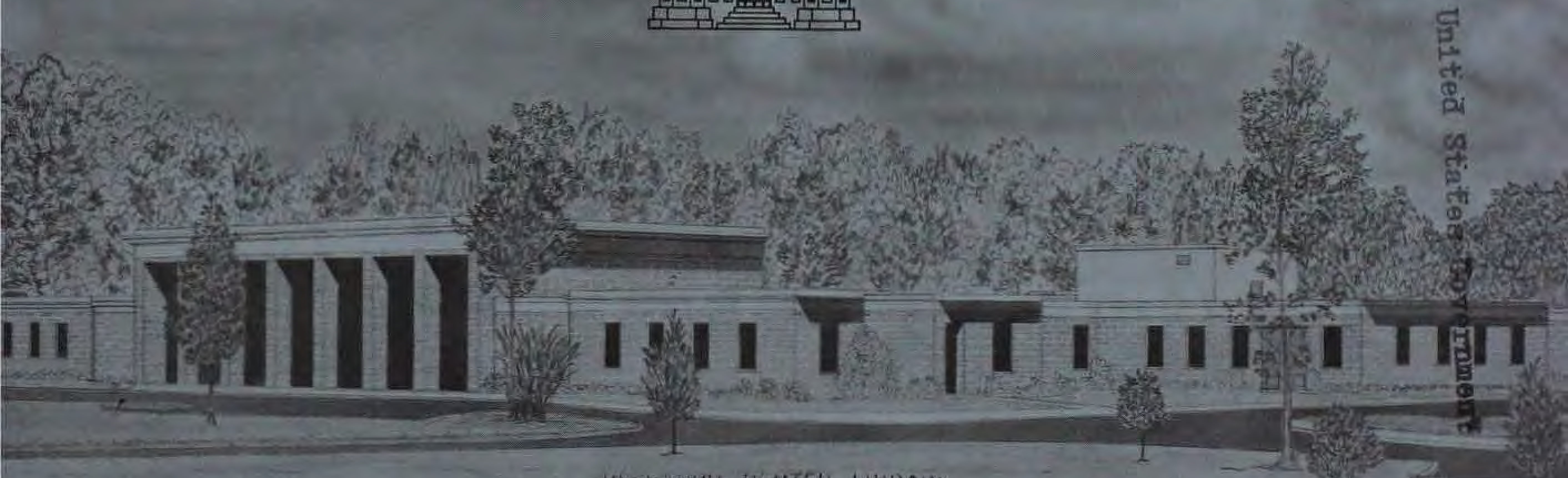
TECHNICAL REPORT H-70-11

DESIGN FOR FLOOD CONTROL AND WAVE PROTECTION, CHAGRIN RIVER EASTLAKE, OHIO

Hydraulic Model Investigation

by

C. E. Chatham, Jr.



September 1970

Sponsored by U. S. Army Engineer District, Buffalo

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

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ARMY-MRC VICKSBURG, MISS.

FOREWORD

A request for the U. S. Army Engineer Waterways Experiment Station (WES) to conduct a hydraulic model investigation of Chagrin River, Eastlake, Ohio, was initiated by the District Engineer, U. S. Army Engineer District, Buffalo (BED), in a letter to the Division Engineer, U. S. Army Engineer Division, North Central (NCD), dated 9 December 1966, subject, "Model Study, Chagrin River, Eastlake, Ohio." Authority to conduct the study was granted by the Office, Chief of Engineers (OCE), on 23 February 1967 by the first indorsement to a letter from the NCD dated 20 February 1967, subject, "Model Study, Chagrin River, Eastlake, Ohio." The model study was conducted during the period November 1968 to October 1969 in the Harbor Wave Action Section, Wave Dynamics 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 Wave Dynamics Branch. The tests were conducted by Mr. C. E. Chatham, Jr., Project Engineer, with the help of Mr. C. W. Brasfeild. This report was prepared by Mr. Chatham.

During the course of the investigation, liaison was maintained between the BED 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. R. E. Emmenegger of the NCD, MAJ Benjamin Schlapak, Deputy District Engineer, and Messrs. R. S. Goodno, G. A. Lynde, S. A. Maiore, and Roger Repp of the BED.

Directors of the WES during the conduct of the study and the preparation of this report were COL Levi A. Brown, CE, and COL Ernest D. Peixotto, CE. Technical Directors were Mr. J. B. Tiffany and Mr. F. R. Brown.

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
miles (U. S. statute)	1.609344	kilometers
square feet	0.092903	square meters
square miles	2.58999	square kilometers
feet per second	0.3048	meters per second
miles per hour	1.609344	kilometers per hour
cubic feet per second	0.02831685	cubic meters per second

SUMMARY

A 1:75-scale model of the lower 2000 ft of the Chagrin River and sufficient offshore area in Lake Erie to permit generation of the required test waves was used to investigate the arrangement and design of certain proposed improvements with respect to wave action and flood control. The proposed improvement plans consisted of (a) arrowhead breakwaters in Lake Erie at the mouth of the river, aggregating about 2360 ft in length; (b) realignment and enlargement of the river channel from Lake Erie through the city of Eastlake, with levees where required to supplement channel enlargement; (c) a spur channel and an access channel for navigation; (d) recreational facilities at the river mouth; and (e) the addition of beach fill and protective groins along the shoreline east of the east breakwater. A 60-ft-long wave machine and electrical wave-height measuring and recording apparatus were utilized in model operation.

It was concluded from test results that (a) the originally proposed plan of improvement, which specified a 275-ft-wide navigation opening and a 230-ft-wide, 10-ft-deep lower river channel, will not provide adequate protection from wave action, and current velocities in the revised channel will exceed the specified criteria; (b) reducing upstream current velocities and wave heights by reducing the width of the navigation opening and outer channel to 150 ft will result in flooding along the west bank of the river; (c) an improvement plan utilizing a 190-ft-wide, 14-ft-deep navigation opening and lower river channel will provide more satisfactory flood flow conditions than any of the other plans tested; (d) while several of the plans tested would provide satisfactory wave conditions at most locations inside the harbor, only plan 6B meets the specified criteria at all locations; and (e) the addition of wave absorbers to the lakeward face of the east breakwater and the installation of groins along the shore eastward of this structure will be beneficial in reducing the magnitude of alongshore littoral currents and should help to reduce beach erosion in this area.

Fig. 1. Aerial view of mouth of Chagrin River, Eastlake, Ohio

DESIGN FOR FLOOD CONTROL AND WAVE PROTECTION CHAGRIN RIVER, EASTLAKE, OHIO

Hydraulic Model Investigation

PART I: INTRODUCTION

DESCRIPTION OF PROTOTYPE

Existing Conditions

1. The Chagrin River is located in northeastern Ohio and flows into Lake Erie at Eastlake, Ohio, about 15 miles* east of Cleveland (plate 1). The river basin contains about 264 square miles and is roughly elliptical in shape, being approximately 30 miles long, north to south, and 17 miles wide, east to west.

2. The lower 1.5 miles of the river and its meandering side channels have been extensively developed for mooring of small boats, and this reach of the river is subject to heavy marine recreational traffic despite poor conditions of passage between the river mouth and Lake Erie (see fig. 1). In 1960, approximately 850 recreational craft of all types were permanently based at Chagrin River.[1] A steam-electric power plant owned by the Cleveland Electric Illuminating Company is located on the shore of Lake Erie approximately 2000 ft west of the mouth of the Chagrin River. The water intake channel of the power plant is protected by a cellular sheet-pile jetty that extends about 1000 ft into the lake.

Proposed Improvements

3. The proposed multiple-purpose plan of improvement for the Chagrin River consists of (a) arrowhead breakwaters in Lake Erie at the mouth of the river to provide wave protection and prevent further erosion of the shoreline inside the arrowhead and formation of the sandbar in the river mouth; these breakwaters would be either rubble-mound or cellular sheet-pile structures with an aggregate length of about 2360 ft; (b) the realignment and enlargement of the river channel from Lake Erie through the city of Eastlake, with levees where required to supplement channel enlargement; (c) a spur channel for navigation, 100 ft wide, 6 ft deep, and about 1500 ft long; (d) an access channel for navigation, 50 ft wide and 5 ft deep, from the -5 ft contour in Lake Erie to the east channel; (e) recreational facilities at the river mouth; and (f) the addition of beach fill and protective groins along the shoreline east of the east breakwater.

THE PROBLEM

4. Major floods occurred in the Chagrin River basin in March 1913, January 1929, June 1931, March 1948, October 1954, January 1959, and January 1968. Due to progressive urban development along the lower 30 miles of the main stream, susceptibility to serious damage from flooding has increased in recent years. In the lowlands along the lower 30 miles of the river,

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

the channel is in good condition, but it is not large enough to contain the higher discharges, and is characterized by numerous sharp bends and meanders. The most damaging floods in the lower reaches of the Chagrin River have resulted when a frontal-type storm occurs in the early spring, accompanied by melting snow over the basin and ice jams in the lower reaches. Many times the mouth of the river has been virtually closed by sandbars formed by river currents and littoral drift due to wave action. The formation of a sandbar at the river mouth affects river stages during high discharges in the summer months and is restrictive to the passage of ice during the spring breakup. Formations of windrowed ice in Lake Erie at the mouth of the river during the winter and ice jams in restricted reaches of the river during spring discharges cause water to impound behind them, which results in flooding with even moderate river discharges.

5. During the past 15 years, the shores on both sides of the river mouth have suffered serious erosion and loss of beaches due to wave action. In addition, difficulty has been experienced in maintaining a navigable channel for small boats through the sandbar at the river mouth. During the navigation season, the east channel is usually blocked, and frequent dredging, usually following major storms, is required to maintain adequate depths in the main channel for use by small craft.

6. In summary, improvements at the mouth and in the lower reaches of the Chagrin River are needed to provide adequate depths throughout the navigation season for use of small craft, to provide adequate channel capacity for flood flows, to reduce erosion conditions along the shore on both sides of the river mouth, and to provide protection from wave action.

PURPOSE OF MODEL STUDY

7. The purpose of the model study was to (a) study wave action and flood flow conditions in the harbor entrance and lower reaches of the river with the proposed improvements and revisions installed in the model; (b) develop remedial plans for the alleviation of undesirable conditions with respect to wave action, navigation, and flood flows in the entrance and lower river as found necessary; and (c) 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 and enhance flood flow conditions.

WAVE-HEIGHT CRITERIA

8. At the present time, completely reliable criteria have not been developed for ensuring that satisfactory navigation and mooring conditions will obtain in small-craft harbors for all types of small craft, mooring conditions, and a wide range of wave periods. For the study reported herein, the U. S. Army Engineer District, Buffalo (BED), specified that, for an improvement plan to be acceptable, maximum wave heights in the harbor should not exceed 2.5 ft in the mouth of the main river channel and 1.5 ft in the entrances to the east channel and the boat mooring basins.

FLOOD FLOW CRITERIA

9. No specific criteria were set with regard to current velocities in the river channel. It was desired, however, that velocities be high enough to allow the passage of ice yet low enough

that extensive bank protection would not be required. Consequently, a current velocity of about 6 fps was selected by the BED as being acceptable. With regard to water surface elevations, it was specified that no flooding should occur along the banks of either the main channel or the boat mooring basins.

PART II: THE MODEL

DESIGN OF MODEL

10. The Chagrin River model (plate 2) was constructed to a linear scale of 1:75, model to prototype. Scale selection was based on such factors as (a) the depth of water required in the model to prevent excessive bottom friction effects; (b) the absolute size of model waves; (c) available shelter dimensions and the area required for constructing the model; (d) efficiency of model operation; (e) capabilities of available wave-generating and wave-measuring equipment; and (f) cost of model construction. A geometrically undistorted model was necessary to ensure accurate reproduction of short-period wave patterns. Following selection of the linear scale, the model was designed and operated in accordance with Froude's model law.[2] The scale relations used for design and operation of the model were as follows:

Characteristic	Dimension*	Model:Prototype Scale Relation
Length	L	$L_r = 1:75$
Area	L^2	$A_r = L_r^2 = 1:5625$
Volume	L^3	$\forall_r = L_r^3 = 1:421{,}875$
Time	T	$T_r = L_r^{1/2} = 1:8.66$
Velocity	L/T	$V_r = L_r^{1/2} = 1:8.66$
Roughness (Manning's n)	$L^{1/6}$	$n_r = L_r^{1/6} = 1:2.054$

* Dimensions are in terms of length and time.

11. The proposed plans of improvement for Chagrin River included the use of rubble-mound wave absorbers and groins. Past experience and experimental research have shown that considerable wave energy passes through the interstices of this type of structure; thus, the transmission and absorption of wave energy became a matter of concern in the design of the 1:75-scale model. In small-scale harbor models, rubble-mound structures reflect relatively more wave energy than geometrically similar prototype structures.[3] Too, the transmission of energy through the structure 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-reflection and wave-transmission characteristics. From previous findings[4,5] at the U. S. Army Engineer Waterways Experiment Station (WES) for cases similar to that at Chagrin River, it was determined that a considerable reduction in scale effects for the reflection and transmission characteristics could be obtained by increasing the size of the rock used in the 1:75-scale model to approximately twice that required for geometric similarity. Accordingly, in constructing the wave absorber and groin structures in the Chagrin River model, the rock sizes were computed linearly by scale, then doubled to arrive at the actual sizes used in the model.

12. The values of Manning's roughness coefficient n used in the design of the improved river channel were calculated by the BED from water surface profiles of known discharges in the

prototype. From these computations and past experience, n values of 0.050 for the existing channel and 0.030 for the improved channel were selected. When the improved channel was constructed in the model, it was given a finish that, according to past experimentation and experience at the WES,[6] would represent a prototype Manning's n of 0.030. Figure 2 shows a typical section of the improved channel with model roughness installed.

Fig. 2. Typical section of improved river channel with model roughness installed

DESCRIPTION OF MODEL AND APPURTENANCES

13. The model, which was molded in cement mortar, reproduced the lower 2000 ft of the river channel and underwater contours in Lake Erie to an offshore depth of 27 ft. Sufficient additional offshore area was included to permit generation of test waves from all critical directions. The total area reproduced in the model was approximately 10,700 sq ft, representing about 2.2 square miles in the prototype. Photograph 1 shows a general view of the model with plan 1 installed. Vertical control for model construction was based on low water datum (lwd), the elevation* of which is 568.6 ft above mean water level at Father Point, Quebec (International Great Lakes Datum 1955).[1] Horizontal control was referenced to the State of Ohio Lambert projection, north zone, U. S. Geological Survey.

14. Model waves were generated to scale by a 60-ft-long wave machine with a

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

trapezoidal-shaped, vertical-motion plunger. The vertical movement of the plunger caused a periodic displacement of water incident to this motion. The length of stroke and the period of the vertical motion were infinitely variable over the range necessary to generate waves with the required characteristics. In addition, the wave machine was mounted on retractable casters, which enabled it to be positioned to generate waves from the required directions. 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 depth of the gage in the water.

15. A water-circulating system (plate 2) consisting of a 6-in. water-intake manifold, a 3-cfs pump, and a 6-in. Van Leer weir[7] was utilized in the model for reproduction of steady-state flows through the river channel and outer harbor area that corresponded to selected prototype river discharges. The direction and magnitude of currents were measured by timing the progress of weighted floats (8-ft submergence) over known distances. Water surface profiles were secured with staff gages mounted on a rail system that extended from the lakeward end of the breakwaters to the upstream limit of the model.

PART III: TEST CONDITIONS AND PROCEDURES

SELECTION OF TEST CONDITIONS

Still-Water Level

16. Still-water levels (swl) for harbor wave action models are selected so that the various wave-induced phenomena that are dependent on 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.

17. Water levels of the Great Lakes vary from year to year and from month to month. Also, at any given location the water level can vary from day to day and from hour to hour. Continuous records of the levels of the Great Lakes have been tabulated since 1860. The usual pattern of seasonal variations of water levels consists of highs in summer and lows in late winter. The highest and lowest monthly average levels in Lake Erie usually occur in June and February, respectively. The average level of Lake Erie during the period of record was +1.8 ft for the entire year and +2.1 ft for the ice-free period (April through November). The highest one-month average level of +4.2 ft occurred in May 1952, and the lowest one-month average level of −1.1 ft occurred in February 1936. The seasonal variation in the mean monthly level of Lake Erie usually ranges between 1 and 2 ft, with an average variation of 1.6 ft.

18. Seasonal and longer variations in the levels of the Great Lakes are caused by variations in precipitation and other factors that affect the actual quantities of water in the lakes. Wind tides and seiches are relatively short-period fluctuations caused by the tractive force of wind blowing over the water surface and differential barometric pressures, and are superimposed on the longer period varations in lake level. Records of short-period fluctuations for the Cleveland area (15 miles west of Chagrin River) show that a rise of 1.0 ft will occur about once in 2.5 months.[8] Large short-period rises in local water level are associated with the most severe storms, which generally occur in the winter months when the lake level is usually low; thus, the probability that a high lake level and a large wind tide or seiche will occur simultaneously is relatively small.

19. Based on the above considerations, an swl of +3.0 ft was selected for use in the wave action phase of the model study. This value was obtained by combining the average water level of Lake Erie during the ice-free period (+2.1 ft) with a 0.9-ft, short-period rise in local water level due to wind tide.

20. The water surface elevation of Lake Erie for the flood flow phase of the model study was selected by the BED. In making this selection, consideration was given to the annual maximum daily mean stage that might be expected once in 2 years and the known lake stages that occurred coincidentally with known flood peaks. An el of +2.8 was assumed to be a conservative estimate for a lake level to occur coincidentally with the design discharge of 27,000 cfs and was therefore selected for use in the model. A lake level of el +1.4 was selected for use in the model with a discharge of 20,000 cfs to obtain velocity measurements for bank riprap design.

Wave Dimensions and Directions

21. *Factors influencing selection of test wave characteristics.* 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 allow a realistic test of the proposed improvement plans and an accurate evaluation of the elements of the various proposals. 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 wave that can be generated by a given storm depends on 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 geographic position of the navigation entrance to the harbor; (d) the alignments, lengths, and locations of the various reflecting surfaces inside the harbor; and (e) the refraction of waves caused by differentials in depth in the area lakeward of the harbor, which may create either a concentration or a diffusion of wave energy at the harbor site.

22. *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 change in wave height and direction can be determined by plotting refraction diagrams and calculating refraction coefficients. For this study, refraction diagrams were prepared by personnel of the WES for representative wave periods for 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 at any point in shallow water (H) is 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 wave length and water depth, can be obtained from reference 9.

23. *Prototype wave data.* Measured wave data on which a comprehensive statistical analysis of wave conditions could be based were unavailable for the Chagrin River area. However, wave hindcast data for station "B" north of Cleveland (approximately 15 miles west of Chagrin River) were secured from reference 10. These data, published in 1953, were based on synoptic weather charts compiled from U. S. Weather Bureau observations for the three-year period 1948-1950 and were computed using hindcasting techniques developed by Sverdrup and Munk[11] in 1947 and revised by Arthur[12] in 1948. These data are summarized in table 1 to show the characteristics and estimated durations of deepwater waves approaching Chagrin River from the various directions. In addition to the data in table 1, wave hindcast data were computed by the BED using wind records from the U. S. Coast Guard Station at Cleveland. These data, which were computed using Bretschneider's method[13] and which cover the past 20 years, are presented in the following tabulation:

Deepwater Wave Direction	Wind Velocity mph	Fetch Length miles	Average Depth ft	Wave Period sec	Wave Height ft
West-northwest	50	50	50	8.5	10
Northwest	45	54	56	8	9.7
North	45	65	63	8.5	10.5
North-northeast	45	89	67	9	10.8

24. *Shallow-water waves.* The deepwater wave data in table 1 were converted to shallow-water values for use in the model by the application of refraction and shoaling coefficients. The results of this conversion are presented in table 2. A similar procedure was used for the additional wave data listed in paragraph 23 with the following results:

Deepwater Wave Direction	Corresponding Shallow-Water Refracted Direction	Shallow-Water Waves	
		Period sec	Height ft
West-northwest (N67°30'W)	North 63°30'West	8.5	11.0
Northwest (N45°00'W)	North 44°52'West	8	10.3
North	North 10°56'West	8.5	10.0
North-northeast (N22°30'E)	North 01°12'East	9	9.7

The shallow-water wave directions were taken to be the average directions of the refracted waves for the significant wave periods noted from each deepwater wave direction.

25. *Selection of test waves.* The deepwater directions and the corresponding shallow-water directions selected for generating test waves in the model are presented in the following tabulation. Also shown are the wave periods and wave heights selected for the various test directions.

Deepwater Wave Direction	Selected Shallow-Water Test Direction	Selected Test Waves	
		Period, sec	Height, ft
West	North 85°00'West	5	5
		5	10
		6	10
North 67°30'West	North 65°00'West	5	5
		5	9
		6	11
		8	8
		9	11
North 45°00'West	North 45°00'West	5	5
		5	8
		8	10
North 22°30'West	North 25°00'West	5	7
		6	10
		6	16
North	North 08°00'West	5	6
		9	10

(Continued)

Deepwater Wave Direction	Selected Shallow-Water Test Direction	Selected Test Waves	
		Period, sec	Height, ft
North 22°30′East	North 06°00′East	5	5
		9	10
North 45°00′East	North 27°00′East	5	5

River Discharges

26. The U. S. Geological Survey has maintained a continuous-recording gage on the Chagrin River at Willoughby, Ohio, since 1939. Records[1] from this gage indicate an average discharge of 313 cfs, a maximum summer discharge (June 26, 1931) of 19,000 cfs, and a maximum recorded discharge (March 22, 1948) of 21,400 cfs. Using the above and other available records, supplemented by estimated discharges based on data from nearby watersheds, the BED selected a design discharge of 27,000 cfs for the Chagrin River. In addition, a discharge of 20,000 cfs was selected for use in determining bank riprap design.

ANALYSIS OF MODEL DATA

27. The relative merits of the various plans tested were evaluated by (a) comparison of wave heights at selected locations in the harbor; (b) comparison of current velocities and water surface profiles in the revised river channel; and (c) visual observations and photographs. In the wave-height data analysis, the average height of the highest one-third of the waves recorded at each gage location was selected. 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.[14] 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

DESCRIPTIONS OF TESTS

Improvement Plans

28. Wave-height, current velocity, and water surface elevation tests were conducted for 18 variations in the design elements of the proposed plans of improvement to the harbor. The plans tested included (a) the basic plan of improvement as proposed by the BED; (b) variations in the width and depth of the proposed channel; (c) variation in the length of the east breakwater and, consequently, in the width of the navigation opening between the breakwaters; (d) the addition of auxiliary structures in the area between the east and main channels; (e) the addition of wave absorbers along the lakeward face of the east breakwater and groins along the shore east of the east breakwater; (f) a change in the alignment of the east breakwater; and (g) combinations of several of these design features. Brief descriptions of the plan elements are given in the following subparagraphs; dimensioned details are presented in plates 3-12. Plate 9 shows typical sections for plans 1-6C, and plate 12 shows typical sections for plans 6D-6H and 7A-7C.

- *a.* *Plan 1 (plate 3)* consisted of the basic plan of improvement proposed by the BED. This plan included arrowhead breakwaters aggregating 2360 ft in length, with a 275-ft navigation opening between the breakwaters, a 230-ft-wide channel with a bottom elevation of −10 ft from the −10 ft contour in Lake Erie to sta 27+50, and a gradual channel transition to a width of 280 ft and a bottom elevation of −7 ft at sta 30+00, the upstream limit of the model.
- *b.* *Plan 2 (plate 3)* involved increasing the length of the east breakwater by 80 ft, thereby reducing the navigation opening between the breakwaters to 200 ft.
- *c.* *Plan 3 (plate 4)* entailed reducing the navigation opening and channel width between sta 0+00 and 27+50 to 190 ft.
- *d.* *Plan 4 (plate 5)* included a 190-ft navigation opening and channel width from sta 0+00 to 10+00, with a transition to a width of 230 ft at sta 11+00. Channel widths upstream from sta 11+00 were the same as for plan 1.
- *e.* *Plan 5 (plate 6)* included a 150-ft navigation opening and channel width from sta 0+00 to 10+00, with a transition to a 230-ft width at sta 12+00. Channel widths upstream from sta 12+00 were the same as for plan 1.
- *f.* *Plan 6 (plate 7)* consisted of a 190-ft navigation opening, a 190-ft-wide channel with a bottom elevation of −14 ft from the −14 ft contour in Lake Erie to sta 25+00, and a transition to a width of 240 ft and a bottom elevation of −9 ft at sta 27+50.
- *g.* *Plan 6A (plate 7)* consisted of the same breakwater and channel configuration as plan 6 with a 330-ft-long rubble-mound wave absorber installed along the shore between the east and main channels.
- *h.* *Plan 6B (plate 7)* entailed the addition of a 150-ft-long rubble-mound groin with an impervious core to the plan 6A wave absorber.
- *i.* *Plan 6C (plate 8)* involved the installation of a sand beach with a slope of 1 vertical on 10 horizontal in the area between the east and main channels.
- *j.* *Plans 6D, 6E, and 6F (plate 10)* entailed the addition of 225-ft, 450-ft, and 675-ft-long wave absorbers, respectively, along the lakeward face of the east breakwater.
- *k.* *Plans 6G and 6H (plate 10)* involved the addition of one and two groins, respectively, along the shore east of the east breakwater with the plan 6B breakwater configuration installed.

l. *Plan 7 (plate 11)* involved a change in the alignment of the east breakwater.

m. *Plan 7A (plate 11)* entailed the addition of a 750-ft-long wave absorber to the lakeward face of the plan 7 east breakwater.

n. *Plans 7B and 7C (plate 11)* involved the addition of one and two groins, respectively, along the shore east of the east breakwater with the plan 7 breakwater configuration installed.

Wave-Height Tests

29. Wave-height tests for the various improvement plans were conducted using test waves from one or more of the test directions listed in paragraph 25. Analysis of the plan 1 data showed that the most critical directions of wave approach with respect to wave action in the inner harbor area were the shallow-water test directions of north 25° west and north 06° east. Consequently, tests involving some of the proposed improvement plans were limited to these test directions only. However, plan 6B was tested comprehensively for all test conditions listed in paragraph 25 to establish a more complete comparison with plan 1. The wave-height gage locations for each improvement plan tested are included in plates 3-8.

Current Velocity and Water Surface Elevation Tests

30. Current velocity measurements and water surface profiles were secured for plans 1-6 using river discharges of 27,000 and 20,000 cfs and lake levels of +2.8 and +1.4 ft, respectively. In addition, measurements were secured for plan 1 using a river discharge of 27,000 cfs and lake levels of 0.0 and +1.4 ft to determine the effects of different lake levels on flow in the revised river channel. Measurements were made at 200-ft intervals along the channel center line and the east and west banks.

TEST RESULTS

31. In the evaluation of test results, the relative efficiency of each of the improvement plans tested was assessed on the basis of an analysis of measured wave heights, current velocities, and water surface elevations. The model wave-height and current velocity data were tabulated to show the measured values at the various locations for each of the improvement plans. The water surface elevations were plotted graphically to show water surface profiles along the channel center line and the east and west banks for each improvement plan tested. An additional comparison of the various plans tested was made by selecting wave-height and current velocity data at corresponding locations and computing a numerical average of the selected data for each plan, then comparing these average values as percentages of wave-height or current velocity increase or reduction as compared with plan 1 results.

Wave-Height Tests

32. The results of wave-height tests with the basic proposed plan of improvement (plan 1) installed in the model are presented in table 3. These data reveal that wave heights ranged up to 3.2 ft in the main channel, 2.7 ft in the entrance to the east channel, and 1.7 ft in the entrances to the boat mooring basins. These values exceed the specified criteria of 2.5 ft in the main channel and 1.5 ft in the east channel and basin entrances.

