Ludington Pumped Storage Project
Lake Front Model

Part I - Report

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INTRODUCTION

The Ludington Pumped Storage Project is being constructed about four miles south of Ludington, Michigan, by the Consumers Power Company and the Detroit Edison Company. During maximum power generation the discharge from the upper lake to Lake Michigan will be 76,000 cfs whereas the maximum flow rate during the pumping stage will be 66,000 cfs. The powerhouse will be located approximately at the present Lake Michigan shore line. The depth of water at the face of the structure will be approximately 50 feet.

The purpose of this model study was to develop a protective harbor for the powerhouse which would keep wave heights at the structure in a safe range while minimizing currents throughout the harbor area. The model also provided some qualitative information on the nature of the sediment and ice movement that might be expected in the prototype.

The model project was undertaken as the result of a contract, dated January 23, 1968, between Ebasco Services, Inc., the designing engineers, and The University of Michigan Office of Research Administration. Work was done under the direction of Dr. E. F. Brater, Professor of Hydraulic Engineering, in The University of Michigan Lake Hydraulics Laboratory.

DESIGN WAVES

Maximum storm waves were determined primarily from two previous investigations carried out by one of the writers[1,2]. The study dealing specifically

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[1]"Investigations of Wave Action and Wave Forces at the Proposed Generating Station near Pigeon Lake, Michigan", E.F. Brater, for Commonwealth Associates, Inc., April 23, 1960.

[2]"Extreme Levels of Lake Erie near Monroe, Michigan", E.F. Brater and H.W. Baynton, for the Detroit Edison Company, July, 1956.

with the eastern shore of Lake Michigan[1] was done to aid in the design of the Consumers Power Company's Campbell plant. For this study the National Weather Records Center was asked to examine the wind records at the locations shown in Table I. The periods for which records were available at each station are also shown. The Weather Records Center was asked to search for the five largest average hourly winds of record which occurred during separate wind storms in the sector from N.W. through W. to S.W. and then to supply the wind speeds for the 10 hours preceding and following the maximum hour. After becoming more familiar with the magnitude of the winds during major storms the procedure was modified to request the wind data for only those storms which had a wind in excess of a selected value for 5 or more consecutive hours.

TABLE I

WIND RECORDS INVESTIGATED

Station	Period of Records
Chicago, Illinois	1872-1959
Green Bay, Wisconsin	1902-1959
Grand Rapids, Michigan	1902-1959
Milwaukee, Wisconsin	1906-1959
Grand Haven, Michigan	1906-1933

Having found all of the largest winds in each of these locations additional data were obtained as needed to estimate the maximum average wind on Lake Michigan for the three directions, N.W., W., and S.W., for durations varying from 6 to 10 hours. Before averaging the velocities at the two sides of the lake the wind velocities were reduced to the probable value 26 feet above the ground (or water) surface so that wave heights and periods could be determined from Bretschneider's curves [3]. Before estimating the corresponding wave heights for the three directions the wind velocities were increased arbitrarily by 10 per cent to allow for the possibility of occurrence of winds larger than those recorded during the periods of records. The largest wind storms for the three directions as determined from this investigation and increased by 10 per cent are shown in Table II.

TABLE II

MAXIMUM WIND VELOCITIES
Averages over the Lake in Knots

Duration in hrs.	Direction and Date of Storm		
	S. W. Nov. 11, 1940	W. Nov. 16, 1955	N. W. April 7, 1909
6	37.3	35.4	34.6
7	37.9	35.7	33.2
8	37.6	35.5	31.5
9	37.1	35.6	31.8
10	36.7	35.4	28.3

The second intensive wind velocity investigation [2] was made for the purpose of calculating the maximum positive and negative wind tides at the Enrico Fermi power plant located on Lake Erie. For that purpose the maximum winds of record near Lake Erie were studied exhaustively and a supplementary search was made to determine if any larger winds had occurred elsewhere in the region bounded on the west by a line from St. Paul, Minnesota, to St. Louis, Missouri, and on the east by the Atlantic Ocean. No winds were found greater than those which occurred over Lake Erie where the maximum southwesterly wind of record was 35 knots for a duration of nine hours. When this velocity is increased by 10 per cent it becomes 38.5 knots which is slightly greater than the values for S.W. shown in Table II.

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[3] "Revisions in Wave Forecasting: Deep and Shallow Water", C. L. Bretschneider, Proc. 6th Conf. on Coastal Engr., Council on Wave Research, 1958, pp. 30-67.

TABLE III

PROBABLE WAVE HEIGHTS AND PERIODS PRODUCED BY MAXIMUM WINDS*

Direction	Fetch in Miles	Wave Height in Feet H_0	Wave Height in Feet H_G	Wave Period (T) Seconds	Depth at Gage (d_G) Feet
N.W.	70	11.0	9.3	8.2	31
W.	65	11.3	10.5	8.2	33
S.W.	98	14.8	12.6	9.7	35

The waves shown in Table III were used as the design waves for the model tests. Wave heights were estimated from the fetches for each direction by selecting the combinations of wind duration and velocity from Table II which gave the maximum wave height[3]. The fetches and maximum wave heights for each direction are shown in Table III. The computed deep water wave heights (H_0) and period (T) (as well as the wave height at the monitoring gages (H_G)) are also shown. For each wind direction the waves generated by the wave machine were monitored at a location outside the harbor. At these locations some changes in the height and orientation of the waves had taken place due to refraction and small shallow depths. Values of H_G, the computed wave heights at these locations, are shown in Table III. The depths at the gage locations (d_G) are shown in Table III.

It should be mentioned that a publication of the Corps of Engineers[4] gives estimated wave heights on Lake Michigan for the years 1948, 1949, and 1950. Wave heights are given for Frankfort and Muskegon, Michigan. Because Ludington lies between these two locations the average might be considered to apply to Ludington. On this basis, it is estimated that the three year

[4] "Wave and Lake Level Statistics for Lake Michigan", Tech. Mem. No. 36, Beach Erosion Board, Corps of Engineers, 1953.

*These waves would result from winds 10 per cent larger than the largest winds of record during periods of records varying from 27 to 87 years at the five gages on Lake Michigan. Comparable periods of records were studied on Lake Erie. One could therefore expect that these wave heights would be very rare occurrences, probably having frequencies larger than 100 years.

frequency wave height for the entire westerly sector is 16.5 feet and the one year wave is 12.5 feet. The writer's much more extensive studies including the years 1948-50 showed no winds capable of producing waves larger than 14.8 feet. It is the opinion of the writer that very conservative techniques were used in interpreting wind data in the Corps of Engineers' publication.

The wave heights (H_0) discussed here and the ones simulated in the model tests are called "significant wave heights". They represent the average of the highest one-third of a group of natural waves. Various studies of wave spectra in nature indicate that the average of the highest ten per cent of the waves is 1.1 H_0, and the average of the highest one per cent of the waves is 1.6 H_0. The test results provide a means of estimating the size within the harbor of waves larger than H_0.

The wave heights and periods used for most of the model tests are those shown in Table III. However, three series of tests were also conducted with waves of about half the size of those shown in Table III.

THE MODEL AND TESTING PROCEDURE

A tank having the dimensions 45 feet by 40 feet was constructed and the model was located at one side of the tank as shown in Fig. 1. Photographs are shown in Plate I and II. The scale ratio of the model was 150 to 1. This scale provided the largest model that could be built while still providing room to maneuver the wave machine into various posiitions so that waves in the sector from S.W. to N.W. could be simulated. The lake bottom was built of concrete grout about one inch thick, placed on compacted sand. The bottom topography was reproduced by means of plywood templates in accordance with soundings provided by EBASCO Services and by data from U.S.

Lake Survey charts in regions not covered by the soundings. The two series of soundings were not in exact agreement and therefore some adjustments were made where the two sets of bottom topography overlapped. A contour map is shown in Fig. 2. The contours were checked at the beginning of the testing program and again near the middle of the testing program. The checks showed contours in their proper relative location but water depths were as much as 0.02 feet too large on the south side of the model. This corresponds to an error of 3 feet in the prototype depths. During the latter part of the testing program, spot checks were made at important locations to be sure that no significant additional changes occurred in the bottom elevation.

The design water surface elevation was 579.5. The model water surface elevation was checked during the tests by means of a hook gage located in one corner of the tank. One series of tests was made with the water surface five feet higher.

The model limits did not extend to deep water; therefore, for each direction it was necessary to compute the changes in the magnitude and orientation of the waves caused by refraction and changes in depth which affect the waves as they travel from deep water to a selected gage location. The wave machine was oriented and adjusted to correctly simulate these computed waves at the gage locations. In this manner the tests reproduced the waves from southwest, west and northwest shown in Table III. Wave absorbers were installed around the walls of the tank to prevent any waves from reflecting from the tank walls.

The wave heights were measured by resistance gages which provide a relation between wave height and the displacement of a recording pen on an oscillograph. The gages were calibrated before each test. Two oscillograph channels provided continuous, simultaneous records of wave height at a location in the harbor and at the selected gage location outside of the harbor. The gage inside the harbor was mounted on a frame which permitted movement to various locations in the harbor.

The discharge through the powerhouse was simulated by means of a pump. The piping system was designed to permit changing from the generating phase (outflow) to the pumping phase (inflow) by operating valves. The water was circulated from the model to the pump through the tank by means of perforated pipes located at the tank walls on both sides of the model. The piping was so arranged that all the water could be drawn in through (or be discharged from) either of the pipes. Thus it was possible to simulate a northerly current during southwesterly storms and a southerly current during northwesterly storms. During westerly storms or during current measurements made without wave action, the flow could be divided between the two perforated pipes.

The model discharge was regulated in accordance with the Froude model law. The discharge was measured by means of a pipe orifice installed in the system.

Currents were measured in two ways. Surface currents were measured by distributing confetti on the water surface and making a time exposure with an elevated camera. A grid of 150 foot squares painted on the model floor (Fig. 3) facilitated the interpretation of these photographs. Sub-surface velocities were measured by injecting small amounts of milk at selected locations by means of a slender hollow needle. The movement of the small volume of white milk solution was timed with a stop watch as it moved across the grid system.

The various parts of the harbor are given consistent names throughout this report. As shown in Fig. 3, the two piers extending outward from the shore on either side of the powerhouse are called "jetties". The inside of the shoreward portion of the jetties are referred to as revetments, and the outer breakwater is simply called the "breakwater". Changes in the manner of construction, length or location of these structural features could be readily made in the model. Low strength concrete was used to construct the vertical walled jetties and rubble mound construction was simulated by typical stones reduced to model size.

THE TESTING PROGRAM

Wave heights in the vicinity of the powerhouse were given the major attention during the first part of the testing program. The original harbor dimensions are shown in Fig. 4 and the various other harbor arrangements tested are shown in Fig. 5. Maximum design wave heights and periods for the three wind directions tested are shown in Table III. For some velocity tests the outer ends of the jetties were curved in various ways as shown in Fig. 6. Many other modifications of the arrangements shown were examined briefly and discarded. The plans differ not only in arrangement of the jetties and the breakwater but in the nature and crest elevations of the structures. The wave action in the vicinity of the powerhouse was primarily caused by reflection from the jetties. It was considered essential to be able to compare results from a vertical walled structure, which produces maximum wave reflection, with those obtained from rubble mound structures, which provide much less reflection. A zig-zag wall on the inner side of the jetties was also tested for a number of conditions.

Three series of tests were repeated with smaller waves having a shorter period to determine whether the particular design wave size or period influenced the selection of the most effective plans. Another series of tests was repeated for approaching waves oriented 10° in either direction from the design direction This was done to be sure that the selection of the most effective breakwater arrangement was not influenced by a particular wave orientation. The effectiveness of various breakwater arrangements is discussed in some detail in the next section.

After learning that wave heights near the structure could be effectively controlled, tests on currents were undertaken. The surface currents were measured with and without waves by broadcasting confetti on the model and taking aerial photographs with two or three second exposure times. These tests were made for both the generating and the pumping phases with and without wave action. When the currents were measured during wave action the model pumping arrangement was adjusted to reproduce currents to the north during southwesterly winds and currents toward the south during northwesterly winds. For westerly winds or no winds the inflow or outflow was divided on the two sides of the model.

As the velocity testing program progressed to a certain point it was decided that subsurface velocities should be measured rather than surface movement. Consequently the procedure was changed and velocities were determined by observing the movements of small amounts of color injected at selected locations. No wave motion was created during measurements of subsurface velocities.

As the current measurements progressed, changes in the breakwater arrangements were made for the purpose of reducing currents near the ends of the jetties. Some of these included openings in the breakwater. When new breakwater arrangements were found to be promising in reducing velocities they were subjected to a wave test to determine if they would produce satisfactory wave conditions near the powerhouse.

TEST RESULTS - WAVE HEIGHTS

The wave heights inside the model were measured at three points 75 feet from the ends of the gate piers of the powerhouse (0.5C, 0.5D, and 0.5E) and at three points 225 feet from the structure (1.5C, 1.5D, and 1.5E). The points may be located by means of the coordinate system shown in Fig. 3. The test results are reported in Tables IV through XIV. At the top of each tabulation is shown the wind direction, the deep water wave heights (H_O), the wave height at the gage used to monitor the incoming wave (H_G) and the wave period (T). Other headings designate the nature of the structure being tested (vertical walls, rubble mound, etc.), the Plan Number, and the flow conditions (pumping, generating, or no flow). The various Plans are shown in Fig. 5. The values shown for each location and condition are the significant wave height converted to prototype conditions and the percentage of the wave height at the monitoring gage (H_G) remaining. The wave height is shown in the table in the lower left hand corner of each space and the per cent remaining in the upper right hand corner. For example, in Table IV for the location 0.5C the wave height is 1.0 foot and the percentage of wave height remaining (100 x 1.0/12.6) is 8. Average values are also shown for each set of three measurements on coordinate lines 0.5 and 1.5. Preceding each table is a short statement giving relevant information and some conclusions about the series of tests presented in the table. The more general conclusions are presented in the next section. An estimate of the size of waves in the harbor for deep water waves other than those tested can be made by assuming that the percentage remaining for large waves will be the same as those obtained in the tests for the design waves. For smaller waves the same procedure can be followed using the percentage obtained in the small wave tests. Because the percentage values given in the tables are based on the wave heights at the gages it is necessary to decrease the percent remaining to compare results with deep water waves. The factors are: S.W., .84; W., .93; N.W., .85. For example if the percentage remaining is 20 for a S.W. wave the percent of the deep water wave remaining is 20 x .84 = 17%.

WAVE HEIGHTS NEAR THE POWER HOUSE

For Three Wind Directions for Various Types of Construction and Flow Conditions

TABLE IV

The original harbor configuration shown in Fig. 4 and in Fig. 5, Plan 1, was tested for waves simulating those generated by winds from the N.W., W., and S.W. The effectiveness of this plan was investigated for three types of jetty construction, with vertical walls, Fig. 5 (Plan 1), rubble mound jetties extending from the end of the revetment to outermost point (Plan 5), and a zig-zag vertical wall from the end of the revetment to outermost point (Plan 4). Table IV provides values for each condition tested. The effectiveness of rubble or zig-zag construction is in the damping action which partially prevents reflection from the inner faces of the breakwaters. If, for example, waves are approaching from the S.W., the major portion of the reflection occurs from the inner face of the N. breakwater. Some tests in this group were conducted with the absorptive type of construction placed only on the side causing the principal reflection (Plans 2 and 3). This was done only in this series, in Table VII and in Table XIV. These results show that when the inner faces of both jetties have rubble construction the resulting wave height is approximately 83 per cent of the wave height when this construction is applied to one side only. When zig-zag walls are used on both sides the wave height is reduced to 87 per cent of the value obtained with zig-zag walls on one side only.

Rubble mound jetties were tested for many other structures in addition to those shown in Table IV providing a total of 30 tests for comparison with vertical walled jetties. The use of rubble jetties caused an average reduction in wave height of 34 per cent over wave heights occurring with vertical walled jetties. For example, if the wave height at a particular location near the

Power House is 3.0 feet for vertical walled jetties it will be approximately $(3.0 - 3.0 \times .34) = 2.0$ feet if rubble jetties are used.

Zig-zag walls are used only in one other series of tests (Table VII), providing a total of nine test conditions from which the effect of zig-zag walls could be determined. The average reduction in wave heights caused by the use of zig-zag walls as compared with vertical walled jetties is 13 per cent.

TABLE IV a

WAVE HEIGHTS NEAR THE POWER HOUSE

For Three Wind Directions for Various Types of Construction and Flow Conditions

Location	S.W. WAVE $H_O = 14.8$ ft. $H_G = 12.6$ ft. T = 9.7 sec.								
	Vertical Walled Jetties Plan 1			Rubble Jetties Plan 5			Rubble Jetties (One Side) Plan 3		
	Pumping	No Flow	Generating	Pumping	No Flow	Generating	Pumping	No Flow	Generating
0.5C	1.0 8	2.6 21	4.3 34	0.3 2	1.0 8	1.4 11	1.0 8	1.8 14	2.1 17
0.5D	0.8 6	1.0 8	2.0 16	2.3 18	1.5 12	1.0 8	1.3 11	0.8 6	1.4 14
0.5E	1.5 12	1.4 11	3.0 24	0.8 6	1.0 8	2.3 18	1.8 14	1.6 13	2.4 19
AVERAGE	1.1 9	1.7 13	3.1 25	1.1 9	1.2 9	1.6 12	1.4 11	1.4 11	2.1 17
1.5C	2.6 21	4.5 36	4.7 37	2.6 21	3.0 24	3.5 28	2.7 21	3.4 27	4.2 33
1.5D	1.4 11	0.8 7	4.7 37	2.5 20	2.8 22	3.3 26	1.8 14	4.2 33	3.7 29
1.5E	3.0 24	4.2 33	4.8 38	2.9 23	4.7 37	3.7 29	3.7 29	5.6 45	5.0 40
AVERAGE	2.3 19	3.2 25	4.7 37	2.7 21	3.5 28	3.5 28	2.7 21	4.4 35	4.3 34

TABLE IVb

WAVE HEIGHTS NEAR THE POWER HOUSE

For Three Wind Directions for Various Types of Construction and Flow Conditions

Location	S.W. WAVE H_O = 14.8 ft. H_G = 12.6 ft. T = 9.7 sec.						N.W. Wave H_O = 11.0 ft. H_G = 9.3 ft. T = 8.2 sec.		
	Zig-Zag Jetties Plan 4			Zig-Zag Jetties (One Side) Plan 2			Vertical Walled Jetties Plan 1		
	Pumping	No Flow	Generating	Pumping	No Flow	Generating	Pumping	No Flow	Generating
0.5C		13 1.7	26 3.3	12 1.5	14 1.8	27 3.4	27 2.5	15 1.4	22 2.0
0.5D		10 1.2	13 1.7	8 1.0	9 1.1	20 2.5	19 1.8	11 1.0	22 2.0
0.5E		18 2.2	17 2.1	15 1.9	19 2.4	21 2.6	27 2.5	22 2.0	22 2.0
AVERAGE		14 1.8	19 2.4	12 1.5	14 1.8	23 2.8	24 2.3	16 1.5	22 2.0
1.5C		26 3.3	29 3.6	21 2.6	33 4.2	33 4.2	19 1.8	34 3.2	43 4.0
1.5D		20 2.5	23 2.9	11 1.4	14 1.8	29 3.7	32 3.0	36 3.3	29 2.7
1.5E		42 5.2	29 3.6	30 3.8	40 5.0	33 4.2	22 2.0	26 2.4	16 1.5
AVERAGE		29 3.7	27 3.3	21 2.6	29 3.7	32 4.0	24 2.3	32 3.0	29 2.7

TABLE IVc

WAVE HEIGHTS NEAR THE POWER HOUSE

For Three Wind Directions for Various Types of Construction and Flow Conditions

Location	N.W. WAVE H_O = 11.0 ft. H_G = 9.3 ft. T = 8.2 sec.						W. WAVE H_O = 11.3 ft. H_G = 10.5 ft. T = 8.2 sec.		
	Rubble Jetties (One Side) Plan 3			Zig-Zag Jetties (One Side) Plan 2			Vertical Walled Jetties Plan 1		
	Pumping	No Flow	Generating	Pumping	No Flow	Generating	Pumping	No Flow	Generating
0.5C	1.3 14	2.0 21	0.9 10	2.5 27	2.0 21		1.5 14	1.3 12	2.7 26
0.5D	1.5 16	1.3 14	0.7 7	1.8 19	2.4 26		0.9 9	1.5 14	3.0 29
0.5E	2.0 22	1.0 11	0.7 7	2.5 27	1.7 18		0.9 9	1.2 11	3.0 29
AVERAGE	1.6 17	1.4 15	0.8 8	2.3 24	2.0 22		1.2 11	1.3 12	3.0 29
1.5C	1.3 14	2.4 26	3.1 33	1.8 19	2.7 29		1.7 16	2.1 20	2.5 24
1.5D	2.0 22	0.7 7	3.3 35	3.0 32	3.2 34		0.9 9	1.6 15	2.8 27
1.5E	1.3 14	1.0 11	1.9 20	2.0 22	2.5 27		0.9 9	0.8 8	3.0 29
AVERAGE	1.5 17	1.4 15	2.8 27	2.3 24	2.8 30		1.2 11	1.5 14	2.8 27

EXTENSION OF BREAKWATER TO THE NORTH

TABLE V

The breakwater was extended 233 feet to the north making the total length 1933 feet. This extension produced a symmetrical harbor arrangement. The data shown in Table V permit the evaluation of the effect of this extension during N.W. waves. The addition of the 233 feet of breakwater produced an average reduction in wave height during northwesterly storms of 45 per cent. Because of the symmetry of the harbor it can be assumed a comparable adverse effect would result during southwesterly waves from reducing the length of the breakwater at the south end. This reasoning would indicate that the wave height could be expected to increase approximately 82 per cent during southwesterly waves if the breakwater is shortened by 233 feet at the south end.

