DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS

THE

BULLETIN

OF THE

BEACH EROSION BOARD

OFFICE, CHIEF OF ENGINEERS
WASHINGTON, D.C.

VOL. 7 OCTOBER 1, 1953 NO. 4

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS

THE

BULLETIN

OF THE

BEACH EROSION BOARD

OFFICE, CHIEF OF ENGINEERS
WASHINGTON, D.C.

VOL. 7 OCTOBER 1, 1953 NO. 4

TABLE OF CONTENTS

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
BEACH EROSION BOARD

Vol. 7 1 OCTOBER 1953 No. 4

COMPARISON OF DEEP WATER WAVE FORECASTS BY THE DARBYSHIRE AND BRETSCHNEIDER METHODS AND RECORDED WAVES FOR POINT ARGUELLO, CALIFORNIA, 26-29 OCTOBER 1950*

There are today essentially two published methods of making wave forecasts. The first is a several times revised version of the Sverdrup-Munk method originally published in the early days of World War II (1)**, the latest revisions being incorporated in a report by Bretschneider(2). The other is an outgrowth of British work during the war. It is given in its most advanced state in a recent article by Darbyshire(3). A short comparison of the waves recorded during a single storm and those forecast by the two methods is believed to be of interest.

A rather extensive forecast by the Bretschneider method had already been made for the storm of 26-29 October for Point Arguello, California(4,5), and an analysis of the waves reaching Point Arguello was available from the University of California pressure recorder situated in a depth of about 80 feet off Point Arguello. The conditions of this storm during a portion of which surface winds in excess of 40 knots were observed were thought to be relatively representative of the winter lows approaching the California coast. This storm was, therefore, chosen for the comparison.

The results of the comparison are shown in Figure 1. Figure 1a shows a comparison of the significant heights and periods as obtained by the two methods with those observed. It will be noted that the Bretschneider forecast results in two wave trains after 0500 on the 28th; therefore a dashed line has been added representing the heights obtained if these two trains are combined by the square root of the sum of the squares; this latter should be more nearly what would be obtained from the recorder. From Figure 1a it is seen that the major disparity between the methods is one of time. Figure 1b has been plotted to show this more fully. In this figure the time bases of both the Bretschneider and Darbyshire forecasts have been shifted to make the time of the peaks correspond with that of the recorded peak; this required a 2-hour retardation (shift to the right on the time base) for the Bretschneider forecast, and a 16-hour advance (shift to the left) for the Darbyshire forecast. When this time shift has been made, it is readily seen (Figure 1b) that both methods of forecasting give the same general curve of wave height, with the Darbyshire method following the observed curve somewhat more closely except for a 2.5-foot error at the storm peak. Both methods give values of the same order of magnitude and, statistically,

* The analysis by the Darbyshire method was made by Mortimer Datz, who also prepared the initial draft of this report. Mr. Datz has since left the staff of the Board to finish his studies abroad, and this final draft of his report has been prepared by the Research Division.

** Numbers in parenthesis refer to the bibliography at the end of the article.

would seem to give roughly the same frequencies -- i.e.; a statistical summary of wave conditions at a point over a long period of time would probably be similar regardless of which method was used.

There appears to be no particular explanation for the 16-hour late arrival time of the waves forecast by the Darbyshire method, and there are, of course, not enough data to show whether this is abnormal, such large deviations occurring only infrequently, or is inherent in the method (resulting perhaps from a speed of travel of the wave components different from that predicted by the group velocity). Since the method has apparently been used quite satisfactorily by the British, it is probably the former.

It is interesting to note that both methods show an increase in wave height from a secondary fetch following the main storm. Apparently this was not recorded, and it is not known whether this was due to refraction (the gage is in a depth of about 80 feet) or to some other cause.

The Darbyshire method is considerably more time consuming than that of Bretschneider (though undoubtedly short cuts in the computations would be found were the method to be adopted for extensive use), but the former does have the advantage of providing a picture of the wave spectrum. Since the different components of the wave train refract differently according to their periods, this spectrum can become very important in predicting inshore conditions. In the general case, results of sufficient accuracy can be obtained from the use of the significant period alone for refraction purposes, but in cases where there is a wide spectrum or the refraction conditions are complex, this may lead to significant errors. A sample of the predicted wave spectrum for the peak of the storm is shown in Figure 2. The peak height during the storm (11.9 feet) is the square root of the sum of the squares of the heights of the individual components shown.

Recent work at New York University by Neuman and Pierson has resulted in a third method of forecasting waves, as yet unpublished. It is planned to make a similar forecast for this same storm by that method, when it becomes available.

BIBLIOGRAPHY

(1) Sverdrup, H. U, and W. H. Munk - Wind Waves and Swell, Principles in Forecasting, H. O. Misc. 11,275, 1944.

(2) Bretschneider, C. L. - Revised Wave Forecasting Relationships, Proc. 2nd Conf. on Coastal Engrg., Council on Wave Research, The Engineering Foundation, 1952.

(3) Darbyshire, J. - The Generation of Waves by Wind, Proc. Roy. Soc. A, n. 1122, v. 215, 1952.

(4) Bretschneider, C. L. - Revised Wave Forecasting Curves and Procedures, Inst. of Engrg. Res., Univ. of Calif., Ser. 29, Issue 155-47, 1951 (unpublished).

(5) Kaplan, K. - Analysis of Moving Fetches for Wave Forecasting, Beach Erosion Board, Technical Memorandum No. 35, 1953.

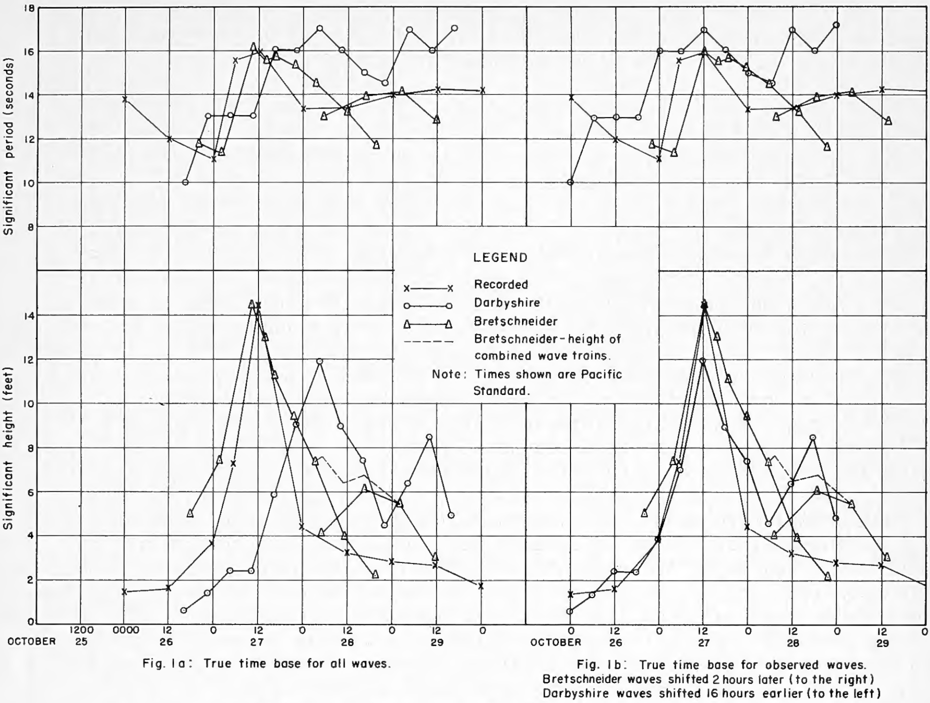

FIG. I: COMPARISON OF DEEP WATER WAVE FORECASTS AND RECORDED WAVES FOR POINT ARGUELLO, CALIFORNIA, 26-29 OCTOBER 1950

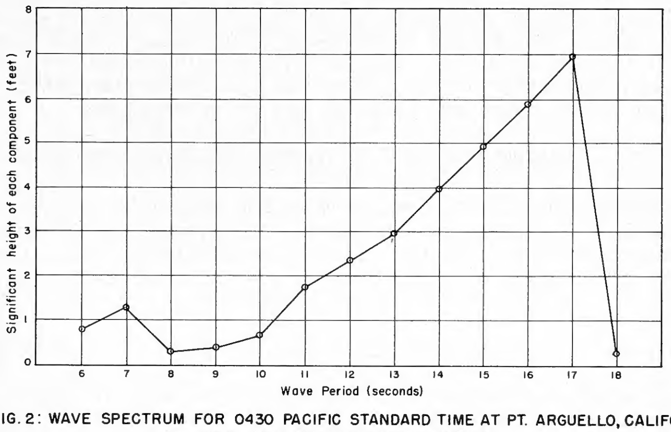

FIG. 2: WAVE SPECTRUM FOR 0430 PACIFIC STANDARD TIME AT PT. ARGUELLO, CALIFORNIA PREDICTED BY DARBYSHIRE METHOD

A COMPARISON OF OBSERVED AND HINDCAST WAVE CHARACTERISTICS OFF SOUTHERN NEW ENGLAND

by
Robert F. Dearduff
Hydraulic Engineer, Research Division
Beach Erosion Board

The Beach Erosion Board in collaboration with the U. S. Navy Hydrographic Office conducted a series of tests to compare wave characteristics obtained simultaneously in deep water, in relatively shallow water, and computed by the revised Sverdrup-Munk method of wave hindcasting.

The records for deep water were to be obtained at the Nantucket Lightship (located some 40 miles southeast of Nantucket in about 180 feet of water) by means of a floating step-resistance gage held stationary by a damping disk; however this gage became inoperative due to short-circuiting during its calibration aboard ship. Consequently, wave measurements were made visually from the lightship. Some 116 observations (on a four-hour-interval basis) were made. Height observations were obtained by noting the rise and fall of the water against the wave staff, length observations by using the length of the ship and its various parts as a reference, and period observations by stopwatch, using the travel time of a floating object from crest to crest. In each case values were obtained as an average over a three-minute period. Although obviously a true average value was not obtained, the very small waves being neglected, this value is still probably somewhat less than the significant values.

The records for relatively shallow water were obtained by a pressure-type recorder installed in 30 feet of water off Martha's Vineyard, Massachusetts. This recorder was operated intermittently over the period September 20 to October 9, 1951. Some 58 seven-minute records were made during this time, each recording representing a four-hour time interval. The locations at which waves were observed are shown on Figure 1.

NANTUCKET LIGHTSHIP OBSERVATIONS*

The Navy Hydrographic Office data are broken down into the two main divisions: Swell Waves and Local Wind Waves. However, an analysis of the data shows that there is no distinct line of demarcation between these, inasmuch as waves of 2, 3 and 4-second periods are included in both divisions.