33. The results of wave-height tests with plans 2-6 installed in the model are presented in tables 4 and 5. These data indicate that reducing the navigation opening from 275 to 200 ft (plan 2) effected a reduction in wave heights inside the harbor (gages 3-12) of about 20 percent. However, wave heights in the vicinity of the east channel (gages 5 and 6) were not reduced significantly. Reducing the channel width and navigation opening to 190 ft (plan 3) also reduced wave heights inside the harbor about 20 percent when compared with plan 1 wave heights; however, as was the case with plan 2, wave heights in the entrance to the east channel exceeded the specified criteria for that area. The data in tables 4 and 5 reveal that the installation of plans 4 and 5 effected reductions in wave heights inside the harbor of about 30 and 50 percent, respectively, when compared with plan 1. Wave heights for these two plans were within the specified criteria at all gages with the exception of those measured at gage 5 at the entrance to the east channel, where maximum wave heights of 1.9 ft were recorded for both plans. The installation of plan 6 (190-ft navigation opening and channel width and 14-ft channel depth) reduced wave heights about 20 percent when compared with plan 1 wave heights. Wave heights were within the specified criteria at all gages except gage 5, where maximum heights ranged up to 2.5 ft.

34. Visual observations revealed that the excessive wave heights in the entrance to the east channel were caused primarily by waves reflecting into that area from the rather steep slope of the shore reach between the east channel and the main channel. To alleviate this action, plans 6A, 6B, and 6C were formulated and tested in the model. The results of wave-height tests for these three auxiliary plans are compared with similar data for plan 6 in table 6. These data reveal that, while significant wave-height reductions were accomplished in the area of the east channel entrance by each of the three auxiliary plans, the specified wave-height criteria at all gage locations were satisfied only by plan 6B.

35. Since plan 6B appeared to be the optimum plan of improvement with respect to wave heights inside the harbor, this plan was subjected to wave attack from all test directions to allow a more comprehensive comparison with plan 1. The results of these tests are presented in table 7 and, when compared with the plan 1 data in table 3, indicate that plan 6B reduced wave heights inside the harbor about 30 percent.

36. During model testing, overhead photographs of wave patterns in the harbor area were secured for several of the proposed improvement plans. Photographs 2-15 present a comparison of wave patterns for plans 1-6 and 6B for representative test waves.

37. Photographs 2-15 indicate that a considerable amount of wave energy reflects off the lakeward faces of the cellular sheet-pile breakwaters. It therefore became a matter of concern that reflected waves from the east breakwater might worsen the existing problem of beach erosion along the shore eastward of that structure. In an effort to reduce wave reflections and wave-induced currents along the beach, various combinations of wave absorbers, groins, and a change in the alignment of the east breakwater were tested.

38. The results of tests to determine the effects of plans 6B, 6D-6H, and 7-7C on wave reflections and wave-induced current patterns (indicated on the photographs by superimposed arrows) are presented in photographs 16-29. These photographs indicate that the 225- and 450-ft-long wave absorbers along the lakeward face of the east breakwater (plans 6D and 6E) had very little effect on reflected wave patterns, but the 675-ft-long wave absorber (plan 6F) almost completely eliminated reflected waves from this structure.

39. The installation of the two groins (plans 6G and 6H) also had little effect on reflected

wave patterns, but these structures interrupted the normal alongshore wave-induced current patterns and tended to cause the formation of eddies that should be beneficial in reducing erosion.

40. Changing the alignment of the east breakwater (plan 7) changed the direction of the reflected waves but had little or no effect on wave-induced current patterns. The addition of a 750-ft-long wave absorber to this structure (plan 7A) effectively reduced the reflected waves and tended to cause the formation of an eddy at the shoreward end of the breakwater.

41. As was the case with plans 6G and 6H, the addition of the two groins with the plan 7 breakwater configuration installed (plans 7B and 7C) had little effect on reflected waves, but current eddies were formed in the vicinity of both structures.

42. The results of wave-height measurements with plans 7 and 7A installed in the model are compared with corresponding data for plan 6B in table 8. These data reveal that plans 7 and 7A effected increases in wave heights of about 9 and 18 percent, respectively, when compared with plan 6B. It should be noted however that wave heights for plans 7 and 7A were within the specified criteria at all locations except at the entrance to the east channel (gage 5), where maximum values of 2.0 and 1.7 ft, respectively, were recorded. The specified wave-height criterion at this location is 1.5 ft.

43. Wave patterns in the harbor area for two representative test waves with plans 7 and 7A installed are presented in photographs 30-33.

Current Velocity and Water Surface Elevation Tests

44. The results of current velocity measurements with plan 1 installed in the model are presented in table 9. These data indicate that for a discharge of 27,000 cfs current velocities increased with each successive reduction in lake level and exceeded the specified velocity criteria at most of the measuring stations. Over the channel reach from sta 30+00 to 0+00 average velocities were 7.2, 8.0, and 8.4 fps for lake levels of +2.8, +1.4, and 0.0 ft, respectively. Maximum current velocities in the entrance to the harbor ranged up to 10.8 fps for a lake level of 0.0 ft. For a discharge of 20,000 cfs and a lake level of +1.4 ft, current velocities with plan 1 installed averaged about 6 fps between sta 30+00 and 0+00.

45. Water surface profiles taken with plan 1 installed are presented in plates 13-16. These data indicate no flooding conditions along the reach of the river reproduced in the model; this was verified by visual observations during model testing. Water surface elevations varied from a maximum of +5.4 ft (Q = 27,000 cfs; lake level = +2.8 ft) to a minimum of −0.5 ft (Q = 27,000 cfs; lake level = 0.0 ft).

46. The results of current velocity measurements with plans 2-6 installed in the model are presented in tables 10 and 11. For each of these plans, measurements were made using discharges of 27,000 cfs (lake level = +2.8 ft) and 20,000 cfs (lake level = +1.4 ft).

47. In an effort to restrict the flow of water, thereby raising the water level in the channel and reducing current velocities, the navigation opening between the breakwaters was reduced from 275 to 200 ft (plan 2). The results of current velocity measurements for this configuration reveal that average current velocities were approximately the same for plans 1 and 2 with a discharge of 20,000 cfs. However, for a discharge of 27,000 cfs, the installation of plan 2 reduced average current velocities by about 10 percent.

48. The water surface profiles for plan 2 (plates 17 and 18) show maximum and minimum

water surface elevations of +5.9 ft (Q = 27,000 cfs) and +0.9 ft (Q = 20,000 cfs), respectively.

49. In order to further restrict flow in the channel, the navigation opening and the channel width from sta 27+50 to 0+00 were reduced to 190 ft (plan 3). Current velocities for plan 3 show average increases over plan 1 of about 7 and 16 percent for discharges of 27,000 and 20,000 cfs, respectively. Maximum current velocities of 14.4 fps were recorded at sta 0+00 with plan 3 installed, indicating the need for breakwater toe protection in this area.

50. Water surface profiles for plan 3 are presented in plates 19 and 20 and show maximum and minimum water surface elevations of +7.0 ft (Q = 27,000 cfs) and +1.6 ft (Q = 20,000 cfs). Visual observations indicated no flooding along either bank; however, bank-full stages were noted at several locations for a discharge of 27,000 cfs.

51. Current velocity data for plans 4 and 5 (tables 10 and 11), when compared with similar data for plan 1, reveal that installation of plan 4 (190-ft channel width from sta 10+00 to 0+00) effected average reductions in current velocities in the river channel between sta 30+00 and 10+00 of about 5 and 3 percent for discharges of 27,000 and 20,000 cfs, respectively. With plan 5 installed (150-ft channel width from sta 10+00 to 0+00), current velocities between sta 30+00 and 10+00 were reduced about 17 and 8 percent for discharges of 27,000 and 20,000 cfs, respectively. Maximum current velocities of 12.4 (plan 4) and 17.3 fps (plan 5) were recorded in the harbor entrance, again indicating the need for breakwater toe protection in this area.

52. Water surface profiles for plans 4 and 5 are presented in plates 21-24. These data show maximum water surface elevations of +6.8 ft (Q = 27,000 cfs) for plan 4 and +8.1 ft (Q = 27,000 cfs) for plan 5. Visual observations indicated no flooding along either bank with plan 4 installed; however, with plan 5 installed and a discharge of 27,000 cfs, flooding was noted along the west bank from sta 30+00 to 14+50 and around the perimeter of the boat mooring basins on the west side of the channel.

53. In an effort to prevent flooding and still keep current velocities at an acceptable level, the main channel was revised to a 190-ft width, with the bottom lowered to el −14 (plan 6). A comparison of the data for plans 1 and 6 reveals that the installation of plan 6 effected average reductions in current velocities in the river channel between sta 30+00 and 10+00 of about 15 and 8 percent for discharges of 27,000 and 20,000 cfs, respectively, and current velocities were either within or close to the specified criteria. Maximum current velocities of 9.6 fps were recorded in the harbor entrance with plan 6 installed.

54. Water surface profiles for plan 6 (plates 25 and 26) show maximum water surface elevations of +4.8 and +2.8 ft for discharges of 27,000 and 20,000 cfs, respectively. Visual observations indicated no flooding with plan 6 installed.

Discussion of Test Results

55. Comparative test results for all plans tested are summarized in the following tabulation. Using the plan 1 test results as a base, percentages of increase or decrease in wave heights and current velocities are presented for the other plans. Also shown for each plan are the maximum current velocities in the harbor entrance and the maximum water surface elevations in the river channel between sta 30+00 and 0+00.

56. The values provided in the following tabulation show that considerable reductions in wave heights were realized when the channel width was reduced below that specified in the original proposed plan of improvement (plan 1). Also, in most cases, current velocities in the river channel

Plan No.	Percent Reduction in Wave Heights	Percent Change in Current Velocities in River Channel (Sta 30+00 to 10+00)	Maximum* Current Velocity in Harbor Entrance fps	Maximum* Water Surface Elevation (Sta 30+00 to 0+00), ft
1	--	--	8.7	+5.4
2	23	8 (reduction)	10.8	+5.9
3	19	7 (increase)	14.4	+7.0
4	30	4 (reduction)	12.4	+6.8
5	50	12 (reduction)	17.3	+8.1
6	19	11 (reduction)	9.6	+4.8
6A	29	--	--	--
6B	31	--	--	--
6C	27	--	--	--
7	26	--	--	--
7A	20	--	--	--

* Maximum values recorded for the design discharge of 27,000 cfs and a lake level of +2.8 ft.

between sta 30+00 and 10+00 showed some reduction. However, maximum velocities at the harbor entrance were excessive in several instances and, referring to tables 4 and 5, it is seen that in some cases wave heights in the east and main channel entrances exceeded the criteria specified, i.e., 2.5 ft at the mouth of the main river channel and 1.5 ft at the entrance to the east channel. In addition, considerable flooding occurred along the west bank when the lower channel width and navigation opening were reduced to 150 ft.

57. When the river channel was revised to a 14-ft depth and 190-ft width (plan 6), considerable improvement was seen in current velocities both in the river channel and in the harbor entrance. Also, no flooding occurred. Although the specified current velocity criteria of 6.0 fps were exceeded in a few places, it appears that satisfactory flood flow conditions will obtain in the river channel and in the harbor entrance. Although the plan 6 revision alone did not solve the wave-height problem in the entrance to the east channel, the installation of the two rubble-mound structures between the main channel and the east channel (plan 6B) resulted in wave heights that were within the criteria specified as acceptable for the installation.

58. The installation of wave absorbers along the entire lakeward face of the east breakwater (plans 6F and 7A) effectively reduced reflected waves from this structure, and the installation of groins at 500-ft intervals along the beach east of the east breakwater interrupted the normal alongshore wave-induced currents and caused the formation of eddies. Changing the alignment of the east breakwater, however, resulted in a slight increase in wave heights inside the harbor and did not significantly alter wave reflections or wave-induced current patterns.

PART V: CONCLUSIONS

59. Based on the results of the investigation reported herein, it is concluded that:

a. The most critical shallow-water directions of wave approach with respect to extreme wave heights in the proposed harbor are north 25° west and north 06° east (corresponding directions of deepwater wave approach are north 22°30′ west and north 22°30′ east).

b. Installation of the originally proposed plan of improvement (plan 1) will allow adequate channel capacity for the passage of flood flows, but current velocities in the revised channel and wave heights at several locations in the harbor will exceed the selected criteria.

c. Current velocities in the river channel and wave heights in the harbor area can be effectively reduced by sufficient reductions in the width of the navigation opening and outer channel (plan 5). However, such revisions will result in flooding along the west bank of the river during flood flows, and exceptionally large current velocities will obtain in the harbor entrance.

d. Of all improvement plans tested, plans involving a 190-ft-wide, 14-ft-deep navigation opening and lower river channel (plans 6-6H and 7-7C) will most nearly meet the selected criteria with respect to current velocities and passage of flood flows. Of these plans, only plan 6B meets the specified criteria with respect to wave protection at all locations inside the harbor.

e. The installation of a wave absorber along the entire lakeward face of the east breakwater will effectively reduce the heights of reflected waves from this structure.

f. The installation of groins at 500-ft intervals along the shore east of the east breakwater will cause the formation of eddies in the wave-induced current patterns alongshore and should help to reduce erosion.

g. Changing the alignment of the east breakwater (plans 7-7C) will result in a slight increase in wave heights inside the harbor.

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Table 1

Estimated Duration and Magnitude of Deepwater Waves*
Approaching Chagrin River from the Various Directions

Wave Height** ft	Duration, hr/yr, for Wave Periods** of 1-2 sec	2-3 sec	3-4 sec	4-5 sec	5-6 sec	6-7 sec	7-8 sec	Total
				West				
0.5- 1	10	6						16
1- 2	8	78	26					112
2- 3		62	62					124
3- 4		10	46	2				58
4- 5			28	12				40
5- 6			6	10				16
6- 7			6	14				20
7- 8								
8- 9				4	2			6
9-10				2				2
10-11					2			2
Total	18	156	174	44	4			396
				West-Northwest				
0.5- 1	20	8						28
1- 2	10	106	42					158
2- 3		60	90					150
3- 4		2	74	2				78
4- 5			38	30				68
5- 6			10	18				28
6- 7			2	14			2	18
7- 8				14				14
8- 9				2	2			4
9-10					2			2
10-11					2			2
Total	30	176	256	80	6		2	550
				Northwest				
0.5- 1	14	12						26
1- 2	4	122	10					136
2- 3		42	74					116
3- 4		2	58					60
4- 5			24	2				26
5- 6			12	8				20
6- 7				12				12
7- 8				8				8
Total	18	178	178	30				404

(Continued)

* Hindcast wave data taken from reference 10 for station B (lat. 41°35'N, long. 81°45'W), located at a water depth of 50 ft.

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

(1 of 3 sheets)

Table 1 (Continued)

Wave Height** ft	Duration, hr/yr, for Wave Periods** of 1-2 sec	2-3 sec	3-4 sec	4-5 sec	5-6 sec	6-7 sec	7-8 sec	Total
				North-Northwest				
0.5- 1	12	16						28
1- 2	8	152	4					164
2- 3		36	58					94
3- 4			56					56
4- 5			20	2				22
5- 6			2					2
6- 7				4				4
7- 8								
8- 9								
9-10					2			2
10-11								
11-12					2			2
12-13								
13-14								
14-15								
15-16								
16-17					2			2
Total	20	204	140	6	6			376
				North				
0.5- 1	22	16						38
1- 2	20	154	24					198
2- 3	2	26	76					104
3- 4		4	30					34
4- 5			6	2				8
5- 6			2	4				6
Total	44	200	138	6				388
				North-Northeast				
0.5- 1	40	34	2					76
1- 2	2	128	70					200
2- 3		30	70					100
3- 4		4	56	6				66
4- 5			18	10				28
5- 6			6	16				22
Total	42	196	222	32				492

(Continued)

Table 1 (Concluded)

Wave Height** ft	Duration, hr/yr, for Wave Periods** of 1-2 sec	2-3 sec	3-4 sec	4-5 sec	5-6 sec	6-7 sec	7-8 sec	Total
				Northeast				
0.5- 1	16	8	2					26
1- 2	4	56	42					102
2- 3		30	58	6				94
3- 4		2	38	8				48
4- 5			8	6				14
5- 6			2					2
6- 7				4				4
Total	20	96	150	24				290

Table 2

Estimated Duration and Magnitude of Shallow-Water Waves Approaching Chagrin River from the Various Directions

Wave Height* ft	Duration, hr/yr, for Wave Periods* of 1-2 sec	2-3 sec	3-4 sec	4-5 sec	5-6 sec	6-7 sec	7-8 sec	Total
				West				
0.5- 1	10	6						16
1- 2	8	78	26					112
2- 3		62	62					124
3- 4		10	46	2				58
4- 5			28	12				40
5- 6			6	10				16
6- 7			6	14				20
7- 8								
8- 9				4	2			6
9-10				2	2			4
Total	18	156	174	44	4			396
				West-Northwest				
0.5- 1	20	8						28
1- 2	10	106	42					158
2- 3		60	90					150
3- 4		2	74	2				78
4- 5			38	30				68
5- 6			10	18				28
6- 7			2	14				16
7- 8				14			2	16
8- 9				2	2			4
9-10					2			2
10-11					2			2
Total	30	176	256	80	6		2	550
				Northwest				
0.5- 1	14	12						26
1- 2	4	122	10					136
2- 3		42	74					116
3- 4		2	58					60
4- 5			24	2				26
5- 6			12	8				20
6- 7				12				12
7- 8				8				8
Total	18	178	178	30				404

(Continued)

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

Table 2 (Concluded)

Wave Height* ft	Duration, hr/yr, for Wave Periods* of 1-2 sec	2-3 sec	3-4 sec	4-5 sec	5-6 sec	6-7 sec	7-8 sec	Total
			North-Northwest					
0.5- 1	12	16						28
1- 2	8	152	4					164
2- 3		36	58					94
3- 4			56					56
4- 5			20	2				22
5- 6			2					2
6- 7				4				4
7- 8								
8- 9								
9-10					2			2
10-11								
11-12					2			2
12-13								
13-14								
14-15								
15-16					2			2
Total	20	204	140	6	6			376
			North					
0.5- 1	22	16						38
1- 2	20	154	24					198
2- 3	2	26	76					104
3- 4		4	30					34
4- 5			6	2				8
5- 6			2	4				6
Total	44	200	138	6				388
			North-Northeast					
0.5- 1	40	34	2					76
1- 2	2	128	70					200
2- 3		30	70					100
3- 4		4	56	6				66
4- 5			24	26				50
Total	42	196	222	32				492
			Northeast					
0.5- 1	16	8	2					26
1- 2	4	56	42					102
2- 3		32	96	14				142
3- 4			8	6				14
4- 5			2	4				6
Total	20	96	150	24				290

Table 3

Wave Heights Obtained with Plan 1 Installed in the Model

Test Direction	Test Wave Period sec	Test Wave Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12
			Wave Height at Indicated Gage Location, ft											
North 85°00' West	5	5	5.1	1.0	0.6	0.7	1.4	0.3	0.8	0.9	0.3	0.2	0.1	0.6
	5	10	10.1	2.5	1.0	2.2	2.0	0.3	1.0	1.0	0.4	0.3	0.3	0.8
	6	10	10.2	3.1	1.8	2.7	1.9	0.3	1.3	0.9	0.4	0.3	0.4	0.8
North 65°00' West	5	5	4.7	1.3	0.5	1.8	1.1	0.3	0.6	0.6	0.3	0.2	0.1	0.5
	5	9	7.5	2.6	0.9	2.7	1.5	0.3	1.1	0.8	0.3	0.3	0.1	0.8
	6	11	12.3	4.2	1.5	2.7	1.6	0.3	1.0	0.8	0.3	0.3	0.6	0.6
	8	8	9.7	4.9	1.5	4.0	1.5	0.5	1.7	1.4	1.3	1.1	1.3	0.9
	9	11	12.0	4.3	2.0	2.7	1.4	0.6	1.7	1.6	1.4	1.2	0.8	1.3
North 45°00' West	5	5	4.7	3.0	1.5	2.3	1.9	0.3	0.9	0.8	0.4	0.5	0.4	0.8
	5	8	8.4	6.2	1.5	4.1	2.0	0.3	1.5	1.0	0.4	0.3	0.6	1.3
	8	10	13.8	7.2	4.1	3.8	1.7	0.8	2.3	1.5	1.4	1.2	1.6	1.1
North 25°00' West	5	7	8.7	6.7	2.8	4.7	2.7	0.8	2.2	1.8	0.9	1.0	0.9	2.1
	6	10	14.3	6.6	4.7	4.2	2.3	0.5	3.0	1.7	0.9	0.6	1.0	1.3
	6	16	10.3	7.4	5.2	3.6	2.3	0.5	2.4	1.4	0.7	0.5	1.0	1.1
North 08°00' West	5	6	6.5	7.1	4.1	2.3	2.2	0.3	3.2	1.7	0.7	1.0	0.7	1.9
	9	10	13.3	6.4	3.9	4.5	2.1	0.9	3.2	1.9	1.4	1.2	1.0	1.8
North 06°00' East	5	5	6.2	7.0	5.5	2.1	1.4	0.3	2.6	1.9	0.7	0.3	0.8	1.0
	9	10	10.1	8.6	6.1	2.1	2.1	1.4	3.0	2.5	1.6	1.1	1.7	2.0
North 27°00' East	5	5	5.5	7.6	4.5	1.5	1.1	0.3	1.8	1.5	0.9	0.8	0.6	1.2

Note: Wave gage locations are shown in plate 3.

Table 4

Wave Heights (in feet) for Test Waves from North 25° West

Test Direction for Plans 2-6

Wave Gage No.	5-sec, 7-ft Test Wave					6-sec, 10-ft Test Wave					6-sec, 16-ft Test Wave				
	Plan 2	Plan 3	Plan 4	Plan 5	Plan 6	Plan 2	Plan 3	Plan 4	Plan 5	Plan 6	Plan 2	Plan 3	Plan 4	Plan 5	Plan 6
1	8.3	7.8	8.5	7.3	7.7	13.4	12.3	12.1	12.5	12.0	13.9	9.2	10.1	11.0	11.6
2	5.4	5.7	5.9	5.3	7.5	7.6	8.7	8.2	4.7	6.4	7.2	7.4	7.4	6.2	8.8
3	2.3	2.3	1.9	1.3	2.7	1.9	2.4	2.4	2.4	4.1	3.0	3.1	3.1	3.0	4.1
4	2.0	2.1	1.9	1.4	2.7	3.6	3.7	3.4	2.7	3.6	3.4	3.1	3.3	2.7	3.8
5	1.7	1.5	1.4	1.4	2.5	2.3	1.7	1.9	1.9	2.2	2.2	1.9	1.7	1.8	2.4
6	0.2	0.3	0.3	0.3	0.6	0.5	0.3	0.3	0.3	0.5	0.5	0.5	0.3	0.3	0.6
7	1.9	1.6	1.6	0.8	2.5	2.8	2.6	2.5	1.4	2.2	2.4	3.0	2.2	1.3	2.3
8	1.2	0.9	1.5	0.7	1.1	1.5	2.0	1.7	0.7	1.6	1.6	2.3	0.9	0.7	1.9
9	0.3	0.3	0.4	0.3	1.0	0.4	0.4	0.5	0.5	0.8	0.7	0.7	0.5	0.4	1.1
10	0.1	0.3	0.4	0.3	0.6	0.3	0.1	0.3	0.3	0.6	0.7	0.4	0.4	0.3	0.7
11	0.8	0.3	0.7	0.5	0.8	0.8	0.4	0.7	0.5	0.5	0.7	0.4	0.7	0.3	0.7
12	0.7	1.0	0.9	0.4	1.0	1.0	0.9	1.2	0.4	0.7	1.2	1.2	0.6	0.4	0.9

Note: Wave gage locations are shown in plates 3-7.

Table 5

Wave Heights (in feet) for Test Waves from North 06° East

Test Direction for Plans 2-6

Wave Gage No.	5-sec, 5-ft Test Wave Plan 2	Plan 3	Plan 4	Plan 5	Plan 6	9-sec, 10-ft Test Wave Plan 2	Plan 3	Plan 4	Plan 5	Plan 6
1	6.0	6.3	6.0	6.2	5.3	10.3	10.1	12.4	13.3	12.1
2	7.0	8.7	7.2	5.8	4.9	6.9	7.6	7.6	8.0	8.3
3	4.5	5.0	4.5	3.2	4.9	3.3	3.8	4.3	2.9	4.5
4	1.2	2.2	2.3	1.3	1.2	4.3	5.2	2.6	2.6	3.6
5	1.3	1.4	1.6	0.9	1.2	2.6	3.3	1.6	1.5	1.9
6	0.3	0.3	0.3	0.3	0.3	1.3	1.0	0.4	0.4	0.6
7	2.0	2.1	1.8	1.3	1.2	1.7	2.3	1.7	0.8	2.3
8	1.5	1.9	1.5	0.7	0.7	1.6	2.3	1.3	0.5	1.6
9	0.3	0.4	0.6	0.3	0.7	1.5	0.9	0.9	0.5	1.1
10	0.3	0.8	0.5	0.3	0.5	0.7	0.4	0.4	0.4	0.4
11	1.0	1.1	0.8	0.4	0.7	1.2	1.5	0.8	0.4	0.9
12	0.9	1.2	0.8	0.8	0.8	1.7	1.7	1.0	0.4	0.7

Note: Wave gage locations are shown in plates 3-7.

Table 6

Wave Heights (in feet) for Plans 6, 6A, 6B, and 6C

Test Wave		Test	Gage 4				Gage 5				Gage 6				Gage 7			
Period sec	Height ft	Direction	Plan 6	Plan 6A	Plan 6B	Plan 6C	Plan 6	Plan 6A	Plan 6B	Plan 6C	Plan 6	Plan 6A	Plan 6B	Plan 6C	Plan 6	Plan 6A	Plan 6B	Plan 6C
5	5	N06°E	1.2	1.2	1.1	1.1	1.2	0.9	0.7	1.0	0.3	0.3	0.3	0.3	1.2	1.5	1.6	1.6
9	10	N06°E	3.6	3.3	3.3	3.2	1.9	1.3	0.9	1.8	0.6	0.4	0.3	0.3	2.3	1.8	1.8	2.3
5	7	N25°W	2.7	2.3	2.0	2.1	2.5	1.7	1.3	1.5	0.6	0.3	0.3	0.3	2.5	2.8	2.5	2.4
6	10	N25°W	3.6	2.3	2.9	3.0	2.2	1.2	0.9	1.8	0.5	0.3	0.3	0.3	2.2	2.2	2.5	2.4
6	16	N25°W	3.8	3.0	3.0	3.0	2.4	1.4	1.0	1.8	0.6	0.3	0.3	0.3	2.3	2.6	2.4	1.8

Note: Wave gage locations are shown in plates 7 and 8.

