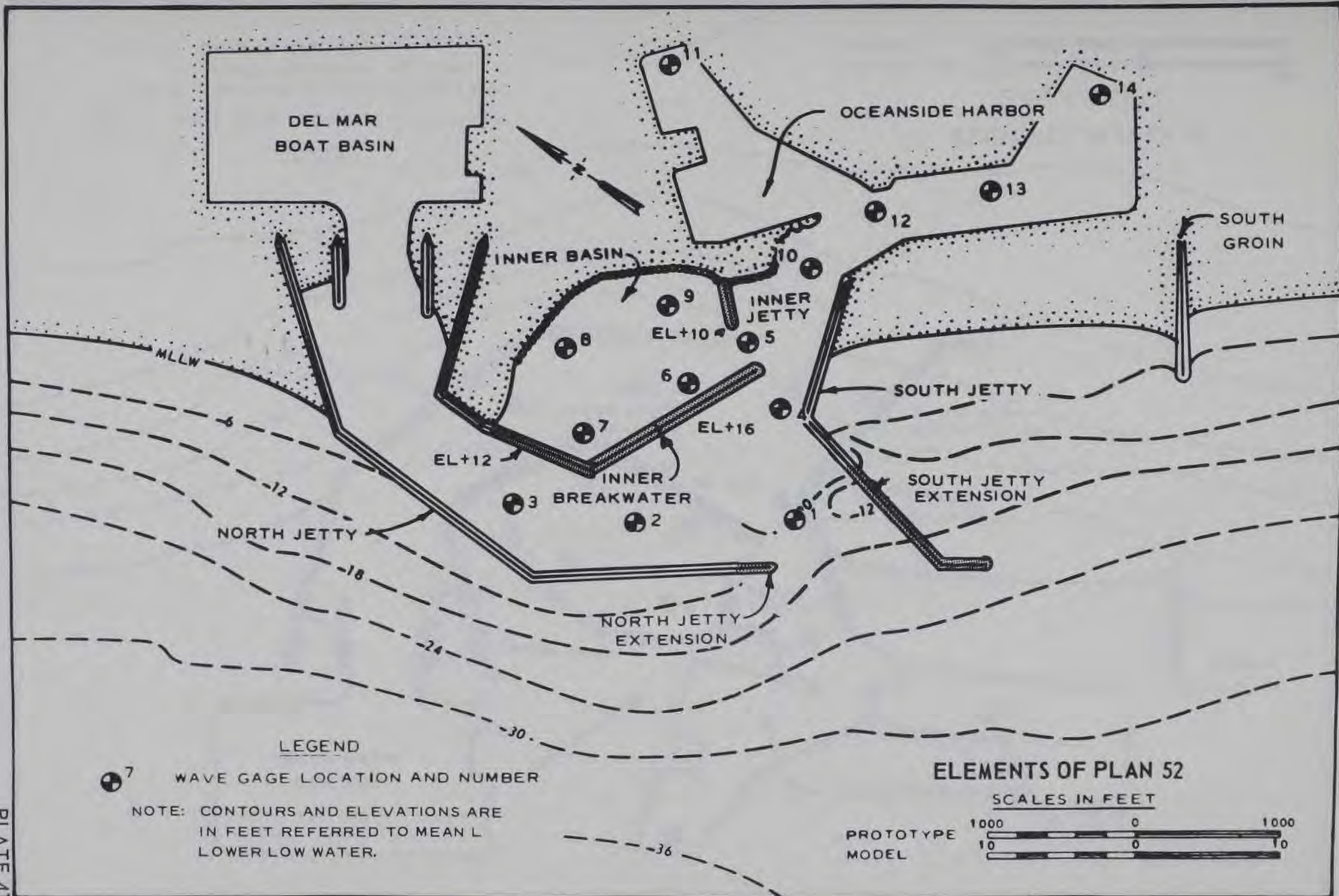

DEL MAR
BOAT BASIN
N
OCEANSIDE HARBOR
INNER BASIN
SOUTH
GROIN
MLLW
SOUTH JETTY
EL+16
STUB
GROIN
EL+12
INNER
BREAKWATER
SOUTH JETTY
EXTENSION
NORTH JETTY
-6
-12
-18
-24
-30
-36
1
2
3
4
5
6
7
8
9
10
11
12
13
14
LEGEND
WAVE GAGE LOCATION AND NUMBER
NOTE: CONTOURS AND ELEVATIONS ARE
IN FEET REFERRED TO MEAN
LOWER LOW WATER.
ELEMENTS OF PLAN 49
SCALES IN FEET
PROTOTYPE
MODEL
1000
0
1000
10
0
10



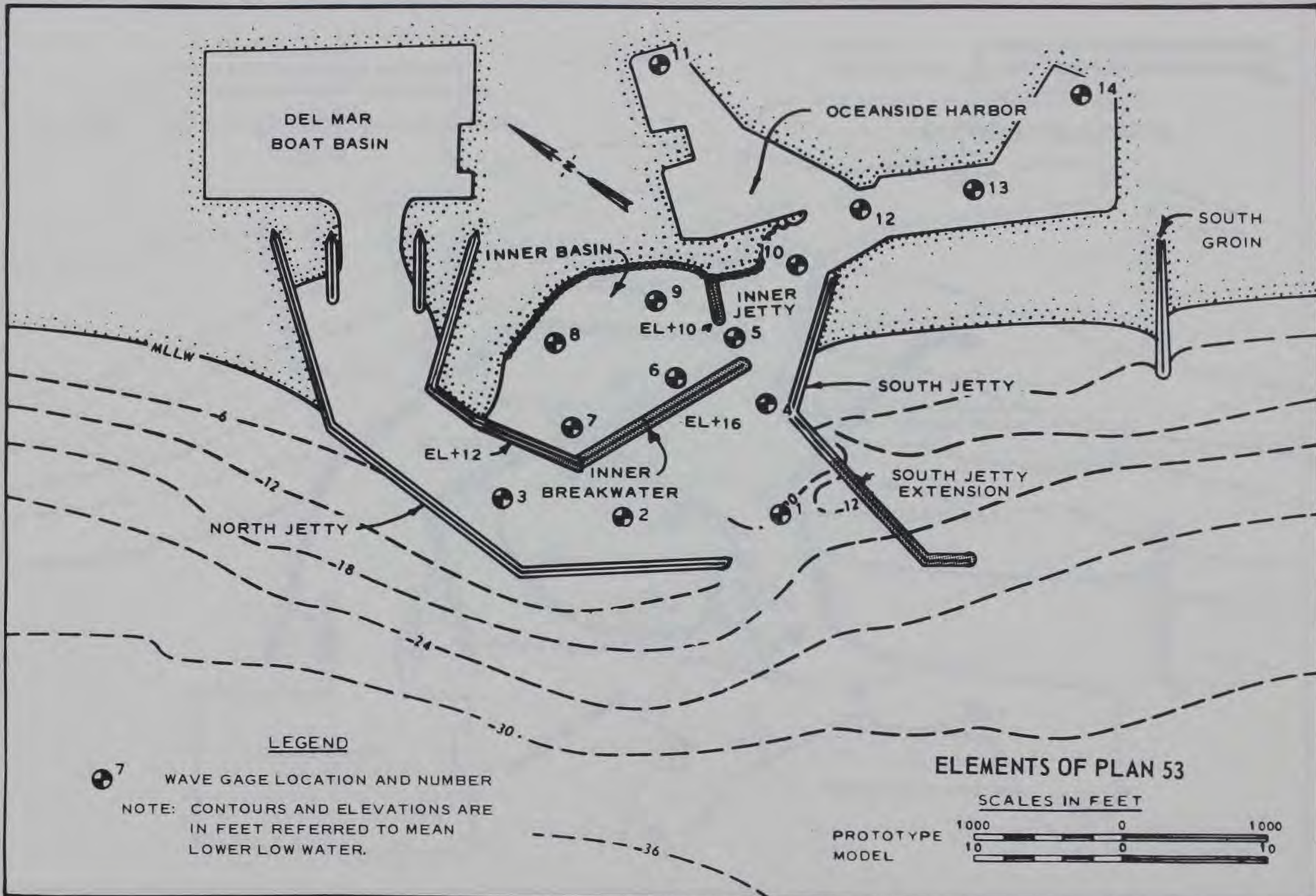
ELEMENTS OF PLANS 50 AND 50A



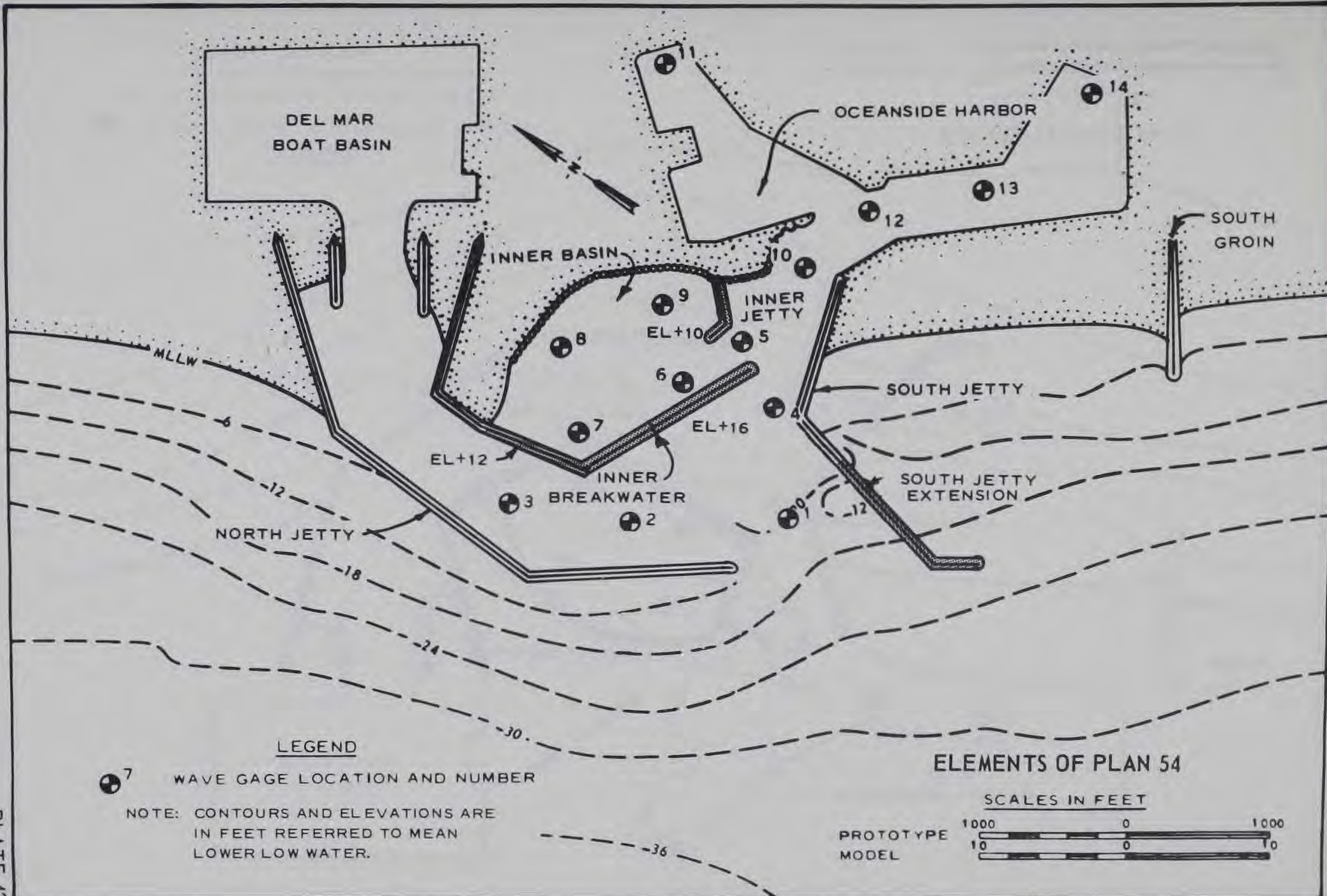
ELEMENTS OF PLAN 51



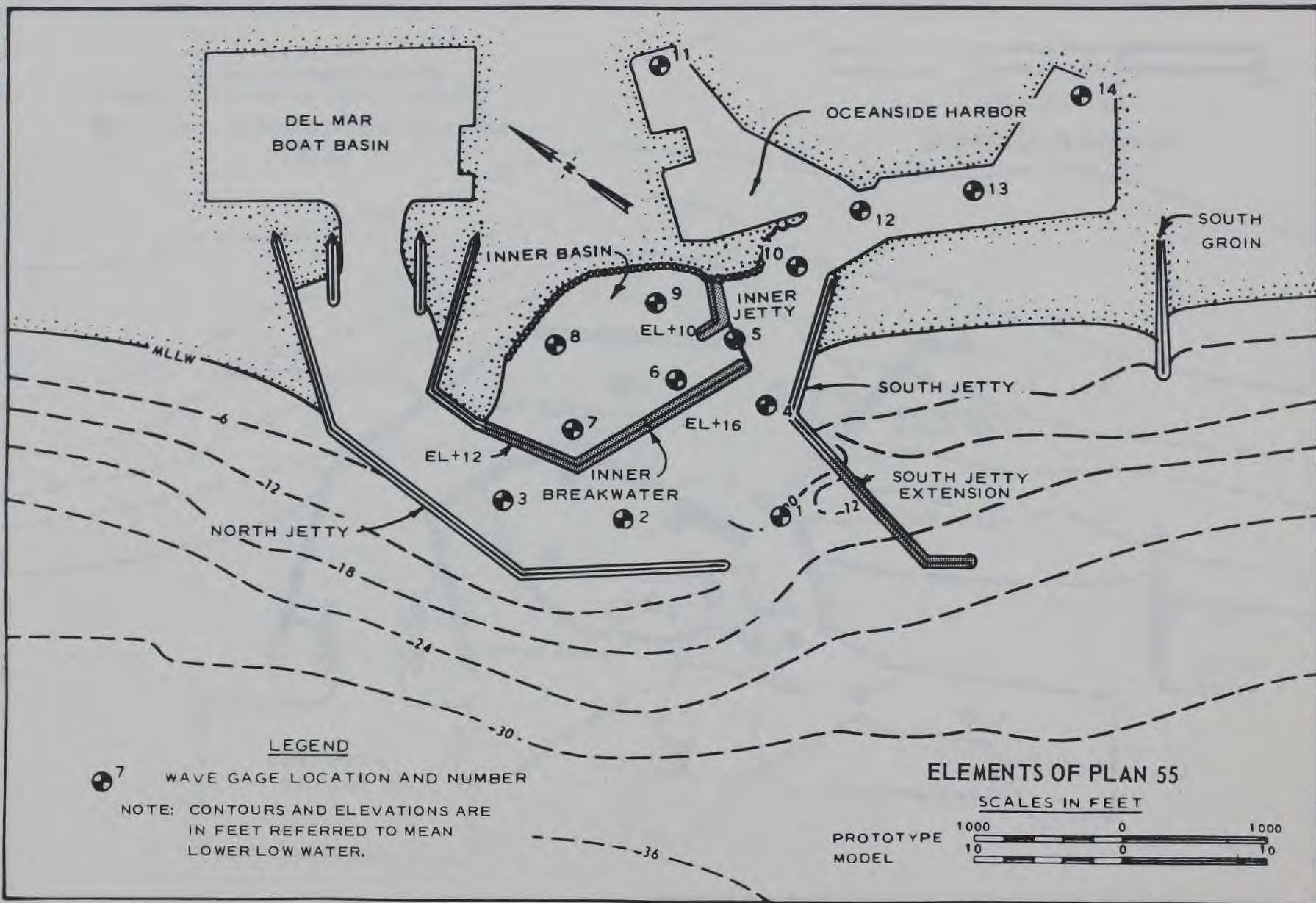
ELEMENTS OF PLAN 52



ELEMENTS OF PLAN 53



ELEMENTS OF PLAN 54



ELEMENTS OF PLAN 55

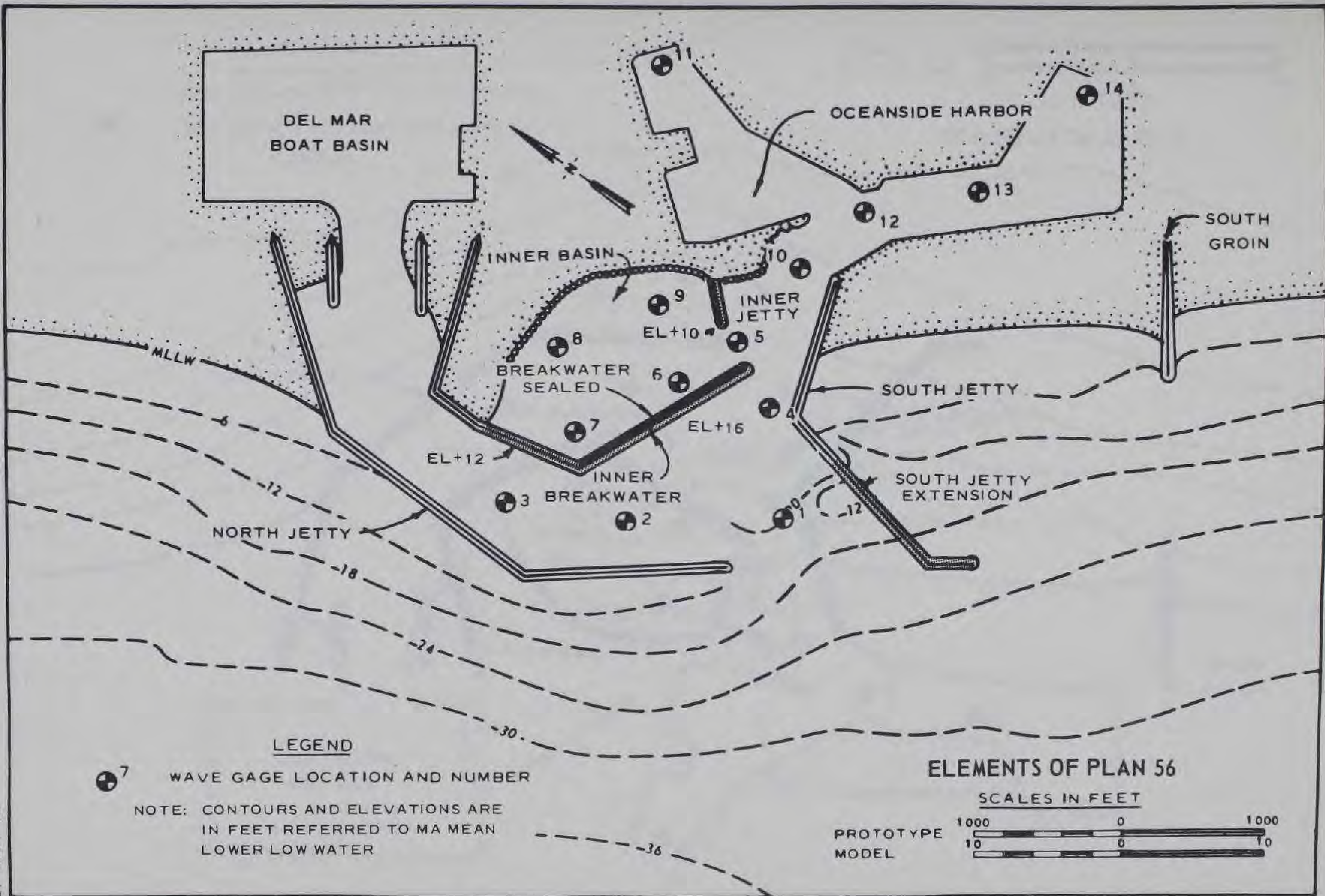
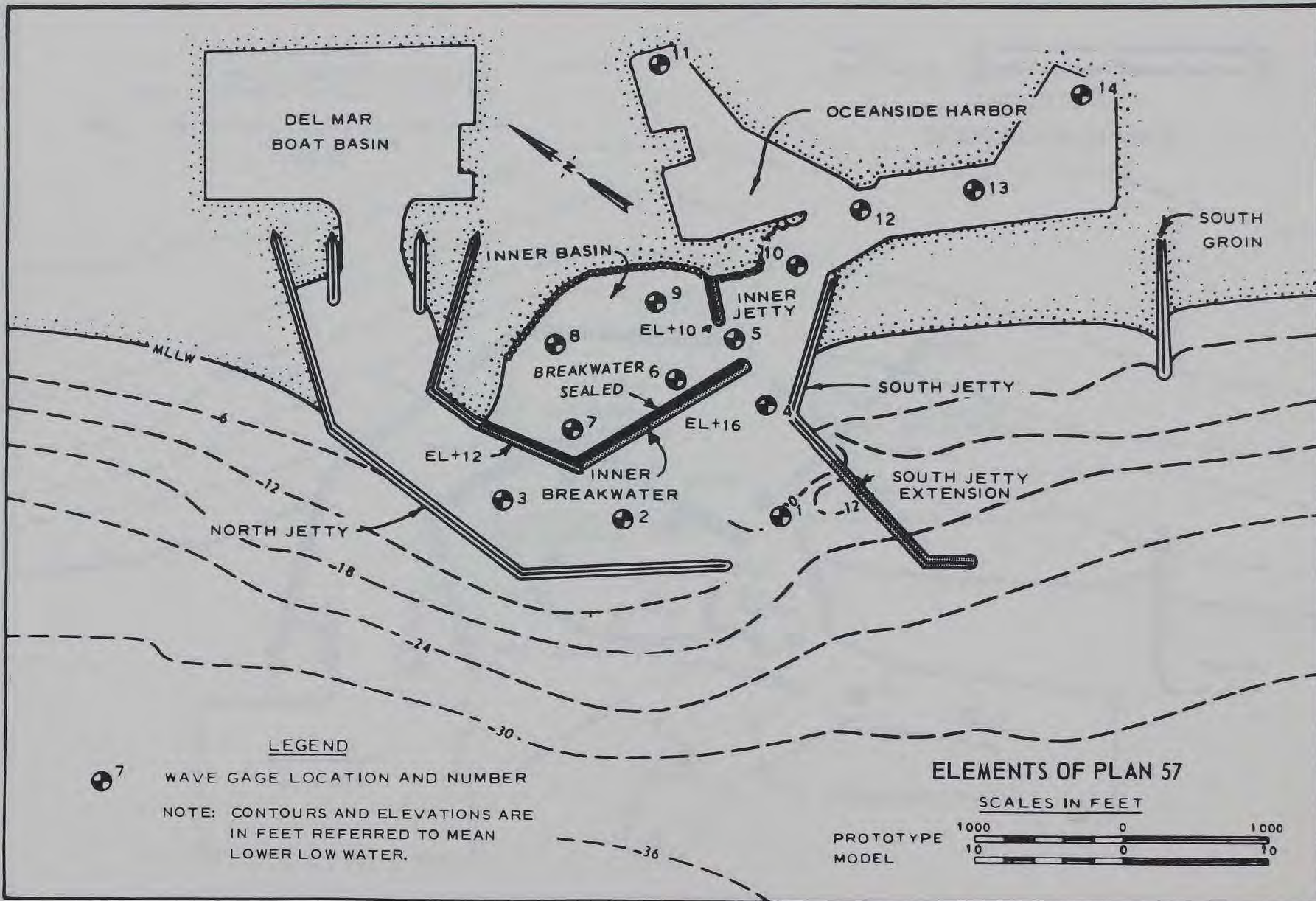
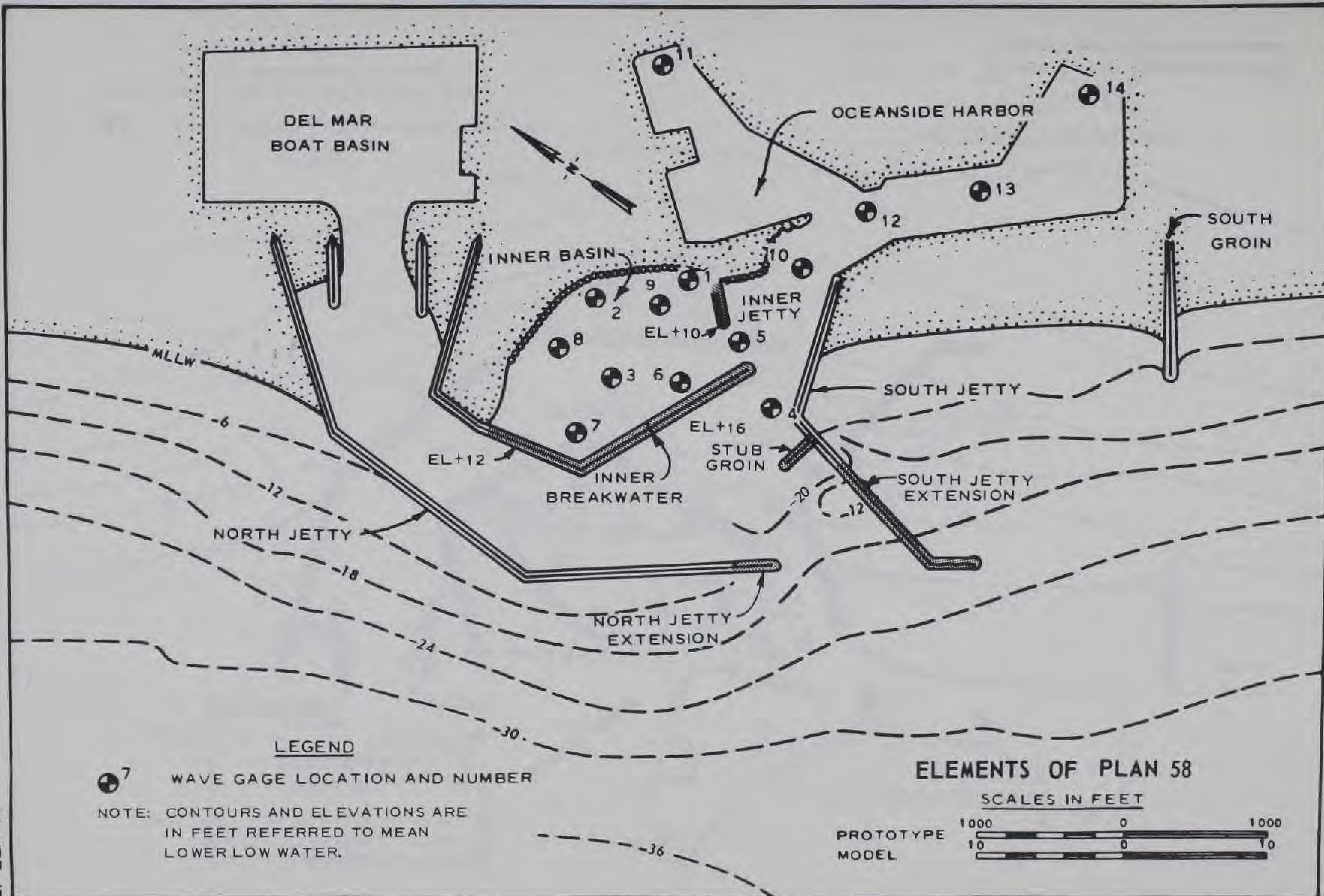