* "Preliminary Report, Nantucket Wave Observation Project, 19 Sept-9 Oct 1951," Navy Hydrographic Office (unpubl).

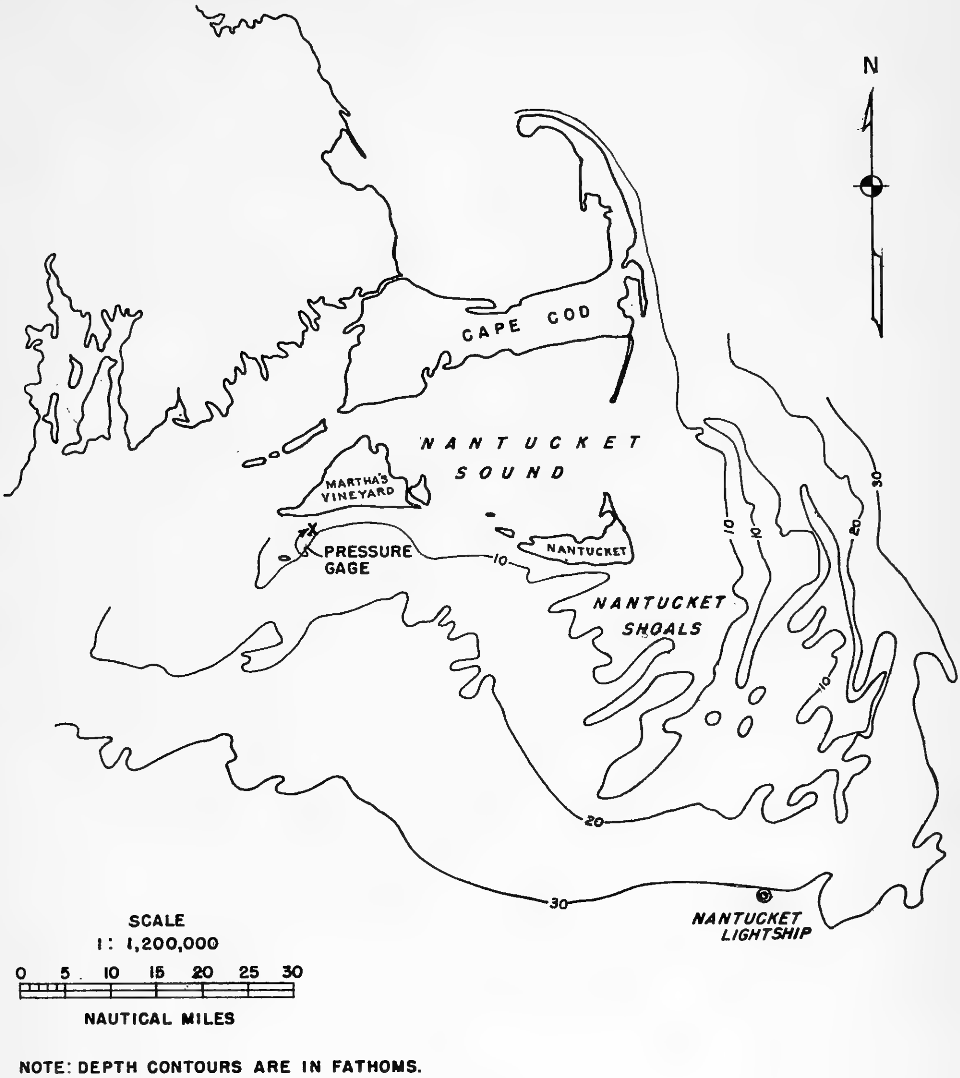

FIG. 1: LOCATIONS OF WAVE OBSERVATIONS

A wave steepness (H/L) probability curve was plotted for all data given under the local wind wave group and it was ascertained that about 70 percent of the waves theoretically could not exist as the wave steepness exceeded 1/7 (0.14). In some cases, the steepness ran as high as 0.4 (Figure 2). (The length, L was calculated on the basis of the observed period, T). As wave periods were obtained from the total number of waves passing the observer in a given time interval, it is thought that the exceedingly high steepness ratios obtained may be attributed to the use of too many waves associated with the smaller heights in determining the average period of the waves of measured height. A breakdown of wave periods for this local wind wave group shows that a total of 68 percent of all the waves had a period of 2 seconds or less (Figure 3). This is manifestly a small period for an ocean wave.

Inasmuch as the observed wave period was determined by measuring with a stopwatch the time interval between two consecutive wave crests passing a floating object, it might be assumed that the time measurement was made not between crests of similar period waves in the group but rather between crests of dissimilar period waves in the group. For example, a wave spectrum under observation might include waves of 7- and 5-second periods and the observer might measure the time interval between the 7- and 5-second wave crests rather than between two consecutive 7-second wave crests, thereby getting a much smaller period than the actual one. Illustrative of this, Figure 4 shows a possible wave train consisting of 7- and 5-second period waves. "A" represents the crest of the 7-second wave and "B" the crest of the 5-second wave; it is quite conceivable that in measuring period the time interval between crest "A" and crest "B" was taken rather than the time required for consecutive crests "A" and "A" to pass a given object.

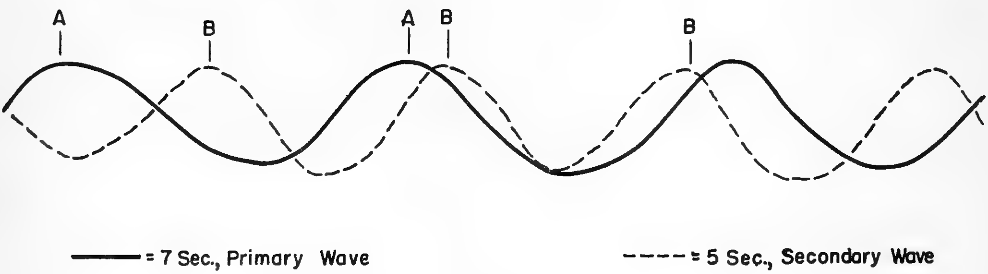

FIG. 4

This same situation could have existed in measurement of wave length, i.e., determining length by the distance between crest "A" and crest "B" rather than between "A" and "A". It is also possible that heights for one set of waves were measured and the periods and lengths for another set and the results of all three grouped together as characteristics of the same wave.

As a result of the above inconsistencies, it was decided, after a talk with the Navy observer, to neglect the local wind wave data and to concentrate solely on the comparison of observed and hindcast swell characteristics.

The steepness ratio of the waves in the observed swell group is more reasonable, none of these exceeding H/L ratios of 1/7. However, it was noticed in this group that the observed wave lengths were as a whole much smaller than the calculated lengths based on the usual formula $L = 5.12T^2 \tanh 2\pi d/L$, obtained from the observed period T. There appears to be an inconsistency here as the observed wave lengths should approximate the calculated wave lengths based on the observed period. This again might indicate measurements involving dissimilar wave period crests in determining the wave length as shown in Figure 4. A curve was plotted (Figure 5) from all the data in the observed swell group and it was ascertained that about 60 per cent of all the recorded waves had a ratio of length measured to calculated of 0.5 or less, and that only about 5 per cent of the waves approximated a unity ratio of measured to calculated length.

Similar discrepancies have been noted in past tests; for example, wave characteristic data taken by M. P. O'Brien* at the mouth of the Columbia River in the early 1930's showed that the measured wave length was approximately 75 percent of the calculated wave length. Dr. Willard J. Pierson, Jr.** of New York University in a recent paper expressed the belief that the wave length in deep water is not correctly given by the equation $L_o = 5.12T^2$, except for a condition of the sea having but one simple wave train. For a "turbulent" sea consisting of various wave trains of many period components from several directions, he determines that the actual "average" length should theoretically be about six-tenths of this value determined from this "average period". This might to some extent explain the predominance of measured wave lengths shorter than calculated wave lengths. The average value of this ratio for these data was 0.493.

An analysis of observed deep water wave directions showed that the greatest frequencies were from the West and South with percentages of total

* M. P. O'Brien "Wave Action and Salinity Currents at the Mouth of the Columbia River" presented before the Oceano. Soc. of the Pacific, Denver, June 21, 1937

** W. J. Pierson, Jr. "An Interpretation of the Observable Properties of Sea Waves in Terms of the Power Spectrum of the Gaussian Record" presented at 34th Annual Meeting of AGU at Wash., D. C., May 4-6, 1953 (to be publ.)

frequency of 21.5 and 17.4 respectively.(Figure 6). The percentages of frequencies derived by hindcasting were as follows: North 17.5%, East 16.5%, South 15.0%, Southwest 16.3%. Relative to the directions Northwest, North, and Northeast, the discrepancy between the observed and hindcast wave frequencies may be accounted for by the fact that there are a series of shoals averaging 35 feet in depth and having minimum depths of 20 feet located some 40 miles from the Lightship in these directions. Waves having periods in excess of 3 seconds would refract in passing over these shoals and would not necessarily approach the Lightship from the original deep water directions; and in some cases (depending on period) would never reach the Lightship. Thus without making numerous wave refraction diagrams covering these directions and all periods in excess of 3 seconds, it is impossible to make a good wave direction comparison between observed and hindcast data.

A comparison of observed and hindcast wave heights and periods (Figure 7) showed very good similarity of phase pattern; for the most part observed peaks and hindcast peaks occurred simultaneously with some individual linear difference in height and period. The average wave height difference (hindcast greater than observed) was 0.66 foot and the average wave period difference was 1.9 seconds over the three week period under observation, where the observed wave heights averaged about $3\frac{1}{2}$ feet (ranging from 1 to 11 feet) and the observed period averaged about 4 seconds (ranging from 2 to 12 seconds). The fact that the hindcast values are consistently greater than the observed may be attributed in part at least to the difference between the significant wave hindcast and the somewhat lower "average" wave observed.

About 50 percent of all swell waves considered had a deviation (hindcast height minus observed height) of plus or minus 1 foot or less, and 73 percent had a deviation of plus or minus 2 feet or less (Figure 8).

As shown in Figure 8a there was a small percentage of waves wherein the differential between hindcast and observed heights was greater than 2 feet up to a maximum of 9 feet; the explanation of this large discrepancy is not immediately apparent as the majority of these waves came from the south and southeast and consequently would not be affected by refraction. No refractive effects have been considered in this comparison of hindcast and observed data; however the use of refraction diagrams would probably result in only a negligible difference in the height deviation already given as, excluding those waves crossing Nantucket Shoals, only those waves with periods of 8 seconds or more would be affected by refraction, as the lightship is moored in 180 feet of water.