Table 7

Wave Heights Obtained with Plan 6B Installed in the Model

Test Direction	Test Wave Period sec	Test Wave Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12
North 85°00'West	5	5	4.5	1.5	0.5	0.5	0.4	0.2	0.4	0.4	0.3	0.3	0.3	0.3
	5	10	9.8	3.0	1.3	1.7	0.9	0.3	1.0	0.5	0.3	0.3	0.4	0.4
	6	10	12.2	3.2	1.8	1.9	0.8	0.2	0.8	0.7	0.3	0.3	0.3	0.3
North 65°00'West	5	5	5.4	1.8	0.5	0.7	0.5	0.2	0.5	0.3	0.3	0.3	0.3	0.3
	5	9	7.8	4.0	1.4	1.4	1.0	0.3	1.3	0.7	0.4	0.3	0.4	0.6
	6	11	11.8	3.3	1.8	1.7	0.8	0.3	1.0	0.4	0.3	0.3	0.3	0.3
	8	8	9.4	3.4	1.7	1.9	0.6	0.3	1.2	0.9	0.5	0.4	0.5	0.4
	9	11	11.2	4.2	2.4	2.1	0.7	0.4	1.2	0.8	0.4	0.3	0.5	0.4
North 45°00'West	5	5	4.9	2.9	0.7	0.8	0.8	0.2	0.7	0.4	0.4	0.3	0.5	0.6
	5	8	9.3	5.7	2.2	1.8	1.4	0.3	1.7	0.7	0.4	0.3	0.8	0.7
	8	10	12.4	6.7	4.2	3.1	1.0	0.4	1.9	1.0	0.5	0.4	0.5	0.7
North 25°00'West	5	7	7.5	9.3	2.7	2.0	1.3	0.3	2.5	1.1	0.5	0.3	1.1	1.2
	6	10	12.3	6.9	4.0	2.9	0.9	0.3	2.5	1.3	0.5	0.3	0.7	1.0
	6	16	11.1	7.0	3.5	3.0	1.0	0.3	2.4	1.1	0.4	0.3	0.5	0.6
North 08°00'West	5	6	5.8	7.0	4.5	2.4	1.0	0.3	1.8	0.7	0.5	0.4	0.7	1.3
	9	10	11.0	6.3	3.9	3.8	1.1	0.6	1.5	1.5	0.7	0.6	0.8	1.1
North 06°00'East	5	5	5.1	4.1	3.3	1.1	0.7	0.3	1.6	0.7	0.4	0.5	0.4	0.9
	9	10	11.1	7.9	6.0	3.3	0.9	0.3	1.8	1.6	0.8	0.3	0.8	0.7
North 27°00'East	5	5	6.8	5.2	2.3	1.9	0.7	0.3	1.5	0.5	0.4	0.3	0.4	0.6

Note: Wave gage locations are shown in plate 7.

Table 8

Wave Heights (in feet) for Plans 6B, 7, and 7A

Wave Gage No.	North 25° West Test Direction									North 06° East Test Direction					
	5-sec, 7-ft Test Wave			6-sec, 10-ft Test Wave			6-sec, 16-ft Test Wave			5-sec, 5-ft Test Wave			9-sec, 10-ft Test Wave		
	Plan 6B	Plan 7	Plan 7A	Plan 6B	Plan 7	Plan 7A	Plan 6B	Plan 7	Plan 7A	Plan 6B	Plan 7	Plan 7A	Plan 6B	Plan 7	Plan 7A
1	7.5	7.2	7.5	12.3	11.8	11.4	11.1	11.1	12.0	5.1	6.4	5.6	11.1	13.1	12.6
2	9.3	6.9	5.9	6.9	5.5	7.2	7.0	8.0	7.5	4.1	4.0	4.7	7.9	8.9	8.5
3	2.7	1.8	1.7	4.0	3.6	4.8	3.5	4.3	4.2	3.3	4.6	4.6	6.0	4.5	4.0
4	2.0	2.7	2.7	2.9	2.9	3.5	3.0	3.3	3.1	1.1	1.9	1.7	3.3	2.5	2.3
5	1.3	2.0	1.7	0.9	0.8	1.2	1.0	1.2	0.9	0.7	0.8	0.9	0.9	1.0	0.9
6	0.3	0.3	0.8	0.3	0.3	0.5	0.3	0.5	0.6	0.3	0.3	0.5	0.3	0.5	0.5
7	2.5	1.9	2.1	2.5	1.5	2.4	2.4	2.2	2.2	1.6	1.9	1.9	1.8	2.2	2.4
8	1.1	1.1	1.1	1.3	1.5	2.3	1.1	2.2	2.2	0.7	0.8	1.0	1.6	1.5	2.0
9	0.5	0.6	0.6	0.5	1.0	1.3	0.4	1.0	1.0	0.4	0.6	0.7	0.8	0.8	1.1
10	0.3	0.5	0.5	0.3	0.9	0.8	0.3	0.8	0.8	0.5	0.6	0.6	0.3	0.4	0.3
11	1.1	1.0	1.1	0.7	0.8	1.1	0.5	1.1	1.0	0.4	0.6	0.9	0.8	0.8	0.8
12	1.2	1.5	1.5	1.0	0.6	1.1	0.6	0.9	0.9	0.9	1.4	1.4	0.7	0.7	1.0

Note: Wave gage locations are shown in plate 7.

Table 9

Current Velocities (fps) for Plan 1

	Discharge = 27,000 cfs									Discharge = 20,000 cfs		
	West Bank			Channel Center Line			East Bank			West Bank	Channel Center Line	East Bank
	swl	swl	swl	swl	swl	swl	swl	swl	swl	swl	swl	swl
Station	0.0	+1.4	+2.8	0.0	+1.4	+2.8	0.0	+1.4	+2.8	+1.4	+1.4	+1.4
30+00	7.9	8.7	7.2	7.2	7.9	7.2	8.7	8.7	8.7	7.2	6.4	8.7
28+00	7.2	7.2	6.7	7.2	6.7	6.7	8.7	7.9	7.2	5.4	5.8	6.2
26+00	6.7	6.7	6.7	7.2	7.2	6.7	8.7	8.7	7.2	5.4	5.8	6.4
24+00	6.7	6.7	6.2	7.2	7.2	7.2	7.9	7.2	7.2	5.4	6.0	5.1
22+00	7.9	6.7	7.2	8.7	8.7	7.2	8.7	8.7	7.2	5.8	5.8	6.9
20+00	7.9	8.7	7.2	8.7	8.7	8.7	8.7	8.7	7.2	6.2	6.7	6.7
18+00	8.7	8.7	7.2	8.7	8.7	8.7	7.9	8.7	5.8	5.8	6.7	6.2
16+00	10.8	8.7	8.7	8.7	8.7	8.7	5.8	5.1	4.3	7.9	7.2	3.5
14+00	10.8	9.6	8.7	9.6	8.7	8.7	5.8	5.1	5.1	6.7	6.9	4.3
12+00	10.8	8.7	7.2	8.7	9.6	8.7	7.9	6.7	5.8	6.2	6.7	5.3
10+00	10.8	9.6	7.2	9.6	8.7	7.9	8.7	7.9	7.2	5.4	6.2	5.1
8+00	8.7	8.7	5.8	9.6	8.7	8.7	8.7	6.7	6.7	6.0	6.0	5.8
6+00	7.9	6.7	5.8	8.7	8.7	7.2	7.9	6.7	5.8	4.0	5.1	5.1
4+00	6.7	6.7	5.1	7.2	8.7	6.7	7.9	6.7	6.7	5.6	5.4	4.8
2+00	8.7	8.7	8.7	9.6	8.7	8.7	10.8	8.7	8.7	6.2	6.7	6.7
0+00	9.6	8.7	8.7	9.6	9.6	8.7	eddy	eddy	eddy	7.2	7.2	eddy

Table 10

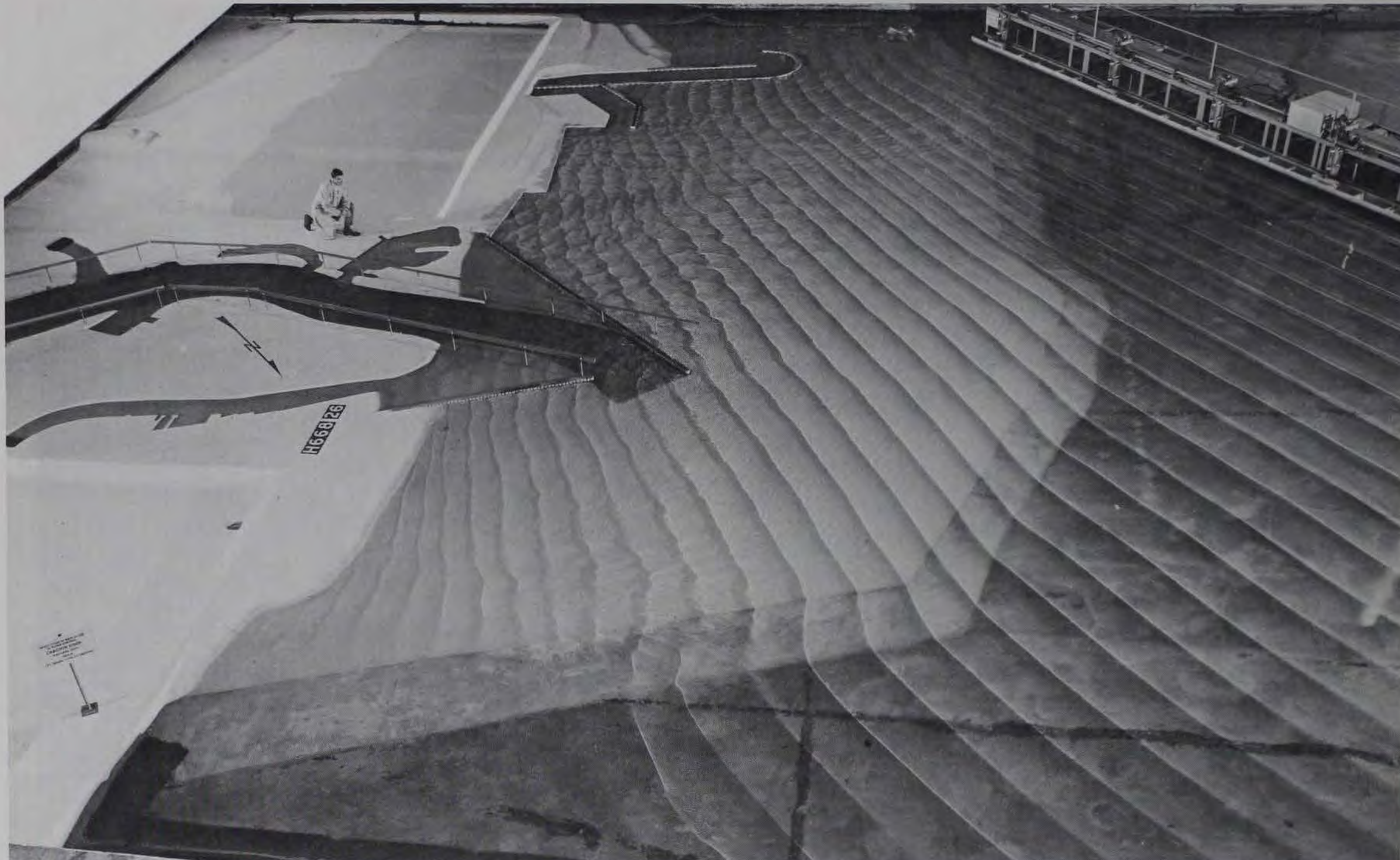
Current Velocities (fps) for Plans 2-6 for a Discharge of 27,000 cfs and a Lake Level of +2.8 ft

	West Bank					Channel Center Line					East Bank				
Station	Plan 2	Plan 3	Plan 4	Plan 5	Plan 6	Plan 2	Plan 3	Plan 4	Plan 5	Plan 6	Plan 2	Plan 3	Plan 4	Plan 5	Plan 6
30+00	6.7	7.2	6.2	5.4	7.2	6.2	7.2	6.2	5.4	7.2	7.2	8.7	7.2	5.4	7.2
28+00	6.7	6.7	6.7	5.4	7.2	6.2	7.2	6.7	5.4	7.2	6.7	8.7	7.2	6.2	7.2
26+00	5.8	6.2	6.2	5.4	6.7	5.8	7.9	6.7	5.8	6.2	7.2	8.7	7.9	6.2	7.2
24+00	5.4	7.2	6.2	4.8	5.8	6.2	7.2	6.2	5.8	5.8	5.8	6.2	6.7	5.4	5.8
22+00	6.2	7.2	6.7	5.8	5.8	7.2	7.9	7.2	6.2	6.2	6.2	5.8	7.2	5.4	5.4
20+00	6.2	7.2	7.2	5.4	5.8	7.2	7.9	7.2	6.2	6.2	6.7	7.2	7.2	6.2	5.1
18+00	6.7	7.2	7.2	5.4	5.4	6.7	7.9	7.9	6.2	6.7	5.4	6.2	5.8	5.4	5.1
16+00	7.2	8.7	7.9	7.2	6.7	7.2	8.7	7.2	6.7	6.2	4.3	5.1	4.3	2.9	3.6
14+00	7.2	8.7	7.9	7.2	7.2	6.7	8.7	7.2	6.2	6.7	4.3	6.7	4.3	3.6	3.9
12+00	6.7	8.7	7.9	7.2	6.7	7.2	8.7	7.2	7.2	6.7	5.4	7.2	4.8	4.6	5.1
10+00	6.2	8.7	8.7	8.7	6.7	6.7	7.9	7.9	8.7	6.7	5.4	6.2	7.9	8.7	6.2
8+00	6.2	7.9	7.9	8.7	6.2	7.2	7.9	8.7	8.7	6.7	5.8	6.7	7.2	7.9	5.8
6+00	5.8	5.4	7.9	8.7	5.8	5.8	7.2	8.7	8.7	7.2	5.8	7.2	7.2	7.9	5.8
4+00	5.4	6.2	6.7	8.7	5.8	5.8	7.2	7.2	8.7	6.2	5.8	6.2	6.7	7.9	5.8
2+00	8.7	9.6	10.8	10.8	7.9	8.7	10.8	9.6	10.8	7.9	8.7	9.6	10.8	10.8	7.9
0+00	10.8	12.4	12.4	17.3	9.6	10.8	14.4	12.4	17.3	9.6	eddy	eddy	eddy	eddy	eddy

Table 11

Current Velocities (fps) for Plans 2-6 for a Discharge of 20,000 cfs and a Lake Level of +1.4 ft

	West Bank					Channel Center Line					East Bank				
Station	Plan 2	Plan 3	Plan 4	Plan 5	Plan 6	Plan 2	Plan 3	Plan 4	Plan 5	Plan 6	Plan 2	Plan 3	Plan 4	Plan 5	Plan 6
30+00	6.7	6.2	6.2	5.4	6.7	5.8	6.2	5.8	5.4	6.7	6.7	7.2	6.2	5.4	6.7
28+00	5.8	7.2	6.2	5.4	6.7	5.8	7.2	5.8	5.4	6.7	6.7	7.2	6.2	5.4	6.7
26+00	5.8	6.2	5.4	5.1	5.8	5.8	7.2	5.8	5.8	5.8	6.2	7.2	6.7	5.8	6.7
24+00	4.8	6.2	5.4	4.8	5.1	5.8	7.2	5.8	5.4	5.1	4.8	6.7	5.4	5.4	5.1
22+00	5.8	6.2	6.2	5.4	5.1	6.2	5.8	5.8	5.4	5.1	5.4	5.4	6.2	5.4	5.1
20+00	5.8	6.7	5.8	5.4	5.1	7.2	7.2	6.2	6.2	5.1	6.2	6.7	6.2	5.4	5.1
18+00	6.2	6.2	5.8	5.4	5.1	6.2	6.7	6.7	5.4	5.8	5.4	5.4	5.4	5.1	4.6
16+00	7.2	7.9	7.2	6.2	5.8	6.2	8.7	6.7	6.2	5.8	3.9	4.8	3.6	3.0	3.5
14+00	7.2	7.2	6.7	7.2	5.8	7.2	7.2	6.2	5.8	6.2	3.9	6.7	3.1	3.3	3.8
12+00	6.2	7.2	6.2	6.7	6.2	6.2	8.7	7.2	5.8	6.2	4.8	6.7	4.3	3.5	4.8
10+00	6.2	7.2	6.7	8.7	6.7	6.2	7.9	7.2	8.7	6.7	5.4	6.2	7.2	7.9	5.4
8+00	5.4	6.7	7.2	8.7	5.8	5.8	7.2	7.2	8.7	6.2	5.4	6.2	6.2	8.7	5.1
6+00	4.8	5.8	6.2	7.9	5.8	5.4	7.2	6.2	8.7	5.8	5.4	6.2	5.8	7.2	5.1
4+00	4.8	5.4	5.4	7.2	5.1	5.1	6.2	6.2	7.9	6.2	4.6	5.4	6.2	7.9	5.8
2+00	8.7	8.7	9.6	10.8	6.2	8.7	8.7	8.7	9.6	6.7	8.7	10.8	8.7	12.4	5.8
0+00	10.8	10.8	10.8	12.4	8.7	8.7	10.8	10.8	10.8	8.7	eddy	eddy	eddy	eddy	eddy

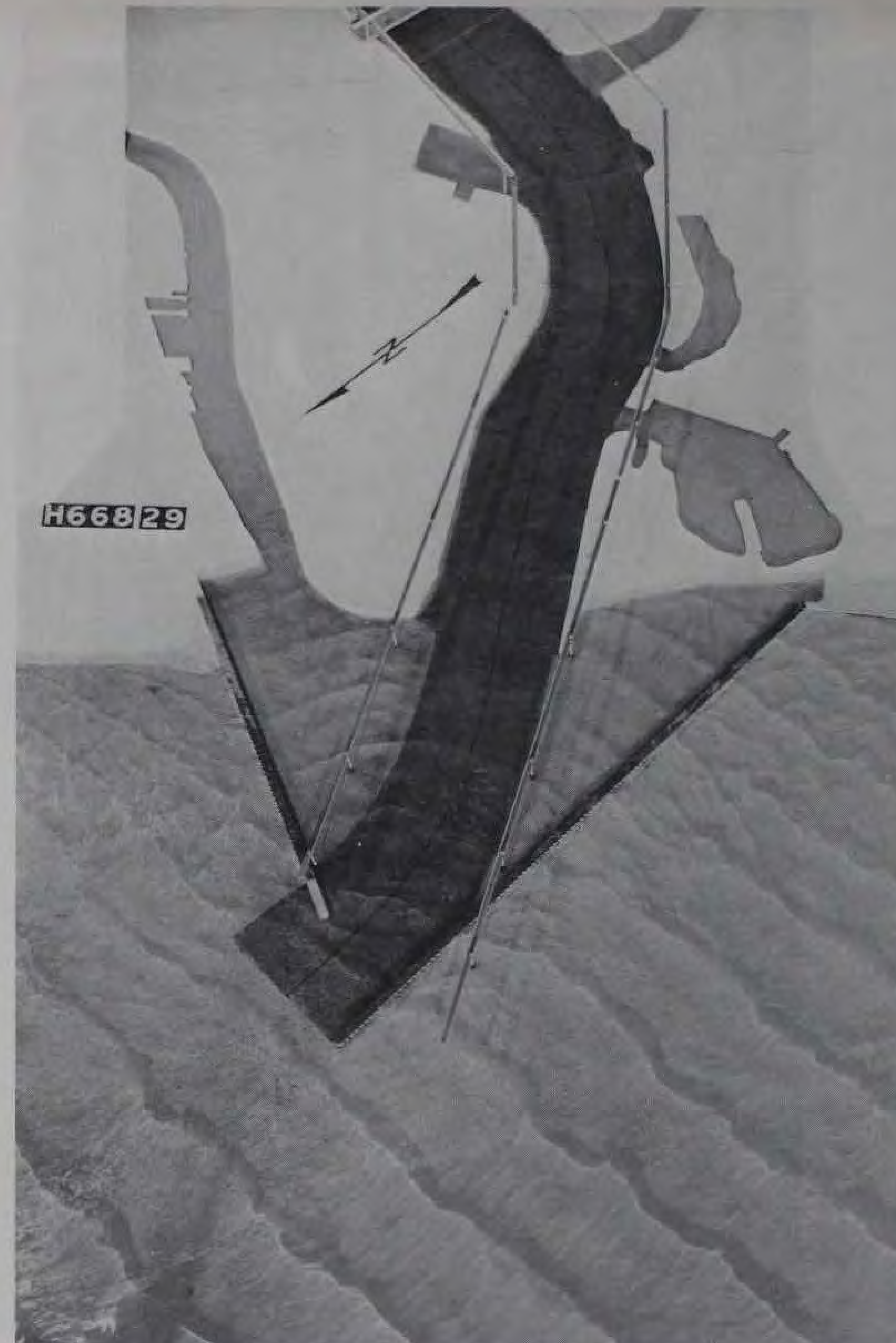


Photograph 1. General view of model; plan 1 installed

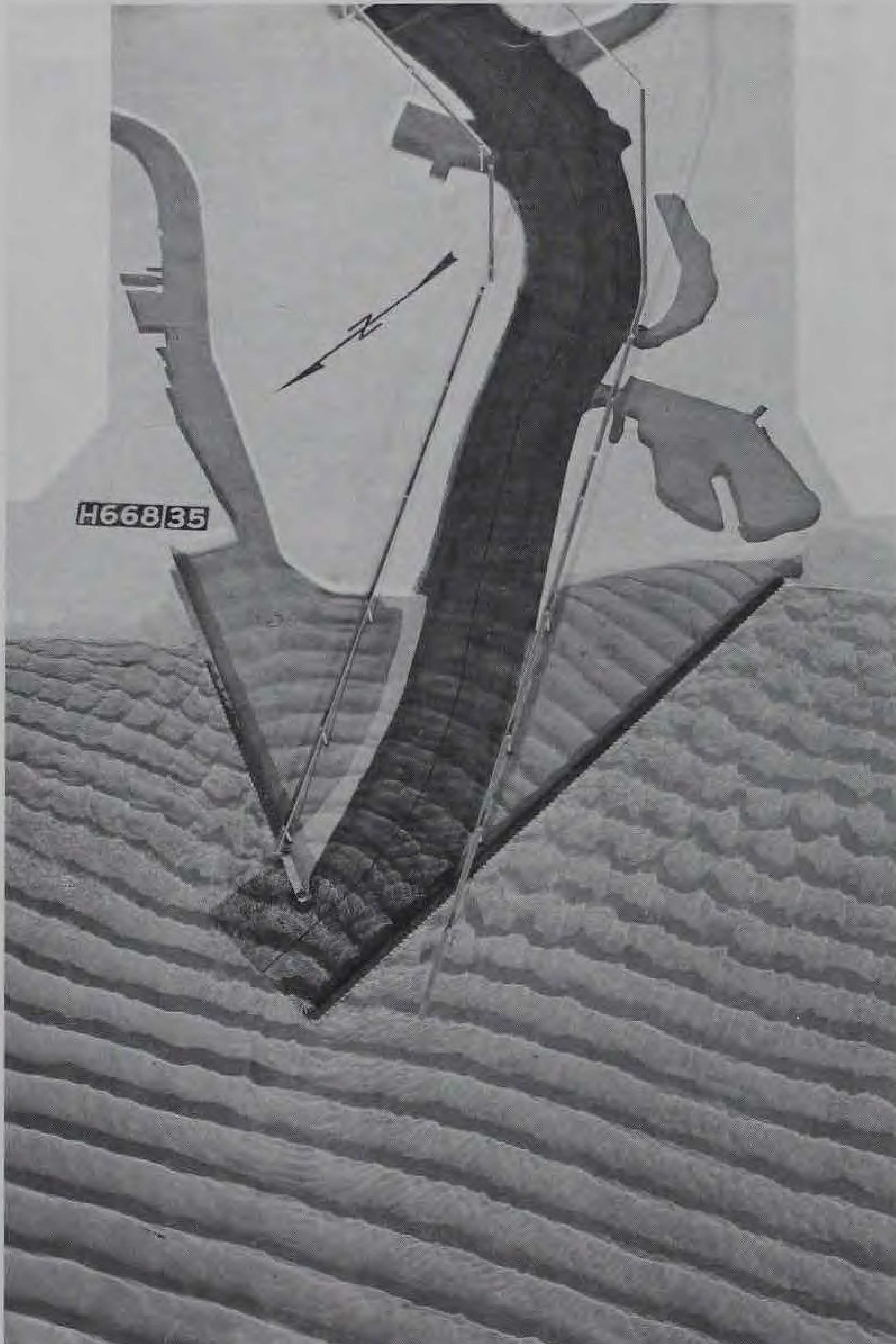
Photograph 2. Plan 1; 5-sec, 7-ft waves from north 25° west

Photograph 3. Plan 1; 9-sec, 10-ft waves from north 06° east

Photograph 4. Plan 2; 5-sec, 5-ft waves from north 06° east

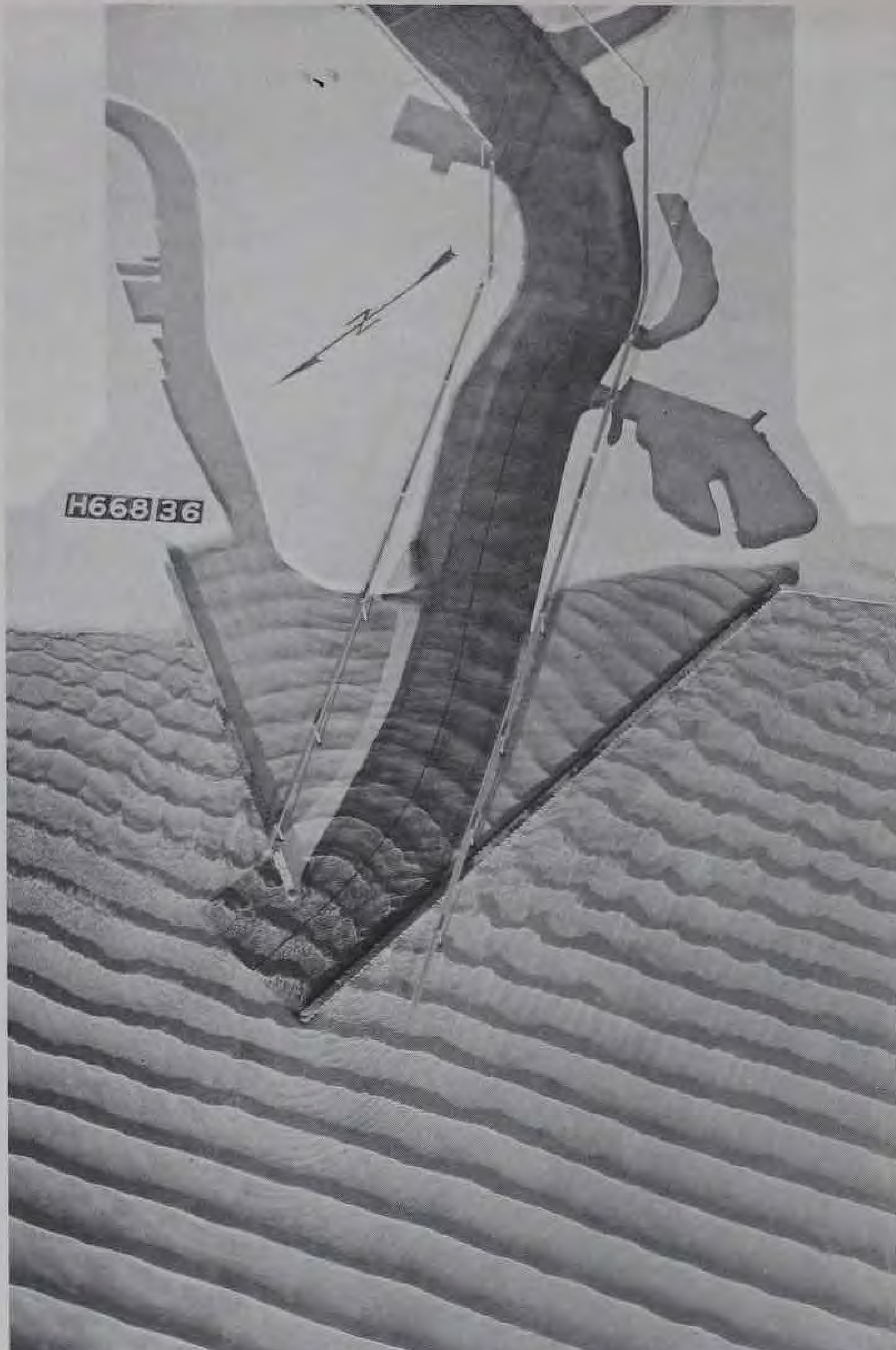


Photograph 5. Plan 2; 9-sec, 10-ft waves from north 06° east



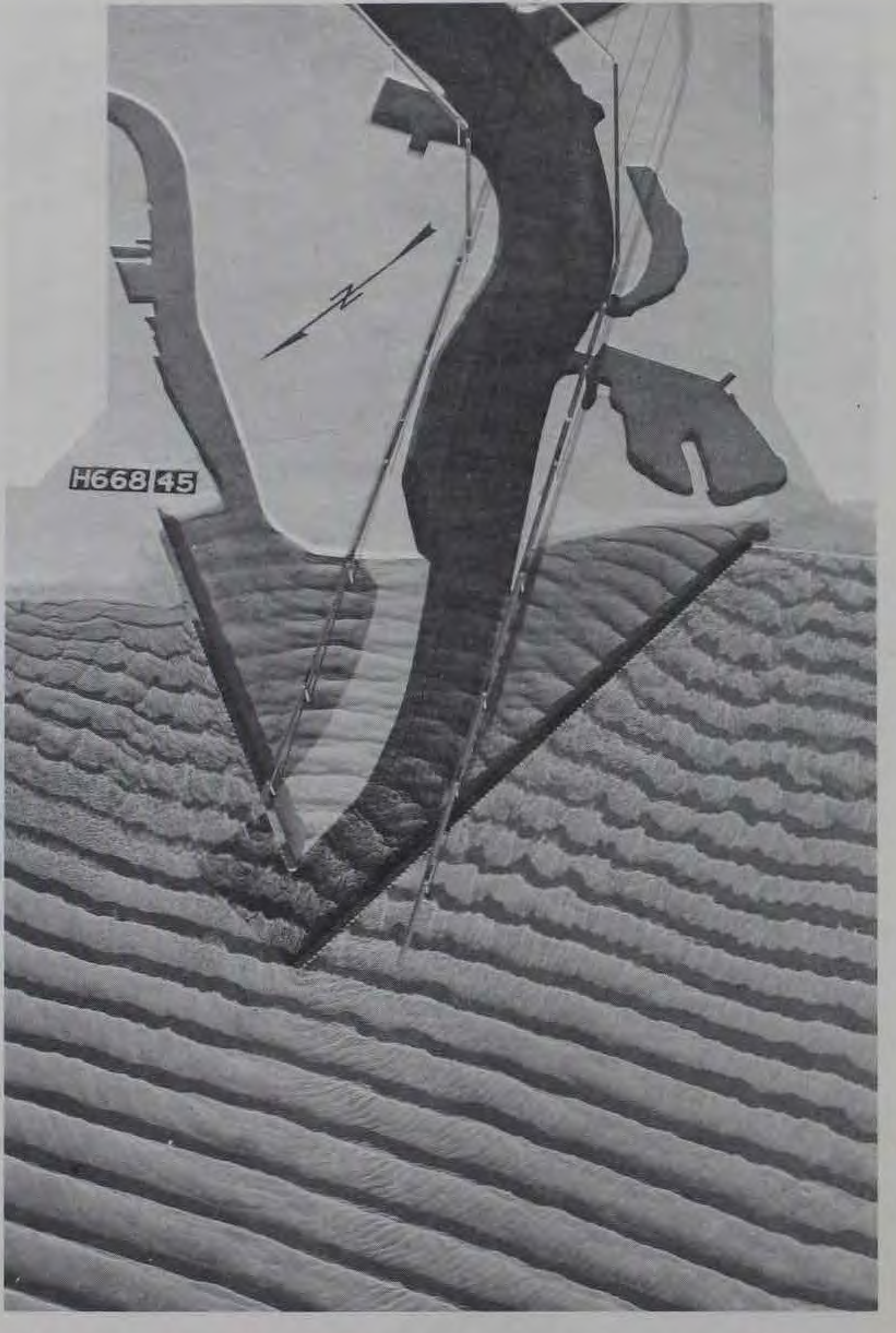
Photograph 6. Plan 3; 5-sec, 7-ft waves from north 25° west

