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CONTENTS

CONTENTS

CONTENTS

PART 4

DESIGN OF SHORELINE STRUCTURES

Part 1

BASIC INFORMATION FOR SHORELINE INVESTIGATION

CHAPTER 1

HINDCAST WAVE STATISTICS FOR THE GREAT LAKES

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The General Investigations program of the Beach Erosion Board comprises investigations, regional rather than local in scope, designed to improve, simplify, and expedite the solution of local problems, by giving a compilation of all existing data pertinent to shore processes in the particular region. As a first step in the compilation of these data, a study of wave and lake level conditions on the Great Lakes is being made. The results of such studies for Lake Michigan, Lake Erie, and Lake Ontario have recently been completed and published as Technical Memorandums of the Beach Erosion Board (Saville, 1953).

Five stations on Lake Michigan, four stations on Lake Erie, and three stations on Lake Ontario were selected for a comprehensive wave analysis, the locations being as shown in Figure 1. These particular stations were selected since it was thought that they would give adequate coverage to the entire United States' shore of the lakes, and permit interpolation of values between stations, thus enabling one to obtain an accurate representation of wave action at any point along the lakes' shore.

Wave characteristics were hindcast from synoptic weather charts for each station for the three-year period 1948-1950. The weather maps used were the United States Surface Synoptic Charts compiled at six-hour intervals by the U. S. Weather Bureau. Fetch areas, and the wind speeds and durations in these areas, were determined directly from the weather maps; these values were used with the curves derived by Sverdrup and Munk (1947) and revised by Arthur (1947) to obtain the hindcast wave characteristics. The revisions in methods recently suggested by Bretschneider (1951) were not employed; hence the wave periods determined may be expected to be slightly low. The only major variation from the usual methods of wave forecasting or hindcasting (Hydrographic Office, 1951) was that the surface wind was determined directly from reported observations rather than from a gradient wind determined from the isobar spacing. It was thought that with the lake area so small in comparison to the area of the pressure cell, the isobaric pattern on the surface would be influenced to a large extent by the surface topography, and gradient winds determined from the isobar spacing would not necessarily give true values of wind velocity over the lake surface. Hence reported values of the surface wind could be expected to give a more realistic figure of the wind velocity. Observations have shown (U. S. Weather Bureau, 1951) that the greater surface friction over land serves to reduce the wind over land from what it may be over water. Since the reported values were almost always obtained at land stations, the wind speeds used in the analysis may have been lower than those actually occurring over the lake in the generating area. Some compensation was made for this by selecting the top speed of the Beaufort range reported rather than the middle value.

The wave characteristics thus determined are for the significant wave -- that is, the period is that of the predominating waves, and the height is the average of the higher one-third of these predominant waves. It should also be noted that these wave conditions are deep water conditions, and must be used in conjunction with refraction diagrams to obtain inshore values. The values obtained have been summarized in tabular form in the Beach Erosion Board Technical Memorandums. These tables show, for each station, the number of hours' duration that deep water waves of any given height, period and direction occurred during any month of the three-year period; and also for each month (as summations) the number of hours' occurrence of waves of any particular height and period exclusive of direction; the number of hours' occurrence of waves of any particular height and direction exclusive of period; and the total number of hours' occurrence of waves of any particular height. In addition tables summarizing these values for an entire year (rather than a single month) are also shown -- as in Table 1.

As an example of the data presented, from Table 1 (for the station at Milwaukee, Wisconsin), waves of 2 to 3-foot height and 3 to 4-second period from the east were hindcast to occur for 72 hours during 1948, 84 hours during 1949, and 54 hours during 1950. Thus, waves of this category were hindcast to occur for a duration of 210 hours during the three-year period and hence can be expected to occur for about 70 hours (on the average) during any year in the future. Waves of 2 to 3-foot height and 3 to 4-second period (from all directions) were hindcast to occur for 1,458 hours over the three-year period, or an average of 486 hours per year. Waves of 2 to 3-foot height from the east (all periods) were hindcast to occur for 306 hours over the three-year period, or an average of 102 hours per year. Waves of 2 to 3-foot height (all periods and all directions) were hindcast to occur for 2,508 hours over the three-year period, or an average of 836 hours per year.

During much of the winter season, portions of the lakes are covered with ice, and fetch areas are limited considerably. In addition, for a somewhat greater portion of the winter season, the coast areas of the lakes are covered with ice, and, even though waves are generated in offshore areas, they never reach the shore, being interrupted by the ice around the rim of the lake. No account of this effect of the ice was taken in the actual hindcasting of the waves, and the durations given for the various winter months are computed as

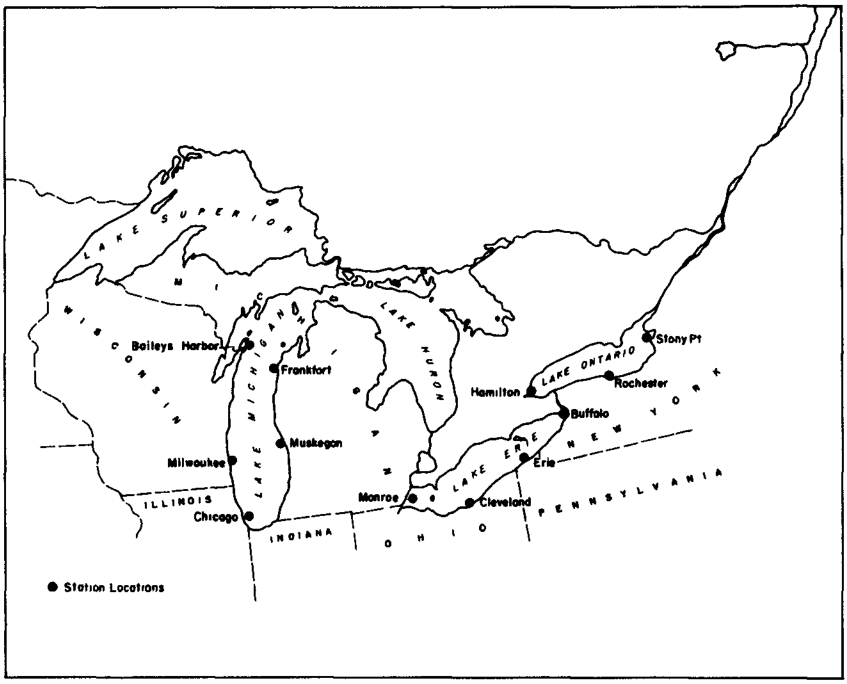

Fig. 1. Location map

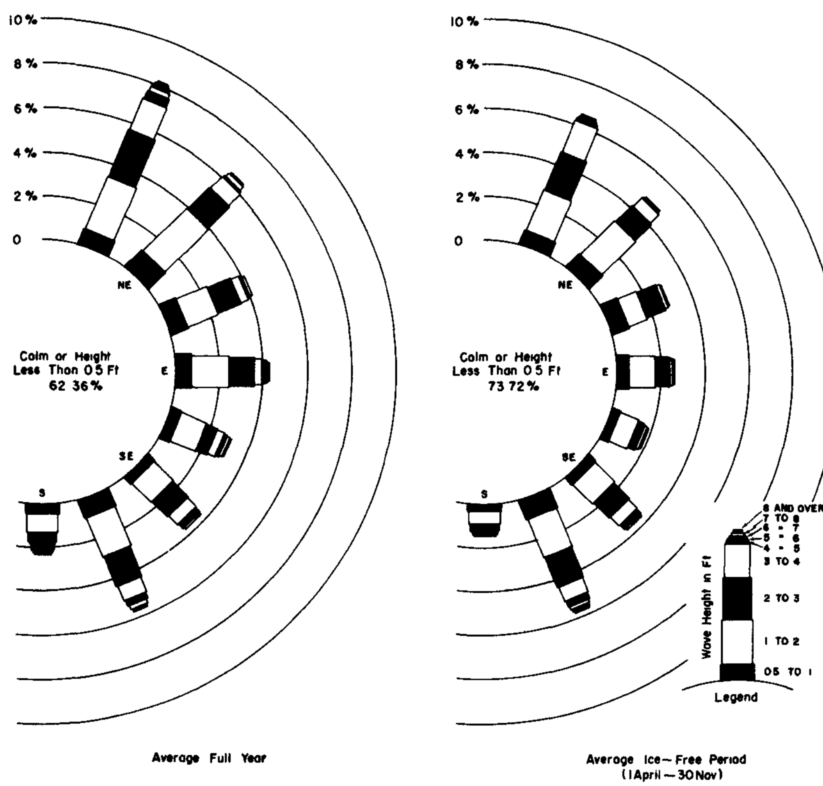

Fig. 2. Deep water wave roses for Milwaukee, Wisconsin.

Table 1.

STATISTICAL HINDCAST DATA FOR LAKE MICHIGAN, STATION 8, MILWAUKEE, WISCONSIN
Durations given in hours Height and period groupings include lower value but not the upper
FULL YEAR

Height (feet)	Period (sec.)	1-2				2-3				3-4				4-5				5-6				6-7				Totals			
		1948	1949	1950	Total	1948	1949	1950	Total	1948	1949	1950	Total	1948	1949	1950	Total	1948	1949	1950	Total	1948	1949	1950	Total	1948	1949	1950	Total
0.5-1	NNE	42	54	42	138	12		18	30	6	6	12	24													60	60	72	192
	NE	24	48	54	126	36	48	54	138		12	6	18													60	108	114	282
	ENE	36	42	42	120	18	30	54	102																	54	72	96	222
	E	30	36	18	84	12	48	48	108																	42	84	66	192
	ESE	30	6	30	66	6	54	6	66																	36	60	36	132
	SE	30	24	6	60	12	24	18	54																	42	48	24	114
	SSE	36	54	54	144	6	60	24	90		6		6													42	120	78	240
	S	48	24	42	114	6			6			6	6													54	24	48	126
	Total	276	288	288	852	108	264	222	594	6	24	24	54													390	576	534	1500
1-2	NNE	12	12	30	54	180	132	120	432	54	18	72	144	6	36	12	54									252	198	234	684
	NE	12	24		36	96	144	204	444	78	90	84	252		6		6									186	264	288	738
	ENE	18	18	24	60	48	84	78	210	30	42	60	132													96	144	162	402
	E	24	6		30	102	120	78	300	54	42	24	120													180	166	102	450
	ESE	6	18		24	66	72	60	198	30	24	54	108													102	114	114	330
	SE	6	12		18	84	84	42	210	30	54	12	96													120	150	54	324
	SSE		12	6	18	144	114	114	372	36	60	24	120													180	186	144	510
	S	18	6	12	36	60	36	54	150	12	12	12	36													90	54	78	222
	Total	96	108	72	276	780	786	750	2316	324	342	342	1008	6	42	12	60									1206	1278	1176	3660
2-3	NNE	6		6	12	60	72	90	222	66	66	90	222	30	54	84	168									162	198	270	624
	NE					42	12	48	102	72	84	96	252	6	6	6	18									120	102	150	372
	ENE			6	6	18	6	24	48	66	78	90	234			6	6									84	84	126	294
	E			6	6	18	36	36	90	72	84	54	210													90	120	96	306
	ESE					6	12	12	30	36	18	60	114													42	30	72	144
	SE					18	42	24	84	84	72	24	180													102	114	48	264
	SSE					48	96	36	180	42	102	60	204		6		6									90	204	96	390
	S					36	18	18	72	6	18	18	42													42	36	36	114
	Total	6		18	24	246	294	288	828	444	522	492	1458	36	66	96	198									732	882	894	2508
3-4	NNE						6	12	18	90	66	72	228	42	48	60	150									132	120	144	396
	NE									78	30	42	150	6	6	24	36									84	36	66	186
	ENE					6	6		12	30	24	18	72	6			6									42	30	18	90
	E									54		36	90													54		36	90
	ESE							6	6	30	30	36	96													30	30	42	102
	SE						6		6	66	12	24	102	18	6	24	48									84	24	48	156
	SSE					12	6		18	48	72	42	162	12	6		18									72	84	42	198
	S						6	6	12		12	12	24														18	18	36
	Total					18	30	24	72	396	246	282	924	84	66	108	258									498	342	414	1254
4-5	NNE									12	6	18	36	18	6	18	42	12		18	30					42	12	54	108
	NE									12			12	12	6		18									24	6		30
	ENE									6	6	18	30	12			12									18	6	18	42
	E									18	6	6	30			18	18									18	6	24	48
	SSE									6	6	12	24			12	12									6	6	24	36
	SE									30	18	6	54			24	24									30	18	30	78
	SSE									30	18	6	54	12		6	18									42	18	12	72
	S									12	30	12	54											6	6	12	30	18	60
	Total									126	90	78	294	54	12	78	144	12		18	30			6	6	192	102	180	474
5-6	NNE									18	6		24	6	12		18		6	12	18					24	24	12	60
	NE											6	6	12	12	12	36	6		6	12					18	12	24	54
	ENE									6			6		6		6									6	6		12
	E									6			6	6			6									12			12
	ESE									6			6	18	12		30									24	12		36
	SE													6		12	18									6		12	18
	SSE									12	6		18	12	12	6	30									24	18	6	48
	S										6	6	12														6	6	12
	Total									48	18	12	78	60	54	30	144	6	6	18	30					114	78	60	252
6-7	NNE														6	6	12		6		6			12	12		12	18	30
	NE															6	6											6	6
	ENE														6	6	12										6	6	12
	E															6	6											6	6
	ESE													6			6									6			6
	SSE													18		6	24		6		6					18	6	6	30
	S													6	6		12									6	6		12
	Total													30	18	30	78		12		12			12	12	30	30	42	102
7-8	NNE															6	6		6	6	12						6	12	18
	NE														6		6										6		6
	ENE													6			6									6			6
	E															6	6											6	6
	ESE													6			6									6			6
	SE															6	6											6	6
	S														6		6	6			6					6	6		12
	Total													12	12	18	42	6	6	6	18					18	18	24	60
8-9	NNE													18			18									18			18
	NE													6			6									6			6
	E													6			6									6			6
	SSE															6	6											6	6
	S													6			6									6			6
	Total													36		6	42									36		6	42
9-10	S													6			6									6			6
10-11	NNE																	6			6	6				12			12
	NE																	6			6					6			6
	Total																	12			12	6			6	18			18
11-12	SSE																	6			6					6			6
12-13	E																	6			6					6			6
14-15	NNE																	6			6					6			6
15-16	NNE																					6			6	6			6
Totals		378	396	378	1152	1152	1374	1284	3810	1344	1242	1230	3816	324	270	378	972	54	24	42	120	12		18	30	3264	3306	3330	9900
Calm																										5520	5454	5430	16404
Hours in year																										8784	8760	8760	26304

though there were no ice on the lake, a fact that should be remembered in using these data.

From yearly records of lake and air temperatures, and the dates of the opening and closing of the lakes for navigation, an average ice-free period was determined for each station. For the three lakes considered this ran, on the average, from April through November, but varies somewhat from station to station. Tabulations of the wave data for this ice-free period were also made and presented in a form similar to that of Table 1.

The durations of waves of particular height and direction have also been tabulated as percentages of time for the three-year period and are shown graphically in wave roses for both the full year and the ice-free period. Sample roses for the station at Milwaukee are shown in Figure 2. It may be noted that, for convenience, as in other computations involving the ice-free year, 100 percent of the time represents 365 days rather than the actual number of days in the ice-free period.

Figures such as that shown in Figure 3 were drawn for each station showing the total percentages of time that the wave height may be expected to be greater than any particular height throughout the year. They thus show the (average) total duration time of specific waves over the year. Two curves are shown in each figure, one based on the data gathered for the entire year's period, and the other on just the average ice-free period (April through November). For example, from Figure 3, at the Milwaukee station the total duration of waves in excess of 10 feet in height during the ice-free period is expected to be 0.055 percent of the time; and 0.214 percent of the time during the full year. Hence waves 10 feet or higher can be expected to occur for a total duration of 19 hours (0.00214 x 365 x 24) over the course of each year, and, of this, 5 hours (0.00055 x 365 x 24) will be during the ice-free portion of the year when the waves will be certain to reach the shore.

Frequency curves, such as Figure 4, were also drawn, showing the frequency with which storms resulting in waves higher than a given height can be expected to occur. For example, from Figure 4, at the Milwaukee station on 1.10 percent of the days each year the waves may be expected to be ten feet or greater in height, and on 0.23 percent of the days they may be expected to reach this height during the ice-free portion of the year. Thus waves ten feet or higher may be expected to occur (on the average) four times each year (0.011 x 365); of these four occurrences, only one (0.0023 x 365) will be expected to occur during the ice-free portion of the year.

Combining the data obtained from the graphs on Figures 3 and 4, waves ten feet and higher may be expected to occur at the Milwaukee station about four times each year, and the average duration of each storm will be about five hours. During the ice-free portion of the year, waves of ten feet and higher may be expected to occur only once, and the duration of this storm is also expected to be about five hours.

There are, in general, two methods of plotting points to obtain frequency curves such as those shown in Figure 4. One, based on the so-called theory of sampling, involves the assumption that the known period of record (three years) is a fair average sample of all similar three-year periods over an infinite number of years, and that therefore the largest storm of this three-year period is the median of all storms of the same class in all other three-year periods. This results in a frequency given by the equation:

$$F = \frac{2N - 1}{2T} \times 100$$

where F = frequency (in percent) of the occurrence of storms equalling or exceeding the given storm

T = number of days of record

N = number of occurrences of a storm equal to or greater than the given storm

The second method essentially considers only the period of record, in which case the frequency becomes

$$F = \frac{N}{T} \times 100$$

Values of F are the abscissas of points on the frequency curve. Using the second equation above, the largest storm which occurred in the known three-year period would have an abscissa of 0.0914 percent and would represent the storm which would most probably occur once in three years, i.e., would be the "three-year storm". But this would be contrary to the theory of sampling, where (above) the assumption is made that the largest storm in the known three-year period was the median of the largest storms in a long succession of three-year periods. Therefore, over a long period such as 300 years, it will be exceeded not 100 times, but 50 times; i.e., it is by definition not a "3-year storm", but a "6-year storm".

Either of the above two equations could be, and have been, used to prepare frequency curves. Although the former is the one most generally used for hydrologic data, the latter method has been used in this case. The use of this formula ($F = 100N/T$) will result in somewhat more conservative interpretation of the data, and was thought justified in view of the extremely short period of record (3 years).

The points plotted may be represented fairly closely by a straight line curve. Actually the published curves were not always drawn as the lines of best fit, but somewhat more weight was sometimes placed on the higher values. This again tends to give a somewhat more conservative curve, but was thought warranted in most cases.

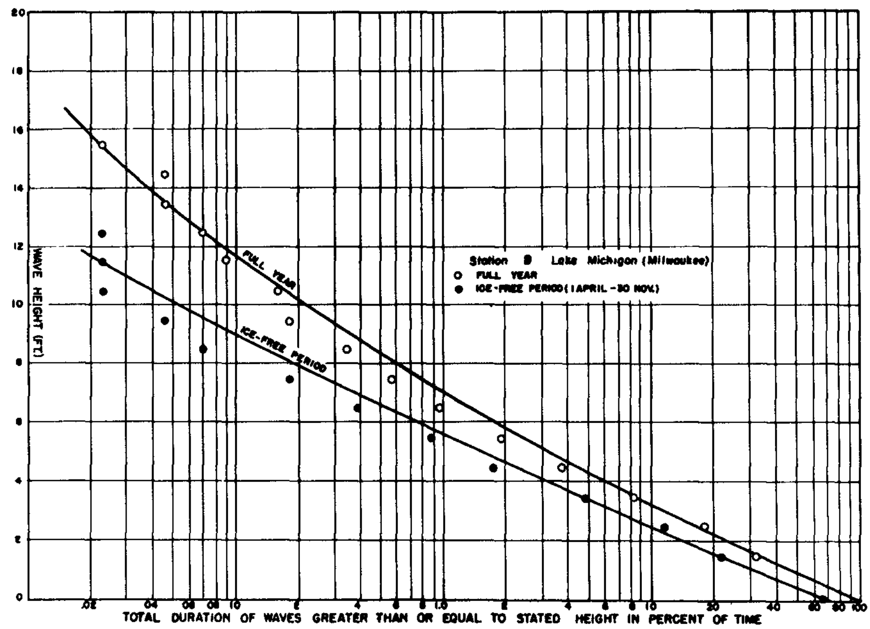

Fig. 3. Duration curves for deep water waves off Milwaukee, Wisconsin.

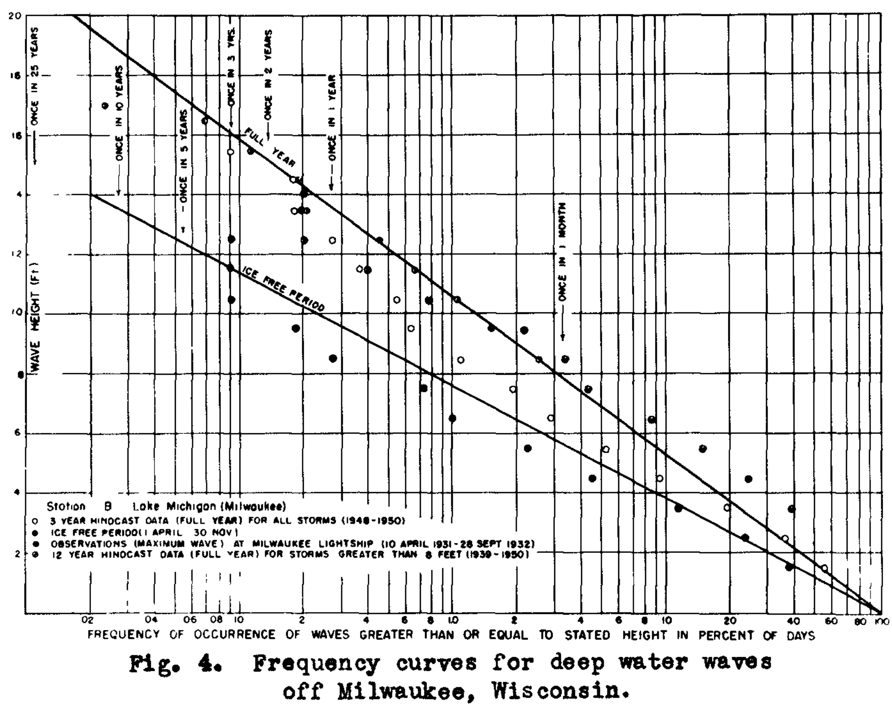

Fig. 4. Frequency curves for deep water waves off Milwaukee, Wisconsin.

In view of the shortness of the period of record, some doubt arose as to the validity of extrapolation from these curves, and as to whether the three-years chosen were representative (i.e., that they represent average conditions, and not three years of abnormally high, or low, waves). Hence hindcasts were made for the Milwaukee station for a period of 12 years (1939-1950) for all storms which were expected to give waves greater than 8 feet. The points determined are also shown in Figure 4. These points fit a straight line curve very closely and, though the points mostly lie somewhat above those determined from the three-year data, the curve is not greatly different from that which would have been drawn from the three-year data (where the greater weight was placed in the higher values). It is interesting to note that though the maximum storm hindcast for the 12-year period was almost 17 feet, three storms were hindcast in the 16 to 17-foot grouping.

Observations of the "average maximum" wave were obtained by the Milwaukee District of the Corps of Engineers (1933) at the Milwaukee lightship over the period of 10 April 1931 to 28 September 1932 and these points are also shown in Figure 4. While the exact correspondence between the significant waves hindcast and the "average maximum" waves observed is not known, values should be closely comparable -- and although the observed points lie somewhat higher for the lower waves, agreement is good for the higher waves. From the above, it is thought that reasonable confidence can be put in the curves obtained, at least for values of the waves occurring with frequencies less than about once in 10 years.

Although for structural design purposes, the important factor is the size of the maximum probable wave (within a certain time period), for computations involving sand movement and littoral drift, a more desirable parameter would be some averaged factor including within it the effect of both height and period, the variation of these parameters, and the duration that waves of each particular category exist. Present day knowledge indicates that sand movement by wave action is best correlated with the amount of energy transmitted forward (and eventually on to the beach) by the waves. The total energy per unit width in each wave is, in deep water

$$E_o = \frac{wLH^2}{8} \left[1-4.93 \left(\frac{H}{L}\right)^2 \right] = \frac{wg}{16\pi} H^2T^2 \left[1-4.93 \left(\frac{H}{L}\right)^2 \right]$$

where w = unit weight of water = 62.4 lbs./cu. ft.
g = acceleration due to gravity = 32.2 ft./sec./sec.
H = wave height (ft.)
T = wave period (sec.)
L = wave length (ft.)

One-half of this energy is transmitted forward from deep water toward the shore, and it is this amount of energy that eventually reaches the shore line. The total energy transmitted forward in any given period of time (E_T) is then $E_o/2$ times the number of waves occurring

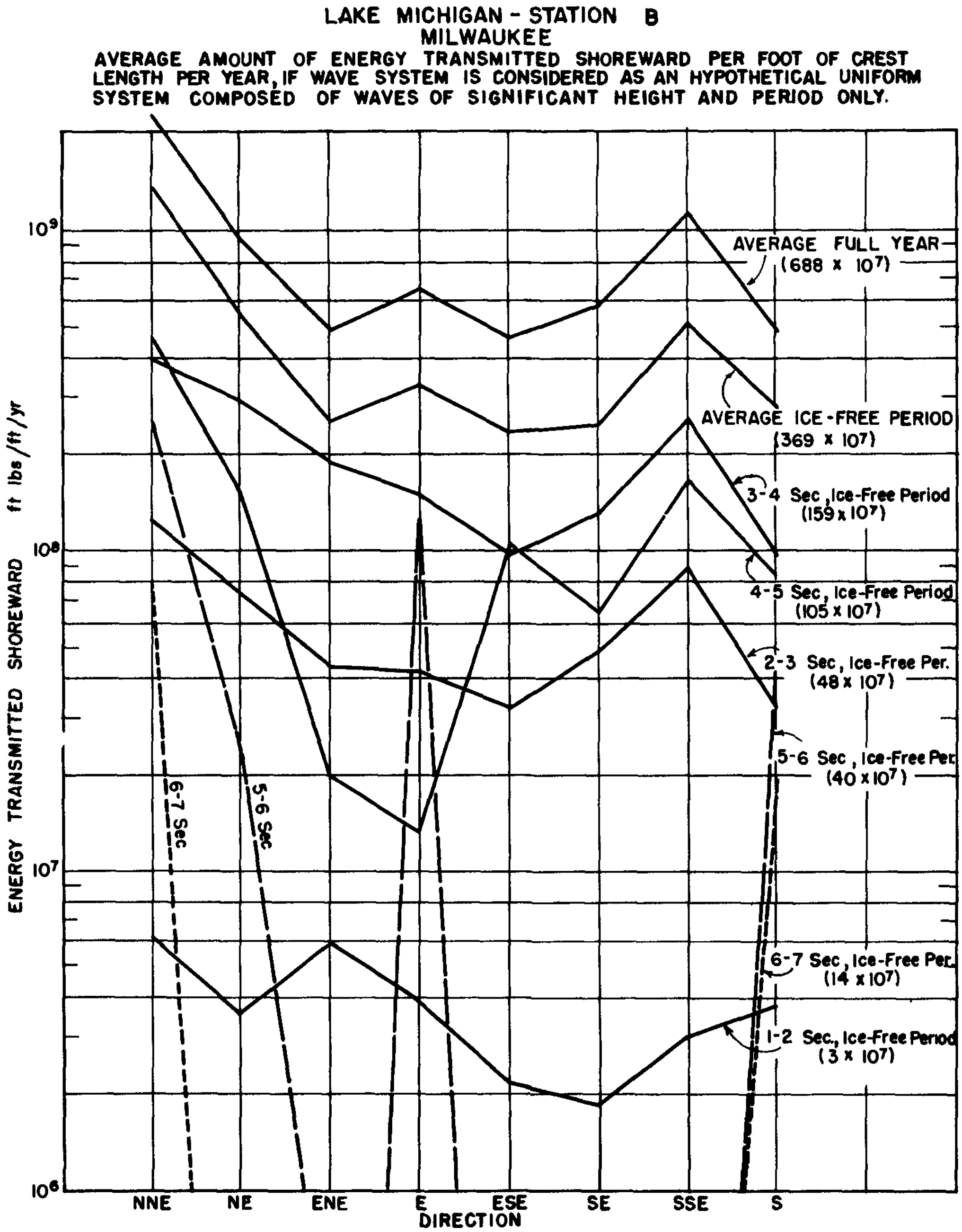

Fig. 5. Deep water energy curves for Milwaukee, Wisconsin.

in that period of time, and

$$E_T = \frac{E_o}{2} \frac{(3600t)}{T} = 7.195 \times 10^4 H^2 T t \left[1 - 4.93 \left(\frac{H}{L}\right)^2 \right]$$

where t is the duration of the waves in hours. If some particular time interval (say, one year) is considered during which waves of varying height and period pass a given point toward shore, then the heights and associated periods may be tabulated and there will be n groups. If the height of the ith group is represented by its class mark H_i, and the wave period in that group denoted by T_i, and the duration of the group by t_i, then the total amount of energy transmitted forward during the entire time interval is

$$E_T = E_{T1} + E_{T2} + E_{T3} + \ldots\ldots + E_{Ti} + \ldots\ldots E_{Tn}$$

and

$$E_T = 7.195 \times 10^4 \sum_{i=1}^{i=n} H_i^2 T_i t_i \left[1 - 4.93 \left(\frac{H_i}{L_i}\right)^2 \right]$$

For each station a tabulation was made of the average energy transmitted forward from deep water toward the shore in each category of height, period, and direction during both the average ice-free period and the entire year. Table 2 shows such a tabulation for Milwaukee for the full year. Since the values in the original tables (as Table 1) represent significant wave height and period, these energy values are those obtained if the wave system is uniform and consists only of waves of significant height and period. Wave trains in nature are, however, exceedingly irregular, and have less energy than that determined by the significant wave concept. The relationship between the actual energy contained in a given wave train and that computed from the significant wave has been examined somewhat by personnel at Scripps Institution of Oceanography (1947) and more recently by Barber (1950) and Darbyshire (1952), and has been found to be very nearly a constant ratio (on the order of 0.58). The energies given, therefore, may be considered to be the true value of the energy multiplied by some nearly constant value, and hence can be used to determine quite accurately ratios of energies from different directions. These latter represent very closely the ratios of the drift-producing forces. Summations of these energies for each direction and period were shown graphically in figures similar to Figure 5.

In utilizing the data from these three publications of the Beach Erosion Board it must be remembered that all the wave data given refer to deep water conditions -- that is, depths greater than one-half the wave length. As such, interpolation between stations to obtain values for other points along the shore is quite valid, and it is felt that adequate deep water hindcast values may be thus obtained

Table 2.

ENFROY TABULATION FOR AVERAGE YEAR FOR LAKE MICHIGAN, STATION B, MILWAUKEE, WISCONSIN
Energies in foot- pounds per foot of crest per year x 10^{-4}. Height and period groupings include upper value but not the lower

FULL YEAR

Height (feet)	Period (sec.)	1-2	2-3	3-4	4-5	5-6	6-7	Total
NNE	NNE	273	101	113				487
	NE	250	464	85				799
	ENE	238	343					581
0.5-1	E	166	363					529
	ESE	131	222					353
	SE	119	181					300
	SSE	285	302	28				615
	S	226	20	28				274
	Total	1688	1996	254				3938
	NNE	401	5767	2712	1310			10190
	NE	267	5927	4745	146			11085
	ENE	445	2804	2486				5735
1-2	E	223	4005	2260				6488
	ESE	178	2643	2034				4855
	SE	134	2804	1808				4746
	SSE	134	4966	2260				7360
	S	267	2003	678				2948
	Total	2049	30919	18983	1456			53407
	NNE	207	8070	11555	11298			31130
	NE		3708	13117	1211			18036
	ENE	104	1745	12180	404			14433
2-3	E	104	3272	10931				14307
	ESE		1091	5934				7025
	SE		3053	9369				12422
	SSE		6543	10618	404			17565
	S		2617	2186				4803
	Total	415	30099	75890	13317			119721
	NNE		1244	23087	19721			44052
	NE			15189	4733			19922
	ENE		829	7291	789			8909
3-4	E			9113				9113
	ESE		415	9721				10136
	SE		415	10329	6311			17055
	SSE		1244	16404	2366			20014
	S		829	2430				3259
	Total		4976	93564	33920			132460
	NNE			5965	9094	7988		23047
	NE			1988	3897			5885
	ENE			4971	2598			7569
	E			4971	3897			8868
4-5	ESE			3977	2598			6575
	SE			8948	5196			14144
	SSE			8948	3897			12845
	S			8948			1892	10840
	Total			48716	31177	7988	1892	89773
	NNE			5864	5795	7138		18797
	NE			1466	11591	4759		17816
	ENE			1466	1932			3398
5-6	E			1466	1932			3398
	ESE			1466	9659			11125
	SE				5795			5795
	SSE			4398	9659			14057
	S			2932				2932
	Total			19058	46363	11897		77318
	NNE				5367	3315	7869	16551
	NE				2683			2683
	ENE				5367			5367
6-7	E				2683			2683
	ESE				2683			2683
	SSE				10732	3315		14047
	S				5367			5367
	Total				34882	6630	7869	49381
	NNE				3549	8801		12350
	NE				3549			3549
	ENE				3549			3549
7-8	E				3549			3549
	ESE				3549			3549
	SE				3549			3549
	S				3549	4401		7950
	Total				24843	13202		38045
	NNE				13571			13571
	NE				4524			4524
8-9	E				4524			4524
	SSE				4524			4524
	S				4524			4524
	Total				31667			31667
9-10	S				5602			5602
10-11	NNE					8529	10193	18722
	NE					8529		8529
	Total					17058	10193	27251
11-12	SSE					10182		10182
12-13	E					11969		11969
14-15	NNE					15923		15923
15-16	NNE						21904	21904
	Totals	4152	67990	256465	223227	94849	41858	688541

for essentially all points on the United States' shores of Lakes Michigan, Erie and Ontario. Standard refraction techniques may then be used to obtain inshore data from this deep water data. While the production of wave statistics by the hindcast technique is still admittedly of undeterminate quantitative accuracy for inland waters such as the Great Lakes, it is thought that these data will nevertheless provide the engineer with better wave data than have heretofore been available.

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CHAPTER 2

STATISTICAL ANALYSIS OF WAVE RECORDS

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ABSTRACT

The establishment of quantitative relationships between recorded wave-system characteristics and other phenomena requires numerical description of the wave record. Current concepts applied to time histories of wave activity at a point are discussed. Characteristic statistical regularities found in wave measurements are described. Examples given show the application of statistical techniques to the description of wave systems in terms of the distribution of spectral energy as well as the distributions of "individual" wave heights and periods. Results from the prediction, from one point to another, of surface time histories illustrate the application of approximate spectral information.

INTRODUCTION

In the study of waves and their interaction with their environment, there has been a need for effective description of observed wave systems. Convenient data (Snodgrass, 1951, 1952) is provided by the recorded time history, taken above some fixed point on the bottom, of the fluctuating pressure below the surface or of the water level at the surface. It will be understood that the length of a wave record selected for analysis is long relative to the periods of the fluctuations of interest, but short compared to the variations in meteorological conditions.

The questions we are concerned with are: What information is contained in a wave record? And how may this information be conveniently extracted? We shall first describe the statistical regularities suggesting a particular mathematical model (Tukey, 1950; Pierson 1952a) found useful for interpreting wave-record data. The two complementary aspects of this model each involve a distribution - one a statistical distribution of wave-record ordinates, the other a spectral distribution of energy. Theoretical relations connecting various model parameters will be illustrated with actual data. Although the experimental results to be presented have been obtained with pressure waves in the ocean for the most part, many of the methods and results are applicable to other kinds of data.

In the lower half of Fig. 1 is shown a short segment of a typical wave record, giving the pressure fluctuation 64 feet below the surface off the California coast. The total time interval shown represents about four and one-half minutes, during which from twenty to twenty-five waves pass the recording point. It will be seen that on the time-history curve the apparent slope and curvature, as well as the ordinate, vary irregularly from one instant to the next and bear little obvious relation to one another. The most noticeable feature may be the tendency for curvature and ordinate, measured from its average level, to have opposite signs.

The upper half of Fig. 1 shows a sample segment generated by the theoretical model in which suitable parameters were chosen to yield an artificial wave record comparable to the observed one.

DISTRIBUTIONS OF ORDINATES AND DERIVED QUANTITIES

Suppose a horizontal line is drawn cutting the wave-record curve at an arbitrary level. The fraction of the time the curve spends below such a line is a function (Birkhoff and Kotig, 1953; Putz, 1953b) increasing with the height of the line known as the distribution function for the curve. A plot of this function for a typical twenty-minute wave record is shown in Fig. 2. Here the number on the vertical scale represents the arbitrary chart level, the horizontal scale, the percent of time that the curve lies below that level. The notion of the probability of finding the curve below a given level may be introduced if one thinks of choosing at random an instant of time on the chart. The location of the plotted points on an approximate straight line is characteristic (Putz, 1953b) and results here from the choice of the horizontal scale which is the familiar Gaussian-distribution or normal-probability scale.

The interesting thing about wave records is the apparent ability of the normal probability scale to rectify very nearly not only the distribution curves for the ordinate and the first derivative, which have been experimentally checked(Rudnick, 1951; Putz, 1953b), but also, according to the theory, the distribution curves for derivatives of all orders. In each case the mean value below which the curve or derivative spends just fifty percent of its time will be zero. For the ordinates, this will be true because of our convention of measuring them from their mean. Since in this way one point is fixed on each curve, the straight-line plots for the various derivatives of the wave-record will differ among themselves only in their slopes. A convenient measure of the slope is the difference between the height of the line at the 84.1 percent level and its height at the 50 percent level. This distance, called the standard deviation, or root-mean-square (r.m.s.) ordinate, when measured for the k^{th} derivative, is denoted by σ_k .

The ordinate distribution concept may be extended by considering more than one ordinate at a time. Given n arbitrarily-selected chart levels, the n-dimensional generalization (Cramer, 1946) of the Gaussian distribution is then applicable to the fraction of the time that, simultaneously, n ordinates, chosen at different time instants on the wave record will lie below their corresponding chart levels. The assumption that for each finite n, the selection of n ordinates actually results in a multidimensional Gaussian distribution is known as the multinormal hypothesis. Such a distribution is characterized by a set of parameters known as the covariances (Cramer, 1946). These parameters may be readily interpreted in terms of the degree to which the various ordinates determine, or are determined by, each other. Computation (Rice, 1944-5) tells us that the covariance of any two quantities so distributed is proportional to $\cos(\pi \cdot P)$, where P is the probability that the two quantities have opposite algebraic signs.

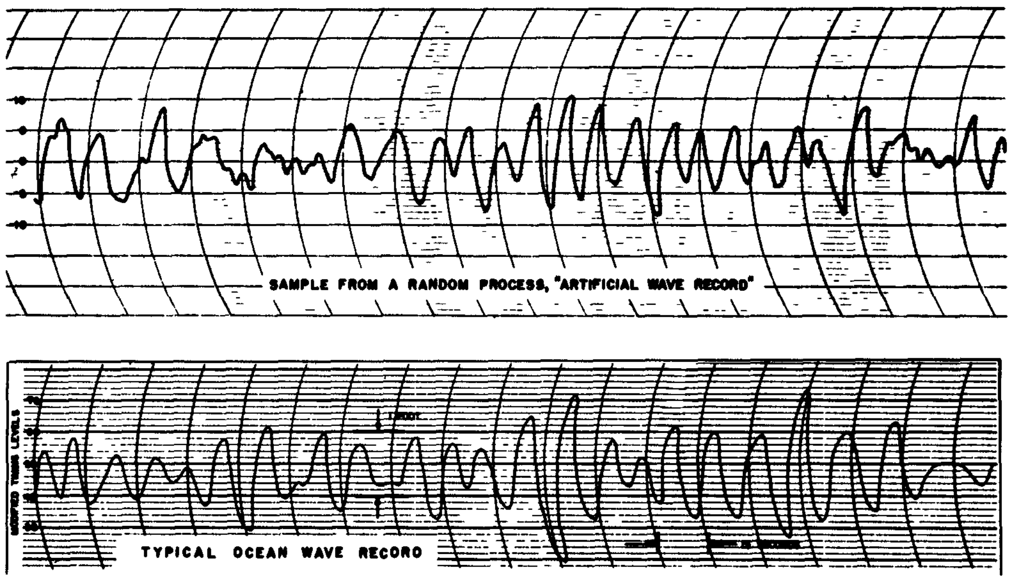


Fig. 1

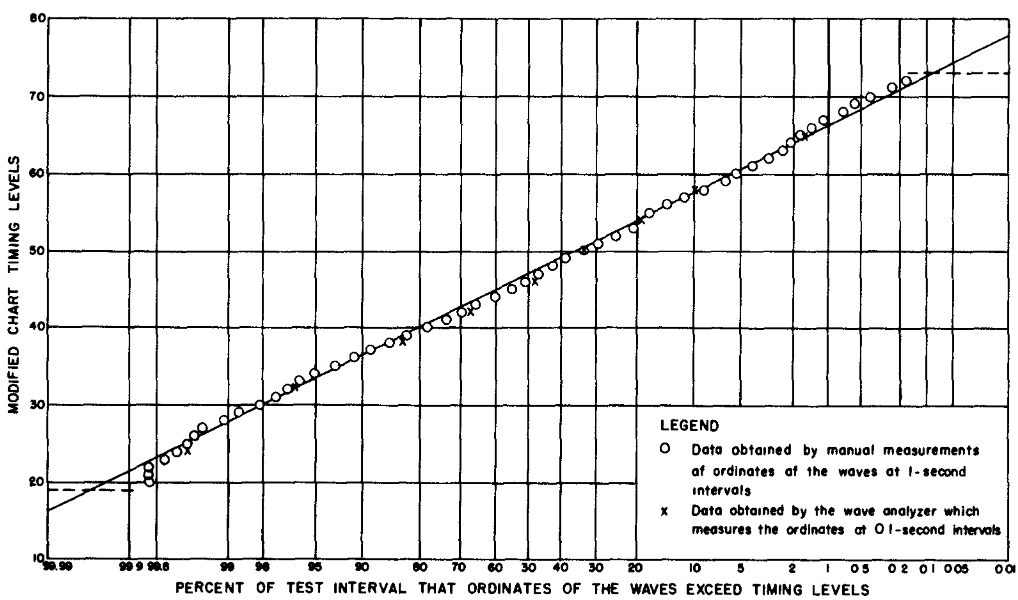


Fig. 2. Cumulative distribution function of ordinates.

A very convenient property of these multiple ordinate distritutions, and one which is physically reasonable except for relatively long records, is the stationary property (Lee, 1949; Putz 1953a), namely that the distribution of any set of ordinates depends only upon the differences in time between the corresponding abscissas. We then have distributions with equal covariances whenever the corresponding two ordinates are separated by the same time lag, τ, i.e., the covariance γ between any two ordinates selected at times t and $t+\tau$ depends only upon τ. The covariance is thus a function of the lag τ, the function $\gamma(\tau)$ being known as the covariance function, (Mann, 1953), and, when divided by its maximum value $\gamma(o)$, as the correlation function, $\rho(\tau)$, the latter being the familiar product-moment correlation coefficient. Once assumed for the ordinates, the stationary and multinormal properties follow for derivatives of all orders.

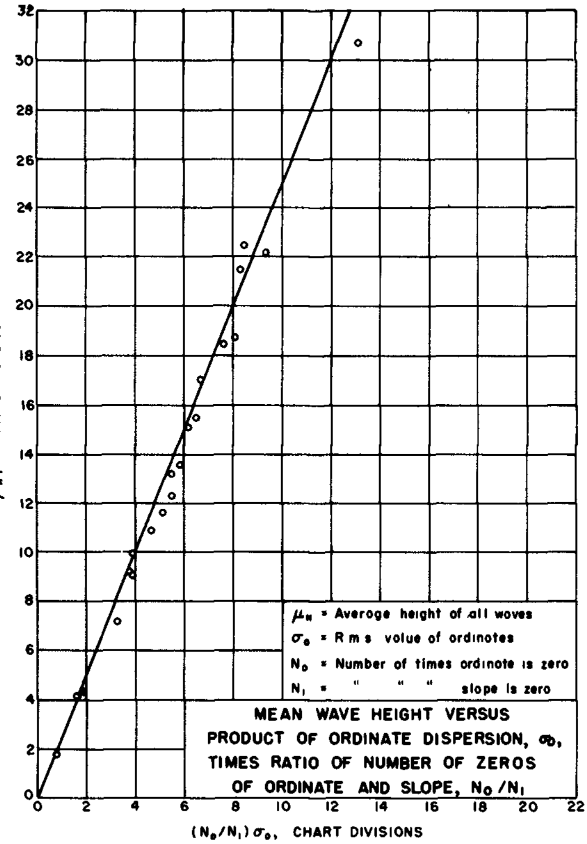
WAVE HEIGHTS

Interpretations for many of the parameters of these underlying probability distributions may be found in the distributions of certain quantities which may be graphically measured on the wave record.

If N_k denotes the number of times the k^{th} derivative passes through the zero level, and if T is the total length of the wave record in seconds, then the average number of zero-level crossings (up or down) per second will be N_k/T. A natural definition of the mean (undirectional) zero-crossing frequency is then $f_k = N_k/(2T)$ in cycles per second, or $\omega_k = 2\pi N_k/(2T) = \pi N_k/T$ in radians per second. Theory predicts that $\omega_k = \sigma_{k+1}/\sigma_k$, which serves to relate the r.m.s. values of the ordinate and of the first and second derivatives to the observed mean zero-crossing frequencies of the ordinate and the first derivative, when k is taken to be zero and one.

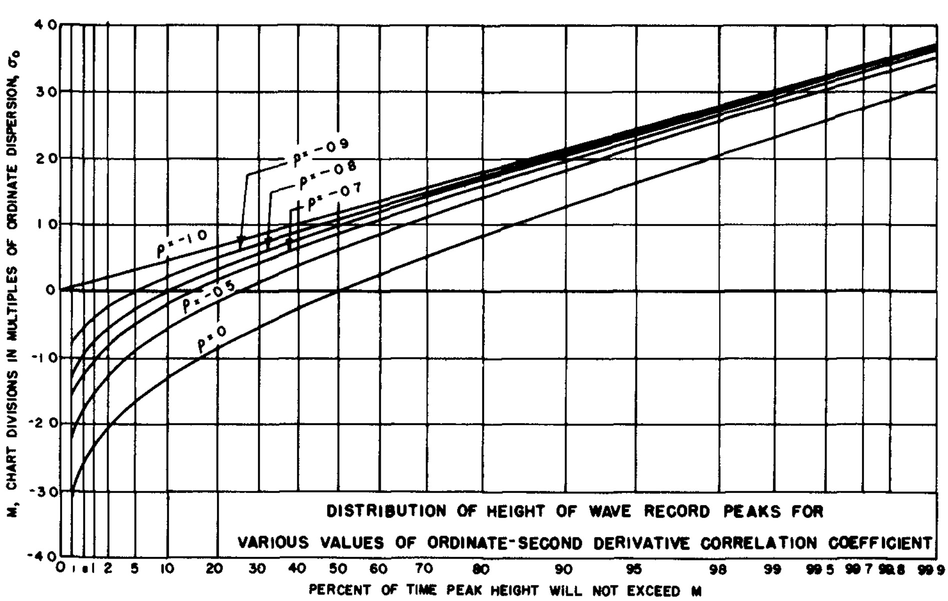
Further relations appear if we consider simultaneous values of ordinate and second derivative. The value of the coefficient of correlation between these two quantities is given by $\rho_o = \cos(\pi \cdot P_o)$, where P_o is the fraction of the time that they have opposite signs. Theory (Rice, 1944-5) predicts further that $\rho_o = -(\omega_o/\omega_1) = -(N_o/N_1)$. A satisfactory experimental check of the corresponding relation $\cos(\pi \cdot P_o) = -(N_o/N_1)$ has been obtained by locating points of inflection on a few twenty-minute records.

We may now consider the distribution of wave heights. For this purpose we shall take as fundamental the heights of the maximum points on the wave record curve, i.e., the points at which the second derivative is negative and the first derivative is zero. The theory applies also to minimum points if the wave-record curve is reflected about its mean level. If these peak heights, measured as directed distances from the mean level, are averaged, the resulting quantity μ_M is just one-half the average trough-to-crest wave height μ_H (Putz, 1952a). The theoretical relation is $\mu_H = (2\pi)^{\frac{1}{2}}(-\rho_o)\,\sigma_o$, illustrated for twenty-minute wave records in Fig. 3, where μ_H, measured in chart divisions, is plotted against $(N_o/N_1)\,\sigma_o$, also in chart divisions. The straight line shown has the slope $(2\pi)^{\frac{1}{2}}$ given by the theory. It is seen that the average of the so-called individual wave heights (Putz, 1953b) can provide



μ_H, CHART DIVISIONS
$(N_0/N_1)\sigma_0$, CHART DIVISIONS
μ_H = Average height of all waves
σ_0 = R m s value of ordinates
N_0 = Number of times ordinate is zero
N_1 = " " " slope is zero
MEAN WAVE HEIGHT VERSUS
PRODUCT OF ORDINATE DISPERSION, σ_0,
TIMES RATIO OF NUMBER OF ZEROS
OF ORDINATE AND SLOPE, N_0/N_1

Fig. 3.



M, CHART DIVISIONS IN MULTIPLES OF ORDINATE DISPERSION, σ_0
PERCENT OF TIME PEAK HEIGHT WILL NOT EXCEED M
$\rho = -1.0$
$\rho = -0.9$
$\rho = -0.8$
$\rho = -0.7$
$\rho = -0.5$
$\rho = 0$
DISTRIBUTION OF HEIGHT OF WAVE RECORD PEAKS FOR
VARIOUS VALUES OF ORDINATE-SECOND DERIVATIVE CORRELATION COEFFICIENT

Fig. 4.

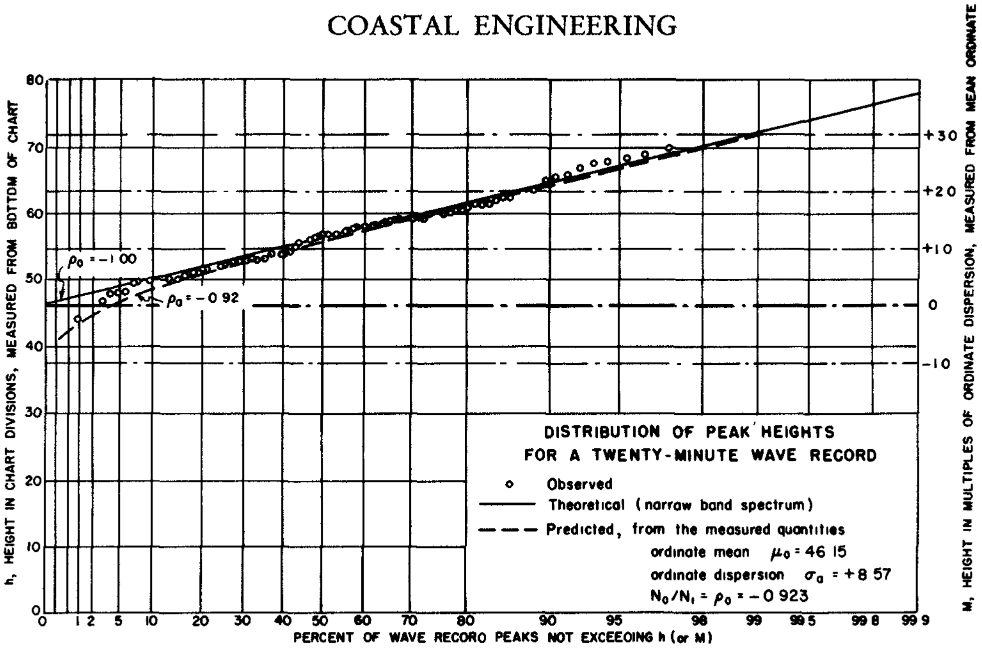

DISTRIBUTION OF PEAK HEIGHTS
FOR A TWENTY-MINUTE WAVE RECORD
Observed
Theoretical (narrow band spectrum)
Predicted, from the measured quantities
ordinate mean μ_0 = 46 15
ordinate dispersion σ_a = +8 57
$N_0/N_1 = \rho_0$ = −0 923
ρ_0 = −1 00
ρ_a = −0 92
h, HEIGHT IN CHART DIVISIONS, MEASURED FROM BOTTOM OF CHART
M, HEIGHT IN MULTIPLES OF ORDINATE DISPERSION, MEASURED FROM MEAN ORDINATE
PERCENT OF WAVE RECORD PEAKS NOT EXCEEDING h (or M)

Fig. 5

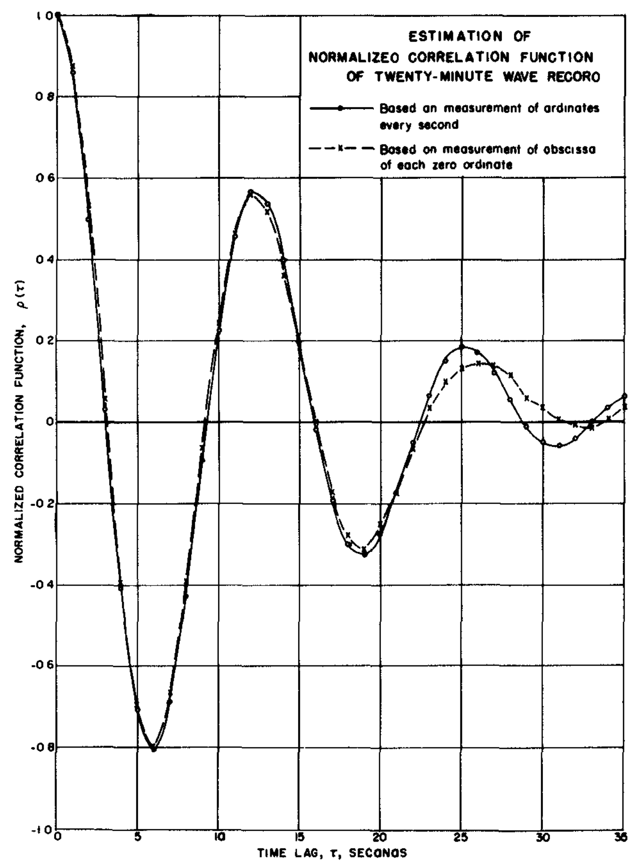

ESTIMATION OF
NORMALIZED CORRELATION FUNCTION
OF TWENTY-MINUTE WAVE RECORD
Based on measurement of ordinates every second
Based on measurement of obscissa of each zero ordinate
NORMALIZED CORRELATION FUNCTION, $\rho(\tau)$
TIME LAG, τ, SECONDS

Fig. 6.

a good estimate of the r.m.s. ordinate when combined with a count of the number of times the ordinate and the slope pass through zero.

The entire distribution of peak (or trough) heights (Rice 1944-5) given by the theory, depends only upon σ_0 and ρ_0, and is shown in Fig. 4. The vertical scale corresponds to the chart level measured from the mean in σ_0- units equal to the r.m.s. ordinate. The percent of peak heights not exceeding this vertical level appears on the horizontal, the scale in this case being chosen to correspond to the so-called Rayleigh distribution (Longuet-Higgins 1952; Lawson and Uhlenbeck; 1950, Knudtzon, 1949) which the peak heights follow more and more closely as the parameter ρ_0 tends to the value minus one. It may be observed that the probability of a low peak situated below the mean level is just $\frac{1}{2}$ $(1+\rho_0)$, which tends to zero as ρ_0 tends to minus one, corresponding to a relatively narrow-band spectrum.

Fig. 5 shows a typical observed distribution of peak heights for a twenty-minute wave record. The straight line represents the asymptotic Rayleigh distribution, while the curve represents the distribution of peak heights actually predicted from measured estimates of σ_0 and ρ_0, as well as of the mean ordinate μ_0. The vertical level is indicated in chart divisions on the left and in σ_0-units on the right.

ENERGY DISTRIBUTION

The statistical analysis of the periods between the zero crossings, or between the peaks on a wave record is closely related to the distribution of energy over the frequency or period spectrum. This assignment of energy to a continuum of superimposed elementary waves is specified by the so-called spectral distribution function or spectrum. The spectrum provides a means of deriving and interpreting the parameters of the distributions of wave-record ordinates and derivatives entering into the distribution of wave heights.

The distribution of energy is of fundamental importance, of course, for the study of the generation, propagation and the effects of waves. While information regarding the frequency of occurrence of wave heights or periods of given magnitudes appears occasionally to be directly useful for design purposes, a knowledge of the spectrum can be used whenever the frequency-response function corresponding to a general filtering action is known.

The usual spectrum analysis of a given wave-record sample results in the assignment of an amplitude and, in addition, a phase angle to each of a certain set of frequencies. However, the direct dependence of such phase information on the time origin to which the spectrum analysis is attached, makes it unsuitable as a possible parameter for a stationary random process. Thus it is natural to take as fundamental the remaining aspect of the spectrum -- i.e., the amplitude, or equivalently, the square of the amplitude, the so-called power spectrum (Rice, 1944-5).

There is a function obtainable from a sample wave record known as the sample covariance function which has for its spectrum just the power spectrum of the wave record itself. In many cases the power spectrum of a given sample wave record is most naturally obtained by first computing the covariance function for the wave record which was observed. However, any problem whose solution involves the prediction of the possible sample wave records which might have been observed before or after the given one will naturally lead to the association of a probabilistic model with each sample wave record. This model may be thought of as comprising a whole set of possible, infinitely long, wave records, each having the same power spectrum or covariance function. The identification of this common covariance function with that described earlier in connection with stationary multinormal ordinate distributions is the step which supplies the relation between these probability distributions and the spectral-energy distribution. The specification of the covariance function, together with the stationary multinormal assumption, defines a so-called stationary Gaussian random (or stochastic) function (or process). (Mann, 1953; Lee, 1950).

The ability of such a theoretical model to describe observed wave records is illustrated by comparing a sample taken from such a random process with a typical wave record. The upper curve in Fig. 1 was generated after first specifying three fixed numbers defining the spectrum and then consulting a table of random numbers (Wold, 1948).

If the mathematical moments of the spectrum exist (Putz, 1953a) , certain of these can be shown to be equivalent to the r.m.s. values of the derivatives of the original wave record, while certain ratios of these moments are essentially the coefficients of correlation between the derivatives. Thus the r.m.s. ordinate σ_0 is the zero-order moment of the spectrum or the total spectral energy, while ρ_0 is related to the second-order spectral moment, or more precisely, to a certain measure of the relative spectral bandwidth when the latter is small.

WAVE PERIODS

It has been seen that the instants of time at which the wave record passes through zero or through a peak enter,by way of the relative spectral bandwith,into a relation between the wave heights and the r.m.s. ordinate. For the analysis of the spectrum, the differences between two successive zero-crossings may be considered the fundamental individual wave periods. According to the theory, for a long record the list of these individual wave periods in the order of their occurrence is completely equivalent to the spectrum. Peak-to-trough or trough-to-peak time intervals, corresponding to the periods between successive zero-crossings of the wave-record derivative, while more sensitive to the influence of noise, are capable in principle of yielding the same information. On the other hand, peak-to-peak or trough-to-trough time intervals (Snodgrass, 1952), corresponding to the periods between alternate zero-crossings of the derivative, would be expected to contain less information.

The significance of the zero-crossings for our model lies in the fact that the abscissas of these points separate the intervals during which the wave record has constant sign. Using these abscissas, we may estimate the correlation function for a given wave record by first determining the fraction $P(\tau)$ of the time that the original wave record and the wave record advanced by a time interval τ have opposite sign. It is from the correlation function $P(\tau) = \cos[\pi P(\tau)]$ that the spectral information is obtained.

Fig. 6 shows a comparison of the estimate of $\rho(\tau)$ obtained for a twenty-minute record by measuring zero-crossing abscissas and the ordinary estimate (Tukey, 1949; Pierson & Marks) obtained by measuring ordinates every second. This agreement between correlation functions has been found sufficiently close to result in a relatively small difference between the corresponding estimates of the wave-record power spectrum from the two methods. It may be observed that in addition to measuring 1200 ordinates on the wave record, nearly 1200 multiplications and 1200 additions are required to obtain each point of the solid-line curve, while for each point of the dotted line curve it is sufficient to measure about 200 abscissas on the wave record and perform about 400 additions and 200 subtractions. While either method leads to a substantial saving in labor over a straightforward numerical Fourier analysis of the original wave record (James, Nichols and Phillips, 1947); the zero-crossing method for the correlation function, requiring essentially no multiplication, may be carried out with only an adding machine.

The degree of reliability of the zero-crossings exhibited by this one experimental check on a twenty-minute wave record may be of some theoretical interest. The close correspondence, particularly for small time lags, seen in Fig. 6 is additional evidence for the general mathematical model we have described, and for the multinormal property in particular. It constitutes evidence also for the sampling adequacy of a wave record twenty minutes in length. The indications are that the so-called individual wave periods, suitably defined, can furnish reliable information about the Fourier spectrum of the wave record.

This kind of approximate spectral information has been used to predict, by a least-squares method (Putz, 1952b), the behavior of ocean waves at one time and place from their behavior at a time and place nearby. The resulting prediction is shown in Fig. 7, where the upper plot represents the time history of the observed surface elevation at a point 1570 feet seaward of the point at which prediction was to be made. The lower part of the figure shows the observed and the predicted wave record at the shoreward point fifty seconds later. The fact that the accuracy of prediction represented here compares favorably with that based upon a more exact spectrum analysis is a further indication of the reliability of the information provided by the zero-crossings.

SUMMARY

There exists an approximate mathematical model for wave records which may be profitably exploited until refinements in it are found necessary. This model specifies simple verifiable relations which may be

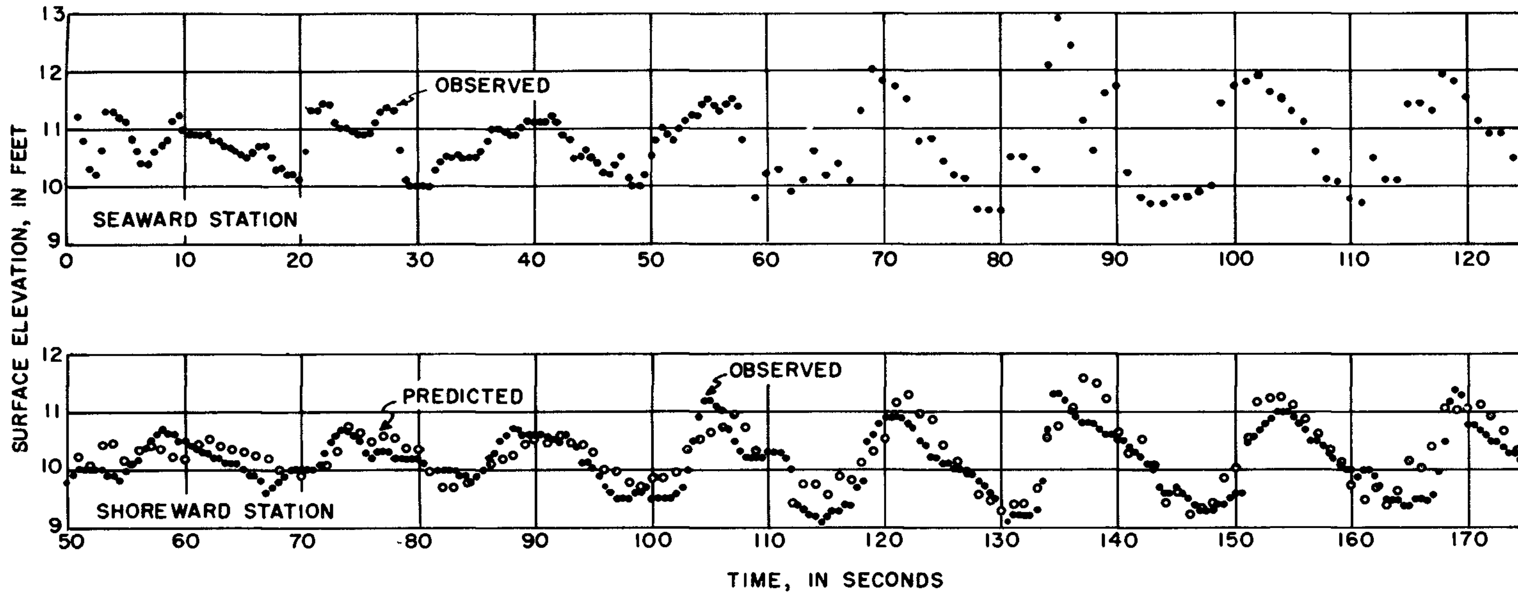


Fig. 7. Least-squares surface prediction over 1570 ft. distance (Four-term kernel based upon approximate spectral information from both positions).

expected to hold between appropriate quantities easily measured on long wave records. The parameters occurring in the model may be estimated from individual wave-height and wave-period distributions. A few such parameters, easily interpreted in terms of the spectrum, are available for the description of a wave record.

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CHAPTER 3

WIND TIDE AND SEICHES IN THE GREAT LAKES

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INTRODUCTION

Because of the unusually high lake stages of recent years, the Weather Bureau was called on to forecast the short period variations of lake level, which were believed to be caused by wind stress and atmospheric pressure gradient. It became necessary to investigate the feasibility of such forecasts. In a review of the available literature, many papers were found which described methods of computing the free periods of oscillation for lakes when no external forces were acting. Other papers were found which described methods of computing the steady state relation between a constant atmospheric force and the lake surface when inertial forces are neglected.

But in nature, a steady state different from the equilibrium state is rarely achieved, and neither the external forces nor the inertial forces can be neglected in any attempt to relate the atmospheric disturbances to lake level disturbances.

A theory which relates the build up of the water level disturbance to the concurrent or previous weather conditions is needed, if forecasts are to be prepared with any degree of confidence.

This report gives the preliminary results of an effort to develop such a theory. Since the theoretical study requires some knowledge of the phenomenon to be explained, the first part of the report is devoted to a description of observed water level disturbances. The middle section gives the essential development of the hydrodynamic equations leading to a more generalized theory of the forced oscillations of a lake. This is followed by a short discussion of the consequences of the extended theory.

Most of the mathematical treatment not required for an understanding of the physics of the problem has been relegated to the appendix.

In scientific discussion the term "seiche" is usually restricted to the inertial oscillations which persist after the external force has ceased to act. In popular usage this term is frequently applied to any disturbance which is believed to be produced by some meteorological force and whose period is longer than that of the surface waves.

In the Great Lakes, these disturbances appear to fall naturally into two classes: those which involve all or a large part of a major lake, and those of a more local character. The first class is illustrated by figure 1. The surface water in Lake Erie is driven toward the eastern end of the lake by the wind. This causes an increase in the water level at Buffalo and a decrease in the water level at the western end of the lake. After the wind shifts or decreases in speed, the lake undergoes a series of damped oscillations as it returns to normal.

The second class is illustrated by figure 2. This is a tracing of the actual water level record for Two Harbors, Minn. for a seven-hour period on May 5, 1950. This oscillation appears to be of a more local character than that shown in figure 1.

For the purposes of this report, disturbances of the first class will be referred to as long period and those of the second type as short period. The division appears to occur with a period of about one hour.

THE DATA

In order to test the various hypotheses that have been proposed for the generation of lake level disturbances, all of the continuous lake level records of the U. S. Lake Survey for the year 1950, and selected portions of the records of several other agencies and for other periods have been examined.

The ten or fifteen most prominent disturbances at each of the Lake Survey gages during 1950 were compared with synoptic weather charts and other meteorological data. The location of gages used in this part of the study is shown in figure 3. It was found that short period disturbances tend to occur in zones of disturbed weather, that is, near fronts, squall-lines, thunderstorms, etc., and that long period disturbances seem to conform to the classical picture of the surface water being driven to the leeward end of the lake by high winds.

Both long and short period disturbances occur on all lakes. However, the long period disturbances are most prominent on Lake Erie, and the short period disturbances are most prominent on the other lakes. Accordingly, the detailed study of long period disturbances was largely confined to Lake Erie. Southern Lake Michigan was selected for a detailed study of the short period disturbances because of the relatively large number of lake level gages in the neighborhood of Chicago.

SHORT PERIOD DISTURBANCES

The location of gages used in this study is shown in figure 4. The Wilson Avenue crib gage is located about 3 miles off shore in the open lake. Two gages, Chicago River and Navy Pier, are located in the Chicago Harbor. The Filtration Plant gage is located in a small basin protected by a breakwater open at both ends. The Calumet Harbor gage is located in the Calumet River. The Waukegan gage, also used in this study, is just inside the Waukegan harbor about 35 miles north of Navy Pier.

Figure 5 shows an example of a short period disturbance as recorded by four of these gages. It should be noticed that the agreement between the two records for Chicago Harbor, is much greater than that between these records and those of the Filtration Plant and Calumet Harbor.

Figure 6 gives a comparison between the records of a disturbance as recorded at the Wilson Avenue Crib in the open lake, and the Calumet and Waukegan harbors. It should be observed that the amplitude of the disturbance is much greater at the harbor gages than at the crib gage. This relation also holds for other harbors in this area and appears to be typical of at least the largest disturbances in the records examined.

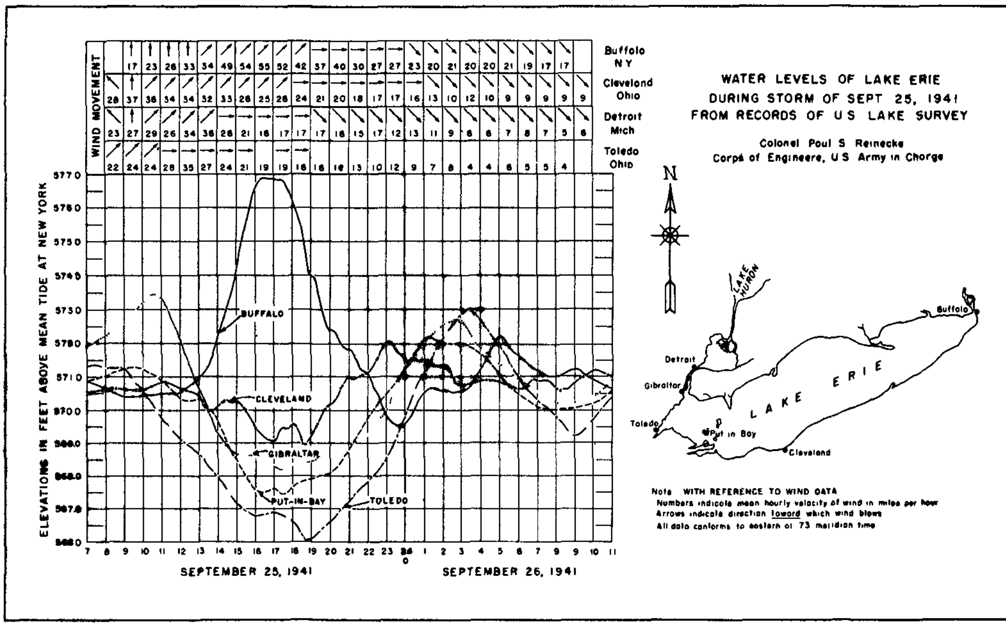

Fig. 1. Lake Erie wind tide, adapted from original drawing by U.S. Lake Survey.

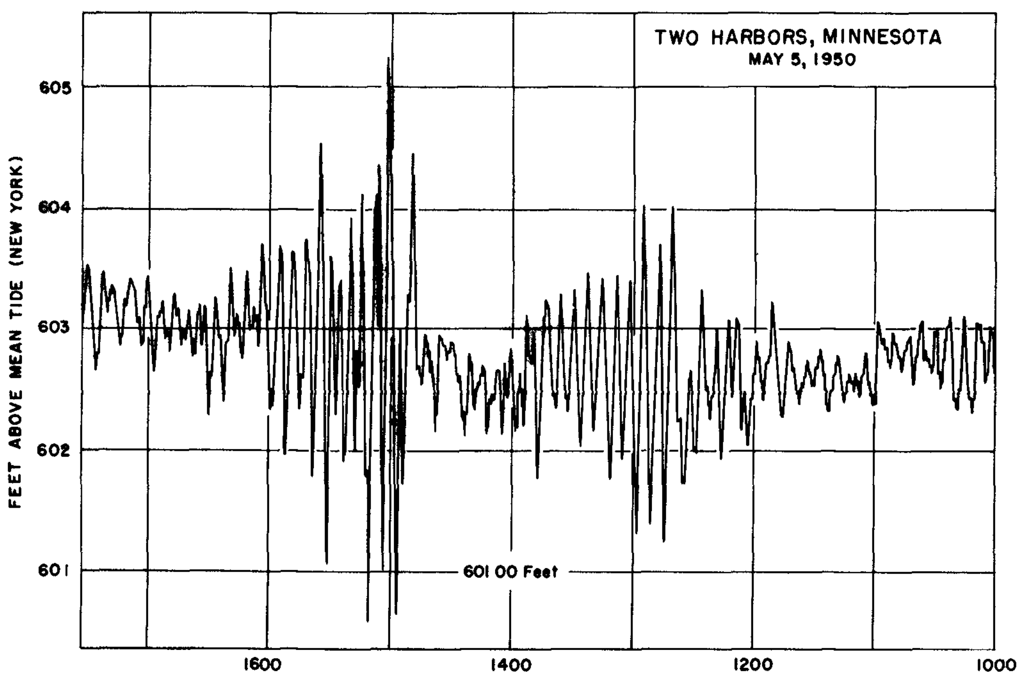

Fig. 2. Example of short period harbor oscillation at Two Harbors, Minn.

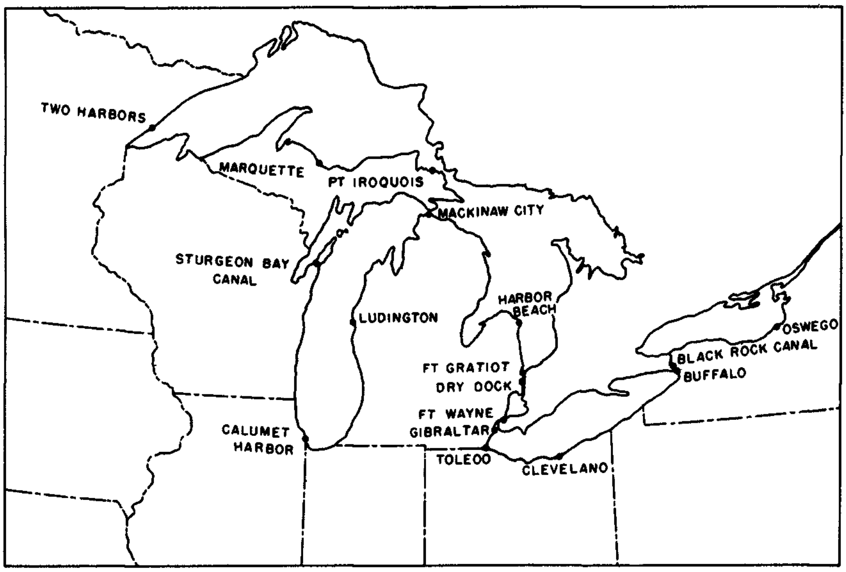

Fig. 3. Locations of U.S. Lake Survey gages used in the study of wind tides and seiches.

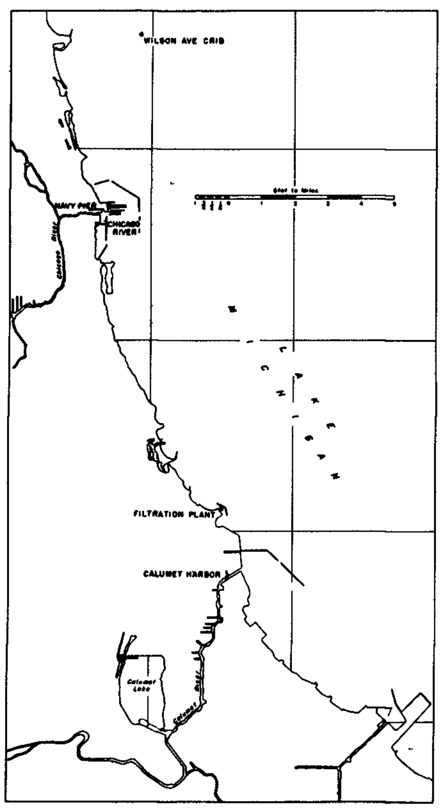

Fig. 4. Location of lake level gages in Chicago.

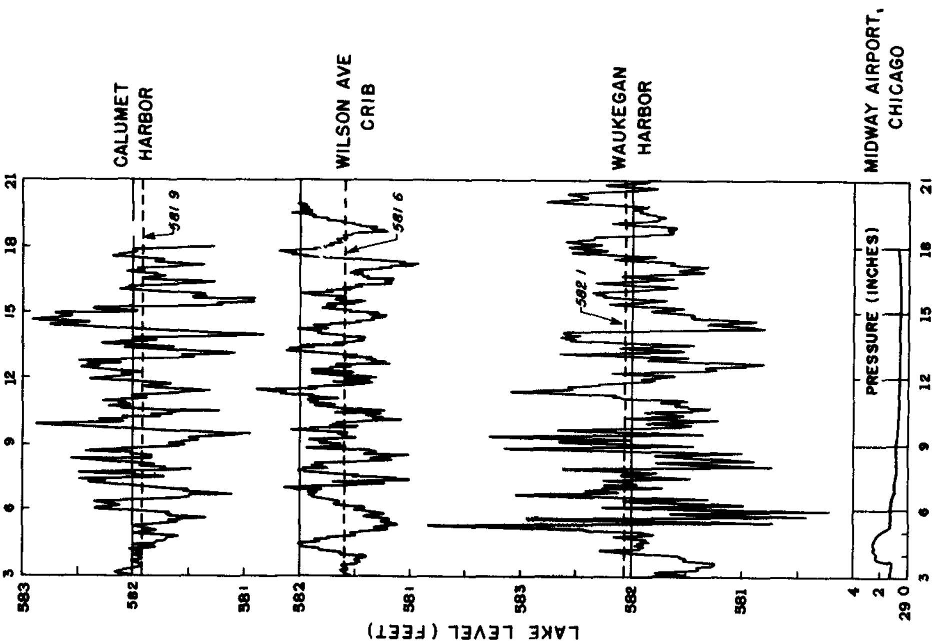

Fig. 6. Comparison of lake level records, as recorded in the open lake and in two harbors, and the barograph record, June 8, 1953.

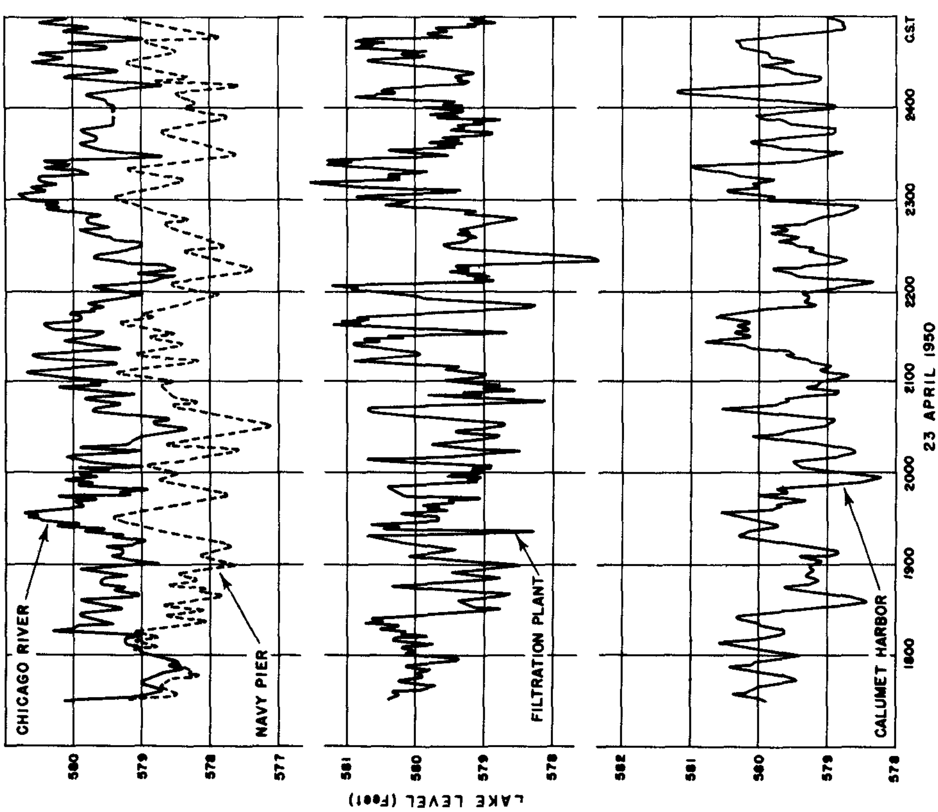

Fig. 5. Comparison of lake level at four Chicago gages.

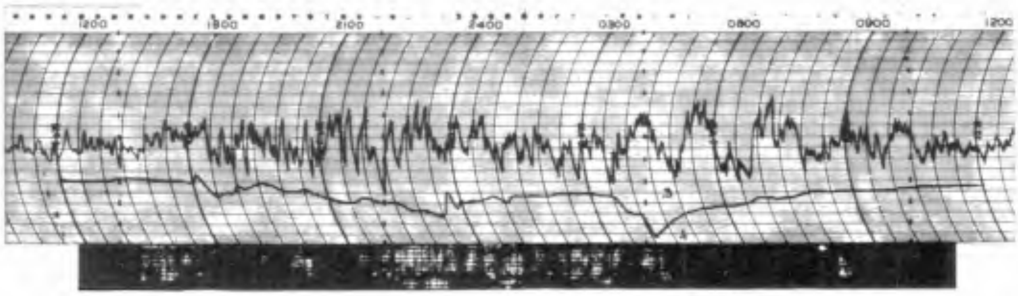

Fig. 7. Comparison of the records of lake level, pressure, and wind speed as recorded at the 79th Street Filtration Plant, Chicago, April 23-24, 1950.

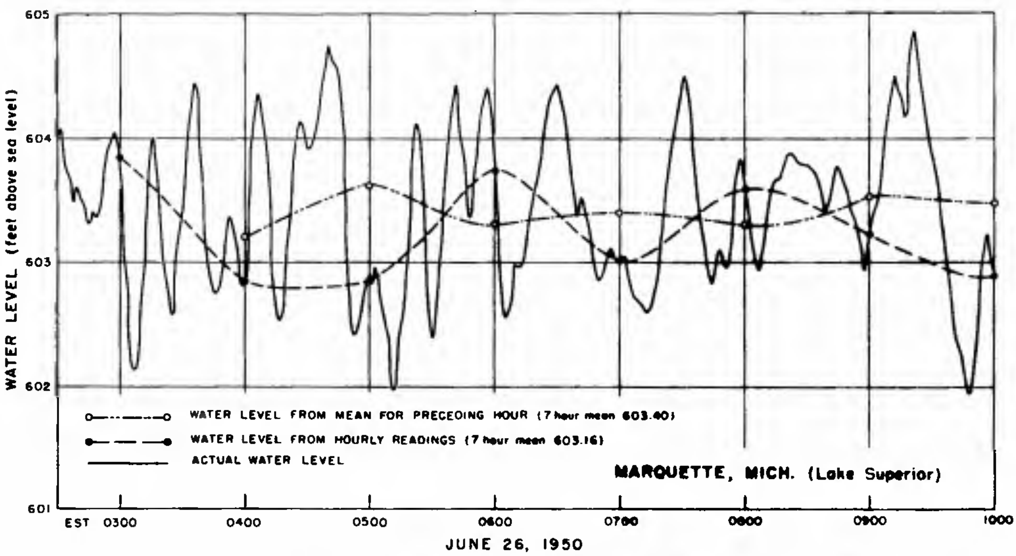

Fig. 8. Water level variations at Marquette, Mich., June 26, 1950.

It should also be noticed that the mean water level at the harbor gages is about .3 to .5 feet higher than that recorded at the crib gage. This tendency for the mean water level to be higher in a harbor than in open water has been reported by McNown (1952) and will be mentioned again.

The possibility that these short period disturbances may be produced by changes in atmospheric pressure has often been mentioned. A well organized atmospheric pressure wave crossed southern Lake Michigan on the morning of June 8, 1953. A copy of the barograph record for Midway Airport, Chicago, is shown on the bottom of figure 6. The more intense pressure change wave passed the western shore of Lake Michigan about an hour before the sudden increase in the amplitude of the lake level oscillations. Supplementary data show that this wave of rising pressure had passed completely across the lake by the time that the large disturbance was recorded on the Waukegan record.

In general all of the gages tend to become excited at about the same time, but the frequency and amplitude, as well as the phase of the most prominent components of the disturbance, vary from one harbor to the next in an apparently random manner.

Figure 7 shows a comparison between the pressure, wind, and water level as recorded at the Filtration Plant on April 23, 1950. The resemblence between the wind and lake records is much greater than that between the pressure and lake records. In neither case, however, is the comparison between either wind or pressure changes and water level changes close enough to imply a one-to-one relationship.

There were other well defined pressure waves unaccompanied by any significant change in the character of the lake records. There were also periods of large amplitude lake oscillations accompanied by a comparatively smooth barograph record. If a time discrepancy of about two hours is allowed, approximately half of the pressure disturbances are accompanied by lake oscillations, and vice versa. Comparisons between the lake records and wind records lead to similar conclusions.

The tendency for these short period oscillations to occur at the approximate time of an atmospheric disturbance indicates that there is some connection between the meteorological disturbances and the lake level fluctuations. However, these data show that the relation cannot be very simple or direct.

So far as could be determined, no wave recorders were in operation in Chicago during 1950, however, the Beach Erosion Board has prepared a "hindcast" of waves in southern Michigan based on the 6-hourly synoptic weather maps prepared by the WBAN Analysis Center in Washington. (Saville; this volume.) Little correlation was found between the amplitude of the observed lake oscillations and the expected amplitude of the surface waves.

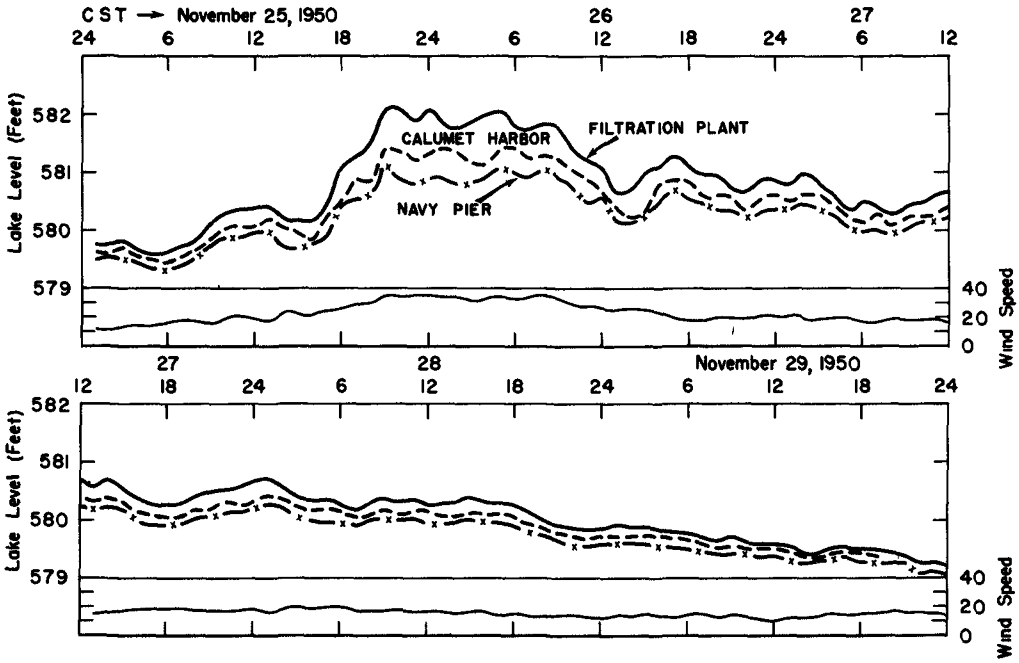

Fig. 9. Hourly mean lake levels, as recorded at 3 Chicago gages, November 25-29, 1950. Northwesterly winds November 25-28, becoming westerly on November 29.

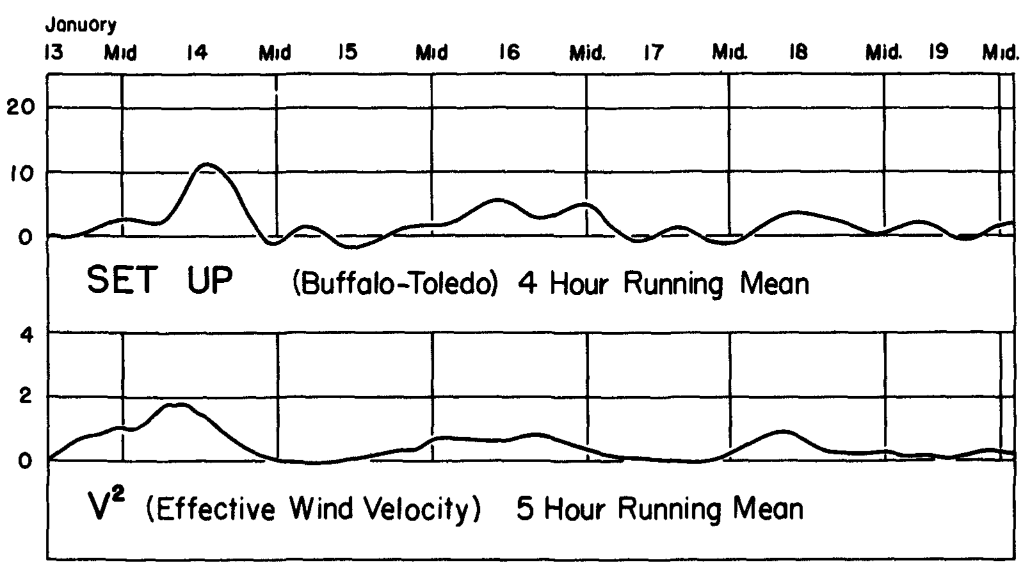

Fig. 10. Set-up on Lake Erie, and V^2(effective wind velocity) for the period, January 13-19, 1953, as defined by Keulegan.

REPRESENTATIVENESS OF OBSERVATIONS

The official records of the U. S. Lake Survey consist, for the most part, of the hourly readings of instantaneous lake level taken from the continuous recorder records. The primary purpose of this record, is the computation of the daily, weekly, and monthly mean lake level. Since each of these means represents the average of a great many individual readings, the method of sampling appears to be adequate for this purpose.

However, these hourly reports are sometimes used as the basis for a study of lake level oscillations. The inadequacy of this procedure is clearly indicated by figure 8. It can be seen that the use of hourly values only gives the appearance of spurious periodicities, and does not represent the true behavior of either the actual water level or the long period trend at the gage. Although these cases represent disturbances of unusual amplitude, the general lack of agreement between the true and apparent periodicities, and the tendency to obscure extreme values are typical of most of the gages in the Great Lakes network.

In an effort to eliminate the effect of harbor oscillations from the lakewide disturbances, hourly means were computed from a number of gages on Lake Michigan for periods when systematic lakewide disturbances were suspected. The hourly mean lake levels for three of the Chicago gages, for the period November 25-29, 1950 are shown in figure 9. It will be noticed that the spread between the record of the three gages increases with the wind speed. Similar studies of other periods show that the sign of this difference depends on the wind direction.

This report is concerned only with cases of unusual lake behavior, and the figures are selected to show extreme cases. It is pointed out that the hourly readings, and even the means computed from a harbor gage, may fail to represent the true behavior of the lake surface during a disturbed period. However, disturbances of the magnitude shown here are of short duration and infrequent occurrence, and these disturbances will not often lead to any significant error in the monthly mean.

LONG PERIOD DISTURBANCES

Keulegan (1951,1952) investigated the wind stress coefficient by studying the relation between the effective wind over Lake Erie, and the set-up. The set-up is defined as the difference in water level at the opposite ends of the lake, as measured by the gages in Buffalo and Toledo. He gave a formula for the effective wind velocity as a function of the observed wind at four stations on the southern shore.

In deriving his formula, Keulegan assumed the existence of a steady state relation between the effective wind velocity and set-up. He sought to offset the error involved in this assumption by time averages of the wind stress and set-up.

Figure 10 is a plot of the set-up and effective wind velocity as defined by Keulegan for the period January 13-19, 1950. It can be seen that a steady state did not exist at any time during this period. The effective wind velocity rises and falls three times during this period.

Keulegan recognized that the steady state could not exist for any protracted period, but he assumed that it would hold at the time midway between the maximum displacement at Toledo and Buffalo. Roughly this corresponds to the peaks in the set-up curve as shown in figure 10.

Using the above assumptions, Keulegan derived the data shown in figure 11 for the relation between the set-up and the effective wind velocity. The linear regression line (dashed line) has been added. The correlation coefficient between this line and the plotted data is .94. The straight line was used because of its greater simplicity. It does not appear that the correlation could be greatly improved by considering the curved line.

It is evident from figure 10, that Keulegan's results can not give all the information needed for forecasting. However, the high correlation between set-up and wind velocity, even though obtained under somewhat special conditions, leads one to believe that a useful forecasting procedure could be obtained from a similar analysis in which changes with respect to time are considered.

HYDRODYNAMIC THEORY

In order to derive an expression relating the atmospheric pressure and wind variations to the behavior of a lake, it is necessary to consider the hydrodynamic equations of motion. Since attention is focussed on the displacement of a free surface it will be convenient to integrate the equations from the bottom to the top of the lake and to consider only the mean motion, averaged through the vertical. The appropriate linearized equations of motion become

$$\partial u/\partial t = -g\partial h/\partial x + F_x + fv \qquad (1)$$

$$\partial v/\partial t = -g\partial h/\partial y + F_y - fu \qquad (2)$$

$$\partial h/\partial t + \partial(Du)/\partial x + \partial(Dv)/\partial y = 0 \qquad (3)$$

where:

h = displacement of water surface

D = mean depth of the water

F_x, F_y = components of atmospheric force

f = Coriolis parameter - effect of earth's rotation.

and the other symbols have the usual meaning.

A slightly different form of these equations is given by Lamb (1932 Chapter VIII). The method of derivation is given in detail by Haurwitz (1951).

SEICHE EQUATIONS -- TWO DIMENSIONAL THEORY

It will be convenient to consider the inertial motion, that is the true seiches, before taking up the forced motions. In order to accomplish this, the applied forces, F_x and F_y, are neglected, and it is assumed that u, v, and h are periodic in time. That is, it is assumed that

$$u = f_1(x,y)e^{i\nu t}$$
$$v = f_2(x,y)e^{i\nu t} \qquad (4)$$
$$h = \Phi(x,y)e^{i\nu t}$$

where f_1, f_2 and Φ are as yet undetermined functions of x, and y and ν is the frequency of the disturbance.

If equations (4) are differentiated and substituted into equations (1-3), u and v can be eliminated from equation (3) to obtain

$$\frac{\partial}{\partial x}\left(D\frac{\partial\Phi}{\partial x}\right) + \frac{\partial}{\partial y}\left(D\frac{\partial\Phi}{\partial y}\right) + \lambda\Phi = i\frac{f}{\nu}\left(\frac{\partial D}{\partial x}\frac{\partial\Phi}{\partial y} - \frac{\partial D}{\partial y}\frac{\partial\Phi}{\partial x}\right) \qquad (5)$$

where

$$\lambda = (\nu^2 - f^2)/g \qquad (6)$$

The terms on the right in equation (5) are at least one or two orders of magnitude smaller than those on the left over most of the lake, and if they are neglected, we obtain

$$\partial(D\partial\Phi/\partial x)/\partial x + \partial(D\partial\Phi/\partial y)/\partial y + \lambda\Phi = 0 \qquad (7)$$

The boundary condition is determined by the requirement that no fluid passes through the sides of the lake. This may generally be expressed in the form

$$\partial\Phi/\partial n = 0 \quad \text{on the boundary}$$

where n is a unit normal to the boundary.

Equation (7) can be solved only for certain discrete values of λ, known as eigenvalues. Corresponding to each eigenvalue, there will be one natural frequency of oscillation and one or more eigenfunctions. Each eigenfunction will describe a different mode of oscillation, and each mode of oscillation will be completely specified by the number and location of the nodes, that is the lines of no vertical displacement.

The effect of the earth's rotation is often neglected in the derivation of the equations for seiches. This is equivalent to assuming that f = 0, or that

$$\lambda = \nu^2/g \qquad (9)$$

The permissible values of λ are obtained by solving the differential equation subject to the appropriate boundary conditions, and the frequencies of free oscillation can then be obtained by means of equations (6) or (9). Since the Coriolis parameter, f, is not involved in the computation of λ, it appears that the principal effect of the rotation of the earth is to increase the frequency of the free oscillations above that which would be experienced on a nonrotating earth. The amount of this increase can be obtained by eliminating λ between equations (6) and (9), or

$$\nu^2(\text{rotating earth}) = \nu^2(\text{nonrotating earth}) + f^2 \qquad (10)$$

Since f is approximately 10^{-4} in middle latitudes, this correction will amount to about 1% for a period of 2.5 hours and to about 10% for a free period of 7.8 hours.

It appears that satisfactory results can be obtained by neglecting the Coriolis term initially and correcting the computed frequency as indicated in equation (10).

The actual solution of equation (7) for an arbitrary lake is quite difficult, but some insight into the character of such a solution may be obtained from an examination of the known solutions for lakes of regular outline. We consider first a circular lake. The nodal lines are given by concentric circles and equally spaced diameters. (See fig. 12a.) If the circle is flattened slightly so as to form an ellipse, the nodal circles become ellipses, and all but two of the nodal diameters break away from the center to form nodal hyperbolas as shown in figure 12b. As the ellipse is flattened still further the hyperbolas tend to become more nearly straight lines perpendicular to the lake axis. This is illustrated in figure 12c which has the approximate proportions of Lake Erie. For rectangular lakes the nodal lines are parallel to the shores as shown in 12d.

SEICHE EQUATIONS -- ONE DIMENSIONAL THEORY

Since the more important nodal lines for long narrow lakes appear to be approximately straight lines at right angles to the lake axis, it appears that further simplification may be gained by reducing the problem to one dimension. This can be accomplished by considering the average value of u and h for each cross section of the lake. The resulting differential equation is

$$\partial(A\partial\phi/\partial x)/\partial x + \lambda b g^{-1}\phi = 0 + \text{higher ordered terms} \qquad (11)^*$$

$A = A(x) =$ Area of cross section
$b = b(x) =$ Width of the lake

The boundary condition is determined by assuming that no fluid flows through the ends of the lake, or

$$\partial\phi/\partial x = 0 \text{ at } x = 0 \text{ and } x = L. \qquad (12)$$

This is equivalent to the equations used by Chrystal (1905) and Defant (1925), and several methods have been given for obtaining a solution.

The effect of the earth's rotation can be taken into account in computing the natural frequency by using the definition of λ obtained in the study of the two dimensional problem in which the Coriolis terms were considered.

It should be emphasized however, that the averaging process eliminates all transverse modes of vibration. It can give only the average value of h as a function of distance along the axis of the lake. The nodal lines predicted by the one dimensional theory are necessarily straight lines crossing the lake. The true nodal lines are, in general, curvilinear, and will not necessarily intersect the shore. The importance of this point is illustrated in figure 13 which shows one mode of oscillation of a circular lake. This mode of oscillation in a laboratory

*Derivation of equation (11) is given in Appendix I

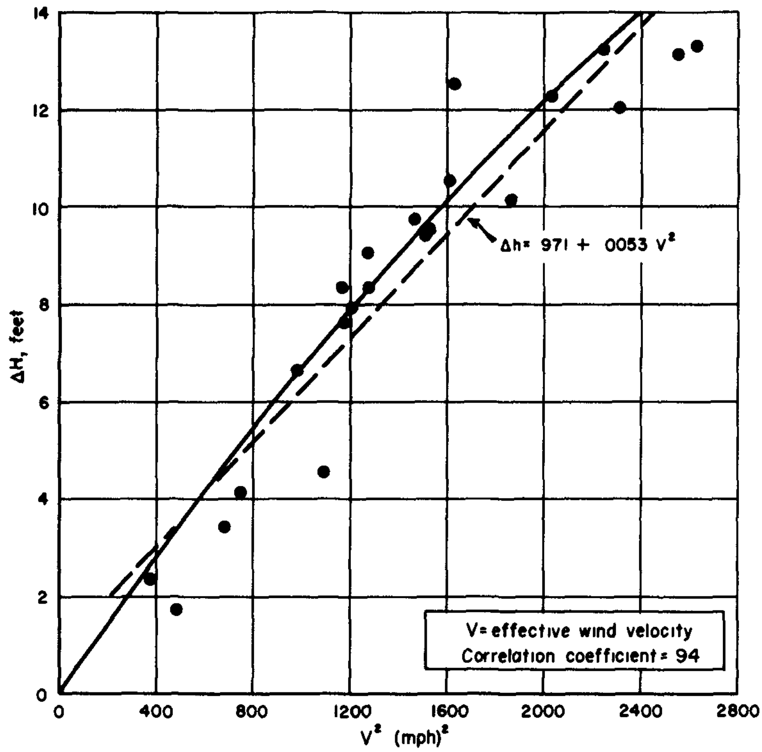

Fig. 11. Relation between set-up and effective wind velocity, from Keulegan.

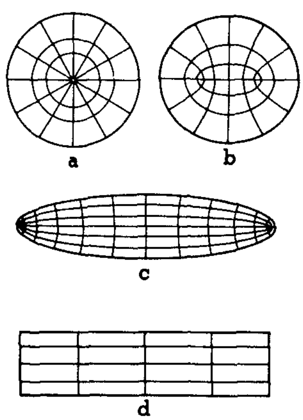

Fig. 12.

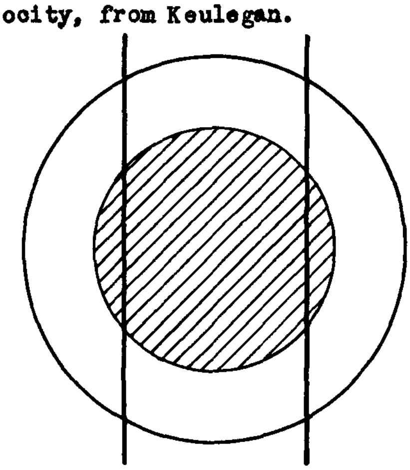

Fig. 13.

Fig. 12. Characteristic nodal lines for lakes of regular outline.

Fig. 13. An example of the true nodal line and the nodal lines as defined by the one dimensional theory for one mode of oscillation of a circular lake.

model has been photographed by McNown (1952). The true nodal line is a circle. The nodal lines in the mean value of h which is to be investigated by the one dimensional theory are given by lines aa' and bb'. It can be shown, however, that all modes of oscillation predicted for a symmetric lake by the one dimensional theory must have a nodal diameter running through the center of the circle. Thus it appears that the one dimensional theory can not give even an approximate description of all modes of oscillation of a lake.

NATURE OF THE EXTERNAL FORCES

Before discussing the equations of forced oscillations of a lake, it may be well to examine the nature of the forces to be considered. The forces considered here are those due to the atmospheric pressure gradient and the frictional stress of the wind on the water surface. We may write

$$F_x = -(\partial p_a/\partial x - \tau x/D)\rho_w \tag{13}$$

where

ρ_w = density of water, assumed constant
p_a = atmospheric pressure
τ_x = component of the wind stress in the x direction.

A similar expression may be given for F_y.

It is generally assumed that τ_x can be expressed in the form

$$\tau_x = \chi \rho_a V^2 \cos\theta \tag{13a}$$

where

χ = wind stress coefficient

V = wind speed

θ = angle between the wind direction and x axis.

We can investigate the approximate ratio between the magnitude of the wind and pressure components of the external force by expressing the pressure gradient in terms of the geostrophic wind by the relation

$$\partial p_a/\partial n = \rho_a f V_g \tag{14}$$

where V_g = geostrophic wind

ρ_a = density of the air,

and writing

$$\frac{\text{Force due to wind stress}}{\text{Force due to pressure gradient}} = \frac{\chi V^2}{fDV_g} \tag{15}$$

Most determinations of the wind stress coefficient show that χ is between 10^{-3} and 3×10^{-3}, and f is approximately 10^{-4} in middle latitudes. Hence if we assume that the true wind is quasigeostrophic we have the approximation

$$\frac{\text{Force due to wind stress}}{\text{Force due to pressure gradient}} = \frac{10V_g}{D}$$

The depth of Lake Erie varies from 10 to 40 meters, and in all lakes the depth within a few miles of the shore is rarely much greater than ten meters. Since important set-ups do not occur until the wind velocity is about 10 m/sec, (20 mph) or more we see that the wind effect will generally be an order of magnitude greater than the pressure effect in the shallow waters near the shores of all lakes, and over all of Lake Erie. In the deeper parts of the other lakes, where the depth may exceed 100 meters, the effects of wind and pressure will be of about equal importance.

In the case of sudden pressure changes such as that shown for Chicago on figures 5 and 6, the actual wind will be much less than the geostrophic value and the pressure gradient term may exceed the wind term.

However, apart from resonance effects, to be discussed later, the effect of the pressure gradient cannot exceed that of the equivalent water barometer, or approximately one foot of water for each inch of pressure. Since even the most violent of these sudden pressure changes rarely exceed .2 or .3 inch it appears unlikely that pressure changes could ever explain as much as one foot of the observed variation in water level.

FORCED MOTION

The mathematics involved in the solution of the equation for forced motion is much simpler in one dimension than in two. However, the physical principles involved are the same. Therefore only the one dimensional problem will be discussed in this paper. The method presented can be extended to cover the two dimensional problem.

The one dimensional differential equation for forced motion is

$$\partial(A\partial h/\partial x)/\partial x - bg^{-1}\partial^2 h/\partial t^2 = g^{-1}\,\partial(AF_x)/\partial x \qquad (17)^*$$

The boundary condition is again determined by the requirement that no fluid can flow through the ends of the lake. This leads to the requirement that the slope of the free surface must be in equilibrium with the applied force at the ends of the lake. This may be expressed in the form

$$\partial h/\partial x = F_x \text{ at } x = 0 \text{ and } x = L. \qquad (18)$$

The solution of this equation can be expressed as a sum of the eigenfunctions obtained in the study of seiches. This solution has the form

$$h = \sum a_n(t)\Phi_n(x) \qquad (19)$$

where the coefficients are functions of time. It is shown in Appendix II that the differential equation for $a_n(t)$ has the form

$$d^2a_n/dt^2 + \lambda a_n = R(t). \qquad (20)$$

This is a standard form of the equation for forced oscillations, and its solution is known for many different kinds of functions R(t).

*Derivation in Appendix I

It is shown in Appendix II, that if

$$F_x = \partial F_x/\partial t = h = \partial h/\partial t = 0 \tag{21}$$

at time t = 0, then

$$h = \frac{1}{g}\sum\left\{\int_0^L \nu_n \int_0^t F \operatorname{Sin} \nu(t-T)dT\, \Phi_n dx\right\} \Phi_n(x) \tag{22}$$

A study of the observations shows that there are frequent periods when these assumptions are justified.

This equation gives the theoretical relation between the applied meteorological forces and the displacement of the free surface of a lake. It provides the connection between the seiche theories and the wind tide theories which is necessary to the development of any logical forecasting system.

BEHAVIOR OF THE LAKE IN RESPONSE TO CERTAIN SIMPLE FORCES

The significance of equation (22) will be clarified by a few simple examples. If a steady state solution exists, it can be found by integrating the first equation of motion in the proper form (see Appendix I). The resulting expression is

$$h(x) = h_0 + F(x)/g. \tag{23}$$

This is equivalent to the wind tide equation given by Keulegan (1951, 1953). This expression can be expanded in a series of the form

$$h = \sum\left[\int_0^L F(x)\,\Phi(x)\,dx\right]\Phi_n(x) \tag{24}$$

If we assume that a constant force is suddenly imposed on a quiet lake, it is found that

$$h = \sum\left[(1-\operatorname{Cos}\nu_n t)\int_0^L F(x)\,\Phi_n(x)\,dx\right]\Phi_n(x) \tag{25}$$

The coefficient of each mode of oscillation fluctuates around its steady state value with an amplitude equal to the steady state value, so that the resulting displacement reachs a maximum of approximately twice the steady state value. In the absence of damping it is not obvious that a steady state condition can ever be achieved.

In figure 10, it is noticed that on Lake Erie, the force function rose from a value near zero to a maximum and returned to zero several times. The simplest mathematical expression for a force of this kind is given by

$$\begin{aligned} F &= 0 && \text{for } t \leq 0 \\ &= F^*(x,y)(1-\cos\omega t) && \phantom{\text{for }} t \geq 0 \end{aligned} \tag{26}$$

In this case we have

$$h = \sum\left\{\left[1-\frac{\omega^2 \operatorname{Cos}\nu t-\nu^2 \operatorname{Cos}\omega t}{\omega^2-\nu^2}\right]\int_0^L F^*\,\Phi(x)\,dx\right\}\Phi(x) \tag{27}$$

The term in square brackets is obviously a function of the ratio ω/ν. If we set $\omega = \theta\nu$, this becomes

$$a_n(t,\theta) = \left[1 - \frac{\theta^2 \cos \nu t - \theta\nu t}{\theta^2 - 1}\right] \tag{28}$$

A plot of $a_n(t,\theta)$ for low values of t and various values of θ is given in figure 14. For small values of θ, corresponding to forces with a period long in proportion to the natural period, the lake is always in approximate equilibrium with the applied force. For large values of θ, corresponding to forces whose period is short compared to the natural period, the lake behavior approximates that which would result from the sudden imposition of a constant force with the same mean value.

As θ approaches unity, the amplitude of $a_n(t,\theta)$ grows because of the term $(\theta^2 - 1)^{-1}$. The apparent period varies with time, but the average period agrees with the period of the forcing mechanism, or with the natural period, whichever is greater, and the water level disturbance often becomes out of phase with the applied force.

In the above discussion, only one harmonic of the atmospheric force has been considered. The natural atmospheric disturbances may be considered as the sum of many component eddies. These eddies vary in size, speed, and other characteristics. Only those eddies whose size is comparable to that of the lake can be efficiently expanded in terms of the eigenfunctions. The effects of those eddies which are much smaller than the lake are observed as "noise" or interference superimposed on the basic pattern of the water level movements determined by the larger eddies. This noise will always lead to some error in the forecast, and useful forecasts will be possible only in regions in which the amplitude of the noise is small compared to that of the basic disturbance.

This noise appears in the lake level records as fluctuations of the lake level with periods that are very short compared to the fundamental natural period of the lake. It is believed that most of the variations in lake level shown in figures 2, 5, 6, 7 and 8 are due to noise of this type.

There is little evidence of noise of this type in the records for Buffalo, Toledo, and Gibraltar at the ends of Lake Erie. Hence it appears that useful forecasts of the lakewide disturbances on Lake Erie should be possible. However, some additional development work is still needed. Equation (22) should provide the starting point for the development of a practical forecasting procedure.

However, the available records for the other Great Lakes indicate that the amplitude of the noise is usually as great as or greater than that of the lakewide oscillations. In view of this fact, it appears unlikely that useful forecasts of the lakewide disturbances on these lakes can be based on the available records.

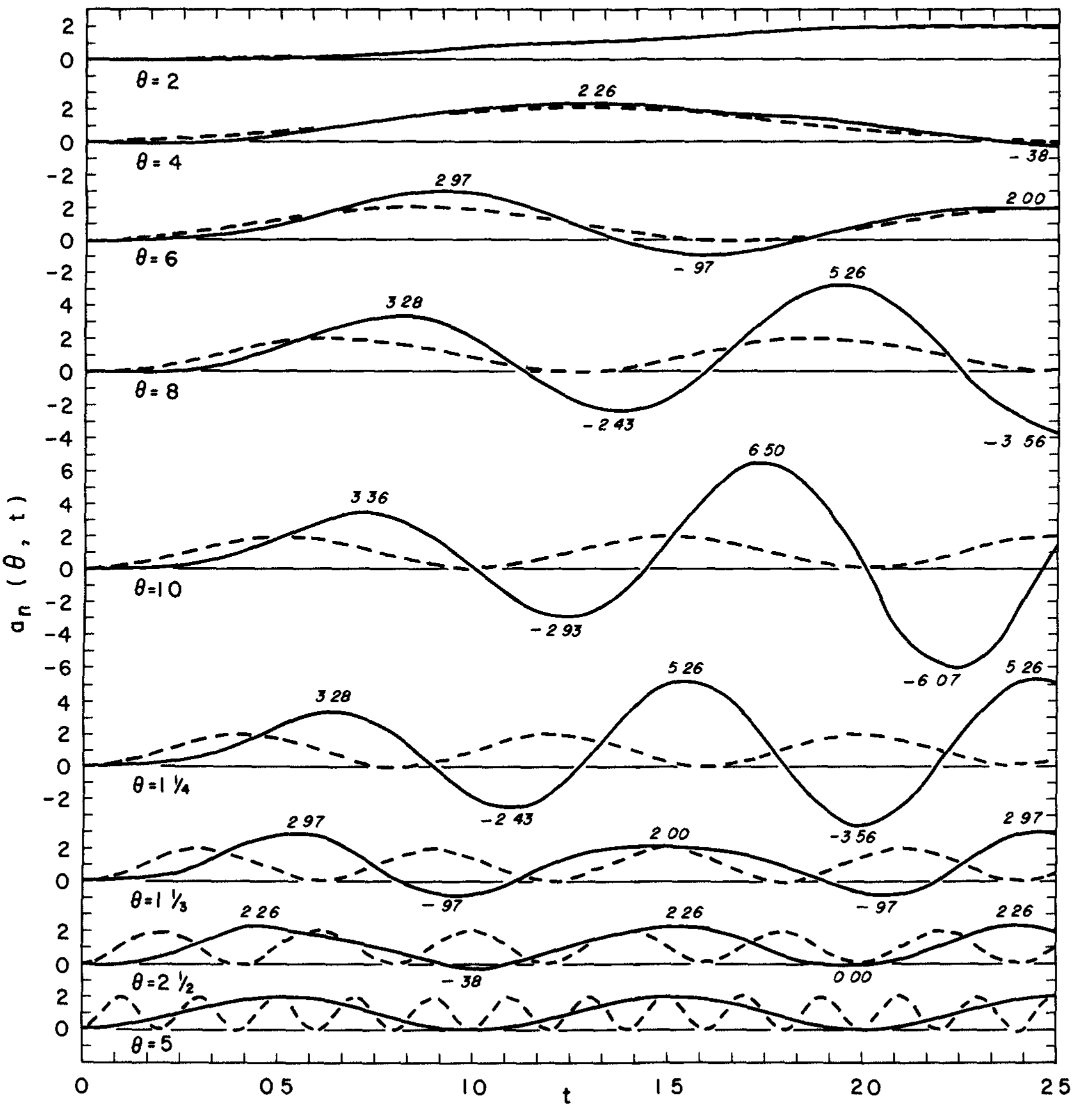

Fig. 14. Plot of $a_n(\theta,t)$ for several values of θ and low values of t. The dashed line gives the value of an applied force. The solid line shows the relative displacement of the lake surface at a point. The unit of time is the free period of oscillation. The slanted numbers inside the chart give the extreme values of the displacement.

MOVING DISTURBANCES

Proudman (1929) has shown that resonance will occur between the atmosphere and a lake of constant depth if the atmospheric disturbance moves with the same speed as the long waves in the water. It can also be shown by the theory developed in this report, that resonance should occur in a rectangular basin of constant depth, if the wave length of the atmospheric disturbance corresponds to the distance between two successive nodes of any mode of oscillation of the lake.

Since the speed of long waves, and the effective wave length of the large scale lake disturbances depend on the depth of the lake, it appears that in a natural lake, these parameters vary from point to point so that a temporary state of resonance between the atmosphere and the lake can occur in some sections of the lake without having much effect on the lake as a whole. These disturbances, once created, are dispersed throughout the lake according to the laws of solitary waves.

HARBOR DISTURBANCES

The differential equation for the disturbances of harbors and other small basins opening into a larger body of water is the sane as that for disturbances in the entire lake. However the boundary conditions are somewhat changed. For seiches in a harbor opening onto a smooth lake, the boundary conditions are

$$\partial h/\partial n = 0 \text{ on the closed boundary} \tag{29}$$

$$h = 0 \quad \text{at the harbor opening} \tag{30}$$

There are mouth effects similar to those found in the study of organ pipes in acoustics, and the effective harbor opening will usually be displaced slightly toward open water, from the geometrical opening of the harbor.

In the general case, however, the lake surface is not smooth, and the boundary condition at the opening becomes

$$h_{harbor} = h_{lake} = h(t) \tag{31}$$

In this case, the oscillations of the lake at the harbor entrance will force an oscillation of the harbor with the same frequency. Since the boundary condition (29) requires that the elevation of the water surface must be maximum or a minimum on the closed boundary, it is evident the amplitude of the disturbance must be greater at the closed boundary of the harbor than at the opening.

By means of a suitable change in the dependent variable, it is possible to transfer the effect of the lake oscillation into the non-homogeneous term of the differential equation of the harbor oscillation. Here it will be added to the effects of the wind stress and the atmospheric pressure gradient to determine a total forcing term. The solution may then be expanded in a series of the eigenfunctions of the harbor defined by the differential equations for seiches and boundary conditions (29), (30), as in the case of an entire lake.

If the periods of any of the individual components of the noise, referred to above, approach any free period of oscillation of any harbor or other restricted basin opening into the lake, that component will be amplified by resonance within the harbor. From the study of a great many records of water level variations in harbors, similar to those shown in the first part of this report, it appears to the writer that most of these disturbances were established in this way. That is to say, atmospheric disturbances give rise to a wide spectrum of disturbances on the open lake. All of these are somewhat amplified by convergence and some are greatly amplified by resonance within the harbor. This hypothesis can explain the following observed characteristics of the short period disturbances.

(1) All harbors in the same area tend to become excited at approximately the same time.

This is to be expected if all harbors are forced by the same agency, either lake or atmosphere.

(2) The characteristic period appears to be different in each harbor.

This is to be expected if the disturbance is due in any way to resonance.

(3) The harbor disturbances may occur as much as an hour or two before or after the apparently associated atmospheric disturbance passes the harbor.

(4) Harbor disturbances may occur when no atmospheric disturbance passes the immediate area of the harbor.

These latter two characteristics imply that the energy of the disturbance must be communicated to the lake at some distance from the harbor and advected to the harbor in the form of a water level disturbance in the lake.

If this theory is correct, it may be impossible to forecast these short period oscillations primarily from meteorological considerations. However, the situation is not altogether hopeless, since it would be possible to minimize the undesirable features of these short period fluctuations of water level in harbors by changes in the harbor design. In order to accomplish this, it would be necessary to observe the spectrum of disturbances occurring in the open lake near the harbor entrance, and to make sure that all changes in harbor design tend to decrease the resonance between the harbor and the more common frequencies of the open lake. The importance of this subject has been discussed more fully by Vanoni and Carr (1951), Carr (1952), McNown (1952), and McNown, Wilson, and Carr (1953).

All disturbances which do not have a node at the harbor entrance, may be regarded as progressive waves which enter the harbor, are reflected by the opposite shore, and after some attenuation in amplitude emerge from the harbor. Since the volume of water carried by the waves is a function of their amplitude, it is evident that the waves carry more water into the harbor than they carry out. While most of this excess water will eventually be carried out of the harbor by a gravity current, the average water level inside the harbor will be greater than at the entrance.

It appears that the water level in all harbors should be greater than that in the open lake, but that the amount of this difference would depend on the amount of water carried into the harbor by wave action. Thus on calm days this difference would be vanishingly small, but on disturbed days it would increase by an amount depending on the meteorological situation and the harbor exposure.