Figure 8b shows a histogram of wave period deviation (hindcast minus observed). About 36 percent of all waves had a deviation of plus or minus 1 second or less and 57 percent had a deviation of plus or minus 2 seconds or less.

MARTHA'S VINEYARD MEASUREMENTS

It is difficult to make a worth-while comparison between the hindcast data and that obtained with the pressure-type recorder due to the sporadic recordings obtained from the latter. During half of the time under observation (September 20 - October 9, 1951) this gage was not functioning properly and no records were obtained. In contrast with the visual observations made aboard the Nantucket Lightship which numbered 116 (each representing a four-hour period), only 58 readings were obtained from the pressure gage records. There were no wave length measurements made and no wave directions given so that only the wave height and period can be compared.

with

A comparison of recorded/hindcast height and period (Figure 7) showed good similarity of phase pattern; however the average difference between hindcast height and recorded height showed the hindcast height greater by 3.2 feet. The average period difference showed the recorded period as being 2.4 seconds greater than the hindcast. About 76 percent of all waves considered had a deviation (hindcast height less recorded height) of 4 feet or less (Figure 9). The height deviation between hindcast and recorded data is not as bad as it appears at first glance. The pressure head was located in 30 feet of water and waves having periods in excess of 3.5 seconds would refract before reaching it, resulting generally in smaller heights. In the recorded data, none of the waves had a period of less than 4 seconds and 50 percent of the waves had periods greater than 8 seconds. An analysis of the hindcast data showed no waves with periods less than 3 seconds and 50 percent of the waves had periods greater than 5 seconds, (Figure 10). Therefore all of the waves under observation would to some extent be affected by refraction. Figure 10 shows, as might be expected, that the hindcast periods fall between those of the deep water observations (where frequently too many waves are counted)and those of the underwater pressure head (where lower period components are reduced by pressure attenuation, and the method of analysis tends to result in higher periods).

ACKNOWLEDGEMENT

The Beach Erosion Board gratefully acknowledges the assistance of the Navy Hydrographic Office and the Coast Guard in this study.

