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WAVE RUN-UP AND OVERTOPPING LEVEE SECTIONS LAKE OKEECHOBEE, FLORIDA

Hydraulic Model Investigation

TECHNICAL REPORT NO. 2-449

January 1957

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

ARMY-MRC VICKSBURG, MISS.

PREFACE

The District Engineer, Jacksonville District, Corps of Engineers, requested the Waterways Experiment Station to conduct the hydraulic model investigation described in this report in a letter dated 17 August 1954. Authority to perform the investigation was granted by the Chief of Engineers, Department of the Army, on 27 August 1954, and tests were conducted during the period September 1954 to September 1955. During the course of the investigation test data were forwarded to the District Engineer immediately upon completion of each phase of the testing program.

Liaison was maintained during the course of the investigation by progress reports and conferences. The following representatives of the Jacksonville District, South Atlantic Division, Office, Chief of Engineers, and Beach Erosion Board attended conferences at the Waterways Experiment Station: Colonel H. W. Schull, Jr., CE, District Engineer, Captain I. A. Hunt, Jr., CE, Mr. E. W. Eden, Jr., Chief Planning and Reports Branch, and Mr. G. F. Snodgrass, engineer, Jacksonville District, Jacksonville, Florida; Mr. C. P. Lindner, Chief Engineer, Mr. E. L. Belladonna, engineer, Mr. L. F. Johnson, Chief, Planning and Reports Branch, and Mr. L. G. Leach, engineer, South Atlantic Division, Atlanta, Georgia; Mr. A. L. Cochran, Chief, Hydrology and Hydraulics Branch, and Mr. W. H. Booth, Jr., engineer, Office, Chief of Engineers, Washington, D. C.; and Mr. Thorndike Saville, Jr., Assistant Chief, Research Division, Beach Erosion Board, Washington, D. C. Also, Mr. C. L. Bretschneider, Consulting Engineer, Texas A & M College, College Station, Texas, and Mr. J. W. Johnson, Consulting Engineer, University of California, Berkeley, California, attended conferences at the Waterways Experiment Station.

The model study was conducted in the Hydraulics Division of the Waterways Experiment Station under the supervision of Mr. E. P. Fortson, Jr., Chief of the Hydraulics Division, Mr. F. R. Brown, Chief of the Hydrodynamics Branch, and Mr. R. Y. Hudson, Chief of the Wave Action Section. Waterways Experiment Station engineers actively connnected with the model study were Messrs. R. Y. Hudson, R. A. Jackson, and R. E. Cuckler.

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LIST OF SYMBOLS AND DEFINITIONS

A_r = area scale of model.

d_1 = water depth at toe of beach in feet.

d_2 = water depth at toe of levee in feet.

d_3 = water depth at break in grade of composite levee slopes in feet.

F = force in pounds.

F_r = force scale of model.

f = reads "function of."

H = wave height, vertical distance from crest to trough in feet.

h = height of wave run-up above wind-tide level in feet.

L = length. Wave length (horizontal distance between consecutive wave crests) in feet.

L_r = linear scale of model.

msl = mean sea level datum.

P_r = pressure scale of model.

Q_{ot} = volume of overtopping water per unit length of wave crest divided by wave period (T) in cubic feet per second.

T = time. Wave period (time required for a wave crest to travel one wave length) in seconds.

T_r = time scale of model.

t = time required for water to travel downslope from point of run-up to water surface in seconds.

V_r = velocity scale of model.

$\overline{V}_r$ = volume scale of model.

W_r = weight scale of model.

y = height of levee crown above wind-tide level in feet.

α = angle between the horizontal and face of continuous levee slope in degrees.

α_1 = angle between the horizontal and levee face below the berms and below water depth d_3 in degrees.

α_2 = angle between the horizontal and levee face above the berms and above water depth d_3 in degrees.

γ_r = specific-weight scale of model.

H/L = wave steepness.

h/H = wave run-up factor.

$\frac{Q_{ot}T}{H^2}$ = wave overtopping factor.

y/H = relative levee-crown elevation.

cot = cotangent.

fetch = horizontal distance, in the direction of the wind, over which the wind blows.

hurricane = tropical storm with wind speeds in excess of 75 mph.

significant wave height = statistical term referring to the average height in feet of the one-third highest waves for a selected interval of time.

wtl = wind-tide level. Elevation of water surface in feet above undisturbed water level resulting from wind forces on water surface.

SUMMARY

A hydraulic model investigation of levee sections proposed for impounding Lake Okeechobee was conducted to determine the effectiveness of different types of sections in preventing excessive overtopping of the protective levees by hurricane-generated waves. Most of the tests were conducted on 1:30-scale models of the levee sections installed in a wave flume 94 ft long with a testing section 1.5 ft deep and 1 ft wide. A few tests were conducted in a larger wave flume (119 ft long, 4 ft deep, and 5 ft wide) using a linear scale of 1:17, model to prototype. The height of wave run-up on and volume of water overtopping levee sections of various slopes, crown elevations, and lakeside berms for several depths of water at beach toe and levee toe were determined. It was concluded from the results of this model study and other related investigations that:

a. The height of wave run-up on levee slopes at Lake Okeechobee during hurricanes is a function of wave steepness, levee slope, shape of levee face, water depth at the toe of the levee, roughness and permeability of the levee slope, and wind speed (effects of the roughness and permeability of the levee slope and wind speed were not studied in this investigation; however, tests conducted at the Waterways Experiment Station for the Beach Erosion Board showed that a rough surface reduced wave run-up appreciably, Waterways Experiment Station tests on rubble-mound breakwaters showed that wave run-up for permeable breakwaters is considerably less than that for the impermeable-type structures used in the present investigation, and tests conducted by Sibul and Tickner at the University of California indicated that wave run-up increases as wind velocity increases).

b. Overtopping of Lake Okeechobee levees during hurricanes is a function of all the variables listed above for wave run-up and, in addition, is a function of levee crown elevation above wind-tide level.

c. The wave run-up data obtained on the 1:30-scale model are adequate to determine the most efficient levee slope.

d. The wave run-up data are also adequate to determine the crown elevation of the levee, assuming small local wind speed; however, some allowance should be made for the fact that model studies at the University of California indicated that wave run-up is a function of local wind speeds.

e. The overtopping data obtained on the 1:30-scale model were apparently affected somewhat by the small scale used, and

the quantities of overtopping water obtained on this model should probably be increased about 10 per cent for the largest quantities obtained, to as much as 100 per cent for the small quantities.

f. Additional model tests, using the largest possible model scale, should be conducted to aid in the design of a levee section that would not fail as the result of erosion caused by overtopping water.

WAVE RUN-UP AND OVERTOPPING,

LEVEE SECTIONS, LAKE OKEECHOBEE, FLORIDA

Hydraulic Model Investigation

PART I: INTRODUCTION

Description of Lake Okeechobee[2*]

1. Lake Okeechobee is a shallow, fresh-water lake situated in the south central portion of the Florida peninsula, at the northern edge of the Everglades (plate 1). The lake is about 31 miles in diameter and its surface area is approximately 730 sq miles. It is comparatively shallow--the minimum lake level is about 15 ft above msl--with the greatest depth near the center. At the deepest point the elevation of the lake bottom is about mean sea level. The east foreshore of the lake is 6 to 12 ft deep; elsewhere depths along the foreshore are shallow with slopes varying from 1 to 2 ft per mile. Prior to an attempt to control the water level of Lake Okeechobee, and to drain the Everglades, the normal elevation of the lake water surface was about 20.5 ft above msl, the normal fluctuation was about 2.5 ft, and during high stages lake waters were discharged over the southern rim into the Everglades.

Existing Improvements**

2. Existing improvements consist of works designed and constructed for flood control, navigation, and irrigation. About 66 miles of levee have been constructed around the south shore of the lake from the St. Lucie Canal on the east to high ground near Lakeport on the west to prevent southward flow into the Everglades. Also, about 16 miles of levee

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* Raised numerals refer to similarly numbered entries in the List of References which follows the text.

** The following discussions of existing and proposed improvements, and the problems at Lake Okeechobee which necessitated the hydraulic model studies, were abstracted from references 3 and 11.

have been constructed on the north shore in the vicinity of the city of Okeechobee. The crown elevations of the levees vary between +32.5 ft msl on the north at the city of Okeechobee and +37.0 ft msl on the south at Lake Harbor. However, these flood-control structures do not adequately protect the land behind them. The present major outlets, the St. Lucie Canal and the Caloosahatchee River, are integral parts of the Lake Okeechobee Waterways and provide a navigable channel with a minimum depth of 8 ft from the Atlantic Ocean to the Gulf of Mexico. Each outlet contains locks and dams and control works for regulating lake outflow.

Proposed Additional Improvements[3,11]

3. Additional works comprising part of the Central and Southern Florida Flood Control Project were authorized by the Flood Control Acts of 1948 and 1950. Because of the importance of protecting the surrounding area from further hurricane damage, enlargement of the Lake Okeechobee levees is a major feature of the over-all plan. The plan of protection includes greater storage capacity for Lake Okeechobee, which requires raising the crown elevation of existing levees, and construction of additional levees for complete encirclement of the lake; protection of the Lake Okeechobee agricultural area by encircling levees and a system of interior canals and levees with pumping facilities for removal of floodwaters; provision for three large conservation areas unsuitable for agriculture; a levee to protect the urban east coast from Everglades floodwaters, with canals for removing local floodwaters; floodway channels in the Kissimmee River Basin, with suitable control structures to prevent overdrainage; and removal of floodwaters from the Upper St. Johns River Basin by diversion canals to Indian River.

The Problem[3,11]

4. The design of a system of encircling levees for Lake Okeechobee presents a difficult hydraulic problem because the levees must be capable of containing the high tides and waves caused by hurricane action on the

water surface. Consequently, the problem involves forecasting wind tides, wave heights, and wave run-up on the levees resulting from hurricanes. Also experience has shown that a large percentage of levee failures have occurred on the landside as a result of overtopping. (A major cause of such failures appears to be lack of adequate drainage at the toe of the landside slope.) Therefore, the design criteria for levee shape, levee height, and resistance to erosion are based on the most critical combination of wind tides, wave action, and wave run-up that may be expected to occur with a frequency sufficient to be considered significant relative to the economic life of the project structures.

Bases for Design Criteria

Hurricane studies

5. Little information is available on Florida hurricanes occurring prior to the twentieth century. During the period 1901-1953 twenty-two hurricanes crossed the Florida coasts. Eighteen of these hurricanes passed within 50 miles of Lake Okeechobee, which is near enough to be considered potentially dangerous to the lake area. Studies indicate that the greatest number of hurricanes in Florida occur in the months of September, October, August, July, November, June, and December, in that order.

6. Studies for the purpose of developing a project design storm based on the principles of hurricane behavior in the project area have been in progress since 1936. These studies are being conducted by the Jacksonville District in collaboration with the Hydrometeorological Section of the United States Weather Bureau, Washington, D. C. Based on the studies the following project design storms were selected for use in the design of the Lake Okeechobee levees:

a. A maximum probable hurricane with a central pressure of 26.75 in. of mercury, a maximum wind radius of 13 miles, and an average velocity over the lake of 95 mph.

b. A standard project hurricane with a central pressure of 27.5 in. of mercury, a maximum wind radius of 30 miles, and an average velocity over the lake of 85 mph.

7. These design hurricanes are used to predict wind tides, wave

heights, and wave periods applicable to the lake-levee designs. Either of the design hurricanes would subject the levees to more critical conditions than any of the 64 hurricanes of record that have struck the United States mainland.

Wind tides

8. A wind tide is the increase in elevation of a water surface caused by the cumulative effects of wind forces acting on the surface of a body of water. Wind tides are generated by surface friction between wind and water, and the return currents are restricted by bottom friction. Maximum wind tides of record on Lake Okeechobee, observed during the hurricanes of 1928 and 1949, produced tide heights that ranged from 10 to 12 ft above still water level. The continuous change in wind directions during hurricanes causes further complications of tidal movement along the shore of the lake. Wind-tide formulas[1,5,7,8, and 9] developed from basic relationships and modified in accordance with results of analyses of observed wind tides on Lake Okeechobee have been used to compute wind-tide elevations for six basic fetch directions on the lake. The computed wind tides have been found to be in good agreement with observed wind tides.

Wave action

9. The problem of wave action is of equal importance with that of wind tide. The incident wind is the controlling factor for both wind tides and wave action. The highest tides on Lake Okeechobee may be expected to occur concurrently with the highest waves. Thus, a solution, in terms of levee design criteria, requires the determination of the probable wave heights and their duration under design wind-tide conditions, the frequency of occurrence of high waves, the height of wave run-up, and the quantity of water overtopping the levee with respect to time.

10. Variables that affect wave height and wave period during a hurricane are wind velocity and duration, fetch, and water depth. The shallow water depths of Lake Okeechobee limit the dimensions of the waves that reach the levees. Variables that may influence wave run-up are wave height, wave period, depth of water at the levee toe, levee

slope, shape, roughness, and permeability. The effects of roughness and permeability on wave run-up were not investigated in the model tests. The design of an effective system of levees is dependent upon accurate information concerning the height of wave run-up on various levee shapes and slopes. However, wave run-up recorders have not been used on Lake Okeechobee, and only minor incidents of run-up have been observed, so very little field data are available.

Tentative Design Criteria

11. Tentative design criteria for the Lake Okeechobee levee system indicate a conservation pool elevation of 16.4 ft above msl. Assuming that a standard project flood for the Lake Okeechobee Basin is routed through the lake during a hurricane, the lake level might rise to an elevation of about 22 ft above msl. For the north-south fetch, the standard project hurricane could cause a water-surface elevation of about 33 ft above msl. Significant waves would be approximately 10-12 ft in height, and assuming a conservative wave run-up of only 11 ft, a levee crown elevation of 44 ft above msl would be required to prevent overtopping. Thus, considering that the present maximum crown elevation is only +37 ft msl, it is apparent that the combination of conditions likely to occur poses a problem requiring very careful consideration in order that an economical, yet effective, levee system be provided. Because of the high cost of levee construction, and the hazards to life and property from flooding, the correct evaluation of all critical factors concerning levee design is very important.

Purpose of Model Study

12. Since accurate field data necessary for levee design were not available, a model study was conducted to obtain information relative to wave run-up and overtopping of levees from which design criteria could be formulated for use in selecting the optimum-type levee cross sections to impound Lake Okeechobee waters during hurricanes. Specifically, it

was desired to obtain information concerning the effects of:

a. Levee slope on wave run-up.

b. Water depth at toe of beach on wave run-up.

c. Water depth at toe of levee on wave run-up.

d. Levee crown elevation on amount of overtopping.

e. Different combinations of berms on wave run-up and overtopping.

f. Different combinations of composite slopes on wave run-up and overtopping.

PART II: THE MODELS

13. Two model scales were used in this investigation: 1:30-scale section models were used for testing all the proposed levee section designs, and 1:17-scale section models were used to determine the effect of scale on the accuracy of selected model results.

1:30-scale Section Models

14. The 1:30-scale models of proposed levee sections were tested in an existing steel-framed flume (fig. 1). One side wall of the flume (designated flume No. 1) is of plate glass, and the other is of aluminum plates. Flume No. 1 is 94 ft long and consists of a vertical-transition section, a horizontal-transition section, and the testing section. The purpose of the vertical-transition section, which is 4 ft wide by 3 ft deep by 18 ft long, is to facilitate the generation of large waves. A wave generator occupies the rear half of this section; the forward 9 ft comprise an adjustable-slope section. The horizontal-transition section which is 4 ft wide, 1.5 ft deep, and 36 ft long, increases the height of

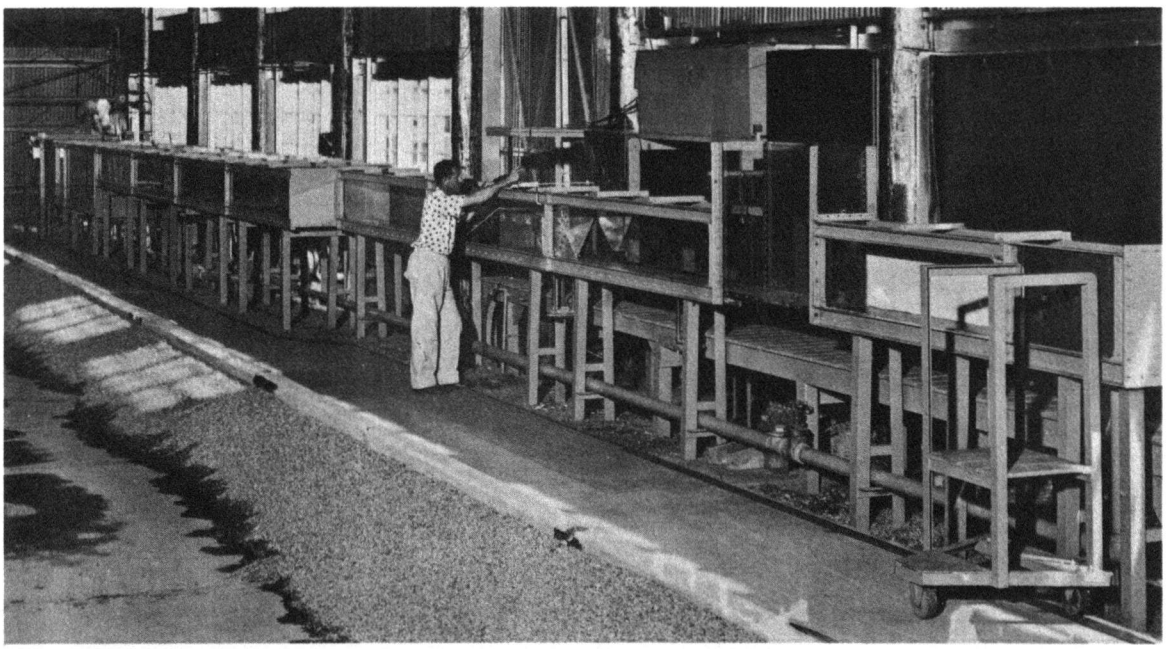

Fig. 1. Wave flume No. 1

waves generated in the deeper portion of the flume, and compensates for the attenuation of wave height due to friction. Adjustable plates 27 ft in length inside the fixed walls of the flume are used to vary the width of this section from 4 to 1 ft. The testing section is 1 ft wide, 1.5 ft deep, and 40 ft long. The levee sections were set in this part of the flume and were modeled of metal plates and plywood boards so that the face shapes of the various designs could be easily formed. The model scale of 1:30 was selected after consideration of the dimensions of the flume, the capabilities of the wave generator with respect to the prototype water depth, dimensions of the prototype levee sections, and dimensions of hurricane-generated waves on Lake Okeechobee.