Photograph 7. Plan 3; 9-sec, 10-ft waves from north 06° east



Photograph 8. Plan 4; 5-sec, 7-ft waves from north 25° west

Photograph 9. Plan 4; 9-sec, 10-ft waves from north 06° east



Photograph 10. Plan 5; 5-sec, 7-ft waves from north 25° west

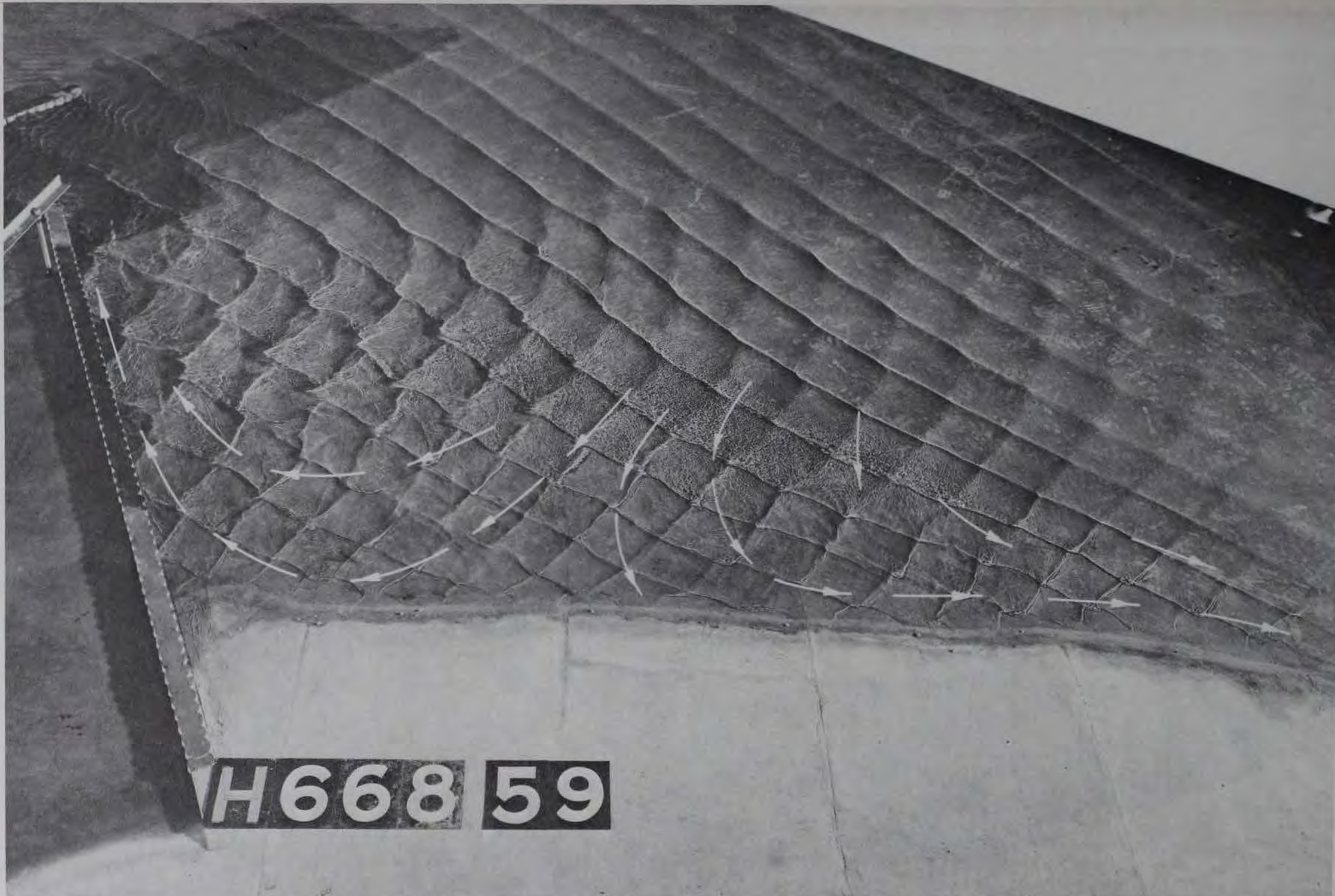
Photograph 11. Plan 5; 9-sec, 10-ft waves from north 06° east

Photograph 12. Plan 6; 5-sec, 7-ft waves from north 25° west

Photograph 13. Plan 6; 9-sec, 10-ft waves from north 06° east

Photograph 14. Plan 6B; 5-sec, 7-ft waves from north 25° west

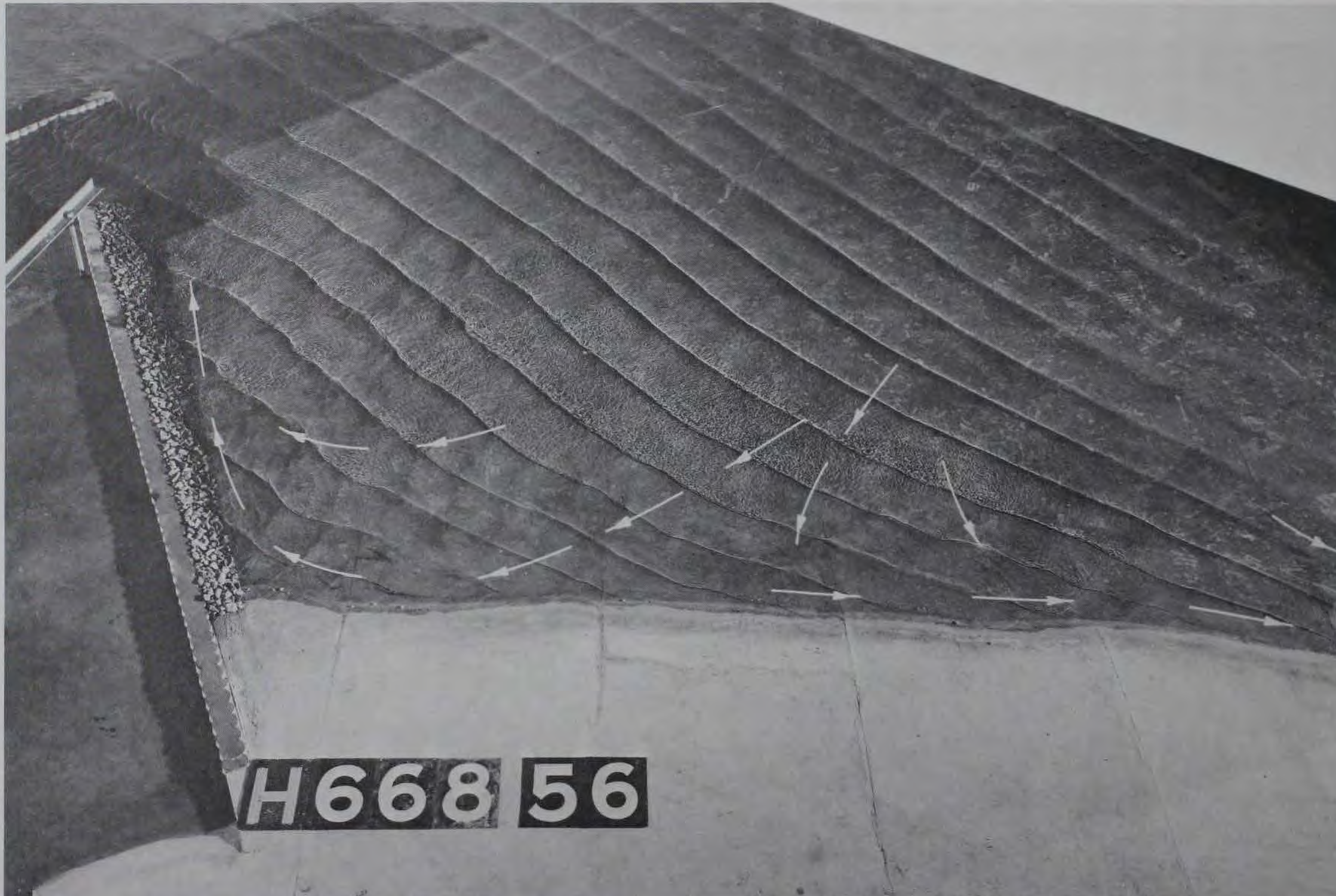
Photograph 15. Plan 6B; 9-sec, 10-ft waves from north 06° east



Photograph 16. Plan 6B; 5-sec, 5-ft waves from north 06° east

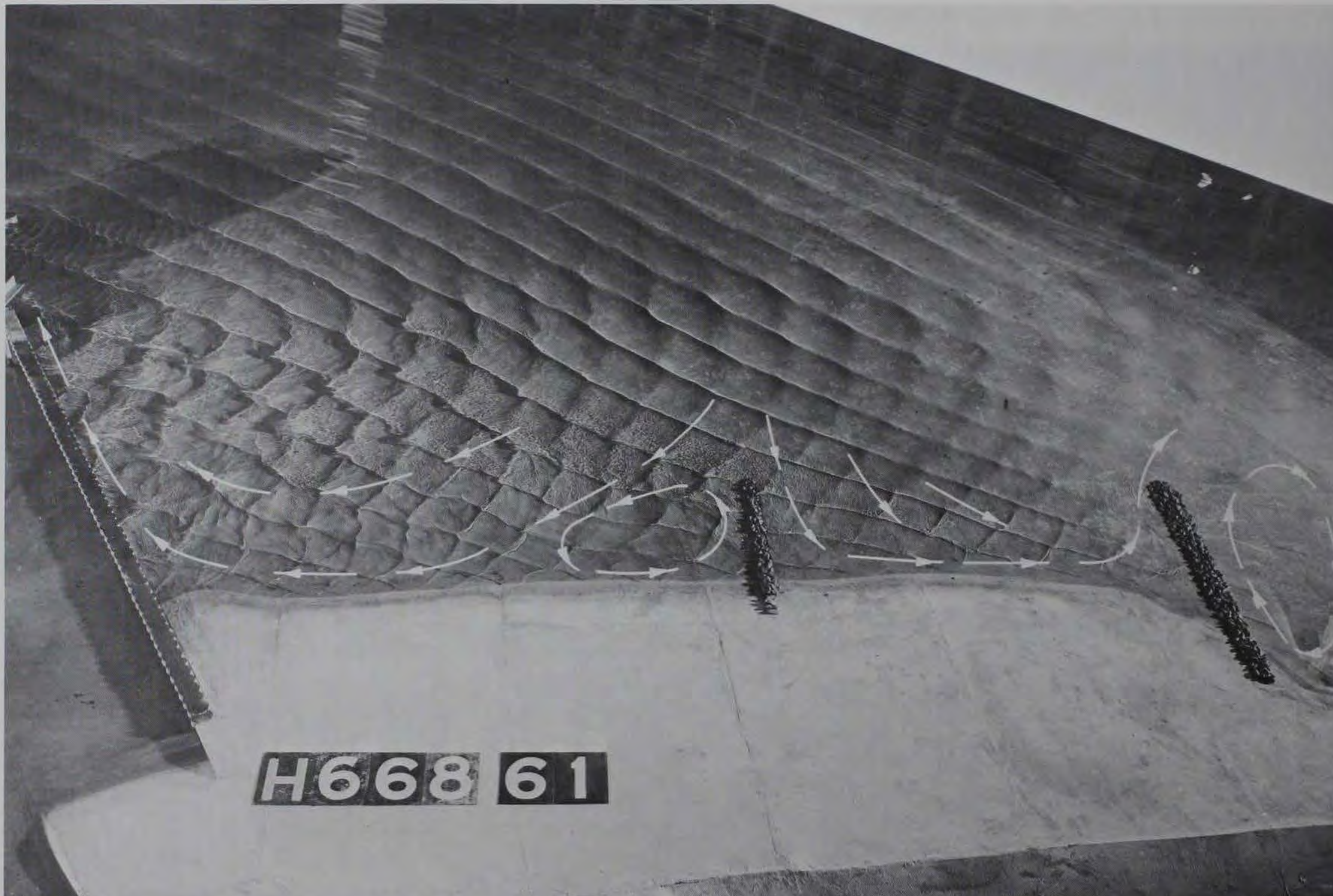
Photograph 17. Plan 6D; 5-sec, 5-ft waves from north 06° east

Photograph 18. Plan 6E; 5-sec, 5-ft waves from north 06° east



Photograph 19. Plan 6F; 5-sec, 5-ft waves from north 06° east

Photograph 20. Plan 6G; 5-sec, 5-ft waves from north 06° east



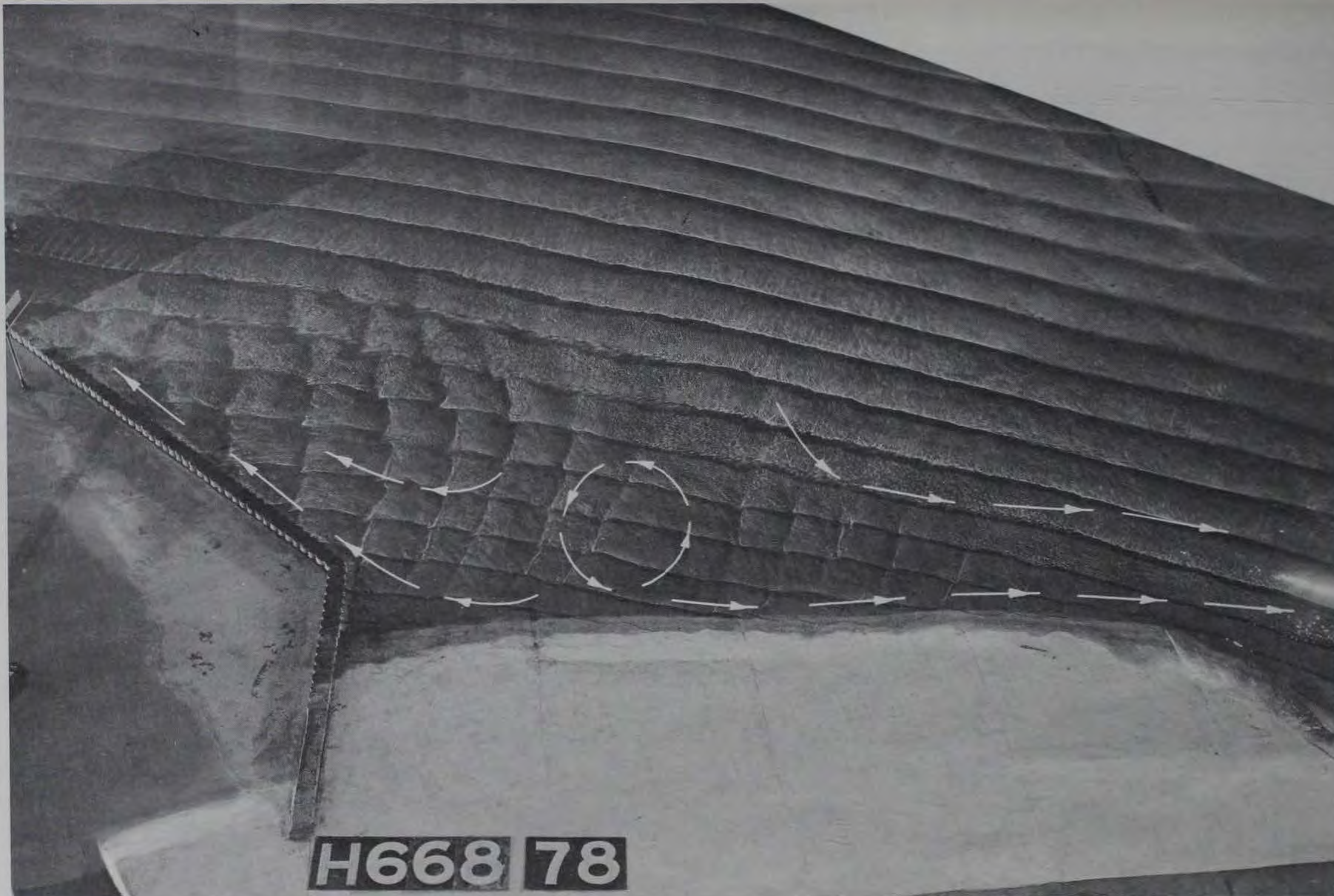
Photograph 21. Plan 6H; 5-sec, 5-ft waves from north 06° east

Photograph 22. Plan 7; 5-sec, 5-ft waves from north 06° east

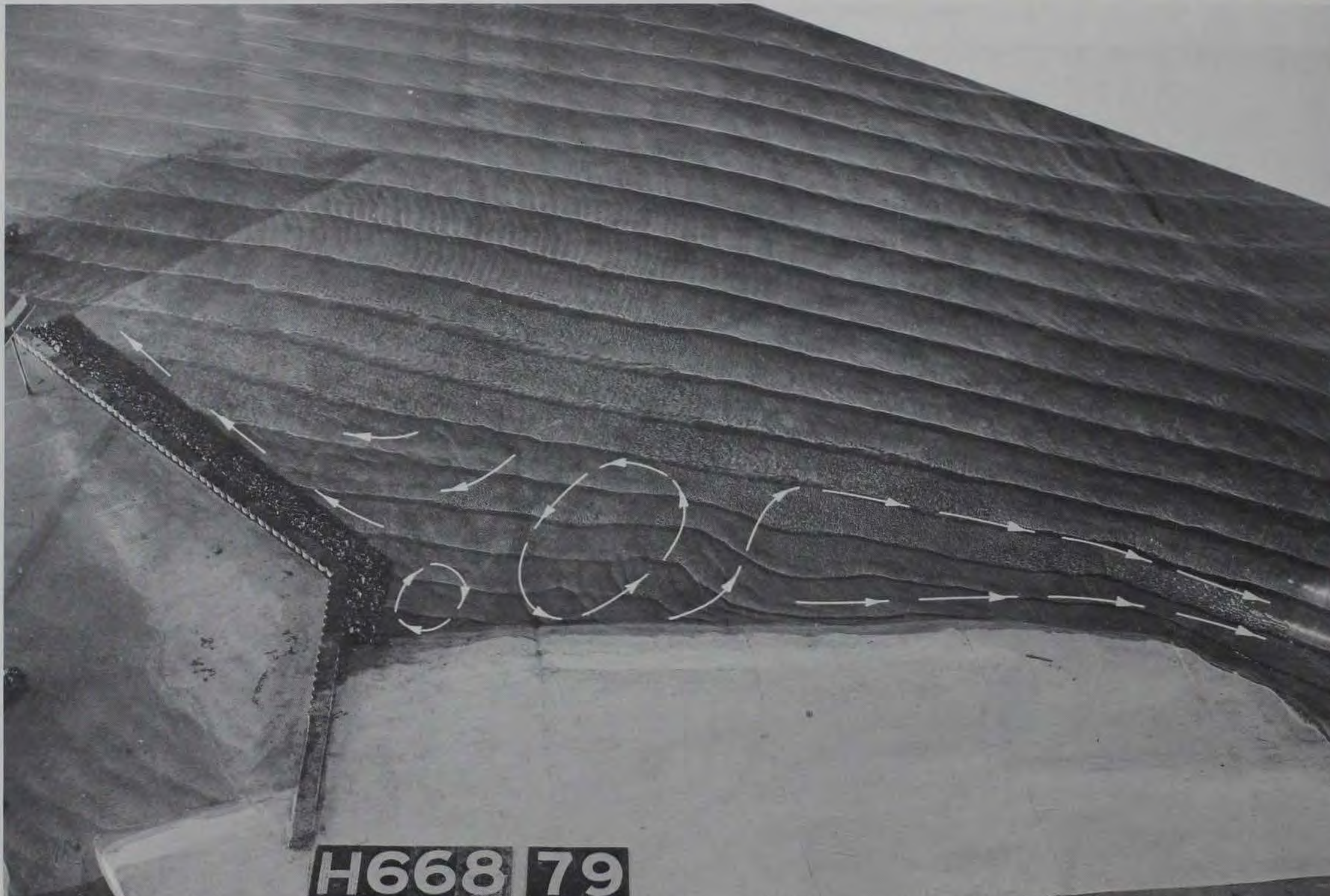
Photograph 23. Plan 7A; 5-sec, 5-ft waves from north 06° east

Photograph 24. Plan 7B; 5-sec, 5-ft waves from north 06° east

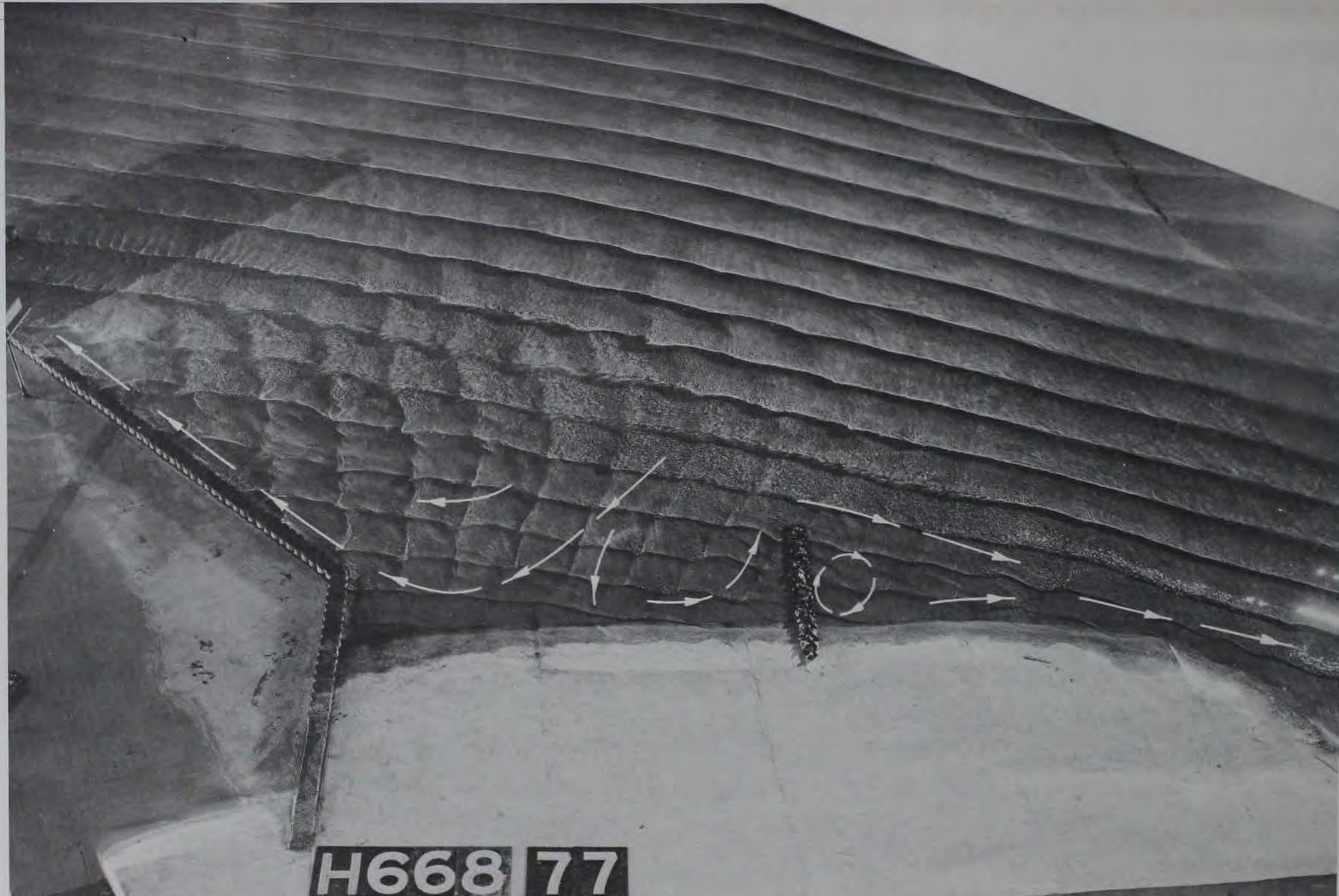
Photograph 25. Plan 7C; 5-sec, 5-ft waves from north 06° east



