TECHNICAL REPORT H-68-6

DESIGN FOR OPTIMUM WAVE CONDITIONS CRESCENT CITY HARBOR, CRESCENT CITY CALIFORNIA

Hydraulic Model Investigation

by

P. K. Senter
C. W. Brasfeild

September 1968

Sponsored by

U. S. Army Engineer District
San Francisco

Conducted by

U. S. Army Engineer Waterways Experiment Station
CORPS OF ENGINEERS
Vicksburg, Mississippi

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FOREWORD

Request for a model investigation of Crescent City Harbor was initiated by the District Engineer, U. S. Army Engineer District, San Francisco (SFD), in a letter to the Division Engineer, U. S. Army Engineer Division, South Pacific, dated 30 March 1965. Authorization for the U. S. Army Engineer Waterways Experiment Station (WES) to perform the study was granted on 9 April 1965 by the Office, Chief of Engineers. Model construction was completed in April 1966, and the tests were conducted from May 1966 through August 1967.

Before the investigation was begun, a WES engineer visited the SFD office to confer with its representatives concerning the prototype problem and the model study. During the course of the study, liaison was maintained between the SFD and WES by means of conferences, telephone communications, and periodic progress reports.

Corps of Engineers personnel who visited WES to attend conferences and witness model demonstrations were: Mr. O. F. Weymouth of the South Pacific Division, and LTC F. C. Boerger, District Engineer, Messrs. G. P. Reilly, P. L. Vredenburg, R. Riddle, R. E. Blyberg, and O. T. Magoon of the SFD. Others who visited WES in connection with the study were: Honorable Donald Clausen, U. S. House of Representatives, from the First Congressional District of California; Mr. T. J. McNamara, Supervisor, Del Norte County, California; Mr. W. C. Peepe, Mayor, Crescent City, California; Messrs. C. A. Brower, President, F. E. Finley, Director, A. J. Phillips, Director, D. G. Richcreek, Harbor Master, M. J. Scavuzzo, Director, J. J. Yarbrough, Director, and T. J. Murray, Consultant, Crescent City Board of Harbor Commissioners.

The investigation was conducted in the Hydraulics Division of WES under the general direction of Mr. E. P. Fortson, Jr., Chief of the Hydraulics Division, and Mr. R. Y. Hudson, Chief of the Water Waves Branch. The model tests were conducted by Mr. P. K. Senter, Project Engineer, assisted by Mr. J. M. Hall, Engineering Technician, under the successive supervision of Messrs. H. B. Wilson, Engineer, and C. W. Brasfeild, Engineering Technician, of the Harbor Wave Action Section. This report was prepared by Messrs. Senter and Brasfeild.

COL John R. Oswalt, Jr., CE, and COL Levi A. Brown, CE, were Directors of WES during the conduct of the model study and the preparation and publication of this report. Mr. J. B. Tiffany was Technical Director.

CONTENTS

CONVERSION FACTORS, BRITISH TO METRIC UNITS OF MEASUREMENT

British units of measurement used in this report can be converted to metric units as follows:

Multiply	By	To Obtain
feet	0.3048	meters
square feet	0.092903	square meters
miles	1.609344	kilometers
square miles	2.58999	square kilometers
tons	0.907185	metric tons

SUMMARY

Tests were conducted on a 1:125-scale model of Crescent City Harbor and sufficient adjacent coastline and offshore bathymetry to permit generation of waves and wave-front patterns from all significant directions of wave approach to the harbor. The hydraulic model, equipped with wave-generating and wave-measuring apparatus, was used to determine the optimum length and location of an extension, or extensions, to the existing breakwater system that would reduce to a tolerable level the present adverse effects of storm waves on navigation and mooring conditions in the harbor.

It was concluded that (a) wave action could be reduced to a satisfactory level in the inner harbor basin by installation of a 400-ft-long northwesterly extension of the inner breakwater; and (b) a 2000-ft extension of the existing outer breakwater to Round Rock, with a 1200-ft-long companion breakwater extending from Whaler Island, would substantially improve navigation and mooring conditions in the harbor.

DESIGN FOR OPTIMUM WAVE CONDITIONS, CRESCENT CITY HARBOR

CRESCENT CITY, CALIFORNIA

Hydraulic Model Investigation

PART I: INTRODUCTION

Description of the Prototype

1. Crescent City Harbor, California (fig. 1), is located on the Pacific Ocean about 320 miles* north of San Francisco and 17 miles south of the Oregon border. As measured along the shoreline, the harbor is about 1 mile long and faces south. The harbor area includes an inner

Fig. 1. Crescent City Harbor, California, October 1956

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* A table of factors for converting British units of measurement to metric units is presented on page vii.

harbor basin and an outer harbor basin (plate 1). The entrance to the harbor is a natural channel about 22 ft deep and 500 ft wide between Fauntleroy Rock and Flat Rock. Existing protective structures (see fig. 1 and plate 1) in the harbor are as follows: (a) the outer breakwater, which is a concrete-capped, rubble-mound structure extending 4700 ft from shore in a southeasterly direction on the west side of the harbor; (b) the inner breakwater, a 1200-ft-long, rubble-mound structure extending northwesterly from Whaler Island; and (c) a rubble-mound sand barrier approximately 2400 ft long, constructed between Whaler Island and the shore on the east side of the harbor to prevent sand movement into the inner harbor.

Proposed Harbor Improvements

2. Several proposals have been advanced for the improvement of Crescent City Harbor. The project plan recommended by the District Engineer, U. S. Army Engineer District, San Francisco (SFD), included in the inner harbor a 1500-ft-long T-shaped basin, with a stem approximately 1000 ft long, dredged to a depth of 20 ft, and a 300-ft-long extension of the existing inner breakwater in a northwesterly direction. Alternative proposals for which consideration was requested by local interests are referred to as a long-range protection plan for the entire harbor, a deep-draft harbor, and an expanded inner harbor plan. The alternative proposals involved (a) constructing an arm of breakwater extending about 2400 ft southwesterly from Whaler Island to provide, with the existing outer breakwater, a navigation entrance about 300 or 400 ft wide; (b) dredging a new basin near Whaler Island for deep-draft vessels; (c) increasing the depth of all navigable water in Crescent City Harbor to 20 ft; (d) extending the outer breakwater along the original alignment to Round Rock; and (e) in conjunction with (d) above, constructing a companion arm of breakwater extending from Whaler Island in a westerly direction.

The Problem

3. The harbor is exposed to wind waves (sea and swell) from all

deepwater directions clockwise between south and west-southwest. These waves, reckoned 3000 to 4000 ft outside the harbor entrance, range in height from 5 to 22 ft and in period from 5 to 17 sec. Specific problems cited by local interests are damage to moored vessels and vessel time lost due to wave action and surge. Also, the present harbor depths preclude usage by fully loaded, deep-draft vessels. These factors contribute to excessive transportation costs for lumber, petroleum products, and other commodities being transported through the harbor.

Purpose of the Model Study

4. The model study was conducted to determine the optimum length and location of an extension to the existing breakwater system that would reduce to a tolerable level the present adverse influence of storm waves on navigation and mooring conditions in the harbor.

Motion Picture

5. At the request of the SFD, several motion picture sequences were secured in connection with the Crescent City Harbor model study. The motion pictures show wave action in the model harbor with existing conditions and with test plans 1, 2, 4, 6A, 7, 8, and 9 installed in the model, and with simulated storm waves from the south and southwest deepwater directions. This film, unedited, was furnished the SFD in February 1968.

PART II: THE MODEL

Design

6. The Crescent City Harbor model (photograph 1) was constructed using a linear scale of 1:125, model to prototype. Selection of this scale was based on such factors as (a) the depth of water required in the model to minimize bottom friction effects; (b) the absolute size of model waves; (c) available shelter dimensions and the area required for the model; (d) efficiency of model operation; (e) characteristics of required wave-generating and wave-measuring equipment; and (f) cost of model operation. A geometrically undistorted model was necessary to ensure accurate reproduction of wave patterns. Following selection of the linear scale, the model was designed and operated in accordance with Froude's model law.[1] The scale relations used for design and operation of the model were as follows:

Characteristic	Dimension*	Model:Prototype Scale
Length	L	$L_r = 1:125$
Area	L^2	$A_r = L_r^2 = 1:15,625$
Volume	L^3	$\forall_r = L_r^3 = 1:1,953,125$
Time	T	$T_r = L_r^{1/2} = 1:11.18$
Velocity	L/T	$V_r = L_r^{1/2} = 1:11.18$

Description

7. The model, which was molded in cement mortar, reproduced to scale the existing prototype harbor and sufficient adjacent coastline and offshore bathymetry to permit generation of waves and wave-front patterns from all significant directions of wave approach to the harbor. The area

* Dimensions are in terms of length and time.

of the model was approximately 10,700 sq ft, representing about 6.0 square miles in the prototype. Vertical control in model construction was based on the mean lower low water (mllw) datum, and all elevations used in this report are in feet referred to this datum (at Crescent City, mllw is 3.8 ft below mean sea level). Horizontal control was referenced to the Lambert Conformal Projection, Zone 1, California, as described in U. S. Coast and Geodetic Survey Special Publication No. 253.[2] Bottom contours were reproduced seaward to prototype elevations ranging to -60.0. A relatively flat-sloped transition extended downward from the contoured area to the wave machine pit, which was at an elevation of -85.0.

8. Model waves were generated to scale by a 60-ft-long wave machine with a vertical-motion plunger, trapezoidal in shape. The vertical movement of the plunger caused a periodic displacement of water incident to this motion. The length of plunger stroke and the period of vertical motion were infinitely variable over the range necessary to generate waves with the required characteristics. The wave machine was mounted on retractable casters that enabled it to be positioned to generate waves from the required directions.

9. Wave heights at selected locations in the model were recorded on photosensitive chart paper by a multichannel, electrically operated oscillograph. The input to the oscillograph was the output of electrical wave height gages, which measured the changes in the water-surface elevation with respect to time. The electrical output of each wave height gage was directly proportional to the submergence of the gage in water.

PART III: THE TEST PROGRAM

Selection of Test Conditions

Still-water level

10. Still-water levels (swl) for harbor wave-action models are selected so that the various wave-induced phenomena that are dependent upon water depths can be reproduced accurately in the model. These phenomena include the refraction of waves in the harbor area, the overtopping of harbor structures by the waves, the reflection of wave energy from nonporous structures, and the transmission of wave energy through porous structures. Some of the most important factors that should be taken into consideration in selection of a model swl are that (a) the maximum amount of wave energy that can reach a coastal area will ordinarily do so during the period of a severe storm that coincides in time with the higher-high-water phase of the astronomical tide cycle; (b) severe storms are characteristically accompanied by an additional increase in the normal water level due to wind tide and mass transport; and (c) a relatively high swl in the model is beneficial in minimizing the effects of bottom friction, which can be excessive in shallow areas of small-scale models. Therefore, with consideration for the various factors contributing to and affected by the static water level in the prototype, and in view of the tendency toward more conservative results from the model investigation, it is desirable that a model swl be selected that closely approximates the higher water stages that normally prevail during severe storms in the prototype. This entails the study of tide height records in the prototype locality, with due attention to the higher levels experienced in the area in the past.

11. The mean diurnal range of the astronomical tide at Crescent City Harbor is 6.9 ft, and the maximum range is 12.5 ft. Mean higher high water (mhhw) is +6.9, the extreme high water stage is about +10, and the lowest stage is -2.5 (see reference 3). In view of the low probability that a maximum astronomical tide stage, a high wind tide, and extreme storm waves will occur simultaneously, a model swl approximating

such a combination of extreme conditions was not considered justifiable. The swl selected, which is considered to be more representative of that which would be expected to occur during a representative, severe storm-wave attack on the harbor, was +7.5. This value corresponds to an assumed wind tide of 0.6 ft superimposed on the mhhw stage of +6.9.

Test waves

12. Factors influencing selection of test waves. In planning a test program for a model investigation of harbor wave-action problems, dimensions and directions for the test waves should be selected that will afford a realistic test of the improvement plans proposed, and thus permit the optimum plan of improvement to be accurately determined. Wind waves are generated by the tangential shear force of the wind on the water surface and the normal force of the wind against the wave crests. The height and period of the maximum wave that can be generated by a given storm depend on the wind speed, the duration for which wind of a given speed continues to blow, and the water distance (fetch) over which it blows. Factors that influence the selection of test waves include: (a) fetch distances in the various directions from which waves can attack the harbor; (b) the frequency of occurrence and the duration of winds of storm intensity blowing from the various directions; (c) the width, alignment, and position of the navigation entrance into the harbor; (d) the alignment, length, and position of reflecting surfaces inside the harbor; and (e) the refraction of waves by differentials in depth in the area seaward of the harbor, which may cause either a concentration or a diffusion of wave energy at the harbor site.

13. Prototype wave data. The northern coast of California is subject to severe winter storms that generate waves from directions ranging clockwise from south to northwest; however, the outer breakwater at Crescent City effectively protects the harbor from west-to-northwest waves. Thus, the evaluation of prototype wave data for the selection of test waves was restricted to waves associated with storms approaching the harbor site from the sector between south and west-southwest. Measured wave data upon which to base a comprehensive statistical analysis of wave conditions were not available for the Crescent City area. However,

statistical wave hindcast data compiled by National Marine Consultants[4] included data for a sea location (Station 1) approximately 50 miles northwest of Crescent City, and it was assumed that waves with similar characteristics could be expected to occur at Crescent City. The hindcast data provide average annual durations (in percentage of time) that waves of specific height and period can be expected to occur at Station 1. The data were grouped into the following directions from which the storm waves can approach the harbor: south, south-southwest, southwest, and west-southwest. The period of record covered by the hindcast data analysis was 1956-1958. The data separate the waves into two categories, "sea" and "swell." The term "swell" refers to waves resulting from storms originating at considerable distances from Station 1; the term "sea" refers to waves resulting from local or near-local storms. For the purpose of the present analysis, data for both sea and swell were combined, and the annual durations were converted from percentage of time to hours per year. Results of the deepwater wave analysis are presented in table 1.

14. Wave refraction. When wind waves move into water of gradually decreasing depth, transformations take place in all wave characteristics except wave period. The most important transformations with respect to the selection of test-wave characteristics are the changes in wave height and direction of travel due to the phenomenon referred to as wave refraction. The changes in wave height and direction can be determined by plotting refraction diagrams and calculating refraction coefficients. For this study, refraction diagrams were prepared by personnel of the SFD for representative wave periods from the critical directions of approach. These diagrams were 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 of energy, 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 $(b_o/b)^{1/2}$, derived from refraction diagram studies, is the refraction coefficient.

The shoaling coefficient (K) is a function of wavelength and water depth, and was obtained from tables compiled by Wiegel.[5] Thus, the refraction coefficient multiplied by the shoaling coefficient provides a conversion factor for the transfer of deepwater wave heights to corresponding shallow-water values.

15. Shallow-water waves. In general, shallow-water waves are those whose velocity is affected by both wavelength (L) and depth of water (d), which occurs when the value of d/L is about 0.5. For the investigation reported herein, the term "shallow-water test waves" refers to waves in the depth of water in which the wave generator was situated during model tests (92.5 ft prototype). After the refraction analysis had been completed, the deepwater wave heights (table 1) were converted to shallow-water values for use in the model. The conversion took into account the refraction and shoaling coefficients as outlined in paragraph 14. The results of the wave height conversion are presented in table 2.