CONCLUSION

This investigation was undertaken to determine the cause of the short period oscillations, and to evaluate the possibility of forecasting oscillations of both long and short periods.

The ultimate cause of the short period oscillations has not been definitely determined. However, a theory which is consistent with the observations and with hydrodynamic principles has been presented. The validity of this theory can be adequately tested only by additional field and laboratory studies planned for this purpose.

It appears that useful forecasts of the larger long period oscillations of Lake Erie should be possible. It appears unlikely that useful forecasts of the short period disturbances or of the long period disturbances on any of the lakes other than Erie will be possible within the next few years.

The discussion of the representativeness of the lake level as measured by gages located in harbors was an unexpected by-product of this study, and no attempt has been made to evaluate the discrepancies between the records of nearby gages in engineering terms. However, it is believed that an effort should be made to locate gages in the open lake whenever this is possible.

ACKNOWLEDGMENTS

The following organizations have assisted in this study by supplying data used in the observational section of this report. The U. S. Lake Survey; Department of Water and Sewers, City of Chicago; Division of Waterways, Department of Public Works and Buildings, State of Illinois; and the Stone Institute of Hydrobiology, Put-in-Bay, Ohio. All of these organizations have cooperated in this study to the fullest extent, and we wish to express our sincere appreciation for their assistance in this project.

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APPENDIX I, DERIVATION OF THE ONE DIMENSIONAL EQUATIONS FOR THE OSCILLATIONS OF A LAKE

By integration from one side of the lake to the other it follows from equations (1) and (3) that

$$\int_{y_1}^{y_2} \frac{\partial u}{\partial t} dy = -g \int_{y_1}^{y_2} \frac{\partial h}{\partial x} dy + \int_{y_1}^{y_2} F_x dy \qquad \text{I-1}$$

$$\int_{y_1}^{y_2} \frac{\partial h}{\partial t} dy + \int_{y_1}^{y_2} \frac{\partial Du}{\partial x} dy + \int_{y_1}^{y_2} \frac{\partial Dv}{\partial y} dy = 0 \qquad \text{I-2}$$

where y_1 and y_2 are single valued functions of x forming the boundary of the lake and the Coriolis term is neglected.

A theorem in advanced calculus states that

$$\frac{\partial}{\partial x} \int_{y_1}^{y_2} G(x,y)dy = \int_{y_1}^{y_2} \frac{\partial G}{\partial x} dy - G(x,y_1)\,\partial y_1/\partial x + G(x,y_2)\,\partial y_2/\partial x \qquad \text{I-3}$$

where G is any function of x and y. The condition that no fluid pass through the boundary requires that

$$v(y_1) = u(y_1)\,\partial y_1/\partial x, \text{ and } v(y_2) = u(y_2)\,\partial y_2/\partial x \qquad \text{I-4}$$

By repeated use of I-3 and I-4, equations I-1 and I-2 may be transformed into

$$\int_{y_1}^{y_2} u\, dy = -g \frac{\partial}{\partial x}\int_{y_1}^{y_2} h\, dy + \int_{y_1}^{y_2} F_x dy + g h(y_1)\partial y_1/\partial x - gh(y_2)\partial y_2/\partial x \qquad \text{I-5}$$

$$\frac{\partial}{\partial t}\int_{y_1}^{y_2} h\, dy + \frac{\partial}{\partial x}\int_{y_1}^{y_2} D u\, dy = 0 \qquad \text{I-6}$$

We introduce the new variables

$$\overline{D} = \frac{1}{b} \int_{y_1}^{y_2} D dy \qquad \overline{F}_x = \frac{1}{b} \int_{y_1}^{y_2} F_x dy$$

$$\overline{u} = \frac{1}{b} \int_{y_1}^{y_2} u dy \qquad \overline{h} = \frac{1}{b} \int_{y_1}^{y_2} h dy$$

where $b = y_2 - y_1$, and set

$$h(y_1) = \bar{h} + \epsilon_1 \qquad\qquad h(y_2) = \bar{h} + \epsilon_2$$

With the aid of these substitutions, we may write equations I-5 and I-6 in the form

$$\partial\bar{u}/\partial t + g\partial\bar{h}/\partial x = \bar{F}_x + gb^{-1}(\epsilon_2\,\partial y_2/\partial x - \epsilon_1\,\partial y_1/\partial x) \qquad \text{I-7}$$

$$b\,\partial\bar{h}/\partial t + \partial(b\bar{D}\bar{u})/\partial x = 0 \qquad \text{I-8}$$

In most cases the term in parentheses in equation I-7 is small compared to the terms on the left and can be neglected. By neglecting this term and eliminating u between these equations, we obtain

$$\partial(A\,\partial\bar{h}/\partial x)/\partial x - bg^{-1}\,\partial^2 h/\partial t^2 - g^{-1}\,\partial(A\bar{F}_x)/\partial x, \qquad \text{I-9}$$

where A is the cross sectional area. Equation (17) is equation I-9 excepting that the bars indicating average values have been omitted. If the force term on the right is omitted, this corresponds to the equation for wave motion in a canal of variable section given by Lamb (1932, p. 273-274).

When the force term is omitted, $\bar{h}$ may be written as the product of two functions of the form

$$\bar{h} = \bar{\Phi}(x)\,\bar{T}(t) \qquad \text{I-10}$$

where $\bar{T}(t)$ is periodic, and the differential equation for Φ is given in the form

$$\partial(A\,\partial\bar{\Phi}/\partial x)/\partial x + \lambda bg^{-1}\,\bar{\Phi} = 0. \qquad \text{I-11}$$

When the bars are omitted, this becomes equation (11).

The steady state solution for one dimension is obtained from I-7 by setting $\partial\bar{u}/\partial t = 0$ and neglecting the term in parentheses. This gives

$$\partial h/\partial x = g^{-1}F_x,$$

or

$$h = g^{-1}\int_0^x F_x dx = g^{-1}F \qquad \text{I-12}$$

which gives equation (23).

If we neglect the terms on the right side of equation I-7 and use I-8 to eliminate h from I-7, we obtain the differential equation for u in the form

$$\partial^2 u/\partial t^2 - (gbA)^{-1}\,\partial^2 u/\partial t^2 = 0 \qquad \text{I-13}$$

By introducing a new independent variable defined by the relation

$$v = \int_0^x b\,dx \qquad \text{I-14}$$

and assuming that u is periodic in time this may be transformed into

$$d^2(Au)dx^2 + \nu^2(gbA)^{-1}(Au) = 0 \qquad \text{I-15}$$

The boundary condition for this equation becomes

$$(Au) = 0 \text{ at } x = 0 \text{ and } x = L \qquad \text{I-16}$$

when (Au) is known, h may be found by the expression

$$h = b^{-1}d(Au)/dx. = d(Au)/dv \qquad \text{I-17}$$

Equations I-15 through I-17 are in the form given by Chrystal. This appears the most convenient form of the equations for the computation of the one dimensional seiche functions. It is not as convenient as equation I-11 for generalization to two dimensions or for the study of forced seiches.

It will be instructive to use equation I-15 for an investigation of the seiches of a circular lake of constant depth and to compare the results with the complete solution as given by Lamb (1932, p. 284 ff) and McNown (1952). If the origin of coordinates is taken at the center of the circle, the width of the circle $2y = 2(a^2-x^2)^{1/2}$, and equation I-15 becomes

$$d^2(Au)/dx^2 + \nu^2\left[Dg(a^2-x^2)\right]^{-1}(Au) = 0 \qquad \text{I-18}$$

with the boundary condition

$$(Au) = 0 \text{ at } x = \pm a.$$

The boundary conditions and the coefficients of the differential equation are all even functions. Hence the solution must be an even function and all solutions will have either maxima or minima at x = 0. The first derivative of the solution therefore must vanish along the y axis. This implies that the vertical displacement at the center of the circle must always be zero. It is well known, however, that there are modes of oscillation in which all nodal lines are circles, and the maximum displacement of the free surface occurs at the center of the circle. This result can be generalized to apply to any basin which is symmetric with respect to some axis. It appears that the one dimensional theory can not even approximate the solution of modes of oscillation, whose nodes do not intersect the shore.

APPENDIX II, SOLUTION OF THE ONE DIMENSIONAL EQUATION FOR THE FORCED OSCILLATION OF A LAKE

The method employed here involves the expansion of the differential equation for forced motion, equation (17), in terms of the eigenfunctions of the lake, obtained as solutions of equation (11). In order to accomplish this we define two new quantities

$$F = \int_0^x F_x d_x \qquad \text{II-1}$$

$$\psi = F - gh \qquad \text{II-2}$$

By means of these substitutions equation (17) is transformed into

$$g\,\partial(A\,\partial\psi)/\partial x - \partial^2\psi/\partial t^2 = -bg^{-1}\,\partial^2 F/\partial t^2 \qquad \text{II-3}$$

with the boundary conditions

$$\partial\psi/\partial x = 0 \quad \text{at } x = 0, \text{ and } x = L \qquad \text{II-4}$$

If we assume that ψ may be expressed in the form

$$\psi = \Phi(x)T(t) \qquad \text{II-5}$$

where Φ is a solution of equation (11), equation (22) may be expressed in the form

$$\Phi(x)\ (\lambda T + \partial^2 T/\partial t^2) = \partial^2 F/\partial t^2. \qquad \text{II-6}$$

We define two functions of n and t by the relations

$$a_n(n,t) = \int_0^L \psi\,(x,t)\,\Phi_n(x)dx \qquad . \quad \text{II-7}$$

$$R_n(n,t) = \int_0^L \left[\partial^2 F(x.t)/\partial t^2\right]\Phi_n(x)dx \qquad \text{II-8}$$

where $\Phi_n(x)$ is the nth normalized eigenfunction defined by equation (11). Thus we see that $\Phi(n,t)$ and $R(n,t)$ are the generalized Fourier coefficients of $\psi(x,t)$ and $\partial^2 F/\partial t^2$, when these quantities are expressed as a series of the eigenfunctions, Φ_n. These coefficients are functions of time. By introduction of equations II-5, II-7, and II-8 into equation II-6 , we obtain the differential equation for $a_n(t)$ in the form

$$d^2a_n/dt^2 + \lambda a_n = R_n(t) \qquad \text{II-9}$$

where we can use the symbol for a total derivative since a_n depends only on t. This is a standard form of the equation for forced oscillations. The general solution of equation II-9 may be given as

$$a_n = a_{n,1} + a_{n,2} \qquad \text{II-10}$$

where

$$a_n = a_n\Big]_{t=0}\cos\nu_n t + da_n/dt\Big]_{t=0}\nu_n^{-1}\sin\nu_n t: \qquad \text{II-11}$$

and depends only on the initial conditions.

$$a_n = \frac{1}{\nu_n}\int_0^t R(\tau)\,\text{Sin}\,(t-\tau)\,d\tau \qquad \text{II-12}$$

and depends only on the applied force.

If the initial conditions are known, $a_{n,1}$ can be evaluated for any future time by standard methods. In the following discussion it will be assumed that $a_{n,1} = 0$, and only $a_{n,2}$ will be discussed. This is equivalent to assuming that at the time $t = 0$, a_n and da_n/dt are both zero. Observations show that there are frequent periods in which this assumption is valid, and we may choose $t = 0$ in any such period.

Equation II-9 is the differential equation for the coefficient of a single mode of vibration. The complete solution is composed of the sum of all modes of vibration. Hence the complete solution for x under the condition assumed here is

$$\psi = \sum\left\{\int_0^t\int_0^L\left[\partial^2 F(T)/\partial t^2\right]\Phi_n(x)\,\mathrm{Sin}\,(t-T)\,dx\,dT\right\}\Phi_n\,(x) \qquad \text{II-13}$$

Since $h = (F - \psi)/g$, and it is permissible to change the order of integration, we have

$$h = \frac{1}{g}\sum\left\{\int_0^h\left[F-\int_0^t\frac{\partial^2 F(T)}{\partial t^2}\,\mathrm{Sin}(t-T)\,dT\right]\Phi_n(x)\,dx\right\}\Phi_n(x) \qquad \text{II-14}$$

After performing the integration this becomes II-15

$$h = \frac{1}{g}\sum\left\{\int_0^h\left(\frac{\partial F}{\partial t}\right]_{t=0}\mathrm{Sin}\,\nu_n t + F\Big]_{t=0}\mathrm{Cos}\,\nu_n t + \nu\int_0^t F\,\mathrm{Sin}\,\nu(t-T)\,dT\right)\Phi_n\,dx\right\}\Phi_n(x)$$

If F and $\partial F/\partial t = 0$ at $t = 0$, this becomes

$$h = \frac{1}{g}\sum\left\{\int_0^L\nu_n\int_0^t F\,\mathrm{Sin}\,\nu\,(t-T)\,dT\,\Phi_n\,dx\right\}\Phi_n(x) \qquad \text{II-16}$$

This is equation (22).

CHAPTER 4

THE MECHANISM OF SEICHES IN TABLE BAY HARBOR, CAPE TOWN

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ABSTRACT

Surging in Table Bay Harbor is shown to be favored by the peculiar location of the harbor within Table Bay and of Table Bay within the South Atlantic Ocean, and by the fortuitous fact that both the external and internal dimensions of the harbor accentuate the development of seiches. The roadstead comprising a three-sided quasi-basin permits resonant augmentation of several bay-seiches, notably those of about 11 and $5\frac{1}{2}$ minutes period, and certain harmonics among these are stimulated sufficiently to communicate themselves into the docks and achieve resonance there also. The periodicities of oscillations found by observation are explained theoretically and confirmed in model experiments. The largest basin of the harbor, during its construction period, exhibited different oscillating properties from those in evidence today. The differences are explained on the basis of the changing dimensions in shape and depth of the enclosed body of water. Model experiments again confirm the general mechanism of behavior.

INTRODUCTION

The occurrence of troublesome Range-action or surging in Table Bay Harbor, Cape Town, South Africa, during critical shipping years of World War II, at a time when this phenomenon was still improperly understood, led to an intensive model study (1943-46) conducted under the auspices of the South African Railways and Harbours Administration. Some results of these researches which have general applicability to harbors or bear more particularly upon Table Bay Harbor itself, have recently been published (Wilson, 1950, 1952, 1953); the present paper may be said to contain another phase of the general findings.

The same problem, as affecting harbors in this country and elsewhere in the world, has received considerable attention in the last decade and the essential principles of the action taking place during surging are now well understood (Neumann, 1948; Irribarren, 1949; Vanoni, 1951; Knapp, 1952; Carr, 1952; McNown, 1952; and Wilson, 1953 (ii)).

Briefly, the phenomenon may be attributed to the development of coastal seiches in bays, inlets or semi-enclosed bodies of water as a result of the ingress and reflection of long waves or ground swells whose energy cannot normally be dissipated in surf and by attrition along the coast.

Every enclosed or semi-enclosed body of water has natural periods of oscillation which depend entirely upon its dimensional characteristics such as the surface configuration and the topographical features of the bed, as affecting the depth. A measure of agreement between the impressed frequency of the pressure or ground-swell excitation and the natural frequency of the water-body may be all that is required to stimulate the development of a seiche. In general seiches tend to occur in families of related frequencies, which, in the special cases of basins with simple geometrical shapes, such as the rectangular forms of uniform depth, constitute a pure harmonic series; ordinarily, however, the relationship is more complex. (Chrystal, 1904-5).

A condition for oscillation of closed bodies of water is that antinodes or loops shall exist always at the extremities of the basins. In the case of basins which open upon larger bodies of water the modes of oscillation generally require nodal conditions at the mouth. However, if the opening to a basin is comparatively small in terms of the basin dimensions, response will be to modes of oscillation which are a combination of the modes for basins fully closed and fully open. The problem is somewhat complicated when the mouth itself attains to appreciable dimensions in length between the connected bodies (Neumann, 1950).

SEICHES OF TABLE BAY

Evidence secured from marigram records at Cape Town suggests that the submerged oceanic basins to the west of the South African coast (Fig 1) permit independent oscillations of the water masses within the pseudo-rectangular trench and triangular canyon (Wilson, 1953 (iii)). The inference is that these oceanic basins are responsive to fluctuating pressure changes arising within the east bound frontal depressions which traverse them in wave-like sequence and contribute a component of motion along the long axes of the basins.

Topographically, Table Bay itself is similar to a semi-ellipsoidal bowl with two openings in its side wall, parallel to its major and minor axes (Fig 1, inset). As the bay lies on the boundary of both the oceanic basins mentioned it would be subject to the forced seiches imposed by some of the higher modes of the oceanic oscillations. Several of these forced seiches seem capable of securing near resonant response from Table Bay, but of its own accord the Bay can beget its own system of free seiches, activated by wave or pressure disturbances whose frequencies are too high to make any sensible impression upon the larger water bodies in the oceanic basins.

Analysis of the oscillating properties of Table Bay from three standpoints, - theoretical, observational and experimental - seems to indicate that the important critical periods conform to the series (Wilson, 1953 (iii)): -

T = 71-66-57; 55-51; 43-36; 33-26; 23-17; 14-12; 11-10;
9.8-9.4; 8.3-7.8; 7.5-7.0; 6.8-6.5; 6.3-5.9; 5.7-5.4;
4.8-4.4; 4.3-4.1; 4.0-3.7; 3.6-3.4; 3.2-3.1; 2.9-2.7;
..... minutes ____ (1)

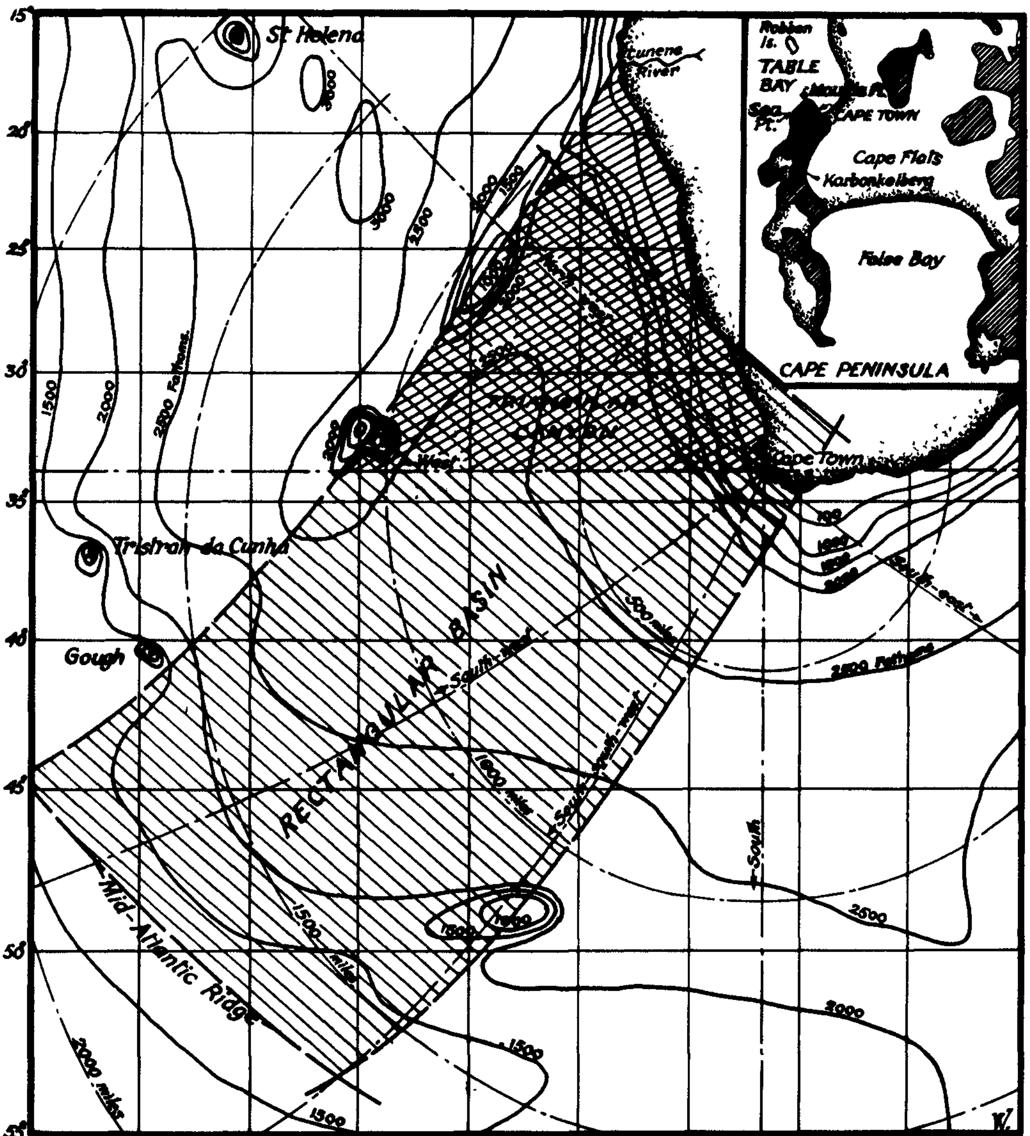


Fig. 1. Configuration of South Atlantic Ocean

These figures give the approximate ranges within which particular periodicities of oscillation tend to occur.

THE BREAKWATER - SHORE OSCILLATING SYSTEM

Just as the majority of the above frequencies cannot much influence the water-mass in the oceanic basins, so the highest frequencies of ground-swells (periods smaller than shown in the above sequence) cannot excite much response in the water-body of Table Bay. It requires a still smaller oscillating basin to encourage the development of such seiches, and this, as it happens, has been provided by the handiwork of man, in the form of an open sided basin between the harbor breakwater and the opposite shore (Fig. 2).

The depth of water and distance between the breakwater and the shoreline, the latter's parallelism with the breakwater, the rectangularity and axial disposition of the quasi-basin, all fortuitously favor the imposition of particular forced seiches of the bay and the reproduction of their families of dependent frequencies. Thus, in the sequence (1) of periodicities given above, the 11-10 minute seiche of the bay, its 22-20 minute overtone, and $5\frac{1}{2}$ minute harmonic, resonate almost perfectly in the breakwater-shore basin, greatly stimulating themselves in that corner of the bay and ensuring the promotion of yet higher-frequency seiches. This fact has now been fairly convincingly demonstrated by model experiments, refraction-diagram analysis of seiches, and by harmonic analysis of seichograms (Wilson 1953 (iii)) with results for the latter such as shown in Table I:

The regular shape and very gradually changing depth of water in the breakwater-shore basin ensure that the natural periods of longitudinal oscillation between the breakwater and shore boundaries will not be far removed from an harmonic series. In the knowledge, then, that the most important fundamental oscillation for the quasi-basin is about 11 mins., we could expect harmonics in the series 1, 1/2, 1/3, 1/4, . . . to be 11, 5.5, 3.7, 2.8, 2.2, 1.8, 1.57, 1.38, 1.22, 1.10, 1.00, 0.92 . . . minutes. A typical harmonic such as the quadrinodal one of 2.8 minutes would accommodate itself in the quasi-basin after the fashion of Fig. 3.

Forced seiches such as the predominant 13/6.5 minute and 9.5/4.7 combinations, falling within the periodic sequence, Equation (1), would find in the breakwater-shore system a receptacle for achieving near-resonance, and could be expected to yield their own harmonic trains to add to or reinforce the possible modes of oscillation of the water-mass in the harbor area.

A seiche of the order of 22 minutes period has been shown to have a node in line with the breakwater, and parallel to the coast (Wilson, 1953 (iii)), and may be likened in its effect on the breakwater-shore system to the fundamental seiche for an open basin with node at the mouth. As a forced seiche, it could thus be expected to impress itself upon the quasi-

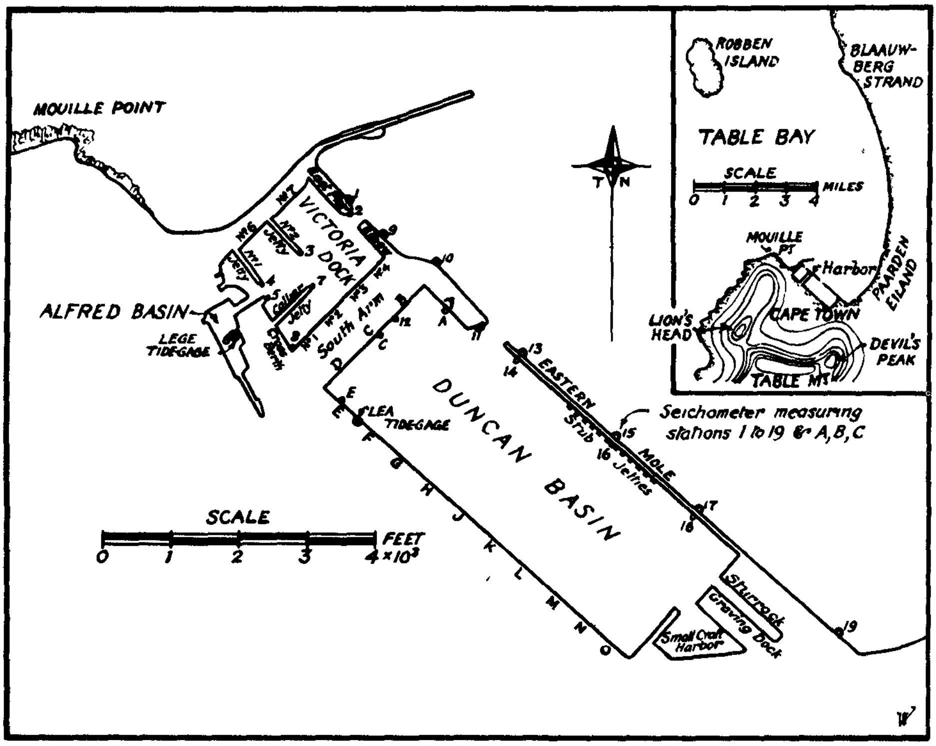


Fig. 2. Table Bay and harbor, Cape Town

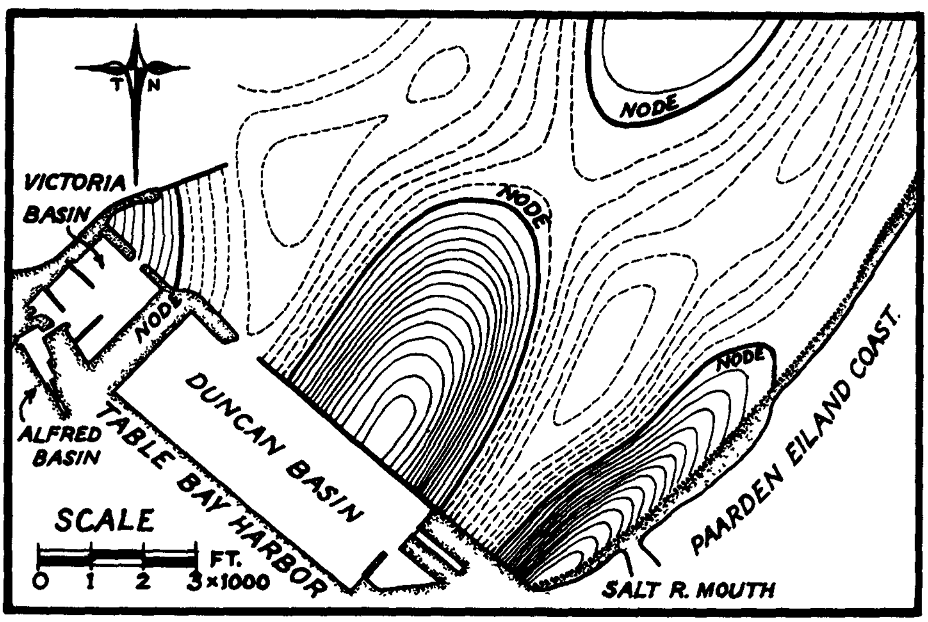


Fig. 3. Approx. instantaneous form of 2 3/4 min. sieche off Table Bay Harbor. (Graphical synthesis)

Table I

Apparent Periodicities of Component Oscillations in Seichograms Obtained by Method of Residuation (of Chrystal, 1906).

General Location	Recorder Station	Date of Record	Apparent Periodicities (Minutes)																						Analyst
Outside Eastern Mole near Shore.	15	Oct. 4/43	—	—	—	25.7	—	—	11.4	—	—	5.7	4.5	3.8	2.8	2.2	—	—	—	—	—	—	—	—	Wilson
	19	" "	—	—	—	25.7	—	—	11.4	8.6	—	5.7	4.5	3.8	2.8	2.2	—	—	—	—	—	—	—	—	Wilson
	19	Nov. 6/44	—	51	—	25.7	—	—	11.5	—	6.9	—	4.9	4.3	2.8	2.0	—	—	—	—	0.75	—	—	0.25	Joosting
	19	" "	58	—	—	29.0	—	—	11.1	—	—	5.6	4.3	—	—	2.2	—	—	1.06	—	0.71	0.62	0.49	—	Wilson
	19	" "	—	—	—	—	—	—	10.2	—	—	5.7	4.2	—	—	2.3	—	1.52	1.10	—	—	—	—	—	de Boer
	19	Nov. 8/44	—	—	—	—	18.6	—	10.9	—	7.4	5.5	—	3.5	—	2.1	—	—	1.06	—	—	—	—	—	Wilson
	19	Aug. 6/46	62	—	—	23.4	—	14.0	—	8.8	—	5.4	—	3.4	2.8	—	1.74	1.54	1.09	—	0.70	—	0.41	0.33	Joosting
	19	" "	—	—	—	—	—	14.5	11.0	—	—	5.5	—	3.8	2.5	—	—	—	1.10	—	—	—	—	0.36	Cowan
	19	Aug.23/46	62	—	—	22.7	—	13.6	—	9.2	—	5.6	—	—	3.0	2.5	1.50	1.33	1.07	—	0.76	—	0.41	—	Joosting
	19	" "	—	—	—	26.0	—	14.0	—	9.1	—	5.6	—	—	2.5	—	—	1.40	—	0.90	—	—	0.45	—	Cowan
Entrance of Duncan Basin	11	Aug. 1/44	—	—	—	—	—	16.0	—	—	—	5.6	4.2	—	2.7	2.0	1.90	—	1.00	—	—	—	0.51	—	de Boer
	11	" "	—	—	—	—	21.0	—	—	—	—	5.4	4.3	3.9	—	2.1	1.97	—	1.02	0.96	—	—	0.53	—	Joosting
	11	Aug. 3/44	—	—	38	—	19.0	—	—	9.5	—	5.3	4.9	—	2.5	2.3	—	1.25	1.00	—	—	0.60	0.50	—	de Boer
	11	" "	—	—	38	—	19.0	—	—	9.5	—	5.5	4.8	—	2.5	2.3	—	1.25	1.02	—	—	0.56	0.50	—	Joosting
	11	Oct. 1/44	—	—	43	29.0	21.3	17.0	—	—	6.8	—	4.3	—	—	2.0	—	1.35	—	—	0.68	—	—	0.25	Joosting
	11	June21/45	—	—	—	—	19.0	—	—	—	—	5.6	—	—	—	2.3	1.87	—	—	0.94	0.67	—	0.49	—	Joosting
	11	July 7/45	—	—	—	—	—	14.0	—	—	7.0	5.6	—	—	2.6	—	1.76	—	1.00	—	0.69	0.54	—	—	Joosting
	11	July31/45	—	—	—	—	21.0	—	—	—	—	5.9	—	3.5	—	—	1.87	—	1.05	—	0.71	—	0.50	—	Joosting
Breakwater Bight outside Victoria Basin	1	Oct. 1/44	62	—	—	31.0	19.2	12.1	10.1	—	—	—	4.1	—	—	2.2	—	—	—	—	—	—	—	—	Joosting
	1	Nov. 6/44	—	51	—	—	20.3	—	—	9.8	—	5.5	—	—	2.8	—	1.60	—	—	—	—	—	—	—	Joosting
	1	Nov. 8/44	—	—	—	—	21.0	—	11.0	—	—	5.8	—	3.5	2.9	—	1.68	—	—	—	—	—	—	—	Wilson
Inside Victoria Basin	4	Oct. 1/44	62	—	—	31.0	18.6	—	11.5	8.7	7.8	—	4.2	—	2.8	—	—	—	—	—	—	—	—	—	Joosting
	4	Nov. 6/44	—	51	—	22.5	—	—	11.2	9.5	—	5.5	4.1	—	2.8	—	—	1.39	—	—	—	—	—	—	Joosting
	8	June20/45	—	—	—	—	—	—	10.8	—	—	5.4	—	3.1	2.3	2.3	1.91	—	1.10	0.87	—	—	—	—	Joosting
Inside Duncan Basin	12	Aug.30/44	—	—	—	28.0	—	14.0	—	—	—	5.6	4.7	3.8	—	2.2	1.90	1.30	1.00	—	—	—	0.50	—	de Boer
	12	" "	—	—	—	28.0	—	14.0	—	—	—	5.5	4.8	3.4	—	2.2	1.89	1.42	1.00	0.91	—	—	0.50	—	Joosting
	12	Nov. 6/44	—	52	—	26.0	18.0	—	11.2	—	6.3	—	4.3	—	—	2.0	—	1.53	—	0.90	—	—	0.45	0.25	Joosting
	12	Nov. 8/44	—	—	—	27.5	—	13.7	—	—	—	5.5	4.1	—	—	2.1	1.68	1.22	—	—	0.71	—	—	—	Wilson
	12	" "	57	—	—	28.3	—	14.1	—	—	6.5	—	4.3	—	—	2.1	—	1.41	1.15	—	0.66	—	—	0.25	Joosting
	12/C	Aug. 6/48	—	—	—	—	—	14.0	—	—	—	5.9	4.0	—	—	—	—	—	—	0.95	—	—	—	—	Cowan
	12/C	Aug.23/48	—	—	—	—	—	—	—	—	—	—	4.0	—	—	—	—	—	1.05	0.80	—	—	—	—	Cowan
	A	Aug. 6/48	—	—	—	—	17.0	—	—	—	—	5.4	—	—	—	2.1	—	1.57	—	—	—	—	—	—	Cowan
	A	Aug.23/48	—	—	—	—	—	13.0	—	—	—	5.5	—	—	—	—	—	1.44	1.00	—	—	—	—	—	Cowan
	E/F	Aug.18/45	—	—	—	—	—	16.0	—	—	—	5.5	—	3.8	2.6	—	—	1.50	—	0.90	—	—	—	—	Cowan
	E/F	Nov.17/45	—	—	—	—	—	—	11.0	—	—	5.6	—	—	2.6	—	1.91	1.50	—	—	0.76	—	—	—	Cowan
	E/F	Dec. 7/45	—	—	—	—	—	—	11.0	—	—	5.6	—	3.5	—	—	1.75	1.55	—	0.93	—	—	—	—	Cowan
	14	Oct. 1/44	—	—	—	29.0	18.6	17.0	—	9.8	6.6	—	—	—	—	—	1.77	—	—	—	—	—	—	—	Joosting
	16	Oct. 1/44	—	—	—	29.0	19.1	12.7	—	—	—	5.0	—	—	—	—	1.66	—	—	0.89	—	—	0.43	—	Joosting
	16	Nov. 6/44	—	51	—	20.4	17.4	12.2	—	—	—	—	4.6	3.8	—	—	1.86	1.29	—	—	0.68	—	—	0.25	Joosting
	18	Oct. 1/44	—	—	—	29.0	19.1	12.2	—	—	—	—	4.7	—	—	—	1.66	—	—	—	—	—	0.43	—	Joosting

basin by begetting higher harmonics in the odd modes only so as to comply always with the requirement of a node lying in line with the breakwater. In this way harmonics would conform to the series 1, 1/3, 1/5, 1/7, . . . yielding periodicities approximating to 22, 7.3, 4.5, 3.1, 2.4, 2.0, 1.69, 1.47, 1.30, 1.16, 1.04, 0.96, ...

The co-existence of these several seiches for Table Bay with their developed families should then produce upon coalescence, in the immediate vicinity of the harbor, seiches of the average periodicities shown at the foot of Table II. The values have been derived from the figures tabulated by weighting twice in favor of the 11-minute seiche to make some arbitrary allowance for its more perfect resonance in the breakwater-shore quasi-basin.

Table II

Natural and Forced Periods of Oscillation for the Breakwater-Shore Quasi-Basin.

Funda-mental Forcd. Oscil.	Higher Harmonics (Minutes).													
22	7.3			4.5	3.1	2.4	2.0		1.69	1.47	1.30	1.16	1.04	0.96
13		6.5		4.3	3.2	2.6	2.2	1.86	1.63	1.44	1.30	1.18	1.08	1.00 0.93
11			5.5		3.7	2.7	2.2	1.83	1.57		1.38	1.22	1.10	1.00 0.92
9.5				4.7	3.2		2.3	1.90	1.58		1.36	1.19	1.05	0.95
All	7.3	6.5	5.5	4.5	3.4	2.6	2.2	1.86	1.61	1.45	1.35	1.19	1.07	0.96

The final row of figures, which, of course, are capable of some fluctuation up or down, bears favorable comparison with the apparent periodicities identified by residuation analysis in the seichograms for the area outside the harbor (Table I) and leads to the inference that this mechanism, in part at least, explains the existence of the multi-period sea movements outside the harbor basins.

PERIODOGRAMS FOR THE HARBOR AREA FROM MODEL TESTS.

The Range-action model of Table Bay Harbor, constructed to a coefficient of distortion (vertical exaggeration) of 8 (horizontal scale 1/1200, vertical scale 1/144), reproduced the whole of Table Bay from Mouille Point to Robben Island and Blaauwberg Strand (Fig 1). Two wave paddles across the west and north channel entrances to the bay were used to simulate the ingress of swells from the outer ocean. In the process of determining the appropriate adjustments for these paddles it was found that the amplitudes of the oscillations in the harbor area varied with periodicity in a manner which could not be attributed wholly to the changes introduced in the settings of the paddle machinery. The model was in fact functioning as an harmonic analyser in revealing the resonant frequencies of the harbor system.

An early experiment, aimed at determining some of the natural frequencies of the harbor, gave the periodograms of amplitude measurements shown in Fig 4 for three stations, one of which was in the Victoria Basin and the others in the Duncan Dock (see Fig 2). At the time that the test was made the best adjustments for the paddles had not been finally determined and the amplitudes of oscillations are not really comparable, quantitatively, over a wide span of periodicities, though comparisons are valid enough over short period-ranges. The remarkable increase in amplitude of the oscillations at certain periods is at once evident from Fig 4. The following critical periods reveal themselves:

$$T = 13.2,\ 11.5,\ 9.5\text{-}9.0,\ 8.5\text{-}7.6,\ 6.8\text{-}6.4,\ 5.7\text{-}5.4,\ 4.8\text{-}4.5,\ 3.6\text{-}3.4,\ 3.0\text{-}2.8,\ 2.4,\ (2.2),\ 2.0,\ 1.9\text{-}1.8,\ 1.6,\ \ldots \text{ minutes} \qquad (2)$$

These experimental results, insofar as they extend, provide good confirmation of the observed periods (Table I) and the theoretical interpretations placed upon them (Equation (1) and Table II). But later experiments, having reference more particularly to a periodic range from 1.0 to 8.0 minutes, afford more material for defining the critical frequencies.

Once the appropriate paddle-settings for the model had been worked out, the search to discover critical periodicities for the model harbor-basins involved recording the water oscillations at four different points in the model harbor for some 80 different wave-periods taken in succession. Each set of observations was repeated four times to simulate the effects of waves from four directions in the outer ocean, south-west, west-southwest, west and west-northwest, the paddles being appropriately slewed or altered to cover these cases. The results, embodied in the composite periodograms, Figs 5 (a) and (b), have been corrected for the distortional effects of the model (time scale) and converted to natural proportions. The range or double-amplitude of oscillations are far in excess of movements measured in the real harbor, but the exaggeration of wave effects in the model does not detract from the usefulness of the results which should be interpreted on a comparative basis only.

Two things are immediately noticeable about Fig 5: the first is that there are relatively isolated frequency bands of response in all the basins; the second, that there is not a great deal of difference in the response to waves from different directions in the external ocean. The critical periods in all cases are much the same, only the overall magnitude of the disturbances being appreciably affected by wave direction.

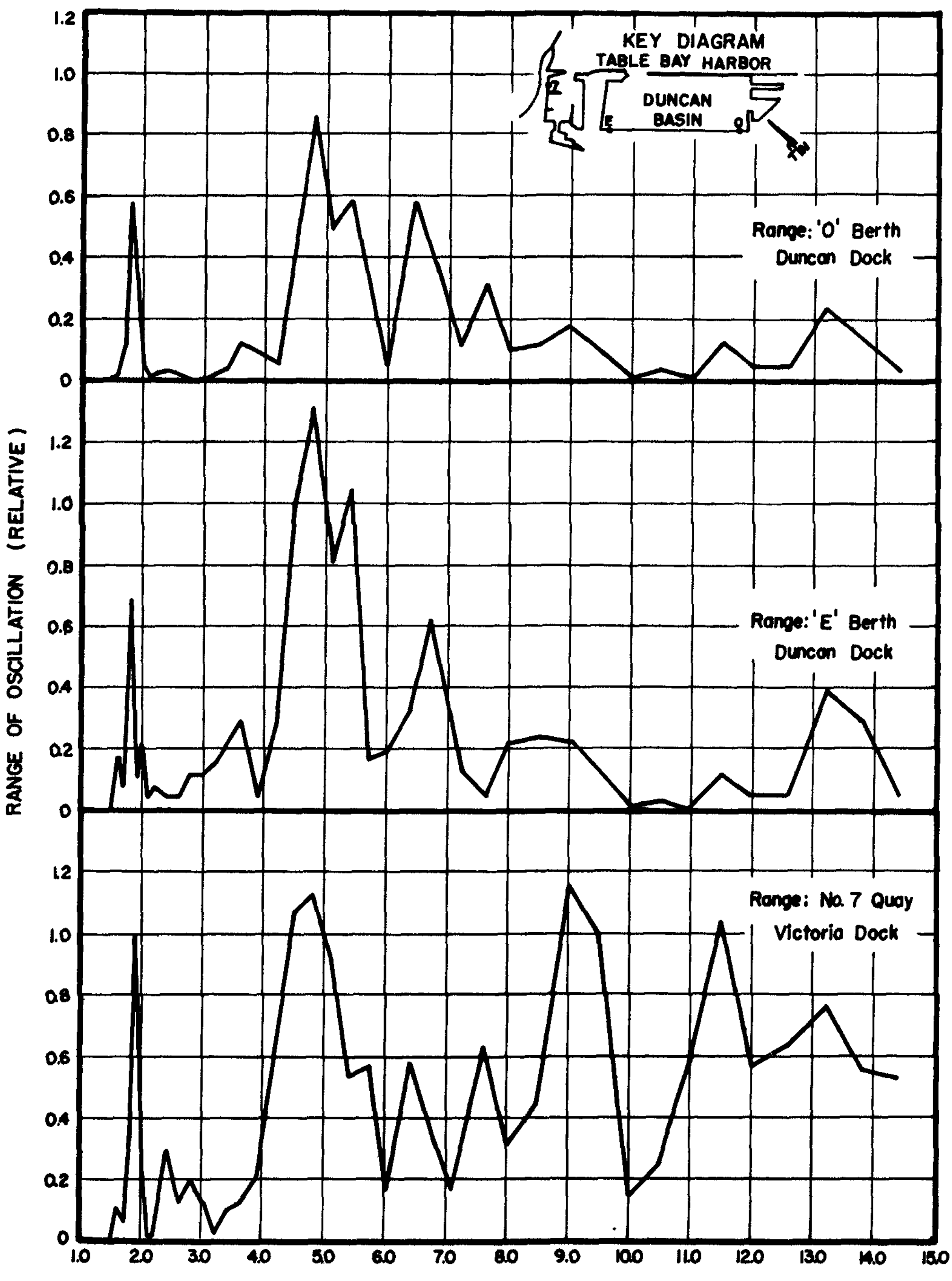
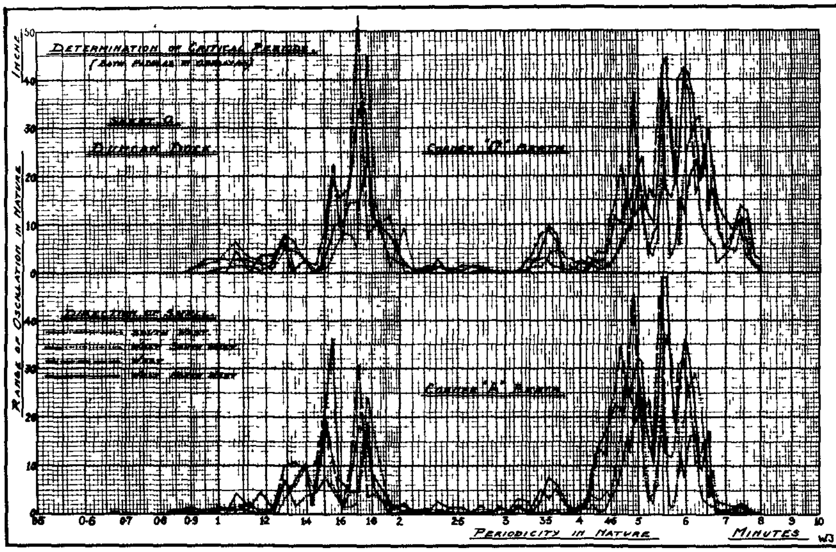
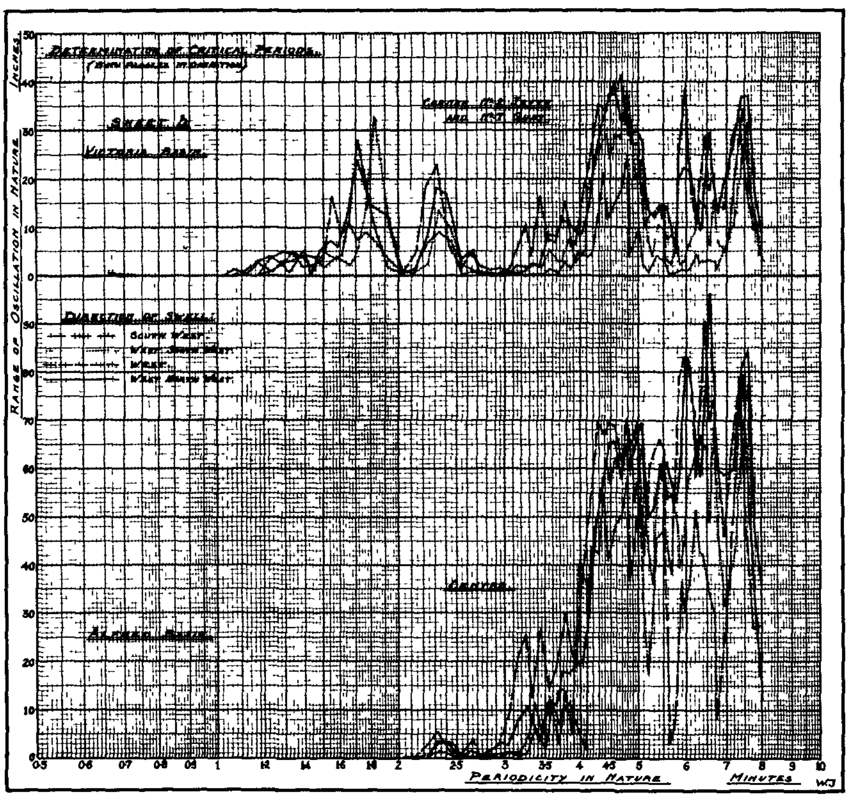


Fig. 4. Periodograms of response of model harbor to different wave frequencies. (Early model experiments)



(a) Duncan Dock



(b) Victoria and Alfred Basins

Fig. 5. Periodograms of response of model harbor to waves of different frequencies.

The explanation for these features is not difficult to descry. The harbor basins, clearly, only respond to those forced seiches of the external breakwater-shore system as agree reasonably closely with their natural frequencies. Further, the relative unimportance of wave direction derives from the refractive influence of Table Bay in causing swells that penetrate to the harbor to have only a small directional variation (Wilson, 1953 (iii)).

Taking the periodograms of Fig 5 collectively and grouping peak periodicities for the different wave directions whenever they are closely knit, we may identify what are obviously the forcing seiches of the breakwater-shore oscillating system. In this process, certain minor peaks justify inclusion since their apparent unimportance can be attributed to the unresponsiveness of the basins to those modes of the external oscillation. Certain other peaks, particularly the three occuring between 3 and 4 minutes in the Victoria and Alfred Basins (Fig 5 (b)), have been unified, since it is known from the model studies that these are resonance peaks for individual compartments of the Victoria Basin (cf Fig 2) and are thus peculiar to the inside of the harbor rather than to the outside. The resulting sequence of critical periods fluctuates round the following most significant values.

$$T = 7.5,\ 6.6,\ 6.0,\ 5.5,\ 4.7,\ 3.6,\ 2.6,\ 2.3,\ 1.81,\ 1.72,\ 1.55,\ 1.40,\ 1.30,\ 1.17,\ 1.07,\ 0.95,\ \ldots \text{ minutes} \tag{3}$$

This series, compared now with Table I, Equation (1) and Table II, but especially the latter, shows good correspondence and confirms the picture of the essential underlying structure of the phenomenon. However, there are one or two notable omissions in Table II, such as those of the 6.0 and 1.72 minute periodicities, which demand an explanation. The existence of the former, as proved by the model, could be attributed to the 6.3-5.9 minute forced seiche of the bay (Equation (1)) and would seem to indicate that a 12/6.0 minute near-resonant combination should have been included in the assembly of Table II. The seventh harmonic of a 12 minute fundamental oscillation would be 1.71 minutes and its comparative isolation from neighboring harmonics of the other fundamentals (22, 13, 11, and 9.5 minutes, Table II) could perhaps explain its independent occurrence. However, it will be shown later that the development in strength of this oscillation in the harbor basins can be aided from another source.

If Table II is amended to include a 12-minute fundamental oscillation and harmonics, as well as the independent higher harmonic seiches of the bay from Equation (1), with the arrangement and weighting shown in Table III, which makes some arbitrary allowance for the degree of 'resonance-fit' of the fundamentals within the breakwater-shore basin, the resulting sequence of arithmetic means gives the probable central values of the critical periods:

These are now the periodicities likely to be most active in producing sympathetic oscillations within the harbor and the bracketed figures or cumulative weights should give some idea (admittedly artificial) of their relative importance and likelihood of occurrence, though not necessarily of their relative amplitudes.

Table III

Periods of Natural and Forced Oscillation
for the Breakwater-Shore Quasi-Basin (Revised)

Fundamental Forced Oscillation (Mins)					Assigned Weight	Harmonics (Minutes)																	
22					(3)		7.3				4.5		3.1	2.4	2.0		1.69		1.47	1.30	1.16	1.04	0.96
	13				(2)			6.5				4.3	3.2	2.6	2.2	1.86		1.63	1.44	1.30	1.18	1.08	1.00 0.93
		12			(3)				6.0			4.0	3.0	2.4	2.0	.	1.71	.	1.50	1.33	1.20	1.09	1.00
			11		(4)					5.5		3.7		2.7	2.2	1.83	.	1.57	.	1.37	1.22	1.10	1.00 0.92
				9.5	(2)						4.7		3.2	.	2.3	1.90	.	1.58	.	1.36	1.19	1.05	0.95
						8.0 (1)	7.3 (3)	6.6 (2)	6.1 (3)	5.6 (4)	4.6 (2)	4.2 3.9 (2)	3.5 3.2 (2)	2.8 (4)									
22 (3)	13 (2)	12 (3)	11 (4)	9.5 (2)		8.0 (1)	7.3 (6)	6.5 (4)	6.0 (6)	5.5 (8)	4.6 (7)	4.0 (13)	3.2 (14)	2.6 (16)	2.1 (14)	1.86 (8)	1.70 (6)	1.59 (8)	1.47 (8)	1.33 (14)	1.19 (14)	1.08 (14)	0.96 (20)

THE NATURAL MODES OF OSCILLATION FOR THE DUNCAN BASIN

The extent to which the forcing seiches just considered will communicate themselves into the harbor will depend very largely upon the oscillating properties of the harbor basins and the location of the latter in relation to the external seiches. The Duncan Dock which is vertical walled, of very regular shape (Fig 2), and of almost uniform depth provides a good case for the application of the hydrodynamical theory of seiches to the determination of the modes of free oscillation. Thus the periods, T_m, of the several modes of oscillation along the axes of a rectangular basin of length, L, breadth, B, and uniform depth, d, are given by the equation (Sverdrup, 1942): -

$$T_m = \frac{1}{m}\cdot\frac{2L\ (\text{or } B)}{gd} \qquad (4)$$

in which g is the gravitational acceleration, and m an integer of successive values 1, 2, 3, . . . , defining the mode. The numerical evaluation of T_m for the Duncan Basin according to (4) is given in Table IV:

Table IV

Natural Periods of Duncan Basin from Hydrodynamical Theory

Direction	Dimensions		Computed Periods (for values of m)-Minutes						
	L or B	d	1	2	3	4	5	6	7
Length	6000	40	5.57	2.78	1.85	1.37	1.11	0.93	0.79
		45	5.26	2.63	1.75	1.31	1.05	0.88	0.75
Breadth (northend)	2100	40	1.95	0.97	0.65	0.49	0.39	0.33	0.28
		45	1.84	0.92	0.61	0.46	0.37	0.31	0.26
Breadth (southend)	2200	40	2.04	1.02	0.68	0.51	0.41	0.34	0.29
		45	1.93	0.96	0.64	0.48	0.39	0.32	0.28

Equation (4) however, is really a special case of a more general relationship for the two dimensional oscillations in a rectangular basin of uniform depth (Lamb, 1932), namely: -

$$\left.\begin{aligned} &\text{(i)} \quad \frac{4L^2}{T_{mn}^{\;2}gd} = m^2 + \frac{n^2}{\beta} \\ &\text{(ii)} \quad \beta = (B/L)^2 \end{aligned}\right\} \qquad (5)$$

Here m and n are integers of possible values 0, 1, 2, 3, . . . defining the nodality along the longitudinal and transverse axes respectively. The natural periods already given in Table IV correspond to the cases, taken separately, of $n = 0$ and $m = 0$. As the length-breadth ratio, L/B , for the Duncan Basin is very closely 3, we find, on taking $\beta = 1/9$ in equation (5), the period-values shown in Table V for some of the remaining cases when m and n are both finite: -

Table V

Natural Periods of Duncan Basin: Two-Dimensional Oscillation

Depth d	Computed Periods (for different values of m and n)-Mins.									
	m = 1	2	3	4	5	6	1	1	1	2
	n = 1	1	1	1	1	1	2	3	4	2
40	1.76	1.55	1.31	1.11	0.96	0.83	0.92	0.62	0.46	0.87
45	1.67	1.46	1.24	1.05	0.90	0.78	0.86	0.58	0.44	0.83

Only the first few important periods are incorporated in the above table, because other values of m and n tend to give overlapping periods which are either indistinguishable from those given or approximate to the values already listed in Table IV.

Besides the natural periods shown in Tables IV and V, which are those for a closed basin, there will exist modes of oscillation for the Duncan Dock as an open basin, the applicable equation in this case being (Lamb, 1932):

$$T_s = \frac{1}{s} \cdot \frac{4L \text{ (or B)}}{gd} \qquad (6)$$

where s is an integer having only odd values 1, 3, 5, ... Equation (6) is strictly applicable to an open-ended rectangular canal of uniform depth, but since the entrance of the Duncan Basin is on the long side towards the north end and the diagonal distance from the mouth to the far corner approximately equals the length of the basin, L , the application of (6) may be considered admissable as an approximation. This is favored too by the fact that the basin mouth has negligible length of its own, and any "mouth correction" (Neumann, 1950) can be disregarded.

For s = 1 , the period of the fundamental oscillation of this type will be double of that for the fundamental seiche in the longitudinal direction given by Table IV, namely from 11.1 to 10.5 minutes, according to the height of the tide. The third harmonic, s = 3 , by the same token will have a period of from 3.71 to 3.51 minutes and will be reinforced by the close congruency of the fundamental oscillation (s = 1) in the direction of the breadth, B , (equation (4)), whose periodicity can vary from 4.08 to 3.68 (double the primary values in Table IV).

For a basin of such a narrow entrance as the Duncan Dock (400 feet in relation to a length of 6000 feet) it may be assumed that the open-mouth type of oscillation cannot develop with ready facility and that higher harmonics than the third will not be of importance. The third harmonic in the direction of the breadth with a period 1.36-1.30 would be perhaps the highest frequency deriving from this stimulant.

Based on equations (4), (5) and (6), then, the critical periods for the Duncan Basin, taken collectively, could be expected on the average to center round the values forming the series: -

$$T = 10.8,\ \underline{5.4},\ \underline{3.8},\ 2.7,\ \underline{2.0},\ \underline{1.85},\ \underline{1.72},\ \underline{1.51},\ 1.34,\ 1.28,\ \underline{1.08},\ \underline{0.98\text{-}0.90},\ 0.80,\ 0.66\text{-}0.60,\ 0.54\text{-}0.45,\ 0.33\text{-}0.30,\ \text{minutes} \qquad (7)$$

Underlined are the periodicities that are likely to be of greatest importance because of the special shape and dimensions of the basin, these being, in general, the lowest modes of the several kinds, longitudinal, transverse and two-dimensional.

THE EXCITATION OF SEICHES IN THE DUNCAN BASIN.

As to whether these natural frequencies will ever be excited depends essentially on the nature of the disturbances infiltrating from outside the basin. Reference to Tables I and II shows that many of the periodicities of the seiches which have been shown to exist in the external oscillating basin between breakwater and shore are very close in value to the natural periods of Equation (7). The 11, 5.5 and 1.86 minute seiches outside the Duncan Basin are almost exactly tuned, and if the antinodal positions are favorably situated to cause a flux through the basin entrance, they will induce completely resonant oscillations inside. The external seiches of periods nearest to these critical values can also be expected to beget forced seiches of near-resonance.

The model tests reflected in Fig 5 (a) prove that the 6.0 and 4.7-4.5 minute external seiches can obtain a fair measure of response from the Duncan Basin. Their co-existence, in fact would be likely to produce a beat oscillation with an apparent periodicity of about 5.3 minutes, which would strongly reinforce the main 5.5 minute forced seiche. The longest periodicity recorded in the Duncan Dock in any given case would depend on how many of these three forced seiches were present outside. If the 6.0 minute seiche were absent, for instance, the 5.5 and 4.6 minute seiches would tend to promote a beat oscillation of about 5.0 minutes period inside the basin, whereas if the 4.6 minute one were missing, the apparent periodicity would run to about 5.8 minutes or possibly higher. On rare occasions, no doubt, one or other of these forcing seiches might operate alone and impress its own periodicity upon the basin. Conformation of this would seem to lie in the fact that actual periodicities secured at random from marigrams for the Duncan Basin show values fluctuating from about 5.0 to 6.0 minutes with the average round 5.6 minutes. Fig 6 (a) gives a frequency diagram of these periodicities as recorded over the year 1945.

Direct and convincing evidence of the combining of the forced seiches in beats is provided by the seichograms of Fig 7 for stations A, C and E at the north end of the Duncan Basin. Attention is drawn in particular in these reproductions to the very obvious beats occurring simultaneously in the oscillations at A and E berths. The oscillations at stations A and E are seen to be directly opposite in phase while the trace for the intermediate station, C, shows a complete absence of the periodicity producing the

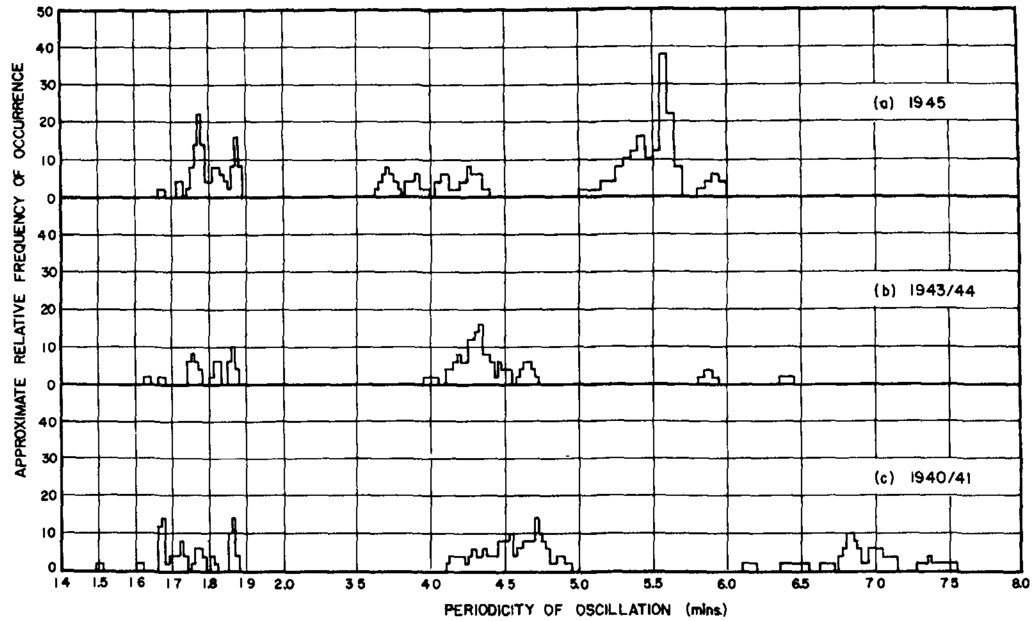


Fig. 6. Periodograms of prominent oscillations in the Duncan Basin at various stages of construction (see also Fig. 9.) as obtained from lea tide gauge at E/F berth location.

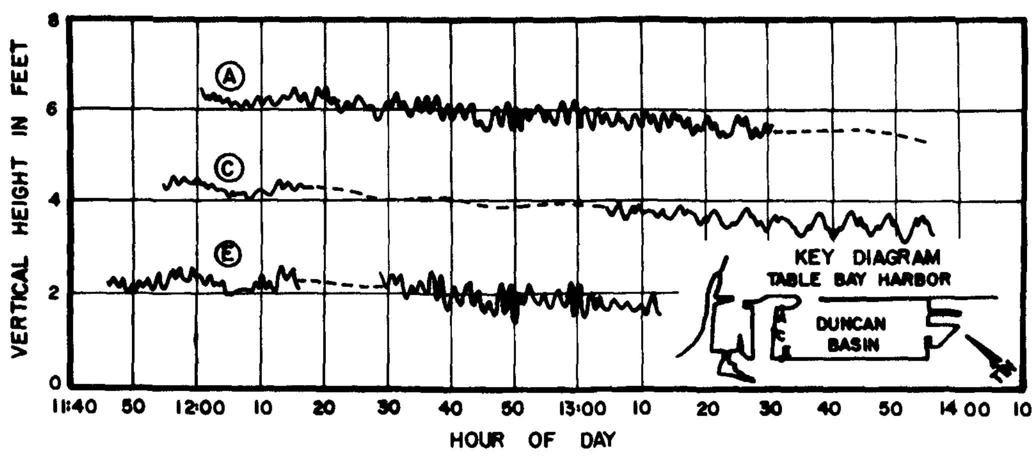


Fig. 7. Synchronised seichograms for North west end of Duncan Basin - July 22, 1946.

beats, though revealing what is clearly the second harmonic of it. All this is consistent with a fundamental transverse seiche across the northern end of the dock, whose apparent periodicity on the average is 1.765 minutes. The average length of the beats in which it occurs is 11 minutes.

Now if the beats are assumed to result from the interaction of two constituent seiches of frequencies p_1 and p_2, thereby producing a resultant of apparent frequency p which decays and grows with a beat frequency ω, it can be shown, if p_1 and p_2 are not markedly different in value, that

$$\left.\begin{array}{ll}\text{(i)} & p_1 = p + \omega \\ \text{(ii)} & p_2 = p - \omega\end{array}\right\} \qquad (8)$$

For the case in point $\omega = 1/22$ and $p = 1/1.765$, whence from (8) the component periods T_1 ($= 1/p_1$) and T_2 ($= 1/p_2$) are found to be respectively 1.63 and 1.92 minutes. These values suggest strongly that the 1.86 (±) and 1.59 (±) minute forced seiches of the breakwater-shore system (Table III) were co-existent (with or without an additional constituent of intermediate period) and impressed themselves as a resultant upon the Duncan Dock, in view of their close congruency with the free periods of transverse oscillation 1.85, 1.72, 1.51 minutes as suggested by equation (7).

Tide-gage marigrams for the Duncan Basin show that periodicities of transverse oscillations at the northern end of the dock vary mainly between 1.7 and 1.9 minutes with the relative frequencies of occurrence shown in Fig 6 (a). The greatest tendency is for the oscillations to have a period of either 1.875 or 1.775 minutes. These are the most important transverse periodicities for the two-dimensional modes of oscillation ($m = 0$, $n = 1$) and ($m = 1$, $n = 1$), respectively (Tables IV and V) and are easily brought into vibration by the outside disturbances; but it must be supposed from the evidence of the more accurate seichograms, such as Fig 7, in which beat oscillations were of common occurrence, that the 1.775 minute oscillation is more often the result of the combining of the ($m = 0$, $n = 1$) and ($m = 1$, $n = 2$) modes of free oscillation of the basin under the stimulus of the 1.86 and 1.59 minute external seiches. The latter frequency is not often found on its own and this, no doubt, is because it becomes merged with its stronger partner, whenever it builds up in amplitude, and completely loses its identity to the latter when it weakens.

Intermediate between the fundamental longitudinal and transverse seiches in equation (7) are natural periods of 3.8, 2.7 and 2.0 (±) minutes. The first is in evidence in Table I and is found also in the model periodograms (Fig 5 (a)), being obviously activated by the 4.0 or 3.2-minute external seiche of Table III. The 2.7-minute oscillation, on the other hand, is rather conspicuously absent. The reason for this must be conjectured but is not difficult to explain.

In the first place the 2.7-minute frequency is the second harmonic or binodal oscillation for the basin in the longitudinal direction, and, as such, has nodes at the quarter-points. The location of the basin entrance, however, is also situated at the eighth-to-quarter point in the long side of the dock, and therefore coincides with a nodal area of the second harmonic. No amount of stimulation from outside can then excite this mode of oscillation even though, as we have seen, a 2.8-minute seiche is prominent outside the harbor. In much the same way, a periodic force applied at a nodal point of a stretched string, cannot animate the particular mode of oscillation for which the string is nodal at that point, despite the period of the force being resonant with it.

In terms of Table IV, the 2.0-minute natural frequency of equation (7) is really the fundamental transverse oscillation for the southern end of the dock. Figs 5 (and also 8, which represents the results of later experiments) however, show that, although it is excited at that end of the dock, it is in rather subdued form. This is no doubt because the 2.1 minute forcing seiche outside the basin is somewhat out of step and is only able to impose upon this mode of oscillation by forming a beat frequency of the right order with the 1.86-minute seiche.

Below the 1.51-minute natural frequency in the periodic scale of equation (7) there are natural periods of 1.34, 1.28, 1.08 and the important group between 0.98 and 0.90 minutes. All of these are to be found in Table I and Figs 5 and 8, and as they correspond with the external forcing seiches of Table III, their generation is satisfactorily explained. All oscillations approaching 1 minute in period may be considered to be binodal transverse seiches.

There is not much corroborative material to confirm the natural periods below about 0.9 minutes, other than is contained in Table I, but this on the whole is favorable. The interpretation of most of the periodicities detected in the records for the Duncan Basin, given above, thus appears to approximate to the truth.

THE CHANGING NATURAL FREQUENCIES OF THE DUNCAN DOCK

The explications for the observed commotion in and round the harbor thus far fit the facts reasonably well. A further test of their reliability however, is posed by data pertaining to the construction period of the Duncan Dock.

Particular stages in the development of the Duncan Basin are recorded in Fig 9. By 1940-41 its configuration was something intermediate between stages (3) and (4) of Fig 9 and in plan appearance it was not altogether dissimilar from a three-petal clover-leaf pattern.

The Lea-tidegage marigrams at the E/F berth location (Fig 2) showed at this time, besides the now familiar transverse oscillations, two principal longitudinal oscillations with periods ranging from about 7.5 to 6.5 minutes and from 4.9 to 4.2 minutes, Fig 6 (c).

For the shape of the basin as it then was there is no really dependable simple formula for computing the fundamental mode of oscillation but a guess may be hazarded by applying the equation which Chrystal (1904-5)

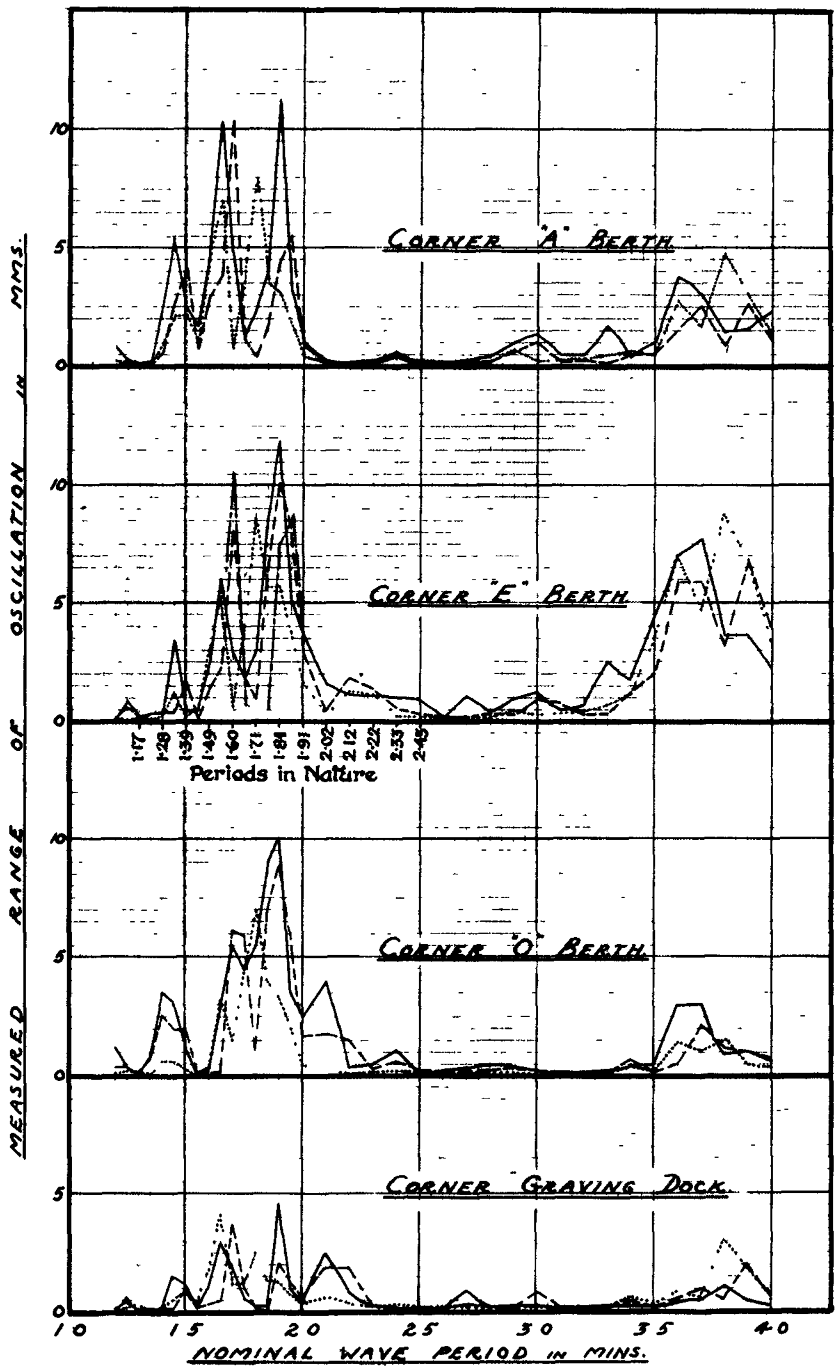


Fig. 8. Periodograms of response of model harbor to waves of different frequencies. (Variations with repeat tests largely unaccounted for - supposed due to slightly different conditions in sand bed of model)

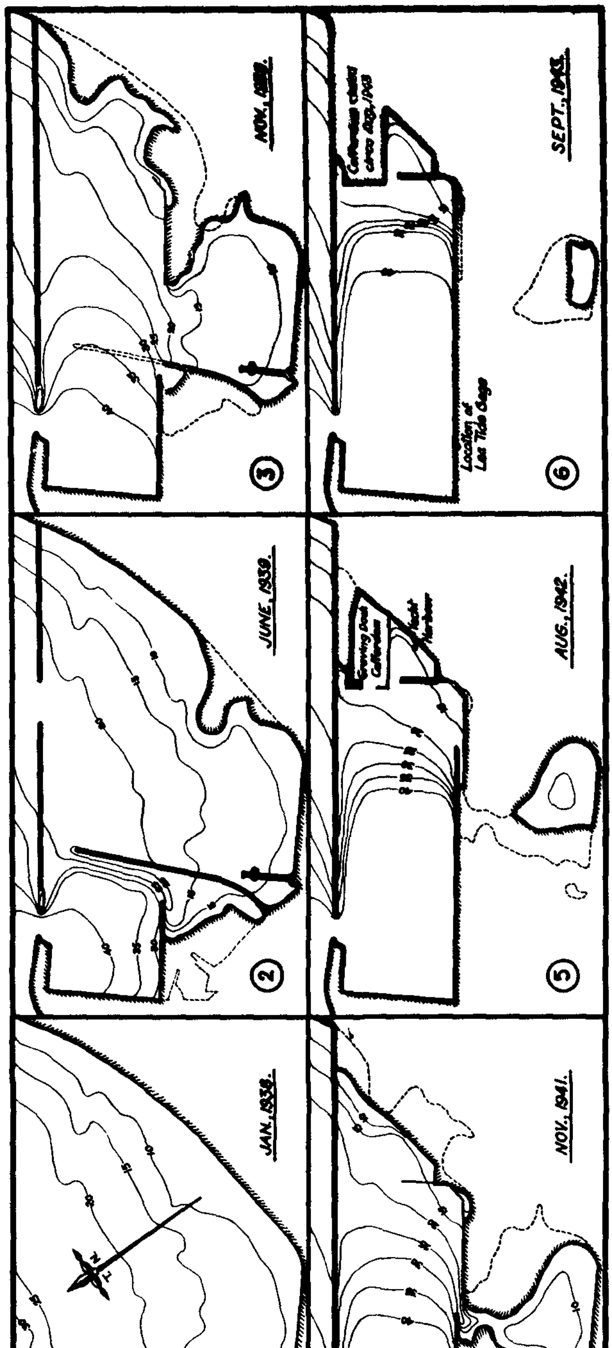


Fig. 9. Growth of the Duncan Basin, 1938-1943.
(Contours in feet approximate only)

found to be so useful in estimating the natural periods of lakes of all shapes of outline and depth, namely:

$$\frac{\pi^2 L^2}{T_m^2 g d_o} = m(m+1) \qquad (9)$$

Ignoring for the moment the lagoon portion of the basin, and taking the mean length of the remainder along its axis as $L = 7200$ feet, with a maximum depth $d_o = 40$ feet, equation (9) gives $T_1 = 7.5$ minutes for $m = 1$, and $T_2 = 4.3$ minutes for $m = 2$. These values tally very well with the observed periods of Fig 6 (c), considering the poor approximation of the assumed parabolic bed of the formula to the true sectional profile of the basin along its long axis.

Another estimate for the fundamental mode of oscillation is possible using the graphical method of wave refraction (Wilson, 1953 (iii)). According to this, the fundamental period of the basin (neglecting the lagoon) will be twice the time taken for a long wave to travel from one extremity to the other, found to be 2 x 3.6, or 7.2 minutes at low tide. At high tide, this figure would be reduced to about 6.9 minutes giving a range in fairly good agreement with Fig 6 (c). These considerations thus leave no doubt but that the longest observed periodicities in 1940-41 correspond with the lowest mode of oscillation for the basin as it then was. The development in strength of this seiche could also logically be ascribed to the 7.3 minute external forced seiche of Table III.

By the end of 1941, the marigrams revealed a definite tendency for the periodicities of the lowest mode of oscillation to incline towards lower values between about 6.8 and 6.0 minutes. Fig 9 shows that the overall length of the basin could not have changed much in this time, but as the dredging and deepening of the basin had been proceeding apace, it is reasonable to attribute the diminution of periods to this cause.

The secondary oscillations of periods from about 5.0 to 4.2 minutes persisted apparently unchanged throughout 1940 and 1941. While the approximation based on equation (9) clearly establishes these vibrations as the second harmonics of the longitudinal oscillation, it is also clear from Fig 9 that the lagoon of water to the south, remaining to be reclaimed from the sea, must have had its due influence upon this mode.

As one means of proving the model the conditions of November 1941 (Fig 9) were reproduced, the lagoon of water on the southern side being included. The critical periods in the range 4 to 8 minutes as measured at three points in the basin are shown in Fig 10.

Two broad bands of critical periods show up in these periodograms, the larger of which incorporates several peaks. These may be identified as frequencies of 7.1, 6.6 and 6.2 minutes period together with minor ones at 6.0 and 5.5 minutes, all of which, significantly, are found in the periodic sequence of external forcing seiches (Table III). It is interesting to note that the 5.5 minute external seiche, which from previous considerations is known to be strong and persistent, was barely able to influence the Duncan Basin because of the latter's non-resonant response at that time.

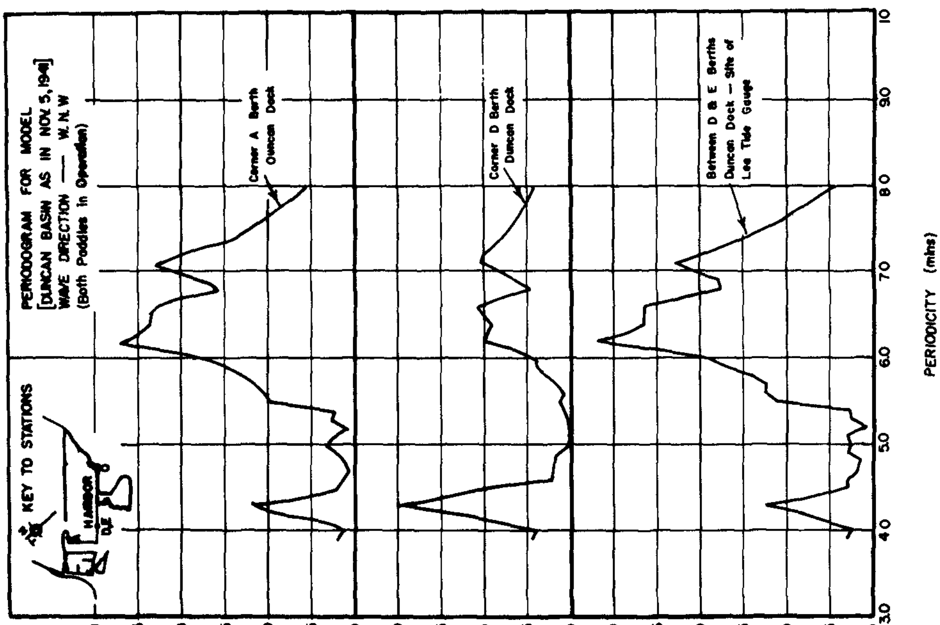


Fig. 10. Periodograms of respones of Duncan Basin to different wave frequencies, (1941 conditions).

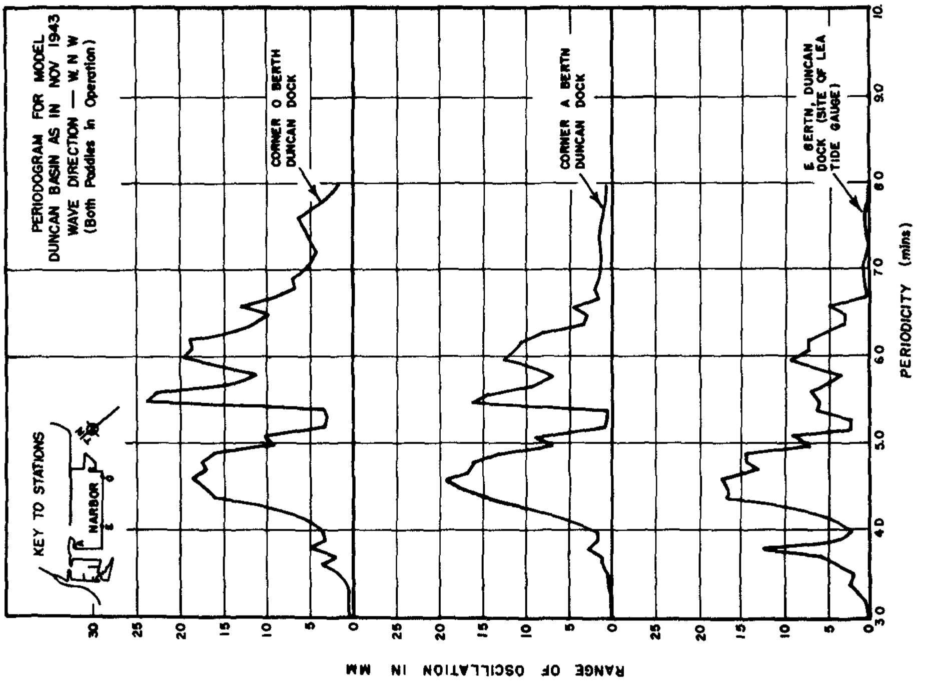


Fig. 11. Periodogram of response of Duncan Basin to different wave frequencies, (1943 conditions).

The smaller band in the periodograms of Fig 10, giving a peak at 4.3 minutes, corresponds well with the equivalent band of Fig 6 (c). In the model this mode of oscillation was observed to involve the lagoon as well as the basin in a common three-cornered 'flapping' of the whole cloverleaf-like body of water contained by the Eastern Mole. An antinode at the 'stem' or common junction of the three 'petals' of the clover-leaf formation was clearly in evidence. There were thus three nodes, one to each petal, two of which, of course, were roughly transverse to the basin and thus accorded with the binodal longitudinal oscillation of that body of water on its own. The motivating seiches in the external breakwater-shore system would presumably be the 4.0 and 4.6 minute periodicities in the sequence of Table III.

In 1942 the quay-wall construction and reclamation works met and sealed off the lagoon of water to the south of the Duncan Basin (Fig 9). The works for the construction of the graving dock at the south-eastern end of the basin meanwhile gradually pushed out and finally cut off the triangular-shaped end. The closing of this area by cofferdam occurred towards the end of July 1943 and caused the sudden disappearance of the mode of oscillation of 7.1 - 6.2 minutes period in the marigrams of the Lea tide-gage. The 4.3-minute seiche, however, persisted in the face of this shortening of the basin and detachment of the lagoon (Fig 6 (b)) and throughout most of 1944 showed itself to be the only prominent longitudinal periodicity in the marigram records.

It is necessary to record in explanation of this that by 1943, the dredging and rock breaking in the basin had extended the 40 depth limit of the bed at low tide to about two-thirds of the finished length of the basin. Of the remaining one-third length about half had been dredged to between 38 and 35 feet, while the remaining length at the southern end formed a shelf only 20 feet deep (Fig 9-(6)). In effect then the dock comprised a submerged basin of length $L = 5000$ feet and depth $d = 40$ to 45 feet, whose fundamental period according to equation (4) would be from 4.6 to 4.4 minutes according to the state of the tide.

The conditions of November 1943 (Fig 9) were examined in the model and yielded the periodograms of Fig 11 for three observing stations. These periodograms, while showing a strong band of critical periodicities near 4.6 minutes in support of a submerged basin oscillation, also show that there must have been considerable response at that time to the forcing seiches of periods 5.5, 6.1 and 6.6 minutes. However, for the particular location of the Lea tide-gage (Fig 9) the latter periodicities are very subdued (Fig 11) and to this fact, very largely, must be ascribed the absence of any indications of them in the marigrams for 1944.

There was, nevertheless, another factor affecting the issue, which only came to the author's attention long after the experiment. This concerned the deposition of the dredgings from the basin in a wide area outside the Eastern Mole. A considerable shoal formed here, which must have had a profound effect upon the seiches of the breakwater-shore oscillating system. When the extent of the shoal was discovered it was realised by the harbor authorities that it constituted a hazard to ships in the roadstead, and it was by degrees removed. The total disappearance of any

6.0 to 5.3 minute oscillations in 1944 and their reappearance in 1945 in the marigrams of the Lea tide gage was probably intimately bound up with the growth and final removal of this sand bank outside the harbor.

By the end of 1944 the Duncan Basin had been dredged to its designed depths throughout most of its area except that immediately adjoining the cofferdam of the graving dock, but in April, 1945, the removal of this cofferdam was commenced and by June the basin had assumed its completed form. This culminated in the sudden and complete cessation of the 4.3 minute frequency and its replacement on rare occasions by the less stable 3.9 - 3.7 minute oscillation identified in equation (7), and evoked full response to the 5.5 minute forcing seiche as shown in Fig 6 (a).

OSCILLATIONS OF THE VICTORIA AND ALFRED BASINS

Considerations of space preclude detailed discussion of the oscillations peculiar to the Victoria and Alfred Basins, which are complex, but some discursive remarks seem necessary.

The behavior of these basins follows the same general mechanism as the Duncan Basin, their response, of course, depending upon the shapes and sizes of the individual compartments into which these docks are divided. The periodograms contained in Fig 5 (b) are generally indicative of the critical frequencies inspired by the external forcing seiches.

Model experiments showed that at periodicities above 4.0 minutes the oscillation in the Victoria Basin consisted of a pumping action of the entire water-body. In the Alfred Basin this action started at periods above about 3 minutes. In the case of the Victoria Basin it is not difficult to see why this should be so, since this dock is roughly square and of dimensions equal to the width of the Duncan Basin. Any uninodal oscillation for the Victoria Dock must therefore be of the same order as the fundamental transverse oscillation for the Duncan Basin, which was found to be 1.85 minutes (equation (7)). Merely by doubling this figure the approximate periodicity (3.7 minutes) of the open-mouth oscillation is obtained, which is nodal at the entrance and increases in amplitude from the mouth to the head of the dock. Higher periodicities must therefore cause a more-or-less universal scend over the entire area of the basin.

SUMMARY AND CONCLUSIONS

The existence of surging of serious magnitude in the harbor basins at Cape Town can be imputed to the development of seiches in the roadstead immediately outside. These forcing seiches are engendered as features both of Table Bay and the three-sided quasi-basin contained between the harbor breakwater and the coast. The latter is so oriented and shaped as to permit of the development of an extensive series of harmonics whose presence is indicated in seichograms and confirmed by model experiments and theoretical considerations.

The nodal lines of the external seiches tend to be normal to the harbor boundary along the line of the Eastern Mole, so that at any particular point along this boundary, away from a node, the water level will tend to rise and fall in the period of the seiche, creating alternately a head of water outside and then inside the harbor basins at their entrances along this boundary. Of necessity this head of water induces a compensating periodic flux through the basin entrance which provides the pulsation for activating oscillations within the harbor.

The fact that the seiches within the harbor at Cape Town attain to serious proportions is largely due in the first place to the circumstance that the location of the breakwater fortuitously aggravates the development of certain seiches of the bay and in the second place to the circumstance that the basin dimensions and entrance sites permit of fully resonant transmission of some of these enhanced forcing seiches of the roadstead. Both the fundamental longitudinal and transverse oscillations in the rectangular Duncan Basin (respectively 5.6 and 1.85 minutes in period) are favored in this way, and to make matters worse, the dimensions of the long and short sides of this basin are so related (ratio 3:1) as to accentuate the transverse frequencies.

In the Victoria Basin the mechanism of the action follows the same pattern as for the Duncan Dock. Here there are more compartments to sustain particular periodicities of oscillation, but owing to the sheltering effect of the breakwater and wave diffraction the higher-frequency disturbances in the roadstead have less chance of penetrating this basin. The breakwater-bight, however, constitutes an antinode or loop-end for most of the seiches of the external roadstead and the Victoria and Alfred Basins are much imposed upon by the lower frequencies. Seiches of more than 3 minutes period in the Alfred Basin and more than 4 minutes period in the Victoria Basin cause a pumping action of water over the entire dock areas. The equivalent effect is produced in the Duncan Basin by all seiches exceeding 11 minutes in period, very notably the strong seiches of the bay of 17-23, 26-33 minutes periodicity.

The behavior of the Duncan Basin during its construction period affords an excellent example of the influences of shape and depth on the oscillating characteristics of an enclosed body of water. Throughout the time of its construction the external seiches, largely unchanged themselves, have played upon it, often with completely different results. The conslusion is clear that a basin will only respond to those impressed frequencies which accord most nearly with its own. Partial resonance seems to be possible when the forcing seiche is sufficiently close in period to that of the natural frequency, there being in general no precipitate transition from complete unresponsiveness to full resonance.

ACKNOWLEDGEMENT

The writer wishes to record his appreciation to the South African Railways and Harbours Administration for the use of the data analysed in this paper.

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Part 2
SHORELINE SEDIMENT PROBLEMS

CHAPTER 5

GEOLOGIC HISTORY OF GREAT LAKES BEACHES

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ABSTRACT

The locations of the Great Lakes and many details of the lake bottom topography bear a distinct relationship to the bed rock structure. Normal stream erosion in pre-glacial time probably etched out the major topographic relief of the region, forming the major basins and even some of the present bays, in the weak rock belts.

Glacial ice, advancing over the region in several stages, followed the lowlands but reshaped them and probably deepened most of them.

The known lake history, beginning with the last retreat of the ice from the southern rims of the Michigan and Erie basins, involves a number of stages at different levels in each of the basins. These lakes discharged at various places at different times, because of readvancement or retreat of the glacial ice front and because of tilting of the earth's surface. The writer's summary of this history is illustrated by a series of sixteen maps.

The practical importance of two extremely low lake stages is pointed out. These have affected foundation conditions in the vicinity of many river mouths. The newly established recency of some of the higher lake stages (Nipissing and Algoma), and the revision of the elevations attained by them, affect estimates of the intensity of beach action and they affect conclusions regarding the time of last discharge of water through the Chicago outlet.

INTRODUCTION

Many details of the geologic history of the Great Lakes are pertinent to the study of present day shore processes and to foundation problems along the lake shores. The Great Lakes have stood at their present elevations for only a few thousand years. Recent studies suggest that Lakes Michigan and Huron reached their present elevations less than two thousand years ago. The shortness of this period should be taken into account in an evaluation of the rate of shore erosion.

Water level in all of the five Great Lakes has been both higher and lower by considerable amounts in the past. The old higher-level stages are recorded by beach deposits, wave-cut cliffs and other features which now are abandoned. The lower stages are recorded by features such as a sand and shell zone in the deep-water clays and by deep valleys in the lower courses of rivers, now partially or wholly filled by fine-grained sediment. This fill generally constitutes poor foundation material for any major structure placed on it.

BED ROCK STRUCTURE OF THE REGION

The locations and shapes of the Great Lakes basins are intimately related to the bed rock structure. This is evident from a cursory examination of the geologic map of the region (Stose, 1946). The very old (pre-Cambrian), generally hard rocks of the Canadian shield lie north of all of the lakes and extend southward through the Superior basin into Wisconsin. These rocks include ancient sedimentary deposits, lava flows and intrusive rocks, all of which have been more or less highly metamorphosed, folded, and welded into a generally resistant mass. The shield has been worn down to a surface of low relief, and then dissected.

In the western half of Lake Superior the shores and the bottom topography are aligned parallel to the trends of bed rock structure in the pre-Cambrian shield rocks.

Outside of the Lake Superior basin, and possibly beginning in the southeastern part of that basin, the surface of the Canadian shield dips under a cover of Paleozoic sedimentary rocks.

The four lower lakes, Michigan, Huron, Erie and Ontario, lie in the Paleozoic rock province. These lake basins are elongated parallel to the strike of the rock and belts of easily eroded shale formations extend through all of the basins.

The Great Lakes basins almost certainly acquired their major topographic forms in pre-glacial time, by normal subaerial erosion acting on a variety of bed rock formations of differing resistance to erosion.

GLACIATION

It is reasonable to assume that the flow patterns of the early Pleistocene glaciers were influenced to a considerable extent by the pre-glacial topography. The ice, however, re-shaped the land somewhat by scour, deepening and smoothing the basins, as well as by deposition. Toward the end of glacial time the ice lobes conformed closely to the shapes of the present lake basins.

GEOLOGIC HISTORY OF GREAT LAKES BEACHES

The glacial stages of the Pleistocene epoch, and some of their substages, are as follows (youngest stage at top);

- Wisconsin glacial stage
    - Mankato substage
    - Cary substage
        - Port Huron
        - Lake Border
        - Defiance-Tinley
        - Valparaiso-Fort Wayne
    - Tazewell substage (Tazewell II?)
    - Iowan substage (Tazewell I?)
    - Farmdale substage
- Illinoian glacial stage
- Kansan glacial stage
- Nebraskan glacial stage

It is generally believed that the Nebraskan glaciation began about one million years ago. The history of lakes in the Great Lakes basins, as presented here, begins within the early Cary substage of the Wisconsin stage, at a date estimated by the writer as about 25,000 years ago (Hough, 1953b). The Mankato substage has been dated as occurring about 11,000 years ago, on the basis of radiocarbon studies (Flint and Deevey, 1951).

HISTORY OF THE LAKES IN THE GREAT LAKES BASINS

During the times between the various major ice advances of the Pleistocene epoch there may have been lakes in the Great Lakes basins, but if such existed there is no clear record of them. Our detailed record of lakes begins toward the end of the glacial period, when the last ice to completely fill the basins melted back, leaving room between the ice front and the divides to the south for impounding of water. A complicated series of lake stages followed, in which lake levels fell and rose, outlets shifted from one place to another, and sometimes back to previously used discharge channels, in response to three causes. Retreat of the glacial ice front exposed new lower outlets to the north and east, and resulted in lowering of lake levels: readvance of the ice part way into the lake basins closed the lower outlets, raised lake levels, and shifted discharge back to the south. Erosion of the beds of the outlet channels lowered lake levels. As the glacial ice front retreated to the north, uptilting of the land caused further changes in lake levels and in points of discharge.

The evidence of former higher lake levels consists mainly of abandoned beach deposits and wave-cut benches lying above the present lake shores, and of former outlet channels connected to these beaches.

The old beaches in the southern half of the Lake Michigan basin, and in the southernmost part of the Lake Huron basin, are horizontal, indicating no tilting in these areas since the beaches were formed. Some of the older, higher level beaches rise to the north, indicating that there has been tilting of the region since those beaches were formed. The amount of uplift recorded at the Straits of Mackinac is 200 feet, and at the north side of Lake Superior and at North Bay, Ontario, northeast of Georgian Bay, the amount of uplift is 600 feet. Some evidence has been found on the bottom of Lake Michigan for a low stage of that lake.

The history of the lake stages presented in the following pages is the writer's interpretation, which is based on the work of Leverett and Taylor (1915), Stanley (1936 and 1937), Deane (1950), Bretz (1951a, 1951b), and on investigations of his own (Hough, 1953). It is emphasized that the history presented here is, in some details, almost certain to be revised as new information is obtained. The writer's proofs or arguments for many details, which are stated rather dogmatically in the present paper, are given in his report on an Office of Naval Research project (Hough, 1953).

LAKE MAUMEE AND EARLY LAKE CHICAGO

To begin the history of the lake stages we must visualize the land in the northern part of the Great Lakes region depressed, and visualize the basins filled with glacial ice. When the ice front retreated from the western end of the Lake Erie basin, a lake (Highest Lake Maumee) formed there and it discharged southwestward to the Wabash River (fig. 1). At the same time the ice front in the Lake Michigan basin probably retreated an appreciable distance northward, allowing a lake (Early Lake Chicago) to form there (fig. 1). There is no direct evidence of this lake, because any beaches which might have been developed would have been destroyed by the subsequent advance of ice in the area.

DEFIANCE AND TINLEY GLACIAL ADVANCES

A readvance of glacial ice in the Erie basin, to the position of the Defiance moraine, then occurred (fig. 2). This reduced the area of Lake Maumee but that lake apparently remained at the same elevation, about 800 feet above sea level. This advance is correlated with the advance of ice in the Michigan basin to the Tinley moraine. The Tinley ice completely filled the basin (fig. 2).

LOWEST LAKE MAUMEE AND THE GLENWOOD STAGE OF LAKE CHICAGO

The next retreat of the ice front (fig. 3) allowed Highest Lake Maumee to expand in the Erie basin, and, possibly at the same time, allowed the Glenwood stage of Lake Chicago to form in the Michigan basin.

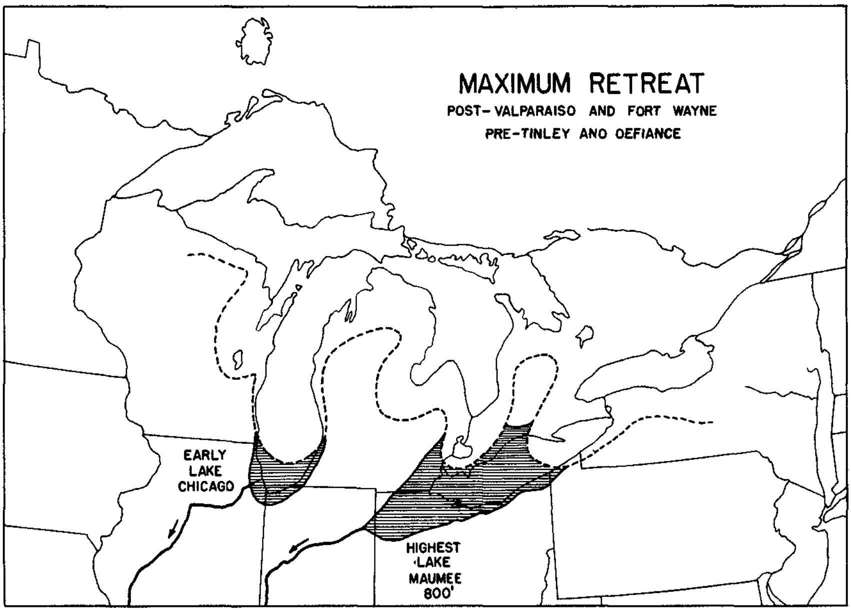

FIG. 1 - **Lake stage map no. 1: Early Lake Chicago and Highest Lake Maumee.**

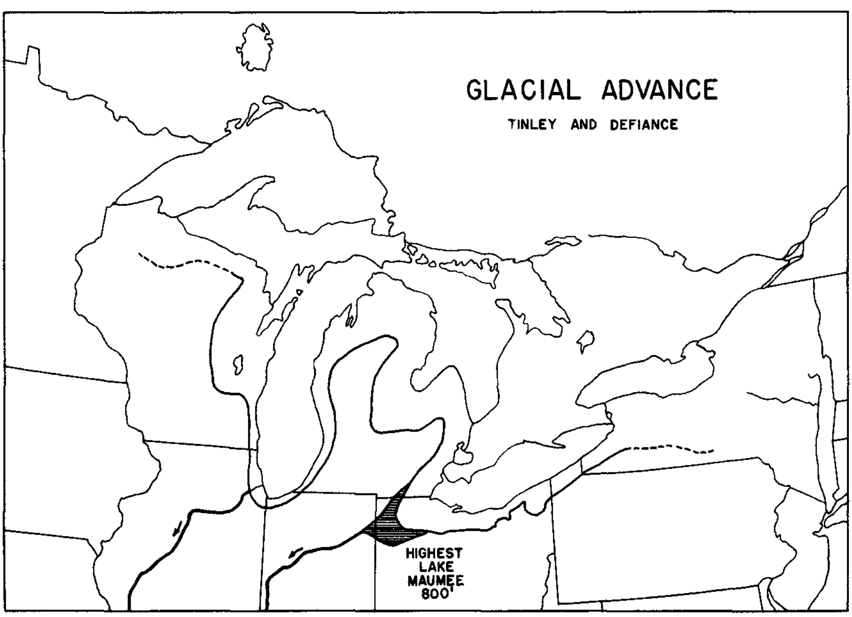

FIG. 2 - **Lake stage map no. 2: Highest Lake Maumee.**

The Glenwood stage stood at an elevation of 640 feet, or 60 feet above the present Lake Michigan level, and it discharged down the Illinois River. At the time of maximum retreat in this phase of the history, Lake Maumee found a dischargeway across the "thumb" of Michigan and thence down the Grand River valley to the Michigan basin. This discharge through a lower outlet lowered the surface of Lake Maumee to an elevation of 760 feet, thus producing the second, and lowest level, Maumee stage.

LAKE BORDER GLACIAL ADVANCE AND MIDDLE LAKE MAUMEE

The advance of the ice front to the Lake Border moraines of the Erie basin (fig. 4) produced the third Maumee stage, at elevation 790 feet. This was accomplished by the ice front riding up the "thumb" of Michigan and displacing the discharge channel to a higher elevation. An advance of the ice front to the Lake Border moraines of the Michigan basin, possibly occurring at the same time as that in the Erie basin just described, reduced Lake Chicago to a narrow crescentic body of water at the southwestern rim of the Michigan basin.

LAKE ARKONA

The next retreat of the ice was more extensive, and the lake in the Erie basin expanded northward into the Huron basin and joined with a growing lake in the Saginaw Bay area to form Lake Arkona (fig. 5). This lake initially stood at an elevation of 710 feet, and it discharged down the Grand River valley to the Michigan basin. During the life of Lake Arkona the lake level was lowered from 710 feet to 695 feet, by downcutting of the outlet. In the Michigan basin, meanwhile, Lake Chicago had expanded northward with the retreat of the ice front to cover approximately the southern half of the basin. Lake Chicago was still at the Glenwood stage (elevation 640 feet), and still discharged down the Illinois River.

PORT HURON GLACIAL ADVANCE AND LAKE WHITTLESEY

An advance of the ice to the Port Huron moraine (fig. 6) filled the southern Huron basin and raised the lake level to the south to the Lake Whittlesey stage (elevation 738 feet). Lake Whittlesey drained across the "thumb" of Michigan, along the ice front, to Lake Saginaw. Lake Saginaw, a remnant of the Arkona 695-foot stage, drained down the Grand River valley to the Michigan basin. The Glenwood stage of Lake Chicago persisted in the Michigan basin, though the extent of the lake was reduced somewhat by the advance of ice to point a little more than half-way down the basin.

EARLY LAKE WARREN AND THE FIRST CALUMET STAGE OF LAKE CHICAGO

During the early stage of retreat of the Port Huron ice, (fig. 7) Lake Whittlesey in the Erie basin was drained down to the

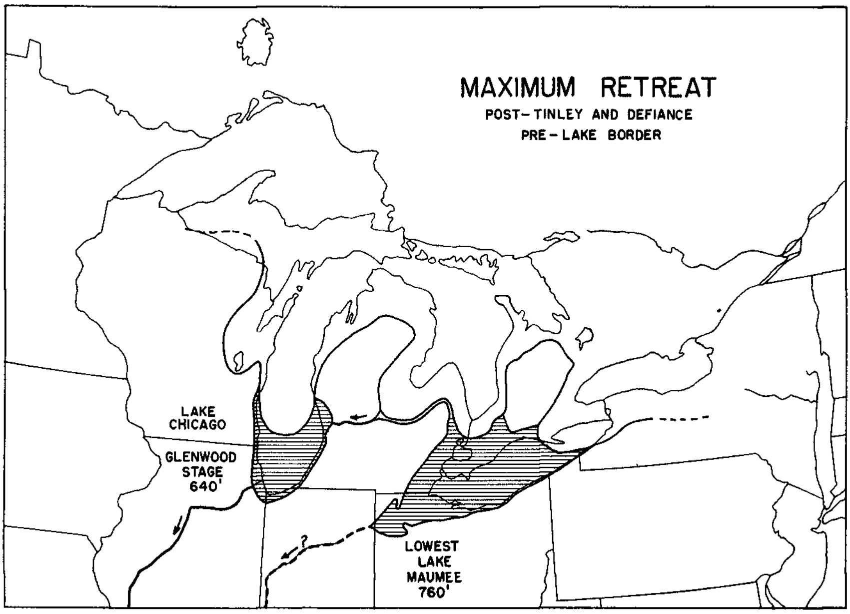

FIG. 3 - **Lake stage map no. 3: Lake Chicago Glenwood stage and Lowest Lake Maumee.**

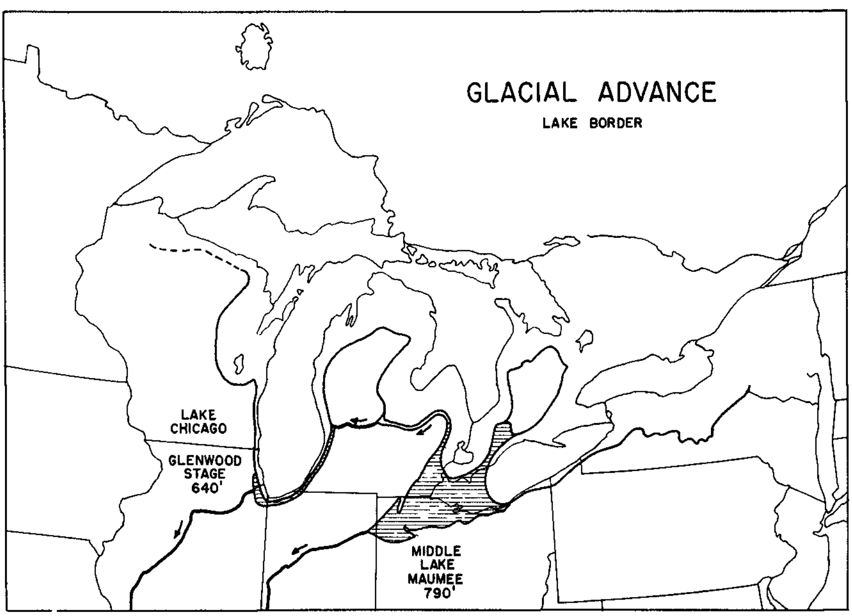

FIG. 4 - **Lake stage map no. 4: Lake Chicago Glenwood stage and Middle Lake Maumee.**

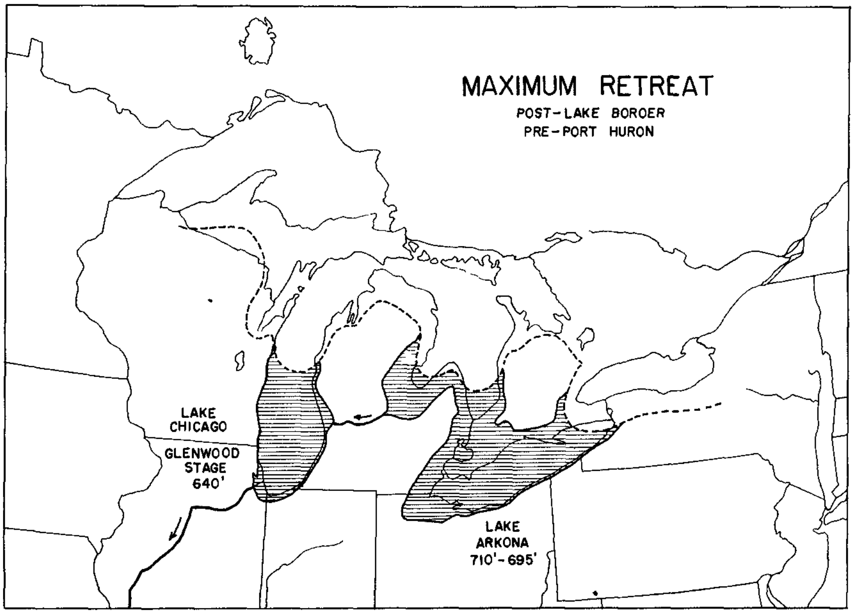

FIG. 5 - **Lake stage map no. 5: Lake Chicago Glenwood stage and Lake Arkona.**

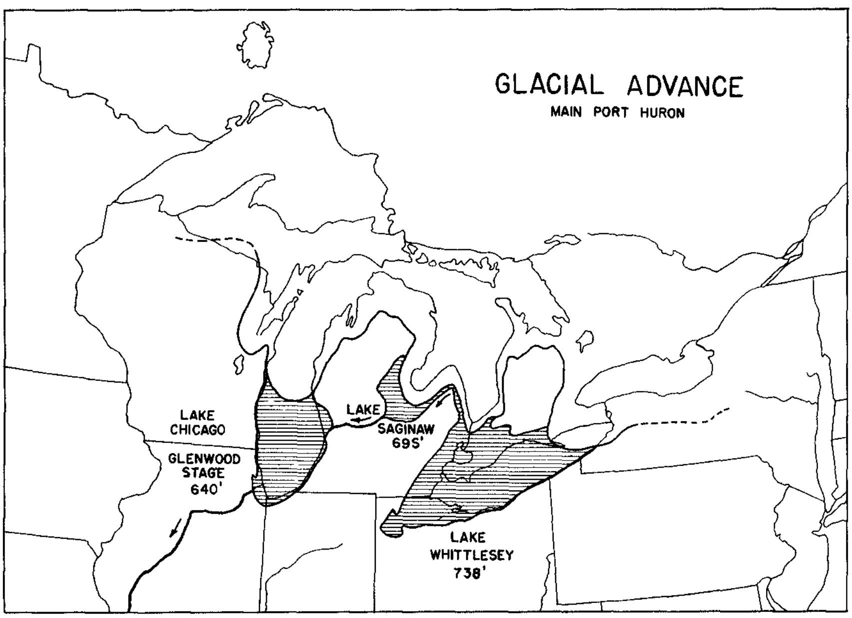

FIG. 6 - **Lake stage map no. 6: Lake Chicago Glenwood stage and Lake Whittlesey.**

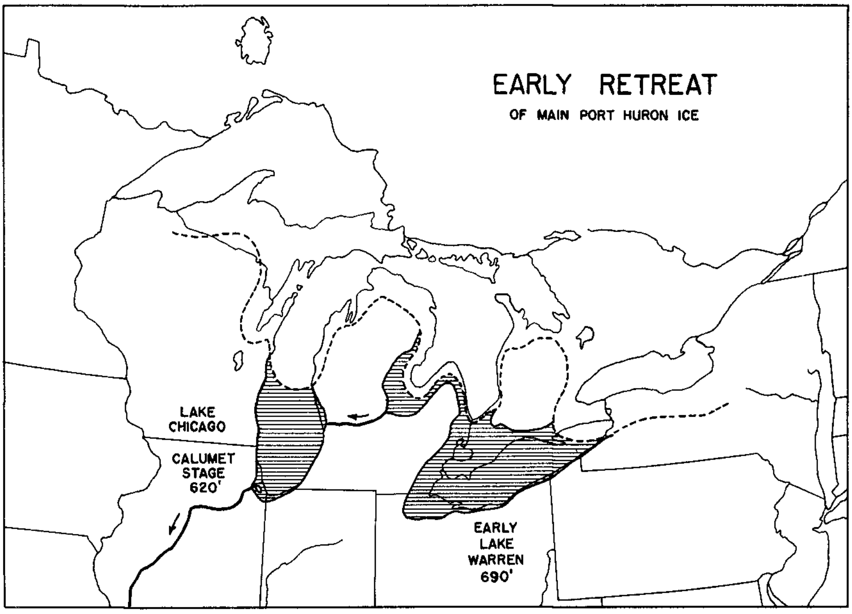

FIG. 7 - **Lake stage map no. 7: Lake Chicago Calumet stage no. 1 and Early Lake Warren.**

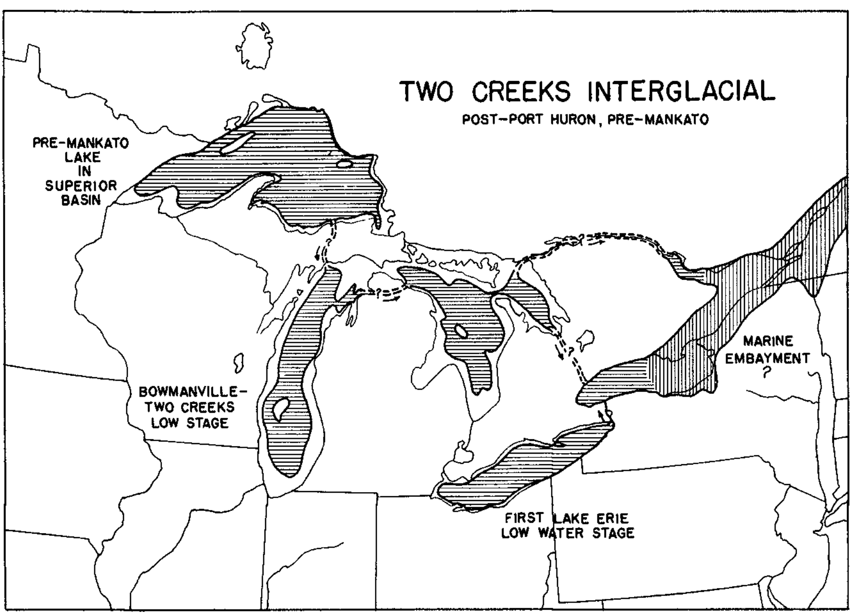

FIG. 8 - **Lake stage map no. 8: Two Creeks interval low stage lakes.**

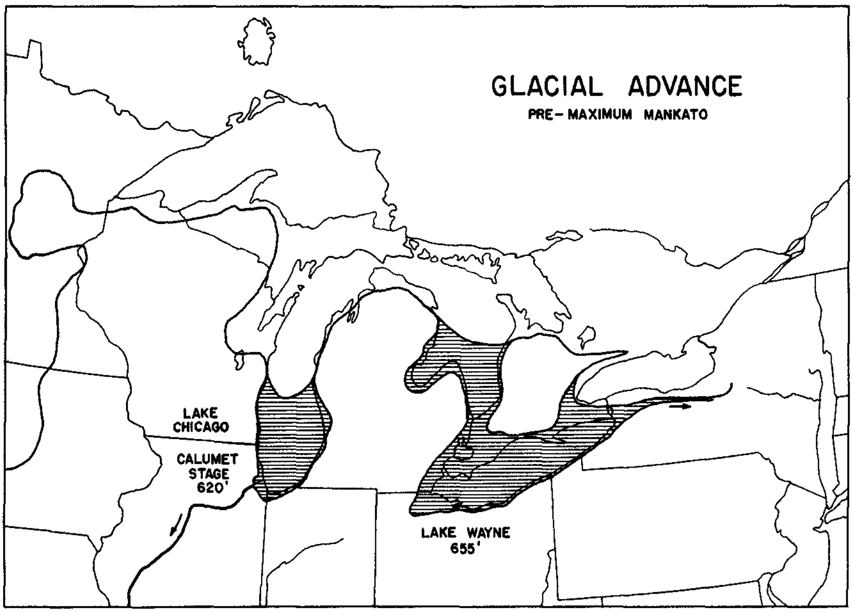

FIG. 9 - **Lake stage map no. 9: Lake Chicago Calumet stage no. 2 and Lake Wayne.**

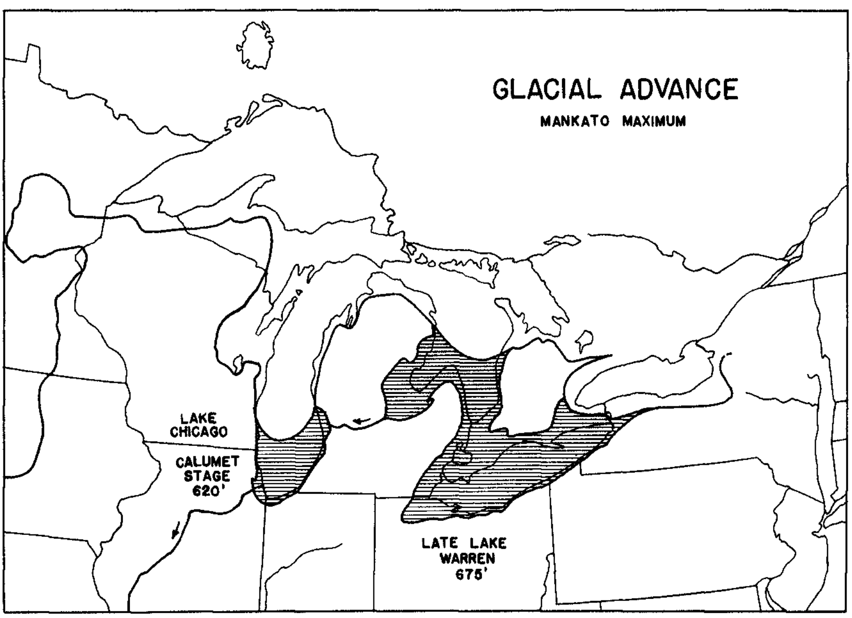

FIG. 10 - **Lake stage map no. 10: Lake Chicago Calumet stage no. 2 and Late Lake Warren.**

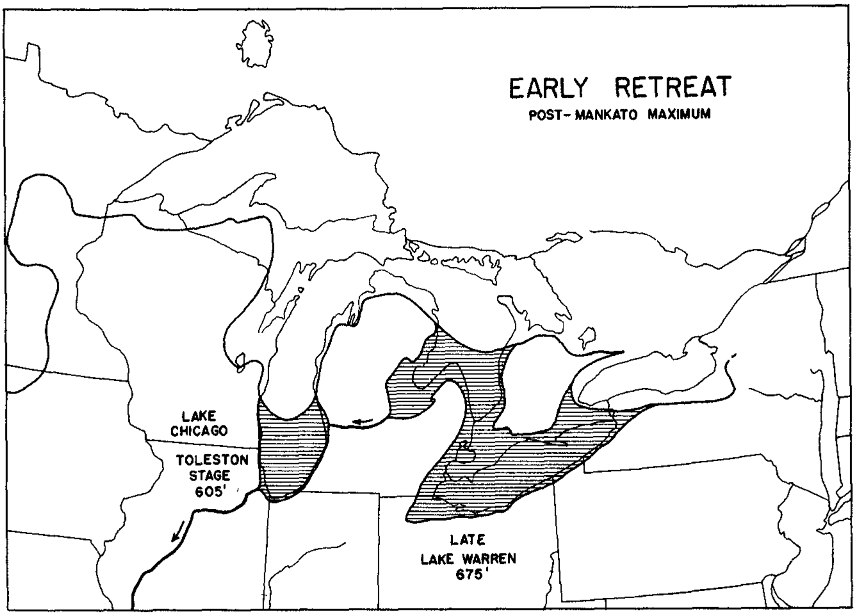

FIG. 11 - **Lake stage map no. 11: Lake Chicago Toleston stage and Late Lake Warren.**

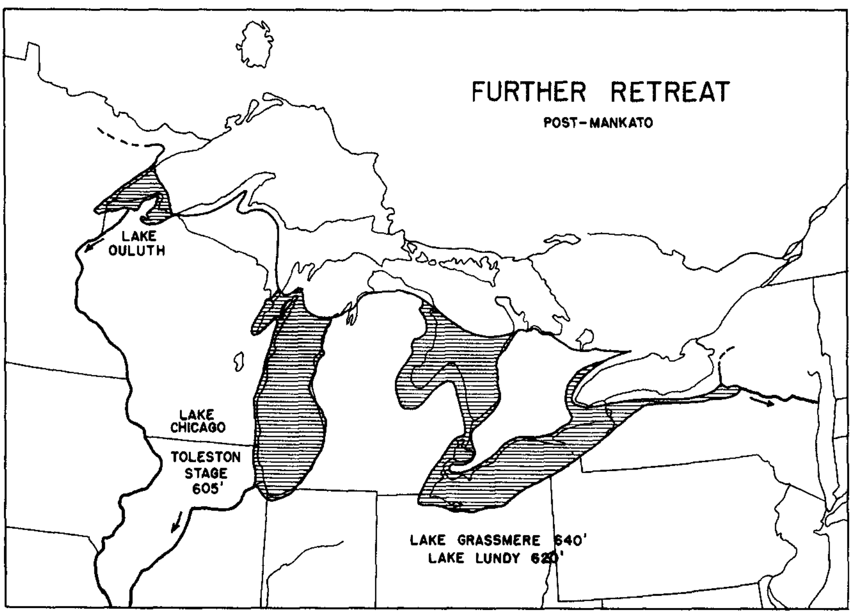

FIG. 12 - **Lake stage map no. 12: Lake Chicago Toleston stage, Lakes Grassmere and Lundy, and Lake Duluth.**

level of the Saginaw Bay outlet, discharging a large volume of water in a relatively short period of time. The outlet apparently was lowered about 5 feet by erosion resulting from the increased discharge, and the next lake level, the Early Warren stage, was developed at an elevation of 690 feet. In the Michigan basin the ice front retreated northward, allowing Lake Chicago to expand. It is likely that the increased discharge from the east, resulting from the drainage of Lake Whittlesey as well as from the melting ice, so swelled the discharge of Lake Chicago that it caused a downcutting of its outlet. Lake Chicago was lowered from the Glenwood stage (640 feet) to the Calumet stage (620 feet) at about this time.

