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Part 1
BASIC OCEANOGRAPHIC INFORMATION

CHAPTER 1

WAVES AND BREAKERS IN SHOALING WATER

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Wiegel and Johnson (1950) summarized useable wave theories for deep and shallow water. Mason (1950) discussed waves in shoaling water and compared theoretical predictions with measurements. The theories are shown to apply, within practical limits, to periodic systems of deep water waves, and to periodic waves progressing over a shoaling bottom to wave positions near the breaking point.

Near and at the breaking position the wave features are not predicted from theory with desired accuracies and measured characteristics are used to describe breakers. The available measurements are limited and do not show the effects of variables such as the beach slope.

Recent work at the University of California has resulted in informati on the limits of applicability of the linearized wave theories as applied to wave transformation in shoaling water, and on breaker shapes and motion including the effect of beach slope.

DEFINITIONS OF TERMINOLOGY AND SYMBOLS

The description of breakers involves the use of terminology which may not be consistent in all wave and breaker studies. In order to avoid mis-interpretation or confusion, the terminology and symbols as shown on Figure 1 are adopted for this discussion. Subscripts are used with the symbols to designate particular locations of the variables. Subscript o refers to dee water wherein the wave form is not affected by the proximity of the bottom. Subscript b refers to the breaker point.

EXPERIMENTAL PROCEDURE

Two sets of experiments were performed, one in which the breaker detai were examined, the other in which wave height transformations in shoaling water were obtained. Both were made in the same laboratory wave channel, Figure 2, which consists of a channel one foot wide by three feet deep rec-tangular cross-section of smooth side walls and smooth bottom with a workin length of 54 feet. Smooth, impervious plane sloping bottoms of reinforced plywood or metal sheeting were placed in the channel to give desired beach slopes. In some arrangements a seaward toe was used with a steeper slope than the normal beach to give a longer constant depth portion of the channe All beaches were sealed at the junction of the beach bottom and side walls.

Waves were generated as a continuous periodic train by a hinged plane flap oscillating with a constant period. The period and amplitude of the

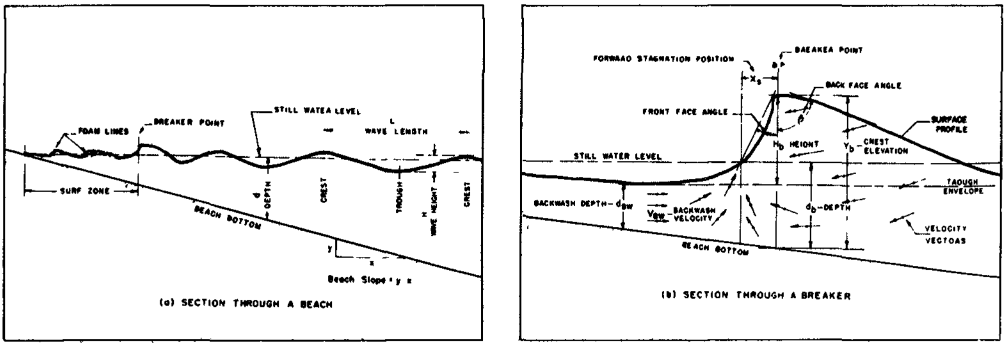

Fig. 1 Wave and Breaker Terminology.

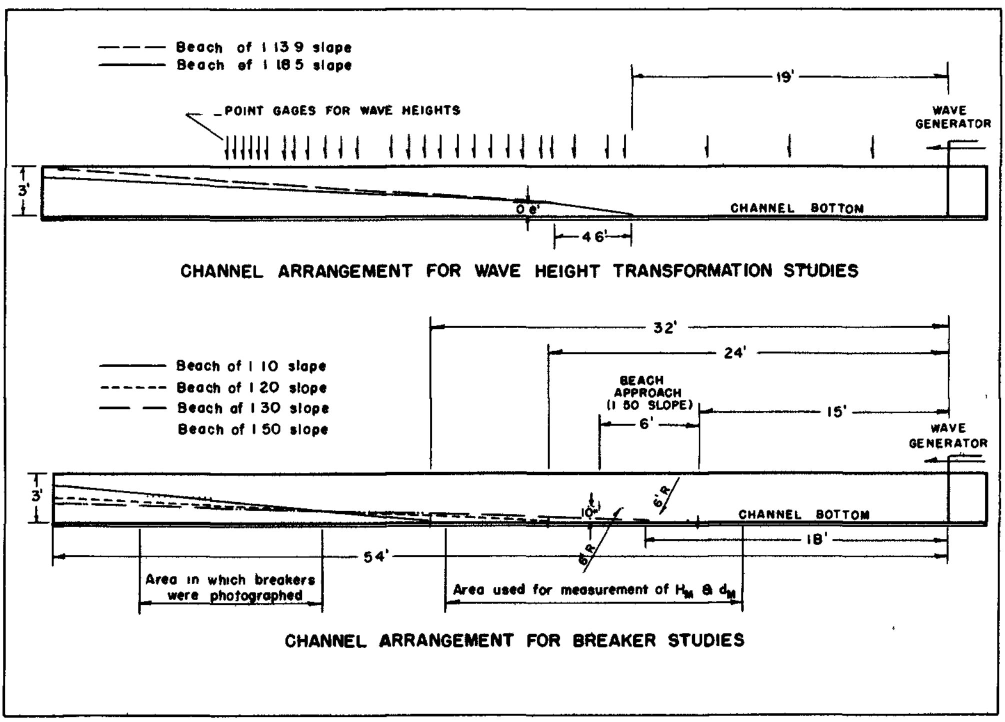

Fig. 2.

flap were adjustable to enable a range of initial wave conditions. For the breaker studies the flap was driven through top and bottom independently adjustable cranks to permit a close approximation of a shallow water wave at the wave generator.

Measurements were made as follows:

(1) Wave height transformation studies. Crest and trough positions at various stations as diagrammed in Figure 2(a) were obtained with vertical point gages. Depth readings at each of the stations also were obtained. The wave period was obtained from the timed oscillations of the wave generator.

(2) Breaker studies. Wave heights in the constant depth portion of the channel were obtained from point gage readings of crest and trough elevation. Movies of the breaker region were taken through the glass walls of the channel with the camera axis at the still-water level. To obtain the kinematics of the water movement in the breaker, particles of a mixture of xylene and carbontetrachloride, with zinc oxide for coloring, with a specific gravity corresponding to that of the water, were introduced in the breaker region. The point to point movement of the particles then was recorded on the movies, from which each particle velocity was obtained by superposition of the projected movie frames to give distance moved and time interval of movement. Complete velocity fields were mapped for each wave for successive positions before and during breaking. The breaker surface profile transformations were also obtained by this procedure.

The limitation of the length of the laboratory wave channel restricted the investigation to beach slopes of 1:50 or steeper. In addition, in order to cover a range of characteristic waves with appreciable heights for reasonable measurements of vertical displacements, the majority of the waves were not generated as deep-water waves due to the depth limitations of the channel. The defining incident waves, characterized by the deep-water wave steepness, the ratio of the deep-water wave height to the deep-water wave length*, were evaluated from the wave heights measured in the constant depth portion of the channel with application of wave height transformation information (Mason, 1950) to obtain deep-water wave heights.

EXPERIMENTAL RESULTS - WAVE HEIGHT TRANSFORMATION STUDIES

Figures 3, 4, 5, and 6 show the measured wave heights related to the still water depth. The small amplitude linear theory corrected for channel wall and bottom frictional effects is shown to be applicable for the test conditions of Runs 1 and 7. The theory for the remainder of the laboratory waves does not predict the measured results with sufficient accuracy for laboratory experimentation. Two conditions contribute to the discrepancy.

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*Deep-water wave steepness = H_o/L_o, or H_o/T^2 where T is the wave period. With L_o in feet and T in seconds, $L_o = 5.12\ T^2$.

The short length of channel from the generator to the initial height measurement stations introduces some doubt that the wave form for the longer waves was completely established at the first few height measuring stations. Since these heights were used as references for the initial wave conditions, a lack of agreement between measured and predicted wave heights along the channel may be expected. In addition, the steep beach toe, as shown on Figure 2, may have produced excessive reflections, particularly for the longer waves, and consequent erroneous wave height measurements.

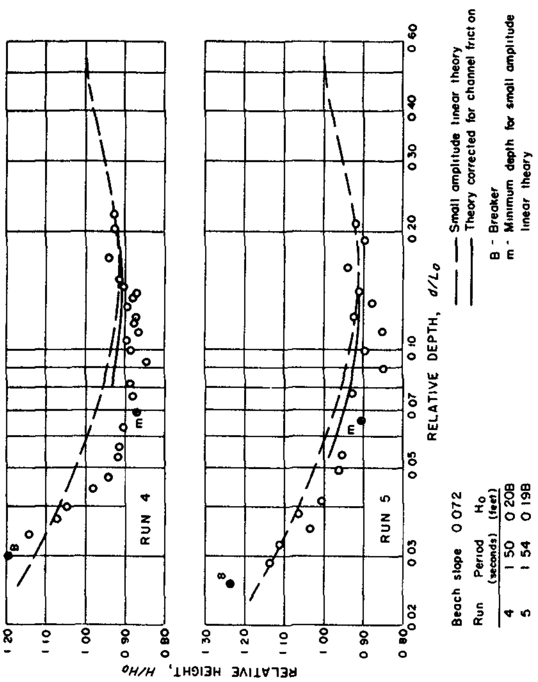
One feature is definitely established in the wave height transformation. The initial portion of the transformation shows a decrease of wave height as the depth becomes smaller. The small amplitude linear theory predicts a change of this nature. The theory then shows a gradual increase of wave height as the depth further decreases. The measurements follow the predicted trend of the wave height decrease with approximately the same rate of change. However, the measured rate of increase of wave height with decreasing depths is much greater than that predicted. The depth at which the disagreement becomes marked is noted on Figures 3, 4, 5, and 6. The choice of limiting depth is admittedly somewhat arbitrary. Results are shown on Figure 7. A curve is drawn through the results as a tentative limiting depth of applicability of the small amplitude linear theory.

EXPERIMENTAL RESULTS: BREAKER CHARACTERISTICS

Results obtained from the breaker studies included the complete geometry of the wave transformation in the region of breaking, and also the complete velocity field from the water surface to the beach bottom at increments of the wave and breaker position in the region of breaking. The complex nature of the transformation of a wave into a breaker, with the dependence of the transformation upon initial wave steepness and beach slope, precludes any simple presentation of the complete experimental results. Certain features pertaining to the breaker point can be correlated. These include the variables as listed on Figure 1(b). Correlations are made as a function of deep water wave steepness and beach slope.

The same limitation was present in the breaker experiments as was present in the wave height transformation experiments, i.e., most of the waves which were generated were not deep water waves in the constant depth portion of the channel. Deep water wave heights were computed from measured wave heights in the constant depth portion of the channel using the transformation results from small amplitude theory. Frictional damping of the wave train between the incident wave measurement station and the breaker was included to obtain the equivalent undamped deep water wave height. All results are shown as a function of the deep water wave characteristic, H_o/T^2, where H_o is the deep water wave height and T is the wave period.

Breaker heights are shown in Figure 8 for beach slopes of 1:10, 1:20, 1:30, and 1:50. The generated waves were between 0.1 to 0.4 feet high, and with periods between 0.8 to 2.5 seconds. Some scatter is noticed for the



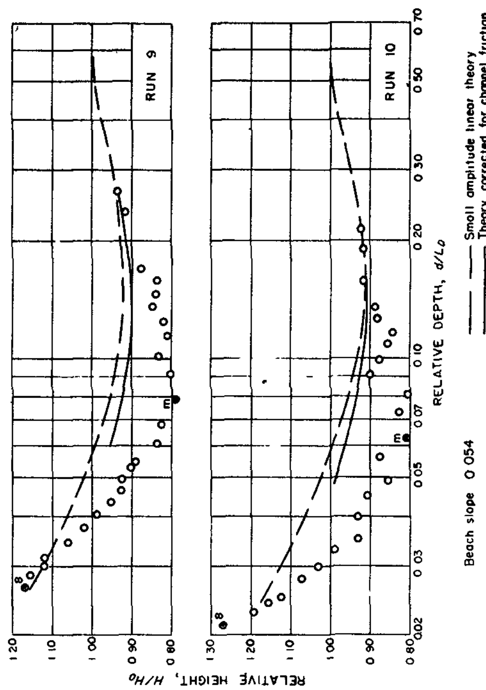


FIGURE 4

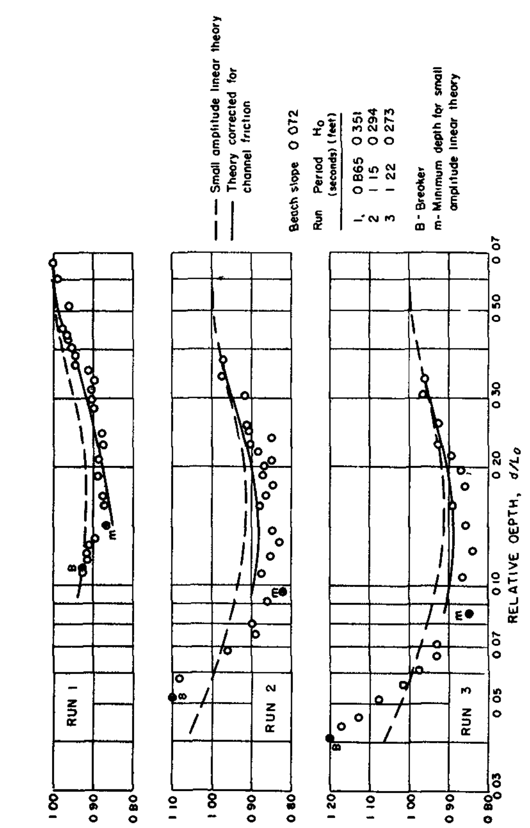


FIGURE 3

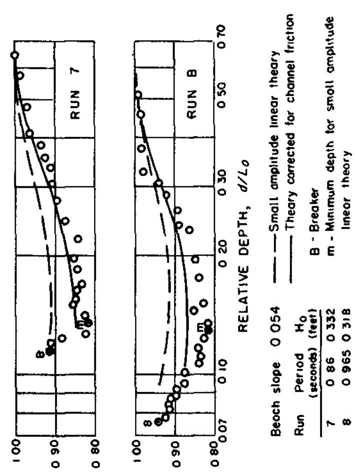


FIGURE 5

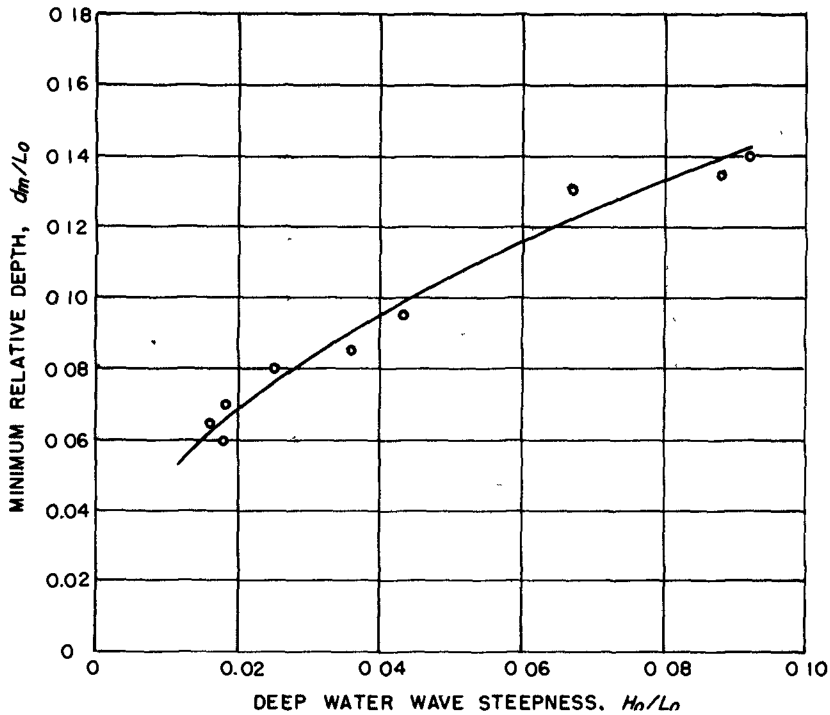


Fig. 7. Tentative limiting depth for application of small amplitude linear theory wave height transformation.

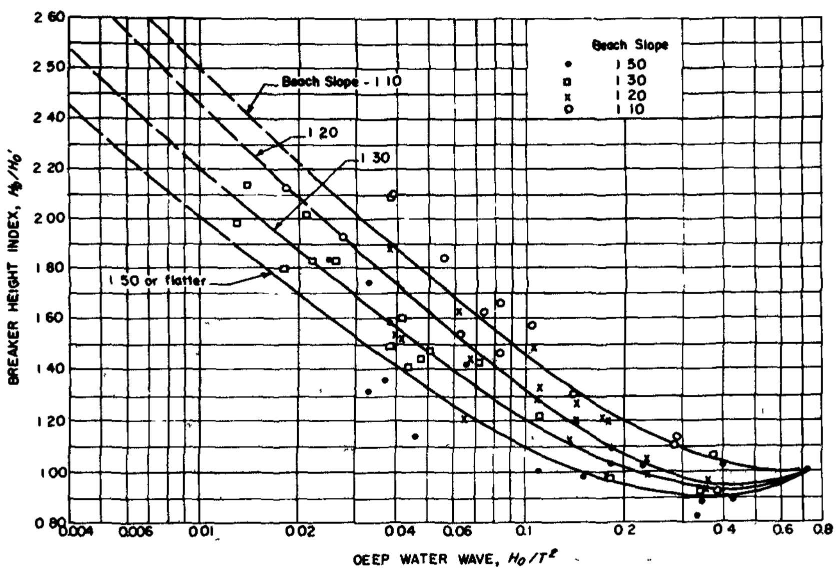


Fig. 8. Breaker height index as a function of deep water wave and beach slope.

results of each beach slope. The scatter is larger than that accounted for by the measurement techniques. No explanation is offered at present to reconcile the scatter.

However, the beach slope effect is apparent on the breaker heights since the same range of wave trains in height and period was generated with each beach slope. Curves have been drawn as representative averages of the results from each beach slope. Breakers on the 1:10 slope are approximately 40% higher than those on the 1:50 slope.

Figures 9, 10, 11, and 12 show breaker geometrical features of depth at breaking, crest height, backwash depth, and forward stagnation position. The breaker height was used to obtain the various dimensionless geometric ratios of the curve ordinates as shown since the breaker heights were measured directly and in the same manner as the other shape variables. Although some experimental scatter is noted, the results describe continuous average curves. Curves also are shown relating the breaker shape variable to the deep water wave height by combining the curves from the plotted results with the curves of Figure 8 - for example, $d_b/H_o = (d_b/H_b)\ (H_b/H_o)$.

Figure 13 shows the breaker face angles. Figure 14 shows the backwash velocity and the crest velocity in dimensionless form with the crest height as representative of a shallow water wave velocity evaluated from a water depth.

Some comments should be made relative to the evaluation of the results which appear in the above figures. The breaker point is, to a certain degree, a matter of judgment which depends upon the type of breaker which is formed. For "spilling" breakers, in which the crest became unstable in a mild fashion with the appearance of "white water" at the crest which expanded down the front face of the breaker, the picture preceding that in which the first white water appeared was taken as the breaker point. For "plunging" breakers, in which the crest overshot the body of the wave to project ahead of the wave face, the picture in which the front face at the crest was vertical was taken as the breaker point. For "surging" breakers, in which the front face of the wave became unstable over the major portion of the face in a large scale turbulent fashion, the picture preceding this action was taken as the breaker point. The movies, from which the results were obtained, were taken at approximately 60 frames per second. The time interval of 1/60th of a second permitted a reasonable approximation of the breaker point.

The depth, crest elevation, backwash depth, and front and back face angles were easily determined from the selected pictures. The forward stagnation point, which was determined from the particle movements, was noted to occur at approximately the intersection of the still water line and the front face of the wave. Backwash velocities were obtained by averaging all particle velocities in the region of minimum depth in the backwash. Crest velocities were obtained from the gradient of the crest position-time history. Small surface irregularities influenced the selection of the crest position in any one picture.

