TECHNICAL REPORT H-73-13

WAVE ACTION AND BREAKWATER DESIGN HAMLIN BEACH HARBOR, NEW YORK

Hydraulic Model Investigation

by

C. W. Brasfeild

August 1973

Sponsored by U. S. Army Engineer District, Buffalo

Conducted by U. S. Army Engineer Waterways Experiment Station
Hydraulics Laboratory
Vicksburg, Mississippi

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ARMY-MRC VICKSBURG, MISS.

FOREWORD

The request for the model investigation reported herein was initiated by the District Engineer, U. S. Army Engineer District, Buffalo (NCB), in a letter to the Division Engineer, U. S. Army Engineer Division, North Central, dated 30 March 1971. Authorization for the U. S. Army Engineer Waterways Experiment Station (WES), Vicksburg, Mississippi, to perform the study was granted by the Office, Chief of Engineers (OCE), in the second indorsement to that letter, dated 12 April 1971.

Model construction was completed in March 1972, and tests were conducted intermittently during the period April 1972-January 1973. During the study, liaison was maintained between NCB and WES by means of conferences, progress reports, and telephone conversations.

Mr. R. S. Goodno and LT Gary Ritchie of NCB visited WES during the course of the study to observe model performance and attend a conference.

The investigation was conducted in the Hydraulics Laboratory at WES under the direction of Mr. H. B. Simmons, Chief of the Hydraulics Laboratory, Mr. R. Y. Hudson and Dr. R. W. Whalin, successive Chiefs of the Wave Dynamics Division, and Mr. C. E. Chatham, Jr., Chief of the Harbor Wave Action Branch. The model tests were conducted by Mr. L. A. Barnes, under the supervision of Mr. C. W. Brasfeild, Project Engineer. The main text of this report was prepared by Mr. Brasfeild. Appendix A was prepared by Mr. Chatham.

BG E. D. Peixotto, CE, and COL G. H. Hilt, CE, were Directors of WES during the conduct of the investigation and the preparation and publication of this report. Mr. F. R. Brown 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
miles (U. S. statute)	1.609344	kilometers
feet	0.3048	meters
square feet	0.092903	square meters
square miles	2.58999	square kilometers
inches	25.4	millimeters

SUMMARY

The 1968 River and Harbor Act authorized construction of a small-boat harbor at Hamlin Beach State Park on Lake Ontario in Monroe County, New York. On the basis of experience at other locations, it was deemed advisable to conduct a hydraulic model investigation of the proposed facilities to determine the most economical breakwater arrangement consistent with the provision of satisfactory entrance conditions and adequate protection to moored boats within the harbor.

The 1:64-scale model was molded in cement mortar and reproduced approximately 3000 ft of the Lake Ontario shoreline on each side of the harbor entrance, the entrance channel, the harbor (approximately 500 by 1150 ft), and sufficient underwater contoured area lakeward of the harbor to permit accurate simulation of storm wave action. A 50-ft-long wave machine and electrical wave height measuring and recording apparatus were utilized in model operation.

It was concluded that the breakwater proposed for the west side of the entrance channel could be reduced in length by 100 ft without sacrificing the full protection desired for the entrance and inner harbor and that a reduction in length of another 100 ft would not seriously impair the desired protection.

WAVE ACTION AND BREAKWATER DESIGN, HAMLIN BEACH HARBOR, NEW YORK

Hydraulic Model Investigation

PART I: INTRODUCTION

Description of Project

1. Hamlin Beach Harbor is proposed for construction on the south shore of Lake Ontario, in Hamlin Beach State Park, which lies approximately 17 miles* northwest of Rochester, New York (plate 1). The harbor will be located in a low, marshy area through which Yanty Creek meanders to the lake.** The authorized improvements consist primarily of breakwaters in Lake Ontario to protect the harbor entrance, an entrance channel, and an interior channel along the berthing area. The cooperating agency will provide berthing piers, a launching ramp, service facilities, and other onshore facilities. Details of the proposed improvements are shown in plate 2. There had been no development at the site prior to inception of the model study.

The Problem

2. The shoreline area proposed for location of the project is subjected in varying degrees to storm-generated waves approaching from directions ranging clockwise from about west-northwest to east. These storm waves, which range up to 10 ft in height, make navigation difficult and dangerous for small craft near shore and can cause serious damage to boats moored inside a harbor unless adequate protection is furnished for the harbor entrance and the interior basin.

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

** Appendix A presents a wave refraction analysis for a possible alternate harbor location.

Purpose of Model Study

3. The purpose of the model study was to determine the optimum length of the protective structures proposed for the entrance to the basin with respect to economics and the reduction of incident wave heights to acceptable levels in the entrance channel and the interior harbor area.

Wave Height Criteria

4. Completely reliable criteria have not yet been developed for ensuring that satisfactory navigation and mooring conditions will be obtained in small-craft harbors during attack by short-period waves. However, based on experience, the U. S. Army Engineer District, Buffalo (NCB), specified that, for an improvement plan to be acceptable, maximum waves should not exceed those indicated below for the various areas of concern:

Location	Maximum Acceptable Wave Height, ft
Harbor entrance	2.5
200-ft-wide dock channel and maneuvering area along east side of basin	1.0
Berthing area (west side of basin)	0.5

PART II: THE MODEL

Design of Model

5. The Hamlin Beach Harbor model was constructed to an undistorted linear scale of 1:64, model to prototype. Selection of this scale was based on such factors as:

- a. Depth of water required in the model to minimize excessive bottom friction effects.
- b. Absolute size of model waves.
- c. Dimensions of the available shelter and the area required for the model.
- d. Efficiency of model operation.
- e. Characteristics of required wave-generating and wave-measuring equipment.
- f. Cost of model construction.

A geometrically undistorted model ensured accurate reproduction of wave patterns and heights in direct proportion to prototype values. After selection of the linear scale, the model was designed and operated in accordance with Froude's model law.[1] The scale relations used for design and operation of the model were as follows:

Characteristic	Dimension*	Scale Relation (Model:Prototype)
Length	L	$L_r = 1:64$
Area	L^2	$A_r = L_r^2 = 1:4,096$
Volume	L^3	$\forall_r = L_r^3 = 1:262,144$
Time	T	$T_r = L_r^{1/2} = 1:8$
Velocity	L/T	$V_r = L_r^{1/2} = 1:8$

* Dimensions are in terms of length and time.

Description of Model and Appurtenances

6. The model was molded in cement mortar and reproduced

approximately 3000 ft of the Lake Ontario shoreline on each side of the harbor entrance, the entrance channel, the interior harbor, and sufficient underwater contoured area lakeward of the harbor to permit generation of waves and wave-front patterns from all significant directions of wave approach to the harbor (plate 1). Vertical control in model construction and operation was based on the low-water datum (lwd)* for Lake Ontario, which is 242.8 ft above mean water level (mwl) at Father Point, Quebec (International Great Lakes Datum (IGLD), 1955). Horizontal control was referenced to coordinates of the New York State Plane Coordinate System, West Zone. Lake-bottom contours were reproduced to a prototype depth of 25 ft, and a sloped transition extended downward from the contoured area to the wave machine pit, which was at el -50. The entire area of the model was approximately 10,600 sq ft, representing nearly 1.6 square miles in the prototype. About half of this area was contoured, the other half being the wave machine pit area. Photo 1 shows a general view of the model prior to construction of the protective breakwaters.

7. Model waves were generated to scale by a 50-ft-long wave machine with a trapezoidal-shaped, vertical-motion plunger. The vertical movement of the plunger caused a periodic displacement of water incident to this motion. The length of the plunger stroke and the period of vertical motion were infinitely variable over the ranges necessary to generate waves with the required characteristics. The wave machine was mounted on retractable casters that enabled it to be positioned to generate waves from the required directions.

8. A 10-channel wave height measuring system was used to secure wave height data during actual test operations. Each of the 10 channels consisted of a wave rod to detect the water level, a remote-controlled, motor-driven assembly to raise and lower the wave rod in the water during calibration, and related cable circuitry connecting the wave rod to a power supply and to a 10-channel, light-beam oscillograph recorder. The resistance-type wave rods consisted of two 0.08-in.-diam parallel wires

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* All elevations (el) cited herein are in feet referred to the lwd.

which formed the legs of an electrical circuit that was closed when the wave rods were partially submerged. During calibration of the system, which was performed in still water prior to each series of test runs (usually about twice daily), each wave rod was moved up and down in the water by the remote-controlled, motor-driven rod assembly, and the resulting light-beam deflection was recorded for calibration data. Vertical travel of each rod was monitored by a linear potentiometer in the mechanical linkage from which the rod was suspended. The travel distance of each potentiometer, which was used as the calibration standard for the attached wave rod, was accurately measured and did not vary throughout the testing period. A capability of doubling the amplitude of the deflection of the light beam was incorporated to facilitate data reduction of the smaller amplitude waves in the more protected areas of the model. In all cases, the wave-trace deflections were directly proportional to the submergence of the respective wave rods in the water. All of the controls for calibration of the wave rods, adjustment and activation of the recorder, and control of the wave machine were located in an instrumentation room adjacent to the model.

PART III: TEST CONDITIONS AND PROCEDURES

Selection of Test Conditions

Still-water level

9. Still-water levels (swl) for harbor wave-action models are selected so that the various wave-induced phenomena that are dependent on water depths can be accurately reproduced in the model. These phenomena include wave refraction, overtopping of harbor structures by waves, reflection of wave energy from harbor structures, and transmission of wave energy through porous structures. Some of the more important factors contributing to selection of the optimum model swl are the following:

- a. The maximum amount of wave energy that can reach a given area will ordinarily do so during the period of a severe storm that coincides in time with the highest water level normally experienced in the area.
- b. Severe storms moving onshore are characteristically accompanied by an increase in the normal water level due to wind tide and mass transport, whereas storms moving offshore tend to lower the water level.
- 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 to the various factors contributing to and affected by the swl in the prototype and in view of the tendency toward more conservative results from model investigations, it is desirable that a model swl be selected that closely approximates the higher water stages that normally prevail during severe storms in the prototype. This procedure entails the study of water-level records in the prototype locality, with due attention being given to the higher levels experienced in the area in the past.

10. Under the lake stage regulation plan currently in effect for Lake Ontario, the monthly mean lake level having a frequency of occurrence of about once in 20 yr is 246.7 ft above mwl (IGLD) or at el 3.9. A temporary rise of about 1.3 ft due to wind setup occurs in the vicinity

of Rochester, New York, with approximately annual frequency. The maximum design lake level due to a combination of these two occurrences thus becomes el 5.2.[2] Based on this information and the considerations in the selection of a model water-level stage, a swl at el 5.0 was selected for use in the model study.

Test waves

11. Factors influencing selection of test waves. In planning the test program for a model investigation of harbor wave-action problems, dimensions and directions for the test waves should be selected that will afford a realistic test of the improvement plans proposed, thus permitting the optimum plan of improvement to be accurately determined. Wind waves are generated by the tangential shear force of the wind on the water surface and the normal force of the wind against the wave crests. The height and period of the maximum wave that can be generated by a given storm depend on the wind velocity, the duration for which wind of a given velocity continues to blow, and the water distance (fetch) over which it blows. Factors that influence the selection of test waves include:

- a. The fetch distances in the various directions from which waves can attack the harbor.
- b. The frequency of occurrence and the duration of winds of storm intensity blowing from the various directions.
- c. The width, alignment, and position of the harbor entrance and the reflecting surfaces inside the harbor.
- d. The refraction of waves caused by differentials in depth in the approaches to the harbor, a factor which may create either a concentration or a diffusion of wave energy at the harbor site.

12. Prototype wave data. The area proposed for the location of the entrance to Hamlin Beach Harbor is exposed in varying degrees to storm-generated waves from directions ranging clockwise from about west-northwest to east. As reported in reference 2, measured wave data upon which to base a comprehensive statistical analysis of wave conditions were not available for the area; however, meteorological records were available from which statistical wave hindcast data could be compiled. Wind records from U. S. Coast Guard Stations at Rochester, Youngstown,

and Oswego, New York, were reviewed by personnel of the NCB to determine the severity, duration, and extent of storm occurrences, with more weight being given to data from the Rochester Station, which is nearest the area of concern. Using the data thus compiled, computations were made to establish the characteristics of waves that could be expected in the harbor area. Two methods of wave hindcasting were used in the computations, and the results were compared with each other and with wave characteristics for Lake Ontario that were derived from still another source.[2] Since there was very little difference in the results from the three sources, a tabulation of wave characteristics combining the results of the three sources was used in the wave refraction study.

13. Wave refraction. When wind waves move into water of gradually decreasing depth, transformations take place in all wave characteristics except the 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 wave refraction diagrams and calculating refraction coefficients. For this study, refraction diagrams were prepared by personnel of NCB using the wave characteristics referred to in the preceding paragraph. 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 $(b_o/b)^{1/2}$, derived from the refraction diagram studies, is the refraction coefficient. The shoaling coefficient K, which is a function of wavelength and water depth, can be obtained from tables compiled by Wiegel.[3] Thus, the refraction coefficient multiplied by the shoaling coefficient provides a conversion factor for transferring deepwater wave heights to corresponding shallow-water values that can be used for model testing.