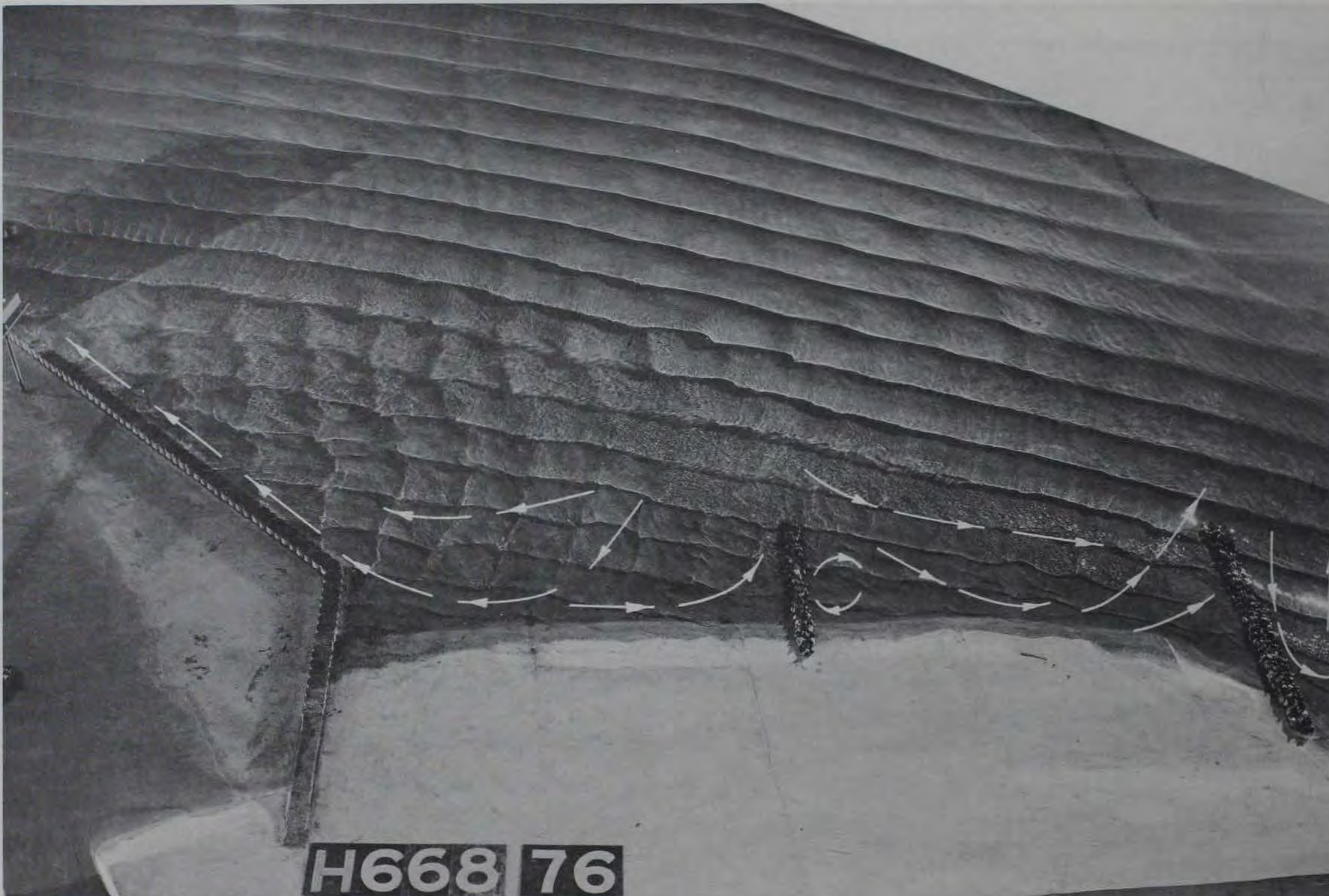
Photograph 26. Plan 7; 5-sec, 7-ft waves from north 25° west



Photograph 27. Plan 7A; 5-sec, 7-ft waves from north 25° west



Photograph 28. Plan 7B; 5-sec, 7-ft waves from north 25° west

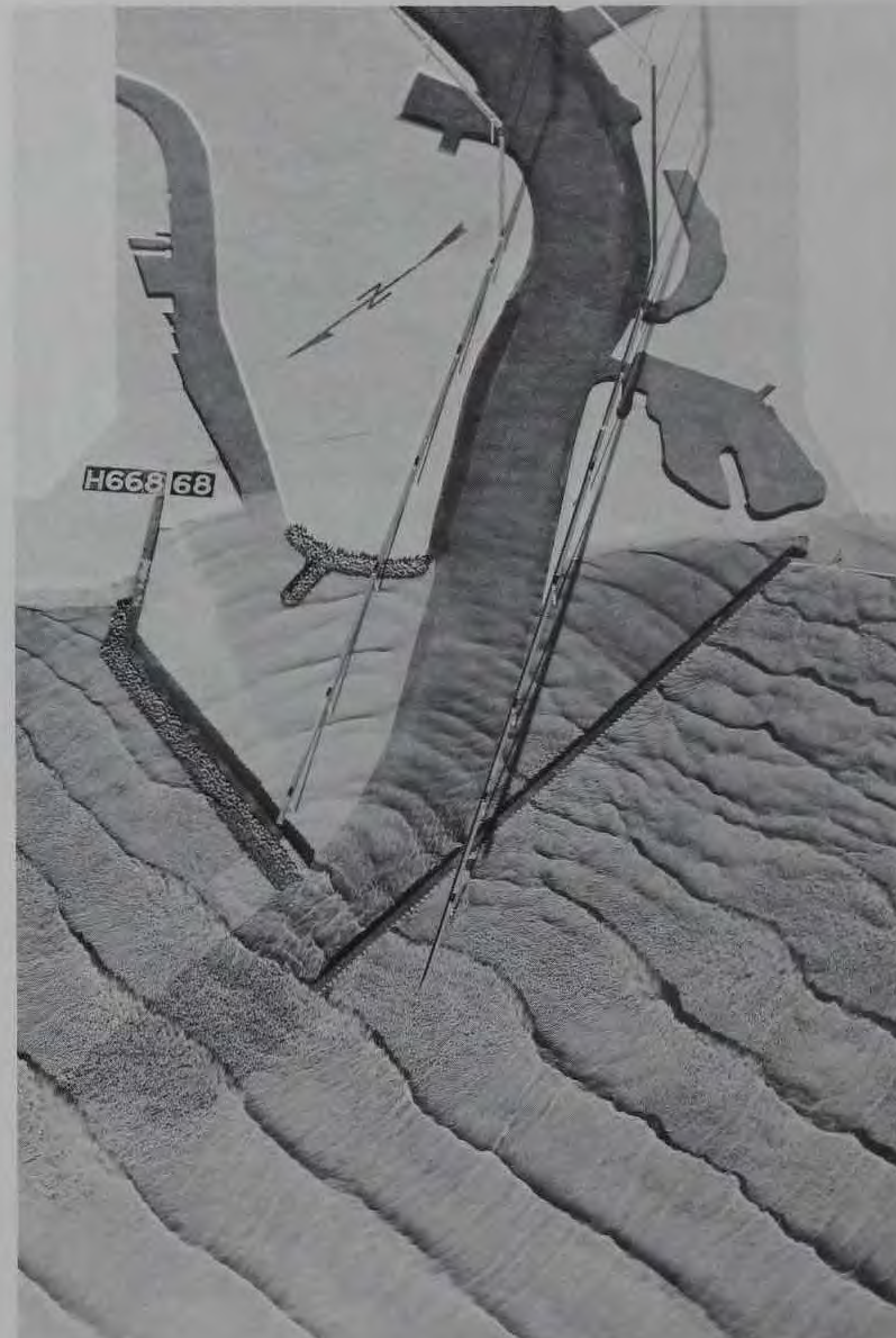


Photograph 29. Plan 7C; 5-sec, 7-ft waves from north 25° west

Photograph 30. Plan 7; 5-sec, 7-ft waves from north 25° west

Photograph 31. Plan 7; 9-sec, 10-ft waves from north 06° east

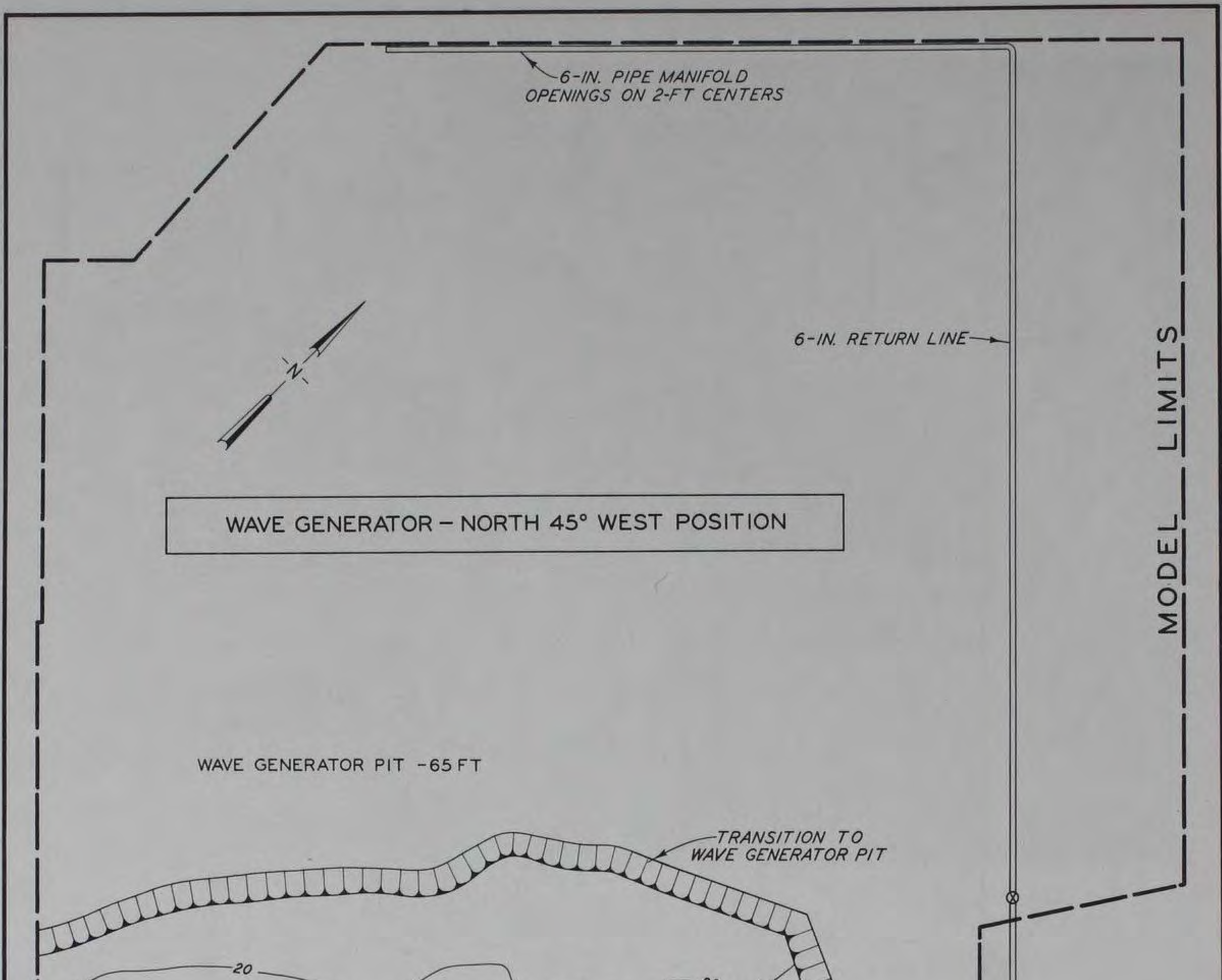
Photograph 32. Plan 7A; 5-sec, 7-ft waves from north 25° west



Photograph 33. Plan 7A; 9-sec, 10-ft waves from north 06° east

PLATE I


43° 00'
83° 00'
82° 00'
81° 00'
80° 00'
42° 00'
MICHIGAN
LAKE HURON
ONTARIO
N
DETROIT
LAKE
ST CLAIR
BUFFALO
CANADA
UNITED STATES
NEW YORK
LAKE ERIE
ERIE
TOLEDO
Eastlake
PENNSYLVANIA
CHAGRIN R.
Sandusky
CLEVELAND
OHIO
VICINITY MAP
SCALE IN MILES
20
0
20
40



NOTE: CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO LOW WATER DATUM (LWD) ELEVATION 568.6 FEET ABOVE MEAN WATER LEVEL AT FATHER POINT, QUEBEC (IGLD 1955).

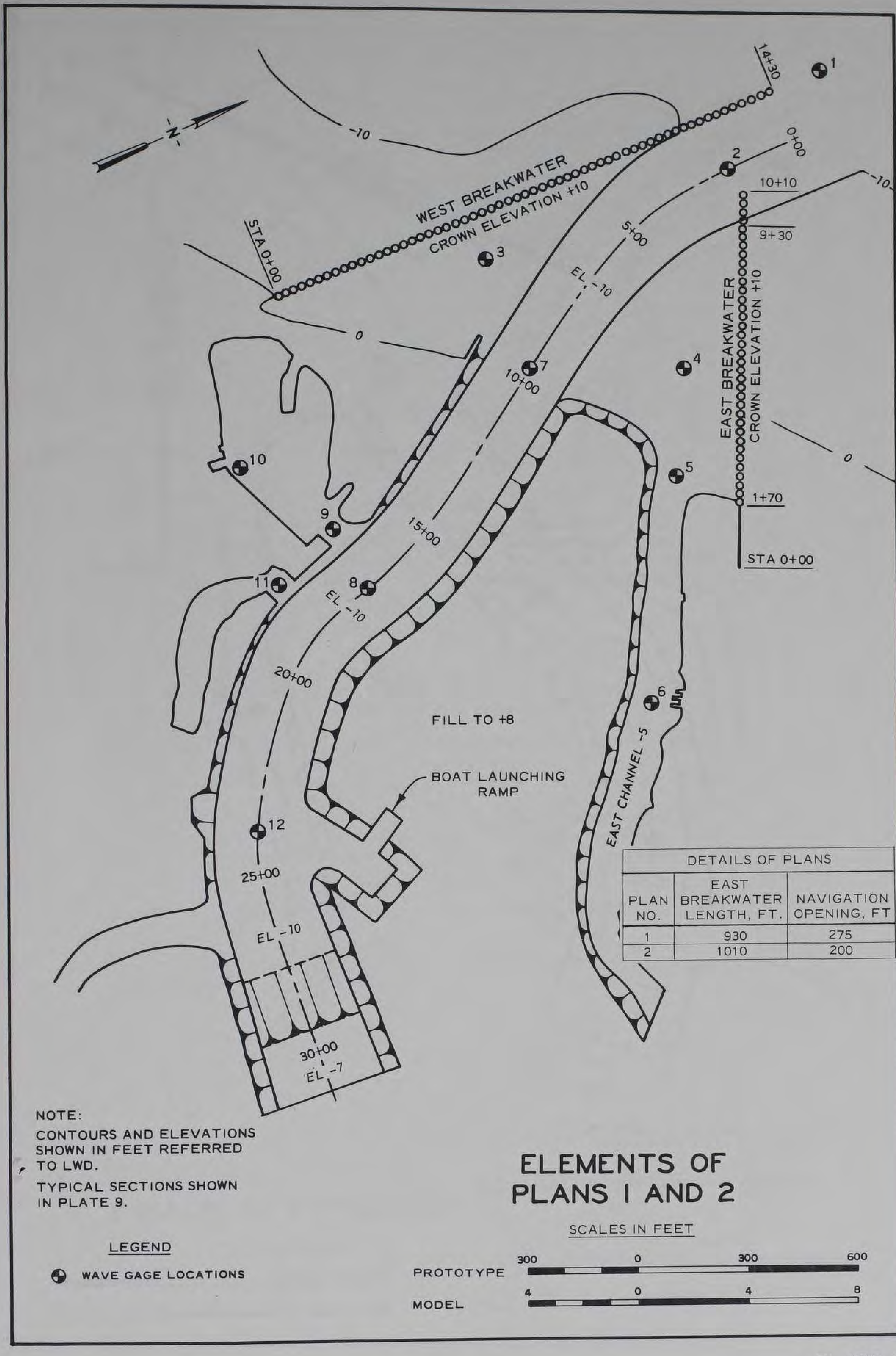
MODEL LAYOUT

SCALES IN FEET

PROTOTYPE 400 0 400 800 1200

MODEL 5 0 5 10 15

PLATE 2



DETAILS OF PLANS		
PLAN NO.	EAST BREAKWATER LENGTH, FT.	NAVIGATION OPENING, FT
1	930	275
2	1010	200

NOTE:

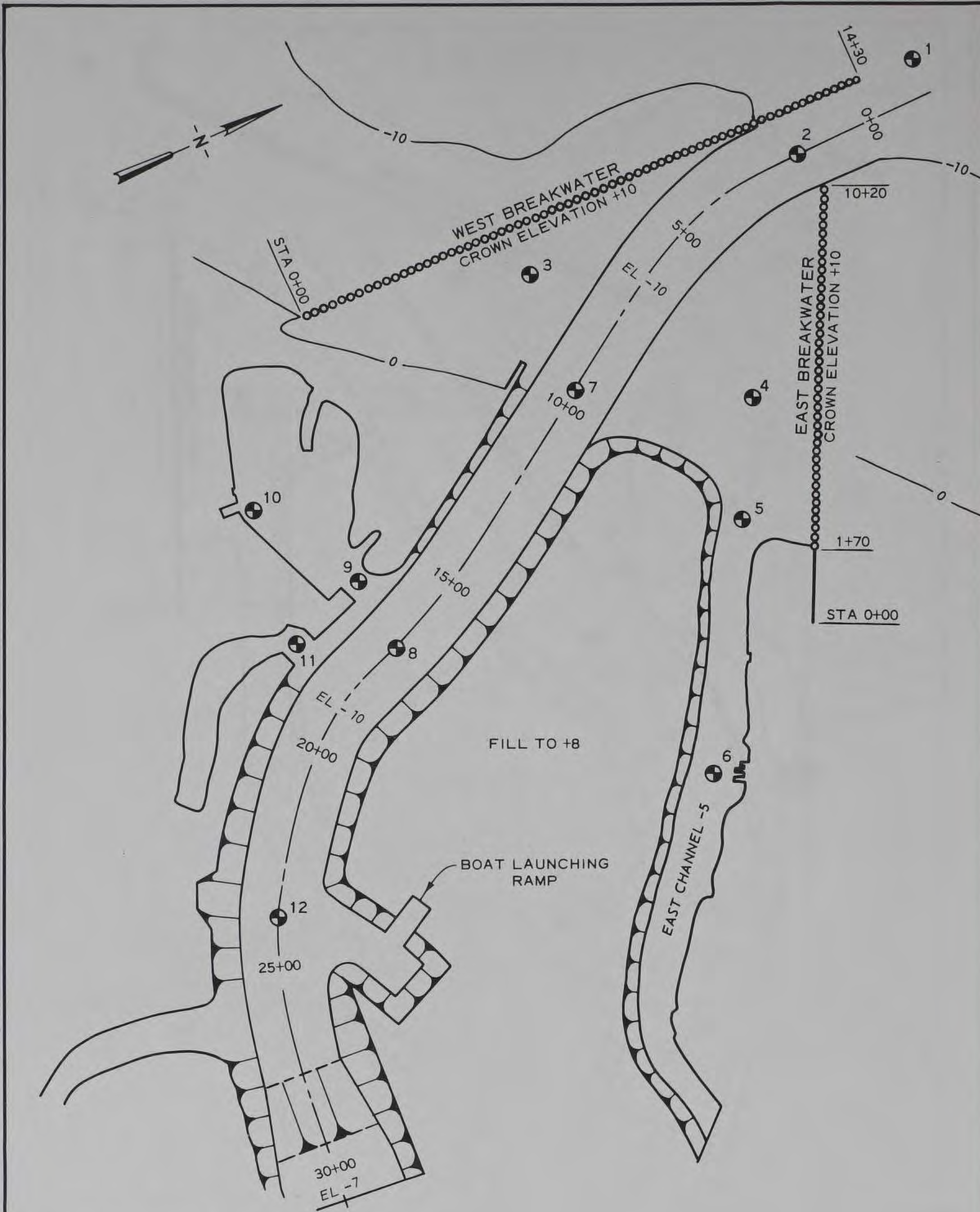
CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO LWD.

TYPICAL SECTIONS SHOWN IN PLATE 9.

LEGEND

WAVE GAGE LOCATIONS

ELEMENTS OF PLANS 1 AND 2



NOTE: CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO LWD.

TYPICAL SECTIONS SHOWN IN PLATE 9.

LEGEND

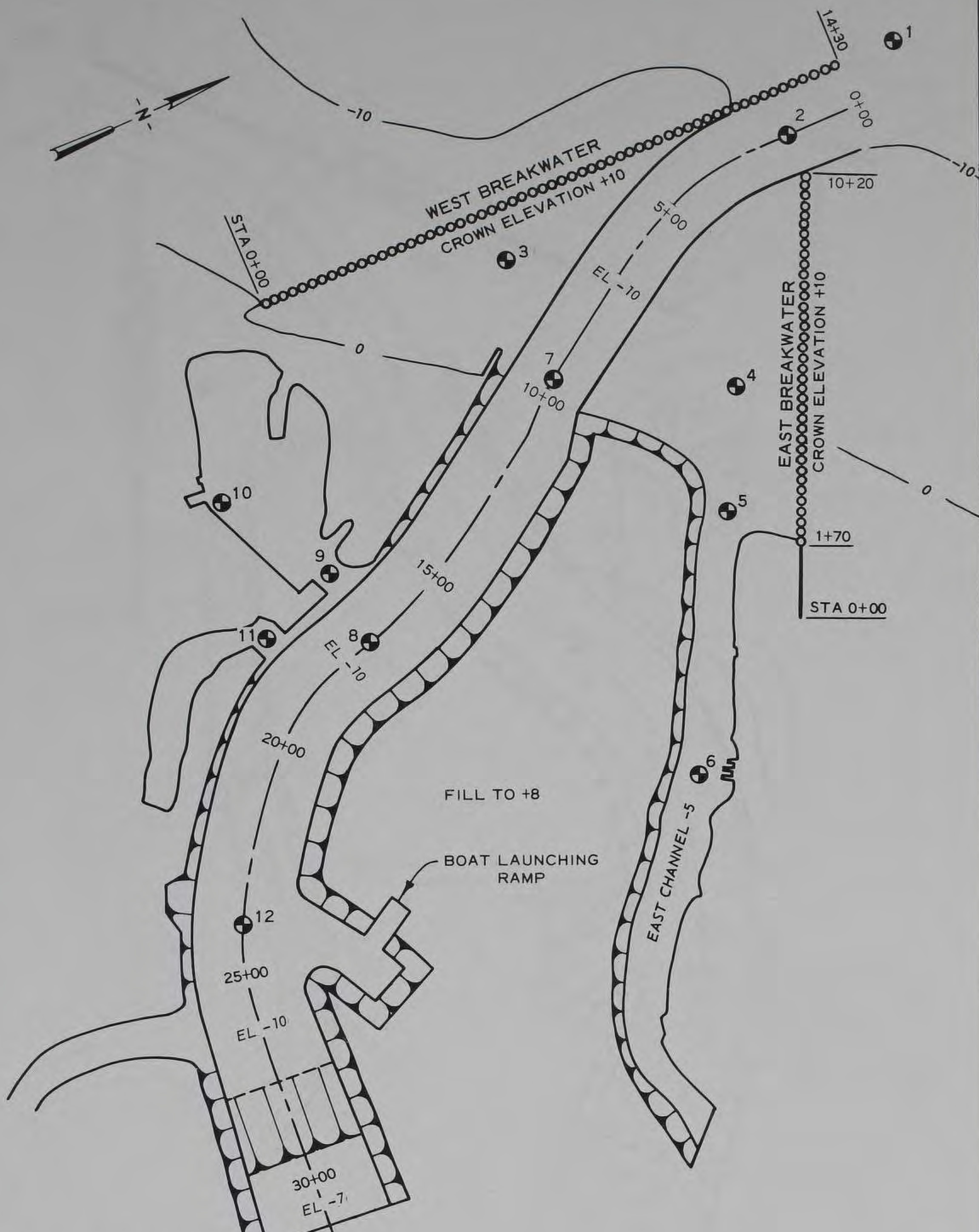
WAVE GAGE LOCATIONS

ELEMENTS OF PLAN 3

SCALES IN FEET

PROTOTYPE 300 0 300 600

MODEL 4 0 4 8



NOTE:
CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO LWD.
TYPICAL SECTIONS SHOWN IN PLATE 9.

LEGEND

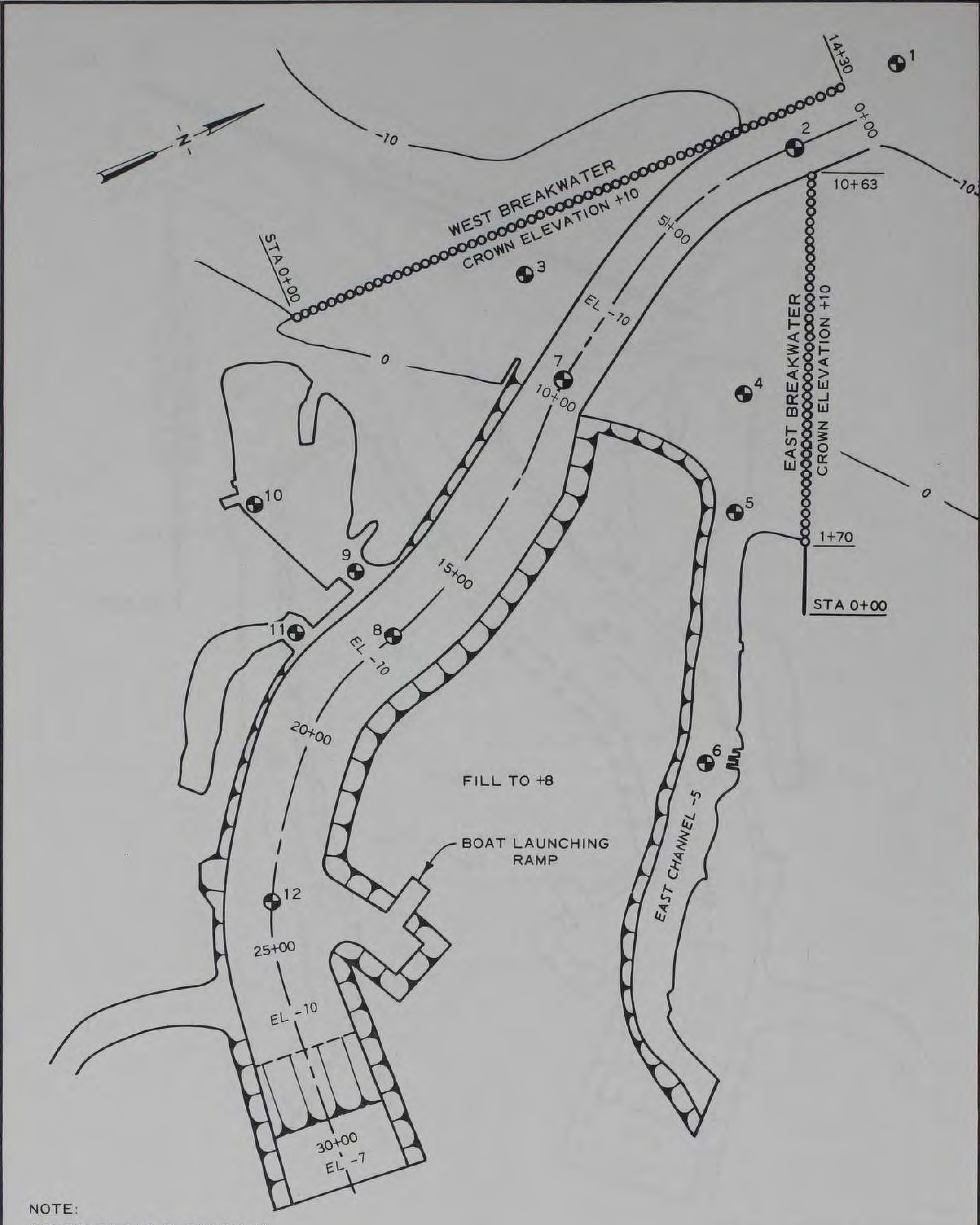
WAVE GAGE LOCATIONS

ELEMENTS OF PLAN 4

SCALES IN FEET

PROTOTYPE 300 0 300 600

MODEL 4 0 4 8



NOTE:
CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO LWD.
TYPICAL SECTIONS SHOWN IN PLATE 9.