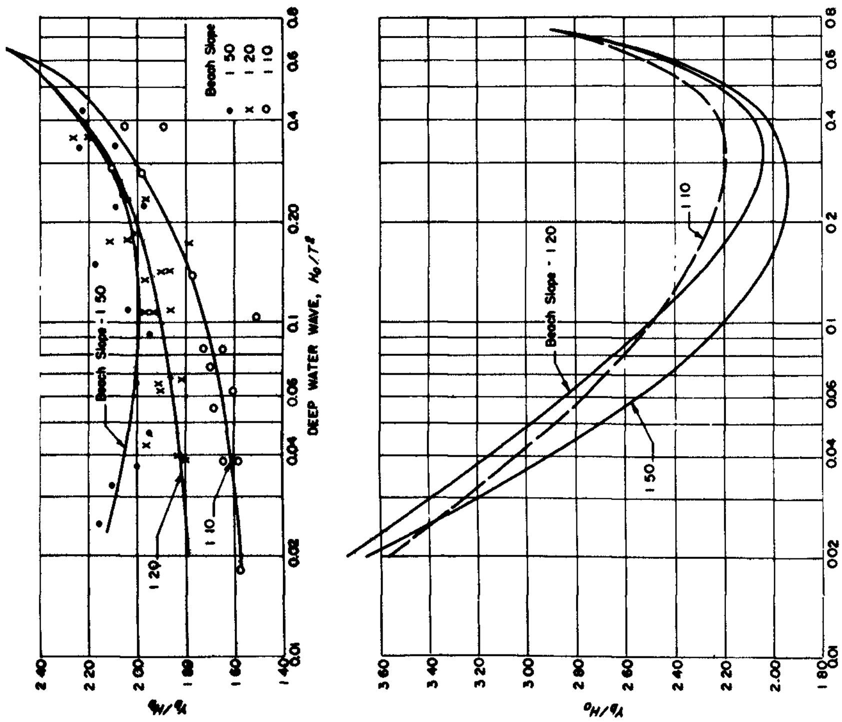


Fig. 10. Breaker Crest Index.

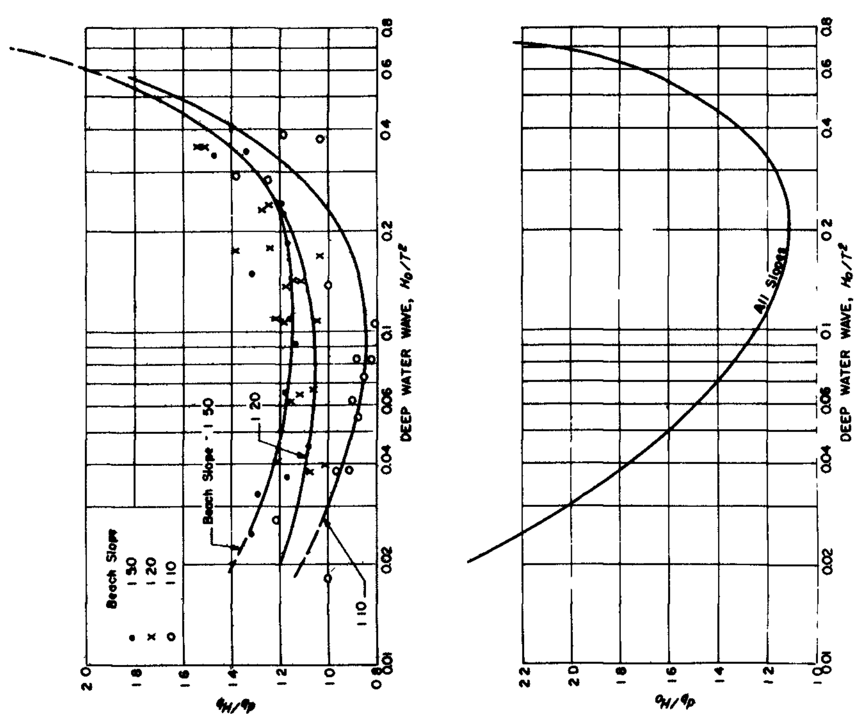


Fig. 9. Breaker Depth Index.

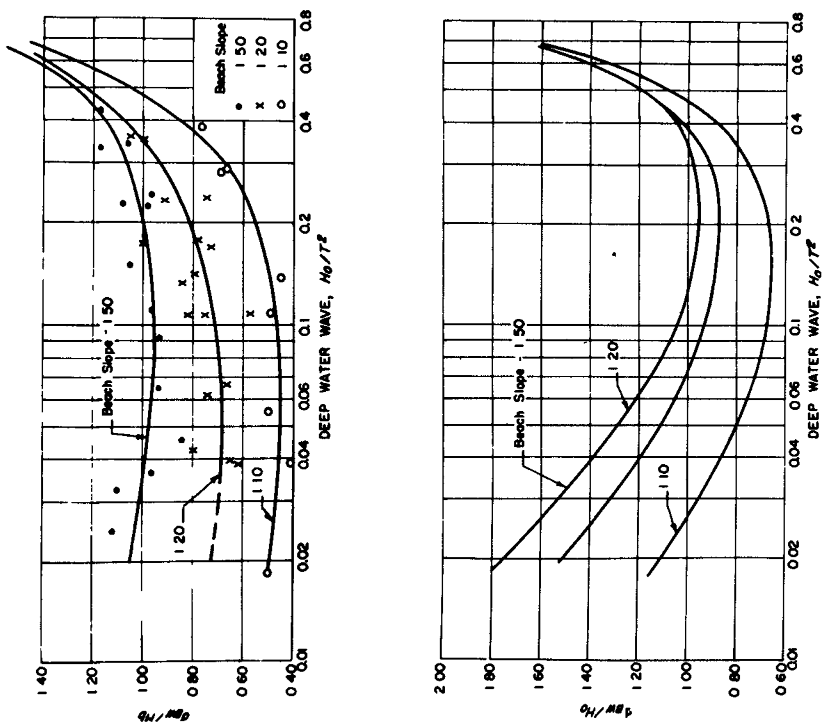


Fig. 11. Backwash depth index.

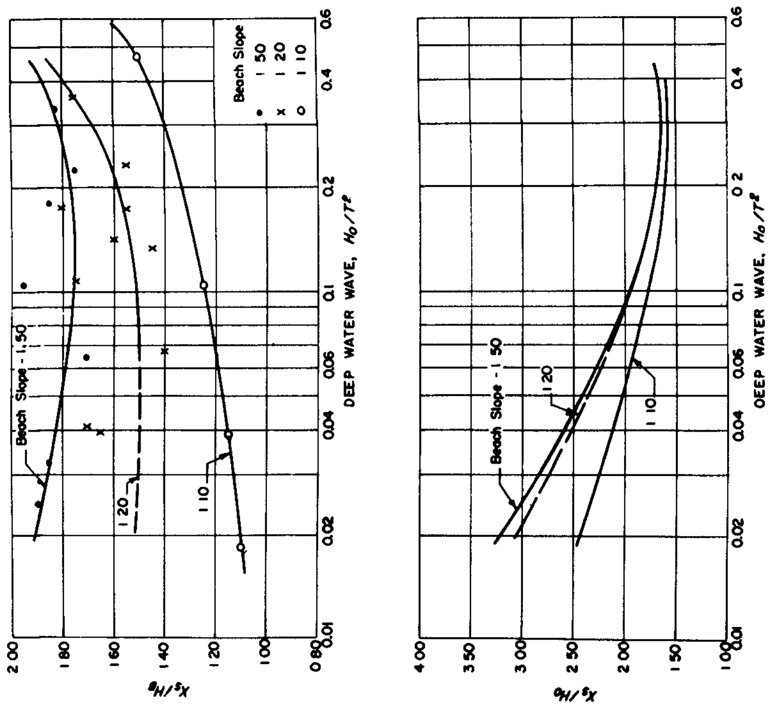
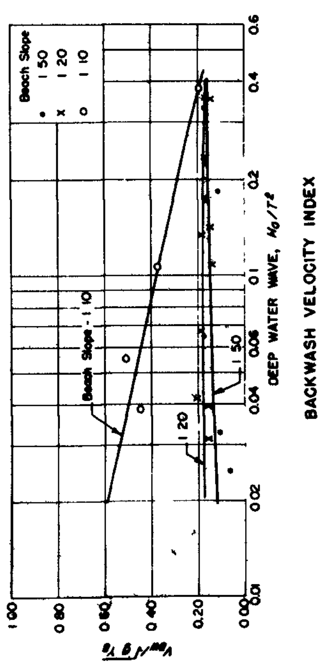


Fig. 12. Forward stagnation position index.



BACKWASH VELOCITY INDEX

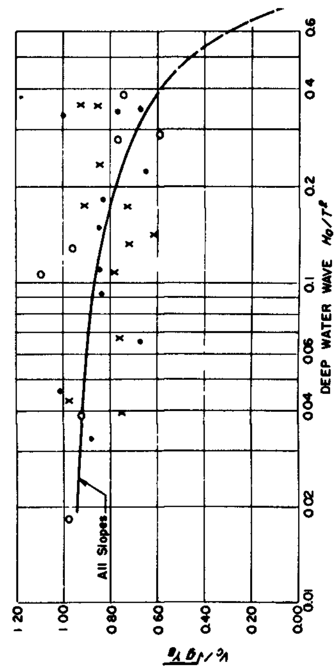
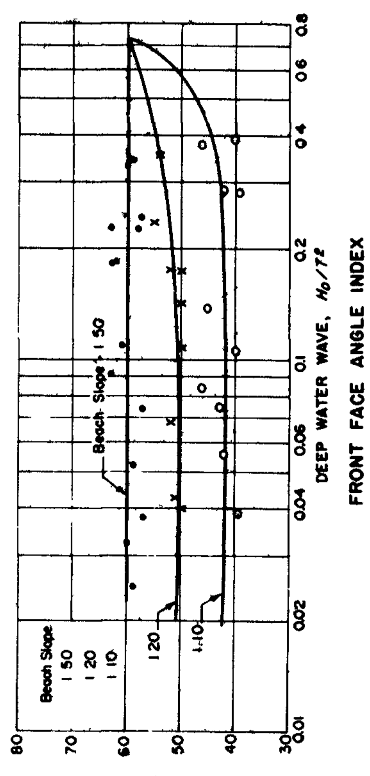


Fig. 14. Crest velocity index.



FRONT FACE ANGLE INDEX

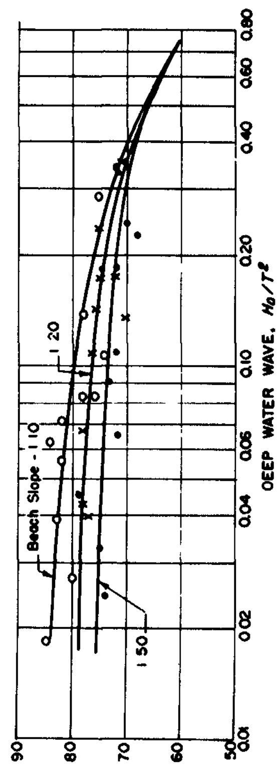


Fig. 13. Backface angle index.

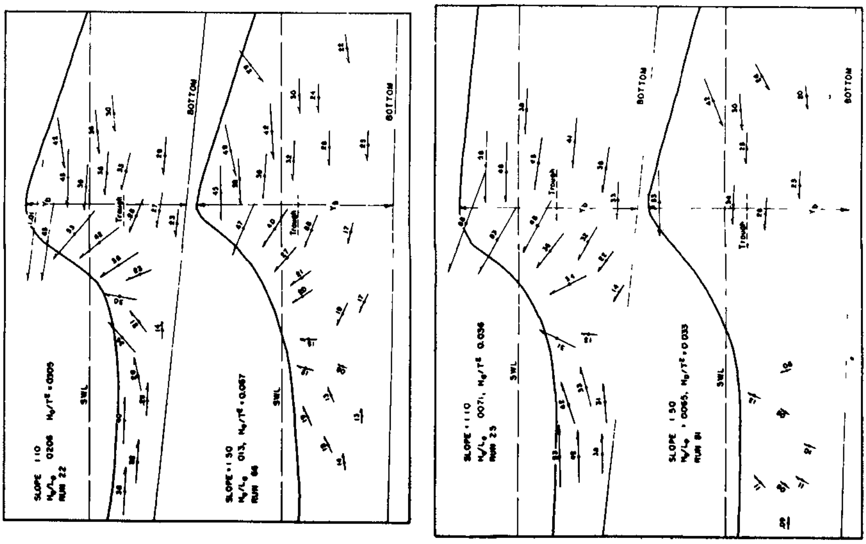


Fig. 16. Kinematics at breaker point.

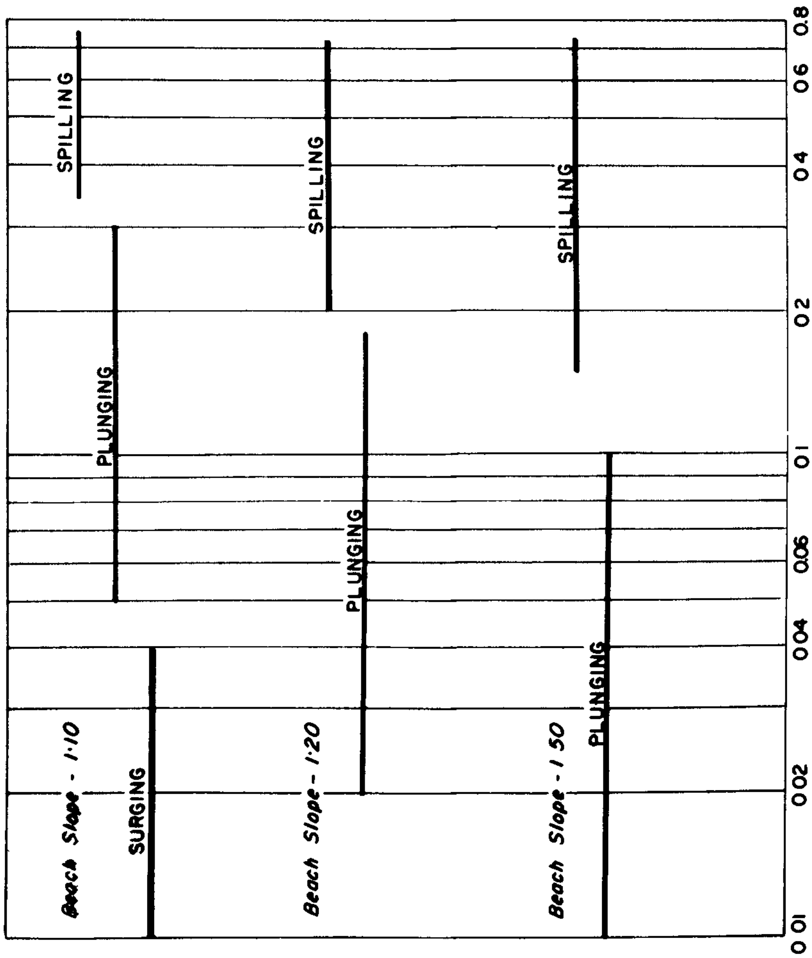


Fig. 15. Breaker type index.

For a given wave train defined by the deep water wave height and period, on a steep beach as compared to a flat beach, the breaker is higher, breaks in the same depth of water with a higher crest elevation, has a flatter back face and a steeper front face, and has a smaller depth in the backwash with a higher backwash velocity.

The backwash, which is a function of events preceding a particular breaker, is a factor in the breaking action. High backwash velocities retard the base of the wave with a consequent tendency to promote a "plunging" breaker. At large values of deep water wave steepness, the breakers on all beaches were "spilling". At smaller values of deep water steepness the waves tend to plunge with greater tendencies on the steeper slopes. At the extreme lower values of the deep water wave steepness, particularly on the 1:10 slope, the breaker tended to "surge". A breakdown of observed spilling, plunging and surging tendencies is shown in Figure 15.

Other features of the breaker, particularly the kinematic field, may be noted in Figure 16. All breakers which were studied showed essentially the same general kinematic field, except for the differences as noted in the fore part of the breaker in terms of the backrush and forward stagnation point.

The laboratory waves were of uniform period and geometry. Natural waves seldom correspond to this condition. What effect the previous and following wave histories have upon a single wave under consideration, if the wave train is irregular, is not known. The effect of bottom friction and percolation also should be included for waves on natural beaches.

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Wiegel, R.L. and Johnson, J.W. (1950), Elements of Wave Theory; Proc. of the First Conf. on Coastal Engineering, pp. 5-21.

Mason, M.A. (1950), The Transformation of Waves in Shallow Water; Proc. of the First Conf. on Coastal Engineering, pp. 22-32.

CHAPTER 2

THE SOLITARY WAVE

Its Celerity, Profile, Internal Velocities and Amplitude Attenuation in a Horizontal Smooth Channel

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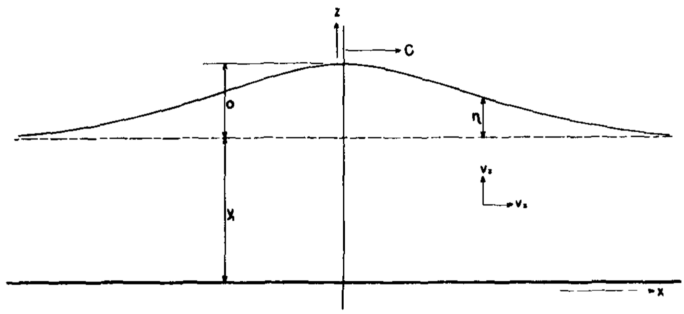
INTRODUCTION

The solitary wave consists of a single elevation of water above the originally undisturbed level as shown in Figure 1. It is translatory, a passing wave causing a definite net horizontal displacement of the liquid. While the characteristics of oscillatory waves depend on wave length as well as wave height and water depth, the solitary wave is apparently described completely by the wave height and water depth so long as attenuation due to friction is unimportant.

It is well known that a close analogy exists between the characteristics of the solitary wave and shallow water waves of long wave length. In recent years this analogy has been used in attempting to predict the characteristics and effects of oscillatory waves moving into shoal water. For this purpose, it has been necessary to use theoretically derived characteristics of the solitary wave, there being very little experimental information beyond J. Scott Russell's (1)(2)* original studies. As a result, there has been a renewed interest in completely determining the properties of the actual wave and checking the rather varied results of the several theoretical analyses.

A summary of the important theoretical analyses for waves moving over a horizontal bottom is given in Table I. The equation for celerity, C, found by Boussinesq (4), Rayleigh (5) and Stoker (14) is the same found empirically by Russell. Keulegan's (12) viscous attenuation equation is for a smooth bottom and agrees with Russell's experimental findings. Munk's (13) work is a restatement of McCowan's analysis using simplifying dimensionless parameters. The coefficients M and N are given graphically in Table I. All the analyses conclude that the wave characteristics are represented by a unique function of the undisturbed depth y_1 and the amplitude-to-depth ratio a/y_1. Aside from these similarities, the several solutions differ in various respects.

*Numbers in parentheses refer to references at end of this chapter.



a - wave amplitude, feet.

a/y_1 - ratio of wave amplitude to undisturbed depth.

b - constant coefficients of series in theory by McCowan.

B - width of wave channel, feet.

C - wave celerity, ft./sec.

k - coefficient of series in theory by Davies.

m - constant coefficient of argument in tangent series by McCowan.

M - dimensionless coefficient, my_1, of McCowan's theory as expressed by Munk.

N - dimensionless coefficient, mb, of McCowan's theory as expressed by Munk.

P - wetted perimeter of channel cross-section, ft.

v_x - x - component of instantaneous unsteady motion fluid element velocity, ft./sec.

v_z - z - component of instantaneous unsteady motion fluid element velocity ft./sec.

x - horizontal coordinate at channel bottom, with origin at axis of symmetry of wave, feet.

X - dimensionless horizontal coordinate in theory by McCowan as expressed by Munk, x/y_1.

y_1 - undisturbed water depth.

z - vertical coordinate at axis of symmetry of wave with origin at bottom of channel, feet.

Z - dimensionless vertical coordinate in theory by McCowan as expressed by Munk z/y_1.

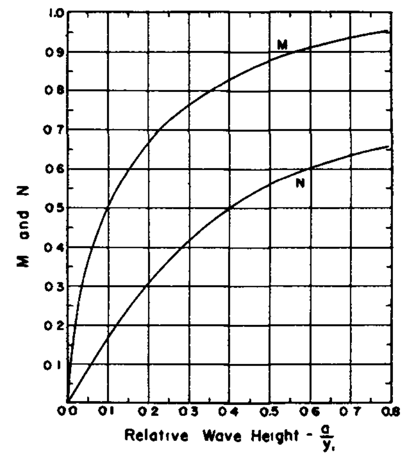
z_o - maximum value of $z = a + y_1$.

η - $(z - y_1)$, elevation of water surface above undisturbed surface level, $\eta_{max} = a$.

ϕ - velocity potential in field of fluid motion.

Fig. 1. Definitions and notations.