DEL MAR
BOAT BASIN
OCEANSIDE HARBOR
N
INNER BASIN
INNER JETTY
EL+10
BREAKWATER
SEALED
EL+16
EL+12
INNER
BREAKWATER
SOUTH JETTY
SOUTH JETTY
EXTENSION
SOUTH
GROIN
NORTH JETTY
MLLW
-6
-12
-18
-24
-30
-36
LEGEND
WAVE GAGE LOCATION AND NUMBER
NOTE: CONTOURS AND ELEVATIONS ARE
IN FEET REFERRED TO MA MEAN
LOWER LOW WATER
ELEMENTS OF PLAN 56
SCALES IN FEET
PROTOTYPE
MODEL
1000 0 1000
10 0 10

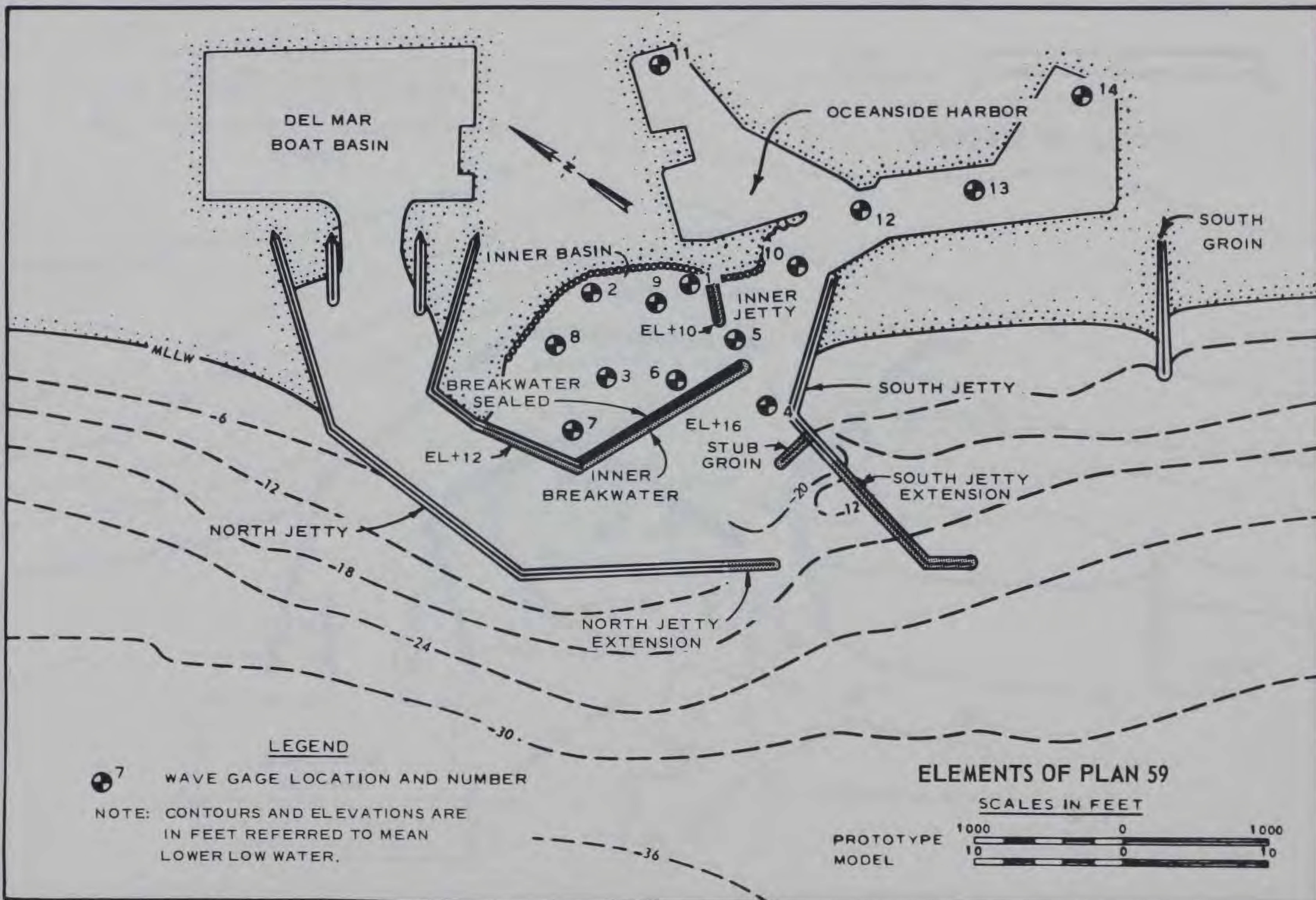
PLATE 45



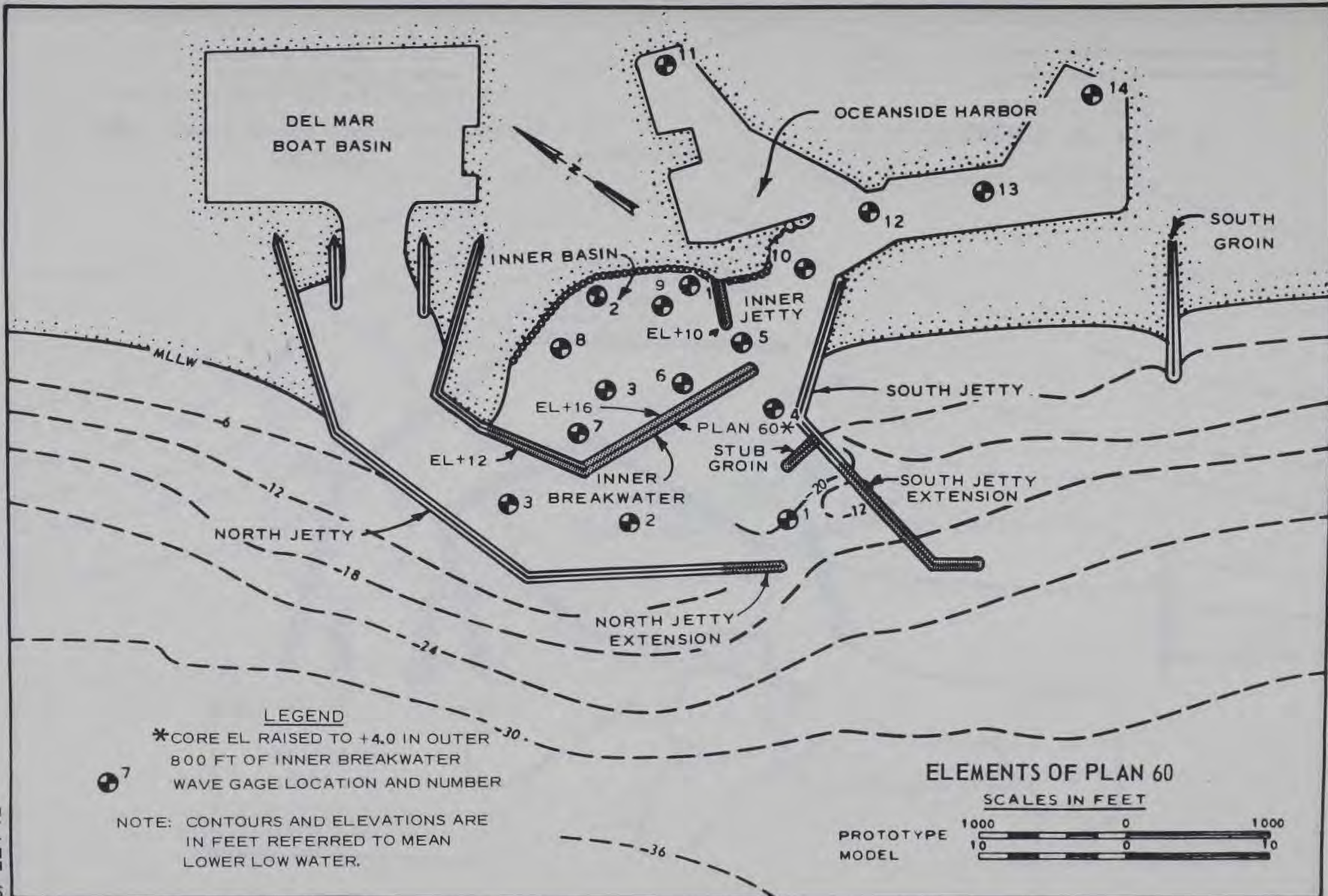
ELEMENTS OF PLAN 57

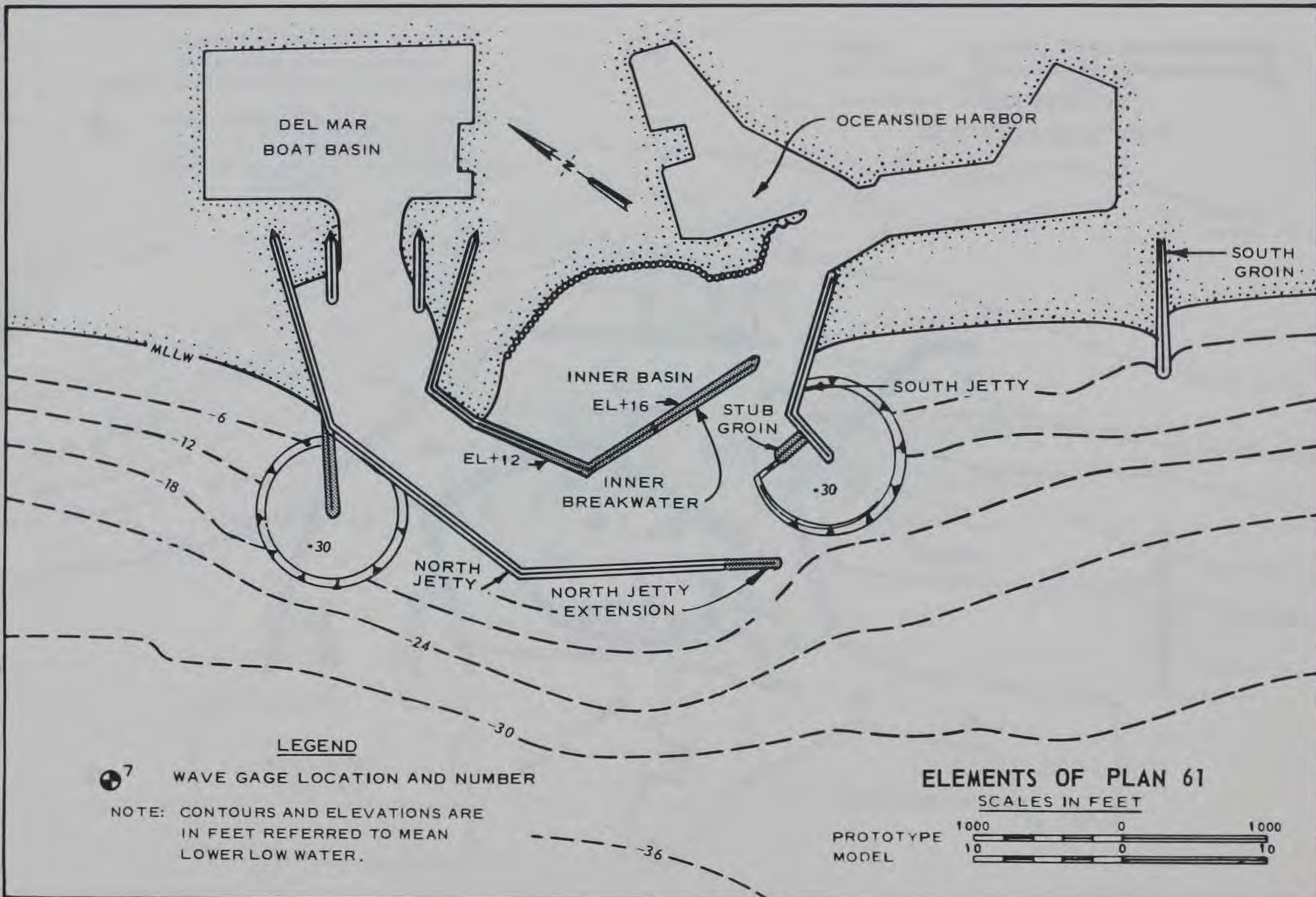


ELEMENTS OF PLAN 58

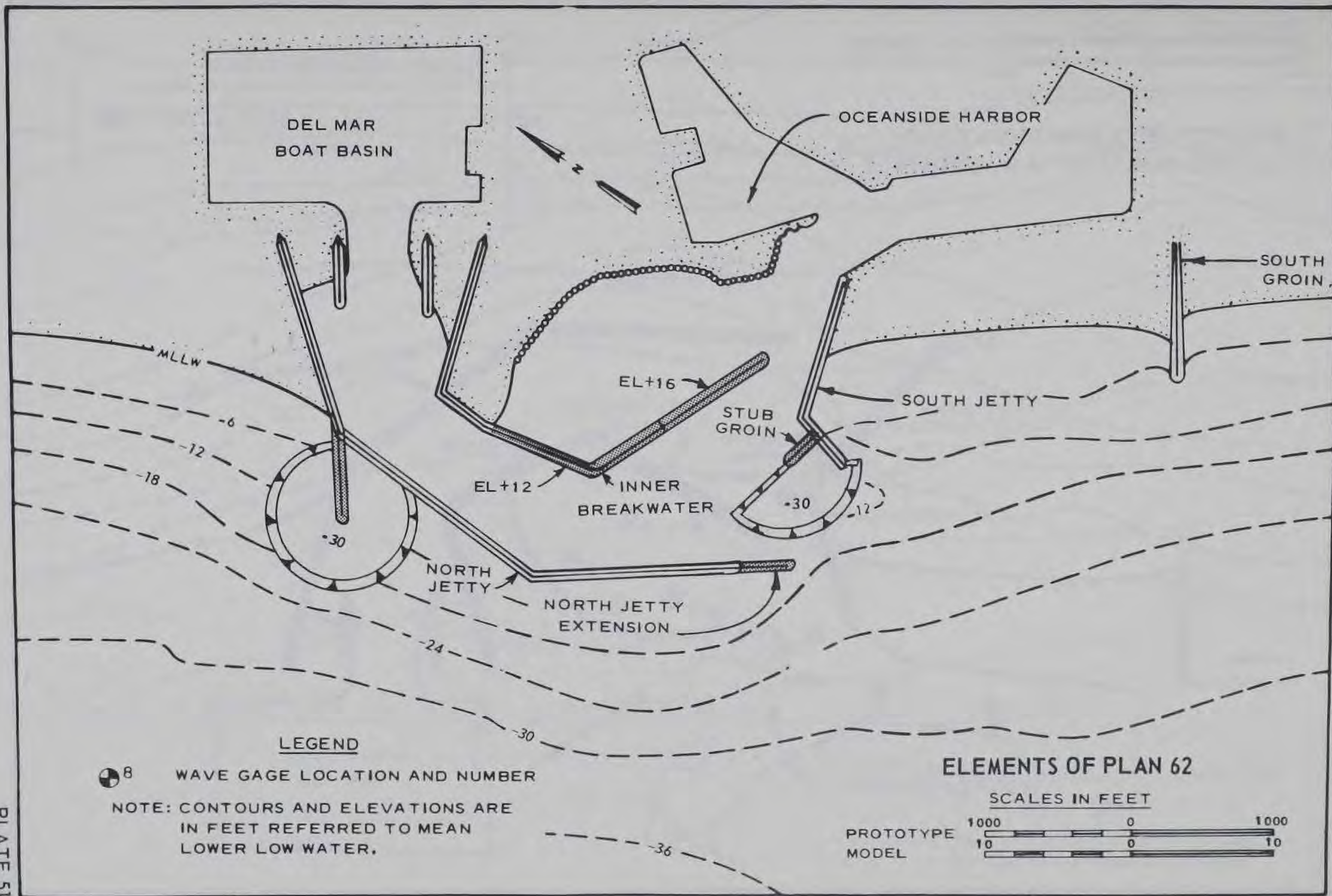


ELEMENTS OF PLAN 59

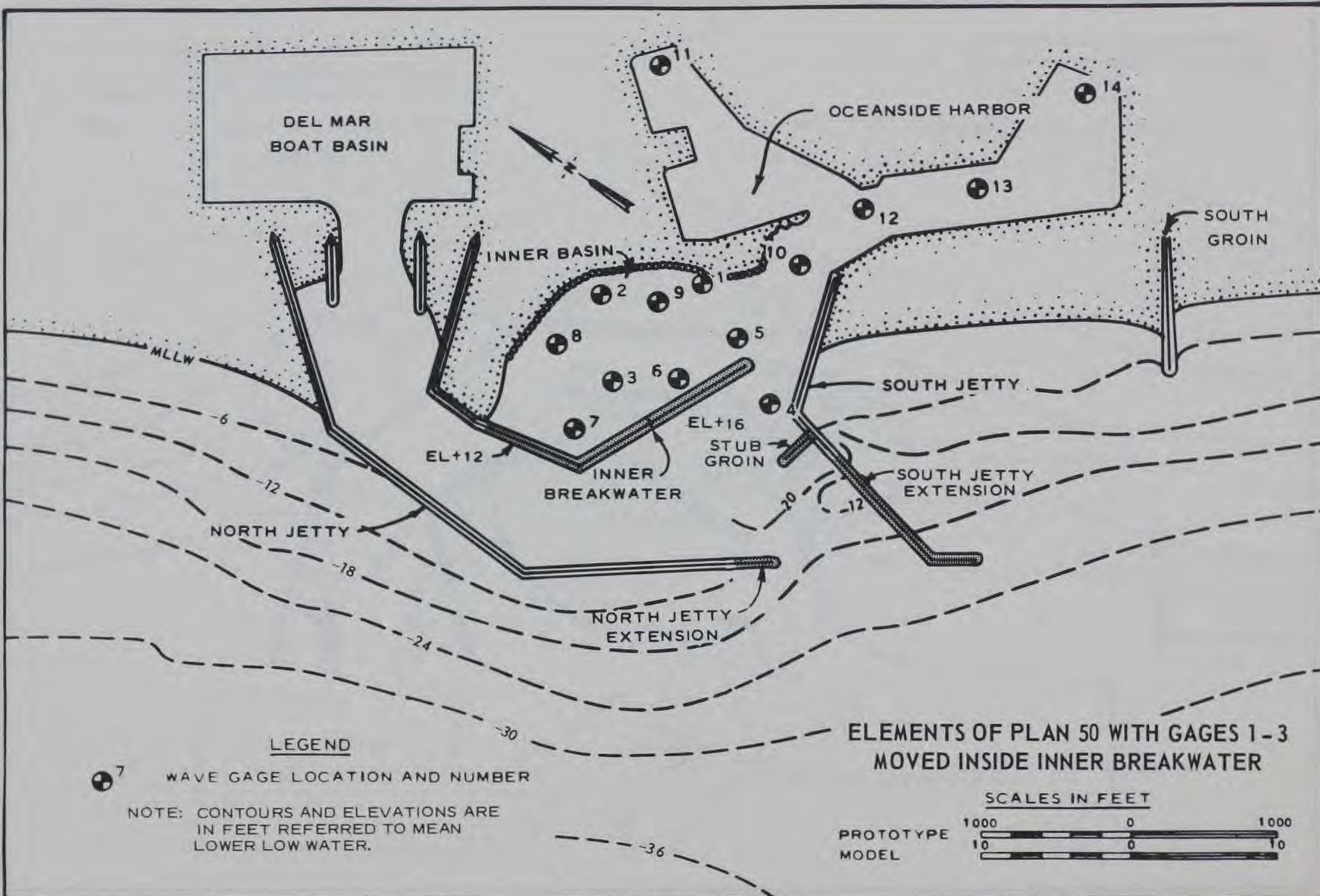

DEL MAR
BOAT BASIN
OCEANSIDE HARBOR
SOUTH
GROIN
INNER BASIN
INNER
JETTY
EL+10
EL+16
EL+12
MLLW
SOUTH JETTY
PLAN 60*
STUB
GROIN
INNER
BREAKWATER
SOUTH JETTY
EXTENSION
NORTH JETTY
NORTH JETTY
EXTENSION
-6
-12
-18
-20
-24
-30
-36
LEGEND
*CORE EL RAISED TO +4.0 IN OUTER
800 FT OF INNER BREAKWATER
WAVE GAGE LOCATION AND NUMBER
NOTE: CONTOURS AND ELEVATIONS ARE
IN FEET REFERRED TO MEAN
LOWER LOW WATER.
ELEMENTS OF PLAN 60
SCALES IN FEET
PROTOTYPE
MODEL
1000
0
1000
10
0
10