TWO CREEKS INTERGLACIAL SUBSTAGE

The post-Port Huron retreat of the ice continued until the Great Lakes basins were nearly, if not entirely, free of ice (fig. 8). This is indicated by forest remains at Two Creeks, Wisconsin (15 miles north of Manitowoc) in which logs, peat, and tree stumps in growth position occur down as low as the present lake surface, about 580 feet A.T. (Wilson, 1932, 1936). The lake must have been drained at least to that level. An outlet from the Lake Michigan basin low enough to permit this drainage can have existed only north of the Port Huron morainic system, somewhere along the northeastern edge of the basin. The most likely location of this outlet is believed to be the lowland extending from Little Traverse Bay eastward to the Huron basin. This lowland is floored by unconsolidated material, including Mankato drift which is younger than the low-water stage. The area in which the lowland lies has been warped upward since late Mankato time. If the area were depressed as much in late Port Huron time as it was in late Mankato time, it would have provided an outlet with a floor at least 100 feet below present lake level.

In order for the water in the Michigan basin to have drained down to 580 feet A.T. (it probably went lower), the water in the Huron basin to the east must have been at least that low, and a drainage course must have been open to the sea. Because there is no outlet as low as 580 feet A.T. anywhere along the western and southern borders of the Great Lakes region, the discharge must have gone down the St. Lawrence river valley. This implies a major retreat of the ice in the east.

The Lake Superior basin also apparently was essentially free of ice at this time. This is indicated by the fact that Mankato glacial till, which is younger than the Two Creeks interval deposits, possesses a strong red color and has a high percentage of clay. It is generally believed that this till is red because the ice which deposited it overrode extensive deposits of Two Creeks interval red lake clay and incorporated that material into the till. The red till occurs around the Lake Superior basin as well as in the northern parts of the Michigan and Huron basins.

During the Two Creeks interval water in the Lake Erie basin drained northeastward, and this discharge must have cut the gorge which lies to the west of the present Niagara Gorge and which is now filled with glacial till. The level of water in the Erie basin at this time must have been at least as low as the present lake surface.

THE MANKATO GLACIAL SUBSTAGE AND LAKE WAYNE, LATE LAKE WARREN, AND THE SECOND CALUMET STAGE

When advancing Mankato ice closed the low northeastern outlet of the Lake Michigan basin, the water rose until it spilled through the old outlet at Chicago. This outlet had been abandoned at the Calumet level, and the new lake undoubtedly rose to the same level. There was, therefore, a second Calumet stage, in Mankato time (fig. 9).

The advancing Mankato ice in the eastern part of the Great Lakes region filled the Ontario basin, and for a period of time the water in the Erie and southern Huron basins stood at the Lake Wayne level, 655 feet, while discharging eastward across New York state (fig. 9).

At the time of maximum advance of the Mankato ice the eastern outlets were overridden and closed, and the water in the Erie and southern Huron basins rose until it reached the Late Lake Warren level, 675 feet above sea level. This lake spilled westward from Saginaw Bay down the old Grand River outlet, to Lake Chicago (fig. 10).

Lake Chicago remained at the Calumet level, 620 feet, during the early retreat of the Mankato ice. This is indicated by the presence of a Calumet level beach on the southern border of the red Mankato till. The absence of a Calumet beach farther north on the red till indicates that the surface of Lake Chicago was lowered during the early stages of retreat of the Mankato ice (fig. 11). A plausible explanation of the lowering is that the volume of discharge through the Chicago outlet was increased greatly by rapid melting of glacial ice throughout the Michigan, Huron and Ontario basins and that this increased discharge cut the outlet to a lower level.

THE TOLESTON STAGE OF LAKE CHICAGO

The lowering of Lake Chicago after the early stages of retreat of the Mankato ice brought the lake down to the Toleston level, 605 feet above sea level (fig. 11). The Toleston stage persisted in the Lake Michigan basin (fig. 12) until the glacial ice front retreated northward far enough to permit a connection between the lakes of the

Michigan and Huron basins. The Toleston stage discharged through the Chicago outlet, which was now at its lowest possible elevation, on a bedrock sill.

LAKES GRASSMERE AND LUNDY

At some time after the Toleston stage of Lake Chicago was formed, the glacial ice front in the Ontario basin retreated to a point where Late Lake Warren (of the Huron and Erie basins) could discharge through a lower outlet to the east. When this occurred, the waters of the Huron and Erie basins were lowered to the Lake Grassmere stage, 640 feet above sea level (fig. 12). Further retreat of the ice front in the east exposed a still lower outlet, and the Huron-Erie waters were lowered to the Lake Lundy level, 620 feet above sea level (fig. 12). Both the Grassmere and Lundy stages discharged eastward across New York state.

EARLY LAKE DULUTH

The history of directly-known lake stages in the Great Lakes region has, up to this point, been limited to the southern part of the region and has involved only the Michigan, Huron and Erie basins. It is inferred that a lake existed in the Superior basin during the Two Creeks interval, but the ice of the Mankato substage completely filled the Superior basin and obliterated any shore features which may have been formed there previously.

When the ice front retreated after the Mankato maximum, probably sometime after the Toleston and Grassmere stages came into existence in the lower lakes, Lake Duluth was formed in the western end of the Superior basin (fig. 12). This lake drained down the St. Croix River and thence to the Mississippi River. The earliest known stage of Lake Duluth stood at an elevation of 1100 feet above sea level. Later stages, which were more extensive because of the northeastward retreat of the ice front, occurred at elevations of 1076, 1044 and 1022 feet. The reduction in elevation recorded by these successively lower stages presumably was brought about by downcutting of the outlet to the St. Croix River.

Lake Duluth here is considered to have ceased to exist when the ice barrier on the east retreated to a point where the lake could drain eastward. Before this occurred, however, another part of the lake history ensued in the lower basins.

LAKE ALGONQUIN

The Algonquin part of the Great Lakes history is complicated, and it is a chapter which has been revised more drastically than any other in the light of recent investigations. Some of the major revisions were made by Stanley (1936, 1937) and essentially corroborated by Deane (1950), while others have been make by the writer (Hough, 1953).

In the Lake Michigan basin, the Toleston stage ended and the Algonquin stage began when the waters of the Michigan and Huron basins were joined by means of retreat of the ice front from the Little Traverse Bay-Lake Huron lowland. No change of water surface elevation occurred in the Michigan basin in this transition, because both stages drained through the stabilized Chicago outlet. Lake Algonquin, therefore, came into existance at the 605-foot level.

In the Huron basin, Lake Algonquin followed Lake Lundy (the 620-foot stage in the Huron-Erie basin). Leverett and Taylor (1915) have stated that Lake Lundy was drained by the opening of a lower outlet somewhere to the east of Lake Erie, and that the water surface in the Erie basin thus was lowered below the divide between the Huron and Erie basins, while glacial ice still blocked the northern part of the Huron basin. The Huron water then drained southward, and the lake surface in the Huron basin was stabilized at 605 feet because erosion of the channel bed (composed of glacial till) ceased. It appears equally possible to the writer that ice retreated from the channel connecting the Huron and Michigan basins before it vacated a lower outlet to the east, and that the Lundy stage thus was terminated by drainage to the north and west rather than to east. Under this interpretation, the initial Algonquin level in both the Michigan and Huron basins was determined by the rock-floored outlet at Chicago. Under the Leverett and Taylor interpretation, the attainment of the 605-foot Algonquin level by two separate bodies of water in the Michigan and Huron basins was a mere coincidence.

As the ice front receded, Lake Algonquin expanded until it extended throughout the Michigan basin and the greater part of the Huron basin (fig. 13), while maintaining its 605-foot elevation. During this stage one of the strongest beaches in the Great Lakes region was formed. The northern shore of the lake was across the glacial ice front in the northeastern part of the Huron basin, but it was on the edge of the crystalline rock upland in the vicinity of Sault St. Marie, Ontario. From there southwestward to Green Bay the shore was along the ice front which stood on the peninsula separating the Michigan and Superior basins. A glacial outwash delta was deposited in the lake along this shore, and the surface of the delta (now elevated) records the level of the Algonquin stage.

The beach of Lake Algonquin was well developed, largely in unconsolidated material, throughout the remainder of the Lake Michigan basin, down the western shore of the Huron basin, and along the southeastern side of the Huron basin to the vicinity of Kirkfield in Georgian Bay. The Algonquin stage lake apparently discharged through two outlets simultaneously; one was the Chicago outlet, floored with bed rock, and the other was the outlet at Port Huron (at the south end of the Huron basin), which was floored with glacial till.

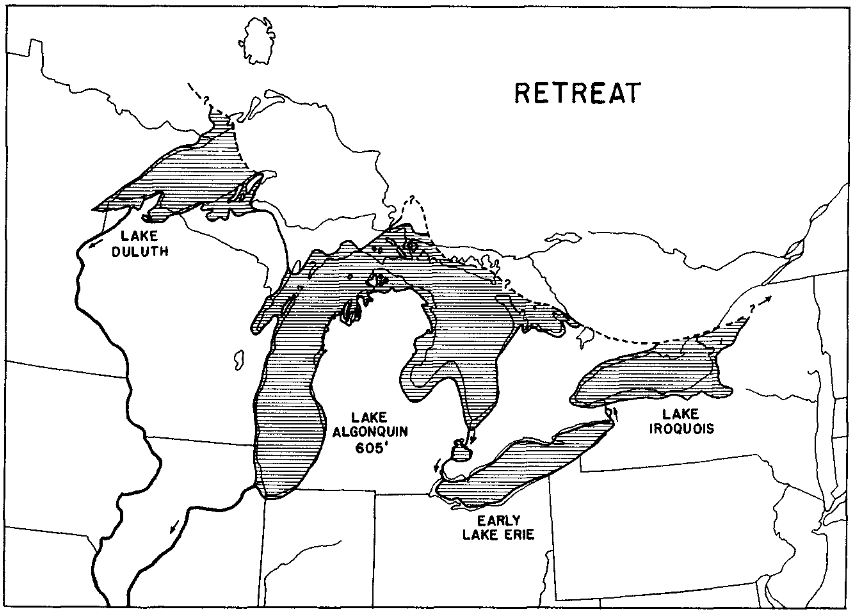

FIG. 13 - **Lake stage map no. 13: Lake Algonquin, Lake Duluth, Early Lake Erie, and Lake Iroquois.**

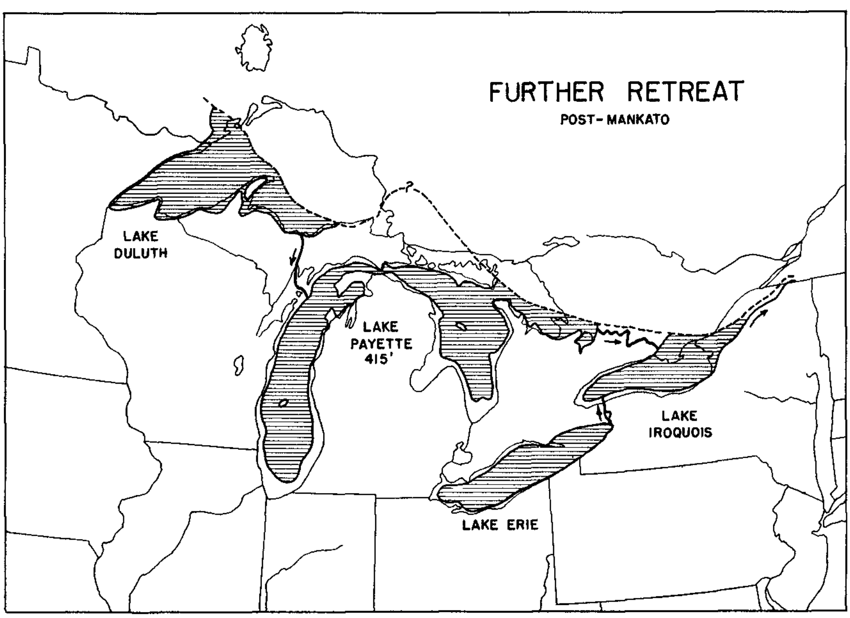

FIG. 14 - **Lake stage map no. 14: Lake Payette, Lake Duluth, Lake Erie and Lake Iroquois.**

During the period of existence of the 605-foot Algonquin stage in the Michigan and Huron basins, other lakes existed in the other Great Lakes basins. Lake Duluth was expanding in the Superior basin, but was still discharging southwestward down the St. Croix River to the Mississippi. Lake Iroquois was in the Ontario basin, discharging eastward, and an early Lake Erie in the Erie basin discharged into Lake Iroquois.

POST-ALGONQUIN LOW STAGES

The Algonquin stage, as defined here, was terminated when retreat of the ice front in an area southeast of Georgian Bay exposed a lower outlet. Drainage of water through this new outlet lowered the lake surface approximately 90 feet, this initiating the first of several low-level stages which discharged through successively lower outlets made available by further retreat of the ice. The lowest of these stages which is represented by recognizable shore features is the Payette stage (fig. 14) which existed at an elevation of approximately 415 feet above sea level.

At some time after the Algonquin stage but before the Payette, the ice barrier between the Superior and Michigan basins retreated, allowing discharge of Lake Duluth water into the Michigan basin. This may be considered as the termination of the Lake Duluth phase of the Superior basin history.

THE CHIPPEWA-STANLEY LOW-WATER STAGE

An extreme low-water stage of the lakes in the Huron and Michigan basins, lower than the Payette stage, was inferred by Stanley (1936, 1938). The writer (Hough, 1952, 1953) found definite evidence for this low-water stage in core samples taken in Lake Michigan. The low stage in Lake Michigan stood at 230 feet above sea level; this has been named Lake Chippewa (fig. 15). The low stage in Lake Huron, which probably stood at least 50 lower, has been named Lake Stanley. The outlet of the Chippewa-Stanley low water stage was at North Bay, Ontario, and down the Mattawa and Ottawa valleys to the St. Lawrence valley. This outlet was made available by retreat of the ice front, and the retreat beyond this outlet marks the beginning of "post-glacial" time in the Great Lakes region. During the Chippewa-Stanley low stage the water surface in the Lake Superior basin probably was extremely low also.

TERMINATION OF THE LOW STAGE BY UPWARP

The extreme low-water stage was brought to a close by upwarp of the outlet area. A part of the upwarp occurred while the ice front was receding through the Georgian Bay area, but the greater part of the upwarp occurred after the North Bay outlet came into use. This

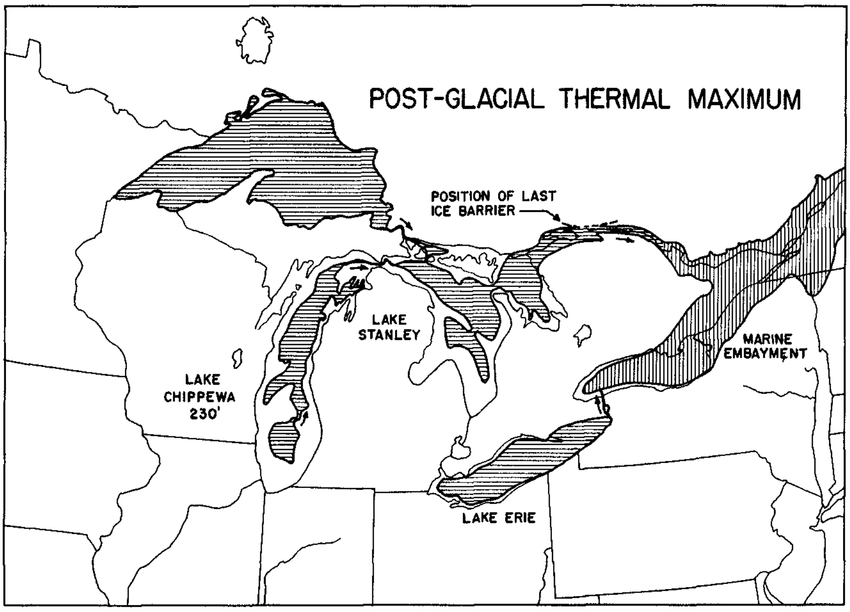

FIG. 15 - **Lake stage map no. 15: Lake Chippewa, Lake Stanley, Early Lake Superior, Lake Erie, Ontario marine embayment.**

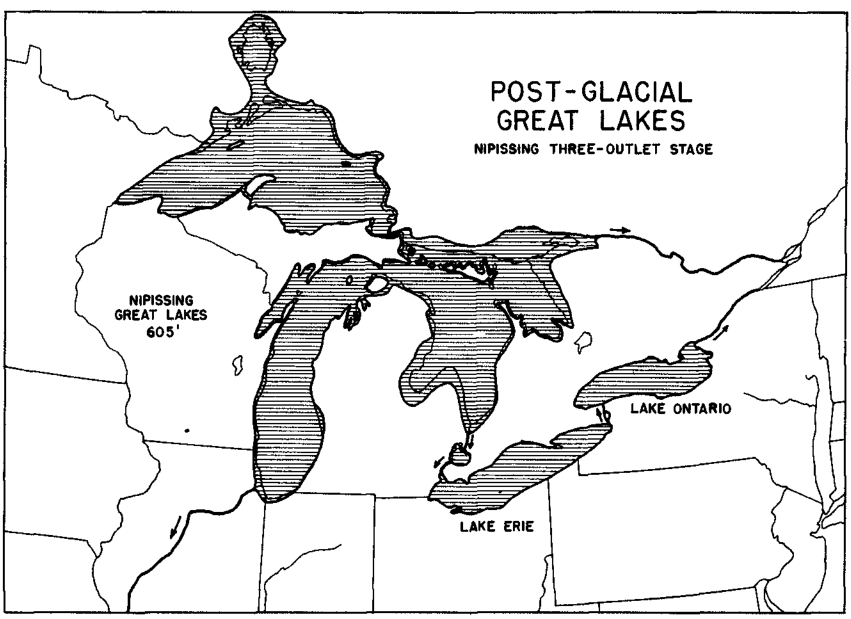

FIG. 16 - **Lake stage map no. 16: Nipissing Great Lakes.**

outlet served as the only dischargeway for the lake in the Huron basin until the water level was raised to the elevation of the old southern outlet channels.

THE NIPISSING GREAT LAKES

When upwarp of the land to the north raised the water level in the Huron and Michigan basins to the level of the old southern outlets at Port Huron and at Chicago, and discharge to the south began, the Nipissing stage was produced.

Because the old southern outlets had been abandoned when the lake was at the 605-foot Algonquin level, it is reasonable to assume that when a portion of the Nipissing discharge reoccupied these outlets the lake level would again stand at 605 feet. By this time glacial ice had disappeared from all of the lake basins, and the Nipissing Great Lakes extended throughout the Superior, Michigan, and Huron basins, as a single body of water. The total volume of its discharge probably was greater than that of the Algonquin stage, because during the Algonquin stage much of the discharge of the Superior basin went southwestward down the St. Croix River.

While the Nipissing stage discharged through three outlets, the rising North Bay outlet and the two stable southern outlets, the volume of its discharge through any one of the outlets was inadequate to deepen the outlet channels by scour. The development of the strong Nipissing beach probably took place during this three-outlet stage. Radiocarbon dating of peat associated with the Nipissing beach in the southwestern part of the Superior basin has given us a date of 3656 ± 640 years ago for its formation (Arnold and Libby, 1951). Similar dating of a beach deposit at the 605-foot level, at the south end of the Lake Michigan basin, has given a date of 3469 ± 230 years ago (Arnold and Libby, 1951).

When continued rise of the land to the north carried the North Bay outlet above the level of the southern outlets, the entire discharge of the Nipissing Great Lakes passed through the two southern outlets. The outlet at Chicago, floored with bed rock, could not be cut down, but the outlet at Port Huron, which was floored with unconsolidated till, was susceptable to downcutting by the increased discharge. Downcutting at Port Huron by this mechanism accounts for the termination of the Nipissing stage.

THE ALGOMA STAGE

During the lowering of lake level from the Nipissing stage to the present level (by downcutting of the Port Huron outlet) there was one well-defined static period, during which the Algoma beach was formed. In the northern, upwarped, part of the region the Algoma beach is a feature of moderately strong development which lies between the Nipissing and the present lake beaches. It, like the Nipissing beach, descends southward toward the unwarped southern areas but can not be traced continuously to a junction with a beach in the south. It appears likely, however, that a moderately strong horizontal beach at elevation 596 in the south is the Algoma beach. Radiocarbon dating of charcoal associated with the 596-foot level deposits (at Port Franks, Ontario) gives a date of 2619 ± 220 years ago for their accumulation (Libby, 1952). These deposits contain evidences of human occupation (Dreimanis, 1952).

ATTAINMENT OF THE PRESENT LEVEL

If the radiocarbon dates quoted above are reliable, the water surface in the Michigan and Huron basins stood at an elevation of 605 feet only about 3500 years ago, and it stood at the 596-foot level only 2600 years ago. It follows, then that the water surface was lowered to the present stage (581 feet) at a time which probably was considerably less than 2000 years ago.

PRACTICAL IMPORTANCE OF THE GREAT LAKES HISTORY

Many highways and railroad rights-of-way in the Great Lakes region are located on beaches which were formed during some of the higher lake stages. Other features, formed during lower stages, are not readily recognizable by casual inspection but nevertheless may be of great importance in engineering design. During low-water stages many of the rivers which discharge into the lakes were deeply entrenched in their valleys; when the lakes rose, the lower courses of these rivers were drowned and deposition of fine-grained materials in the resulting deep channels then occurred.

A typical example of a construction problem resulting from the lowering and raising of lake level is the bridge project at Saugatuck, Michigan (reported by Stanley, 1938). Borings at the bridge site were made in the bed of the Kalamazoo River, near its mouth, and they penetrated 85 feet in an apparently recent fill of fine sand and peat which constituted very poor foundation material. Similar valley-fill problems in the Chicago area will be described in another paper on the present program (Peck, R. B., "The influence of subsurface conditions on the design of foundations for water front structures in the Great Lakes area").

Many of the streams tributary to the Great Lakes may be expected to contain deep fill in their mouths. The problem of interpreting borings made in such fills is complicated, however, by the occurrence of more than one period of low water. Two periods of extreme low water were described in the foregoing history of the lakes. The first of these occurred during the Two Creeks interval, prior to the advance of the Mankato ice which extended part way down the Michigan and Huron basins and which completely filled the Superior and Ontario basins. In these areas covered by the Mankato ice, glacial till may have been deposited in the entrenched valleys, possibly overlying fine-grained alluvial fill. South of the Mankato glacial boundary (fig. 10), however, no till will be found overlying the alluvium in the valleys which were entrenched during the Two Creeks interval.

The second extreme low water period occurred after the Mankato glacial substage, during the Chippewa-Stanley stage. Entrenchment of streams at this time would remove at least a part of the pre-existing valley fill (alluvial, or alluvial and glacial where both existed). Subsequent drowning of the valleys during the Nipissing stage would permit the deposition of new alluvial fill. Again, in the valleys south of the Mankato glacial boundary (fig. 10) no till will be found overlying alluvium deposited after the Two Creeks interval. North of the Mankato boundary, however, the following sequence is possibly present in some of the valleys; Nipissing to recent alluvium at the top, Mankato glacial till, alluvium, then older glacial till or bed rock. In other words, the first till encountered in a boring in a valley north of the Mankato boundary may be thin and it may have alluvium under it. The alluvium may be expected to have a relatively small bearing capacity.

The relationships just described may be complicated further by other low water stages which occurred at earlier dates, previous to the known history of the lakes. Pre-Cary valleys cut in bed rock or in older glacial deposits are known in the area of the Cary drift in northeastern Illinois. Such valleys may contain Cary glacial till overlying alluvium deposited during some early lake stage.

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CHAPTER 6

SOURCE MATERIALS FOR LAKE MICHIGAN BEACHES

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The extensive wave erosion which has occurred in many parts of the Lake Michigan shoreline since 1943 has centered interest on the materials composing the bluffs above the water line and on the amount and grain size of beach sediments yielded by wave erosion of these bluffs. The present study grew out of a preliminary mapping of the landform types and shore materials of the Lake Michigan basin carried out in 1952-53 for the Beach Erosion Board of the United States Army Corps of Engineers. The report prepared for the Beach Erosion Board summarized the geology of the Lake Michigan shoreline. Types of bedrock were indicated not only where such rocks form cliffs or reefs above water level but also beneath the unconsolidated materials which form three-quarters of the actual shoreline. Such unconsolidated materials were also mapped as to type and present form, and on these bases the shoreline was divided into physiographic units. These physiographic units and materials will be evaluated with respect to the total proportion of each type along the shoreline and the sediments yielded to the beach zone by wave erosion.

COMPLEXITY OF LAKE MICHIGAN SHORE MATERIALS

Bedrock forms about twenty-five per cent of the Lake Michigan shoreline, almost entirely in the northern third of the lake basin. Although the Lake Michigan basin and Green Bay lowland are eroded mainly along the outcrop of shale formations, the latter are generally covered by unconsolidated deposits, and rocks exposed along the shore consist almost wholly of dolomites of Ordovician, Silurian, Devonian, and Mississippian geologic age. Although these dolomites differ considerably in thickness of individual beds and amount of interbedded shale, they all retreat but slowly under wave attack and yield coarse slabby gravel or large stone blocks which effectively resist the waves. Such bedrock forms cliffs or reefs along the east shore of Door Peninsula north of Jacksonport and the western coast of the same peninsula, most of the shores of the Stonington and Garden Peninsulas north of Green Bay, considerable stretches of the north shore of the lake between Manistique and St. Ignace, much of the shore from Mackinaw City westward to Waugoshance Point, and the south shore of Little Traverse Bay. Other scattered exposures of bedrock include reefs at Sheboygan, Wind Point north of Racine, and Fifty-first and Seventy-ninth Streets in Chicago. No exposures are known to occur on the east coast between Chicago and Grand Traverse Bay, but reefs are present near Norwood and Charlevoix farther north.

Three-fourths of the Lake Michigan shoreline consists of unconsolidated deposits resting on and concealing the bedrock. Such deposits belong to three groups: those of glacial origin, those of lacustrine origin deposited when the lake waters stood at higher levels, and sands dropped by wind action along both the present lake shore and the shorelines of former higher stages of the lake.

Surface glacial deposits belong to the Cary and Valders-Port Huron substages of the last, or Wisconsin, glacial stage. Deposits of the three pre-Wisconsin glacial stages may and probably do underlie the Wisconsin, but such older glacial materials have not yet been positively identified along the present shoreline. During the Wisconsin glacial stage, glacial lobes from eastern Canada advanced down the Lake Michigan and Green Bay lowlands. Along the Door Peninsula, a dolomite ridge, these two opposing glacial lobes merged. Unstratified glacial till and washed deposits of sand and gravel were dropped by these glaciers as, through melting, they shrank back through the Lake Michigan and Green Bay lowlands to some position north of the Lake Michigan basin. While receding, they impounded a glacial lake which stood sixty feet above present Lake Michigan in the Chicago area. Later the same two glacial lobes readvanced as far as Fond du Lac south of Green Bay, and in the Lake Michigan basin as far as Milwaukee and Muskegon. These deposits mark the Valders-Port Huron glacial substage and consist also of glacial till, sand, and gravel. These glacial lobes also receded or melted back toward their source region northeast of the Great Lakes.

As the glaciers last receded, water was impounded between the shrinking ice front and the glacial drift deposits forming the margins of the Lake Michigan basin. These lake waters like the earlier pre-Valders lake overflowed first southwest of Chicago to the Illinois River. During and following the glacial episode, the region northeast of the Great Lakes was lower than now, and the waters of the post-glacial lakes submerged extensive areas in the northern part of the Lake Michigan basin. At an earlier lake stage, the Algonquin, the lake waters stood 220 feet higher than Lake Michigan in the Straits of Mackinac area, and at a later lake stage, the Nipissing, more than 50 feet higher in the same area. Deposits made in these former lakes are extensive in the northern part of the Lake Michigan basin area, and consist of sand and gravel where the water was shallow, clay and silt in deeper off-shore areas.

Sand dunes line long stretches of the present shoreline and are also associated with the sandy plains and shores of the higher lake stages. Such dune areas are extensive along the east shores of Lake Michigan from Gary northward to Sleeping Bear Point and at a few places on the north shore of the basin. Sand dunes occur at very few places along the west shore of Lake Michigan and Green Bay. Their absence on the west and presence on the east of the lake basin is attributed to the fact that strongest and driest winds in this region commonly blow from the west.

PHYSIOGRAPHIC UNITS OF THE LAKE MICHIGAN SHORE

Physiographic units of the Lake Michigan shore have been established on the basis, first, of shore materials and, second, of coastal landform types. Shore materials consist of the following:

Table 1. Materials Composing Lake Michigan Shores Above Beach Zone.

Glacial till -- clayey
Glacial till -- sandy
Sand and gravel -- either glacial or lacustrine
Gravel -- either glacial or lacustrine
Dune sand -- eolian
Silt and clay -- lacustrine
Bedrock

Glacial till is an unstratified glacial deposit consisting of boulders, gravel, sand, and grit distributed through a matrix of fine ground-up rock. Where the matrix is sticky when damp, the glacial till is described as clayey. Where the matrix is sandy and has less coherence when damp, the till is classified as sandy. Stratified sand and gravel deposits may be of either glacial or lacustrine origin. Although the two may be readily distinguished by such properties as their bedding, a distinction between them in the present study serves no purpose. Gravel with most particles above four millimeters in size may also be either glacial or lacustrine in origin. Dune sand is remarkable for its very large proportion of particles between one millimeter and one-fourth millimeter in size. Silt and clay deposits are thinly stratified and represent the deeper water sediments of the lake at its former higher stages.

The landform types above the beach zone may be classified as follows:

Table 2. Landform Types Of Lake Michigan Shores, Other Than Beaches.

High bluff composed of (clayey till
(sandy till
(sand and gravel
(gravel
(dune sand
(silt and clay
Low bluff composed of same six materials
Low plain composed of (sand and gravel
(gravel
(silt and clay
Dune, no bluff
Steep cliff composed of bedrock
Low rock reefs in water, below inland plain composed of sand and gravel
Sand bar or spit, associated with beach

Where the bluff is not more than twenty feet high, it is classed as Low Bluff; if higher than twenty feet, as High Bluff. Bluffs, both the high and low, are associated with wave erosion and recession of the shore. Low Plain, on the contrary, generally marks areas that are stable or aggrading. Sand bars or spits are dominant elements of the shore at a few places where they have closed off lagoons or inland lakes. They mark aggradation of the shore.

Although the above table lists nineteen physiographic types on the basis of form and material, some are too limited in extent to appear on a small scale map of the lake basin. However, thirteen of the types occur frequently and comprise extensive proportions of the shoreline as listed in Table 4 of this report. On the basis of these thirteen types of physiographic units, 196 shoreline units were recognized and indicated on a map of scale 1:500,000 for the Beach Erosion Board. By using appropriate combinations of symbols, it was possible to indicate on the map both the coastal landform type and the nature of the underlying materials for each of these 196 units.

SEDIMENTS YIELDED BY SHORE EROSION OF THE SEVEN TYPES OF SHORELINE MATERIALS

Bedrock cliffs commonly shed coarse rock fragments varying from flat platy shingle fragments a few inches in size to massive blocks several feet in dimension. All these materials remain on the beach and effectively protect the cliffs behind from rapid erosion inasmuch as the wave energy is largely expended in chewing these rock fragments. Where gravel forms the shoreline practically all of the gravel remains on the beach where it also effectively retards wave erosion. No analyses were made of the large fragment sizes composing such gravel and bedrock rubble.

The other five types of shore materials form extensive portions of the shoreline and in many places are undergoing erosion. Grain size analyses were run on representative samples from these five types of materials in order to determine the proportions likely to remain in the shore zone and thus contribute to the beach. A summary of these grain size analyses is given in Table 3 below:

Table 3. Sediments Yielded by Wave Erosion Of Shore Materials:

MATERIAL	NO. OF SAMPLES	SIZE, IN MM. ABOVE 4	4-1	1-¼	¼-1/16	BELOW 1/16
CLAYEY TILL	5	2.4%	2.9	8.7	11.4	74.7
SANDY TILL	3	23.4	7.8	16.3	17.7	34.6
SAND & GRAVEL	11	0.9	2.1	77.8	17.2	2.0
DUNE SAND	5	0.0	0.0	73.3	25.1	1.7
CLAY & SILT	2	0.0	0.2	0.6	2.8	96.5

The results summarized above indicate that sandy till contains a large proportion of sediment above four millimeters in size. Clayey till, on the contrary, consists predominantly of matrix material below one-sixteenth millimeter. Both sand and gravel and dune sand consist predominantly of particles between one millimeter and one-fourth millimeter with most of the remainder between one-fourth and one-sixteenth millimeter. Clay and silt are composed almost entirely of particles of less than one-sixteenth millimeter in size. During disintegration and sorting by wave erosion, most of the clay and silt and three-fourths of the clayey till may be expected to be removed by the lake waters in suspension. Sand and gravel and dune sand, although relatively fine, consist mainly of particles that will remain on the beach.

PROPORTION OF SHORELINE FORMED BY EACH OF THE THIRTEEN PHYSIOGRAPHIC UNIT TYPES

The shoreline of Lake Michigan and Green Bay is approximately 1,216 miles in length. The proportion of this shore composed of each of the thirteen principal types of physiographic units is indicated in the following table:

Table 4. Proportion Of Lake Michigan Shoreline Formed By Each Principal Type Of Physiographic Unit:

	TOTAL LENGTH, MILES	% OF TOTAL
HIGH BLUFF OF CLAYEY TILL	193.0	15.9
HIGH BLUFF OF SANDY TILL	16.5	1.4
HIGH BLUFF OF SAND AND GRAVEL	43.1	3.5
HIGH BLUFF OF CLAY AND SILT	11.8	1.0
HIGH BLUFF OF DUNE SAND	127.5	10.5
LOW BLUFF OF CLAYEY TILL	18.1	1.5
LOW BLUFF OF SAND AND GRAVEL	162.0	13.3
LOW BLUFF OF CLAY AND SILT	8.6	0.7
LOW PLAIN OF SAND AND GRAVEL	258.7	21.3
DUNES, NO BLUFF	61.2	5.0
BAR OR SPIT OF SAND OR GRAVEL	6.3	0.5
CLIFF OF BEDROCK	257.1	21.8
LOW BEDROCK REEFS IN FRONT OF SANDY PLAIN	52.2	4.3
	1,216.1 MILES	100.7 %

It will be noted that about 32 per cent of the shoreline consists of high bluffs, mainly of clayey till and dune sand. More than 15 per cent of the shore is low bluffs, mainly of sand and gravel marking ancient lake bottom. More than 21 per cent of the shore is a low plain of sand and gravel. This landform type is most extensive along the west shore of Green Bay and in the Chicago area. Dunes without a bluff and thus indicating more or less stable shore conditions form only 5 per cent of the shoreline. Bedrock cliffs and low reefs are present along more than 26 per cent of the shoreline.

SIGNIFICANCE OF BEACH MATERIALS YIELDED BY EROSION OF BLUFFS

The grain size distribution of the unconsolidated materials forming three-fourths of the Lake Michigan shoreline is significant with respect to the proportion of such material likely to remain in the beach zone. Material remaining on the beach builds up the latter and retards erosion of the bluffs behind. Particles less than one-sixteenth millimeter in diameter are suspended by wave agitation and float out into deeper portions of the lake where they settle. Particles within this grain size, therefore, contribute almost nothing to the beach. Only materials greater than one-sixteenth millimeter will stay on the beach to protect the shore. Table 3 indicates what the principal types of unconsolidated shore materials are likely to contribute to the beach under wave erosion and sorting in the beach zone.

In November, 1952, and September, 1953, 139 shore localities along Lake Michigan shores were visited; and at most, samples of shore materials were collected. At 90 of the 139 localities, shore erosion appeared to be in progress; and in a number of cases, rapid retreat of the bluffs was indicated. 34 localities appeared to be stable; and at 14 localities, the shore seemed to be aggrading. At those many points where erosion is occurring under present conditions, protection of the shore from further destruction must be based on an intelligent utilization of beach materials furnished by the waves themselves in their attack on the shore.

CHAPTER 7

SOME CHARACTERISTICS OF BOTTOM SEDIMENTS ALONG THE ILLINOIS SHORE LINE OF LAKE MICHIGAN

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The State of Illinois is engaged in a long range study of shore erosion phenomena along the Illinois shore of Lake Michigan in order to obtain more definite information on the numerous engineering, geologic and meteorologic factors involved in the erosion process.

One phase of this study is devoted to the characteristics of the bottom sediments of the lake based upon samples taken annually along certain selected range lines. These range lines were first established for the 1946 cooperative study with the Beach Erosion Board, Corps of Engineers and were used that year by the Corps and again in 1950, 1951, 1952 and 1953 by the Division of Waterways. The ranges are spaced approximately one mile apart commencing at the Wisconsin-Illinois State line and extending southward through Chicago to the Indiana-Illinois line. At the present time, the sampling program is confined to that reach of the lake from the Wisconsin line to the northern limits of Chicago. (Fig. 1.) Each range line azimuth was established at approximately 90° to the general shore line at that locality.

Approximately 22 lake bottom samples are taken each year on each range. An attempt has been made to obtain a sample at each 2 ft. change in depth from the shore line to about the 15 ft. depth, and then at each 3 ft. to 5 ft. depth change thereafter to a depth of 60 feet, or three miles distance from shore, depending on which is reached first.

The samples are taken from an amphibian truck and the course of the truck and the location of each sample is controlled by two beach parties operating transits for fix angles, etc. Communications during the sampling are maintained by two-way radio operations between the truck and the beach crews. The samples thus secured are surface samples and are taken with a drag-type sampler based upon recommendations of the Beach Erosion Board, (Figs. 2 and 3). The sampler consists essentially of a short section of pipe equipped with a cutting lip, a pipe cap for emptying and a bail for attaching the cable. Stability while in motion through the water was considerably improved by the addition of a vertical fin to the top of the sampler. Very satisfactory results have been obtained in sand bottoms, but considerable difficulty has been experienced in beds of large gravel and on clay or fill bottoms. A drop-type sampler has been used to a limited extent in clay and fill areas with some degree of success.

Beach samples are collected on the shore end of each of the ranges. In general, samples are taken at the water's edge, the toe of the bluffs and at a point between.

Laboratory work on samples has included the following:

1. Sieve analysis, using the $\sqrt{2}$ series sieves throughout the

range down through the No. 325 mesh.

2. Separation of light and heavy minerals, using bromoform.

3. Magnetic mineral separation, using a weak magnet.

4. Carbonate separation, using diluted hydrochloric acid.

The analysis for heavy minerals, magnetic minerals and carbonates was made in the hope that movement of the sediments might be traced. However, no unusual concentrations were found which could be attributed to any one source material. The glaciation which occurred over this area with the resultant extremes of water depths during those periods, has evidently so sorted the bluff, beach and bottom material that the presence in various amounts of heavy minerals, magnetic minerals or carbonates gives no apparent indication of sediment movement or littoral drift. Therefore, in 1952 the magnetic and carbonate analyses were discontinued, although portions of each sample have been prepared and sorted should these analyses ever need to be resumed.

The results of the grain size analysis of each sample are plotted in the form of a cumulative curve from which the median and quartile diameters and other descriptive parameters may be obtained.

With the thought in mind that a definite pattern of sediment size distribution might be present along the Illinois shore line, plan sheets for each year's survey have been prepared upon which are plotted the location and median diameter of each offshore sample. There were then drafted thereon lines of equal median diameter, much in the same manner as the drafting of contour lines on a topographic map. It is recognized that this is not a precise method in view of the relatively small number of samples and the wide spacing of the ranges, but it is felt that such a procedure does serve to present a general pattern of the size distribution of the material on the lake bottom. Fig. 4 is the plot of the median diameters from the Wisconsin State line to Waukegan for the 1950 and 1952 surveys. This area is in the Northern Lake Plain section, is subject to marked erosion at the north, and due to the presence of Waukegan Harbor and the breakwater at Northern Illinois Public Service Company, is accreting at the south. For simplicity, the size distributions have been broken into three groups: (1) Sizes up to 0.25 mm median diameter, called fine sand or under; (2) Sizes from 0.25 to 1.00 mm designated as medium sand, and (3) Sizes above 1.00 labelled as gravel or above. As can be seen, the 1950 and 1952 plots show a marked similarity and this same similarity was also present for the 1946 survey.

This reach of the shore is practically in its original state with few man-made structures present except at Waukegan. Fig. 5 shows a further breakdown of the median diameters in the inshore area into medium sand (0.25 mm or above), fine sand (.25 mm to .10 mm), and very fine sands or under (.10 mm or below). There is a narrow band of fine sand (.25 to .10 mm median diameter) present along the immediate shore line, outside of which very fine sand is present out to about the 60 ft. depth. Using P.D. Trask's formula for sorting coefficient to indicate the slope of the mechanical analysis curve of each sample, the fine sands and very

SOME CHARACTERISTICS OF BOTTOM SEDIMENTS ALONG THE ILLINOIS SHORE LINE OF LAKE MICHIGAN

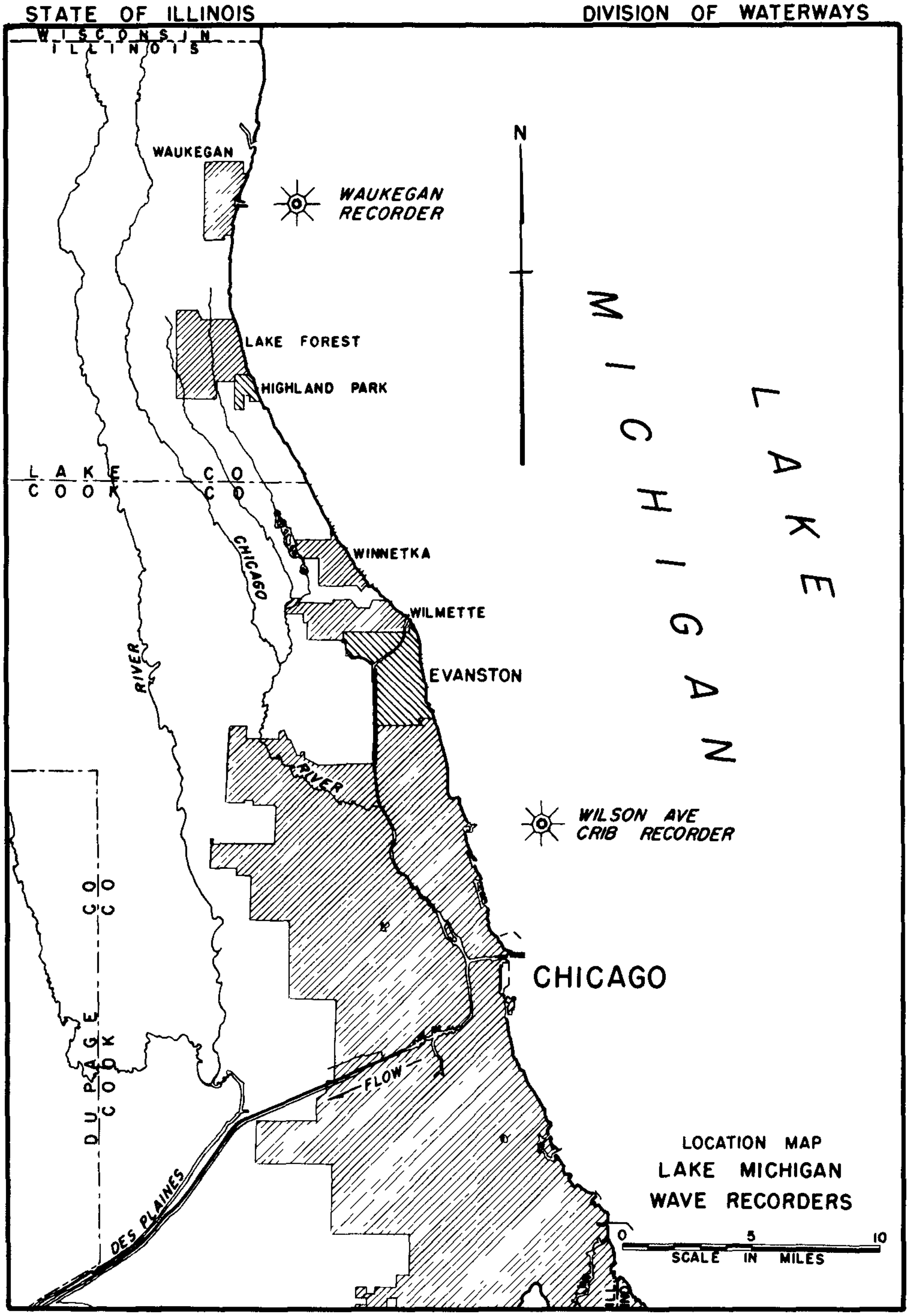

Fig. 1.

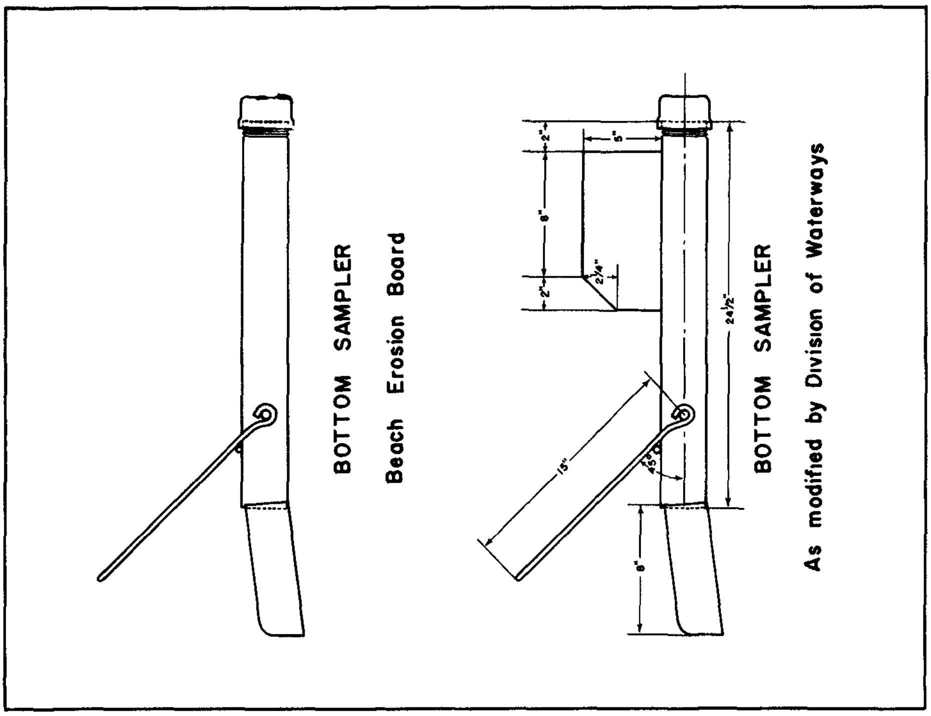

Fig. 3

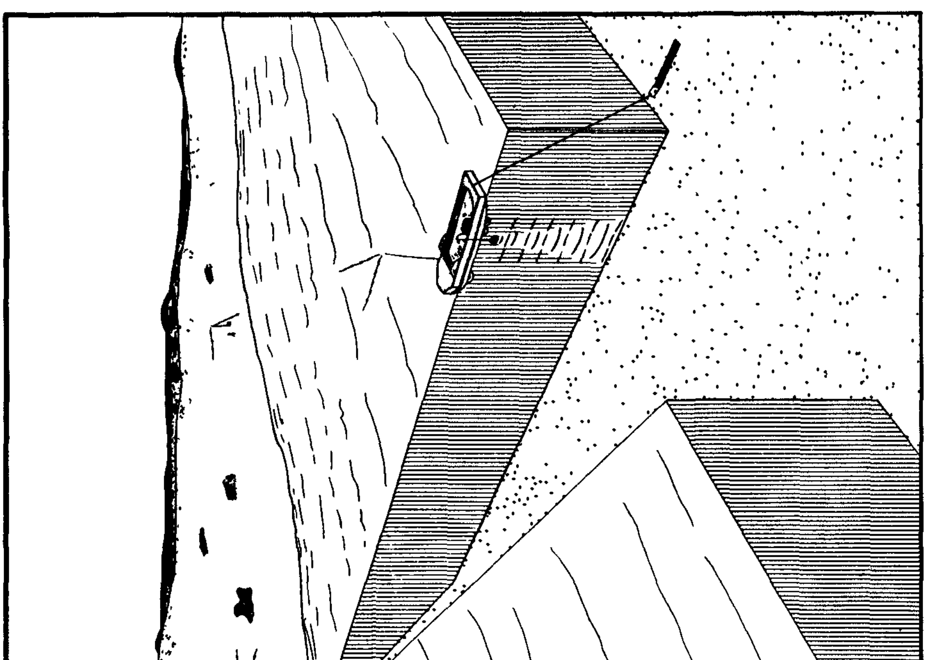

Fig. 2. Diagram of sounding and bottom sampling operation.

SOME CHARACTERISTICS OF BOTTOM SEDIMENTS ALONG THE ILLINOIS SHORE LINE OF LAKE MICHIGAN

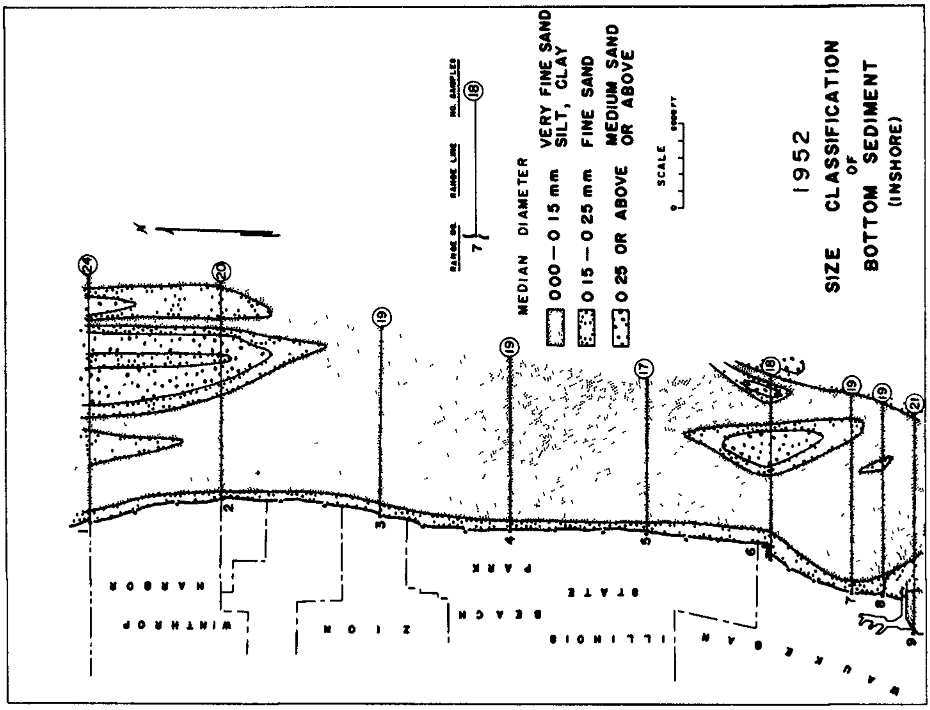

Fig. 5.

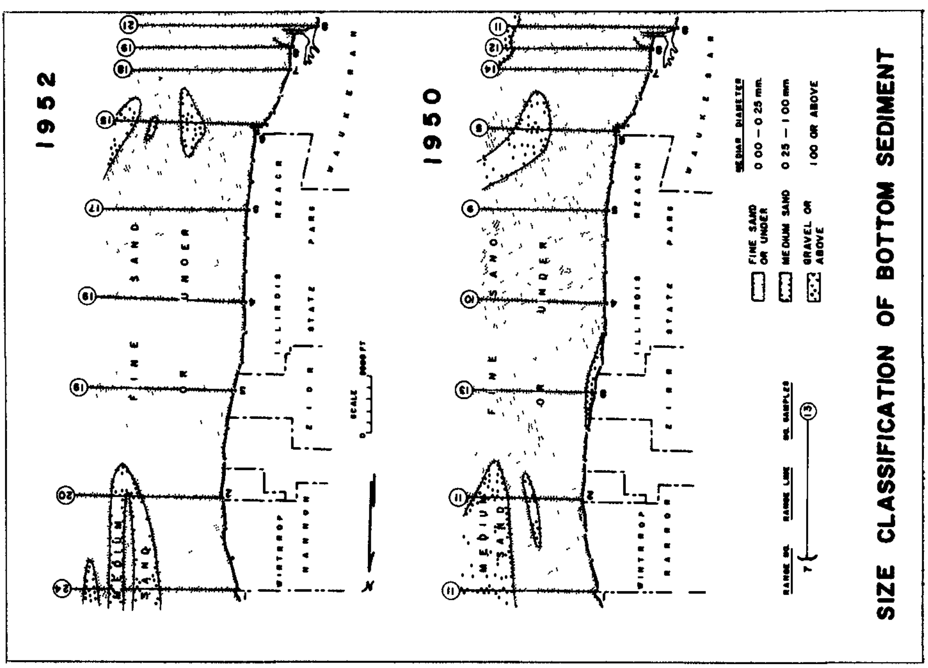

Fig. 4.

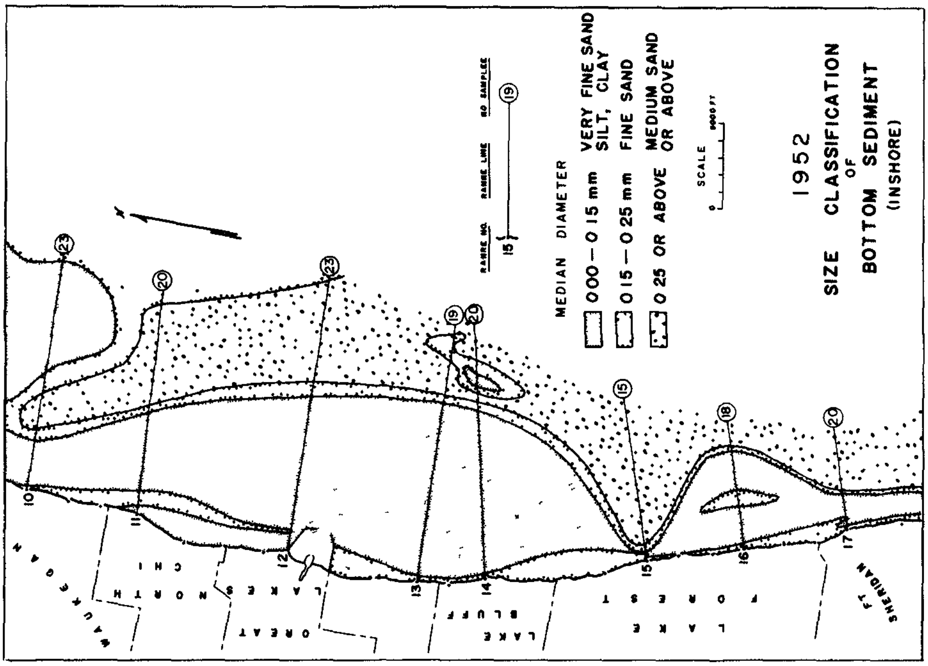
1952
SIZE CLASSIFICATION OF BOTTOM SEDIMENT
(INSHORE)
MEDIAN DIAMETER
0 00 – 0 15 mm VERY FINE SAND SILT, CLAY
0 15 – 0 25 mm FINE SAND
0 25 OR ABOVE MEDIUM SAND OR ABOVE
RANGE NO.
RANGE LINE
NO SAMPLES
SCALE
WAUKEGAN
NORTH CHI
GREAT LAKES
LAKE BLUFF
LAKE FOREST
FT SHERIDAN

Fig. 7.

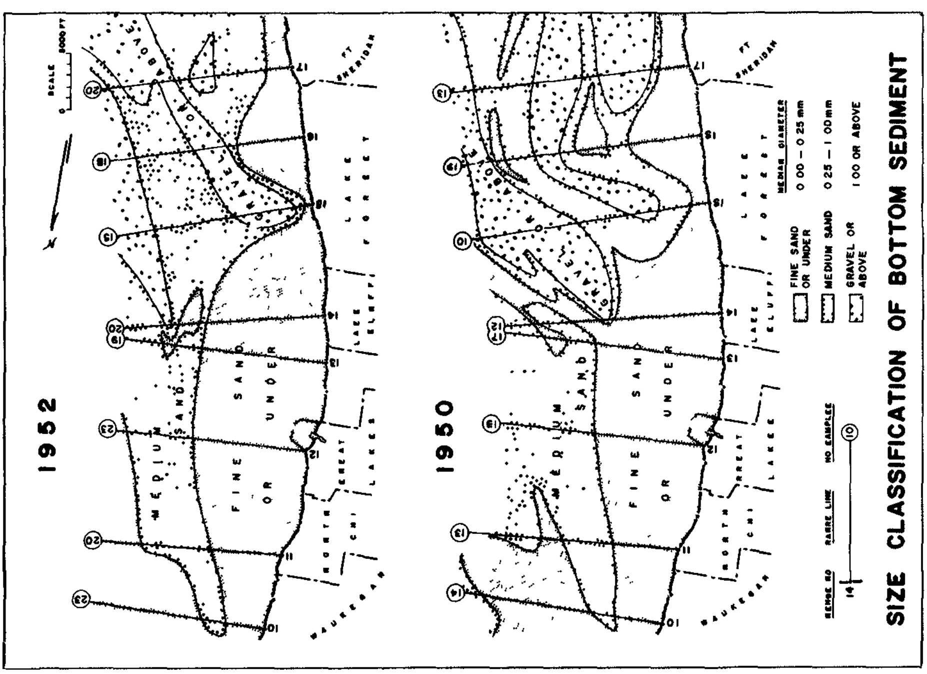
1952
1950
SIZE CLASSIFICATION OF BOTTOM SEDIMENT
MEDIAN DIAMETER
FINE SAND OR UNDER 0 00 – 0 25 mm
MEDIUM SAND 0 25 – 1 00 mm
GRAVEL OR ABOVE 1 00 OR ABOVE
RANGE NO
RANGE LINE
NO SAMPLES
SCALE
MEDIUM SAND
FINE SAND OR UNDER
GRAVEL OR ABOVE
WAUKEGAN
NORTH CHI
GREAT LAKES
LAKE BLUFF
LAKE FOREST
FT SHERIDAN

Fig. 6.

fine sands are well sorted, having coefficients of from 1.1 to about 1.6. The foreshore bottom slopes vary in this section from about 1 on 25 at the Wisconsin State line to about 1 on 65 immediately above Waukegan Harbor. From the records of accretion above the harbor, it is evident that the predominant sand movement is in a generally southward direction and is of considerable amount. Fig. 6 is the plot of the median diameters from Waukegan through Fort Sheridan. This area is in the Lake Border Moraine section and is characterized by erosion along the entire reach except immediately above Great Lakes Harbor. As shown by the plot, the inshore area is composed entirely of fine sand or under, with a greater amount of medium sand offshore and the presence of gravel or above showing up off Lake Bluff, Lake Forest and Fort Sheridan. Again the similarity between the 1950 and 1952 survey results are apparent. Fig. 7 shows the further breakdown of the fine sands in the inshore area. As in the reach from the Wisconsin State Line to Waukegan, there is a narrow band of fine sand present along the immediate shore. However, it can be noted that this sand is apparently not a continuation of that occurring along the shore above Waukegan. Waukegan Harbor has seemingly diverted the sand above the harbor offshore with a portion of it extending both north and south of the harbor. Very fine sand or under occurs off Great Lakes and this covers a considerable area. Again the fine sands show good sorting with coefficients ranging from 1.1 to 1.7. The medium sands and the gravels' have a coefficient of from 2.0 to about 6.0. The shore line from Waukegan to Great Lakes is largely protected by rip rap but the natural foreshore slopes in this reach were very steep. The foreshore slopes immediately above Great Lakes are about 1 on 55 and these steepen to about 1 on 20 or below near Fort Sheridan.

Fig. 8 is a continuation of the median diameter plot and covers that reach from Fort Sheridan south to Kenilworth. The 1950 Survey, due to unfavorable weather, was discontinued at Highland Park, so for comparison purposes, the 1946 Survey was plotted for the area southward of that point. Sufficient samples were not taken during this survey to adequately define this area, and therefore, the comparison from this point southward will not be in complete agreement.

This reach is a continuation of the Lake Border Moraine Section. The inshore area is still characterized by a narrow band of fine sand or under, but the offshore area shows an increasing amount of medium sands and gravels in apparently no explainable pattern. Southward from the northern limits of Glencoe, the drag sampler did not obtain samples of sufficient quantity for analysis in the areas shown as medium sands and gravels. A drop type sampler similar in design to that used by Dr. Hough of the University of Illinois was developed and samples of about 2 inches in depth were obtained in these areas. The interesting feature of these samples is that this offshore area is shown to be composed of clay overlain with a very thin layer of medium sands and gravels on the surface. Therefore, the pattern of offshore gravels and medium sands is possibly in error and misleading. However, the size distribution as shown seems to be indicative of the characteristics of the surface materials along this reach.

Fig. 9 shows the further classification of the inshore sediments. The immediate offshore area is again composed of fine sand with very fine

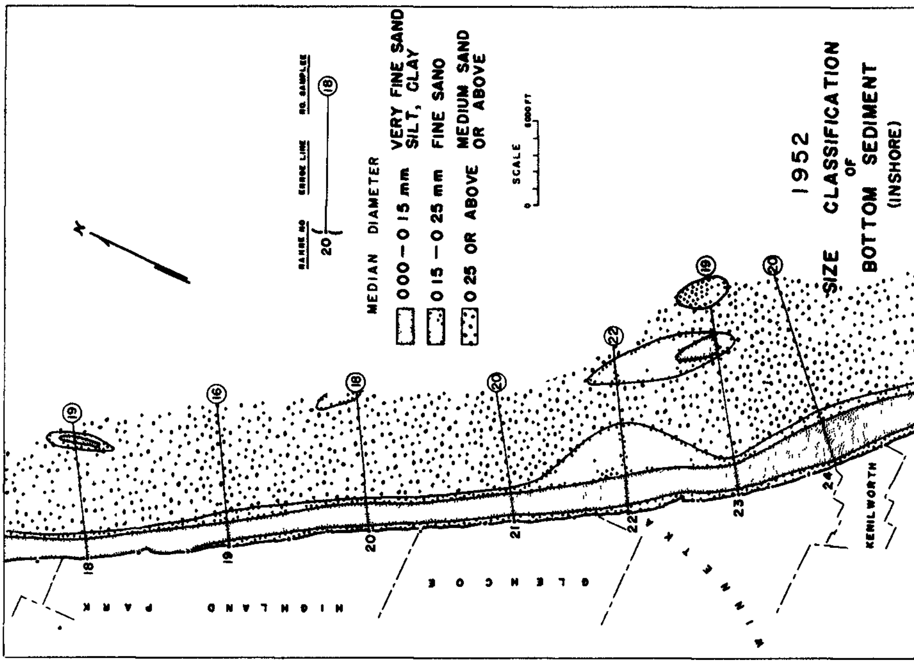
MEDIAN DIAMETER
000 – 0 15 mm VERY FINE SAND SILT, CLAY
0 15 – 0 25 mm FINE SAND
0 25 OR ABOVE MEDIUM SAND OR ABOVE
SCALE
1952
SIZE CLASSIFICATION OF BOTTOM SEDIMENT (INSHORE)
HIGHLAND PARK
GLENCOE
WINNETKA
KENILWORTH

Fig. 9.

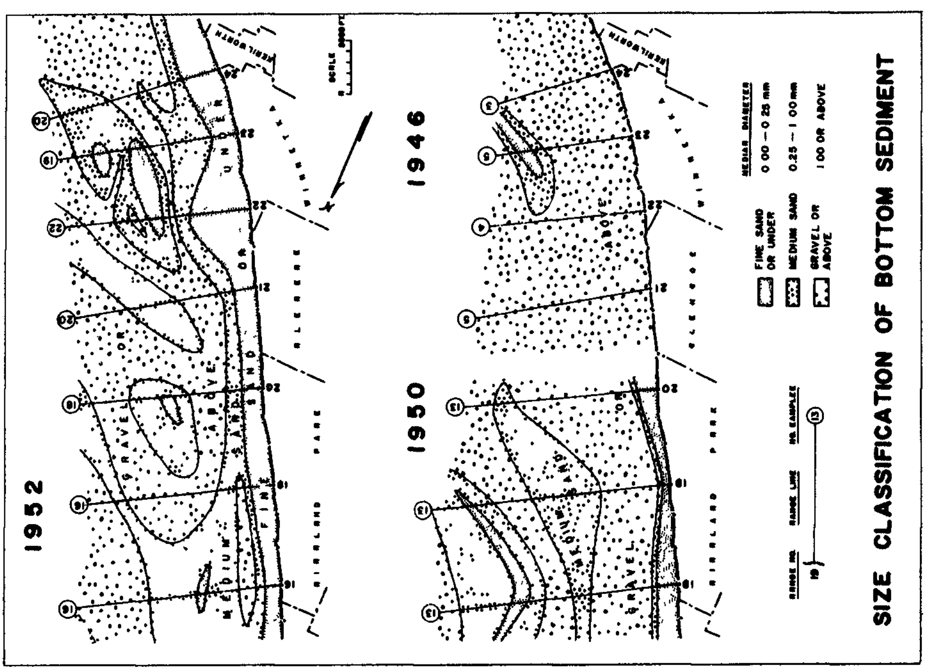
1952
1950
1946
MEDIAN DIAMETER
FINE SAND OR UNDER 0.00 – 0.25 mm
MEDIUM SAND 0.25 – 1.00 mm
GRAVEL OR ABOVE 1.00 OR ABOVE
SCALE
SIZE CLASSIFICATION OF BOTTOM SEDIMENT

Fig. 8.

SOME CHARACTERISTICS OF BOTTOM SEDIMENTS ALONG THE ILLINOIS SHORE LINE OF LAKE MICHIGAN

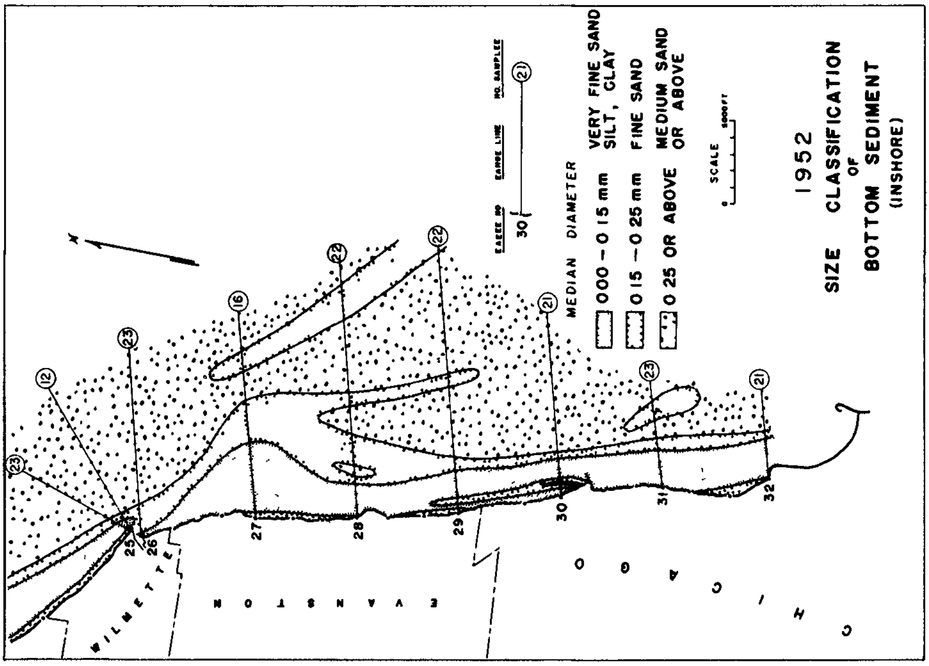

Fig. 11.

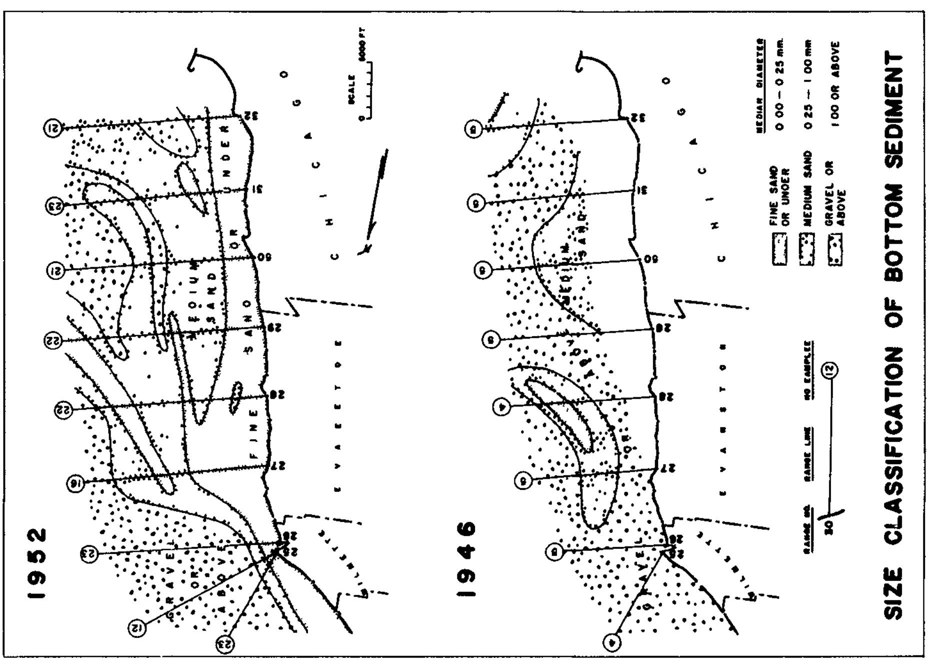

Fig. 10.

sand further offshore. However, in this reach, the area of very fine sand is relatively narrow and is completely bounded on the east by another strip of fine sand. The sorting coefficients again vary from about 1.1 to 1.5 for the fine sands and from 2.0 to 6.0 for the medium sands and gravels. Foreshore bottom slopes in this area are from 1 on 12 to 1 on 20 and are fairly uniform from north to south.

Fig. 10 is the median diameter plot from Kenilworth to Foster Avenue at Chicago. The reach from Kenilworth to Wilmette Harbor is a continuation of the Lake Border Moraine Section and that below Wilmette is in the Southern Lake Plain Section. This reach again has the inshore area of fine sands or under and the offshore has more medium sands, fine sands and less gravel than the area immediately north. The drop sampler was also used in this area for the gravel and some of the medium sand areas and these areas were also found to be composed of clay overlain with very thin layers of sand and gravel on the surface.

Fig. 11 shows the further classification of the inshore sediments. Below Wilmette Harbor, the inshore area of fine sand is not continuous along the beach as was the case in the areas to the north. The continuation of the fine sands above Wilmette Harbor are now offshore about 2000 ft. Except in isolated locations, the very fine sands are immediately adjacent to the shore. The shore immediately below Wilmette Harbor is not largely protected by bulkheads and rip rap but in the unprotected areas, the foreshore slopes are on the order of 1 on 10 to 1 on 15. The sorting coefficients of the fine sands and under are still on the order of 1.1 to 1.6 and the medium sands and gravels from 2.0 to 7.0.

Fig. 12 shows three typical profiles of the lake bottom with the complete median diameter size classification of the samples taken. This classification is that used by the U.S. Bureau of Soils. The first profile was taken off the Illinois Beach State Park, the second at Fort Sheridan and the third at the southern city limits of Evanston.

The above comments and illustrations have been concerned with the general overall characteristics of the bottom surface sediments in Lake Michigan along the Illinois shore line. In addition to this study, a detailed sampling program has been in progress for certain selected groin systems. Samples and soundings have been taken at regular intervals on 8 groins in the Lake Bluff area. These groins were constructed by a landowner who was suffering extensive loss of land due to erosion of the high bluff fronting on the lake. The groins were constructed in 1952 and have, as yet, only partially controlled the bluff erosion. Eight sampling ranges and seven additional sounding ranges were established. Ten samples per range were taken out to a depth of 15 feet. Fig. 13 shows the median diameter plot of these samples.

Prior to the groin construction, surveys indicated that only fine and very fine sands were present. It will now be noted that the groins have sorted and held inshore those medium gravels of 1.00 mm or above in median diameter with a narrow band of coarse and medium sand offshore of the gravel, and fine sand adjacent to the coarse and medium sand. Very fine sand is still present further offshore outside of the action of the groins.

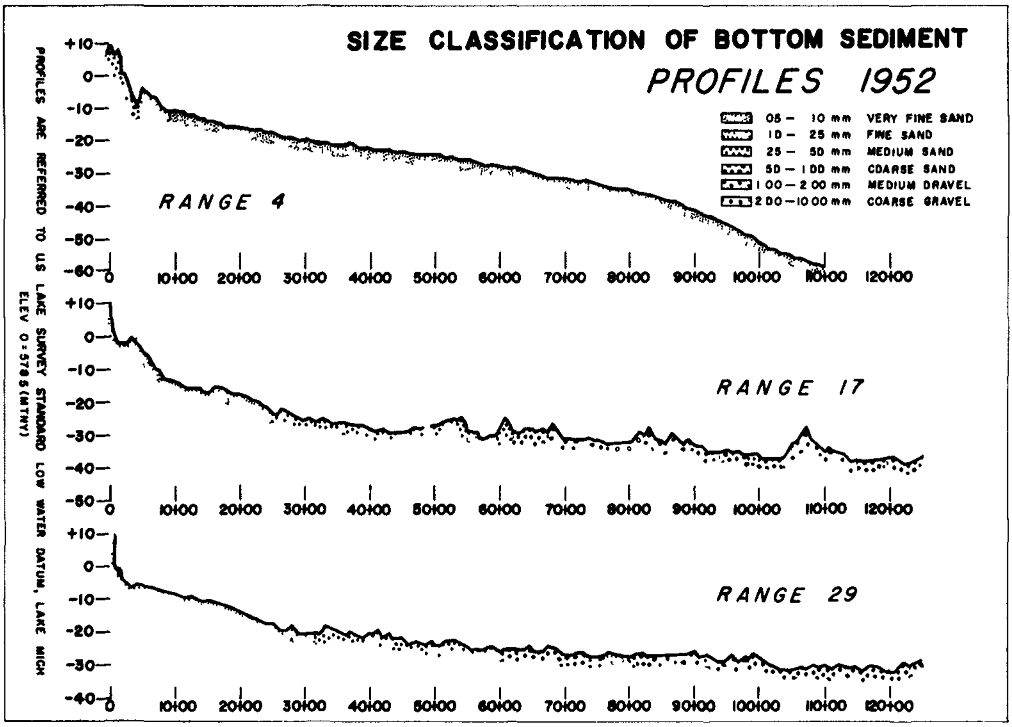
SIZE CLASSIFICATION OF BOTTOM SEDIMENT
PROFILES 1952
05 – 10 mm VERY FINE SAND
10 – 25 mm FINE SAND
25 – 50 mm MEDIUM SAND
50 – 100 mm COARSE SAND
100 – 200 mm MEDIUM GRAVEL
200 – 1000 mm COARSE GRAVEL
RANGE 4
RANGE 17
RANGE 29
PROFILES ARE REFERRED TO US LAKE SURVEY STANDARD LOW WATER DATUM, LAKE MICH
+10
0
-10
-20
-30
-40
-50
-60
0
10+00
20+00
30+00
40+00
50+00
60+00
70+00
80+00
90+00
100+00
110+00
120+00

Fig. 12.

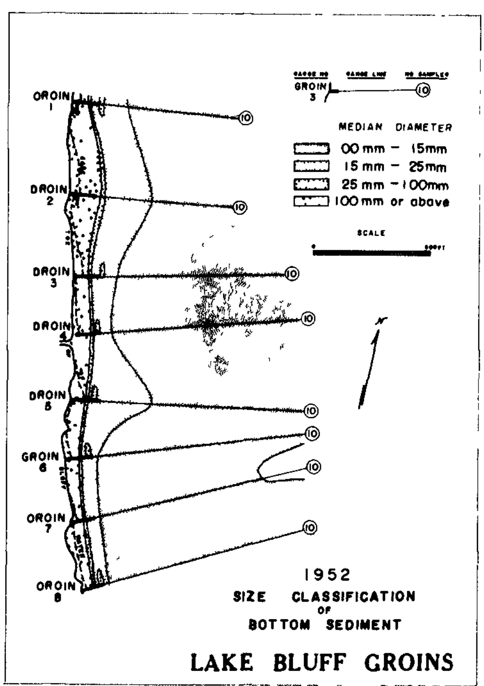
GROIN 1
GROIN 2
GROIN 3
GROIN 4
GROIN 5
GROIN 6
GROIN 7
GROIN 8
MEDIAN DIAMETER
00 mm – 15mm
15 mm – 25mm
25 mm – 100mm
100 mm or above
SCALE
1952
SIZE CLASSIFICATION OF BOTTOM SEDIMENT
LAKE BLUFF GROINS

Fig. 13.

The work which has been done to date on bottom sediment characteristics has been confined to the sounding, sampling, and analysis mentioned, plus limited studies of results. At present, it appears desirable to continue the present survey program, with such modification as may be indicated from time to time, and in addition it seems desirable to make a determination of offshore sources of beach material as to areal extent and depth of various materials.

Some of the things which may be determined by study of the surveys are:

1. The behaviour of bottom sediments when acted upon by the various physical forces.

2. The effects of the characteristics of bottom materials upon the stability of beaches.

3. A more intelligent design of shore protection structures and protective beaches.

4. The availability of materials for maintaining or replenishing beaches.

CHAPTER 8

THE MOTION OF SEDIMENT ALONG THE SOUTH SHORE OF LAKE ERIE

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INTRODUCTION

Much of Lake Erie's southern shoreline displays fairly uniform properties with respect to shore processes. However, detailed studies of selected strips of shore areas often reveal characteristics which are so distinctive that problems of control require special attention to local characteristics.

The purpose of this paper is to present some generalizations and some detailed comments on the motion of sediment along the south shore of Lake Erie, to outline the results of some detailed studies of small areas, and to evaluate the types of evidence used in such studies.

The term "motion of sediment", can be assigned several distinctive meanings, each meaning being fixed by the type of evidence used in inferring motion and, where possible, direction and rate. Certainly, "motion of sediment" has one meaning when inferred from long period accretion patterns, another meaning when inferred from systematic variations along the shoreline in grain size or mineralogy, and still another meaning when inferred from observations of beach drifting for a few hours. The phrase, "along the shore", as used in this context, is taken to mean any (inferred) motion, which has a longshore component, whether that motion applies to an individual particle or to groups of particles, to net displacement or to total displacement, for any specified time interval, within any specified area. Obviously, understanding of movement along the shore requires also consideration of normal components, therefore one considers also motion into and out of offshore areas, and landward movement of barrier beaches.

The effectiveness of ice as a longshore transporting agent is not here considered. On the basis of observations of ice action in several of the study areas, it is not believed that a serious error is committed by allowing this omission here.

Wind may be an effective transporting agent locally, but its total importance as such is believed to be insignificant.

In fact, wind or ice may obliterate or distort evidences of movement by water.

Comparisons of inferences drawn from several sources point up the possible differences in conclusions based upon evidence from the several sources.

THE PHYSICAL SETTING

BEDROCK GEOLOGY

From Sandusky eastward, the southern shore of Lake Erie is underlain by non-resistant rocks, mostly shales, of Devonian age; west of Sandusky, the shoreline is underlain by resistant limestones and dolomites of Silurian age (Fig. 1,2).

The lake is shallower in the western end than it is in the central and eastern portions, possibly because of differences in the resistance of the bedrock to glacial scour. The easternmost part of the basin is also the deepest, possibly because here the non-resistant rocks dip more steeply than the equivalent rocks underlying the central basin to the west; this steeper dip would have presented a greater vertical thickness of weak rock for excavation by the southwestward moving ice tongue (Carman, 1946).

Only a small fraction of the shoreline is bedrock. Bedrock is exposed along 34 miles of Ohio's 184 miles of shoreline; resistant rocks account for 11 of the 34 miles (White and Gould, 1945).

Bedrock is exposed on the lake bottom in many places, especially in the western end, where the relatively resistant Silurian dolomites and limestones appear at the surface at Catawba, Marblehead, and on the islands. So-called "reefs", which provide excellent sport fishing, are lake bottom exposures of these rocks. Shales are frequently exposed farther east, but the relief of these exposures is quite low.

The types of bedrock exposed along the shore and underwater contain, in general, very little material in the beach-building size range.

UNCONSOLIDATED SHORE MATERIALS

While the Lake Erie basin appears to have been the site of excavation by ice, many of the features of the southern shore, especially in the central and western portions, are glacial or glacial lake deposits.

Boulder Clays

Much of the bluff material is unstratified glacial drift, usually a tough boulder clay, containing mixtures of local and foreign debris. A large variety of rock types is represented in the larger fragments of these till mixtures, accounting, at least in part, for the heterogeneous assemblages of minerals commonly found in beach deposits.

The fresh boulder clay, typically blue to gray, has been oxidized to a yellowish color from the top to depths exceeding ten feet, in some areas. Possibly ten percent of the boulder clay is potential beach material, therefore considerable erosion of the bluffs is required to produce appreciable amounts of beach-building sediments. The 40-65' high bluffs from Cleveland to the Pennsylvania border are cited here as an example of the boulder clay deposits in Ohio.

The till bluffs are attacked by waves, slaking, frost action, rain wash, ice push, and sapping.

Glacial Lake Deposits

Some of the lakes preceding Lake Erie have left their mark in the form of lake bottom deposits and ancient beach ridges.

The lake bottom deposits are often found lying upon the boulder clay. In other places, where the shore relief is low, only the old lake deposits are under wave attack.

Such a stretch of low shoreline runs along the western part of the lake, in Lucas and Ottawa Counties, Ohio. Here, some areas are so low that marshes front on the lake, with barrier beaches intermittently marking their lakeward edges.

These deposits, typically silty clays, and usually patchy blue-gray and brown, contain so little sand size material that their contribution to the supply of beach material can be disregarded.

Old beach ridges mark the margins of earlier higher lakes formed as the water level rose and fell with changes in the lake's outlet during its complex history. Some of their sandy constituents may be contributed to the lake by streams cutting through the ridges, and, in time, by wave attack. The total contribution of such materials at the present time is believed to be very small (Kleinhampl, 1952).

EXAMPLES OF SHORE MATERIALS

Some of the types of shore materials observed are illustrated in Figs. 4, 8, 9, 11, 15, 17, and 19.

TOPOGRAPHY

The southern shore of the lake, for the great part of its length, lies between flat surfaces: landward, glacial deposits and the beds of the higher predecessors of the modern lake present a smooth topography, usually sloping gently to the north; lakeward, the bottom generally drops very gradually toward the lake's center, rising again gradually toward the Canadian side. The bottom relief is very gentle, regardless of the type of bottom material; bedrock bottoms are occasionally slightly irregular.

In almost any profile cutting the landward plains and the lake bottom, the zone of maximum relief is the narrow shore area, in which the bluffs join the landward and lakeward surfaces. Changes in relief along the shoreline are usually very gradual.

The shoreline displays some conspicuous signs of submergence, as for example, the flooded embayment at Sandusky. Moore (1948) has presented quantitative evidence indicating submergence of Lake Erie's shores with respect to the lake's outlet.

Some areas, particularly those west of Port Clinton, where the bottoms of older, higher lakes now make up the shore, show the effects of recent submergence superimposed upon the earlier emergence.

Aspects of the topography well worth noting are the orientation and regularity of the shoreline. The orientation is particularly important when considered in connection with wind data and directions of large fetch. At Ceylon Junction, approximately 12 miles southeast along the shore from the Cedar Point jetty, the general trend of the shore changes quite markedly: to the west, the shore trends generally northwest, and to the east, it trends generally northeast.

Irregularities in the shore arise from such factors as the submergence mentioned earlier, exposure of relatively resistant rock, as at Catawba, and large-scale deposition, as at Presque Isle Peninsula, Erie, Pennsylvania.

DRAINAGE

Most of the water in Lake Erie comes from the upper lakes, through the Detroit River; only 12% of the water comes from the lake's drainage basin (Moore, 1946).

Apparently, tributary streams contribute little beach material. Some of the streams transporting sand size materials are used for commercial navigation; harbor areas are generally enclosed in such a way that upstream materials settle out in the harbor basin, not reaching the lake proper. The dredging of channels may place some of these materials into the general longshore circulation.

UNCONSOLIDATED BOTTOM DEPOSITS

Most of the lake areas studied to date are floored with clays and silty clays much like those found in the bottom deposits of the older, higher lakes. Many of the deposits in deeper water have apparently settled out of suspension very slowly, forming quite uniform layers.

Large areas of sand and gravel on the lake bottom are not common. The large Lorain-Vermilion sand and gravel area (7x10 miles) lying 6 miles and more northwest of Lorain, consists of relatively poorly sorted material, probably of glacial origin. Probing operations by the Lake Erie Geological Research Program have revealed thicknesses of 8 and 10 feet in some places. This deposit does not appear to be directly connected in any way with shoreline sand deposits in the Lorain and Vermilion areas. Exploration as far north as the International Boundary has revealed similar coarse materials, but the detailed distribution on the lake bottom is not yet known.

Other large sand deposits, like those in outer Sandusky Bay and off the west Fairport breakwater (4x10 miles), are well sorted and apparently continuous with nearshore and shore deposits. The outer Sandusky Bay deposits have been probed to depths of over 20 feet by the Lake Erie Geological Research Program.

Peaty and mucky bottom deposits often lie offshore from marshy areas. Off Magee Marsh, 18 miles east of Toledo, the barrier beach is migrating landward, exposing old marsh deposits on its lakeward side. Such deposits provide no beach building materials, except, perhaps, for small accumulations of sand blown or washed into the ancient marsh in which the materials were formed.

METEOROLOGICAL FACTORS

Southwest winds prevail in the Lake Erie Basin. However, there are sufficient west, northwest and northeast storms to utilize large fetches, thereby producing waves (Saville, 1953) capable of causing much shore damage and longshore, lakeward, and landward transportation of sediment.

Transient changes in lake level produced by pressure gradients or wind stress frequently bring vulnerable shore materials under the attack of large waves, operating over an offshore profile which dissipates very little of the wave energy.

The seasonal variation in lake levels is usually between one and two feet, the highest and lowest levels occuring in June and February, respectively. The average level for the period 1860-1952 is 572.31 feet above mean tide at New York City. The difference between the highest one-month average (574.60', April 1952) and the lowest one-month average (569.43', February 1936) is 5.17' (Saville, 1953).

GENERAL COMMENTS ON SHORE PROCESSES

The combination of wind directions, available fetches, and shoreline trends apparently add up to net generalized littoral drifts which flow to the west and to the east for given wind directions from neutral points whose positions vary with the wind directions. Neutral points have been reported (House Doc. No. 32, 83d,, Congress, 1st Session, 1953; House Doc. No. 220, 79th Cong., 1st Session, 1946) at, or in the general vicinity of the previously mentioned break in the main trends of the shoreline, viz., Ceylon Junction.

Shore materials, whether consolidated or unconsolidated, are generally very poor sources of potential beach materials. The boulder clays are usually the most prolific sources, but they are usually made up of no more than 10% of potential beach materials. Occasionally, as in the Cedar Point area (Metter 1952), beach materials may be exposed on the lake bottom as an active modern beach moves landward; or, suitable materials from older lake deposits may be exposed in shore bluffs, providing localized shoreline sources of material (Kleinhampl, 1952).

Large offshore deposits apparently must be ruled out as sources of beach materials because there is little evidence to indicate that they are contributing appreciably to nearshore and shoreline deposits.

The generalized picture of littoral drift is based upon availability of energy from different sources (Saville, 1953), and upon patterns of accretion (usually with respect to structures) and erosion, recorded over periods large enough to minimize the effects of seasonal and apparently random fluctuations.

COMMENTS ON SELECTED STRIPS OF SHORELINE

INTRODUCTION

Brief discussions follow of some of the significant features of selected areas studied by the personnel of the Lake Erie Geological Research Program. These discussions are set in the larger framework of the very competent reports prepared by the U. S. Army Corps of Engineers for publication as Congressional documents on the subject of beach erosion control.

SELECTED AREAS

Magee Marsh, Eastern Lucas County, Ohio (Fig. 2)

A westward littoral drift along the barrier beach fronting Magee Marsh (State property 18 miles east of Toledo) is reported by Savoy (1953, p. 26). The evidence upon which this statement is based is the westward migration of the mouths of two creeks flanking the study area, accretion on the eastern side of structures, and a pattern of bay filling.

Some accretion patterns have indicated movement from the west, but these have been only short period reversals of the dominant trend.

The source of beach materials is apparently the line of till bluffs to the southeast. Littoral drift may actually be depleting the Magee area, and depositing material to the northwest against structures on private property.

The barrier beach is migrating slowly landward, exposing peaty and mucky deposits in various places offshore.

The sand stream moving northwest is very lean, possibly because of damming by several structures to the southeast.

Savoy's determinations of drift are in general agreement with those reported in House Document No. 177, 79th Congress, 1st Session (1945). This document, which also presents additional details on drift between the Ohio-Michigan line and Marblehead, Ohio, utilizes patterns of accretion as the principal evidence for detecting littoral drift. Somewhat east of Magee Marsh, slight eastward drifts are indicated, but the movement here is possibly better classified as "variable". Movement of dunes is reported to be insignificant in this area.

Lee's (1953, Fig. 1) map shows a southeastward drift in the area just northwest of Magee Marsh; this apparently conflicts with the information given in the preceding paragraphs.

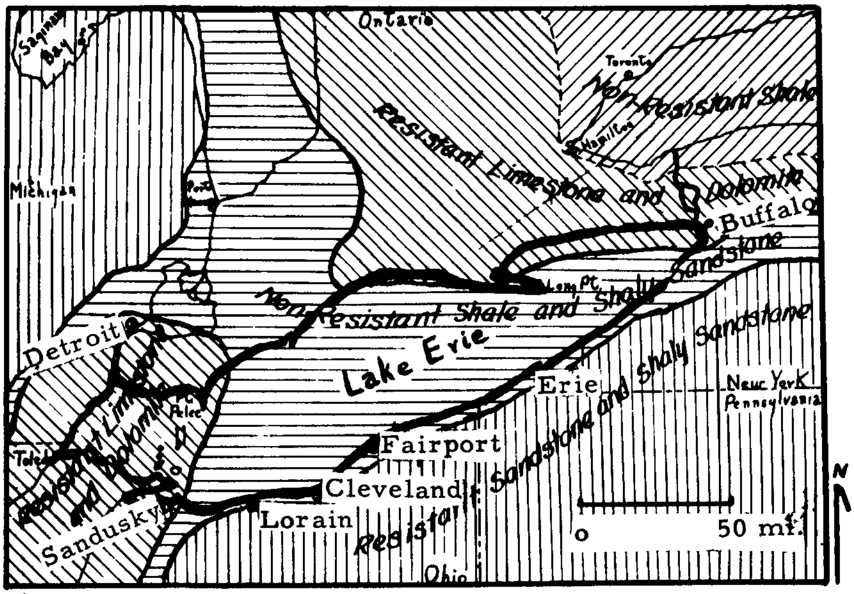

Fig. 1. Map of Lake Erie region, showing the distribution of resistant and non-resistant types of bedrock. From Carman (1946).

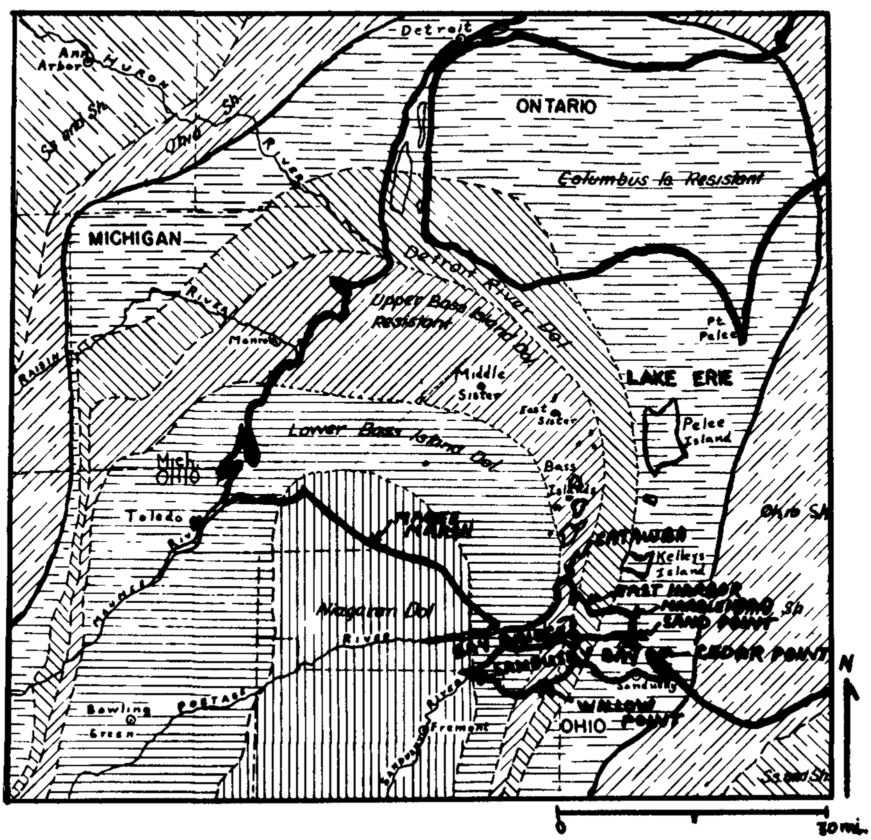

Fig. 2. Geologic map of the western end of Lake Erie, showing the arcuate pattern of the rocks. Study areas are shown in heavily inked letters. Geology from Carman (1946).

East Harbor Beach, Ottawa County, Ohio (Fig. 2)

Mechanical, mineralogical, and carbonate analyses of samples collected on a 1950 reconnaissance of the East Harbor Beach fail to show significant systematic variations which might yield some inferences about motion of sediment along this strip (Pincus, Roseboom and Humphris, 1951).

The Congressional document (1945) previously cited notes that just northwest of the East Harbor Beach, motion toward the southeast is indicated by the drifting of sand southeast across the mouth of West Harbor; east of West Harbor, indications of movement are nil. This report's historical work shows that progression of the shoreline in this area has been accompanied by a general deepening in offshore areas.

Although some of the dunes in this area are over 10' high, abundant plant cover has apparently reduced dune drift to zero.

Thus, the available data on longshore drift in this area, as derived from two types of evidence, i.e., accretion patterns and sedimentary analyses, yield equally inconclusive results. This is not equivalent to saying that the two types of evidence support each other.

Marblehead to Sand Point to Bay Bridge, Northern Sandusky Bay, Ottawa County, Ohio (Fig. 2,3)

Sand Point is a compound spit, growing southeastward (Humphris, 1952). Beach ridges along the eastern shore, near the southern tip of the spit, attest to the persistence of this growth pattern for some time. From Marblehead Lighthouse to the base of the spit, the shore is divided into three parts: 1. the northern third consists of bedrock (Columbus limestone) and limestone cobble beaches; 2. the central third has been covered with protective structures; 3. the southern third consists of till, with some limestone cobble and pebble beaches. Most of the north shore of Sandusky Bay, lying west of the spit, consists of lake clay lying on top of boulder clay. The tills in Ottawa County may contain as much as 20% sand size particles.

Humphris (1952) reports that longshore currents move south along the east shore of Sand Point, north along the west shore, and east along the north shore of Sandusky Bay, from Bay Bridge to the spit. These statements are based upon observations of accretion patterns and of the development of minor beach features, and upon consideration of wind-fetch relationships.

Systematic variations in mechanical and mineralogical characteristics and in carbonate content with respect to location along the spit have not been recognized (Pincus, Roseboom, and Humphris, 1951; Humphris, 1952), although carbonate content and certain mineral abundances appear to vary systematically with grain size. The sediments on Sand Point are finer grained and better sorted than those to the west, along the north shore of the bay. Humphris (1952) attributes this contrast to derivation of Sand Point materials from Sandusky Bay, and north shore materials from erosion of the local shore. He points out that the median size of material found along Sand Point is that size most easily moved by wave and current action, according to Inman (1949).

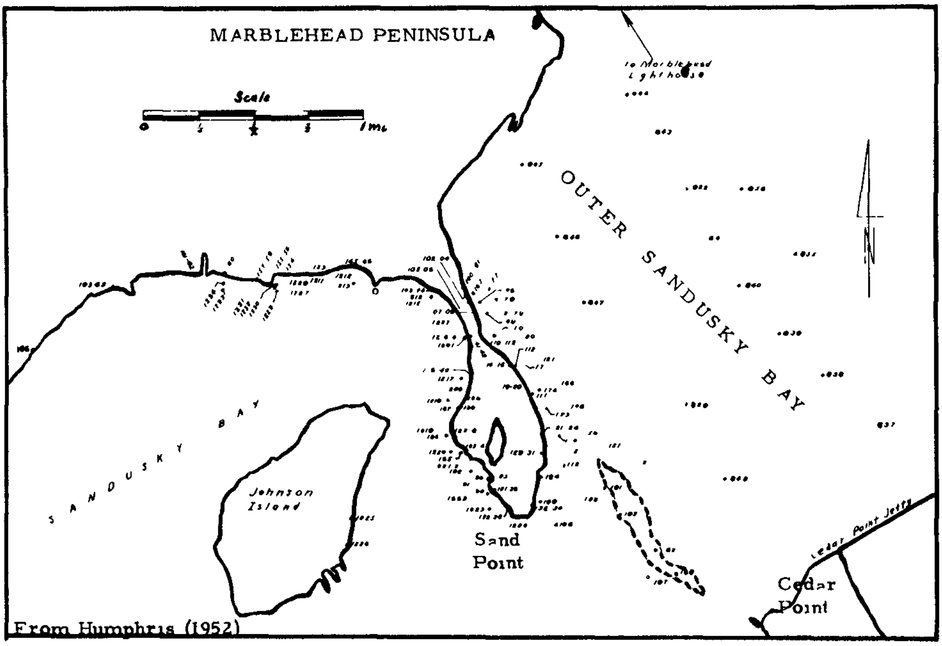

Fig. 3. Map of Marblehead Peninsula, Sand Point, and outer Sandusky Bay. Note location of Cedar Point jetty in lower right hand corner. Small numbers designate study localities, not discussed in this paper.

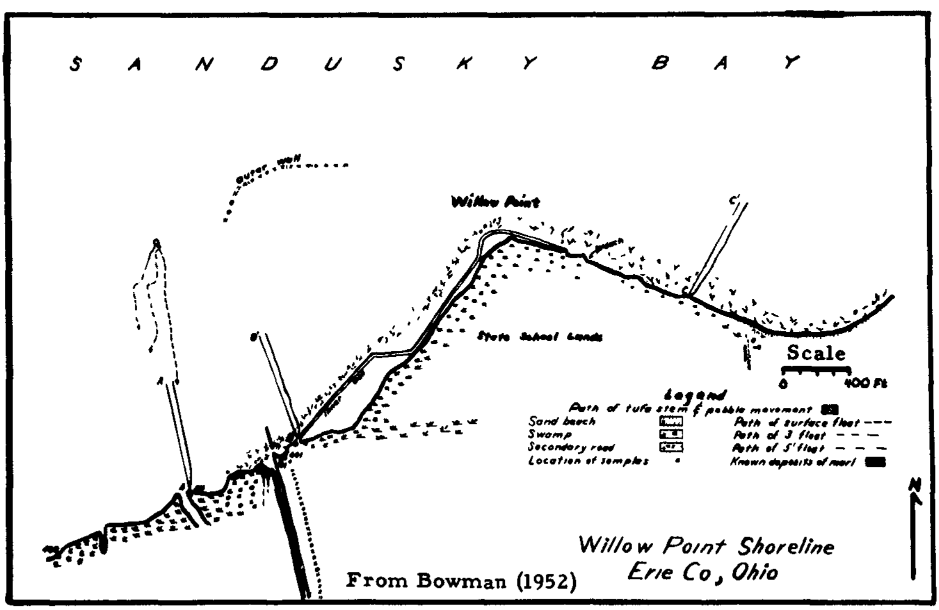

Fig. 4. Map of Willow Point area. Little Pickerel Creek runs just west of dirt road in lower left of map. Vertical exaggeration of profiles AA', BB' and CC' is 10X.

Shale fragments, $\frac{1}{4}$-1" in diameter, occur in great quantities along the southern and western shores of the spit. Shale fragments have not been observed in any nearby till exposures, and the bedrock is not shale (Fig. 2). The nearest known exposure of shale along the shore is about 9 miles east of Sand Point (Metter, 1952). Humphris (1952, p. 25) suggests that perhaps the shale fragments have been swept westward past Cedar Point, around the pierhead light, then to the southwest past the tip of the spit, and up its western side. House Document No. 32, 83d Congress, 1st Session (1953), p. 23, par. 59 reports a strong reversing current in the channel running northeast-southwest between the lake and the inner bay. The currents are attributed to changes in lake levels arising from wind action; velocities of over three miles per hour are attained, reducing channel maintenance considerably. Dominant longshore drift from the supposed source of the shale to Cedar Point is westward; the number of shale pebbles on the beach increases toward the east (Metter, 1952).

Outer Sandusky Bay, between Marblehead and Cedar Point (Fig. 3)

Sedimentary processes in outer Sandusky Bay appear to be directly related to those involving the eastern side of Sand Point. Grain size becomes smaller in an apparently continuous series from the spit out into deeper water (Pincus, Roseboom, and Humphris, 1951; Humphris, 1952). Trends of median grain size run parallel to the shore in the Marblehead area.

The sand on the bottom is more than 20' thick in some places, thinning toward the northwest. Beneath the sand lies a gray clay, lying, in turn, upon bedrock.

Data derived from mineralogical, mechanical, and carbonate analyses do not seem to throw any light upon the mechanisms of sediment transport.

An observation by the author in the autumn of 1950 may have some bearing upon this problem. The Program's research vessel was anchored west of the channel in the outer bay, near the pierhead light, with fairly high waves coming in from the northeast. Although the northeast winds blew the vessel southwest of the anchor, the vessel moved into the wind, drawn by the taut anchor line. The author interpreted this as the result of northeastward creeping of the bottom sediments, possibly in conjunction with a returning hydraulic current. However, coring operations during the following summer revealed that the bottom was covered with a crust which might inhibit or prevent such creeping; operators of sand-suckers have sadly confirmed the existence of this crust.

Some sand undoubtedly passes westward through or over the Cedar Point jetty; other materials probably are swept around the jetty and into the bay (Humphris, 1952). Metter (1952) reports an increase in grain size near the end of the jetty on the east side, apparently reflecting the effects of strong, observed currents entering or leaving Sandusky Bay (House Doc. No. 32, 1953). Contributions from the inner bay are probably minor.

Willow Point, South Shore of Sandusky Bay (Fig. 2,4)

Bowman (1952) has reported a net eastward drift along the shore in the vicinity of Willow Point on the south shore of Lake Erie. He cites the conspicuous eastward decrease in abundance of tufa pebbles washed into the bay from Little Pickerel Creek. Tufa stems are present in the west bank of the creek and in the shoreline property immediately to the west of the creek (Fig. 5). The band of tufa widens past Willow Point, possibly because the current continues eastward while waves spread the deposit southward toward the shore, which swings to the southeast east of the point (Fig. 4). Variations in grain size of the tufa material on either side of Little Pickerel Creek are very strikingly illustrated in Fig. 6. In this area it is possible to outline a path of sediment drift with considerable confidence, because of the apparently unique source of easily recognizable materials. In particular, the tufa pebbles, which seem to come from the creek alone, are especially reliable tracers.

In a survey of the shoreline to the east of the school lands, Pincus (1953) observed that both the amount of tufa and the particle size of beach materials diminished toward the east.

Beach materials in this area are continually being driven landward into the marshy areas so persistent along this part of the shoreline. Bowman (1951) has shown that variations in bay level, which depend upon wind direction, determine which layer of shore material is attacked by waves (Fig. 5). A layer of marl is particularly susceptible to such attack. Thus, water level affects the type of material introduced into the longshore currents. On the north shore of the bay, a glass wool plant had been dumping waste silicate beads and splinters for at least five years preceding Bowman's study. Bowman found these materials in every beach sample he collected along the south shore of the bay. Particle sizes of the artificial materials range from 4.0 to .064 mm; hardness and specific gravity are slightly less than that of quartz. The greatest inferred straight-line distance of cross-bay transportation is 5.9 miles. The actual paths of these materials are not known. Most of the bay in this area is less than 6' deep, and the bottom is largely composed of very fine materials.

Both the tufa and the artificial grains provide excellent means for tracing movement, in that distinctive materials are supplied at a "point source", or an approximation thereto. These observations have led to a program of developmental work in using "tracer grains"; the principal unsolved problem to date is that of obtaining sufficient quantities of innocuous, inexpensive materials, with adequate physical and chemical properties.

Cedar Point to Huron, Erie County, Ohio (Fig. 7)

House Document No. 32, 83d Congress, 1st Session (1953), treats the 20-mile strip of shoreline from Cedar Point to Vermilion Harbor; the Cedar Point to Huron strip, discussed in this section, and studied by Metter (1952), is the western half of the 20-mile strip.

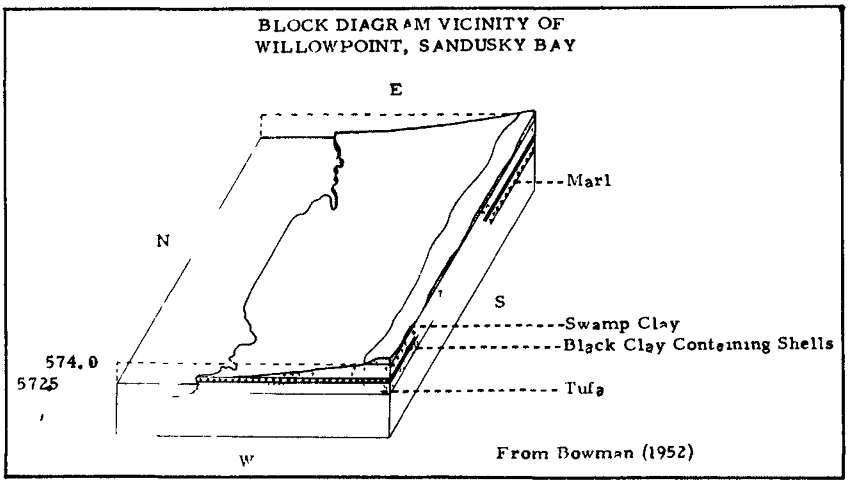

Fig. 5. Shore materials just west of Little Pickerel Creek . Note how small variations in water level bring different materials into zone of wave attack.

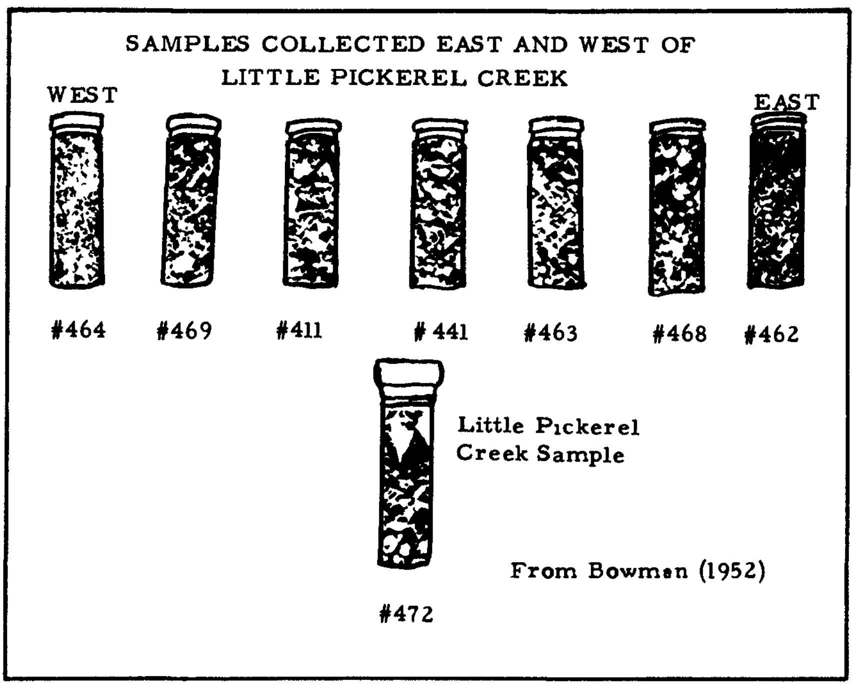

Fig. 6. Samples of tufa collected east and west of the mouth of Little Pickerel Creek. Note the decrease in size with increase in distance along the shore from the mouth.

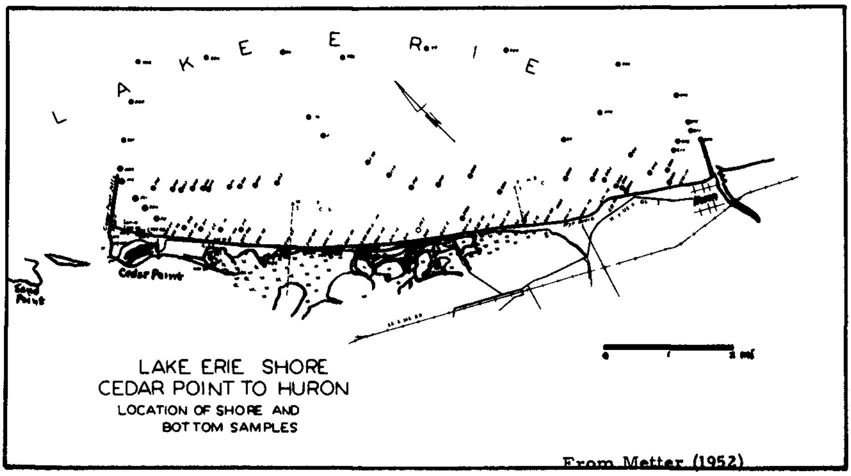

Fig. 7. Lake Erie shore from Cedar Point to Huron. Small numbers designate study localities, not discussed in this paper.

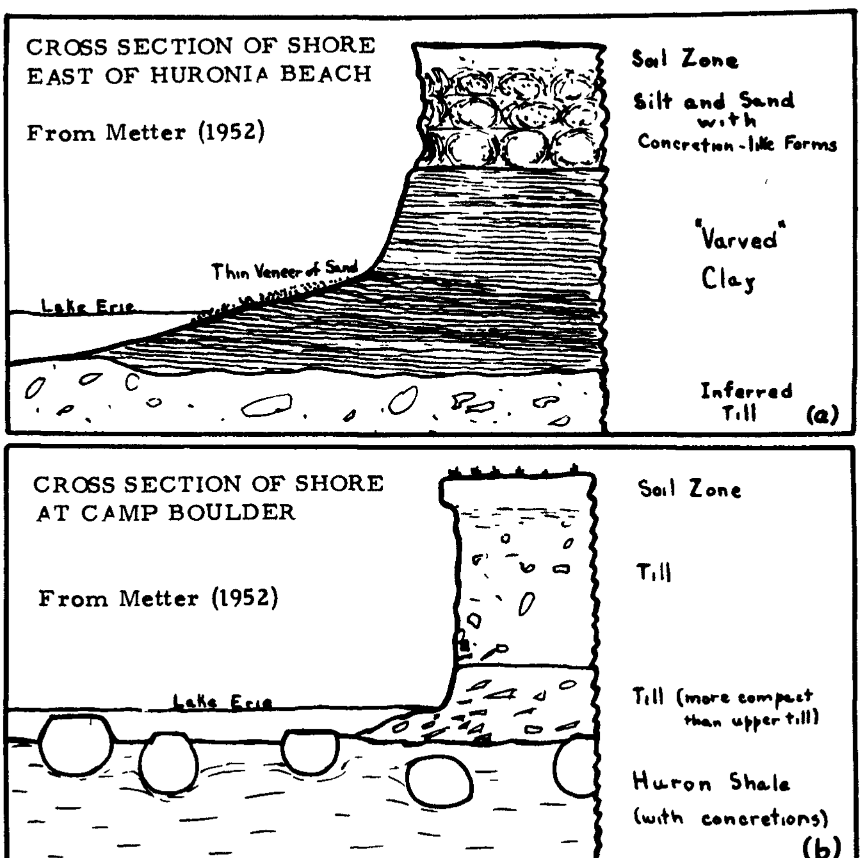

Fig. 8. Shore and bluff materials in the eastern part of the Cedar Point-Huron study area.

According to Document No. 32, longshore drift along this strip of coast is predominantly east to west. Accretion on the east side of the Cedar Point jetty and the east breakwater at Huron are cited as evidence for this statement. It is also pointed out that there is a westward movement of points east of these structures where wide beaches give way to beaches narrow enough to permit bluff erosion.

This document also carries information on littoral currents predicted from refraction diagrams for 4-second waves. While east and east-northeast storms generate east to west littoral currents throughout the area, and northwest storms generate west to east currents, northeast and north storms develop currents flowing away from each other at neutral points. The neutral point for northeast storms is about 3 miles east of the Cedar Point jetty; the neutral point for northern storms is Ceylon Junction, discussed earlier in this paper.

An earlier report, House Document No. 220, 79th Congress, 1st Session (1946), gives results of current observations with dyes and subsurface floats. These observations agree fairly well with the predictions of the more recent (1953) documents. Current velocities up to 60 ft/min. were observed during comparatively calm weather.

The 1946 document points out that approximately 5 miles east of Vermilion, there is a division between dominant east and west littoral drift, as evidenced by accretion patterns. West of this point, accretion is on the east side of structures; east of the point, accretion is on the west side of structures. Such a division point is not the same as the previously mentioned neutral point, for the latter is determined with respect to a specific wind direction. From the present compilation of data, and according to Lee's (1953) generalized map, the division point lies east of the point cited in the 1946 report. Possibly, the division point, or zone, lies at Lorain, or even eastward toward Avon Point.

According to the 1946 document, no littoral drift passes Huron Harbor. Thus, there are no natural outside sources of material in this area. To the west of Huron, erosion is indicated by changes in the shoreline and 6,12, and 18 foot depth contours, until 6000 feet east of the jetty; the shoreline and the 18 foot contours show accretion from this point to the jetty, while the 6 and 12 foot contours indicate erosion.

Metter's (1952) study shows a net westward drift at the western end of the spit, based on the accretion east of the Cedar Point jetty, and upon the fact that the Cedar Point spit was formed long before the construction of the jetty. His observations and inferences on sediment drift agree fairly well with those of the 1953 document.

Metter reports that a series of short groins about 6 miles east of the jetty have not collected much sediment. At the western end of this series, sand has accumulated on the western side; at the eastern end, sand has accumulated on the eastern side. Interestingly, this area lies between the neutral points for north and northeast storms, as described in the 1953 document. This groin series lies very near the neutral point for the northeast storms, which are far more severe than any other storms.

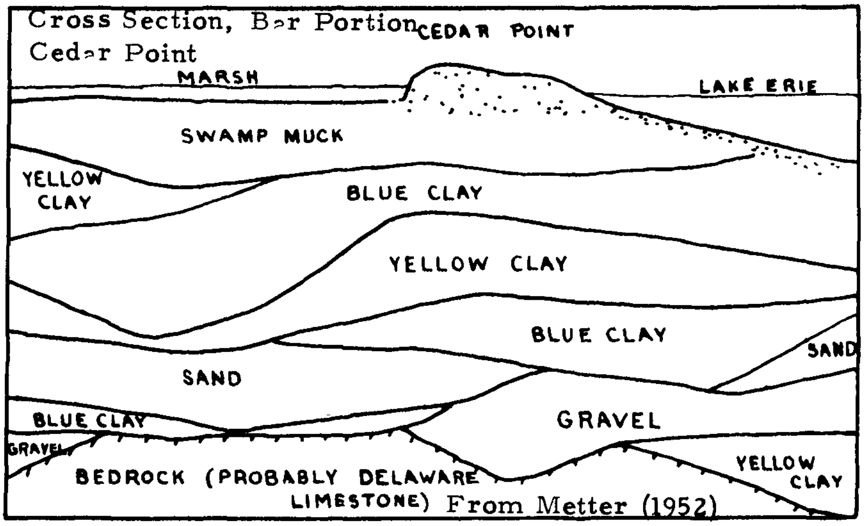

Fig. 9. Shore and subsurface materials in central part of Cedar Point-Huron study area. Subsurface structure inferred from borings by Sandusky City Engineer.

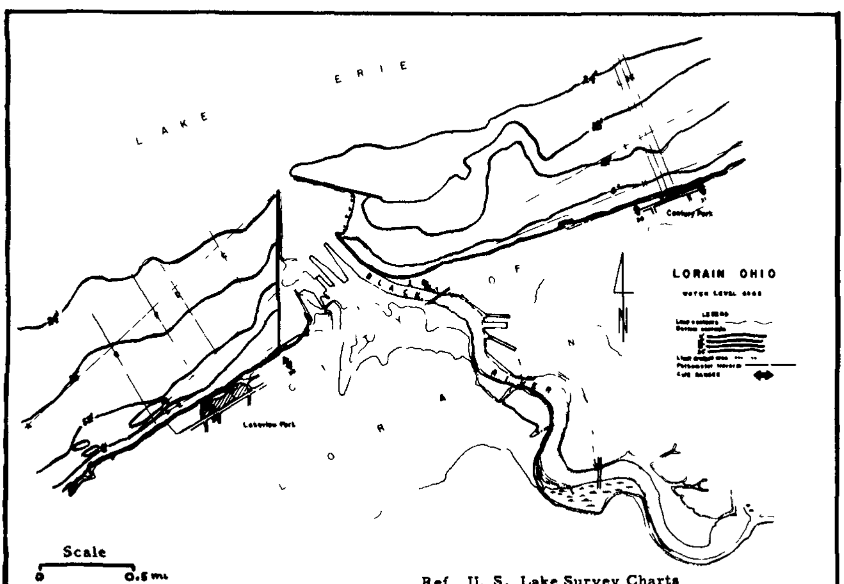

Fig. 10. Map of Lorain, Ohio, showing location of Lakeview and Century Parks. Note the apparent effect of the harbor breakwaters on bottom topography.

The previously mentioned eastward increase in the number of black shale pebbles is to be expected, if their source were the shale outcrops 9 miles east of the jetty.

Near the center of Metter's study area, the sediment near shore is coarser than to the east or west; he attributes this to the erosion of coarse material from the lake bottom in this area. Sand and gravel 15-20 feet below the central part of the bar portion of the Cedar Point peninsula could provide coarse materials on the lake bottom, as the bar moves landward (Fig. 9).

Sorting of sediments along the water's edge increases with decreasing grain size. Dune materials become finer westward from the approximate center of the study area.

It is conceivable that the opposite offshore trends reported by Metter are caused, at least in part, by currents on opposite sides of a neutral point.

