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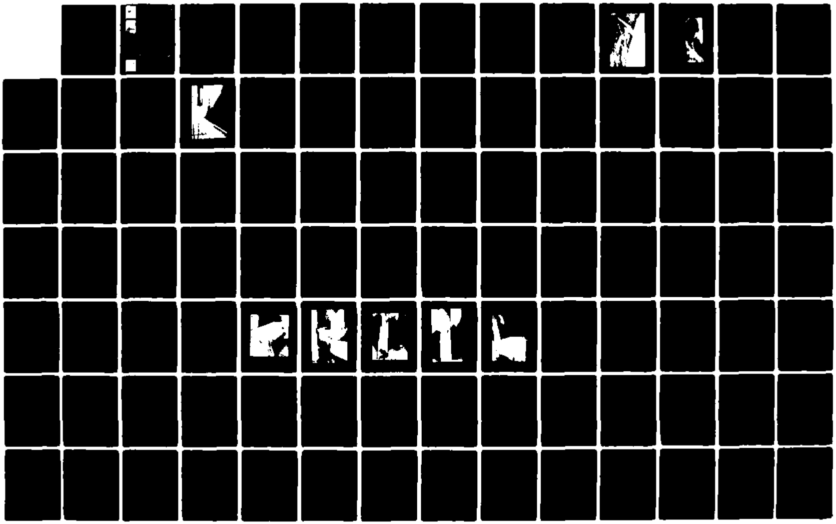

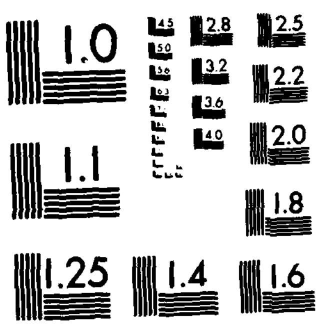

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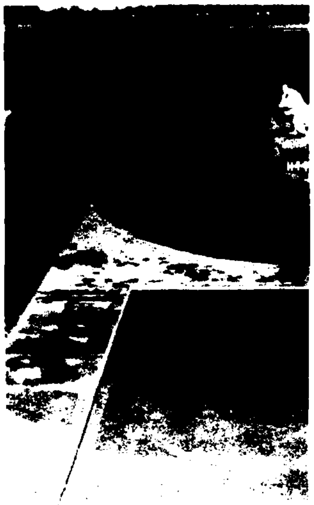

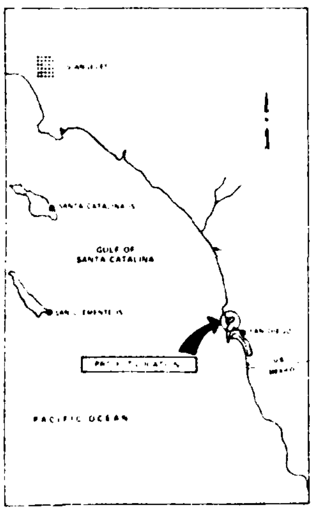

HYDRAULICS
LABORATORY

(12)

TECHNICAL REPORT HL-83-17

MISSION BAY HARBOR, CALIFORNIA, DESIGN FOR WAVE AND SURGE PROTECTION AND FLOOD CONTROL

Hydraulic Model Investigation

by

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June 1983
Final Report

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

A 1:100-scale (undistorted) hydraulic model, reproducing Mission Bay Harbor, approximately 3 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 hazardous entrance conditions, (b) reducing surge inside the harbor, and (c) eliminating potential flood hazards. The original proposal for harbor improvement consisted of a 2,200-ft-long offshore breakwater. The proposed structures for river flood control consisted of (a) a 1,200-ft-long weir in the middle jetty, (b) various south jetty extensions, and (c) diversion dikes on the middle jetty. Nonstructural flood-control measures consisted of incremental sand plug removal and pilot channels. Two wave generators (70 and 80 ft long), crushed coal tracer material, Styrofoam surface floats, and an Automated Data Acquisition and Control

(Continued)

20. ABSTRACT (Continued).

System (ADACS) were used during model operation. For the harbor, it was concluded from model test results that:

a. Existing conditions are characterized by strong longshore currents that are redirected seaward by the north and middle jetties for moderate to large wave conditions. In general, clockwise eddies form north of the north jetty and counterclockwise eddies form south of the middle jetty. No shoaling of the harbor entrance was observed. Wave heights in the entrance channel were frequently excessive but were largely dissipated upon reaching the small-boat basins. Long-period wave (surge) tests revealed substantial oscillations in the entrance channel and the small-boat basins for a number of incident wave periods.

b. The original improvement plan for wave protection for Mission Bay Harbor (i.e., the offshore breakwater of Plan 1) was ineffective in reducing wave heights in the bay entrance to an acceptable level.

c. Moving the breakwater into shallower water (Plan 2 series) decreased wave heights in the entrance channel to a more acceptable level, but the 1.5-ft criterion for the entrance channel still was exceeded. It was apparent that excessive wave energy was being transmitted through the voids of the breakwater.

d. By sealing the core of the offshore breakwater (Plan 3 series), wave energy that had passed through the voids of the structure was largely eliminated. Of the plans tested, Plan 3G provided the most effective reduction of wave energy with a reduction of the volume of rock required for construction of 50 percent when compared with the originally proposed Plan 1. This plan was effective even under the most extreme conditions (i.e., removal of all revetment within the bay and an increase in swl to +7.6 ft. This plan also considerably reduced long-period wave amplification (generally 50 percent or more) in the channel and basins. Shoaling of the harbor entrance was very slight and occurred only for one extreme test condition.

e. The Plan 9 offshore breakwater allowed slightly more short-period wave energy to enter the entrance channel than did Plan 3G but long-period responses within the bay were generally slightly less. Plan 9 reduced the volume of rock required for construction by 54 percent, when compared with the originally proposed plan, and should be considerably easier and less expensive to construct than Plan 3G.

f. Based on the results of all model tests, Plans 3G and 9 are considered as viable alternatives for providing wave and surge protection to Mission Bay.

For the San Diego River, it was concluded from model test results that:

a. The flood-control channel at project depth is prone to severe shoaling for waves from any direction, but particularly for waves from the southwest. The flood-control channel at project depth is also quite capable of discharging the maximum flood flow tested (97,000 cfs) without causing flooding upstream.

b. Tests of the flood-control channel with a +10 ft elevation sand plug (Plan 4A), representative of that presently blocking the river mouth, indicated a flooding hazard for the 49,000- and 97,000-cfs riverflows.

c. A reduction of the elevation of the sand plug to +6 ft reduced the flooding hazard. However, this plan would be difficult to maintain.

d. Removal of sections of the sand plug by dredging (Plans 5-5H) proved quite effective in reducing the flood hazard. Again, this plan may be difficult to maintain.

e. Tests conducted with a +6 ft elevation weir built into the middle jetty for a +10 ft elevation sand plug (Plan 6C) showed significantly reduced water-surface elevations for all river discharges.

f. Of the plans tested to prevent the formation of the sand plug, Plan 7J (2,373-ft-long south jetty extension) was effective in preventing all wave-induced river shoaling. However, because of the length of structure required, this plan would be quite expensive. Plan 7C (1,273-ft-long south jetty extension) would eliminate channel shoaling by nearshore material.

g. All plans involving a pilot channel cut into the sand plug worked well in preventing river flooding.

h. Plan 8A (400-ft-long diversion structure on the middle jetty) was the optimum plan tested for preventing shoaling of the south entrance to the bay during blowout of the sand plug by flood conditions.

PREFACE

A request for a model investigation of Mission Bay Harbor, 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 (OCE), U. S. Army. Initial funds were authorized by SPL on 8 January 1979, with subsequent installments authorized through 14 November 1981.

The model study was conducted at the U. S. Army Engineer Waterways Experiment Station (WES) during the period January 1979 through March 1982 under the direction of Mr. H. B. Simmons, Chief of the Hydraulics Laboratory, and Dr. R. W. Whalin and Mr. C. E. Chatham, former and acting Chiefs of the Wave Dynamics Division, respectively. Tests were conducted by Mr. C. R. Curren, Project Engineer, with the assistance of Messrs. R. E. Ankeny, Computer Technician, and L. L. Friar, Electronics Technician. This report was prepared by Mr. Curren. During the course of investigation, liaison was maintained with SPL by means of conferences, telephone communications, and monthly progress reports. Messrs. Chatham and Curren and Drs. Whalin and L. Z. Hales visited Mission Bay to confer with representatives of 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:

Mr. Ted Albrecht	SPD
Mr. Bob Edmisten	SPD
Mr. Ted Durst	SPD
Mr. Charles Fisher	SPL
Mr. Tad Nizinski	SPL
Mr. Bob Koplin	SPL
Mr. Ed Chew	SPL
Mr. Mauricio Munoz	SPL
Mr. John H. Lockhart, Jr.	OCE

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

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
acres	4046.856	square metres
cubic feet per second	0.02831685	cubic metres per second
feet	0.3048	metres
feet per second	0.3048	metres per second
miles (U. S. statute)	1.609344	kilometres
square feet	0.09290304	square metres
square miles (U. S. statute)	2.589988	square kilometres

MISSION BAY HARBOR, CALIFORNIA, DESIGN FOR WAVE AND SURGE PROTECTION AND FLOOD CONTROL

Hydraulic Model Investigation

PART I: INTRODUCTION

The Prototype

1. Mission Bay is located on the coast of southern California about 10 miles* north of the entrance to San Diego Bay (Figure 1). The complex

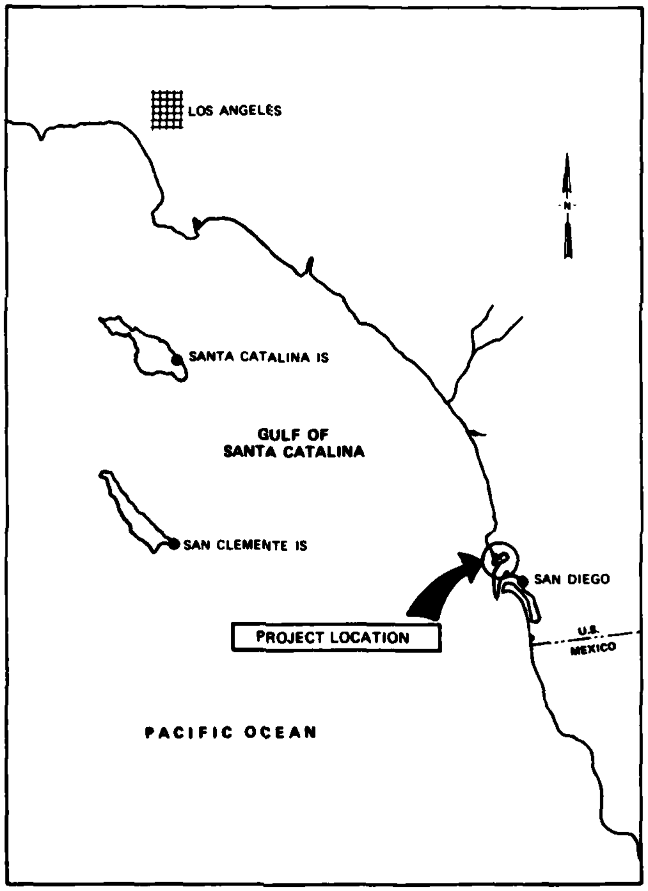

Figure 1. Project location

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

covers an area of approximately 4,000 acres and is used entirely for recreational purposes. The entrance to the bay is protected by a 3,800-ft-long north jetty and a 4,600-ft-long middle jetty. The middle jetty also serves to separate the navigation channel from the San Diego River flood-control channel. A 2,000-ft-long south jetty forms the southern border to the San Diego River (Figure 2). The sea floor is characterized by gently sloping contours that bend around the entrance and increase somewhat in slope north of the entrance.

The Problem

2. In the course of this study, three major problems were investigated.

   a. Hazardous conditions at the entrance to the harbor due to large short-period (7 to 20 sec) waves.

   b. Surge due to long-period (30 to 140 sec) waves causing damage to boats and facilities inside the harbor.

   c. Potential flood hazards due to a sand plug at the mouth of the San Diego River flood-control channel (Figure 3).

Proposed Improvements

3. Improvements for Mission Bay, originally proposed by the U. S. Army Engineer District, Los Angeles (SPL), were as follows:

   a. The harbor. The original proposal for wave and surge protection consisted of a 2,200-ft-long offshore breakwater seaward of the entrance channel in approximately 30 ft of water with a crown elevation of +22.5 ft.*

   b. The flood-control channel. The original proposals for solving the potential flood hazard for the San Diego River called for either:

      (1) Removal of the sand plug by dredging and the construction of a 1,073-ft-long south jetty extension to prevent the plug from re-forming.

      (2) Installation of a 1,200-ft-long weir with a +6 ft crown elevation in the existing middle jetty to act as an emergency relief until the plug breaches.

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

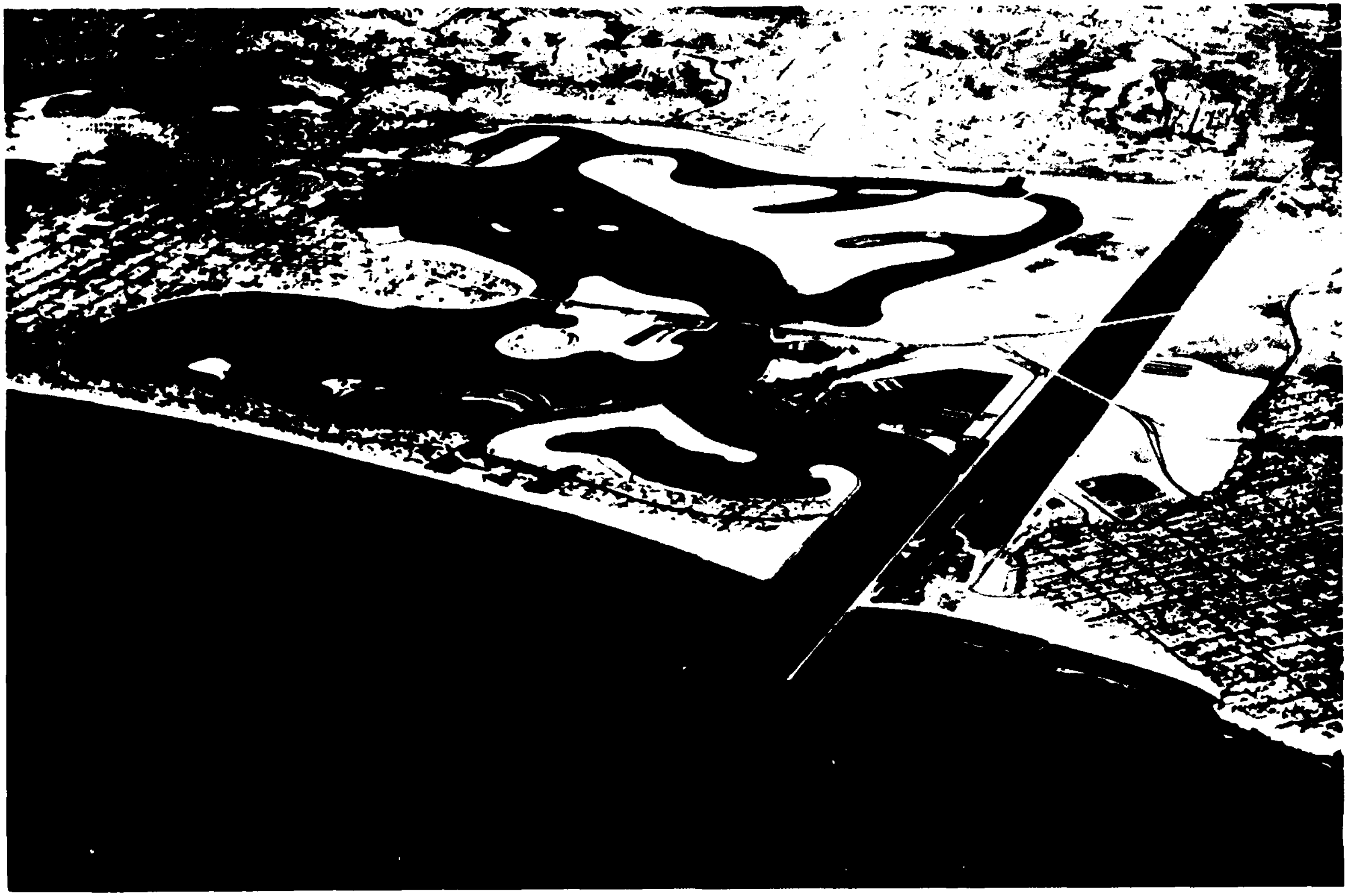

Figure 2. Aerial view of Mission Bay on 11 August 1961

Figure 3. Aerial view of sand plug on 5 March 1978

Purposes of the Model Study

4. Purposes of the model study were to:

   a. Determine existing long- and short-period wave conditions at the bay entrance and inside Quivira and Mariners Basins and conditions that cause shoaling of the river mouth.

   b. Study long- and short-period wave conditions and shoaling with the proposed improvement plans installed in the model.

   c. Develop alternative remedial plans for 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 perform adequately.

Wave-Height Criteria

5. Completely reliable criteria have not yet been developed for ensuring satisfactory navigation and berthing in small-craft harbors. However, for the study reported herein, SPL specified that for an improvement plan to be acceptable, maximum wave heights in the harbor entrance should not exceed 1.5 ft for deepwater waves of 6 ft or less and maximum wave heights in the basins should not exceed 1.0 ft for all wave conditions.

PART II: THE MODEL

Design of the Model

6. The Mission Bay model (Figure 4) was constructed to an undistorted 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.
- b. Absolute size of model waves.
- c. Available shelter dimensions and area required for model construction.
- d. Efficiency of model operation.
- e. Available wave-generating and wave-measuring equipment.
- f. Model construction costs.

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

Characteristic	Dimension*	Model:Prototype Scale Relation
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	$V_r = L_r^{1/2} = 1:10$
Discharge	L^3/T	$Q_r = L_r^{5/2} = 1:100,000$

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

7. Ideally, a quantitative, three-dimensional, movable-bed model investigation would best reproduce the formation of the sand plug across the San

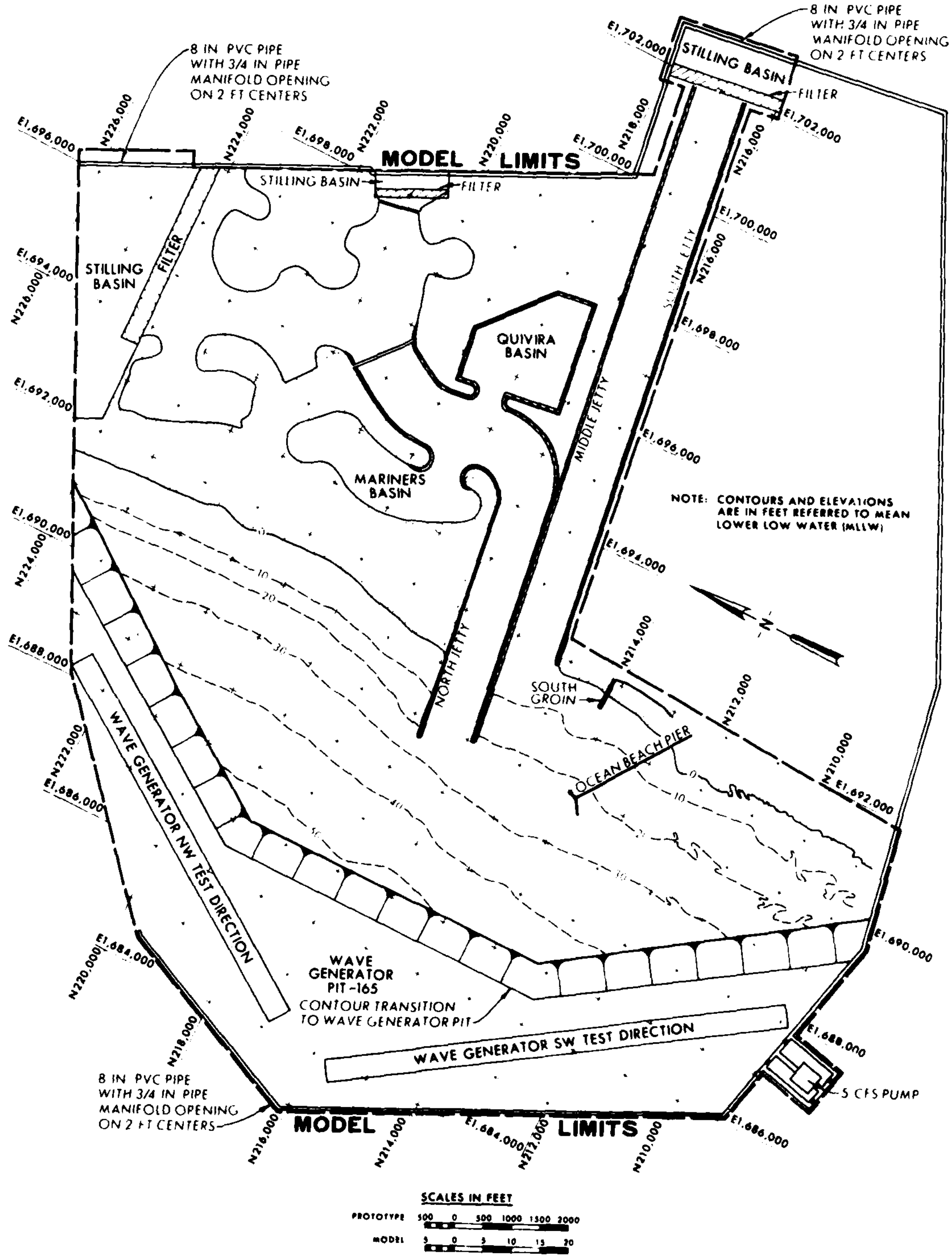

Figure 4. Model layout

Diego River mouth and indicate the effectiveness of various project plans to prevent the plug from re-forming. 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; Hales 1979):

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 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 Mission Bay project, the model was molded in cement mortar (fixed bed) at an undistorted scale of 1:100 and a tracer material was obtained to determine qualitatively the degree of sediment movement for various plans.

9. 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.

10. The originally proposed improvement plans for Mission Bay submitted by SPL included the use of rubble-mound breakwaters, weirs, diversion dikes, 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 (LeMehaute 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 the U. S. Army Engineer Waterways Experiment Station (WES) (Brasfeild 1965, 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 Mission Bay, 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 Mission Bay 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

11. The model reproduced approximately 3 miles of shoreline and underwater contours to offshore depths ranging from 40 to 54 ft, with a sloping transition to the wave generator pit elevation of -165 ft. The total model area of 17,500 sq ft represented about 6.3 square miles in the prototype. A general view of the model is shown in Figure 5. Vertical control for model construction was based on the mean lower low water (mllw) elevation of

Figure 5. General view of model

0.0 ft. Horizontal control was based on a local prototype grid system.

12. Model waves were generated by two wave generators (80 and 70 ft long) each with trapezoidal-shaped, vertical-motion plungers. The vertical motion of each 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 generators were mounted on retractable casters which enabled them to be positioned to generate waves from the required directions.

13. A water circulating system (Figure 4) consisting of intake and discharge pipes, a centrifugal pump, four valves, and an electronic flowmeter were used in the model to reproduce maximum steady-state ebb and flood tidal flows in the entrance to the bay, and river flood flows in the San Diego River.

14. An Automated Data Acquisition and Control System (ADACS), designed and constructed at WES (Figure 6), was used to secure wave-height data at

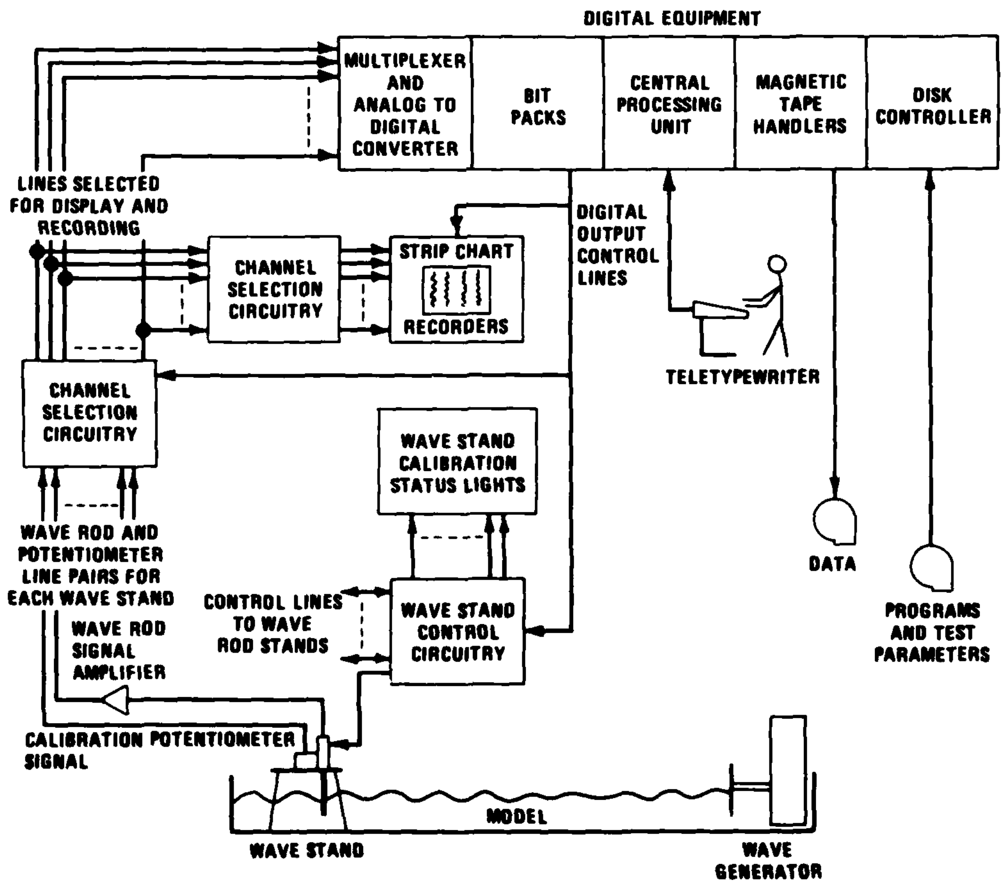

Figure 6. Automated Data Acquisition and Control System (ADACS)

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.

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

16. As previously discussed 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 (Bottin and Chatham 1975, Curren and Chatham 1977, Bottin 1977, Curren and Chatham 1979, Curren and Chatham 1980), 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 η_D ; and the relative specific weight ratio η'_γ (Figure 7). These relations were determined experimentally using a wide range of wave conditions and beach materials and are valid mainly for the breaker zone.

17. Noda's scaling relations indicate that movable-bed models with scales in the vicinity of 1:100 (model to prototype) should be distorted (i.e., they should have different horizontal and vertical scales). Since the fixed-bed model of Mission Bay 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

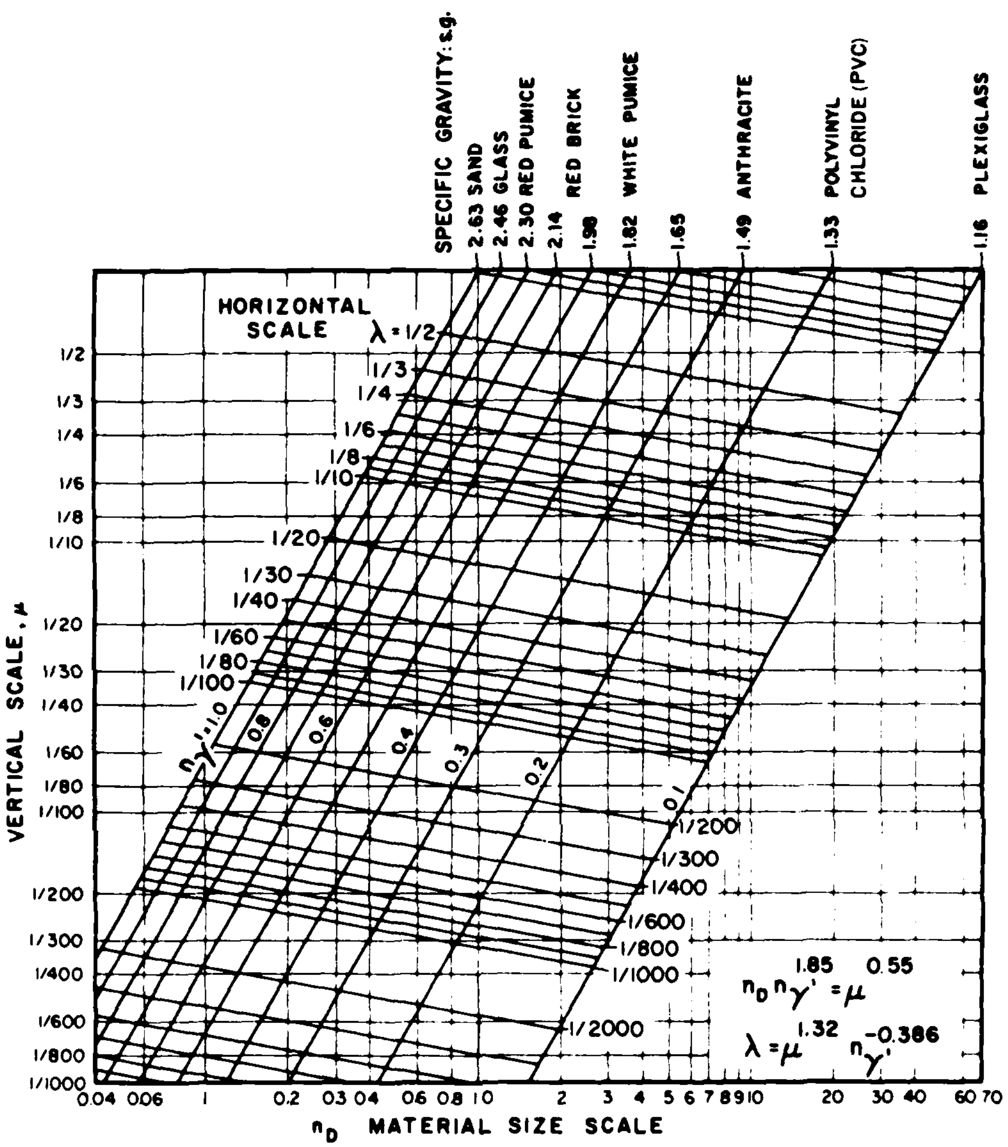

Figure 7. Graphical representation of model law (Noda 1971)

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. Hereinafter, the use of the term "tracer" will refer to this crushed coal tracer material.

PART III: TEST CONDITIONS AND PROCEDURES

Selection of Still-Water Levels

18. Still-water levels (swl's) for wave-action models are selected so that various wave-induced phenomena that are dependent on water depths are accurately reproduced in the model. These phenomena include refraction of waves 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.

19. From U. S. Coast and Geodetic Survey records of 1950-1961 (now, National Ocean Survey), the mllw level at Mission Bay is 0.0 ft, and the mean higher high water (mhhw) level is +5.4 ft. The mhhw stage was considered to be representative 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 was selected for use in the model to determine if the relative effectiveness of various plans was sensitive to the swl. A median swl of +2.7 was selected for maximum steady-state ebb and flood tidal flows.

Wave Dimensions and Directions

Factors influencing selection of test-wave characteristics

20. 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

21. 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 to 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 (U. S. Army CERC 1977).

22. A wave-refraction analysis, conducted by WES for a previous investigation, was used for deepwater wave directions ranging from 225 to 315 deg and wave periods from 6 to 19 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 -165 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

23. Estimated durations and magnitudes of deepwater waves approaching Mission Bay, California, obtained from wave hindcasts by National Marine Consultants (1960) and Marine Advisors (1961) as in the previous Mission Bay model study (Ball and Brasfeild 1969). These data were consolidated into deepwater test directions of northwest, west, and southwest and are summarized in Table 1. Using refraction coefficients from the refraction analysis discussed in paragraph 22 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 deg	Selected Shallow-Water Wave Test Direction deg	Selected Test Wave Period sec	Selected Test Wave Height ft
Northwest (315)	294	7	9
		9	9
			13
		11	9
			15
		13	11
			17
		15	11
			17
		17	11
			15
		19	6
West (270)	267	7	9
		9	7
			11
		11	7
			13
		13	7
			15
		15	7
			13
		17	5
			13
		19	5
Southwest (225)	234	7	7
		9	11
		11	11
		13	11
		15	9
		17	5
		19	7

Prototype Flood Flows for the San Diego River

24. Prototype flood flows for the San Diego River were provided by SPL for various exceedance intervals. The flows selected for testing in the model are as follows:

Peak Discharge cfs	Exceedance Interval years
11,000	25
27,000	50
49,000	100
97,000	SPF*

* SPF designates Standard Project Flood.

Steady-State Tidal Flows

25. Existing conditions and various improvement plans were tested using maximum steady-state ebb and flood tidal flows. The discharges were determined by multiplying the cross-sectional area of the inlet by the maximum current velocity. For a cross-sectional area of 1.98×10^4 sq ft and a velocity of 1.9 fps (Herron 1972), a flow of 37,620 cfs was calculated. A corresponding discharge of 0.38 cfs to be used in the model was determined by the scale relationship for model discharges of 1:100,000.

Analysis of Model Data

26. 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) comparisons of tracer patterns, (d) comparisons of resultant tracer deposits, 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.

PART IV: HARBOR TEST AND RESULTS

Description of Tests

Base tests

27. Prior to tests of various improvement plans, comprehensive tests were performed for five base test conditions in an effort to select the side slope revetment which most closely represented existing prototype conditions. Wave-height data were obtained for various stations within the entrance channel, Quivira Basin, and Mariners Basin for the test conditions listed in paragraph 23. Wave-induced current patterns and current magnitudes and tracer patterns also were secured for Base Tests 1 and 5 for representative waves from the three selected test directions. At the request of SPL, a timber pile breakwater, located at the entrance to Quivira Basin, was not included in any model tests. Since the breakwater was installed as a temporary measure only, it would not be included in future long-term solutions. Brief descriptions of the base tests are presented below; dimensional details are presented in Plates 1-5.

- a. Base Test 1 (Plate 1) consisted of existing conditions with all rock revetments reproduced.
- b. Base Test 2 (Plate 2) entailed the elements of Base Test 1 with the curved rubble-mound section of the middle jetty replaced with a concrete slope.
- c. Base Test 3 (Plate 3) entailed the elements of Base Test 2 with the revetment inside Quivira Basin removed.
- d. Base Test 4 (Plate 4) involved the elements of Base Test 3 with all the remaining revetment within the bay removed.
- e. Base Test 5 (Plate 5) involved the elements of Base Test 1 with the revetments within the bay replaced with a thin veneer of rock and the revetment within Quivira Basin removed.

Harbor improvement plans

28. Wave-height, current pattern and magnitude, and tracer and/or confetti tests were conducted for 30 plan variations. These variations consisted of changes in lengths and alignments of the breakwater structures, and changes in the breakwater cross section. Photographs of wave patterns, current 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 6-12 and in Table 27. Breakwater rock volumes are listed in Table 28.

a. Plan 1 (Plate 6) consisted of the elements of Base Test 5 with the removal of 220 ft from the end of the north jetty and the addition of a 2,200-ft-long rubble-mound breakwater with a crown elevation of +22.5 ft positioned 900 ft seaward of the harbor entrance.

b. Plan 1A (Plate 6) entailed the elements of Plan 1 with 100 ft of the breakwater removed from each end (total breakwater length 2,000 ft).

c. Plan 1B (Plate 6) entailed the elements of Plan 1A with an additional 100 ft removed from each end of the breakwater (total breakwater length 1,800 ft).

d. Plan 1C (Plate 6) consisted of the elements of Plan 1B with an additional 100 ft removed from each end of the breakwater (total breakwater length 1,600 ft).

e. Plan 1D (Plate 6) involved the elements of Plan 1C with an additional 100 ft removed from each end of the breakwater (total breakwater length 1,400 ft).

f. Plan 1E (Plate 6) entailed the elements of Plan 1D with an additional 100 ft removed from each end of the breakwater (total breakwater length 1,200 ft).

g. Plan 1F (Plate 6) involved the elements of Plan 1E with an additional 100 ft removed from each end of the breakwater (total breakwater length 1,000 ft).

h. Plan 1G (Plate 7) entailed the elements of Plan 1C with the crown elevation reduced to +20.0 ft.

i. Plan 1H (Plate 7) entailed the elements of Plan 1G with the crown elevation lowered to +15.0 ft.

j. Plan 1I (Plate 7) involved the elements of Plan 1H with the crown elevation raised to +17.5 ft.

k. Plan 1J (Plate 7) involved the elements of Plan 1I with the northern end of the breakwater lengthened 100 ft (total breakwater length, 1,700 ft).

l. Plan 1K (Plate 7) entailed the elements of Plan 1J with the northern end of the breakwater extended an additional 100 ft (total breakwater length 1,800 ft).

m. Plan 1L (Plate 7) entailed the elements of Plan 1K with the northern end of the breakwater extended an additional 100 ft (total breakwater length 1,900 ft).

n. Plan 1M (Plate 7) involved the elements of Plan 1L with the crown elevation raised to +22.5 ft.

o. Plan 1N (Plate 8) consisted of the elements of Plan 1M with the crown elevation of the middle and southern sections of the breakwater lowered to +17.5 ft.

p. Plan 1O (Plate 8) involved the elements of Plan 1N with the southern end of the breakwater lengthened 100 ft (total breakwater length 2,000 ft).

q. Plan 1P (Plate 8) consisted of the elements of Plan 1O with the southern end of the breakwater lengthened an additional 100 ft (total breakwater length 2,100 ft).

r. Plan 2 (Plate 9) involved the elements of Plan 1P with the offshore breakwater repositioned 375 ft shoreward of its original position. The south and middle sections of the breakwater were 450 and 900 ft long, respectively, with a crown elevation of +17.5 ft. The north section was 450 ft long with a crown elevation of +22.5 ft (total breakwater length 1,800 ft).

s. Plan 2A (Plate 9) entailed the elements of Plan 2 with the south end of the breakwater lengthened 100 ft (total breakwater length 1,900 ft).

t. Plan 2B (Plate 9) involved the elements of Plan 2A with the north end of the breakwater lengthened 100 ft (total breakwater length 2,000 ft).

u. Plan 2C (Plate 9) involved the elements of Plan 2B with the north end of the breakwater lengthened an additional 100 ft (total breakwater length 2,100 ft).

v. Plan 2D (Plate 9) entailed the elements of Plan 2C with the south end of the breakwater extended 100 ft (total breakwater length 2,200 ft).

w. Plan 3 (Plate 10) entailed the elements of Plan 2D with the core of the structure made impervious to an elevation of +7.5 ft and the crown elevation of the northern section of the breakwater lowered to +17.5 ft.

x. Plan 3A (Plate 10) involved the elements of Plan 3 with the northern section of the breakwater shortened 100 ft (total breakwater length 2,100 ft).

y. Plan 3B (Plate 10) involved the elements of Plan 3A with the northern section of the breakwater shortened an additional 100 ft (total breakwater length 2,000 ft).

z. Plan 3C (Plate 10) involved the elements of Plan 3B with the northern section of the breakwater shortened an additional 100 ft (total breakwater length 1,900 ft).

aa. Plan 3D (Plate 10) entailed the elements of Plan 3C with the northern section of the breakwater shortened an additional 100 ft (total breakwater length 1,800 ft).

bb. Plan 3E (Plate 11) entailed the elements of Plan 3C with the southern section of the breakwater shortened 100 ft (total breakwater length 1,800 ft).

cc. Plan 3F (Plate 11) involved the elements of Plan 3E with the southern section of the breakwater shortened an additional 100 ft (total breakwater length 1,700 ft).

dd. Plan 3G (Plate 11) involved the elements of Plan 3F with the southern section of the breakwater shortened an additional 100 ft (total breakwater length 1,600 ft).

ee. Plan 9 (Plate 12) involved a revised offshore breakwater cross section incorporated into the breakwater configuration of Plan 3G. The impervious core of the structure was removed and the front and back slopes steepened from 1V on 2H to 1V on 1.5H and 1V on 1.5H to 1V on 1.25H, respectively.

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

Harbor wave-height tests

29. Wave-height tests for base test conditions and various improvement plans were conducted using test waves from one or more of the test directions listed in paragraph 23. As an expedient, tests involving certain proposed improvement plans were limited to one or two critical directions of approach. After the development of a promising plan, wave-height tests then were conducted from the remaining directions of approach to assure that the specified wave-height criteria were met for all wave conditions. The wave gage locations for base tests and each improvement plan are shown in the referenced plates.

Long-period wave tests

30. Long-period (30 to 140 sec) wave tests were conducted for Base Test 5 and the best breakwater plan (with respect to short-period wave protection) using waves from the west test direction. The two types of tests involved with investigating long-period waves are as follows:

a. Frequency response tests involved the placement of wave sensors at strategic locations throughout the harbor to measure the amplitude of the oscillations (Plates 13-15). An array of 12 wave gages at the harbor entrance was used to determine the amplitude of incident waves. By plotting the ratio of the measured wave height at each gage to the incident wave height (response factor) versus the wave periods tested, frequency response curves showing resonant peaks were obtained.

b. Surface-float tests were conducted using small white squares of of Styrofoam "confetti" and time-lapse photography to determine oscillation patterns. The confetti was spread over the surface of the channel and basins and subsequent movement by each wave period was photographed by a series of overhead cameras with shutter openings equal to the wave period being tested. The resulting mosaics (Appendix B) show the oscillation patterns and location of nodes and antinodes.

Harbor current pattern and magnitude tests

31. Wave-induced current patterns and magnitudes were determined at selected locations by timing the progress of a dye solution relative to a known distance on the model surface. These tests were conducted for base tests and various improvement plans using the same test directions and waves as for wave-height tests.

Harbor tracer tests

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

Test Results

33. 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, current pattern and magnitudes, and measured frequency response of the two basins. From this evaluation, the best improvement plans were selected.

Base Tests

Base Test 1

34. Wave-height tests for Base Test 1 were conducted using 14 wave gages arranged as shown in Plate 1. Results for waves from the southwest, west, and northwest deepwater directions at mllw and mhhw are compiled in Tables 3 and 4, respectively, and show large heights at the bay entrance (gage 1). Gage 1 is situated at approximately the -25 ft contour which allows a maximum nonbreaking wave height of 19.5 ft (mllw) or 23.7 ft (mhhw) using the generally accepted criterion of $H_b = 0.78 d_b$. Values that exceed these limits indicate waves which are peaking and breaking directly on the wave gage. Wave heights in the navigation channel (gage 2) are much less severe due to dissipation of wave energy by the middle and north jetties. The maximum wave height recorded in the bend of the navigation channel (gage 3) was 3.7 ft resulting from an

11-sec, 11-ft wave from the southwest deepwater direction at mhhw. The same wave produced maximum wave heights in the entrance to and within Quivira Basin of 1.6 ft (gage 4) and 1.0 ft (gage 7), respectively. The maximum wave height in the entrance to Mariners Basin (gage 12) was 1.0 ft for the 19-sec, 6-ft wave from the northwest test direction at mhhw while none of the wave heights within Mariners Basin exceeded 0.7 ft. Wave-height tests for waves from the west deepwater direction showed wave height at the entrance in excess of 20.0 ft. Observations showed that wave energy that entered Quivira Basin resulted primarily from waves which had diffracted around the curved section of the north jetty. Conversely, wave energy in Mariners Basin was due primarily to the reflection of wave energy from the curved section of the middle jetty.

35. Current patterns and magnitudes for Base Test 1 (typical example shown in Photo 1) using waves from the northwest deepwater direction showed the formation of strong longshore currents (as high as 5 fps) north of the north jetty and curving seaward toward the end of the north jetty. For large waves, these currents moved across the entrance channel and to the south. Generally, when combined with waves breaking along the middle jetty, this current produced a counterclockwise eddy in the entrance. Maximum velocities were about 3.3 fps. A counterclockwise eddy also was formed in the lee of the middle jetty with maximum velocities also about 3.3 fps. Tests conducted using waves from the southwest deepwater direction (typical example shown in Photo 2) showed strong northerly longshore currents (as high as 5 fps) moving seaward past the end of the middle jetty. These currents tended to dissipate seaward of the middle jetty rather than move across the entrance. Currents in the entrance tended to flow seaward for this condition with little or no eddying. Longshore currents in the north side of the entrance generally curved from the end of the north jetty to the north along Mission Beach. A clockwise eddy usually was formed in the lee of the north jetty with velocities as high as 5 fps. Currents in Quivira and Mariners Basins were too small to be accurately measured (<0.1 fps). Current pattern and magnitude tests for waves from the west deepwater direction showed a general clockwise eddy north of the north jetty with maximum velocities of about 3 fps. Northerly longshore currents up to 4 fps were observed north of the eddy. A counterclockwise eddy was located south of the middle jetty with maximum velocities of about 4 fps. A weak clockwise eddy also was observed between the south jetty and the south groin. Currents south of the south groin were

generally confused. At the bay entrance, currents formed along the outside of the north and middle jetties and flowed seaward past the ends of the jetties. Velocities were generally less than 3 fps. Current velocities in the basins were too small to be accurately measured (<0.1 fps).

36. Tracer tests for Base Test 1, conducted using waves from the northwest deepwater direction (typical example shown in Photo 3), showed a strong movement of tracer south along Mission Beach and out toward the end of the north jetty with little or no movement of tracer into the entrance. An eddy frequently formed at the shoreward terminus of the north jetty. Tracer material placed south of the middle jetty moved partially to the south and partially into a counterclockwise eddy in the lee of the middle jetty. For waves from the southwest deepwater direction (typical example shown in Photo 4), tracer tests showed movement of tracer into the flood-control channel and out to the end of the middle jetty. No tracer entered the navigation channel. Tracer placed north of the entrance moved into a clockwise eddy or north along Mission Beach. For waves from the west deepwater direction, tracer tests showed a clockwise eddy trapping tracer north of the north jetty. North of this eddy, longshore currents carried tracer to the north. A counterclockwise eddy formed south of the middle jetty. Tracer placed south of this eddy moved to the south past the south groin. Tracer moving seaward along the outside of the north and middle jetties was pushed back into the eddies allowing no tracer past the ends of the jetties.

Base Test 2

37. Wave heights recorded in Quivira and Mariners Basins were relatively low for Base Test 1 when compared with the subjective estimates of wave heights by the harbor patrol (i.e., it had been estimated that wave heights were sometimes as large as several feet). Also, since smaller wave heights make comparisons of the effectiveness of various improvement plans more difficult, it was desirable that wave heights be as large as possible. Therefore, Base Test 1 was modified in an effort to increase wave heights in the basins. Base Test 2 entailed the elements of Base Test 1 with the curved rubble-mound section of the middle jetty replaced with a concrete slope (Plate 2). This had little effect on the wave heights within Quivira Basin; however, wave heights in Mariners Basin increased (Table 5). It was observed that the concrete slope reflected more wave energy to the entrance of Mariners Basin.

Base Test 3

38. For Base Test 3, the revetment within Quivira Basin also was removed (Plate 3). This resulted in increased wave heights in Quivira Basin (Table 5) due to decreased absorption of wave energy within the basin.

Base Test 4

39. For Base Test 4, all revetments within the bay were removed (Plate 4). Results of wave-height tests for this configuration (Table 5) showed markedly increased wave heights in the basins due to decreased absorption of wave energy within the bay.

Base Test 5

40. Base Test 5 entailed the elements of Base Test 1 with the revetments inside the bay replaced with a thin veneer of rock and the revetment within Quivira Basin removed (Plate 5). This condition was felt to be more representative of the true conditions in the bay. It minimized absorption of wave energy while allowing surface friction to prevent the formation of excessive wave-induced currents along the smooth concrete slopes. Wave-height tests, for waves from the west deepwater direction (Table 6), showed maximum wave heights for Quivira Basin of 1.7 ft in the basin entrance and 1.3 ft inside the basin. For Mariners Basin, maximum wave heights were 1.1 ft in the basin entrance and 1.0 ft inside the basin. Wave-height tests using maximum steady-state ebb and flood tidal flows at the midtide level of +2.7 ft (Table 7) showed a general increase in wave heights in the navigation entrance for maximum ebb and a decrease in wave heights in the navigation entrance for maximum flood when compared with tests run at mhhw with no flow. Observations indicated that ebb currents opposed the incoming waves and forced them to peak. For the flood flow, incoming waves were accelerated, increasing the wavelength and thereby reducing the wave height. Wave heights in Quivira Basin decreased for maximum ebb and increased for maximum flood when compared with tests run at mhhw with no flow. This may be attributed to the fact that waves break in the entrance sooner for an ebb flow and later for a flood flow than for a slack-water condition. Wave heights in Mariners Basin demonstrated the same tendency.

41. Wave-induced current patterns and magnitudes for Base Test 5 using maximum steady-state ebb and flood tidal flows showed little change with respect to a "no-flow" condition, other than in the navigation channel. Current velocities in the navigation channel averaged about 2 fps. For an ebb

flow, currents in the navigation channel flowed seaward. Littoral currents on either side of the north and middle jetties remained essentially the same. For a flood flow, currents entered the navigation channel from the seaward end causing some increase in littoral current velocities at the heads of the north and middle jetties. Typical examples of wave and current patterns and magnitudes for Base Test 5 without tidal flows are presented in Photos 5 and 6.

42. Tracer tests for Base Test 5 using maximum steady-state ebb and flood tidal flows showed virtually no change from tests at slack water. Typical examples are shown in Photos 7-10.

Improvement Plans

Plan 1

43. Wave-height tests were conducted for Plan 1 (2,200-ft-long breakwater) using waves from the west deepwater direction and the results are presented in Table 8. In order to more effectively determine the entrance conditions for this plan, gages 11 and 14 were repositioned in the middle of the south and north entrances, respectively. Wave heights in the entrance channel, Quivira Basin, and Mariners Basin were substantially reduced when compared with Base Test 5. Maximum wave heights at gage 1 in the entrance channel were reduced from 22.9 to 3.9 ft, maximum wave heights in Quivira Basin were reduced from 1.4 to 0.4 ft, and maximum wave heights in Mariners Basin were reduced from 1.1 to 0.3 ft.

44. Current patterns and magnitudes for Plan 1 were obtained for no-flow conditions at mhhw and mllw and for maximum ebb and flood tidal flows at midtide level (+2.7 ft). Waves were from the west deepwater direction. Except in the immediate entrance area, current patterns for the two slack-water levels were similar to those for Base Test 5. However, south to north longshore currents deflected seaward along the south side of the middle jetty were "funneled" or deflected across the navigation channel by the offshore breakwater. Maximum current velocities reached 10 fps. A strong clockwise eddy was noted north of the north jetty. The currents moving north across the navigation channel contributed to this circulation. The maximum ebb condition forced the currents south of the middle jetty to eddy in a counterclockwise direction. No currents were observed moving across the navigation channel. Currents for the maximum flood condition seemed to enter the bay

from the south with little change in northern eddy. Maximum current velocities in the south entrance approached 10 fps. Typical wave and current patterns and current magnitudes for Plan 1 are presented in Photos 11 and 12.

45. Tracer tests for Plan 1 (Photos 13-16) showed a counterclockwise eddy south of the middle jetty and a clockwise eddy north of the north jetty with no tracer moving into the entrance for any of the waves tested.

46. As an expedient, a number of plans after Plan 1 were tested only for extreme storm conditions occurring from one direction with a swl of +5.4 ft (mhhw). When a promising plan was found, it was tested from all directions.

Plans 1A-1F

47. In optimizing the length of the offshore breakwater of Plan 1, each end of the breakwater was shortened in increments of 100 ft. Plans 1A, 1B, 1C, 1D, 1E, and 1F correspond to total breakwater lengths of 2,000, 1,800, 1,600, 1,400, 1,200, and 1,000 ft. All of these plans were tested using 11- to 15-ft waves of various periods from the west deepwater direction at mhhw Results of wave-height tests for Plans 1-1F are shown in Table 9. An examination of these data reveals that Plan 1C eliminates the greatest length of structure without causing a significant increase in wave heights.

Plans 1G-1I

48. In optimizing the height of the offshore breakwater, the crown elevation of the Plan 1C breakwater was lowered in 2.5-ft increments. Plans 1G, 1H, and 1I correspond to crown elevations of +20.0, +15.0, and +17.5 ft. These plans were tested under the same conditions as described in the preceding paragraph. Results of wave-height tests for Plains 1G-1I also are shown in Table 9. An examination of these data showed, in general, an increase in entrance and basin wave heights as the height of the breakwater is decreased. In telephone conversations discussing test results, SPL supplied tentative wave-height criteria of 3 to 4 ft in the entrance and navigation channel and 1.5 ft in Quivira and Mariners Basins. Since none of the wave heights in the two basins approached 1.5 ft, selection of the optimum plan was based on its effectiveness in reducing entrance wave conditions. On this basis, Plan 1I was tentatively selected to represent the optimum plan.

49. Plan 1I was tested using all waves at mhhw for the three test directions--northwest, west, and southwest; results of these tests are shown in Table 10. It was observed that waves from the west and southwest

directions presented no drastic problems. However, for waves from the northwest, wave heights at gage 1 were excessive (up to 17.8 ft).

Plans 1J-1L

50. The northern end of the breakwater was lengthened in increments of 100 ft (Plans 1J-1L) and tested using waves from the northwest deepwater direction. Plans 1J, 1K, and 1L correspond to breakwater lengths of 1,700, 1,800, and 1,900 ft. Results of wave-height tests are shown in Table 11. An analysis of the data showed decreasing entrance wave heights with increasing breakwater length. However, wave heights in the basins were observed to increase with increased breakwater length. This may be due to reduced interference between the waves transmitted through the breakwater and waves diffracting around the ends of the breakwater. In any case, wave heights in the basins did not exceed the 1.5-ft criterion.

Plans 1I-1L

51. Plans 1I-1L were retested using waves somewhat smaller than those tested to date to more closely represent normal to moderate wave conditions. Test waves were from the southwest and northwest deepwater directions at mhhw. Test results (Table 12) showed a decrease in entrance wave heights as the breakwater length increased. Again, wave heights in the basins appear to increase with increasing breakwater length. In any case, wave heights in the basins did not exceed 0.5 ft. Using the tentative wave criteria supplied by SPL of 3 to 4 ft in the entrance and navigation channel and 1.5 in the basins, Plan 1L was selected as providing the optimum entrance and navigation channel wave conditions.

Plan 1M

52. The crown elevation of Plan 1L was raised to +22.5 ft (Plan 1M) to determine its effect on entrance and navigation channel wave conditions. All of the waves from the northwest deepwater direction were tested (Table 13). Test results showed a significant reduction in entrance and navigation channel wave heights. Plan 1M then was tested using waves from the southwest deepwater direction and wave heights in the entrance and navigation channel met the specified criteria except for one test wave.

Plans 1N-1P

53. From the previous tests, it seemed clear that more protection was required for waves from the northwest than for waves from the southwest. Therefore, the crown elevation of the north breakwater section was left at

+22.5 ft and the south and middle sections of the Plan 1M breakwater were lowered to +17.5 ft (Plan 1N) in an effort to save rock. For comparison, 100-ft increments were added to the south end of the breakwater resulting in total breakwater lengths for Plans 1N-1P of 1,900, 2,000, and 2,100 ft. These plans were tested using waves from the southwest deepwater direction and results are shown in Table 14. Wave heights were within the criteria for Plan 1P except for two instances where 5-ft waves were recorded in the entrance channel. Plan 1P was then tested using waves from the northwest deepwater direction. Averages for all waves for gages 1, 2, and 3 showed values of 3.9, 2.3, and 0.9 ft, but the maximum wave measured was 11 ft.

54. In the previous tests, evaluation of plans was based primarily on the averages of wave heights in the entrance and navigation channels and basins. Some individual wave heights at certain gages were in excess of the criteria, but these may be tolerated due to their infrequent occurrence.

55. At this point, information was received from SPL that contained revised wave-height criteria. These criteria stated that wave heights in Quivira Basin should not exceed 1.0 ft for any wave condition and that wave heights in the lee of the offshore breakwater should not exceed 1.5 ft for deepwater waves of 6 ft or less.

56. Wave-height tests were conducted for Plan 1P using 6-ft waves of different periods and directions. Test results (Table 15) indicated wave heights in excess of the 1.5-ft criterion at gage 1 for the 17-sec waves from northwest and for the 15-, 17-, and 19-sec waves from southwest.

Plan 2

57. In an effort to reduce the volume of rock required for constructing the offshore breakwater and at the same time improve wave protection, the offshore breakwater was moved 375 ft shoreward into shallower water. Both the north and south entrances were made 450 ft wide. This plan was designated Plan 2 (total breakwater length 1,800 ft) and was tested using all waves from the southwest deepwater direction. Results (Table 16) showed wave heights much less than 1.0 ft in Quivira Basin. However, for the 6.0-ft incident waves, the 13-, 17-, and 19-sec periods exceeded the criterion at gage 1.

Plan 2A

58. In an attempt to reduce wave heights at gage 1, the south section of the breakwater was extended 100 ft (Plan 2A) and tested using the three waves that exceeded the criterion for Plan 2. Test results showed, in general,

only a slight reduction in wave heights (Table 16). Plan 2A then was tested using all waves from the northwest deepwater direction. Test results (Table 17) showed no wave heights greater than 0.5 ft in Quivira Basin. However, most of the 6.0-ft test waves generated wave heights in excess of 1.5 ft at gage 1.

Plan 2B

59. The north section of the breakwater was lengthened 100 ft (Plan 2B) to a total breakwater length of 2,000 ft and tested using the three most critical waves from the northwest. Test results (Table 17) showed a marked reduction in wave heights recorded at gage 1; however, they still exceeded the 1.5-ft criterion.

Plan 2C

60. An additional 100 ft was added to the north end of the breakwater (Plan 2C) for a total breakwater length of 2,100 ft and tested for the three most critical waves from the northwest. Test results (Table 17) showed that all wave heights recorded at gage 1 were within the 1.5-ft criterion.

Plan 2D

61. At this point, the south end of the breakwater was extended 100 ft (Plan 2D) for a total breakwater length of 2,200 ft and tested using the most critical waves from the southwest. Test results (Table 16) showed that the 17- and 19-sec waves still exceeded the criterion.

62. An analysis of the Plan 2 series revealed that for waves from the southwest, there was relatively little reduction in wave height with increasing breakwater length. This indicated that more wave energy was being transmitted over and/or through the breakwater than around it and that raising the crown elevation and/or sealing the structure to make it impervious might reduce wave energy more effectively than lengthening the structure. Indications were that the raised northern section of the breakwater (crown el +22.5 ft) was allowing little wave energy to be transmitted over and/or through it. On the other hand, most of the wave energy recorded at gage 1 from northwest waves appeared to have diffracted around the north end of the breakwater and lengthening the breakwater reduced significantly wave energy entering the harbor.

Plan 3

63. In an effort to reduce the transmission of wave energy through the voids of the breakwater, the core stone of the Plan 2D structure was made

impervious to an elevation of +7.5 ft. To save rock, the crown elevation of the north section of the breakwater was lowered to +17.5 ft. This plan (Plan 3) was tested using the entire range of test waves from the southwest and northwest deepwater directions and test results are shown in Table 18. Wave heights exceeded the 1.5-ft criterion at gage 1 (for 6-ft incident waves) only once (i.e., a 1.6-ft wave height was recorded for the 17-sec, 6-ft test wave from the northwest deepwater direction). This is only slightly over the criterion and it would seem reasonable to consider this an acceptable condition.

Plan 3A

64. To determine if the structure could be shortened without significantly increasing entrance wave conditions, the north end of the breakwater was reduced 100 ft (Plan 3A, total length 2,100 ft) and tested using the four most critical 6-ft incident waves from the northwest (as determined from the previous test). Test results (Table 19) showed that wave heights were reduced to within the criterion with a maximum wave height of 1.4 ft. This slight reduction is most likely due to interference of wave energy passing around the ends of the breakwater with energy passing through the voids of the armor stone; but a change this small also could be attributed to experimental uncertainty.

Plans 3B-3D

65. The north end of the breakwater was then shortened in 100-ft increments (Plans 3B, 3C, and 3D) until the criterion was exceeded significantly (Table 19). From these data, it was concluded that Plan 3C (total breakwater length of 1,900 ft) was the optimum with respect to waves from the northwest.

Plans 3E-3G

66. Attention was then directed to the southern end of the breakwater and waves approaching from the southwest. The three most critical 6-ft waves tested for Plan 3 were used as test waves. The south end of the Plan 3C breakwater first was shortened 100 ft (total breakwater length 1,800 ft) and designated Plan 3E. Test results (Table 19) showed that wave heights at gage 1 were well within the criterion. Plan 3F involved shortening the south end of the breakwater an additional 100 ft (total breakwater length 1,700 ft). Test results (Table 19) showed that wave heights at gage 1 were still within the 1.5-ft criterion. Plan 3G involved shortening the south end of the breakwater an additional 100 ft (total breakwater length 1,600 ft). Test results

(Table 19) showed that wave heights at gage 1 were slightly in excess (1.7 ft) of the 1.5-ft criterion. From the above data, it was concluded that Plan 3F should be the optimum plan with respect to waves approaching from the southwest direction.

67. Plan 3F was tested using the entire range of test waves from the southwest, west, and northwest test directions. Test results (Tables 20 and 21) showed that the 1.5-ft criterion at gage 1 was met for all 6-ft incident waves. Also, for all waves tested, the 1.0-ft criterion for Quivira Basin was not exceeded.

68. Results of wave-height tests for Plans 3F and 3G, using waves from the southwest deepwater direction, were discussed with SPL personnel; and it was decided that the savings of 100 ft of breakwater (Plan 3G) outweighed the disadvantage of slightly exceeding (0.2 ft for one test wave) the 1.5-ft criterion. As a result, Plan 3G was also tested using waves from the west deepwater direction. Test results (Table 22) indicated that for the 6-ft incident waves, the maximum wave height recorded at gage 1 was 1.5 ft for a 17-sec, 6-ft wave at mhhw. The slight difference in maximum wave height (1.5 ft versus 1.7 ft) between this and the previous test is attributed to experimental error. Wave heights in Quivira Basin did not exceed 0.4 ft.

69. In view of the data discussed above and the reduced volume of rock (compared with Plan 3F) required for construction, Plan 3G was considered to be the optimum plan tested to date. Wave-height tests were performed with maximum steady-state ebb and flood tidal flows at midtide level using selected test waves shown in Table 22. For 6.0-ft incident waves, test results showed a maximum wave height at gage 1 of 1.2 ft for maximum flood and 0.9 ft for maximum ebb flows. Wave heights in Quivira Basin did not exceed 0.2 ft. For waves from the southwest test direction at mhhw (Table 23), test results showed that wave heights at gage 1 (for 6.0-ft incident waves) exceeded the 1.5-ft criterion only once. The 19.0-sec, 6.0-ft wave at mhhw produced a 1.7-ft wave. This small increase in desired wave height in the entrance with the low frequency of occurrence of this incident wave (about 4 hr/year) should create no problems. Wave heights in Quivira Basin did not exceed 0.2 ft.

70. Current pattern and magnitude and tracer tests were conducted for Plan 3G (Photos 17-30) for the northwest, west, and southwest deepwater directions; tests were made at mhhw and mllw. For waves from the west deepwater direction, additional tracer tests were performed with maximum steady-state

ebb and flood tidal flows. For waves from the northwest (Photos 23 and 25), longshore currents moved south to the north jetty where they split upon reaching the north entrance. One component moved seaward past the north end of the breakwater and back to the north to form a large eddy. The other component moved into the north entrance and exited through the south entrance. Current velocities reached 4.0 fps in the north entrance for the larger storm waves. For 6.0-ft incident waves, maximum current velocities were 1.9 fps in the north entrance and 2.0 fps in the south entrance. For all waves, currents south of the middle jetty formed a counterclockwise eddy. Currents in the south entrance reached 3.3 fps. For waves from the southwest deepwater direction (Photos 27 and 29), a counterclockwise eddy was formed south of the middle jetty. Currents entering the south entrance reached 5.0 fps for the larger storm waves but did not exceed 2.2 fps for the 6.0-ft incident waves. Currents north of the north jetty formed a clockwise eddy and maximum velocities of 4.0 fps were observed in the north entrance. Current patterns for waves from the west deepwater direction (Photos 17 and 21) showed the formation of a counterclockwise eddy south of the middle jetty and a clockwise eddy north of the north jetty for all wave, swl, and tidal flow conditions tested. Maximum current velocities observed in the entrances were 2.5 fps in the south entrance and 3.3 fps in the north entrance.

71. Tracer tests were conducted for Plan 3G for the same test conditions listed in the preceding paragraph. Each tracer test was run for about 15 min (2.5 hr prototype). Tracer tests using waves from the northwest deepwater direction (Photos 24 and 26) showed tracer moving south in the surf zone pushed shoreward by wave forces as it approached the north jetty. It was observed that tracer material actually moved opposite to the current flow in some cases. This was verified by simultaneous injection of dye and tracer into the model. Apparently, at some points, bed-load movement is opposite the movement of surface currents. Waves diffracting around the south end of the breakwater set up a counterclockwise eddy south of the middle jetty. Because of the sheltering effect of the breakwater and jetties, movement of tracer was slow. For waves from the southwest (Photos 28 and 30), movement of tracer north of the north jetty was very slow. Tracer tended to collect in a clockwise eddy. South of the middle jetty, tracer in the surf zone moved north to the middle jetty, then seaward along the jetty. Upon nearing the end of the jetty, the tracer curved south and formed a counterclockwise eddy. For waves

from the west direction (Photos 18-20 and 22), tracer material collected in two eddies--a clockwise eddy north of the north jetty and a counterclockwise eddy south of the middle jetty. This occurred for all wave, swl, and tidal flow conditions tested. Of all the waves and directions tested, only one wave moved any tracer into the harbor entrance. The 9.0-sec, 13.0-ft wave from the northwest deepwater direction at mllw (Photo 26) allowed a small amount of some of the finer particles of the coal into the north entrance. Since this condition occurs on the average of only 2 hr/year, this should not be a problem.

72. In order to provide more conservative data on the effect of the best breakwater plan (Plan 3G) on inner harbor wave heights, all the rock revetment within the bay was removed and tests were conducted using waves from the northwest, west, and southwest deepwater directions. Results of these tests at a swl of +5.4 ft are shown in Table 24. Wave heights within Quivira Basin increased slightly relative to the plan tested with the interior revetment; however, none of the test waves produced wave heights greater than 1.0 ft.

73. At the request of SPL, Plan 3G (with no rock revetment in the bay) was tested with a swl of +7.6 ft. This represents a 2.2-ft storm surge superimposed on an astronomical tide level of +5.4 ft (mhhw). This extreme condition was tested using waves from the northwest, west, and southwest deepwater directions and results are shown in Table 25. The higher water level tended to allow more wave energy to pass over and through the breakwater, particularly for the large waves. However, wave heights recorded at gage 1 for 6-ft deepwater waves exceeded the 1.5-ft criterion only three times. Wave heights within Quivira Basin were still within the 1.0-ft criterion. These tests indicate that the Plan 3G offshore breakwater provides protection to the inner basins even under the most extreme conditions.

74. Long-term tracer tests were conducted for Plan 3G using waves from the northwest deepwater direction. Tracer material was injected into the surf zone north of the north jetty and 9-sec, 3-ft waves at mhhw were used to build an initial beach face. To prevent model circulation effects, each continuous test run was limited to about 30 min model time. As the waves continued to run, the tracer "beach" grew toward the south. The 9-sec, 3-ft wave was run a total of 4 hr model time (40 hr prototype), but movement of tracer was slow. To increase the rate of tracer movement, wave heights and periods were varied and observations were made of the rate of beach growth. The 11-sec, 6-ft wave

was found to accrete tracer more readily than larger or smaller waves. As the beach grew south toward the north jetty, the rate of accumulation began to decrease. Waves breaking along the north jetty combined with waves reflected off the north jetty to impede the progress of tracer. This increased the time required for testing, but the tracer eventually built a fillet against the north jetty as shown in Photo 31. The total amount of time required for various waves to build this fillet was 33 hr model time (330 hr prototype)

75. A long-term tracer test of Plan 3G using the 9-sec, 13-ft waves from the northwest at mllw was conducted with the fillet shown in Photo 31 as the beginning condition in the model. These were the only test conditions (observed from previous tests) that moved tracer into the harbor entrance. Test results after 11 hr model time (110 hr prototype) are shown in Photo 32. It was observed that most of the tracer in the surf zone collected in a counterclockwise eddy north of the north jetty. Tracer accumulated in this eddy until the water depth decreased to the point that wave forces exceeded the current forces and tracer migrated shoreward. Some tracer, however, did reach the end of the north jetty. A rip current moved tracer along the north jetty to the jetty head where currents, combined with waves diffracted around the end of the offshore breakwater, moved material into the entrance. The quantity of tracer in the entrance was measured and amounted to about one percent of the total tracer introduced into the model. While these tests are of a qualitative nature and no actual quantities can be determined, they provide some indication of the relative magnitude of accumulation.

76. A long-term tracer test also was conducted using 13-sec, 15-ft waves at mllw and test results after 8 hr model time (80 hr prototype) are shown in Photo 33. As tracer moved toward the north jetty, breaking waves along the side and north of the north jetty forced tracer shoreward and into a counterclockwise eddy. As tracer accumulated in this eddy and depths became shallower, wave forces eventually exceeded the current forces and tracer was pushed shoreward.

77. Long-term tracer tests were conducted for Plan 3G using waves from the southwest direction. This was primarily in an effort to build a sand plug across the San Diego River mouth and will be discussed in more detail in PART V. Following the formation of the sand plug using a 9-sec, 6-ft wave, Photo 34 was taken showing a very small amount of fine coal dust in the south entrance. A more detailed view of the plug is shown in Photo 35. During a

test which used this deposit as the starting condition and a 13-sec, 11-ft wave, some of the finer dust present in the coal tracer also moved into the south entrance; but the quantity was very small relative to the amount of tracer fed into the system (Photo 36).

Plan 9

78. Following transmittal of the original draft of this report, SPL requested additional tests for a revised offshore breakwater cross section. This plan, designated Plan 9, is described in paragraph 28ee. Short-period wave-height tests were conducted using waves from the west deepwater direction at mhhw only. Test results (Table 26) showed an increase in wave height at gage 1 when compared with Plan 3G. The maximum wave height for Plan 9 for the 6-ft deepwater wave was 2.0 ft for the 17-sec wave. This exceeds the 1.5-ft criterion by 0.5 ft but may be acceptable, considering the infrequent occurrence of this 17-sec wave.

Long-Period Wave Tests

79. Long-period (30 to 140 sec) wave tests were conducted for existing conditions, Plan 3G, and Plan 9 using waves from the west deepwater direction at mhhw. The gage arrangements for these tests are shown in Plates 13-15. To ensure an accurate determination of incident wave heights, at the harbor entrance for existing conditions, the first 12 gages were placed in an array to measure the nodes and antinodes of possible standing waves in the entrance channel. The incident wave height was then calculated from the following relationship:

$$H_i = \frac{H_a + H_n}{2}$$

where

H_i = incident wave height

H_a = wave height at antinode

H_n = wave height at node

The gage array was used to determine incident wave heights in the entrance channel and corresponding wave machine settings. Following tests of existing conditions, the gage array was removed and Plan 3G was tested under the same wave conditions as existing conditions. The placement and numbering of the remaining gages were the same. Measured wave heights at a particular gage location were divided by the incident wave height for that wave period to obtain the response factor, $R = H/H_i$. Frequency response (response factor versus wave period) curves for gages 13-26 for existing conditions, Plan 3G, and Plan 9 are presented in Plates 16-29.

Existing conditions

80. Test results for existing conditions indicate:

a. Channel gages 13 and 14 exhibit definite standing wave characteristics (Plates 16 and 17). The maximum response was 1.42 for gage 13 at a 105-sec period. Periods that exhibited peaks common to both gages were 30, 72, 95, and 105 sec.

b. Quivira Basin gages 15-22 exhibited sharp peaks for several periods (Plates 18-25). The maximum response was 2.22 at gage 16 for the 86-sec period. Periods that produced peaks for most Quivira Basin gages were 37, 50, 62, 68, 76, 86, 100, 122, and 134 sec. The plots for gages 16 and 22 are very similar, suggesting an oscillation between the northwest corner and the southeast corner of Quivira Basin. The plots of gages 18 and 21 are also similar, suggesting an oscillation between the northeast corner and the southwest corner of Quivira Basin.

c. Mariners Basin gages 23-26 showed major peaks at periods of 76 and 88 sec (Plates 26-29). The maximum response was 3.53 at gage 24 for the 88-sec period. This may be due to a coupling of oscillations between Quivira Basin (where a strong 86-sec response was observed) and Mariners Basin.

Plan 3G

81. Test results for Plan 3G indicate:

a. The effect of the offshore breakwater on channel gages 13 and 14 was a reduction of channel response by 50 percent or more (Plates 16 and 17).

b. Peak responses in Quivira Basin (gages 15-22) were reduced by over 50 percent (Plates 18-25) in almost all cases. Some peaks shifted slightly but, in most cases, occurred at the same periods. It also was observed that the widths of the peaks were significantly reduced. This reduces the frequency of occurrence of waves that might cause significant harbor oscillations.

c. Peak responses in Mariners Basin (gages 23-26) were, in most cases, reduced by about 50 percent or more (Plates 26-29). In most cases, peaks occurred at or very near the same period, and widths of peak responses were significantly reduced.

82. Squares of Styrofoam "confetti" were spread on the water surface, and time-lapse photographs were taken for selected wave periods for existing conditions and Plan 3G using eight overhead cameras. Resulting photographs

were assembled in mosaics and are presented in Appendix B. Areas of maximum horizontal movement (nodes) and minimum horizontal movement (antinodes) and the resulting oscillation patterns are shown in the photographs. A comparison of these mosaics with the frequency response curves shows, in general, good correlation between the positions of nodes and antinodes for each gage for the selected wave periods. Also, when comparing existing conditions with Plan 3G, the magnitude of the horizontal displacement at the nodes is markedly reduced for Plan 3G.

Plan 9

83. Test results for Plan 9 indicate:

a. The effect of the Plan 9 offshore breakwater on channel gages 13 and 14 was very similar to that of Plan 3G. Responses for incident wave periods below 80 sec were slightly higher than those for Plan 3G while responses for incident wave periods above 80 sec were slightly lower than those for Plan 3G (Plates 16 and 17).

b. Peak responses in Quivira Basin (gages 15-22) were generally reduced when compared with Plan 3G (Plates 18-25). Some peaks were eliminated (in particular, the peak occurring at 86 sec) while some peaks shifted slightly.

c. Peak responses in Mariners Basin (gages 23-26) were generally reduced when compared with Plan 3G (Plates 26-29). As in Quivira Basin, some peaks were eliminated and some peaks shifted slightly.

Discussion

Harbor test results

84. A comparison of Base Tests 1-4 showed increased wave energy in the basins with decreasing revetment in the bay. Base Test 5 used a thin veneer of rock for portions of the bay revetment to allow for a more realistic condition. Wave heights, current patterns and magnitudes, and tracer patterns in the bay entrance and seaward were not affected by these changes. For waves from the three test directions, the general current movements showed a clockwise eddy north of the north jetty and a counterclockwise eddy south of the middle jetty. In general, tracer material followed the same pattern. No significant shoaling of the harbor entrance was observed.

85. Attempts to optimize the length and crown elevation of the breakwater in its original location revealed that a very massive and expensive

structure would be required to meet the specified wave-height criteria. Moving the structure 375 ft shoreward (Plan 2 series) provided better overlap with the north and middle jetties requiring less breakwater length. In addition, the breakwater was located in shallower depths, reducing the height of the structure about 5 ft.

86. The revised maximum wave-height criterion of 1.5 ft in the entrance channel for 6-ft incident waves was difficult to achieve due to transmission of wave energy through the structure. Discussions with SPL personnel revealed that the elevation of the core stone could be raised to +7.5 ft (effectively sealing the structure to that elevation) without adversely affecting prototype construction techniques. The model breakwater, therefore, was redesigned and constructed to this new specification (Plan 3 series). Tests to optimize the length of the Plan 3 breakwater resulted in recommended lengths of 350 ft for both the north and south wings (total structure length = 1,600 ft at a crest elevation of +17.5 ft). Maximum wave heights in the entrance channel for 6-ft incident waves were 1.7 ft. In discussions with SPL, this was considered close enough to the 1.5-ft criterion to be acceptable. When Plan 3G was tested with all revetment within the bay removed and an extreme swl of +7.6 ft, results showed that wave heights slightly exceeded the channel criterion but were within the basin criterion (i.e., less than 1.0 ft). Of the improvement plans tested, Plan 3G appears to be the most effective alternative with respect to short-period wave-height reduction.

87. Tracer tests for Plan 3G showed that material generally moved into eddies north and south of the harbor entrance. Only one wave from the northwest moved a small amount of tracer into the north entrance. Since this was a very small percentage of the tracer introduced into the model and was for a wave with a very low frequency of occurrence, this was not considered a problem.

88. Wave-height tests for the revised breakwater of Plan 9 showed slightly larger wave heights in the lee of the breakwater than those for Plan 3G. This is due to a more porous structure that allows increased transmission of wave energy through the structure and a steeper frontal slope that allows more overtopping. While Plan 9 is less effective than Plan 3G in reducing short-period wave heights in the entrance (maximum of 2.0 ft as compared with 1.5 ft), it would be considerably cheaper to build. The location of the Plan 3G and Plan 9 breakwaters is shown in Plate 72.

89. A comparison of long-period test results for Base Test 5 and Plan 3G reveals that the sealed (+7.5 ft elevation core) breakwater effectively reduced long-period wave energy in the entrance channel and mooring basins. In most cases, response peaks were reduced by 50 percent or more in both magnitude and width. Some coupling of oscillations between Quivira and Mariners Basins was noted at 86 to 88 sec and corner-to-corner oscillations appeared to dominate in Quivira Basin.

90. A comparison of long-period test results for Plan 3G and Plan 9 reveals that the more porous breakwater of Plan 9 apparently allows wave energy to radiate out of the harbor more efficiently, thereby reducing oscillations within the harbor. The significant response at 86 sec for Base Test 5 and Plan 3G was eliminated by Plan 9. With this exception, responses for Plan 9 were similar to, or generally less than, Plan 3G.

Prototype long-period wave data

91. The Scripps Institution of Oceanography (SIO), under contract with the U. S. Army Corps of Engineers, undertook a project to collect and analyze prototype long-period wave data at Mission Bay, California (Castel and Seymour 1981). The primary purpose of this study was to determine the cause, pattern, and magnitude of surge in the basins, if possible.

92. Of primary interest to the model study were results of waves at gages located in Quivira Basin north (corresponding to gage 17, Plate 13, in the model) and Quivira Basin south (corresponding to gage 21, Plate 13, in the model). The data covered the period 1 August 1980 to 30 April 1981. Following analysis of these data, plots were made of energy density versus a dimensionless frequency. The dimensionless frequency consisted of dividing the calculated fundamental period of oscillation at the gage by the measured period at that gage.

$$\text{frequency*} = \frac{T_{(n,m)}}{T}$$

where

frequency* = the dimensionless frequency

$T_{(n,m)}$ = the natural period of oscillation at modes n and m

T = the measured wave period at the gage

In determining the natural period of oscillation, SIO used an expression developed by Sorenson in the case of a basin where the width and length are of comparable size and n,m are the oscillating modes

$$T_{(n,m)} = \frac{\left[2\left(\frac{n}{x}\right)^2 + \left(\frac{m}{y}\right)^2\right]^{1/2}}{(gd)^{1/2}}$$

where

x and y = the length and width of the basin

d = the water depth at the gage

g = the gravitational constant

SIO chose the fundamental frequencies using

$$n = 1$$

$$m = 1$$

$$x = 1{,}870 \text{ ft} \quad y = 1{,}870 \text{ ft}$$

$$d_{(\text{Quivira south})} = 14.3 \text{ ft}$$

$$d_{(\text{Quivira north})} = 19.9 \text{ ft}$$

For Quivira Basin south

$$T_{(n,m)} = 123.2 \text{ sec}$$

For Quivira Basin north

$$T_{(n,m)} = 104.4 \text{ sec}$$

By converting the dimensionless frequencies at which peaks occurred on the SIO graphs to wave periods, Quivira Basin south yielded periods of oscillation of approximately 615, 114, 107, 67, 59, 46, 36, 29, 25, and 21 sec. Likewise, Quivira Basin north yielded periods of oscillation of approximately 653, 131, 75, 60, 47, 36, 32, 28, and 24 sec.

93. Since tests conducted on the model were run at mhhw (swl +5.4 ft) and since the periods of oscillation vary with varying depths, in order to compare model tests with prototype data it became necessary to convert wave periods at one depth of water to equivalent periods at another depth. A list of model peak periods, their equivalent periods at prototype depths, and the periods recorded by SIO are presented below for Quivira Basin south and north.

Quivira Basin South			Quivira Basin North		
Model Period, sec		Prototype	Model Period, sec		Prototype
Actual	Equivalent	Period, sec	Actual	Equivalent	Period, sec
36	48	46	36	41	
46	61	59	44	50	47
50	67	67	50	56	60
61	81		62	70	
67	89		68	77	75
75	100	107	76	86	
90	120	114	86	97	
122	162		102	115	
135	180		122	138	131
			135	153	

These values appear to correspond favorably especially considering that the prototype plots are difficult to read precisely, and small deviations in the dimensionless frequency may result in a significant change in period.

94. It also should be noted that oscillations observed in the model were not necessarily from one side of the basin to the other. In fact, the largest peaks were observed to oscillate from corner to corner (i.e., the southwest corner to the northeast corner and the northwest corner to the southeast corner). This could explain why the large 86-sec oscillation at Quivira Basin north observed in the model did not show up in the prototype data.

Effect of proposed offshore breakwater on surfing

95. The entire southern California region is surfing country, and the Mission Bay area is no exception. The conditions along the coast are excellent for the sport as the mild climate makes surfing possible almost year-round. Waves are the right shape for surfing along Ocean Beach and Mission Beach, as the offshore topography causes many waves to lose their energy gradually but steadily as they move shoreward. Such a spilling wave gives the surfer the right pattern he needs to stand and ride the board across the nearshore region until the wave dies out in the last upwash of the surf. The surfers probably spend more time on the beach than any other residents of the area. It is not surprising, therefore, that great concern can arise in this community regarding any man-made structural measures which might potentially alter the surfing environment.

96. Because of the intimate relationship of surfing with the wave climate, the effect of the proposed offshore breakwater on surfing can be ascertained by determining the effect of the structure on the resulting wave

characteristics. At the present time, surfing activity exists along the Mission Beach region immediately north of the north jetty to Mission Bay, and immediately south of the middle jetty to the bay. Because the entrance to the San Diego River floodway is usually plugged with a littoral sand deposit, the floodway essentially becomes an extension of the Ocean Beach region to the south. Hence, surfing can be enjoyed both along the Ocean Beach area and along the floodway section.

97. Waves and current patterns and magnitudes for the existing condition (Base Test 1, Photo 1) using waves from the northwest deepwater direction showed the formation of strong longshore currents north of the north jetty and curving seaward toward the end of the north jetty. For large waves, these currents moved across the entrance channel and to the south. In general, this current, when combined with waves breaking along the middle jetty, produced a counterclockwise eddy in the entrance. A counterclockwise eddy also was formed in the lee of the middle jetty. The waves from the northwest direction propagated along the upcoast side of the north jetty, diffracted around the middle jetty, and propagated as a diffracted wave toward the San Diego River floodway.

98. Tests conducted using the existing condition with waves from the southwest deepwater direction (Photo 2) showed strong northerly longshore currents moving seaward past the end of the middle jetty. These currents tended to dissipate seaward of the middle jetty rather than move across the entrance. Currents in the entrance tended to flow seaward for this condition with little or no eddying. Longshore currents in the north side of the entrance generally curved from the end of the north jetty to the north along Mission Beach. A clockwise eddy was formed in the lee of the north jetty; a counterclockwise eddy was located south of the middle jetty.

99. Wave and current patterns and magnitudes were obtained for the recommended offshore breakwater configuration for the west, northwest, and southwest deepwater directions at mhhw and mllw (Photos 21, 23, 25, 27, and 29). For waves from the northwest, longshore currents moved south to the north jetty where they split upon reaching the outer end of the north jetty. One component moved seaward past the north end of the offshore breakwater and back to the north to form a large eddy. The other component moved into the north entrance and exited through the south entrance. For all waves, currents south of the middle jetty formed a counterclockwise eddy. Current patterns for waves from

the west deepwater direction showed the formation of a counterclockwise eddy south of the middle jetty and a clockwise eddy north of the north jetty for all wave, swl, and tidal flow conditions tested. For waves from the southwest deepwater direction, a counterclockwise eddy was formed south of the middle jetty. Currents north of the north jetty formed a clockwise eddy.

100. The ends of the offshore breakwater cause a shielding of the outer ends of the north and middle jetties as waves diffract around the breakwater. This condition is more pronounced at the end of the middle jetty for waves from the northwest, and is more pronounced at the end of the north jetty for waves from the southwest. However, as waves from each of these directions continue to propagate shoreward, diffraction effects tend to cause the wave crest to bend and become attached to the jetties. Hence, except for that region immediately behind and very near the offshore breakwater, the wave crest patterns of Photos 21, 23, 25, 27, and 29 are quite similar to the existing condition. Since it is extremely doubtful that surfing would occur immediately adjacent to either the north or middle jetty even under existing conditions, it appears that the existence of the recommended offshore breakwater would have minimal (negligible) effect on surfing activities. Surfers do not enter the entrance channel to Mission Bay between the north and middle jetties at the present time; hence the fact that the offshore breakwater would shelter the entire entrance channel from wave effects is of no consequence to surfing. At the same time, the return flow (rip currents) which develop along the outside of both the north and middle jetties for all wave conditions (as indicated by the physical model tests) will probably create a desirable avenue to be used by the surfers as they return to the sea (Hales and Curren, in preparation).

Stability tests

101. Throughout the mode' study reported herein, various breakwaters were tested to determine their effects on wave conditions. Certain structure parameters (i.e, crest elevation, length of structure, location, etc.) were optimized to obtain maximum performance at minimum cost. However, it was beyond the capability of this 1:100-scale, three-dimensional model to determine the stability of these structures under severe wave conditions. Therefore a larger scale, two-dimensional model was built and tested (Markle, in preparation) to determine the stability of the structures recommended by the three-dimensional model study.

PART V: SAN DIEGO RIVER FLOOD-CONTROL CHANNEL TESTS AND RESULTS

Description of Tests

Existing (or project) river channel

102. Prior to and in conjunction with tests of various improvement plans, tests were performed for the existing (or project) river channel. These tests included water-surface profiles, long-term tracer tests, and patterns of sand plug blowouts. The river discharges used during water-surface profiles and sand plug blowouts are listed in paragraph 24. Long-term tracer tests were conducted using waves from the southwest deepwater direction and no river discharge.

River improvement plans

103. Water-surface profile, current pattern and magnitude, and/or tracer tests were conducted for 29 plan variations. These variations consisted of changes in the plug elevations and widths, construction of a weir in the middle jetty, a diversion dike tied into the middle jetty, and south jetty extensions. Photographs of tracer movement and/or current patterns were obtained for all major improvement plans. Brief descriptions of the river improvement plans are presented below; dimensional details are presented in Plates 30-35:

a. Plan 4 (Plate 30) consisted of the elements of Plan 3G with the sand plug in the mouth of the river molded in cement mortar to an elevation of +6 ft.

b. Plan 4A (Plate 30) consisted of the elements of Plan 4 with the elevation of the sand plug raised to +10 ft.

c. Plan 4B (Plate 30) consisted of the elements of Plan 4A with the elevation of the sand plug raised to +14 ft.

d. Plan 5 (Plate 31) involved the elements of Plan 4A with the removal of 100 ft of the sand plug adjacent to the middle jetty.

e. Plan 5A (Plate 31) entailed the elements of Plan 5 with an additional 100 ft of sand plug removed (total channel width 200 ft).

f. Plan 5B (Plate 31) involved the elements of Plan 5A with an additional 100 ft of the sand plug removed (total channel width 300 ft).

g. Plan 5C (Plate 31) involved the elements of Plan 5B with an additional 100 ft of the sand plug removed (total channel width 400 ft).

h. Plan 5D (Plate 31) consisted of the elements of Plan 5C with an additional 100 ft of the sand plug removed (total channel width 500 ft).

i. Plan 5E (Plate 31) entailed the elements of Plan 5D with an additional 100 ft of the sand plug removed (total channel width 600 ft).

j. Plan 5F (Plate 31) involved the elements of Plan 5E with an additional 100 ft of the sand plug removed (total channel width 700 ft).

k. Plan 5G (Plate 31) involved the elements of Plan 5F with an additional 100 ft of the sand plug removed (total channel width 800 ft).

l. Plan 5H (Plate 31) entailed the elements of Plan 5G with the remainder of the sand plug removed.

m. Plan 6 (Plate 32) involved the elements of Plan 4 with the middle jetty made impervious from the sand plug to the shoreward terminus.

n. Plan 6A (Plate 32) entailed the elements of Plan 4 with a 1,200-ft-long weir (+6 ft crown elevation) built into the unsealed middle jetty.

o. Plan 6B (Plate 32) consisted of the elements of Plan 6 with a 1,200-ft-long weir (+6 ft crown elevation) built into the sealed middle jetty.

p. Plan 6C (Plate 32) involved the elements of Plan 4A with a 1,200-ft-long weir (+6 ft crown elevation) built into the unsealed middle jetty.

q. Plan 7 (Plate 33) involved the elements of Plan 3G with the sand plug removed and a 1,073-ft-long curved extension of the south jetty.

r. Plan 7A (Plate 33) entailed the elements of Plan 7 with the south jetty extension lengthened an additional 300 ft (total extension length 1,373 ft).

s. Plan 7B (Plate 33) entailed the elements of Plan 7A with the south jetty extension lengthened an additional 200 ft (total extension length 1,573 ft).

t. Plan 7C (Plate 34) consisted of the elements of Plan 7 with a 200-ft-long dogleg added to the south jetty extension (total extension length 1,273 ft).

u. Plan 7D (Plate 34) involved the elements of Plan 7C with the trunk of the south jetty extension lengthened an additional 100 ft which effectively moved the 200-ft-long dogleg seaward (total extension length 1,373 ft).

v. Plan 7E (Plate 34) involved the elements of Plan 7D with the trunk of the south jetty extension lengthened an additional 100 ft (total extension length 1,473 ft).

w. Plan 7F (Plate 34) entailed the elements of Plan 7E with the trunk of the south jetty extension lengthened an additional 100 ft (total extension length 1,573 ft).

x. Plan 7G (Plate 34) entailed the elements of Plan 7F with the trunk of the south jetty extension lengthened an additional 200 ft (total extension length 1,773 ft).

y. Plan 7H (Plate 34) involved the elements of Plan 7G with the trunk of the south jetty extension lengthened an additional 200 ft (total extension length 1,973 ft).

z. Plan 7I (Plate 34) entailed the elements of Plan 7H with the trunk of the south jetty extension lengthened an additional 200 ft (total extension length 2,173 ft).

aa. Plan 7J (Plate 34) involved the elements of Plan 7I with the trunk of the south jetty extension lengthened an additional 200 ft (total extension length 2,373 ft).

bb. Plan 8 (Plate 35) consisted of the elements of Plan 3G with no sand plug and the addition of a 200-ft-long diversion dike on the middle jetty 800 ft from the jetty head at an angle of 30 deg to the center line of the middle jetty and a crown elevation of +14 ft.

cc. Plan 8A (Plate 35) entailed the elements of Plan 8 with the diversion dike lengthened to 400 ft.

Typical sections of the various structures described above are shown in Appendix A. The location of the weir and diversion dike are shown in Plate 72.

Water-surface profile tests

104. Water-surface profile tests were conducted for the San Diego River Flood-Control Channel with the channel at project depths and for various improvement plans. The water-surface elevations were measured at selected locations for the river discharges mentioned in paragraph 24. The resulting elevations were plotted versus prototype station number to give a profile of the river under various conditions (Plates 36-71). These tests were performed with the ocean level at mhhw representing the worst case with respect to flooding.

River current patterns and magnitude tests

105. River current pattern 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 primarily for the weir for use in design of toe protection.

River tracer tests

106. Basically, three types of tracer tests were conducted during

this portion of the study. They are as follows:

a. Short-term tracer tests were conducted to determine the effectiveness of various south jetty extension plans in preventing tracer from moving into the river channel. These tests involved the introduction of small amounts of tracer into the model to determine the general pattern of movement.

b. Long-term tracer tests were conducted in an effort to reproduce shoaling of the river mouth and subsequent formation of a sand plug. Tracer was continuously fed into the updrift surf zone where wave-induced currents moved it alongshore and into the river mouth, forming the sand plug.

c. Sand plug blowout tests were conducted to study the effects of various plans and river discharges on the sand plug. During these tests, observations were made on where and how readily the sand plug washed out and the pattern of dispersal. Photographs taken during and after these tests illustrated the relative effectiveness of various improvement plans and potential problems encountered.

In evaluating the tracer test results, it should be kept in mind that there is no accepted time scale for bed evolution (i.e., development of the plug); therefore model times cannot be converted to equivalent prototype values. Relative comparisons of times among plans should be valid, however.