TABLE V

EXTENSION OF BREAKWATER TO THE NORTH

Location	N.W. WAVE $H_O = 11.0$ ft. $H_G = 9.3$ ft. $T = 8.2$ sec.					
	Rubble Jetties (One Side)				Vertical Jetties	
	Generating		No Flow		Generating	
	Original Plan 3	Extended Plan 6	Original Plan 3	Extended Plan 6	Original Plan 1	Extended Plan 7
0.5C	10 0.9	7 0.7	21 2.0	7 0.7	19 1.8	13 1.2
0.5D	7 0.7	7 0.7	14 1.3	7 0.7	19 1.8	14 1.4
0.5E	7 0.7	7 0.7	11 1.1	9 0.8	26 2.4	19 1.7
AVERAGE	8 0.8	7 0.7	16 1.5	8 0.7	21 2.0	15 1.4
1.5C	33 3.1	11 1.0	26 2.4	17 1.6	63 5.9	19 1.8
1.5D	35 3.3	15 1.4	7 0.7	11 1.1	29 2.7	16 1.5
1.5E	20 1.9	11 1.1	11 1.1	9 0.8	26 2.4	11 1.1
AVERAGE	29 2.7	12 1.2	15 1.4	12 1.2	39 3.7	16 1.4

VARYING DIRECTION OF APPROACH OF SOUTHWESTERLY WAVES

TABLE VI

These tests were run to determine if the selection of a wind exactly from the S.W. produced typical conditions in the harbor during a southwesterly storm. This guards against the possibility that the harbor is especially sensitive to the particular direction of approach. Sets of tests were conducted with the orientation of the wave crests at the gage location 10° in each direction from the design direction. The southwesterly design waves are affected by refraction in such a manner that at the wave gage the angle with a north-south line (α) is 26°. These check tests were conducted with $\alpha = 36°$ and $\alpha = 16°$ respectively. The same design wave height was maintained at the gage location (H_G). The design wave is somewhat high for $\alpha = 36°$. Waves approaching from this direction would be generated over a longer fetch than the design condition (135 miles instead of 98 miles) and would be about 16 feet high in deep water. However, the refraction would be so much greater for this direction that the wave height at the gage location would be about 10.9 feet rather than 12.6 feet. Therefore it can be concluded that our values of $\alpha = 36°$ are about 13 per cent too high. The data shown in Table VI are arranged so that they can be compared with the design wave ($\alpha = 26°$) for vertical and rubble mound jetties for three flow conditions. The results of these tests indicate that using $\alpha = 26°$ provides results which are similar to those obtained from the two other directions.

TABLE VI

VARYING DIRECTION OF APPROACH

Location	S.W. WAVES (α = 26°) H_O = 14.8 ft. H_G = 12.6 ft. T = 9.7 sec.											
	Vertical Walled Jetties Plan 1							Rubble Jetties Plan 5				
	Generating		No Flow			Pumping		Generating		No Flow		
	α=26°	α=16°	α=36°	α=26°	α=16°	α=36°	α=26°	α=36°	α=26°	α=36°	α=26°	α=16°
0.5C	4.3 34	4.0 32	1.1 9	2.6 21	2.0 16	2.3 18	1.0 8	1.5 12	1.4 11	1.1 9	1.0 8	1.8 14
0.5D	2.0 16	2.9 23	1.4 11	1.0 8	1.9 15	2.6 21	0.8 6	1.5 12	1.0 8	0.9 7	1.5 12	1.4 11
0.5E	3.0 24	1.4 11	0.9 7	1.4 11	1.6 12	2.3 18	1.5 12	1.4 11	2.3 18	1.1 9	1.0 8	2.0 16
AVERAGE	3.1 25	2.8 22	1.1 9	2.7 13	1.8 14	2.4 19	1.1 9	1.5 12	1.6 12	1.0 8	1.2 9	1.7 14
1.5C	4.7 37	3.6 28	2.7 22	4.5 36	3.3 26	4.6 37	2.6 21	2.1 17	3.5 28	1.8 14	3.0 24	2.4 19
1.5D	4.7 37	4.0 32	2.4 19	0.8 7	3.6 28	3.4 27	1.4 11	1.9 15	3.3 26	2.4 19	2.8 22	3.2 25
1.5E	4.8 38	3.0 24	2.3 18	4.2 33	3.6 28	3.8 30	3.0 24	2.4 19	3.6 29	2.3 18	4.7 37	2.8 22
AVERAGE	4.7 37	3.5 28	2.5 20	3.2 25	3.5 27	3.9 31	2.3 19	2.1 17	3.5 28	2.2 17	3.5 28	2.8 22

COMPARISON OF REDUCTION IN WAVE HEIGHT FOR WAVES OF TWO SIZES

TABLE VII

For the S.W. direction, measurements were made with wave heights approximately half the size and with a smaller period than the design waves. The purpose of these tests was to determine if this harbor arrangement is as effective in reducing wave heights for waves of a smaller height and period as it is for the design waves and to determine to what extent decisions based on these tests are sensitive to the selected design wave heights. The smaller wave had a height at the gage of 6.0 feet and a period of 8.4 seconds. The corresponding deep water wave height is approximately 6.8 feet. Waves of this size can be expected a number of times each year. Results are presented in Table VII in a form which permits convenient comparison. The average wave height remaining at the three gage positions located 75 feet from face of the structure (.5C, .5D, and .5E) was 17% for the larger waves and 10% for the smaller waves. The corresponding remaining wave height at gage positions 225 feet from the structure (1.5C, 1.5D, and 1.5E) was 29% for the larger waves and 18% for the smaller waves. Thus indicating that this harbor is more effective for smaller more frequent waves than for the rarer large waves.

Another method of analyzing the data in this table is to determine if the smaller wave would have led to the same conclusion regarding the effectiveness of the four types of construction used in this series of tests. This was done by adding the percentage of wave height remaining in the various positions. These results are shown below. The three major types of construction are arranged in the order of decreasing effectiveness as determined by the large waves. It may be seen that the same order would be obtained with the smaller waves except that the zig-zag walled jetties are slightly more effective than the rubble mound jetties.

	Large Wave	Small Wave
Rubble mound jetties	78	47
Zig-zag walled jetties	84	45
Vertical walled jetties	100	72

TABLE VIIa

COMPARISON OF REDUCTION IN WAVE HEIGHT FOR WAVES OF TWO SIZES

Location	S.W. WAVE Small Wave: H_0 = 6.8 ft. H_G = 6.0 ft. T = 8.4 sec. Large Wave: H_0 = 14.8 ft. H_G = 12.6 ft. T = 9.7 sec.							
	Rubble Jetties Plan 5				Vertical Walled Jetties Plan 1			
	Generating		No Flow		Generating		No Flow	
	H_G=12.6	H_G=6.0	H_G=12.6	H_G=6.0	H_G=12.6	H_G=6.0	H_G=12.6	H_G=6.0
0.5C	11 1.4	10 0.6	8 1.0	12 0.8	34 4.3	13 0.8	21 2.6	10 0.6
0.5D	8 1.0	10 0.6	12 1.5	5 0.3	16 2.0	13 0.8	8 1.0	13 0.8
0.5E	18 2.3	13 0.8	8 1.0	3 0.2	24 3.0	10 0.6	11 1.4	10 0.6
AVERAGE	13 1.6	11 0.7	9 1.2	7 0.4	25 3.1	12 0.7	13 1.7	11 0.7
1.5C	28 3.5	10 0.6	24 3.0	13 0.8	37 4.7	28 1.7	36 4.5	13 0.8
1.5D	26 3.3	15 0.9	22 2.8	17 1.1	37 4.7	32 2.0	7 0.8	37 2.3
1.5E	29 3.7	18 1.1	37 4.7	15 0.9	38 4.8	17 1.1	33 4.2	20 1.2
AVERAGE	28 3.5	14 0.9	28 3.5	15 0.9	37 4.7	26 1.6	25 3.2	23 1.4

TABLE VIIb

COMPARISON OF REDUCTION IN WAVE HEIGHT FOR WAVES OF TWO SIZES

Location	S.W. WAVE Small Wave: H_O = 6.8 ft. H_G = 6.0 ft. T = 8.4 sec. Large Wave: H_O = 14.8 ft. H_G = 12.6 ft. T = 9.7 sec.							
	Zig-Zag Jetties (One Side) Plan 2				Zig-Zag Jetties Plan 4			
	Generating		No Flow		Generating		No Flow	
	H_G=12.6	H_G=6.0	H_G=12.6	H_G=6.0	H_G=12.6	H_G=6.0	H_G=12.6	H_G=6.0
0.5C	3.4 27	0.8 13	1.8 14	0.3 5	3.3 27	0.8 12	1.7 13	0.5 8
0.5D	2.5 20	1.1 17	1.1 9	0.3 5	1.7 13	0.5 8	1.2 10	0.5 7
0.5E	2.6 21	0.8 13	2.4 19	0.6 16	2.1 17	0.8 12	2.2 18	0.3 5
AVERAGE	2.8 27	0.9 14	1.8 14	0.4 7	2.4 19	0.7 11	1.7 14	0.4 7
1.5C	4.2 23	1.4 22	4.2 33	0.6 10	3.6 29	0.8 12	3.3 26	0.3 5
1.5D	3.7 29	1.8 30	1.8 14	1.4 23	2.9 23	0.3 5	2.5 20	1.5 25
1.5E	4.2 33	1.1 18	5.0 40	0.8 12	3.6 29	0.8 13	5.2 42	1.2 20
AVERAGE	4.0 32	1.4 23	3.7 29	0.9 15	3.4 27	0.6 10	3.7 24	1.0 17

DIVERGING WALLS

TABLE VIII

Wave tests were made for the S.W. direction with the large wave (H_G = 12.6) after installing vertical diverging walls which extended in straight lines from the north and south ends of the gate openings to the jetties near Station 5 (Plan 8). The diverging walls were installed to eliminate the large eddies which were generated in the inner corners of the harbor. The results of the wave height measurements are shown in Table VIII. It may be seen that the presence of these vertical walls created higher waves during the no flow and pumping phases of operation.

TABLE VIII

DIVERGING WALLS

Location	S.W. WAVE H_O = 14.8 ft. H_G = 12.6 ft. T = 9.7 sec.					
	Vertical Walled Jetties					
	Generating		No Flow		Pumping	
	Original Conditions Plan 1	Diverging Walls Plan 8	Original Conditions Plan 1	Diverging Walls Plan 8	Original Conditions Plan 1	Diverging Walls Plan 8
0.5C	4.3 34	3.0 24	2.6 21	4.0 32	1.0 8	2.6 21
0.5D	2.0 16	3.0 24	1.0 8	4.8 38	0.8 6	3.4 27
0.5E	3.0 24	2.5 20	1.4 11	3.3 27	1.5 12	2.3 18
AVERAGE	3.3 25	2.8 23	1.7 13	4.0 32	1.1 9	2.8 22
1.5C	4.7 37	4.4 35	4.5 36	7.0 55	2.6 21	5.0 40
1.5D	4.7 37	4.4 35	0.8 7	4.8 38	1.4 11	4.6 37
1.5E	4.8 38	5.1 40	4.2 33	3.3 26	3.0 24	3.8 30
AVERAGE	4.7 37	4.6 37	3.2 25	5.0 40	2.3 19	4.5 36

RUBBLE REVETMENTS

TABLE IX

All tests prior to this series were made with the revetments (see Fig. 4) constructed of smooth concrete. In this series the vertical walled jetties were tested with the smooth concrete revetments replaced by rubble (Plan 11) and with the revetment removed entirely (Plan 13). The rubble jetties were tested with the smooth revetments replaced by rubble (Plan 12).

In the case of vertical walled jetties (Plan 1) the use of rubble revetments (Plan 11) or no revetments (Plan 13) reduced wave heights by approximately 20 per cent. For rubble mound jetties (Plan 5) the tests showed a reduction of about 45 per cent due to the use of rubble revetments (Plan 12). Compared another way, waves measured near the powerhouse with rubble jetties and rubble revetments (Plan 12) were approximately 60 per cent less than those measured with vertical jetties and smooth concrete revetments (Plan 1).

TABLE IX

RUBBLE REVETMENTS

Location	S.W. WAVE H_0 = 14.8 ft. H_G = 12.6 ft. T = 9.7 sec.									
	Vertical Walled Jetties						Rubble Jetties			
	Generating			No Flow			Generating		No Flow	
	Concrete Revetments Plan 1	Rubble Revetments Plan 11	No Revetment Plan 13	Concrete Revetments Plan 1	Rubble Revetments Plan 11	No Revetment Plan 13	Concrete Revetments Plan 5	Rubble Revetments Plan 12	Concrete Revetments Plan 5	Rubble Revetments Plan 12
0.5C	4.3 / 34	2.5 / 20	3.3 / 26	2.6 / 21	1.5 / 12	2.4 / 19	1.4 / 11	1.4 / 11	1.0 / 8	0.6 / 5
0.5D	2.0 / 16	3.0 / 24	2.8 / 22	1.0 / 8	1.4 / 11	1.5 / 12	1.0 / 8	1.1 / 9	1.5 / 12	0.6 / 5
0.5E	3.0 / 24	2.5 / 20	2.3 / 18	1.4 / 11	1.8 / 14	1.0 / 8	2.3 / 18	1.4 / 11	1.0 / 8	1.1 / 9
AVERAGE	3.1 / 25	2.6 / 21	2.8 / 22	1.7 / 13	1.5 / 12	1.6 / 13	1.6 / 12	1.3 / 10	1.2 / 9	0.8 / 6
1.5C	4.7 / 37	2.5 / 20	3.4 / 27	4.5 / 36	3.0 / 24	3.3 / 26	3.5 / 28	1.9 / 15	3.0 / 24	1.5 / 12
1.5D	4.7 / 37	3.8 / 30	3.3 / 26	0.8 / 7	2.3 / 18	2.4 / 19	3.3 / 26	1.4 / 11	2.8 / 22	0.9 / 7
1.5E	4.8 / 38	3.8 / 30	2.3 / 18	4.2 / 33	1.4 / 11	2.5 / 20	3.7 / 29	1.1 / 9	4.7 / 37	1.9 / 15
AVERAGE	4.7 / 37	3.4 / 27	3.0 / 24	3.2 / 25	2.3 / 18	2.8 / 22	3.5 / 28	1.5 / 12	3.5 / 28	1.4 / 11

RUBBLE MOUND BREAKWATER

TABLE X

A number of tests were made for the S.W. direction to determine whether wave conditions would be improved by changing the breakwater from a vertical walled structure to a rubble mound structure. For two of the three conditions tested the waves were greatly reduced, but the third condtition showed an increase in wave height. The average of all tests shows a reduction in wave heights of 21 per cent due to the rubble construction of the breakwater.

TABLE X

RUBBLE MOUND BREAKWATER

Location	S.W. WAVE $H_0 = 14.8$ ft. $H_G = 12.6$ ft. T = 9.7 sec.					
	Vertical Walled Jetties				Rubble Jetties	
	Generating		No Flow		No Flow	
	Vertical Breakwater Plan 1	Rubble Breakwater Plan 9	Vertical Breakwater Plan 1	Rubble Breakwater Plan 9	Vertical Breakwater Plan 5	Rubble Breakwater Plan 10
0.5C	4.7 38	2.7 22	2.6 21	4.4 35	0.9 7	0.9 7
0.5D	3.7 29	2.5 20	1.0 8	1.5 12	1.9 15	0.9 7
0.5E	2.6 20	1.6 13	1.4 11	1.5 12	1.3 10	1.0 8
AVERAGE	3.7 29	2.3 18	1.7 13	3.7 29	1.4 11	0.9 7
1.5C	5.1 41	3.7 29	4.5 36	5.5 43	1.4 11	1.4 11
1.5D	5.5 43	2.5 20	0.8 7	2.6 21	1.6 13	0.3 2
1.5E	2.9 23	2.0 16	4.2 33	1.8 15	2.5 20	1.6 13
AVERAGE	4.5 36	2.7 22	3.2 25	3.3 26	1.9 15	1.1 9

LOWER BREAKWATER AND JETTIES

TABLE XI

This series of tests was made to determine what effect lowering the breakwater and jetties had on the wave heights near the powerhouse. The breakwater was lowered five feet, from Elev. 596 to 591, and wave heights near the powerhouse were measured for Plan 14, 15 and 16 during SW waves. These measurements are shown in Table XIa, XIb, and XIc, along with the wave heights measured for a harbor having the same geometry and the original breakwater elevation (Plans 1, 5, and 12). The last four columns of Table XIc compare the lower breakwater (Plan 16) and the original breakwater (Plan 1) during the small SW wave (H_0 = 6.8 ft.). Wave heights with the lower breakwater were also measured during W. waves (Plans 15, 16). These measurements are shown in Table XId along with the wave heights measured for a harbor having the same geometry and the original breakwater elevation (Plan 1).

Four additional tests were made with the breakwater lowered, from Elev. 596 to 591,and the jetties lowered six feet, from Elev. 596 to 590. These tests were made for Plan 17 and 18 during SW waves. They are shown with all comparable tests in Table XIa and XIb.

The tests indicate that lowering the crest elevations as indicated does not cause any increase in wave heights near the powerhouse for the range of wave heights used in these tests.

TABLE XIa

LOWER BREAKWATER AND JETTIES

Location	S.W. WAVE H_O = 14.8 ft. H_G = 12.6 ft. T = 9.7 sec.					
	Vertical Walled Jetties					
	Generating			No Flow		
	Original Plan 1	Lower Breakwater Plan 16	Lower Brw. & Jetties Plan 17	Original Plan 1	Lower Breakwater Plan 16	Lower Brw. & Jetties Plan 17
0.5C	4.3^{34}	4.5^{36}	4.2^{33}	2.6^{21}	3.3^{26}	3.5^{28}
0.5D	2.0^{16}	2.4^{19}	3.0^{24}	1.0^{8}	1.4^{11}	2.0^{16}
0.5E	3.0^{24}	3.3^{26}	2.1^{17}	1.4^{11}	1.5^{12}	1.8^{14}
AVERAGE	3.1^{25}	3.4^{27}	3.2^{25}	1.7^{13}	2.5^{20}	2.4^{19}
1.5C	4.7^{37}	2.8^{22}	2.8^{22}	4.5^{36}	3.7^{29}	2.3^{18}
1.5D	4.7^{37}	4.2^{33}	3.3^{26}	0.8^{7}	2.7^{21}	2.3^{18}
1.5E	4.8^{38}	4.8^{38}	4.7^{37}	4.2^{33}	2.3^{18}	2.1^{17}
AVERAGE	4.7^{37}	3.9^{31}	3.5^{28}	3.2^{25}	2.9^{23}	2.3^{18}

TABLE XIb

LOWER BREAKWATER AND JETTIES

Location	S.W. WAVE H_0 = 14.8 ft. H_G = 12.6 ft. T = 9.7 sec.					
	Rubble Jetties					
	Generating			No Flow		
	Original Plan 5	Lower Breakwater Plan 15	Lower Brw. & Jetties Plan 18	Original Plan 5	Lower Breakwater Plan 15	Lower Brw. & Jetties Plan 18
0.5C	11 1.4	10 1.3	17 2.1	8 1.0	14 1.8	11 1.4
0.5D	8 1.0	6 0.8	15 1.9	12 1.5	11 1.4	11 1.4
0.5E	18 2.3	12 1.5	15 1.9	8 1.0	4 0.5	13 1.6
AVERAGE	12 1.6	9 1.2	16 2.0	9 1.2	10 1.3	12 1.5
1.5C	28 3.5	10 1.3	18 2.3	24 3.0	14 1.8	17 2.1
1.5D	26 3.3	11 1.4	13 1.6	22 2.8	4 0.5	14 1.8
1.5E	29 3.7	15 1.9	17 2.1	37 4.7	15 1.9	21 2.6
AVERAGE	28 3.5	12 1.5	16 2.0	28 3.5	11 1.4	17 2.1

TABLE XIc

LOWER BREAKWATER AND JETTIES

Location	S.W. WAVE H_0 = 14.8 ft. H_G = 12.6 ft. T = 9.7 sec.				S.W. WAVE H_0 = 6.8 ft. H_G = 6.0 ft. T = 8.4 sec.			
	Rubble Jetties and Revetments				Vertical Jetties			
	Generating		No Flow		Generating		No Flow	
	Original Plan 12	Lower Breakwater Plan 14	Original Plan 12	Lower Breakwater Plan 14	Original Plan 1	Lower Breakwater Plan 16	Original Plan 1	Lower Breakwater Plan 16
0.5C	1.4 11	0.9 7	0.6 5	1.5 12	0.8 13	1.4 24	0.6 10	0.5 9
0.5D	1.1 9	0.9 7	0.6 5	1.5 12	0.8 13	1.4 24	0.8 13	0.5 9
0.5E	1.4 11	0.8 6	1.1 9	1.4 11	0.6 10	0.7 13	0.6 10	0.1 2
AVERAGE	1.3 10	0.9 7	0.8 6	1.5 12	0.7 12	1.2 20	0.7 11	0.4 7
1.5C	1.9 15	1.0 8	1.5 12	2.1 17	1.7 28	1.9 31	0.8 13	0.7 13
1.5D	1.4 11	1.1 9	0.9 7	0.4 3	2.0 32	1.4 24	2.3 37	1.1 18
1.5E	1.1 9	1.1 9	1.9 15	0.9 7	1.1 17	1.7 28	1.2 20	1.1 18
AVERAGE	1.5 12	1.1 9	1.4 11	1.1 9	1.6 26	1.7 28	1.4 23	1.0 16

TABLE XId

LOWER BREAKWATER AND JETTIES

Location	W. WAVE H_O = 11.3 ft. H_G = 10.5 ft. T = 8.2 sec.							
	Vertical Walled Jetties						Rubble Jetties	
	Generating		No Flow		Pumping		Generating	No Flow
	Original Plan 1	Lower Breakwater Plan 16	Original Plan 1	Lower Breakwater Plan 16	Original Plan 1	Lower Breakwater Plan 16	Lower Breakwater Plan 15	Lower Breakwater Plan 15
0.5C	2.7 26	1.8 17	1.3 12	1.6 15	1.5 14	0.6 6	1.5 14	1.5 14
0.5D	3.0 29	2.5 24	1.5 14	1.4 13	0.9 9	1.1 10	1.5 14	1.2 11
0.5E	3.4 32	1.9 18	1.2 11	2.1 20	0.9 9	1.3 12	1.6 15	0.7 7
AVERAGE	3.0 29	2.1 20	1.3 12	1.7 16	1.2 11	0.9 9	1.5 14	1.2 11
1.5C	2.5 24	2.0 19	2.1 20	1.5 14	1.7 16	1.3 12	1.9 18	1.4 13
1.5D	2.8 27	2.0 19	1.6 15	1.6 15	0.9 9	1.1 10	1.6 15	0.7 7
1.5E	3.0 29	2.1 20	0.8 8	1.2 11	0.9 9	1.1 10	1.7 16	1.3 12
AVERAGE	2.8 27	2.0 19	1.5 14	1.4 13	1.2 11	1.2 11	1.7 16	1.2 11

THREE 100 FOOT OPENINGS IN BREAKWATER

TABLE XII

This arrangement (Plans 25 and 26) was studied because it provided some beneficial effects on currents between the jetties and the breakwater. Two openings were placed opposite the ends of the jetties and one at the center line of the harbor. Table XIIa gives results for a W. wind and Table XIIb for waves approaching from the S.W. Table XIIc provides a means of comparing the wave heights with the most similar tests without openings in the breakwater. The comparisons were made with test results from Plans 15 and 16 which do not have the 90° extensions on the ends of the jetties. However, these extensions are believed to have little effect on wave heights in the harbor. The values in Table XIIc are the sum of the two average residual percentages for each condition. For example, the value for Vertical Jetties with 3 Openings during the generating phase is 39. This is obtained by adding the two average residual percentages 16 and 23 from the first column of Table XIIa. During the generating phase the results are inconclusive, but during no flow and pumping phases the wave heights near the structure are considerably greater with the three openings in the breakwater, particularly during southwesterly storms. For southwesterly storms the average increase in wave height for all three flow phases is approximately 70 per cent.