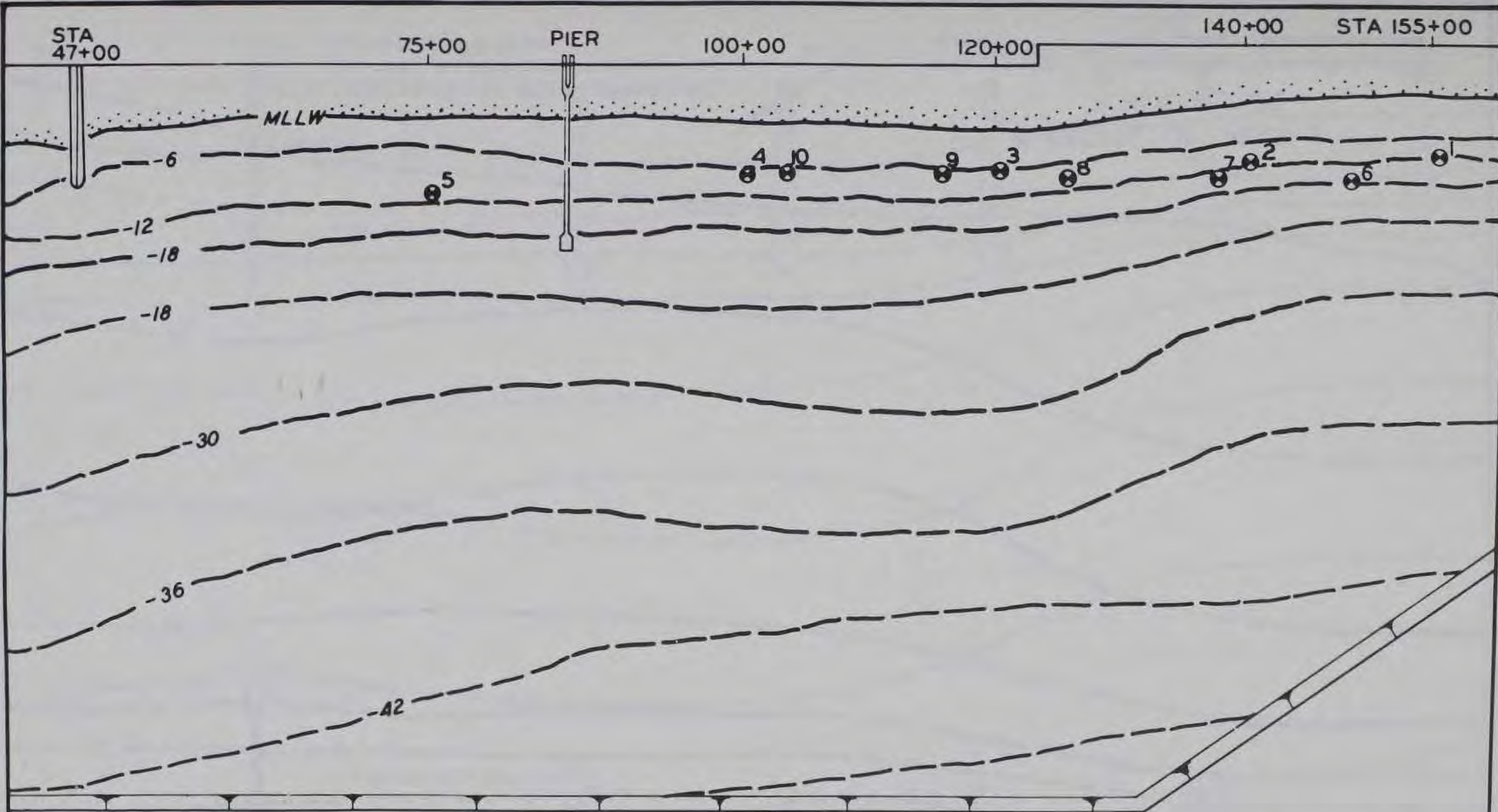
ELEMENTS OF PLAN 61



ELEMENTS OF PLAN 62



ELEMENTS OF PLAN 50 WITH GAGES 1-3
MOVED INSIDE INNER BREAKWATER

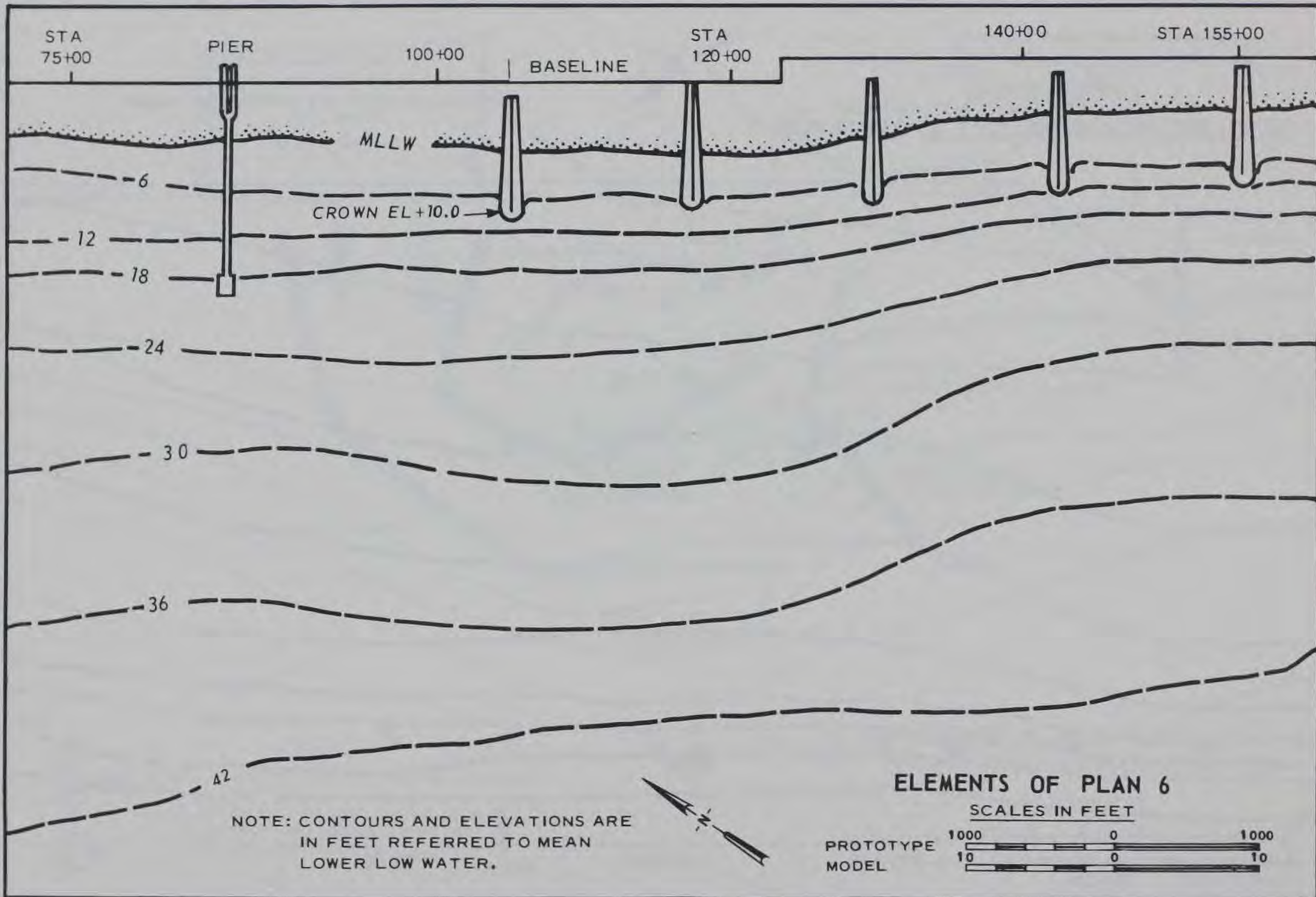


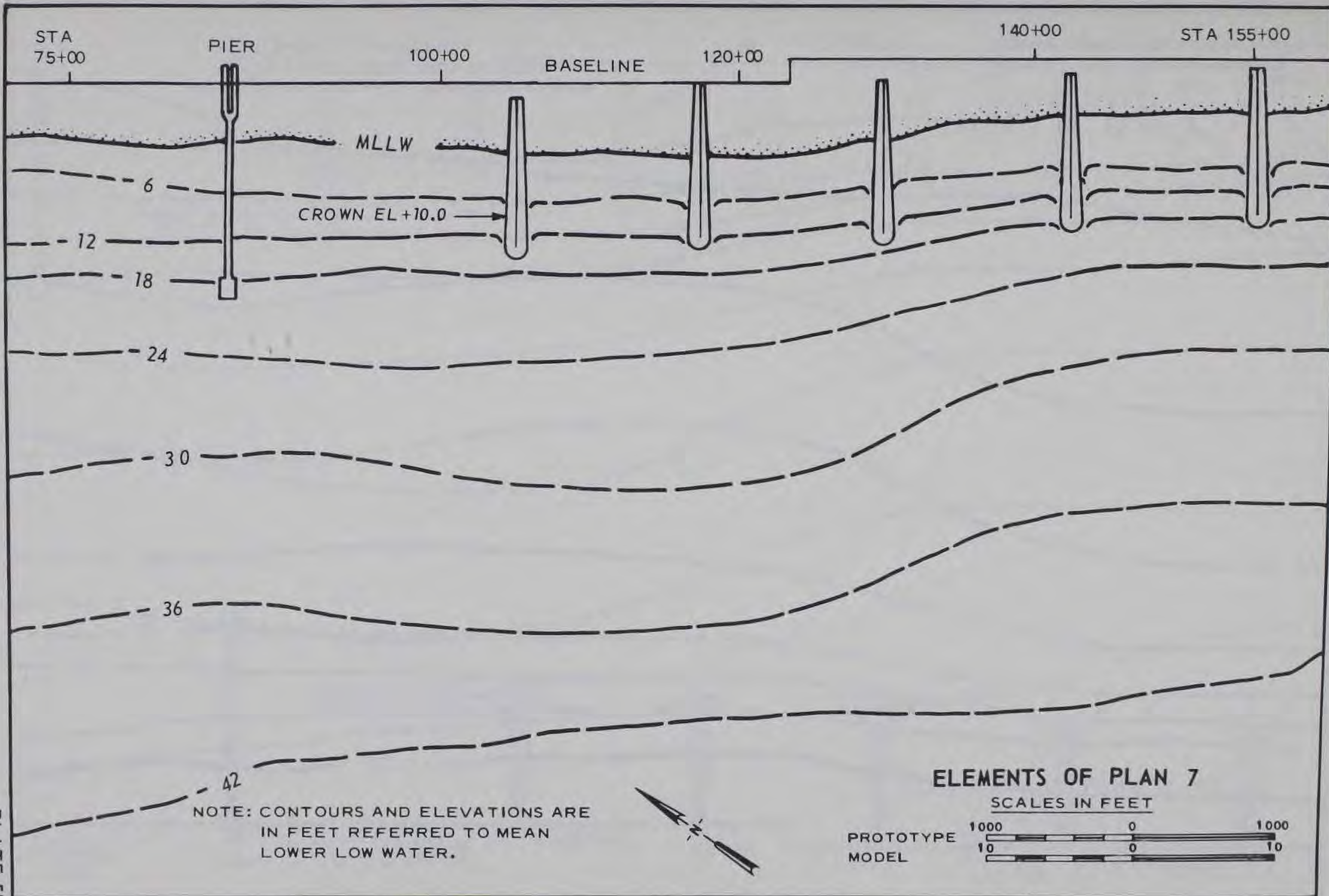
ELEMENTS OF EXISTING BEACH CONDITIONS

SCALES IN FEET

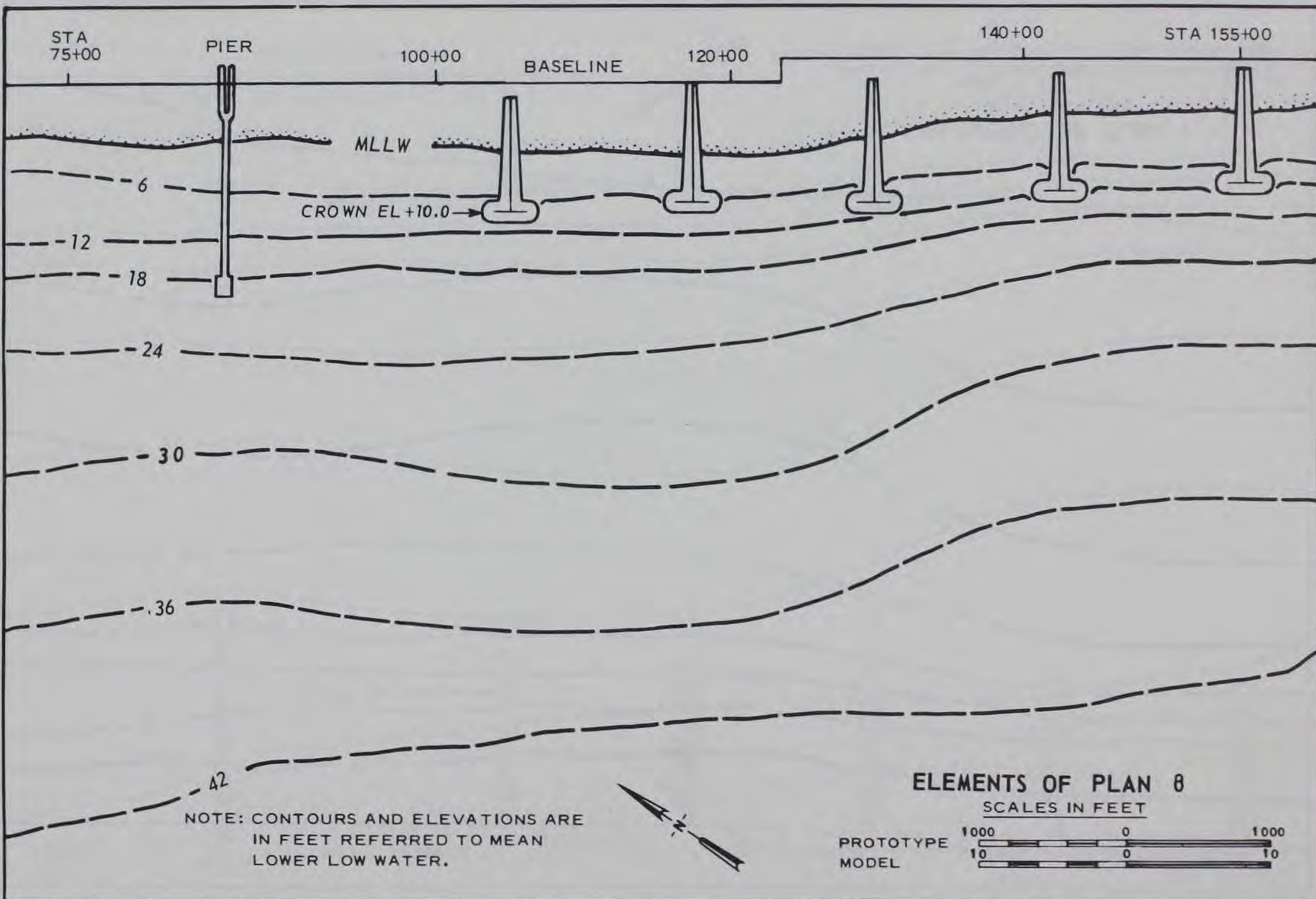
PROTOTYPE 1000 0 1000
MODEL 10 0 10

NOTE: CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER.

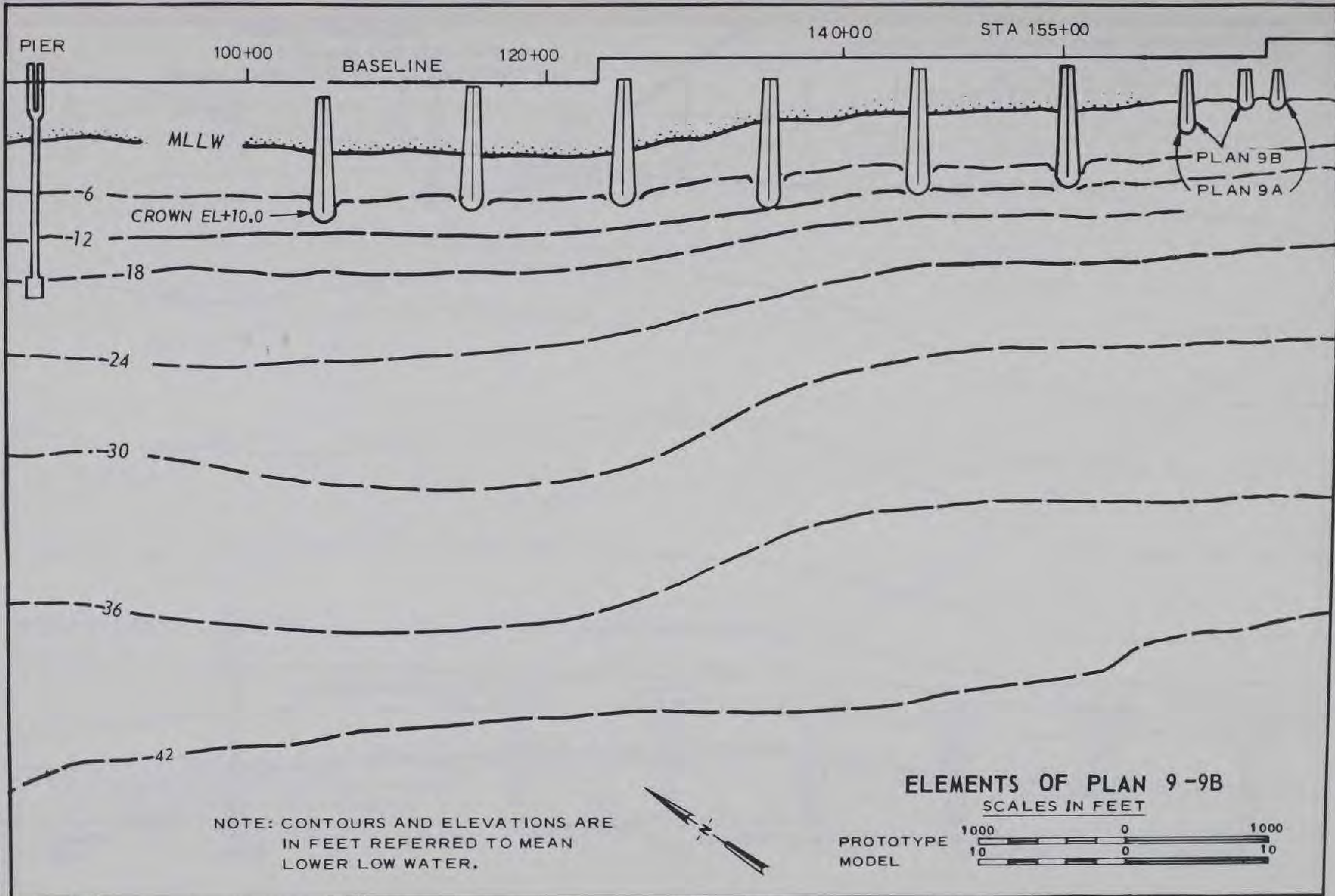

STA 75+00
PIER
100+00
BASELINE
STA 120+00
140+00
STA 155+00
MLLW
CROWN EL +10.0
6
12
18
24
30
36
42
N
ELEMENTS OF PLAN 6
SCALES IN FEET
PROTOTYPE
1000
0
1000
MODEL
10
0
10
NOTE: CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER.



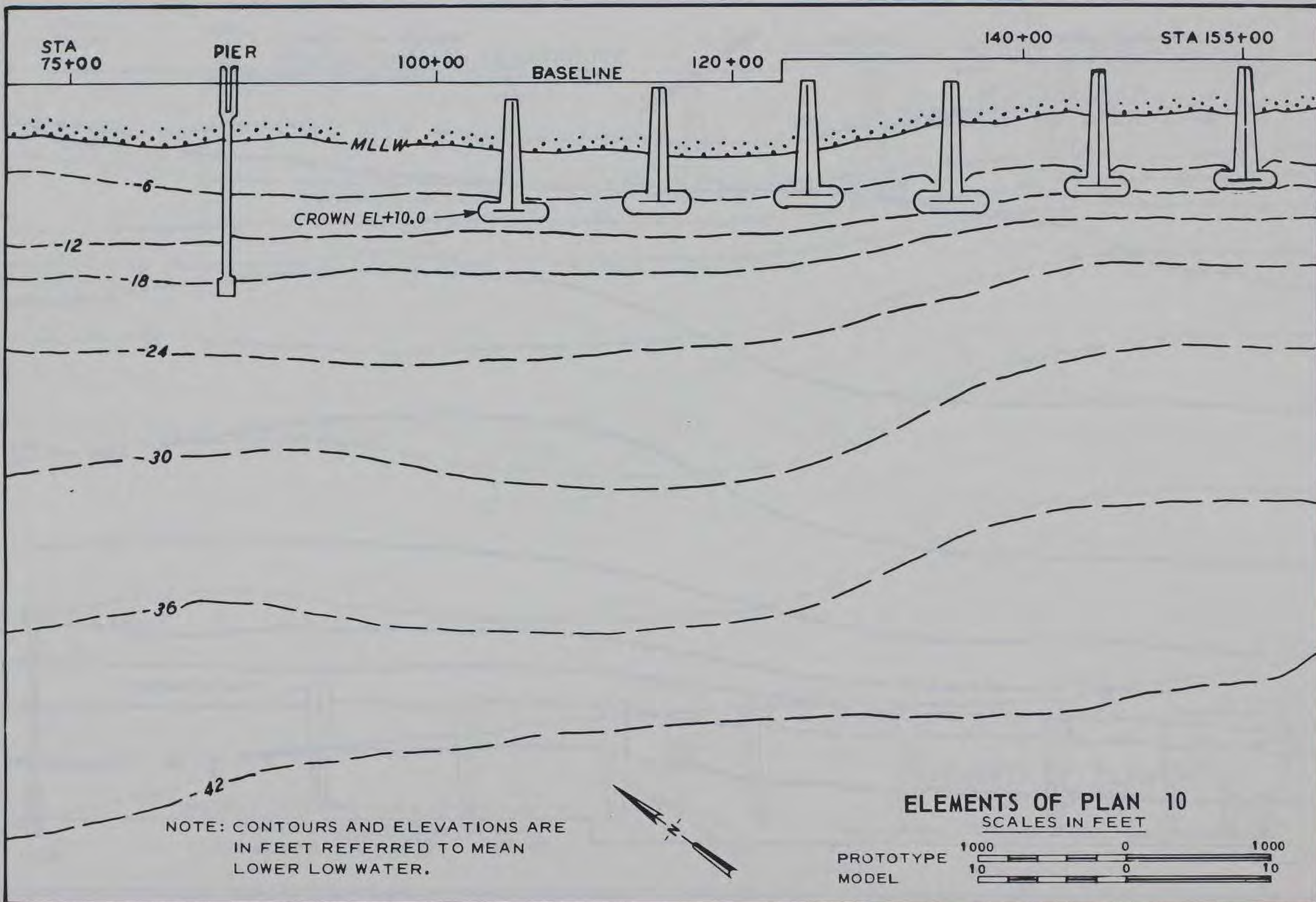
ELEMENTS OF PLAN 7

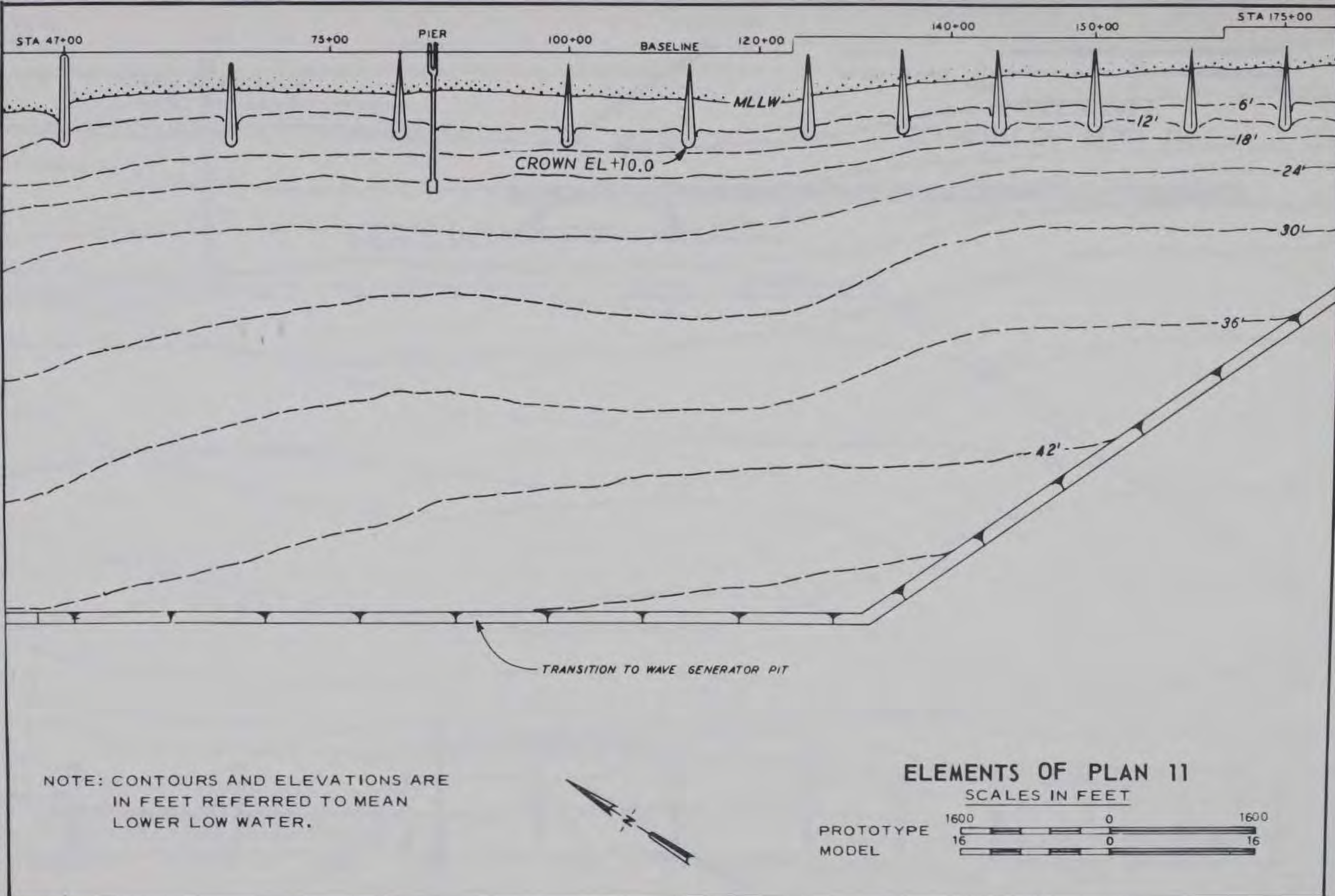


ELEMENTS OF PLAN 8



ELEMENTS OF PLAN 9-9B

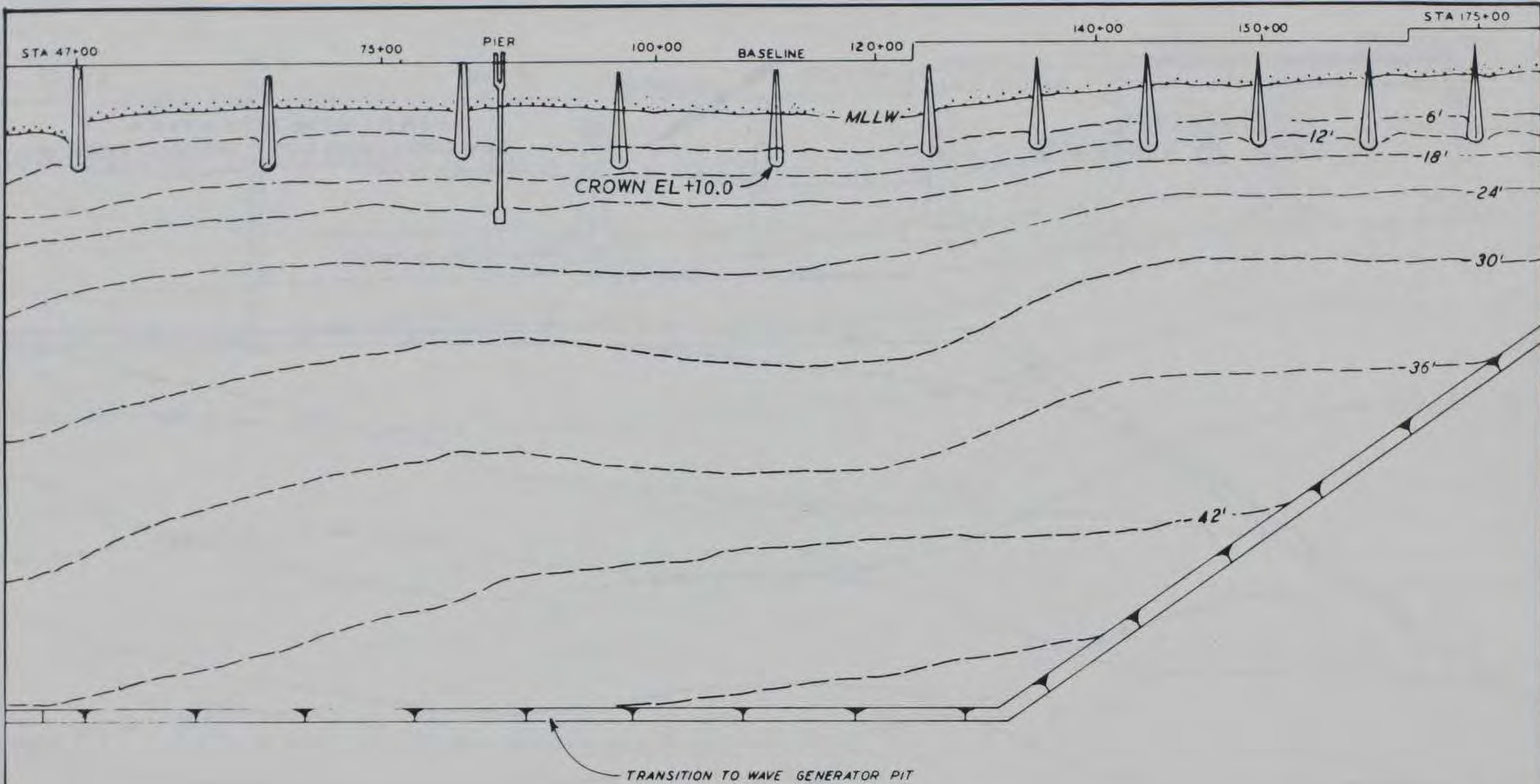

STA 75+00
PIER
100+00
BASELINE
120+00
140+00
STA 155+00
MLLW
-6
CROWN EL+10.0
-12
-18
-24
-30
-36
-42
NOTE: CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER.
N
ELEMENTS OF PLAN 10
SCALES IN FEET
PROTOTYPE
MODEL
1000
0
1000
10
0
10



NOTE: CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER.

ELEMENTS OF PLAN 11

PLATE 59



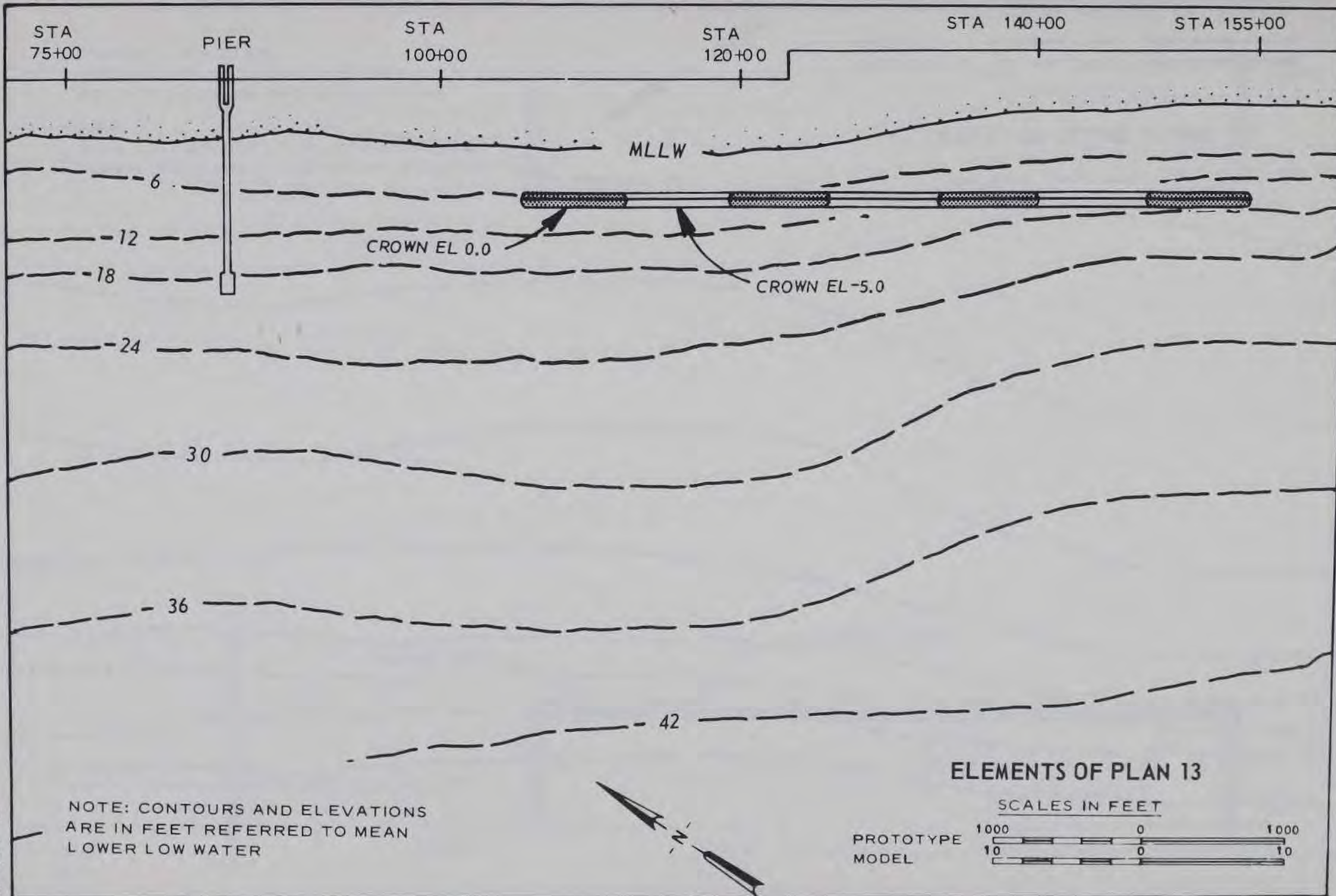
NOTE: CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER.