14. Test waves. From the prototype wave studies and the refraction-shoaling analysis and application, test waves were selected for use in the model. Since the wave machine pit area was constructed to el -50, this depth was taken as shallow water for the refraction-shoaling analysis. The following tabulation presents the pertinent characteristics, expressed in prototype dimensions, of the test waves selected for use in the model:

Wave Period T, sec	Wave Height, ft: Deep Water H_o	Wave Height, ft: Shallow Water H	Direction of Wave Approach: Deep Water	Direction of Wave Approach: Shallow Water	Direction of Wave Approach: Used in Model
7	9.0	7.6	N48°30'W	N38°10'W	N39°00'W
7	9.0	8.6	N07°30'E	N07°30'E	N07°30'E
7	9.0	8.3	N44°00'E	N40°30'E	N39°00'E
8	11.0	9.3	N60°00'W	N40°00'W	N39°00'W
8	11.5	9.6	N43°00'E	N37°30'E	N39°00'E
8	11.5	10.2	N87°00'E	N61°30'E	N61°30'E

Model input wave characteristics corresponded to the values given in the first, third, and sixth columns in the tabulation. It can be seen that, in two cases, the shallow-water directions of approach for two test waves were close enough to permit selection of an average direction for the wave machine position, thereby reducing the time involved in testing.

Test Data Obtained

15. Data obtained during the testing program included wave height measurements at several selected locations in and near the approach channel and throughout the inner harbor area, photos of wave-front patterns for all test waves, and visual observations of model action. The wave height gage locations selected for use in the investigation are shown in the plates, which are referred to in Part IV of this report, depicting the various model configurations. Wave heights measured in the model were corrected to compensate for the increased rate at which bottom friction attenuates waves in the model as compared with the prototype by applying Keulegan's attenuation equation[4] to calculate the correctional coefficients.

PART IV: PLANS TESTED AND TEST RESULTS

Description of Plans

16. Tests were conducted for four model configurations, which were designated the base test and plans 1, 2, and 3. Each of the four plans was subjected to the test waves described in paragraph 14. A description of the plans tested is presented in the following tabulation:

Designation	Physical Description of Model
Base test (plate 3)	Model constructed according to the proposed design of the harbor. Both east and west revetments were installed, but no breakwater structures were installed.
Plan 1 (plate 4)	Similar to the base test, except both east and west breakwaters were installed as proposed. Total length of east breakwater was 270 ft; total length of west breakwater was 950 ft.
Plan 2 (plate 5)	Similar to plan 1, except that the length of the lakeward arm of the west breakwater was reduced by 100 ft.
Plan 3 (plate 6)	Similar to plan 2, except that an additional 100 ft was removed from the lakeward arm of the west breakwater.

Results of Tests

17. The test results are presented in table 1 in the form of wave heights measured at various locations in the harbor area for the model configurations tested. To supplement the measured data, views of wave-action patterns that occurred during model testing are presented in photos 2-7.

Discussion of Test Results

18. Model testing of the base test configuration (no breakwaters installed, plate 3) was conducted to determine the wave regime for the

unprotected harbor. The results of these tests showed that waves from the test direction of N39°00'W caused higher waves in the harbor basin than those from any of the other test directions, thereby confirming the proposed orientation of the breakwaters. This configuration resulted in the entrance channel receiving the maximum protection for waves from the northwest direction.

19. The wave height criteria specified in paragraph 4 are repeated here for convenience:

Location	Maximum Acceptable Wave Height, ft
Harbor entrance	2.5
200-ft-wide dock channel and maneuvering area along east side of basin	1.0
Berthing area (west side of basin)	0.5

The results of tests with the breakwaters installed as originally proposed (plan 1, plate 4) revealed that resulting wave heights in the basin did not exceed 0.5 ft except at the gage 6 location in the docking channel, where a maximum height of 0.8 ft was observed. In the harbor entrance, the maximum wave height noted was 1.9 ft. Thus, the plan 1 configuration apparently more than satisfied the specified wave height criteria. Therefore, additional testing was directed toward the reduction in length of the west breakwater in an attempt to balance the economics of construction with the desired degree of protection to the harbor. Due to the relatively short length of the east breakwater and based on visual observations of wave action in the area, it was not considered feasible to reduce the length of the east breakwater.

20. For plan 2, a 100-ft (prototype) length was removed from the outer end of the lakeward arm of the west breakwater (plate 5). Results of the ensuing tests indicated that the plan 2 configuration also satisfied the specified wave height criteria except in one case when waves at the gage 10 location reached a maximum height of 0.6 ft. However, because the location of gage 10 was near the center of the basin and because the criterion was exceeded by only 0.1 ft, it appeared that plan 2 could also be considered as satisfactory.

21. An inspection of the plan 2 data in table 1 reveals an apparent anomaly in the wave height values. With 8-sec, 9.3-ft waves approaching from the N39°00'W direction, the wave height at the gage 1 location is given as 14.6 ft; however, the still-water depth at that location was equivalent to about 14.0 ft, which should not, theoretically, sustain a wave as high as that reported. A thorough check of the data acquisition and analysis procedures revealed no error therein; however, model observations and a close inspection of the photo of the wave conditions involved (photo 3c) showed that there was a strong convergence of wave energy immediately lakeward of the gage 1 location that resulted in abnormally high peaking waves passing directly through the gage location before and during breaking. A small black circle in photo 3c shows the location of gage 1.

22. An additional 100-ft (prototype) length was removed from the outer end of the west breakwater (plan 3, plate 6). Results of the plan 3 tests revealed several instances in which the wave heights exceeded the criteria specified for the berthing area; however, wave heights in the harbor entrance and docking channel did not vary substantially from those obtained with plan 2 installed in the model.

23. From a review of the model test results, it can be seen that plan 1 apparently offers more protection than needed, plan 2 appears to be the optimum plan tested, and plan 3 does not seem to afford the desired protection. However, when the wave heights presented herein for the three breakwater plans tested are examined closely, it can be seen that test waves approaching from the easterly directions resulted in considerably higher waves than did the test waves from the north and west directions. These observations suggest that further study of the magnitude and estimated durations of storm waves that can be expected from the easterly directions could possibly be beneficial in determining the actual length of the lakeward arm of the west breakwater necessary to provide the desired protection to the harbor (i.e., if these easterly storms occur infrequently, further consideration might be given to plan 3).

PART V: CONCLUSIONS

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

- a. The location of the proposed Hamlin Beach Harbor is exposed to storm wave attack in varying degrees of severity from all directions clockwise from about west-northwest to east. Resulting wave heights in the harbor area can make navigation difficult and dangerous for small craft near shore and can cause serious damage to boats moored inside the harbor unless adequate protection is furnished for the installation.
- b. The full length of the lakeward arm of the west breakwater as proposed (400 ft, as specified in plan 1) is not necessary to afford the full protection desired for the harbor.
- c. Plan 2, wherein the west breakwater was reduced 100 ft in length at its outer end, will provide adequate protection for the entrance channel and inner harbor.
- d. An additional 100-ft reduction in length of the west breakwater, designated herein as plan 3, will not seriously impair the protection desired for the harbor; however, the specified wave height criteria will be slightly exceeded during storm wave attack from the easterly directions.
- e. Following construction of the protective breakwaters proposed for the harbor (either plan 1, 2, or 3), the most severe wave action that will occur in the harbor will result from attack on the area by storm waves approaching from the easterly directions.

LITERATURE CITED

1. Committee of the Hydraulics Division on Hydraulic Research, "Hydraulic Models," Manuals of Engineering Practice No. 25, Jul 1942, American Society of Civil Engineers, New York, N. Y.

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3. Wiegel, R. L., "Gravity Waves; Tables of Functions," 1954, Engineering Foundation, Council on Wave Research, Berkeley, Calif.

4. Keulegan, G. H., "The Gradual Damping of a Progressive Oscillatory Wave with Distance in a Prismatic Rectangular Channel" (unpublished report), May 1950, National Bureau of Standards, Washington, D. C.

5. Dobson, R. S., "Some Applications of a Digital Computer to Hydraulic Engineering Problems," M.S. Thesis, 1967, Stanford University, Stanford, Calif.

Table 1

Comparison of Wave Height Data for Base Test and Plans 1, 2, and 3

Wave Gage No.	Wave Heights* at Gage Locations**, ft, for Various Test Conditions†											
	7-sec, 7.6-ft Waves from N39°00'W				8-sec, 9.3-ft Waves from N39°00'W				7-sec, 8.6-ft Waves from N07°30'E			
	Base Test	Plan 1	Plan 2	Plan 3	Base Test	Plan 1	Plan 2	Plan 3	Base Test	Plan 1	Plan 2	Plan 3
1	4.9	5.7	8.1	4.0	11.6	11.0	14.6	11.5	11.0	10.6	10.2	10.6
2	6.5	--	--	--	8.6	--	--	--	6.5	--	--	--
3	4.5	2.1	4.2	2.2	9.8	4.6	6.7	8.5	6.5	6.8	6.3	6.5
3A	--	0.9	2.0	4.4	--	1.7	1.7	3.8	--	2.0	4.5	8.1
4	5.5	0.8	1.1	2.0	5.8	1.2	2.1	2.3	5.0	1.6	2.2	3.2
5	3.6	0.3	0.4	0.3	3.3	0.8	0.4	0.5	3.2	0.8	0.5	0.7
6	3.2	0.3	0.4	0.3	3.0	0.6	0.4	0.3	1.9	0.6	0.7	0.8
7	1.4	0.2	0.2	0.1	1.3	0.3	0.1	0.2	0.8	0.3	0.4	0.3
8	1.0	0.2	0.2	0.1	1.0	0.2	0.2	0.1	0.6	0.3	0.2	0.3
9	2.0	0.2	0.3	0.1	1.0	0.2	0.2	0.1	1.0	0.3	0.4	0.4
10	1.6	0.2	0.2	0.1	1.4	0.3	0.2	0.1	1.0	0.3	0.4	0.4

Wave Gage No.	Wave Heights* at Gage Locations**, ft, for Various Test Conditions†											
	7-sec, 8.3-ft Waves from N39°00'E				8-sec, 9.6-ft Waves from N39°00'E				8-sec, 10.2-ft Waves from N61°30'E			
	Base Test	Plan 1	Plan 2	Plan 3	Base Test	Plan 1	Plan 2	Plan 3	Base Test	Plan 1	Plan 2	Plan 3
1	8.9	11.1	9.8	11.2	10.0	11.9	12.1	12.7	10.0	11.2	10.8	12.2
2	9.5	--	--	--	8.7	--	--	--	8.2	--	--	--
3	10.5	8.2	10.6	9.8	9.1	8.1	9.3	9.5	9.4	6.9	12.6	9.2
3A	--	3.8	5.4	7.7	--	4.3	6.4	7.9	--	3.8	6.0	5.0
4	6.8	1.6	3.2	4.7	7.9	1.8	3.3	4.3	6.8	1.9	5.0	6.6
5	3.6	0.5	0.7	0.9	3.7	1.0	0.9	0.9	3.2	0.5	0.9	1.1
6	1.3	0.5	0.7	1.0	1.8	0.8	0.7	0.7	1.3	0.5	0.9	0.8
7	0.7	0.3	0.3	0.6	0.6	0.5	0.4	0.5	0.7	0.2	0.3	0.6
8	0.4	0.1	0.3	0.4	0.5	0.5	0.4	0.4	0.4	0.2	0.4	0.4
9	0.7	0.3	0.4	0.5	0.7	0.4	0.4	0.4	0.5	0.2	0.4	0.3
10	0.7	0.3	0.4	0.7	1.0	0.5	0.6	0.6	0.7	0.2	0.5	0.4

* Wave heights adjusted in accordance with Keulegan's equation.[4]
** Locations shown in plates 3-6.
† All tests conducted using still-water level of +5.0 ft lwd.