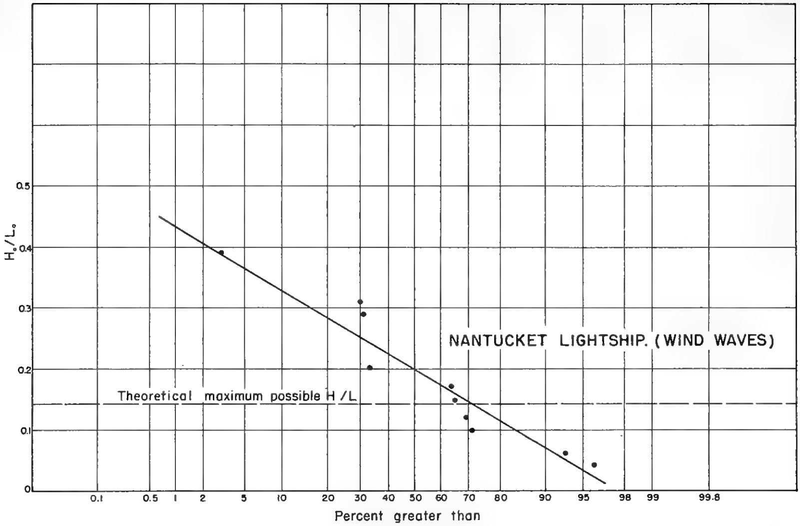

FIG. 2. WAVE STEEPNESS FREQUENCY

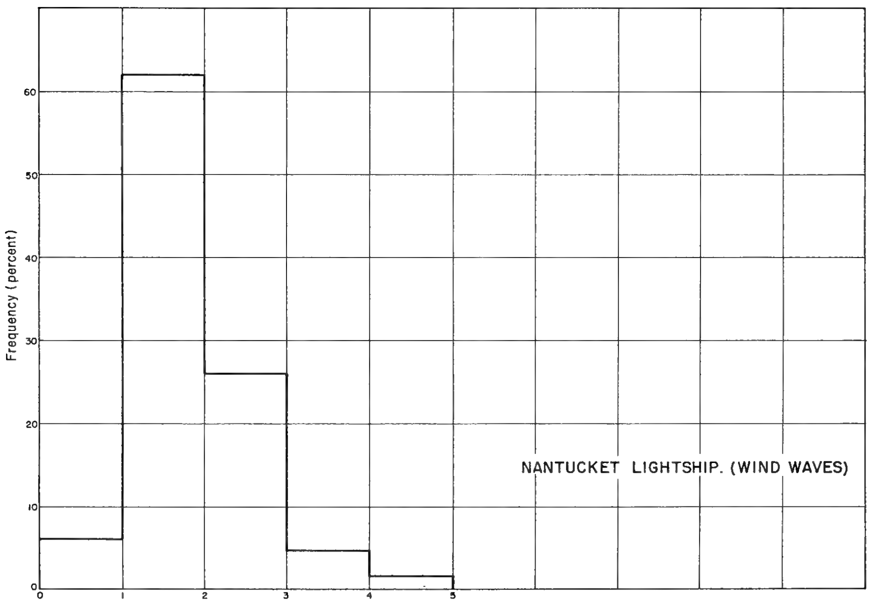

FIG. 3. FREQUENCY OF PERIOD SPREAD

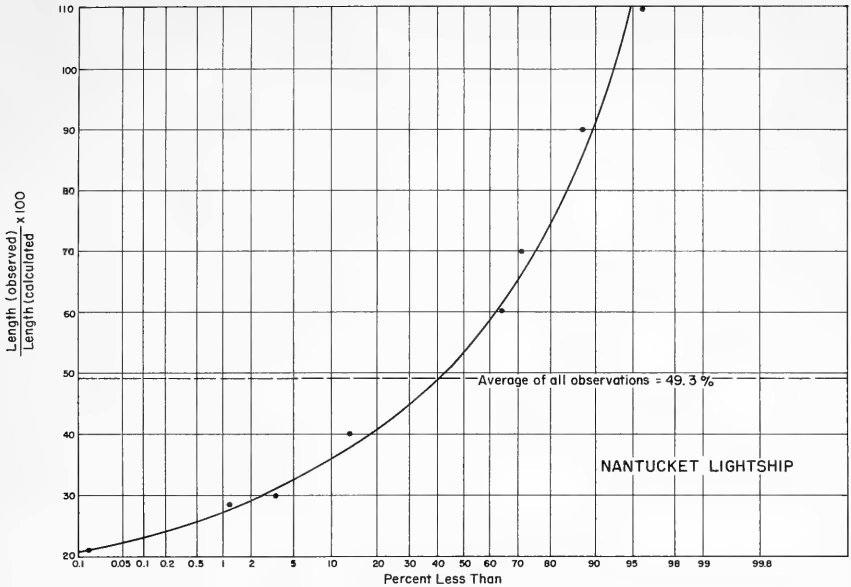

FIG. 5. RELATION BETWEEN OBSERVED AND CALCULATED WAVE LENGTH AND FREQUENCY

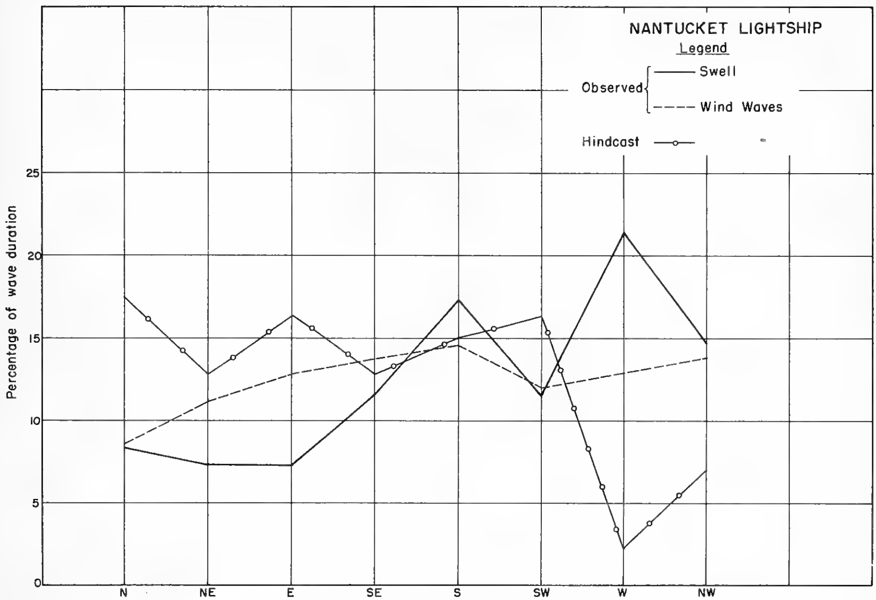

FIG. 6. WAVE DIRECTION FREQUENCY

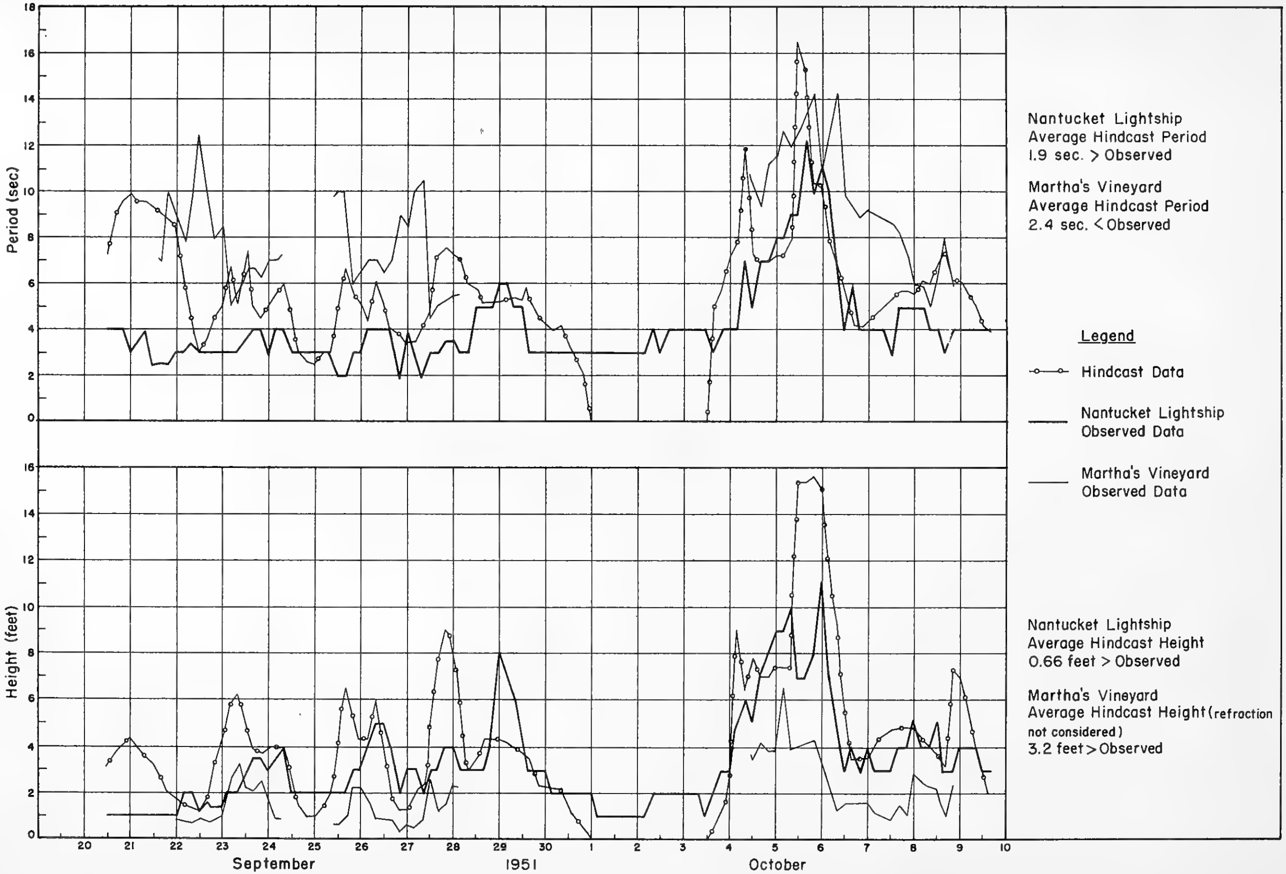

FIG. 7. HEIGHT AND PERIOD AS A FUNCTION OF TIME

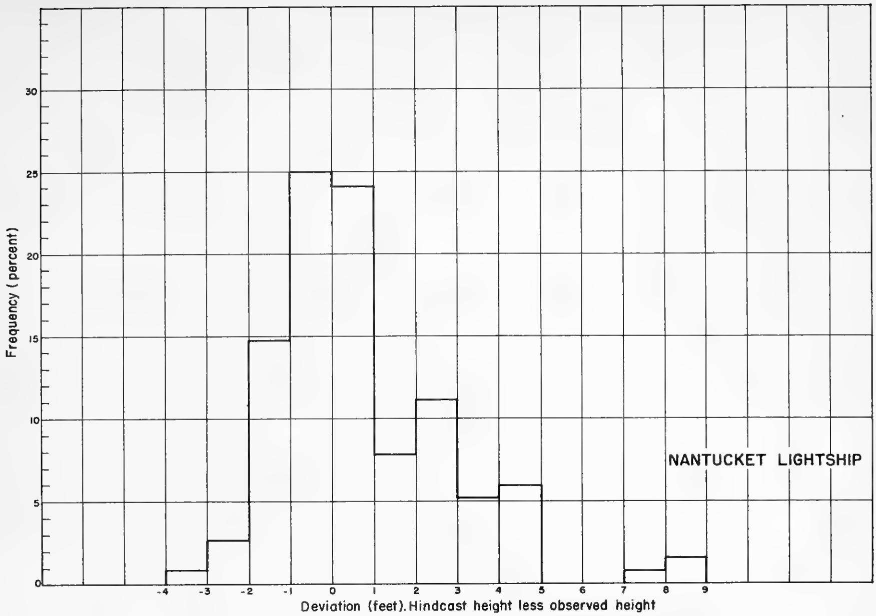

FIG. 8a. FREQUENCY OF HEIGHT DEVIATION-HINDCAST LESS OBSERVED

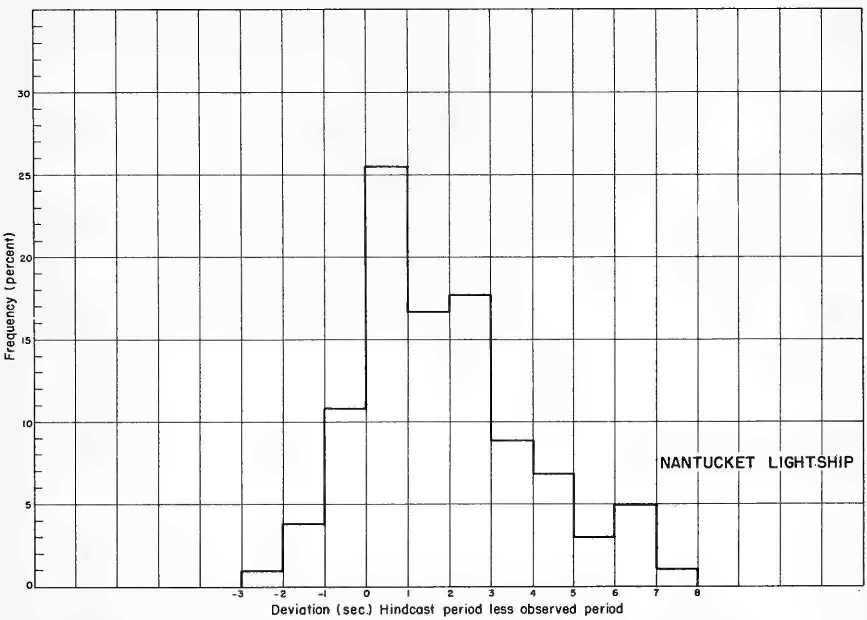

FIG. 8b. FREQUENCY OF PERIOD DEVIATION-HINDCAST LESS OBSERVED

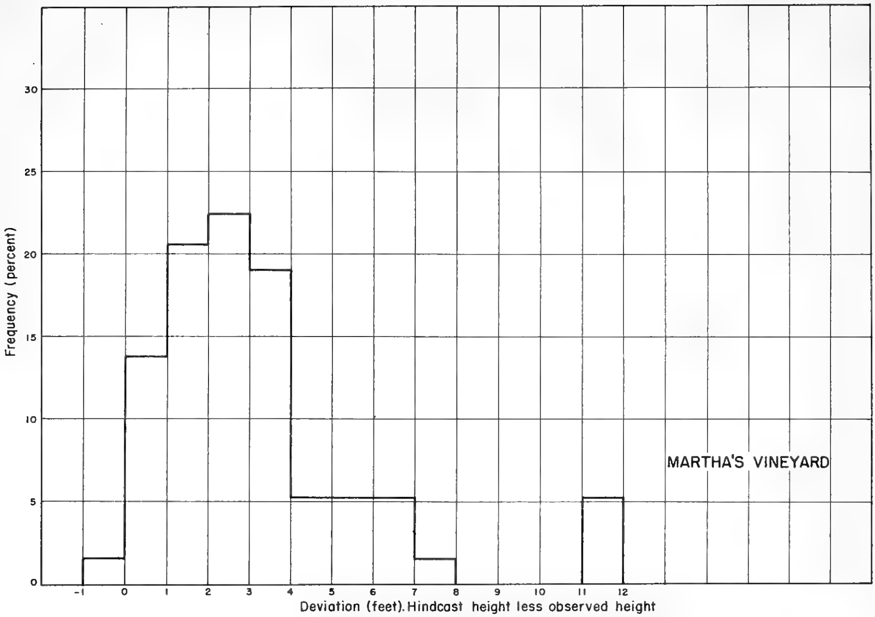

FIG. 9 . FREQUENCY OF HEIGHT DEVIATION-HINDCAST LESS OBSERVED

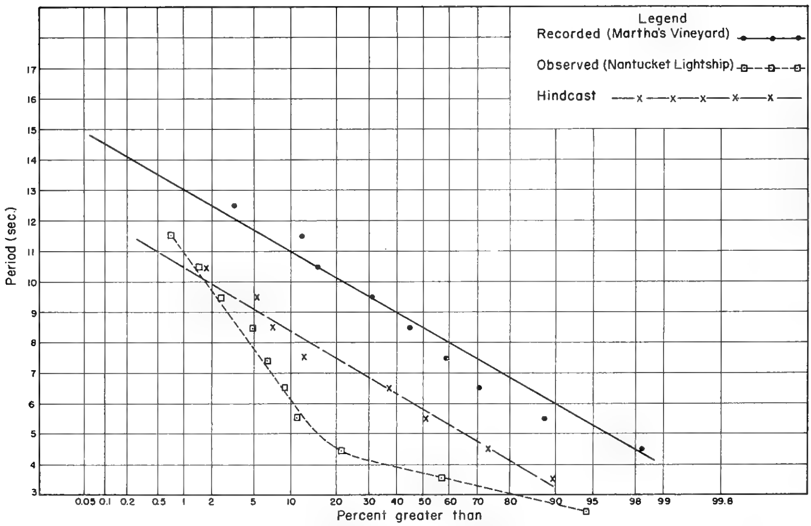

FIG. 10. WAVE PERIOD FREQUENCY

PROGRESS REPORTS ON RESEARCH SPONSORED BY
THE BEACH EROSION BOARD

Abstracts from progress reports on several research contracts in force between universities or other insitutions and the Beach Erosion Board, together with brief statements as to the status of research projects being prosecuted in the laboratory of the Beach Erosion Board, are presented as follows:

I. University of California, Contract No. DA-49-055-eng-8, Status Report No. 10 - 1 June through 31 July 1953.

The mechanical analysis of the 60 core samples collected during March from the filled area West of Santa Barbara breakwater has now been completed. The data obtained are now being compiled in the forms of tables and graphs.

II. University of California, Contract No. DA-49-055-eng-17, Status Report - October 1, 1952 to June 10, 1953.

This project is concerned primarily with an investigation of the fundamental mechanics of sediment movement by wave action, with particular attention being given to the problem of movement in depths of water beyond the surf zone. Evidence indicates that appreciable movement of sediment appears to take place in water depths as great as 60 feet and possibly greater. One of the first steps in placing an outer limit to the depths at which sand movement along the ocean bed by wave action might be expected to occur, is the formulation of a criterion for the condition at which flow at or near the bed is unstable (turbulent). The initial work in this study consisted of a theoretical analysis on the stability of oscillatory flow along a wall.

During the period in which this theoretical study was in progress, the experimental equipment was being constructed. This equipment consists of a tank with an oscillating bottom which can be given a motion relative to still water comparable to water motions occurring in the ocean. The initial set of experiments were made with both a smooth bed and a rough bed. The bed roughness was accomplished by fastening various types of roughness to the plate. Materials used consisted of sand, pea gravel, and half-round wooden strips of various sizes. The condition at which flow changed from laminar to turbulent flow was observed by the use of dye.