16. Test waves selected. Wave height and period characteristics for the waves used in the testing program were selected on the basis of the height-period-duration data shown in table 2. Two to four test-wave heights were selected for representative wave periods for each deepwater wave direction. The model directions of shallow-water wave approach were determined from the refraction diagram study described in paragraph 14. The characteristics of the test waves selected are as follows:

Wave Period, sec	Deepwater Waves Direction	Deepwater Waves Height, ft	Selected Shallow-Water Test Waves Direction	Selected Shallow-Water Test Waves Height, ft
7.0	South	6, 10	S6°35'W	4, 8
9.0		8, 14, 16, 20	S10°22'W	6, 12, 14, 18
12.0		12, 20	S27°45'W	8, 14
9.0	South-southwest	6, 12, 16	S28°20'W	6, 10, 14
12.0		6, 14, 20	S35°40'W	6, 12, 18
14.0		14, 20	S38°30'W	12, 18
9.0	Southwest	8, 14	S47°15'W	8, 14
12.0		10, 16	S50°00'W	10, 16
14.0		8, 14, 18	S49°15'W	10, 16, 22
16.0		4, 8, 16	S49°15'W	6, 12, 22

(Continued)

Wave Period, sec	Deepwater Waves Direction	Height, ft	Selected Shallow-Water Test Waves Direction	Height, ft
12.0	West-southwest	8, 14	S63°25'W	8, 14
14.0		10, 18, 24	S63°25'W	10, 16, 22
16.0		10, 18, 24	S63°25'W	10, 16, 22

Test Data

17. The data obtained during the testing program included (a) wave height measurements at several locations inside and outside the harbor, (b) photographs showing wave-front patterns, and (c) visual observations. The locations of all wave height gages used during the course of the investigation are shown in plate 2. Data were first obtained for base test (existing) conditions, and then with the proposed improvement plans installed in the model. Comparison of the wave height data obtained from the various tests with the selected wave height criteria (paragraph 18) permitted evaluation of the effectiveness of each proposed improvement plan and provided a basis for selecting the optimum plan. Wave heights measured in the model were corrected to compensate for the increased rate of wave height attenuation in the model, due to bottom friction, compared to the amount of attenuation in the prototype. Keulegan's attenuation equation[6] was used to calculate attenuation coefficients for the model waves.

Wave Height Criteria for Evaluation of Improvement Plans

18. The wave height criteria used in this study to judge plan adequacy required that waves in the inner harbor basin not exceed 2 ft in height more than 24 hr/yr, and that waves in the outer harbor basin not exceed 3 ft in height more than 24 hr/yr.

PART IV: PLANS TESTED AND TEST RESULTS

Base Test

19. The term "base test," as used in this report, denotes a test performed with existing prototype conditions simulated in the model. These conditions are shown in plate 1. The sand barrier on the east side of the inner harbor has been damaged several times by storm waves that have lowered the crown to an elevation of about +6 to +8 in several reaches of the barrier. However, since the exact limits of the damaged sections were not known, the sand barrier was constructed to simulate the initial crown elevation of +10 for the model tests.

Description of Plans

20. As previously noted (paragraph 2), several proposals were made for improvement of the harbor, which varied in types and locations of protective structures with the particular purpose of the individual proposal. Details of the various proposals are enumerated in the following paragraphs, and all of the plan elements described are shown in plate 3.

Recommended project plan (plan 6)

21. The project plan was designated plan 6 of the model testing program. It included dredging the T-shaped basin in the inner harbor area to a depth corresponding to -20, as shown in plate 3, and extending the inner breakwater 300 ft in a northwesterly direction. The elements of plan 6A were the same as those of plan 6 except that the proposed inner breakwater extension was 400 ft long.

Long-range plans for harbor protection (plans 1, 4, 5, 7, 8)

22. Tests were conducted of several breakwater plans proposed to provide future protection to the entire harbor. Elements of the individual plans were as follows:

- a. Plan 1 entailed extending the existing outer breakwater 2000 ft in a southeasterly direction from the junction of the dogleg portion of the existing structure out to Round Rock.

b. Plan 4 elements consisted of the plan 1 breakwater with the addition of a companion breakwater extending from Whaler Island 1200 ft in a southwesterly direction out to Flat Rock.

c. Plans 5 and 5A contained the same elements as plan 4 plus a northwesterly extension of the inner breakwater to lengths of 300 and 400 ft, respectively.

d. Plan 7 involved the same elements as plan 4 with the addition of a rubble-mound wave absorber, parallel to and 100 ft (center line to center line) harborward of the existing outer breakwater. The absorber was approximately 2200 ft long, starting at a point opposite sta 15+40 of the outer breakwater and extending seaward to an intersection with the dogleg portion of the existing structure.

e. Plan 8 entailed construction of an arm of breakwater extending 1200 ft from Whaler Island in a southwesterly direction out to Flat Rock. Plans 8A and 8B involved 1000-ft-long alternate alignments of the breakwater arm of plan 8, with the plan 8A structure being rotated 22 deg northwesterly from plan 8 and the plan 8B structure being rotated an additional 22 deg northwesterly from plan 8A.

Deep-draft harbor (plans 2, 3)

23. The following plans were tested to provide information pertinent to the possible creation of a deep-draft harbor.

a. Plan 2 entailed the construction of a breakwater extending in a southwesterly direction from Whaler Island 1800 ft, then angling in a westerly direction and extending an additional 600 ft to form a navigation opening of 400 ft with the dogleg portion of the existing outer breakwater. For plan 2A, the westerly portion of the proposed structure was extended to a length of 700 ft, thereby reducing the width of the navigation opening to 300 ft.

b. Plan 3 involved extending the dogleg portion of the existing outer breakwater 1270 ft in a straight line, then angling the extension east-northeasterly and continuing about 630 ft, in conjunction with a 550-ft-long arm of breakwater stemming from Whaler Island in a southwesterly direction. Plan 3 provided a 400-ft-wide navigation entrance to the harbor.

Expanded inner harbor (plan 9)

24. Tests were conducted to determine the wave conditions that would obtain in a proposed northwesterly expansion of the inner harbor. Plan 9 consisted of a 400-ft-long northwesterly extension of the existing inner breakwater (as used for plan 6A) in conjunction with a breakwater extending from the +7.5 contour in the Elk Creek vicinity south for approximately

1700 ft, then angling southeasterly and continuing an additional 700 ft to form a 400-ft-wide navigation entrance to the inner harbor.

Test Results

25. The test results are presented in tables 3-12, which include wave height data and the estimated durations of waves of various heights and periods that can be expected to occur at selected locations inside and outside the harbor for the plans tested. Photographs 2-22 show wave patterns within the harbor for the various plans and test waves. In preparing the wave height duration data, model wave heights were used to compute wave reduction coefficients, H/H_w , where H is the adjusted wave height at the specified gage locations, and H_w is the wave height at the wave machine (corresponding to the shallow-water test-wave heights tabulated in paragraph 16). These coefficients were then applied to the shallow-water wave duration values contained in table 2. The results of this application are summarized in tables 3-10 and table 12, which show the estimated durations of waves of various magnitudes that may occur in the problem areas for the plans tested in the model.

Base test

26. Results of the tests conducted to determine the severity of wave action that may occur in the existing harbor due to storm waves approaching from the deepwater directions between south and west-southwest are summarized in table 3. These data indicate that severe wave conditions can obtain inside the harbor for all but the lowest magnitude storms. Significant waves ranging from 10 to 18 ft in height occur in the navigation entrance more than 150 hr/yr. It is estimated from the model test results that waves greater than 2 ft in height occur at the entrance to the inner harbor basin more than 400 hr/yr, and that waves higher than 3 ft occur in the outer harbor basin about 450 hr/yr. Photographs 2-5 show wave patterns that obtained with base test conditions installed in the model; the degree of wave severity visible in these photographs is indicative of the need for increased harbor protection for navigation and mooring.

Recommended project plan (plan 6)

27. Results of the wave height tests of the recommended project plan are presented in table 4. These data indicate that wave heights at the entrance to the inner harbor basin would be reduced considerably by the installation of plan 6; however, there would still be about 70 hr/yr when waves would reach a height of 2 ft or more. Results of the tests of plan 6A (table 4) indicate that the design criterion for the inner harbor is met by the installation of this alternate plan. Photograph 6 shows that wave conditions at the entrance to the inner harbor basin were improved by the installation of plan 6, and photograph 7 depicts the effectiveness of plan 6A in providing protection to the inner harbor basin.

Long-range plans for harbor protection (plans 1, 4, 5, 7, 8)

28. Results of wave height tests of plan 1 are presented in table 5 and photographs 8 and 9. These results reveal that plan 1 improved navigation conditions in the harbor considerably. However, the design criteria are not met by this plan because severe storm waves still overtopped the existing outer breakwater, and plan 1 did not provide any obstruction to waves that (a) are propagated from the south deepwater direction, (b) are diffracted around Round Rock from the south-southwest and southwest deepwater directions, or (c) overtop the plan 1 breakwater.

29. Results of the wave height tests of plan 4 (table 5) indicate that the addition of the companion breakwater from Whaler Island was effective in further reducing wave heights in the harbor. Photographs 10 and 11 show the effectiveness of plan 4 in providing protection to the harbor. Although the design criteria for the inner harbor basin were met with plan 4 installed, portions of the outer harbor basin would still be subjected to about 150 hr/yr of waves 3 ft high or higher. Plan 4 appears to provide satisfactory navigation conditions from Round Rock to the mooring area.

30. Results of tests of plans 5 and 5A (table 6) indicate that combining the elements of plan 4, which met the criteria for the inner basin, with the inner breakwater extensions would further decrease the wave heights at the inner harbor entrance. As shown in table 6, it is estimated

that waves higher than 1 ft would occur at the entrance (gage 6) approximately 180 hr/yr with plan 5 installed and approximately 80 hr/yr with plan 5A installed, while waves higher than 2 ft would occur approximately 12 hr/yr and 6 hr/yr with plans 5 and 5A, respectively, installed. Photographs 12 and 13 depict the effectiveness of these plans against a severe storm from the southwest deepwater direction.

31. Due to the relatively localized influence of the plan 7 breakwater structure, wave height data obtained during tests of that plan were not developed into wave height duration form. Instead, a comparison of wave heights measured at selected gage locations for base test, plan 4, and plan 7 for the test waves that caused severe overtopping of the outer breakwater is tabulated below:

	Wave Heights at Selected Gage Locations,* ft					
Gage	Base Test	Plan 4	Plan 7	Base Test	Plan 4	Plan 7
	14-sec, 22-ft Shallow-Water Waves from S49°15'W			16-sec, 22-ft Shallow-Water Waves from S49°15'W		
4	7.0	3.3	2.3	8.4	3.0	2.7
5	3.9	1.4	1.1	4.4	2.2	0.9
6	3.2	1.7	1.4	3.4	2.6	1.5
9	5.9	5.7	1.3	7.4	5.2	2.2
10	7.0	4.6	1.6	7.9	4.8	2.5
	14-sec, 22-ft Shallow-Water Waves from S63°25'W			16-sec, 22-ft Shallow-Water Waves from S63°25'W		
4	6.4	3.7	2.6	7.5	3.0	2.6
5	3.9	2.5	1.4	2.9	1.6	0.8
6	3.9	3.2	1.4	3.3	2.6	1.2
9	5.4	4.0	2.1	4.9	4.1	1.9
10	8.7	5.8	2.4	6.2	6.8	2.2

* Locations shown in plate 2.

These data and photograph 14 indicate that the plan 7 structure will effectively absorb the waves that overtop the outer breakwater. The optimum crown elevation of the plan 7 breakwater required to eliminate overtopping of this structure was determined by observational tests to be about +17.

32. Results of the wave height tests of plans 8, 8A, and 8B are

presented in tables 7-9 and photographs 15-17, respectively. Although these plans did not fulfill the design criteria requirements or provide navigation improvements from Round Rock to the breakwater, they did provide improved navigation and mooring conditions in the harbor. Tests were not conducted with these plans combined with the recommended extension of the inner breakwater or the plan 7 breakwater, but indications were that such a combination would provide satisfactory wave conditions in the harbor. The model data reveal that none of the three alignments has a definite advantage over the others for all parts of the harbor; therefore, it appears that plan 8 would be the most desirable of the three plans because it makes use of Flat Rock and provides the most useful protected harbor area. Also, Flat Rock would probably have to be excavated if one of the other two alignments was used.

Deep-draft harbor (plans 2, 3)

33. During the tests with plan 2 installed in the model, wave heights were measured at 25 locations (gages A1-A5, B1-B5, C1-C5, D1-D5, and E1-E5 as shown in plate 2), in addition to the ten basic gage locations, in the area enclosed by the plan 2 breakwater to obtain sufficient data for preparation of wave height contours for the proposed deep-draft mooring area. The wave height contours for three test-wave conditions are presented in plate 4. Wave heights measured at the basic gage locations provided the basis for the test results that are presented in table 10. These data indicate that wave heights that obtained in the area enclosed by the plan 2 structure would still be detrimental to mooring of deep-draft vessels. Examples of the unfavorable wave conditions that would occur are depicted in photographs 18 and 19. Narrowing the navigation opening width to 300 ft (plan 2A) did not appear to provide any additional protection. Test data for plan 2A are not presented herein; however, photograph 20 shows plan 2A being attacked by severe storm waves from the southwest deep-water direction.

34. Results of the wave height tests of plan 3 (table 11) indicate that, with this plan installed, the harbor would be protected sufficiently for development of a deep-draft mooring area, except for waves that overtop the protective structure. Photograph 21 shows plan 3 being attacked

by severe storm waves from the southwest deepwater direction, causing overtopping of the breakwater. This plan provided considerable wave protection; however, it is doubtful that an eastern entrance to the harbor would be satisfactory from the standpoint of navigation.

Expanded inner harbor (plan 9)

35. Results of the wave height tests of plan 9 (table 12) indicate that wave heights would be substantially decreased in the area enclosed by the plan 9 structures. However, it is estimated that waves higher than 2 ft would still obtain in the midharbor area (gage 7) approximately 480 hr/yr and in the west side (gage K) approximately 300 hr/yr. The estimated duration of waves 2 ft or higher at the existing inner harbor entrance (gages 5 and 6) was increased slightly for plan 9 compared to plan 6A. This was probably due to waves being reflected into this area by the plan 9 breakwater, which extends from shore. Photograph 22 shows plan 9 being attacked by storm waves from the south deepwater wave direction.

Sand barrier

36. A tentative proposal had been made to repair the sand barrier on the east side of the harbor and to raise its crown elevation to +15. Due primarily to a lack of available data on locally generated waves approaching the harbor from the south or slightly east of south directions, no tests were scheduled to determine the degree of protection that the proposed revision would afford the harbor. However, during the conduct of the scheduled tests, visual observations were made and manual measurements taken of wave heights that obtained inside the harbor in the vicinity of the sand barrier. For the wave conditions used in the model tests, no consequential disturbances were observed in the inner harbor because of waves overtopping or being transmitted through the sand barrier as constructed in the model (with a crown elevation of +10).