ELEMENTS OF PLAN 12

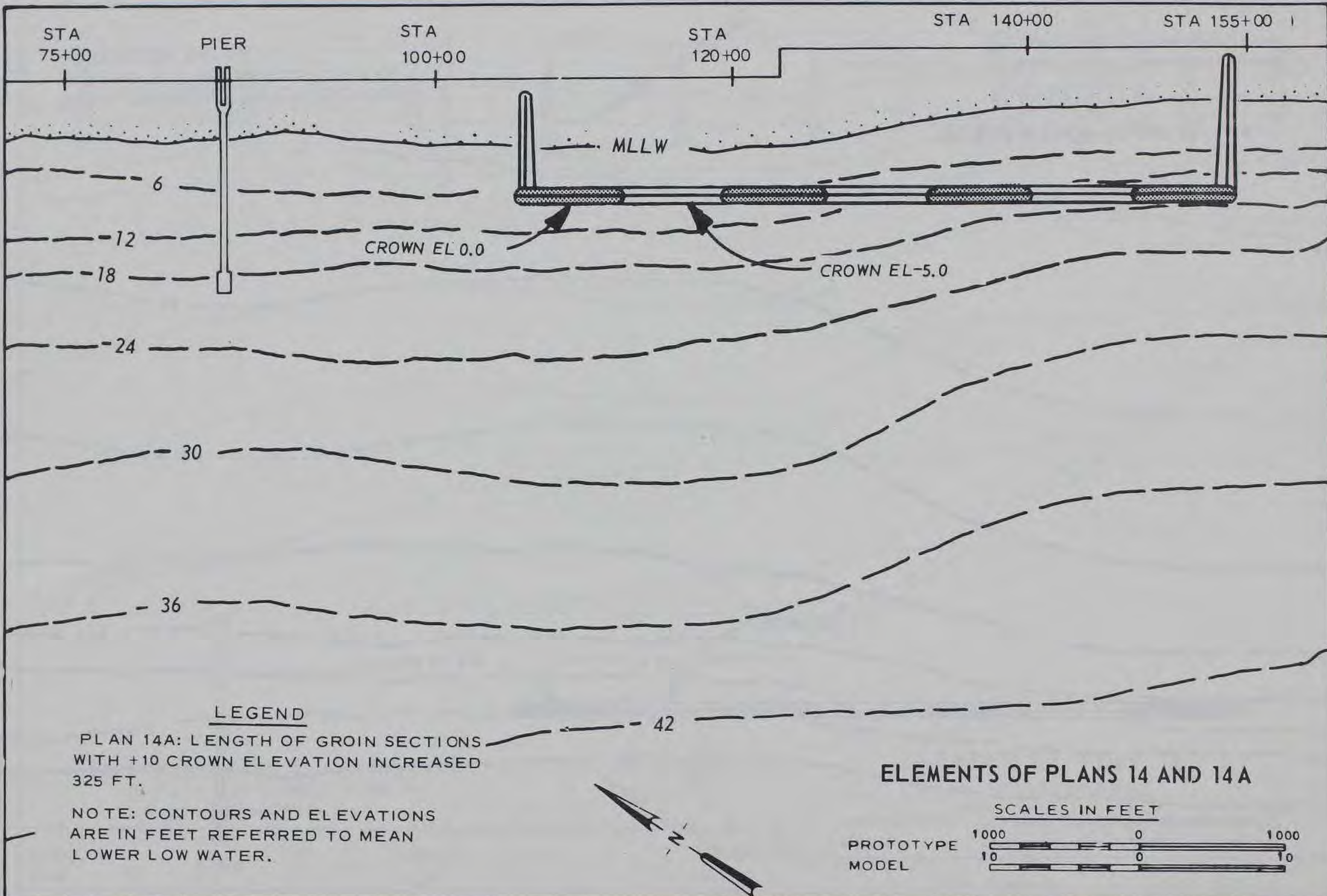
SCALES IN FEET

PROTOTYPE 1600 — 0 — 1600

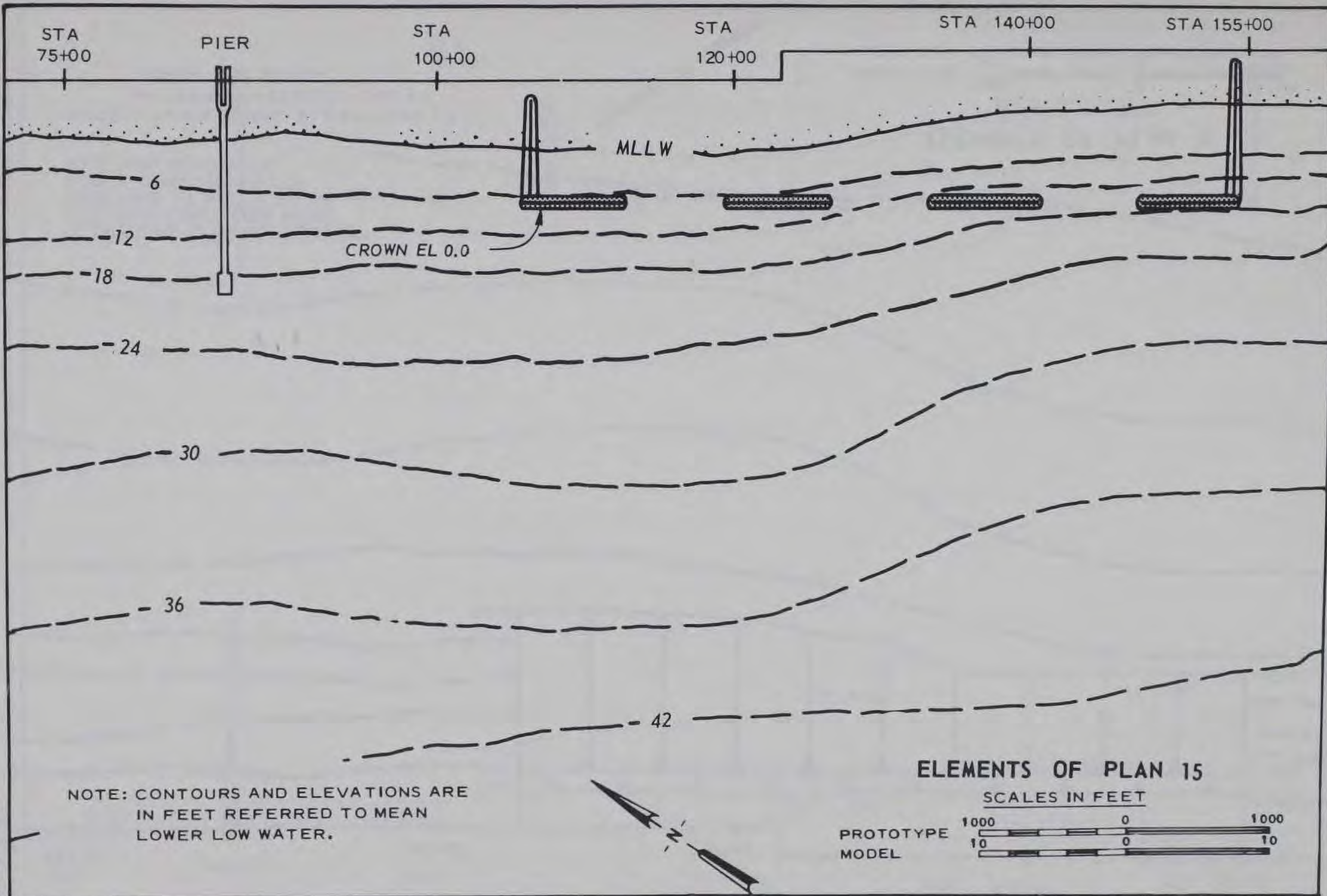
MODEL 16 — 0 — 16

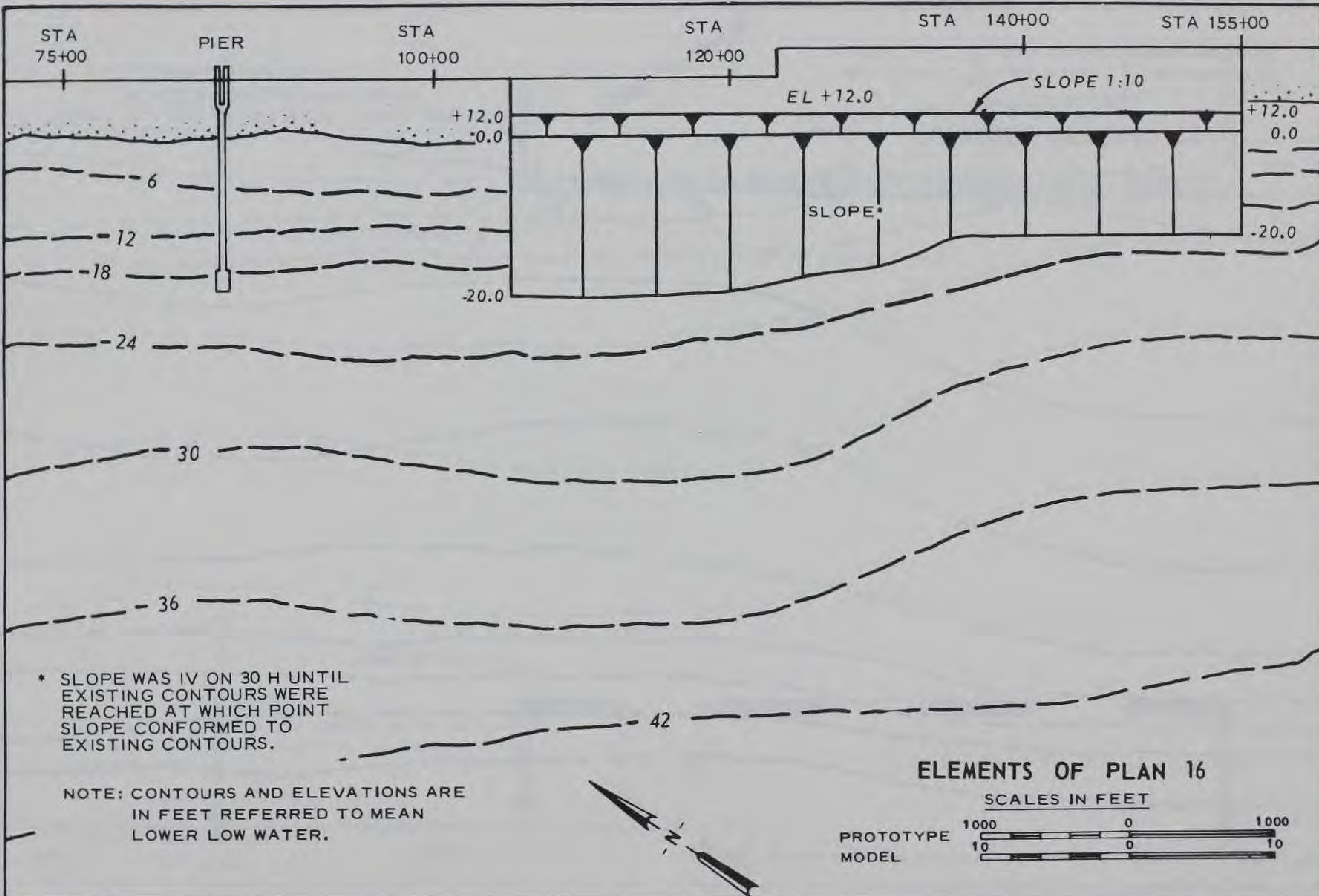


ELEMENTS OF PLAN 13



ELEMENTS OF PLANS 14 AND 14A

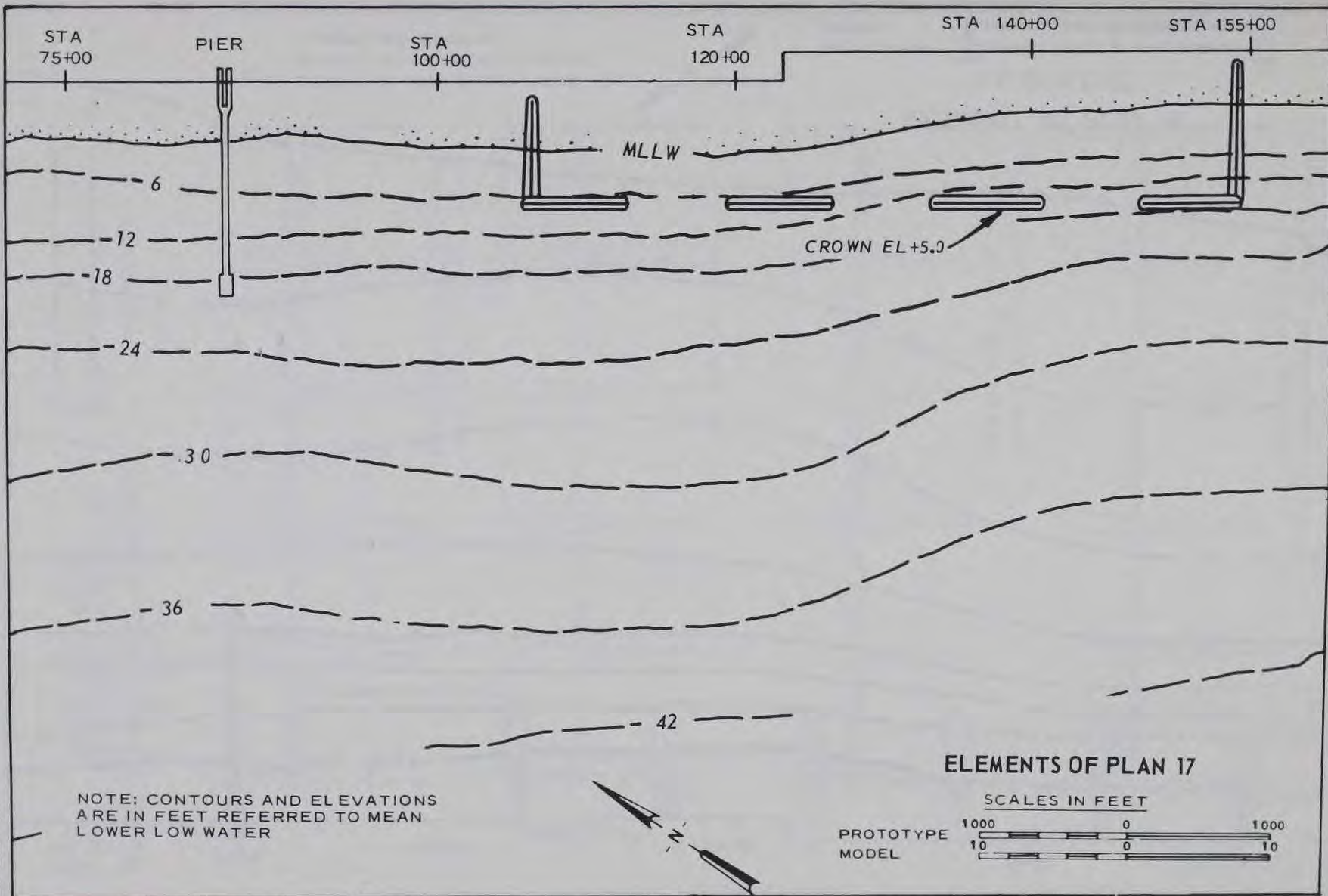

STA 75+00
PIER
STA 100+00
STA 120+00
STA 140+00
STA 155+00
MLLW
6
12
18
24
30
36
42
CROWN EL 0.0
N
NOTE: CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER.
ELEMENTS OF PLAN 15
SCALES IN FEET
PROTOTYPE 1000 0 1000
MODEL 10 0 10



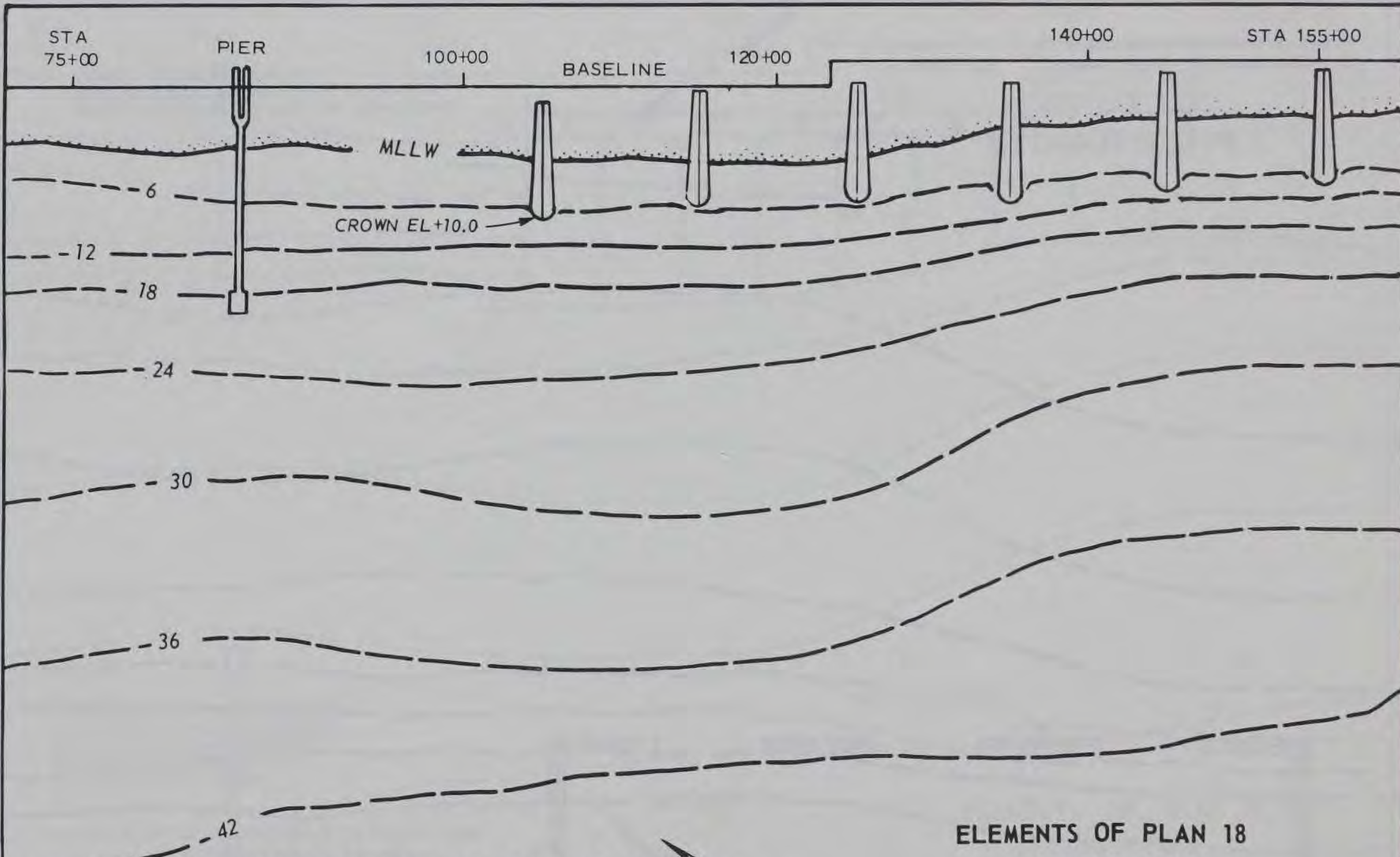
* SLOPE WAS IV ON 30 H UNTIL EXISTING CONTOURS WERE REACHED AT WHICH POINT SLOPE CONFORMED TO EXISTING CONTOURS.

NOTE: CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER.

ELEMENTS OF PLAN 16



ELEMENTS OF PLAN 17



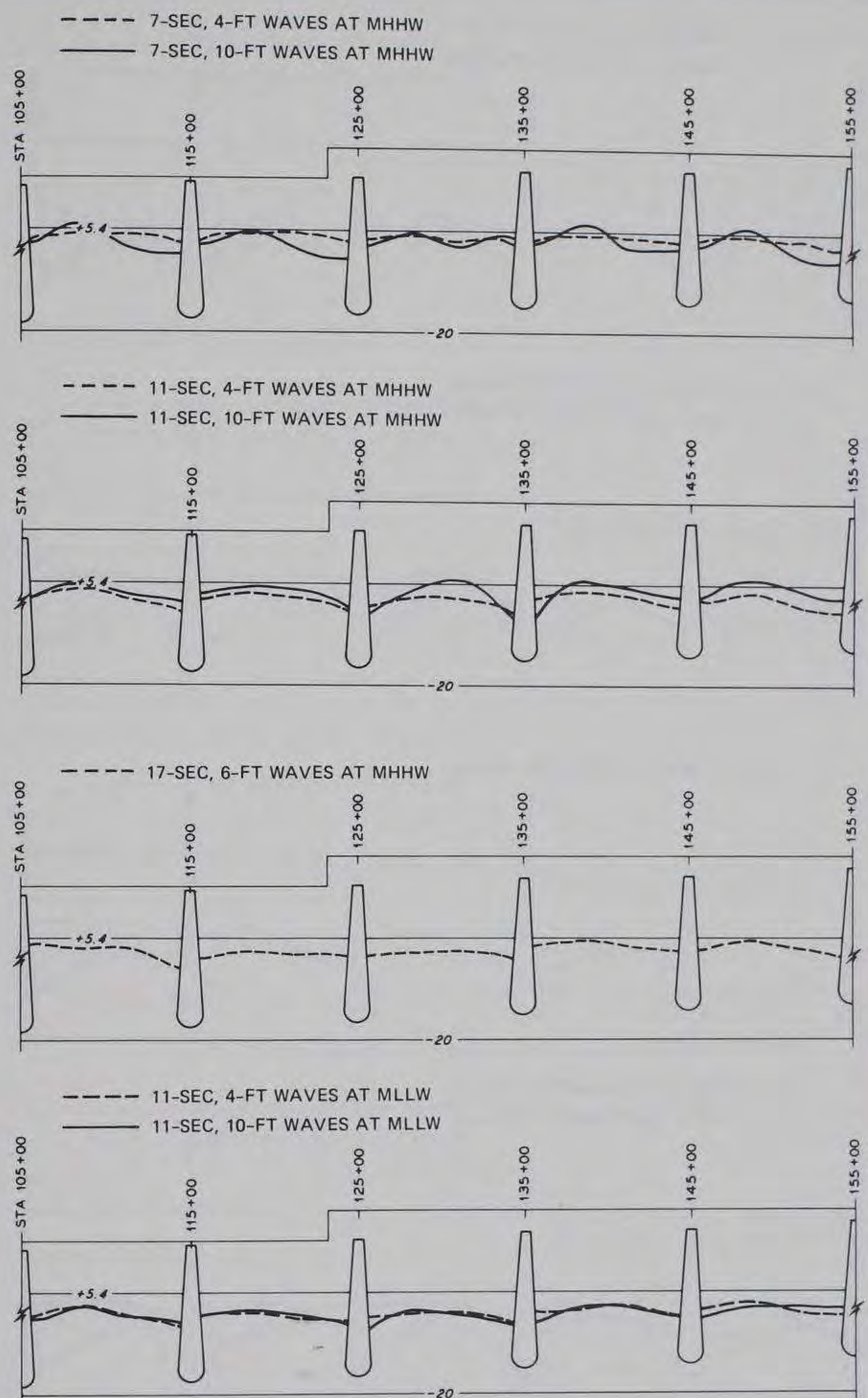
NOTE: CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER.