Test Results

107. In evaluating test results, the relative merits of each plan were based primarily on an analysis of water-surface profiles and/or tracer tests. From this evaluation, the best improvement plans were selected.

108. As mentioned in paragraph 76, long-term tracer tests were conducted for Plan 3G using 9-sec, 6-ft waves from the southwest deepwater direction in an effort to build a plug across the mouth of the San Diego River. By continuously feeding tracer into the surf zone, the shoreline between the south jetty and south groin built out to a point wherein all tracer subsequently fed into the model migrated past the end of the south jetty and into the river mouth. After 32 hr of model testing time, the plug had extended about halfway across the channel; after 55 hr of model testing time the plug merged with the middle jetty, as shown in Photo 34. Photo 35 is a closeup of the resulting plug viewed with the south jetty at the top and the middle jetty at the bottom. The bump in the shoreline next to the south jetty appeared to be a function of this particula. test wave.

109. The long-term tracer test was continued using a 13-sec, 11-ft wave

from southwest at mhhw and test results after 8 hr of model testing time are shown in Photo 36. The plug remained intact and actually accreted. Some of the finer coal dust present in the tracer moved into the south entrance, but the quantity was very small relative to the amount of tracer fed into the system. Photo 37 is a closeup of the plug showing results after the 13-sec, 11-ft wave. The bump adjacent to the south jetty is removed and the seaward face of the plug is steeper and more uniform.

110. A profile was made of the plug which showed elevations of the main body of the plug to be about +7.0 ft. At the shoreline, the elevation of the beach berm was about +10.0 ft. This profile allowed the plug to be reconstructed for subsequent tests.

111. The plug was subjected to various riverflows and the washout was observed and photographed. An 11,000-cfs flow (25-year recurrence) was tested first and the plug remained intact with flow passing through the voids of the middle jetty. Next, a flow of 49,000 cfs (100-year recurrence) was tested and the first breach in the plug was noted near the middle jetty after 4 min of model testing time. Three minutes later, the plug developed a second breach about 350 ft north of the south jetty as shown in Photo 38. The riverflow then was increased to 97,000 cfs (the SPF). Photographs taken after 3 and 45 min into the flood are shown in Photos 39 and 40. An examination of these photographs shows that the dispersal of tracer was generally to the southwest which prevented large quantities of tracer from shoaling the south entrance channel. Dry-bed photographs were taken after dewatering the model in order to better illustrate the dispersal pattern (Photos 41 and 42).

112. The tracer plug was replaced with concrete mortar plugs of various elevations and riverflow tests were conducted. Water-surface elevations were measured for the four river flood flows mentioned in paragraph 24 (11,000, 27,000, 49,000, and 97,000 cfs) for three different plug elevations [+6 ft (Plan 4), +10 ft (Plan 4A), and +14 ft (Plan 4B)]. Test results for Plans 4-4B are presented in Plates 36-39. In general, as the elevation of the sand plug increased and as the flow rate increased, the water-surface elevation increased. Gage 2 was located on the plug; thus some of the elevations plotted for this location represent the top of the plug rather than the water surface. All riverflows caused overtopping of the Plan 4 plug; the 49,000- and 97,000-cfs flows caused overtopping of the Plan 4A plug; and only the 97,000-cfs riverflow caused overtopping of the Pl· . 4B plug. In cases where the plug was not

overtopped, the flow exited through the voids of the middle jetty.

113. Riverflow tests were conducted with 100-ft increments of the Plan 4A plug removed (Plans 5-5H). Test results were plotted as before and are presented in Plates 40-43. Gage 2 was moved to the center of the channel created by the incremental plug removals. For the 11,000-cfs flow, the change in water-surface elevation, as more of the plug was removed, was small. As the flow rate increased, the differences in water-surface elevation became apparent. Only the 97,000-cfs flow caused any overtopping of the plug.

114. Test results of Plans 6-6B were plotted as before and are presented in Plates 44-47. For each flood flow, water-surface elevations were highest for Plan 6 because the entire riverflow was forced to exit over the plug. Conversely, for each flood flow, water-surface elevations were lowest for Plan 6A because the riverflow could exit through the voids of the middle jetty and over the weir as well as over the plug. A calculated water-surface profile (provided by SPL) for Plan 6B with a 97,000-cfs flood flow is compared with model data for Plan 6B in Plate 47. The calculated profile runs about 1.5 to 2 ft higher than the model test data. This probably is due to a difference in roughness factors used (i.e., SPL used a Manning's friction factor of 0.03 for the calculations while the friction factor of the slick concrete used in the model is approximately 0.015). The entire model was built with a slick concrete finish to minimize friction effects for wave-height tests. The model does, however, give a valid indication of relative water levels for various plans.

115. Photographs of Plan 6B for the 49,000- and 97,000-cfs flood flows are presented in Photos 43 and 44, respectively. A water-soluble dye was injected into the model to illustrate the pattern of flow over the weir and sand plug.

116. Tracer tests were conducted for various south jetty extensions using waves from the southwest deepwater direction in an effort to prevent wave-induced shoaling of the river entrance. Plan 7 consisted of a 1,073-ft-long curved extension of the south jetty. Tracer material was introduced into the surf zone in the vicinity of Sunset Cliffs. Tracer moved along the outer portion of the surf zone, past the south jetty extension head, and into the river mouth.

117. The south jetty extension was lengthened an additional 300 ft (Plan 7A) making the total extension length 1,373 ft. Test results showed a

continued movement of tracer past the head of the extension.

118. The south jetty extension then was lengthened an additional 200 ft (Plan 7B) making the total extension length 1,573 ft. Tracer material continued to move around the head of the extension, assisted by rip currents moving seaward along the structure.

119. At this point, it was felt that a realignment of the jetty head might induce currents and tracer material to eddy. A 200-ft-long dogleg was added to the end of the Plan 7 south jetty extension (Plan 7C) making the total extension length 1,273 ft. Tracer material in the surf zone moved past the jetty extension and into the river channel.

120. Plans 7D-7J involved additions to the trunk of the south jetty extension which effectively moved the 200-ft-long dogleg seaward. Total extension lengths of the plans are as follows: Plan 7D, 1,373 ft; Plan 7E, 1,473 ft; Plan 7F, 1,573 ft; Plan 7G, 1,773 ft; Plan 7H, 1,973 ft; Plan 7I, 2,173 ft; Plan 7J, 2,373 ft. As the south jetty length increased and extended farther through the surf zone, the buildup of water against the jetty extension increased, magnifying the seaward-flowing rip current adjacent to the jetty. The tracer in the surf zone appeared to move from the point of injection in a straight line to the jetty where it received an additional boost from the rip current to push it past the jetty head. As the jetty length increased, the rate of movement around the head decreased until with Plan 7J, the tracer material formed a counterclockwise eddy. This plan effectively prevented tracer material in the surf zone and shoreward of the surf zone from entering the river channel for tests at both mhhw and mllw. However, the large increase in jetty extension over that proposed (2,373 ft as compared with 1,073 ft) would make this plan expensive.

121. For the previous tests, tracer material was placed in the surf zone. Since the surf zone for large waves at mllw is located quite some distance offshore, the question was raised concerning the quantity of sand available in this area. Therefore it was decided to investigate a solution based on the movement of tracer material placed shoreward of the surf zone (for waves approximating less severe conditions). When Plan 7 was reinstalled in the model and tested for these new conditions, tracer material moved along the south jetty extension and past the jetty head (Photos 45 and 46).

122. Plan 7C was reinstalled in the model and tested with the revised injection procedure. Results showed that nearshore tracer material moved

along the south jetty extension and was forced by the 200-ft-long dogleg into a counterclockwise eddy (Photo 47). When tested with some of the larger waves, tracer material placed nearshore generally moved shoreward and to the south (Photo 48). Tracer material shown in the river channel in this photograph was a result of the emergence of tracer trapped in the voids of the structure during previous tests.

123. Based on the above tests, it appears that a 2,373-ft-long jetty extension will be required to eliminate all wave-induced shoaling of the San Diego River entrance, while a 1,273-ft-long extension will eliminate shoaling by nearshore (along the beach) material.

124. Tests of sand plug formation in the San Diego River entrance were conducted for Plan 3G with detailed photographic coverage using waves from the southwest deepwater direction (Photos 49-78). To prevent model circulation, each continuous test was limited to 15 min model time. Tracer was introduced continuously into the surf zone south of the south groin. Waves and currents transported this material past the end of the south groin where waves moved some of it shoreward to accrete the shoreline and some of it remained in the surf zone. The tracer that had remained in the surf zone was pushed by waves directly into the river mouth. As the shoreline between the south groin and the south jetty continued to build, material began to move past the end of the south jetty. As the test continued, the shoreline between the south groin and the south jetty stabilized. All additional tracer fed into the model moved along the shoreline and into the river mouth. Waves diffracting around the south jetty head moved this tracer into the lee of the jetty where it accreted. Eventually, the deposit rose above the waterline, as indicated by the white string line in Photo 53. As the tracer continued to deposit, a spit was formed which extended upriver (Photo 54). As more material entered the river mouth, it appeared that wave conditions began to change. The spit was breached by waves (Photo 55) and then re-formed (Photos 56-60). At this point, the spit began to widen (Photos 61-64). Eventually, another spit branched off the first one and moved across the river channel until it merged with the middle jetty (Photos 65-67) to form a plug. Waves again breached the plug (Photo 68) then sealed it (Photo 69). At this point, subsequent waves steadily widened the plug (Photos 70-78). Photo 78 shows the final configuration after 40 hr of model testing. A comparison of the model plug formation (from the above photographs) with the prototype sand plug formation (Figures 8-12)

Figure 8. Sand plug formation in the San Diego River entrance on 8 March 1951

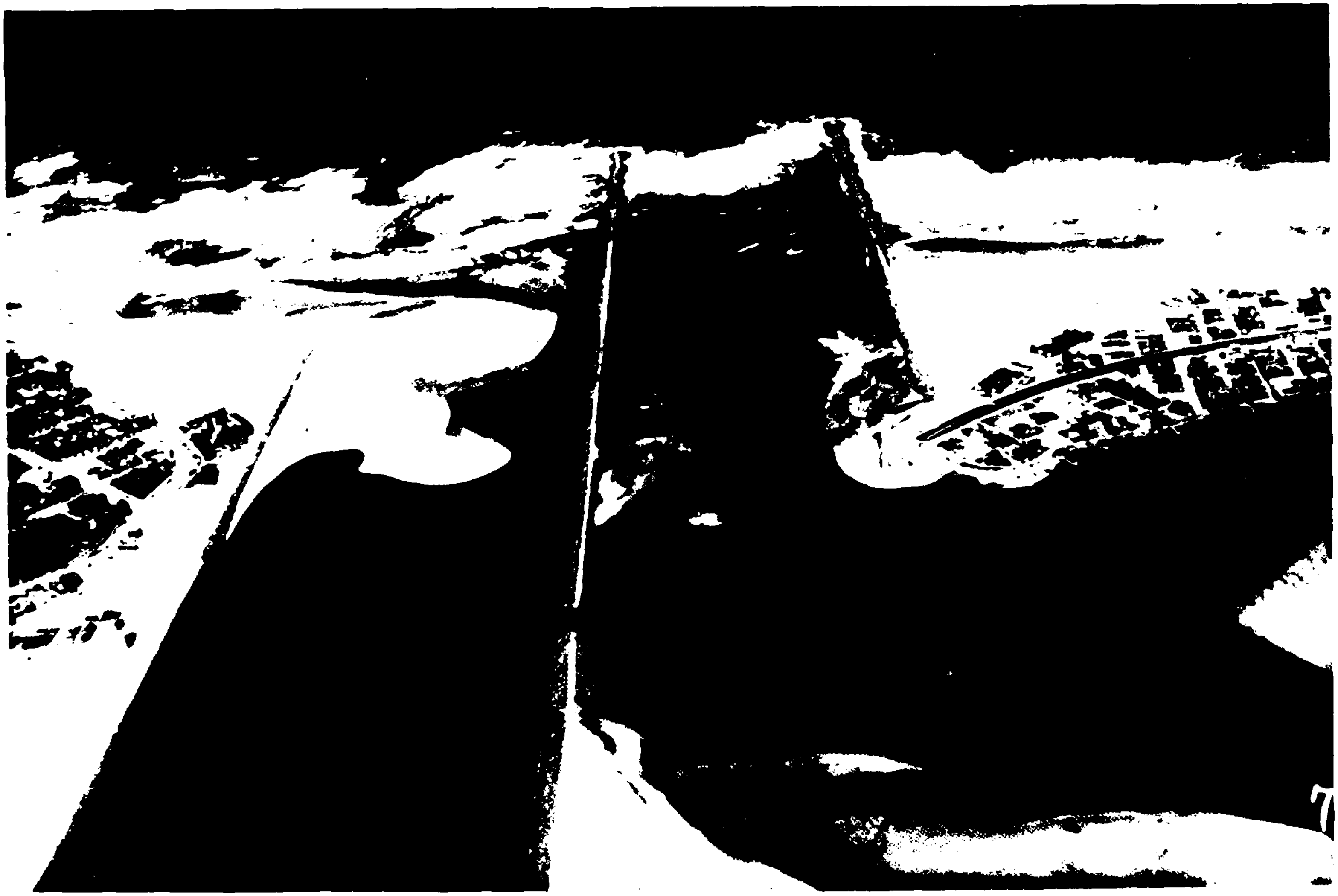

Figure 9. Sand plug formation in the San Diego River entrance on 30 April 1951

Figure 10. Sand plug formation in the San Diego River entrance on 28 November 1951

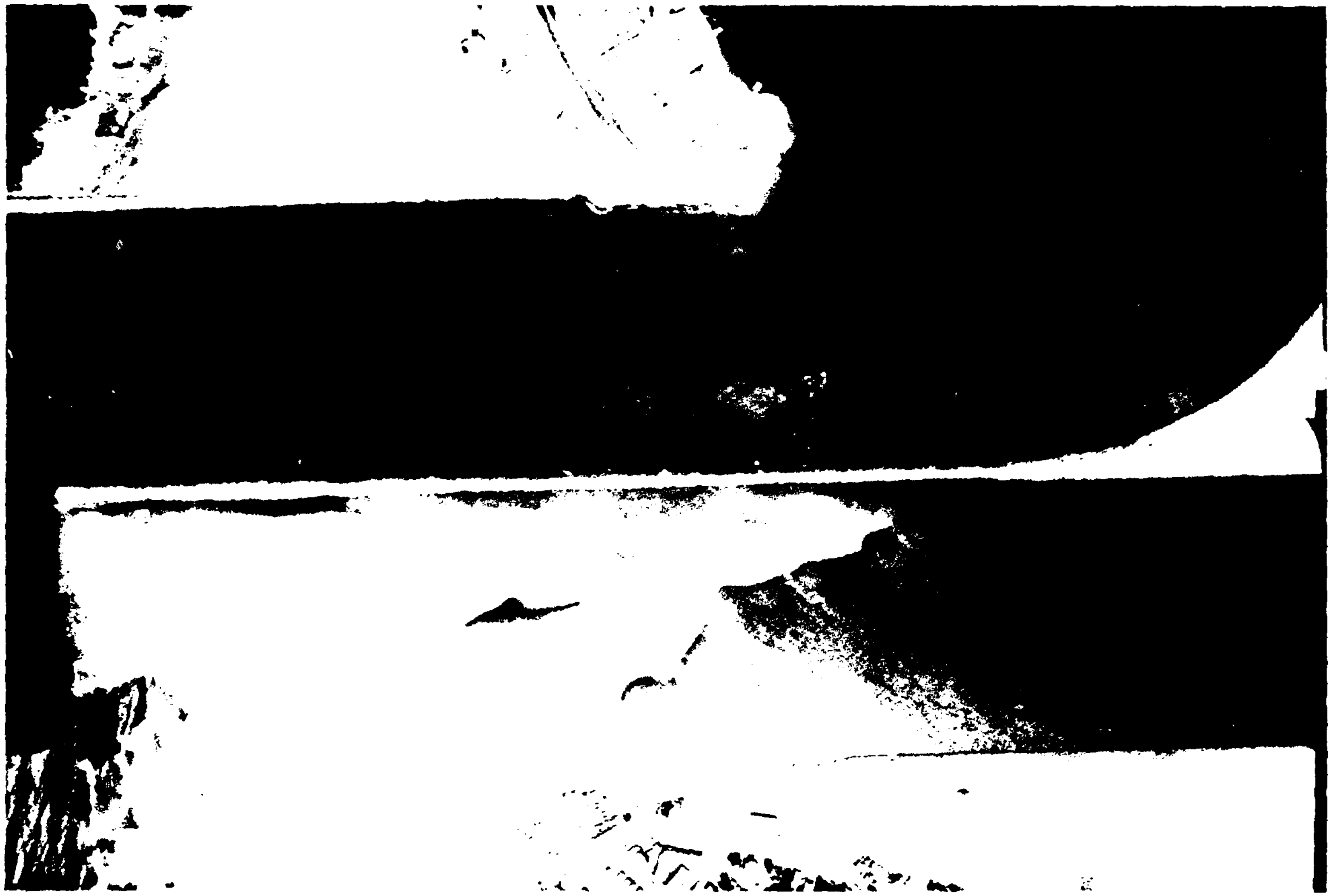

Figure 11. Sand plug formation in the San Diego River entrance on 9 March 1953

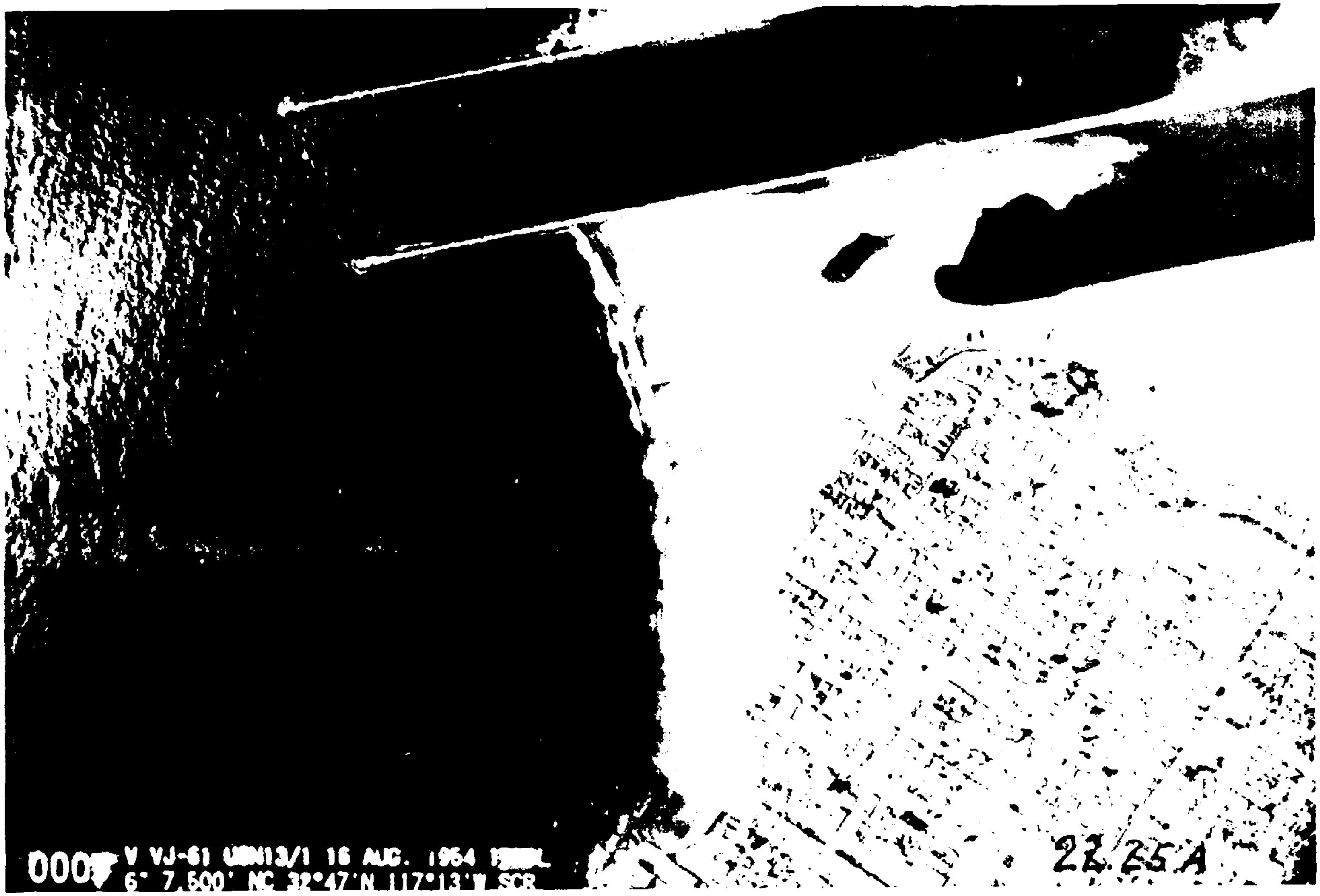

Figure 12. Sand plug formation in the San Diego River entrance on 16 August 1954

reveals very similar features, including the initial lobe formation (Figure 8 and Photo 64), the formation of a second lobe (Figure 9 and Photo 65), and the final shoreline configuration (Figure 12 and Photo 78). Surveys of the model and prototype plugs revealed very similar elevations (about +7 ft for the main body and about +10 ft for the berm in both cases).

125. Following formation of the plug, it was subjected successively to the four river test flows. While there was no time scale for erosion of the plug, each flow was run for a sufficient time to allow a stable flow and/or erosion pattern to develop (i.e., steady flow through voids of middle jetty for flows not overtopping plug and steady erosion of plug for overtopping flows). Photos 79 and 80 show the southern portion of the plug rapidly eroding away for the 49,000-cfs flow and the entire plug beginning to wash out for the 97,000-cfs flow in Photos 81 and 82. After 1 hr of testing, the model was drained and dry-bed photographs were taken of the resulting deposits (Photos 83 and 84). The general movement of tracer was to the southwest. There was a significant amount of tracer in the south entrance of the bay, but only a thin layer moved across the entrance channel.

126. The plug was rebuilt to an elevation of about +10 ft and a 100-ft-wide pilot channel (el +5.4 ft) was cut in the center of the plug to act as a release valve (Photo 85). As the smaller flood flows were run (11,000 and 27,000 cfs), water ran through the channel without overtopping the remainder of the plug (Photo 86). The channel began to steadily erode, making it deeper and wider. As the larger flood flows were run, the channel eroded faster (Photos 87 and 88). Because the channel continued to increase in width and depth, the amount of flow it would handle increased. Neither the 49,000- nor the 97,000-cfs flows overtopped the remainder of the plug. Tracer material that moved seaward moved in a more orderly fashion than before. Tracer moved directly seaward with little tracer entering the entrance channel. Dry-bed photographs are shown in Photos 89 and 90.

127 The plug was re-formed and a similar pilot channel cut next to the middle jetty. To prevent tracer from entering the south entrance to the bay, a 200-ft-long diversion dike was installed on the middle jetty 800 ft from the jetty head at an angle of 30 deg to the center line (Plan 8). This 200-ft-long dike proved insufficient in totally diverting the flow of tracer from the south entrances as shown in Photos 91 and 92.

128. The plug was reconstructed as before and the length of the dike

was increased to 400 ft (Plan 8A). Tracer moving seaward was successfully diverted around the south end of the Plan 3G offshore breakwater (Photos 93-95).

129. As discussed in paragraph 114, the initial model channel roughness was not the same as that used in SPL's calculations. Therefore, SPL requested that additional friction be installed in the river channel to make the two compatible and water-surface profiles be obtained with the fixed-bed sand plug installed. To obtain this additional roughness, sheets of expanded metal were anchored to the river channel bottom as illustrated in Photo 96. From past investigations at WES, it was determined that the equivalent Manning's friction factor should be very close to 0.03 (that used by SPL in their calculations). Also, since the calculations of water-surface elevations by SPL assumed the middle jetty to be impervious, this series of tests was performed with an impervious middle jetty. Water-surface profiles were obtained for Plans 6, 6B, and 6C for each flood flow (Plates 56-59). Plate 59 shows a comparison of measured and calculated (by SPL) profiles for Plan 6B. It can readily be seen that the two profiles are very close. The spike at gage 2 for Plan 6C in Plates 56 and 57 merely reflects the higher elevation of the sand plug (i.e. +10 ft).

130. Plates 48-51 show water-surface profiles for Plans 4-4B and 5H for each flood flow. Because the middle jetty was sealed, all flood flows were forced to exit over the plug except for Plan 5H which was a test with no plug. Tests with the middle jetty sealed resulted in consistently higher profiles than previous tests with a pervious middle jetty. For the higher riverflows, severe upstream flooding was noted for the higher plug elevations.

131. Incremental removals of 100 ft of the +10 ft elevation plug (Plans 5-5G) with the middle jetty sealed were tested for the four flood flows. Results (Plates 52-55) show a rapid reduction in water-surface elevations as the first few sections of the sand plug were removed. The difference in profiles decreases as more of the plug is removed.

132. SPL then requested retesting of the plans with the middle jetty made pervious and model channel roughness installed. This was felt to be more representative of actual conditions in the prototype. Plans 4-4B were retested and results are presented in Plates 60-63. Gage 2 was positioned in the middle of the sand plug; thus some of the elevations plotted at this location represent the top of the plug rather than the water surface. The plots show an increase in water-surface elevation with an increase in river

discharge. Since the middle jetty was pervious, some of the flood flows passed entirely through the voids of the middle jetty rather than over the plug. For Plan 4A (+10 ft plug elevation), only the 49,000- and 97,000-cfs flood flows overtopped the plug. For Plan 4B (+14 ft plug elevation) only the 97,000-cfs flood flow overtopped the plug.

133. River profile tests were conducted for Plans 5-5H (incremental removals of 100 ft from the north end of the +10 ft elevation fixed-bed sand plug) with channel roughness installed and a pervious middle jetty. Plan 5H was a "no plug" condition. Gage 2 was located in the center line of the channel formed by the incremental plug removals. Test results are shown in Plates 64-67. In general, as the gap widened, the influence of the sand plug on the restriction of the flow from the river channel was reduced. As the river channel became less constricted, water-surface elevations decreased. This change was much more significant for the higher flows than for the lower flows.

134. River profile tests were conducted for Plans 6A (+6 ft elevation fixed-bed plug) and 6C (+10 ft elevation fixed-bed plug) with the channel roughness installed and a pervious middle jetty.

a. Results of Plan 6A (with roughness, unsealed middle jetty) are compared with Plan 6A (without roughness, unsealed middle jetty) in Plates 68-71. The roughness caused an increase in water-surface elevations (1.5 to 2.0 ft) with a corresponding increase in slope. The slope also increased with increasing river discharge.

b. Tests for Plan 6C are compared with a previous test of Plan 6C with an impervious middle jetty and no roughness installed in Plates 68-71. These results show that water-surface elevations for Plan 6C (with roughness, unsealed middle jetty) are higher than for the previous test but not as high as for other plans. This may be due to the passage of water through the voids of the middle jetty preventing as large a buildup of water upstream. For the 49,000- and 97,000-cfs flows, gages 2 and 3 consistently recorded lower elevations. This also may be due to water escaping through the voids of the middle jetty. For the lower flows, the plug was not overtopped.

135. Current patterns and magnitudes were obtained for the Plan 6C (with roughness) weir for the four riverflows at swl's of 0.0 (mllw) and +5.4 (mhhw). These tests were to determine the current velocities in the vicinity of the toe of the weir for various river discharges and tide stages. Current velocities shown in Photos 97-100 were measured within 100 ft of the weir in the navigation channel. For riverflows of 11,000 and 27,000 cfs, velocities were

relatively slow (less than 2 fps). Photos 97 and 98 show the 49,000- and 97,000-cfs flows at mllw which were the worst conditions. For these conditions, the head difference was the greatest and the velocities the fastest (i.e., as high as 6.7 fps). Photos 99 and 100 show the 49,000- and 97,000-cfs flows at mhhw. The head difference was less and velocities only reached 5.0 fps. Care should be exercised in the interpretation of these velocity results. Velocities were obtained by timing the progress of a water-soluble dye with a stopwatch. Water quickly ran down the backside of the weir, then slowed as it entered the deeper water of the navigation channel. The resultant current measurements, therefore, are an average between the faster currents before the toe of the weir and slower currents beyond the toe of the weir. The average of the two combined should be fairly representative of the current velocities at the toe of the weir.

136. Test results obtained for the river entrance revealed a strong tendency for waves approaching from any direction (but especially for waves from the southwest) to move tracer material into the river mouth.

137. As designed, the river channel appeared to be able to handle all flows tested as evidenced by the plots of Plan 5H. The flood-control channel was designed with a depth of 4.64 ft at sta 30+00 (location of the south jetty head) and sloping upward upstream at a slope of 0.00072. With the river mouth blocked by a sand plug, the potential for flooding was greatly increased. Test results showed that fairly small flood flows (less than 27,000 cfs) may be accommodated with water exiting through the voids of the middle jetty. Larger flows may cause flooding upstream. A water-surface elevation in excess of +14 ft seaward of sta 68+00 was considered a potential flood hazard. Results for Plans 4-4B showed that as the sand plug elevation increases, the potential for upstream flooding also increases.

138. Results for Plans 5-5H showed that as more of the sand plug was removed, the potential for upstream flooding decreased. There was not a linear relationship between amount of plug removed and water-surface elevations. Indications are that a small amount of sand plug removal resulted in a large reduction in water-surface elevations.

139. Results for Plans 6A-6C showed the weir to be effective in reducing water-surface elevations with the sand plug in place. This should act as a release valve allowing the water to exit before building up to a level sufficient to cause upstream flooding. The best of these plans, with regard to

water-surface elevations, was Plan 6A with a sand plug elevation of +6 ft. A more representative plan was Plan 6C where the sand plug had a more realistic elevation of +10 ft.

140. Results for Plans 7-7J revealed that a 2,373-ft-long south jetty extension (Plan 7J) will be required to eliminate all wave-induced shoaling of the river mouth. A 1,273-ft-long south jetty extension (Plan 7C) will eliminate shoaling by nearshore material.

141. Results for Plans 8 and 8A showed the 200-ft-long spur jetty of Plan 8 to be insufficient in preventing harbor entrance shoaling due to river flood flows. The 400-ft-long spur jetty (Plan 8A) was required to divert currents and current-borne tracer away from the south entrance to the bay.

142. Following the conduct of the model study, a question arose concerning the effect of floodwaters passing over the weir on currents within the basins. Observations during the testing of the weir showed the presence of no adverse currents within or at the entrances to the basins. Floodwaters simply flowed over the weir and out the entrance channel. Also, since the location of the offshore breakwater was chosen so that the sum of the cross-sectional areas of the entrances between the jetties and breakwater equaled the cross-sectional area of the existing entrance channel, no restriction of flood flows out of the bay by the offshore breakwater was observed.

PART VI: CONCLUSIONS

The Harbor

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

- a. Existing conditions are characterized by strong longshore currents which are redirected seaward by the north and middle jetties for moderate to large wave conditions. In general, clockwise eddies form north of the north jetty and counter-clockwise eddies form south of the middle jetty. No shoaling of the harbor entrance was observed. Wave heights in the entrance channel were frequently excessive but were largely dissipated upon reaching the small-boat basins. Long-period wave tests revealed substantial oscillations in the entrance channel and the small-boat basins for a number of incident wave periods.
- b. The original improvement plan for wave protection for Mission Bay Harbor (i.e., the offshore breakwater Plan 1) was ineffective in reducing wave heights in the bay entrance to an acceptable level.
- c. Moving the breakwater into shallower water (Plan 2 series) decreased wave heights in the entrance channel to a more acceptable level, but the 1.5-ft criterion in the entrance channel still was exceeded. It was apparent that excessive wave energy was being transmitted through the voids of the breakwater.
- d. By sealing the core of the offshore breakwater (Plan 3 series), wave energy that had passed through the voids of the structure was largely eliminated. Of the plans tested, Plan 3G provided the most effective reduction of wave energy with a reduction of the volume of rock required for construction of 50 percent when compared with the originally proposed Plan 1. This plan was effective even under the most extreme conditions (i.e., removal of all revetment within the bay and an increase in swl to +7.6 ft). This plan also considerably reduced long-period waves (generally 50 percent or more) in the channel and basins. Shoaling of the harbor entrance was very slight and only for one extreme test condition.
- e. The Plan 9 offshore breakwater allowed slightly more short-period wave energy to enter the entrance channel than did Plan 3G but long-period responses within the bay were generally slightly less. Plan 9 reduced the volume of rock required for construction by 54 percent, when compared with the originally proposed plan, and should be considerably easier and less expensive to construct than Plan 3G.

f. Based on the results of all model tests, Plans 3G and 9 are considered as viable alternatives for providing wave and surge protection to Mission Bay.

The River

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

a. The river channel at project depth is prone to severe shoaling for waves from any direction, but particularly for waves from the southwest. The river channel at project depth is also quite capable of discharging the maximum flood flow tested (97,000 cfs) without causing flooding upstream.

b. Tests of the river channel with a +10 ft elevation sand plug (Plan 4A), representative of that presently blocking the river mouth, indicated a flooding hazard for the 49,000- and 97,000-cfs riverflows.

c. A reduction of the elevation of the sand plug to +6 ft reduced the flooding hazard. However, this plan would be difficult to maintain.

d. Removal of sections of the sand plug by dredging (Plans 5-5H) proved quite effective in reducing the flood hazard. Again, this plan may be difficult to maintain.

e. Tests conducted with a +6 ft elevation weir built into the middle jetty for a +10 ft elevation sand plug (Plan 6C) showed significantly reduced water-surface elevations for all river discharges.

f. Of the plans tested to prevent the formation of the sand plug, Plan 7J (2,373-ft-long south jetty extension) was effective in preventing all wave-induced river shoaling. However, because of the length of structure required, this plan would be quite expensive. Plan 7C (1,273-ft-long south jetty extension) would eliminate channel shoaling by nearshore material.

g. All plans involving a pilot channel cut into the sand plug worked well in preventing river flooding.

h. Plan 8A (400-ft-long diversion structure on the middle jetty) was the optimum plan tested for preventing shoaling of the south entrance to the bay during flood conditions.

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Table 1

Estimated Duration And Magnitude Of Deepwater Waves

Approaching Mission Bay from Various Directions

Wave Height ft	Duration, hr/yr, Wave Periods* of, sec								
	<4	4–6	6–8	8–10	10–12	12–14	14–16	16–18	18–20
					Northwest				
0–1									
1–2	254	503	20	1080	792	140	50	11	6
2–3		761	251	899	597	136	124	65	14
3–4		260	464	601	247	127	63	51	12
4–5		62	749	260	171	122	69	26	4
5–6			406	200	131	67	17	8	
6–7			156	197	111	37	23	6	
7–9			32	657	76	49	15	2	
9–11				176	77	15	6		
11–13				20	56	2			
13–15				4	21	2			
15–17					2		2		
					West				
0–1		9	158	614	885	701	307	96	26
1–2	228	88	157	61	149	324	491	254	88
2–3		96	147	225	93	59	35	96	105
3–4		53	43	94	62	31	2	8	44
4–5		26	41	42	20	9	11	2	18
5–6		6	20	25	28	12	9	4	
6–7				27	28	11	6		
7–9			9	25	28	18	2		
9–11				2	20	14	9		
11–13					15	2	61	2	
13–15						8			

(Continued)

Note: Since two or more well-developed wave trains may exist simultaneously, the total duration for a given period may exceed 100 percent.

* Wave-height and wave-period groupings include the lower but not the upper values.

Table 1 (Concluded)

Wave Height ft	<4	4-6	6-8	8-10	10-12	12-14	14-16	16-18	18-20
	Duration, hr/yr, Wave Periods* of, sec								
					Southwest				
0-1			9	35	18	745	720	150	35
1-2	221	140	44	55	71	1586	1113	316	53
2-3		138	41	15	28	527	420	114	9
3-4		45	55	19	4	79	123	124	9
4-5		9	43	9	2	9	26	9	9
5-6			15	9	9			18	
6-7			8	6					9
7-9				17	11		18		
9-11				2	4				

Table 2

Estimated Duration And Magnitude Of Shallow-Water Waves Approaching Mission Bay from Various Directions

Wave Height ft	Duration, hr/yr, Wave Periods* of, sec <4	4-6	6-8	8-10	10-12	12-14	14-16	16-18	18-20
					Northwest				
0-1									
1-2	254	503	20	1082	792				
2-3		761	251	899	597	140	50		6
3-4		260	464	628	247	136	124	11	14
4-5		62	749	259	171	127	63	65	12
5-6			406	326	131	122	69		4
6-7			156	674	111	67	17	51	
7-9			32	207	129	37	23	26	
9-11				34	99	49	15	14	
11-13				6	2	15	6		
13-15					2	2		2	
15-17						2			
17-19							2		
					West				
0-1		9	158	614	885	701	307	96	26
1-2	228	88	157	61	149	324	491	254	88
2-3		96	147	225	93	59	35	96	105
3-4		53	43	94	62	31	2	8	44
4-5		26	41	42	20	9	11	2	18
5-6		6	20	25	28	12	9	4	
6-7				27	28	11	6		
7-9			9	25	28	18	2		
9-11				2	20	14	9		
11-13					15	2	61	2	
13-15						8			

(Continued)

Note: Since two or more well-developed wave trains may exist simultaneously, the total duration for a given period may exceed 100 percent.

* Wave-height and wave-period groupings include the lower but not the upper values.

Table 2 (Concluded)

Wave Height ft	Duration, hr/yr, Wave Periods* of, sec								
	<4	4-6	6-8	8-10	10-12	12-14	14-16	16-18	18-20
					Southwest				
0-1			9	35	18	745	720	150	35
1-2	220	140	55	55	71	1665	1174	334	53
2-3		138	40	17	32	448	359	105	9
3-4		45	81	17	2	79	123	115	9
4-5		9	31	11		9	26	9	9
5-6			6	9	9			18	
6-7			6	6					9
7-9				19	11		18		
9-11					4				

TABLE 3

WAVE HEIGHTS FOR BASE TEST 1

SWL = 0.0 FT

TEST WAVE			WAVE HEIGHT, FT				
DIRECTION	PERIOD SEC	HEIGHT FT	GAGE 1	GAGE 2	GAGE 3	GAGE 4	GAGE 5
NW	7.0	9.0	[illegible]	2.3	0.3	<0.1	<0.1
	9.0	9.0	10.7	2.7	0.2	0.1	<0.1
		13.0	18.0	4.6	1.0	0.5	0.1
	11.0	9.0	13.8	4.3	1.2	0.3	<0.1
		15.0	20.9	4.9	1.1	0.3	0.1
	13.0	11.0	21.4	6.5	0.6	0.6	0.2
		17.0	14.4	6.6	0.8	0.7	0.2
	15.0	11.0	21.7	7.1	1.8	0.8	0.2
		17.0	14.7	7.1	1.6	0.7	0.2
	17.0	11.0	19.6	4.5	0.6	0.3	0.1
		15.0	19.0	5.6	0.8	0.6	0.2
	19.0	6.0	8.5	3.9	0.6	0.5	0.2
W	7.0	9.0	5.6	2.5	0.3	0.1	<0.1
	9.0	7.0	5.6	1.1	0.4	0.2	<0.1
		11.0	10.4	4.8	1.0	0.5	0.1
	11.0	7.0	8.6	4.6	1.0	0.4	<0.1
		13.0	13.5	4.6	1.1	0.2	<0.1
	13.0	7.0	7.9	4.6	0.4	0.3	<0.1
		15.0	11.8	5.2	0.5	0.5	0.1
	15.0	7.0	7.4	4.4	1.1	0.4	0.1
		13.0	14.0	5.5	1.7	0.6	0.2
	17.0	5.0	6.3	3.1	0.5	0.3	<0.1
		13.0	18.9	5.1	0.6	0.4	0.2
	19.0	15.0	5.5	2.3	0.3	0.4	0.2
SW	7.0	7.0	8.4	3.5	0.7	0.2	0.1
	9.0	11.0	13.2	4.5	1.4	0.6	0.1
	11.0	11.0	15.6	4.2	1.7	0.5	0.1
	13.0	11.0	16.2	7.8	0.7	0.6	0.2
	15.0	9.0	18.2	7.5	2.1	0.8	0.2
	17.0	5.0	8.0	4.8	0.7	0.4	0.1
	19.0	7.0	13.9	6.7	1.0	1.2	0.5

(CONTINUED)

(SHEET 1 OF 3)

TABLE 3 (CONTINUED)

TEST WAVE			WAVE HEIGHT, FT				
DIRECTION	PERIOD SEC	HEIGHT FT	GAGE 6	GAGE 7	GAGE 8	GAGE 9	GAGE 10
NW	7.0	9.0	<0.1	<0.1	<0.1	<0.1	<0.1
	9.0	9.0	<0.1	<0.1	<0.1	<0.1	<0.1
		13.0	0.1	0.1	0.1	0.1	0.1
	11.0	9.0	0.1	0.2	0.3	0.1	<[illegible]
		15.0	0.2	0.2	0.3	0.1	[illegible]
	13.0	11.0	0.3	0.2	0.2	<0.1	0.2
		17.0	0.3	0.2	0.2	0.1	[illegible]
	15.0	11.0	0.2	0.1	[illegible]	<0.1	[illegible]
		17.0	0.2	0.3	[illegible]	0.1	[illegible]
	17.0	11.0	0.2	0.1	0.2	0.1	[illegible]
		15.0	0.3	0.2	0.3	0.1	[illegible]
	19.0	9.0	0.2	0.1	<0.1	<0.1	<[illegible]
W	7.0	9.0	<0.1	<0.1	<0.1	<0.1	<0.1
	9.0	[illegible]	<0.1	<0.1	<0.1	<0.1	<[illegible]
		13.0	0.2	0.1	0.1	[illegible]	[illegible]
	11.0	7.0	0.2	0.2	0.4	0.2	<[illegible]
		13.0	0.1	0.1	0.2	<[illegible]	<[illegible]
	13.0	[illegible]	0.1	<0.1	<0.1	<[illegible]	[illegible]
		15.0	0.2	0.1	[illegible]	[illegible]	[illegible]
	15.0	[illegible]	0.2	0.1	[illegible]	<[illegible]	[illegible]
		13.0	0.2	0.2	0.2	[illegible]	[illegible]
	17.0	5.0	0.1	0.1	0.1	<0.1	<[illegible]
		13.0	0.2	0.1	0.3	0.1	0.1
	19.0	15.0	0.2	0.1	<0.1	<0.1	<[illegible]
SW	7.0	7.0	0.1	<0.1	<0.1	<0.1	<0.1
	[illegible]	11.0	0.3	0.2	0.1	0.1	<[illegible]
	[illegible]	11.0	0.3	0.3	0.5	[illegible]	[illegible]
	[illegible]	11.0	0.3	0.3	[illegible]	0.1	[illegible]
	[illegible]	9.0	0.3	0.3	[illegible]	0.1	[illegible]
	[illegible]	5.0	0.1	0.1	0.2	<0.1	<[illegible]
	[illegible]	[illegible]	0.3	0.3	0.2	0.1	[illegible]

(CONTINUED)

TABLE 3 (CONCLUDED)

Test Wave			Wave Height, ft			
Direction	Period sec	Height ft	Gage 11	Gage 12	Gage 13	Gage 14
NW	7.0	9.0	<0.1	<0.1	<0.1	<0.1
	9.0	9.0	0.1	<0.1	<0.1	<0.1
		13.0	0.3	0.2	<0.1	<0.1
	11.0	9.0	0.4	0.4	<0.1	<0.1
		15.0	0.3	0.4	<0.1	0.1
	13.0	11.0	0.2	0.2	0.2	<0.1
		17.0	0.4	0.2	0.3	0.1
	15.0	11.0	0.5	0.3	0.3	0.2
		17.0	0.4	0.4	0.3	0.2
	17.0	11.0	0.3	0.1	0.1	<0.1
		15.0	0.3	0.2	0.1	<0.1
	19.0	6.0	0.3	0.3	0.1	<0.1
W	7.0	9.0	<0.1	<0.1	<0.1	<0.1
	9.0	7.0	0.2	<0.1	<0.1	<0.1
		11.0	0.4	0.2	0.2	<0.1
	11.0	7.0	0.4	0.4	0.3	<0.1
		13.0	0.2	0.3	0.2	<0.1
	13.0	7.0	0.1	0.1	0.3	<0.1
		15.0	0.2	0.2	0.3	0.1
	15.0	7.0	0.3	0.3	0.2	0.2
		13.0	0.4	0.4	0.3	[illegible]
	17.0	5.0	0.3	0.1	<0.1	<0.1
		13.0	0.3	0.2	0.1	<0.1
	19.0	15.0	0.2	0.2	0.1	<0.1
SW	7.0	7.0	<0.1	<0.1	<0.1	<0.1
	9.0	11.0	0.5	0.2	0.2	<0.1
	11.0	11.0	0.5	0.5	0.4	<0.1
	13.0	11.0	0.4	0.3	0.3	0.1
	15.0	9.0	0.5	0.4	0.3	0.2
	17.0	5.0	0.3	0.2	0.1	<0.1
	19.0	7.0	0.4	0.5	0.2	0.1

(SHEET 3 OF 3)

TABLE 4

WAVE HEIGHTS FOR BASE TEST 1

SWL = +5.4 FT

TEST WAVE			WAVE HEIGHT, FT				
DIRECTION	PERIOD SEC	HEIGHT FT	GAGE 1	GAGE 2	GAGE 3	GAGE 4	GAGE 5
NW	7.0	9.0	9.2	1.6	0.3	<0.1	<0.1
	9.0	9.0	13.3	1.6	0.5	0.3	<0.1
		13.0	22.0	5.4	2.3	0.7	0.1
	11.0	9.0	13.3	2.3	0.9	0.3	<0.1
		15.0	26.8	5.2	2.4	1.1	0.2
	13.0	11.0	19.7	5.7	1.9	0.9	0.3
		17.0	23.3	9.4	2.7	1.4	0.5
	15.0	11.0	17.4	8.2	2.0	0.6	0.1
		17.0	22.1	7.6	1.5	0.5	0.2
	17.0	11.0	21.8	5.2	1.2	1.3	0.5
		15.0	18.4	6.0	0.9	1.1	0.4
	19.0	6.0	16.0	6.5	1.1	1.0	0.2
W	7.0	9.0	8.4	1.2	1.1	0.8	0.2
	9.0	7.0	5.9	2.9	1.4	0.3	0.1
		11.0	10.5	6.2	2.4	0.7	0.3
	11.0	7.0	7.3	3.5	1.3	0.5	<0.1
		13.0	13.8	7.6	1.9	1.2	0.2
	13.0	7.0	8.3	4.9	1.2	0.5	0.1
		15.0	21.7	7.4	1.6	0.9	0.2
	15.0	7.0	8.3	4.0	1.3	0.4	<0.1
		13.0	16.2	8.1	2.4	1.1	0.3
	17.0	5.0	5.9	2.3	0.5	0.3	0.2
		13.0	17.0	6.2	1.5	0.9	0.4
	19.0	15.0	5.9	1.8	0.3	0.3	<0.1
SW	7.0	7.0	8.0	0.9	0.4	0.4	<0.1
	9.0	11.0	11.5	5.5	2.7	0.9	0.2
	11.0	11.0	19.3	8.1	3.7	1.6	0.3
	13.0	11.0	21.4	9.4	2.2	1.1	0.5
	15.0	9.0	10.1	5.5	1.1	0.4	0.2
	17.0	5.0	8.3	3.4	0.8	0.8	0.4
	19.0	7.0	14.7	5.0	1.4	1.0	0.3

(CONTINUED)

TABLE 4 (CONTINUED)

TEST WAVE			WAVE HEIGHT, FT				
DIRECTION	PERIOD SEC	HEIGHT FT	GAGE 6	GAGE 7	GAGE 8	GAGE 9	GAGE 10
NW	7.0	9.0	<0.1	<0.1	<0.1	<0.1	<0.1
	9.0	9.0	0.1	<0.1	<0.1	<0.1	0.1
		13.0	0.3	0.3	0.1	0.2	0.2
	11.0	9.0	0.2	0.2	0.2	0.2	<0.1
		15.0	0.5	0.6	0.4	0.5	0.2
	13.0	11.0	0.3	0.3	0.2	0.2	0.3
		17.0	0.6	0.7	0.5	0.2	0.5
	15.0	11.0	0.4	0.4	0.2	0.1	0.2
		17.0	0.3	0.3	0.2	0.1	0.1
	17.0	11.0	0.4	0.2	0.2	0.2	0.1
		15.0	0.4	0.3	0.2	0.1	0.1
	19.0	6.0	0.5	0.3	0.2	0.2	0.2
W	7.0	9.0	0.3	0.2	<0.1	<0.1	<0.1
	9.0	7.0	0.2	0.2	0.1	0.2	0.2
		11.0	0.4	0.4	0.3	0.3	0.3
	11.0	7.0	0.2	0.3	0.2	0.3	0.1
		13.0	0.4	0.7	0.5	0.6	0.3
	13.0	7.0	0.2	0.2	0.3	0.2	0.2
		15.0	0.4	0.3	0.2	0.2	0.3
	15.0	7.0	0.2	0.3	0.3	<0.1	0.1
		13.0	0.6	0.4	0.4	0.1	0.3
	17.0	5.0	<0.1	0.1	0.1	<0.1	<0.1
		13.0	0.3	0.3	0.3	0.2	0.2
	19.0	15.0	0.1	0.1	<0.1	0.1	0.1
SW	7.0	7.0	0.2	0.1	<0.1	<0.1	<0.1
	9.0	11.0	0.4	0.5	0.3	0.4	0.4
	11.0	11.0	0.7	1.0	0.6	0.9	0.4
	13.0	11.0	0.5	0.6	0.4	0.8	0.4
	15.0	9.0	0.2	0.2	0.2	0.1	0.1
	17.0	5.0	0.2	0.2	0.1	0.1	0.1
	19.0	7.0	0.3	0.4	0.2	0.2	0.2

(CONTINUED)

(SHEET 2 OF 3)

TABLE 4 (CONCLUDED)

Test Wave			Wave Height, ft			
Direction	Period sec	Height ft	Gage 11	Gage 12	Gage 13	Gage 14
NW	7.0	9.0	0.1	<0.1	<0.1	<0.1
	9.0	9.0	0.3	0.2	0.1	[illegible]
		13.0	0.6	0.9	0.6	[illegible]
	11.0	9.0	0.4	0.2	0.1	[illegible]
		15.0	0.7	0.6	0.4	[illegible]
	13.0	11.0	0.4	0.4	0.2	[illegible]
		17.0	0.7	0.7	0.6	[illegible]
	15.0	11.0	0.5	0.7	0.4	[illegible]
		17.0	0.7	0.3	0.3	[illegible]
	17.0	11.0	0.4	0.3	0.3	[illegible]
		15.0	0.4	0.4	0.3	[illegible]
	19.0	6.0	0.8	1.0	0.3	[illegible]
W	7.0	9.0	0.2	0.2	0.2	0.1
	9.0	7.0	0.3	0.3	0.2	[illegible]
		11.0	0.5	0.8	0.4	[illegible]
	11.0	7.0	0.2	0.3	0.1	[illegible]
		13.0	0.9	0.7	0.5	[illegible]
	13.0	7.0	0.3	0.4	0.2	[illegible]
		15.0	0.5	0.5	0.3	[illegible]
	15.0	7.0	0.4	0.4	0.1	[illegible]
		13.0	0.6	0.9	0.4	0.2
	17.0	5.0	0.2	0.2	0.1	0.1
		13.0	0.5	0.4	0.3	0.2
	19.0	15.0	0.2	0.4	0.1	0.2
SW	7.0	7.0	0.2	<0.1	<0.1	<0.1
	9.0	11.0	0.7	0.8	0.7	[illegible]
	11.0	11.0	0.8	0.8	0.7	[illegible]
	13.0	11.0	0.7	0.7	0.5	[illegible]
	15.0	9.0	0.5	0.2	0.3	[illegible]
	17.0	5.0	0.3	0.2	0.2	[illegible]
	19.0	7.0	0.7	0.6	0.3	[illegible]

TABLE 5

COMPARISON WAVE HEIGHTS FOR BASE TESTS 1-5

FOR TEST WAVES FROM WEST, SWL = +5.4 FT

Plan	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
BT 1	11.0	13.0	13.8	7.6	1.9	1.2	0.2	0.4	0.7	0.5	0.6	0.3	0.9	0.7	0.5	0.5
	15.0	13.0	16.2	8.1	2.4	1.1	0.3	0.6	0.4	0.4	0.1	0.3	0.6	0.9	0.4	0.2
BT 2	11.0	13.0	13.8	8.3	2.0	0.8	0.2	0.4	0.3	0.5	0.4	0.4	1.3	1.0	1.2	1.0
	15.0	13.0	13.8	6.9	2.3	1.0	0.2	0.3	0.3	0.3	0.2	0.3	1.2	0.7	0.4	0.3
BT 3	11.0	13.0	12.8	7.0	2.2	1.0	1.0	1.0	0.9	1.1	1.0	1.2	1.2	0.7	1.1	1.0
	15.0	13.0	12.9	6.0	2.1	1.1	0.8	0.8	0.7	0.7	0.5	0.6	1.1	0.5	0.3	0.2
BT 4	11.0	13.0	14.0	8.1	7.0	2.2	1.3	2.2	2.3	1.3	1.3	1.5	1.6	3.4	2.0	1.7
	15.0	13.0	12.4	6.8	2.8	1.0	0.7	0.7	0.7	0.6	0.4	0.4	1.6	1.6	2.0	1.3
BT 5	11.0	13.0	15.0	7.1	3.0	1.2	0.7	1.1	1.2	1.0	1.0	1.1	1.3	1.1	0.8	0.5
	15.0	13.0	14.7	6.3	2.5	0.7	0.6	0.4	0.4	0.5	0.3	0.3	1.3	0.7	0.4	0.3

TABLE 6

WAVE HEIGHTS FOR BASE TEST 5

FOR 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
				SWL = 0.0 FT					
W	7.0	9.0	6.5	3.3	0.4	0.2	0.2	0.2	0.1
	9.0	7.0	5.3	0.5	0.4	0.4	0.1	0.2	0.2
		11.0	12.4	2.9	0.8	0.8	0.5	0.8	0.4
	11.0	7.0	6.1	3.5	0.6	0.6	0.2	0.2	0.5
		13.0	13.3	4.8	1.2	0.5	0.4	0.5	0.4
	13.0	7.0	8.0	4.7	2.0	0.4	0.3	0.3	0.2
		15.0	12.7	4.0	1.0	0.7	0.3	0.5	0.3
	15.0	7.0	7.4	4.3	2.2	0.3	0.3	0.3	0.5
		13.0	15.7	5.8	2.6	0.5	0.5	0.5	0.5
	17.0	5.0	6.1	3.1	1.6	0.3	0.2	0.2	0.1
		13.0	13.4	4.5	1.8	0.8	0.5	0.5	0.4
				SWL = +5.4 FT					
W	7.0	9.0	6.8	3.4	0.1	0.2	0.2	0.2	0.1
	9.0	7.0	6.6	1.0	0.8	0.7	0.3	0.3	0.2
		11.0	10.8	5.1	2.0	1.4	0.8	0.6	0.4
	11.0	7.0	7.2	3.6	1.8	0.3	0.4	0.3	0.4
		13.0	15.0	7.1	3.0	1.2	0.7	1.1	1.2
	13.0	7.0	8.6	5.4	1.8	0.7	1.3	1.2	0.6
		15.0	22.9	7.3	1.9	1.1	1.2	1.0	0.7
	15.0	7.0	6.8	3.9	2.0	0.3	0.2	0.2	0.3
		13.0	14.7	6.3	2.5	0.7	0.6	0.4	0.4
	17.0	5.0	4.5	1.8	0.7	0.8	0.3	0.2	0.2
		13.0	17.0	4.0	1.5	1.7	0.7	1.2	0.5

(CONTINUED)

TABLE 6 (CONCLUDED)

Test Wave Direction	Test Wave Period, sec	Test Wave Height, ft	Wave Height, ft: Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 14
			SWL = 0.0 FT						
W	7.0	9.0	0.2	0.2	0.1	0.1	0.1	<0.1	<0.1
	9.0	7.0	0.3	0.2	0.1	0.2	<0.1	0.1	<0.1
		11.0	0.8	0.5	0.5	0.4	0.2	0.2	<0.1
	11.0	7.0	0.3	0.2	0.3	0.6	0.5	0.5	0.1
		13.0	0.5	0.4	0.2	0.4	0.5	0.3	0.1
	13.0	7.0	0.5	0.3	0.3	0.8	0.4	0.6	0.2
		15.0	0.4	0.4	0.4	1.1	0.7	0.5	0.2
	15.0	7.0	0.2	0.4	0.3	0.7	1.1	0.5	0.2
		13.0	0.7	0.4	0.4	0.9	1.0	0.5	0.3
	17.0	5.0	0.2	0.1	0.2	0.5	0.2	0.1	<0.1
		13.0	0.7	0.4	0.4	0.8	0.5	0.3	0.1
			SWL = +5.4 FT						
W	7.0	9.0	0.2	0.2	0.3	0.1	<0.1	<0.1	<0.1
	9.0	7.0	0.2	0.2	0.2	0.3	0.1	0.1	<0.1
		11.0	0.5	0.5	0.5	1.1	0.3	0.3	0.2
	11.0	7.0	1.0	0.4	0.4	1.0	0.8	0.4	0.1
		13.0	1.0	1.0	1.1	1.3	1.1	0.8	0.5
	13.0	7.0	1.1	0.3	0.7	0.8	0.6	0.5	0.3
		15.0	0.8	0.6	0.8	0.8	0.6	1.0	0.7
	15.0	7.0	0.2	0.1	0.2	0.8	0.8	0.8	0.3
		13.0	0.5	0.3	0.3	1.3	0.7	0.4	0.3
	17.0	5.0	0.2	0.2	0.2	0.3	0.3	0.2	0.2
		13.0	0.5	0.5	0.5	0.6	0.5	0.4	0.3

TABLE 7

WAVE HEIGHTS FOR BASE TEST 5

WITH AND WITHOUT TIDAL FLOWS

FOR TEST WAVES FROM WEST

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
SWL = 0.0 FT									
W	9.0	11.0	12.4	2.9	0.8	0.8	0.5	0.8	0.4
	11.0	13.0	13.3	4.8	1.2	0.5	0.4	0.5	0.4
	13.0	15.0	12.7	4.0	1.0	0.7	0.3	0.5	0.3
	15.0	13.0	15.7	5.8	2.6	0.5	0.5	0.5	0.5
SWL = +2.7 FT(MAXIMUM EBB)									
W	9.0	11.0	18.0	3.4	1.2	1.1	0.3	0.3	0.3
	11.0	13.0	22.0	3.5	1.1	0.6	0.4	0.4	0.4
	13.0	15.0	16.4	6.1	1.2	0.8	0.9	0.6	0.7
	15.0	13.0	21.2	6.5	1.5	1.4	1.2	0.9	1.0
SWL = +2.7 FT(MAXIMUM FLOOD)									
W	9.0	11.0	9.2	4.7	2.8	1.7	0.8	1.2	0.7
	11.0	13.0	13.4	4.7	2.7	1.4	0.6	1.1	1.0
	13.0	15.0	20.4	4.0	1.1	0.8	0.7	0.6	0.5
	15.0	13.0	16.6	6.6	4.2	1.5	0.5	0.5	0.6
SWL = +5.4 FT									
W	9.0	11.0	10.8	5.1	2.0	1.4	0.8	0.6	0.4
	11.0	13.0	15.0	7.1	3.0	1.2	0.7	1.1	1.2
	13.0	15.0	22.9	7.3	1.9	1.1	1.2	1.0	0.7
	15.0	13.0	14.7	6.3	2.5	0.7	0.6	0.4	0.4

(CONTINUED)

TABLE 7 (CONCLUDED)

Test Wave Direction	Test Wave Period, sec	Test Wave Height, ft	Wave Height, ft: Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 14
SWL = 0.0 FT									
W	9.0	11.0	0.8	0.5	0.5	0.4	0.2	0.2	<0.1
	11.0	13.0	0.5	0.4	0.2	0.4	0.5	0.3	0.1
	13.0	15.0	0.4	0.4	0.4	1.1	0.7	0.5	0.2
	15.0	13.0	0.7	0.4	0.4	0.9	1.0	0.5	0.3
SWL = +2.7 FT(MAXIMUM EBB)									
W	9.0	11.0	0.5	0.4	0.3	0.6	0.3	0.3	0.2
	11.0	13.0	0.5	0.4	0.4	0.6	1.0	0.5	0.2
	13.0	15.0	1.0	0.4	1.0	0.8	0.6	0.6	0.4
	15.0	13.0	1.1	0.4	0.6	1.2	0.7	0.4	0.4
SWL = +2.7 FT(MAXIMUM FLOOD)									
W	9.0	11.0	1.0	1.0	0.8	1.3	0.5	0.3	0.2
	11.0	13.0	1.4	1.1	1.0	1.6	1.8	1.1	0.4
	13.0	15.0	0.5	0.4	0.7	1.1	0.6	0.7	0.4
	15.0	13.0	0.8	0.4	0.5	1.9	1.7	0.7	0.6
SWL = +5.4 FT									
W	9.0	11.0	0.5	0.5	0.5	1.1	0.3	0.3	0.2
	11.0	13.0	1.0	1.0	1.1	1.3	1.1	0.8	0.5
	13.0	15.0	0.8	0.6	0.8	0.8	0.6	1.0	0.7
	15.0	13.0	0.5	0.3	0.3	1.3	0.7	0.4	0.3

TABLE 8

WAVE HEIGHTS FOR PLAN 1 FOR TEST WAVES FROM WEST DIRECTION

Test Wave: Direction	Test Wave: Period, sec	Test Wave: Height, ft	Wave Height, ft: Gage 1	Gage 2	Gage 3	Gage 4	Gage 5
			SWL = 0.0 FT				
W	7.0	9.0	0.3	0.2	<0.1	<0.1	<0.1
	9.0	7.0	0.7	0.1	0.1	<0.1	<0.1
		11.0	1.4	0.2	0.2	<0.1	<0.1
	11.0	7.0	1.4	0.2	<0.1	<0.1	<0.1
		13.0	2.5	0.5	0.1	0.1	<0.1
	13.0	7.0	1.1	1.0	0.4	<0.1	<0.1
		15.0	2.2	1.4	0.6	0.2	0.1
	15.0	7.0	0.9	1.0	0.3	0.1	<0.1
		13.0	2.1	1.6	0.6	0.2	0.1
	17.0	5.0	1.0	0.7	0.4	<0.1	<0.1
		13.0	1.9	1.0	0.5	0.2	0.1
			SWL = +2.7 FT(MAXIMUM EBB)				
W	9.0	11.0	0.7	0.3	<0.1	<0.1	<0.1
	11.0	13.0	2.2	0.5	0.3	<0.1	0.1
	13.0	15.0	1.6	1.4	0.5	0.2	0.2
	15.0	13.0	2.9	1.2	0.9	0.2	0.1
			SWL = +2.7 FT(MAXIMUM FLOOD)				
W	9.0	11.0	1.2	0.3	0.2	0.1	<0.1
	11.0	13.0	1.7	1.1	0.8	0.2	0.1
	13.0	15.0	1.7	1.6	0.4	0.2	0.2
	15.0	13.0	3.4	1.5	0.8	0.1	0.1
			SWL = +5.4 FT				
W	7.0	7.0	0.4	0.2	<0.1	<0.1	<0.1
	9.0	7.0	0.9	0.2	0.2	0.1	<0.1
		11.0	1.5	0.3	0.3	0.1	<0.1
	11.0	7.0	0.5	1.0	0.3	<0.1	<0.1
		13.0	1.4	1.9	0.4	0.3	0.1
	13.0	7.0	0.9	1.1	0.3	0.1	0.2
		15.0	3.5	2.2	0.5	0.3	0.2
	15.0	7.0	1.9	0.8	0.3	0.1	<0.1
		13.0	3.2	2.0	0.3	0.2	0.2
	17.0	5.0	1.6	0.6	0.2	0.2	<0.1
		13.0	3.9	2.5	0.5	0.9	0.2

(CONTINUED)

TABLE 8 (CONTINUED)

Test Wave			Wave Height, ft				
Direction	Period sec	Height ft	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10
			SWL = 0.0 FT				
W	7.0	9.0	<0.1	<0.1	<0.1	<0.1	<0.1
	9.0	7.0	<0.1	<0.1	<0.1	<0.1	<0.1
		11.0	<0.1	<0.1	<0.1	0.1	<0.1
	11.0	7.0	0.1	<0.1	<0.1	0.1	<0.1
		13.0	0.1	0.1	0.1	0.1	<0.1
	13.0	7.0	<0.1	<0.1	0.2	<0.1	0.1
		15.0	0.1	0.1	0.2	0.2	0.2
	15.0	7.0	<0.1	<0.1	<0.1	<0.1	<0.1
		13.0	<0.1	<0.1	<0.1	0.1	0.1
	17.0	5.0	<0.1	<0.1	<0.1	<0.1	<0.1
		13.0	0.2	0.1	0.2	0.1	<0.1
			SWL = +2.7 FT(MAXIMUM EBB)				
W	9.0	11.0	<0.1	<0.1	<0.1	<0.1	<0.1
	11.0	13.0	0.2	<0.1	0.1	0.2	0.1
	13.0	15.0	0.2	0.2	0.2	0.1	0.3
	15.0	13.0	<0.1	<0.1	0.1	<0.1	<0.1
			SWL = +2.7 FT(MAXIMUM FLOOD)				
W	9.0	11.0	0.1	<0.1	<0.1	0.1	<0.1
	11.0	13.0	0.2	0.2	0.2	0.1	0.2
	13.0	15.0	0.2	0.1	0.3	0.1	0.2
	15.0	13.0	<0.1	0.1	0.1	<0.1	0.2
			SWL = +5.4 FT				
W	7.0	7.0	<0.1	<0.1	<0.1	<0.1	<0.1
	9.0	7.0	<0.1	<0.1	<0.1	0.1	<0.1
		11.0	<0.1	0.1	<0.1	0.2	0.1
	11.0	7.0	<0.1	0.1	<0.1	0.1	0.1
		13.0	0.1	0.2	0.2	0.3	0.1
	13.0	7.0	0.2	0.3	0.2	<0.1	0.1
		15.0	0.3	0.4	0.2	0.2	0.2
	15.0	7.0	<0.1	0.1	<0.1	<0.1	<0.1
		13.0	0.2	0.3	0.1	0.1	0.1
	17.0	5.0	0.1	<0.1	<0.1	<0.1	<0.1
		13.0	0.3	0.1	0.1	0.2	0.1

(CONTINUED)

TABLE 8 (CONCLUDED)

TEST WAVE			WAVE HEIGHT, FT			
DIRECTION	PERIOD SEC	HEIGHT FT	GAGE 11	GAGE 12	GAGE 13	GAGE 14
			SWL = 0.0 FT			
W	7.0	9.0	1.0	<0.1	<0.1	1.7
	9.0	7.0	0.6	<0.1	<0.1	2.1
		11.0	1.6	<0.1	<0.1	3.4
	11.0	7.0	1.2	<0.1	<0.1	2.4
		13.0	3.0	0.1	0.1	4.1
	13.0	7.0	1.4	0.1	<0.1	5.5
		15.0	3.0	0.1	0.1	6.4
	15.0	7.0	1.4	0.2	0.1	4.8
		13.0	4.2	0.3	0.1	6.8
	17.0	5.0	0.9	<0.1	<0.1	2.2
		13.0	2.9	0.2	0.1	4.2
			SWL = +2.7 FT(MAXIMUM EBB)			
W	9.0	11.0	0.7	<0.1	<0.1	1.8
	11.0	13.0	1.8	0.2	0.1	2.9
	13.0	15.0	2.9	0.1	0.3	5.0
	15.0	13.0	1.8	0.2	0.1	5.0
			SWL = +2.7 FT(MAXIMUM FLOOD)			
W	9.0	11.0	1.7	<0.1	<0.1	2.9
	11.0	13.0	2.0	0.2	0.1	4.6
	13.0	15.0	4.1	0.2	0.2	5.4
	15.0	13.0	2.9	0.3	<0.1	6.5
			SWL = +5.4 FT			
W	7.0	7.0	0.7	<0.1	<0.1	1.8
	9.0	7.0	0.5	<0.1	0.1	1.8
		11.0	1.1	0.1	<0.1	2.6
	11.0	7.0	1.1	<0.1	0.1	2.8
		13.0	1.6	0.2	0.1	5.4
	13.0	7.0	0.9	0.1	0.2	2.7
		15.0	3.0	0.3	0.2	5.7
	15.0	7.0	1.4	0.1	0.1	2.9
		13.0	2.6	0.2	0.1	4.8
	17.0	5.0	1.2	0.2	0.1	2.1
		13.0	3.3	0.4	0.3	5.4

(SHEET 3 OF 3)

TABLE 9

COMPARISON WAVE HEIGHTS FOR PLANS 1-11 FOR TEST WAVES

FROM WEST DIRECTION, SWL = +5.4 FT

PLAN	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
1	9.0	11.0	1.5	0.3	0.3	0.1	<0.1	<0.1	0.1
	11.0	13.0	1.4	1.9	0.4	0.3	0.1	0.1	0.2
	13.0	15.0	3.5	2.2	0.5	0.3	0.2	0.3	0.4
	15.0	13.0	3.2	2.0	0.3	0.2	0.2	0.2	0.3
1A	9.0	11.0	1.7	0.5	0.2	0.1	<0.1	0.1	0.1
	11.0	13.0	1.7	1.6	0.9	0.2	0.1	0.1	0.2
	13.0	15.0	3.0	2.5	0.5	0.2	0.3	0.2	0.1
	15.0	13.0	4.6	2.1	1.2	0.4	0.2	0.2	0.2
1B	9.0	11.0	1.3	0.7	0.4	0.1	0.1	0.2	0.1
	11.0	13.0	4.4	1.0	0.4	0.1	0.1	0.1	0.1
	13.0	15.0	5.9	2.1	1.0	0.6	0.3	0.2	0.3
	15.0	13.0	6.4	2.5	1.4	0.3	0.2	0.3	0.3
1C	9.0	11.0	3.2	0.8	0.3	0.2	0.1	0.1	<0.1
	11.0	13.0	4.1	1.1	0.5	0.2	0.2	0.3	0.2
	15.0	15.0	4.9	2.3	0.4	0.3	0.2	0.2	0.2
		13.0	4.0	2.1	0.8	0.2	0.2	0.2	0.2
1D	9.0	11.0	2.1	1.6	0.7	0.3	0.2	0.2	0.4
	11.0	13.0	2.1	2.2	0.5	0.3	0.3	0.2	0.2
	13.0	15.0	4.5	2.7	1.0	0.4	0.3	0.3	0.4
	15.0	13.0	5.4	2.6	1.1	0.5	0.2	0.3	0.4
1E	9.0	11.0	2.3	1.6	0.4	0.3	0.1	0.2	0.3
	11.0	13.0	4.1	2.3	1.2	0.3	0.2	0.2	0.2
	13.0	15.0	5.7	2.9	0.7	0.4	0.4	0.3	0.4
	15.0	13.0	3.0	2.6	0.9	0.5	0.3	0.3	0.5
1F	9.0	11.0	3.7	1.3	0.5	0.3	0.1	0.3	0.3
	11.0	13.0	5.0	2.7	0.8	0.4	0.2	0.4	0.3
	13.0	15.0	8.2	1.7	0.8	0.5	0.3	0.3	0.4
	15.0	13.0	4.8	3.2	1.2	0.5	0.4	0.5	0.3
1G	9.0	11.0	3.4	1.3	0.3	0.2	<0.1	0.2	0.2
	11.0	13.0	3.9	1.5	0.7	0.3	0.2	0.4	0.3
	13.0	15.0	4.2	2.4	0.8	0.3	0.4	0.4	0.2
	15.0	13.0	2.9	2.7	0.9	0.4	0.3	0.3	0.2
1H	9.0	11.0	4.6	1.7	0.7	0.4	0.2	0.3	0.2
	11.0	13.0	4.8	3.2	0.7	0.6	0.2	0.3	0.3
	13.0	15.0	8.1	4.6	1.3	0.6	0.4	0.4	0.4
	15.0	13.0	5.8	3.1	1.3	0.3	0.3	0.5	0.2
1I	9.0	11.0	3.6	1.6	0.6	0.3	0.2	0.2	0.2
	11.0	13.0	5.2	1.9	0.9	0.3	0.2	0.5	0.3
	13.0	15.0	6.3	2.6	0.8	0.5	0.4	0.3	0.3
	15.0	13.0	4.0	2.9	0.9	0.3	0.2	0.5	0.3

(CONTINUED)

TABLE 9 (CONCLUDED)

PLAN	TEST WAVE PERIOD SEC	TEST WAVE HEIGHT FT	WAVE HEIGHT, FT GAGE 8	GAGE 9	GAGE 10	GAGE 11	GAGE 12	GAGE 13	GAGE 14
1	9.0	11.0	<0.1	0.2	0.1	1.1	0.1	<0.1	2.6
	11.0	13.0	0.2	0.3	0.1	1.6	0.2	0.1	5.4
	13.0	15.0	0.2	0.2	0.2	3.0	0.3	0.2	5.7
	15.0	13.0	0.1	0.1	0.1	2.6	0.2	0.1	4.8
1A	9.0	11.0	<0.1	<0.1	0.1	3.4	<0.1	<0.1	5.0
	11.0	13.0	0.2	0.3	0.3	5.3	0.2	0.3	6.4
	13.0	15.0	0.4	0.2	0.4	5.9	0.2	0.3	7.7
	15.0	13.0	0.2	0.1	0.1	6.5	0.4	0.2	8.1
1B	9.0	11.0	0.1	0.1	0.1	4.5	0.1	<0.1	6.4
	11.0	13.0	0.2	0.1	0.2	5.4	0.2	0.1	7.0
	13.0	15.0	0.2	0.2	0.3	6.6	0.7	0.4	11.5
	15.0	13.0	0.2	0.1	0.2	7.3	0.5	0.4	9.9
1C	9.0	11.0	<0.1	0.2	0.1	4.5	0.1	0.1	6.3
	11.0	13.0	0.3	0.2	0.2	7.3	0.3	0.1	7.4
	15.0	15.0	0.2	0.3	0.3	8.7	0.3	0.3	11.3
		13.0	0.3	0.2	0.2	9.4	0.3	0.3	13.5
1D	9.0	11.0	0.7	0.3	0.3	3.9	0.5	0.2	7.1
	11.0	13.0	0.2	0.2	0.3	6.3	0.1	0.3	8.5
	13.0	15.0	0.4	0.2	0.3	9.6	0.4	0.3	12.6
	15.0	13.0	0.3	0.2	0.3	11.2	0.4	0.3	14.9
1E	9.0	11.0	0.4	0.3	0.2	4.9	0.3	0.2	7.9
	11.0	13.0	0.3	0.2	0.4	8.9	0.2	0.3	12.4
	13.0	15.0	0.3	0.2	0.3	9.7	0.3	0.2	17.8
	15.0	13.0	0.3	0.3	0.3	12.8	0.4	0.3	13.1
1F	9.0	11.0	0.3	0.3	0.3	8.1	0.3	0.2	11.6
	11.0	13.0	0.3	0.3	0.3	9.8	0.2	0.2	11.1
	13.0	15.0	0.3	0.3	0.3	11.1	0.2	0.2	18.6
	15.0	13.0	0.4	0.3	0.3	13.6	0.5	0.3	14.0
1G	9.0	11.0	0.2	0.2	0.1	5.1	0.2	<0.1	6.6
	11.0	13.0	0.3	0.2	0.4	7.0	0.2	0.2	7.3
	13.0	15.0	0.5	0.2	0.4	8.4	0.3	0.2	10.7
	15.0	13.0	0.3	0.2	0.2	8.8	0.3	0.2	15.6
1H	9.0	11.0	0.2	0.3	0.2	5.8	0.2	0.2	6.8
	11.0	13.0	0.3	0.4	0.3	6.4	0.3	0.2	8.9
	13.0	15.0	0.5	0.5	0.4	11.3	0.5	0.5	12.8
	15.0	13.0	0.3	0.2	0.3	9.1	0.5	0.2	14.4
1I	9.0	11.0	0.2	0.3	0.2	5.3	0.1	<0.1	6.7
	11.0	13.0	0.4	0.3	0.3	8.1	0.2	0.2	8.2
	13.0	15.0	0.2	0.2	0.3	10.1	0.2	0.3	12.9
	15.0	13.0	0.2	0.2	0.2	9.9	0.3	0.2	13.6

TABLE 10

WAVE HEIGHTS FOR PLAN 11, 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
NW	7.0	9.0	3.1	1.8	0.2	0.2	0.1
	9.0	9.0	4.1	0.9	0.6	0.5	0.1
		13.0	6.2	1.0	0.9	0.5	0.3
	11.0	9.0	4.3	1.5	0.6	0.2	<0.1
		15.0	8.2	2.3	1.0	0.2	0.1
	13.0	11.0	5.6	1.2	0.5	0.2	0.2
		17.0	12.3	2.8	0.9	0.4	0.4
	15.0	11.0	3.4	2.2	0.7	0.2	0.2
		17.0	12.5	3.1	1.3	0.5	0.4
	17.0	11.0	9.0	2.6	1.2	0.6	0.4
		15.0	17.8	5.7	2.3	0.8	0.5
	19.0	6.0	4.6	4.0	1.1	1.4	0.6
W	7.0	9.0	1.5	0.3	<0.1	<0.1	<0.1
	9.0	7.0	2.4	0.4	0.3	<0.1	<0.1
		11.0	3.6	1.6	0.6	0.3	0.2
	11.0	7.0	2.6	1.0	0.5	0.1	<0.1
		13.0	5.2	1.9	0.9	0.3	0.2
	13.0	7.0	2.0	0.9	0.3	0.2	0.2
		15.0	6.3	2.6	0.8	0.5	0.4
	15.0	7.0	1.7	1.2	0.2	0.2	0.2
		13.0	4.0	2.9	0.9	0.3	0.2
	17.0	5.0	1.1	0.6	0.2	0.2	0.1
		13.0	5.6	2.7	0.5	0.4	0.2
	19.0	5.0	2.0	0.9	0.3	0.5	0.1
SW	7.0	7.0	2.2	0.4	0.1	0.1	<0.1
	9.0	11.0	3.2	1.1	0.5	0.3	0.2
	11.0	11.0	3.9	3.3	2.0	0.6	0.3
	13.0	11.0	7.2	4.5	1.1	0.6	0.4
	15.0	9.0	4.9	3.6	1.6	0.4	0.3
	17.0	5.0	3.0	1.4	0.4	0.2	0.1
	19.0	7.0	4.9	1.9	0.9	0.4	0.2

(CONTINUED)

TABLE 10 (CONTINUED)

TEST WAVE			WAVE HEIGHT, FT				
DIRECTION	PERIOD SEC	HEIGHT FT	GAGE 6	GAGE 7	GAGE 8	GAGE 9	GAGE 10
NW	7.0	9.0	0.2	0.1	0.1	<0.1	<0.1
	9.0	9.0	0.2	0.1	0.2	0.3	0.2
		13.0	0.4	0.3	0.3	0.5	0.2
	11.0	9.0	0.2	0.1	0.1	0.1	0.2
		15.0	0.2	0.1	0.3	0.2	0.2
	13.0	11.0	0.2	0.1	0.2	0.2	0.2
		17.0	0.4	0.2	0.3	0.2	0.3
	15.0	11.0	0.2	0.1	0.3	0.1	0.2
		17.0	0.4	0.3	0.4	0.3	0.3
	17.0	11.0	0.4	0.4	0.4	0.2	0.2
		15.0	0.5	0.4	0.4	0.3	0.4
	19.0	6.0	0.7	0.3	0.2	0.4	0.2
W	7.0	9.0	<0.1	<0.1	<0.1	<0.1	<0.1
	9.0	7.0	0.2	<0.1	<0.1	0.2	0.1
		11.0	0.2	0.2	0.2	0.3	0.2
	11.0	7.0	0.1	0.2	0.1	0.2	0.1
		13.0	0.5	0.3	0.4	0.3	0.3
	13.0	7.0	<0.1	0.1	<0.1	<0.1	0.2
		15.0	0.3	0.3	0.2	0.2	0.3
	15.0	7.0	0.1	0.2	0.2	0.1	<0.1
		13.0	0.5	0.3	0.2	0.2	0.2
	17.0	5.0	<0.1	<0.1	0.2	<0.1	<0.1
		13.0	0.4	0.2	0.3	0.2	0.3
	19.0	5.0	0.2	<0.1	<0.1	<0.1	<0.1
SW	7.0	7.0	<0.1	<0.1	0.1	<0.1	<0.1
	9.0	11.0	0.2	0.1	0.1	0.2	0.2
	11.0	11.0	0.6	0.4	0.5	0.5	0.5
	13.0	11.0	0.4	0.3	0.3	0.4	0.4
	15.0	9.0	0.4	0.3	0.4	0.2	0.2
	17.0	5.0	0.2	<0.1	<0.1	0.1	0.1
	19.0	7.0	0.3	0.3	0.2	0.1	0.2

(CONTINUED)

TABLE 10 (CONCLUDED)

Test Wave: Direction	Test Wave: Period, sec	Test Wave: Height, ft	Wave Height, ft: Gage 11	Wave Height, ft: Gage 12	Wave Height, ft: Gage 13	Wave Height, ft: Gage 14
NW	7.0	9.0	1.6	<0.1	0.1	11.2
	9.0	9.0	2.5	0.2	0.1	10.8
		13.0	3.8	0.2	0.1	15.6
	11.0	9.0	4.7	<0.1	0.1	11.1
		15.0	6.2	0.2	0.2	19.1
	13.0	11.0	5.3	0.2	0.2	16.3
		17.0	9.6	0.2	0.4	26.9
	15.0	11.0	4.2	0.4	0.4	9.8
		17.0	9.8	0.7	0.5	23.6
	17.0	11.0	6.6	0.4	0.3	19.4
		15.0	9.7	0.7	0.5	29.3
	19.0	6.0	3.5	0.8	0.6	10.5
W	7.0	9.0	3.1	<0.1	<0.1	3.8
	9.0	7.0	3.2	<0.1	<0.1	3.1
		11.0	5.3	0.1	<0.1	6.7
	11.0	7.0	3.5	0.3	<0.1	4.0
		13.0	8.1	0.2	0.2	8.2
	13.0	7.0	3.3	<0.1	0.1	6.9
		15.0	10.1	0.2	0.3	12.9
	15.0	7.0	5.0	0.1	<0.1	5.3
		13.0	9.9	0.3	0.2	13.6
	17.0	5.0	3.3	0.2	<0.1	3.2
		13.0	12.0	0.3	0.2	10.3
	19.0	5.0	5.5	0.2	0.2	6.9
SW	7.0	7.0	4.8	<0.1	<0.1	1.9
	9.0	11.0	7.7	0.1	0.1	6.5
	11.0	11.0	10.0	0.5	0.4	6.7
	13.0	11.0	11.1	0.3	0.3	9.3
	15.0	9.0	8.2	0.4	0.3	6.9
	17.0	5.0	5.3	0.2	0.1	4.0
	19.0	7.0	10.0	0.2	0.1	5.7

TABLE 11

COMPARISON WAVE HEIGHTS FOR PLANS 1I-1L FOR TEST WAVES

FROM NORTHWEST DIRECTION, SWL = +5.4 FT

PLAN	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
1I	9.0	13.0	6.2	1.0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.3	0.5	0.2	3.8	0.2	0.1	15.6
	11.0	15.0	8.2	2.3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.3	0.2	0.2	6.2	0.2	0.2	19.1
	13.0	17.0	12.3	2.8	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.3	0.2	0.3	9.6	0.2	0.4	26.9
	15.0	17.0	12.5	3.1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.4	0.3	0.3	9.8	0.7	0.5	23.6
	17.0	15.0	17.8	5.7	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.4	0.3	0.4	9.7	0.7	0.5	29.3
1J	9.0	13.0	4.1	1.0	0.3	0.1	0.1	[illegible]	[illegible]	0.2	0.2	0.1	3.1	0.1	<0.1	15.4
	11.0	15.0	3.7	3.2	[illegible]	0.3	[illegible]	[illegible]	0.3	0.3	0.3	0.3	6.9	0.2	0.3	17.0
	13.0	17.0	9.3	3.3	1.1	0.5	0.4	[illegible]	0.3	0.4	0.3	0.4	7.7	0.3	0.3	23.0
	15.0	17.0	10.6	3.7	1.6	0.6	0.4	[illegible]	0.4	0.4	0.3	0.3	6.5	1.0	0.5	23.1
	17.0	15.0	16.0	8.0	2.2	[illegible]	1.0	[illegible]	0.5	[illegible]	0.5	0.5	8.9	1.4	0.9	18.7
1K	9.0	13.0	5.1	1.0	0.5	0.4	0.2	0.3	0.2	0.3	0.4	0.2	3.6	0.1	<0.1	8.8
	11.0	15.0	5.7	2.9	0.9	0.3	0.2	0.3	0.4	0.3	0.3	0.3	5.7	0.3	0.2	13.8
	13.0	17.0	6.8	4.0	1.1	0.5	0.4	0.4	0.4	0.3	0.3	0.4	7.5	0.3	0.3	22.3
	15.0	17.0	8.2	4.8	1.8	0.7	0.4	0.5	0.4	0.4	0.2	0.2	6.1	0.8	0.5	23.6
	17.0	15.0	17.3	7.9	2.0	2.4	0.9	1.2	0.6	0.5	0.6	0.5	9.4	1.5	0.5	16.9
1L	9.0	13.0	3.8	1.1	0.4	0.4	0.2	0.3	0.1	0.2	0.2	0.2	3.1	0.1	<0.1	C.9
	11.0	15.0	6.3	2.5	0.5	0.3	0.1	0.3	0.3	0.3	0.3	0.2	5.5	0.2	0.1	11.1
	13.0	17.0	10.2	3.6	1.3	0.8	0.7	0.6	0.5	0.4	0.3	0.5	7.9	0.3	0.4	11.9
	15.0	17.0	7.4	3.3	1.5	0.6	0.5	0.6	0.3	0.5	0.2	0.3	6.3	0.7	0.5	16.7
	17.0	15.0	13.2	7.9	1.9	2.3	1.0	1.1	0.6	0.6	0.6	0.5	8.1	1.3	1.0	14.6

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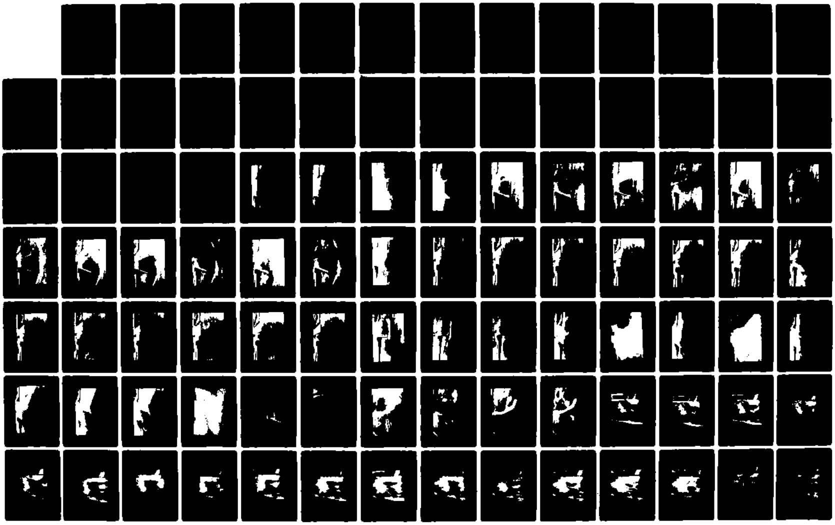

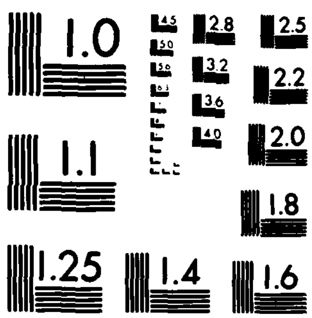

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TABLE 12

COMPARISON WAVE HEIGHTS FOR RETESTING PLANS 1I-1L FOR TEST WAVES

FROM NORTHWEST DIRECTION, SWL = +5.4 FT

PLAN	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
1I	9.0	9.0	4.1	0.9	0.6	0.5	0.1	0.2	0.1	0.2	0.3	0.2	2.5	0.2	0.1	10.8
	11.0	9.0	4.3	1.5	0.6	0.2	<0.1	0.2	0.1	0.1	0.1	0.2	4.7	<0.1	0.1	11.1
	13.0	11.0	5.6	1.2	0.5	0.2	0.2	0.2	0.1	0.2	0.2	0.2	5.3	0.2	0.2	16.3
	15.0	11.0	3.4	2.2	0.7	0.2	0.2	0.2	0.1	0.3	0.1	0.2	4.2	0.4	0.4	9.8
	17.0	11.0	9.0	2.6	1.2	0.6	0.4	0.4	0.4	0.4	0.2	0.2	6.6	0.4	0.3	19.4
1J	9.0	9.0	3.4	0.8	0.4	0.1	0.1	0.3	0.1	0.1	0.3	0.3	1.5	<0.1	0.1	10.1
	11.0	9.0	2.6	2.2	0.8	0.2	<0.1	0.2	0.3	0.2	0.2	0.2	3.1	0.1	0.2	11.4
	13.0	11.0	4.3	1.7	0.8	0.6	0.4	0.2	0.3	0.2	0.1	0.4	6.0	0.2	0.2	13.1
	15.0	11.0	2.3	2.0	0.9	0.3	0.3	0.4	0.1	0.3	0.1	0.2	3.7	0.3	0.2	9.8
	17.0	11.0	9.9	6.1	1.8	2.0	0.7	0.9	0.4	0.3	0.5	0.3	6.5	1.0	0.3	18.2
1K	9.0	9.0	1.5	0.8	0.4	0.2	0.2	0.3	0.2	0.2	0.3	0.3	1.4	<0.1	<0.1	7.6
	11.0	9.0	1.7	1.5	0.6	0.2	0.1	0.3	0.3	0.2	0.2	0.2	2.6	0.1	0.2	9.7
	13.0	11.0	4.1	1.4	0.8	0.4	0.3	0.3	0.3	0.3	0.2	0.4	6.0	0.2	0.2	15.5
	15.0	11.0	1.7	2.1	0.9	0.4	0.3	0.3	0.2	0.3	0.2	0.2	4.2	0.2	0.2	9.5
	17.0	11.0	7.7	4.9	1.8	2.2	0.7	0.9	0.4	0.4	0.5	0.3	6.4	1.1	0.4	16.8
1L	9.0	9.0	1.4	0.8	0.3	0.2	0.1	0.3	0.1	0.1	0.3	0.2	2.0	<0.1	<0.1	6.3
	11.0	9.0	1.7	1.3	0.6	0.1	<0.1	0.2	0.2	0.1	0.1	0.2	3.6	<0.1	0.1	7.3
	13.0	11.0	3.1	2.7	0.8	0.3	0.4	0.3	0.2	0.3	0.2	0.4	5.5	0.2	0.3	10.4
	15.0	11.0	2.3	1.9	0.5	0.3	0.2	0.1	0.2	0.1	0.1	0.1	3.9	0.4	0.4	8.4
	17.0	11.0	7.7	4.4	2.1	1.3	0.6	0.6	0.4	0.4	0.3	0.3	6.3	0.9	1.1	12.8

TABLE 13

WAVE HEIGHTS FOR PLAN 1M FOR TEST WAVES

FROM DIRECTIONS NORTHWEST AND SOUTHWEST, 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
NW	7.0	9.0	1.3	0.7	<0.1	<0.1	<0.1	<0.1	<0.1
	9.0	9.0	1.2	0.6	0.2	0.2	<0.1	0.2	<0.1
		13.0	2.5	0.8	0.4	0.3	0.1	0.2	0.1
	11.0	9.0	1.4	1.0	0.3	0.1	<0.1	0.1	0.2
		15.0	3.6	1.5	0.5	0.2	0.1	0.3	0.2
	13.0	11.0	2.5	1.6	0.7	0.5	0.4	0.2	0.3
		17.0	5.4	2.6	1.0	0.4	0.4	0.3	0.3
	15.0	11.0	2.4	1.3	0.4	0.3	0.3	0.3	0.2
		17.0	7.2	3.0	1.2	0.5	0.4	0.5	0.3
	17.0	11.0	7.6	3.8	1.8	1.4	0.7	0.7	0.4
		15.0	11.7	6.4	2.2	2.4	1.0	1.1	0.5
	19.0	6.0	1.0	1.1	0.3	0.3	0.1	0.2	0.1
SW	7.0	7.0	1.1	0.4	<0.1	0.1	<0.1	<0.1	<0.1
	9.0	11.0	1.7	0.5	0.2	0.1	<0.1	0.1	0.1
	11.0	11.0	3.4	2.0	1.3	0.4	0.2	0.2	0.2
	13.0	11.0	5.7	1.8	0.5	0.2	0.3	0.2	0.2
	15.0	9.0	3.1	1.4	0.8	0.2	0.2	0.2	0.2
	17.0	5.0	1.2	0.5	0.3	0.3	0.1	<0.1	<0.1
	19.0	7.0	2.4	1.4	0.8	0.4	0.2	0.2	0.2