Design Wave Heights for Stability of Structures

37. A series of tests was performed to determine (a) the maximum wave heights along selected reaches at the proposed locations of the structures included in plans 1 and 2, and (b) the heights of waves that

approach the existing outer breakwater between sta 20+00 and sta 46+70 (seaward head, as shown in plate 1). The maximum wave heights were obtained for use as design wave heights for the proposed structures and for repair of sections of the existing outer breakwater. The tests were performed using a range of wave heights for selected wave periods and directions that had been found to cause extreme wave heights in the area of the structures in previous tests. In the maximum wave height tests for plan 1, waves were measured at several selected positions along a line representing the seaward toe of the proposed breakwater (plate 2), both with and without the breakwater installed in the model. Maximum wave heights were measured for the plan 2 breakwater (plate 3) along a line representing the center line of that structure (plate 2) without the structure installed in the model. Then, with the plan 2 breakwater installed, wave heights were secured along two ranges in order to determine comparative magnitudes of waves making a frontal attack on the breakwater. Range 1 was located along a line parallel to the breakwater center line and near the seaward toe of the structure; range 2 was parallel to and 62 ft seaward of range 1. In the tests to determine maximum wave heights for the existing outer breakwater, wave heights were measured along two ranges located (with respect to the outer breakwater) similar to the ones used with the plan 2 breakwater installed. The maximum wave heights (H_{max}) that were measured at the selected locations, based on all tests performed for each plan, are presented in the following tabulation:

Plan 1

	H_{max}, ft	
Gage	Breakwater Removed	Breakwater Installed
11	26.1	25.0
12	29.2	22.0
13	29.2	25.6
14	34.2	27.3
15	33.7	22.5
16	35.8	30.5
17	31.1	30.8
18	23.7	22.7
19	27.2	32.3

(Continued)

Plan 2

Breakwater Removed		Breakwater Installed			
		Range 1		Range 2	
Gage	H_{max}, ft	Gage	H_{max}, ft	Gage	H_{max}, ft
20	22.0	1C	15.2	1D	18.9
21	23.2	2C	13.7	2D	19.4
22	28.4	3C	14.4	3D	16.4
23	23.8	4C	13.2	4D	20.2
24	18.6	5C	11.9	5D	18.9
25	16.3	6C	12.3	6D	16.4
26	15.3	7A	17.4	7B	19.2
27	13.0	8A	22.4	8B	26.2
		9A	17.7	9B	18.2

Existing Outer Breakwater Installed

Breakwater Station	Range 1		Range 2	
	Gage	H_{max}, ft	Gage	H_{max}, ft
20+00	28	18.5	28A	24.6
23+12	29	15.2	29A	20.6
26+24	30	18.8	30A	20.2
29+36	31	23.0	31A	27.7
32+48	32	25.0	32A	29.6
36+70	11	26.1	11A	27.3
39+20	3A	15.6	3B	18.9
41+70	4A	17.6	4B	22.0
44+20	5A	14.2	5B	18.8
46+70	6A	14.0	6B	15.4

Note: Breakwater stations are shown in plate 1.
Gage locations are shown in plate 2.

38. The maximum wave heights measured with the breakwaters installed may not correspond to the dimensions of the actual waves that attack the structure because of the effects of wave reflection from the structures. However, the maximum wave that can attack a breakwater is a function of the reflection and overtopping characteristics of the structure. Thus, it is believed that the wave dimensions determined with the breakwater removed should be used for design. Unfortunately, design wave tests were not conducted with the existing breakwater removed. For use in the design of repair sections for this structure, it is suggested that the larger of the two wave heights obtained for ranges 1 and 2 be selected.

PART V: CONCLUSIONS

39. Based on the results of the hydraulic model study presented in this report, it is concluded that:

- a. Installation of a 400-ft-long northwesterly extension of the inner breakwater (plan 6A) will afford the desired protection for the existing inner harbor basin.
- b. Of the several plans tested, plan 4 would provide the best entrance conditions and protection for the overall harbor.
- c. A rubble-mound wave absorber installed parallel to and harborward of the existing outer breakwater (structure G, plan 7) would provide adequate protection from waves that overtop the existing outer breakwater.
- d. Plans 8, 8A, and 8B offer improved navigation and mooring conditions in the harbor. These plans all provide about the same degree of protection to the harbor, but plan 8 would provide the largest protected area of the three.
- e. Unfavorable wave conditions would still exist in the proposed deep-draft mooring area with the installation of plan 2 or 2A.
- f. An expanded inner harbor basin would be provided a fair degree of protection by the installation of plan 9, but waves with heights that exceed the criteria for the inner harbor basin would still obtain in a considerable portion of the newly formed inner basin.

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Table 1

Estimated Duration and Magnitude of Deepwater Storm Waves Approaching Station 1 from Directions Between South and West-Southwest

Wave Height ft	Duration, hr/yr, for Wave Periods* of 4 to 6 sec	6 to 8 sec	8 to 10 sec	10 to 12 sec	12 to 14 sec	14 to 16 sec	16 to 18 sec	18+ sec
				South				
1-2.9	272	31	46	26		2		
3-4.9	69	200	56	4	2			
5-6.9		119	12	2				
7-8.9		15	163	9				
9-10.9			72	10				
11-12.9			19	18	4			
13-14.9			15	38				
15-16.9			6	18				
17-18.9				26	4			
19-20.9			2	4	8			
				South-Southwest				
1-2.9	36	18	28	54	2	2		
3-4.9	15	76	16	6	8			
5-6.9		24	8	11	2			
7-8.9		2	55					
9-10.9			18	2				
11-12.9			4	6				
13-14.9			6	11				
15-16.9			4	2				
17-18.9				6				
19-20.9				11	10			
				Southwest				
1-2.9	186	54	93	110	11	2		
3-4.9	28	213	53	16	4	4		
5-6.9		57	26	16				
7-8.9		4	96	11	2	2	2	2
9-10.9			18	23				
11-12.9			6	4	4			
13-14.9			6	10	4		2	
15-16.9				2		2		
17-18.9					8			
				West-Southwest				
1-2.9	23	71	124	81	32	18	6	
3-4.9	8	122	112	58	13	11	2	
5-6.9		31	49	39	20	11	2	
7-8.9		2	68	25	2	6		
9-10.9			20	6	6	2		
11-12.9			4	8	8			
13-14.9				4	2			
15-16.9								
17-18.9					2		2	
19-20.9								
21-22.9								
23-24.9					6	2		

* Wave period groupings include the lower but not the upper values.

Table 2

Estimated Duration and Magnitude of Shallow-Water Waves Approaching Crescent City Harbor from Deepwater Directions Between South and West-Southwest

Wave Height ft	Duration, hr/yr, for Wave Periods* of 4 to 6 sec	6 to 8 sec	8 to 10 sec	10 to 12 sec	12 to 14 sec	14 to 16 sec	16 to 18 sec	18+ sec
				South				
1-2.9	272	31	46	30	2	2		
3-4.9	69	319	56	2				
5-6.9		15	175	9				
7-8.9			72	28	4			
9-10.9			19	38				
11-12.9			15	44	4			
13-14.9			6	4	8			
15-16.9								
17-18.9			2					
				South-Southwest				
1-2.9	36	18	28	54	2	2		
3-4.9	15	76	16	6	8			
5-6.9		24	8	11	2			
7-8.9		2	55					
9-10.9			22	2				
11-12.9			6	17				
13-14.9			4	2				
15-16.9				6				
17-18.9				11	10			
				Southwest				
1-2.9	186	54	93	110	11	2		
3-4.9	28	213	53	16	4			
5-6.9		57	26	16		4		
7-8.9		4	96	11				
9-10.9			18	23	2			
11-12.9			6	4		2	2	2
13-14.9			6	10	4			
15-16.9				2	4			
17-18.9								
19-20.9								
21-22.9					8	2	2	
				West-Southwest				
1-2.9	23	71	124	81	32	18	6	
3-4.9	8	122	112	58	13	11	2	
5-6.9		31	49	39	20	11	2	
7-8.9		2	68	25	2	6		
9-10.9			20	6	6	2		
11-12.9			4	8	8			
13-14.9				4	2			
15-16.9					2		2	
17-18.9								
19-20.9								
21-22.9					6	2		

* Wave period groupings include the lower but not the upper values.

Table 3

Estimated Duration* of Wave Heights for All Test

Directions Combined, Base Test

Wave Height** ft	Estimated Duration of Wave Heights at Gage Locations,† hr/yr									
	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10
0-1	34	26	547	1434	3035	2350	1177	2358	1492	1874
1-2	478	224	1378	976	580	1039	1903	917	1361	815
2-3	952	751	846	905	110	316	294	318	484	615
3-4	385	470	317	113	45	89	186	172	265	214
4-5	484	665	161	157	26	10	70	19	127	156
5-6	541	45	256	102			106	18	49	55
6-7	261	708	158	37			6	2	18	55
7-8	252		22	32			10		6	12
8-9	118	121	41	4	8		52		2	8
9-10	128	292	36	34						
10-11	28	139	16							
11-12	25	76	10	10						
12-13	49	31	16							
13-14	18	12								
14-15	20	33								
15-16	15	6								
16-17	8	85								
17-18	8	4								
18-19		54								
19-20		60								
20-21		2								

* Based on model wave reduction coefficients applied to the prototype wave data presented in table 2.
** Wave height groupings include the lower but not the upper values.
† Gage locations are shown in plate 2.

Table 4

Estimated Duration* of Wave Heights for All Test Directions Combined, Plans 6 and 6A

Wave Height** ft	Estimated Duration of Wave Heights at Gage Locations,† hr/yr Gage 2	Gage 4	Gage 5	Gage 6	Gage 9
			Plan 6		
0-1		1334	3206	3355	1231
1-2	368	1532	529	425	1290
2-3	979	452	69	24	632
3-4	334	260			346
4-5	883	141			97
5-6	141	69			26
6-7	250	16			44
7-8	196				69
8-9	131				69
9-10	177				
10-11	41				
11-12	103				
12-13	25				
13-14	75				
14-15	16				
15-16	14				
16-17	46				
17-18	23				
18-19	2				
			Plan 6A		
0-1		2785	3548	3212	1334
1-2	460	757	256	582	1528
2-3	891	221		10	521
3-4	346	37			179
4-5	867	4			161
5-6	169				32
6-7	321				18
7-8	129				29
8-9	242				2
9-10	74				
10-11	86				
11-12	87				
12-13	37				
13-14	28				
14-15	18				
15-16	18				
16-17	2				
17-18	23				
18-19	6				

* Based on model wave reduction coefficients applied to the prototype wave data presented in table 2.

** Wave height groupings include the lower but not the upper values.

† Gage locations are shown in plate 2.

Table 5

Estimated Duration* of Wave Heights for All Test Directions Combined, Plans 1 and 4

Wave Height** ft	Estimated Duration of Wave Heights at Gage Locations,† hr/yr									
	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10
					Plan 1					
0-1	1116	800	1289	1856	2600	2080	1939	2993	2038	2238
1-2	1065	815	1200	1132	894	1499	1039	608	1029	612
2-3	774	327	213	360	292	129	490	135	376	539
3-4	303	645	549	157	18	76	166	56	148	259
4-5	246	322	216	145		14	75	10	148	110
5-6	151	26	123	44		6	12		57	26
6-7	23	65	82	21			69		6	10
7-8	67	462	44	6			2		2	8
8-9	27	14	54	48				2		2
9-10	22	76	12	21			12			
10-11	2	29								
11-12	6	19		2						
12-13	2	33	2	12						
13-14		29								
14-15		44								
15-16										
16-17		98								
					Plan 4					
0-1	1245	3451	2158	2826	3688	3519	2756	3106	2681	2628
1-2	1189	290	1216	724	106	263	796	495	714	812
2-3	196	29	211	116	10	16	113	183	255	202
3-4	560	22	89	136		6	79	18	90	136
4-5	274	12	20	2			50	2	34	18
5-6	135		90				10		22	6
6-7	24		18						8	
7-8	65									2
8-9	21		2							
9-10	75									
10-11	18									
11-12										
12-13										
13-14	2									

* Based on model wave reduction coefficients applied to the prototype wave data presented in table 2.

** Wave height groupings include the lower but not the upper values.

† Gage locations are shown in plate 2.

Table 6

Estimated Duration* of Wave Heights for All Test Directions Combined, Plans 5 and 5A

Wave Height** ft	Estimated Duration of Wave Heights at Gage Locations,† hr/yr Gage 1	Gage 3	Gage 4	Gage 5	Gage 6	Gage 9
			Plan 5			
0-1	951	1420	2671	3706	3625	2457
1-2	1399	1132	971	98	167	900
2-3	709	650	150		12	265
3-4	369	399	6			126
4-5	156	158	6			50
5-6	27	13				2
6-7	75	30				
7-8	12					4
8-9	12	2				
9-10	92					
10-11						
11-12						
12-13	2					
			Plan 5A			
0-1	947	1664	2599	3756	3722	2194
1-2	1266	1427	1102	48	76	1054
2-3	790	534	82		6	320
3-4	441	108	21			165
4-5	159	30				65
5-6	71	10				6
6-7	6	18				
7-8	85	11				
8-9	6	2				
9-10	2					
10-11	29					
11-12						
12-13						
13-14	2					

* Based on model wave reduction coefficients applied to the prototype wave data presented in table 2.

** Wave height groupings include the lower but not the upper values.

† Gage locations are shown in plate 2.

Table 7

Estimated Duration* of Wave Heights for All

Test Directions Combined, Plan 8

Wave Height** ft	Estimated Duration of Wave Heights at Gage Locations,† hr/yr								
	Gage 1	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10
0-1	136	951	1360	3649	3321	1364	2946	1944	1334
1-2	928	1187	1271	125	451	1464	749	1310	1669
2-3	464	784	626	30	22	464	79	344	436
3-4	704	442	373			274	30	140	185
4-5	480	258	91		10	120		44	158
5-6	251	102	49			95		22	12
6-7	219	28	14						10
7-8	218	20	20			13			
8-9	104								
9-10	132	22							
10-11						10			
11-12	85								
12-13	20	10							
13-14									
14-15	31								
15-16	20								
16-17									
17-18									
18-19	12								

* Based on model wave reduction coefficients applied to the prototype wave data presented in table 2.

** Wave height groupings include the lower but not the upper values.

† Gage locations are shown in plate 2.

Table 8

Estimated Duration* of Wave Heights for All

Test Directions Combined, Plan 8A

Wave Height** ft	Estimated Duration of Wave Heights at Gage Locations,† hr/yr								
	Gage 1	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10
0-1		951	1485	3595	3414	1401	3038	2122	2229
1-2	951	1399	1620	183	358	1635	697	1301	1152
2-3	1136	715	505	26	22	508	19	215	183
3-4	306	166	68		10	103	28	105	104
4-5	511	276	94			125	22	31	100
5-6	242	110	12			22		28	34
6-7	280	137	14					2	2
7-8	51	18	6						
8-9	138	24				10			
9-10	65	2							
10-11	92	6							
11-12									
12-13	2								
13-14									
14-15	14								
15-16	6								
16-17									
17-18									
18-19									
19-20	10								

* Based on model wave reduction coefficients applied to the prototype wave data presented in table 2.

** Wave height groupings include the lower but not the upper values.

† Gage locations are shown in plate 2.

Table 9

Estimated Duration* of Wave Heights for All

Test Directions Combined, Plan 8B

Wave Height** ft	Estimated Duration of Wave Heights at Gage Locations,† hr/yr								
	Gage 1	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10
0-1	652	951	1933	3601	3652	2011	3080	2836	2420
1-2	734	1181	1492	197	111	1230	419	646	1003
2-3	758	281	221	6	41	264	157	184	149
3-4	189	723	108			179	134	77	102
4-5	710	127	28			65	14	17	31
5-6	132	242	2			25		10	97
6-7	258	87	14			6		32	2
7-8	48	172	6			24			
8-9	32	18						2	
9-10	241								
10-11	18	11							
11-12		11							
12-13	12								
13-14									
14-15									
15-16									
16-17									
17-18									
18-19									
19-20									
20-21	14								
21-22									
22-23									
23-24	6								

* Based on model wave reduction coefficients applied to the prototype wave data presented in table 2.

** Wave height groupings include the lower but not the upper values.

† Gage locations are shown in plate 2.

Table 10

Estimated Duration* of Wave Heights for All

Test Directions Combined, Plan 2

Wave Height** ft	Estimated Duration of Wave Heights at Gage Locations,† hr/yr Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10
0-1		431	1630	1786	2570	2265	1757	2688	1876	1496
1-2	1334	1488	1278	1414	1044	1258	1390	905	1151	1144
2-3	893	614	578	346	141	238	496	197	437	720
3-4	381	377	170	177	29	33	87	14	211	173
4-5	361	192	45	34	10	10	26		69	209
5-6	361	296	37	14	10		19		20	50
6-7	86	47	35	12			2		28	12
7-8	165	165	2	11			6			
8-9	44	71					11			
9-10	57	40	8	10			10		12	
10-11	27	28	21							
11-12	18	14								
12-13	10	18								
13-14	26									
14-15	11	12								
15-16	6	11								
16-17	10									
17-18	2									
18-19										
19-20										
20-21	12									

* Based on model wave reduction coefficients applied to the prototype wave data presented in table 2.