ELEMENTS OF PLAN 18

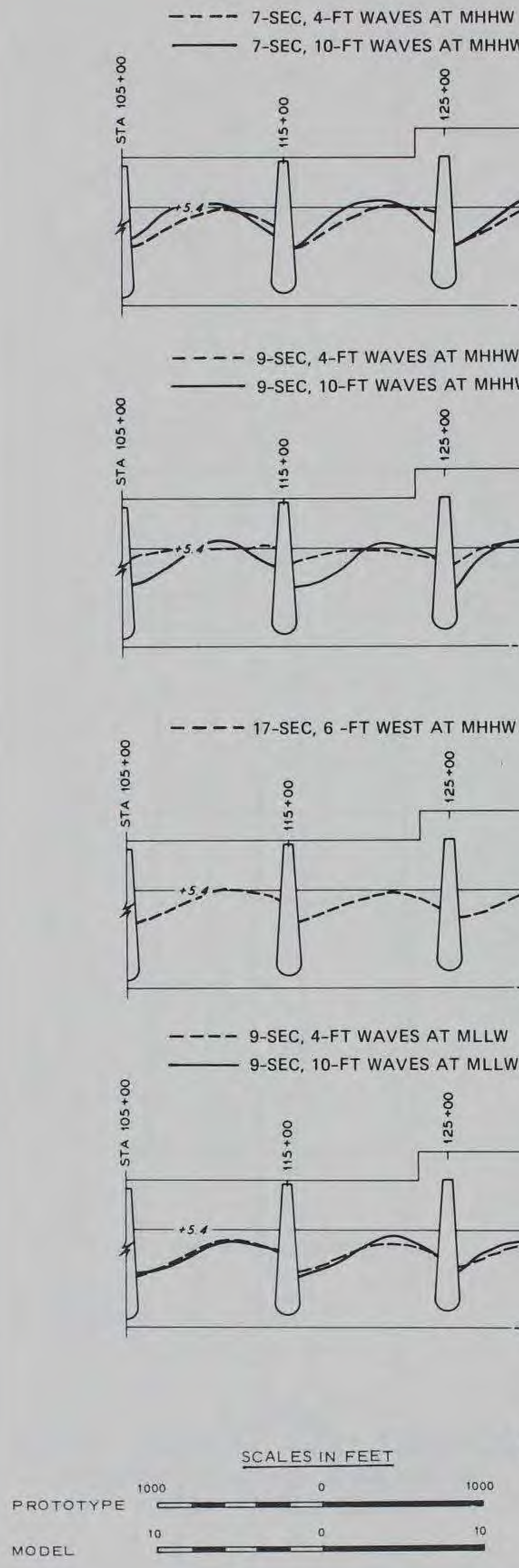
SCALES IN FEET

PROTOTYPE 1000 0 1000

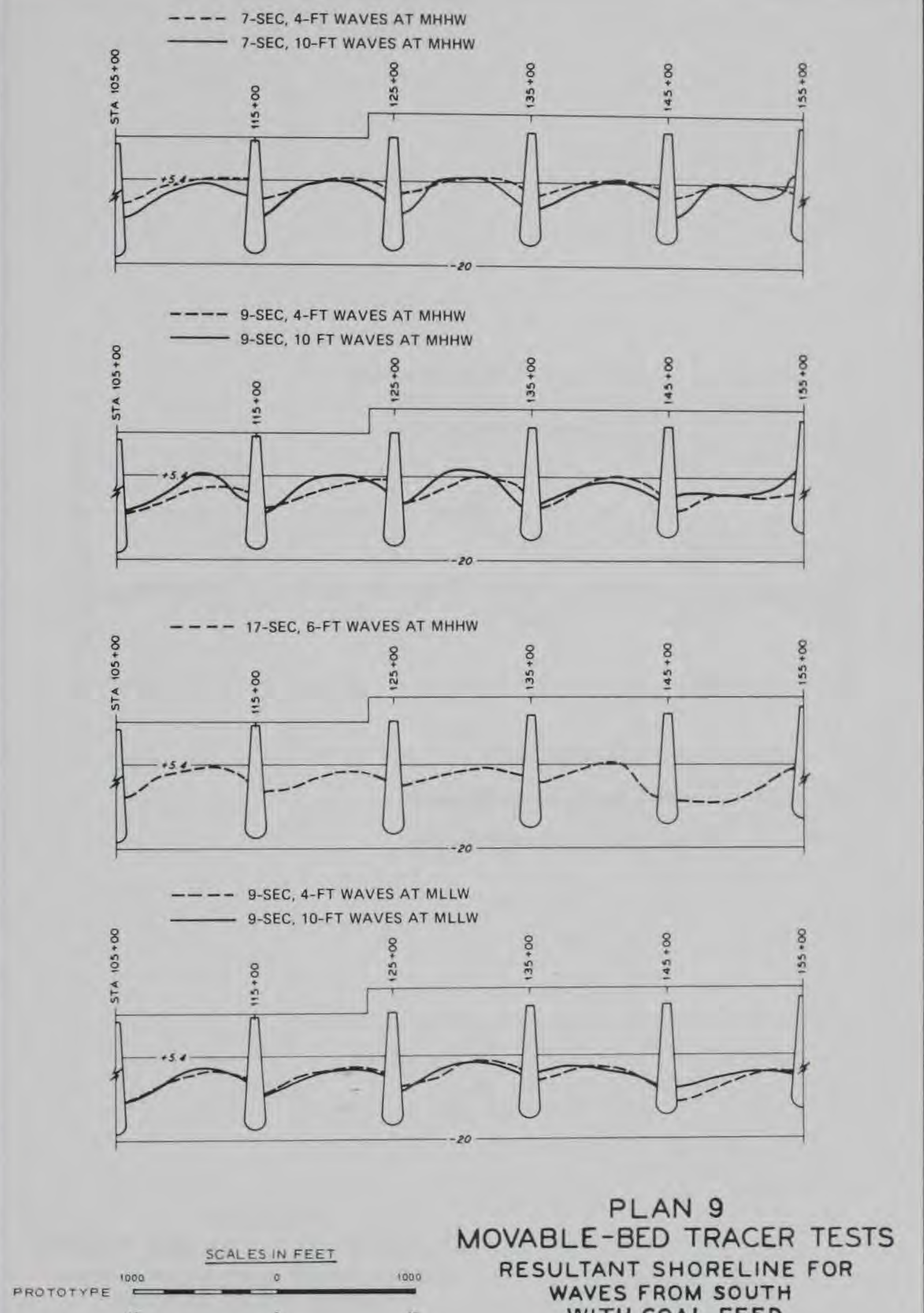
MODEL 10 0 10



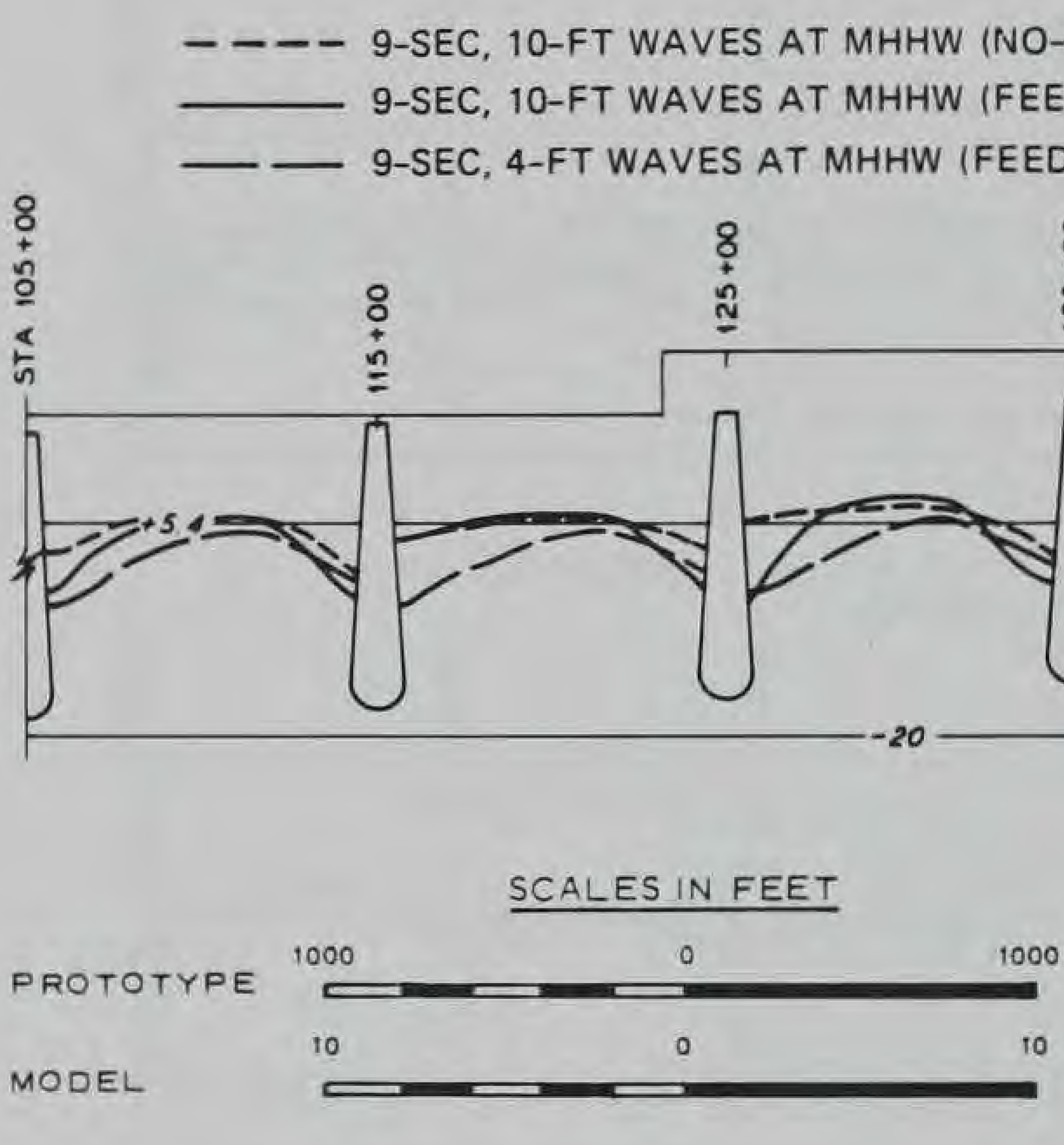
PLAN 9
MOVABLE-BED TRACER TESTS
RESULTANT SHORELINE FOR WAVES FROM WEST



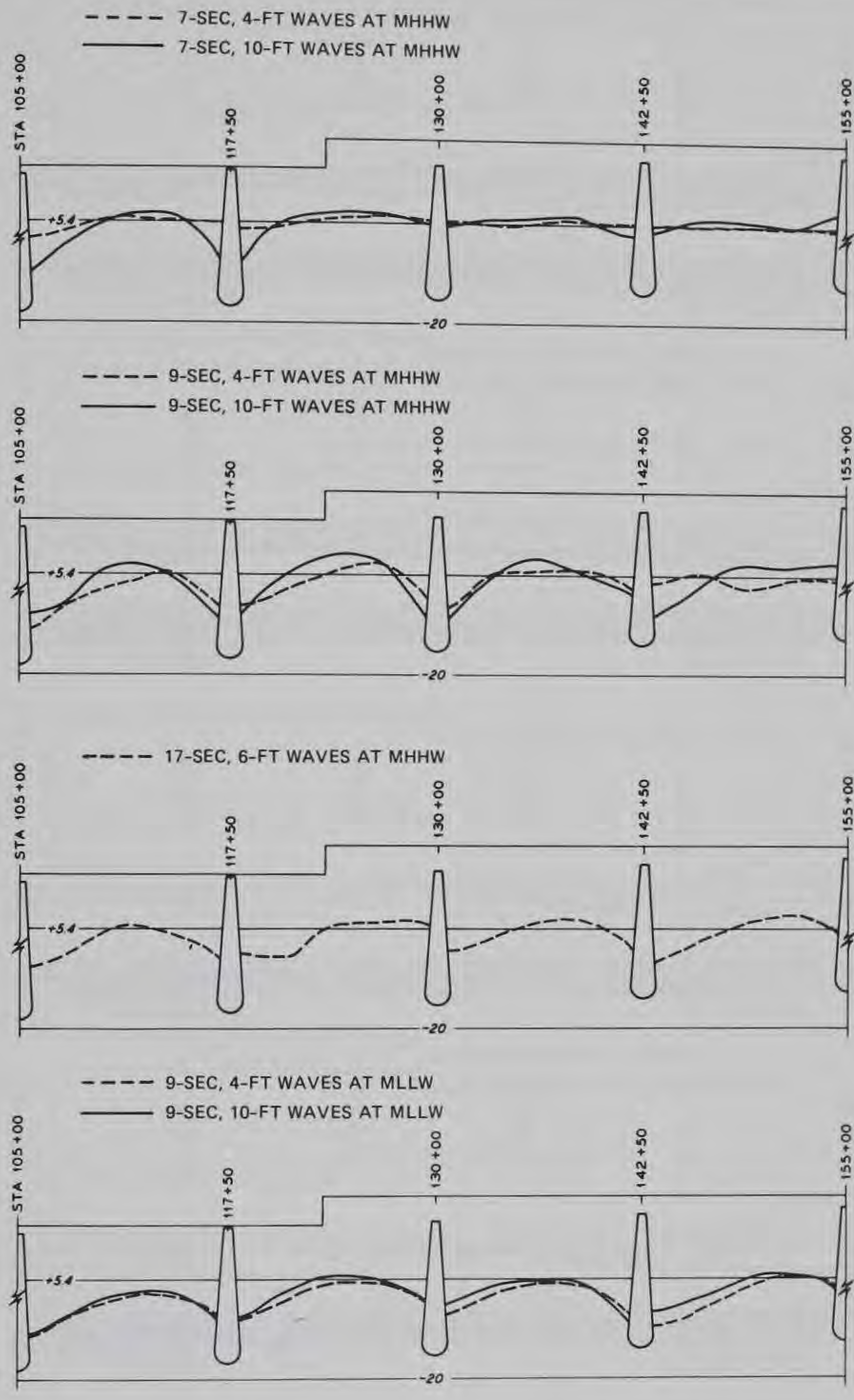
PLAN 9
MOVABLE-BED TRACER TESTS
RESULTANT SHORELINE FOR WAVES FROM SOUTH WITHOUT COAL FEED

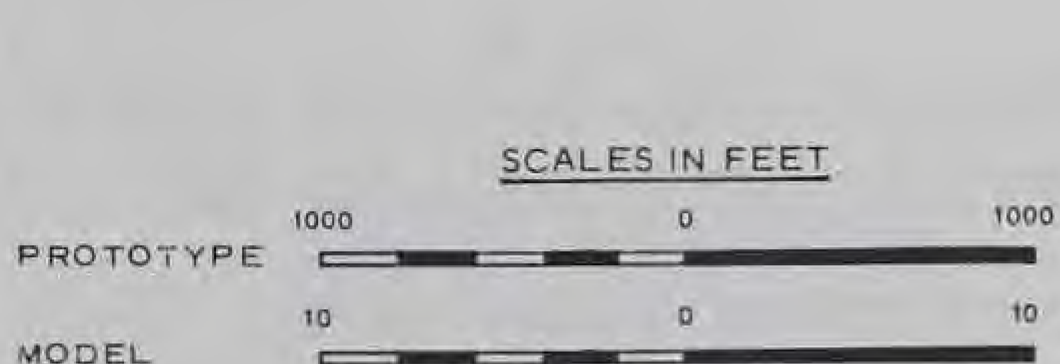


PLAN 9
MOVABLE-BED TRACER TESTS
RESULTANT SHORELINE FOR WAVES FROM SOUTH WITH COAL FEED

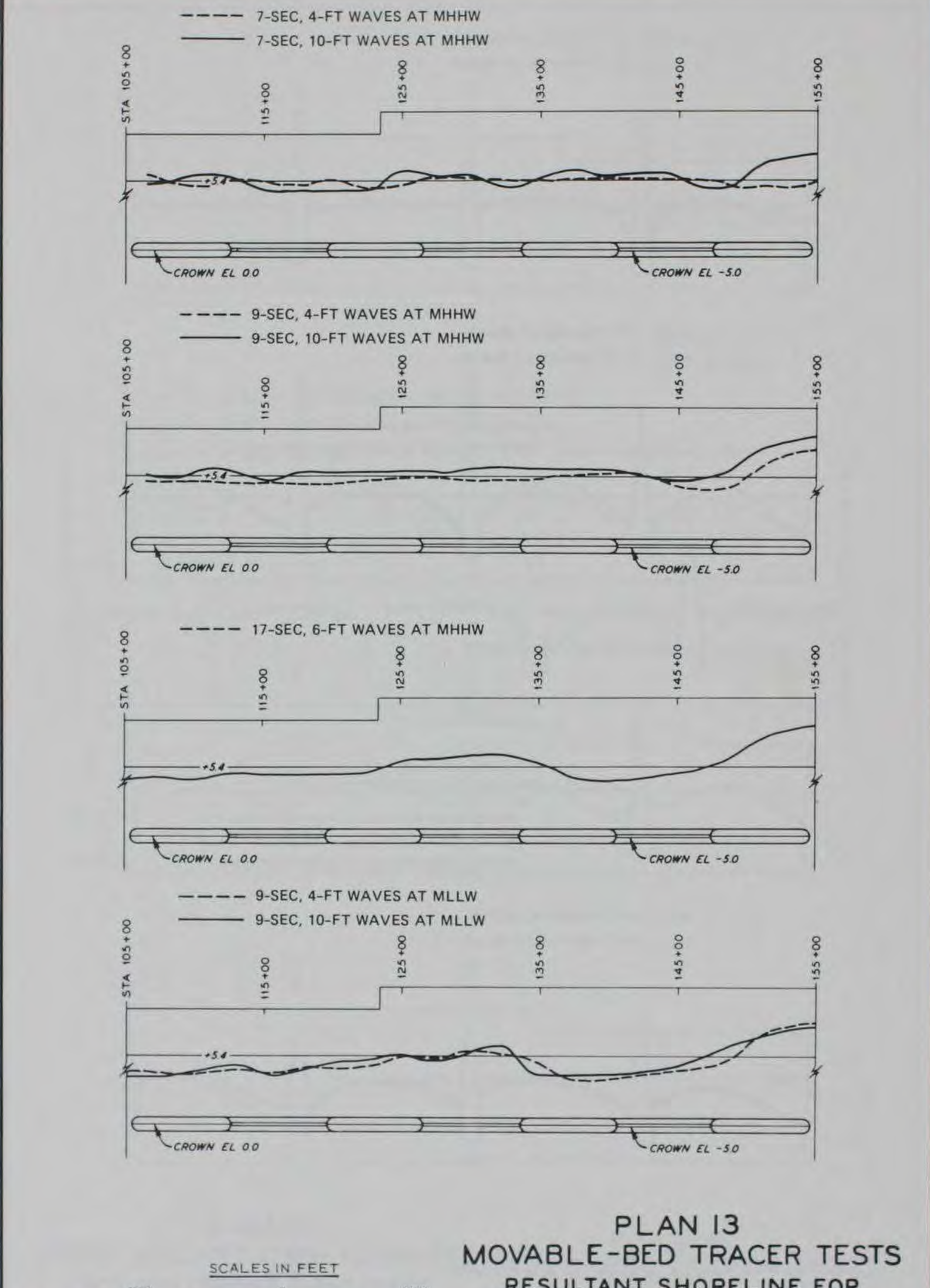


PLAN 9
MOVABLE-BED TRACER TESTS
RESULTANT SHORELINE FOR WAVES FROM SOUTH
FEED AND NO-FEED

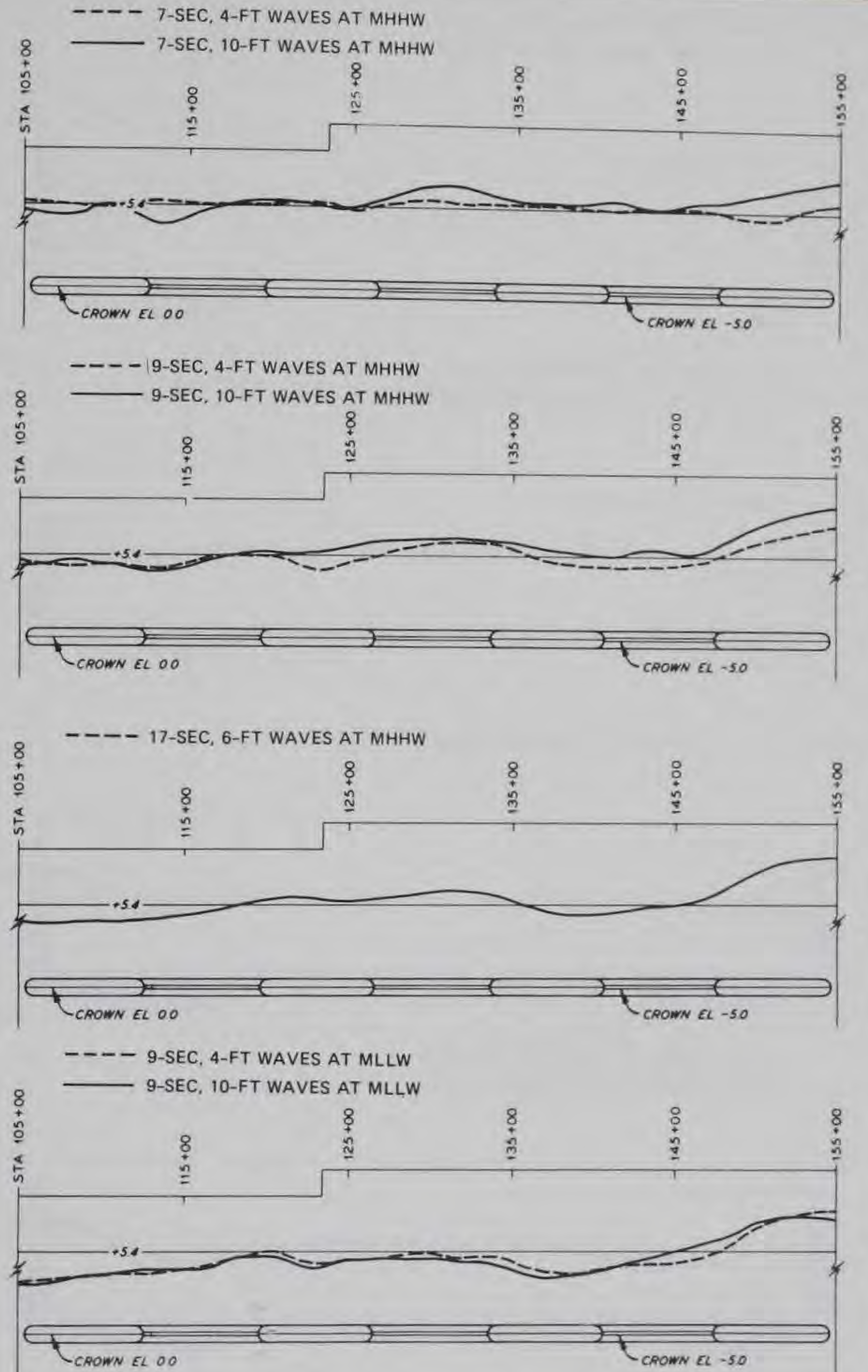


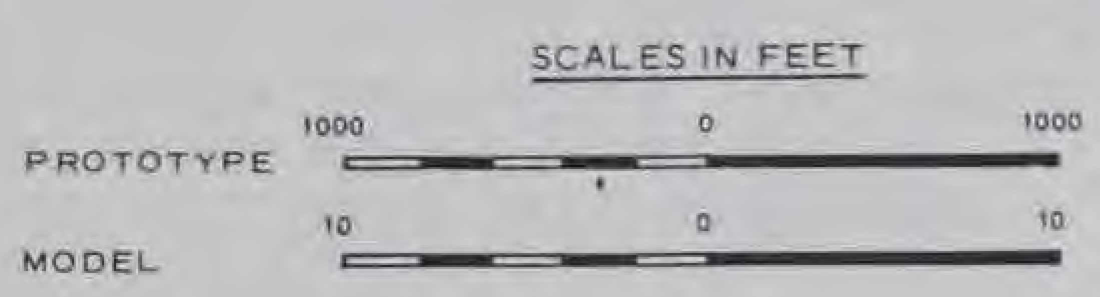


PLAN 6
MOVABLE-BED TRACER TESTS
RESULTANT SHORELINE FOR WAVES FROM SOUTH

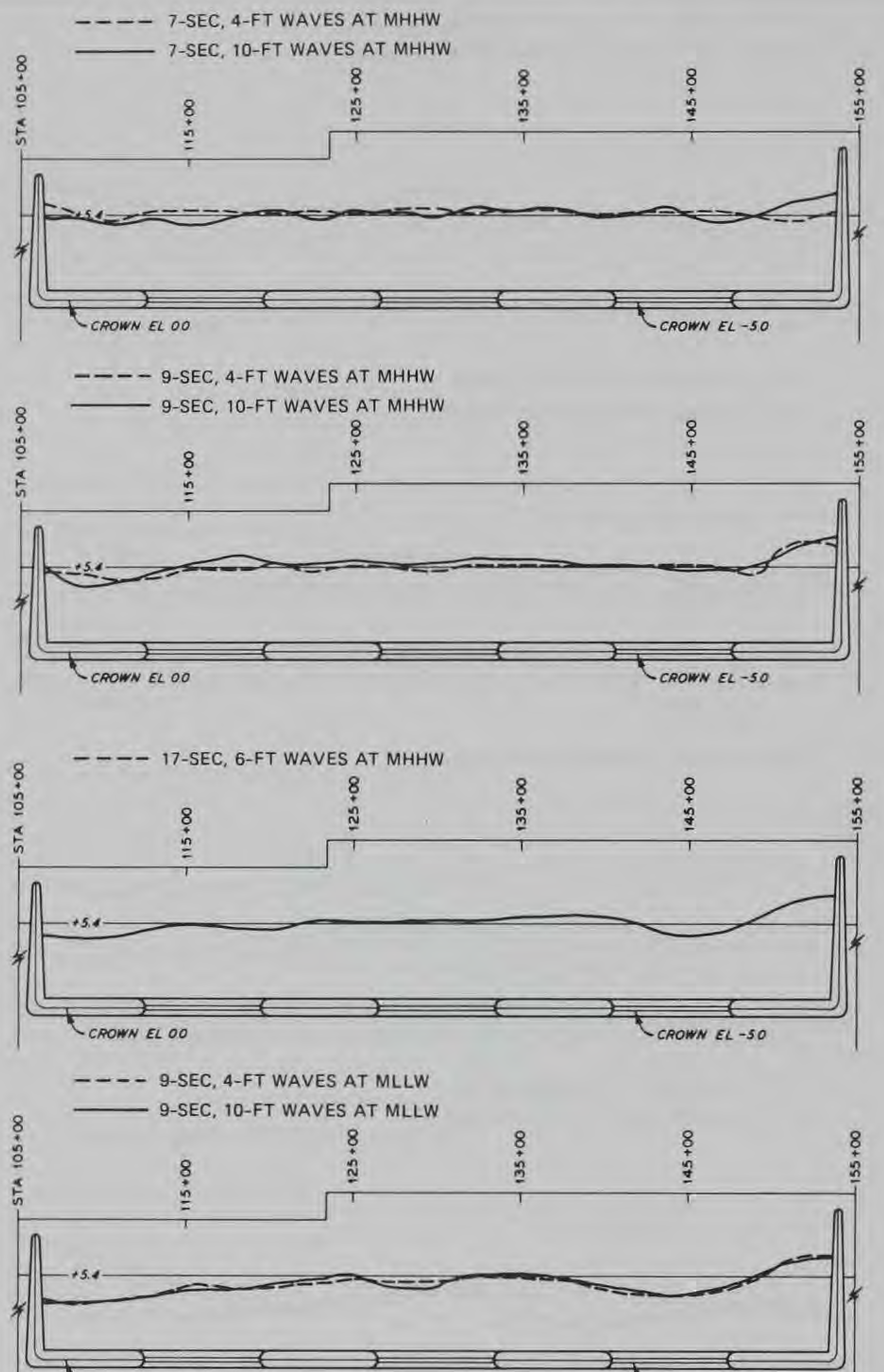


PLAN 13
MOVABLE-BED TRACER TESTS
RESULTANT SHORELINE FOR WAVES FROM SOUTH WITHOUT COAL FEED

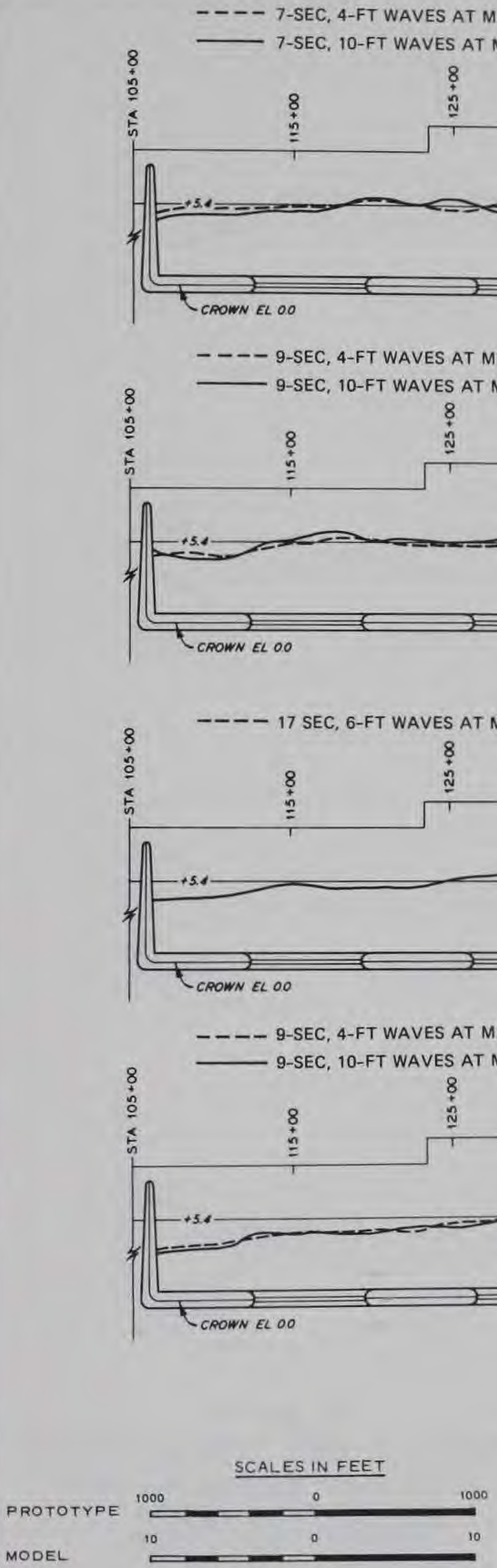




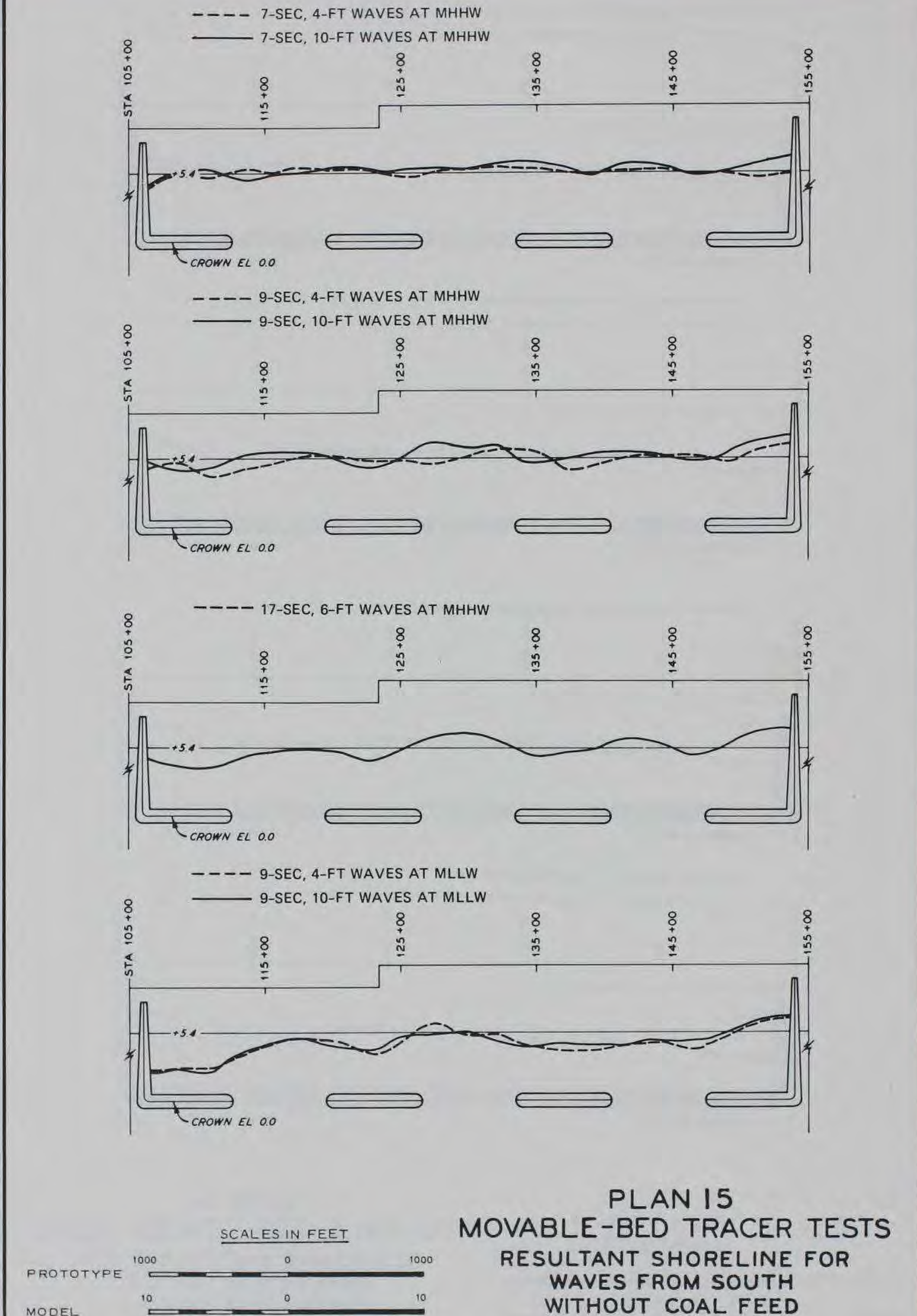
PLAN 13
MOVABLE-BED TRACER TESTS
RESULTANT SHORELINE FOR WAVES FROM SOUTH WITH COAL FEED