(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
NW	7.0	9.0	<0.1	<0.1	<0.1	1.4	<0.1	<0.1	6.1
	9.0	9.0	0.1	0.2	0.1	1.8	<0.1	<0.1	6.1
		13.0	0.2	0.2	0.2	2.0	0.1	<0.1	8.5
	11.0	9.0	0.1	0.1	0.2	2.9	0.1	0.1	7.6
		15.0	0.2	0.1	0.3	4.3	0.2	0.1	10.9
	13.0	11.0	0.2	0.1	0.4	4.1	0.1	0.3	9.6
		17.0	0.3	0.2	0.5	5.7	0.2	0.3	16.7
	15.0	11.0	0.3	<0.1	0.2	3.7	0.3	0.1	9.7
		17.0	0.4	0.2	0.3	6.0	0.5	0.4	20.6
	17.0	11.0	0.3	0.4	0.3	6.6	1.0	0.8	14.3
		15.0	0.4	0.6	0.5	7.9	1.3	0.7	16.8
	19.0	6.0	0.2	0.1	<0.1	2.7	0.4	0.1	4.7
SW	7.0	7.0	<0.1	<0.1	<0.1	4.1	<0.1	<0.1	1.7
	9.0	11.0	0.1	<0.1	<0.1	5.4	<0.1	<0.1	1.9
	11.0	11.0	0.3	0.2	0.4	10.0	0.2	0.3	2.4
	13.0	11.0	0.2	0.1	0.4	7.7	0.2	0.2	3.0
	15.0	9.0	0.1	<0.1	0.2	8.1	0.3	0.1	3.1
	17.0	5.0	<0.1	<0.1	<0.1	5.4	0.3	<0.1	1.7
	19.0	7.0	0.2	<0.1	0.1	6.6	0.4	0.1	2.2

TABLE 14

COMPARISON WAVE HEIGHTS FOR PLANS 1N-1P FOR TEST WAVES FROM DIRECTIONS SOUTHWEST AND NORTHWEST, SWL = +5.4 FT

PLAN	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
			SOUTHWEST DIRECTION						
1N	7.0	7.0	1.4	0.5	0.1	<0.1	0.1	<0.1	<0.1
	9.0	11.0	3.2	1.1	0.4	0.2	<0.1	0.2	0.2
	11.0	11.0	5.2	2.5	1.6	0.4	0.2	0.4	0.3
	13.0	4.0	1.4	0.9	0.2	0.1	<0.1	<0.1	<0.1
		11.0	7.8	2.9	0.9	0.4	0.3	0.3	0.5
	15.0	4.0	1.8	0.3	0.2	<0.1	<0.1	<0.1	<0.1
		9.0	4.3	2.3	1.2	0.3	0.2	0.3	0.3
	17.0	5.0	1.2	0.5	0.3	0.2	<0.1	0.1	<0.1
	19.0	7.0	3.8	2.1	0.8	0.5	0.2	0.3	0.3
1O	7.0	7.0	1.6	0.4	<0.1	<0.1	<0.1	<0.1	<0.1
	9.0	11.0	3.0	0.6	0.5	0.3	0.2	0.2	0.2
	11.0	11.0	5.5	2.5	1.5	0.6	0.2	0.3	0.3
	13.0	11.0	8.1	3.4	0.7	0.5	0.4	0.4	0.3
		4.0	1.4	0.6	<0.1	<0.1	0.1	<0.1	<0.1
	15.0	4.0	1.5	0.4	0.2	0.1	<0.1	<0.1	<0.1
		9.0	4.1	1.3	0.9	0.4	0.2	0.2	0.2
	17.0	5.0	2.6	0.7	0.4	0.4	0.2	0.2	<0.1
	19.0	7.0	6.3	2.8	1.1	0.5	0.3	0.4	0.5
1P	7.0	7.0	0.8	0.3	0.1	0.1	<0.1	<0.1	<0.1
	9.0	11.0	2.1	0.7	0.4	0.1	0.1	0.2	0.1
	11.0	11.0	3.6	2.1	1.3	0.4	0.2	0.2	0.3
	13.0	4.0	0.9	1.1	0.3	0.1	<0.1	<0.1	0.1
		11.0	5.8	3.1	1.3	0.5	0.3	0.3	0.3
	15.0	4.0	1.3	0.4	0.2	<0.1	<0.1	<0.1	<0.1
		9.0	3.7	1.8	0.7	0.2	0.2	0.2	0.1
	17.0	5.0	1.2	0.7	0.4	0.2	0.1	0.1	0.1
	19.0	7.0	5.3	1.8	1.0	0.4	0.2	0.3	0.4
			NORTHWEST DIRECTION						
1P	7.0	9.0	1.2	0.8	<0.1	<0.1	<0.1	<0.1	<0.1
	9.0	9.0	1.1	0.6	0.2	0.2	<0.1	0.2	<0.1
		13.0	3.2	0.8	0.5	0.3	0.1	0.2	0.1
	11.0	4.0	0.4	0.9	0.5	<0.1	<0.1	0.1	<0.1
		9.0	1.4	1.6	0.7	0.1	0.1	0.2	0.1
		15.0	5.5	3.0	1.3	0.2	0.1	0.2	0.2
	13.0	4.0	0.3	1.0	0.3	0.3	0.2	<0.1	0.1
		11.0	4.0	1.6	0.9	0.5	0.4	0.3	0.3
		17.0	8.5	3.4	1.2	0.7	0.6	0.4	0.5
	15.0	4.0	0.4	0.9	0.4	<0.1	<0.1	<0.1	<0.1
		11.0	2.8	2.0	0.9	0.3	0.3	0.4	0.2
		17.0	8.5	4.3	1.9	0.8	0.5	0.4	0.6
	17.0	11.0	7.4	5.0	1.8	2.0	0.8	1.0	0.5
		15.0	11.0	6.9	1.9	2.9	0.8	1.2	0.4
	19.0	6.0	2.0	1.2	0.4	0.3	0.1	0.1	0.1

(CONTINUED)

TABLE 14 (CONCLUDED)

PLAN	TEST WAVE PERIOD SEC	TEST WAVE HEIGHT FT	WAVE HEIGHT, FT GAGE 8	GAGE 9	GAGE 10	GAGE 11	GAGE 12	GAGE 13	GAGE 14
			SOUTHWEST DIRECTION						
1N	7.0	7.0	<0.1	<0.1	<0.1	4.3	<0.1	<0.1	1.3
	9.0	11.0	0.1	0.3	0.2	7.2	0.2	0.1	1.7
	11.0	11.0	0.4	0.4	0.4	11.0	0.2	0.4	2.1
	13.0	4.0	<0.1	<0.1	0.1	3.1	<0.1	0.1	1.7
		11.0	0.3	0.2	0.2	10.4	0.3	0.3	3.4
	15.0	4.0	<0.1	<0.1	<0.1	3.6	<0.1	<0.1	1.9
		9.0	0.2	0.2	0.2	9.0	0.2	0.2	3.1
	17.0	5.0	<0.1	<0.1	<0.1	5.2	0.2	0.1	1.5
	19.0	7.0	0.2	<0.1	0.1	8.5	0.2	0.1	2.7
1O	7.0	7.0	<0.1	<0.1	<0.1	3.5	<0.1	<0.1	1.6
	9.0	11.0	0.1	0.3	0.2	9.7	0.2	0.1	2.2
	11.0	11.0	0.4	0.4	0.5	10.2	0.3	0.4	2.3
	13.0	11.0	0.3	0.3	0.4	10.0	0.3	0.3	3.7
		4.0	<0.1	<0.1	0.1	1.5	<0.1	0.1	0.9
	15.0	4.0	<0.1	<0.1	<0.1	2.6	<0.1	<0.1	2.4
		9.0	0.2	0.2	0.2	5.5	0.2	0.1	3.0
	17.0	5.0	<0.1	0.1	<0.1	6.5	0.3	0.1	1.9
	19.0	7.0	0.3	0.2	0.2	10.5	0.2	0.2	2.5
1P	7.0	7.0	<0.1	<0.1	<0.1	2.5	<0.1	<0.1	1.2
	9.0	11.0	0.1	0.2	0.1	5.7	0.1	<0.1	1.8
	11.0	11.0	0.3	0.4	0.4	7.3	0.3	0.3	2.1
	13.0	4.0	<0.1	0.1	0.1	2.8	<0.1	0.1	1.3
		11.0	0.4	0.3	0.3	7.1	0.5	0.2	2.9
	15.0	4.0	<0.1	<0.1	<0.1	1.9	<0.1	<0.1	1.7
		9.0	0.1	0.1	0.1	5.9	0.2	0.1	2.7
	17.0	5.0	<0.1	0.1	<0.1	4.1	0.2	0.4	1.6
	19.0	7.0	0.2	<0.1	0.1	8.5	0.2	0.1	2.3
			NORTHWEST DIRECTION						
1P	7.0	9.0	<0.1	<0.1	<0.1	1.2	<0.1	<0.1	5.3
	9.0	9.0	0.1	0.2	0.1	0.9	<0.1	<0.1	6.5
		13.0	0.2	0.1	0.2	1.9	<0.1	<0.1	9.4
	11.0	4.0	<0.1	0.1	<0.1	0.9	<0.1	0.1	3.4
		9.0	0.1	0.2	0.2	1.3	0.2	0.2	7.7
		15.0	0.2	0.2	0.2	3.0	0.2	0.3	12.8
	13.0	4.0	<0.1	<0.1	0.2	1.0	<0.1	0.2	2.9
		11.0	0.2	0.2	0.3	2.7	0.2	0.2	12.0
		17.0	0.5	0.3	0.5	4.9	0.3	0.3	23.3
	15.0	4.0	<0.1	<0.1	<0.1	0.6	0.2	<0.1	2.2
		11.0	0.3	0.1	0.2	2.6	0.2	0.2	10.1
		17.0	0.5	0.3	0.3	4.2	0.6	0.5	20.2
	17.0	11.0	0.4	0.5	0.4	6.1	1.1	0.4	14.1
		15.0	0.4	0.5	0.4	7.6	1.1	0.9	15.6
	19.0	6.0	0.2	<0.1	<0.1	2.6	0.3	0.1	3.4

TABLE 15

WAVE HEIGHTS FOR PLAN 1P FOR TEST WAVES

FROM DIRECTIONS NORTHWEST AND SOUTHWEST 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
NW	7.0	6.0	0.8	0.5	<0.1	<0.1	<0.1	<0.1	<0.1
	9.0	6.0	0.7	0.5	0.3	0.2	0.1	0.1	0.1
	11.0	6.0	0.7	0.6	0.5	<0.1	<0.1	0.2	0.1
	13.0	6.0	0.7	1.4	0.4	0.2	0.2	0.1	0.1
	15.0	6.0	0.9	1.2	0.5	0.1	0.1	<0.1	<0.1
	17.0	6.0	1.3	2.4	0.8	0.7	0.4	0.2	0.2
SW	7.0	6.0	0.9	0.7	<0.1	<0.1	<0.1	<0.1	<0.1
	9.0	6.0	0.8	0.6	0.5	0.2	0.1	0.1	<0.1
	11.0	6.0	1.0	1.2	0.8	0.2	0.1	0.2	0.2
	13.0	6.0	1.2	1.0	0.3	0.2	0.2	0.1	0.1
	15.0	6.0	1.7	0.9	0.3	0.1	0.2	0.1	<0.1
	17.0	6.0	1.7	0.7	0.4	0.4	0.2	0.2	<0.1
	19.0	6.0	4.4	1.5	0.6	0.2	0.2	0.2	0.2

(CONTINUED)

TABLE 15 (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
NW	7.0	6.0	<0.1	<0.1	<0.1	0.5	<0.1	<0.1	3.3
	9.0	6.0	<0.1	<0.1	0.1	0.8	0.1	<0.1	3.4
	11.0	6.0	<0.1	0.1	0.1	1.4	<0.1	0.2	4.6
	13.0	6.0	<0.1	<0.1	0.2	1.3	<0.1	0.1	3.9
	15.0	6.0	0.1	<0.1	<0.1	0.9	0.3	0.1	3.5
	17.0	6.0	0.1	0.2	0.1	2.4	0.5	0.6	7.4
SW	7.0	6.0	<0.1	<0.1	<0.1	2.5	<0.1	<0.1	1.1
	9.0	6.0	<0.1	<0.1	0.2	2.6	0.1	0.1	1.0
	11.0	6.0	0.2	0.1	0.2	3.0	0.1	0.2	1.4
	13.0	6.0	<0.1	<0.1	0.2	3.5	<0.1	0.2	1.4
	15.0	6.0	<0.1	<0.1	<0.1	3.2	0.1	<0.1	1.7
	17.0	6.0	<0.1	0.1	<0.1	4.9	0.3	<0.1	1.6
	19.0	6.0	0.2	0.1	0.1	7.9	0.3	0.1	2.3

TABLE 16

COMPARISON WAVE HEIGHTS FOR PLANS 2,2A AND 2D FOR TEST WAVES

FROM SOUTHWEST DIRECTION, SWL = +5.4 FT

PLAN	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
2	7.0	6.0	0.5	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	0.5	<0.1	<0.1	0.5
	9.0	6.0	0.8	0.3	0.4	0.2	<0.1	0.1	0.1	<0.1	0.2	0.1	1.5	0.1	<0.1	0.8
		11.0	1.4	0.7	0.3	0.3	<0.1	0.2	0.2	0.2	0.2	0.2	1.9	0.1	<0.1	1.5
	11.0	6.0	1.4	0.8	0.5	0.1	<0.1	0.1	0.2	0.1	<0.1	0.1	0.9	<0.1	0.2	0.5
		11.0	4.4	1.5	0.7	0.2	0.1	0.2	0.2	0.2	0.2	0.2	5.1	0.1	0.2	2.1
	13.0	6.0	1.6	1.0	0.4	0.1	0.1	0.2	0.2	0.2	0.1	0.1	2.1	0.2	0.1	0.8
		11.0	5.8	2.3	0.7	0.4	0.2	0.3	0.3	0.2	0.3	0.2	6.5	0.3	0.2	2.4
	15.0	6.0	1.5	0.8	0.5	0.2	0.1	<0.1	<0.1	0.1	0.1	<0.1	2.7	<0.1	<0.1	0.6
		9.0	3.2	1.4	0.7	0.2	0.1	0.2	0.3	0.1	0.1	0.2	3.5	0.1	<0.1	1.6
	17.0	6.0	1.9	0.9	0.4	0.4	0.2	0.2	<0.1	<0.1	0.1	0.1	2.4	0.2	0.1	1.3
	19.0	6.0	2.1	1.0	0.4	0.2	0.1	0.2	0.2	0.1	<0.1	<0.1	2.9	0.2	<0.1	0.7
2A	13.0	6.0	1.4	0.8	0.5	0.2	<0.1	<0.1	0.1	<0.1	<0.1	<0.1	1.5	0.2	<0.1	0.8
	17.0	6.0	2.0	0.7	0.3	0.2	0.1	0.1	0.1	<0.1	<0.1	<0.1	1.6	0.2	0.3	1.4
	19.0	6.0	1.9	0.9	0.3	0.2	<0.1	0.1	0.1	0.2	0.1	0.1	2.2	0.3	0.1	1.0
2D	13.0	6.0	1.2	0.9	0.3	<0.1	0.2	<0.1	<0.1	0.2	<0.1	0.2	2.1	0.1	0.1	0.9
	15.0	6.0	1.1	0.4	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	1.2	<0.1	<0.1	1.4
	17.0	6.0	1.6	0.7	0.3	0.3	0.1	0.1	<0.1	<0.1	<0.1	<0.1	1.8	0.3	0.2	1.6
	19.0	6.0	1.8	0.7	0.2	0.1	0.1	0.1	0.2	0.1	<0.1	<0.1	1.7	0.1	<0.1	1.3

TABLE 17

COMPARISON WAVE HEIGHTS FOR PLANS 2A-2C FOR TEST WAVES

FROM NORTHWEST DIRECTION, SWL = +5.4 FT

PLAN	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
2A	7.0	6.0	0.6	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	0.3	<0.1	<0.1	1.4
		9.0	0.9	0.5	<0.1	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	0.7	<0.1	<0.1	2.8
	9.0	6.0	1.8	0.6	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	0.9	<0.1	<0.1	3.6
		9.0	2.5	0.6	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	1.1	<0.1	<0.1	3.6
		13.0	2.6	0.5	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	0.1	<0.1	1.9	<0.1	<0.1	6.2
	11.0	6.0	2.1	0.4	0.3	0.1	<0.1	<0.1	0.1	0.3	<0.1	0.1	1.6	0.2	<0.1	3.8
		9.0	2.5	0.5	0.4	0.2	0.1	0.1	0.2	0.3	0.1	0.1	1.8	0.1	<0.1	5.0
		15.0	4.0	1.3	0.7	0.2	0.1	0.2	0.3	0.2	0.2	0.1	2.5	<0.1	0.1	9.8
	13.0	6.0	2.0	1.0	0.3	<0.1	0.2	0.1	<0.1	0.1	<0.1	<0.1	1.4	0.1	<0.1	3.4
		11.0	3.6	1.3	0.5	0.2	0.2	0.2	0.2	0.1	0.1	0.1	2.4	<0.1	<0.1	8.9
		17.0	7.2	2.7	1.0	0.5	0.5	0.3	0.3	0.3	0.2	0.5	3.4	0.2	0.3	10.0
	15.0	6.0	2.3	1.0	0.5	<0.1	<0.1	<0.1	0.1	<0.1	<0.1	<0.1	1.2	0.1	0.1	4.1
		11.0	3.6	1.6	0.5	0.2	0.2	<0.1	0.2	0.1	<0.1	0.1	2.2	0.2	0.2	8.4
		17.0	5.4	2.3	0.8	0.3	0.3	0.3	0.2	0.3	0.1	0.2	4.6	0.4	0.3	12.7
	17.0	6.0	1.8	1.1	0.4	0.3	0.2	0.2	<0.1	0.2	<0.1	<0.1	1.7	0.2	0.2	4.3
		11.0	4.2	2.2	0.8	0.6	0.4	0.4	0.2	0.4	0.2	0.2	3.9	0.4	0.4	9.4
		15.0	7.4	2.9	1.0	0.7	0.4	0.5	0.3	0.3	0.3	0.3	4.9	0.5	0.6	13.0
	1[illegible].0	6.0	1.4	0.5	0.2	0.2	<0.1	<0.1	0.1	0.1	<0.1	<0.1	1.3	0.2	<0.1	2.2
2B	11.0	6.0	1.7	0.2	0.3	<0.1	<0.1	<0.1	0.1	0.2	0.1	0.1	1.4	0.2	<0.1	2.9
	13.0	6.0	1.5	1.1	0.2	<0.1	0.1	0.1	<0.1	0.1	<0.1	<0.1	1.7	<0.1	<0.1	2.2
	15.0	6.0	1.9	1.0	0.7	0.1	<0.1	<0.1	0.1	0.1	<0.1	<0.1	1.3	0.2	0.2	2.4
2C	11.0	6.0	0.6	0.3	0.4	<0.1	<0.1	<0.1	0.2	0.2	<0.1	0.1	0.7	0.1	<0.1	0.9
	13.0	6.0	0.8	0.8	0.1	0.1	<0.1	<0.1	0.1	0.2	<0.1	0.2	1.6	0.1	0.1	1.0
	15.0	6.0	1.2	0.8	0.5	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	1.1	0.2	0.2	1.9

TABLE 18

WAVE HEIGHTS FOR PLAN 3 FOR TEST WAVES FROM

DIRECTIONS NORTHWEST AND SOUTHWEST, 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
NW	7.0	6.0	1.1	0.2	<0.1	<0.1	<0.1	<0.1	<0.1
		9.0	1.3	0.3	<0.1	<0.1	<0.1	<0.1	<0.1
	9.0	6.0	0.8	0.5	0.2	<0.1	<0.1	0.1	<0.1
		9.0	0.8	0.5	0.2	<0.1	<0.1	0.1	<0.1
		13.0	0.8	0.5	0.2	<0.1	<0.1	0.1	<0.1
	11.0	6.0	0.7	0.1	0.3	0.1	<0.1	0.1	0.2
		9.0	1.4	0.2	0.2	<0.1	<0.1	<0.1	0.1
		15.0	2.7	0.8	0.4	0.2	0.1	0.2	0.2
	13.0	6.0	0.8	0.7	<0.1	<0.1	<0.1	<0.1	<0.1
		11.0	3.0	1.5	0.4	0.1	0.1	0.1	<0.1
		17.0	5.9	3.4	1.0	0.5	0.5	0.3	0.3
	15.0	6.0	1.0	0.5	0.3	0.1	<0.1	<0.1	<0.1
		11.0	2.7	1.6	0.7	0.2	0.1	0.1	0.1
		17.0	7.5	3.4	1.5	0.4	0.3	0.4	0.3
	17.0	6.0	1.6	0.4	0.1	0.1	0.1	0.1	<0.1
		11.0	5.4	2.7	0.8	0.7	0.5	0.4	0.3
		15.0	9.4	2.7	1.1	1.1	0.5	0.6	0.3
	19.0	6.0	1.5	0.4	0.1	0.1	<0.1	<0.1	<0.1
SW	7.0	6.0	0.3	0.3	<0.1	<0.1	<0.1	<0.1	<0.1
	9.0	6.0	0.3	0.3	0.1	<0.1	<0.1	<0.1	<0.1
		11.0	1.1	0.3	0.1	0.1	<0.1	0.1	0.1
	11.0	6.0	0.4	0.5	0.2	<0.1	<0.1	<0.1	<0.1
	13.0	11.0	1.5	0.7	0.4	0.1	0.1	0.1	<0.1
		6.0	0.5	0.5	0.2	<0.1	0.1	<0.1	<0.1
		11.0	3.4	1.4	0.6	0.2	0.2	0.2	0.2
	15.0	6.0	0.8	0.4	0.2	<0.1	<0.1	<0.1	<0.1
		9.0	1.7	0.6	0.4	0.2	0.1	0.2	0.2
	17.0	6.0	0.9	0.6	0.3	0.1	<0.1	<0.1	<0.1
	19.0	6.0	1.3	0.8	0.4	0.2	<0.1	0.1	<0.1

(CONTINUED)

TABLE 18 (CONCLUDED)

Test Wave: Direction	Test Wave: Period, sec	Test Wave: Height, ft	Wave Height, ft: Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 14
NW	7.0	6.0	<0.1	<0.1	<0.1	0.9	<0.1	<0.1	1.2
		9.0	<0.1	<0.1	<0.1	1.0	<0.1	<0.1	1.3
	9.0	6.0	<0.1	0.1	0.1	1.0	<0.1	<0.1	1.8
		9.0	<0.1	0.1	<0.1	0.9	<0.1	<0.1	2.8
		13.0	<0.1	0.1	0.1	0.8	<0.1	<0.1	2.6
	11.0	6.0	0.2	0.1	<0.1	0.5	<0.1	<0.1	1.6
		9.0	0.2	<0.1	<0.1	1.0	0.1	<0.1	3.1
		15.0	0.2	0.2	0.2	1.6	0.1	0.1	4.6
	13.0	6.0	<0.1	<0.1	<0.1	0.6	<0.1	<0.1	1.4
		11.0	0.1	0.1	0.1	1.5	0.1	0.1	4.9
		17.0	0.3	0.3	0.4	2.3	0.3	0.4	7.6
	15.0	6.0	0.1	<0.1	<0.1	0.7	0.1	<0.1	2.0
		11.0	0.1	<0.1	0.1	1.5	0.2	0.2	3.8
		17.0	0.3	0.2	0.2	1.7	0.7	0.4	8.0
	17.0	6.0	<0.1	<0.1	<0.1	0.7	<0.1	0.1	1.3
		11.0	0.2	0.2	0.2	1.8	0.3	0.4	7.7
		15.0	0.3	0.4	0.3	2.9	0.7	0.4	8.5
	19.0	6.0	0.1	<0.1	<0.1	1.4	0.1	<0.1	1.9
SW	7.0	6.0	<0.1	<0.1	<0.1	0.8	<0.1	<0.1	0.5
	9.0	6.0	<0.1	<0.1	<0.1	0.7	<0.1	<0.1	0.4
		11.0	0.1	0.2	<0.1	2.1	0.1	<0.1	1.0
	11.0	6.0	<0.1	<0.1	<0.1	0.7	<0.1	<0.1	0.8
	13.0	11.0	0.2	0.1	0.2	2.5	0.1	0.1	0.8
		6.0	<0.1	<0.1	<0.1	1.3	<0.1	0.1	0.7
		11.0	0.2	0.2	0.2	4.0	0.3	0.2	1.9
	15.0	6.0	<0.1	<0.1	<0.1	1.4	<0.1	<0.1	1.0
		9.0	0.1	0.1	0.1	2.3	0.2	0.1	1.8
	17.0	6.0	<0.1	<0.1	<0.1	1.3	0.1	<0.1	1.0
	19.0	6.0	0.1	<0.1	0.1	1.5	0.2	<0.1	0.8

TABLE 19

COMPARISON WAVE HEIGHTS FOR PLANS 3A-3G FOR TEST WAVES

FROM DIRECTIONS NORTHWEST AND SOUTHWEST, SWL = +5.4 FT

PLAN	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
								NORTHWEST DIRECTION								
3A	7.0	6.0	0.6	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	0.5	<0.1	<0.1	1.8
	15.0	6.0	1.3	0.7	0.3	0.2	0.2	0.2	<0.1	0.2	<0.1	<0.1	0.8	<0.1	<0.1	3.0
	17.0	6.0	1.4	0.6	0.2	0.2	0.2	0.1	<0.1	<0.1	<0.1	<0.1	0.8	0.1	0.2	2.6
	19.0	6.0	1.1	0.6	0.1	0.1	<0.1	<0.1	<0.1	0.1	<0.1	<0.1	1.1	0.2	<0.1	1.8
3B	7.0	6.0	1.0	0.3	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	0.4	<0.1	<0.1	2.5
	15.0	6.0	1.2	0.7	0.4	0.1	<0.1	<0.1	0.1	0.1	<0.1	<0.1	0.9	0.1	<0.1	3.5
	17.0	6.0	1.5	0.9	0.4	0.2	0.2	0.1	<0.1	0.1	0.1	<0.1	1.0	0.2	0.2	4.6
	19.0	6.0	1.3	0.8	0.2	0.2	<0.1	<0.1	0.1	0.2	<0.1	<0.1	0.7	0.3	<0.1	2.2
3C	7.0	6.0	0.9	0.4	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	0.7	<0.1	<0.1	4.6
	15.0	6.0	1.1	0.7	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	0.7	<0.1	<0.1	4.2
	17.0	6.0	1.5	1.3	0.5	0.2	0.1	0.1	0.1	0.1	0.1	<0.1	0.7	0.2	0.2	6.1
	19.0	6.0	0.8	0.8	0.2	0.2	<0.1	<0.1	0.1	0.2	<0.1	<0.1	0.8	0.3	<0.1	2.7
3D	7.0	6.0	1.4	0.5	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	0.6	<0.1	<0.1	6.5
	15.0	6.0	1.4	1.0	0.3	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	1.0	0.2	0.2	6.6
	17.0	6.0	2.1	1.6	0.6	0.3	0.2	0.2	<0.1	0.1	0.1	<0.1	1.5	0.3	0.3	10.6
	19.0	6.0	0.6	1.0	0.2	0.2	<0.1	<0.1	0.1	0.2	0.1	<0.1	1.1	0.4	<0.1	3.5
								SOUTHWEST DIRECTION								
3E	15.0	6.0	0.5	0.3	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	0.9	<0.1	<0.1	1.5
	17.0	6.0	1.0	0.6	0.3	0.2	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	1.1	0.1	0.1	2.8
	19.0	6.0	1.2	0.8	0.3	0.1	<0.1	0.1	<0.1	0.1	<0.1	0.1	1.5	0.2	<0.1	2.0
3F	15.0	6.0	0.5	0.4	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	1.5	<0.1	<0.1	1.8
	17.0	6.0	1.1	0.6	0.3	0.2	0.2	0.1	<0.1	<0.1	<0.1	<0.1	2.2	0.1	0.2	2.8
	19.0	6.0	1.4	0.8	0.3	0.1	<0.1	0.1	0.2	<0.1	<0.1	<0.1	2.1	0.1	<0.1	1.4
3G	15.0	6.0	0.6	0.4	0.3	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	2.1	<0.1	<0.1	1.6
	17.0	6.0	0.9	0.6	0.3	0.5	0.2	0.2	<0.1	<0.1	0.1	<0.1	3.4	0.3	0.2	2.5
	19.0	6.0	1.7	0.8	0.4	0.2	<0.1	0.1	0.1	0.2	<0.1	0.1	4.8	0.4	<0.1	1.9

TABLE 20

WAVE HEIGHTS FOR PLAN 3F FOR TEST WAVES FROM

DIRECTIONS NORTHWEST AND SOUTHWEST, 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
NW	7.0	6.0	0.9	0.3	<0.1	<0.1	<0.1	<0.1	<0.1
		9.0	0.8	0.5	<0.1	<0.1	<0.1	<0.1	<0.1
	9.0	6.0	0.9	0.6	0.1	<0.1	<0.1	<0.1	<0.1
		9.0	0.9	0.6	0.2	<0.1	<0.1	<0.1	<0.1
		13.0	2.3	0.8	0.2	0.1	<0.1	0.1	<0.1
	11.0	6.0	1.0	0.4	0.2	0.1	<0.1	<0.1	<0.1
		9.0	1.8	0.7	0.1	<0.1	0.1	<0.1	<0.1
		15.0	4.1	1.3	0.4	0.2	0.1	0.2	0.1
	13.0	6.0	1.3	0.7	0.3	0.1	0.1	<0.1	<0.1
		11.0	2.3	0.9	0.6	0.3	0.2	<0.1	0.2
		17.0	5.3	2.5	0.9	0.4	0.3	0.2	0.2
	15.0	6.0	1.1	0.7	0.3	0.1	0.1	0.1	<0.1
		11.0	2.8	1.0	0.3	0.2	0.3	0.3	<0.1
		17.0	5.2	2.5	1.1	0.4	0.3	0.4	0.2
	17.0	6.0	1.4	1.1	0.5	0.4	0.1	0.1	0.1
		11.0	6.0	3.5	1.1	1.2	0.5	0.5	0.3
		15.0	7.9	4.3	1.5	1.7	0.5	0.9	0.3
	19.0	6.0	0.5	0.3	0.1	0.1	<0.1	<0.1	<0.1
SW	7.0	6.0	0.4	0.3	<0.1	0.1	<0.1	<0.1	<0.1
	9.0	6.0	0.4	0.4	0.1	<0.1	<0.1	<0.1	<0.1
		11.0	1.0	0.4	<0.1	<0.1	<0.1	<0.1	<0.1
	11.0	6.0	1.1	0.6	0.3	<0.1	<0.1	<0.1	<0.1
		11.0	2.1	1.0	0.4	<0.1	<0.1	<0.1	<0.1
	13.0	6.0	0.9	0.3	0.2	0.1	0.1	<0.1	<0.1
		11.0	3.8	1.4	0.7	0.2	0.2	0.2	0.3
	15.0	6.0	0.5	0.4	0.2	<0.1	<0.1	<0.1	<0.1
		9.0	1.4	1.1	0.6	0.2	0.1	0.1	0.2
	17.0	6.0	1.1	0.6	0.3	0.2	0.2	0.1	<0.1
	19.0	6.0	1.4	0.8	0.3	0.1	<0.1	0.1	0.2

(CONTINUED)

TABLE 20 (CONCLUDED)

Test Wave Direction	Test Wave Period sec	Test Wave Height ft	Wave Height, ft Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 14
NW	7.0	6.0	<0.1	<0.1	<0.1	0.6	<0.1	<0.1	3.4
		9.0	<0.1	<0.1	<0.1	0.6	<0.1	<0.1	5.2
	9.0	6.0	<0.1	0.1	<0.1	0.9	<0.1	0.1	4.4
		9.0	<0.1	0.2	<0.1	1.1	<0.1	0.1	6.0
		13.0	0.1	0.2	0.1	1.4	0.1	<0.1	9.0
	11.0	6.0	<0.1	0.1	0.1	1.0	0.2	<0.1	5.4
		9.0	0.2	<0.1	0.1	1.7	0.1	<0.1	7.9
		15.0	0.2	0.1	0.2	2.2	0.2	0.1	14.0
	13.0	6.0	<0.1	<0.1	0.1	1.1	<0.1	<0.1	5.4
		11.0	<0.1	<0.1	0.2	1.6	<0.1	0.1	12.2
		17.0	0.2	0.2	0.3	3.3	0.2	0.2	15.4
	15.0	6.0	0.1	<0.1	<0.1	1.0	<0.1	<0.1	4.0
		11.0	0.3	0.1	0.1	2.0	0.2	0.1	12.9
		17.0	0.3	0.2	0.2	3.4	0.4	0.3	16.2
	17.0	6.0	0.2	<0.1	0.1	0.7	0.3	0.2	6.2
		11.0	0.3	0.4	0.2	3.1	0.7	0.5	17.2
		15.0	0.3	0.4	0.3	4.3	0.8	0.7	15.4
	19.0	6.0	<0.1	<0.1	<0.1	0.5	0.2	<0.1	1.9
SW	7.0	6.0	<0.1	<0.1	<0.1	1.2	<0.1	<0.1	1.2
	9.0	6.0	<0.1	<0.1	<0.1	0.9	<0.1	<0.1	1.9
		11.0	<0.1	<0.1	<0.1	2.5	<0.1	<0.1	3.0
	11.0	6.0	<0.1	0.1	<0.1	1.9	<0.1	0.1	1.8
		11.0	<0.1	0.1	0.1	3.1	<0.1	0.1	3.7
	13.0	6.0	<0.1	<0.1	0.1	1.6	<0.1	0.1	1.8
		11.0	0.2	0.1	0.2	4.9	0.3	0.1	4.0
	15.0	6.0	<0.1	<0.1	<0.1	1.5	<0.1	<0.1	1.8
		9.0	0.1	<0.1	0.1	2.8	0.1	0.1	2.9
	17.0	6.0	<0.1	<0.1	<0.1	2.2	0.1	0.2	2.8
	19.0	6.0	<0.1	<0.1	<0.1	2.1	0.1	<0.1	1.4

TABLE 21

WAVE HEIGHTS FOR PLAN 3F FOR TEST WAVES

FROM WEST DIRECTION, 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
W	7.0	6.0	0.5	0.3	<0.1	<0.1	<0.1	<0.1	<0.1
		9.0	0.8	0.5	<0.1	<0.1	<0.1	<0.1	<0.1
	9.0	6.0	0.4	0.2	<0.1	<0.1	<0.1	<0.1	<0.1
		11.0	1.1	0.5	0.3	0.1	<0.1	0.1	<0.1
	11.0	13.0	0.4	0.3	0.2	<0.1	<0.1	<0.1	<0.1
		13.0	1.8	1.1	0.5	0.2	0.1	0.2	0.2
	13.0	6.0	0.9	0.5	<0.1	<0.1	0.1	0.1	<0.1
		15.0	3.3	2.1	0.6	0.4	0.4	0.2	0.2
	15.0	6.0	1.3	0.4	0.2	<0.1	<0.1	<0.1	<0.1
		13.0	3.7	1.8	0.7	0.2	0.2	0.2	0.1
	17.0	6.0	1.4	0.5	0.2	0.1	0.1	<0.1	<0.1
		13.0	4.4	1.9	0.6	0.5	0.3	0.4	0.2
	19.0	6.0	1.1	0.5	0.2	0.1	<0.1	<0.1	<0.1

(CONTINUED)

TABLE 21 (CONCLUDED)

Test Wave Direction	Test Wave Period, sec	Test Wave Height, ft	Wave Height, ft: Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 20
W	7.0	6.0	<0.1	<0.1	<0.1	0.9	<0.1	<0.1	1.9
		9.0	<0.1	<0.1	<0.1	1.1	<0.1	<0.1	2.6
	9.0	6.0	<0.1	<0.1	<0.1	0.7	<0.1	<0.1	[illegible]
		11.0	<0.1	0.1	0.2	1.5	<0.1	<0.1	[illegible]
	11.0	13.0	<0.1	<0.1	<0.1	0.5	<0.1	<0.1	[illegible]
		13.0	0.2	0.2	0.2	1.0	0.1	0.2	[illegible]
	13.0	6.0	<0.1	<0.1	<0.1	1.5	<0.1	<0.1	[illegible]
		15.0	0.3	0.2	0.4	2.1	0.2	0.2	[illegible]
	15.0	6.0	<0.1	<0.1	<0.1	1.5	0.1	<[illegible]	[illegible]
		[illegible]	0.2	0.1	0.2	[illegible]	0.2	[illegible]	[illegible]
	17.0	6.0	<0.1	<0.1	<0.1	1.8	<[illegible]	<[illegible]	[illegible]
		[illegible]	0.2	0.2	0.3	[illegible]	[illegible]	0.1	[illegible]
	19.0	6.0	0.1	<0.1	0.1	[illegible]	[illegible]	<0.1	[illegible]

TABLE 22

WAVE HEIGHTS FOR PLAN 3G FROM WEST DIRECTION

Test Wave Direction	Test Wave Period sec	Test Wave Height ft	Wave Height, ft Gage 1	Gage 2	Gage 3	Gage 4	Gage 5
			SWL = +2.7 FT (MAXIMUM EBB)				
W	7.0	8.0	0.5	0.1	<0.1	<0.1	<0.1
	11.0	8.0	0.9	0.2	<0.1	<0.1	<0.1
		12.0	0.9	0.5	0.2	<0.1	<0.1
	15.0	8.0	0.8	0.2	0.1	<0.1	<0.1
		12.0	1.9	0.5	0.2	0.1	0.1
	17.0	8.0	0.9	0.2	0.1	<0.1	<0.1
	19.0	2.0	0.3	0.2	<0.1	<0.1	<0.1
			SWL = +2.7 FT (MAXIMUM FLOOD)				
W	7.0	8.0	0.6	0.2	<0.1	<0.1	<0.1
	11.0	[illegible]	0.9	0.2	<0.1	<0.1	<0.1
		12.0	0.2	0.5	0.3	0.1	0.1
	15.0	8.0	0.9	0.3	<0.1	<0.1	0.1
	[illegible]	[illegible]	1.1	0.8	0.1	<0.1	<0.1
	[illegible]	[illegible]	1.2	[illegible]	0.2	0.1	<0.1
	19.0	[illegible]	0.0	[illegible]	0.1	0.1	<0.1
			SWL = +5.4 FT				
W	7.0	8.0	0.5	0.2	<0.1	<0.1	<0.1
		[illegible]	[illegible]	0.1	<0.1	<0.1	<0.1
	9.0	8.0	1.0	0.2	0.1	<0.1	<0.1
		11.0	1.8	0.6	0.4	0.1	0.1
	11.0	6.0	0.7	0.3	0.1	<0.1	<0.1
		12.0	1.3	1.4	0.6	0.2	0.1
	13.0	8.0	1.2	0.4	0.1	<0.1	<0.1
		12.0	4.6	2.5	0.3	0.4	0.4
	15.0	8.0	1.0	0.2	0.1	<0.1	<0.1
		12.0	3.0	1.8	0.7	0.2	0.2
	17.0	8.0	1.5	0.4	0.2	0.1	<0.1
		12.0	4.0	1.3	0.5	0.4	0.2
	19.0	8.0	0.9	0.9	0.1	0.1	<0.1

(CONTINUED)

TABLE 22 (CONTINUED)

TEST WAVE			WAVE HEIGHT, FT				
DIRECTION	PERIOD SEC	HEIGHT FT	GAGE 6	GAGE 7	GAGE 8	GAGE 9	GAGE 10
SWL = +2.7 FT(MAXIMUM EBB)							
W	7.0	6.0	<0.1	<0.1	<0.1	<0.1	<0.1
	11.0	6.0	<0.1	<0.1	<0.1	<0.1	<0.1
		13.0	<0.1	<0.1	<0.1	<0.1	0.1
	15.0	6.0	<0.1	<0.1	<0.1	<0.1	<0.1
		13.0	0.1	<0.1	<0.1	<0.1	0.1
	17.0	6.0	<0.1	<0.1	0.1	<0.1	<0.1
	19.0	6.0	<0.1	<0.1	<0.1	<0.1	<0.1
SWL = +2.7 FT(MAXIMUM FLOOD)							
W	7.0	6.0	<0.1	<0.1	<0.1	<0.1	<0.1
	11.0	6.0	<0.1	<0.1	<0.1	<0.1	<0.1
		13.0	0.1	0.1	0.2	0.1	0.2
	15.0	6.0	<0.1	<0.1	<0.1	<0.1	<0.1
	7.0	6.0	<0.1	<0.1	<0.1	<0.1	<0.1
	17.0	6.0	0.1	<0.1	0.1	<0.1	<0.1
	19.0	6.0	0.1	<0.1	<0.1	<0.1	<0.1
SWL = +5.4 FT							
W	7.0	6.0	<0.1	<0.1	<0.1	<0.1	<0.1
		9.0	<0.1	<0.1	<0.1	<0.1	<0.1
	9.0	6.0	<0.1	<0.1	<0.1	<0.1	<0.1
		11.0	0.1	0.1	0.1	0.1	0.1
	11.0	6.0	<0.1	<0.1	<0.1	<0.1	<0.1
		13.0	0.3	0.2	0.2	0.2	0.2
	13.0	6.0	<0.1	<0.1	<0.1	<0.1	<0.1
		15.0	0.3	0.3	0.3	0.3	0.4
	15.0	6.0	<0.1	<0.1	<0.1	<0.1	<0.1
		13.0	0.2	0.1	0.1	0.1	0.2
	17.0	6.0	<0.1	<0.1	<0.1	<0.1	<0.1
		13.0	0.3	0.1	0.1	0.2	0.2
	19.0	6.0	<0.1	<0.1	<0.1	<0.1	<0.1

(CONTINUED)

TABLE 22 (CONCLUDED)

TEST WAVE			WAVE HEIGHT, FT			
DIRECTION	PERIOD SEC	HEIGHT FT	GAGE 11	GAGE 12	GAGE 13	GAGE 14
			SWL = +2.7 FT(MAXIMUM EBB)			
W	7.0	6.0	0.9	<0.1	<0.1	1.8
	11.0	6.0	1.6	<0.1	<0.1	2.6
		13.0	2.7	<0.1	<0.1	4.6
	15.0	6.0	2.0	<0.1	<0.1	3.3
		13.0	4.1	0.1	<0.1	5.5
	17.0	6.0	2.4	<0.1	<0.1	3.8
	19.0	6.0	2.1	0.1	<0.1	2.5
			SWL = +2.7 FT(MAXIMUM FLOOD)			
W	7.0	6.0	1.1	<0.1	<0.1	1.7
	11.0	6.0	1.7	<0.1	<0.1	2.8
		13.0	4.1	0.1	<0.1	4.9
	15.0	6.0	2.0	<0.1	<0.1	3.9
		6.0	2.8	<0.1	<0.1	4.5
	17.0	6.0	3.1	0.1	<0.1	4.0
	19.0	6.0	2.6	0.1	<0.1	2.8
			SWL = +5.4 FT			
W	7.0	6.0	1.9	<0.1	<0.1	2.2
		9.0	2.2	<0.1	<0.1	2.6
	9.0	6.0	1.8	<0.1	<0.1	2.6
		11.0	2.8	<0.1	<0.1	4.4
	11.0	6.0	2.2	<0.1	<0.1	2.5
		13.0	4.2	0.1	0.1	6.3
	13.0	6.0	2.1	<0.1	<0.1	3.2
		15.0	5.1	0.2	0.2	8.2
	15.0	6.0	2.8	0.1	<0.1	3.6
		13.0	5.8	0.3	0.1	8.1
	17.0	6.0	3.1	<0.1	<0.1	4.0
		13.0	4.8	0.3	0.1	7.9
	19.0	6.0	3.2	<0.1	<0.1	3.3

TABLE 23

WAVE HEIGHTS FOR PLAN 3G FOR TEST WAVES

FROM SOUTHWEST DIRECTION, 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
SW	7.0	6.0	0.5	0.2	<0.1	<0.1	<0.1	<0.1	<0.1
	9.0	6.0	0.8	0.3	0.2	<0.1	<0.1	<0.1	<0.1
		11.0	1.5	0.4	0.2	<0.1	<0.1	<0.1	<0.1
	11.0	6.0	0.7	0.4	0.2	<0.1	<0.1	<0.1	<0.1
		11.0	1.9	0.9	0.4	0.2	0.1	0.1	0.1
	13.0	6.0	0.8	0.3	0.1	<0.1	0.1	<0.1	<0.1
		11.0	3.4	1.0	0.2	0.1	0.1	0.1	<0.1
	15.0	6.0	0.6	0.4	0.3	<0.1	<0.1	<0.1	<0.1
		9.0	1.0	1.0	0.4	0.1	0.2	0.1	<0.1
	17.0	6.0	0.9	0.6	0.3	0.5	0.2	0.2	<0.1
	19.0	6.0	1.7	0.8	0.4	0.2	<0.1	0.1	0.1

(CONTINUED)

TABLE 23 (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
SW	7.0	6.0	<0.1	<0.1	<0.1	1.7	<0.1	<0.1	0.9
	9.0	6.0	<0.1	<0.1	<0.1	1.9	<0.1	<0.1	1.5
		11.0	<0.1	<0.1	<0.1	4.1	<0.1	<0.1	2.8
	11.0	6.0	<0.1	<0.1	<0.1	2.7	<0.1	<0.1	2.4
		11.0	0.2	0.1	0.2	5.1	<0.1	0.1	3.3
	13.0	6.0	<0.1	<0.1	<0.1	2.1	<0.1	<0.1	1.8
		11.0	0.1	<0.1	0.1	7.3	<0.1	0.1	3.0
	15.0	6.0	<0.1	<0.1	<0.1	2.1	<0.1	<0.1	1.6
		9.0	0.1	<0.1	0.1	3.1	0.2	<0.1	2.6
	17.0	6.0	<0.1	0.1	<0.1	3.4	0.3	0.2	2.5
	19.0	6.0	0.2	<0.1	0.1	4.8	0.4	<0.1	1.9

TABLE 24

WAVE HEIGHTS FOR PLAN 3G (NO REVETMENT), SWL = +5.4 FT

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	13.0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	15.0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.2
	[illegible]	[illegible]	[illegible]	[illegible]	0.4	[illegible]	0.1

(CONTINUED)

(SHEET 1 OF 3)

TABLE 24 (CONTINUED)

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	<0.1
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	<0.1
	[illegible]		[illegible]	[illegible]	[illegible]	[illegible]	<0.1
		[illegible]	[illegible]	[illegible]	[illegible]	0.2	<0.1
		[illegible]	[illegible]	[illegible]	[illegible]	0.2	0.1
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.2
			[illegible]	[illegible]	[illegible]	0.2	0.2
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.1
			[illegible]	[illegible]	[illegible]	[illegible]	0.2
		[illegible]	0.1	0.1	[illegible]	[illegible]	0.4
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	<0.1
		[illegible]	[illegible]	[illegible]	[illegible]	<0.1	[illegible]
		[illegible]	[illegible]	[illegible]	[illegible]	0.8	0.8
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.1	<0.1
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.4
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.1	<0.1
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	<0.1
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	<0.1
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	<0.1
		[illegible]	[illegible]	[illegible]	[illegible]	0.2	0.1
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	<0.1
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.2
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	<0.1	<0.1
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	<0.1
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.2
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	<0.1
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.3
	[illegible]	[illegible]	[illegible]	<0.1	[illegible]	<0.1	<0.1
SU	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	<0.1	<0.1
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	<0.1
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	<0.1
	[illegible]	[illegible]	[illegible]	[illegible]	<0.1	0.1	<0.1
		[illegible]	[illegible]	[illegible]	0.2	0.2	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	<0.1	<0.1	0.1
		[illegible]	[illegible]	[illegible]	0.3	0.2	0.2
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	<0.1	[illegible]
		[illegible]	[illegible]	[illegible]	0.2	0.1	0.2
	17.0	[illegible]	[illegible]	[illegible]	0.1	0.1	<0.1
	19.0	[illegible]	0.1	0.2	0.3	0.1	0.1

(CONTINUED)

(SHEET 2 OF 3)

TABLE 24 (CONCLUDED)

(SHEET 3 OF 3)

TABLE 25

WAVE HEIGHTS FOR PLAN 3G (NO REVETMENT), SWL = +7.6 FT

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
NW	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
W	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
SW	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

(CONTINUED)

(SHEET 1 OF 3)

TABLE 25 (CONTINUED)

[illegible]			WAVE HEIGHT, FT				
DIRECTION	[illegible]	[illegible]	GAGE 6	GAGE 7	GAGE 8	GAGE 9	GAGE 10
NW	7.0	[illegible]	<0.1	<0.1	<0.1	<0.1	<0.1
		[illegible]	<0.1	<0.1	<0.1	<0.1	<0.1
	9.0	[illegible]	0.1	<0.1	0.1	<0.1	0.1
		[illegible]	0.1	<0.1	0.1	0.1	0.1
		[illegible]	0.3	[illegible]	0.2	0.3	0.2
	11.0	[illegible]	0.3	[illegible]	0.1	0.2	0.2
		[illegible]	[illegible]	[illegible]	0.1	0.3	0.3
		[illegible]	0.4	[illegible]	0.3	0.3	0.4
	13.0	[illegible]	[illegible]	0.3	0.3	0.1	0.2
		[illegible]	[illegible]	0.6	0.4	0.1	0.2
		[illegible]	[illegible]	[illegible]	0.4	0.4	0.3
	15.0	[illegible]	[illegible]	0.2	<0.1	0.1	<0.1
		[illegible]	[illegible]	0.3	[illegible]	0.3	0.2
		[illegible]	[illegible]	[illegible]	0.5	0.5	0.3
	17.0	[illegible]	[illegible]	[illegible]	0.2	<0.1	0.1
		[illegible]	[illegible]	0.5	0.6	0.3	0.3
		[illegible]	0.7	0.4	0.6	0.3	0.4
	19.0	[illegible]	0.1	0.2	0.1	<0.1	<0.1
W	7.0	[illegible]	<0.1	<0.1	<0.1	<0.1	<0.1
		[illegible]	<0.1	[illegible]	<0.1	<0.1	<0.1
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	<0.1
		[illegible]	[illegible]	[illegible]	0.1	0.1	0.2
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	<0.1	0.1
		[illegible]	0.3	0.2	[illegible]	0.2	0.3
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	<0.1	<0.1
		[illegible]	0.3	[illegible]	[illegible]	0.2	0.2
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	<0.1	<0.1
		[illegible]	0.2	0.3	0.3	[illegible]	0.2
	[illegible]	[illegible]	[illegible]	<0.1	<0.1	<0.1	<0.1
		[illegible]	[illegible]	0.3	0.3	0.3	0.3
	[illegible]	[illegible]	[illegible]	0.1	<0.1	<0.1	<0.1
SW	[illegible]	[illegible]	[illegible]	<0.1	<0.1	<0.1	<0.1
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	<0.1	<0.1
		[illegible]	[illegible]	<0.1	<0.1	<0.1	<0.1
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.1	0.2
		[illegible]	[illegible]	[illegible]	0.1	0.2	0.2
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	<0.1	<0.1
		[illegible]	0.2	0.2	0.2	0.3	0.2
	[illegible]	[illegible]	[illegible]	[illegible]	<0.1	<0.1	<0.1
		[illegible]	[illegible]	[illegible]	0.2	<0.1	<0.1
	[illegible]	[illegible]	0.3	[illegible]	0.2	<0.1	<0.1
	[illegible]	[illegible]	0.3	0.2	0.1	0.1	0.1

(CONTINUED)

TABLE 25 (CONCLUDED)

NE	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	9.0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	25.0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
E	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	<0.1	[illegible]
SW	[illegible]	[illegible]	[illegible]	<0.1	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	11.0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	13.0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	15.0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	17.0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	19.0	[illegible]	[illegible]	[illegible]	[illegible]	2.1

(SHEET 3 OF 3)

Table 26

Wave Heights for Plan 9

SWL = + 5.4 Ft

Test Wave Direction	Period (Sec)	Height (ft)	Gage Number 1	2	3	4	5	6	7	8	9	10	11	12	13	14
W	7	6	.8	.2	.1	<.1	.1	<.1	<.1	<.1	<.1	<.1	1.8	<.1	<.1	1.9
	7	9	1.3	.2	.1	.1	.1	<.1	.1	<.1	.1	.1	2.7	<.1	.1	2.8
	9	6	1.4	.9	.6	.2	.1	.1	.2	.5	.2	.1	1.6	.5	.2	2.0
	9	11	3.5	1.1	.8	.3	.2	.2	.3	.6	.2	.2	3.9	.7	.2	4.0
	11	6	.8	.5	.6	.1	.1	.2	.1	.1	.2	.1	2.4	.2	.1	2.6
	11	13	4.8	1.1	1.1	.3	.2	.3	.3	.3	.3	.2	4.7	.3	.3	5.9
	13	6	1.3	.5	.4	.3	.3	.2	.2	.2	.1	.2	1.7	.2	.1	2.1
	13	15	6.8	1.3	.5	.5	.4	.3	.2	.4	.2	.2	6.0	.2	.2	6.5
	15	6	1.4	.7	.2	.1	.1	.1	.1	.1	.1	.1	1.9	.1	.1	2.6
	15	13	6.7	.9	.3	.2	.2	.2	.1	.1	.1	.2	5.1	.1	.1	9.9
	17	6	2.0	.9	.3	.2	.1	.3	.1	.1	.2	.2	2.9	.1	.1	3.6
	17	13	7.4	.7	.2	.2	.1	.2	.1	.1	.1	.2	7.5	.1	.1	9.0
	19	6	1.6	.4	.2	.2	.1	.2	.1	.1	.1	.1	3.4	.1	.1	4.0

Table 27

Dimensional Details for Various Harbor Improvement Plans

		Length				Crown Elevation			Distance from Entrance		Elevation of Impervious
Plate	Plan	North Dogleg	Trunk	South Dogleg	Total	North Dogleg	Trunk	South Dogleg	ft	Impervious	Core
6	1	650	900	650	2200	22.5	22.5	22.5	900	No	--
6	1A	550	900	550	2000	22.5	22.5	22.5	900	No	--
6	1B	450	900	450	1800	22.5	22.5	22.5	900	No	--
6	1C	350	900	350	1600	22.5	22.5	22.5	900	No	--
6	1D	250	900	250	1400	22.5	22.5	22.5	900	No	--
6	1E	150	900	150	1200	22.5	22.5	22.5	900	No	--
6	1F	50	900	50	1000	22.5	22.5	22.5	900	No	--
7	1G	350	900	350	1600	20.0	20.0	20.0	900	No	--
7	1H	350	900	350	1600	15.0	15.0	15.0	900	No	--
7	1I	350	900	350	1600	17.5	17.5	17.5	900	No	--
7	1J	450	900	350	1700	17.5	17.5	17.5	900	No	--
7	1K	550	900	350	1800	17.5	17.5	17.5	900	No	--
7	1L	650	900	350	1900	17.5	17.5	17.5	900	No	--
7	1M	650	900	350	1900	22.5	22.5	22.5	900	No	--
8	1N	650	900	350	1900	22.5	17.5	17.5	900	No	--
8	1O	650	900	450	2000	22.5	17.5	17.5	900	No	--
8	1P	650	900	550	2100	22.5	17.5	17.5	900	No	--
9	2	450	900	450	1800	22.5	17.5	17.5	525	No	--
9	2A	450	900	550	1900	22.5	17.5	17.5	525	No	--
9	2B	550	900	550	2000	22.5	17.5	1.5	525	No	--
9	2C	650	900	550	2100	22.5	17.5	17.5	525	No	--
9	2D	650	900	650	2200	22.5	17.5	17.5	525	No	--
10	3	650	900	650	2200	17.5	17.5	17.5	525	Yes	+7.5
10	3A	550	900	650	2100	17.5	17.5	17.5	525	Yes	+7.5
10	3B	450	900	650	2000	17.5	17.5	17.5	525	Yes	+7.5
10	3C	350	900	650	1900	17.5	17.5	17.5	525	Yes	+7.5
10	3D	250	900	650	1800	17.5	17.5	17.5	525	Yes	+7.5
11	3E	350	900	550	1800	17.5	17.5	17.5	525	Yes	+7.5
11	3F	350	900	450	1700	17.5	17.5	17.5	525	Yes	+7.5
11	3G	350	900	350	1600	17.5	17.5	17.5	525	Yes	+7.5
14	9	350	900	350	1600	17.5	17.5	17.5	525	No	--

Table 28

Calculated Offshore Breakwater Rock Volumes

Plan No.	Volume cu yd	Plan No.	Volume cu yd
1	481,000	1P	410,000
1A	438,000	2	322,000
1B	395,000	2A	336,000
1C	353,000	2B	355,000
1D	310,000	2C	374,000
1E	267,000	2D	388,000
1F	224,000	3	336,000
1G	323,000	3A	320,000
1H	268,000	3B	304,000
1I	295,000	3C	288,000
1J	314,000	3D	272,000
1K	334,000	3E	274,000
1L	354,000	3F	260,000
1M	424,000	3G	246,000
1N	379,000	9	220,000
10	395,000		

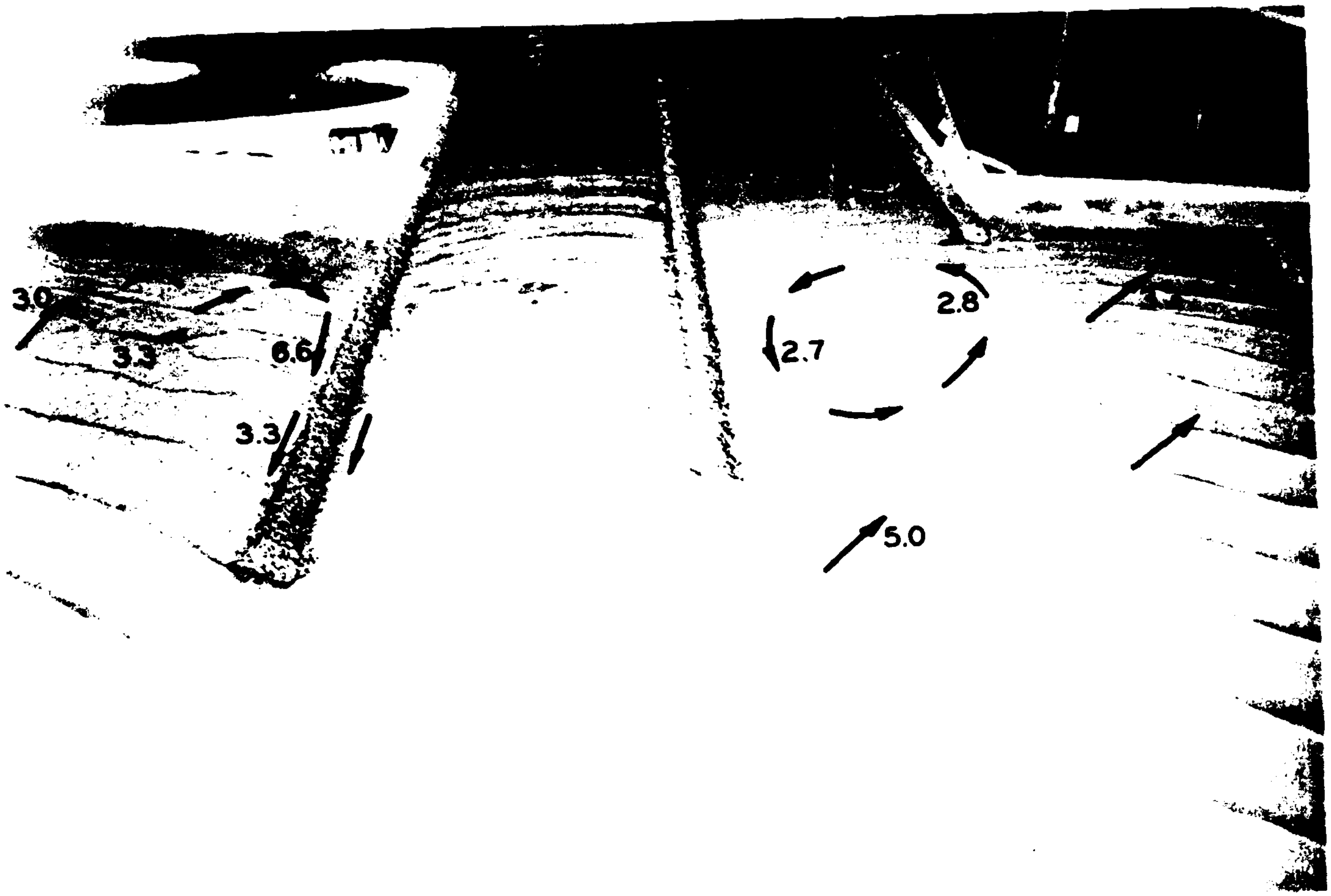


Photo 1. Typical wave and current patterns and current magnitudes (prototype feet per second) for Base Test 1; 9-sec, 13-ft waves from northwest at mllw

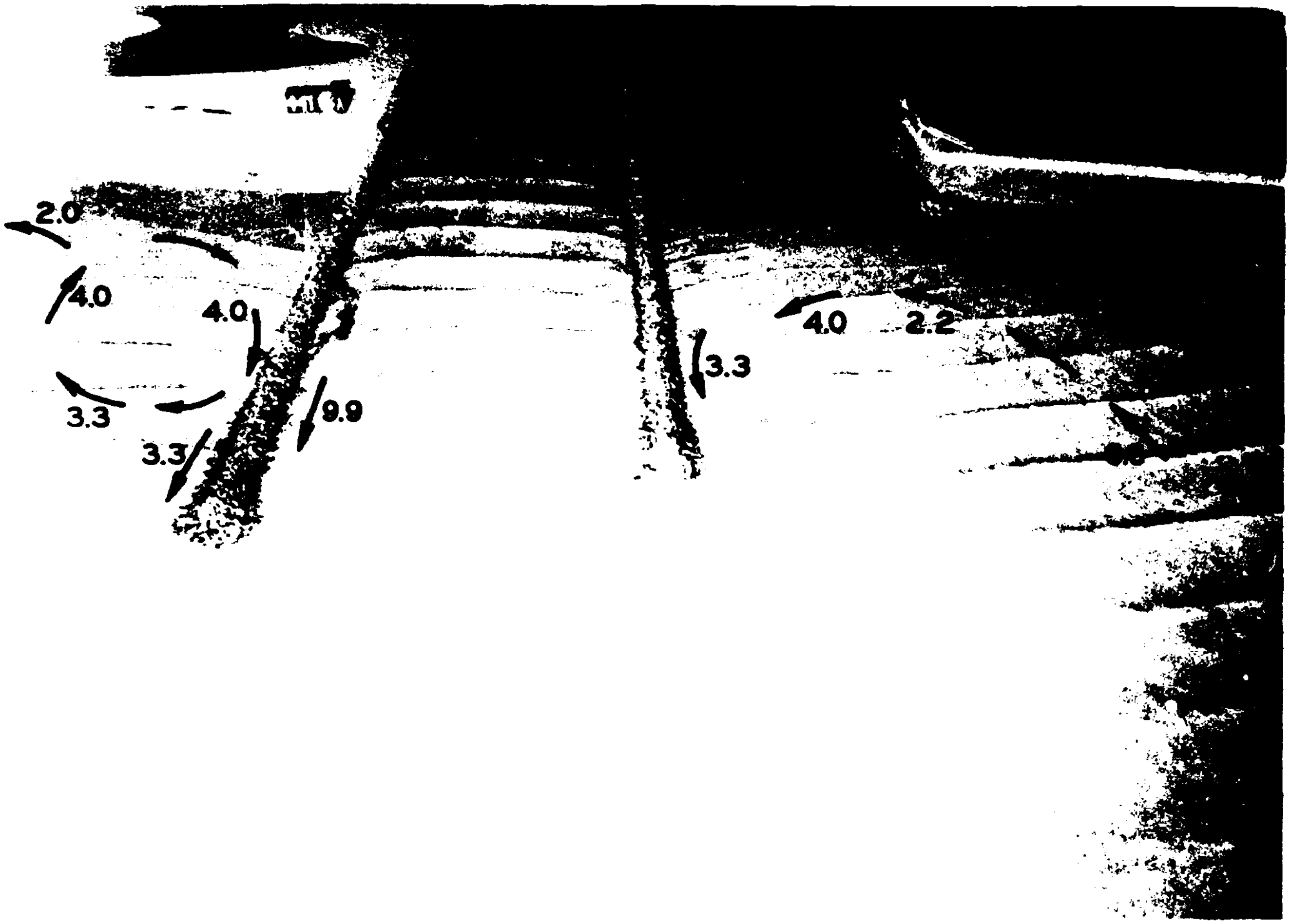


Photo 2. Typical wave and current patterns and current magnitudes (prototype feet per second) for Base Test 1; 9-sec, 11-ft waves from southwest at mllw

Photo 3. Typical tracer movement for Base Test 1 resulting from 9-sec, 13-ft waves from northwest at mllw

Photo 4. Typical tracer movement for Base Test 1 resulting from 9-sec, 11-ft waves from southwest at mllw

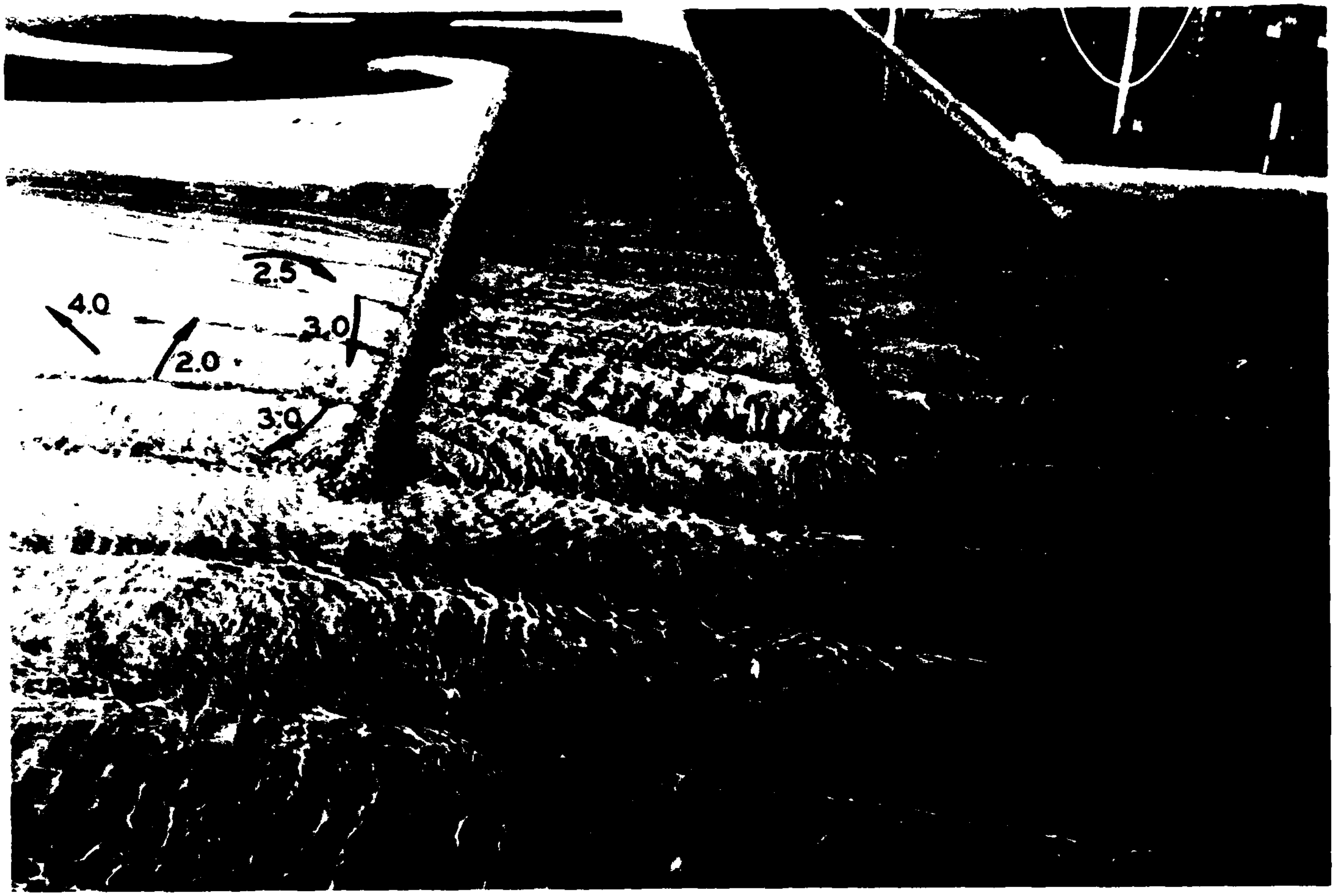


Photo 5. Typical wave and current patterns and current magnitudes (prototype feet per second) for Base Test 5; 11-sec, 13-ft waves from west at mhhw

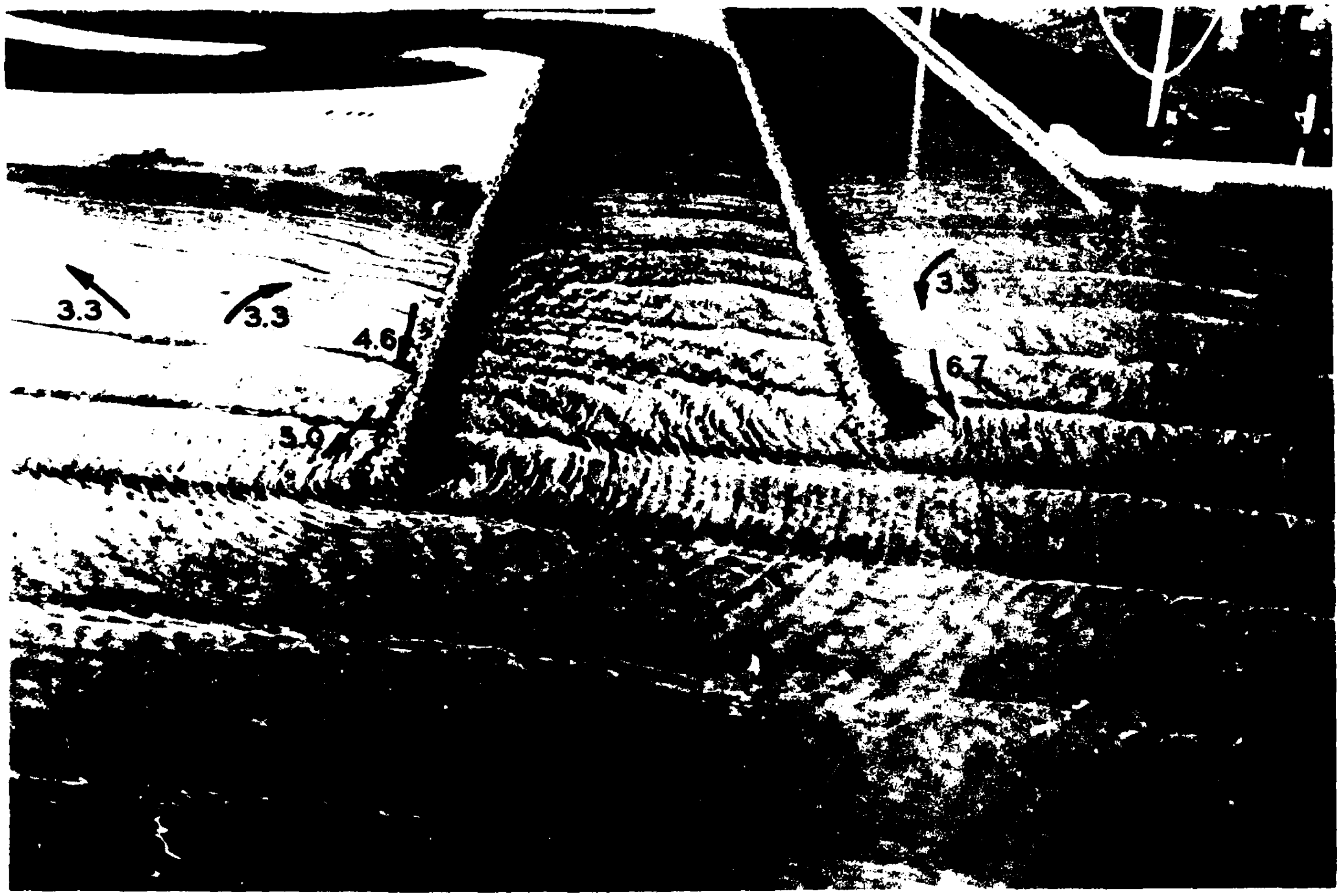


Photo 6. Typical wave and current patterns and current magnitudes (prototype feet per second) for Base Test 5; 11-sec, 13-ft waves from west at mllw

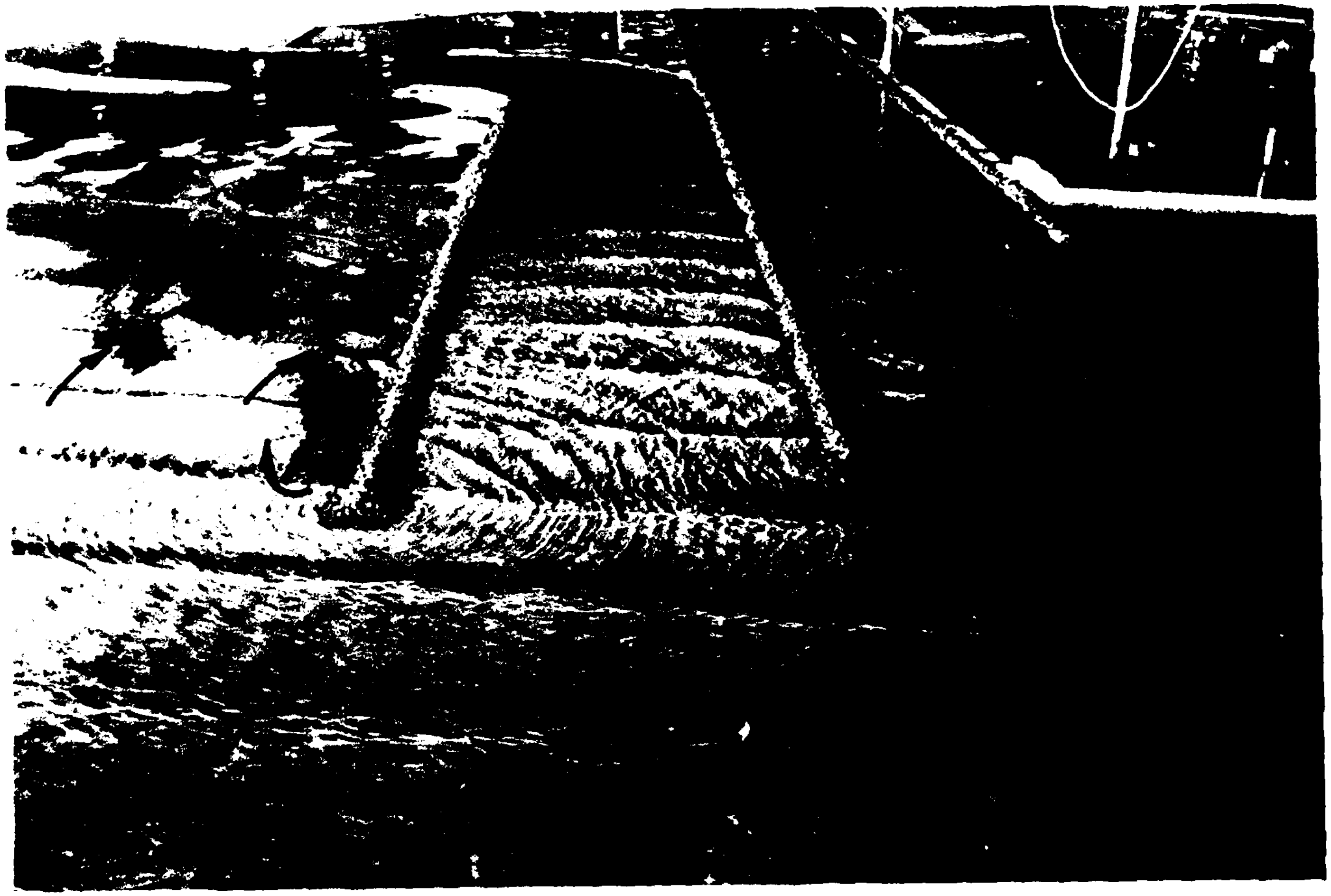


Photo 7. Typical tracer movement for Base Test 5 resulting from 11-sec, 13-ft waves from west at mhhw

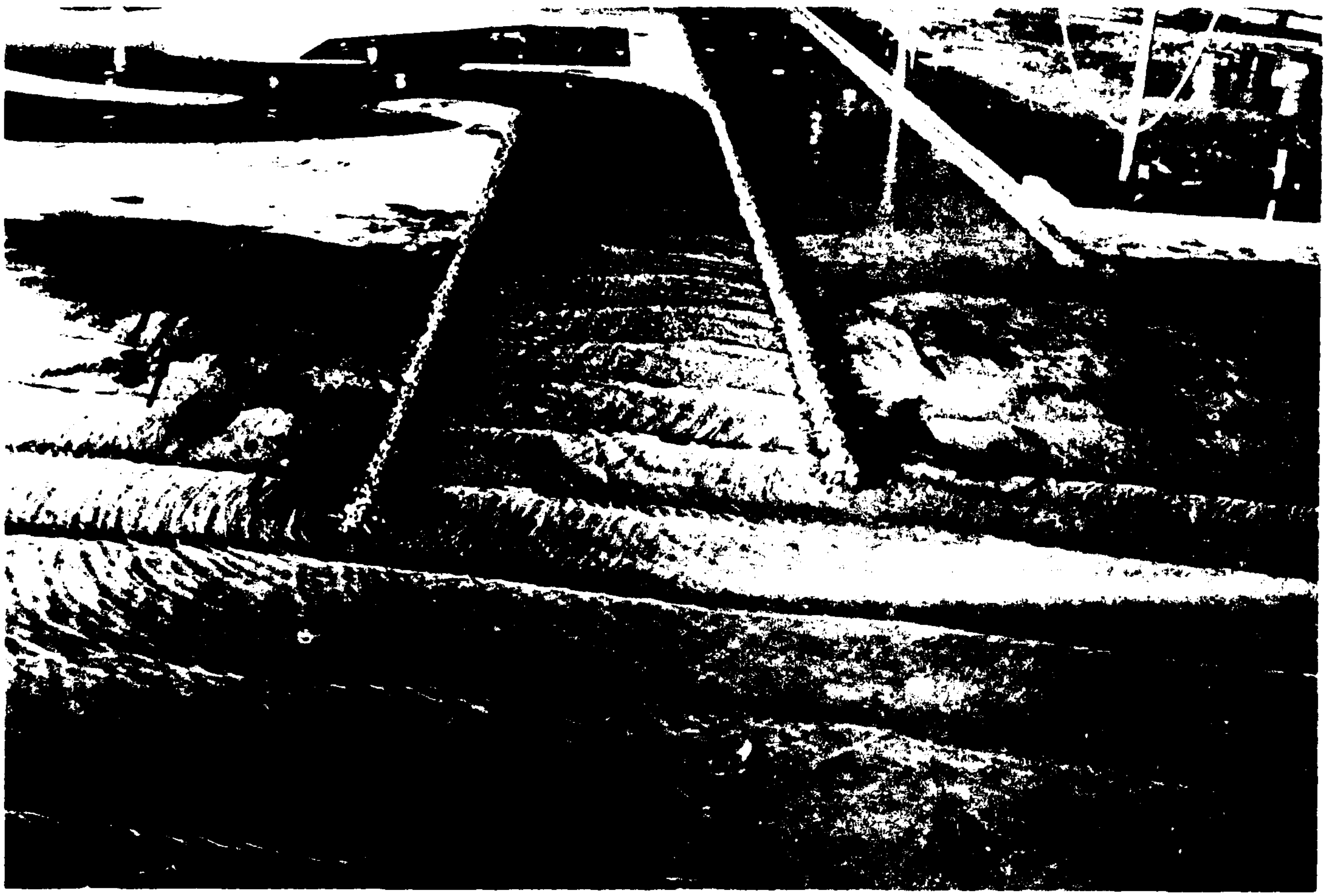


Photo 8. Typical tracer movement for Base Test 5 resulting from 11-sec, 13-ft waves from west at mllw

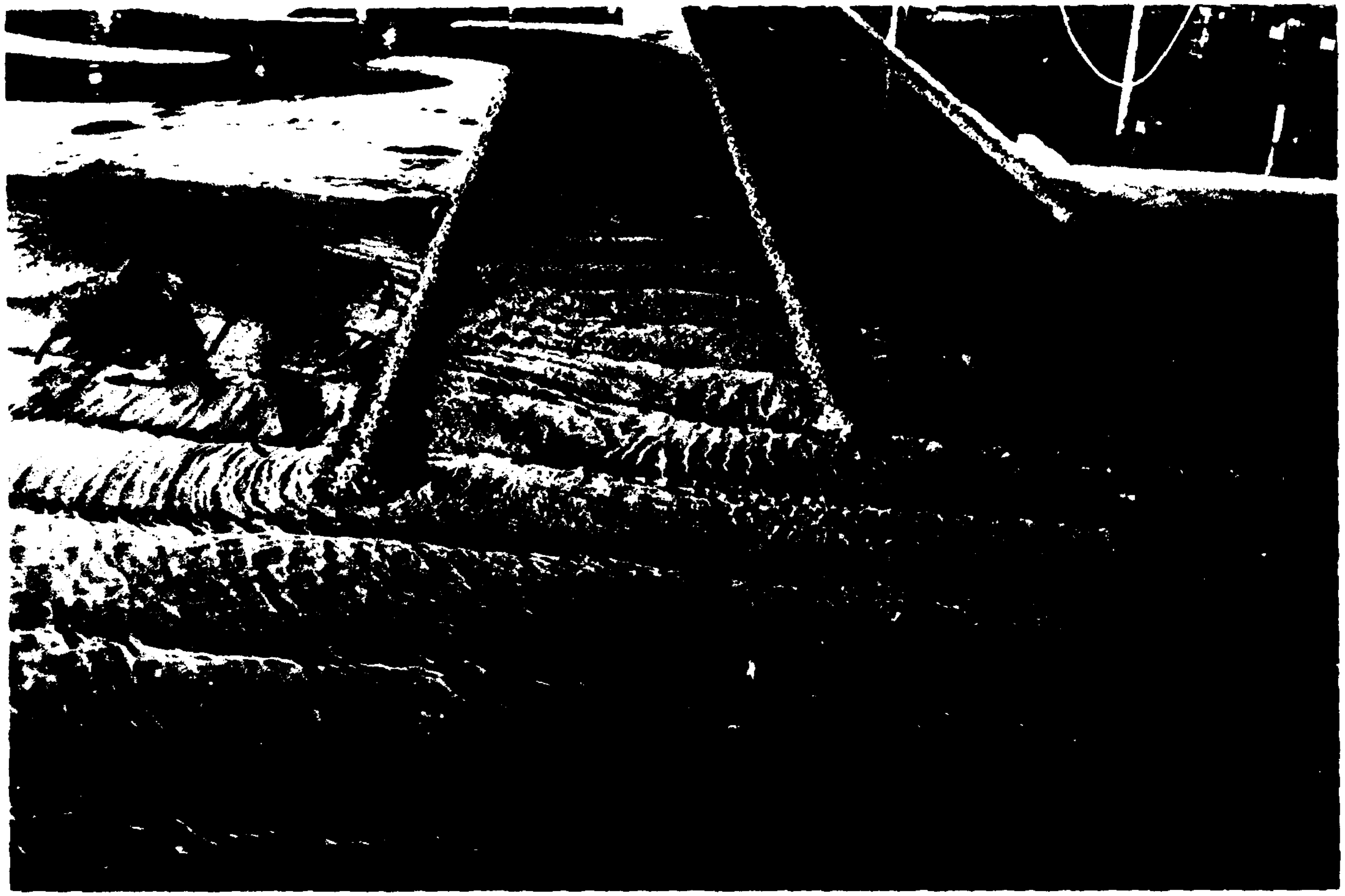


Photo 9. Typical tracer movement for Base Test 5 resulting from 11-sec, 13-ft waves from west for maximum ebb

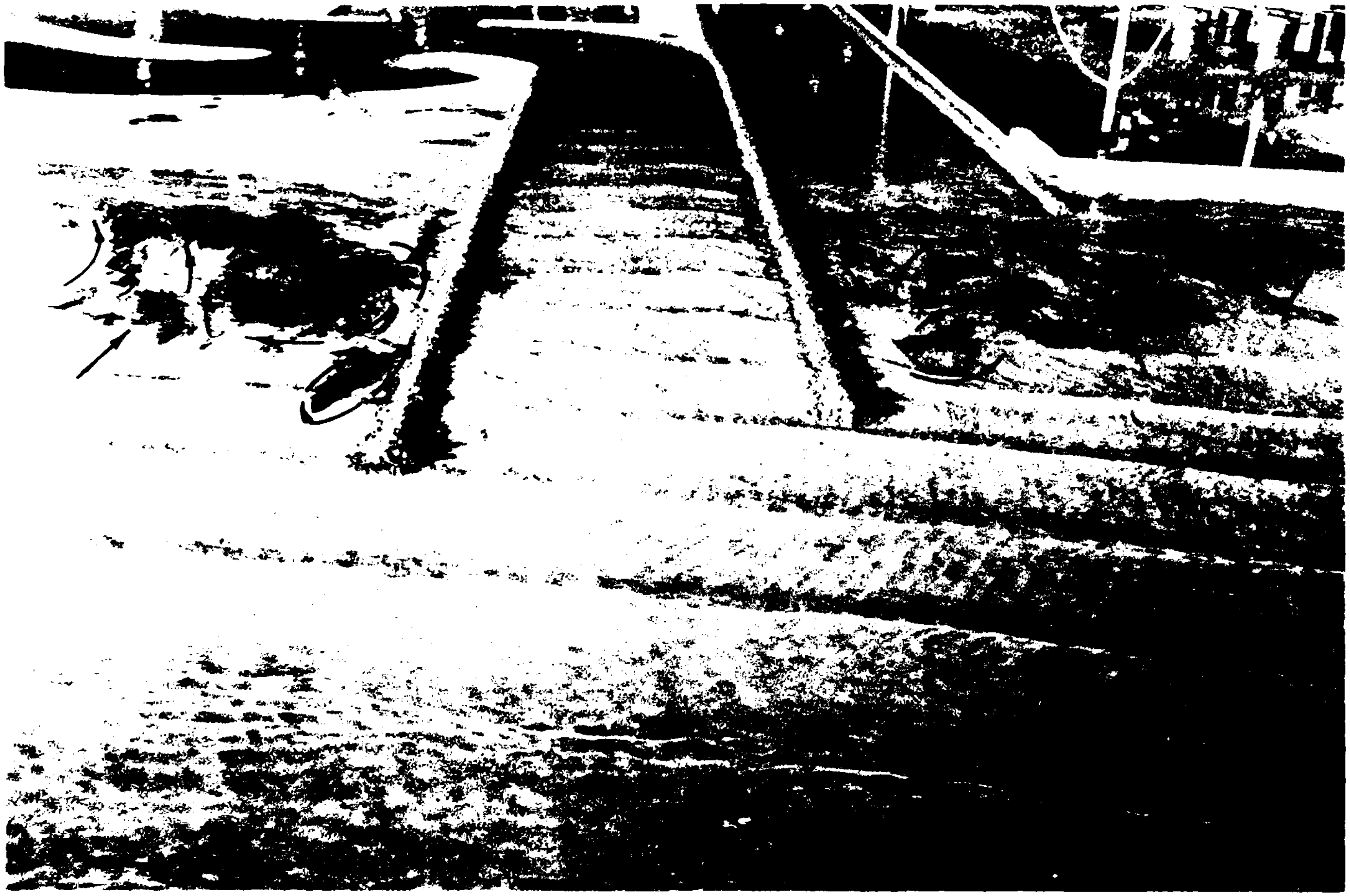


Photo 10. Typical tracer movement for Base Test 5 resulting from 11-sec, 13-ft waves from west for maximum flood

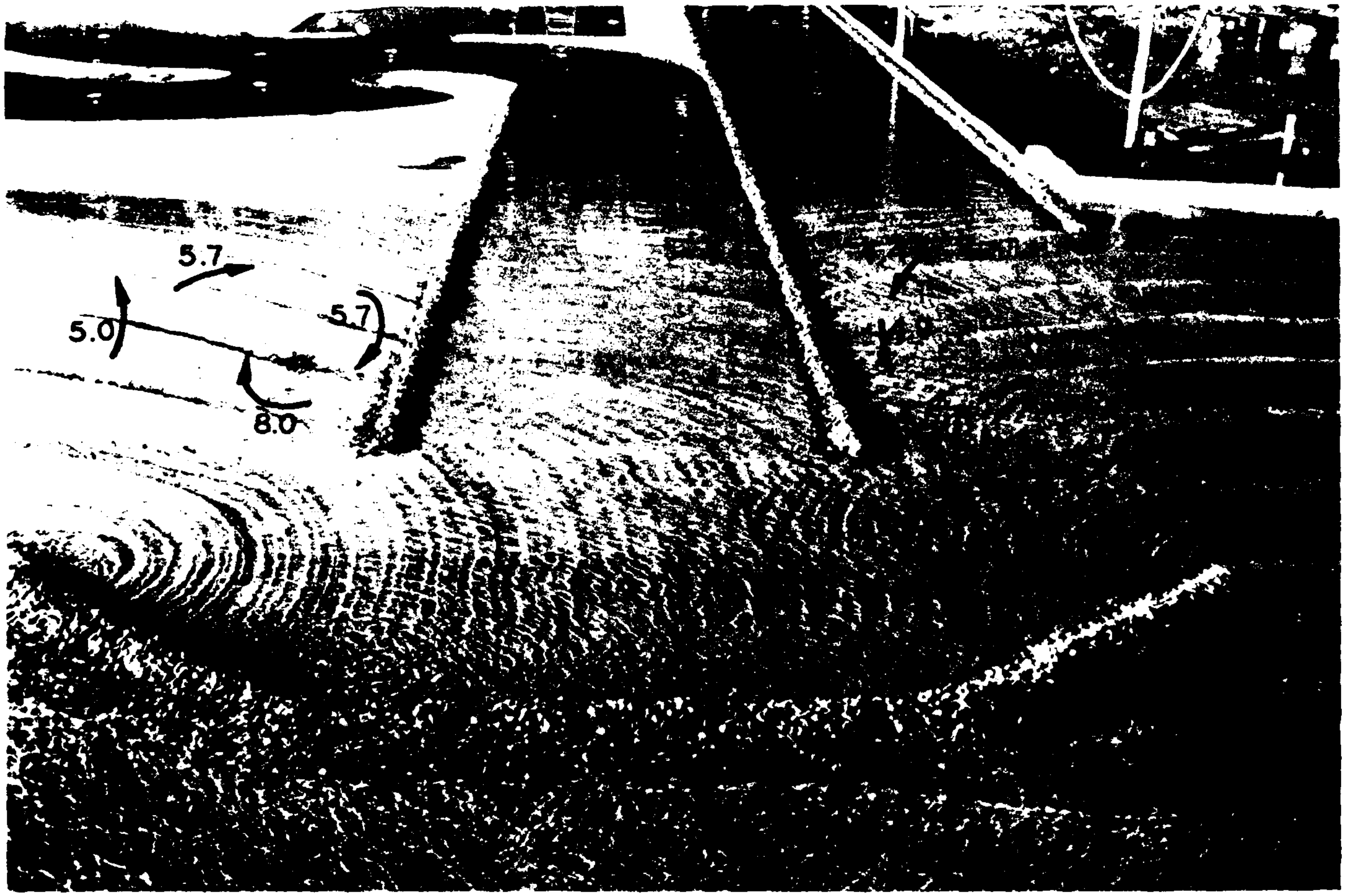


Photo 11. Typical wave and current patterns and current magnitudes (prototype feet per second) for Plan 1; 11-sec, 13-ft waves from west at mhhw

Photo 12. Typical wave and current patterns and current magnitudes (prototype feet per second) for Plan 1; 11-sec, 13-ft waves from west at mllw