** Wave height groupings include the lower but not the upper values.

† Gage locations are shown in plate 2.

Table 11

Wave Height Data, Plan 3

Test Waves		Wave Heights at Gage Locations,* ft																
Period sec	Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage A	Gage B	Gage C	Gage D	Gage E	Gage F	Gage G
						Shallow-Water Test Direction S10°22'W												
9	8	1.2	4.0	2.4	0.9	0.6	0.8	1.2	1.8	1.3	2.1	2.1	1.5	3.6	1.9	8.3	9.7	12.2
	12	1.4	5.0	3.1	1.2	0.9	0.8	1.6	2.2	1.7	2.4	2.5	2.8	4.3	2.5	11.0	12.2	15.2
	14	1.9	5.3	3.4	1.2	0.9	1.0	2.0	2.4	1.7	2.4	3.0	4.7	6.0	2.5	13.9	11.7	15.8
						Shallow-Water Test Direction S47°15'W												
9	8	0.8	0.8	1.2	1.1	0.8	1.0	0.8	0.5	1.5	0.8	0.7	1.3	2.1	1.0	3.7	1.4	5.8
	14	1.8	1.8	2.1	2.4	0.9	1.3	0.8	0.5	2.8	1.2	1.5	3.2	2.7	1.2	8.9	8.3	12.6
						Shallow-Water Test Direction S49°15'W												
14	10	2.2	1.4	3.7	3.6	1.2	1.2	2.9	0.8	4.6	3.2	2.8	3.6	3.3	1.2	7.9	9.1	10.4
	16	6.2	6.8	5.9	4.7	1.9	1.9	2.4	2.5	8.6	5.3	7.5	7.6	6.2	3.6	6.0	7.2	7.4
	22	7.6	8.6	5.9	3.6	1.4	1.9	2.7	3.2	5.0	4.9	7.6	9.9	7.9	6.7	5.8	6.3	8.6
16	6	1.7	1.1	1.6	2.3	1.0	1.4	2.3	0.9	2.2	2.9	1.5	1.5	2.2	1.1	3.0	4.8	4.8
	12	6.9	4.0	3.7	5.1	1.2	1.7	4.5	2.0	4.0	3.9	2.9	4.1	3.3	3.4	6.7	9.3	10.6
	22	11.0	8.1	8.5	6.3	2.2	2.7	5.8	2.2	5.9	6.8	10.4	10.5	8.0	6.3	6.3	7.2	9.3
						Shallow-Water Test Direction S50°00'W												
12	10	2.1	1.9	2.5	3.8	0.9	1.6	2.0	0.8	2.4	2.8	3.3	2.2	2.9	3.3	4.7	6.6	7.9
	16	6.6	6.0	4.2	4.6	1.5	2.0	2.9	1.8	4.6	4.4	7.6	5.7	5.2	7.9	9.5	12.7	14.2

* Gage locations are shown in plate 2.

Table 12

Estimated Duration* of Wave Heights for All

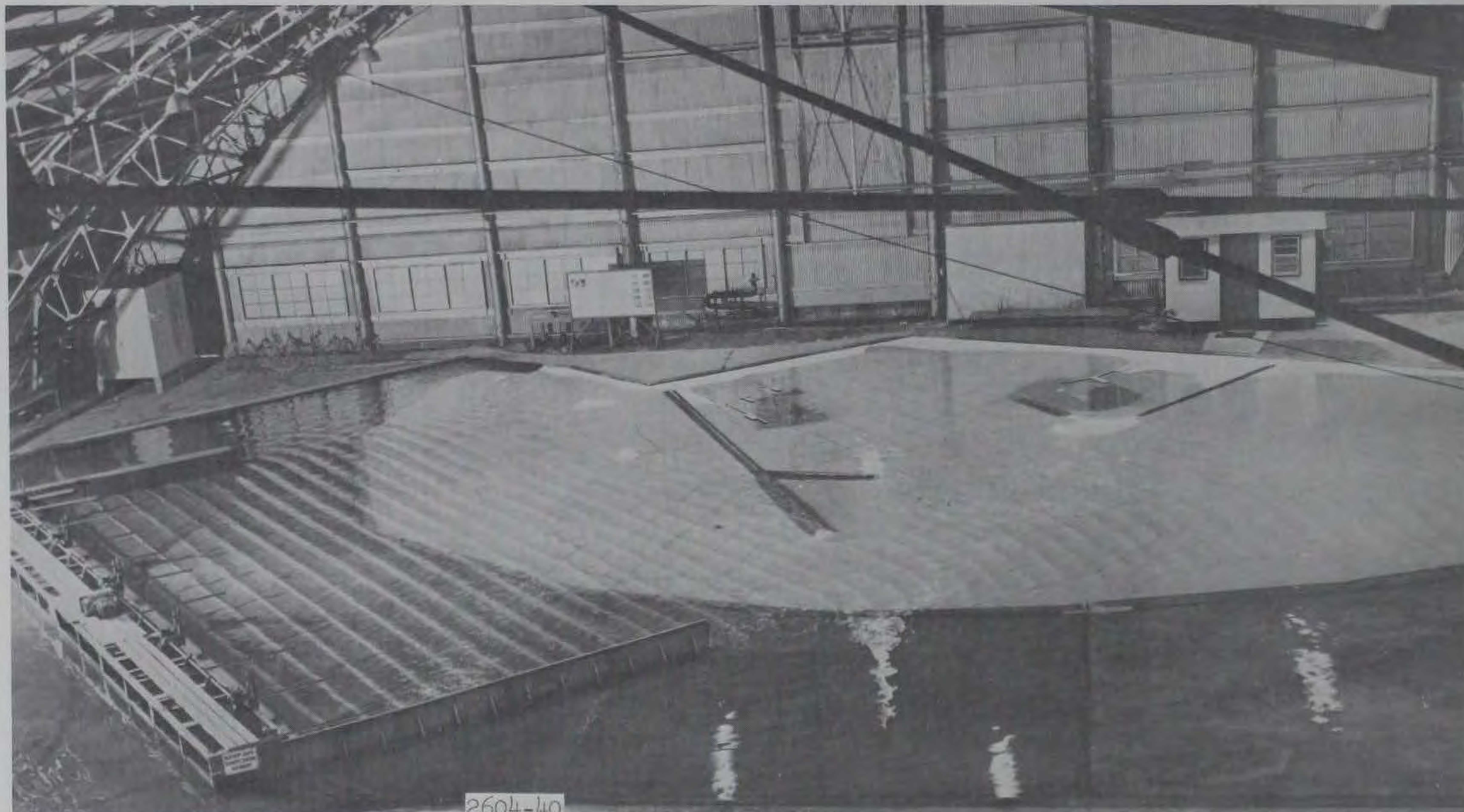
Test Directions Combined, Plan 9

Wave Height** ft	Estimated Duration of Wave Heights at Gage Locations,† hr/yr									
	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage K	Gage 9	Gage 10
0-1			1042	624	3000	3250	2190	2192	1887	1334
1-2	869	318	1394	1464	773	503	1136	1310	1063	1387
2-3	503	568	686	871	31	51	290	296	338	535
3-4	633	566	295	329			180		283	229
4-5	581	553	60	211			6	6	106	193
5-6	233	94	118	173			2		77	69
6-7	237	756	39	92					26	24
7-8	375	2	95	32					18	17
8-9	87	213	38	2					6	14
9-10	144	235	29							2
10-11	31	346								
11-12	50		6	6						
12-13	14	52								
13-14	14	16								
14-15	11	22	2							
15-16	16	4								
16-17		46								
17-18										
18-19		11								
19-20										
20-21	6	2								

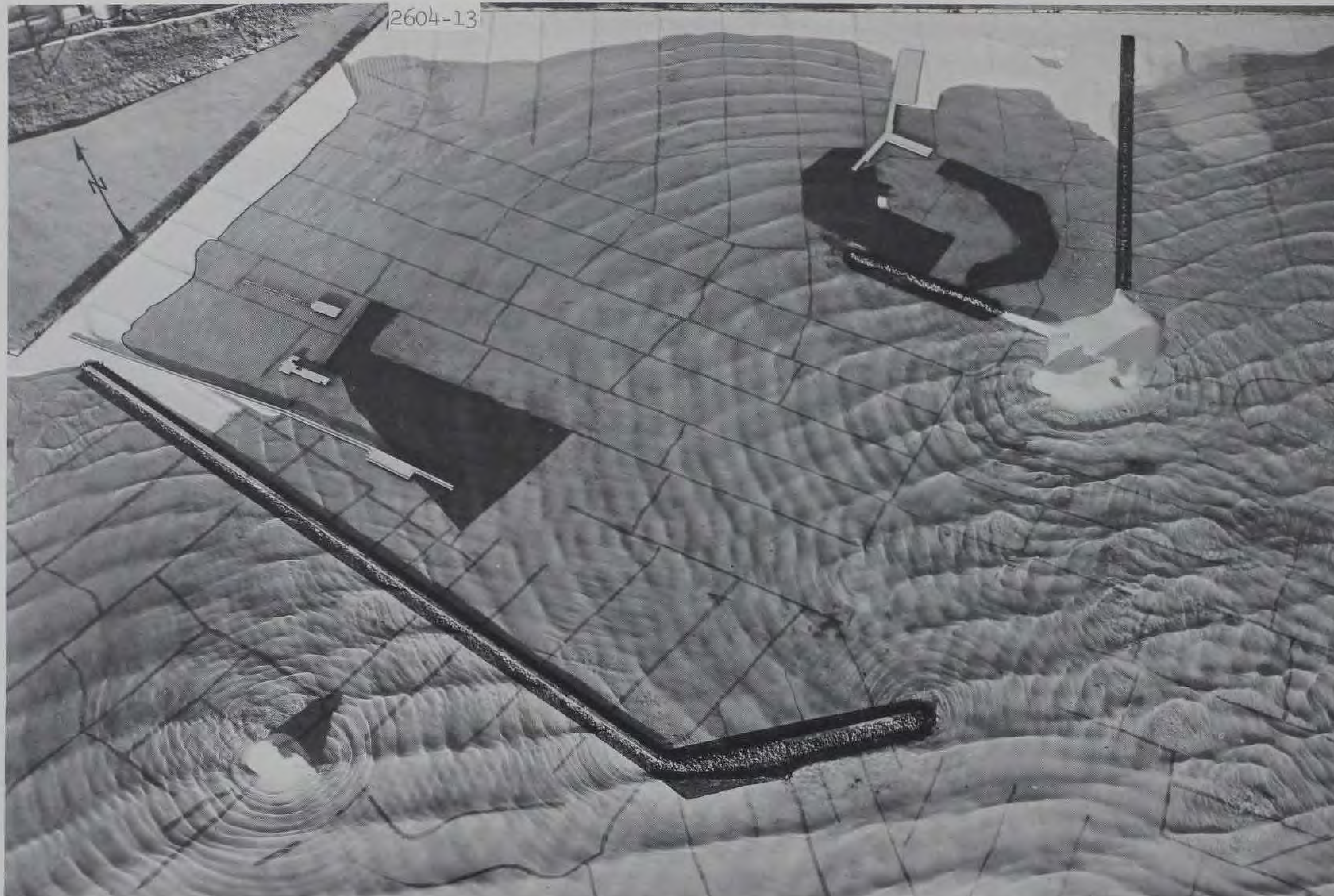
* Based on model wave reduction coefficients applied to the prototype wave data presented in table 2.

** Wave height groupings include the lower but not the upper values.

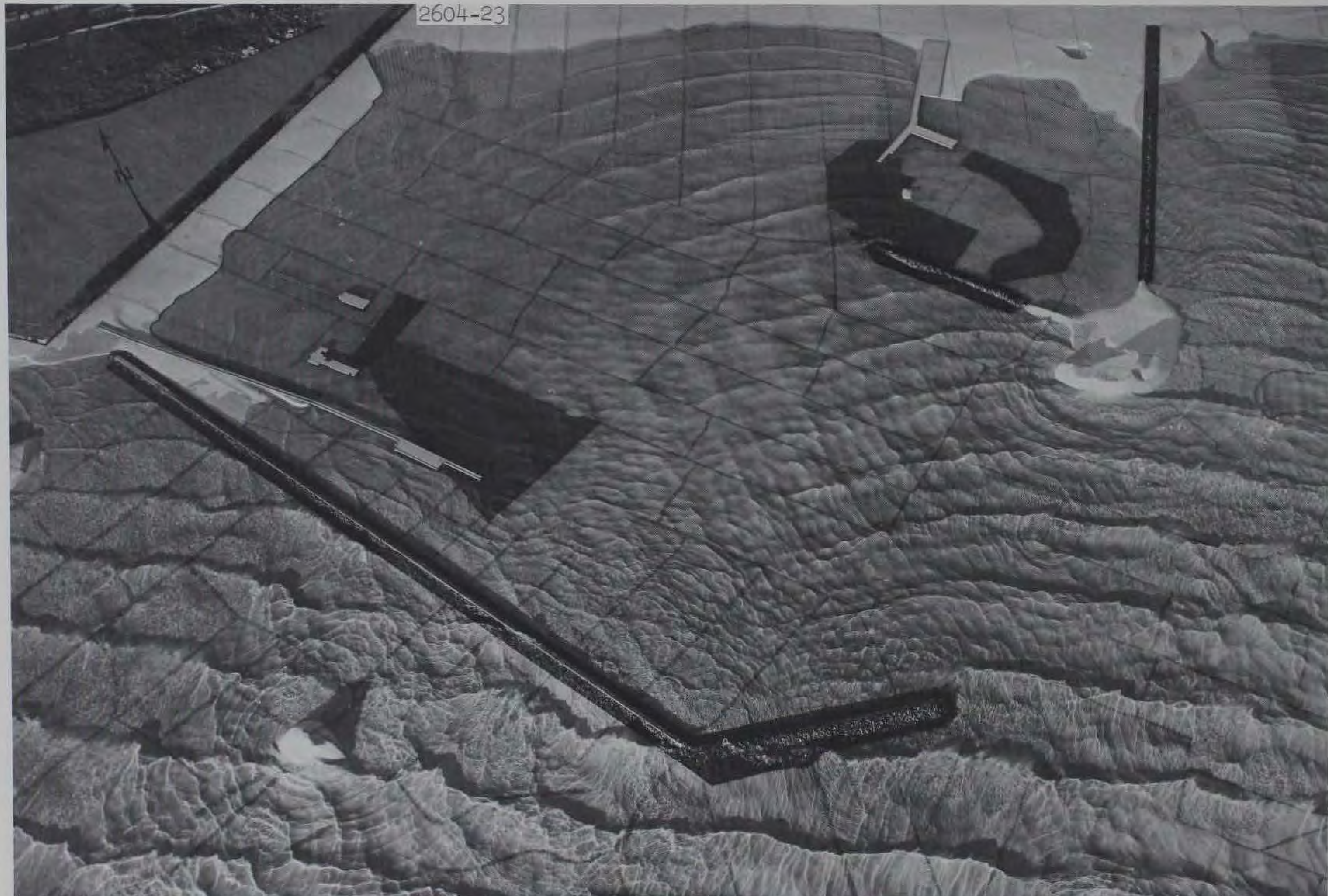
† Gage locations are shown in plate 2.