TABLE I

INVESTIGATOR	CELERITY	PROFILE	FLUID ELEMENT VELOCITY	ATTENUATION
St Venant (3)	$C^2 = gy_1\left(1+\frac{3a}{4y_1}\right)$			
Boussinesq (4)	$C^2 = gy_1\left(1+\frac{a}{y_1}\right)$	$\eta = a\,\mathrm{sech}^2\frac{x}{y_1}\sqrt{\frac{3a}{4y_1}}$	$v_x = \sqrt{\frac{g}{y_1}}\left[\eta-\frac{\eta^2}{4y_1}+\left(\frac{y_1^2}{3}-\frac{z^2}{2}\right)\frac{d^2\eta}{dx^2}\right]$ $v_z = -z\sqrt{\frac{g}{y_1}}\left[1-\frac{\eta}{2y_1}\frac{d\eta}{dx}+\frac{1}{3}y_1^2-\frac{z^2}{2}\frac{d^3\eta}{dx^3}\right]$ $U = C\frac{\eta}{z}$	
Rayleigh (5)	$C^2 = gy_1\left(1+\frac{a}{y_1}\right)$	$\eta = a\,\mathrm{sech}^2\frac{x}{y_1}\sqrt{\frac{3}{4}\frac{a}{a+y_1}}$		
Mc Cowan (7)	$C^2 = \frac{g\tan my_1}{m}$ $C^2 = gy_1\left(1+\frac{a}{y_1+19/12\,a}\right)$	$\eta = \frac{a}{\tan\frac{mz_0}{2}}\frac{\sin mz}{\cos mz+\cosh mx}$ $\eta = a\,\mathrm{sech}^2\frac{x}{y_1}\sqrt{\frac{3}{4}\frac{a}{y_1+19/12\,a}}$	$v_x = \frac{Cma}{\tan\frac{mz_0}{2}}\frac{1+\cos mz\cosh mx}{(\cos mz+\cosh mx)^2}$ $v_z = \frac{Cma}{\tan\frac{mz_0}{2}}\frac{\sin mz}{(\cos mz+\cosh mx)^2}$	
Stokes (6)	$C^2 = gy_1\frac{\tan my_1}{my_1}$			
Weinstein (8)	$C^2 = gy_1\left[1+\frac{a}{y_1}-\frac{21}{20}\left(\frac{a}{y_1}\right)^2\right]$			
Starr (9)	$C^2 = gy_1\left(1+\frac{3}{2y_1}\frac{\int\eta^2dx}{\int\eta\,dx}\right)$			
Davies (10)	$C^2 = gy_1\frac{\tan kCy_1}{kCy_1}$	$Cx = \phi+\frac{2}{3k}\sin^2 kCy_1\frac{\sinh k\phi}{\cosh k\phi+\cos kCy_1}+\cdot$ $Cz = \frac{2}{3k}\sin^2 kCy_1\frac{(\cosh k\phi-1)\tan\frac{1}{2}kCy_1}{\cosh k\phi+\cos kCy_1}+$		
Keulegan (12)				$\left(\frac{a}{y_1}\right)^{-\frac{1}{4}}-\left(\frac{a_0}{y_1}\right)^{-\frac{1}{4}} = \frac{1}{12}\left(1+\frac{2y_1}{B}\right)\sqrt{\frac{\nu}{g^{1/2}y_1^{3/2}}}\frac{x}{y_1}$
Munk (13)	$C^2 = gy_1\frac{\tan M}{M}$	$\eta = Ny_1\frac{\sin MZ}{\cos MZ+\cosh MX}$	$v_x = CN\frac{1+\cos MZ\cosh MX}{(\cos MZ+\cosh MX)^2}$ $v_z = CN\frac{\sin MZ}{(\cos MZ+\cosh MX)^2}$	
Stoker (14)	$C^2 = gy_1\left(1+\frac{a}{y_1}\right)$			

Results of theoretical analyses of the solitary wave.

In general, the theoretical analyses have been based on the assumptions of an infinite wave length, potential motion and a permanent wave form. The solutions have been in the form of various series expansions to different degrees of approximation. In addition, boundary condition assumptions affect the particular solution. An actual solitary wave, on the other hand, has a finite wave length and a non-permanent wave form since from any initial condition viscous attenuation always reduces the amplitude to zero.

Several questions arise then:

(a) If not permanent, can a wave, nevertheless, be described by a unique function of amplitude to depth ratio?

(b) To what degree will any existing theory describe an actual wave in which at best the motion is only approximately irrotational?

(c) Is agreement with theory dependent upon the rate of attenuation, i.e., can different results be expected for different bottom roughness and friction effects?

A program has been underway at M.I.T. under the sponsorship of O.N.R. to provide an experimental foundation for a solidification of the theory and to answer these particular questions. The program has had two phases, studies of waves in a channel with smooth walls and bottom such that friction effects were a minimum, and studies with definite roughnesses. This chapter is a report on the first phase.

SPECIFICATIONS AND EQUIPMENT FOR AN EXPERIMENTAL STUDY

In order to verify any one of the theoretical results, two things are required:

(a) It is necessary to duplicate as nearly as physically possible the basic boundary conditions inherent in the definition of the wave.

(b) It is necessary to make measurements with sufficient precision to differentiate between the various theoretical results.

Item (a) concerns the wave channel and generation of the wave. The channel used for these experiments is 13 inches deep, 16-1/2 inches wide and 32 feet long with lucite walls and bottom (Fig. 2). The friction effec which is a minimum with smooth walls and bottom can be varied by coating th bottom with sand or gravel. The width is large with respect to any boundar layer development assuring essentially two-dimensional motion. It has been found universally by investigators that to within the accuracy of experiments made to date, the properties of a wave are indifferent to the mode of generation after the wave has travelled some distance from its origin and providing secondary excitations and disturbances are avoided in the

Fig. 2. Wave channel with plunger type wave generator installed.

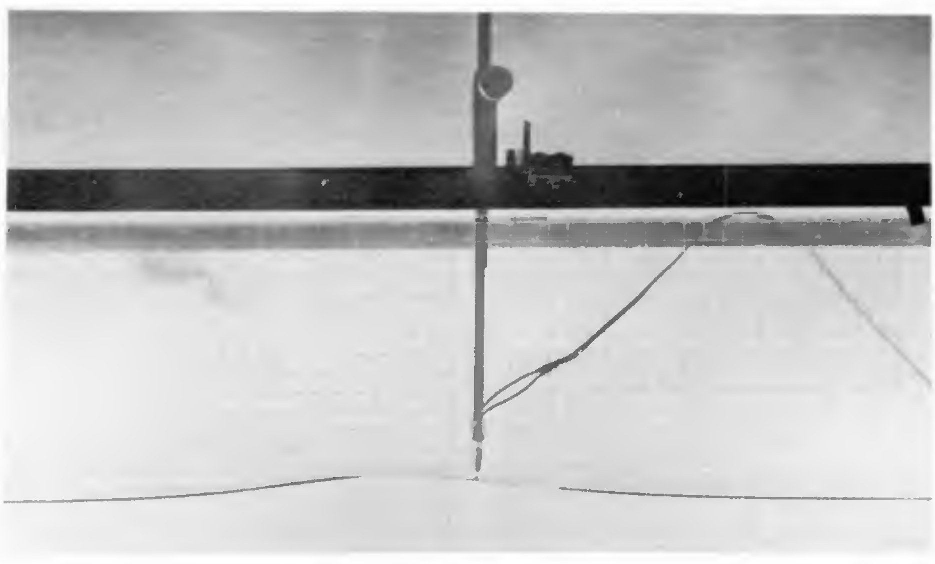


Fig. 3. Solitary wave propagating on 0.2 feet undisturbed depth.

generation. Because of the limited length of channel, extreme care was taken to generate a "clean" wave which would assume its characteristic shape in a short distance. Two methods were used. In one, the rapid immersion of a plunger displaced a quantity of water equal to the initial wave volume. As the wave surged away from the point of generation, it assumed the characteristic solitary wave form. In the second, more effective method, water in a small reservoir in one end of the channel was released into the channel by raising a sluice gate. This flood of water exerted pressure on a wall, perpendicular to the channel axis and free to move in a horizontal direction, causing it to move as a piston a short distance along the channel. Again the displacement equalled the initial volume of the wave.

The required precision of measurements depends upon the magnitude of differences between expressions for any wave property and upon the actual scale of the experimental waves.

A numerical comparison of the relations in Table I reveals that for the amplitude-to-depth ratio equal to 0.1, the minimum percentage difference between theoretical celerities is about 0.2%. This difference increases rapidly for a/y_1 greater than 0.1. The difference between the two closest theoretical profiles is a maximum of only about 3% of the amplitude when a/y_1 is 0.1. This difference also increases with increasing a/y_1. Fortunately too, the several profiles vary in such a manner that this extreme precision was not necessary for proper matching to experimental surface curves.

The celerity is determined by timing the wave over a distance of approximately 12 feet. An electronic device triggered by the wave at the beginning and end of the test distance starts and stops an electric timer and simultaneously flashes electronic speed lamps for two photographs of the wave. The travel time can be read from the electric timer to ±0.001 sec. From photographs, the initial and final positions of the wave axis and hence the travelled distance is established ±0.005 feet. The resulting accuracy of the celerity measurements is better than ±0.1%. However, as the celerity determined is actually the mean celerity for the timing distance of wave travel and as each celerity must be associated with a measured amplitude to be of any value, the accuracy of measurement of the amplitudes becomes the criterion to be considered in evaluating the experimental data. Also, since the amplitude attenuation over this distance is nearly linear and the magnitude decrease is small, it is felt that, within the experimental accuracy of measurement, the mean value of amplitude can be associated with the measured mean celerity.

The photographs are made through a grid inscribed on a sheet of transparent lucite as shown in Figure 3. From enlargements approximately one and one-half natural size, measurements of the profile and amplitude are obtained to within ±0.001 foot. The undisturbed depth can be measured to within ±0.0005 foot. Thus the absolute accuracy of amplitude or surface profile elevation measurements is about ±0.001 foot.

In the experiments, a maximum depth of about 0.4 feet was used, as this gave sufficient freeboard and provided a good width-to-depth ratio. The minimum depth was set at 0.2 feet principally because of the inability of the generating apparatus to produce waves without secondary disturbances at the very low depths. No amplitudes less than 0.04 foot were used at the 0.4 foot depth and none less than 0.05 foot at the 0.2 foot depth. Thus for these ranges of amplitudes and depths, the resulting combined error in comparing calculated and measured celerities is approximately ±0.3%. The resulting error for profile measurements reported here is of the order of ±1.5% of the crest amplitude.

Studies of the motion of the fluid elements are conducted by injecting small droplets of η -butyl thalate and xylene colored red into the water at several depths. By adjusting the specific gravity of the mixture to match that of the water in the channel, the droplets remain suspended in the water and move freely with any motion of the water. Using a 35 mm. movie camera and a stroboscopic flash lamp, photographs at timed intervals are taken of the droplets in their various positions during a wave passage. The positions of the droplets are measured from the frames of the movie film which are projected with a grid superimposed. Knowing the time interval of the frames, the average velocity of the droplets between each position is determined. From several such droplets within the water depth, the velocity distribution is obtained.

WAVE CHARACTERISTICS

PROFILE

Typical experimental profiles selected from the numerous runs are shown in Figures 4 and 5, where points from the photographic records (small circles) are compared with the Boussinesq theoretical profile (solid line). In Figures 4 and 5a the entire experimental range of the amplitude-to-depth ratio, a/y_1, is represented, while, except for two cases, the undisturbed depth y_1 is nearly constant.

Although no theoretical function was found to represent the wave profile exactly, it was determined that those equations involving the use of the square of the hyperbolic secant more nearly approximated the experimental data than did the others. In particular, the wave profile as determined by Boussinesq (4) gives the most consistent agreement and was used in Figures 4 and 5.

In making a comparison of the experimental profile measurements with the theoretical wave profile as described by Boussinesq's analysis, consistent deviations from the theory are observed as follows. At the higher wave amplitudes for any depth, the experimental profile has a sharper crest with more gentle side slopes and a broader base, while the lower amplitudes exhibit characteristics of an opposite nature, being flatter at the crest with steeper side slopes and a shorter base than given by theory. Several profiles are shown in Figure 5b with approximately the same a/y_1 value, but for different values of y_1. Here a tendency is shown for the base width of a wave propagated in the smaller undisturbed depths to be proportionately longer than a wave at the larger depths when they are compared with the theoretical profiles for the respective a/y_1 values.

Thus, for the amplitudes and depths examined in this investigation, the experimental profile varies systematically about the theoretical profile as described by Boussinesq.

CELERITY

The results of the experimental measurements of wave celerity are shown in Figures 6 and 7. It is seen in Figure 6 that a smooth curve through the experimental data describing the dimensionless relationship between $C/\sqrt{gy_1}$ and a/y_1 falls roughly at a constant angle to the curve representing the relationship $C/\sqrt{gy_1} = \sqrt{1 + a/y_1}$. This latter expression is the theoretical celerity function as derived empirically by Russell (1) and theoretically by Rayleigh (5) and Boussinesq (4) and as a first approximation by later investigators. At the lower a/y_1 values, the experimental points give a celerity that is about 99% of that given by theory with experimental values decreasing to about 97.5% of the theoretical at the highest a/y_1 used in this investigation. Major deviations from a smooth curve occur at low values of a/y_1, where experimental accuracies are appreciably lower, due to the small magnitudes to be measured.

In Figure 7, a comparison of the absolute experimental celerity values with those as given by the Boussinesq expression is shown. A smooth curve through the experimental data on this plot gives an experimental celerity that is 99.4% of the theoretical at the low celerity values and an experimental celerity that is 98.7% of the theoretical at the high celerity values. For presenting the results of the experimental investigation, the representation in Figure 6 is a more sensitive expression of the celerity relationship.

The equations using higher degrees of approximation obtained by other investigators deviate sufficiently from the equation of the first approximation to be usable with increased accuracy only in restricted ranges. Thus, as is shown in Figure 6, the curves representing the analytical results of McCowan and Weinstein depart radically from the experimental results as a/y_1 is increased. The results of McCowan and Weinstein represent the limiting examples of the equations from the so-called higher approximation. The other analyses result in curves that lie between the latter two curves.

On the basis of the comparison shown in Figure 6, it is felt that the equation of the solitary wave celerity as presented by the first-

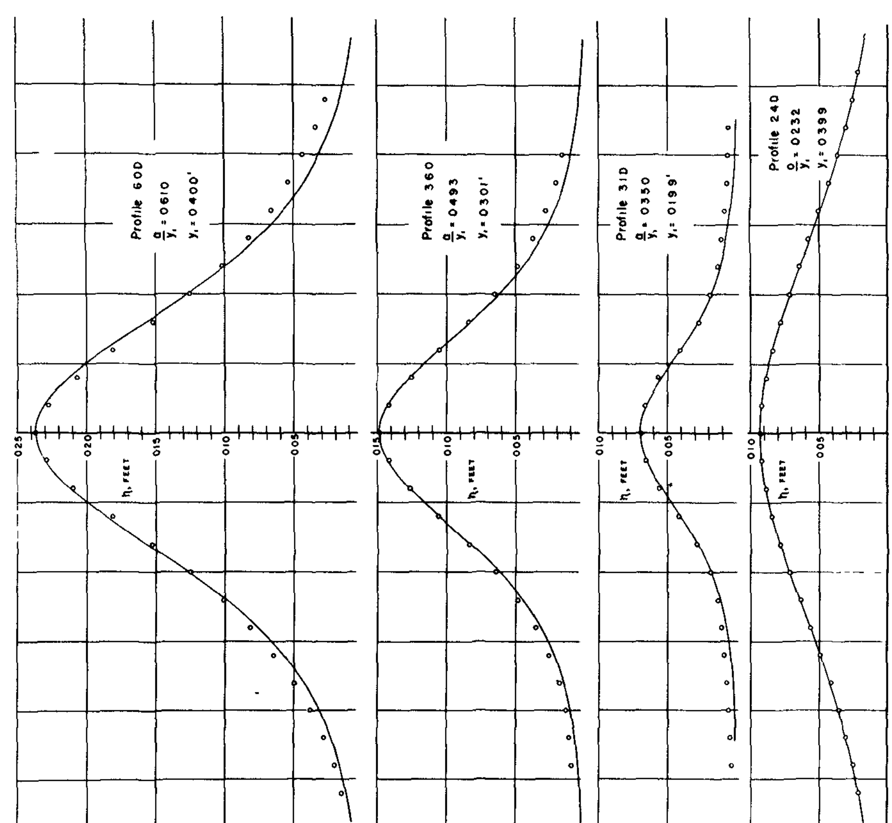
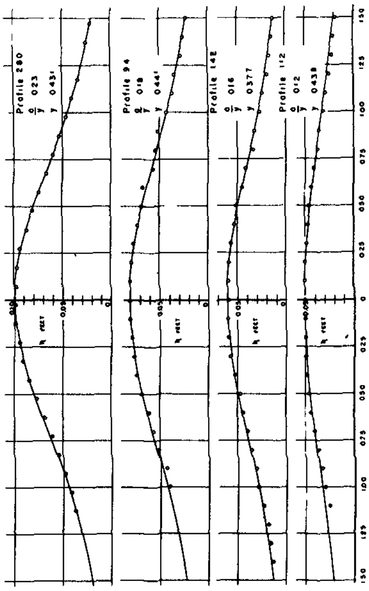
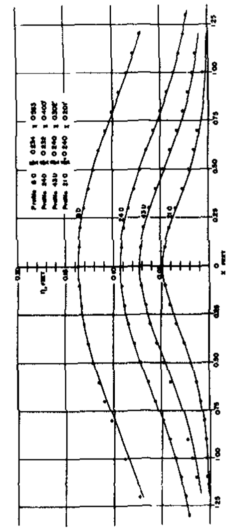


Fig. 4. Experimental profiles (circles) compared with Boussinesq's theoretical profile (solid line).



a. a/y_1 variable at constant depth.



b. a/y_1 constant at variable depth.

Fig. 5. Experimental profiles (circles) compared with Boussinesq's theoretical profile (solid line).

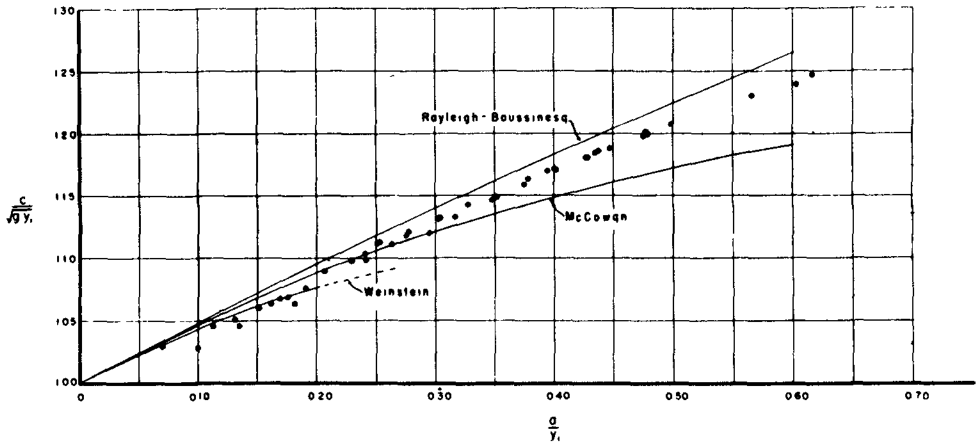


Fig. 6. Dimensionless comparison of experimental celerity data with theoretical celerity curves.

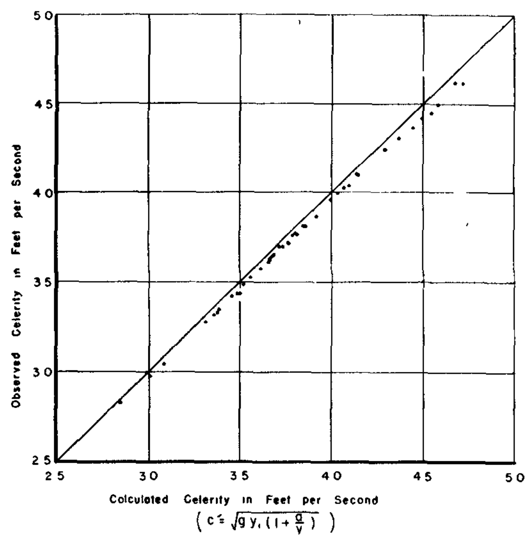


Fig. 7. Measured absolute celerity compared with Rayleigh-Boussinesq values.

approximation analysis of Rayleigh and Boussinesq is one that will give very close results for practical application to all values of a/y_1.

FLUID ELEMENT VELOCITY

The motion of suspended droplets was recorded on 35 mm. motion-picture film, using an open shutter and stroboscopic lighting with a flash period of 0.055 seconds. The position of each particle in successive frames was measured and the horizontal and vertical components of the mean velocity between each position were calculated. Typical sets of particle position and velocity component data are shown in Figures 8 and 9. Figure 8 shows the absolute paths of particles at different levels. The effect of the passing wave in producing a mass transport is clearly shown. Figure 9 which shows v_y plotted against v_x, is a useful diagram for correlating the components. It will be noted that there is some irregularity in these derived velocity components. Although the droplet positions were measured from enlargements of about two times natural size, the accuracy of measurement remained at about ±0.001 feet. With a maximum displacement of the droplets of about 0.05 feet in the time interval of 0.055 seconds, the minimum percentage error obtained was approximately ±2%, which became greater as the displacement distances decreased. Since no discontinuities are observed in the motion of the fluid, a continuous smooth relationship must exist between v_x and v_y. The solid curves drawn in Figure 10 represent such a correlation. Working backward then, adjustments of the measured positions of the droplets can be made for a more precise determination of the velocity-space distribution.

