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TECHNICAL REPORT H-68-9

WAVE AND SURGE CONDITIONS AFTER PROPOSED EXPANSION OF MONTEREY HARBOR, MONTEREY, CALIFORNIA

Hydraulic Model Investigation

by

C. E. Chatham, Jr.

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

A request for the U. S. Army Engineer Waterways Experiment Station (WES) to conduct a hydraulic model investigation of Monterey Harbor, California, 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 (SPD), dated 7 June 1963, subject, "Monterey Bay (Monterey Harbor), California, Request for Authority to Perform Model Study." Authority to conduct the study was granted by the Office, Chief of Engineers (OCE), on 25 June 1963, by the 2d indorsement to this letter.

The model study was conducted during the period May 1966 to November 1967 in the Harbor Wave Action Section, Water Waves Branch, Hydraulics Division, WES, under the direction of Mr. E. P. Fortson, Jr., Chief of the Hydraulics Division, and Mr. R. Y. Hudson, Chief of the Water Waves Branch. The tests were conducted by Messrs. H. B. Wilson and C. E. Chatham, Jr., project engineers, assisted by Messrs. N. R. Oswalt, engineer, and E. H. Brasfield, engineering technician, under the supervision of Dr. A. M. Kamel, consultant to the Hydraulics Division. This report was prepared by Mr. Chatham. Appendix A was prepared by Dr. Kamel.

During the course of the investigation liaison was maintained between the SFD and the WES by means of conferences, telephone communications, and monthly progress reports.

The following personnel visited the WES to observe model operation and participate in conferences: Mr. C. E. Lee, OCE; Mr. O. F. Weymouth, SPD; LTC H. McK. Roper, Jr., Deputy District Engineer, and Messrs. H. E. Pape, Jr., R. E. Blyberg, O. T. Magoon, N. Shimizu, and E. M. Huggins, SFD; Mr. J. M. Caldwell, Coastal Engineering Research Center; Dr. B. W. Wilson,

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Science Engineering Associates; Dr. R. E. Kent, Oceanographic Services, Inc.; Mr. R. P. Lundin, Koebig and Koebig, Inc.; Mr. F. D. Eastwood, 12th U. S. Coast Guard District; Dean C. E. Menneken, CDR W. S. Mitter, and CDR D. W. Urish, U. S. Navy Postgraduate School; Dr. J. P. Murtha, University of Illinois; Mr. L. M. Richards, Division of Harbor and Water Craft, State of California; Mrs. M. D. Coyle, Mayor, and Messrs. J. G. Ansel, W. D. Curtis, J. H. Nail, J. E. Logan, L. B. Bowhay, and L. W. McIntyre, City of Monterey; and Mr. F. K. Arthur, Jr., Publisher of the Monterey Peninsula Herald.

COL John R. Oswalt, Jr., CE, and COL Levi A. Brown, CE, were Directors of the WES during the conduct of the model study and the preparation 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
inches	2.54	centimeters
feet	0.3048	meters
square feet	0.092903	square meters
acres	4046.9	square meters
square miles	2.58999	square kilometers
tons	907.185	kilograms
feet per second	0.3048	meters per second
miles	1.609344	kilometers

SUMMARY

A 1:120-scale model of Monterey Harbor, California, and sufficient offshore area to permit generation of the required test waves was used to investigate the arrangement and design of certain proposed harbor improvements with respect to wave and surge action and to determine current conditions in the navigation entrances to the harbor and its basins. The proposed harbor improvements consisted of (a) enlarging the present harbor by construction of a detached north breakwater, approximately 3350 ft in length, and a companion east breakwater connected to shore and extending approximately 1100 ft seaward; and (b) development of the inner-harbor area by constructing moles to form two additional basins for the anchorage of small pleasure craft.

A 56-ft-long wave machine and electrical wave height measuring and recording apparatus were utilized in model operation. Base tests were conducted with existing prototype conditions installed in the model. Results of tests involving the various improvement plans were compared with base test results to determine the relative effectiveness of the various plans.

It was concluded from the test results that (a) either the single-entrance or the double-entrance plan will provide an improvement over existing conditions with respect to long-period surge in the harbor; (b) although the harbor basins respond to several of the wave periods tested, no serious cases of resonance were noted; and (c) either the single-entrance or the double-entrance plan will provide sufficient protection to the inner basins from short-period (5 to 20 sec) waves, except in a portion of the east basin.

An analytical study of long-period sea-energy oscillations in the vicinity of Monterey Bay with respect to the possibility of related response in Monterey Harbor was conducted, and the results of that study are presented in Appendix A.

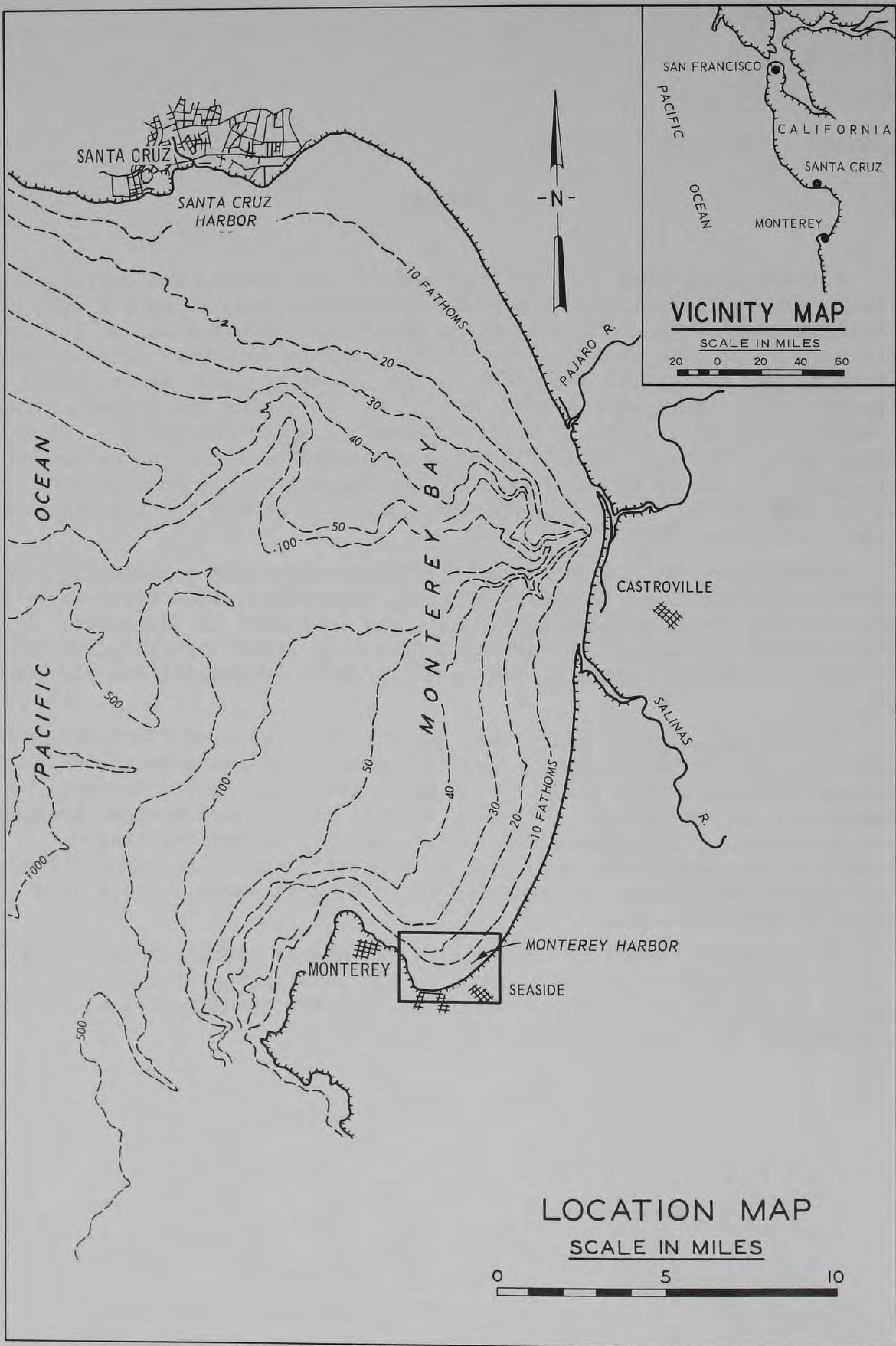

Fig. 1. Location and vicinity maps of Monterey Bay and surrounding area

WAVE AND SURGE CONDITIONS AFTER PROPOSED EXPANSION OF MONTEREY HARBOR MONTEREY, CALIFORNIA

Hydraulic Model Investigation

PART I: INTRODUCTION

Description of Prototype

Existing conditions

1. Monterey Harbor is located at the southern end of Monterey Bay and is about 100 miles* south of San Francisco (fig. 1). Monterey Bay is a large semielliptical body of water open to the Pacific Ocean on the west; hence, to varying degrees, shoreline locations are exposed to waves arriving from several directions. An important feature of Monterey Bay is that the deep trough of the North Pacific basin south of the Mendocino sea scarp approaches closer to shore at Monterey than at any other point along the North American coastline. This trough is bounded on the southern side by the Murray sea scarp and is therefore something of a deep-walled channel running in a west-east direction up to the comparatively narrow continental shelf off Monterey.

2. In the early 1900's, two wharfs were constructed to provide support to the fishing fleet of Monterey. However, these wharfs were fully exposed to the weather, and it was not until 1934 that a 1700-ft breakwater was constructed by the Federal Government. With this basic construction, Monterey Harbor took form, encompassing about 80 acres of water area that was utilized primarily for offshore mooring of the fishing fleet. However, due to the reduced fish catch in recent years, the activities of Monterey Harbor have been reoriented to increased recreational boating and tourist attractions in addition to the existing commercial and sports fishing activities. In 1960, an impervious bulkhead

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

wall with a 42-ft navigation opening was constructed between Municipal Wharfs No. 1 and No. 2 to provide a sheltered 350-berth marina for the anchorage of small pleasure craft. The principal features of the present harbor are shown in plate 1.

Background information

3. Following construction of the existing breakwater in 1934, it became evident that the 1700-ft structure did not provide sufficient mooring area for the large number of fishing craft based in the harbor at that time, nor was adequate protection afforded from wave and surge action. Several proposals were advanced over the ensuing years for expansion of the harbor and increased protection for marine craft and harbor facilities. One such proposal resulted in the conduct of a wave-action model study of Monterey Harbor[1] at the U. S. Army Engineer Waterways Experiment Station (WES) during the period 1946-1948, the purposes of which were to provide design data on means of protecting the harbor from both long- and short-period waves and to determine the optimum orientation and configuration of a proposed breakwater system for enlarging the harbor. Results of that investigation, which was primarily concerned with protection of the fishing fleet, indicated that adequate protection from short-period waves could be obtained, but means of providing adequate protection from long-period waves remained unresolved. Later, a plan for harbor expansion and improvement similar to one deemed satisfactory by the model study reported in reference 1 was recommended[2] and authorized for construction. However, no construction has been initiated as yet. This may be a fortunate situation in view of the reorientation of the principal harbor usage from commercial fishing to pleasure boating, because of the fact that pleasure craft require a greater degree of protection from wave action and a more tranquil mooring area than do commercial fishing craft.

Proposed improvements

4. It is presently proposed that the existing harbor at Monterey be enlarged by the construction of one or more additional breakwater structures to provide safe anchorage for the commercial fishing fleet and to provide additional berthing facilities for pleasure craft. The inner-harbor area would be developed by the construction of several moles that

would provide additional shelter to the small-craft berthing facilities and provide land areas suitable for resort motels, restaurants, and related commercial activities. The inner-basin improvement plans for Monterey Harbor were developed by Koebig and Koebig, Inc., Los Angeles, California.

The Problem

5. Monterey Harbor is exposed to short-period, distant-storm waves from the deepwater directions clockwise between west and northwest and local-storm waves from the north direction. These waves sometimes are of sufficient magnitude to damage fishing boats and harbor facilities, and cause mooring difficulties for small craft in exposed areas of the harbor. Also, long-period waves of considerable magnitude do occur in Monterey Bay, and it is known that such waves are capable, under certain circumstances, of a substantial increase in amplitude in some harbor areas due to resonance phenomena. Thus, it was desirable that the proposed breakwaters and inner-harbor structures be designed to provide the maximum protection from short-period waves at minimum cost, and further, that the proposed construction not amplify the long-period surge waves that occur in the harbor at the present time. Since it is still not possible to accurately predict the behavior of waves in a harbor by analytical methods, a recommendation for a hydraulic model investigation was included with the present proposal for expansion of the harbor.

6. Because of (a) the recognized need to resolve the wave and surge problems in Monterey Harbor prior to embarking upon a large construction program, (b) the increasing attention being focused on long-period wave phenomena in harbors in the last few years, and (c) the rapidly developing science of long-period wave and surge-action modeling and measurement, the San Francisco District (SFD) convened a meeting of several experts in July 1963 to devise a program of study, the results of which could be used to determine the feasibility of a hydraulic model study. As a consequence of this meeting, arrangements were made to have surface-wave motions recorded at three key locations in the harbor (sensors 1, 2, and 3; plate 1).

This study was conducted by Marine Advisers, Inc.,[3] in 1964. A further consequence of this meeting was the conduct of an analytical study by Dr. B. W. Wilson[4] to determine the feasibility of conducting a model study to resolve the long-period surge problems, using, among other things, data obtained in the 1964 field survey. Following these studies, a wave-action model of Monterey Harbor was constructed at the WES in 1966.

Purpose of Model Study

7. The purpose of the model study reported herein was to determine whether the proposed harbor revisions would provide adequate protection from both long- and short-period wave and surge action. It was desired that the long-period waves that occur in the harbor area not be amplified by resonance to such an extent that the resulting wave heights and currents in the navigation openings and inner-harbor basins would constitute a hazard to small craft. An additional objective of the model investigation was to determine whether suitable design modifications of the proposed plans could be made that would reduce construction costs significantly and still provide adequate protection from wave action.

Criteria for Judging the Adequacy of Harbor Plans

Long-period waves

8. At the present time, no established criteria are available from which satisfactory conditions in a small-craft harbor can be assured with respect to long-period wave action (waves with periods greater than about 25 sec). However, observations by SFD personnel in the small-craft harbor at Santa Cruz, California, indicate that waves with periods from 100 to 1000 sec and with heights from about 1.0 to over 4.0 ft occur frequently in that area, but that mooring conditions are considered satisfactory. Further, although there have been reported instances of difficulties resulting from surge currents in the entrance to the existing marina in Monterey Harbor, it is understood that, in general, navigation and mooring conditions in the marina are considered to be acceptable. Thus, for this

investigation, it was assumed that surge conditions in the existing marina, and in the proposed additional small-craft basins in Monterey Harbor, would be satisfactory if long-period wave heights and resulting currents in the existing and proposed basins and entrances do not exceed those that occur at the present time in the existing marina.

Short-period waves

9. Completely reliable criteria have not yet been developed that will ensure that satisfactory navigation and mooring conditions will obtain in small-craft harbors for short-period waves (waves with periods from about 5 to 20 sec). However, it is known that when resonant surge conditions occur for small craft moored in present-day marinas, small wave heights can result in the breaking of mooring lines when the craft are not moored correctly. For this study, it was assumed that satisfactory conditions would obtain if short-period wave heights do not exceed 1.5 ft in the inner basins and 4.0 ft at the basin entrances and in the fairway.

PART II: THE MODEL

Design of Model

10. At the time the present model investigation was authorized, it was believed that the science of designing and operating long-period wave models was not sufficiently advanced to ensure that sufficiently accurate results could be obtained. For this reason, the feasibility study conducted by Wilson[4] was authorized. The purpose of Wilson's study was to determine (a) the type of model that should be used to investigate the surge problems in Monterey Harbor; (b) how the model should be designed to ensure accurate results; (c) how to use the available prototype data to formulate a model testing program; and (d) how best to analyze the model test results to ensure selection of a satisfactory plan of harbor improvement.

11. From Wilson's results, it was concluded that the wave periods of concern were likely to be less than 3 min and certainly less than 7 min. Therefore, the vicinity of Mussel Point (about halfway between Point Pinos and Monterey, fig. 2) was selected as the seaward limit of the model. If the seaward boundary were taken closer to the harbor than this, the longer period oscillations (near 7 min) would have to be generated in tidal fashion by introducing and withdrawing water from the model. It was further concluded that the long-period wave energy coming across the rim of the deep submerged canyon on the northern edge of the continental shelf (for the southern part of the bay) is insignificant and that it would be unnecessary to generate long-period waves from this direction. Thus, a side boundary for the model, normal to the coast from near the inlet to the Laguna del Rey, would not seriously interfere with the oscillating regime, provided that sufficient wave-filter material was installed along the wall to prevent wave reflection. For the same reason, it was recommended that wave-filter material be placed in front of the wave generator. The recommended limits for a surge-action model of Monterey Harbor were established as shown in fig. 2 with two wave-generator units to reproduce the correct directions of approach of the long-period waves south of Mussel Point.

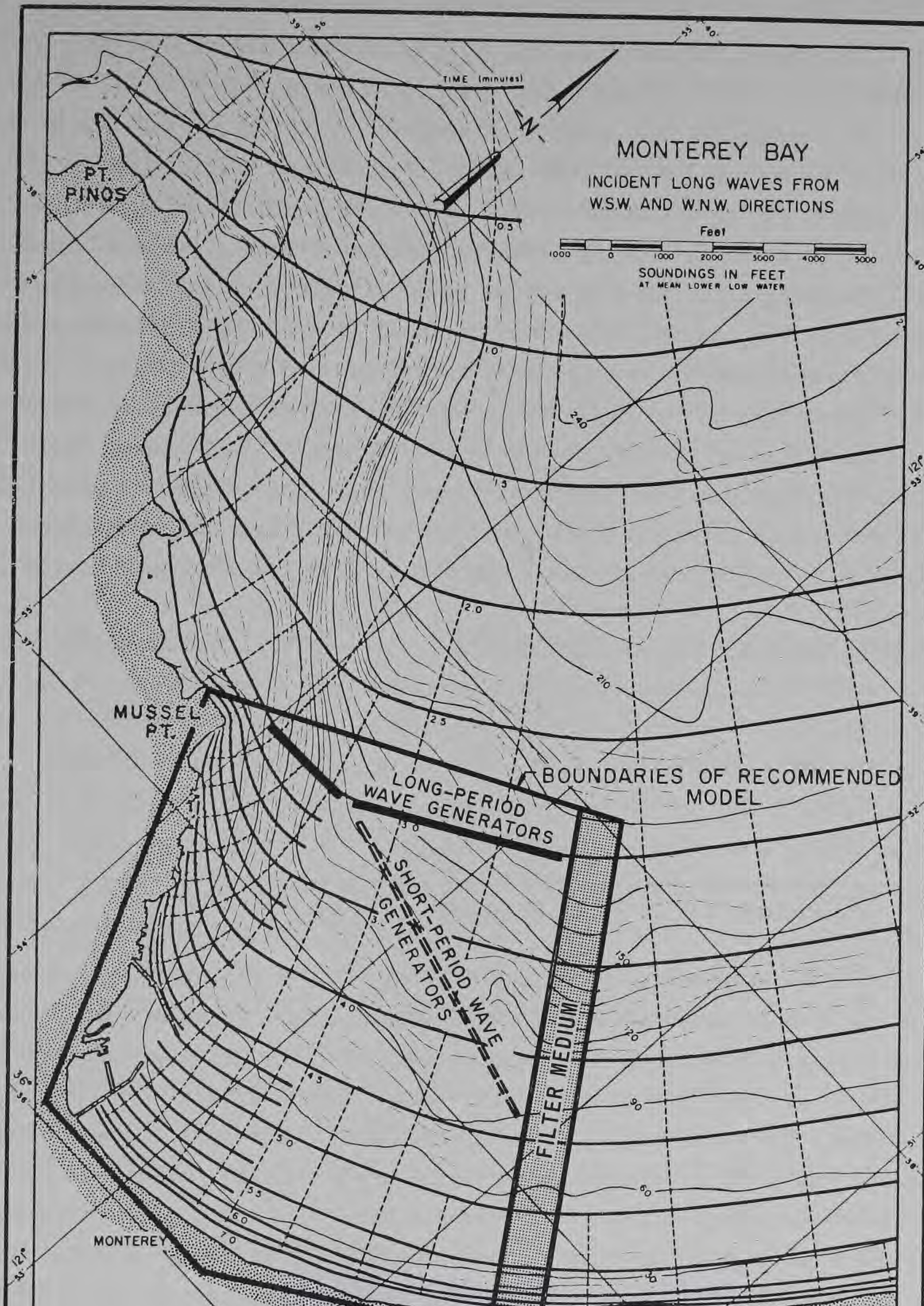

Fig. 2. Recommended boundaries of surge-action model for Monterey Harbor (after Wilson[4])

12. On the basis that the harbor area to be modeled was about 10^4 by 10^4 ft (prototype), as shown in fig. 2, a horizontal linear scale of 1:200 was suggested in order to bring the model to a convenient size of 50 by 50 ft. Also suggested was a vertical scale of 1:120, which would give a distortion factor of 1.67 for the model. The maximum water depth in the marina (about 16 ft) would then be about 0.13 ft in the model, which is considered an adequate working depth in that inner basin. Wilson[4] considered the 1.67 distortion factor as being satisfactory for reliable reproduction of long-period waves down to about 30 sec. However, because the model would also be used to study the effects of short-period waves (in the range of 5 to 20 sec), and because such waves can best be investigated in undistorted models, it was decided to use a 1:120 linear scale for both the horizontal and vertical dimensions. The design and operation of the model were based on the recommendations of Wilson and were in accordance with Froude's model law;[5] the scale relations used were as follows:

Characteristic	Dimension*	Model:Prototype Scale
Length	L	$L_r = 1:120$
Area	L^2	$A_r = L_r^2 = 1:14,400$
Volume	L^3	$V_r = L_r^3 = 1:1,728,000$
Time	T	$T_r = L_r^{1/2} = 1:10.95$
Velocity	L/T	$V_r = L_r^{1/2} = 1:10.95$

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

13. The proposed plans of improvement for Monterey Harbor included the use of rubble-mound breakwater structures. Past experience and experimental research have shown that considerable wave energy passes through the interstices of this type of structure. Thus, the transmission of wave energy through the rubble-mound structures became a matter of concern in the design of the 1:120-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,[6] but the absolute value of this model scale effect has not yet been established. Also, the transmission of energy through the breakwater is less (relatively) for

the small-scale model than for the prototype. Consequently, some adjustment in small-scale-model, rubble-mound structures is needed to ensure satisfactory reproduction of wave-transmission characteristics. On several occasions during past investigations at the WES, this adjustment has been accomplished as follows. The wave-energy transmission characteristics of the proposed prototype structure were determined in a two-dimensional model using a scale large enough to ensure negligible scale effect. Then, a breakwater section that would provide essentially the same relative transmission of wave energy was developed for the small-scale three-dimensional model. However, this was not done for the Monterey Harbor study, because it was believed that the proposed structures and the incident wave characteristics at Monterey were sufficiently similar to those of other harbors for which such studies had been made to allow application of the results of those studies to the Monterey case. Therefore, a study was made of the previous findings in those cases that were similar to Monterey, and it was found that a close approximation of the correct wave-energy transmission characteristics could be obtained for short-period waves by increasing the size of the rock used in the small-scale model to approximately twice that required for geometric similarity. Accordingly, in constructing the breakwater structures in the Monterey Harbor model, the rock sizes were computed linearly by scale, then doubled to arrive at the actual sizes used in the model. Based on the work of Le Méhauté,[6] it is considered that the scale effects with respect to the transmission of long waves were negligible for the model breakwaters used in this study.

Description of Model and Appurtenances

14. The model was molded in cement mortar and reproduced the entire harbor area and underwater contours to an offshore depth of 160 ft. Sufficient additional offshore area was included to permit generation of both long-period and short-period test waves from the selected model directions of wave approach (see paragraphs 22 and 26). The total area reproduced in the model was approximately 7800 sq ft, representing about 4 square miles in the prototype. Fig. 3 shows a general view of the model with existing

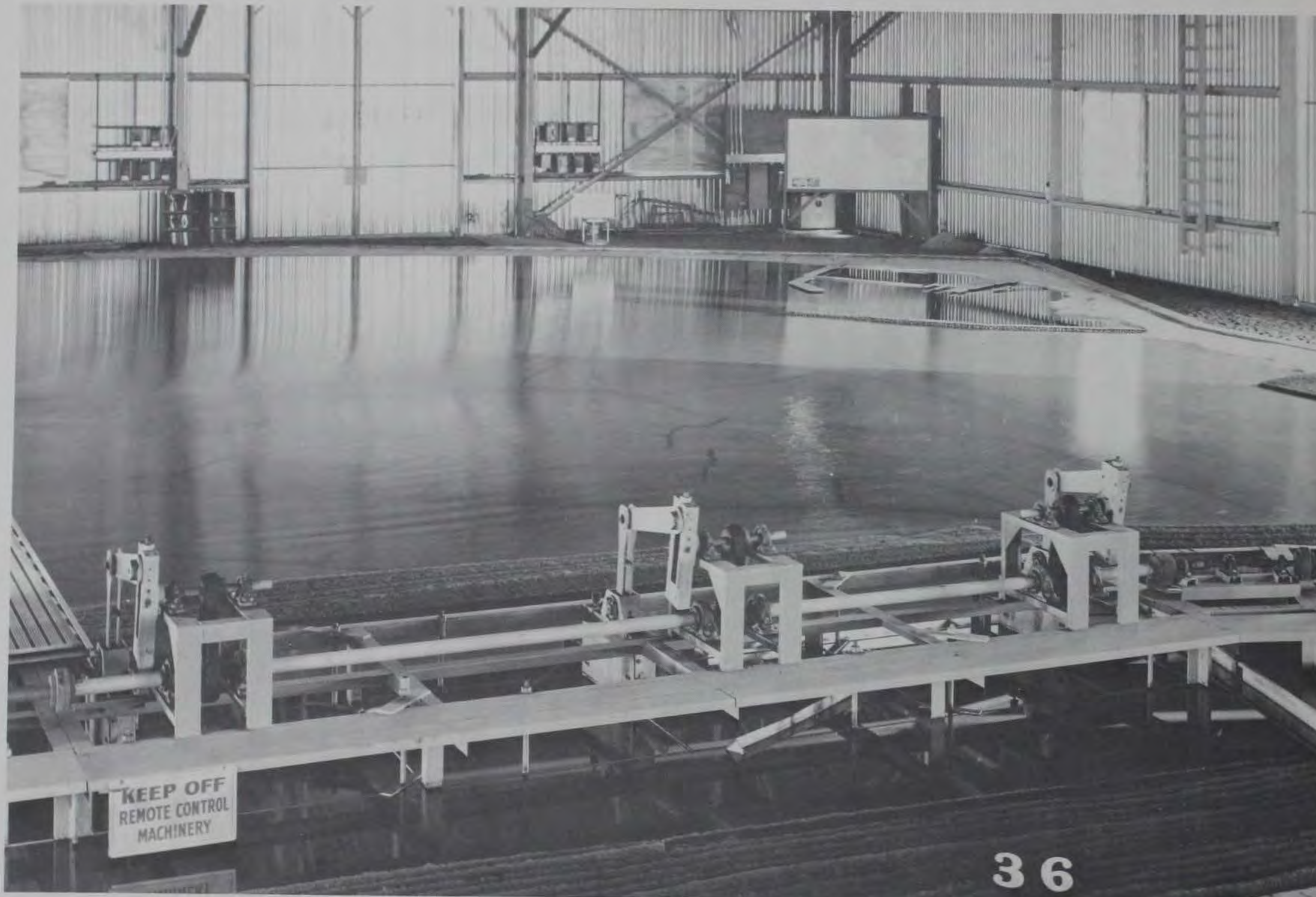

Fig. 3. General view of model, existing prototype conditions

conditions installed. Model construction was based on the mean lower low water (mllw) datum, and all elevations used in this report refer to mllw. Several layers of wave absorber material were placed around the seaward boundaries of the model and in front of the vertical-faced wave generator to reduce the effects of wave reflection on model test results.

15. Long-period model waves were generated to scale by two sections of a vertical-bulkhead wave generator. The two sections had a total length of 56 ft and were positioned to reproduce the average curvature of a long-period wave front bent by refraction as it traveled through shallow water to the harbor area (plate 2). The generator, by use of universal couplings between sections, operated from a single power source. The horizontal movement of the vertical bulkhead caused a periodic displacement of water incident to this motion. The bulkhead speed and displacement were infinitely variable over the range necessary to permit generation of model test waves. For the short-period phase of the investigation, the two wave generator sections were combined into one straight 56-ft generator that was mounted on retractable casters enabling it to be positioned to generate waves from more than one test direction. In order to provide room for the wave generator to generate test waves from the north (azimuth 360 deg) direction, the outer reaches of the molded area were modified so that underwater contours were reproduced only to an offshore depth of 120 ft (plate 3).

16. The direction and magnitude of surface currents in the model were measured by taking time-exposure photographs of surface floats from camera positions directly above the harbor area. From these photographs, the progress of the floats over one wave cycle was measured relative to a horizontal grid system painted on the model floor, and the corresponding velocities were computed. Wave heights at selected locations in the model were recorded on chart paper by an electrically operated oscillograph. The input to the oscillograph was the output of electrical wave-height gages that 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 the water.

PART III: TEST CONDITIONS AND DATA ANALYSIS

Selection of Test Conditions

Still-water level

17. Still-water levels (swl) for harbor wave-action models are selected so that the various wave-induced phenomena that are dependent upon water depths are accurately reproduced 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 harbor structures, and the transmission of wave energy through porous structures. Some of the more important factors influencing 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 usually accompanied by some 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 of 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 was desirable that a model swl be selected that closely approximated the higher water stages that normally prevail during severe storms in the prototype. This entailed the study of tide height records in the prototype locality, with due attention being given to the higher levels experienced in the area in the past.

18. The mean diurnal range of the astronomical tide at Carmel, California, near Point Pinos and adjacent to Monterey Harbor, is 5.2 ft, and the maximum range is 9.7 ft. Mean higher high water (mhhw) is 5.2 ft above mllw. Because of the low probability that an extreme wind tide, a high astronomical tide, and extreme storm waves would occur simultaneously, it appeared reasonable to select a swl somewhat less than the maximum

recorded tide. Accordingly, the mhhw stage of +5.2 ft was selected as being representative of conditions normally expected to occur during a severe storm, and this swl was used for all tests conducted in the model.

Wave dimensions and directions

19. In planning the test program for a model investigation of harbor wave-action problems, it is necessary to select dimensions and directions for the test waves that will afford a realistic test of the proposed improvement plans and allow an accurate evaluation of the elements of the various proposals. Since Monterey Harbor is subject to the action of both long-period and short-period waves, it was necessary to incorporate both of these wave types into the testing program.

20. Long-period waves. Very little is known about the basic causes of surging in Monterey Bay, but indications are that it is more likely to be the result of genuine long-period waves from the open ocean than the result of surf beats generated locally by swells. A detailed study was made of the manner of propagation of these long-period waves into Monterey Bay by Wilson.[4] Wave refraction diagrams that were drawn for incident waves from SSW clockwise to WNW indicated that regardless of the deepwater direction all long-period waves reach Monterey Harbor from practically the same direction; however, there are comparatively large differences in energy content. Accordingly, the face of the wave generator was angled to reproduce a wave front closely approximating the average curvature of the long-period waves following refraction, as determined by Wilson, and the generator was positioned to propagate model waves from the average direction of the refracted long-period waves.

21. Long-period prototype wave data. Under contract to the SFD, Marine Advisers, Inc., of La Jolla, California, installed and operated a group of three wave sensors at critical locations in Monterey Harbor (plate 1) for a period of six months. Sensors 1 and 2 were arranged to filter sea and swell from the records and therefore functioned as long-period wave recorders. The results of the measurements made by sensors 1 and 2 are summarized in plate 4.

22. Selection of long-period test waves. An analytical study of long-period sea-energy oscillations in the vicinity of Monterey Bay with

respect to the possibility of related response in Monterey Harbor was conducted, and the results of that study are presented in Appendix A. Based on these results, the following wave periods were selected for the long-period phase of the model study:

T = 35, 38, 41, 44, 47, 51, 55, 60, 66, 72, 80, 88, 97,
100.2, 114, 124, 132, 138, 144, 158, 172, 185, 205,
225, 234, 257, 280, 305, 330, 360 sec (prototype)

Prior to the conduct of the analytical study, preliminary tests were made using ten arbitrarily selected long-period waves in the range of 35 to 255 sec (see paragraph 33).

23. Short-period waves. Surface-wind waves are generated by the tangential shear force of the wind blowing along the water surface and the normal force of the wind against the wave crests. The magnitude of the maximum waves that can be generated by a given storm depends upon the wind speed, the length of time that wind of a given speed continues to blow, and the water distance (fetch) over which the wind blows. Selection of test wave conditions entails evaluation of such factors as (a) the fetch and decay distances (the latter being the distance over which waves travel after leaving the generating area) for the various directions from which waves can attack the problem area; (b) the frequency of occurrence and duration of storm winds from the different directions; (c) the alignment, width, and relative geographical position of the navigation entrance to the harbor; (d) the alignment, length, and location of various reflecting surfaces inside the harbor; and (e) the refraction of waves caused by differentials in depth in the area seaward of the harbor, which may create either a concentration or a diffusion of wave energy at the harbor site.

24. Short-period 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, short-period wave refraction diagrams were

prepared by the SFD for representative wave periods from the critical directions of approach. These diagrams were constructed by plotting the positions 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 $(b_o/b)^{1/2}$ is the refraction coefficient; K is the shoaling coefficient. Thus, the refraction coefficient multiplied by the shoaling coefficient gives a conversion factor for transfer of deepwater wave heights to shallow-water values. The shoaling coefficient, which is a function of wavelength and water depth, can be obtained from reference 7.

25. Short-period prototype wave data. Wave hindcast data for sea and swell at station 3 off the central California coast were furnished for a three-year period (1956-1958) by National Marine Consultants.[8] These data, computed in accordance with the theory of wave spectra and statistics as presented by Pierson, Neumann, and James,[9] were analyzed to establish the characteristics and estimated duration of deepwater sea and swell approaching Monterey Harbor from the west, west-northwest, and northwest directions. The results of this analysis are presented in table 1. Local sea, generated in Monterey Bay itself, approaches Monterey Harbor from the north direction with periods up to about 8 sec and heights up to about 12 ft.

26. Selection of short-period test waves. After completion of the short-period wave refraction analysis, the deepwater wave values were converted to shallow-water values for use in the model (table 2). Two shallow-water test wave directions were selected. One was the average direction of the refracted waves for the significant wave periods noted for the three deepwater wave directions used in the analysis, and which represented waves from the open ocean. The other test direction represented the locally generated waves from the north. The following tabulation shows the wave periods selected, the deepwater wave directions and

heights, and the corresponding shallow-water wave directions and heights selected for testing in the model.

Wave Period	Deepwater Waves		Selected Shallow-Water Test Waves	
T , sec	Direction	Height, ft	Direction	Height, ft
7	WNW	11	N35°W	7
	NW	9	N35°W	7
9	NW	13	N35°W	9
11	W	19	N35°W	7
	WNW	15	N35°W	7
	NW	9	N35°W	7
	NW	17	N35°W	13
13	W	21	N35°W	7
	WNW	17	N35°W	7
	NW	9	N35°W	7
	NW	19	N35°W	13
15	W	25	N35°W	7
	WNW	11	N35°W	7
	NW	9	N35°W	7
	NW	17	N35°W	13
17	W	25	N35°W	7
	WNW	11	N35°W	7
	WNW	21	N35°W	13
	NW	9	N35°W	7
8	N	9	N	9
	N	12	N	12

Analysis of Model Data

Long-period waves

27. In evaluating the various design plans tested, corresponding model data, i.e. results of tests using similar input test conditions with different plans installed, were compared to determine the relative effectiveness of each individual plan. For the long-period wave phase of the study, this included the comparison of: (a) both maximum and average wave heights recorded in the individual harbor basins; (b) current velocities in the harbor basins and entrances thereto; (c) modes of oscillation in the bay area; (d) frequency-response data for the various basins; and (e) time-exposure photographs of float movement in critical areas. Visual observations during model testing and test notes aided in the analysis.

Short-period waves

28. In the short-period wave phase of the study, the relative merits of the various plans tested were evaluated by (a) comparison of wave heights at selected locations in the harbor; and (b) extension of the wave height data into tables showing the estimated duration of waves of various magnitudes that can be predicted at the selected locations. Visual observations, photographs of wave-front patterns, and test notes were also utilized in the short-period wave test analysis. In the wave height data analysis, the average height of the highest one-third of the waves recorded at each gage location was selected for the computations. All wave heights thus selected were then adjusted to compensate for the greater rate of wave height attenuation in the model, as compared with the prototype, by the application of Keulegan's equation.[10] From this equation, the reduction of wave heights in the model due to bottom friction can be calculated as a function of water depth, width of wave front, wave period, water viscosity, and distance of wave travel.

PART IV: TESTS AND RESULTS

Description of Plans Tested

Base test

29. The term "base test" as used herein denotes a test performed with existing prototype conditions (plate 5) installed in the model. Prior to tests of the various improvement plans, comprehensive base test data were obtained in the harbor and bay area. These data were then used as a base to determine the relative efficacy of the various improvement plans.

Improvement plans

30. Model tests were conducted using the two basic harbor configurations designed by the firm of Koebig and Koebig, Inc. In this report, these plans are designated plan 1 (double entrance - Koebig and Koebig, Inc. plan 5) and plan 2 (single entrance - Koebig and Koebig, Inc. plan 5A). The various elements of plans 1 and 2 are shown in plates 6 and 7, respectively, and layout details of the proposed structures are shown in plate 8. Typical sections of the various breakwater and revetment structures are shown in plate 9. For convenience in reporting model test results, the various harbor areas were designated the west basin (the area between the existing breakwater and Municipal Wharf No. 2, excluding the marina); the marina (the area bounded by Municipal Wharf No. 1, Municipal Wharf No. 2, and the existing fishing pier); the mid-basin (the area bounded by moles A, B, and C and Municipal Wharf No. 2); the east basin (the area bounded by moles B, D, and E and the east breakwater); and the fairway (the area between the north breakwater and moles C, D, and E). Breakwater, mole, and basin designations are shown in plate 10.

Minor modification of plans

31. During the conferences of 23-24 May and 31 August-1 September 1967, several special tests involving minor modifications of plans 1 and 2 were conducted at the request of the visiting conferees. These tests involved such changes as removing the marina frontal wall and the cutoff

wall under Wharf No. 2; enlarging moles A, C, and F; installing a bulkhead wall in the east basin and in the marina; and sealing the existing breakwater. Since none of these schemes effected a significant change in the degree of protection afforded by either plan 1 or plan 2, no data for these special tests are included in this report.

Description of Tests

Long-period waves

32. Calibration of wave generator. For the long-period wave phase of the investigation, the wave generator was calibrated using the Marine Advisers' data for sensor No. 1 shown in plate 4. For each individual wave period, the wave generator stroke was adjusted until the required wave height was recorded at the location of sensor No. 1 (gage 37 in the model). The corresponding input wave at the wave generator was then used for all subsequent base tests and tests of the two improvement plans.

33. Modes of oscillation. Preliminary tests were conducted in which wave heights were measured over the entire bay area of the model for base test and plan 1 conditions to determine if installation of the plan would cause any significant changes in the modes of oscillation in the model bay area. Measurements were made over a horizontal grid system, and contours of equal wave heights were drawn so that the various loop and node points could be distinguished. Wave periods of 35, 45, 55, 120, 130, 170, 180, 200, 230, and 255 sec were used in these tests.

34. General wave height and surface current tests. Wave heights and surface currents were measured in the harbor basins and their entrances for base test, plan 1, and plan 2 over the entire range of wave periods from 35 to 360 sec. For the measurement of wave heights, gage locations for the various model configurations are shown in plates 5, 6, and 7. For each test in which surface currents were measured, several hundred surface floats approximately 1 in. square and 1/8 in. thick were distributed throughout the harbor area, and time-exposure photographs were taken of their movement over one wave cycle (one wave period). From these photographs, the movement of the individual floats was measured relative to a

model grid system, and the corresponding average surface velocities over one wave cycle were computed. Due to the complex current patterns in and around the entrance to the marina, it was impossible to measure accurately the currents in this area over a complete wave cycle. Therefore, separate measurements were made, and the floats were timed only during the time that they were in the actual entrance (a distance of approximately 100 ft prototype). These measurements were taken during periods of peak flow through the entrance (both in and out) and represent maximum values.

35. Frequency-response tests. Frequency-response tests were conducted to determine if any of the harbor basins responded to specific wave periods. In these tests, comprehensive wave height measurements were made in each of the basins for the entire range of wave periods (35 to 360 sec). Time-exposure photographs, as described in paragraph 34, were also used to determine the degree of excitement of the harbor basins for the various wave periods.

Short-period waves

36. General wave height tests. Short-period wave tests were conducted with base test conditions and plans 1 and 2 installed in the model for the entire range of test waves listed in paragraph 26. Wave height measurements were made in all of the harbor basins and the entrances thereto. Specific wave height gage locations for the various model configurations are shown in plates 5, 6, and 7.

37. Shortening of north breakwater. Additional short-period wave tests were conducted to determine the feasibility of shortening the east end of the north breakwater as used in plan 1. Six plans were tested with the following increments removed from the structure length: plan 1A, 50 ft; plan 1B, 100 ft; plan 1C, 150 ft; plan 1D, 200 ft; plan 1E, 250 ft; and plan 1F, 300 ft. Elements of plans 1A through 1F are shown in plate 11. Test waves from the north had periods of 8 sec and heights of 9 and 12 ft, representing nonovertopping and overtopping conditions, respectively. Test waves from the N35°W direction had periods of 11 and 17 sec with heights of 7 and 13 ft for each period. For both wave periods, the 7-ft test wave represented little or no overtopping of the structure, whereas the 13-ft wave represented significant overtopping.

38. East entrance current tests. Photographic and observational tests were conducted to determine the direction and magnitude of wave-induced currents in the vicinity of the east entrance to the proposed harbor. Overhead photographs of surface-float movement were taken to show current patterns in the east entrance and in the area immediately east of the east breakwater resulting from 17-sec, 13-ft waves from the N35°W test direction for plans 1, 1A, and 2. Also, photographs of current patterns were obtained with two separate groin structures installed in conjunction with plans 1, 1A, and 2 as follows: groin G, a 400-ft groin extending seaward from shore, parallel to and 500 ft east of the east breakwater; and groin H, a 400-ft groin originating at a point on the east breakwater 500 ft from the seaward end of the breakwater and extending eastward parallel to the shore. The model floor in this area was painted black to contrast with the white surface floats.

39. Design-wave height tests. Wave heights for use in designing the breakwaters to withstand the forces of short-period waves were measured at the location of both the proposed north breakwater (plate 12) and the existing breakwater (plate 13). For these tests, the breakwaters were removed and wave absorbers were placed along the shore to reduce the effects of wave reflection on test results. The largest significant shallow-water wave height for each of six representative wave periods was selected for testing. The shallow-water test waves are tabulated below with the corresponding deepwater wave characteristics.