Abundance patterns of the heavy minerals simply and collectively suggest a central source someplace toward the center of Metter's study area; although these patterns are not too clear-cut in indicating an overall drift, there is the suggestion of more movement to the west than to the east.

In the postulated source area, the 1/8-1/4 mm. size grade contains as much as or more heavy minerals than the 1/8-1/16 mm. size grade; elsewhere the finer grade contains more heavy minerals. These relations are consistent with the hypothesis of transport away from the designated source area.

From the mineral assemblages observed in the beach materials, it appears likely that glacial till along the shores is probably an important source of sediments (Fig. 8). Although many of these minerals are considered to be easily weathered, fresh angular grains are usually observed in beach deposits. Metter proposes that continued abrasion in transport could account for this condition.

The possibility that some sources are on the lake bottom has been presented earlier.

The types of evidence used here seem to indicate multiple sources and fairly complex to-and-fro motion along the shore. Inferences drawn from the several types of evidence yield results which are fairly consistent with each other.

Lakeview and Century Parks, Lorain, Ohio (Fig. 10)

According to the U. S. Army Corps of Engineers report on the Vermilion to Sheffield Lake strip (1949), the predominant drift here is from east to west as indicated by accretion at Beaver Creek jetty (west of Lorain) and against small structures east of Lorain Harbor. Since the eastern fetch is much larger than the western fetch, this direction of drift is to be expected. This report notes an apparent local reversal of predominant drift immediately adjacent to and west of Lorain Harbor, inferred from accretion patterns. Also noted are these observations: 1. there has been little accretion adjacent to the many shore structures in the area; 2. the greatest loss has occurred immediately east of Lorain Harbor; and 3. east of Lorain there has been a net accretion offshore, possibly resulting from the eastward transportation of Black River silt.

Apparently beach material is derived through erosion of local materials, particularly the till bluffs (Fig. 11). Sea walls in this area cut down on the effectiveness of this source.

In a survey of Lakeview and Century Parks (Fig. 10), Pincus (1952-3) observed accretion on the eastern sides of Century Park structures (Fig.13), and, in 1952, accretion on the western sides of Lakeview Park (Fig. 12) structures. This agrees with patterns of bottom contours adjacent to the Lorain breakwater (Fig. 10). In 1953, however, accretion adjacent to the eastern structures at Lakeview had shifted to eastern sides. These eastern structures are longer than the structure to the west, and are closer to the Black River and its surrounding, large protective structures. The reason for this local shift in accretion is not clear, for during the same period there was an accumulation of material inside the permeable base of the west harbor breakwater.

Regarding mechanical, total heavy mineral, and carbonate analyses, Pincus (1952-3) finds no meaningful patterns. Abundance data for individual minerals are now being compiled. One interesting observation coming out of the analytical results is that in several of the areas bounded by two groins and the shoreline, the sedimentsin the center are finer than the materials around the inside of the area's edges. This might indicate an energy gradient toward the center of such systems, but what this means in terms of motion of sediment is not clear. Observations of currents with subsurface floats indicate complex water movements in the groin fields.

Thus, the only recognized patterns in the Pincus (1952-3) report i.e., accretion, agree generally with the observations of the Army report, but seem to indicate complications. Possibly the Lorain harbor structures are trapping sand moving alternately east and west, and are also reflecting significant amounts of wave energy.

Apparently, the Lorain-Vermilion sand and gravel deposit, 6 miles to the northwest, has no direct connection with the Lorain beach materials.

Avon Lake and Vicinity, Lorain and Cuyahoga Counties, Ohio (Fig. 14)

In a detailed study of the 9½ mile strip from Sheffield Lake (4 miles east of Lorain) to Huntington Park (west of Cleveland), Kleinhampl (1952) reports that most accretion,although not large, occurs on the west side of structures. This seems to indicate a reversal of the dominant direction of drift for Lorain as recorded by the 1948 report, and as recorded in this paper for areas west of Lorain. As mentioned earlier, it is possible that the division point, or zone, for littoral drift lies at Lorain or slightly eastward. Occasionally, there are accretions on the east and also equal accretions between two groins. Some of these apparently erratic patterns may be caused by reflecting surfaces. Newberry (1873) noted a

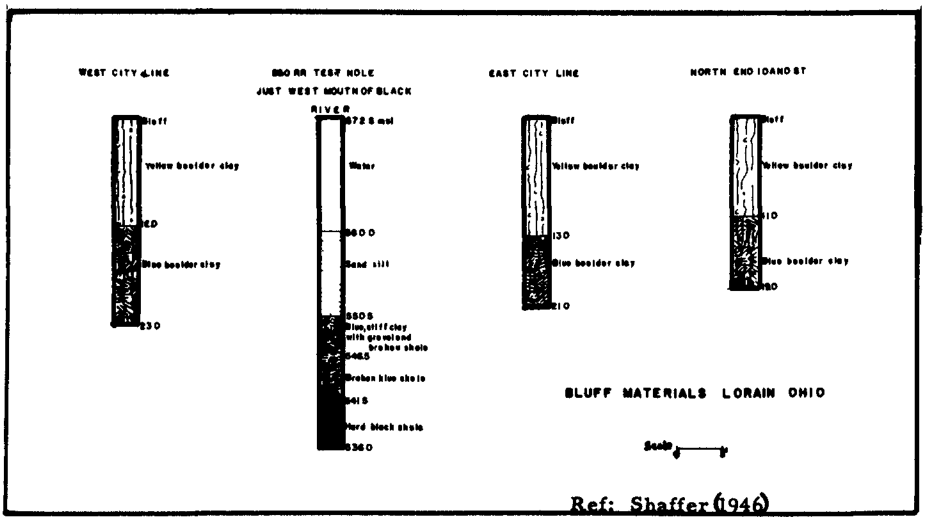

Fig. 11. Bluff materials along Lorain shore. Contrast with materials logged in test hole just west of the mouth of the Black River.

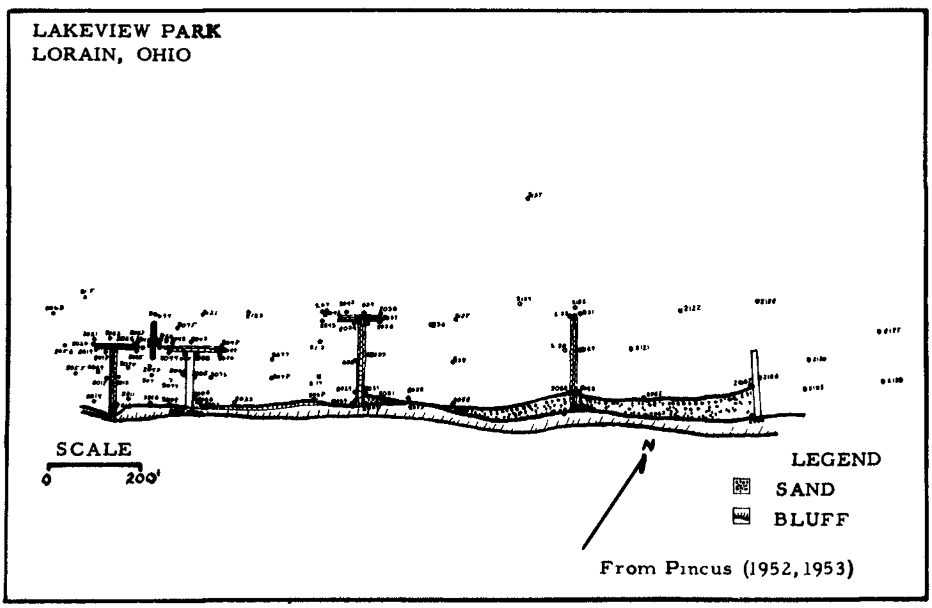

Fig. 12. Lakeview Park, Lorain, Ohio. Note areas of sand accretion. Small numbers apply to study localities, not discussed in this paper.

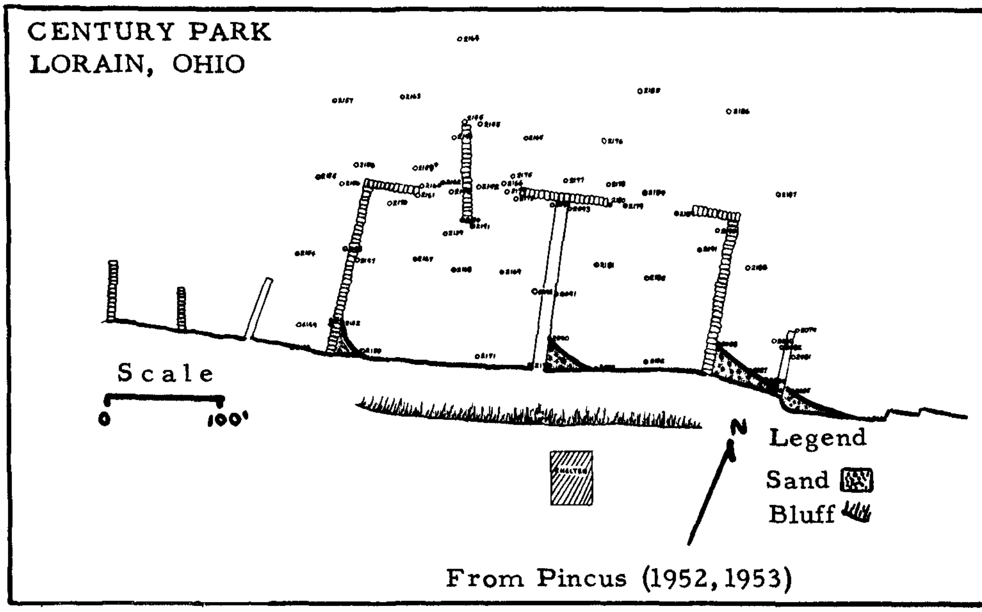

Fig. 13. Century Park, Lorain, Ohio. Note areas of sand accretion. Small numbers apply to study localities, not discussed in this paper.

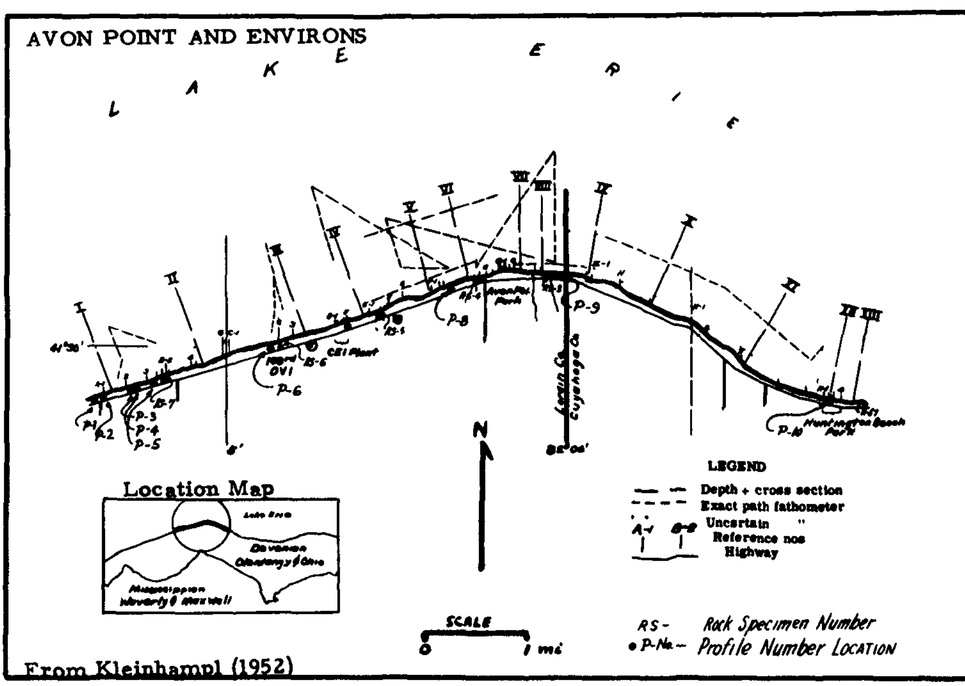

Fig. 14. Map of Avon Point and environs. Survey data in fine print are not discussed in this paper.

westward deflection of the turbid outflow of the Cuyahoga River; Kleinhampl (1952) reports that air photos show a westward deflection of Rocky River's outflow. However, such deflections could have been short-period events, of less long-time consequence as evidence of drift than the patterns of accretion cited above. Kleinhampl notes the association between the composition of the cliffs (Fig. 15) and the beach materials. The mineral composition and pebble counts of beach materials point particularly to a till source. The exposed shales and siltstones contribute shingle and fines. The largest single source of sand along the shore is believed to be the sandy bottom material of an older lake, just west of Huntington Park. Two creeks in the area cut through some old beach ridges; during heavy flow, they could carry gravel and sand into the lake.

Mineralogical and mechanical analyses reveal no systematic variations along the shore. Local supply appears to balance movement and thus to obscure evidence of movement in this area. Along the shore sorting is good, and median diameters approach that of Inman's "size most easily moved" (1949).

Some of the analytical values show conspicuous fluctuations slightly west of Avon Point. Possibly these fluctuations result from the action in this area of the many protective structures, some of which are very large, or from proximity to a point or zone of division of littoral drift.

House Document No. 502, 81st Congress, 2d Session (1950), reports on an 18-mile strip of shoreline from the west city line of Lakewood to the east city line of Cleveland. This area is almost adjacent to the eastern end of Kleinhampl's study area. This document reports a predominance of littoral current from west to east. Wind-fetch relations predict such currents, and patterns of accretion and changes in bottom contours attest to this type of movement.

Mouth of Chagrin River, Eastlake, Lake County, Ohio (Fig. 16)

In a reconnaissance report on shore processes in the immediate vicinity of the mouth of the Chagrin River, roughly midway between Cleveland and Fairport, Pincus and Hartley (1953) report littoral drift from the west. They cite as evidence the accretion on the west side of a large water intake structure to the west, and on the west side of the groin at the mouth of the river. Opposing weaker drift is indicated by the westward growth of small spits on the east side of the mouth of the Chagrin River.

Beach materials are apparently derived from the pebbly tills in the high bluffs; the lower-lying pebble-free clays and silty clays (which could be the remains of sediment laid down during an invasion of the river valley by an older, higher lake) are not effective sources of beach materials. Some patches of sandy materials on top of the high bluffs could contribute only very small total quantities to the longshore drift (Fig. 16,17).

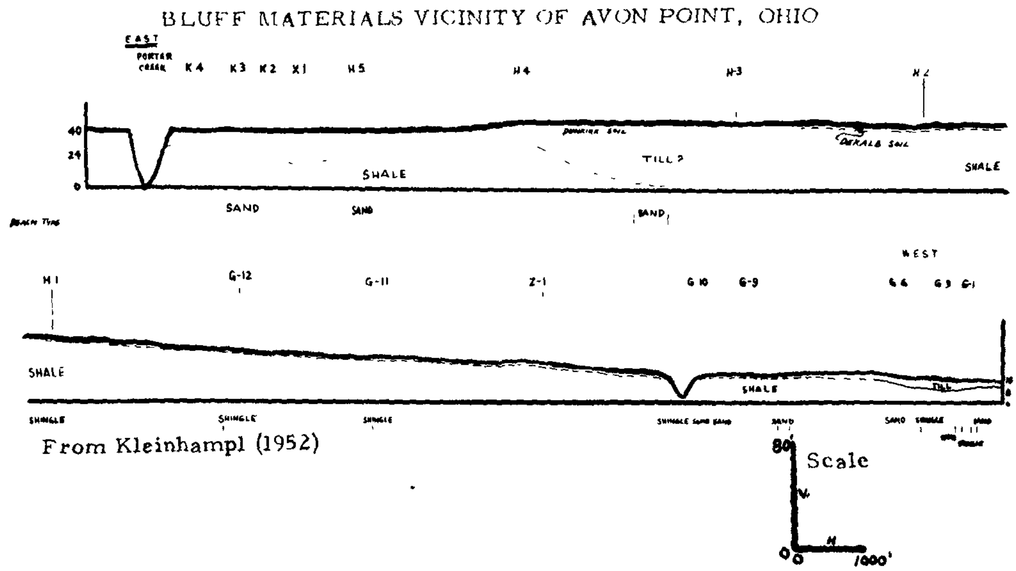

Fig. 15. Shore and bluff materials in the vicinity of Avon Point, as viewed from offshore. Vertical exaggeration 20X. Note relief on the bedrock surface.

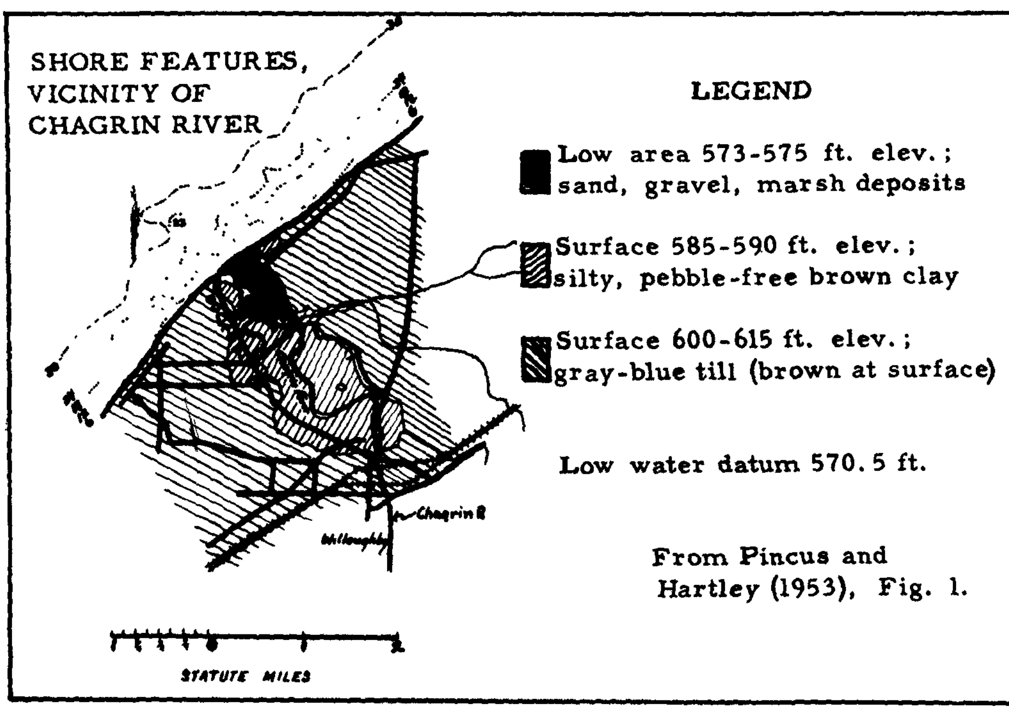

Fig. 16. Shore materials and offshore topography, mouth of Chagrin River.

The Chagrin River is not believed to be a significant source of beach materials, principally because it reaches lake level some distance south of the lake shore, depositing much of its load upstream. Observations of materials in the bed of the stream support this contention.

Some beach materials are migrating inland, over marsh deposits in old channels of the river, and into the base of the low bluffs of pebble-free clay and silty clay.

House Document No. 596, 81st Congress, 2d Session (1950), which treats the Lake County shoreline, reports littoral drift from the west. This drift is attributed to wind-fetch factors; accretion patterns confirm the analysis.

This document reports the occurrence of offshore bars off the mouth of the Chagrin River, and immediately west of the west breakwater off Fairport Harbor.

The Fairport commercial sand deposit, northwest of the harbor, is possibly connected with longshore sedimentation west of Fairport. These sands are fairly fine and fairly well sorted.

Madison Township Sewage Disposal Plant, Eastern End of Lake County, Ohio (Fig. 18)

Just offshore from the Madison Township Sewage Disposal Plant (15 miles east of Fairport) a thin veneer of sand (Fig. 19) trends roughly northeast from the small point of land protected by sheet steel piling (Pincus and Blackburn, 1953). In recent months, erosion has occurred on the west side of the point, and accretion has occurred on the east side. A small spit at the mouth of Arcola Creek was observed growing westward (Fig. 18, 19).

According to House Document No. 351, 82d Congress, 2d Session (1952), treating the shoreline between Fairport and Ashtabula, littoral currents are dominantly from the west, with temporary reversals, depending on the wind. The evidence for the statement is, once again, accretion patterns. The observations of Pincus and Blackburn (1953) appear to be in agreement with those reported by this document.

According to the same document, little material comes from outside the area (i.e., Fairport to Ashtabula) due to damming effects of harbor structures at each end. This implies that much of the beach materials have been confined to and reworked in this area, and therefore that processes of selection might have operated with considerable effect. This appears to be consistent with the observation (Pincus and Blackburn, 1953) that unusually large accumulations of heavy minerals occur here; there is no reason to suppose that the source areas are unusually rich in these minerals, although this is certainly possible.

House Document 350, 82d Congress, 2d Session (1952), treating the strip of shore from Ashtabula to the Pennsylvania state line, reports predominant currents from the west, with temporary reversals. This statement is based on consideration of wind and fetch, and upon observed

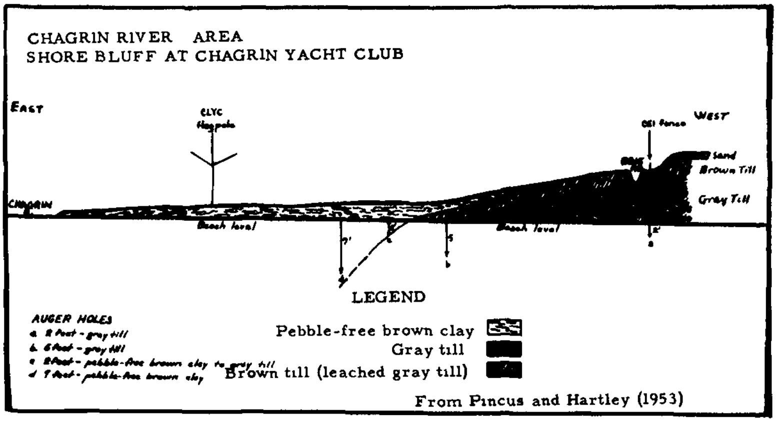

Fig. 17. View from offshore of shore and bluff materials just west of the mouth of the Chagrin River.

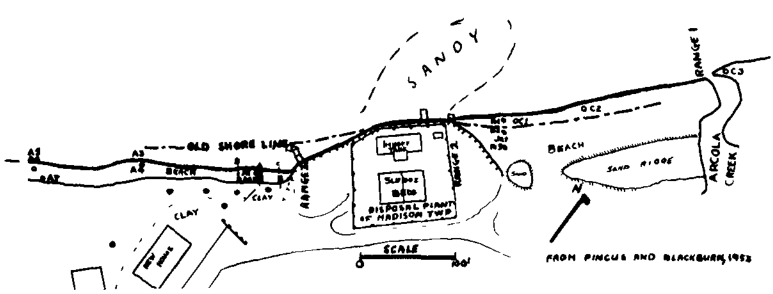

Fig. 18. Madison Township Sewage Disposal Plant at eastern end of Lake County. Note "old shoreline" and split at mouth of Arcola Creek.

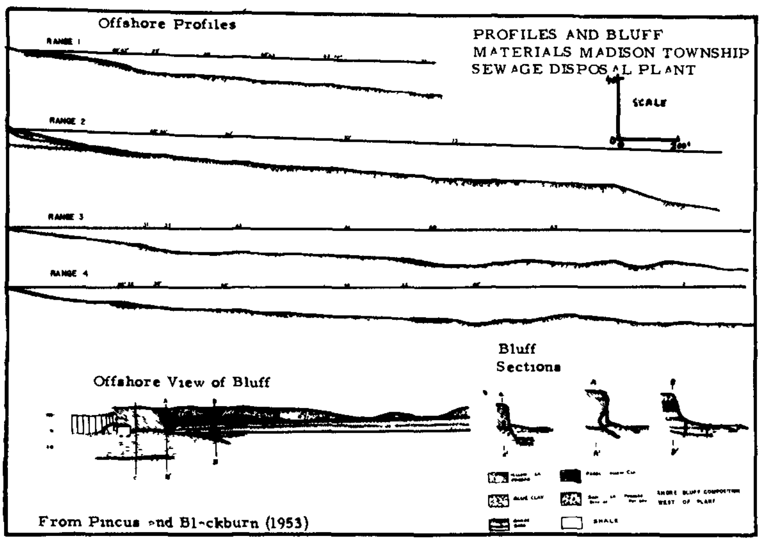

Fig. 19. Bluff materials and inferred offshore profiles, Madison Township Sewage Disposal Plant. Note that unconsolidated materials are apparently veneers covering bedrock (shale).

accretion at Conneaut, near the state line. The Harbor structures at Ashtabula and Conneaut appear to confine much of the shore material between the two cities.

Lee's (1953) detailed study of Presque Isle reports a predominant drift from the west; this direction is evident from accretion on the west side of structures and from the shape of Presque Isle. The source material apparently comes from the bluffs between Conneaut and Presque Isle. Wind-fetch factors favor an eastward drift.

SUMMARY AND CRITICISM OF THE EVIDENCE USED FOR DETECTING SEDIMENT DRIFT

The types of evidence useful for detecting sediment drift are:

1. Topographic changes: This category really deals with changes such as:

   a) Bluff retreat
   b) Shoreline changes, adjusted for comparable water levels, where possible.
   c) Accretion patterns, generally adjacent to structures of known age. (Age of structure allows estimation of rate of accretion.) These are tied to erosion patterns, where possible. Both erosion and accretion patterns can be mapped out (as in the House documents cited) from changes in bottom contours.

Regarding all topographic changes, the specification of the time during which the change has taken place is extremely important in estimating rates. Topographic changes usually reflect long-range changes, some of which are irreversible (viz., bluff retreat).

None of these changes defines the path of sediment transport. They indicate only effects which apparently show a predominance of drift in one direction. Combinations of sources of evidence will sometimes point to a to-and-fro or reversing motion, such as that inferred for Lorain Harbor.

c) may be tied to b) or a), or to some other possible source of sediment, such as streams, to work out an inferred path of transport. But here, there is always the possibility that there may be another "source", or there may be groups of sources as yet unidentified as such, the knowledge of which could yield a totally indifferent interpretation of sediment drift.

These changes also tell only the "algebraic" sum of all changes at a point---they do not indicate how much material has been moved in a period of time. Again, this is another way of saying that the path of transport has not been uniquely determined.

2. Changes in attributes of the sediments (mineralogical, chemical, and carbonate analyses):

a) Changes in mechanical properties, such as median grain size, sorting, skewness, and kurtosis, apparently indicate transportation only when some aspect of the transportation process or processes serves to modify the sediment with respect to one or more of these properties along the path of transport, and only when there are no contributions to the material along this path. Since much of the Lake Erie shore is actively eroding, there are relatively few longshore paths which would not be subject to such "pollution". And, of course, the use (in a problem) of a "variation series" for indicating path implies that something is known about the mechanism of transport.

Changes in mechanical properties which can be attributed to contributions from a distinctive source area may be used for indicating an overall direction of transport; this method, however, is subject to the limitations stated in the previous section, viz., the uniqueness of the source is assumption, not fact.

Combinations of mechanical properties may allow deductions which will yield clearer insight into possible modes of transportation by limiting the number of possible mechanisms of transport.

b) Mineralogical or compositional variations present similar problems of interpretation. Of course, such factors as mineral densities and shapes may provide insight into transport mechanisms, and in turn, into directions of movement. Into the problem of mineral assemblages come such factors as weathering. Are easily weathered minerals removed while in the till bluffs, in transport, or on the beach? Are weathered coatings removed by abrasion in transport? Also, there are sampling problems, such as those involving heavy mineral laminae, variations within single layers depending on distance from water's edge, etc.

3. Indirect Indicators: Wind direction, fetch, orientation of shoreline and wave patterns deal with the way in which energies are to be applied to systems of this kind. By themselves, they tell little about the motion itself: they merely restrict the possible directions and intensities which one can apply to descriptions of sediment movement. Of course, carefully thought through refraction and diffraction diagrams give considerable help in apportioning wave energy to various stretches of shoreline, and in combination with other types of evidence may prove to be fairly powerful tools for inquiry. The Lake Erie Geological Program has just instituted a project involving construction of refraction and diffraction diagrams for some of the crucial areas studied during recent years.

4. Scale models: Models which might prove very helpful, have not yet been used in the work of the Lake Erie Geological Research Program.

5. Observations of single grains, either in the field or in models: This technique, if successful, would yield behaviors of individual grains, which might be of about the same use here as the description of the supposed behavior of single gas molecules is useful in understanding some aspects of the behavior of a gas.

Combinations of these and other types of evidence provide important checks on interpretations, and the discrepancies arising from such comparisons may lead to a more complete understanding of the nature of the evidence.

ACKNOWLEDGMENTS

The investigations of the Lake Erie Geological Research Program have been conducted under the principal sponsorship of the Ohio Division of Shore Erosion, of which Mr. F. O. Kugel is Chief. Aid to the Program has come also from the Ohio State University through its Graduate School and Department of Geology and from the Ohio Departments of Geological Survey, Water, and Wildlife.

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The personnel of the Program are very grateful for this help, and for the friendly spirit of cooperation in which it has been offered.

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Chapter 9

STATISTICAL PROBLEMS OF SAMPLE SIZE AND SPACING ON LAKE MICHIGAN BEACHES

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INTRODUCTION

Beaches along Lake Michigan display a wide range in texture and composition. Some are composed of fairly uniform sand or gravel, others display an irregular mixture of both. Most beach sand is predominantly quartz with variable amounts of calcite, feldspar, and heavy minerals. Some gravel beaches are composed wholly of limestone or dolomite pebbles of local origin; in others the pebbles have the varied composition of their parent glacial materials.

Lake Michigan beaches may be narrow or wide, steep or gently sloped, soft or firm, and so on, depending in part upon their orientation, the season of the year, their nearness to natural source areas, and other factors. A single beach may show a wide variety of characteristics, with notable changes in texture and composition over short distances or during short time intervals.

The problem of sampling such nonhomogeneous deposits for geological or engineering purposes has challenged numerous workers; and to the writer's knowledge no completely satisfactory sampling technique has yet been developed. How large should a single sample be? How deeply should it penetrate the beach? How far apart should samples be collected? How often should the sampling process be repeated?

Specific answers to these questions must depend on the objectives of the beach study and on the nature of particular beach areas. It is common knowledge that material transported along beaches is subject to abrasional and selective sorting processes which produce systematic changes in particle size, shape, and other attributes. The objective of many studies is to measure these systematic changes and relate them to the physical processes of the beach and nearshore zone. The information so obtained is of value in geological interpretation, engineering design, and other fields.

In the present paper a somewhat general approach is used, to bring out principles which apply to all beaches. From these, it is possible to develop specific techniques for most common situations. Sand beaches afford a suitable starting point, both because more is known about them than about gravel or mixed beaches, and because sampling problems are relatively simpler. It is known, for example, that closely spaced samples over a small area on sand beaches have similar characteristics in terms of particle size, shape, composition, and other attributes. In other words, within a local area the particle population of a beach is homogeneous. That is, random samples from the small area satisfy the statistical condition that they have similar properties at some selected level of significance. Moreover, average values from the samples tend to distribute themselves as a normal probability curve.

As the area of sampling is enlarged, or as the depth of sampling increases, the variability among the samples increases, and the more outlying or deeper samples begin to display average properties significantly different from those in the smaller area. Statistically this means that the larger population is no longer homogeneous at the selected significance level. On this basis a beach may be considered as having a series of homogeneous populations which grade or abut into each other as the deposits are followed along or across the beach. Each individual population has a variability due mainly to random sampling fluctuations, whereas statistically significant variations from one population to the next may be considered as being caused by variations in the "natural treatments" imposed by waves, currents, and other geological agencies.

In some instances the homogeneous areas grade imperceptibly into others, as when particle size is studied along an extensive sand beach. In other instances there are abrupt population differences, as when the texture changes abruptly from uniform sand on the backshore to stringers and patches of gravel on the foreshore. In general, changes across the beach are more pronounced than along the beach, and most beaches display a series of bands or zones parallel to the shore, each of which is homogeneous for an appreciable distance along the beach.

The several homogeneous populations that comprise a beach may be considered as a universe or set of populations which characterize the beach as a whole. The number of such populations and the relations among them, are partly a function of the significance levels selected, and partly a function of the natural gradations and abrupt changes which occur on the beach. It is also possible to consider the gradational types of population as single populations with linear or exponential gradients.

In statistical practice it is recognized that sampling procedures should be adapted to the kinds of populations being sampled. A few random samples from a homogeneous population may be sufficient to estimate the characteristics of the population, but when heterogeneity is present, the sampling plan usually involves some type of systematic or stratified sampling. Sampling procedures are designed to ensure that samples are representative of their populations, and that more than one population is not inadvertently included in any one sample.

The purpose of this paper is to show that definite answers to some problems of sampling heterogeneous beaches can be given. Sampling plans suitable for various beach conditions are discussed in the light of standard statistical procedures for sorting out and evaluating variabilities among samples.

STATISTICAL PROBLEMS OF SAMPLE SIZE AND SPACING ON LAKE MICHIGAN BEACHES

BEACH POPULATIONS

The writer and his students, with collaboration of E. C. Dapples, have conducted many beach sampling experiments during the past seven years. Some of the results are reviewed here, supplemented by other data to enlarge the frame of discussion. Most of the experiments were conducted with sampling grids laid over the beaches. The grid spacing varied from less than 10 feet to as much as 100 feet between samples. Most grids involved segments of beach 200 feet or more long, commonly covering the entire width of exposed beach. In some experiments a single sample was collected at each grid point; in others multiple samples were collected around each stake.

The samples were analyzed for moisture content, particle size, particle shape, heavy mineral content, etc. In many instances a parallel set of beach firmness readings was made with a penetrometer. The analytical data were plotted on maps, and almost all of the maps show a distinct zoning or banding parallel to the shore, with each band relatively homogeneous over its length. Some beach attributes do not display this marked banding; for example, maps of particle roundness may be relatively irregular across the beach, although in some studies a marked increase in average roundness is found on the landward side in the dune sands. On the whole, however, most properties that are related to the dynamics of beach processes (such as average particle size, degree of sorting, sphericity, and density as shown by heavy mineral content) do display systematic patterns.

The concept of beach populations was suggested by the systematic map patterns and by the generally known geological processes occurring on beaches. Waves wash up to the berm but seldom cross it. Hence, the berm is a natural subdivision between a lakeward part of the beach subject to rhythmic wave action, and a backshore acted upon seldomly by waves. The backshore consists mainly of dry sand and is subject to winnowing action by the wind. Farther inland the winnowed sand may accumulate in a belt of dunes. These geological processes vary with the seasons and contribute to cross-beach variability. Similarly, currents along the shore tend to carry some particles farther than others, so that average particle size and other properties vary from one end of the beach to the other. The along-beach changes are generally more gradual than the cross-beach changes, however.

The data from the grid experiments were analyzed statistically to determine whether the observed variability of beach properties could occur within a single homogeneous population, or whether a more suitable approach would be to interpret the data as representing several populations distinct from each other. Numerous analyses showed that statistically significant differences are present for a number of properties on the conventional 5 per cent significant level. The method used is <u>analysis of variance</u>, a statistical technique for sorting out and evaluating variabilities. In its simplest form the method compares variances within and between groups of data. If the ratio of the between-

LANDWARD

56 58 58 50	54 58 54 58	52 62 56 64	56 48 64 62	54 66 56 58
60 62 70 62	62 58 62 50	58 60 60 62	58 64 58 56	54 54 58 56
58 52 54 52	60 56 62 48	68 64 56 56	64 56 44 52	54 54 48 52

N LAKEWARD 0 2 FT.

Fig. 1. Penetrometer grid on beach foreshore, Evanston, Illinois. Dial penetrometer, 6-inch penetration.

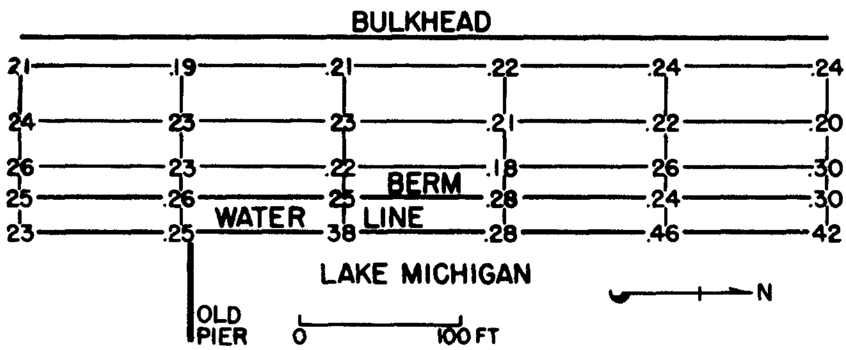

Fig. 2. Sampling grid, Lee Street Beach, Evanston, Ill. Numbers are geometric mean diameters of sand size distribution curves at sampling sites.

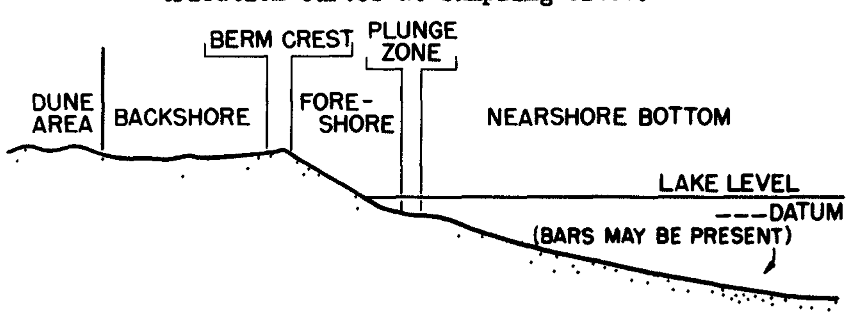

Fig. 3. Cross-section showing "population bands" on Lake Michigan beaches.

group variance to the within-group variance exceeds a selected critical value, the difference between the groups may be considered statistically significant. In terms of the present approach, such differences may be interpreted as indicating that the populations represented by the several groups are also different.

Statisticians have developed a number of analysis of variance models during the past two decades. Several of these are directly applicable to beach problems. Full description of the methods are given in numerous recent statistics texts, such as Cochran and Cox (1950), Goulden (1952), and Kempthorne (1953). Dixon and Massey (1951) provide an excellent introduction to the subject. Five models which apply to a variety of geological problems are discussed by Krumbein and Miller (1953), and several of these are included in the following examples.

BEACH FIRMNESS

Beach firmness is a convenient property for introducing analysis of variance of beach characteristics. Penetrometers of several kinds are available, the experiments are readily performed, and the data are fairly reproducible. The experimental grid may be laid out with stakes; readings may be taken at each stake, or distributed over the rectangular areas defined by the stakes. In one study of local variability of firmness, stakes were set at 2-foot intervals over an area 6 feet wide across the beach and 10 feet wide along the beach. Each "cell" between the stakes was a 4-foot square, and four readings were made in each cell, as shown in Figure 1.

The penetrometer used in the experiment consisted of a steel rod with a gauge attached. The rod was inserted to a fixed depth in the sand, and the limiting pressure for additional penetration was read from the gauge. The larger the reading, the firmer the beach. Table 1 shows the analysis of variance of the penetration data. An intermediate step, not shown, is preparation of a "condensed table" in which the four values of each cell are combined into a single value. This analysis of variance model is a two factor form with replication, and details of computation are given in Dixon and Massey (1951, p. 134) and in Krumbein and Miller (1953). Essentially the computation consists in squaring all items in the original cells and in the condensed cells. In addition, column and row totals are squared, as is the grand total. These values, converted to sums of squares, are arranged in the analysis of variance table with appropriate degrees of freedom. The mean square or variance is then found by dividing the sums of squares by corresponding degrees of freedom. In Table 1 the total variance has been separated into four parts, representing variances due to row, column, "interaction", and residual effects.

The mean square for interaction is first evaluated against residual error at the 5 per cent confidence level, and in this instance it is non-significant. The interaction sum of squares is then added to the residual, and the total is divided by the combined degrees of freedom to give a net

TABLE 1

ANALYSIS OF VARIANCE OF PENETROMETER DATA FROM CAMPUS BEACH. ORIGINAL VALUES SHOWN IN FIGURE 1

Source	Sum of Squares	Degrees of Freedom	Variance	F
Columns (along beach)	131	4	32.75	1.33 N.S.
Rows (across beach)	138	2	69.00	2.80 N.S.
Column x row interaction	229	8	28.63	1.20 N.S.
SUBTOTAL	498	14		
Residual	1077	45	23.93	
TOTAL	1575	59		

N.S. = non-significant

(Combined error = 1306/53 = 24.64)

TABLE 2

ANALYSIS OF VARIANCE OF GEOMETRIC MEAN DIAMETERS OF SAND SAMPLES FROM LEE STREET BEACH, EVANSTON, APRIL, 1950

Source	Sum of Squares	Degrees of Freedom	Variance	F
Columns (along beach)	0.0173	5	0.00346	1.62 N.S.
Rows (across beach)	.0562	4	.01405	6.49 **
Residual	.0428	20	.00214	
TOTAL	0.1163	29		

N.S. = non-significant
** = significant at 1% level

error term. When the main effects (along-beach variance and across-beach variance) are tested against the combined error, neither is significant at the 5 per cent level.

The result of this experiment is that the small grid area may be considered as having a homogeneous population of beach firmness. The variability within the rows and columns is of the same order as the variability between rows or columns, indicating that there are no significant row or column effects. In other words, there is no significant variation along or across the beach within this grid area.

PARTICLE SIZE

A second example of beach populations is furnished by Figure 2, which shows a grid 500 feet long and 100 feet wide on Lee Street Beach, Evanston. The grid extends from the backshore across the berm crest to the water line. A sample was collected at each grid point and analyzed for particle size; the figure shows the geometric mean diameters at the sampling points. Inspection of the figure shows that mean particle size increases lakeward across the grid. The berm crest was sampled in this experiment to see whether its characteristics were transitional between backshore and foreshore.

The analysis of variance is shown in Table 2. This is a two factor form with single entry (Dixon and Massey, 1951, p. 127). This simpler form does not permit evaluation of interaction effects. As the table shows, there is a highly significant difference between the rows (across beach), but no significant difference between the columns (along beach). Additional analysis indicates that the three landward rows on the backshore are homogeneous among themselves, and that the foreshore may be considered as a different population, with the berm crest representing a transition between the two. If the values in the landward three rows are averaged, the mean size is found to be 0.228 mm. The samples in the row nearest the lake average 0.337 mm. These are unbiased estimates of the two population means for particle size. The berm samples may be treated as a third average if desired.

An extension of the method of analysis also permits evaluation of the population standard deviation for sample means. Details are not given here, but the subject is discussed in Dixon and Massey (1951, p. 142). An estimate of the backshore standard deviation by these methods is 0.026 mm, and for the foreshore it is 0.096 mm. These values suggest that the backshore in this instance is finer and more homogeneous than the foreshore.

OTHER BEACH POPULATIONS

Numerous analysis of variance studies similar in form to Tables 1 and 2 have been made, on grids varying in size as previously mentioned (Krumbein and Miller, 1953; Krumbein, 1953). These studies show that each attribute of the beach, such as firmness, particle size, particle shape, moisture content, thickness of bedding, etc., has variations along and across beaches. It may be concluded, therefore, that beaches consist of a number of populations of different attributes, which together comprise some universe or super-population of attributes representative of beaches as a whole.

The problem of beach sampling thus appears to resolve itself to the sampling of variable populations, and the practical solution requires some method which provides representative samples of the populations or of "strata" within the super-population. In most geological or engineering studies particle size is the main attribute investigated, with mineral composition, firmness, and other characteristics important in special studies. Such attributes as moisture content, though interesting, have been studied mainly for teaching purposes.

The writer's studies on Lake Michigan sand beaches indicate that several areas of relatively homogeneous population can be discerned on most sand beaches. These population zones or bands may be made the basis for sampling the beach. Figure 3 shows the zones on a completely developed sand beach. The landward area of windblown sand, where present, grades gradually to the backshore. The backshore is relatively flat, sometimes with a gentle landward slope. The berm crest, represented as a narrow band in the figure, is a transitional area having a mixture of backshore and foreshore characteristics. This transitional zone may vary in position and width seasonally. The foreshore may be homogeneous in particle size, but usually has several moisture content populations. The foreshore extends to the average zone of breaking waves (also subject to seasonal change) where rapid and marked population changes may occur. The nearshore bottoms may be homogeneous along fairly extensive stretches of shore in moderate depths. In local instances population changes may occur near jetties and breakwaters.

The relative simple pattern of Figure 3 is subject to wide variations, especially on mixed sand and gravel beaches. Even on sand beaches the zoning may be seriously disturbed by storm waves which wash across the backshore. A major storm in Spring, 1949, scattered very coarse sand and pebbles over most of the beach at Wilmette, Illinois, in an apparently random manner. Samples collected shortly after the storm showed a wide variability in average particle size, although there were no significant row or column effects in a sampling grid 500 feet long and 150 feet wide. Apparently the storm had developed a wide but variable "foreshore" over the whole beach. In time more normal conditions redeveloped a new foreshore and berm nearer the lake, leaving a somewhat patchy but homogeneous backshore inland.

Gravel beaches present their own problems in terms of zoning. A relatively recent gravel beach at Dempster Street, Evanston, showed several steep gravel berms with distinctly different degrees of coarseness visible to the eye. Surprisingly enough, a set of samples collected on a grid which included the berms showed no significant across-beach variance in average particle size, but did show a marked size gradient along the beach.

STATISTICAL PROBLEMS OF SAMPLE SIZE AND SPACING ON LAKE MICHIGAN BEACHES

CYCLICAL CHANGES ON BEACHES

The foregoing discussion considers beach populations as they are at a given moment. In many studies it is desirable to record changes on the beach during storm cycles, seasons, or over longer time periods. Such changes may be noted by collecting samples on the same grids or along the same profiles at specified times. Maps of the successive beach patterns yield a visual picture of the changes, and significant differences from one pattern to the next can be evaluated by conventional geological analysis or by analysis of variance.

In 1947 the writer and his assistants, E. J. Herbaly and Carl Setzer, made a daily study of changes along a beach segment fronting Northwestern University. The segment is 225 feet long and is enclosed at both ends by pre-cast permeable groins. Maps were made each day during most of October and November. The maps showed a systematic series of sand and gravel distribution patterns during and between storm cycles.

The maps were generalized into several stages of beach development during the storm cycle. Figure 4 shows these stages. The upper map (A) shows the normal beach pattern after several days of quiet water (no waves exceeding 6 inches in height). A landward band of uniform sand gives way abruptly to a low ridge of coarse gravel marking the main berm. Lakeward of the coarse gravel berm were bands of medium and fine gravel, each marking a minor berm. No sand was exposed at the water's edge, although sand bottom was present a few feet from shore. The quiet water beach thus apparently displayed four particle populations, usually with abrupt contacts, although some gradations attributed to boundary conditions could be seen near the groins.

Several moderate storms occurred during the period of study. Some lasted for more than a day, with a gradual increase in wave height and period, followed by a tapering off to quiet water again. The first effect of a storm, observable when the waves reached a height of about 12 inches, was the stripping away of the finer gravel berm, leaving a foundation of uniform sand, as shown map B of Figure 4. As the waves increased in height to about 24 inches, the medium gravel berm was stripped off, leaving a sand zone from the water's edge to the coarse gravel berm, as shown in map C of Figure 4. In some instances the coarse gravel berm was locally breached at this stage. If the storm continued to a wave height of about 48 inches, most of the coarse gravel was stripped off, leaving a uniform sand beach to the foot of the bank. Map D of Figure 4 shows the effects of a storm during which the waves averaged about 54 inches high at the breaking point.

As the storm subsided, the coarse gravel berm was restored first, then the medium gravel berm, and finally the fine gravel, leading to a map much like the initial stage of Figure 4. This sequence of events was observed on several occasions during the study, and although no underwater samples were taken during the storms, it appeared evident that the gravel was stored offshore in one or more bars, which moved shoreward as the storm died down.

The important point for the present discussion is that the beach underwent a systematic series of population changes during the storm cycle. How should such a beach be characterized? During calm weather it is essentially a pebble beach; during severe storms it is entirely sand. One may perhaps argue that the beach can best be characterized by a composite sample from all four bands. Such a composite, however, would not by itself indicate the extreme variability of the beach. A preferable method would perhaps be to take samples from each band separately, weighting the averages in terms of band width and length. For detailed study it seems preferable to use principles of stratified or systematic sampling, as discussed in a later section.

SAMPLE SIZE AND DEPTH OF PENETRATION

Common practice on sand beaches is to collect samples of shallow depth and relatively small diameter. An average of present practice would perhaps be to collect a cylindrical sample from 1.5 to 3 inches in diameter and about 2 or 3 inches deep. In some instances the upper half inch of sand is first scraped off to remove stray debris. Samples of beach gravel commonly vary in volume, depending on the size of the larger pebbles present, but the writer knows of no average or standard practice for pebble beaches.

Inasmuch as a sample should be representative of the population at the point of sampling, experiments can be designed to determine the extent to which observed characteristics vary as sample size or depth are varied. Elsewhere the writer describes such an experiment on a sand beach (Krumbein, 1953), and in this paper similar data are given for medium gravel with mean size between 8 and 16 mm. The experimental design involves using a series of concentric rings which are forced to successively greater depths as sampling proceeds. The gravel in each annular space is collected and analyzed separately, and the data are combined to develop a composite cylindrical sample which grows in diameter and depth.

For the gravel sample the writer used five welded metal rings of diameters 3, 6, 9, 12, and 15 inches. The rings were forced into the beach to a depth of 2 inches, and the process was repeated four times, yielding four sets of five samples with a total cylindrical volume 15 inches in diameter and 8 inches deep. The size data for analysis of variance as shown in Table 3, which lists the geometric mean diameters in mm. of the successive analyses. The total sample weighed 27 kilograms, roughly 60 pounds. The lower part of Table 3 shows the analysis of variance as a two factor form with single cell entries. The between-column variance (area of sample at given depth) is not significant at the 5 per cent confidence level, but the between-row variance (depth of sample for given area) is significant at the 1 per cent level. This implies that for gravel of the size studied, the smaller rings for any given depth are not significantly different from the larger rings, but that a shallow sample of given area is significantly different from a deeper one of the same area.

STATISTICAL PROBLEMS OF SAMPLE SIZE AND SPACING ON LAKE MICHIGAN BEACHES

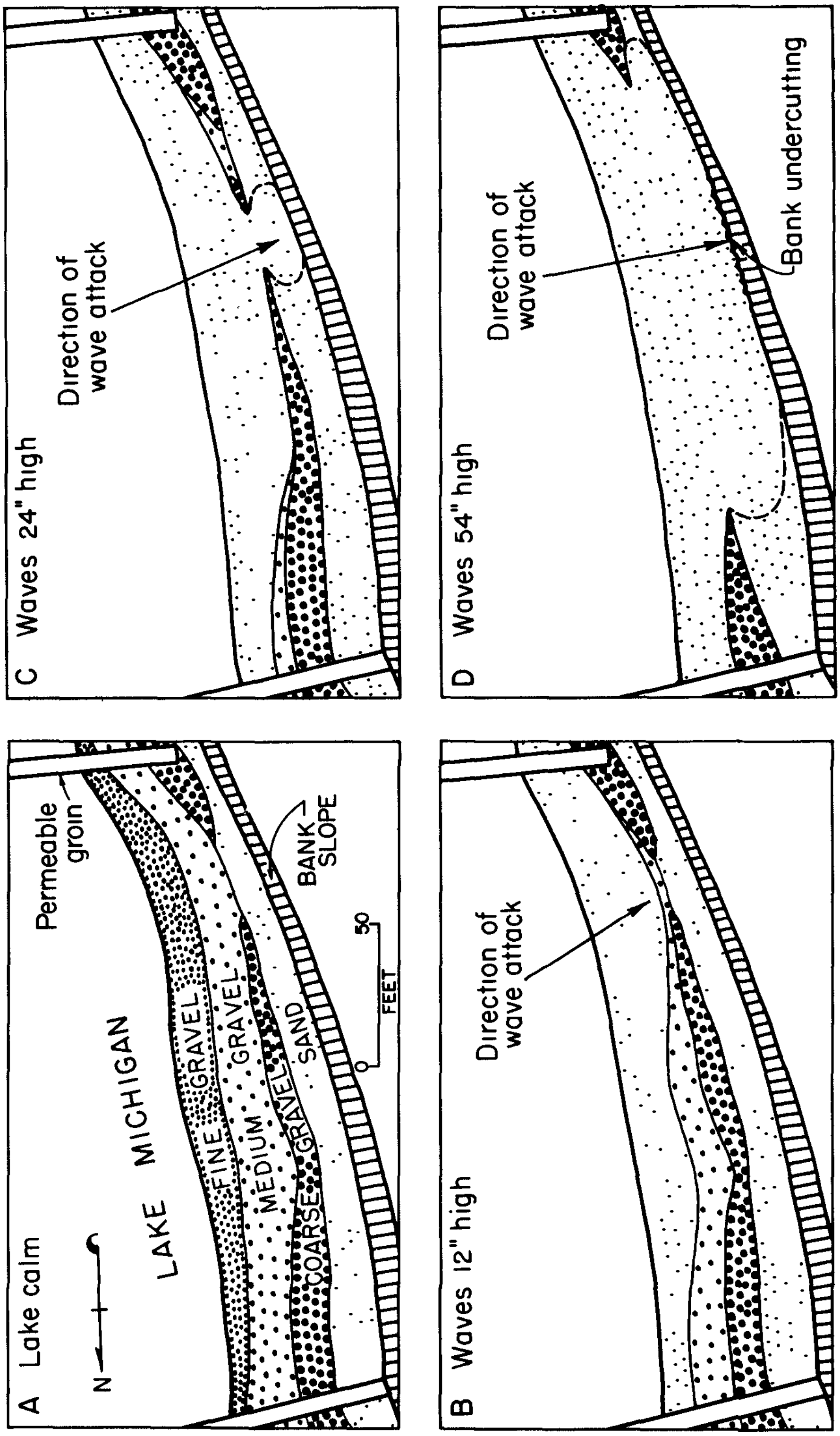

Fig. 4. Maps of small campus beach at Northwestern University, showing changes during storm cycles.

Table 3.

ANALYSIS OF VARIANCE OF GEOMETRIC MEAN DIAMETERS OF GRAVEL FROM CONCENTRIC RING SAMPLER, DEMPSTER STREET BEACH, EVANSTON

Ring Diameter

Depth	3"	6"	9"	12"	15"
2"	11.7	11.5	11.5	11.0	11.3
4"	10.3	10.3	10.3	10.3	10.6
6"	9.5	9.6	9.7	9.6	9.8
8"	8.9	9.1	9.2	9.2	9.3

ANALYSIS OF VARIANCE

Source	Sum of Squares	Degrees of Freedom	Variance	F
Columns (sample area)	0.12	4	0.030	1 N.S.
Rows (sample depth)	14.43	3	4.81	150.3 **
Residual	0.38	12	0.032	
TOTAL	14.93	19		

N.S. = non-significant
** = significant at 1% level

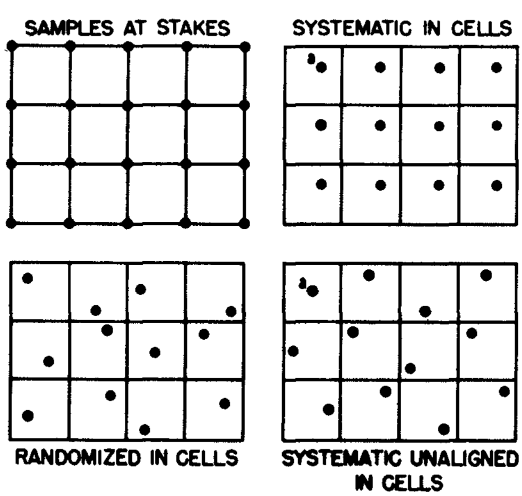

Fig. 5. Diagram showing four methods of arranging samples on a beach grid. (Adapted from Quenouille, 1949)

The results of the gravel analysis were the same as for the earlier sand analysis, which showed that a sample 1.5 or 3 inches in diameter to a given depth was not significantly different from larger samples up to 12 inches in diameter. As with the gravel, there was a significant difference between samples of different depth. Both of these studies are only exploratory, but they suggest that a minimum and perhaps an optimum size of sample could be determined for a variety of deposits. For gravel of the size studied, the writer believes that a cylindrical sample 6 inches in diameter is amply large. The depth of penetration is more a matter of opinion, and must be decided on the basis of the purpose of the study.

The composite ring sample was collected in an area from which additional samples 6 inches in diameter and 2 inches deep were also collected. The ring sample gave essentially the same population mean as the other samples in its vicinity for samples collected in the 2-inch depth.

BEACH SAMPLING PLANS

In an earlier paper the writer suggested using each beach population or zone as a separate sampling area (Krumbein, 1953). For any one beach profile or traverse, the several zones normally present above water may be recognized by noting berm position, textural changes, relative firmness, and the like. A single sample collected at random from the central portion of each zone provides data for an estimate of the population mean. Underwater samples are collected either in terms of water depth or fixed distances from shore. Control by depth appears to be favored in practice.

The sampling procedure is improved if a minimum of four samples is collected from each population band. In this manner sufficient data are obtained to estimate population variance as well as the mean. Even two samples from each zone are preferable to single ones, if time or expense prevent collection of four. Multiple underwater samplers can be designed which provide two samples at each station.[1]

The use of beach zones as a basis for sampling is a form of stratified sampling, in which the zones represent "strata" of the main population or universe, each conceivably with its own mean and variance. If the samples in each "stratum" are selected by a random process, the samples are "stratified random samples." On the other hand, if the samples in the first stratum (say the backshore) are randomized, and samples in succeeding strata are collected in the same relative positions, the samples are "systematic samples." The simplest case of systematic sampling is to collect a sample from the center of each zone or stratum.

1. The writer had the privilege of using a two-tubed sampler developed at his request by the Beach Erosion Board. The sampler consisted of two parallel pipe segments with bell-shaped mouths. When cast overboard and pulled back to the ship, two samples were collected about 16 inches apart.

Multiple samples in each zone may be collected at random from segments of a circle drawn around the central stake. A 1-foot circle centered at the stake is divided into 45° segments numbered 1 to 8. Four numbers are drawn from a random number table (say 2, 3, 5, 8), and samples are taken 1 foot from the stake at these prenumbered positions. An alternative method is to subdivide each stratum or zone into several substrata, with a sample taken from each substratum by some random or systematic process.

If the sampling plan includes an appreciable width of beach on either side of the profile line, the problem becomes one of plane sampling. The commonest example is the square staked grid in which samples are collected at each stake. The problem may be approached from a more general viewpoint, however, as Quenouille (1949) showed. The grid may be arranged as a pattern of squares, rectangles, or other shapes. In the present discussion the square grid is used as illustration, following the essential development of Quenouille.

Figure 5 is an adaptation of a similar figure from Quenouille, showing four possible sampling plans out of a large number discussed in the original paper. Each grid is composed of square "cells" which may be considered as the sampling units. In the upper left diagram the samples are taken at the grid stakes, and in the upper right diagram they are taken with the same spacing, but at some fixed position within the cell. Both are "systematic aligned samples", with the position in cell a of the upper right diagram determined by drawing a random number for position. Once the value is selected, all subsequent samples are taken from the same cell positions.

The lower left diagram is an example of stratified sampling in which the position of the sample is separately randomized for each cell. This spreads the samples over the cells in the manner shown. The lower right diagram is an example of "unaligned systematic sampling" in which the sample in square a is randomized, succeeded by randomization of vertical co-ordinates in the remaining upper row cells, and randomization of horizontal coordinates in the remaining first column cells. Cochran (1953, p. 183) describes the process. By extending this pattern over the cells a systematic arrangement without alignment is obtained. The unaligned systematic samples have some advantages over the simpler systematic plans of the first two diagrams, in that they supply additional data on gradients in the population (Cochran, 1953, p. 184).

The writer has extended some of these plans to as many as four samples per cell. The additional samples are located by repetition of the same randomizing or systematizing process as is used for the single cell samples. For analysis of variance studies involving evaluation of interactions, multiple samples from each cell provide more information than single samples. Controlled sampling plans of this type can readily be applied to the exposed beach, but the collection of underwater samples is more complex if the ship has to be maneuvered into specific grid positions. Aligned systematic samples can be taken conveniently, however, inasmuch as the ship may follow a straight course during the sampling.

Quenouille (1949) tested the relative efficiency of the several sampling plans for various populations, and pointed out that the efficiency varies somewhat depending on the population characteristics, and presumably on the objectives of the study. If the grids are confined to homogeneous zones on the beach, any of several may be satisfactory. If the grid straddles population zones the same cell may contain samples from more than a single population. On the other hand, if the zones are thought of as units within some "super-population" with linear or exponential gradient, then a grid which brings out the details of the gradient is presumably the best. A study of this problem for beaches by geologists or engineers in collaboration with a mathematical statistician would clarify several important details.

The use of sampling grids, especially with fairly close spacing, is perhaps more appropriate for detailed beach studies than for conventional beach surveys. In most applied problems the objective is to obtain data on beach characteristics with a minimum expenditure of time and cost. For these studies the collection of samples from beach profiles located at convenient distances along the beach appears to be suitable (Krumbein, 1953). When the areal variations on the beach are to be related more specifically to beach processes and wind winnowing, however, more detailed population studies are indicated.

The several sampling plans described above are sufficiently general to apply to most sand beach situations. The specific problem of sampling Lake Michigan beaches, with their relatively large variability of particle size, may require adjustments in grid spacing, in sample size, and in repetitions of the sampling process. Each beach presents its own special problems, but it is believed that the sampling plans described can be adapted to most situations. Beaches with mixed sand and gravel, either as patches of gravel on an otherwise sand beach, or sand beaches with scattered isolated pebbles appear to present the greatest difficulties for sampling. More study of these conditions is needed.

CONCLUDING REMARKS

The present paper is a progress report on some experimental approaches to the problem of sampling Lake Michigan beaches. It is evident that much remains to be learned about the most efficient sampling plan for any specific situation. It is believed that many of the problems can be solved by suitably designed experiments. A large background of sampling theory is available as a guide for such investigations. Cochran's book on sampling (1953) is a basic reference in this connection.

In the long run the sampling process represents an initial stage in acquiring geological and engineering information on beaches. After the samples are analyzed and the population parameters estimated, there remains the question of the relation between these parameters and the natural processes which control them. Experimental design, which is rapidly expanding in the earth sciences, provides one way of attacking the general problem. The basic importance of sampling in all subsequent interpretation of the data suggests that the sampling plan itself be an integral part of the over-all experimental design.

This paper does not include a review of the somewhat voluminous literature on Lake Michigan beaches, inasmuch as it is directed toward the more general problem of sampling design. However, the reader is referred especially to the paper by Fisher (1954) in this symposium for an example of a comprehensive study of size and other properties along the Lake Michigan shore of Illinois. The study involves repetitive sampling along selected profiles, and includes both exposed beach and underwater samples. The maps accompanying the paper show the distribution of underwater areas of uniform and variable sand characteristics, useful in a study of population changes in the nearshore zones.

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CHAPTER 10

CHARACTERISTICS OF NATURAL BEACHES

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The subject of beach characteristics is a complex one which is obviously worthy of book-length treatment. The object of this paper is to describe briefly the shapes which groups of beach-forming particles take and the mechanisms by which the forces of nature have so arranged them. Using these reactions as criteria, the coastal engineer who is confronted with the problem of replenishing beach materials or of altering some existing sand flow with a shoreline structure will be able to understand (and predict) the rapid readjustment of the particles to their new environment. There are, of course, no unnatural beaches; the implication is that artificially created or nourished beaches and otherwise altered shorelines will respond in the same manner as those untouched beaches which were studied to assemble the information presented here.

The most important characteristic of a beach is its dynamic nature; beaches are restless, ever-shifting groups of particles which respond with great sensitivity to small changes in the natural forces that are quite imperceptable to man. A concept of this ceaseless change which reflects the wave characteristics is absolutely essential to the understanding of beach problems.

The actual shifting of the beach materials takes place under water and since the human mind cannot very well remember and compare the previous position of the surface with some later position, it may appear that little or no change is taking place. However, where there are rigid structures, such as pilings, projecting thru the beach for sand-level comparisons or where detailed surveys indicate that material has moved, the total volume in transport may prove to be astonishing. For example, the daily accretion to the sand spit in Santa Barbara harbor in some seasons is of the order of 700 yards a day (Bascom 1951) or equivalent to a large truck dumping 10 yards of sand there three times an hour, 24 hours a day continuously. Even though the area is small, this rapid growth is discernable only over an extended period of time or by survey and plotting which foreshortens the time scale. Other cases of longshore sand movement are rarely so conveniently measured but such magnitudes are probably not unusual.

The other principal motion of beach material is off-shore - - onshore which is represented by the migration of the materials from bar to

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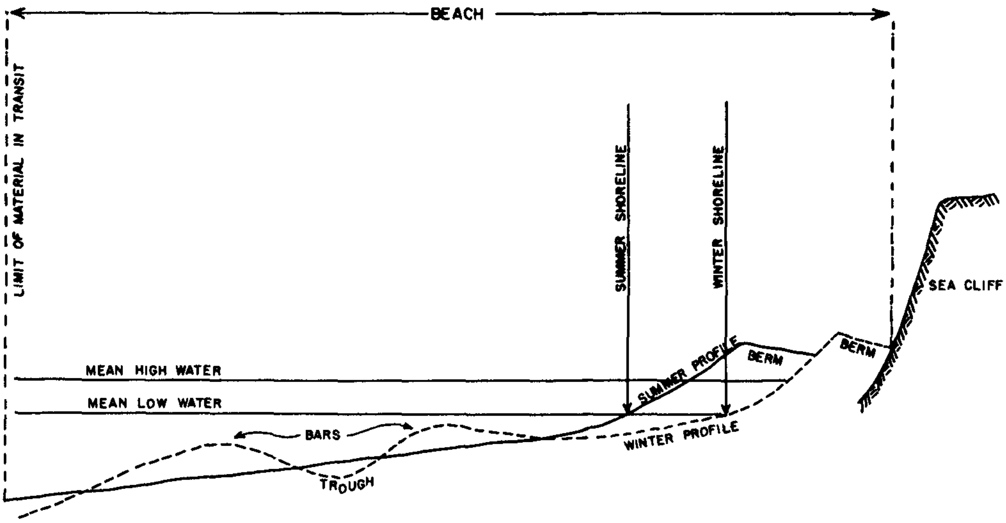

Fig. 1. Beach profile showing seasonal distribution of sand.

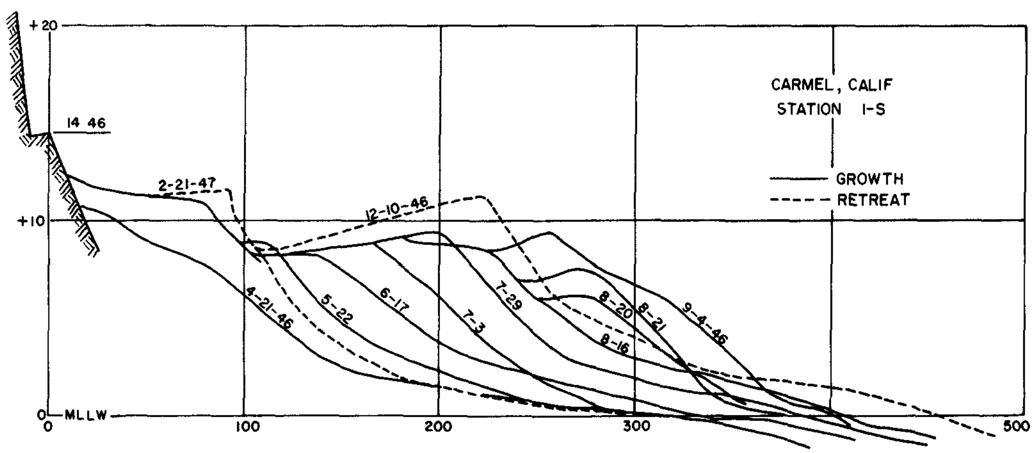

Fig. 2. Growth and retreat of a berm.

berm and visa-versa. The most noticeable form that this transfer takes is the removal by storm waves of the part of the beach above water. The arrival of such waves after a quiescent period may move as much as 2,000 cubic yards overnight from a 100 yard length of berm.

What are the materials involved and how do the natural forces shift them? This paper will be restricted to the classical situation which probably represents over 90% of the beaches, and thus the problems, of the United States coasts. The forces are those of waves and wave-caused currents; the material is sand (dominantly quartz) with a median diameter of from 0.2 to 1.0 mm.

There are other forces such as the winds which create local wind-driven currents and act sub-aerially on the part of the beach above water, and there are other materials: fine sands, pebbles, shingle and boulders. (In fact the word beach originally meant the tabular-rounded sedimentary pebbles which so commonly compose the shorelines of the British Isles). For the sake of brevity, a detailed discussion of these comparatively minor forces and unusual materials will be omitted. It is important to note however, that the sandy beach forms which will be described are also seen, equally well developed, in pebbles and cobbles.

BEACH FORMS

"A beach is a deposit of material which is in transit either along shore or off-and-on shore". This dynamic concept of many particles in continual motion is attributable to G. K. Gilbert (1890) and D. W. Johnson (1919). Notice particularly that it includes particles seaward of the shoreline; the criteria is only that they be "in transit". It is so worded as to include longshore bars, troughs and rip channels which are quite as important to the student of beach processes as the above-water features.

Figure 1 makes use of stylized winter and summer profiles of the beach at Carmel, California to illustrate the more important features and indicate typical seasonal changes. Note that the volume of sand involved is a constant since the areas under winter and summer sections are about equal; the material merely shifts from berm to bar and back again.

Various major coastal features such as spits, offshore bars, baymouth bars and tombolos are composed of beach materials and, of course, have a present-day beach at their seaward edges. Such features are of greater interest to geologists than engineers and, since their overall rate of change is quite slow, will not be discussed here.

LONGSHORE BARS

Longshore or beach bars are ridges in the material of which the inshore bottom is composed and are usually thought of as being parallel to the shore and maintaining the same section for a considerable distance.

This ideal condition may exist if wave and current forces remain constant for a considerable time, however, a change in wave direction will rapidly change neat lines of bars into a chaos of short irregular ridges with meandering troughs between. The origins of longshore bars have long been the subject of study and speculation and still there is no general agreement as to the details of the process by which they are formed and maintained. They form during storm conditions in which seaward flowing bottom currents remove material from the beach face and carry it seaward. The number of bars on a beach depends on the size of the waves, the general bottom slope and the tide range.

Consider a beach with an even sloping bottom (barless beach profile) which is subject to light wave action; sand is moved slowly ashore by the mechanisms discused later under Forces. A storm arises and the waves become much higher; more bottom is affected because the waves feel bottom sooner and sand continues to move towards shore from comparatively deep water. When these waves break and become translatory, the water moves forward with the wave form; its natural return is a seaward-flowing bottom current which transports sand eroded from the beach face. This seaward-moving sand meets the shoreward-moving sand under the line of breakers (where the transformation from oscillatory to translatory waves takes place); at this point the opposing currents neutralize each other and the sand deposits to form a bar. Once the process starts the effect is progressive. The rising mound of sand (the bar) causes more waves to break since it is shallower; it also concentrates the breakers of slightly varying height (which formerly broke at their respective depths) in one place. Waves are thus filtered by the bar; only those less than some critical height pass across without breaking - - when these reach some appropriate depth, they also break and an inner bar may be formed. When bar formation has progressed sufficiently, the bar projects well above the surrounding sand and the trough to its landward side is frequently wide and deep. This results in a further adjustment since broken waves are now progressing in deepening water rather than gradually shoaling water. The translatory waves created by breaking are sometimes able to reform into oscillatory waves and although a great deal of energy has been lost in the original breaking, the reformed wave may be as large as many of the smaller ones which passed over the bar without breaking; these waves break again on the inner bar. For this reason, practically all waves break on the inner bar with breakers which are much smaller than those on the outer bar; for those waves which break twice, the difference in height between the two breakers is a measure of the amount of energy expended in the outer breaking. (Note that the height of a breaker is proportional to the square root of the energy).

Small waves are not able to obliterate the deep bars created by large storm waves since they do not affect the bottom sufficiently; however, as already mentioned, they do move sand ashore from shallow water and thus will tend to plane off bar tops and fill in troughs. This is abetted by the absence of scouring rip currents in light surf. The modification of the inner bar and the filling of the trough between it and the beach face occasionally results in a nearly flat surface intermediate between a bar and a berm which is called a low-tide terrace. As might be expected, these low-tide terraces are generally found on partly protected beaches or on exposed beaches which have experienced a long period of small waves. Thus, a large storm may create a deep bar which will remain largely unchanged through months of calmer weather (until another group of large waves exists to change it) although at the same time there may be considerable change in shallower bars and the beach inshore. This means that the depths of the several bars are not necessarily related except immediately after a large storm and that valid comparisons of bars and waves require continued observation of the beach before, during and after changes. In an attempt to discover the grosser mechanics of sand movement the Beach Erosion Board has conducted experiments with artificial bars at Long Branch, New Jersey and Santa Barbara, California which have given some negative information. In the hope of nourishing these sand-starved beaches, underwater sand ridges were placed in 38 and 18 feet of water respectively, the idea being that the wave action would bring the sand onto the beach. According to Hjulstom's formula for threshold velocity, the sand should have been moved by the waves that existed; however, the sand did not move appreciably and the mounds remained for a number of years. It should be noted, however, that no bars are known to exist even on those beaches of the United States Pacific Coast most exposed to violent storms which have crests more than 20 feet below mean lower low water. Figures obtained by averaging tabulated data from 29 surveys of exposed United States Pacific Coast beaches with two or more bars have turned up some interesting facts:

<u>Bar Depths</u>

Depth of inner bar	-	1.0 ft.	
			6.5 ft. difference
Depth of second bar	-	7.5 ft.	
			5.5 ft. difference
Depth of third bar	-	13.0 ft.	

<u>Beach Slopes</u> (measured between MLLW and -30 ft. or 3000 ft. offshore)

Less than 1/75	three or more bars
1/75 to 1/50	two bars
Greater than 1/50	one bar

Note: All exposed beaches had at least one bar.
No individual beach varied greatly.

The phrases "exposure" and "protection" are conveniently used to describe large differences in the amount of wave energy reaching a beach; it is to be expected that beach changes are of greater magnitude where violent waves occasionally impinge; the application of this to bars is of interest.

Like other beach features, bars have lower relief and are composed of smaller particles as protection is reached. Where protection is at a maximum as in the lee of a headland, bars rarely exist because waves never get steep enough to form them. Attempts to correlate bar spacing with wave length or other characteristics have been unsuccessful to date but there is a relationship between bar depth and wave height as already indicated. This cannot yet be stated quantitatively because of the large number of variables involved. Kuelegan (1945) compared bar depths with trough depths and decided that the depth of bars to the depth of troughs was a fairly constant relation; i.e. the depth of the trough divided by the depth of the bar (both measured from still water level) is approximately 1.69. Shepard (1950) measured additional profiles and found that the ratio was generally less (1.16 to 1.63).

Since bars appear on tideless seas, obviously no tides are required for their formation; however, the previous statistics show that the difference in elevation between pairs of bars is nearly the same; this happens to be about the same as the tide range at those beaches. Water level is continuously changing because of the tide's surface but for about an hour before and after high and low tides, the water level is fairly constant. This means that there is considerable time for a bar to develop at a particular elevation during the high and low periods; in the transition period, the level changes too fast for the sand to follow. Because of this, the waves are able to form bars which are in use at both high and low tide; the outer bar at low tide will be the inner bar at high tide. At intermediate stages of the tide, unusually large waves will break three or more times, on each of the bars and the beach face. At lowest tide, the inner bar may be above water, or nearly so, and occasionally a ridge of sand appears on the seaward side of this bar which may be a vestigal remnant of a berm which started to form at extreme low tide. The tide range probably influences the number of bars that can exist since a large range would bring the wave action into contact with widely separated bottom areas. Steep beaches exposed to the same wave action as flat ones have fewer bars because even at lower tides the rapidly deepening bottom is below the limit of wave action except on the flanks of the existing bars. The remarks about the depth of water in which waves influence the bottom must also be modified to take into account the usual rise in actual water level during a storm above that predicted. Although the average depth of the third bar is given as 13.0 feet below MLLW, the waves that cause that bar may be several feet more. Moreover, the slope of the bottom influences the depth of breaking and a bar obviously offers a steeper slope than the barless profile. These comments are based on information obtained by making detailed soundings of bars before, during and after heavy surf conditions on a variety of beach types.

Bars have been observed on beaches ranging in size, and subject to the wave action of, model tanks, lakes, seas and oceans. Many types and sizes have been described; Hagen (1863) who first suggested their origin saw five on a single beach; Gilbert (1885) mentions bars on the shores of Lake

Michigan* that could be traced for "hundreds of miles" (he probably did not mean that these bars were continuous, but even so, it is quite remarkable). Substantially unbroken bars 20 miles long have been seen on the Washington Coast (Isaacs, 1947). It should be pointed out though that it is hard for an observer ashore to say just what the length of a bar is, since bars are most easily judged by the breakers on them; breakers are highly variable in height along the crests and a large number of waves would have to be observed in order to say whether a low point was in the bar or in the waves.

BERMS

A berm is a nearly horizontal formation of beach material brought ashore by the waves. Berms are, in a sense, the opposite of bars since they are the depositional sand form and bars are the erosional sand form; as described in detail elsewhere, the wave steepness appears to control which of the two will form. There is no general agreement as to the criteria for determining whether any beach has a berm, for on some beaches berms are difficult to recognize. If it is necessary to have a flat surface or a well-defined crest, many beaches will never have a recognizable berm. On the other hand, if the accretion of sand above water is to be the standard, obviously every beach must have a berm part of the time in order to exist above water. There appears to be no basic difference between the sand deposits that waves leave on very flat beaches (which are said by some to have no berms) and the sand deposits on intermediate slope beaches which have well-defined berms by any standard. On beach areas which have greater protection from wave action at one end than the other this is readily observed. On the most exposed portion, such a beach may have a definite berm with a rather ill-defined crest (coarse grained beaches rarely have clean-cut berms); on the part of the beach where the sand is intermediate in size and exposure is moderate, there will be a sharp crest and a flat berm top; as the most protected area is approached, this berm fades into a fairly flat beach and is no longer discernable as a definite form. It will be noted that each of these areas falls within the definition and has a "nearly horizontal formation of beach material brought ashore by the waves" - - the point is that on the beach which is always "nearly horizontal" the berm is difficult to see.

Since berms are formed by wave action, height of the crest of the berm is a function of the height of the forming waves above the sea level at the time of formation. Experiments in a wave channel by Bagnold (1940) indicate that the height of the berm is 1.3 x H_0 of the waves that formed it.

* The level of Lake Michigan varies as much as seven feet because of rainfall, evaporation, seiching and wind tides; these bars therefore are not necessarily related to the water level as observed.

Although this research has not been extended sufficiently to determine a factor for ocean beaches as yet, the author's observations (Bascom 1953) indicate that such a relationship will eventually be shown in which some factor (possibly 1.3) multiplied by the refraction coefficient will give the height of the crest of the berm at any point. For this reason, the berm formed by any one set of waves is lower in protected areas than on exposed beaches. This effect has been observed at a number of places including Monterey Bay, California where, on occasion, the crest of the berm is found to be 16 feet above MLLW on the exposed beach at Fort Ord, decreasing in the shelter of Point Pinos to 10 feet at Del Monte Creek and becoming perhaps half of that in Monterey Harbor.

Berms are depositional features placed above water by wave uprushes. The action of the wave is something like this: Great turbulence exists in the surf zone, particularly at the line of breakers, which churns up the bottom sand and maintains it in suspension. After a wave breaks, the water rushes forward up the beach carrying the suspended sand, losing velocity as it goes because it is opposed by both gravity and friction, and at some point stops completely. When the velocity drops below the threshold of transport, the suspended material deposits. Since beaches are permeable, some of the water sinks down through the sand leaving a lesser amount to return along the surface as backrush. The water that does return on the surface must start from zero velocity; consequently a large part of the suspended sand which was carried up the beach remains near the termination of the uprush. The backrush reaches a high enough velocity to remove some sand from the lower part of the beach but the balance is in favor of deposition. The saturation of the beach face largely controls the relationship between the volumes of uprush and backrush water and it is readily seen that the short period waves keep the beach wetted best because the intervals between saturations are smaller. As already noted, the short period waves increase the H/L values which are the criteria for erosion or deposition. It may be reasoned then, that a most important erosional characteristic of waves is their ability to keep the beach wetted face saturated; by the same token, berms build more readily on "dry" beaches since there is less water returning along the surface to transport the sand seaward again.

As this process continues, the sand builds seaward; since the height of the waves above sea level controls the height of the berm crest, the tide has a considerable influence on berm height. If ocean waves were all of the same height, a rapidly growing berm surface might show undulations in response to the tides. However, berms grow evenly because of the variation in the heights of waves and although the seaward growth results from the combined work of the average waves, the upward growth depends only on the largest waves. Where the berm crest is well developed, the uprushes of these largest waves pass completely over the crest and deposit the bulk of their sand load on the landward side. Since this can only occur on the highest tides, the crest grows higher as it builds seaward and the shoreward side of the crest slopes inland. This has two effects on the growth of the berm.

(1) The water rushing shoreward down the gently sloping landward side levels off irregularities by depositing new sand and scouring off high points with the result that the berm may be quite flat.

(2) This water eventually saturates the beach completely and then stands in large puddles; eventually, when enough water gathers it is able to breach the crest of the berm at the low point and flow back out to sea. Future uprushes crossing the crest deepen this cut and the result is a drainage channel, something like a "rip channel" in a bar, but forming on the upper beach.

This vertical growth of the berm caused by large waves creates an interesting paradox. Storm waves which erode a beach also build a berm during the erosion. The uprushes of the large waves carry sand up and deposit it on top of the berm, adding vertically to its top even while the face is eroding so that when the storm condition subsides, a narrow but higher berm remains. Since large storms frequently occur in the winter when the beach is narrow, these higher berms are called winter berms and are found at the landward extremity of the beach. They will survive until subaerial erosion or a larger storm removes them. A prograding beach may have a series of abandoned berms at its landward side.

Since a berm is a depositional feature, its width is dependent on the sand supply and the length of time that waves exist which are capable of moving sand shoreward. Because of the effects just described the most recent addition will frequently cover and obliterate traces of the previous berm limits. The example of the growth of the berm at Carmel Beach, California (Figure 2) illustrates this point. This particular berm disappears completely in the winter time and builds out as much as 300 feet by the end of summer.

The Growth and Retreat of a berm (Figure 2) (Vertical exaggeration 1:10) is worthy of additional explanation since it nicely demonstrates beach changes which are seldom so large or so well documented.

(1) The berm built gradually seaward at the rate of about forty feet a month during most of the summer when the light waves existed; in August and September the rate increased. No further surveys were made until December and February; by then the beach was found to have retreated almost to the cliff.

(2) The winter berm crests, formed as the beach retreated before high storm seas, were several feet higher than the summer berm crests.

(3) The growth of the berm from 20 August 1946 to 4 September 1946 was upward and outward partly because the height of high tide increased during that period (until the 26th) and partly because the waves were slightly higher.

(4) Each successive addition to the berm was higher so that the surface sloped landward to a run-off channel.

Newly formed additions to the berm usually have a rather soft surface compared to recently eroded surfaces of about the same slope; this softness is sometimes helpful in deciding whether the beach is eroding or building. At Clatsop Plains, Oregon, a very flat beach where clear cut berm features rarely if ever exist, it was noticed one day that a DUKW traveled 20 mph slower along the beach than it had at the same place on the previous day although no beach changes were evident visually. A survey showed that a foot of new sand had come ashore during the night.

RIP AND RUN-OFF CURRENTS

Two transverse depressions that often dissect beaches are the rip channel and the run-off channel which cross the bar and the berm, respectively. These channels are cut by currents which perform parallel functions (one above water, one below) i.e., the return seaward of water raised above average water level by wave action. Potential energy gained by this thin sheet of water raised along a length of beach is spent in eroding a narrow channel as it runs back to sea.

The actual amount of sand moved by these currents is secondary compared to their effect on the flow mechanism in the surf zone. In a sense they act as safety valves which drain off the excess head. The run-off channel behind the berm crest, for example, does not operate until the berm is completely saturated and water stands in a large puddle or lagoon. When the water surface has reached sufficient height to breach the lowest part of the berm, the lagoon water rushes out to sea reducing the head on the water percolating thru the beach. The rip channel and its feeders have a more serious effect on the inshore bottom. A bar must form much more rapidly if no rip current exists since all the water moved shoreward in translation must flow back out to the bar carrying sand with it. As soon as the rip forms, however, the returning water and suspended sand will flow seaward only as far as the trough where they will join the rip feeder current and move parallel to the shore until the rip itself is reached. The rip current flows seaward thru its channel in the bar (at velocities as much as four knots); outside the surf zone it slows down, breaks into eddies and drops its sand to form a delta.

CHARACTERISTICS OF NATURAL BEACHES

NATURAL FORCES

The offshore-onshore motion of sand which has just been discussed is more easily observed and described than the longshore motion. This is because one can think of offshore-onshore motion as operating within a closed system, that is, the amount of sand involved is constant. Sand measurements plotted as profiles do not give information about the flow of sand across the profiles, they merely indicate the height of the "standing crop" and on a yearly basis the profile will show no net movement, Longshore sand movements are nearly always irreversable and therefore usually result in large net movements of sand from a length of coast which is eroding to a comparatively small place where deposition takes place (usually a sand spit). Such sand movements characterize coasts which make a large angle with the fronts of incompletely refracted swell approaching from the usual storm centers or which have local winds that blow steadily enough to maintain wind-waves with a constant direction. It is the coastwise momentum given to the inshore water by these waves striking the beach at an angle that drives the current which moves the sand. The development of means for dealing with this littoral movement caused by waves striking the beach at an angle is the principal coastal engineering problem today.

Longshore currents need not be of sufficient velocity themselves to pick up and transport sand, for in the surf zone the sand is put into suspension by the forces accompanying wave transformation and very small currents are effective in moving the suspended material sidewise. Wave steepness, discussed in detail later, is an important factor in determining the rate of littoral drift. Higher waves effect the bottom deeper and keep the sand in suspension over a wider area; shorter period waves will be less refracted and strike the beach at a greater angle. The volume of material transported is therefore seen to be dependent on the height, period and attack angle of the waves as well as on the nature of the materials involved and the variations in the character of the zone of transport.

The paths of the materials in transport are of interest since the motion takes place on the beach as well as below water. Underwater the suspended materials sway back and forth in the wave orbits and presumably move in a roughly sinusoidal path at nearly the velocity of the littoral water current. On the beach face, particles oscillate with the uprush and backrush to create a figure which is best described as a series of skewed parabolas connected at their limbs.

Munch-Petersen (1938) has likened this littoral process to a conveyor belt the width of the surf zone whose belt speed is the current velocity. The accumulated deposit at the end of the belt may be very large. For example, Rockaway Spit at the entrance to New York Harbor grew at the rate of 200 feet per year for a long period (one mile in 23 years).

Man, in building shoreline structures that obstruct or influence littoral sand flows almost invariably creates a dual problem: (1) sand will

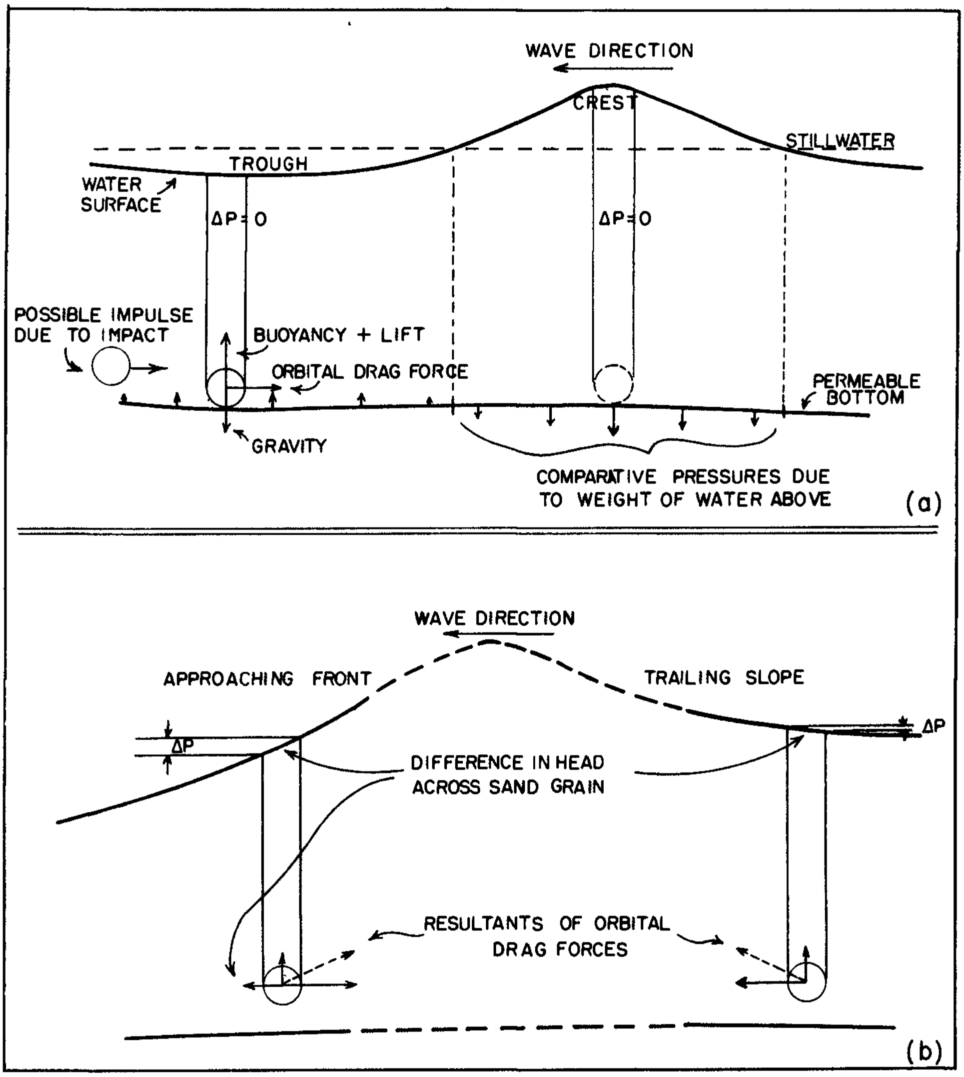

Fig. 3. Forces on a sand particle.

deposit wherever comparatively quiet water is created and (2) cut off the supply of sand to beaches downstream from the structure which will retreat.

FORCES ON A SAND PARTICLE

A description of the characteristics of beaches is made difficult by their constant change and although it is an easy matter to define or describe the sand forms, it is not yet possible to say exactly which forces cause the shifting or how. In spite of the great mass of wave records, surf observations, and detailed beach surveys obtained in the last few years the level of knowledge is not much above that which has existed for a hundred years: "storm" waves move sand from berm to bar; "calm" seas move it back.

Since the means by which the individual sand grains are moved about is the heart of the problem it is perhaps worthwhile to examine the forces in some detail.

Consider the complex environment of a single grain of sand on the surface of the inshore bottom subject to the action of almost-breaking waves. (See Figure 3). Although it is held down by gravity it is buoyed up by the medium it displaces (a slurry of sand and water with a density greater than that of water) and consequently the net downward force is comparatively small. A wave approaches. As the trough passes the pressure on the sand grain is relaxed, because of the decrease in head, while at the same time the bottom under the crest is subjected to additional pressure. Consequently there is flow through the bottom depending on its permeability and water will emerge under the wave trough and tend to lift the particle. This flow, when it exists, is reflected in the next-to-bottom orbits. Immediately above an impervious bottom these orbits will parallel the bottom; above a permeable bottom through which there is flow the orbit will be warped and take on a vertical dimension. At the same time drag forces resulting from the orbital current acts to move the grain seaward and lift forces (due to the particle's convex upper surface) tend to raise it. Velocity is now at a maximum, acceleration a minimum, and the water surface above the grain nearly horizontal. This grain, on the bottom may also receive an impulse by being struck by other grains slightly above the bottom and moving with higher velocity.

A few seconds later the steep face of the approaching wave is directly over the grain which is now slightly above the bottom. At this time there is differential pressure on opposite sides of the sand grain due to the slope of the water surface above; this will have a tendency to move it landward with the wave form. The horizontal seaward velocity has now diminished and a vertical component of orbital force influences the particle to move upward.

As the crest passes, upward velocity of the orbital currents is a maximum as is pressure on the bottom; here again no pressure gradient due to surface slope exists. Horizontal acceleration is at a maximum as the velocity goes to zero and then reverses direction.

The trailing slope of the wave crest, less steep than its front, creates a reverse but somewhat smaller pressure gradient across the sand grain; at this time high orbital velocities landward and slightly upward are attained.

As the cycle is completed the orbit of the water particles starts downward again; its velocity (and consequently its available drag force) is reduced and the sand grain is returned to the bottom. Does it return to its point of beginning each cycle?

Additional complexities are caused by waves (often several trains arriving at once), which are highly variable in height and period and strike the beach at various angles. The roughness and slope of the bottom, the size, shape, and weight of the particle and the depth each has a certain effect on the flow and the intensity of wave forces on the bottom.

The problem is to decide whether the sand grain sustained any net movement or not and in which direction.

It is evident from the changes in the beach forms that the net force must change, causing sand to migrate; however as yet one cannot say definitely which combination of these minute forces is dominant at any given time. Certain generalizations are possible about the relative magnitude of forces and the chances of their ascendency.

Most important is the fact that objects immersed in water tend to do what the water around them does --- especially particles whose resistance is high compared with the net downward force (low settling velocity). Thus, a grain of sand churned into suspension by the highest breakers of a group of waves will often settle slowly enough to partake of the orbits (and the mass transport) of small succeeding waves. In addition the particle will be subject to any alongshore motion which the water mass may have as it is driven by wind or by translatory waves moving at an angle with the shore.

Waves which are actually breaking must be of considerable importance in moving sand grains; here again one is limited to qualitative descriptions. Two events associated with breaking are readily recognizable as being influential. The first of these is the effect of plunging breakers in sending a jet of water down to the bottom through the preceeding trough; that is, the water mass flung forward by the plunge maintains enough integrity to reach the bottom and churn up sand (a motion readily observed by swimmers).

The other effect of breaking is the change from the orbital state where comparatively little forward motion (mass transport) takes place, to translatory motion in which the water moves forward with the wave form. The water in translation moves rapidly and turbulently and carries with it whatever items may be in suspension. Since the waves which run up the beach face are almost always translatory, this probably accounts for most of the actual motion of sand upwards onto the berm.

WAVE STEEPNESS

There is some evidence that sand always moves in the same direction as the wave form unless the usual driving force is overridden by some temporarily stronger force in the opposite direction but no critical experiments in actual wave conditions have yet proven this point. As previously discussed, this usual driving force is probably attributable to either a mass transport effect or to the differential pressures due to the slope of the water surface. The opposing forces, whatever they may be, are obviously dominant at the time of storm waves (usually defined by their steepness, H_o/L_o). According to Johnson (1949), model tank experiments show that a wave steepness of greater than 0.03 will always cause a bar to form and a steepness of less than 0.025 will never cause one to form. The validity of these numbers at any actual beach is somewhat doubtful but the trend seems quite clear. It seems unlikely that the steepness itself is effective in reversing the sand flow; more probably some other characteristic acting at the same time is responsible.

The meaning of wave steepness is deserving of discussion since it is generally considered to be an important criteria in determining the direction of transfer of beach materials. Steepness (H_o/L_o) is the ratio of the deep water height to the square of the period (T^2) times a constant ($g/2\pi$) (shallow water-steepness is proportional). Therefore the steepness can increase either with an increase in height or a decrease in period. A greater steepness means a greater slope on the wave front and consequently a greater pressure gradient between trough and crest and a greater driving force on the water in the orbit. This gradient, as previously indicated, must be increasingly effective in (1) causing flow through the bottom and (2) in exerting differential pressure on the individual sand grains (in both directions but with a net force landward). Another result of steepness is the rate of delivery of water to the beach as indicated by a decrease in T. Short period waves allow little time for settling between waves and consequently a greater amount of material is maintained in suspension.

An even more important effect of steepness is the comparative unstableness of the crest which makes it more likely to break. The greater percentage of breakers results in a greater transport of water to the beach by translatory motion which results in the raising of a "head" of water in the nearshore zone. Since the rate of delivery of water to this area is high, the surf zone water level actually slopes upward to the shoreline;

this water naturally seeks to return seaward and there are two paths by which it travels. It either flows transversely outward through or along the bottom, or it moves parallel to the beach in the innermost trough (as a feeder current) until it reaches a rip channel whence it flows to sea through a breach in the bar. Both of these currents must be responsible for sand movement but only in the case of the rip current (which sometimes makes a delta) is it readily observed.

On the beach face the importance of wave steepness is related to the permeability of the sand. Think of a single uprush moving up a dry beach face; it is a small translatory wave and carries sand grains with it. As it goes some of the water sinks through the sand and percolates down to join the water-table. At the maximum extent of the uprush, little water is left and consequently there is no transporting medium to move the sand grains back down the beach face. The water that does return along the beach face must start from zero velocity (below the threshold of sand transport) and consequently a large part of the suspended sand which was carried upward remains near the termination of the uprush. If the rate of delivery of water and sand is slow (small steepness), there is time for this percolation to take place and each uprush adds to the sand on the beach. If the steepness is large, the rapid succession of waves maintains a saturated beach and the uprush water returns down the beach face carrying much of its load with it and scouring additional sand from the lower face as it goes.

SUMMARY

In summary, beaches may be said to have the following principal characteristics:

1. They are composed of constantly shifting groups of particles which move and orient themselves to fit changing waves and currents.

2. These particles make up the two major beach forms: the berm (above water) and the bar (below water). The transfer of material between the two seems to be dependent on the wave steepness.

3. The relief of these features and the slope of the beach face is related primarily to the particle size but also to the character of the waves which formed it.

4. Sand will move in the same direction as the wave form except when some local condition, such as a bottom current flowing in the opposite direction, becomes predominant.

5. Where the dominant wave system strikes the beach at an angle,

a littoral current is set up that maintains an irreversable flow of sand along the coast. This sand will deposit when it reaches comparatively quiescent water---thus causing serious coastal engineering problems.

The great number of small forces which act in various ways on the sand grains make the study of beach dynamics a complex one and the lack of definite quantitative information at the present time about the causes of sand migration make a paper such as this a mere "state of the art" discussion. It is hoped that engineers and observers will be challenged sufficiently by the indicated missing pieces of information to devise critical field experiments that will end speculation about these unscalable phenomena. The answers must be sought in the natural environment where the engineer must work; persistent effort will some day give man control over the beach environment.

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CHAPTER 11

FIELD INVESTIGATION OF SUSPENDED SEDIMENT IN THE SURF ZONE

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INTRODUCTION

Purpose - This paper presents the results of a number of field observations made at Pacific Beach near San Diego, California with a suspended sediment sampler. A detailed description of the laboratory and field development of the sampler is presented in the Beach Erosion Board Technical Memorandum No. 34 entitled "Development and Field Tests of a Sampler for Suspended Sediment in Wave Action". The laboratory development involved: a circulating system which provided various current velocities and concentration patterns at the test section; the testing of various size nozzles; the study of particle size distribution of samples obtained by the nozzles; and the development of correction factors for field conditions. It was concluded from the laboratory study that a pump-type sampler could be adapted to the study of suspended material movement in wave action. The principal result from the laboratory tests was a tentative finding that by pumping through a vertically disposed 1/2-inch nozzle with a velocity approximately twice the maximum orbital current velocity in a wave, samples could be obtained which were representative in weight (even without a correction factor) to within about 15 per cent of the true suspension.

When the Field Research Group of the Beach Erosion Board was making a study of shore line changes in the Mission Bay area, near San Diego, California, from March 1949 to March 1951, opportunity was afforded to make field tests of a suspended sediment sampler designed in accordance with the laboratory findings. The purpose of the field program was threefold, as follows:

a. To test the adaptability of the suspended sediment sampler to use off a pier in open water;

b. To determine the suspended sediment concentration at various points in and immediately outside the surf zone over as wide a range of wave conditions as practicable; and

c. To analyze the results of sampling to obtain an indication of whether or not the suspended load is of sufficient magnitude to play a significant role in the alongshore transport of littoral materials.

Description of Area - The Mission Bay area is shown on Figure 1. It lies between the La Jolla and Point Loma headlands. The shore area includes Pacific, Mission and Ocean Beaches. Pacific and Mission Beaches, which appear to be essentially stable, extend southward from the La Jolla headland in a gently curving arc to the jetties at the entrance to Mission Bay. Throughout most of that length the beach is broad and flat with the crest of the beach berm approximately 11 feet above mean lower low water. Seaward from the beach berm the slope is relatively steep, gradually becoming flatter at the approach to the low tide terrace. A low bar is usually present seaward of the terrace.

Tidal Data - The tides are of the mixed type with a diurnal inequality. The mean range of tide in this locality is about 3.6 feet, and the mean diurnal range is about 5.2 feet.

Wave Characteristics - From January to December 1950, visual observations were made twice daily of the wave height, period, and direction. During part of 1950 an underwater pressure-type wave gage was operated from Crystal Pier at Pacific Beach. Observed and recorded wave data were supplemented by hindcast wave data using synoptic weather charts for 6-hour intervals. The results have been compiled into a wave diagram (Figure 2), which presents an estimate of deep water wave conditions for 1950. Because of the lack of weather data for the region south of latitude 15° North, the southern limit of hindcast waves was about 260° azimuth. Also, the northern limit of observed directions was approximately 290° azimuth; waves on the graph with directions north of 290° azimuth were hindcast. Since these latter waves were not observed, it is possible that diffraction around offshore islands altered the direction of waves before they reached Pacific Beach. Thus the sector of wave approach actually observed has as its limits azimuths of 180° and 290°. Percentages of time shown total more than 100 percent since often two or more wave systems occurred simultaneously.

Suspended Sediment Sampler - The sampler was designed to gather a sediment sample by pumping a quantity of sediment-laden water from a selected point. The water was discharged back into the ocean after passing through a filter which removed the sediment. The amount of water pumped was measured by a meter connected in series with the filter.

The sampler and appurtenances are shown on Figure 3. The apparatus consists of a 1/2-inch intake nozzle, a filter case, a modified filter core, standard check valve, a submersible pump, a standard pipe tee and plug for priming, and a water meter (modified by filling the dial chamber with light oil and replacing the glass face plate with a lucite face plate). The filter paper was 10 ply, Z-fold embossed; the openings in the paper being rated as passing only solids of less than 25 microns diameter. When within 1 to 4 feet of the bottom, the intake nozzle opening was positioned with respect to the ocean bottom by means of a positioning unit which consisted of a round plate attached to the sampler by iron supports. The plate had a spur which penetrated the ocean bottom thereby eliminating any latteral movement of the sampler during operation. The sampling unit was lowered into and removed from the water by means of a block and tackle.

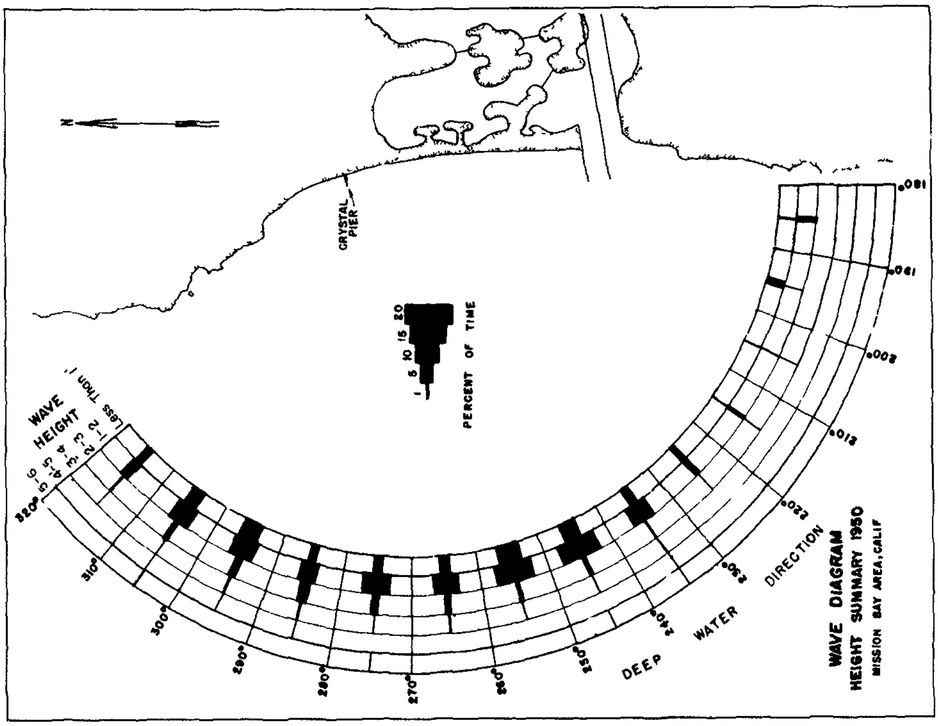

WAVE DIAGRAM
HEIGHT SUMMARY 1950
MISSION BAY AREA, CALIF
DEEP WATER DIRECTION
WAVE HEIGHT
CRYSTAL PIER
PERCENT OF TIME

Fig. 2.

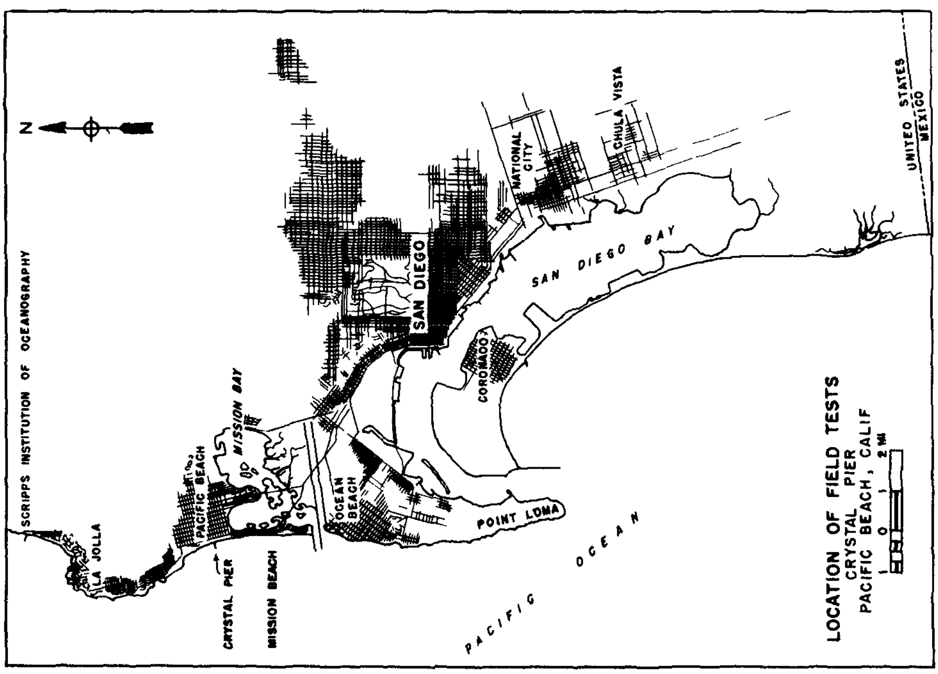
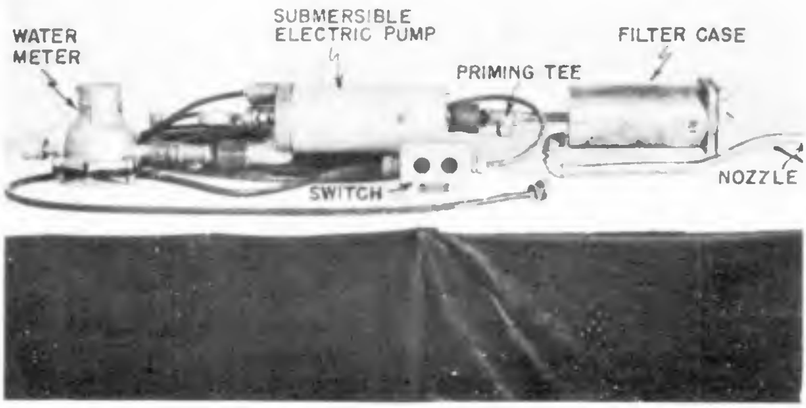

LOCATION OF FIELD TESTS
CRYSTAL PIER
PACIFIC BEACH, CALIF
SCRIPPS INSTITUTION OF OCEANOGRAPHY
LA JOLLA
PACIFIC BEACH
CRYSTAL PIER
MISSION BEACH
MISSION BAY
OCEAN BEACH
POINT LOMA
PACIFIC OCEAN
SAN DIEGO
SAN DIEGO BAY
CORONADO
NATIONAL CITY
CHULA VISTA
UNITED STATES
MEXICO

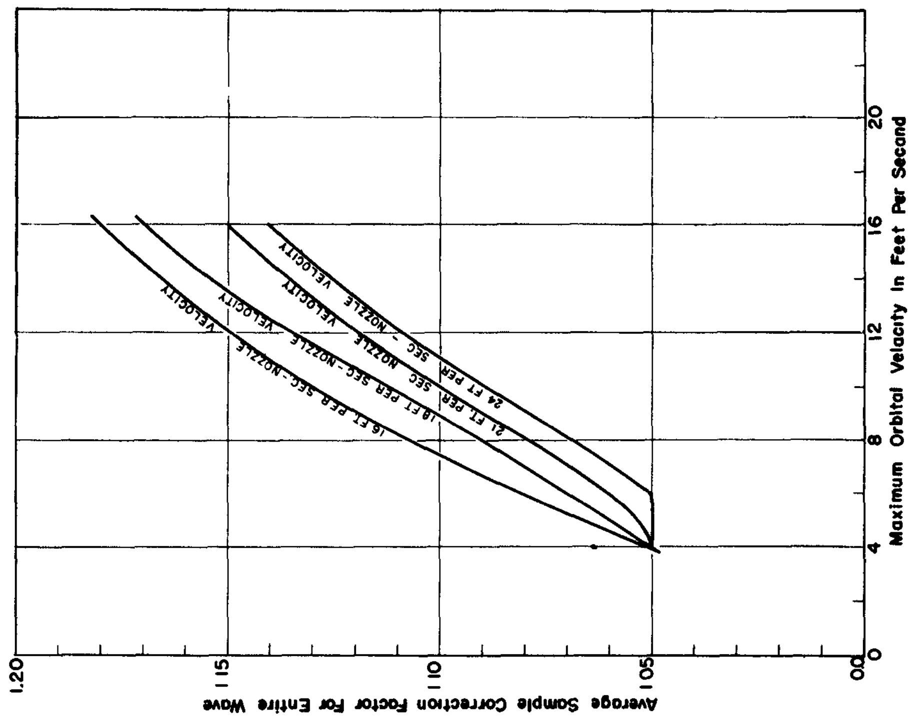
Fig. 1.



3-a. Component Parts Of Sampler.

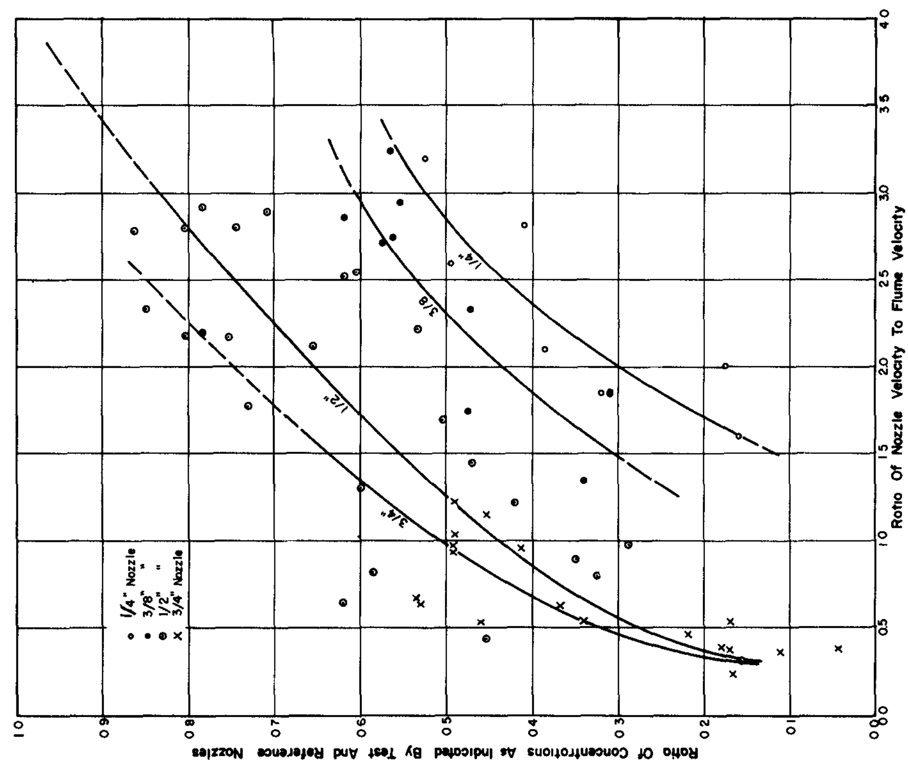
3-b. Sampler And Hoist

SUSPENDED SEDIMENT SAMPLER
Fig. 3.



CORRECTION FACTOR FOR ENTIRE WAVE IN BREAKER ZONE

Fig. 5.



SAMPLING EFFICIENCY

Fig. 4.

The efficiency curve for the sampler equipped with a 1/2-inch diameter intake nozzle was determined from laboratory tests and is shown on Figure 4. The sampler was designed to pump with a nozzle velocity of about 18 feet per second. The laboratory development tests indicated that the sampler will pump at an average sampling efficiency of 94 percent when the internal orbital velocity of the wave is from 0 to 5 feet per second. Therefore, for that part of the wave cycle in which the internal orbital velocity is less than 5 feet per second, a sampling efficiency of 94 percent can be assumed. For nozzle velocity-current velocity ratios less than 3.5 the sampling efficiency falls off rapidly, being only 44 percent when the ratio is unity. In view of these findings a correction factor was developed (Figure 5) which varied with the internal current velocity in the wave.

FIELD TESTS

Procedure - All suspended sediment samples were taken from Crystal Pier, a structure located at Pacific Beach. The pier, shown in an aerial photograph on Figure 6, is approximately 1,000 feet in length. Due to limitations of personnel and time for hydrographic survey work at Mission Bay, the suspended sediment sampling program was conducted only on days of poor visibility, excessively rough seas, or when it was impractical to attempt hydrographic work. Consequently, sampling was done only for a limited number of wave conditions. Samples were taken on 23 days between 15 January 1950 and 15 May 1951. The total number of samples procured was 290. Samples taken with an intake nozzle velocity less than 15 feet per second were not included in the compilation since these low intake velocities were generally the result of seaweed or debris clogging the nozzle which would greatly influence the accuracy of the indicated sample concentration. Most of the samples were obtained landward of the breaker line since the waves generally broke before reaching the seaward end of the pier. Although 52 samples were obtained seaward of the breaker line, 30 had intake nozzle velocities less than 15 feet per second. The 22 acceptable samples were insufficient in number to make any detailed study in this zone. Of the 238 samples taken landward of the breaker line, 170 were acceptable. A typical field data sheet, Figure 7, illustrates the information recorded for each sample.

Pumping Time Per Sample - The influence of pumping time for an individual sample was given careful consideration. It was believed that sampling should be continuous during the passage of at least 15 to 20 wave crests to obtain a representative sample. As noted in the wave summary study there were frequent times when inconsistent or combination wave trains approached the shore thereby creating a rather complex wave period record. However, it appears on the average that a wave period of approximately 13 to 15 seconds prevailed. On this basis it was assumed that a sampling duration of 5 minutes would extract samples of the suspended material from the sea over approximately 15 to 20 wave passages and should provide a representative sample for the prevailing wave characteristics. An analysis of samples taken over a period of approximately 10 minutes indicated a considerable reduction in intake nozzle velocity due generally to the head loss in overloading the sampler filter.

1 FEB. 1951 SCALE 1:5,000

CRYSTAL PIER, PACIFIC BEACH, CALIF.

Figure 6

SUSPENDED SAND DETERMINATION
Crystal Pier
Mission Bay, California

Sta. of observation 9+75 — Sample No. 63
Sta. of breaker line 8+50 — Date 7 Mar. 50
Sta. of uprush limit 2+50 — Time 1340
Sample taken (~~shoreward~~) (seaward) of breaker line.
Estimated wave height at sampling point 2 ft
Estimated wave period at sampling point 15 sec
Water depth at sampling point 11.3 ft
Height of intake nozzle above bottom 9 ft
Duration of run 5 min 0 sec (5.00 min)
Meter reading after run 637.9
Meter reading before run 631.0

Water pumped 6.9 cu ft (x 64.0 =) 442 lbs sea water
Rate of pumping 1.38 cu ft per min
Intake nozzle velocity (cu ft per min x 12.3) 17.0 ft per sec
Max. orb. wave velocity (from curves) 1.7 ft per sec (from ~~recorded~~ estimated wave data)
Correction factor for this sample (from curves) 1.05
Weight of sample less foreign matter, oven-dry 21.6 grams (x 0.0022) 0.0475 lbs
Corrected weight of sample 22.7 grams (x 0.0022) 0.0499 lbs
Parts of sand per thousand parts of water by weight 0.113
Parts of sand per thousand parts of water by volume 0.043
(by weight x 0.379, assuming sp. gr. sand at 2.70 and sea water at 1.025)

Recorded wave height 1.0 ft
Recorded wave period 14.1 sec
Time of wave record 1500
Depth of water at recorder 29.5
Wave direction (observed from shore station) 270°
Time of wave direction observation 1500
Type of breaker Plunging
Littoral drift direction N
Littoral drift velocity 15 ft per min
Time of littoral drift observation 1200
Median grain size of nearest bottom sample 0.165 mm
Sample number of nearest bottom sample 69
Other data on bottom sample
Median grain size of nearest beach sample 0.170 mm
Sample number of nearest beach sample 68
Other data on beach sample
Median grain size of suspended sand sample 0.117 mm
Description of foreign material in sample

REMARKS

FIELD DATA SHEET

FIG. 7

Data Obtained - All data obtained on sediment concentrations in individual samples taken landward of the breaker line are given in Table 1. In view of the inaccuracies of estimated wave heights, it was necessary to group the data into classes. The data in Table 1 are grouped by classes of wave heights, water depths and sampling elevations from the bottom. An arithmetical mean concentration is derived for each water depth - sampling elevation - wave height combination.