ELEMENTS OF PLAN 5

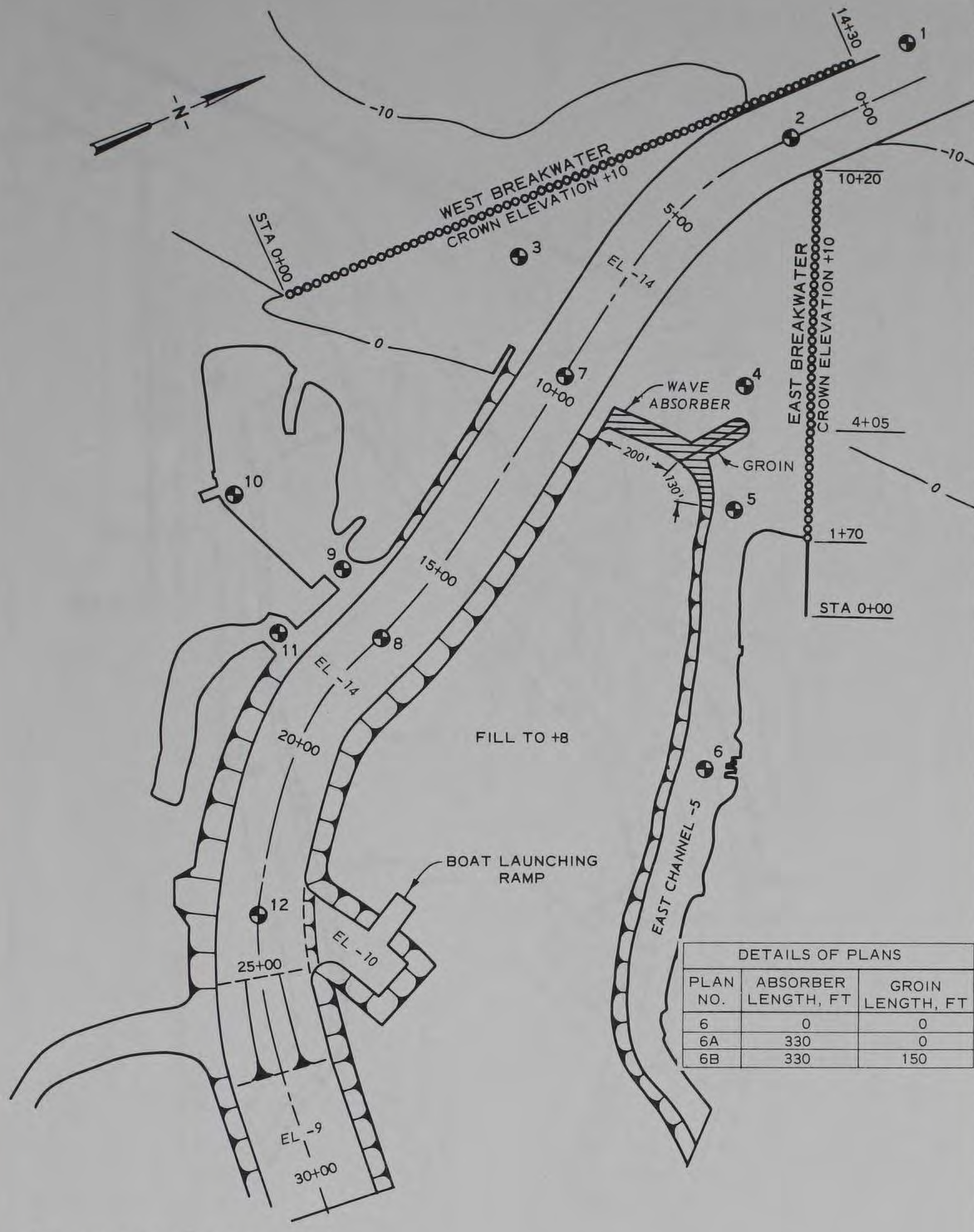
LEGEND

WAVE GAGE LOCATIONS

SCALES IN FEET

PROTOTYPE 300 0 300 600

MODEL 4 0 4 8



DETAILS OF PLANS		
PLAN NO.	ABSORBER LENGTH, FT	GROIN LENGTH, FT
6	0	0
6A	330	0
6B	330	150

NOTE: CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO LWD.

TYPICAL SECTIONS SHOWN IN PLATE 9.

LEGEND

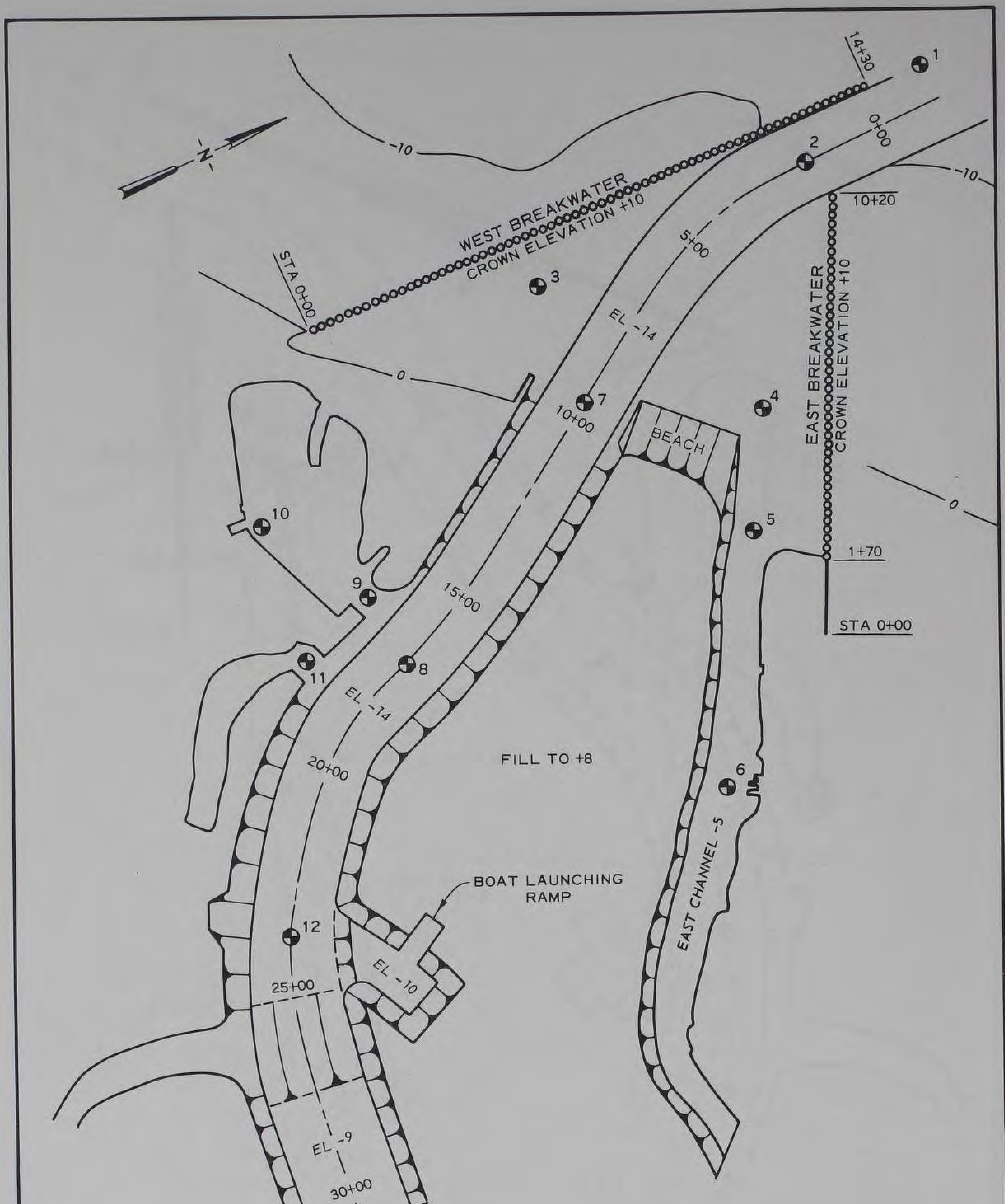
WAVE GAGE LOCATIONS

ELEMENTS OF PLANS 6, 6A, AND 6B

SCALES IN FEET

PROTOTYPE 300 0 300 600

MODEL 4 0 4 8



NOTE: CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO LWD.

TYPICAL SECTIONS SHOWN IN PLATE 9.

LEGEND

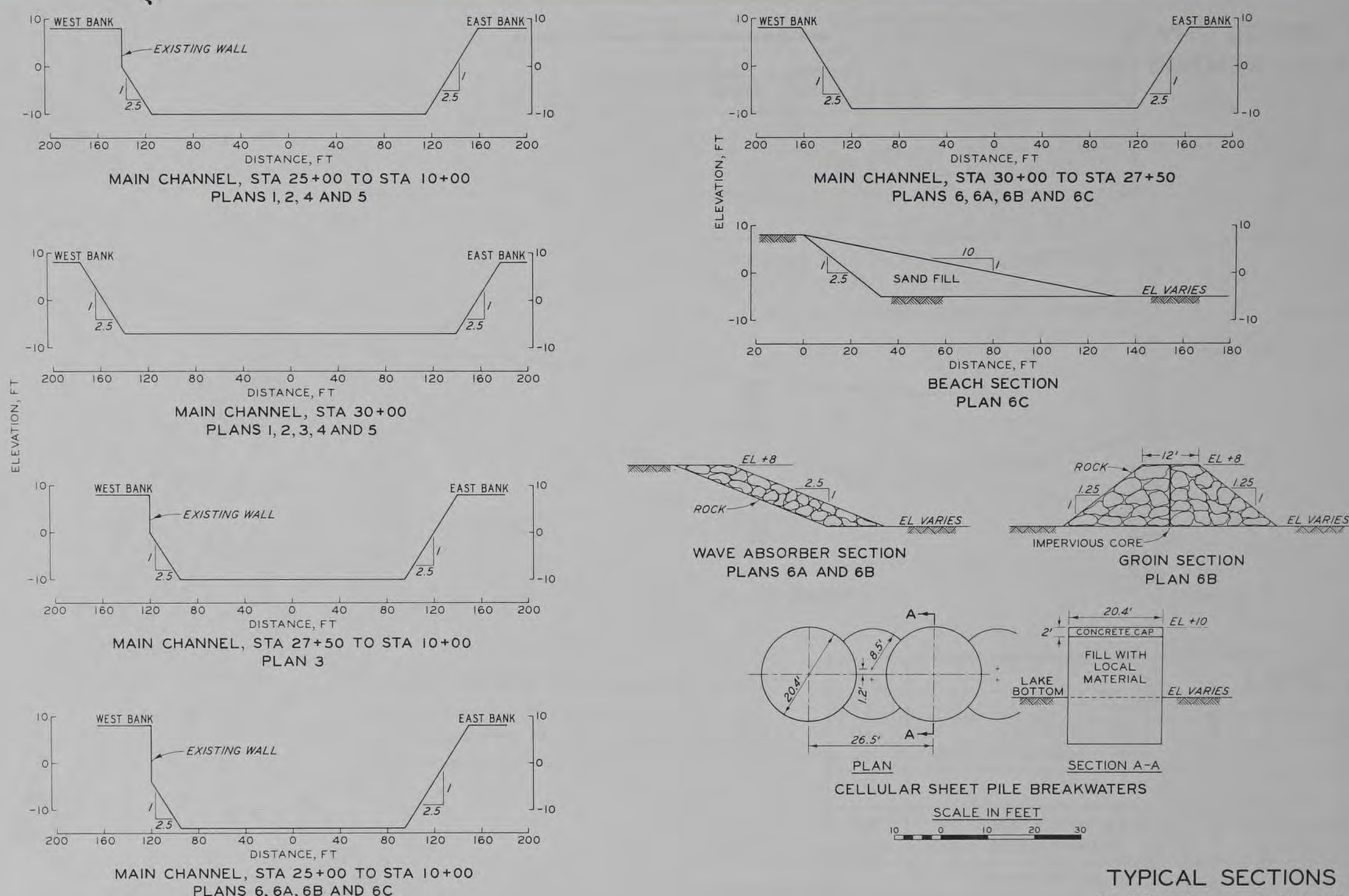
WAVE GAGE LOCATIONS

ELEMENTS OF PLAN 6C

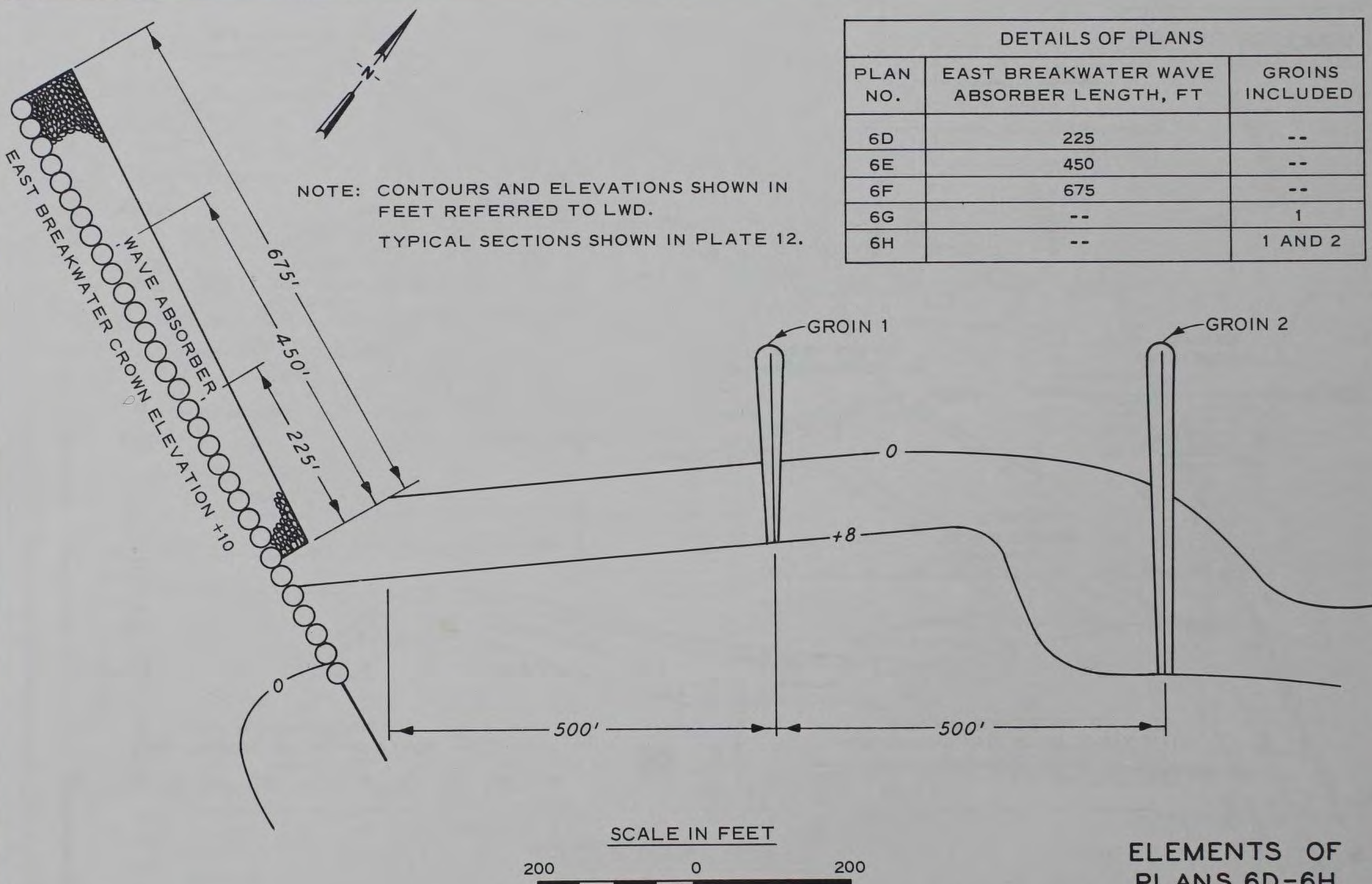
SCALES IN FEET

PROTOTYPE 300 0 300 600

MODEL 4 0 4 8



TYPICAL SECTIONS
PLANS 1-6, 6A, 6B AND 6C



NOTE: CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO LWD.

TYPICAL SECTIONS SHOWN IN PLATE 12.

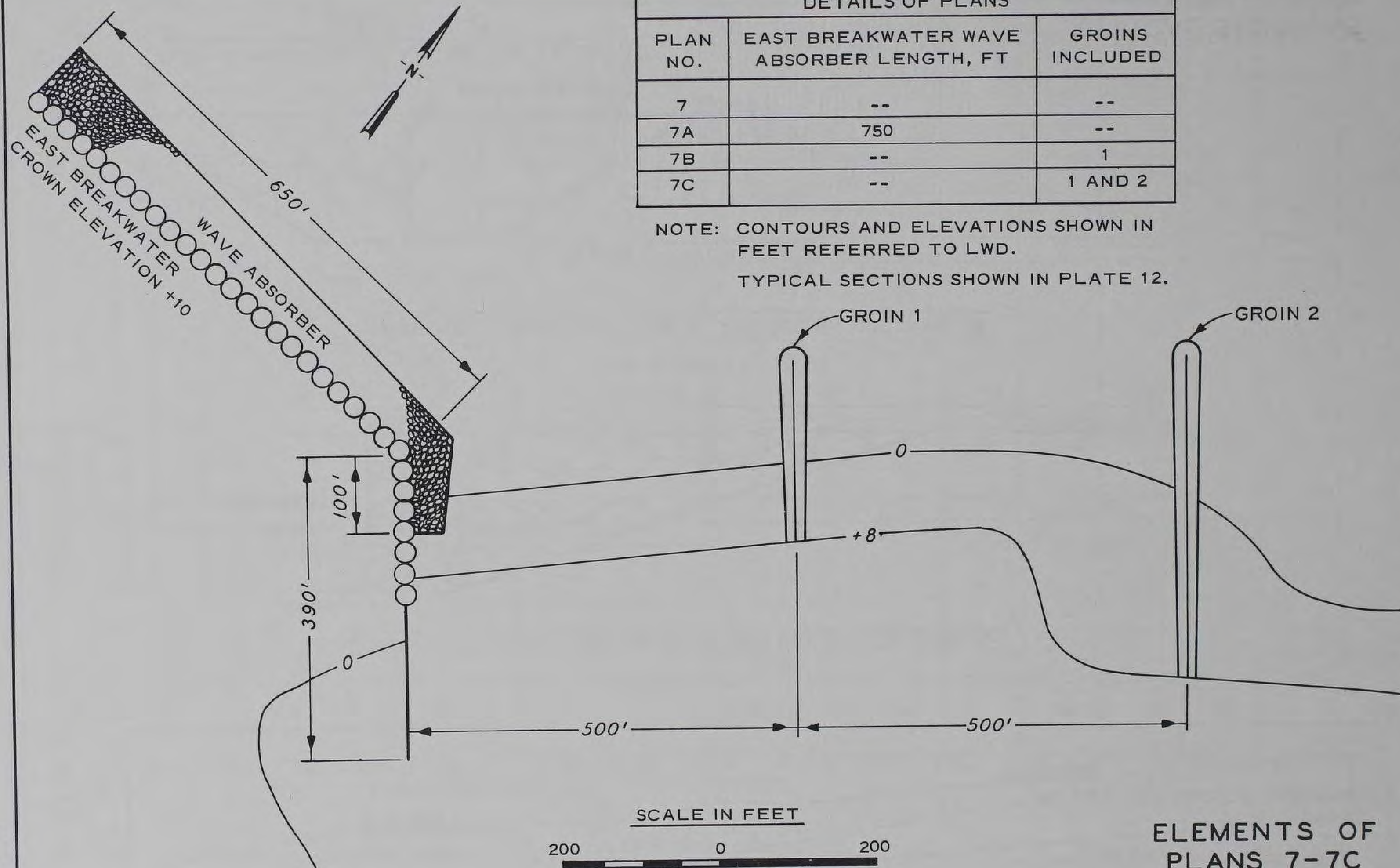
DETAILS OF PLANS		
PLAN NO.	EAST BREAKWATER WAVE ABSORBER LENGTH, FT	GROINS INCLUDED
6D	225	--
6E	450	--
6F	675	--
6G	--	1
6H	--	1 AND 2

ELEMENTS OF PLANS 6D-6H

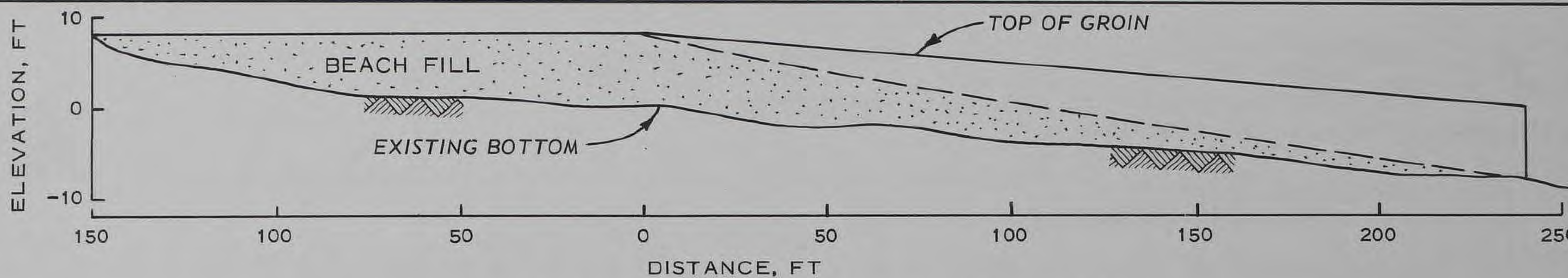
DETAILS OF PLANS		
PLAN NO.	EAST BREAKWATER WAVE ABSORBER LENGTH, FT	GROINS INCLUDED
7	--	--
7A	750	--
7B	--	1
7C	--	1 AND 2

NOTE: CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO LWD.

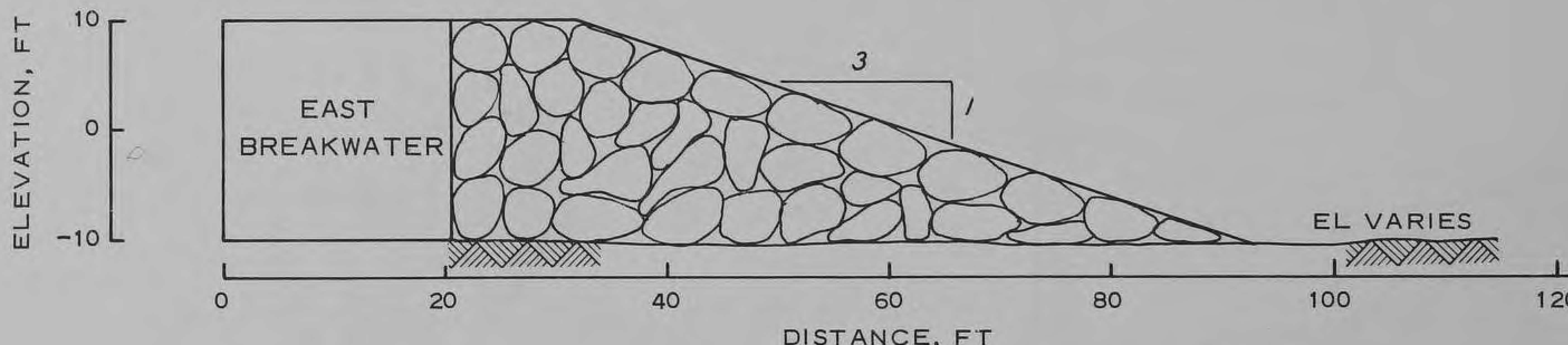
TYPICAL SECTIONS SHOWN IN PLATE 12.