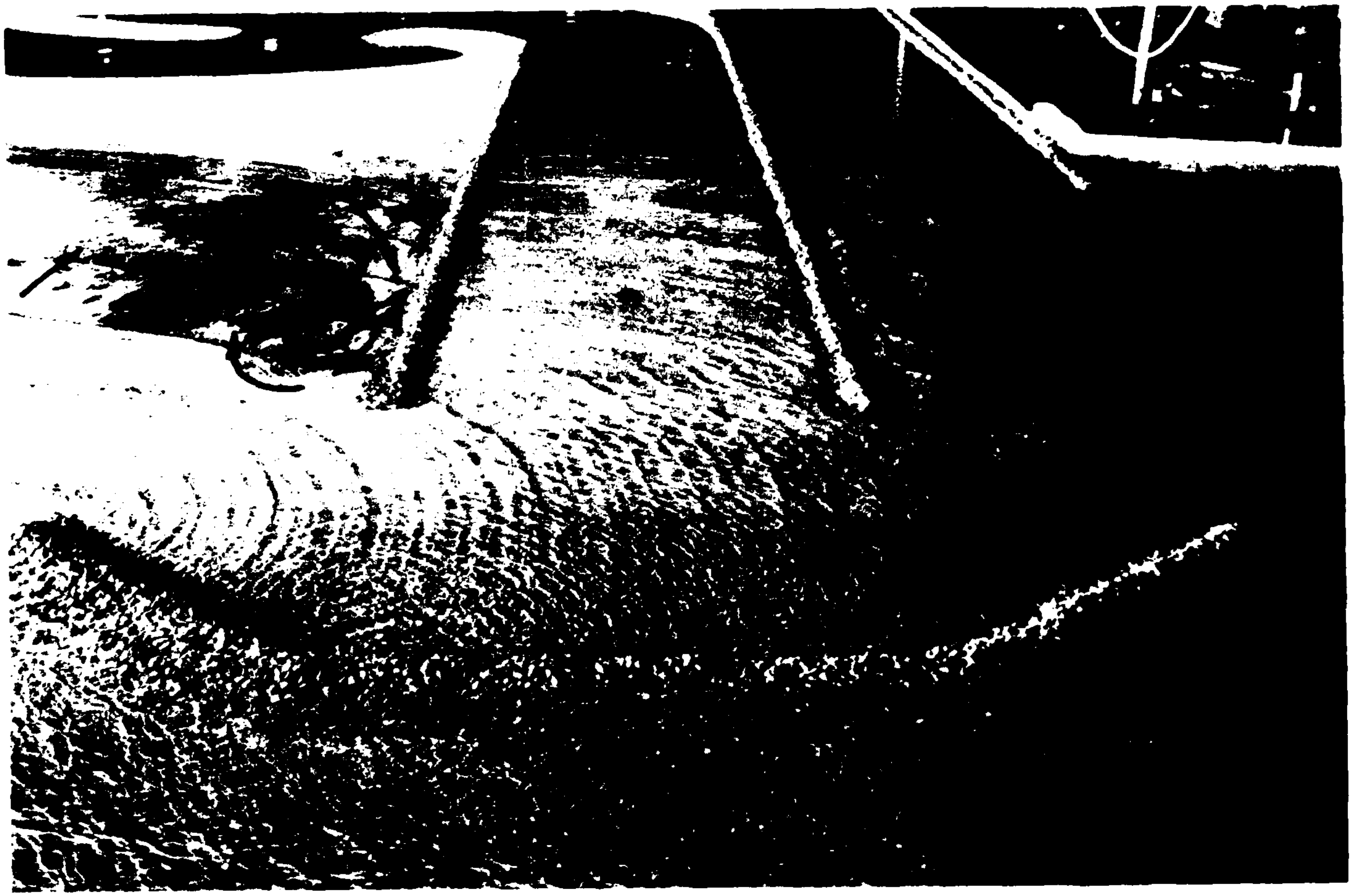


Photo 13. Typical tracer movement for Plan 1 resulting from 11-sec, 13-ft waves from west at mhhw

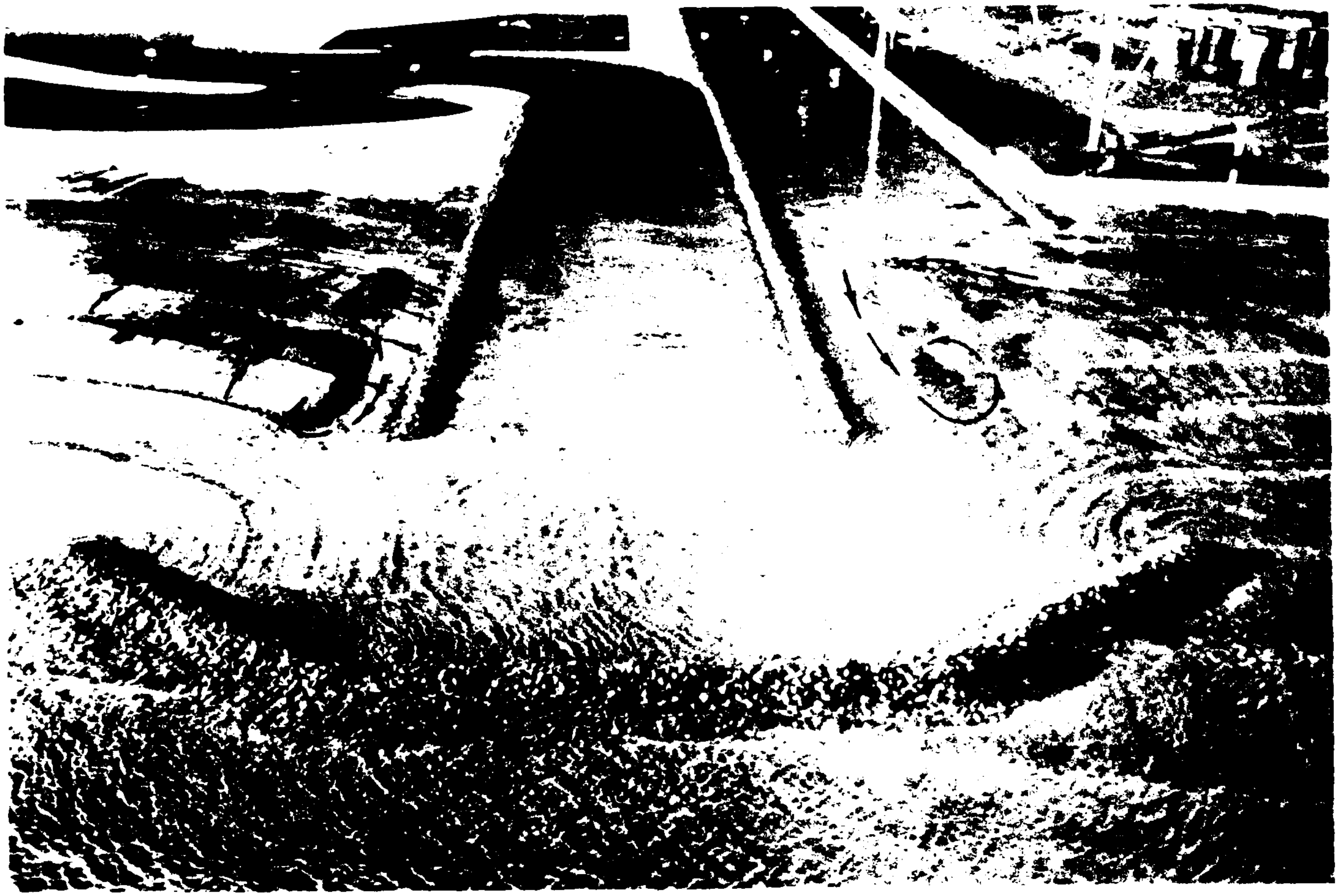


Photo 14. Typical tracer movement for Plan 1 resulting from 11-sec, 13-ft waves from west at mllw

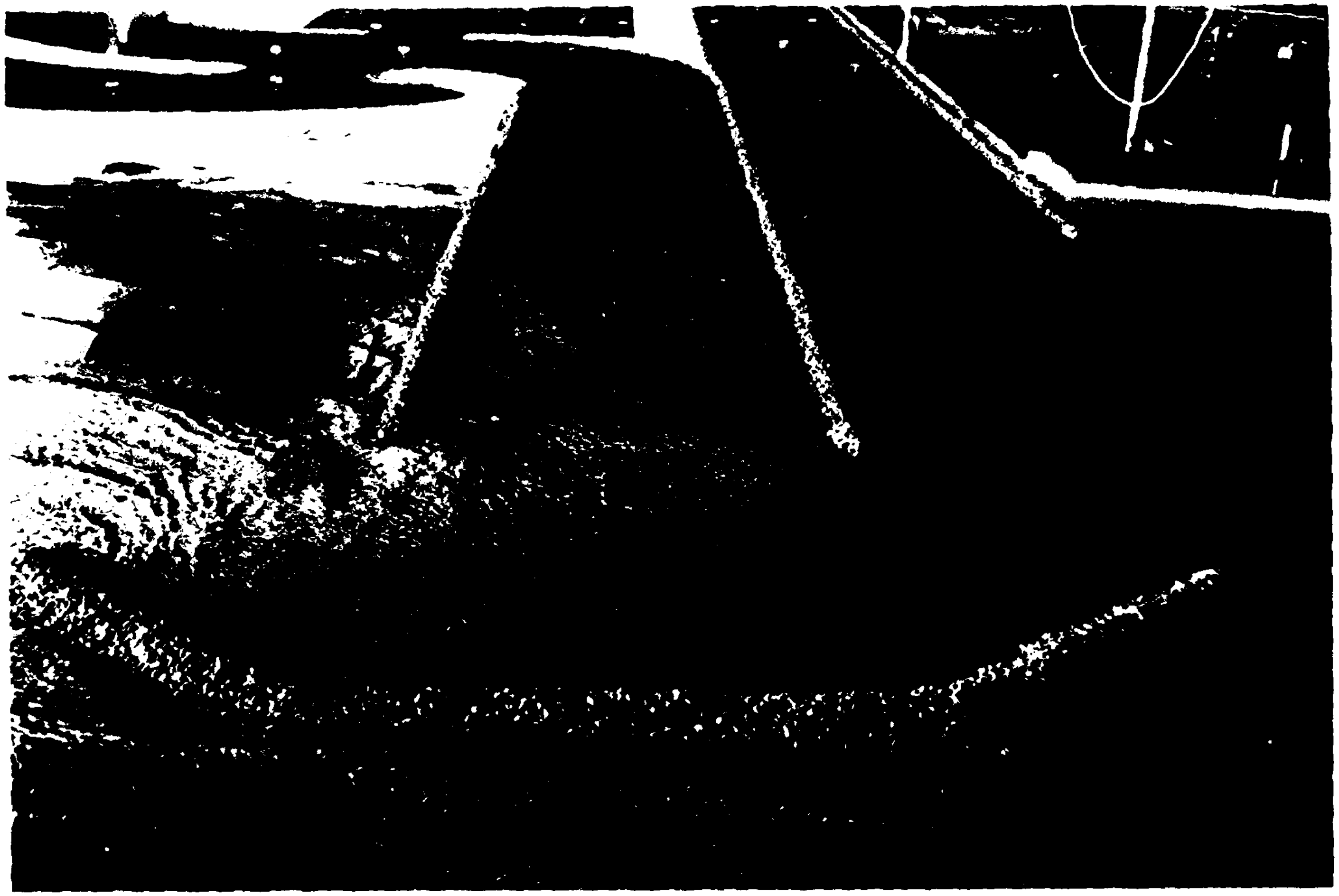


Photo 15. Typical tracer movement for Plan 1 resulting from 11-sec, 13-ft waves from west for maximum ebb

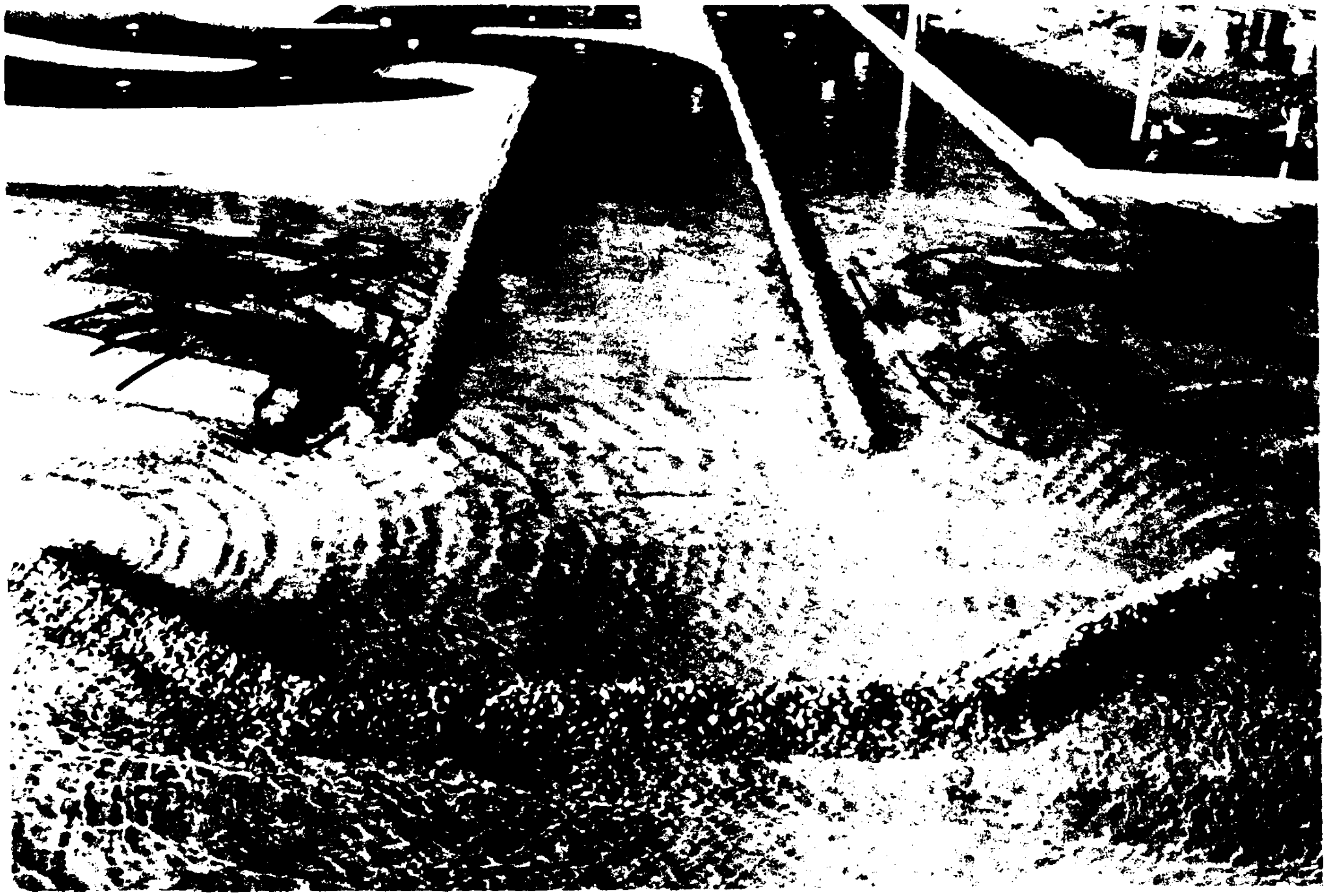


Photo 16. Typical tracer movement for Plan 1 resulting from 11-sec, 13-ft waves from west for maximum flood

Photo 17. Typical wave and current patterns and current magnitudes (prototype feet per second) for Plan 3G; 11-sec, 13-ft waves from west at mhhw

Photo 18. Typical tracer movement for Plan 3G resulting from 11-sec, 13-ft waves from west at mhhw

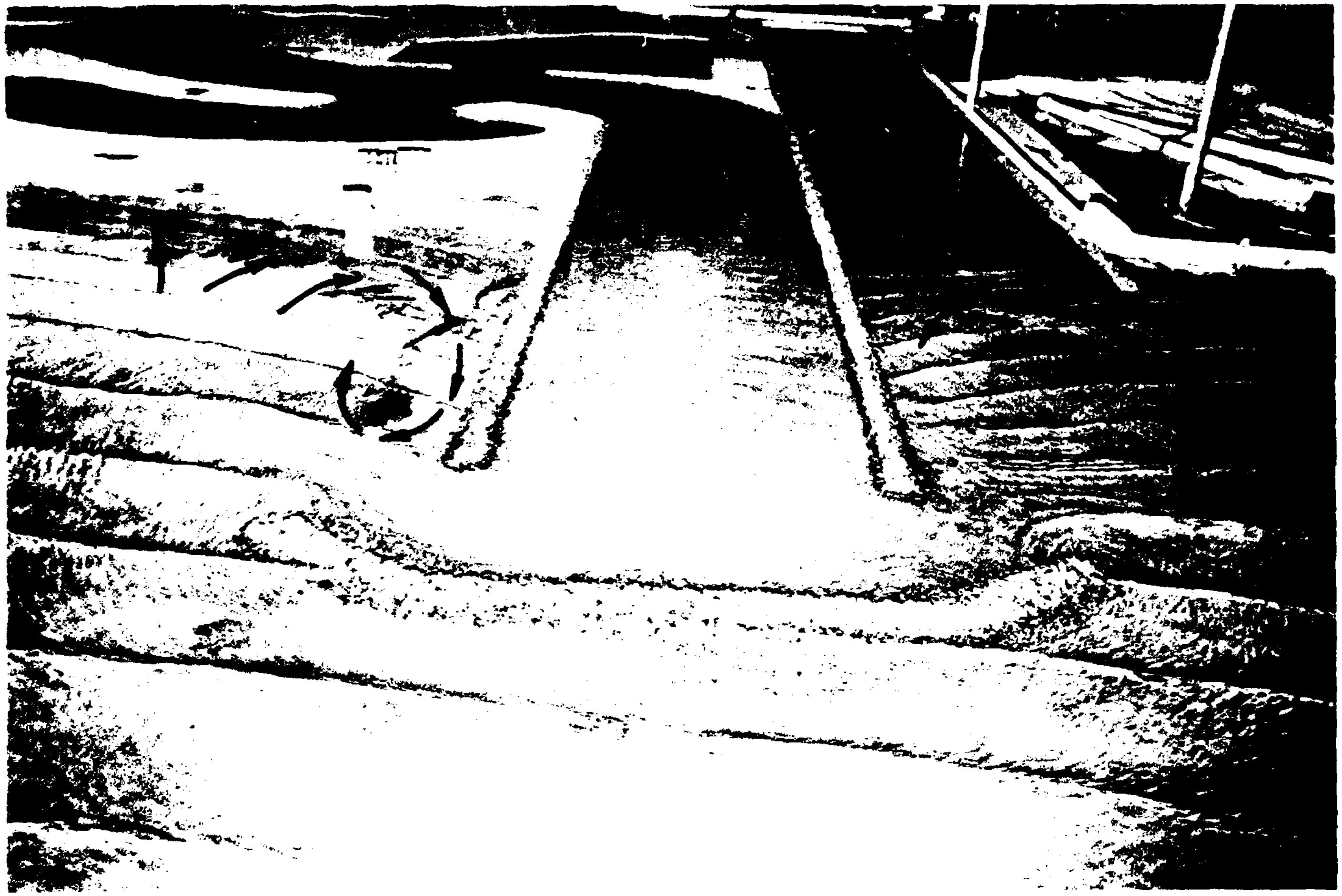


Photo 19. Typical tracer movement for Plan 3G resulting from 11-sec, 13-ft waves from west for maximum ebb

Photo 20. Typical tracer movement for Plan 3G resulting from 11-sec, 13-ft waves from west for maximum flood

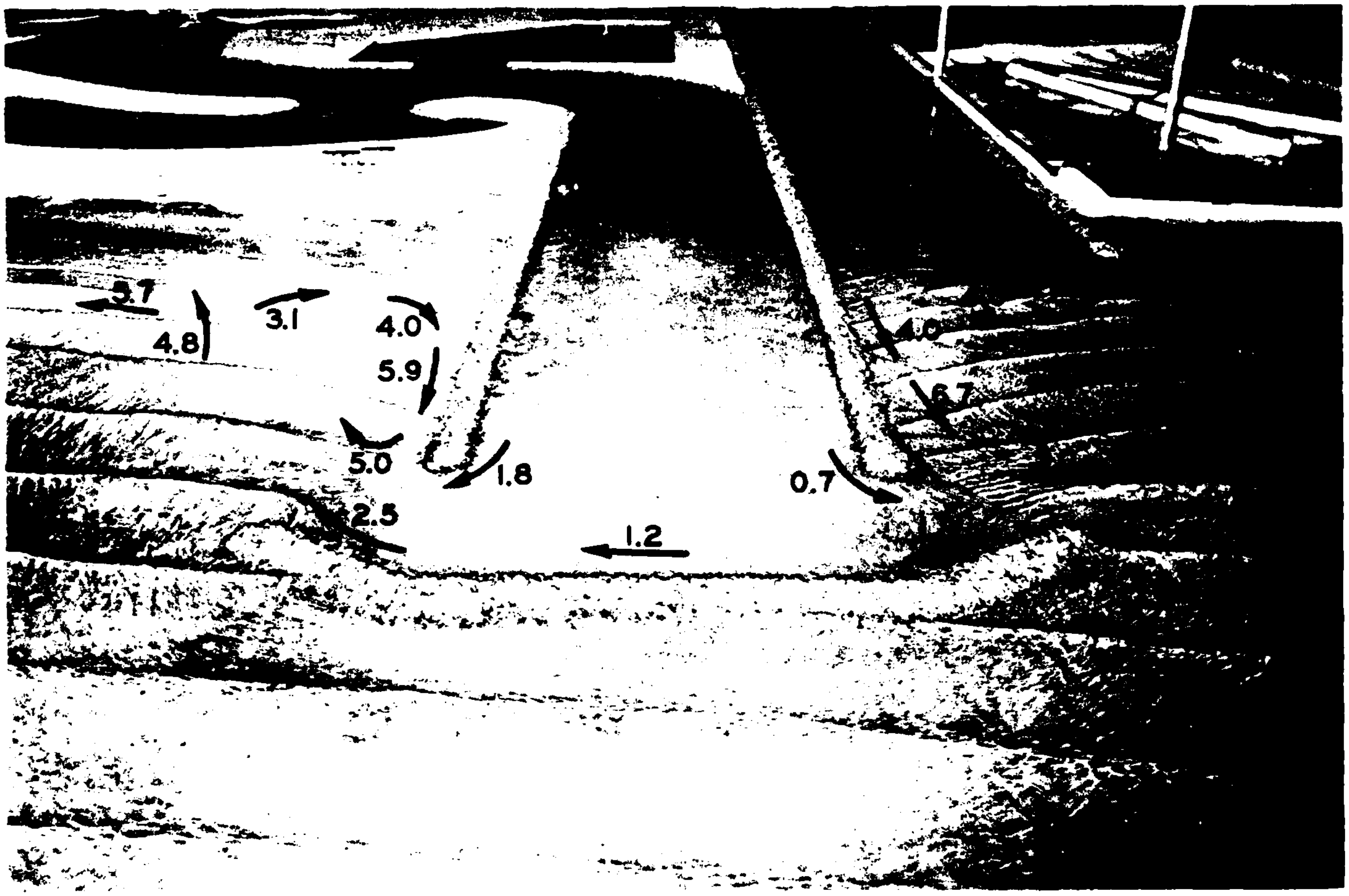


Photo 21. Typical wave and current patterns and current magnitudes (prototype feet per second) for Plan 3G; 11-sec, 13-ft waves from west at mllw

Photo 22. Typical tracer movement for Plan 3G resulting from 11-sec, 13-ft waves from west at mllw

Photo 23. Typical wave and current patterns and current magnitudes (prototype feet per second) for Plan 3G; 11-sec, 15-ft waves from northwest at mhhw

Photo 24. Typical tracer movement for Plan 3G resulting from 11-sec, 15-ft waves from northwest at mhhw

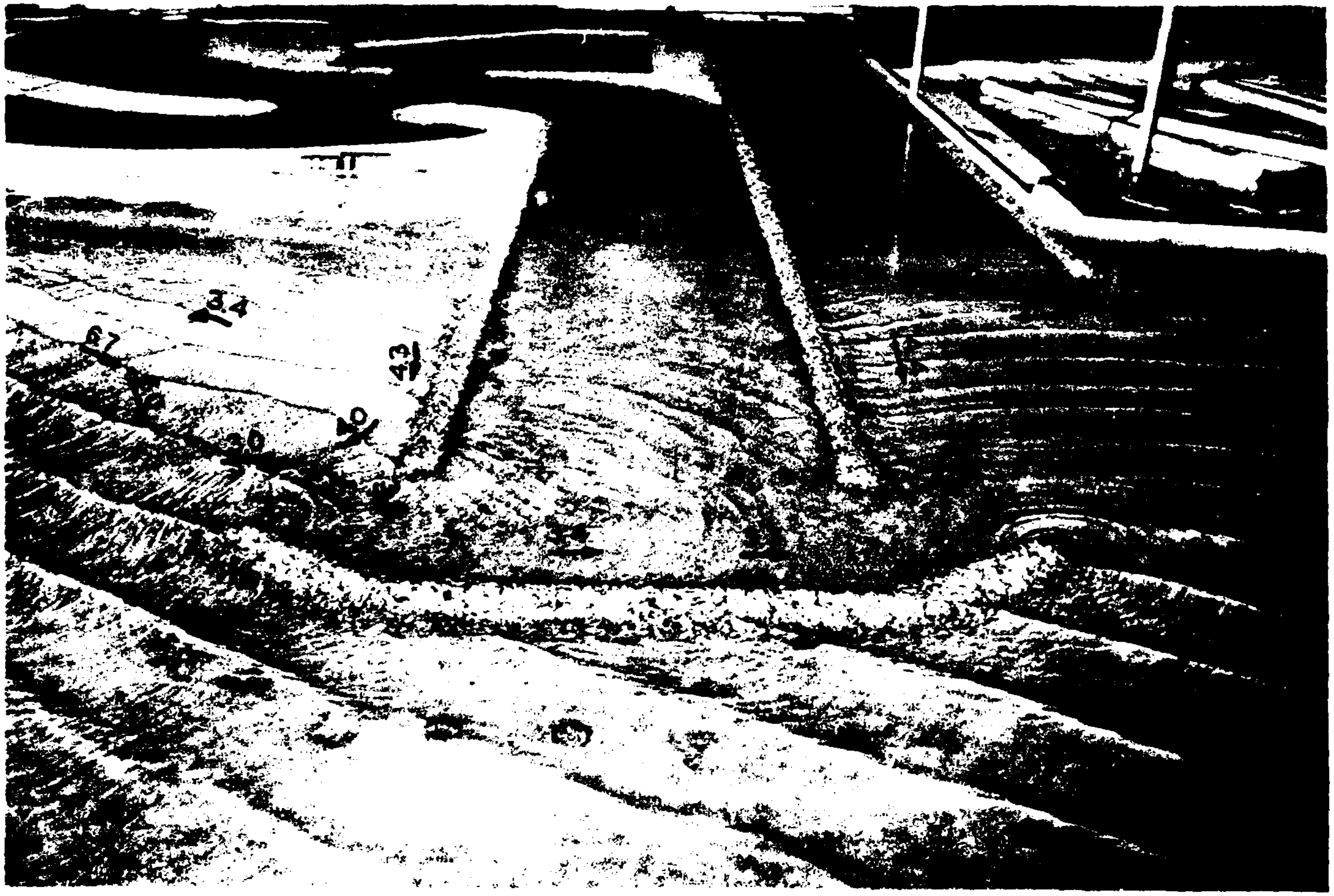


Photo 25. Typical wave and current patterns and current magnitudes (*prototype feet per second*) for Plan 3G; 9-sec, 13-ft waves from northwest at mhhw

Photo 26. Typical tracer movement for Plan 3G resulting from 9-sec, 13-ft waves from northwest at mhhw

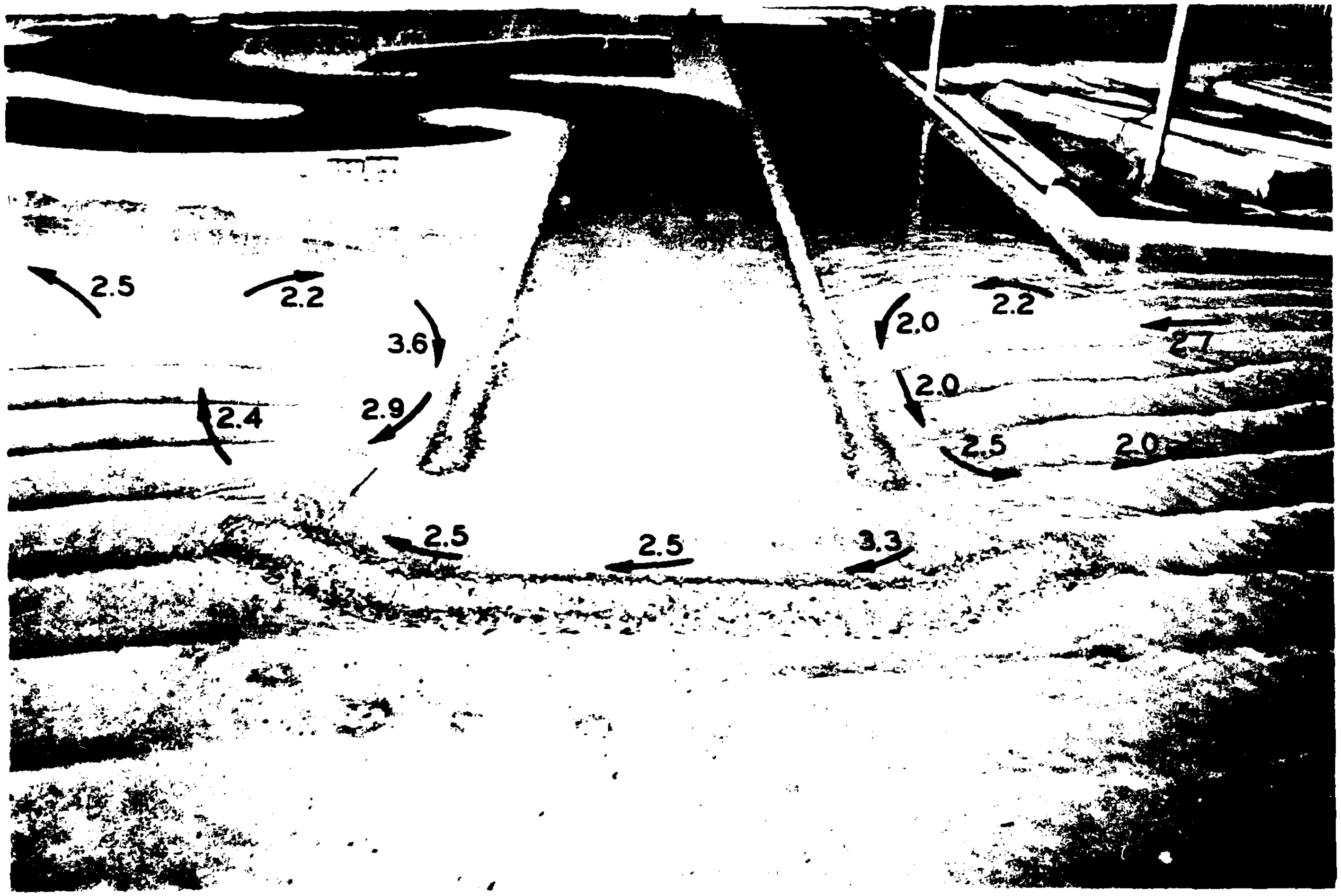


Photo 27. Typical wave and current patterns and current magnitudes (prototype feet per second) for Plan 3G; 9-sec, 11-ft waves from southwest at mhhw

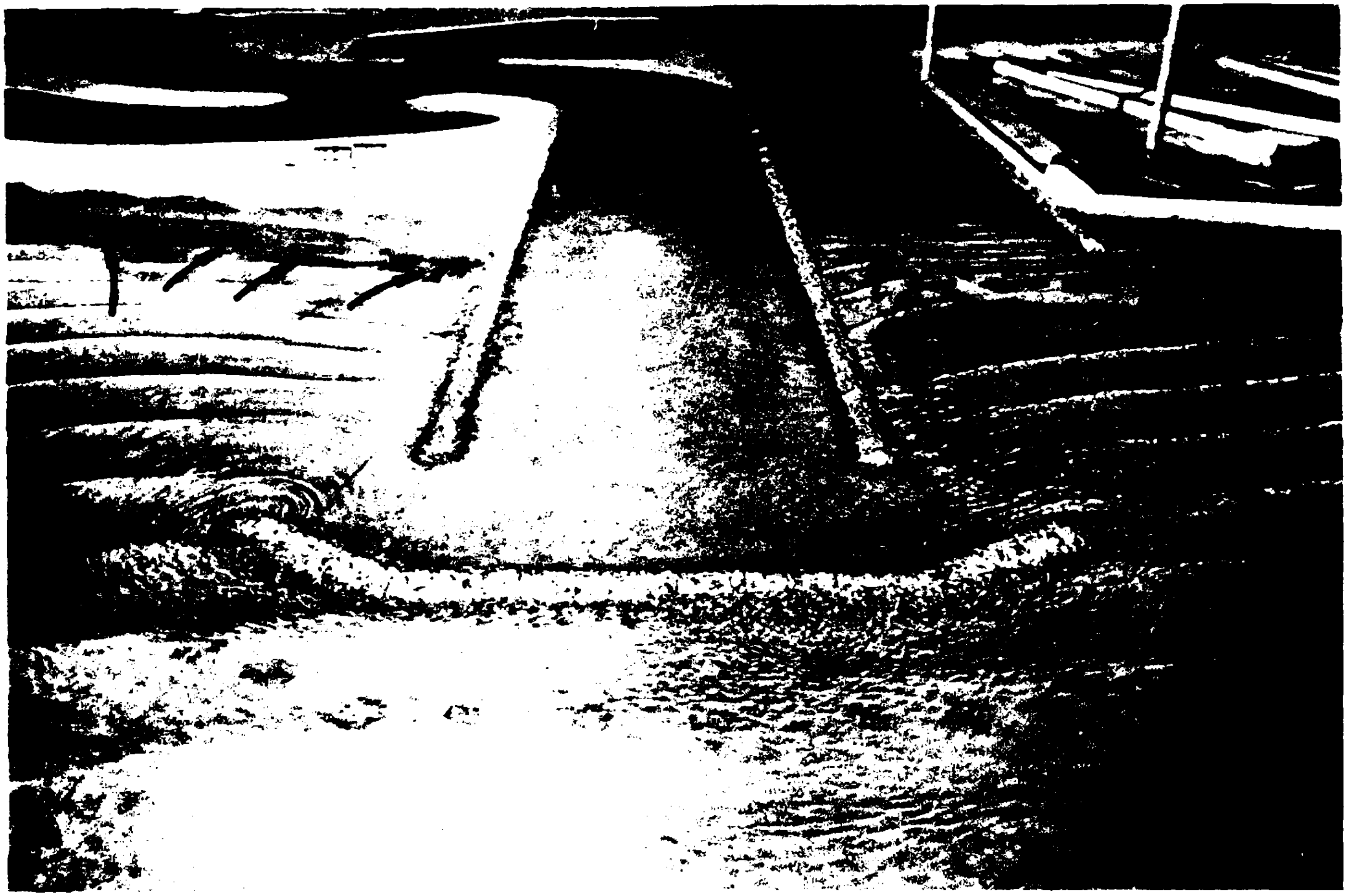


Photo 28. Typical tracer movement for Plan 3G resulting from 9-sec, 11-ft waves from southwest at mhhw

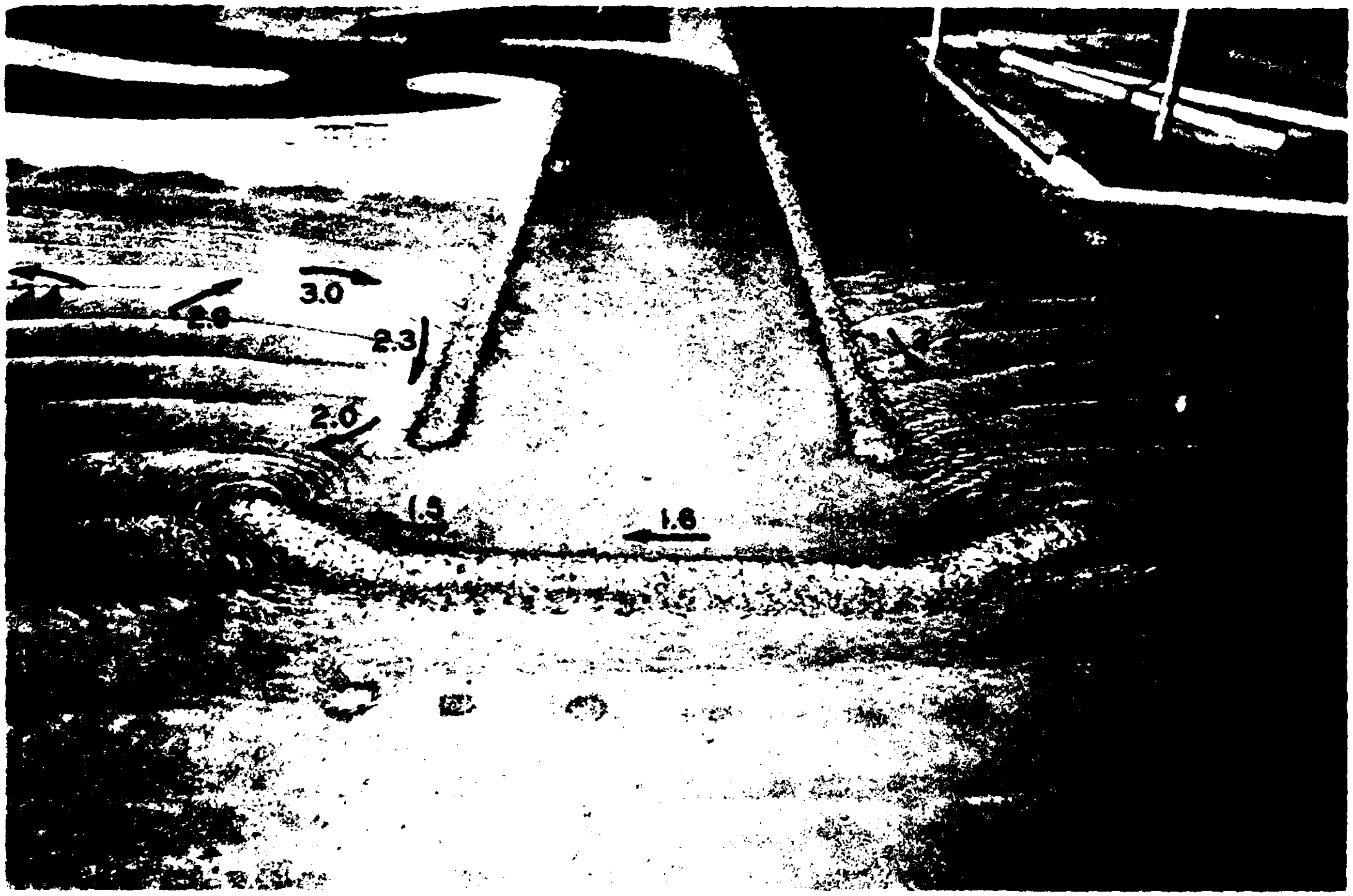


Photo 29. Typical wave and current patterns and current magnitudes (prototype feet per second) for Plan 3G; 9-sec, 11-ft waves from southwest at mllw

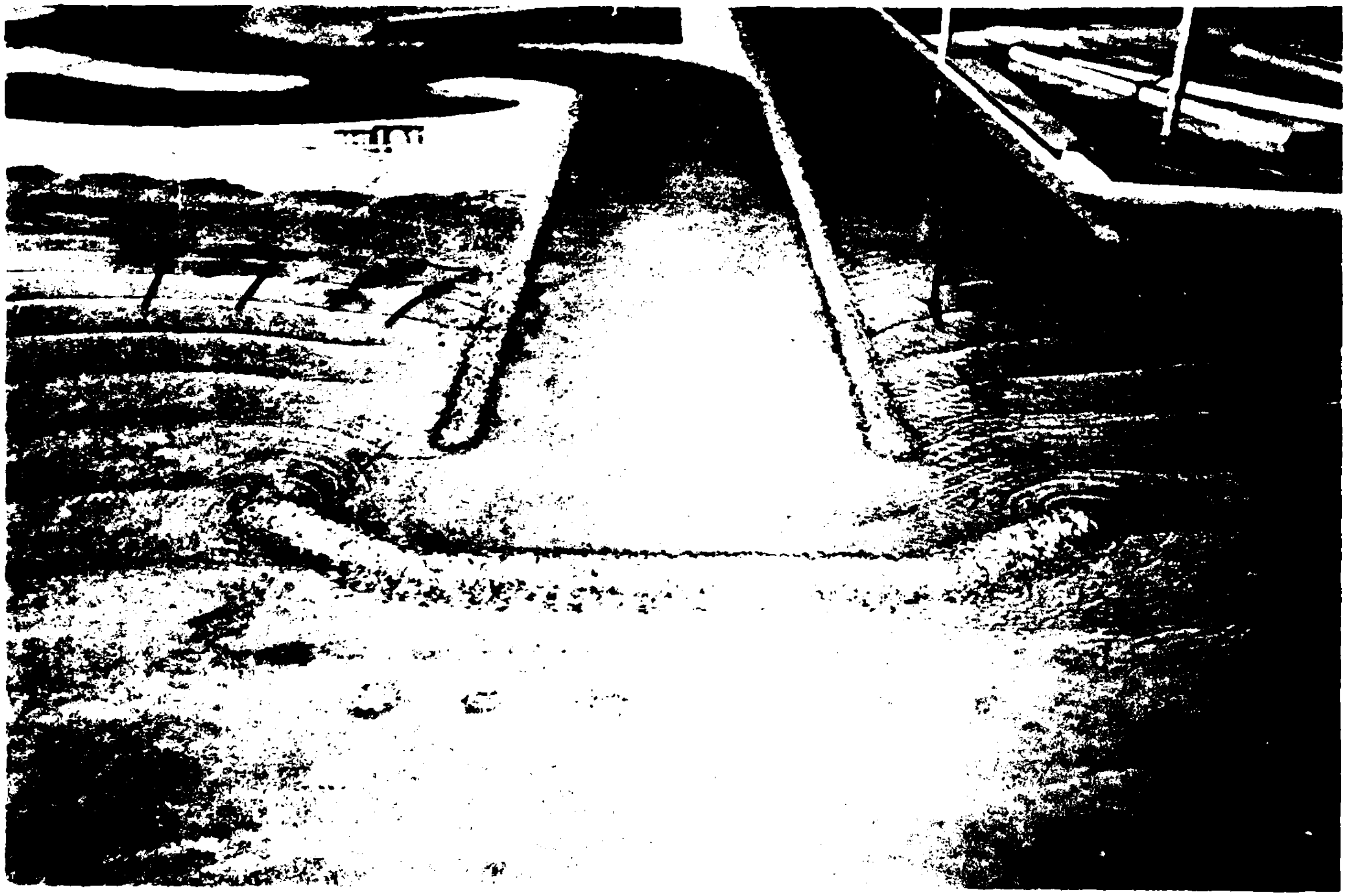


Photo 30. Typical tracer movement for Plan 3G resulting from 9-sec, 11-ft waves from southwest at mllw

Photo 31. Typical tracer movement for Plan 3G resulting from 11-sec, 6-ft waves from northwest at mllw (tested for 33 hr)

Photo 32. Typical tracer movement for Plan 3G resulting from 9-sec, 13-ft waves from northwest at mllw (tested for 11 hr starting with fillet in Photo 31)

Photo 33. Typical tracer movement for Plan 3G resulting from 13-sec, 15-ft waves from northwest at mllw (tested for 8 hr)

Photo 34. Typical tracer movement for Plan 3G resulting from 9-sec, 6-ft waves from southwest at mhhw (tested for 55 hr)

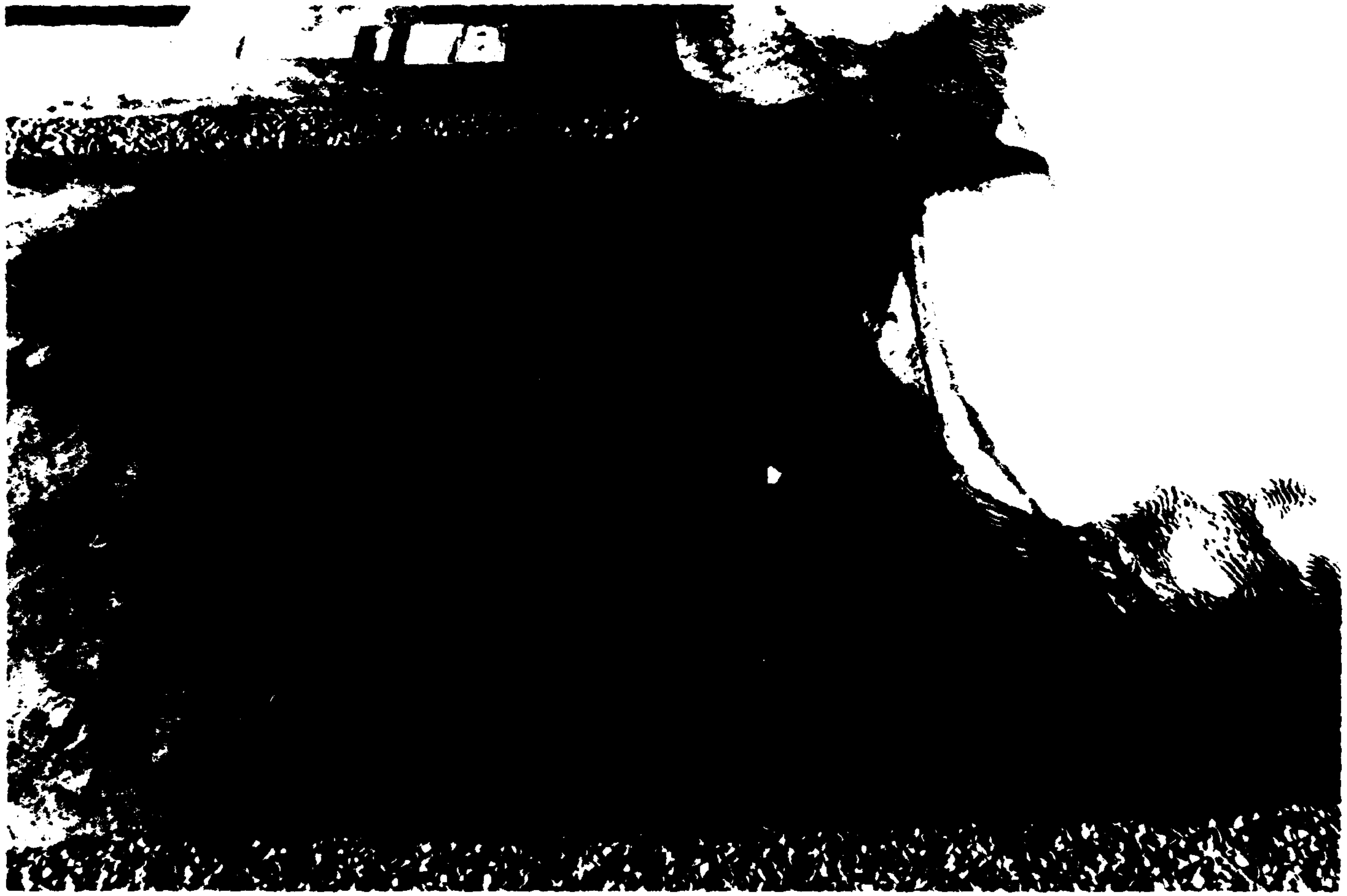


Photo 35. Typical tracer deposits for Plan 3G resulting from 9-sec, 6-ft waves from southwest at mhhw (after 55 hr)

Photo 36. Typical tracer movement for Plan 3G resulting from 13-sec, 11-ft waves from southwest at mhhw (tested for 8 hr starting with deposit in Photo 34)

Photo 37. Typical tracer deposits for Plan 3G resulting from 13-sec, 11-ft waves from southwest at mhhw (after 8 hr)

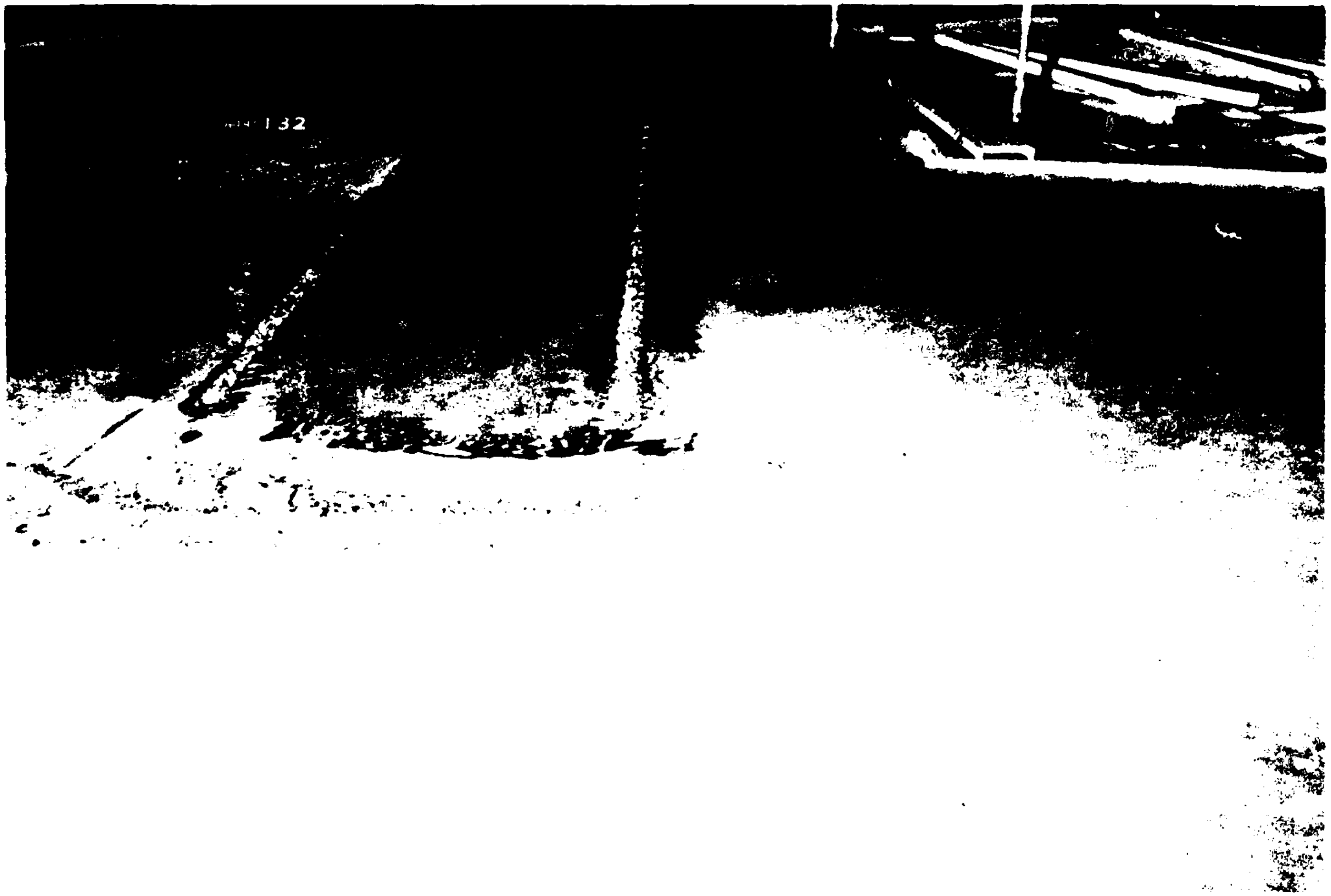


Photo 38. Typical tracer movement for Plan 3G resulting from 49,000-cfs river discharge (after 7 min)

Photo 39. Typical tracer movement for Plan 3G resulting from 97,000-cfs river discharge (after 3 min)

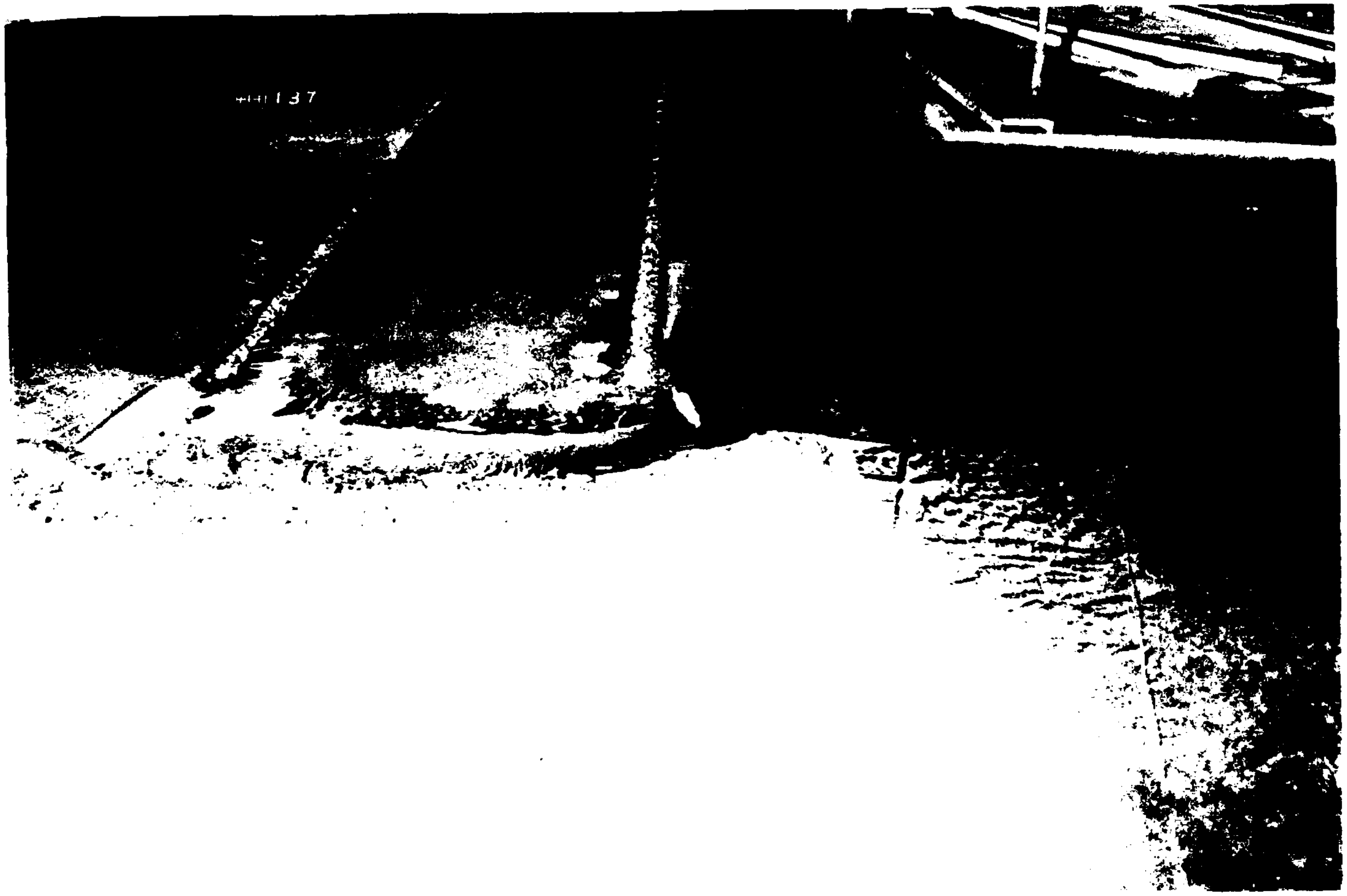


Photo 40. Typical tracer movement for Plan 3G resulting from 97,000-cfs river discharge (after 45 min)

Photo 41. Dispersal pattern of tracer plug for Plan 3G resulting from 97,000-cfs river discharge (after 45 min)

Photo 42. Close-up of dispersal pattern of tracer plug for Plan 3G resulting from 97,000-cfs river discharge (after 45 min)

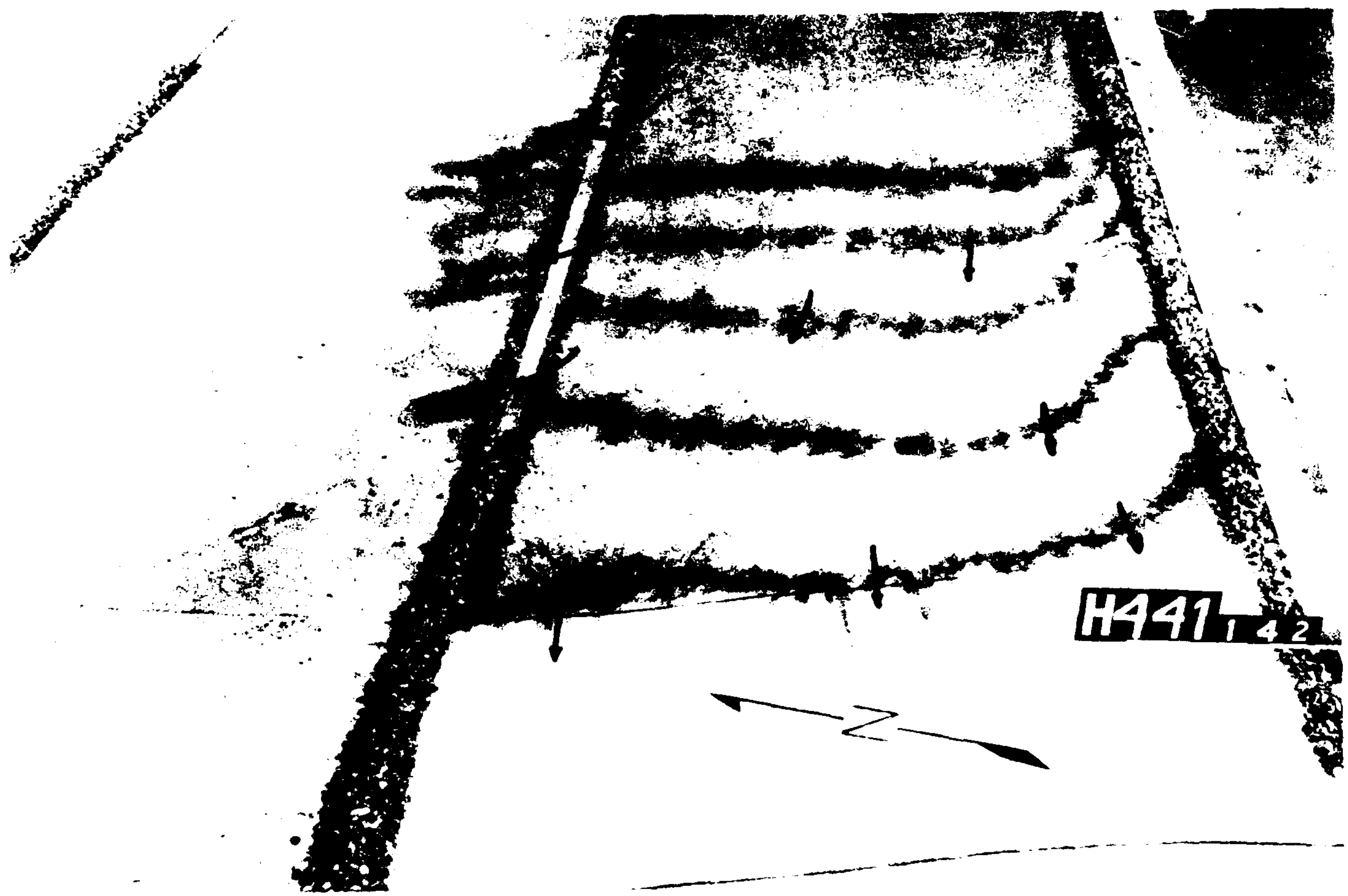


Photo 43. Typical current patterns for Plan 6B resulting from 49,000-cfs river discharge

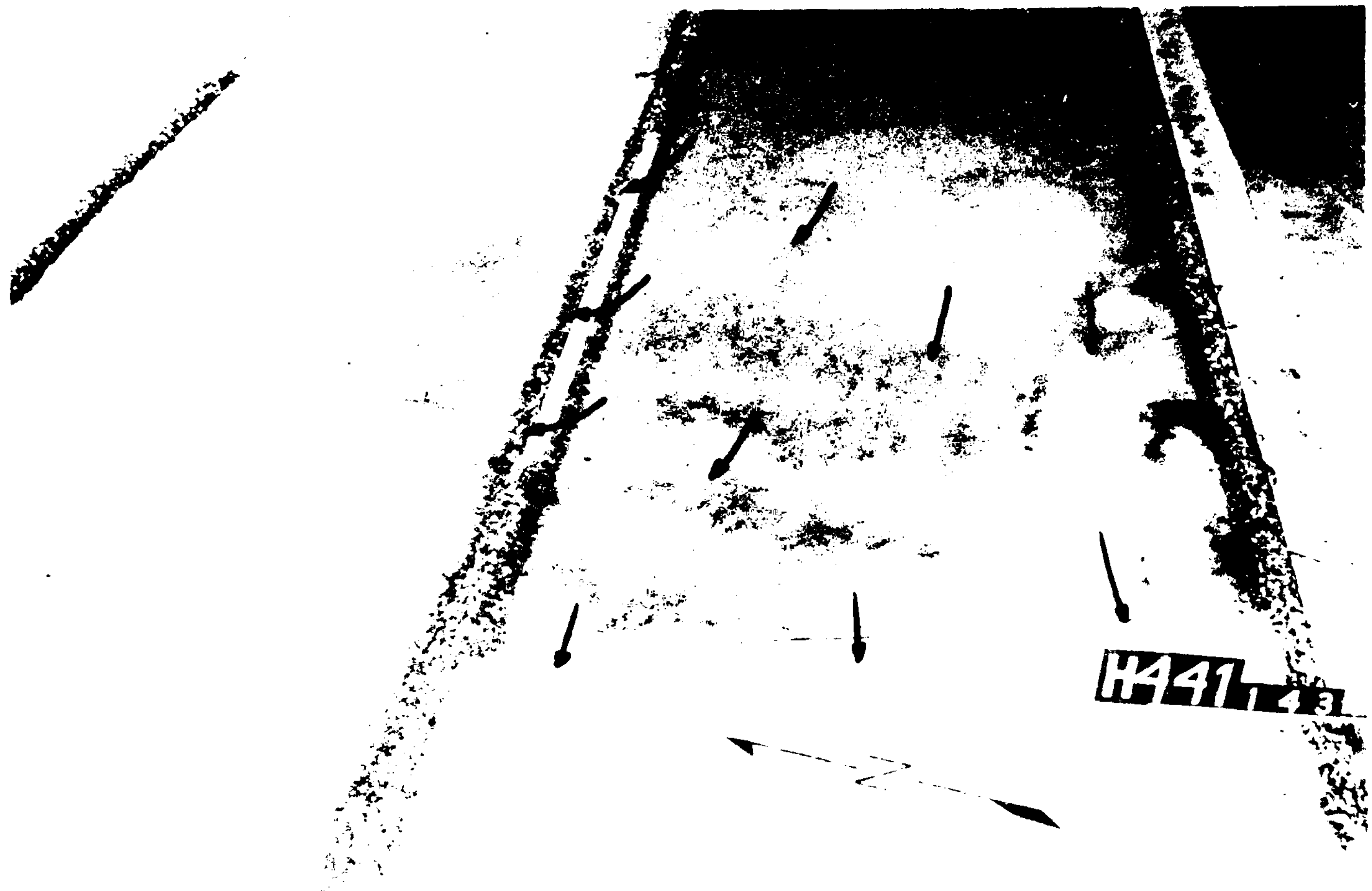


Photo 44. Typical current patterns for Plan 6B resulting from 97,000-cfs river discharge

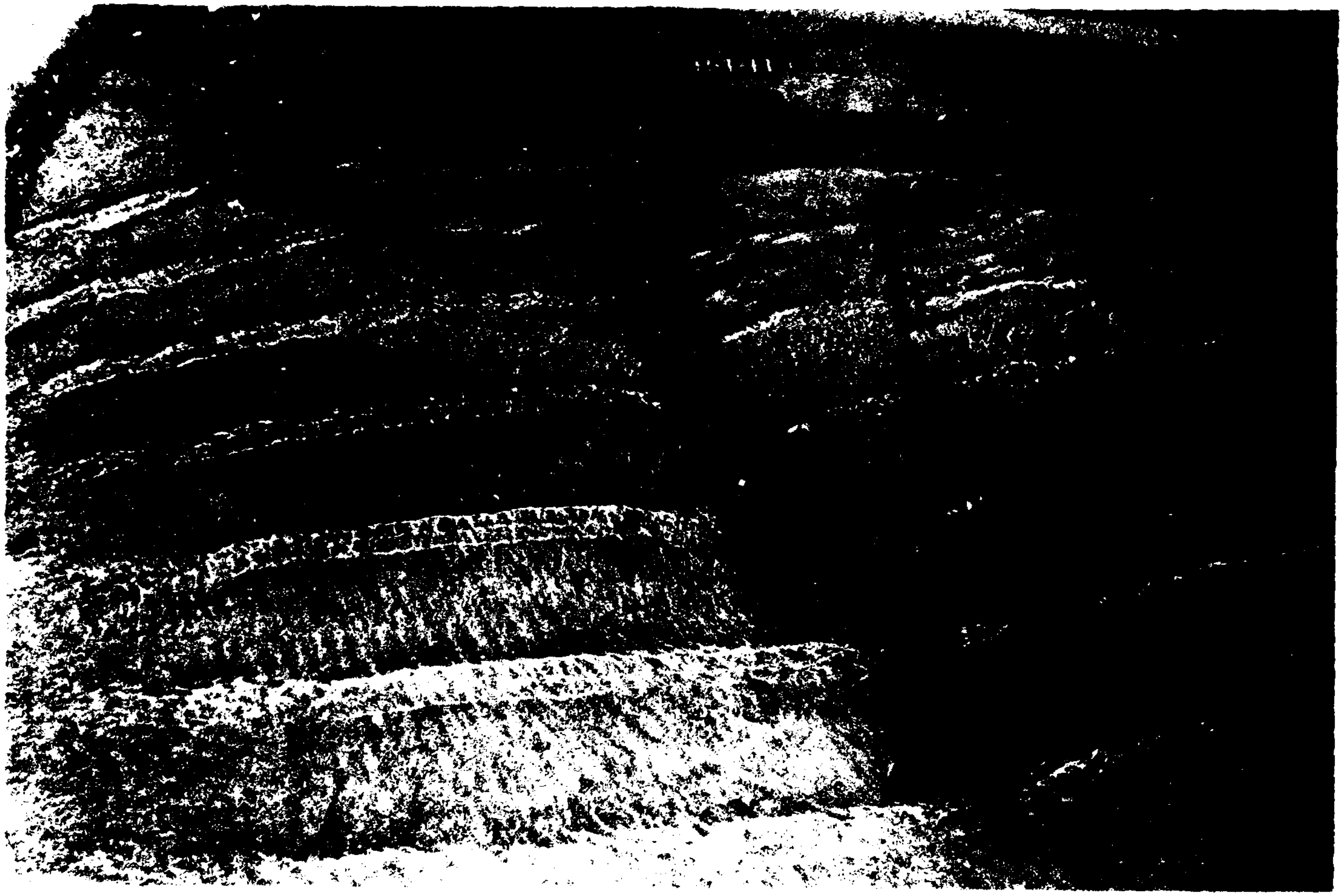


Photo 45. Typical tracer movement for Plan 7 resulting from 9-sec, 11-ft waves from southwest at mhhw

Photo 46. Typical tracer movement for Plan 7 resulting from 11-sec, 11-ft waves from southwest at mhhw

Photo 47. Typical tracer movement for Plan 7C resulting from 9-sec, 11-ft waves from southwest at mhhw

Photo 48. Typical tracer movement for Plan 7C resulting from 11-sec, 11-ft waves from southwest at mhhw

Photo 49. Formation of tracer plug at San Diego River mouth resulting from 9-sec, 6-ft waves from southwest at mhhw (after 3 hr)

Photo 50. Formation of tracer plug at San Diego River mouth resulting from 9-sec, 6-ft waves from southwest at mhhw (after 4 hr)

Photo 51. Formation of tracer plug at San Diego River mouth resulting from 9-sec, 6-ft waves from southwest at mhhw (after 5 hr)

Photo 52. Formation of tracer plug at San Diego River mouth resulting from 9-sec, 6-ft waves from southwest at mhhw (after 6 hr)

Photo 53. Formation of tracer plug at San Diego River mouth resulting from 9-sec, 6-ft waves from southwest at mhhw (after 7 hr)

Photo 54. Formation of tracer plug at San Diego River mouth resulting from 9-sec, 6-ft waves from southwest at mhhw (after 8 hr)

Photo 55. Formation of tracer plug at San Diego River mouth resulting from 9-sec, 6-ft waves from southwest at mhhw (after 9 hr)

Photo 56. Formation of tracer plug at San Diego River mouth resulting from 9-sec, 6-ft waves from southwest at mhhw (after 10 hr)

Photo 57. Formation of tracer plug at San Diego River mouth resulting from 9-sec, 6-ft waves from southwest at mhhw (after 11 hr)

Photo 58. Formation of tracer plug at San Diego River mouth resulting from 9-sec, 6-ft waves from southwest at mhhw (after 12 hr)

Photo 59. Formation of tracer plug at San Diego River mouth resulting from 9-sec, 6-ft waves from southwest at mhhw (after 13 hr)

Photo 60. Formation of tracer plug at San Diego River mouth resulting from 9-sec, 6-ft waves from southwest at mhhw (after 14 hr)

Photo 61. Formation of tracer plug at San Diego River mouth resulting from 9-sec, 6-ft waves from southwest at mhhw (after 15 hr)

Photo 62. Formation of tracer plug at San Diego River mouth resulting from 9-sec, 6-ft waves from southwest at mhhw (after 16 hr)

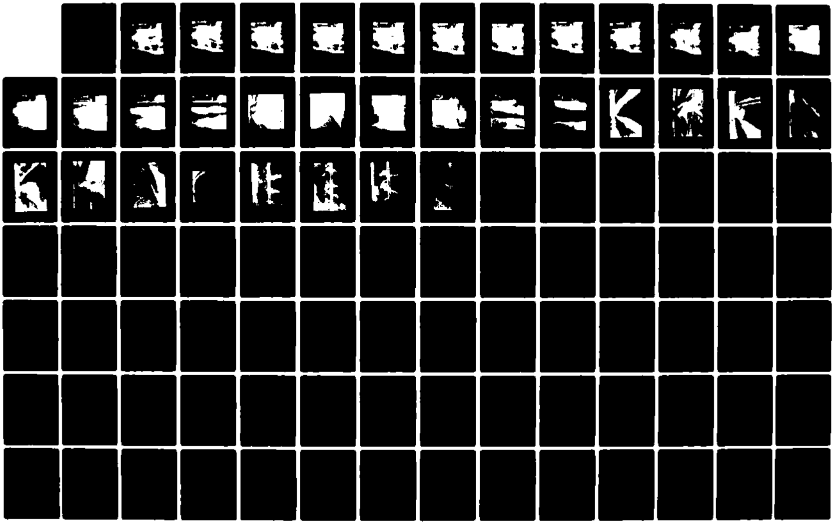
Photo 63. Formation of tracer plug at San Diego River mouth resulting from 9-sec, 6-ft waves from southwest at mhhw (after 17 hr)

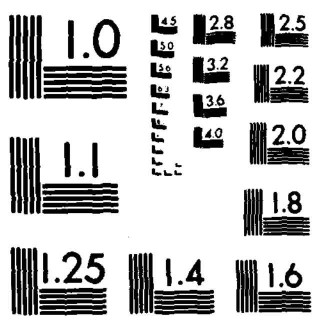
Photo 64. Formation of tracer plug at San Diego River mouth resulting from 9-sec, 6-ft waves from southwest at mhhw (after 18 hr)

Photo 65. Formation of tracer plug at San Diego River mouth resulting from 9-sec, 6-ft waves from southwest at mhhw (after 19 hr)

Photo 66. Formation of tracer plug at San Diego River mouth resulting from 9-sec, 6-ft waves from southwest at mhhw (after 20 hr)

AD-A136 371 MISSION BAY HARBOR CALIFORNIA DESIGN FOR WAVE AND SURGE 3/4
PROTECTION AND FL..(U) ARMY ENGINEER WATERWAYS
EXPERIMENT STATION VICKSBURG MS HYDRA.. C R CURREN
UNCLASSIFIED JUN 83 WES/TR/HL-83-17 F/G 13/2 NL





MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

Photo 67. Formation of tracer plug at San Diego River mouth resulting from 9-sec, 6-ft waves from southwest at mhhw (after 21 hr)

Photo 68. Formation of tracer plug at San Diego River mouth resulting from 9-sec, 6-ft waves from southwest at mhhw (after 22 hr)

Photo 69. Formation of tracer plug at San Diego River mouth resulting from 9-sec, 6-ft waves from southwest at mhhw (after 23 hr)

Photo 70. Formation of tracer plug at San Diego River mouth resulting from 9-sec, 6-ft waves from southwest at mhhw (after 24 hr)

Photo 71. Formation of tracer plug at San Diego River mouth resulting from 9-sec, 6-ft waves from southwest at mhhw (after 27 hr)

Photo 72. Formation of tracer plug at San Diego River mouth resulting from 9-sec, 6-ft waves from southwest at mhhw (after 28 hr)

Photo 73. Formation of tracer plug at San Diego River mouth resulting from 9-sec, 6-ft waves from southwest at mhhw (after 29 hr)

Photo 74. Formation of tracer plug at San Diego River mouth resulting from 9-sec, 6-ft waves from southwest at mhhw (after 30 hr)

Photo 75. Formation of tracer plug at San Diego River mouth resulting from 9-sec, 6-ft waves from southwest at mhhw (after 31 hr)

Photo 76. Formation of tracer plug at San Diego River mouth resulting from 9-sec, 6-ft waves from southwest at mhhw (after 32 hr)

Photo 77. Formation of tracer plug at San Diego River mouth resulting from 9-sec, 6-ft waves from southwest at mhhw (after 33 hr)

Photo 78. Formation of tracer plug at San Diego River mouth resulting from 9-sec, 6-ft waves from southwest at mhhw (after 40 hr)

Photo 79. Washout of tracer plug (no pilot channel) for Plan 3G resulting from 49,000-cfs river discharge

Photo 80. Washout of tracer plug (no pilot channel) for Plan 3G resulting from 49,000-cfs river discharge

Photo 81. Washout of tracer plug (no pilot channel) for Plan 3G resulting from 97,000-cfs river discharge

Photo 82. Washout of tracer plug (no pilot channel) for Plan 3G resulting from 97,000-cfs river discharge

Photo 83. Dispersal pattern (looking east) of tracer plug (no pilot channel) for Plan 3G resulting from 97,000-cfs river discharge (after 1 hr)

Photo 84. Dispersal pattern (looking north) of tracer plug (no pilot channel) for Plan 3G resulting from 97,000-cfs river discharge (after 1 hr)

Photo 85. Tracer plug with center pilot channel before testing

Photo 86. Washout of tracer plug (with center pilot channel) for Plan 3G resulting from 27,000-cfs river discharge

Photo 87. Washout of tracer plug (with center pilot channel) for Plan 3G resulting from 49,000-cfs river discharge

Photo 88. Washout of tracer plug (with center pilot channel) for Plan 3G resulting from 97,000-cfs river discharge

Photo 89. Dispersal pattern (looking west) of tracer plug (with center pilot channel) for Plan 3G resulting from 97,000-cfs river discharge (after 1 hr)

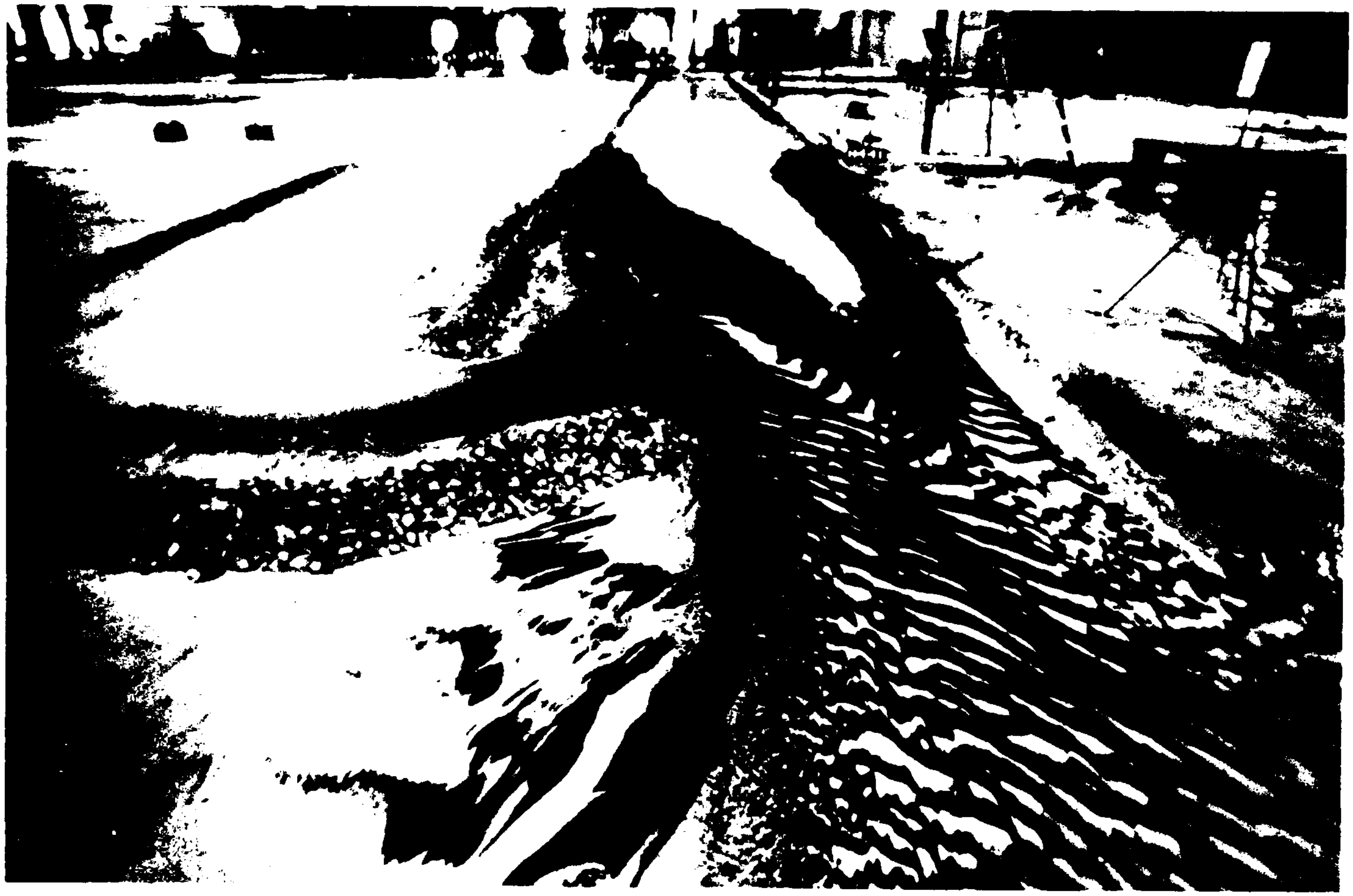


Photo 90. Dispersal pattern (looking east) of tracer plug (with center pilot channel) for Plan 3G resulting from 97,000-cfs river discharge (after 1 hr)

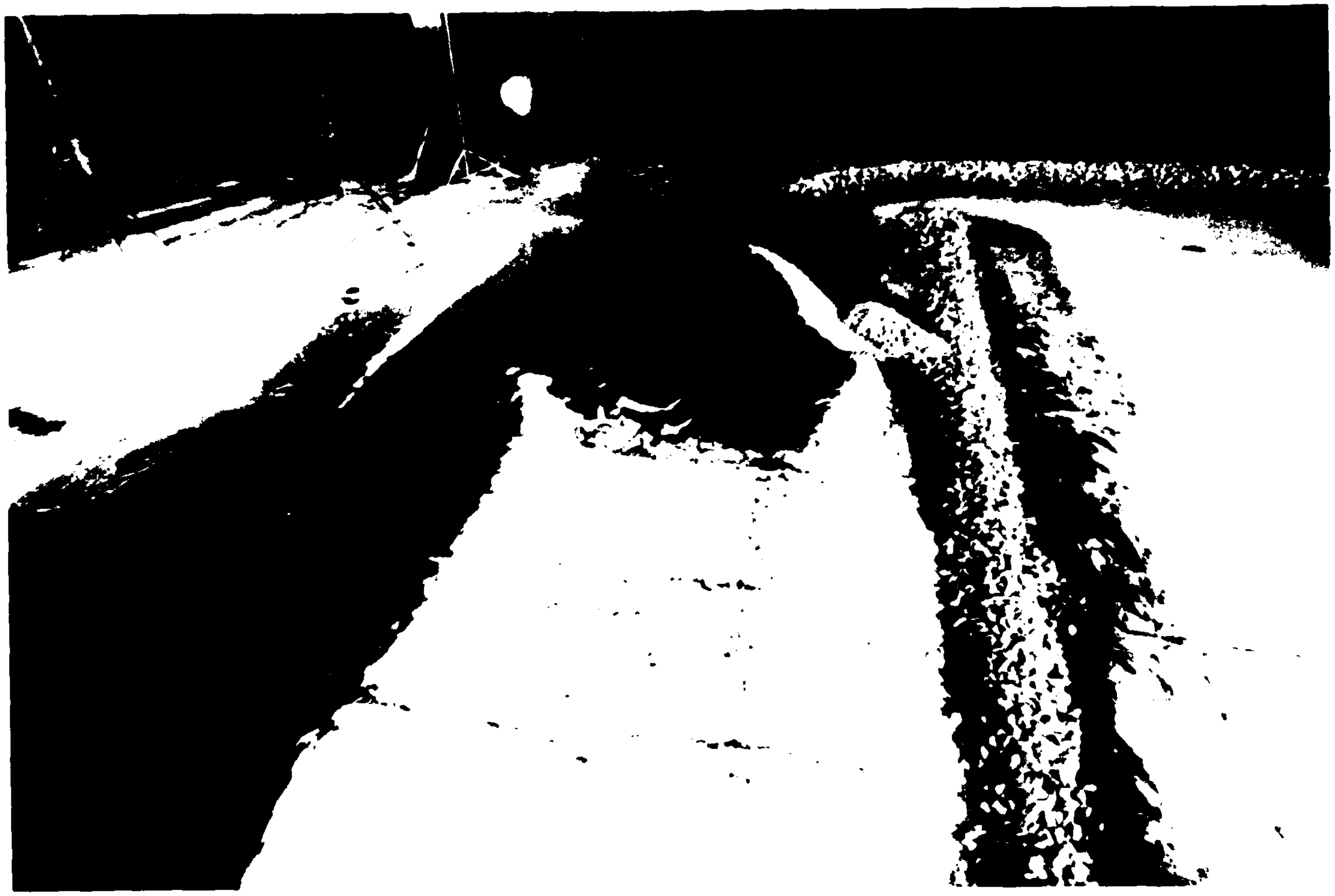


Photo 91. Dispersal pattern (looking west) of tracer plug for Plan 8 resulting from 97,000-cfs river discharge (after 1 hr)

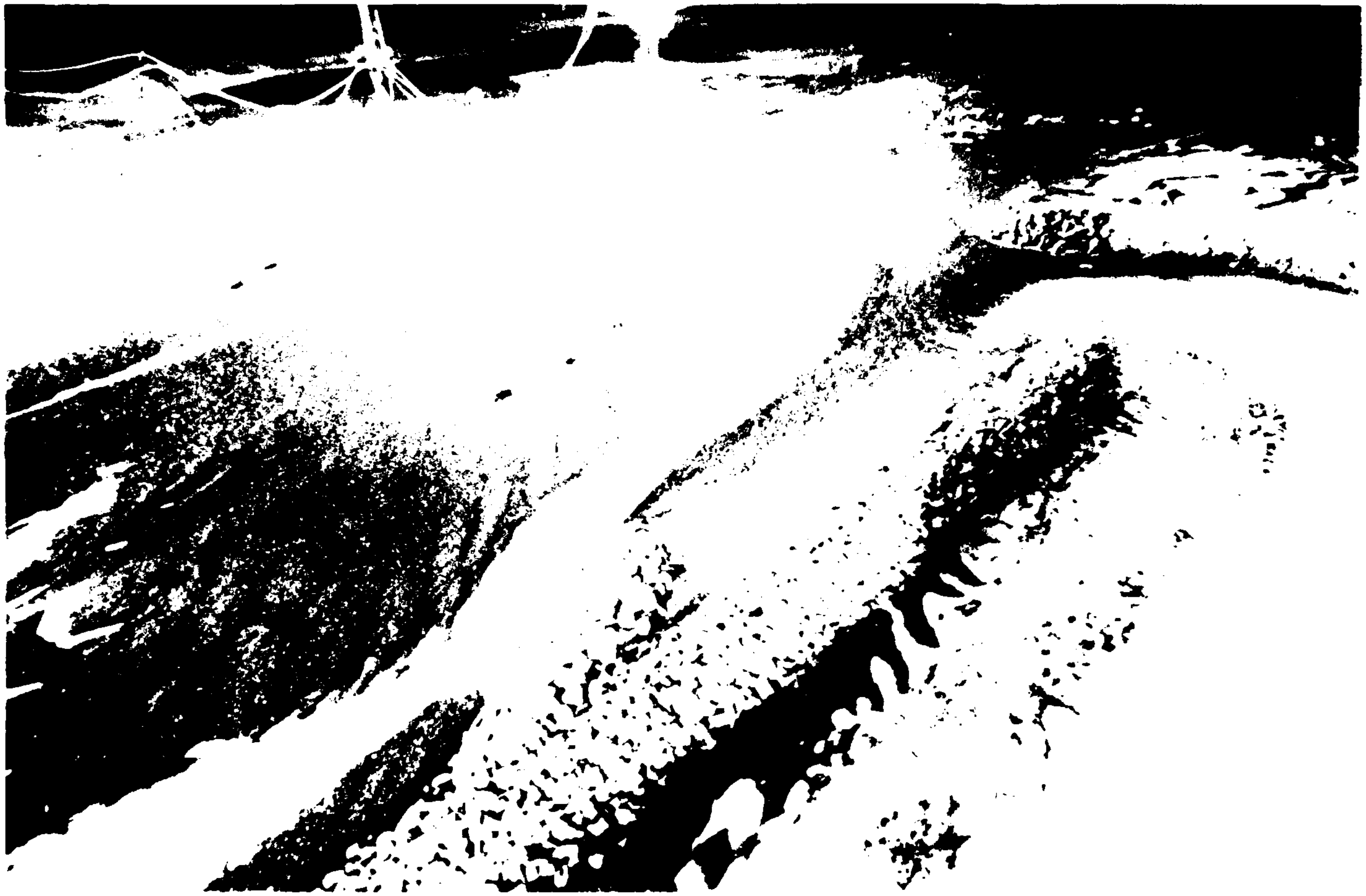


Photo 92. Dispersal pattern (looking southwest) of tracer plug for Plan 8 resulting from 97,000-cfs river discharge (after 1 hr)

Photo 93. Dispersal pattern (looking west) of tracer plug for Plan 8A resulting from 97,000-cfs river discharge (after 1 hr)

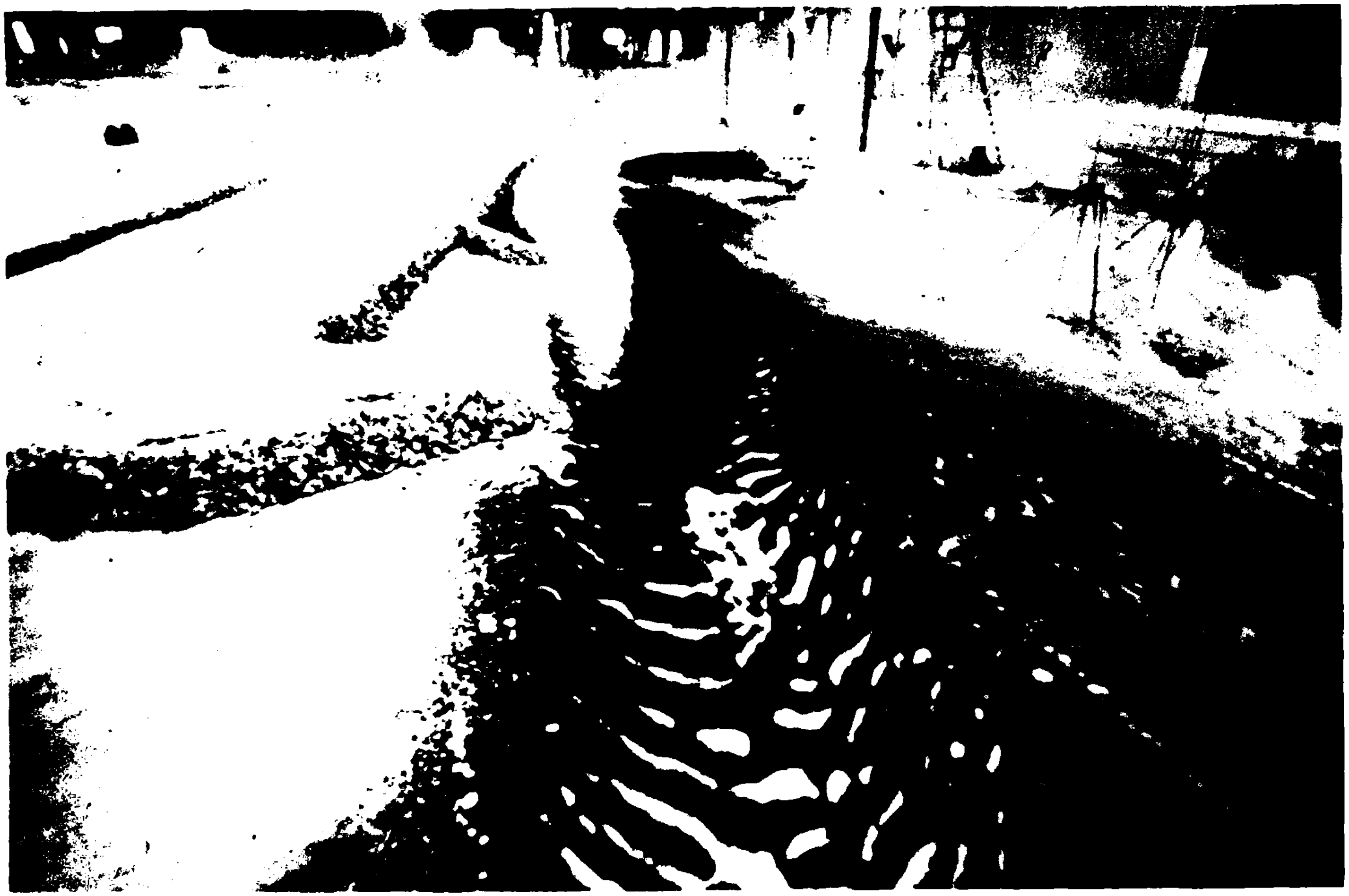


Photo 94. Dispersal pattern (looking east) of tracer plug for Plan 8A resulting from 97,000-cfs river discharge (after 1 hr)

Photo 95. Dispersal pattern (looking northeast) of tracer plug for Plan 8A resulting from 97,000-cfs river discharge (after 1 hr)