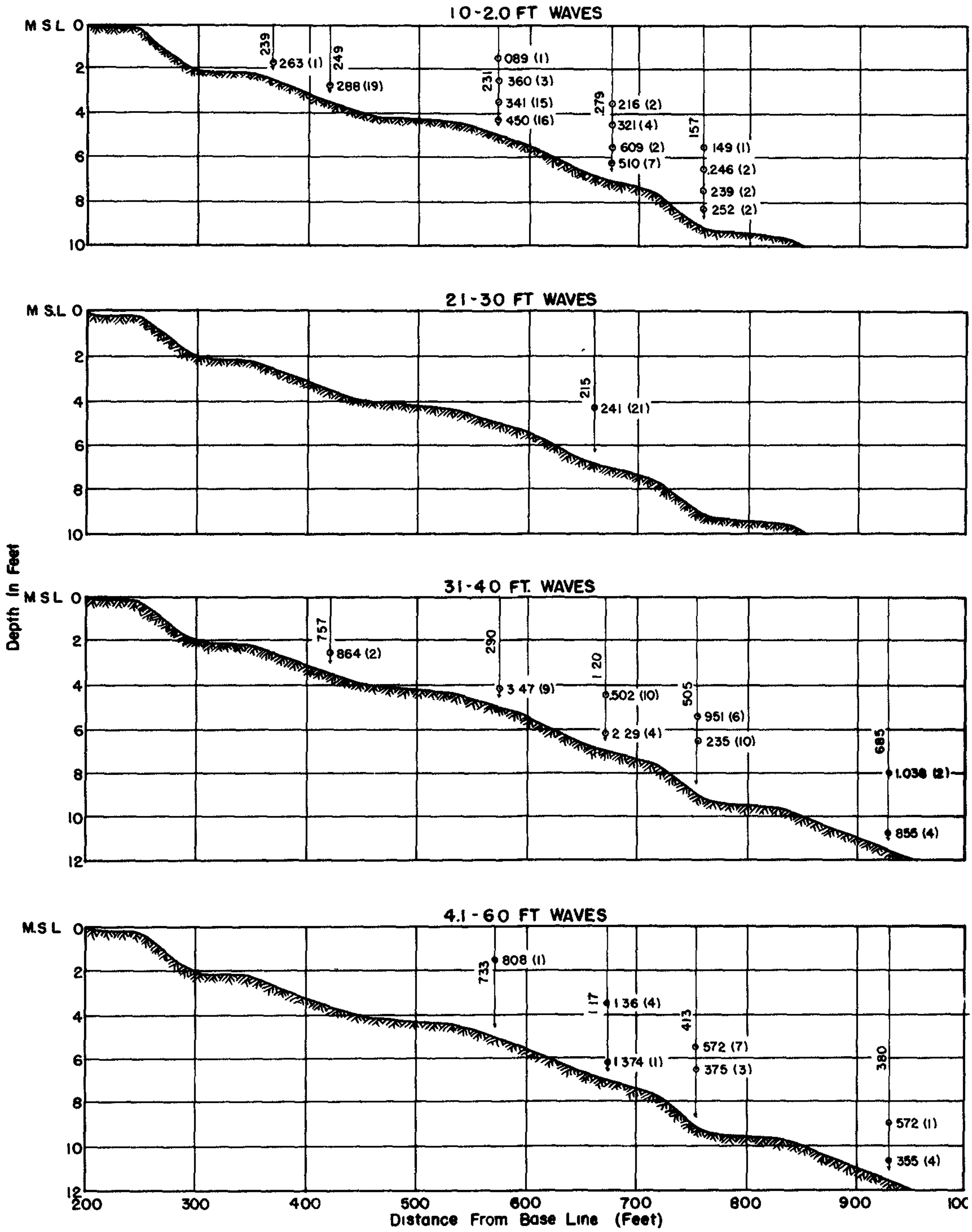
PRESENTATION OF DATA

Concentration Distributions Along Profile - The mean concentration values from Table 1 were plotted (according to their respective water depth -- sampling elevation -- wave height classes) in relation to a hydrographic profile which is representative of the Crystal Pier location. Figure 8 shows the plotted concentration values for four wave height classes. Where concentration values are indicated at the various depths at a station on Figure 8, an average concentration value for that station also is indicated. This average concentration value for each indicated station was derived by plotting the concentration values between the water surface and 0.5 foot from the bottom, (plots with individual values not shown) then drawing a curve to define the vertical concentration distribution. In establishing the curve, consideration was given to the evidence found in Figure 10 which indicates that the concentration is fairly constant between about two-tenths and six-tenths of the depth from the bottom.

The average concentration at each station for each wave height class as established in Figure 8 was plotted as shown on Figure 9. These plots of the suspended sediment data represent the average concentration profiles. Curves of visual best fit have been drawn for various wave height classes between the limits of available data. They have been extrapolated thence to the mean sea level shore line and to the 11-foot depth contour for purposes of estimating material movement past the profile.

Data for all acceptable samples are given in Table 2 by Z/H ratios and wave height classes; Z being the distance from the bottom to the nozzle intake, and H being the total water depth at the sampling station. The arithmetic means of the concentration values for each wave height class were then plotted against Z/H as shown in Figure 10. These plots represent the vertical concentration profile when all Z/H values are considered for each wave height class.

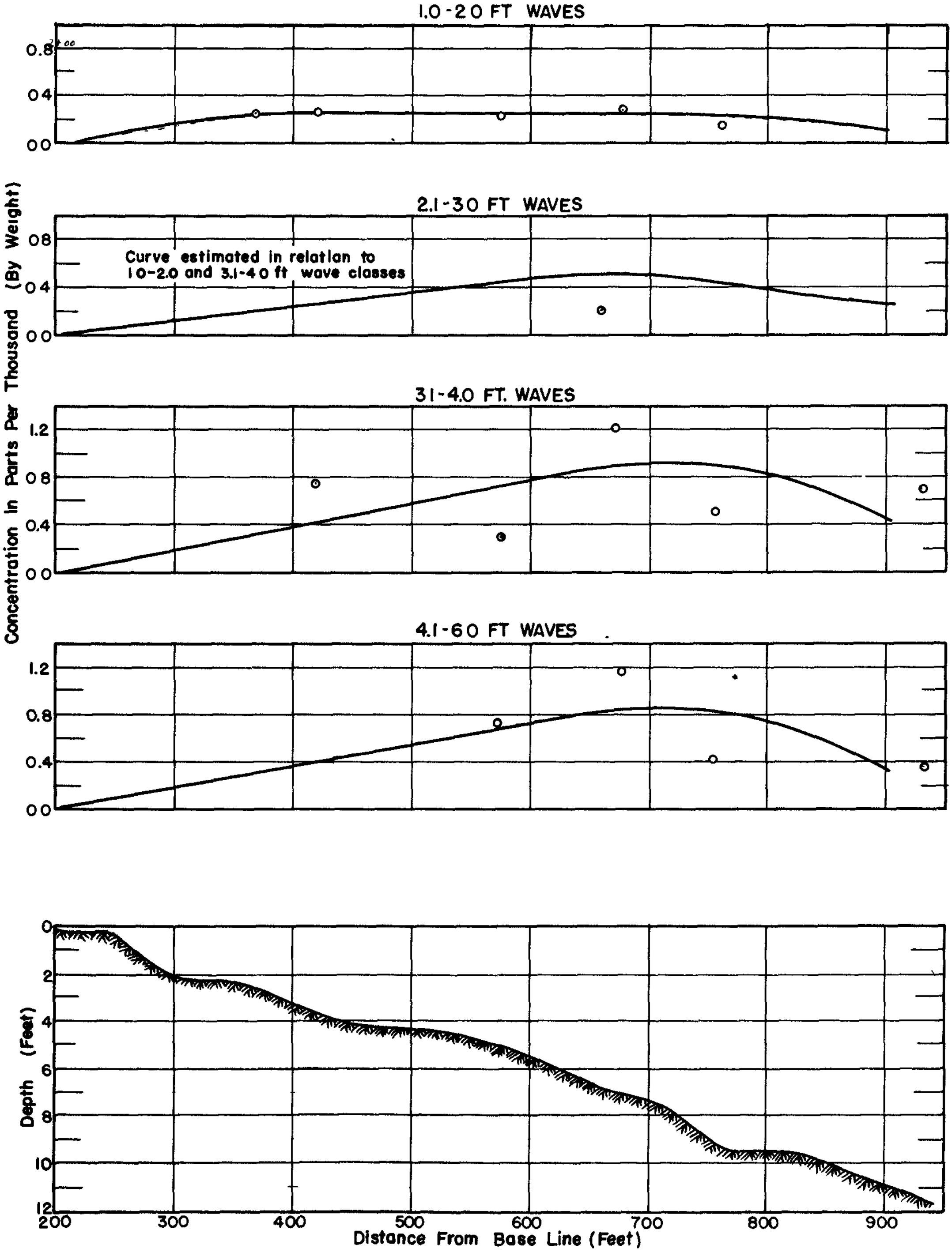
Total Material in Suspension - In order to investigate the average concentration distribution shown on Figure 9 in terms of total material in suspension, a tabulation of volumes of sand per linear foot of shore, between the shore and the 10-foot depth (Stations 250 to 850), is presented in Table 3. As can be seen Table 3 utilizes the suspended sediment concentrations, as arrived at from the sampling program, to deduce the average amount of material in suspension in the surf zone, (between the shore and the 10-foot depth contour) for various wave height classes. The volume of material in suspension was computed as $V\rho_w \not{c}/\rho_s$



NOTE Each point is the average concentration in P.P.T of all samples taken at that particular water depth and elevation above the bottom. The number in brackets by each concentration value is the number of samples averaged to arrive at the indicated concentration. The average concentration between the water surface and 0.5 ft from the bottom is indicated at each sampling station. (see fig. 19)

SUSPENDED SEDIMENT CONCENTRATIONS ALONG PROFILE

Figure 8



NOTE: Plotted points indicate average concentrations between the water surface and 0.5 ft from the bottom. (From fig. 18)

CONCENTRATION DISTRIBUTION ALONG PROFILE
Figure 9

Table I

Concentrations of Individual Samples For Various Combinations of Water Depth - Sampling Elevation - Wave Height Classes

Water Depth Class (ft)	Sampling Elev. Class (ft. from bottom)	Wave Height Class (ft.)	Concentrations of Individual Samples In Parts Per Thousand by Weight	Arithmetic Mean
2.0 - 3.0	0.6 - 1.0	1.0 - 2.0	0.263	0.263
3.1 - 4.0	0.6 - 1.0	1.0 - 2.0	0.098, 0.142, 0.154, 0.180, 0.183, 0.234, 0.235, 0.266, 0.268, 0.273, 0.287, 0.290, 0.295, 0.318, 0.324, 0.367, 0.399, 0.420, 0.740	0.288
	0.6 - 1.0	3.1 - 4.0	0.651, 1.077	0.864
4.1 - 6.0	0.6 - 1.0	1.0 - 2.0	0.031, 0.086, 0.122, 0.131, 0.163, 0.193, 0.224, 0.225, 0.274, 0.324, 0.352, 0.430, 0.773, 0.867, 1.442, 1.570	0.450
	0.6 - 1.0	3.1 - 4.0	1.460, 1.980, 2.170, 2.340, 2.910, 3.410, 4.430, 4.610, 7.910,	3.47
	1.1 - 2.0	1.0 - 2.0	0.070, 0.072, 0.083, 0.093, 0.168, 0.204, 0.219, 0.237, 0.287, 0.288, 0.329, 0.455, 0.860, 0.874, 0.877	0.341
	2.1 - 3.0	1.0 - 2.0	0.123, 0.429, 0.529	0.360
	3.1 - 4.0	1.0 - 2.0	0.089	0.089
	3.1 - 4.0	4.1 - 6.0	0.808	0.808
6.1 - 8.0	0.6 - 1.0	1.0 - 2.0	0.179, 0.252, 0.416, 0.450, 0.582, 0.646, 1.045	0.510
	0.6 - 1.0	3.1 - 4.0	1.150, 2.500, 2.540, 2.960	2.29
	0.6 - 1.0	4.1 - 6.0	1.374	1.374
	1.1 - 2.0	1.0 - 2.0	0.558, 0.661	0.609
	2.1 - 3.0	1.0 - 2.0	0.194, 0.234, 0.333, 0.524	0.321
	2.1 - 3.0	2.1 - 3.0	0.155, 0.165, 0.171, 0.172, 0.175, 0.177, 0.181, 0.204, 0.206, 0.208, 0.230, 0.232, 0.238, 0.244, 0.247, 0.252, 0.274, 0.316, 0.358, 0.384, 0.482	0.241
	2.1 - 3.0	3.1 - 4.0	0.182, 0.197, 0.240, 0.330, 0.426, 0.427, 0.565, 0.590, 0.736, 1.330	0.502
	3.1 - 4.0	1.0 - 2.0	0.188, 0.244	0.216
	3.1 - 4.0	4.1 - 6.0	1.002, 1.290, 1.310, 1.840	1.36
8.1 - 10.0	0.6 - 1.0	1.0 - 2.0	0.233, 0.272	0.252
	1.1 - 2.0	1.0 - 2.0	0.120, 0.358	0.239
	2.1 - 3.0	1.0 - 2.0	0.026, 0.467	0.246
	2.1 - 3.0	3.1 - 4.0	0.126, 0.152, 0.182, 0.220, 0.221, 0.244, 0.246, 0.257, 0.314, 0.388	0.235
	2.1 - 3.0	4.1 - 6.0	0.324, 0.375, 0.424	0.375
	3.1 - 4.0	1.0 - 2.0	0.149	0.149
	3.1 - 4.0	3.1 - 4.0	0.712, 0.756, 0.908, 1.058, 1.133, 1.140	0.951
	3.1 - 4.0	4.1 - 6.0	0.363, 0.443, 0.471, 0.577, 0.703, 0.715, 0.732	0.572
10.1 - 13.0	0.6 - 1.0	3.1 - 4.0	0.403, 0.541, 0.547, 1.930	0.855
	0.6 - 1.0	4.1 - 6.0	0.244, 0.258, 0.318, 0.604	0.355
	2.1 - 3.0	4.1 - 6.0	0.572	0.572
	3.1 - 4.0	3.1 - 4.0	0.880, 1.196	1.038

Table 2

SEDIMENT CONCENTRATIONS OF SAMPLES BY Z/H AND WAVE HEIGHT CLASSES
Note: Z = Distance from bottom to nozzle intake; H = Water depth at sampling point;
Concentrations in part per thousand by weight

Z/H Class	0.06 to 0.10	0.11 to 0.15	0.16 to 0.20	0.21 to 0.25	0.26 to 0.30	0.31 to 0.35	0.36 to 0.40	0.41 to 0.45	0.46 to 0.50	0.51 to 0.55	0.56 to 0.60	0.61 to 0.65
					1.0 - 2.0 Ft. Wave Class							
	0.233	0.252	0.031	0.120	0.098	0.026	0.083	0.072	0.070	0.123	0.188	0.089
	0.366	0.272	0.086	0.122	0.154	0.142	0.237	0.149	0.093		0.244	
	0.377	0.461	0.163	0.131	0.180	0.168	0.287	0.194	0.204		0.429	
	0.768	0.646	0.179	0.224	0.235	0.183	0.327	0.234	0.288			
	[illegible]	1.016	0.193	0.225	0.290	0.263	0.860	0.333	0.329			
			0.274	0.266	0.318	0.273		0.542	0.529			
			0.324	0.268	0.324	0.455		0.874	0.877			
			0.358	0.287	0.324	0.661						
			0.430	0.295	0.420							
			0.450	0.352	0.558							
			1.442	0.367	0.740							
			1.570	0.399								
				0.773								
				0.867								
Avg	0.436	0.535	0.458	0.335	0.331	0.271	0.359	0.342	0.341	0.123	0.287	0.089
					2.1 - 3.0 Ft. Wave Class							
	0.133						0.149	0.155	0.238			
							0.177	0.165				
							0.206	0.171				
							0.230	0.172				
							0.232	0.175				
							0.244	0.181				
							0.247	0.204				
							0.252	0.208				
							0.274	0.358				
							0.316	0.482				
							0.384					
Avg	0.133						0.246	0.227	0.238			
					3.1 - 4.0 Ft. Wave Class							
	0.403	1.150	1.46	0.651	1.077	0.126	0.152	0.590	0.712			
	0.541	2.50	1.98	2.34		0.182	0.182	0.756	1.058			
	0.547	2.54	2.17	3.41		0.220	0.197	0.908				
	1.93	2.96	2.91	4.43		0.221	0.240	1.130				
				4.61		0.246	0.244	1.140				
				7.91		0.257	0.330	1.33				
						0.314	0.426					
						0.388	0.427					
							0.565					
							0.736					
							0.880					
							1.196					
Avg	0.855	2.288	2.13	3.892	1.077	0.244	0.465	0.976	0.885			
					4.1 - 6.0 Ft. Wave Class							
	0.244	1.374			0.572	0.324	0.042	0.363		1.002		0.808
	0.258					0.375	0.471	0.443		1.29		
	0.318					0.424	0.577	0.703		1.31		
	0.604							0.715		1.84		
								0.732				
Avg	0.356	1.374			0.572	0.374	0.363	0.591		1.361		0.808

Table 3

MATERIAL IN SUSPENSION PER LINEAR FOOT OF SHORE BY WAVE HEIGHT CLASSES

Station Limits Along Profile	Average Water Depth Within Station Limits	Volume of Water Within Station Limits	WAVE HEIGHT CLASSES 1.0 - 2.0 ft. Avg. Conc.	1.0 - 2.0 ft. Mat'l in Susp.	2.1 - 3.0 ft. Avg. Conc.	2.1 - 3.0 ft. Mat'l in Susp.	3.1 - 4.0 ft. Avg. Conc.	3.1 - 4.0 ft. Mat'l in Susp.	4.1 - 6.0 ft. Avg. Conc.	4.1 - 6.0 ft. Mat'l in Susp.
Ft.	Ft.	Cu. Yds.	P.P.T.	Cu. yds.	P.P.T.	Cu. Yds.	P.P.T.	Cu. yds.	P.P.T.	Cu. Yds.
				$x10^{-4}$		$x10^{-4}$		$x10^{-4}$		$x10^{-4}$
250 - 350	0.9	3.33	0.15	3.2	0.12	2.6	0.19	4.1	0.19	4.1
350 - 450	2.5	9.27	0.23	13.8	0.25	15.0	0.38	22.7	0.38	22.7
450 - 550	4.0	14.80	0.23	22.0	0.35	33.4	0.57	54.4	0.57	54.4
550 - 650	4.8	17.80	0.23	26.4	0.45	51.7	0.79	90.7	0.70	80.4
650 - 750	6.5	24.00	0.23	35.6	0.50	77.4	0.90	139.3	0.85	131.6
750 - 850	8.5	31.40	0.20	40.5	0.39	79.0	0.82	166.1	0.75	151.9
			Totals	141.5		259.1		477.3		445.1

Table 4

INDICATED ANNUAL SUSPENDED LITTORAL DRIFT FOR ASSUMED NET LITTORAL CURRENT VELOCITIES

Velocity of Littoral Current (Assume Direction Constant)	WAVE HEIGHT CLASSES 1.0 - 2.0 ft. Suspended Material Passing Unit Width (%Yearly Occurrence of Wave Ht. Class = 60)	2.1 - 3.0 ft. Suspended Material Passing Unit Width (%Yearly Occurrence of Wave Ht. Class = 19)	3.1 - 4.0 ft. Suspended Material Passing Unit Width (%Yearly Occurrence of Wave Ht. Class = 4)	4.1 - 6.0 ft. Suspended Material Passing Unit Width (%Yearly Occurrence of Wave Ht. Class = 1.3)	Total Littoral Drift
Ft. Per Min.	Cu. Yds. Per Year	Cu. Yds Per Year	Cu. Yds. Per Year	Cu. Yds. Per Year	Cu. Yds. Per Year
1	4462	2587	1000	304	8353
5	22310	12935	5000	1520	41765
10	44620	25870	10000	3040	83530
15	66930	38805	15000	4560	125295
20	89240	51740	20000	6080	167060
25	111550	64675	25000	7600	208825
30	133860	77610	30000	9120	250590
40	178480	103480	40000	12160	334120
50	223100	129350	50000	15200	417650
60	267720	155220	60000	18240	502180
70	312340	181090	70000	21280	584710

Note: The percentages of occurrence of wave height classes total 84.3% of time, the remaining percentage comprises

Table 5

MEDIAN DIAMETERS OF SUSPENDED SEDIMENT SAMPLES

Z/H Class	0.00 to 0.05	0.06 to 0.10	0.11 to 0.15	0.16 to 0.20	0.21 to 0.25	0.26 to 0.30	0.31 to 0.35	0.36 to 0.40	0.41 to 0.45	0.46 to 0.50	0.51 to 0.55	0.56 to 0.60	0.61 to 0.65	0.66 to 0.70	0.71 to 0.75	0.76 to 0.80	0.81 to 0.85
1.0 - 2.0 Ft. Wave Class																	
	0.118	0.116	0.131	0.160	0.136	0.120	0.130	0.120	0.112	0.120	0.162	0.108	0.130	0.138	0.170		0.141
	0.090	0.139	0.113	0.143	0.125	0.150	0.163	0.140	0.130	0.172		0.119					
	0.180	0.145	0.155	0.184	0.093	0.145	0.115	0.152	0.122			0.130					
	0.187	0.100	0.118			0.093	0.118		0.164								
	0.096	0.098	0.112				0.092		0.085								
	0.103	0.097	0.135														
	0.098	0.099	0.150														
	0.099		0.096														
	0.107		0.098														
Avg	0.120	0.113	0.123	0.162	0.118	0.127	0.124	0.137	0.123	0.146	0.162	0.119	0.130	0.138	0.170		0.141
2.1 - 3.0 Ft. Wave Class																	
	0.200	0.120	0.122	0.150	0.141			0.141	0.114	0.170		0.163					0.152
	0.160	0.122			0.143					0.157							0.149
										0.141							
Avg	0.180	0.121	0.122	0.150	0.142			0.141	0.114	0.156		0.163					0.151
3.1 - 4.0 Ft. Wave Class																	
		0.170	0.190	0.200	0.195	0.189	0.170	0.190	0.190								
		0.173		0.160	0.172		0.169		0.170								
Avg		0.172	0.190	0.180	0.184	0.189	0.170	0.190	0.180								
4.1 - 6.0 Ft. Wave Class																	
		0.190	0.184			0.190	0.160	0.185	0.165		0.160						
			0.184				0.160		0.165		0.157						
							0.190		0.190								
Avg		0.190	0.184			0.190	0.170	0.185	0.173		0.174						

V - Volume of water in cubic yards
ρ_w - Density of sea water in lbs per cubic yard
ϕ - Concentration in parts by weight
ρ_s - Bulk density of sand, taken as 2700 lbs per cubic yard

Although the degree of accuracy of this computation cannot be stated with certainty at this time, it is believed that the amount of material in suspension indicated by these computations is of the correct order of magnitude.

Indicated Annual Suspended Littoral Drift - The next step, shown in Table 4, introduces the yearly percentages of occurrence of the various wave height classes and an assumed net rate of alongshore current to illustrate the net rate of alongshore drift which could be attributed to suspended material (as contrasted to creep, or bed load transport). It is recognized that the assumptions behind these computations are rather broad and it is not intended that these results be accepted for quantitative application to shore erosion studies. However, it is believed that the rate of longshore drift indicated by the computations serves to show that the suspended load is potentially a sizeable factor in the longshore drift picture.

Grain Size of Suspended Sediment - Data on the median diameters of a number of samples obtained are given in Table 5. The presentation of data in Table 5 is similar to that in Table 2 with respect to Z/H ratios and wave height classes.

ANALYSIS OF RESULTS

Adaptability of Sampler - In this series of tests approximately 71 percent of the total number of samples obtained were employed in studying the suspended material movement. This could be considered as a relatively poor sampling efficiency; however, the sampling efficiency for this particular type of sampler will be a function of the local conditions. In the area where the samples were taken, there was on occasions a considerable amount of eel grass in suspension which clogged the sampler intake nozzle and reduced the intake nozzle velocity so that the sample was of questionable value. The number of acceptable samples would be increased where only a nominal amount of this type of foreign material was in suspension at the sampling point.

No satisfactory operational procedure has been developed for this sampler which would facilitate the procurement of samples other than from a fixed structure. The shore structure undoubtedly had some influence on the sample results, but the magnitude of such influence could not be evaluated. Precautions were taken to obtain samples at a point as far from a structural member of the pier as possible, the sampler also being positioned about 10 feet away from the pier. Sampling was done from the side of the pier toward the direction from which waves were approaching.

Table 1 shows that when a number of samples were taken with a specific water depth, sampling elevation, and wave height class, the maximum and minimum values of sample concentrations frequently differed by a factor of 3 to 5. This difference is appreciable and serves to show that the suspended concentration pattern, in relation to time, must be exceedingly complex. The spread in concentration values might be expected to become somewhat less if the class limits (water depth, sampling elevation, and wave height) were decreased. However, the following tabulation is presented to illustrate that repetitive sampling (samples taken as often as possible, under essentially identical conditions) seems to indicate a similiar spread in concentration values, therefore the order of magnitude of spread in concentration values for the class limits used could be expected.

TABLE 6 - REPETITIVE SAMPLING DATA

Estimated wave height at sampling point - 3 feet
Water depth at sampling point - 6.8 feet
Height of intake nozzle above bottom - 3 feet

22 Jan 1951, Time	1354	1404	1414	1428	1445	1454	1505
Concentration of Sample (P.P.T. by Wt.)	0.204	0.155	0.175	0.208	0.482	0.171	0.165

Although the concentration values vary appreciably, it is believed that they indicate the range between the limits of which the true value probably lies; the true value probably not being greatly different than the mean of the group.

When the mean values of concentration for various depth and wave height classes are plotted as shown in Figure 8, it can be seen that many more samples would be desirable in order to establish the average concentration profile at each station for each wave height class. Nevertheless, the average concentration value computed for each station indicated in Figure 8 seems to provide a logical and reasonable concentration pattern when plotted as shown in Figure 9. Approximately 70 percent of the suspended sediment data falls into the 1.0 to 2.0 and 3.1 to 4.0-foot wave height classes and there seems to be a reasonable correlation in Figure 9 for these wave height classes. The 2.1 to 3.0-foot wave height class contains only one point and this falls slightly above the average concentration for the 1.0 to 2.0-foot wave height class. The line indicating an average concentration profile for this wave height class is undoubtedly questionable, but was sketched in relation to the 1.0 to 2.0 and 3.1 to 4.0-foot classes, for use in the computations in Table 3. The scattered data for the 3.1 to 4.0-foot and 4.1 to 6.0-foot wave classes in Figure 9 do not indicate any significant difference between the average concentration profiles for the two classes. The fact that the 4.1-6.0 wave class concentration profile does not indicate greater concentrations is probably due to lack of data.

The concentration profiles in Figure 9 indicate that the greatest amount of material, in this area, is thrown into suspension between the 4 and 8-foot depth contours which is the area slightly landward of the breaker line. There is some evidence that the difference of the average concentration of suspended material at any station between the breaker line and approximately the 2-foot depth is not great; rather it could be more of a uniform concentration of suspended material between these two points. This fact seems to be brought out in the data tabulated in Table 2 and plotted on Figure 10. Here concentration values at all stations on the profile for each wave height class have been plotted against Z/H classes. For each wave height class, this plot tends to indicate that at any station along the profile there is a depth range where the concentration is fairly uniform and since Z/H values are used for all depths, this uniform concentration zone would extend throughout the surf area. For the 1.0 to 2.0-foot wave height class the range of uniform concentration extends from the two-tenths to the six-tenths depth; the 2.1 to 3.0-foot wave height class has an insufficient number of points and the limits of the range cannot be established; for the 3.1 to 4.0 and 4.1 to 6.0-foot wave height classes the lower portion of the range is indicated to be around the three or four-tenths depth, the upper limit cannot be established due to lack of data.

Littoral Drift Computations - Although additional samples would have been desirable, the 170 acceptable samples used for this study seem to present a reasonable concentration distribution when the results are resolved into averages. The overall accuracy of the average concentration values cannot be evaluated at this time, therefore the accuracy of the quantitative computations in Tables 3 and 4 cannot be assessed. As far as is known, no other method has been developed to date, that will give any indication as to the magnitude of the concentrations of suspended material in the nearshore zone. The results, as shown in Table 3, indicate that the suspended load is potentially a sizeable factor in the movement of material alongshore.

Grain Size of Samples - The data were studied to determine if a correlation between grain size in suspension and distance from the bottom could be established for various wave characteristics. No trend toward any relationship of this type is apparent from the number of observations taken in this study. It appears that many samples must be taken at each station along a profile under a wide variation of wave characteristics in order to establish this relationship. An analysis of the beach and bottom samples at or near Crystal Pier indicates that the beach and offshore bottom sediments can be divided into three size classes. At the intersection of the plane of mean tide level with the beach the median diameter of the sand is about 0.22 millimeter; from this zone to the 20-foot depth the median diameter of the sand is about 0.15 millimeter; and from the 20-foot to the 50-foot depth the median diameter is about 0.10 millimeter. The suspended sediment samples were taken, in general, landward of the 13-foot depth and analysis of all the suspended samples indicates an average median diameter of about 0.14 millimeter, which compares favorably with the 0.15 millimeter sand size found from mean tide to the 20-foot depth contour.

CONCLUSIONS

The field tests of the suspended sediment sampler indicate that the number of acceptable samples procurable with the sampler is dependent on local conditions. Where excessive foreign material is present in suspension, it may clog the nozzle and make the sample unusable. It was found that only 71 percent of the total samples procured at Crystal Pier could be employed in evaluating the data. By averaging the data from acceptable samples a reasonably logical correlation between suspended sediment concentration, water depth, and wave height can be made. The results presented for this series of observations are not of a high degree of accuracy but tend to indicate that the total suspended material movement can be an important factor in a littoral drift analysis.

CHAPTER 12

THE DEVELOPMENT OF A SAND BEACH BY DEEP-WATER WAVES

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ABSTRACT

In a previous paper**, it was shown that the mechanism of the trochoidal waves can be used to determine the equilibrium slope of a sand beach under any wave conditions. As a start it was assumed that the beach material was of uniform grain size, and that the waves approached the beach directly with all motion in planes at right angles to the shore line.

In the present paper, the application of the theory is shown in the development of various sand and gravel beaches. The equilibrium theory is studied in the light of the fact that there is usually considerable transportation of material along the shore. In particular, attention is called to the characteristics of beaches with rounded or pointed contours, of beaches whose ends are closed off by rocks or cliffs, or whose ends are open and extend into deep water without barriers of any kind.

A method of study and analysis is demonstrated which can be applied to all beaches. Finally, it is shown that an accurate forecast of the natural development of a beach can be made on the basis of the equilibrium slope equation, as well as a forecast of the effect of any structure placed in a naturally developing beach.

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* Manuscript not submitted

** The Effects of Waves on a Sand Beach, by Harold Flinsch, Proceedings Minnesota International Hydraulics Convention, Minneapolis, Minn., September 1953.

CHAPTER 13

EFFECT OF ICE ON SHORE DEVELOPMENT

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INTRODUCTION

During the course of investigations of ice forms on the Great Lakes bordering the state of Michigan, the writers had occasion to observe several shore areas under winter conditions. The following paper is a general consideration of these casual observations and includes suggestions of the probable relationship between ice conditions at the shore line and in the surf zone to the normal shore processes effective during the ice-free year.

No systematic investigations of ice and its role in shore processes have been made by the writers; it is to be emphasized that the data recorded here were obtained in connection with other investigations not directly concerned with the processes of beach development, and are presented here only because they seem to indicate a field in which little or no previous studies have been made.

GENERAL ICE CONDITIONS OF THE GREAT LAKES BORDERING MICHIGAN

None of the Upper Great Lakes (Superior, Michigan, and Huron) is known to have frozen over completely within historical time, although the ice-free part of the lakes may be of greater or lesser areal extent in different years. Some bays freeze over entirely, and frequently an ice cover will form along a straight coast for a considerable distance out from shore. The thickness of this ice cover is also variable but usually ranges from 6 inches to 2 feet, depending on latitude and local climatic factors. The ice season begins in late November or December, reaches a peak sometime in late January or February, and ends in late March or April. One can thus expect ice conditions to prevail during a period of $3\frac{1}{2}$ to $4\frac{1}{2}$ months of the year.

Between 1949 and 1953, the writers have observed segments of shore under normal winter conditions of Lakes Superior, Michigan, and Huron as a by-product of investigations sponsored by the Snow, Ice, and Permafrost Research Establishment (SIPRE) of the U. S. Corps of Engineers. On Lake Huron, observations in the vicinity of Alpena, Rogers City, Cheboygan and Mackinaw City were made (fig. 1). On Lake Michigan, ice conditions on the north shore between St. Ignace and Manistique were seen; and in Lake Superior, winter shore conditions were recorded for the south shore of Whitefish Bay between Brimley and Point Iroquois.

FORMATION OF ICE-FOOT

The geographic location of the upper Great Lakes and their unusual size combine to produce a characteristic winter condition along "normal" shores. Normal shores are those with an underwater profile approaching equilibrium conditions, that is, they are not of the type bordered by steep bedrock cliffs, nor are they of the unusually flat and swampy kind.

Beginning with the period of sub-freezing temperatures, spray produced in the surf zone is blown onto the foreshore and frozen. Eventually, through a continuation of this process, the frozen spray produced a mass of ice firmly attached to the foreshore. This is called the <u>ice-foot</u> (fig. 2A).[1] Requirements for its formation are twofold: (1) sub-freezing atmospheric temperature; (2) open water bodies that, because of their size, remain ice free well into the season of sub-freezing temperatures. The second factor precludes any formation of an ice-foot on inland lakes, even large ones such as Houghton, Mullet and Black; because the time between the beginning of sub-freezing temperatures and the time of the complete freeze-up of the lake is not sufficiently long to permit the development of an ice-foot.

After the ice-foot becomes firmly established, it may be subjected to some modification by wave action. This involves either continued accretion of frozen spray or erosion by waves. Eventually, however, the water surface in contact with the base of the ice-foot may become frozen so that all further processes effective against the foot will cease. At the water edge of the ice-cover that now extends out from shore, a new ice-foot may develop. Sometimes the new ice-foot is initiated by the presence of an ice ridge formed by the shingling or jamming of broken ice blocks along the open water edge of the sheet ice. Spray on this ridge cements it firmly together, thus producing a new ice foot. For the purposes of easy reference, this second type of ice-foot is called the <u>off-shore ice-foot</u> in contrast to the <u>shore ice-foot</u> (fig. 2B). If the ice-foot borders expanse of open lake water, it is an <u>active ice-foot</u>; an ice-foot locked firmly in frozen lake ice is <u>inactive</u> (fig. 2C).

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1 The term <u>ice foot</u> was used to designate "the part of the fast-ice (ice <u>in situ</u> along a seacoast) immediately close to shore that is not affected by the rise and fall of the tide." <u>Parentheses are ours</u>. Although this definition is not strictly applicable to fresh water ice, it is the only term previously used that seems appropriate for the ice form herein described as the <u>ice-foot</u>.

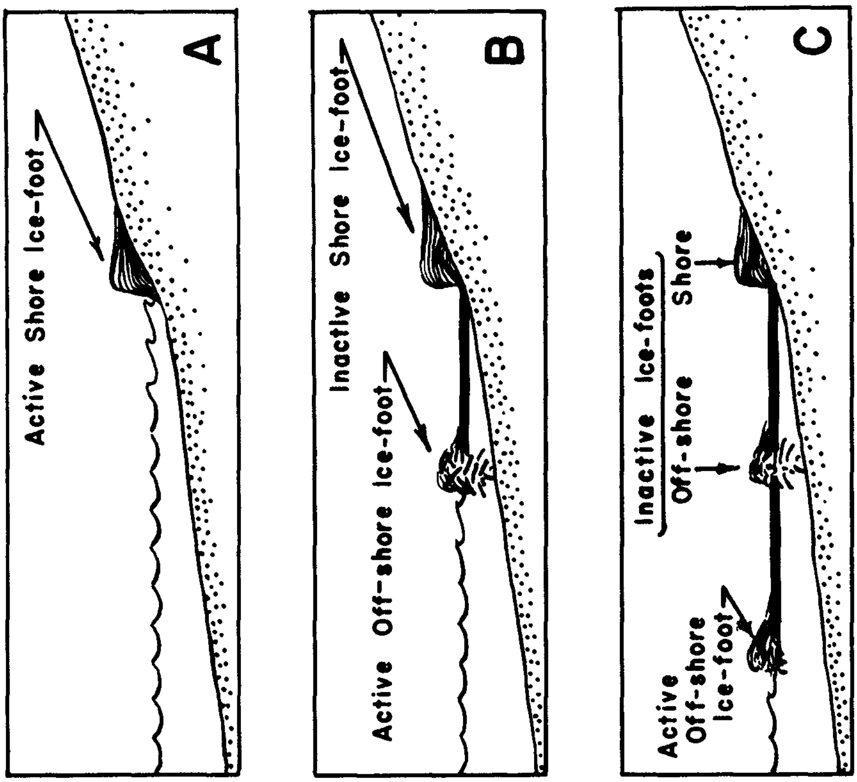

Fig. 2 Diagramatic sketch showing the development of the various types of ice-foots.

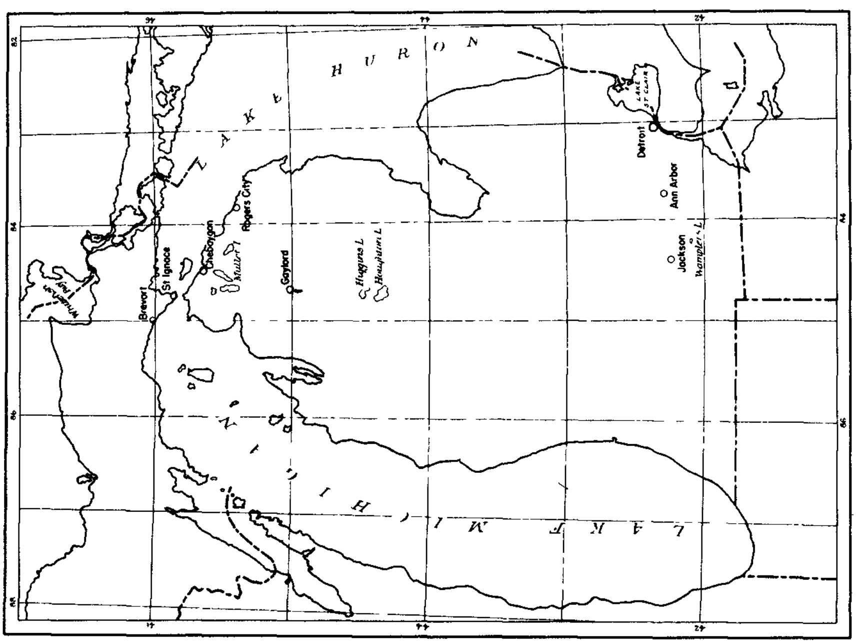

Fig. 1 Index map.

In shallow water of the surf zone the off-shore ice-foot may be grounded. Furthermore, the off-shore ice-foots may be multiple, since the first off-shore foot is liable to stabilization in the same manner as the shore ice-foot, thus permitting growth of a second off-shore foot at some point further out. It is not known whether the position of the off-shore ice-foots is controlled by water depth, or whether their place of occurence is a random event.

POSSIBLE EFFECT OF THE ICE-FOOT ON SHORE PROCESSES

Waves breaking on shore or currents moving parallel to the shore account for most of the processes of shore development. The extent to which the presence of an active ice-foot of the shore or off-shore type modifies normal shore development is not well known, but some theoretical considerations are noteworthy in that they have been supported by casual observation.

Effect of ice-foot on wave attack — Once established, the active shore ice-foot can be regarded as a protective feature since it takes the bulk of the impact of the breaking wave. Sandy beaches and beaches consisting of unconsolidated bedrock are thus given some degree of protection against wave attack. The protective action of the ice-foot is not entirely effective, however, because sand grains imbedded in the ice-foot indicated that some wave scour does take place at the base of ice-foot. This sand is apparently derived from the bottom of shallow water zones and is incorporated in the ice-foot by waves breaking against it. Even on the active off-shore ice-foots, similar evidences of wave scour have been noticed.

The question arises as to the magnitude of this scour in comparison with the scour that is produced under ice free conditions. One conclusion seems justified, even on theoretical considerations alone; the development of multiple ice-foots is tantamount to the moving of the zone of wave scour away from the shore line. Or conversely stated, no wave scour is possible on an inactive ice-foot.

The presence of an active shore ice-foot must upset the equilibrium of the subaqueous profile, because it renders a large part of the available sand immobile as far as movement by direct wave or current action is concerned. This condition requires a readjustment of the bottom profile, since less sand is available for the wave system than during ice-free periods.

Assuming, then, that the profile is altered because of the presence of an ice-foot, the question immediately arises as to the degree of alteration and permanency of the change. Is the change in profile insignificant insofar as the "normal" profile is concerned? How quickly is the altered winter profile reverted to the profile of the ice free year? Is it possible that the conditions imposed upon the regimen of the wave system by the ice-foot could have permanent effects on the general nature of the shoreline? In other words, how do shore lines, formed under conditions of a three to four month ice season, differ from shore lines formed under exactly the same conditions except for the ice?

A closer examination of ice-foots as well as an investigation of ice conditions in general with respect to the shore processes might yield the necessary data to permit the answering of these and other questions.

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Part 3
SHORELINE PROTECTION PROBLEMS

CHAPTER 14

PRINCIPLES OF SHORE PROTECTION FOR THE GREAT LAKES

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The Great Lakes region is one of quiet challenge and absorbing interest to the coastal engineer. From the broad pattern of its morphologic history to the detail of its present-day shoreline it presents a fascinating variety of natural phenomena and man-imposed regimen that has controlled and still conditions its shoreline behavior. Many of the features of the behavior pattern and its controls are recognized and subject to beneficial management; some of these are either not recognized or are ignored by coastal experts who should know better; while others are as yet beyond the capabilities of the methods of beneficial management available today.

It is the purpose of this paper to examine and describe some of the elements controlling the shore environment of the Great Lakes region, to the end that a better understanding of the problems in coastal engineering in this region may be had. That such understanding is needed must be apparent when one reviews the relics of attempts to control beneficially the shores of the Great Lakes, or when one attempts to evaluate the formerly existing resources now hidden by the waters that ruthlessly exploited the weaknesses of coastal works of the past. In fact, the distinguishing achievement of coastal engineering in the Great Lakes region probably has been the almost unvaried attainment of inefficient or ineffective coastal works. When one applies the rigorous criterion of engineering excellence - that of requiring a completely satisfactory technical solution at a minimum cost - to the works in this region one is lead to concede that the engineering of works in the Lakes shore has been something less than distinguished. The few notable exceptions to this vicious generalization are outstanding; among them one must cite certainly the exceptionally successful treatment of the Chicago shore, and the extensive but largely unused harbor at Cleveland.

These remarks are not necessarily a diatribe in support of a contention that past designers of Great Lakes coastal works have not been adequate to the task - though this must have been true in many cases. They are rather more a model of the passionate feelings of many property owners and taxpayers whose scanty financial resources have been expended on structures that either didn't perform their intended function, or did so at the expense of aggravating other problems.

Can the coastal engineer today serve more adequately the needs of his clients than did his predecessors? The answer is affirmative, if he avails himself of existing knowledge and analyzes the problems involved by engineering methods rather than by analogy or pseudo-scientific hocus-pocus. What, then, are the features of the Great Lakes that are pertinent to the coastal engineer, and what are the design principles he must employ?

Basically the Great Lakes coastal engineer has to solve a management problem, in that he must so guide, control, and manipulate a very few dynamic forces as to achieve an equilibrium of the type he desires. Not only in the Great Lakes region - but as well anywhere, the shore situation is the result of the dynamic action of two fluids, air and water, on an essentially static body, the land.

Whatever the coastal engineer's problem in the Great Lakes, be it protection against erosion, provision of a beach, design of a harbor, or maintenance of a navigable channel, the solution usually lies in management of the movement of sand and similar material on the shore face, and in control of wave action. Other features, such as drainage, enter some problems; however, the great majority fall in the general category described.

Before we can manage the natural regimen of the Lakes to our benefit we must know the regimen intimately. Let us then first define the dynamic features, i.e. those features that provide the energy to move the inherently static shorematerial and to damage coastal works. Initially it must be recognized that the Lakes are essentially calm most of the time. For four to six months of the year ice on the lakes or at the shore prohibits wave and current action. During the open or non-ice season the wave action in the Lakes is strictly limited. According to statistics published by the Beach Erosion Board, waves are less than 0.5 ft. high in Lake Erie for about 80% of the open season; in Lake Michigan calm prevails for about 60% of the open season; and in Lake Ontario it is calm about 80% of the time.

When waves do occur they appear to be related closely to local storms over the Lakes and exist for about the same length of time. Waves of height 6 to 8 ft. occur on the average not oftener than once a month and exist in terms of hours rather than days. Rarely, say once or less a year on the average, waves as high as about 10 ft. occur and exist for relatively short periods.

Thus, for a preponderant part of the time the shore environment is practically static, and any significant movement of material or stress due to waves must occur during the isolated short intervals when wave action is relatively violent. Further it is recognized that the structural design of any works to be built must be based on two types of forces; the ice forces (viz. loading, uplift, pressure movement); and the wave forces (impact and water flow). Therefore, the coastal engineer's problem is to manage wave forces of moderate magnitude that endure for short periods, and ice forces that may be active continuously for long periods. Regarding these latter our state of ignorance is appalling.

A second important feature of the shore environment is the nature of the land, which is a resource to be conserved and the source of the material whose beneficial or contrary influences are to be managed. A review of the morphologic history of the area, of which the most important period to our discussion is the glacial lake period, establishes the following features as pertinent to the coastal engineer. In general the land is heterogeneous in character; at the shore it may be erosion-resistant rock, or glacial lake moraine, or lacustrine deposits. The two latter are easily eroded, they have been the source in the past and presently provide all the material now moving on the lake shores, since the contribution of material by drainage features can be shown to be negligible. The coastal engineer must define the nature and source of the material in his area of interest before he is in position to approach the analysis of his problem.

The principal conclusion to be drawn from past studies of the shore geology in the Lakes region is that, as a general rule, the economy of the material situation is one of scarcity. Local abundance of material occurs rarely and is usually obvious because of its rarity.

Another pertinent feature is that of the effects of wind. Aside from its obvious contribution in the generation of lake waves, the wind is important in the Lakes area as the cause of short-term variations in lake level elevation, and as a medium for the removal of material by wind transportation from the shore face. Of these the wind set-up contribution is believed presently to be the more important.

The last pertinent feature of the Lakes environment is the variation in lake level elevations. This is important in that it defines the water-land boundary of prime concern to the coastal engineer. Various patterns of behavior of lake level elevation have been formulated and are available from the Corps of Engineers or the Beach Erosion Board. Their chief utility derives from the fact that a shift in lake level moves the locus of action on the shore, and by this motion alone may occasion or remedy a shore problem. From the point of view of design of shore works the one element of importance is the short-term duration and frequency of occurrence of the levels of elevation. Here published data are deficient, for it is to be noted that the important consideration is the probability of short-term (i.e. order of hours) concurrent occurrence of wave action and high water. Thus design based on average lake levels may be seriously deficient for the few hours of concurrent occurrence of high waves and wind set-up or raised lake level that is required to cause damage. Perhaps the destructive combination may exist for only a few hours in a long period of years. The engineer's problem is to decide what combination to select for design purposes that is neither too conservative nor dangerously radical and yet economical.

The author suggests that a probability of one occurrence of two hours duration in 10 years represents one of several reasonable design criteria. Suppose, for example, that at Erie, Pennsylvania 10 ft. high waves may be expected to occur for a two hour period once in ten years and at the same location, a short period rise of lake level of about two ft. above the maximum monthly average level of the open season may be expected once in 10 years. Following the suggested criterion the acceptable design conditions would be 10 ft. high waves, occurring concurrently with a lake level two ft. above the maximum monthly average lake level of the open season.

In summary the author believes that the problem of design of Great Lakes coastal works is one of design for some selected, infrequent, concurrent occurrence of maxima of wave action and lake level elevation in reference to a localized area of action, and with consideration given to long-period ice effects as they may control structural requirements. The works may serve little or no useful purpose for a major part of their life, perhaps as much as 80% of the time, yet their construction be justified by their benefits during periods as short as two hours in an interval of ten years. The close correlation of economic analysis and engineering design is apparent and represents one of the very difficult problems to be solved.

Let us turn our attention now to some particulars of the Great Lakes shore environment and coastal works design.

It has been mentioned that wave action in the Great Lakes is characterized by its sporadic occurrence related closely to the storm regimen. Available evidence leads to the belief that waves are predominantly steep, short in length, and of short duration. The probability is very good that the maximum design waves are of the order of 10 ft. high, 250 ft. long, and with durations of the order of 12 hours maximum. Appreciable wave action seldom persists for longer than two days at a time, followed usually by a period of essential calm. Swell, in the sense applicable to oceanic wave phenomena, probably never occurs although some "ground swell" may occur as a short time forerunner or remnant of a lake storm. In general the wave action builds and subsides quickly, following the generating storm life closely. For design purposes it appears that information on the duration times and frequency of occurrence of various wave heights is indispensable to sound engineering of coastal works in the Lakes region. Such data is not available generally but its importance would seem to warrant some appreciable effort toward its development.

It is not sufficient for purposes of rigorous design analysis to know only duration times and frequency of occurrence. Equally important is knowledge of the time element involved in wave damage to shores or structures. For example, is a jetty or seawall damaged by the occurrence of one 10 ft. high wave, or must 10 ft. high waves act on the structure for an hour, or three hours, or a day before damage occurs. If material movement is the problem, how much material is removed or deposited in a given time by a given regimen of wave action.

Almost no useful information on the time factor in shore problems is available. Here again one may insist that such data is prerequisite to excellence in design, and to suggest that coastal engineers encourage research on this problem even at the expense of less difficult and less important research.

The short, steep waves characteristic of the Lakes are considered usually to be associated with lakeward net movement of material, i.e. they are degenerative in that they carry material from the shore to the lake. The shore area subject to such erosive movement extends from the limit of uprush of the waves on the shore to about the point of breaking of the waves offshore. As a general rule it can be considered that waves break in depths approximating 1.3 the wave height, thus permitting determination of the offshore limit of movement. Maximum protection against material movement therefore demands lakeward extension of structures inhibiting movement, such as groins, to lengths of the order of the distance offshore of breaking waves. Economic considerations may, however, dictate shorter lengths.

Once the design wave characteristics are selected (as shown this is in large part a question of engineering economics) the top elevations of structures, whether they be jetties, bulkheads, seawalls, or others, is determined. In general, vertical face structures will not be overtopped by solid water if their elevation is set above the average water level during wave action a distance equal to the total wave height. A sloping face requires, in general terms, a distance equal to 1.5 the wave height because of wave run-up on the face.

The requirements for structural stability are defined by Sainflou's or Iribarren's methods, both of which are described in Hudson's paper, "Wave Forces on Breakwaters", ASCE Separate No. 113, January 1952.

It must be noted additionally that because of the general paucity of material sources the structures must be impermeable in locations where retention of shore material is an important part of the function of the structure. Permeable structures appear to be poorly suited to this area and probably result in a profligate waste of scarce natural shore material; whether the structures be groins or armoring. It is highly doubtful that they can be considered adequate for any protective purpose in the Great Lakes or similar areas of limited natural resources of shore building materials. Insofar as groins are concerned the requirement of impermeability is absolute. Feeding of down-drift beaches often claimed as a unique benefit of permeable groins can be achieved better by employing impermeable groins with low top elevations, that allow material to pass over the structure once it has impounded its designed capacity. This represents sound engineering management of natural forces in accordance with the present state of our knowledge of shore processes. In the Lakes area there seems to be no effective substitute for impermeable groins whose top elevations essentially follow the profile of the beach desired. Consideration must be given to varying lake elevations in determining the profile, but a straight-line approximation of the profile is acceptable if it is desirable for ease of construction.

In many Lakes locations the deficiency of material suitable for protective or recreational beaches is extreme. In these locations armoring of the shore face may be the best engineering solution. However, when beaches are desired nevertheless, artificially supplied material is required. The design methods described in Technical Memo No. 29 of the Beach Erosion Board represent the best technology now available in this field. Direct placement methods probably are best for Lakes locations.

The relatively high density of shorefront occupation and the general scarcity of natural shore protection in these normally eroding Lake areas combine to require unusually careful evaluation of the effects on adjacent shorefronts of coastal works. Harbor jetties, or extensive groins, unless otherwise designed, may act so effectively as sand traps as to starve down-drift areas depending upon up-drift sources of material for complete or partial protection. Armoring of an eroding bluff may eliminate the bluff as a source of nourishment for a beach. An acceptable engineering solution in such cases must include an evaluation of these effects. Since the effects easily could impose very large burdens for protective works both the economic and engineering implications of protective works should be a part of every coastal study. Although it is not within the province of this paper to discuss the legal aspects of these questions they do exist.

The advantages of some sort of control by a technically competent and responsible group over coastal works design and construction seem apparent. The assignment of such responsibility as an extra activity to highway departments, park authorities, or municipal engineers can be successful only when the technical personnel in these offices are adequately trained in the highly specialized field of coastal engineering knowledge, or the services of qualified personnel can be obtained otherwise. In few engineering activities can so much damage be done by well-meaning, hard working personnel whose coastal engineering judgment is faulty only because they lack intimate knowledge of the complex of natural processes at a shore.

In closing, a few words must be directed to those who are most intimately concerned with shore problems in the Lakes area, the property owners, the taxpayers, the municipalities and the states. Without intelligent action on their part in insisting on competent technical analysis of their problems none of the mass of useful technology now available can be of value to them. So as the first principle in the design and construction of coastal works in the Great Lakes I suggest the principle of obtaining the best technical assistance that can be found and abiding by their recommendations. Without this elementary but necessary approach there is no hope for satisfactory solution of their coastal problems.

CHAPTER 15

LOW COST SHORE PROTECTION USED ON THE GREAT LAKES

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The purpose of this paper is to present the results of three years of field observations on low cost beach protection structures in use on the Great Lakes. The structures were studied in regard to their effectiveness as beach building and protective devices and with respect to their durability in resisting ice and wave forces. The term "low cost" refers to structures which cost between $10 and $30 per foot of frontage at 1952 prices.

INTRODUCTION

As early as 1947 the Michigan Department of Conservation became concerned over the increasing rate of beach erosion on the Michigan shore line. It was anticipated that a period of high lake levels would cause severe damage because of the extensive amount of lake front development that had taken place during the previous thirty years of relatively low levels. Many homes, resorts, public buildings, roads, and bridges had been constructed during this 30 year period without a full realization that lake levels were in the lower portion of their 5 or 6 foot range. During the period from 1950 to 1953 the lakes did rise and extensive damage has occurred.

In 1948 the University of Michigan Lake Hydraulics Laboratory was established to help in solving the problem. An extensive bibliography on beach erosion (1950a) and a booklet entitled "Beach Erosion in Michigan" (1950b) were published in 1950. The latter publication undertook to explain some of the beach erosion processes and to summarize and make readily available pertinent information from other publications. The U. S. Corps of Engineers Manual for Civil Works (1947) was extensively drawn upon. While these publications served a useful purpose, it was soon found that they did not meet the needs of the majority of the people who needed help. It was found that the Michigan shore erosion problems differed from those encountered on the sea shores not only because of a difference in the hydraulic factors but because lake shore property was predominantly in private ownership. This latter factor raised the difficult but challenging problem of finding adequate methods of protection at a minimum cost. The Lake Hydraulics Laboratory undertook a project to study this problem in cooperation with the Michigan Water Resources Commission and the Michigan Department of Conservation. The studies were started early in 1950.

LOW COST SHORE PROTECTION USED ON THE GREAT LAKES

THE NATURE OF THE PROBLEM

It was apparent from the beginning of these studies that much money was being spent by property owners on structures which were inadequate structurally and that often times the methods used were ineffective or even harmful hydraulically. Sea walls were used where groins would have been more effective. Many sea walls were being built with insufficient strength to withstand the back pressure of the saturated soil or without cut-offs to prevent seepage from the shoreward side which ultimately undermined the walls. Efforts were being made to hold banks at slopes much steeper than the natural angle of repose. A typical example of completely inadequate and incorrect wall construction is shown in Figure 1. This wall has no tie backs, the sheeting is on the wrong side of the piles, the sheeting does not penetrate into the ground a sufficient distance to prevent seepage and provide strength, a double row of sheeting should be used rather than a single row and, finally, the angle of the bank is much too steep to be stable. Many walls were built without provision for preventing long shore currents from developing behind the wall as the result of overtopping by waves. The use of permeable walls and groins was widespread. Groins were often not extended far enough inshore to prevent by-passing. An example of groin and wall construction which embodies some of the errors mentioned above is shown in Figure 2. Many walls were being built parallel to shore in off-shore locations where they were ineffective and easily damaged. Revetments were frequently constructed of such coarse material that erosion continued at nearly unhampered rates between the stones.

Many homes and cottages had been built and were still being built in locations that could not be protected at a reasonable cost. Some locations were found where groups of cottages, and even entire villages, had been built on low filled land. These areas were sometimes so low, even after filling, that waves were able to beat directly against the buildings during major storms. In many areas, where high bluffs exist near the shore line, homes were built on lots so shallow that there was no room to move the buildings back as the bank receded. In some of these locations the banks have advanced and receded from time to time during the past hundred years. This caused little concern until homes or highways were built on the fluctuating portion of the land.

In order to get reasonably correct information to the public as soon as possible, preliminary results of the studies were published a year ago under the heading "Low Cost Shore Protection for the Great Lakes" (1952). An additional year of observations had been obtained at the time of the preparation of this paper. Any conclusions must still be regarded as tentative because the total period of observations is less than four years. It is expected that the observations will be continued for a number of years, or until the lake levels recede.

Fig. 1. An example of inadequate sea wall construction.

Fig. 2. An example of ineffective sea wall and groin construction.

RESEARCH PROCEDURE

The problem is being studied by making a series of observations on the protective structures of various types which have been built by individuals and groups on the shore lines of the Great Lakes. Reconnaissance surveys have been made over nearly the entire 3100 miles of Michigan shore line by engineers of the Water Resources Commission. Approximately 25 reaches of shore line, each several miles long, have been chosen for closer observation. These areas were chosen not only because they provided examples of serious erosion problems but also because the property owners in those areas have constructed interesting types of protective structures.

The observations consisted primarily of obtaining photographs of the various locations at intervals of one year or less plus additional inspections to detect any changes that might be taking place at a rapid rate. The observations are being made with two distinct objectives: one, to determine the effectiveness of different types of protective methods under varying conditions and, two, to determine the durability of structures of different types and costs under varying degrees of exposure to wave and ice action.

The great difficulty in making field studies of this type is in evaluating the effect of the many uncontrollable variables. Such factors as fetch, storm frequency, and the general nature of the beach material could be readily determined. However, in some research areas it would be desirable to determine lake bottom topography, prepare refraction diagrams and make observations on currents and littoral drift. Such an elaborate program is obviously out of the question because of the great number of locations being studied. Insofar as possible, the nature of the unknown variable factors has been estimated on the basis of observations and previous experience. It is expected that the field observations will be followed by a series of model tests in the wave tank where the effect of some of the variables can be observed under controlled conditions.

CONCLUSIONS

Although the present high water period has produced serious inroads on the shore line, more serious than any other for perhaps thirty or fifty years, there is evidence that the shores have receded even farther during the past hundred and fifty years. The natural beach and dune building processes have repaired and covered up many of these older eroded areas during low water periods. Therefore, wherever possible a practical defense against erosion damage would be to move homes, cottages, or highways back and let nature take it's course. This has been done in many locations. However, where lots are not deep enough or where rights of way are not wide enough, it is necessary to provide protective measures.

Fig. 3
Beach resulting from the use of rock filled timber crib groins.

Fig. 4.
Beach resulting from the use of concrete groins.

The protective devices in use are groins of various types, sea walls, and revetments. Protection by means of artificial sand fill, often in conjunction with a groin system, is also a standard procedure.

GROINS

It has been found that groin systems provide excellent protection in the majority of locations studied. This is because of the generous supply of sand which is available along most of the Michigan shore line. Groins temporarily hold some of the littoral drift,thus raising the beach sufficiently to break the waves before they are able to attack the steeper banks. Protection by means of groins has the great advantage, in recreational areas, of not destroying the bathing beach as is the case where sea walls are built. Typical examples of beach building by low cost groins are shown in Figures 3, 4, and 5. Figure 3 shows a series of timber crib groins on Lake Huron. In Figure 4, the entire beach outside of the cement block wall was built up after the construction of a series of concrete groins. This beach, located in the Saginaw Bay area, has been maintained in the present condition for two years. A beach in the Little Point Sable area on Lake Michigan is shown in Figure 5. Timber sheet piling groins were used in this case. Water several feet deep occurs on the lake side of the timber sea wall shown in the background of Figure 5. The observations indicate that groins should extend from 50 to 70 feet into the lake from the shore line and should extend well back into the bank at their shoreward ends. Good results have been obtained by constructing the groins so that their top edges are located one foot above highest lake levels to be expected. It has been found that groins must be of tight construction to be effective. Only in locations where wave action is mild or where the beach material is very coarse do permeable groins hold beach material. Examples of ineffective permeable groins are shown in Figure 2.

In locations where wave action is severe and where the beach material is fine, the groins must be placed approximately the same distance apart as they extend out from the shore line, thus forming squares in the water area. For less severe wave conditions or where coarse sand or gravel appears in the beach material, they may be placed much farther apart, sometimes twice as far apart as they extend outward from the shore line. In practice, the groin spacing is usually related to the locations of property boundaries. Figure 6 shows timber crib groins in an exposed portion of the Lake Huron shore where the spacing is too great. Figure 7 shows groins that are longer than necessary or desirable. Figure 8 shows groins that have not been extended into the bank a sufficient distance and are therefore in danger of being breached at their inner ends. This group of groins could well have been constructed with the same total length but with their locations shifted fifteen or twenty feet in the shoreward direction. An extensive groin system, such as is shown in Figure 8, usually requires artificial nourishment not only to supply beach material to the groin area but to keep from starving down drift locations.

Groin Construction - Three types of groin construction have been found to give good results upon the basis of the observations carried out to this time. Timber sheet piling groins are the most widely used. Groins of this type are shown in Figures 5 and 9. As previously mentioned, these are modifications of the type of construction recommended by the Corps of Engineers (1947). Credit for the original design belongs to them. They consist of two rows of white oak plank sheeting with joints overlapping, walers on both sides, and round piling spaced at approximately 8 foot intervals on one side. The piles and sheeting are installed by jetting with a stream of water from a small portable pump. It is recommended that the sheeting be driven twice as far beneath the ground as it will extend above the ground, however, many timber groins which do not quite meet these specifications have been standing for several years.

Where it is impossible to jet in timber sheeting, because of the presence of large stones or hard clay, rock filled timber crib or concrete groins are built. Timber crib groins are shown in Figures 3, 6, and 10. They are constructed by building log cribs in shallow trenches and filling with large stones after placing a layer of brush. The logs are fastened together with steel pins.

Figures 4 and 11 show concrete groins. They are built in shallow trenches and must be sufficiently massive to resist the overturning effect of the waves.

The three types of groins described and illustrated above have come to be considered the standards in low cost construction in this area. They represent the minimum standards of constructions needed to provide reasonable durability, based on observations to this time. Many other types of groin construction have been used. Usually these are cheaper variations of the types described above as, for example, timber groins with one row of sheeting rather than two. One type, illustrated in Figure 2, consisting of a stockade of cedar posts has been used so extensively in some areas that it deserves to be mentioned. This type of construction is much cheaper than the three basic types described previously. However, except perhaps under very mild conditions, it is not effective, because it is impossible to drive the posts with a tight fit. In some areas, groins of this type have been built in the form of Y's with the open V in the upper portion of the Y facing toward the lake. These have appeared to be slightly more effective than straight permeable groins

Fig. 5
Beach resulting from the use of timber sheet piling groins.

Fig. 6
Timber crib groins spaced too far apart.

Fig. 7
Examples of groins that are longer than necessary or desirable.

Fig. 8
Examples of groins which do not extend into the bank a sufficient distance.

Fig. 9
Timber sheet piling groins.

Fig. 10
Rock filled timber crib groins.

but much less effective than straight impermeable groins.

SEA WALLS

The observations have shown that sea walls accelerate erosion in front of the walls, thus partially destroying the recreational value of the area. It has also been found that sea walls present a much more difficult structural problem than do groins. Consequently, the use of walls is recommended only for those cases where there is little or no littoral drift or where valuable property is so vulnerable to wave action that it is in immediate danger of destruction.

Sea Wall Construction - The most successful type of inexpensive sea wall construction is timber sheet piling. The walls are constructed in the same manner as the groins illustrated in Figure 9, except that ties to anchor piles must be installed at least every eight feet. The sheeting should be jetted six or eight feet below the lake bottom to insure that seepage will be cut off and that piping will be prevented. Groins must be placed on the inside of the walls to prevent erosion behind the wall, due to long shore currents fed by overtopping waves. The walls must be tight to prevent the leaching out of the back fill. Whenever possible, groins should also be constructed on the lakeward side of sea walls to minimize the erosion that would otherwise occur in front of the wall.

Where very high walls are required, standard designs recommended by the Corps of Engineers (1947) are recommended. Such construction does not fall in the "low cost" category.

REVETMENTS

Revetments are less objectionable than sea walls because the turbulence in front of a revetment is much less intense than in the case of a vertical wall. Revetments are successfully used in conjunction with groin systems to protect some particularly valuable portion of a bank. It is common practice to construct stone revetments around the bases of trees.

Revetment Construction - Stone revetments have been used successfully and can be constructed at reasonable cost. Care must be taken to mix stones of various sizes to provide a filter action and thus prevent the erosive power of the waves from penetrating to the beach material under the revetment.

An interesting type of revetment built in the Saginaw Bay area is shown in Figure 12. This construction consists of concrete poured into

Fig. 11
Concrete groin.

Fig. 12
Concrete revetment.

canvas bags and placed as shown. Each bag is connected with the one below by means of a short piece of reinforcing steel. The toe of the revetment is placed in a trench several feet deep. The revetment is "four bags thick" at the toe and tapers to a thickness of one bag as it rises against the bank. Although the revetment shown in Figure 12 is two years old, it has not yet been subjected to the severe ice conditions which sometimes occur in that area.

SAND FILL

The use of sand fill to provide artificial nourishment for beaches is a standard method of shore protection. Raising the flat beach several feet is usually sufficient to cause the waves to break before attacking the vulnerable steep banks. However, if fill is placed over a limited area, it is quickly carried away by long shore currents unless it is used in conjunction with a groin system. In many cases where this method has been used, the fill was borrowed from a near shore location by means of drag lines. This practice is dangerous for bathers and is usually ineffective because the wave action tends to carry the sand back to the deep areas where it was obtained. Several locations have been observed where a supply of sand was dumped in one location on a beach as a result of a land clearing and levelling operation. In such cases the beaches for as far as 1,000 feet in the downstream direction from the clearing operation have been well supplied with sand.

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CHAPTER 16

FILLING PATTERN OF THE FORT SHERIDAN GROIN SYSTEM

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ABSTRACT

An analysis of the action of a groin system through the 14 months following construction is reported herein. Six surveys were made during this period which included onshore elevations to the bluff line and offshore soundings to approximately the seven foot depth contour. Wind records during the period of record were studied and hindcasts of wave conditions made. Analysis of the basic data reveals that the cumulative volume of impoundment at the system may be expressed approximately by $K(I \cos B)^{0.97}$.

GENERAL

The groin system under consideration is located on the shoreline of Lake Michigan within the reservation of the Fort Sheridan Army Post. Fort Sheridan is located along the west shore of Lake Michigan between Lake Forest and Highland Park in Lake County, Illinois about 30 miles north of Chicago and 45 miles north of the south end of the Lake. The area is a part of the Highland Park Lake Border Moraine. The bluffs which vary in height from 50 to 70 feet above the beach are composed of glacial till and contain about 10 to 15 percent, by weight, of beach building material. Bluffs of this general composition are typical throughout the general area.

The reservation occupies about 1-3/4 miles of shore with the beach alongshore ranging up to 150 feet wide at low water datum. (Low water datum is a plane of reference 578.5 feet above mean tide at New York and is referred to herein after as LWD.) LWD is about 2 feet below mean lake level. In the area protected by the groin system the present development along the beach includes a water pumping station, an officer's beach, and an enlisted men's beach. The area along the top of the bluff behind the groin system contains family quarters and other post buildings.

Lake Michigan is the third largest of the Great Lakes and has a water area of approximately, 22,400 square miles (see fig. 1). The Lake has a maximum length of 307 miles with the major axis lying in a north-south direction. The maximum width, which lies in an east-west direction, is about 118 miles. Maximum deep-water wave height, as determined by hindcast methods and use of the Bretschneider (1)* curves, is considered to be about 25 feet. Information concerning Lake Michigan and Fort

* Numbers in parenthesis throughout this paper refer to references at end of paper.

Sheridan is given in other chapters of the proceedings of this conference and in references (2, 3, 4, 5). General information in regard to the variations in Lake Michigan levels during the period of record 1860 - 1952 is tabulated below.

TABLE 1

VARIATIONS IN LEVEL OF LAKE MICHIGAN

	DATE	ELEVATION*	FEET
Highest Monthly Mean Stage (1860 - 1953)	June 1886	583.68	
Lowest Monthly Mean Stage (1860 - 1953)	Feb. 1926	577.35	
Mean Stage (1860 - 1953)		580.58	
Maximum Temporary Rise:			
at Milwaukee			2.3
at Calumet Harbor			2.8

* Referred to mean tide at New York.

The general alignment of the shore line at Fort Sheridan is in a north-south direction and permits waves from the north through east to south-southeast to affect the shore. The available fetches in nautical miles are approximately 110, 210, 90, 75, 60, 60, 45 and 40 from North, NNE, NE, ENE, East, ESE, SE and SSE, respectively.

In the fall of 1950 in a report by the District Engineer, Chicago District, Corps of Engineers to the Chief of Engineers, based on information gathered for a Cooperative Beach Erosion Control Study with the State of Illinois of the Illinois Shore of Lake Michigan (4), it was recommended that the shore within the reservation of Fort Sheridan could best be protected by a system of 21 groins. However, it was also pointed out that the rate of erosion and the degree of development of the shore at that time justified only the protection of the central portion by the construction of 10 groins. After consideration of the volume of available drift it was decided that the initial phase of the construction should include not more than five groins. Five groins were constructed and comprise the system discussed herein.

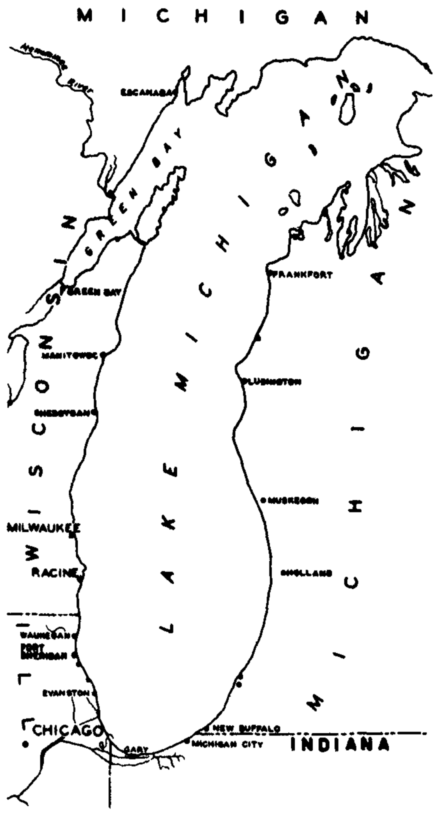

Fig. 1. Lake Michigan

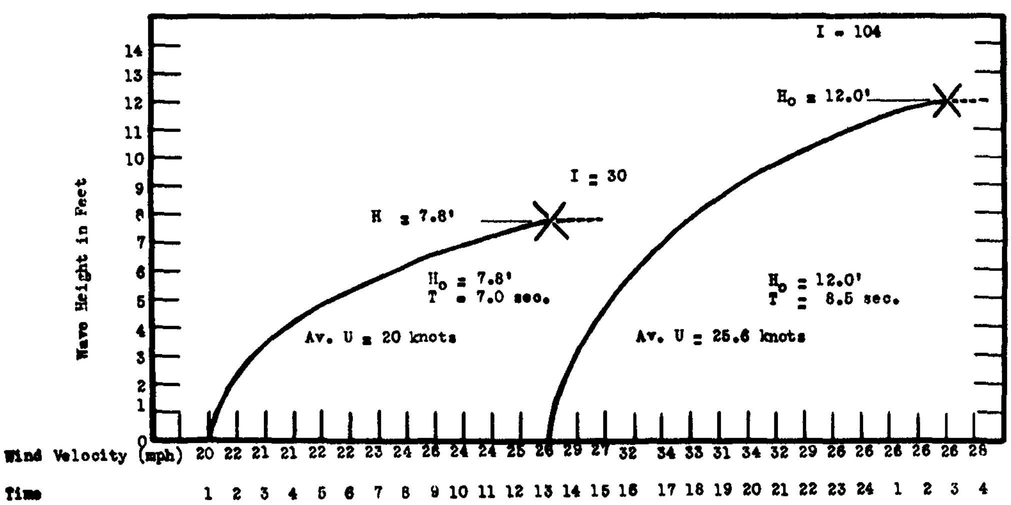

Fig. 2. Computation index for storm on October 23-24, 1951.

The groins of the comprehensive system are identified by consecutive numbers, the most southerly or downdrift numbered 1 and the most northerly or updrift numbered 21. The five groins under consideration in this paper are numbered 9 to 13 and will be referred to by number. Construction of groin 9 was begun on May 26, 1951, construction of groin 13, and therefore this system, was completed on August 8, 1951. The initial plans included the construction of groins 14-18 during the spring of 1952. However, due to the small amount of impoundment at groins 9 and 10 at that time, further construction was posponed until October 1952, following the final survey considered in this paper.

The groins are of cantilever type construction consisting of singlerow, interlocking steel sheet piling. The shoreward ends of the groin extend into the bluff or past the line of considered maximum erosion of the backbeach. The tops of the groins at their shoreward ends are 5.5 feet above LWD and extend horizontally from the bluff to about 100 to 150 feet lakeward thereof, then slope on a 1 on 30 slope to an elevation of 0.5 feet above LWD at their outer ends. The groins are so designed that the breaks in elevation of the top edges form a smooth convex curve and are coincident with the crest of the design forebeach. The groins are 250 to 300 feet in total length and are spaced from 350 to 390 feet apart.

SURVEYS

On August 14, 1951 following the groin construction, a sounding survey was made by Chicago District, Corps of Engineers. However, the observations were taken only from the forebeach to the lakeward end of the groins and the space between observations was too great to furnish sufficient information for true evaluation of the action of the groins. Beginning with the survey of October 12, 1951 the program of observations was extended and remained consistent throughout the remaining surveys.

Observations were taken on lines adjacent to each side of the groins, from the outer ends of the groins lakeward to approximately the 6.5 foot depth contour, and midway between each groin from a point shoreward of the forebeach crest to the 6.5 foot depth contour. Except for those lines adjacent to the groins the observations were taken at 20 foot intervals. The surveys were plotted and submitted in map form. Contour maps prepared from these data are presented herewith as Plates 1, 2, 4, 6, 8 and 10. Maps were also made showing change in bottom elevation between surveys and are presented as Plates 3, 5, 7, 9 and 11. Discussion of the pattern of filling of the groins is presented in a later section of this paper. By use of a planimeter and the Plates showing change in bottom elevation the volume of accretion or scour that occurred between the surveys was obtained. Table No. 2 presents a summary of impoundment to the ends of the groins and to the 6 foot depth contour.

TABLE 2

VOLUME OF IMPOUNDMENT BY GROIN SYSTEM

VOLUME - Bluff to 6-foot depth in Cubic Yards

AREA OF IMPOUNDMENT GROIN NUMBER	10/12/51 to 12/4/51	12/4/51 to 4/18/52	4/18/52 to 5/28/52	5/28/52 to 8/12/52	8/12/52 to 10/8/52	TOTAL Per Area
200 ft. North of 13 to 13	- 400	-1,820	- 970	+1,380	+ 680	-1,130
13 to 12	+6,100	+ 740	- 320	+ 250	+ 790	+7,560
12 to 11	+2,690	+2,140	+4,850	+1,120	- 630	+10,170
11 to 10	- 780	+3,360	+ 380	+ 50	+1,440	+4,450
10 to 9	-1,240	+ 890	+2,030	-1,680(1)	+2,120	+2,120
9 to 200 ft. South of 9	- 440	- 590	- 30	- 600(1)	+ 90	-1,570
TOTAL	+5,930	+4,720	+5,940	+ 520	+4,490	+21,600

VOLUME - Bluff to End of Groins in Cubic Yards

AREA OF IMPOUNDMENT GROIN NUMBER	10/12/51 to 12/4/51	12/4/51 to 4/18/52	4/18/52 to 5/28/52	5/28/52 to 8/12/52	8/12/52 to 10/8/52	TOTAL Per Area
200 ft. North of 13 to 13	+ 400	-2,790	- 640	+ 730	+ 260	-2,040
13 to 12	+6,660	- 870	- 430	+ 240	-2,120	+3,480
12 to 11	+1,030	+3,460	+5,300	+ 420	- 200	+10,010
11 to 10	- 570	+3,460	+ 120	+ 200	+1,180	+4,390
10 to 9	-1,370	+ 140	+ 490	-1,080(1)	+1,200	- 620
200 ft. South of 9 to 9	- 720	- 590	- 60	- 600(1)	+ 60	-1,910
TOTAL	+5,430	+2,810	+4,780	- 90	+ 380	+13,310

(1) Groin 9 Flanked during this period.

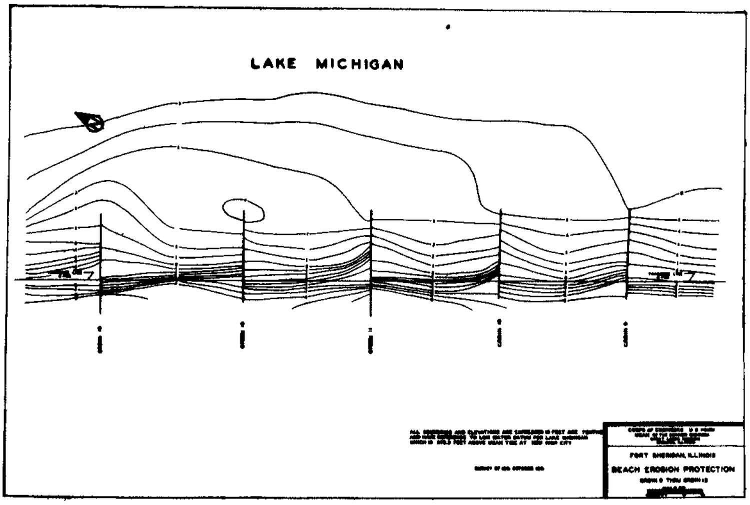

Plate 1

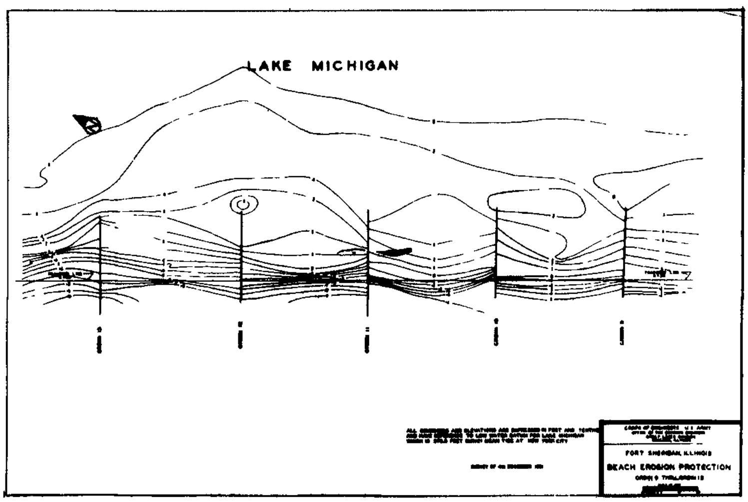

Plate 2

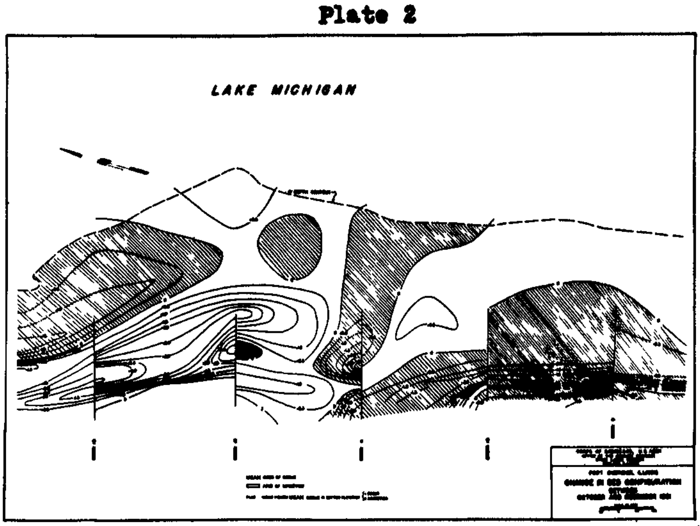

Plate 3

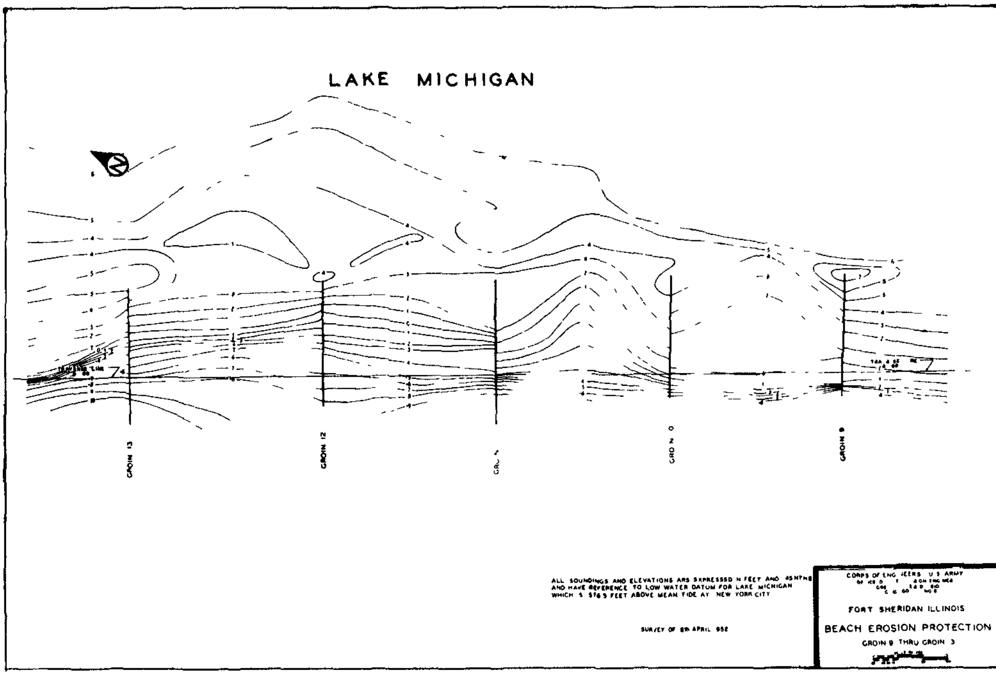

Plate 4

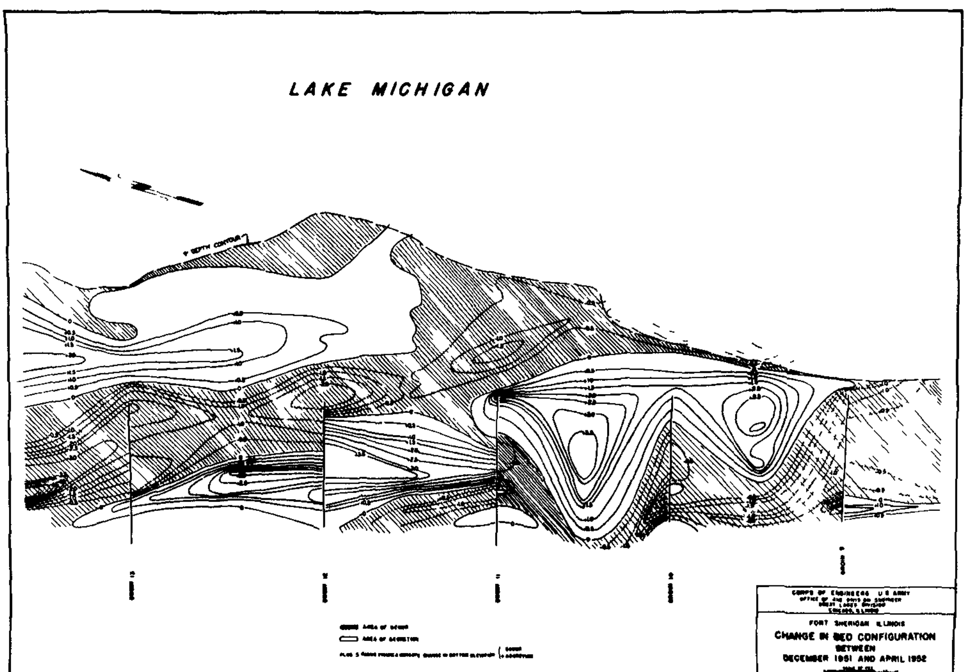

Plate 5

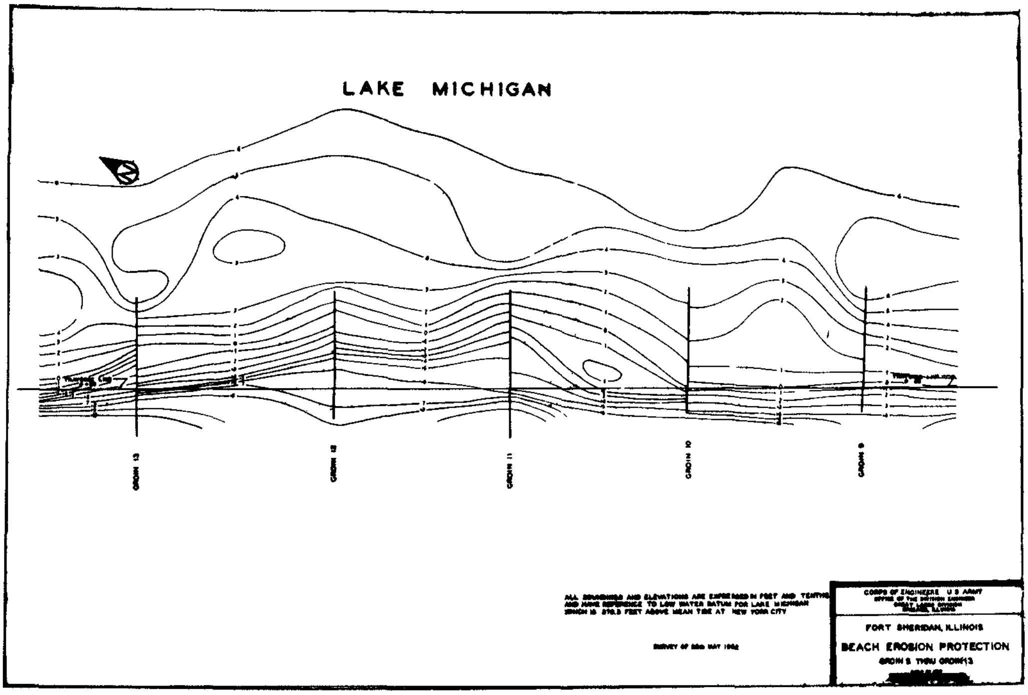

Plate 6.

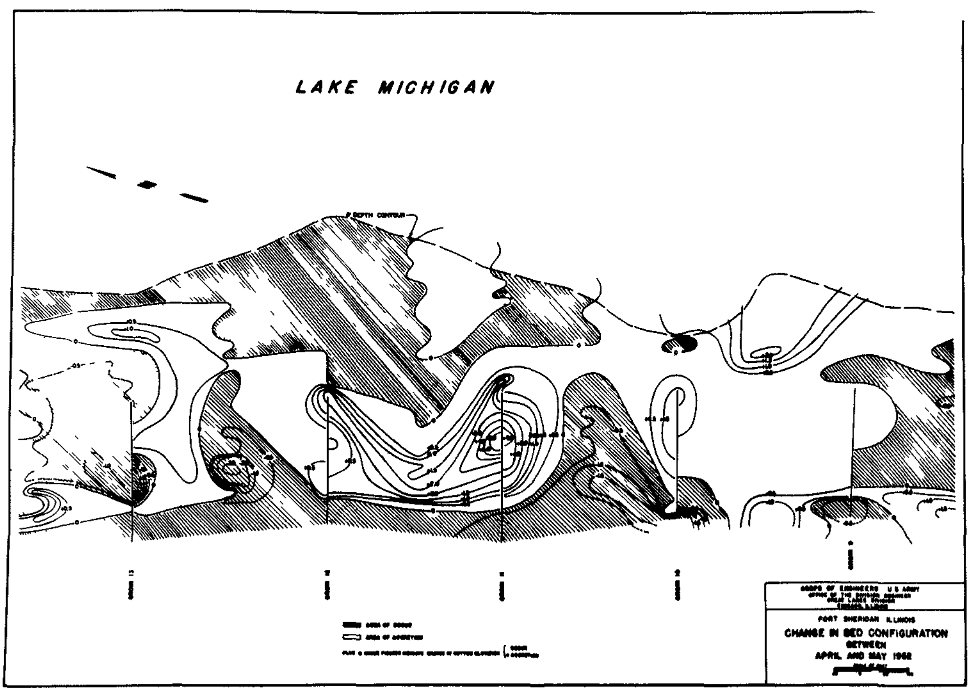

Plate 7.

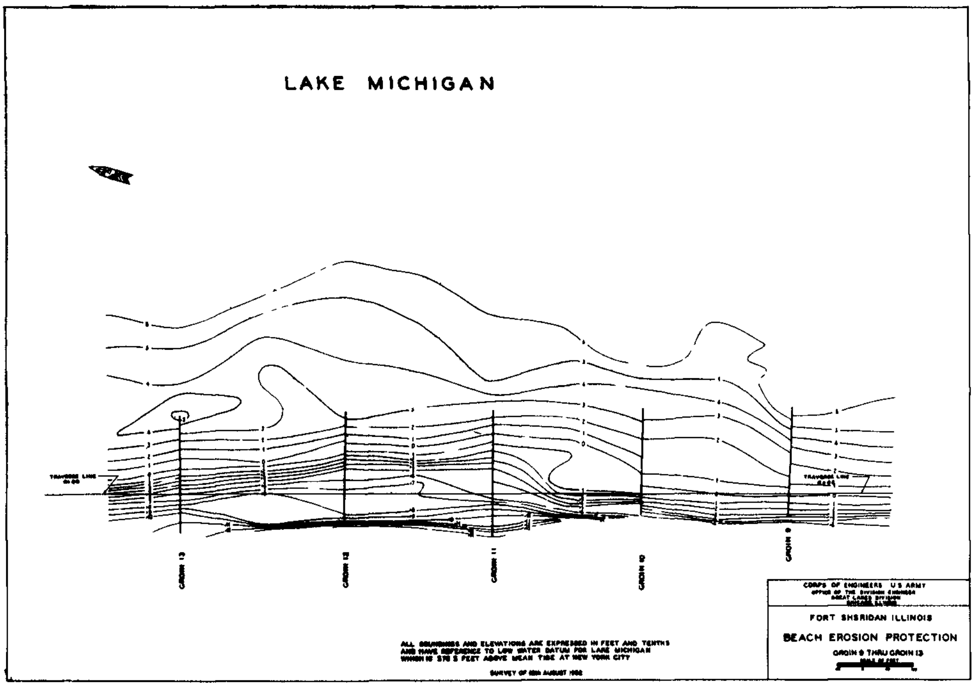

Plate 8.

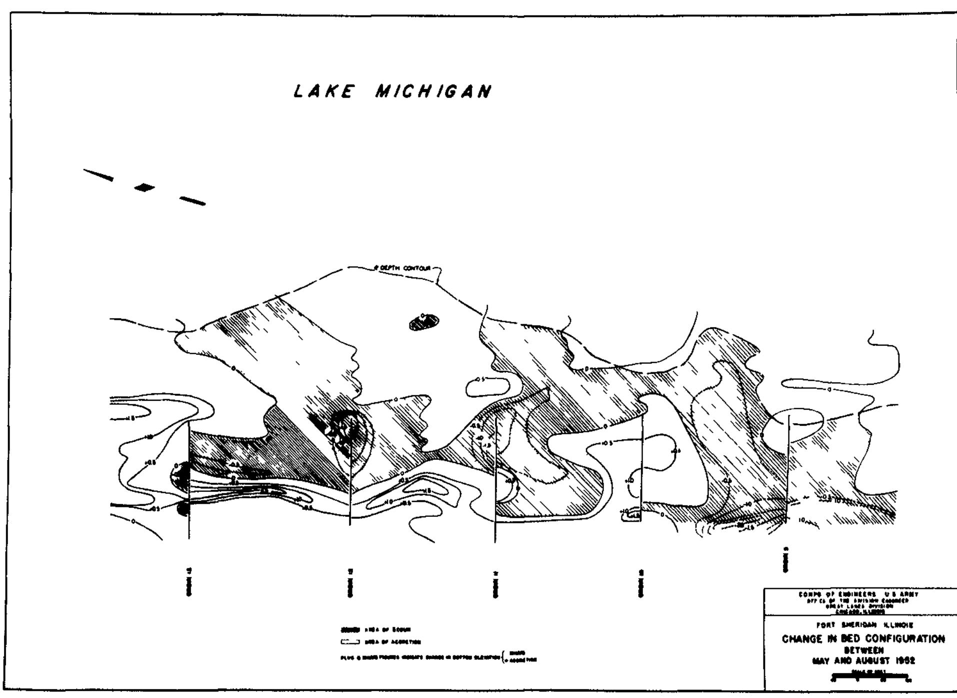

Plate 9.

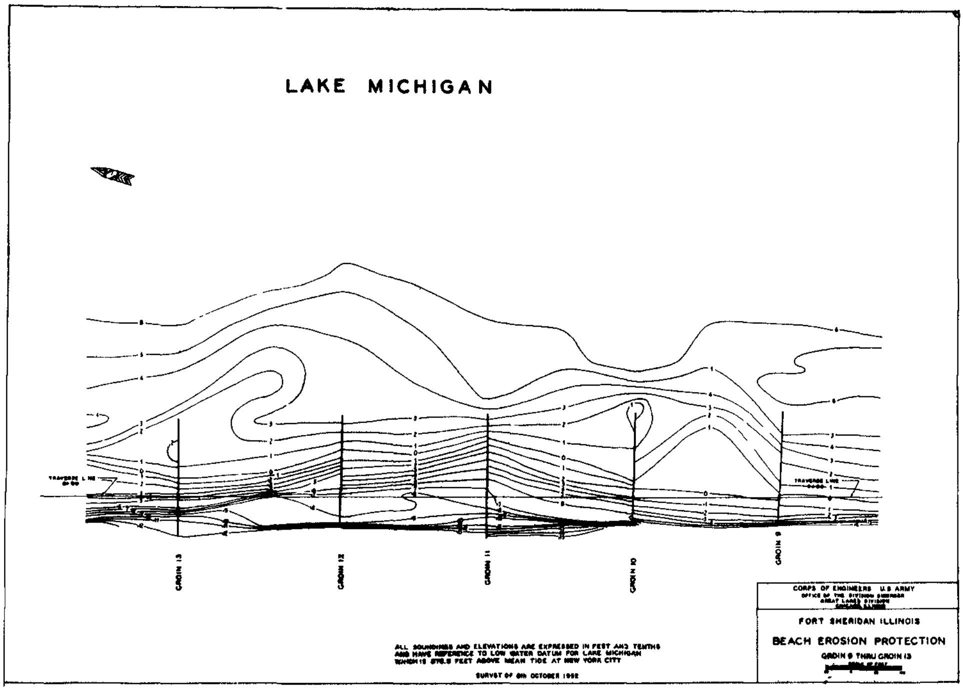

Plate 10.

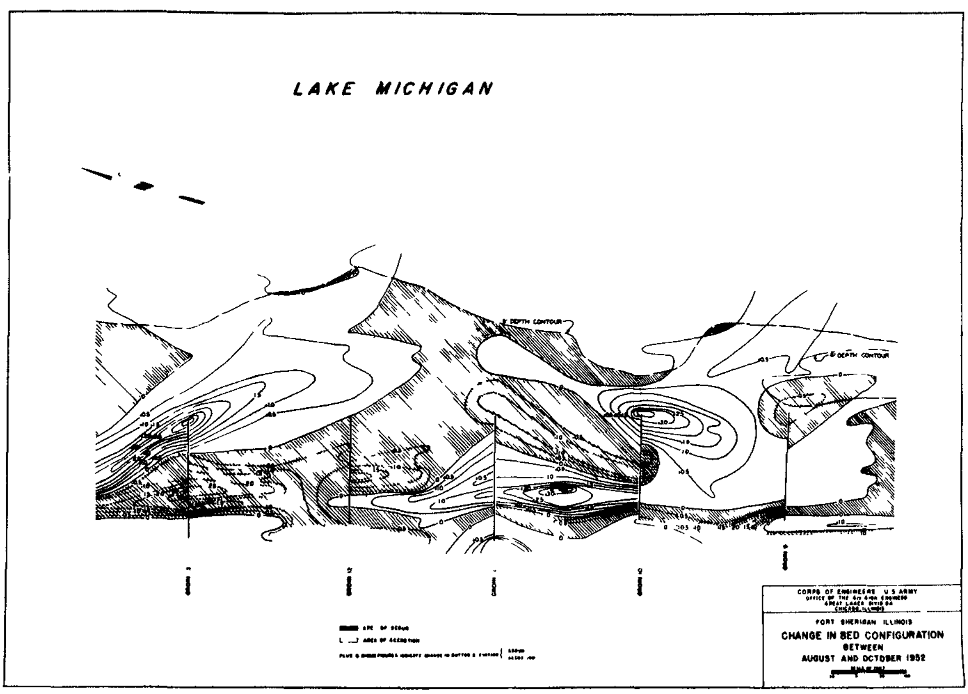

Plate 11.

ENERGY INDEX

It was considered possible to express impoundment of the littoral drift by the groin system in terms of an index derived from a combination of the effects of the various pertinent meteorological factors. It was also considered that such a description could be an index of the energy transmitted along the water surface. This index would therefore be an adaptation of the equation for total energy in a unit length of wave front. The equation for total energy in deep water may be written:

$$E_t = \frac{W \frac{g}{2\pi} T^2 H^2}{8} \left(1 - 4.93 \frac{H^2}{L^2}\right)$$

in which E_t = total energy, W = specific weight of water in pounds per cubic foot, g = accelleration of gravity in feet per second per second, T = wave period in seconds, H = wave height (trough to crest) in feet, and L = wave length in feet.

The shape term, $\left(1-4.93\frac{H^2}{L^2}\right)$, is negligible in such an approximation and of the remaining terms of the formula, $\frac{W \frac{g}{2\pi}}{8}$, are constant. Therefore, it is considered that the principal variables, T^2H^2, of the energy formula would provide an adequate approximate measure for comparative purposes.

The energy index was computed for each storm throughout the period October 12, 1951 through October 8, 1952 in which a deepwater wave height of 3 feet, or more, was generated. The wave characteristics were determined by hindcast methods (1) from wind data recorded at automatic recording gages located at Navy Pier, about 30 miles south of the area, for 1951 and at the South Side Filtration Plant, about 35 miles south of the area, for 1952. The wind data were supplied by the Division of Water Purification, City of Chicago.

In order to assure a consistent evaluation, the wind record for each storm was divided in such a manner that a maximum value is obtained as the index for the storm. The divisions are of two general classes. First is the principal portion, or that part of the storm which could, by itself, generate a wave height equal to the maximum of the entire storm; and, second are the remaining portions as delimited by the principal portion or by the effective fetch. The H^2 T^2 is computed for the significant wave of each portion, summed, and divided by 100 to obtain the index of the storm. The duration of the storm is thus given recognition in the index since the energy transmitted to the water by the portions of the storm superfluous to that theoretically necessary for production of the maximum wave height is included in the index as additional H^2T^2. Figure 2 is a graphic evaluation of the storm which occurred on October 23-24 1951.

In figure 2, U = wind velocity in knots and I = energy index. The principal portion of the storm, e.i. from hour 1400 to hour 0300, would generate a deepwater wave 12 feet high at Fort Sheridan; also, preceding this portion of the storm, the wind occurring from hour 0 to hour 1400 would produce a wave 7.8 feet in height. After hour 0300 on October 24 the velocity of the wind decreased rather rapidly and the direction changed to northwest, which is an offshore direction. The wind after hour 0300 was therefore not effective. The energy index for the storm would then be:

$$H^2T^2 = I = \frac{(7.8)^2\ (7.0)^2 + (12.0)^2\ (8.5)^2}{100} = 134$$

Energy index, I, referred to throughout this paper was determined thus.

RELATIONSHIP OF ENERGY INDEX TO VOLUME OF IMPOUNDMENT.

The index as described in preceding paragraphs was determined for the period of time between surveys and tabulated according to date and direction of propagation of the wave.

TABLE 3

SUMMARY OF ENERGY DATA

PERIOD COVERED	ENERGY INDEX I BY DIRECTION									
	NORTH	NNE	NE	ENE	EAST	ESE	SE	SSE	TOTAL I	cos E
Oct.12, 1951 Dec. 4, 1951	320 (52%)	247 (40%)	-	-	16 (3%)	-	13 (2%)	19 (3%)	615	584
Dec. 4, 1952 - Apr. 18, 1952	319 (17%)	688 (38%)	113 (6%)	91 (5%)	229 (12%)	193 (11%)	101 (6%)	90 (5%)	1824	1680
April 18, 1952 - May 28, 1952	3 (1%)	167 (76%)	10 (4%)	15 (7%)	26 (12%)	-	-	-	221	212
May 28, 1952 - Aug 12, 1952	4 (6%)	22 (32%)	-	-	19 (28%)	14 (20%)	8 (14%)	-	67	64
Aug 12, 1952 - Oct 8, 1952	70 (32%)	46 (21%)	72 (33%)	19 (9%)	11 (5%)	-	-	-	218	198
TOTAL	716 (24%)	1170 (40%)	195 (7%)	125 (4%)	301 (10%)	207 (7%)	122 (4%)	109 (4%)	2945	

The percentages shown in parenthesis are the portion of the total index for the survey period that the value for the specific direction comprises.

However, in order to relate the index to impoundment it was necessary to resolve the index value to the direction of wave travel which would produce maximum transport. This direction in which maximum drift will occur is assumed to be when the deep water waves approach the generalized shore line at an angle of 45°, since maximum drift will occur when the combination of the velocity of littoral current and the erosion of updrift bluffs and beaches is maximum. Saville (6,7) determined that maximum littoral transport occurred along an infinitely long straight beach when deepwater waves approach at an angle of 43° to the beach. Bruun (8) estimated that maximum transport occurs when the waves approach at an angle of 40° to 50°. Figure 3 shows the relative location of the maximum drift producing directions M_1 and M_2 with respect to the recorded wind directions, when M_1 and M_2 are at an angle of 45° with the generalized shore line.

On the figure 3, B is the angle between a line parallel to the actual direction of wave propagation and a line parallel to the direction of propagation which would produce maximum transport. The corrected energy index value was obtained by calculation according to the formula:

$$I \text{ corrected} = I \cos B$$

A study of the values of the energy index and the impoundment of the groin system measured between the bluff and the 6 foot depth contour indicates direct relationship. A plot was made in which impoundment and 10 (I cos B) was compared. Inspection indicated that a fractional exponent of 10 (I cos B) would approximately reproduce the impoundment curve except for irregularities due to other variables.

In addition to energy considerations the measures deemed necessary to consider in describing the filling of the groins would be elevation of the lake surface during the storms, the capacity of the groins to impound the available drift, and ice effect on erosion and accretion on the shore. From the available basic data it appears that cumulative volume of impoundment at a 5 groin system at Fort Sheridan may be expressed,

$$\text{Cum. Imp.} = 10C_1 \, (I \cos B)^{0.97} \, C_2 \, C_3.$$

and simplified to,

$$K \, (I \cos B)^{.97}$$

wherein $K = 10C_1 \, C_2 \, C_3$.

The three coefficients, C_1, C_2 and C_3 are discussed in the following paragraphs.

The coefficient C_1 gives consideration to the ability of the groins to impound a constant portion of the drift passing the area, and is variable with time. Throughout the period of record of the groins C_1 is assumed to equal unity. However, as the system fills the retaining ability decreases and therefore C_1 would decrease.

C_2 is a coefficient to correct for the effect of ice on erosion of, and accretion on the shore line. Usually in the latter part of December ice begins to form and the spray freezes on the shore, both having the effect of decreasing the erodibility of the source of the drift. Also as the ice thickens and windrows form, waves cannot attack the shore. The ice also forms a barrier to the shoreward movement of littoral drift. Usually in the latter part of March or early in April the ice melts and the necessity for this correction is nullified. Inspection of the available data indicates that a value of 0.7 for C_2 is appropriate for the ice effect on the shore during January - April 1952.

C_3 is the coefficient to adjust the energy index to conditions of change in Lake level. The monthly lake level in October 1951 was 581.54 feet above mean tide at New York, rising slightly to 581.66 in March 1952. From March to August 1952 the level rose rapidly to 582.69, then declined to 581.91 in October 1952. Since the higher elevation could only have the effect of increasing the amount of littoral drift made available by energy acting on the shore, the shape of the curve reveals no appreciable effect caused by the higher lake level. Therefore, C_3 is considered to be unity for the specific conditions during this study. However, if at a later date the lake level decreased appreciably so that the waves could not progress inland to erode the bluffs it appears that this coefficient would decrease to reflect the loss of efficiency of the energy index.

Considering the above evaluation of the coefficients the formula expressing the cumulative impoundment for the period January-April may be reduced to $7\ (I \cos B)^{.97}$ and for the remaining portion of the record is expressed by $10(I \cos B)^{.97}$. Figure 4 indicates that these values reproduce the cumulative curve within about 10%.

DISCUSSION OF PATTERN OF GROIN SYSTEM FILLING

Considering the survey of October 12, 1951 as a base, the patterns of filling of the system as shown by the surveys are discussed. Assumptions are also made as to the causes of the particular patterns.

Survey of December 4, 1951. During the period of 12 October to 4 December 1951 the monthly average level of Lake Michigan varied from 581.54 to 581.58, a very small change. The energy factor for the period was 616 with 320, or 52 percent from the north and 247, or 40 percent from the north-northeast. The groins impounded a total of 5930 cu. yds., an average of 3120 cu. yds. per month. The area of greatest accretion was between groins 13 and 12 where 6100 cu. yds. were impounded. The greatest loss occurred between groins 10 and 9 and amounted to about 1240 cu. yds. All volumes of accretion given herein are measured between the bluff and the 6-foot depth contour unless otherwise indicated.

As indicated above, the action of the groins was generally positive in nature (see Plate 3). The greater bulk of the accretion formed a

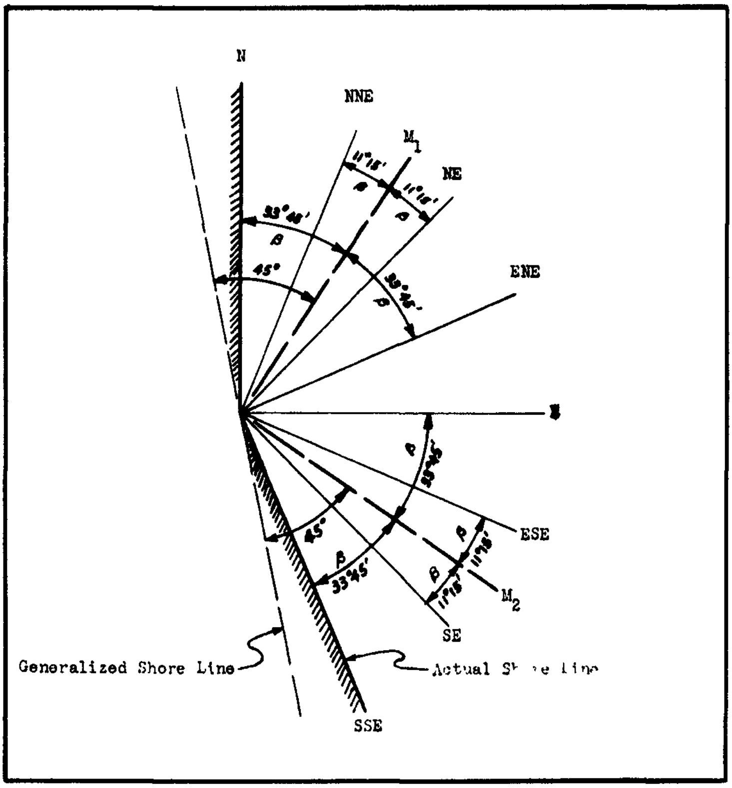

Fig. 3. Relationship of direction of maximum drift production, M_1 and M_2, to direction of wave propagation.

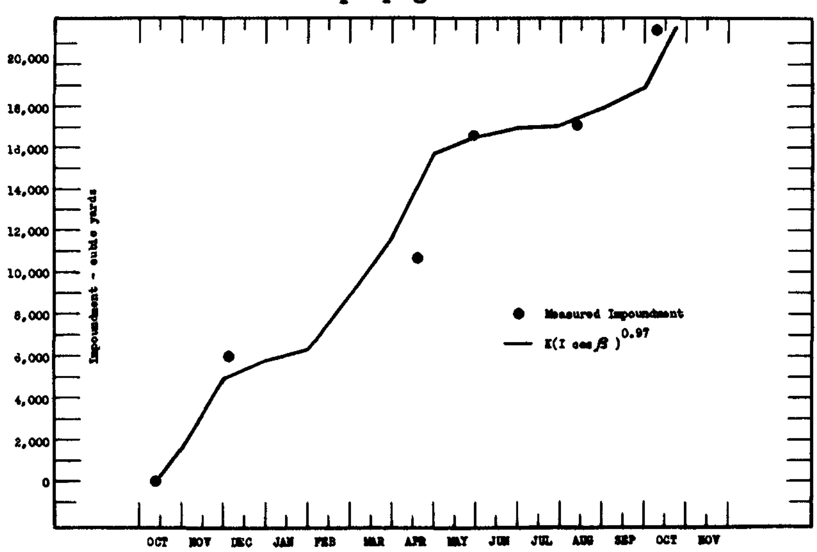

Fig. 4. Comparison of computed to measured impoundment.

finger shaped area beginning near the inner end, and north of groin 13, and extending in a slightly eastward direction past 13 and 12 to about 3/4 the distance between 12 and 11. The maximum depth of impoundment was 4.7 feet and was adjacent to the south side of groin 13. The depth of impoundment decreased to 3.2 feet at the lakeward end of groin 12, and to zero 290 feet southward of the groin. It would appear that the predominance of southward moving wave energy moved a considerable amount of material from the north. This material formed a beach north of groin 13 with a berm 10 feet above low water datum or about 8.5 feet above the average lake level with the forebeach extending lakeward with a 1 on 16 slope. The sand was impounded about 4.5 feet above the top of the groin therefore spilling over the groin and piling on the south side. Since the predominant wave was diffracted by groin 13 much less energy was expended on this impoundment. The circulation (9) between groins 13 and 12 distributed the sand in a south-southeast direction around the end of groin 12, then diffraction around the end of groin 12 directed it in a south direction. It is evident that the portion of the energy from the east (3%) was of sufficient intensity to distribute a portion of the shoal farther onto the shore.

From about 120 feet north of groin 11 to the south end of the study area the principal action was negative. The cause is assumed to be a paucity of material reaching the downdrift groins, due to the simultaneous construction of the five groins. The points of maximum erosion and the points of minimum accretion, where there is accretion adjacent to the north side of the groins, occurs between 100 and 200 feet landward from the outer ends of the groins. Specifically, the points adjacent to groins 13, 12, 11, 10 and 9 occur at a distance of 120, 100, 150, 190 and 200 feet, respectively, shoreward from the lakeward ends. It would appear then that this is the point of maximum energy impingement on the groin. It would also appear that a rip action occurs along the updrift side of the groin during this particular predominent direction of wave approach. The variance in distance of the erosion points from outer ends of the groins is apparently due to local refraction characteristics.

At the time of this survey, the beaches appeared to be rather straight and sloping without noticeable bar formation as the impoundment of material generally flattened the beach gradient. This is especially apparent between the 1 and 5 foot depth contours in the area from groin 13 to groin 10. The elevation of the beach berms decreased from $+$ 10 feet at groin 13 $+$ 5 feet south of groin 9. The beach slopes formed by groins 13, 12, 11, 10 and 9, between the berm and the flatter offshore gradient were about 1 on 16, 1 on 14, 1 on 10, 1 on 8 and 1 on 12, respectively, the slope south of groin 9 was about 1 on 10.

<u>Survey of April 18, 1952</u>. The period December 4 to April 18 includes that portion of the year in which ice forms a barrier on the shore. This decreases the effect of the large energy index of 1324 for the period. Several rather severe storms occurred during this period

which generated maximum wave heights of 9.8 feet, 15 feet and 13 feet from north-northeast, and one storm from the north which developed a maximum wave height of about 10.5 feet. The energy index was distributed as follows: North 319 (17.5%); North-Northeast 688 (38%), Northeast 113 (6%), East-Northeast 91 (5%), East 229 (12.5%), East-Southeast 193 (10.5%), Southeast 101 (5.5%), and South-Southeast 90 (5%). The elevation of the lake level varied from 581.58 to 582.06 during the time period. The greatest impoundment occurred between groins 11 and 10 where a volume of 3360 cu.yds. was impounded, and between groins 12 and 11 where a volume of 2140 cu.yds. accreted. Loss of volume occurred north of groin 13 by an amount of 1820 cu.yds. and south of groin 9 by an amount of about 600 cu.yds. The total increased volume in the system amounted to 4720 cu.yds. during this period of time, an average 1020 cu.yds. per month.

In examining the patterns of change in bottom elevation, it is of interest to note the resemblance between the band of accretion from north of groin 13 to groin 11 on the map of the October-December change (Plate 3) and the band of erosion on the map of the December-April change (Plate 5). Referring to Table 2 and Plates 3 and 5, it would appear that the large volume of material impounded between groins 13 and 12 during the period October-December 1951 migrated to the area between groins 12 and 11. Assuming this to be the manner in which the large accretion occurred between groins 12 and 11, the areas of maximum influx were in the 3 to 6 foot depths lakeward of groins 13 and 12, between groins 11 and 10 and between 10 and 9. The geometrical designs formed by the impoundment between groins 11 and 10, and 10 and 9 indicate that a reverse circulation may have been set up which caused erosion adjacent to the groins and deposits in the center of the area. However, no observations were made during storm periods to substantiate such an assumption. It also appears that a considerable littoral current was produced which moved material from the area immediately north of groin 13 and near and beyond the outer ends of the remaining groins. Considerable scour may be noted near the outward extremity of each groin. Continued erosion is noted south of groin 9.

At the time of the April survey the beach was more irregular than at the time of the December survey. This is accredited to the fact that the predominant direction of energy approach changed from north to north-northeast and that a greater number of easterly and southerly storms occurred.

Survey of 28 May 1952. The fourth survey of the study was completed on May 28, 1952. Therefore, the period of time covered by this survey was 1.3 months. The monthly mean lake level for April was 582.06 and for May was 582.32. By reference to Table 3 it may be noted that $I = 221$ and $I \cos B = 212$. It may also be noted that 76% of the energy index was from NNE which is an angle of approximately 34° with the shoreline, therefore, approaching closely the direction of maximum littoral transport.

The highest rate of impoundment that occurred during the period of study took place between the April and May surveys. The groins accumulated 5940 cubic yards of material, which is 4570 cubic yards per month for the 1.3 month period. The major portion of the accumulation, 4850 cubic yards, occurred between groins 12 and 11. Groin 9 collected 2030 cubic yards, which was the first large impoundment at that groin. Possibly a considerable portion of the groin 9 accumulation is migration from groin 10.

The general pattern of change in the bottom configuration was one of smoothing. The predominance of short period waves had the effect of slight erosion in the deeper water beyond the lakeward ends of groins 13 through 10. The ridge of sand midway between groins 11 and 10 was eroded, and filling occurred adjacent to the groins. There was also some filling downdrift of groin 10, but the general configuration remained constant. Gentle erosion continued downdrift of groin 9, probably due to a paucity of littoral drift. The accretion occurring below groin 9, landward of the LWD shoreline, is evidently migration from the north side of the groin. The slight erosion along the berm may have been caused by the westerly, or offshore, wind movement which consisted of approximately 10,000 miles of the 22,000 miles occurring during the months of April and May.

Survey of 12 August 1952. This increment of the time was the least productive of any portion of the study as the total gain in impoundment was only 520 cubic yards, which is an average of 210 cubic yards per month. There are two principal reasons for the small impoundment, the small amount of energy developed and the fact that groin 9 was outflanked. The energy index for the period was 67, an average of 27 per month.

The date of the actual flanking of the groin is not known but it appears reasonable to assume that the storm of June 11-12 may have culminated the action following the constant erosion of the toe of the bluff on the south side of the groin. The storm of June 11-12 was the most severe of this time increment. The computed maximum wave height and period was 7.1 feet and 6.2 seconds, respectively, from the east. This direction of propagation would mean that waves approached perpendicular to the shore causing severe erosion and little littoral current.

The wave action was sparse consisting of low, short period waves which varied from about 4 to 7 feet in height and 5 to 6 seconds in period. It is also of interest to note that 62% of the energy index came from the southerly direction, opposite to the predominant direction of littoral drift.

The general pattern of bottom change was to further smooth the beach except in the vicinity of groin 9 where an exceptional amount of erosion occurred because of the outflanking of the groin. In the area bounded by the bluff and the 6-foot depth contour, and lines 200-feet north and 200-feet south of groin 9, a loss of about 2280 cubic yards was experienced. In the remainder of the system the beach prograded. The area of largest accretion was north of groin 13 where approximately 1380 cubic yards accumulated. The prograding of this area is accredited to the smaller waves occuring during the period which would cause less turbulence and also would cause less overtopping of the groin. The greatest depths of accretion, exclusive of that near the shoreline, was at the lakeward end of groin 13 and about 80 feet lakeward of the outer end of groin 11, where in both cases a depression in the lake bottom had previously existed. The areas of greatest erosion were along the lakeward ends of groins 12 and 11 where small mounds of sand had existed at the time of the May survey.

The lake level during the period June through August varied from elevation 582.48 to elevation 582.69. The monthly mean elevation for August was the highest that had occurred since June 1886. This high lake level would tend to make the smaller waves more effective in eroding the shore than if they had occurred at a lower level.

Survey of October 8, 1952. This is the final survey under consideration in this paper and completes one year of study of the five-groin system. The energy index for this period was 218, an average of 109 per month. The north component comprised 32% and the NE 33 % of the index. The system impounded 4490 cubic yards, an average of 2245 cubic yards per month.

The wave heights contained in the index were greater than for the preceding two surveys. The wave heights varied from about 5 to 10.5 feet and the wave periods from about .5 to 8 seconds. There were two rather severe storms, one occurring on August 21-22 that generated a maximum wave height of about 10 feet with a 7.5 second period, the second occurring on September 6th produced a wave 10.6 feet high with a period of about 8 seconds.

The larger waves involved were evidently in part responsible for the irregular beach configuration existing at the time of the survey. A formation very similar to that observed on the plot of the December 1951 survey is evident. The formation consists of a cigar-shaped impoundment beginning north of groin 13 and extending southward and lakeward, forming an angle of about 60° with the groin. The maximum depth of the accretion was 4 feet, occurring at the lakeward end of groin 13. The other points of maximum depth of accretion were midway between groins 11 and 10, about 60 feet lakeward of the landward end of groin 10, and at the lakeward end of groin 10, both deposits were about 4 feet deep. In noting the similarity in the impoundment formations at the time of the December 1951 survey and the October 1952 survey, it may also be noted that at only these two survey times were there a larger portion of the energy index resulting from waves from the north rather than north-northeast. The reason is not apparent for the clearly defined erosion and accretion areas. For instance, at the lakeward ends of groins 13, 12 and 10 there is definite accretion while at the lakeward ends of groins 11 and 9 there is definite erosion.