Both the theoretical and experimental study with a fixed bed are presented in the report "Stability of Oscillatory Laminar Flow Along a Wall" by Huon Li. This report will be distributed in July 1953.

Experimental work with a movable sand bed has been started and will be summarized in the next Status Report.

III. Scripps Institution of Oceanography, Quarterly Progress Report No. 16, Contract No. W-49-055-eng-3, April-June 1953.

The field phase of the investigation of orbital velocities has been completed. Preliminary investigation of wave data indicates that the relation between mean water depth d, depth under the wave trough h, and the wave height H can be expressed empirically as $d-h = kH$, where k is a constant for a given series of observations. To date k has been found to range from about 0.1 to 0.2, which means that for a given height, measured from wave trough to wave crest, approximately 80 to 90 percent is above mean water level.

The program of underwater observations by swimmers equipped with self-contained diving gear is being expanded. At present, objectives are (1) studies of the mechanics of sand transport, (2) accurate measurements of cut and fill by means of reference rods, and (3) development and improvement of equipment and techniques for underwater studies. Cut and fill of the sandy bottom in water depths of 52 and 70 feet based on averages of six rods at each station, has been negligible ($\pm$ 0.02 feet). Measurements in shoaler areas indicate somewhat greater changes.

Comparison has been made between the amount of shoaling and deepening of the valley axes at the head of Scripps Submarine Canyon between surveys. The surprising result is that there are indications that shoaling takes place at a relatively rapid rate following deepening. After a submarine landslide, rapid fill takes place at the very head of the valley and then progresses gradually down the valley axis.

A study of suspended sediment in and near the surf zone has been initiated. It is planned to sample suspended material in situ both by pumping and by introducing a measuring instrument into the suspension. A photometric technique similar to that used in studies of back scattering in the atmosphere is being considered.

A report, "Areal and Seasonal Variation in Beach and Nearshore Sediments at La Jolla, California" was published as Beach Erosion Board Technical Memorandum No. 39.

IV. New York University, Contract No. DA-49-055-eng-32, Third Quarterly Report, September 8, 1953.

Hindcast Data. Dr. Neumann and Mr. James have prepared a paper on the methods they are using to make hindcasts for selected points on the east coast. The procedures they are using are extensive, detailed and complete.

Wave Analyser. The wave analyser is finished and it has been calibrated against speech records and artificial signals. It has yet to be tested on some actual wave records, but with the general increase in wave activity on the east coast some tape recorder wave records should soon be available for this last step.

One additional change in design is contemplated in which the recorder which graphs the wave spectrum will be changed to a type which will present a larger and more detailed spectrum. This particular type of recorder is in short supply and it will not be available until mid-October Thus the wave analyser in final form should be completed by mid-November.

V. The Agricultural and Mechanical College of Texas, Contract No. DA-49-055-eng-18, Quarterly Report for period ending 31 July 1953.

At the end of the first 13 months of the extended contract, Project 50 was renewed for 12 additional months and the title was changed to "Wave Energy Losses in Shallow Water Ocean Waves".

During the quarter, May 1, 1953 to July 31, 1953 the following progress was made:

1. Planning of Operations. As a result of the gas well fire at Pure Oil structure A, all Research Foundation instruments, including the pressure head wave recorders were removed from structure A and also structure B. Permission was sought and obtained from the Sun Oil Company, to reinstall the above mentioned instruments on the Sun Oil Pier, located about 18 miles east of Galveston. This pier extends 2600 feet into the Gulf. An instrument shack sponsored by other Research Foundation projects was constructed at the end of the Sun Oil Pier. (The electrical wiring of the shack has been completed and actual installation of Research Foundation instrumentation will soon begin).

During the first week of July permission was obtained from the Pure Oil Corporation to reinstall a wave recorder at structure A and also one at structure B.

Since this project has three pressure type wave recorders and four ideal sites for installation, it was planned to install permanently, one wave recorder at Pure Oil Structure A, and also one at the seaward end of Sun Oil pier. It is planned to alternate periodically the installation of the third wave recorder between Pure Oil structure B and the shoreward end of the Sun Oil pier.

2. Field Operations. Actual field operations were resumed at Pure Oil structures A and B during the third week of July. One pressure head support and pressure head was installed permanently at structure A and the second one was installed at structure B.

3. Interim Technical Report. Work was continued on the Technical Report "Change in Wave Height due to Bottom Friction, Percolation and Refraction". Originally it had been intended to complete this report by 1 May 1953. However, additional graphs and examples are being included in order that the report be more complete. It is now anticipated that the report will be released within 30 days.

4. Theoretical Investigations. A theoretical investigation of the effect of bottom fluctuations on the change in wave height has been initiated.

Preliminary computations involving the combination of wave generation and wave energy loss due to bottom friction have begun.

VI. Massachusetts Institute of Technology, Contract No. DA-49-055-eng-16 Progress Report dated September 17, 1953.

1. Wave Channel. The channel is completed having been caulked and water tested so as to be without leaks under the full head of three feet.

2. Wave Generator. The hydraulic components, carriage, rails and feedback mechanism are completed and installed. The piston is under construction at the present time.

The manufacturer of the permanent, adjustable speed, cam drive has indicated a further delay in delivery, the data now being 15 October 1953. Construction of the adjustable stroke mechanism to be incorporated with the above drive will be completed by 1 October 1953. However, a temporary non-adjustable cam drive has been installed which produces the maximum generator stroke of 2 feet at a period of 1.90 seconds. The generator has been operated under these conditions with a still-water depth of 20 inches in the flume and has been found to be mechanically satisfactory.

3. Baffles. Pilot tests were conducted in one of the laboratory's smaller wave channels to determine the most effective material to be used in dissipation of the wave energy behind the wavemaking piston. Tests on rock filled cribs, logs and automobile radiator cores in various positions and combinations indicated the latter to be the most effective. It was found that an optimum spacing of two such vertical baffles existed at which point the percent energy transmitted and reflected were both a minimum.

The baffles and a beach of 15 to 1 slope have been installed in the wave channel.

4. The false bottom, fixed beach at a slope of 15 to 1 is being coated with Ottawa sand of the size range passing the No. 20 sieve and retained on No. 25. This sand, the representative diameter of which is about 0.03 inches, will be glued to the beach to provide roughness. The type and size range of the sand was chosen because it corresponds with that used in previous bed load studies in this laboratory. The steady-state hydraulic roughness of this sand has already been determined, and it is expected that valuable comparisons can be drawn between the steady and unsteady characteristics of bed load movement.

5. A still-water depth of approximately 18 inches will be used in the wave channel and short waves of low amplitude will be generated. These waves will be referred to the equivalent deep water wave and their effect on the distribution of statistical samples of spherical particles will be studied.

VII. Waterways Experiment Station, Vicksburg, Mississippi, Progress Report for Period ending 31 August 1953.

Wave Run-up on Shore Structures - Overtopping tests on a smooth faced pavement with a sea-side slope of 1 on $1\frac{1}{2}$ and a beach slope of 1 on 10 were completed for water depths of 25 and 34 feet, using crown elevations from +3 to +39 feet.

Effect of Inlets on Adjacent Beaches - A test is in progress with the lagoon (forced) tidal apparatus adjusted so that the inlet will remain open. The unbroken beach was stabilized in 120 tidal cycles, the inlet cut through, and the test run for 320 additional tidal cycles. The inlet is tending to migrate, breakthroughs having occurred after 70, 200, and 320 cycles.

VIII. Beach Erosion Board, Research Division, Project Status Report for Quarter ending 18 September 1953.

In addition to the research projects under contract to various institutions which are reported on above, the Research Division of the Beach Erosion Board is carrying out certain projects with its own facilities. The main unclassified projects were described in Volume 6, No. 4 of the Bulletin (October 1952) and a short description of some of the work accomplished through the last quarter is given below.

Project ESMOND. The laboratory testing for this project is about 95 percent complete. The testing involved the dropping of various shaped and weighted sounding leads into a sediment tank containing Delaware River low-density material. A report concerning the results of these tests is now being prepared. The report will also include the results of at least two field investigations which were made with the experimental leads.

Study of Effect of Tsunamis. The problem of tsunami run-up is being attacked through the analysis of the run-up of solitary waves since it may be expected that at least the first wave of a tsunami group should act similar to a solitary wave. Results previously obtained (BEB Tech. Memo. No. 33) for slopes of between 5 and 45 degrees are being extended to flatter slopes (1 on 30, 1 on 60). In addition the effects of combinations of slopes will be tested. It is expected that a preliminary report will be completed by October 15..

Routine progress, testing and analysis has been made on the other projects being carried out by the Research Division. Project reports on Wave and Lake Level Statistics for Lakes Michigan, Erie, and Ontario were published as Technical Memorandums No. 36, 37 and 38 and on "Development and Field Tests of a Sampler for Suspended Sediment in Wave Action" as Technical Memorandum No. 34. In addition two short reports on waves off Martha's Vineyard and on a comparison of a wave forecast by the Darbyshire method with one by the Sverdrup-Munk method as revised by Bretschneider, are contained in this issue of the Bulletin.

BEACH EROSION STUDIES

Beach erosion control studies of specific localities are usually made by the Corps of Engineers in cooperation with appropriate agencies of the various States by authority of Section 2 of the River and Harbor Act approved 3 July 1930. By executive ruling the costs of these studies are divided equally between the United States and the cooperating agencies. Information concerning the initiation of a cooperative study may be obtained from any District or Division Engineer of the Corps of Engineers. After a report on a cooperative study has been transmitted to Congress, a summary thereof is included in the next issue of this bulletin. A summary of reports transmitted to Congress since the last issue of the Bulletin and a list of authorized cooperative studies follow:

SUMMARIES OF REPORTS TRANSMITTED TO CONGRESS

PLUM ISLAND, MASSACHUSETTS

The area studied comprises the northerly 4.7 miles of the Atlantic Ocean shore of Plum Island within the limits of the City of Newburyport and Town of Newbury. The island extends southward from the mouth of the Merrimack River for a total length of about 8 miles. Its northern end is about 3.7 miles south of the Massachusetts-New Hampshire border. Development which includes a United States Coast Guard Station is concentrated in the northern 2 miles of the island. Other than the Coast Guard Station property, this shore frontage is almost entirely privately owned. The remainder of the island to the south, with minor exceptions, is a wild life sanctuary under Federal control. The population of the City of Newburyport and the Town of Newbury in 1950 were 14,111 and 1,944 respectively.

Plum Island is a sandy barrier island with a narrow sand beach backed by dunes. It is separated from the mainland by Plum Island River and a wide marsh. The northern end of the island is split by a shallow body of water known as the Basin which projects southward from the Merrimack River estuary. The width of the island between the Basin and the Atlantic Ocean is about 350 feet at the narrowest part. The north end of the island at the Merrimack River has a width of about 0.6 mile.