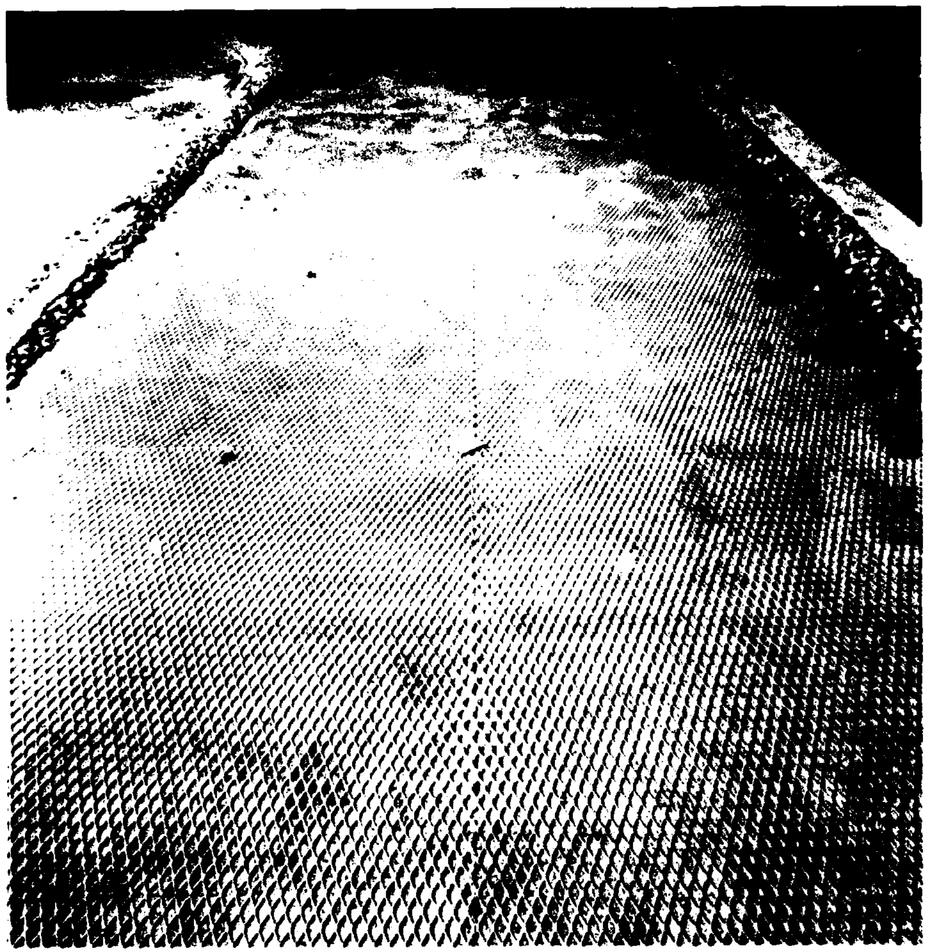


Photo 96. Model roughness installed in the San Diego River channel

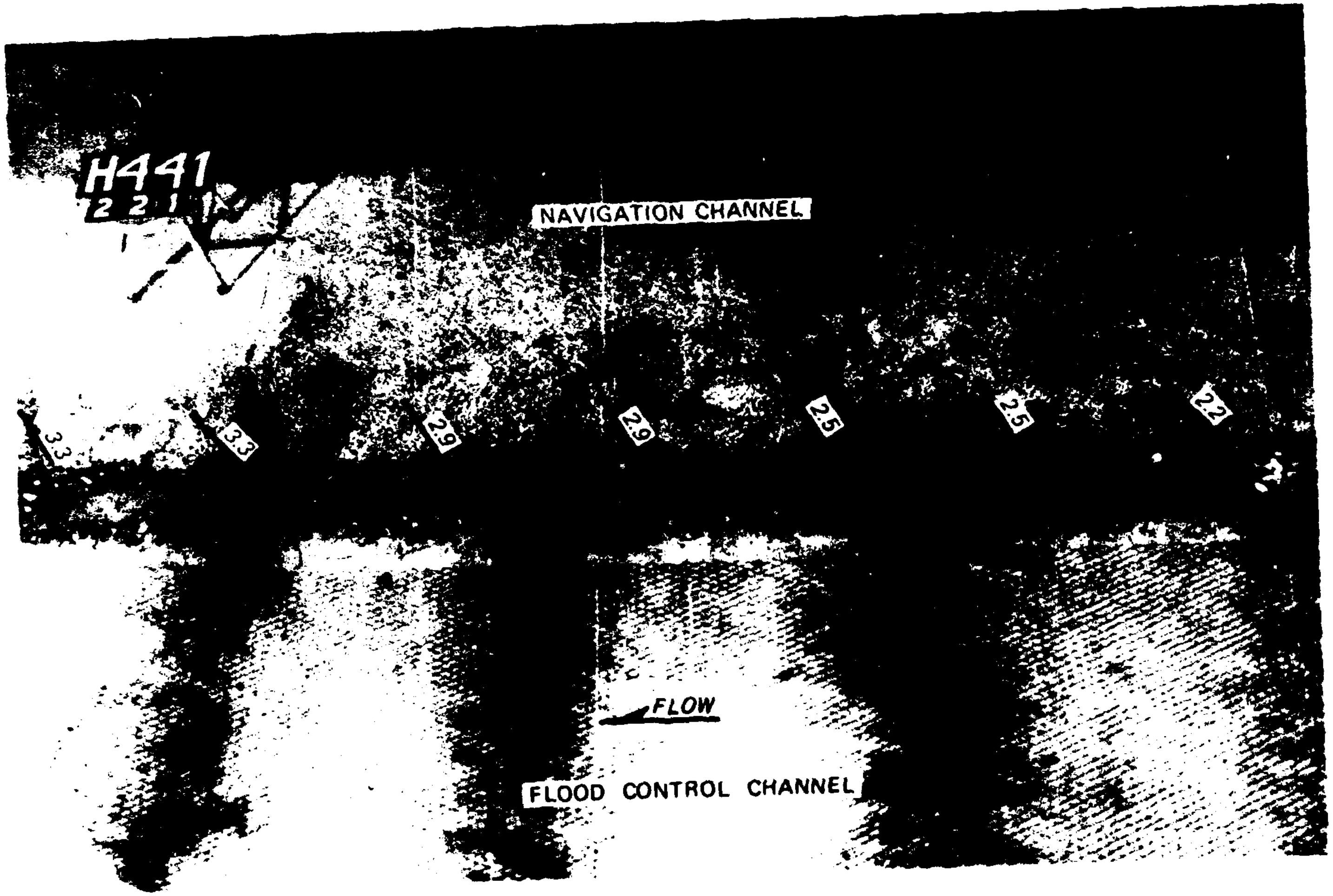


Photo 97. Typical current patterns and magnitudes (prototype feet per second) for Plan 6C resulting from a 49,000-cfs river discharge at mllw

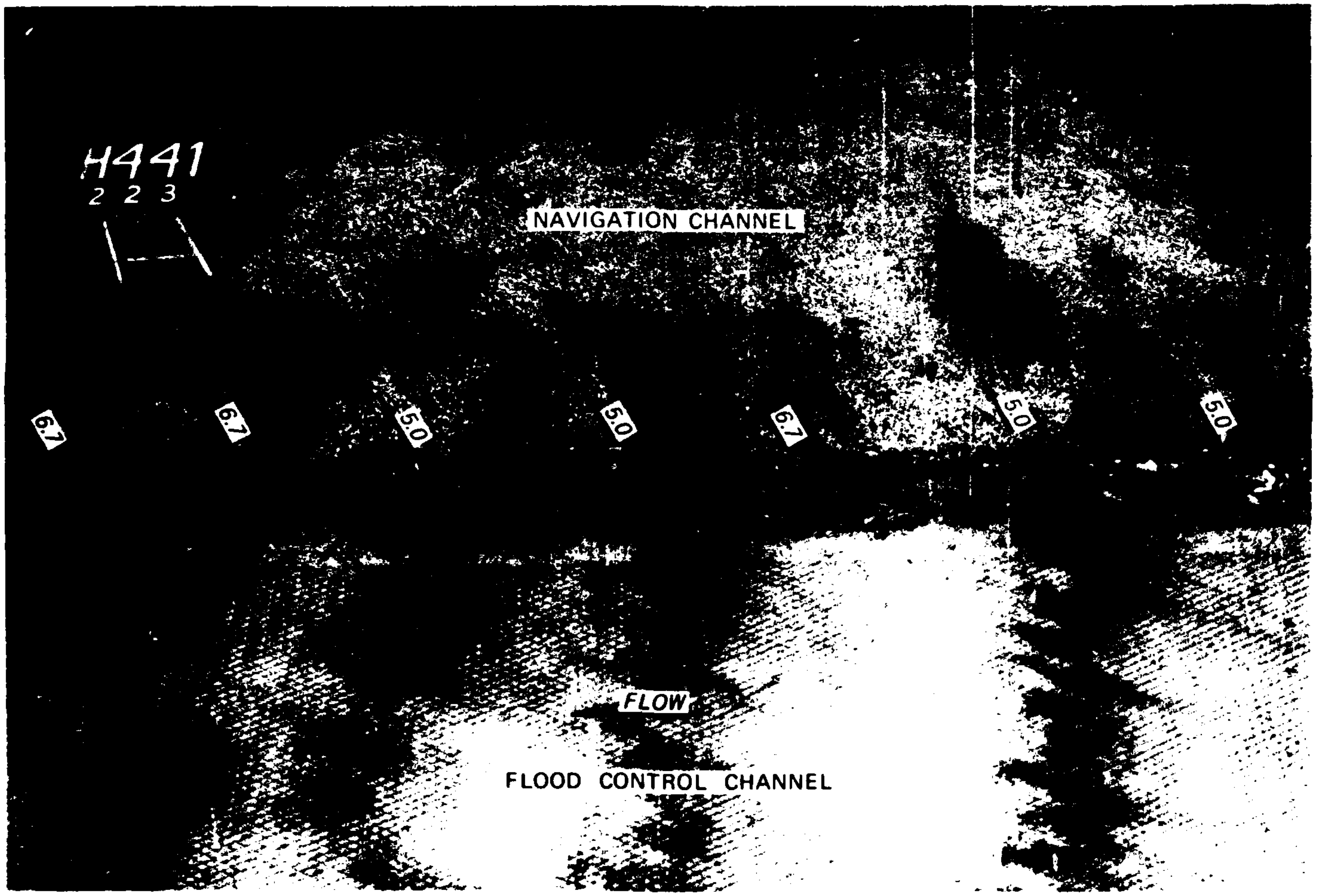


Photo 98. Typical current patterns and magnitudes (prototype feet per second) for Plan 6C resulting from a 97,000-cfs river discharge at mllw

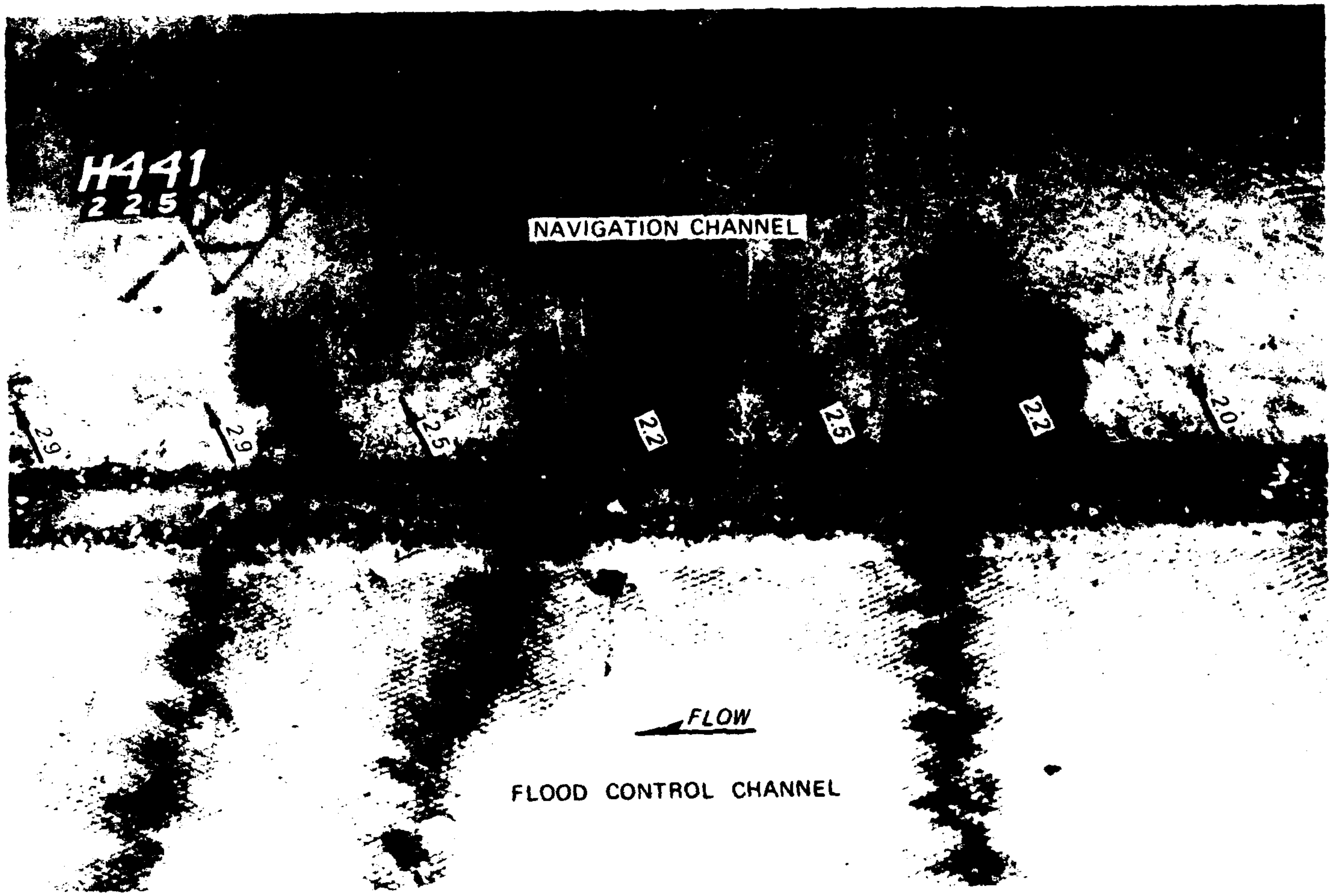


Photo 99. Typical current patterns and magnitudes (prototype feet per second) for Plan 6C resulting from a 49,000-cfs river discharge at mhhw

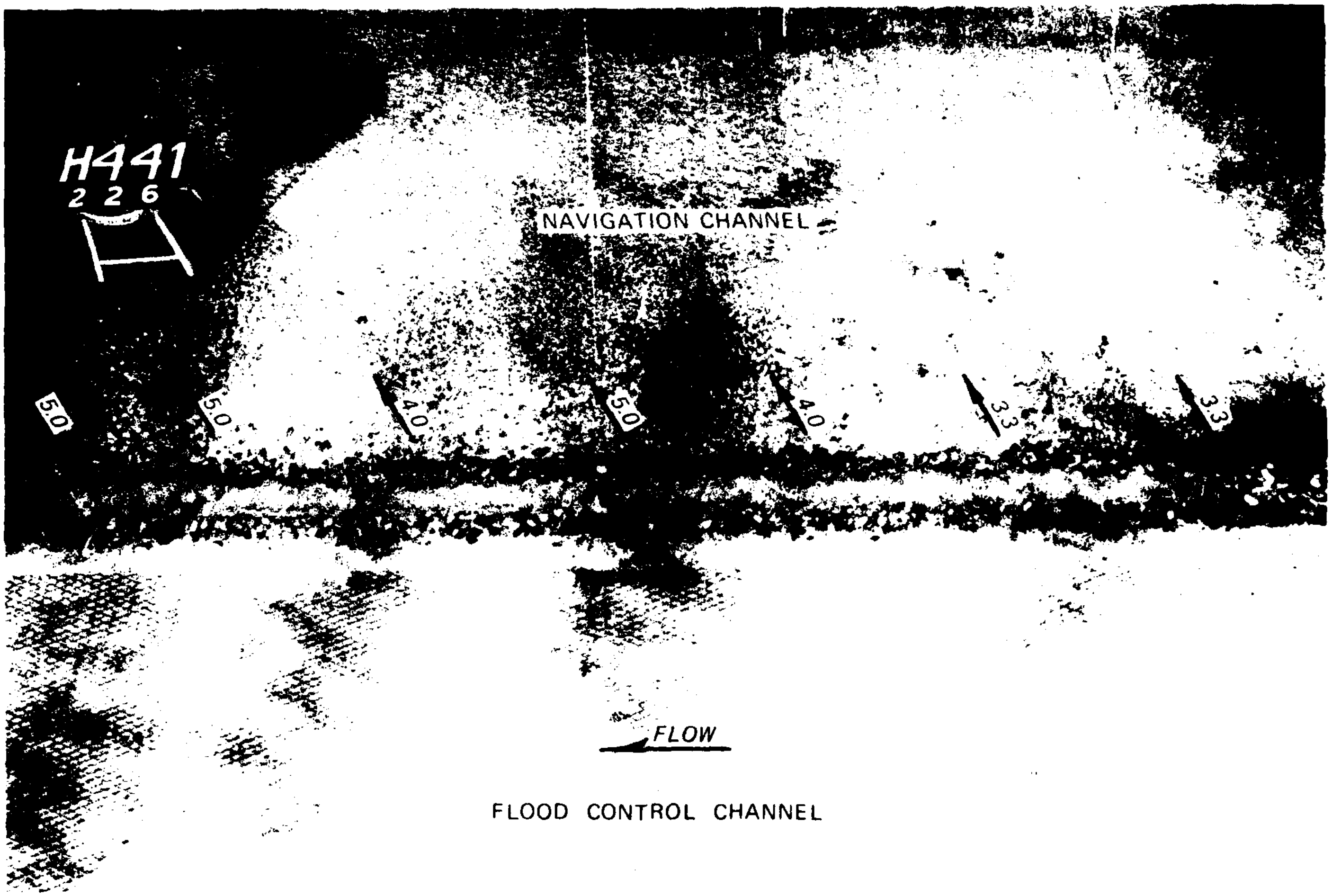
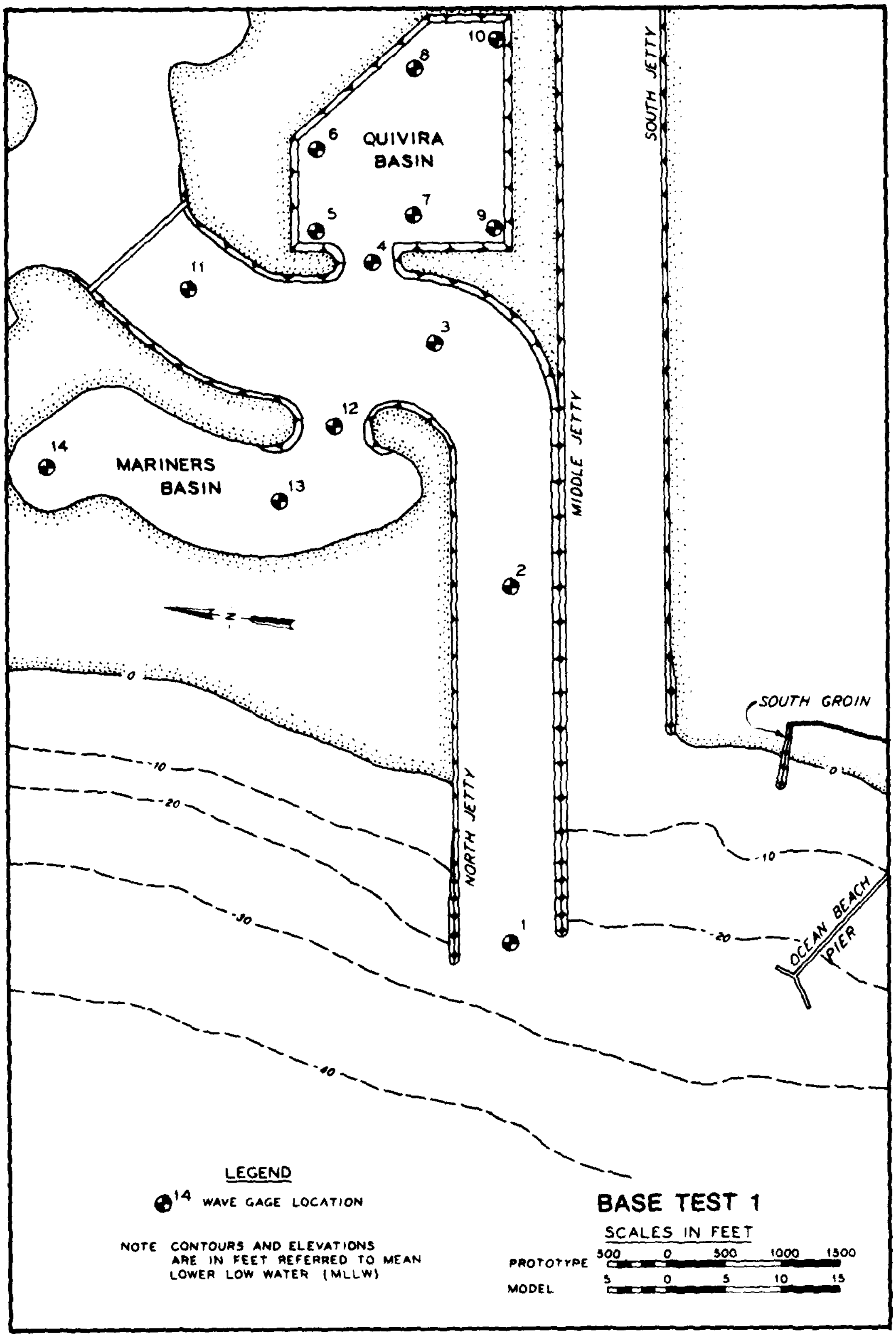


Photo 100. Typical current patterns and magnitudes (prototype feet per second) for Plan 6C resulting from a 97,000-cfs river discharge at mhhw


QUIVIRA BASIN
MARINERS BASIN
SOUTH JETTY
MIDDLE JETTY
NORTH JETTY
SOUTH GROIN
OCEAN BEACH PIER
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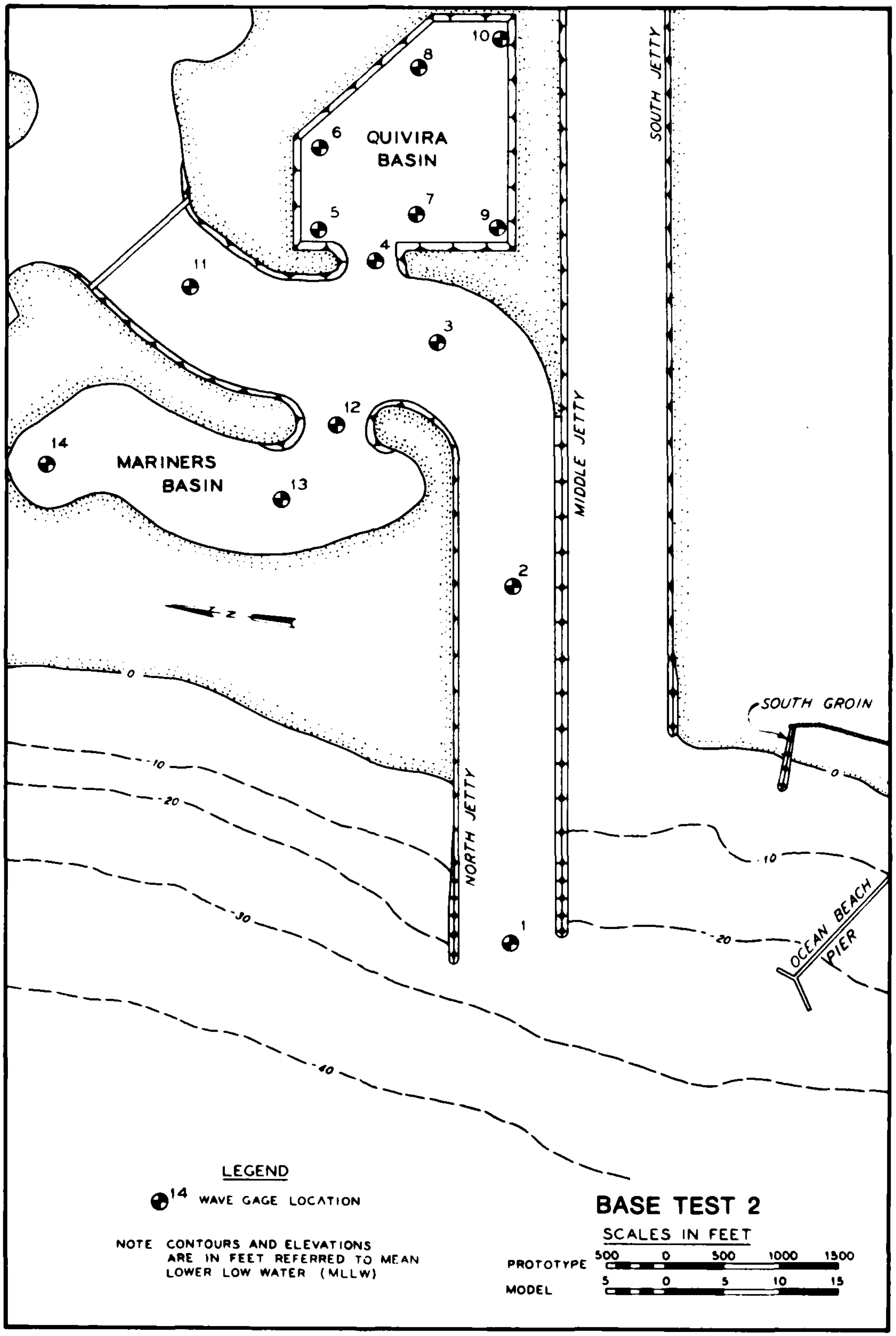
11
12
13
14
0
-10
-20
-30
-40
LEGEND
14 WAVE GAGE LOCATION
NOTE CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER (MLLW)
BASE TEST 1
SCALES IN FEET
PROTOTYPE 500 0 500 1000 1500
MODEL 5 0 5 10 15

PLATE 1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MIDDLE JETTY
NORTH JETTY
SOUTH GROIN
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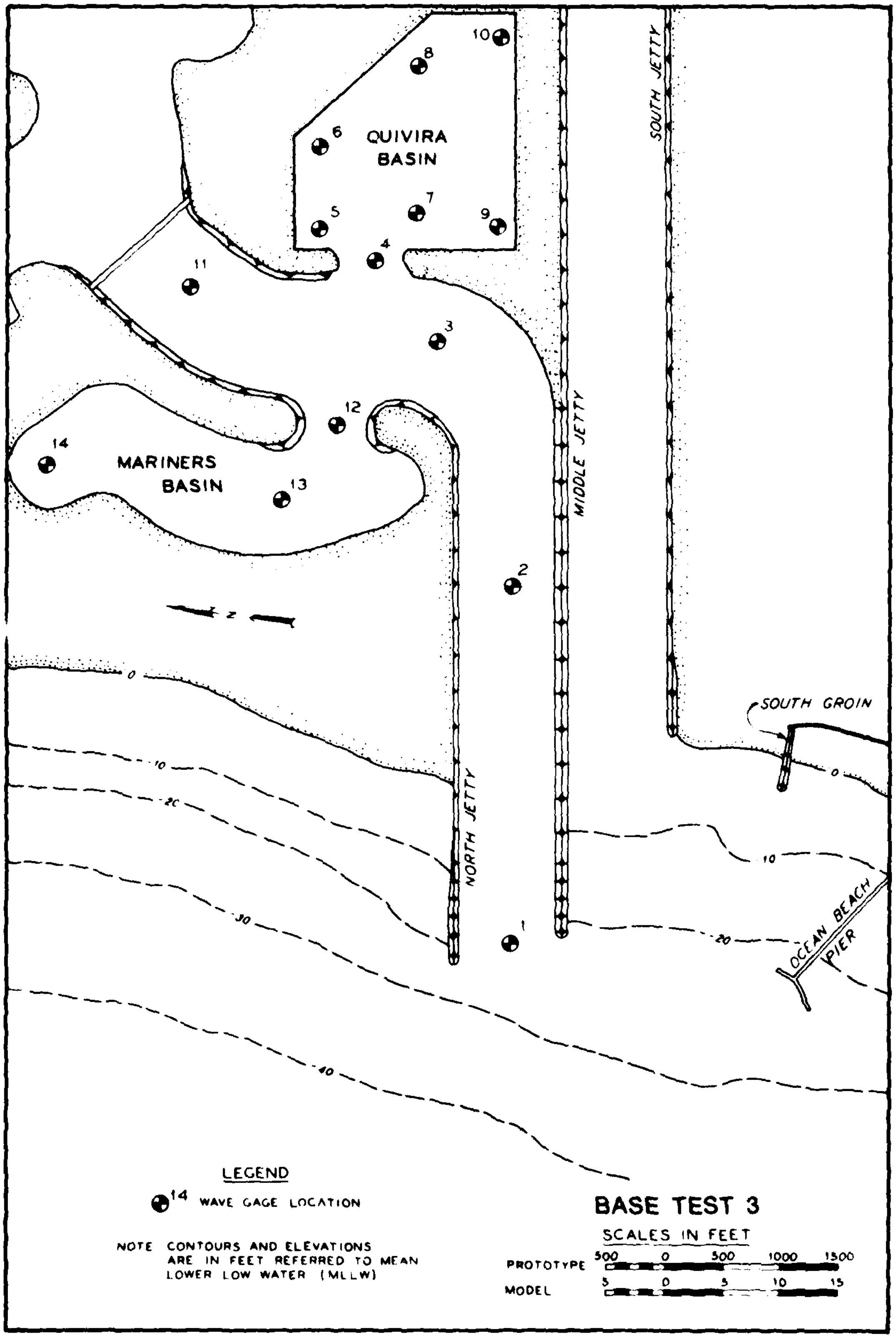
LEGEND
14 WAVE GAGE LOCATION
NOTE CONTOURS AND ELEVATIONS
ARE IN FEET REFERRED TO MEAN
LOWER LOW WATER (MLLW)
BASE TEST 2
SCALES IN FEET
PROTOTYPE 500 0 500 1000 1500
MODEL 5 0 5 10 15

PLATE 2



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MARINERS BASIN
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MIDDLE JETTY
NORTH JETTY
SOUTH GROIN
OCEAN BEACH PIER
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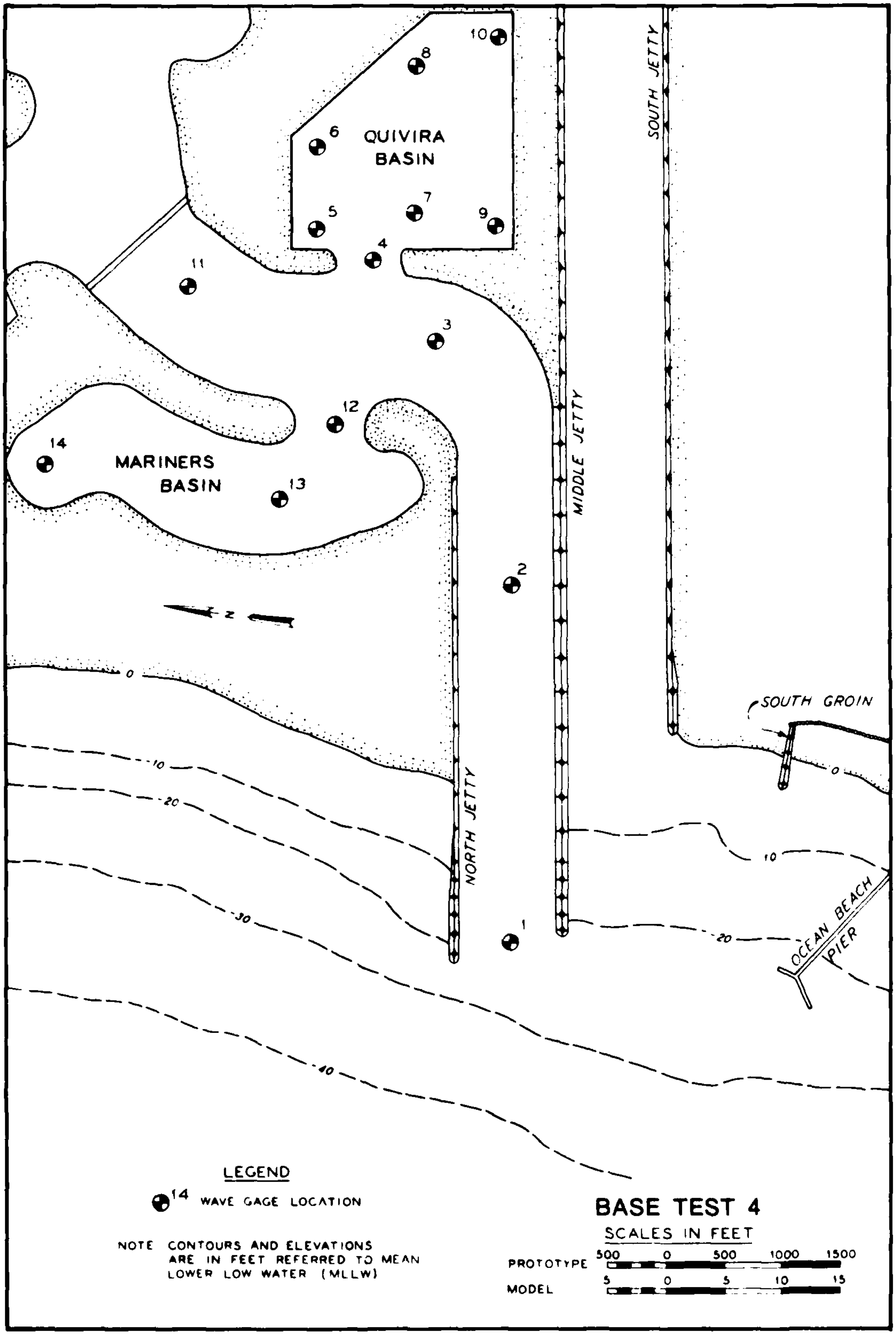
LEGEND
14 WAVE GAGE LOCATION
NOTE CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER (MLLW)
BASE TEST 3
SCALES IN FEET
PROTOTYPE 500 0 500 1000 1500
MODEL 5 0 5 10 15

PLATE 3


QUIVIRA
BASIN
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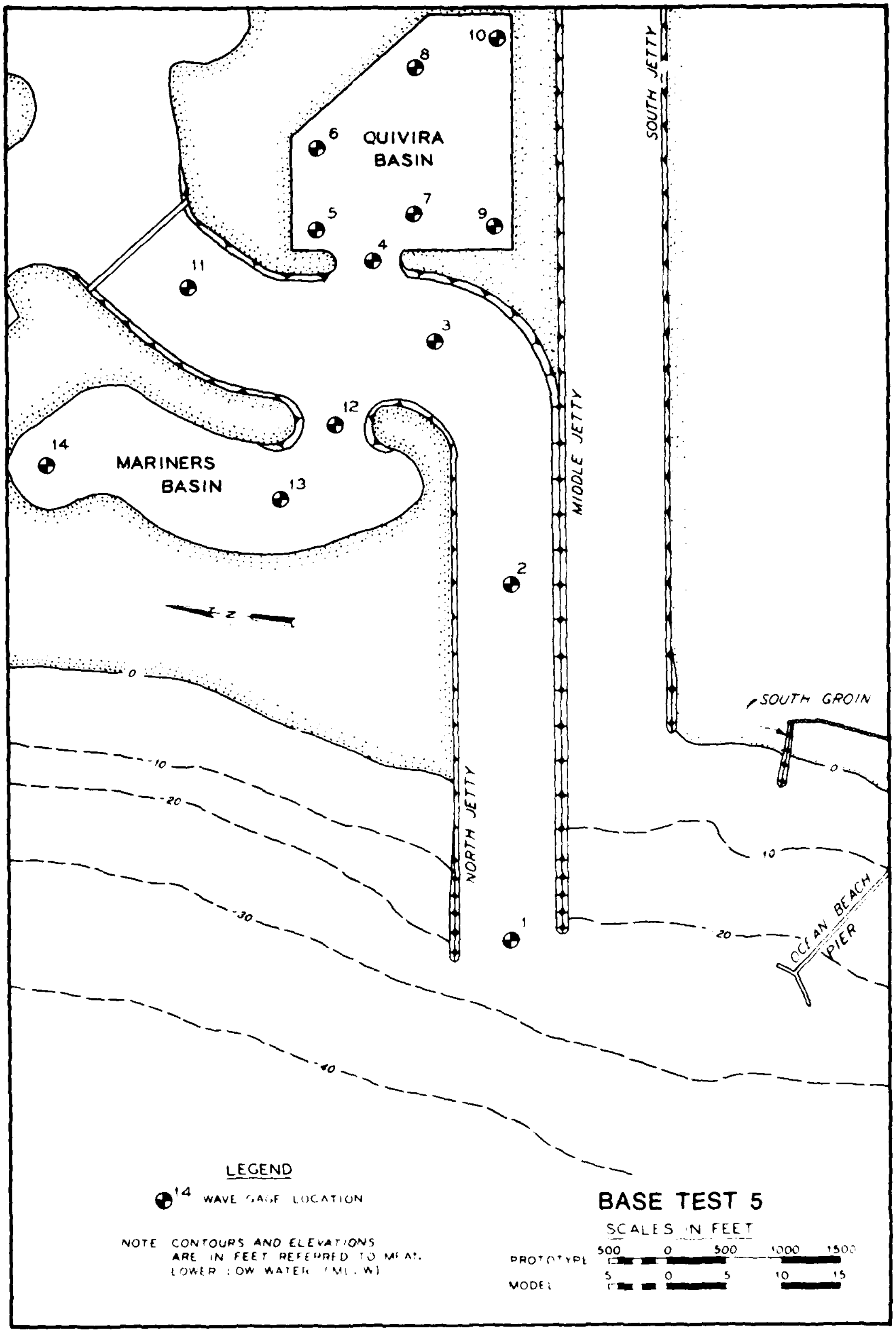
BASIN
SOUTH JETTY
MIDDLE JETTY
NORTH JETTY
SOUTH GROIN
OCEAN BEACH
PIER
LEGEND
14 WAVE GAGE LOCATION
NOTE CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER (MLLW)
BASE TEST 4
SCALES IN FEET
PROTOTYPE 500 0 500 1000 1500
MODEL 5 0 5 10 15

PLATE 4


QUIVIRA BASIN
MARINERS BASIN
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MIDDLE JETTY
NORTH JETTY
SOUTH GROIN
OCEAN BEACH PIER
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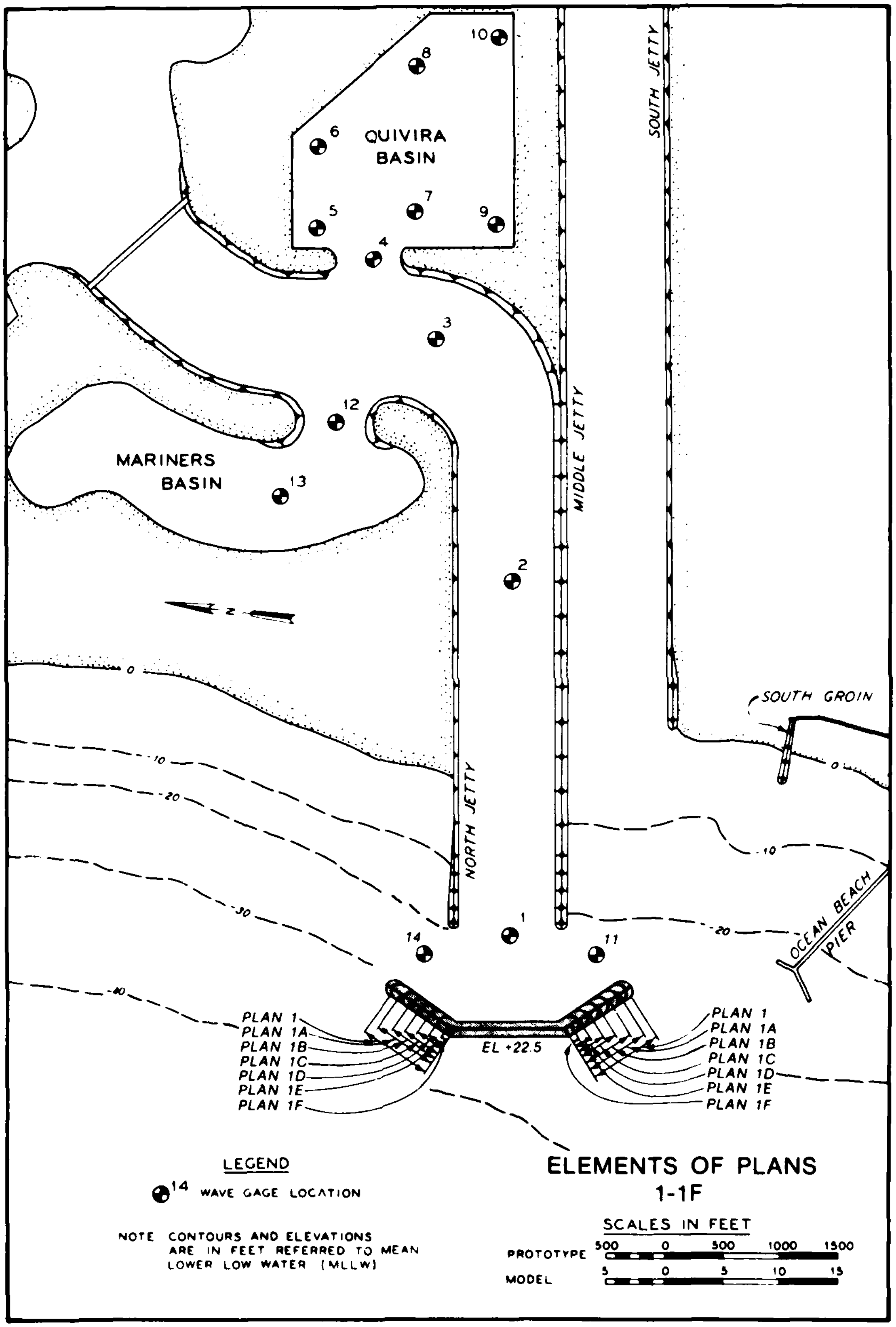
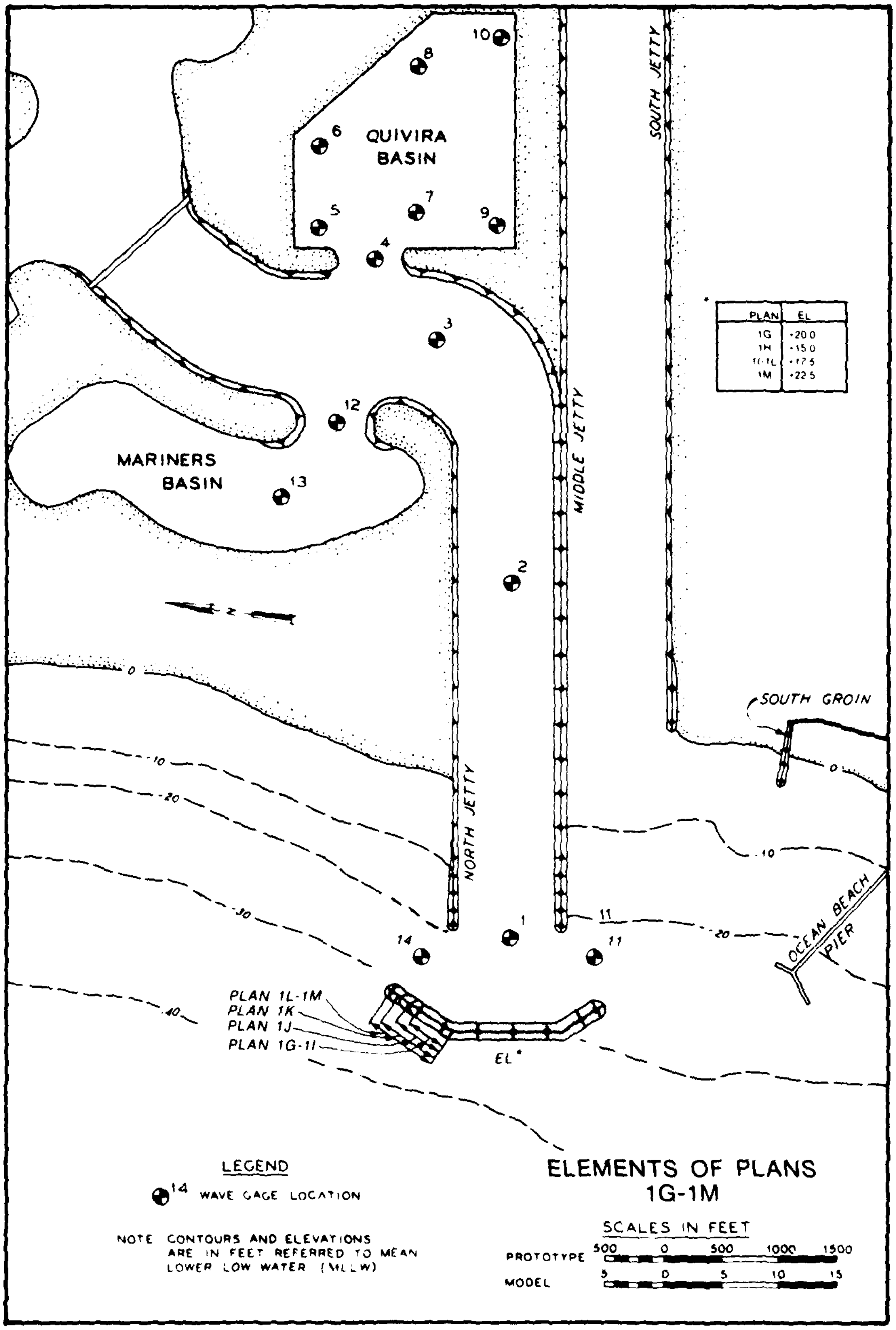
14
LEGEND
14 WAVE GAGE LOCATION
NOTE CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER (MLLW)
BASE TEST 5
SCALES IN FEET
PROTOTYPE 500 0 500 1000 1500
MODEL 5 0 5 10 15

PLATE 5



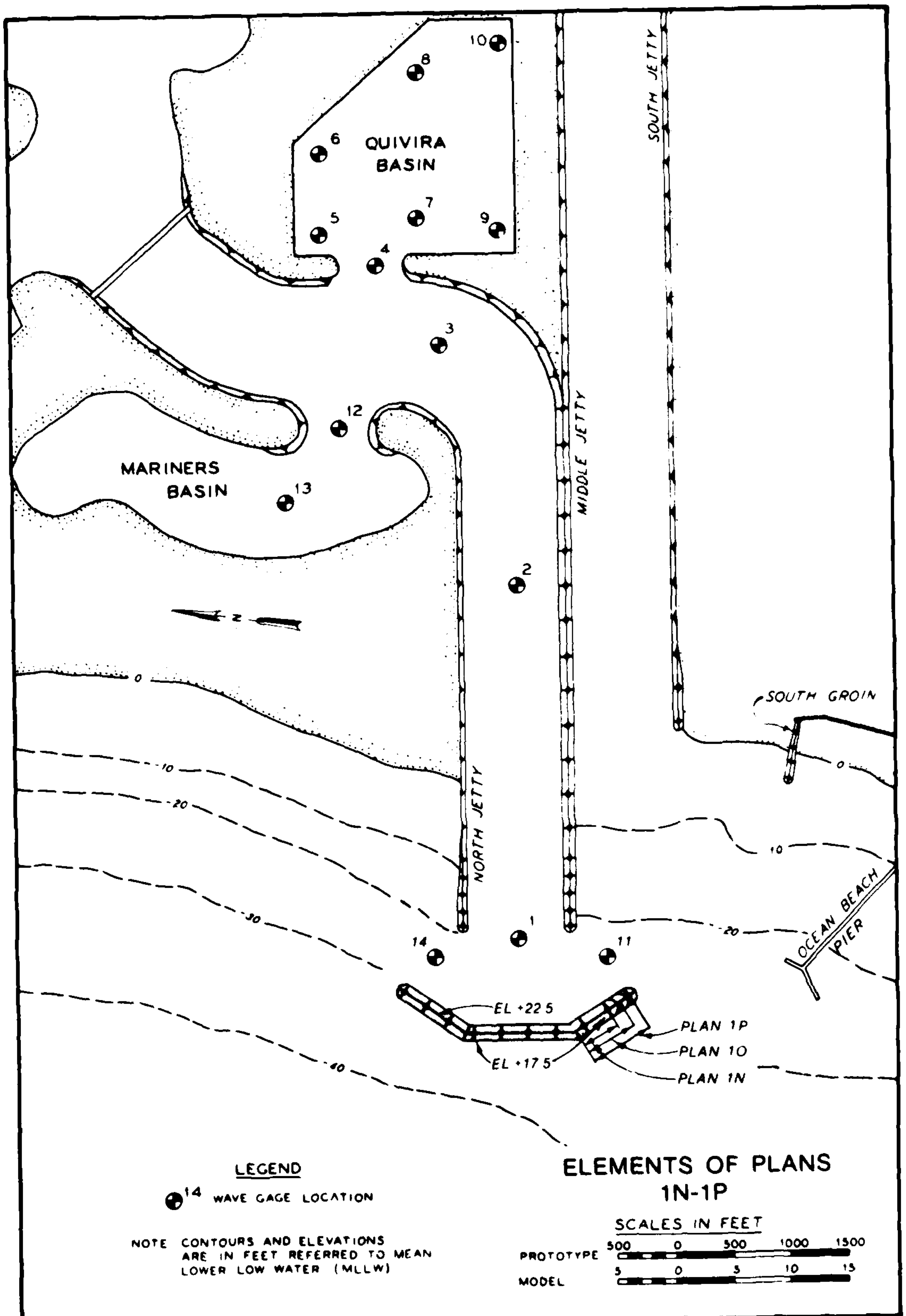
ELEMENTS OF PLANS
1-1F

PLATE 6



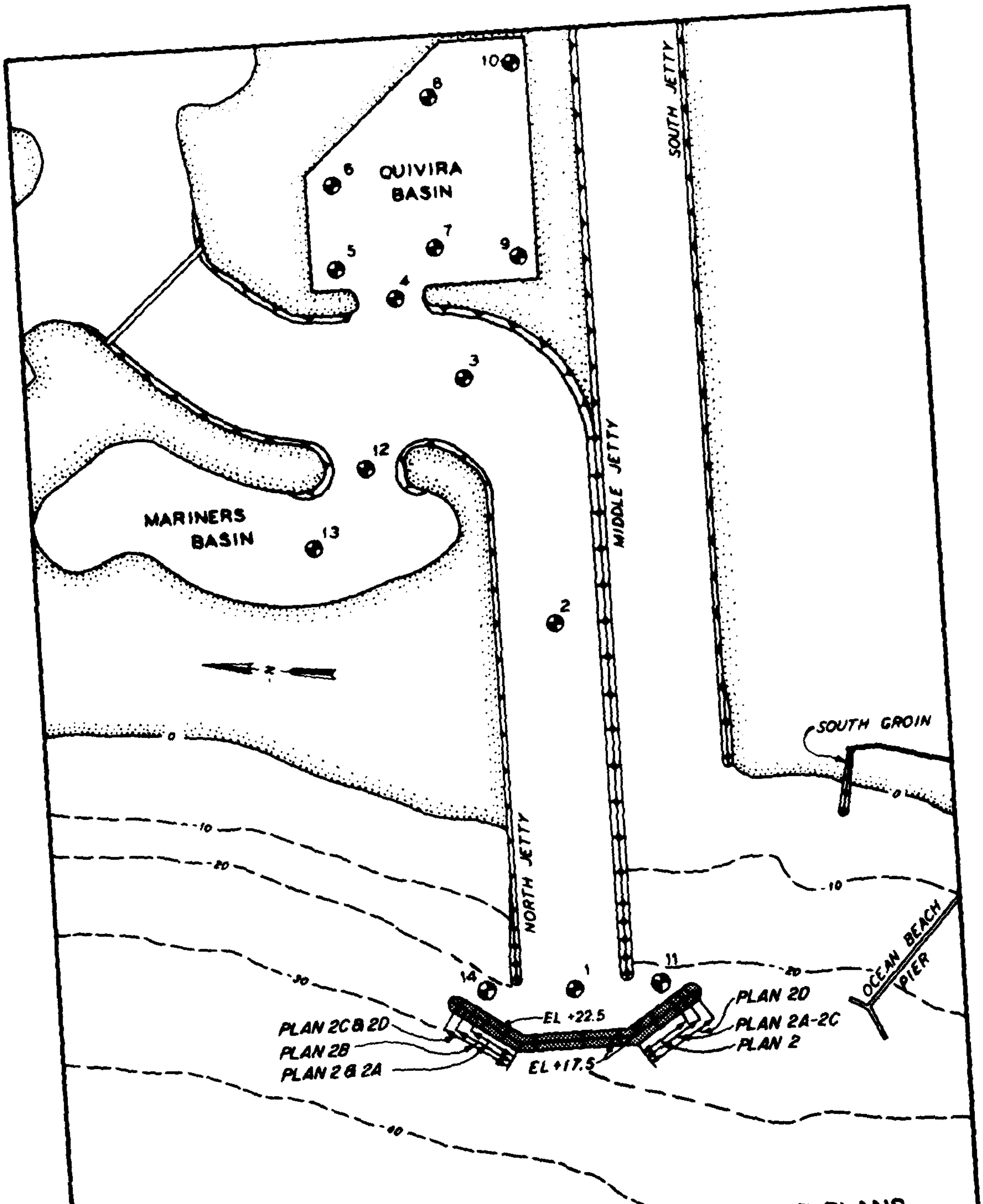
ELEMENTS OF PLANS 1G-1M

PLATE 7



QUIVIRA
BASIN
MARINERS
BASIN
SOUTH JETTY
MIDDLE JETTY
NORTH JETTY
SOUTH GROIN
OCEAN BEACH PIER
EL +22 5
EL +17 5
PLAN 1P
PLAN 1O
PLAN 1N
LEGEND
WAVE GAGE LOCATION
NOTE CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER (MLLW)
ELEMENTS OF PLANS
1N-1P
SCALES IN FEET
PROTOTYPE
MODEL

PLATE 8



ELEMENTS OF PLANS 2-2D

PLATE 9

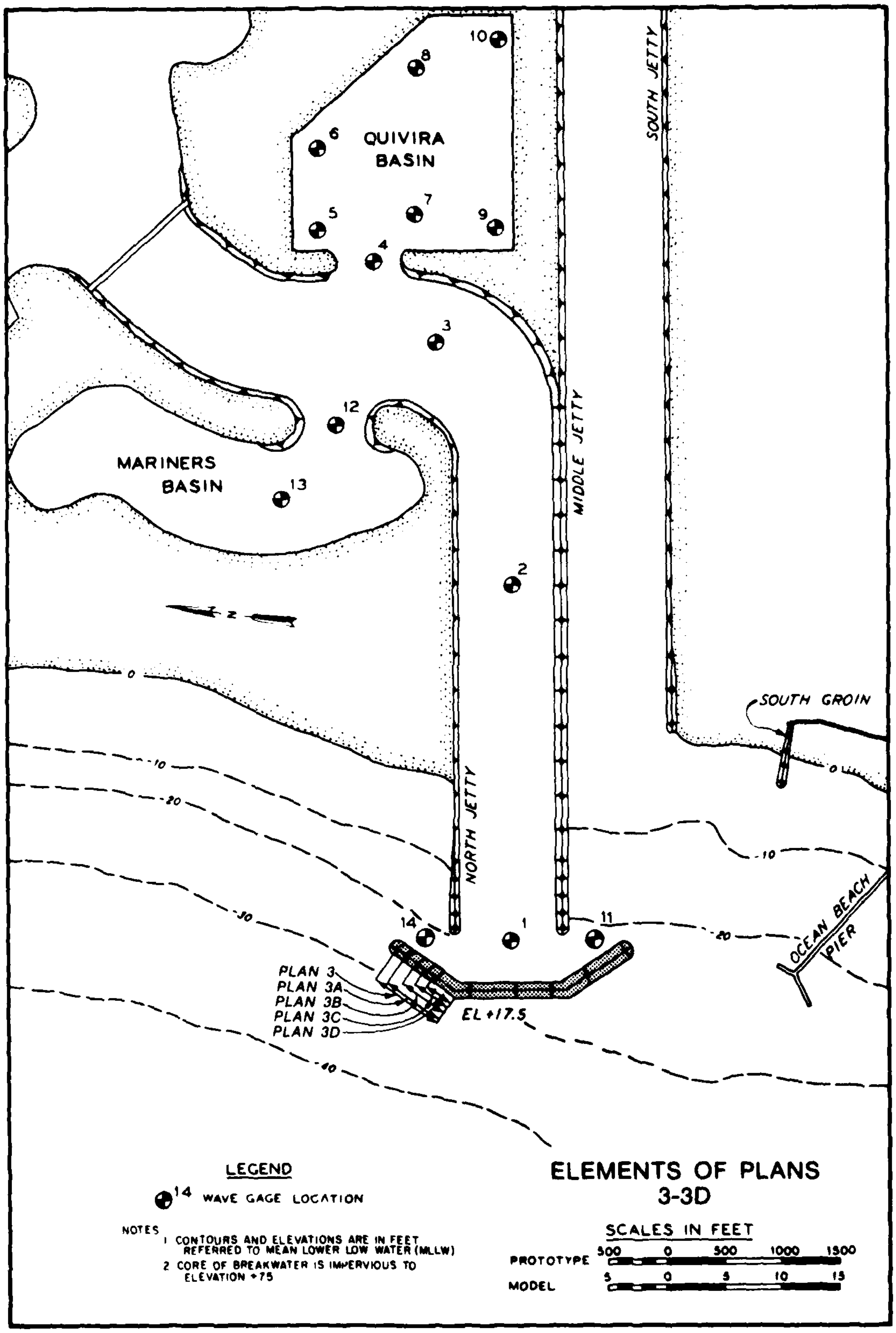
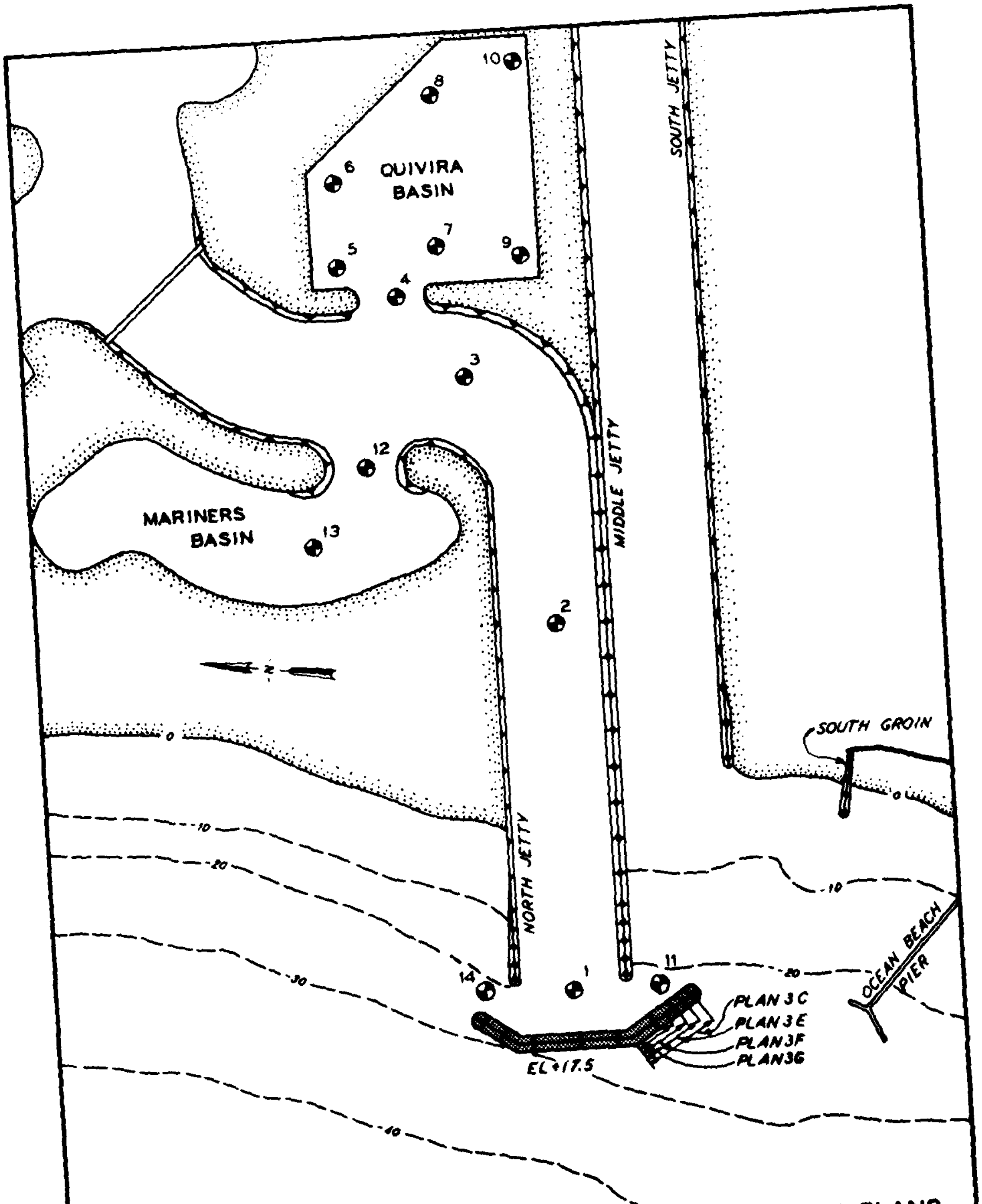

QUIVIRA BASIN
MARINERS BASIN
SOUTH JETTY
MIDDLE JETTY
NORTH JETTY
SOUTH GROIN
OCEAN BEACH PIER
PLAN 3
PLAN 3A
PLAN 3B
PLAN 3C
PLAN 3D
EL +17.5
LEGEND
14 WAVE GAGE LOCATION
NOTES
1 CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER (MLLW)
2 CORE OF BREAKWATER IS IMPERVIOUS TO ELEVATION +7.5
ELEMENTS OF PLANS 3-3D
SCALES IN FEET
PROTOTYPE 500 0 500 1000 1500
MODEL 5 0 5 10 15

PLATE 10



ELEMENTS OF PLANS 3C, 3E-3G

LEGEND

14 WAVE GAGE LOCATION

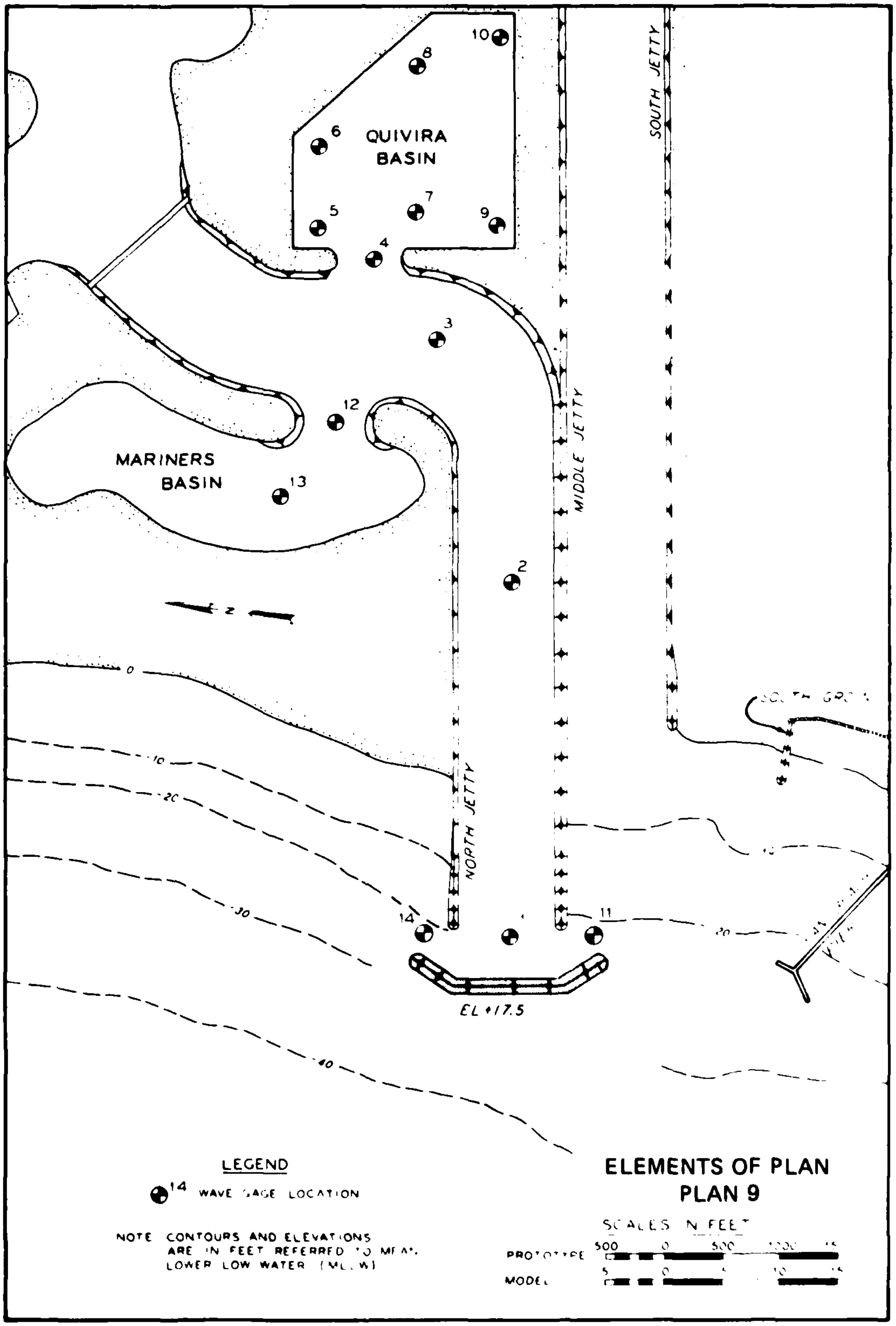
NOTES:
1. CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER (MLLW)
2. CORE OF BREAKWATER IS IMPERVIOUS TO ELEVATION +7.5

SCALES IN FEET

PROTOTYPE 500 0 500 1000 1500

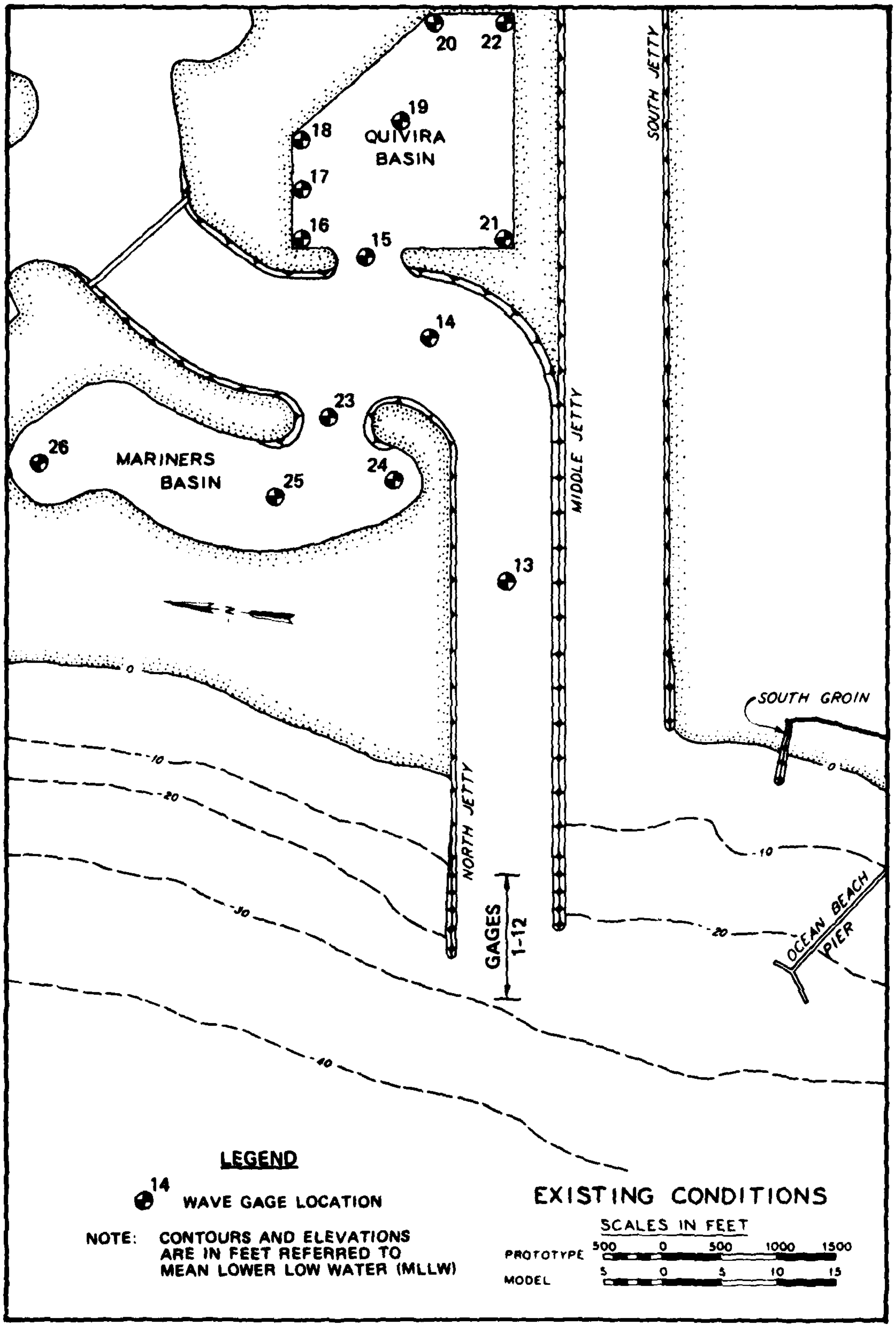
MODEL 5 0 5 10 15

PLATE 11



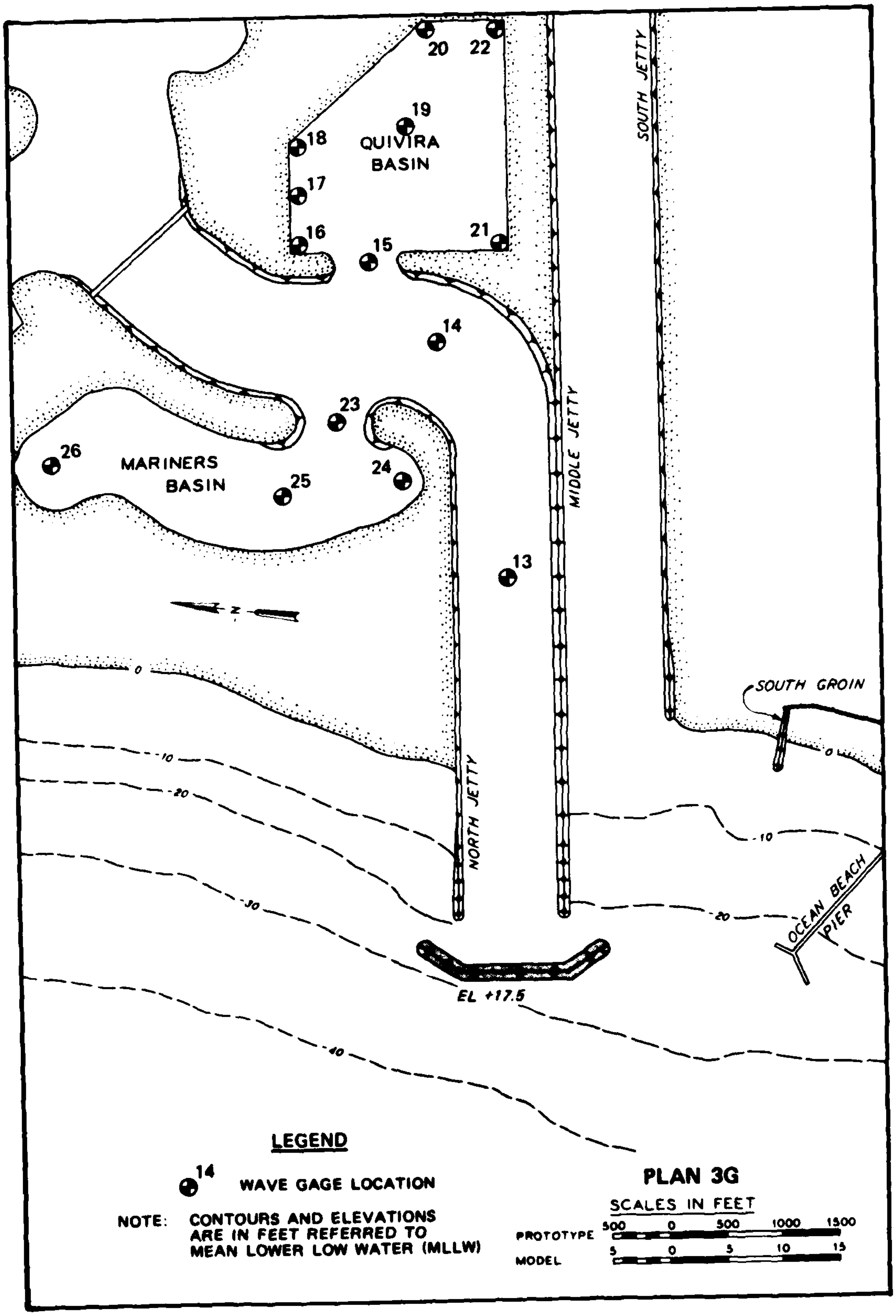
QUIVIRA
BASIN
MARINERS
BASIN
SOUTH JETTY
MIDDLE JETTY
NORTH JETTY
EL +17.5
10
8
6
5
7
9
4
3
12
13
2
14
11
0
10
20
30
40
LEGEND
14 WAVE GAGE LOCATION
NOTE CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER (MLLW)
ELEMENTS OF PLAN
PLAN 9
PROTOTYPE
MODEL

PLATE 12

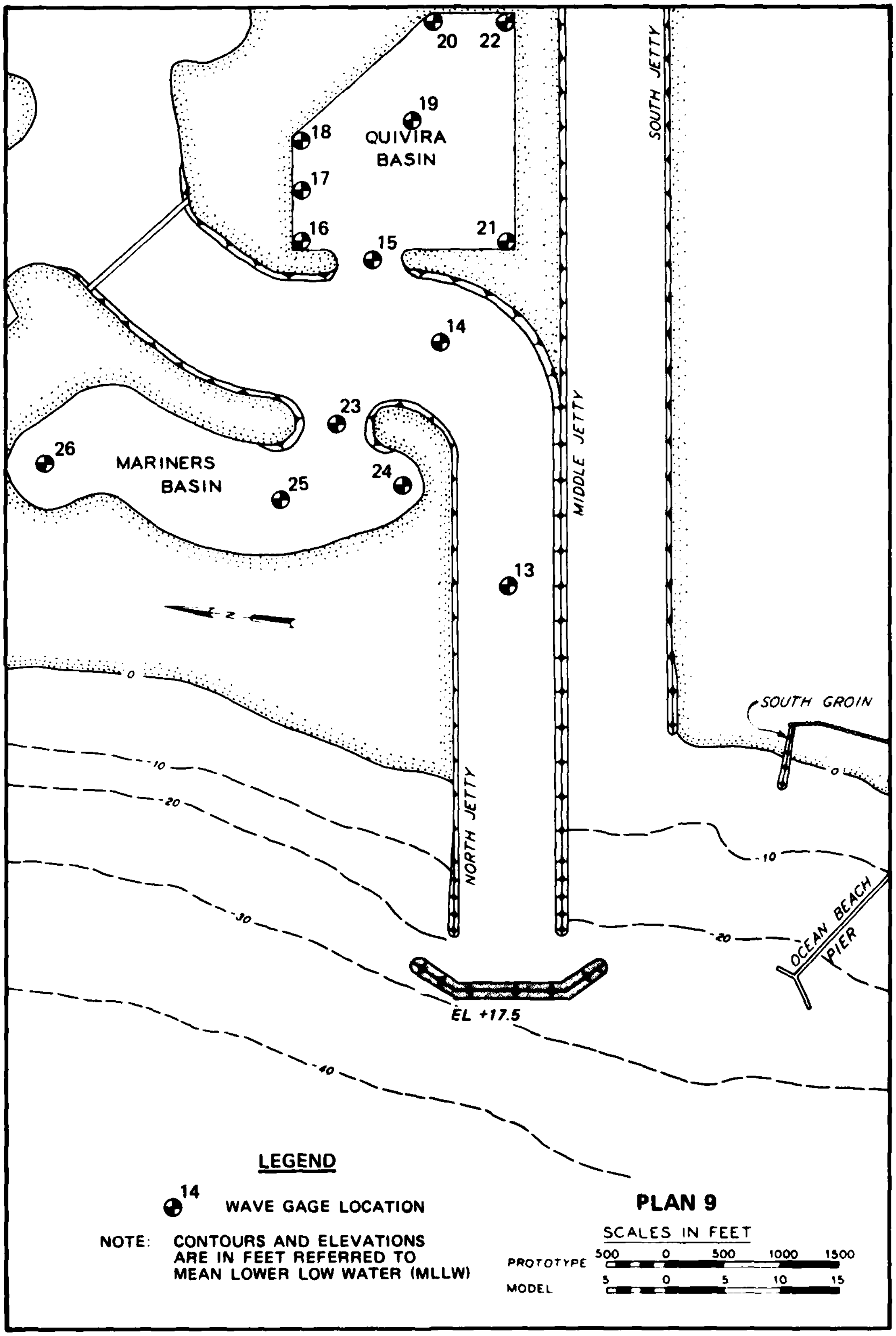


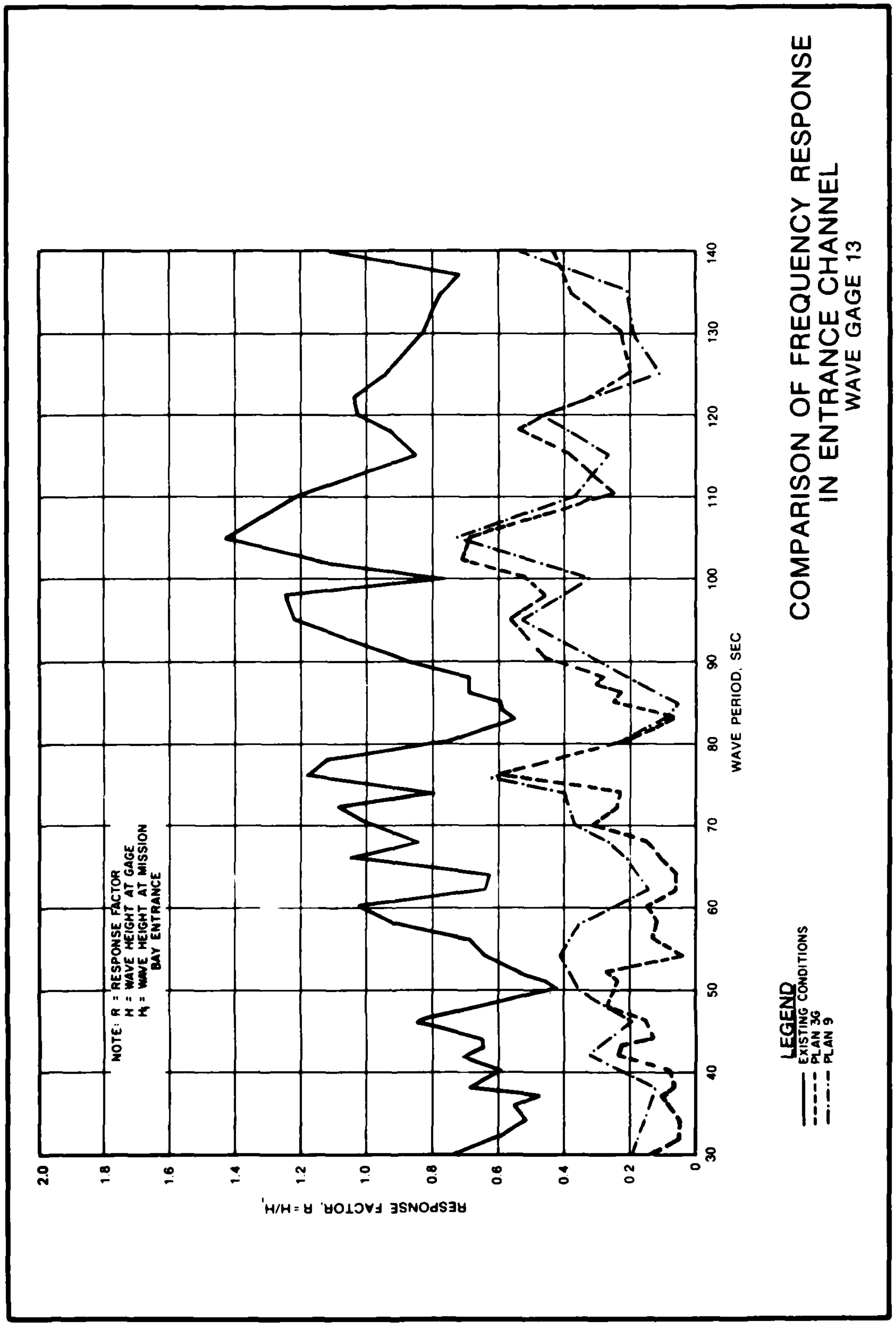
EXISTING CONDITIONS

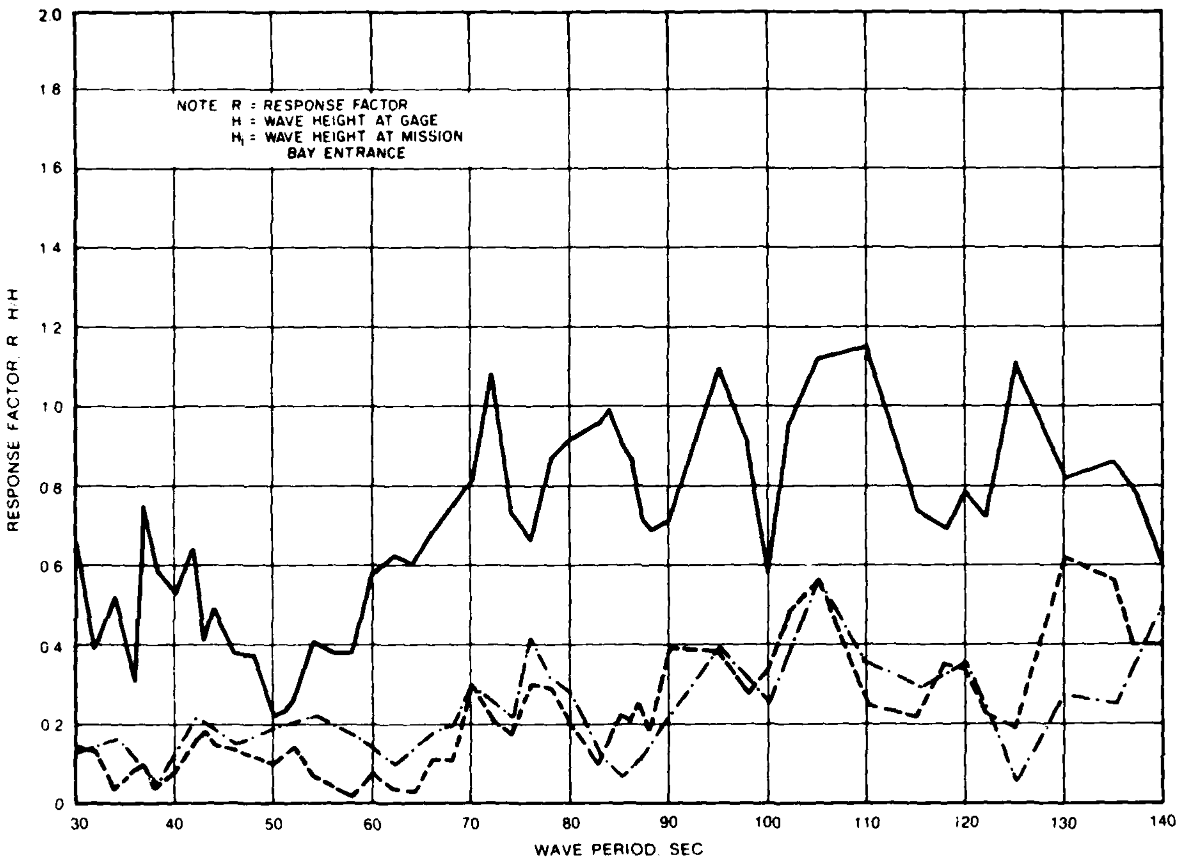
PLATE 13



20
22
19
18
QUIVIRA
BASIN
17
16
15
21
SOUTH JETTY
14
23
26
MARINERS
BASIN
25
24
MIDDLE JETTY
13
N
0
SOUTH GROIN
10
20
NORTH JETTY
10
30
20
OCEAN BEACH
PIER
EL +17.5
40
LEGEND
14
WAVE GAGE LOCATION
NOTE: CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER (MLLW)
PLAN 3G
SCALES IN FEET
PROTOTYPE 500 0 500 1000 1500
MODEL 5 0 5 10 15


QUIVIRA BASIN
MARINERS BASIN
SOUTH JETTY
MIDDLE JETTY
NORTH JETTY
SOUTH GROIN
OCEAN BEACH PIER
EL +17.5
13
14
15
16
17
18
19
20
21
22
23
24
25
26
0
10
20
30
40
LEGEND
14 WAVE GAGE LOCATION
NOTE: CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER (MLLW)
PLAN 9
SCALES IN FEET
PROTOTYPE 500 0 500 1000 1500
MODEL 5 0 5 10 15


COMPARISON OF FREQUENCY RESPONSE
IN ENTRANCE CHANNEL
WAVE GAGE 13
NOTE: R = RESPONSE FACTOR
H = WAVE HEIGHT AT GAGE
H_I = WAVE HEIGHT AT MISSION
BAY ENTRANCE
LEGEND
EXISTING CONDITIONS
PLAN 36
PLAN 9
RESPONSE FACTOR, R = H/H_I
WAVE PERIOD, SEC
0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
30
40
50
60
70
80
90
100
110
120
130
140



LEGEND

EXISTING CONDITIONS
PLAN 3G
PLAN 9

COMPARISON OF FREQUENCY RESPONSE IN ENTRANCE CHANNEL
WAVE GAGE 14

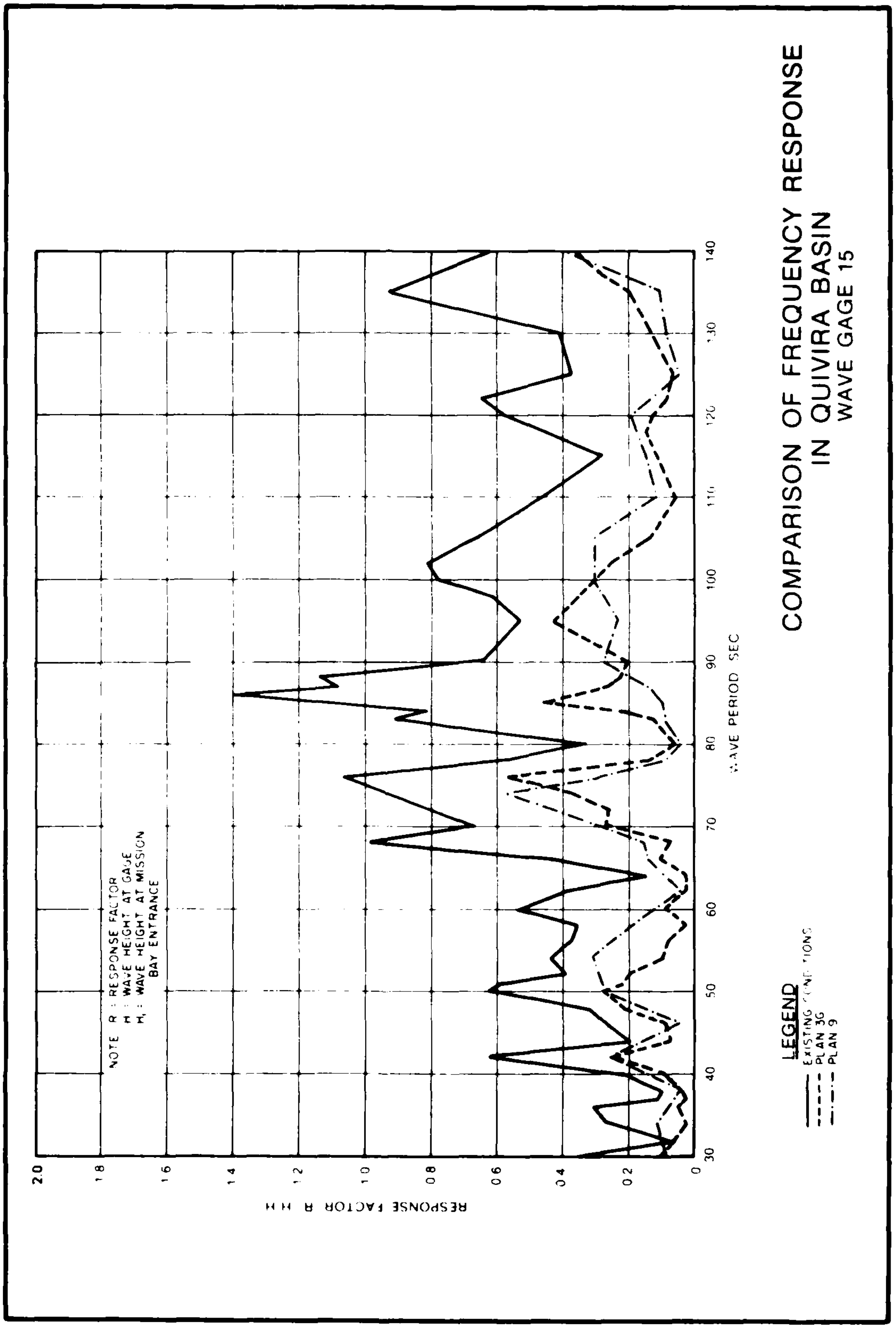
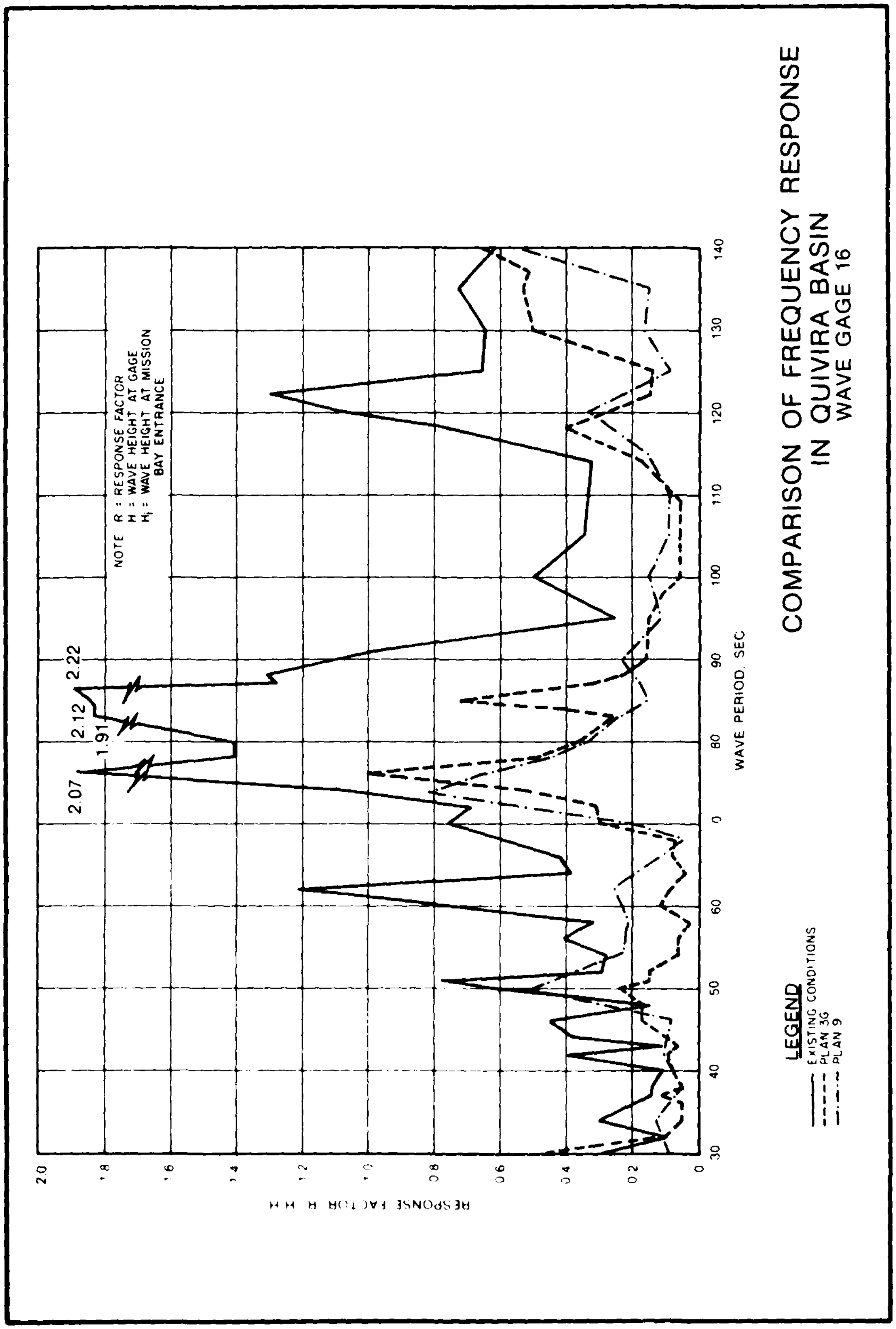

COMPARISON OF FREQUENCY RESPONSE
IN QUIVIRA BASIN
WAVE GAGE 15
NOTE R = RESPONSE FACTOR
H = WAVE HEIGHT AT GAGE
H1 = WAVE HEIGHT AT MISSION
BAY ENTRANCE
RESPONSE FACTOR R H H
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0
30
40
50
60
70
80
90
100
110
120
130
140
WAVE PERIOD SEC
LEGEND
EXISTING CONDITIONS
PLAN 3G
PLAN 9

PLATE 18



COMPARISON OF FREQUENCY RESPONSE
IN QUIVIRA BASIN
WAVE GAGE 16
NOTE R = RESPONSE FACTOR
H = WAVE HEIGHT AT GAGE
Hi = WAVE HEIGHT AT MISSION BAY ENTRANCE
LEGEND
EXISTING CONDITIONS
PLAN 3G
PLAN 9
RESPONSE FACTOR R, H/Hi
WAVE PERIOD, SEC
2.07
1.91
2.12
2.22