Photograph 1. General view of model

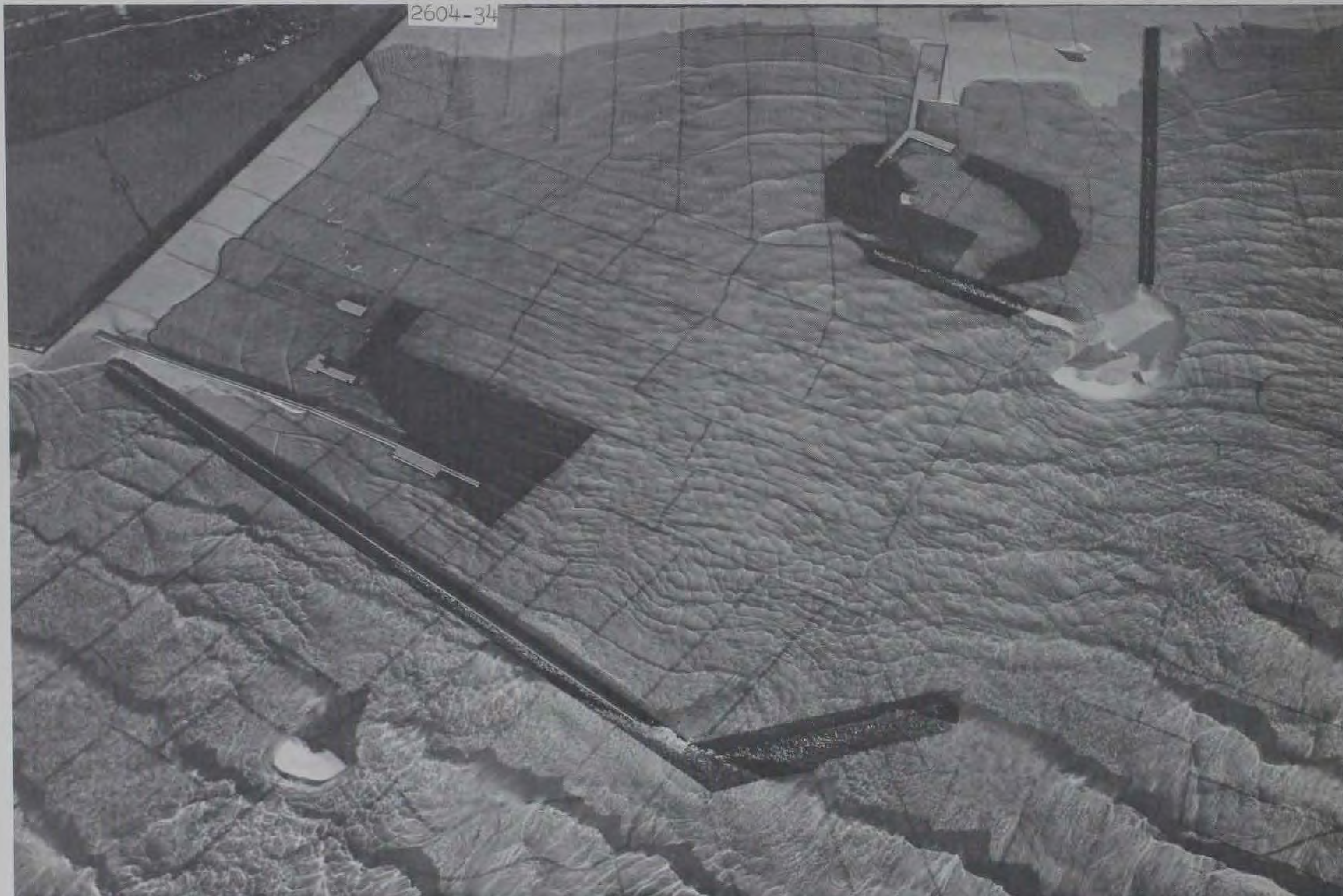


Photograph 2. Wave patterns, base test; 9-sec, 12-ft shallow-water waves from S10°22'W

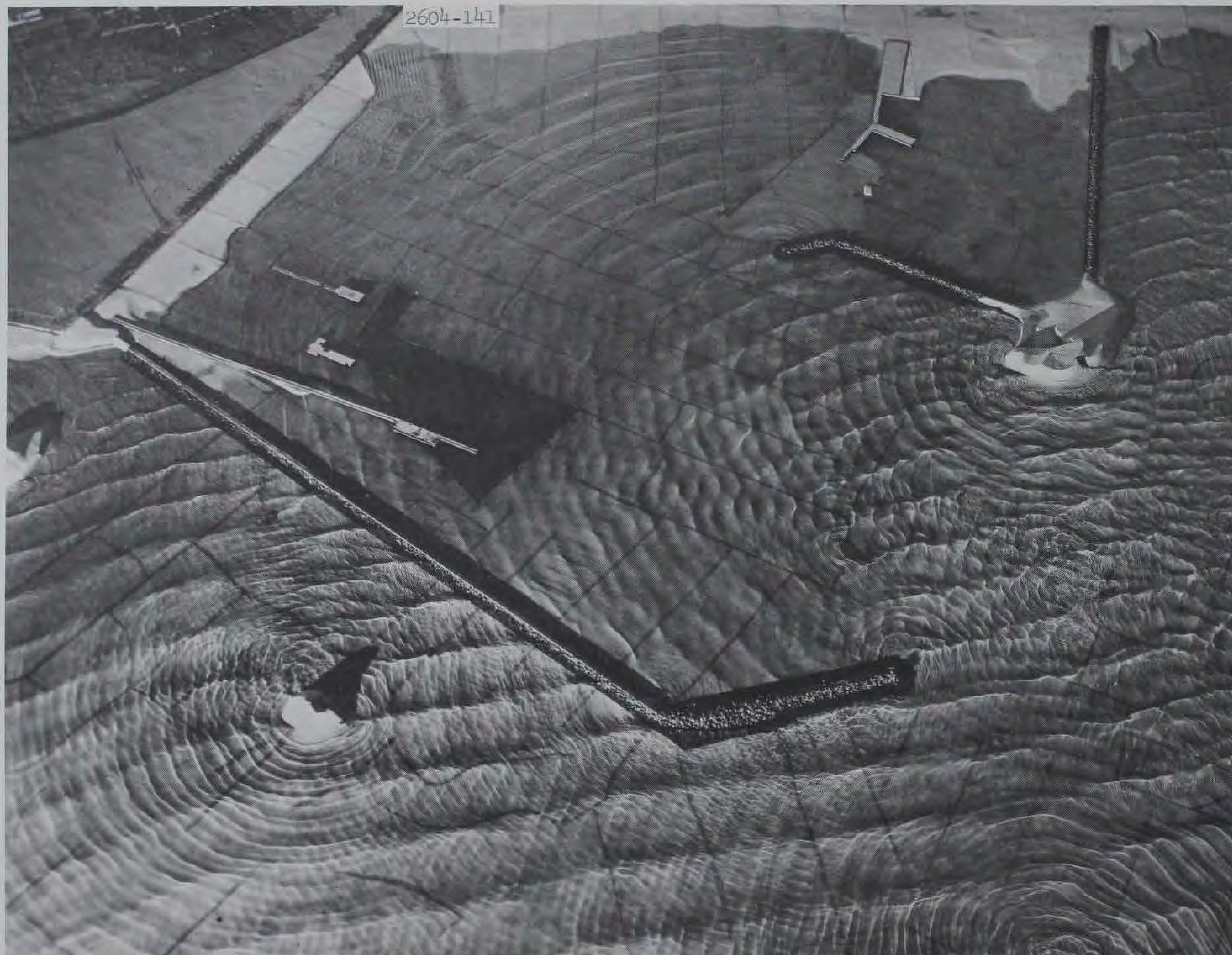


Photograph 3. Wave patterns, base test; 12-sec, 18-ft shallow-water waves from S35°40'W

Photograph 4. Wave patterns, base test; 14-sec, 22-ft shallow-water waves from S49°15'W



Photograph 5. Wave patterns, base test; 14-sec, 22-ft shallow-water waves from S63°25'W



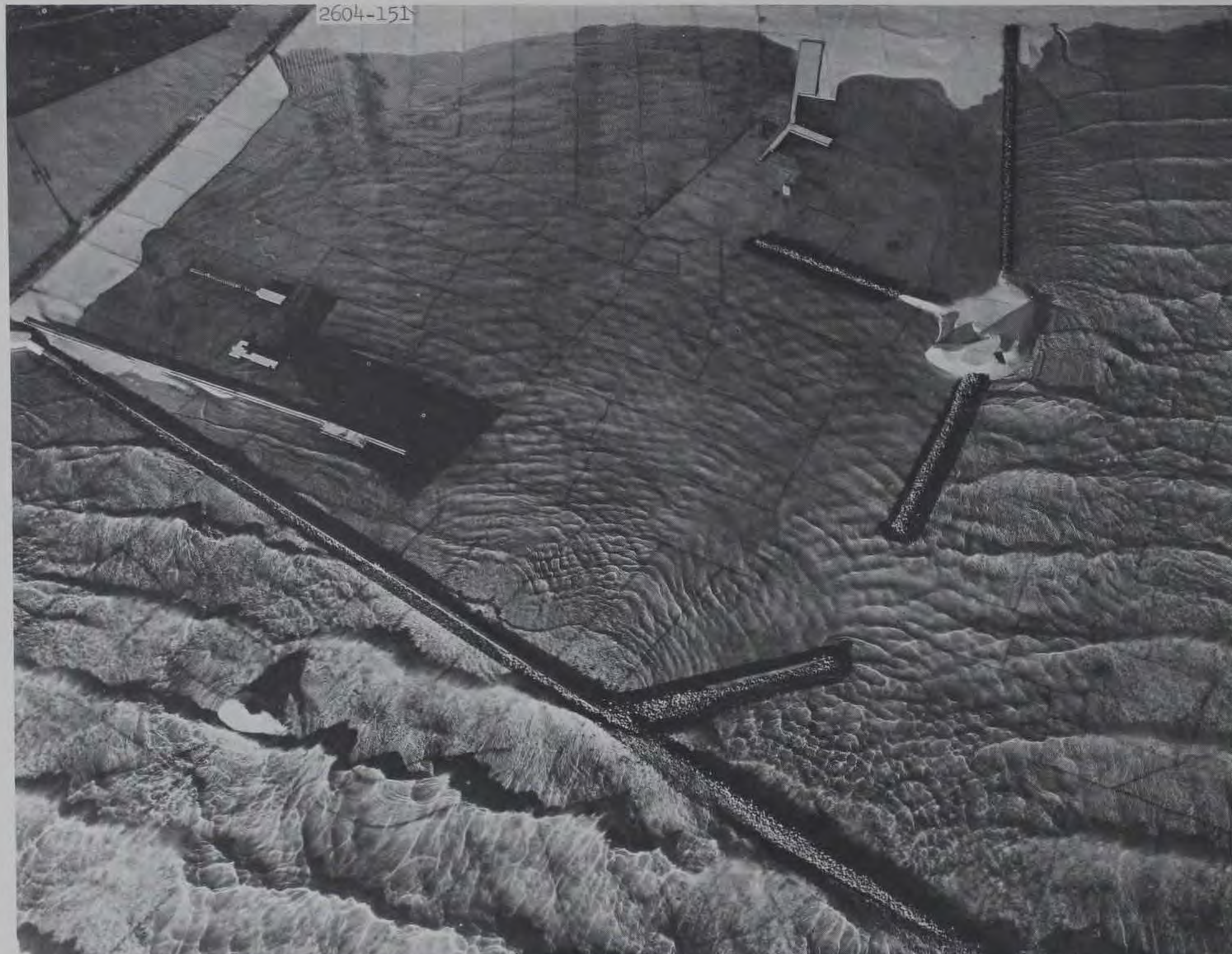
Photograph 6. Wave patterns, plan 6; 9-sec, 12-ft shallow-water waves from S10°22'W

Photograph 7. Wave patterns, plan 6A; 9-sec, 12-ft shallow-water waves from S10°22'W

Photograph 8. Wave patterns, plan 1; 9-sec, 12-ft shallow-water waves from S10°22'W

Photograph 9. Wave patterns, plan 1; 14-sec, 22-ft shallow-water waves from S49°15'W

Photograph 10. Wave patterns, plan 4; 9-sec, 12-ft shallow-water waves from S10°22'W



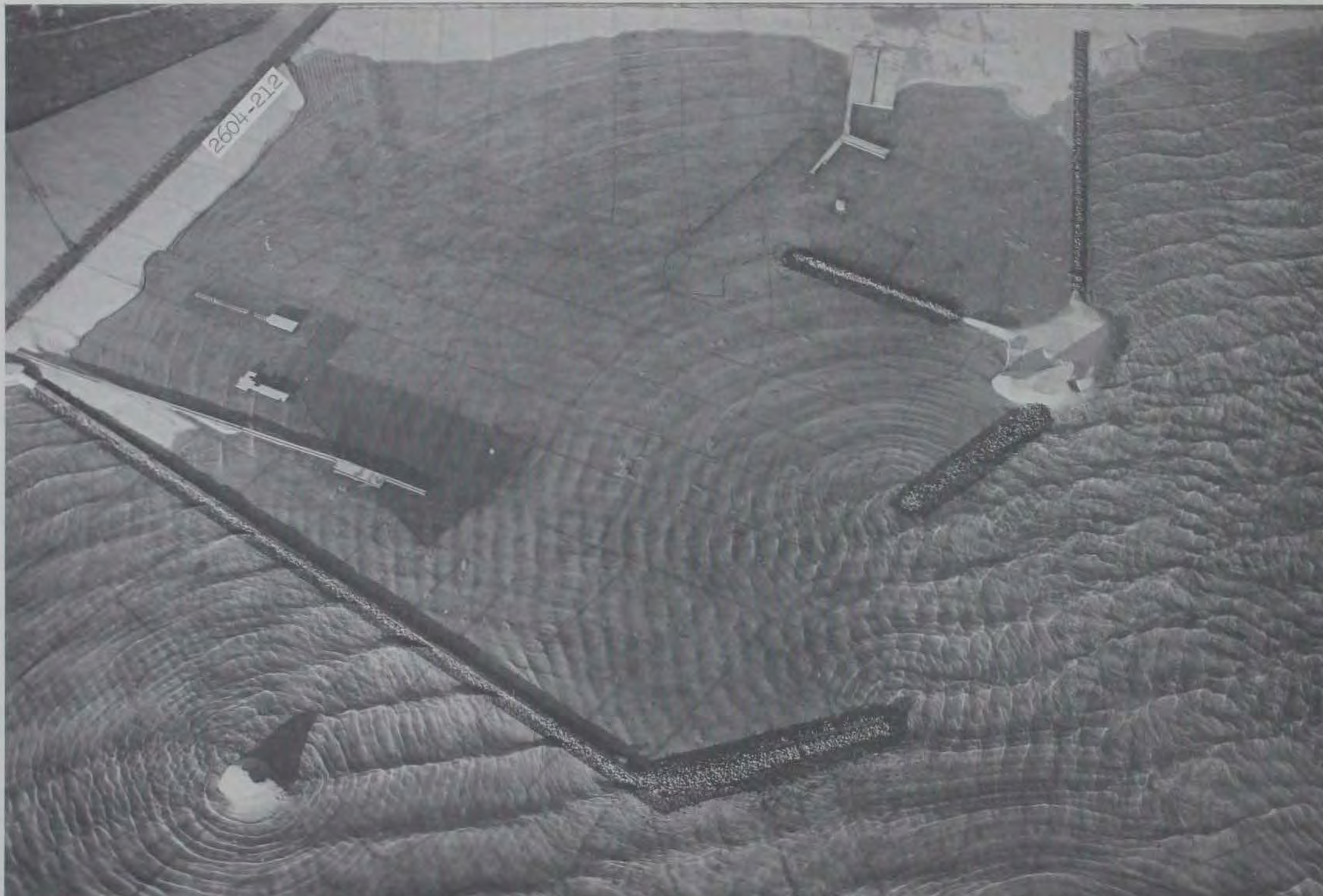
Photograph 11. Wave patterns, plan 4; 14-sec, 22-ft shallow-water waves from S49°15'W

Photograph 12. Wave patterns, plan 5; 14-sec, 22-ft shallow-water waves from S49°15'W