Wave Period	Significant Deepwater Waves		Significant Shallow-Water Waves	
T, sec	Direction	Height, ft	Direction	Height, ft
7	WNW	11	N35°W	7
7	NW	9	N35°W	7
9	NW	13	N35°W	9
11	NW	17	N35°W	13
13	NW	19	N35°W	13
15	NW	17	N35°W	13
17	WNW	21	N35°W	13

Test Results

Long-period waves

40. Modes of oscillation. The results of preliminary tests to determine the modes of oscillation in the bay area for base test conditions are presented in the form of wave height contours in plates 14-23. Similar data with plan 1 installed in the model are presented in plates 24-33. When the wave height contours for plan 1 are compared with those of base test, it can be seen that the plan installation causes no major dissimilarity in the modes of oscillation in the bay area. There is a moderate change in the modes of oscillation for wave periods greater than about 130 sec, but this is due primarily to the fact that wave heights were smaller for the longer periods, making it harder to differentiate between the loop and node points. Thus, it is believed that wave input into the harbor was modeled with sufficient accuracy for each test condition.

41. General wave height and surface current tests. The long-period average and maximum wave heights with base test conditions and plans 1 and 2 installed in the model are presented in tables 3 and 4, respectively. These data allow a direct comparison of both average and maximum wave heights for existing conditions and the two improvement plans for the entire range of test waves. The surface current velocities in the harbor basins and their entrances are presented in tables 5 and 6. The wave height and current data show that both plans 1 and 2 offer approximately the same degree of protection for the harbor, and, when compared with similar base test data, indicate that the installation of either plan will result in some improvement over existing conditions. Wave heights and currents in the newly formed basins of plans 1 and 2 compare favorably with those in the existing small-boat marina, indicating that, to the extent that present wave conditions in the existing marina are considered acceptable, conditions in the proposed additional basins will also be satisfactory.

42. Frequency-response tests. The results of frequency-response tests that were conducted to determine if any of the harbor basins respond to specific wave periods are presented in plates 34-38. These data are

presented in the form of line diagrams that show the wave amplification factor (H/H_c) versus wave periods in the various basins for base test, plan 1, and plan 2, where H is the average wave height in the basin and H_c is the corresponding wave height at the wave generator. The value of H_c is the calculated wave height based on the following relation by Biésel;[11] $H_c = \frac{2\pi dS}{L}$, where d is the water depth at the wave generator, S is the wave generator stroke, and L is the wavelength at the wave generator. The frequency-response line diagrams show that the harbor basins respond to some extent to several of the wave periods tested, particularly 225 sec. It should be noted, however, that the magnitude of the amplification factor H/H_c is less than 1.0 in all cases. This indicates that, although there is some response in the basins for some of the wave periods, serious resonance does not occur.

Short-period waves

43. With respect to short-period wave heights measured in the harbor, two methods of analysis were used. These methods are described as follows:

- a. For those test waves that were derived from the prototype wave hindcast data and that represent open-ocean storm waves (see paragraph 26), the model data were used to compute wave-reduction coefficients, H/H_w, where H is the wave height at specified locations in the harbor, and H_w is the wave height at the wave generator (shallow-water test wave height). These coefficients were then applied to the shallow-water wave duration values presented in table 2. The results of this application were summarized in tabular form to show the estimated duration of waves of various magnitudes that can be expected to occur in the specified harbor areas for existing conditions and for each improvement plan tested. For additional information, the individual wave heights used in this analysis are also included in the presentation of data.
- b. For the series of test waves that did not appear in the hindcast data analysis, i.e. the locally generated waves from the north, the model wave height data were tabulated to show the measured wave heights at the various gage locations for base test and the two improvement plans.

44. General wave height tests. Those results of the analysis of wave height data that were applicable to the shallow-water wave duration data (table 2) are presented in tables 7 and 8, and the individual wave

heights from which those data were derived appear in tables 9-13. A comparison of the wave-duration data in table 7 indicates that both plans 1 and 2 offer an improvement over existing wave conditions in the harbor, but neither plan appears significantly better than the other. These data also indicate that short-period wave heights in the marina, west basin, and mid-basin should be acceptable for either plan. In the east basin and fairway, however, wave heights for both plans are larger in some cases than those generally accepted as being safe for small boats. This is due primarily to overtopping of the eastern end of the north breakwater. In order to allow a more detailed evaluation of the seriousness of the overtopping problem, the east basin was divided into three mooring areas (plate 10), and the estimated duration and magnitude of waves from the directions of west, west-northwest, and northwest that would occur in these areas are presented in table 8. These data indicate that mooring area I, which comprises a large portion of the east basin, would be safe for the anchorage of small boats for either plan 1 or plan 2. However, the larger short-period test waves resulted in wave heights in mooring areas II and III that exceed those usually considered safe for small-boat mooring. The results of tests with locally generated waves from the north are presented in table 14. A comparison of these data for base test and plans 1 and 2 indicates that both plans offer considerable improvement over existing conditions, and short-period wave heights in the harbor resulting from locally generated waves from the north should be acceptable with either plan.

45. During model testing, overhead photographs of short-period wave patterns in the harbor area were taken for base test conditions and for each of the improvement plans. Photographs 1-9 present a comparison of wave patterns for base test, plan 1, and plan 2 for several representative test waves.

46. Shortening of north breakwater. The results of tests to determine the feasibility of shortening the east end of the north breakwater are presented in tables 15-17. These data show that none of the reductions in breakwater length effected a noticeable change in wave heights in the east basin. Wave heights in the fairway also remained largely unchanged

except in the immediate vicinity of the breakwater revisions. In the east entrance to the harbor (gage 28), wave heights increased with each decrease in breakwater length and reached a maximum value of 14.5 ft for plan 1F. Wave heights of this magnitude would create a serious navigation hazard in this area. Further, visual observations during these tests showed that with each reduction in breakwater length the north side of mole E became increasingly more exposed to large waves. Overtopping of this structure by the 17-sec, 13-ft test wave was observed in the model as follows:

Plan No.	Length of Mole E Overtopped, ft
1	0
1A	0
1B	120
1C	120
1D	180
1E	235
1F	260

47. East entrance current tests. Photograph 10 shows wave-induced surface current patterns in the east entrance to the harbor and in the area east of the east breakwater for plan 1. Photographs 11 and 12 show corresponding current patterns with groins G and H (designated plans 1G and 1H, respectively) installed in conjunction with plan 1. A comparison of these photographs shows that surface-float movement along the beach was from east to west, and movement along the east breakwater was from south to north in all cases. It can also be seen that installation of the two groins (plans 1G and 1H) caused the formation of eddies. Average surface current velocities in the area covered by the photographs reached prototype magnitudes of approximately 4 fps. Further examination reveals that none of the surface floats moved into the east entrance to the harbor during any of these tests. From visual observations of the 17-sec, 13-ft test wave, it was noted that overtopping of the proposed north breakwater and transmission through this structure raised the water level inside the harbor, creating outward currents of about 1 fps in both the east and west entrances. Observational tests were made to determine subsurface currents by injecting a dye solution into the model, and it was found that the subsurface current patterns and velocities were essentially the same as those

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obtained with the surface floats. The photographs showing current patterns with the groins installed in conjunction with plans 1A and 2 are not included in this report because of the close similarity to photographs 11 and 12.

48. Design-wave height tests. The results of tests to determine the design-wave heights for the north (detached) breakwater and the existing breakwater are presented in tables 18 and 19, respectively. These tables show the measured wave heights at stations along the center-line locations of each structure for each test wave. The maximum measured wave heights for each wave period in tables 18 and 19 are also shown as curves of H/L versus d/L in plate 39. In this plot, d is the water depth at the position of the breakwater, H is the maximum wave height recorded at any point along the center-line location, and L is the wavelength measured in depth d. Also included in plate 39 for comparison are curves showing the maximum breaking and nonbreaking waves that can attack rubble-mound breakwaters with beach slopes and breakwater side slopes similar to those at Monterey. These latter data are results taken from a report[12] concerning recent tests conducted at WES in connection with ES 815, "Stability of Rubble-Mound Breakwaters." The maximum breaking and nonbreaking waves obtained from the ES 815 data shown in plate 39 are compared with the maximum measured significant waves obtained on the Monterey Harbor model in the following tabulation:

	Monterey Harbor Model Data		WES ES 815 Data	
Wave Period, sec	Max Significant Wave Height Measured at North Breakwater, ft	Max Significant Wave Height Measured at Existing Breakwater, ft	Max Breaking* Wave, ft	Max Nonbreaking* Wave, ft
7	5.5	4.3	21.1	18.0
9	7.4	5.9	25.7	21.9
11	15.8	8.0	27.9	24.2
13	17.7	9.0	31.4	27.0
15	17.8	7.6	33.0	28.3
17	16.4	10.4	33.0	28.2

* Based on an average water depth of 41 ft (-36 ft mllw and +5 ft still-water level).

It can be seen that the maximum measured wave heights are less than the maximum waves that can attack the structures without breaking before reaching the structures. Therefore, the maximum significant design-wave heights for the proposed north breakwater and the existing breakwater are 17.8 ft and 10.4 ft, respectively. However, the data in table 18 show that the maximum wave heights along the location of the proposed north breakwater vary from 6.0 ft at the west end to 17.8 ft at the east end, a variation large enough to suggest that certain reaches of the proposed structure could be designed using design-wave heights somewhat less than the maximum recorded value of 17.8 ft. Accordingly, further examination of the data in table 18 indicates that for design purposes this breakwater can be divided into three reaches, the division being based on the range of maximum wave heights recorded at the various gage locations. The following tabulation presents data relative to the three reaches so selected.

	Reach A* (370 ft)		Reach B* (1490 ft)				Reach C* (1490 ft)			
Gage No.	46	47	48	49	50	51	52	53	54	55
Maximum wave height at each gage, ft	6.0	7.3	9.9	10.8	11.2	11.5	16.2	17.7	14.1	17.8
Range of maximum wave heights in reach, ft	1.3		1.6				3.7			
Maximum wave height in reach, ft	7.3		11.5				17.8			

* Reach locations are shown in plate 12.

Therefore, based on the foregoing analysis, the significant design-wave heights for the proposed north breakwater are 7.3 ft for reach A, 11.5 ft for reach B, and 17.8 ft for reach C. In the case of the existing breakwater, the range of maximum wave heights recorded along the location of this structure was 3.6 ft (table 19), and the significant design-wave height is 10.4 ft.

PART V: CONCLUSIONS

49. Based on the results of the hydraulic model study, it is concluded that:

- a. The modes of oscillation in the bay area for base test conditions and with the proposed plan 1 installed were generally similar, and the wave input into the harbor was reproduced with sufficient accuracy.
- b. Long-period wave and current conditions in the harbor will be approximately the same for either plan 1 or plan 2, and either plan will offer a slight improvement over conditions in the existing harbor.
- c. Long-period wave and current conditions in the newly formed basins of plans 1 and 2 compare favorably with those in the existing marina, indicating that, to the extent that present conditions in the existing marina are considered acceptable, conditions in the proposed additional basins will also be satisfactory.
- d. The harbor basins respond to some extent to several of the long-period waves tested; however, no serious resonance was noted.
- e. With regard to short-period wave heights, both plan 1 and plan 2 offer an improvement over existing conditions and neither plan appears significantly better than the other.
- f. Short-period wave heights in the marina, west basin, midbasin, and mooring area I of the east basin should be acceptable for either plan 1 or plan 2.
- g. During periods of attack by exceptionally high short-period storm waves, wave heights in mooring areas II and III of the east basin and in the fairway will exceed those generally accepted as being safe for the navigation and the anchorage of small boats.
- h. Reducing the length of the detached north breakwater by amounts up to 300 ft will have little effect on wave heights in the east basin. However, wave heights in the east entrance to the harbor will be increased considerably, and serious overtopping of mole E will occur for all reductions in length greater than about 50 ft.
- i. The design-wave heights for the proposed north breakwater are 7.3 ft for reach A, 11.5 ft for reach B, and 17.8 ft for reach C. The design-wave height for the existing breakwater is 10.4 ft. These are significant wave heights corresponding to the average heights of the highest one-third waves in the storm-wave trains.

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Table 1

Estimated Duration and Magnitude of Short-Period Deepwater Waves Approaching Monterey Harbor from West, West-Northwest and Northwest

Wave Height* ft	Duration of Short-Period Deepwater Waves for Various Wave Periods,* hr/yr								
	4-6 sec	6-8 sec	8-10 sec	10-12 sec	12-14 sec	14-16 sec	16-18 sec	18+ sec	Total
				West					
1-3	233	145	260	192	89	36	9	2	966
3-5	58	198	239	160	46	31	9	2	743
5-7		32	155	96	44	32	11		370
7-9		2	57	48	42	12			161
9-11			11	33	16	4			64
11-13			9	37	11	6	2		65
13-15				4	9	2			15
15-17				11	2				13
17-19				4	4	2			10
19-21					4	2	2		8
21-23									
23-25					4	2	2		8
Total	291	377	731	585	271	129	35	4	2423
				West-Northwest					
1-3	174	258	608	240	107	73	31	22	1513
3-5	51	264	446	277	144	78	63	6	1329
5-7	6	38	171	193	94	53	20	4	579
7-9		4	62	100	54	20	8		248
9-11		2	8	28	24	8			70
11-13				16	8	2	4		30
13-15				4	8	6			18
15-17					6	4			10
17-19				2					2
19-21							2		2
Total	231	566	1295	860	445	244	128	32	3801
				Northwest					
1-3	979	288	851	737	117	72	4	2	3050
3-5	174	1203	531	241	113	39	18	2	2321
5-7		284	284	131	50	24	12		785
7-9		8	539	79	28	19	6		679
9-11			113	45	21	18	4		201
11-13			10	38	21	6	6		81
13-15				22	22	6			50
15-17				12	2	2			16
17-19					6				6
Total	1153	1783	2328	1305	380	186	50	4	7189

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

Table 2

Estimated Duration and Magnitude of Short-Period Shallow-Water Waves Approaching Monterey Harbor from West, West-Northwest and Northwest

Wave Height* ft	Duration of Short-Period Shallow-Water Waves for Various Wave Periods,* hr/yr								
	4-6 sec	6-8 sec	8-10 sec	10-12 sec	12-14 sec	14-16 sec	16-18 sec	18+ sec	Total
				West					
0-3	291	377	722	496	221	115	29	4	2255
3-5			9	74	36	8	2		129
5-7				15	10	6	4		35
7-9					4				4
Total	291	377	731	585	271	129	35	4	2423
				West-Northwest					
0-3	225	522	1054	710	345	151	94	28	3129
3-5	6	42	233	128	78	53	20	4	564
5-7		2	8	20	22	28	8		88
7-9				2		8	4		14
9-11						4			4
11-13							2		2
Total	231	566	1295	860	445	244	128	32	3801
				Northwest					
0-3	979	288	851	737	117	72	4	2	3050
3-5	174	1487	815	372	163	63	30	2	3106
5-7		8	539	79	28	19	6		679
7-9			123	83	42	24	10		282
9-11				22	22	6			50
11-13				12	8	2			22
Total	1153	1783	2328	1305	380	186	50	4	7189

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

Table 3

Average Long-Period Wave Heights Measured in Monterey Harbor for Base Test and Plans 1 and 2

Wave Period sec	Wave Ht at Wave Machine H_c , ft	Average Wave Heights* in the Various Basins, ft											
		West Basin			Marina			Mid-Basin		East Basin		Fairway	
		Base Test	Plan 1	Plan 2	Base Test	Plan 1	Plan 2	Plan 1	Plan 2	Plan 1	Plan 2	Plan 1	Plan 2
35	3.2	0.5	0.3	0.4	0.4	0.1	0.3	0.3	0.5	0.1	0.4	0.4	0.4
38	2.4	0.4	0.2	0.4	0.4	0.1	0.2	0.2	0.4	0.1	0.4	0.2	0.4
41	2.0	0.4	0.1	0.3	0.3	0.1	0.3	0.3	0.3	0.2	0.5	0.2	0.4
44	1.6	0.3	0.2	0.3	0.3	0.1	0.2	0.3	0.3	0.1	0.4	0.2	0.4
47	2.2	0.4	0.4	0.4	0.3	0.1	0.3	0.3	0.4	0.1	0.4	0.3	0.4
51	4.0	0.8	0.6	0.5	0.3	0.1	0.2	0.5	0.5	0.3	0.5	0.8	0.5
55	6.8	1.1	0.7	0.6	0.6	0.3	0.4	0.4	0.4	0.6	0.8	1.2	0.7
60	6.1	1.4	0.5	0.5	0.5	0.1	0.3	0.3	0.6	0.7	0.7	0.9	0.7
66	5.7	1.1	1.0	0.9	0.9	0.6	0.5	0.6	0.9	0.8	0.8	1.0	1.0
72	5.4	0.9	1.0	1.0	0.9	0.7	0.6	0.8	0.8	0.5	0.6	0.6	0.6
80	6.0	1.9	1.1	0.7	0.7	0.4	0.3	0.7	0.8	0.6	0.5	1.3	0.9
88	6.7	2.1	1.3	1.0	1.0	0.5	0.5	1.0	0.9	0.7	0.6	1.6	1.0
97	8.1	3.3	0.8	0.8	1.5	0.6	0.5	0.5	0.7	0.6	0.7	1.4	1.2
100.2	8.5	1.6	1.0	0.9	0.9	0.7	0.6	0.9	0.9	0.6	0.7	1.7	1.4
114	9.6	2.6	1.3	1.4	2.1	0.9	1.0	0.8	0.8	1.2	1.0	2.7	2.0
124	10.1	2.5	1.8	1.6	2.4	1.4	1.2	0.8	1.0	0.6	0.8	2.1	2.0
132	10.8	1.1	1.6	1.3	0.7	0.9	0.6	0.8	1.1	0.7	1.0	2.1	2.1
138	10.2	1.0	1.2	0.9	1.2	0.8	0.8	0.7	0.8	1.2	1.4	2.0	2.1
144	9.9	1.3	0.9	0.8	1.4	0.8	0.8	0.5	0.6	1.5	1.5	1.6	1.4
158	7.0	1.2	0.5	0.5	0.8	0.4	0.5	0.3	0.4	1.0	0.9	0.7	0.7
172	4.9	1.2	1.0	0.9	1.1	0.8	0.8	0.3	0.5	0.8	1.0	1.2	0.9
185	4.6	1.4	1.6	1.6	1.3	1.3	1.5	0.7	1.0	0.5	1.0	0.9	0.9
205	5.0	1.9	2.2	2.3	1.8	2.0	2.4	1.3	1.6	0.6	0.5	0.8	0.6
225	5.8	2.5	2.9	3.0	2.2	2.6	3.0	2.4	2.9	0.9	0.7	1.1	1.0
234	6.1	1.7	2.6	2.8	1.7	2.1	2.6	2.3	2.9	1.1	0.9	1.0	0.9
257	5.6	1.5	1.3	1.3	1.7	1.3	1.4	1.6	1.9	1.7	0.9	1.3	0.7
280	5.0	0.9	1.1	0.8	0.7	1.1	0.9	2.4	1.9	2.1	1.1	1.3	0.6
305	4.3	0.8	1.1	1.0	1.6	1.6	1.6	2.2	2.4	1.3	1.1	0.5	0.5
330	3.7	0.9	0.5	0.7	2.2	1.4	1.5	1.3	1.3	0.7	0.7	0.4	0.3
360	2.7	0.8	0.3	0.5	1.9	1.0	1.0	0.5	0.7	0.5	0.5	0.3	0.4

* The average wave height is the average of the values recorded at all gages in a particular basin.

Table 4

Maximum Long-Period Wave Heights Measured in Monterey Harbor for Base Test and Plans 1 and 2

Wave Period sec	Wave Ht at Wave Machine H_c, ft	Maximum Wave Heights* in the Various Basins, ft											
		West Basin			Marina			Mid-Basin		East Basin		Fairway	
		Base Test	Plan 1	Plan 2	Base Test	Plan 1	Plan 2	Plan 1	Plan 2	Plan 1	Plan 2	Plan 1	Plan 2
35	3.2	0.7	0.5	0.5	0.5	0.2	0.3	0.6	0.9	0.4	0.6	0.7	0.5
38	2.4	0.4	0.3	0.4	0.4	0.4	0.3	0.3	0.5	0.3	0.6	0.3	0.5
41	2.0	0.4	0.2	0.3	0.5	0.1	0.3	0.3	0.4	0.3	0.7	0.3	0.6
44	1.6	0.4	0.5	0.5	0.4	0.1	0.3	0.3	0.4	0.3	0.5	0.3	0.4
47	2.2	0.6	0.9	0.6	0.4	0.2	0.3	0.4	0.6	0.3	0.5	0.5	0.5
51	4.0	1.6	0.9	0.7	0.4	0.2	0.3	0.8	0.8	0.6	0.8	1.2	0.8
55	6.8	1.9	1.0	1.2	0.8	0.4	0.5	0.7	0.6	1.4	1.5	2.1	0.9
60	6.1	3.3	0.8	0.7	0.6	0.2	0.4	0.4	1.4	1.6	1.5	1.9	1.0
66	5.7	2.5	2.2	1.9	1.7	0.9	0.7	1.6	2.9	2.2	2.1	1.7	1.5
72	5.4	1.5	2.2	2.6	1.4	1.1	0.9	2.3	2.6	1.2	1.1	1.1	0.7
80	6.0	3.9	2.8	1.7	1.1	0.6	0.4	1.7	1.8	2.0	1.0	2.3	1.4
88	6.7	4.2	3.5	2.7	1.5	0.8	0.7	2.4	2.2	2.3	1.5	3.5	2.3
97	8.1	6.2	1.3	1.7	2.6	1.0	0.7	1.3	1.4	1.7	1.7	3.1	2.7
100.2	8.5	3.3	1.8	1.9	1.4	1.0	0.8	1.8	2.0	1.9	1.5	3.8	2.5
114	9.6	6.0	2.5	2.7	3.6	1.5	1.8	1.6	1.4	2.7	2.2	4.1	3.0
124	10.1	4.3	3.6	3.3	4.3	2.4	2.3	1.4	2.0	1.5	1.8	3.1	2.5
132	10.8	1.5	3.9	3.2	1.1	1.6	0.9	1.7	2.3	1.3	1.8	2.9	3.1
138	10.2	1.9	2.0	1.4	2.2	1.4	1.5	1.0	1.3	2.0	2.4	3.3	3.3
144	9.9	2.3	1.5	1.4	2.5	1.4	1.4	0.6	0.9	2.7	2.6	2.5	2.4
158	7.0	1.7	0.8	1.0	1.3	0.9	0.9	0.4	0.6	1.7	1.5	1.7	0.9
172	4.9	2.0	1.6	1.4	1.8	1.4	1.6	0.5	0.8	1.3	1.8	2.0	1.3
185	4.6	1.9	2.4	2.3	2.2	2.6	2.8	0.9	1.4	0.7	1.8	1.5	1.2
205	5.0	2.5	3.2	3.2	3.1	3.4	3.9	1.9	2.4	0.8	0.7	1.0	1.3
225	5.8	3.6	3.9	4.1	3.5	4.4	4.8	3.5	4.2	1.6	1.2	2.0	1.9
234	6.1	2.3	3.6	3.7	2.9	3.9	4.3	3.3	4.2	1.9	1.6	2.5	1.3
257	5.6	2.0	1.8	1.9	2.7	2.1	2.1	2.3	2.4	3.0	1.9	3.5	0.9
280	5.0	1.7	2.0	1.6	0.9	1.6	1.3	3.2	2.7	3.3	2.2	3.8	0.8
305	4.3	1.6	1.8	1.8	2.2	2.2	2.1	2.9	3.6	2.1	1.9	1.1	0.7
330	3.7	1.4	0.8	1.1	3.0	1.8	2.1	1.7	1.8	1.0	1.0	0.7	0.4
360	2.7	1.0	0.5	0.7	2.7	1.3	1.3	0.6	1.0	0.8	0.6	0.4	0.4

* The maximum wave height is the maximum value recorded at any gage in a particular basin.

Table 5

Long-Period Wave-Current Velocity Data in the Marina and West Basin for Base Test and Plans 1 and 2

Wave Period sec	Surface Velocities* at the Various Locations, fps										
	Marina						West Basin				
	Entrance			Mooring Area			Entrance		Mooring Area		
	Base Test	Plan 1	Plan 2	Base Test	Plan 1	Plan 2	Plan 1	Plan 2	Base Test	Plan 1	Plan 2
35	0.2	0.2	0.1	0.3	0.0	0.0	0.0	0.3	0.3	0.0	0.3
38	0.1	0.1	0.1	0.3	0.0	0.0	0.0	0.3	0.3	0.0	0.3
41	0.1	0.0	0.1	0.3	0.0	0.0	0.0	0.6	0.3	0.0	0.3
44	0.2	0.1	0.0	0.3	0.0	0.0	0.0	0.5	0.3	0.0	1.1
47	0.3	0.1	0.2	0.3	1.0	0.5	0.5	1.0	0.5	1.0	1.0
51	0.3	0.2	0.3	0.5	0.5	0.5	0.7	0.5	0.7	0.9	0.9
55	1.7	0.8	0.5	1.5	1.3	1.3	0.7	0.9	2.2	0.9	0.9
60	1.8	0.4	0.2	1.2	1.0	0.8	0.8	1.0	1.4	0.8	1.4
66	6.0	2.5	2.5	1.8	1.4	1.2	0.7	0.9	1.4	2.2	2.2
72	3.2	2.2	2.2	1.8	1.5	1.3	0.7	1.0	1.5	1.8	1.7
80	1.9	0.6	0.5	1.2	0.7	0.6	0.9	0.9	2.1	2.1	0.9
88	4.0	2.5	1.1	1.1	0.8	0.7	1.1	1.0	2.1	2.3	1.0
97	3.7	1.8	1.5	1.5	1.2	0.6	1.2	1.2	2.5	1.5	2.2
100.2	4.1	3.9	4.1	1.2	0.9	0.7	1.2	1.1	1.9	2.2	1.5
114	7.6	1.4	0.7	1.5	1.3	1.1	1.1	0.8	1.5	1.5	1.7
124	9.1	1.3	1.6	1.9	2.1	1.0	1.2	1.2	3.1	2.3	1.5
132	5.2	1.0	1.6	1.5	1.5	0.7	0.7	0.7	1.8	2.0	1.5
138	2.0	1.0	2.7	1.0	0.5	0.5	0.9	0.5	1.6	0.7	0.9
144	2.2	0.9	0.9	1.2	0.7	0.7	0.6	0.7	1.6	1.1	1.2
158	2.0	0.3	0.9	0.7	0.6	0.6	0.5	0.6	0.9	0.6	0.6
172	1.8	1.1	1.3	0.8	1.3	0.8	1.1	1.0	0.9	0.8	0.7
185	4.9	3.9	3.7	0.9	1.4	1.2	1.4	1.3	0.8	1.2	1.0
205	8.5	4.3	4.6	1.2	1.2	1.4	1.1	1.2	1.2	0.9	1.1
225	6.9	6.3	5.5	1.4	1.5	1.4	1.2	1.1	1.1	1.3	1.2
234	6.3	3.5	5.8	1.3	1.4	1.3	1.0	0.9	1.0	0.9	0.9
257	6.3	3.8	5.0	0.9	0.9	0.9	0.5	0.7	0.7	0.6	0.7
280	2.3	3.7	3.3	0.5	0.8	0.6	0.7	0.9	0.7	0.6	0.7
305	5.9	4.9	4.8	0.9	0.7	0.8	0.6	0.9	0.9	0.8	0.6
330	6.3	4.1	5.9	0.9	0.9	0.7	0.6	0.7	0.8	0.7	0.4
360	6.7	2.5	3.5	0.7	0.4	0.6	0.6	0.5	0.7	0.5	0.4

* Surface velocities given are the largest mean velocities measured over one complete wave cycle at each location with the exception of the entrance to the marina. Here the velocities were not measured over one complete cycle but only during the time that the float was in the actual entrance (a distance of approximately 100 ft).

Table 6

Long-Period Wave-Current Velocity Data in the Mid-Basin, East Basin, and Fairway for Plan 1 and Plan 2

	Surface Velocities* at the Various Locations, fps									
	Mid-Basin				East Basin					
Wave	Entrance		Mooring Area		Entrance		Mooring Area		Fairway	
Period sec	Plan 1	Plan 2	Plan 1	Plan 2	Plan 1	Plan 2	Plan 1	Plan 2	Plan 1	Plan 2
35	0.0	0.0	0.0	0.3	0.0	0.3	0.0	0.3	0.0	0.3
38	0.0	0.0	0.0	0.3	0.0	0.3	0.0	0.3	0.0	0.3
41	0.0	0.6	0.0	0.6	0.0	0.3	0.0	0.9	0.0	0.3
44	0.0	0.5	0.0	0.5	0.0	0.8	0.0	1.1	0.0	0.3
47	0.0	0.5	0.0	0.5	1.0	0.5	1.5	1.0	0.0	0.3
51	0.9	0.5	0.9	0.5	0.9	0.5	0.9	1.2	0.9	0.2
55	0.7	0.4	0.9	0.4	0.9	0.7	1.3	1.3	1.1	0.7
60	0.4	0.6	0.8	1.0	1.2	0.6	2.4	1.6	0.8	0.6
66	0.5	0.9	0.7	1.3	0.7	0.7	2.2	2.5	0.4	0.7
72	0.5	0.7	0.7	1.2	0.7	0.7	1.5	1.8	0.3	0.7
80	0.6	0.6	0.9	1.0	1.1	0.8	1.2	0.9	1.2	0.6
88	1.0	0.8	1.4	1.0	1.1	0.7	1.9	0.8	0.8	0.5
97	0.7	0.5	1.0	0.7	1.5	1.0	1.5	1.0	0.6	0.7
100.2	0.6	0.7	1.1	0.7	1.8	1.1	2.6	1.2	0.6	0.6
114	0.8	0.6	0.8	0.6	2.2	1.3	1.8	1.4	1.7	0.7
124	1.0	0.8	0.8	0.8	1.8	1.2	1.2	1.3	0.8	0.8
132	0.9	0.9	0.7	0.9	1.5	0.9	1.2	1.0	0.9	0.7
138	0.7	0.7	0.5	0.7	0.9	0.9	1.6	0.9	0.5	0.9
144	0.6	0.7	0.3	0.5	1.2	1.0	2.2	0.8	0.5	0.7
158	0.3	0.3	0.3	0.3	0.3	0.3	1.1	0.6	0.3	0.5
172	0.5	0.6	0.4	0.3	0.8	0.6	0.9	0.7	0.3	0.4
185	0.5	0.5	0.7	0.6	0.8	0.6	0.7	0.9	0.2	0.4
205	1.5	1.2	1.3	1.0	0.6	0.4	0.5	0.6	0.5	0.2
225	1.6	1.3	1.6	1.6	0.7	0.6	0.7	0.6	0.5	0.2
234	1.4	1.3	1.3	1.4	1.2	0.7	0.8	0.7	0.4	0.2
257	0.8	1.1	0.6	0.9	1.6	0.9	1.4	0.7	0.5	0.3
280	1.4	1.0	0.8	0.9	1.4	0.9	1.3	0.9	0.5	0.2
305	1.0	1.1	1.0	0.9	1.0	0.8	0.6	0.7	0.2	0.2
330	0.9	0.7	0.4	0.6	0.7	0.7	0.4	0.5	0.4	0.1
360	0.5	0.5	0.3	0.3	0.4	0.3	0.4	0.3	0.1	0.1

* Surface velocities given are the largest mean velocities measured over one complete wave cycle at each location.

Table 7

Estimated Duration and Magnitude of Short-Period Waves from the West, West-Northwest, and Northwest Combined, Occurring in Monterey Harbor for Base Test and Plans 1 and 2

Wave Height* ft	Duration of Short-Period Waves at the Various Locations, hr/yr					
	Entrance			Mooring Area		
	Base Test	Plan 1	Plan 2	Base Test	Plan 1	Plan 2
			Marina			
0-1.5	13,259	13,411	13,411	13,411	13,413	13,413
1.5-2	96	2				
2-3	58		2	2		
			West Basin			
0-1.5	10,143	11,675	11,392	8,550	13,105	13,112
1.5-2	1,115	592	1,416	3,561	266	259
2-3	1,787	983	396	952	42	34
3-4	244	119	165	280		8
4-5	100	36	36	44		
5-6	24	8	8	24		
6-7						
7-8				2		
			Mid-Basin			
0-1.5		13,401	13,145		13,337	13,337
1.5-2		12	266		76	76
2-3			2			
			East Basin			
0-1.5		13,199	13,154		13,259	13,206
1.5-2		110	123		122	149
2-3		70	92		24	50
3-4		24	32		8	8
4-5			2			
5-6		10	10			
			Fairway			
0-1.5		10,872	12,503			
1.5-2		1,736	649			
2-3		564	125			
3-4		165	92			
4-5		32	22			
5-6		32	10			
6-7		12	8			
7-8			4			

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

Table 8

Estimated Duration and Magnitude of Short-Period Waves from the West, West-Northwest, and Northwest Combined, Occurring in the East Basin for Plans 1 and 2

Wave Height* ft	Duration of Short-Period Waves at the Various Locations, hr/yr							
	Entrance		Mooring Area I		Mooring Area II		Mooring Area III	
	Plan 1	Plan 2	Plan 1	Plan 2	Plan 1	Plan 2	Plan 1	Plan 2
0-1.5	13,199	13,154	13,411	13,413	13,309	13,323	13,259	13,206
1.5-2	110	123	2		72	56	152	171
2-3	70	92			24	26	2	36
3-4	24	32			8	8		
4-5		2						
5-6	10	10						

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

Table 9

Comparison of Short-Period Wave Height Data for 7- and 9-sec Test Waves from North 35° West

Wave Gage No.	Wave Gage Location	Comparison of Wave Heights for Base Test and Plans 1 and 2, ft					
		7-sec, 7-ft Test Wave			9-sec, 9-ft Test Wave		
		Base Test	Plan 1	Plan 2	Base Test	Plan 1	Plan 2
5	Marina	0.5	0.3	0.5	0.4	0.4	0.3
6	Marina	1.0	0.9	0.8	1.2	1.0	0.8
7	Marina	0.3	0.3	0.3	0.4	0.3	0.3
8	Marina	0.4	0.3	0.2	0.3	0.3	0.3
9	Marina	0.2	0.2	0.2	0.2	0.2	0.2
10	Mid-Basin		0.2	0.3		0.3	0.3
12	Mid-Basin	1.2	0.7	0.7	2.9	1.3	1.5
14	Mid-Basin		0.6	0.6		0.7	0.6
16	Mid-Basin		0.3	0.3		0.3	0.3
18	Mid-Basin		0.7	0.8		0.7	0.9
19	East Basin		0.3	0.3		0.3	0.3
21	East Basin		0.6	0.6		0.6	0.6
23	East Basin		0.3	0.3		0.3	0.3
25	East Basin		0.7	0.8		1.0	0.9
26	East Basin		0.6	0.6		1.0	0.8
27	East Basin		0.6	0.5		0.6	0.6
30	East Basin		1.2	1.5		1.2	1.4
28*	Fairway		1.3	1.2		2.4	1.2
33	Fairway		1.0	1.1		0.9	1.3
35	Fairway		1.0	1.1		1.6	1.6
36	West Basin	2.0	0.8	0.8	2.7	0.6	0.7
37	West Basin	1.4	0.8	1.1	1.2	1.1	1.3
38	West Basin	0.6	0.7	0.9	1.2	1.2	1.2
39	West Basin	1.2	0.9	0.9	1.1	1.0	0.6
41	West Basin	2.5	1.1	1.0	0.8	0.8	0.6
42	West Basin	0.5	0.8	0.6	1.2	0.8	0.7
44	West Basin	1.6	0.8	0.6	3.3	0.8	1.1
45	West Basin	1.8	1.2	1.4	2.5	2.6	2.3

Note: Gage locations are shown in plates 5-7.
* Gage 28 was used for plan 1; gage 29 was used for plan 2.

Table 10

Comparison of Short-Period Wave Height Data for 11-sec Test Waves from North 35° West

Wave Gage No.	Wave Gage Location	Comparison of Wave Heights for Base Test and Plans 1 and 2, ft					
		11-sec, 7-ft Test Wave			11-sec, 13-ft Test Wave		
		Base Test	Plan 1	Plan 2	Base Test	Plan 1	Plan 2
5	Marina	0.6	0.3	0.5	0.7	0.5	0.9
6	Marina	1.3	1.0	0.8	1.2	1.0	1.4
7	Marina	0.4	0.4	0.3	0.4	0.4	0.3
8	Marina	0.4	0.3	0.3	0.5	0.3	0.3
9	Marina	0.2	0.2	0.2	0.2	0.2	0.2
10	Mid-Basin		0.6	0.5		0.7	0.5
12	Mid-Basin	1.6	1.2	1.5	3.1	1.4	2.0
14	Mid-Basin		0.6	0.7		1.1	1.1
16	Mid-Basin		0.3	0.5		0.6	0.8
18	Mid-Basin		0.7	0.6		0.7	0.7
19	East Basin		0.3	0.2		0.3	0.3
21	East Basin		0.7	0.7		0.8	0.8
23	East Basin		0.6	0.3		0.6	0.5
25	East Basin		1.0	0.6		1.3	1.2
26	East Basin		1.0	1.0		1.2	1.0
27	East Basin		0.9	0.8		1.2	0.9
30	East Basin		0.9	0.7		1.4	1.4
28*	Fairway		2.1	1.2		3.7	2.2
33	Fairway		1.1	1.2		1.8	2.1
35	Fairway		1.2	0.9		1.7	1.4
36	West Basin	2.8	0.6	0.7	5.4	0.7	0.9
37	West Basin	1.1	1.2	1.2	1.5	2.0	1.6
38	West Basin	0.6	1.0	0.7	1.2	1.3	1.3
39	West Basin	1.7	1.4	1.4	2.0	2.0	1.9
41	West Basin	1.3	0.7	0.7	2.4	1.0	1.1
42	West Basin	1.5	1.0	1.0	1.8	1.6	1.3
44	West Basin	2.6	1.1	0.8	4.0	1.7	1.2
45	West Basin	1.9	2.4	2.5	3.0	4.9	4.4

Note: Gage locations are shown in plates 5-7.
* Gage 28 was used for plan 1; gage 29 was used for plan 2.

Table 11

Comparison of Short-Period Wave Height Data for 13-sec Test Waves from North 35° West

Wave Gage No.	Wave Gage Location	Comparison of Wave Heights for Base Test and Plans 1 and 2, ft					
		13-sec, 7-ft Test Wave			13-sec, 13-ft Test Wave		
		Base Test	Plan 1	Plan 2	Base Test	Plan 1	Plan 2
5	Marina	0.4	0.3	0.5	0.9	0.6	0.6
6	Marina	1.4	0.8	0.7	2.7	1.2	1.4
7	Marina	0.3	0.4	0.3	0.4	0.5	0.3
8	Marina	0.5	0.3	0.3	0.5	0.3	0.3
9	Marina	0.2	0.2	0.2	0.2	0.2	0.2
10	Mid-Basin		0.5	0.9		0.7	1.2
12	Mid-Basin	4.9	0.7	0.7	11.8	1.9	1.6
14	Mid-Basin		1.4	1.3		1.9	1.9
16	Mid-Basin		0.8	0.9		1.2	1.3
18	Mid-Basin		0.7	0.6		0.7	0.7
19	East Basin		0.4	0.3		0.3	0.7
21	East Basin		0.6	0.6		0.9	0.8
23	East Basin		0.6	0.5		1.1	1.0
25	East Basin		1.2	1.0		3.4	3.1
26	East Basin		1.3	1.2		1.8	2.2
27	East Basin		1.0	0.4		1.8	1.8
30	East Basin		1.9	1.5		5.1	5.1
28*	Fairway		2.6	1.8		5.8	6.0
33	Fairway		2.2	2.4		6.6	6.5
35	Fairway		2.2	1.7		4.1	4.0
36	West Basin	3.4	1.4	1.5	5.3	2.8	3.0
37	West Basin	1.5	1.2	1.3	2.0	1.9	2.5
38	West Basin	1.2	1.4	1.4	2.0	2.0	2.2
39	West Basin	1.9	1.3	1.0	2.9	2.1	1.9
41	West Basin	1.7	0.9	1.3	5.6	1.9	1.7
42	West Basin	1.0	0.7	1.0	1.9	1.9	1.8
44	West Basin	1.7	1.4	1.4	4.3	2.5	2.7
45	West Basin	1.9	2.0	2.4	4.0	5.1	5.0

Note: Gage locations are shown in plates 5-7.
* Gage 28 was used for plan 1; gage 29 was used for plan 2.

Table 12

Comparison of Short-Period Wave Height Data for 15-sec Test Waves from North 35° West

Wave Gage No.	Wave Gage Location	Comparison of Wave Heights for Base Test and Plans 1 and 2, ft					
		15-sec, 7-ft Test Wave			15-sec, 13-ft Test Wave		
		Base Test	Plan 1	Plan 2	Base Test	Plan 1	Plan 2
5	Marina	0.6	0.5	0.5	1.3	0.6	1.0
6	Marina	1.4	1.1	0.8	2.5	1.2	1.3
7	Marina	0.3	0.4	0.3	0.6	0.3	0.5
8	Marina	0.4	0.2	0.3	0.4	0.3	0.3
9	Marina	0.2	0.2	0.2	0.3	0.3	0.2
10	Mid-Basin		0.6	0.6		0.9	1.1
12	Mid-Basin	1.8	0.9	1.0	4.9	1.6	1.5
14	Mid-Basin		0.7	0.7		0.7	0.7
16	Mid-Basin		0.6	0.6		0.9	1.0
18	Mid-Basin		0.7	0.6		0.7	1.0
19	East Basin		0.3	0.2		0.3	0.4
21	East Basin		0.8	0.6		0.8	0.8
23	East Basin		0.4	0.3		1.1	1.1
25	East Basin		1.0	0.7		2.0	2.0
26	East Basin		1.3	1.7		1.7	2.4
27	East Basin		1.0	0.7		1.6	0.7
30	East Basin		1.1	1.5		3.5	4.0
28*	Fairway		4.0	2.4		6.7	5.9
33	Fairway		2.2	2.4		6.8	7.0
35	Fairway		1.4	1.5		2.9	2.2
36	West Basin	3.0	1.2	1.4	5.4	2.0	2.3
37	West Basin	1.0	1.2	1.4	1.6	1.8	2.4
38	West Basin	1.3	1.2	1.1	2.1	1.4	1.4
39	West Basin	0.6	1.6	1.1	1.6	2.4	1.9
41	West Basin	1.7	1.2	1.1	3.9	1.5	1.6
42	West Basin	1.2	1.4	1.0	1.3	1.8	1.6
44	West Basin	2.9	1.4	1.2	5.5	1.9	1.8
45	West Basin	2.9	2.0	2.0	5.0	4.7	4.1

Note: Gage locations are shown in plates 5-7.
* Gage 28 was used for plan 1; gage 29 was used for plan 2.