It was found that with three openings in the breakwater overtopping had an effect on the wave heights in the harbor. Raising the breakwater to prevent overtopping reduced the wave height at station 0.5D by 30 per cent.

TABLE XIIa

THREE 100 FOOT OPENINGS IN BREAKWATER

Location	W. WAVE H_0 = 11.3 ft. H_G = 10.5 ft. and H_0 = 6.8 ft. H_G = 6.0 ft. T = 8.4 sec.											
	Vertical Walled Jetties Plan 25						Rubble Jetties Plan 26					
	Generating		No Flow		Pumping		Generating		No Flow		Pumping	
	H_G=10.5	H_G=6.0	H_G=10.5	H_G=6.0	H_G=10.5	H_G=6.0	H_G=10.5	H_G=6.0	H_G=10.5	H_G=6.0	H_G=10.5	H_G=6.0
0.5C	21 2.2	13 0.8	18 1.9	11 0.7	16 1.7	6 0.4	12 1.3	11 0.6	12 1.3	7 0.4	5 0.5	5 0.3
0.5D	15 1.6	23 1.4	18 1.9	15 0.9	18 1.9	13 0.8	12 1.3	17 1.0	19 2.0	7 0.4	8 0.8	8 0.5
0.5E	13 1.4	24 1.4	24 2.5	16 1.0	27 2.8	12 0.7	13 1.4	18 1.1	9 0.9	7 0.4	5 0.5	8 0.5
AVERAGE	16 1.7	20 1.2	20 2.1	14 0.8	19 2.0	10 0.6	12 1.3	15 0.9	13 1.4	7 0.4	6 0.6	7 0.4
1.5C	16 1.7	12 0.7	32 3.4	8 0.5	16 1.7	6 0.4	10 1.1	13 0.8	17 1.8	11 0.6	11 1.2	8 0.5
1.5D	21 2.2	19 1.1	18 1.9	22 1.3	32 3.4	7 0.4	13 1.4	11 0.6	23 2.4	11 0.6	25 2.6	11 0.6
1.5E	31 3.3	35 2.1	17 1.8	7 0.6	23 2.4	7 0.4	23 2.4	28 1.7	9 0.9	11 0.6	12 1.3	15 0.9
AVERAGE	23 2.4	22 1.3	22 2.3	12 0.7	24 2.5	7 0.4	15 1.6	17 1.0	16 1.7	11 0.6	16 1.7	11 0.6

TABLE XIIb

THREE 100 FOOT OPENINGS IN BREAKWATER

Location	S.W. WAVE H_0 = 14.8 ft. H_G = 12.6 ft. T = 9.7 sec.					
	Vertical Walled Jetties Plan 25			Rubble Jetties Plan 26		
	Generating	No Flow	Pumping	Generating	No Flow	Pumping
0.5C	2.9 23	3.8 30	3.3 26	4.2 33	5.5 44	1.9 15
0.5D	3.8 30	5.5 44	4.8 38	4.4 35	5.2 41	1.9 15
0.5E	3.8 30	5.4 43	8.0 63	2.6 21	2.3 18	2.3 18
AVERAGE	3.5 28	4.9 39	5.3 42	3.8 30	4.3 34	2.0 16
1.5C	3.3 26	5.5 44	3.3 26	3.0 24	5.6 44	1.5 12
1.5D	3.9 31	4.7 37	5.2 41	4.1 33	3.5 28	4.7 37
1.5E	3.3 26	8.3 66	8.5 67	3.3 26	3.7 29	4.8 38
AVERAGE	3.5 28	6.2 49	5.7 45	3.5 28	4.3 34	3.7 29

TABLE XIIc

THREE 100 FOOT OPENINGS IN BREAKWATER

Residual Percentages of Wave Heights

Structure	Plan No.	Wind Direction	Table	Generating	No Flow	Pumping
Vertical Jetties 3 Openings	25	W	XII	39	42	43
Vertical Jetties No Openings	1	W	IV	56	26	22
Rubble Jetties 3 Openings	26	W	XII	27	29	
Rubble Jetties No Openings	15	W	XI	30	22	
Vertical Jetties 3 Openings	25	SW	XII	56	88	87
Vertical Jetties No Openings	1	SW	IV	62	38	28
Rubble Jetties 3 Openings	26	SW	XII	58	68	45
Rubble Jetties No Openings	5	SW	IV	40	37	30

TWO 100 FOOT OPENINGS IN THE BREAKWATER

Measurements showed that the center opening of the three openings described in the previous section could be closed without greatly affecting the currents. A number of wave height measurements were made with vertical jetties for southwesterly winds under no flow condition with the center opening closed. These tests showed that closing the center opening reduced the wave height at stations 0.5C and 0.5D by 65 per cent. It was shown in the previous section by means of the test results and analysis in Table XII that with three openings the wave heights near the powerhouse were increased by about 70 per cent over the conditions with no openings. Therefore, closing the center opening reduces the waves to nearly what they were with no openings.

It was also determined that when the center opening was closed preventing overtopping had a negligible effect.

ONE 200 FOOT OPENING IN BREAKWATER

This arrangement (Plans 19 to 24) was tested in the hope that currents would be reduced. However, the effects on currents were not substantial and because wave heights near the structure were large, this plan was abandoned. Wave height and current data are on file and could be readily made available.

SPECIAL TESTS SOUTH OF SOUTH JETTY
AND IN THE VICINITY OF THE SOUTH JETTY

These tests were made with the smaller wave height approaching from the S.W. (H_G = 6.0 ft.) at a number of locations near shore in the region south of the South Breakwater and at selected locations along the South Breakwater. The measurements were made with a point gage because the water was too shallow for the recording wave instruments. Results are shown in Fig. 7. Values are wave height remaining at the various locations, expressed in per cent of wave height at wave gage (H_G).

TESTS ON A BREAKWATER WITH ITS ENDS FLARED 200 FT.

TABLE XIII

These tests were made to determine the wave height near the powerhouse when the ends of the breakwater were located 200 feet farther offshore than the apex of the breakwater (Plans 51, 55, 56, 57, 58). Before moving the breakwater's ends 200 ft. farther offshore the breakwater was made approximately 200 feet longer than in the original design shown in Fig. 4. The breakwater used in these tests measured 825 feet from the harbor centerline to its north end and 1070 feet from the harbor centerline to its south end. Wave heights near the powerhouse were measured with the apex of the breakwater located in its original position, 2340 feet offshore, (Plans 51, 55, 56) during SW, W and NW waves. In most cases only the mode of operation (generation, etc.) which had caused the largest waves in previous tests was used. Wave heights near the powerhouse were also measured with the apex of the breakwater 100 feet farther offshore (Plans 57, 58) during NW waves. Tests during the SW and NW wave (Plans 55, 56 and 57, 58) indicate wave conditions near the powerhouse are the same for type B and D jetty ends (Fig. 6). The shape of this breakwater was expected to cause large standing waves on the windward side of the breakwater near its apex. When waves approached the breakwater from the West the largest standing waves were formed. The maximum standing wave measured on Plan 56 during W. waves was 42 feet from crest to trough. Next, the shape of the breakwater was altered by replacing the center 400 feet with a straight section (Plan 62). The maximum standing wave measured near the south apex on Plan 62 during the same W. wave was 28 feet high. Standing waves will form on the windward side of any breakwater. They will be smallest for breakwaters which dissipate wave energy and scatter the reflected wave in many directions. Rubble mound breakwaters are of this type. Larger standing waves, approximately twice the size of the incoming wave, will be formed on the windward side of a straight breakwater with vertical walls. A vertical wall breakwater with its ends flared such as the one tested here (Plans 51, 55, 56, 57, 58) amplifies standing waves, making them larger than those on the windward side of a straight breakwater. Such breakwaters are subject to more scour and greater stress than a straight breakwater with vertical sides. The areas of the breakwater and jetties which were overtopped and some unusual harbor conditions were recorded during these tests. The information for each test is included in the bottom row of Table XIII. The letters in this row refer to a location or note on Fig. 8. In addition, breaking waves and wave runup were observed in areas H and I shown on Fig. 8.

TABLE XIII

TESTS ON A BREAKWATER WITH ITS ENDS FLARED 200 FT.

Location	S.W. WAVE H_O = 14.8 ft. H_G = 12.6 ft. T = 9.7 sec.		W. WAVE H_O = 11.3 ft. H_G = 10.5 ft. T = 8.2 sec.	N.W. WAVE H_O = 11.0 ft. H_G = 9.3 ft. T = 8.2 sec.					
	Breakwater Center at Original Location		Breakwater Center at Original Location	Breakwater Center at Original Location	Breakwater Center at Original Location	Breakwater Center 100 Ft. Farther Offshore			Breakwater Center 100 Ft. Farther Offshore
	Jetty Ends B* Plan 55	Jetty Ends D Plan 56	Jetty Ends A Plan 51	Jetty Ends B Plan 55	Jetty Ends D Plan 56	Jetty Ends B Plan 57			Jetty Ends D Plan 58
	Generating	Generating	No Flow	Generating	Generating	Pumping	No Flow	Generating	Generating
0.5C	3.7 / 29	3.7 / 29	1.7 / 16	3.9 / 42	3.2 / 34	4.5 / 48	6.0 / 65	3.4 / 37	6.0 / 65
0.5D	3.1 / 25	4.3 / 34	0.8 / 8	3.2 / 34	3.4 / 37	3.1 / 33	7.3 / 78	4.9 / 53	3.4 / 37
0.5E	2.5 / 20	1.9 / 15	1.2 / 11	3.6 / 39	3.5 / 38	3.1 / 33	3.0 / 32	5.2 / 56	3.7 / 40
AVE.	3.1 / 25	3.3 / 26	1.3 / 12	3.5 / 38	3.3 / 36	3.5 / 38	5.4 / 58	4.5 / 49	4.4 / 47
1.5C	4.4 / 35	4.4 / 35	2.2 / 21	6.1 / 66	5.6 / 60	7.5 / 80	4.9 / 53	5.2 / 56	4.7 / 51
1.5D	6.5 / 51	6.0 / 48	2.5 / 24	3.6 / 39	3.9 / 42	4.7 / 51	4.7 / 51	4.9 / 53	4.0 / 43
1.5E	6.9 / 55	6.2 / 49	0.8 / 8	5.2 / 56	3.4 / 37	4.0 / 43	4.5 / 48	4.2 / 45	4.9 / 53
AVE.	5.9 / 47	5.5 / 44	1.9 / 18	5.0 / 54	4.3 / 46	5.4 / 58	4.7 / 51	4.7 / 51	4.5 / 49
AREAS OF OVER-TOPPING (Fig. 8)	A,B,E	A,B,E,	A	A,C			A,C,G	A,C,F	A,C,D

STRAIGHT BREAKWATER 300 FT. FARTHER OFFSHORE

TABLE XIV

The straight breakwater was moved 300 ft. farther offshore than in the original design (Fig. 4) and its length was increased approximately 200 feet. Thus the standard length breakwater used in these tests was located 2640 ft. offshore and measured 825 ft. from the centerline of the harbor to its north end and 1070 ft. from the centerline of the harbor to its south end (Plan 54). Wave tests were made on this breakwater because the currents measured between the end of each jetty and the breakwater were lower and more uniform in those plans where the breakwater end had been moved 300 ft. farther offshore than in the original plan (Fig. 4) Tests during the S.W. wave are shown in Table XIVa. Table XIVa shows wave heights measured near the powerhouse with the standard length breakwater (Plan 54) during the "generating" and "no flow" modes of operation. This Table also shows the effect of using different jetty constructions (Plans 65, 66, 69) on one side. The results should be reduced by 13 per cent and 17 per cent respectively to obtain values for the case where the zig-zag wall or rubble mound is used on both sides of the harbor. This correction was previously explained in the discussion on Table IV (page 11). If the reductions are made the use of zig-zag walls causes an average reduction in wave height somewhat higher than the 13% figure explained in the discussion of Table IV, and the use of rubble mound jetties causes an average reduction in wave height equal to the 34 per cent figure explained in the discussion of Table IV (p. 11). Tests on the standard length breakwater (Plan 54) during the W. wave are shown in Table XIVb. Results of tests made during a N.W. wave are also shown in Table XIVb. The standard length breakwater was not tested during the N.W. wave. However, the tests on Plan 57 which are shown in Table XIII can be used to give this information since the ends of the breakwater in Plan 57 are approximately in the same location as in Plan 54. The maximum standing wave on the windward side of the breakwater was measured to provide comparison with those presented in the discussion of Table XIII. The maximum standing wave measured on Plan 54 during W. waves was 22 ft. The areas of the breakwater and jetties which were overtopped were recorded during these tests. The information for each test is included in the bottom row of Table XIV. The letters in this row refer to a location of note on Fig. 9. In addition, breaking waves and wave runup were observed in areas H and I shown on Fig. 9.

TABLE XIVa

STRAIGHT BREAKWATER 300 FT. FARTHER OFFSHORE

Location	S.W. WAVE H_0 = 14.8 ft. H_G = 12.6 ft. T = 9.7 sec.						
	Standard Length Breakwater Plan 54		337 ft. Zig Zag Wall (one side) Plan 65	675 ft. Zig Zag Wall (one side) Plan 66	675 ft. Rubble Mound (one side) Plan 69	Breakwater Extended 193 Feet South Plan 67	Breakwater Extended 290 Feet South Plan 68
	Generating	No Flow	No Flow	No Flow	No Flow	No Flow	No Flow
0.5C	8.1 64	8.8 70	7.7 61	6.5 53	4.4 35	6.7 53	4.8 38
0.5D	4.4 35	5.9 47	4.4 35	6.4 51	4.8 38	4.7 37	3.7 29
0.5E	5.7 45	5.3 42	3.7 29	4.8 38	5.5 44	3.5 28	2.9 23
AVE.	6.0 48	6.7 53	5.3 42	5.9 47	4.9 39	4.9 39	3.8 30
1.5C	5.7 45	7.8 62			3.1 25	6.3 50	3.8 38
1.5D	4.8 38	4.8 38			4.8 38	1.9 15	3.0 24
1.5E	7.7 61	6.1 48			7.7 61	5.5 44	4.8 38
AVE.	6.0 48	6.2 49			5.2 41	4.6 36	4.2 33
AREAS OF OVER-TOPPING (Fig. 9)	A,B,G	A,B,E			A,B		A,B

TABLE XIVb

STRAIGHT BREAKWATER 300 FT. FARTHER OFFSHORE

Location	W. WAVE H_O = 11.3 ft. H_G = 10.5 ft. T = 8.2 sec.			N.W. WAVE H_O = 11.0 ft. H_G = 9.3 ft. T = 8.2 sec.	
	Standard Length Breakwater Plan 54			Breakwater Extended 193 Ft. North Plan 63	Breakwater Extended 290 Ft. North Plan 64
	Generating	Pumping	No Flow	No Flow	No Flow
0.5C	24 2.5	24 2.5	24 2.5	34 3.2	44 4.1
0.5D	26 2.7	24 2.5	20 2.1	36 3.3	39 3.6
0.5E	20 2.1	24 2.5	18 1.9	47 4.4	39 3.6
AVE.	23 2.4	24 2.5	21 2.2	39 3.6	41 3.8
1.5C	36 3.8	30 3.1	28 2.9	57 5.3	44 4.1
1.5D	28 2.9	24 2.5	24 2.5	72 6.7	23 2.1
1.5E	24 2.5	16 1.7	14 1.5	52 4.8	62 5.8
AVE.	29 3.0	23 2.4	22 2.3	60 5.6	43 4.0
AREAS OF OVER-TOPPING (Fig. 9)				A,C,F	

VELOCITY MEASUREMENTS

Currents were measured in the harbor area for 72 different conditions. Two groups of measurements were made. The first group consisted of surface velocity measurements made photographically. These measurements provided information on the general movements of water near the surface and showed the effect of waves and direction of littoral current on velocities in the harbor area. The second group consisted of measurements beneath the surface made by injecting a small amount of milk near the bottom and timing the movement of the colored mass. This group of tests was used to determine the effect of jetty length, configuration and construction and breakwater length, configuration and location on velocities. The objective of the velocity tests was to determine which combination of jetty and breakwater arrangements would give the best overall flow pattern for inlet and outlet velocities and an average inlet velocity of approximately 1.5 fps. The value of 1.5 fps was established by Ebasco Services, Inc. to protect small craft.

The conditions tested are summarized in Table XV for surface velocity measurements and in Table XVI for sub-surface velocities. The magnitudes and directions of surface velocities are shown on Plates 3 thru 19 in the Appendix. The magnitudes and directions of sub-surface velocities are shown on Plates 20 thru 77 in the Appendix. Because the Appendix is bound separately one example of the surface velocity results (Plate 3) and one example of sub-surface velocity results (Plate 20) are also included as part of the main body of this report. The harbor arrangements used in the tests and more detailed information on the various plans tested are provided in Fig. 5. Cross references showing the correspondence between the various Plan numbers shown in Fig. 5 and the Plate numbers is given in the first two columns of Tables XV and XVI.

Velocity profiles were prepared for sections extending from near the ends of the jetties to a point on the breakwater. These show the variation in velocity as the water passes in or out of the harbor on either side. The profiles are presented in Table XVII. The portions of the section where velocities exceeded 1.5 feet per second are cross-hatched. The sections along which the velocity profiles are drawn are not the same in every case. For this reason the velocity profiles should be used in conjunction with the data presented in the Plates. For each pair of profiles the Plate number showing the complete set of velocities as well as the corresponding Plan number (Fig. 5) are shown. The mode of operation and the discharge are also indicated in Table XVII.

Checks on the model elevations and orifice calibration made at the end of the testing program showed that all sub-surface velocities presented on the Plates and in the profiles of Table XVII should be reduced. These check tests are described later in the report. Correction factors for all tests are shown in the last column of Table XVI. The correction is not the same for all of the tests but it will be seen that for most tests the velocities must be reduced by 15 per cent. Although no direct checks were made it is believed that the surface velocities would also respond in the same manner and that these should also be reduced by 15 per cent.

TEST RESULTS - SURFACE VELOCITIES

Surface velocities were measured for plans 1 thru 5 and 8 (Fig. 5) for a variety of wave conditions during both the generating and pumping modes of operation at the powerhouse. Results are shown on Plates 3 through 19. The conditions are summarized in Table XV. The measuremements were made by taking a 3 second time exposure of particles floating on the water surface. Care was taken to remove surface films and establish steady state conditions before making the photo. Investigation indicated these velocities exist in a thin surface layer only.

To compute the prototype velocity from the data, the following procedure should be followed:

1. Measure the length of the path made by a particle being careful to differentiate between overlapping paths.
2. Since grid lines were one foot apart in the model the length of path can be computed as a proportional part of one foot.
3. Calculate the model velocity by dividing the distance travelled (length of path) by 3 seconds.
4. Convert to prototype velocity by multiplying by the square root of the scale ratio, 12.25.
5. Based on two correction factors discussed later in the report it appears that all surface velocities determined from Plates 3 thru 19 should be reduced by 15 per cent.

On each Plate is shown a test number which refers to the original data on file in the laboratory. For example the information on Plate 3 indicates that the original data are on the seventh negative of film roll number one.

TEST RESULTS - SUB-SURFACE VELOCITIES

These tests were made by injecting milk near the bottom through a small needle and noting the direction and length of the path of the milk during a selected time interval. Magnitudes and direction of velocities are presented on harbor plans in Plates 20 thru 77. The corresponding Plan numbers in Fig. 5 are shown in the second column of Table XVI. The velocities are in feet per second in the prototype but subject to the reduction shown in the last column of Table XVI. Prototype velocities were calculated from model velocities using the Froude relationship. Many plans were tested in this series to investigate the advantages of various harbor arrangements. Note that no measurements were made with a littoral current superimposed. Littoral currents can be expected to occur during periods of wave attack. While the magnitude of such currents in 30 feet of water is expected to be relatively small no data from the prototype were obtained. Any longshore current would cause the percentage of total discharge thru the downstream opening to increase and thereby increase velocities. The opposite would be true of the upstream opening.

There is considerable difficulty in making velocity measurements of this type in a small model. A good appreciation of this can be developed by observing the accuracy with which measurements can be reproduced exactly. Two tests with identical conditions (Plan 26) were made with a one day interval between tests. They are shown on Plates 27 and 28. Another pair of tests with identical conditions (Plan 30) were made with a four day interval between tests. These are shown on Plates 31 and 32. Considerable difference in the data observed at individual points under identical conditions is obvious. Most measurements made in these 57 tests were made within or very close to the harbor. In some tests velocities were measured at points as far as 600 feet from the harbor entrance. Measurements at such a location must be considered less reliable as they were made closer to the water distribution system. Two special series of tests were made to determine the effect of water surface elevation on velocity, and the changes in velocities caused by changing the nature of the water distribution system used to simulate lake currents. A detailed description of the tests is given in the following paragraphs.

Plate No. 20 thru Plate No. 23

These initial sub-surface velocity measurements were made in May 1969 on Plans 1 and 10 with both the generating and pumping modes of operation at maximum discharges. These data indicated three undesirable conditions; high average velocities across a line connecting each jetty with the breakwater, an uneven distribution of velocities along this line, and high local velocities near the offshore end of each jetty during pumping and near the ends of the breakwater during generation.

Plate No. 24 thru Plate No. 39

A second set of tests was made during June 1969 to investigate methods of reducing the velocities and improving the velocity distribution. An attempt was made to correct these problems by providing openings in the breakwater, shortening the jetties, changing the shape of the jetties and by using various combinations of these modifications. Measurements of the velocities and their distribution during 2/3 maximum pumping flow (44,000 cfs) were also made. A comparison of the velocities during maximum pumping flow (Plates 27, 28, 34, 35) and the velocities during 2/3 maximum pumping flow (Plates 30, 36, 37) indicated the average reduction in velocity was in proportion to the reduction in total flow. At the conclusion of these tests a check on the existing model contours was made by lowering the water surface in 6 foot increments and marking the "shoreline" on the model (Fig. 2).

Plate No. 40 thru Plate No. 50

This third set of measurements made during October 1969 investigated the effects of a breakwater with its end sections flared, shortening the jetties, changing the shape of each jetty's end, using vanes in the harbor to redistribute the flow, and using a breakwater with the center projecting into the harbor.

Plate No. 51 thru Plate No. 59

These measurements were made during December 1969 to determine the effect of moving the breakwater farther offshore, flaring the breakwater in an offshore direction from the centerline of the harbor and various combinations of both. Jetties with long parabolic ends (see Fig. 6) were used during this series. At this time the test shown on Plate No. 59 was made to provide a better understanding of the velocities outside the harbor. Plate No. 59 indicated that modifications in the water distribution system (A, Fig. 1) might have considerable effect on velocities measured outside the harbor. This possibility was examined by extending the water distribution system 8 feet farther into the lake on both sides of the tank and performing the tests shown on Plate No. 60 thru Plate No. 63.