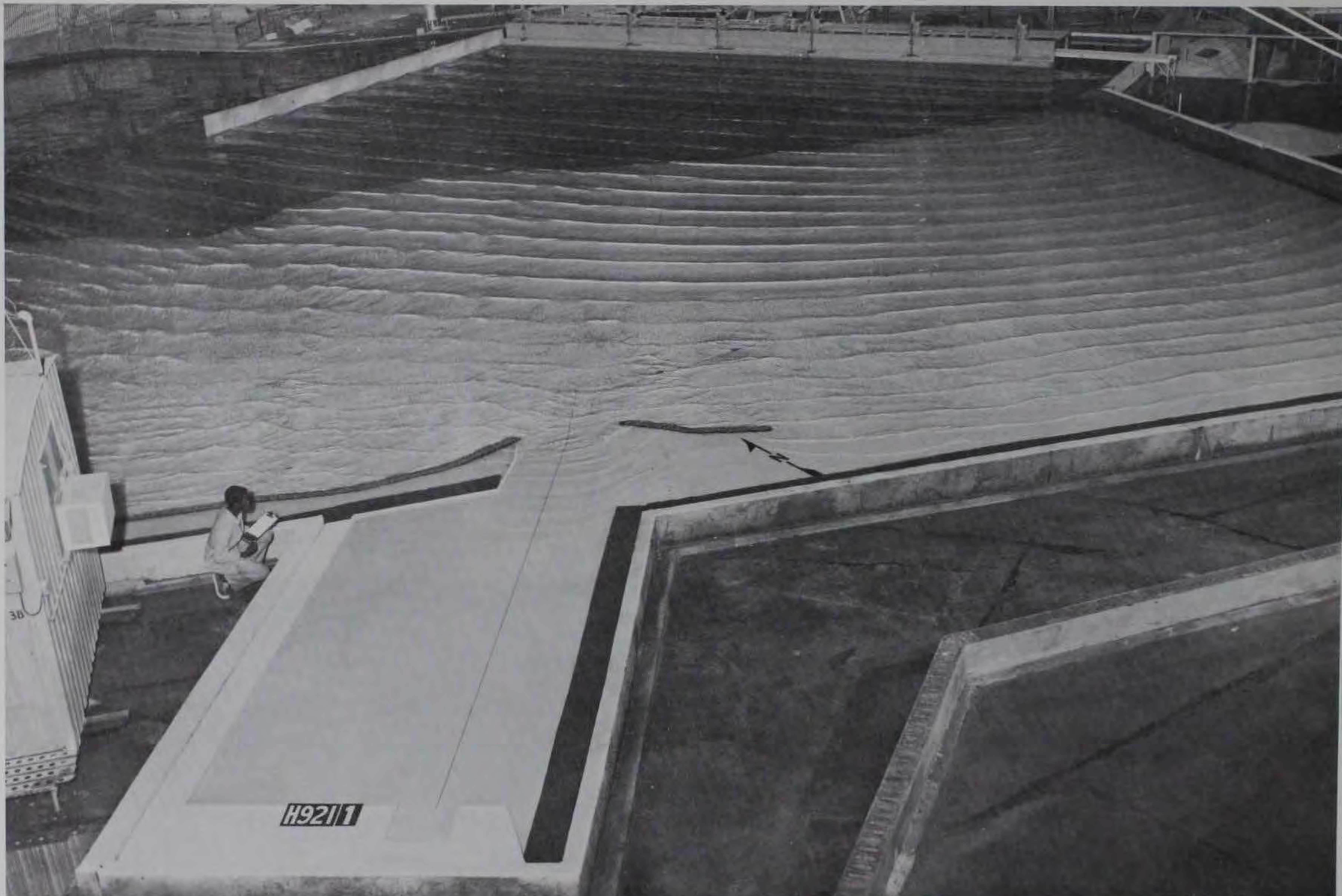
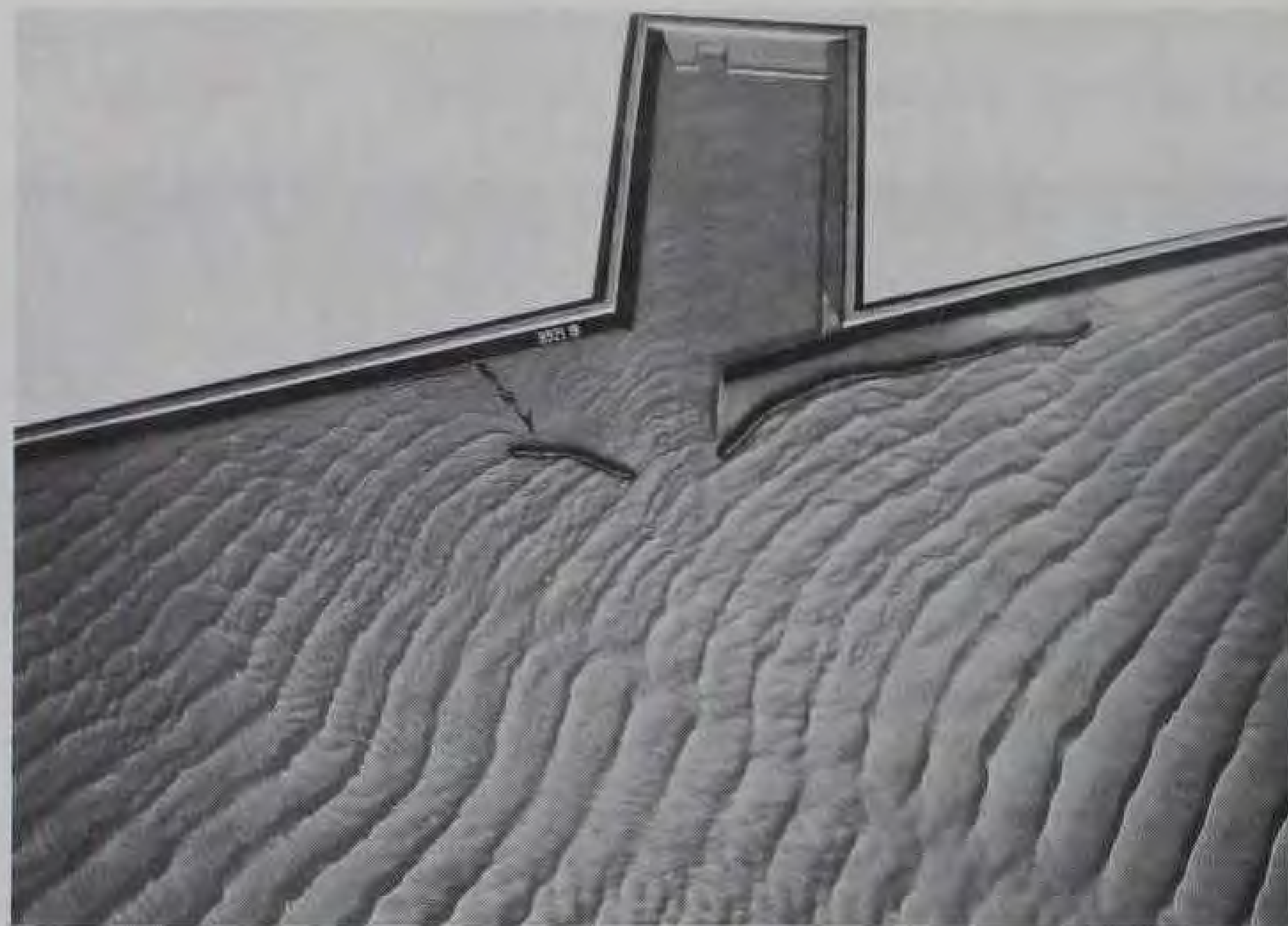


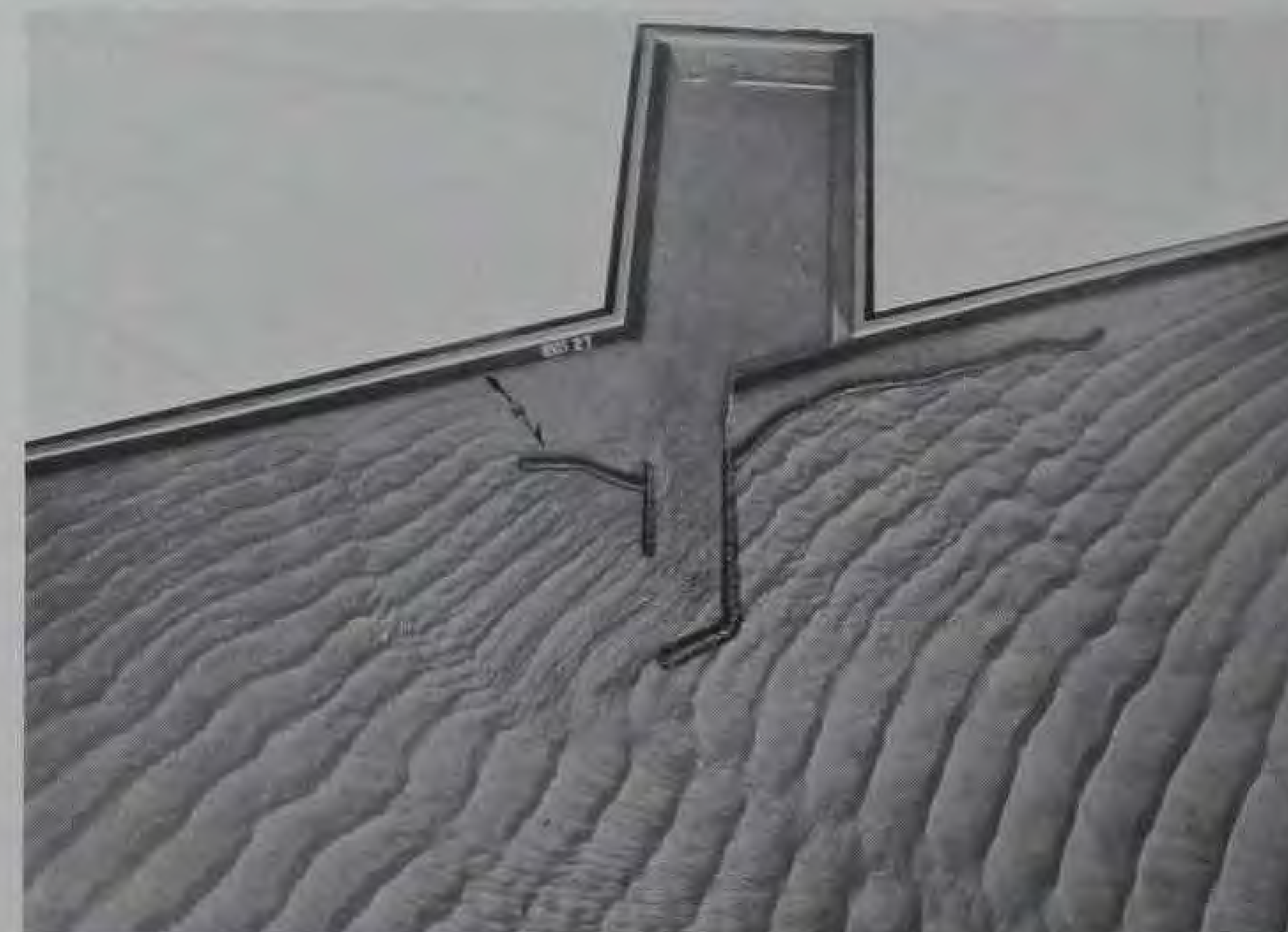
Photo 1. General view of model as originally constructed. Wave machine in background