15. Model waves were reproduced by a plate-type wave machine, 4 ft in width, which generates waves by periodic displacement of the water coincident with the horizontal motion of the plate (fig. 2). Wave heights were determined by measurement of the water-surface elevations with respect to time. The complete apparatus consists of a parallel-rod-type wave gage, balancing circuit, Brush Universal Analyzer, and a Brush pen-motor-type magnetic oscillograph (fig. 3).

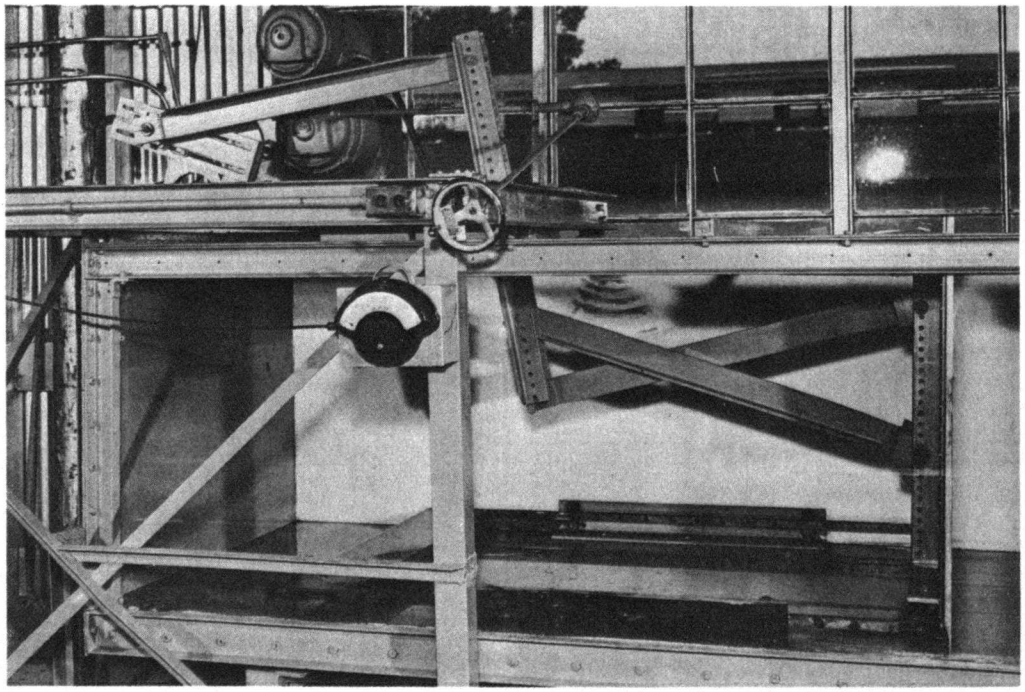

Fig. 2. Wave generator (flume No. 1)

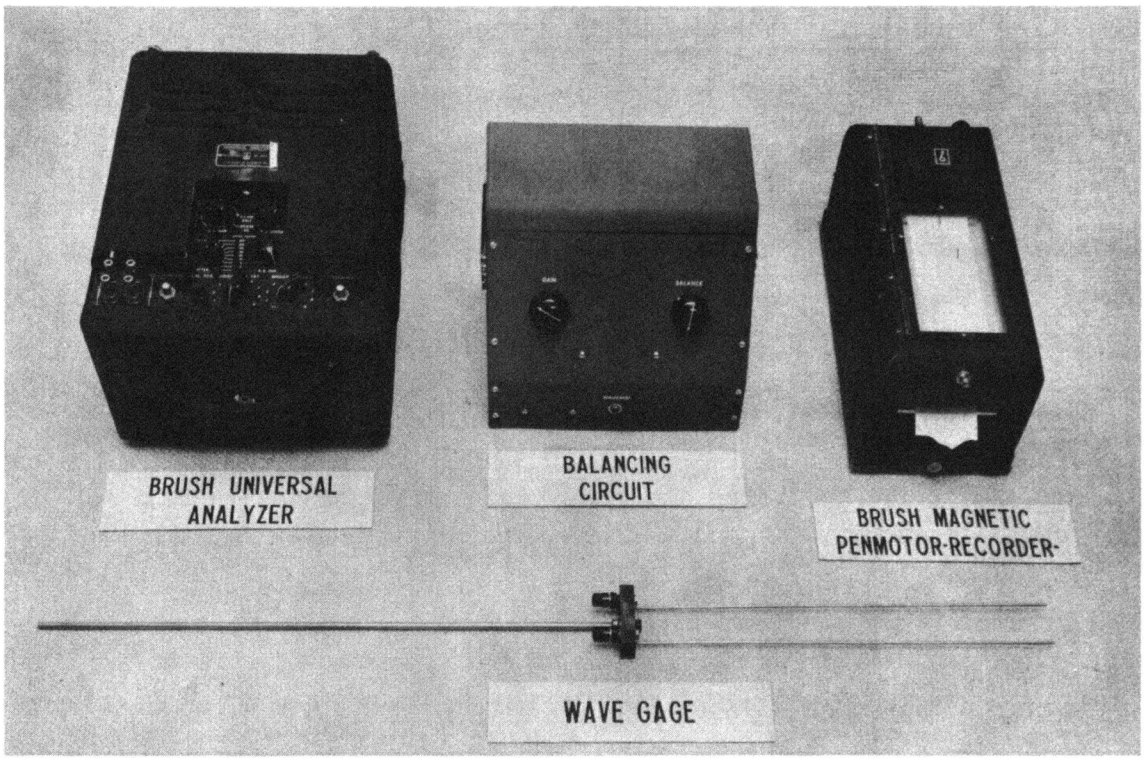

Fig. 3. Wave-height measuring apparatus

16. After the linear scale had been selected, the model was designed and operated in accordance with Froude's model law. Based upon Froude's law, a linear scale (L_r) of 1:30 and a specific weight scale (γ_r) of 1:1, the following model-prototype relationships were derived:

Characteristics	Dimensions*	Model-prototype Scales
Area	L^2	$A_r = L_r^2 = 1:900$
Volume	L^3	$\overline{V}_r = L_r^3 = 1:27,000$
Time	T	$T_r = L_r^{1/2} = 1:5.48$
Velocity	L/T	$V_r = L_r^{1/2} = 1:5.48$
Unit pressure	F/L^2	$P_r = L_r\gamma_r = 1:30$
Force	F	$F_r = L_r^3\gamma_r = 1:27,000$
Weight	F	$W_r = L_r^3\gamma_r = 1:27,000$

* In terms of force, length, and time.

1:17-scale Section Models

17. The 1:17-scale levee sections were constructed of steel plate, and were installed in a concrete wave flume 5 ft wide, 4 ft deep, and 119 ft long (designated flume No. 2, see fig. 4). Waves were generated by a plunger-type wave machine (fig. 5). Wave heights were measured and recorded with equipment similar to that shown on fig. 3. The test sections were installed in the flume 90 ft from the wave generator.

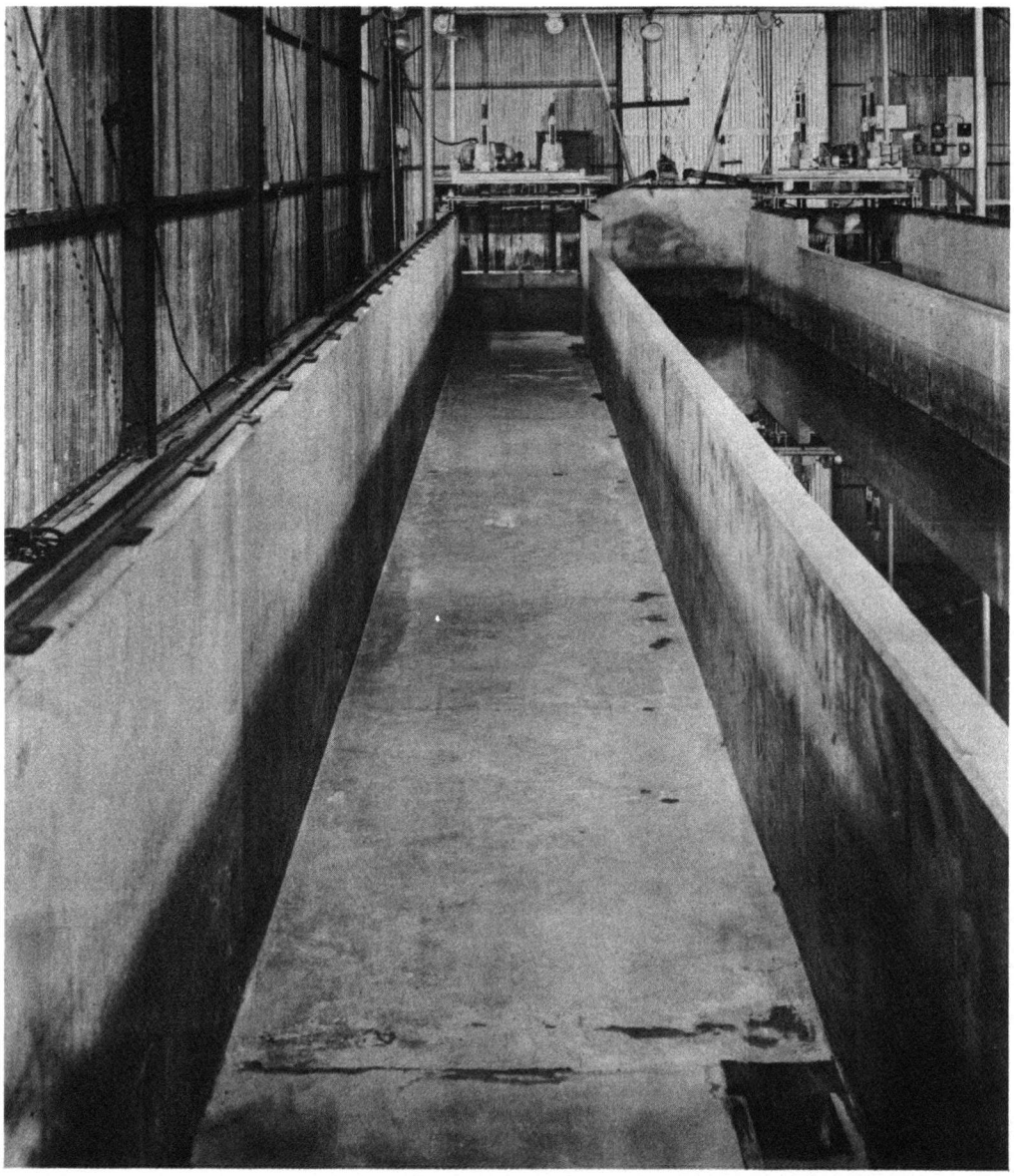

Fig. 4. Wave flume No. 2

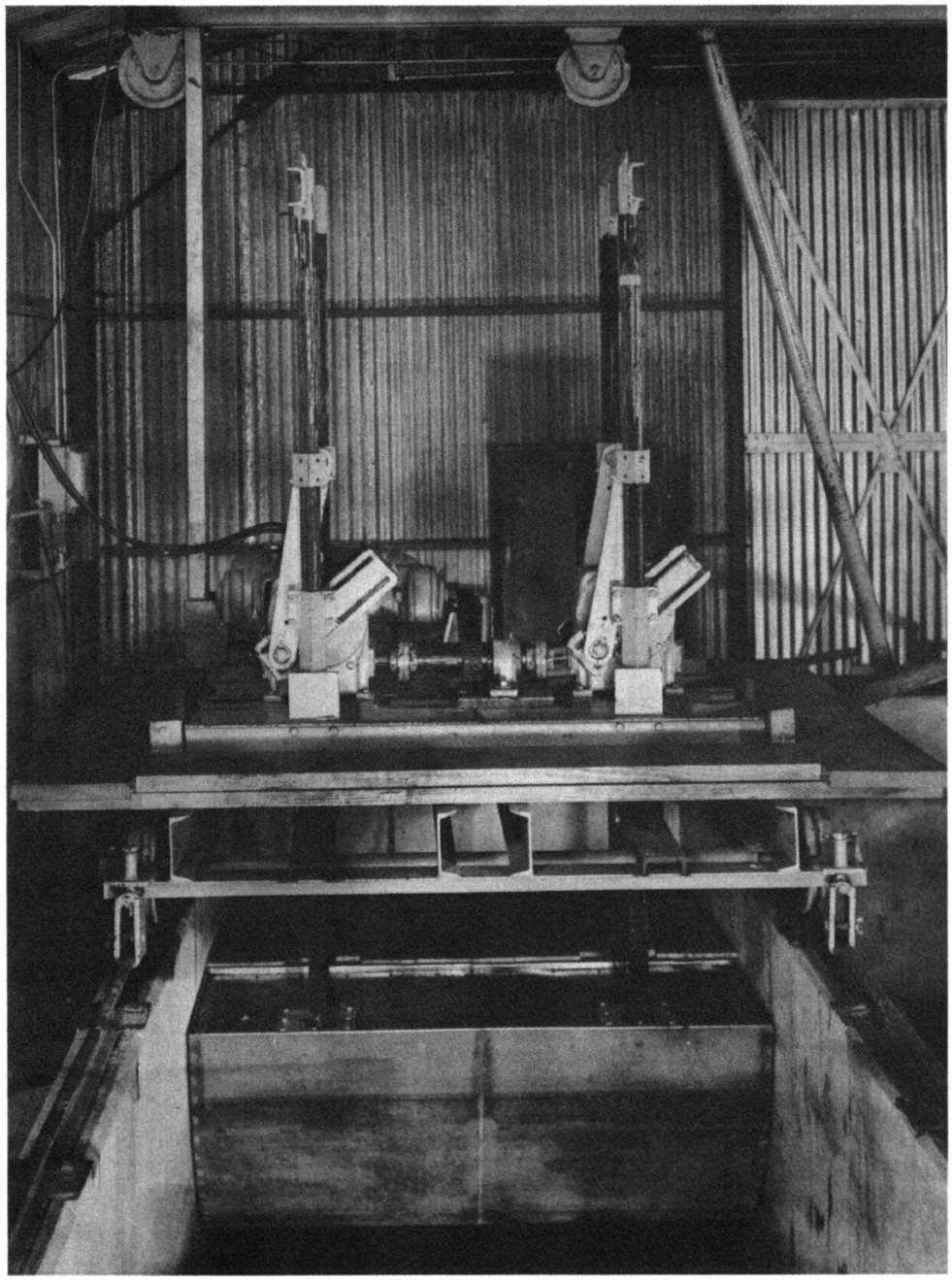

Fig. 5. Wave generator (flume No. 2)

18. Based upon a linear scale (L_r) of 1:17, a specific weight scale (γ_r) of 1:1, Froude's model law, and simple geometric considerations, the following model-prototype relationships were derived:

Characteristics	Dimensions	Model-prototype Scales
Area	L^2	$A_r = L_r^2 = 1:289$
Volume	L^3	$\bar{V}_r = L_r^3 = 1:4913$
Time	T	$T_r = L_r^{1/2} = 1:4.12$
Velocity	L/T	$V_r = L_r^{1/2} = 1:4.12$
Unit pressure	F/L^2	$P_r = L_r \gamma_r = 1:17$
Force	F	$F_r = L_r^3 \gamma_r = 1:4913$
Weight	F	$W_r = L_r^3 \gamma_r = 1:4913$

PART III: THE TESTING PROGRAM

Testing Procedure

19. Tests were conducted on various levee designs to determine the height of wave run-up and the volume of overtopping water. The height of wave run-up was measured by visual observations of a scale scribed on the lakeside face of the levee section. Each test run consisted of determining the average run-up for a train of 10 to 15 waves. The average of the results of five test runs was taken as the height of wave run-up for a particular levee design. The volume of overtopping water was measured in a calibrated tank located immediately behind the test sections, and the volume of overtopping water for a given levee section was determined by averaging the results of three test runs. For each test run the first 3 to 5 waves in the train were wasted and the next 3 to 10 waves were drained into the measuring tank.