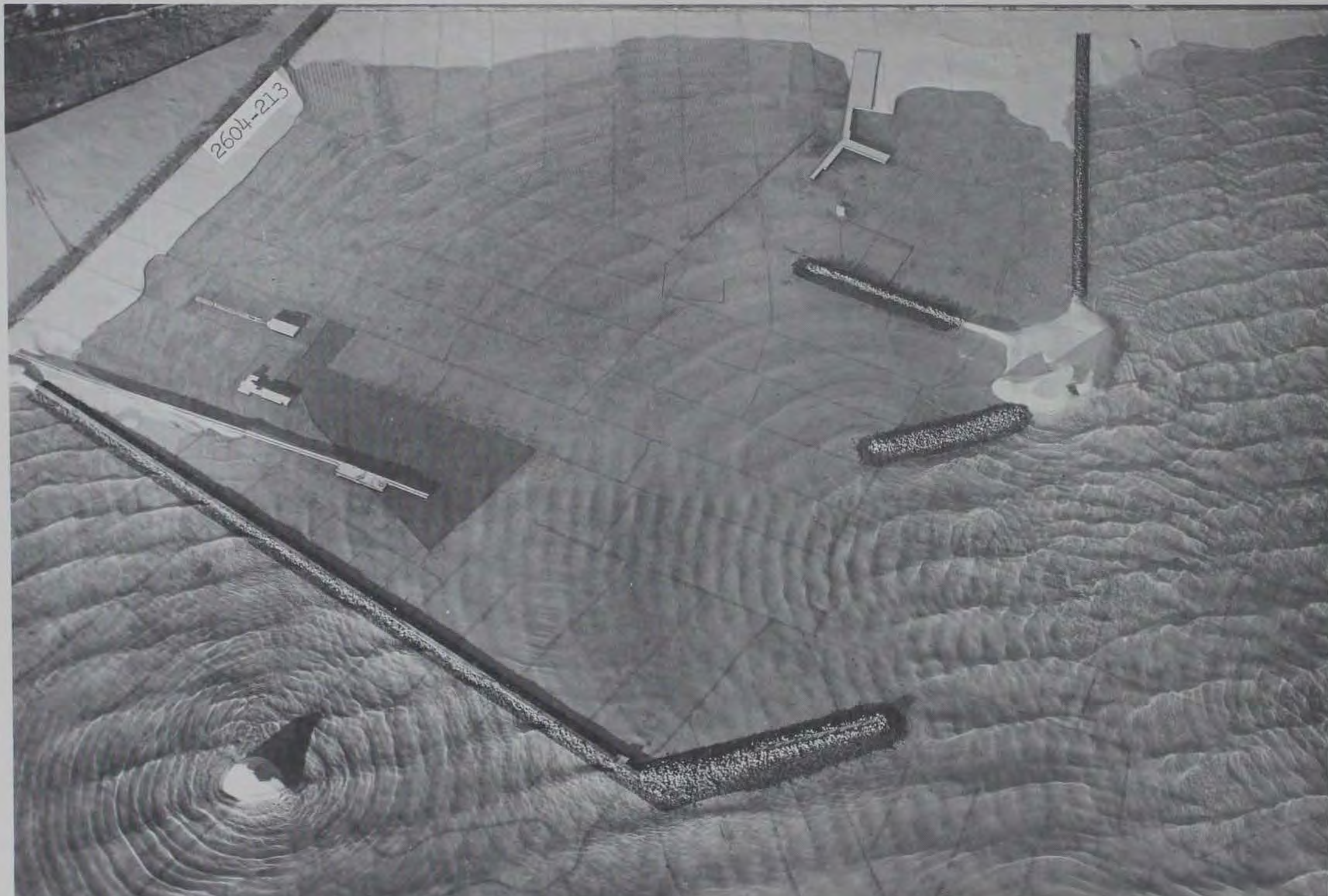
Photograph 13. Wave patterns, plan 5A; 14-sec, 22-ft shallow-water waves from $S49^{\circ}15'W$

Photograph 14. Wave patterns, plan 7; 14-sec, 22-ft shallow-water waves from S49°15'W

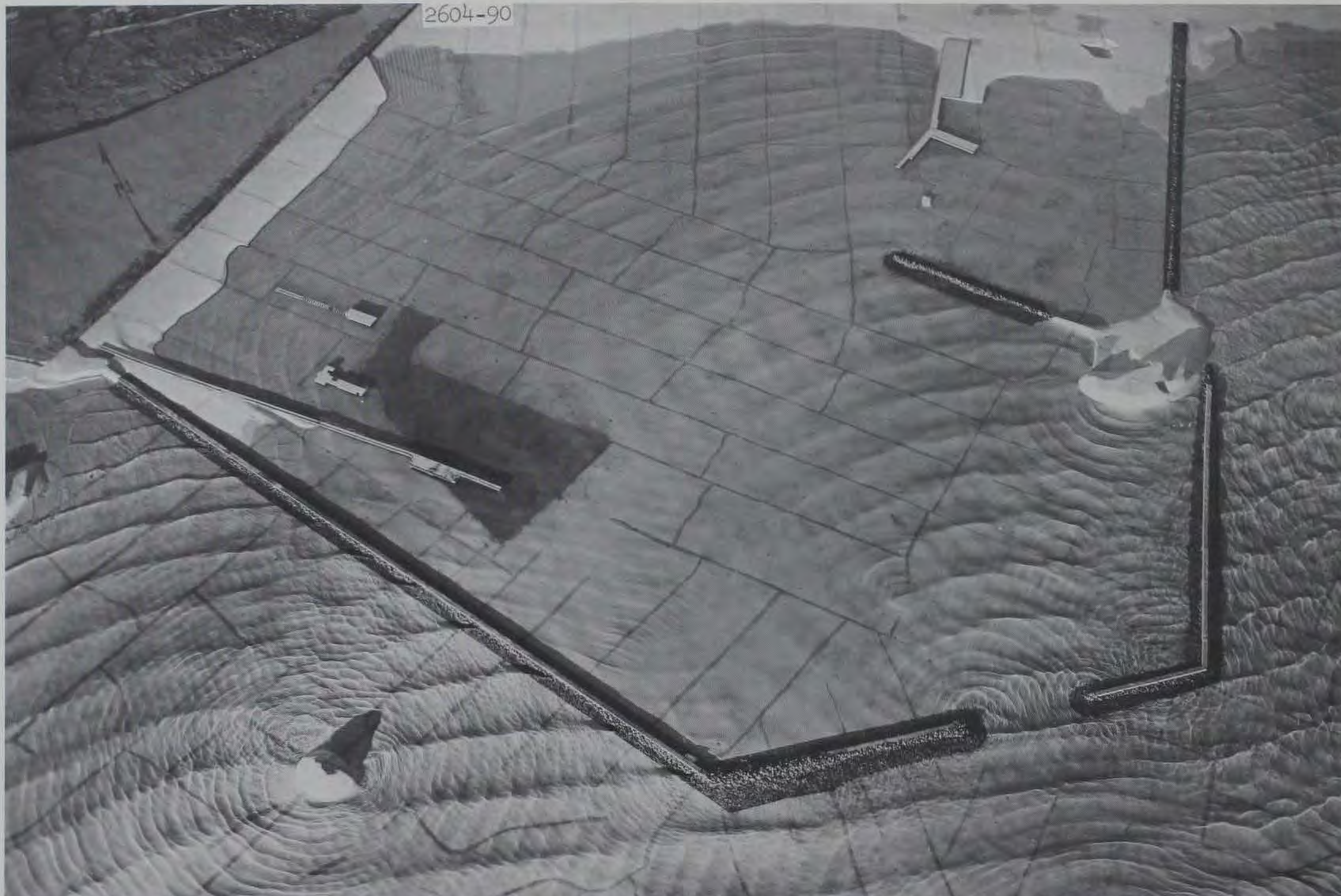
Photograph 15. Wave patterns, plan 8; 9-sec, 12-ft shallow-water waves from S10°22'W



Photograph 16. Wave patterns, plan 8A; 9-sec, 12-ft shallow-water waves from S10°22'W



Photograph 17. Wave patterns, plan 8B; 9-sec, 12-ft shallow-water waves from S10°22'W

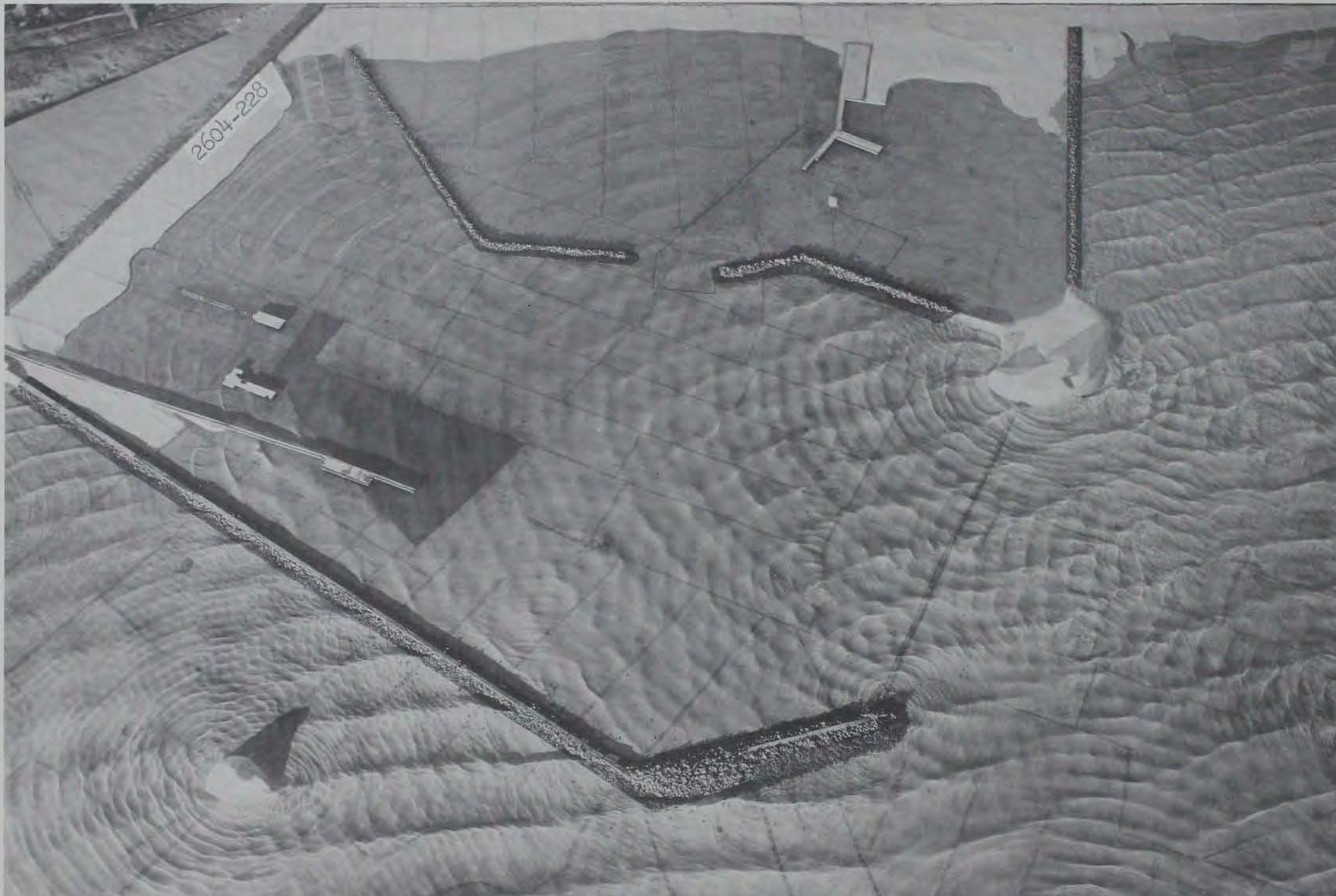


Photograph 18. Wave patterns, plan 2; 9-sec, 12-ft shallow-water waves from S10°22'W

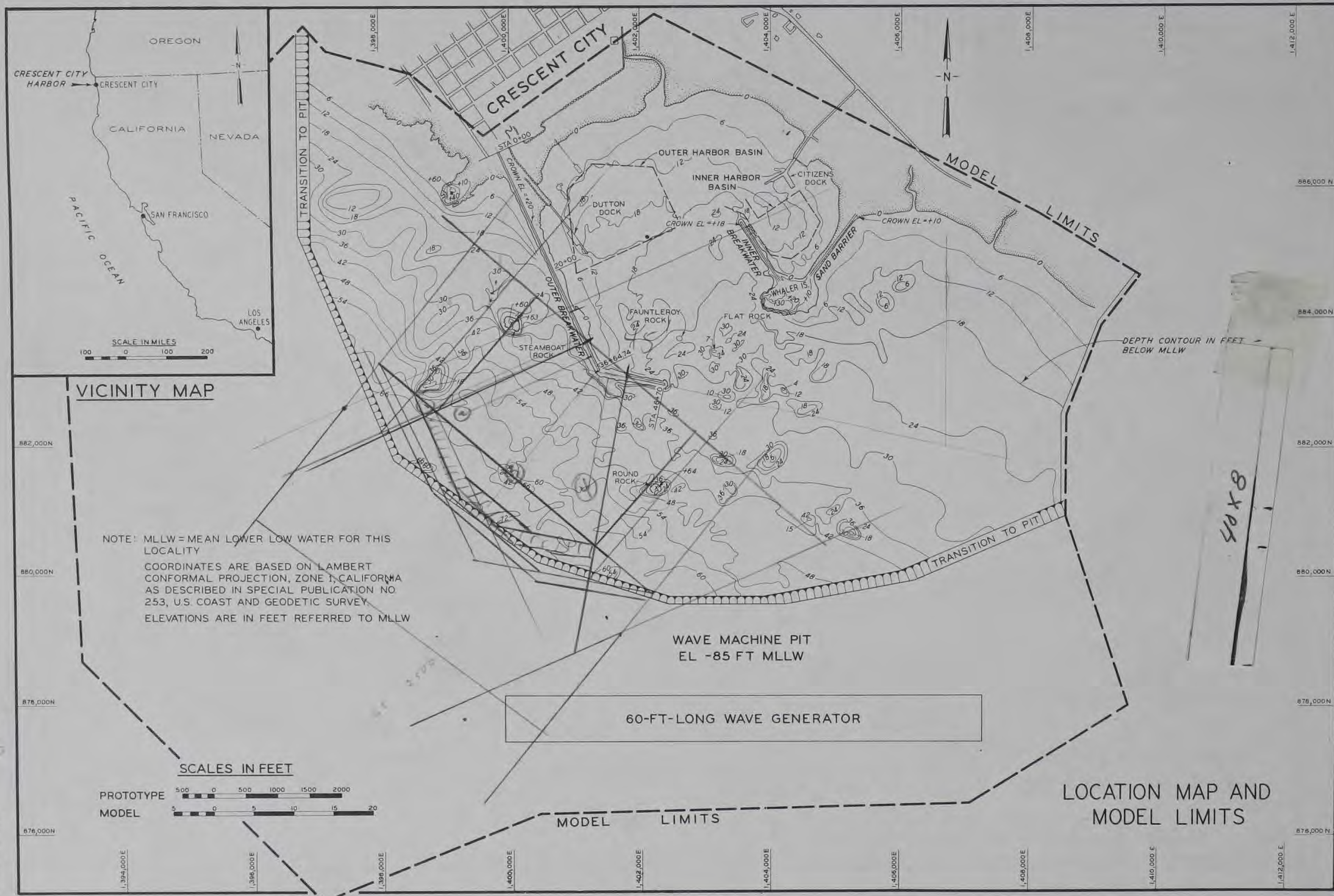
Photograph 19. Wave patterns, plan 2; 14-sec, 22-ft shallow-water waves from S49°15'W