a. Base Test

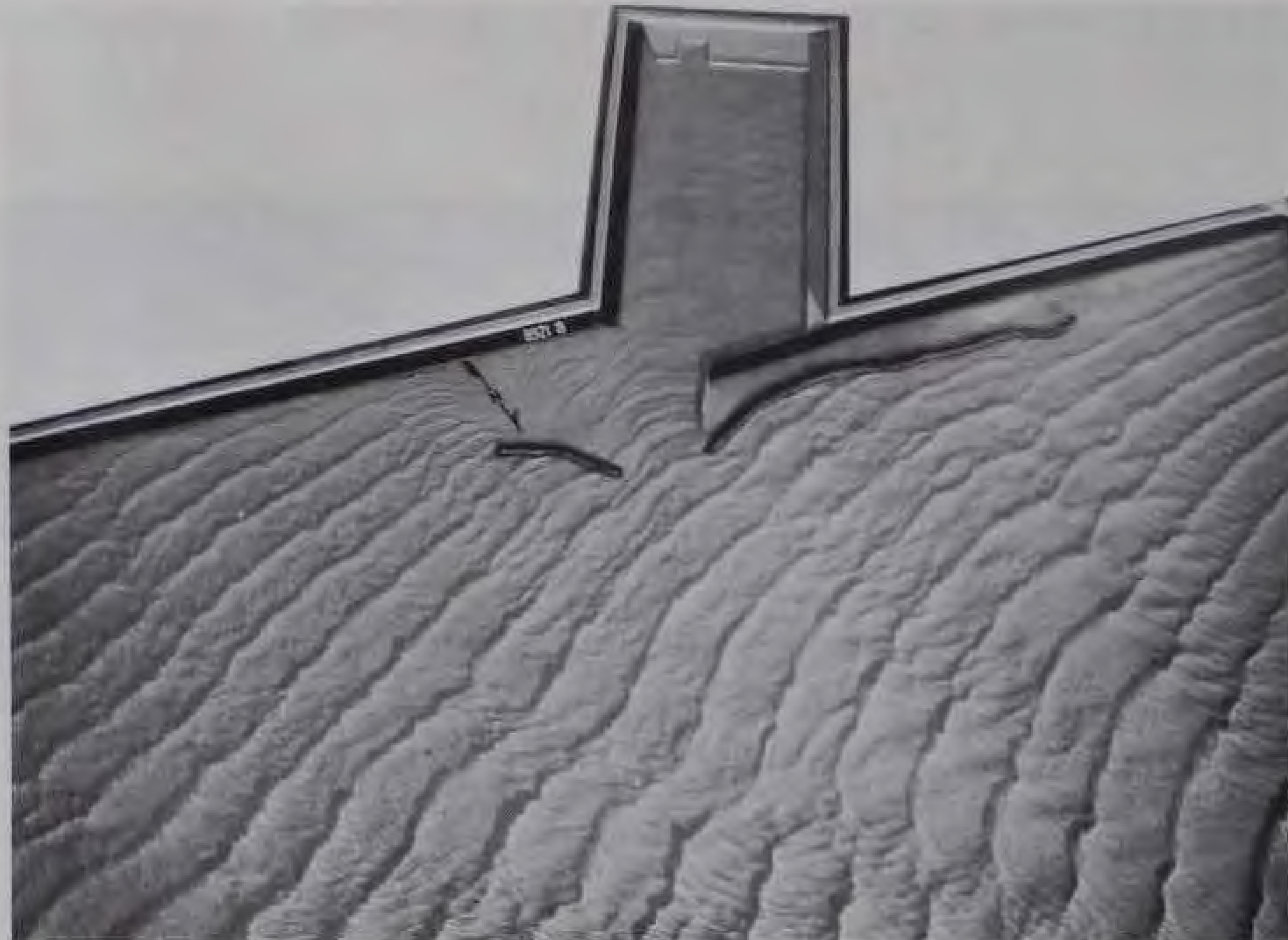
b. Plan 1

c. Plan 2



d. Plan 3

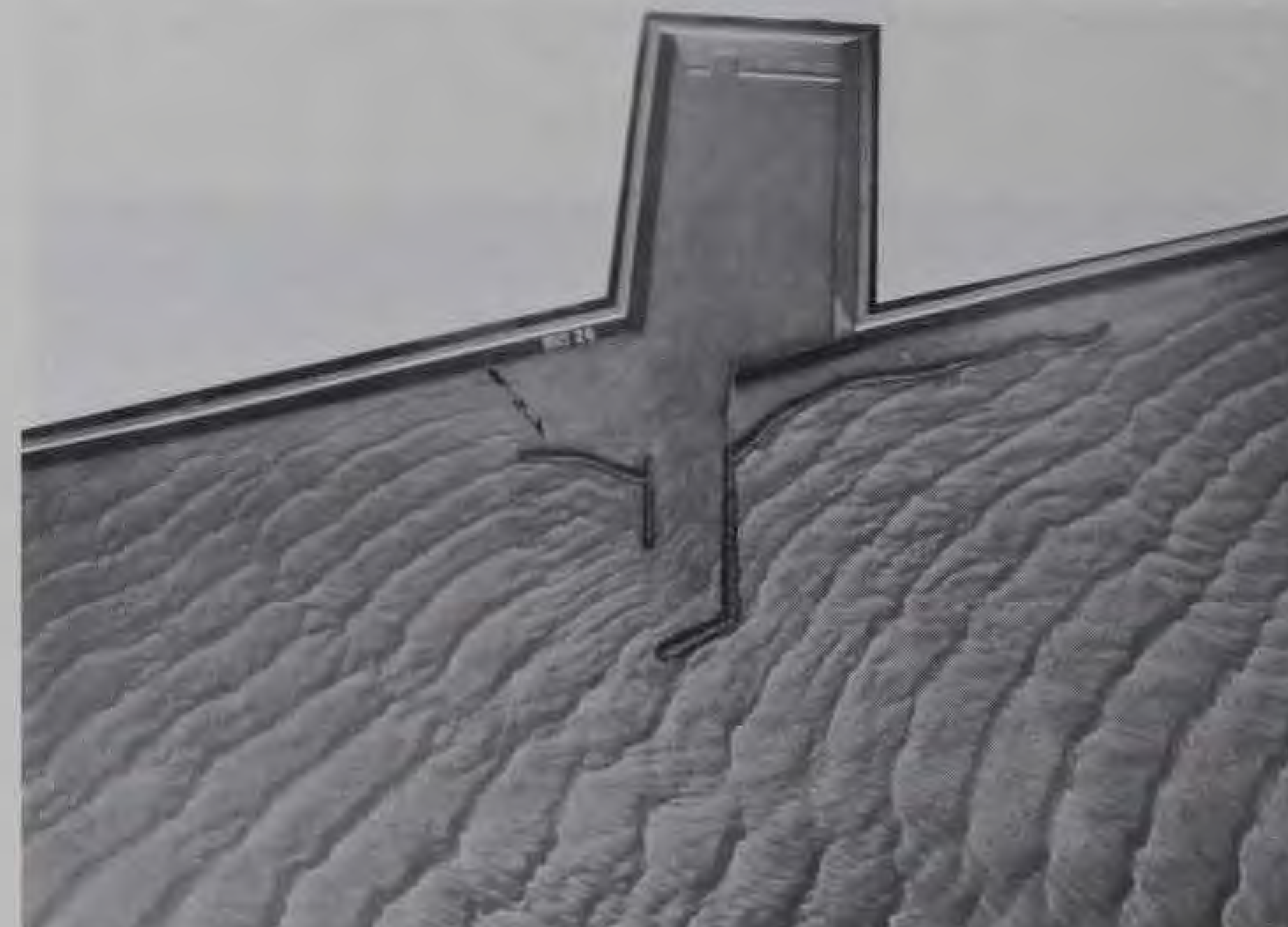
Photo 2. Comparison of wave patterns; 7-sec, 7.6-ft waves from N39°00'W