20. Complete wave trains were recorded by means of the wave gage and oscillograph. Accurate determination of the height of test waves was complicated by the fact that the wave train contained waves of abnormal height as a result of starting and stopping the wave machine. These larger waves, which occurred at the ends of each wave train, influenced to some extent the height of wave run-up and the quantity of overtopping water. Therefore, they were not considered as an integral part of the wave train in the data analyses. The heights of test waves were defined statistically as the average height of the one-third highest waves, excluding the larger waves at the ends of the wave trains.

Test Conditions

21. The wave characteristics, water depths, and beach slope to be used in the tests were selected during a conference[6] at the Waterways Experiment Station before the model study was undertaken. The water depths selected were 10.0, 17.5, and 25.0 ft measured at the toe of the beach slope (d_1), and corresponded to depths in the horizontal portion

of the wave flume. Water depths at the toe of the levee slope (d_2) were 5.0 ft and 10.0 ft. A beach slope of 1 on 10 was used for all tests. The water depths selected represented estimates of average depths at various localities along the lake shore that require additional protection during hurricanes.

22. The dimensions of test waves and the corresponding water depths d_1 used for testing are as follows:

Wave Period T, sec	Wave Height H, ft	Water Depth d_1, ft
6.0	12.0	25.0
7.0	12.0	25.0
5.5	10.0	25.0
6.5	10.0	25.0
5.0	8.0	25.0 and 17.5
6.0	8.0	25.0 and 17.5
4.5	6.0	25.0 and 17.5
5.5	6.0	25.0 and 17.5
4.5	4.0	25.0, 17.5, and 10.0
5.5	4.0	25.0, 17.5, and 10.0

The initial selection of test waves, based on studies by Bretschneider,[1] included all waves listed above except those 4 ft in height. The 4-ft waves were added to the testing program after it was found that the wave machine in flume No. 1 would not generate waves higher than 4 ft in water as shallow as d_1 = 10.0 ft.

Levee Sections Tested

23. The details of the levee sections tested in the eight test series are shown on plate 2. Of the seven test series conducted on the 1:30-scale models, wave run-up was measured in four (test series 1-4) and overtopping was measured in three. Test series 8 was performed on the 1:17-scale models, and included measurements of both run-up and overtopping.

1:30-scale model, test series 1-7

24. Test series 1. Test series 1 involved the measurement of wave run-up on the face of levee sections with lakeside slopes of 1 on 2, 1 on 3, 1 on 4, 1 on 6, and 1 on 10. Water depths d_1 were 10.0, 17.5,

and 25.0 ft; water depth d_2 was 10.0 ft. Figs. 6 and 7 show levee sections with slopes of 1 on 3 and 1 on 6 installed in flume No. 1.

25. Test series 2. This series of tests was similar to test series 1 except that d_2 was 5.0 ft instead of 10.0 ft.

26. Test series 3. In test series 3 wave run-up on various berm designs constructed on the lakeside of the levee was measured. Berm widths of 30.0, 50.0, and 70.0 ft were tested with levee slopes of 1 on 6 below wtl and 1 on 3 above wtl, and with levee slopes of 1 on 3 both above and below wtl. Fig. 8 shows the test section with the former slopes and a 70-ft berm. Water depths d_1 and d_2 were 25.0 ft and 10.0 ft, respectively. The slope of the berm face was 1 on 20 and the lakeside toe of the berm was at elevation 0.0 ft wtl.

27. Test series 4. These tests involved determination of the height of wave run-up on composite levee slopes of 1 on 3 and 1 on 6, and 1 on 3 and 1 on 10. Water depths at the change in slope of the levee face (d_3) were 0.0 and 3.0 ft. Water depths d_1 and d_2 were 25.0 and 10.0 ft, respectively. Fig. 9 shows composite levee slopes of 1 on 3 and 1 on 10 with a d_3 of 3.0 ft.

28. Test series 5. Test series 5 involved measurement of quantities of overtopping water for levee slopes of 1 on 3 and 1 on 6, and several levee crown elevations. These crown elevations were 1/4 h, 1/2 h, 3/4 h, and h with h equal to the heights of wave run-up determined from test series 1. The width of the crown of the levee sections was 13.0 ft. Water depths d_1 and d_2 were 25.0 and 10.0 ft, respectively. The quantity of overtopping water for each test wave was measured for each of the four different crown elevations.

29. Test series 6. These tests were made to determine the quantities of overtopping water for a 1-on-3 levee slope with a 30-ft-wide berm constructed on the levee face. The toe of the berm was at elevation 0.0 ft wtl. The width of the levee crown was 13.0 ft, and the crown elevations were 1.5 ft, 1/4 h, 1/2 h, 3/4 h, and h; h was determined from test series 3. Water depths d_1 and d_2 were 25.0 and 10.0 ft, respectively.

30. Test series 7. In test series 7 the quantities of overtopping

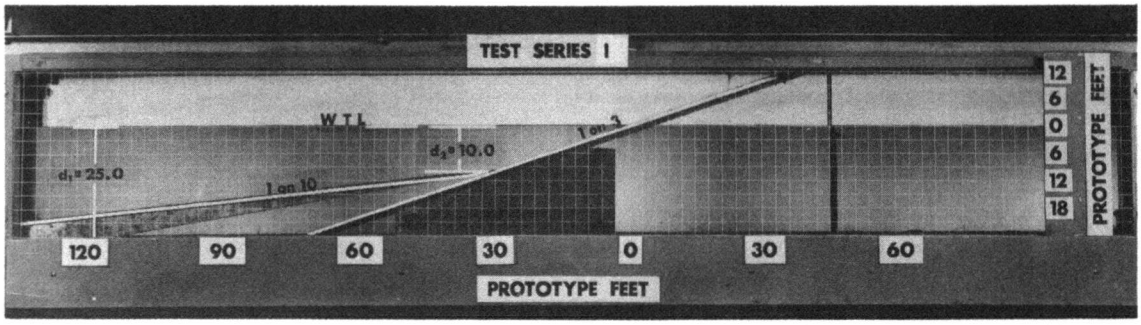

Fig. 6. 1-on-3 levee slope

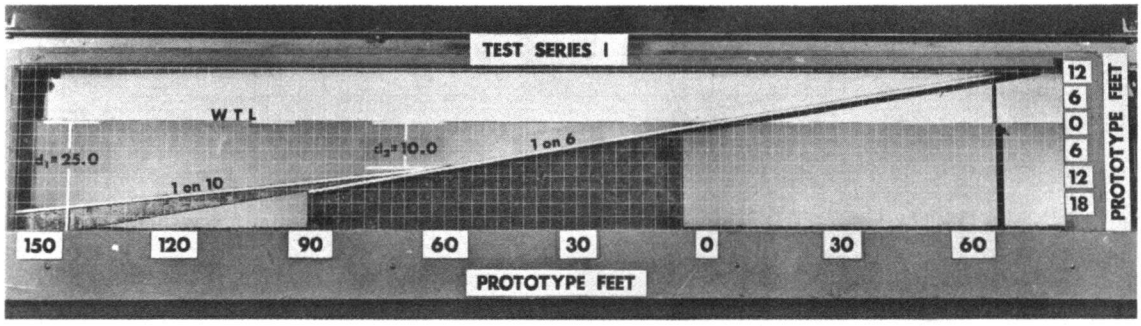

Fig. 7. 1-on-6 levee slope

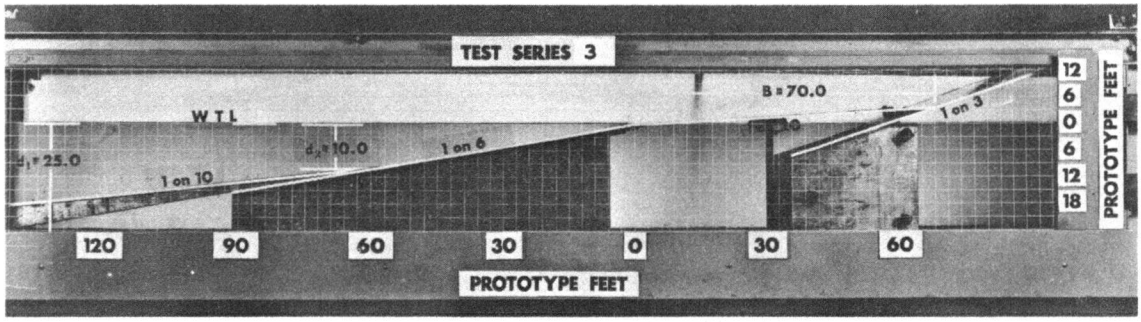

Fig. 8. 1-on-6 levee slope below wind-tide level; 1-on-3 levee slope above wind-tide level; 70-ft-wide berm

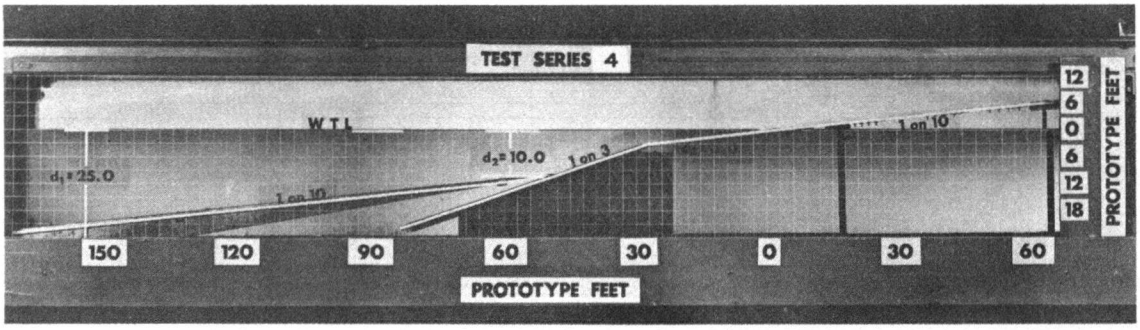

Fig. 9. Composite levee slopes of 1 on 3 and 1 on 10; water depth d_3 = 3.0 ft

water were measured for a composite levee slope of 1 on 3 and 1 on 6, with the change in slope at elevation 0.0 ft wtl, and crown elevations of 1/4 h, 1/2 h, 3/4 h, and h , where h was determined from test-series 4 levee designs. Water depths d_1 and d_2 were 25.0 ft and 10.0 ft, respectively.

1:17-scale model, tests series 8

31. Test series 8 was concerned with the height of wave run-up and quantities of overtopping water for a 1-on-6 levee slope. Water depths d_1 and d_2 were 34.0* ft and 10.0 ft, respectively. These tests were conducted to check similar tests conducted on the 1:30-scale model, i.e., the run-up tests on the 1-on-6 levee slope of test series 1 and the overtopping tests on the 1-on-6 levee slope of test series 5.

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* Corresponding tests were conducted on the 1:30-scale model using depth d_1 of 25.0 ft. This change in depth was necessary because the wave machine in the larger flume was not capable of generating waves of sufficient height in the 25.0-ft water depth.

PART IV: RESULTS OF TESTS

Methods of Presenting Test Data

32. The results of tests conducted in this investigation are presented in tables 1-9 and on plates 3-15. Tables 1-4 and plates 3-8 present wave run-up data obtained on the 1:30-scale model. On plates 3-8 the wave run-up factor (h/H) is plotted against wave steepness (H/L), with the different features of the levee face (various continuous slopes, berm widths, and composite slopes) as parameters. The wave run-up factor is the ratio of the vertical height of run-up above wtl (h) to the height (H) of the test wave. Wave steepness is the ratio of wave height to wave length. The wave heights and lengths were determined at the toe of the beach slope, in water depth d_1, corresponding to the level portion of the wave flume. Tables 5-7 and plates 9-12 present data pertaining to wave overtopping of the levee crown. The wave overtopping factor ($Q_{ot}T/H^2$) is plotted against the relative crown height (y/H) with wave steepness as the parameter. The overtopping factor is dimensionless. Q_{ot}, a volume rate of flow, is the volume of overtopping water per unit length of wave crest, or levee length, divided by the wave period (T). The family of curves on plates 9-12 were determined by interpolation of the plotted data points, and represents approximate "best-fit" curves. Because of the small quantities of water involved, and the possible effect of surface tension for the smaller quantities of overtopping, the quantitative accuracy of the overtopping results decreases as the quantity of overtopping approaches zero.

33. Table 8 and plate 13 show a comparison of the wave run-up data for a 1-on-6 levee slope obtained from the 1:30- and 1:17-scale models. Table 9 shows a comparison of overtopping data for a 1-on-6 levee slope obtained from the 1:30- and 1:17-scale models, and plate 14 shows the overtopping data for a 1-on-6 levee slope obtained on the 1:17-scale model. Plate 15 summarizes all wave run-up data obtained on the 1:30-scale model.

Analysis of Test Data

Test series 1 and 2:
$h/H = f(H/L, \alpha, d_1 \text{ and } d_2)$

34. The first two series of tests were performed to determine the effects of levee slope, water depth at toe of beach, and water depth at toe of levee on wave run-up on the levee. The data obtained (tables 1-2, and plates 3-4) show that levee slope is a very important variable with respect to wave run-up. For all three water depths d_1 (10.0, 17.5, and 25.0 ft), and both water depths d_2 (5.0 and 10.0 ft), the wave run-up factor decreased as cot α increased. Within the range of conditions tested, which corresponds to the range of conditions existing in the prototype, d_1 affected wave run-up only slightly. The effect on wave run-up of decreasing d_2 is illustrated by the comparison of curves shown on plate 5. The decrease in d_2 from 10.0 ft to 5.0 ft resulted in an appreciable decrease in wave run-up, especially for the higher wave-steepness values, because the decrease in water depth at the levee toe caused the steeper, more unstable waves to break at a greater distance from the levee toe. Plate 3 shows that when d_2 is 10.0 ft, values of wave run-up for the steeper levee slopes (cot α = 2 and 3) fell below the straight lines for values of wave steepness less than about 0.045 to 0.050. This variation in run-up data from the "best-fit" curves is not important in this investigation because it occurred for small wave-steepness values; the larger wave-steepness values are more important for hurricane winds over shallow water and short fetches. Also, the largest variation was obtained for a levee slope of 1 on 2, and the steepest practical slope for earth-fill levees is within the range of 1 on 3 to 1 on 3-1/2.

Test series 3:
$h/H = f(H/L, \alpha_1, \text{ and berm width})$

35. The third series of tests was performed to determine the effect of berms of various widths on wave run-up for levee slopes of 1 on 3 and 1 on 6, and various wave-steepness values. Slopes of 1 on 3 and 1 on 6 were selected for these tests.

36. The results of these tests are summarized in table 3 and on

plates 6 and 7. They show that wave run-up decreases as berm width increases. Also, as before, wave run-up is less for the flatter slope and for the higher wave-steepness ratios. The combination of a 1-on-6 slope below and 1-on-3 slope above the berm, and a 70-ft-wide berm (fig. 8) resulted in smaller run-up factors than any other type of section tested at d_1 = 25.0 ft and d_2 = 10.0 ft except the 1-on-10 levee slope.

Test series 4:
$h/H = f(H/L, \alpha_2, \text{ and } d_3)$

37. The results of these tests (table 4 and plate 8) show the effects on wave run-up of different composite levee slopes for water depths d_3 of 0.0 and 3.0 ft. The slope below the break in grade was 1 on 3, and the slopes above the break in grade were 1 on 6 and 1 on 10. The wave run-up factor was decreased by increasing d_3 from 0.0 to 3.0 ft. The effect of the top slope (α_2) on wave run-up was more pronounced than the effect of the change in d_3. The composite slopes did not result in a decrease in wave run-up compared with run-up on the corresponding continuous slopes of 1 on 6 and 1 on 10.

Test series 5:
$Q_{ot} = f(y/H, H/L, \text{ and } \alpha)$

38. The results of these tests, in which continuous levee slopes of 1 on 3 and 1 on 6 were tested with four different crown elevations, are listed in table 5 and presented graphically on plates 9 and 10. The tests showed that the overtopping factor varies appreciably with crown elevation, wave steepness, and levee slope. As in the wave run-up data, there is a considerable scatter of points. This variability is probably inherent in the phenomena of wave run-up and overtopping, and it is believed that an increase in accuracy can be obtained only at the expense of performing a large number of very carefully controlled tests. In the analysis of the overtopping data, and in the application of these data to the selection of the optimum levee slope and crown elevation, it should be remembered that the data were obtained with mechanically generated wave trains of nearly uniform height and constant period. During a hurricane,

wave overtopping will be very erratic with respect to time because the wind-generated wave trains consist of a complex spectrum of wave heights and periods. Thus, for the prototype structures, the volume of overtopping water as a function of time will depend on the variation of wave heights and periods in the wave trains. The use of the so-called significant or one-third-highest wave as a measure of the ability of wave trains to overtop the levee is questionable. Some form of an integration method, which will take into consideration the variation of wave height and period in wave trains, and the variation of overtopping with wave height and wave period as shown by the model test results, should be developed to insure as accurate an analysis of the prototype problem as possible.