Using the correlated results, dimensionless diagrams such as those in Figures 10ab-12ab were prepared showing isovels of fluid elements and corresponding paths of the particles all relative to the wave axis. Additional diagrams appear in Ref. (15). Figures 10b-12b illustrate the apparent paths of the particles for the condition of steady motion. In Figures 10a-12a, the dimensionless isovels are the ratio of the local velocity to the wave celerity, and elevation and horizontal distance are expressed as fractions of the undisturbed depth y_1. For purposes of comparison, theoretical isovels are shown in Figures 10a-12a using Munk's formulation of McCowan's results (see Table I). In these figures, the heavy solid and dashed curves represent lines of equal vertical velocity and the light curves are lines of equal horizontal velocity. McCowan's results are used because only his analysis leads to explicit expressions for velocity components throughout the field of motion. While several other theoretical investigations of the particle paths and local velocities have been made, all, with this exception, seem to have by-products in the general scheme of analysis giving results for only special conditions. Examples of the latter are surface and bottom element velocities or the mean velocity over a vertical cross section such as determined by Boussinesq.

Examination of the isovel plots in comparison with the correlation diagram shows that the best accuracy of measurement was obtained at the

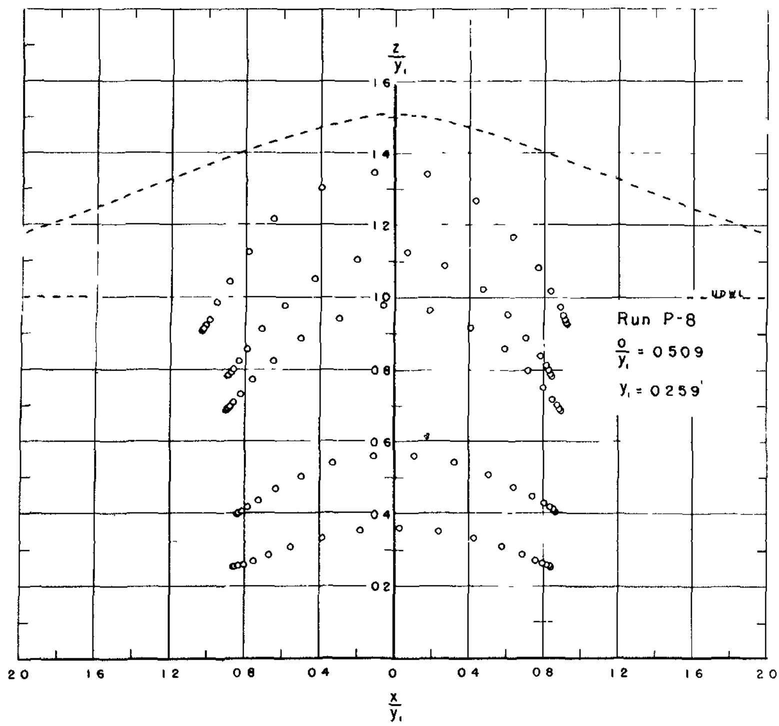


Fig. 8. Absolute paths of suspended droplets shown as they appear to a stationary coordinate system. The dashed wave profile is just for comparison with paths of droplets.

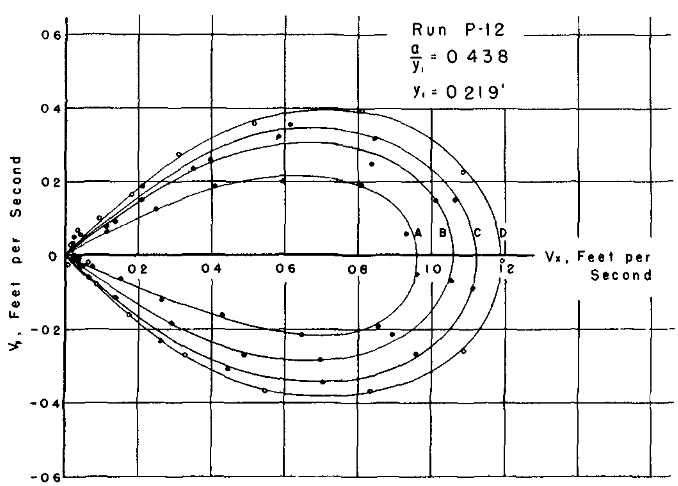


Fig. 9. Illustrating the use of the correlation between the components of fluid velocity to smooth out the irregularities in the experimental data. The solid curve is drawn as a best fit and values of components read from it. The points are the experimental data for four droplets at different elevations.

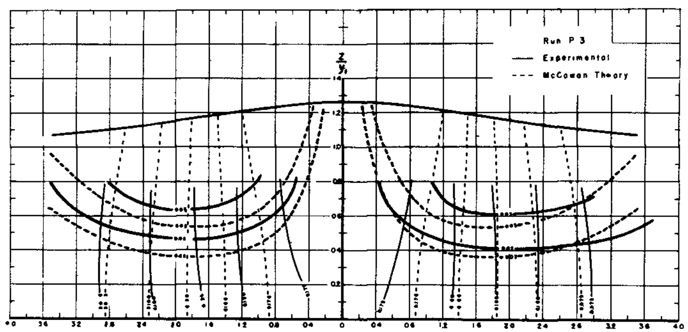


Fig. 10a. Comparison of components of local fluid element velocity according to McCowan with the components determined by experiment. Heavy isovels show equal vertical velocity. Light isovels show equal horizontal velocity. a/y_1 = 0.265, y_1 = 0.345 feet, C = 3.709 ft/sec. Values of isovels are ratios of element velocities to wave celerity.

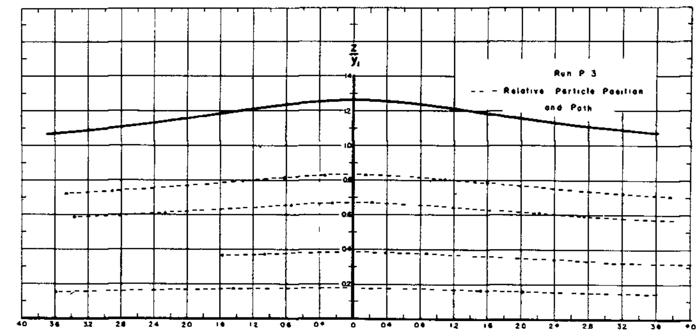


Fig. 10b. Positions of suspended droplets relative to the wave profile and axis at intervals of 0.055 second, illustrating the paths for the condition of steady motion.

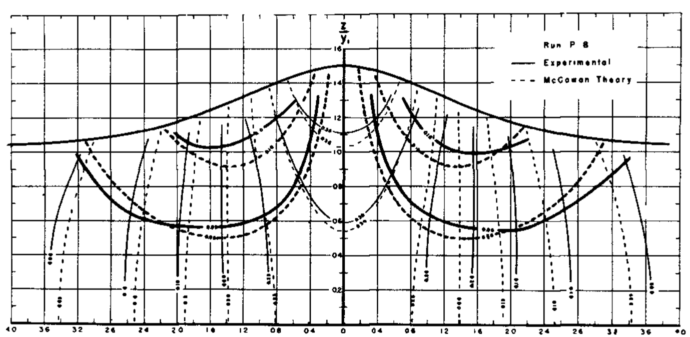


Fig. 11a. Comparison of components of local fluid element velocity according to McCowan with components determined by experiment. Heavy isovels show equal vertical velocity. Light isovels show equal horizontal velocity. a/y_1 = 0.509, y_1 = 0.259 feet, C = 3.502 ft/sec. Values of iso-

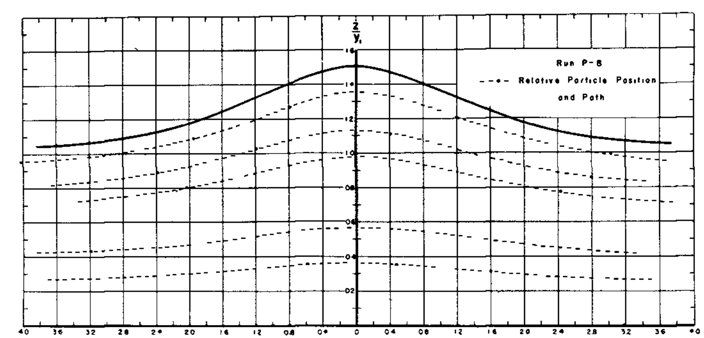


Fig. 11b. Positions of suspended droplets relative to the wave profile and axis at intervals of 0.055 second, illustrating the paths for the condition of steady motion.

wave axis. Actual observations of the droplet positions were made over a much longer period than is represented by the extent of the isovel plots. However, near the ends of the wave, the irregularities in the measurement became so large relative to the displacement distances that no satisfactory analysis could be made.

As McCowan's results show, a different distribution characterizes each amplitude-to-depth ratio. However, the experimental results otherwise do not agree with McCowan's. Towards the ends of the wave, the theoretical fluid velocity is less than the observed velocity. As the wave axis is approached, the situation becomes reversed, the theoretical values equalling the experimental at approximately the inflection point of the surface curve. These differences are consistent for the range of a/y_1 values investigated and, furthermore, are consistent with the differences between the actual and McCowan's theoretical profile. The latter tends to be more peaked with steeper side slopes above and more gentle side slopes below the point of inflection of the surface curve. Boussinesq's analysis yields the logical conclusion that the rate of increase or decrease of velocities is a function of rate at which the surface profile rises or falls. Assuming this holds the observed trends of actual velocities are logical.

In general, it was observed that the droplets move from an initial position of rest to a final position of rest without any orbital component. However, some exceptions were noted in which a backward component of motion was indicated. This cannot be construed as positive evidence, because of possible secondary influences, such as residual motion in the fluid or disturbances accompanying the main wave. However, there is this indication that the motion of the fluid element in the solitary wave merely represents the extreme limiting condition of the motion observed for wave trains. A more accurate method of measurement of the droplet displacement distances would be required to positively evaluate any such motion.

ATTENUATION

The attenuation of the wave amplitude is determined from photographs of the wave taken at various intervals of travel as it is reflected up and down the length of the channel. Figure 13 shows the results for tests with undisturbed depths of 0.2, 0.3 and 0.4 feet on a smooth channel bottom. The data for each depth is a composite of several runs. With the first experimental run at a given depth having the maximum possible initial amplitude, the subsequent runs were made with each wave amplitude slightly less than the preceding. This was necessary because only twelve passages of the wave could be made without reloading the cameras. By adjusting the origin of the abscissa for each run, a single curve was obtained for each depth.

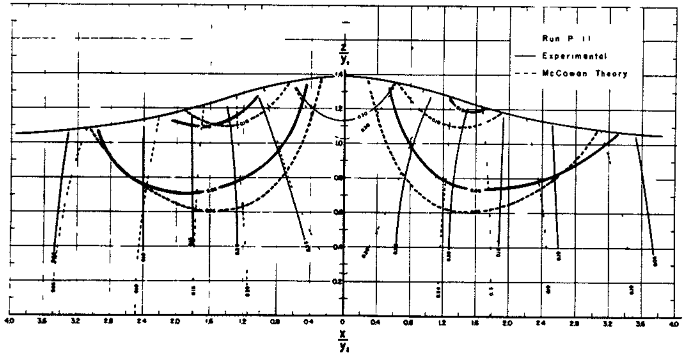


Fig. 12a. Comparison of oomponents of looal fluid element velooity aooording to McCowan with oomponents determined by experiment. Heavy isovels show equal vertioal velocity. Light isovels show equal horizontal velooity. $a/y_1 = 0.388$, $y1 = 0.207$ feet, $C = 3.003$ ft/seo. Values of isovels are ratios of element velocities to wave celerity.

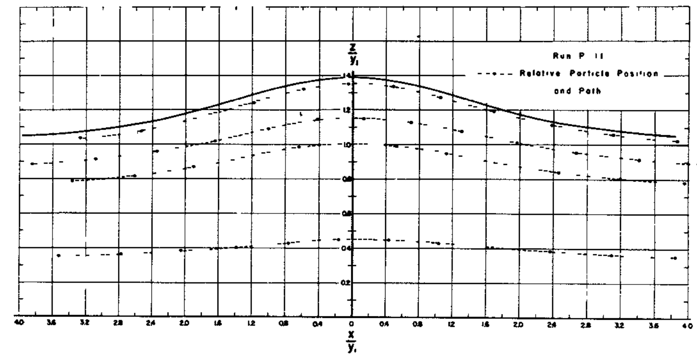


Fig. 12b. Positions of suspended droplets relative to the wave profile and axis at intervals of 0.055 seoond, illustrating the paths for the oondition of steady motion.

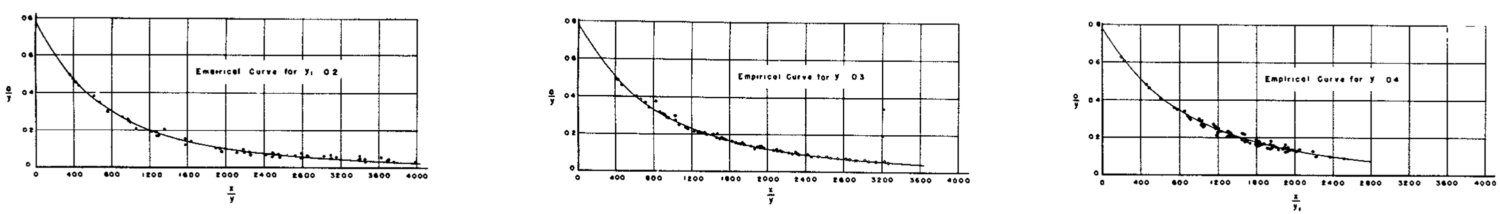


Fig. 13. Attenuation on a smooth bottom. Eaoh empirical ourve begins at the arbitrary value $a/y_1 = 0.78$.

A dimensional analysis gives for the rate of change of amplitude with a smooth bottom

$$\frac{\Delta a}{\Delta x} = \text{const} \times \left(\frac{a}{y_1}\right)^m \left(\frac{\gamma}{g^{1/2}\, y_1^{3/2}}\right)^q \left(\frac{B}{y_1}\right)^t \left(\frac{P}{y_1}\right)^v$$

where in addition to terms already defined, x = distance along channel, B = channel width, P = wetted perimeter, γ = kinematic viscosity, and m, q, t, v are exponents. Empirically, the data represented in Figure 13 corresponds to $m = 5/4$, $q = 1/2$, $t = -2$, $v = 2$, and the constant $= 1/20$, so that on integrating between an initial amplitude a_o and a we get

$$\left(\frac{a}{y_1}\right)^{-1/4} - \left(\frac{a_o}{y_1}\right)^{-1/4} = \frac{1}{20}\left(1 - \frac{2y_1}{B}\right)^2 \left(\frac{\gamma}{g^{1/2}\, y_1^{3/2}}\right)^{1/2} \frac{x}{y_1}$$

This gives slightly lower attenuation rates than Keulegan's (12) theoretical equation.

SUMMARY OF OBSERVATIONS

These experiments conducted in a 16-1/2 inch wide horizontal smooth bottomed channel with undisturbed water depths between 0.2 and 0.4 feet give the following:

1. The celerity is adequately described for practical applications by the equation derived empirically by Russell and theoretically by Boussinesq and Rayleigh and given as

$$C = \sqrt{g y_1}\,\sqrt{1 + \frac{a}{y_1}}$$

The theoretical celerity is approximately 2.5% higher than the experimental value at $a/y_1 = 0.6$ with the difference decreasing as a/y_1 increases. Experimental celerities were determined over the range of a/y_1 from 0.07 to 0.61.

2. The wave profile is closely approximated by the theoretical profile given by Boussinesq as

$$\eta = a\,\mathrm{sech}^2\, \frac{x}{y_1}\sqrt{\frac{3}{4}\,\frac{a}{y_1}}$$

The experimental profiles were determined over the range of a/y_1 from 0.12 to 0.61. At high a/y_1, the theoretical profile exhibits a slightly flatter crest than the experimental with the opposite true at low a/y_1. Best agreement is at $a/y_1 = 0.23$.

3. The experimentally determined fluid element velocity distribution depends upon a/y_1 in manner similar to that predicted by McCowan's

solution. The differences between the measured and the theoretical values increase with increasing a/y_1, but are consistent with the differences between the actual and the McCowan profile.

4. The attenuation with a smooth horizontal channel bottom was found to be described by the expression

$$\left(\frac{a}{y_1}\right)^{-1/4} - \left(\frac{a_o}{y_1}\right)^{-1/4} = \frac{1}{20}\left(1 + \frac{2y_1}{B}\right)^2 \left(\frac{\gamma}{g^{1/2}\, y_1{}^{3/2}}\right)^{1/2} \frac{x}{y_1}$$

This gives slightly lower attenuation rates than predicted by Keulegan.

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CHAPTER 3

ACCURACY OF HYDROGRAPHIC SURVEYING IN AND NEAR THE SURF ZONE

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INTRODUCTION

The analysis and solution of most beach erosion problems are based to a significant degree on the quantitative changes in the bottom hydrography as observed in successive surveys. Critical decisions as to the dominant direction of littoral drift, the average rate of this drift, and the onshore-offshore movement of material are based largely on such hydrographic surveys. As the net changes between successive surveys are usually small compared to the area being studied, the degree of accuracy or comparability of the hydrographic surveys is of considerable importance. For instance, a net change of 100,000 cubic yards over one square mile of beach represents an average change in depth of only about 0.1 feet. Thus, it can be seen that uncompensated errors in depth measurement of as little as 0.1 feet can produce indications of significant littoral sand movement which might not exist in reality.

The errors involved in hydrographic work may be attributed almost entirely to two different causes. The first of these, a sounding error, results from errors inherent in the sounder and the methods involved in reducing the sounder data to an actual bottom profile (i.e. tide corrections, elimination of the effect of waves, water temperature corrections, etc.). The second, a spacing error, results from the fact that a particular profile may not be entirely representative of its assigned section of beach.

The sounding error is a measure of the accuracy (or inaccuracy) with which the profile deduced from the sounder record actually represents the bottom hydrography along the particular range being sounded; as such it may be determined as a function of the reproducibility of this profile by the repetition of a series of soundings. The spacing error is a measure of the accuracy (or inaccuracy) with which the particular profile portrays the characteristics of the contiguous beach area; as such it may be determined as a function of the reproducibility of the hydrography of a beach area by using various spacings between adjacent profiles.

It was the purpose of this study to determine on a statistical basis the degree of accuracy that could be expected in hydrographic survey work where comparability of successive surveys is a prime consideration. Tests to determine the magnitude of these two types of error were made at Mission Beach, California. Mission Beach is a relatively long, straight beach, with essentially parallel contours, and no radical changes of bottom hydrography along its length, and as such, is representative of many of the southern California beaches. The results of these tests may be expected to apply to other beaches of the same type.

The tests were made under normal operating conditions by the Field Research Group of the Beach Erosion Board; i.e., standard Beach Erosion Board procedures were used in checking the tide, the sounding instruments, and the position of the survey boat so that the results could be considered applicable to actual hydrographic surveys made by the Field Group. A description of the standard survey techniques used by the Field Research Group is given in The Bulletin of the Beach Erosion Board, July 1947.

DETERMINATION OF THE SOUNDING ERROR

DESCRIPTION OF TEST

The test to determine sounding error involved the repeated sounding of a single profile eight times successively in a five-hour period. The survey extended from the shore line to the -50-foot mean lower low water contour on Beach Erosion Board profile range 136 at Mission Bay, California. This range is about 5500 feet north of the Mission Bay jetties and the -50-foot contour is about 4250 feet offshore. The range was established by the Field Research Group in connection with other work in the area. The test was made on 3 November 1950 while swells of about two feet in height were running. The tide variation was 0.4 feet during the 5-hour period; corrections of the sounding records were made for this variation. An amphibious truck, DUKW, was used as the floating equipment for the survey. In making the tests, a Bludworth NK-2 echo sounder was used while the DUKW was floating; a lead line was used while the wheels of the DUKW were grounded in traversing the shallow water section of the profile.

ANALYSIS OF ECHO SOUNDER DATA

The echo-sounder data and the lead-line soundings were analyzed separately. The echo-sounder charts were first corrected for tide elevation and the soundings taken off at 250-foot intervals starting at a point 750 feet from the base line. The tabulation of results is shown on Table 1. This table shows the corrected soundings for the eight test runs and covers the area from about the -7-foot to the -50-foot mean lower low water contour, a distance of about 3500 feet. The table also shows an "average" profile column obtained by averaging the eight separate profiles.