a. Base test

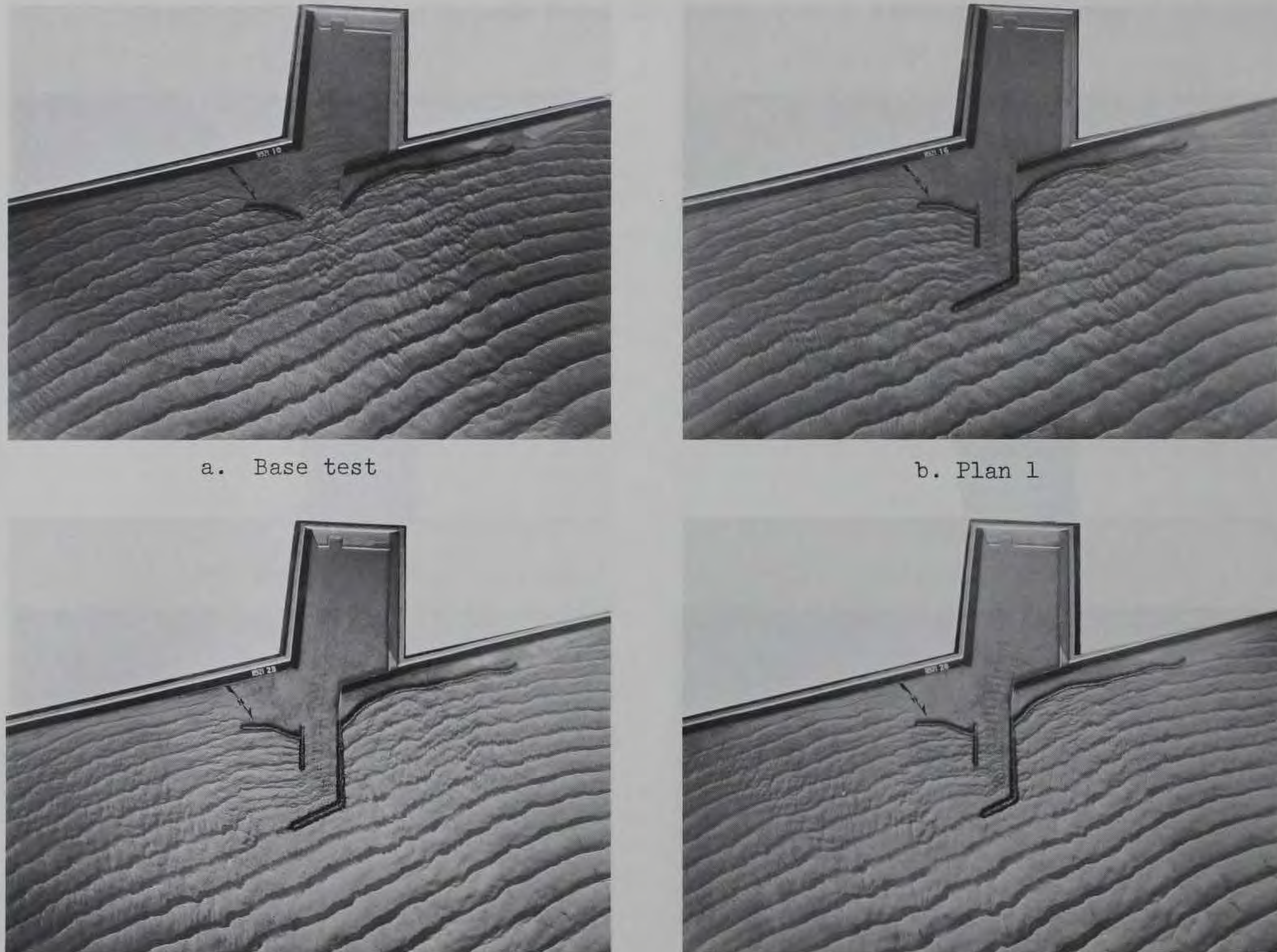
b. Plan 1

c. Plan 2



d. Plan 3

Photo 3. Comparison of wave patterns; 8-sec, 9.3-ft waves from N39°00'W



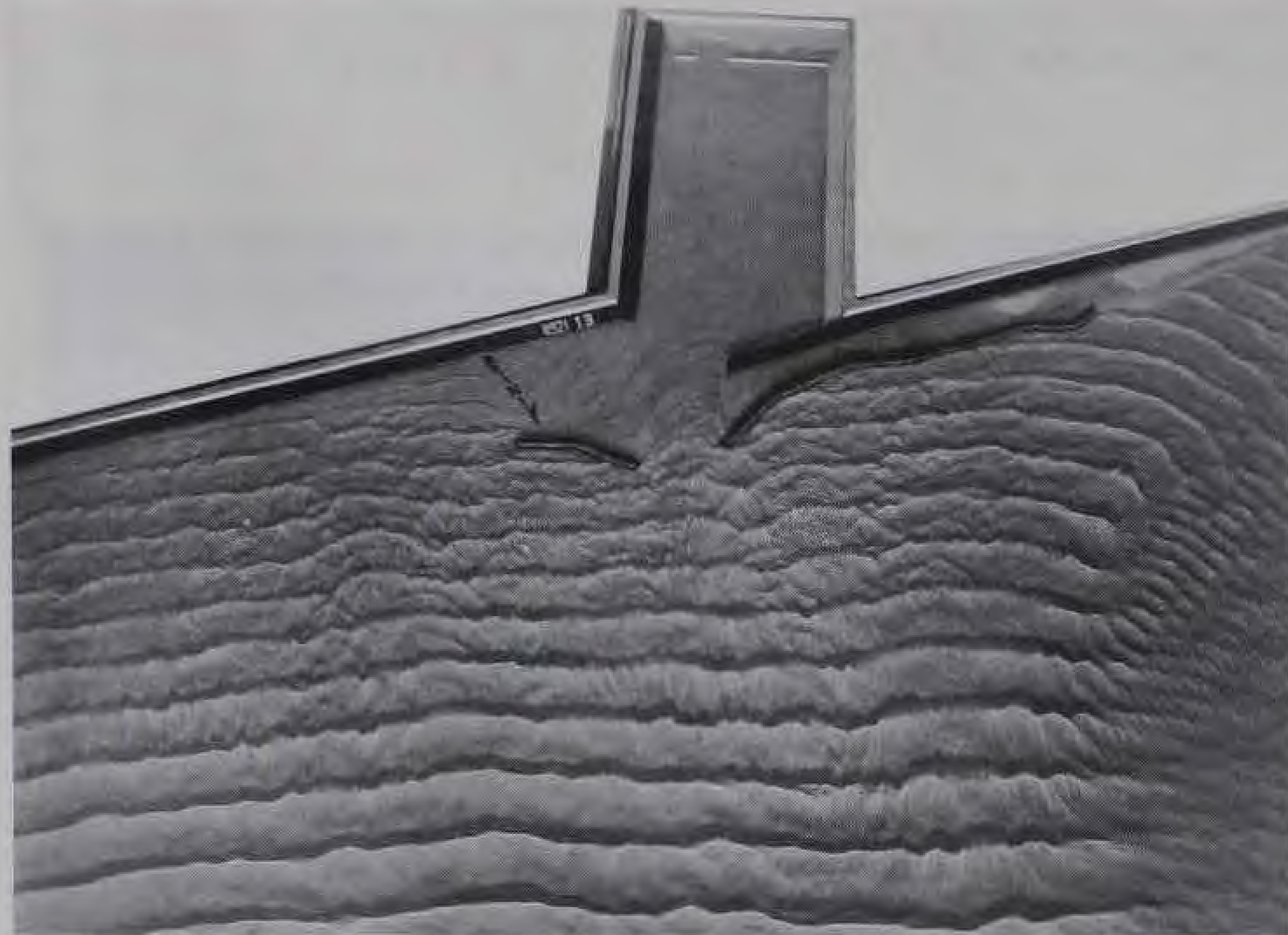
a. Base test

b. Plan 1

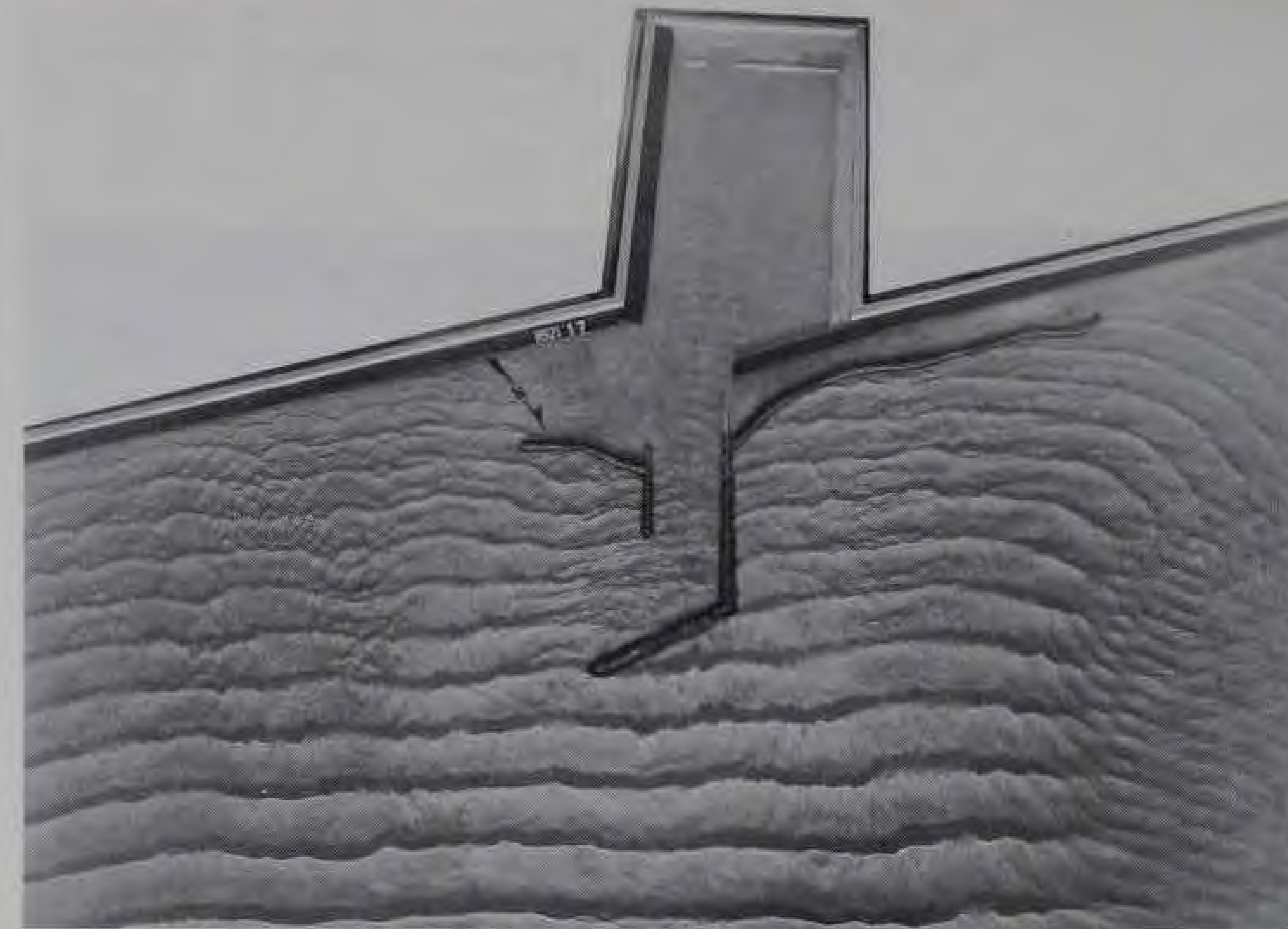
c. Plan 2

d. Plan 3

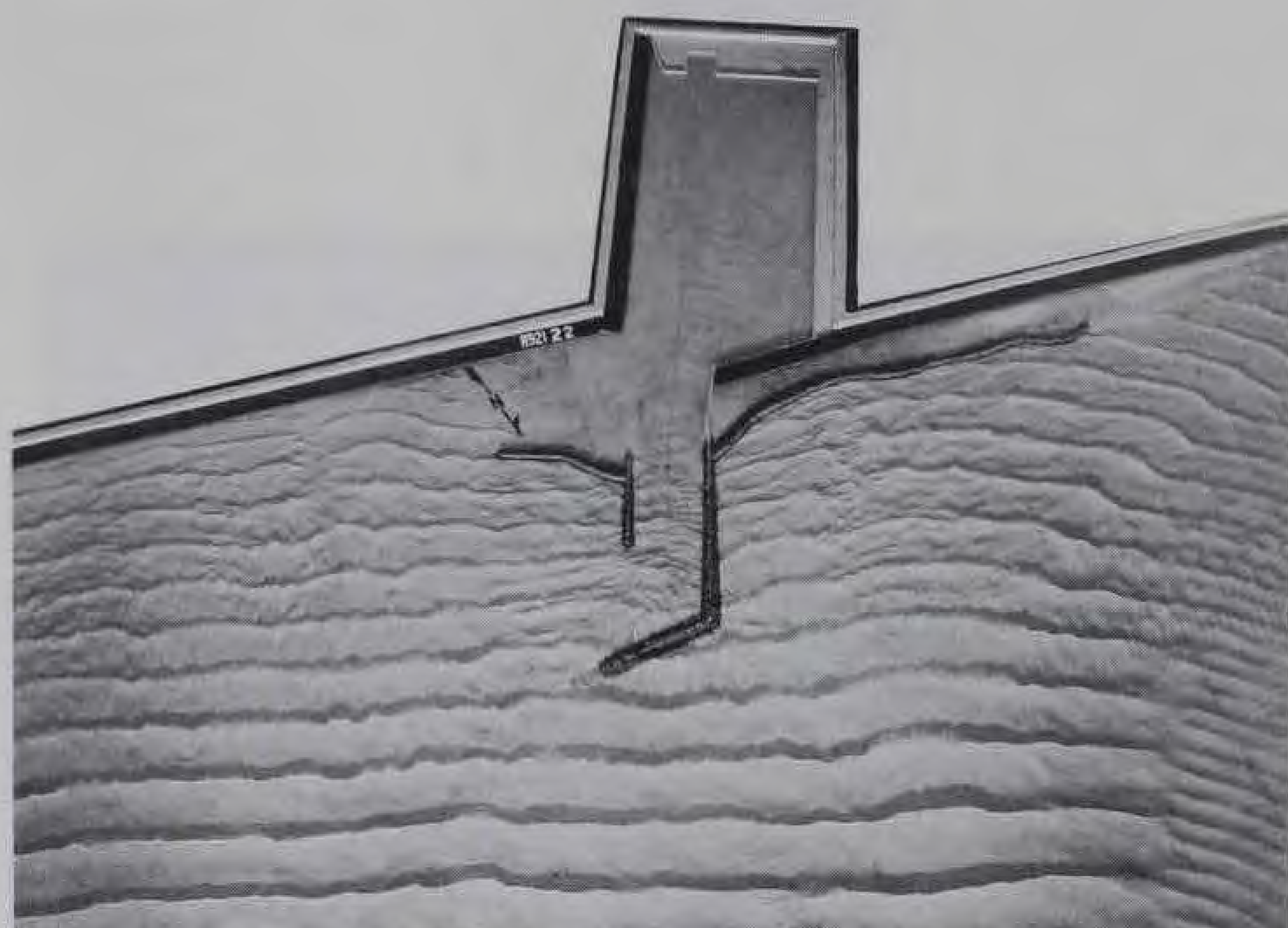
Photo 4. Comparison of wave patterns; 7-sec, 8.6-ft waves from N07°30'E