Test series 6 and 7:
Q_{ot} = f(y/H, H/L, and slope geometry)

39. Test series 6 and 7 were conducted to determine whether the use of a berm or composite slope would effectively reduce wave overtopping as compared with a continuous levee slope. The results of these tests are presented in tables 6 and 7 and are shown graphically on plates 11 and 12. The 30-ft berm reduced overtopping considerably compared with the continuous 1-on-3 levee slope (compare data on plates 9 and 11). The composite levee slopes (test series 7) of 1 on 3 and 1 on 6 resulted in less overtopping than a continuous slope of 1 on 3, and more overtopping than the continuous 1-on-6 slope. Like all the other types of levee sections tested, both the berm- and composite-slope-type levee faces must be compared on a basis of cost versus protection provided.

Test series 8:
Determination of scale effect

40. Wave run-up. The results of run-up tests on a continuous 1-on-6 levee slope in both the 1:30- and 1:17-scale models are presented in table 8 and on plate 13. The test conditions and procedures used for the 1:30- and 1:17-scale tests were the same except for the depth of water in the level portion of the flumes. The 1-on-6 levee slope was tested on the 1:30-scale model using a maximum d_1 of 25.0 ft, whereas in the 1:17-scale model d_1 was 34.0 ft. This increase was necessary

because the wave machine in flume No. 2 was not capable of generating the larger test waves in the 25.0-ft water depth. The results of previous tests (test series 1 and 2) showed that water depth at the toe of the beach had very little effect on the height of wave run-up. The slight effect of the change in depth in test series 1 and 2 can be attributed to the effect of depth on wave length, which in turn affects wave steepness. The results of the scale-effect tests presented in table 8 show that either the change in depth or the change in scale causes a general increase in wave run-up of about 9 per cent. It is believed that the scale effect is negligible. When the results of these tests are shown graphically using the H/L ratio as a parameter, as on plate 13, the apparent effects of both depth of water and scale of model are obscured by the normal scatter of data points.

41. Overtopping. The results of overtopping tests on the 1-on-6 levee slope in both the 1:30- and 1:17-scale models are shown in table 9. Plate 14 presents the overtopping data for the 1:17-scale model. These data show that larger volumes of overtopping water were obtained in the 1:17-scale model. For quantities greater than about 4 cfs, the volume of overtopping water was about 13 per cent greater in the 1:17-scale model than in the 1:30-scale model. The difference increased as the quantity of overtopping water and the wave-steepness ratio decreased. Considerable scatter of data points was obtained in plotting results from both models (plates 10 and 14). The extremely large differences in results for the small quantities of overtopping can be attributed to the fact that the accuracy of the model results decreased as the quantities of overtopping water approached zero. However, for the larger volumes of overtopping water the differences in test results are appreciable, which indicates that there was some scale effect. Part of the difference may be due to the change in water depth from 25.0 to 34.0 ft. However, it is not possible to separate the two effects because of the amount of scatter in the test data obtained from the limited number of tests conducted.

Discussion of Results

42. The wave run-up data from the 1:30-scale tests are summarized

on the qualitative-type graph of plate 15. Since the scale-effect tests indicated that the wave run-up data are reliable (plate 13), it is believed that the data on plate 15, together with the curves of plates 3-8, are adequate for use in selecting the most economical type levee slope. The overtopping curves on plates 9-12 are not as reliable as the wave run-up data, but are probably sufficiently accurate for use in estimating the quantity of overtopping water for a selected levee type and crown elevation. If more accurate data are desired, additional tests of selected plans should be performed on a larger scale model.

43. The wave run-up data on plate 15 show that for all levee sections tested with a 7.0-sec by 12.0-ft wave (largest test wave used) the maximum run-up resulted, except for the 1-on-2 levee slope with water depths d_1 and d_2 of 25.0 ft and 10.0 ft, and 25 ft and 5.0 ft, respectively. However, the data indicate that the 1-on-6 levee slope will provide a very efficient levee section. The most economical slope will vary slightly from reach to reach along the lake perimeter, depending on the slope and crown elevation of the existing levee and the selected crown elevation of the new levee considered necessary to provide adequate protection. Although the use of berms was effective in reducing wave run-up, their construction in the prototype would require a larger volume of fill, and the simple, continuous-slope levee will probably be found to be more economical.

44. In the application of the test results in the design of an optimum levee system for impounding Lake Okeechobee, allowances should be made for the facts that (a) the prototype levee slope will be or can be made considerably rougher than the smooth slopes used in the model, and it has been shown[4, 10] that roughness and permeability affect wave run-up appreciably, (b) model tests conducted at the University of California laboratory,[12] in which wind-generated waves were used, showed that wave run-up increases with wind velocity, (c) wave trains in nature are much more complex and contain a larger variation in wave heights with respect to time than the wave trains generated in the Waterways Experiment Station flumes, and (d) tests on the 1:17-scale model indicated that the result of overtopping tests on the 1:30-scale model were affected to some

extent by model scale. Therefore, considering the complexities of selecting the maximum design hurricane and the corresponding wind-tide level, together with the problems of analysis and selection listed above, it is apparent that the final design of an optimum levee system will be extremely difficult. However, the results of the wave run-up tests presented in this report are believed to provide information of sufficient accuracy to allow selection of the most economical levee slope for a given crown elevation. The optimum crown elevation cannot be selected directly from the results of model tests presented in this report, but the model data provide information that should be of considerable help in the solution of this problem.

PART V: CONCLUSIONS AND RECOMMENDATIONS

Conclusions

45. It is concluded from the results of this model study and from a review of the available literature on the subject of wave run-up and overtopping that:

a. The wave run-up factor generally is a function of wave steepness, slope of levee, geometry of levee face, water depth at toe of beach slope, water depth at toe of levee, roughness and permeability of levee face, wind speed, and time relative to wave period required for water that runs upslope for a given wave to return downslope.

b. The wave run-up factor for the specific conditions of the Lake Okeechobee levees is a function of wave steepness, levee slope, geometry of levee face, water depth at levee toe, roughness of levee face, and wind speed.

c. Overtopping of the Lake Okeechobee levees is a function of the variables listed in b plus the height of levee crown above wtl.

d. Within the range of conditions tested, which corresponds to the range of conditions in the problem area, the depth of water at the beach toe affects wave run-up only slightly.

e. Within the range of wave steepness tested ($0.03 < H/L < 0.08$) wave run-up increases with decreasing steepness values.

f. A decrease in water depth at the levee toe from 10.0 to 5.0 ft results in a decrease in wave run-up -- the amount of decrease becomes larger as wave steepness is increased.

g. Wave run-up decreases as the berm width is increased.

h. An increase in the water depth from 0.0 to 3.0 ft at the break in grade of composite levee slopes results in a decrease in run-up.

i. Wave run-up decreases as levee slope flattens.

j. Within the range of relative crown elevations with which overtopping occurs, the quantity of overtopping increases as crown elevation and wave steepness decrease.

k. A 1-on-6 levee slope provides considerably more protection against overtopping than a 1-on-3 levee slope.

l. The wave run-up data obtained on the 1:30-scale model were

not affected to a measurable extent by scale effect and are considered adequate for selecting the most efficient levee slope.

m. The wave run-up data are also adequate for determining the crown elevation of the levee assuming small local wind speeds (however, some allowance should probably be made for the fact that model studies conducted at the University of California indicated that wave run-up is a function of local wind speeds).

n. The wave overtopping data obtained on the 1:30-scale model were apparently affected somewhat by the model scale.

Recommendations

46. In order to determine the absolute quantities of overtopping water as an aid in the design of a levee section that will not fail as a result of erosion from overtopping, additional model tests should be conducted at a much larger scale which would eliminate one of the variables in the test results and make it possible to determine scale effect directly.

PART VI: METHOD OF SELECTING LEVEE SECTION DESIGN

47. A method for selecting the optimum designs for the different reaches of levee at Lake Okeechobee, based on the model test data, is outlined briefly in the following subparagraphs:

a. First the design pool elevation at time of hurricane, the design hurricane and resulting wind-tide levels, and the design wave spectrums are selected on the basis of the best available information. Then the most economical levee slopes for different reaches of the lake perimeter are determined based on the relationships between water depth, wave height, wave period, slope of levee, and wave run-up developed from the 1:30-scale model and presented in this report. Selection of the most economical levee slopes may be based in each case on the assumption that the optimum height of levee crown is the minimum elevation that results in no overtopping (e.g., for a 7.0-sec by 12.0-ft design wave and a 1-on-6 levee slope, a crown elevation of h = +7.5 ft wtl would be selected).

b. Careful consideration should be given to the possible difference in height of wave run-up for mechanically generated waves versus wind-generated waves. If it is decided that the effects of wind on wave run-up are sufficient to necessitate adding correction factors to the wave run-up data presented in this report, the selected increases of run-up would be converted to estimated quantities of overtopping water for the levee slopes and crown elevations previously selected as described in subparagraph a. The quantities of overtopping water for the previously selected optimum crown elevations would be estimated from plates 9 and 10 (continuous slopes of 1 on 3 and 1 on 6) applying correction factors to the overtopping data as follows: the volume of overtopping water (Q_{ot}) would be increased from about 10 per cent for the largest value of Q_{ot} obtained on the model to as much as 100 per cent for the small values obtained on the model. If the estimated values of Q_{ot} are appreciable, the possible necessity of raising the previously selected crown elevations should be considered.

c. To aid in the design of a levee section which would not fail as the result of erosion due to overtopping water, additional model tests should be conducted. These tests might be performed in the large wave tank at the Beach Erosion Board laboratory, Washington, D. C., using the largest possible model scale within the limitations of the wave tank and wave generator.

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Table 1

Wave Run-up with Levee Slopes and d_1 Varied

Test Series 1

d_2 = 10.0 ft

Beach slope = 1 on 10

Wave Dimensions		Height of Wave Run-up, ft above wtl				
Period sec	Height ft	1-on-2 Slope	1-on-3 Slope	1-on-4 Slope	1-on-6 Slope	1-on-10 Slope
		d_1 = 25.0 ft				
6.0	12.0	17.7	12.2	9.2	6.0	4.1
7.0	12.0	20.5	15.2	12.0	7.3	5.4
5.5	10.0	18.8	11.7	8.8	5.1	3.1
6.5	10.0	23.2	14.6	11.1	7.0	4.1
5.0	8.0	20.1	11.9	8.5	4.6	2.7
6.0	8.0	22.8	14.3	10.1	6.8	3.5
4.5	6.0	13.4	9.1	7.5	4.2	2.1
5.5	6.0	16.2	12.0	8.9	6.0	2.8
4.5	4.0	8.3	6.3	4.8	3.5	1.6
5.5	4.0	8.5	7.9	6.2	3.9	2.1
		d_1 = 17.5 ft				
5.0	8.0	14.8	11.5	7.9	4.5	2.8
6.0	8.0	21.6	15.2	10.1	6.4	3.6
4.5	6.0	12.4	10.6	7.1	3.7	2.2
5.5	6.0	16.7	14.9	9.3	5.4	2.8
4.5	4.0	8.7	6.5	4.9	3.7	2.2
5.5	4.0	9.5	8.7	6.2	4.1	2.4
		d_1 = 10.0 ft				
4.5	4.0	9.9	6.3	5.2	3.2	1.9
5.5	4.0	9.3	9.1	6.8	4.3	2.3

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Table 2

Wave Run-up with Levee Slopes and d_1 Varied

Test Series 2

d_2 = 5.0 ft

Beach slope = 1 on 10

Wave Dimensions		Height of Wave Run-up, ft above wtl				
Period sec	Height ft	1-on-2 Slope	1-on-3 Slope	1-on-4 Slope	1-on-6 Slope	1-on-10 Slope
		d_1 = 25.0 ft				
6.0	12.0	14.0	10.5	8.1	5.9	4.1
7.0	12.0	12.8	12.5	10.6	7.4	5.4
5.5	10.0	10.6	9.0	6.8	4.8	3.1
6.5	10.0	12.1	10.4	8.6	6.0	4.1
5.0	8.0	8.7	7.9	6.0	4.1	2.7
6.0	8.0	12.6	9.6	7.8	5.4	3.5
4.5	6.0	11.3	7.4	5.2	3.4	2.1
5.5	6.0	13.7	9.3	7.6	4.7	2.8
4.5	4.0	11.0	8.0	5.5	3.5	1.6
5.5	4.0	11.6	9.6	7.0	4.6	2.1
		d_1 = 17.5 ft				
5.0	8.0	11.3	7.6	4.8	4.1	2.8
6.0	8.0	10.7	9.1	7.9	5.3	3.6
4.5	6.0	9.0	6.5	4.7	3.5	2.2
5.5	6.0	11.4	8.3	6.5	4.2	2.8
4.5	4.0	11.7	8.2	5.9	3.6	2.2
5.5	4.0	11.9	9.8	7.1	4.0	2.4
		d_1 = 10.0 ft				
4.5	4.0	9.7	6.7	4.8	3.0	1.9
5.5	4.0	13.3	9.3	6.6	3.9	2.3

Table 3

Wave Run-up on Berms of Various Widths and Levee Slopes of 1 on 3 and 1 on 6

Test Series 3

d_1 = 25.0 ft, d_2 = 10.0 ft Berm slope = 1 on 20

Wave Dimensions		Height of Wave Run-up, ft above wtl		
Period sec	Height ft	Berm Width 30 ft	Berm Width 50 ft	Berm Width 70 ft
		Cot α_1 = 3; Cot α_2 = 3		
6.0	12.0	11.0	8.4	7.1
7.0	12.0	15.6	10.6	9.0
5.5	10.0	8.9	7.5	6.2
6.5	10.0	12.1	9.2	8.1
5.0	8.0	6.3	5.8	5.4
6.0	8.0	8.1	6.9	6.0
4.5	6.0	4.7	4.1	4.3
5.5	6.0	6.8	5.3	5.0
		Cot α_1 = 6; Cot α_2 = 3		
6.0	12.0	7.4	5.7	4.6
7.0	12.0	9.8	7.5	6.1
5.5	10.0	5.4	4.7	3.6
6.5	10.0	8.2	6.3	5.2
5.0	8.0	5.2	4.3	3.6
6.0	8.0	7.1	5.6	4.7
4.5	6.0	3.9	3.4	2.9
5.5	6.0	5.7	4.3	3.6
4.5	4.0	2.3	No tests	
5.5	4.0	3.3		

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Table 4

Wave Run-up on Composite Levee Slopes with

d_3 = 0.0 and 3.0 ft

Test Series 4

d_1 = 25.0 ft, d_2 = 10.0 ft

Wave Dimensions		Height of Wave Run-up, ft above wtl	
Period sec	Height ft	1-on-3 and 1-on-6 Levee Slopes	1-on-3 and 1-on-10 Levee Slopes
		d_3 = 0.0 ft	
6.0	12.0	9.9	7.5
7.0	12.0	12.1	8.9
5.5	10.0	9.2	6.7
6.5	10.0	10.5	7.7
5.0	8.0	8.3	6.2
6.0	8.0	9.3	6.9
4.5	6.0	6.0	4.8
5.5	6.0	7.4	5.6
		d_3 = 3.0 ft	
6.0	12.0	8.6	6.3
7.0	12.0	10.3	7.3
5.5	10.0	7.4	5.3
6.5	10.0	8.6	6.2
5.0	8.0	6.0	4.5
6.0	8.0	7.4	5.2
4.5	6.0	4.4	3.4
5.5	6.0	6.0	4.1

H = H1/3 at 1 ft 13-14

1:30 P.32 TM64

Table 5

Wave Overtopping of Levees with 1-on-3 and 1-on-6 Slope

Test Series 5

d_1 = 25.0 ft, d_2 = 10.0 ft

Wave Dimensions		Q_{ot} per ft of Wave Crest, cfs, for Crown El ft above wtl			
Period sec	Height ft	1/4 h*	1/2 h*	3/4 h*	h*
		Levee Slope = 1 on 3			
6.0	12.0	11.10	5.40	1.95	0.90
7.0	12.0	10.80	6.43	2.32	0.51
5.5	10.0	7.21	4.42	1.31	0.65
6.5	10.0	8.88	5.00	1.53	0.14
5.0	8.0	5.94	2.52	1.08	0.18
6.0	8.0	6.60	3.45	1.35	0.45
4.5	6.0	4.20	1.80	0.40	0.00
5.5	6.0	4.58	1.80	0.33	0.00
4.5	4.0	2.60	1.60	0.20	0.00
5.5	4.0	2.62	1.15	0.16	0.00
		Levee Slope = 1 on 6			
6.0	12.0	8.10	3.90	1.50	0.60
7.0	12.0	10.69	5.27	1.54	0.51
5.5	10.0	6.71	4.09	1.47	0.49
6.5	10.0	7.20	3.05	0.83	0.28
5.0	8.0	4.68	2.16	0.72	0.00
6.0	8.0	5.85	2.25	0.30	0.00
4.5	6.0	2.60	1.40	0.40	0.00
5.5	6.0	4.25	1.64	0.16	0.00
4.5	4.0	1.80	0.80	0.20	0.00
5.5	4.0	2.13	0.98	0.16	0.00

* h determined from test series 1.