Table 13

Comparison of Short-Period Wave Height Data for 17-sec Test Waves from North 35° West

Wave Gage No.	Wave Gage Location	Comparison of Wave Heights for Base Test and Plans 1 and 2, ft					
		17-sec, 7-ft Test Wave			17-sec, 13-ft Test Wave		
		Base Test	Plan 1	Plan 2	Base Test	Plan 1	Plan 2
5	Marina	0.9	0.5	0.6	1.7	0.7	1.3
6	Marina	1.7	1.2	1.2	2.9	1.9	2.1
7	Marina	0.6	0.3	0.3	0.9	0.4	0.5
8	Marina	0.3	0.3	0.3	2.2	0.3	0.3
9	Marina	0.2	0.2	0.2	0.4	0.3	0.2
10	Mid-Basin		0.2	0.4		0.7	0.7
12	Mid-Basin	5.3	1.2	1.2	10.4	1.9	2.4
14	Mid-Basin		1.0	0.7		1.3	1.1
16	Mid-Basin		0.7	0.5		1.1	1.1
18	Mid-Basin		0.7	0.7		1.1	0.8
19	East Basin		0.3	0.3		0.4	0.8
21	East Basin		0.6	0.8		0.8	0.9
23	East Basin		0.5	0.4		1.8	1.1
25	East Basin		1.2	1.0		2.5	2.1
26	East Basin		1.2	1.7		2.1	2.7
27	East Basin		1.0	1.0		2.2	2.6
30	East Basin		1.9	1.4		5.9	5.7
28*	Fairway		3.3	1.8		6.6	7.1
33	Fairway		1.3	1.6		4.6	4.1
35	Fairway		1.7	1.7		3.7	3.8
36	West Basin	2.7	0.8	0.6	5.4	1.2	1.5
37	West Basin	1.4	0.7	0.8	2.0	1.8	2.0
38	West Basin	1.0	1.2	1.1	1.7	1.4	1.3
39	West Basin	1.2	1.4	1.2	2.3	1.8	1.7
41	West Basin	3.7	0.7	1.0	7.9	1.7	1.4
42	West Basin	1.0	1.0	1.0	1.4	1.3	1.7
44	West Basin	2.0	1.2	0.8	4.5	1.8	1.4
45	West Basin	2.3	1.6	1.5	4.8	3.9	3.5

Note: Gage locations are shown in plates 5-7.

* Gage 28 was used for plan 1; gage 29 was used for plan 2.

Table 14

Comparison of Short-Period Wave Height Data for Locally Generated Waves from North

Wave Gage No.	Wave Gage Location	Comparison of Wave Heights for Base Test and Plans 1 and 2, ft					
		8-sec, 9-ft Test Wave			8-sec, 12-ft Test Wave		
		Base Test	Plan 1	Plan 2	Base Test	Plan 1	Plan 2
5	Marina	1.5	0.7	1.0	1.6	1.1	1.0
6	Marina	2.6	1.2	1.5	3.9	1.6	2.1
7	Marina	0.5	0.3	0.4	0.6	0.3	0.5
8	Marina	0.6	0.2	0.3	0.6	0.3	0.3
9	Marina	0.4	0.1	0.2	0.5	0.2	0.2
10	Mid-Basin		0.3	0.4		0.5	0.5
12	Mid-Basin	13.2	1.7	1.4	14.0	2.3	2.6
14	Mid-Basin		0.6	0.5		0.7	0.6
16	Mid-Basin		0.2	0.4		0.3	0.3
18	Mid-Basin		1.0	1.0		1.1	1.1
19	East Basin		0.3	0.4		0.6	0.3
21	East Basin		0.6	0.6		0.5	0.7
23	East Basin		0.6	0.4		0.6	0.4
25	East Basin		0.8	0.8		1.0	1.1
26	East Basin		1.0	1.1		1.1	1.5
27	East Basin		0.7	0.5		0.9	0.5
30	East Basin		1.5	1.7		3.1	3.8
28	Fairway		5.2			6.6	
33	Fairway		1.7	1.8		3.2	5.0
35	Fairway		2.7	2.9		4.5	5.2
36	West Basin	4.3	0.9	1.4	6.8	1.5	1.8
37	West Basin	1.5	1.3	1.3	2.0	1.8	1.4
38	West Basin	2.1	2.1	2.3	2.1	2.6	2.3
39	West Basin	1.8	1.6	2.0	2.8	2.5	2.5
41	West Basin	7.6	1.1	1.5	9.9	1.7	2.3
42	West Basin	1.8	0.9	1.2	2.3	1.7	1.4
44	West Basin	3.3	0.7	0.7	6.0	1.5	1.5
45	West Basin	4.5	3.7	4.6	5.8	5.3	6.6

Note: Gage locations are shown in plates 5-7.

Table 15

Comparison of Short-Period Wave Height Data for 8-sec Test Waves from North

Wave Gage No.	Wave Gage Location	Comparison of Wave Heights for Plans 1, 1A, 1B, 1C, 1D, 1E, and 1F, ft: 8-sec, 9-ft Test Wave, Plan 1	Plan 1A	Plan 1B	Plan 1C	Plan 1D	Plan 1E	Plan 1F	8-sec, 12-ft Test Wave, Plan 1	Plan 1A	Plan 1B	Plan 1C	Plan 1D	Plan 1E	Plan 1F
19	East Basin	0.3	0.5	0.3	0.3	0.4	0.2	0.2	0.6	0.3	0.3	0.3	0.3	0.2	0.2
21	East Basin	0.6	0.5	0.4	0.2	0.4	0.2	0.3	0.5	0.3	0.5	0.4	0.2	0.2	0.2
22	East Basin	0.5	0.6	0.4	0.4	0.5	0.3	0.3	0.6	0.7	0.5	0.4	0.4	0.4	0.4
23	East Basin	0.6	0.3	0.5	0.5	0.6	0.3	0.4	0.6	0.6	0.6	0.6	0.5	0.4	0.4
24	East Basin	0.4	0.3	0.4	0.4	0.5	0.3	0.3	0.3	0.4	0.4	0.4	0.5	0.4	0.3
25	East Basin	0.8	0.8	0.7	0.7	0.8	1.0	1.0	1.0	1.0	1.3	1.0	1.3	1.1	1.1
26	East Basin	1.0	1.0	0.7	1.0	0.9	0.9	0.9	1.1	1.3	1.3	1.1	1.1	1.2	1.2
27	East Basin	0.7	0.7	0.4	0.5	0.5	0.6	0.5	0.9	0.5	0.7	0.4	0.5	0.5	0.5
30	East Basin	1.5	1.4	1.4	1.7	1.7	1.8	1.6	3.1	2.1	2.4	2.5	2.5	2.5	2.2
28	Fairway	5.2	5.1	5.0	5.1	6.7	9.2	9.8	6.6	6.5	8.3	8.3	9.3	13.1	14.5
29	Fairway	0.8	1.5	2.5	2.2	2.2	1.5	2.0	1.3	2.3	3.1	3.1	2.9	2.1	2.4
31	Fairway	2.2	2.3	1.9	1.7	1.3	1.6	1.8	2.5	3.0	3.1	2.4	2.1	2.2	2.7
32	Fairway	1.7	1.5	1.3	1.1	1.3	1.5	1.6	2.3	2.6	2.9	2.5	2.8	2.8	2.9

Note: Gage locations are shown in plates 6 and 7.

Table 16

Comparison of Short-Period Wave Height Data for 11-sec Test Waves from North 35° West

Wave Gage No.	Wave Gage Location	Comparison of Wave Heights for Plans 1, 1A, 1B, 1C, 1D, 1E, and 1F, ft													
		11-sec, 7-ft Test Wave							11-sec, 13-ft Test Wave						
		Plan 1	Plan 1A	Plan 1B	Plan 1C	Plan 1D	Plan 1E	Plan 1F	Plan 1	Plan 1A	Plan 1B	Plan 1C	Plan 1D	Plan 1E	Plan 1F
19	East Basin	0.3	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.2	0.2	0.2	0.2	0.2	0.3
21	East Basin	0.7	0.2	0.2	0.2	0.2	0.2	0.3	0.8	0.2	0.2	0.2	0.3	0.3	0.2
22	East Basin	0.3	0.3	0.3	0.2	0.3	0.3	0.4	0.2	0.3	0.2	0.3	0.3	0.3	0.4
23	East Basin	0.6	0.3	0.3	0.6	0.4	0.5	0.5	0.6	0.3	0.4	0.4	0.5	0.6	0.6
24	East Basin	0.2	0.3	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.2	0.3	0.3	0.4
25	East Basin	1.0	0.7	0.6	0.5	0.5	0.6	1.0	1.3	1.2	1.0	0.9	0.7	1.1	1.2
26	East Basin	1.0	0.6	0.5	0.8	0.5	0.6	0.6	1.2	0.8	0.6	0.5	0.8	0.8	0.8
27	East Basin	0.9	0.6	0.4	0.3	0.4	0.4	0.6	1.2	0.5	0.4	0.5	0.6	0.8	0.9
30	East Basin	0.9	0.7	0.7	1.2	1.0	0.6	1.2	1.4	1.4	1.5	1.5	1.3	1.7	1.7
28	Fairway	2.1	2.9	3.2	3.6	4.0	4.0	5.4	3.7	6.0	6.7	6.6	8.0	9.0	10.8
29	Fairway	0.6	1.2	1.3	1.9	1.9	1.6	2.0	1.4	2.2	2.7	3.3	3.2	2.9	2.9
31	Fairway	0.7	0.9	0.7	1.1	1.2	1.4	1.5	2.0	1.7	1.6	2.0	2.5	3.2	2.7
32	Fairway	1.6	1.6	1.5	1.3	1.5	1.5	2.1	3.3	3.1	3.3	3.2	3.1	3.2	3.6

Note: Gage locations are shown in plates 6 and 7.

Table 17

Comparison of Short-Period Wave Height Data for 17-sec Test Waves from North 35° West

Wave Gage No.	Wave Gage Location	Comparison of Wave Heights for Plans 1, 1A, 1B, 1C, 1D, 1E, and 1F, ft — 17-sec, 7-ft Test Wave: Plan 1	Plan 1A	Plan 1B	Plan 1C	Plan 1D	Plan 1E	Plan 1F	17-sec, 13-ft Test Wave: Plan 1	Plan 1A	Plan 1B	Plan 1C	Plan 1D	Plan 1E	Plan 1F
19	East Basin	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.4	0.4	0.4	0.5
21	East Basin	0.6	0.4	0.4	0.4	0.4	0.9	0.4	0.8	0.4	0.4	0.5	0.5	0.6	0.5
22	East Basin	0.4	0.5	0.5	0.6	0.6	0.5	0.4	0.6	0.9	0.8	0.9	0.8	0.9	0.8
23	East Basin	0.5	0.5	0.5	0.6	0.6	0.6	0.5	1.8	1.2	1.3	1.4	1.4	1.4	1.2
24	East Basin	0.5	0.7	0.7	0.8	0.6	0.6	0.5	1.3	1.5	1.6	1.6	1.6	1.5	1.2
25	East Basin	1.2	1.0	1.2	1.5	1.1	1.1	1.2	2.5	2.5	2.5	2.7	2.8	2.8	2.3
26	East Basin	1.2	1.3	1.4	1.5	1.4	1.4	1.5	2.1	2.1	1.9	2.0	2.4	2.2	2.2
27	East Basin	1.0	0.5	0.5	0.4	0.4	0.6	0.5	2.2	1.4	1.2	1.5	1.4	1.6	1.4
30	East Basin	1.9	2.2	1.9	1.7	1.6	1.7	1.6	5.9	5.6	5.6	5.9	5.4	5.3	4.6
28	Fairway	3.3	5.1	5.3	6.4	6.4	6.6	8.1	6.6	9.9	12.0	12.8	13.2	13.6	13.5
29	Fairway	1.2	1.8	1.6	2.0	1.9	1.9	2.4	5.7	6.1	6.4	6.4	5.6	5.3	5.9
31	Fairway	0.8	0.8	1.1	1.0	1.1	1.2	1.2	3.1	3.0	3.2	4.0	4.1	4.2	4.2
32	Fairway	0.9	0.8	0.8	0.8	0.8	0.9	1.1	3.4	2.9	3.5	3.8	4.0	3.9	3.5

Note: Gage locations are shown in plates 6 and 7.

Table 18

Short-Period Wave Heights at Center Line of North Breakwater*

Test Wave		Significant Wave Heights Along Breakwater Center Line, ft									
Period sec	Height ft	Gage 46	Gage 47	Gage 48	Gage 49	Gage 50	Gage 51	Gage 52	Gage 53	Gage 54	Gage 55
7	7	0.6	2.3	3.0	2.9	4.2	4.7	5.5	5.4	4.1	5.0
9	9	2.1	2.1	4.1	5.4	5.4	6.8	6.7	4.6	6.6	7.4
11	13	1.8	6.3	6.8	10.8	11.0	9.4	11.7	11.7	7.4	15.8
13	13	5.5	6.4	8.1	10.6	11.2	10.9	11.8	17.7	13.0	16.6
15	13	6.0	7.3	9.1	8.8	9.7	11.5	10.0	11.7	11.8	17.8
17	13	3.5	6.4	9.9	10.7	10.5	9.7	16.2	14.1	14.1	16.4

Note: Gage locations are shown in plate 12.
* The breakwater structure was removed for these tests.

Table 19

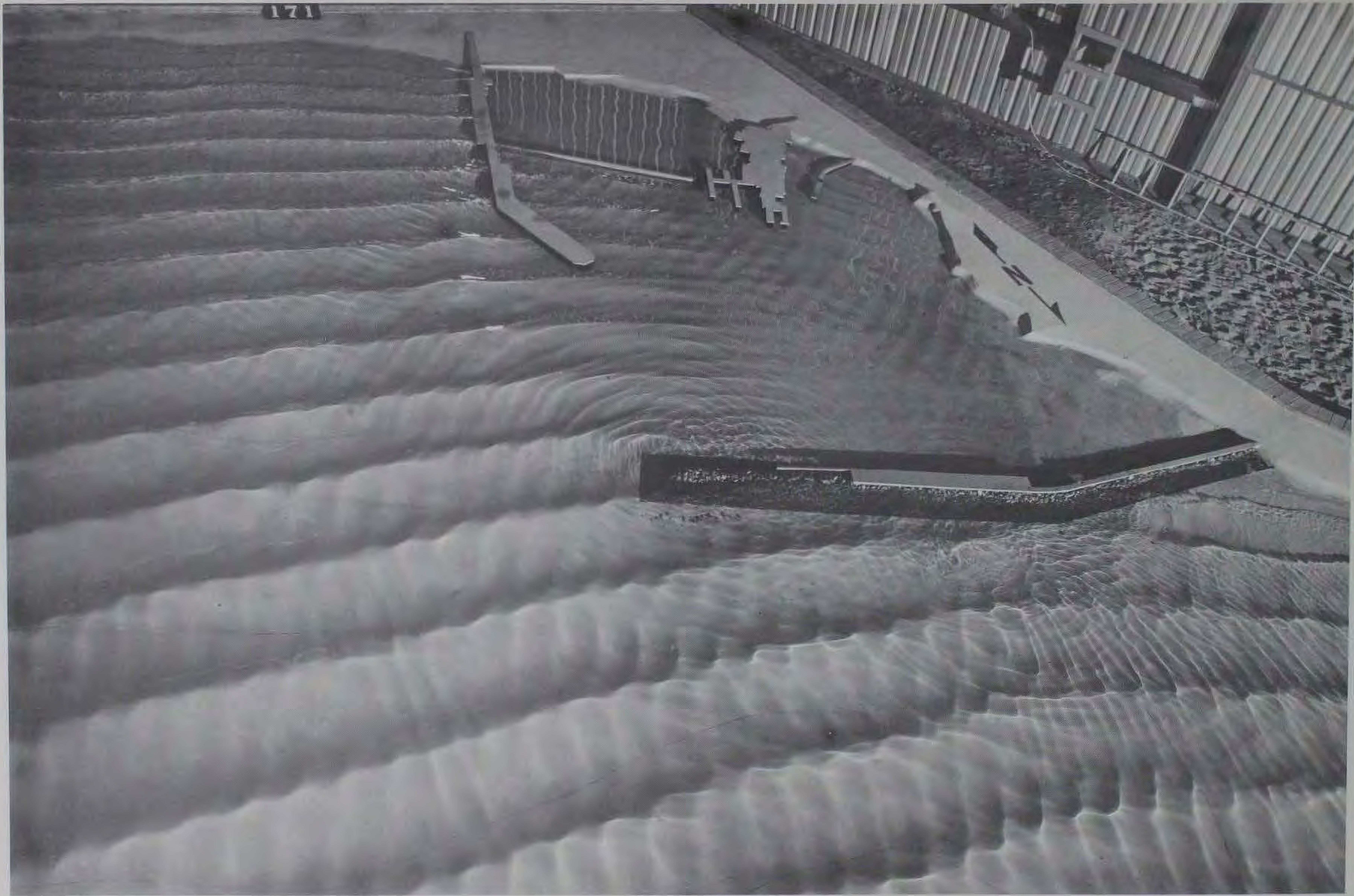
Short-Period Wave Heights at Center Line of Existing Breakwater*

Test Wave		Significant Wave Heights Along Breakwater Center Line, ft				
Period sec	Height ft	Gage 56	Gage 57	Gage 58	Gage 59	Gage 60
7	7	4.3	3.0	3.0	2.6	3.0
9	9	5.9**	3.9	4.0	2.8	4.3
11	13	7.5**	6.9	6.1	7.5	8.0
13	13	7.4**	9.0	6.8	7.4	8.9
15	13	7.3**	7.1	6.7	6.9	7.6
17	13	9.1**	10.4	6.8	8.5	8.5

Note: Gage locations are shown in plate 13.

* The breakwater structure was removed for these tests.

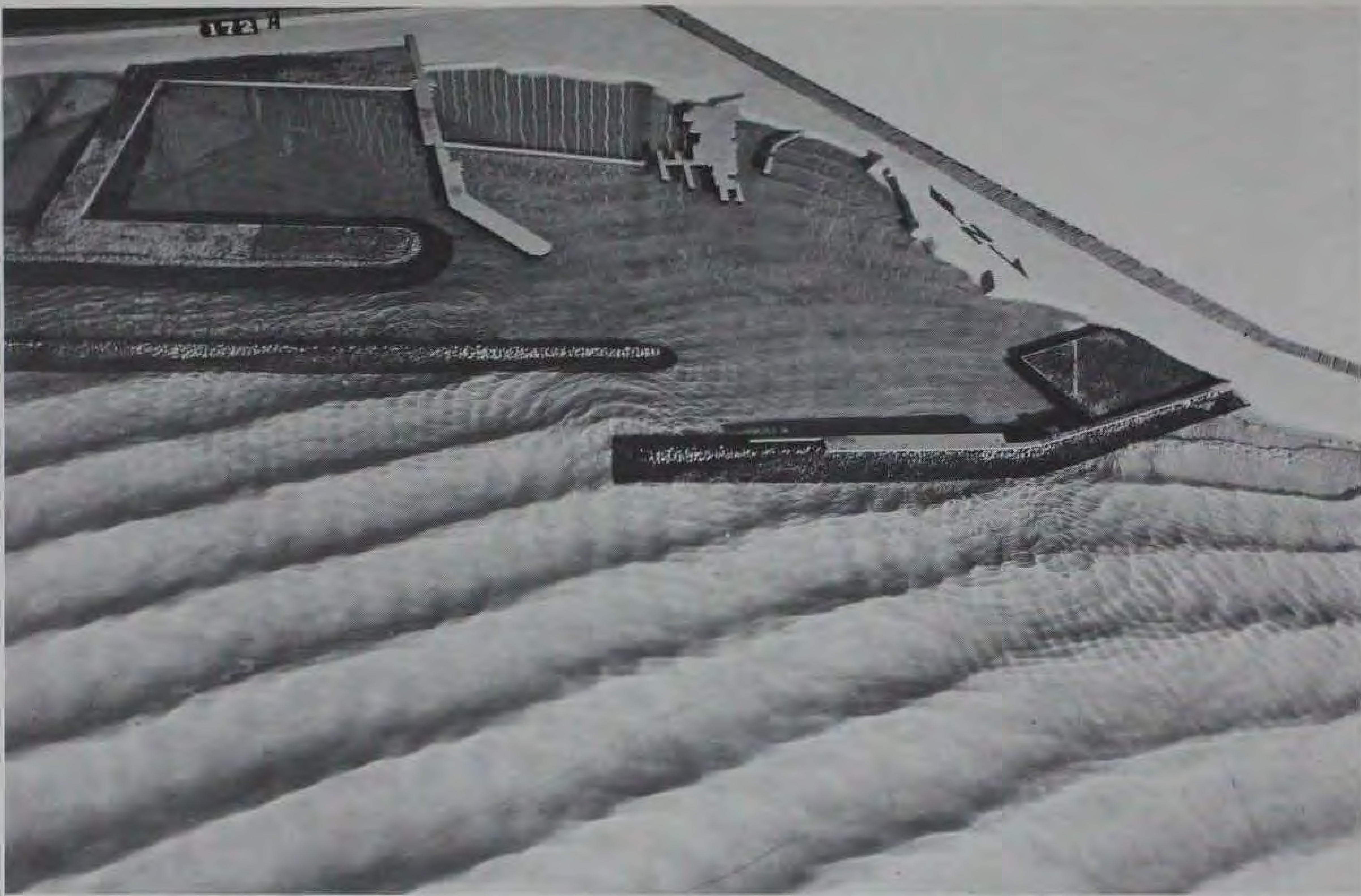
** Test wave broke before reaching gage 56.



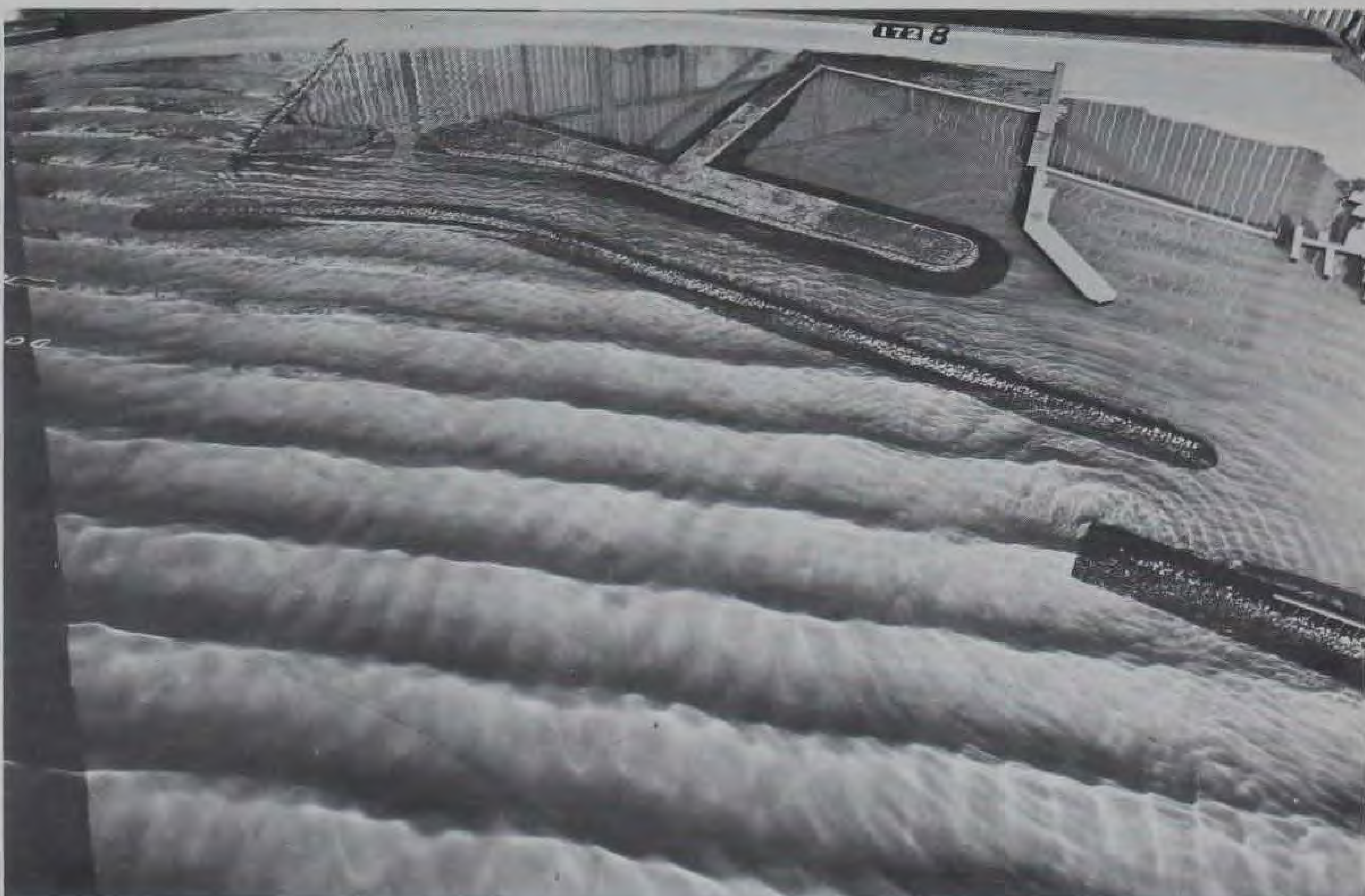
Photograph 1. Base test; 8-sec, 12-ft waves from north

Photograph 2. Base test; 9-sec, 9-ft waves from north 35° west

Photograph 3. Base test; 17-sec, 13-ft waves from north 35° west



a. West view



b. East view

Photograph 4. Plan 1; 8-sec, 12-ft waves from north

a. West view

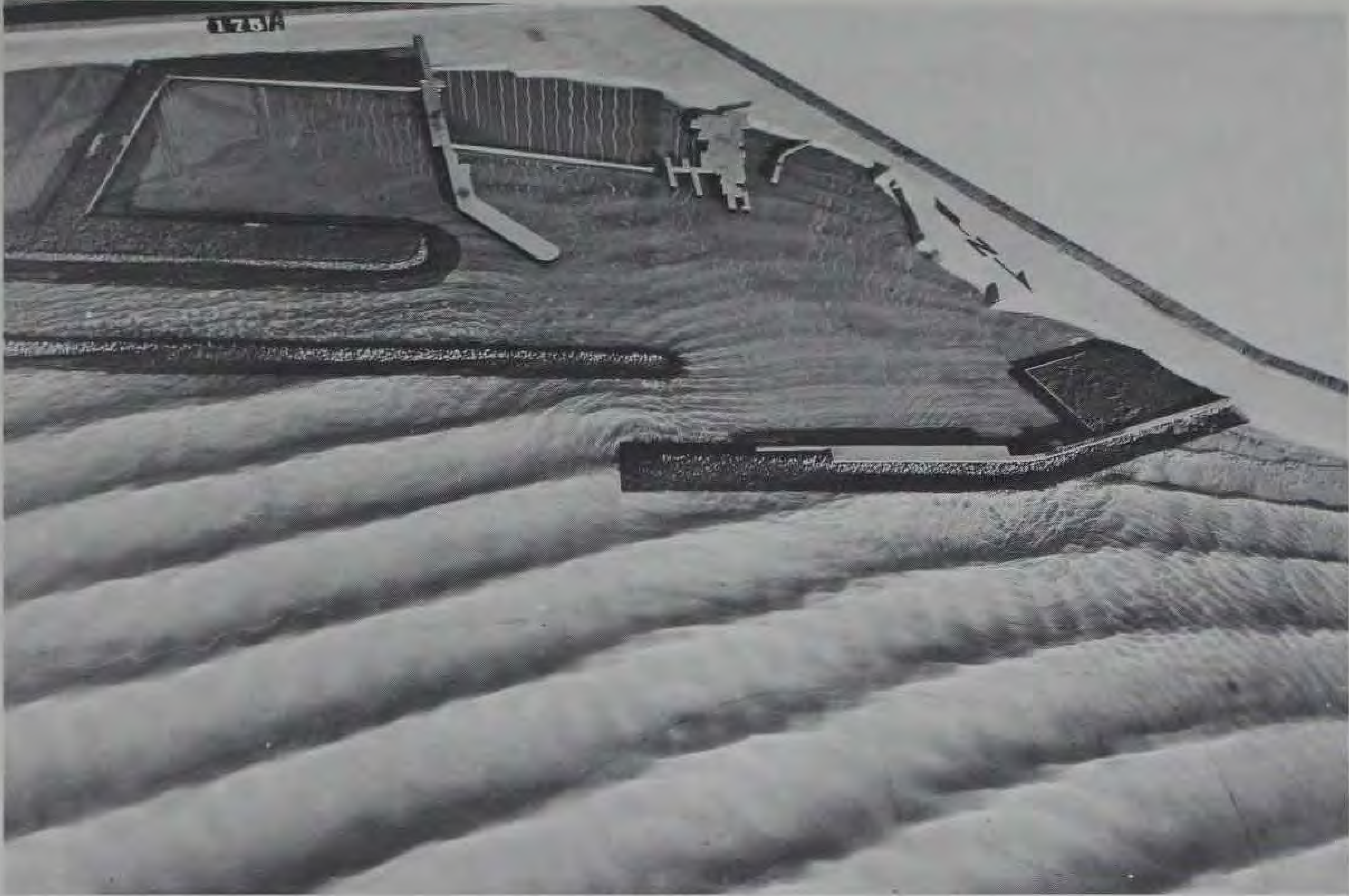
b. East view

Photograph 5. Plan 1; 9-sec, 9-ft waves from north 35° west

a. West view

b. East view

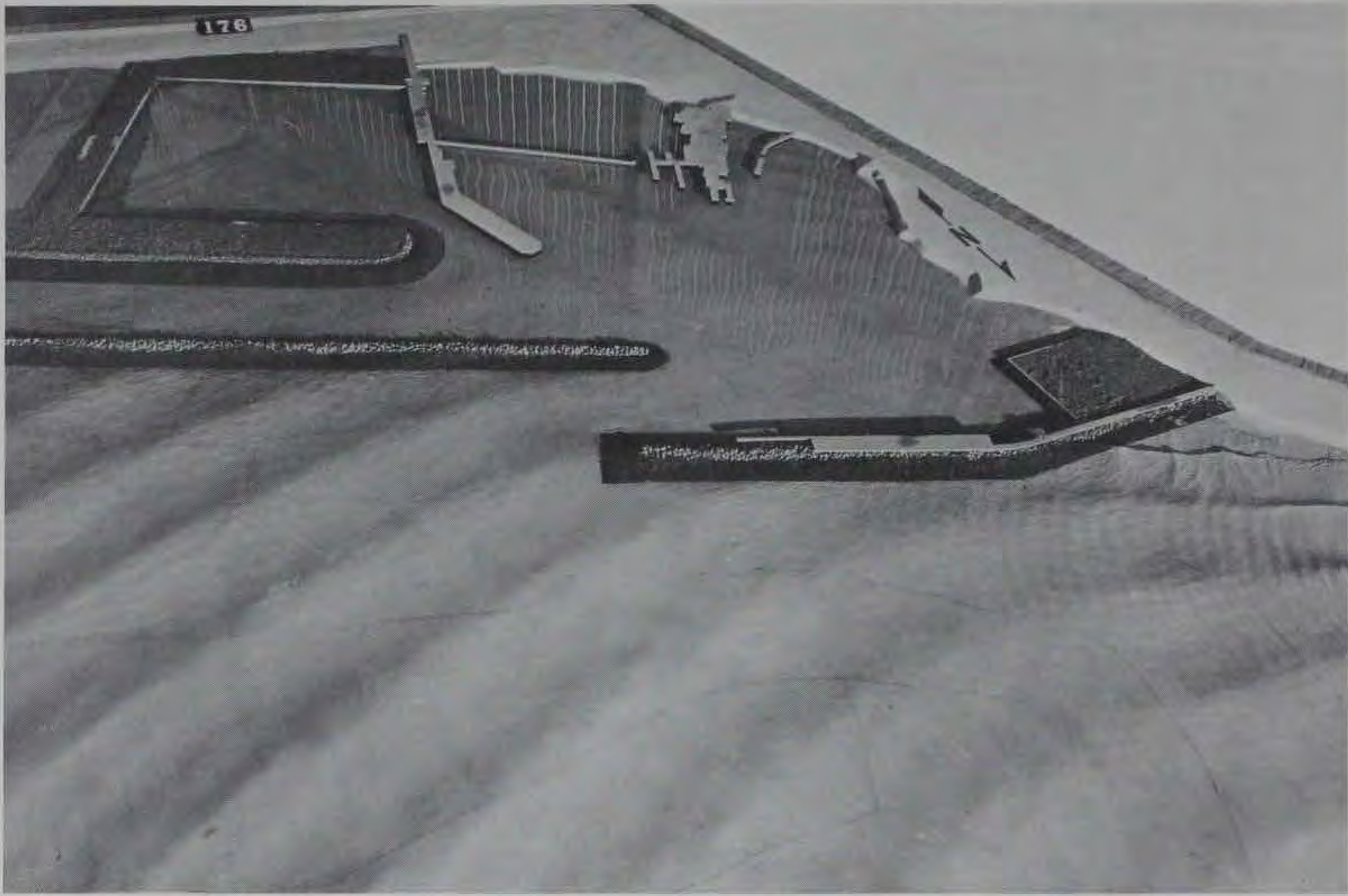
Photograph 6. Plan 1; 17-sec, 13-ft waves from north 35° west



a. West view

b. East view

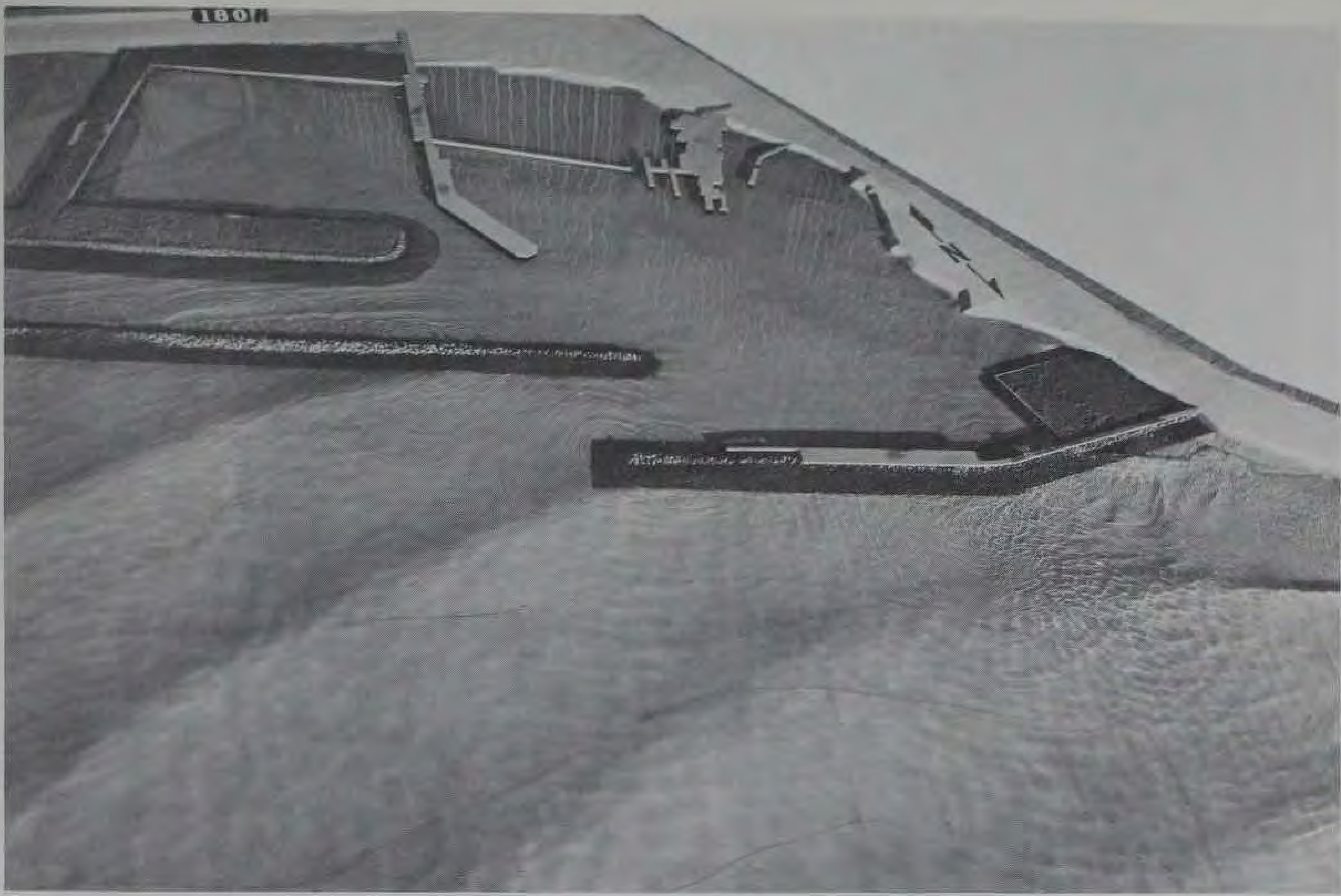
Photograph 7. Plan 2; 8-sec, 12-ft waves from north



a. West view

b. East view

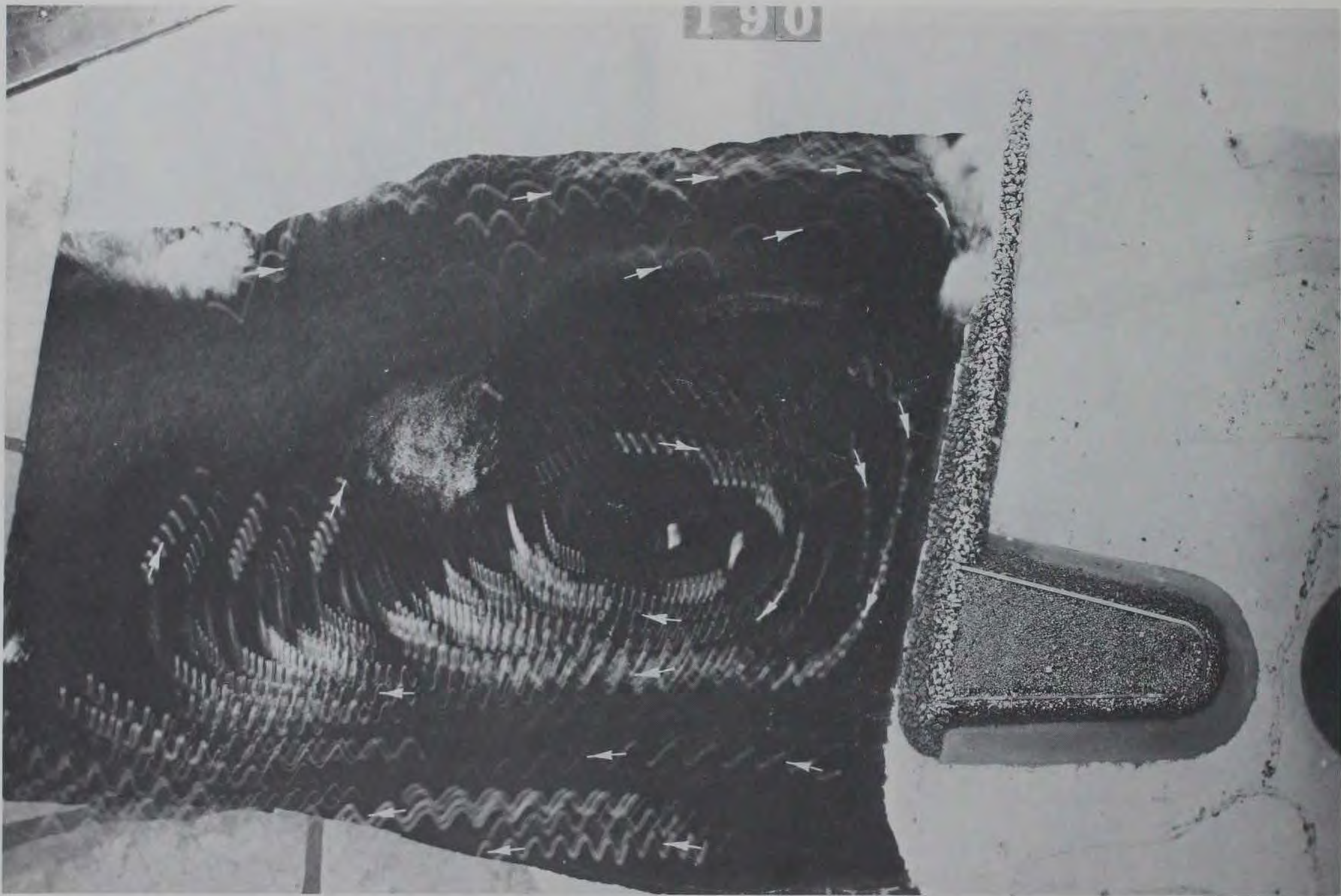
Photograph 8. Plan 2; 9-sec, 9-ft waves from north 35° west