PLATE 19

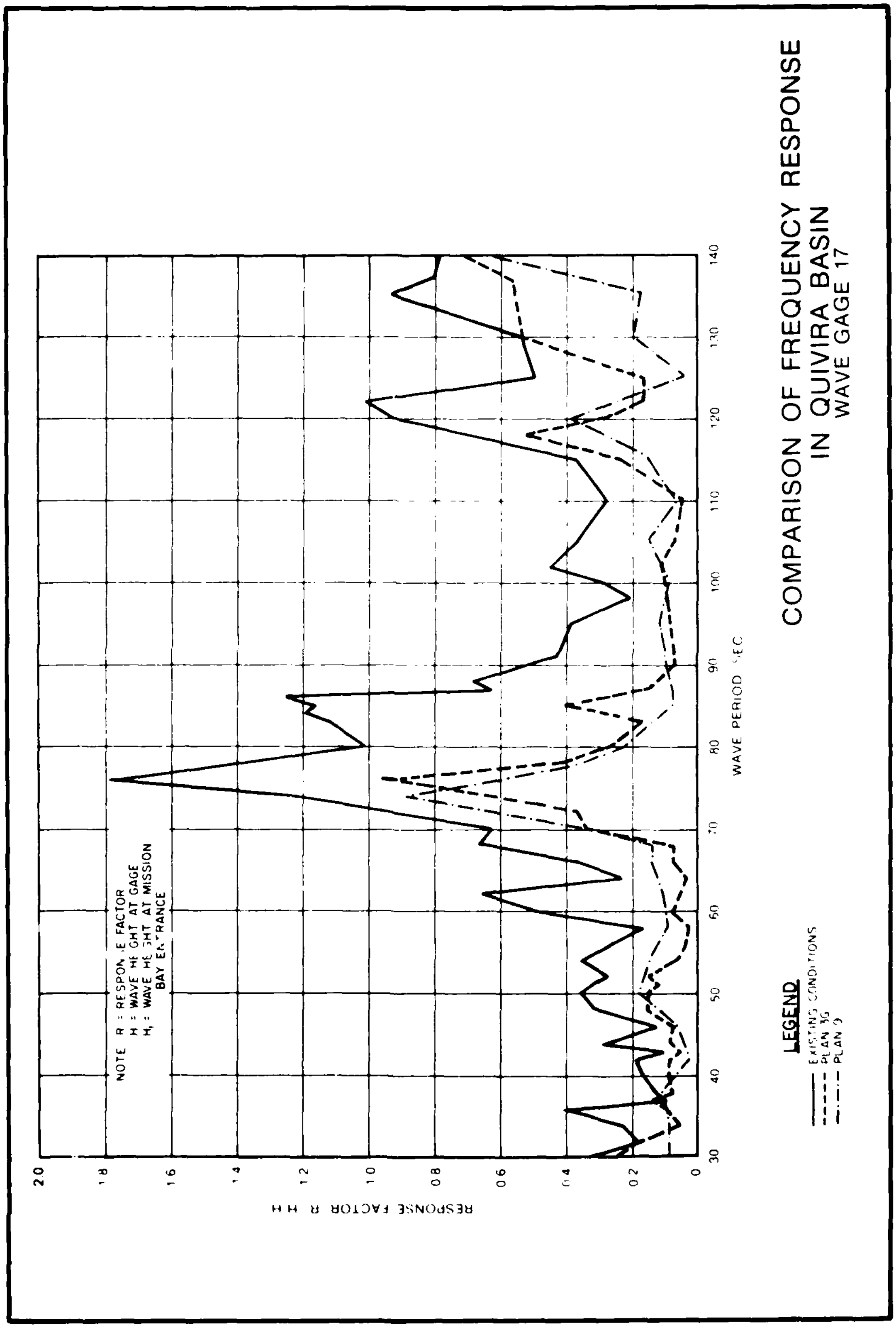
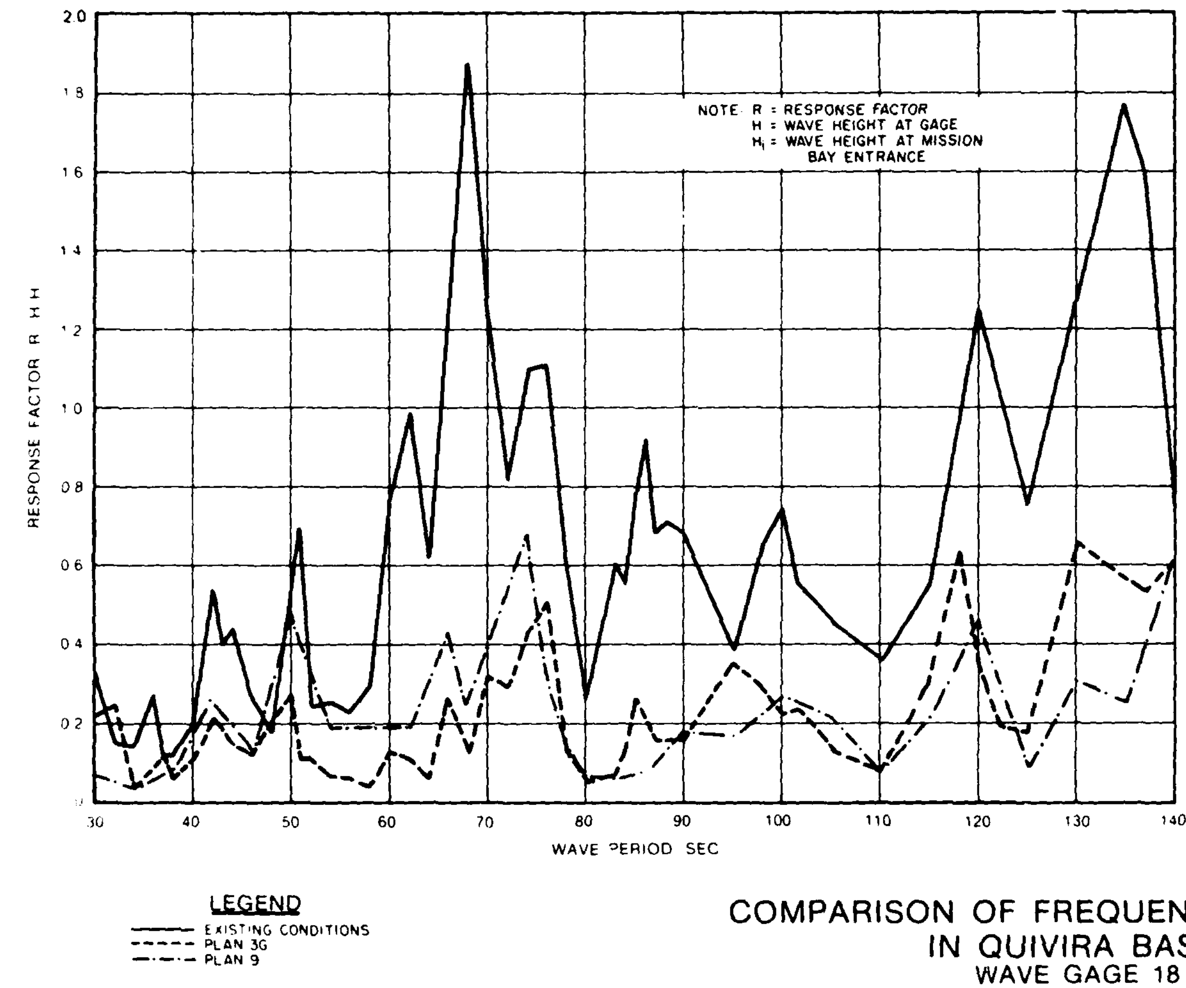
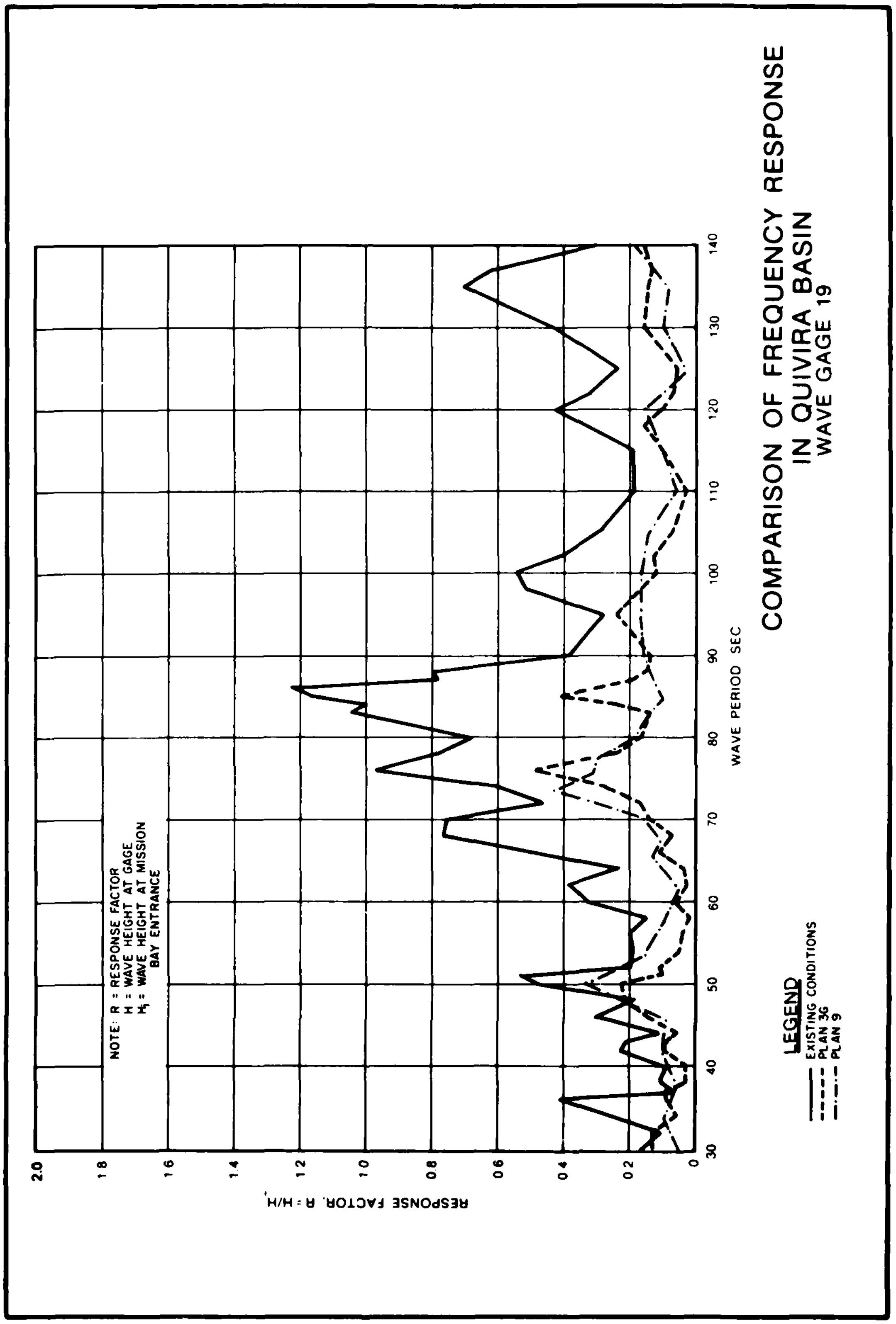

COMPARISON OF FREQUENCY RESPONSE
IN QUIVIRA BASIN
WAVE GAGE 17
NOTE R = RESPONSE FACTOR
H = WAVE HEIGHT AT GAGE
H₁ = WAVE HEIGHT AT MISSION
BAY ENTRANCE
LEGEND
EXISTING CONDITIONS
PLAN 3G
PLAN 9
WAVE PERIOD, SEC
RESPONSE FACTOR R = H/H₁
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0
30
40
50
60
70
80
90
100
110
120
130
140

PLATE 20

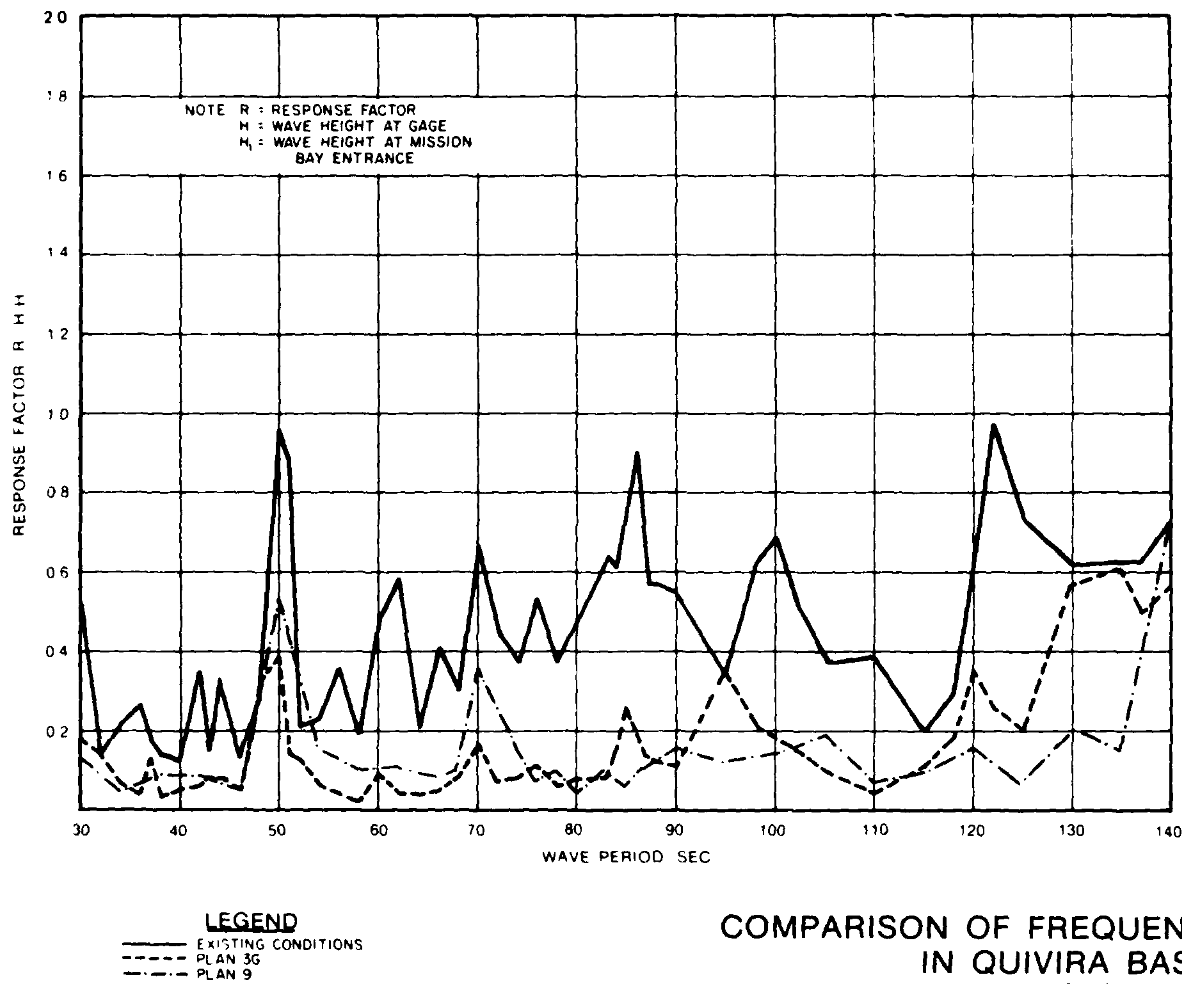


COMPARISON OF FREQUENCY RESPONSE
IN QUIVIRA BASIN
WAVE GAGE 18

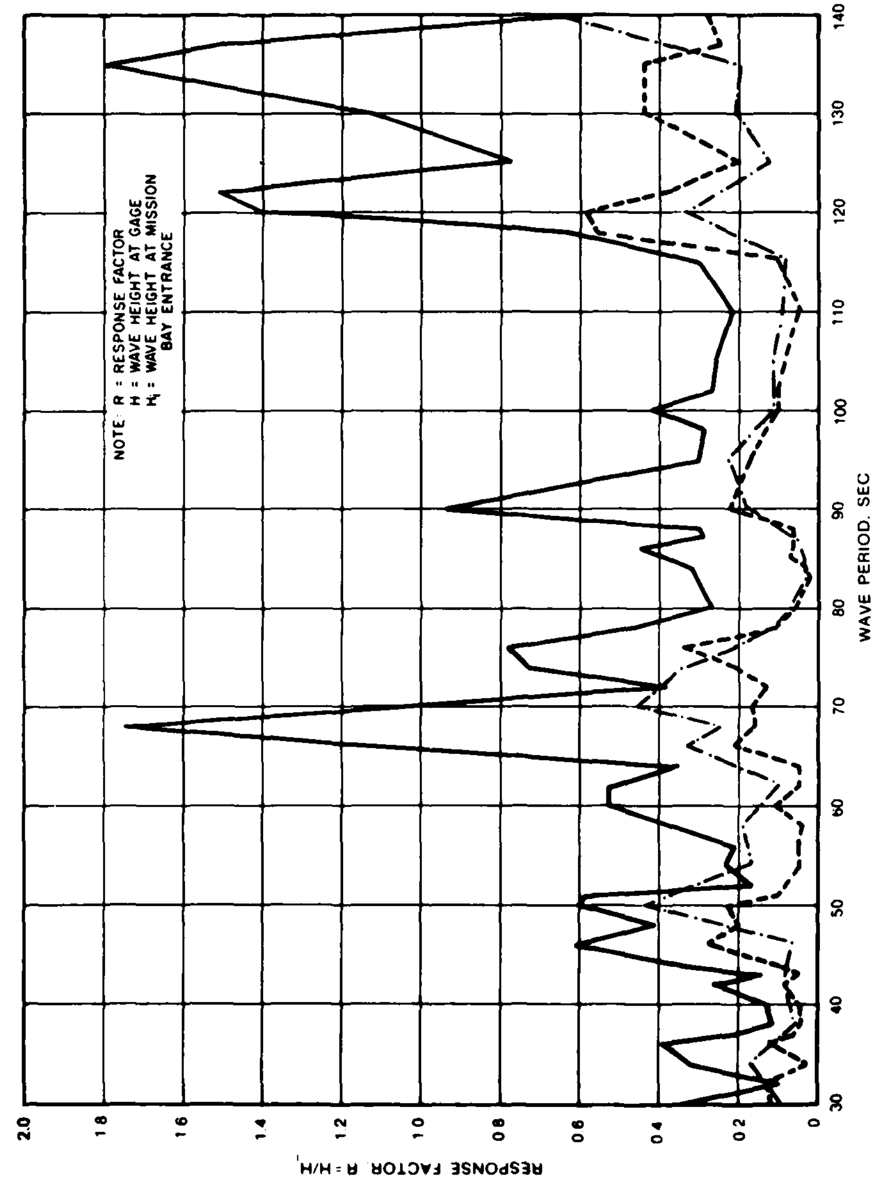


COMPARISON OF FREQUENCY RESPONSE
IN QUIVIRA BASIN
WAVE GAGE 19
NOTE: R = RESPONSE FACTOR
H = WAVE HEIGHT AT GAGE
H_i = WAVE HEIGHT AT MISSION
BAY ENTRANCE
RESPONSE FACTOR, R = H/H_i
WAVE PERIOD, SEC
0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
30
40
50
60
70
80
90
100
110
120
130
140
LEGEND
EXISTING CONDITIONS
PLAN 36
PLAN 9

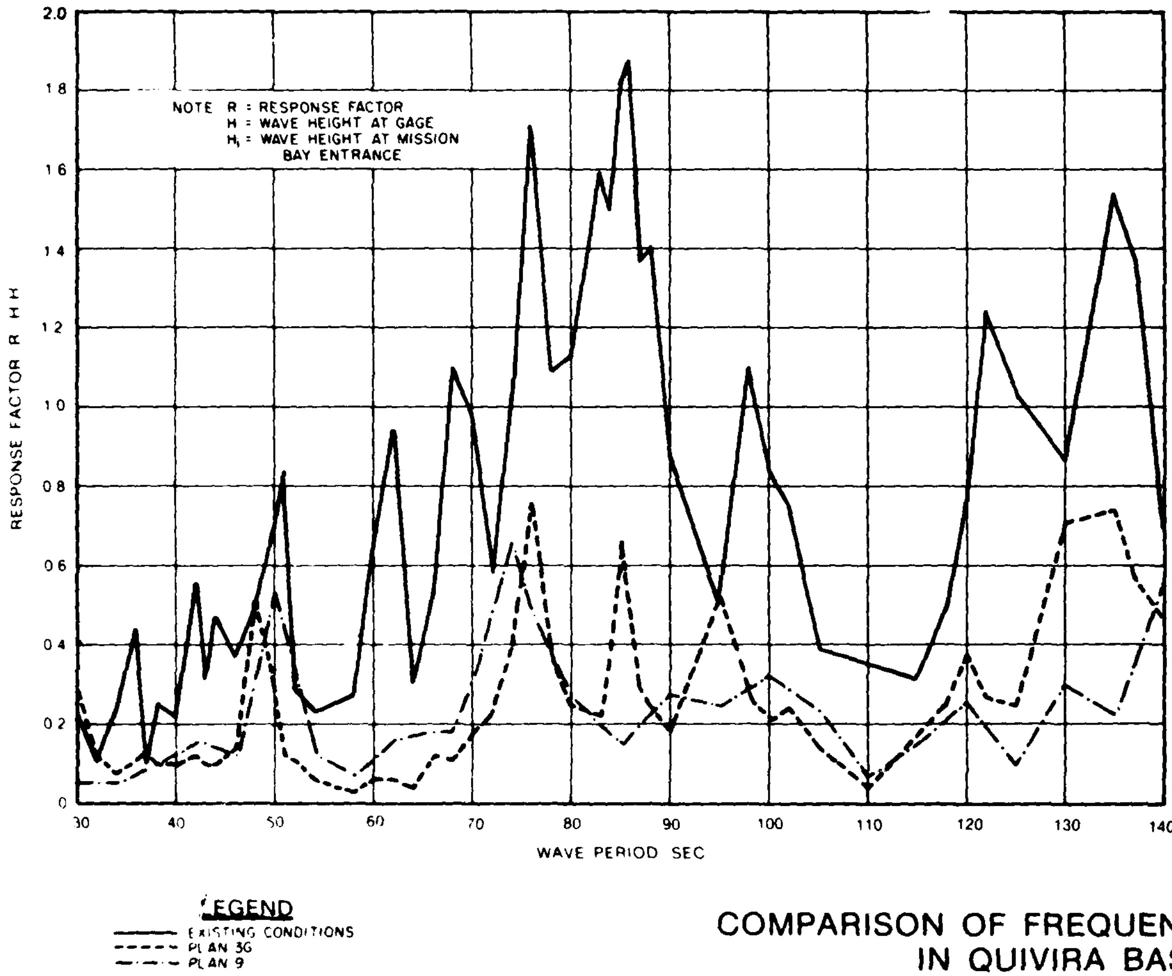
PLATE 22



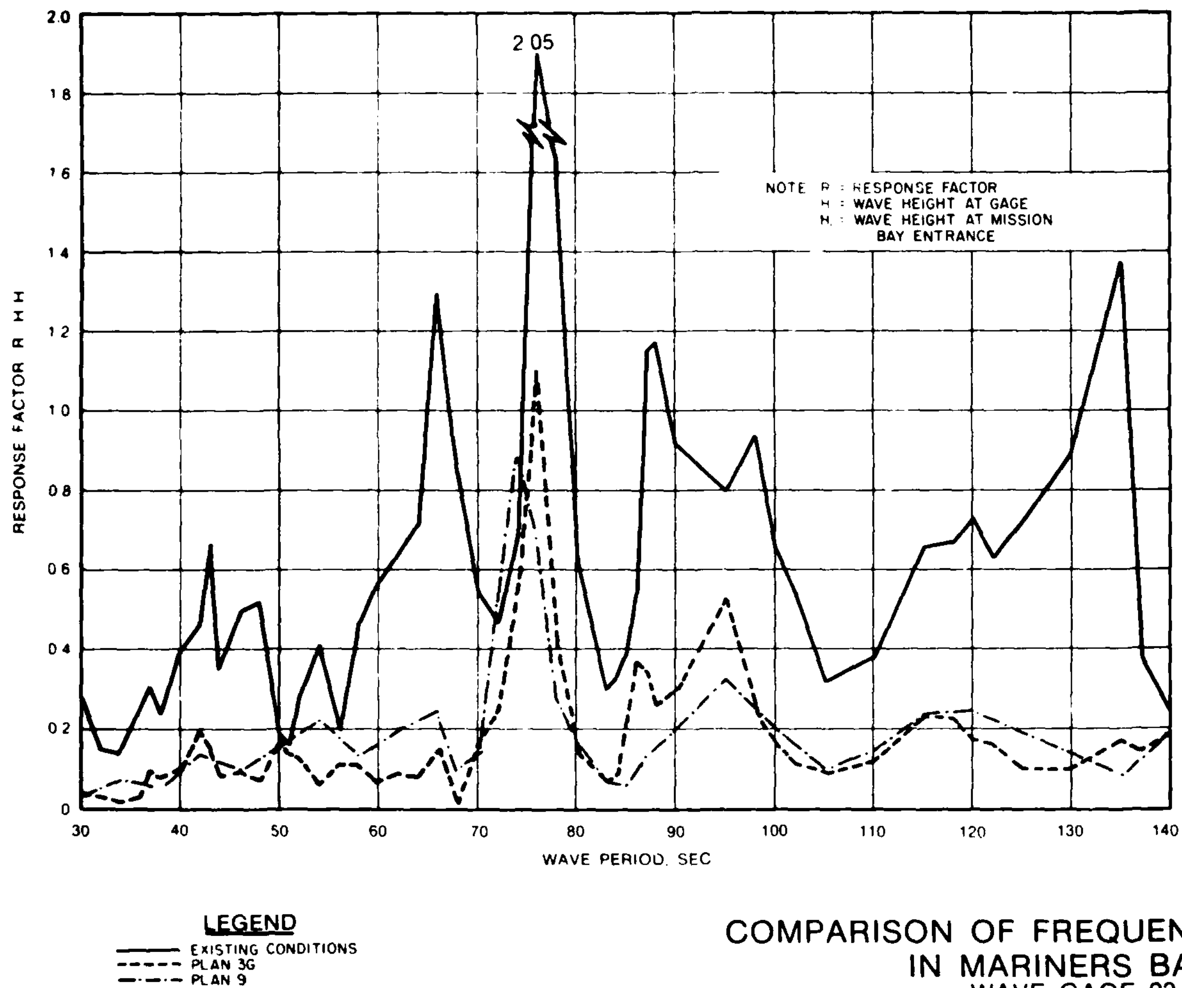
COMPARISON OF FREQUENCY RESPONSE
IN QUIVIRA BASIN
WAVE GAGE 20



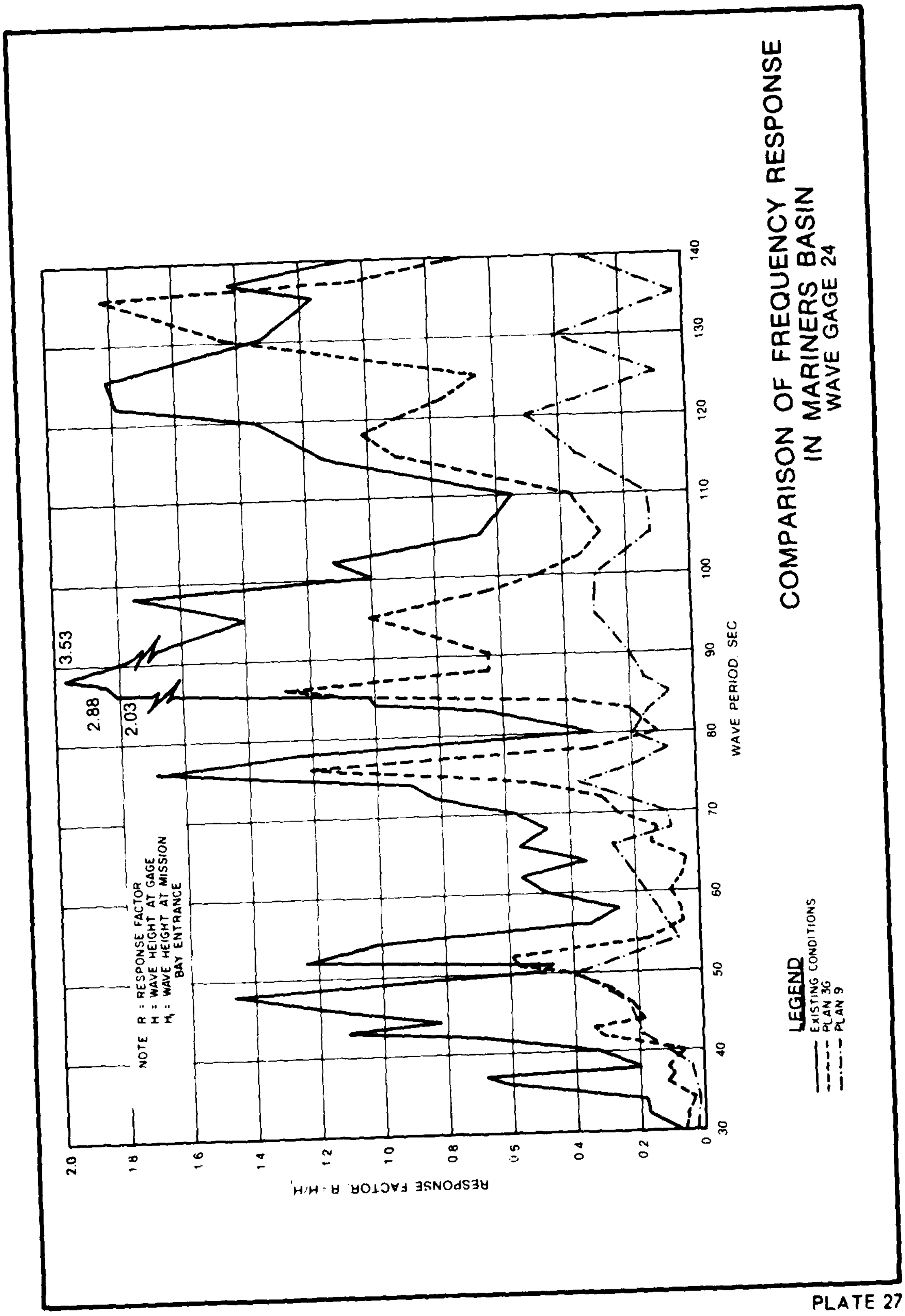
COMPARISON OF FREQUENCY RESPONSE
IN QUIVIRA BASIN
WAVE GAGE 21

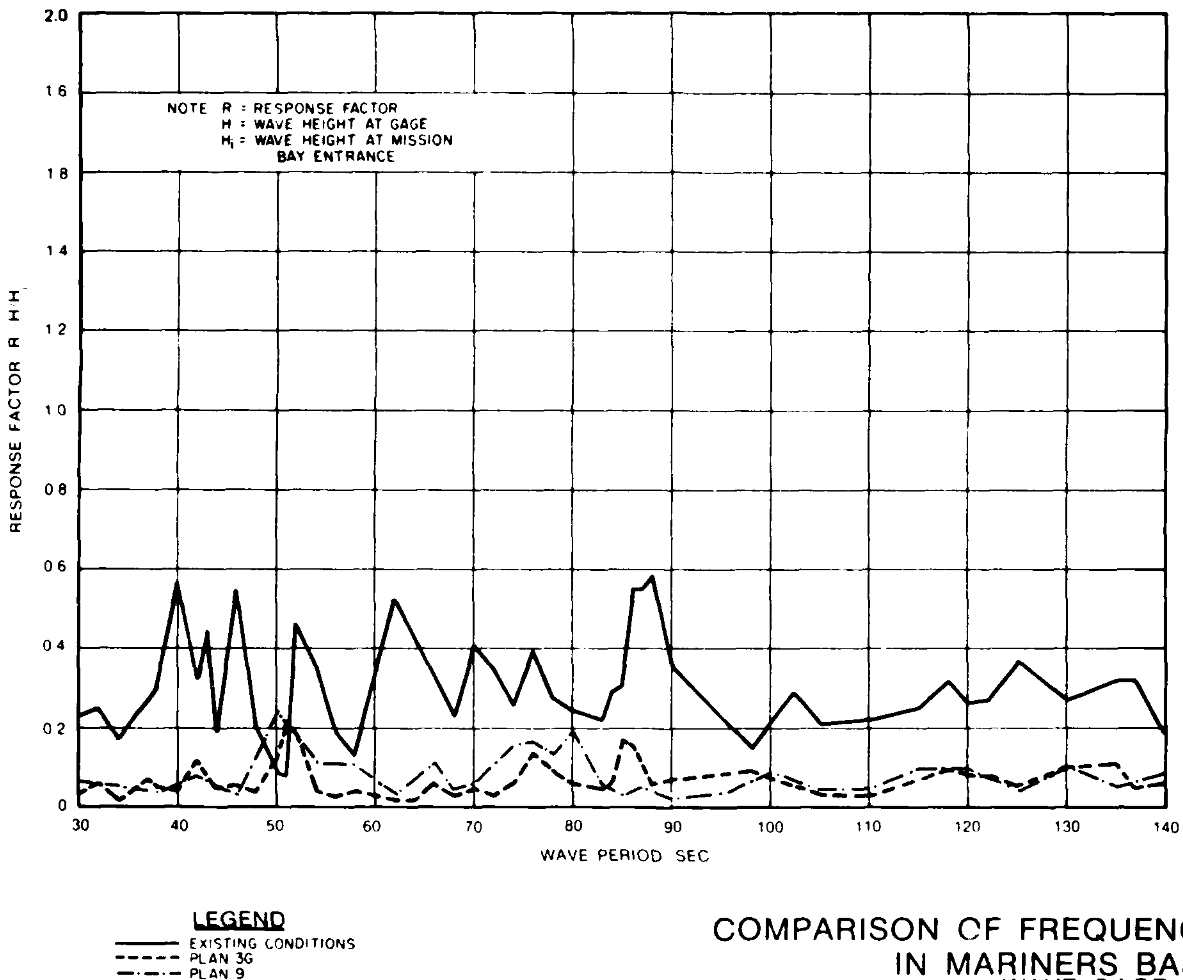


COMPARISON OF FREQUENCY RESPONSE
IN QUIVIRA BASIN
WAVE GAGE 22

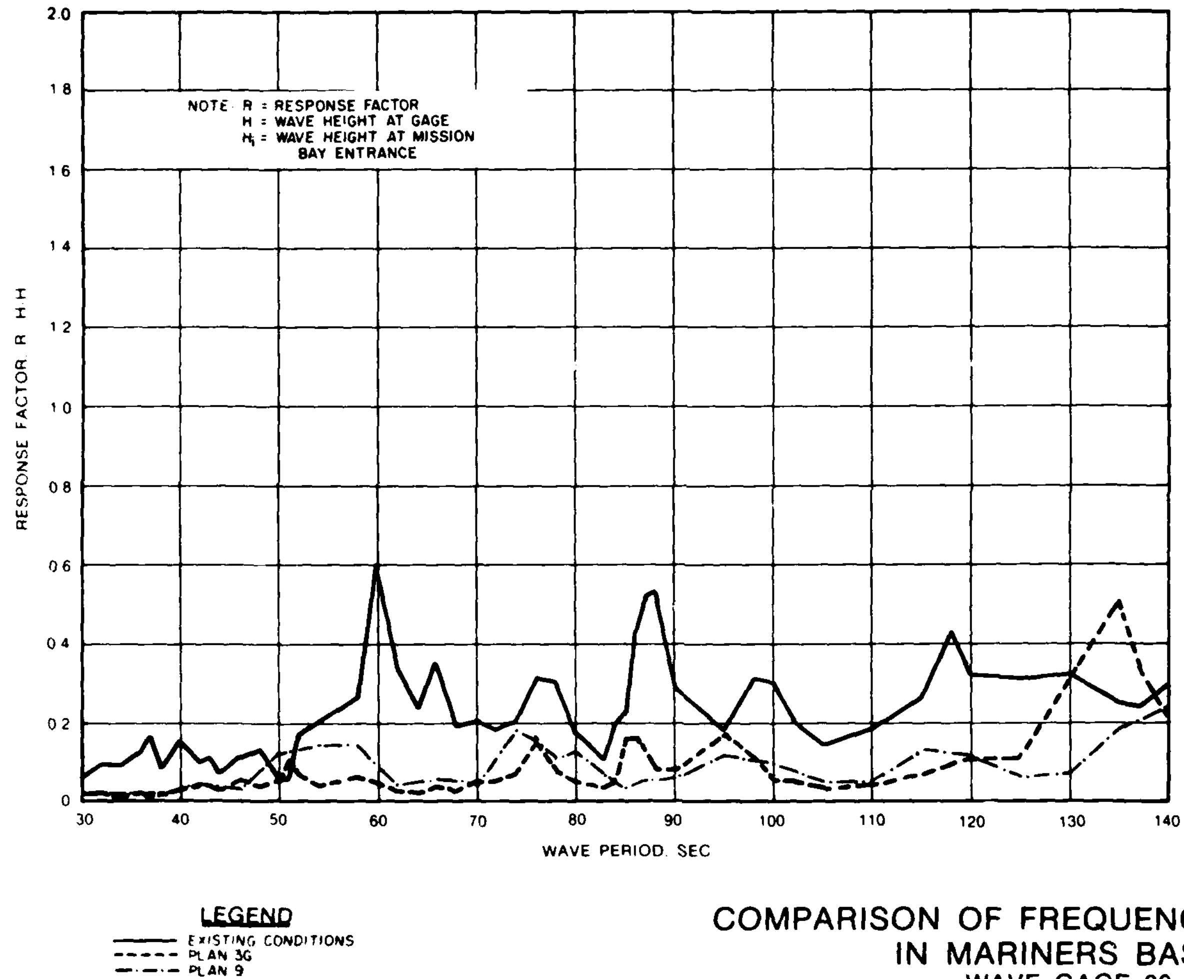


COMPARISON OF FREQUENCY RESPONSE
IN MARINERS BASIN
WAVE GAGE 23

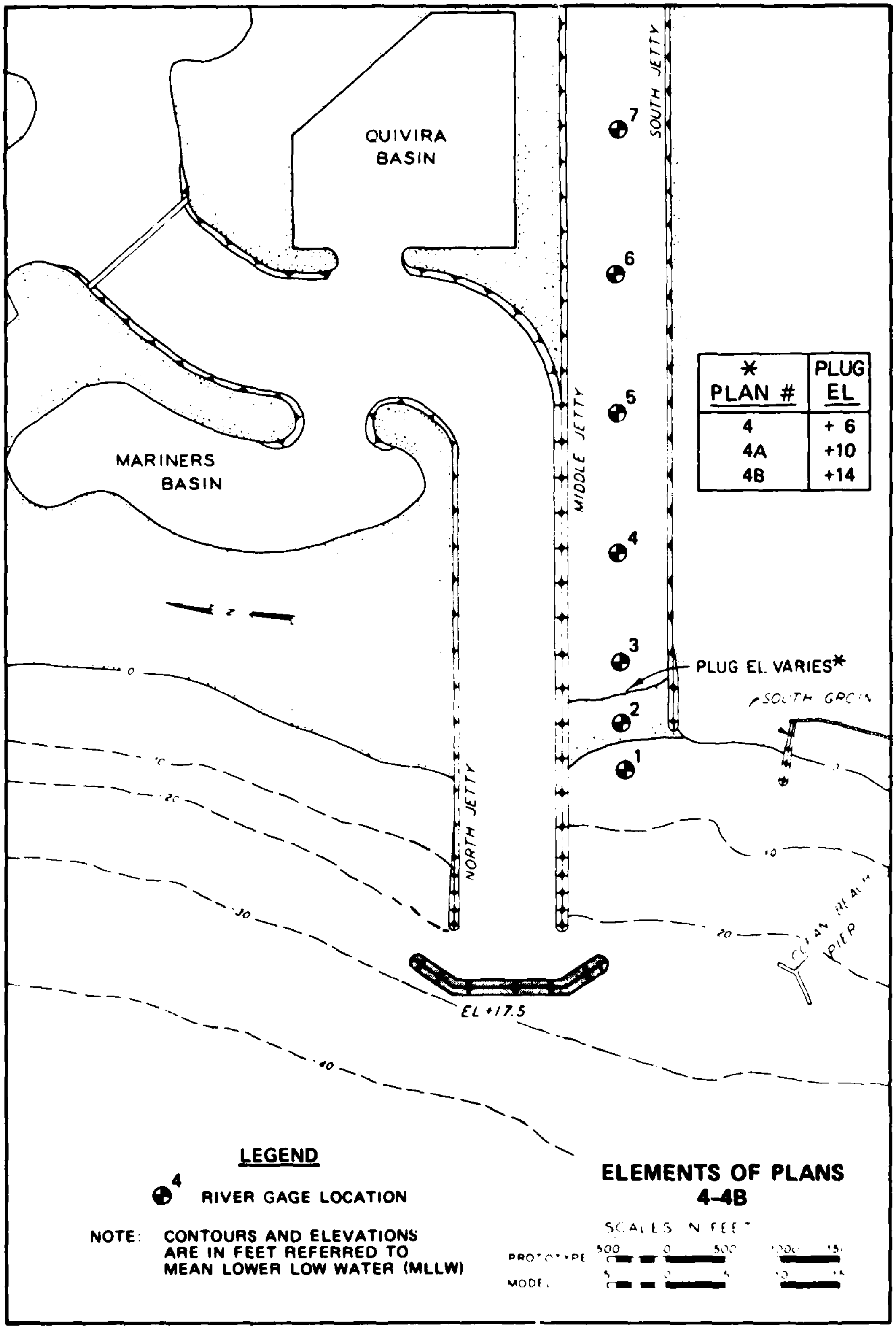

COMPARISON OF FREQUENCY RESPONSE
IN MARINERS BASIN
WAVE GAGE 24
NOTE R : RESPONSE FACTOR
H : WAVE HEIGHT AT GAGE
H, : WAVE HEIGHT AT MISSION BAY ENTRANCE
LEGEND
EXISTING CONDITIONS
PLAN 3G
PLAN 9
RESPONSE FACTOR, R=H/H,
WAVE PERIOD, SEC
3.53
2.88
2.03
0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
30
40
50
60
70
80
90
100
110
120
130
140



COMPARISON OF FREQUENCY RESPONSE
IN MARINERS BASIN
WAVE GAGE 25

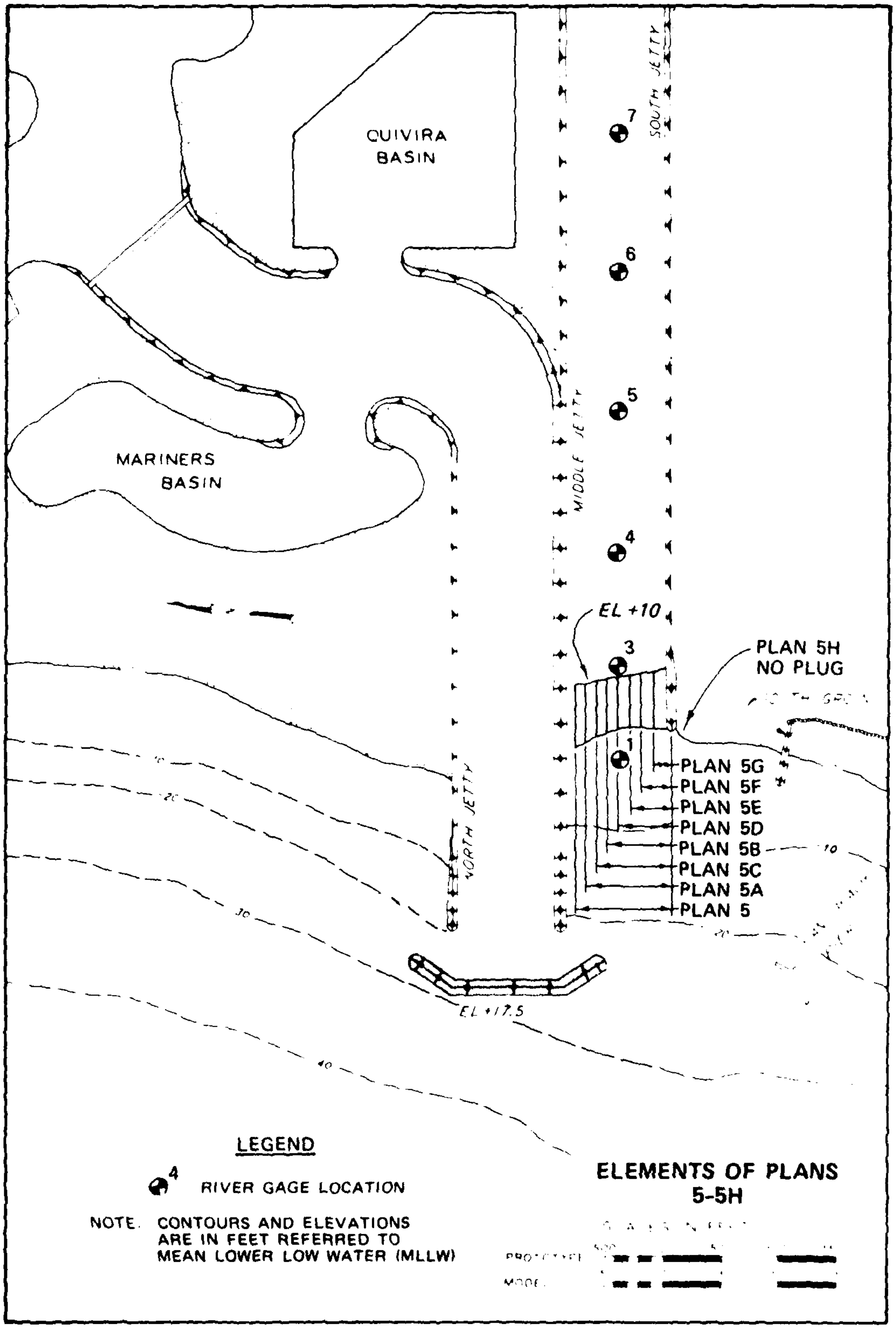


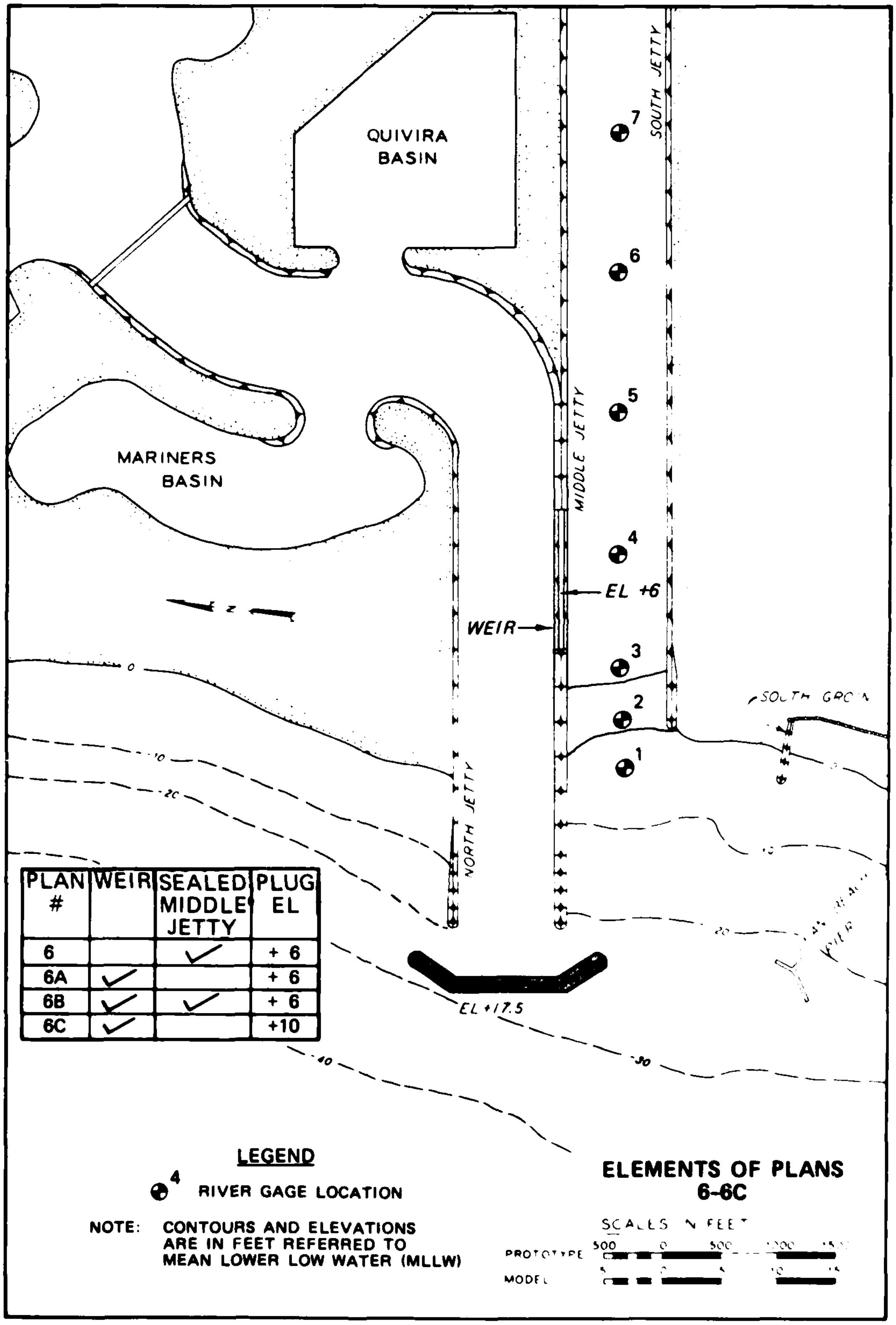
COMPARISON OF FREQUENCY RESPONSE
IN MARINERS BASIN
WAVE GAGE 26



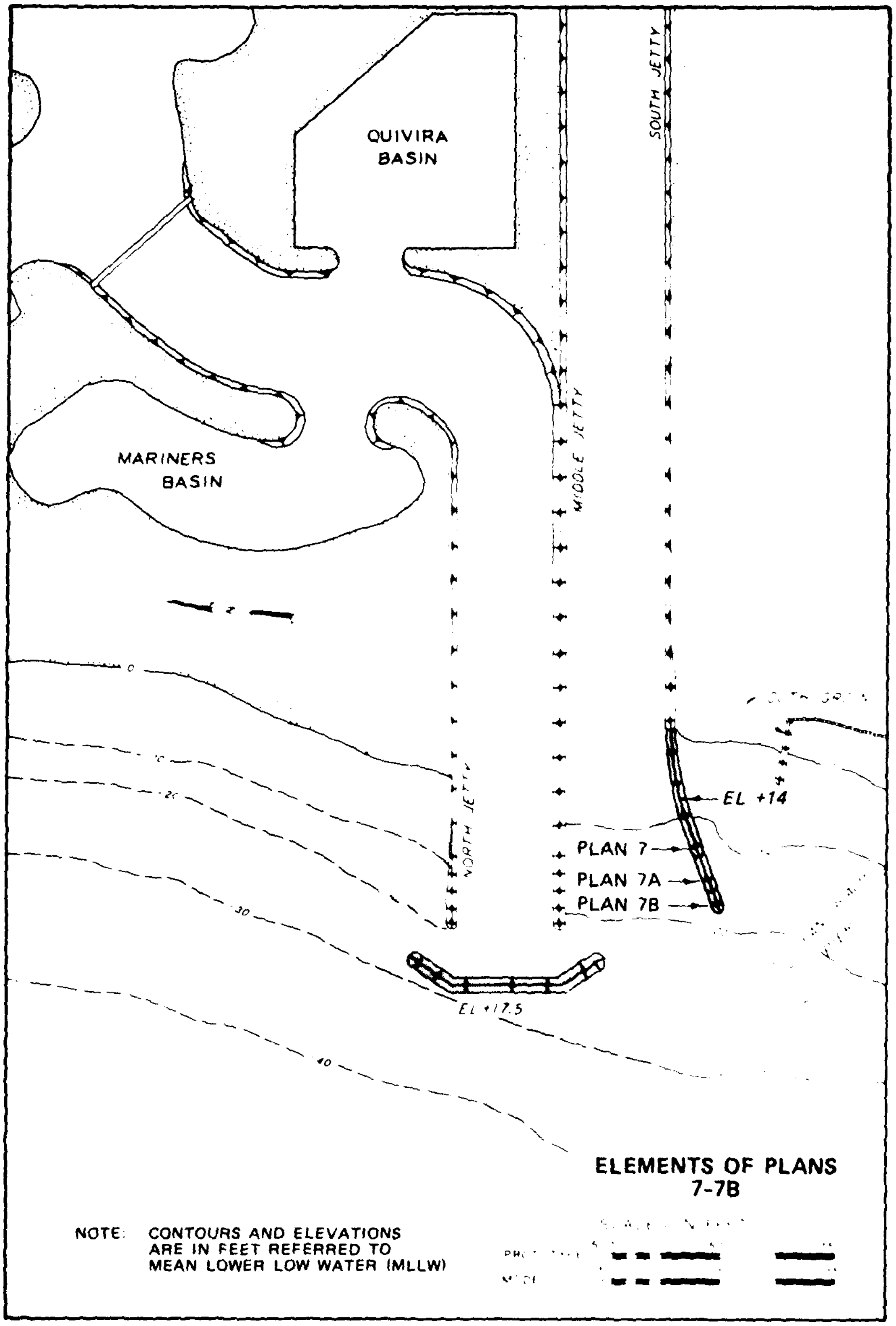
* PLAN #	PLUG EL
4	+ 6
4A	+10
4B	+14

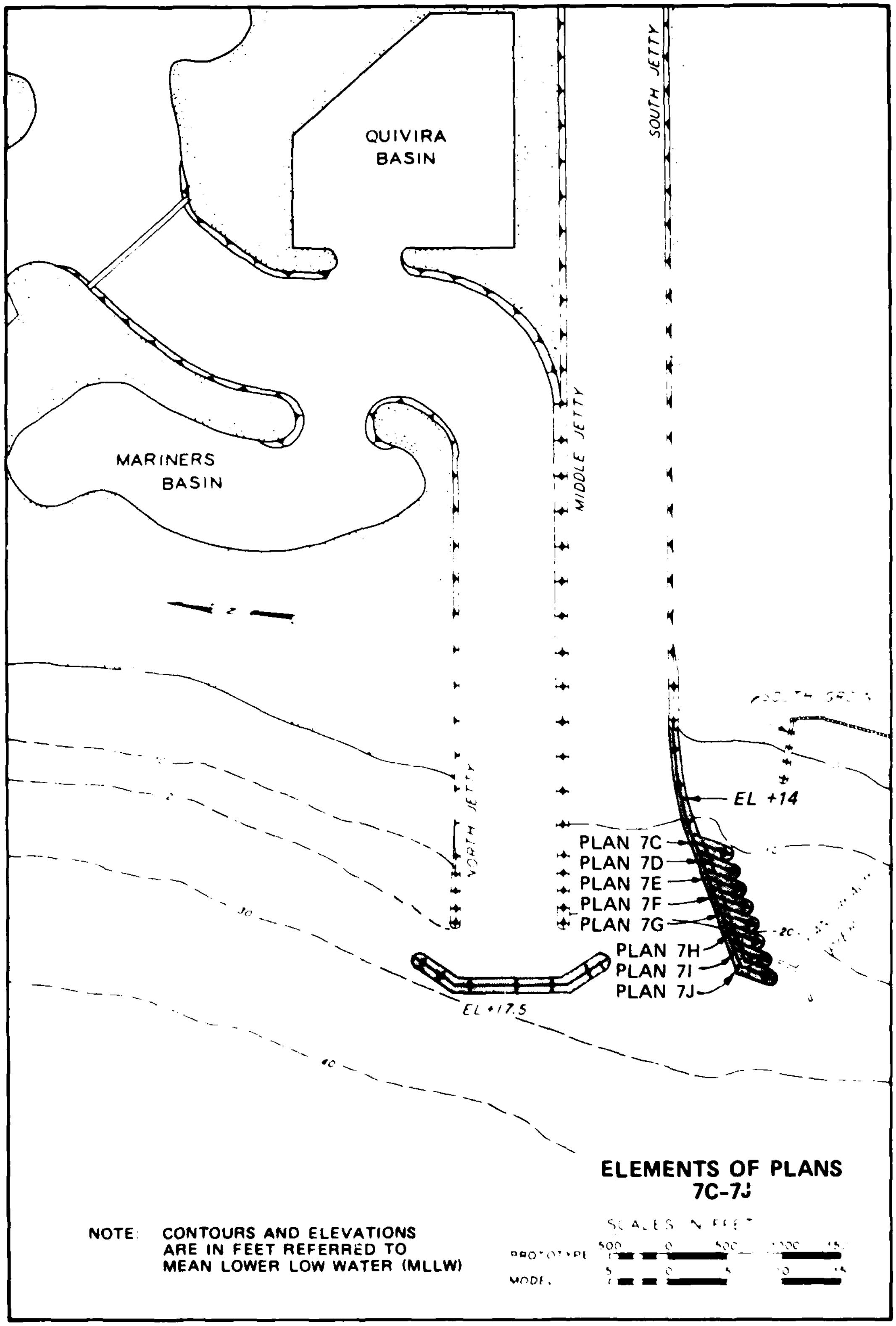
PLATE 30


QUIVIRA BASIN
MARINERS BASIN
SOUTH JETTY
MIDDLE JETTY
NORTH JETTY
EL +10
EL +17.5
PLAN 5H NO PLUG
PLAN 5G
PLAN 5F
PLAN 5E
PLAN 5D
PLAN 5B
PLAN 5C
PLAN 5A
PLAN 5
LEGEND
RIVER GAGE LOCATION
NOTE. CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER (MLLW)
ELEMENTS OF PLANS 5-5H

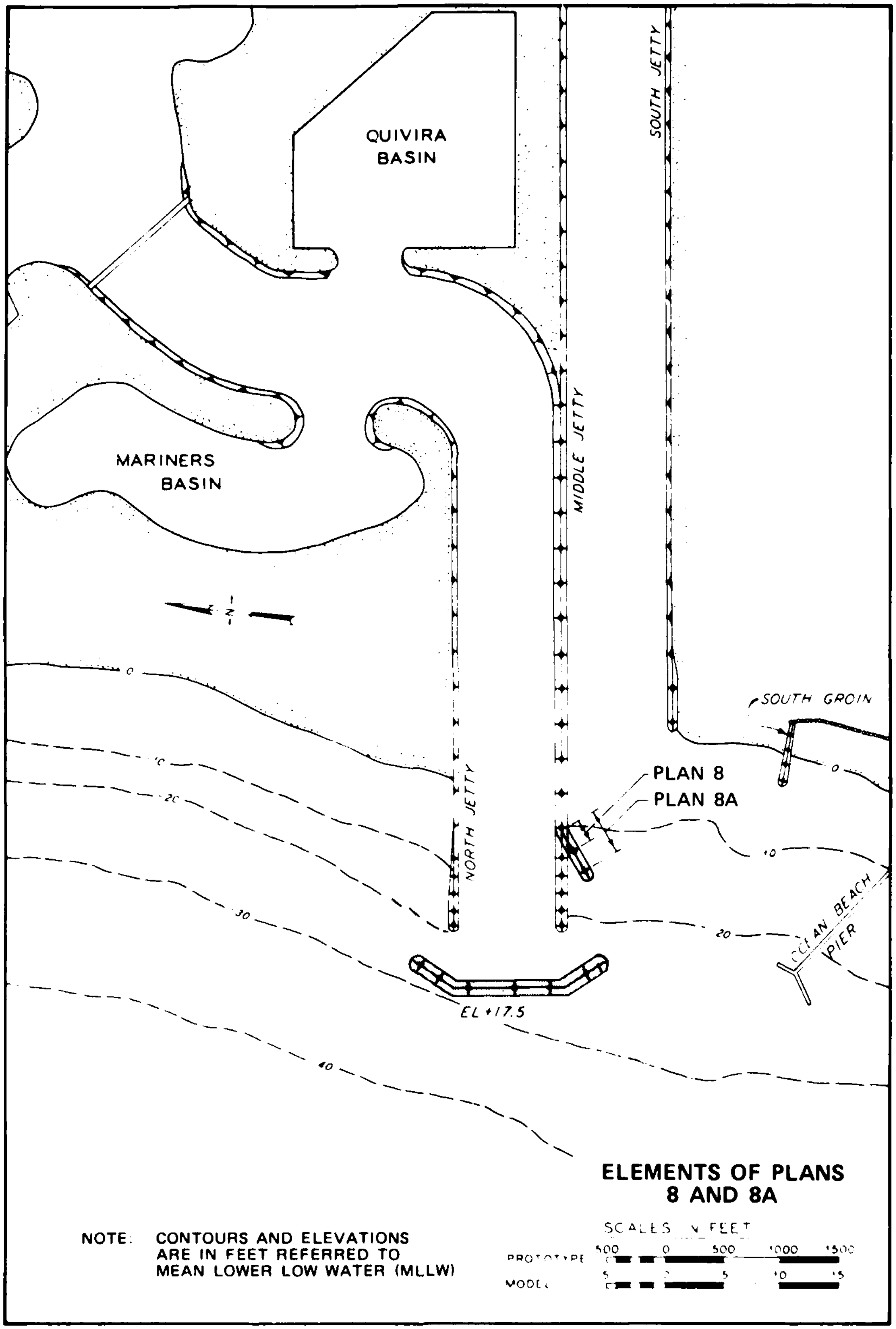


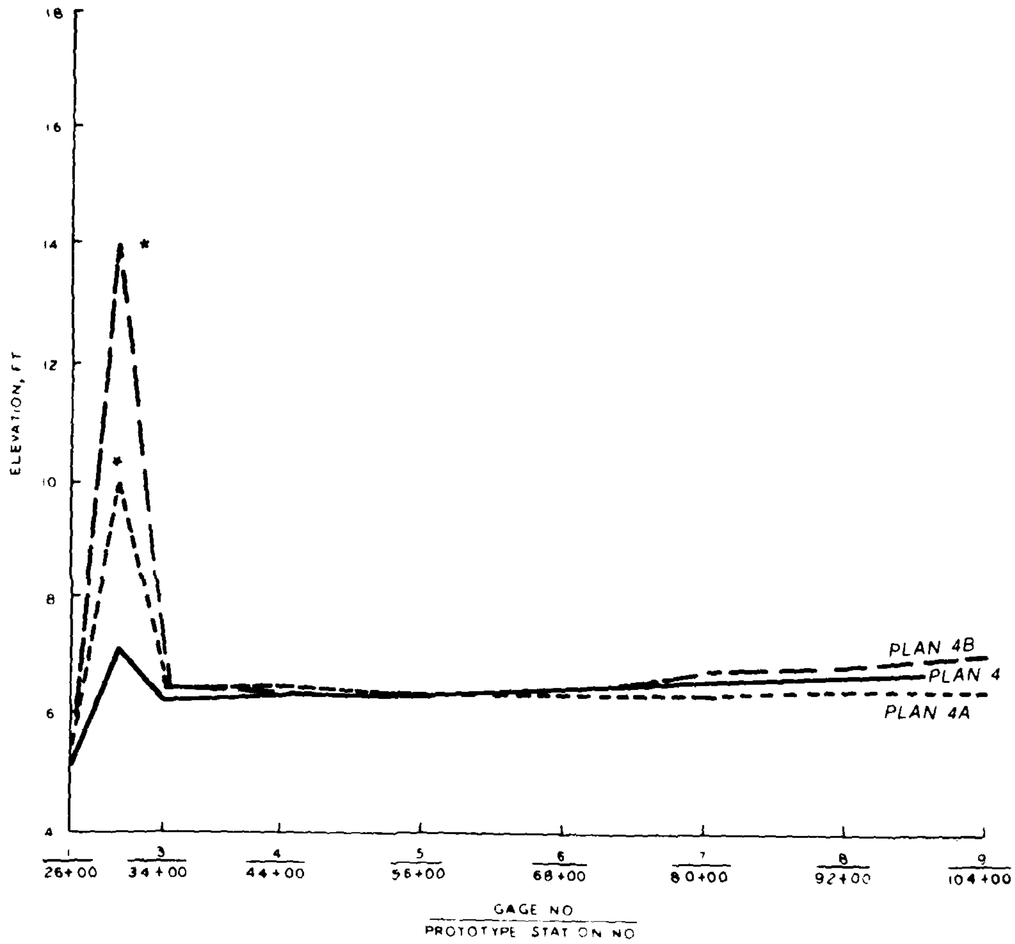
PLAN #	WEIR	SEALED MIDDLE JETTY	PLUG EL
6		✓	+ 6
6A	✓		+ 6
6B	✓	✓	+ 6
6C	✓		+10


QUIVIRA BASIN
SOUTH JETTY
MIDDLE JETTY
MARINERS BASIN
NORTH JETTY
0
10
20
30
40
EL +14
PLAN 7
PLAN 7A
PLAN 7B
EL +17.5
ELEMENTS OF PLANS 7-7B
NOTE: CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER (MLLW)



ELEMENTS OF PLANS 7C-7J

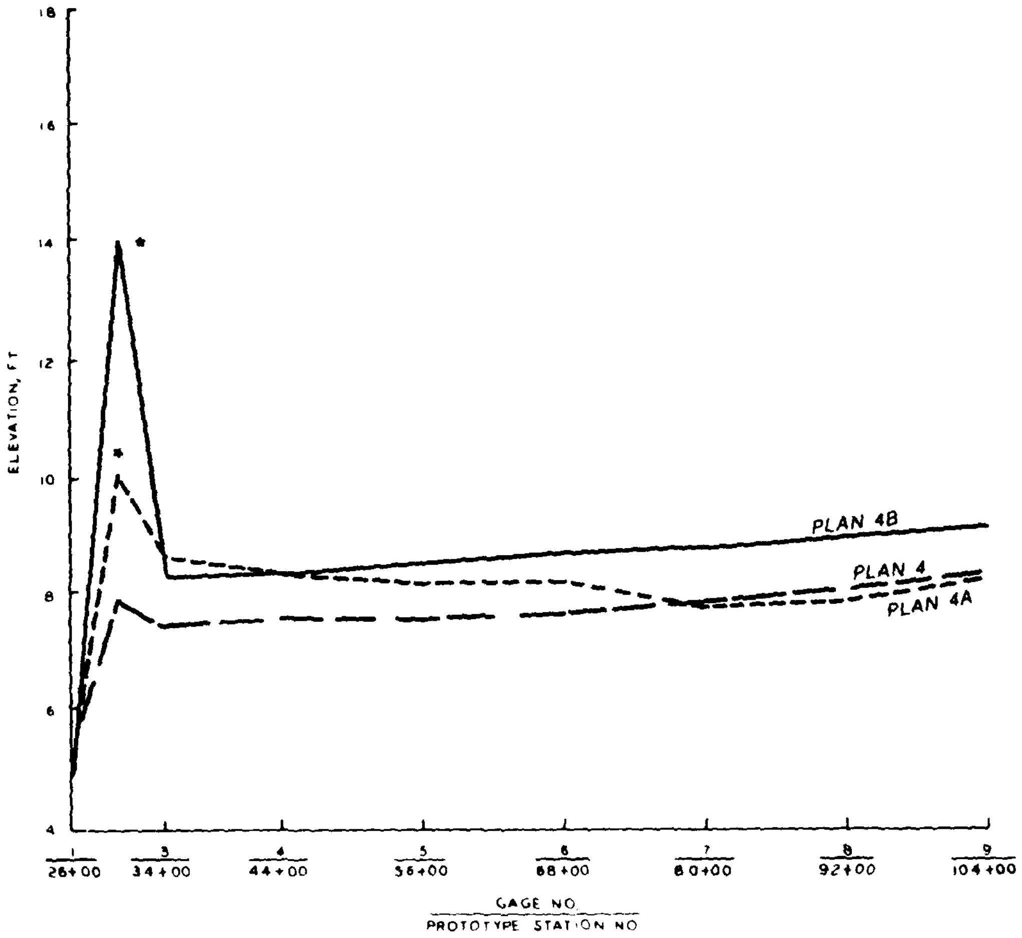

QUIVIRA
BASIN
SOUTH JETTY
MARINERS
BASIN
MIDDLE JETTY
NORTH JETTY
SOUTH GROIN
PLAN 8
PLAN 8A
OCEAN BEACH
PIER
EL +17.5
0
10
20
30
40
ELEMENTS OF PLANS
8 AND 8A
NOTE: CONTOURS AND ELEVATIONS
ARE IN FEET REFERRED TO
MEAN LOWER LOW WATER (MLLW)
SCALES IN FEET
PROTOTYPE 500 0 500 1000 1500
MODEL 5 0 5 10 15



*CREST ELEVATION OF PLUG

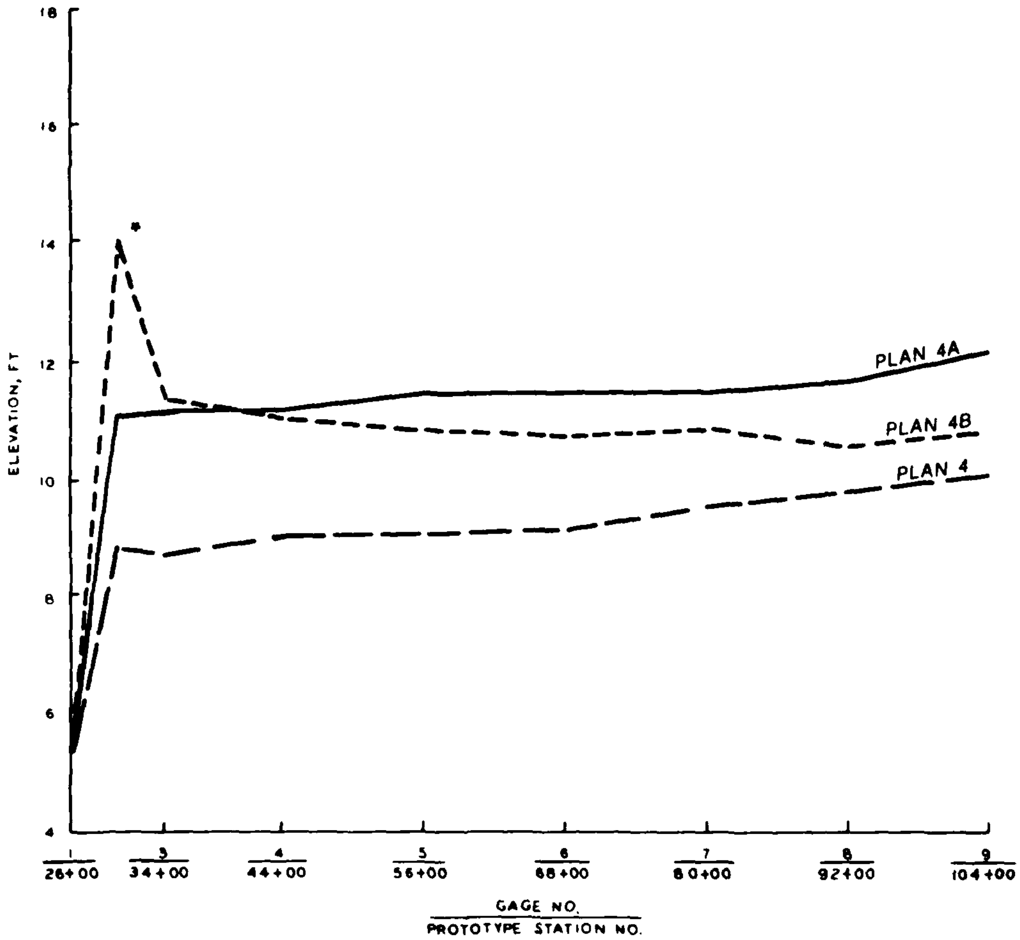
SAN DIEGO RIVER PROFILE
(WITHOUT MODEL ROUGHNESS, UNSEALED MIDDLE JETTY)
PLANS 4-4B, 11,000-CFS DISCHARGE

PLATE 36



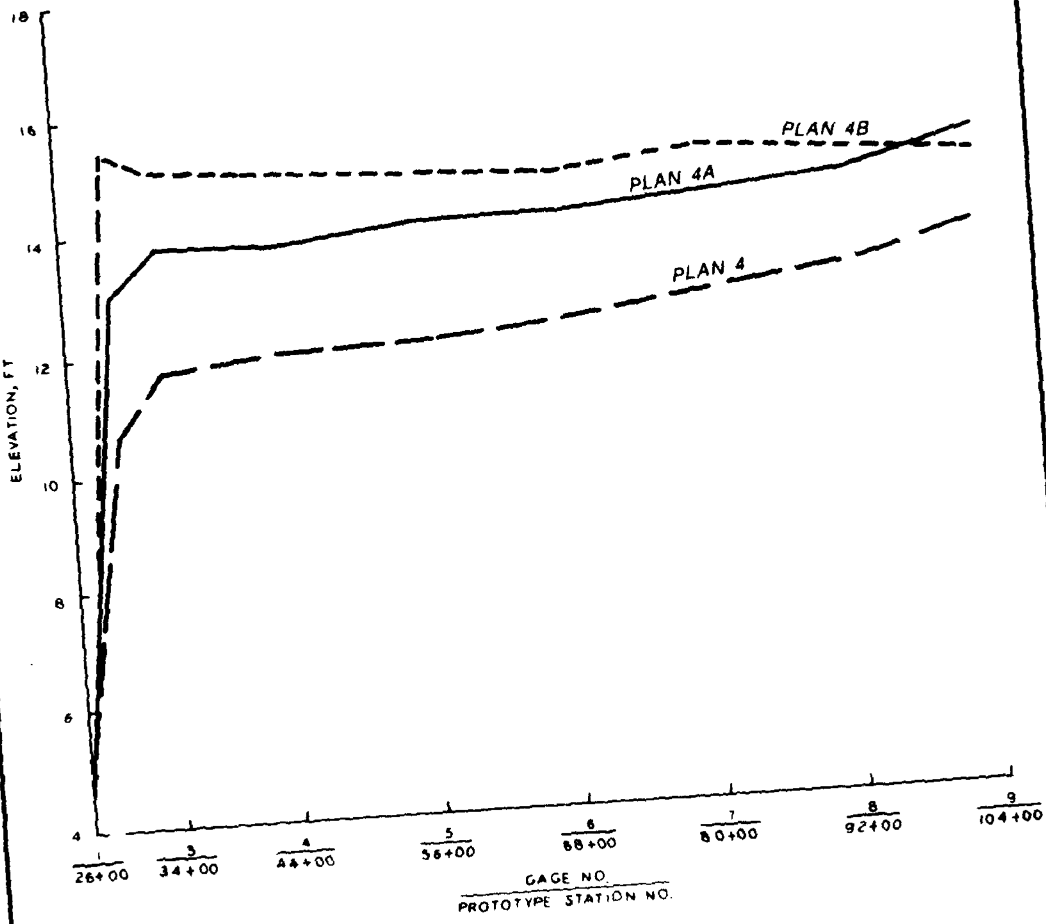
*CREST ELEVATION OF PLUG

SAN DIEGO RIVER PROFILE
(WITHOUT MODEL ROUGHNESS, UNSEALED MIDDLE JETTY)
PLANS 4-4B, 27,000-CFS DISCHARGE

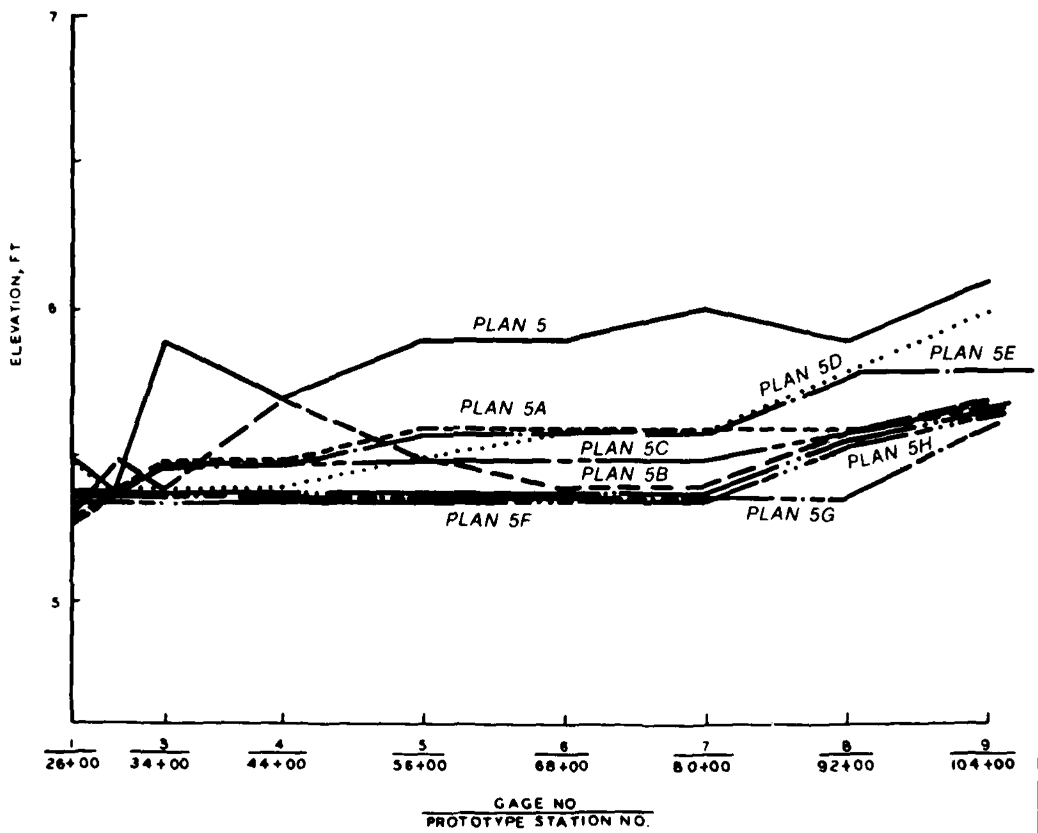


SAN DIEGO RIVER PROFILE
(WITHOUT MODEL ROUGHNESS;
UNSEALED MIDDLE JETTY)
PLANS 4-4B; 49,000-CFS DISCHARGE

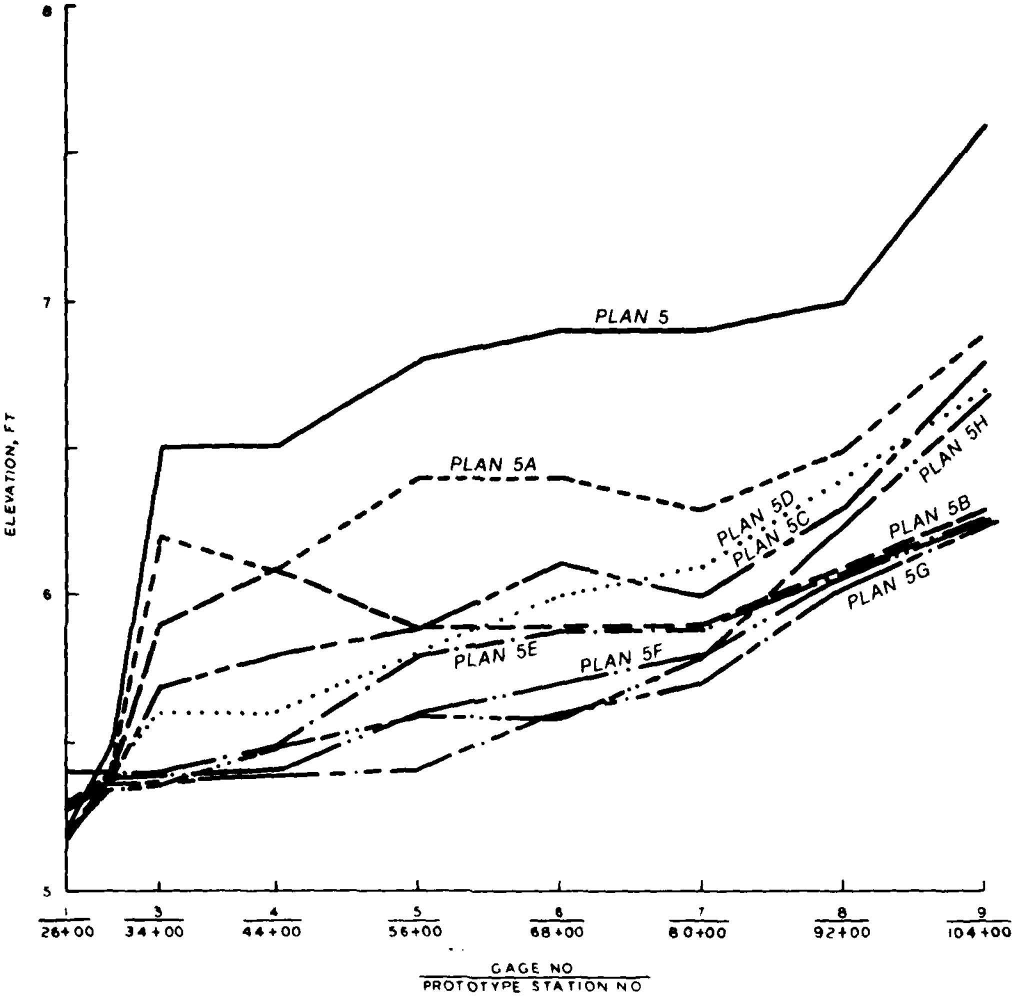
*CREST ELEVATION
OF PLUG



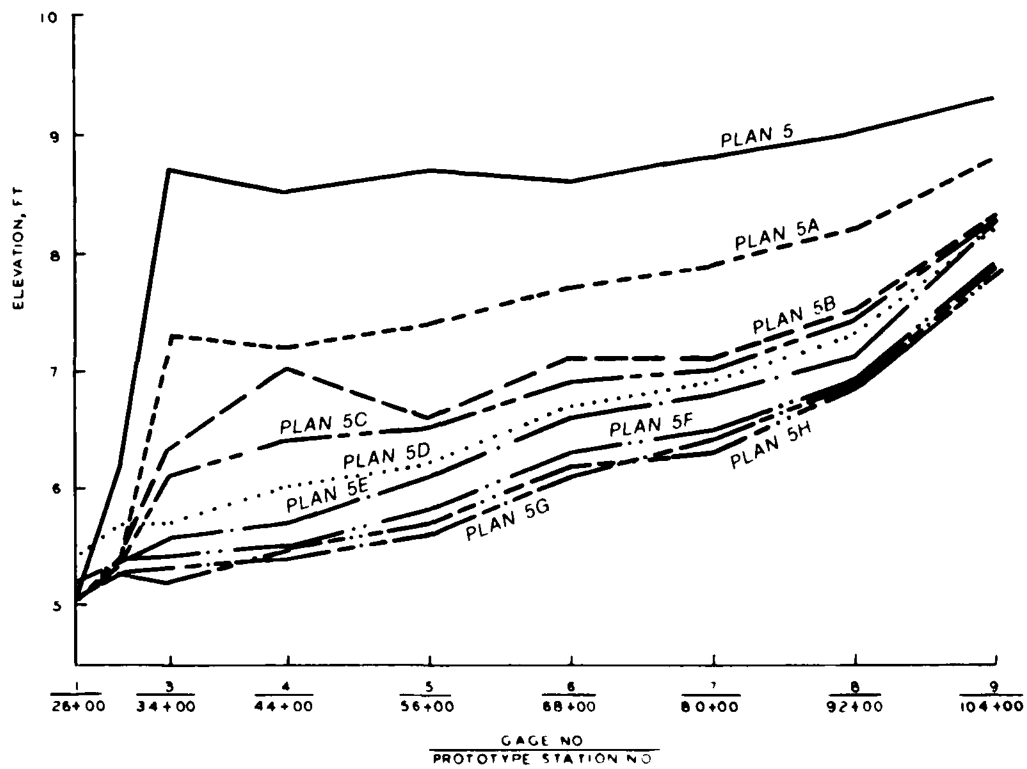
SAN DIEGO RIVER PROFILE
(WITHOUT MODEL ROUGHNESS, UNSEALED MIDDLE JETTY)
PLANS 4-4B, 97,000-CFS DISCHARGE



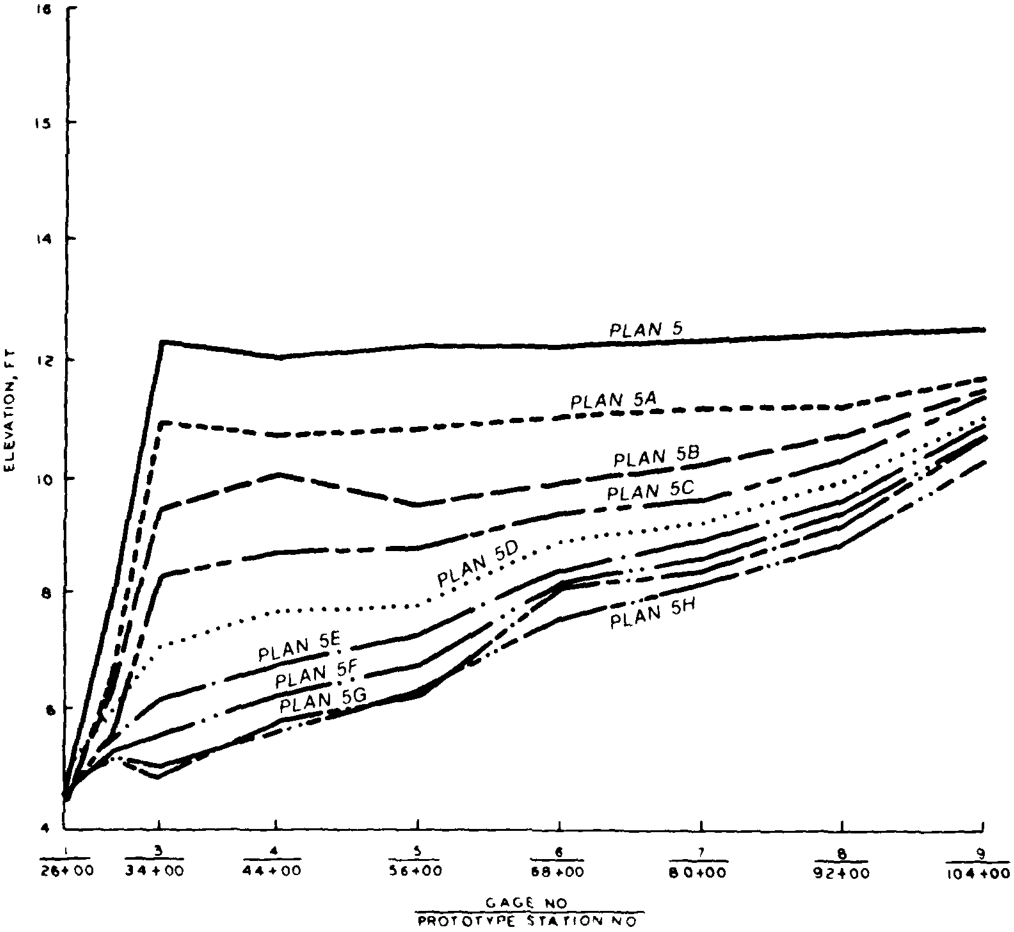
SAN DIEGO RIVER PROFILE
(WITHOUT MODEL ROUGHNESS; UNSEALED MIDDLE JETTY)
PLANS 5-5H; 11,000-CFS DISCHARGE



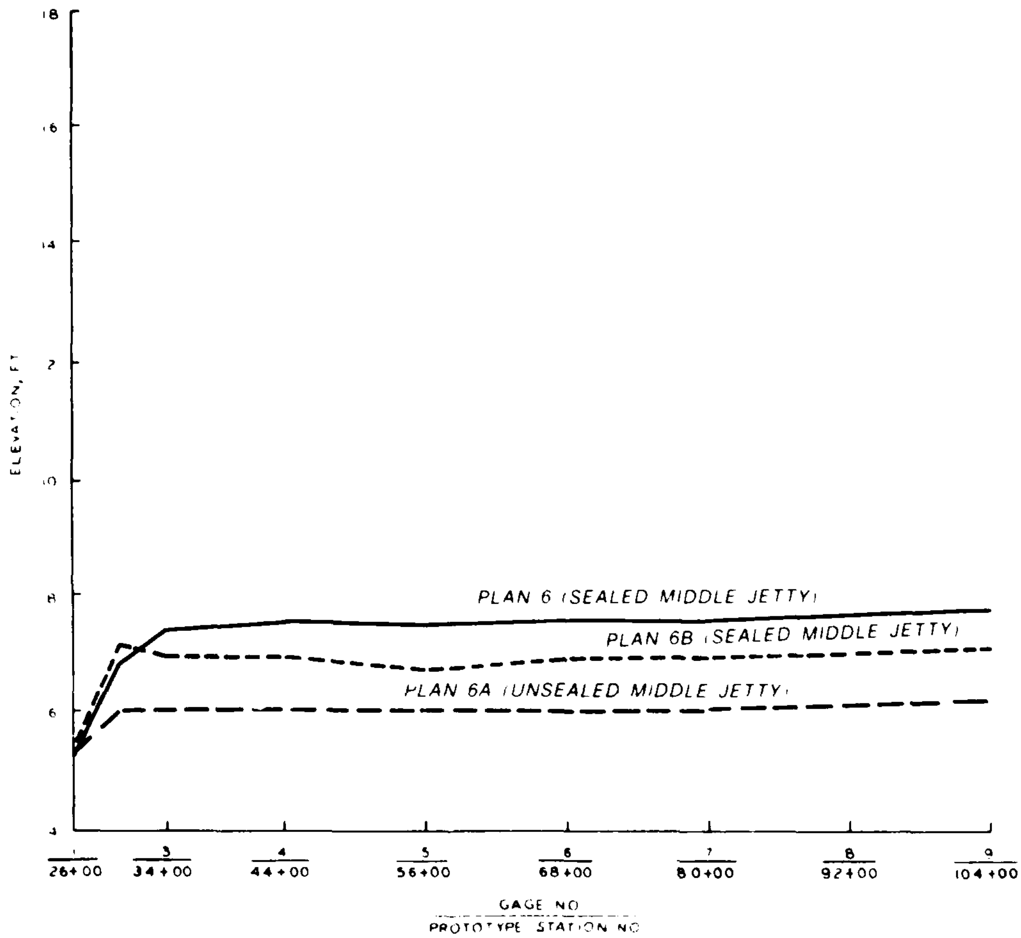
SAN DIEGO RIVER PROFILE
(WITHOUT MODEL ROUGHNESS; UNSEALED MIDDLE JETTY)
PLANS 5-5H, 27,000-CFS DISCHARGE