Photograph 20. Wave patterns, plan 2A; 14-sec, 22-ft shallow-water waves from S49°15'W

Photograph 21. Wave patterns, plan 3; 12-sec, 16-ft shallow-water waves from S50°00'W



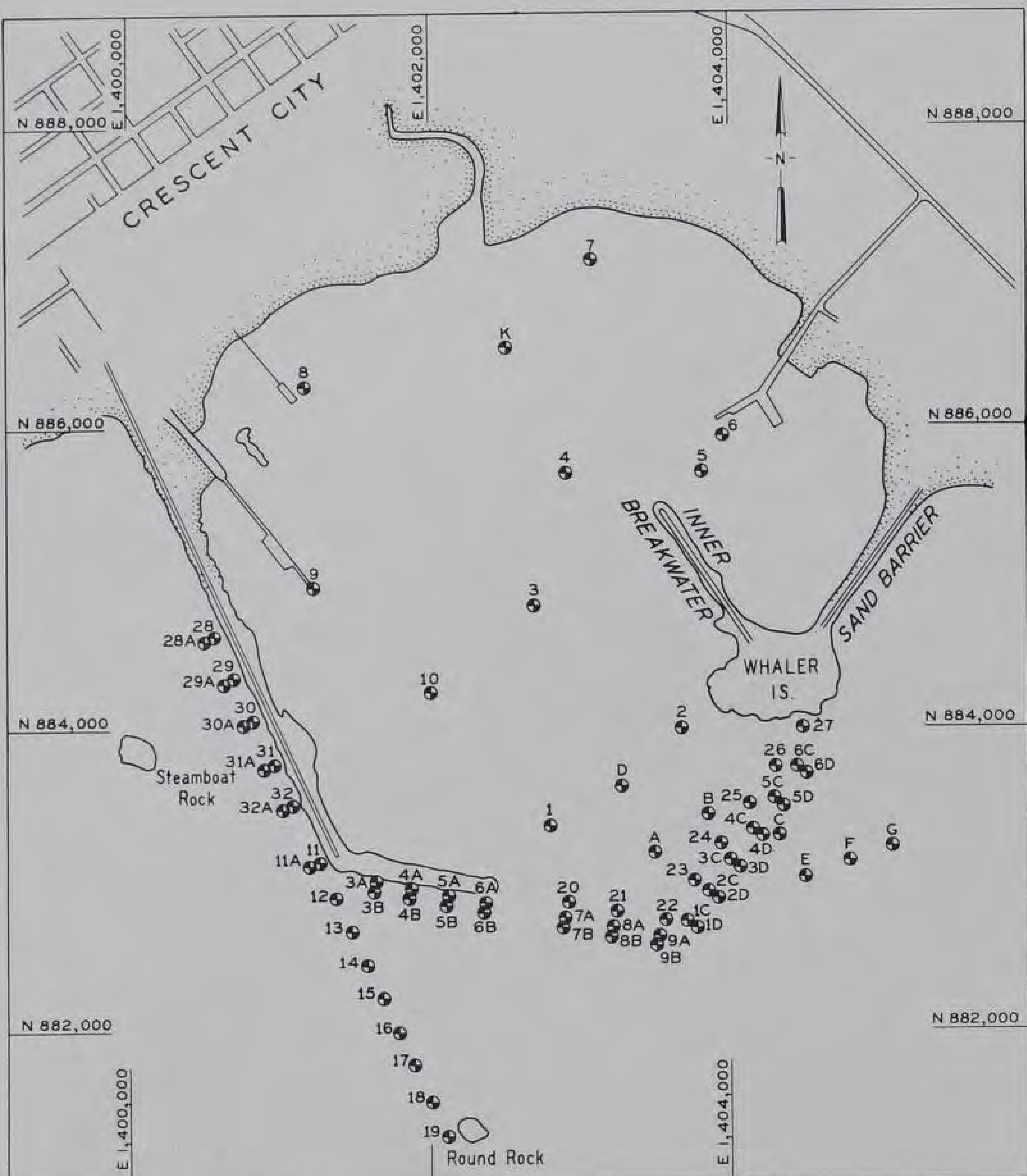
Photograph 22. Wave patterns, plan 9; 9-sec, 12-ft shallow-water waves from S10°22'W



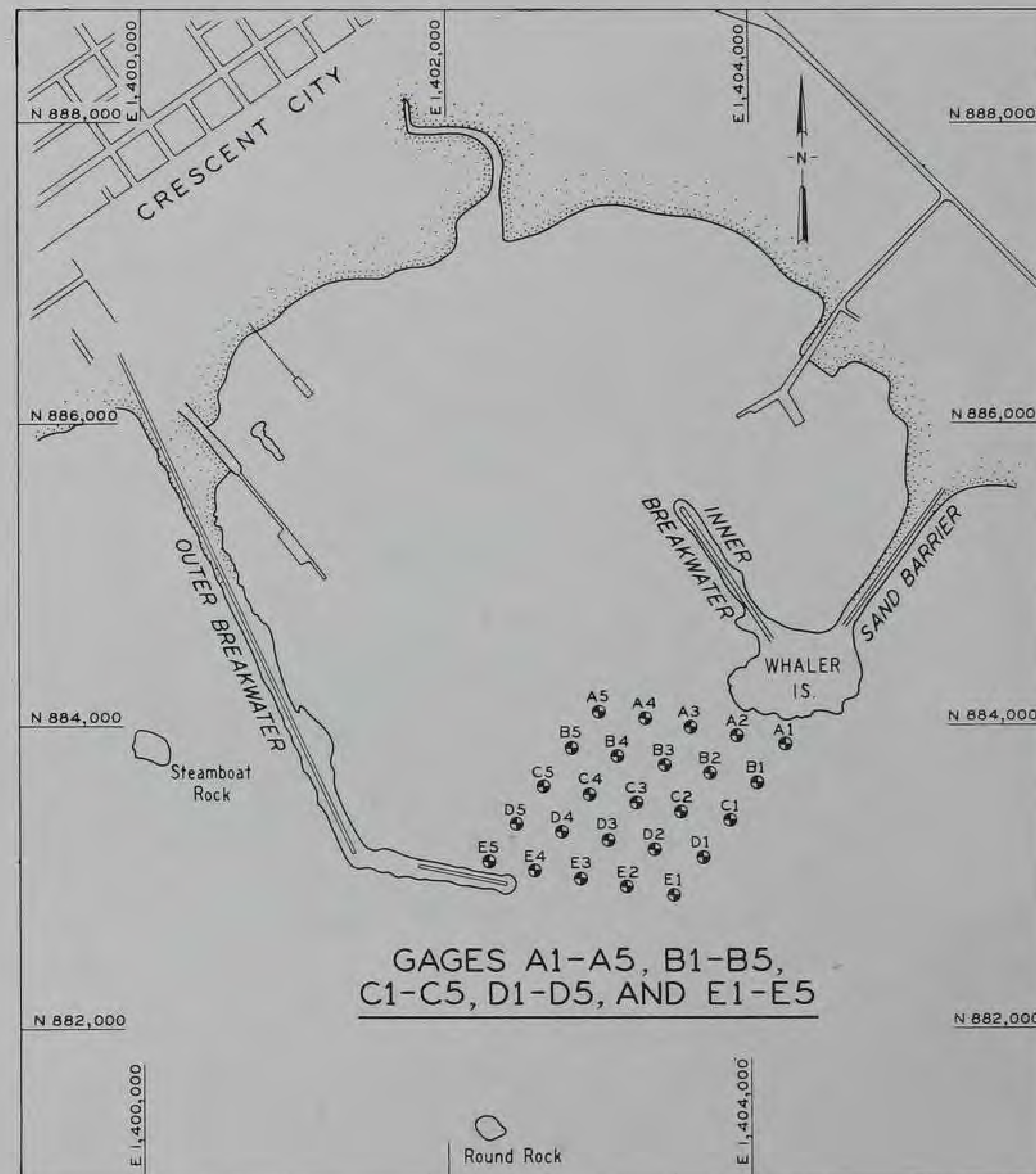
LOCATION MAP AND MODEL LIMITS

PLATE 1

PLATE 2



GAGES 1-32, A-G, K, 3A-9A, 11A, 28A-32A, 3B-9B, 1C-6C, AND 1D-6D



GAGES A1-A5, B1-B5, C1-C5, D1-D5, AND E1-E5

LEGEND

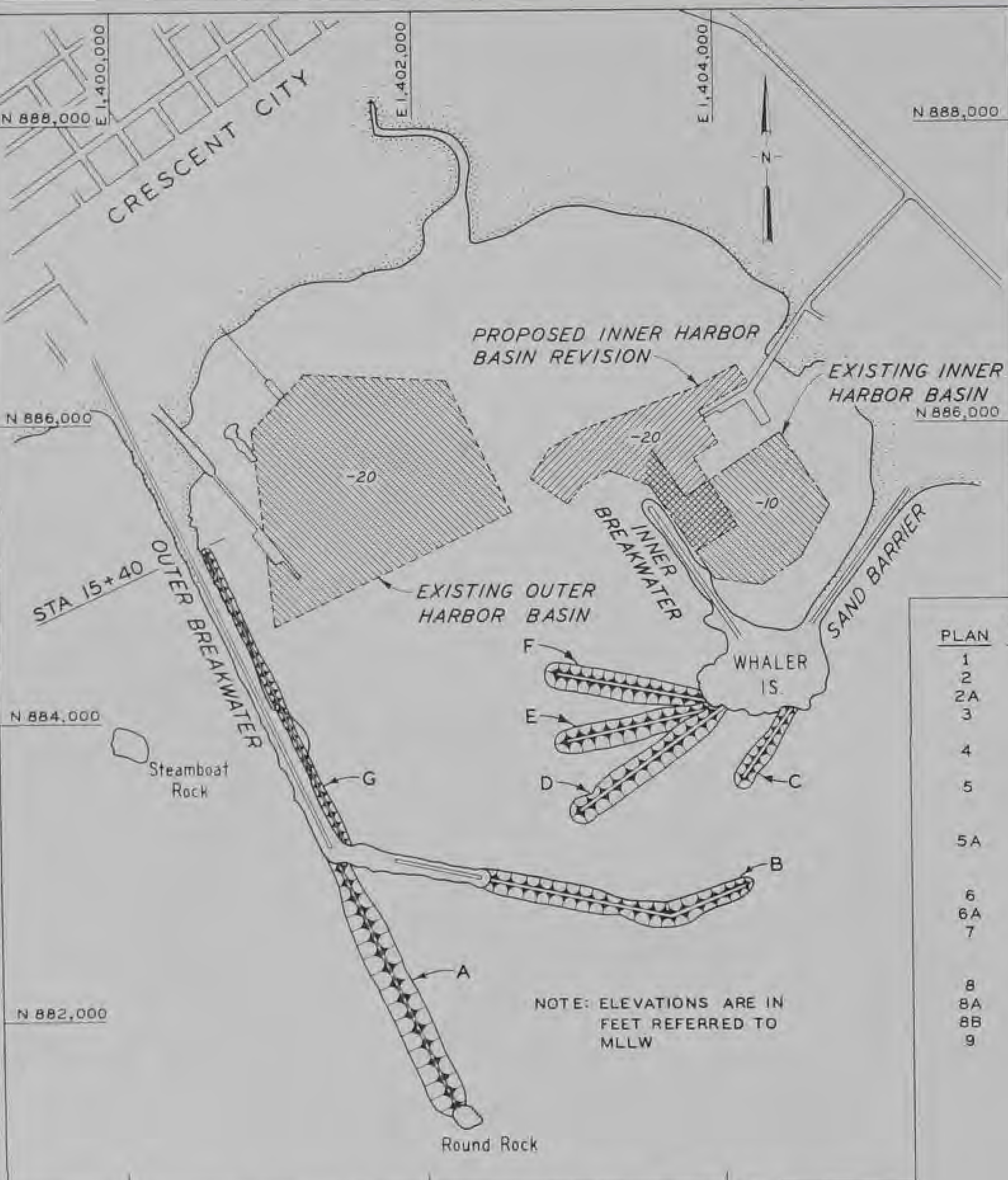
3 WAVE GAGE

LOCATION OF WAVE GAGES

SCALES IN FEET

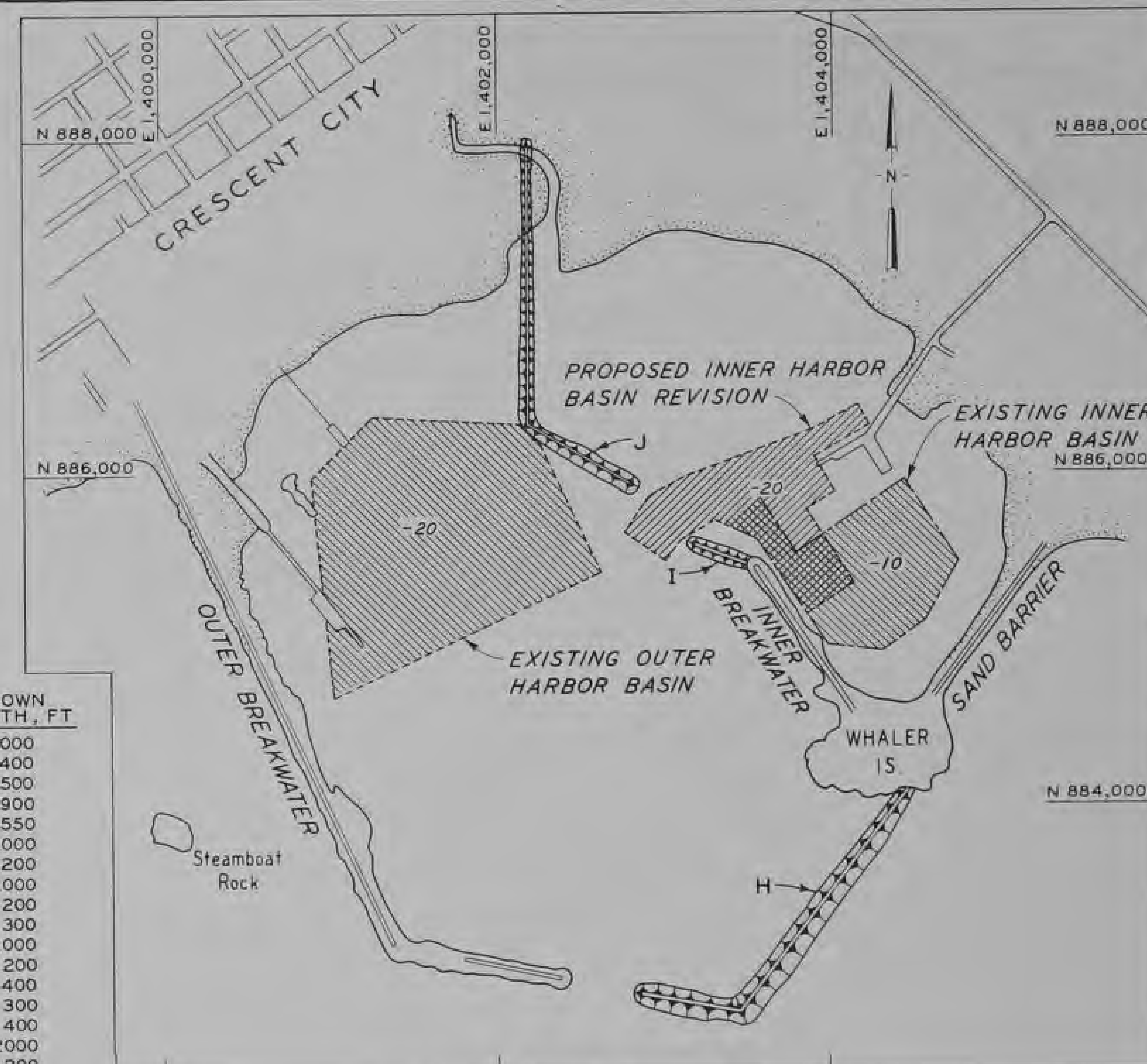
PROTOTYPE 500 0 500 1000 1500 2000

MODEL 5 0 5 10 15

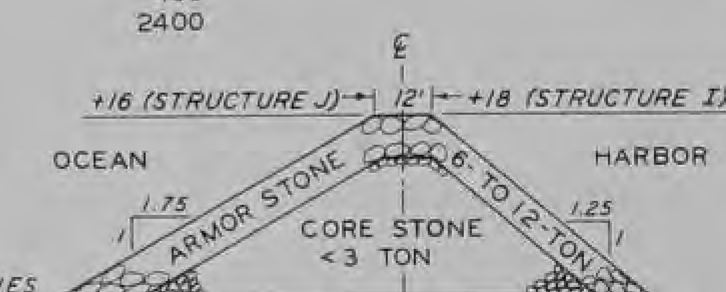


STRUCTURES A, B, C, D, E, F, & G

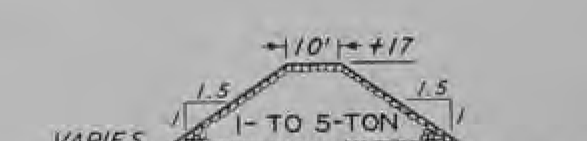
PLAN	STRUCTURES INCLUDED	CROWN LENGTH, FT
1	A	2000
2	H	2400
2A	H	2500
3	B	1900
	C	550
4	A	2000
	D	1200
5	A	2000
	D	1200
	I	300
5A	A	2000
	D	1200
	I	400
6	I	300
6A	I	400
7	A	2000
	D	1200
	G	2200
8	D	1200
8A	E	1000
8B	F	1000
9	I	400
	J	2400