Table 6

Wave Overtopping with 30-ft Berm and 1-on-3 Levee Slope

Test Series 6

d_1 = 25.0 ft, d_2 = 10.0 ft Berm slope = 1 on 20

	Wave Dimensions		Q_{ot} per ft of Wave Crest, cfs, for Crown El ft above wtl				
	Period sec	Height ft	1.5	1/4 h*	1/2 h*	3/4 h*	h*
.01035	6.0	12.0	12.90	8.40	3.75	0.60	0.00
.0076	7.0	12.0	14.80	7.98	3.22	0.77	0.13
.0103	5.5	10.0	10.81	9.01	3.11	0.49	0.33
.00735	6.5	10.0	11.92	7.21	2.77	0.83	0.28
.00994	5.0	8.0	8.10	8.28	2.52	0.36	0.00
.0069	6.0	8.0	9.30	7.80	3.15	1.35	0.30
.0092	4.5	6.0	4.20	No test	1.80	0.40	0.00
.00616	5.5	6.0	5.74	5.08	1.15	0.16	0.00
.0013	4.5	4.0	0.40	No test	No test	0.20	0.00
.0041	5.5	4.0	1.15	No test	0.98	0.16	0.00

* h determined from test series 3.

Table 7

Wave Overtopping with Composite Levee Slope of 1 on 3 and 1 on 6 and d_3 = 0.0 ft

Test Series 7

d_1 = 25.0 ft, d_2 = 10.0 ft

Wave Dimensions		Q_{ot} per ft of Wave Crest, cfs, for Crown El ft above wtl			
Period sec	Height ft	1/4 h*	1/2 h*	3/4 h*	h*
6.0	12.0	10.20	4.50	0.60	0.00
7.0	12.0	12.21	5.66	0.65	0.00
5.5	10.0	8.18	4.59	0.66	0.00
6.5	10.0	9.56	4.99	0.69	0.00
5.0	8.0	6.48	3.22	0.54	0.00
6.0	8.0	7.20	3.90	0.45	0.00
4.5	6.0	4.60	3.00	0.60	0.00
5.5	6.0	5.07	1.97	0.16	0.00
4.5	4.0	No test	2.60	1.40	0.40
5.5	4.0	3.61	1.97	0.66	0.00

* h determined from test series 4.

H = $H_{1/3}$ at d_1 (pp. 12-14)

Table 8

Comparison of Heights of Wave Run-up in 1:30- and 1:17-scale Models

Test Series 8

1-on-6 Levee Slope
d_2 = 10.0 ft

d_1 = 25.0 ft, 1:30-scale model
34.0 ft, 1:17-scale model

Wave Dimensions		Height of Wave Run-up, ft above wtl		
Period sec	Height ft	1:30 Scale	1:17 Scale TR-4	Difference %*
6.0	12.0	6.0	6.7	+12
7.0	12.0	7.3	7.8	+7
5.5	10.0	5.1	5.5	+8
6.5	10.0	7.0	7.2	+3
5.0	8.0	4.6	5.2	+13
6.0	8.0	6.8	6.9	+1
4.5	6.0	4.2	4.5	+7
5.5	6.0	6.0	6.3	+5
4.5	4.0	3.5	4.1	+17
5.5	4.0	3.9	4.7	+20

* Percentage of difference in height of wave run-up is based on data obtained from tests conducted on the 1:30-scale model.

Table 9

Comparison of Volume of Wave Overtopping in 1:30- and 1:17-scale Models

Test Series 8

1-on-6 Levee Slope
d_2 = 10.0 ft

d_1 = 25.0 ft, 1:30-scale model
34.0 ft, 1:17-scale model

$\frac{H}{gT^2}$ (not H_o)	Wave Dimensions		Q_{ot} per ft of Wave Crest, cfs, for Crown El ft above wtl											
			1/4 h			1/2 h			3/4 h			h		
	Period sec	Height ft	1:30 Scale	1:17 Scale	Difference %*	1:30 Scale	1:17 Scale	Difference %*	1:30 Scale	1:17 Scale	Difference %*	1:30 Scale	1:17 Scale	Difference %*
.01035	6.0	12.0	8.10	8.81	+9	3.90	4.61	+18	1.50	2.04	+36	0.60	0.94	+57
.0076	7.0	12.0	10.69	11.09	+4	5.27	5.86	+11	1.54	2.20	+43	0.51	0.89	+75
.01027	5.5	10.0	6.71	7.72	+15	4.09	4.46	+9	1.47	1.76	+20	0.49	0.62	+27
.00735	6.5	10.0	7.20	8.84	+23	3.05	4.23	+39	0.83	1.48	+78	0.28	0.39	+39
.00994	5.0	8.0	4.68	5.60	+20	2.16	3.38	+57	0.72	1.79	+149	0.00	0.30	
.0069	6.0	8.0	5.85	6.45	+10	2.25	2.86	+27	0.30	0.96	+220	0.00	0.08	
.0092	4.5	6.0	2.20	2.99	+36	1.40	2.65	+89	0.40	0.94	+135	0.00	0.05	
.00616	5.5	6.0	4.25	4.74	+12	1.64	2.57	+57	0.16	0.78	+387	0.00	0.04	
.00613	4.5	4.0	1.80	2.38	+33	0.80	1.70	+112	0.20	0.64	+220	0.00	0.04	
.0041	5.5	4.0	2.13	3.81	+79	0.98	2.41	+146	0.16	1.32	+725	0.00	0.33	

* Percentage of difference in volumes of overtopping water is based on data obtained from tests conducted on the 1:30-scale model.

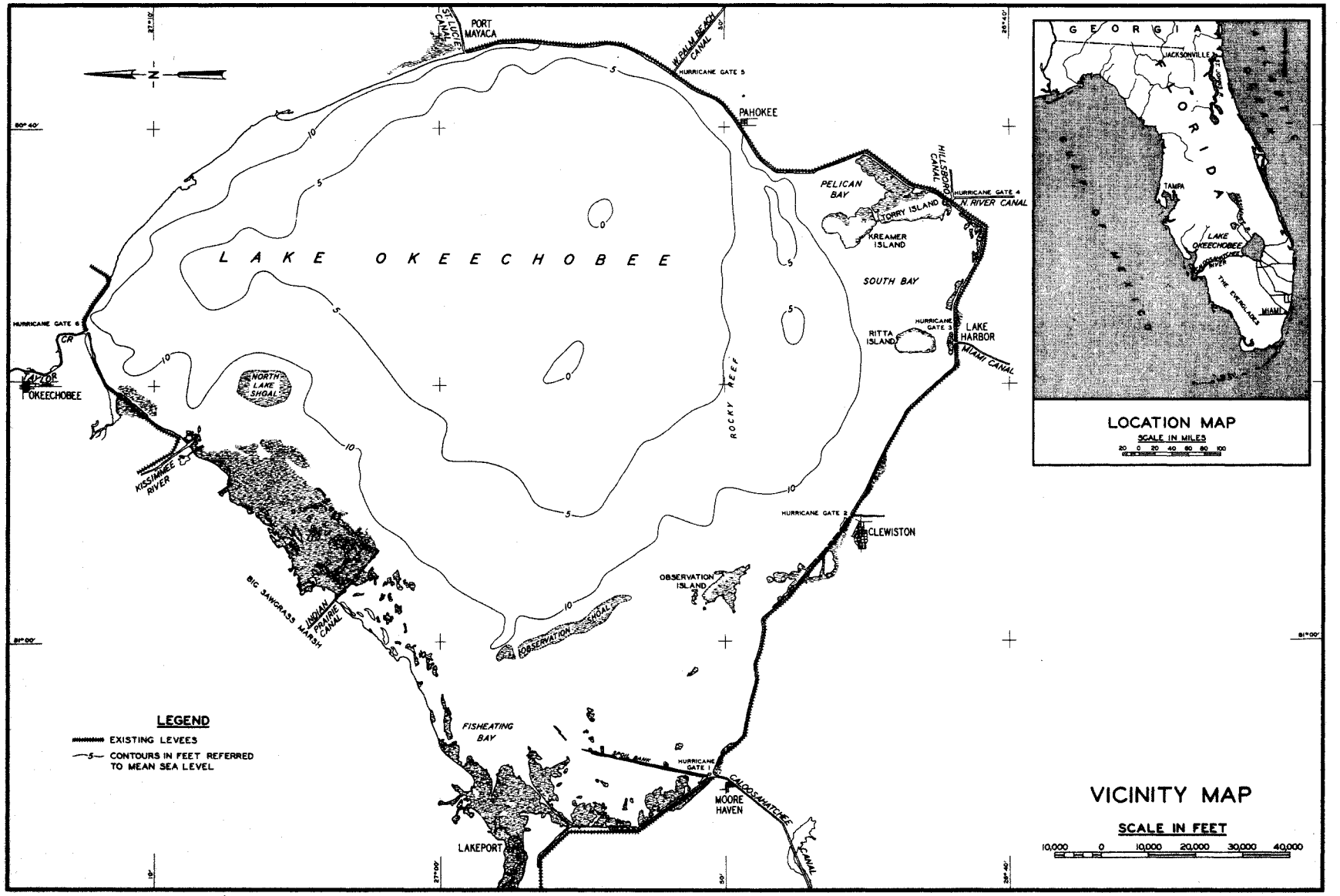
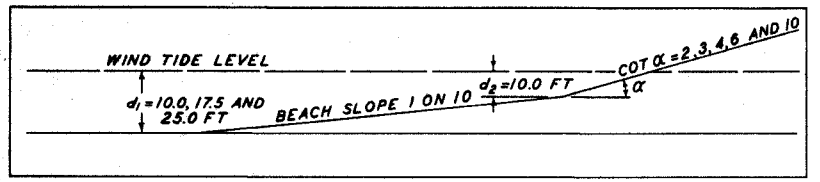
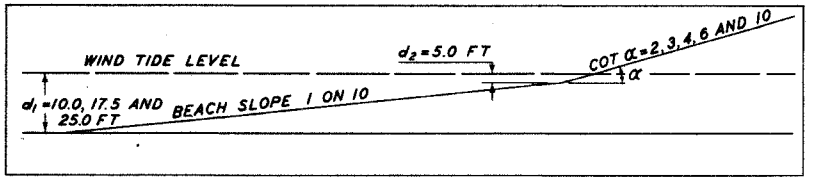

LAKE OKEECHOBEE
PORT MAYACA
ST. LUCIE CANAL
W. PALM BEACH CANAL
HURRICANE GATE 5
PAHOKEE
HILLSBORO CANAL
HURRICANE GATE 4
N. RIVER CANAL
PELICAN BAY
TORRY ISLAND
KREAMER ISLAND
SOUTH BAY
HURRICANE GATE 3
LAKE HARBOR
RITTA ISLAND
MIAMI CANAL
ROCKY REEF
HURRICANE GATE 6
TAYLOR CR
OKEECHOBEE
NORTH LAKE SHOAL
KISSIMMEE RIVER
HURRICANE GATE 2
CLEWISTON
OBSERVATION ISLAND
OBSERVATION SHOAL
BIG SAWGRASS MARSH
INDIAN PRAIRIE CANAL
FISHEATING BAY
HURRICANE GATE 1
MOORE HAVEN
CALOOSAHATCHEE CANAL
LAKEPORT
LEGEND
EXISTING LEVEES
5 CONTOURS IN FEET REFERRED TO MEAN SEA LEVEL
VICINITY MAP
SCALE IN FEET
10,000 0 10,000 20,000 30,000 40,000
GEORGIA
JACKSONVILLE
FLORIDA
TAMPA
LAKE OKEECHOBEE
THE EVERGLADES
MIAMI
LOCATION MAP
SCALE IN MILES
20 0 20 40 60 80 100

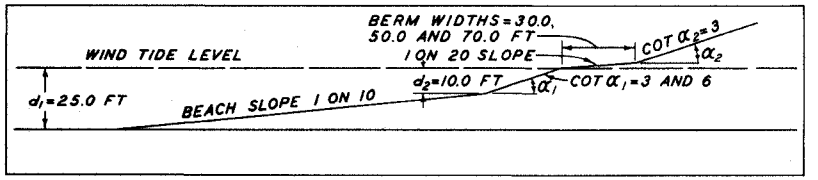
PLATE 1



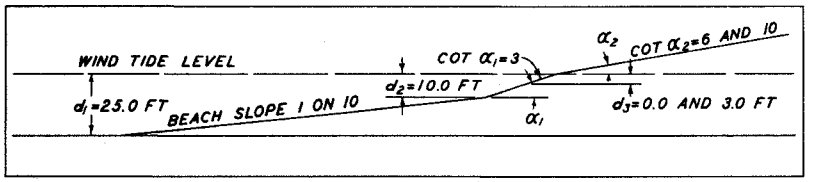
TEST SERIES 1



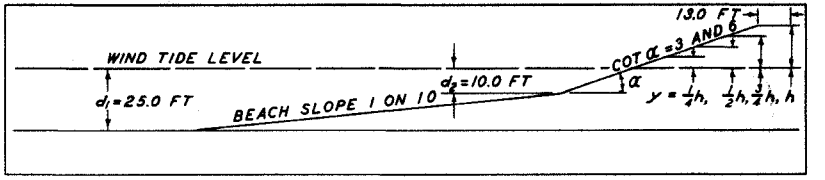
TEST SERIES 2



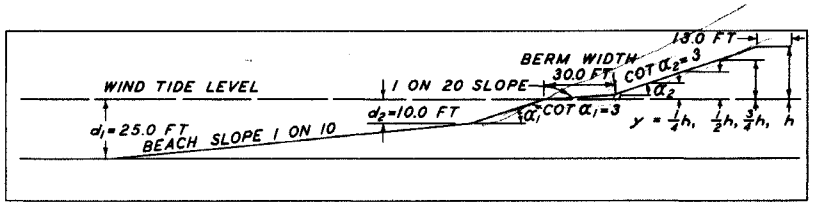
TEST SERIES 3



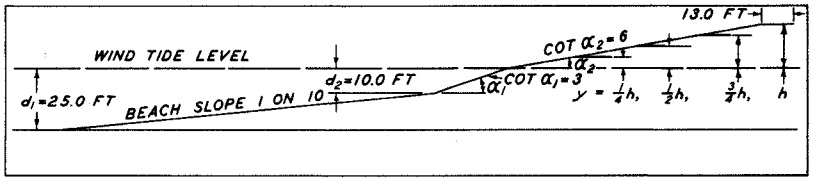
TEST SERIES 4



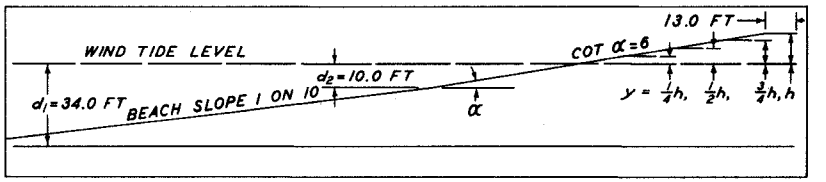
TEST SERIES 5



TEST SERIES 6



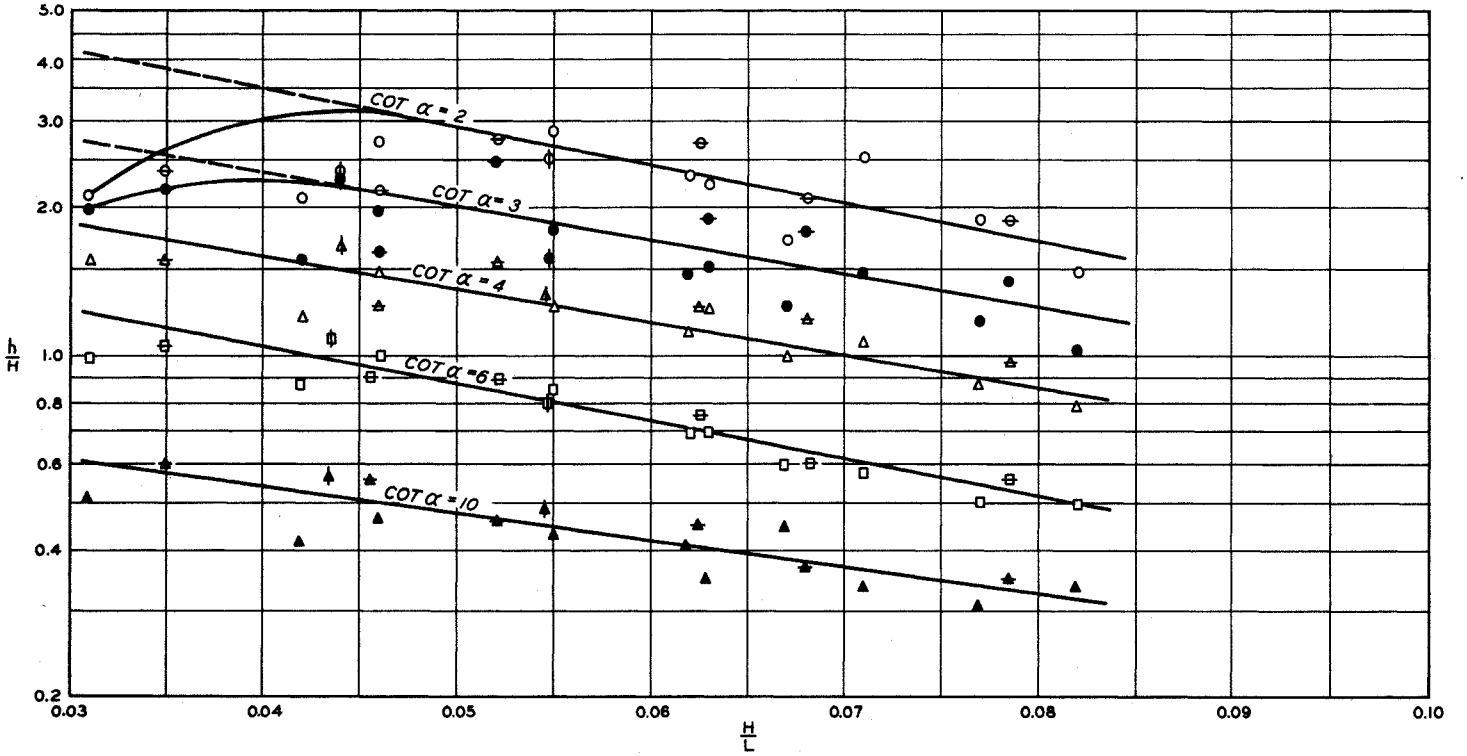
TEST SERIES 7



TEST SERIES 8

d_1=WATER DEPTH AT TOE OF BEACH SLOPE (FT).
d_2=WATER DEPTH AT TOE OF LEVEE SLOPE (FT).
d_3=WATER DEPTH AT CHANGE IN SLOPE OF COMPOSITE LEVEE SLOPES (FT).
h =HEIGHT OF WAVE RUN-UP ON LEVEE FACE (FT ABOVE WTL).
y =HEIGHT OF LEVEE CROWN (FT ABOVE WTL).
TEST SERIES 1 TO 7-MODEL SCALE=1:30.
TEST SERIES 8-MODEL SCALE=1:17.