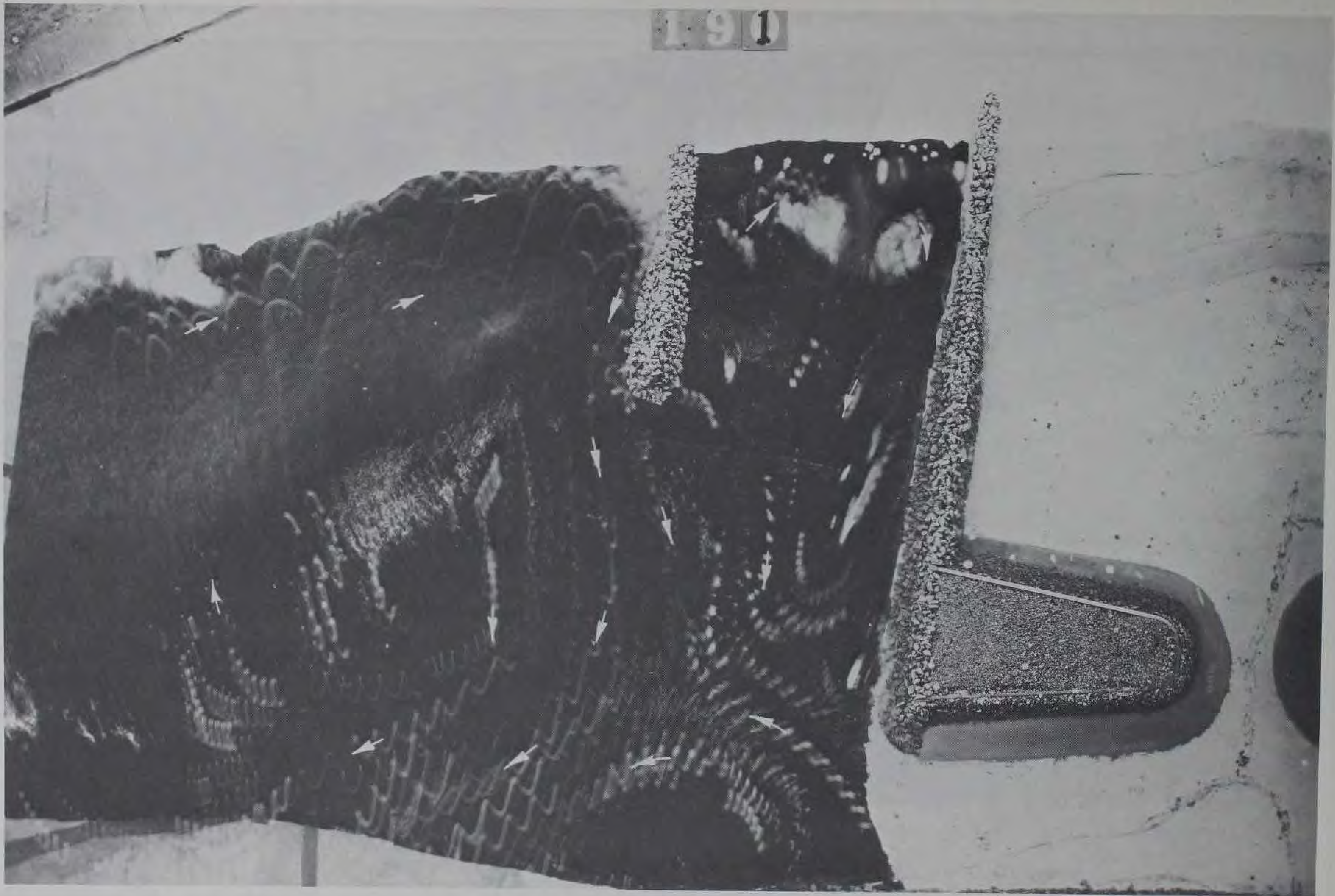
a. West view

b. East view

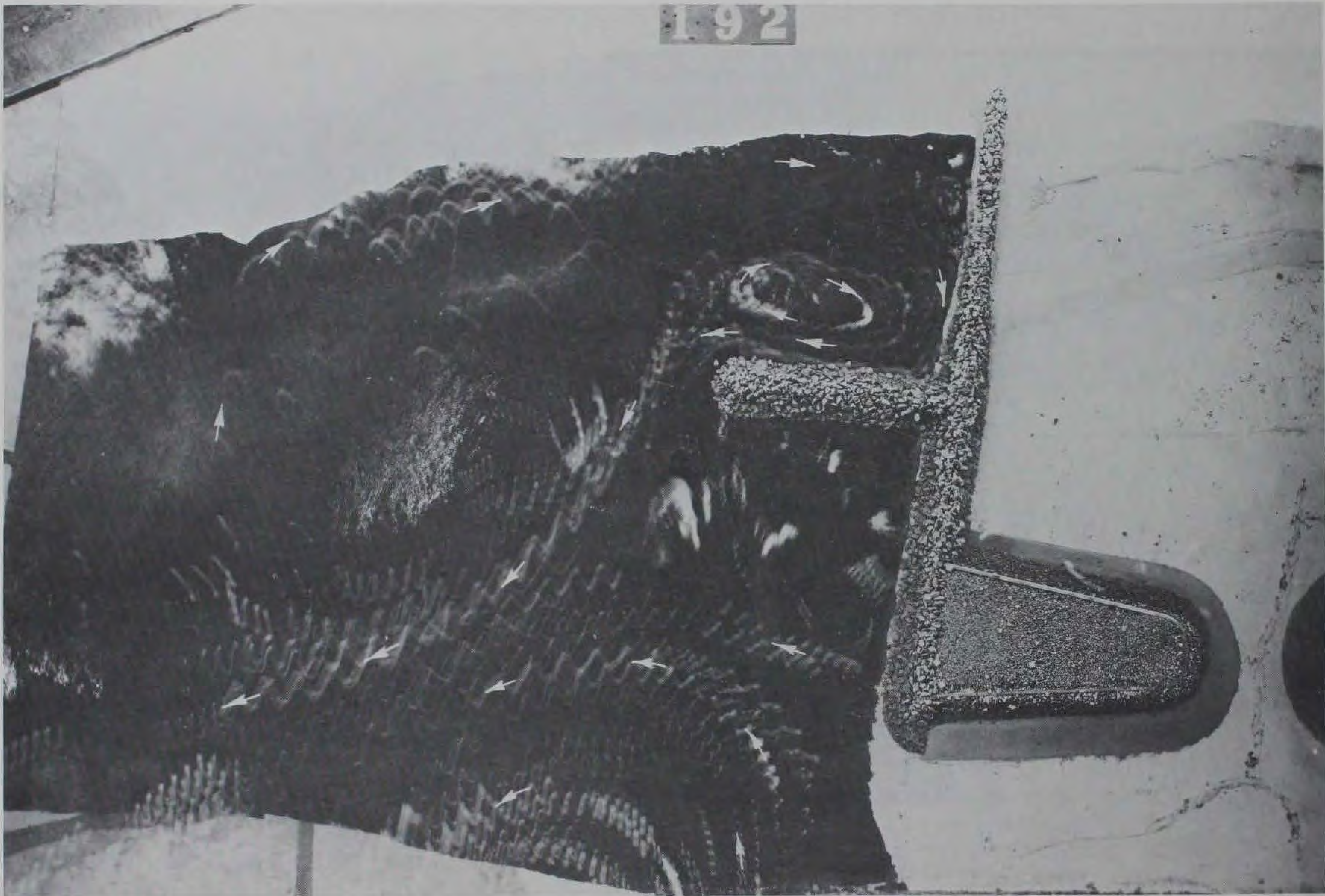
Photograph 9. Plan 2; 17-sec, 13-ft waves from north 35° west



Photograph 10. Plan 1; 17-sec, 13-ft waves from north 35° west



Photograph 11. Plan 1G; 17-sec, 13-ft waves from north 35° west



Photograph 12. Plan 1H; 17-sec, 13-ft waves from north 35° west

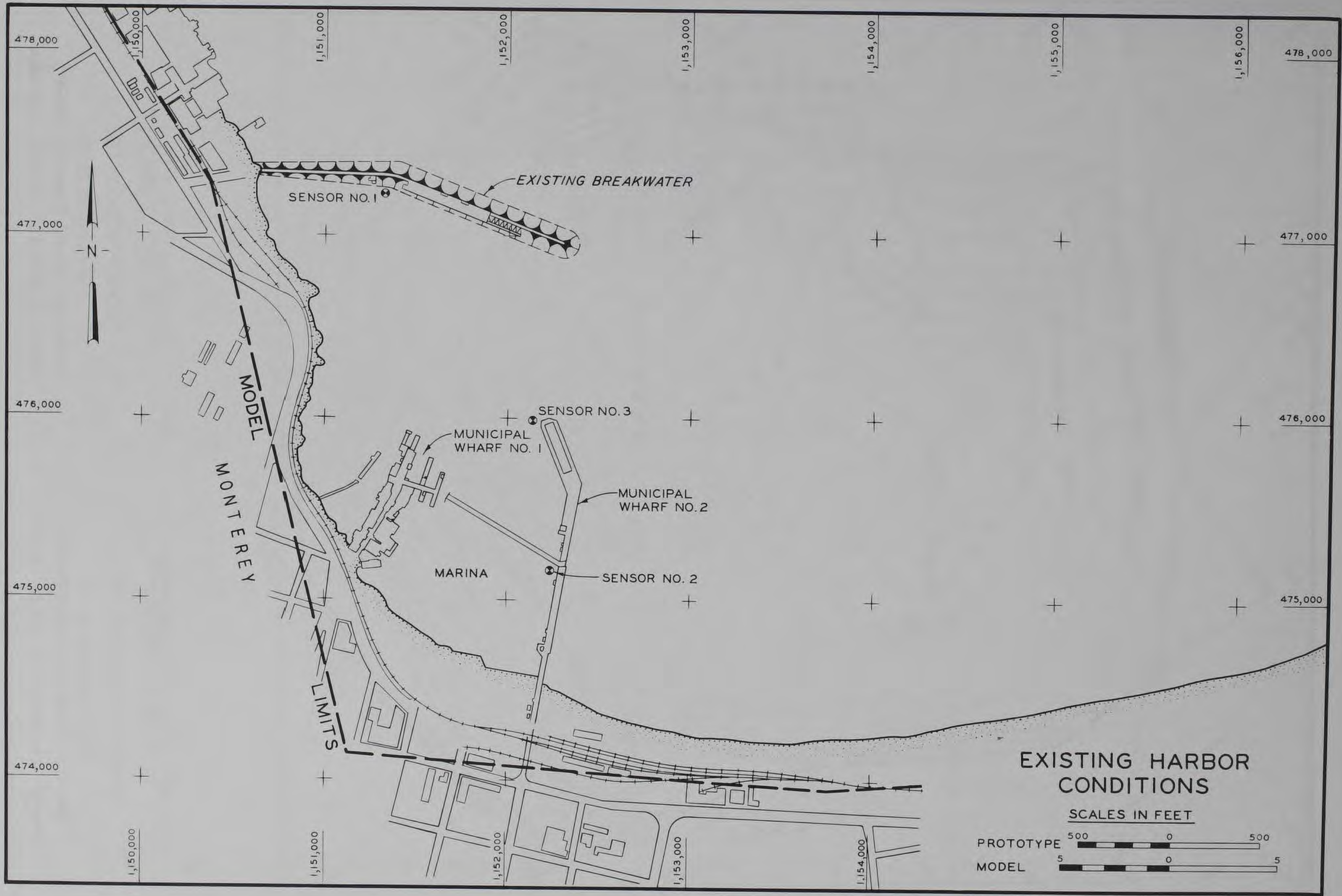
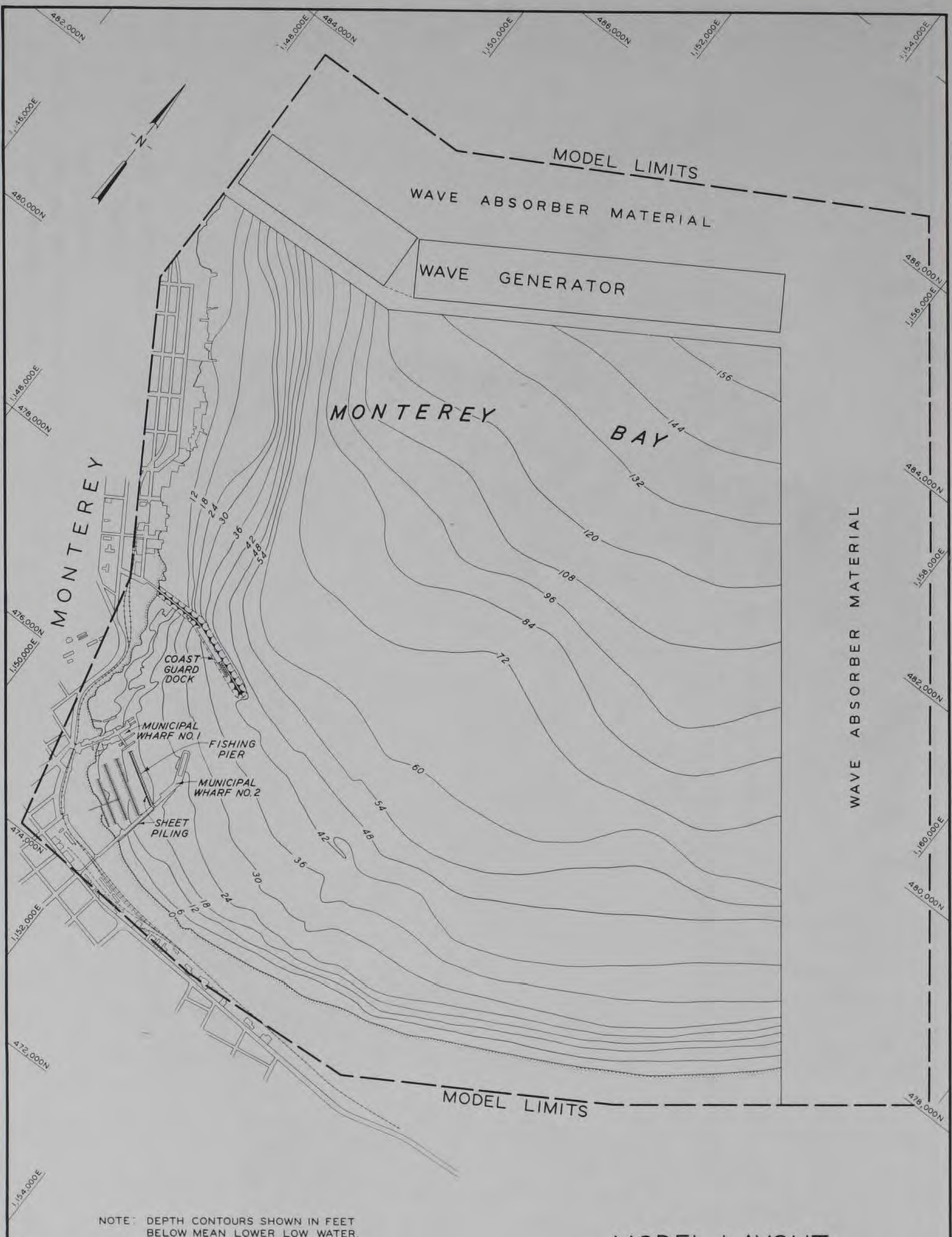

EXISTING BREAKWATER
SENSOR NO. 1
SENSOR NO. 3
SENSOR NO. 2
MUNICIPAL WHARF NO. 1
MUNICIPAL WHARF NO. 2
MARINA
MODEL LIMITS
MONTEREY
N
478,000
477,000
476,000
475,000
474,000
1,150,000
1,151,000
1,152,000
1,153,000
1,154,000
1,155,000
1,156,000
EXISTING HARBOR CONDITIONS
SCALES IN FEET
PROTOTYPE 500 0 500
MODEL 5 0 5

PLATE 1

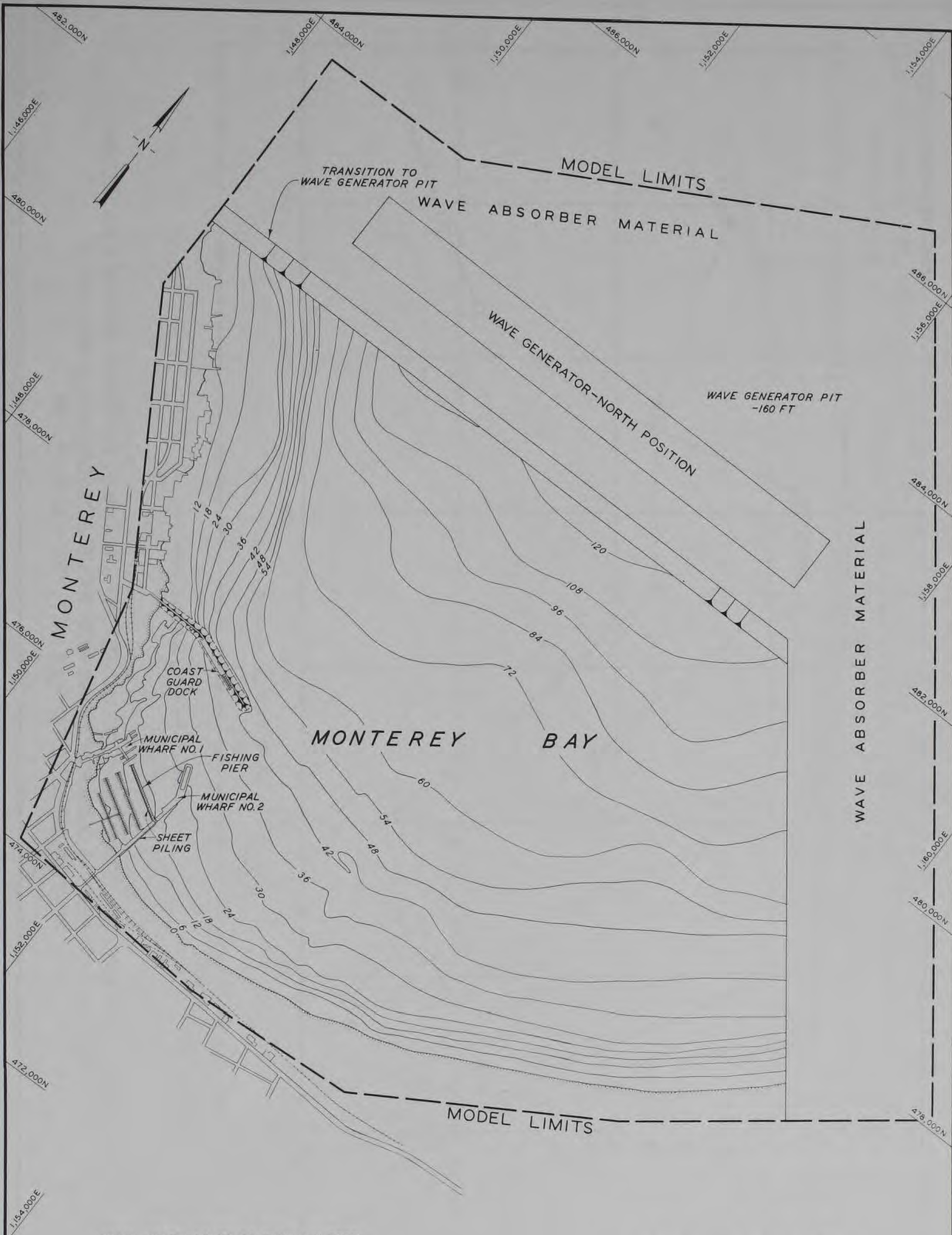


MODEL LAYOUT
LONG-PERIOD TESTS

SCALES IN FEET

PROTOTYPE 500 0 500 1000 1500 2000

MODEL 5 0 5 10 15 20

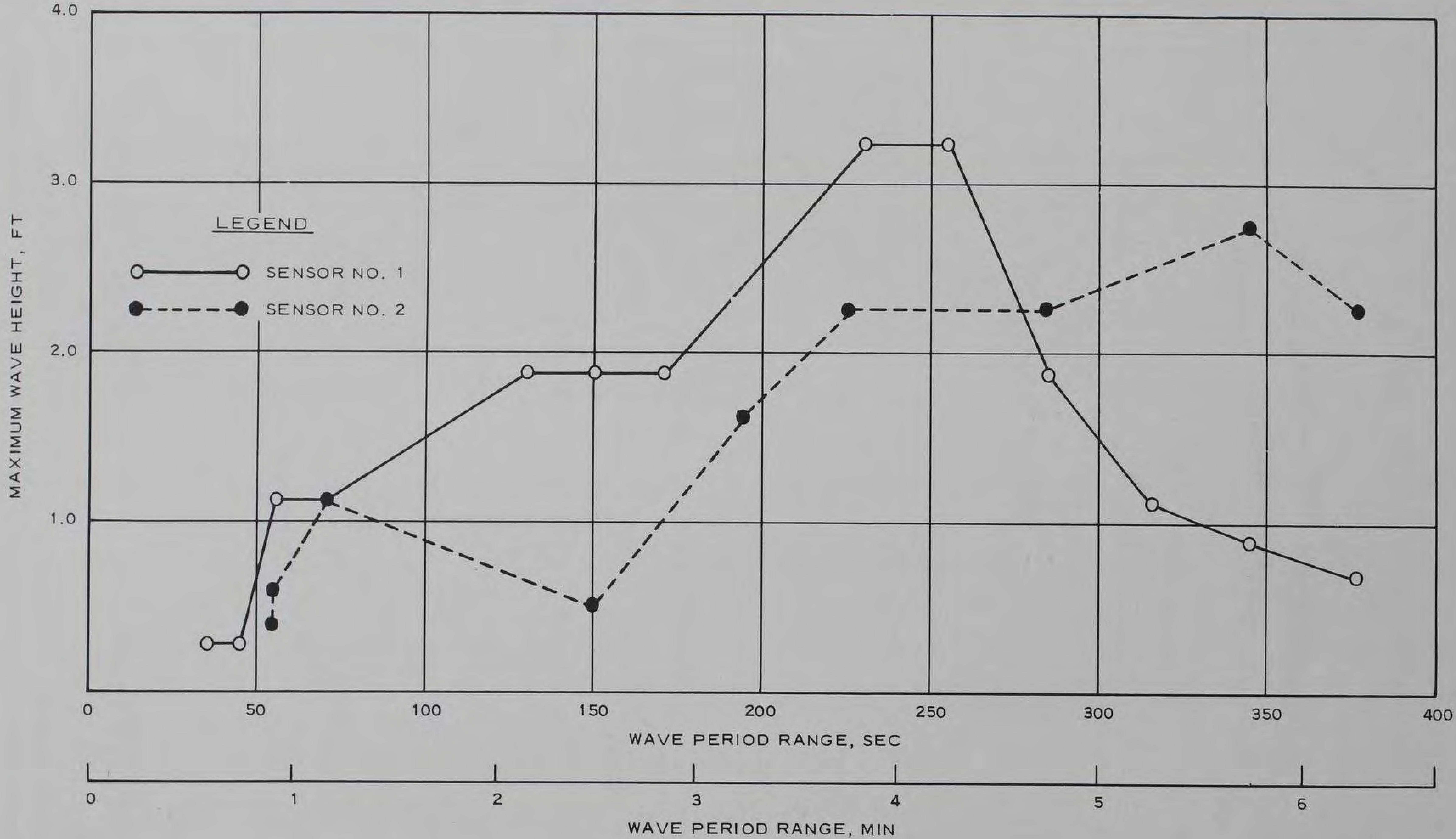


MODEL LAYOUT
SHORT-PERIOD TESTS

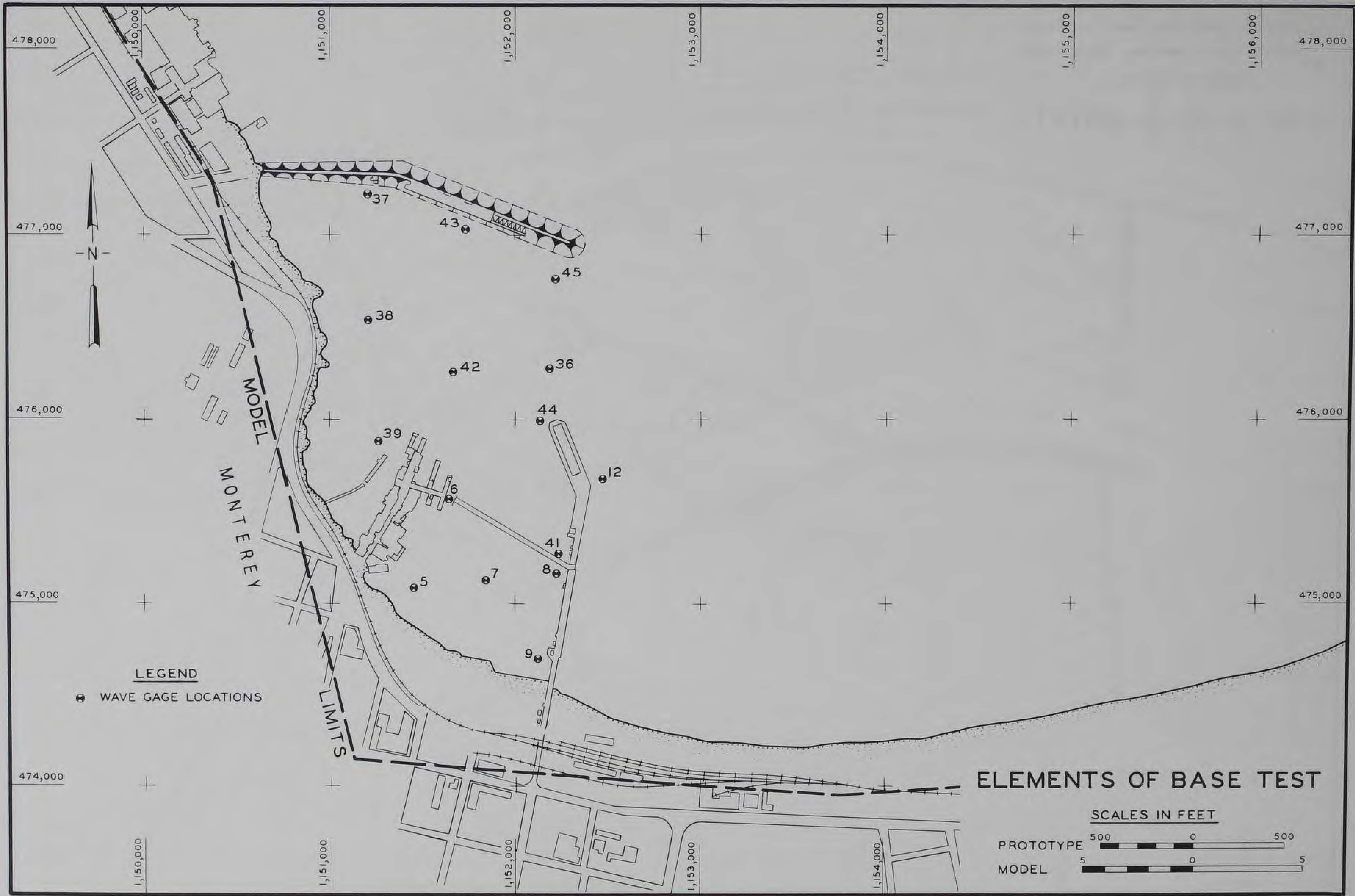
SCALES IN FEET

PROTOTYPE 500 0 500 1000 1500 2000

MODEL 5 0 5 10 15 20



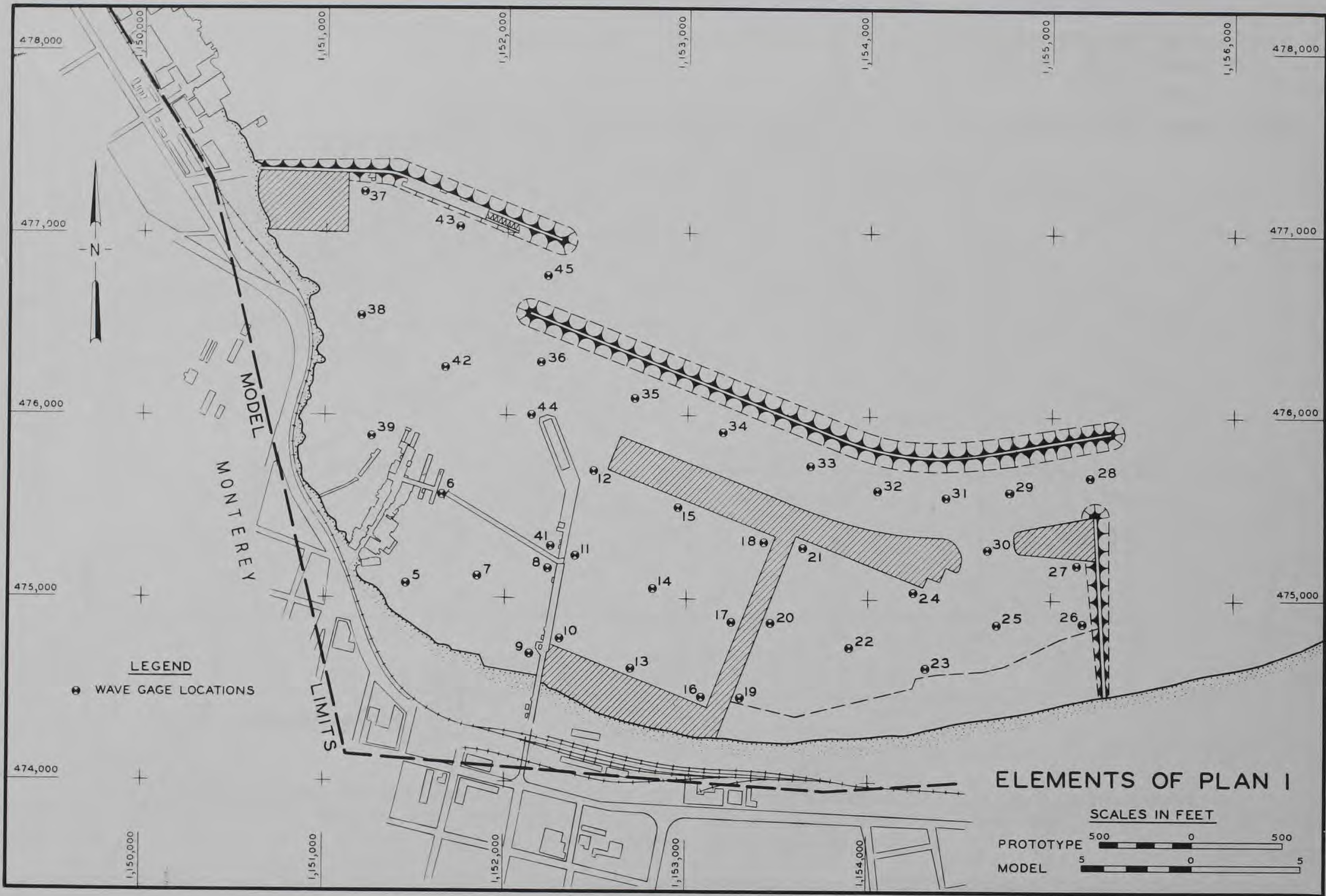
COMPARISON OF MAXIMUM SIGNIFICANT WAVE HEIGHTS AND WAVE-PERIOD RANGES BASED ON MARINE ADVISER'S OBSERVATIONS FOR WAVE SENSORS NO. 1 AND NO. 2
PERIOD OF RECORD = 23 OCT. '63 – 30 APR. '64



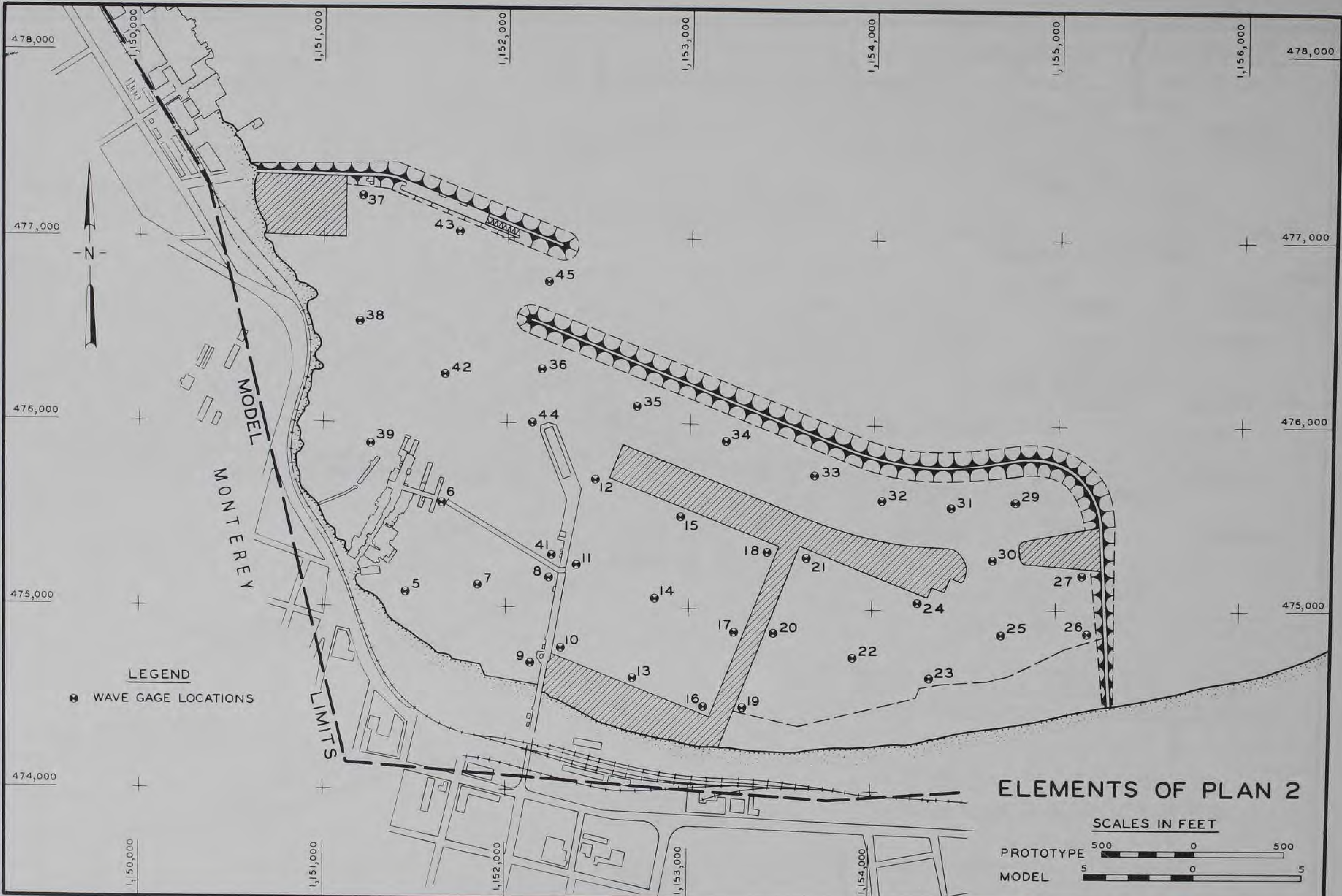
478,000
477,000
476,000
475,000
474,000
1,150,000
1,151,000
1,152,000
1,153,000
1,154,000
1,155,000
1,156,000
N
37
43
45
38
42
36
44
39
12
6
41
8
7
5
9
MODEL
MONTEREY
LIMITS
LEGEND
WAVE GAGE LOCATIONS
ELEMENTS OF BASE TEST
SCALES IN FEET
PROTOTYPE
500
0
500
MODEL
5
0
5

PLATE 5

PLATE 6



ELEMENTS OF PLAN I


ELEMENTS OF PLAN 2
SCALES IN FEET
PROTOTYPE 500 0 500
MODEL 5 0 5
LEGEND
WAVE GAGE LOCATIONS
MODEL LIMITS
MONTEREY
N
478,000
477,000
476,000
475,000
474,000
1,150,000
1,151,000
1,152,000
1,153,000
1,154,000
1,155,000
1,156,000
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
26
27
29
30
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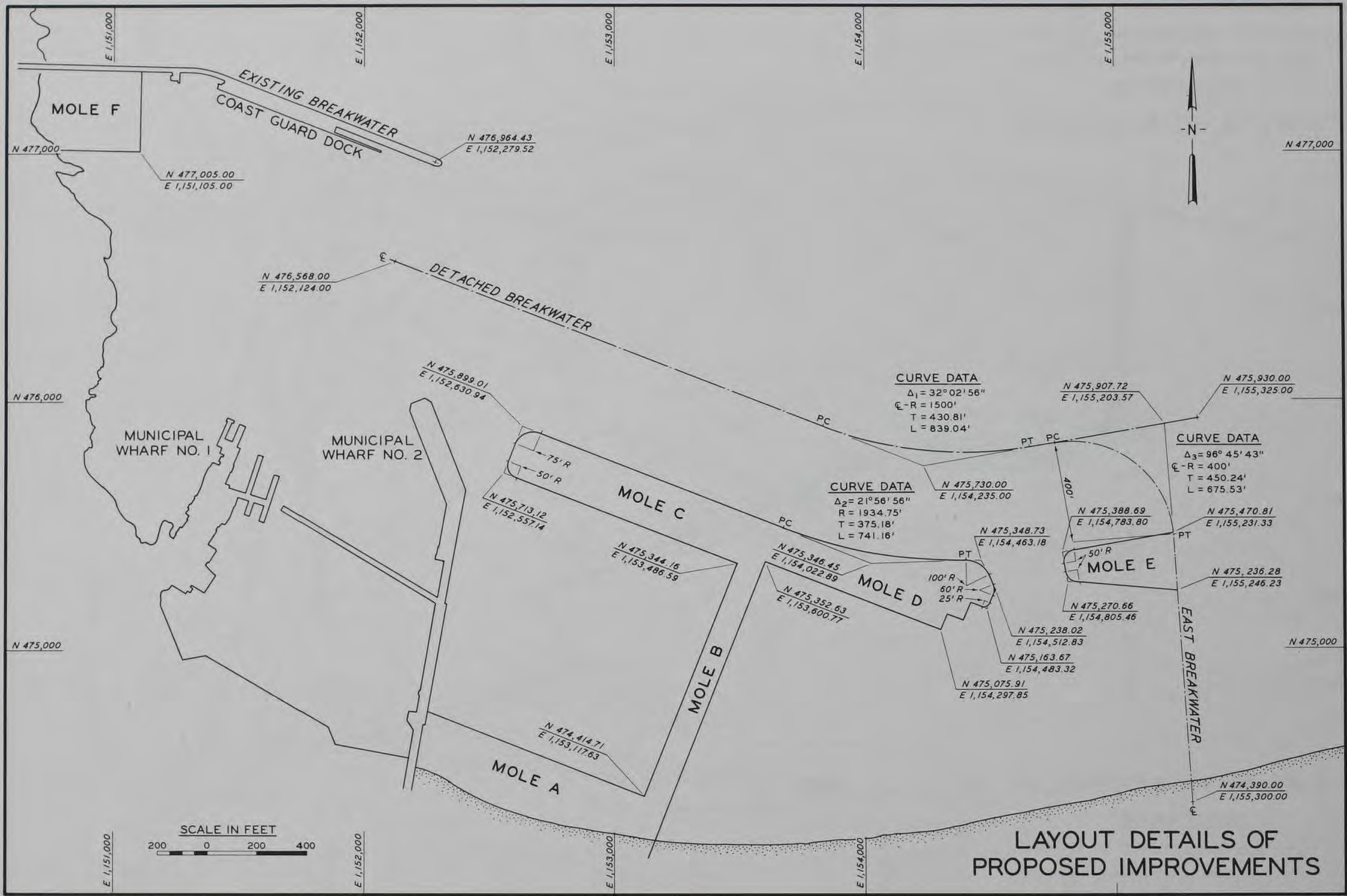
32
33
34
35
36
37
38
39
41
42
43
44
45

PLATE 7

PLATE 8



LAYOUT DETAILS OF PROPOSED IMPROVEMENTS

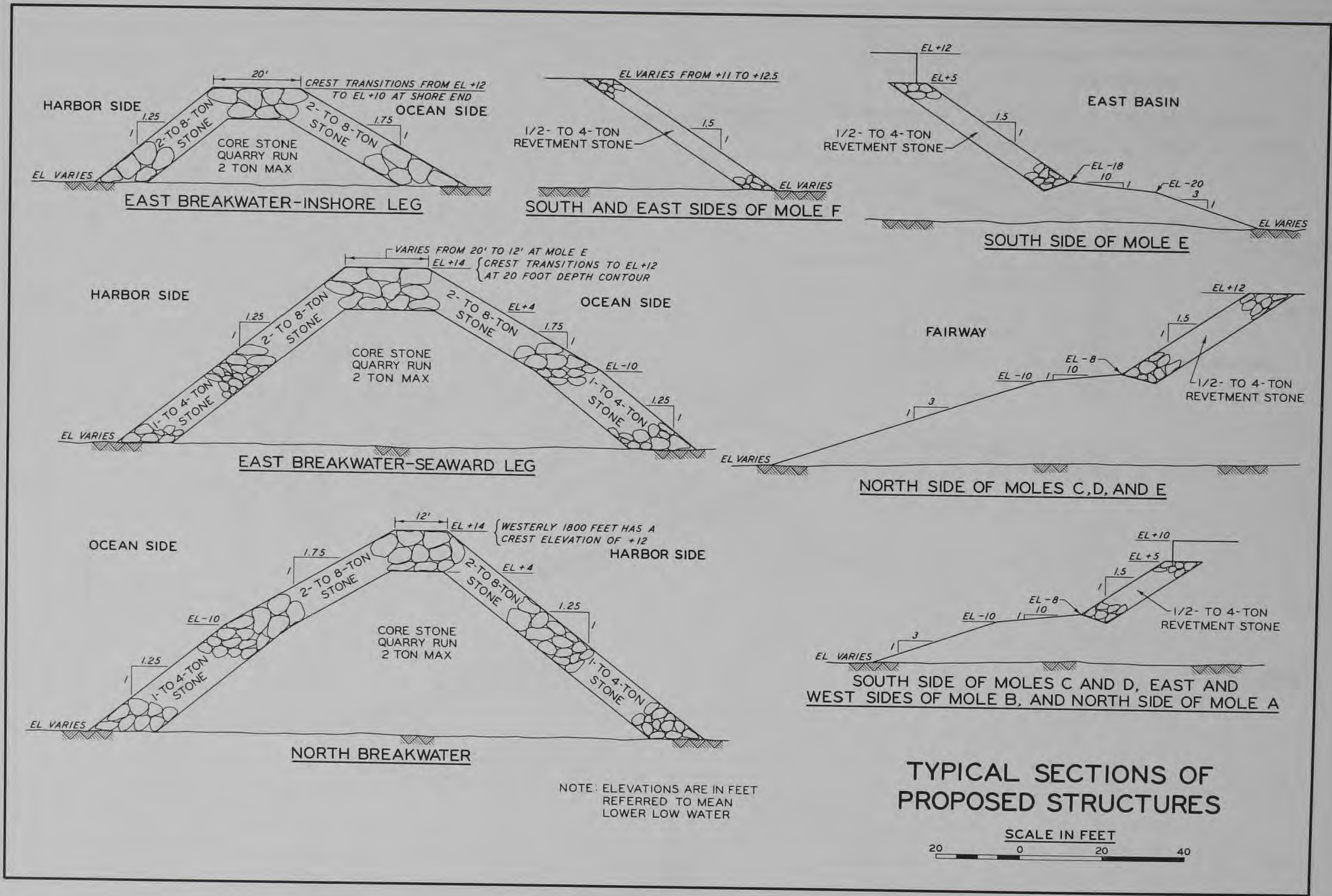
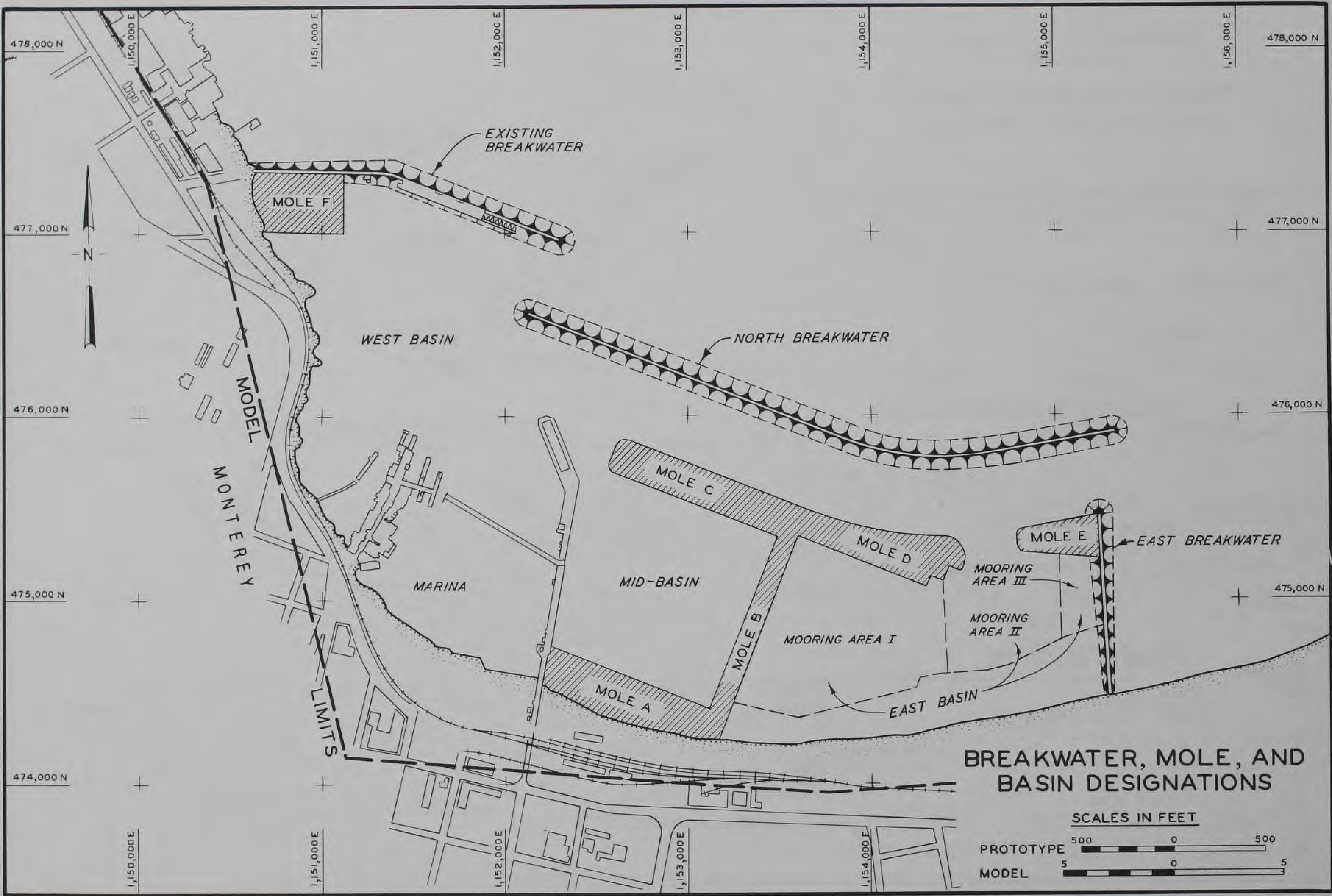