As with most statistical data, there are several ways of effecting an analysis. However, only two methods appeared to have enough engineering significance in the present case to warrant a set of calculations. The first method assumes that the "average" profile is the correct profile for the 5-hour period and then studies the deviation of each of the eight profiles from the average. The second method assumes that the deviation of one profile from the succeeding profile is a better measurement of the degree of accuracy with which successive surveys can be compared. The data has been analysed in both ways.

ACCURACY OF HYDROGRAPHIC SURVEYING IN AND NEAR THE SURF ZONE

TABLE 1

Soundings Taken at Range 136, Mission Bay, California
3 November 1950

Distance from Base Line (ft.)	Soundings in feet MLLW for Run Number 1	2	3	4	5	6	7	8	Average for all runs
	Sonic Soundings								
750	-7.4	-7.5	-7.5	-7.4	-7.3	-7.6	-7.7	-7.5	-7.49
1000	14.3	14.2	14.2	14.5	14.2	14.1	14.2	14.0	14.21
1250	19.1	18.9	19.1	19.4	18.9	18.7	18.9	18.8	18.98
1500	23.6	23.4	23.8	23.6	23.3	23.7	23.4	23.7	23.56
1750	26.9	27.3	27.4	27.6	26.9	27.1	27.1	27.5	27.22
2000	30.0	30.4	30.3	30.3	30.3	30.3	30.1	30.3	30.25
2250	32.7	33.0	32.9	33.2	33.0	32.9	32.5	32.8	32.88
2500	35.4	35.4	35.2	35.6	35.2	35.3	35.0	35.3	35.30
2750	37.7	37.4	37.6	37.9	37.6	37.4	37.5	37.5	37.57
3000	39.8	39.8	40.1	39.9	39.7	39.6	39.8	39.7	39.80
3250	41.8	41.8	42.3	42.4	41.8	41.7	41.8	41.9	41.94
3500	44.1	44.1	44.7	44.5	44.1	44.1	43.8	44.3	44.21
3750	46.6	46.5	46.8	46.5	46.5	46.5	46.4	46.5	46.54
4000	48.9	48.8	48.8	48.7	48.7	48.6	48.5	48.6	48.70
4250	50.9	51.2	50.8	50.9	50.8	50.8	50.8	51.0	50.90
	Lead-line Soundings								
250	2.1	1.8	1.5	2.4	1.8	2.4	2.5	1.9	2.05
300	1.1	1.4	1.4	2.2	1.0	1.1	1.4	1.5	1.39
350	0.9	0.5	0.6	0.8	+0.3	+0.8	+0.8	0.6	0.66
400	+0.3	+0.3	+0.2	+0.4	-0.4	-0.3	+0.5	+0.3	+0.16
450	-0.7	-0.7	-0.5	-0.6	-1.1	-1.4	-0.5	-0.6	-0.76
500	-1.5	-1.5	-1.5	-2.0	-2.3	-2.0	-1.7	-1.5	-1.75

Note: Soundings were taken over a 5-hour period and have been corrected for tide

TABLE 2

Deviation (in feet) of Actual Profiles from Average Profile

Distance from Base Line (ft.)	Profile being compared to average profile 1	2	3	4	5	6	7	8
	Sonic Soundings							
750	+.09	-.01	-.01	+.09	+.19	-.11	-.21	-.01
1000	-.09	+.01	+.01	-.29	+.01	+.11	+.01	+.21
1250	-.12	+.08	-.12	-.42	+.08	+.28	+.08	+.18
1500	-.04	+.16	-.24	-.04	+.26	-.14	+.16	-.14
1750	+.32	-.08	-.18	-.38	+.32	+.12	+.12	-.28
2000	+.25	-.15	-.05	-.05	-.05	-.05	+.15	-.05
2250	+.18	-.12	-.02	-.32	-.12	-.02	+.38	+.08
2500	-.10	-.10	+.10	-.30	+.10	0	+.30	0
2750	-.13	+.17	-.03	-.33	-.03	+.17	+.07	+.07
3000	0	0	-.30	-.10	+.10	+.20	0	+.10
3250	+.14	+.14	-.36	-.46	+.14	+.24	+.14	+.04
3500	+.11	+.11	-.49	-.29	+.11	+.11	+.41	-.09
3750	-.06	+.04	-.26	+.04	+.04	+.04	+.14	+.04
4000	-.20	-.10	-.10	0	0	+.10	+.20	+.10
4250	0	-.30	+.10	0	+.10	+.10	+.10	-.10
Total d	-0.35	-0.15	-1.95	-2.85	+1.25	+1.15	+2.05	+0.15
Ave. d_2	-.023	-.010	-0.130	-0.190	+0.083	+0.077	+0.137	+0.010
d^2	.000545	.0001	.0169	.0361	.006944	.005878	.018678	.0001

$\Sigma d^2 = 0.085245 \quad n = 8 \quad \sigma = \sqrt{\frac{0.085245}{8}} = \sqrt{0.010656} = 0.103$ ft.

Probable error (sonic soundings) = (0.6745) (0.103) = 0.069 ft.

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Distance from Base Line (ft.)	1	2	3	4	5	6	7	8
	Lead-line Soundings							
250	+.05	-.25	-.55	+.35	-.25	+.35	+.45	-.15
300	-.29	+.01	+.01	+.81	-.39	-.29	+.01	+.11
350	+.24	-.16	-.06	+.14	-.36	+.14	+.14	-.06
400	+.14	+.14	+.04	+.24	-.56	-.46	+.34	+.14
450	+.06	+.06	+.26	+.16	-.34	-.64	+.26	+.16
500	+.25	+.25	+.25	-.25	-.55	-.25	+.05	+.25
Total d	+.45	+.05	-.05	+1.45	-2.45	-1.15	+1.25	+.45
Ave. d_2	+.075	+.008	-.008	+0.242	-0.408	-0.192	+0.208	+.075
d^2	.005625	.000069	.000069	.058403	166736	036736	043403	005625

$\Sigma d^2 = 0.316666 \quad n = 8 \quad \sigma = \sqrt{0.316666/8} = \sqrt{0.03953} = 0.199$ ft.

Probable error (lead-line soundings) = (0.6745)(0.199) = 0.134 ft

TABLE 3

Deviation (in feet) of Each Profile from the Succeeding Profile

Distance from Base Line in ft.	Profiles being compared 1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-1
	Sonic Soundings							
750	+.1	0	-.1	-.1	+.3	+.1	-.2	-.1
1000	-.1	0	+.3	-.3	-.1	+.1	-.2	+.3
1250	-.2	+.2	+.3	-.5	-.2	+.2	-.1	+.3
1500	-.2	+.4	-.2	-.3	+.4	-.3	+.3	-.1
1750	+.4	+.1	+.2	-.7	+.2	0	+.4	-.6
2000	+.4	-.1	0	0	0	-.2	+.2	-.3
2250	+.3	-.1	+.3	-.2	-.1	-.4	+.3	-.1
2500	0	-.2	+.4	-.4	+.1	-.3	+.3	+.1
2750	-.3	+.2	+.3	-.3	-.2	+.1	0	+.2
3000	0	+.3	-.2	-.2	-.1	+.2	-.1	+.1
3250	0	+.5	+.1	-.6	-.1	+.1	+.1	-.1
3500	0	+.6	-.2	-.4	0	-.3	+.5	-.2
3750	-.1	+.3	-.3	0	0	-.1	+.1	+.1
4000	-.1	0	-.1	0	-.1	-.1	+.1	+.3
4250	+.3	-.4	+.1	-.1	0	0	+.2	-.1
Total d	+.5	+1.8	+0.9	-4.1	+0.1	-0.9	+1.9	-0.2
Ave d_2	+0.033	+0.12	+0.06	+0.273	+0.007	-0.06	+0.127	-0.013
d^2	0.0011	0.0144	0.0036	0.0747	0.0000	0.0036	0.0160	0.0001

$d^2 = 0.1135 \quad n = 8 \quad \sigma = \sqrt{\frac{0.1135}{8}} = \sqrt{0.0142} = 0.119$ feet.

Probable error = (0.6745)(0.119) = 0.080 feet.

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Distance from Base Line in ft.	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-1
	Lead-line Soundings							
250	+.3	+.3	-.9	+.6	-.6	-.1	+.6	-.2
300	-.3	0	-.8	+1.2	-.1	-.3	-.1	+.4
350	+.4	-.1	-.2	+.5	-.5	0	+.2	-.3
400	0	+.1	-.2	+.8	-.1	-.8	+.2	0
450	0	-.2	+.1	+.5	+.3	-.9	+.1	+.1
500	0	0	+.5	+.3	-.3	-.3	-.2	0
Total d	+.4	+.1	-1.5	+3.9	-1.3	-2.4	+.8	0
Ave d_2	+.067	+.017	-0.25	+0.65	-0.217	-0.4	+0.133	0
d^2	0.0044	0.0003	0.0625	0.4225	0.0469	0.16	0.0177	0

$d^2 = 0.7143 \quad n = 8 \quad \sigma = \sqrt{\frac{0.7143}{8}} = \sqrt{0.893} = 0.299$ feet.

Probable error = (0.6745)(0.299) = 0.201 feet.

TABLE 4

Study of relation of number of profiles used to the average accuracy of the profiles

Number of profiles used at a time	Number of combinations possible and used	Computed on basis of average for eight profiles: Standard Deviation (feet)	Computed on basis of average for eight profiles: Probable error (feet)	Error Theory: Standard deviation (feet)	Error Theory: Probable error (feet)
		(For offshore section sounded by echo-sounder)			
1	8	0.103	0.069	0.103	0.069
2	28	0.068	0.046	0.072	0.049
3	56	0.050	0.034	0.059	0.040
4	70	0.039	0.026	0.051	0.035
5	56	0.030	0.020	0.046	0.031
6	28	0.023	0.016	0.042	0.028
7	8	0.015	0.010	0.039	0.026
8	1	0	0	0.036	0.024
		(For inshore section sounded by lead-line)			
1	8	0.199	0.134	0.199	0.134
2	28	0.130	0.088	0.141	0.095
3	56	0.097	0.065	0.115	0.077
4	70	0.075	0.051	0.099	0.067
5	56	0.058	0.039	0.089	0.060
6	28	0.043	0.029	0.081	0.055
7	8	0.028	0.019	0.075	0.051
8	1	0	0	0.070	0.047

The deviation of the individual soundings from the average sounding for the comparable station is shown in Table 2. The deviations for each profile are summarized algebraically on the table; each summation is in turn divided by the number of stations, 15, in order to establish the average deviation, d, of the profile from the average profile. This average deviation is a measure of the error that would be introduced in a set of computations by using a single profile instead of the average profile; thus Run 3 gives a profile for the echo-sounder portion of the record which averages 0.130 feet below the average profile. These average profile deviations, d, can be handled collectively by the statistical formula

$$\sigma = \sqrt{\frac{\Sigma d^2}{n}}$$

where σ is the standard deviation and n is the number of observations. The result is

$$\sigma = \sqrt{\frac{0.08524}{8}} = 0.103 \text{ feet}$$

The probable error, P.E., in any one profile is given by

$$\text{P.E.} = 0.6745\sigma = 0.069 \text{ feet. (say 0.07 feet)}$$

This indicates that any one profile obtained by the echo sounder can be expected to have an uncompensated error averaging 0.07 feet.

The second method of analysis involves comparing each profile with the succeeding profile. In this manner, no attempt is made to establish the absolute profile as was done with the "average" profile in the preceeding paragraph; rather the comparison is on the basis of the comparability of successive profiles. The statistical analysis based on this reasoning is given in Table 3. In this case it can be seen that the profile of Run 1 is compared to Run 2, then Run 2 to Run 3, and so on. Finally, Run 8 is compared back to Run 1. The summation and statistical handling is the same as used previously and shows for the echo-sounder portion of the record a standard deviation, σ, of 0.119 feet, and a probable error of 0.08 feet. It is to be noted that the probable error indicated by this analysis is of the same order as for the first analysis (0.08 feet against 0.07 feet). Attention is also called to the fact that the deviation for the comparison of Run 8 to Run 1 was well below the average deviation, indicating that there was no systematically increasing error over the 5-hour test period.

In considering this indication of an 0.07 to 0.08 foot uncompensated error it should be kept in mind that this figure is probably an optimistic one due to the fact that the comparative profiles were taken on the same day with the same personnel and equipment and with a relatively small tide variation. These factors would tend to make the error somewhat less than would be the case if the surveys were taken several weeks or months apart. Also, any constant error that might have been effective on the day of the soundings, such as in the instruments, the submergence of the sounder, or the tide adjustment, is not included in the 0.07 foot figure.

ANALYSIS OF LEAD-LINE SOUNDING DATA

A lead-line was used for sounding whenever the wheels of the DUKW were grounded. Table 1 shows the lead-line soundings as well as the sonic soundings taken during the running of the eight test profiles. These soundings were analyzed statistically in the same manner as the echo sounder records and it was found that:

(a) A comparison of profile deviation against the "average" profile showed an uncompensated probable error of 0.13 feet.

(b) A comparison of successive profiles showed an uncompensated probable error of 0.20 feet.

It is seen that these probable errors with the lead-line are considerably greater than the probable errors for that portion of the profile sounded by echo sounder. However, the portion of the profile covered by lead-line is generally a minor portion of the entire profile so that the quantitative error is usually not as great in the overall picture. In the Mission Bay tests, about 4,000 feet of profile was sounded by echo sounder and about 300 feet by lead-line.

The fact that the actual beach profile for the eight test runs was probably slightly different for each run is appreciated. However, this does not change the analysis given above, as no hydrographic survey is made simultaneously over all profiles. Instead the profiles are run successively as in the test and the test runs would appear to indicate the degree of comparability of the profiles, which was the purpose of the test.

Of some significance in considering the results of the analysis given above is the fact that the portable echo-sounders used in most beach profile work are rated as having an accuracy of $\pm \frac{1}{2}$-foot at a 50-foot depth. It should be noted that the sounder accuracy is expressed in feet at 50 feet and not as a percentage; this is done because some of the errors in the sounder vary with depth whereas others are independent of depth. Thus the error could be expected to be less at 10 feet than at 50 feet but not as much less as the ratio of depths might indicate. The fact that during the eight test runs discussed above the same echo-sounder was used by the same crew and the entire test covered only a 5-hour period would tend to hold the sounder error to a minimum. The usual bar checks were made to adjust the sounder before starting the tests.

APPLICATION TO A SURVEY CONSISTING OF MORE THAN ONE PROFILE

The preceding discussion applies to the sounding error to be expected over a single profile. Most hydrographic surveys involve the use of a number of profiles to determine the hydrography of a given area. The use of multiple profiles makes it likely that the uncompensated errors in one profile will be somewhat compensated by a similar error opposite in sign on another profile. The eight profiles used in the preceding discussion were accordingly analyzed toward the end of discovering the sounding error to be expected in the use of multiple profiles.

In making this analysis, the eight profiles of Table 1 were compared to the average profile shown in the same table. The eight profiles were compared individually to the average and the resultant deviations compared statistically; the results of this comparison have already been discussed and are shown on Table 2. The results indicated for the sonic-sounder portion a standard deviation of 0.102 feet based on the use of a single profile on which to establish a comparison.

The indicated errors for every possible combination of two profiles were then averaged. The results established a standard deviation for the offshore portion of 0.0676 feet based on the use of two profiles. The comparison was continued for all possible combinations of three, four, five, six, seven, and eight profiles with the results shown in Table 4. In using these results, two factors must be kept in mind:

(1) That the results should not be construed as indicating to what degree the profiles are representative of the section of beach which they are assumed to represent. The present portion of this memorandum is pointed toward indicating the "surveying" errors; the degree to which a selected profile may be considered representative will be discussed later in this memorandum.

(2) That the entire set of computations is influenced by the fact that only eight profiles were used and that these eight were averaged to give the reference or base profile. This condition effects the lower end of the curve much more than the upper end; for instance Table 4 indicates a zero deviation if eight profiles are used, which is obviously unrealistic. However, it is believed that the figures for the use of one or two profiles are not too greatly influenced by the fact that only eight profiles were used as a basis for the computations.

If the value based on the use of the single profiles is assumed to be correct, then values for the use of any number of profiles may be derived from error theory to give

$$\sigma_n = \frac{\sigma_1}{\sqrt{n}}$$

where σ_n represents the standard deviation to be expected from the use of n profiles; and σ_1 is the standard deviation for a single profile. σ_1 was previously shown to be 0.103 feet for the sonic portion of the profile and 0.199 for the lead-line portion. Values for the probable error may be derived similarly, and

$$P.E._n = \frac{P.E._1}{\sqrt{n}}$$

Values for the standard deviation and probable error computed by this

TABLE 5

Probable Sounding Errors in Beach Surveys Made with Sonic Sounder

Number of Profiles used	Standard deviation (feet)	Probable error (feet)	Probable error in cu. yd. per linear foot of shore,when using profilee with an average length of ------------- 1 ft.	100 ft.	500 ft.	1000 ft.	5000 ft.
1	0.103	0.069	0.00255	0.255	1.27	2.55	12.7
2	0.072	0.049	0.00182	0.182	0.91	1.82	9.1
3	0.059	0.040	0.00148	0.148	0.74	1.48	7.4
4	0.0510	0.0342	0.00127	0.127	0.63	1.27	6.3
5	0.0457	0.0308	0.00114	0.114	0.57	1.14	5.7
6	0.0418	0.0280	0.00104	0.104	0.52	1.04	5.2
8	0.0361	0.0243	0.00090	0.090	0.45	0.90	4.5
10	0.0321	0.0217	0.00080	0.080	0.40	0.80	4.0
15	0.0264	0.0177	0.00066	0.066	0.33	0.66	3.3
20	0.0229	0.0154	0.00057	0.057	0.29	0.57	2.9
30	0.0186	0.0126	0.00047	0.047	0.23	0.47	2.3
40	0.0161	0.0110	0.00041	0.041	0.20	0.41	2.0
50	0.0145	0.0097	0.00036	0.036	0.18	0.36	1.8
75	0.0118	0.0079	0.00029	0.029	0.15	0.29	1.5
100	0.0102	0.0069	0.00026	0.026	0.13	0.26	1.3
150	0.0084	0.0056	0.00021	0.021	0.10	0.21	1.0
200	0.0072	0.0049	0.00018	0.018	0.09	0.18	0.9
500	0.0046	0.0031	0.00011	0.011	0.06	0.11	0.6
1000	0.0032	0.0022	0.00008	0.008	0.04	0.08	0.4

TABLE 6

Probable Sounding Errors in Beach Surveys Made by Lead-Line

Number of Profilee used	Standard Deviation (feet)	Probable error (feet)	Probable error in cu. yd. per linear foot of shore, when using profiles with an average length of -------------- 1 ft.	100 ft.	500 ft.	1000 ft.	5000 ft.
1	0.199	0.134	0.00496	0.496	2.48	4.96	24.8
2	0.141	0.094	0.00348	0.348	1.74	3.48	17.4
3	0.115	0.076	0.00282	0.282	1.41	2.82	14.1
4	0.099	0.068	0.00252	0.252	1.26	2.52	12.6
5	0.088	0.059	0.00218	0.218	1.09	2.18	10.9
6	0.081	0.054	0.00200	0.200	1.00	2.00	10.0
8	0.070	0.047	0.00174	0.174	0.87	1.74	8.7
10	0.063	0.042	0.00156	0.156	0.78	1.56	7.8
15	0.051	0.034	0.00126	0.126	0.63	1.26	6.3
20	0.044	0.030	0.00111	0.111	0.56	1.11	5.6
30	0.036	0.025	0.00093	0.093	0.46	0.93	4.6
40	0.031	0.021	0.00078	0.078	0.39	0.78	3.9
50	0.027	0.019	0.00070	0.070	0.35	0.70	3.5
75	0.023	0.016	0.00059	0.059	0.30	0.59	3.0
100	0.020	0.013	0.00050	0.050	0.25	0.50	2.5
150	0.016	0.011	0.00041	0.041	0.20	0.41	2.0
200	0.014	0.009	0.00035	0.035	0.17	0.35	1.7
500	0.009	0.006	0.00022	0.022	0.11	0.22	1.1
1000	0.006	0.004	0.00011	0.016	0.08	0.16	0.8

formula are also shown in Table 4. Figure 1 shows the variation of the sounding error as computed by error theory if it is assumed that the value for a single profile is correctly obtained from the average of the eight test profiles. Also shown are the points obtained from using all the possible combinations of the test profiles for the sonic portion of the test. As may be seen the points obtained for the combination of two and three profiles do not differ greatly from the error theory curve, and this supports strongly the assumption that the value for the single profile is very nearly correct.