ELEMENTS OF TEST SECTIONS



SYMBOL	COT α	d_1 (FT)
○	2	25.0
⊖	2	17.5
ϕ	2	10.0
●	3	25.0
⊖	3	17.5
ϕ	3	10.0
△	4	25.0
☆	4	17.5
▲	4	10.0
□	6	25.0
⊖	6	17.5
ϕ	6	10.0
▲	10	25.0
☆	10	17.5
▲	10	10.0

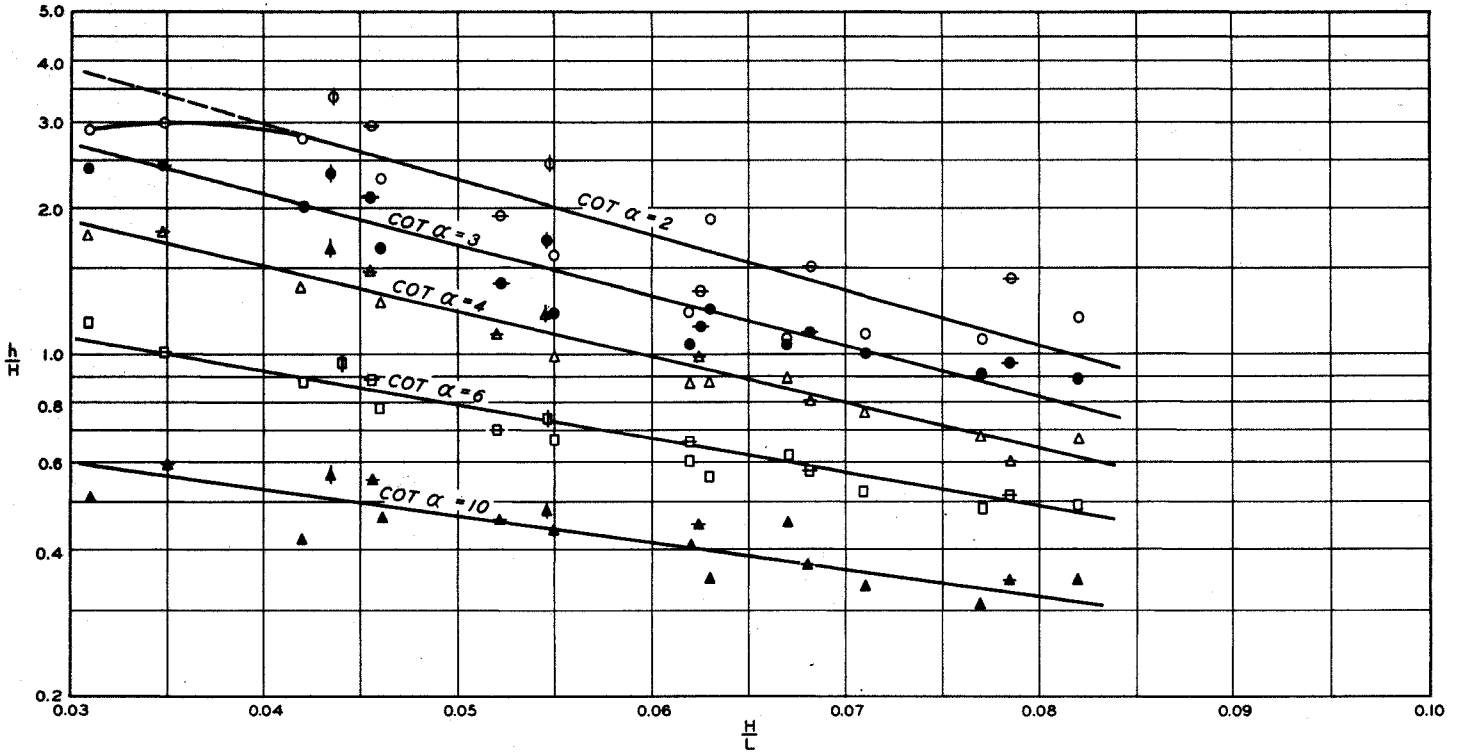
TEST SERIES I

RUN-UP AS A FUNCTION OF $\frac{H}{L}$, α, AND d_1

d_2 = 10.0 FT

PLATE 3

PLATE 4

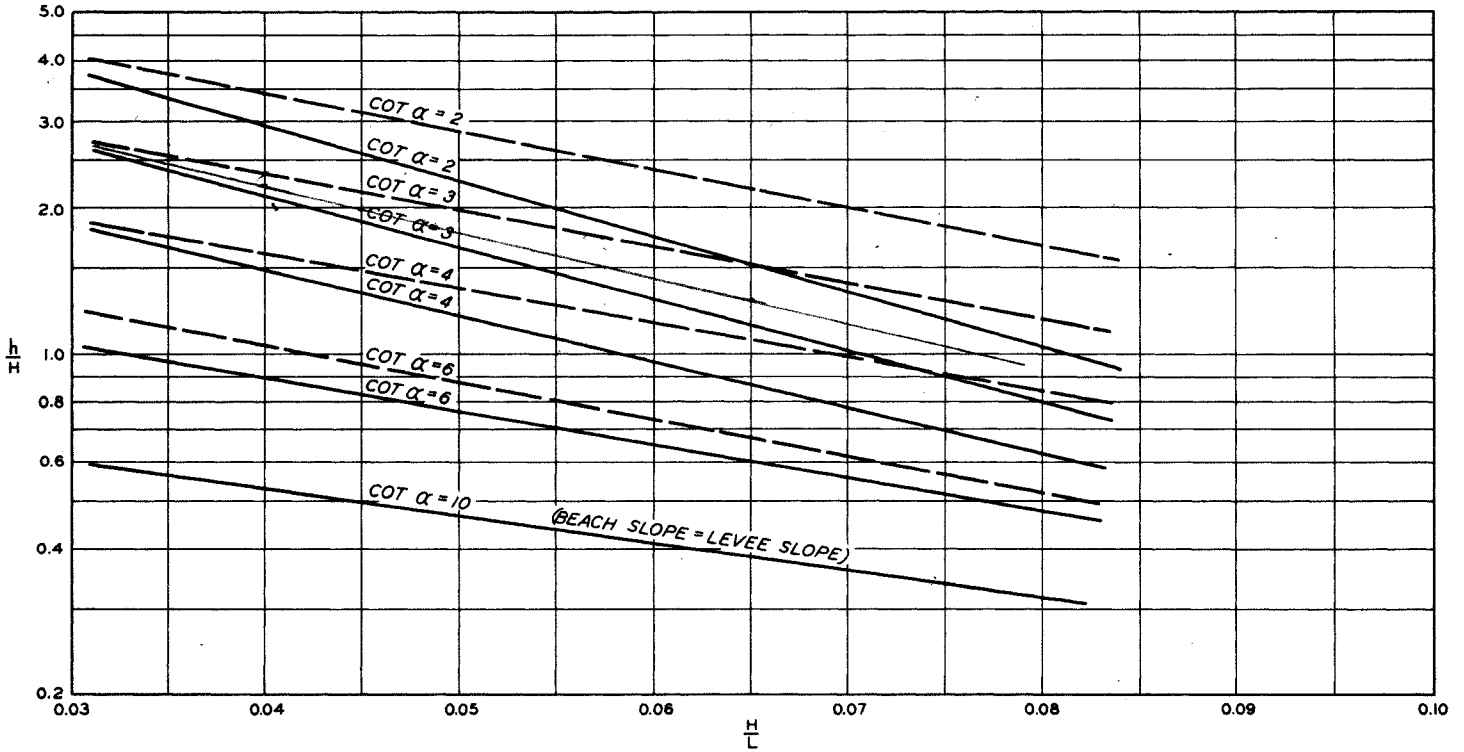


SYMBOL	COT α	d_1 (FT)
○	2	25.0
⊖	2	17.5
⊕	2	10.0
●	3	25.0
⊖	3	17.5
⊕	3	10.0
△	4	25.0
☆	4	17.5
▲	4	10.0
□	6	25.0
⊟	6	17.5
⊞	6	10.0
▲	10	25.0
★	10	17.5
▲	10	10.0

TEST SERIES 2

RUN-UP AS A FUNCTION OF $\frac{H}{L}$, α, AND d_1

d_2 = 5.0 FT



LEGEND

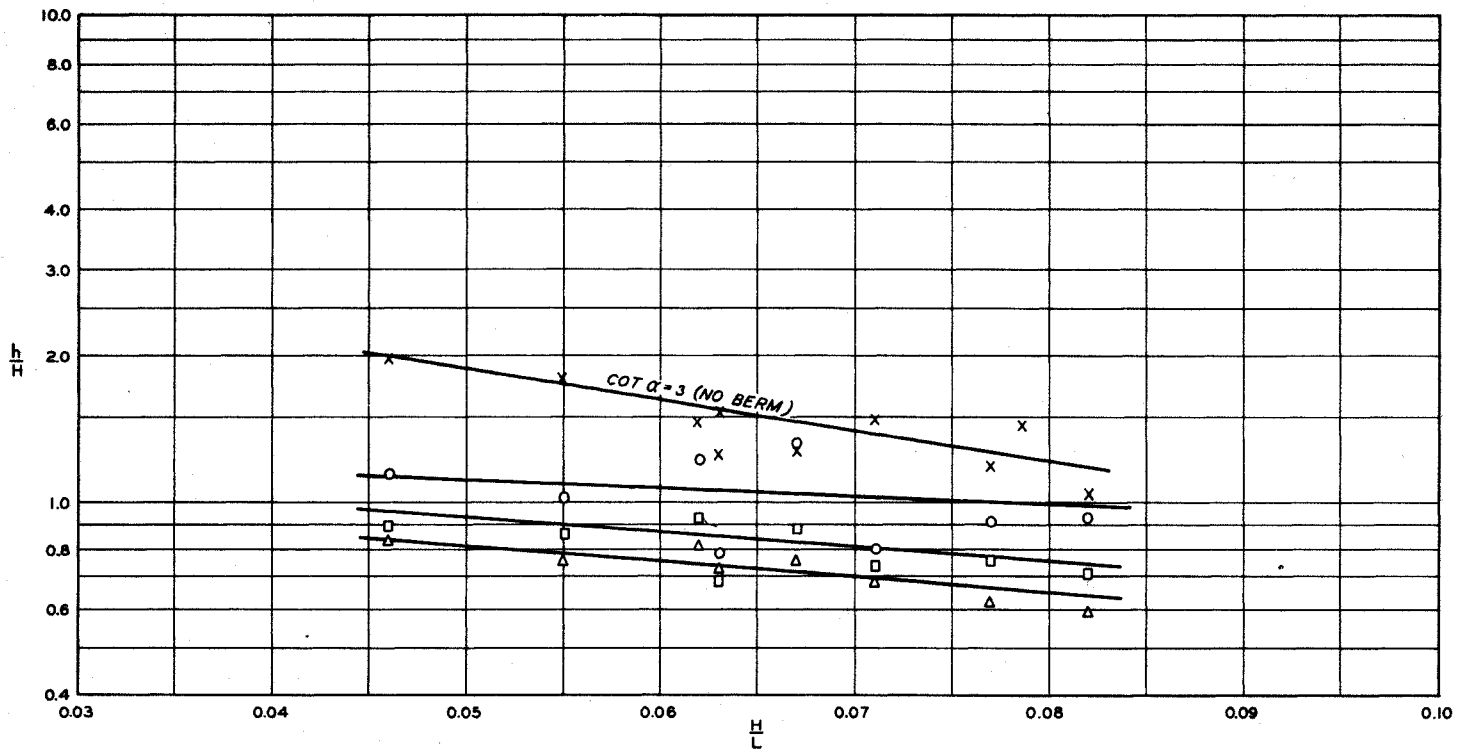
——— AVERAGE CURVE FROM TEST SERIES 2 DATA. $d_2 = 5.0$ FT