In recent years, the ocean shore line along the northern end of Plum Island has been subject to intermittent erosion and accretion, with shifting of the eroding area alongshore and a resulting gradual recession of the shore as a whole. Storm waves accompanying high tides pass over the beach berm and erode the dunes. Large volumes of material are removed in localized areas during severe storms. Shifting of the areas of erosion is associated with changes in the offshore bar. Eroding areas are incompletely restored due to a deficiency in the rate of supply of material to the area. The deficiency in supply is estimated at about 36,000 cubic yards annually. Adequate protection can be provided to the shore in the problem area by restoring a suitable protective beach by artificial placement of sand fill.

The restored beach can be maintained by artificially placing beach material as required to offset the deficiency in supply.

The Division Engineer and the Beach Erosion Board concluded that the most suitable method of stabilizing the shore and protecting shore front cottages and Northern Boulevard consists of direct placement of sand fill to widen the beach fronting cottages and dunes along that portion of the seaward shore of Plum Island between points approximately 3,000 and 6,000 feet south of the Merrimack River south jetty and raising the inshore portion of the south jetty as may be required to act as a barrier to northward drifting and loss of beach material into Merrimack River. They also found that the extent of publicly owned shore to be protected is minor, and as Federal assistance in the cost of protecting privately owned shores would not be in accordance with existing laws, no Federal assistance for protection of the shore of Plum Island was recommended at this time.

In accordance with existing statutory requirements, the Beach Erosion Board stated its opinion that:

a. since the problem area is privately owned and Federal statute provides no basis for Federal aid for the protection of privately owned shore lines, it is inadvisable for the United States to adopt at this time a project authorizing Federal participation in the cost of shore protection at Plum Island, Massachusetts;

b. the public interest involved in the proposed improvements is small, being limited to the value of a publicly owned street end and rights-of-way being lost through erosion; and

c. since there is no basis for the adoption of a Federal project, no share of the expense should be borne by the United States.

The Chief of Engineers concurred generally in the views and recommendations of the Beach Erosion Board.

STATE OF CONNECTICUT - NEW HAVEN HARBOR TO HOUSATONIC RIVER

The study area comprised the shore of Long Island Sound between the entrance to New Haven Harbor and the mouth of Housatonic River. It included the shores of the towns of West Haven and Milford, a total length of about 16 miles. This shore area is adjacent to and west of New Haven, Connecticut, and is about 50 miles east of New York City. It is extensively developed as a resort and residential area, with improvements ranging from small cottages to small estates. A number of small town-owned beaches are included in the area.

The eastern portion of the study area from the root of Sandy Point to Bradley Point consists of relatively low flat ground. Thence westward to Milford Harbor, headlands of glacial material with numerous rock outcrops extend to the shore. Between these headlands wave-built bars have been formed and the shoreward areas generally have filled and become marshy. West of Milford Harbor to Housatonic River, the terrain is again relatively low and flat. Charles Island, lying off this area, supplies material to the shore. The shores of this island are now covered with boulders remaining from erosion. A low water tombolo connects the island to the mainland, and the shore end of the tombolo includes a large marsh area.

The headlands of unconsolidated glacial till formerly supplied ample material to the intervening beaches. Due to the development of the area, the headlands now are generally protected by seawalls and the supply of material is thus reduced or eliminated. Consequently the beaches have slowly deteriorated. Long Island affords considerable protection and wave action in the sound is generally not severe. Ordinary storm waves cause littoral drift and offshore loss of beach material. Absence of swells probably precludes the possibility of return of material from offshore by wave action. The prevailing winds over Long Island Sound are from the westerly quadrant. The direction of littoral movement is dependent upon the orientation of the shore.

The tides in the area are semi-diurnal. The mean ranges are 6.2 feet at the entrance to New Haven Harbor and 6.6 feet at Milford Harbor. The highest tide of record occurred during the 1938 hurricane and was about 13 feet above mean low water. Tides in excess of 3 feet above mean high water occur infrequently.

The Division Engineer concluded that the most suitable plans of protection and improvement are as follows:

a. Bradley Point (West side), West Haven - Widening beach to 100-foot width by direct placement of sand and construction of one impermeable groin;

b. Prospect Beach, West Haven - Widening to a 100-foot width by direct placement of sand, 6,000 feet of shore from a point about 350 feet south of South Street northerly to Ivy Street with an added 50-foot widening at the south end of the fill and construction of 8 impermeable groins each 330 feet long;

c. Oyster River Point to Oyster River - Widening the beach to 100-to 150-foot width with groins as deferred construction, if needed.

d. Woodmont Shore, Milford - Widening to a 100-foot width by direct placement of sand, 500 feet of shore in the first pocket beach west of Merwin Point; widening to a 100- to 150-foot width, 3,500 feet of shore from Chapel Street northerly to a point about 400 feet north of Anderson Avenue and construction of 5 impermeable groins 300 to 400 feet long;

e. Burwell Beach, Milford - Widening the beach to 100-foot width by direct placement of sand and construction of one impermeable groin.

f. Gulf Beach, Milford - Widening to a 100-foot width, 1,200 feet of Gulf Beach by direct placement of sand;

g. Silver, Myrtle, Walnut, Laurel and Cedar Beaches and Meadows End, Milford - Widening to a 100-foot width by direct placement of sand, 15,600 feet of shore along Silver, Myrtle, Walnut, Laurel and Cedar Beaches and Meadows End, with an added widening of 150 feet around Meadows End and the construction of 11 impermeable groins 350 to 400 feet long.

The Board concurred generally in the views of the Division Engineer. In considering the plan of protection and improvement for the section from Silver Beach to Cedar Beach, it noted that the rate of recession of the shore west of Myrtle Beach had been relatively low over the period of record and considered that the rate of loss of fill from that area might also be low. Consequently, the Board believed that groins may not be justified by the reduction in maintenance that they would effect, and that maintenance by artificial placement alone may be more economical. Accordingly the Board considered that groins should be constructed only after experience with maintenance without groins indicates that they are needed and will result in lower annual costs.

In accordance with existing statutory requirements, the Board stated its opinion that:

a. It is advisable for the United States to adopt projects authorizing Federal participation in the cost of protecting and improving the publicly owned shores at Prospect Beach, West Haven, Woodmont Shore, Milford,Gulf Beach, Milford and Silver Beach to Cedar Beach, Milford, Connecticut.

b. The public interest involved in the proposed measures for these shores is substantial. It is associated with prevention of direct damages to publicly owned property, increased earning power of adjacent land, and recreational benefits to the public.

c. The share of the expense which should be borne by the United States is one-third of the first cost of the proposed work for the protection of those portions of the shore which are publicly owned at the time of completion of the work on a project.

The Board recommended that separate projects be adopted by the United States authorizing Federal participation by the contribution of Federal funds in an amount equal to one-third of the first cost of the measures for the protection and improvement of those portions of the shores of the Towns of West Haven, and Milford, Connecticut, which are publicly owned as follows:

a. Prospect Beach, West Haven - Widening to a 100-foot width by direct placement of sand, 6,000 feet of shore from a point about 350 feet south of South Street northerly to Ivy Street with an added 50-foot widening at the south end of the fill and construction of eight impermeable groins each 330 feet long;

b. Woodmont Shore, Milford - Widening to a 100-foot width by direct placement of sand, 500 feet of shore in the first pocket beach west of Merwin Point; widening to a 100 to 150-foot width, 3,500 feet of shore from Chapel Street northerly to a point about 400 feet north of Anderson Avenue and construction of 5 impermeable groins 300 to 400 feet long;

c. Gulf Beach, Milford - Widening to 100-foot width about 1,200 feet of beach by direct placement of sand;

d. Silver Beach to Cedar Beach, Milford - Widening to a 100-foot width by direct placement of sand, 15,600 feet of shore along Silver, Myrtle, Walnut, Laurel and Cedar Beaches and Meadows End, with an added widening of 150 feet around Meadows End, and the construction of eleven impermeable groins 350 to 400 feet long as deferred construction.

The Chief of Engineers concurred in the views and recommendations of the Beach Erosion Board.

COLD SPRING INLET (CAPE MAY HARBOR), NEW JERSEY

Cold Spring Inlet is the navigable channel between the Atlantic Ocean and Cape May Harbor, which is the most southerly of the tidal lagoons on the New Jersey Coast. It is about 6 miles east of Cape May at the entrance to Delaware Bay. The route of the Intracoastal Waterway from the north traverses Cape May Harbor, thence runs westward through the Cape May Canal to Delaware Bay. Cold Spring Inlet has been improved and is maintained by the United States under a project adopted in 1907 and modified in 1945. The project includes an entrance channel 25 feet deep protected by two parallel stone jetties. The shore for about 5,750 feet west of the inlet is Federally owned, being the frontage of an air and marine base of the United States Coast Guard. Thence westward lies Cape May City with a frontage of about 3 miles, of which about 55.5 per cent is publicly owned. The shore between Cape May City and Cape May Point is sparsely settled. A portion of the frontage is owned by the United States. The shore of Cape May Point is developed for private use. Cape May City has been a summer resort for over a century. It is extensively developed with residences, hotels, a boardwalk and recreational features. The summer population is about 20,000.

The instability of the shore west of Cold Spring Inlet has necessitated construction of protective works, despite which the beaches have deteriorated and portions of the shore have receded. Local interests stated that a beach sufficiently wide and flat for recreational and protective purposes existed along this frontage prior to improvement of Cold Spring Inlet.

The District and Division Engineers concluded that the most suitable plan of restoration and protection comprises groin construction and artificial placement of sand fill along a portion of the frontage of Cape May City.

The Beach Erosion Board concurred in the plan for groin construction and artificial placement of fill along a portion of the Cape May City frontage, except that it believed that a stockpile of beach material, deposited on the shore east of the area to be restored and replenished periodically, will probably maintain the restored shore without additional shore structures. Accordingly, it recommended a plan of restoration and protection of the shore of Cape May City from Wilmington Avenue to a point 3,300 feet west of Windsor Avenue comprising artificial placement of approximately 832,000 cubic yards of beach material on the shore to create a beach 100 to 200 feet wide above mean high water, artificial placement of 300,000 cubic yards of beach material on the adjoining 3,000 feet of shore to the east, construction of 5 new timber groins and extension of 5 existing stone groins, the groin construction and extensions to be deferred pending demonstration of need.

In accordance with statutory requirements, the Board stated its opinion that:

a. It is advisable for the United States to adopt a project for protection and improvement of the shore at Cape May City, New Jersey;

b. The public interest involved in the proposed work is substantial. It is associated with prevention of damages to public property and recreational benefits to the general public;

c. The United States should bear a share of the expense computed by totalling the cost applicable to protecting the Federally owned frontage plus 1/3 of the first cost of measures for the restoration and protection of the other publicly owned portions of the shore and based on the percentage of public ownership of the shore frontage at the time of completion of the work.