The data from Figure 1 have the dimensions of feet, and can be expressed as cubic feet per lineal foot of shore per foot of profile and hence can be reduced to a relationship of probable cubage error per foot of shore as related to the number of profiles utilized in the survey under consideration. A tabulation of this relationship for the sonic sounder, as computed from Figure 1, is given in Table 5, and for lead-line soundings in Table 6. The relationships for both lead-line and sonic portions are shown as a series of curves in Figure 2. The values given in Tables 5 and 6 or Figure 2 are readily applied to the analysis of the probable surveying error inherent to a given survey of a beach. Knowing the number of profiles used, and the average length of these profiles, the cubage error per foot of beach can be computed. The product of this unit error and the length of beach gives the probable cubage error over the study area. It should be kept in mind that the cubage errors indicated in Tables 5 and 6 are <u>per linear foot of beach</u>. As an example, for a 10,000 foot section of beach, surveyed by 20 profiles each 4,000 feet long, the total probable sounding error would be (0.57) (4) (10,000) = 22,800 cubic yards.

From the above it can be seen that surveying errors may enter the analysis of a beach problem to a significant degree if too few profile lines are used in the study. It should again be emphasized that these errors represent "sounding error" alone and take no account of a spacing error.

It should be noted that the computations discussed above and tabulated in Tables 2 and 4 were based on the use of fifteen soundings for the sonic sounder section of each profile. The question arises as to the effect on the comparative accuracy of the profile line of increasing the number of soundings. This effect was investigated by taking the same eight profiles previously used and picking off soundings at 125-foot intervals instead of 250-foot intervals; this resulted in thirty soundings for comparison, or double the number originally used. An intercomparison of these eight profiles with thirty soundings each was then worked out on the same basis as described above. Table 7 shows a comparison of the results using 30 soundings per profile with the results using 15 sounding per profile; the very close agreement in the results indicates that the use of 15 soundings per line was sufficient to establish the accuracy characteristics of the profile and that nothing would be gained by increasing the number of soundings utilized in the comparison.

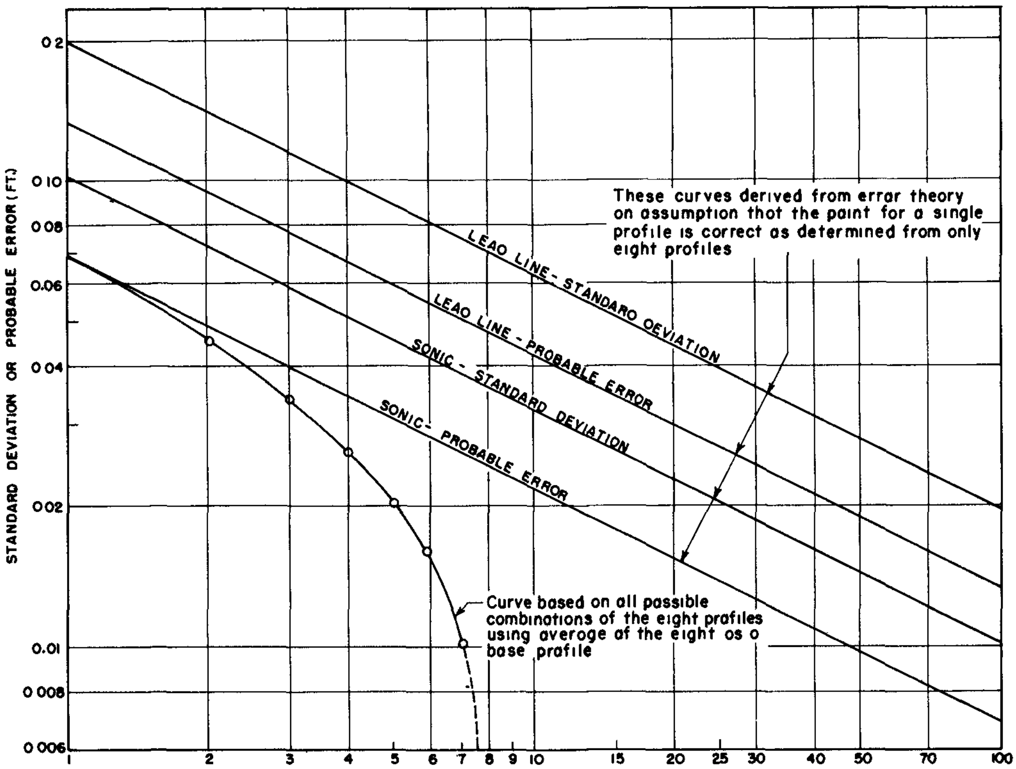

Fig. 1. Variation of sounding error with number of profiles sounded.

TABLE 7

Study of Effect of Number of Soundings per Profile on the Average Accuracy of a Profile

Number of profiles used at a time	Standard deviation* in feet using 15 soundings per profile	30 soundings per profile
1	0.103	0.103
2	0.0676	0.0675
3	0.0504	0.0503
5	0.0302	0.0302
6	0.0225	0.0224
7	0.0147	0.0147

* In computing these deviations, the various profiles and combinations were compared to the average profile of the eight profiles as was done in Tables 2 and 4. When succeeding profiles were compared in the manner done in Tables 3, the use of 30 soundings per profile showed a standard deviation of 0.0118 feet which is identical with the results shown in Table 3 for 15 soundings per profile.

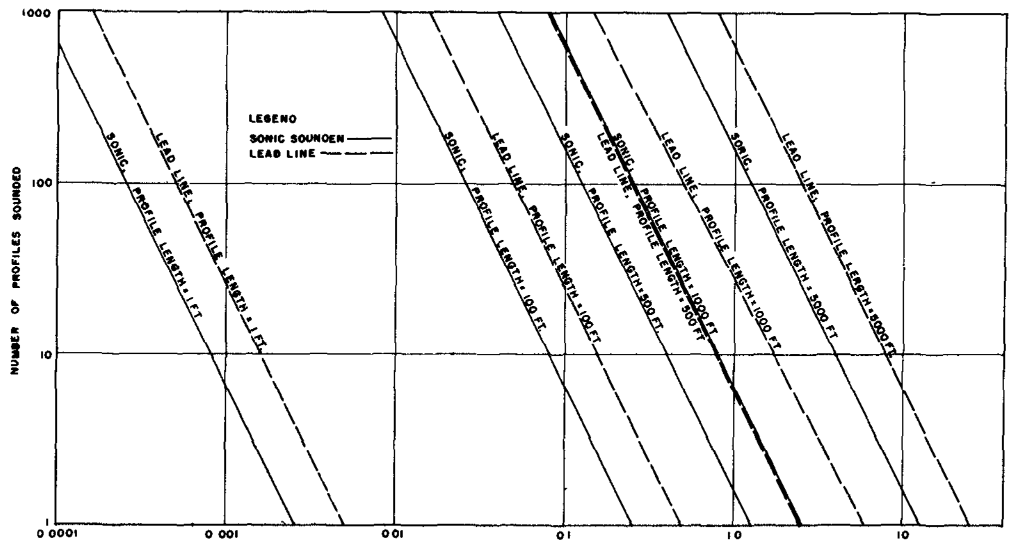

Fig. 2. Relation of probable sounding error in a beach survey to the number of profiles sounded for different profile lengths (does not include spacing error).

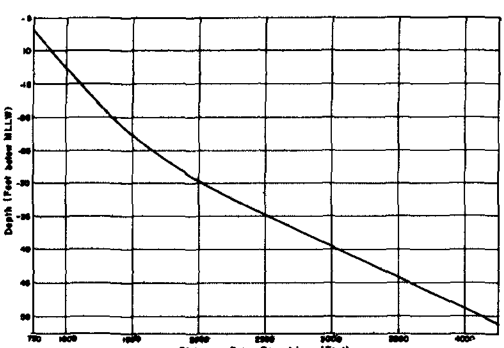

Fig. 3a. Average profile for entire test section 9 June 1950 (sonic data).

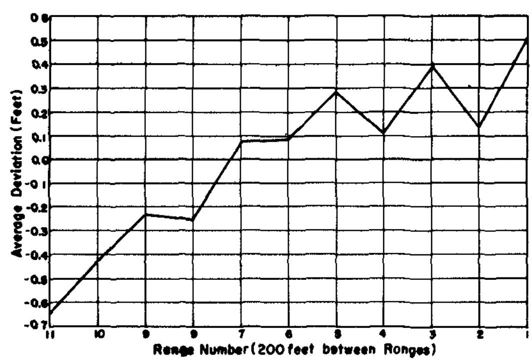

Fig. 3b. Average deviation of each profile from the average profile for the entire test section, 9 June 1950 (sonic data).

ACCURACY OF HYDROGRAPHIC SURVEYING IN AND NEAR THE SURF ZONE

DETERMINATION OF THE SPACING ERROR

DESCRIPTION OF TEST

As stated in the introduction, the spacing error is considered as the error resulting from the fact that a particular profile may not be entirely representative of its assigned section of beach. The tests to determine spacing error involved the use of data obtained from two different sets of surveys. These were:

(a) The sounding at Mission Beach of a 2,000-foot test section consisting of eleven ranges spaced two hundred feet apart at approximately one week intervals between 12 May and 8 September 1950. In addition, three surveys were made in April 1951, making a total of nineteen surveys. The ranges involved were established by the Field Research Group of the Beach Erosion Board in connection with other work, and were designated Beach Erosion Board ranges 126-146. The mid-range of the section was about 5,500 feet north of the Mission Bay jetties and the -50-foot contour is about 4,250 feet offshore. All surveys extended from the shore line to the -50-foot mean lower low water contour.

(b) The sounding at Mission Beach of a 9,200-foot section of beach consisting of 47 ranges spaced two hundred feet apart at approximately three month intervals between June 1949 and April 1951. A total of eight surveys were involved. Again, all surveys extended to the -50-foot mean lower low water contour. The ranges involved were Beach Erosion Board ranges 78-170; range 170 is about 2,100 feet north of the Mission Bay jetties; range 78 is slightly over two miles north of the jetties, and about 2,000 feet south of Crystal Pier.

The entire beach in the Mission Beach area is sand and has essentially straight and parallel contours, with no radical changes in underwater hydrography along its length; this uniformity of the beach was considered desirable for this study as the profiles might reasonably be expected to be representative of an extensive section of beach.

ANALYSIS OF THE ECHO SOUNDER DATA

The echo sounder data and the lead-line soundings were analysed separately. The echo sounder charts were corrected for tide elevation, and as in the analysis for "sounding error", soundings were taken off at 250-foot intervals along each range starting from a point 750 feet from the baseline. A tabulation of the soundings of the eleven profiles for the 2,000-foot test section for the survey of 12 May 1950 is shown in Table 8, as is an "average" profile obtained by averaging the eleven separate profiles. The deviation of any particular profile from this "average" profile is a measure of the error involved if only that profile were used to determine the hydrography of the area. Similarly, the error involved in using any particular set of profiles to indicate this hydrography may be measured as the sum of the deviations of the profiles from the average profile, if these deviations are weighted according to the area which each profile is assumed to represent. For the 12 May 1950 survey of the 2,000-foot test section, a tabulation of the deviation of

TABLE 8

SOUNDINGS TAKEN ON TEST SECTION AT MISSION BAY, CALIFORNIA
12 MAY 1950

SOUNDINGS IN FEET MLLW FOR RANGE NUMBER

Distance from Base Line (ft.)	(1) R-126	(2) R-128	(3) R-130	(4) R-132	(5) R-134	(6) R-136	(7) R-138	(8) R-140	(9) R-142	(10) R-144	(11) R-146	Average
					Sonic Soundings							
750	-8.3	-8.5	-9.5	-6.2	-6.2	-6.7	-6.6	-6.7	-6.5	-6.0	-6.0	7.02
1000	15.3	15.3	16.1	13.0	11.7	12.3	12.1	12.1	11.7	11.3	11.7	12.96
1250	21.0	21.3	21.2	18.7	18.7	18.3	18.3	17.8	17.8	17.5	17.8	18.95
1500	25.2	25.0	25.2	23.2	23.3	22.9	22.8	22.6	22.6	22.4	22.2	23.40
1750	28.3	28.3	28.4	26.9	26.8	26.8	26.8	26.4	26.2	26.0	26.1	27.00
2000	31.2	31.4	31.2	30.6	30.0	29.9	29.7	29.8	29.4	29.1	29.8	30.19
2250	33.3	33.4	33.1	32.6	32.5	32.3	32.4	32.3	31.7	31.7	32.1	32.49
2500	35.8	35.7	35.8	35.0	35.0	35.0	35.2	34.9	34.2	34.2	34.3	35.01
2750	38.0	38.0	38.3	37.3	37.1	37.2	37.4	37.1	37.0	36.4	36.5	37.30
3000	40.3	40.4	40.6	40.0	39.6	39.7	39.7	39.7	39.7	38.9	39.2	39.80
3250	42.5	42.6	42.9	41.9	42.0	42.1	42.0	42.1	41.9	41.9	42.0	42.17
3500	44.6	44.9	44.9	44.4	44.7	44.3	44.5	44.4	44.1	44.1	43.9	44.44
3750	46.8	47.0	47.4	46.8	46.6	46.9	46.9	46.7	46.3	46.6	46.3	46.75
4000	48.8	49.3	49.5	49.0	48.9	49.3	49.1	49.0	48.2	49.2	48.8	49.01
4250	51.5	51.7	52.4	51.2	51.2	51.8	51.1	51.4	50.9	51.3	51.0	51.41
					Lead-line Soundings							
250	+0.1	+0.7	+0.6	+2.2	+2.0	+2.0	+2.2	+0.8	+0.9	+2.0	+1.1	+1.327
300	-0.9	-0.5	-0.4	-1.1	+1.9	+0.3	0.0	0.0	+0.5	+0.8	+0.5	+0.100
350	-1.5	-1.3	-1.6	-2.0	-0.3	+0.1	-0.1	-0.9	-0.3	-0.1	-0.8	-0.800
400	-2.3	-2.5	-3.0	-1.0	-1.2	-0.7	-0.8	-1.8	-1.5	0.0	-1.5	-1.482
450	-4.1	4.[illegible]	4.7	2.0	1.3	1.7	1.7	2.8	2.0	0.3	0.9	-2.345
500	-5.3	5.5	6.1	3.2	2.0	2.5	2.7	3.0	3.0	3.0	1.7	-3.455

TABLE 9

DEVIATION (in feet) OF ACTUAL PROFILE FROM AVERAGE PROFILE (12 MAY 1950)
PROFILE BEING COMPARED TO AVERAGE PROFILE

Distance from Base Line (ft)	(1) R-126	(2) R-128	(3) R-130	(4) R-132	(5) R-134	(6) R-136	(7) R-138	(8) R-140	(9) R-142	(10) R-144	(11) R-146
						Sonic Soundings					
750	-1.28	-1.48	-2.48	+0.82	+0.82	+0.32	+0.42	+0.32	+0.52	+1.02	+1.02
1000	-2.34	-2.34	-3.14	-0.04	+1.26	+0.66	+0.86	+0.86	+1.26	+1.66	+1.26
1250	-2.05	-2.35	-2.25	+0.25	+0.25	+0.65	+0.65	+1.15	+1.15	+1.45	+1.15
1500	-1.80	-1.60	-1.80	+0.20	+0.10	+0.50	+0.60	+0.80	+0.80	+1.00	+1.20
1750	-1.30	-1.30	-1.40	+0.10	+0.20	+0.20	+0.20	+0.60	+0.80	+1.00	+0.90
2000	-1.01	-1.21	-1.01	-0.41	+0.19	+0.29	+0.49	+0.39	+0.79	+1.09	+0.39
2250	-0.81	-0.91	-0.61	-0.11	-0.01	+0.19	+0.09	+0.19	+0.79	+0.79	+0.39
2500	-0.79	-0.69	-0.79	+0.01	+0.01	+0.01	-0.19	+0.11	+0.81	+0.81	+0.71
2750	-0.70	-0.70	-1.00	0.00	+0.20	+0.10	-0.10	+0.20	+0.30	+0.90	+0.80
3000	-0.50	-0.60	-0.80	-0.20	+0.20	+0.10	+0.10	+0.10	+0.10	+0.90	+0.60
3250	-0.33	-0.43	-0.73	+0.27	+0.17	+0.07	+0.17	+0.07	+0.27	+0.27	+0.17
3500	-0.16	-0.46	-0.46	+0.04	-0.26	+0.14	-0.06	+0.04	+0.34	+0.34	+0.54
3750	-0.05	-0.25	-0.65	-0.05	+0.15	-0.15	-0.15	+0.05	+0.45	+0.15	+0.45
4000	+0.21	- .29	-0.49	+0.01	+0.11	-0.29	-0.09	+0.01	+0.81	-0.19	+0.21
4250	-0.09	-0.29	-0.99	+0.21	+0.21	-0.39	+0.31	+0.01	+0.51	+0.11	+0.41
Total d	-13.00	-14.90	-18.60	+1.10	+3.60	+2.40	+3.30	+4.90	+9.70	+11.30	+10.20
Ave. d	-.8667	-.9933	-1.2400	+.0733	+.2400	+.1600	+.2200	+.3267	+.6467	+.7533	+.6800
						Lead-line Soundings					
250	+1.23	+0.63	+0.73	-0.87	-0.67	-0.67	-0.87	+0.53	+0.43	-0.67	+0.23
300	+1.00	+0.60	+0.50	+1.20	-1.80	-0.20	+0.10	+0.10	-0.40	-0.70	-0.40
350	+0.70	+0.50	+0.80	+1.20	-0.50	-0.90	-0.70	+0.10	-0.50	-0.70	0.0
400	+0.82	+1.02	+1.52	-0.48	-0.28	-0.78	-0.68	+0.32	+0.02	-1.48	+0.02
450	+1.75	+1.95	+2.35	-0.35	-1.05	-0.65	-0.65	+0.45	-0.35	-2.05	-1.45
500	+1.85	+2.05	+2.65	-0.25	-1.45	-0.95	-0.75	-0.45	-0.45	-0.45	-1.75
Total d	+7.35	+6.75	+8.55	+0.45	-5.75	-4.15	-3.55	+1.05	-1.25	-6.05	-3.35
Ave. d	+1.22	+1.09	+1.42	+0.07	-0.96	-0.69	-0.59	+0.17	-0.21	-1.01	-0.56

ACCURACY OF HYDROGRAPHIC SURVEYING IN AND NEAR THE SURF ZONE

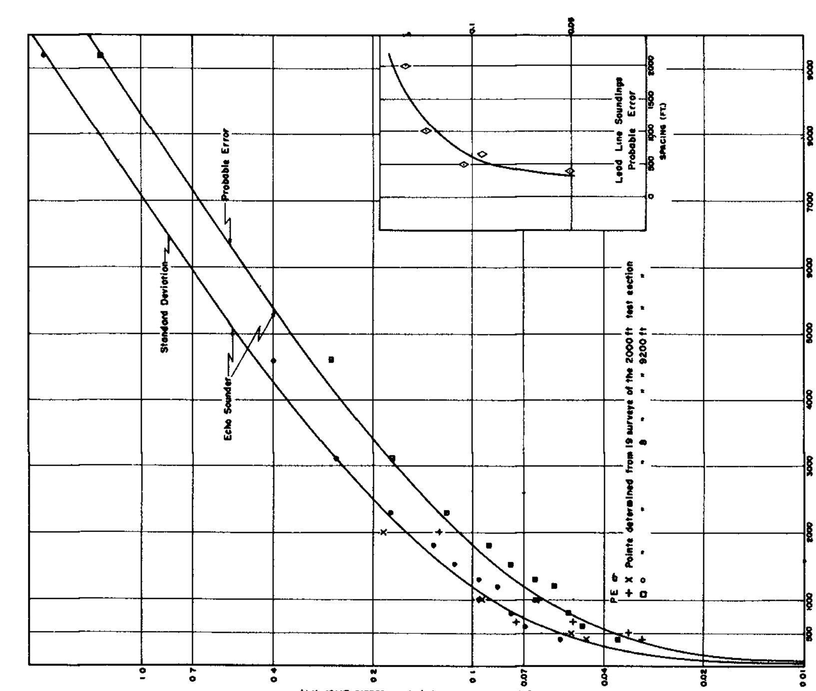

Fig. 4. Spacing error.