Plate No. 60 thru Plate No. 63

These tests were made in December 1969 to determine the effect of alterations in the original water distribution system, system A in Fig. 1. The three distribution systems (A, B, C in Fig. 1) should be compared by the effect that the changes had on the measured velocities. The Plates which can be compared are: Plates 53, 54, and 59 made using distribution system A; Plates 60 and 61 made using distribution system B; Plates 62, 63, and 64 made using distribution system C.

The effect of changing from distribution system A to distribution system C on the velocities measured along a line connecting the end of the jetty and the end of the breakwater can be seen by comparing Plates 53 and 63, 54 and 62, 59 and 64. The effect of this change on the velocities measured approximately 600 feet to the north and to the south of the breakwater end can be seen by comparing Plates 59 and 64. Changing from distribution system A to distribution system C had the following effects.

1. Significant changes occurred in the direction and distribution of velocities recorded approximately 600 feet north of and 600 feet south of the breakwater ends.
2. A noticeable change occurred in the direction of velocities from about mid opening to the breakwater and the velocities were generally more evenly distributed.

Distribution system B was not used because it appeared to be a less realistic arrangement than system C. All tests made prior to Plate 60 were made with distribution system A. All tests made after Plate 63 were made with distribution system C.

Plate No. 64 thru Plate No. 69

This fifth set of measurements was made in December 1969 with distribution system C (Fig. 1). The velocities within 600 feet of the harbor were measured for Plans 51, 53, 46 (Fig. 5) during the maximum generating and maximum pumping modes of operation. Discussion of the curved ends of the jetties are shown in Fig. 6.

Plate No. 70 thru Plate No. 73

These measurements were part of the fifth set made in December 1969 with distribution system C. These were special tests made to further compare the advantages of the four end configurations shown in Fig. 6. The location of the breakwater during these tests is shown in Plan 46.

Plate No. 74 thru Plate No. 79

Information concerning the bottom contours in the harbor area obtained after the model was constructed indicated the correct depth of water midway between the north jetty and the breakwater was 30 feet. This condition would be obtained in the model by raising the SWL such that there was a 36 foot depth at pt. A (Fig. 2). These final measurements were made during February 1970 to determine the effect water depth has on the velocities measured. Velocities were measured with the depth of water used throughout most of the testing program (Plate 74) and with the water five feet deeper (Plate 75). Plate No. 78 is a comparison of the data obtained at the two depths. These measurements were repeated in a duplicate set of tests (Plates 76, 77) which are compared on Plate No. 79. This was done to have a larger number of observations for obtaining an average value. It may be seen from Plates 78 and 79 that there was a large random variation in the velocity changes at the various points with no apparent pattern related to the location of the points. The average unweighted reduction in velocity from the 36 comparisons was 8 per cent. If all of the velocities were weighted according to the depth at the various points this result would have been larger. Because the concern here is for the actual magnitudes of the individual velocities the 8 per cent reduction is deemed to be the appropriate one. There was no significant variation in the direction of the velocities due to the increase in depth.

Calibration of Orifice

At the conclusion of the tests the piping system was dismantled and the orifice used to set the model discharge was calibrated. This was not done at the beginning of the model program due to the fact that the determination of wave heights in the harbor was given the highest initial priority and because it was expected that the orifice coefficients determined elsewhere could be applied to give nearly correct discharges. As it became apparent that the velocity measurements were of great impor tance it became essential to check the orifice calibration. The calibration was carried out for both the pumping and generating modes without distrubing the section of pipe near the orifice measuring the time required to fill a 30 gallon container. Several test runs were made for both the pumping and generating modes and consistent results were obtained. The results showed that when operating at maximum capacity the discharge used in the model tests was 7 per cent higher than the design values. Therefore, all measured velocities should be reduced by seven per cent to correct for this higher discharge.

Velocity Correction Factors

In the previous paragraphs it has been shown that if the model had been operated with a water surface five feet higher and with the correct discharge the velocities would have been reduced by eight per cent and by seven per cent respectively. Therefore for most tests the measured velocities should be reduced by 15 per cent. Because the water surface was at a slightly different location for some tests this correction is not completely uniform. The correction factors to be applied to the various tests are shown in the last column in Table XVI. As previously stated, it can be assumed that this same 15 per cent reduction can also be applied to the surface velocities summarized in Table XV.

INDEX OF SURFACE VELOCITY MEASUREMENTS

TABLE XV

Plate No	Plan No	Jetty Construction				Wave			Littoral Current			Generating @ 76,000 cfs	Pumping @66,000 cfs	Velocity Reduction Factor α
		Vertical	Rubble	Zig Zag	Diverging Walls	None	N.W.	S.W.	None	S.	N.			
3	1	X				X			X			X		0.85
4	1	X					X			X		X		0.85
5	1	X						X			X	X		0.85
6	3		X			X			X			X		0.85
7	3		X				X			X		X		0.85
8	5		X					X			X	X		0.85
9	2			X		X			X			X		0.85
10	2			X			X			X		X		0.85
11	4			X				X			X	X		0.85
12	8	X			X			X			X	X		0.85
13	1	X				X			X				X	0.85
14	1	X						X			X		X	0.85
15	5		X			X			X				X	0.85
16	5		X					X			X		X	0.85
17	4			X		X			X				X	0.85
18	4			X				X			X		X	0.85
19	8	X			X			X			X		X	0.85

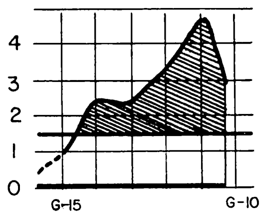
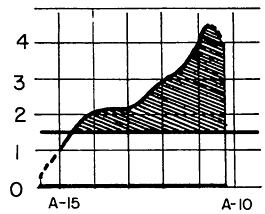
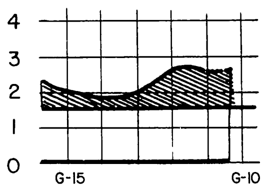
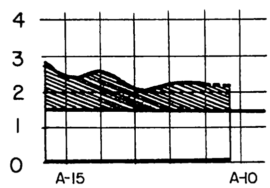
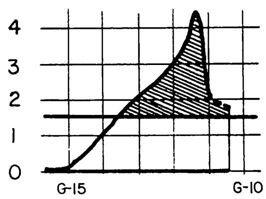
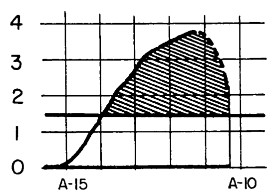
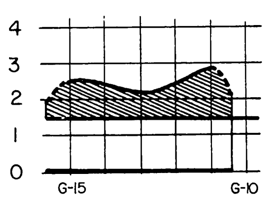
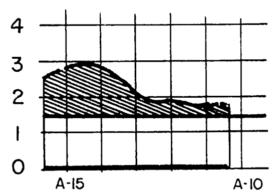
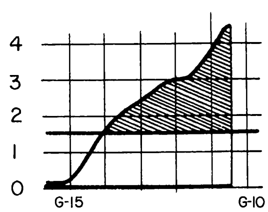
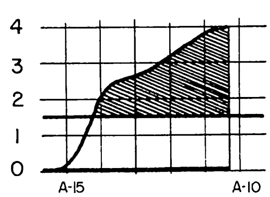
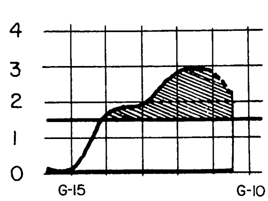
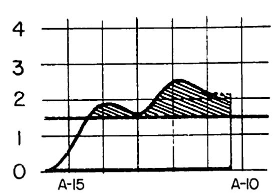
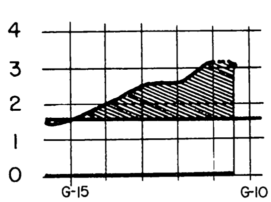
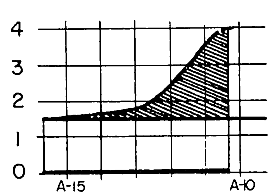
TABLE XVI

INDEX OF SUB-SURFACE VELOCITY MEASUREMENTS

Plate Number	Plan Number	JETTIES														Number of vanes within harbor	BREAKWATER																		Generating @ 76,000 cfs	Pumping @ 66,000 cfs	Depth of water measured at Pt. A (Fig. 2)			Velocity Reduction Factor α
		Construction			Length								End Extension				Construction		Openings		Distance offshore along ℄ of harbor				Flare				Length												
		Vertical Walls	Rubble Walls	Zig Zag Walls	projected perpendicular to shoreline		projected parallel to shoreline						163', 90° vertical	long parabolic	short parabolic		Vertical	Rubble	3-100'	2-100'	2340'	2440'	2540'	2640'	end sections		additional distance offshore to end of B.W.		North of ℄			South of ℄					30'	35'	37.7'	
					1585'	1350'	30'	110'	193'	300'	380'	410'													15°	30°	100'	200'	730'	825'	925'	980'	1070'	1170'						
20	1	X			X		X										X				X								X			X				X	X			0.85
21	1	X			X		X										X				X								X			X			X		X			0.85
22	10		X		X												X			X								X			X				X	X			0.85
23	10		X		X												X			X								X			X			X		X			0.85
24	27	X			X			X						X		X				X								X			X				X	X			0.85
25	28	X			X				X				X				X				X								X			X				X	X			0.85
26	29	X			X		X										X		X		X								X			X				X	X			0.85
27	26	X			X				X				X				X		X		X								X			X				X	X			0.85
28	26	X			X				X				X				X		X		X								X			X				X	X			0.85
29	26	X			X				X				X				X		X		X								X			X			X		X			0.85
30	26	X			X				X				X				X		X		X								X			X				2/3	X			0.92
31	30	X				X	X										X				X								X			X				X	X			0.85
32	30	X				X	X										X				X								X			X				X	X			0.85
33	31	X				X			X				X				X				X								X			X				X	X			0.85
34	32	X				X	X										X		X		X								X			X				X	X			0.85
35	33	X				X			X				X				X		X		X								X			X				X	X			0.85
36	32	X				X	X										X		X		X								X			X				2/3	X			0.92
37	33	X				X			X				X				X		X		X								X			X				2/3	X			0.92
38	34	X	X		X						X			X			X		X		X								X			X				X	X			0.85
39	35	X			X				X				X				X			X	X								X			X				X	X			0.85
40	35	X			X				X				X				X			X	X								X			X				X			X	0.97
41	36	X			X				X				X				X			X	X				X				X			X				X			X	0.97
42	37	X			X				X				X				X			X		X			X				X			X				X			X	0.97
43	38	X			X				X				X				X			X		X				X			X			X				X			X	0.97
44	39	X			X				X				X			2	X			X		X				X			X			X				X			X	0.97
45	40	X			X				X				X			6	X			X		X				X			X			X				X			X	0.97
46	41	X				X			X				X				X			X		X				X			X			X				X			X	0.97
47	42	X				X				X				X			X			X								X			X				X	X			0.85
48	43	X				X				X				X			X			X	X								X			X				X	X			0.85
49	44	X				X				X				X			X			X		X							X			X				X	X			0.85
50	44	X				X				X				X			X			X		X							X			X			X		X			0.85
51	50	X			X								X		X			X				X						X			X			X				X	X			0.85
52	52	X			X								X		X			X					X						X			X			X				X	X			0.85
53	53	X			X								X		X			X					X							X		X			X				X	X			0.85
54	45	X			X								X		X			X						X									X			X				X	X			0.85
55	46	X			X								X		X			X						X						X			X			X				X	X			0.85
56	47	X			X								X		X			X						X							X		X			X				X	X			0.85
57	48	X			X								X		X			X							X								X			X				X	X			0.85
58	49	X			X								X		X			X							X					X			X			X				X	X			0.85
59	51	X			X								X		X			X				X								X		X			X				X	X			0.85
60	45	X			X								X		X			X						X									X			X				X	X			0.85
61	45	X			X								X		X			X						X									X			X				X	X			0.85
62	45	X			X								X		X			X						X									X			X				X	X			0.85
63	53	X			X								X		X			X					X							X		X			X				X	X			0.85
64	51	X			X								X		X			X				X								X		X			X				X	X			0.85
65	51	X			X								X		X			X				X								X		X			X			X		X			0.85
66	53	X			X								X		X			X					X							X		X			X				X	X			0.85
67	53	X			X								X		X			X					X							X		X			X			X		X			0.85
68	46	X			X								X		X			X						X						X			X			X				X	X			0.85
69	46	X			X								X		X			X						X						X			X			X			X		X			0.85
70	46	X			X								X		X			X						X									X			X				X	X			0.85
71	59	X			X								X		X			X						X									X			X				X	X			0.85
72	60	X			X								X		X			X						X									X			X				X	X			0.85
73	61	X			X								X		X			X						X									X			X				X	X			0.85
74	70	X			X								X		X			X							X							X			X		X	X			0.85
75	70	X			X								X		X			X							X							X			X		X		X		0.93
76	70	X			X								X		X			X							X							X			X		X	X			0.85
77	70	X			X								X		X			X							X							X			X		X		X		0.93

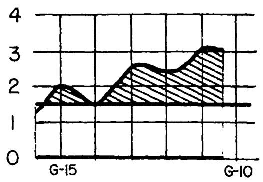
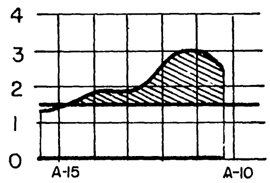
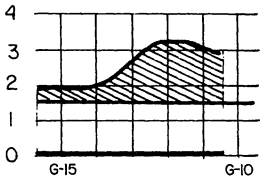
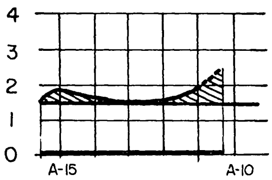
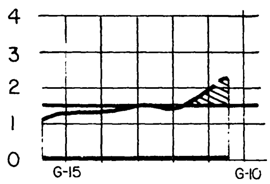
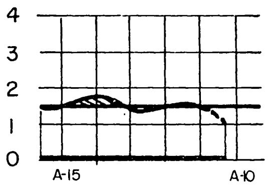
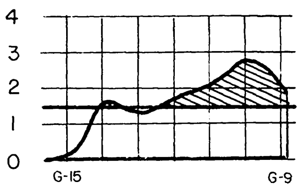
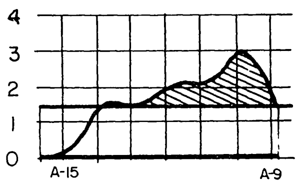
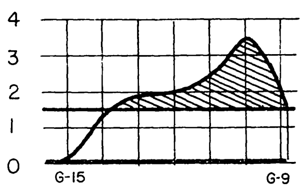
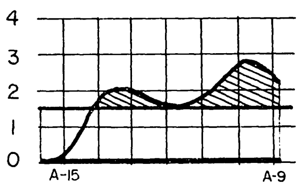
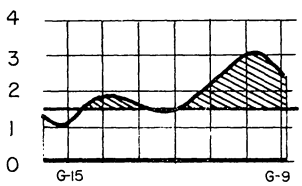
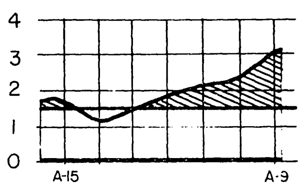
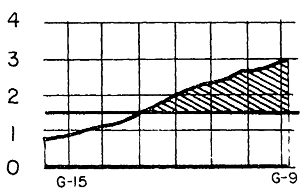
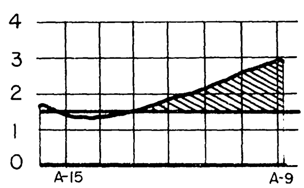
TABLE XVII

SUB-SURFACE VELOCITY PROFILES BETWEEN END OF JETTY AND BREAKWATER

PLATE No.	PLAN No.	NORTH PROFILE	SOUTH PROFILE	OPERATION
20	1			PUMPING @ 66,000 c.f.s.
21	1			GENERATING @ 76,000 c.f.s.
22	10			PUMPING @ 66,000 c.f.s.
23	10			GENERATING @ 76,000 c.f.s.
24	27			PUMPING @ 66,000 c.f.s.
25	28			PUMPING @ 66,000 c.f.s.
26	29			PUMPING @ 66,000 c.f.s.

NOTE: All velocity values in this table must be reduced by the factors given in Table XVI. Plates 20-60 were made using distribution system A; Plates 61-63 were made using distribution system B; Plates 64-77 were made using distribution system C; see discussion on pg. 55 and Fig. 1.

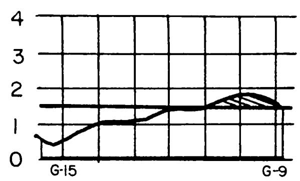
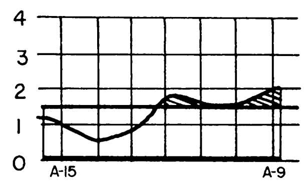
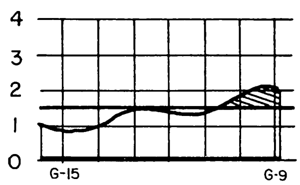
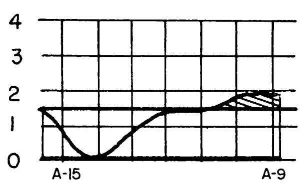
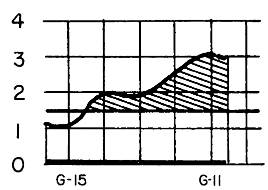
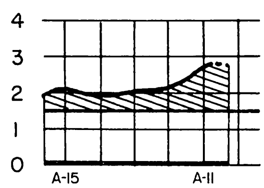
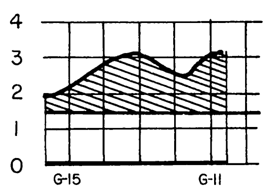
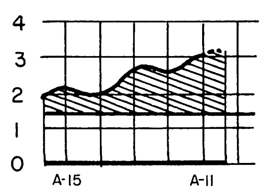
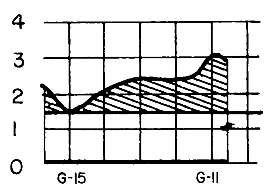
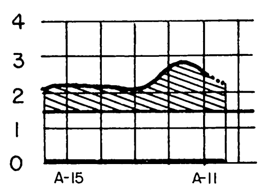
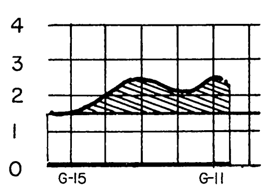
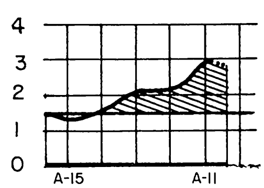
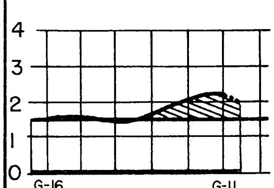
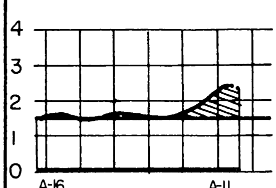
TABLE XVII
SUB-SURFACE
VELOCITY PROFILES BETWEEN END OF JETTY AND BREAKWATER

PLATE No.	PLAN No.	NORTH PROFILE	SOUTH PROFILE	OPERATION
27	26			PUMPING @ 66,000 c.f.s.
29	26			GENERATING @ 76,000 c.f.s.
30	26			PUMPING @ 44,000 c.f.s.
31	30			PUMPING @ 66,000 c.f.s.
33	31			PUMPING @ 66,000 c.f.s.
34	32			PUMPING @ 66,000 c.f.s.
35	33			PUMPING @ 66,000 c.f.s.

NOTE: All velocity values in this table must be reduced by the factors given in Table XVI. Plates 20-60 were made using distribution system A; Plates 61-63 were made using distribution system B; Plates 64-77 were made using distribution system C; see discussion on pg. 55 and Fig. 1.

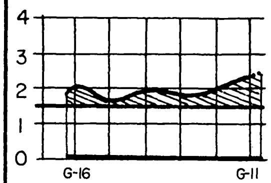
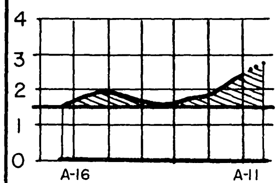
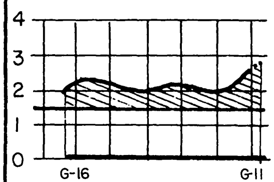
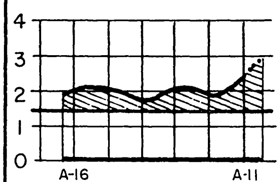
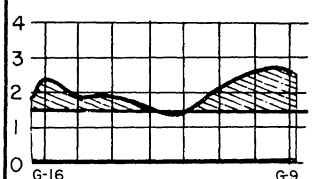
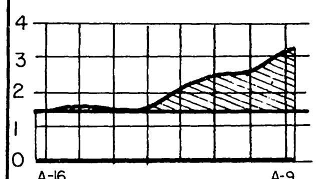
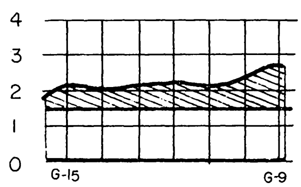
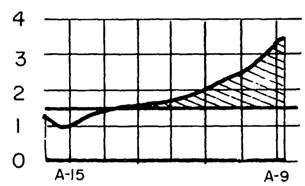
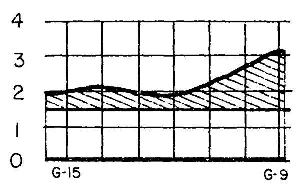
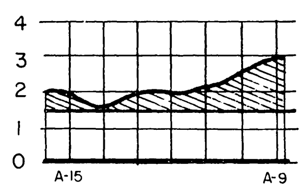
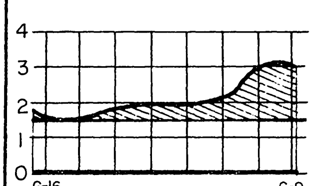
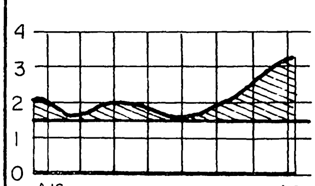
TABLE XVII

SUB-SURFACE
VELOCITY PROFILES BETWEEN END OF JETTY AND BREAKWATER

PLATE No.	PLAN No.	NORTH PROFILE	SOUTH PROFILE	OPERATION
36	32			PUMPING @ 44,000 c.f.s.
37	33			PUMPING @ 44,000 c.f.s.
38	34			PUMPING @ 66,000 c.f.s.
39	35			PUMPING @ 66,000 c.f.s.
40	35			PUMPING @ 66,000 c.f.s.
41	36			PUMPING @ 66,000 c.f.s.
42	37			PUMPING @ 66,000 c.f.s.

NOTE: All velocity values in this table must be reduced by the factors given in Table XVI. Plates 20-60 were made using distribution system A; Plates 61-63 were made using distribution system B; Plates 64-77 were made using distribution system C; see discussion on pg. 55 and Fig. 1.