The Board recommended that a project be adopted by the United States authorizing Federal participation, subject to certain conditions, by the contribution of Federal funds in an amount equal to the portion of the cost applicable to protecting the Federally owned frontage plus 1/3 of the first cost of measures for the restoration and protection of other publicly owned portions of the shore of Cape May City between Wilmington Avenue and a point 3,300 feet west of Windsor Avenue.

The Chief of Engineers concurred in the recommendations of the Beach Erosion Board.

PRESQUE ISLE PENINSULA, PENNSYLVANIA

Presque Isle is located on the south shore of Lake Erie at Erie, Pennsylvania, about 78 miles southwest of Buffalo, New York, and 102 miles northeast of Cleveland, Ohio. The peninsula is a compound recurved sandspit projecting a maximum distance of about 2.5 miles from an otherwise straight mainland shore. From its root to its distal end, it has a lake shore line over six miles in length. The large landlocked bay between the peninsula and the mainland provides a spacious harbor which has been improved by the Federal government under the navigation project for Erie Harbor. The peninsula provides valuable protection to the harbor.

Presque Isle Peninsula is generally low in elevation except for beach ridges or dunes which rise to an average elevation of 20 feet above Lake Erie low-water datum. The spit varies in width from about 250 feet near its root or neck to a maximum of about $1\frac{1}{4}$ miles toward the distal end. The lakeward shore of the spit is, in general, a flat sandy beach except where the neck is protected by seawalls. Its regularity and continuity are broken only at points where protective works have altered the natural contour of the shore line. Presque Isle State Park comprising about 3,200 acres occupies practically the entire peninsula. The State has provided adequate access roads, but has left the area for the most part in its natural condition. The park is a popular area for bathing, boating, fishing and other outdoor forms of recreation. Its large attendance, totaling over 1,500,000 persons annually, is drawn mostly from western New York, Pennsylvania, and eastern Ohio. The public has free and unrestricted access to the park. The Erie City Water Works and U. S. Coast Guard also have installations on the peninsula. No pollution likely to endanger the health of bathers exists, except possibly on the eastern end of the spit near the harbor entrance, where bathing has been restricted at times by park officials. Future pollution along the lake shore is possible as residential development along the shore to the west increases.

The lake shore of the peninsula is exposed to wave attack from the southwest through north to northeast. The greater frequency and severity of storms from the westerly quadrant and the greater fetch in that direction cause a predominant eastward littoral drift. During the period of record the supply of beach material from bluffs and streams west of the spit has been insufficient to replace material eroded from the neck of the spit. Recession of the shore line has been greatest at the root of the peninsula gradually decreasing to a nodal point about two-thirds of the length of the peninsula from the root, from which point accretion has occurred as the eroded material was deposited in that area. On several occasions the narrow neck of the peninsula was breached by storm wave action. The earlier breaches were closed by natural processes. The Federal government closed a breach in 1920-1922 and since has built seawalls and bulkheads on the lake shore of the neck to preserve it and thus prevent the loss of protection it affords to Erie Harbor. After this portion of the shore was protected, the resulting decrease in material supply to more northerly portions increased the rate of recession of those areas. Successive northward extensions of protective bulkheads and groins were made by the State until such measures passed the former nodal point. The reduced supply of material moving alongshore caused the nodal point to move northward. The most northerly sections of bulkhead have been generally ineffective. Recession has continued and the highway in that area was destroyed in 1946.

The District and Division Engineers concluded that the most suitable plan of protection and improvement comprises the construction of a continuous sand beach created by artificial placement of fill, groins to reduce the rate of loss of the proposed sand fill and bulkheads to serve as a last line of defense in case of temporary loss of fill.

It was the opinion of the Board that the plan of protection and improvement for the neck of the peninsula should be designed to provide uninterrupted access to the main portion of the park. The proposed wider beach to be placed by artificial means was considered the most effective method of preventing wave action from throwing sand and water across the access road. In order to increase the volume of littoral drift, a feeder beach should be provided at the updrift end of the neck by additional widening. The past rate of erosion of this shore along the neck and the high cost of replacing beach material in this locality indicated the justification of groins to retard loss of the fill. Experience with the experimental groins constructed in 1943 indicated the ability of similar structures to minimize the loss of beach material. Their general accumulation and retention of trapped sand indicated that similar groins would stabilize the artificial beach. The Board believed that the seawall and bulkhead recommended by the District Engineer will provide insurance against breaching of the neck and interruption of traffic in case of a severe storm which might seriously erode the beach. They would also provide protection during the period after such a severe storm until replacement of the beach could be effected. The Board therefore concurred in the plan of protection for the neck recommended by the District and Division Engineers.

The Board found that the placement of beach material artificially and its retention by a groin system, as recommended by the district engineer, would be a complete and effective method of protection and improvement of the shore of the peninsula north of the low area on the Water Works Reservation. However, it noted that part of the area is in reasonably good condition, partially protection by a groin system. It believed that the shore north of the neck can be stabilized by supplying an adequate volume of littoral drift, provided the groin system in the area is filled initially. The Board considered that sand fill placed as a feeder beach at the low area of the Water Works Reservation and replenished as needed would provide adequate littoral drift to stabilize the shore north thereof. The Board believed that the feeder beach method would result in lower annual costs than the complete plan recommended by the District Engineer. Accordingly the Board considered that the proposed groins and bulkheads along this shore will not be essential, if the supply of material is restored, and as the consequences of temporary loss of beach fill would not be sufficiently serious to warrant the added expenditure. Therefore, the Board did not concur in recommending the groins and bulkheads in sections north of the neck.

In accordance with existing statutory requirements, the Board stated its opinion as follows:

a. it is advisable for the United States to adopt a project authorizing Federal participation in the cost of protecting the shore of Presque Isle Peninsula, Pennsylvania;

b. the public interest involved in the proposed improvement and protective measures results from elimination of damage to public property, the elimination of maintenance costs of existing protective works, and the recreational benefits that will accrue to the general public;

c. the share of the expense of the project which should be borne by the United States is one-third of the first cost of the work.

The Board recommended that a project be adopted by the United States authorizing Federal participation by the contribution of Federal funds in the amount of one-third of the first cost of the following measures for the protection and improvement of the shore of Presque Isle Peninsula. These measures comprise artificial placement of approximately 1,100,000 cubic yards of sand fill and construction of a seawall, bulkhead, and a groin system along the neck and for the remainder of the Peninsula placement of approximately 1,000,000 cubic yards of beach material as a feeder beach at the Water Works Reservation, distribution of approximately 400,000 cubic yards additional material in the remainder of the area and removal of portions of the Lighthouse jetty and of the existing bulkhead.

The Chief of Engineers concurred in the views and recommendations of the Beach Erosion Board.

OHIO SHORE LINE OF LAKE ERIE - VERMILION TO SHEFFIELD LAKE VILLAGE

The area studied is located in Lorain County on the south shore of Lake Erie from 23 to 37 miles west of Cleveland, Ohio. It lies between Vermilion and Sheffield Lake Village, a distance of about 14 miles. Lorain Harbor, which has been improved by the United States for navigation is located at the mouth of the Black River, in the eastern part of the study area.

Lorain County had a population of about 112,000 in 1940. The principal centers of population are the cities of Lorain and Elyria, which had a combined population of about 69,000. The property along the shore line of the study area has been developed mainly for private residential and recreational purposes. Inland areas are devoted mainly to agricultural uses. The shore is publicly owned at Waverly, Lakeview, Riverside and Century Parks in Lorain. All are open to the public. Lakeview and Century Parks have narrow beaches and bathing facilities. The shore at Elyria Water Works is also publicly owned, but it is protected by a stone breakwater and no beach exists at that locality.

The shore line of the study area consists principally of eroding bluffs of clay, silt and sand averaging 30 feet in height fronted by narrow beaches of sand and gravel. Shale outcrops in the bluff just east of Vermilion. Analysis of samples of bluff material indicated that in general not more than 12 per cent of the material is suitable for beach building. Erosion of the bluffs makes available a small volume of beach material. East of Vermilion River and Beaver Creek good beaches have formed by accretion caused by the jetties at the east sides of these streams. Miscellaneous groins and walls have been constructed in an ateempt to prevent erosion of the shore. Short groins have generally caused minor accretion on their east sides and have reduced recession of the bluffs to some extent. The accretion east of the jetties at Beaver Creek and Vermilion Harbor indicate a westward predominance of littoral drift. In the eastern part of the study area no marked predominance in direction of littoral drift is apparent.

The District and Division Engineers concluded that the most economical and practical general plans of protection for the privately owned shore line consist in grading, and draining of the bluffs, revetment of the toe of the slope, and maintenance of existing narrow beaches by means of short groins. They presented an alternative plan consisting of the same treatment of the bluff, and in lieu of the groins, the construction of a cellular steel sheet pile seawall. They recommended that owners of private property adopt the proposed plan of improvement best suited to the desired utilization of their shore front property. They further concluded that the best plans of protection and improvement for the proposed Beaver Creek State Park and Lakeview Park comprise artificial fill and groin construction, and in addition extension of the seawall at Lakeview Park.

The Board noted that the reporting officers recommended effective methods for protecting the shores of privately owned property. Typical protective measures include sloping and draining the bluff, protecting the toe of the slope by armor or bulkheads, and the construction of groins. The Board agreed that these measures will be entirely adequate, will interfere to a minimum degree with the recreational or other use of the shore, and will affect to a minimum degree its natural beauty. On the other hand the Board was of the opinion that protection against erosion could be attained by less elaborate and possibly less costly methods. In its simplest form this protection would consist of a continuous belt of heavy riprap or stone pavement covering the zone of destructive wave action and laid on a stable slope. The bluff behind this armored zone would not need to be sloped or drained. In that case however progressive recession of the top of the bluff would be expected until the slope of the bluff approximates that of the natural angle of repose of the material composing it. The groins may be omitted unless property owners desire to maintain beaches in front of their properties. The omission of groins will, however, probably result in the loss of beach in front of the belt of revetment or pavement. This will in turn result in an intensification of the wave attack at the toe of the belt of stone protection with consequent settlement of stone. The stone lost by such settlement should be replaced without delay to prevent breaching the protective belt.