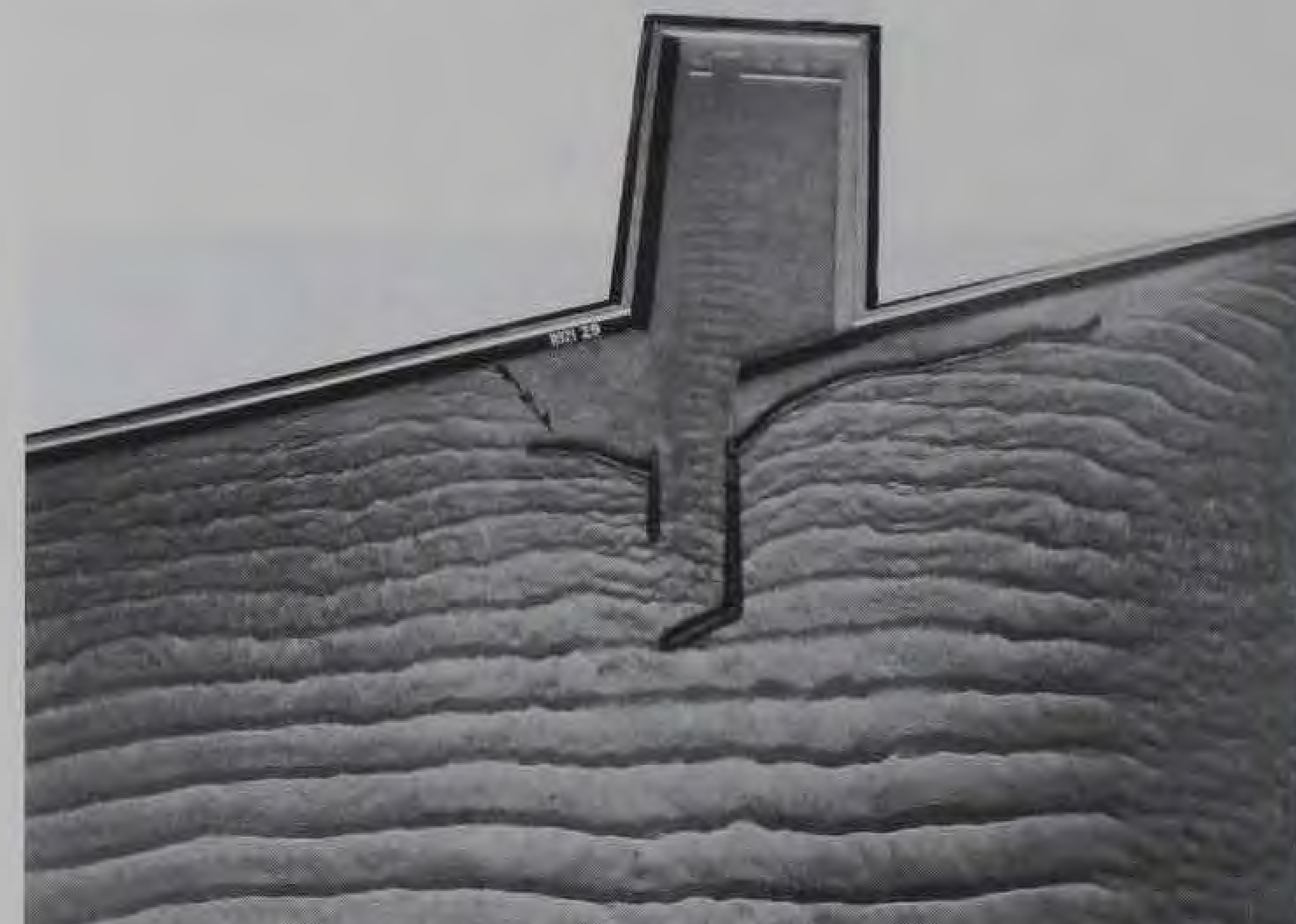
a. Base test



b. Plan 1



c. Plan 2



d. Plan 3

Photo 5. Comparison of wave patterns; 7-sec, 8.3-ft waves from N39°00'E

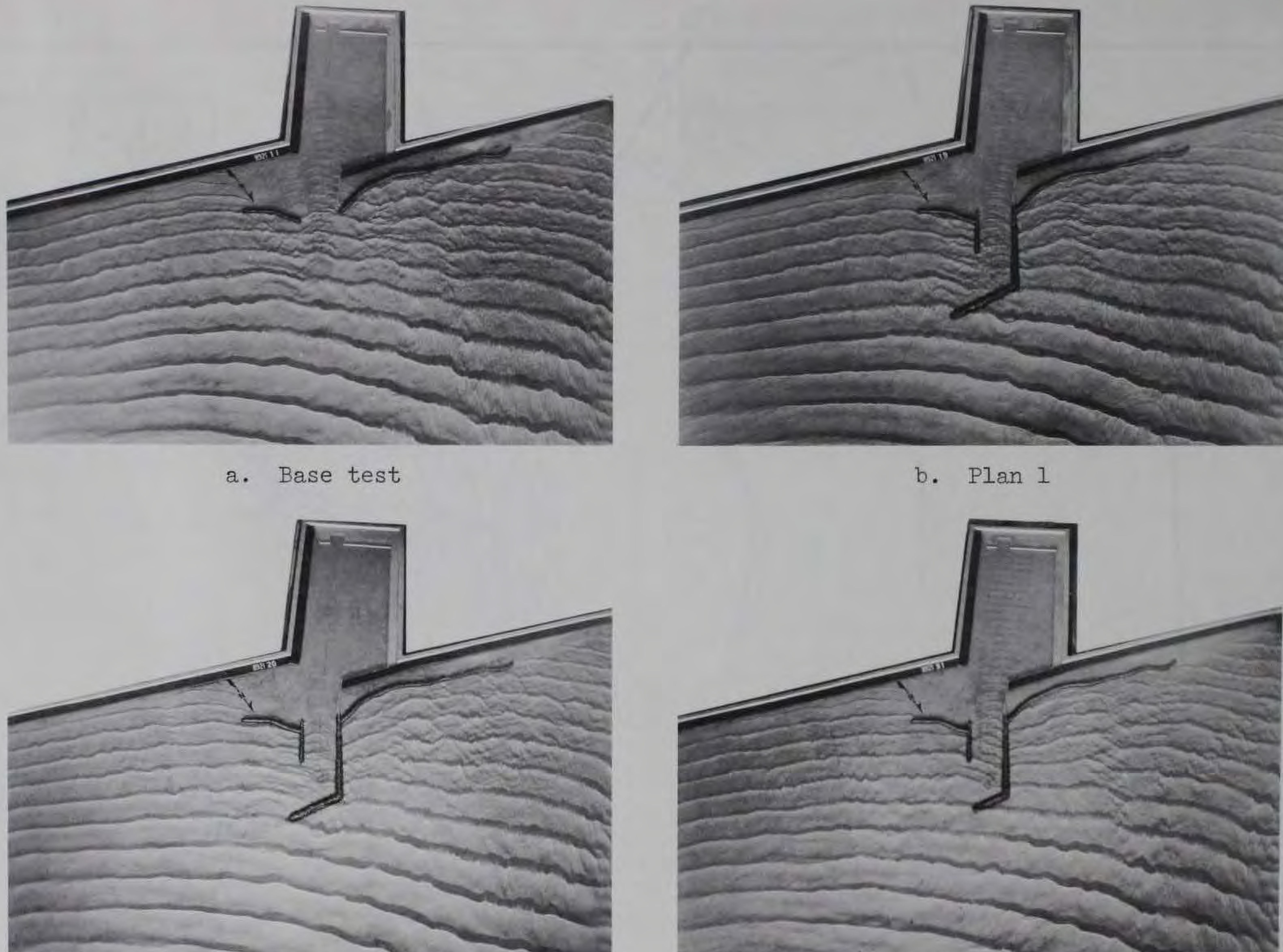
a. Base test

b. Plan 1

c. Plan 2

d. Plan 3

Photo 6. Comparison of wave patterns; 8-sec, 9.6-ft waves from N39°00'E



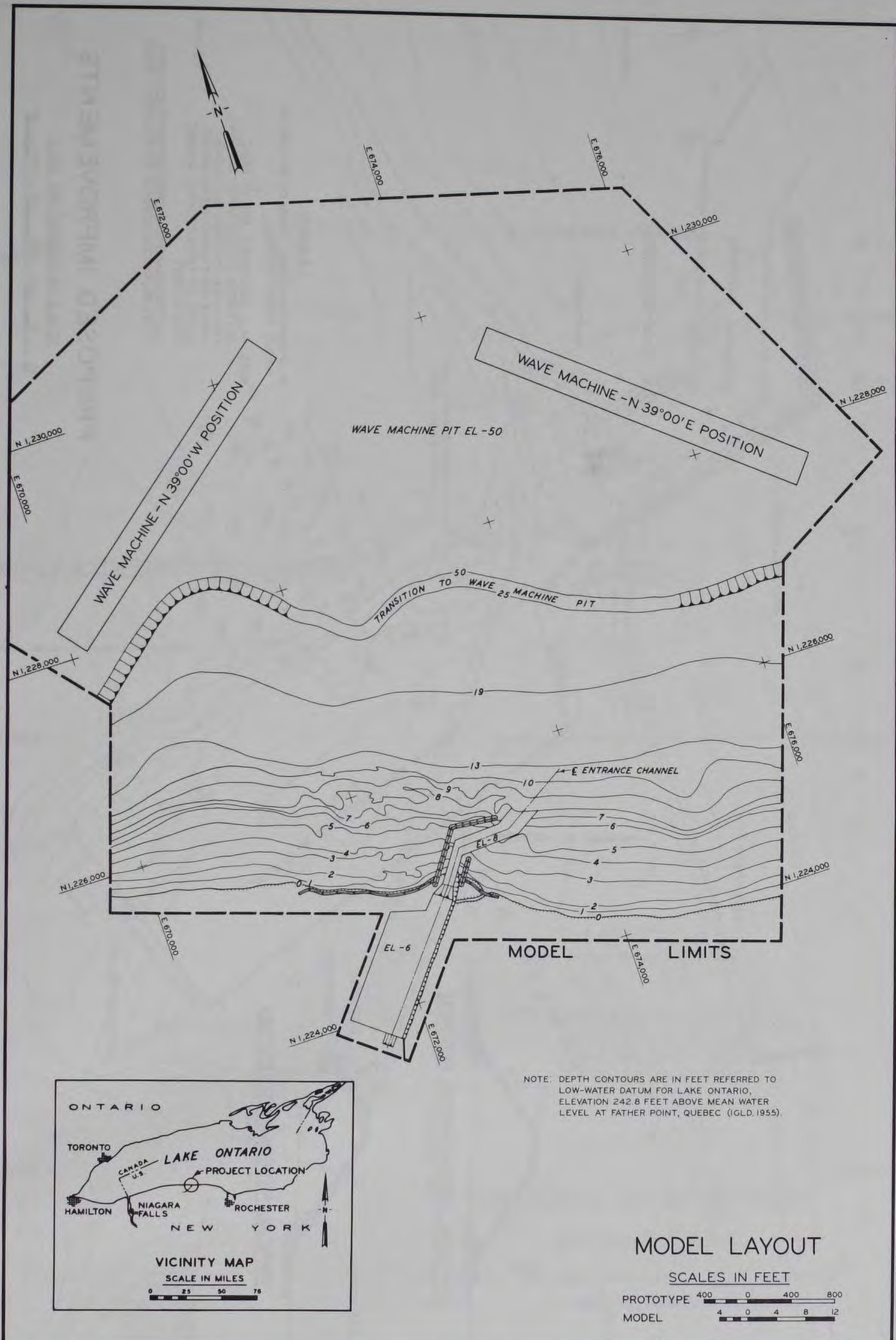
a. Base test

b. Plan 1

c. Plan 2

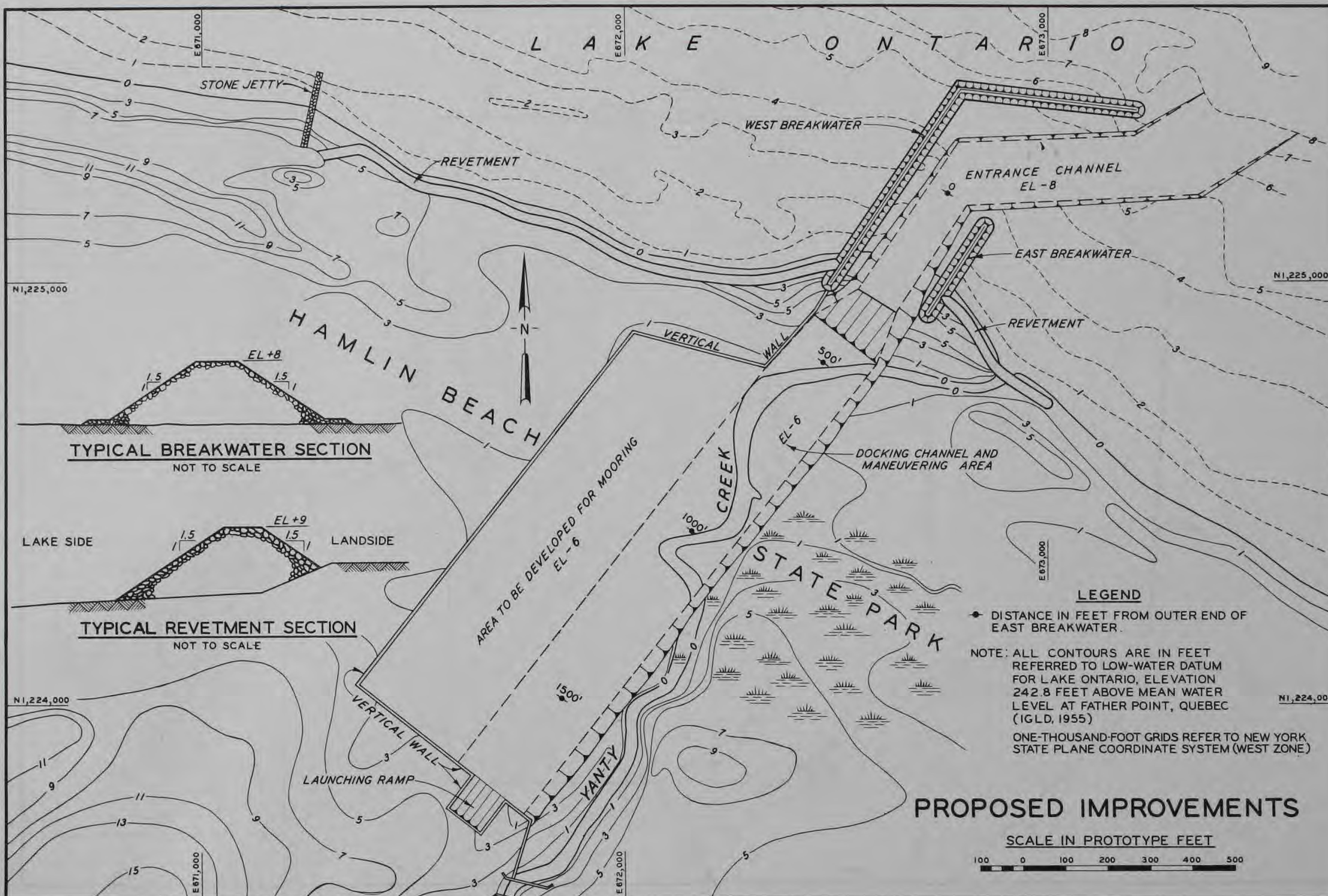
d. Plan 3

Photo 7. Comparison of wave patterns; 8-sec, 10.2-ft waves from N61°30'E

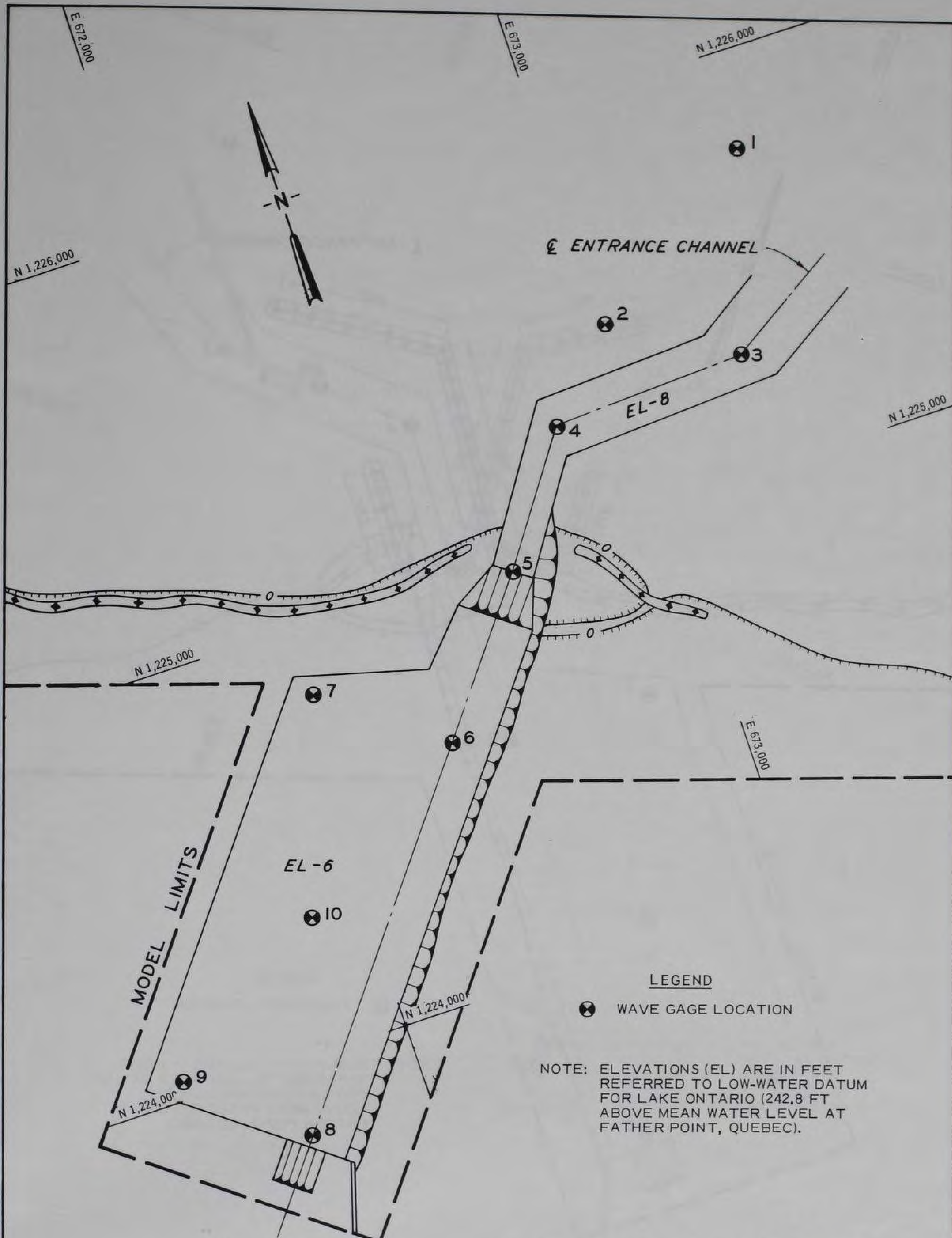

N
E 672,000
E 674,000
E 676,000
N 1,230,000
N 1,228,000
WAVE MACHINE - N 39°00'W POSITION
WAVE MACHINE - N 39°00'E POSITION
WAVE MACHINE PIT EL -50
E 670,000
TRANSITION TO WAVE MACHINE PIT
50
25
N 1,228,000
N 1,226,000
19
13
10
9
8
7
6
5
4
3
2
1
0
℄ ENTRANCE CHANNEL
EL -8
EL -6
N 1,226,000
N 1,224,000
E 676,000
E 670,000
E 674,000
E 672,000
N 1,224,000
MODEL LIMITS
NOTE: DEPTH CONTOURS ARE IN FEET REFERRED TO LOW-WATER DATUM FOR LAKE ONTARIO, ELEVATION 242.8 FEET ABOVE MEAN WATER LEVEL AT FATHER POINT, QUEBEC (IGLD. 1955).
ONTARIO
TORONTO
LAKE ONTARIO
CANADA
U.S.
PROJECT LOCATION
HAMILTON
NIAGARA FALLS
ROCHESTER
NEW YORK
VICINITY MAP
SCALE IN MILES
0 25 50 75
MODEL LAYOUT
SCALES IN FEET
PROTOTYPE 400 0 400 800
MODEL 4 0 4 8 12

74642

PLATE 2



PROPOSED IMPROVEMENTS

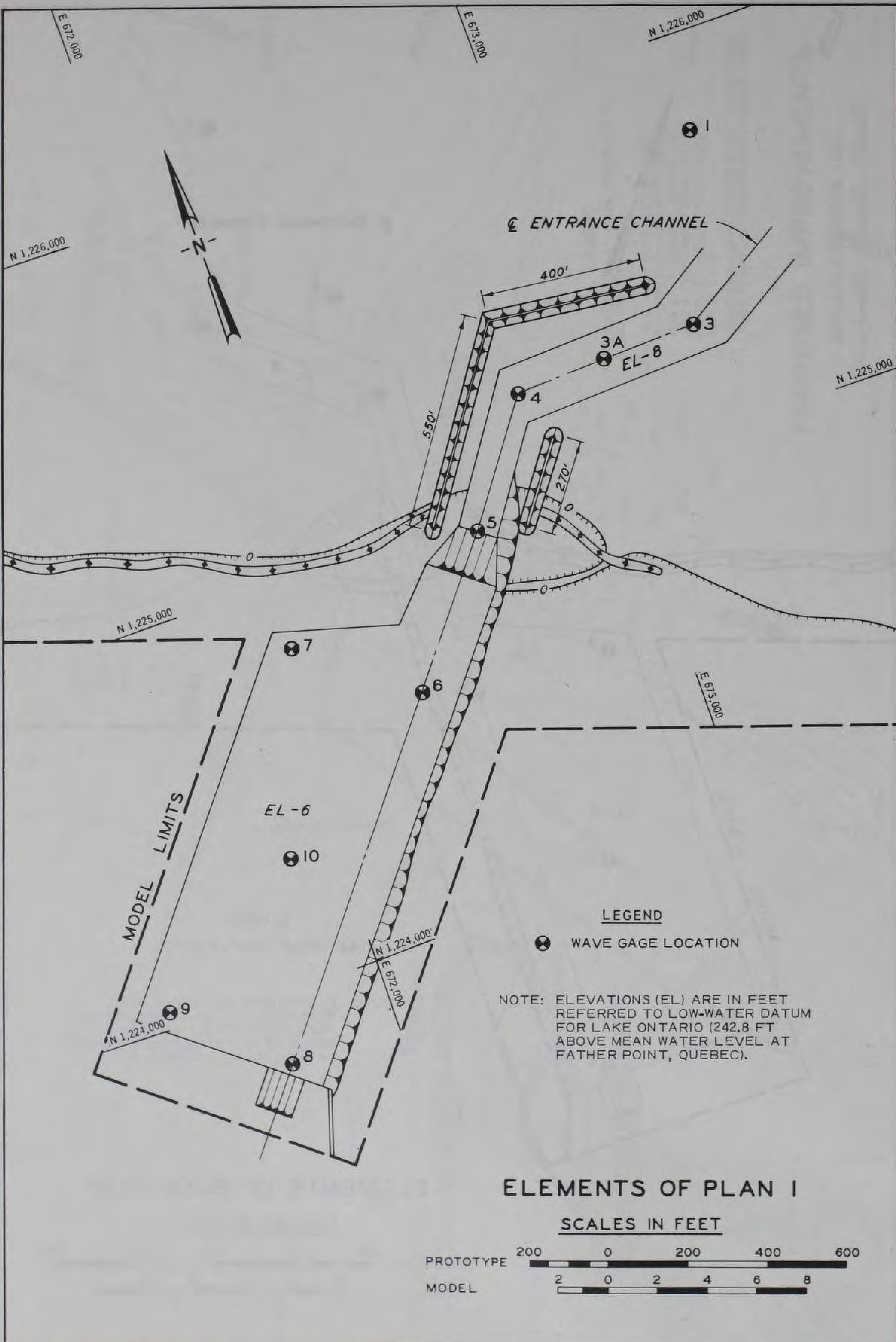


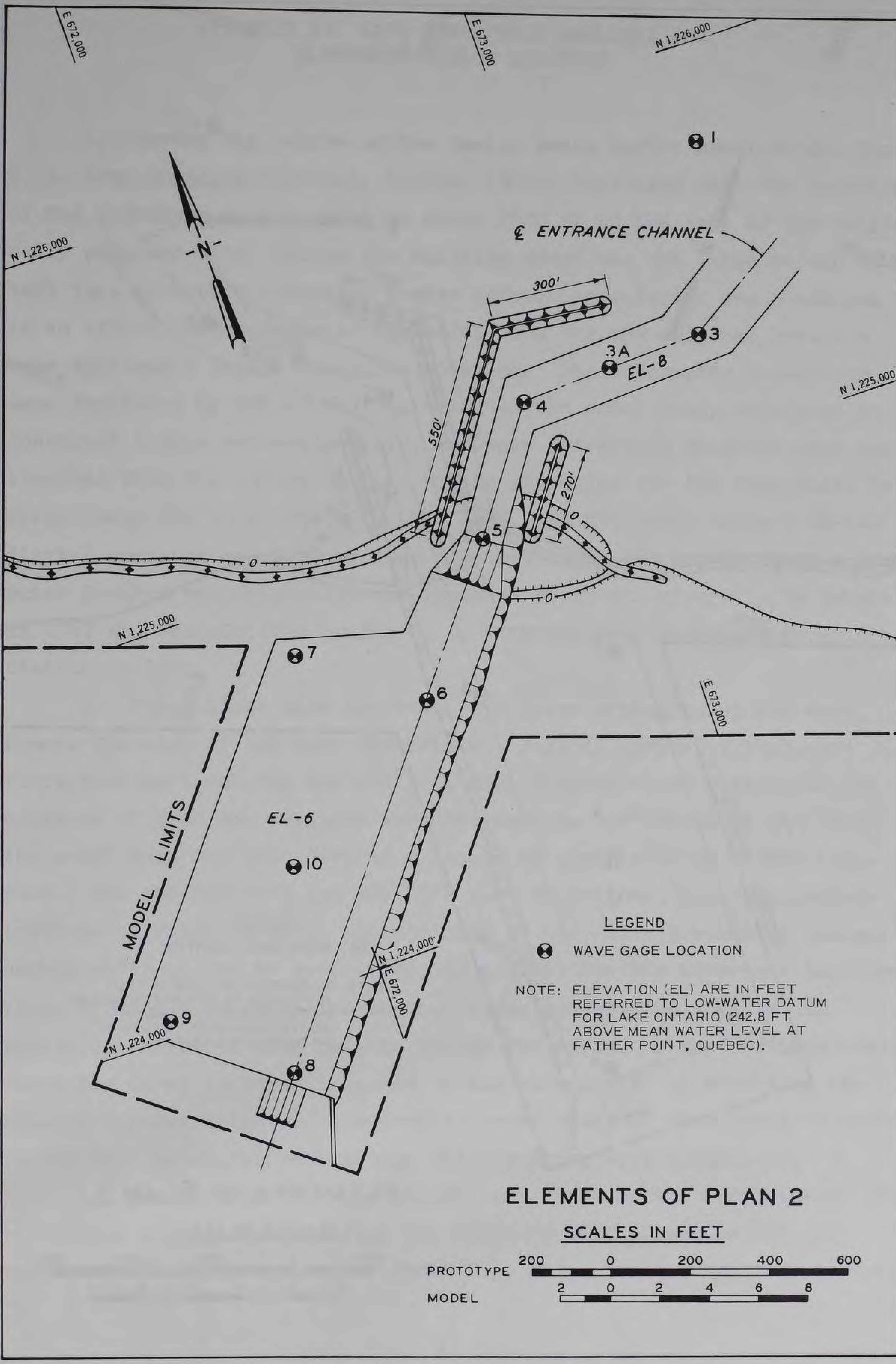
ELEMENTS OF BASE TEST

SCALES IN FEET

PROTOTYPE 200 0 200 400 600

MODEL 2 0 2 4 6 8


E 672,000
E 673,000
N 1,226,000
N 1,226,000
N 1,225,000
N 1,225,000
E 673,000
N 1,224,000
E 672,000
N 1,224,000
-N-
℄ ENTRANCE CHANNEL
400'
550'
270'
0
0
0
EL-8
EL -6
1
3
3A
4
5
6
7
8
9
10
MODEL LIMITS
LEGEND
WAVE GAGE LOCATION
NOTE: ELEVATIONS (EL) ARE IN FEET REFERRED TO LOW-WATER DATUM FOR LAKE ONTARIO (242.8 FT ABOVE MEAN WATER LEVEL AT FATHER POINT, QUEBEC).
ELEMENTS OF PLAN 1
SCALES IN FEET
PROTOTYPE 200 0 200 400 600
MODEL 2 0 2 4 6 8



ELEMENTS OF PLAN 2

ELEMENTS OF PLAN 3

APPENDIX A: WAVE REFRACTION ANALYSIS FOR ALTERNATE HARBOR LOCATION