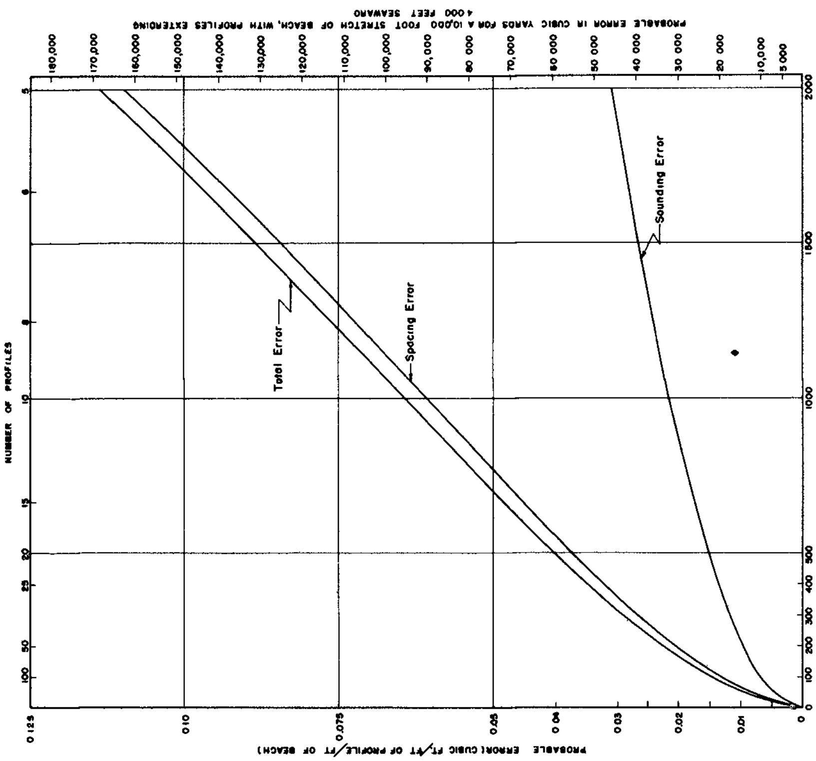

Fig. 5. Total probable error in surveying by echo-sounder a 10,000-foot stretch of beach; profile length equal 4000 feet.

each sounding and the overall deviation of each range from the average profile is shown in Table 9. Similar tabulations were made for each of the nineteen surveys of the 2,000-foot test section and each of the eight surveys of the 9,200-foot section. Figure 3 shows a typical average profile, and the average deviation of each individual profile from this average profile.

The error involved in using a number of different combinations of profiles rather than the full number of profiles was determined for each survey. The combined error for a series of evenly spaced profiles was determined as the algebraic sum of the deviations of each individual profile from the average profile determined from full survey data. This gave a variation of profile spacing of 400 to 2,000 feet for the test section and 400 to 9,200 feet for the full section. A tabulation of these errors (for the combinations of profiles selected) for the test section surveys is shown in Table 10, and for the full section survey in Table 11. The nineteen different values (one for each survey) involved in the test section and the eight different values involved in the full survey may be analyzed statistically to obtain a standard deviation and a probable error by the formulae used in the preceding section, and these values are also shown in Tables 10 and 11.

Several of the combinations of profile lines used have the same spacing, and these may be combined to give a single value of the standard deviation for each spacing. For example, in the test section, using a combination of ranges 3 and 9 gives a 1,000-foot spacing, as does also the combination of ranges 1, 6, and 11. The former results in a probable error of 0.072 and the latter in one of 0.053. These may be combined by taking the square root of the sum of the squares to give a single, more accurate value of 0.064 for the probable error. This combining has been done for both the test section and the full survey, and values of standard deviation and probable error for the various spacings are shown in Table 12. These values have been plotted in Figure 4, and curves drawn to fit the points. The scatter is surprisingly small, and it is thought that the curve represents fairly accurately the error which may be expected due to profile spacing on beaches having a hydrography generally similar to that of Mission Beach and sounded by sonic methods.

Due to the large number of surveys and profiles used, the sounding error (discussed previously) is negligible (each point plotted represents the results from the combination of a minimum of 57 profiles, and most points are obtained from several hundred profiles) -- and hence the error determined by this method may be attributed entirely to spacing error. This source of error is of greater magnitude than the sounding error, and may reach considerable values if the spacing is large.

That portion of the curves for spacings between 100 and 2,500 feet may be represented very closely by the linear functions

$$\sigma = 0.02 + 7.2s \times 10^{-5}$$

and

$$P.\ E. = 0.013 + 4.84s \times 10^{-5}$$

TABLE 10

ERROR INTRODUCED BY USING GIVEN PROFILES ONLY RATHER THAN ALL 47 PROFILES (9200' Section)
(CUBIC FEET PER FOOT OF PROFILE PER FOOT OF BEACH)

Lines Number	Average Spacing (feet)	Jun 1949	Oct 1949	Feb 1950	Apr 1950	Jun 1950	Sep 1950	Dec 1950	Apr 1951	Standard Deviation (feet)
1,3,5,7,...47	400	-0.0964	+0.0274	-0.0513	+0.0565	+0.0385	+0.0417	+0.0521	+0.0633	.0568
2,4,6,8, ..46	400	+0.0033	+0.0743	+0.0759	-0.0491	+0.0477	+0.0466	+0.0492	+0.0306	.0519
2,5,8,11,...47	600	+0.1062	+0.0402	+0.0072	+0.0716	+0.0707	+0.0473	+0.0700	+0.0089	.0615
1,4,7,10,...46	600	+0.0485	+0.0632	+0.1007	+0.0350	+0.0507	+0.0372	-0.0054	+0.0726	.0581
3,6,9,12, .45	600	+0. 012	+0.0488	+0.0659	+0.0649	+0.0804	+0.0522	+0.0147	+0.0541	.0539
1,4,7,. 22,26,29,32,.. 47	600	-0.0762	-0.1004	-0.1023	-0.1051	-0.1087	-0.0750	-0.0579	-0.1197	.0952
1,5,9,13,. .45	800	+0.0204	-0.0306	-0.0198	+0.0552	+0.0541	+0.0087	+0.0203	+0.1556	.0637
2,6,10,14,...46	800	-0.0560	+0.0786	+0.0243	-0.0006	+0.0910	+0.0239	+0.0449	+0.1006	.0622
3,7,11,15,...47	800	+0.1723	+0.0853	+0.0681	+0.0578	+0.0230	+0.0747	+0 0840	-0.0325	.0860
1,5,9,..21,24,27,31,35, 47	800	+0.1696	-0.0032	+0.1006	+0.1002	+0.0562	+0.0962	+0.0511	-0.0365	.0903
1,6,11,16,...46	1000	+0.0903	+0.0006	+0.0133	-0.0649	+0.0515	-0.0057	-0.0680	+0.0109	.0500
2,7,12,17,...47	1000	-0.1041	-0.0247	+0.0282	-0.0677	+0.1690	+0.0300	-0.0414	+0.1457	.0930
1,6,11,16,21,27,32,37,42,47	1000	+0.3141	-0.0595	-0.0376	-0.0142	+0.0444	+0.0177	-0.0871	+0.0381	.120
4,9,14,...44	1000	+0.1332	+0.0533	+0.0381	+0.2733	+0.0448	+0.0487	+0.0949	+0.0403	.104
1,7,13,19,24,29,35,41,47	1200	+0.0944	-0.0202	+0.0317	-0.0032	+0.0126	+0.1253	-0.0252	-0.0553	.0611
4,10,16,22,26,32,38,44	1200	+0.0769	+0.1072	+0.0981	+0.1400	+0.0713	+0.0232	+0.0801	+0.0520	.0876
4,10,16,21,26,32,38,44	1200	+0.0633	+0.1066	+0.0924	+0.2138	+0.0801	+0.0211	+0.0887	+0.0001	.103
4,10,16,22,27,32,38,44	1200	+0.0745	+0.0904	+0.0735	+0.1058	+0.0757	+0.0408	+0.0445	-0.1039	.0794
1,8,15,21,27,33,40,47	1300	+0.0577	+0.0883	+0.1244	+0.0884	+0.1297	+0.1011	+0.0032	+0.1339	.0996
4,10,17,24,31,38,44	1350	+0.1293	+0.0327	+0.0341	+0.1008	+0.1136	+0.0922	+0.1286	-0.0313	.0922
1,8,16,24,32,40,47	1550	+0.0428	+0.0229	+0.0397	+0.1207	+0.1198	+0.1029	-0.0023	-0.0009	.0737
5,13,21,28,35,43	1600	-0.0947	-0.0194	+0.2304	+0.1104	-0.0370	+0.0786	-0.0716	-0.0744	.108
5,13,20,27,35,43	1600	+0.0302	+0.0033	+0.2401	+0.1797	-0.0600	+0.2006	+0.1268	-0.1380	.146
1,10,19,29,38,47	1800	+0.0705	+0.1106	-0.1924	-0.0378	+0.1524	+0.0562	-0.3023	+0 1174	.145
6,15,24,33,42	1800	-0.0037	-0.0478	+0.0452	+0.1646	+0.1239	+0.1656	+0.1931	+0.0224	.118
1,12,24,36,47	2300	-0.2072	-0.0432	+0.1793	-0.1527	+0.1822	+0.0045	-0.2654	-0.2633	.185
1,13,24,35,47	2300	+0.2162	-0.0548	+0.2481	-0.0143	+0.0478	+0.1328	-0.1233	-0.3937	.194
7,18,30,41	2300	+0.0303	+0.0287	-0.0543	+0.0337	+0.0483	+0.1787	+0.3220	-0.2090	.153
1,16,32,47	3100	-0.3222	-0.2343	+0.0363	-0.0792	+0.1716	-0.0434	-0.2871	-0.1785	.197
9,24,39	3100	+0.4806	-0.3031	+0.2817	+0.3678	+0.4065	+0.1808	+0.0499	+0.1995	.312
1,20,47	4600	-0.2780	-0.4947	-0.7433	-0.3263	-0.0833	-0.0780	+0.2187	-0.5777	.415
13,35	4600	+0.3387	-0.4653	-0.6673	-0.3120	+0.1900	+0.4920	+0.2820	-0.1427	.396
12,36	4600	-0.9133	-0.3287	+0.3373	-0.1547	+0.2867	+0.0720	-0.1647	-0.0320	.387
1,47	9200	-2.3080	-2.3280	-2.2993	-2.4447	-1 9867	-2.0247	-2.3313	-2.1193	2.236
24	9200	+1.7520	+1.3387	+2.1507	+1.7920	+1.9701	+1.7353	+1.4353	-0.9640	1.680

TABLE 11

ERROR INTRODUCED BY USING GIVEN PROFILES ONLY, RATHER THAN ALL ELEVEN PROFILES (2000' Test Section)
(CUBIC FEET PER FOOT OF PROFILE PER FOOT OF BEACH)

Lines #	6	1,11	3,9	1,6,11	3,6,9	1,4,8,11	2,5,7,10	1,4,6,8,11	1,3,6,9,11	2,4,6,8,10	1,3,5,7,9,11
Average Spacing (feet)	2000	2000	1000	1000	700	600	500	500	500	400	400
12 May 1950	-.1600	+.0934	+.2966	-.0333	+.1596	-.1120	-.0550	-.1040	+.1190	-.0640	+.0454
18 May	+.1840	-.1993	-.0860	-.0076	-.0050	+.0037	+.0506	+.0223	-.0277	+.0240	+.0153
26 May	+.1980	-.0020	-.0953	+.0080	-.0073	+.0400	-.0294	+.0680	+.0113	+.0047	-.0047
9 June	+.0827	-.0740	+.0760	+.0044	+.0780	-.0740	+.0160	+.0427	+.0480	- 0720	+.0873
16 June	+.0513	-.3387	+.0180	-.1436	+.0280	-.1450	+.0547	-.1223	-.0433	+.0167	+.0507
21 June	+.1547	-.2887	-.0987	-.0669	-.0227	-.0753	+.0380	+.0600	-.0606	-.0907	-.0340
23 June	+.2693	-.2240	-.0264	+.0227	+.0992	-.0210	+.0110	+.0197	+.0492	+.0653	-.0105
30 June	+.2233	-.1700	+.0333	-.2283	+.0903	-.0440	+.0100	-.0013	+.1497	+.0353	+.0007
7 July	+.0900	+.0200	-.0767	+.0450	-.0267	+.0293	-.0100	+.0273	+.0144	+.0260	-.0293
21 July	+.2420	-.2780	+.0220	-.0180	+.0880	-.0190	+.0220	+.0110	+.0280	+.0487	+.0073
4 August	+.0187	-.0698	+.0287	-.0255	+.0257	+.0132	+.0037	+.0071	+.0059	-.0160	+.0396
11 August	+.2620	-.1713	+.0154	+.0453	+.0893	-.0453	+.0087	+.0053	+.0520	-.0233	+.0580
18 August	+.0993	-.2107	+.1727	-.0556	+.1507	-.0263	-.0323	-.0170	+.0740	-.0727	+.1147
25 August	+.0353	-.1380	+.1220	-.0513	+.0960	-.0283	-.0097	+.0248	+.0452	+.0153	+.0127
1 September	+.1153	-.2414	+.0787	-.0631	+.0897	-.0650	+.0470	-.0443	+.0257	+.0033	+.0453
8 September	+.2073	-.1094	+.0006	+.0489	+.0627	-.0113	-.0127	+.0240	+.0406	+ 0167	+.0053
24 April 1951	+.2247	-.4353	+.1180	-.1052	+.1600	-.0410	+.0380	-.0217	+.0393	+.0473	+.0393
27 April	-.0560	-.1526	+.1473	-.1043	+.0863	-.0476	+.0190	- 0584	+.0263	-.0120	+.0433
28 April	+.1680	-.1086	-.0487	+.0347	+.0163	-.0807	+.0713	-.0354	+.0044	-.0414	+ 0633
Stan. Devia.	.169	.205	.107	.079	.086	.060	.034	.056	.058	.045	.047
Probable Error	.114	.139	.072	.053	.058	.041	.023	.038	.039	.030	.032

TABLE 12

Spacing Error

	Spacing	Sonic Standard deviation (ft.)	Sonic Probable error (ft.)	Lead Line Standard Deviation (ft.)	Lead Line Probable error (ft.)
2000-Foot Test Section	2000	0.188	0.127	0.236	0.159
	1000	0.094	0.064	0.206	0.139
	650	0.074	0.050	0.140	0.094
	500	0.051	0.034	0.158	0.107
	400	0.046	0.031	0.0751	0.051
9200-Foot Full Section	9200	1.977	1.333		
	4600	0.399	0.269		
	3100	0.260	0.175		
	2300	0.178	0.120		
	1800	0.132	0.089		
	1525	0.113	0.076		
	1300	0.096	0.065		
	1200	0.084	0.057		
	1000	0.095	0.064		
	800	0.077	0.052		
	600	0.069	0.047		
	400	0.054	0.037		

where s is the spacing in feet.

It was suspected that the spacing error might decrease somewhat as the number of profiles at that spacing was increased -- the spacing error between one set of profiles tending to compensate somewhat for the spacing error between the next set of profiles. If this were true, then the points obtained from the 9,200-foot section, having many more profiles, should lie somewhat beneath the points determined from the 2,000-foot test section. Such is not the case however, and it is thought that the curve shown is an accurate portrayal of the spacing error.

ANALYSIS OF LEAD-LINE DATA

A similar analysis was performed on the lead-line data for the 2,000-foot test section. The points determined are also shown in Table 12 and are plotted on Figure 4, where a curve of best fit has been drawn in. As with the sonic data the number of profiles used to determine the points is large, and the sounding error is therefore negligible. The analysis for the lead-line data for the full 9,200-foot section has not yet been completed, and with the relatively small number of points used to determine the curve, it is not thought that as complete reliance should be placed in this curve as in that for the echo-sounder data.

It may be noted that the spacing error as determined from the lead-line data is larger than (roughly twice) that determined from the sonic data. This if, of course, not due to the different methods of surveying, but to the fact that the inshore portion of the beach (where the lead-line data was taken) is much less regular than the offshore portion, and a particular profile there would be expected to be much less representative of the surrounding area than it would farther offshore where the hydrography is more regular.

APPLICATION TO ACTUAL SURVEY

The total error to be expected in any particular survey will be a combination of the sounding error and the spacing error, and may be determined, for beaches similar in hydrography to Mission Beach, from the curves shown herein. If e denotes the total probable error, e_s the probable spacing error, and e_a the sounding error, then

$$e = \sqrt{e_a^2 + e_s^2}$$

and the probable yardage error is

$$E = \frac{\sqrt{(e_a^2 + e_s^2)}\,LL'}{27}$$

where L is the length of the beach in feet and L' the length of the profile in feet.

An example of the combined error for a 10,000-foot stretch of beach is shown in Figure 5. As may be readily seen, the probable error for a large spacing reaches a quite considerable cubage. It is interesting to note that, at least for this particular case, while the sounding error is quite appreciable, it is so small in comparison to the spacing error that it has only a relatively small effect on the total error.

The above analysis appears to demonstrate that the cubage errors -- due to the fact that profiles of a hydrographic survey are not strictly comparable either among themselves or to a previous survey (sounding error), and that any particular profile does not necessarily represent accurately the bottom area which it is assumed to describe (spacing error) -- can introduce serious misinterpretations as to the rate and direction of movement of littoral drift. For instance, in the Mission Bay area, for a 10,000-foot stretch of beach, it is seen that for a very small range spacing (200 feet) an error of almost 35,000 cubic yards can still be more or less expected in the cubage computations; while for the relatively large spacing of 1,000 feet, an error of almost 100,000 yards can be expected. In many beach studies errors of these magnitudes could produce completely misleading interpretations of the test data. It is therefore recommended that the presence of such errors be considered as a distinct possibility in the interpretation of test data based on the comparison of successive hydrographic surveys.

CHAPTER 4

ENVIRONMENTAL ASPECTS OF THE EBB SIDE AND FLOOD SIDE OF TIDAL ESTUARIES AS A FACTOR IN HARBOR LOCATIONS

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In a previous paper presented at the Second Conference on Coastal Engineering concerning the environmental aspects of harbors, mention was made of the fact that owing to the differences between the ebb and flood side of an estuary, the ebb side presented certain advantages which might make it the more suitable location for a harbor. No expanded discussion was attempted at that time, but the problem seemed to be of sufficient interest to warrant further study.

Generally speaking, the establishment of a new harbor or the development of an old one depends too often on local interests, adequate transportation, or access to the hinterland. Even though exhaustive studies of the phenomena of sedimentation, currents, salinity, and fouling at the proposed location are sometimes made, the viewpoint is quite often that remedial measures will be taken to combat these phenomena where they are found to be detrimental. In general, these forces of nature can be modified or at least ameliorated, but the possibility of cooperating with natural forces rather than combating them is often overlooked. It would seem more logical to choose a harbor site or develop a harbor where sedimentation is at a minimum rather than one where sedimentation is so rapid as to require almost constant dredging. Also it would be more logical to select a site where the incidence of marine borers and fouling organisms is slight rather than to develop and apply artificial measures which guarantee no permanent immunity from attack. It is the purpose of this paper to show that environmental characteristics of the ebb side of an estuary may provide a more suitable location for a port than the flood side.

The ebb side of an estuary in the northern hemisphere is considered to be the right side looking seaward toward the mouth of the estuary and the flood side the left bank looking in the same direction. Because of the Coriolis force, moving objects in the northern hemisphere are deflected to the right, therefore, it should be expected that the right side of an estuary would exhibit certain peculiarities. Recent observations have shown considerable differences in the two sides of most estuaries. Probably the clearest example is in the salinity distribution patterns of two bays along the east coast of the United States. It has been shown by Pritchard (1952) that surface isohalines in the Chesapeake Bay (Fig. 1) run obliquely across the estuary rather than running perpendicular to its long axis. The resulting effect is that salinities are lower on the ebb side and higher on the flood side. Although the salinity gradient of the ebb side is probably accented by the excess of fresh water inflow from the rivers of the western shore of the bay it should not be supposed that this is completely the cause of the lowered salinities. It has been pointed out (Pritchard, 1952) that even in narrow estuaries where the river inflow from both shores is approximately equal, the lateral gradient shows the salinities of less value to be on the ebb side. A similar large spacing reaches a quite considerable cubage. It is interesting to note that, at least for this particular case, while the sounding error is quite appreciable, it is so small in comparison to the spacing error that it has only a relatively small effect on the total error.

The above analysis appears to demonstrate that the cubage errors -- due to the fact that profiles of a hydrographic survey are not strictly comparable either among themselves or to a previous survey (sounding error), and that any particular profile does not necessarily represent accurately the bottom area which it is assumed to describe (spacing error) -- can introduce serious misinterpretations as to the rate and direction of movement of littoral drift. For instance, in the Mission Bay area, for a 10,000-foot stretch of beach, it is seen that for a very small range spacing (200 feet) an error of almost 35,000 cubic yards can still be more or less expected in the cubage computations; while for the relatively large spacing of 1,000 feet, an error of almost 100,000 yards can be expected. In many beach studies errors of these magnitudes could produce completely misleading interpretations of the test data. It is therefore recommended that the presence of such errors be considered as a distinct possibility in the interpretation of test data based on the comparison of successive hydrographic surveys.

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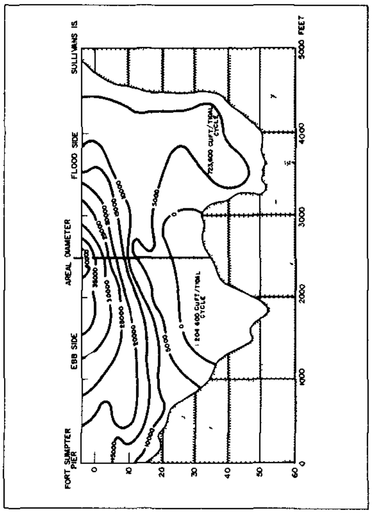

Fig. 2. Net outflow through the mouth of Charleston Harbor during one tidal cycle on 26 October 1950.