The Board emphasized the necessity for complete uniform coverage of the zone to be protected from wave attack and the importance of insuring that the riprap of stone pavement be laid on a stable slope. The practice of dumping stone of miscellaneous sizes, broken concrete, other building materials, brush, etc. over the bluff is not normally an effective method of preventing erosion of the shore. The dumped material frequently fails to reach the zone of active wave attack in sufficient quantity to form an effective and continuous protective belt. The Board noted that the reporting officers proposed that dumped waste materials (stone, broken concrete, bricks, etc.) be utilized as protection for the Riverside Park and Waverly Park shore lines. In these two instances the use of such materials is open to the objections noted above and special care should be taken to insure complete and uniform coverage. The Board did not recommend dumping of miscellaneous materials as an effective or standard method of beach protection, but in these cases it did not object to it.

The Board called attention to the desirability of coordinated action by owners to protect a stretch of frontage under any plan of protection for privately owned shores, and the necessity of adequately protecting the ends of the work to prevent flanking. The Board recommended that private owners adopt one of the plans of protection presented by the District Engineer, or the plan without groins described above, selecting that most suitable to the desired use of their shore frontages. Existing Federal law includes no policy for Federal assistance in the cost of protecting privately owned shores, consequently no Federal participation in the cost of the foregoing work was recommended.

In accordance with existing statutory requirements, the Board stated its opinion that;

a. It is advisable for the United States to adopt projects authorizing Federal participation in the cost of protecting and improving the Lake Erie shore of Ohio at Lakeview Park in Lorain and at the proposed Beaver Creek State Park;

b. The public interest involved in the proposed measures at Lakeview Park is substantial. At the Beaver Creek State Park, the public interest will be substantial after acquisition of the shore by the State. These interests are associated with the direct damages to public property prevented and the recreational benefits to the general public;

c. The share in the cost which in the opinion of the Board should be borne by the United States is one-third of the first cost of construction for Lakeview Park, and for the proposed Beaver Creek State Park, one-ninth of the first cost of the proposed fill and in addition one-third of the first cost of the proposed groins.

The Board recommended that projects be adopted by the United States authorizing Federal participation, subject to certain conditions by the contribution of Federal funds toward the cost of the measures for the protection and improvement of the Lake Erie shore of Ohio as follows:

a. At the proposed Beaver Creek State Park, artificial placement of approximately 580,000 cubic yards of sand fill and construction of nine steel sheet piling groins, the Federal contribution to be 1/9 of the cost of the fill and 1/3 of the cost of the groins;

b. At Lakeview Park, Lorain, artificial placement of approximately 100,000 cubic yards of sand fill, construction of one new steel sheet piling groin and adjoining section of seawall and reconstruction and extension of 3 existing groins; the Federal contribution to be 1/3 of the cost thereof.

The Chief of Engineers concurred generally in the views and recommendations of the Beach Erosion Board, except for the project at Beaver Creek State Park. As the State of Ohio had formally notified the District Engineer subsequent to the Beach Erosion Board's report that the State project for a park in the vicinity of Beaver Creek had been abandoned because of excessive land and development costs, and as Federal participation in this proposed project was recommended by the Beach Erosion Board contingent upon prior acquisition of necessary lands by the State, the Chief of Engineers did not recommend adoption of a Federal project for the proposed Beaver Creek State Park.

WAIKIKI BEACH, ISLAND OF OAHU, T. H.

Waikiki Beach is located in Honolulu on the south shore of the island of Oahu between Diamond Head and the entrance to Honolulu Harbor. The study area comprised a shore frontage of about $3\frac{1}{2}$ miles. In addition to Waikiki Beach proper, it included the Ala Wai Yacht Harbor and Ala Moana Park shores at the northwestern end of the area. Honolulu had an estimated population of 245,500 in 1950. Waikiki Beach is the principal recreational center for the city and tourist trade. The additional seasonal population, consisting mainly of tourists numbered 42,000 in 1948. The tourist season includes practically the entire year. Highly developed residential, hotel, business, and park areas are contiguous to the shore area. Waikiki Beach proper includes the shore from the Elks Club to the Ala Wai Yacht Harbor. Two sections of this shore are publicly owned and occupied by Kapiolani and Kuhio Parks. Another section, Fort DeRussy, is owned by the United States and used as a recreational facility for service personnel. Northwest of Waikiki Beach proper are the Ala Wai Yacht Harbor and Ala Moana Park frontages, both publicly owned. The former is excluded for consideration pending development of the harbor. The mean tidal range at Waikiki is 1.24 feet and the extreme spring range is 4.2 feet. Higher tides are rare. Waves affecting the study area are caused by atmospheric disturbances at some distance from the Hawaiian Islands. Waves approach predominantly from the south-southwest. The average height of waves observed 2,300 feet offshore was 1.5 to 3 feet. Waves 4 to 5 feet high were observed on several occasions and during one storm reached a height of 11 feet. The energy of waves is rapidly dissipated as they cross the gently sloping coral shelf near shore. The direction of littoral drift is northwestward, but due to the change in shore alignment and the extensive reef, little material moves into the Ala Wai Harbor or Ala Moana Park areas. The Waikiki shore is subject to cyclical and seasonal onshore and offshore movements of sand. In its natural state the shore consisted of a continuous sand beach, the crest of which advanced and retreated with the shifting of material on and offshore. With the development and improvement of the upland areas, seawalls,were constructed to limit the retreat of the shore. Since construction of the seawalls the beaches deteriorated until only scattered remnants remained. Much of the Waikiki shore had no beach at any season. Beaches that remained were of inadequate width to survive erosional cycles and were at times completely washed away. The beach is the principal attraction for tourists visiting Oahu. The tourist trade is a major factor in the economy of the island. Existing beaches, even under the most favorable seasonal conditions, are inadequate to meet public demand. The Territorial Government desires a restored and protected beach at Waikiki to preserve and encourage tourist trade.

The District and Division Engineers and the Beach Erosion Board concluded that the most suitable plan for restoring and improving Waikiki

Beach comprised widening of the beaches to widths of 75 to 150 feet, constructing a terrace wall, 1,200 feet long , 2 groins and appurtenant drainage facilities, construction of the groins and 800 feet of the wall to be deferred pending demonstration of their need.

In accordance with existing statutory requirements, the Board stated its opinion that:

a. It is advisable for the United States to adopt a project authorizing Federal participation in the cost of protecting and improving the shores at Waikiki Beach;

b. The public interest involved in the proposed improvement is substantial. It is associated with prevention of damages to public property and recreational benefits to the general public; and

c. The share of the expense which should be borne by the United States is one-third of the first cost of the work applicable to the publicly owned shore.

The Chief of Engineers concurred in the recommendations of the Beach Erosion Board that a project be adopted by the United States authorizing the contribution of Federal funds in an amount equal to one-third of the first cost of the aforegoing work applicable to the publicly owned portions of the shore.

****

AUTHORIZED COOPERATIVE BEACH EROSION STUDIES

NEW HAMPSHIRE

HAMPTON BEACH. Cooperating Agency: New Hampshire Shore and Beach Preservation and Development Commission.

Problem: To determine the best method of preventing further erosion and stabilizing and restoring the beaches, also to determine the extent of Federal aid in any proposed plans of protection and improvement.

MASSACHUSETTS

PEMBERTON POINT TO GURNET POINT. Cooperating Agency: Department of Public Works.

Problem: To determine the best methods of shore protection, prevention of further erosion and improvement of beaches, and specifically to develop plans for protection of Crescent Beach, the Glades, North Scituate Beach and Brant Rock.

CONNECTICUT

STATE OF CONNECTICUT. Cooperating Agency: State of Connecticut (Acting through the Flood Control and Water Policy Commission).

Problem: To determine the most suitable methods of stabilizing and improving the shore line. Sections of the coast are being studied in order of priority as requested by the cooperating agency until the entire coast has been included.

NEW YORK

N. Y. STATE PARKS ON LAKE ONTARIO. Cooperating Agency: Department of Conservation, Division of Parks.

Problem: To determine the best method of providing and maintaining certain beaches and preventing further erosion of the shores at Selkirk Shores, Fair Haven Beach and Hamlin Beach State Parks, and the Braddock Bay area owned by the State of New York.

NEW JERSEY

STATE OF NEW JERSEY. Cooperating Agency: Department of Conservation and Economic Development.

Problem: To determine the best method of preventing further erosion and stabilizing and restoring the beaches, to recommend remedial measures, and to formulate a comprehensive plan for beach preservation or coastal protection.

NORTH CAROLINA

CAROLINA BEACH. Cooperating Agency: Town of Carolina Beach

Problem: To determine the best method of preventing erosion of the beach.

ALABAMA

PERDIDO PASS AND ALABAMA POINT. Cooperating Agency: Alabama State Highway Department.

Problem: To determine the best method of preventing further erosion of Alabama Point, for stabilizing the inlet, and for determining the extent of Federal aid, if any, in the cost of such proposed plans for protection and improvement as may be recommended.

LOUISIANA

GRAND ISLE. Cooperating Agency: Department of Public Works, State of Louisiana.

Problem: To determine the best methods of preventing further erosion of the beaches along the Gulf shore of Grand Isle.

CALIFORNIA

STATE OF CALIFORNIA. Cooperating Agency: Division of Beaches and Parks, State of California.

Problem: To conduct a study of the problems of beach erosion and shore protection along the entire coast of California. The current studies cover the Santa Cruz and San Diego areas.

WISCONSIN

KENOSHA. Cooperating Agency: City of Kenosha.

Problem: To determine the best method of shore protection and beach erosion control.

TERRITORY OF HAWAII

WAIMEA & HANAPEPE, KAUAI. Cooperating Agency: Board of Harbor Commissioners, Territory of Hawaii.

Problem: To determine the most suitable method of preventing erosion, and of increasing the usable recreational beach area, and to determine the extent of Federal aid in effecting the desired improvement.

The third national conference on Coastal Engineering, sponsored jointly by the Council on Wave Research and the Massachusetts Institute of Technology, with support also given by the Northeastern Section of the ASCE, the Boston Society of Civil Engineers and the Boston Section of the ASME, was held at Boston, Massachusetts in October 1952. The Proceedings of this conference published by the Council on Wave Research as well as those of the two earlier conferences (see Bulletin of the Beach Erosion Board, Vol. 6, Nos. 1 and 4), may be ordered from Professor J. W. Johnson, Waves Council, 245 Hesse Hall, University of California, Berkeley 4, California. A fourth conference will be held in Chicago on 29-31 October 1953.

The contents of the proceedings of the third conference are as follows:

Part I
BASIC OCEANOGRAPHIC INFORMATION

Part 2
COASTAL SEDIMENT PROBLEMS

Part 3
DESIGN OF COASTAL WORKS

Part 4
FACTORS AFFECTING THE LIFE OF COASTAL STRUCTURES

Part 5
CASE HISTORIES OF COASTAL PROJECTS