PLAN 14
MOVABLE-BED TRACER TESTS
RESULTANT SHORELINE FOR WAVES FROM SOUTH WITHOUT COAL FEED



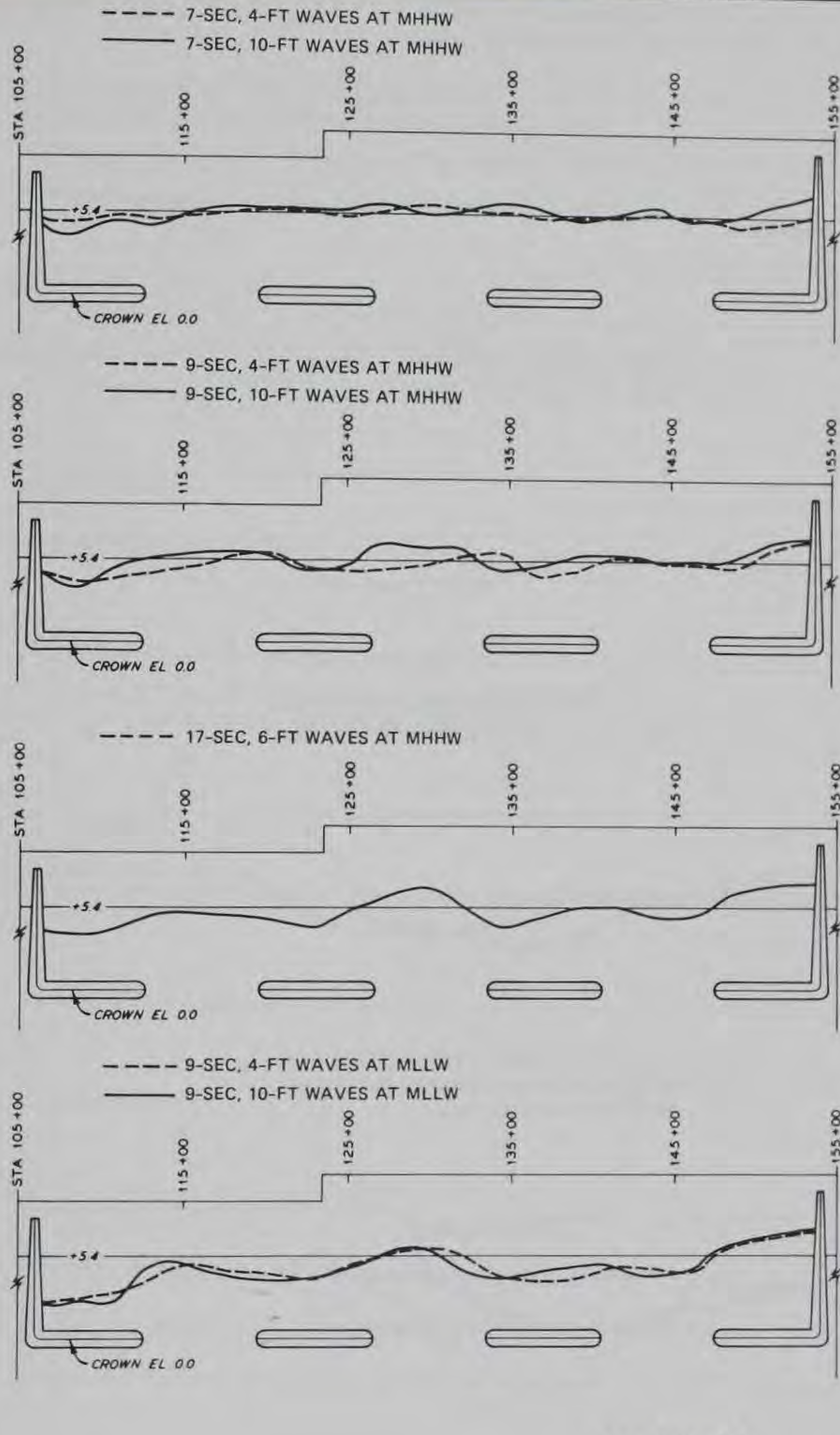
PLAN 14
MOVABLE-BED TRACER TESTS
RESULTANT SHORELINE FOR WAVES FROM SOUTH WITH COAL FEED



PLAN 15

MOVABLE-BED TRACER TESTS

RESULTANT SHORELINE FOR WAVES FROM SOUTH WITHOUT COAL FEED

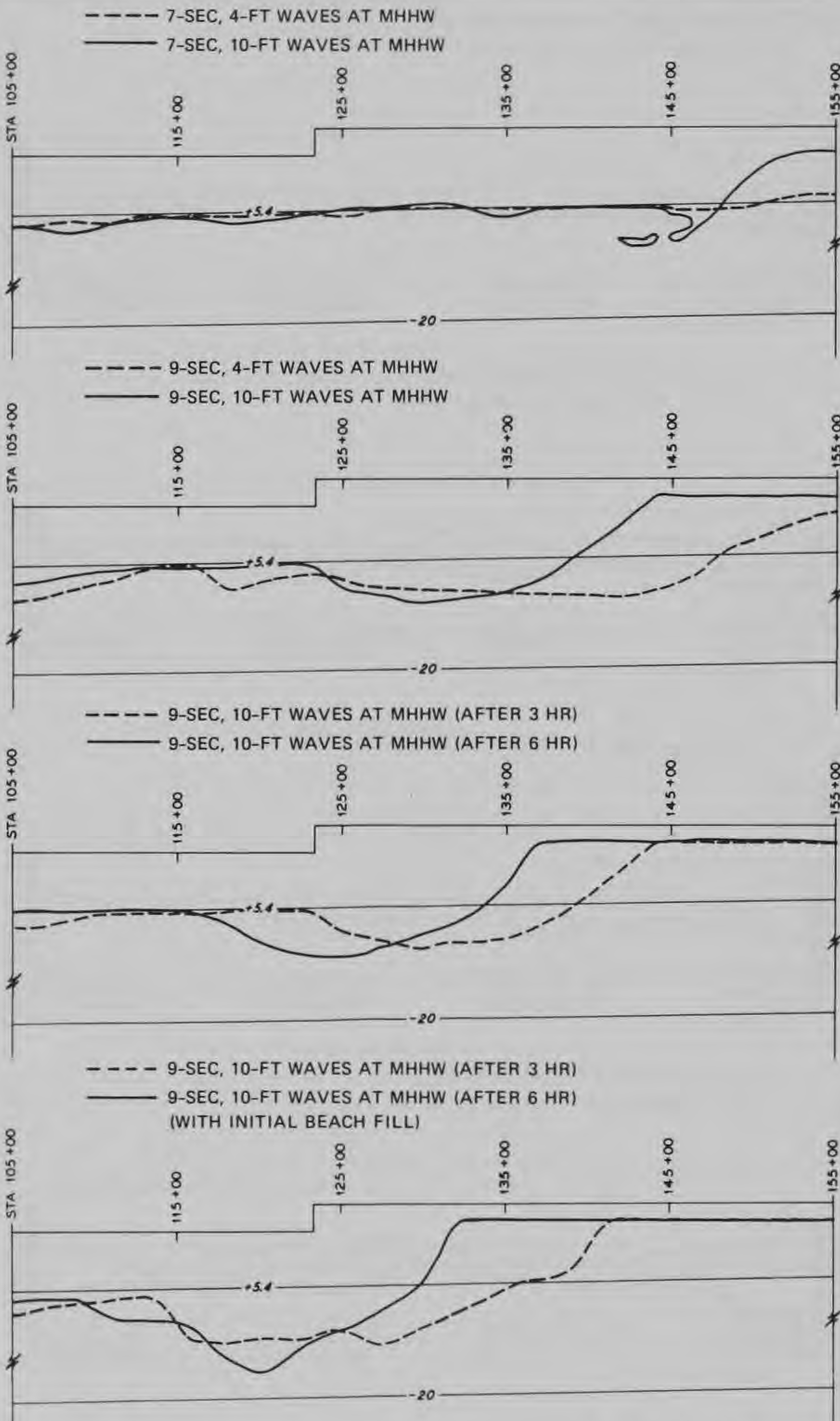


SCALES IN FEET

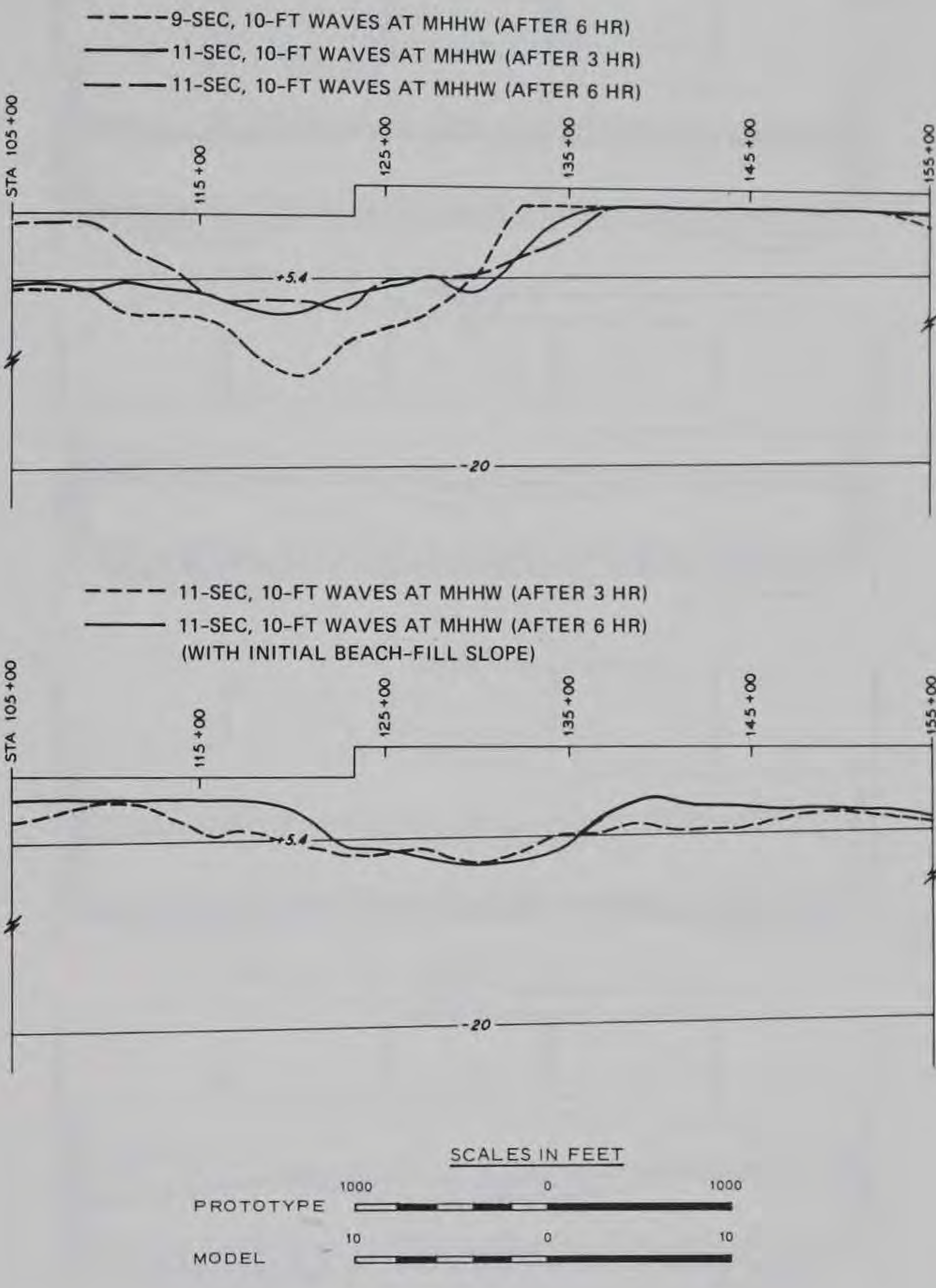
PROTOTYPE 1000 0 1000

MODEL 10 0 10

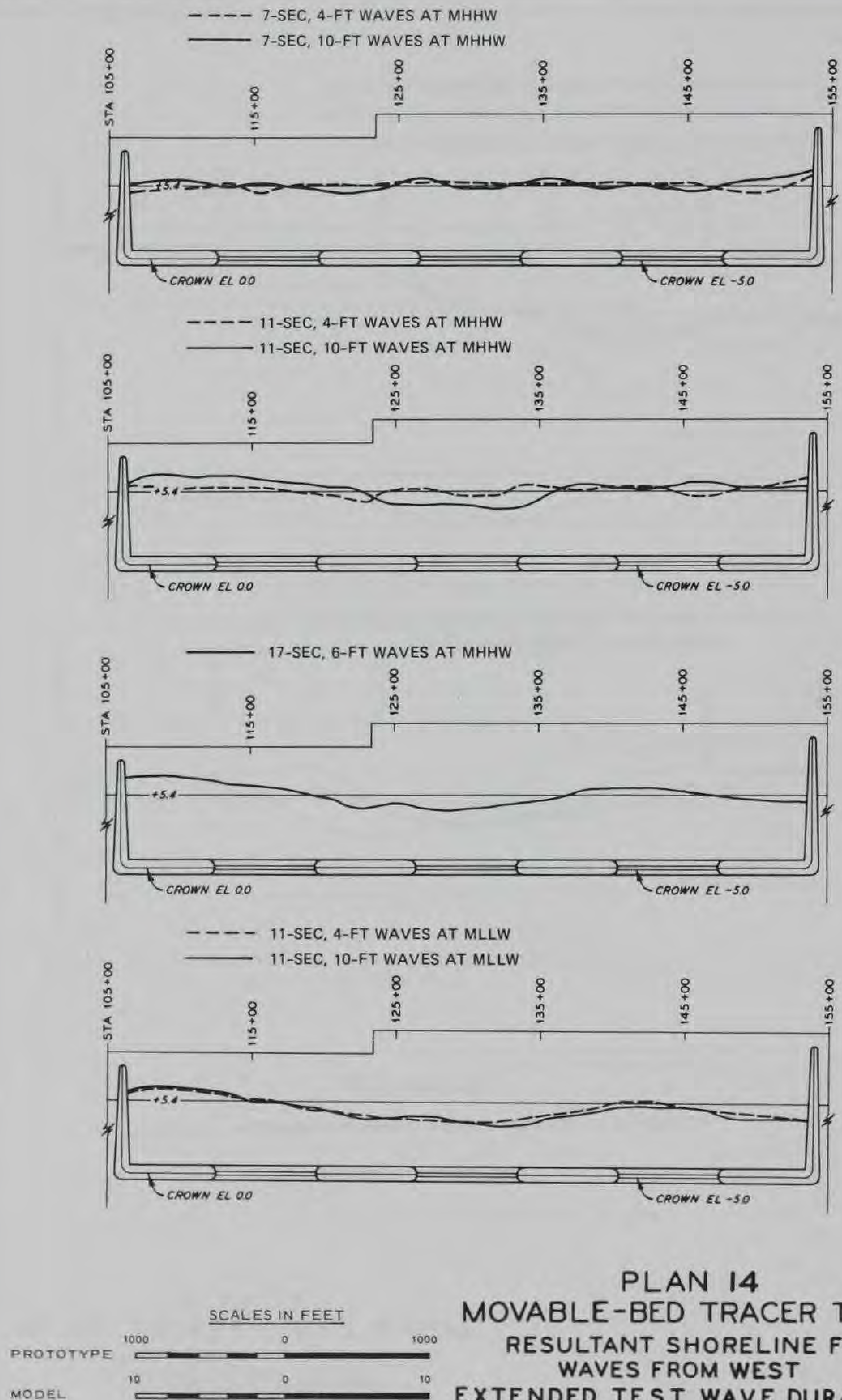
PLAN 15
MOVABLE-BED TRACER TESTS
RESULTANT SHORELINE FOR WAVES FROM SOUTH WITH COAL FEED