HARBOR SIDE
20'
CREST TRANSITIONS FROM EL +12 TO EL +10 AT SHORE END
OCEAN SIDE
1.25
1
2-TO 8-TON STONE
2- TO 8-TON STONE
1.75
1
CORE STONE QUARRY RUN 2 TON MAX
EL VARIES
EAST BREAKWATER-INSHORE LEG
EL VARIES FROM +11 TO +12.5
1/2- TO 4-TON REVETMENT STONE
1.5
1
EL VARIES
SOUTH AND EAST SIDES OF MOLE F
EL +12
EL +5
EAST BASIN
1.5
1
1/2- TO 4-TON REVETMENT STONE
EL -18
10
1
EL -20
3
1
EL VARIES
SOUTH SIDE OF MOLE E
VARIES FROM 20' TO 12' AT MOLE E
EL +14
CREST TRANSITIONS TO EL +12 AT 20 FOOT DEPTH CONTOUR
HARBOR SIDE
OCEAN SIDE
1.25
1
2- TO 8-TON STONE
2- TO 8-TON STONE
EL +4
1.75
1
CORE STONE QUARRY RUN 2 TON MAX
EL -10
1- TO 4-TON STONE
1- TO 4-TON STONE
1.25
1
EL VARIES
EAST BREAKWATER-SEAWARD LEG
EL +12
FAIRWAY
1.5
1
EL -8
10
EL -10
1
1/2- TO 4-TON REVETMENT STONE
3
1
EL VARIES
NORTH SIDE OF MOLES C, D, AND E
12'
EL +14
WESTERLY 1800 FEET HAS A CREST ELEVATION OF +12
OCEAN SIDE
HARBOR SIDE
1.75
1
2- TO 8-TON STONE
2-TO 8-TON STONE
EL +4
1.25
1
EL -10
CORE STONE QUARRY RUN 2 TON MAX
1.25
1
1-TO 4-TON STONE
1-TO 4-TON STONE
EL VARIES
NORTH BREAKWATER
EL +10
EL +5
1.5
1
EL -8
10
EL -10
1
1/2- TO 4-TON REVETMENT STONE
3
1
EL VARIES
SOUTH SIDE OF MOLES C AND D, EAST AND WEST SIDES OF MOLE B, AND NORTH SIDE OF MOLE A
NOTE: ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER
TYPICAL SECTIONS OF PROPOSED STRUCTURES
SCALE IN FEET
20
0
20
40

PLATE 9

PLATE 10



BREAKWATER, MOLE, AND BASIN DESIGNATIONS

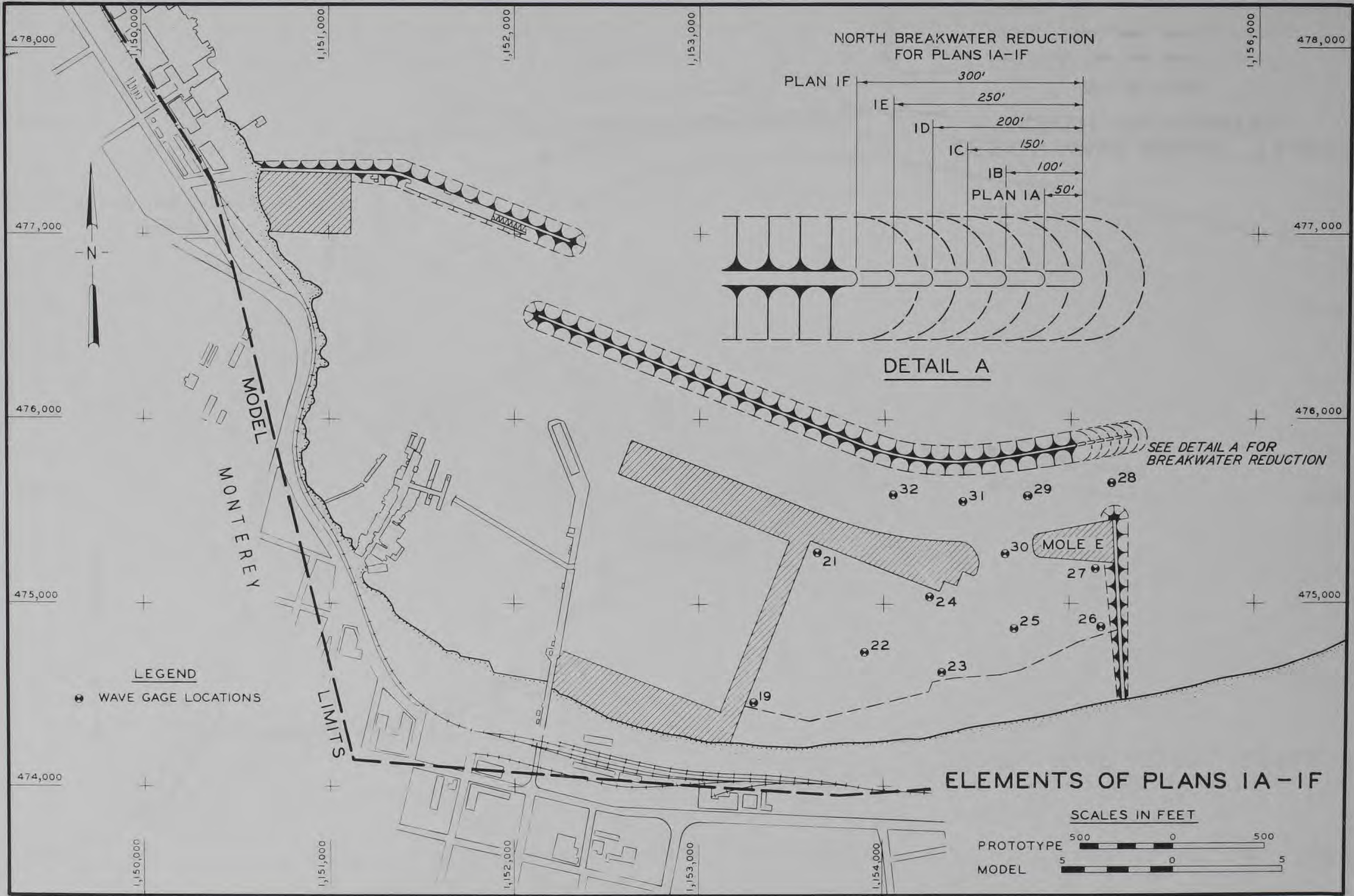
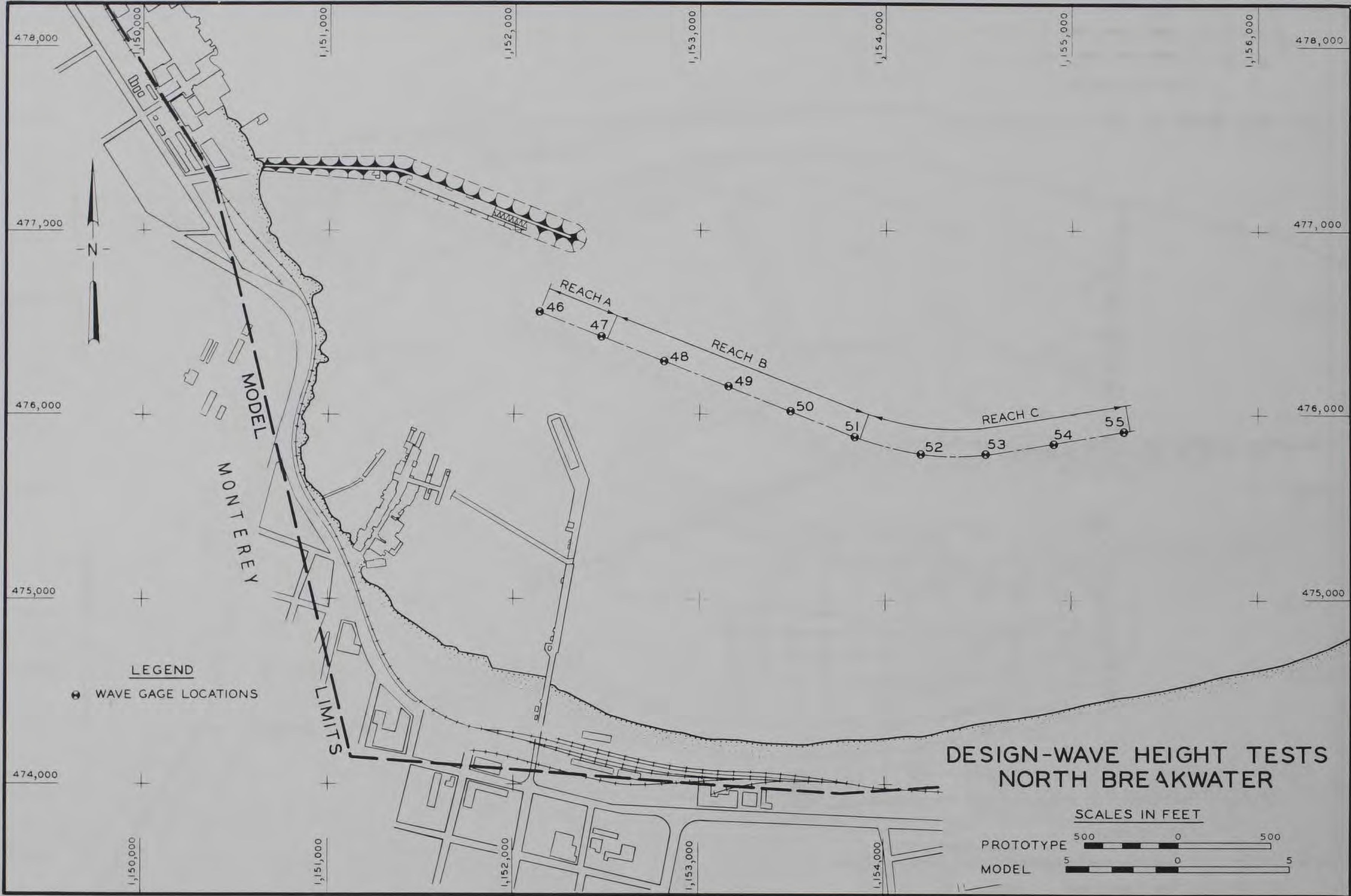
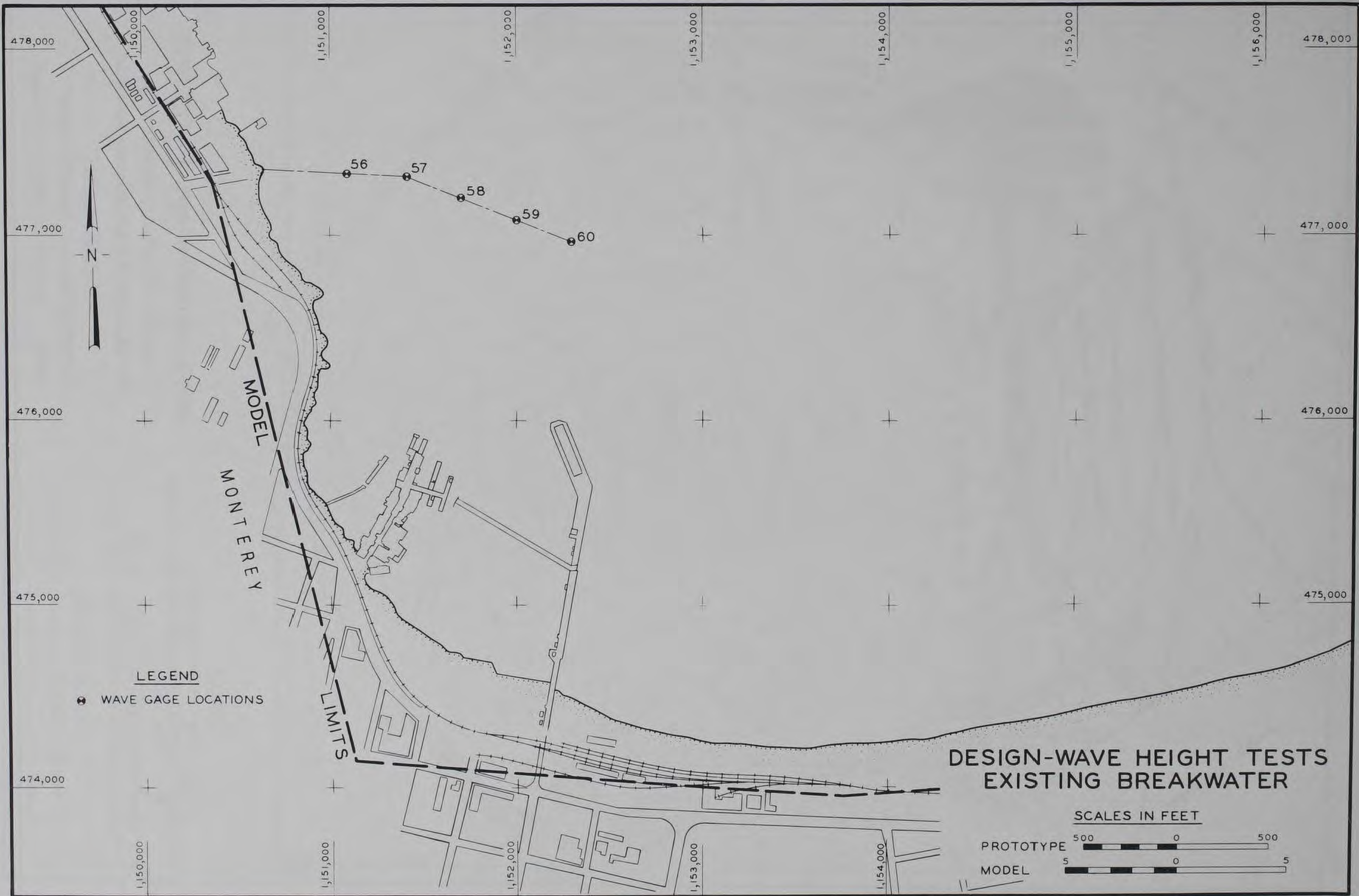
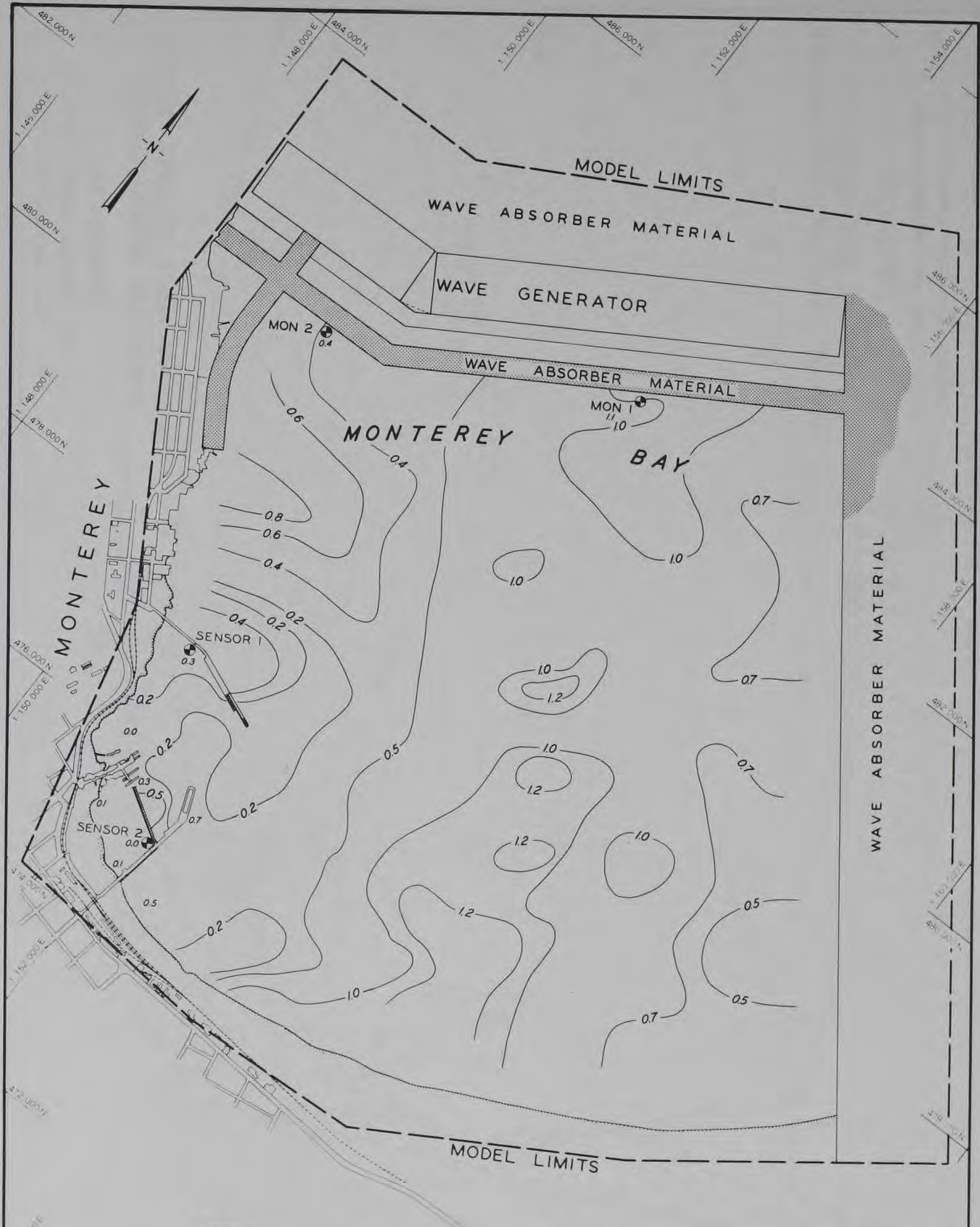

NORTH BREAKWATER REDUCTION
FOR PLANS 1A-1F
PLAN 1F
300'
1E
250'
1D
200'
1C
150'
1B
100'
PLAN 1A
50'
DETAIL A
SEE DETAIL A FOR
BREAKWATER REDUCTION
MOLE E
MODEL LIMITS
MONTEREY
LEGEND
WAVE GAGE LOCATIONS
ELEMENTS OF PLANS 1A-1F
SCALES IN FEET
PROTOTYPE
500
0
500
MODEL
5
0
5
478,000
477,000
476,000
475,000
474,000
1,150,000
1,151,000
1,152,000
1,153,000
1,154,000
1,156,000
19
21
22
23
24
25
26
27
28
29
30
31
32

PLATE 12


DESIGN-WAVE HEIGHT TESTS
NORTH BREAKWATER
SCALES IN FEET
PROTOTYPE 500 0 500
MODEL 5 0 5
LEGEND
WAVE GAGE LOCATIONS
MODEL LIMITS
MONTEREY
REACH A
REACH B
REACH C
46
47
48
49
50
51
52
53
54
55
N
478,000
477,000
476,000
475,000
474,000
1,150,000
1,151,000
1,152,000
1,153,000
1,154,000
1,155,000
1,156,000