1. During the course of the Hamlin Beach Harbor model study, the U. S. Army Engineer District, Buffalo (NCB), indicated that the location of the prototype harbor might be moved 2400 ft to the west of the originally proposed site. Since the existing model was not large enough to test this alternate location, a wave refraction analysis was conducted in an effort to determine if the model data for the original location were applicable to the alternate location. The underwater topography maps furnished by NCB at the beginning of the model study were used to construct a wave refraction grid, and wave refraction diagrams were constructed from the -50-ft contour to the shoreline for the four model test directions, N61°30'E, N39°00'E, N07°30'E, and N39°00'W, using a GE-440 digital computer and Calcomp drum plotter. The refraction diagram computer program was originally developed at Stanford University by Dobson[5] in 1967 and was modified by the U. S. Army Engineer Waterways Experiment Station in 1971.

2. Figs. A1-A4 show the wave rays (wave orthogonals) and wave fronts for each of the test directions. Fig. A1 indicates that wave refraction patterns for the N61°30'E test direction are similar at the entrance of both the original and alternate harbor locations and that the model data for this direction should be applicable to either location. For the N39°00'E and N07°30'E test directions, however, concentrations of wave energy in the vicinity of the outer breakwater arm and harbor entrance can be seen (figs. A2 and A3) for the alternate location. It is difficult to determine whether these convergence zones would result in increased wave heights inside the harbor or whether they would cause the waves to break lakeward of the harbor with no resulting increases in wave heights. A hydraulic model study of this location would be the best means for determining the resulting wave conditions.

3. Due to a lack of underwater topography on the west side of the refraction grid, wave rays from the N39°00'W test direction did not reach the alternate harbor location (fig. A4). It is suspected, however,

that refraction patterns from this direction would also indicate possible concentrations of wave energy at the alternate location.