Groin 9 was not tied back to the bluff prior to this survey. Therefore, erosion along the base of the bluff, and between the landward end of the groin and the bluff continued. However, debris and bluff material closed the breech to the extent that accretion in amount of about 2120 cubic yards occurred between groins 10 and 9.

SUMMARY OF FINDINGS

It has been attempted herein to follow the filling of a five groin system, to point out some of the major patterns in bottom changes caused by the groins, theorize as to elements of the cause of the patterns, and to develop an empirical expression to describe the volume change in the system. It may be noted that impoundment is related to an index of energy and no attempt is made to separate the effect of the various characteristics of the waves which contain the energy. The waves considered are storm-generated, deep-water waves whose generation occurred during 993 hours of the 8,760 hours comprising the study time period, or about 11% of the total time.

It is interesting to note that groin 13, which is the extreme updrift groin, did not accumulate the largest impoundment. One feature of the filling characteristics of the system was the initial action of this groin. For the first month or so after construction a prograding of the beach updrift occurred, thereafter retrogression followed for about 7 months. Rapid prograding did not occur until after the completion of this study when a short groin was constructed to the updrift. This feature is considered characteristic since following construction of additional groins to the system updrift, the area north of groin 13 began rapid prograding and the new extreme updrift groin is following a pattern of retrogression. This pattern of action indicates that the extreme updrift groin acts as a wave disipator for the system and that diffraction of the waves and currents by this groin aids in the filling of those downdrift.

Groin 12 impounded considerable material between the surveys of October and December 1951 and the flow of material was directed around to groins 11 and 10 thereafter. Both groins 11 and 10 collected a sizeable impoundment between the surveys of December 1951 and April 1952, and groin 11 was very effective between the surveys of April and May 1952. It is believed that groin 9 would have accreted beach between the surveys of May and August 1952 had not the flanking of the groin occurred. The first positive action of groin 9 occurred between the surveys of August and October 1952.

The filling of a groin system is not necessarily a persistent smooth filling process but is generally one of pulsating movement forming areas of alternate accretion and erosion. The actual configuration of the bottom at any specific time depends largely on the condition of the sea existing for an undetermined period of time preceeding the survey. The data analyzed for this study indicates that the larger, longer period waves form a very irregular beach configuration while the short period waves smooth the contours and form a regular-shaped beach. The irregular beach pattern caused by the larger waves is assumed to be related to two apparent characteristics of the impinging wave. First, the larger waves contain greater energy which causes the suspension of a larger quantity of sand thereby making it available for distribution, and second, the larger waves transport a larger quantity of water to the beach resulting in more intense rip currents which, together with circulation between the groins causes confused patterns of deposition. Such an assumption appears consistent with the findings of Bruun (9).

It may also be seen that the accretion is not confined to the area shoreward of the lakeward ends of the groins. It is true that in the initial stages the majority of available material is impounded within the limits of the groins, but as the system fills the accretion extends outward into deeper water. Between the surveys of October and December 1951 92% of the impoundment occurred within the groin confines, between December 1951 and April 1952 the amount was 60%, from April to May 1952 80%, May to August 1952 a loss of 20%, August to October 1952 a gain of 8%. On the basis of cumulative amount of impoundment the amount within the groins confines at the respective survey times listed previously was 92%, 77%, 78%, 75% and 62% of the total impoundment. Therefore 38% of the total impoundment as of October 1952 was lakeward of the groin system and was generally in a crescent shaped area extending from approximately groin 13 to groin 11.

It is considered that comparison of detail prototype studies of other groin systems are necessary to determine identities of the process of groin action. Also, it is hopes that further study of this system can be made. The recently constructed groins updrift of the five reported on herein will allow study of the effect of simultaneous groin construction on a somewhat stabilized system downdrift. Further study is also necessary to substantiate the validity of direct relationship between a storm wave energy index and volume of impoundment.

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CHAPTER 17

VARIATION IN GREAT LAKES LEVELS IN RELATION TO ENGINEERING PROBLEMS

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INTRODUCTION

Throughout the recorded history of the Great Lakes, the fluctuation in their water levels has created engineering problems generally unique in relation to coastal engineering. In periods of low water, demands are heard from navigation and power interests to raise the levels. In periods of high water appeals are made by shore property owners to lower the levels. Such conflicting interests present major engineering problems, the nature of which during a given period of time reflect the long-range upward and downward trend in lake levels due to natural phenomena.

High waters on the Great Lakes during the past few years, particularly in 1951 and 1952, have again focused attention on the fluctuation of the levels of the Great Lakes and the effects of such fluctuations on the three major interests concerned with the waters of these lakes, namely navigation, power, and owners of shore properties.

In this paper I wish to present first a general description of the Great Lakes system and an explanation of the fluctuation in lake levels with reasons therefor. With this background in mind, I wish to conclude my discussion by explaining in general terms the engineering studies related to the variations in lake levels now being undertaken by the Corps of Engineers. Other papers which follow will, I understand, discuss in more detail some of the engineering problems and possible solutions.

IMPORTANCE OF GREAT LAKES

From an historic, geographic, and economic standpoint the Great Lakes with their connecting channels form the most important bodies of fresh water in the world and these waters have played a vital part in developing the midwest into an industrial empire. The enormous deposits of iron ore found in the Lake Superior region, the large deposits of lime-stone located along the lake shores, together with the vast coal fields of the Upper Ohio River Valley, are the major sources of the raw materials required in the production of our basic industrial commodity steel.

The exploitation of all the natural resources in the Great Lakes Basin paralleled the growth and development of the country. The Great Lakes and their connecting rivers formed a natural artery for the economic transportation of raw materials which comprise the basis of the present vast industrial complex of the region.

As industry expanded and the population grew, there followed logically the development of hydroelectric power plants at the outlets of Lake Superior and Lake Erie and also increasing use of the attractive lake shores for residential and recreational use.

The Great Lakes - St. Lawrence River drainage system extends from the Gulf of St. Lawrence to the headwaters of the St. Louis River in northern Minnesota, a distance of some 2,000 miles or almost half-way across the North American continent. The entire drainage basin has an area of approximately 325,000 square miles of which nearly one-third is water surface. The five Great Lakes constitute one-third of all of the fresh water area in the world and cover an area nearly twice the size of Illinois.

Lake Superior, the largest and deepest of the lakes, has a water surface area of 31,820 square miles. The outlet of Lake Superior is the St. Marys River which flows for a distance of some 70 miles with a drop of 23 feet before emptying into Lake Huron. The average discharge from Lake Superior through the St. Marys River is 73,000 cubic feet per second.

Lakes Michigan and Huron are one lake hydraulically because the Straits of Mackinac which connects the two lakes are so broad and deep the water surfaces of the two lakes stand at the same level. Lake Michigan has a surface area of 22,400 square miles and Lake Huron has an area of 23,000 square miles. The outflow from Lake Huron passes through the St. Clair River, Lake St. Clair, and Detroit River to Lake Erie, a distance of 87 miles with a drop in water surface of about 8 feet. The average flow through these connecting channels is 175,000 cubic feet per second.

Lake Erie, the shallowest of all the Great Lakes, is considerably smaller than the three lakes above it, having an area of only 9,940 square miles. The Niagara River, connecting Lakes Erie and Ontario, drops 326 feet in its length of 36 miles. Fifty (50) feet of this drop occur in the Cascades immediately above the Falls, 172 feet occur at the Falls, and 93 feet from the foot of the Falls to Lake Ontario. The Niagara River averages 194,000 cubic feet per second.

Lake Ontario, 326 feet below Lake Erie and 356 feet below Lake Superior, is the smallest of the Great Lakes with a surface area of 7,540 square miles. It may be of interest to note here that the deepest point of Lake Ontario is some 525 feet below sea level and the deepest point of Lake Superior is about 700 feet below sea level.

The outlet of all of the Great Lakes is the St. Lawrence River. The first 116 miles of the river is on the international boundary between Canada and the United States, the remainder of its course is in Canadian territory. The fall from Lake Ontario to Montreal Harbor is approximately 226 feet of whic some 92 feet occur in the 46-mile long International Rapids section. The flow of the St. Lawrence River at Ogdensburg, N.Y. averages 231,000 cubic feet per second.

The flow in all of the outlet channels is remarkably uniform, the ratio between the maximum and minimum being only in the order of about two to one.

LAKE LEVEL VARIATIONS

The levels of all of the lakes fluctuate from year to year. The over-all long-range fluctuation over a period of years based on the range between the high and low monthly average levels varies from a little over 4 feet on Lake Superior to over 6.5 feet on Lake Ontario. The levels of the lakes also follow a yearly seasonal pattern with highs occuring in the summer and lows in the winter or early spring months. These seasonal variations average from 1.1 feet on Lakes Michigan-Huron to 1.8 feet on Lake Ontario. Through a period of record since 1860 the Lakes have departed widely, however, from their average seasonal behavior both as to the magnitude of the fluctuations and the timing of the highs and lows.

The principal natural factors which affect seasonal and yearly fluctuations of the levels of each of the Great Lakes are precipitation, evaporation, and flow in the rivers connecting the lakes.

Precipitation falling directly on the lake surfaces immediately becomes a part of the volume of water in the lakes and has a direct effect of raising levels by the depth of precipitation. Precipitation falling on land surfaces of the drainage basin produces a variable effect on lake levels due to the variations in runoff.

Evaporation from the lake surfaces lowers the lake levels by the number of inches of evaporation, but unfortunately no accurate means has been devised for measuring this evaporation. The depth of water removed directly from the lakes each year by evaporation is estimated to vary from approximately 1.5 feet on Lake Superior to as much as 3 feet on Lake Erie.

Flows in the connecting rivers have direct effects on levels of the lakes, inflow from the lake above tending to raise levels and outflow to the lake below tending to lower levels.

The abnormally high stages prevailing in 1951-1952, when Lake Erie and Ontario reached all-time highs, are attributable to the persistent trend of above-normal rainfall during the preceding 10 years. During this 10-year period the annual precipitation was 2.23 inches above the average for the years since 1900, and in 1950 and 1951 the precipitation was nearly 6 inches above the average.

A number of attempts have been made to find cycles in the rise and fall of lake levels and to correlate them with cycles of other phenomena. Our studies have failed to find any consistent cycle pattern in the long-range rise and fall of lake levels nor has any evidence been found to relate the lake level fluctuations to the waxing and waning of sun spots or other physical phenomena.

Short-period fluctuations are superimposed on the long-range and seasonal fluctuations. They result from an unbalance or tilting of the lake surfaces induced primarily by winds and differential barometric pressures. These fluctuations reach their maximum in periods ranging from a few minutes to several days and remain at or near the maximum for about the same periods of time. The maximum temporary rises recorded range from 8.4 feet on Lake Erie to 2.5 feet on Lake Huron. Fluctuations of this magnitude occur at infrequent intervals but temporary changes in levels of from 1 to 2 feet are common.

Another natural factor which must be considered in relation to lake levels is the crustal movement of the earth. It has been shown rather conclusively that the coasts of all of Lake Ontario, all of Lake Erie, most of Lake Michigan, and the southerly shore of Lake Superior are sinking in relation to the lake outlets, due to a crustal movement of the earth, so lake levels there are rising. The coasts of most of Lake Huron, the northeasterly part of Lake Michigan, and the northerly shore of Lake Superior are rising in relation to the lake outlets and the lake levels there are lowering. The maximum changes in lake levels due to this cause occur at Port Dalhousie on the westerly end of Lake Ontario where the water is rising at a rate of 1.1 feet per hundred yards and at French River in Georgian Bay on Lake Huron where the water is falling at a rate of 0.7 foot per hundred years. Rates at many other places on the Lakes are almost as great.

Artificial factors affecting the levels of the Great Lakes are diversions into and out of the lakes, and alterations of flows in the connecting channels by artificial means such as dredging or by compensating or regulating works.

Diversions in the Great Lakes today consist of the Long Lake-Ogoki diversions from Hudson Bay watershed into Lake Superior, the Chicago Sanitary and Ship Canal diversion from Lake Michigan to the Mississippi River basin; and the Welland Canal and the New York State Barge Canal diversions out of Lake Erie into Lake Ontario. Regulating works are in operation at St. Marys Falls at the Soo to control the levels of Lake Superior. The other four lakes are not regulated.

The net total effect of all existing diversions and control structures on lake levels is in the order of less than 2 inches, except on Lake Ontario where the Gut Dam in the Galop Rapids of St. Lawrence River raised Lake Ontario levels approximately an additional 7 inches. However, this structure was removed in January 1953 so the levels of Lake Ontario are dropping accordingly.

The net effect of artificial factors is of extremely small magnitude when compared with the natural phenomena of long-period fluctuation of from 4 to 6 feet and when compared with short-period fluctuations.

ENGINEERING AND ECONOMIC MATTERS RELATED TO VARIATIONS IN LAKE LEVELS

With this condensed explanation of the various natural and artificial factors that affect the variations in lake levels, and bearing in mind that higher lake stages are favorable to navigation and power but that lower lake stages are favorable to shore interests, let us now briefly examine the economic effect of fluctuations in lake levels on each of these three major uses of water and then consider the engineering studies related to each.

LAKE LEVELS AND DAMAGES TO SHORE PROPERTY

The damages which occur to shore property by wave action are dependent not only on levels of the Lakes but also on the number and severity of storms.

VARIATION IN GREAT LAKES LEVELS IN RELATION TO ENGINEERING PROBLEMS

Damage to shore properties is relatively minor at low lake stages since the uplands are generally protected by wide beaches at these stages. However, during high lake stages, the beaches are largely under water and the back shore is subject to direct wave attack from even moderate storms. The maximum damage occurs when a severe storm is accompanied by a large temporary rise in lake levels at a time when the lake levels are on the high side of their fluctuation range as they have been during the past two or three years.

During high lake stages many low-lying areas along the lake shores are flooded. In general, many of these areas were developed during extended periods of low lake levels which condition furnished a false sense of security from the hazards of future high water stages. As an example, extensive areas around the westerly end of Lake Erie, including several hundred homes, were flooded in 1952.

During the recent period of high lake levels when unusually stormy weather prevailed from the spring of 1951 to the spring of 1952, the damage along the United States shores of the Great Lakes is estimated to have been in excess of $61,000,000, of which $50,000,000 was caused by wave action and $11,000,000 by flooding of low lying areas.

The present and prospective future increased extensive utilization of shores of the Great Lakes make the matter of damage to shore properties a subject of serious concern in the entire Great Lakes region. It affects directly hundreds of thousands of people. Consequently, the damaging effect of high lake stages on shore properties is more apparent to the general public than are the effects of variations in lake levels on the two other major water uses of navigation and power.

Unlike a flood from a river when the damage comes and goes in a short period of time, lake shore owners are faced with floods or are threatened with floods for many consecutive months. Accordingly, the interests of shore property owners are best served by regulating lake stages within narrower limits than occur naturally so that some optimum upper limit will not be exceeded.

LAKE LEVELS AND NAVIGATION

Extensive improvements have been underway by the Corps of Engineers since 1825 to provide adequate through connecting channels and harbors for the lake fleet. One of the most vital of these improvements has been the Soo Locks for by-passing the rapids in the St. Marys River.

The channels in the St. Marys River as well as in the St. Clair River, Lake St. Clair, and in the Detroit River have been deepened to 25 feet in downbound and 21 feet in upbound channels. Deepening of these connecting channels has been a major undertaking.

To by-pass the Niagara River the Canadian Government has built the Welland Canal with 7 lift locks with depths of 30 feet over the sills and with the canal presently dredged to 25 feet to accomodate modern lake freighters. Canada has also provided a series of canals and locks for

14-foot navigation to by-pass the rapids in the St. Lawrence River between Lake Ontario and Montreal.

Today we have 60 active commercial harbors on the Great Lakes, 14 of which handle in excess of 7,500,000 tons of cargo annually. As compared to this, the active commercial salt water ports of the United States number about 155 of which 31 handle more than 7,500,000 tons annually. Based on the total commerce handled in 1950, four of the ten largest ports in the United States are on the Great Lakes.

During the calendar year 1950 the total waterborne commerce in the United States was 820,600,000 tons. Of this total, 199,200,000 tons or approximately 24 per cent was carried on the Great Lakes. The bulk of the traffic is carried during the eight and one-half month navigation season from about April 1 to December 15 at a rate of over 20,000,000 tons per month.

Most of the commerce on the Great Lakes consists of bulk raw materials such as iron ore, coal, stone, grain, and petroleum products. Of the total of 199,200,000 tons carried in 1950 about 44 per cent, or about 87,000,000 tons was iron ore carried from upper lake ports to the lower lakes. In the 1953 navigation season it is expected that about 100,000,000 tons of iron ore will be transported on the lakes.

Prior to World War II the largest vessels on the lakes were about 600 feet long with maximum drafts of about 22 feet. The connecting channels and major harbors served by this fleet had adequate depths except when the lakes were at low stages. Since 1942, 43 U.S. vessels have been built or are under construction for the Great Lakes with maximum permissible drafts of from 24 feet 6 inches to 26 feet 10 inches. It is expected that this trend toward construction of larger vessels and retirement of the smaller vessels will continue. These larger vessels have a cargo capacity of up to about 20,000 tons as compared to about 12,000 tons for the larger boats of a few years ago.

It is apparent that these vessels can load to their maximum drafts only when the lakes are at extreme high stages. When lakes are at average or low stages they must reduce their drafts by as much as 3 or 5 feet because of the restricted depths in the connecting channels and in the improved harbor channels. The larger vessels carry over 100 tons per inch of draft so at low lake level stages they are required to lighten their loads upwards of 4,000 tons per trip.

Thus we see that the fluctuation in lake levels have a significant effect on navigation. During high lake stages the carrying capacity of the fleet is much greater than when they are low with resulting savings in the cost of transportation. It follows logically then that the interests of navigation are served best by high lake stages.

LAKE LEVELS AND POWER DEVELOPMENT

Hydroelectric power is generated in the St. Marys River at Sault Ste. Marie and in the Niagara River at Niagara Falls. The large and remarkably steady flow from the Great Lakes resulting from their tremendous storage capacity makes the outlet rivers from these lakes extremely valuable as a source of water power. The present installed capacity at Niagara Falls is 445,000 KW in the United States and 848,000 KW in Canada for a total of 1,293,000 KW. Canada now has under construction an additional development of 1,200,000 KW. In a study in 1951 the District Engineer, Corps of Engineers, at Buffalo concluded that an additional development of 1,300,000 KW could be made in the United States at the Falls. In the International Rapids section of the St. Lawrence River a proposed development of about 1,880,000 KW is to be divided equally between the United States and Canada. It is needless to say that power interests are best served by high lake levels.

ENGINEERING STUDIES RELATED TO LAKE LEVELS

Taking into account the physical and economic factors of the Great Lakes which I have discussed, we see that natural variations in lake levels, both from a long-range and seasonal point of view, prove to be favorable at times to some interests and yet are also unfavorable to others at the same time. Since high and low lake stages will continue to occur under natural conditions in the future at various intervals, the economic impact therefrom will become increasingly more significant in relation to the growing need for more power, the construction of larger vessels for navigation, and desirability of relief from damages to shore properties.

In recognition of these circumstances, certain comprehensive engineering studies have been initiated in an effort to reduce the growing economic losses caused by the fluctuations in lake levels.

In response to directives of the Congress, the Corps of Engineers is now engaged in two major studies of the Great Lakes problems. One deals directly with the variations in lake levels, the other is concerned with navigation.

The first study is concerned with determining the feasibility of a plan of regulation of the levels of the Great Lakes that will best serve the interests of all water uses, particularly in regard to the reduction of damages to shore properties, the use of the Great Lakes for navigation, and the use of the storage and outflows from the Great Lakes for power development. It is significant to note that numerous lake regulation studies have been made in the past 57 years by various Boards, Commissions, private individuals and others which contemplated regulation to maintain high lake levels in the interests of navigation and power. The current study is the first to give full consideration also to shore property owners.

Since the Great Lakes form one of the most complex hydraulic systems in the world and since the requirements of navigation, power, and shore property uses, in respect to lake levels and outflows from the lakes, are not entirely compatible, considerable time will be required to complete these lake regulation studies.

These studies are being coordinated with the eight other Great Lakes states and interested Federal Agencies to arrive at an overall solution which will serve best the interests of all concerned. In this regard before any system of lake regulation can be adopted it will of course be necessary to coordinate and obtain agreement with Canada as all of the lakes except Lake Michigan are boundary waters between the two countries.

Another phase of this investigation will consider the advisability of providing local flood protection works to protect low-lying areas from inundation. An interim report on this phase of the study has been submitted to the Congress recommending Federal participation in the cost of providing protection to three localities on the western end of Lake Erie.

Associated with these hydraulic studies are such investigations as long-range forecasting of lake levels; and the analysis of the individual factors governing supplies of water to the lakes, such as precipitation on land and water surfaces, runoff, groundwater, outflow in connecting channels, and evaporation. Because both the United States and Canada have each measured the hydraulic factors of the Great Lakes independently, the Corps of Engineers and technical agencies in Canada are now in the process of seeking agreement on the basic hydraulic and hydrological data.

The second major study by the Corps of Engineers concerns consideration of the need for improvements in the connecting channels to accommodate the increasing number of large vessels of the United States Great Lakes fleet. The Congress has directed that previous reports by the Corps of Engineers be reviewed to determine the advisability of proceeding at this time with improvements in the Great Lakes connecting channels to provide a channel depth of at least 27 feet below low water datum and to prepare up to date estimates of the costs of such improvement.

As mentioned above the connecting channels include the St. Marys River, between Lakes Superior and Huron; the Straits of Mackinac, between Lakes Michigan and Huron; and the St. Clair River, Lake St. Clair and Detroit River between Lakes Huron and Erie. Deep draft navigation between Lakes Erie and Ontario is now provided by the Welland Canal. These channels now provide 25 feet in downbound and 21 feet in upbound channels. The Welland Canal is dredged to 25 feet and has depths of 30 feet over the lock sills.

Allowing 2 to 3 feet of underclearance for safe navigation it is clear that the present project depths of 21 feet and 25 feet are no longer adequate for the growing number of vessels with designed drafts of 24 feet or more.

The study now underway to consider deepening of the connecting channels is of course immediately concerned with the fluctuation of lake levels, the present and future make-up of the fleet, the compensation needed in the channels to balance the deepening effect so as to not adversely affect lake levels and comprehensive economic analysis to determine the economic justification of the improvements at this time.

One other type of study should be mentioned, that is, studies for protection of the shores and beaches by means other than by regulation of the levels of the lakes. Cooperative beach erosion control studies are authorized by Public Law 520, 71st Congress, approved July 3, 1930, as amended and supplemented, wherein the United States may cooperate with States, Municipalities, or other political subdivisions in making studies with a view to devising effective means of preventing erosion of the shore of coastal and lake waters by waves and currents. In these co-operative studies the cost is divided equally between the United States and the cooperating agency. Public Law 727, 79th Congress, approved 13 August 1946 established policy whereby, with the purpose of preventing damage to public property and promoting and encouraging healthful recreation of the people, the United States may assist in construction, but not the maintenance, of works for the improvement and protection against erosion by waves and currents of the shores of the United States owned by States, Municipalities, or other political subdivisions; Provided, that the Federal contribution toward the construction of protective works shall not in any case exceed one-third of the total cost. There is no provision for Federal participation in the cost of construction of protective works for privately-owned property.

Of the 4,000 miles of mainland shoreline of the Great Lakes in the United States, about 450 miles have been studied and reported on by the Corps of Engineers. Each report provides type-designs for works to protect privately owned property and detail designs for the protection of the publicly-owned property. The shoreline covered by the reports are: On Lake Michigan, the entire shoreline state of Illinois, and Milwaukee and Racine Counties, Wisonsin; on Lake Erie, the entire shoreline of the State of Ohio and Presque Isle Peninsula, Pa.; and, on Lake Ontario, Niagara County, N.Y. Now under study is the shoreline of four State of New York Parks on Lake Ontario; and,the shoreline of Kenosha, Wisconsin and that from the North City limits of Two Rivers, Wisconsin to the south city limits of Manitowok, Wisconsin on Lake Michigan.

CONCLUSION

In conclusion we in the Great Lakes region have been confronted continuously with coastal and related engineering problems of major importance to all users of the Great Lakes including shore property owners. These problems are aggravated by the large fluctuations of lake levels.

These fluctuations may be reduced on certain of the lakes as a result of some of the studies now underway and also on Lake Ontario as a result of the works proposed in the plans for the development of the St. Lawrence River. Although the fluctuation range may be reduced, the present pattern will always remain, that is, high stages in summer and low stages in winter and with cycles of generally high or low stages over a period of years depending on the amount of precipitation in the basin. It is too early to predict the outcome of our lake regulation studies but certainly we know that any action which can be taken to reduce the natural fluctuation ranges of the lakes will have the effect of mitigating our coastal engineering problems by only a relatively minor degree. We do believe, however, that any reduction in the fluctuation ranges of the lakes which can be justified by lake regulation will provide wide-spread benefits to shore properties and to other users of the Great Lakes waters.

Chapter 18

THE DISASTER IN THE NETHERLANDS CAUSED BY THE STORM FLOOD OF FEBRUARY 1, 1953

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SITUATION OF THE NETHERLANDS WITH REGARD TO THE NORTH SEA.

It will be useful to show first a sketch of the situation of our low lying area with regard to the North Sea (Fig. 1). We might idealize the North Sea in the shape of a rectangular pocket, nearly 400 miles wide and 500 miles long, open to the Atlantic in the line Duncansby (Scotland) - Bergen (Norway). For its size the North Sea is relatively shallow. The depth is of the order of magnitude of 60 m. The funnel-like Southern part, between England and the Netherlands, is not more than 40 m deep. The average width of this part is 250 miles.

Figure 2 shows the peculiar topographic nature of the Netherlands. The more elevated Southeastern part may be regarded as the real delta of the rivers Rhine and Meuse. This area is entirely above the highest floods of the North Sea. The lower part initially was a lagoon, sheltered from the sea by a barrier beach built up by the sea. Originally this barrier was a riff of elongated sand banks, gradually they were heightened by waves and wind. Behind this protecting barrier the sea and the rivers deposited sand and silt, on which a marshy vegetation flourished. All the time the vegetation was submerged by the sea and successfully struggling to keep above the water. In this way the lagoon has been built up by formations of peat and marine clay and sand to about the level of normal astronomical high water. This is today the low part of our country, defended against the sea by 2000 miles of dikes. An important part of this area consists of reclaimed lakes, with the height of the land from 2 to 5 m below mean sea level. This situation has not been permanently established and it cannot be considered as stabilized. There are two factors which not only keep the menace of the sea alive but even strengthen it gradually. These are:

a. The gradual settling of the layers of soil which have filled the lagoon; as a result the level of the land, including the bases of the dikes and the dikes themselves, is lowered slowly. In some cases the crest of the dike has subsided 2 m in 400 years. That indicates an average sinking of 50 cm per century by a process of soil mechanics.

b. The territory of the Netherlands is subject to a gradual sinking relative to the sea level of about 15 cm per century. Therefore the height of the stormsurges slowly raises.

It is clear that on account of these two causes the situation of the Dutch polderland has steadily grown more difficult during the centuries. This has become apparent with fearful clarity in the recent years. A great handicap for a thorough investigation of this vital problem is the lack of accurate records from the past.

THE DISASTER IN THE NETHERLANDS CAUSED BY THE STORM FLOOD OF FEBRUARY 1, 1953

Fig. 1. Low lying area of the Netherlands with regard to the North Sea.

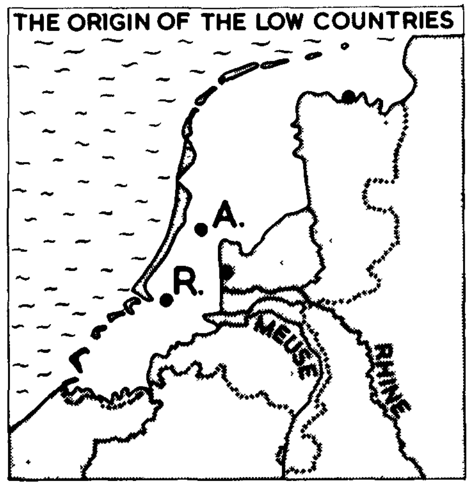

Fig. 2. Peculiar topographic nature of the Netherlands.

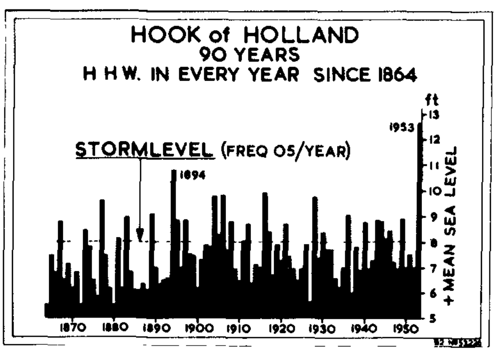

Fig. 3. Diagram of the yearly maxima of storm surges from 1864 to 1953.

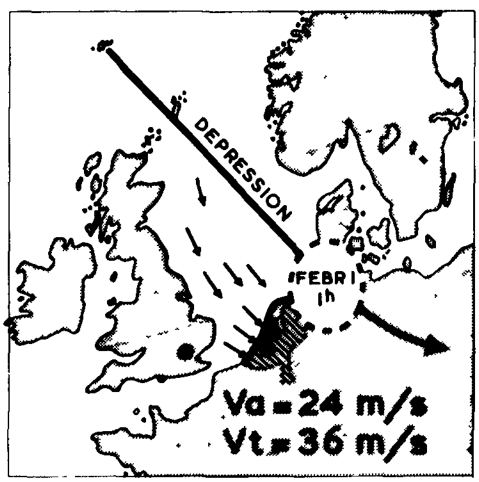

Fig. 4. Track of the depression and position of the storm center at the moment of the beginning of the disaster.

Moreover, extreme floods are not well defined fixed phenomena. They range within a wide scale of possibilities. A systematic change in the pattern of this scale can only be detected by means of an extensive statistical study. Without such a study it might escape the attention altogether.

The insight necessary for a methodical treatment of this problem has been developed only during the last 10 or 15 years. This has led to a new search for data from the past. Included is an investigation of the atomic structure of Carbon in organic deposits from old settlements, N_{13}, C_{14}, C_{12}, by which the age of layers can be detected within an accuracy of two or three hundred years.

The tidal motion of clearly semi-diurnal type is naturally accompanied by powerful tidal currents in the estuaries. Near the mouth of some of the estuaries the discharge at the moment of maximum flood or ebb current is 100,000 m^3/sec. Twice a day these huge tidal currents run in and out the estuaries. They have a maximum velocity of $1\frac{1}{2}$ to 2 m per sec. They scour channels in the sand to a depth of 30 to 40 m and in some places even more. Until recently technical possibilities were entirely inadequate for closing off inlets of this size. Moreover the discharge of the rivers Rhine and Meuse has to find its way seaward through these estuaries. It is clear there was no choice but to accept the situation handed down to us by history.

The normal tidal motion is subject to continuous disturbance by wind. As a rule this amounts to but a few inches and is seldom greater than two feet.

Regular daily observations were not started before the middle of the nineteenth century. If we prepare a diagram (Fig. 3) of the yearly maxima for the 90 year period, 1864 to 1953, we obtain an insight into the varying characteristics of the storm surges. Sometimes long periods pass without any serious storm surge. For tens of years the serious gale of 1894 has been considered as an extremely high one. In any case it has been made painfully clear to us in February 1953 that nature does not recognize any limit that has been prematurely impressed on the human mind.

From statistical considerations developed in recent years* we know that theoretically we will have to reckon with much higher floods than have been known in the past, and which will surpass even the 1953 flood.

On the strength of this consideration, the work of re-establishing the decreased safety had already been started. Newly constructed dikes and the large enclosure dam of the Zuiderzee, for example, already had been given considerably higher crests. But the disastrous storm surge in the beginning of this year overtook us before we were ready to meet it.

THE GALE OF FEBRUARY FIRST

From a meteorological point of view the gale of 1953 was different

*P.J. Wemelsfelder - Wetmatigheden in het optreden van Stormvloeden. (A statistical investigation on the probability of storm surges) De Ingenieur 1939 No. 9

from other heavy gales in two respects:

a. It had an extremely long duration.
b. The track of the storm was very unfavorable for our country

If we investigate the tracks of storms which caused earlier high storm surges we find tracks traveling over Great-Britain, particularly over Scotland, in an eastward direction and disappearing over Norway or Denmark. The gale of February 1 is markedly different from this well known type (Fig. 4). This time the center of the depression of the storm has crossed the North Sea diagonally from Scotland to Hamburg. During all this time the wind to the right hand of the center was Northwest and consequently it was directed straight toward our coast. The sketch gives the position of the center of the depression at the moment of the beginning of the disaster. Evidently the track of this storm was extremely bad for piling up water against the Dutch coast. As long as meteorological data has been gathered, never has such an undesirable storm been observed. However, this gives information over only half a century.

On February 1 the wind velocity in the vicinity of the coast was about 24 m/sec. with gusts up to 36 m/sec. If we draw a diagram of the wind effect at the time of high water at different stations we obtain a fair picture of the geographic features of the storm surge (Fig. 5). On the East Coast of Great Britain, the wind effect is little more than 2 ft. at Leith in Scotland. Going South, the effect increases, reaching 8 ft. in the Washbay and 7 ft. in the Thames Estuary at Chatham, both enlarged in comparison with the raise in the line of the coast due to the normal funnel effect in bays. Toward Southampton the curve shows a sharp decline, as has been represented by the dotted line. But if we cross the Channel from Dover to Calais and proceed in a northeast direction along the other shore, we see a sharp rise past Ostend and Flushing to the Dutch Coast. Here the increase in level over the astronomical height of the tide extends to 10 ft. The summit of this curve extends all along the West Coast of Holland. At Den Helder the coastline curves to the east and from here we find a considerable decrease of the wind effect. In the German Bight it is but 4 to 6 ft., and in Denmark it is less than 3 ft.

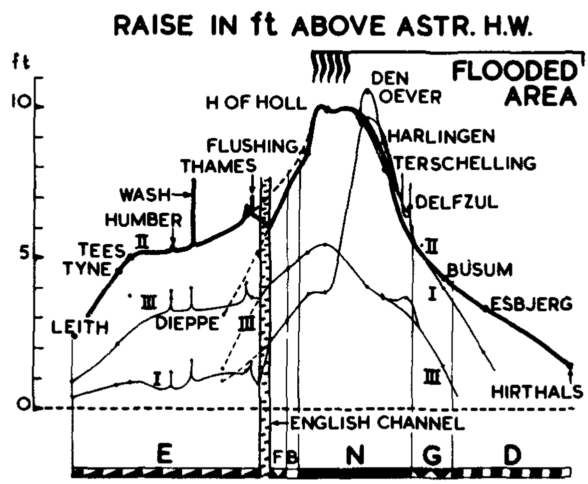

Fig. 5. Diagram of the wind effect at the time of high water at different stations.

At the bottom of the diagram in Fig. 5 there is given an indication of which parts of the coast belong to England, France, Belgium, the Netherlands, Germany and Denmark. As one can see the highest

Fig. 6. Photograph of a model showing, to an exaggerated vertical scale, the elevation of the water level at the time of highest high water above the astronomical high water. Note the extreme height along the Dutch Coast, the Wash, the Thames and the English Channel.

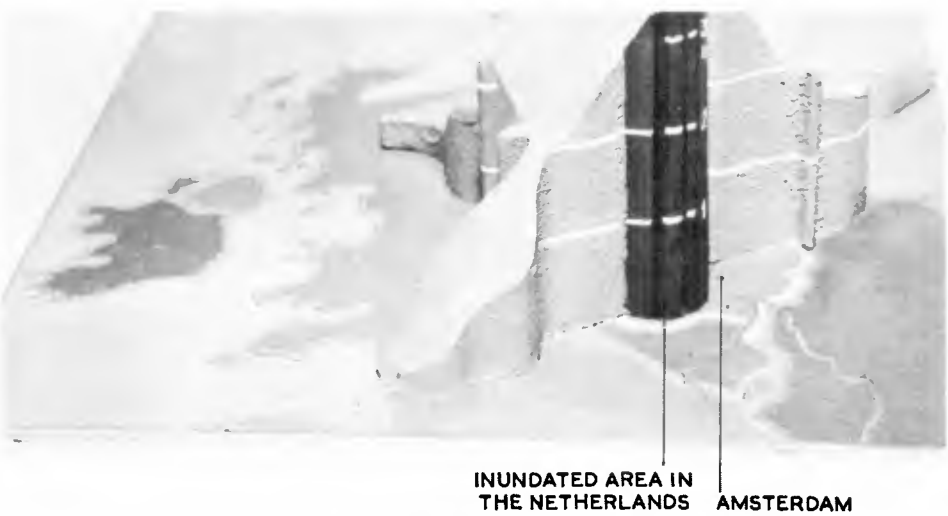

Fig. 7. The same model as in Fig. 6, seen in the northern direction. The dark part represents the estuaries in Zeeland and Suidholland, the center of the catastrophe.

elevation above the normal tide took place just along the coast of the Netherlands.

As an aid to our discussion of the problem a small model of the North Sea was constructed. The same figures of the rise above astronomical high water of Fig. 5 are in this model, placed vertically (Figs. 6 and 7). What is seen here is not the true level of the sea at the moment of the disastrous flood, but the excess elevation due to the wind.

It should be noted that this model does not represent the inclination of the sea at one single moment during the storm surge, since the moments of the astronomical high water do not occur simultaneously at all points. Every high tide enters the North Sea north of Scotland. It then moves along the East Coast of England in a southward direction. It crosses the Seven Straights, at the same moment joining with the tidal wave coming from the Channel, the latter being only a small part of the whole tidal motion. Then the tidal wave turns along the coast of the Netherlands to the northeast and up to the German Bight and Denmark.

The surge appeared at 4 PM on January 31 in Leith. It then passed to the Straits and arrived there at midnight. It reached the greatest elevation about Hook of Holland at 2 o'clock in the early morning Sunday, February 1, 1953. At 8 o'clock the surge was at Den Helder and then disappeared toward the east.

You will remember that the British East Coast and the Belgium Coast along the Scheldt also were stricken by the flood. It was the Netherlands, however, that had to bear the brunt of its violence. It is clear from Figs. 1, 6 and 7 that the attack of the storm surge was concentrated particularly on the Dutch coast. Taken roughly the wind effect here was twice that on the East Coast of Britain. As you know, the catastrophe occurred in the dead of night from Saturday to Sunday. Eyewitness reports are few. A photograph (Fig. 8) made at daybreak at Flushing near the mouth of the Wester Scheldt, conveys a feeble impression of the violence of a turbulent sea assaulting the shore defences.

Fig. 8. An illustration of the violence of the turbulent sea, made at daybreak near the mouth of the Wester Scheldt at Flushing.

The effect of the gale on the water level as a function of time can be seen from the gage records at Rotterdam (Fig. 9). The diagram shows the recorded curve together with the predicted tidal curve. It is seen that:

a. The wind effect increases and decreases roughly linearly. The irregularities are of little importance. There are no sharp peaks discernable.

b. The maximum storm effect fortunately did not coincide with astronomic high water, but it occurred near low water. This maximum amounts to 3.70 m. At high water it was but 3.0 m.

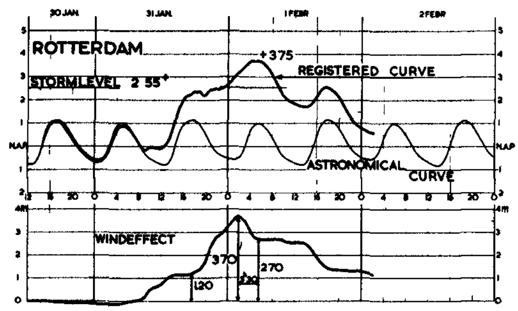

Fig. 9. Diagram of recorded and predicted tidal curve at Rotterdam.

The corresponding diagram of Hook of Holland shows a similar shape. At the moment of high water, the wind effect amounts to 3.0 m The largest wind effect occurred 3½ hours before high water. It amounted to 3.3 m (11 ft.). There was an additional set-up between Hook and Rotterdam of 0.4 m, partly due to the piling up of the Rotterdam Waterweg by wind and partly due to funnel effect.

The exceptional character of the storm surge appears from its location on a frequency curve. The diagram (Fig. 10) shows frequency curves for three stations along the Scheldt. The highest levels for a period of 90 years are all represented in the diagram. The height, H, is entered on a linear scale in a vertical direction, the number, N, on a logarithmic scale in a horizontal direction. The frequency curves in this diagram show a fairly smooth curve. They are not entirely straight, nor should they be straight on a Gaussian scale, a Gumbel scale or any other scale. No theoretical discussion will be presented; however, the single example we have here is sufficient to show the inadequacy of any simplified method to represent the rather complicated true condition. For practical reasons we often prefer the semi-logarithmic paper as being the most simple to use.

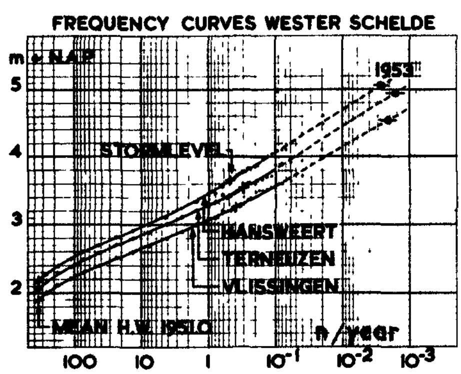

Fig. 10. Diagram of frequency curves for three stations along the Scheldt, showing the extent to which the 1953 level exceeded the levels recorded before.

A plausible extrapolation on this paper would lead us to assume for this super-gale a frequency of 0.0025, or once in 400 years. The diagram shows the extent to which the 1953-level exceeded the levels recorded before. In this light it is admitted that practically nobody expected the occurrence of such a storm. The disastrous consequences will no longer be surprising. In the entire territory of the Southwest Netherlands the levels were 50 to 70 cm above the highest levels that have ever been recorded.

THE ATTACK ON THE DIKES AND THE INUNDATION

During the gale of February 1 the water levels in themselves exceeded by 50 to 70 cm the design levels of the dikes. Hence the water level came near to the crest of the dikes and overflowed it at many places, especially in harbors and other sheltered spots where strong waves never penetrated and the safety margin of the crest level was small. As a result of this the dikes were damaged over many miles, and in many places they were washed away completely.

The damage practically never started at the outer side of the dike as one would usually think, but always at the inner side. Fig. 11 shows a dike that has been damaged but not pierced. The sea is to the right, the land was to the left. As you can see, the inner face has been eroded away by the water flowing over the crest. This occurred not at only a single point, but over tens of miles one could find the same condition.

From this and similar evidence it appears that the catastrophe was not the result of insufficient maintenance. The dike faces opposed to the enemy are still intact. The extreme high water level has led to an attack from the rear for which the earth-dikes were not adequate.

The first stage of the attack can be seen in Fig. 12. The sea is to the left, the land to the right. On the crest of the dike a low concrete wall has been constructed, in order to prevent the waves from overtopping the structure. It was not sufficient for this super-flood At the landward side of the crest a lengthwise fissure can be seen. It can be interpreted as the start of a slide in the water-saturated earth of the dike, caused by the strong pressure difference in the ground water. The water dashing over the crest penetrates into these fissures and this has frequently resulted in a rapid collapse of the dike (Fig. 13). In many places the dike has been washed away to the base, sometimes over several miles at a stretch.

Still much more serious were those cases where the jetlike flow bursting through the dikes cut out a gully straight to the interior. Where that occurred the sea had not access merely to the polder for one tide, but a fatal connection had been established between the estuary and the polder (Fig. 14).

The strong tidal motions in the estuaries made millions of cubic meters of water flow twice daily in and out of the polders. The differences in level at the breaches were 2, 3 or even 4 ft. giving rise to flow velocities of 4 to 5 m/sec. It is not surprising that in one case in a short time a channel was scoured 30 to 35 m in depth and 500 m in width, through which four times a day powerful currents occur to fill up and to empty again the polder.

Fig. 11. A damaged dike. The sea is on the right and the land <u>was</u> to the left. Damage occurred at the inner side.

Fig. 12. The first stage of the attack. The fissure on the crest is caused by the strong pressure difference in the ground water.

THE DISASTER IN THE NETHERLANDS CAUSED BY THE STORM FLOOD OF FEBRUARY 1, 1953

Fig. 13. The jetlike flow cut out a gully straight to the interior.

Fig. 14. Collapse of a dike.

THE DISASTER IN THE NETHERLANDS CAUSED BY THE STORM FLOOD OF FEBRUARY 1, 1953

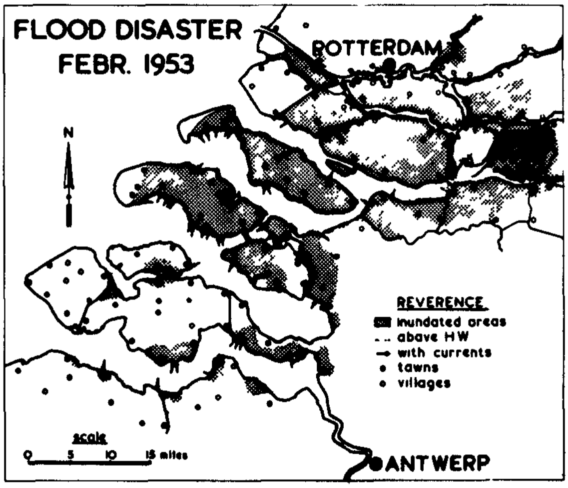

Fig. 15. Flooded Areas

The map (Fig. 15) shows the flooded areas. The stretches showing serious damage are indicated by small black squares. The arrows indicate the 67 breaches with fierce tidal currents. Where the sea defences gave way over a great length, the impact of an advancing steep waterfront struck down several houses and farms (Fig. 16). Other buildings were damaged later because of wave action in the inundated polders. Interior dikes, dividing the polders to a certain extent into smaller sections, did not protect the polders situated farther from the shore. The sections immediately behind the dikes were filled rapidly to sea level (Fig. 17) and from these the water overflowed the secondary dikes into the adjoining polders. This explains the enormous extent of the inundations.

A summary of the damage gives the following figures:

200,000 ha (800,000 acres) were flooded;
1783 men, women and children drowned;
100,000 people were evacuated;
47,300 houses were damaged, from which
9215 were badly or irrepairably damaged.

The damage to dikes, buildings, agriculture, livestock, etc. is estimated at 250 million dollars. The tidal currents, continually flowing in and out destroyed in little time, often on the first day of the disaster, the roads in the polders (Fig. 18) rendering it impossible to transport heavy equipment for rescue purposes. Hence it was extremely difficult to reach the villages and give immediate aid so urgently needed.

I feel compelled to mention here the extensive help which was offered in those days from all over the world. Especially I may bring here into remembrance the rescue effected by a large number of helicopters of the United States and other countries by which the lives of more than 2000 people were saved who could not have been helped in any other way. These signs of a growing brotherhood among men have met in the Netherlands deep feelings of rejoicing and gratitude.

Fig. 16. Damaged houses and farms.

Fig. 17. Water overflowing the secondary dikes.

Fig. 18. The tidal currents destroyed
the roads in the polders in a short time.

THE DISASTER IN THE NETHERLANDS CAUSED BY THE STORM FLOOD OF FEBRUARY 1, 1953

PROJECTS

After the sea had displayed its power of aggression, with not a little more fury than we had thought possible, the general opinion favors radical measures for an adequate protection of the Low Countries in the future. For this purpose extensive studies are in progress, under the direction of a board of prominent experts called the Delta-Commissie. The answer of the Low Countries to the challenge of nature in the beginning of this year will be:

1. The reclamation of the flooded areas, the rebuilding of the damaged farms, buildings and houses and the restoration of the agricultural possibilities of these fertile soils.

2. An investigation into the possibility of the enclosure of three of the five large estuaries in the southwestern part of Holland.

3. An investigation of the strongest possible gale and highest possible storm surge with modern oceanographic and hydrodynamic scientific means and the fixation of the design storm to some 2 or 3 ft. higher than the level of 1953.

4. The enlargement and heightening of the dikes as far as they remain exposed to gale effects.

5. Improvement of the dike-army of the local residents who come into action when there is any danger to limit destruction in the very beginning.

6. Improvement of the storm-flood-warning system and the forecasting from meteorological data.

These points give us a program for tens of years. Concerning the first point, the reclaiming of the inundated areas, we have already made great progress. From the 67 breaches with fierce tidal currents, 66 are already closed and we hope that the last one will be closed in the near future.*

As to the other five points of this large program, the reports of the Delta-Board will give further information from time to time.

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* The last gap "Ouwerkerk" (200 m wide, 20 m deep) was closed on 6 November 1953.

CHAPTER 19

THE RECONSTRUCTION OF THE NETHERLAND DIKES AFTER THE STORM OF FEBRUARY 1953

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INTRODUCTION

The reconstruction of the damage to the dikes by the flood of February 1953 presented an enormous task. From the hydraulic engineer's point of view the most interesting part was the closing of the major or tidal breaches, that is to say, the places where a dike for a certain length was totally destroyed and where, therefore, the tides had free entrance to the inundated interior, scouring out deep gullies. This called into action the resources of tidal hydraulics, theoretical considerations, and model experiments.

Throughout the region where the gaps occurred, the soil consists of a fairly resistant layer of clay of a few meters thickness at the surface, below which there is a fine sand of a very erodible nature with occasional lenses of clay or peat. Once the upper layer of clay has been cut through, the breaches as a rule deepen and widen rapidly by scouring of the fine sand. As the breaches become wider, the tidal movement in the flooded areas increases. The height of the land in the flooded areas is nearly everywhere below mean sea-level and often at or even below low water (say $1\frac{1}{2}$ to 2 m below mean sea-level). The areas are intersected by roads, usually somewhat higher than the land, and by ditches and canals. Villages and small towns are mostly situated on somewhat higher ground.

During most of the tidal cycle the land is covered with water and the tide propagates all over the area. Naturally the tidal flow tends to concentrate in the lower sections and in particular in ditches and canals which happen to have a suitable direction. These accordingly are scoured out and in some cases they have become powerful tidal creeks, starting from the breaches and gradually lengthening by regressive erosion. At several of the largest gaps, therefore, a system of radially diverging gullies had developed after some time.

In some of the breaches the upper layer of fairly resistant clay was cut through immediately by the inflowing water when the dike was breached. In many other cases, however, the gap remained relatively shallow at first and it took some time for the erosive action of the currents to remove the clayey sill. It was especially in these cases, as will be readily understood, that speed in undertaking and executing the closing was essential. It was, as a matter of fact, possible in a number of places to prevent the breaches from becoming really dangerous by a speedy attack and improvised methods. The principal material used in these activities has been the common sandbag. Especially during the first period after the catastrophe this was an article in great demand. Altogether 15 million sandbags

Fig. 1. Placing sandbags.

were used. They were transported by land, by water and by air to the danger spots. Not only sandbags were used, but also the stones from the revetment of neighboring dikes, the paving from the streets and many other improvised materials. Fig. 1 serves to give an impression of this activity.

In this way many smaller breaches were - at least provisionally - filled before they had a chance of developing into tidal channels. Not in all places, however, was this possible; not everywhere was sufficient labor available, or means of transport of material. Some breaches were inaccessible. Moreover during the first days and even weeks in several regions there was no time for any activity but saving as many lives as possible.

Also, of course, quite a number of breaches were so wide or so deep from the start as to render entirely futile any attempt at closing or even controlling them with improvised methods.

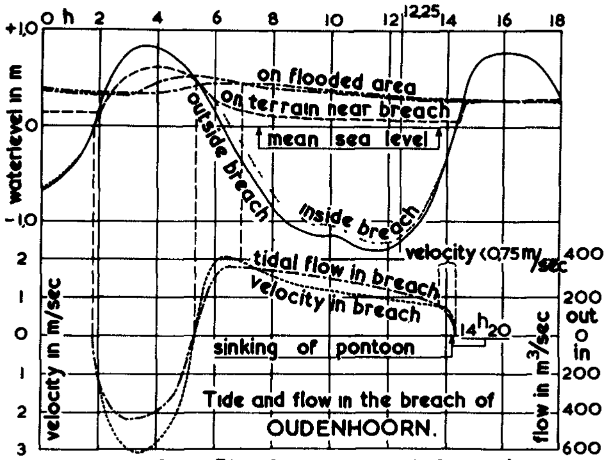

Fig. 2. Tidal curves at breach of Oudenhoorn.

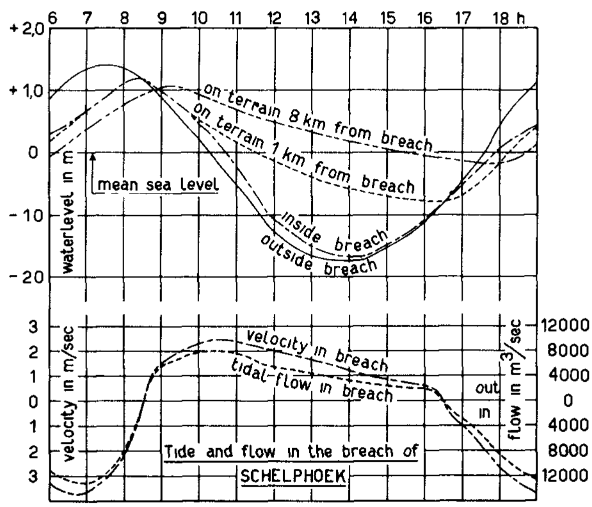

Fig. 3. Tidal curves at breach of Schelphoek.

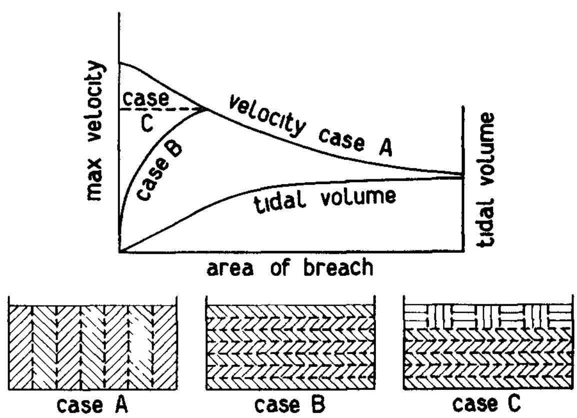

Fig. 4. Tidal volume and flow velocity as function of breach area.

After this first stage, say at the end of February, 67 tidal breaches existed, the largest of which was about 400 m wide, with a maximum depth of about 35 m, giving access to a storage area of 900 sq. km., and having a tidal volume varying between 100 and 150 million m^3, according to the variations of the tidal amplitude. This, as an illustration, is equal to the volume entering and leaving the mouth of the Rotterdam waterway in every tidal cycle.

TIDAL MOTION

The first task everywhere was to prevent the breaches from further scouring by protecting the bottom and the sides with fascine matresses and stone, the traditional Dutch practice, which has been followed for centuries without essential changes. At the same time an extensive reconnaissance by sounding, current measuring and observation of the tides in the flooded areas was carried out. By means of model experiments and tidal calculations these data (subject to continual and sometimes even fairly rapid changes) were correlated in order to obtain a comprehensive picture of the hydrographic and hydraulic situation, which could be used as a base for planning the strategy and tactics of the closing operations. This planning should be understood to be rather like the planning of a chess player who has to be ready to revise his strategy and tactics according to the moves of his opponent,and not the drawing up of a definite plan to be followed in detail to the very end.

In this stage it may be useful to make a few remarks of a general nature on the procedure of closing off a tidal storage area. Given a gap of a certain cross-sectional area feeding a certain area of storage lying at a certain level with regard to the mean water level outside, with a certain amplitude of the tide and a certain value of the resistance in the breach, in the gullies and on the terrain, a definite picture can be made of the tidal movement in the storage area and the associated flood and ebb flow through the gap. Figs. 2 and 3 show this condition for two of the breaches. If the gap is gradually narrowed the amplitude of the tides on the terrain will diminish and also the total flow will diminish, but the flow velocity in the gap will increase. This is presented for a hypothetical case in Fig. 4 (case A). When the gap is nearly closed, the rise and fall inside will be practically nil, but the velocity in the gap will be at its maximum. Given sufficient data at the start, it is possible to predict by calculation or model experiment to a sufficient degree of accuracy the trend of the changes and the values to be expected in every stage of the closing. For the sake of simplicity certain complicating factors are left out of consideration for the moment, such as the fluctuation of spring and neap tides, abnormal high tides by wind action and the opportunity that presents itself in some cases of eliminating a part of the storage area. These factors will find their place later, when a few cases are discussed.

GENERAL PROCEDURE OF CLOSING

With the tidal ranges encountered in the Southwestern estuaries of the Netherlands (3 to 5 m) it is found that the flow velocities at the end will be too high for any of the available materials (sand, clay) except heavy stones. A dam of stones, however, besides being expensive, takes a long time to build, especially as the slack water periods at the transition between flood and ebb flow, and vice versa, become progressively short. Moreover, the continuous transport of large quantities of stone presents difficulties. It should be remembered that stone has to be imported from other countries (Belgium or Germany) and that also much stone is needed for ballasting the brush-wood mattresses.

In most cases, therefore, another method has been followed. This consists in narrowing the gap to a certain width, wide enough for the velocities to remain below a critical value. This value is not a constant, but depends on the local conditions, the nature of the soil, etc. In general it should not be over 3 to 3.5 m/sec. at an average tide. The gap thus left is then closed in a single operation. In this way the last stage of the closing, the stage in which the flow velocities grow to dangerous values, is, as it were, cut off. Our ancestors have followed a similar method. They used a ship for the final operation. This has also been done now in some cases, but mostly use is made of concrete pontoons.

There was one case, however, in which this pontoon method could not solve the problem. This was the largest breach of all. Here, in order to keep the flow velocities below the critical value, a final gap of something like 3000 m^2 would have had to be closed in one operation. This would have meant the use of, for instance, a pontoon 200 m long reaching to 15 m below mean sea level, which was out of the question.

Here it was necessary to follow a different method, a method which would avoid increasing the flow velocities to excessive values. This can be done if the gap is narrowed by building up uniformly from the bottom. In this case, when the crest reaches a certain level a state of maximum overflow prevails. Then the downstream water level has no influence on the velocity of the flow any more, and further restriction of the gap does not lead to higher velocities (Fig. 4, cases B and C).

The crux of this method is to have sufficient width so that the velocity at the stage where the maximum overflow occurs is below or equal to the critical velocity which can be accepted under the prevailing conditions. The need for the instantaneous closing of the remaining gap is thus eliminated. This might be termed the high crest closing method. A sufficient width for this purpose is usually not to be found in the breach itself. Moreover, the actual breach is as a rule not suitable for building up a dam with a fairly high crest, among other reasons, because of its great initial depth. When applying this method of closing, one is therefore led to leave the breach alone and to construct a new embankment some distance inland, encircling the breach and thereby sacrificing temporarily a certain area of land. The base of this embankment is then on the level of the terrain, so that a crest of sufficient height for insuring maximum overflow is more easily obtained. As will be shown later, in the case of Schelphoek, our largest breach, a retreat of about 1 km was made and an embankment of 4 km length was constructed instead of directly closing the 500 m wide gap. It will also be shown that as the situation developed, it was not a high crest closing procedure in the full sense, as had originally been planned, but a case of the chess player on the other side of the board obstructing the well thought-out moves.

After this discussion on general procedure, three breaches have been selected for a more detailed treatment, first one of the smaller ones that could be finished relatively early, the largest one, and the one that unexpectedly got out of control and consequently was the last gap of all to be closed.

THE BREACH OF OUDENHOORN

A week after the gale, the breach of Oudenhoorn had a width of about 70 m and a depth of $5\frac{1}{2}$ m below mean sea level (Fig. 5). At that depth a layer of fairly resistant clay was encountered which for the time being prevented further erosion. At first the area flooded by this breach was 37 sq. km, but by repairing some inland embankments it was soon possible to reduce the area to 26 sq. km. The terrain was on the average situated slightly above mean sea-level. The tidal volume was about 10 million m^3, which caused maximum velocities in the gap of about 3 m /sec. Fig. 2 shows a few typical tidal curves and the velocity curve in the gap. These curves represent a neap tide. At spring tide the tidal differences and the velocities are higher.

For closing the breach two concrete ships were at hand, which had been used as floating tanks. They had a length of 25 m each and by combining the two by means of steel girders and sheets, a unit could be obtained of 51 m length, 5.8 m in width and 5.6 m in height. At both ends a steel scaffolding was erected supporting steel sheet piles which could be dropped by removing a ratchet. This construction was christened, "guillotine".

In the time needed to lash together the two hulls and to equip this unit with the guillotine and other necessities, the bottom of the gap was raised by brush mattersses and stone to 4.4 m below mean sea-level. The top of the pontoon (5.6 m high) therefore reached to 1.2 m above mean sea-level; that is, the top was slightly above mean high water. The ends of the dike at both sides of the gap were more or less trimmed by stone.

The closing operation had to be carried out at slack tide, either at high water or at low water (Fig. 2). For two reasons the low slack tide, that is, at the end of the ebb flow was indicated. In the first place, the period of moderate velocities during which the unit has to be maneuvered into position and sunk was somewhat longer. In fact, for about fifteen minutes the flow was below 3/4 m/sec. In the second place, less time would be needed for sinking, because of the lower initial level. As the draft of the pontoon was 2.2 m and the expected level at the slack time would be about mean sea-level, the pontoon had to sink over 4.4 - 2.2 or 2.2 m. For sinking the pontoon, the sides of the hulls were pierced and the holes were closed by means of flaps which could be opened by removing a ratchet.

Everything was ready for the closing operation on February 28.

Fig. 5. Breach at Oudenhoorn.

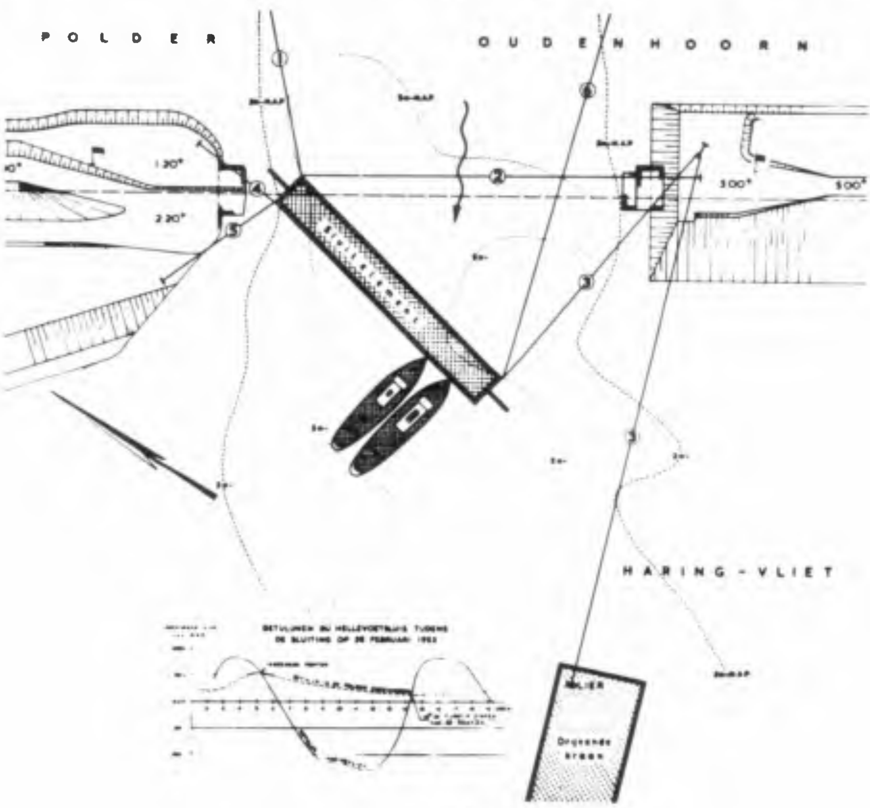

Fig. 6. Closing scheme Oudenhoorn.

Experiments had been carried out in the hydraulic laboratory from which the method shown in Fig. 6 was chosen. The pontoon was moored beforehand at one end by means of cables 1, 4, and 5. The actual maneuvering into position was effected mainly by cable 3, controlled by a winch of a floating crane at anchor. Cables 2 and 6, as well as 1, served only for emergency purposes.

As soon as the ebb flow had slackened sufficiently, the operation was started. There was a slight delay because the pontoon touched bottom on a protruding edge of a fascine mattress which had not sunk properly. As the tide was rising during the operation, the pontoon floated free just in time. It was maneuvered exactly into position and sunk within a few minutes (Fig. 7).

This however was by no means the end of the operation. The pontoon with its equipment had a weight of about 700 tons. As can be seen from the tidal curve, the water was rising rather rapidly at the moment of closing and very soon at high water the pontoon would have to withstand a head of say 3/4 m, that is to say a horizontal pressure of some 200 tons. Moreover the outward pressure at low water would be nearly twice as much. As from previous experience and from special experiments it was established that the friction coefficient of a concrete pontoon on a bed of dumped stone is about 1 : 3, the total weight had to be brought within a few hours to at least 1200 tons. This was done by pumping sand onto the hulls, for which purpose a pipeline was laid on the pontoon, ready to be connected to a pipeline along the dike immediately after sinking. At the same time stones were dumped between the ends of the pontoon and the dike ends, and both in front of and behind the pontoon, to be followed later by clay in order to seal off the leaks left by the imperfect fitting of the concrete pontoon on the rough bed.

Upon completion of the above operation,a full dike of sand covered by clay and stone revetment was built around and on top of the pontoon and this breach was completely closed.

THE BREACH OF SCHELPHOEK

The largest and most difficult breach was that of Schelphoek on the island of Schouwen (Fig. 9). At the time of the first survey it was 300 m wide and the greatest depth was 30 m (Fig. 8). Later it grew to a width of more than 500 m with a maximum depth of 35 m. Gullies, developing into large tidal creeks, were eating their way inland at a rate of 300 to 400 m per week (compare Figs.10 and 11). The tidal volume was, after some time, between 120 and 150 million cubic meters, with a maximum flow of some 14,000 cubic meters per sec, which is more than the maximum discharge of the Rhine. A picture of the tidal motion is shown in Fig. 3.

It was quite out of the question to deal with this grandfather of all breaches in the same way as at Oudenhoorn. Therefore, it was decided

Fig. 7. Pontoon under way.

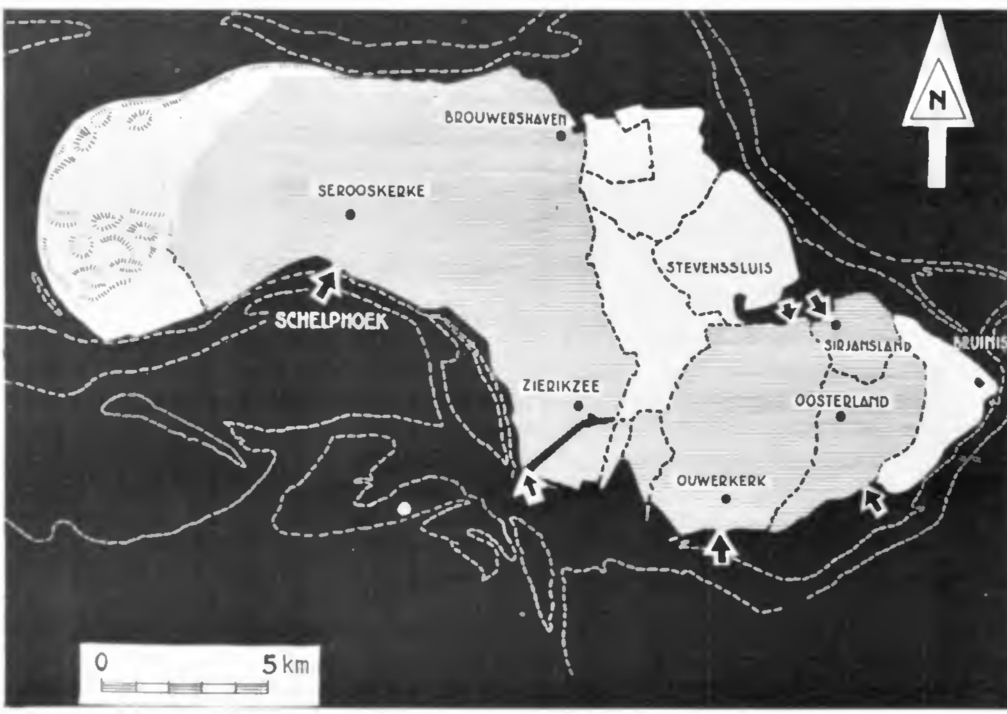

Fig. 8. Island Schouwen-Duiveland.

Fig. 9. Breach Sohelphoek at high water.

to encircle the breach by a high crest overflow dam. Calculation showed that its length should have to be about 3 km. In the center, just opposite the breach, was the village of Serooskerke, at a distance of nearly 1½ km. This was situated slightly higher than the general level of the terrain, whioh varied between 1 and 1½ m below mean sea level. As the higher ground around the village did not contribute materially to the flow to and from the breach,and in order to keep the village inland of the embankment, this was fixed at a short distanoe in front of the village at about 1 km from the gap. From there an alignment was projected, ourving gradually to the dike at both sides of the breach (Fig. 12). The embankment was to join the dikes at looations which for practical reasons offered suitable positions for floating pumping stations delivering sand for the embankment by means of pipelines. On the higher ground in front of Serooskerke the embankment was built immediately to above the high-tide level by hydraulic fill, using a suotion dredger with floating pipeline, pumping sand between clay mounds built with olay from the site by clam shell dredges. The same was done for a short distanoe from the dikes at both sides of the breach in order to oreate starting

Fig. 10. Aerial view May 1, 1953.

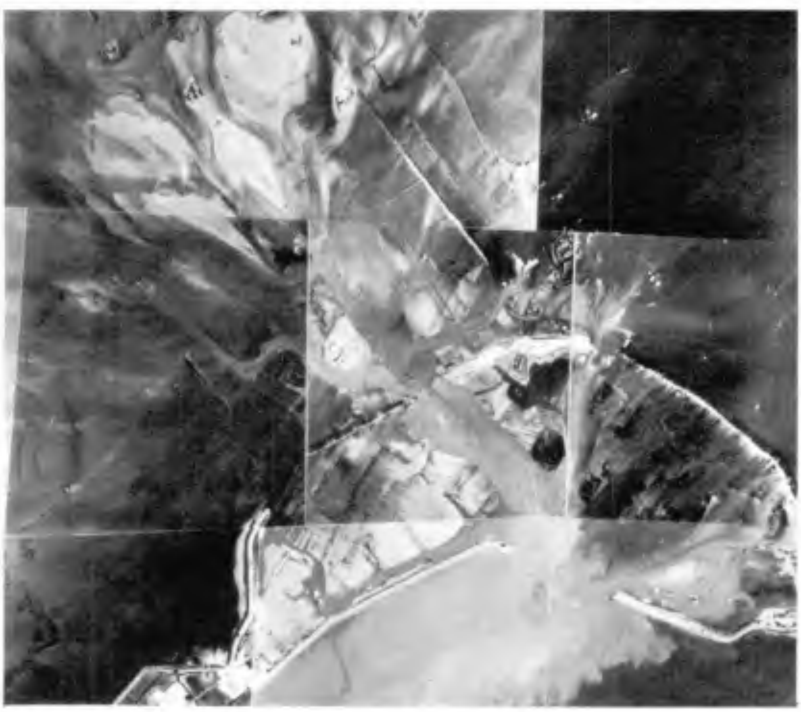

Fig. 11. Aerial view August 1, 1953.

Fig. 12. Alignment of embankment at Schelphoek.

points for the construction of the final embankment. Both east and west of the village in this way there remained some 1500 m, making 3000 m altogether for the overflow crest, and the total length of the embankment was to be 4 km.

According to the plans a 40 m wide strip of brushwood mattress had to be laid and loaded with stone over the entire length of 3 km. This would form a crest, varying in level from mean sea level to 1/2 m below it, sufficient to create maximum overflow conditions everywhere (Fig. 13). On this foundation concrete pontoons 11 m long, 7.5 m wide and 2 to 3 m high were finally placed so as to reach everywhere to at least 2 m above mean sea level (slightly above spring high water). When sunk and filled with water, these pontoons were calculated to be able to withstand the pressure of the tidal differences. They were to form the nucleus of the final embankment.

The project however could not be carried out unchanged. While the construction of the embankment at the center and at the flanks, and at the same time the sinking and loading of the brushwood strip was progressing, two of the gullies eroded their way inland beyond the projected alignment of the dam. One of them eroded at such a rate in the northwesterly direction that a shifting of the alignment of the dam in order to keep ahead of it was out of the question. Therefore, the project had to be revised in the following way. The embankment was to cross the gullies in a suitable place. The bottom and sides of the gullies were to be covered by fascine mattresses in order to prevent further deepening and widening at the sites of the crossings. Before starting to place pontoons on the brushwood strip the gullies were to be closed by means of concrete pontoons.

As this revision of the project entailed an increase in the amount of fascine work and therefore prolonged the time needed for this part of the project, it was decided meanwhile to extend the embankment at full height at the two extreme ends of the alignment. This was made possible from a hydraulic point of view because the cross-sections of the gullies were added to the total cross-section area available for the flow. Accordingly the length of the high crest dam could be safely diminished to 900 m at the west side and 1100 m at the east side of Serooskerke.

It took some time to get the work under way. There was no harbor or shelter within easy distance to serve as a base and as a refuge in case of bad weather for the fleet of dredges, cranes, barges, tugs, survey-vessels and other floating equipment needed for a job of this scale. There was no accomodation for the laborers, no suitable site for workshops, office huts, no so called "zate" for construoting the brushwood mattresses. A "zate" is a beach or other fairly even surface, dry at low water and just submerged at high water to allow for floating the finished mattresses. All this had to be created first. Two harbors, one to the west and one to the east of the breach, were dredged out in bights sheltered by protecting ends of abandoned dikes from flood disasters in the past (in this region history records repeated retreats from the assaults of the sea). The spoils from the dredging were used for raising areas for huts, workshops, etc. Because of these necessary preparations it was near the end of May when the activities were in full swing. By that time there were in action around the island of Schouwen-Duiveland alone, 5 bucket-dredges, 33 suction dredges, 126 tugs, 266 barges of different description, 17 floating cranes, 105 draglines and numerous smaller units. The greater part of this equipment was employed at the Schelphoek breach.

In the operation 15,000 tons of stone, 30,000 m^3 of clay and 300,000 m^3 of sand were delivered and used per week. Also 15 to 20,000 m^2 of brushwood mattresses were oonstructed weekly, sunk and loaded with stone. Near mid-August the 2 km brushwood sill was put in place, the two gullies were provided with a protection against further erosion of the bottom and the sides, and the embankments in the center of the two ends of the alignment were in suitable condition. The velocities in the gullies had inoreased by then to $3\frac{1}{2}$ to 4 m per sec. It was time to start the closing operations of the gullies.

The biggest type of concrete pontoon available was the Phoenix Ax from Mulberry Equipment, a small number of which had been left in England from the war. They were 62 m long, 18 m broad and 18 m high, when floating they had a displacement of 7000 metric tons. They were brought over from England for use in closing the largest breaches.

Fig. 13. Overflow crest.

Fig. 14. Closing east gap.

The gully west of Serooskerke was 100 m wide, and the base of dumped stone on the brushwood mattresses which was to receive the pontoon was at 10 m below mean sea level. Before placing the pontoons, the gap had to be trimmed to the length of the pontoon. This was done by placing smaller pontoons which had been constructed since February. It was realized immediately after the flood that concrete pontoons in great quantities would be needed for the closing of the numerous breaches and accordingly orders were placed with a group of several large contracting companies for the construction of standardized units which could be combined into pontoons of different sizes, according to the needs. The dimensions chosen for the units were 11 m long and 7.5 m wide. The assembled pontoons accordingly could have lengths of either n x 7.5 or n x 11 m and heights of 2, 4, 6 etc. up to 14 m. It was also possible by slight modifications to have intermediate heights.

By means of combining suitably chosen pontoons from this stock and dumped stone, the width of the west gully was restricted to 60 m. This immediately resulted in a local increase of the flow velocities and in increased erosion along the edges of the protective layer of the brushwood mattresses. The most dangerous factors were the vortex trails starting from the edges of the outer pontoons. The protection, originally practically flush with the bed of the gully, formed a sill with undesirably steep slopes. In the extremely fine sand slides began to occur, which resulted in subsidence of some of the concrete pontoons restricting the width. At that moment an important decision had to be made. Either the big pontoon for closing off the gully would have to be put into place immediately or this would have to be delayed until the solidity of the base layer could be improved by extending and reinforcing the protective layer and dumping more stone.

If the first course was to be followed, further erosion could be stopped but the danger of the entire construction sliding down into one of the deep excavations of 30 m at both sides of the sill was a possibility. Moreover, the closing of the west gully would have to be followed with a minimum of delay by the closing of the east gully, because the increased flow would increase erosion in that place.

On the other hand, it was by no means certain that it would be possible by continued sinking of mattresses and dumping stones to gain the upper hand of the rapidly progressing erosion. Moreover, the end of August was nearing, which would bring high spring tides and the possibility of an early autumn gale - they have been known to occur in the first days of September

It was a matter of weighing chances, none of which could be assessed quantitatively. Yet it was felt by everyone concerned that the bold course had to be taken; accordingly, on the 18th of August the big pontoon was maneuvered into place and sunk, as usual at the end of the ebb tide. Again the maneuver had been carefully rehearsed in the hydraulic laboratory and the operation went exactly according to schedule.

Two days later a pontoon was placed in the east gully. Although this was a smaller pontoon, the operation was difficult because it had to be carried out at the slack tide near high water as the depth at low water was insufficient. The time available was very short, and on this occasion the pontoon had to be held by cables and eased-off with the decreasing flood current instead of pulled against the last ebb flow. However, once again all went well and the most dangerous part of the closing of the Schelphoek breach was past.

Then followed a time of feverish activity in consolidating the position of the two pontoons and sealing the gaps, left below and at the sides, with stone, clay and ultimately sand. At the same time the two stretches of the alignment on the terrain were closed by means of a large number of smaller pontoons (2 to 3 m high) mostly operated in strings of 4 to 6 (Fig. 14). By the end of September the entire ring was closed, although it still took much labor and care to seal all leaks and to convert the slender line of rather flimsy concrete boxes into an embankment capable of withstanding the winter gales. By that time the hope of also closing the breach itself before winter had been given up. It had been planned to use for that purpose 4 or 5 of the big Phoenix pontoons, but as it turned out they were needed at another place. This was the breach at Ouwerkerk, the last one to be closed and the last one we will discuss.

THE BREACHES AT OUWERKERK

Ouwerkerk is a small village in the eastern half of the twin island Schouwen-Duiveland (Fig. 8). This half is divided by two inland embankments into three polders, all of which were flooded. The sea entered through five tidal breaches, two of which were situated near to each other just south of Ouwerkerk. The three other breaches all could be closed without too great difficulties. Also the two embankments separating the polders had been repaired by the beginning of August. This had been a much more troublesome affair because of the difficult transport and the impossibility of putting big equipment to work.

In front of the dike at Ouwerkerk there was a shallow shelf or foreland several hundred meters wide. As the dike between and adjoining the gaps was badly damaged also, it was decided to construct a new dike in front of the old one over a length of 2.2 km (Fig. 15). Starting from the breaches in the dike, gullies had been scoured out both in the polder and through the foreland. It was decided to build the parts of the dike on the higher stretches of the foreland first and to leave the gullies for the final closing. Of course the bottom and sides of the gullies were protected first with brushwood mattresses.

The area served by the two gaps was 27 m^2 with a tidal volume of about 40 million m^3. It was calculated that a total area of the two final gaps of 1200 m^2, 800 m^2 east and 400 m^2 west, would be sufficient to keep the maximum velocities below the limit of $3\frac{1}{2}$ m/sec. Both gaps were to be closed by means of concrete pontoons assembled from standard units on two successive days in a week of neap tides.

Fig. 16. General View.

Fig. 17. East gap closed with four Phoenix pontoons.

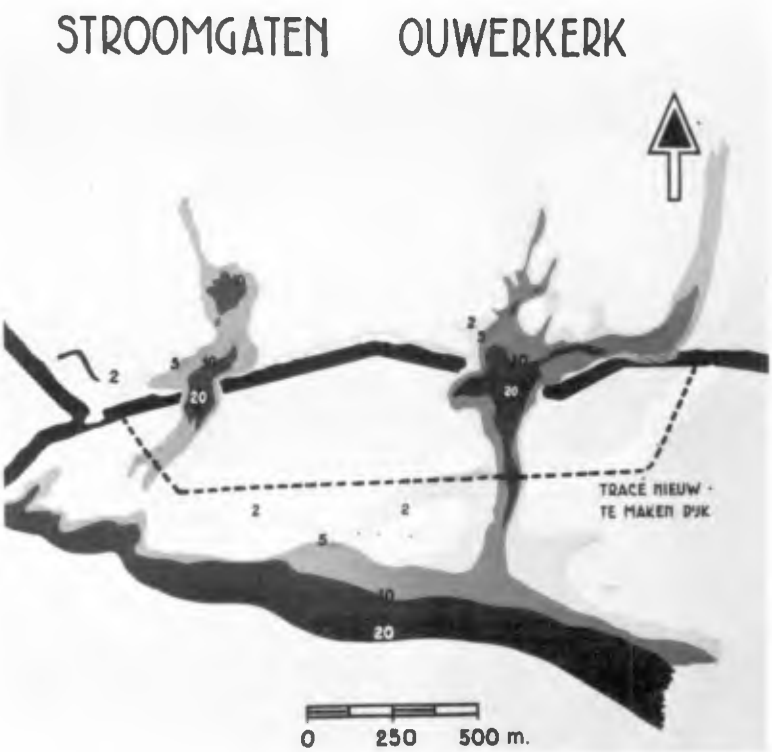

Fig. 15. Scheme for closing breaches at Ouwerkerk.

Compared with Schelphoek it seemed to be a simple and straightforward affair, which proceeded exactly to schedule nearly to the very end. With the approach of the closing operations which had been planned for the week of 20-25 August, difficulties began to arise. Here again pontoons were placed by way of abutments, in order to trim the gap down to the proper size for the closing pontoons. Almost immediately afterwards, the sill formed by dumping stone on mattresses in the west gap on which the pontoon was to rest began to settle at the edges. It became dangerously narrow and the dumping of additional stone did not yield material results. It seemed as if sand flowed from under the mattresses into the deep excavations at both sides of the sill. On the 22nd of August the abutment pontoons one after the other began to settle and to tilt.

As at Schelphoek, it was another case of now or much later, and as it was felt that with the available facilities for constructing mattresses and handling stone (it should be remembered that this coincided with the crucial period at Schelphoek) did not insure gaining advantage over the erosion - especially with the spring tide approaching - it was again decided to venture on the bold course.

On Sunday 23 August 1953 two strings of five standard units each, 6 m high, were let down into the west gap at slack tide between flood and ebb. It was a delicate operation because the time for maneuvering was short and the narrow sill made it imperative to place both strings at a slight angle to each other with an accuracy of less than one meter. In the utmost tension the operation was carried out. After the pontoons had been sunk, it soon became evident that in the center three units were slightly outward of their true position. They slid obliquely down, but then were held in a position with the top at about low water level.

In these circumstances it was not possible to close the east gap on the next day, as had been planned. This would have meant an important increase in the pressure on the pontoons in the west gap, because the level within the polder then would have remained practically constant somewhat above mean sea level, and it was practically a certainty that at low water outside the entire pontoon barrier would have been swept away.

In that case there would have been left a 400 m^3 gap with maximum velocities of $5\frac{1}{2}$ or 6 m/sec. There was no other possibility but to try to consolidate the barrier in the west gap first. This was eventually done by means of additional pontoons, stone, clay and finally sand. But meanwhile the flow through the east gap increased to 5 m/sec and in a few days the bed protection together with the abutment pontoons became a shambles. Every thought of using this gap as a site for a closing operation by pontoons had to be abandoned. Fig. 16 shows a general view of the situation at this stage.

After careful survey of the new state of affairs by sounding, gauge-reading and flow measuring, an entirely new scheme was eventually developed. This consisted in preparing a new site for placing a pontoon barrier just outside the breach in the old dike. Four Phoenix AX pontoons were to be sunk on a bed of brushwood mattresses and stone at 10 to 15 m depth. As the enclosure dam at Schelphoek by this time was fairly well consolidated, it was possible to concentrate a larger portion of the equipment and manpower on Ouwerkerk. Favored by the exceptionally fair autumn weather, the work proceeded practically without incident so that in the night of 6 to 7 November the last pontoon could be placed according to schedule (Fig. 17).

The old motto of the Province Zealand: "Luctor et Emergo", once more had been made good.

Part 4

DESIGN OF SHORELINE STRUCTURES

CHAPTER 20

THE INFLUENCE OF SUBSURFACE CONDITIONS ON THE DESIGN OF FOUNDATIONS FOR WATERFRONT STRUCTURES IN THE GREAT LAKES AREA

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The purpose of this paper is to describe subsurface conditions beneath the principal waterfront areas in the Great Lakes region and to discuss certain foundation problems associated with waterfront structures in this locality.

GEOLOGICAL SETTING

INTRODUCTION

During the slow and oscillating withdrawal of the continental ice sheets from the locality of the Great Lakes, various bodies of water existed in the basins now filled by the Great Lakes themselves. At different times each of the basins was occupied by larger lakes or by lakes standing at higher levels than at present. Therefore, the waterfront cities of the present time, now located along the shores of the existing bodies of water, lie almost without exception in areas once occupied by glacial lakes. This fact may be observed in Fig. 1, which shows the maximum extent of the former glacial lakes and the locations of selected cities to be discussed in more detail in this paper.

In some instances the glaciers deposited clayey tills directly from the ice onto the existing ground surface or onto the bottom of the glacial lakes. In other localities, or at other times in the same locality, the glaciers discharged their debris into the waters of the lakes, and the materials were sorted by sedimentation through the lake waters. Thus, most of the important waterfront cities on the Great Lakes are underlain by deposits of clay, some of which are directly of glacial origin, and others of which should be classed as lacustrine glacial clays.

Inasmuch as the lake levels occasionally stood nearly constant for considerable periods of time, the clay deposits are frequently covered by sandy deposits representing old beaches. In some of the Great Lake cities, the sand deposits are of great thickness and importance; in others they are absent or insignificant.

During periods of exceptionally low water levels the drainage systems eroded substantial valleys into the terrain. If the lake levels then rose slowly, the mouths of these valleys often became filled with swamp vegetation mixed with silt brought in by the streams. Thus, valleys filled with soft organic deposits are often encountered in the clays. These filled valleys are sometimes covered by beach deposits, but their existence is often revealed by depressions in the existing land surface, and they are often approximately followed by the courses of present rivers. Since many of the Great Lakes cities are established at the mouths of rivers, many of the most important industrial locations are underlain by such buried valleys.

Thus, it is seen that the subsurface materials beneath the more important ports in the Great Lakes area are likely to consist of clay tills, of lacustrine clays, or of various organic shore deposits.

CONSISTENCY OF DEPOSITS

The principal problems associated with waterfront construction in the Great Lakes area arise from the presence of deep deposits of clay or organic materials. Until recently the consistency of such materials was described by such terms as very soft, soft, stiff, etc. These classifications were subject to misunderstanding and to various interpretations. Therefore, the ability to deal with foundation problems in the region depended to a great extent on the experience and personal judgment of those charged with the responsibility for the work.

Since the advent of soil mechanics about 30 years ago, the consistency of these materials has been determined quantitatively by means of a simple test known as the unconfined compression test. A cylindrical sample of the soil, in virtually intact state, is inserted in a testing machine and subjected to vertical load until it fails. The pressure on a horizontal cross section of the sample at the time of failure is designated as the unconfined compressive strength. A general impression of the meaning of the various numerical values for the unconfined compressive strength can be obtained from Table 1.

Numerous borings have been made in the region of the Great Lakes from which relatively undisturbed samples have been obtained and unconfined compressive strengths determined. Diagrams representing the results of borings typical of various important industrial regions are shown in Figs. 2, 3 and 4.

In the discussion of the boring logs, one should bear in mind that all the deposits adjacent to the Great Lakes are of extremely erratic character. Therefore, in one sense, there is no such thing as a typical boring. Nevertheless, each of the Great Lakes cities has subsurface characteristics that differentiate it from the others. The boring logs shown in Figs. 2 to 4 have been chosen to illustrate the most prevalent or characteristic conditions.

THE INFLUENCE OF SUBSURFACE CONDITIONS ON THE DESIGN OF FOUNDATIONS FOR WATERFRONT STRUCTURES IN THE GREAT LAKES AREA

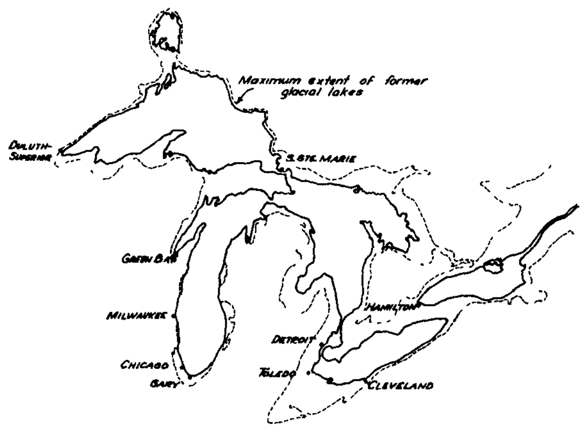

Fig. 1. Location map of locations discussed.

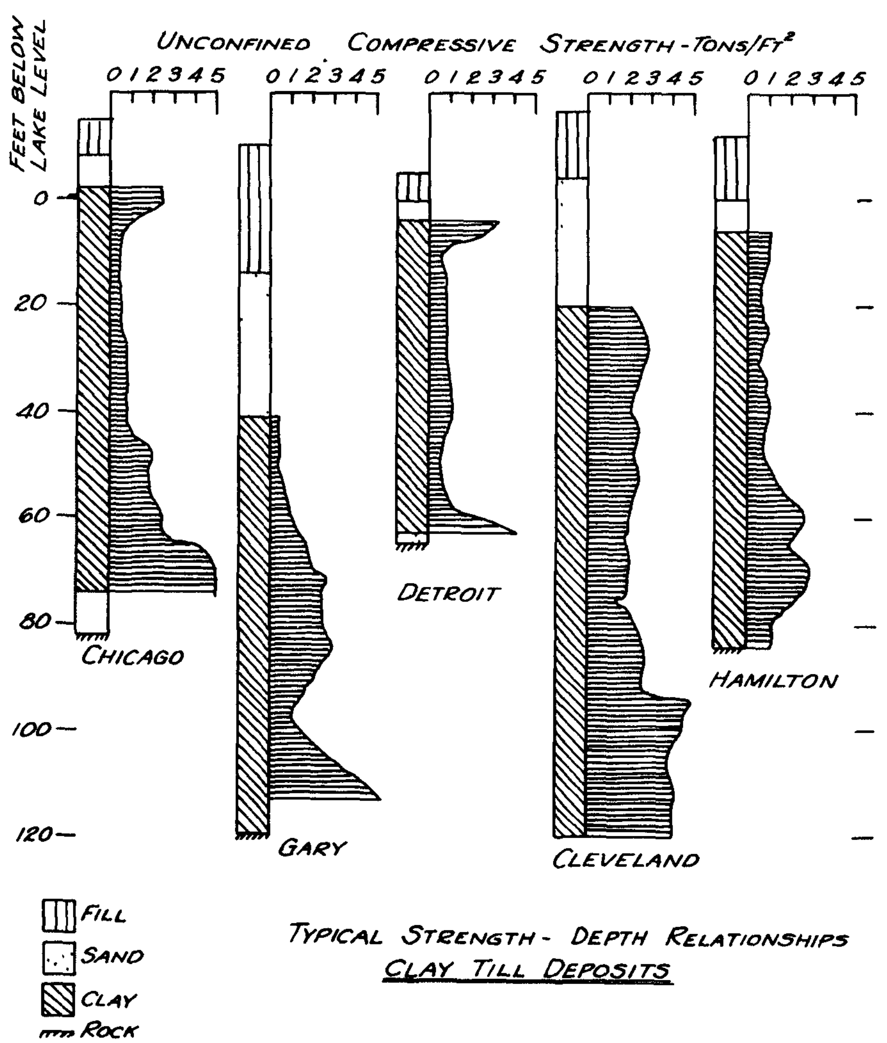

Fig. 2.

Table 1. Qualitative and Quantitative Expressions for Consistency of Clays

Consistency	Field Identification	Unconfined Compressive Strength (tons/sq ft)
Very soft	Easily penetrated several inches by fist	Less than 0.25
Soft	Easily penetrated several inches by thumb	0.25-0.5
Medium	Can be penetrated several inches by thumb with moderate effort	0.5 -1.0
Stiff	Readily indented by thumb but penetrated only with great effort	1.0 -2.0
Very stiff	Readily indented by thumbnail	2.0 -4.0
Hard	Indented with difficulty by thumbnail	Over 4.0

CLAY TILL DEPOSITS

The localities of Chicago, Gary, Detroit, Cleveland and Hamilton are underlain primarily by glacial clay till. In Chicago, Fig. 2, there are found a few feet of fill, a few feet of sand, and a relatively deep deposit of clay. It may be seen that the upper 3 to 4 ft of the clay constitute a stiff crust having an unconfined compressive strength of about 2.5 tons per sq ft. The crust merges rapidly into relatively soft to medium clays having a strength not greater than 1 ton per sq ft for a depth of approximately 40 ft. With increasing depth, the stiffness of the clay increases in rather well defined steps. As a rule, each deposit of clay having a characteristic compressive strength represents one unique sheet of till associated with a specific advance and retreat of one of the ice sheets. At a depth greater than 60 ft the clay becomes very hard and may be underlain by granular materials. The bedrock consists of limestone. It is commonly encountered about 80 ft below lake level.

It is obvious that the principal difficulties with foundations in the locality of Chicago are the result of the 20 to 50 ft of soft and medium clays directly beneath the stiff crust. Under moderate load these clays are relatively compressible and cause large settlements. They are also responsible for large lateral pressures against bulkheads and wharves. Piles or piers find their support in the stronger underlying materials.

THE INFLUENCE OF SUBSURFACE CONDITIONS ON THE DESIGN OF FOUNDATIONS FOR WATERFRONT STRUCTURES IN THE GREAT LAKES AREA

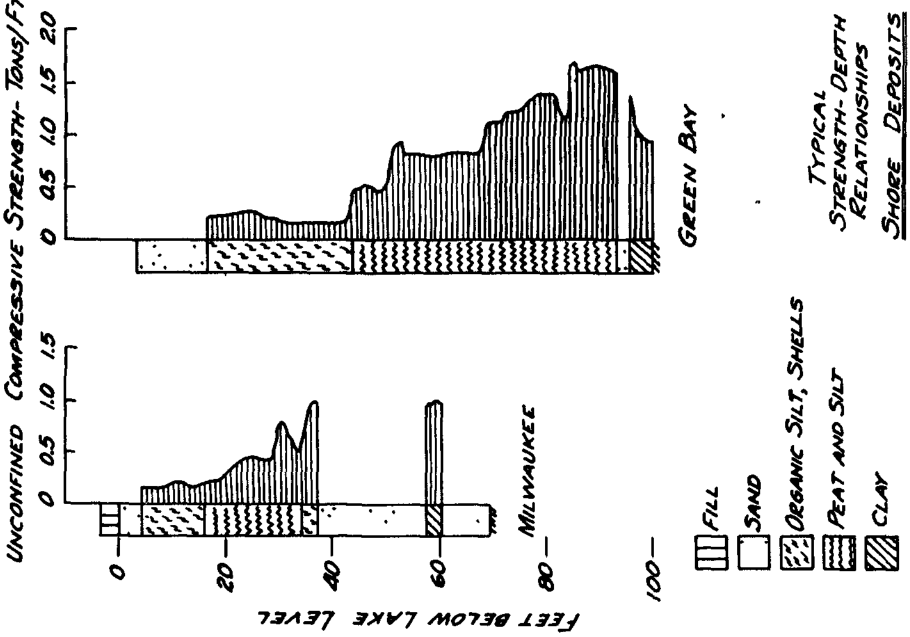

Fig. 4.

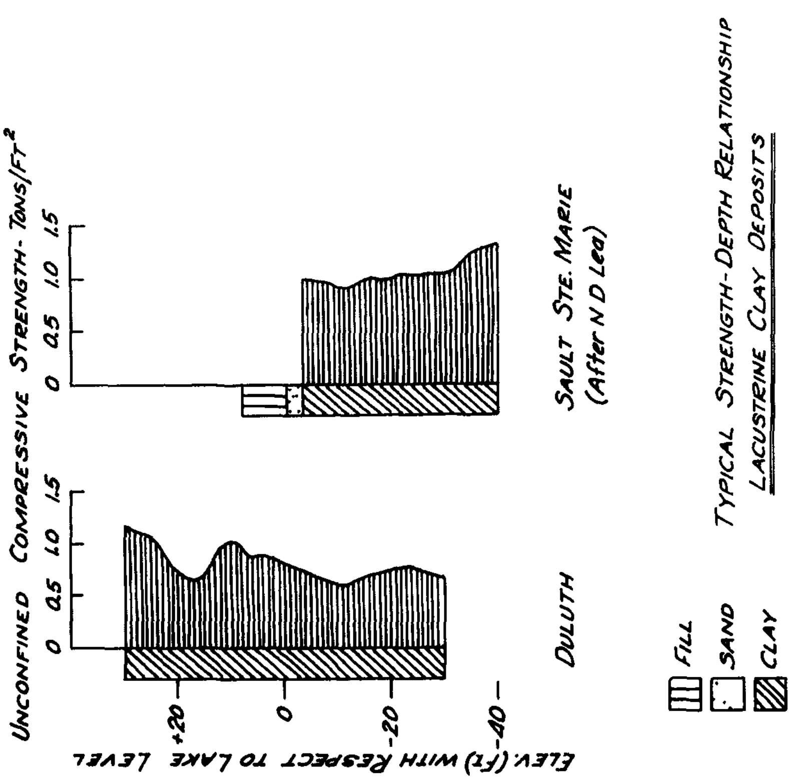

Fig. 3.

The subsurface conditions in northern Indiana are exemplified by the boring log from Gary. The clays at Gary are somewhat similar to those in the Chicago area, but the surface of the clay is approximately 40 ft below lake level. Above the clay is found sand, or fill which itself usually consists of sand. The sand is relatively dense and forms a blanket capable of supporting most light structures. Beneath the sand is encountered clay which may be of soft to medium consistency, but which usually is stiff to very·stiff. Only the heaviest and largest structures experience excessive settlement as a result of the presence of the underlying bodies of clay. Bedrock is somewhat deeper than in Chicago, generally about 120 ft below lake level.

The subsurface conditions in Detroit also have a general similarity to those in Chicago except for the fact that clays having an unconfined compressive strength less than 1 ton per sq ft extend almost the full distance from the clay surface to the bedrock. Therefore, waterfront structures in Detroit are usually more expensive to construct than those in the Chicago area, because the clays do not become stiffer with increasing depth.

In the main industrial section of Cleveland, located in the valley of the Cuyahoga River, the subsurface conditions are somewhat different from those previously described. Some 20 to 30 ft of sand and fill lie on top of the deposits of clay. However, at one time the sand was at least 100 ft thick and the underlying clays were subjected to much greater vertical pressures than they now experience. Therefore, they are all relatively stiff. The thickness of the clays is locally very great, because this part of Cleveland is underlain by a pre-glacial valley excavated into bedrock to depths as great as 600 ft below present lake level. It is obvious from a study of Fig. 2 that only the heaviest and largest structures in Cleveland should experience difficulty on account of the presence of the underlying clay.

The boring log characteristic of Hamilton, Ontario, shows many of the characteristics of that of Chicago and the subsurface conditions are quite similar.

LACUSTRINE CLAY DEPOSITS

The clays laid down in glacial lakes differ from the clay tills in that they do not contain stones or pebbles of large size, are likely to consist of laminations or bands of clay and silt, and are in general somewhat softer than the till deposits. Typical of these formations are borings from Duluth and Sault Ste. Marie, Fig. 3.

The boring at Duluth penetrated clays of medium consistency to a depth of about 50 ft, but the boring was not carried to the full depth of the clay deposits. It is probable that the compressive strength decreases still further as the depth increases. At Sault Ste. Marie the lacustrine clay appears to have an almost constant strength of about one ton per sq ft for a depth of about 40 ft. The clays at both localities are relatively compressible and are not blanketed by deposits of sand or stiff clay. Therefore, difficulties with waterfront structures are not uncommon.

SHORE DEPOSITS

The most significant characteristic of the various shore deposits or buried valleys is their extreme variability. The subsoil profiles are likely to contain sand, organic silt, shells, peat, and lacustrine clays in a completely heterogeneous array. The examples chosen, Fig. 4, from Milwaukee and Green Bay are illustrative of the great variability of the unconfined compressive strength of the organic silty deposits. It is seen that the compressive strength in both localities is at some levels somewhat less than 0.2 ton per sq ft. Such materials are incapable of supporting even the lightest structures and exert practically fluid pressures against bulkheads. Therefore, foundation problems are serious in both cities, and numerous examples of excessive settlement and of the lateral movement of retaining structures could be cited.

ORE STORAGE AREAS

INTRODUCTION

All standard types of waterfront construction are used along the shores of the Great Lakes. Most of these present no unusual features and do not deserve special discussion. In one respect, however, the waterfront facilities of the Great Lakes area are unique. Because the lakes are frozen for several months during the year, such heavy commodities as iron ore and limestone cannot be brought continually to the steel mills or other manufacturing centers where they are utilized. Therefore, storage spaces must be provided for these commodities in order to keep the plants in operation for a period of four to five months each year. Since space along the waterfront in the industrialized areas is at a premium, there is a natural desire to store such materials to the greatest possible height. At the same time, the unit weight of iron ore is relatively great (about 160 lbs per cu ft). Hence, the storage areas constitute large tracts of ground subjected to exceptionally high unit pressures. Many fields for the storage of iron ore are customarily piled to a height of at least 50 ft and exert a pressure of from 4 to 5 tons per sq ft over areas as great as 300 by 1000 ft. Since the storage areas are close to the waterfront, the lateral forces against dock structures may be very great. Therefore, it is not surprising that the areas for the storage of iron ore and for other heavy commodities have from time to time experienced large movements or even catastrophic failures. Several of these are described in the paper by Mr. E. J. Fucik.

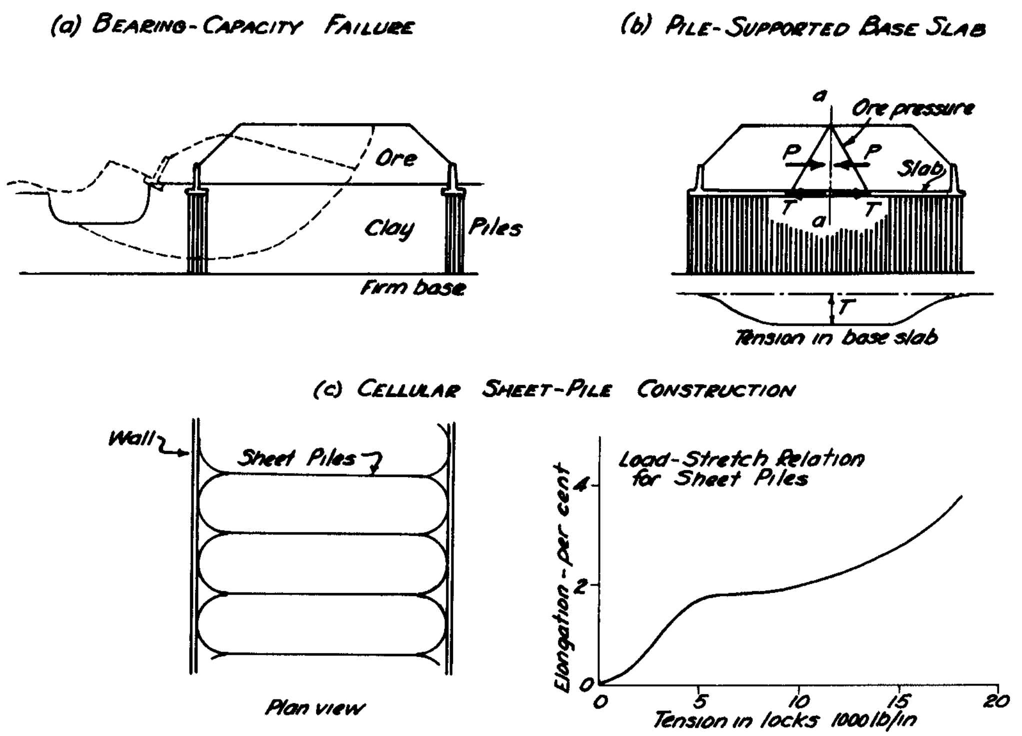

Fig. 5. General features of ore storage areas.

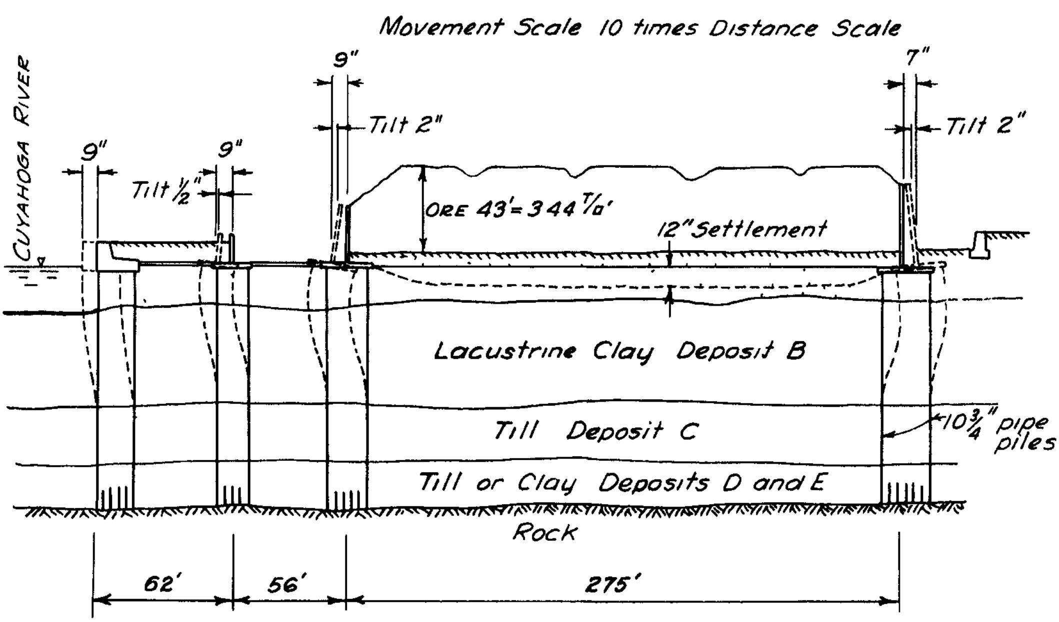

Fig. 6. Deformations of modern ore dock in Cleveland.

The type of complete failure most commonly observed is that shown in Fig. 5a, wherein a large portion of the stored ore subsides, pushes the retaining structures laterally, and heaves the ground in front of the retaining structures in a vertical direction. If outright failure does not occur, large progressive movements may develop over a period of 20 to 30 years. Most of the ore docks in the Great Lakes region have experienced lateral movement on the order of several feet.

Methods of analysis have been developed for estimating the load that may be placed on the surface of a deposit of clay without incurring a failure such as that shown in Fig. 5a. These methods have been described by Terzaghi (1943). As a simple and crude approximation, however, the load that is likely to cause failure of a clay subsoil may be taken as approximately 2.57 times the unconfined compressive strength. This extremely simple rule indicates, for example, that iron ore could be piled only to a height of 16 ft before failure if the subsoil had an unconfined compressive strength of 0.5 tons per sq ft, whereas it could be piled to a height of 64 ft if the strength were as great as 2.0 tons per sq ft. In practice, of course, a suitable factor of safety is required.

CONVENTIONAL METHODS OF AVOIDING FAILURE

Several of the ore docks in the Greal Lakes region have been supported by vertical piles capped by a reinforced concrete raft as shown in Fig. 5b. The piles have been driven to the firm base beneath the weak clays and have been designed to carry the entire vertical weight of the ore. The provision of such vertical support is often considered sufficient to insure the stability of the storage area. However, reference to Fig. 5b indicates that against any vertical section, such as a-a, there is a large lateral force representing the internal pressure built up within the ore itself. The magnitude P of this force can be computed with considerable accuracy by means of ordinary earth pressure theory. This force tends to cause the base of the ore pile to spread. If it is not resisted by adequate tensile reinforcement in the raft on top of the piles, the raft may split and the ore storage structure may fail by lateral movement. The tension T for which the slab must be reinforced is equal to the pressure P, per foot of length, exerted by the ore. Ordinarily an extremely high percentage of reinforcement is required to carry such loads.

The lateral ore pressures that cause the tension in the slab occur not only in the direction at right angles to the waterfront, but also in the longitudinal direction. Several ore storage areas have experienced failure or very large movements because the slabs were reinforced only in the direction at right angles to the dock and not longitudinally. One of the most recent examples of unsatisfactory behavior of an ore yard can be attributed to the lack of adequate longitudinal steel.

A pile-supported adequately reinforced slab constitutes a satisfactory structural solution to the ore storage problem, but is not likely to be attractive economically. Therefore, other procedures have often been advocated.

SHEET-PILE DIAPHRAGM CONSTRUCTION

Several ore storage areas have been constructed by driving diaphragms of sheet piles transverse to the waterfront and tied together by semicircular arcs at the ends as shown in Fig. 5c. It is assumed that the vertical load on the clay within the sheet pile enclosures will produce lateral forces against the sheet piles which will tend to place each ring of sheet piles in tension. The tension in the sheet piles acts as a hoop tending to confine the enclosed clay and assures the stability of the structure. Construction of this type has occasionally been successful where the sheet pile cells were partly excavated and backfilled with granular material, but the method of construction has often been disappointing when no such precautions were taken.

One fallacy in the reasoning leading to the sheet-pile diaphragm construction consists in ignoring the fact that the sheet-pile walls must elongate appreciably in order to develop significant tension in the interlocks. Figure 5c contains a diagram showing for typical sheet piles the percentage elongation required to develop various amounts of tension in the interlock. To develop even a tension of 5000 lbs per lineal inch, a value that must be realized if the type of construction is to be at all economical, requires an elongation of about 2 per cent. Since the usual width of an ore storage structure is about 250 ft, the required extension of the sheet-pile diaphragm is on the order of 5 ft. Moreover, sheet-pile walls cannot be driven without a considerable amount of clearance in the interlocks of the piling. Such clearance probably amounts to at least 1/16 in. per interlock or a total of about 1 ft in 250 ft. Thus, before sheet-pile diaphragms could become effective in carrying a substantial part of the superimposed load, a spread of the base of the storage area of about 6 ft would be required. Movements of this magnitude are commonly considered excessive.

Furthermore, the ore stored in such areas is commonly placed in the form of pyramids which exert pressures longitudinally as well as transversely to the waterfront. A long transverse diaphragm affords no protection against longitudinal forces near its center. Therefore, it is quite possible that such diaphragms may be seriously displaced or ruptured in a direction at right angles to that for which they are designed.

Finally, it should be noted that the integrity of the sheet-pile diaphragm system depends upon the driving of every sheet-pile properly interlocked with its neighbor. A single pile driven out of lock renders an entire diaphragm useless. Where glacial tills are involved, the presence of pebbles and occasional boulders must be taken for granted. Therefore, an element of risk always exists in the driving of a continuous line of sheet piles. Furthermore, no method exists for detecting whether or not a pile has driven out of interlock. Therefore, the sheet-pile cellular construction, although extremely expensive, may not actually possess all the advantages that have been claimed for it.

On some projects, the sheet-pile cells have not been extended entirely across the storage area, but have been restricted to a narrow retaining wall close to the waterfront. The cells in such a wall are often nearly circular in shape. Unless the clay in these cells is removed and replaced by granular material, the cells have negligible resistance to shearing deformations and do not serve their purpose. Indeed, even with granular fill, it is usually impracticable to satisfy the requirements of stability.

STRENGTHENING SUBSOIL BY CONTROLLED LOADING

The development of soil mechanics has led to the introduction of a basically different approach to the foundation problems of ore storage areas. It is well known that the application of a load to clay subsoils initiates a process of consolidation by which water is gradually squeezed from the pores of the clay. As this process continues, the strength of the clay increases appreciably. The increase occurs most rapidly when relatively large loads are placed on the clay. On the other hand, if the load applied to the clay produces large shearing forces in the subsoil, a failure may occur. Therefore, in order to achieve the maximum rate of increase of strength, a load only slightly less than the failure load should be applied, but great care must be exercised to make sure that a failure is not induced.

This principle may be utilized for the strengthening of the subsoils for ore storage areas, provided the quantity of ore to be stored in the first few years of the life of the facility can be restricted. Although it is undesirable to reduce the quantity of ore that may be stored from the point of view of operation of the blast furnaces, the cost of bringing in the required supplementary ore by rail may be substantially less than the cost of providing the additional initial strength by such artificial means as pile-supported rafts, etc.

Figure 6 shows a cross section of a modern ore dock constructed in Cleveland in accordance with the principle mentioned in the preceding paragraph. The retaining walls for the storage structure are established on steel piles to rock. No foundation was provided, however, for the ore storage area proper. Appreciable deformation of the structure was anticipated, and actually occurred. The deformations after a period of operation of four years are indicated in Fig. 6. It is seen that the walls moved out on the order of 9 in. The movements still continue but at a relatively slow rate. By controlling the rate of loading during the first four years the strength of the subsoil was built up to the point where virtually no restriction need now be placed upon the operation of the yard.

The process of strengthening of the foundation is illustrated in Fig. 7. The vertical pressure exerted by the ore on a strip 90 ft wide adjacent to one retaining wall is plotted as a function of the outward movement of the wall. During the first year of loading the average pressure increased from zero to approximately 2 tons per sq ft. During this period the deflection in the wall increased to about 4 in., and the shape of the load-deflection curve indicates that if the load had been carried appreciably beyond 2 tons per sq ft a failure would have undoubtedly occurred, because the curve was approaching a vertical tangent at about that value of loading. When the ore was used during the winter season the storage yard was almost completely unloaded. No appreciable recovery of the deflection took place. The second year of loading involved relatively small loads on the ore yard for operational reasons. Therefore, the second year of loading did not produce appreciable additional deflection of the wall. However, during the third year the load was increased to almost three tons per sq ft. The load-deflection curve indicates that the strength of the clay had been increased at least 50 per cent by the loading during the first two years. The load placed during the third year further increased the strength of the clay so that during the fourth year a pressure of about 3.3 tons per sq ft was applied with safety.

Figure 7 indicates forcefully and graphically that the strength of the clay beneath the storage area was nearly doubled in a period of four years by a process of increased loading carefully controlled to avoid failure. The avoidance of failure was guaranteed by a variety of types of field observations concerning the movements of the walls, the settlement of the clay surface, the excess water pressure developed in the pores of the clay, and detailed records of the position and amount of the various loads. Such control can hardly be attempted except under expert supervision, but once the strengthening has been achieved the structure is capable of carrying the desired loads with complete safety. Obviously, the cost of the construction is less than that of any of the types mentioned under the preceding subheading.

BEHAVIOR OF ANCHORED BULKHEADS

PRINCIPLES OF DESIGN

Numerous bulkheads consisting of vertical sheet piles tied back to anchorages have been constructed along all the Great Lakes industrial waterfronts. These have usually been designed in accordance with generally accepted procedures, and have for the most part performed in a satisfactory manner. Inasmuch as the latest information concerning the principles of design for anchored bulkheads has recently been summarized by Terzaghi (1953), no discussion of these principles will be attempted in the present paper.

However, it may be pointed out that experience indicates that the most frequent shortcoming of anchored bulkheads is the inadequate resistance of the soil in front of the buried part of the sheet-piles. The depth of the sheet piling is usually estimated on the basis of the assumed passive resistance of the soil in front of the bulkhead. If the strength of this soil is overestimated, failure of the bulkhead is extremely likely to occur.

THE INFLUENCE OF SUBSURFACE CONDITIONS ON THE DESIGN OF FOUNDATIONS FOR WATERFRONT STRUCTURES IN THE GREAT LAKES AREA

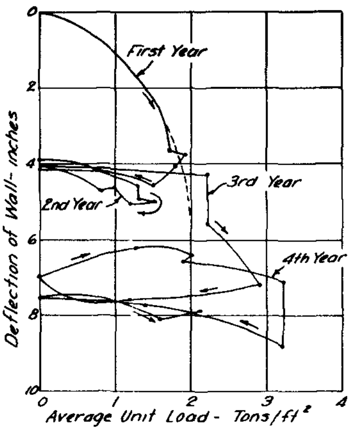

Fig. 7. Relation between wall deflection and load on 90-ft. strip adjacent to wall, Cleveland ore storage yard.

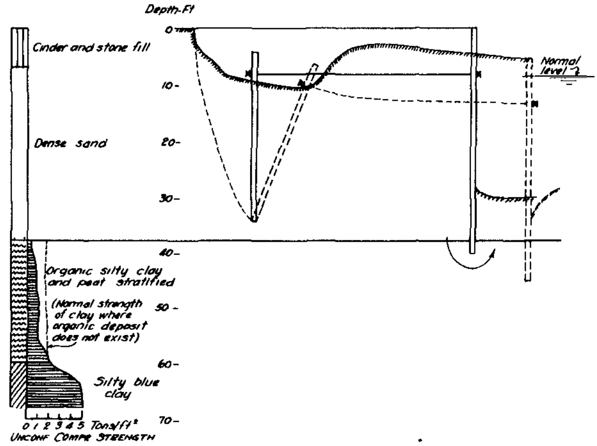

Fig. 8. Failure of sheet-pile bulkhead, Northern Indiana.

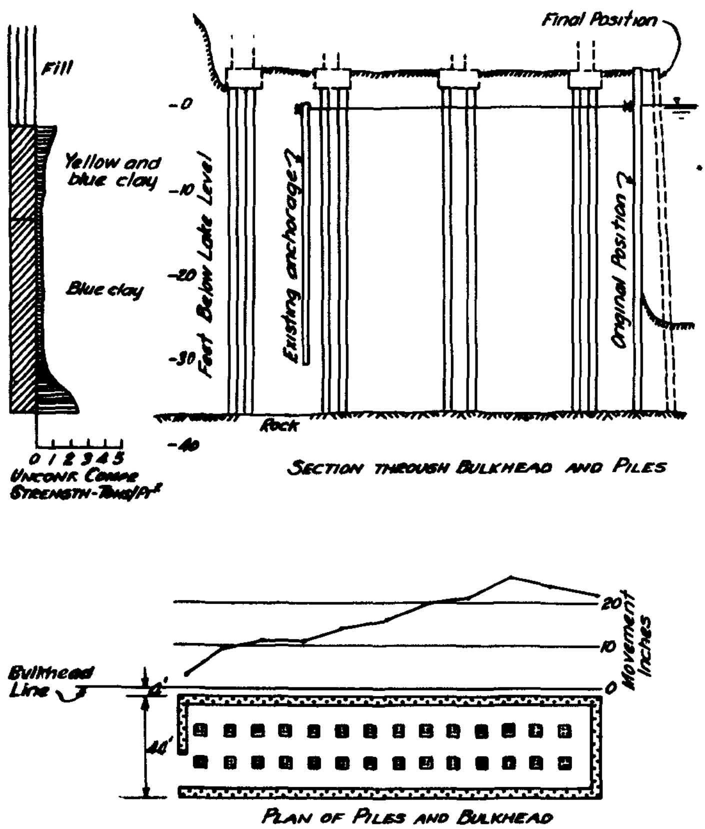

Fig. 9. Bulkhead movements caused by pile driving.