DESIGN-WAVE HEIGHT TESTS
EXISTING BREAKWATER



WAVE HEIGHTS AND PATTERNS

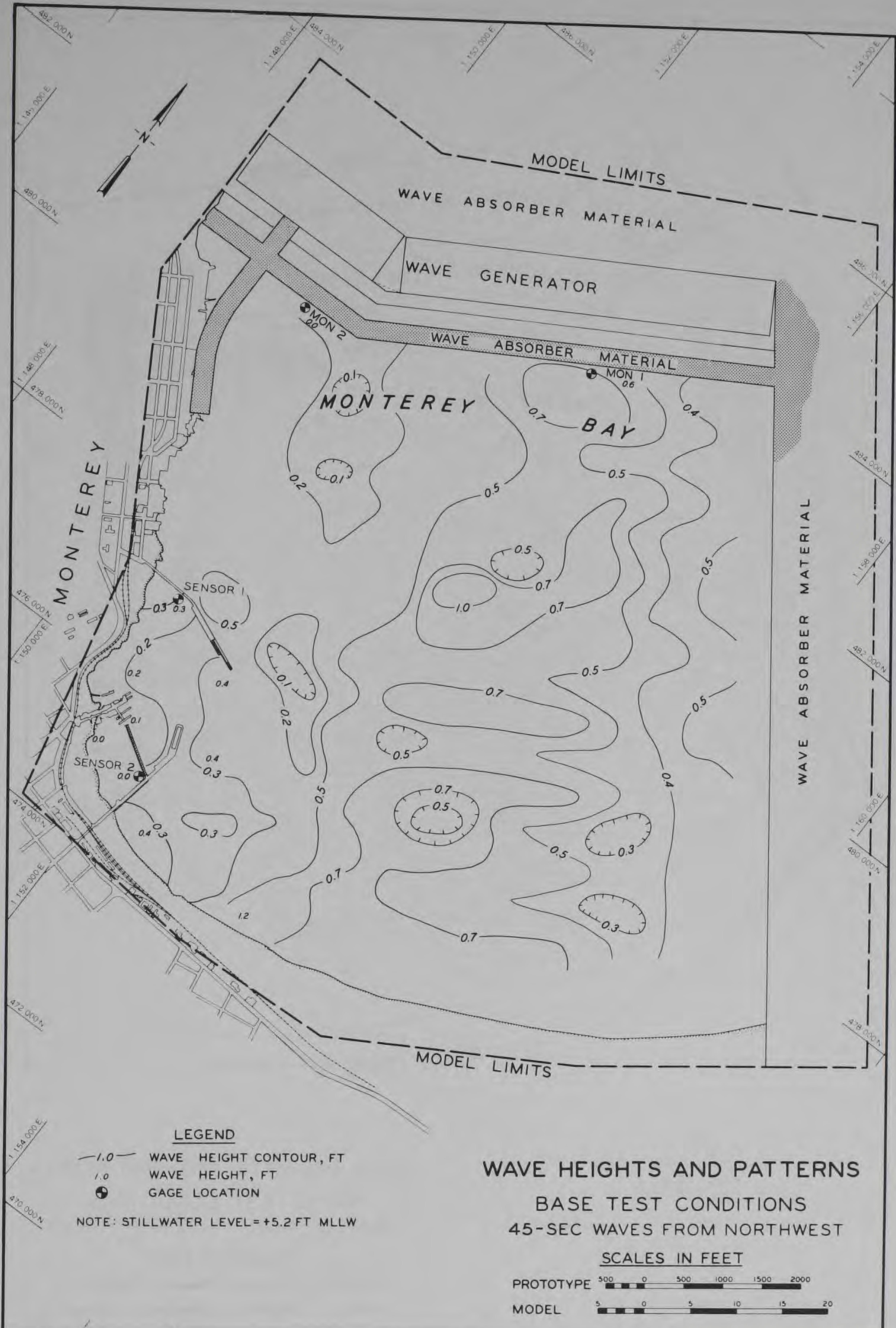
BASE TEST CONDITIONS

35-SEC WAVES FROM NORTHWEST

SCALES IN FEET

PROTOTYPE 500 0 500 1000 1500 2000

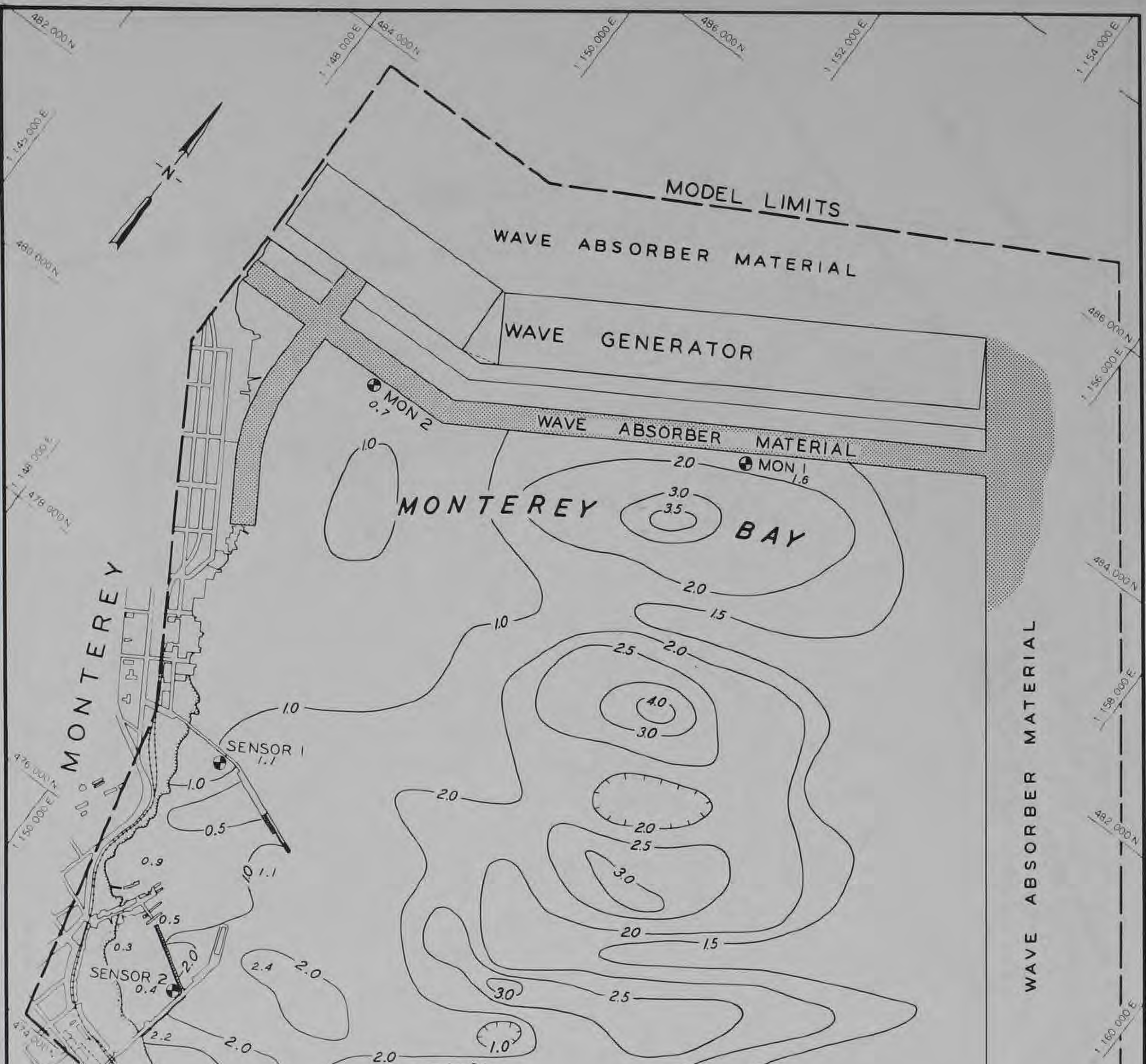
MODEL 5 0 5 10 15 20



WAVE HEIGHTS AND PATTERNS

BASE TEST CONDITIONS

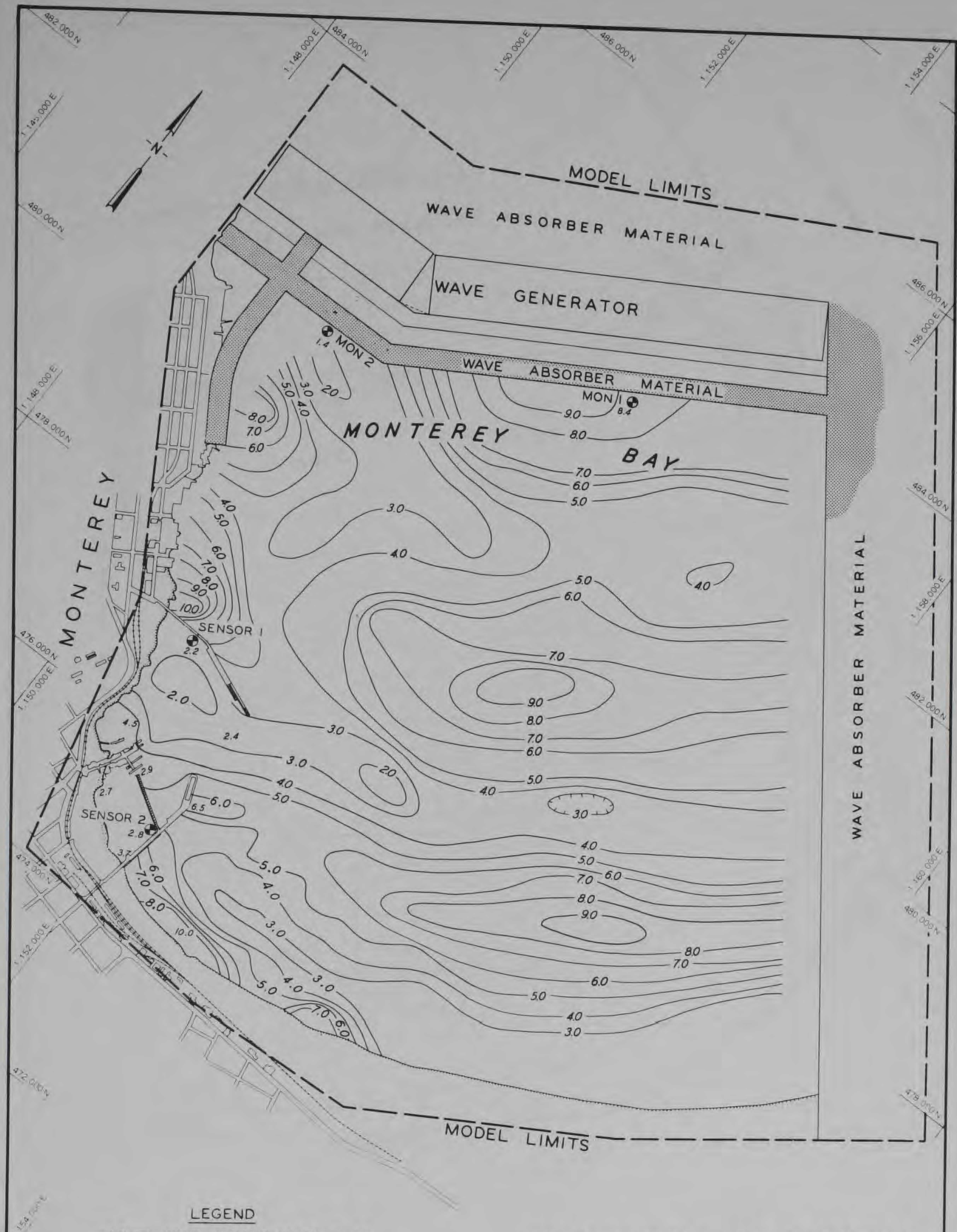
45-SEC WAVES FROM NORTHWEST



WAVE HEIGHTS AND PATTERNS

BASE TEST CONDITIONS

55-SEC WAVES FROM NORTHWEST



LEGEND

—1.0— WAVE HEIGHT CONTOUR, FT

1.0 WAVE HEIGHT, FT

GAGE LOCATION

NOTE: STILLWATER LEVEL = +5.2 FT MLLW

WAVE HEIGHTS AND PATTERNS

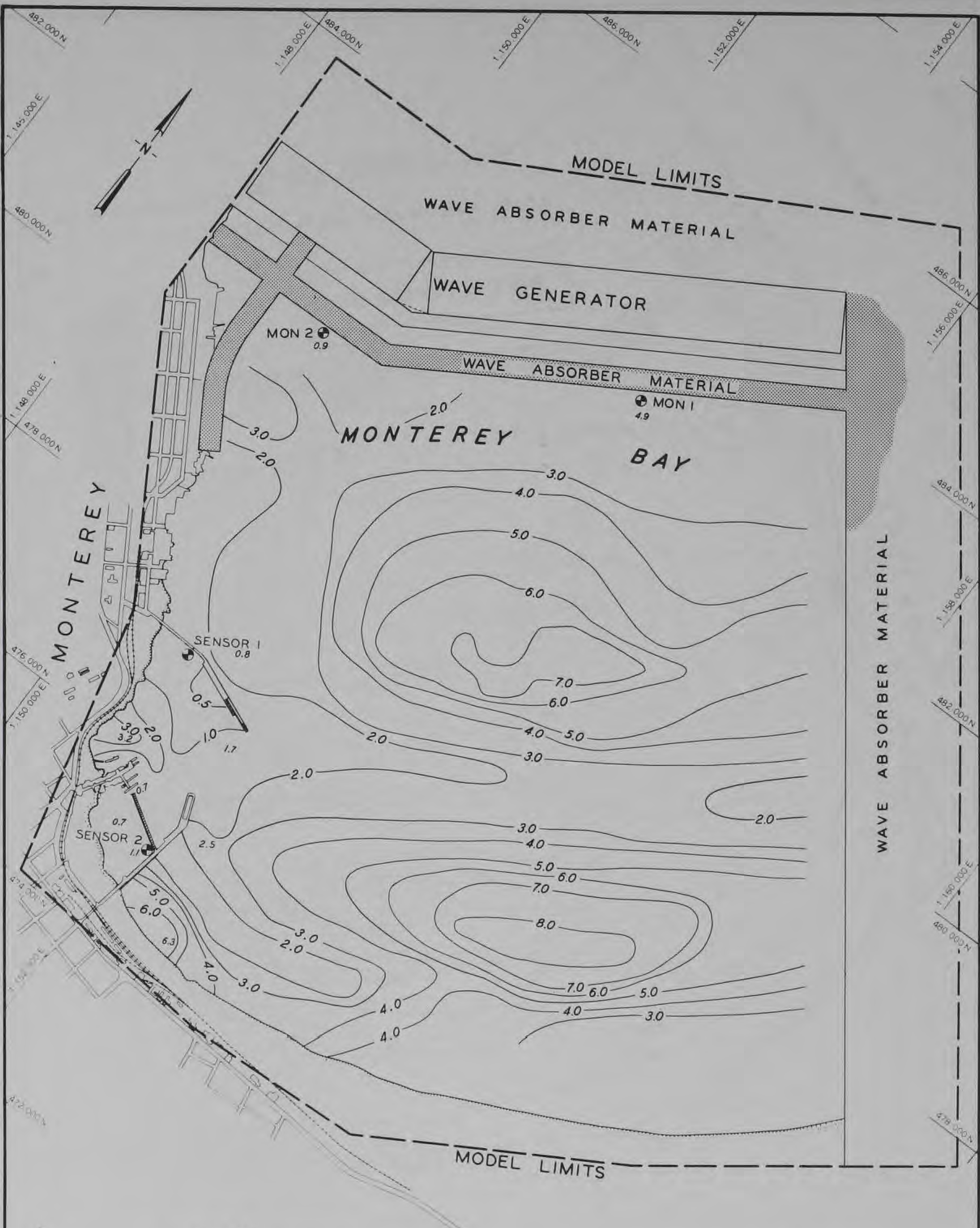
BASE TEST CONDITIONS

120-SEC WAVES FROM NORTHWEST

SCALES IN FEET

PROTOTYPE 500 0 500 1000 1500 2000

MODEL 5 0 5 10 15 20

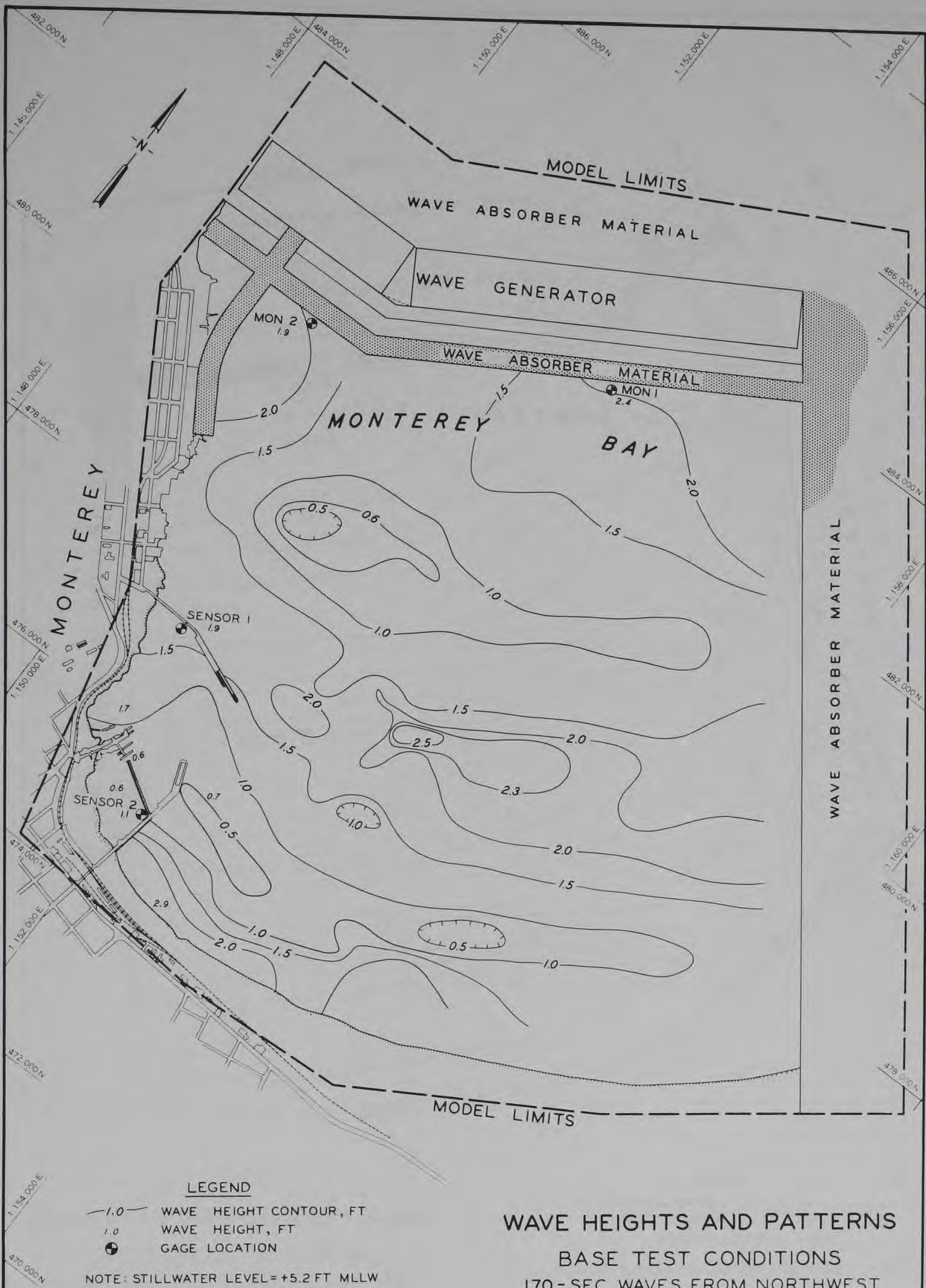


WAVE HEIGHTS AND PATTERNS

BASE TEST CONDITIONS

130-SEC WAVES FROM NORTHWEST

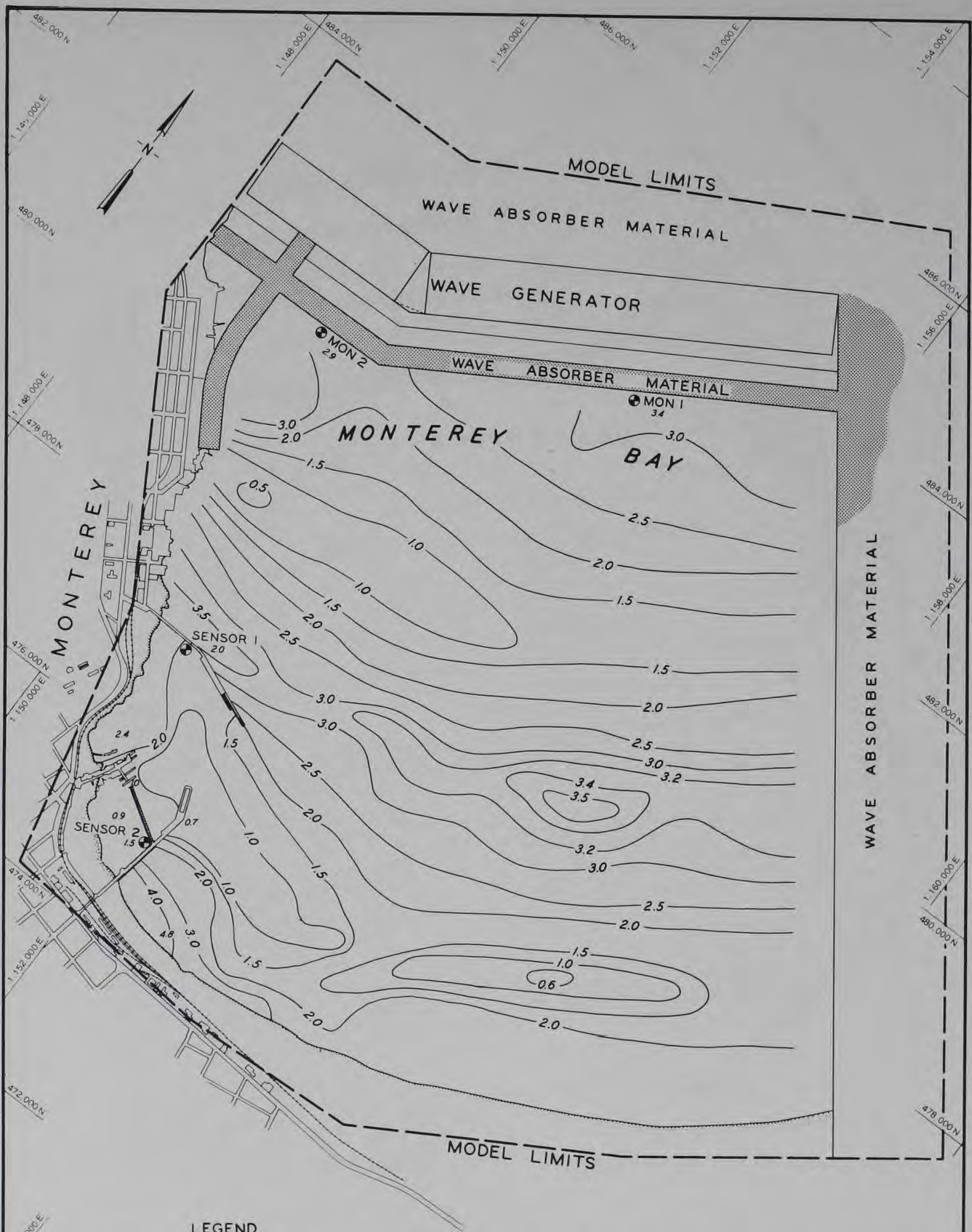
PLATE 18



WAVE HEIGHTS AND PATTERNS

BASE TEST CONDITIONS

170-SEC WAVES FROM NORTHWEST



LEGEND

—1.0— WAVE HEIGHT CONTOUR, FT

1.0 WAVE HEIGHT, FT

⊕ GAGE LOCATION

NOTE: STILLWATER LEVEL = +5.2 FT MLLW

WAVE HEIGHTS AND PATTERNS

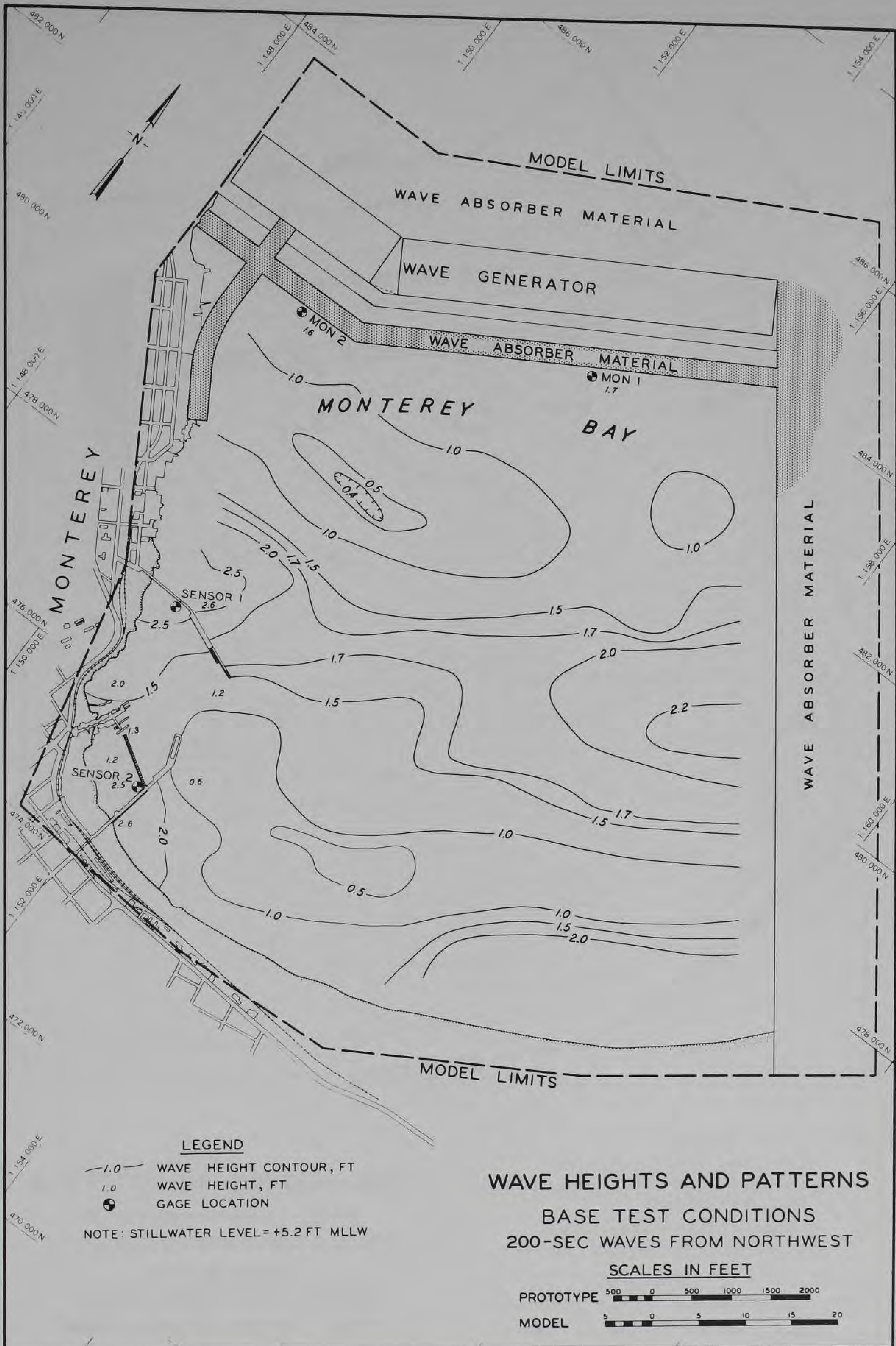
BASE TEST CONDITIONS

180-SEC WAVES FROM NORTHWEST

SCALES IN FEET

PROTOTYPE 500 0 500 1000 1500 2000

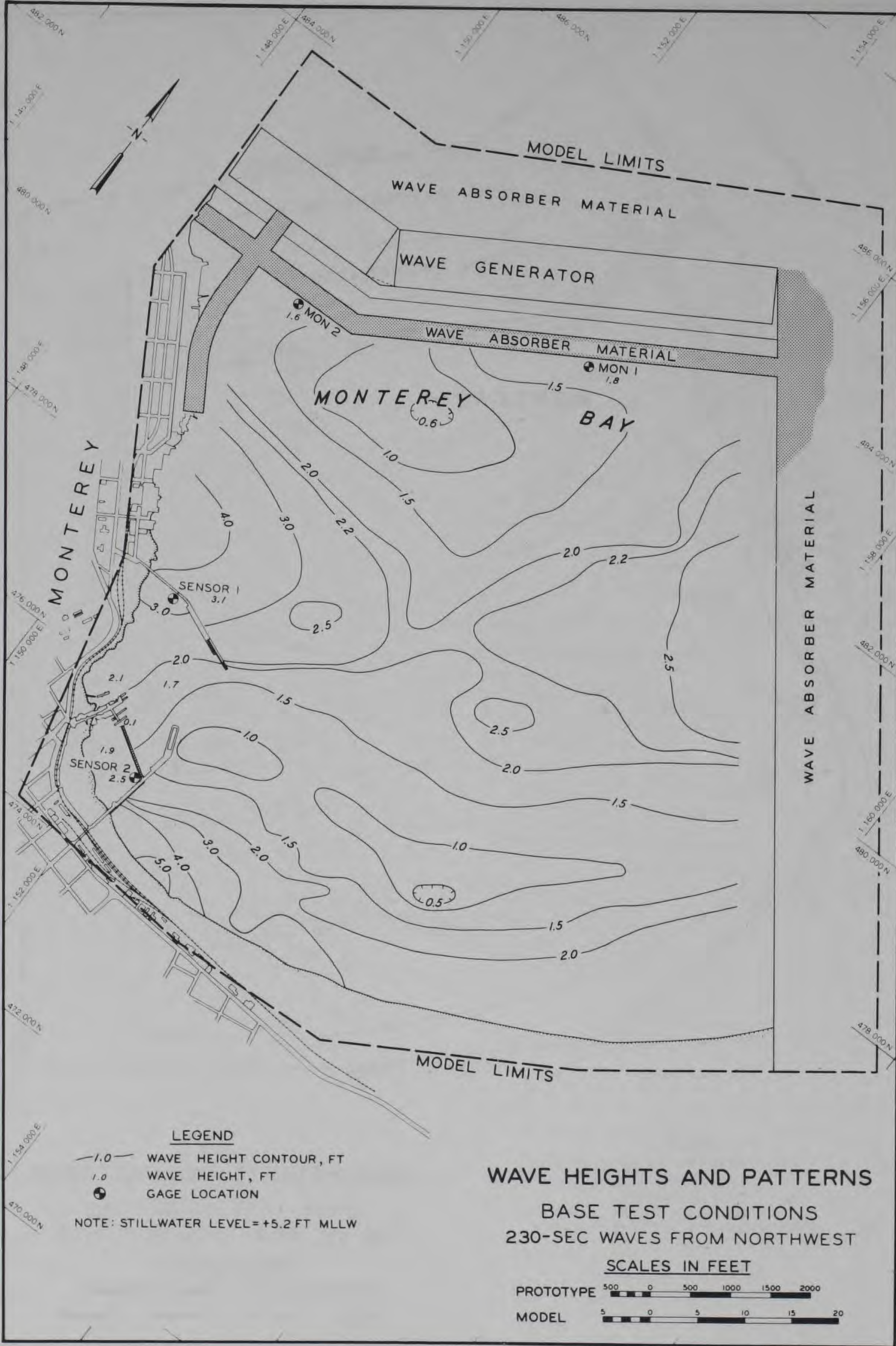
MODEL 5 0 5 10 15 20



WAVE HEIGHTS AND PATTERNS

BASE TEST CONDITIONS

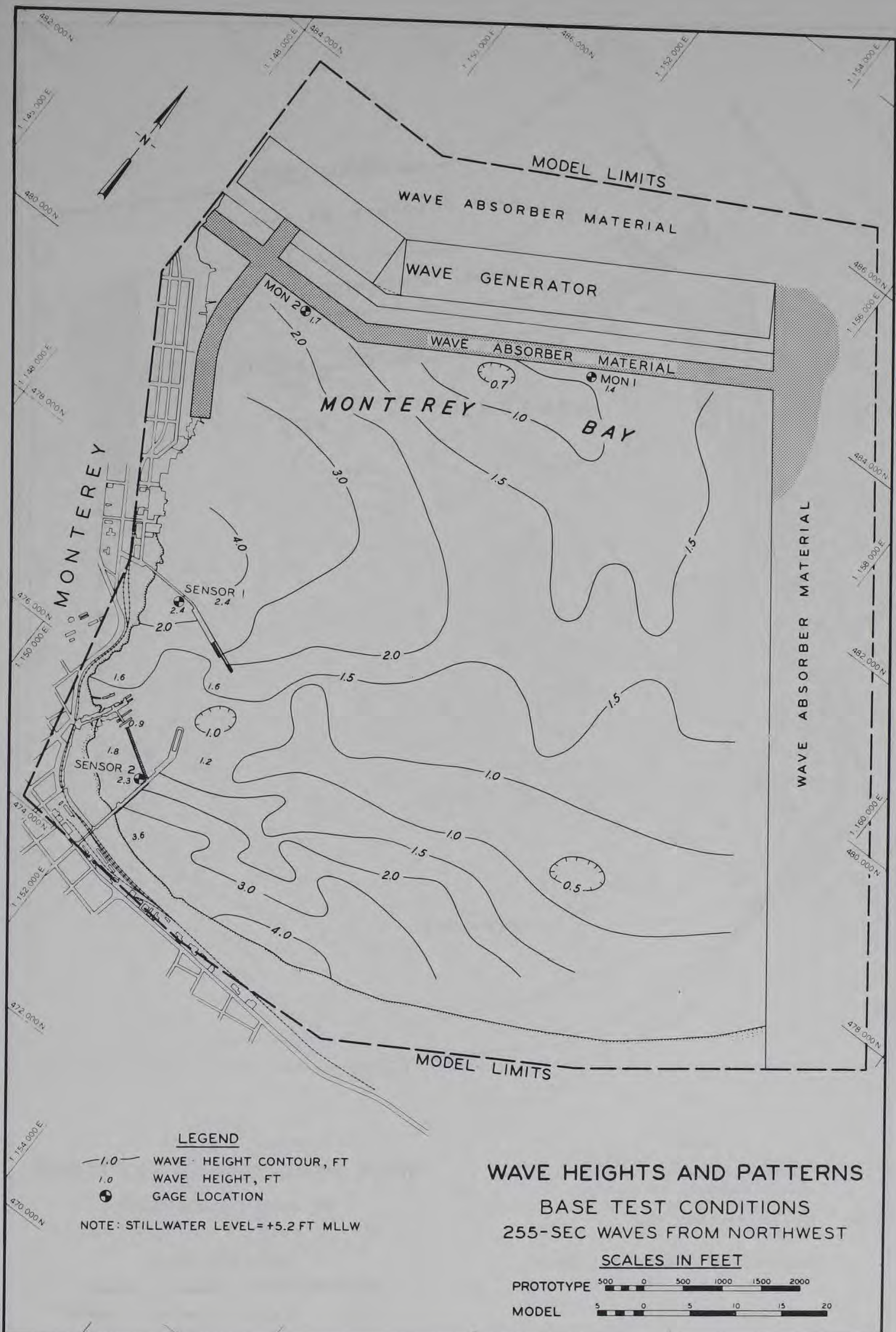
200-SEC WAVES FROM NORTHWEST



WAVE HEIGHTS AND PATTERNS

BASE TEST CONDITIONS

230-SEC WAVES FROM NORTHWEST



WAVE HEIGHTS AND PATTERNS
BASE TEST CONDITIONS
255-SEC WAVES FROM NORTHWEST

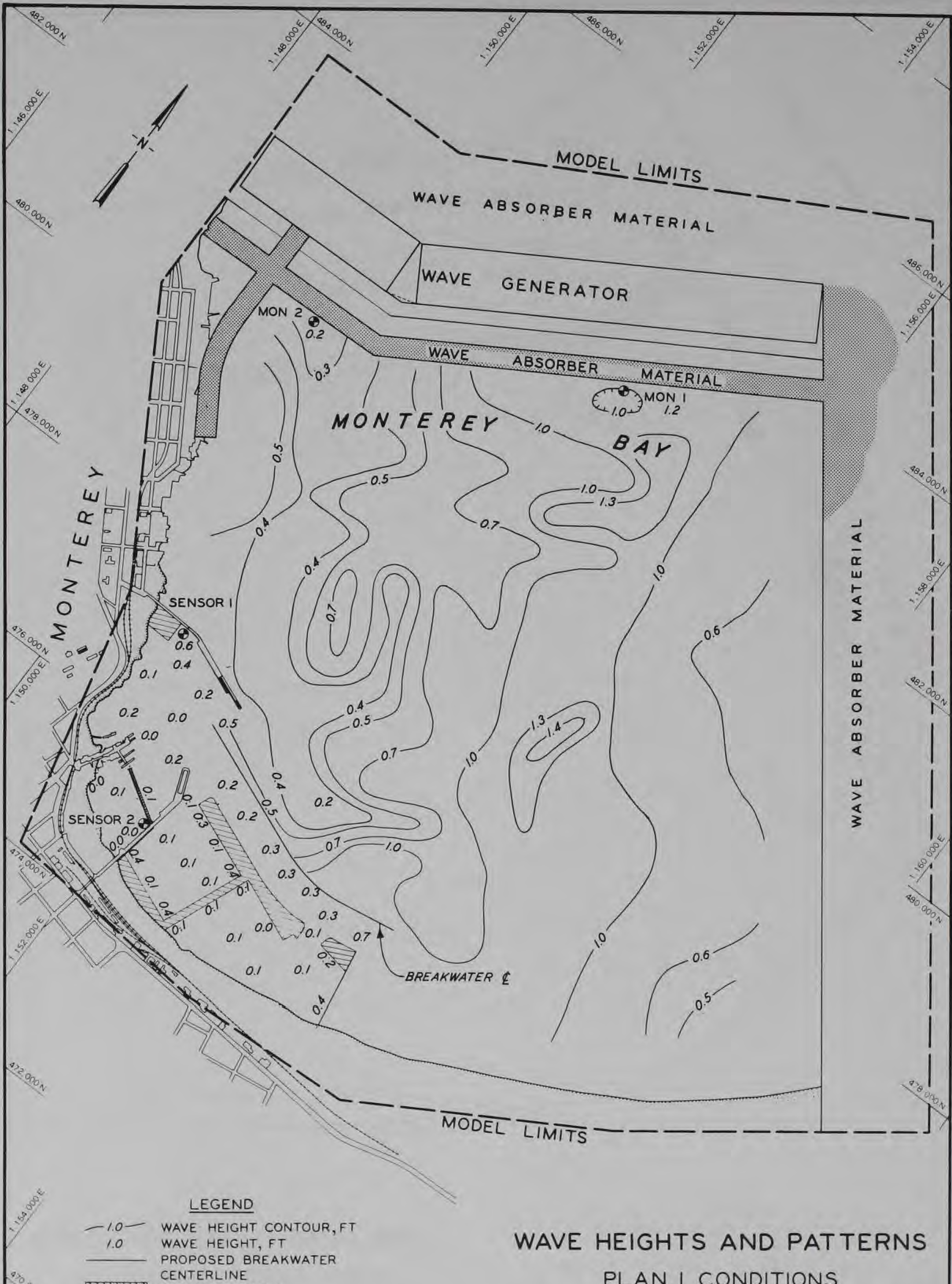
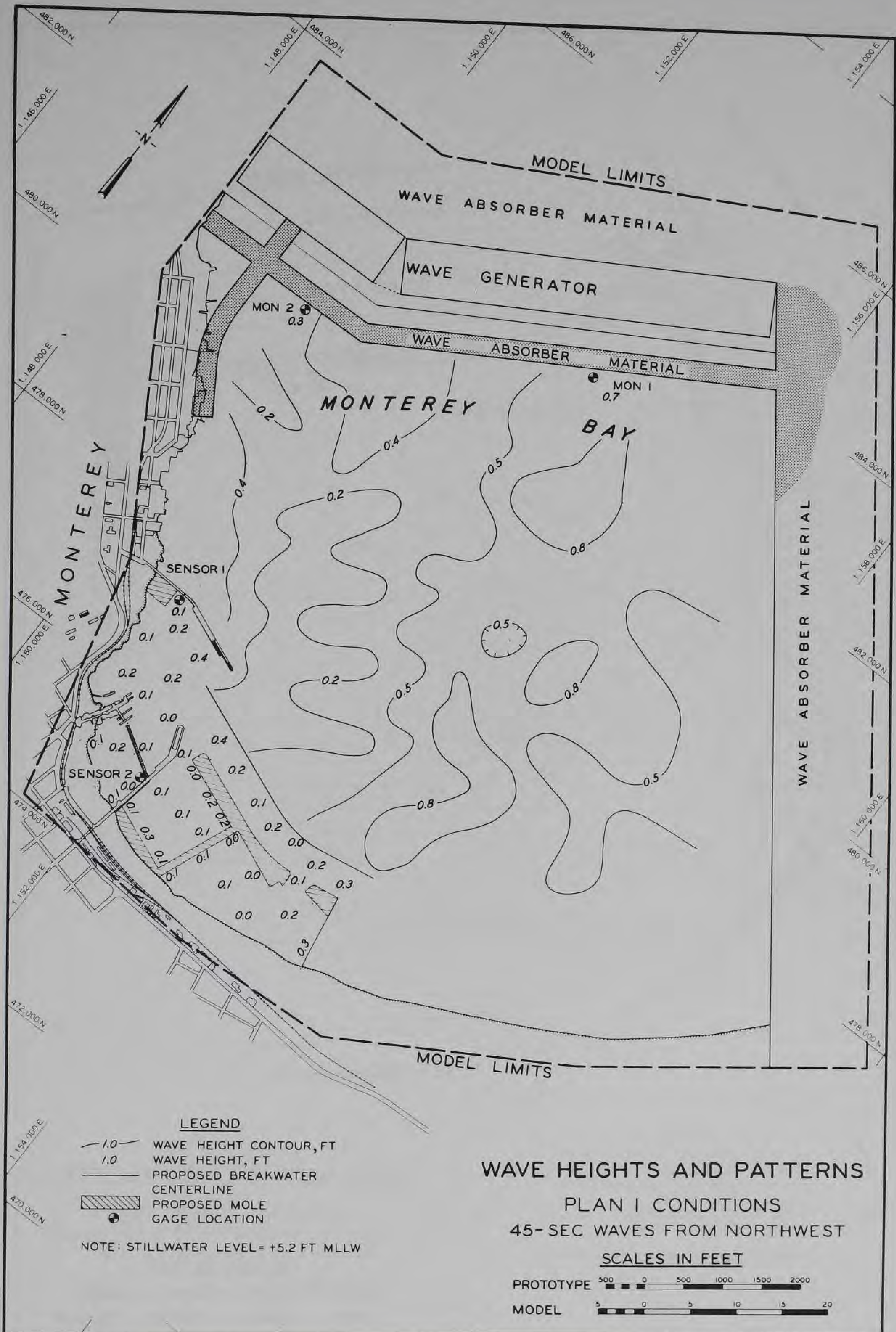

MODEL LIMITS
WAVE ABSORBER MATERIAL
WAVE GENERATOR
WAVE ABSORBER MATERIAL
MON 2
MON 1
MONTEREY BAY
MONTEREY
SENSOR 1
SENSOR 2
BREAKWATER ℄
WAVE ABSORBER MATERIAL
MODEL LIMITS
LEGEND
1.0 WAVE HEIGHT CONTOUR, FT
1.0 WAVE HEIGHT, FT
PROPOSED BREAKWATER CENTERLINE
PROPOSED MOLE
GAGE LOCATION
NOTE: STILLWATER LEVEL = +5.2 FT MLLW
WAVE HEIGHTS AND PATTERNS
PLAN 1 CONDITIONS
35-SEC WAVES FROM NORTHWEST
SCALES IN FEET
PROTOTYPE 500 0 500 1000 1500 2000
MODEL 5 0 5 10 15 20

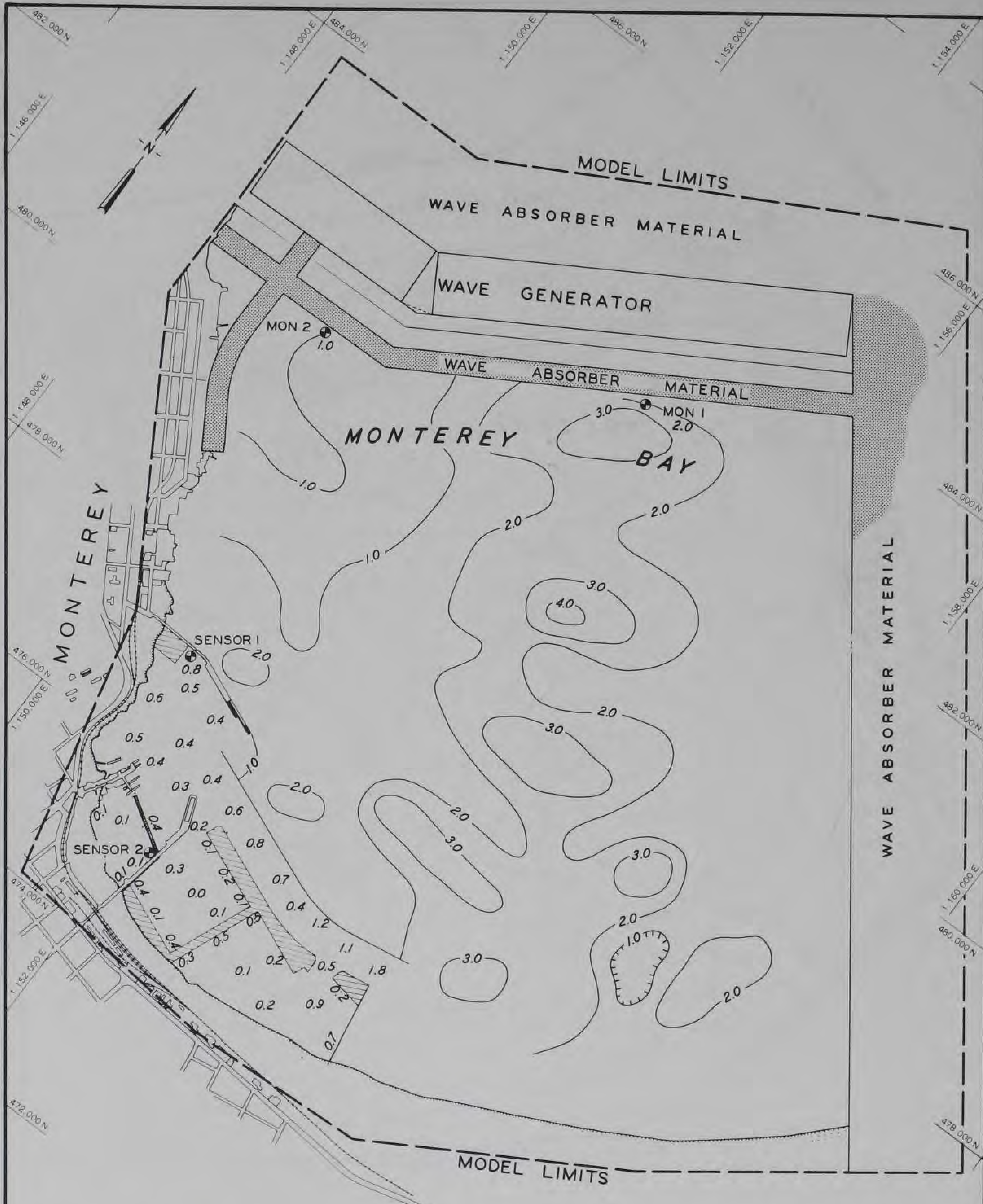
PLATE 24



WAVE HEIGHTS AND PATTERNS

PLAN 1 CONDITIONS

45-SEC WAVES FROM NORTHWEST



WAVE HEIGHTS AND PATTERNS

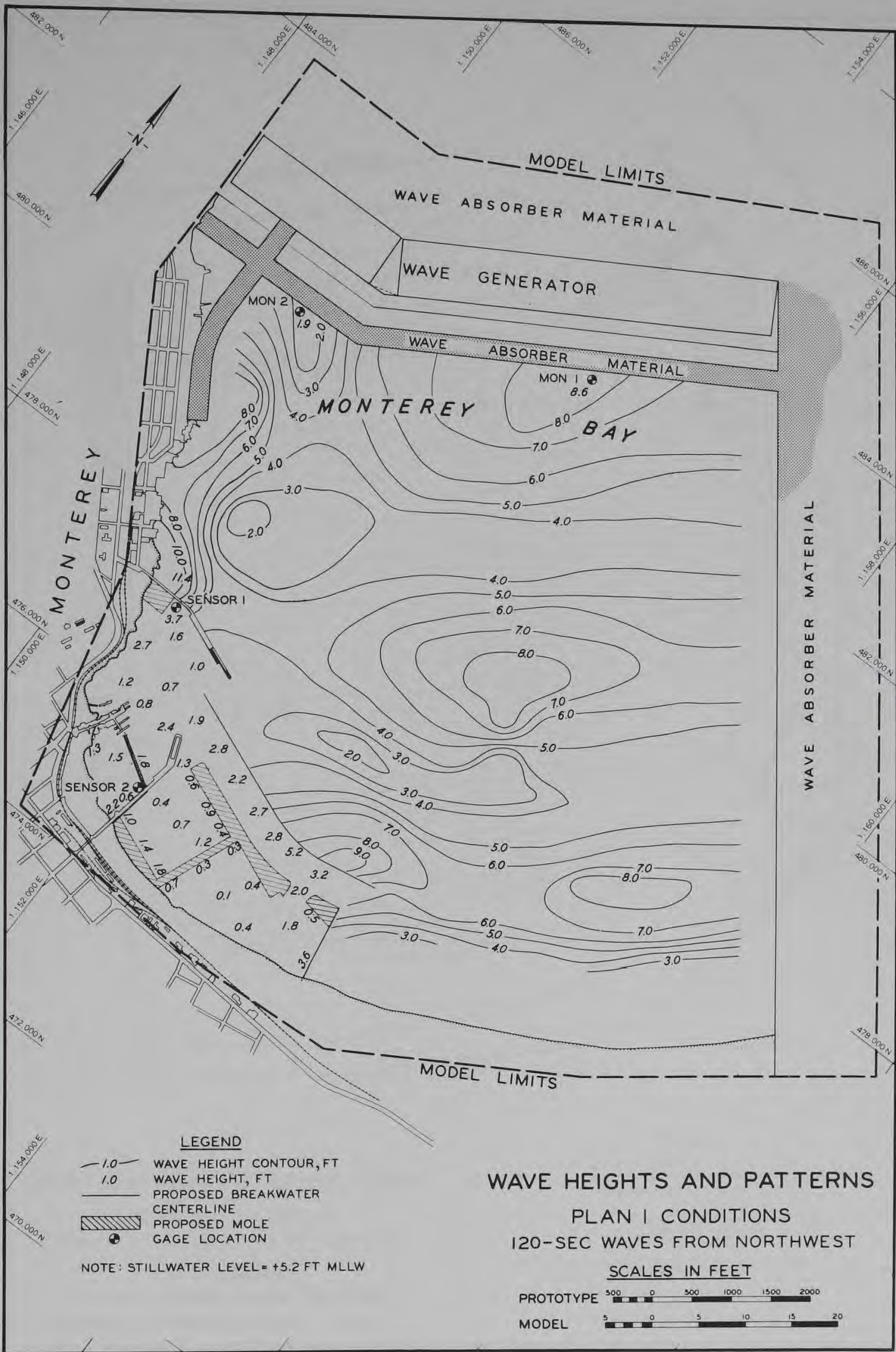
PLAN 1 CONDITIONS

55-SEC WAVES FROM NORTHWEST

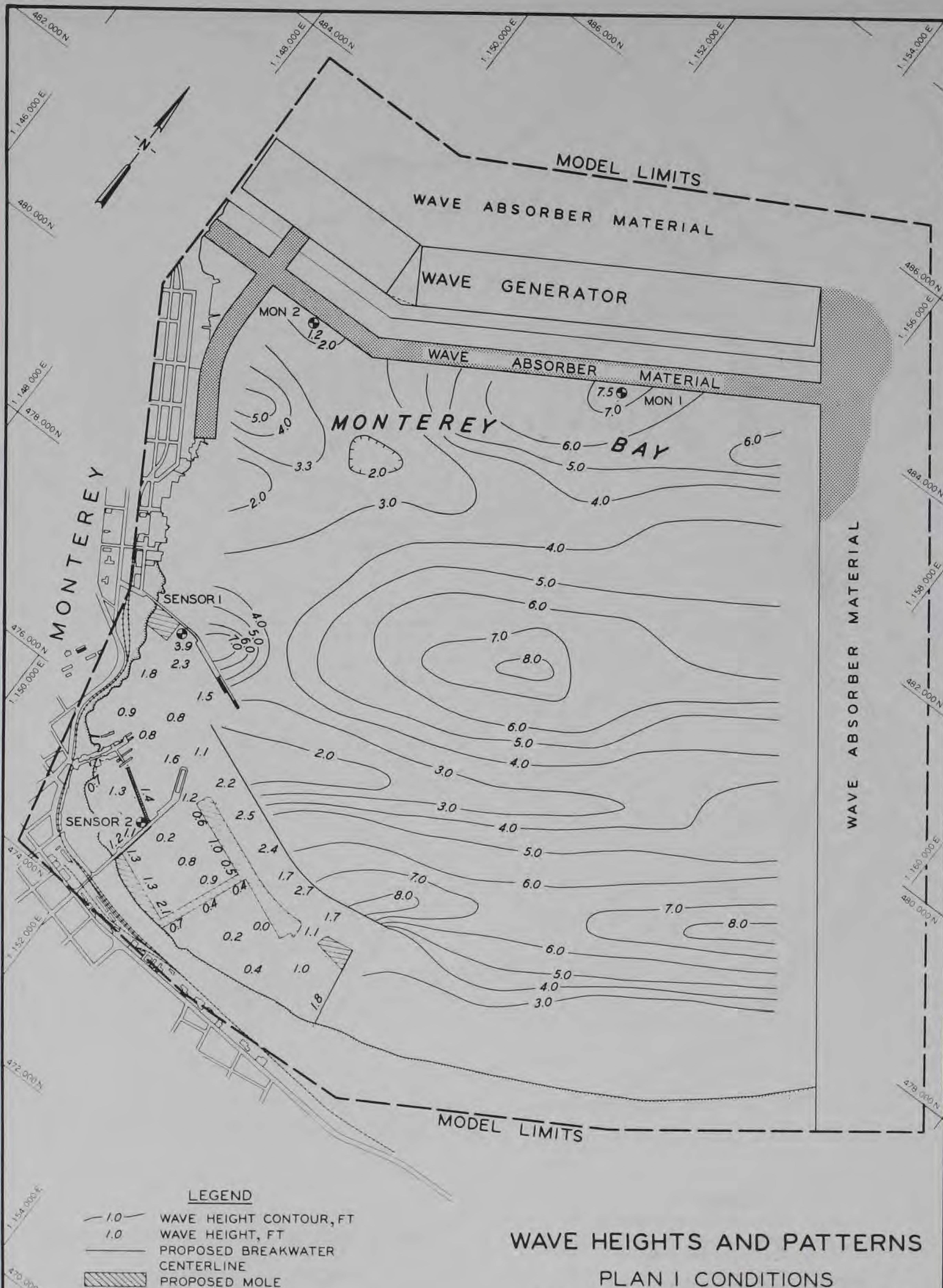
SCALES IN FEET

PROTOTYPE 500 0 500 1000 1500 2000

MODEL 5 0 5 10 15 20



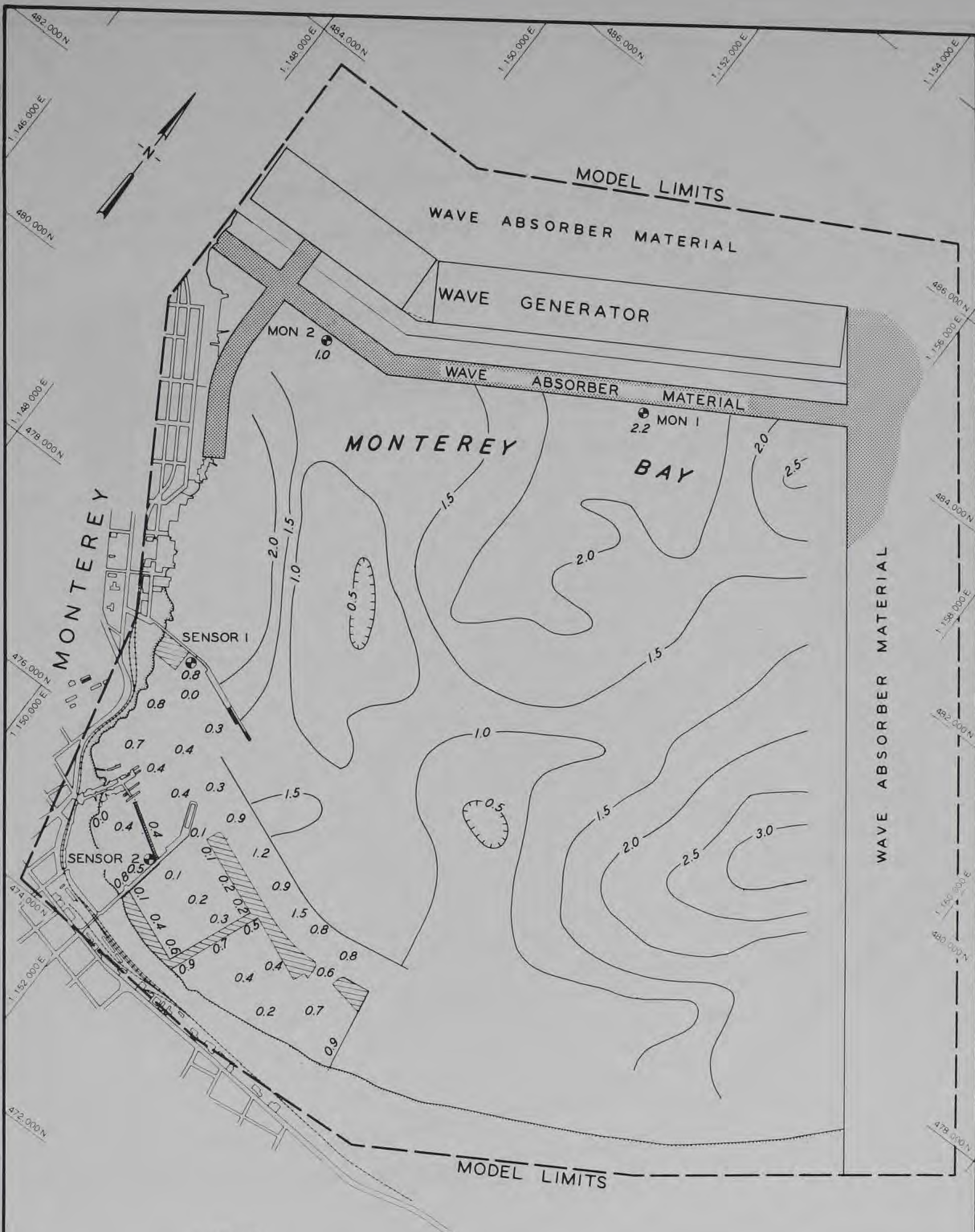
MODEL LIMITS
WAVE ABSORBER MATERIAL
WAVE GENERATOR
WAVE ABSORBER MATERIAL
MON 2
MON 1
SENSOR 1
SENSOR 2
MONTEREY BAY
MONTEREY
MODEL LIMITS
LEGEND
1.0 WAVE HEIGHT CONTOUR, FT
1.0 WAVE HEIGHT, FT
PROPOSED BREAKWATER CENTERLINE
PROPOSED MOLE
GAGE LOCATION
NOTE: STILLWATER LEVEL = +5.2 FT MLLW
WAVE HEIGHTS AND PATTERNS
PLAN 1 CONDITIONS
120-SEC WAVES FROM NORTHWEST
SCALES IN FEET
PROTOTYPE 500 0 500 1000 1500 2000
MODEL 5 0 5 10 15 20