4. It should be noted that all the refraction patterns indicate a diffusion of wave energy in the area from about 1500 to 2000 ft west of the original harbor location. In lieu of another model study, this area might be well worth considering as an alternate location for the harbor. Regardless of the final location chosen for the harbor, it should be noted that the low marshy area on the east side of the inner basin acts as a spending beach that is very effective in reducing wave heights (especially since the other perimeter walls are vertical) and thus a comparable side slope must be reproduced in any alternate harbor location.

Fig. A1. Wave refraction patterns for the N61°30'E test direction

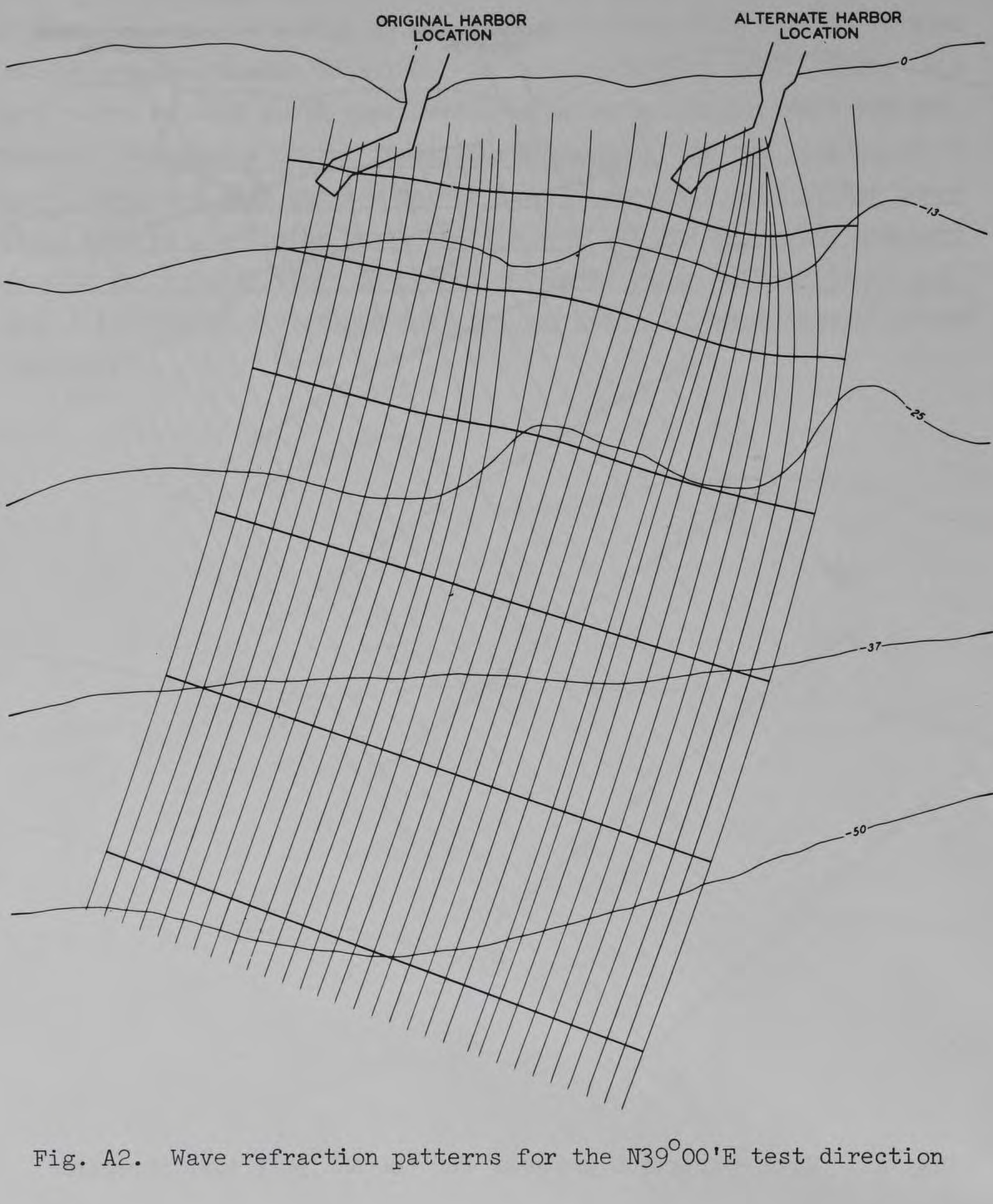


Fig. A2. Wave refraction patterns for the N39°00'E test direction

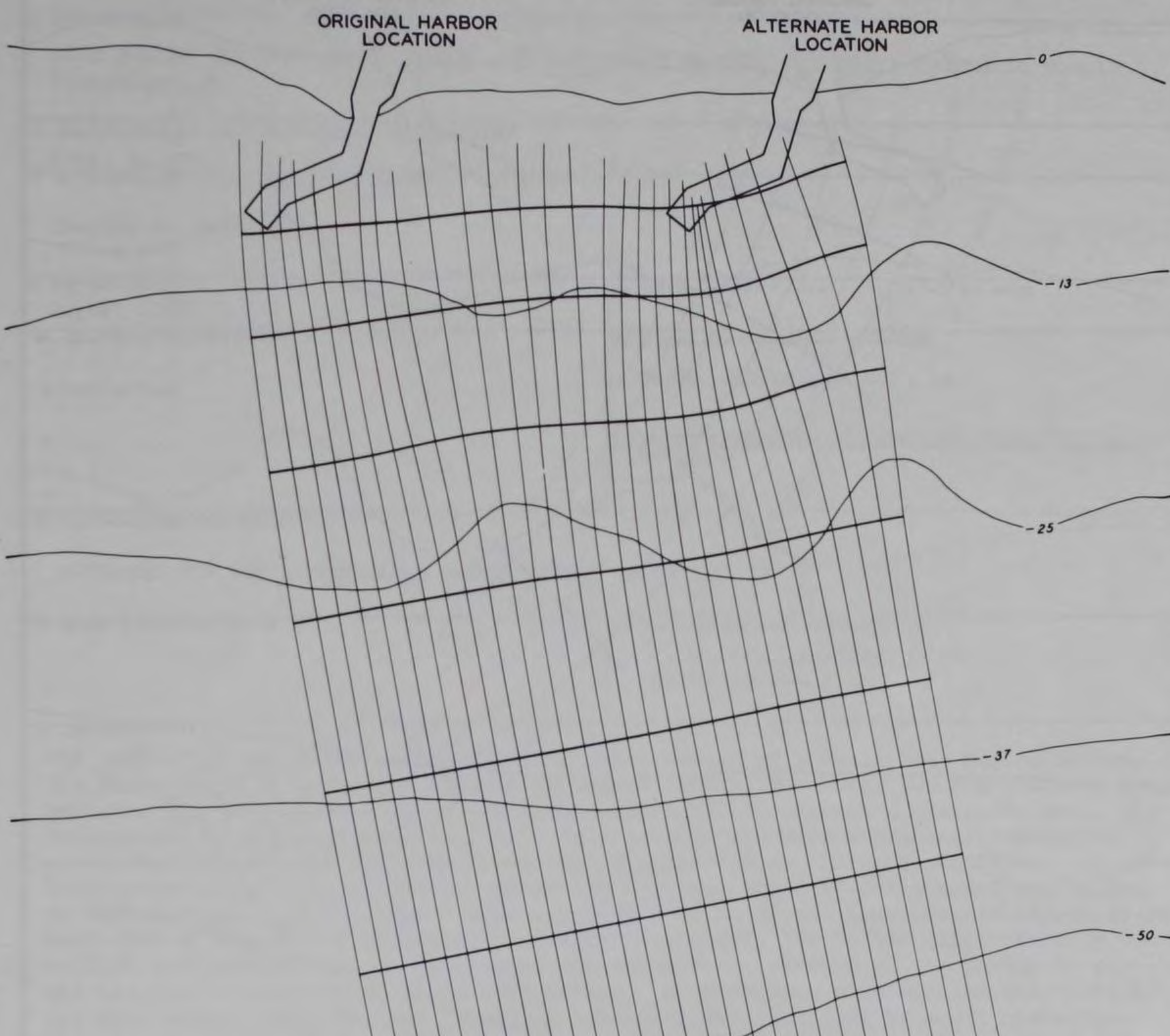


Fig. A3. Wave refraction patterns for the N07°30'E test direction

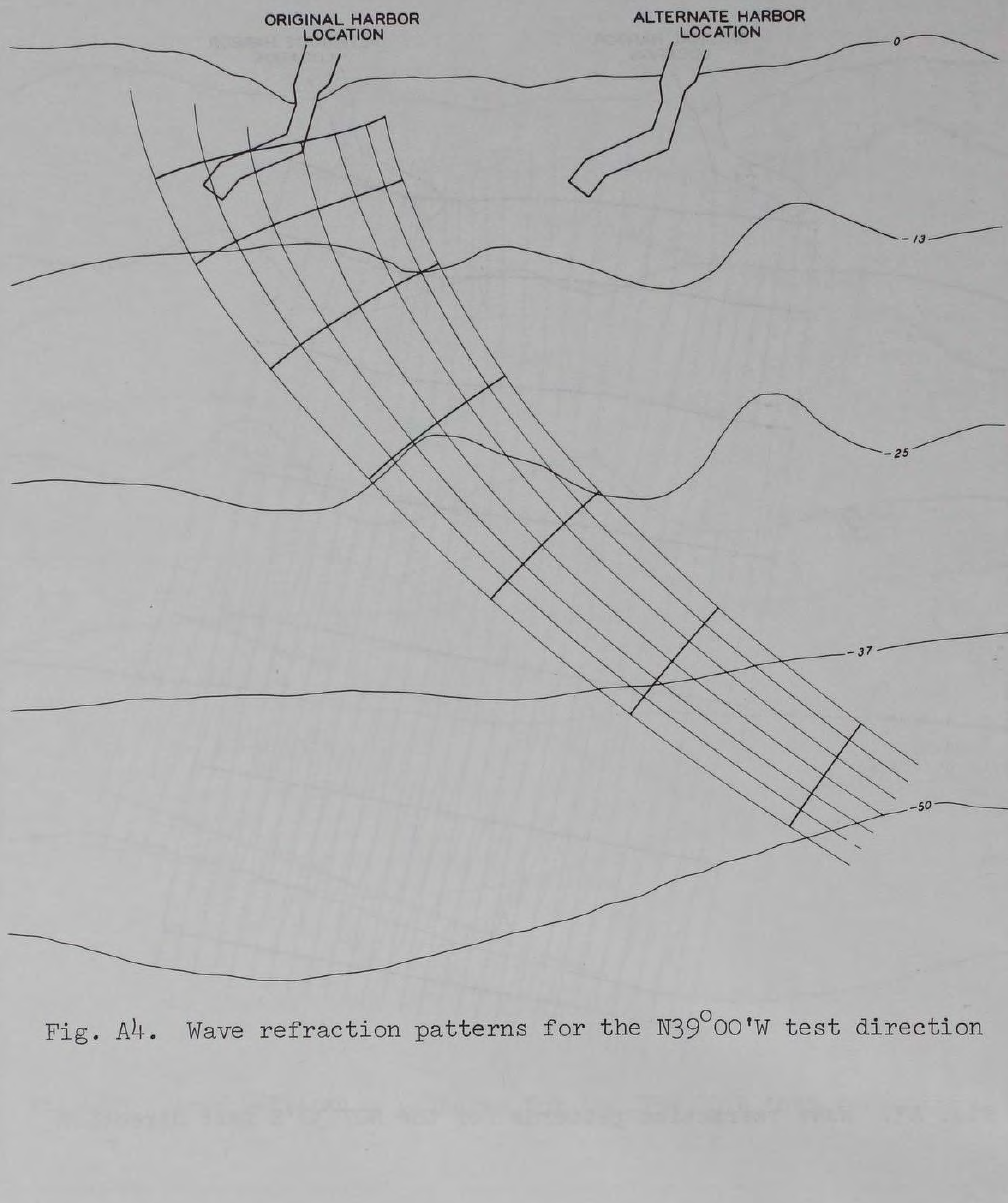


Fig. A4. Wave refraction patterns for the N39°00'W test direction

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13. ABSTRACT

The 1968 River and Harbor Act authorized construction of a small-boat harbor at Hamlin Beach State Park on Lake Ontario in Monroe County, New York. On the basis of experience at other locations, it was deemed advisable to conduct a hydraulic model investigation of the proposed facilities to determine the most economical breakwater arrangement consistent with the provision of satisfactory entrance conditions and adequate protection to moored boats within the harbor. The 1:64-scale model was molded in cement mortar and reproduced approximately 3000 ft of the Lake Ontario shoreline on each side of the harbor entrance, the entrance channel, the harbor (approximately 500 by 1150 ft), and sufficient underwater contoured area lakeward of the harbor to permit accurate simulation of storm wave action. A 50-ft-long wave machine and electrical wave height measuring and recording apparatus were utilized in model operation. It was concluded that the breakwater proposed for the west side of the entrance channel could be reduced in length by 100 ft without sacrificing the full protection desired for the entrance and inner harbor and that a reduction in length of another 100 ft would not seriously impair the desired protection.

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Harbors						
Hydraulic models						
Storm waves						
Water waves						

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