STRUCTURES H, I, & J

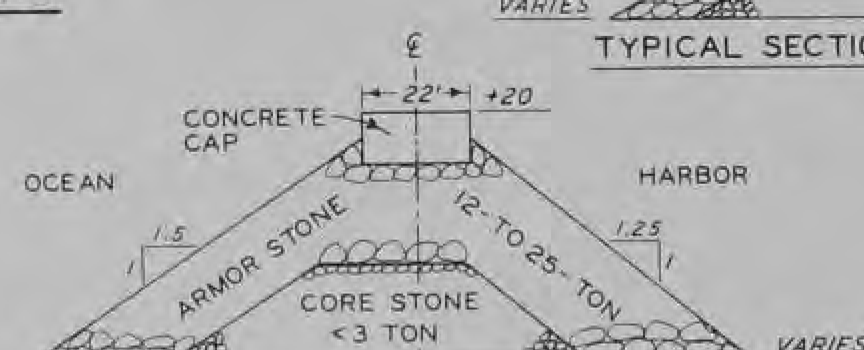


TYPICAL SECTION STRUCTURES I&J



TYPICAL SECTION STRUCTURE G

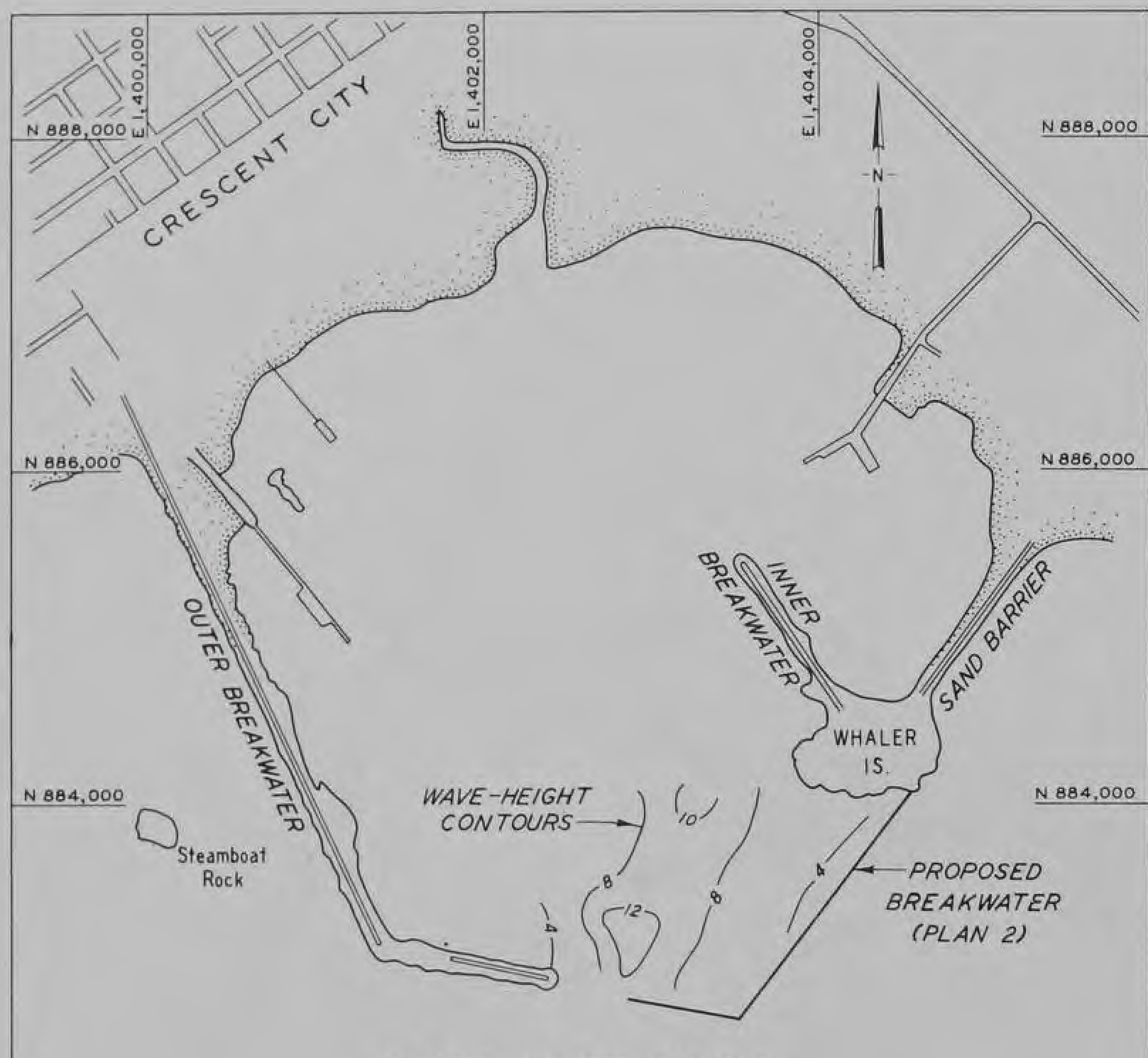
TYPICAL SECTION STRUCTURES A,D,E,&F



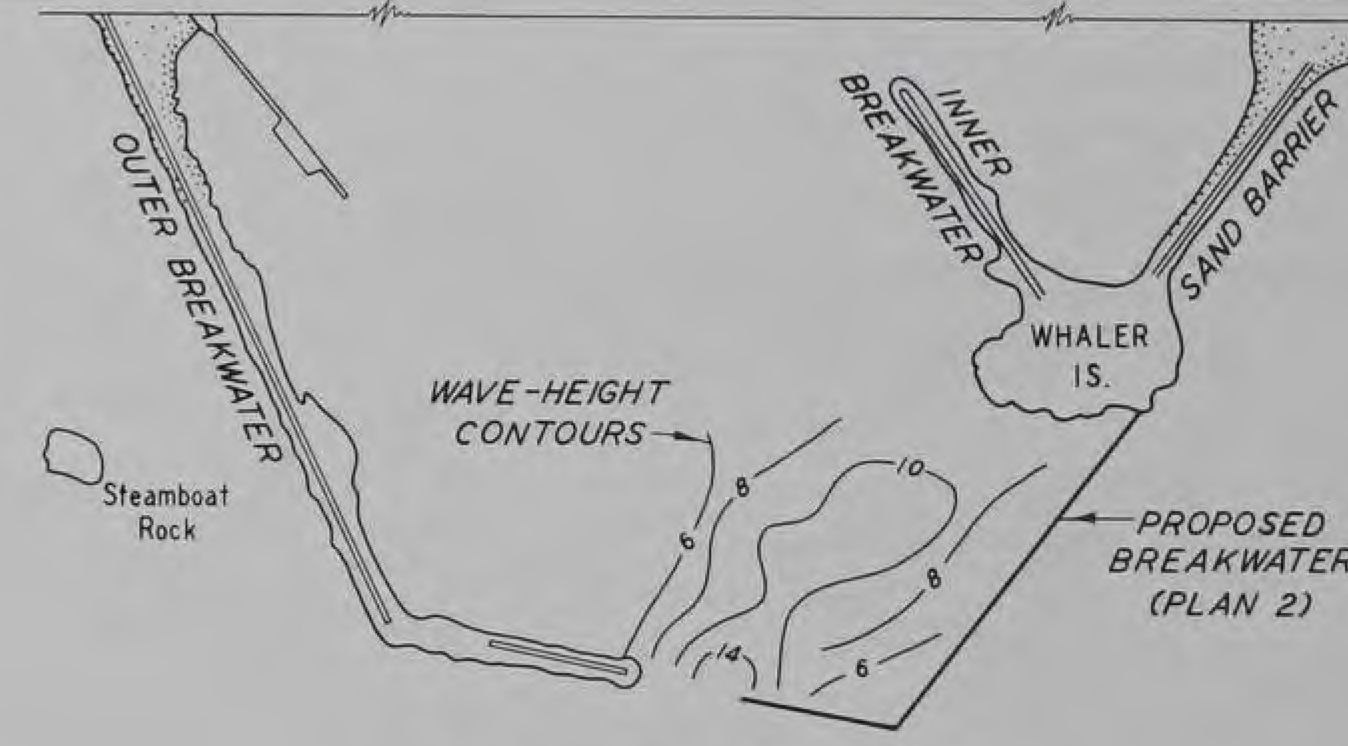
TYPICAL SECTION STRUCTURES B,C,&H

ELEMENTS OF BREAKWATER PLANS

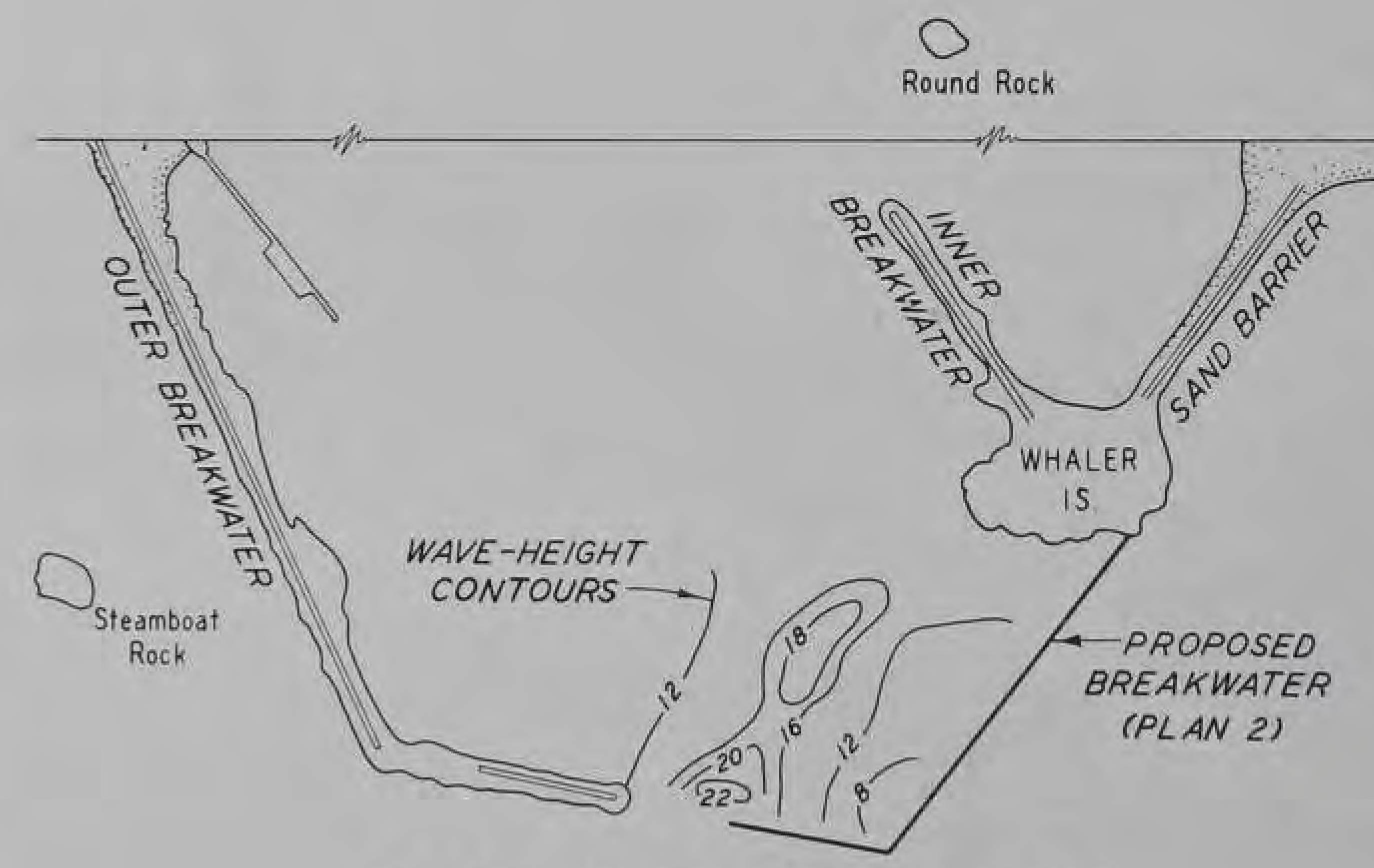
SCALES IN FEET



9-SEC, 14-FT WAVES FROM SOUTH

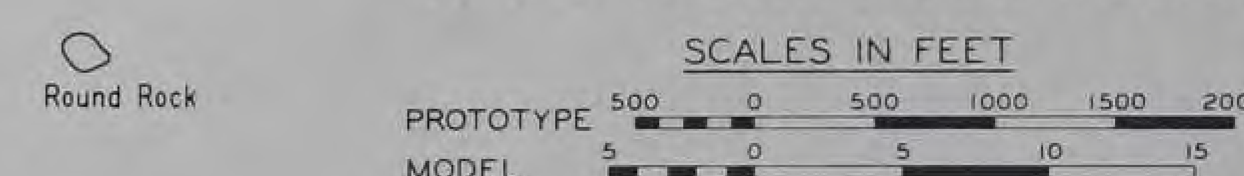


9-SEC, 14-FT WAVES FROM SOUTHWEST



14-SEC, 18-FT WAVES FROM SOUTH-SOUTHWEST

WAVE HEIGHT PATTERNS



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13. ABSTRACT

Tests were conducted on a 1:125-scale model of Crescent City Harbor and sufficient adjacent coastline and offshore bathymetry to permit generation of waves and wave-front patterns from all significant directions of wave approach to the harbor. The hydraulic model, equipped with wave-generating and wave-measuring apparatus, was used to determine the optimum length and location of an extension, or extensions, to the existing breakwater system that would reduce to a tolerable level the present adverse effects of storm waves on navigation and mooring conditions in the harbor. It was concluded that (a) wave action could be reduced to a satisfactory level in the inner harbor basin by installation of a 400-ft-long northwesterly extension of the inner breakwater; and (b) a 2000-ft extension of the existing outer breakwater to Round Rock, with a 1200-ft-long companion breakwater extending from Whaler Island, would substantially improve navigation and mooring conditions in the harbor.

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14. KEY WORDS	LINK A		LINK B		LINK C	
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Crescent City, Calif.--Harbor						
Hydraulic Models						
Water Wave Experiments						