SAN DIEGO RIVER PROFILE
(WITHOUT MODEL ROUGHNESS;
UNSEALED MIDDLE JETTY)
PLANS 5-5H 49,000-CFS DISCHARGE

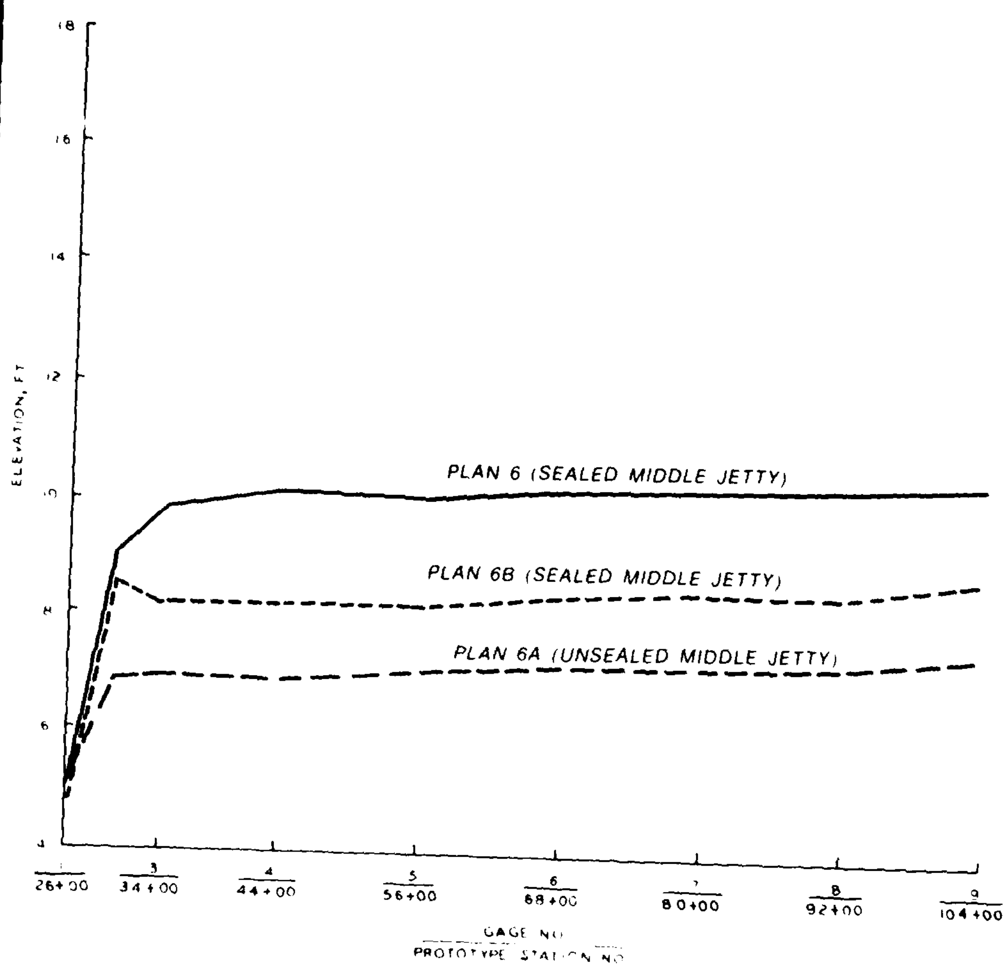


SAN DIEGO RIVER PROFILE
(WITHOUT MODEL ROUGHNESS;
UNSEALED MIDDLE JETTY)
PLANS 5-5H, 97,000-CFS DISCHARGE

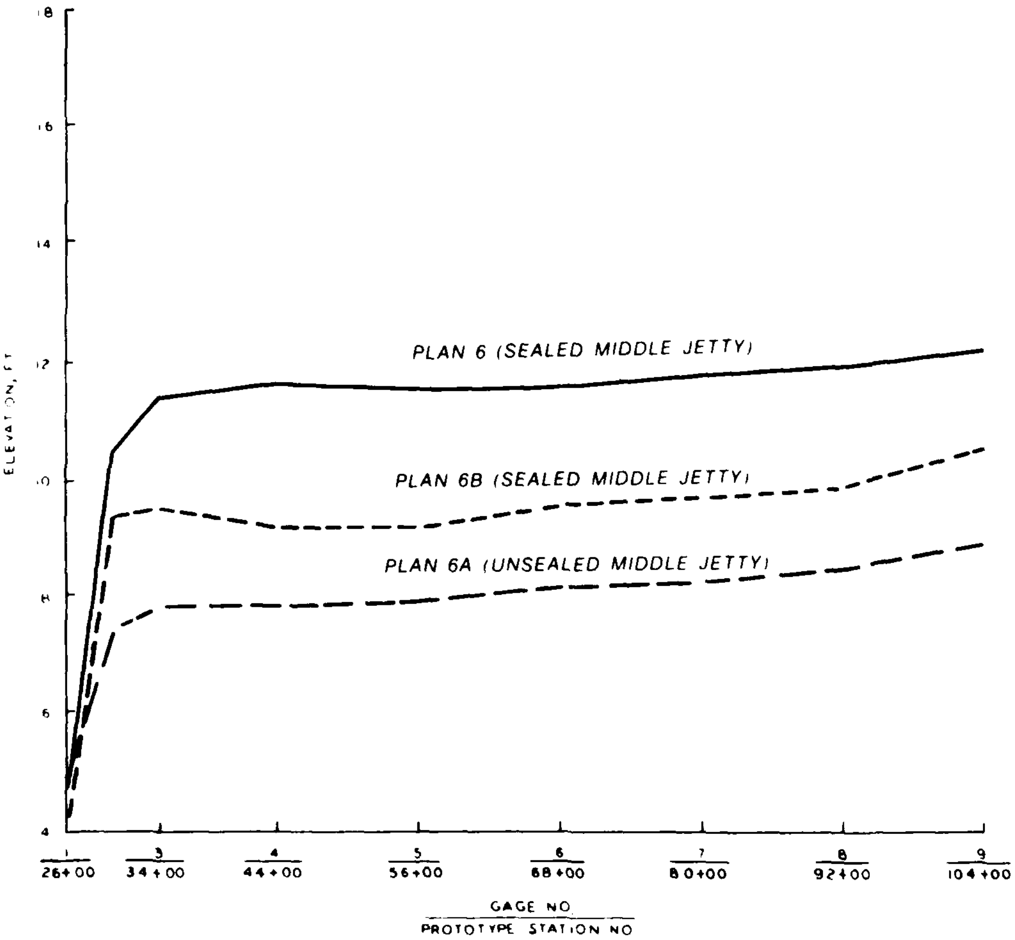


SAN DIEGO RIVER PROFILE
(WITHOUT MODEL ROUGHNESS)
PLANS 6-6B, 11,000-CFS DISCHARGE

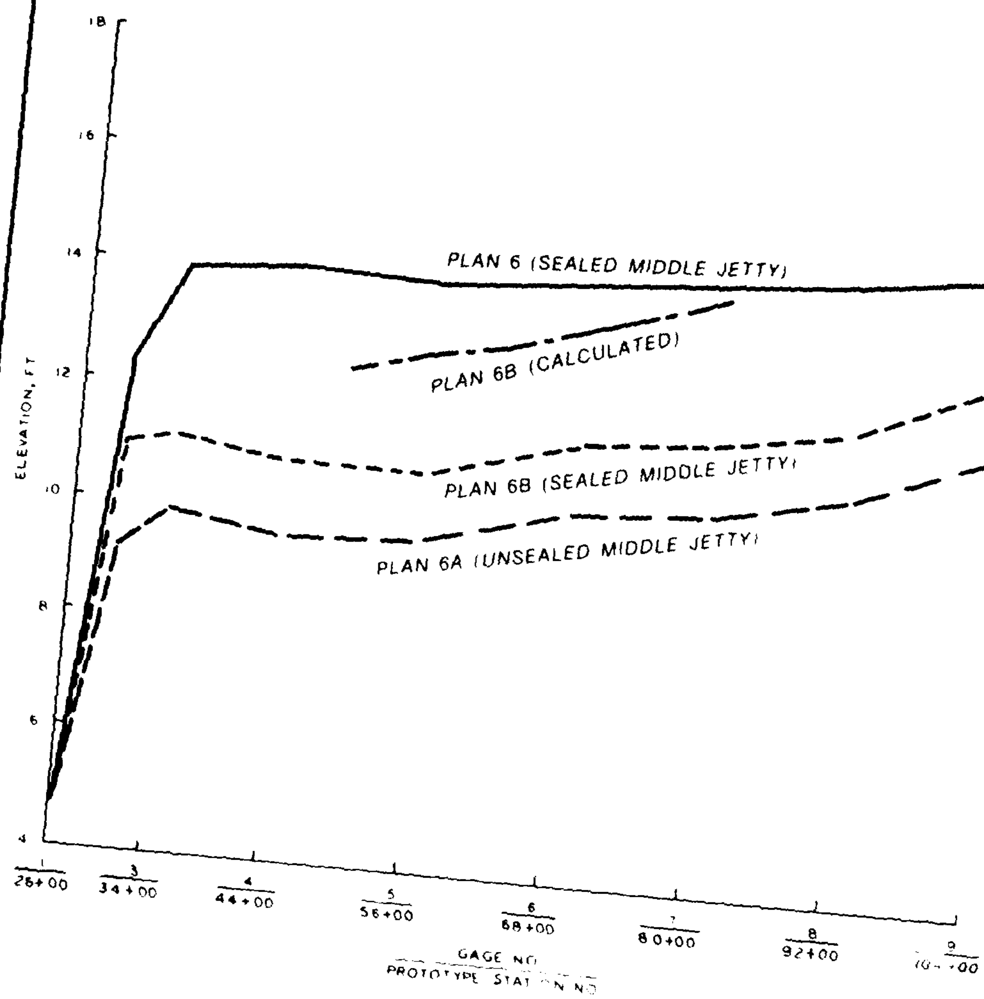
PLATE 44



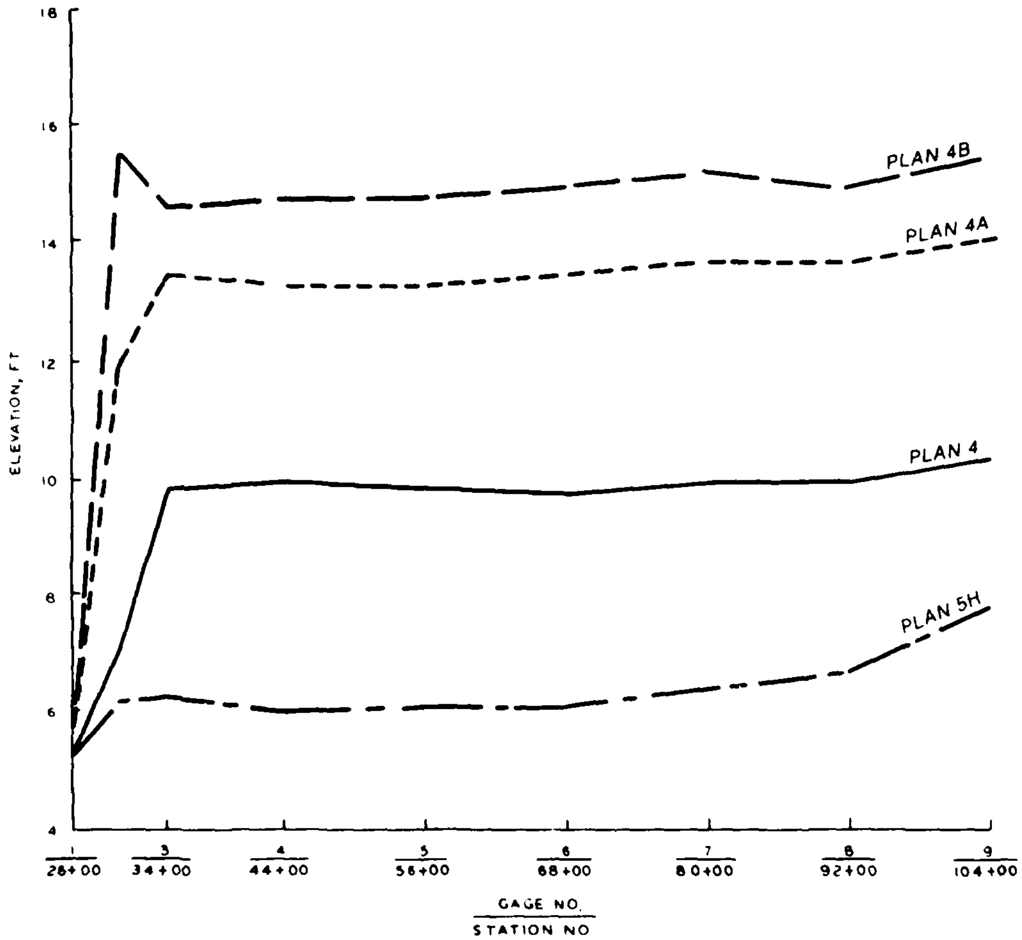
SAN DIEGO RIVER PROFILE
(WITHOUT MODEL ROUGHNESS)
PLANS 6-6B 27,000-CFS DISCHARGE



SAN DIEGO RIVER PROFILE
(WITHOUT MODEL ROUGHNESS)
PLANS 6-6B, 49,000-CFS DISCHARGE

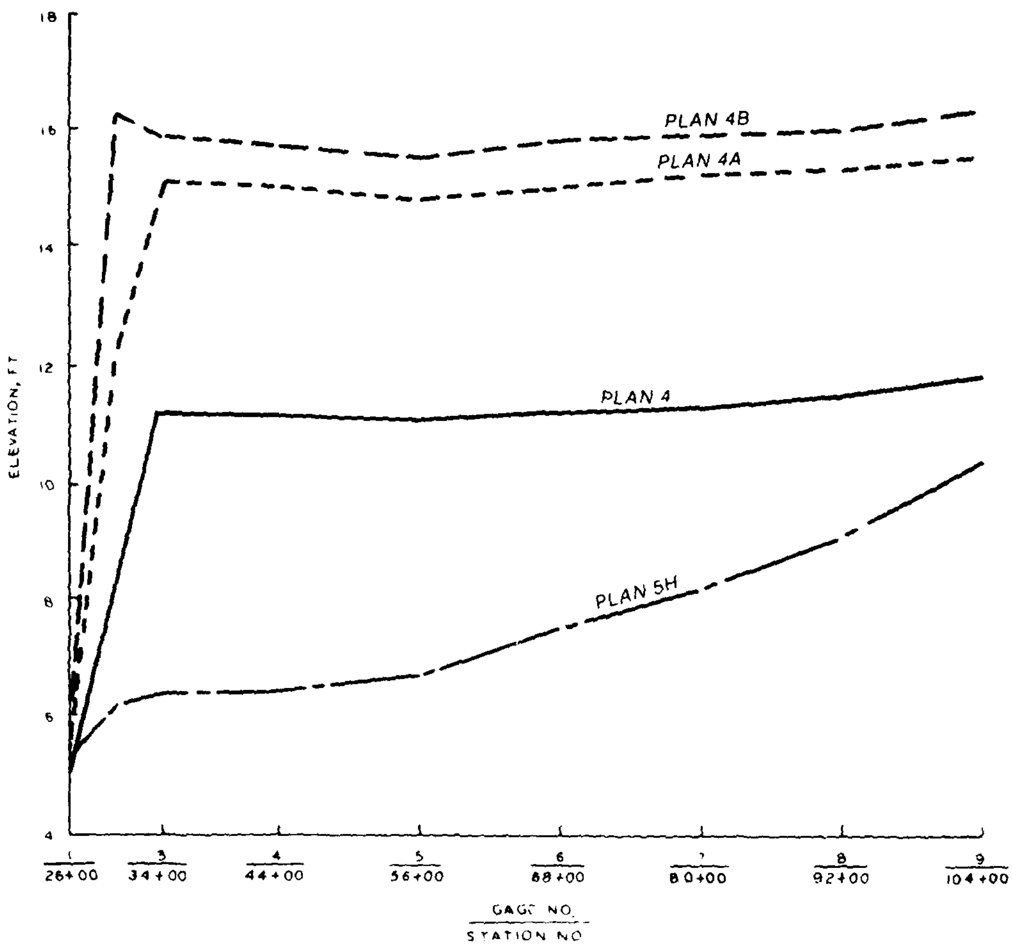


SAN DIEGO RIVER PROFILE
(WITHOUT MODEL ROUGHNESS)
PLANS 6-6B 97,000-CFS DISCHARGE

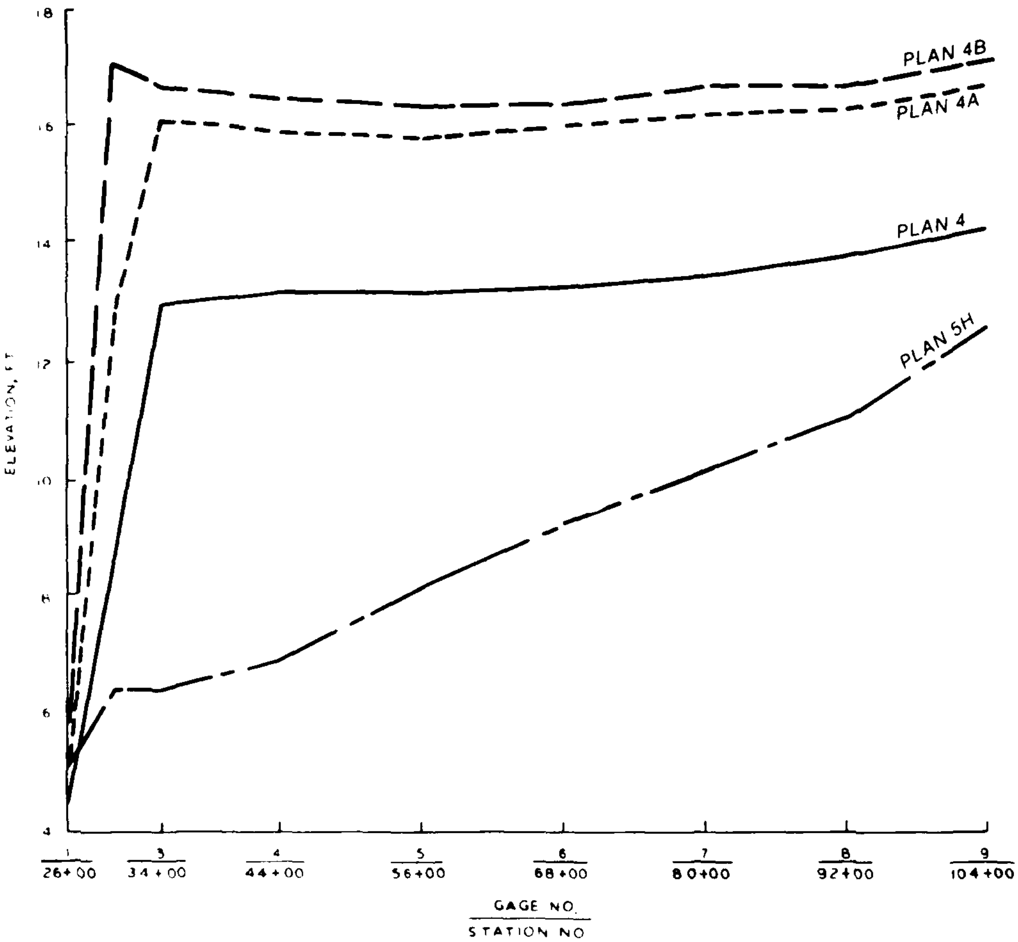


SAN DIEGO RIVER PROFILE
(WITH MODEL ROUGHNESS,
SEALED MIDDLE JETTY)
PLANS 4-4B, AND 5H, 11,000-CFS DISCHARGE

PLATE 48

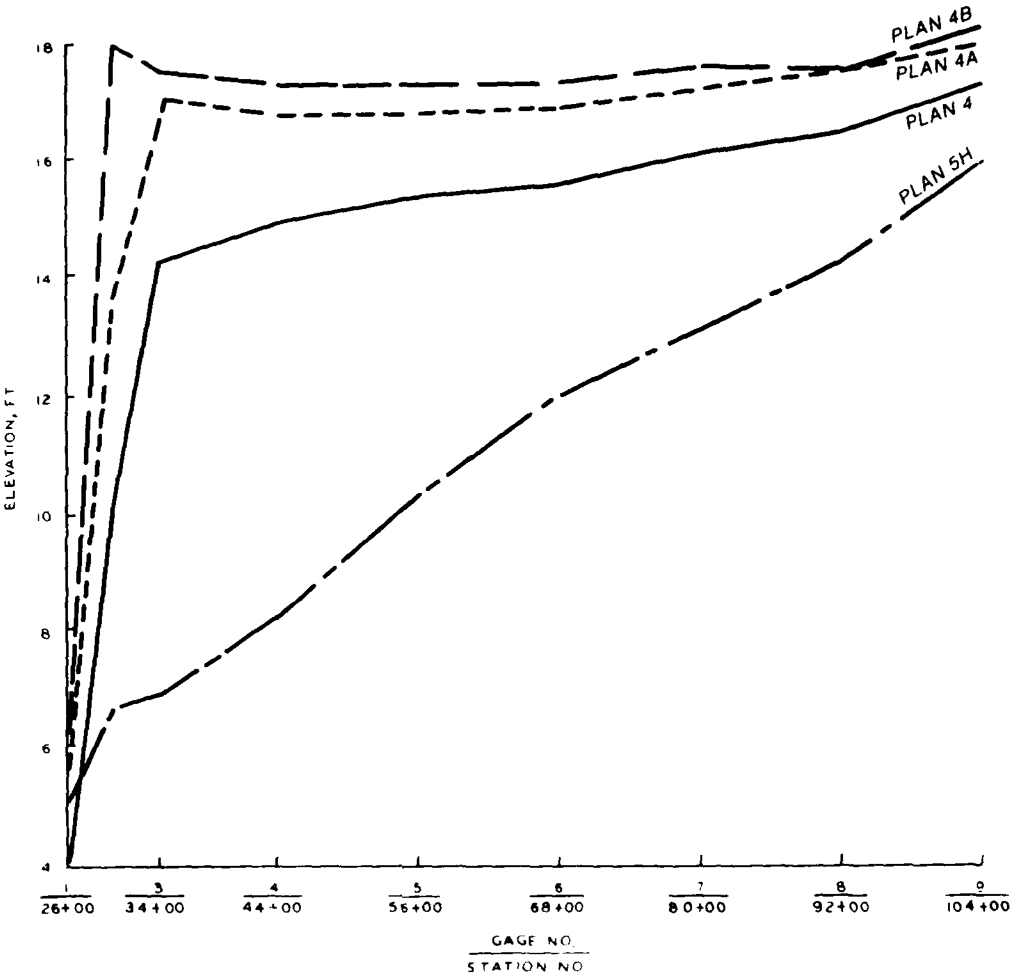


SAN DIEGO RIVER PROFILE
(WITH MODEL ROUGHNESS;
SEALED MIDDLE JETTY)
PLANS 4-4B, AND 5H, 27,000-CFS DISCHARGE

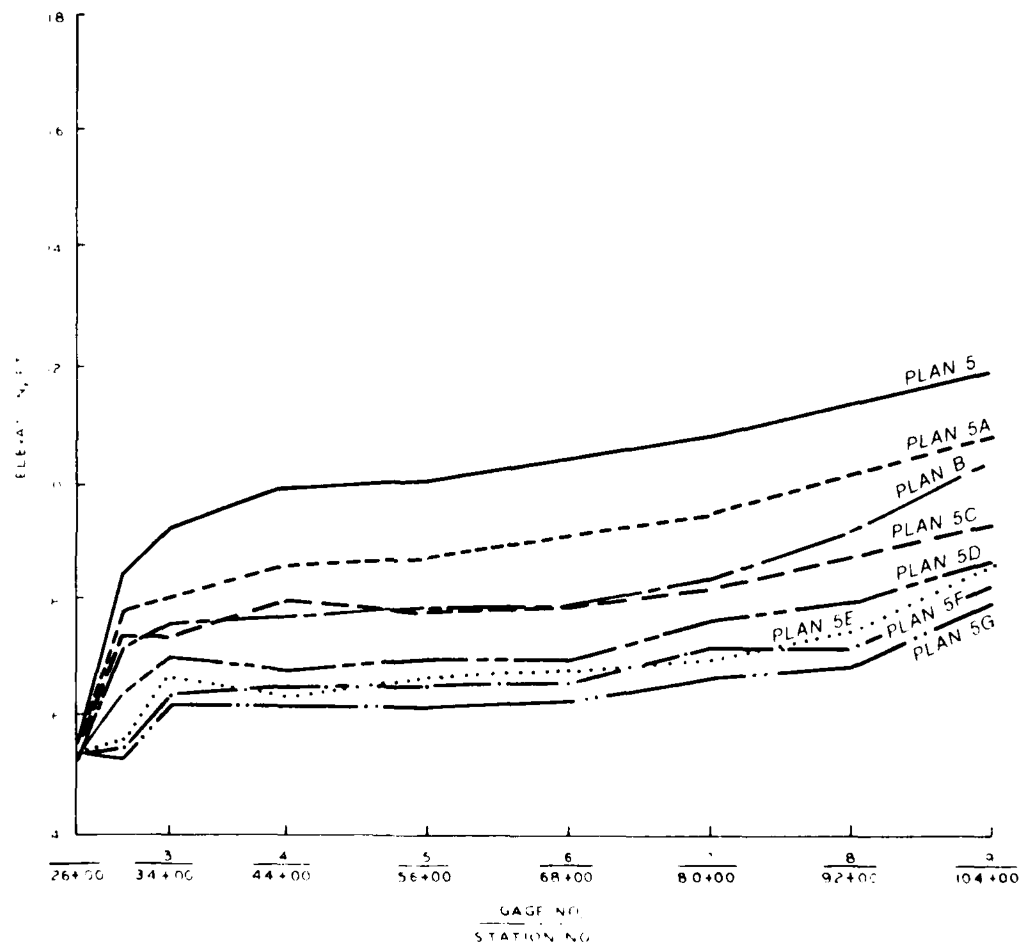


SAN DIEGO RIVER PROFILE
(WITH MODEL ROUGHNESS.
SEALED MIDDLE JETTY)
PLANS 4, 4B AND 5H 49,000 CFS DISCHARGE

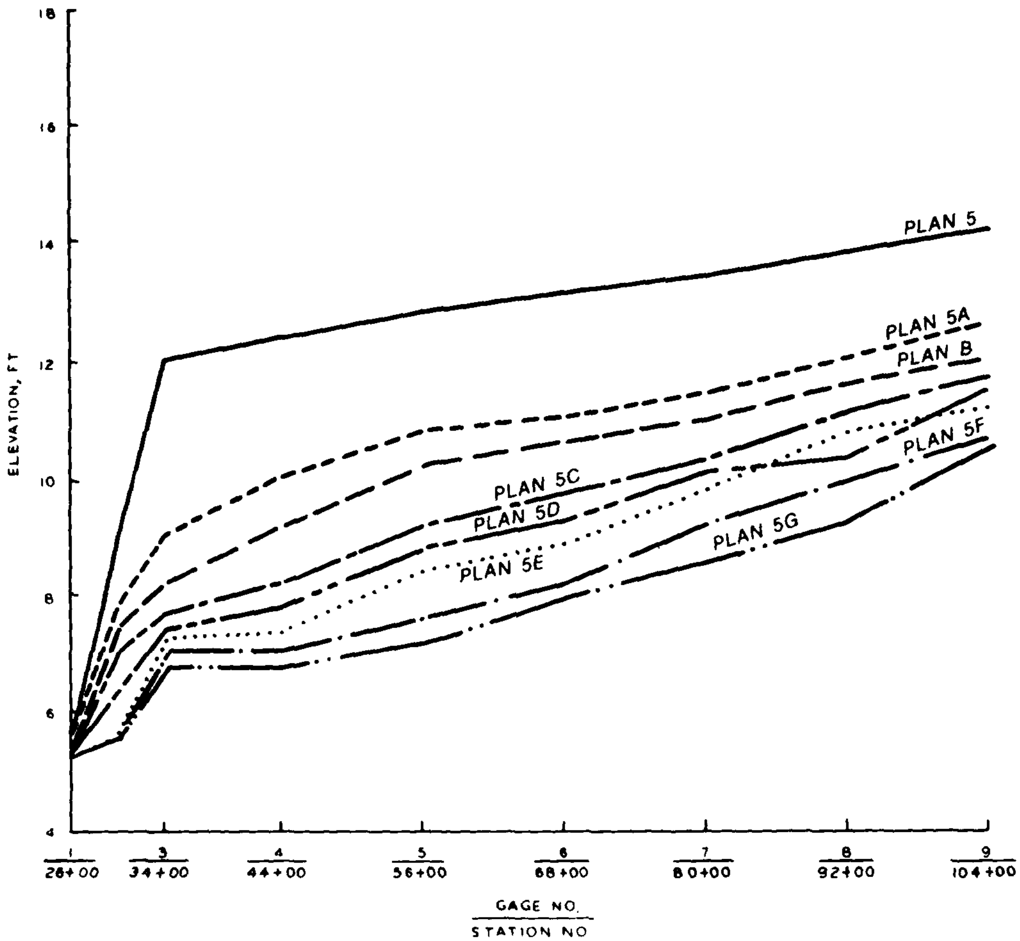
PLATE 50



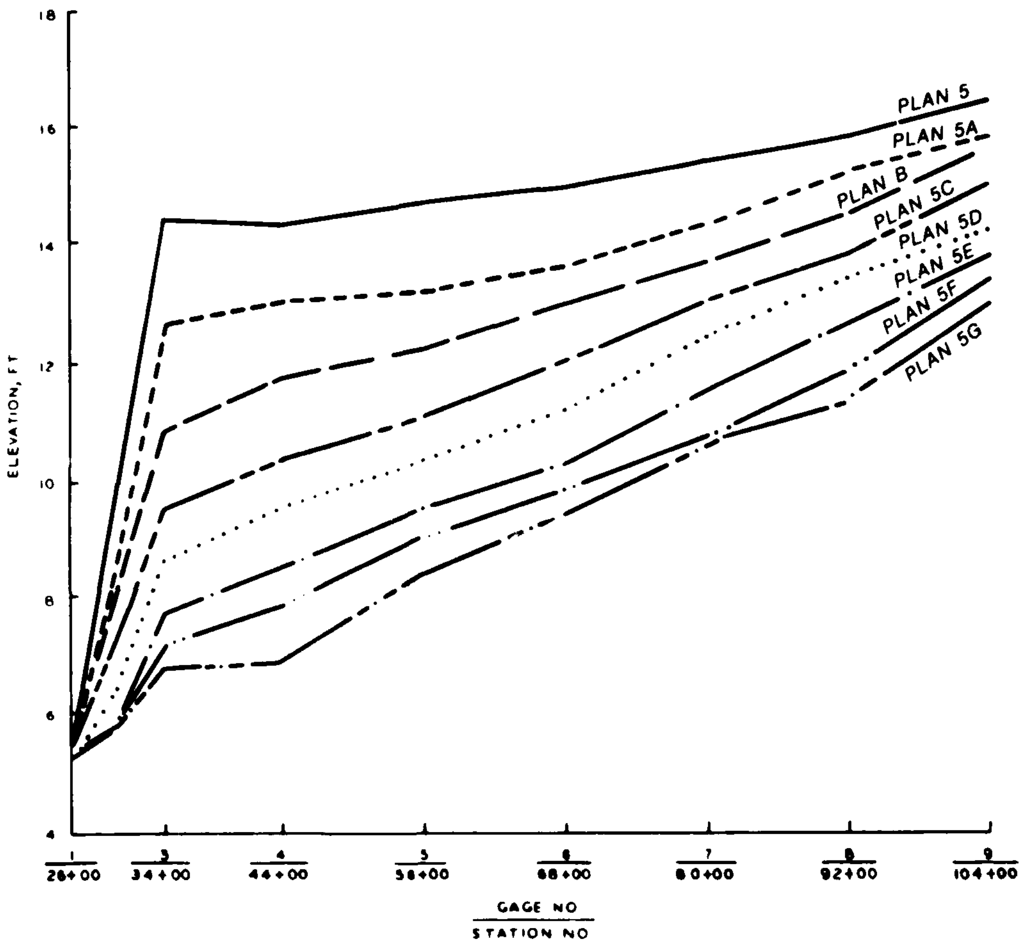
SAN DIEGO RIVER PROFILE
(WITH MODEL ROUGHNESS,
SEALED MIDDLE JETTY)
PLANS 4-4B, AND 5H, 97,000-CFS DISCHARGE



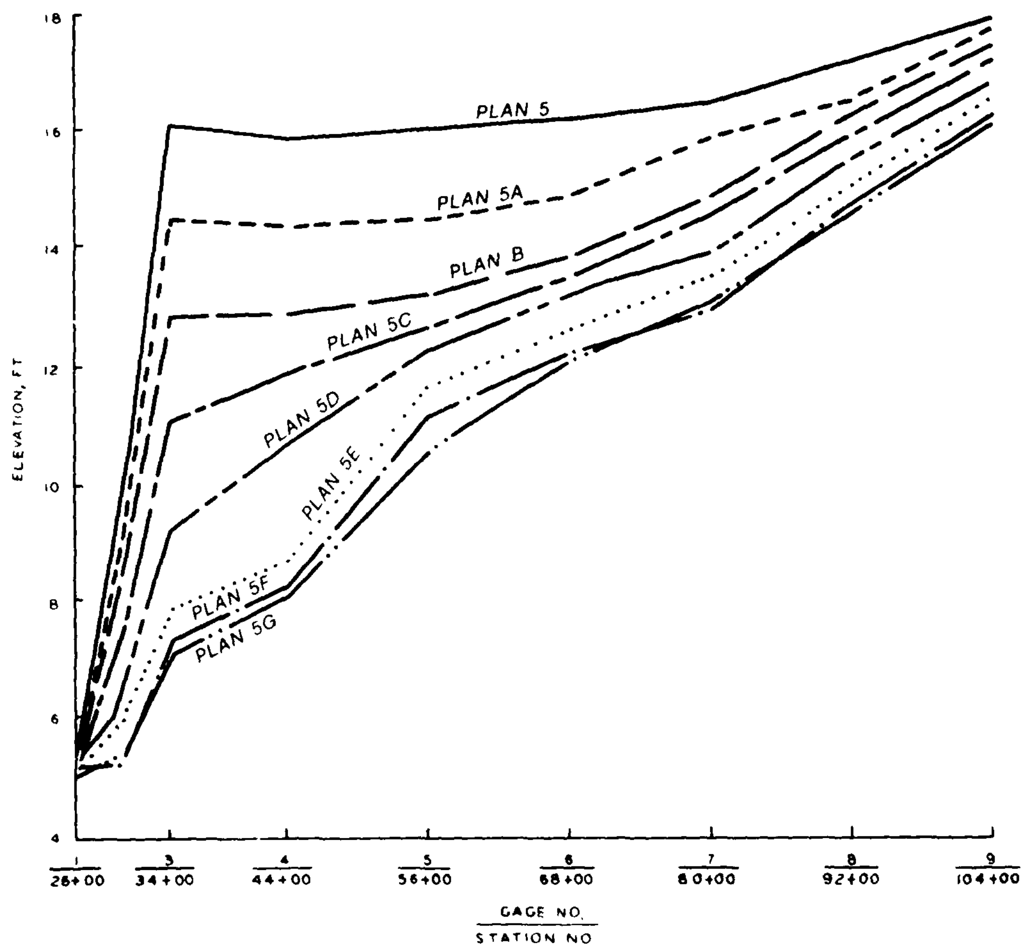
SAN DIEGO RIVER PROFILE
(WITH MODEL ROUGHNESS,
SEALED MIDDLE JETTY)
PLANS 5-5G 11,000-CFS DISCHARGE



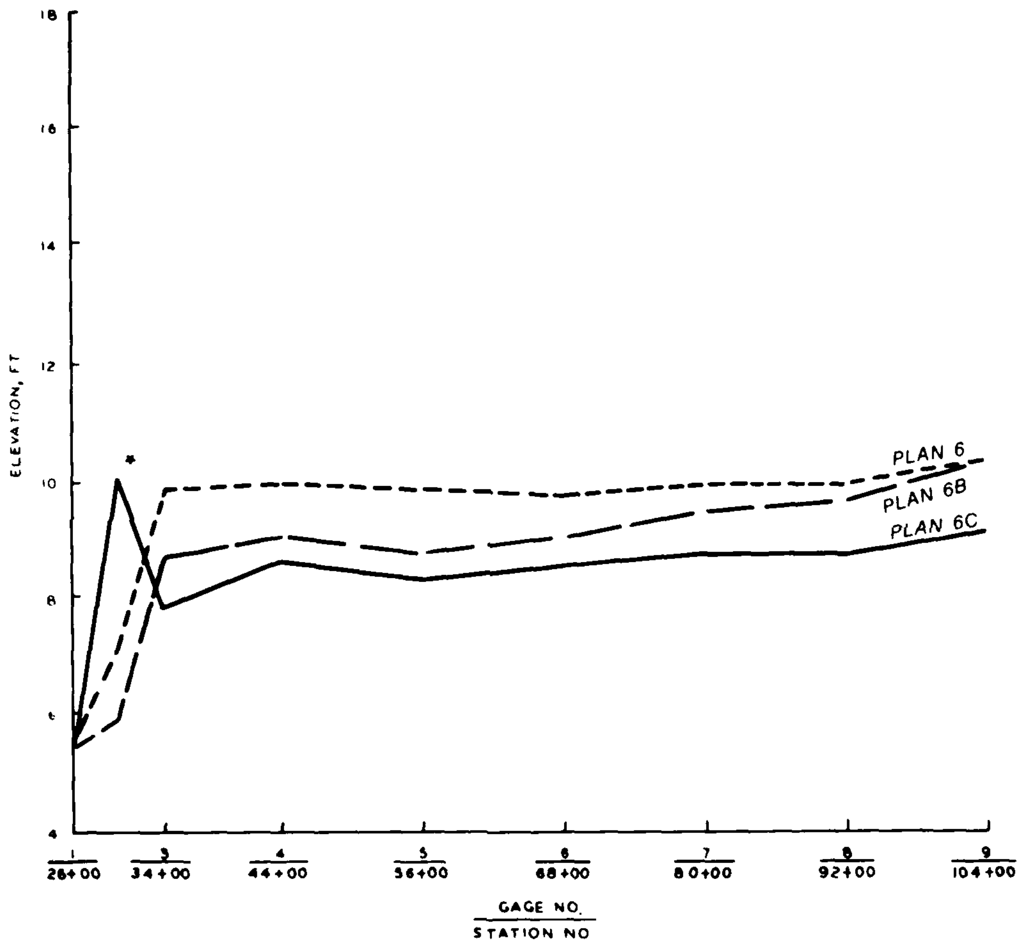
SAN DIEGO RIVER PROFILE
(WITH MODEL ROUGHNESS;
SEALED MIDDLE JETTY)
PLANS 5-5G 27,000-CFS DISCHARGE



SAN DIEGO RIVER PROFILE
(WITH MODEL ROUGHNESS; SEALED MIDDLE JETTY)
PLANS 5-5G, 49,000-CFS DISCHARGE

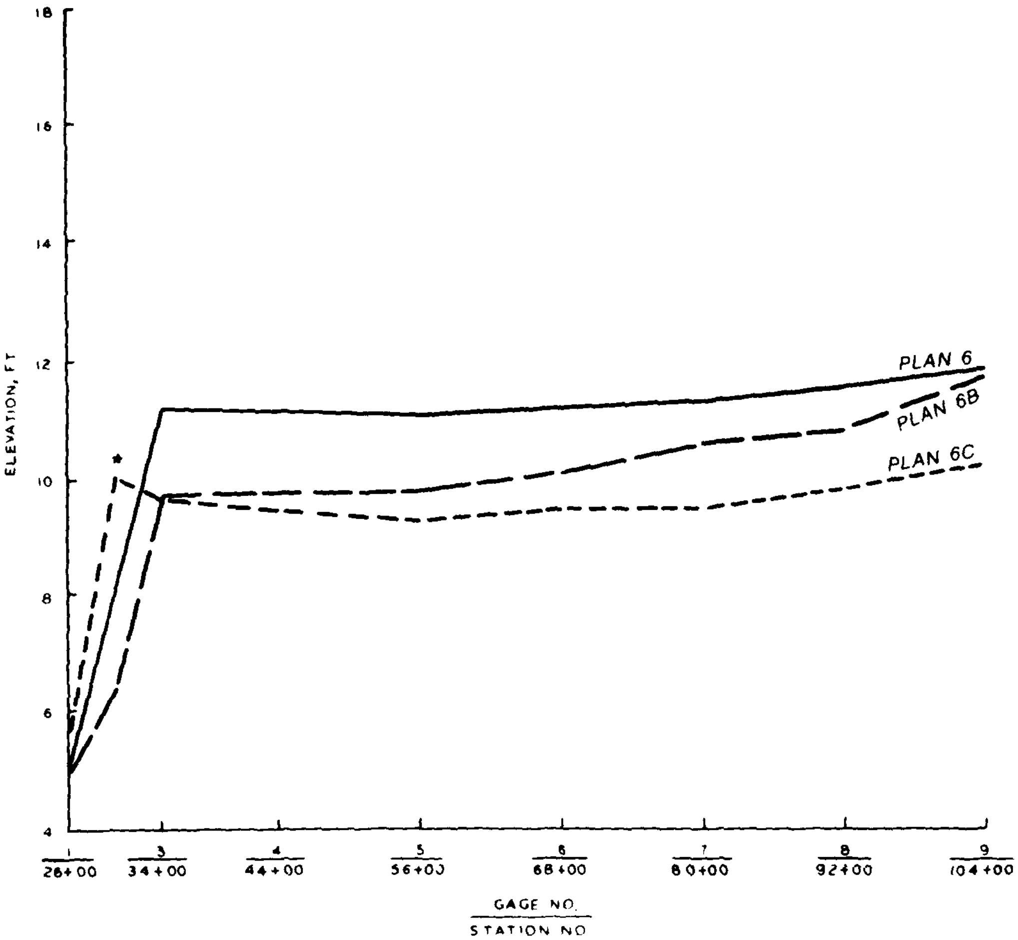


SAN DIEGO RIVER PROFILE
(WITH MODEL ROUGHNESS, SEALED MIDDLE JETTY)
PLANS 5-5G 97,000-CFS DISCHARGE



SAN DIEGO RIVER PROFILE
(WITH MODEL ROUGHNESS;
SEALED MIDDLE JETTY)
PLANS 6, 6B, AND 6C. 11,000-CFS DISCHARGE

***CREST ELEVATION OF PLUG**

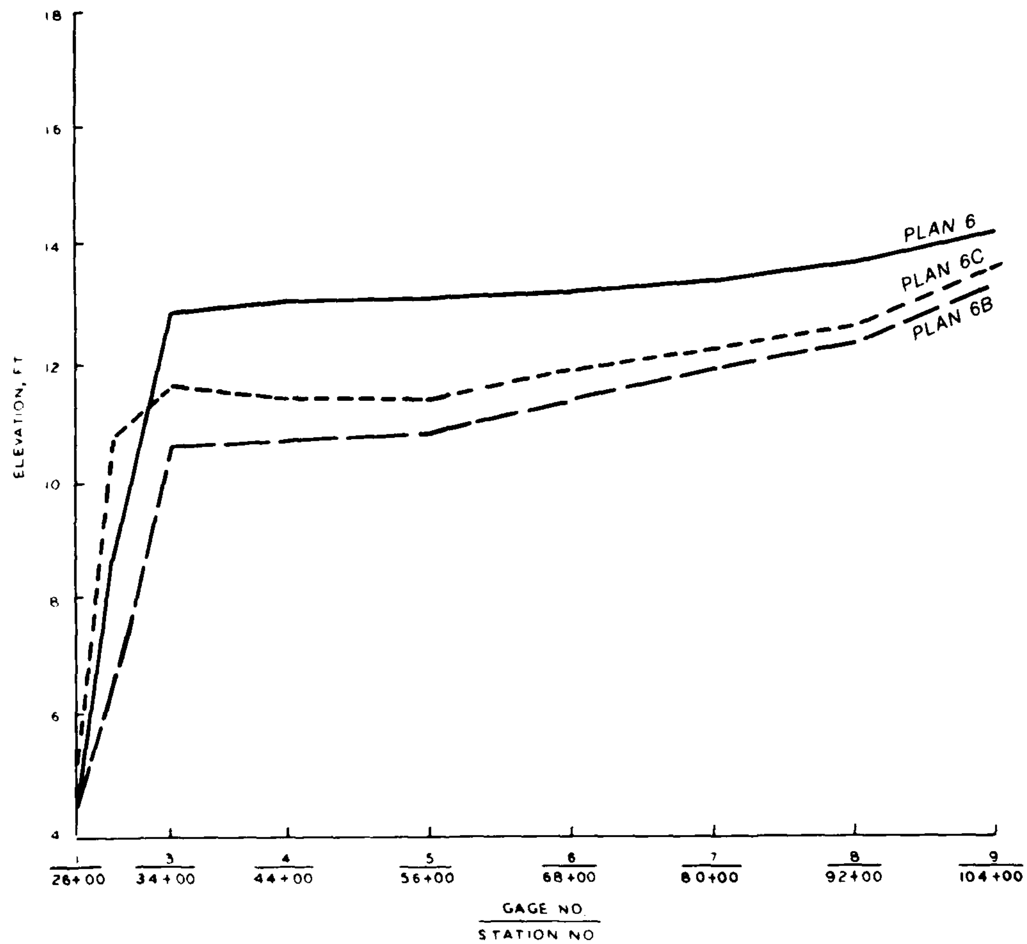


SAN DIEGO RIVER PROFILE

(WITH MODEL ROUGHNESS; SEALED MIDDLE JETTY)

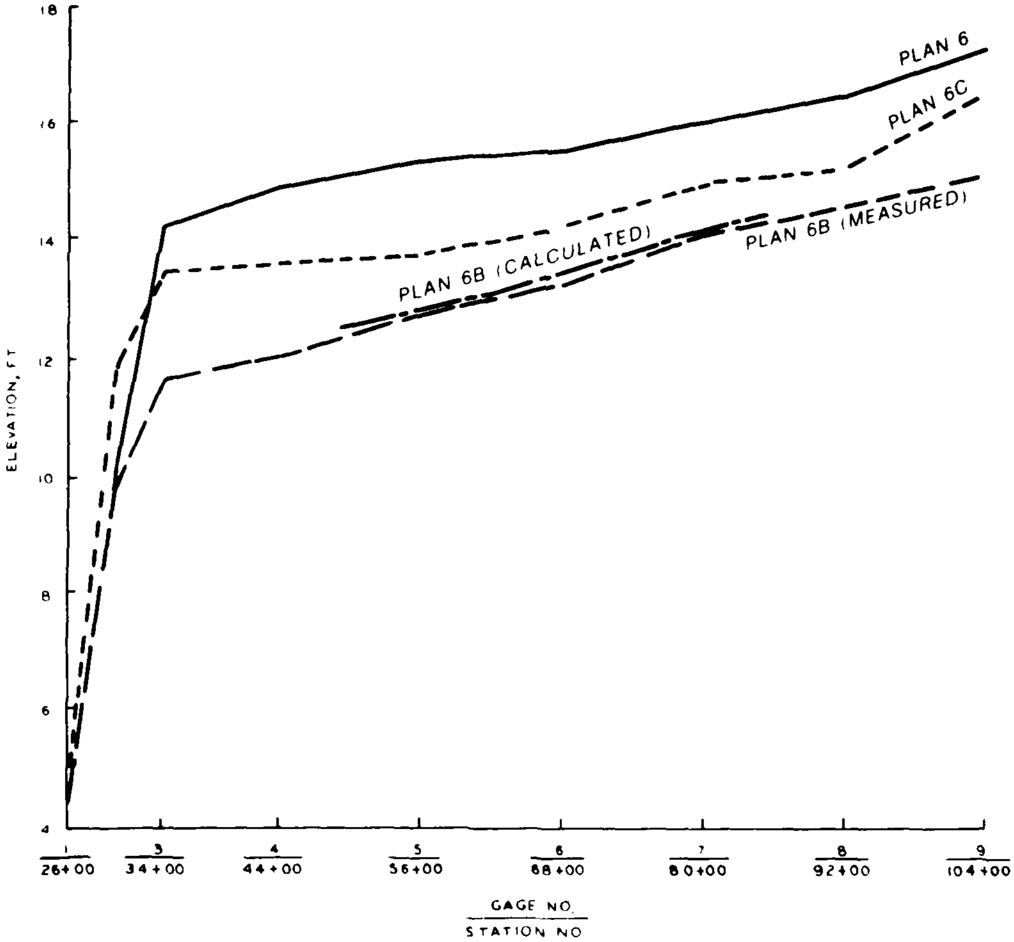
PLANS 6, 6B, AND 6C. 27,000-CFS DISCHARGE

*CREST ELEVATION OF PLUG

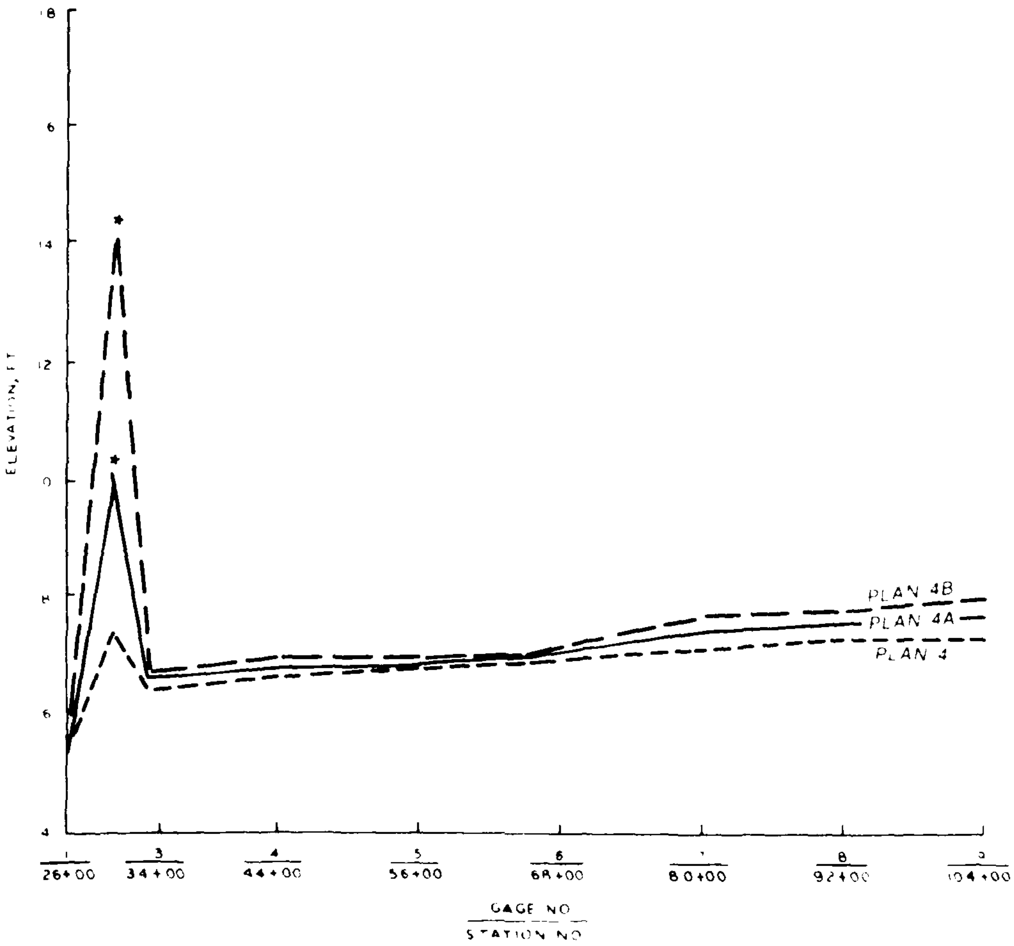


SAN DIEGO RIVER PROFILE
(WITH MODEL ROUGHNESS;
SEALED MIDDLE JETTY)
PLANS 6, 6B, AND 6C, 49,000-CFS DISCHARGE

PLATE 58

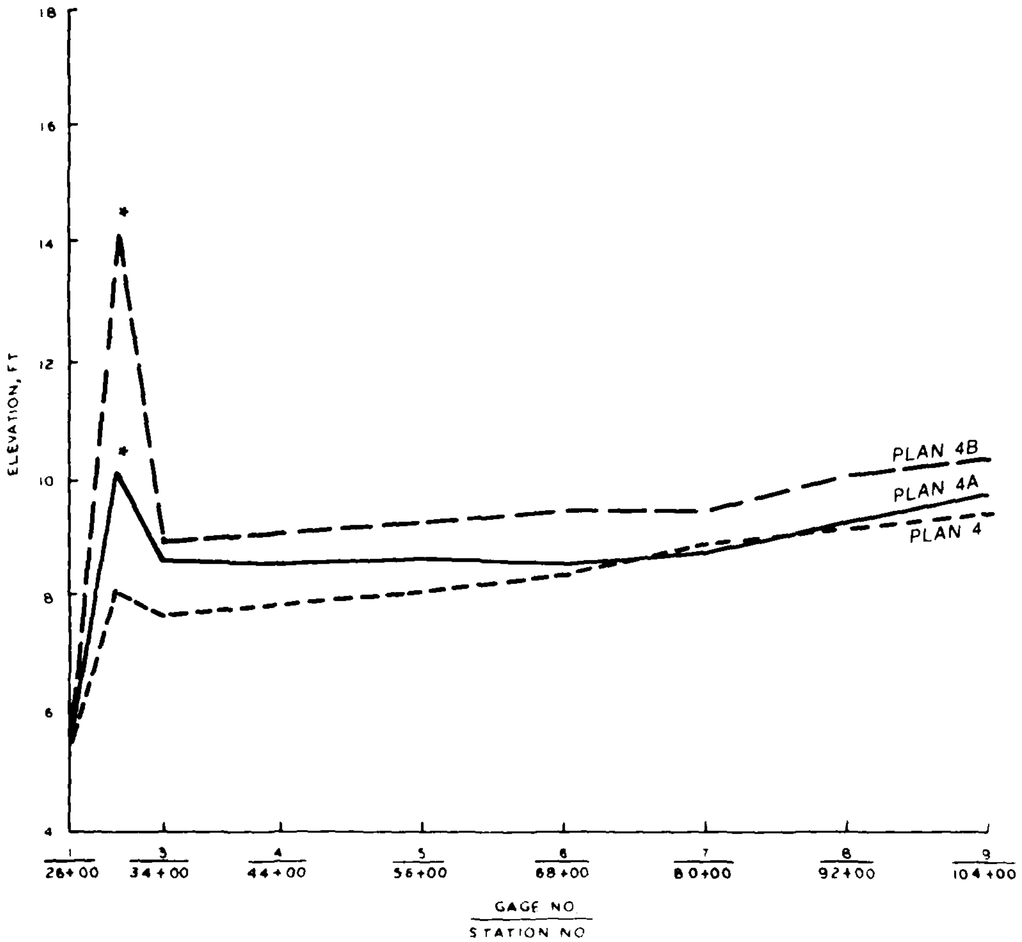


SAN DIEGO RIVER PROFILE
(WITH MODEL ROUGHNESS,
SEALED MIDDLE JETTY)
PLANS 6, 6B, AND 6C, 97,000-CFS DISCHARGE



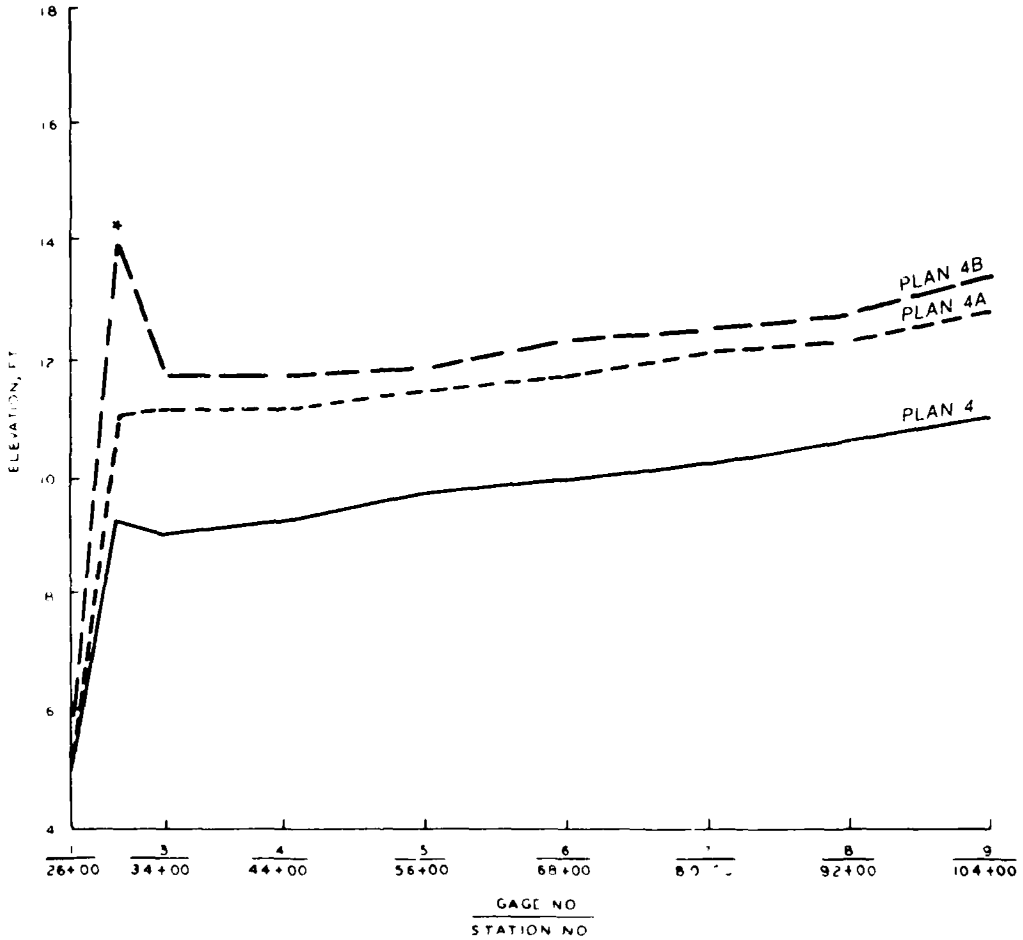
*CREST ELEVATION OF PLUG

SAN DIEGO RIVER PROFILE
(WITH MODEL ROUGHNESS
UNSEALED MIDDLE JETTY)
PLANS 4-4B 11,000 CFS DISCHARGE



***CREST ELEVATION OF PLUG**

SAN DIEGO RIVER PROFILE
(WITH MODEL ROUGHNESS, UNSEALED MIDDLE JETTY)
PLANS 4-4B 27,000-CFS DISCHARGE



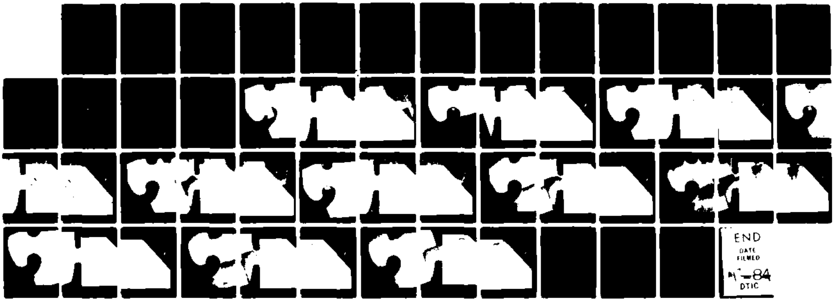
*CREST ELEVATION OF PLUG

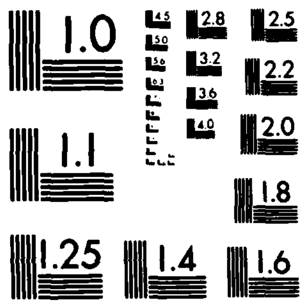
SAN DIEGO RIVER PROFILE
(WITH MODEL ROUGHNESS, UNSEALED MIDDLE JETTY)
PLANS 4-4B 49 000-CFS DISCHARGE

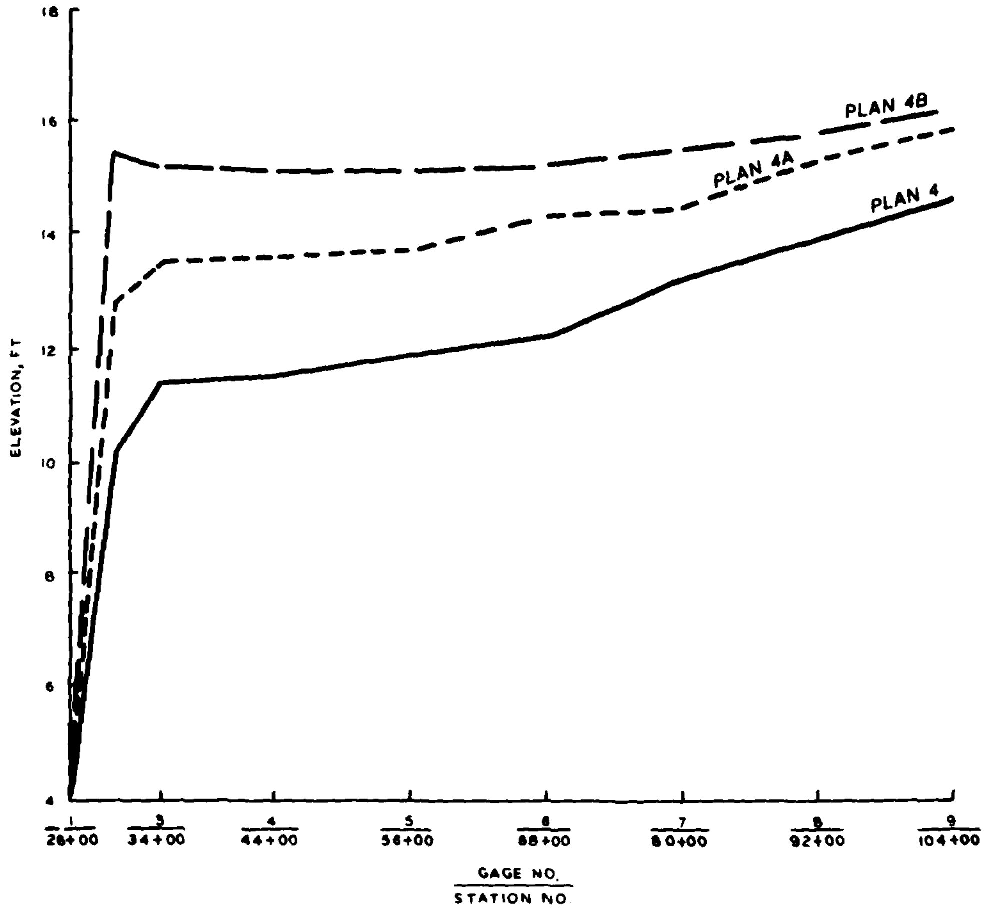
PLATE 62

AD-A136 371 MISSION BAY HARBOR CALIFORNIA DESIGN FOR WAVE AND SURGE PROTECTION AND FL..(U) ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MS HYDRA.. C R CURREN 4/4

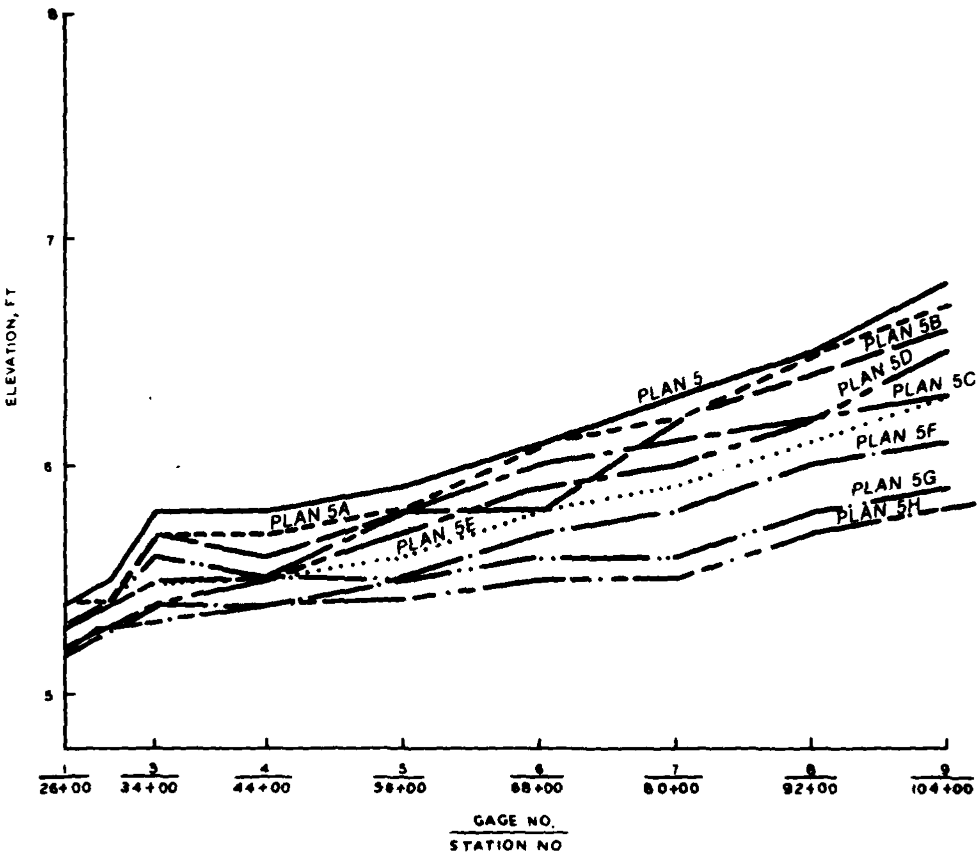
UNCLASSIFIED JUN 83 WES/TR/HL-83-17 F/G 13/2 NL



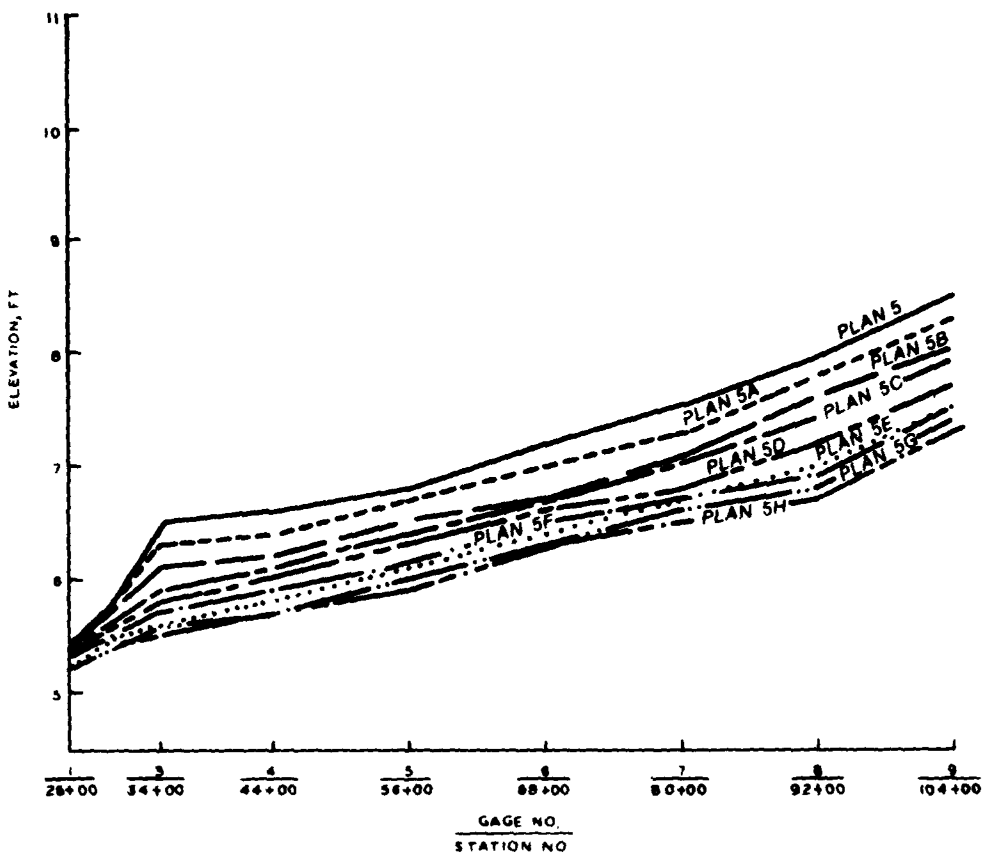

1.0
1.1
1.25
1.4
1.6
1.8
2.0
2.2
2.5
2.8
3.2
3.6
4.0



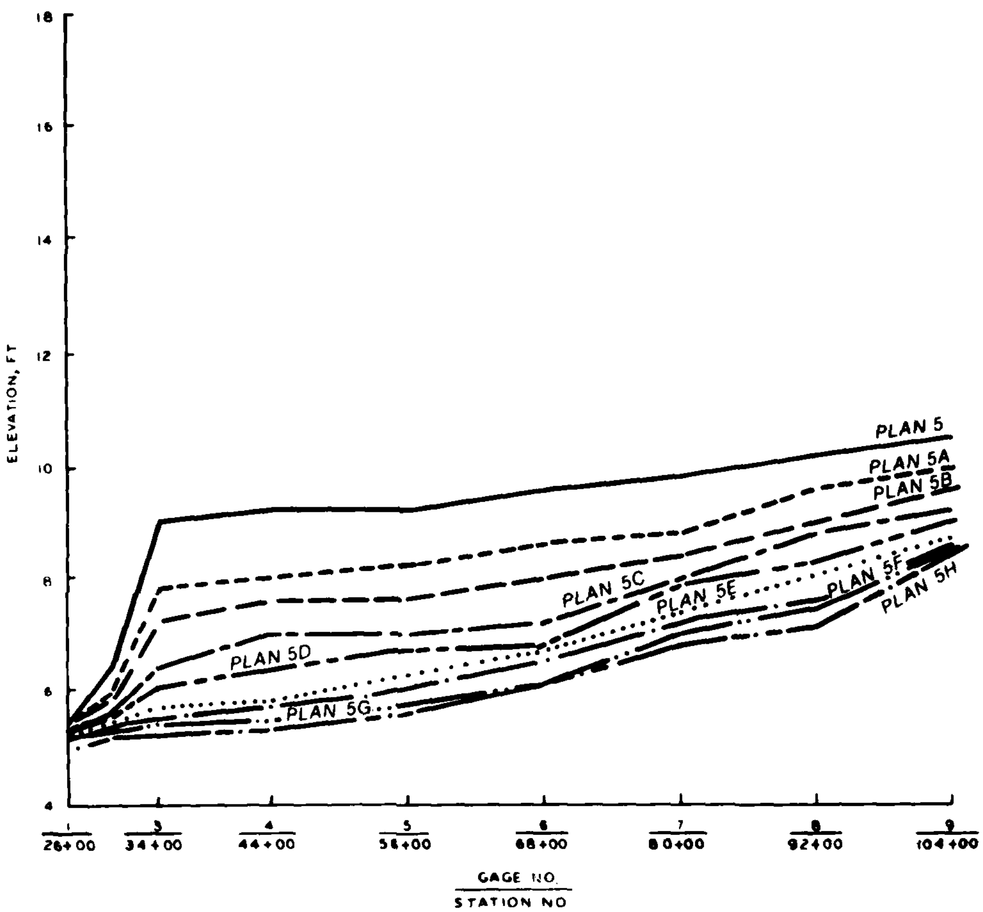
SAN DIEGO RIVER PROFILE
(WITH MODEL ROUGHNESS;
UNSEALED MIDDLE JETTY)
PLANS 4-4B. 97,000-CFS DISCHARGE



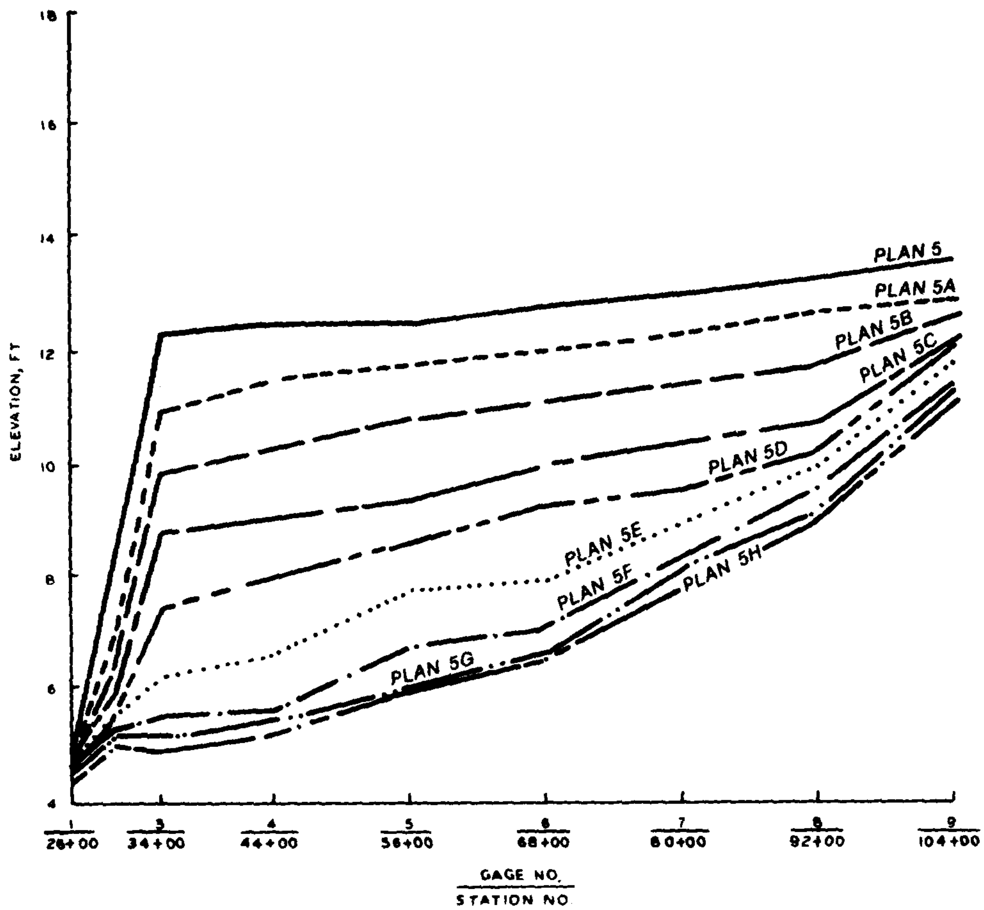
SAN DIEGO RIVER PROFILE
(WITH MODEL ROUGHNESS;
UNSEALED MIDDLE JETTY)
PLANS 5-5H; 11,000-CFS DISCHARGE



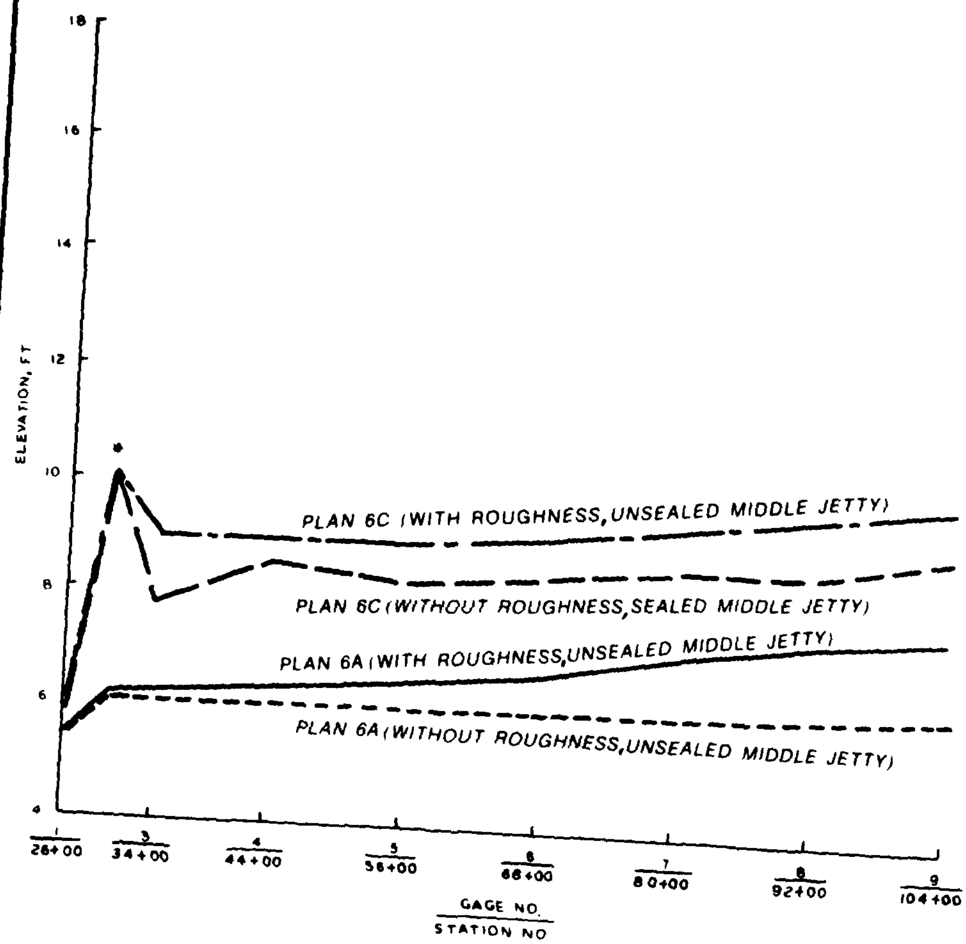
SAN DIEGO RIVER PROFILE
(WITH MODEL ROUGHNESS;
UNSEALED MIDDLE JETTY)
PLANS 5-5H; 27,000-CFS DISCHARGE



SAN DIEGO RIVER PROFILE
(WITH MODEL ROUGHNESS; UNSEALED MIDDLE JETTY)
PLANS 5-5H, 49,000-CFS DISCHARGE

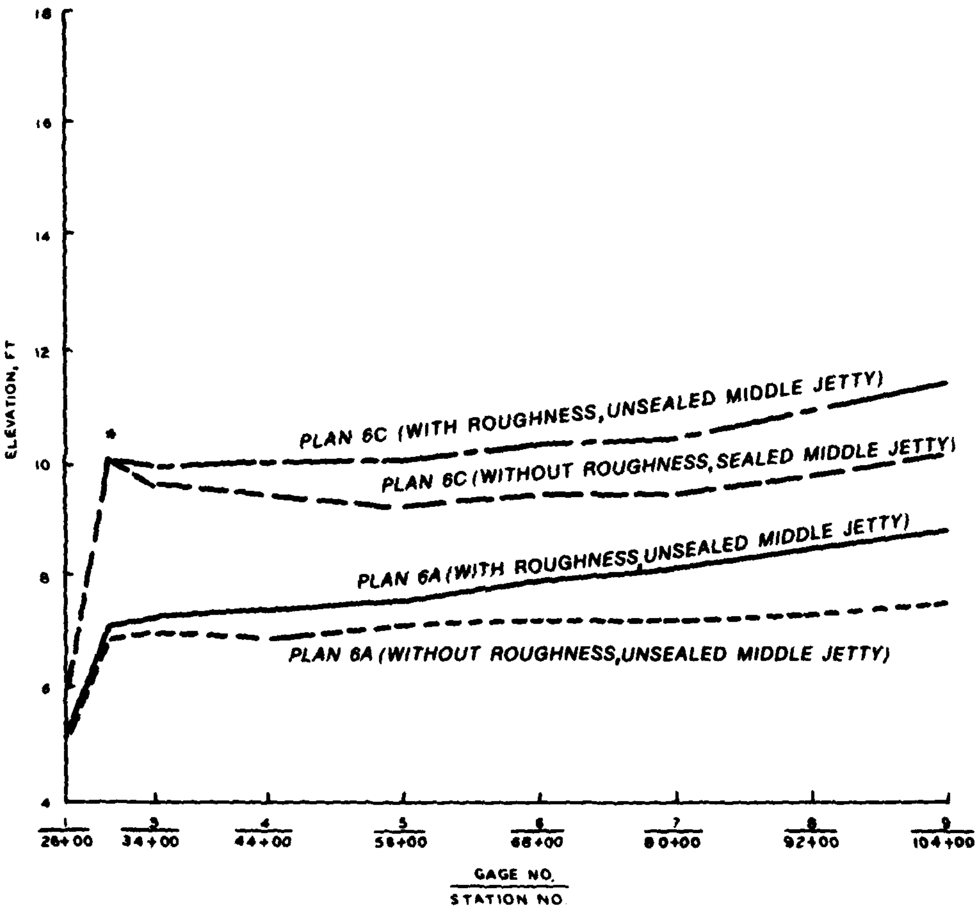


SAN DIEGO RIVER PROFILE
(WITH MODEL ROUGHNESS; UNSEALED MIDDLE JETTY)
PLANS 5-5H: 97,000-CFS DISCHARGE



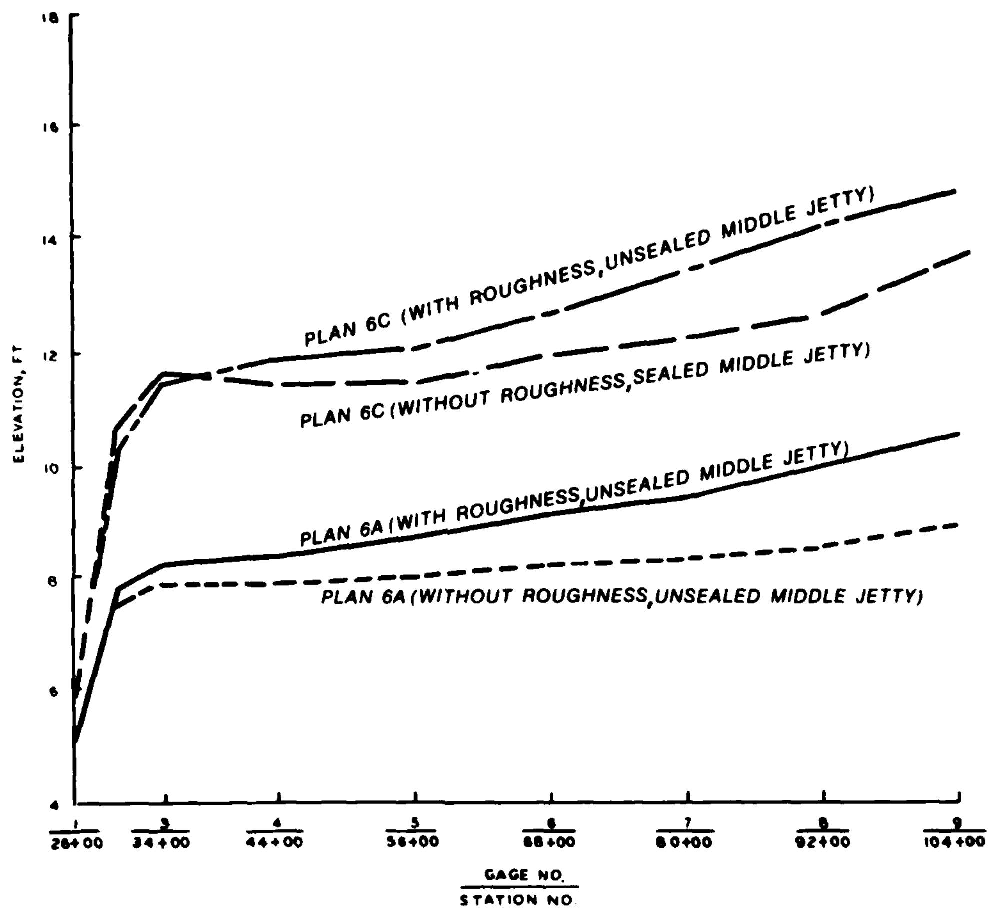
*CREST ELEVATION OF PLUG

SAN DIEGO RIVER PROFILE
PLANS 6A - 6C
11,000-CFS DISCHARGE

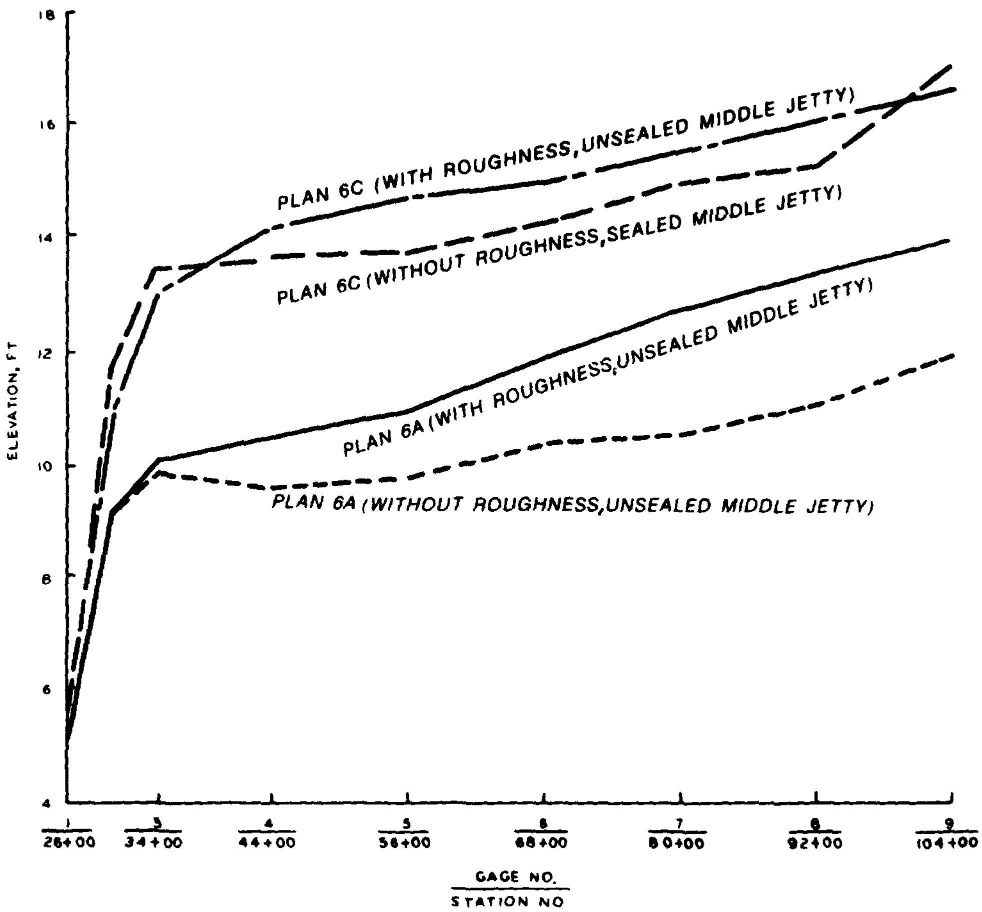


*CREST ELEVATION OF PLUG

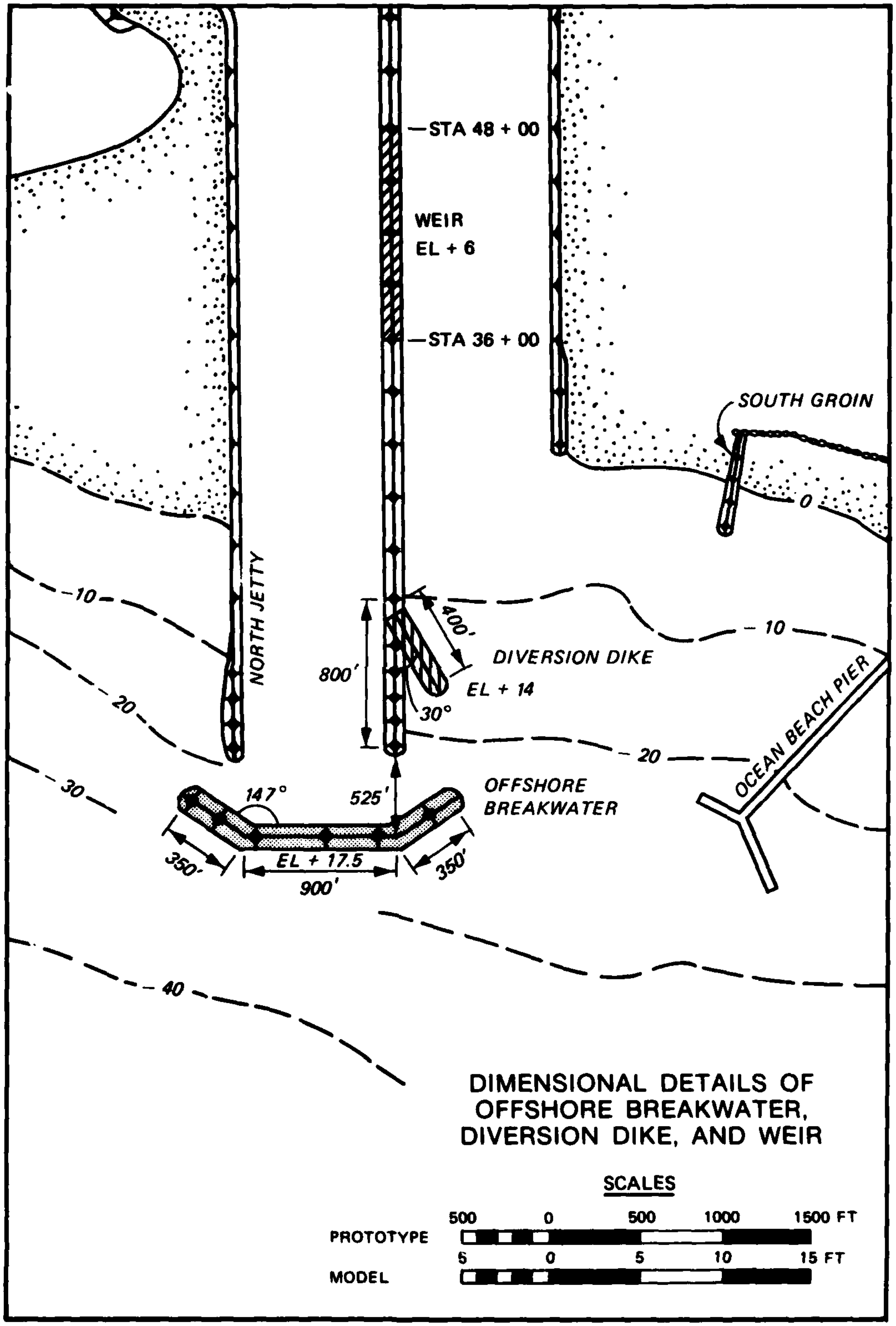
SAN DIEGO RIVER PROFILE
PLANS 6A - 6C
27,000-CFS DISCHARGE



SAN DIEGO RIVER PROFILE
PLANS 6A - 6C
49,000-CFS DISCHARGE



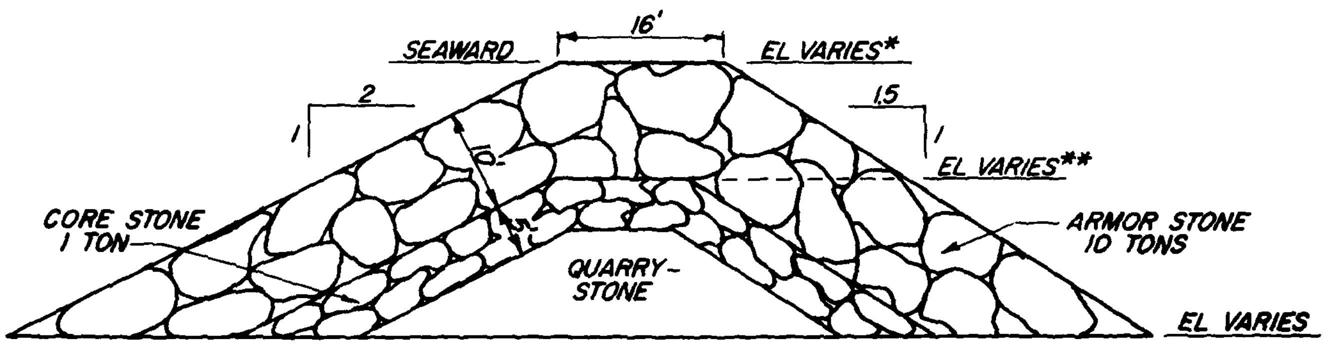
SAN DIEGO RIVER PROFILE
PLANS 6A - 6C
97,000-CFS DISCHARGE



DIMENSIONAL DETAILS OF OFFSHORE BREAKWATER, DIVERSION DIKE, AND WEIR

APPENDIX A

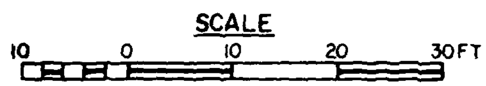
TYPICAL SECTIONS OF VARIOUS STRUCTURES TESTED IN THE MODEL