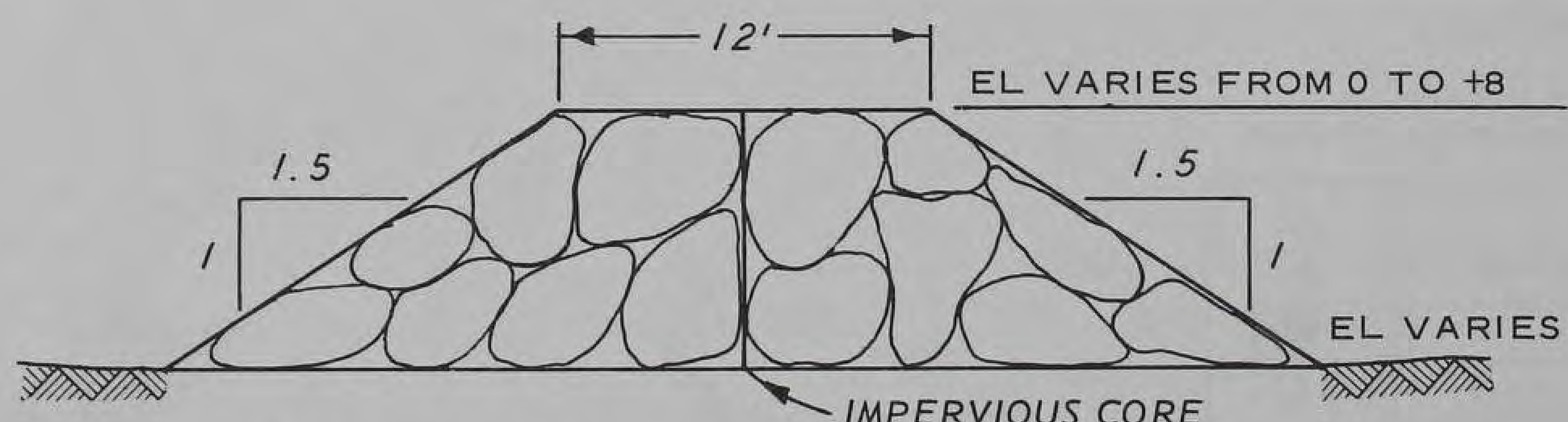
ELEMENTS OF PLANS 7-7C



GROIN AND BEACH PROFILE, PLANS 6G, 6H, 7B, AND 7C



WAVE ABSORBER SECTION, PLANS 6D, 6E, 6F, AND 7A



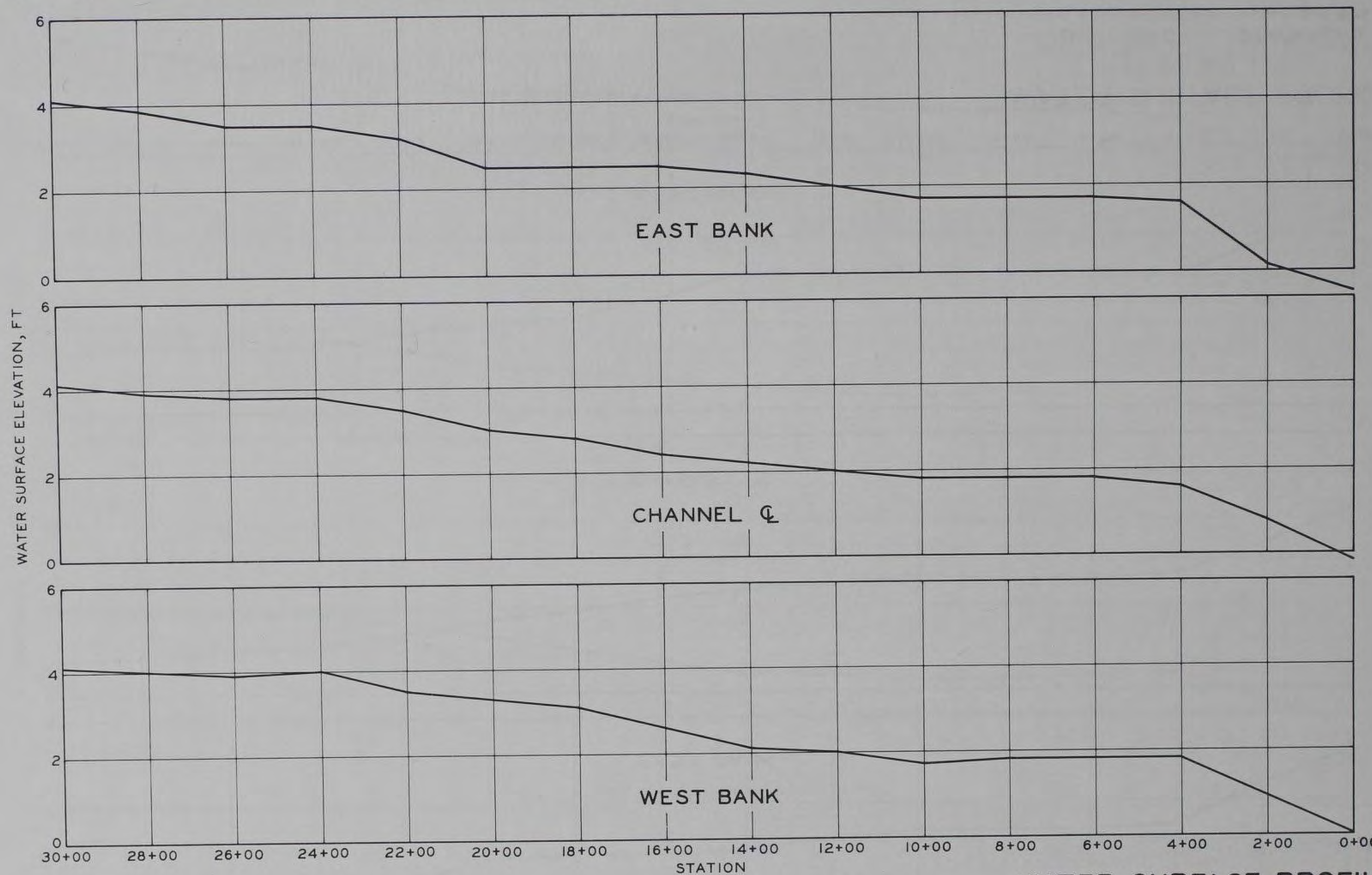
GROIN SECTION, PLANS 6G, 6H, 7B, AND 7C

SCALE IN FEET

10 0 10 20

TYPICAL SECTIONS

PLANS 6D-6H AND 7A-7C

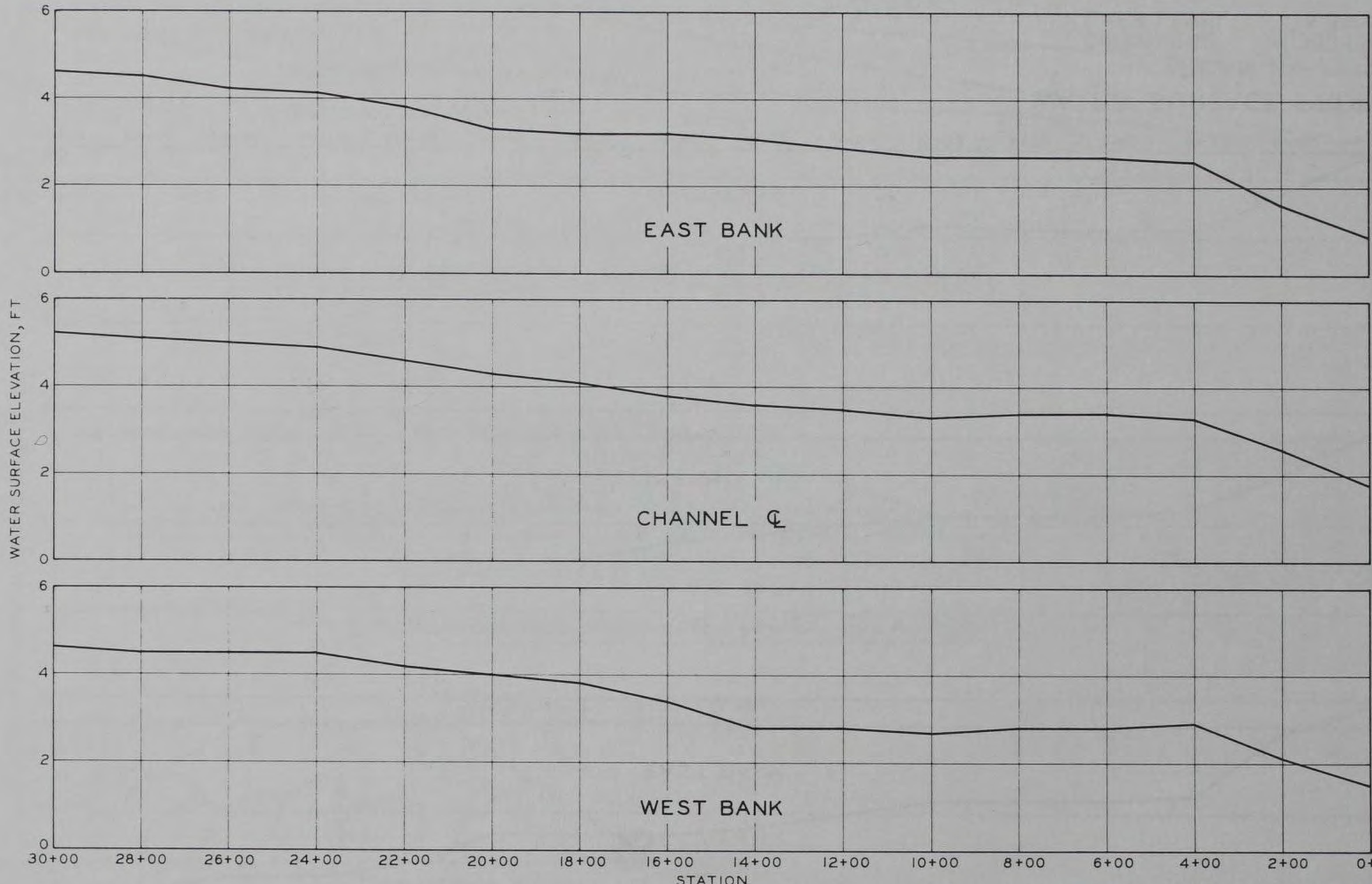


WATER SURFACE PROFILES

PLAN I

DISCHARGE 27,000 CFS

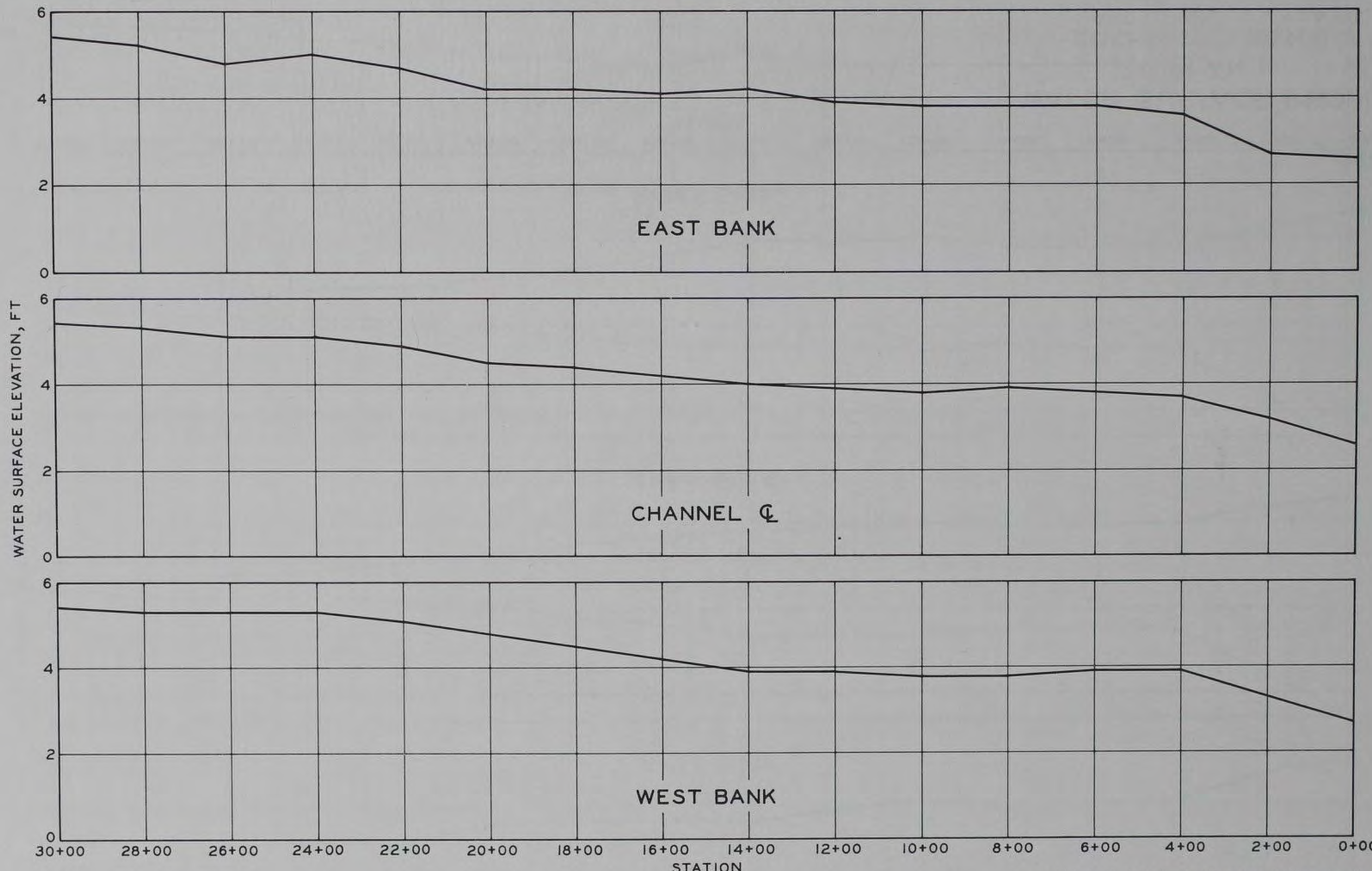
LAKE LEVEL 0.0 FT



WATER SURFACE PROFILES

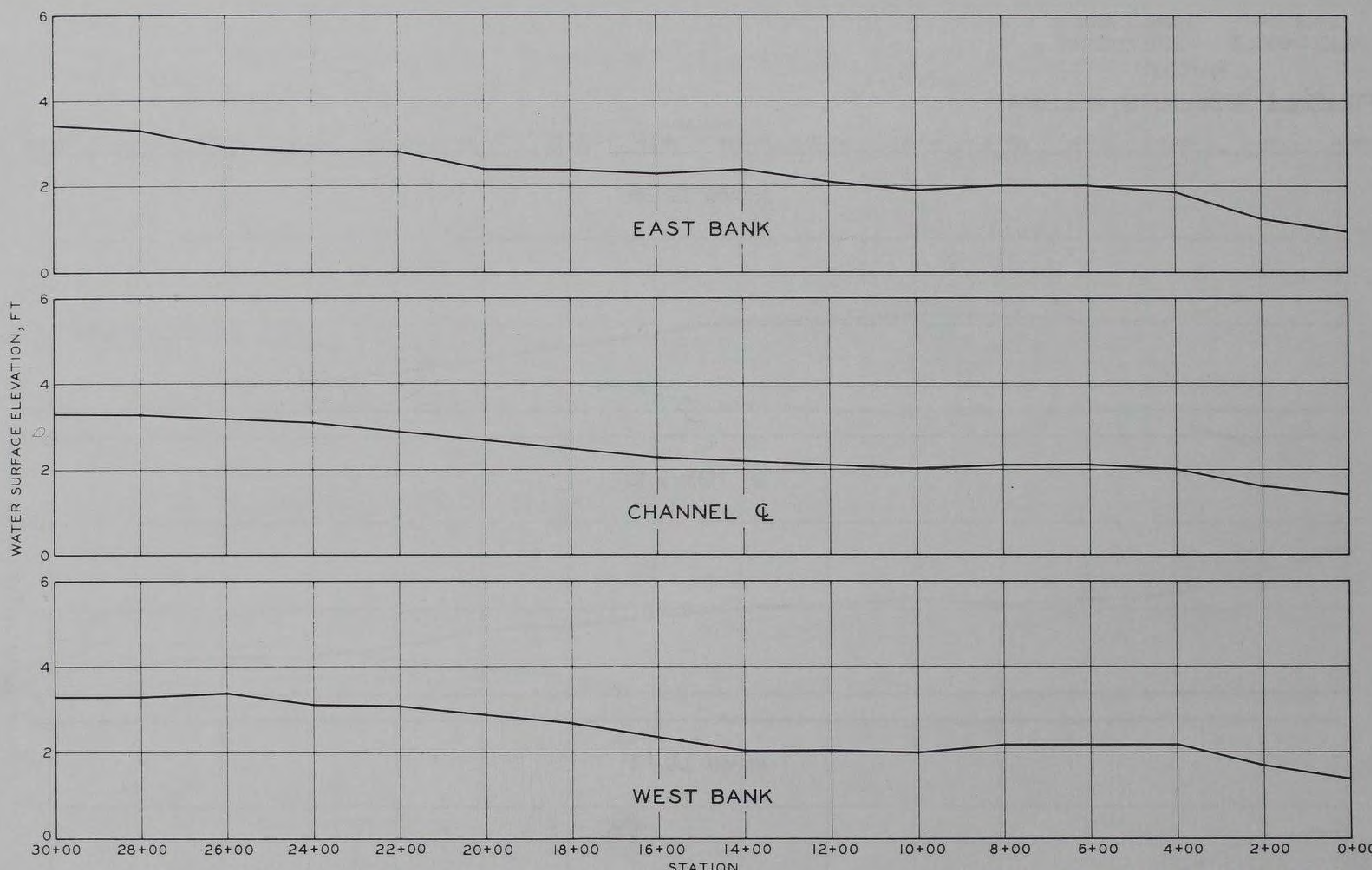
PLAN 1

DISCHARGE 27,000 CFS
LAKE LEVEL +1.4 FT

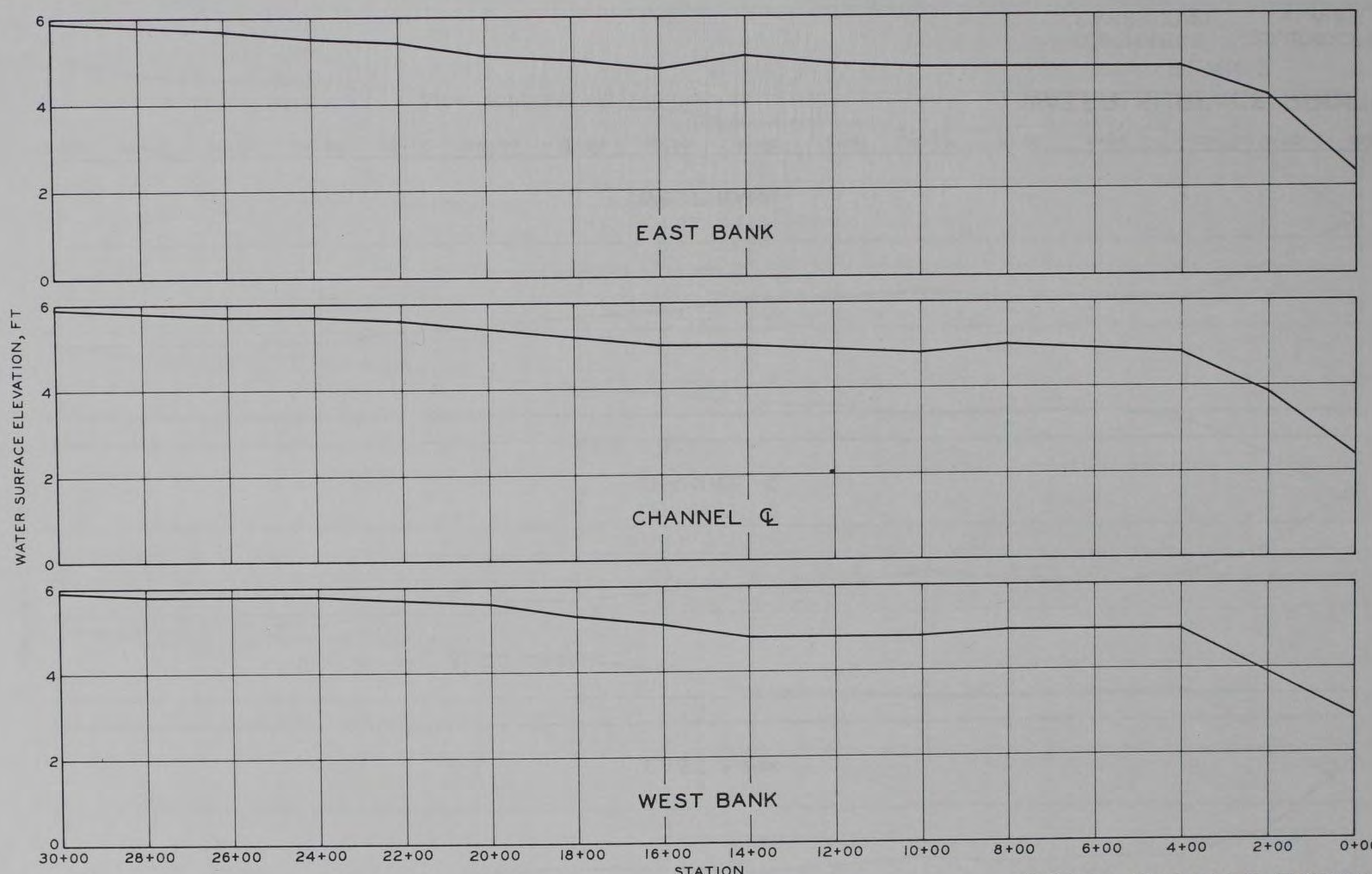


WATER SURFACE PROFILES
PLAN I

DISCHARGE 27,000 CFS
LAKE LEVEL +2.8 FT



WATER SURFACE PROFILES
PLAN I
DISCHARGE 20,000 CFS
LAKE LEVEL +1.4 FT

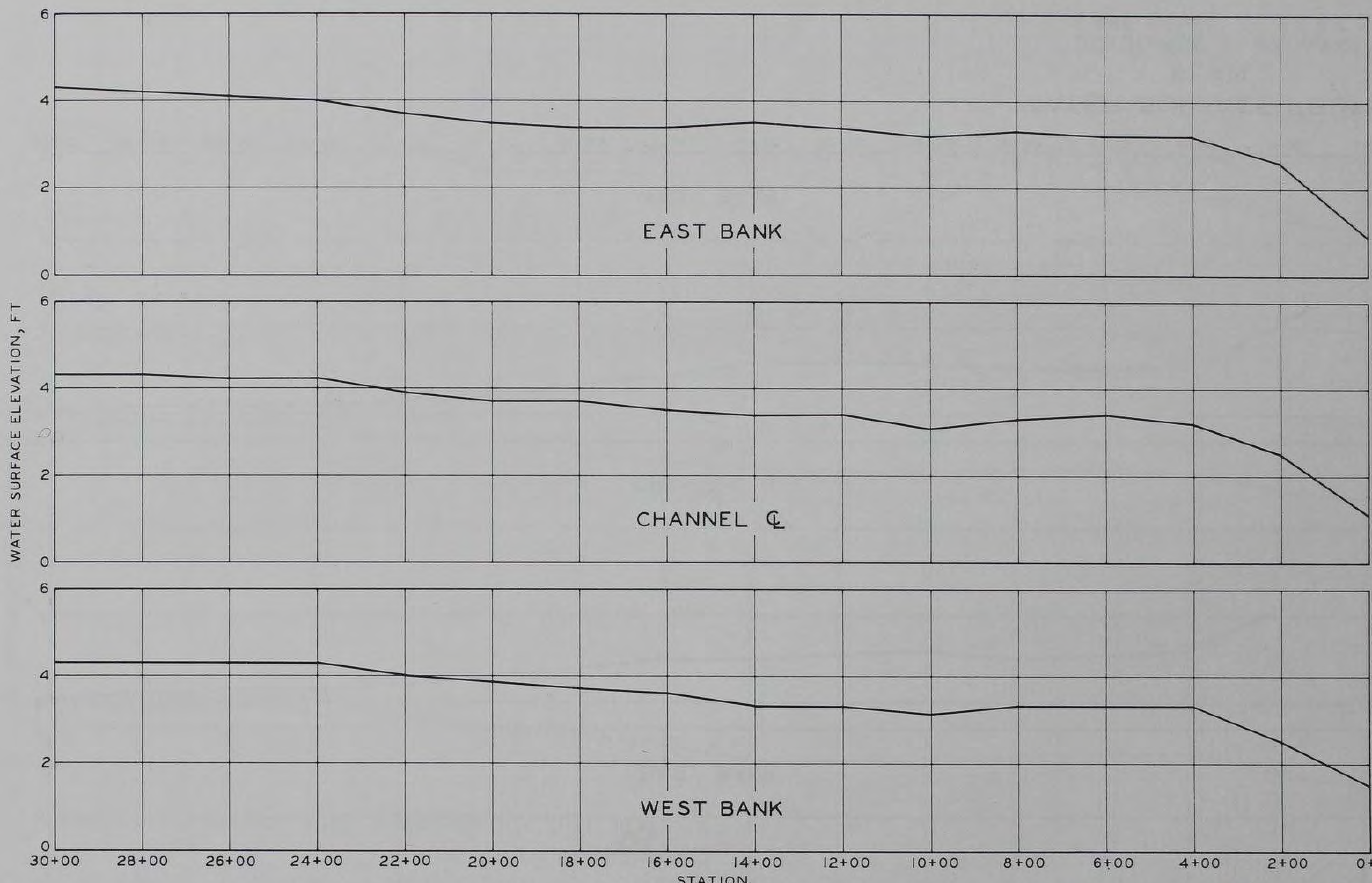


WATER SURFACE PROFILES

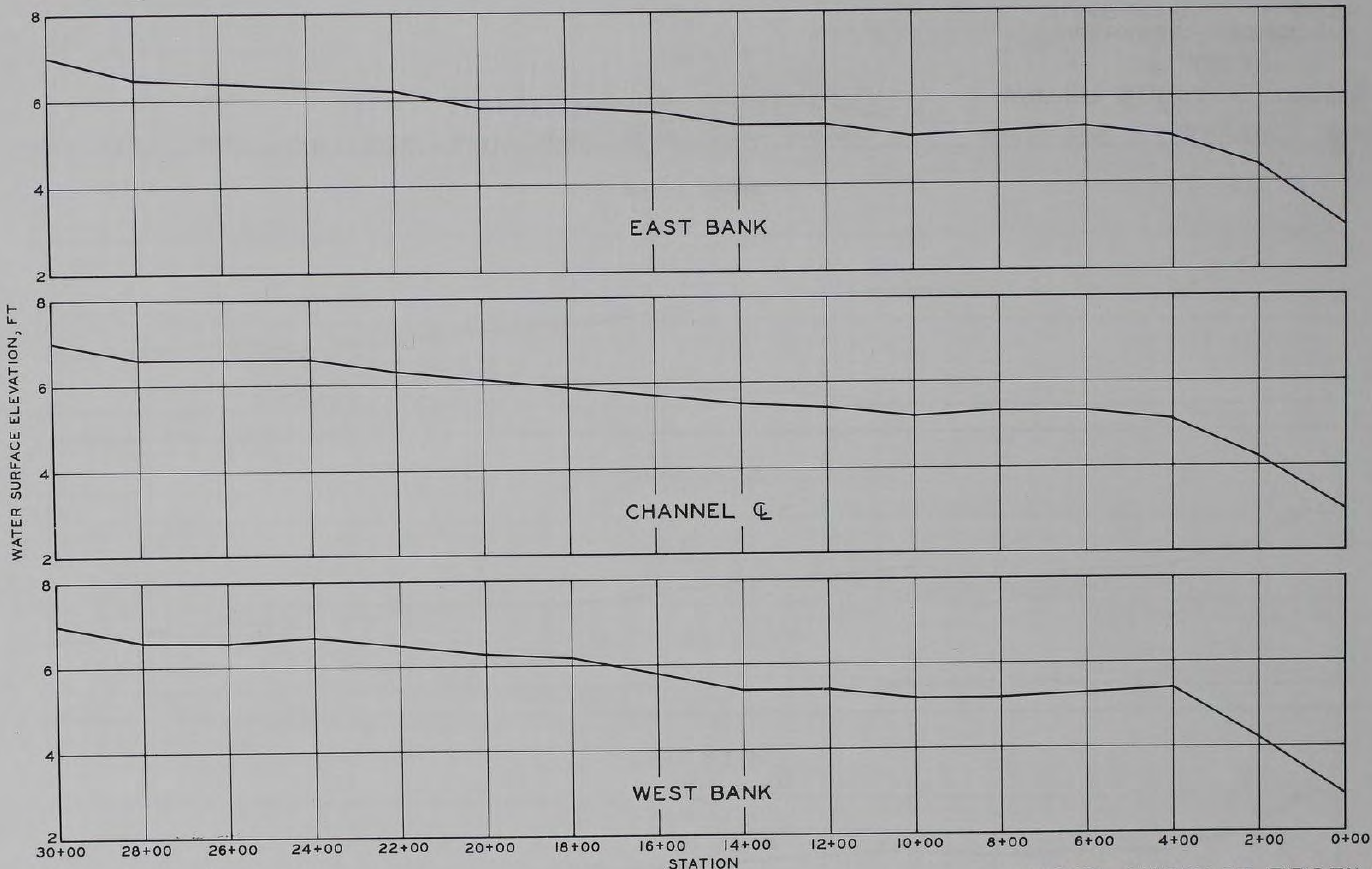
PLAN 2

DISCHARGE 27,000 CFS
LAKE LEVEL +2.8 FT

PLATE 18

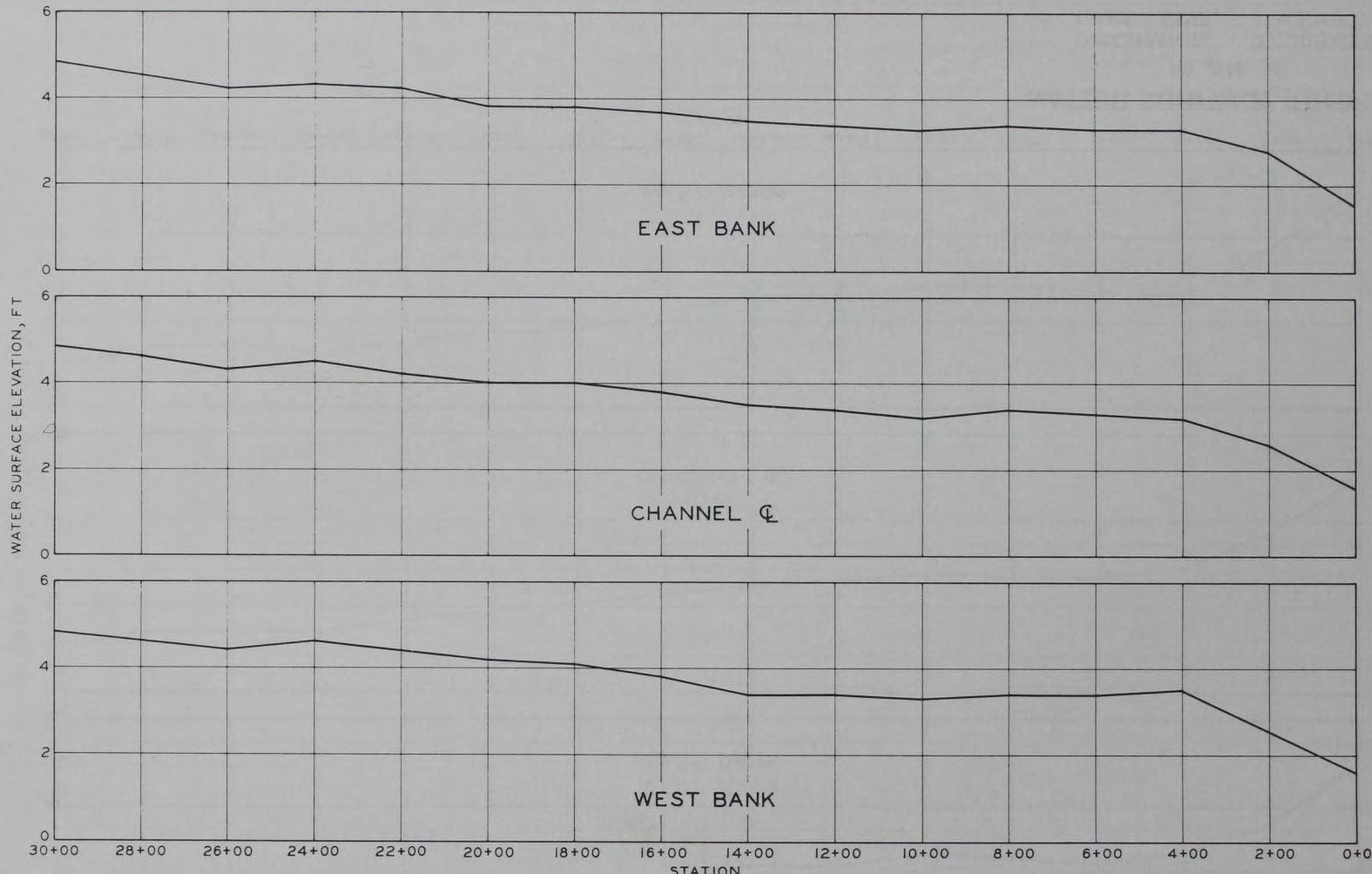


WATER SURFACE PROFILES
PLAN 2
DISCHARGE 20,000 CFS
LAKE LEVEL +1.4 FT



WATER SURFACE PROFILES
PLAN 3

DISCHARGE 27,000 CFS
LAKE LEVEL +2.8 FT

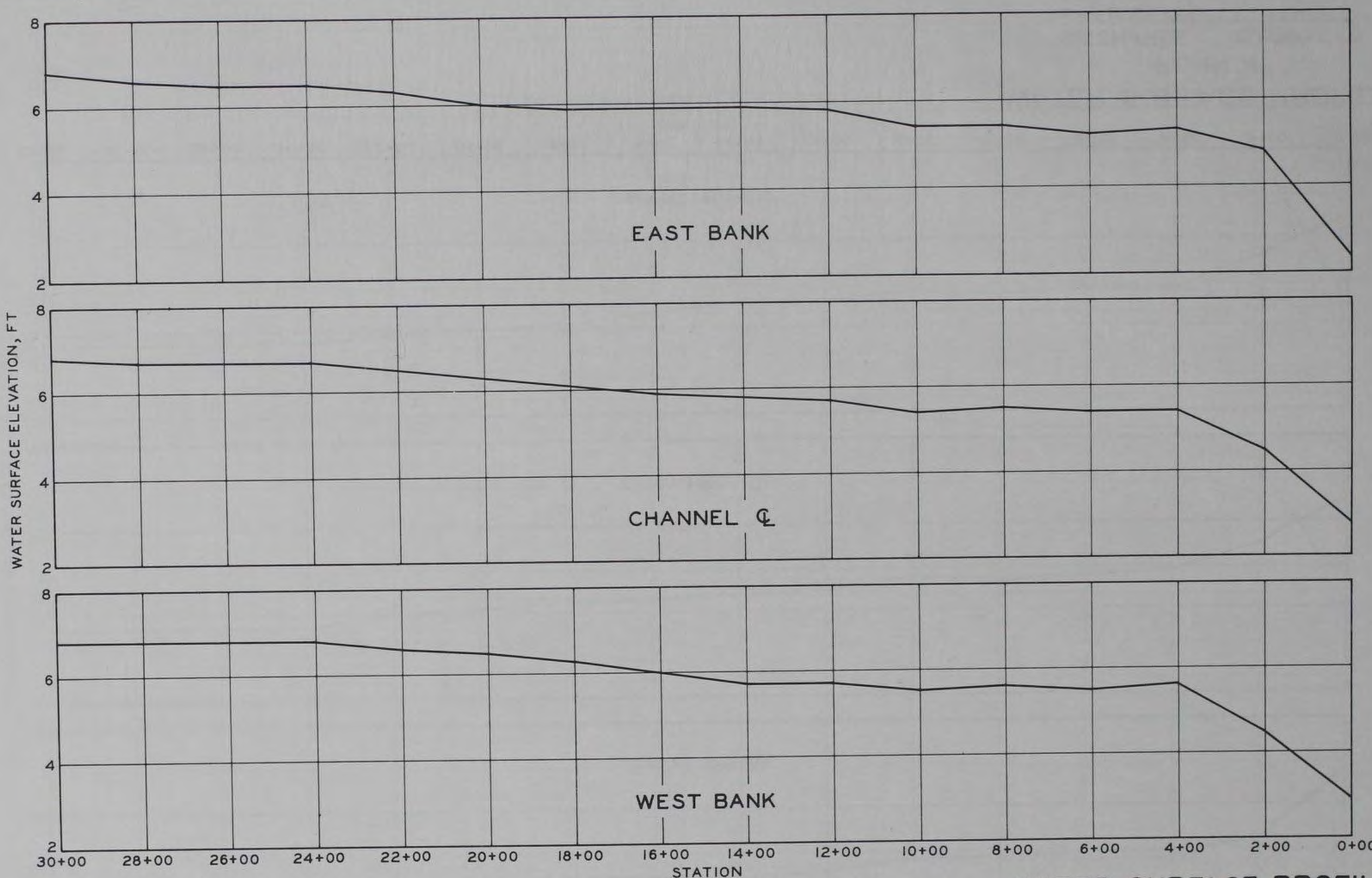


WATER SURFACE PROFILES

PLAN 3

DISCHARGE 20,000 CFS

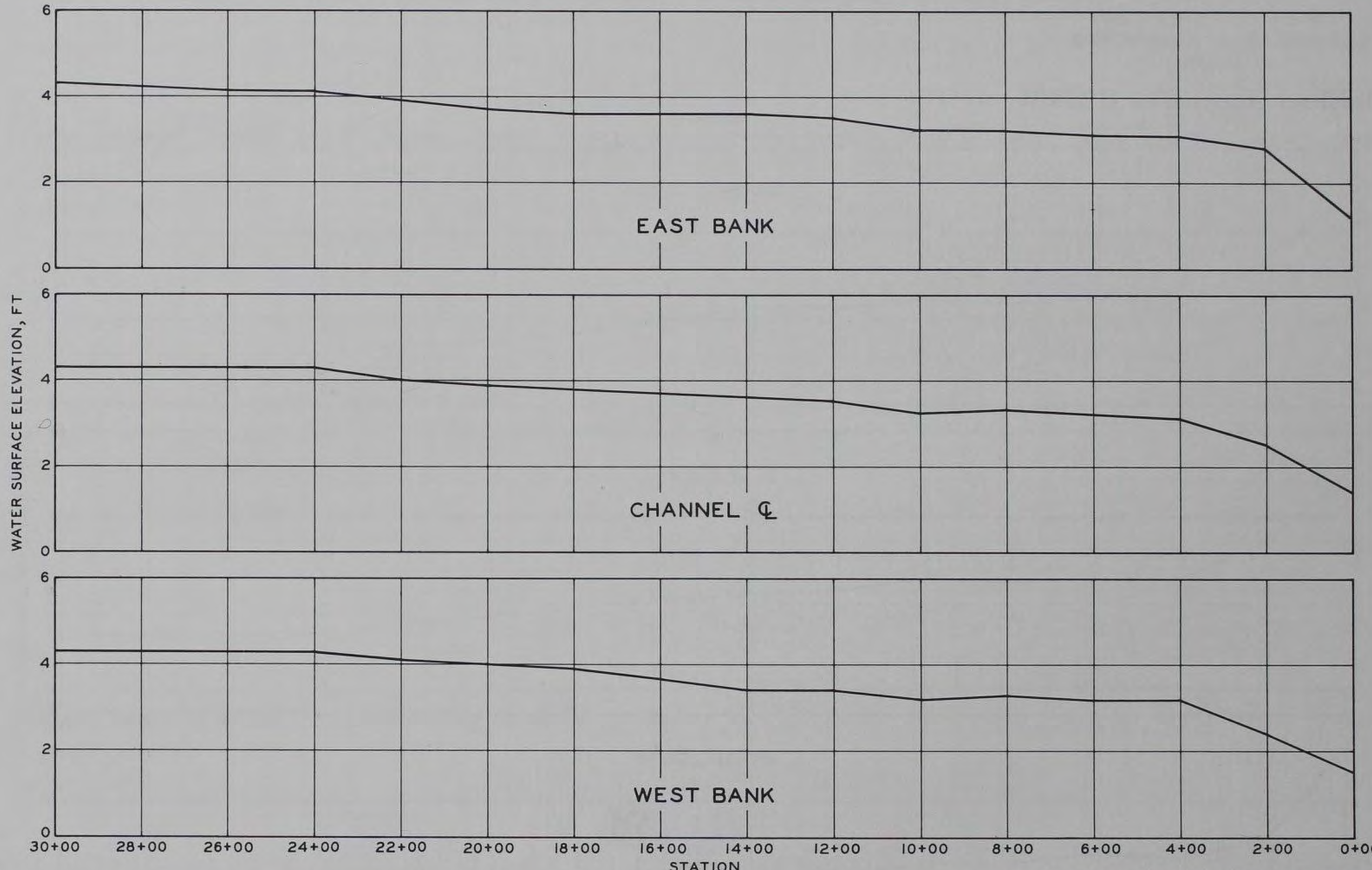
LAKE LEVEL +1.4 FT



WATER SURFACE PROFILES

PLAN 4

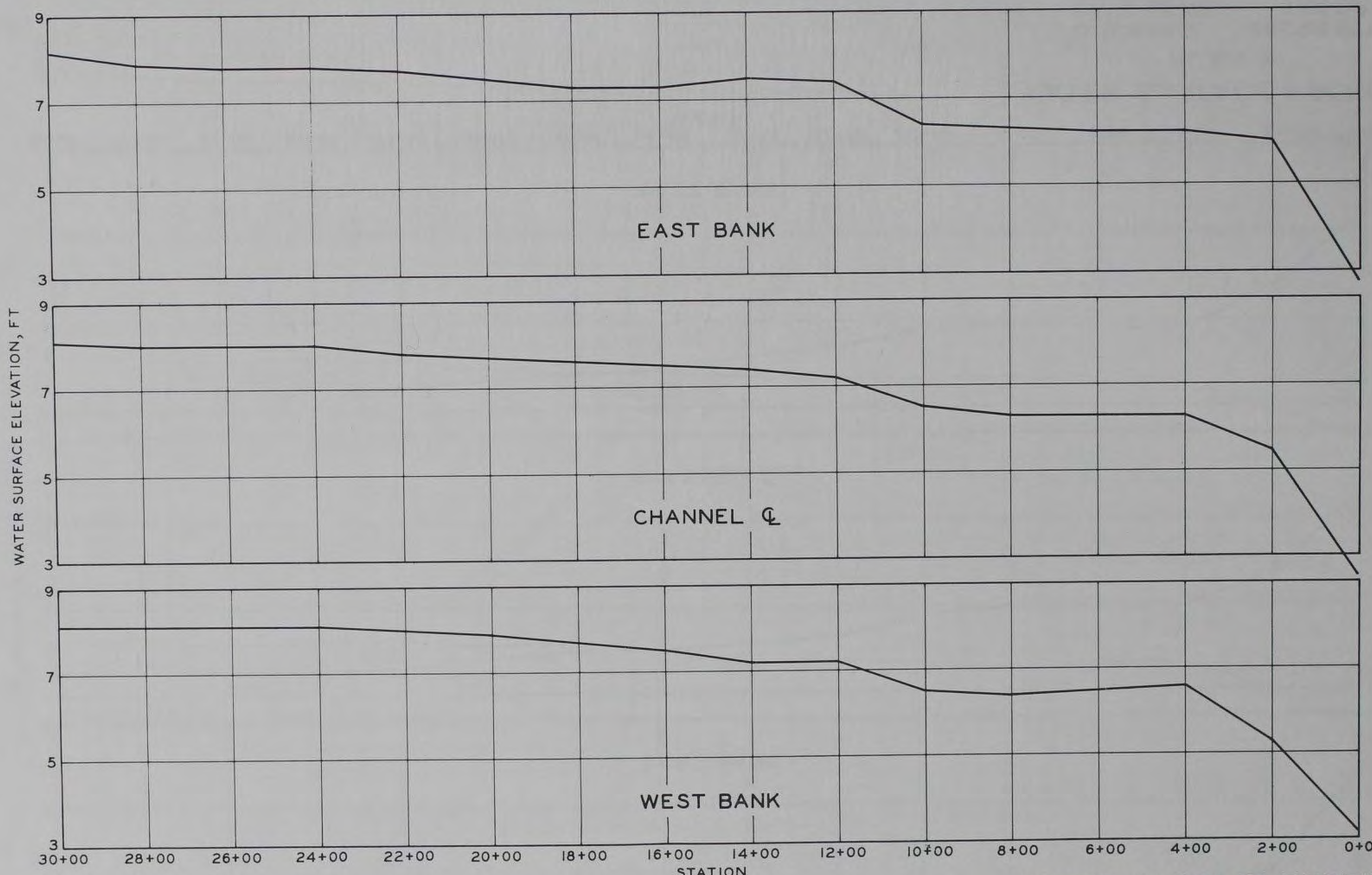
DISCHARGE 27,000 CFS
LAKE LEVEL +2.8 FT



WATER SURFACE PROFILES

PLAN 4

DISCHARGE 20,000 CFS
LAKE LEVEL +1.4 FT



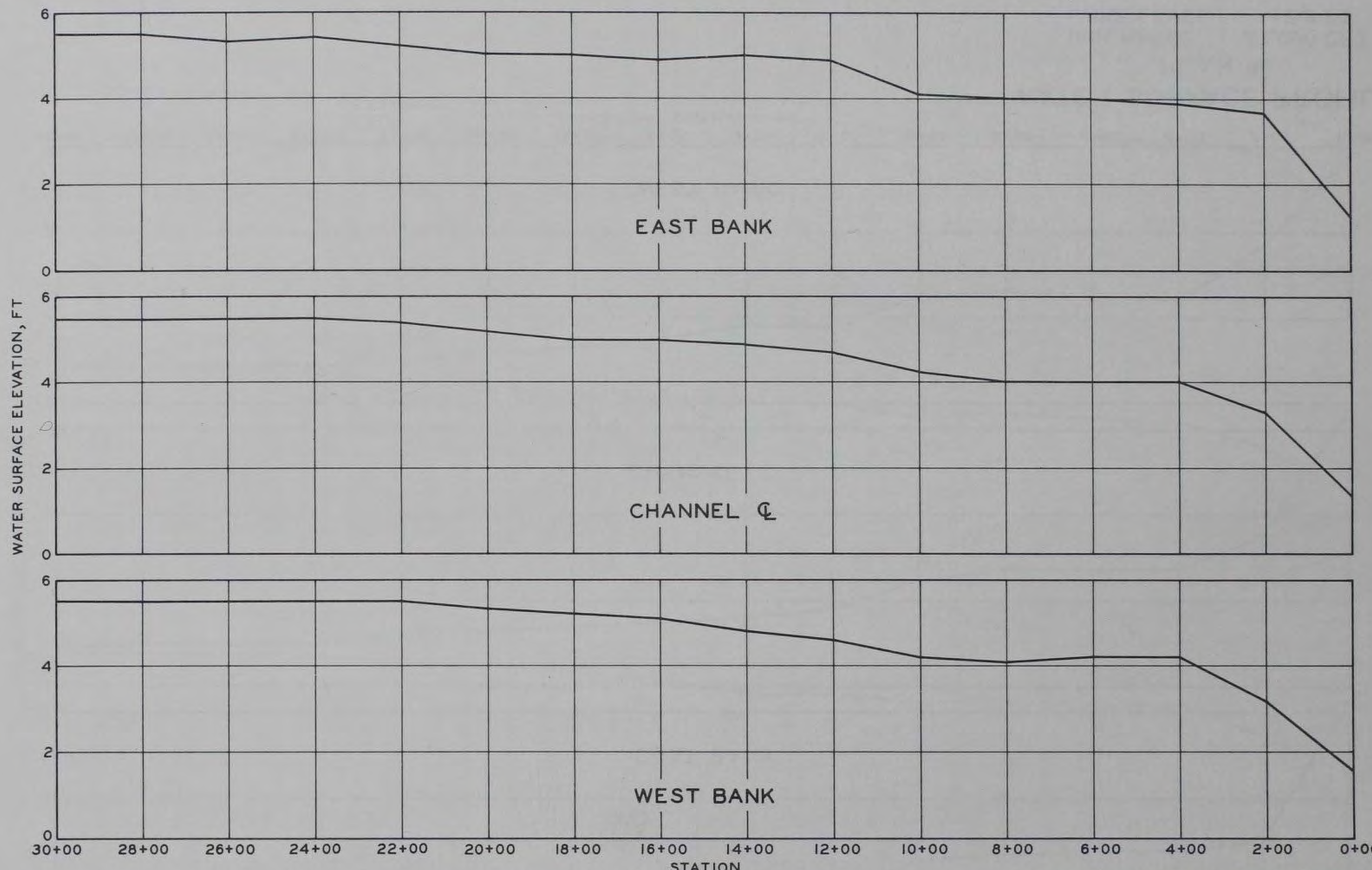
WATER SURFACE PROFILES

PLAN 5

DISCHARGE 27,000 CFS

LAKE LEVEL +2.8 FT

PLATE 24

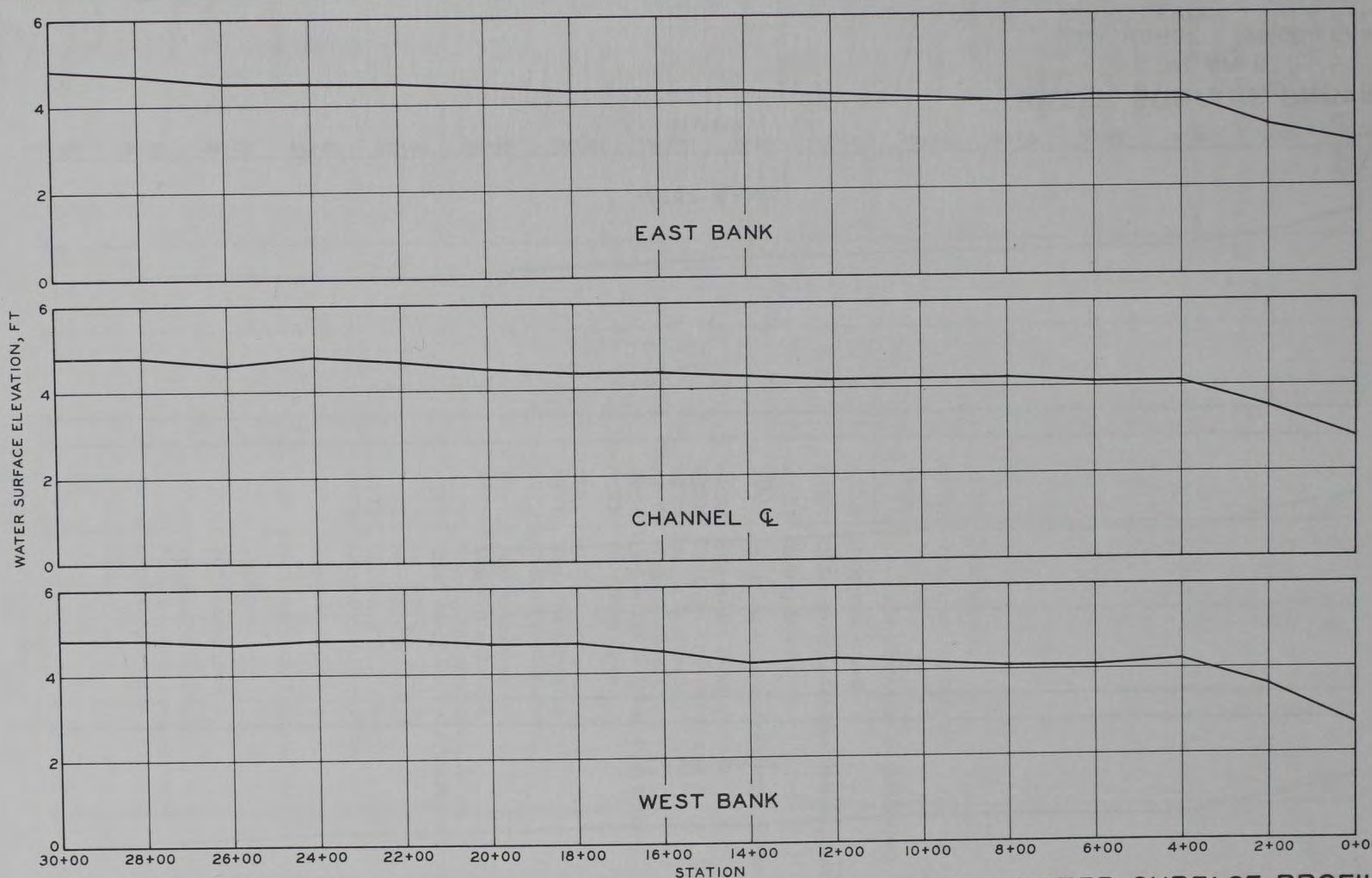


WATER SURFACE PROFILES

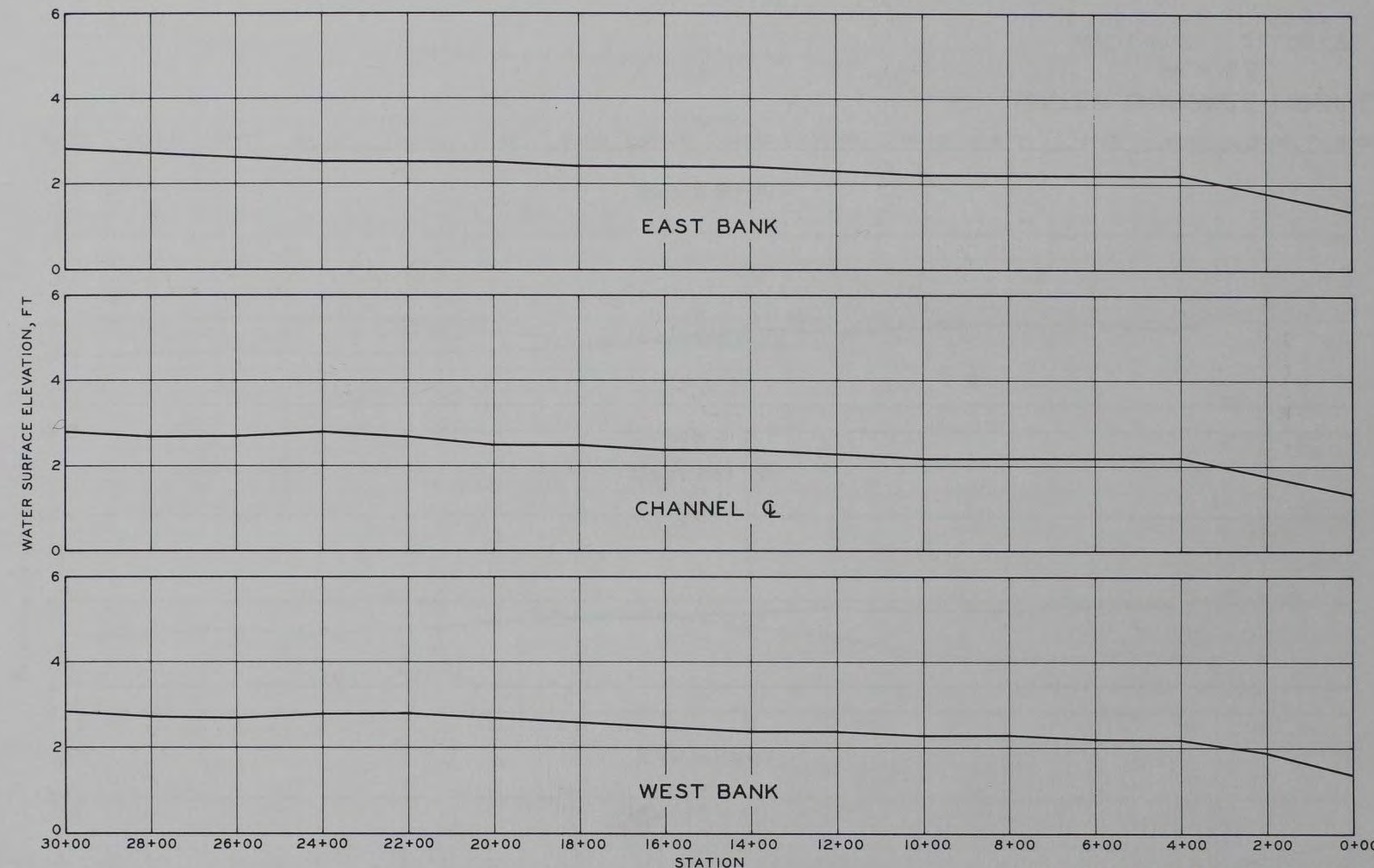
PLAN 5

DISCHARGE 20,000 CFS

LAKE LEVEL +1.4 FT



WATER SURFACE PROFILES
PLAN 6
DISCHARGE 27,000 CFS
LAKE LEVEL +2.8 FT



WATER SURFACE PROFILES
PLAN 6
DISCHARGE 20,000 CFS
LAKE LEVEL +1.4 FT

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11. SUPPLEMENTARY NOTES	12. SPONSORING MILITARY ACTIVITY
	U. S. Army Engineer District Buffalo, New York

13. ABSTRACT