EXAMPLE OF TYPICAL FAILURE

Figure 8 represents one of several bulkhead failures in northern Indiana. The design of the bulkheads is conventional, although not overly conservative. Failures have been uncommon, and have occurred over relatively short distances. The failure illustrated in Fig. 8 took place when a strong and steady wind from the south blew the water at the southern end of Lake Michigan to the north with sufficient energy to lower the water level in front of the bulkhead by about 3 ft in a few hours. At the period of the lowest water level, excessive movements occurred. From the character of the movements shown in Fig. 8 it is obvious that the toe resistance of the sheet piles was inadequate and that the sheeting moved out by translation almost without rotation. Ultimately, of course, the anchorage also proved inadequate, but it is doubtful that the anchorage would have failed had the toe resistance been great enough.

Borings indicated that the sheet piles were buried not in the clay normally found in the locality, with an unconfined compressive strength on the order of 2 tons per sq ft, but in a buried valley containing organic silty clay and peat with unconfined compressive strengths on the order of 0.4 to 0.6 ton per sq ft. The failure of the bulkhead was confined strictly to the locality where the organic deposits existed. Similarly, the extent of other failures in northern Indiana has been defined by the presence of organic deposits of soft consistency which replaced the stiffer clays normally encountered.

These examples indicate the necessity for at least a simple but a systematic soil survey to determine the toe resistance of the materials in which the sheet piles will be embedded. Where buried valleys are likely to be encountered, particular care must be taken to make sure that unanticipated soft spots will not be overlooked. The existence of such soft materials is a far more significant fact than the elaborate or precise determination of the physical properties of the normal soils if the occasional weak deposits are overlooked.

BUILDING FOUNDATIONS NEAR WATERFRONT

INTRODUCTION

The customary types of foundations are commonly utilized in the waterfront areas of the Great Lakes. Where structures would be inadequately supported by a crust of stiff clay or a blanket of dense sand, piles or piers are used to transfer the load to stiffer materials at greater depths.

Construction of pile-supported structures behind existing bulkheads, however, may be accompanied by substantial lateral movement of the bulkheads. This fact has often been ignored in the planning stages of substructures and has led to unpleasant surprises during the construction period. Therefore, this subject will be selected for somewhat more detailed discussion.

MOVEMENTS DUE TO PILE DRIVING

As an example that may be regarded as fairly typical, the lateral movements of a bulkhead due to the driving of piles for a building are illustrated in Fig. 9. The soil profile, including the variation in unconfined compressive strength, is shown in the diagram. It is seen that much of the structure is underlain by clay having an unconfined compressive strength of about 0.5 ton per sq ft, and that the clay is in a sense confined vertically by a blanket or crust of stiff yellow and blue clay.

During the driving of foundation piles it was observed that the bulkhead moved into the river as much as 2 ft at the top, and generally somewhat more at the bottom. The movement was progressive and tended to increase in the general direction in which pile driving took place.

The plan of the piles and bulkhead, together with a cross section through the foundation, indicates that the density of piles was not abnormally large. The average diameter of piles was approximately 1 ft. Nevertheless, the driving of these piles produced large and objectionable movements of the bulkhead and required expensive redesign of the foundation of the structure.

Such movements are most likely to occur when the piles are driven into saturated clays, and are less likely to occur if the subsoil consists of peats or organic silts. Although no strictly rational procedure can be suggested for determining the amount of movement to be expected during the driving of piles behind an existing retaining structure, the possibility of such movement should be kept in mind and may have a decisive effect upon the choice of foundation.

The importance of at least recognizing the possibility of such movements is attested by the fact that the cost of redesign and remedial measures has in numerous instances been far greater than the extra expense of installation of some nondisplacement type of foundation. If displacement piles are used the undesirable consequences may often be avoided or reduced by coring or other methods of preexcavating part of the material occupying the space in which the piles are to be located.

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CHAPTER 21

THE HYDRAULIC DREDGING AND PUMPING OF LAKE AREA DEPOSITS

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First let me say that it is a sincere pleasure and an honor to address such a distinguished audience. More people should be made aware of the work that you are doing in the protection of our shores and coastline. It is a great satisfaction to me to be associated with an industry that is instrumental in rebuilding our shores along the lines of your extensive studies and recommendations. I recall laboring over the trochoidal wave theory in my study of naval architecture quite a few years ago. Frankly, it is not used very much in dredge design except in the larger molded form sea going hopper dredges. There is no doubt that you have made great strides in the theory of wave motion and transmitted energy since that time.

When Professor Hamilton first called upon me to speak to you regarding hydraulic dredging, he felt that you would be interested in a description of various types of hydraulic dredges and their fields of application. It occurred to me then that you would also be interested in a brief description of the modern dredge pump which actually constitutes the heart of the dredge.

While my talk is principally confined to dredging operations in this lake area, the various fields and the scope for the application of the hydraulic dredge is so interesting that with your permission, I should first like to cover the subject in a general way.

Since about 1870, the hydraulic method of excavation and transportation of solids has proven itself a necessary adjunct to our economic and cultural life - - - indeed an important factor to our existence.

In peace time, hydraulic dredging makes possible the continued flow of maritime commerce by the maintenance of our navigable channels. It makes possible the beautification of our parks, our lakes and cities, the elimination of insect ridden swamps and lowlands, the building of levies to protect our homes and property, the digging of channels for proper run-off in flooded areas, the restoration of our shores.

It makes possible the depositing and building up of hydraulic fills as a base for highways, airports, industrial sites and homes. It has contributed to the successful completion of some of our earth-filled dams. It deepens our lakes for the better propagation of marine life. It is one of the most economical methods of excavating for pipeline river crossings. It excavates and often transports in one operation oyster shell for the manufacture of cement and the building of roads, sand and gravel for the production of the screen classified and washed product, it reclaims coal from coal refuse areas, it procures our lake sands from depths sometimes exceeding 100 feet for the production of core and foundry sands for our automotive industry. It excavates ores and minerals for the production of gold, tin and other metals.

THE HYDRAULIC DREDGING AND PUMPING OF LAKE AREA DEPOSITS

Hydraulic dredges are in use for the commercial dredging of peat and humus. One of the most unusual applications of the dredging method occurs in off-shore fishing operations where fish such as menhaden are pumped from the purse seines into the holds of fishing boats and then dredged out of the holds as soon as the vessels are moored to the dock.

In time of war, hydraulic dredging has been indispensable as a fighting tool. Portable dredges are designed which can be shipped in knocked-down condition to almost any part of the world and then reassembled for strategic purposes.

DREDGE CLASSIFICATIONS

It is customary to classify hydraulic dredges broadly into two groups; first, those that are self propelled similar to many of the dredges operated by the U. S. Engineers; secondly, those that are not.

SELF PROPELLED DREDGES:

Hopper Dredges - An interesting example of a self-propelled dredge is the U. S. Engineer twin screw Hopper Dredge Hains now engaged in widening the channel in the Detroit River. A rather light silt is being dredged from a depth of 30 feet and pumped directly into built-in hoppers. The dredge is equipped with a single 18" dredge pump directly connected to a 400 Horsepower electric motor. The main power plant is diesel electric developing 1400 HP. The hopper capacity in this dredge is 730 cubic yards. The Hains maintains a speed of two to four miles an hour while dredging and is capable of a speed of 14 miles per hour in fully loaded condition.

This type of dredge is equipped with two suction drags, one on each side of the hull. The drags are shaped much like a shoe and the bottom of the drag is equipped with a cellular bar screen to exclude oversized foreign matter. The drag is also equipped with a flap valve on its upper surface to admit water at times when it may become clogged.

The Hains is arranged so that the pump can either discharge into the hoppers on board or ashore as the operation may dictate. It is equipped with a bin pump out system in which the dredge pump is used to pump out the eight hoppers. The most common means of unloading the hopper, of course, is to dump the material through hopper gates at the bottom. Under favorable dredging conditions, this dredge has pumped solids into the hoppers at a rate as high as 2160 cubic yards per hour. This is exceptionally good performance for a pump of this size.

Another interesting feature about the design of the Hains and for that matter, most other self-propelled hopper dredges, is the twin rudder design. Each of the screws has a rudder located immediately behind it. The twin rudders are necessary to maintain steerage-way while dredging at the extremely low speeds.

The dredge pump is equipped with a degassifier and it might be well to dwell for a few moments on this rather important device. The function of the degassifier is to remove the entrained gases in the dredging mixture from the suction nozzle of the dredge pump. The output of the dredge pump is reduced considerably with air, vapor or gas present in the mixture. A dredge pump is highly efficient as an air separator -- in fact, too much so in many instances for profitable operation. The entrained air entering the suction of the pump separates immediately and remains in the eye of the impeller until it is removed. The heavier dredging mixture swirles around the periphery of the pump casing. If a sufficient volume of air is permitted to accumulate, the pump will stop functioning, the flow will cease, the solids will drop out of suspension, often causing clogging.

In general, the degassifying system consists of a surge tank, the bottom of which is connected as close to the eye of the impeller as possible. Either a vacuum pump with a priming valve or an educator connected to the top of the tank is then used to exhaust the air from the tank and as a result, from the eye of the impeller. In many instances, the use of the degassifier has resulted in the success of a dredging operation which otherwise might have been a complete failure. This is particularly true in the more gaseous deposits found in marsh lands and sewage sludges.

While the Hains is now engaged in widening the channel of the Detroit River, it has been used in a variety of dredging work here on the lakes. This same type of dredge performs all of the U. S. Engineer harbor maintenance work in the lake area and often is called upon to remove bars at harbor mouths occasioned by storm. The Hains has also aided in the restoration of the lake shore in Rochester, Erie and Ashtabula by dumping sand through the hopper bottom gates into currents on the down-drift side which eventually deposit the sand in the needed location on the shore.

When we speak of self-propelled dredges, we think mostly in terms of molded form hulls such as the dredge just mentioned.

<u>Cutter Dredges</u> - There are only a few self-propelled cutter type dredges in this country. Usually, the expense connected with the propulsion unit is not warranted, unless the dredge is a large one operating and travelling in open waters.

NON-PROPELLED DREDGES:

Non-propelled dredges are of the following types:

1. Pipeline dredges, used either in general contracting service or for the production of sand and gravel and other mineral products.

2. Dredges discharging directly overboard into barges used mostly in the production of sand and gravel.

3. Dredges with complete or partial screening and classifying equipment on board used mostly in the production of sand and gravel.

All three types can be equipped with or without agitating equipment. The most common mechanical agitators are:

1. The rotating basket type cutter.

2. The travelling suction screen.

Hydraulic agitation is sometimes resorted to in the form of high pressure jets at the suction mouth. However, jet agitation has not proven very successful, and is entirely ineffectual in clay or hard pan deposits.

Pipeline Dredges - The most common type of pipeline dredge is the hydraulic cutter dredge consisting essentially of a box-shaped floating hull with rotating cutter, cutter shafting, bearings, reduction gears and, in most cases, the driving electric motor located on a structural steel ladder hinged on trunnions at the bow of the dredge. It is customary to provide a wound rotor motor with variable speed control. The most efficient dredging angle of the cutter ladder is 45°. This type of dredge is usually equipped with a 5-drum dredge hoist. In the larger sizes, dredge hoist brakes and frictions are controlled by pneumatic or hydraulic controls. The center drum on the hoist serves the purpose of raising and lowering the cutter ladder. Two of the drums are used for swinging the dredge to either side and the two remaining drums for raising and lowering each of the two spuds at the stern. The swing lines are usually anchored at a considerable distance each side of the dredge and pass through swing sheaves located at the bottom end of the ladder. Dredge hoists are designed to swing the dredge at speeds varying from 50 to 80 feet per minute.

The usual method of operating a cutter dredge after the cut has been established is to start near the top of the face with cutter projecting into the deposit for approximately its own length, then swinging the dredge from side to side, with the working spud acting as a pivot. The cutter itself rotates only in one direction. The ladder is lowered at each swing on a radius about the working spud until the dredging depth has been reached. The operator then "steps ahead" by dropping the auxiliary spud, and again lowering the working spud when the dredge has been advanced into the cut by approximately the length of the cutter.

Often the operator will undercut the bank to produce caveins for higher production.

When the dredge has advanced to a point where the swingline anchors at each side are so far aft of the ladder that the swinging motion of the dredge is impaired, the anchors are moved again to a point somewhat ahead of and sufficiently outboard of the dredge to permit full freedom in swinging.

The modern pipeline cutter dredge used in general contracting service usually has a main dredge pump directly driven by a diesel engine or it is diesel driven through reduction gears. All dredge auxiliaries such as the raw water circulating pumps, service pumps, hoist, cutters are usually driven by electric motors with current supplied by a diesel engine driven generator.

One of the many interesting hydraulic cutter dredge projects performed on the Great Lakes is the 2500 ft. long cellular steel sheeting break-water built by The Great Lakes Dredge and Dock Company in 1950 for the Youngstown Sheet and Tube Company at the West entrance to the Indiana Harbor Basin here on Lake Michigan. This breakwater extends Northwest from the entrance to the harbor, and is 2500 ft. long. The breakwater was built as a retaining wall for a fill which was being made at the time. It consists of partially intersecting circular steel cells which were filled very nearly to the top with sand dredged and discharged through about 1000 ft. of pipeline from the dredge.

The radius of each cell is about 25 ft. The sheeting used was 50 ft. long. The depth of the water varied from 23 to 29 ft. and the bottom was found to be soft clay. The sheeting was driven to a penetration depth of 15 to 21 ft. After the cells were filled and the sand allowed to settle, a reinforced concrete cap was poured to prevent the washing out of sand cores.

There is an interesting side light regarding this breakwater. After completion, it was found that it deflected currents toward the channel which interferred with the navigation of ships approaching the basin from the north. As a result, the U. S. Engineers, after careful study, required the Youngstown Sheet and Tube Company to riprap the lake side of the cells completely from top to bottom at an angle to deflect these currents clear of the channel.

Some pipeline dredges are entirely electric motor driven with current supplied through a cable from the shore. As you can surmise, this type of dredge is suitable only in locations where the operation is permanent or of long duration. An electrically driven dredge is usually lower in price than one that is diesel driven. It is also more maneuverable because of its smaller physical dimensions.

Striking examples of this type of all electric pipeline cutter dredge are the two dredges which will be built by Construction Aggregates Corporation for operation at Steep Rock Lake, Atikokan, Canada.

As you probably know, a high quality iron ore lies beneath the lake bottom.

The purpose of these dredges is to remove the lake bottom and to expose the ore body. The entire lake was drained several years ago to uncover the bottom. Dredge design data are as follows:

Size 36 inches
Hull - 165 ft. x 30 ft. x 10 ft. deep
Expected output about 6000 cubic yards per hour
Main dredge pump motor 10,000 horsepower
Length of pontoon piping 5,000 feet
Length of shore piping 24,000 feet
Static discharge head about 600 feet

Each dredge will have two 36" booster pumps in the pipeline, one afloat, the other ashore. Each booster will be directly connected to a 10,000 HP motor making a total of 60,000 connected HP for the two dredges and the two sets of boosters.

160,000,000 yards of material will have to be removed and this will be one of the largest dredging operations ever attempted.

<u>The Overboard Discharge Dredge</u> - This type is used principally for sand and gravel production and is found mostly in the rivers.

The dredged material is discharged directly into barges alongside. Since there is no floating pontoon line, there is less interference with navigation. But the most important reason for operating a dredge of this type is to reduce maintenance costs.

The river deposits in this area are, as you know, of glacial origin, containing high percentages of gravel. It is not uncommon to find deposits containing as much as 50 to 60% gravel.

It is not economical to pump this destructive and heavy material through long lengths of pipeline and against high static heads. Abrasion of the pump parts and pipeline is so rapid that costs skyrocket and shutdowns are frequent.

Pumping directly overboard to the barges increases the solids output and by reason of the low pump head and speed greatly lengthens the life of the pump parts.

Most of these glacial deposits are semi-compacted requiring an agitator to obtain reasonable production.

<u>Sand and Gravel Dredges with Screening Plants on Board</u> - This type of dredge is used where it is impractical to locate the screening and classifying plant ashore. It is a completely self contained unit and is designed for operation in glacial gravel deposits.

The dredge pump discharges to a head box containing a scalping screen which separates the oversized stones and discharges them overboard.

The material then passes over the screens needed to produce the desired sizes of gravel. The sand also is separated and both gravel and sand are discharged through chutes into barges along side.

Often the dredge will discharge to four barges simultaneously. While the static discharge head on the pump is 40 ft. to 50 ft., the pipeline is short and pump speeds and maintenance costs low.

METHODS OF AGITATION

Rotating Cutters - Different designs of cutters are used to suit the type of deposit. For sand and gravel, the basket type cutter is used. Cutting edges of the blades may be plain, serrated or toothed.

For clay, a modification of the basket type cutter is used. The blades are narrower, the cutter is more open. Serrated cutting edges are needed to break up the clay.

For conglomerate or soft rock, the cutter is equipped with heavy teeth.

Manganese steel has been found to be the best for cutters operating in gravel or rock.

The Travelling Suction Screen - Many sand and gravel dredges are equipped with a travelling suction screen which, in effect, is a double moving chain connecte by cross links which pass the suction mouth over its full width. It is endless - - - moving upwards toward the bow of the dredge over the suction pipe and downwards toward the nozzle under the suction piping. The travelling screen is fitted with hooks or lugs which remove oversized stones or boulders from the suction nozzle and act as a conveyor to carry the stones upward and away from the digging operation.

The Plain Suction Nozzle - The plain suotion pipeline dredge as the name implies, has no means of agitating the material. It is not recommended for cemented, compacted or semi-compacted deposits, and it economical only in deposits of a free flowing nature.

DREDGE PUMPS

Characteristics and Performance - The design and performance of the dredge pump is the most important factor in the success of the dredge. It must be designed for extremely high suction lifts with a minimum of cavitation. Clearances through the impeller must be large to pass stones of maximum size. It is essential that the vane curvature be correct to produce a smooth flow through the passages, free from excessive eddying and turbulence. We find that overlapping of the vanes definitely improves hydraulic efficiency and that when we turn down the impellers destroying the overlapping feature, we greatly reduce the pump efficiency.

THE HYDRAULIC DREDGING AND PUMPING OF LAKE AREA DEPOSITS

One of the few patent claims allowed in centrifugal pumps in recent years is the pressure balancing feature in our impeller design which tends to reduce circulation of the abrasive solids from the pressure passages in the shell, past the suction disc liner and into the eye of the impeller. This is accomplished by making the suction shroud of the impeller larger in diameter than the vanes. The outer surface of the impeller also has integrally cast vanes which are quite narrow but take the same curvature and position as the main vane inside. These vanes being longer develop a counteracting pressure at the periphery of the suction shroud to balance as nearly as possible the pressure in the volute. By retarding this internal circulation, we are obtaining a notable increase in the life of the pump wearing parts.

An important factor in providing long life of the dredge pump is the speed of operation. Low speeds and low angular velocity means lesser impact of the solids against the surfaces of the vanes and consequently longer life of the parts.

The modern dredge pump is designed with extremely heavy casings or volutes, the periphery uniformly increasing in thickness from the cutwater to the point of discharge. The casing is definitely considered as an expendable part and our object in varying the thickness at the periphery of the volute is to place the metal where, in our experience, we have found the wear is greatest. The ideal design is such as to have the least amount of weight and scrap left when shell finally wears through. All modern dredge pumps have casings of volute design. When we first started the manufacture of dredge pumps in 1870, our casings were circular. Through succeeding years of research and development, we have found that volute casings definitely increases the hydraulic efficiency and the wear characteristics. We take the position that it is just as important to keep on with this development as it is to continually improve the design of our water pumps.

You may be interested in knowing that before we start the manufacture of a new line of dredge pumps, we build a homologous model which is usually an 8" unit. The design of our latest line of heavy duty dredge pumps is based upon exhaustive model tests which extended over a period of one year. Various combinations of impellers and casings were tested. Each combination required six complete tests, each with different diameter impeller. We were extremely fortunate as a result of these model tests to obtain efficiencies with full diameter impellers as high as 75%. This is not much lower than accepted efficiencies for water pumps where there is unlimited freedom in design.

Basically, the dredge pump differs from a water pump in the following ways:

The dredge pump has heavier and more rugged parts. It has internal liner protection and lower operating speeds.

In many cases, a dredge pump, developing the same head and capacity as a water pump, will operate only at 1/4 its speed.

On the suction side of the pump, the sealing or wearing surfaces are structly in a vertical plane. Adjusting features are provided on most dredge pumps to move the impeller toward the suction disc liner of the pump at times when the clearance between the wearing surfaces becomes excessive. This clearance between the impeller and the liners on the suction side should be maintained at 1/32" for best performance.

Bearings of any kind in contact with the dredging mixture cannot be tolerated. The dredge pump shafts, therefore, are supported by heavy bearings entirely external of the water end itself. The shaft must be heavy enough and the center distance between the bearings great enough to provide rigidity and negligible deflection of the overhanging shaft under the worst operating conditions.

You may be interested in the size boulders which these pumps will pass:

36" Pump ---------	Will pass stones approximately 26" in diameter.
20" Pump ---------	14" in diameter.
15" Pump ---------	11" in diameter.
12" Pump ---------	9½" in diameter.

The pump must be designed for this severe service without danger of bending or breaking the shaft, pump parts, or damaging the bearings.

The size of the dredge is usually designated by the size of the pump discharge nozzle. You may like to know what output can be expected from various sizes of hydraulic dredges provided that agitators will excavate the solids at the rate of which the pump is capable.

Let us assume first, that the dredges are operating in ideal deposit such as a fine beach sand. The approximate maximum output, when operating at reasonable dredging depth, based upon the mixture containing 30% solids by apparent volume with pipeline velocity 20 ft. per second, is:

36" dredge ---------------	6000 yards per hour.
24" dredge ---------------	3000 yards per hour.
20" dredge ---------------	2100 yards per hour.
15" dredge ---------------	800 yards per hour.
12" dredge ---------------	600 yards per hour.

We are considering here a free-flowing deposit with particles well rounded and not exceeding, say 50 mesh in size. Naturally, this output would be considerably reduced in the presence of clay.

Let us assume next that these same cutter dredges are operating in a compacted gravel deposit with few stones exceeding the size which will pass through the pump, the deposit containing 50% gravel and 50% sand - pipeline velocity the same as before. The maximum output that could then be expected would be only about one third of the output in free-flowing sand. Output is often further reduced by the presence of oversized stones and boulders. Where a high percentage of oversized stones is contained in the deposit, the dredge will remove smaller stones and the sand leaving the oversized material at the bottom. It is not uncommon to eventually pave the bottom with oversized material to a point where the dredge is no longer able to penetrate the layer. Often times, harbor bottoms are covered with trash such as old tires, scrap iron, water-logged timbers, long branches and roots. These too can be an enigma, often to the extent of abandoning hydraulic dredge operation.

CONCLUSION

It is obvious that each dredge must be specifically designed for the job it has to do. It should be placed in operation only after careful consideration has been given to the depth of dredging below water level, the length of the discharge piping, the type of deposit, whether loose, compacted, cemented or gaseous, the static discharge head, the maximum size of stones in the deposit, the amount of trash to be encountered.

In general, the dredge must be large enough to pass practically all of the material in the deposit to be really successful.

CHAPTER 22

THE MAIN ORE UNLOADING DOCK FAILURES AND THEIR CORRECTION 1909 - 1925, GREAT LAKES REGION

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To start at the beginning of Chicago's lake structures (and first attempts to hold back wave action) we will go back about 120 years, to the year 1833. At that time an Illinois Congressman put through Congress an appropriation of $25,000 to construct a harbor at the south end of Lake Michigan (Fig. 1). At that time the Chicago River was navigable only by canoes. That Congressman, however, argued that the harbor should be located eleven miles south at the Calumet River. He was out-argued by a Captain of the Corps of Engineers, U.S. Army, who prevailed, and two 500 foot piers were built at the mouth of the Chicago River. The Congressman's name was Stephen A. Douglass and the Captain's name was Jefferson Davis, who evidently was a very persuasive fellow. These piers were either wooden cribs filled with stone, or stone-filled pile piers.

These two types of piers and breakwaters protected Chicago and the south end of Lake Michigan for over 80 years until the advent of Rubble Mound, Concrete Caissons and Interlocking Steel Cellular Breakwaters. These stone-filled wooden pile piers (Figs. 2 and 3) consisting of two rows of closely driven wood piles, filled with stone and tied together with a few rods and timbers, are a type peculiar to the south end of Lake Michigan, due to its geological formation. Sand from 5 to 20 feet in depth, overlaying a stiff clay, with stone and timber and piles at hand, enabled these structures to be built cheaply, quickly, and with a minimum of skilled labor. These piers or bulkheads now line practically all of Chicago and the south end of Lake Michigan for about 40 miles of shoreline. The stone-filled pile pier enabled the steel companies to expand into the Lake, to make additional land and provide easy economical dumping grounds right at their door. This process is still going on.

Gary Breakwater (built by the Great Lakes Dredge and Dock Co. in 1905-06) is a 24 foot wide stone-filled timber crib extending on a curve into Lake Michigan from Gary Harbor at the extreme south end of Lake Michigan. It is exposed to Northwest, Northeast, and North winds with a 300 mile fetch, with winds ranging from 20 to 60 miles per hour. This breakwater protects a 250 foot wide canal about 5000 feet long. The west, or ore-unloading side of the canal, parallel to a row of twelve blast furnaces, was lined by a dock or bulkhead (Fig. 2). This dock consisted of a 9 inch Wakefield line of sheet pile and 4 rows of wooden piles driven into the clay (which was found at about -28 feet) and the piles and sheet piles capped with a heavy mass of concrete, all anchored back by two inch steel rods to anchor piles. A portion of this bulkhead moved out into the canal about three feet fifteen years ago, so the whole 5000 feet was further protected by a line of heavy steel Z piles and re-anchored.

THE MAIN ORE UNLOADING DOCK FAILURES AND THEIR CORRECTION 1909 - 1925, GREAT LAKES REGION

Six miles west of Gary lies Indiana Harbor and Canal which serves Inland Steel Company, Youngstown Sheet and Tube Company, Standard Oil Co. of Indiana and scores of other industries. In 1913, the Inland Steel Co., at the entrance to the harbor, had two blast furnaces, an ore dock and wooden bulkheads along the canal. The canal entrance and dock was protected by a stone-filled pier, its only protection from 300 miles of Lake Michigan (Fig. 5). The bulkhead or dock along the canal was a line of wood piles in front of a line of wooden sheet piles with three rows of wooden piles all capped with concrete and anchored back 80 feet to anchor piles. In midwinter of that year about 500 feet of the ore yard and dock slid out into the canal, raising the bottom of the canal about ten feet. (Fig. 6).

The Inland Steel Company requested bids and designs, and the one accepted was that of the Great Lakes Bridge and Dock Company. The principle feature of the design is the carrying of the ore load, 560 tons per lineal foot of dock, on wooden piling down to hard clay. The piles were from 45 to 115 feet long and capped with a mat of concrete, 21 inches thick, in the ore yard. Since that time, the ore yard and dock have been extended into Lake Michigan about 3500 feet, using the same design, protected by a class A breakwater. One section about 300 feet long settled 12 inches and moved out slightly, but the dock is still in use. Figure 7 shows final development of this design.

Later the Buffalo Union Furnace Company, the Donner Steel Company, and the Kelly Island Limestone Company docks, all of Buffalo, failed by sliding out and were corrected in the same manner as the Inland Steel Co. dock.

With the introduction of interlocking steel piling, steel foundation piling, concrete and reinforcing, practically all ore unloading docks followed the general desing of the 1913 Inland Steel design.

In my opinion, the most interesting of the ore dock failures and its correction was the failure of the Algoma Steel Company's dock and ore yard at Sault Ste. Marie, Ontario, in 1913. As the Great Lakes Dredge and Dock Company destroyed all of its files except those of the last 25 years, the sketch shown is drawn from my own notes and recollections. Just recently I received from Mr. K.J. Kenyon, Chief Engineer of the Algoma Steel Co., blue prints of the dock and ore yard section with borings which checks closely the sketch prepared from memory (Fig. 8). This ore yard and dock consisted of a platform dock on rows of wooden piles carrying the ore unloaders with the canal sloping up underneath from -25 feet to about + 3.0 feet, and the ore carried on natural ground for about 250 feet back. The slide ran back about half the width of the ore piles (about 125 feet) destroying the platform dock. Soundings showed soft clay to -60 feet with gravel and sand to rock at -70 feet along the dock line. Competitive designs and figures were received by Julian Kennedy of Pittsburg (at that time considered the Father of Blast Furnace design), and the one submitted by the Great Lakes Dredge and Dock Co. was accepted and a contract was drawn up for about 500 lineal feet of dock. This was later extended for another 500 feet.

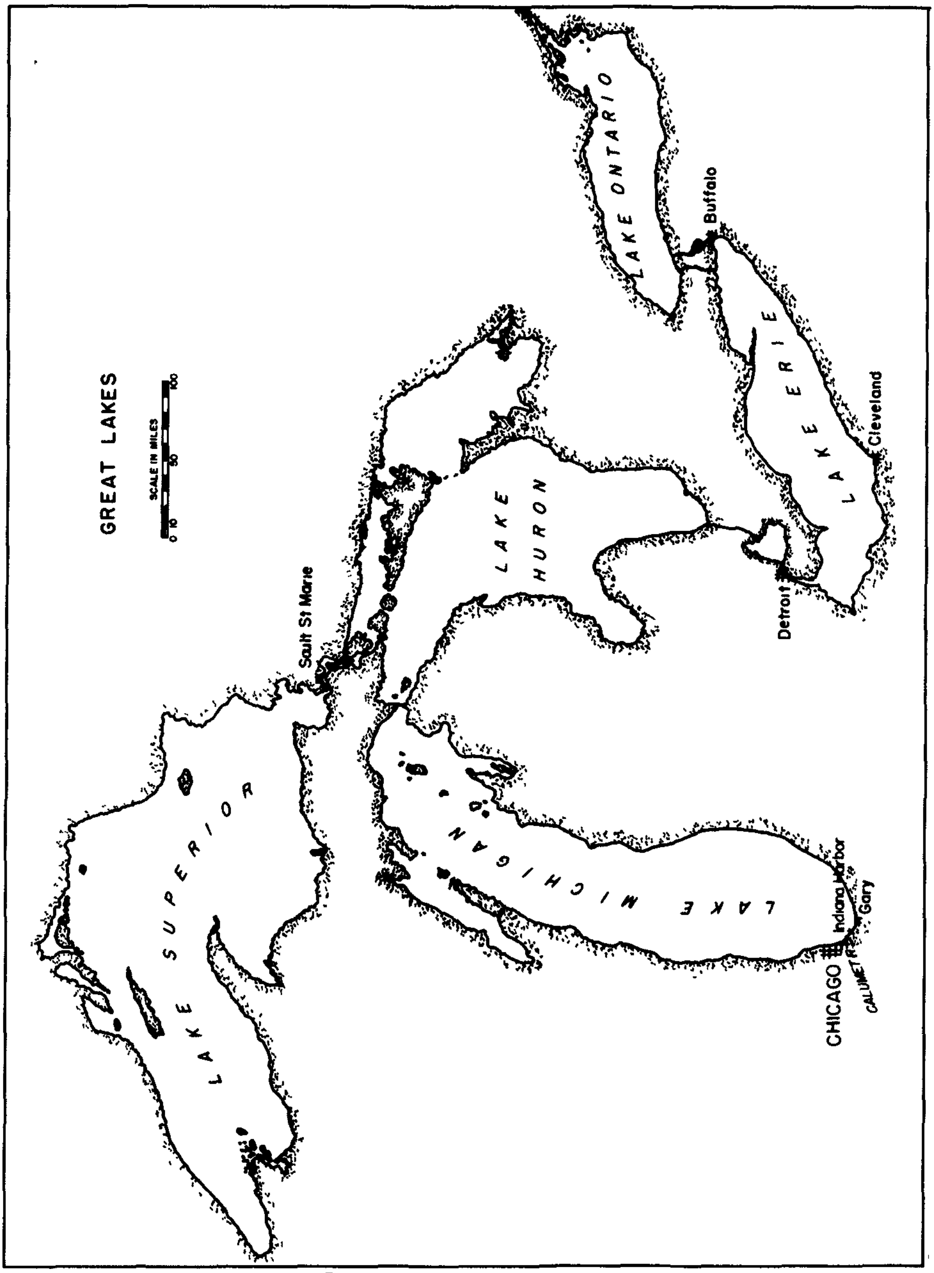
GREAT LAKES
SCALE IN MILES
0
10
50
100
LAKE SUPERIOR
Sault St Marie
LAKE MICHIGAN
LAKE HURON
LAKE ERIE
LAKE ONTARIO
Buffalo
Cleveland
Detroit
CHICAGO
CALUMET R
Indiana Harbor
Gary

Fig. 1.

THE MAIN ORE UNLOADING DOCK FAILURES AND THEIR CORRECTION 1909 - 1925, GREAT LAKES REGION

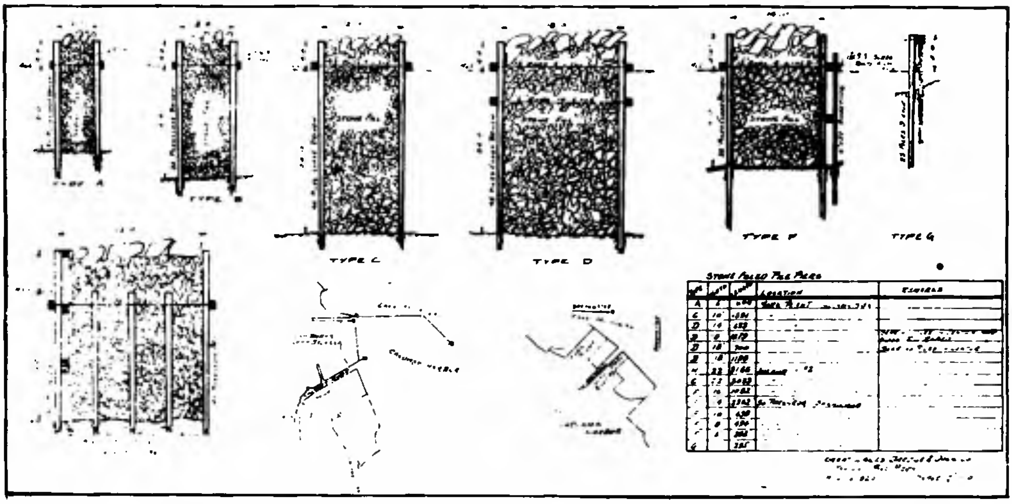

Fig. 2.

Fig. 3. Wilmette, Illinois: Stone-filled pile breakwater (1,700 lineal feet), built in Lake Michigan to protect lake end or intake of the Evanston Sanitary Canal. Other similar pile piers built at Calumet Harbor, Ill., for the Illinois Steel Works, 2,500 lineal ft. Calumet Harbor, Ill., for the Iriquois Iron Co., 3,700 lineal ft., Indiana Harbor, Ind., for the Inland Steel Company, 1,500 lineal ft.

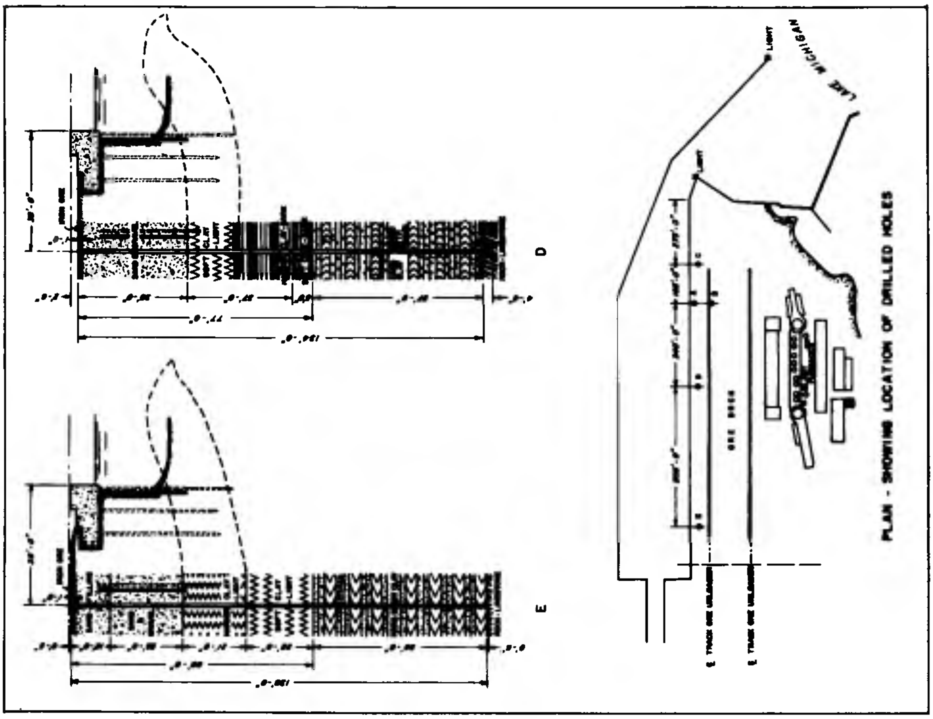
PLAN - SHOWING LOCATION OF DRILLED HOLES
LAKE MICHIGAN
LIGHT
LIGHT
D
E

Fig. 5.

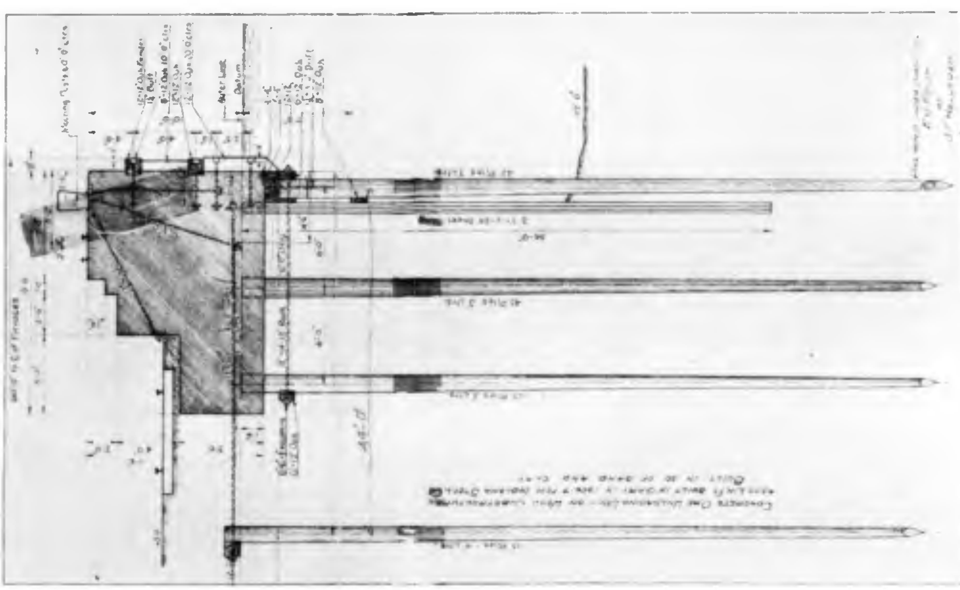

Fig. 4.

THE MAIN ORE UNLOADING DOCK FAILURES AND THEIR CORRECTION 1909 - 1925, GREAT LAKES REGION

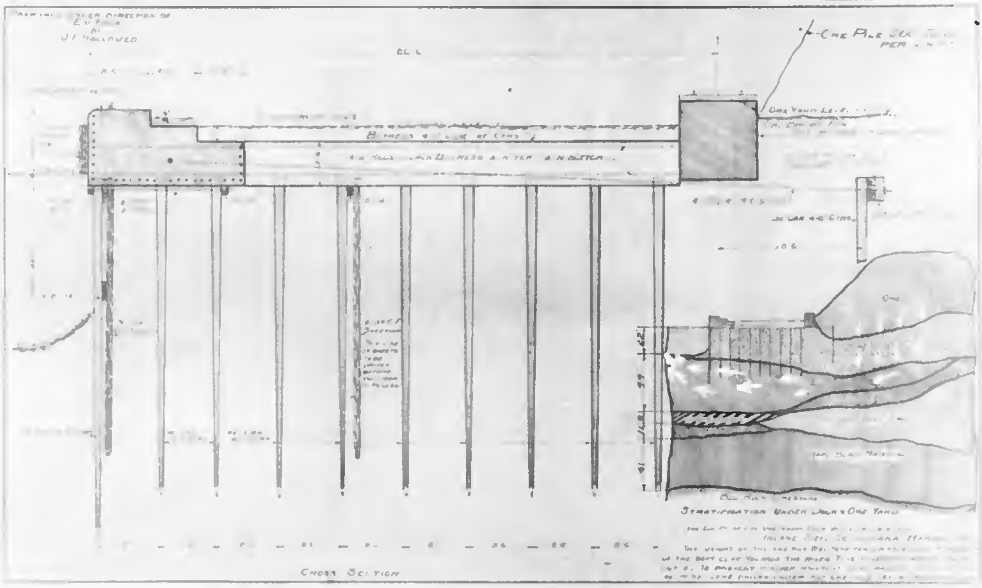

Fig. 6.

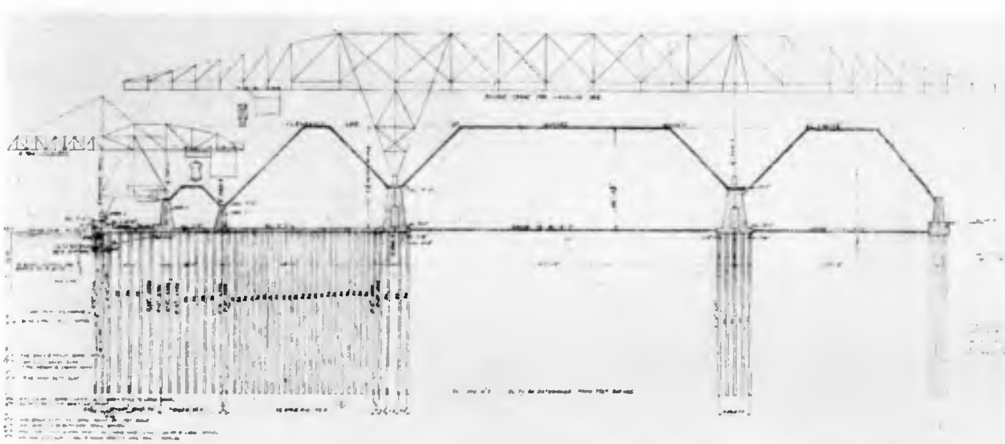

Fig. 7.

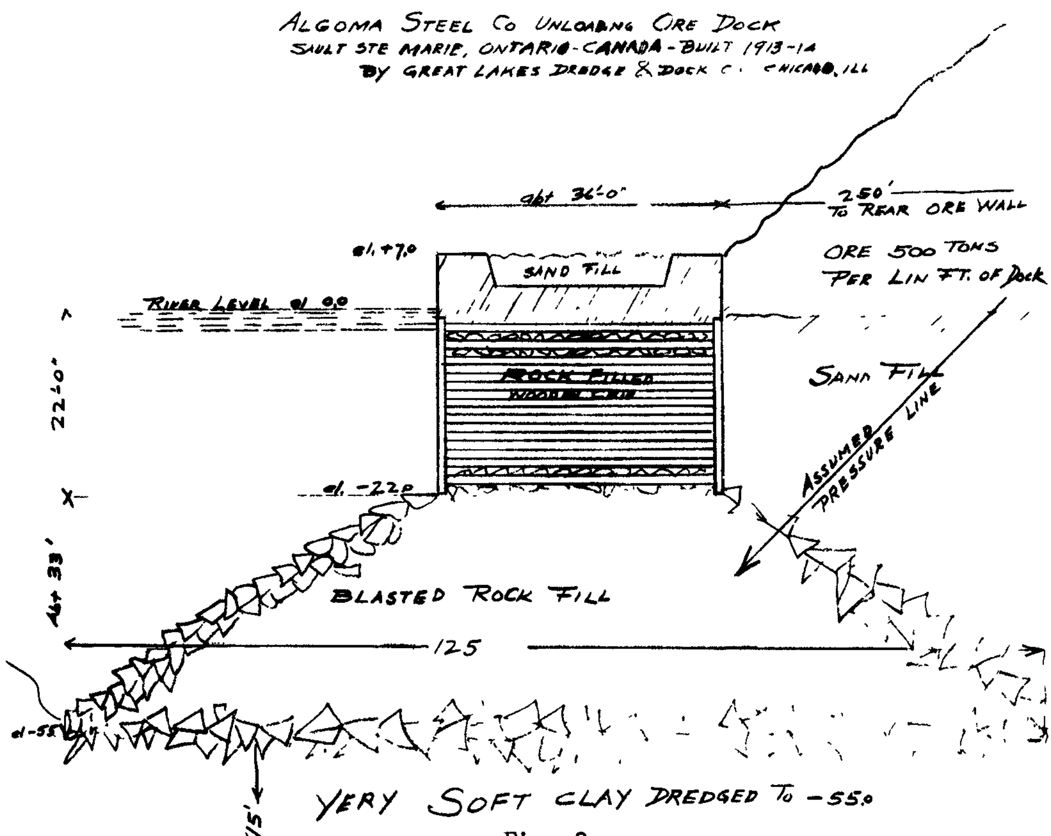

Fig. 8.

As can be seen by the drawing, the design called for dredging with a dipper dredge down to -55 feet (an unusual depth in 1913) and consolidating the soft clay below that elevation by a pile of dumped blasted rock from -55 feet to -22 feet making a trapezoidal section 33 feet thick, 125 feet wide at -55 feet and about 40 feet wide at -23 feet. It was assumed that by the time the dumped rock had been built up from -55 feet to -22 feet, the soft clay would be stiffened or compacted sufficiently below to hold the stone-filled wooden crib. Moreover, it was assumed the mass of rock 33 feet thick and 125 feet wide, topped with a 36 foot wooden crib filled with rock and capped with six feet of concrete would act as a unit against the ore load which started about 40 feet back of the dock face. This was actually what occurred.

The design was considered ingenious as no anchorage was used, and for the first time the shear factor of soft clay was considered in heavy construction. This, we think, was soil mechanics in its earliest stage. The structures described in this paper are still in use after 40 years.

CHAPTER 23

SHOCK PRESSURE OF BREAKING WAVES

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INTRODUCTION

Most of you have observed waves breaking against rocks or structures, and have noted that water is frequently thrown high in the air. The pressure required to project water in a vertical direction is about one half pound per square inch for each foot of height. We may therefore expect to find substantial pressure involved in the mechanics of a wave breaking against a structure.

Pressures as great as 100 psi caused by waves breaking on a structure at Dieppe, France, have been observed by Besson and Petry (1938).

Bagnold (1939) has observed pressures as high as 80 psi caused by 10-inch waves in a wave tank.

The Beach Erosion Board, being especially interested in the subject of wave pressures on shore structures, has continued the study of the pressures of breaking waves. This study was designed to investigate the high-intensity shock pressures on the structures as contrasted to the much smaller hydrostatic pressures developed by the rise on the wave against the face of the structure.

EQUIPMENT USED IN THE TESTS

The pressure sensitive elements of the pressure gauges consist of plates of tourmaline crystal. This material is sensitive to hydrostatic pressure changes. The plates are separated from the water by only thin layers of wax, rubber, and shellac. The element, consisting of four one-inch disks or wafers, is set in and backed by a strong metal case. The possibility of spurious signals caused by resonance or of loss of sensitivity in connecting parts is greatly reduced by the simple and strong construction of the gauges.

The application of pressure to the gauges produces a small charge of electricity: 34 micro-micro coulombs for one pound per square inch change in pressure. The surfaces of the tourmaline disks are covered by thin conducting coatings which collect the charge. A voltage is produced which is inversely proportional to the capacitance of the gauge and leads. It is necessary that the resistance or the insulation be in the order of 1000 megohms to prevent the charge from leaking away too quickly. The voltage is carried to the grid of a radio tube by coaxial cable. The tube is biased so that the grid draws no current. The output of the radio tube can then be connected to the oscilloscope which has an input resistance of only 2 megohms.

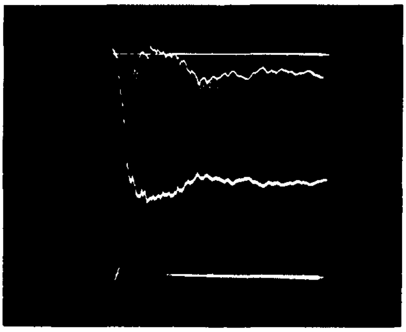

Figure 1

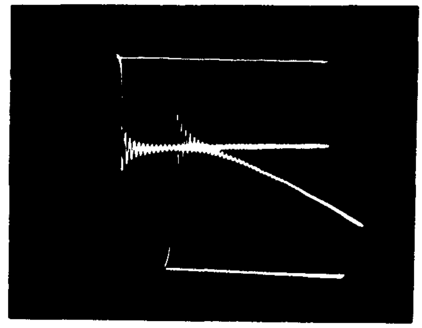

Figure 2

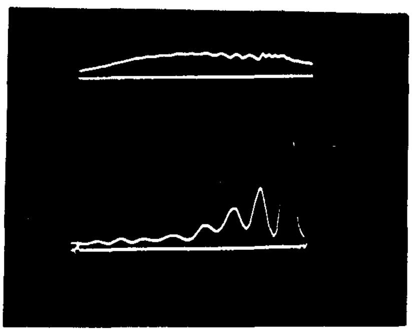

Figure 4

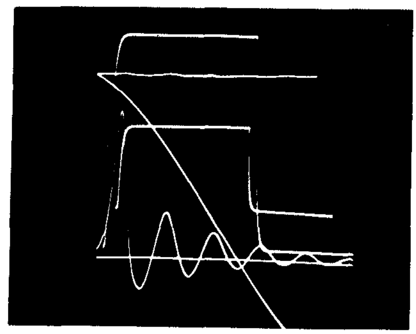

Figure 5

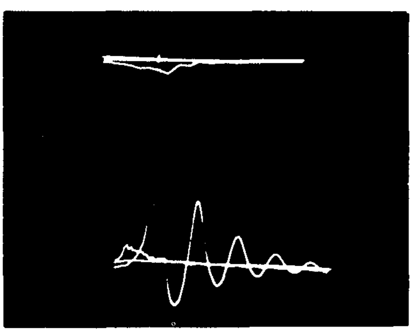

Figure 6

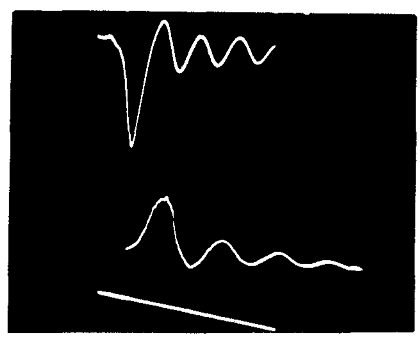

Figure 7

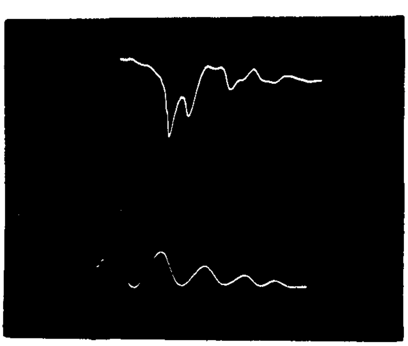

Figure 8

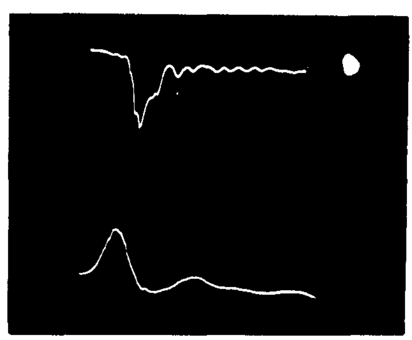

Figure 9

DESCRIPTION OF FIGURES

Figure 1

The traces represent the release of a pressure of 43 psi by the rupture of a 2-inch diaphragm of 0.001-inch steel. There are two traces from one pressure cell, one being through the D.C. amplifier and the other through a delay line and the A.C. amplifier. The delay line inverts one trace with respect to the other. The sweep time is 1/720 second. The pressure was released in slightly more than 1/10,000 second. It is difficult to relsase pressure quickly without oscillations. These may be noted in the traces.

Figure 2

These traces represent the release of 36 psi pressure by the expulsion of a cork. The sweep time is 1/14 second. The pressure release occurred in slightly more than 1/1,000 second. The traces indicate the error caused by the loss of signal with time. The D.C. amplifier trace falls only about 6 per cent in 1/14 second. This drop is caused by the loss of charge through the insulation of the cell and leads. The A.C. amplifier will hold the signal for only about 1/60 second with a similar loss of signal.

Figure 3

Figure 3 is a diagram of the wave tank in which the tests were made.

Figure 4

These traces represent the pressures of two breaking waves. The larger one represents a pressure of 13.5 psi and is number 8 in Table 1. The smaller one represents a pressure of 2.4 psi. The time-pressure integrals are the same, however, 0.011 pound-seconds per square inch. The sweep time is 1/120 second.

Figure 5

This trace represents a pressure of 13.2 psi and is number 25 in Table 1. The sweep time is 1/60 second. The trace of the square wave which is used to calibrate the amplifiers is also present. By comparison with the square wave the voltage represented by the wave trace can be found. A comparison with the results of the calibration tests with release of pressure then indicates the pressure in psi.

Figure 6

The larger trace represents a pressure of 18.9 psi. This is number 39 in Table 1.

Figure 7

These traces represent pressures of 8.5 and 4.9 psi. The pressure cells were at different elevations. The sweep time was 1/60 second. This wave is number 47 in Table 1.

Figure 8

These traces represent pressures of 6.3 and 6.5 psi caused by a breaking wave. The sweep time was 1/60 second. They are number 51 in Table 1.

Figure 9

These traces represent pressures of 6.3 and 5.5 psi. The sweep time was 1/60 second. They are number 55 in Table 1.

Two pressure cells were used with a dual channel oscilloscope. One of the amplifiers of the oscilloscope was an AC type and the other a DC type. The sweep of the oscilloscope was triggered by the signal.

A 4 micro-second delay line and special triggering device were used in many of the tests to insure that we were not losing the initial rise of the signal.

The traces on the oscilloscope screen were recorded by an oscilloscope camera.

The cells and apparatus were calibrated by placing the cells in a small chamber and releasing air pressure by the breaking of a diaphragm or the expulsion of a cork. The first method released the pressure in about 1/10,000 second and the other in about 1/1,000 second. The electric charge produced by tourmaline is linear with changing pressure so that the calibration with the release of pressure may be used with the increase of pressure produced by the breaking wave. The oscilloscope traces from two calibration tests are shown in Figures 1 and 2.

The tests were made in an indoor tank with a length of 96 feet, a depth of 2 feet, and a width of 1 1/2 feet. A diagram of the wave tank is shown in Figure 3.

The waves were generated by a wave machine of the moving bulkhead type. The bulkhead was caused to move by a crank wheel and connecting rod. The speed of rotation of the wheel could be varied to give waves with periods from 1 to 5 seconds. The length of the crank arm was variable in 1/2-inch steps from 2 to 11 inches to give waves of various heights.

The pressure cells were mounted in two 1/2-inch steel plates. Each plate formed one half of the bulkhead representing the vertical structure against which the waves were to break. The sensitive faces of the cells were flush with the surface of the plates. The plates were mounted on a steel frame and could be raised or lowered to change the vertical position of the gauges.

The waves were caused to break by a beach formed of concrete slabs. The height at the bulkhead was 10 inches. Various beach slopes were used from 0.078 to 0.176.

A recording wave gauge was used to give a time profile of many of the waves. It was located about 30 feet in front of the bulkhead.

TYPES OF BREAKING WAVES

Three types of wave conditions were used in the tests.

First, a small wave is formed by starting the wave machine at a certain point in its cycle. If the size of the small wave is regulated correctly by the starting position of the machine, the backwash of this small wave will cause the next or first full-sized wave to break against the bulkhead.

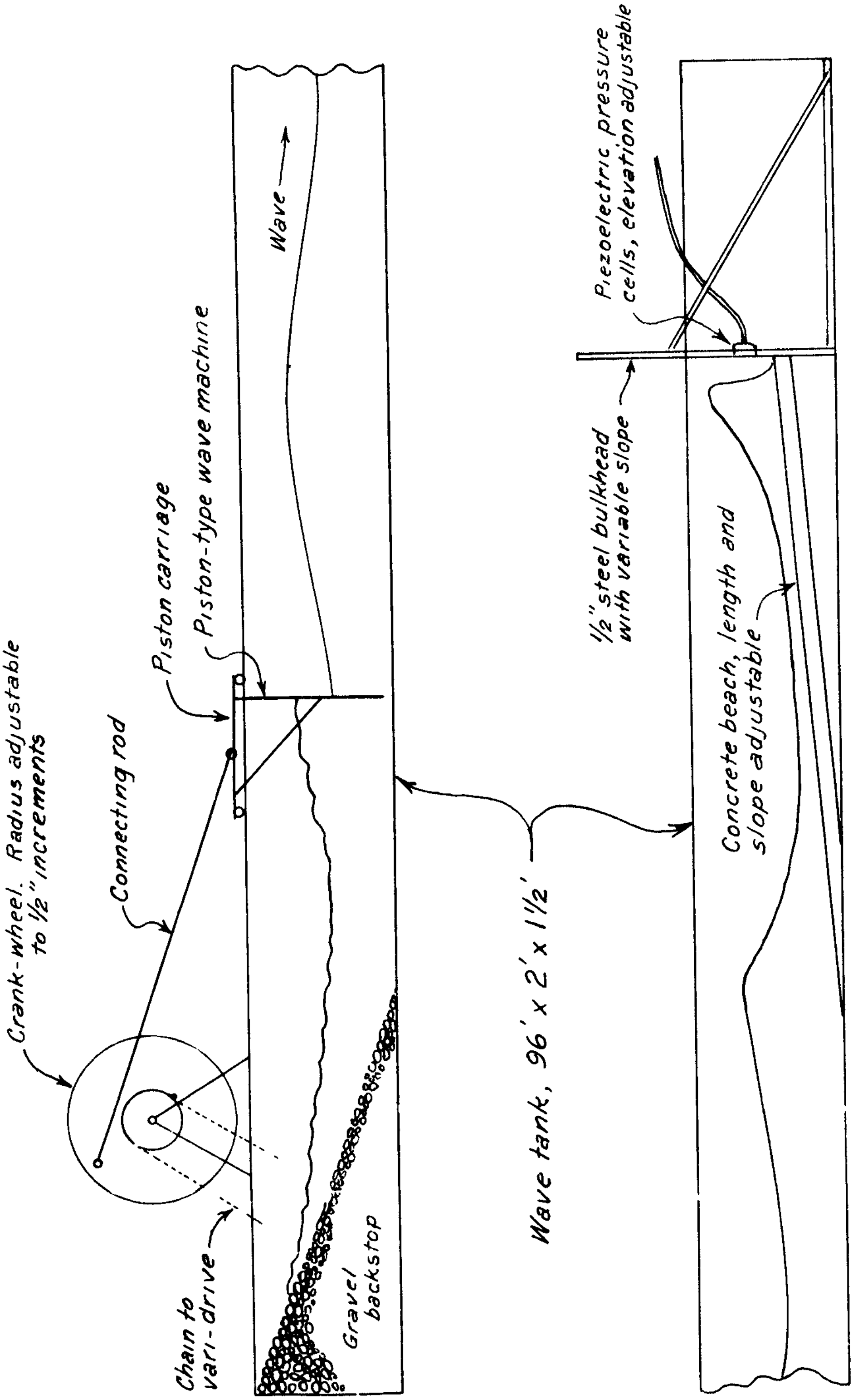

Figure 3. Diagram of wave tank

Waves starting with the second and ending with roughly the tenth are called early waves. These early waves were caused to break in proper position by adjustment of the water depth. At this depth the backwash of the preceding wave is such as to cause each wave to break in proper position.

After 8 to 12 waves have broken on the bulkhead, the influence of the reflected waves traveling from the bulkhead to the wave machine and then back to the bulkhead causes variation in height of the incident waves. The total distance is about 160 feet. These are called late waves.

The first two conditions are reproducible and reliable pressure producers. A slight variation of the period, depth, etc. would cause the waves to break too early or too late to produce much pressure. For the third wave condition, the production of pressure was infrequent. Sometimes very high pressures were produced, however.

The height of the waves varied from 3 to 7.5 inches as measured 30 feet from the bulkhead. The height of the third wave was measured because in subsequent waves the record was complicated by the reflected wave.

PRESSURES PRODUCED BY BREAKING WAVES

Four wave pressures were recorded greater than 18 psi, twenty-one greater than 10 psi, and more than 300 greater than 5 psi. The maximum pressure which would be produced by hydrostatic pressure (clapotis) is only about 0.5 psi.

High pressures occured more frequently and over a larger area of the bulkhead with larger waves. However, sometimes a large wave would break with a big bang and produce a pressure of one or two psi and then a small parasitic wave between the larger waves would slap the bulkhead lightly and produce a pressure of seven or eight psi.

Twenty-two consecutive tests of the first wave type gave 44 values of the pressure with an average of 5 psi. Simultaneous pressures of from 3 to 5 psi occured with the gauges separated vertically by 2 inches. The gauges are separated 9 inches horizontally. This indicates that high pressures occurred over a relatively large area of the bulkhead at the same time.

Most of the shock pressures were observed when the pressure gauges were between one inch below and three inches above the still water line.

A sample of the wave data with experimental conditions is given in Table 1.

Some oscilloscope traces indicating the shock pressures are shown in Figures 4 to 9.

The durations of the shock pressures are short. Bagnold has noted that the time integral of the pressures seems to approach an upper limit. Numerous measurements were made of the time integral of the pressures produced by the waves in these tests. The maximum values found were slightly greater than 0.02 psi-seconds.

SHOCK PRESSURE OF BREAKING WAVES

EFFECT ON STRUCTURES

If we consider the pressure of 18 psi developed by 7-inch laboratory waves, we may well be interested in what full-scale ocean waves may do to a structure.

In these tests, the larger pressures are of too short duration for a structure of much weight to be moved appreciably. At model scale, most types of pressure gauges have too much inertia and resiliency for them to even detect these pressures.

Measurements have shown that the velocity of the peak of a breaking wave approaches the wave velocity. The velocity of the face of the wave decreases rapidly with lower elevation. If we assume a horizontal thickness of the breaking wave as 3 inches and its velocity at the same elevation as 3 feet per second, we find a momentum of 0.32 lbs-ft/sec-in.^{2}, equivalent to an impulse or a pressure-time integral of 0.01 psi-second. If the velocity is reversed, this amount is doubled. This is the approximate magnitude for the higher values of the impulse measured. It may be remembered that a variable amount of the momentum will be overcome by hydrostatic pressure which may account for the lower values. Some of the momentum is not reversed but is converted to turbulence and into the vertical motion of the spray.

If air were not present, we might well expect the pressure to approach the pressure of water hammer. However, some air is always trapped by the breaking wave. The more air trapped, the lower the shock pressure and the longer its duration.

When we consider waves of larger size, we may expect the shape to be similar. The corresponding horizontal section of the breaking wave will be increased in length by the scale ratio.

The velocity of waves in shallow water is proportional to the square root of the depth of the water. Since the larger waves may be expected to break in proportionally greater depth, the velocity will then be greater by the square root of the scale ratio.

The scale ratio for the impulse should be the product of these or the scale ratio raised to the three-halves power. If 7-inch waves produce an impulse of 0.02, a 14-foot wave should then produce an impulse of $0.02 \times 24^{3/2}$ or 2.35 psi-seconds.

The range of wave sizes in these tests was not large enough to check this scale ratio. We hope to make some tests on a much larger scale when our wave machine is completed for our 635-foot tank.

Pressure measurements have been made with full-scale waves at Dieppe, France. The pressure records of 7 waves give values of from 0.38 to 0.73 psi-seconds for the impulse. When corrected for the scale of the wave, these values become 0.003 to 0.010 psi-seconds as 7-inch waves.

Bagnold found the time-integral of the pressure for his 10-inch waves to approach 0.018 psi-seconds. This becomes 0.010 when reduced to the scale of these tests.

TABLE I

DATA FOR SOME OF THE WAVES PRODUCING SHOCK PRESSURES

No.	Wave period	Stroke of crank	Still Water depth	Pres. cell height	Max. pres.	Pressure time integral	Type of wave	Approx. wave height	Beach slope
	sec.	in.	in.	in.	psi	psi-sec.		in.	
1	3.5	4	12.4	14.5	5.9	0.0078	E	4.0	0.094
2	4.0	4	13.0	12.9	4.2	0.0064	L	4.0	0.094
				14.5	9.1	0.0142			
3	4.0	4 1/2	13.2	13.4	9.0	0.0132	L	4.0	0.176
4	4.0	5	13.4	12.9	10.6	0.0098	E	4.0	0.094
5	4.0	5	14.0	14.5	10.1	0.0130	L	4.5	0.094
6	4.0	5	12.7	15.0	7.0	0.0057	L	4.0	0.078
7	5.0	5	12.6	14.5	7.5	0.0083	E	4.0	0.094
8	4.0	5 1/2	12.2	13.4	13.5	0.0110	L	4.5	0.176
9	4.0	5 1/2	13.8	14.5	8.3	0.0150	L	4.5	0.094
10	3.0	6	13.4	12.9	10.4	0.0100	L	5.0	0.094
11	4.0	6	14.0	14.5	10.0	0.0114	E	5.0	0.094
12	4.0	6	13.0	14.5	12.2	0.0099	E	5.0	0.094
13	4.0	6	13.4	12.9	21.4	0.0114	L	5.0	0.094
14	5.0	6	13.0	14.5	10.4	0.0190	L	3.5	0.094
15	5.0	6	13.1	14.5	7.2	0.0182	L	3.5	0.094
16	4.0	7	14.0	12.9	15.7	0.0165	L	5.5	0.094
17	4.0	7	14.0	12.9	6.1	0.0129	L	5.5	0.094
18	4.0	7	14.2	13.3	14.6	0.0147	E	5.5	0.144
19	4.0	7	14.2	13.3	10.8	0.0094	E	5.5	0.144
20	5.0	7	13.4	14.5	11.1	0.0036	L	5.0	0.094
21	5.0	7	12.7	14.5	9.1	0.0114	F	5.0	0.094
22	3.0	7 1/2	12.7	12.6	6.0	0.0134	E	6.0	0.176
23	4.0	7 1/2	14.0	13.3	7.6	0.0150	E	5.5	0.144
24	4.0	7 1/2	14.0	13.3	8.8	0.0215	E	5.5	0.144
25	4.0	8	13.8	14.5	13.2	0.0120	F	6.0	0.094
26	4.0	8	13.2	14.5	5.4	0.0199	E	6.0	0.094
27	5.0	8	13.1	15.2	7.5	0.0164	F	5.0	0.094
28	5.0	8	13.7	15.2	5.9	0.0130	F	5.0	0.094
29	5.0	8	13.1	15.2	9.4	0.0147	F	5.0	0.094
30	5.0	8	12.0	13.3	6.5	0.0128	F	4.5	0.078
31	5.0	8	12.4	14.5	7.1	0.0133	F	4.5	0.094
32	5.0	8	11.9	12.9	2.6	0.0096	F	4.5	0.078
33	4.0	9	13.7	14.4	4.4	0.0048	F	6.5	0.094
				15.2	5.6	0.0189			
34	5.0	9	13.2	14.5	6.8	0.0022	L	6.5	0.094
35	5.0	9	13.2	14.5	5.3	0.0024	E	6.5	0.094
36	5.0	9	13.6	14.5	11.5	0.0206	L	6.5	0.094
37	5.0	9	12.0	13.6	4.8	0.0056	F	6.0	0.078
				15.0	5.9	0.0226			
38	5.0	9	12.0	13.6	5.6	0.0086	F	6.0	0.078
				15.0	3.0	0.0153			
39	5.0	9	14.6	14.5	18.9	0.0182	L	7.0	0.094
40	3.7	9 1/2	10.7	14.5	16.7	0.0088	E	7.0	0.094
41	3.7	9 1/2	10.7	14.5	8.0	0.0087	F	7.0	0.094
42	4.0	9 1/2	14.0	14.1	7.6	0.0096	L	7.0	0.144
43	5.0	10	12.2	14.2	3.3	0.0117	F	7.5	0.078
				15.0	4.7	0.0088			

TABLE 1 (Continued)

No.	Wave period	Stroke of crank	Still water depth	Pres. cell height	Max. pres.	Pressure time integral	Type of wave	Approx. wave height	Beach slope
	sec.	in.	in.	in.	psi	psi-sec.		in.	
44	5	10	12.2	14.2	4.3	0.0127	F	7.0	0.078
				15.0	6.1	0.0132			
45	5	10	12.2	14.2	7.5	0.0116	F	7.0	0.078
				15.0	2.3	0.0105			
46	5	10	12.2	14.2	6.1	0.0136	F	7.0	0.078
				15.0	6.0	0.0125			
47	5	10	12.2	14.2	8.5	0.0137	F	7.0	0.078
				15.0	4.9	0.0093			
48	5	10	12.2	14.2	4.5	0.0116	F	7.0	0.078
				15.0	5.8	0.0090			
49	5	10	12.2	14.2	8.3	0.0172	F	7.0	0.078
				16.0	5.9	0.0091			
50	5	10	12.2	14.2	4.8	0.0142	F	7.0	0.078
				16.0	4.3	0.0090			
51	5	10	12.2	14.2	6.5	0.0129	F	7.0	0.078
				16.0	6.3	0.0061			
52	5	10	12.2	13.5	6.1	0.0112	F	7.0	0.078
				15.3	5.0	0.0150			
53	5	10	12.2	13.5	3.3	0.0097	F	7.0	0.078
				14.4	5.9	0.0135			
54	5	10	12.3	14.4	6.4	0.0139	F	7.0	0.078
				13.5	3.5	0.0103			
55	5	10	12.3	14.4	6.3	0.0152	F	7.0	0.078
				13.5	5.5	0.0099			
56	5	10	12.3	14.4	5.9	0.0168	F	7.0	0.078
				13.5	4.4	0.0106			
57	5	10	12.3	13.3	4.3	0.0135	F	7.0	0.078
				13.0	3.1	0.0071			
58	5	10	12.2	14.2	10.2	0.0119	F	7.0	0.078
59	5	10	12.2	14.2	10.9	0.0075	F	7.0	0.078
60	5	10	13.0	14.5	6.5	0.0021	L	7.5	0.078
61	5	10	13.0	14.5	2.6	0.0181	F	7.5	0.078

TABLE 1 - NOTES

The data in Table 1 is selected to show conditions which gave high wave pressures. Many tests were made in which low pressures or no pressures were produced. For many conditions (wave periods, wave size, beach slope, etc.) the waves could be caused to break and give pressures by adjusting the water depth in the wave tank. The depth for pressure depended on the type of wave.

The motion of the bulkhead producing the waves is about twice the length of the crank arm.

"F" indicates that the wave causing the pressure was the first full wave; "E" indicates a wave after the first but before the influence of any reflection of the first wave travels from the bulkhead to the wave machine and back again; and "L" indicates a wave after the heights of the waves becomes somewhat variable because of variation of depths caused by reflected waves at the wave machine.

Because of the inertia of the water, pressure is not quickly released downward or backward as a wave breaks against a vertical structure. However, a substantial amount of water is usually thrown upward because nothing but air interferes with its upward escape. Therefore, the tops of vertical structures must be designed to withstand the force of falling water.

CONCLUSIONS

The time integral of the pressure approaches a value of 0.02 psi-seconds for 7-inch waves. This limit depends on the three halves power of the scale ratio for waves of other sizes.

No limit on the magnitude of the pressure developed can be given except that which would develop with water hammer. However, the elasticity of the trapped air should always preclude pressures of such high values.

Design of structures should normally be based on the total momentum to be reversed. Usually the high pressures will not be important.

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CHAPTER 24

SOME DYNAMIC ASPECTS IN THE DESIGN OF MARINE STRUCTURES ON THE GREAT LAKES

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INTRODUCTION

An investigation of the failures of a number of marine structures located in the southern portion of Lake Michigan has shown that forces must have existed at the time of failure of a type and of a magnitude whose significance to adequate design had not always been fully realized. The purpose of this paper is to indicate the probable nature of these forces and in one or two instances to estimate their magnitudes.

NATURE OF FORCES

The forces believed to be critical in the structures studied are the following:

1. Up-pressures acting on component elements of composite type breakwaters. These forces appear to be the result of hydrostatics as well as of the dynamic action of waves.

2. Horizontal dynamic pressures of high intensity resulting from breaking waves which act in the vicinity of the mean storm water surface.

3. Vibration resonance phenomena in certain structures which occur when the natural period of the structural system and sustained storm wave action coincide.

TYPES OF FAILURE

Rubblemound breakwaters. Repair and maintenance records for rubblemounds in the Chicago area indicate that this type of structure is subjected to disintegrating forces acting more or less continuously. The zone of marked destructive influence extends downward to approximately 12 feet below the mean lake level. The average annual loss of rock is often nearly as great for periods of little storm activity as for periods which included severe storms. Frequently the loss of stone is of the same order of magnitude for rubblemounds whose orientation and exposure would prevent direct storm attack as for breakwaters that are aligned and exposed so as to insure more direct storm wave action. Although no measurements are available it is believed that the disintegration is caused by long continued action of up-pressures induced by direct as well as reflected waves.

Sheet piling retaining walls. Two instances of failure of sheet piling walls so located as to expose them to frequent and sustained storm wave action shows the decided possibility of the existence of action of resonance between the applied period of the storm waves and of the natural period of the structural systems. In both cases the structures failed during storms of moderate intensity but of sustained duration (30 to 42 hours). The structural and corresponding storm wave periods were in the range of 6.5 to 7.0 seconds.

Walls on composite breakwaters. The displacement of very large cap stones on rubblemound breakwaters and the rupture of heavy concrete foot blocks on timber crib structures during intense storms supports the contention that severe dynamic forces must have occurred in the vicinity of the mean storm water surface. Evidence also indicated that dynamic wave action took place not only on sloping structures in relatively shallow water but occurred as well on breakwaters with vertical faces in water as deep as 35 feet. The concurrent action of up-pressures and of horizontal wave forces was evident in the cases investigated.

Cellular breakwaters. There has been only one case of complete failure of a cellular steel sheet piling breakwater in the lower Lake Michigan region (1949). These cells were sand filled with a water depth of 25 feet. An analysis of the causes of failure showed that wave forces of high intensity probably existed near the mean water surface and that vibration and wave overtopping caused loss of the sand fill. The resulting reduction of friction between the fill and the walls of the cells allowed slippage to occur along the vertical interlocks of the piling so that the structure was caused to fail by tipping. The possibility of this form of failure of sand filled cellular breakwaters had been pointed out by Dr. Karl Terzaghi in 1945.

DYNAMIC WAVE FORCE

The large forces exerted on breakwaters by storm waves in Lake Michigan, as is borne out by the destructive effects noted at or near the mean storm water surface, precludes the use of any design criteria which does not include the dynamic effects of breaking waves. Formulas such as those developed by Sainflou find application in the Chicago area only in those rare instances in deepest water where waves do not readily break. Perhaps this is due to the fact that the relatively short period waves representative of the Great Lakes break more frequently in moderately deep water than do the longer period waves of the oceans.

The following linear relationship between wave height and dynamic wave pressure was developed from the hydraulic bore equation, as modified by velocity coefficients evaluated from the solitary wave profile curves of Munk, and the dynamic resisting force of a stationary flat plate.

$$p_{max} = 133.6\ H \text{ (lbs./sq. ft., in fresh water)} \qquad (1)$$

Concerning factual information on maximum dynamic wave pressures, the measurements made by Capt. D. D. Gaillard, U.S.A., represents the only observations of prototype forces to be expected in the Great Lakes. A comparison of those observations and the pressures predicted by equation (1) is shown in Figure (1).

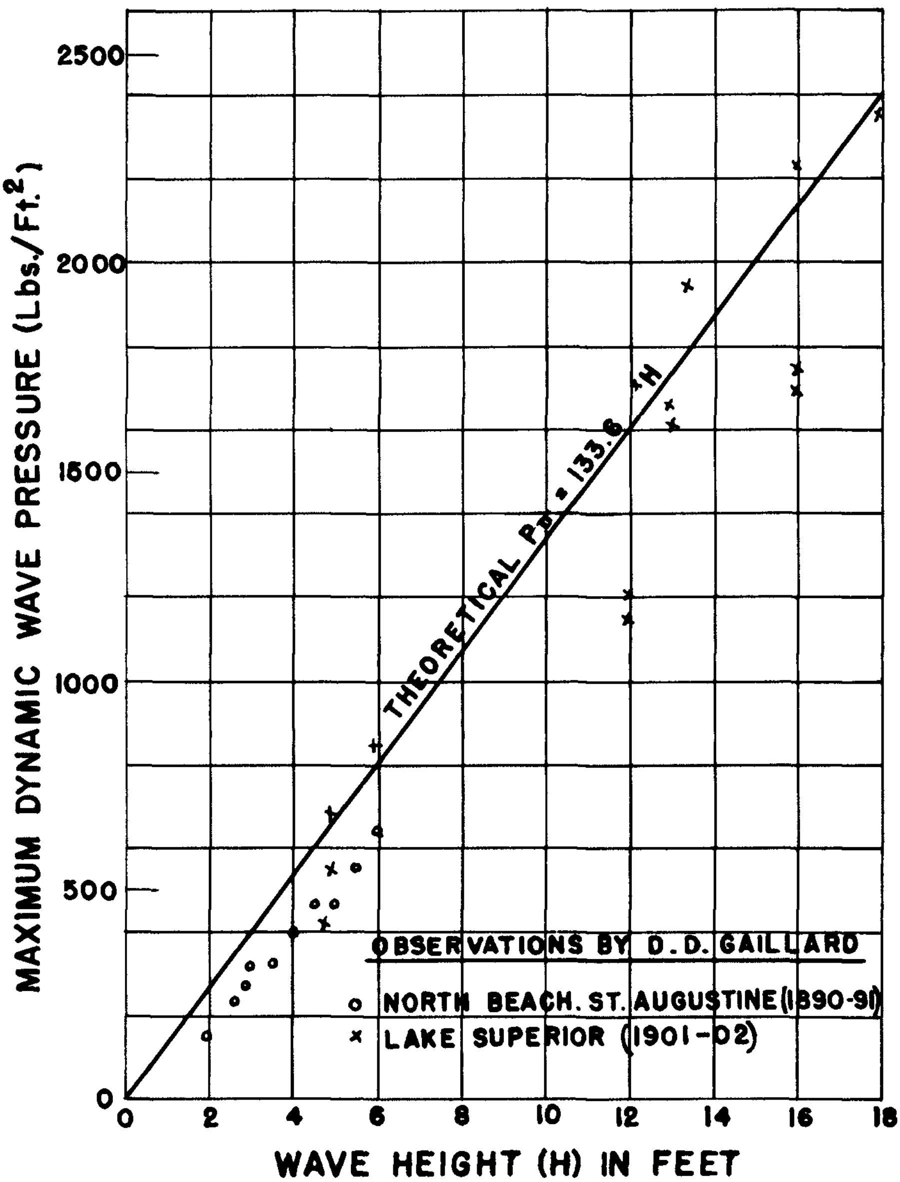

Fig. 1. Relationship between wave heights
and maximum dynamic wave pressures.

EXAMPLES OF FAILURE

Gary Breakwater. As an example of the failure of a composite breakwater the United States Steel Company's structure at Gary, Indiana (Figure 2) will be analyzed so as to estimate the forces which had acted. This breakwater, see Figure 3, is a rock filled timber crib that had been capped with concrete. As a result of a severe storm on 25 and 26 November 1950 a 200 foot long section (2 crib lengths) of concrete cap slid laterally a distance of 3 to 4 feet. (Figure 5).

An anemometer located on the U. S. Steel unloading bridge at Gary, adjacent to the breakwater, recorded wind from NNW averaging 36 m.p.h. for 18 consecutive hours. During this period wave heights of 14 feet were estimated by attendants at the Chicago Water Intake Crib (Dunne). Recording wind gages at the South Side Filtration Plant and at the Navy Pier, Chicago, showed wind velocities similar to those at Gary.

Using wind duration-velocity and wave height curves it was estimated that the deep water waves were 17 feet in height with a 7.2 second period. The corresponding wave height at Gary breakwater (water depth of 35 feet) was computed to be 13.5 feet.

The concrete cap (Figure 4) weighs 25,600 lbs. per lineal ft. On the basis of a coefficient of friction = 0.6 (probably higher at failure), the force required to displace cap = 0.6 x 25,600 = 15,630 lbs. per lineal ft. Additional resistance to displacement was provided by $1\frac{1}{8}$ " drift bolts spaced at 3'-0" centers in both walls. A conservative value for the resistance of drift bolts would be 1000 lbs. per lineal ft. based on the fact that they were probably stressed to the yield point before displacement of the cap could have occurred. Therefore, the total resistance to sliding was 16,630 lbs. per lineal ft.

Depending upon the magnitude and distribution of uplift that acted, it appears that the wave force acting on the Gary breakwater to cause failure had a peak intensitv of from 1440 lbs. per sq. ft. to 2500 lbs. per square foot (based on a horizontal force pressure distribution curve in which the average intensity is 50% to 67% of peak intensity).

Cellular Breakwater, Indiana Harbor. During the same storm of November 1950, a sand filled steel sheet piling cellular breakwater failed by tipping at Indiana Harbor (located NW of Gary, Figure 2). In this instance the maximum dynamic pressure to cause failure for a computed wave height at the breakwater (depth = 25 ft.) of 12 feet was approximately 1800 lbs. per sq. ft.

Other Cases. Captain Gaillard has reported similar earlier failures of marine structures in the Great Lakes. A notable example occurred at the north breakwater, Buffalo Harbor, New York, during the storm of 12 September 1900. The breakwater was of the crib type with capstones 5 feet high, and bases 7 ft. x 8 ft. Each block weighed 27,800 lbs. A rough computation indicates that in the vicinity of the mean storm water surface the average wave force required to displace the stones must have been in the range 600-1000 lbs. per square foot with corresponding peak pressures

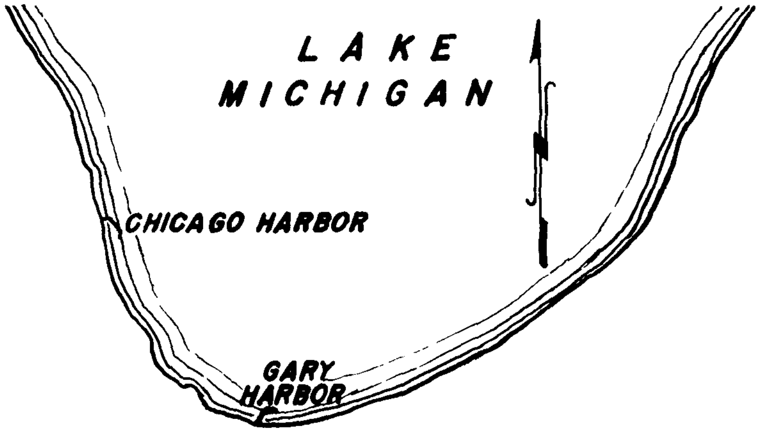

Fig. 2. Vicinity map, Gary Harbor, Indiana.

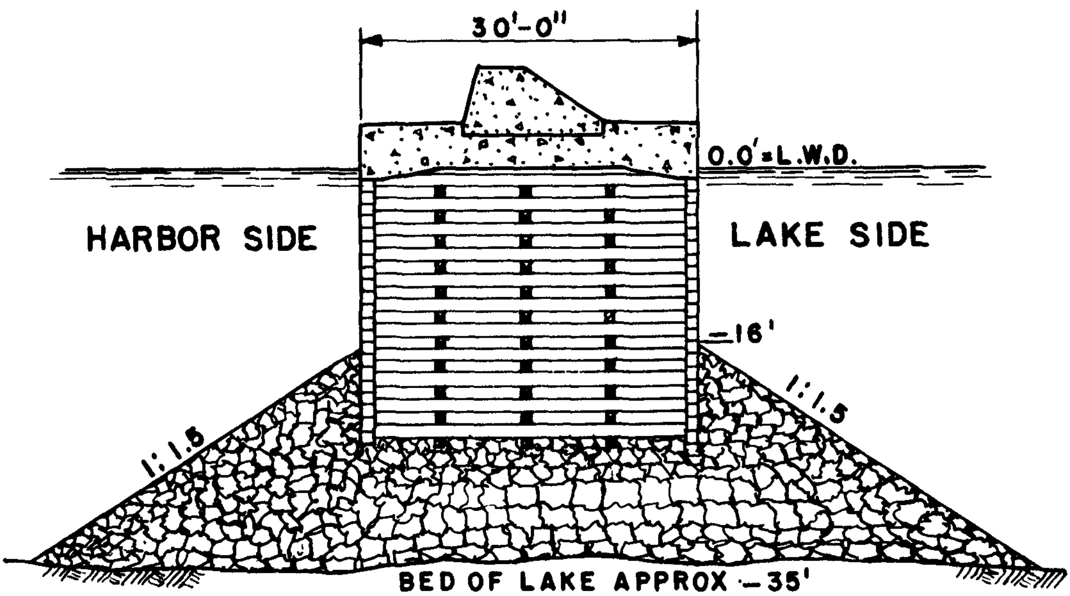

Fig. 3. Cross-section of Gary breakwater

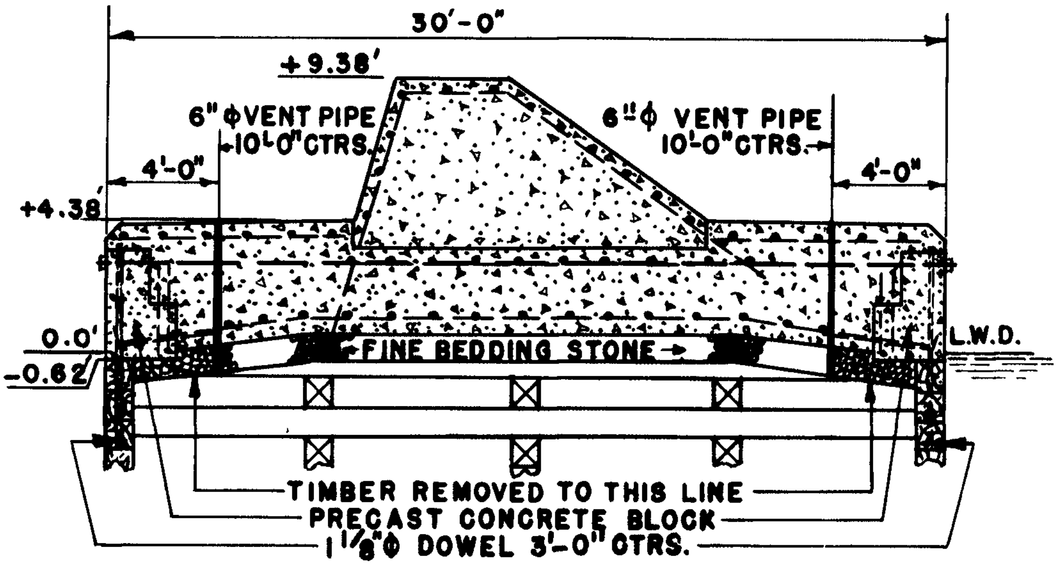

Fig. 4. Details of cap stone construction.

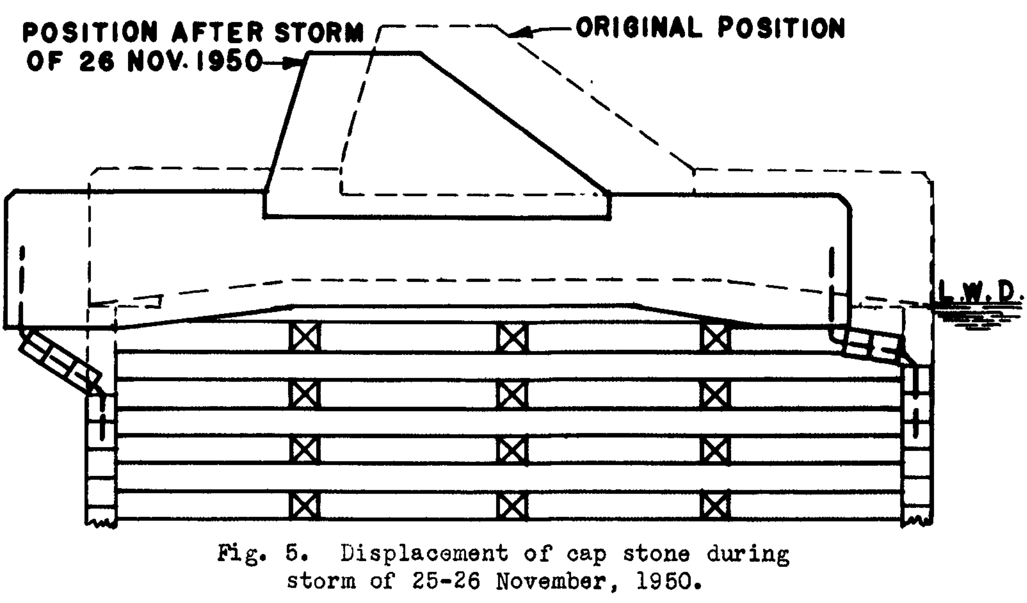

Fig. 5. Displacement of cap stone during storm of 25-26 November, 1950.

of 1000 to 1900 lbs. per square foot, depending upon the uplift assumed acting concurrently.

CONCLUSIONS

The foregoing description and analyses of failures of marine structures in the Great Lakes indicates the not infrequent occurrence of heavy wave pressures (and concurrent uplifts) that can only be adequately accounted for as resulting from forces set up by waves breaking in relatively deep water. Studies of this type indicate the desirability of further study of the nature and measurements of the magnitude of these forces so as to be able to determine adequate design criteria.

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CHAPTER 25

EXPERIMENTAL STUDIES OF FORCES ON PILES

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INTRODUCTION

In the design of a pile structure exposed to surface waves of a given height and period, some of the factors involved in the problem and studied herein are the size, shape and spacing of the piles and the moment distribution on uniform and non-uniform piles. Theoretical and experimental investigations have shown that the force exerted by surface waves on a pile consists of two components -- a drag force and an inertia force. The drag force is proportional to the fluid density, the projected area and the square of the fluid particle velocity. The inertia force, including the virtual mass, is proportional to the fluid density, the volume of the object and the fluid particle acceleration. The virtual mass is the apparent increase of the displaced mass of fluid necessary to account for the increase in force resulting from the acceleration of the fluid relative to the object. This factor is included in the coefficient of mass term in the force calculations.

The experimental and analytical approaches to the pile problem presented in this paper have been based on the total moment about the bottom of the pile and the moment distribution over the length of the pile. In order to calculate a theoretical moment it is necessary to obtain from the experimental results two empirical coefficients -- a drag coefficient and a mass coefficient (Morison, O'Brien, Johnson and Schaaf, 1950). The theoretical equations of total moment corresponding to the crest, trough, and still-water level positions along the surface wave are used to compute these coefficients from the measured total moments at the same positions. Using these coefficients and the theory, a comparison to experimental results is made by comparing the maximum moments, the phase relationships of maximum moments to the surface wave crest, and comparing the calculated and measured total moment time histories. A comparison of the coefficients obtained by these experiments to other published coefficients obtained in different manners, some being steady-flow values, shows that the results herein are of the right order of magnitude but have considerable variability.* Further investigation of the problems would clarify the reasons for the scatter of the coefficients.

Using the experimentally determined coefficients, the moment distributions on uniform diameter and variable diameter round piles were computed and compared to the measured distributions. The computed results are shown to predict the moment distribution with reasonable accuracy for design purposes.

* Errors occurred in Chapter 28, "Design of Piling" in the Proceedings, First Conference on Coastal Engineering and are corrected in the Appendix of this Chapter.

The effects of size, shape and spacing of piles were obtained experimentally. Sheltering and mutual interference effects were found for piles arranged in rows or columns. Results are presented in comparative form as moment ratios with respect to a single cylindrical pile. Center piles in rows of piles aligned parallel to the wave crests showed maximum moments that were higher than those for a single isolated pile. The moment depended upon the relative clearances. Moments on piles arranged in columns parallel to the direction of the wave travel showed a sheltering effect on the center piles in the columns with moments less than those for a single isolated pile.

Moments on piles such as an H - section and a flat plate section were larger than those for cylindrical piles of the same projected area.

THEORETICAL CONSIDERATIONS

The dynamic force on an object in fluid moving with a steady-state velocity relative to the object is given by the expression

$$F = \frac{1}{2} C_D \rho A u^2 \qquad (1)$$

where

C_D = coefficient of drag.
ρ = fluid density.
A = projected area of object perpendicular to the velocity.
u = undisturbed fluid velocity relative to the object.

The coefficient of drag must be determined experimentally. It includes the dynamic effects of frictional drag and of form drag resulting from the disturbance of the fluid in the vicinity of the body.

In steady state fluid flow the drag coefficient is related to the flow by the Reynolds number given by the expression

$$R_e = \frac{Du}{\gamma} \qquad (2)$$

where

D = characteristic length of the object.
γ = kinematic viscosity of the fluid.

When the fluid is in non-steady motion past an object, the acceleration or deceleration of the fluid in the vicinity of the object produces a force component. Adding this force due to the fluid inertia to the frictional force, the total force is given by the expression (O'Brien and Morison, 1950),

$$F = \frac{1}{2} C_D \rho A u^2 + C_M \rho V_m \frac{du}{dt} \qquad (3)$$

where

C_M = coefficient of mass.
V_m = volume of the displaced fluid.
$\frac{du}{dt}$ = acceleration of the fluid relative to the object.

The coefficient of mass must be determined experimentally. This total force does not include any hydrostatic forces. The system under consideration is essentially in a balanced hydrostatic field.

A pile, extending vertically in a fluid in motion due to oscillatory waves, is in a non-uniform flow field with respect to time and to the submerged pile length. Consider a pile at any instant of time. Equation (3) must be written in the differential form and integrated over the pile length in order to obtain the total resultant force on the pile. In Equation (3) the area A is D dS and the displaced volume V_m is $(\pi D^2/4)$ dS. Thus, the differential force on the pile is given by the expression

$$dF = \left(\frac{1}{2} C_D \rho D u^2 + C_M \rho \frac{\pi D^2}{4} \frac{du}{dt}\right) dS \qquad (4)$$

where

D = pile diameter.
S = distance above the bottom into fluid.

Equation (4) may be integrated if C_D, C_M, and u, and du/dt are known as functions of time (t), or the phase angle, and of the position S. Taking $S = (d + y + \eta)$ where d = depth of still water, y = depth below the mean water surface to the mean particle position (measured negatively downward), and η = vertical particle displacement about the mean position, and assuming that the horizontal particle velocity is zero when $\eta = 0$, then the horizontal velocity and acceleration of the fluid in wave action are given by the expressions (Stokes, 1901),

$$u = \frac{\pi H}{T} \frac{\cosh \frac{2\pi S}{L}}{\sinh \frac{2\pi d}{L}} \cos\theta \qquad (5)$$

and

$$\frac{du}{dt} = \frac{2\pi^2 H}{T^2} \frac{\cosh \frac{2\pi S}{L}}{\sinh \frac{2\pi d}{L}} \sin\theta \qquad (6)$$

where

H = wave height.
T = wave period.
L = wave length
θ = $2\pi t/T$, angular position of particle in its orbit measured counter-clockwise from the crest position at time t = 0.

The coefficients C_D and C_M depend upon the state of the fluid motion with respect to the object motion. Little is known about either of the coefficients in accelerated systems. As a first approximation they are considered as constant with respect to time and position to enable integration of Equation (4). Thus, C_D and C_M become overall coefficients.

This study is based on the total moment about the bottom of the pile, or the total moment contributed by the wave motion above any level, S_i, above the bottom. This moment is given by the expression

$$M_i = \int_{S_i}^{S_s} (S - S_i)\, dF \qquad (7)$$

In order to simplify the calculations of the first few experiments made, it was assumed that the wave elevation above or below mean water level contributed little to the total moment about the bottom; that is, η at the surface was small compared to d. Hence in Equation (7) the wave surface S_s is reduced to d and $S = d + y$. By making the necessary substitutions into Equations (4) and (7) and integrating, we have

$$F_i = \pi\rho\,\frac{D\,H^2 L}{T^2}\left\{\pm C_D\,k_1\,\mathrm{Cos}^2\,\theta + C_M k_2\,\frac{\pi D}{4H}\,\mathrm{Sin}\,\theta\right\} \tag{8}$$

$$M_i = \rho\,\frac{D\,H^2\,L^2}{T^2}\left\{C_D\,k_3\,\mathrm{Cos}^2\theta + C_M k_4\,\frac{\pi D}{4\,H}\,\mathrm{Sin}\,\theta - \frac{2\pi S_i}{L}\left[C_D\,\frac{k_1}{2}\,\mathrm{Cos}^2\,\theta + \frac{k_2}{2}\,\frac{\pi D}{4\,H}\,\mathrm{Sin}\,\theta\right]\right\} \tag{9}$$

The line of action of the resultant total thrust, F_i, above the level, S_i is given by the expression

$$\overline{S} = \frac{M_i}{F_i} \tag{10}$$

where

$$k_1 = \frac{\frac{4\pi d}{L} - \frac{4\pi S_i}{L} + \mathrm{Sinh}\,\frac{4\pi d}{L} - \mathrm{Sinh}\,\frac{4\pi S_i}{L}}{16\left(\mathrm{Sinh}\,\frac{2\pi d}{L}\right)^2} \tag{11}$$

$$k_2 = \frac{\mathrm{Sinh}\,\frac{2\pi d}{L} - \mathrm{Sinh}\,\frac{2\pi S_i}{L}}{\mathrm{Sinh}\,\frac{2\pi d}{L}} \tag{12}$$

$$k_3 = \frac{\frac{1}{2}\left(\frac{4\pi d}{L}\right)^2 - \frac{1}{2}\left(\frac{4\pi S_i}{L}\right)^2 + \frac{4\pi d}{L}\,\mathrm{Sinh}\,\frac{4\pi d}{L} - \frac{4\pi S_i}{L}\,\mathrm{Sinh}\,\frac{4\pi S_i}{L} - \mathrm{Cosh}\,\frac{4\pi d}{L} + \mathrm{Cosh}\,\frac{4\pi S_i}{L}}{64\left(\mathrm{Sinh}\,\frac{2\pi d}{L}\right)^2} \tag{13}$$

$$k_4 = \frac{\frac{2\pi d}{L}\,\mathrm{Sinh}\,\frac{2\pi d}{L} - \frac{2\pi S_i}{L}\,\mathrm{Sinh}\,\frac{2\pi S_i}{L} - \mathrm{Cosh}\,\frac{2\pi d}{L} + \mathrm{Cosh}\,\frac{2\pi S_i}{L}}{2\,\mathrm{Sinh}\,\frac{2\pi d}{L}} \tag{14}$$

Equation (9) for the total moment contains sine and cosine terms which are functions of the angular position, θ. Thus, a phase angle is indicated which depends upon the relative magnitude of the sine and cosine terms. The wave equations (5) and (6) are referenced at a wave crest at time $t = 0$. The phase angle, β, of the maximum moment in relationship to the wave crest is determined by differentiating Equation (9) with respect to θ and setting the results equal to zero; thus,

$$\beta = \mathrm{Sin}^{-1}\left\{\frac{\pi D\,C_M\left(k_4 - \frac{2\pi S_i}{L}\,\frac{k_2}{2}\right)}{8\,H\,C_D\left(k_3 - \frac{2\pi S_i}{L}\,\frac{k_1}{2}\right)}\right\} \tag{15}$$

The phase angle of Equation (15) shows that the maximum moment usually does not occur when a wave crest passes a pile. When the pile is in water which is shallow compared to the wave length (d/L small), the phase angle approaches zero. When the pile diameter is small compared to the wave height (D/H small) the phase angle also approaches zero. The phase angle approaches 90° for piles in deep water (d/L large) or for large piles in small waves (D/H large).

Measured moment-time histories on the pile and wave surface-time histories at the pile are used to determine C_D and C_M from Equation (9). Two variables are involved which necessitate selection of two times with the corresponding two moments. The solution is simplified if the selected times are zero (crest or trough at the pile) and the one-quarter or three-quarter wave length time (surface profile at the mean water level). These times result in Sin θ = 0, and Cos θ = 0, respectively. Thus, the selected points reduce Equation (9) to two equations, each with but one unknown, C_D and C_M, respectively.

The moment distribution on a non-uniform pile, that is a pile which consists of various lengths of different diameters (Fig. 1) results from a summation of the moments contributed by each section. The solution of Equation (9) for this system is given by the expression,

$$M_n = \sum_{j,i} \rho \frac{D_j H^2 L^2}{T^2} \left\{ C_D\, k_{3_i} \cos^2\theta + \frac{\pi D_j}{4H} C_M\, k_{4_i} \sin\theta - \frac{2\pi S_n}{L} \right.$$

$$\left. \left[C_D \frac{k_{1_i}}{2} \cos^2\theta + \frac{\pi D_j}{4H} C_M \frac{k_{2_i}}{2} \sin\theta \right] \right\} \tag{16}$$

where

$S_n = S_2, S_3, S_4, \text{---}\, S_{11}$, the elevation at which the total moment is calculated.

$D_j = D_1, D_2, - - - D_5$, diameter of pile of various sections

k_{1_i} etc. $= k_{1_1}, k_{1_2}, - - k_{1_n}, k_{2_1}, k_{2_2} - - k_{2_n}$ the elevation above the bottom is being summed.

The conditions imposed upon Equation (16) in order to perform the summation resulting effect on Equations (11) to (14) are summarized for the conditions illustrated in Figure 1 as follows:

1. If $j = 1$ then $i = 2, 3$
   $j = 2$ $i = 4, 5$
   $j = 3$ $i = 6, 7$
   $j = 4$ $i = 8, 9$
   $j = 5$ $i = 10, 11$

2. For any $n = 2, - - -, 11$, the summation is carried out for successive values of j and the corresponding values of i until $i = n$. (See Fig. 1)

3. The expressions for k_{1i}, k_{2i}, k_{3i}, k_{4i} are the same as k_1, k_2, k_3, k_4 (Equations 11 to 14) where d in the numerator is changed to S_{i-1}. For example, Equation (11) becomes

$$k_{1_i} = \frac{\frac{4\pi S_{i-1}}{L} - \frac{4\pi S_i}{L} + \text{Sinh}\frac{2\pi S_{i-1}}{L} - \text{Sinh}\frac{4\pi S_i}{L}}{16\left(\text{Sinh}\frac{2\pi d}{L}\right)^2}$$

The calculation of moments on piles in shallow water must include the effect of the variation of the lever arm between the crest and trough of the wave. The equation for the total moment about any level S_i is the same as Equation (9) where the expression of k_1, k_2, k_3, and k_4 (Equations 11, 12, 13 and 14) have S_s in the numerator instead of d. For example Equation (11) becomes

$$k_1 = \frac{\frac{4\pi S_s}{L} - \frac{4\pi S_i}{L} + \text{Sinh}\frac{4\pi S_s}{L} - \text{Sinh}\frac{4\pi S_i}{L}}{16\left(\text{Sinh}\frac{2\pi d}{L}\right)^2}$$

where S_s is the elevation to the water surface above the bottom.

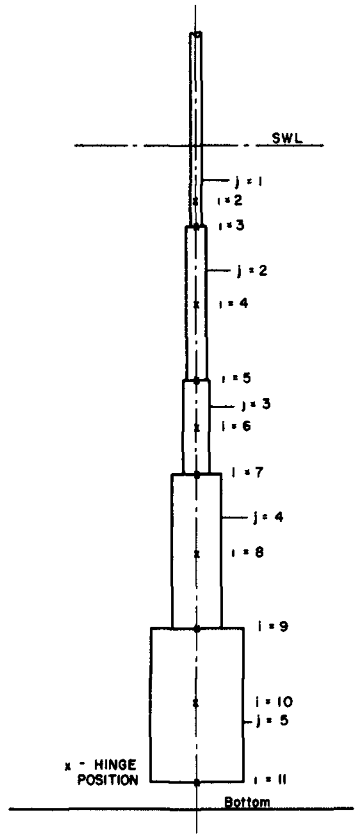


Fig. 1

The calculation of an explicit expression for the phase angle, similar to Equation (15) when considering the change in surface elevation is impossible so that it becomes necessary to plot a graph of equations or use approximate methods to obtain the phase angle of the total moment with respect to the wave crest (See Fig. 10).

In order to evaluate the total moment exerted on a pile subjected to a known wave condition, the coefficients C_D and C_M must be known. Measurements of the moment time history of piles subject to known wave conditions enable evaluation of C_D and C_M. The established coefficients then can be used to predict moments on piling for any pile and imposed wave conditions subject to the limitations and approximations of the analysis which leads to Equations (8), (9) and (15).

The drag coefficient, C_D, in Equations (8), (9) and (15) is comparable in significance to the steady state drag coefficient of Equation (1). Thus, comparisons may be made between the drag coefficients which result from measurements on piling subject to the periodic motion of wave action and those reported in the literature for the same geometrical systems in a steady state fluid stream. The steady state drag coefficients are functions of the Reynolds number, Equation (2), in addition to the geometrical shape. In periodic motion the Reynolds No. varies from zero to a maximum. The maximum influence of the motion of a wave past a pile occurs near the surface in the regions of the highest velocity. Hence, the crest particle velocity is assumed to be most nearly representative of the velocity to be used in the Reynolds number. This results from Equation (5) with $S = d$ and $\theta = 0$.

EXPERIMENTAL INVESTIGATIONS ON MODEL PILES

Experiments were designed to measure the moment history on piles of constant and variable diameter about hinge points in the piles when subjected to wave action. The wave shape was measured simultaneously to determine the height, velocity, and period of the wave at the pile. The wave length is related to the velocity and period as follows;

$$L = C\,T \tag{17}$$

From the measurements of the variables, the coefficients C_D and C_M were obtained from Equation (9). Once having determined the coefficient, then evaluation of the moments was possible for a given pile subjected to known wave action.

Experiments were conducted in the wave channel at the University of California (Morison, 1950a, 1950b, 1950c.

<u>Tests on single circular piles</u>: Moments were measured on single piles hinged at the bottom as well as at various elevations, (Fig. 2). In one instance a 1 inch diameter pile was hinged only at the bottom and subjected to a large range of wave conditions. In another series of tests, piles of 1/2, 1 and 2 inches in diameter were subjected to three different wave conditions, and the moments were obtained at hinge points located at various elevations to obtain moment distributions. A summary of test conditions is presented in Table 1, and a summary of the test results is given in Table 2.

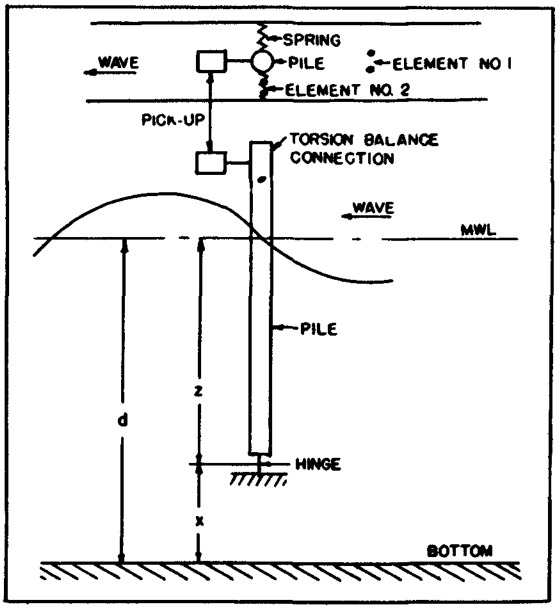


Fig. 2

Table 1.

Summary of test conditions on circular piles

Pile No.	D inches	d ft.	Wave Characteristics T-sec.	H-ft.	L-ft.	Remarks
1	1	2	(variable		)	Moments measured only at the bottom
2	½	1.96	0.98	0.184	4.91	Moments measured at 7 elevations
3	1	1.96	0.98	0.179	4.98	Moments measured at 6 elevations
4	2	1.95	0.98	0.186	4.96	Moments measured at 6 elevations

Table 2.

Summary of test results on circular piles

Variable	Pile No. (See Table 1) 1	2	3	4
H/L	0.009 to 0.114	0.038	0.036	0.038
d/L	0.102 to 0.529	0.400	0.393	0.395
d/H	4.81 to 18.15	10.700	10.900	10.500
D/H	0.212 to 0.758	0.226	0.465	0.898
D/L	0.009 to 0.042	0.009	0.017	0.034
D/d	0.041 to 0.083	0.021	0.042	0.085
Re	2,000 to 11,100	2,300	4,500	9,300
C_D	1.6 ± 0.4	2.7	2.5	4.4 ($\beta = 80°$)
C_M	1.5 ± 0.2	1.2	1.8	1.8

Some results were obtained for a pile placed in breaking waves. The departure of actual conditions from the assumed conditions as stated in the development of Equation (9) was too great to justify use of this equation in the interpretation of results in breakers. The results showed maximum moments produced by a breaker or incipient breaker greatly in excess of the forces corresponding to the orbital motion described by Equation (9)

The coefficients as determined for any one wave condition were used with Equation (9) to compute the complete moment history over the cycle from one wave crest to the next. A typical comparison is shown in Fig. 3.

Moment distribution comparisons were made for piles 2, 3, and 4 (Table 1). Equation (9), with values of C_D and C_M from the measured moment history at the bottom of the pile, was used to compute the moment ratio as a function of depth for comparison with the experimental ratio. Results are shown in Fig. 4.

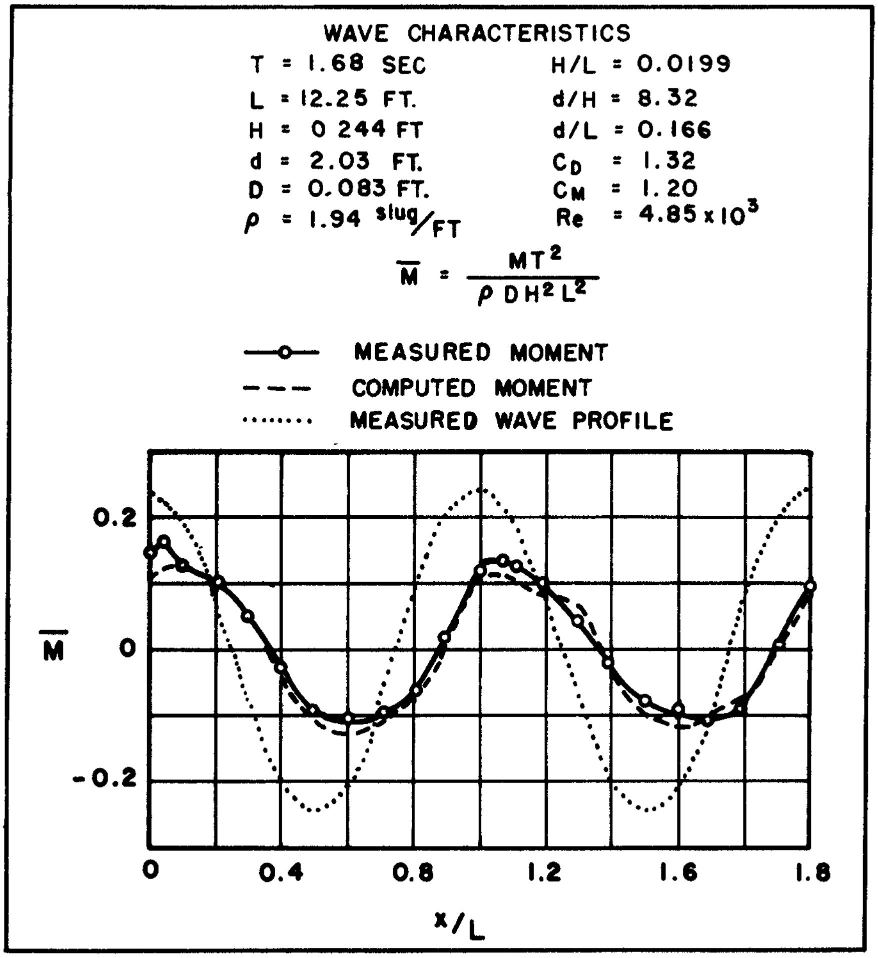


Fig. 3. Total moment about the bottom of a single circular pile.

In Fig. 4 certain features should be noted. The coefficients C_M and C_D were evaluated from the moment history at values of 0, $\pi/2$, π, $(3/2)\pi$ of the angular particle position with respect to the wave crest. Thus, the computed maximum moment may be different from the measured maximum moment for these conditions where the phase angle between the wave crest and maximum moment is different from zero. The computed curves, Fig. 3, show this difference. That is, at $y/d = 1.00$ (bottom), the maximum measured moment and the maximum computed moment do not coincide. However, the shape of the moment distribution as a function of depth, using the average values of C_D and C_M from the measured moment at the bottom to compute the moment at any depth, follows the trend of the measured moment distribution.

A further comparison may be made of the effect of pile diameter on the moment distribution by reducing the moment distribution to a ratio in terms of the maximum moments. Results are shown in Fig. 5 for one wave condition. The computed moment ratio and the experimental moment ratio are in agreement within the limits of experimental error. The pile diameter does not have any influence on the moment distribution. Hence, attention can be concentrated on obtaining moments about one hinge point to establish the necessary criteria to enable prediction of the moments on a pile due to wave action.

Within the accuracy of values of C_D and C_M, the resultant force as a function of time or wave position relative to the pile may be obtained from Equation (8). The action line of the total resultant force is obtained from

$$\overline{S} = \frac{M_d}{F}$$

where $\overline{S}$ is the location of the action line above the bottom and M_d is the moment about a hinge point at the bottom. The resultant force on a pile above a hinge point at any position in the pile may be obtained in a similar manner except for the section of the pile near the water surface.

In these tests forces were not computed, since attention was concentrated on obtaining reliable values of C_D and C_M from moment histories.

<u>Tests on a variable diameter pile:</u> The total moments exerted by waves on a pile which had varied steps of diameters was determined by a model study. The dimensions of the model are shown in Fig. 6. No attempt was made to determine the coefficients, C_D and C_M from the results on the stepped pile.

Three conditions of the stepped pile were investigated with respect to the coefficients C_D and C_M as determined in the discussions above for single cylindrical piles. The moment contributed for each section of the pile was computed from Equation (16) using $C_D = 1.63$, $C_M = 1.51$, and the experimentally measured phase angle, β_d, of the total moment about the bottom. Comparison of the moment distribution in the form of the ratio of the moment resulting from the wave action

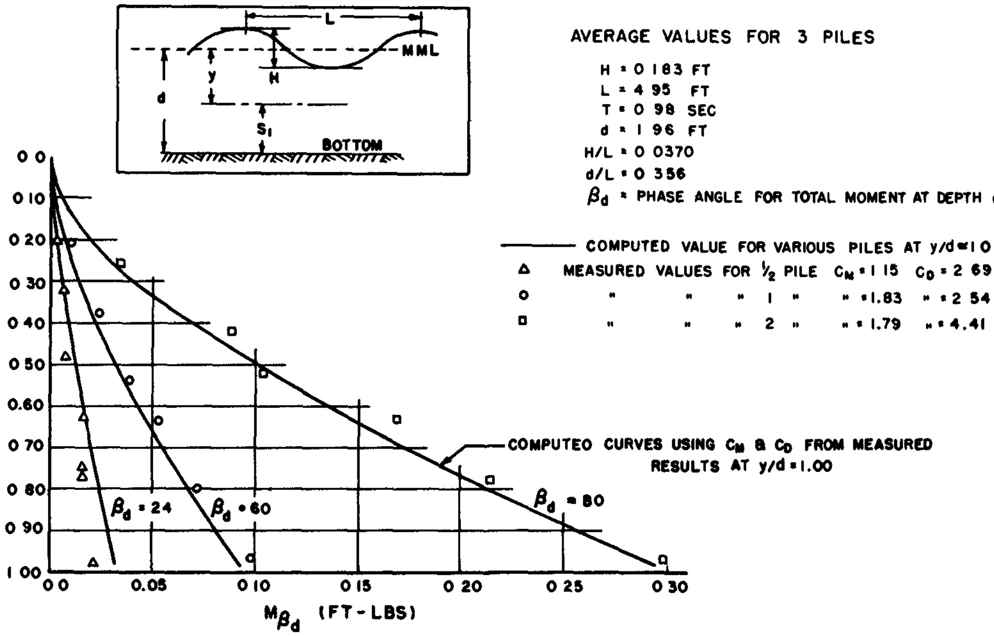


Fig. 4. Moment distribution on uniform pile - Laboratory results.

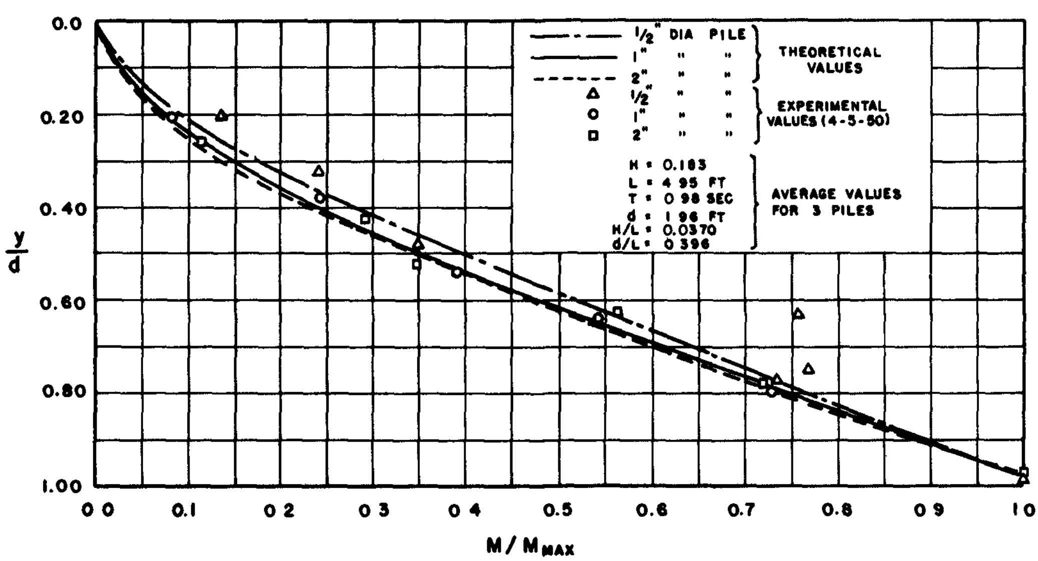


Fig. 5. Dimensionless moment distribution of uniform pile.