— — — AVERAGE CURVE FROM TEST SERIES 1 DATA. $d_2 = 10.0$ FT

COMPARISON OF RESULTS
TEST SERIES 1 AND 2

RUN-UP AS A FUNCTION OF $\frac{H}{L}$, α, AND d_2

PLATE 5

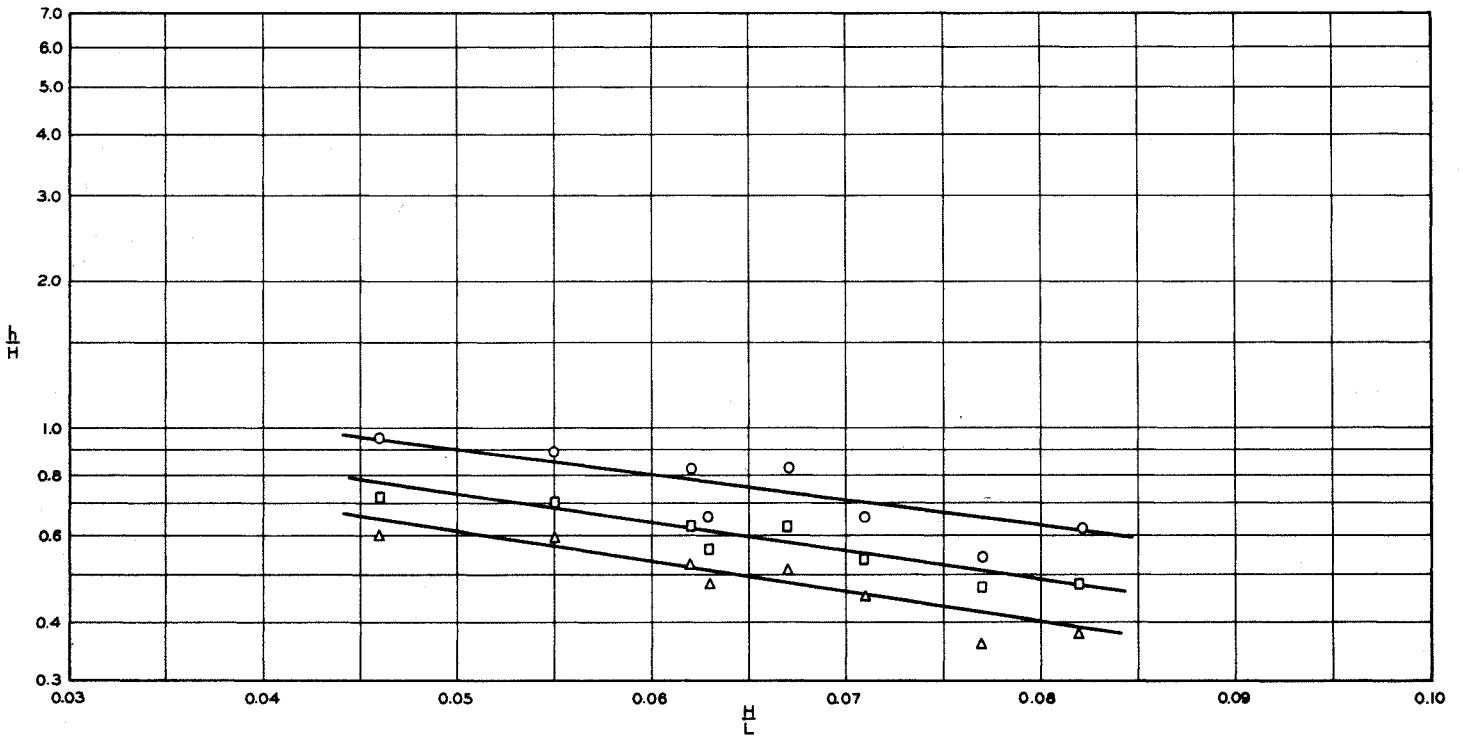


SYMBOL	BERM WIDTH (FT)
X	0.0
O	30.0
□	50.0
Δ	70.0

TEST SERIES 3

RUN-UP AS A FUNCTION OF $\frac{H}{L}$ AND BERM WIDTH

LEVEE SLOPE BELOW BERM = 1 ON 3
LEVEE SLOPE ABOVE BERM = 1 ON 3
d_1 = 25.0 FT d_2 = 10.0 FT



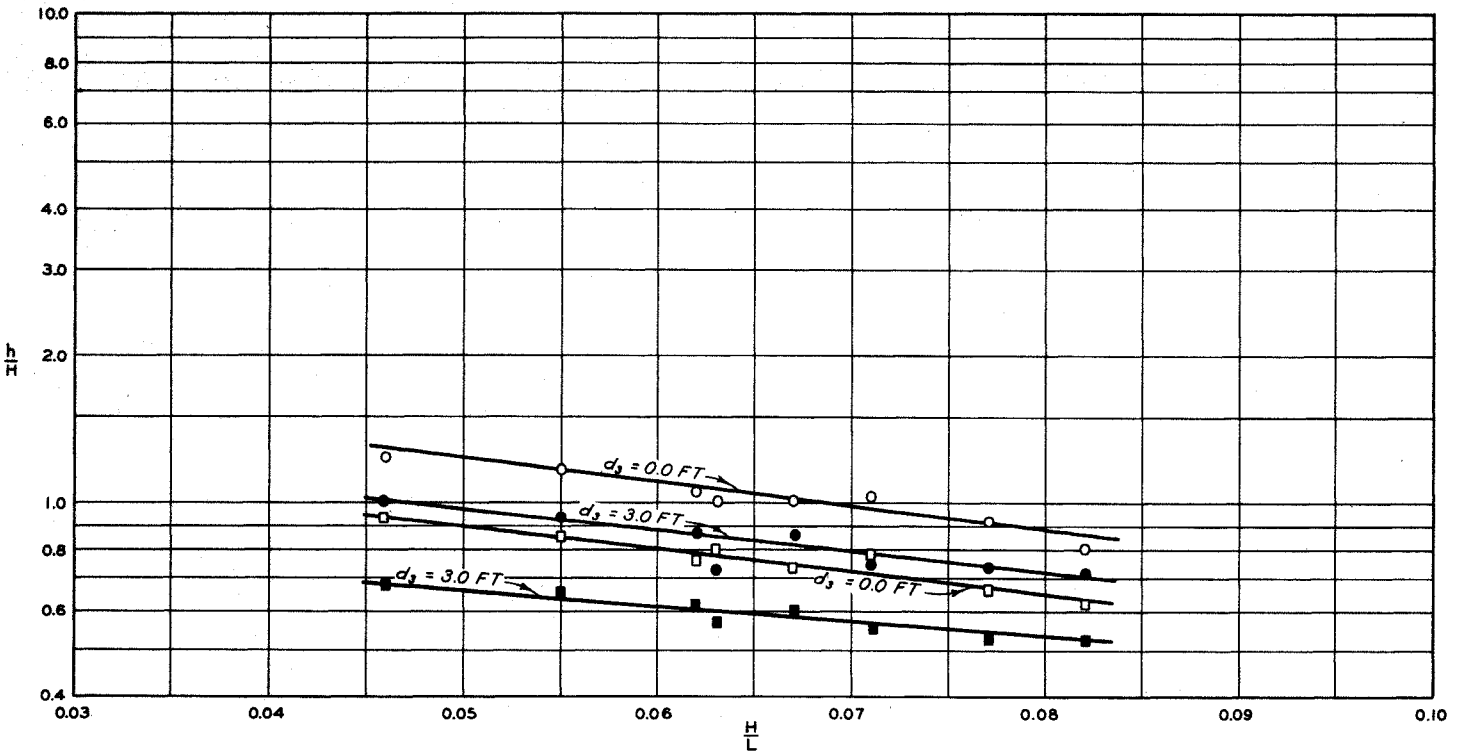
SYMBOL	BERM WIDTH (FT)
○	30.0
□	50.0
△	70.0

TEST SERIES 3

RUN-UP AS A FUNCTION OF $\frac{H}{L}$ AND BERM WIDTH

LEVEE SLOPE BELOW BERM = 1 ON 6
LEVEE SLOPE ABOVE BERM = 1 ON 3
d_1 = 25.0 FT d_2 = 10.0 FT

PLATE 8



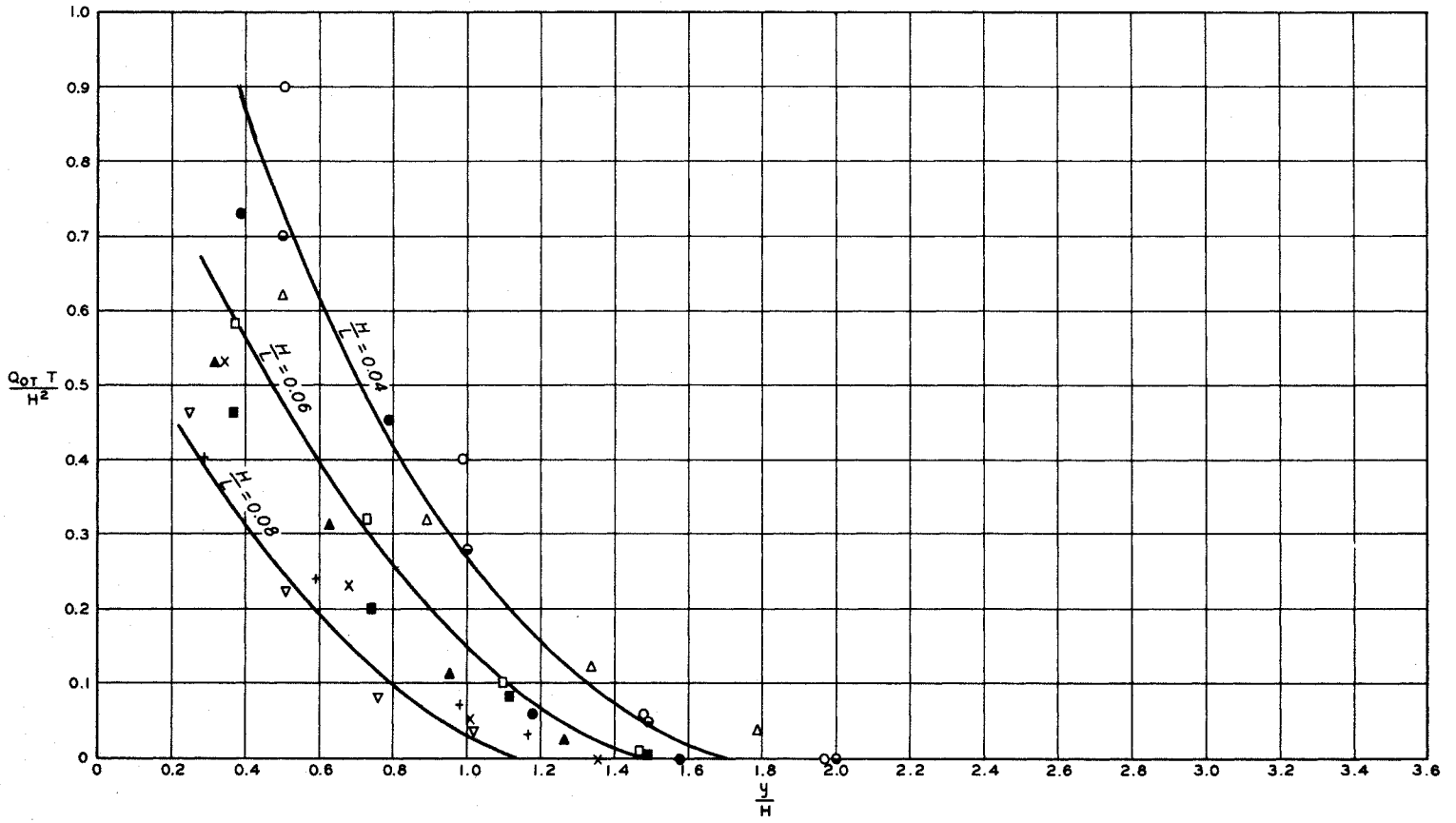
SYMBOL	COT α_1	COT α_2	d_3 (FT)
○	3	6	0.0
●	3	6	3.0
□	3	10	0.0
■	3	10	3.0

TEST SERIES 4

RUN-UP AS A FUNCTION OF $\frac{H}{L}$, α_2, AND d_3

d_1 = 25.0 FT d_2 = 10.0 FT

COT α_1 = 3



SYMBOL	$\frac{H}{L}$
○	0.031
●	0.042
⊖	0.046
△	0.055
□	0.062
×	0.063
▲	0.067
■	0.071
+	0.077
▽	0.082

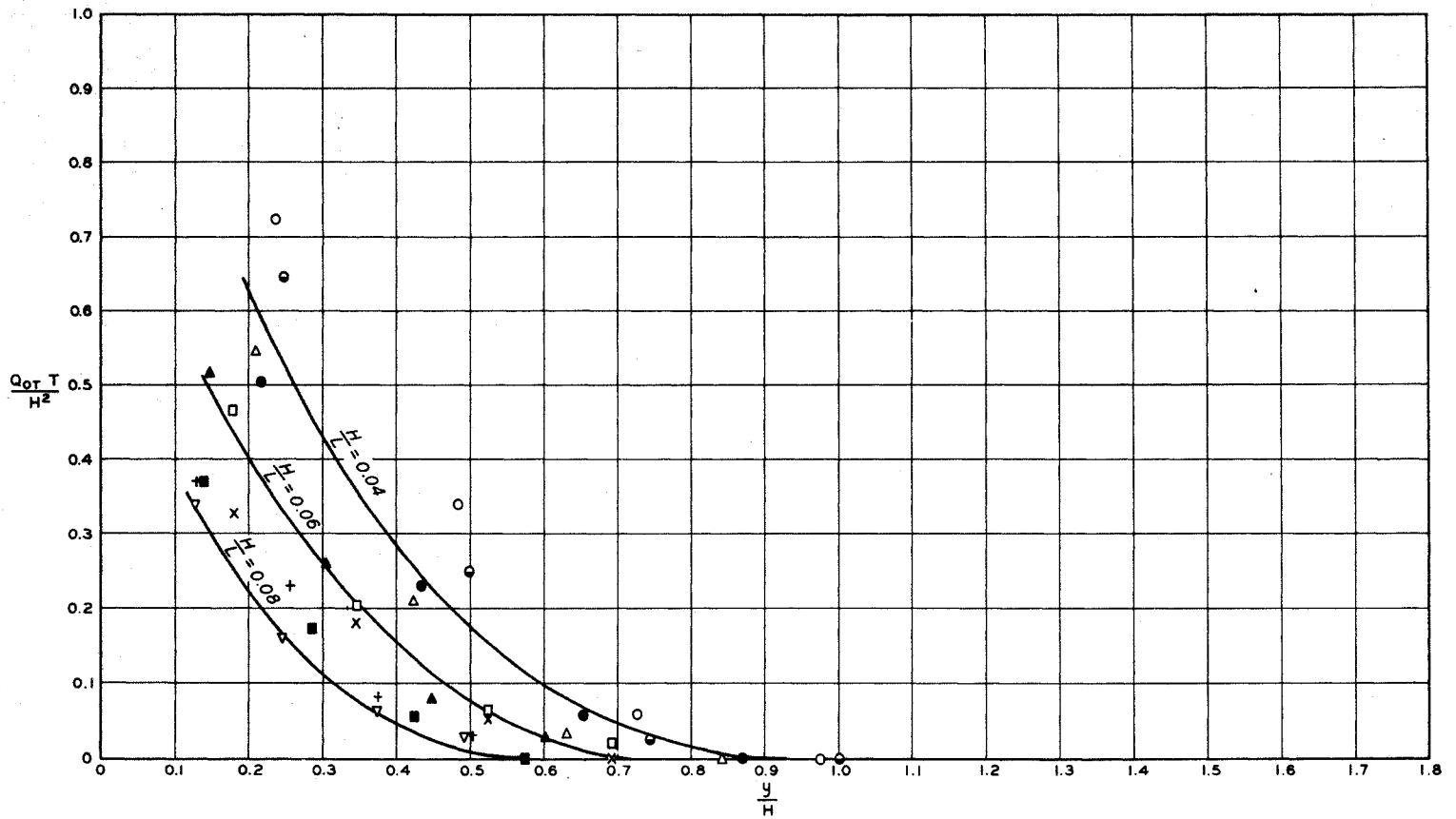
TEST SERIES 5

OVERTOPPING AS A FUNCTION OF $\frac{y}{H}$ AND $\frac{H}{L}$

LEVEE SLOPE = 1 ON 3

d_1 = 25.0 FT d_2 = 10.0 FT

PLATE 9



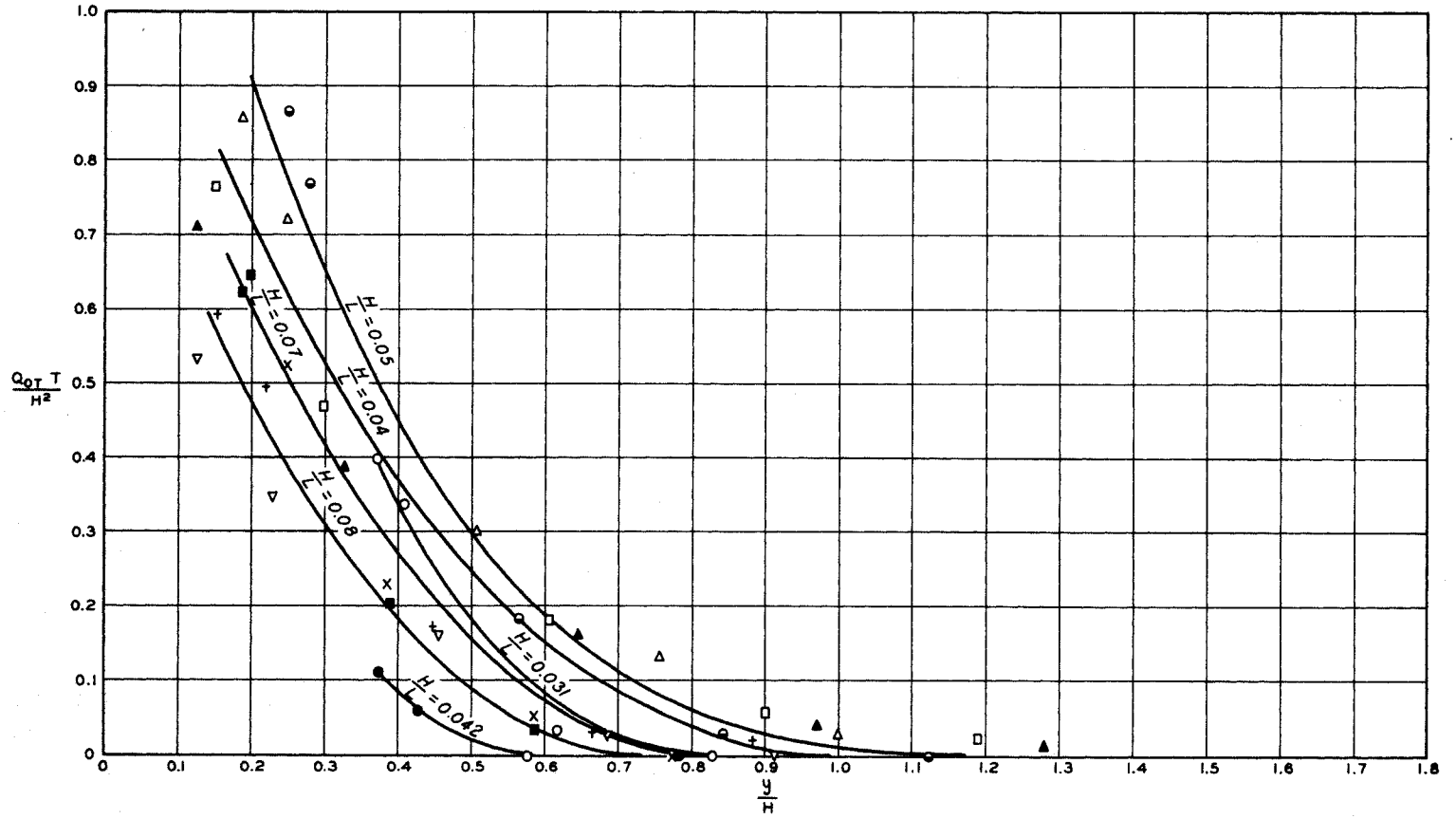
SYMBOL	$\frac{H}{L}$
○	0.031
●	0.042
⊖	0.046
△	0.055
□	0.062
×	0.063
▲	0.067
■	0.071
+	0.077
▽	0.082