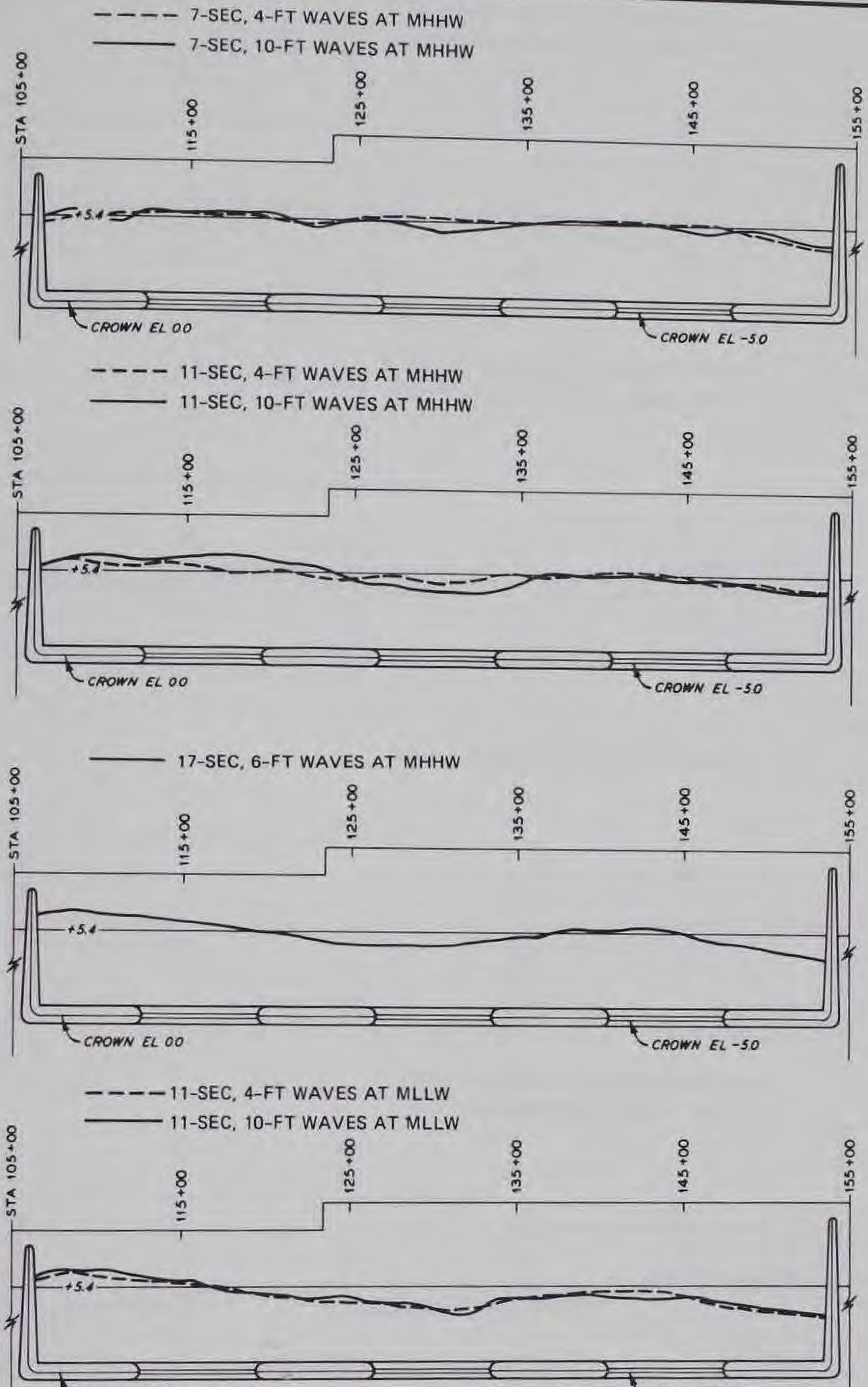
PLAN 16
MOVABLE-BED TRACER TESTS
RESULTANT SHORELINE FOR WAVES FROM SOUTH



PLAN 16
MOVABLE-BED TRACER TESTS
RESULTANT SHORELINE FOR
WAVES FROM SOUTH



PLAN 14
MOVABLE-BED TRACER TESTS
RESULTANT SHORELINE FOR WAVES FROM WEST EXTENDED TEST WAVE DURATION

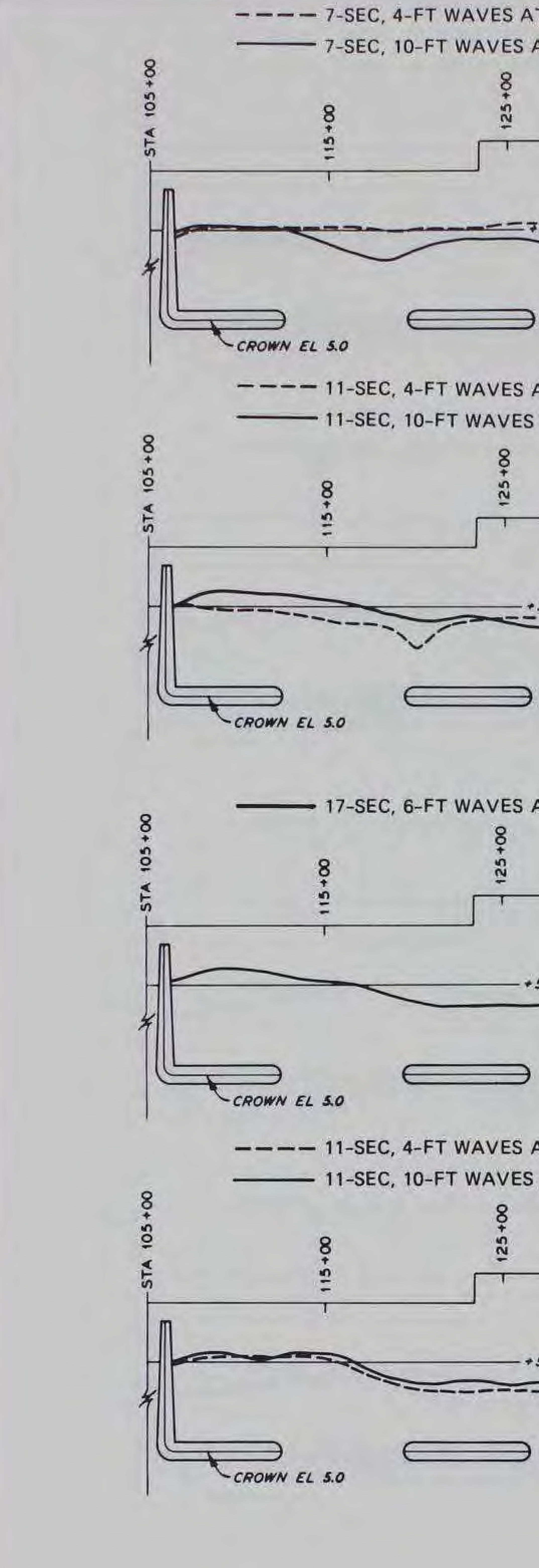


SCALES IN FEET

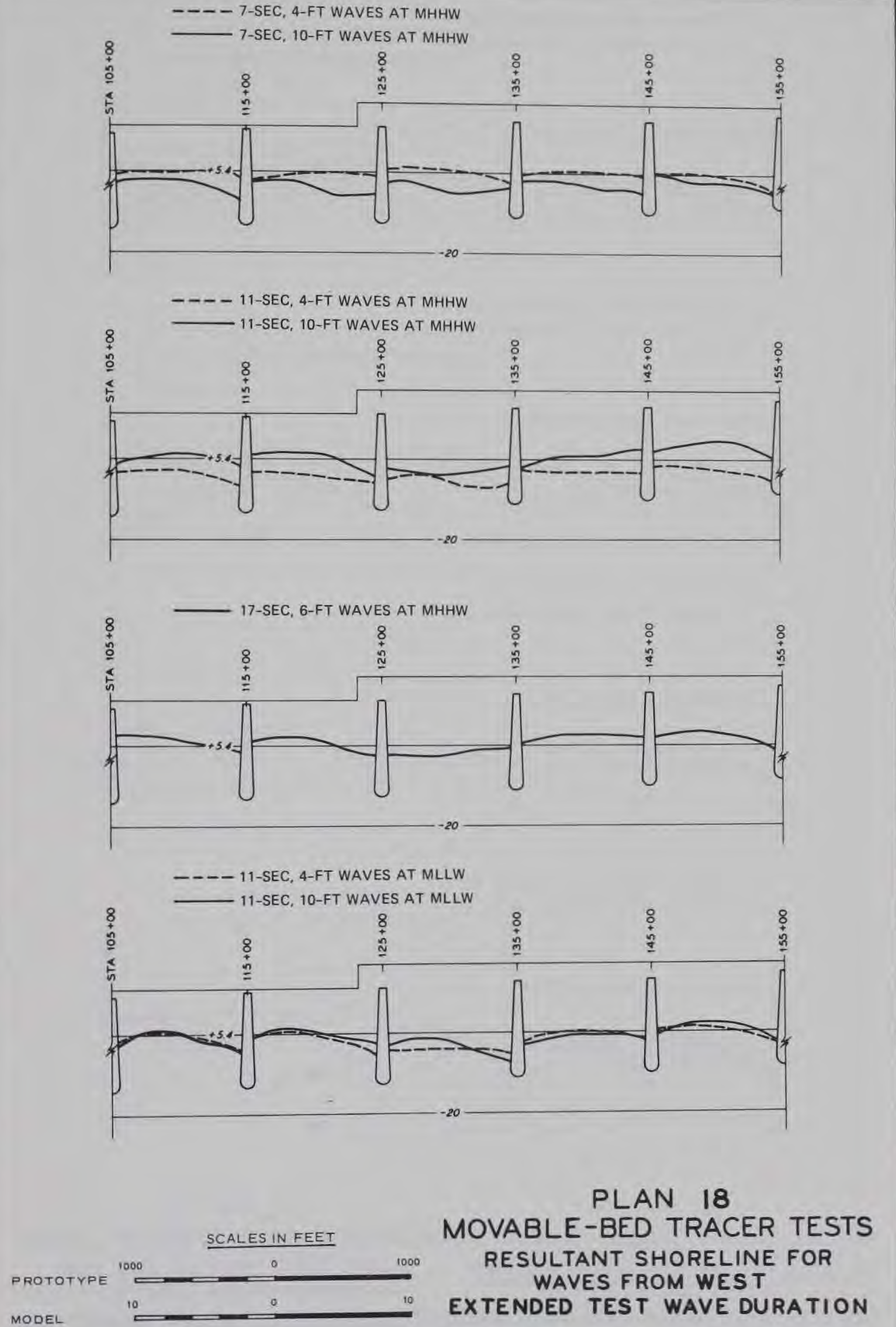
PROTOTYPE 1000 0 1000

MODEL 10 0 10

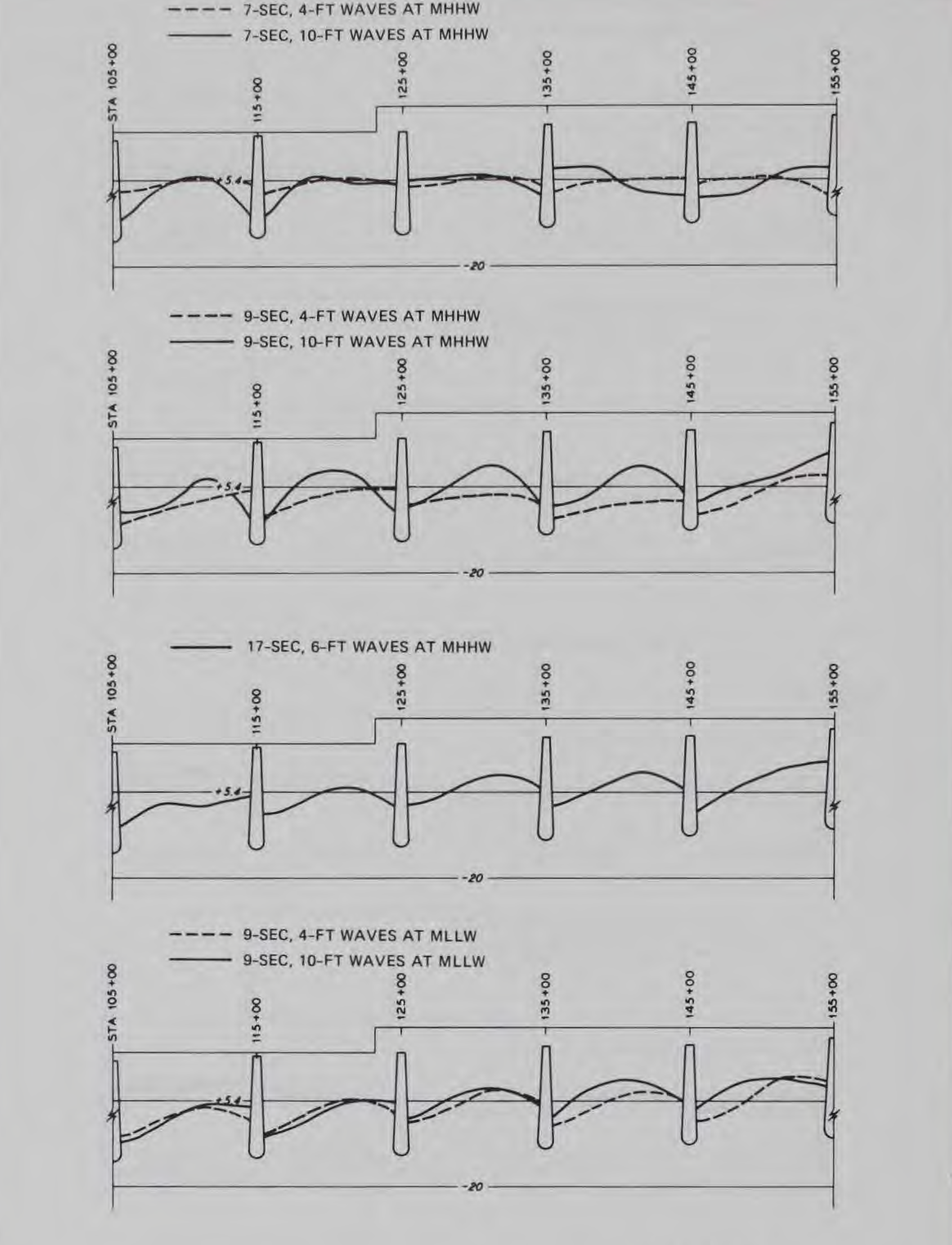
PLAN 14A
MOVABLE-BED TRACER TESTS
RESULTANT SHORELINE FOR WAVES FROM WEST
EXTENDED TEST WAVE DURATION



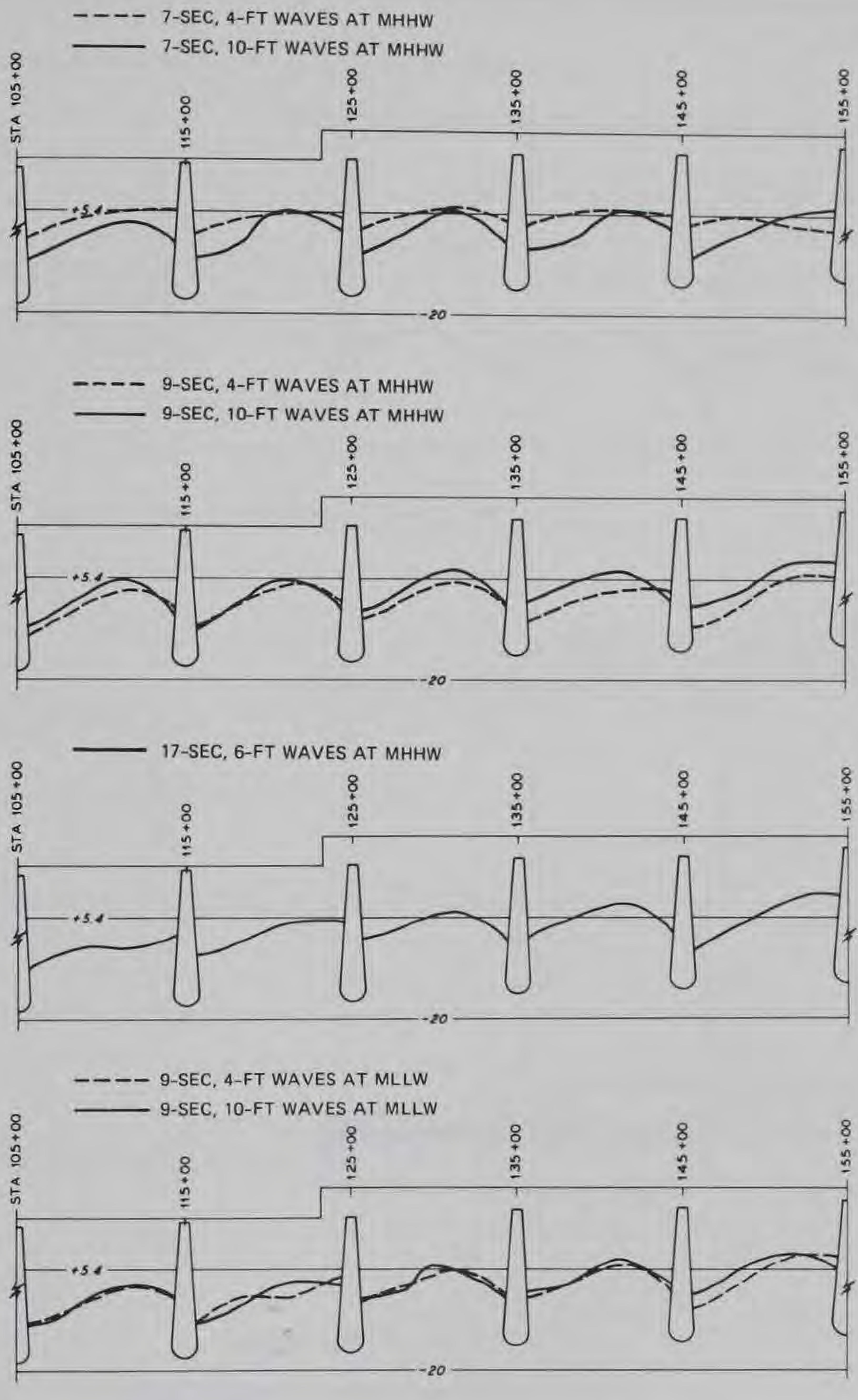
PLAN 17
MOVABLE-BED TRACER TESTS
RESULTANT SHORELINE FOR WAVES FROM WEST EXTENDED TEST WAVE DURATION

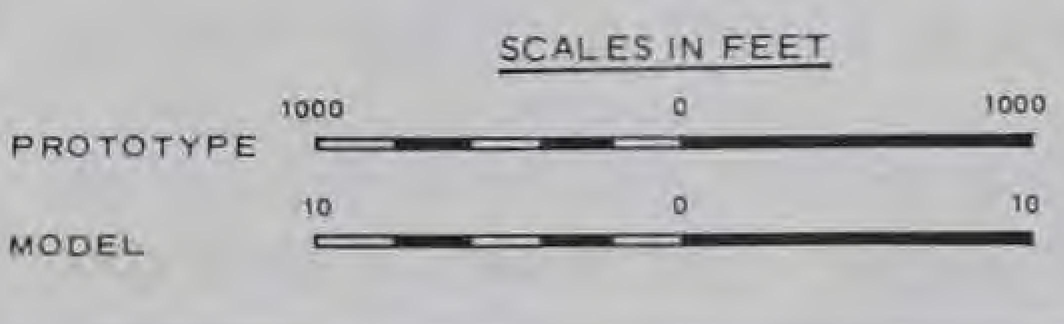


PLAN 18
MOVABLE-BED TRACER TESTS
RESULTANT SHORELINE FOR WAVES FROM WEST EXTENDED TEST WAVE DURATION

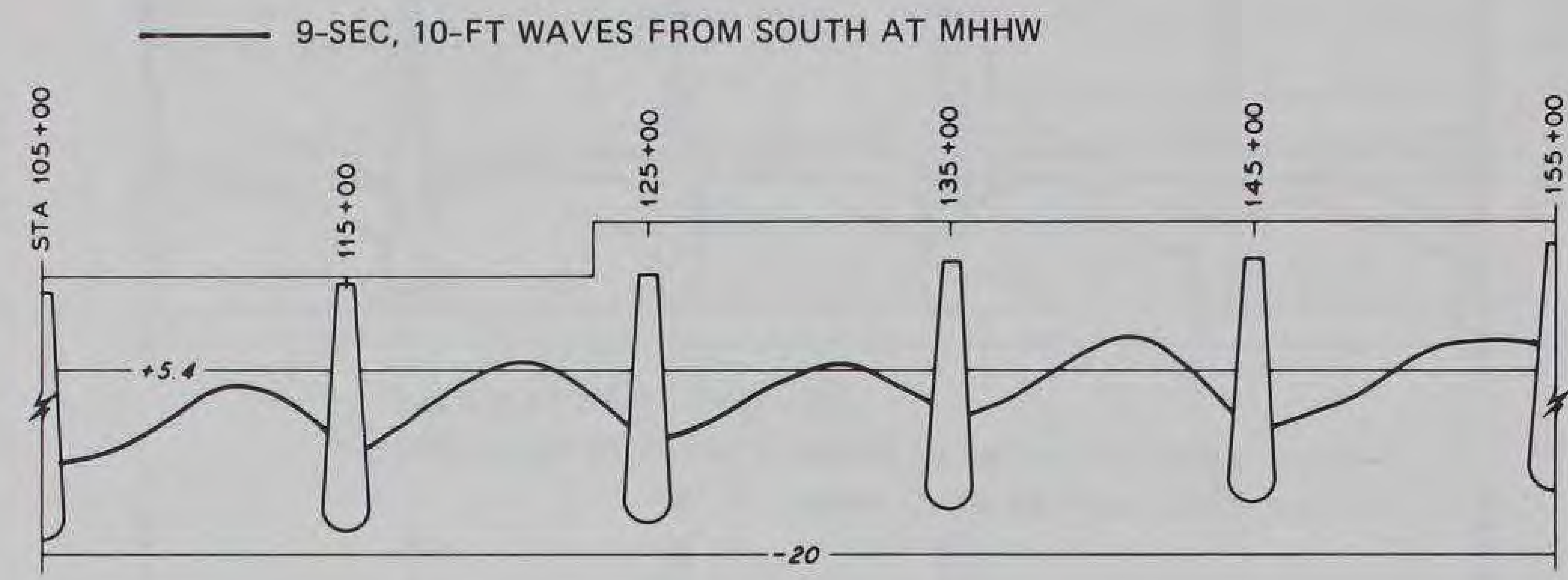


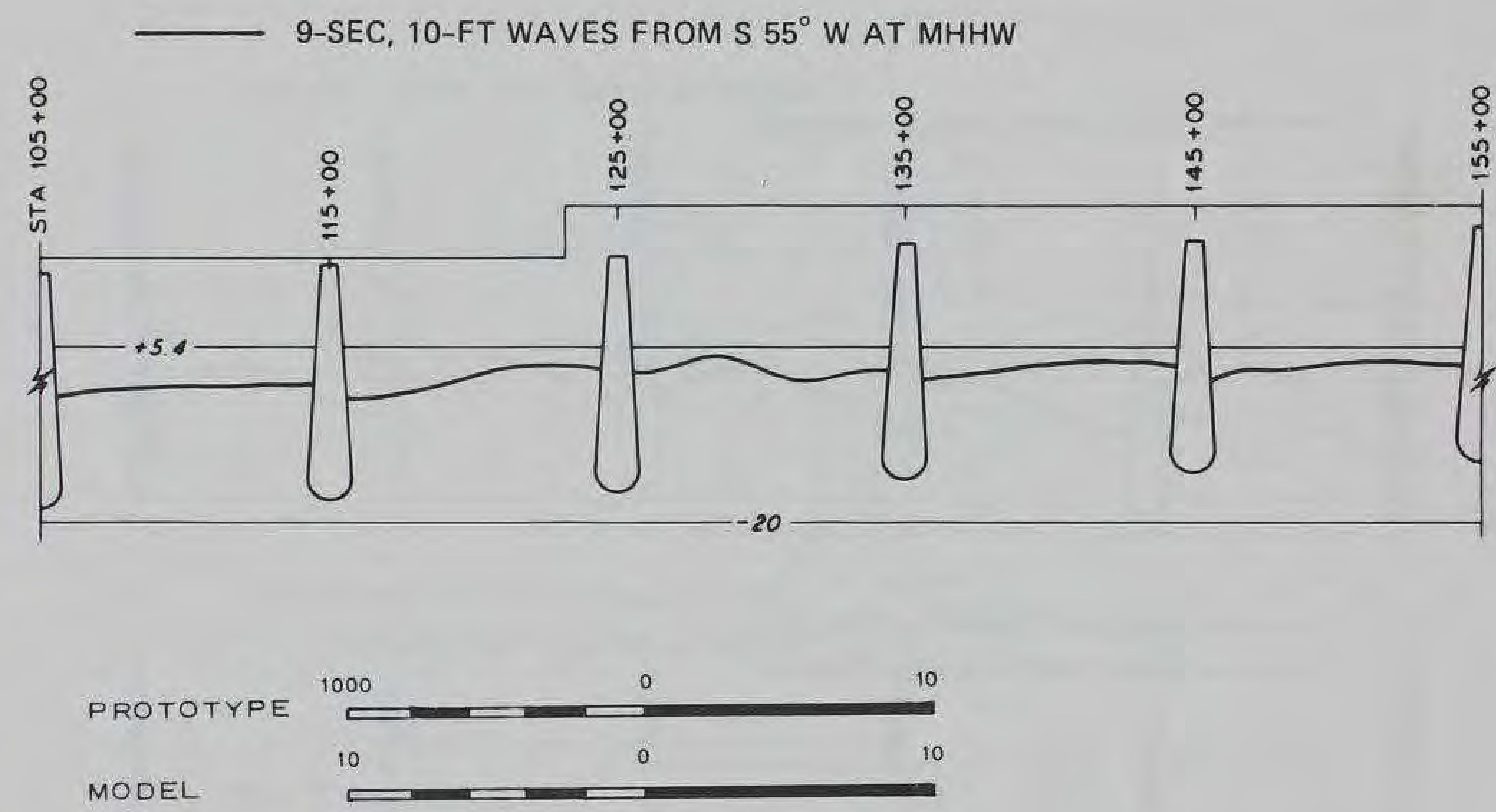
PLAN 18
MOVABLE-BED TRACER TESTS
RESULTANT SHORELINE FOR WAVES FROM SOUTH
EXTENDED TEST WAVE DURATION



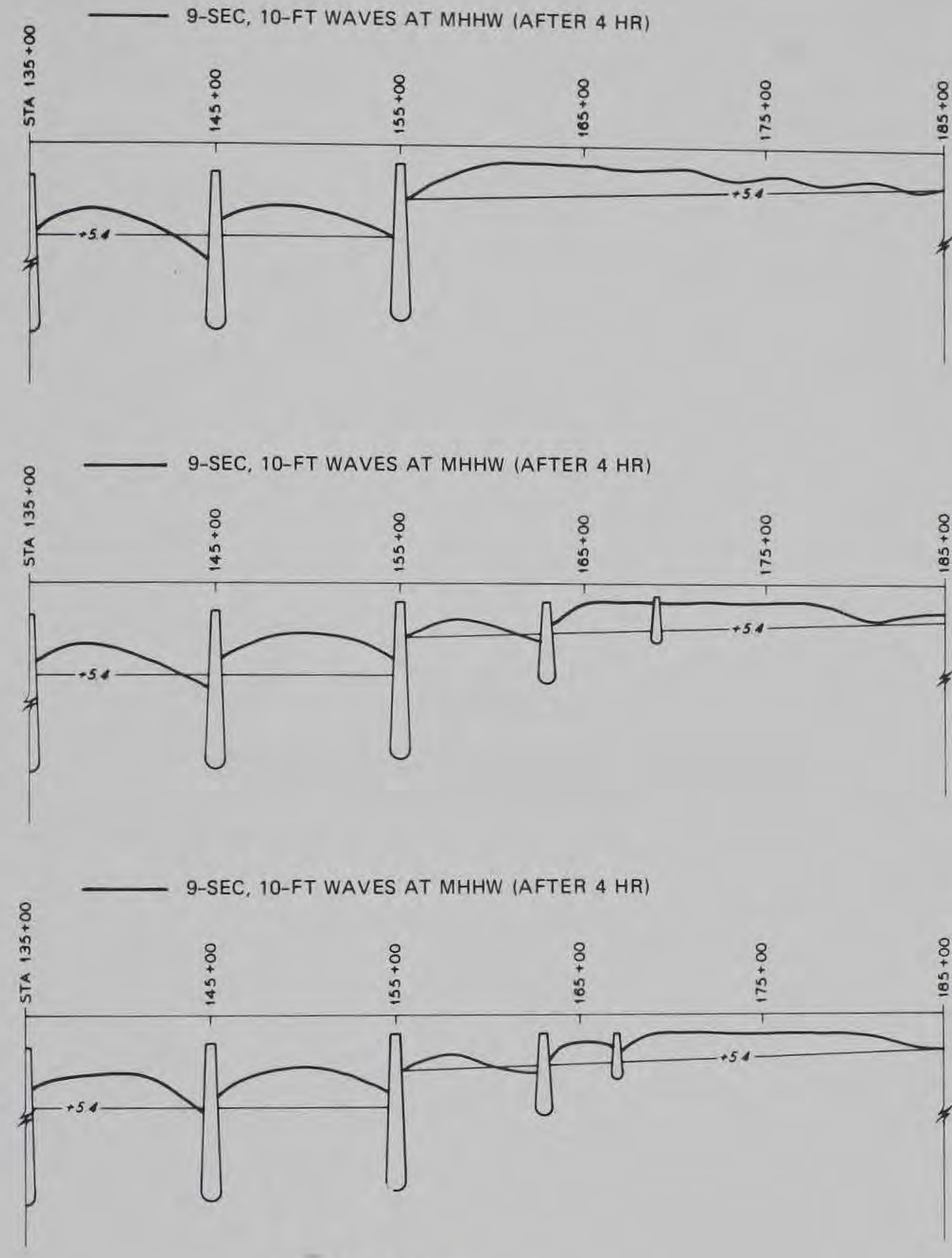


PLAN 9
MOVABLE-BED TRACER TESTS
RESULTANT SHORELINE FOR WAVES FROM SOUTH
EXTENDED TEST WAVE DURATION





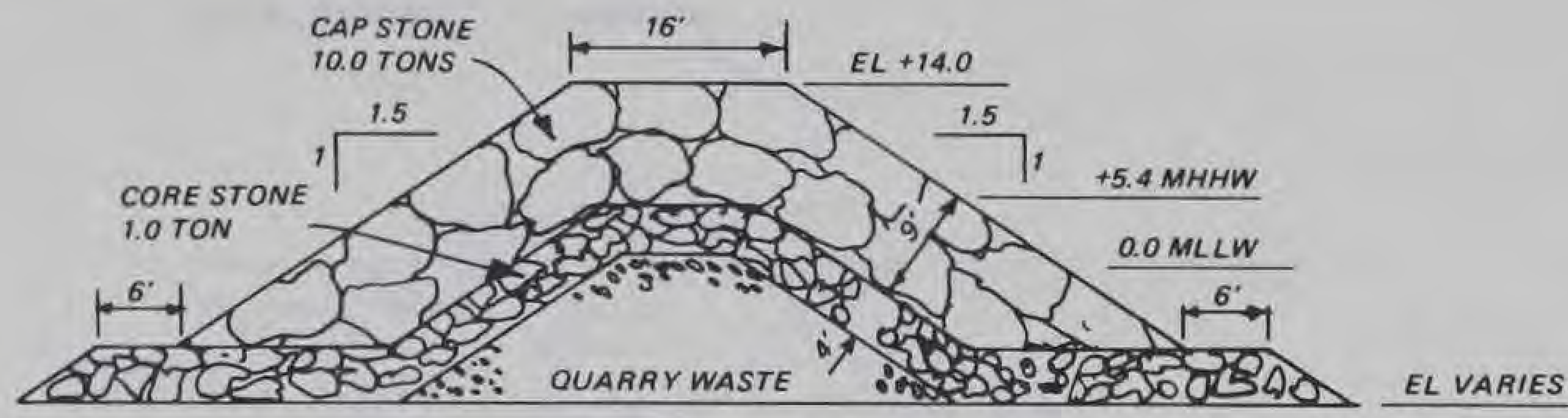
PLAN 9
REVISED BEACH SLOPE
(1:10 ABOVE AND 1:15 BELOW MLLW)
MOVABLE-BED TRACER TESTS
RESULTANT SHORELINE FOR
WAVES FROM SOUTH AND S 55° W
EXTENDED TEST WAVE DURATION



PLAN 9, 9A, AND 9B
MOVABLE-BED TRACER TESTS
RESULTANT SHORELINE FOR WAVES FROM NORTHWEST

APPENDIX A

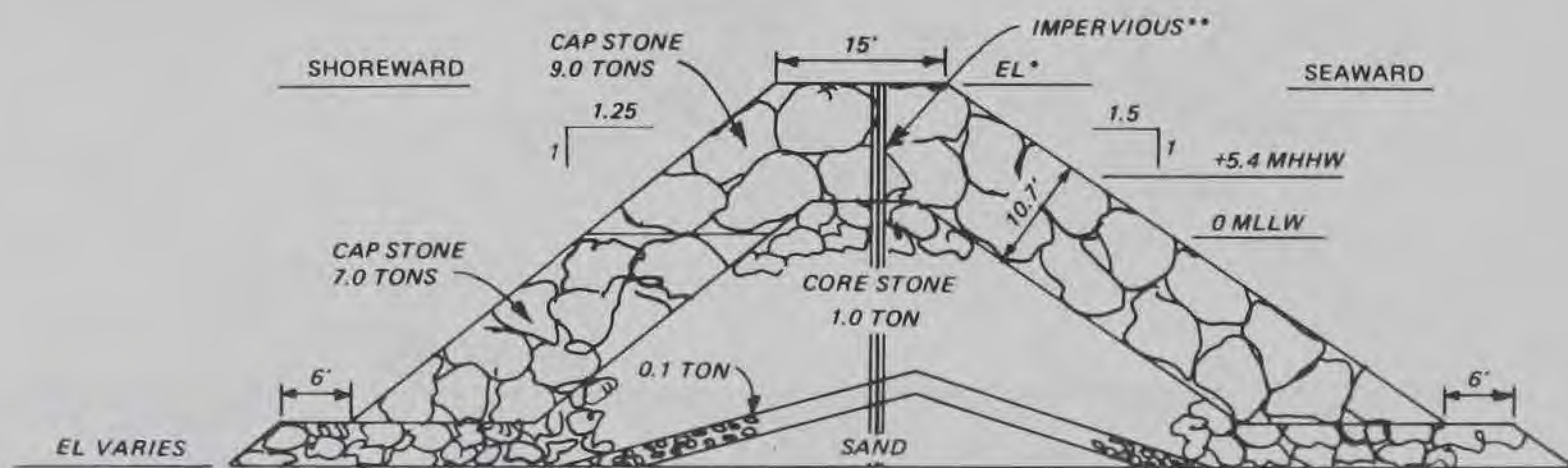
TYPICAL SECTIONS OF VARIOUS STRUCTURES TESTED IN THE MODEL