PLAN*	CROWN ELEVATION, FT
1-1F, 1M-1P, 2-2D	+22.5
1G	+20.5
1I-1L, 1N-1P, 2-2D, 3-3G	+17.5
1H	+15.0

PLAN**	CORE ELEVATION, FT
1-1F, 1M-1P, 2-2D	12.5
1G	10.0
1I-1L, 1N-1P, 2-2D, 3-3G	7.5
1H	5.0

NOTE: PLANS 3-3G WERE WITH IMPERVIOUS CORE TO +7.5' ELEVATION



TYPICAL OFFSHORE BREAKWATER SECTIONS
PLANS 1-3G

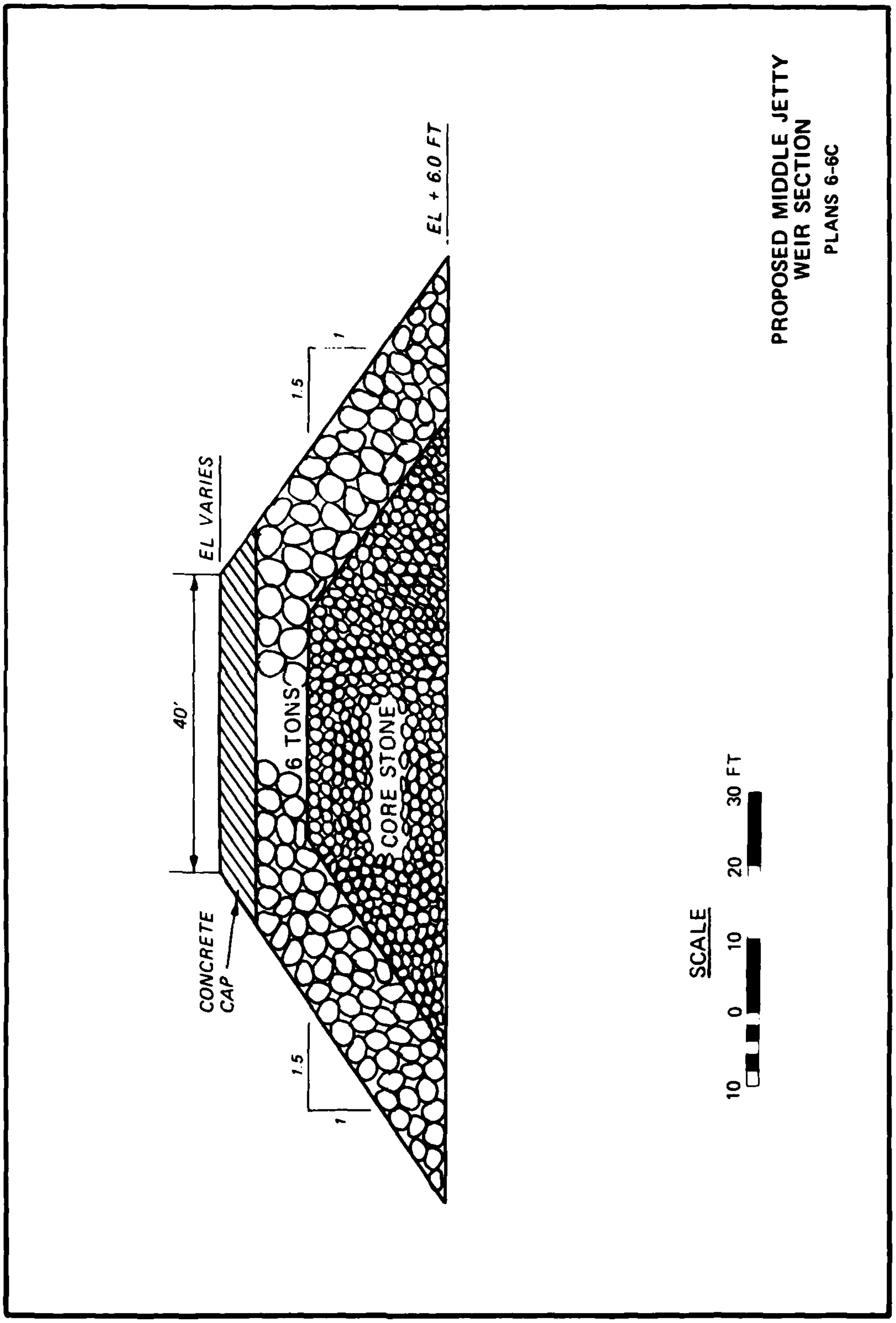
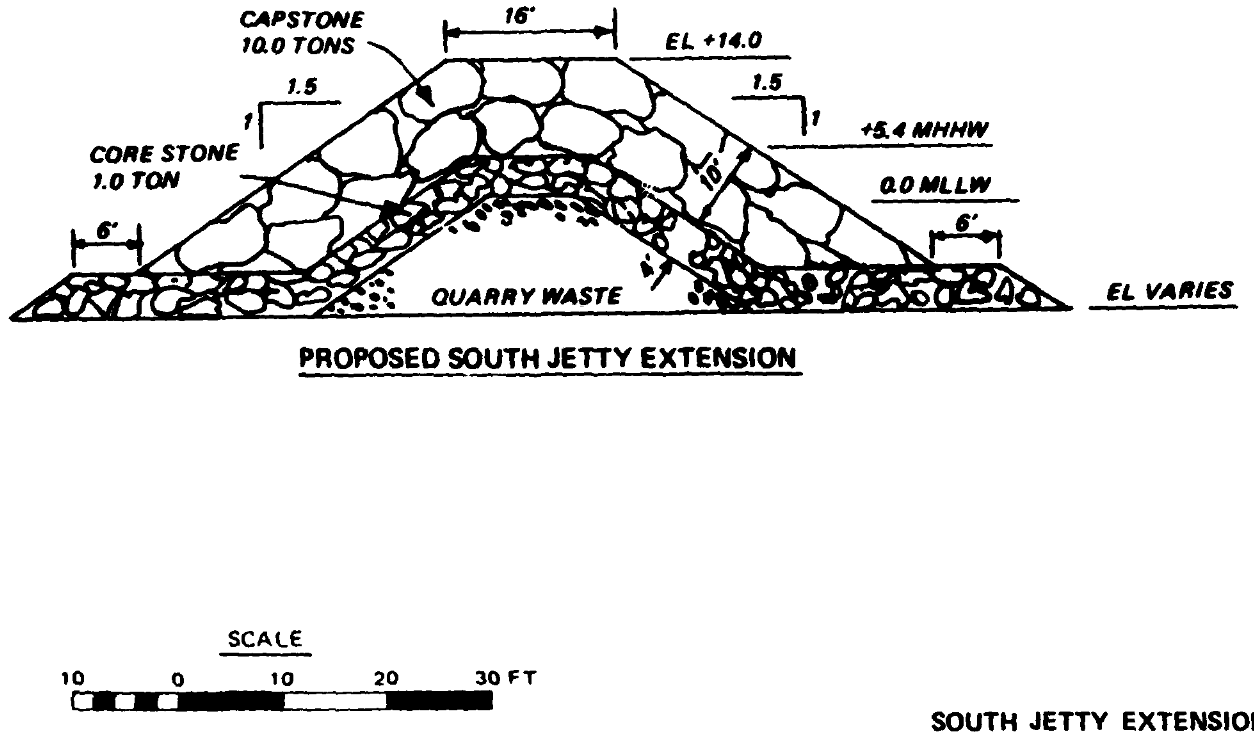

PROPOSED MIDDLE JETTY
WEIR SECTION
PLANS 6-6C
EL + 6.0 FT
EL VARIES
40'
1.5
1
1.5
1
CONCRETE CAP
6 TONS
CORE STONE
SCALE
10 0 10 20 30 FT

PLATE A2



PROPOSED SOUTH JETTY EXTENSION

SOUTH JETTY EXTENSION
AND DIVERSION DIKE SECTIONS
PLANS 7-8A

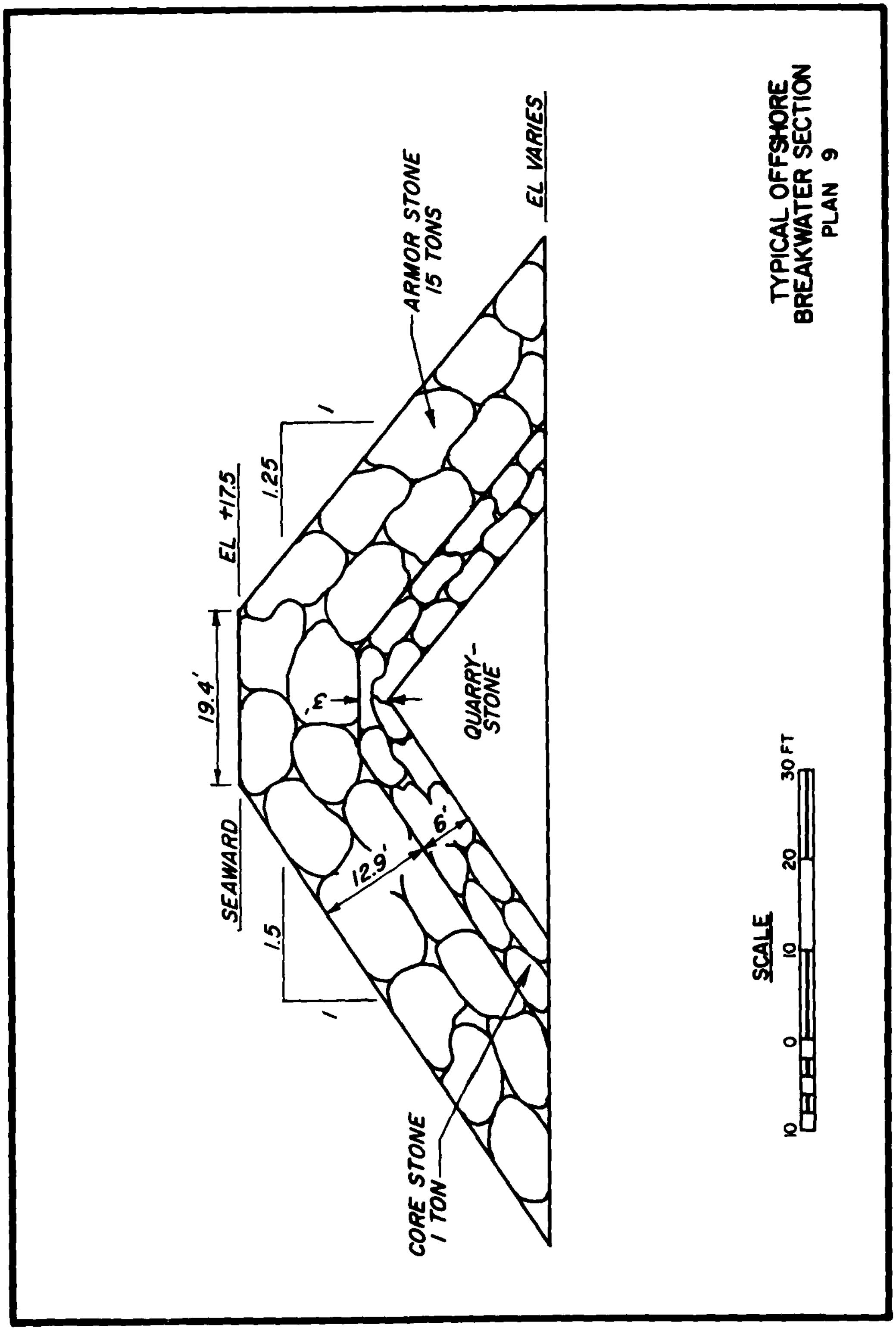

TYPICAL OFFSHORE
BREAKWATER SECTION
PLAN 9
ARMOR STONE
15 TONS
EL VARIES
EL +17.5
1.25
1
19.4'
3'
QUARRY-
STONE
SEAWARD
12.9'
6'
1.5
1
CORE STONE
1 TON
SCALE
10 0 10 20 30 FT

PLATE A4

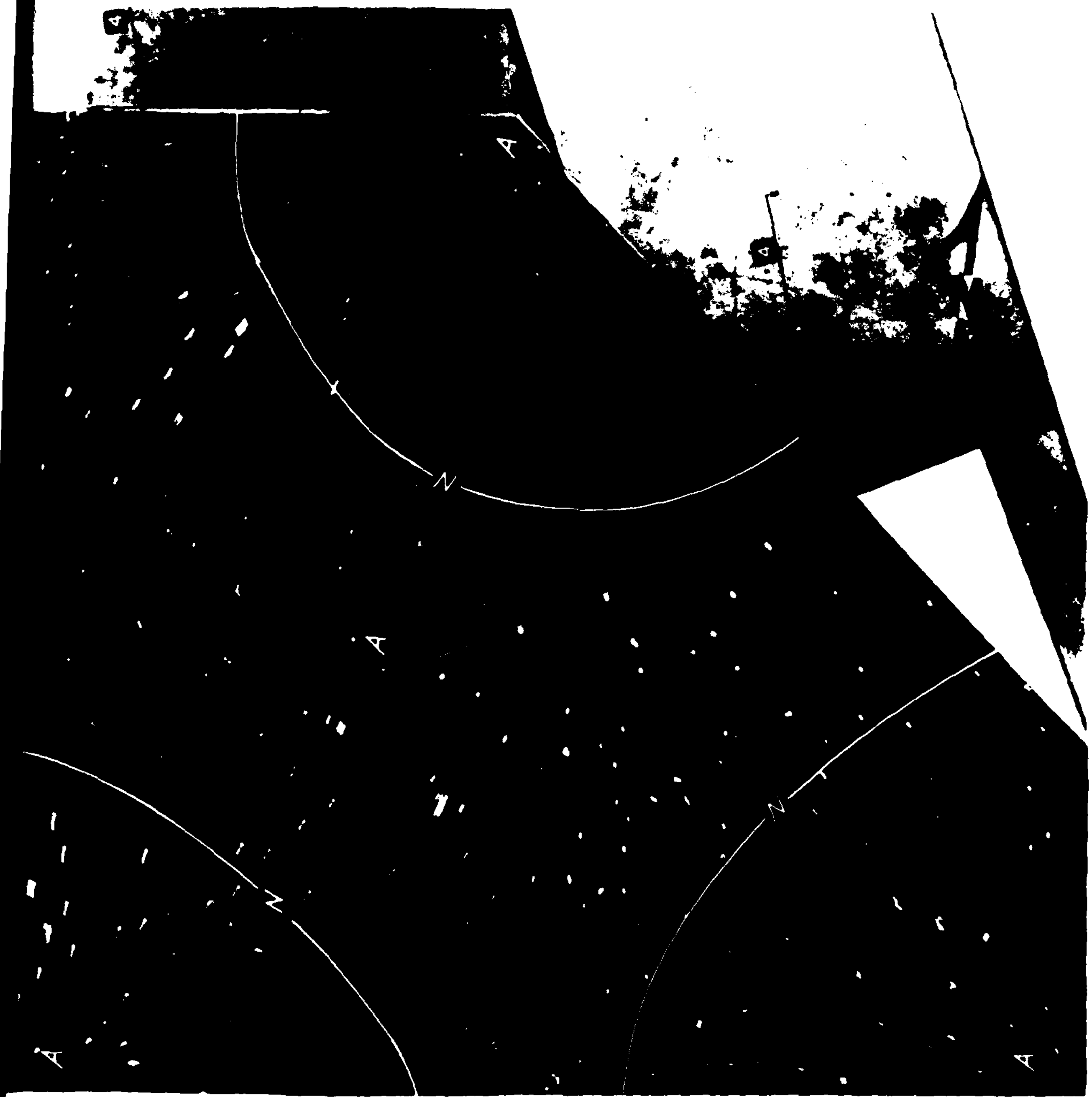
APPENDIX B

LONG-PERIOD WAVE MOSAICS

Photo B1. Harbor oscill

lations for existing conditions resulting from 68-sec waves

2



lting from 68-sec waves

3

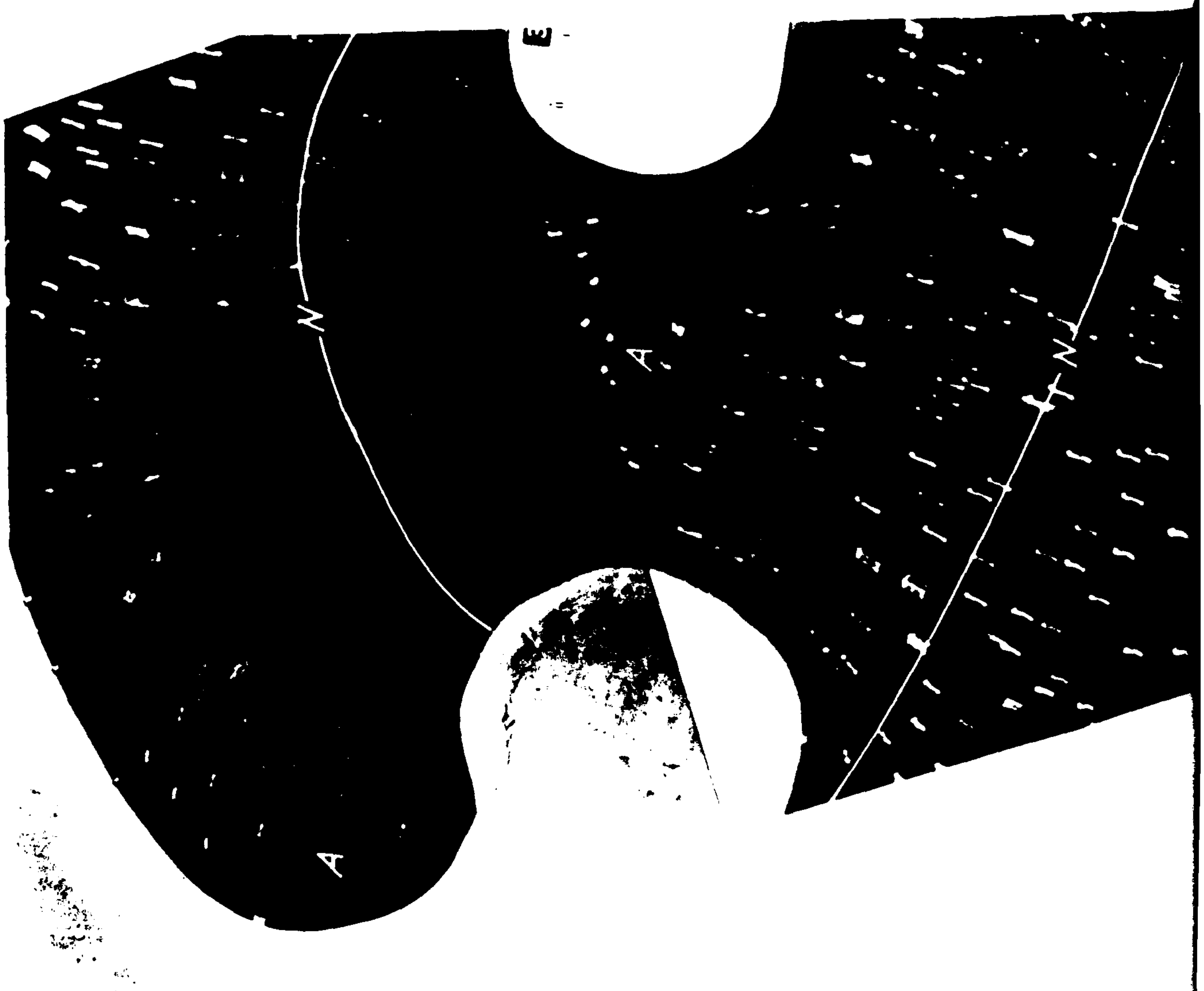


Photo B2. Harb

Harbor oscillations for existing conditions resulting from 76-sec waves

2

ulting from 76-sec waves

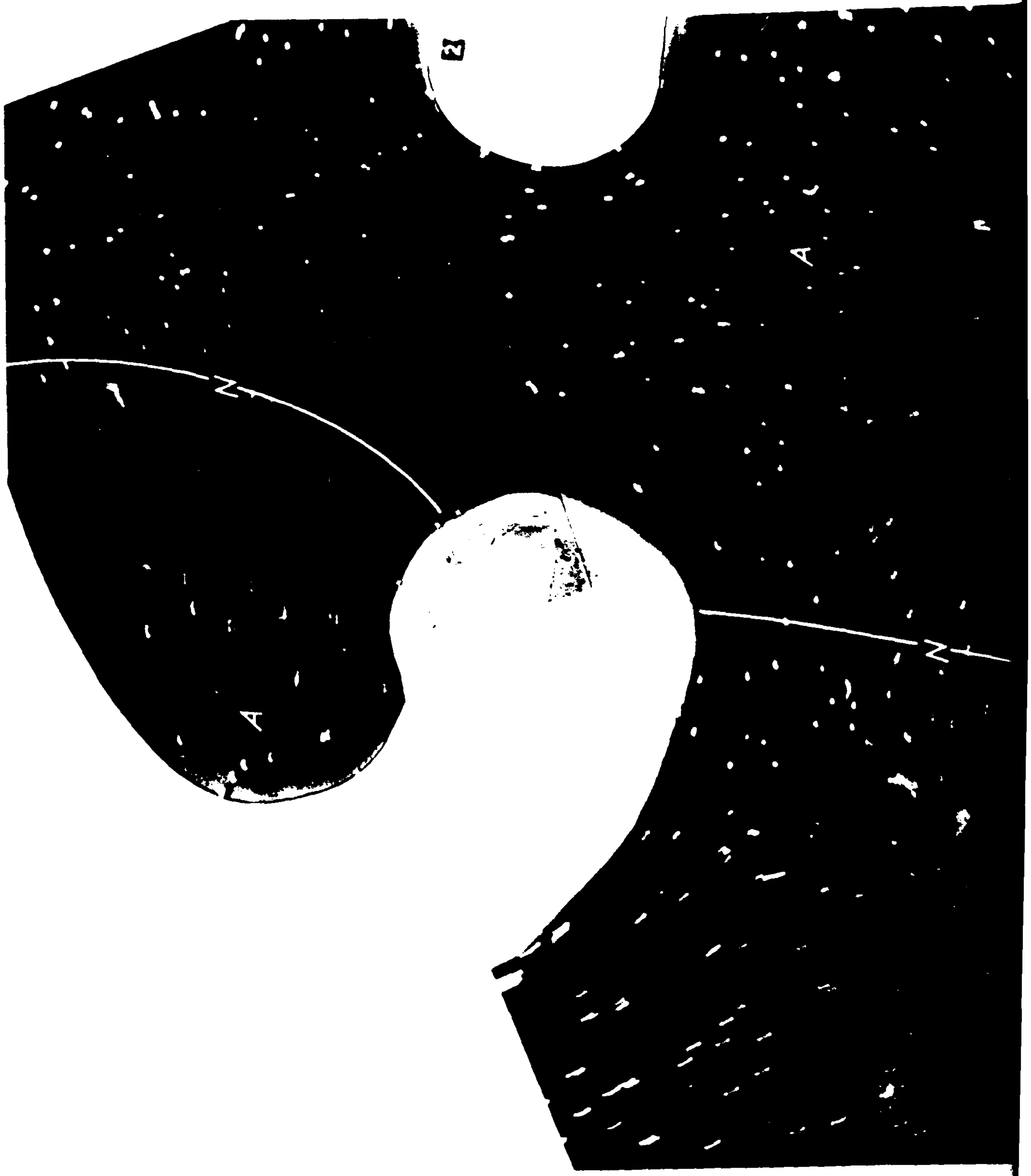
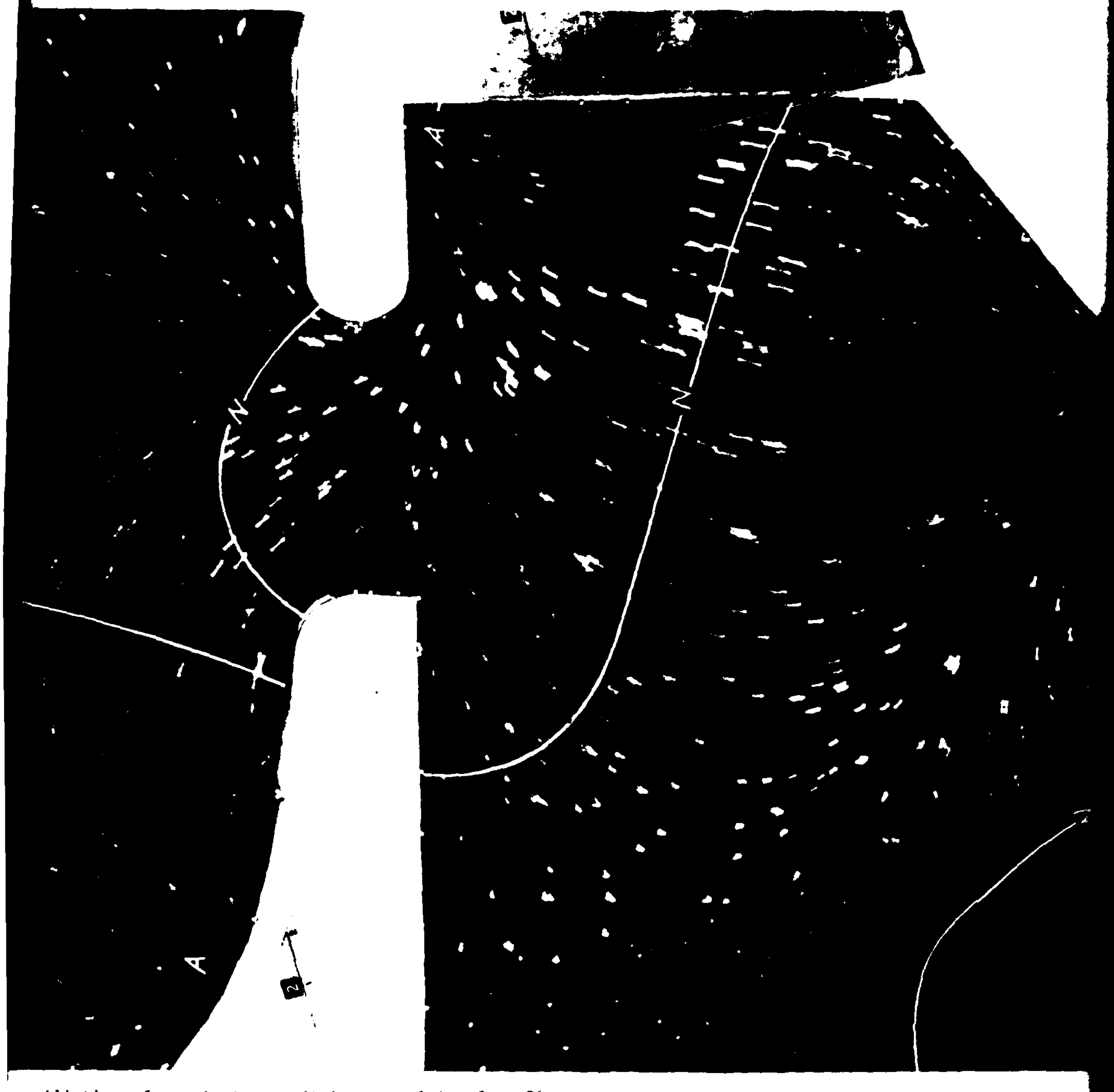


Photo B3. Harbor osc



oscillations for existing conditions resulting from 86-sec waves

2

lting from 86-sec waves

3

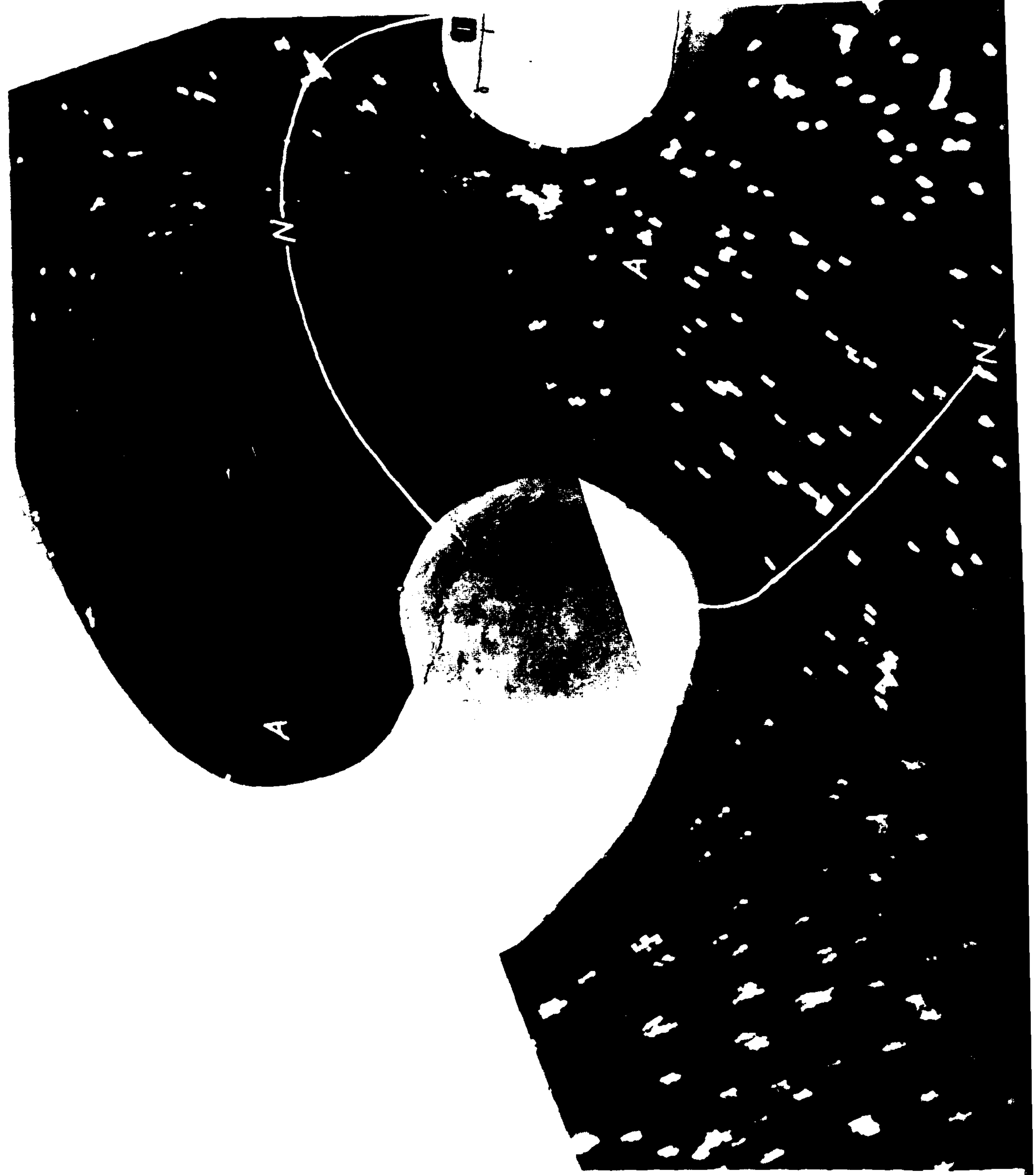
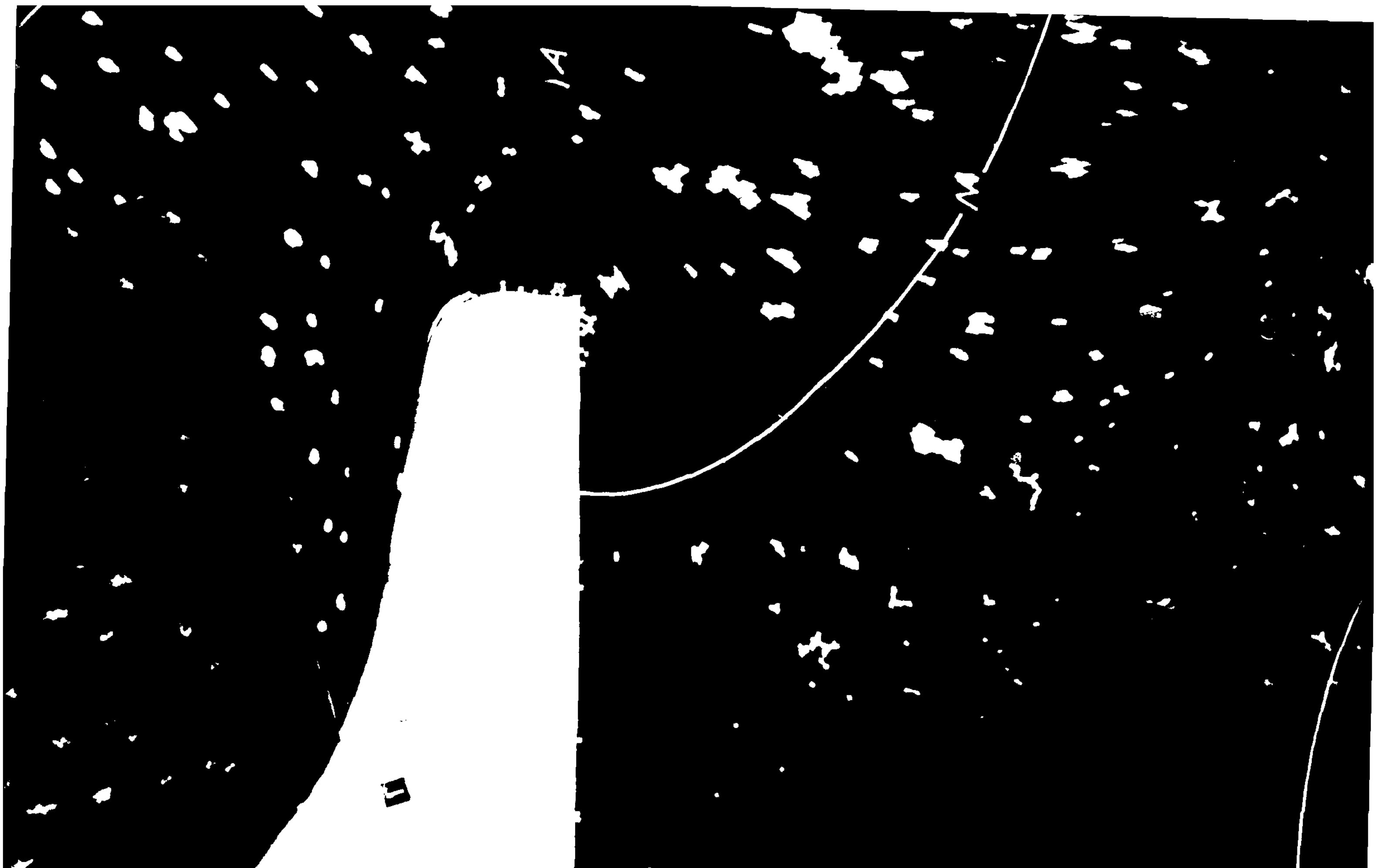
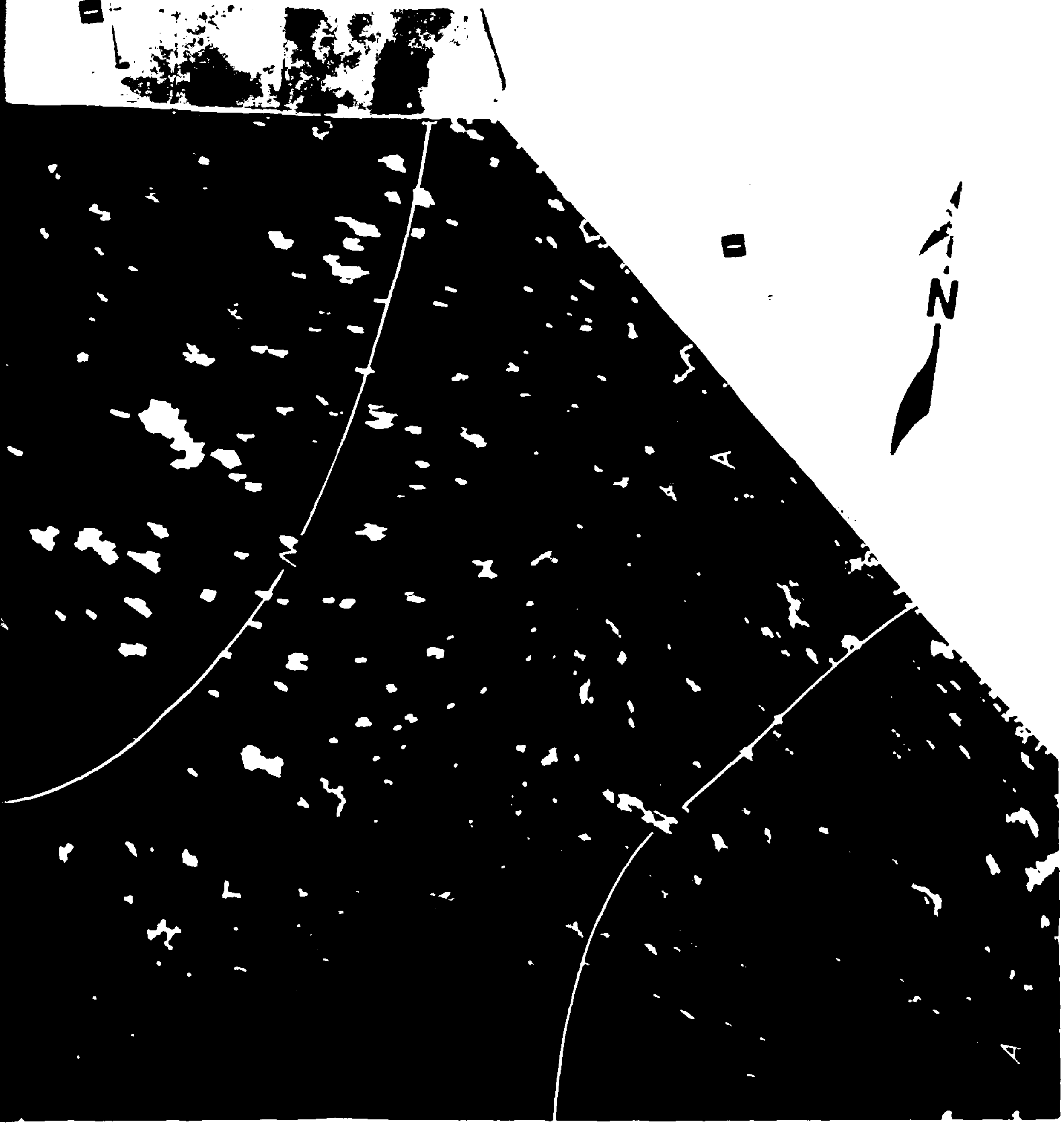


Photo B4. Harbor



Harbor oscillations for existing conditions resulting from 88-sec waves



ulting from 88-sec waves

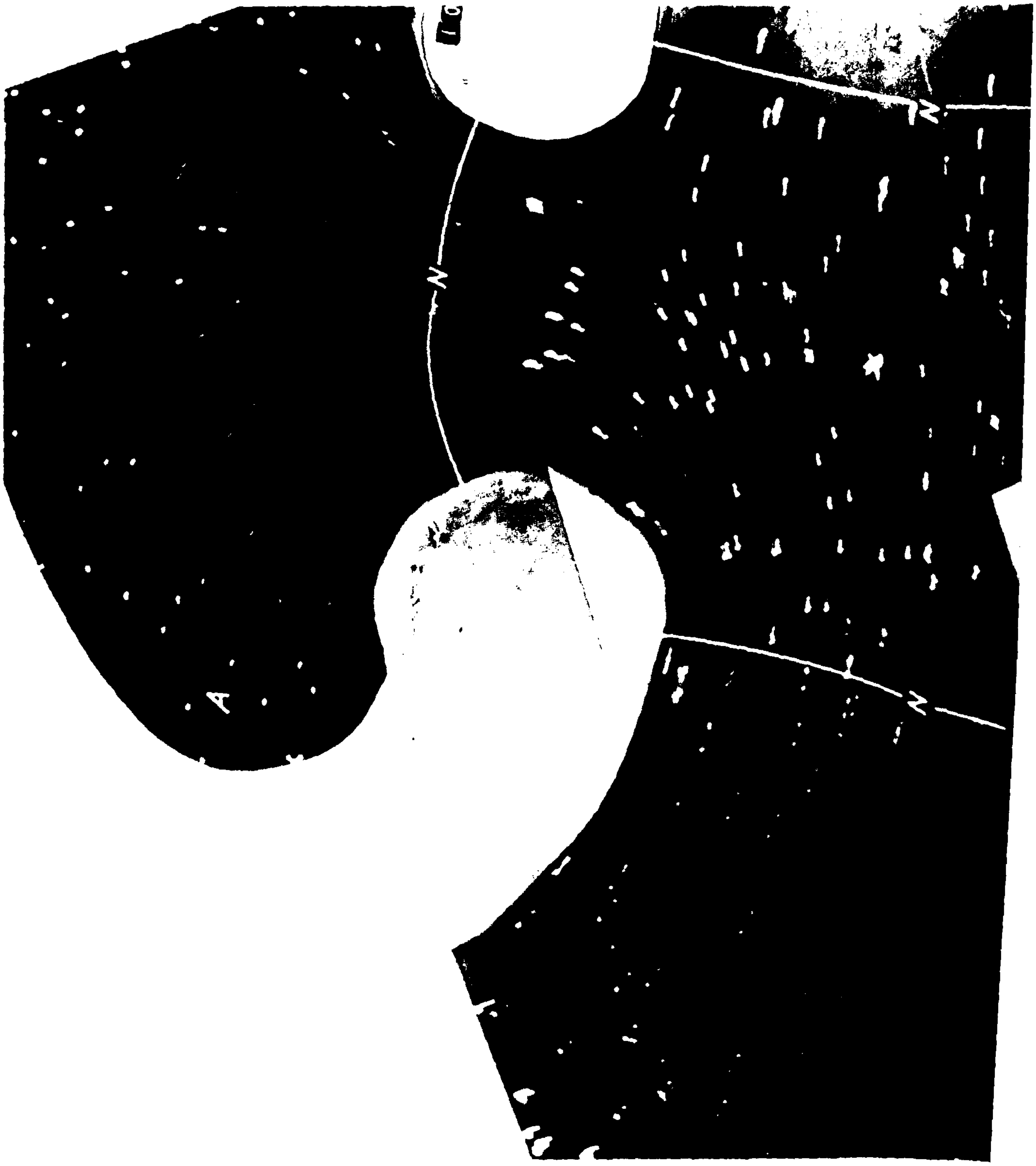
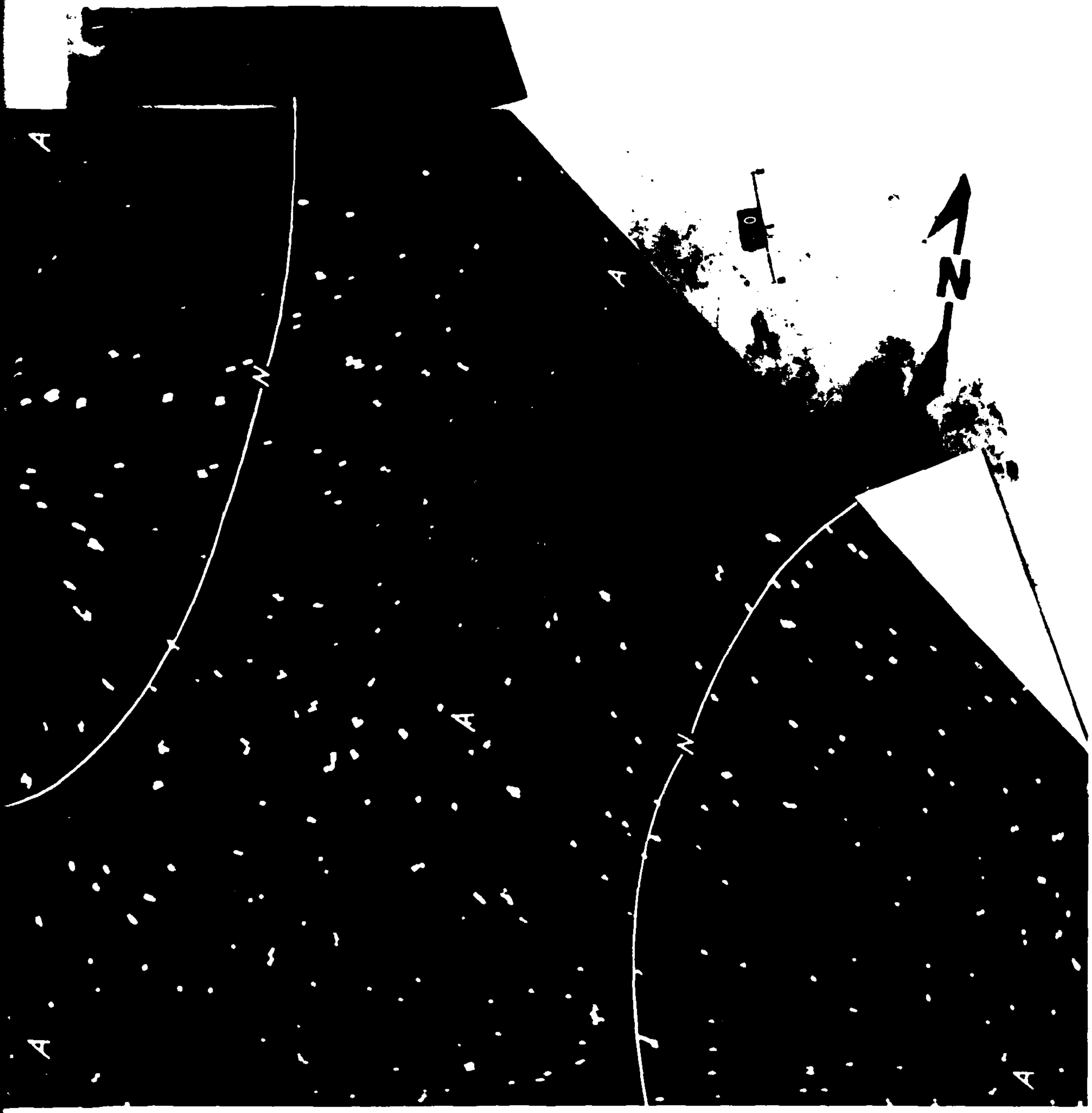


Photo B5. Harbor o

or oscillations for existing conditions resulting from 122-sec waves

2

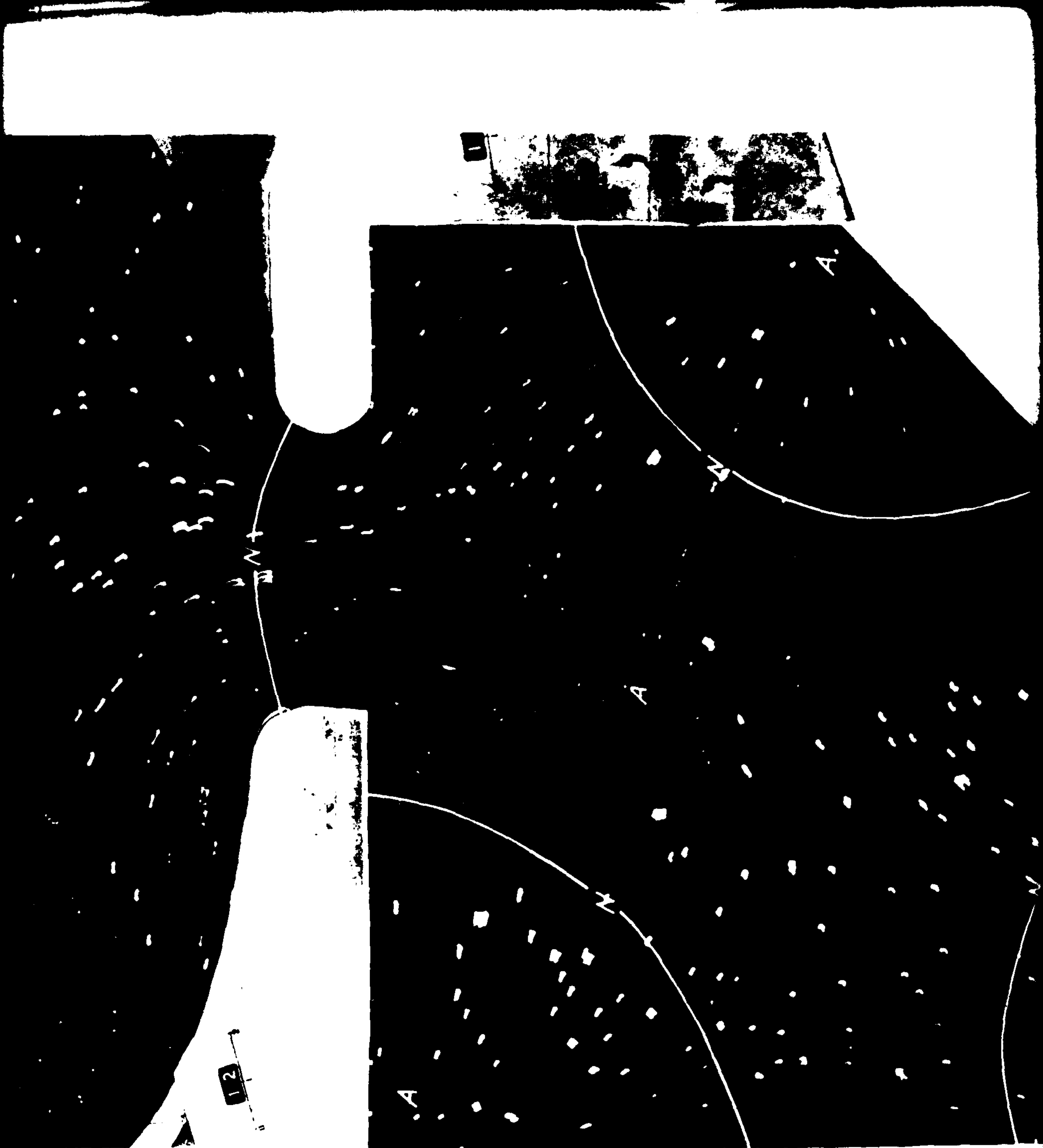


ılting from 122-sec waves

Photo B6. Harbor osc

lting from 68-sec waves

3



r oscillations for existing conditions resulting from 135-sec waves

2

ulting from 135-sec waves

3

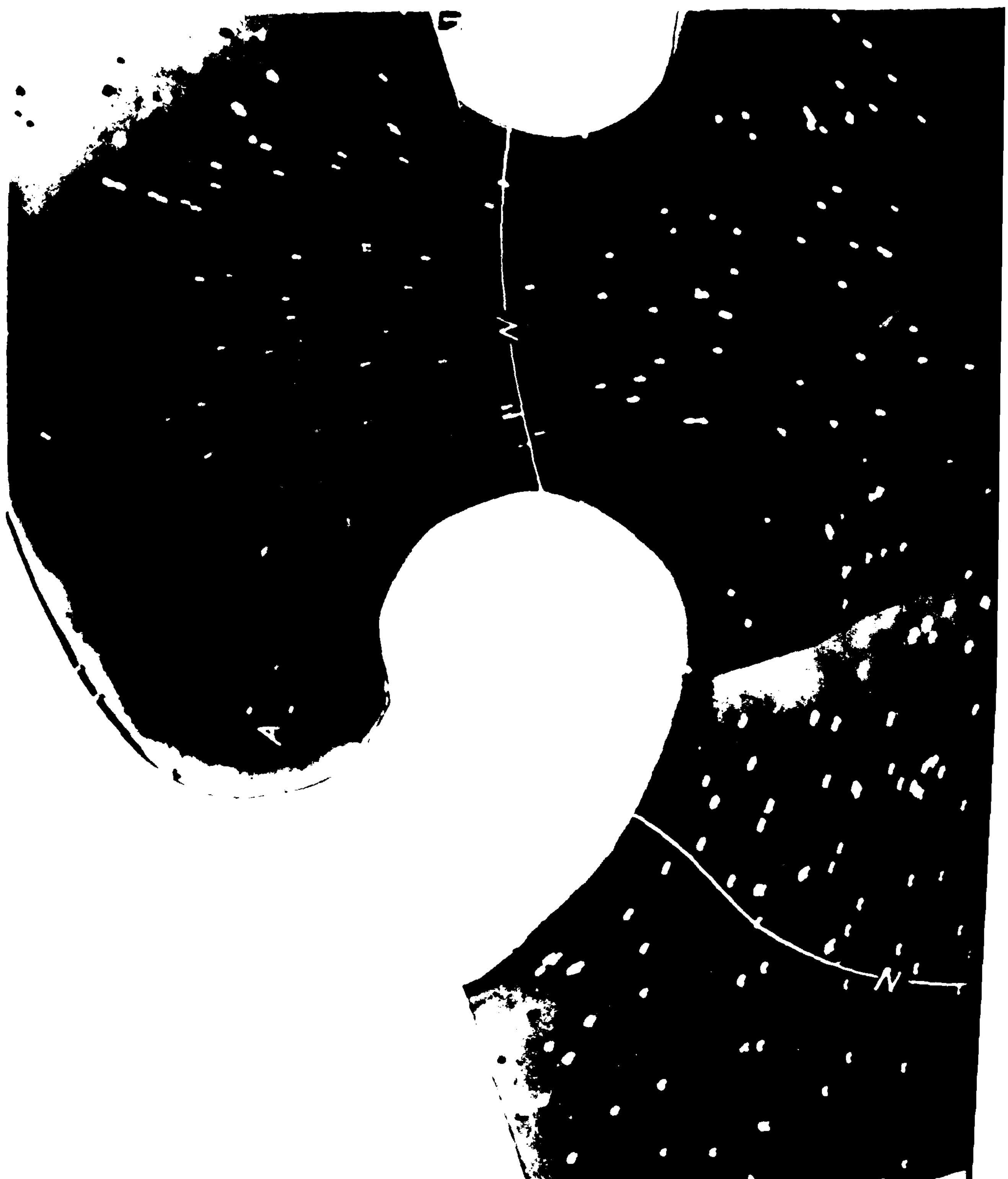
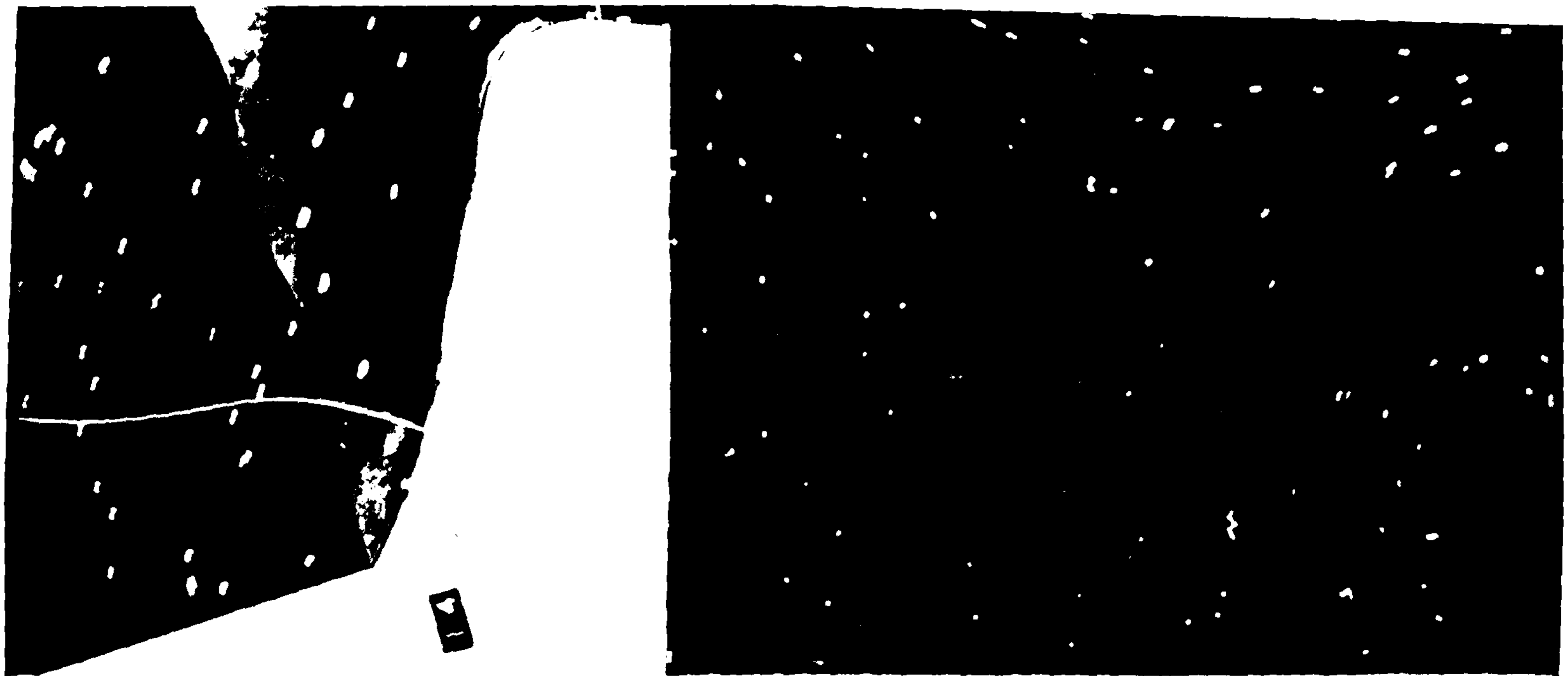
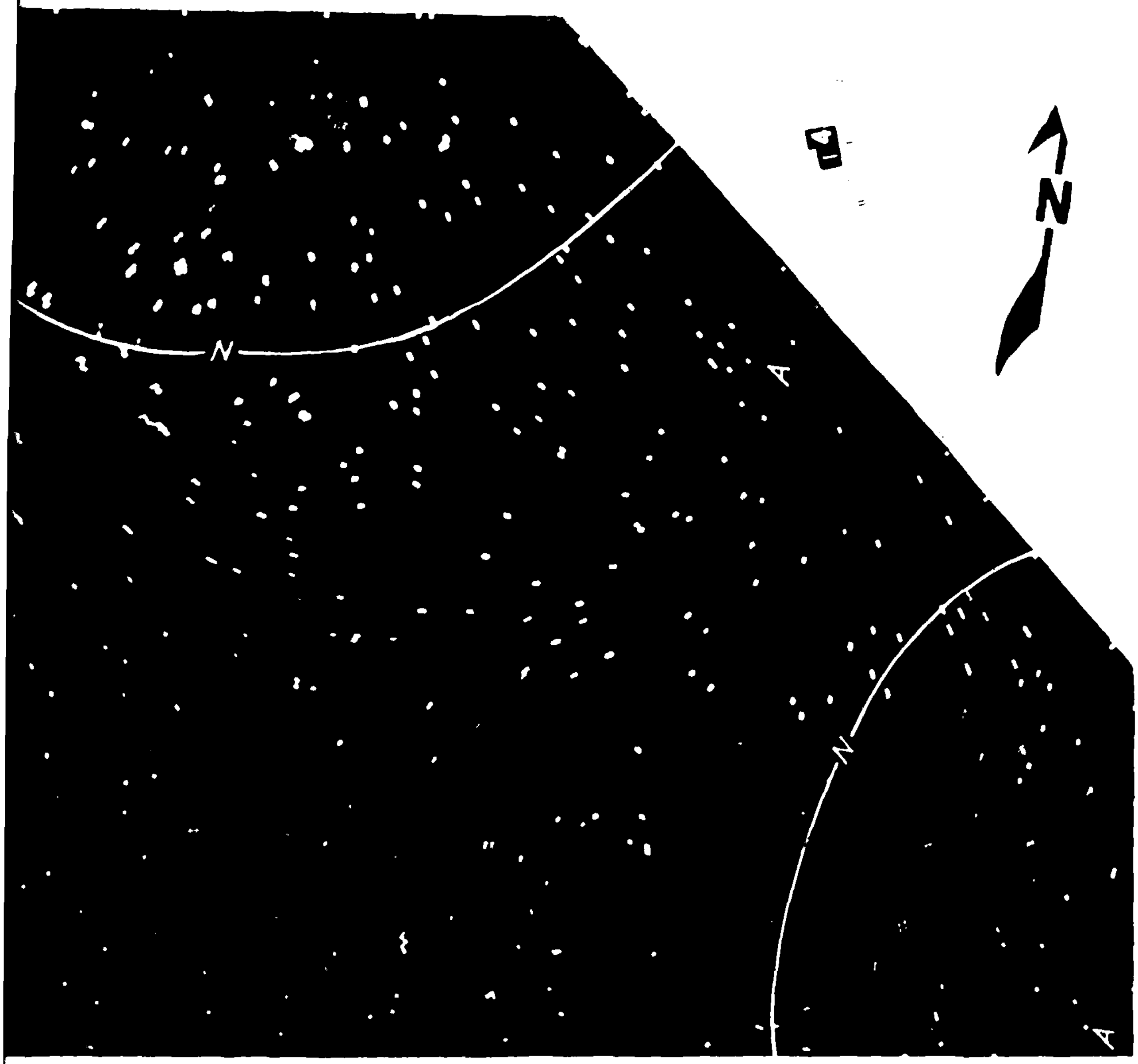


Photo B7. Ha



'. Harbor oscillations for Plan 3G resulting from 76-sec waves

2



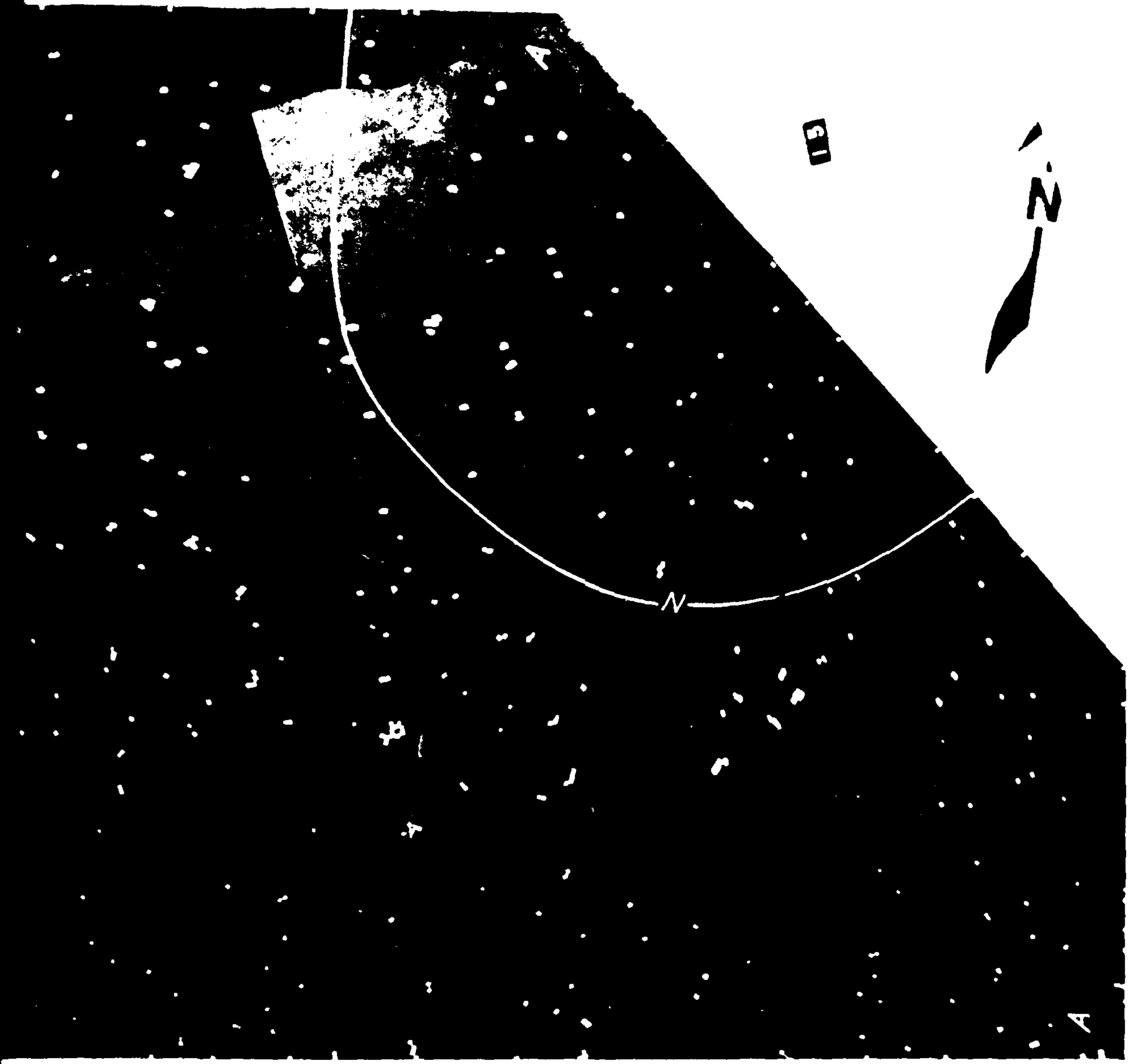
76-sec waves

Photo B8. Harb

3

Harbor oscillations for Plan 3G resulting from 95-sec waves

2



95-sec waves

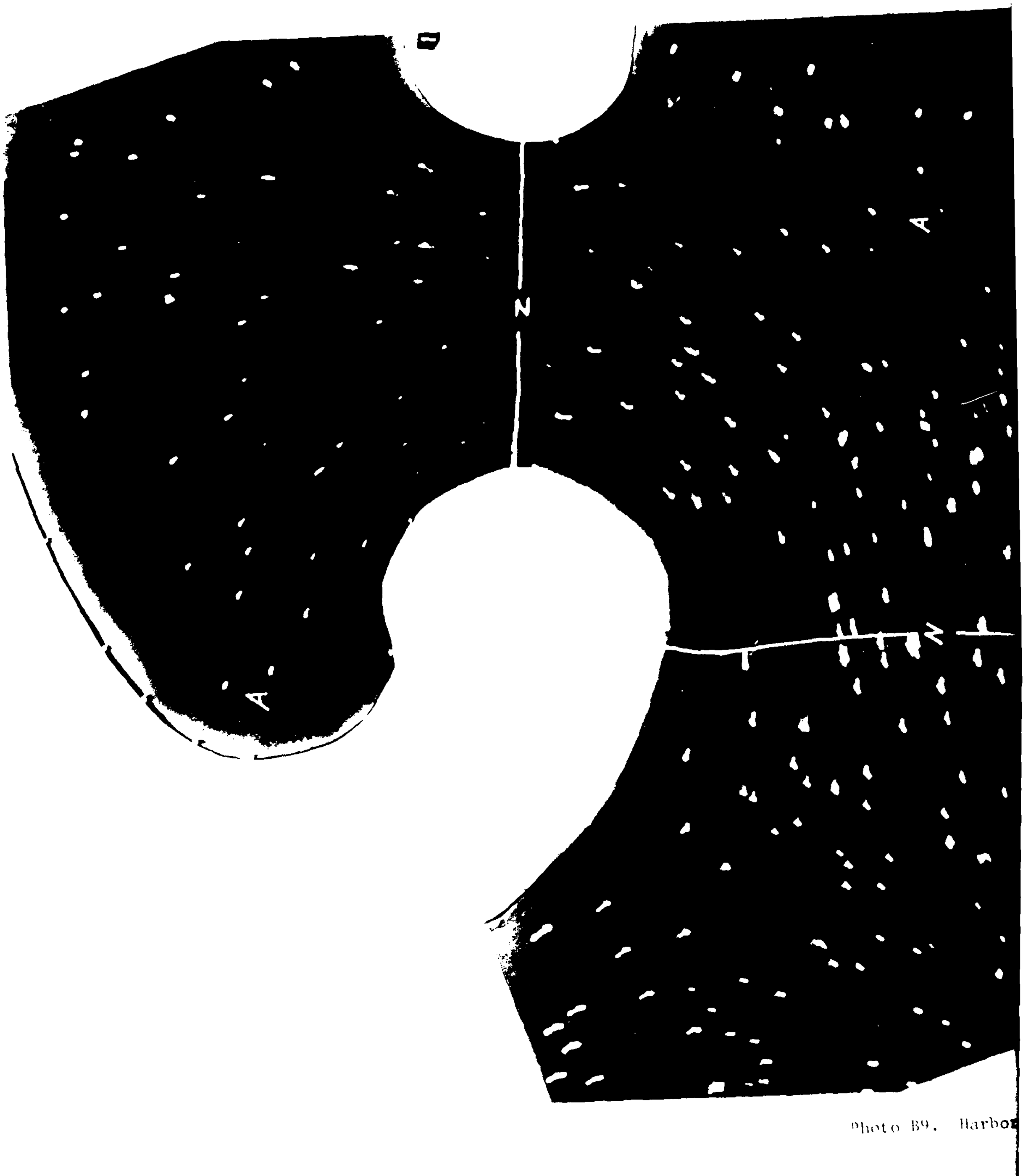
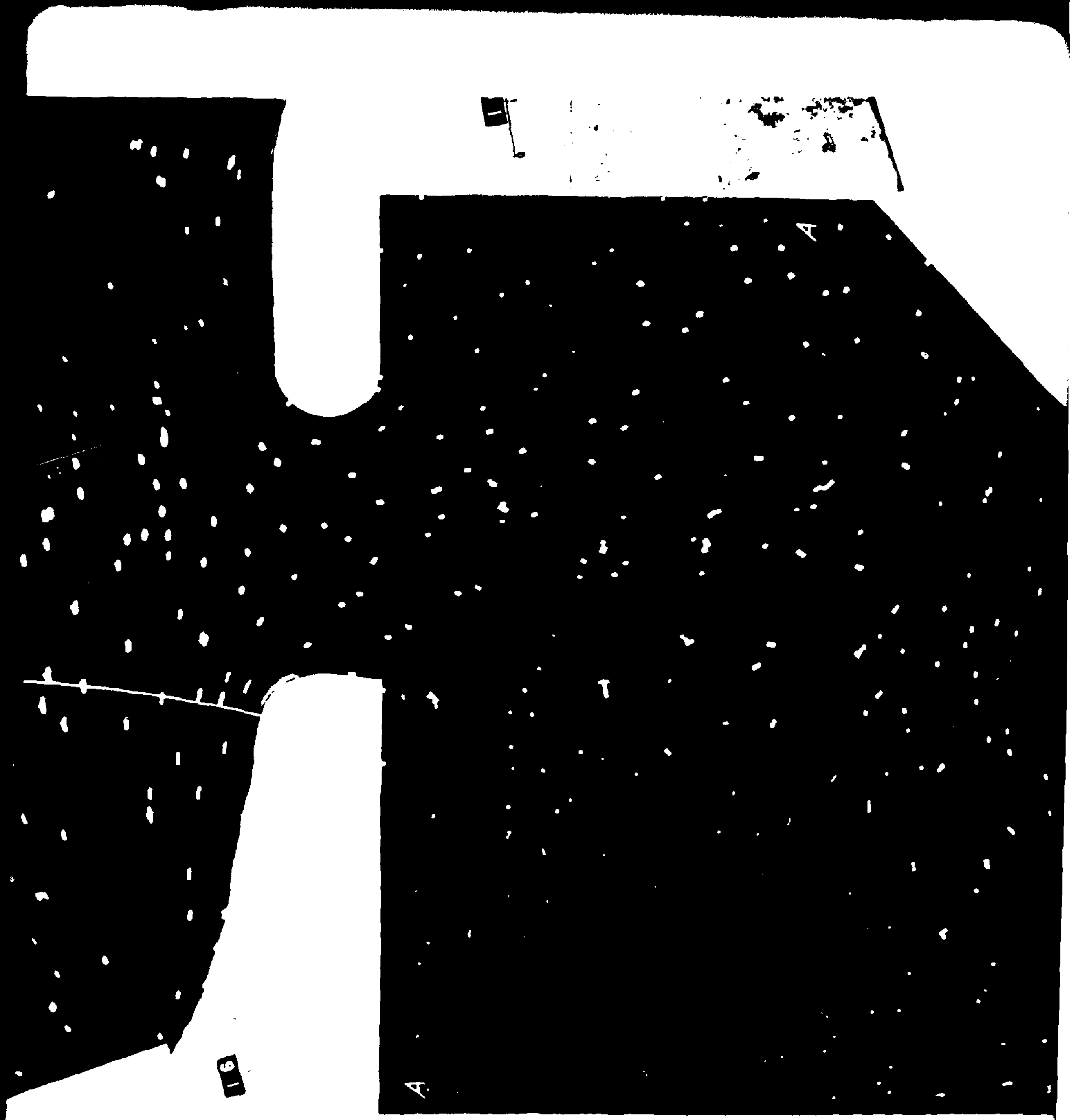


Photo B9. Harbor



Harbor oscillations for Plan 3G resulting from 117-sec waves

2

17-sec waves

3

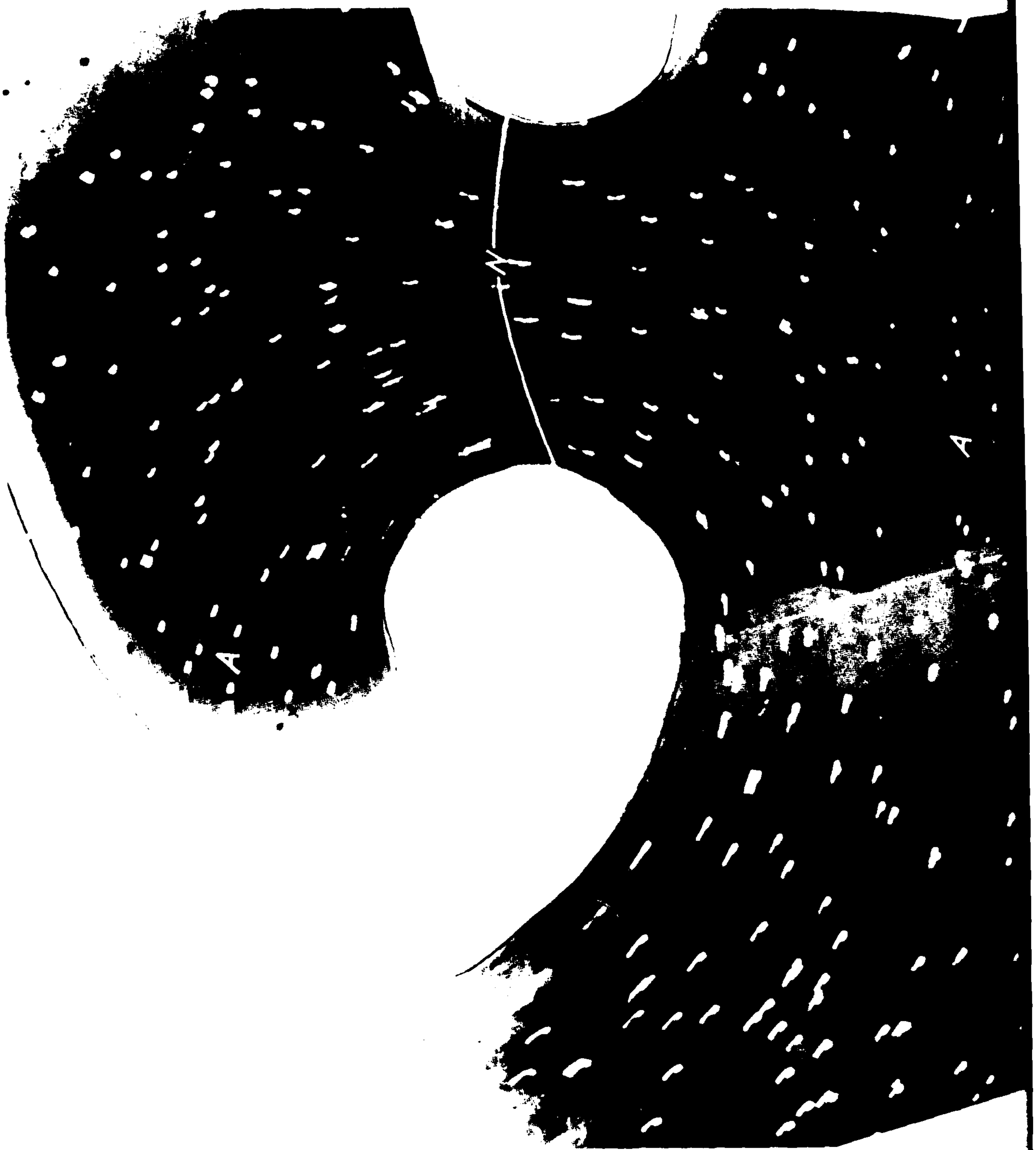
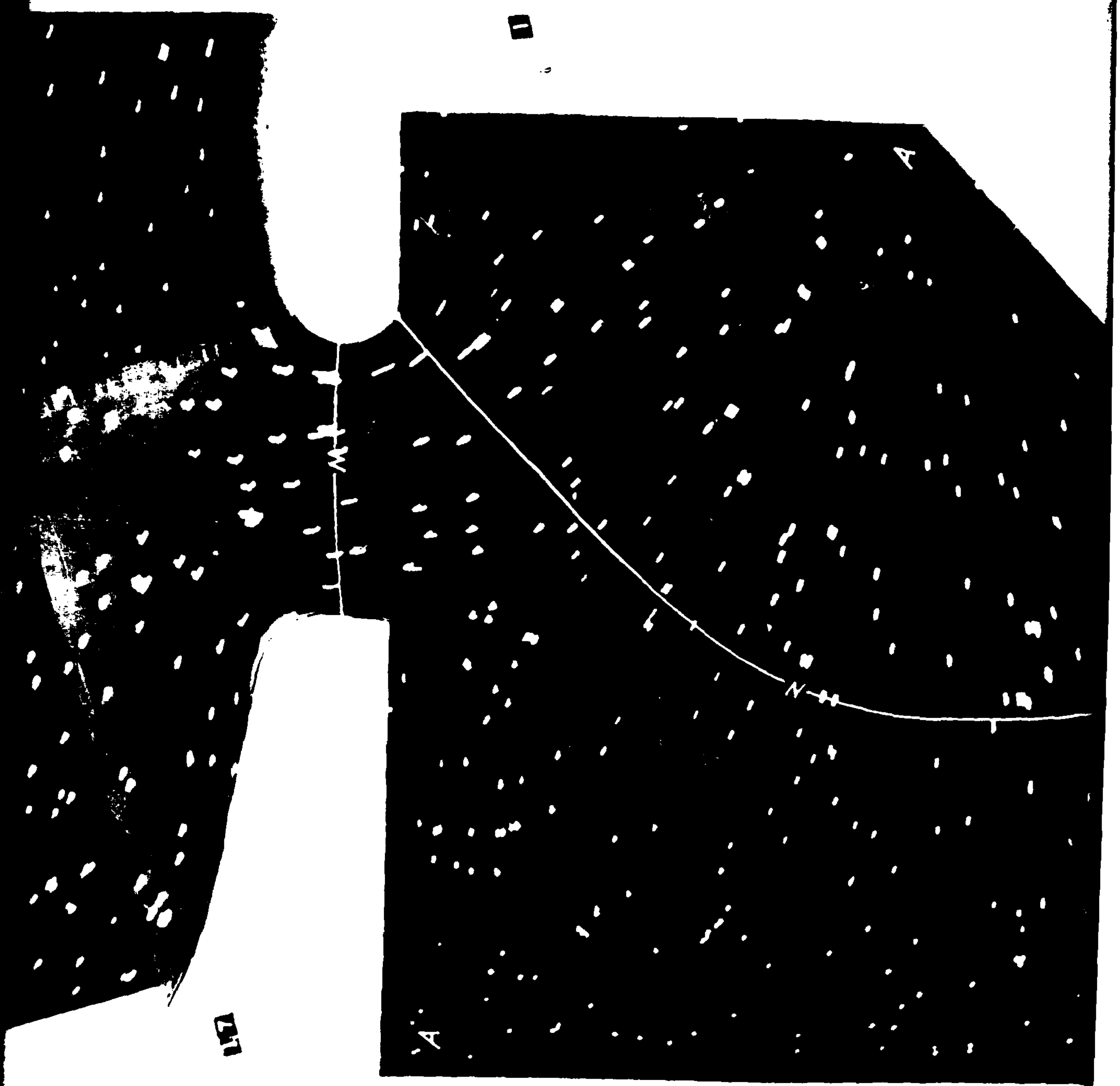
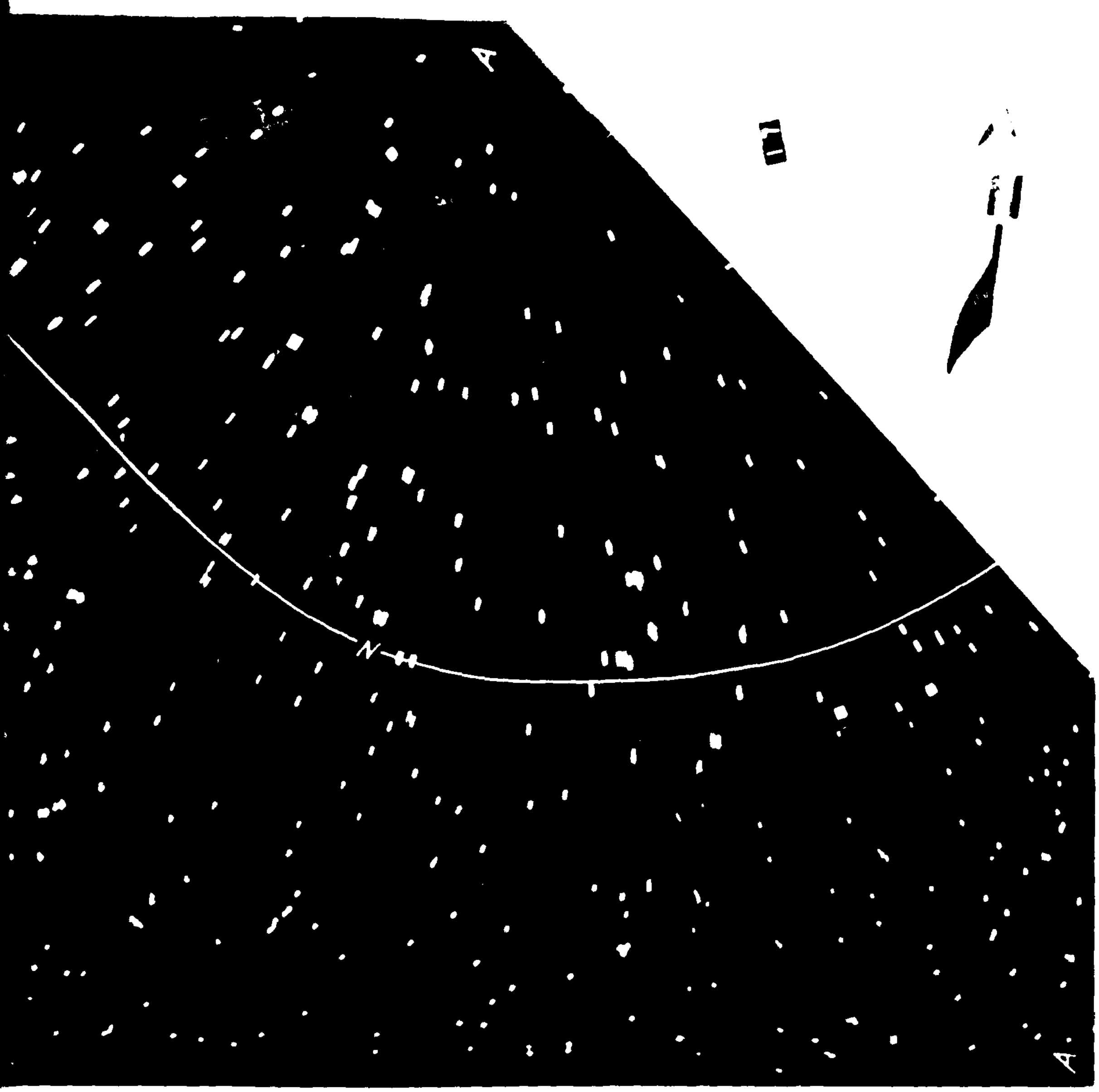


Photo B10. Har



Harbor oscillations for Plan 3G resulting from 120-sec waves

Photo B6. Harbor osc



120-sec waves

3

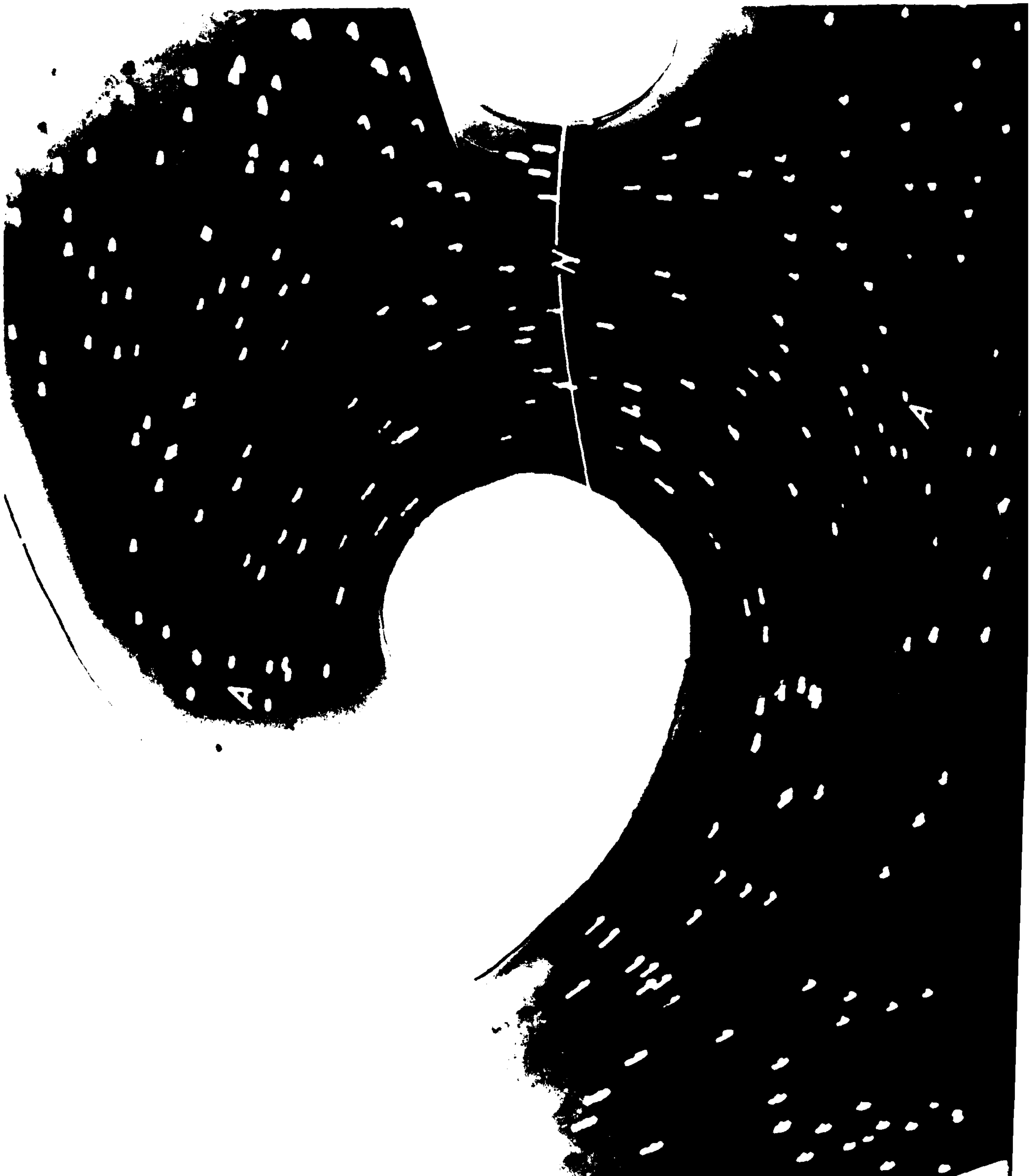
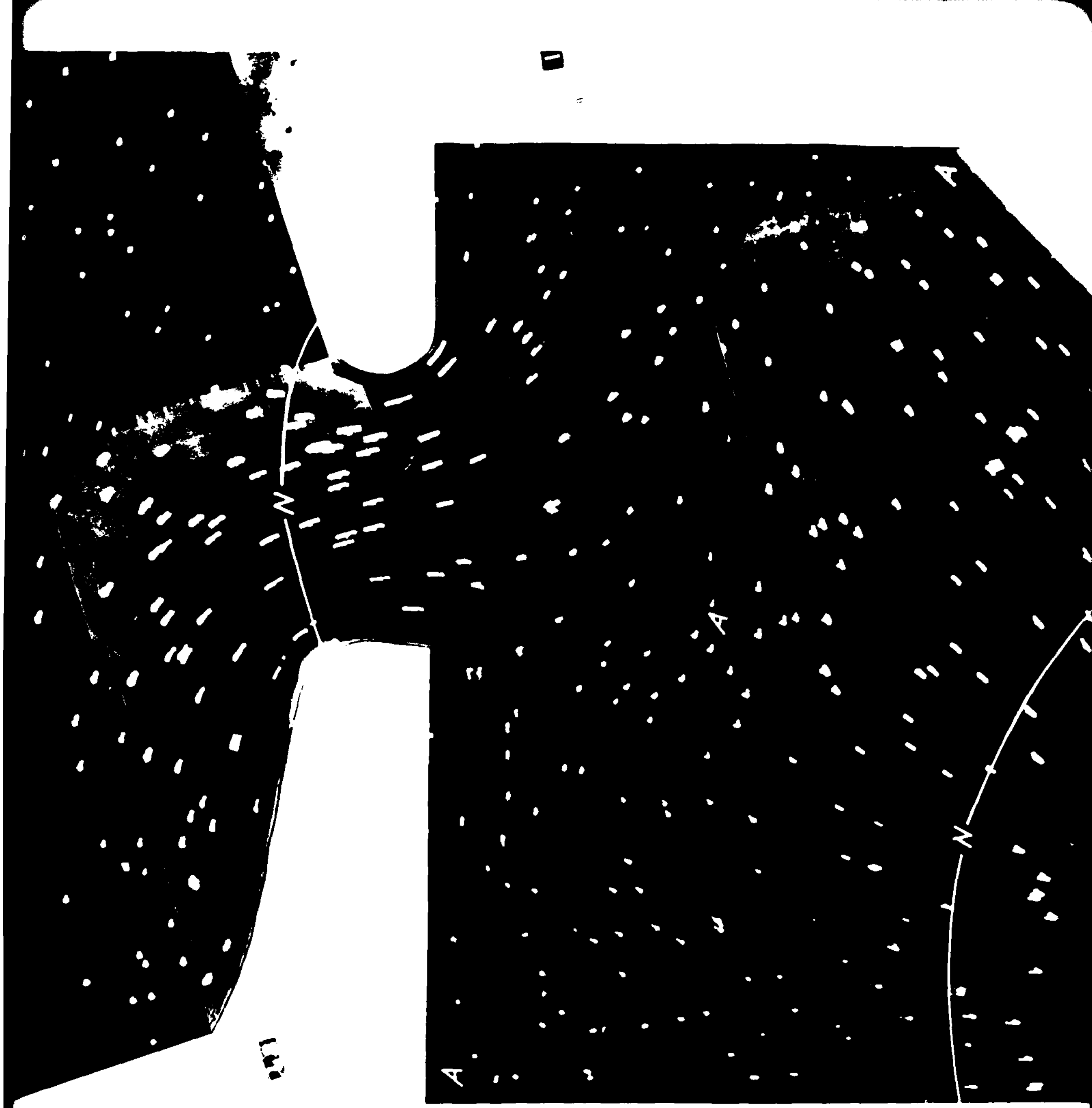


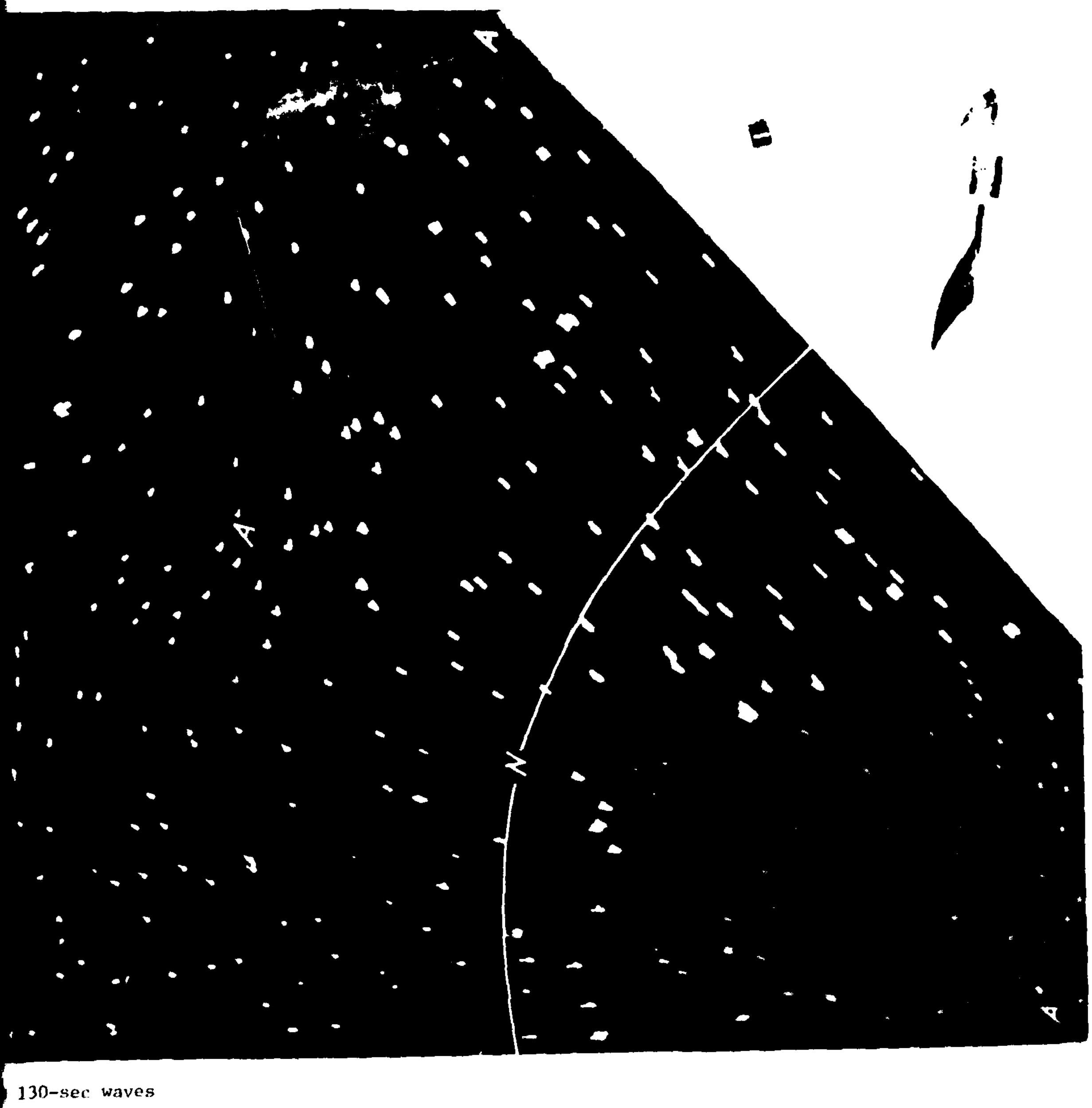
Photo B11.

3



Harbor oscillations for Plan 3G resulting from 130-sec waves

2



130-sec waves

3

APPENDIX C

NOTATION

A	Area
b	Shallow-water orthogonal spacing
b_o	Deepwater orthogonal spacing
$(b_o/b)^{1/2}$	Refraction coefficient
d	Water depth at the gage
d_b	Breaking depth
D_{50}	Median particle diameter
frequency*	Dimensionless frequency
g	Gravitational constant
H	Shallow-water wave height
H_a	Wave height at antinode
H_b	Maximum nonbreaking wave height
H_i	Incident wave height
H_n	Wave height at node
H_o	Deepwater wave height
K	Shoaling coefficient
L	Length
Q	Discharge
R	Response factor
T	Measured wave period at the gage
T	Time
$T_{(n,m)}$	Natural period of oscillation at modes n and m
V	Velocity
$\forall$	Volume
x	Length of basin
y	Width of basin
γ	Specific weight

η_D Ratio of median particle diameter

η'_γ Specific weight ratio

λ Horizontal scale

μ Vertical scale

DA

FIL