WAVE HEIGHTS AND PATTERNS

PLAN 1 CONDITIONS

130-SEC WAVES FROM NORTHWEST



WAVE HEIGHTS AND PATTERNS

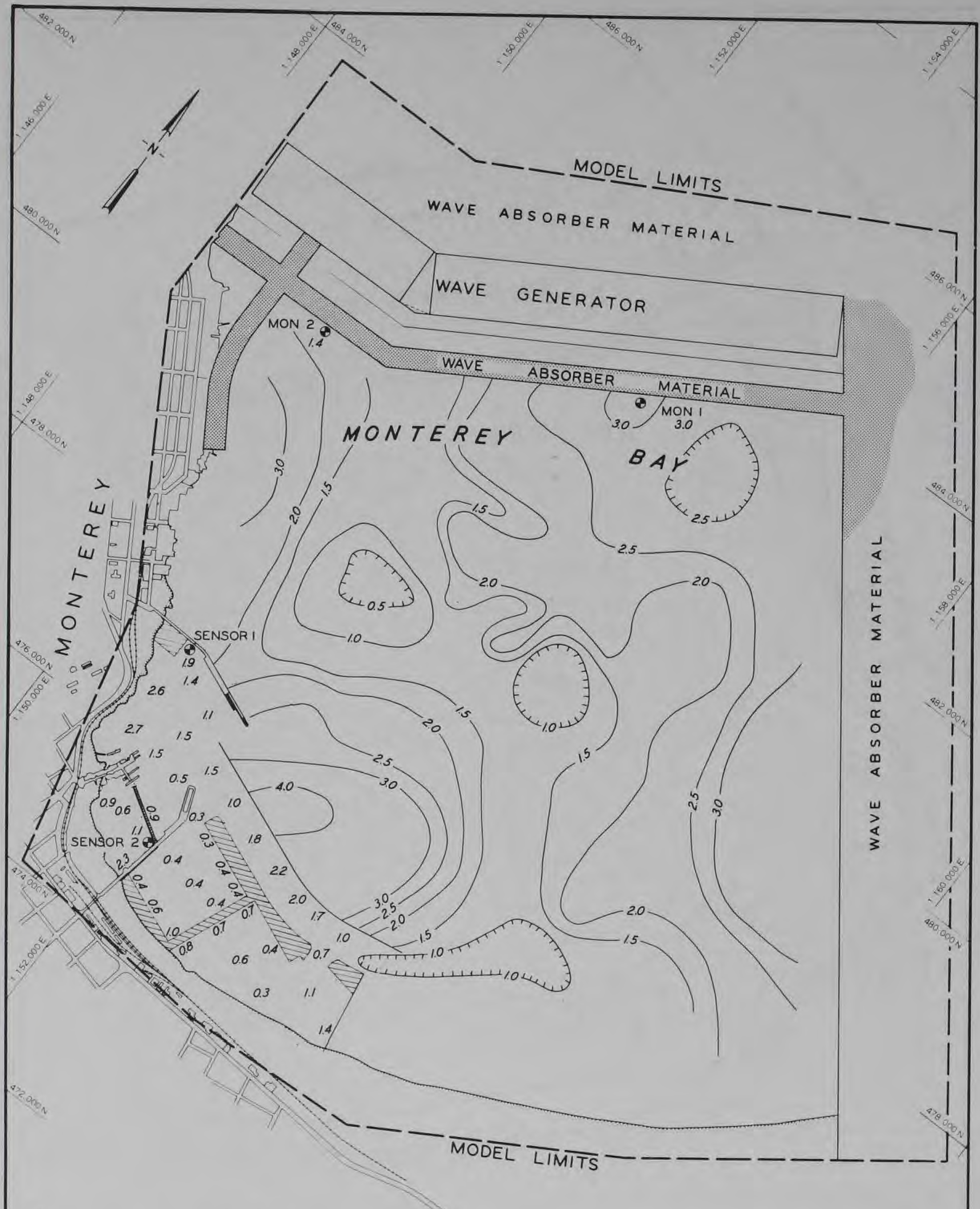
PLAN 1 CONDITIONS

170-SEC WAVES FROM NORTHWEST

SCALES IN FEET

PROTOTYPE 500 0 500 1000 1500 2000

MODEL 5 0 5 10 15 20



LEGEND

—1.0—	WAVE HEIGHT CONTOUR, FT
1.0	WAVE HEIGHT, FT
———	PROPOSED BREAKWATER CENTERLINE
▨	PROPOSED MOLE
⊕	GAGE LOCATION

NOTE: STILLWATER LEVEL = +5.2 FT MLLW

WAVE HEIGHTS AND PATTERNS

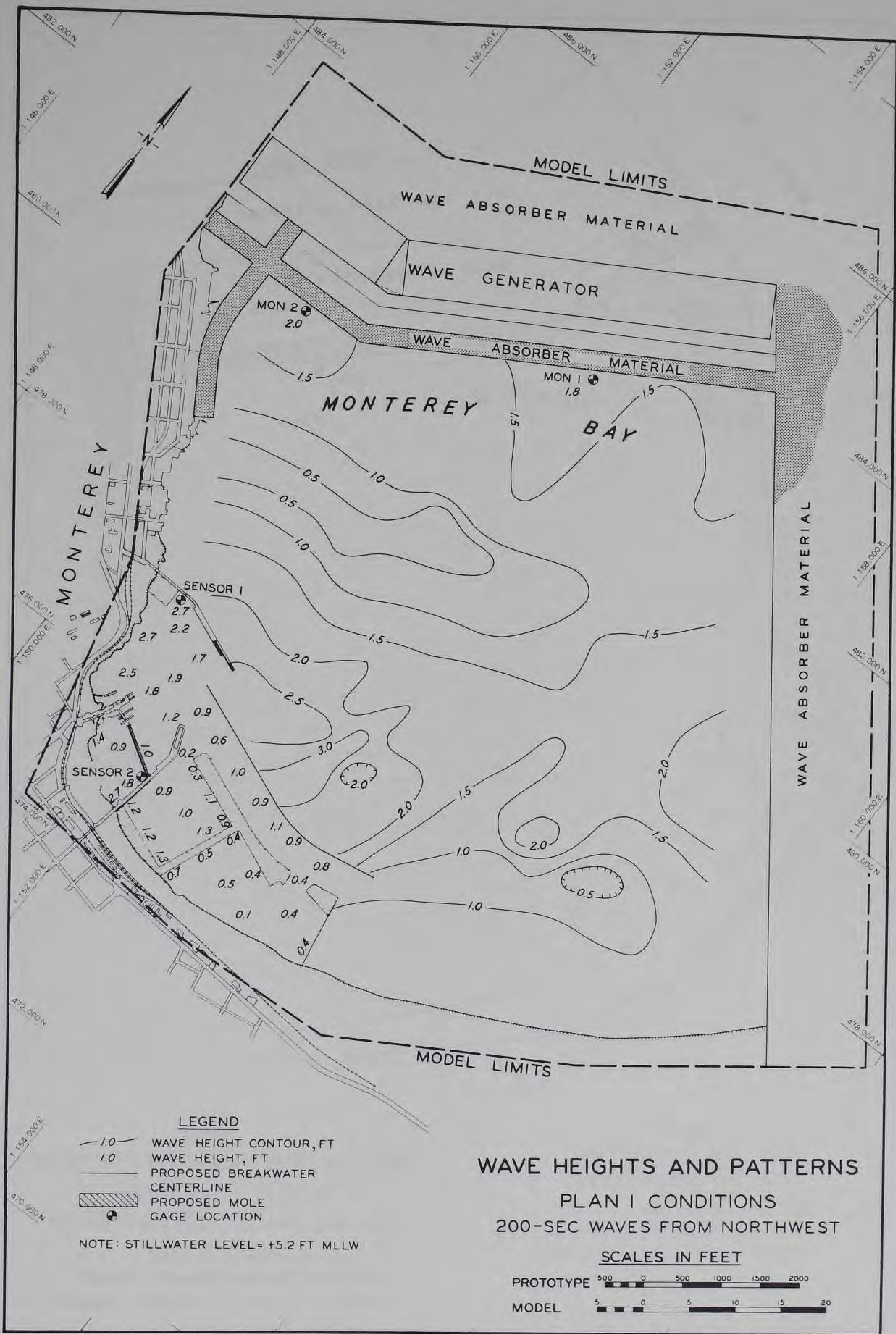
PLAN I CONDITIONS

180-SEC WAVES FROM NORTHWEST

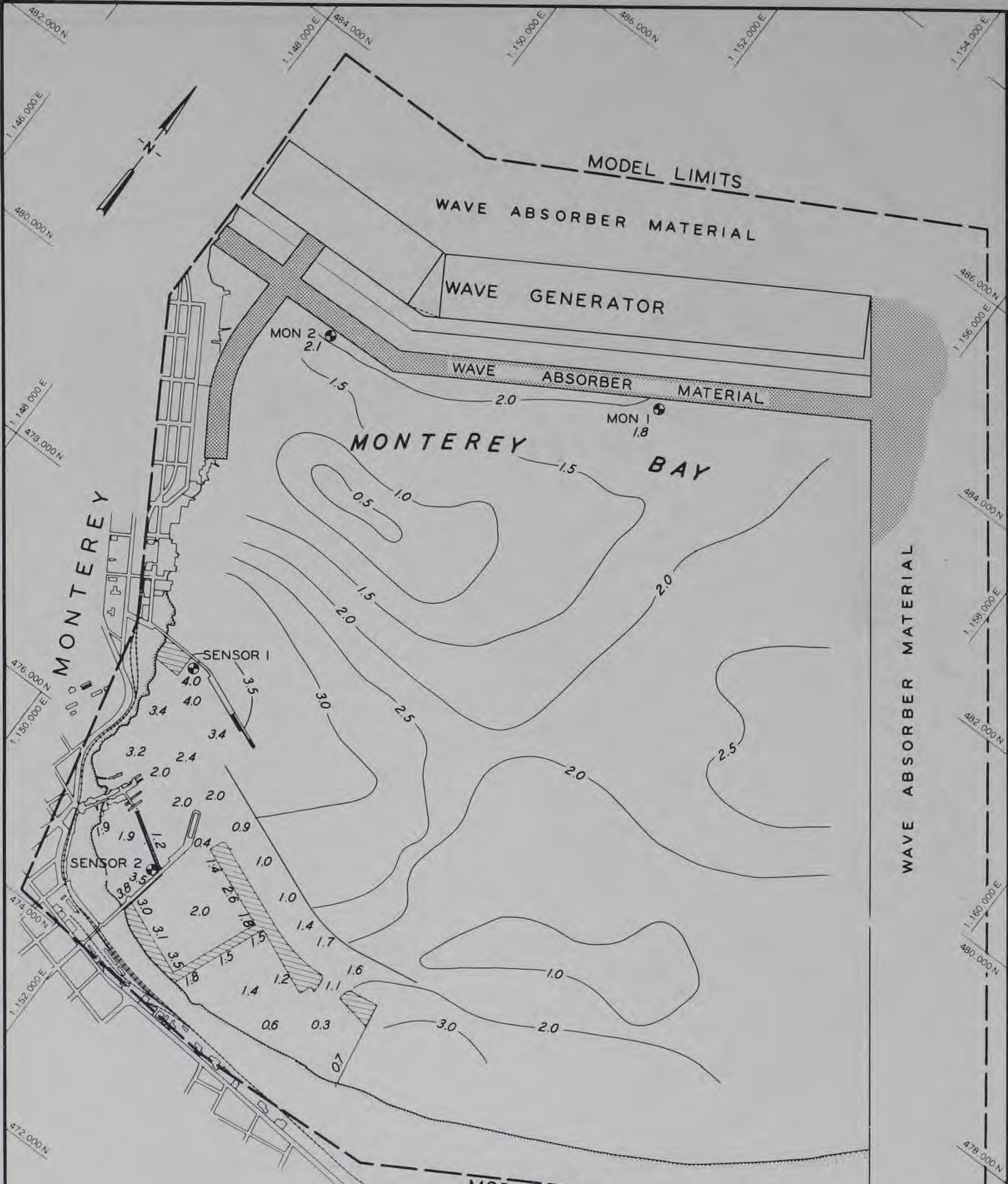
SCALES IN FEET

PROTOTYPE 500 0 500 1000 1500 2000

MODEL 5 0 5 10 15 20



WAVE HEIGHTS AND PATTERNS
PLAN 1 CONDITIONS
200-SEC WAVES FROM NORTHWEST



LEGEND

- 1.0 — WAVE HEIGHT CONTOUR, FT
- 1.0 WAVE HEIGHT, FT
- PROPOSED BREAKWATER CENTERLINE
- PROPOSED MOLE
- GAGE LOCATION

NOTE: STILLWATER LEVEL = +5.2 FT MLLW

WAVE HEIGHTS AND PATTERNS

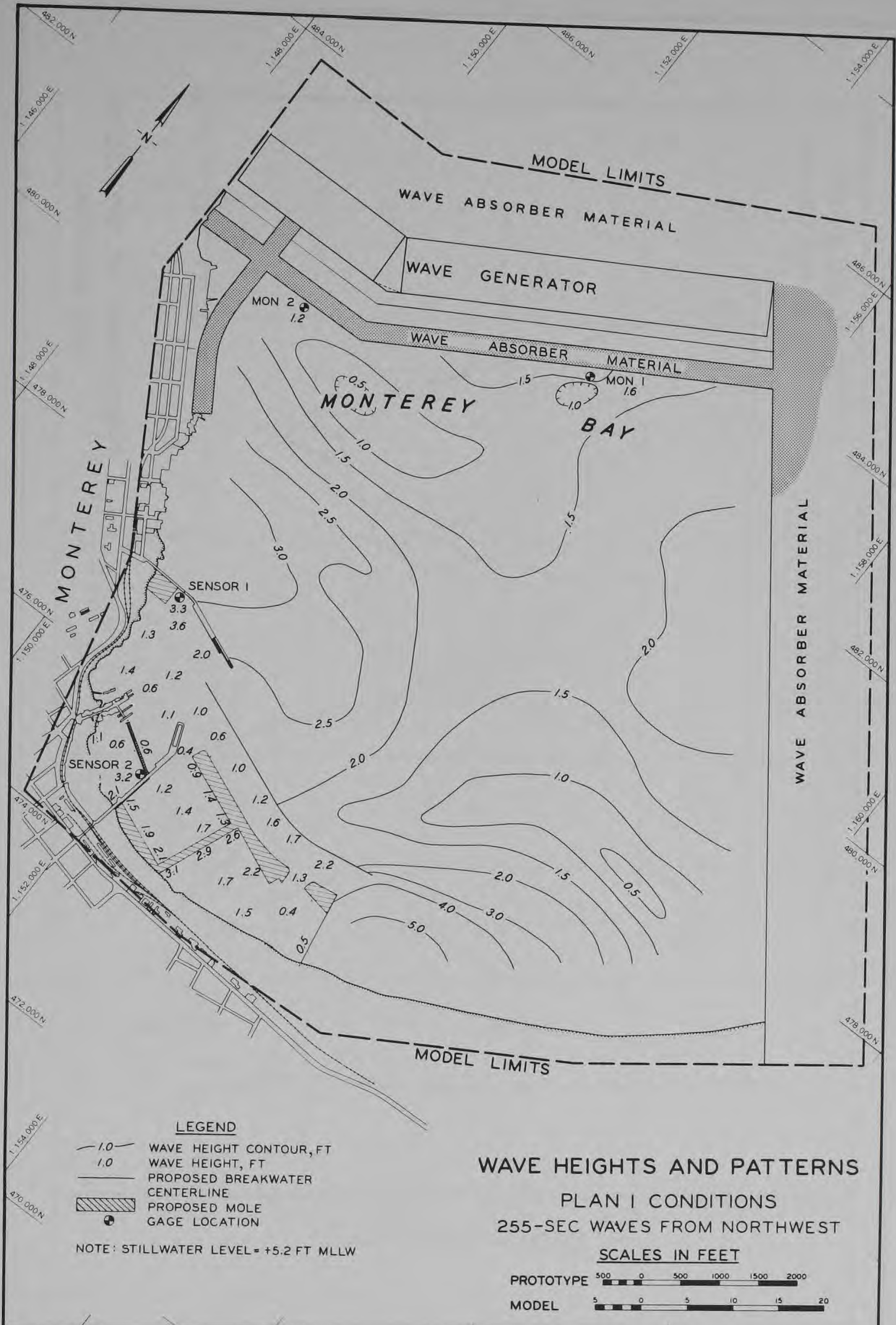
PLAN 1 CONDITIONS

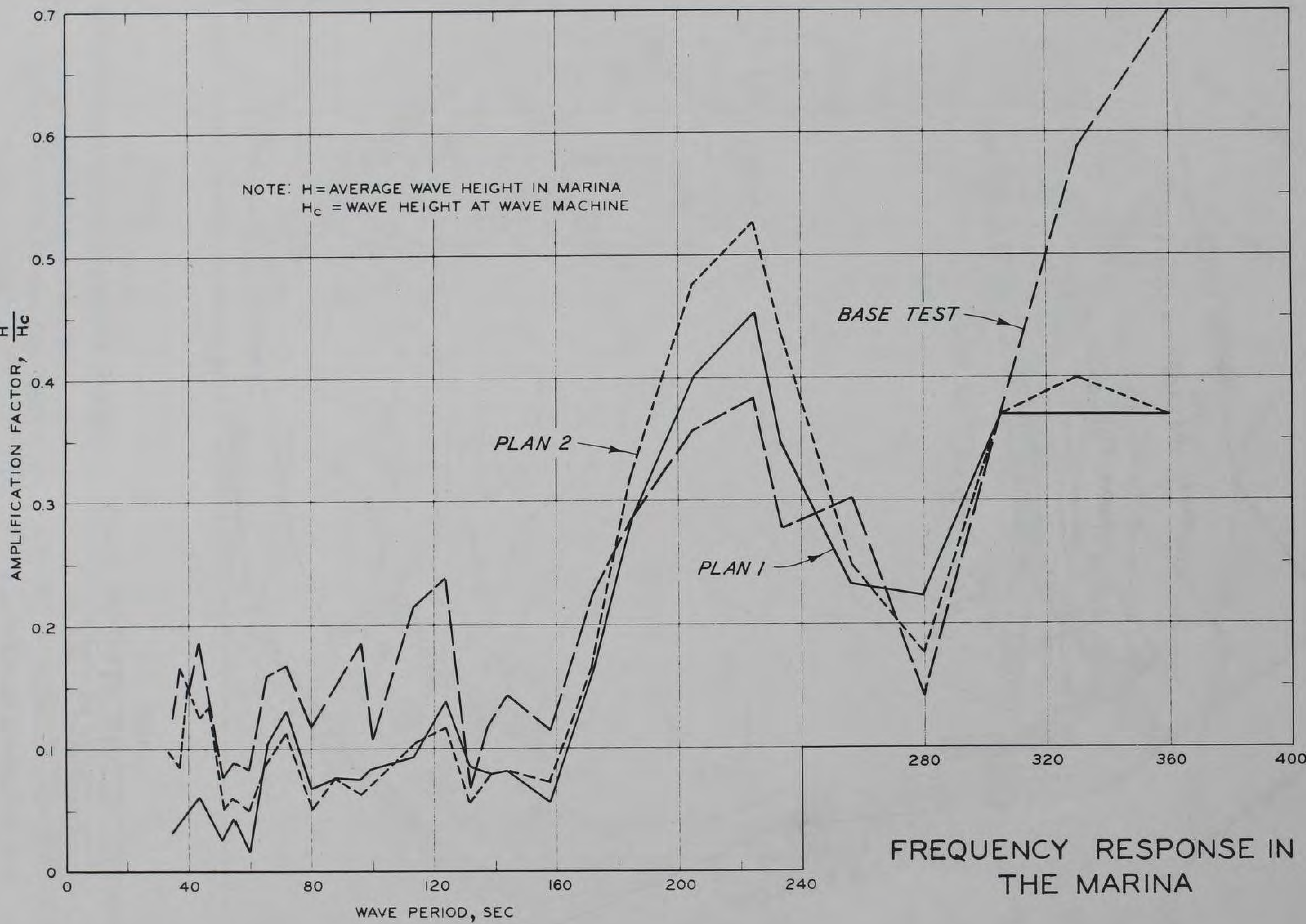
230-SEC WAVES FROM NORTHWEST

SCALES IN FEET

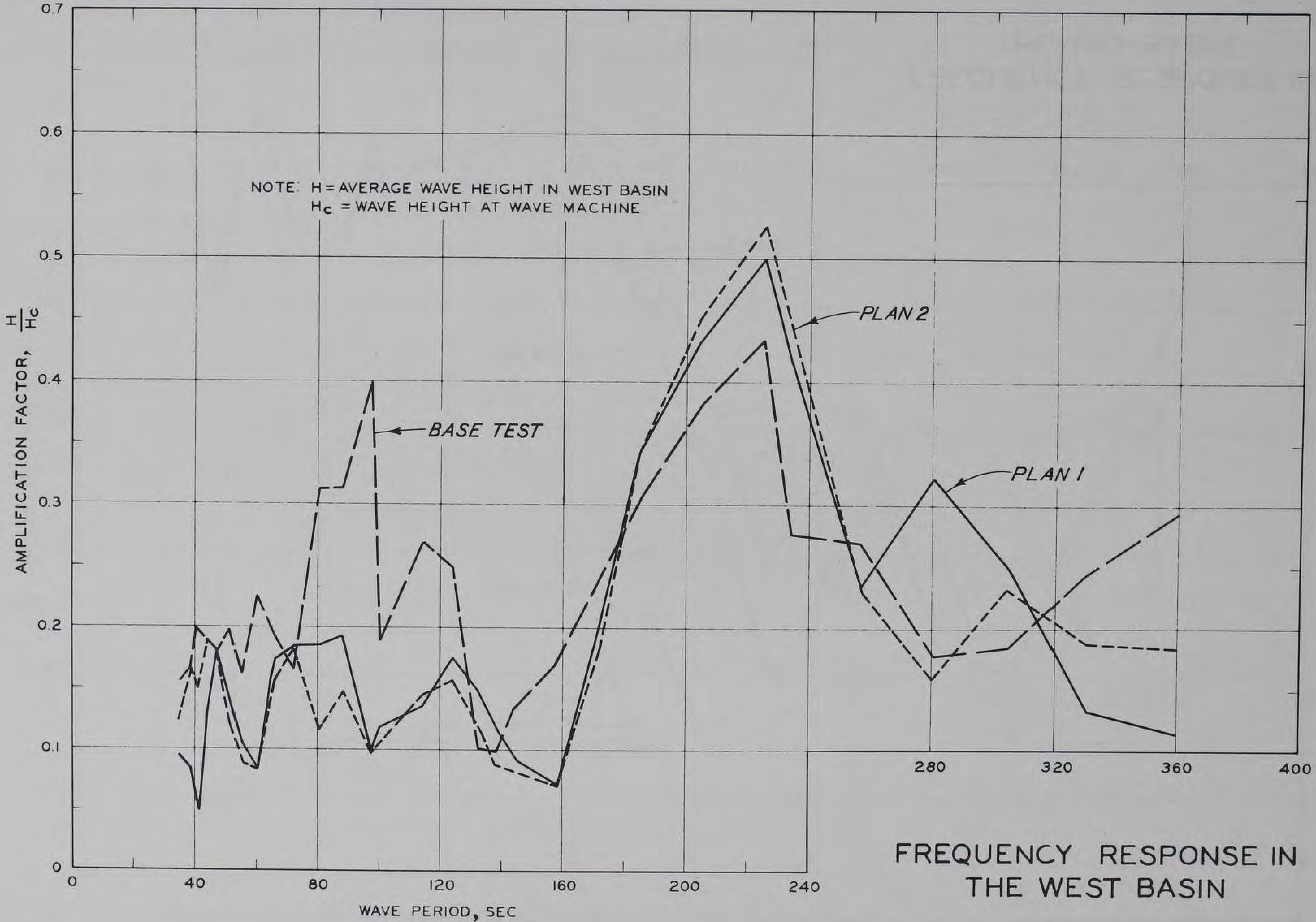
PROTOTYPE 500 0 500 1000 1500 2000

MODEL 5 0 5 10 15 20

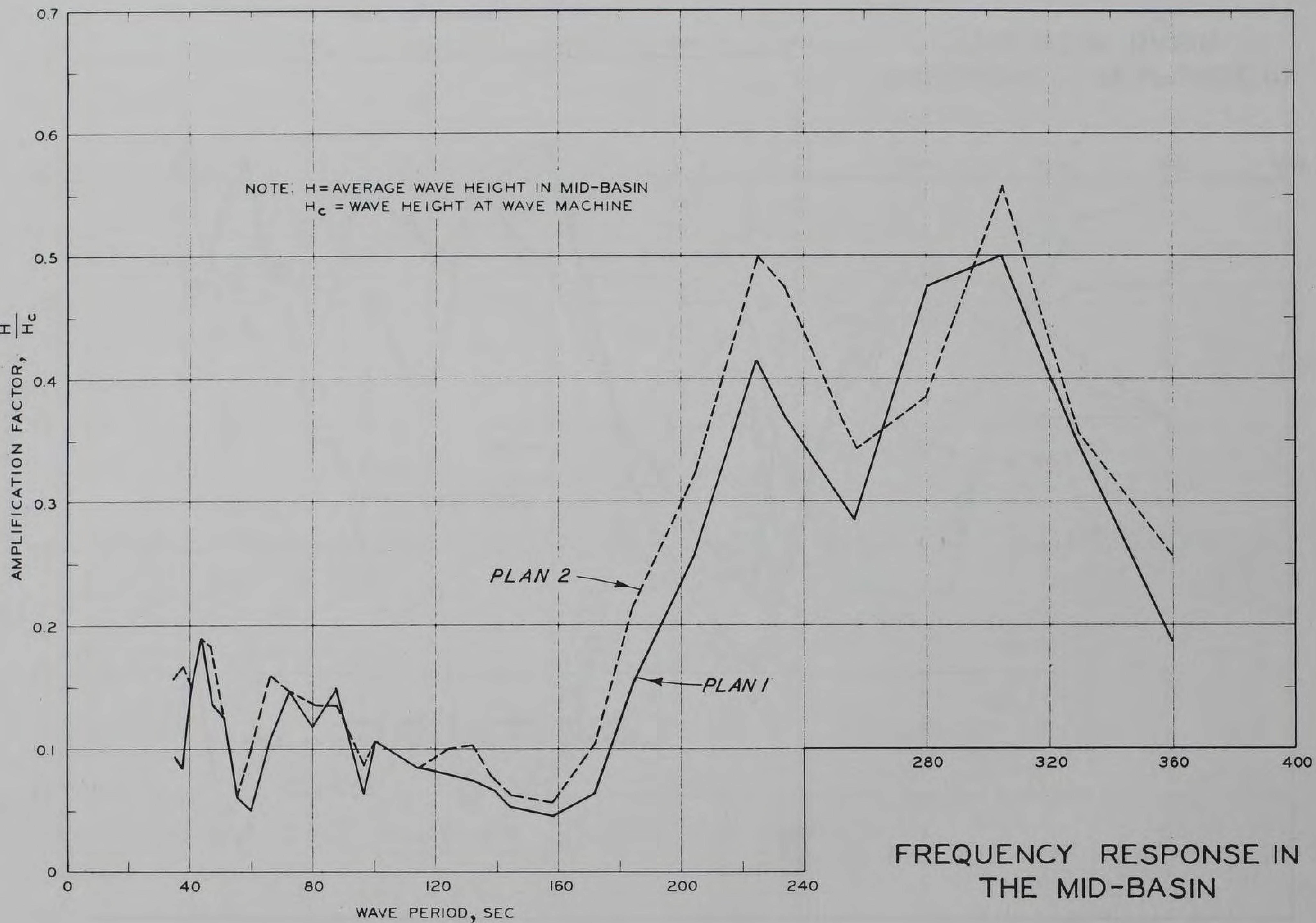

MODEL LIMITS
WAVE ABSORBER MATERIAL
WAVE GENERATOR
WAVE ABSORBER MATERIAL
MON 2
MON 1
MONTEREY BAY
MONTEREY
SENSOR 1
SENSOR 2
WAVE ABSORBER MATERIAL
MODEL LIMITS
LEGEND
1.0 WAVE HEIGHT CONTOUR, FT
1.0 WAVE HEIGHT, FT
PROPOSED BREAKWATER CENTERLINE
PROPOSED MOLE
GAGE LOCATION
NOTE: STILLWATER LEVEL = +5.2 FT MLLW
WAVE HEIGHTS AND PATTERNS
PLAN 1 CONDITIONS
255-SEC WAVES FROM NORTHWEST
SCALES IN FEET
PROTOTYPE 500 0 500 1000 1500 2000
MODEL 5 0 5 10 15 20



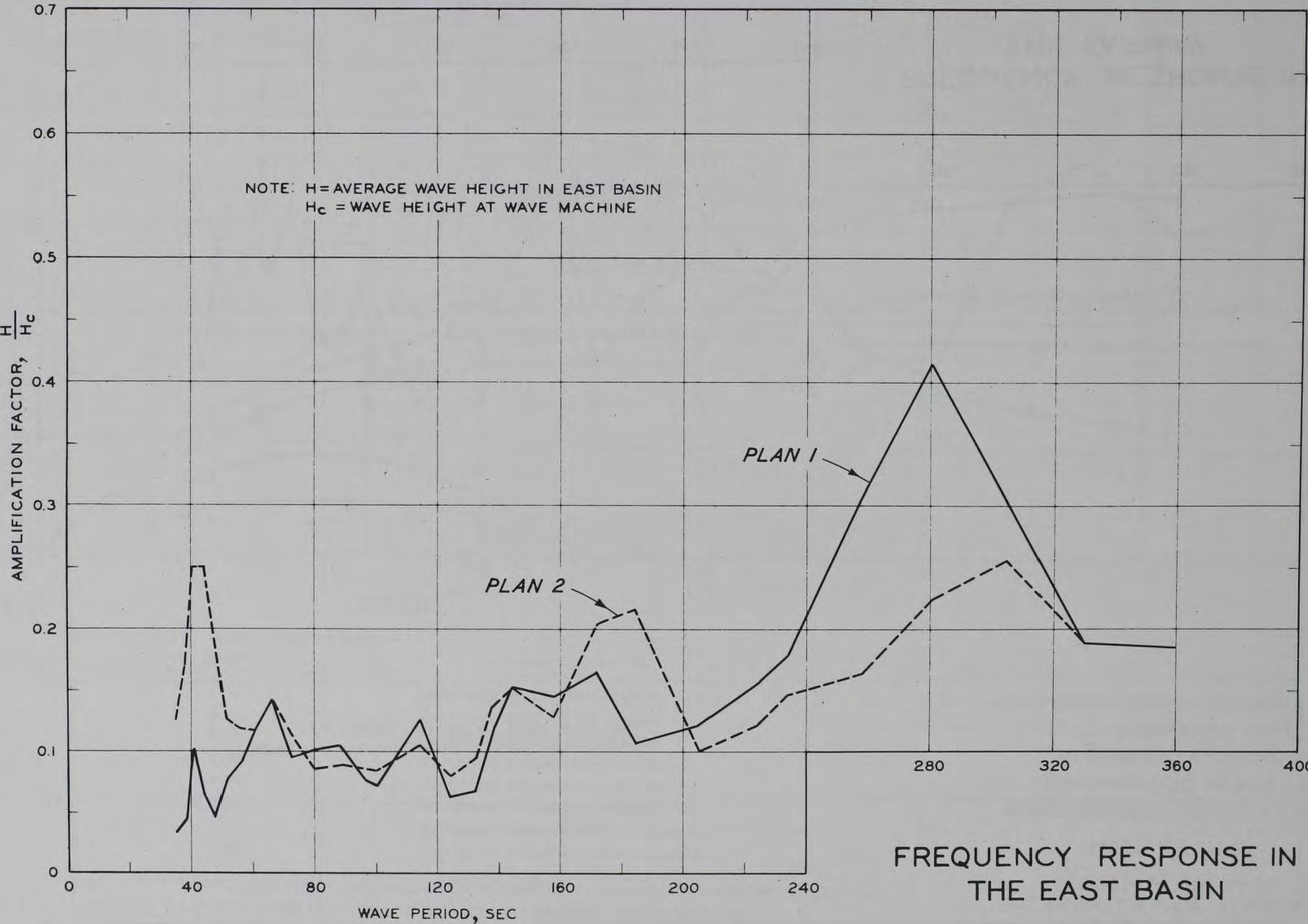
FREQUENCY RESPONSE IN THE MARINA



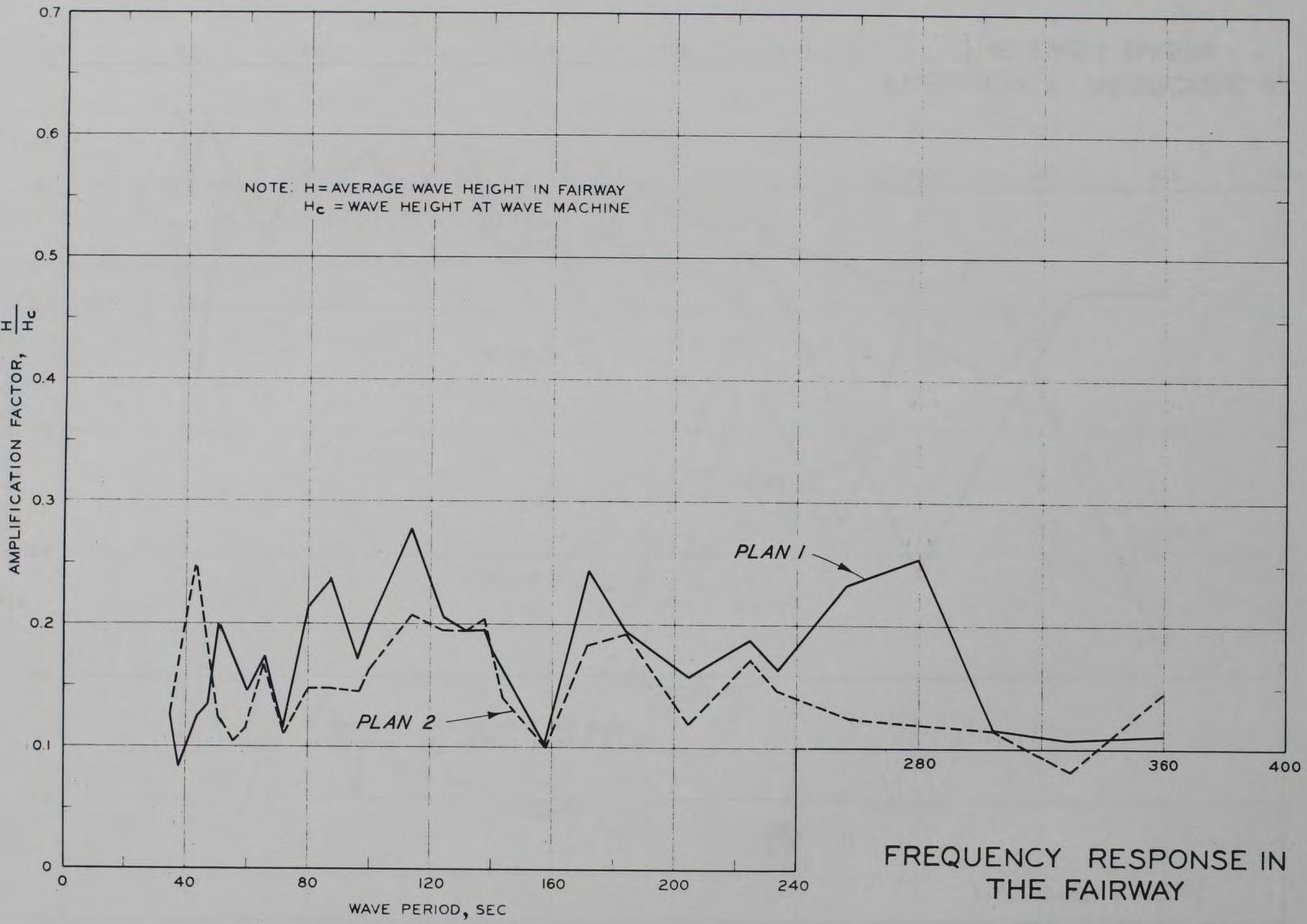
FREQUENCY RESPONSE IN THE WEST BASIN



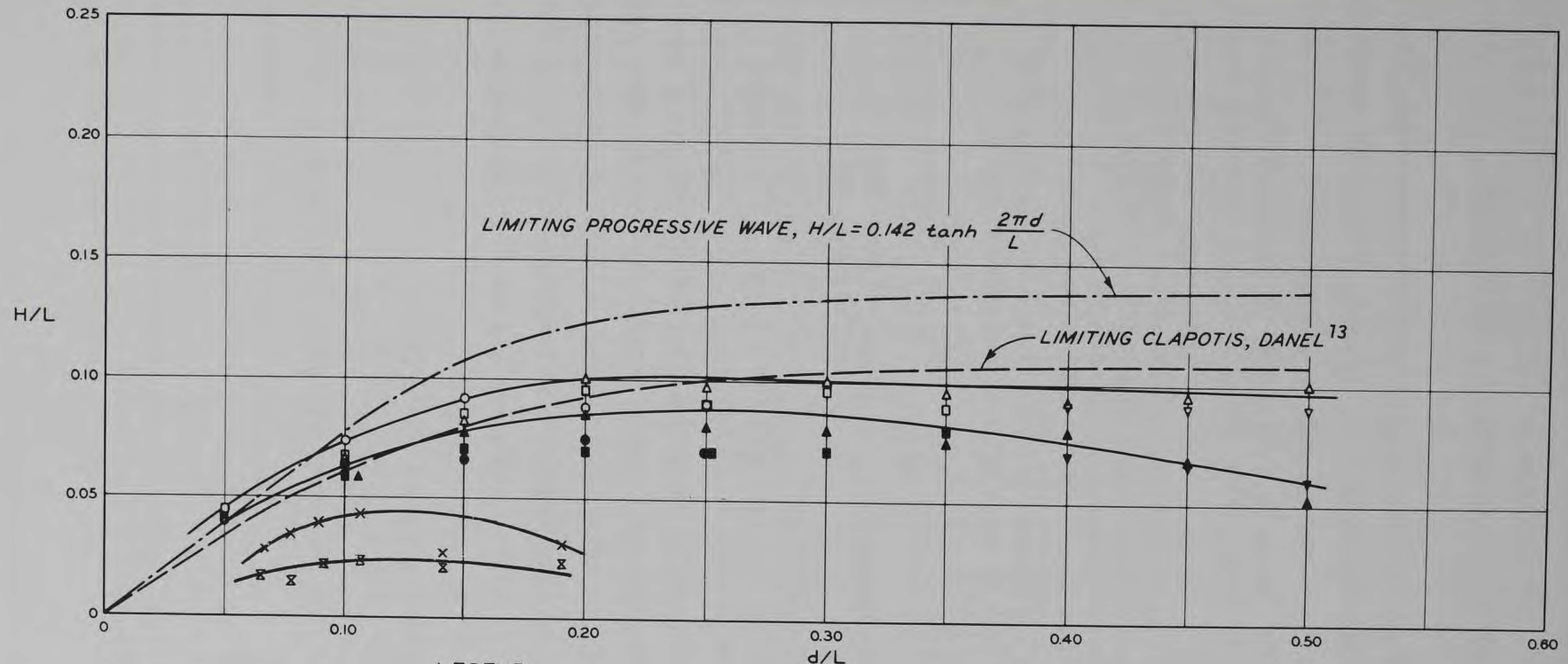
FREQUENCY RESPONSE IN THE MID-BASIN



FREQUENCY RESPONSE IN THE EAST BASIN



FREQUENCY RESPONSE IN THE FAIRWAY



LEGEND

SYMBOL	WATER DEPTH IN FEET REFERRED TO STILL-WATER LEVEL	WAVE FORM
○	0.50	MAXIMUM BREAKING WAVES
□	0.75	MAXIMUM BREAKING WAVES
△	1.25	MAXIMUM BREAKING WAVES
▽	1.50	MAXIMUM BREAKING WAVES
●	0.50	MAXIMUM NONBREAKING WAVES
■	0.75	MAXIMUM NONBREAKING WAVES
▲	1.25	MAXIMUM NONBREAKING WAVES
▼	1.50	MAXIMUM NONBREAKING WAVES

× MONTEREY PROPOSED NORTH BREAKWATER (SIGNIFICANT HINDCAST WAVE HEIGHT)

⧗ MONTEREY EXISTING BREAKWATER (SIGNIFICANT HINDCAST WAVE HEIGHT)

MONTEREY DESIGN-WAVE HEIGHT DATA COMPARED WITH LIMITING BREAKING AND NONBREAKING WAVES ON RUBBLE-MOUND BREAKWATERS

H/L VS d/L

BEACH SLOPE = 1:50
BREAKWATER SIDE SLOPE = 1:1.5

APPENDIX A: SELECTION OF LONG-PERIOD TEST WAVES

Excitation and Response of Monterey Harbor

1. In a comprehensive feasibility study for a surge-action model of Monterey Harbor, California,[4*] the following remarks were made regarding excitation and response of the harbor.

a. Periods T_n = 13.3, 6.8, 4.52, 3.56, 3.02, 2.58, 2.22, 1.94, 1.68 min (sequence 1)

are apparently representative of modes of oscillations in which the southern portion of Monterey Bay responds to stimulation. The above periods (sequence 1) were obtained by a two-dimensional numerical "talweg" solution of the oscillating properties of the southern portion of the bay.

b. Peaks on the sea-energy spectra (fig. A1) showed the following congruencies with some of the above figures.

T_n = 13.3, 5.8-6.7, 3.9-4.4, 2.9-3.5, 2.4-2.6, 2.0, 2.2, 1.7-1.9 min (sequence 2)

c. Residuation analyses of long-wave records yield the following similarities:

T_n = 11.0-13.6, 6.1-6.3, 3.8-4.6, 2.2-2.9, 1.7-1.96 min (sequence 3)

2. Items b and c above indicate that several of the periods in sequence 1 are important to the regime of oscillations affecting Monterey Harbor. The nature of some of these periods in the immediate harbor vicinity was explored with the aid of wave-refraction diagrams. Periods selected for study by refraction diagrams were those that seem to be consistently strong in the records, namely

$$T_n = 13.3, 6.1, 4.3, 2.5 \text{ min} \qquad \text{(sequence 4)}$$

Periods at or near those given in sequence 4 will be resonant for the near-harbor area.

3. The wave-refraction diagram shown in fig. A2 shows that the

---

* Raised numerals refer to items in the Literature Cited at end of main text of this report.

2.5-min oscillation external to the harbor has a node directly in line with the mouth of the marina. It was noted that a periodicity of about 2.5 min will excite an open-mouth fundamental oscillation in the marina (section 2 of reference 4).

4. The next external resonant oscillation of 4.3 min (fig. A3) has a node which will penetrate the entrance of the harbor close to the end of Municipal Wharf No. 2. The outer-harbor basin tends to have a fundamental period of oscillation of its own of precisely 4.3 min as indicated in section 2 of reference 4. The 4.3-min periodicity is not otherwise crucial to the marina.

5. The wave-refraction diagram in fig. A4 shows that the 6.1-min oscillation has a node slightly seaward of the breakwater. The harbor will therefore be in the antinodal area, and response will be moderate. If the external oscillation has a slight off-resonance period of 5 to 6 min, the node would probably be moved shoreward from the position shown in fig. A4, to a position approximately in line with the breakwater, therefore making the external basin of the harbor antinodal at this period. However, since the 5- to 6-min oscillation is approximately twice the fundamental open-mouth period for the marina, it would tend to "pump" the harbor, and some moderate to strong flushing of the marina entrance could be expected. Fig. A1 shows that sensor 2 response to 5.2- to 5.8-min frequencies in the marina is pronounced, as might be expected.

6. The oscillation of 13.3 min (fig. A5) shows the whole region near the harbor to be antinodal. The harbor itself does not have any tendency to echo this frequency, and consequently the response registered on all sensors will be nominal as shown in fig. A1. Based on the above discussion, Wilson[4] concluded that the wave periods of concern to Monterey Harbor are likely to be less than 3 min and certainly less than 7 min.

Modeling the Resonance Phenomenon in the Laboratory

7. It was decided to conduct a model study for Monterey Harbor to investigate the arrangement and design of certain proposed harbor improvements with respect to wave and surge action and to determine current conditions in the navigation entrances to the harbor and its basins. In

such a model study it is important to use small time increments in varying the wave period to ensure that the complete response is being evaluated. Just how small the time increments should be is not known; however, previous studies on the problem of harbor resonance such as that by Ippen[14] provide some guidance.

8. Ippen[14] studied the wave-induced oscillations for a rectangular harbor connected to the open sea with a straight coastline. The harbor is fully or partially open with a symmetric or asymmetric entrance. Using the notations shown in fig. A6, the following assumptions were made in the analysis.

- a. The harbor is excited by a regular wave train moving normal to the coastline.
- b. All boundaries are perfectly reflecting.
- c. The water depth is constant over the whole region in consideration.
- d. Small-amplitude wave theory is applicable.
- e. The harbor entrance is small compared to the wavelength ($kd \leq 1$, where k is the wave number). For the detailed mathematical treatment the interested reader is referred to reference 14; only the most important results pertinent to the Monterey Harbor model study are presented here as figs. A7-A10.

9. Fig. A7 shows a comparison between the theoretical and experimental response curves for a fully open harbor where $\mathfrak{R}$ is the amplification factor at a corner of the back wall of the harbor (fig. A6). It can be seen from fig. A7 that the experimental frequency-response curve is flatter than the theoretical one. Therefore, utilization of the theoretical frequency-response curves (figs. A8-A10) in the selection of a band width $\Delta k\ell$ which appears to be narrow enough to show any significant changes in $\mathfrak{R}$ will lead to a conservative time increment ΔT for varying the wave period T in the model to ensure that the complete response is being evaluated.

10. The four basins of Monterey Harbor can be schematized to rectangular response basins as follows:

Basin	Description	2b ft	ℓ ft	2d ft	$2b/\ell$	d/ℓ	h , water depth, ft
West Basin	Fully open harbor (fig. A8)	2400	1300	2400	1.8	0.92	32
Marina	Fully open harbor (fig. A8)	650	1000	650	0.65	0.325	19
Mid-Basin	Partially open harbor (fig. A9)	1000	1000	550	1.0	0.275	29
East-Basin	Partially open harbor (fig. A10)	1900	950	300	2.0	0.16	30

After comparing the dimensions of the Monterey basins with the dimensions of the basins whose theoretical frequency-response curves are shown in figs. A8-A10, a band width $\Delta k\ell$ of 0.2 was selected and was considered adequate so that the complete response would be evaluated in the Monterey Harbor model.

11. For a $\Delta k\ell$ value of 0.2, fig. A11 shows the relation between test wave periods and their corresponding increments for each of the four basins of Monterey Harbor. The following tabulation is an extract from fig. A11:

Basin	T , sec (prototype)	$(T + \Delta T)/T$
West Basin	60-360	1.05-1.39
Marina	60-360	1.05-1.39
Mid-Basin	60-360	1.06-1.54
East Basin	60-360	1.06-1.64

Based on the data shown in fig. A11 and on recommendations given by Wilson[4] regarding wave periods that are likely to be critical for Monterey Harbor, the following long-period test waves were selected.

T = 35, 38, 41, 44, 47, 51, 55, 60, 66, 72, 80, 88,
97, 100.2, 114, 124, 132, 138, 144, 158, 172,
185, 205, 225, 234, 257, 280, 305, 330, 360 sec (sequence 5)

An upper limit of 360 sec and a wave period accuracy of about one percent were imposed by the capability of the model wave machine.

12. In order to check the accuracy in evaluating the basins' responses for the time increments selected above, additional frequency-response tests were conducted at smaller time increments for the portion

of the marina's frequency-response curve between 205 and 234 sec (model results suggested that the marina may be responsive to a 225-sec oscillation). These additional frequency-response tests were conducted using the following wave periods intermediate between the 205- and 234-sec periods: 210, 215, 220, 223, 225, 227, and 230 sec. The results indicated that no serious resonant peaks occurred between the selected periods of 205, 225, and 234 sec. Similar tests for other wave period intervals and for other basins produced similar results.