A 1:75-scale model of the lower 2000 ft of the Chagrin River and sufficient offshore area in Lake Erie to permit generation of the required test waves was used to investigate the arrangement and design of certain proposed improvements with respect to wave action and flood control. The proposed improvement plans consisted of (a) arrowhead breakwaters in Lake Erie at the mouth of the river, aggregating about 2360 ft in length; (b) realignment and enlargement of the river channel from Lake Erie through the city of Eastlake, with levees where required to supplement channel enlargement; (c) a spur channel and an access channel for navigation; (d) recreational facilities at the river mouth; and (e) the addition of beach fill and protective groins along the shoreline east of the east breakwater. A 60-ft-long wave machine and electrical wave-height measuring and recording apparatus were utilized in model operation. It was concluded from test results that (a) the originally proposed plan of improvement, which specified a 275-ft-wide navigation opening and a 230-ft-wide, 10-ft-deep lower river channel, will not provide adequate protection from wave action, and current velocities in the revised channel will exceed the specified criteria; (b) reducing upstream current velocities and wave heights by reducing the width of the navigation opening and outer channel to 150 ft will result in flooding along the west bank of the river; (c) an improvement plan utilizing a 190-ft-wide, 14-ft-deep navigation opening and lower river channel will provide more satisfactory flood flow conditions than any of the other plans tested; (d) while several of the plans tested would provide satisfactory wave conditions at most locations inside the harbor, only plan 6B meets the specified criteria at all locations; and (e) the addition of wave absorbers to the lakeward face of the east breakwater and the installation of groins along the shore eastward of this structure will be beneficial in reducing the magnitude of alongshore littoral currents and should help to reduce beach erosion in this area.

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14. KEY WORDS	LINK A		LINK B		LINK C	
	ROLE	WT	ROLE	WT	ROLE	WT
Breakwaters						
Chagrin River						
Channel improvements						
Eastlake, Ohio						
Flood protection						
Hydraulic models						
Lake Erie						
Water waves						

Unclassified

Security Classification