PROPOSED SOUTH JETTY EXTENSION

PLAN*	EL, FT
1-5	+14.0
29	+14.0
30-31F	+22.0

PLAN**	IMPERVIOUS
1-5	NO
29-30A	NO
31-31F	YES

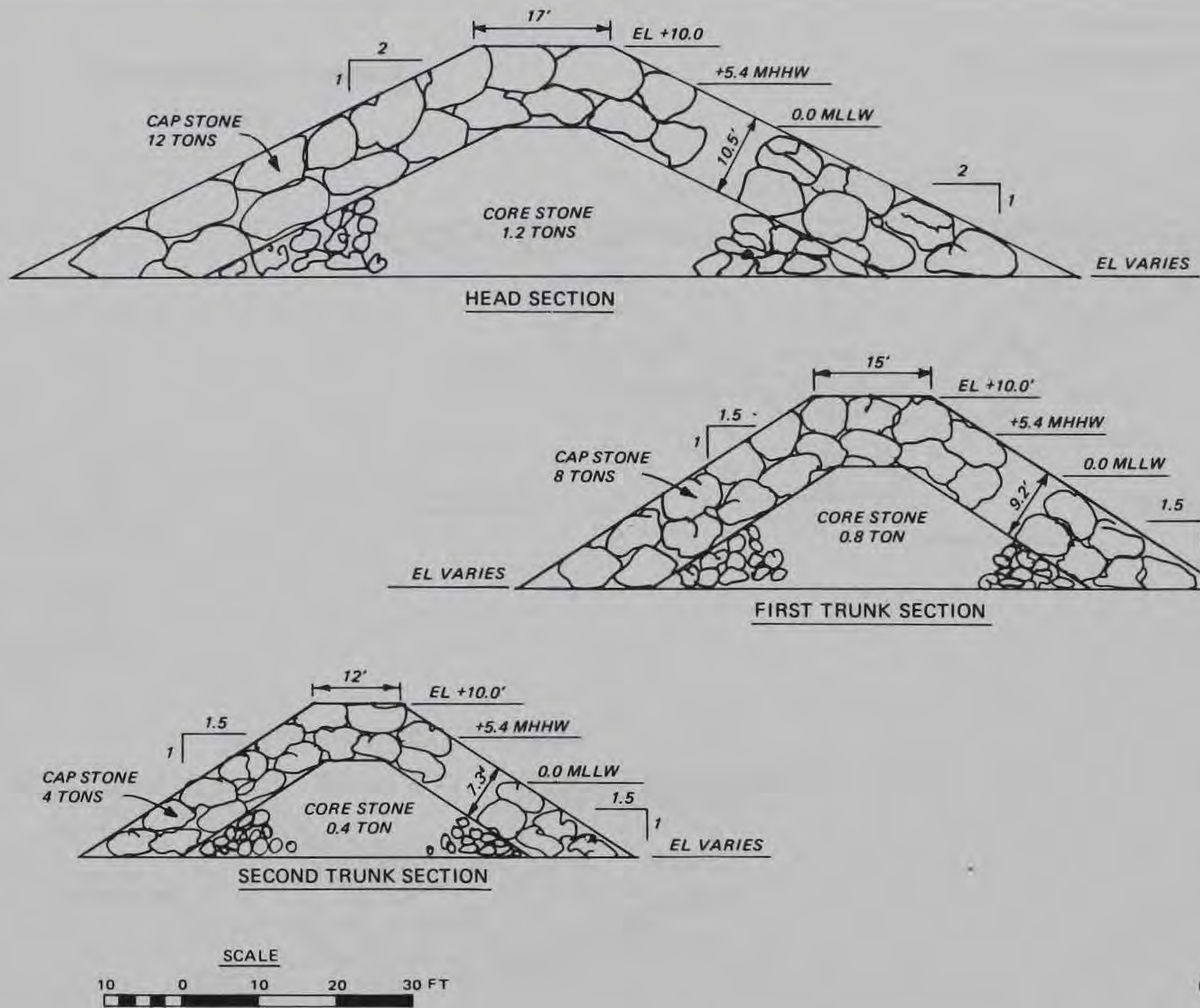


PROPOSED OFFSHORE BREAKWATER

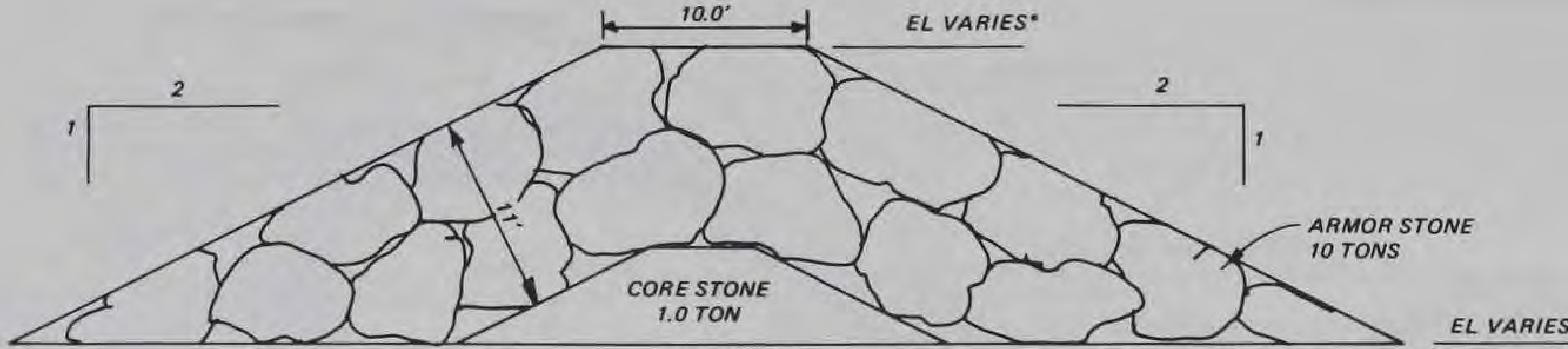
SCALE

JETTY AND BREAKWATER SECTIONS

PLANS 1-5, 29-31F

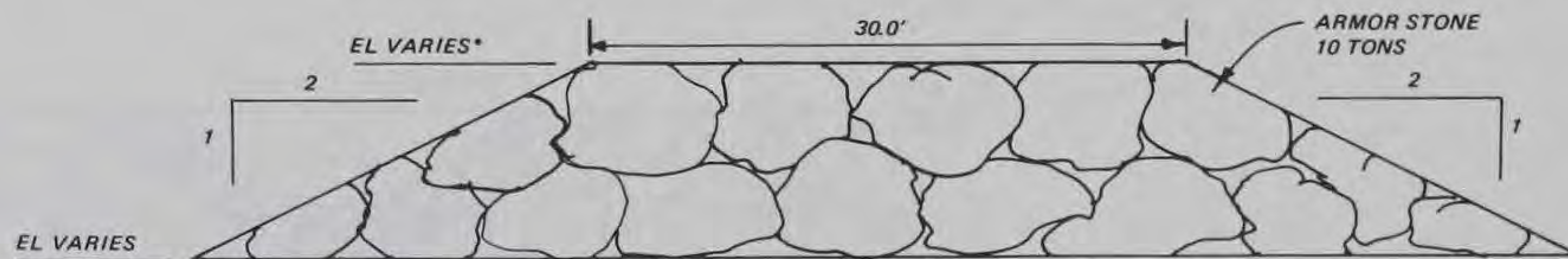


GROIN SECTIONS
PLANS 6-12, 18

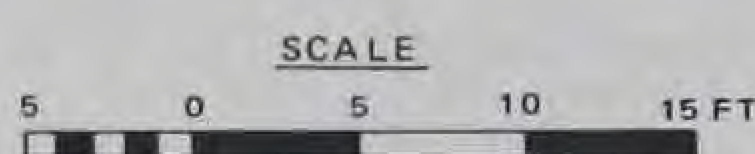


BREAKWATER

PLAN*	BREAKWATER CROWN ELEVATION, FT	LOW-SILL CROWN ELEVATION, FT
13	0.0	-5.0
14	0.0	-5.0
15	0.0	--
17	+5.0	—

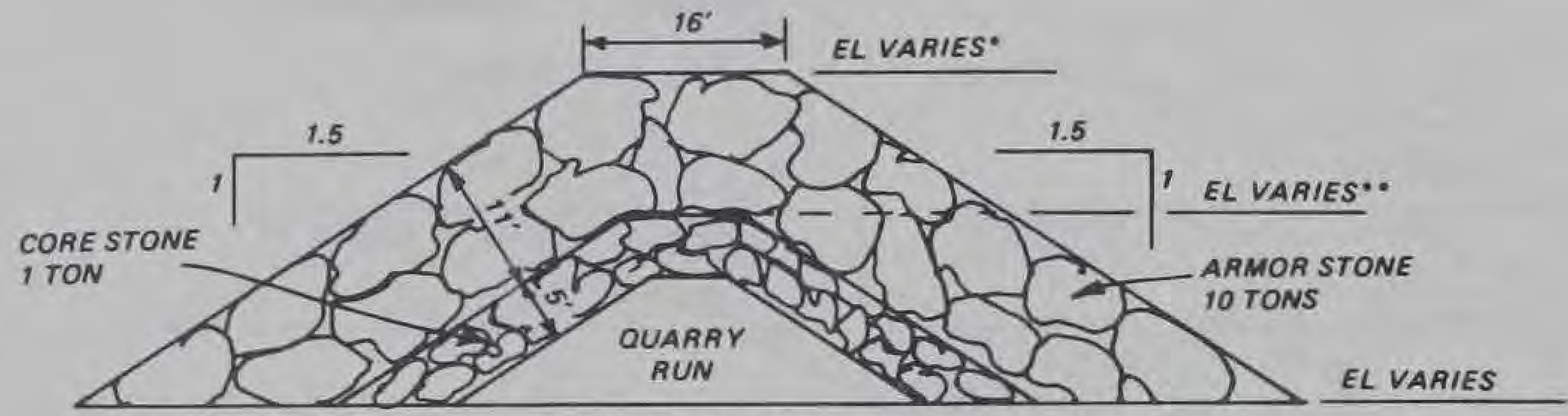


LOW SILL



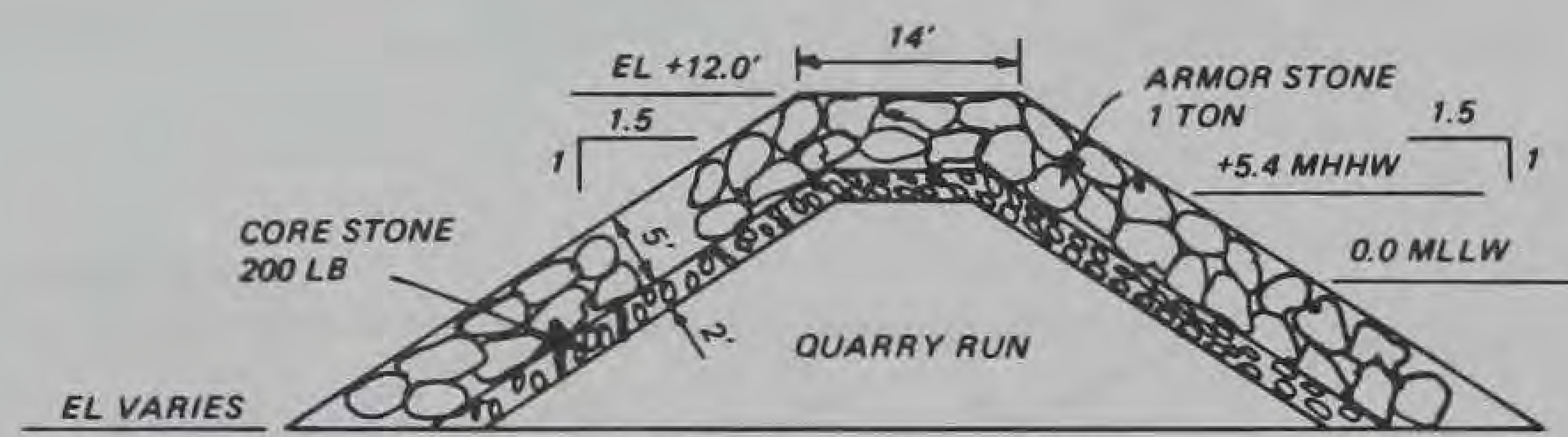
BREAKWATER AND LOW-SILL SECTIONS

PLANS 13-15, 17



SEAWARD SECTION†

PLAN*	CROWN ELEVATION, FT
19-21E	+16.0
21F	+14.0
22-25	+16.0
25A-26A	+14.0
27-28	+16.0
46-62	+16.0



SHOREWARD SECTION

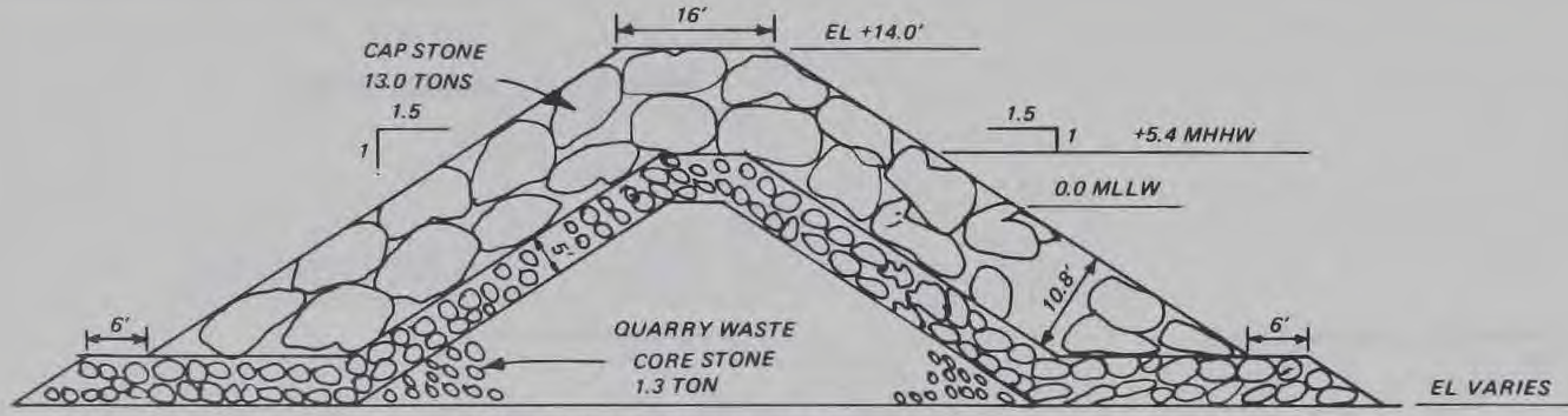
PLAN**	CORE ELEVATION, FT
19-28	0.0
46-59	0.0
60-62	+4.0

NOTE: FOR PLAN 57, ENTIRE INNER BREAKWATER WAS MADE IMPERVIOUS. FOR PLANS 56 AND 59, ONLY LAST LEG OF INNER BREAKWATER WAS IMPERVIOUS.

SCALE

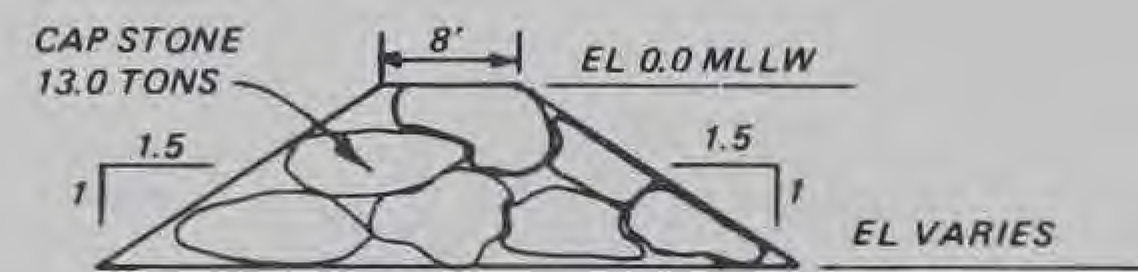
10 0 10 20 30 FT

INNER BREAKWATER SECTIONS
PLANS 19-28, AND 46-62

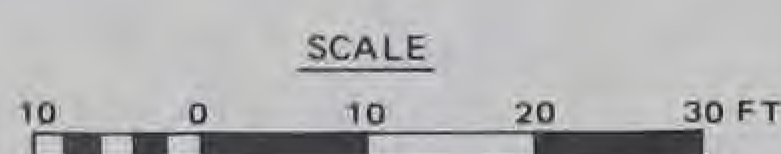


PROPOSED OFFSHORE BREAKWATER

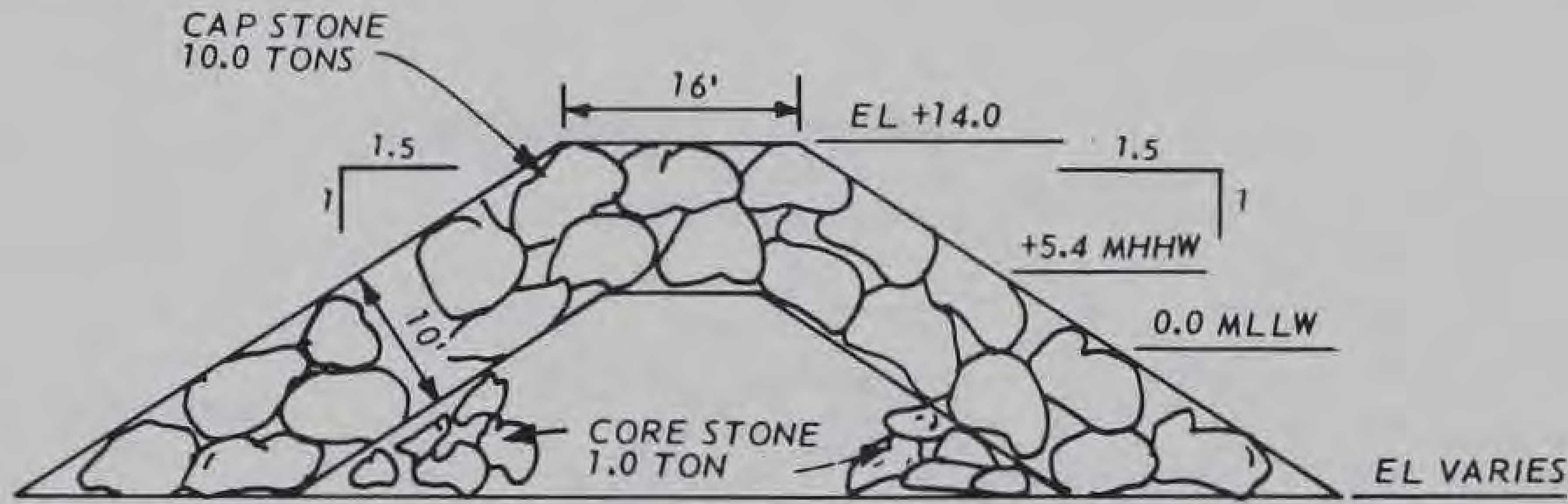
PLAN*	LOW-SILL
32	YES
33	NO
34-36	YES
37-38	NO
38A	YES
38B-39	NO
39A-40B	YES
44	NO



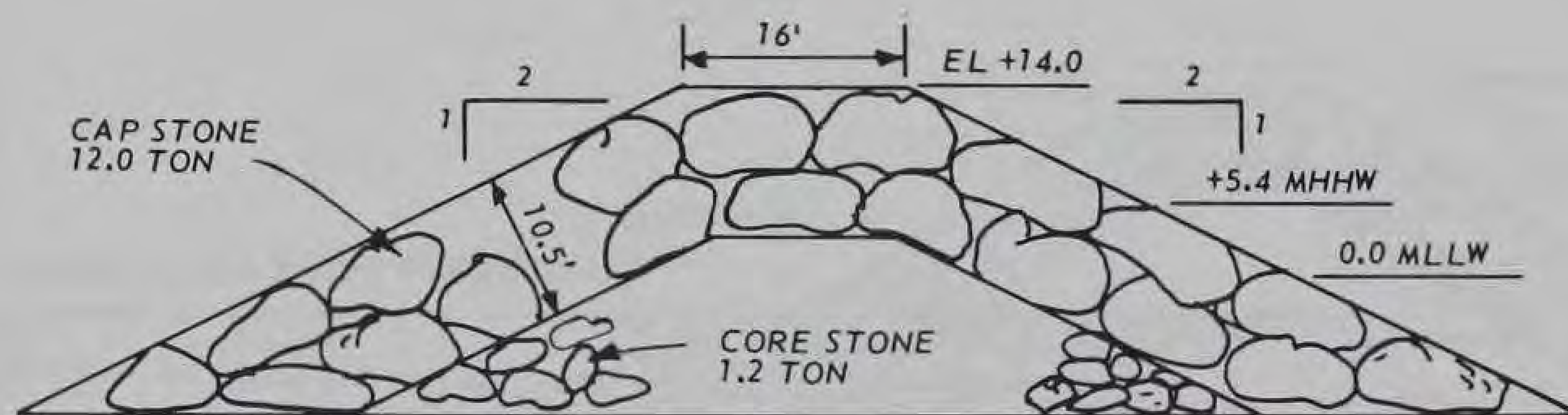
LOW-SILL STRUCTURE*



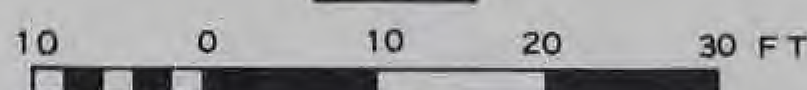
OFFSHORE BREAKWATER AND LOW-SILL SECTIONS
PLANS 32-40B, AND 44



TRUNK SECTION

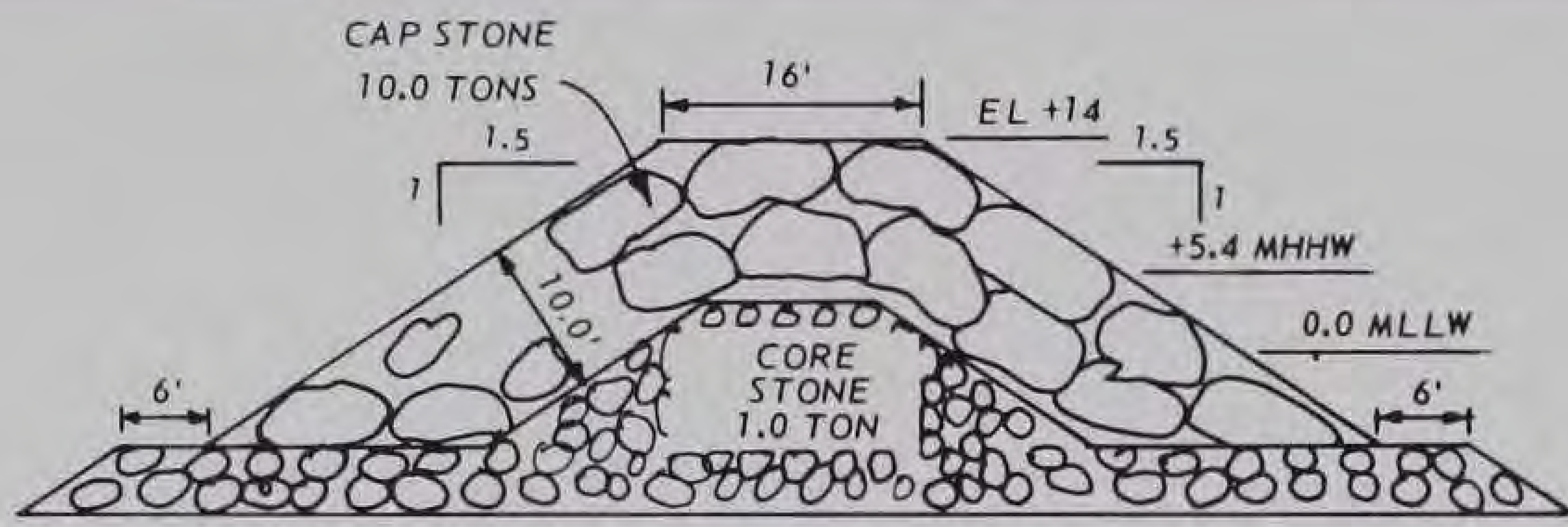


HEAD SECTION



GROIN SECTION

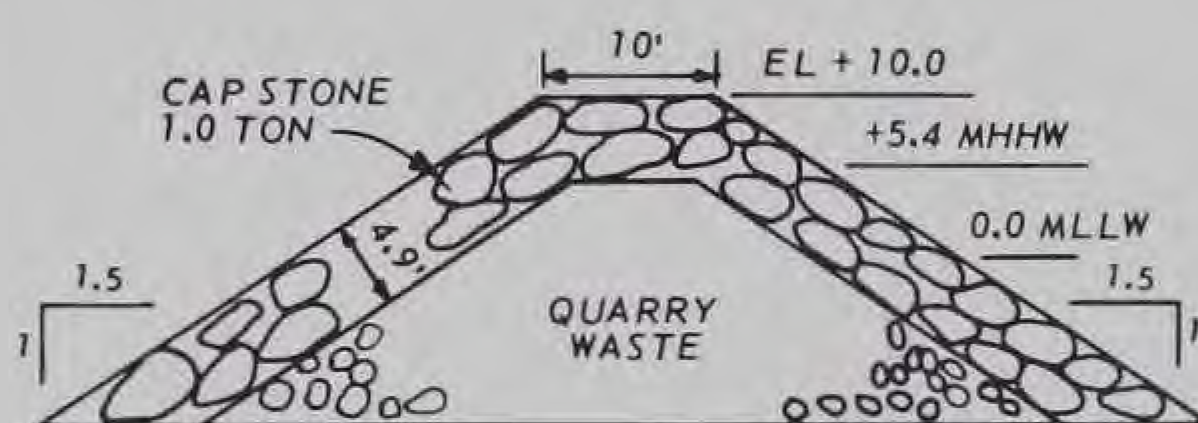
PLANS 35-45



STUB GROIN*

PLAN*	STUB GROIN
46	NO
47-50A	YES
51-57	NO
58-62	YES

PLAN**	INNER JETTY
46-48A	YES
48B-51	NO
52-59	YES
60-62	NO



INNER JETTY**

STUB GROIN AND INNER JETTY SECTIONS

PLANS 46-62

APPENDIX B

NOTATION

A	Area
b	Shallow-water orthogonal spacing
b_o	Deepwater orthogonal spacing
$(b_o/b)^{1/2}$	Refraction coefficient
d_b	Water depth at breaking
D_{50}	Median particle diameter
f	Darcy-Weisbach friction factor
H	Shallow water wave height
H_b	Wave height at breaking
H_o	Deepwater wave height
k_*	Absolute roughness of the beach surface
K	Shoaling coefficient
L	Length
n_D	Ratio of median particle diameter
n'_λ	Ratio of apparent specific weight
T	Time
u_L	Longshore current velocity
V	Velocity
$\forall$	Volume
x	Distance
α	Beach slope
γ	Specific weight
θ_b	Angle of incident wave at breaking
λ	Horizontal scale
μ	Vertical scale