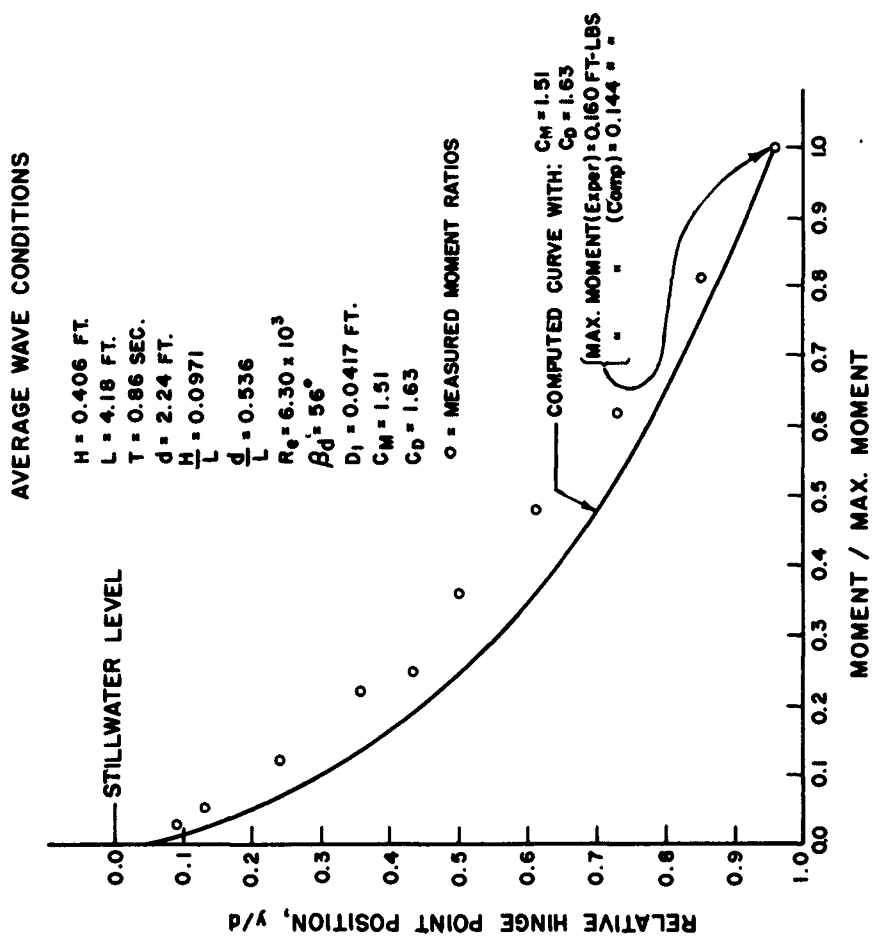


Fig. 7. Comparison of computed and measured moment distribution on a variable diameter pile.

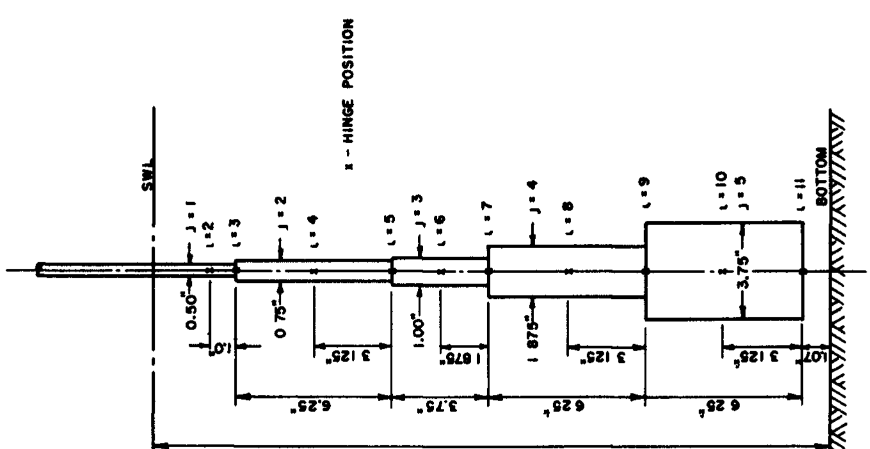


Fig. 6.

above any selected point to the maximum moment about the hinge point at the bottom is shown in Fig. 7.

Tests on piles of various cross-sectional shapes: The moment history of piles with various cross-sectional shapes was determined in the laboratory with the equipment shown in Fig. 1. The pile cross-sections were circular, flat plates and H - sections with one-inch projected width in the normal dimension as detailed in Fig. 8. Results were interpreted as ratios of the maximum moment for any given shape to the maximum moment for the circular shape (Table 4). The H - section was oriented in three different directions as shown in the table. All piles were subjected to the same wave conditions as indicated in Table 3.

Table 3

Wave conditions in tests on circular piles, flat plates and H - sections.

Parameter	Wave 1	Wave 2	Wave 3
H, ft.	0.681	0.342	0.454
L, ft.	7.54	3.87	5.39
T, sec.	1.27	0.88	1.09
d, ft.	1.55	1.50	0.83
H/L	0.0903	0.0884	0.0843
d/L	0.206	0.388	0.154

Table 4

Effect of pile shape on maximum moment.

Pile type and size	Orientation	Ratio: $\frac{\text{Maximum moment for given pile type}}{\text{Maximum moment for circular pile}}$		
1 inch round	→ ○	1.00 (β = 5°)	1.00 (β = 20°)	1.00 (Breaker) (β = 26°)
1 inch H-section	→ H α = 0	1.52 (β = 14°)	2.46 (β = 35°)	2.19 (β = 16°)
1 inch H-section	→ I α = 90°	1.42 (β = 10°)	2.08 (β = 43°)	2.58 (β = 41°)
1 inch H-section	→ α = 45°	2.44 (β = 17°)	3.50 (β = 55°)	2.22 (β = 15°)
1 inch flat plate	→ |	1.28 (β = 4°)	1.17 (β = 37°)	1.37 (β = 9°)

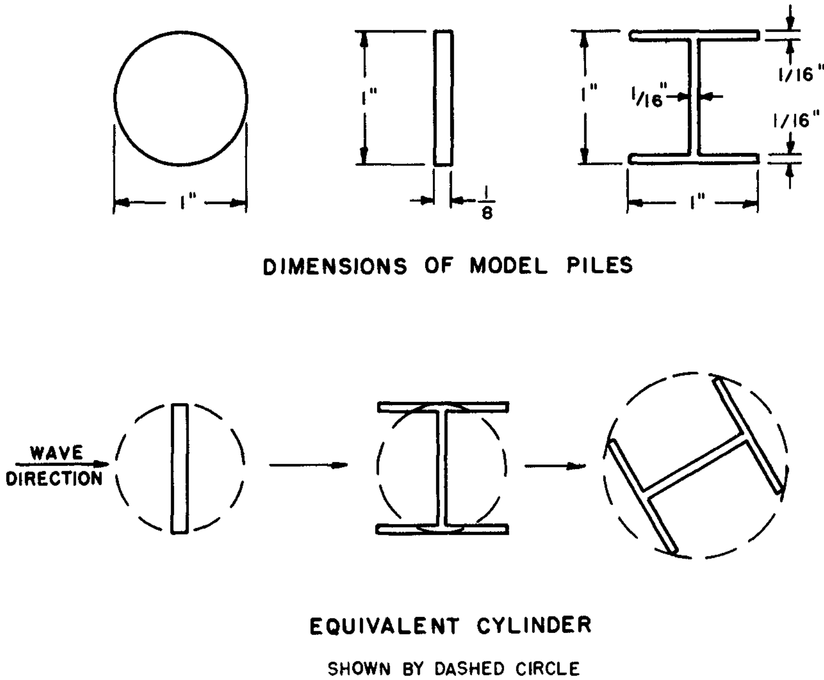


Fig. 8. Cross sections of piles.

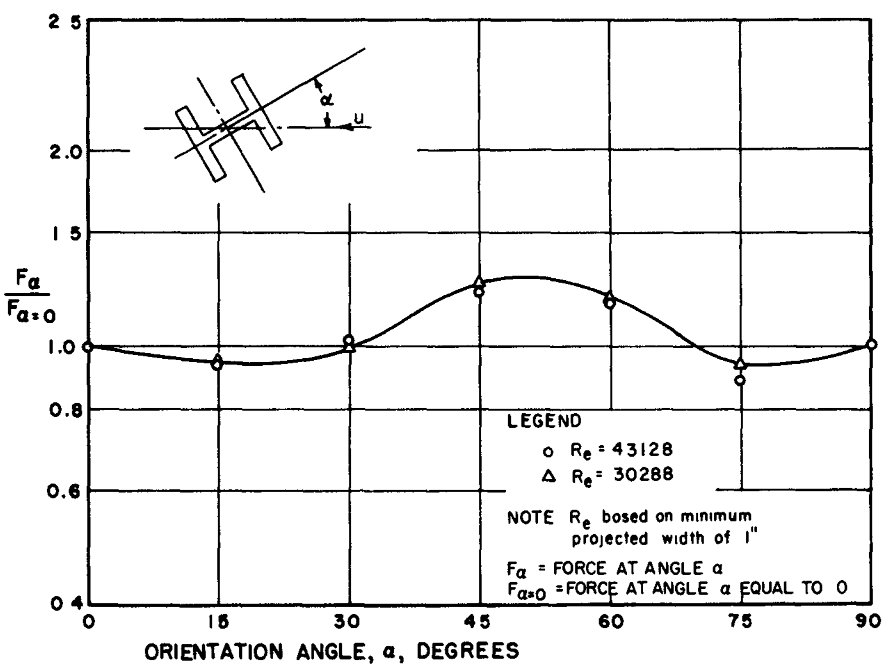


Fig. 9. Measured H-section drag force in steady, uniform flow as a function of orientation.

The force on the H - section was determined in a wind tunnel under steady-state condition as a function of orientation of the section. The maximum force resulted at approximately the 45° orientation as is shown in Fig. 9. Thus the pile results for that orientation were considered as giving the maximum moment (primarily because this orientation gave the greatest projected area); consequently, under wave action the orientation of the H-section was not varied over angles other than the 45° with respect to the direction of wave travel.

One comparison can be made using the H-section results of the steady-state force ratio and the maximum moment ratio in the wave action. Ratios of the maximum moment of the H-section oriented with values of α other than zero to the maximum moment with $\alpha = 0$ may be compared to the corresponding steady-state force ratios. (Note that the moment arm is constant in the comparison, hence moments should be in the same ratio as forces assuming the force distribution is similar and not a function of orientation.)

This comparison is shown in Table 5.

Table 5

Effect of orientation on forces on H-section in steady flow and in oscillatory flow.

Wave Steepness	Orientation of pile		
	→ H $\alpha = 0$	→ I $\alpha = 90°$	→ $\alpha = 45°$
	Ratio: $\frac{\text{Force (or Moment) at orientation shown}}{\text{Force (or Moment) at } \alpha = 0}$		
0.0903	1.00	0.93	1.61
0.0884	1.00	0.85	1.42
0.0843 (Breaker)	1.00	1.17	1.02
Steady Flow	1.00	1.00	1.26

Differences between force ratios in steady state and in oscillatory flow are noted in some cases which are greater than any experimental error. Thus, the steady-state drag forces (hence steady-state drag coefficients) are not the complete criteria by which to evaluate moments of sections which differ from the circular section. This comparison would indicate the presence of the inertia force component, a fact which is confirmed by the differences in phase angles listed in Table 4.

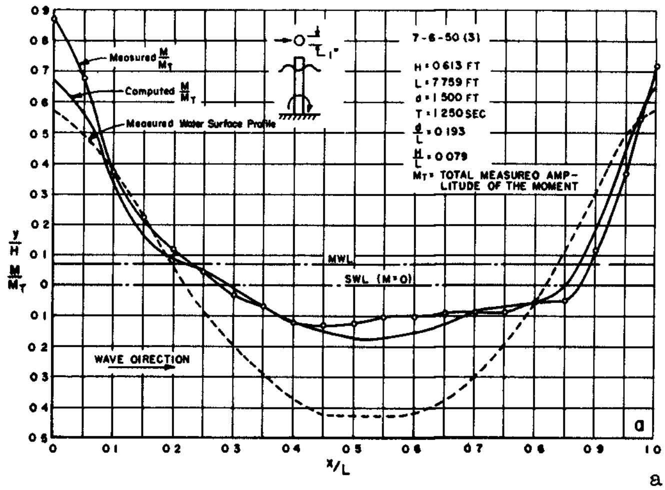
The plots shown in Fig. 10 are computed, and measured moment-time histories of a circular, an H-section and a flat plate pile in shallow water where the effect of the variable lever arm has been considered by using S_s instead of d in Equations 11, 12, 13 and 14. The coefficients of drag and mass computed from the measured curve are given in Table 6 along with the wave characteristics.

Table 6

Coefficients of drag and mass for shallow water waves

Variable	Pile type		
	1 inch circular	1 inch H-section → H	1 inch flat plate → I
H, feet	0.613	0.600	0.705
L, feet	7.76	7.36	8.00
T, sec.	1.25	1.27	1.27
d, feet	1.50	1.46	1.45
H/L	0.079	0.082	0.088
d/L	0.193	0.198	0.181
β, degrees	5	14	0
R_e	15,000	15,000	15,000
C_D	1.78	2.44	1.20
C_M	0.44	1.92	0.42

One feature of the interpretation of the equations from which the coefficients of mass and drag were computed is evident in the results shown in Table 6. When the phase angle is small, the mass coefficient is evaluated from moments which are near the point of zero moment. Small experimental errors become significant and reduce the reliability of the value of the mass coefficient. The mass coefficients for the circular pile and the flat plate pile are small as compared to those reported in Table 2. These low coefficients are not representative.



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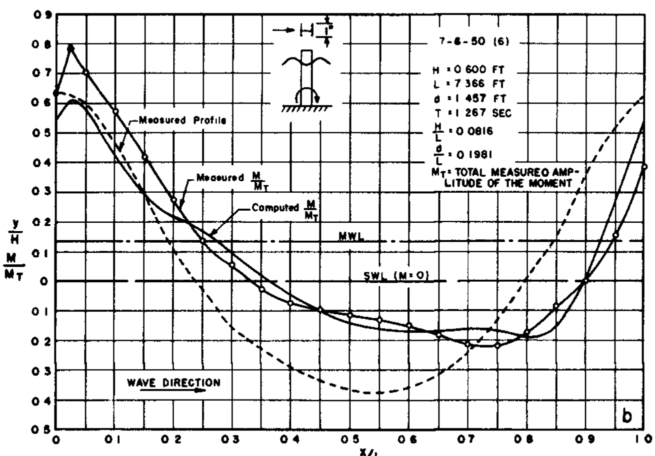

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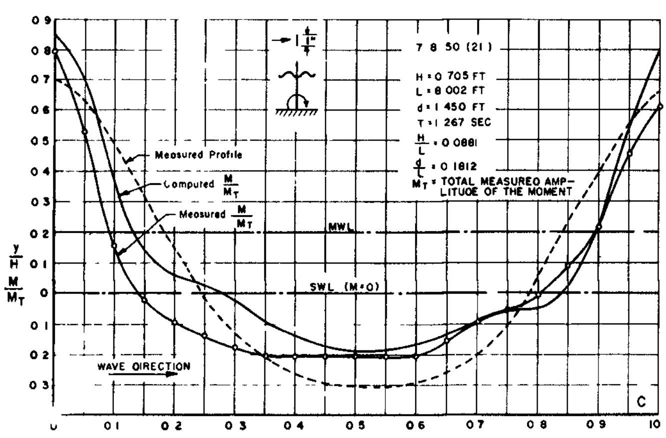

c.

Fig. 10. Computed and measured time history of total moment on circular, H-section, and flat plate piling.

Effect of mutual interference of piles: The one-inch circular and flat plate piles were arranged in rows parallel to the wave direction and in columns perpendicular to the wave direction (see Fig. 11). Three piles were used in each case with moment measurements made on the center pile (Fig. 12). Spacings between the piles were ½D, D and 1½D, where D is the pile diameter. Results are shown in Table 7. The ratio of the maximum moment on the center pile of the column or row to the maximum moment on a single pile shows the results of interference effects. The wave conditions used were the same as listed in Table 3.

Table 7

Effect of mutual interference of piling

Wave Steepness, H/L (See Table 3)	Ratio: $\frac{\text{Moment on center pile}}{\text{Moment on single pile}}$		
	Pile Gap*		
	½ D	D	1½ D
	Row of circular pile perpendicular to wave travel		
0.0903	---	1.42	1.04
0.0884	2.43	0.90	0.94
0.0843 (Breaker)	1.69	1.14	1.23
	Row of flat plates perpendicular to wave travel		
0.0903	1.49	1.46	1.54
0.0884	1.93	1.40	1.17
0.0843 (Breaker)	2.22	1.72	1.31
	Column of circular pile parallel to wave travel		
0.0903	0.39	0.71	0.72
0.0884	0.60	0.71	0.74
0.0843 (Breaker)	0.95	0.75	0.87

* D = 1 inch for all piles.

The results show that, at spacings of less than 1½ D in the row arrangement, interference effects are noticeable. Higher moments are experienced by the center pile as contrasted to a single pile. The blocking effect of adjacent piles increases the force and resulting moment on an individual pile. The blocking effect decreases as the spacing between piles increases. For the limited range of the tests, the blocking effect is concluded to be negligible for spacings of 1½ D or greater.

Results from the piling arranged in columns show a sheltering effect, (Table 7) in that moments were less than those represented by a single pile. The maximum spacing at which the sheltering effect is negligible was not reached in these tests.

Forces on cross members: The measurement of the horizontal force on cross-members was made on a force balance apparatus. The cross-member was mounted on a rod which was pivoted near its center and restrained by calibrated springs at one end (Fig. 13). The submerged part of the rod was shielded from the wave action so that a tare test, without the cross-member attached, showed only about one-percent deflection. The force and the wave characteristics were recorded in the same manner as in the case of the single piles. Three lengths ($2\frac{1}{2}$, 5 and 10 inches) of cross-members were used so that the end effects could be determined.

The measurement of the vertical force on cross-members was made directly by a calibrated spring system. The cross-member was placed at the end of a vertical rod that was attached to springs (Fig. 14). The submerged part of the rod was shielded and held in guides near the cross-member. A tare test showed less than one-percent deflection. The wave characteristics were measured $1\frac{1}{2}$ feet in front and $1\frac{1}{2}$ feet behind the cross-member with a reference measurement of the wave crest being made directly above the cross-member. The force and wave characteristics were recorded simultaneously on the same oscillograph record. The same wave conditions were reproduced as those used for the measurement of the horizontal forces on the cross-members. In both the tests of the horizontal and of the vertical forces, the same wave conditions were used for the horizontal and inclined members at the 1/3 and 2/3 positions of water depth.

The horizontal force per unit length on a cross-member (Tables 8 and 9) indicated that the orientation of the cross-member is not critical for model studies. The test showed also that the end effects are not appreciable. The vertical force per unit length on a cross-member (Table 10) indicated some effects due to orientation. The magnitudes of the forces were about half those for the horizontal direction.

FIELD PILE TESTS

The model tests, as described above, yielded a considerable amount of information on the moments and forces on piles subjected to a wide range of wave conditions and depths of immersion. The limited size of the model system introduces a possible scale effect in the direct application of the model results to predict prototype behavior. Thus, prototype tests were made in an attempt to correlate model and prototype behavior to substantiate the analysis and results from the model tests (Snodgrass, Rice, and Hall, 1951).

The field tests were conducted near shore at Monterey, California, with a cylindrical pile of $3\frac{1}{2}$ inch outside diameter. The pile was hinged at the bottom at approximately sand level. Restraining bars at the top of the pile were arranged with strain gage elements connected to recording equipment. The strain records yielded the force-time history of the pile under the action of the incident waves. Calibrations of the strain recording equipment were made both in the laboratory and in the field.

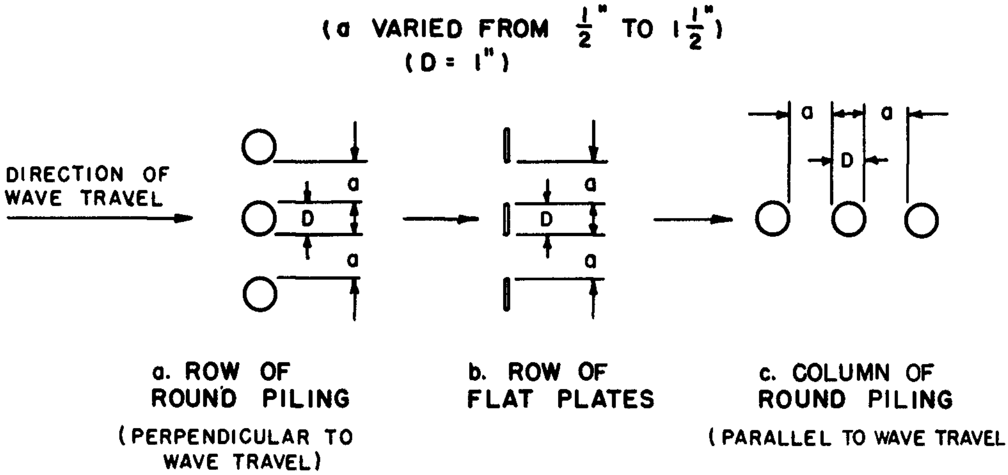

Fig. 11. Arrangement of piling for tests
on mutual interference.

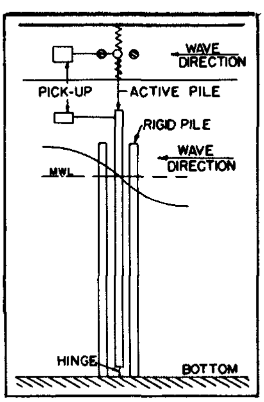

Fig. 12

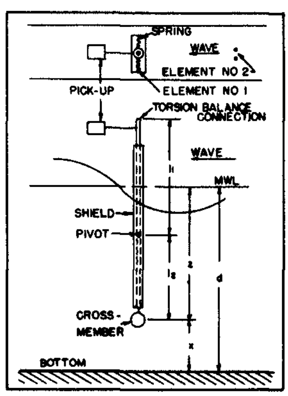

Fig. 13.

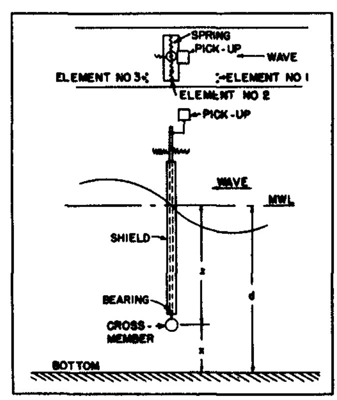

Fig. 14.

Table 8

Horizontal force on cross members.

Code			s ft.	d ft.	s/d	H ft.	L ft.	T sec.	F_{max} lbs.	F_{max} lbs/ft.	β_o deg.	d/L	H/L	C_{M90}	C_{M270}	C_{D_c}	C_{D_t}	R_e	I
8-15-50(14)	10	Hor.	1.054	1.543	0.683	0.615	6.172	1.200	0.05566	0.0669	56	0.250	0.0996	1.245	1.494	0.868	1.548		
8-15-50(17)	10	45 Cross	1.040	1.530	0.680	0.612	6.125	1.208	0.06465	0.0776	44	0.250	0.0999						
8-15-50(18)	10		1.054	1.543	0.683	0.622	6.084	1.200	0.05315	0.0638	84	0.254	0.1022						
8-15-50(19)	10		1.057	1.547	0.683	0.621	6.260	1.200	0.05818	0.0698	74	0.247	0.0992						
8-15-50(5)	5	Hor.	1.055	1.545	0.683	0.620	6.378	1.196	0.02345	0.0563	50	0.242	0.0972	1.103	1.163	0.776	1.505		
8-15-50(9)	5	45 Cross	1.050	1.540	0.682	0.602	6.172	1.200	0.02266	0.0544	53	0.250	0.0975						
8-15-50(10)	5		1.036	1.525	0.679	0.600	6.396	1.208	0.02328	0.0559	64	0.238	0.0938						
6-15-50(11)	5		1.052	1.543	0.682	0.606	6.172	1.200	0.02739	0.0657	90	0.250	0.0980						
6-15-50(23)	2½	Hor.	1.055	1.545	0.683	0.601	6.216	1.200	0.00858	0.0412	62	0.249	0.0967	0.892	1.486	0.588	1.832		
8-15-50(26)	2½	45 Cross	1.039	1.528	0.680	0.605	6.172	1.200	0.00925	0.0444	60	0.248	0.0980						
6-15-50(28)	2½		1.039	1.528	0.680	0.636	6.353	1.200	0.01189	0.0571	60	0.241	0.1001						
8-15-50(27)	2½		1.039	1.528	0.680	0.602	6.353	1.200	0.00925	0.0444	76	0.241	0.0948						
8-15-50(31)	10	Hor.	0.550	1.528	0.360	0.613	6.353	1.200	0.07008	0.0841	6	0.241	0.0965	0.618	0.847	0.633	1.061		
8-15-50(34)	10	45 Cross	0.570	1.549	0.368	0.612	6.172	1.200	0.08586	0.1030	6	0.251	0.0992						
8-15-50(35)	10		0.555	1.534	0.362	0.608	6.353	1.200	0.06186	0.0742	57	0.241	0.0957						
8-15-50(36)	10		0.570	1.550	0.368	0.620	6.171	1.200	0.09140	0.1097	86	0.251	0.1005						
8-15-50(39)	5	Hor.	0.570	1.550	0.368	0.618	6.259	1.183	0.04296	0.1031	30	0.248	0.0987	1.156	0.939	0.910	1.046		
8-15-50(42)	5	45 Cross	0.565	1.544	0.366	0.633	6.349	1.200	0.04160	0.0998	22	0.243	0.0997						
8-15-50(43)	5		0.560	1.539	0.364	0.600	6.548	1.183	0.03850	0.0924	46	0.235	0.0916						
8-15-50(44)	5		0.582	1.561	0.373	0.629	6.545	1.200	0.03550	0.0852	69	0.239	0.0961						
6-15-50(47)	2½	Hor.	0.545	1.524	0.358	0.630	6.548	1.183	0.01980	0.0950	43	0.233	0.0962	0.982	0.488	0.723	1.049		
6-15-50(50)	2½	45 Cross	0.545	1.524	0.358	0.616	6.451	1.200	0.02980	0.1430	33	0.236	0.0955						
6-15-50(51)	2½		0.550	1.529	0.360	0.600	6.451	1.200	0.03450	0.1656	49	0.237	0.0930						
6-15-50(52)	2½		0.564	1.543	0.366	0.636	6.260	1.200	0.02540	0.1219	62	0.246	0.1016						
Average			1.047 0.561	1.538	0.681 0.365	0.614	6.305	1.198				0.244	0.0974					13,580	3.68

Table 9

Horizontal force on cross members.

Code		s ft.	d ft.	s/d	H ft.	L ft.	T sec.	F_{max} lbs.	F_{max} lbs/ft.	β deg.	d/L	H/L	$C_{M_{90}}$	$C_{M_{270}}$	C_{D_c}	C_{D_t}	R_e	I
8-16-50(8)	10" Hor.	0.518	1.503	0.345	0.324	3.204	0.783	0.03946	0.0474	53	0.469	0.1011	0.870	1.232	1.429	2.223		
8-18-50(9)	10" 45° Cross	0.519	1.503	0.345	0.340	3.204	0.783	0.03820	0.0458	44	0.469	0.1061						
8-16-50(10)	10"	0.514	1.499	0.343	0.337	3.204	0.783	0.04680	0.0562	90	0.468	0.1052						
8-16-50(11)	10"	0.507	1.492	0.340	0.327	3.241	0.783	0.05047	0.0606	90	0.460	0.1009						
8-16-50(14)	5" Hor.	0.500	1.485	0.337	0.331	3.167	0.775	0.01881	0.0451	53	0.469	0.1045	0.828	1.034	1.296	1.512		
8-16-50(17)	5" 45° Cross	0.500	1.485	0.337	0.317	3.314	0.792	0.02181	0.0523	61	0.448	0.0957						
8-16-50(1[illegible])	5"	0.501	1.485	0.337	0.317	3.314	0.792	0.02113	0.0507	77	0.448	0.0957						
8-18-50(18)	5"	0.504	1.489	0.338	0.317	3.276	0.792	0.02317	0.0556	90	0.455	0.0968						
8-18-50(22)	2½" Hor.	0.526	1.510	0.348	0.340	3.241	0.783	0.00788	0.0378	71	0.468	0.1049	0.818	0.680	0.846	1.550		
8-16-50(25)	2½" 45° Cross	0.512	1.496	0.342	0.330	3.207	0.775	0.01072	0.0515	72	0.466	0.1029						
8-16-50(28)	2½"	0.508	1.493	0.340	0.326	3.168	0.783	0.00946	0.0454	79	0.471	0.1029						
8-16-50(27)	2½"	0.500	1.485	0.337	0.339	3.204	0.783	0.01072	0.0515	90	0.463	0.1058						
8-16-50(30)	10" Hor.	1.000	1.493	0.670	0.350	3.277	0.783	0.01935	0.0232	90	0.456	0.1068	1.153	1.121	1.701	2.410		
8-16-50(32)	10" 45° Cross	0.998	1.493	0.668	0.343	3.275	0.790	0.02279	0.0273	90	0.456	0.1047						
8-16-50(33)	10"	1.001	1.496	0.669	0.334	3.178	0.750	0.01882	0.0223	83	0.471	0.1051						
8-18-50(34)	10"	1.001	1.496	0.668	0.334	3.278	0.783	0.01779	0.0213	90	0.457	0.1020						
8-16-50(37)	5" Hor.	0.991	1.486	0.867	0.305	3.204	0.783	0.00773	0.0186	88	0.464	0.0952	1.091	1.133	2.640	3.522		
8-16-50(40)	5" 45° Cross	0.999	1.494	0.669	0.350	3.069	0.750	0.00950	0.0228	83	0.487	0.1140						
8-18-50(41)	5"	0.992	1.487	0.667	0.337	3.205	0.783	0.00920	0.0221	83	0.464	0.1051						
8-16-50(42)	5"	0.999	1.494	0.669	0.350	3.133	0.783	0.00690	0.0166	90	0.477	0.1117						
8-18-50(45)	2½" Hor.	0.999	1.494	0.669	0.315	3.204	0.783	0.00205	0.0098	88	0.466	0.0983	0.568	0.649	0.834	1.669		
8-16-50(48)	2½" 45° Cross	1.004	1.499	0.670	0.312	3.209	0.767	0.00480	0.0230	80	0.467	0.0972						
8-16-50(49)	2½"	0.987	1.482	0.666	0.323	3.139	0.767	0.00440	0.0211	90	0.472	0.1029						
8-16-50(50)	2½"	0.994	1.489	0.668	0.326	3.139	0.767	0.00560	0.0269	88	0.474	0.1039						
Average		0.509	1.493	0.341	0.330	3.211	0.779				0.465	0.1028					10,290	1.98
		0.997		0.668														

Table 10

Vertical force on cross members

Code		s ft.	d ft.	s/d	H ft.	L ft.	T sec.	F_{max} lbs.	F_m lbs/ft.	β_c deg.	d/L	H/L	C_{M_c}	C_{M_t}	$C_{D_{90}}$	$C_{D_{270}}$	R_e	I
9-7-50(3)	5 Hor.	1.063	1.537	0.692	0.568	6.452	1.183	0.01567	0.0376	70	0.238	0.0880	0.467	1.649	3.954	3.051		
9-7-50(6)	5 45° Cross	1.010	1.484	0.681	0.521	7.002	1.167	0.01098	0.0262	66	0.212	0.0744						
9-7-50(8)		1.045	1.519	0.688	0.520	6.357	1.183	0.01518	0.0364	118	0.239	0.0818						
9-7-50(7)		1.072	1.546	0.693	0.545	6.360	1.200	0.02125	0.0510	110	0.243	0.0857						
9-7-50(11)	5 Hor.	0.537	1.527	0.352	0.540	6.452	1.183	0.03053	0.0733	98	0.237	0.0837	0.0686	1.309	1.425	0.969		
9-7-50(14)	5 45° Cross	0.524	1.514	0.342	0.509	6.878	1.183	0.02856	0.0686	70	0.219	0.0740						
9-7-50(16)	5	0.547	1.537	0.356	0.525	6.755	1.200	0.03116	0.0748	104	0.228	0.0777						
9-7-50(15)	5	0.539	1.529	0.353	0.559	5.652	1.183	0.03765	0.0904	98	0.230	0.0840						
Average		1.048	1.524	0.688	0.536	6.614	1.185				0.230	0.0810					12,210	3.22
		0.537		0.352														
9-7-50(20)	5 Hor.	0.564	1.554	0.363	0.325	3.138	0.758	0.02133	0.0512	110	0.495	0.1036	1.554	0.225	1.903	4.469		
9-7-50(24)	5 45° Cross	0.509	1.499	0.340	0.322	3.247	0.767	0.01476	0.0354	107	0.462	0.0992						
9-7-50(23)		0.509	1.499	0.340	0.306	3.102	0.758	0.02276	0.0546	124	0.483	0.0986						
9-7-50(25)		0.499	1.489	0.335	0.310	3.209	0.767	0.02860	0.0686	130	0.464	0.0966						
Average		0.520	1.510	0.344	0.316	3.174	0.763				0.476	0.0996					10,050	1.90

The wave height history was obtained from a recording pressure actuated diaphragm type wave meter which was located approximately two feet above the sand bottom and adjacent to the pile. Two auxiliary graduated piles were placed seaward of the measuring pile. The measuring pile and bracing structure also were painted with alternate black and white bands, each one foot high. Motion pictures taken from the beach recorded the surface profile of the waves as they passed the pile. A clock was suspended in the field of view of the camera to provide timing intervals between successive frames of the film. The wave velocity at the pile was obtained from the distance between the seaward auxiliary pile and the measuring pile (19.8 feet), and the time interval of the wave crest travel between these points. The motion pictures also recorded wave heights at the measuring pile. Trough and crest elevations of each wave were obtained from the intersection of the water profile with the graduated vertical piles. The record from the wave meter also gave wave heights and periods.

Analysis of data: The analysis as presented previously in this paper includes the two resistance terms that contain C_D and C_M, and also the phase angle relationship, β, between the two resistance terms. In the analysis of the field pile results, the timing accuracy was not precise enough to determine the time comparison between the water surface profile and the moment history.

The data and results were obtained for waves in various conditions depending on the stage of the tide. Some data were obtained with the pile in a foam line shoreward of the breaker. Other data were obtained with the pile in the smooth unbroken swell seaward of the breaker. The data have been segregated with respect to the wave condition at the pile into the following groups: (1) foam line; (2) foam line immediately shoreward of the breaker point; (3) breaker; (4) sharp peaked swell at incipient break: (5) sharp peaked swell immediately seaward of the breaker point; and (6) swell some distance seaward of the breaker point. The data are summarized in Table 11.

The wave force, which is actually a distributed force extending from the ocean bottom to the water surface, was recorded as an equivalent force at the calibration point. By multiplying the recorded force by the calibration-point lever-arm (9 feet 8 inches) the total moment of the wave force about the bottom hinge was determined. When the maximum force exists (approximately at the time the wave crest passes the pile), the centroid of the wave force was assumed to be located near the mean height of the wave. This location of the centroid was estimated by considering the horizontal component of the particle motion as observed in model studies. By computing the wave force at the mean wave height, as defined above, the data were found to be reasonably consistent. The values obtained from the computation indicate that waves of a given size and shape will exert the same force at the centroid independent of water-level changes over the range encountered in the tests, although the moment about the hinge point varied considerably due to variation of the effective lever arm as the water depth changed. A graph of the wave force at mean wave height is shown in Fig. 15.

Table 11
Test data on field pile

WAVE NO.	WAVE TYPE*	WAVE HEIGHT H Ft.	WAVE PERIOD T Sec.	WAVE VELOCITY (MEASURED) C_f Ft./Sec.	ELEVATION OF CREST ON PILE S_c Ft.	ELEVATION OF TROUGH ON PILE S_t Ft.	STILL WATER LEVEL $d = S_t + \frac{H}{3}$ Ft.	MEASURED FORCE F_r Lbs.	TOTAL MOMENT M Ft-Lbs.	COEFFICIENT OF DRAG C_D
1	FL	4.6	10.7	—	8.0	3.4	5.20	87	562	-
2	FL	4.5	12.1	18.8	8.0	3.5	5.00	57	552	1.05
3	FL	4.2	11.7	17.8	8.0	3.8	4.83	64	625	1.71
4	FL	4.2	9.3	17.5	7.5	3.4	4.80	64	629	1.34
5	FL	4.0	8.3	19.6	7.4	3.4	4.73	34	330	0.63
6	FL	3.8	12.0	17.6	7.0	3.4	4.93	51	496	1.59
7	FL	3.6	7.7	15.8	5.8	2.2	3.40	53	515	1.62
8	FL	3.5	10.3	15.7	6.9	3.4	4.57	33	320	1.46
9	FL-B	4.1	12.1	16.7	7.4	3.3	4.67	48	475	1.91
10	S-FL	4.8	9.3	20.1	8.5	3.7	5.10	51	496	0.59
11	S	5.0	10.1	21.8	8.2	3.2	4.87	55	535	0.49
12	S	4.8	12.2	23.5	8.3	3.5	4.60	57	552	0.53
13	S	3.9	11.2	14.9	5.7	2.8	4.20	32	310	1.05
14	S	3.8	10.5	18.7	7.0	3.1	4.40	27	266	0.73
15	B	3.8	8.4	17.9	6.4	2.8	4.00	34	330	0.91
16	S	3.3	11.0	14.4	5.2	1.9	3.00	28	279	1.31
17	S	3.3	10.3	14.2	5.7	3.4	4.50	23	225	1.28
18	S	3.0	—	—	5.5	3.5	4.50	23	226	-
19	SP-B	3.4	11.0	14.8	5.8	2.4	3.53	17	155	0.85
20	SP-B	3.3	10.8	15.5	7.0	3.7	4.80	19	185	0.88
21	SP-B	3.3	9.0	14.4	6.0	2.7	3.80	13	124	0.64
22	SP-B	2.3	13.1	14.3	5.0	2.7	3.47	3	33	0.39
23	SP-B	2.0	8.5	15.9	4.8	2.8	3.47	4	43	0.57
24	SP	4.5	—	—	7.8	3.5	4.80	5	46	-
25	SP	3.7	—	—	6.2	2.5	3.73	12	113	-
26	SP	3.8	12.1	18.8	7.4	3.8	4.80	17	165	0.44
27	SP	3.5	12.1	20.1	5.9	3.4	4.57	11	109	0.28
28	SP	3.3	10.8	14.8	8.0	2.7	3.70	14	134	0.64
29	SP	3.5	9.1	14.8	5.8	3.3	3.70	8	62	0.26
30	SP	3.1	13.0	17.7	6.8	3.7	4.75	18	175	0.78
31	SP	3.1	10.0	14.2	5.8	2.8	3.83	10	93	0.49
32	SP	3.0	11.0	15.5	6.5	3.6	4.87	9	88	0.46
33	SP	2.9	10.3	13.0	5.4	2.5	3.47	8	75	0.60
34	SP	2.8	11.7	14.8	8.1	3.3	4.23	8	83	0.68
35	SP	2.7	—	—	5.0	3.3	4.20	5	53	-
36	SP	2.6	—	—	5.9	3.3	4.50	5	51	-
37	S	3.4	9.3	15.7	5.7	3.3	4.43	9	85	0.32
38	S	3.3	9.4	12.5	5.2	2.9	4.00	15	155	0.98
39	S	3.0	8.4	18.8	6.4	3.4	4.40	12	130	0.59
40	S	2.9	4.5	16.7	5.5	3.8	4.67	5	58	0.27
41	S	2.7	9.3	14.1	5.4	3.7	5.00	8	57	0.53
42	S	2.7	7.5	15.1	5.9	3.2	4.10	7	70	0.54
43	S	2.5	9.4	13.5	5.9	3.3	4.17	7	72	0.80
44	S	2.5	—	—	5.9	3.4	4.23	7	70	-
45	S	2.4	11.8	16.6	5.7	3.3	4.10	3	33	0.29
46	S	2.4	10.7	18.8	5.0	2.6	3.40	8	75	0.57
47	S	2.4	8.5	16.6	6.0	3.6	4.40	8	52	0.57
48	S	2.4	8.2	15.8	5.6	3.2	4.00	5	45	0.43
49	S	2.3	11.8	18.4	6.7	4.4	5.07	11	108	0.86
50	S	2.3	6.8	—	5.7	3.4	4.17	3	28	-
51	S	2.2	8.4	—	5.9	3.7	4.43	4	41	-
52	S	2.2	8.8	13.7	5.2	3.0	3.73	6	50	0.74
53	S	2.1	13.1	—	5.5	3.4	4.10	[illegible]	56	-
54	S	2.1	7.7	13.8	4.9	2.8	3.50	2	15	0.26
55	S	2.1	7.5	14.1	5.2	4.1	4.67	4	35	0.63
56	S	2.1	7.0	21.4	5.3	3.2	3.90	4	38	0.29
57	S	2.1	—	—	6.5	4.4	5.10	6	74	-
58	S	2.0	11.4	12.5	4.8	2.8	3.55	3	28	0.68
59	S	2.0	7.5	19.1	5.2	3.2	3.87	6	54	0.58
60	S	2.0	6.0	14.4	5.5	3.5	4.17	5	47	0.86
61	S	2.0	4.4	18.8	5.2	4.2	4.87	[illegible]	62	0.88
62	S	2.0	—	—	5.2	3.2	3.87	4	35	-
63	S	1.8	10.0	14.8	5.8	3.9	4.43	2	21	0.45
64	S	1.9	—	—	6.4	4.5	5.13	[illegible]	41	-
65	S	1.8	12.4	21.4	5.2	3.4	4.00	[illegible]	23	0.24
66	S	1.8	10.0	11.0	4.5	2.7	3.30	[illegible]	25	0.94
67	S	1.8	8.8	21.7	8.0	4.2	4.80	[illegible]	79	0.88
68	S	1.8	9.1	20.2	5.7	3.9	4.50	4	35	0.43
69	S	1.7	11.5	22.4	8.2	4.5	5.07	7	68	0.88
70	S	1.7	10.3	—	6.4	4.7	5.27	5	47	-
71	S	1.7	—	—	4.4	2.7	3.27	2	17	-
72	S	1.7	—	—	5.4	3.7	4.27	[illegible]	33	-
73	S	1.8	11.1	—	4.2	2.6	3.13	[illegible]	4	-
74	S	1.6	10.8	14.4	4.8	3.2	3.73	[illegible]	17	0.54

* FL (Foam line); B (Breaker); SP-B (Sharp peak swell starting to break): FL-B (Breaker with some foam): SP (Sharp peak swell); S (Swell) .

Table 11 cont'd.

Test data on field pile

WAVE NO.	WAVE TYPE	WAVE HEIGHT H Ft.	WAVE PERIOD T Sec.	WAVE VELOCITY (MEASURED) C_f Ft./Sec.	ELEVATION OF CREST ON PILE S_c Ft.	ELEVATION OF TROUGH ON PILE S_t Ft.	STILL WATER LEVEL d $S_t+\frac{H}{3}$ d Ft.	MEASURED FORCE F_r Lbs.	TOTAL MOMENT M Ft-Lbs.	COEFFICIENT OF DRAG C_D
75	S	1.6	8.8	----	5.2	3.6	4.13	3	30	----
76	S	1.6	7.8	14.9	5.3	3.7	4.23	3	33	0.96
77	S	1.8	4.2	----	5.5	3.9	4.13	3	27	----
78	S	1.6	----	----	5.4	3.8	4.33	3	33	----
79	S	1.5	10.6	16.7	5.3	3.8	4.30	2	19	0.52
80	S	1.5	9.0	----	5.0	3.5	4.00	1	5	----
81	S	1.5	8.3	17.3	4.4	2.9	3.40	1	14	0.37
82	S	1.5	----	----	5.1	3.6	4.10	3	33	----
83	S	1.4	12.2	19.2	4.2	2.8	3.27	3	29	0.63
84	S	1.4	10.0	----	5.8	4.4	4.60	2	21	----
85	S	1.4	----	----	5.4	4.0	4.47	4	37	----
86	S	1.4	----	----	5.9	4.5	4.97	3	34	----
87	S	1.3	10.2	----	4.7	3.4	3.83		29	----
88	S	1.3	5.8	----	5.5	4.2	4.63	2	17	----
89	S	1.3	----	----	6.0	4.7	5.13	3	29	----
90	S	1.3	----	----	6.0	4.7	5.13	6	46	----
91	S	1.3	----	----	5.8	4.3	4.73	4	37	----
92	S	1.3	----	----	5.0	3.7	4.13	1	29	----
93	S	1.2	13.3	----	4.6	3.4	3.80	2	21	----
94	S	1.2	12.4	12.3	5.8	4.8	5.00	1	29	2.72
95	S	1.2	11.4	15.1	4.9	3.7	4.10	2	19	1.14
96	S	1.2	10.3	10.4	6.0	4.8	5.17	3	34	----
97	S	1.2	----	----	4.7	3.5	3.93	2	22	----
98	S	1.1	11.8	12.5	3.9	2.8	3.17	1	12	1.06
99	S	1.0	12.7	----	4.7	3.7	4.03	1	10	----
100	S	1.0	12.2	13.7	3.7	2.7	3.03	2	17	1.60
101	S	1.0	----	----	4.3	3.5	3.83	1	10	----
102	S	0.9	9.8	18.5	3.8	2.7	3.00	1	8	1.23
103	S	0.8	6.8	----	3.2	4.4	4.53	1	13	----
104	S	0.8	----	----	3.4	4.8	4.87	1	12	----
105	S	0.7	----	----	4.9	4.2	4.43	1	10	----
106	S	0.7	----	----	4.8	3.9	4.13	1	8	----
107	S	0.6	7.1	19.8	5.0	4.4	4.60	2	18	2.38

* S (Swell).

One feature becomes apparent in reviewing the data that permits a comparison between the model results and the field test results. The majority of the field test conditions were obtained with samll ratios of the pile diameter to the wave height, and with small ratios of the water depth at the pile to the wave length. Under these conditions, the phase angle as given by Equation (15) approaches zero and the maximum moment of Equation (9) occurs when the time angle, θ, is zero.

Equation (9) for a pile hinged at the bottom then reduces to

$$M_{max} = \rho \frac{D H^2 L^2}{T^2} C_D k_3' \qquad (18)$$

k_3' is introduced as a refinement of k_3 to include an approximation of velocity distributions in a wave of finite height in shallow water; that is,

$$k_3' = \frac{\frac{1}{2}\left(\frac{4\pi S_c}{L}\right)^2 + \frac{4\pi S_c}{L} \operatorname{Sinh} \frac{4\pi S_c}{L} - \operatorname{Cosh} \frac{4\pi S_c}{L} + 1}{64 \left(\operatorname{Sinh} \frac{2\pi d}{L}\right)^2} \qquad (19)$$

where $d = S_t + 1/3\ H$ (assumed still-water level)
S_c = wave crest elevation above the bottom
S_t = wave trough elevation above the bottom
H = wave height

For small values of d/L, Sinh $4\pi S_c L$ is approximated by $4\pi S_c/L$, and Sinh $2\pi d/L$ is approximated by $2\pi d/L$. These approximations result in

$$k_3' = \frac{3}{32} \frac{S_c}{d} \qquad (20)$$

and

$$C_D = \frac{32}{3} \frac{M_{max} T^2 d}{\rho D H^2 L^2 S_c} \qquad (21)$$

As the wave velocity is related to the length and period by $C = L/T$, we find that

$$C_D = \frac{32}{3} \frac{M_{max} C^2 d}{\rho D H^2 S_c} \qquad (22)$$

All variables on the right side of Equation (22) were measured and C_D then computed. C_D is a drag coefficient which depends upon the state of the disturbance of the wave motion due to the movement of the wave past the pile. For shallow-water waves, the velocity distribution from the crest of the wave to the bottom is a function of the ratio of wave height to water depth, and is essentially independent of the wave length or period. The resulting moment on the pile, and hence C_D, are functions of this ratio, H/d. The results are shown in Fig. 16 on this basis, with segregation of the results according to wave type. The field pile results were obtained for wave conditions of d/L less than 0.06, with the majority of the waves characterized by d/L less than 0.03.

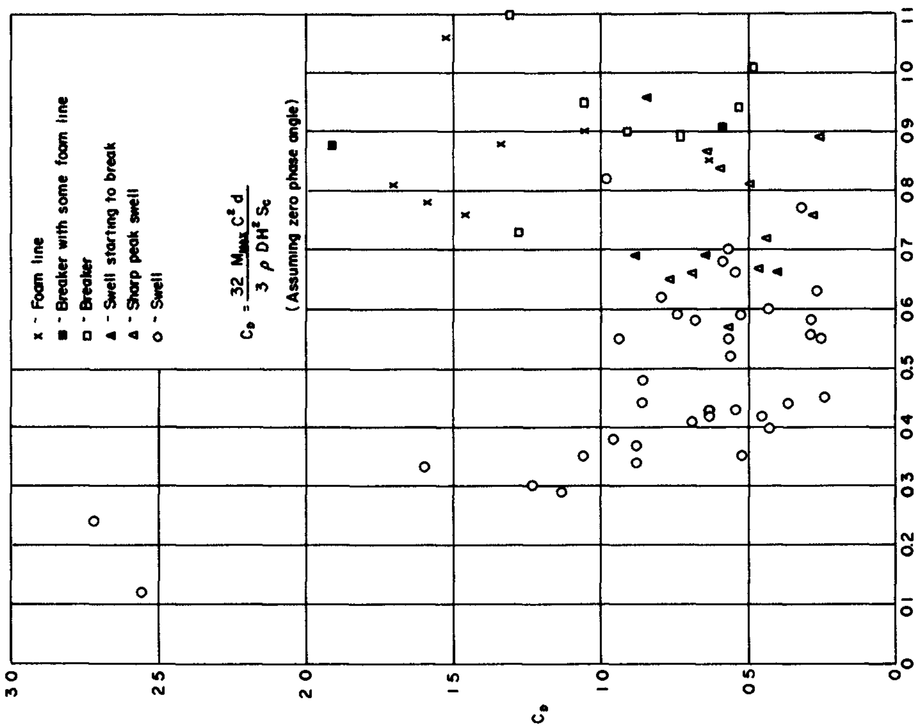

Fig. 16. Coefficient of drag computed from field tests on a circular pile.

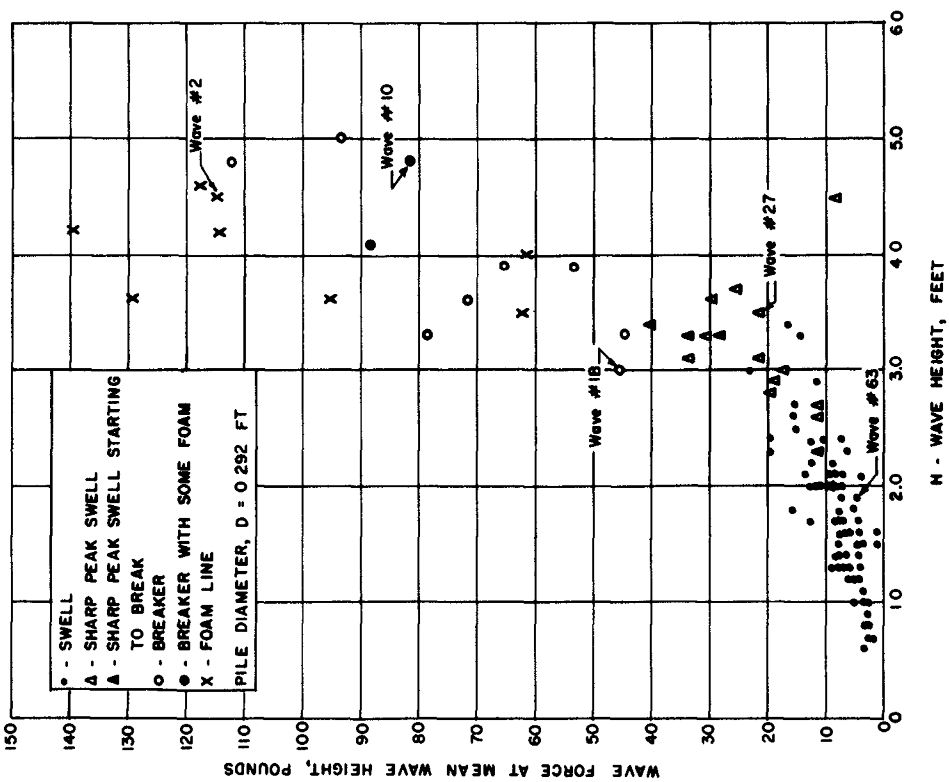

Fig. 15. Wave force at mean wave height as a function of wave height.

The scatter of the results reflects the accuracy of the data and the accuracy of the assumptions of Equation (22), C_D is computed from Equation (22) which contains the square of the wave velocity and the square of the wave height. Small discrepancies of these variables may produce appreciable differences in C_D. The maximum moment was obtained from the force, which was measured to within one pound. Many of the measured forces were from one to five pounds. Some scatter of results is necessarily expected.

Enough data were taken to permit the following general observations.

1) Foam lines and breakers produce higher values of C_D than unbroken swells.

2) For values of H/d greater than 0.4, an average value of C_D equal to 0.50 best represents the results.

3) For values of H/d less than 0.4, C_D becomes larger than 0.50. The assumptions of Equation (22) become invalid in this range of H/d.

A direct comparison of the model test results with the field test results cannot be made. The same range of the governing parameters was not covered in the two series of tests, particularly the ranges of d/L and H/d. In Fig. 16 drag coefficients of 1.0 to 2.5 are shown for values H/d between 0.4 and 0.1. These magnitudes of the drag coefficients are in the same range as those obtained from the model studies. However, the values of d/L of the field tests were not the same as the model tests. As mentioned in the model test summary, complete correlations including all defining parameters have not yet been attained. No attempts have been made to carry the field results beyond Fig. 16.

CONCLUSIONS

The analysis of forces and moments on piles as summarized herein contains two coefficients which must be determined experimentally: the coefficient of mass and the coefficient of drag. The results so far obtained indicate that the theoretical value of 2.0 for the coefficient of mass is adequate for computing the forces on circular piling. For the coefficient of drag, however, additional results are needed with a large range of the variables of pile diameter, wave height, wave length, and water depth.

The results show that moments measured about a single hinge point will suffice in establishing the magnitudes of the coefficients. The moment distribution from coefficients obtained from moments about a bottom hinge point agree with measured moment distributions.

Measured moments on piles of cross-sectional shape other than circular show coefficients which are a function of the shape of the pile. Steady-state drag coefficients can not be used as drag coefficients in the analysis of periodic motion.

Results of the interference effects of rows of circular piling, while limited in scope, indicated that for clearances greater than $1\frac{1}{2}$ pile diameters the interference effects are negligible. Moments on center piles of a row are increased as compared to moments on an isolated pile for spacings less than $1\frac{1}{2}$ pile diameters.

Moments on circular piles arranged in columns are decreased as compared to moments on an isolated pile. No limits were determined at which the moment became independent of the spacing.

RECOMMENDATIONS

The following experiments on model piles are recommended for comparison purposes with theoretical work and prototype tests.

1. Measurement of wave force distribution on single piles of various diameters are needed in order to compare with Equations (4) and (8).

2. Experiments with a greater number of wave conditions on circular piles, H-sections, flat plates and various other objects are needed in order to establish the relationship of the coefficients of drag and mass to the wave characteristics.

3. Investigation should be made of the mathematical theories pertaining to piles and other objects subject to wave action with respect to force, wave reflection, wave diffraction and flow conditions in the vicinity of the object.

4. Investigation should be made of breaking waves on model structures including the development of force recording equipment.

ACKNOWLEDGMENTS

The above investigations were sponsored by the Office of Naval Research, Bureau of Yards and Docks, The California Company, and International Marine Platforms, Inc.

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APPENDIX

Corrections to "Design of Piling". Chapter 28, Proceedings of the First Conference on Coastal Engineering:

Page 257, line 13

$$C_M = 2.0 \quad \text{(use theoretical value of 2.0)}$$

Page 257, line 24

$$M_z = \rho \frac{H^2L^2D}{T^2}\left\{\pm C_D K_2 \cos^2\theta + \frac{\pi}{4}\frac{D}{H} K_1 C_M \sin\theta\right\}$$

$$= \frac{(2.0)\ (10)^2\ (452)^2\ (1.5)}{(10)^2}\left\{+1.6\ (0.0837)(0.9848)^2\right.$$

$$\left. + \frac{\pi}{4}\ \frac{1.5}{10}\ (0.395)\ (2.0)\ (0.1737)\right\}$$

$$= 612{,}000\left\{0.1294 + 0.0162\right\} = \underline{\underline{89{,}000 \text{ ft. lbs.}}}$$

Page 258, line 15;

$$M = \rho \frac{H^2\ L^2\ D}{T^2}\left\{\frac{\pi}{4}\frac{D}{H}\ K_1\ C_M\ (1)\right\}$$

$$= \frac{(2.0)\ (10)^2\ (452)^2\ (6)}{10^2}\left\{\frac{\pi}{4}\ \frac{6}{10}\ (0.395)\ (2.0)\right\}$$

$$= \underline{\underline{916{,}000 \text{ ft. lbs.}}}$$

CHAPTER 26

HYDRAULIC MODEL TESTS FOR INDIANA HARBOR DEVELOPMENT

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INTRODUCTION

Plant facilities of the Inland Steel Company, East Chicago, Indiana, and the East Chicago plant of the Youngstown Sheet and Tube Company, occupy the entire perimeter and adjacent shoreline of Indiana Harbor (Figs. 1, 2, and 3). Major changes in the plants of these companies required revisions in the shoreline and the building of various bulkheads within the harbor and in Lake Michigan outside the harbor breakwater. Small scale hydraulic model studies of the harbor and portions of certain harbor structures were conducted to aid in the solution in the wave action problems arising from the construction programs of the two steel companies. Investigations concerned construction projects recently completed, projects under construction during the course of the model studies, and proposed future plans for plant expansion and harbor development.

Permits issued by the Federal Government for construction projects involving encroachments on lake areas and changes in harbor boundary conditions are granted on the condition that such constructions do not adversely affect wave action and navigation conditions in and adjacent to the harbor. The primary purpose of the Indiana Harbor model (Figs. 2 and 3) was to determine the effects of present and future construction projects of the two steel companies on wave and navigation conditions and, where necessary, to devise modified designs which would satisfy permit requirements. Section models were also used to provide functional and economic design information on specific aspects of certain of the structures, such as breakwater, bulkheads, and wave energy absorbers.

Hydraulic model investigations included (Fig. 3) studies of a 2500 ft steel sheet pile cellular bulkhead (Project Area I) and a 33 acre pier expansion plan for the Youngstown Sheet and Tube Company (Project Area II), and a 5200 ft steel sheet pile cellular bulkhead (Project Area III), a 1200 ft realignment of a dock (Project Area IV), and a 53 acre mooring area with adjacent docking facilities (Project Area V) for the Inland Steel Company. Features of Indiana Harbor as they existed before construction of the projects with which the model studies were concerned are shown in Fig. 2. The currently projected and potential developments are indicated in Fig. 3; project areas I, III, and IV have been built. Typical breakwater sections are shown in Fig. 4.

Model studies were started in March, 1951, at the Waterways Experiment Station, a Laboratory of the Corps of Engineers, Department of the Army, located in Vicksburg, Mississippi. The studies of project areas I, III, IV, and V have been completed as of September, 1953; some further studies are to be made to establish optimum design for area II.

GENERAL PROCEDURE OF THE MODEL STUDIES

PURPOSE OF THE INVESTIGATION

The specific purpose of the model studies was to determine the effects on wave action and navigation conditions in the harbor and harbor approach area of construction in progress or planned by The Youngstown Sheet and Tube Company and the Inland Steel Company. The performing of model tests for all major harbor projects of the Corps of Engineers has been standard practice for several years, and the advantages of conducting such studies have been proven many times. Thus, the use of small-scale hydraulic models to determine the effects of proposed plans on harbor conditions was a logical method of solving the problems which arose in planning for harbor and plant developments, while providing evidence that permit requirements were fulfilled. As the investigation progressed, the purpose and use of the models were enlarged to include the development of the most economical plans which would satisfy permit requirements, the providing of design information concerning wave forces on breakwaters and wave absorbers, and the economical development of future plans for which permits might be desired at a later date.

TYPES OF MODELS USED IN THE INVESTIGATION

In general there are two types of wave-action models. These are the harbor wave-action model and the wave-force or stability model. Harbor models usually reproduce the entire harbor area with the various shoreline structures, and sufficient area seaward to allow proper generation and refraction of waves. Section models usually reproduce only a portion of a harbor or only a section of a harbor structure, such as a breakwater or wave absorber. Section models are used to determine the reflection, absorption, and stability characteristics of breakwaters, wave absorbers, and other harbor structures, as a function of wave height, wave length, and wave direction.

Both types of models were used in the Indiana Harbor investigation. The harbor model was constructed of concrete to a linear scale of 1:150, model to prototype. The section models had linear scales of 1:50 and 1:55.

DESIGN OF WAVE-ACTION MODELS

The Indiana Harbor models were designed to investigate the effects on harbor structures of wave action resulting from storms which generate surface wind waves. Surface wind waves in nature are generated by the tractive force of wind over water, and are propagated by the restoring force of gravity. Model waves are also propagated by the restoring force of

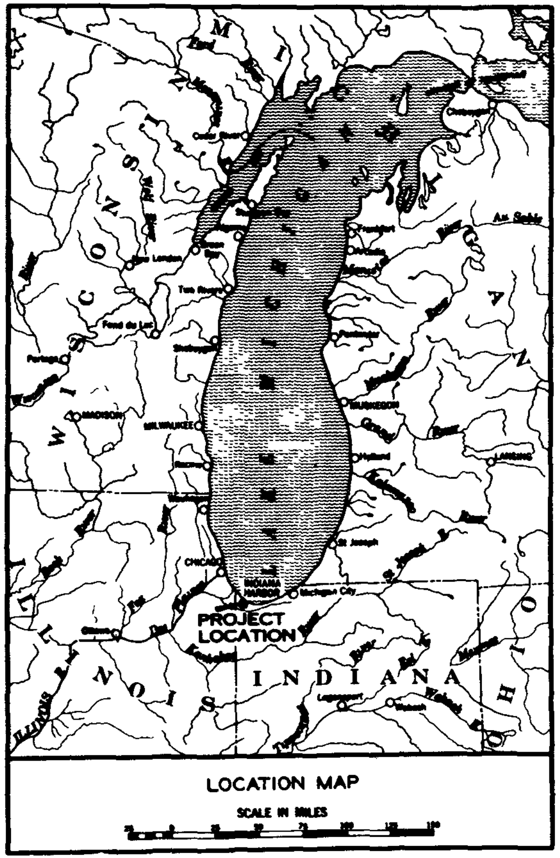

Fig. 1. Map showing location of Indiana Harbor in the southernmost boundary of Lake Michigan. Plants of Inland Steel Co., and the Youngstown Sheet and Tube Co. border completely the periphery of the harbor.

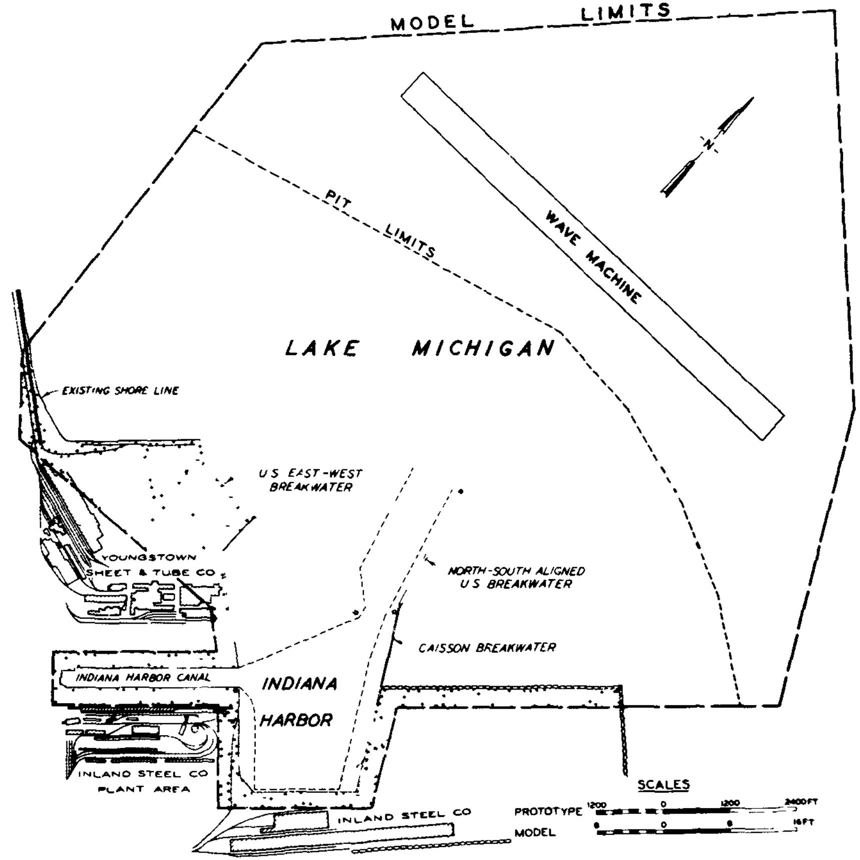

Fig. 2. Diagram showing boundary limits of 1:150-scale model constructed to establish optimum design for harbor development by the steel companies.

gravity. However, in hydraulic models, the waves are generated by the displacement incident to vertical movements of a plunger in the water.

Economical considerations require selection of the smallest model consistent with the required accuracy of test results. If the model is too small, the forces of surface tension and friction may become sufficient in magnitude, relative to the force of gravity, to preclude accurate reproduction of wave phenomena. The type of instrumentation necessary, and the required degree of accuracy of test measurement, are also primary factors in selection of model scale. Thus, in the case of the Indiana Harbor model, where it was only necessary to obtain overall harbor wave heights and current patterns, a linear scale of 1:150, model to prototype, was selected. Fig. 2 shows the extent of the harbor and lake areas contained within the 1:150-scale model limits. For the section models which involved measurement of wave forces, the absorption of wave energy and the stability of rock in the breakwaters and wave absorbers, linear scales of 1:50 and 1:55 were selected. The change in scale from 1:50 and 1:55 was for convenience of model operation only, and is not significant with respect to accuracy of results.

Both types of models were designed and operated in accordance with Froude's[1] model law, and constructed geometrically similar to the prototype harbor and harbor structures to insure dynamic similarity of motion occurrences. The following scale relationships (model to prototype transference equations) were derived based on Froude's model law, a specific weight scale of 1:1, and linear scales of 1:150, 1:55, and 1:50 (Table 1).

TABLE 1

TABULATION OF MODEL TO PROTOTYPE SCALE RELATIONSHIPS
FOR VARIOUS LENGTH RATIOS

Characteristic	1:150-Scale Model	1:55-Scale Model	1:50-Scale Model
Length	1:150	1:55	1:50
Area	1:22,500	1:3025	1:2500
Volume	1:3,375,000	1:166,375	1:125,000
Time	1:12.25	1:7.42	1:7.07
Velocity	1:12.25	1:7.42	1:7.07
Unit Pressure	1:150	1:55	1:50
Force	1:3,375,000	1:166,375	1:125,000
Weight	1:3,375,000	1:166,375	1:125,000

SELECTION OF TEST-WAVE DIMENSIONS AND DIRECTIONS

Judicious selection of test waves requires adequate data of wave-period spectrums and corresponding wave heights which occur at the harbor from different directions of approach. In many similar model studies it is necessary to know the frequency of occurrence of storm waves of different magnitudes from the different directions. However, in this instance, it was not necessary to determine frequency of occurrence of storm waves from the different directions because the primary purpose of the study was such that the analysis of model test results could be made on a comparison basis. It was sufficient for purposes of this study to select test waves based on the estimated average storm waves from the different directions at Indiana Harbor.

Wave dimensions in nature have never been recorded systematically as has been the case for storm winds. Surface water waves are the result of wind blowing over bodies of water, and the magnitude of waves is a function of wind speed, wind duration, and the fetch or distance over which the wind blows. The lack of adequate wave records made it necessary to estimate the characteristics of waves at Indiana Harbor from available wind data and theoretical and empirical wave charts and formulas. Available wind data consisted of a wind rose for the years 1932-1946, and a record of 13 severe storms. These data were furnished by the Chicago District, Corps of Engineers, Department of the Army.

After waves are generated by storms over deeper portions of the lake, they travel into shallow water where they "touch" bottom and are influenced in their travel toward shore by the pattern of bottom contours. Where the depth of water is considerably less than one-half the wave length, the direction of travel of wave fronts is affected by changes in depths of water. Thus, when waves approach shore at an angle, the crests bend because the portion that is nearer shore is in shallower water and travels more slowly than the more off-shore portion. The bending of wave crests in this manner is called "refraction" because of its analogy to the bending of light rays in optical systems. The results of refraction are changes in wave height and the direction of travel of wave crests. Depth contours can be such that they cause waves to be focused on a harbor, or a particular portion of a harbor structure. Also, depth contours can be such that large waves are both reduced in height and changed in direction to the extent that only the smaller waves reach the harbor from a particular deep-water storm direction. The hydrography lakeward of Indiana harbor is relatively shallow for a considerable distance. Therefore, the effects of wave refraction played an important role in the selection of model test waves.

The effects on waves of changes in depth contours are best determined graphically by wave-refraction diagrams. Before performing the Indiana harbor model studies, the deep-water wave dimensions were selected with the aid of Stevenson's formula[2] and the Sverdrup-Munk curves,[3] which show quantitatively the relationship between wind speed, wind duration, fetch, wave heights, and wave periods. Shallow-water wave dimensions and directions, corresponding to selected deep-water wave characteristics, were estimated by charting the deep-water waves into shallow water, to the outer areas reproduced in the model, by refraction diagrams.[4] Wave heights, periods, and directions determined in this manner were generated in the model by wave machines. The depth contours in the harbor model between the wave machine and the harbor and harbor structures were reproduced accurately in concrete. Therefore, the effects on wave characteristics caused by changes in depth, from deep water thence through shallow water to the harbor, were reproduced with considerable accuracy.

Table 2 shows the selected test waves in deep water and after refraction for the different storm directions (H = wave height, T = wave period):

TABLE 2

TABULATION OF CHARACTERISTIC WAVES ESTABLISHED FOR HYDRAULIC MODEL STUDIES OF INDIANA HARBOR

Deep-Water Waves			Shallow-Water Waves*		
Direction	H (Ft)	T (Sec)	Direction	H (Ft)	T (Sec)
E	5.5	3.5	E	5.0	4.0
N 67½° E	10.0	5.0	N 55° E	9.5	5.0
NE	13.0	6.0	N 35° E	9.0	6.0
NE	--	--	NE	10.0	4.0
N 30° E	--	--	N 25° E	15.0	7.0
N 15° E	17.0	8.0	N 15° E	21.0	8.0
N 7½° E	17.0	8.0	N 10° E	16.0	8.0
N	17.0	7.5	N 5° E	11.0	7.5
N	--	--	N	9.0	4.0
N 10° W	15.0	6.5	N 3° E	6.0	6.5
N 10° W	--	--	N 10° W	8.0	4.0
N 15° W	13.0	5.5	N 5° W	7.0	5.5
N 15° W	--	--	N 15° W	7.5	4.0
N 20° W	11.5	5.0	N 16° W	7.0	4.0
N 30° W	7.0	3.8	N 30° W	6.0	4.0
NW	4.0	3.0	NW	4.0	4.0

*At position of wave machine

In those instances where the selected shallow-water wave period in the above tabulation is less than the corresponding deep-water period, the wave height in shallow water corresponding to the smaller period is larger than that of the larger period. In those instances where the selected shallow-water period is greater than the corresponding deep-water period, the value of the deep-water period was so small that the RPM of the available wave machine was exceeded.

OPERATION OF THE MODELS

The 1:150-scale harbor model was first operated with conditions in the harbor reproducing those which existed prior to installation of problem structures under consideration. These initial investigations are called "base tests." The purpose of base tests is to obtain basic data for use as a reference to which the results of tests of various proposed plans may be compared. Using selected test waves from the critical directions, wave heights, and current patterns were determined throughout the harbor and in the navigation channel for base-test conditions and with the different proposed plans installed. Wave heights in the model were measured with wave-height gages,[5] or pick-up units, in connection with a recording oscillograph. A wave-height gage consists of series-connected resistors installed in a direct-current circuit with the resistors so calculated that the current varies directly with submergence of the gage in water. Currents were determined by small pole-type floats.

The 1:50- and 1:55-scale section models were constructed in existing wave flumes. The breakwater and wave-absorber sections were constructed in the flumes using crushed rock and molded concrete blocks to simulate the prototype core material and cap rock. The test sections were placed at one end of the flumes and waves were generated at the other end. The stability of breakwater sections, the absorption and reflection characteristics of wave absorbers, and wave forces on a proposed vertical-wall parapet were determined in these tests. Wave forces on the parapet were recorded electrically with a pressure cell connected to a recording oscillograph.

SOLUTION OF INDIVIDUAL PROBLEMS

The elements of plans and changes in harbor boundary conditions for each problems studied in the Indiana Harbor investigation are shown by Fig. 3. The following presents a brief description of each major problem, outlines the tests performed, and summarizes salient test results.

YOUNGSTOWN SHEET AND TUBE COMPANY'S NORTHWEST BULKHEAD (PROJECT AREA I, FIG. 3)

The Youngstown Sheet and Tube Company constructed a sheet steel pile, cellular bulkhead 2500 ft long, west of the navigation channel and aligned in a northwest direction parallel with the shoreline. The area behind the bulkhead is to be filled and reclaimed for use in future plant development. Immediately after construction of this bulkhead was completed, ships' captains began complaining of unusually difficult navigation conditions in the entrance channel, although waves in the vicinity had been comparatively small in magnitude. It was surmised that the sudden change in navigation conditions was caused by waves, which previously had expended their energy harmlessly on the beach, being reflected from the vertical-wall bulkhead across the navigation channel. The reflected waves were thought to impinge on the north-south aligned breakwater and reflect back across the channel, thus causing standing waves with resultant surging of the ships in a direction perpendicular to the navigation channel.

Tests were performed on the 1:150-scale harbor model using waves from several storm directions to determine which direction, if any, resulted in waves being reflected across the navigation channel. These tests were made (a) for base-test conditions with the northwest bulkhead omitted; (b) with the bulkhead installed as it was originally constructed; and (c) with a rubble-mound wave absorber installed along the bulkhead on the lake side. It was found that the northwest bulkhead reflected waves across the navigation channel sufficiently to cause adverse navigation conditions, and that the remedy consisted in placing a rubble wave absorber along the lakeside of the bulkhead. The optimum design of a stable wave-absorber section was determined by model tests on the 1:50-scale section model.

DESIGN OF NORTHWEST BULKHEAD WAVE ABSORBER
(PROJECT AREA I, FIG. 3)

Tests on the 1:150-scale harbor model showed that a rubble wave absorber along the northwest bulkhead would prevent waves being reflected from the bulkhead into the navigation channel. However, the 1:150-scale model was too small in size to allow determination of the optimum design of the wave absorber. It was desired to determine the smallest quantity of rock which would absorb sufficient incident-wave energy to reduce reflected wave heights satisfactorily.

Wave-absorber sections using different quantities of rock were tested on the 1:50-scale section model. Three sizes of prototype rock were simulated. The rock sizes tested were 1-4 ton, 3-10 ton, and 7-20 ton. Prototype rock of these weights were available locally in adequate quantities. The volume of rock in each wave-absorber section was determined by the berm height and width and slope of the rubble mound. The side slopes used to insure stability of the rock during wave attack were calculated using the generalized Iribarren formula,[6] and coefficients established in previous model tests conducted at the Waterways Experiment Station. The efficiency of each wave-absorber section was determined as a function of rock size and volume by measuring the incident and reflected wave heights. Elements of typical wave-absorber sections (1-4 ton rock only) are shown on Fig. 5. Fig. 6 shows the per cent of incident wave energy absorbed as a function of volume of wave absorber and rock size. Based upon the results of these tests, a wave-absorber section having from 30-40 cu yd of rock per ft was recommended. The rubble mound has now been placed in the field along this bulkhead and has eliminated the wave problem in the navigation channel.

INLAND STEEL COMPANY'S TIN-MILL DOCK (PROJECT AREA IV, FIG. 3)

The Inland Steel Company's realigned tin-mill dock, located in the southeast corner of the harbor, was tested to determine whether realignment of the dock increased wave-action conditions in the harbor. Wave-height and current-pattern tests were performed on the 1:150-scale harbor model, with and without the realigned dock installed in the harbor, and it was determined that the realigned dock had no measurable effect on wave-action conditions in the harbor.

YOUNGSTOWN SHEET AND TUBE COMPANY'S PIER EXPANSION
(PROJECT AREA II, FIG. 3)

The Youngstown Sheet and Tube Company plans to increase their plant area to facilitate railway materials handling by constructing a bulkhead

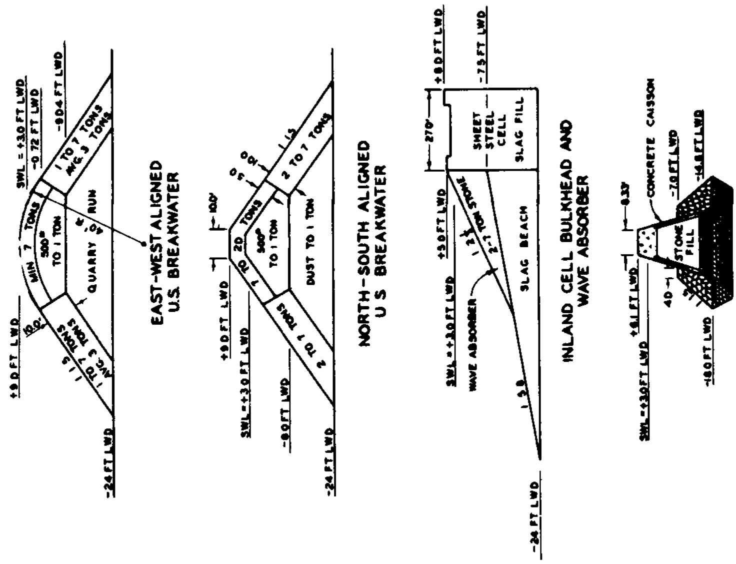

Fig. 4. Typical breakwater sections at Indiana Harbor. (Note LWD = 578.5 ft. above mean tide at New York City; SWL = still water level).

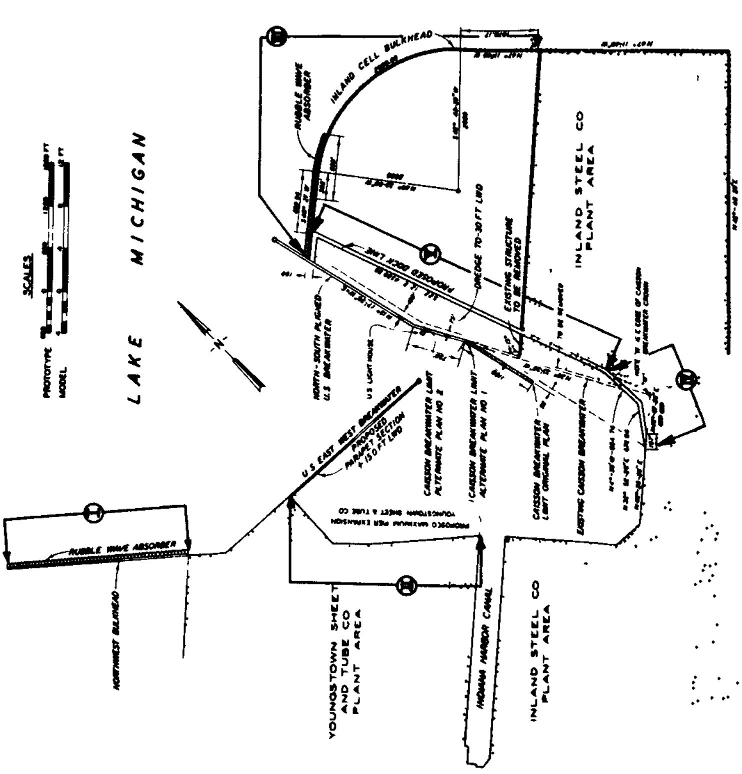

Fig. 3. Diagrammatic sketch of Indiana Harbor model showing principal project areas investigated; I - 2500 ft. steel sheet pile cellular bulkhead for the Youngstown Sheet and Tube Co.; II - 33 acre pier expansion for Youngstown; III - 5200 ft. steel sheet pile cellular bulkhead for Inland Steel Co.; IV - 1200 ft. realignment of a dock for Inland; V - 53 acre mooring area with docking facilities for Inland.

in the northwest portion of the harbor, and again it was desired to determine whether this installation increased wave action inside the harbor. Wave-height and current-pattern tests were conducted with base-test conditions, and with the proposed plan of pier expansion installed in the harbor. It was found that while installation of the pier did somewhat increase wave action inside the harbor, various means of remedying the situation can be established. Tests were conducted which showed that the bulkhead could be constructed in the harbor without increasing wave action if a rubble wave absorber were installed along the bulkhead. However, with a view toward economy and also the potential desirability of using the bulkhead as a mooring dock for ships, which precluded the placing of a rubble mound along the pier, other means of rectifying the wave action have been explored. Tests were conducted to determine the minimum length of parapet on the east-west aligned U. S. breakwater required to reduce overtopping of storm waves sufficiently to allow construction of the bulkhead without an increase in wave action in the harbor. Results of these tests showed that the status quo with respect to wave conditions in the harbor could be maintained if overtopping were reduced to a satisfactory minimum along the western 1200 feet of the east-west aligned U. S. breakwater. The problem was then reduced to that of designing a revised section of the east-west breakwater which would fulfill the minimum requirements of the permit at a minimum cost. A series of model tests on the 1:55-scale flume model was conducted to determine the required design section of (a) a rubble-mound addition lakeward to the breakwater; (b) a concrete parapet surmounting the breakwater; and (c) a sheet-steel pile wall inside the breakwater.

INLAND STEEL COMPANY'S NORTHEAST BULKHEAD (PROJECT AREA III, FIG. 3)

The Inland Steel Company constructed a sheet steel pile, cellular bulkhead 5200 ft long, east of the north-south aligned U. S. breakwater and north of their existing plant area. The area behind the bulkhead is to be filled and reclaimed for use in future plant development. A portion of the bulkhead perimeter reflected waves across the navigation channel. It was desired to determine by model tests the length of bulkhead which reflected waves across the channel, and the minimum volume of rock required to construct a wave absorber adequate to reduce wave reflection to a satisfactory minimum. The problem was similar to the northwest bulkhead problem, except that the Inland Steel bulkhead was curved convex to the waves. This convexity limited the length of bulkhead which could reflect waves across the channel. Also, the Inland Steel bulkhead was provided considerable protection by the installation of an underwater slag fill shortly after the bulkhead was constructed. Another problem requiring solution was that of waves reflecting from the bulkhead and impinging directly onto the north end of the north-south aligned U. S. breakwater. It was feared that these waves, which were larger than those which had attacked the breakwater prior to construction of the bulkhead, would endanger the stability of the breakwater.

Tests were conducted on the 1:150-scale model to determine which portion of the Inland Steel bulkhead reflected waves across the navigation channel. Tests were conducted on the 1:50- and 1:55-scale models to

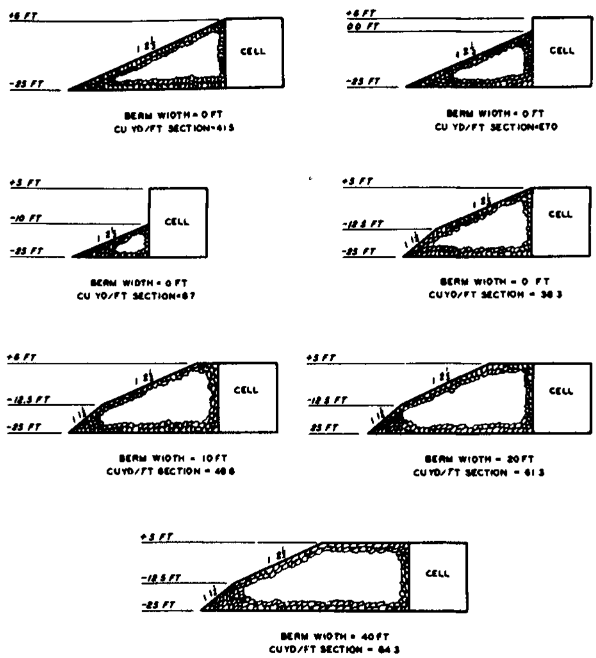

Fig. 5. Diagrammatic sections showing typical wave absorbers tested in 1:50-scale model to establish optimum design for absorbing waves otherwise reflected from northwest bulkhead across navigation channel. All tests in this series of experiments simulated to 1 to 4 ton rock; the elevations are referred to low water datum on Lake Michigan.

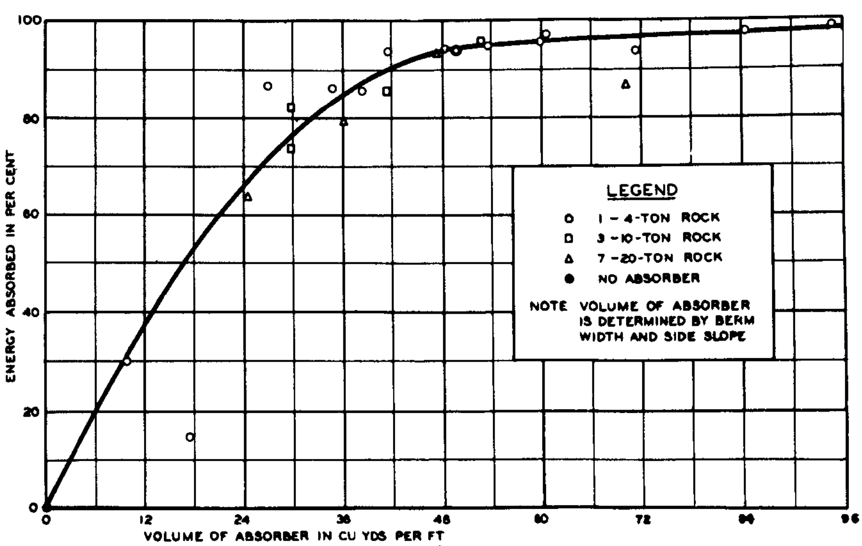

Fig. 6. Results of experimental study of efficiency of wave absorbers for northwest bulkhead. Experiments conducted on 1:50-scale model.

determine the minimum amount of rock necessary to provide an adequate wave absorber, and the effects of reflected waves on the stability of the adjacent U. S. breakwater. It was found that a 900-ft reach of the bulkhead perimeter reflected waves across critical reaches of the navigation channel. The minimum design section of the combined slag-rock fill devised to absorb incident wave energy sufficiently to provide adequate protection to the navigation channel from reflected waves is shown by Fig. 4, 3rd section. The results of the stability tests of the adjacent north-south aligned breakwater with and without an added wave absorber in the corner, formed by the breakwater and bulkhead, are very significant; the reflected waves from the bulkhead without the added wave absorber did considerable damage to the breakwater. After the wave-absorber section was extended from the west end of the 900-ft absorber to the corner, the reflected wave energy was not sufficient to endanger the stability of the breakwater. Construction of a rubble wave absorber from the west end of the 900-ft absorber to the corner was established as an optimum solution.

INLAND STEEL COMPANY'S PROPOSED INNER-HARBOR SLIP AND DOCK (PROJECT AREA V, FIG. 3)

The Inland Steel Company took advantage of the existing harbor model to test the effects on harbor wave action of a proposed layout for an inner-harbor slip and docking area envisioned in possible plans for future harbor and plant development. Tests were also desired to determine whether the protecting breakwater proposed for the slip entrance was necessary to provide adequate protection to the slip. Wave-height and current tests were conducted with and without the slip installed in the model, and with various lengths of the originally proposed protecting breakwater at the entrance to the slip. Wave-action conditions in the slip were found to be satisfactory for all conditions tested. Test results also showed that the 1100-ft breakwater at the slip entrance could be eliminated without increasing wave conditions in the slip. The breakwater was also found to reflect waves across the harbor onto the proposed Youngstown Sheet and Tube Company piers. Therefore, both in the interest of economy and better navigation conditions, construction of this inner-harbor breakwater is not recommended.

CONCLUSIONS

The use of hydraulic models as a tool of the design engineer is feasible in many types of engineering projects. Where the problems of design have to do with the effects of wave action on harbors and harbor works, or where it is necessary to determine the effects of changes in harbor boundary conditions, resulting from a proposed harbor-development project, it is not only feasible but, in many instances, quite essential to conduct model studies if optimum designs are obtained. The complexity of wave-action phenomena and the complicated geometry of most harbors usually make it impossible to obtain adequate answers to problems of design by a purely analytical approach. The general experience and judgment of the engineer cannot be utilized effectively except for the most elementary wave-action problems. Indeed, this method of approach has led to designs which not only failed to reduce wave-action conditions, but instead, actually intensified them. Therefore, the hydraulic model technique of such problems becomes indispensable to insure that the safest, most effective, and most economical harbor and breakwater design is obtained.

The Indiana Harbor model investigation is an excellent example of the value of model studies in providing design engineers and planning experts answers to difficult wave action problems pertaining to harbor and plant development. Results of the model tests provided direct and accurate answers to problems of harbor construction involving the expenditure of several millions of dollars. The cost of the model studies was within about one per cent or less of the estimated total construction costs.

ACKNOWLEDGMENTS

Of the two authors of this paper, Dr. Straub acted in the capacity of engineering consultant to both the Inland Steel Company and the Youngstown Sheet and Tube Company in correlating results of the model tests and with the overall planning for the plant expansion. The model studies were conducted under the direct supervision of Mr. Hudson. The engineering program of Inland Steel Company was under Fred H. Johnson, Chief Engineer; Tom Myhre, Planning Engineer; and James Howard, Assistant Chief Engineer; for Youngstown Sheet and Tube Company it was A. J. Hulse, Chief Engineer. The hydraulic model studies were conducted at the Waterways Experiment Station of the Corps of Engineers.

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CHAPTER 27

MAINTENANCE OF A NAVIGABLE CHANNEL THROUGH A BREAKTHROUGH AREA

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INTRODUCTION

The engineering problem described is one which is encountered on a portion along the Northeastern coastline where tidal currents and shifting sands are predominant features. Due to the extreme local variability of such phenomena, generalizations and similar precedents from which a solution might be adopted are seldom available. It is therefore necessary to attempt to reduce such problems to elements which can be treated by fundamental concepts. With this viewpoint, the proposed solution has been developed by applying elementary principles of energy dissipation to the local situation of a natural water passage, in which flow due to a tidal differential is essentially in one direction.

BACKGROUND AND GEOGRAPHICAL FEATURES

The area under consideration is located in the town of Chatham at the southeast "elbow" of Cape Cod, Massachusetts. Fig. 1 shows the breakthrough which separates the Monomoy Peninsula extending 10 miles south from the mainland. The eastern entrance to the breakthrough is on Pleasant Bay and is protected from the Atlantic Ocean by Nauset Beach. The westerly end of the breakthrough leads into Stage Harbor and thence via the harbor entrance into Nantucket Sound on the south shore of the Cape.

The present breakthrough dates from the hurricane of 1944, before which Monomoy Peninsula was joined to the mainland by a low, narrow beach. The area has a history of earlier breakthroughs and the entire region is noted for marked seasonal changes in sand-bar configurations, especially at the lower end of Nauset Beach. The eastern entrance of the break is approximately 1000 feet wide and tapers to 500 feet at its narrowest section. Observations over several years have shown that the entrance is becoming wider due to progressive erosion on the Monomoy side,and that the depth in the narrow section is increasing as a consequence of the increasing flow, which due to the existing tidal differential between Pleasant Bay and Stage Harbor is essentially in one direction, from east to west.

STATEMENT OF PROBLEM

The present investigation was initiated because the quantity of sand moving through the break and its subsequent deposition is reapidly threatening the usefulness of the Stage Harbor anchorage and maneuvering area. There is a definite incentive for early action with a decision to be made whether to close the break entirely or to maintain navigation through the cut due to its demonstrated usefulness as a 20-mile nautical short-cut from Stage Harbor to the boating and fishing areas off the east shore of the Cape. The specific requirements of the solution may be summarized as follows:

* Presented at the Third Conference on Coastal Engineering.

MAINTENANCE OF A NAVIGABLE CHANNEL THROUGH A BREAKTHROUGH AREA

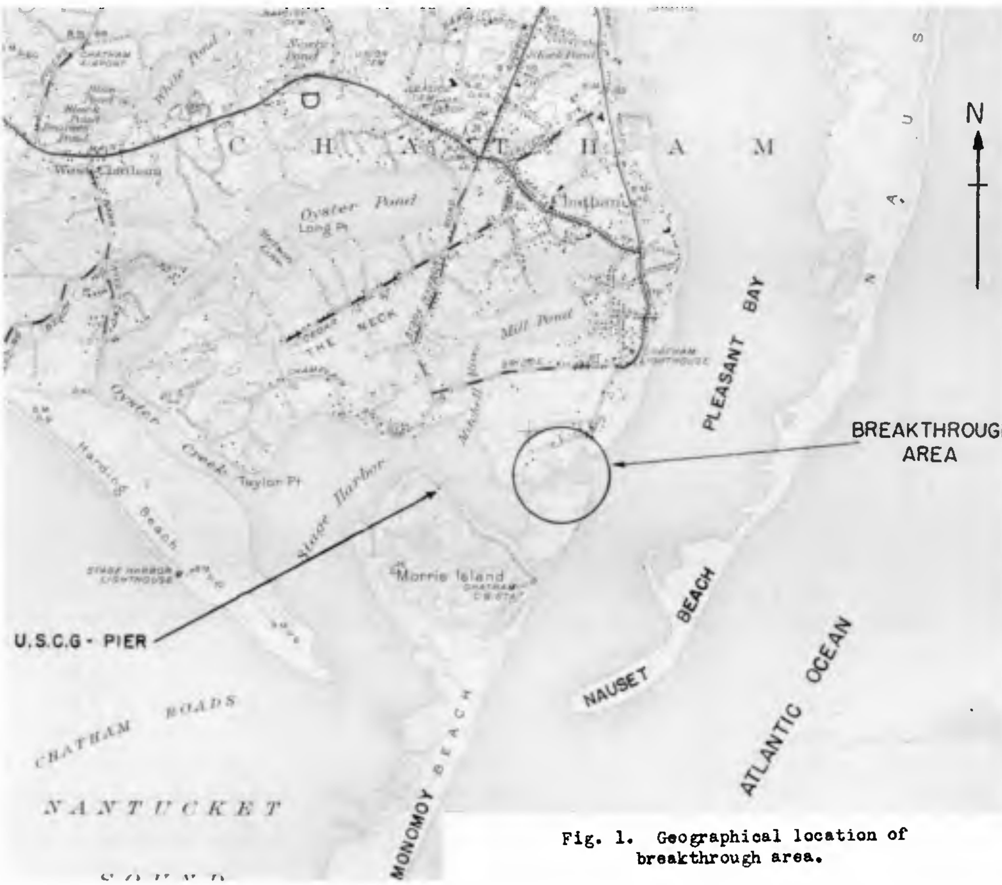

Fig. 1. Geographical location of breakthrough area.

Figure 1.

(a) The sand movement through the break is to be reduced to an amount consistent with maintaining Stage Harbor.

(b) A navigable waterway is to be provided if compatible with (a).

(c) Any proposed structures should in no way endanger present low-level buildings in the adjacent areas.

SUMMARY OF OBSERVATIONS

Simultaneous water-level observations during one tidal cycle established the fact that under mean tidal considerations, a water elevation differential of approximately four inches exists at high tide between the extreme ends of the breakthrough. The mean tide range for this area is 3.6 feet. Since the higher elevation occurs on the eastern or Atlantic side, a strong westerly current transporting large quantities of sand is produced. Fig. 2 is an aerial photograph looking west from the breakthrough entrance toward Stage Harbor showing the direction of the tidal current as determined from the orientation of sand ripples along the bottom. Velocities in excess of three knots have been observed in the narrow portion under mean tidal conditions. Fig. 3 is a similar photograph looking from Stage Harbor in an easterly direction toward the breakthrough showing the position of the shelf of deposited sand as of February 1952. A comparison of previous surveys indicates a yearly deposition of 50,000 cubic yards of sand and a movement of the shelf into Stage Harbor at the rate of 100 feet per year. At low tide, the shelf is dry except for a narrow channel containing less than one foot of water, and under present conditions the breakthrough is navigable to small boats from about two hours before to two hours following high tide, at which time there is approximately 4 - 5 feet of water in the channel. The Coast Guard launching pier in the right center of Fig. 3 has been abandoned due to the presence of the sand shelf. Large steep-fronted sand ripples measuring 7 - 10 feet in length and 3 - 6 inches in height have been observed in this area during low tide.

PROPOSED SOLUTION

It is evident that the amount of sand transport into the harbor is related to the volume rate of flow through the break and that its reduction is dependent on decreasing this flow rate during the high-tide periods. Any solution of course must consider the economic aspects of the problem. It is therefore proposed that the discharge be reduced by minimizing both the cross-sectional area and the velocities of flow in the following manner:

Reduction of cross section can be achieved by construction of dikes from either shore leaving an opening to be determined by navigation requirements. Reduction of velocities on the other hand cannot be brought about by a simple reduction of area, inasmuch as the tidal head differential is essentially independent of the total rate of flow through the break. Velocities, therefore, can be reduced only by increasing energy dissipation such as might be obtained by greater channel roughness or by making use of turbulent dissipation at flow expansions. The latter method can be effectively achieved by constructing several dikes with navigation openings in series with subsequent energy dissipation across each flow constriction. The number of dikes is necessarily limited by economical considerations and the present proposal incorporates two such structures. Each dike would have a 30-foot navigation opening protected by wingwalls and a bottom sill of proper height to prevent under-cutting and scour at the constructed section, as well as to prevent sand movement without obstructing passage of small boats. Assuming that one-half of the total four-inch head difference occurs across each constriction, a velocity of approximately two knots may be expected locally in these openings as compared with velocities in excess of three knots under present conditions or with a single dike. It is estimated that 33 percent reduction in velocity coupled with the decrease in cross section would result in a flow of the order of 10 percent of the present rate under similar tidal conditions. It is probable that the rate of sand transport would be reduced to a greater extent than is indicated by the decrease in the flow rate. In addition, the deep pool between the two dikes would act as a sand trap for some of the material entering through the outer dike. Fig. 4 is a schematic view of the general area showing the location of the proposed dikes in relation to the breakthrough. A navigable waterway which will permit passage of small craft throughout the tidal cycle could be obtained by dredging a channel on the existing sand shelf as indicated.

Fig. 5 shows a cross section along the navigation channel extending from Stage Harbor through the break into Pleasant Bay. The function of the sills in reducing sand movement and the deep portion between the dikes are clearly illustrated.

Fig. 2. Entrance to breakthrough looking toward Stage Harbor at low tide.

Fig. 3. Position of sand shelf at Stage Harbor at end of breakthrough (Feb. 1952).

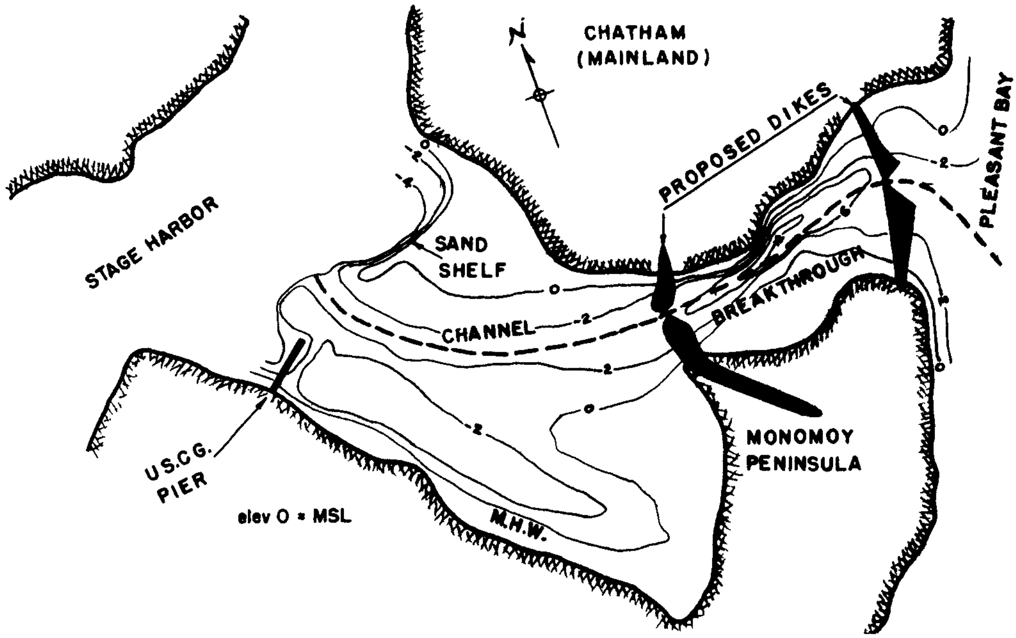

Fig. 4. Schematic view of breakthrough showing location of proposed dikes.

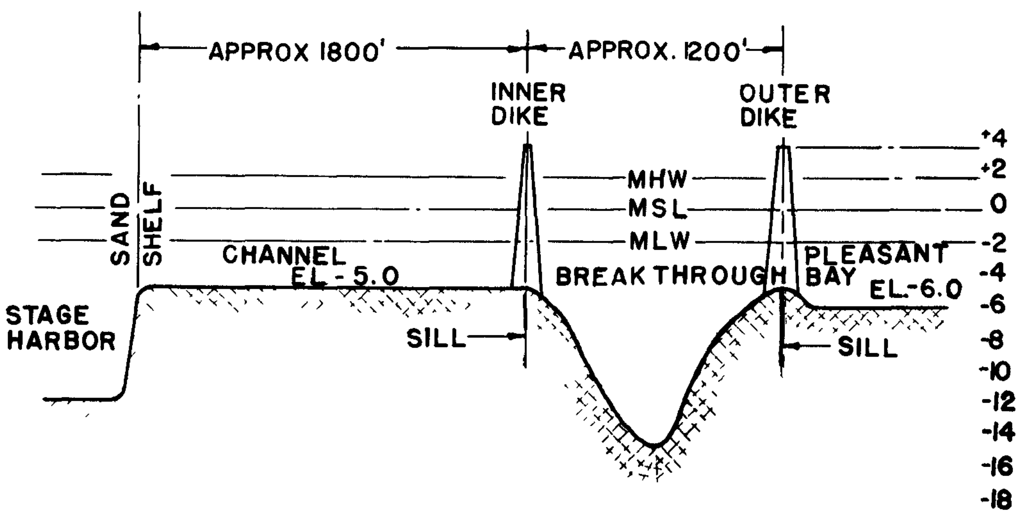

Fig. 5. Cross section along channel between outer dike and Stage Harbor.

The elevation of the top of the dikes would be two feet above mean high water, due to the presence of existing buildings at elevation 3.5 feet above M.H.W. It is therefore necessary that storm tides overtop the dikes so as not to endanger this property. Under these conditions, the outer dike would act as a weir having a crest length of over 1000 feet. The dikes are well protected against wave action from the open ocean by Nauset Beach. Observations during a severe storm showed waves breaking across this offshore beach and very little wave action in the vicinity of the breakthrough.

In a small engineering project such as this, estimated costs had to be held to an absolute minimum so therefore the dikes are proposed to be constructed as treated timber bulkheads of the type commonly used in beach groin construction. Sand obtained from dredging would be used as fill on either side of the sheet piling.

CHAPTER 28

TETRAPODS*

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INTRODUCTION

The tetrapod is no longer entirely a new-comer in the field of marine construction. The sea water intake of the Roches Noires thermic power station at Casablanca, North Africa, the first structure for which tetrapods were used, was constructed nearly four years ago. However, it is of interest to recall the ideas which stimulated the invention of this new type of protection.

The following remark made by engineers specializing in marine work undoubtedly points to the origin of this invention: "The rectangular or cubic artificial blocks normally used are nearly always less stable than natural rocks of the same unit weight, and of random shape, used under the same conditions." This fact, observed after systematic studies and comparisons and both confirmed and amplified by scale model tests, obviously constituted a real challenge to the science of engineering. To take up the challenge, it was sufficient to state clearly the question: what would be the best form for an artificial protecting block? It was also necessary to have the tools needed to make a practical study of the different block shapes under consideration. These conditions were fulfilled in the Neyrpic Hydraulics Laboratory. It was obviously impossible to test a series of chosen shapes in a haphazard manner. On the contrary, one of the first steps taken by the laboratory technicians was to establish a list of the properties desired in the new block. A brief review of these properties is given below.

CHARACTERISTICS OF THE TETRAPOD

PERMEABILITY

Protecting revetments for marine structures should not be impervious because of the possible occurrence of internal pressures which may cause considerable disturbance to the structure. Systematic tests, made during the study of the Mers-el-Kebir naval base, have shown that an impervious revetment could be lifted almost bodily by internal pressures, even when the revetment is constructed of 400-ton blocks. In addition, a permeable revetment is desirable in order to absorb incidental swell and thus reduce wave overtopping and reflection. These two occurrences may constitute a considerable drawback to navigation and use of the harbor. Besides, waves overtopping a structure may attack its more or less unprotected inner slope and, in this way, cause a part of or even the whole structure to collapse. A new block must therefore permit construction of a pervious facing and the perviousness obtained must not be reduced or eliminated by subsequent alterations which may be

* Presented at the Third Conference on Coastal Engineering.

caused by sea action against the structure. This condition, therefore, eliminates blocks which may provide a continuous facing; and consequently, the ideal block should have as few large plane surfaces as possible.

ROUGHNESS

It is desirable for a marine structure revetment to have a rough surface so that it can dissipate the energy of incidental waves and slow down the water masses which tend to rise along the facing and to pass over the structure. The "friction" of water against the mound is thus increased. On the other hand, it is of advantage to increase "internal friction" of the mound by promoting the interlocking of the blocks of which it is made. These results can be obtained with blocks having projections which will ensure both a rough external facing and satisfactory interlocking of the blocks.

RESISTANCE

Finally the block must have maximum resistance, and therefore, the study was passed on to one of the laboratory's experts on the resistance of materials. It seemed difficult to satisfy all of these conditions. However, after various preliminary studies the outline took shape: a sort of sea monster with four tentacles which was patented under the name "tetrapod" (Fig. 1).

Hydraulic tests were made to check the properties of the new block and perfect its design; in this respect one of the essential points was to find the most suitable proportion between the length of the four prongs and the size of the body: the prongs could not be too short because the blocks would then not interlock properly and the revetment would be less stable, they could not be too long because then they would be too fragile. The correct design was finally established.

Once these general proportions were found, further details were settled by considering not only the resistance and hydraulic problems but also the ease of manufacture of the tetrapod casting forms. The result was to give prongs the shape of a truncated cone, of which the angle was determined so as to favor the wedging effect during the first settling movements they make after they are placed into position. The tetrapod with four equidistant prongs gives, to our knowledge, the best combination of those qualities required for a block used to protect marine structures.

A revetment constructed of tetrapods is pervious, and therefore, on the one hand there is no risk whatever of under-pressure, and on the other, wave energy is satisfactorily absorbed and overtopping is reduced. In addition the block projections provide the revetment with the required roughness.

But before deciding that the tetrapod was the perfect answer, the block in its final shape was again the subject of other experiments. Tests of resistance were made. Sample blocks of several weights were cast in concrete,reinforced or not,and of several mixes, and then tested for resistance to shocks (Fig. 2). Then followed systematic scale model tests on the behavior of the blocks when they undergo prolonged attack by the sea. The tetrapod came through all of these tests in a completely satisfactory manner and it was shown to have some great advantages over other blocks in current use; i.e.,

a. The same degree of stability can be achieved with smaller unit weights and steeper facings;

b. The structures can be built lower because overtopping is reduced;

c. Maintenance and repairs are relatively easy as is the case of pell-mell construction.

Once these advantages of the tetrapod were established on scale models, it was deemed possible to use them in actual construction.

USE OF TETRAPODS

There are obviously several ways in which tetrapods may be used for building new structures, or for strengthening and repairing existing ones. In each case where tetrapods are used it is necessary to make a special study,and just as it should be done for any large marine project, it is often helpful to make a scale model study,as this will give the best solution from both technical and economical points of view.

Nevertheless, by referring to experience acquired from many scale model studies, it is possible to specify the most frequently recommended and particularly favorable designs. In fact, two of these are in the majority: the "mound" and the "double layer". A mound of tetrapods is generally used when a vertical structure is to be protected by placing against its seaward face a fill of natural or artificial rocks having a sufficient weight to resist wave action (Fig. 3).

This fill must satisfy several requirements:

a. It protects the wall against any direct wave attack which may endanger its stability;

b. It reduces overtopping;

c. It reduces wave reflections.

If the water at the structure is not too deep, it is often desirable to design this protection in the form of a mound composed completely of tetrapods. This type of construction provides the maximum use of the tetrapods properties, i.e, high stability is obtained, although the seaward slope is very steep (about 1:1). Despite this slope, the mounds have low reflection coefficients because they are pervious and round. To avoid "slaps" against the upper part of the wall,

Fig. 1. A tetrapod

Fig. 2. A four-ton tetrapod
after a 3 ft. fall.

a slight berm is often provided at the upper part of the tetrapod mound. The berm obviously increases the volume of the mound a small amount.

When large water depths are reached, the mound, whose volume increases roughly as the square of the depth, would become too expensive if it was made entirely of tetrapods. The alternative is therefore a core of natural rocks and a revetment of tetrapods. The best design of this revetment is the "double layer" previously mentioned. The part of the core laying directly under the facing should be constructed of rocks having a unit weight not less than 1/10 of that of the tetrapods in order that these rocks may not pass through the gaps. This foundation cannot, as a rule, have a slope as steep as 1:1, because this would be destroyed by any sea disturbance occurring before the tetrapods are placed into position. Thus it is necessary to give a gentler slope to the facing; this will increase the volume, and consequently the expense, but as against this, the method will allow tetrapods to be replaced by small, less expensive natural rocks and will finally result in an economy in comparison to a complete mound of tetrapods. On such a foundation the double layer is built as follows:

a. A first layer of tetrapods is laid down so that three prongs rest on the mound with the fourth pointing outward. This disposition is not strict but simply indicates the general orientation of the tetrapods in the first layer.

b. The second layer is then laid down pointing in the opposite direction to the first, i.e., with one prong directed inward. Tetrapods in the second layer have a natural tendency to settle in this manner, but they are, of course, still less regularly placed than the first because these rest on a relatively plane foundation.

c. When the second layer has been laid down, there is a rather compact revetment which has few recesses open for additional tetrapods. Thus it is difficult to add a few more tetrapods, which would in any case be useless or even harmful. Scale model tests have shown that such additional tetrapods are the first to be torn away by wave action when this reaches the limit of resistance of the facing.

Therefore, the only way to increase the thickness of the revetment would be by constructing a second "double layer". But then the first becomes useless and it is not advisable to use such a method because the upper "double layer" will then rest on a less even and more expensive base than that made of small rocks.

The "double layer" revetment therefore constitutes a well adapted constructional method. It has even better stability than a mound of tetrapods. This can be explained by the fact that tetrapods in a mound are placed quite haphazardly, while those in the "double layer", although largely laid down at random, nevertheless have a general rational organization; because of this tetrapods on the surface offer the smallest possible hold to lifting forces and have the maximum points of contact

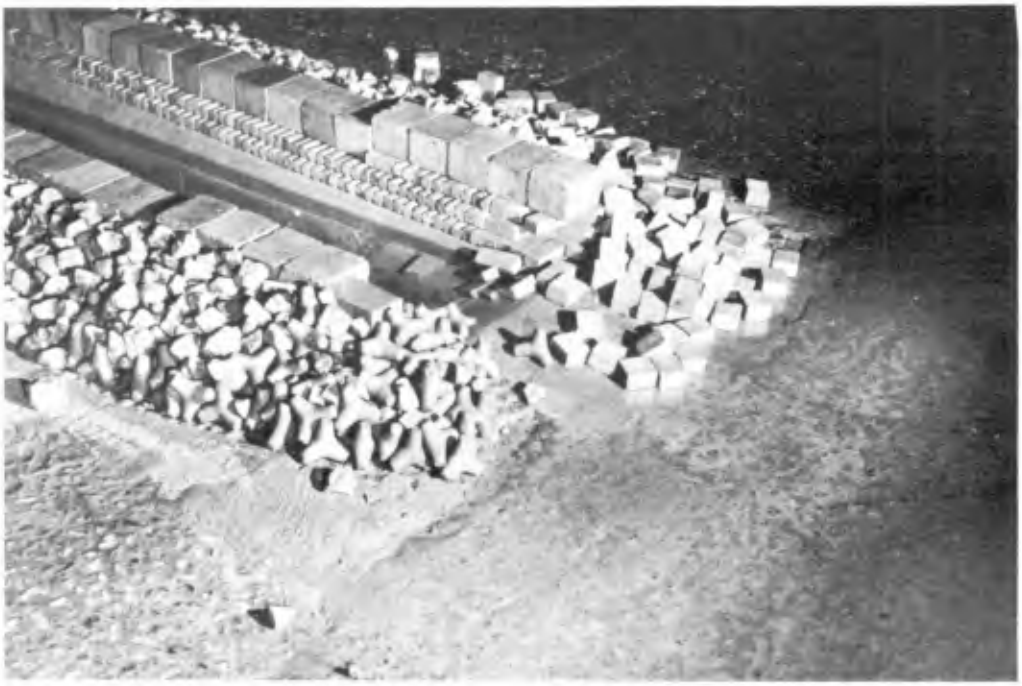

Fig. 3. Comparative study between a tetrapod mound and classic blocks used for protecting structures.

Fig. 4. Study in a wave channel for a revetment made of a double layer of tetrapods.

with those in the second layer.

When the double layer is used, thickness of the revetment varies with the unit weight of the tetrapods (Table 1). For a given revetment surface, the required quantity of these blocks at a determined unit weight is almost constant. As a first approximation, the percentage of space in a mound of tetrapods is approximately 50 percent or a little higher. The quantity of tetrapods required to fill a given volume can thus be calculated both easily and quickly.

Table 1

Characteristics for use of tetrapods in double layers

Unit weight* T	50	40	32	25	20	16	12.5	8	4	2	1	0.5	0.25
Volume m^3	20	16	12.5	10	8	6.3	5	3.2	1.6	0.8	0.4	0.2	0.1
Height of tetrapod mm	4155	3860	3550	3300	3060	2830	2620	2260	1790	1420	1130	900	710
Thickness of double layer mm	5600	5200	4900	4500	4100	3800	3500	3000	2400	1900	1500	1200	950
Quantity on 100 m^2	14	16	19	22	26	30	35	47	74	120	190	300	470

* Density of concrete = 2.5
T (tonne) = 0.984 long ton = 1.102 U.S. ton
m^3 = 35.314 cu. ft.; mm = 0.039 inch; m^2 = 10.764 sq. ft.

In very deep water it is not necessary to construct a protection of tetrapods down to the sea-bed. In such cases, the toe of the facing rests on a rock footing built up to a certain level (Fig. 4). This level obviously depends on the amplitude of waves which attack the structure concerned and on the unit weight of the rocks available. Another equally important question is to establish the height above sea level to which the tetrapod protection must be built. This height also depends on the wave amplitude and on the amount of overtopping which can be allowed.

Systematic studies are now in progress and it will soon be possible to give rapidly a rough estimate for any project. However, it will always be of value to make a scale model study of a projected structure in order to find the most economical and the most technically satisfactory solution.

In particular, the most important question is to decide the proper unit weight for tetrapods so that they can resist waves of a certain amplitude with a given slope. The Neyrpic Hydraulics Laboratory has

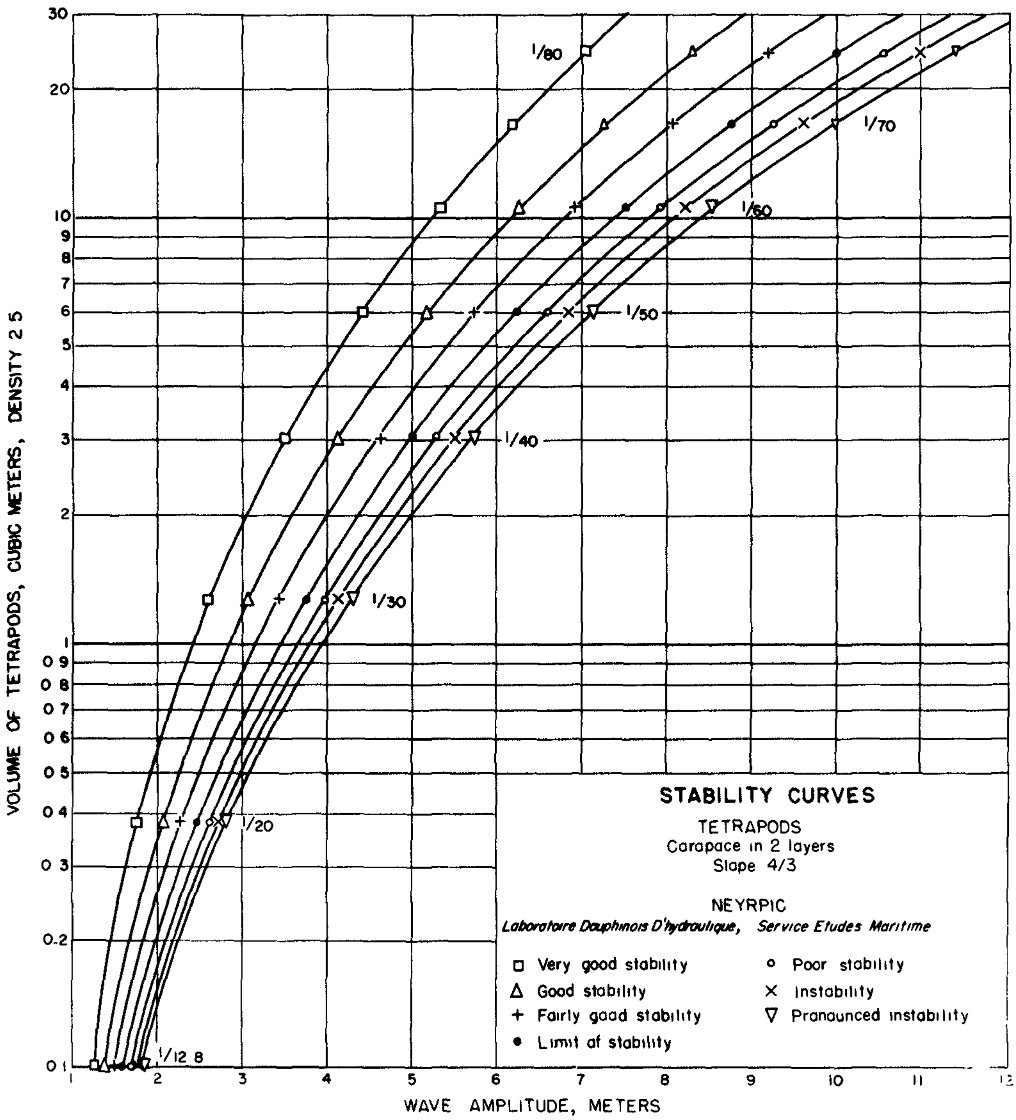

Fig. 5.

made a series of tests to find the necessary rules for determination of the unit weights necessary. These tests do not as yet cover all possible cases because a really complete series of tests is very long and expensive to conduct. The results obtained to date nevertheless allow useful conclusions in a great many problems. Graphs plotted from these tests (Fig. 5) provide a good approximation for proposed projects and enable possible special studies to be greatly curtailed.

CONCLUSION

Several structures using tetrapods already have been built, or are in the course of construction. Tetrapods have produced the results expected of them and experience has shown that they have the qualities anticipated. For instance, the first structure for which tetrapods were used, the sea-water intake for the Roches Noires thermic power station at Casablanca, North Africa, has withstood four hard winters. The 15-ton tetrapods protecting the pier-heads have not moved, although they are exposed to formidable Atlantic waves; when photographs taken at two or three years interval are examined, all the blocks above low water mark are seen to be in their original position.

In all the cases studied to date, tetrapods have proved themselves more efficient and economical than ordinary rectangular or cubic blocks. They have undoubtedly enabled marine construction engineers to widen their field of activity, because the tetrapod offers a new answer to the problem of protecting structures in the sea.