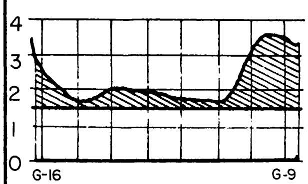
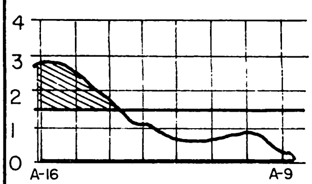
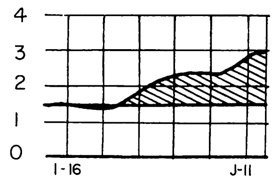
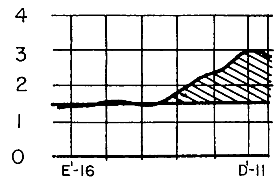
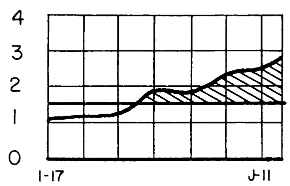
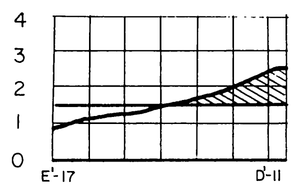
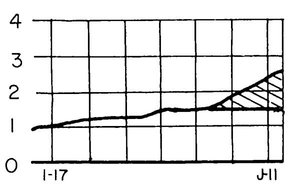
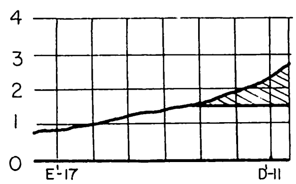
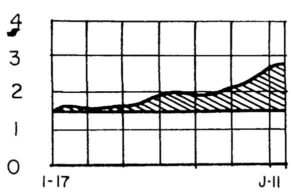
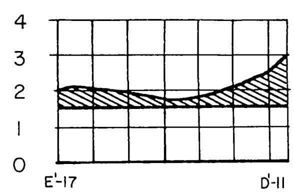
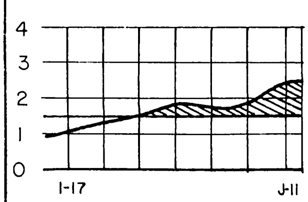
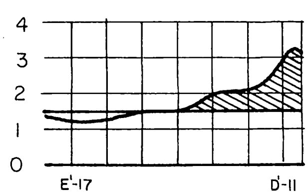
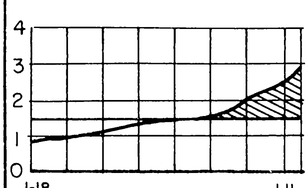
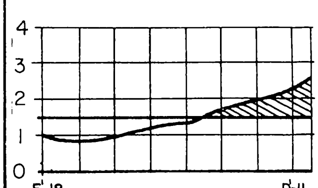
TABLE XVII
SUB-SURFACE
VELOCITY PROFILES BETWEEN END OF JETTY AND BREAKWATER

PLATE No.	PLAN No.	NORTH PROFILE	SOUTH PROFILE	OPERATION
43	38			PUMPING @ 66,000 c.f.s.
44	39			PUMPING @ 66,000 c.f.s.
45	40	NOT SUITABLE FOR THIS TYPE OF ANALYSIS		PUMPING @ 66,000 c.f.s.
46	41			PUMPING @ 66,000 c.f.s.
47	42			PUMPING @ 66,000 c.f.s.
48	43			PUMPING @ 66,000 c.f.s.
49	44			PUMPING @ 66,000 c.f.s.

NOTE: All velocity values in this table must be reduced by the factors given in Table XVI. Plates 20-60 were made using distribution system A; Plates 61-63 were made using distribution system B; Plates 64-77 were made using distribution system C; see discussion on pg. 55 and Fig. 1.

TABLE XVII

SUB-SURFACE VELOCITY PROFILES BETWEEN END OF JETTY AND BREAKWATER

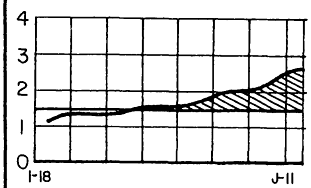
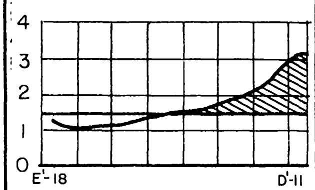
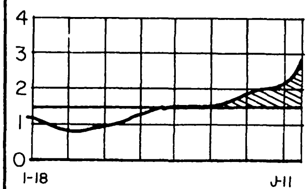
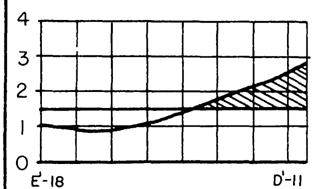
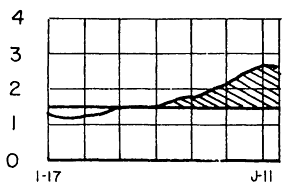
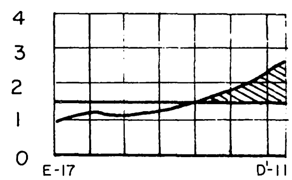
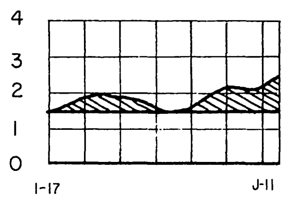
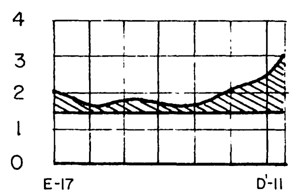
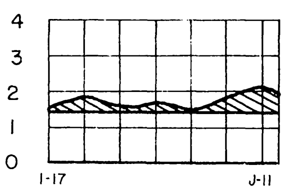
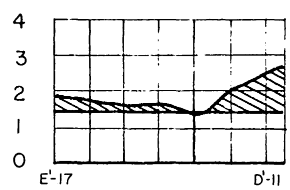
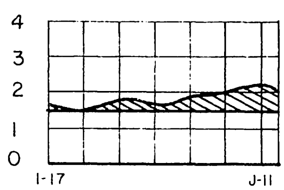
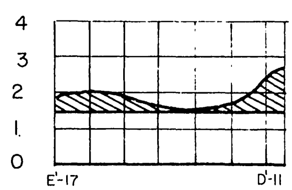
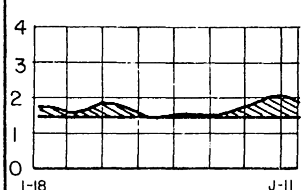
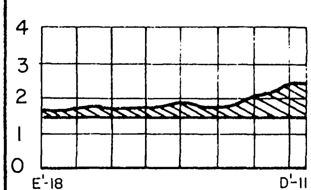
PLATE No.	PLAN No.	NORTH PROFILE	SOUTH PROFILE	OPERATION
50	44			GENERATING @ 76,000 c.f.s.
51	50			PUMPING @ 66,000 c.f.s.
52	52			PUMPING @ 66,000 c.f.s.
53	53			PUMPING @ 66,000 c.f.s.
54	45			PUMPING @ 66,000 c.f.s.
55	46			PUMPING @ 66,000 c.f.s.
56	47			PUMPING @ 66,000 c.f.s.

NOTE: All velocity values in this table must be reduced by the factors given in Table XVI. Plates 20-60 were made using distribution system A; Plates 61-63 were made using distribution system B; Plates 64-77 were made using distribution system C; see discussion on pg. 55 and Fig. 1.

TABLE XVII

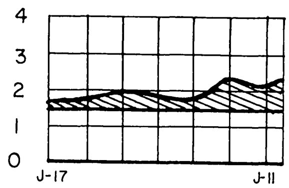
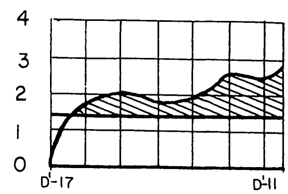
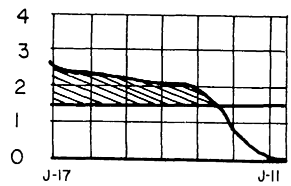
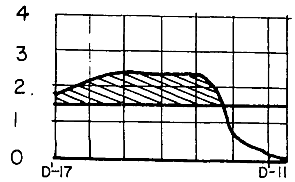
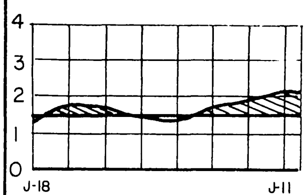
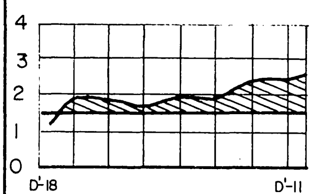
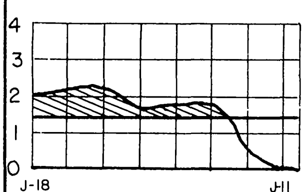
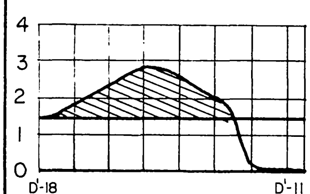
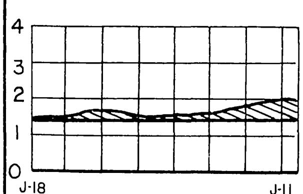
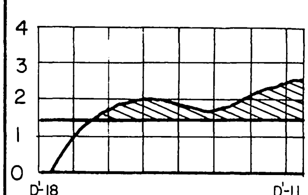
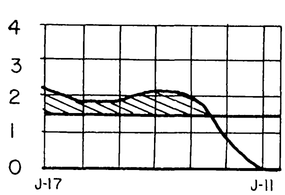
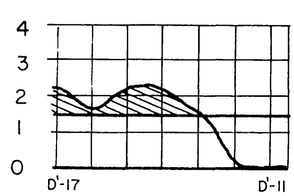
SUB-SURFACE

VELOCITY PROFILES BETWEEN END OF JETTY AND BREAKWATER

PLATE No.	PLAN No.	NORTH PROFILE	SOUTH PROFILE	OPERATION
57	48			PUMPING @ 66,000 c.f.s.
58	49			PUMPING @ 66,000 c.f.s.
59	51			PUMPING @ 66,000 c.f.s.
60	45			PUMPING @ 66,000 c.f.s.
61	45			PUMPING @ 66,000 c.f.s.
62	45			PUMPING @ 66,000 c.f.s.
63	53			PUMPING @ 66,000 c.f.s.

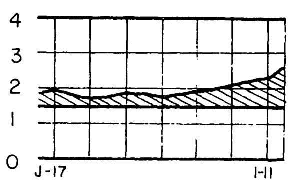
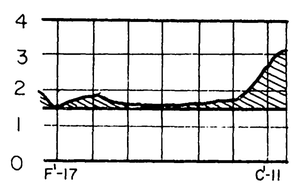
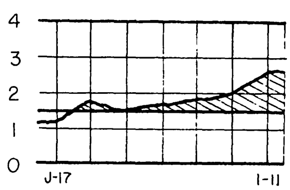
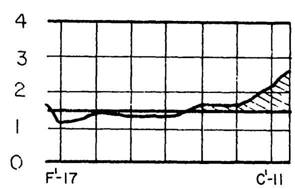
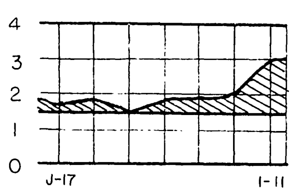
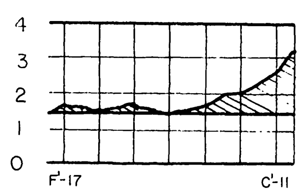
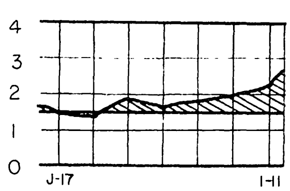
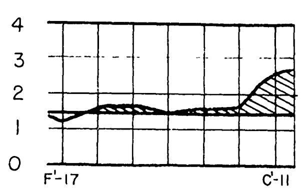
NOTE: All velocity values in this table must be reduced by the factors given in Table XVI. Plates 20-60 were made using distribution system A; Plates 61-63 were made using distribution system B; Plates 64-77 were made using distribution system C; see discussion on pg. 55 and Fig. 1.

TABLE XVII
SUB-SURFACE
VELOCITY PROFILES BETWEEN END OF JETTY AND BREAKWATER

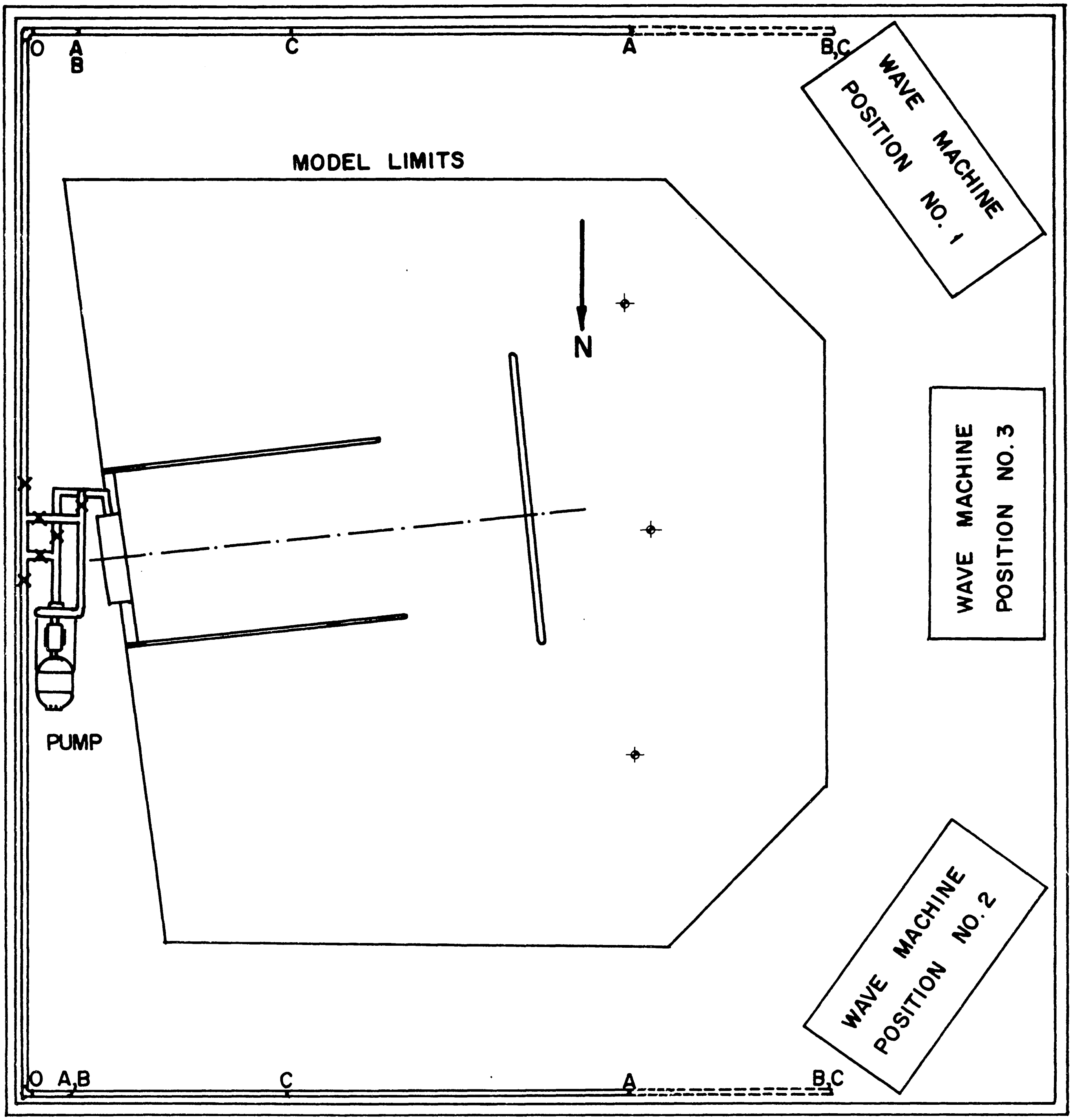
PLATE No.	PLAN No.	NORTH PROFILE	SOUTH PROFILE	OPERATION
64	51			PUMPING @ 66,000 c.f.s.
65	51			GENERATING @ 76,000 c.f.s.
66	53			PUMPING @ 66,000 c.f.s.
67	53			GENERATING @ 76,000 c.f.s.
68	46			PUMPING @ 66,000 c.f.s.
69	46			GENERATING @ 76,000 c.f.s.
70		NOT SUITABLE FOR THIS TYPE OF ANALYSIS		PUMPING @ 66,000 c.f.s.

NOTE: All velocity values in this table must be reduced by the factors given in Table XVI. Plates 20-60 were made using distribution system A; Plates 61-63 were made using distribution system B; Plates 64-77 were made using distribution system C; see discussion on pg. 55 and Fig. 1.

TABLE XVII
SUB-SURFACE
VELOCITY PROFILES BETWEEN END OF JETTY AND BREAKWATER

PLATE No.	PLAN No.	NORTH PROFILE	SOUTH PROFILE	OPERATION
71		NOT SUITABLE FOR THIS TYPE OF ANALYSIS		PUMPING @ 66,000 c.f.s.
72		NOT SUITABLE FOR THIS TYPE OF ANALYSIS		PUMPING @ 66,000 c.f.s.
73		NOT SUITABLE FOR THIS TYPE OF ANALYSIS		PUMPING @ 66,000 c.f.s.
74	70			PUMPING @ 66,000 c.f.s.
75	70			PUMPING @ 66,000 c.f.s.
76	70			PUMPING @ 66,000 c.f.s.
77	70			PUMPING @ 66,000 c.f.s.

NOTE: All velocity values in this table must be reduced by the factors given in Table XVI. Plates 20-60 were made using distribution system A; Plates 61-63 were made using distribution system B; Plates 64-77 were made using distribution system C; see discussion on pg. 55 and Fig. 1.



⌖ POSITION OF WAVE GAGES

CONDITION	DISTANCE TO DIFFUSER	LENGTH OF DIFFUSER
A	OA = 2 ft.	AA = 22 ft.
B	OB = 2 ft.	BB = 30 ft.
C	OC = 10 ft.	CC = 22 ft.

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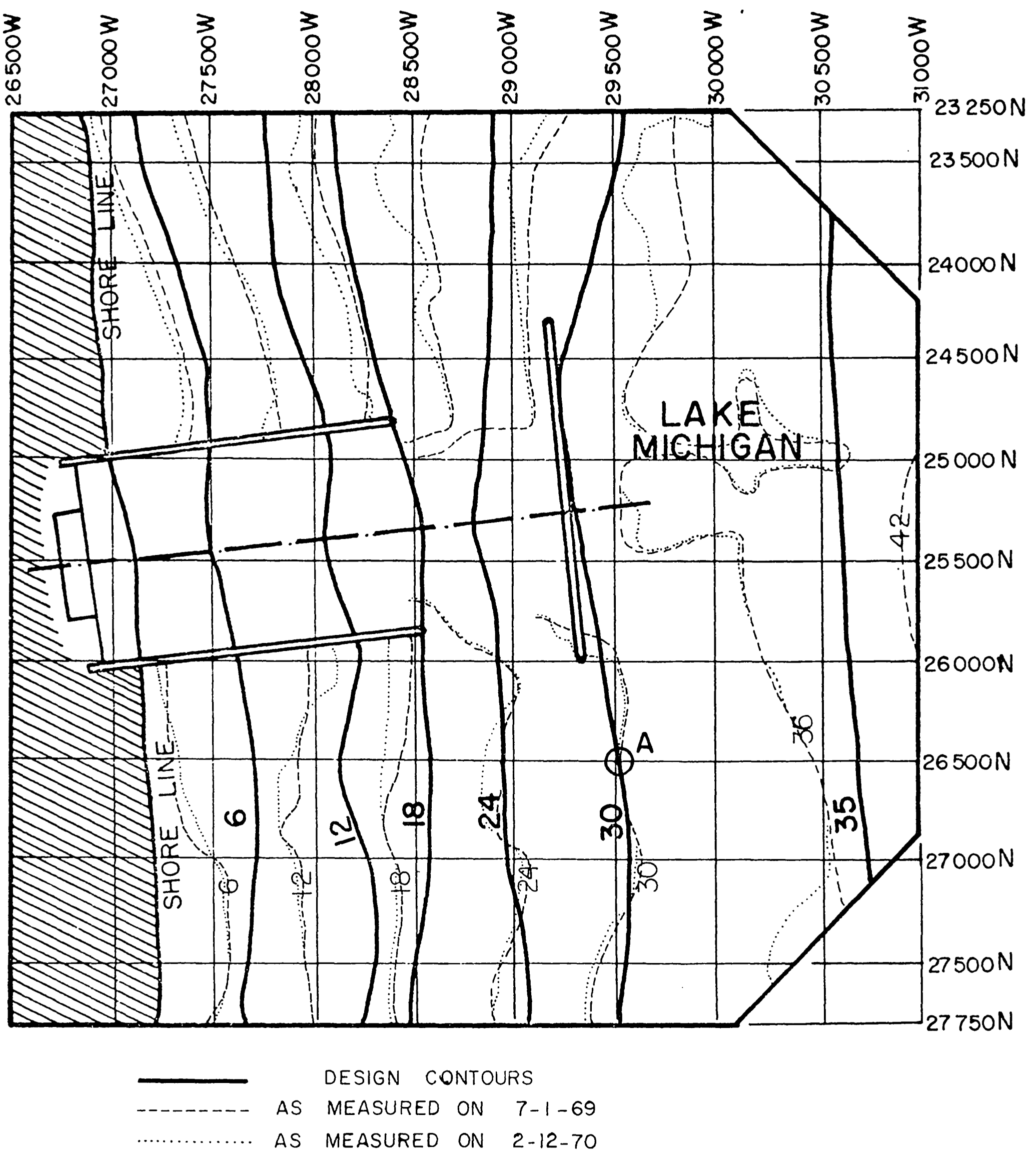
LUDINGTON PUMPED STORAGE PROJECT
LAKE FRONT MODEL
FIGURE No. 1