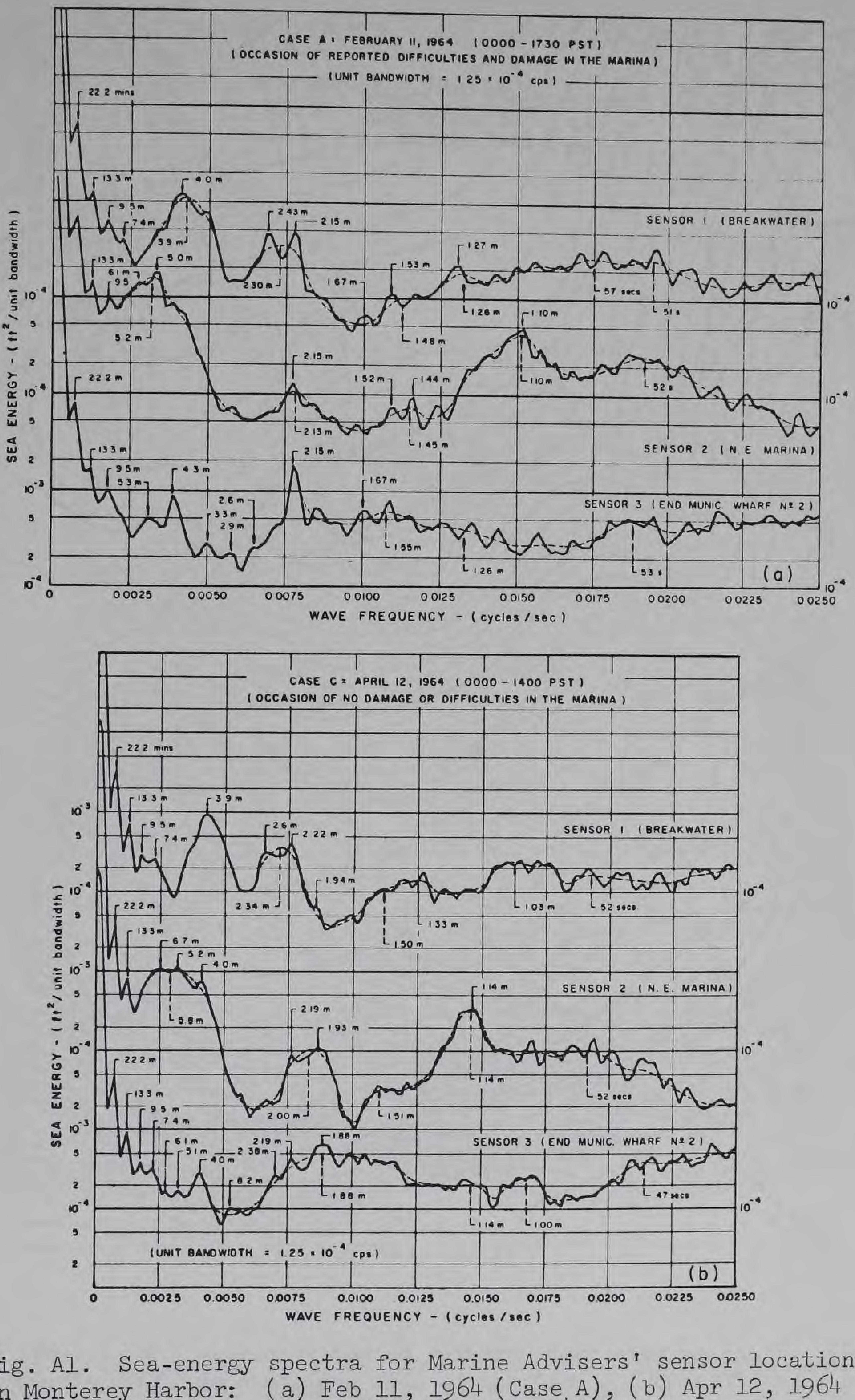


Fig. A1. Sea-energy spectra for Marine Advisers' sensor locations in Monterey Harbor: (a) Feb 11, 1964 (Case A), (b) Apr 12, 1964 (Case C) (after Wilson[4])

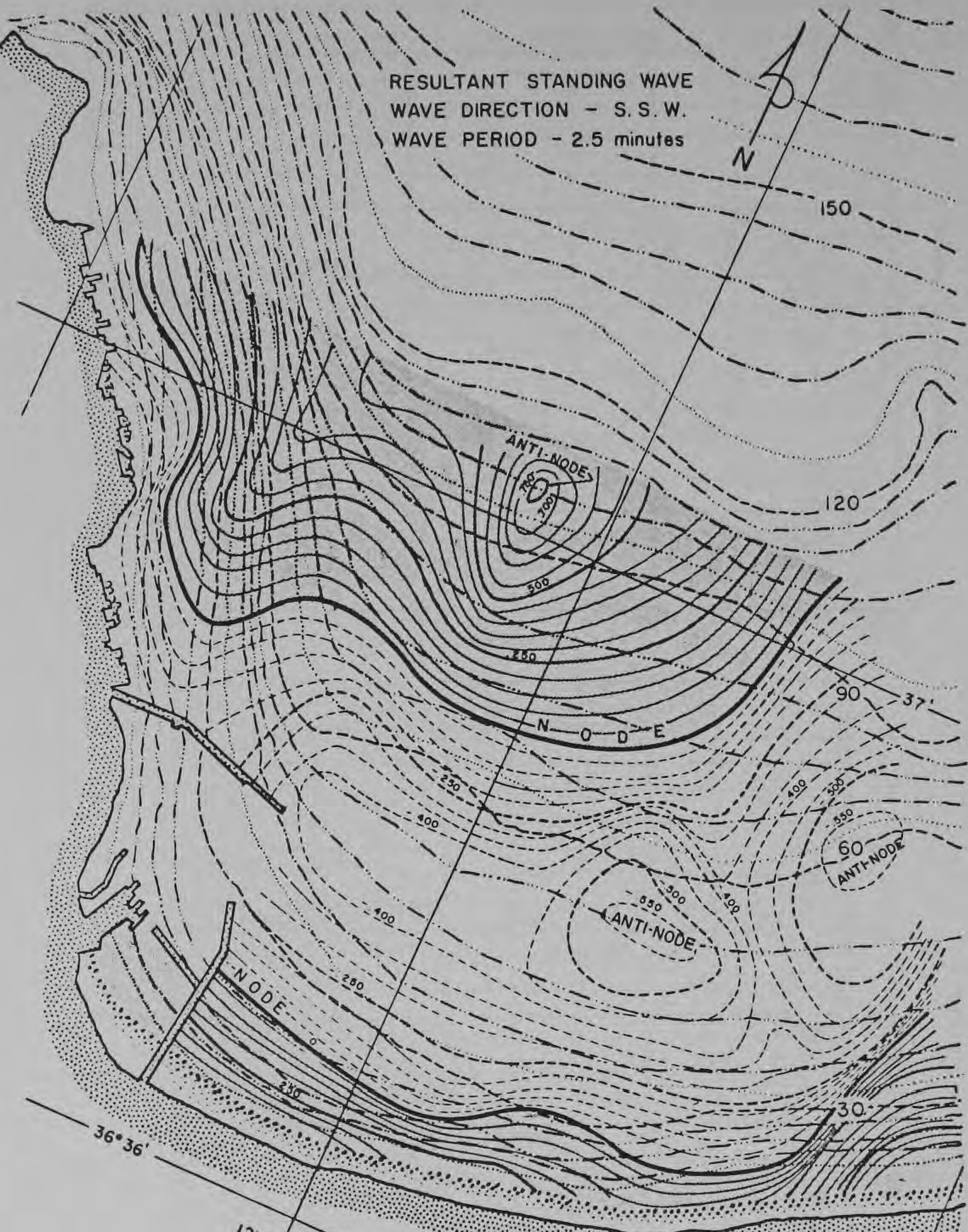


Fig. A2. Standing wave near Monterey Harbor resulting from 2.5-min long-period waves from SSW, normalized to unit amplitude at 10,000-ft water depth (after Wilson[4])

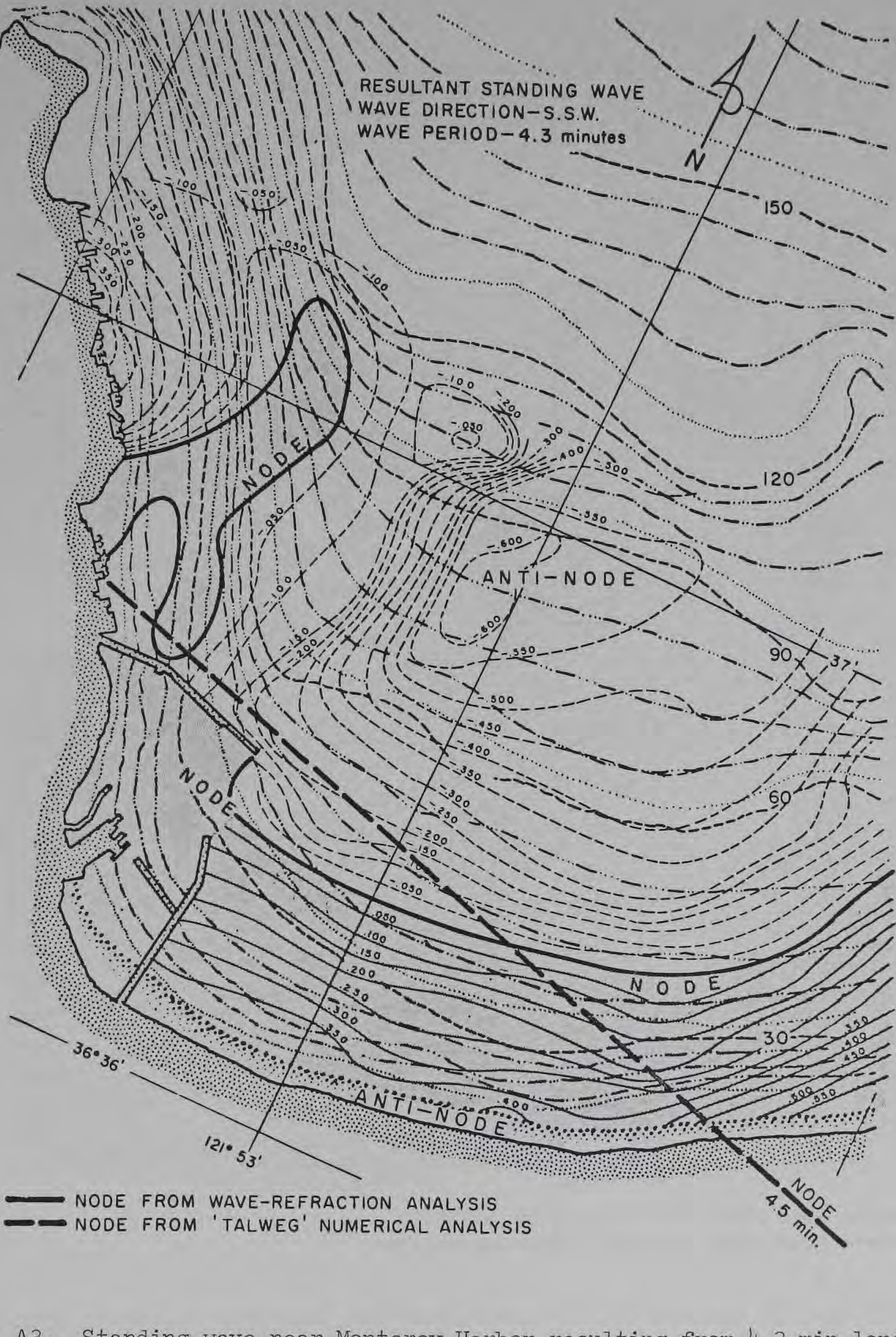


Fig. A3. Standing wave near Monterey Harbor resulting from 4.3-min long-period waves from SSW, normalized to unit amplitude at 10,000-ft water depth (after Wilson[4])

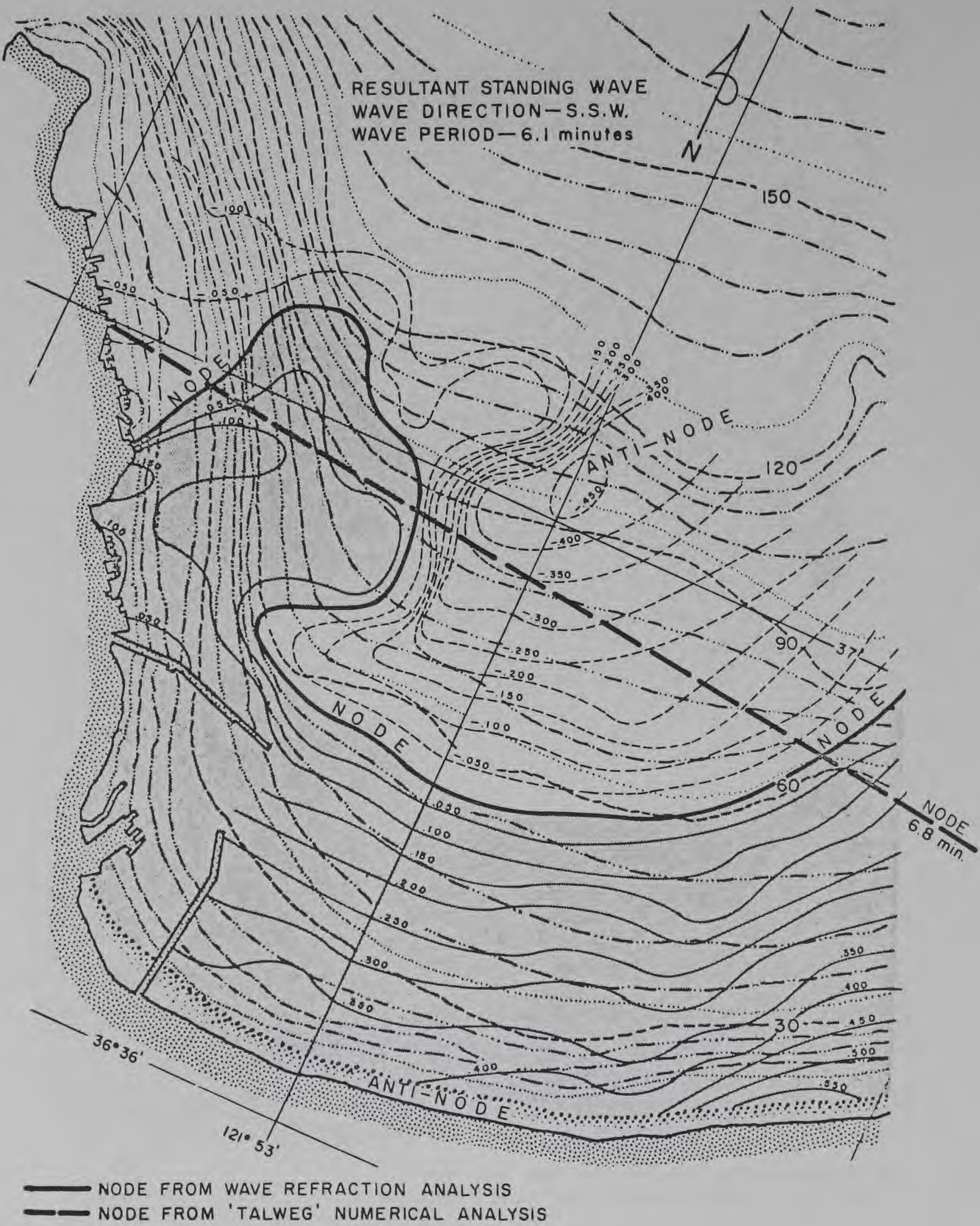


Fig. A4. Standing wave near Monterey Harbor resulting from 6.1-min long-period waves from SSW, normalized to unit amplitude at 10,000-ft water depth (after Wilson[4])

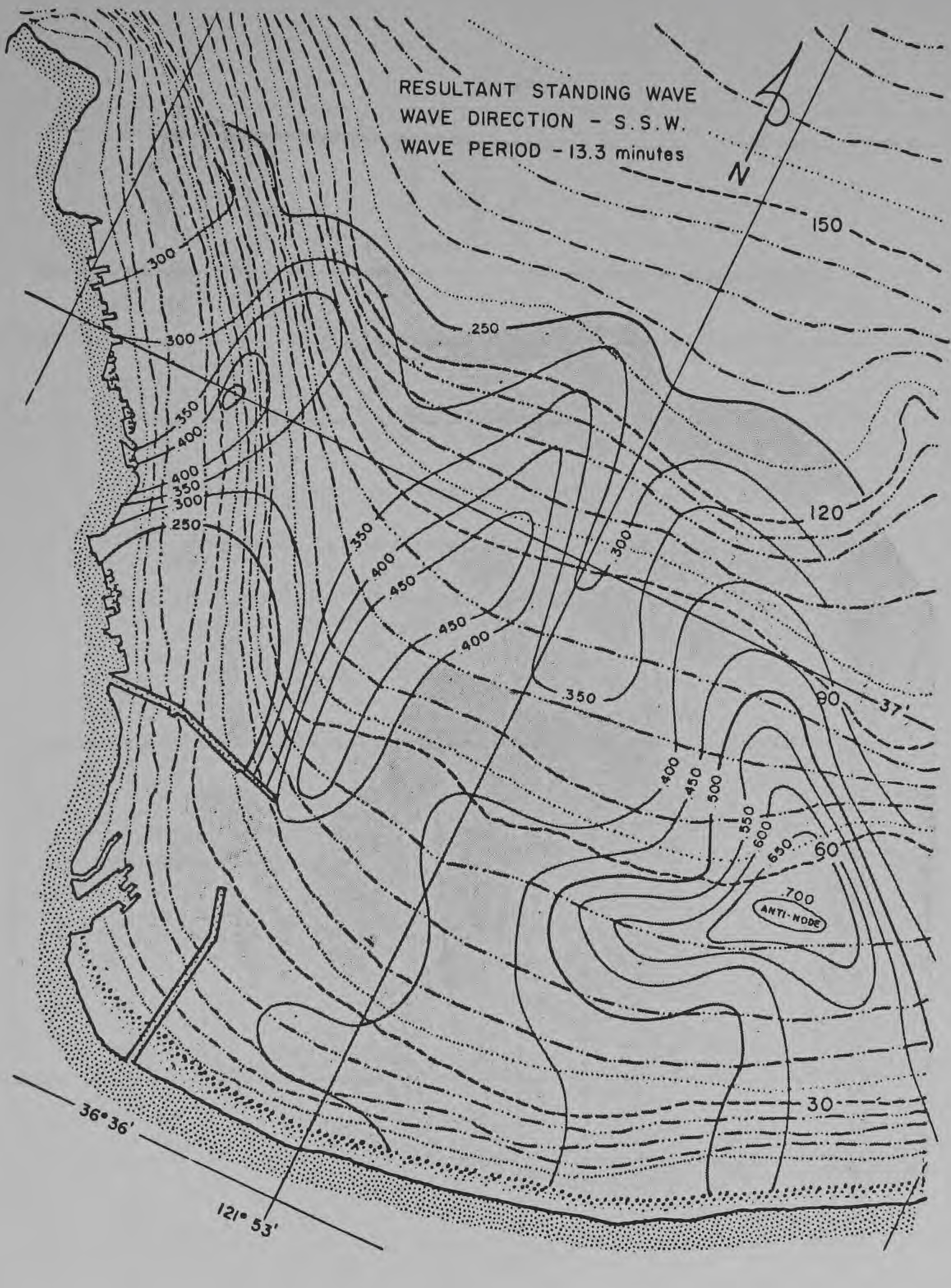


Fig. A5. Standing wave near Monterey Harbor resulting from 13.3-min long-period waves from SSW, normalized to unit amplitude at 10,000-ft water depth (after Wilson[4])

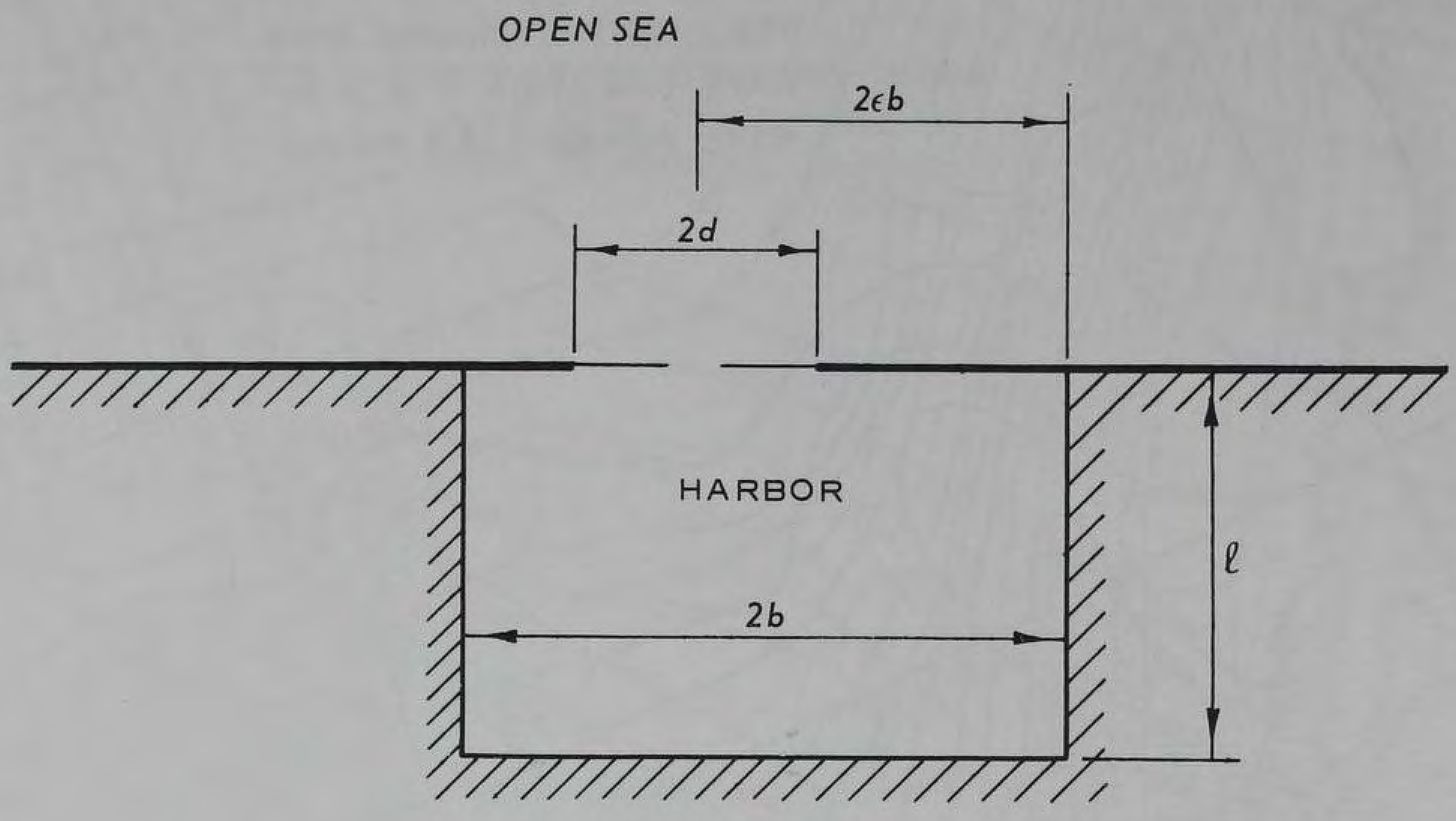


Fig. A6. Definition sketch of the model harbor (after Ippen[14])

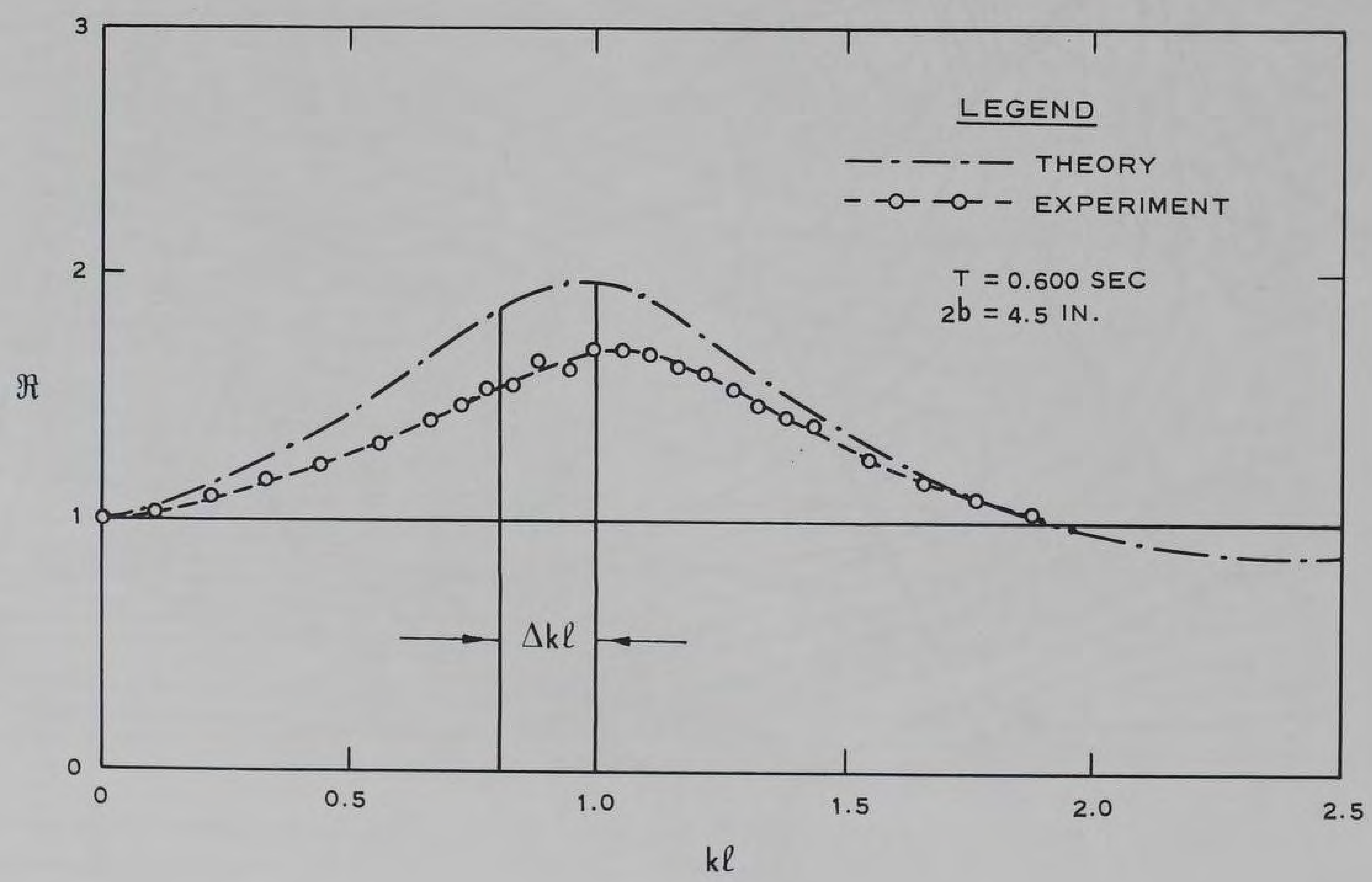


Fig. A7. Typical geometry responses of fully open harbors (after Ippen[14])

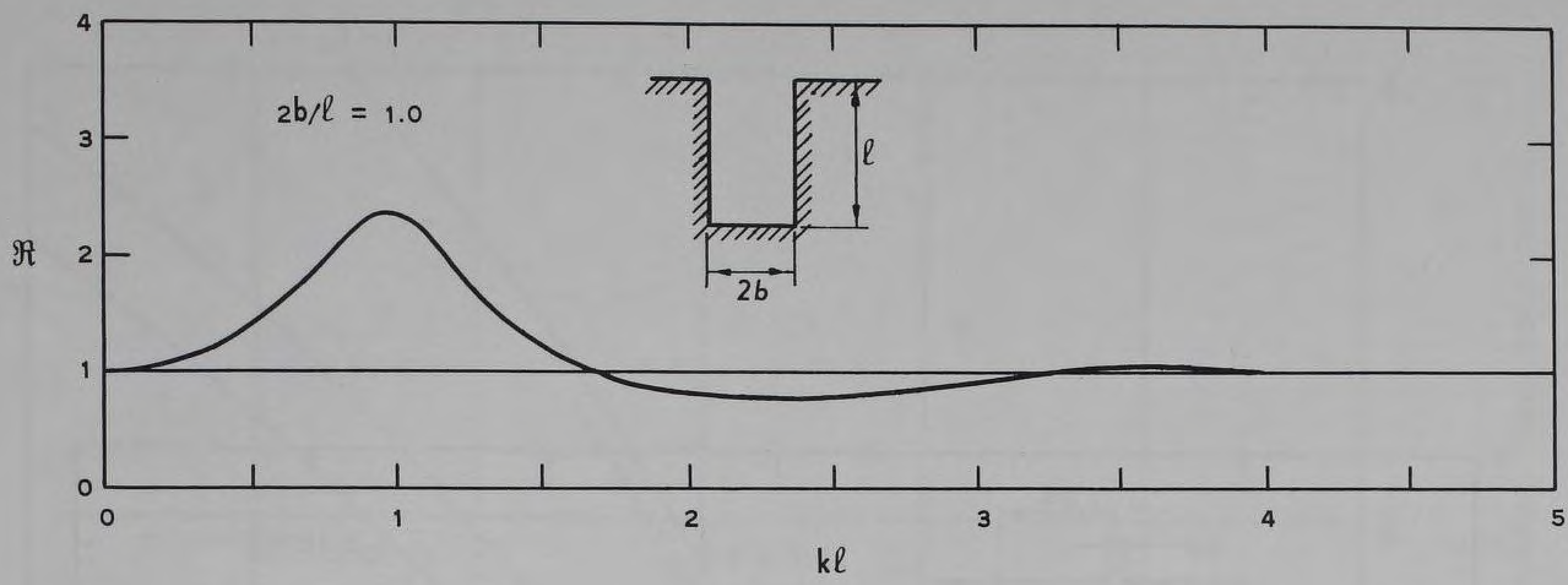


Fig. A8. Frequency-response curves of fully open harbors (after Ippen[14])

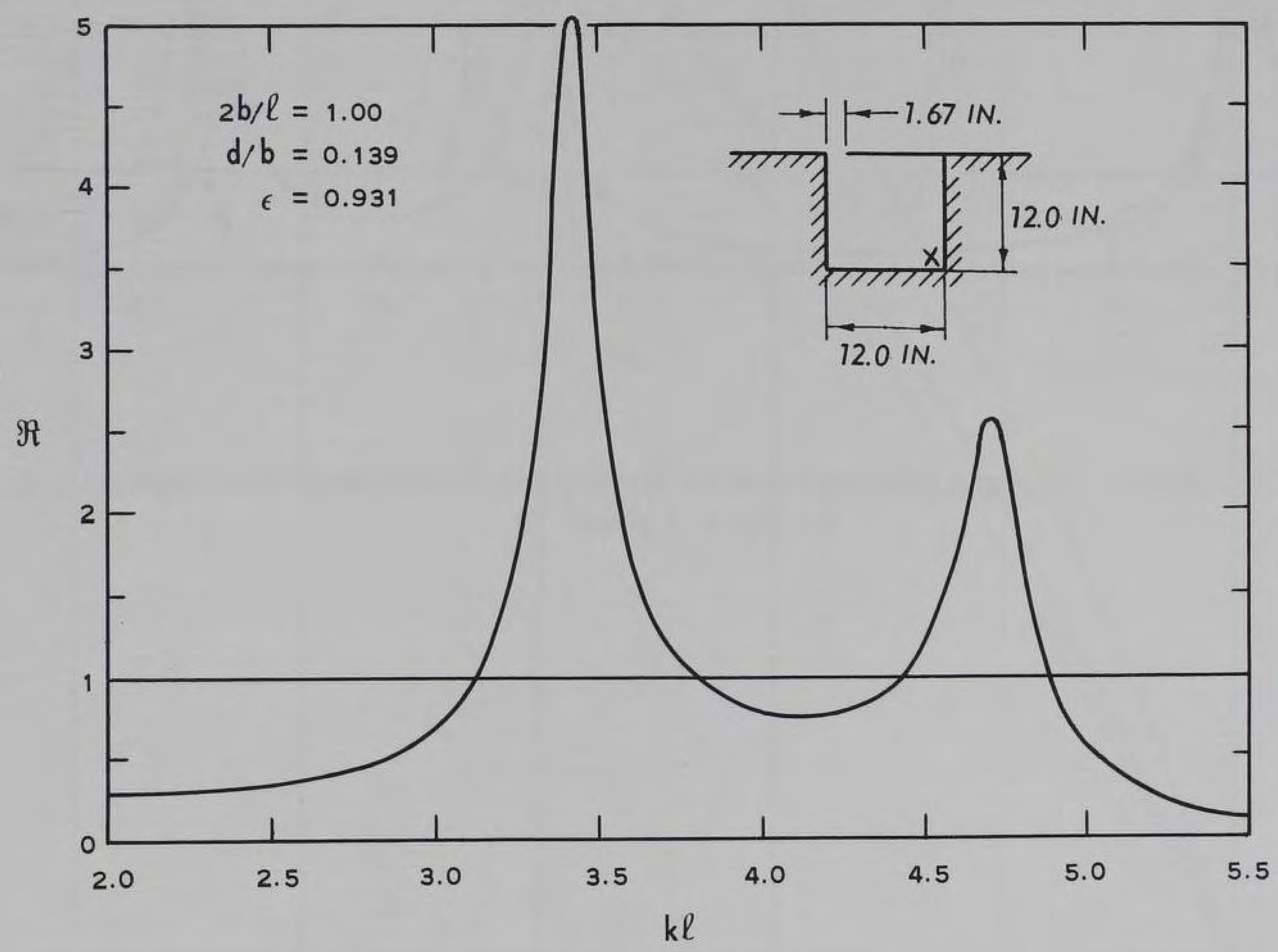


Fig. A9. Frequency response of an asymmetric harbor (after Ippen[14])

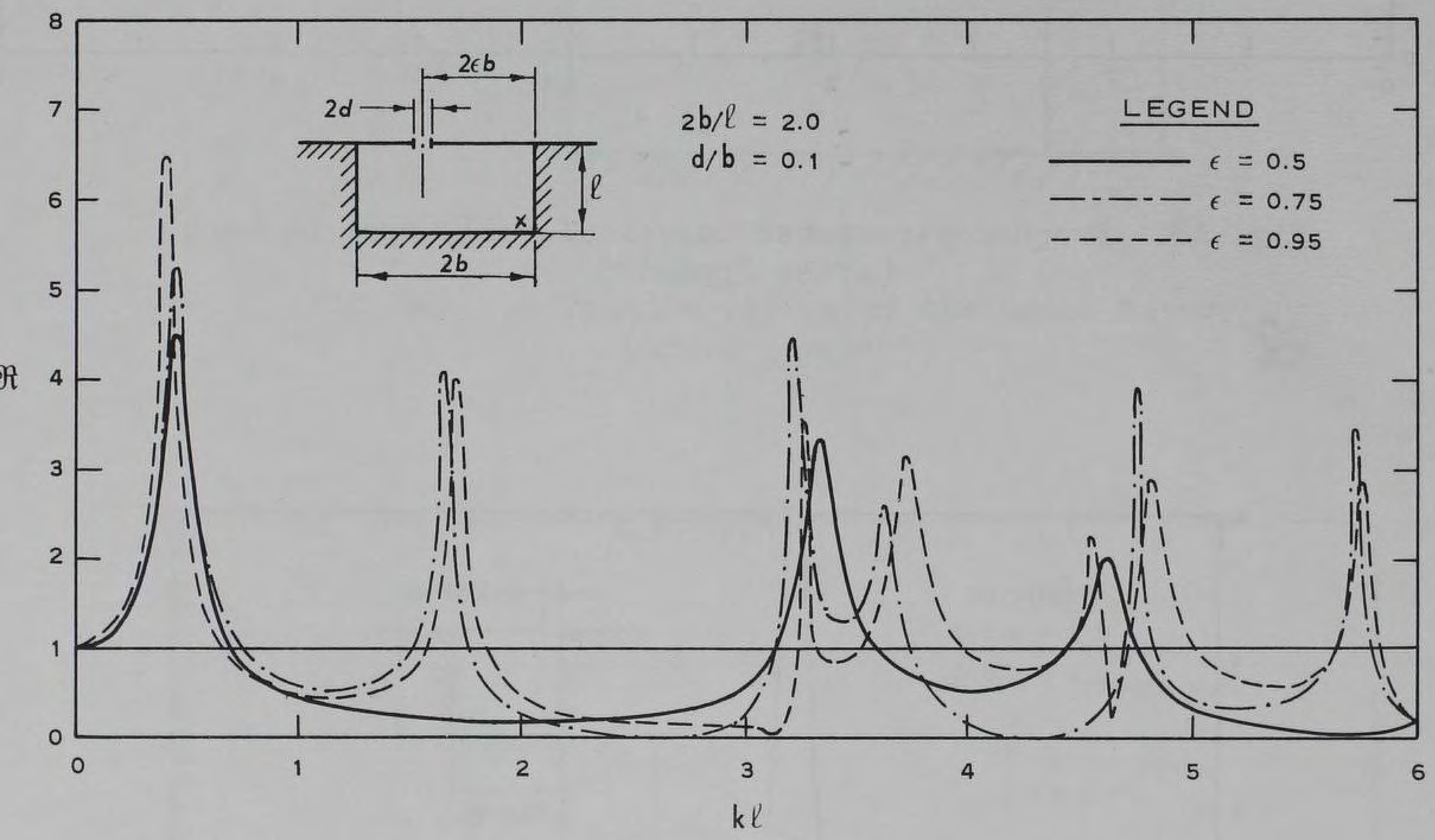


Fig. A10. Frequency-response curves of asymmetric harbors (after Ippen[14])

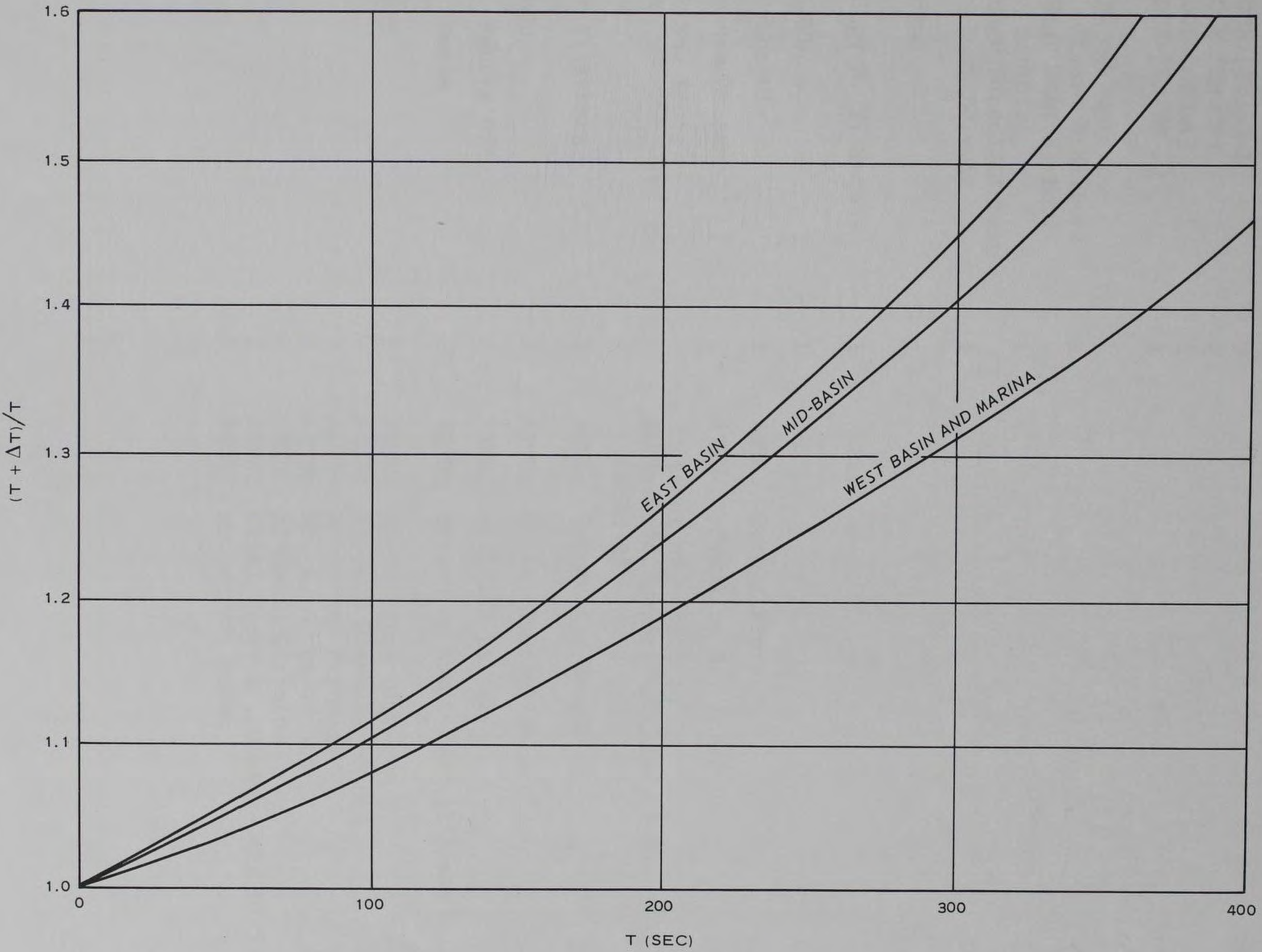


Fig. A11. Relation between test wave periods and their corresponding time increments

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13. ABSTRACT A 1:120-scale model of Monterey Harbor, California, and sufficient offshore area to permit generation of the required test waves was used to investigate the arrangement and design of certain proposed harbor improvements with respect to wave and surge action and to determine current conditions in the navigation entrances to the harbor and its basins. The proposed harbor improvements consisted of (a) enlarging the present harbor by construction of a detached north breakwater, approximately 3350 ft in length, and a companion east breakwater connected to shore and extending approximately 1100 ft seaward, and (b) development of the inner-harbor area by constructing moles to form two additional basins for the anchorage of small pleasure craft. A 56-ft-long wave machine and electrical wave height measuring and recording apparatus were utilized in model operation. Base tests were conducted with existing prototype conditions installed in the model. Results of tests involving the various improvement plans were compared with base test results to determine the relative effectiveness of the various plans. It was concluded from the test results that (a) either the single-entrance or the double-entrance plan will provide an improvement over existing conditions with respect to long-period surge in the harbor; (b) although the harbor basins respond to several of the wave periods tested, no serious cases of resonance were noted; and (c) either the single-entrance or the double-entrance plan will provide sufficient protection to the inner basins from short-period (5 to 20 sec) waves, except in a portion of the east basin. An analytical study of long-period sea-energy oscillations in the vicinity of Monterey Bay with respect to the possibility of related response in Monterey Harbor was conducted, and the results of that study are presented in Appendix A.

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