TEST SERIES 5

OVERTOPPING AS A FUNCTION OF $\frac{y}{H}$ AND $\frac{H}{L}$

LEVEE SLOPE = 1 ON 6

d_1 = 25.0 FT d_2 = 10.0 FT

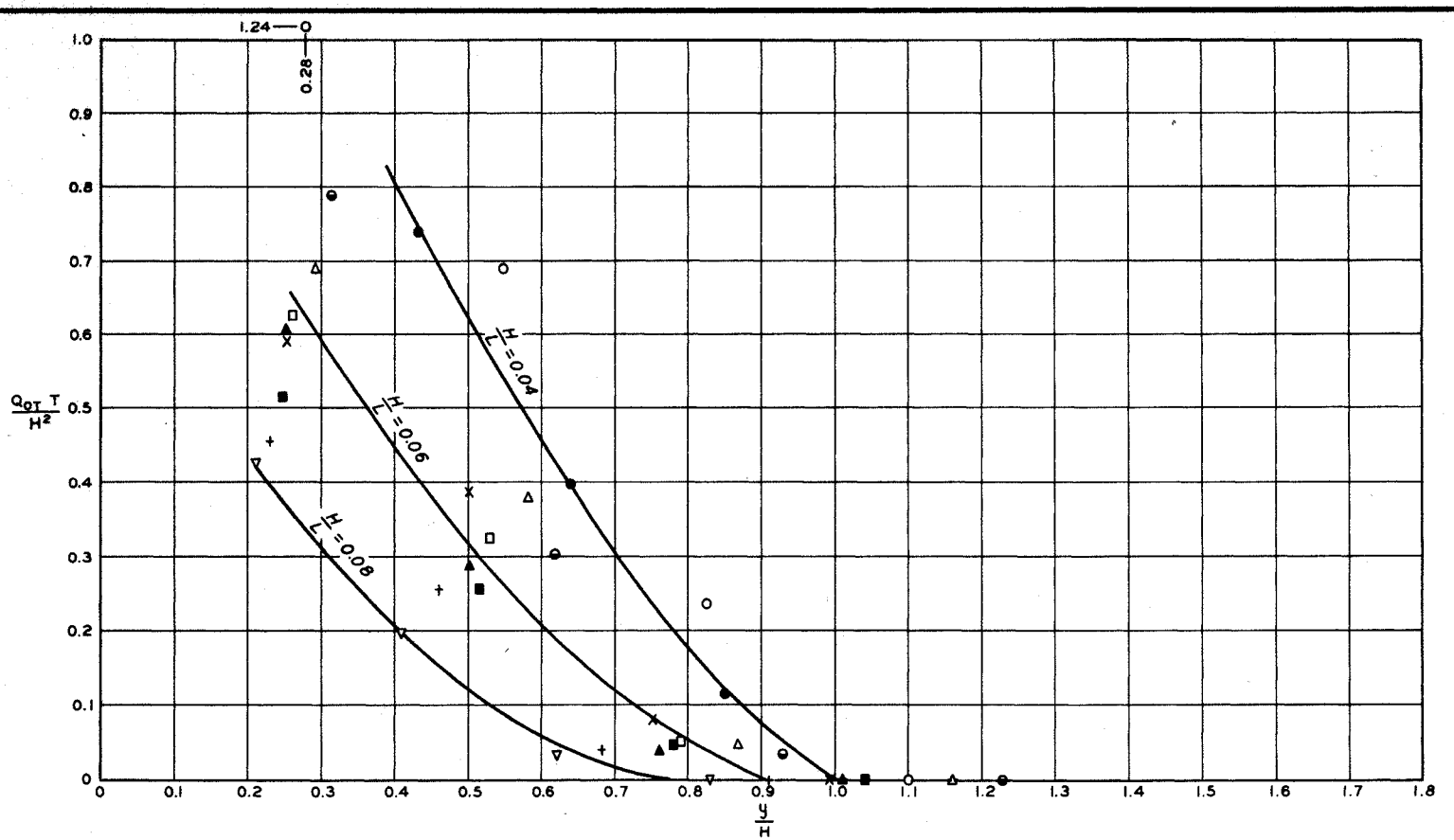


SYMBOL	$\frac{H}{L}$
○	0.031
●	0.042
⊖	0.046
△	0.055
□	0.062
×	0.063
▲	0.067
■	0.071
+	0.077
▽	0.082

TEST SERIES 6

OVERTOPPING AS A FUNCTION OF $\frac{y}{H}$ AND $\frac{H}{L}$

LEVEE SLOPE BELOW BERM = 1 ON 3
LEVEE SLOPE ABOVE BERM = 1 ON 3
WIDTH OF BERM = 30.0 FT
d_1 = 25.0 FT d_2 = 10.0 FT



SYMBOL	$\frac{H}{L}$
○	0.031
●	0.042
⊖	0.046
△	0.055
□	0.062
×	0.063
▲	0.067
■	0.071
+	0.077
▽	0.082

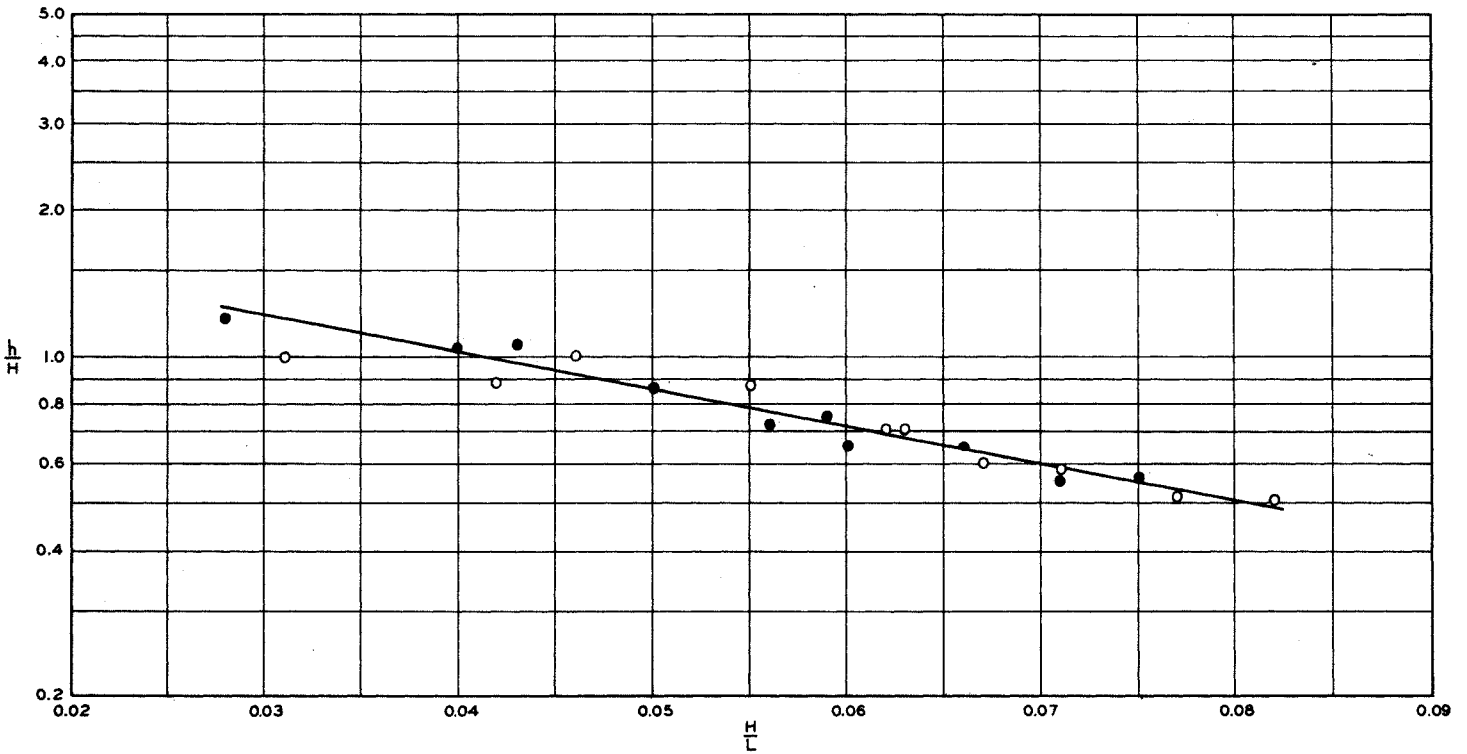
TEST SERIES 7

OVERTOPPING AS A FUNCTION OF $\frac{y}{H}$ AND $\frac{H}{L}$

COT $\alpha_1 = 3$, COT $\alpha_2 = 6$

AND d_3 = 0.0 FT (COMPOSITE LEVEE SLOPE)

d_1 = 25.0 FT d_2 = 10.0 FT



LEGEND

● MODEL SCALE = 1:17. WATER DEPTH AT TOE OF BEACH SLOPE (d_1) = 34.0 FT.

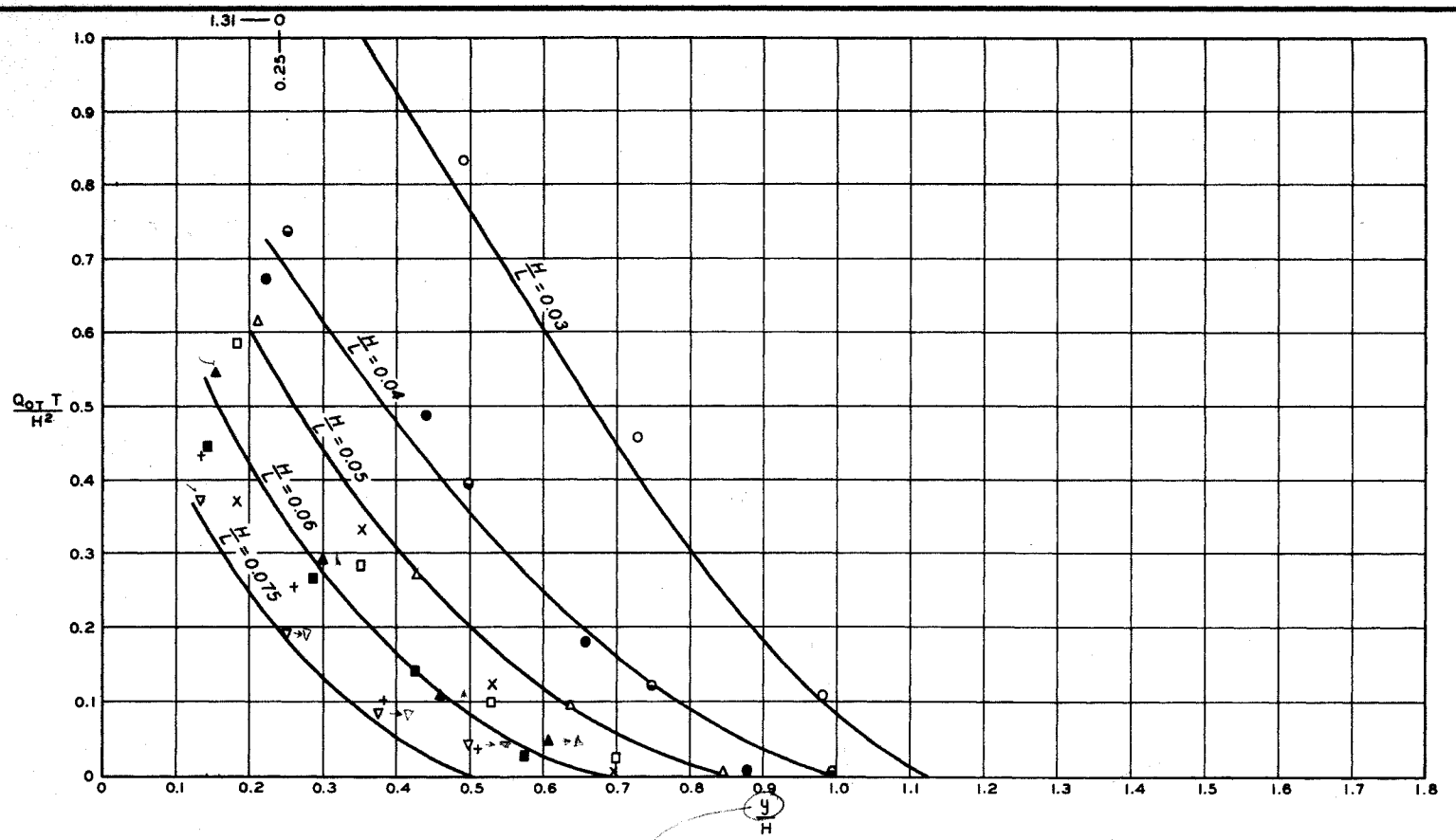
○ MODEL SCALE = 1:30. WATER DEPTH AT TOE OF BEACH SLOPE (d_1) = 25.0 FT.

TEST SERIES 8

RUN-UP AS A FUNCTION OF $\frac{H}{L}$ AND MODEL SCALE

LEVEE SLOPE = 1 ON 6

d_1 = 25.0 AND 34.0 FT d_2 = 10.0 FT



SYMBOL	$\frac{H}{L}$
○	0.028
●	0.040
⊖	0.043
△	0.050
□	0.056
×	0.059
▲	0.060
■	0.066
+	0.071
▽	0.075

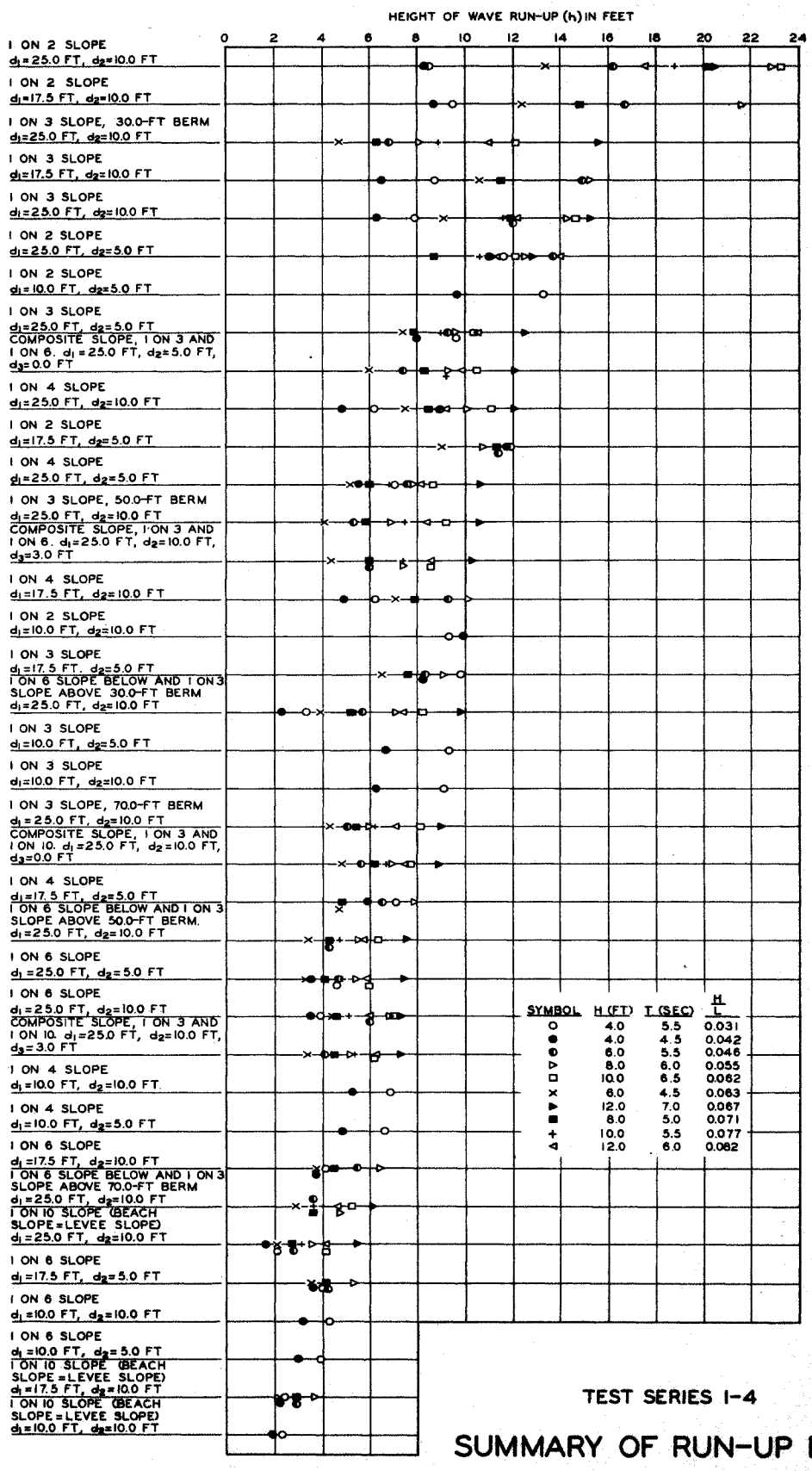
uses runup value from 1:30 scale not 1:17 as is implied by d_1 = 34.0 ft !
effect is to shift the lower portions of the curves to right.

TEST SERIES 8

OVERTOPPING AS A FUNCTION OF $\frac{y}{H}$ AND $\frac{H}{L}$

LEVEE SLOPE = 1 ON 6

d_1 = 34.0 FT d_2 = 10.0 FT



TEST SERIES 1-4

SUMMARY OF RUN-UP DATA