SCALE: 1" = 6' NOV. 1969



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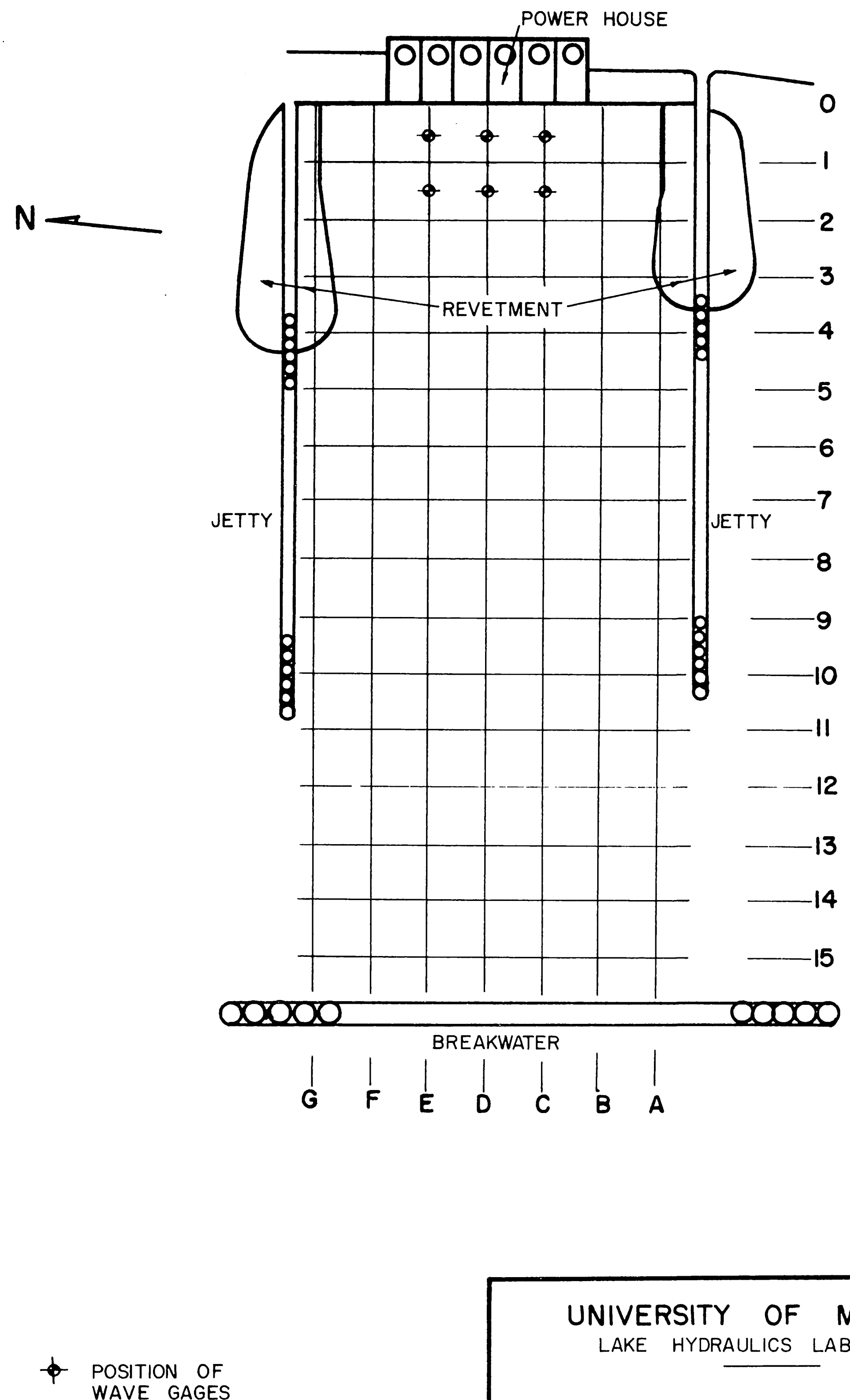
LUDINGTON PUMPED STORAGE PROJECT

LAKE FRONT MODEL

FIGURE No. 2

SCALE: 1" = 6'

NOV. 1969



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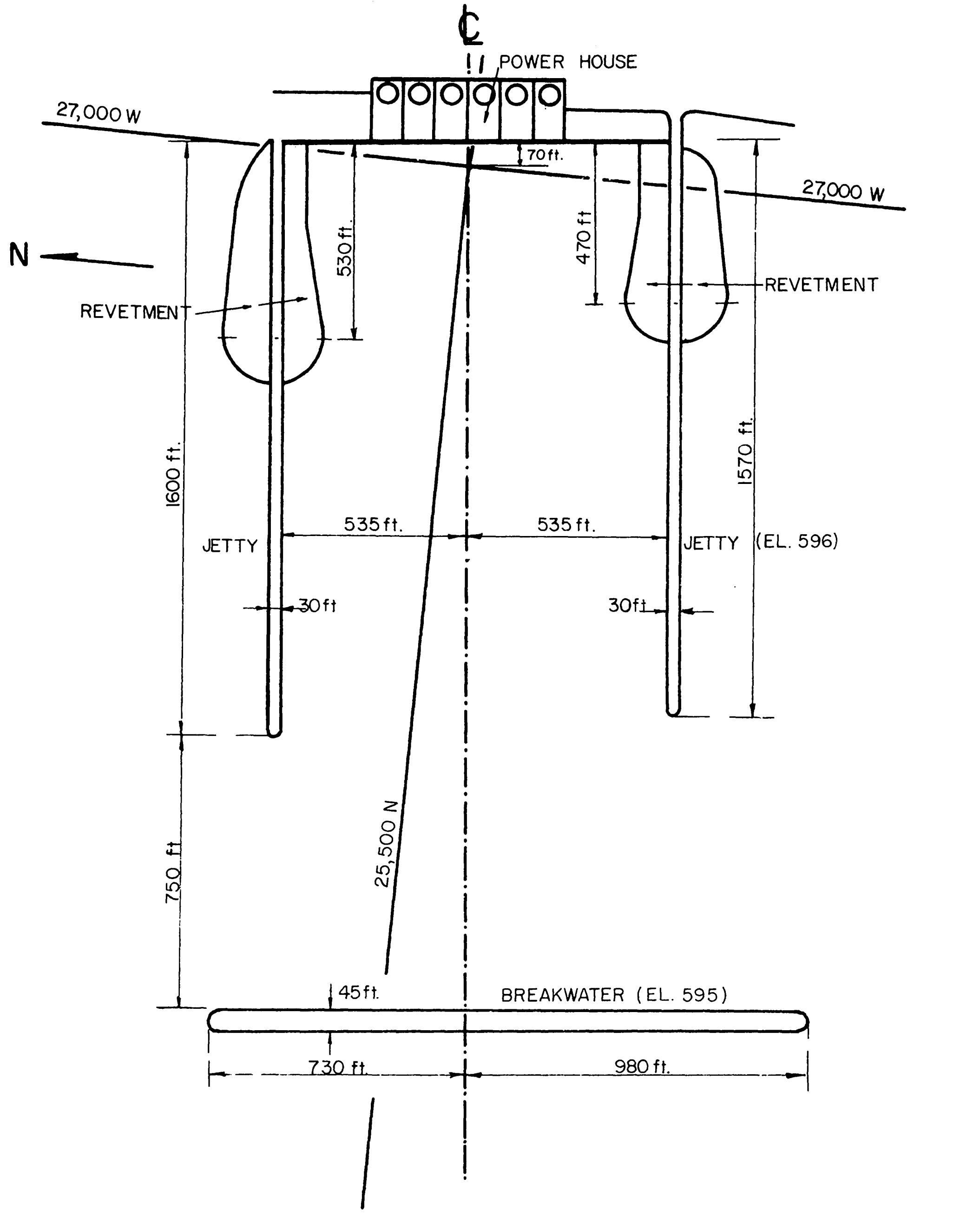
LUDINGTON PUMPED STORAGE PROJECT

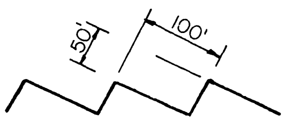
LAKE FRONT MODEL

FIGURE No. 3

SCALE: Grid system is in 150' increments NOV. 1969



ALL DIMENSIONS SCALED FROM "LAKE FRONT PLAN & SECTIONS" EBASCO EXHIBIT "L" SHEET 5 OF 16 DATED FEB. 2nd, 1968.



Zig-zag wall

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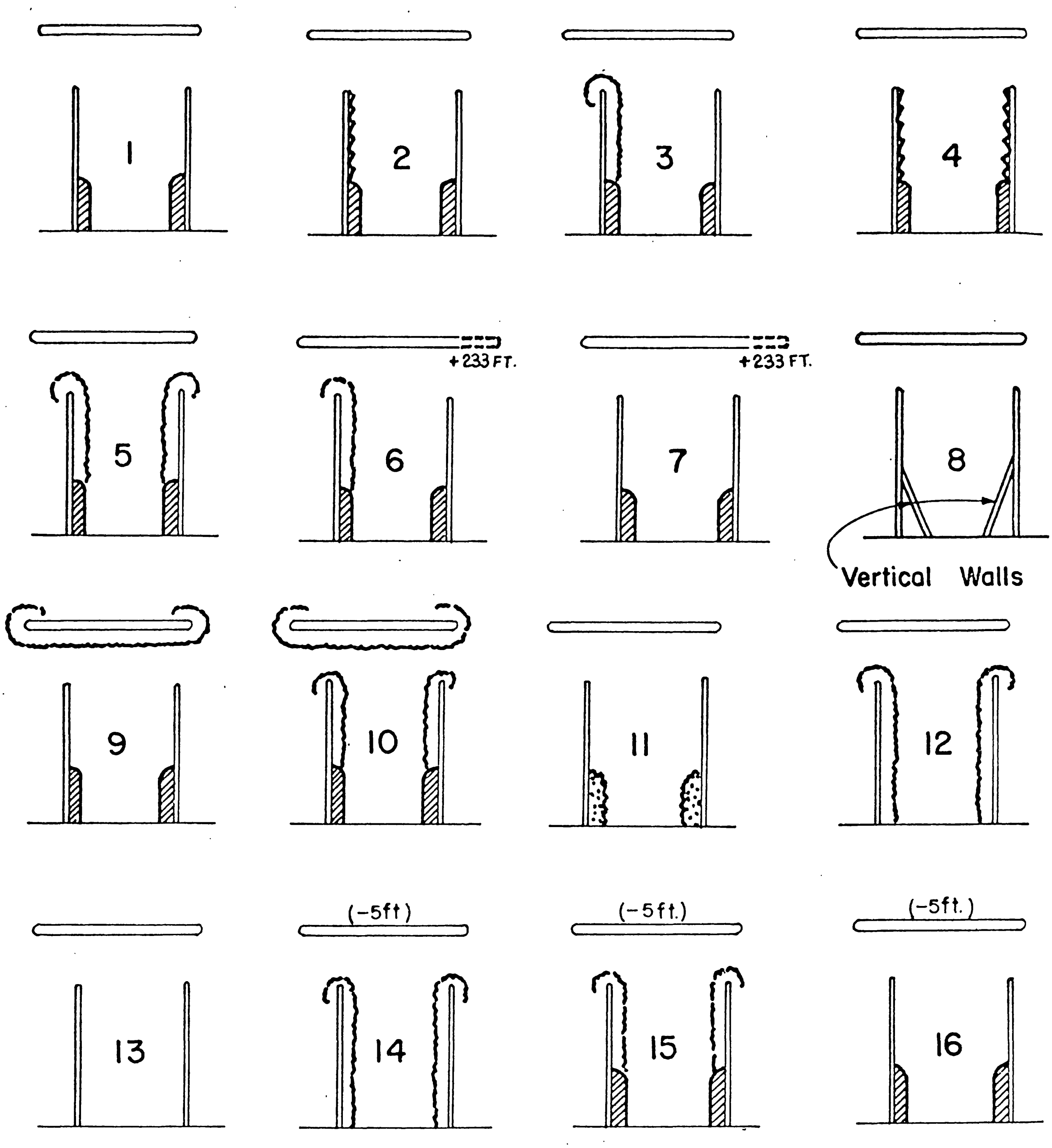
LUDINGTON PUMPED STORAGE PROJECT
LAKE FRONT MODEL
FIGURE No. 4

SCALE: as shown NOV. 1969



KEY

VERTICAL WALLS
RUBBLE MOUND
ZIG-ZAG WALLS
GROUTED REVETMENT
RUBBLE REVETMENT
NUMBER ( ) INDICATES CHANGES IN ELEVATION FROM ORIGINAL PLAN EL.595

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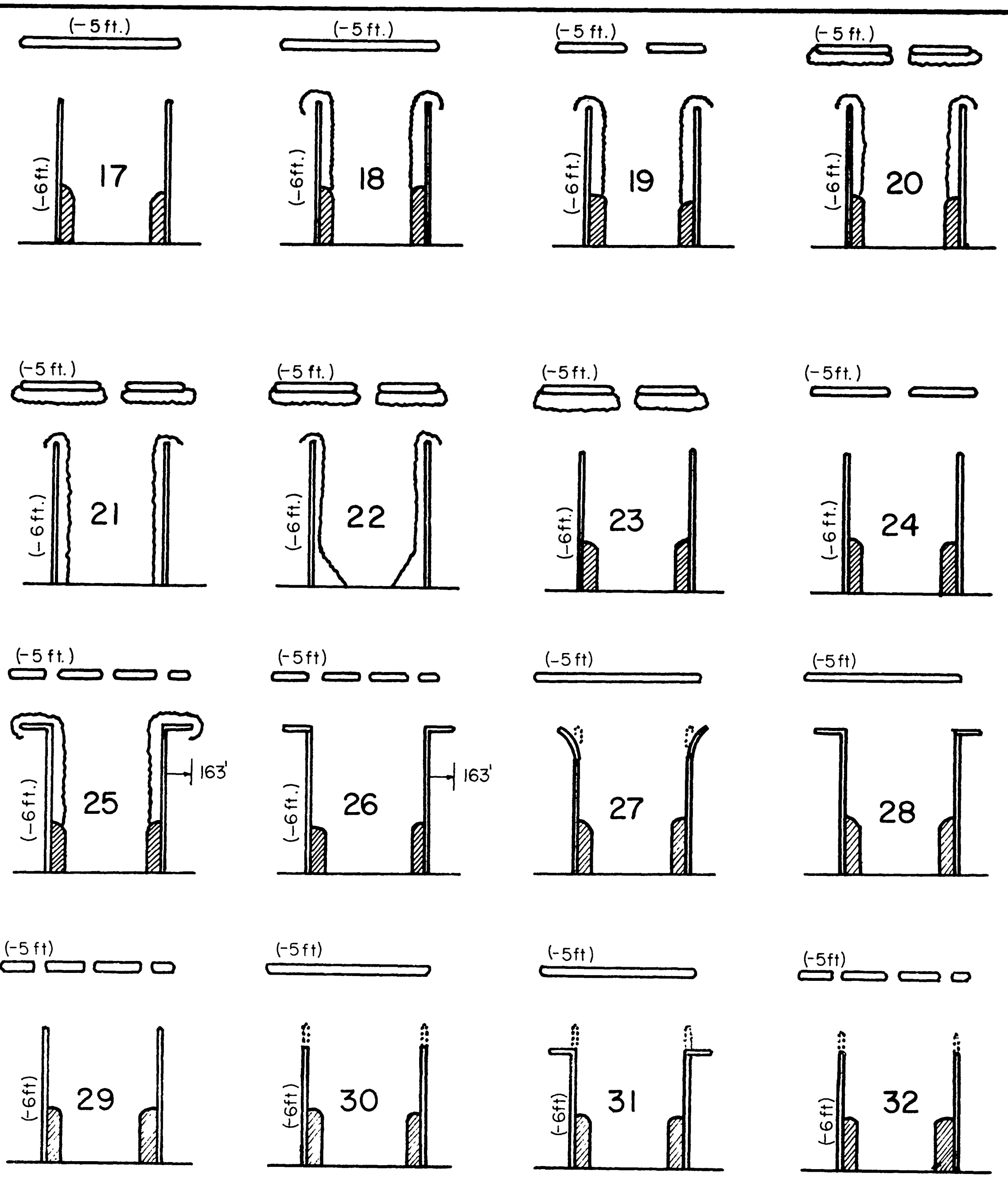
LUDINGTON PUMPED STORAGE PROJECT
LAKE FRONT MODEL
FIGURE No. 5

NOV. 1969



KEY

VERTICAL WALLS
RUBBLE MOUND
ZIG-ZAG WALLS
GROUTED REVETMENT
RUBBLE REVETMENT
NUMBER( ) INDICATES CHANGES IN ELEVATION FROM ORIGINAL PLAN EL.595

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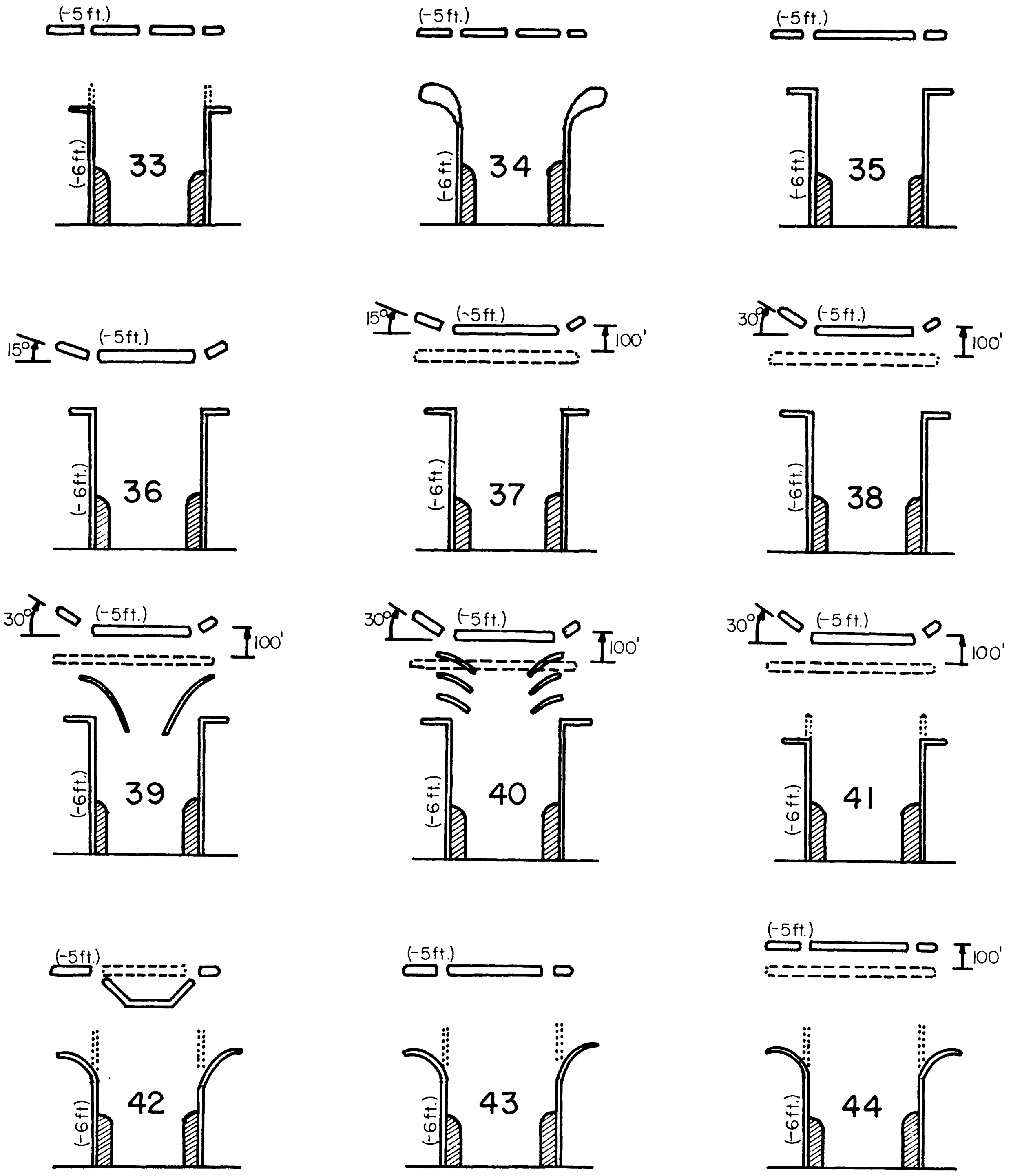
LAKE HYDRAULICS LABORATORY

LUDINGTON PUMPED STORAGE PROJECT
LAKE FRONT MODEL
FIGURE No. 5

NOV. 1969



KEY

VERTICAL WALLS
RUBBLE MOUND
ZIG-ZAG WALLS
GROUTED REVETMENT
RUBBLE REVETMENT
NUMBER ( ) INDICATES CHANGES IN ELEVATION FROM ORIGINAL PLAN EL.595

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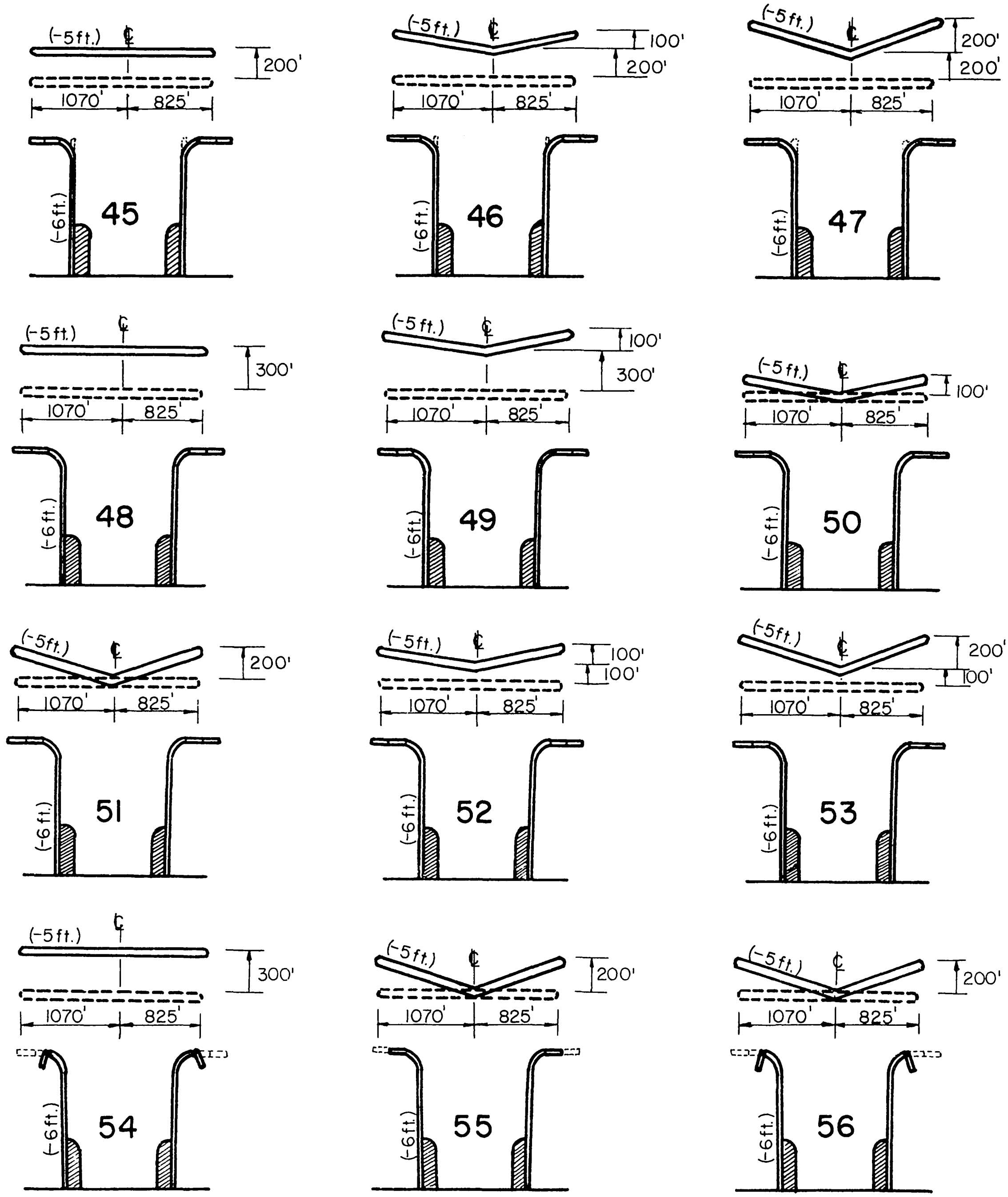
LAKE HYDRAULICS LABORATORY

LUDINGTON PUMPED STORAGE PROJECT
LAKE FRONT MODEL
FIGURE No. 5

NOV. 1969

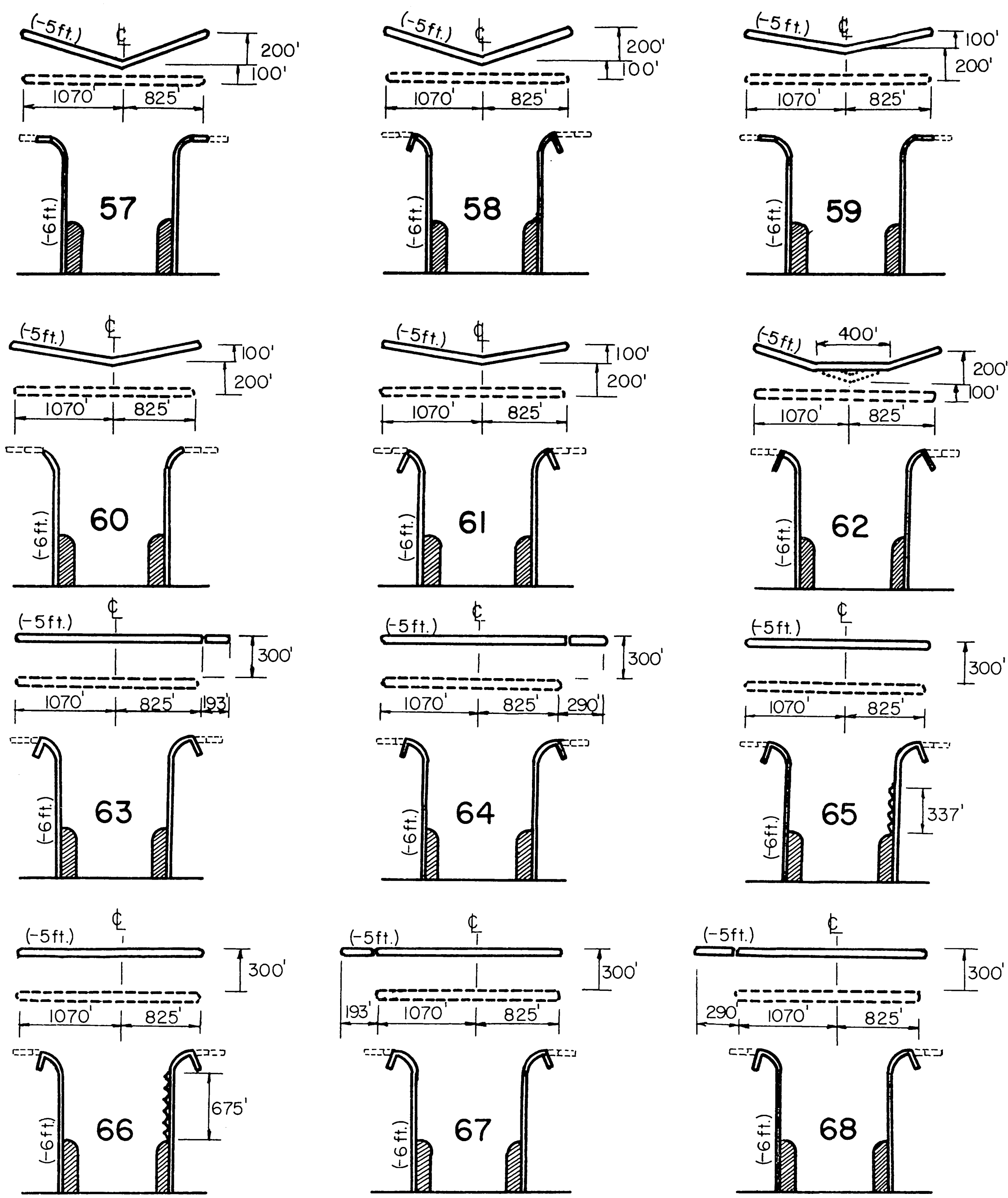


KEY

VERTICAL WALLS
RUBBLE MOUND
ZIG-ZAG WALLS
GROUTED REVETMENT
RUBBLE REVETMENT
NUMBER ( ) INDICATES CHANGES IN ELEVATION FROM ORIGINAL PLAN EL.595

UNIVERSITY OF MICHIGAN
LAKE HYDRAULICS LABORATORY

LUDINGTON PUMPED STORAGE PROJECT
LAKE FRONT MODEL
FIGURE No. 5



KEY

VERTICAL WALLS

RUBBLE MOUND

ZIG-ZAG WALLS

GROUTED REVETMENT

RUBBLE REVETMENT

NUMBER ( ) INDICATES CHANGES IN ELEVATION FROM ORIGINAL PLAN EL. 595

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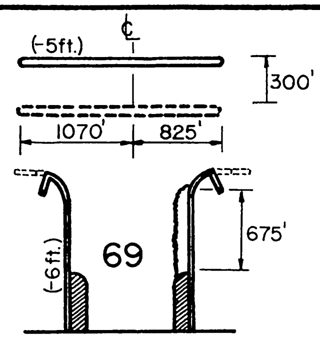
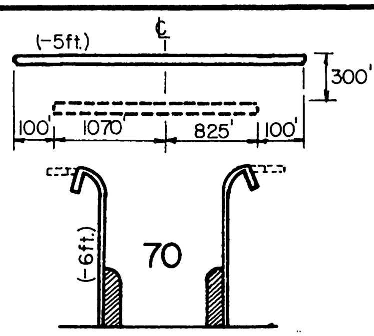
LAKE HYDRAULICS LABORATORY

LUDINGTON PUMPED STORAGE PROJECT

LAKE FRONT MODEL

FIGURE No. 5





KEY

VERTICAL WALLS

RUBBLE MOUND

ZIG-ZAG WALLS

GROUTED REVETMENT

RUBBLE REVETMENT

NUMBER ( ) INDICATES CHANGES IN ELEVATION FROM ORIGINAL PLAN EL.595

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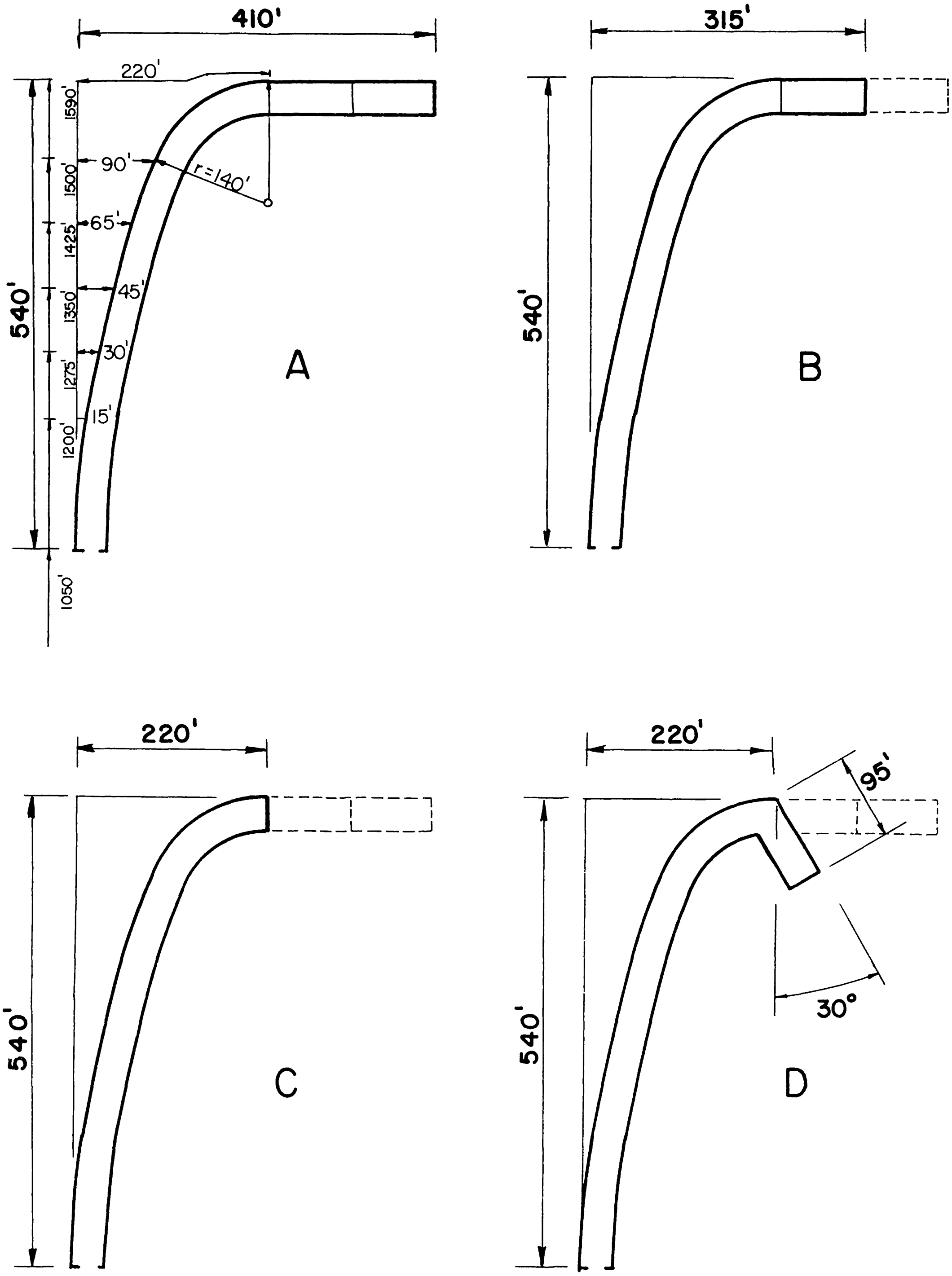
LUDINGTON PUMPED STORAGE PROJECT

LAKE FRONT MODEL

FIGURE No. 5



DETAILS OF JETTY ENDS USED
IN PLANS 45 THRU 70

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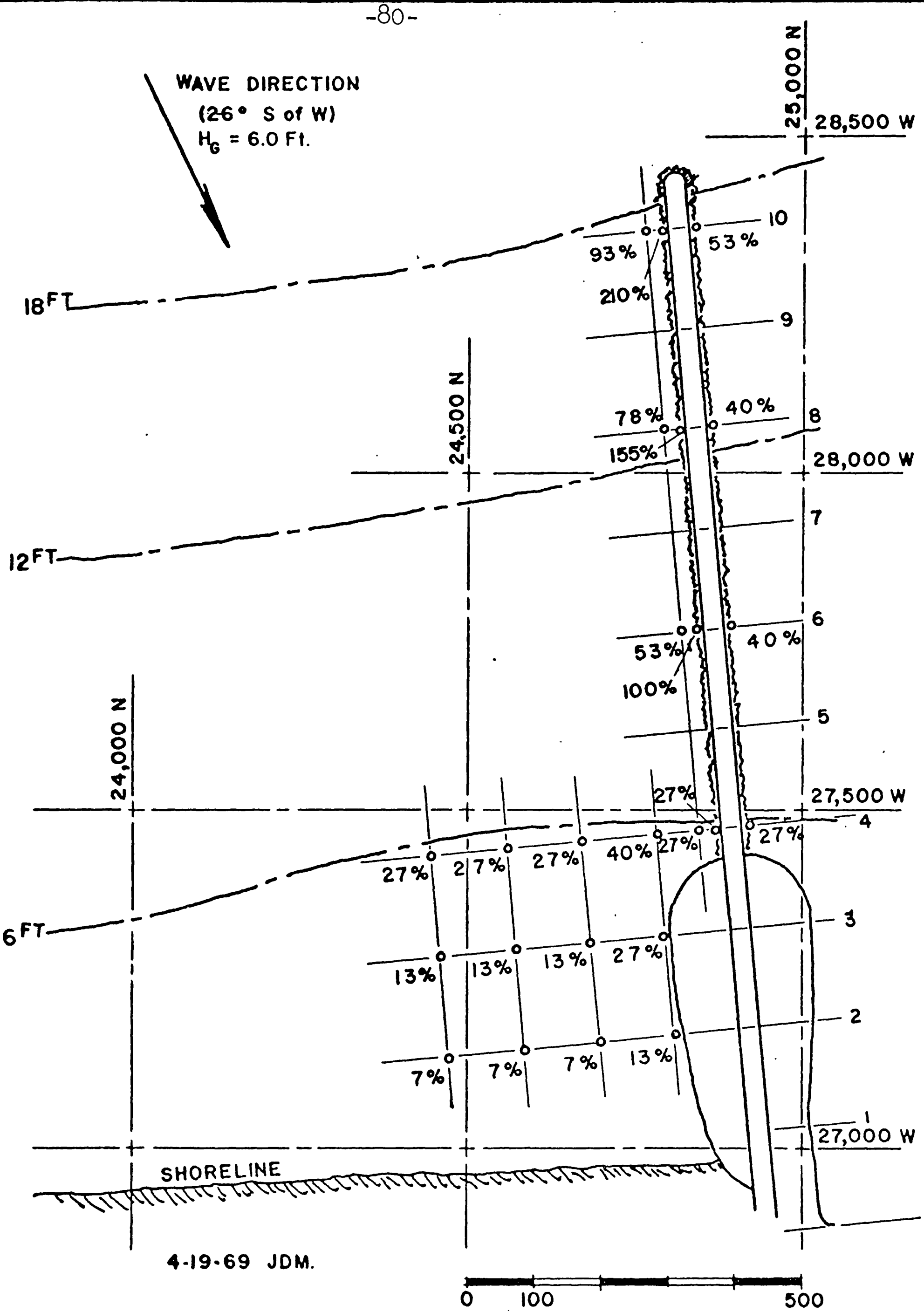
FIGURE No. 6



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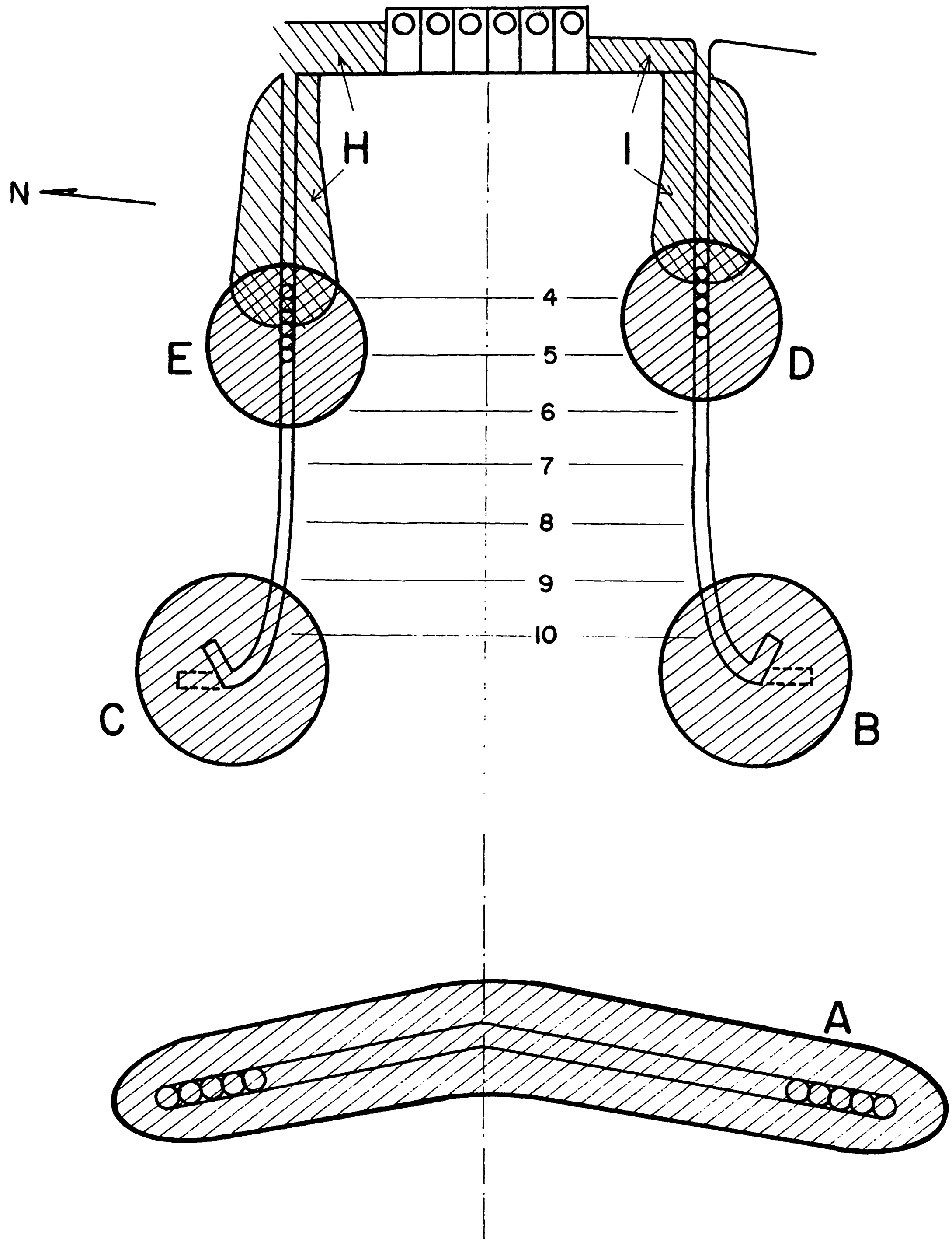
LUDINGTON PUMPED STORAGE PROJECT
LAKE FRONT MODEL
FIGURE No. 7

NOV. 1969



F : OVERTOPPING SOUTH JETTY APPROXIMATELY BETWEEN LINES 4.5 & 6.5, EXACT LOCATION AND MEASUREMENT NOT RECORDED.

G : OVERTOPPING BETWEEN END OF REVETMENT AND LINE 7, WORST AREA BETWEEN LINES 5 & 6.

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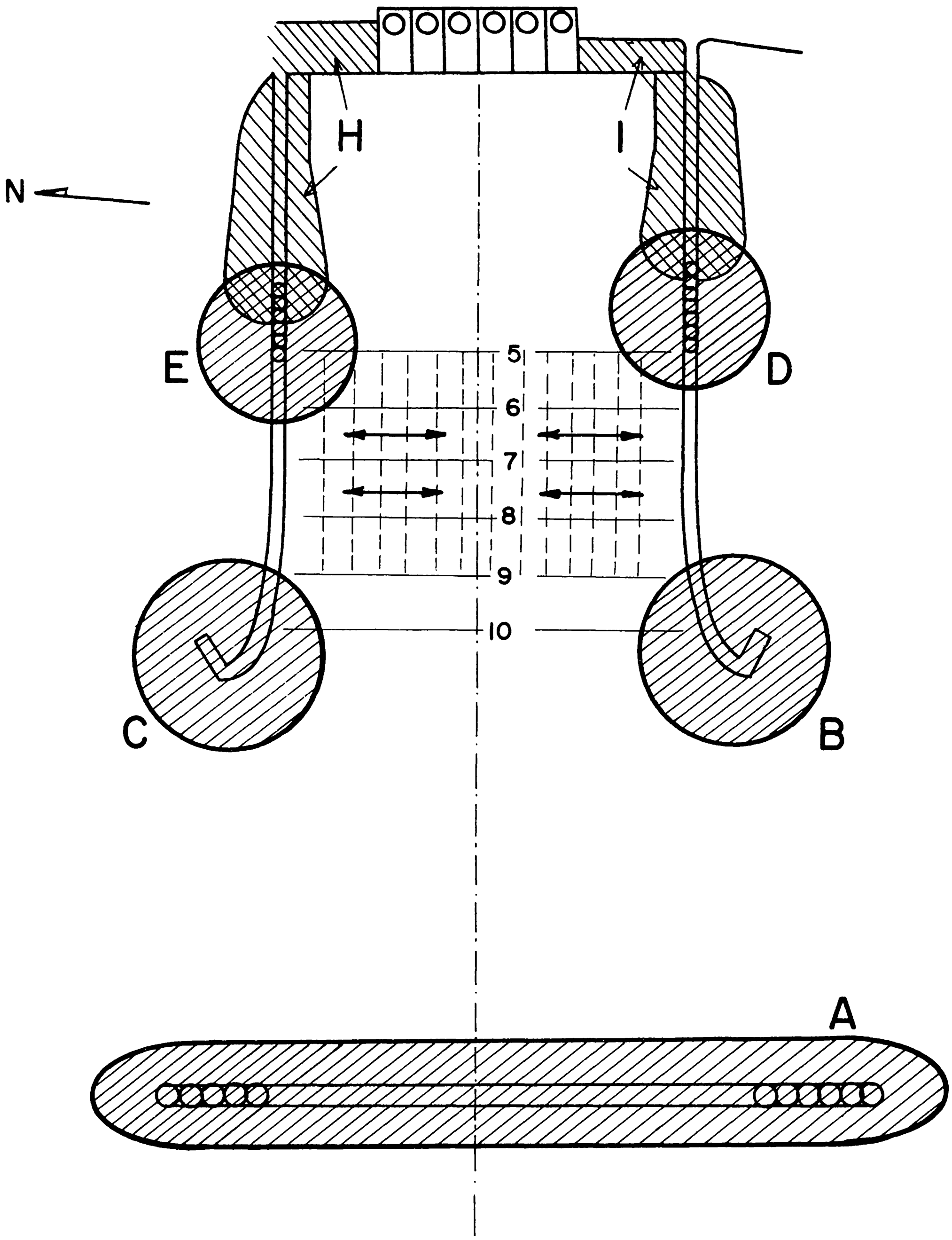
LAKE FRONT MODEL
FIGURE No. 8



F: OVERTOPPING SOUTH JETTY BETWEEN LINES 5 & 8.

G: STANDING WAVES FORMED BETWEEN JETTIES OVERTOPPING BOTH JETTIES BETWEEN LINES 5 & 9.

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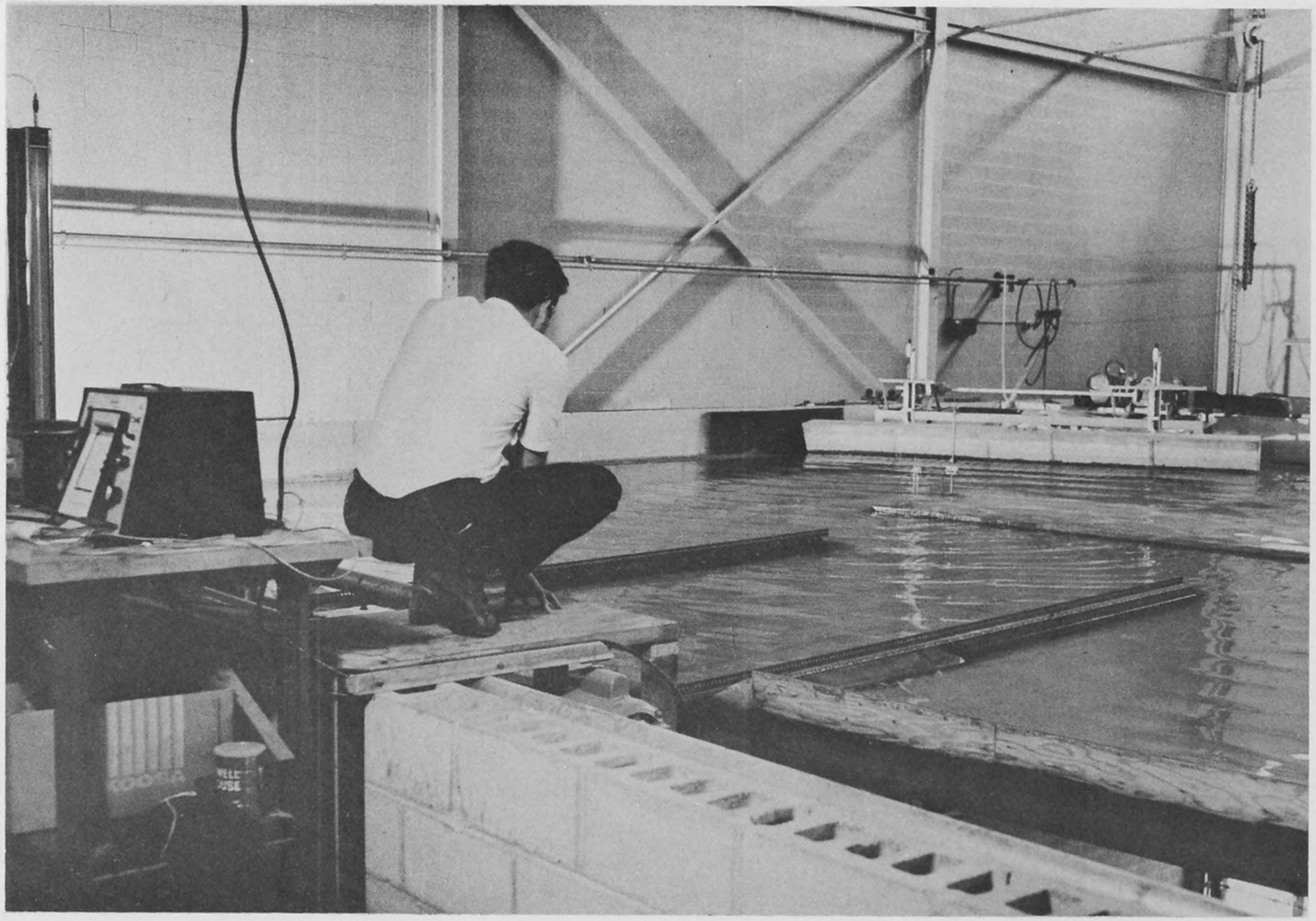
LAKE FRONT, MODEL
FIGURE No. 9



View from a point on shore just north of the harbor during southwesterly waves. The wave machine is in the background and part of the instrumentation is shown in the left foreground.

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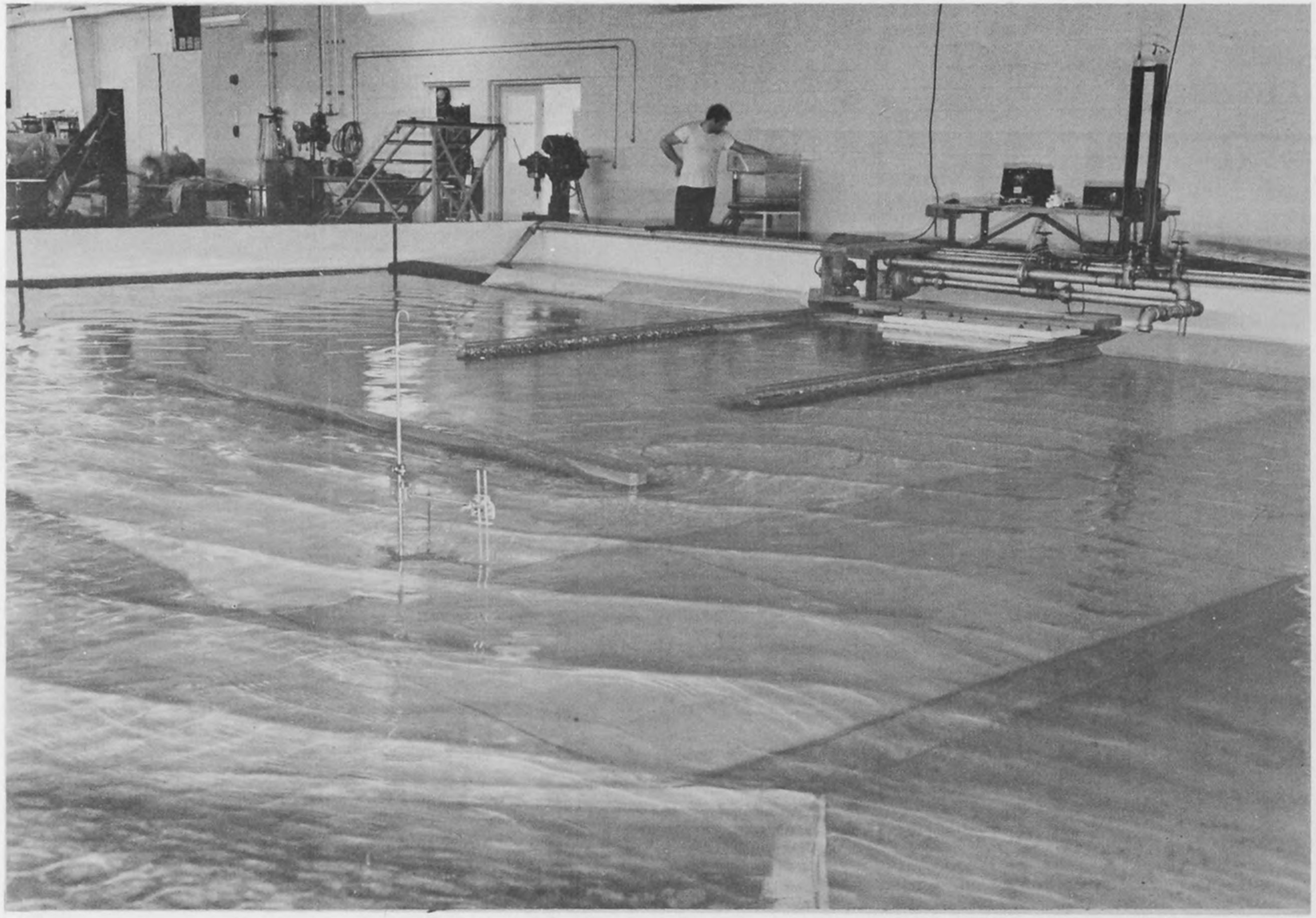
LAKE FRONT MODEL

PLATE No. 1



View from wave machine showing wave approaching from the S.W. A wave gage is shown in the foreground.

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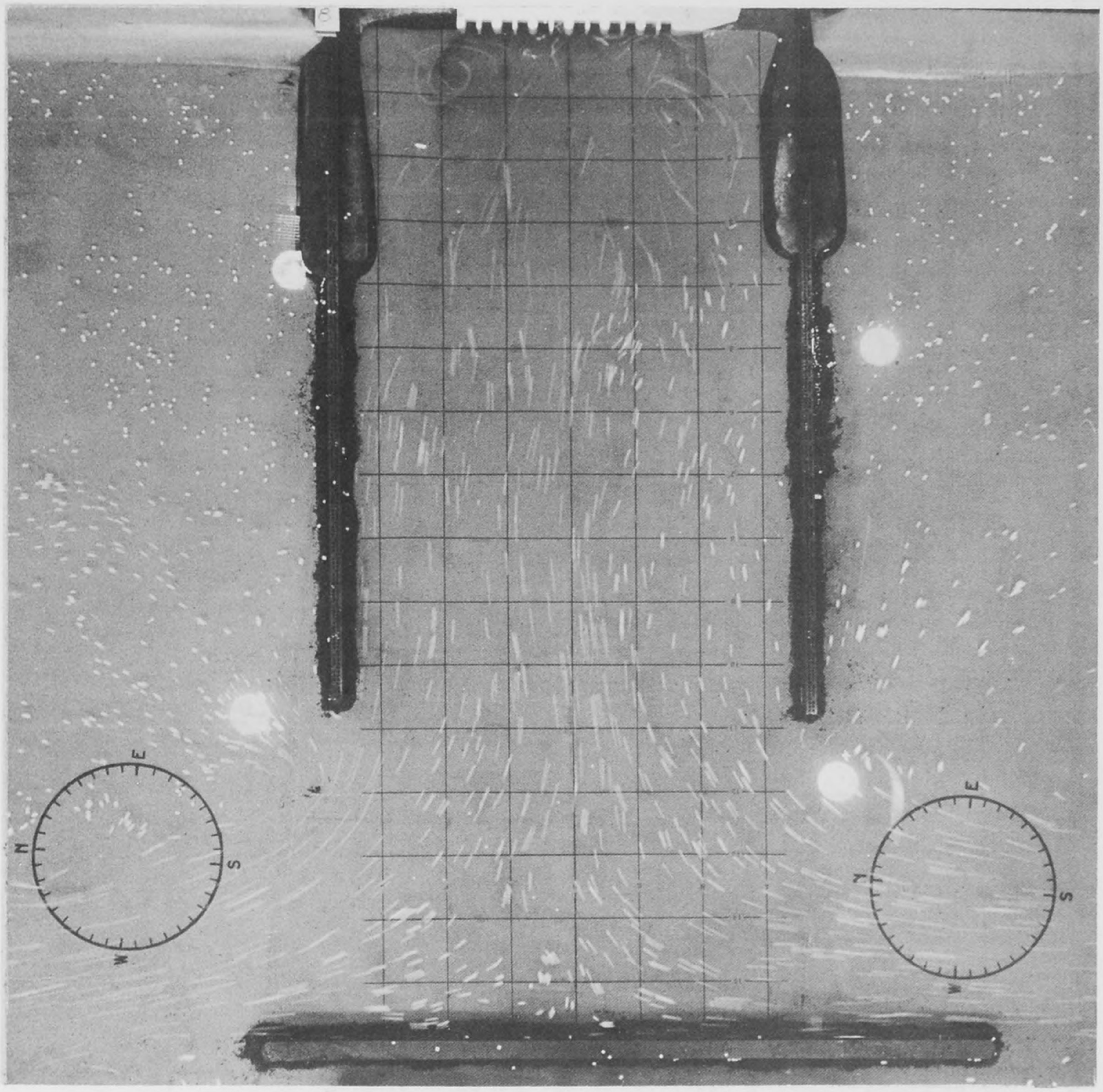
LUDINGTON PUMPED STORAGE PROJECT
LAKE FRONT MODEL
PLATE No. 2



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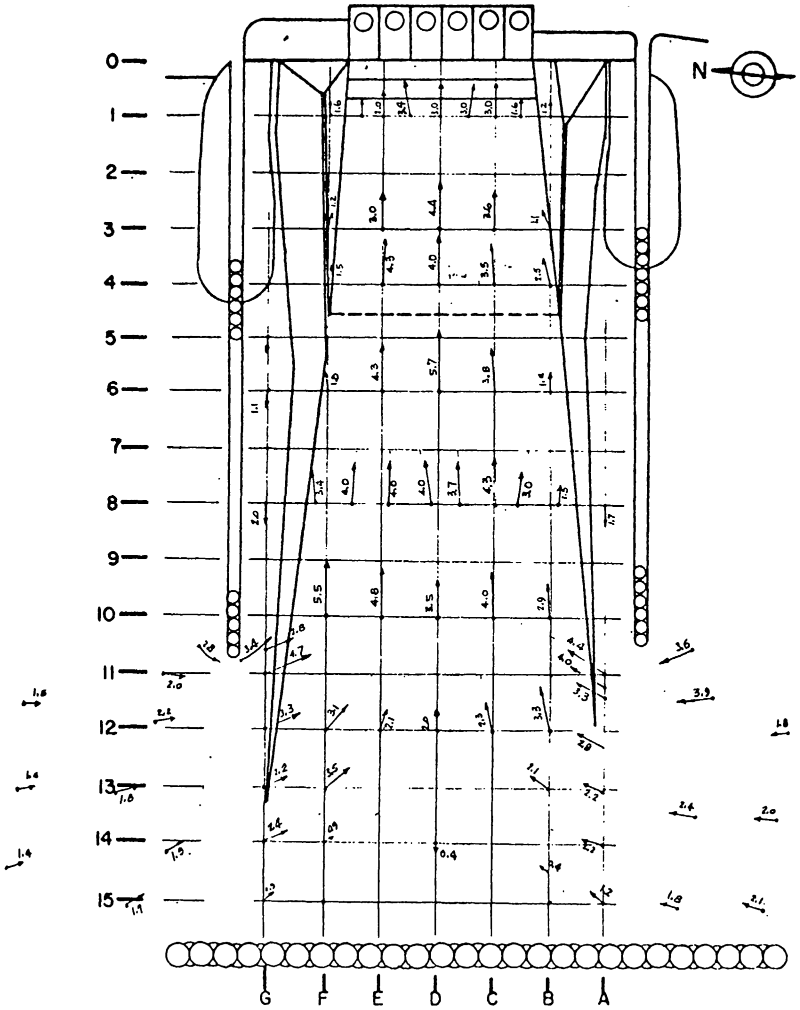
LUDINGTON PUMPED STORAGE PROJECT
LAKE FRONT MODEL
PLATE No. 3

TEST No. 1-7
SURFACE VELOCITIES: Generating @ 76,000
WAVES: None
LITTORAL CURRENT: None
SCALE: Grid system is in 150' increments NOV. 1969



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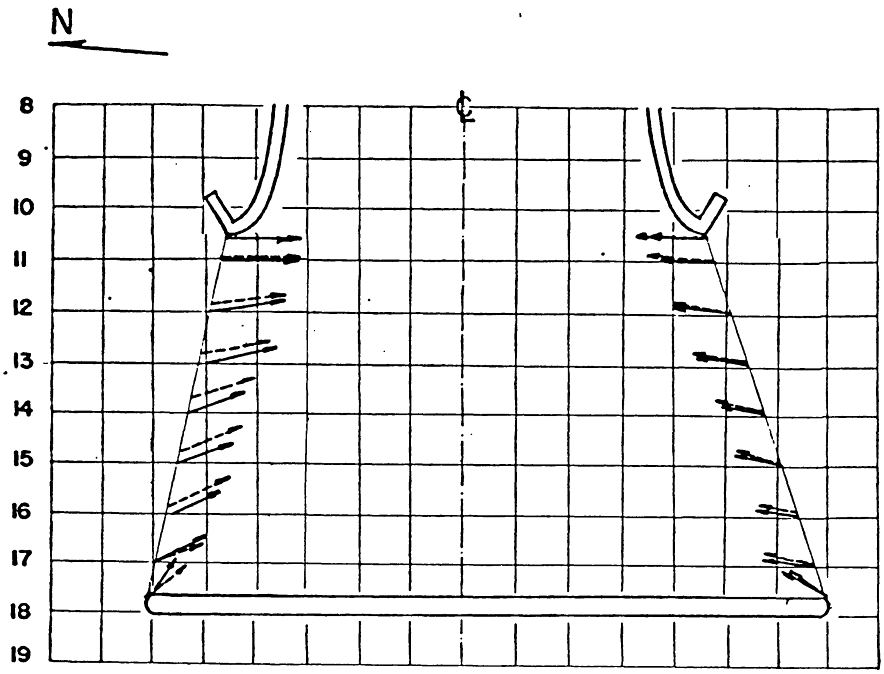
LUDINGTON PUMPED STORAGE PROJECT
LAKE FRONT MODEL

PLATE No. 20

TEST No. 1
BOTTOM VELOCITIES: Pumping @ 66,000 c.f.s.
SCALE: Grid system is in 150' increments NOV. 1969

Comparison of Plate No. 74 and Plate No. 75



Direction SWL EL. 579.5
Direction SWL EL. 574.5

Line No	North Opening			South Opening		
	Velocity		Velocity Reduction %	Velocity		Velocity Reduction %
	Plate 75	Plate 74		Plate 75	Plate 74	
-	2.66	2.69	+ 1.12	2.62	3.06	+14.40
11	2.66	2.44	- 9.00	2.24	2.69	+16.70
12	1.97	2.12	+ 7.06	1.60	1.73	+ 7.50
13	1.81	1.78	- 1.68	1.57	1.67	+ 6.00
14	1.70	1.73	+ 1.73	1.43	1.51	+ 5.30
15	1.48	1.81	+18.20	1.43	1.56	+ 8.35
16	1.72	1.56	-10.30	1.48	1.81	+18.20
17	1.33	1.78	+25.00	1.19	1.49	+20.10
-	1.20	1.75	+31.40	1.56	2.04	+23.50
Average	(1.81)	(1.89)	+ 7.06	(1.68)	(1.95)	+13.32

NOTE: + SIGN INDICATES REDUCTION

− SIGN INDICATES INCREASE

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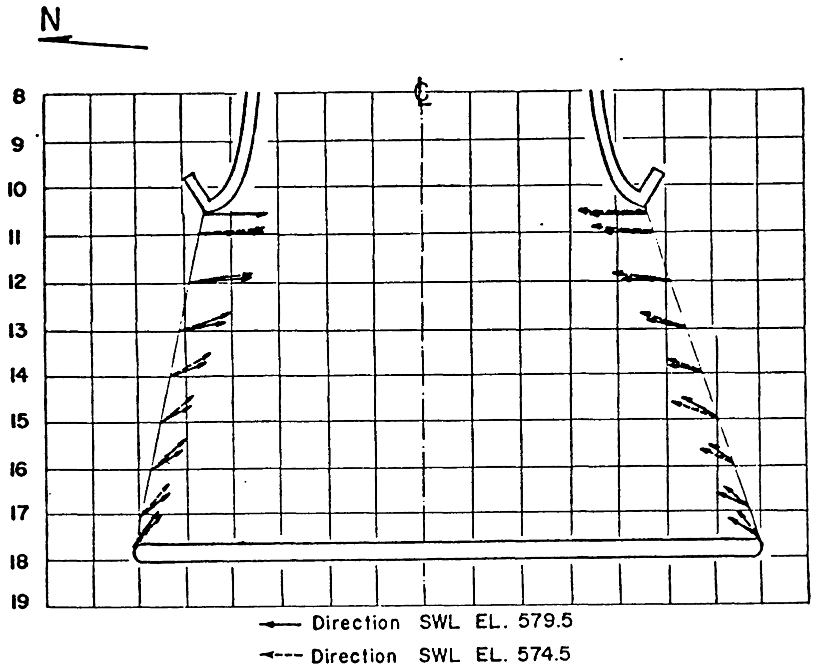
LUDINGTON PUMPED STORAGE PROJECT
LAKE FRONT MODEL

PLATE No. 78

TEST No.
BOTTOM VELOCITIES: Pumping @ 66,000 c.f.s.
SCALE: Grid system is in 150' increments NOV. 1969

Comparison of Plate No. 76 and Plate No. 77



Line No	North Opening			South Opening		
	Velocity		Velocity Reduction %	Velocity		Velocity Reduction %
	Plate 77	Plate 76		Plate 77	Plate 76	
-	2.69	2.51	-7.18	2.62	3.05	+14.10
11	2.30	2.46	+6.50	2.56	2.62	+ 2.29
12	1.97	2.01	+2.00	1.55	2.00	+22.50
13	1.83	1.75	-4.38	1.57	1.62	+ 3.08
14	1.62	1.75	+7.42	1.49	1.53	+ 2.61
15	1.81	1.53	-15.4	1.60	1.67	+ 4.35
16	1.43	1.80	+20.5	1.55	1.53	- 1.31
17	1.47	1.70	+13.5	1.16	1.57	+27.3
-	1.57	1.82	+13.7	1.39	1.51	+ 7.95
Average	1.85	1.92	+ 5.18	1.72	1.90	+ 9.20

NOTE: + SIGN INDICATES REDUCTION

— SIGN INDICATES INCREASE

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LUDINGTON PUMPED STORAGE PROJECT

LAKE FRONT MODEL

PLATE No. 79

TEST No.

BOTTOM VELOCITIES: Pumping @ 66,000 c.f.s.

SCALE: Grid system is in 150' increments NOV. 1969

AIIM SCANNER TEST CHART # 2

Spectra

4 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
6 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
8 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
10 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789

Times Roman

4 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
6 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
8 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
10 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789

Century Schoolbook Bold

4 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
6 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
8 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
10 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789

News Gothic Bold Reversed

4 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
6 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
8 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
10 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789

Bodoni Italic

4 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
6 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
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10 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789

Greek and Math Symbols

4 PT ΑΒΓΔΕΞΘΗΙΚΛΜΝΟΠΦΡΣΤΥΩΧΨΖαβγδεξθηικλμνοπφρστυωχψζ≧∓",./≦±=≠°><⋗⋖⋗⋜≡
6 PT ΑΒΓΔΕΞΘΗΙΚΛΜΝΟΠΦΡΣΤΥΩΧΨΖαβγδεξθηικλμνοπφρστυωχψζ≧∓",./≦±=≠°><⋗⋖⋗⋜≡
8 PT ΑΒΓΔΕΞΘΗΙΚΛΜΝΟΠΦΡΣΤΥΩΧΨΖαβγδεξθηικλμνοπφρστυωχψζ≧∓",./≦±=≠°><⋗⋖⋗⋜≡
10 PT ΑΒΓΔΕΞΘΗΙΚΛΜΝΟΠΦΡΣΤΥΩΧΨΖαβγδεξθηικλμνοπφρστυωχψζ≧∓",./≦±=≠°><⋗⋖⋗⋜≡

White

Black

Isolated Characters

e	m	1	2	3	a
4	5	6	7	o	•
8	9	0	h	I	B

MESH HALFTONE WEDGES

65

85

100

110

133

150

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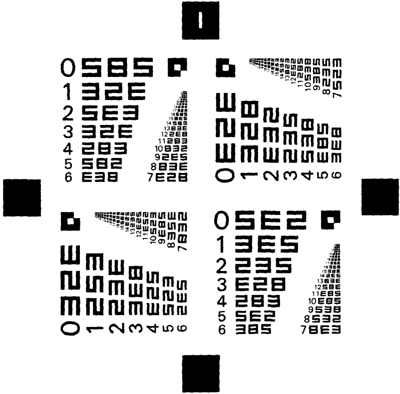
RIT ALPHANUMERIC RESOLUTION TEST OBJECT, RT-1-71



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