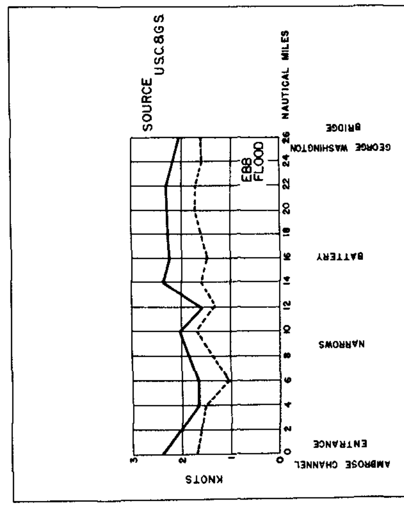

Fig. 3. Maximum ebb and flood current velocities in New York Harbor.

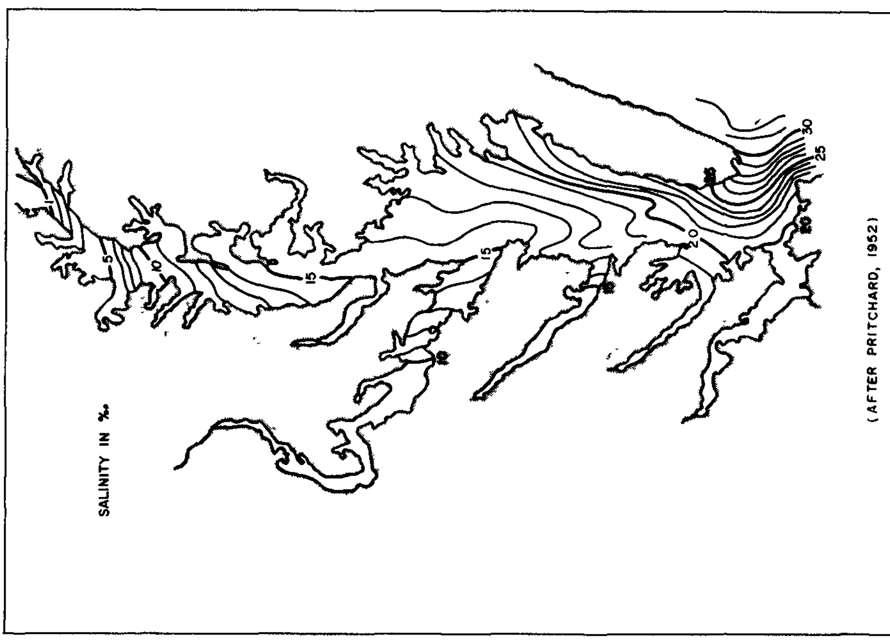

Fig. 1. Typical surface salinity pattern in the Chesapeake Bay.

condition obtains in Delaware Bay (Nelson, 1947) where the deflection of the ebb tide resulting from the earth's rotation carries out along the Delaware coast (ebb side) approximately five times as much fresh water as along the New Jersey shore (flood side). High salinities on the flood side near the mouth of Delaware Bay are further accented by excessive evaporation over the exposed flats in hot weather.

A further difference in the characteristics of the flood and ebb sides is in the large volume of water transported along the latter. This is strikingly shown in the net discharge through the mouth of Charleston Harbor. From data of the U. S. Corps of Engineers the net outflow through the mouth of Charleston Harbor has been computed for one tidal cycle (Fig. 2). The total transport during ebb tide and during flood tide has been measured in linear feet between Sullivans Island (flood side) and Fort Sumter (ebb side) and the net transport per tidal cycle is shown in the diagram. The areal diameter divides the total area into equal parts. When the net discharge volume is computed it is found that the total discharge from the ebb side is in the magnitude of 1,204,600 cubic feet per tidal cycle and from the flood side, 723,600 cubic feet per tidal cycle. This difference of 481,000 cubic feet shows the ebb side to carry out almost 25% more water than the flood side.

As a consequence of a greater amount of water to be discharged on the ebb side, the current velocities here must be greater during the tidal cycle. Higher ebb velocities are apparent in most inlets. At the entrance to Charleston Harbor, for example, maximum ebb velocities at the surface are consistently higher than the maximum flood velocities with the former varying between 1.8 to 3.3 knots and the latter varying only between 0.7 to 2.5 knots. This is further evidenced in New York Harbor (Fig. 3). In a section from above the George Washington Bridge to the Ambrose Channel Entrance, maximum ebb current velocities are greater than flood velocities at all stations. The maximum flood velocity is 1.7 knots and the maximum ebb velocity 2.4 knots (U. S. Coast and Geodetic Survey, 1951).

An additional important characteristic of the tide which will affect the ebb and flood sides is the duration of the respective flows. In general it may be said that the duration of ebb is greater than that of flood. Water particles or contaminants, therefore, experience a net seaward gain during the ebb tide. Even in places where the two tides are equivalent in duration or the flood tide continues slightly longer than the ebb tide, the greater velocities of the ebb current should compensate for this difference and particles will still experience a net seaward gain.

What then is the significance of these facts in choosing one or the other side of an estuary as a harbor site? From the standpoint of biological implications alone the ebb side would seem worthy of consideration. The problems encountered with fouling and boring organisms are of economic importance in the maintenance of a harbor. Although various methods have been developed to discourage both fouling and boring organisms the more logical approach would be to choose a location which would be less susceptible to infestation. The ebb side appears to present an environment which is unfavorable for foulants and borers. In the lower salinity it is to be expected that there will be a less favorable environment for the development of mussels, oysters, barnacles, and borers. Although these will be present in some amount on the ebb side they will probably not be as numerous or grow as rapidly and well as those on the flood side. For example, in San Francisco Bay, creosoted wooden pilings may be expected to last only from 15-25 years on the San Francisco (flood)side, while elsewhere they may be expected to last from 20-30 years (Hcronjeff and Patrick, 1951).

Greater current speed may also tend to reduce the fouling problem on the ebb side. Fouling organisms have limiting current tolerance for the attachment of their larval forms. Three species of barnacles[1] have been shown to be unable to attach themselves at speeds greater than two knots and at least one of these[2] is unable to attach at speeds greater than 0.8 knots (Doochin, 1949). It has been shown (Smith, 1946) that after attachment of the larvae the growth rate is increased by currents less than 1.5 knots and decreased by currents in excess of this. Six hours after the larval form has attached, the growth rate is stopped by a current of 3 knots.

The problem of sedimentation is as important as the biological consideration. In this case the ebb side seems again to be a more suitable location. It has been shown (Sverdrup, Johnson, and Fleming, 1942) that higher current velocities favor the deposition of coarse particles rather than fine particles in suspension. Since the higher velocities occur on the ebb side it is logical to assume that the fine sediments would not be deposited here and would reduce dredging requirements. Strong currents however do not guarantee that only the coarse sedimentary materials will be deposited since they may still be accompanied by some small amount of fine materials. For example in Massachusetts Bay there are several estuaries where the current moves from 2-3 miles per hour. In one of these, Beverly Harbor, there is a tidal current channel with steep sides. In sediments examined from the shore to the north out into the channel there is a decided change found at the channel's edge. The strong current here does not eliminate the fine material but brings a mixture of coarse and some fine material together (Trowbridge and Shepard, 1932).

In areas where man disturbs the natural environment, the conditions postulated here are also upset, but farther removed from man's activity, where natural forces are allowed to act, the conditions are restored. Such a case occurred in San Pablo Bay where a considerable shoal was built up on the ebb shore in the years 1921 to 1931 due to the extensive government dredging in the vicinity, However, there is evidence that much of the light silt from the dredging here was carried farther downstream into San Francisco Bay and deposited on the eastern (flood) shores. This is further supported by the fact that between the years 1859 and 1903 there was marked shoaling south of Point Richmond (flood side). The sedimentary materials which occasioned this shoaling were probably the result of

1 *Balanus amphitrite*, *B. eburneus*, and *B. improvisus*
2 *Balanus eburneus*

hydraulic mining operations during the period (Beebe, 1931). The action of the tide is probably not the sole reason for deposition on the eastern (flood) shores of San Francisco Bay since the prevailing west and southwest winds keep the shoal water constantly agitated, thereby carrying silt in suspension on to the shore or into dredged channels.

Another important factor to be considered in the establishment of a harbor or the development of an already existing one is the problem of flushing. Even though theories of flushing are in a state of conflict and data are limited we may still make the conjecture at this point that the ebb side of an estuary should have a faster flushing time than the flood side. At least several of the conditions stated before seem to point to this. Since the greater volume of water is deflected to the ebb side, since the current velocity is greater, and since the ebb tide is of longer duration it seems that the net seaward gain of a water particle is greater and any contaminant would be more readily flushed from this side.

Little information is available to conclude definitely that the greater scouring action of the tidal forces would be on the ebb side but this is quite possible in view of the higher current velocities lasting over a longer period of time.

Unfortunately, we cannot say that the ebb side would be the best location for a harbor in every instance since various local factors may interfere with the expected conditions. The facts, however, indicate that it would be better to take advantage of the opportunity of cooperating with natural forces rather than fighting the battle of continued expense in harbor maintenance.

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CHAPTER 5

THE SALT WEDGE

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INTRODUCTION

The "salt wedge" is an idealized model of a phenomenon encountered in some estuaries. It consists of a layer of fresh river water flowing over and past an underlying layer of heavier salt water. The following study is an attempt to predict the shape of the salt wedge and its length measured from some significant point in the estuary in terms of quantities, such as the densities of the fresh and salt water, the average river depth and velocity, etc., which are usually available.

THEORETICAL ANALYSIS[2]

To simplify the mathematical treatment we assume that all variations in the direction of the estuary or channel width are negligible and that there is no velocity component in this direction. The channel has a plane horizontal bottom and its cross-section is rectangular and uniform.

We stipulate the existence of a sharp interface between the fresh water and the salt wedge, i.e., a density discontinuity, and the absence of mixing between the layers.

The equations of motion can then be written for each layer in the following form (Fig. 1):

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[1]Contribution No. 638 from the Woods Hole Oceanographic Institution. This work was supported from funds made available by the Office of Naval Research, through their Contract No. N6onr-27701 (NR-083-004) with the Woods Hole Oceanographic Institution.

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[2]For a more detailed account of the theory and additional investigations, see: L. Sanders, L. C. Maximon, G. W. Morgan, On the stationary "salt wedge" - a two layer free surface flow. Technical Report No. 1, Contract Nonr-56202 (NR-083-067). The theoretical results presented in this paper were obtained in the course of research sponsored by the Office of Naval Research under the above Contract with Brown University.

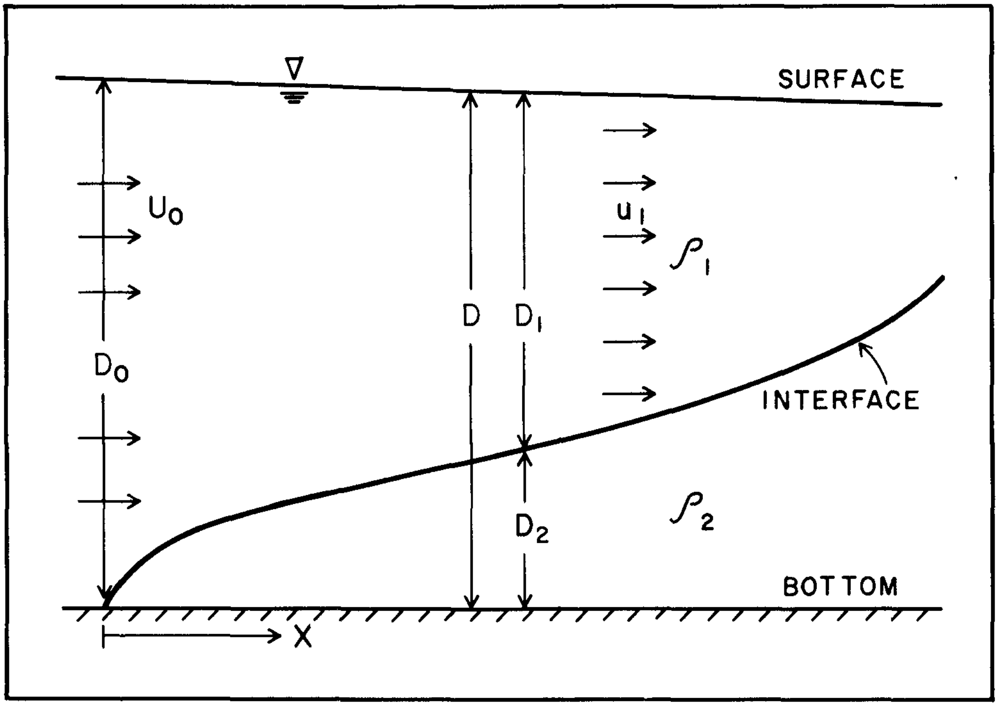

Fig. 1. Schematic diagram of salt wedge.

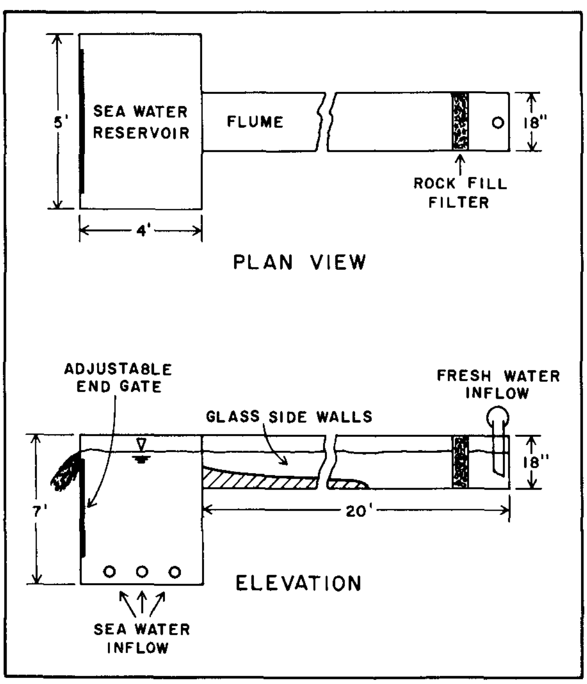

Fig. 2. Schematic diagram of experimental apparatus.

$$\frac{\partial}{\partial x}(p_i + \rho_i u_i^2) + \frac{\partial}{\partial z}(\rho_i u_i w_i) = \frac{\partial}{\partial x}(A_i \frac{\partial u_i}{\partial x}) + \frac{\partial}{\partial z}(A_i \frac{\partial u_i}{\partial z}) \tag{1}$$

$$\frac{\partial}{\partial z}(p_i + \rho_i w_i^2) + \frac{\partial}{\partial x}(\rho_i u_i w_i) = -\rho_i g + \frac{\partial}{\partial x}(A_i \frac{\partial w_i}{\partial x}) + \frac{\partial}{\partial z}(A_i \frac{\partial w_i}{\partial z}) \tag{2}$$

$$\frac{\partial u_i}{\partial x} + \frac{\partial w_i}{\partial z} = 0 \tag{3}$$

where the subscript i takes on the values 1 or 2 referring to the upper or lower layer, respectively;

- ρ is the density,
- p is the pressure,
- u is the horizontal velocity component,
- w is the vertical velocity component,
- g is the gravitational acceleration, and
- A is an eddy viscosity.

These equations apply to turbulent flow, the quantities u, w, p representing time averages of the actual time-dependent quantities.

If we neglect the inertia and shear forces in the vertical direction we can integrate the vertical momentum equation. Applying the condition $p = 0$ at the free surface $z = D$, we obtain

$$p_1 = \rho_1 g\ (D - z) \tag{4}$$

and

$$p_2 = \rho_2 g\ (D_2 - z) + \rho_1 g D_1 \tag{5}$$

where $z = D_2$ is the interface and $D_1 = D - D_2$, see Figure 1.

The solution is too difficult because of the presence of the unknown boundaries D, D_2. We therefore make a further approximation in our analysis by demanding that the laws of conservation of mass and momentum be satisfied only in the large rather than at every point of the flow, i.e., we integrate the equations over the depth. We then obtain the following equations from (1)

$$\rho_1 g D_1 \frac{d}{dx}(D_1 + D_2) + \rho_1 \frac{d}{dx}\int_{D_2}^{D} u_1^2 \, dz = \tau_1 - \tau_2 \tag{6}$$

$$g D_2 \frac{d}{dx}(\rho_1 D_1 + \rho_2 D_2) + \rho_2 \frac{d}{dx}\int_{o}^{D_2} u_2^2 \, dz = \tau_2 - \tau_o \tag{7}$$

Here τ_o, τ_1, τ_2 are the values of $A \frac{\partial u}{\partial z}$ at $z = 0$, D, D_2 respectively, and we have neglected the term $\frac{\partial}{\partial x}(A \frac{\partial u}{\partial x})$ compared with $\frac{\partial}{\partial z}(A \frac{\partial u}{\partial z})$.

The continuity equation (3) gives

$$\frac{d}{dx}\int_{D_2}^{D} u_1 dz = 0 \tag{8}$$

or

$$\int_{D_2}^{D} u_1 dz = q \tag{8a}$$

and

$$\frac{d}{dx}\int_{o}^{D_2} u_2 dz = 0 \tag{9}$$

or

$$\int_{o}^{D_2} u_2 dz = 0 \tag{9a}$$

since there can be no net flow within the stationary wedge.

Equations (6) to (9) can now be treated in an approximate manner by assuming plausible velocity distributions u_1, u_2. Since the flow is turbulent it is reasonable to suppose that u_1 is fairly uniform over the depth except near the interface where sharp velocity gradients may be expected; i.e., $u_1 = \frac{q}{D_1}$. Since there is zero net flow across any section of the wedge and since the velocities in the wedge are caused by friction at the interface, we may expect that u_2 will be small compared with u_1 except possibly near the interface and that, therefore, the integral and the term τ_o in equation (7), representing inertia forces and bottom shear forces, respectively, are negligible. We may also drop the free surface friction term τ_1.

We still know nothing concerning the nature of the interface shear force τ_2. At this point we content ourselves with developing a semi-empirical theory and take recourse to experimental results to determine an approximate law for τ_2.

It is convenient to rewrite equations 6, 7, 8a, 9a in non-dimensional terms by means of the following definitions:

$$D_1 = D_o n_1, \qquad D_2 = D_o n_2, \qquad z = D_o z', \qquad D = D_o D'$$

$$x = Lx', \qquad u_1 = U_o u_1', \qquad \tau_2 = \rho_2 U_o^2 \tau'$$

$$\beta = \frac{\rho_2 - \rho_1}{\rho_2}, \qquad \alpha = \frac{U_o^2}{g D_o \beta} \text{ and } \lambda = \frac{L}{D_o}$$

where D_o is a reference depth, taken to be the total depth at the tip of the wedge, L is the total length of the wedge, (to be defined later), and U_o is the uniform velocity of the fresh water at the tip of the wedge.

Incorporating the assumptions just discussed equations (6) and (7) become:

$$\frac{1-\beta}{\beta\alpha\lambda} n_1 \frac{d}{dx'}(n_1 + n_2) - \frac{(1-\beta)}{\lambda n_1^2}\frac{dn_1}{dx'} = -\tau' \qquad (10)$$

$$\frac{1}{\beta\alpha\lambda} n_2 \frac{d}{dx'}\left[(1-\beta)\, n_1 + n_2\right] = \tau' \qquad (11)$$

after putting

$$u_1' = \frac{1}{n_1} \qquad (12)$$

To determine the shear law we insert observed values of n_1, n_2, etc. in equations (10) and (11) and calculate τ'.

The values of τ' found in this manner for various positions along a given wedge and for various wedges are found to be somewhat erratic, but it appears that over the range of α and β covered by our experiments τ' can be taken as approximately constant, say equal to K. This says that the shear force at the interface varies from wedge to wedge as the square of the fresh water velocity at the tip and is approximately constant over a given wedge.

Now, recalling that u_1' is constant with depth, equations (10) to (12) can be converted into the following two non-linear equations for the two unknowns n_1 and n_2.

$$(1-\beta)\, n_2\, (n_1^3 - \alpha) \frac{dn_1}{dx'} = -\lambda \alpha K n_1^2 \left[(1-\beta)\, n_1 + n_2 \right] \tag{13}$$

$$n_2 \frac{d}{dx'} \left[(1-\beta)\, n_1 + n_2 \right] = \lambda \alpha \beta K \tag{14}$$

To solve these we make use of the fact that β is very small (approximately 0.025) and develop n_1, n_2 in power series of β; i.e., we assume the following forms of solutions:

$$n_1 = n_{10} + \beta n_{11} + \beta^2 n_{12} + \ldots \tag{15}$$

$$n_2 = n_{20} + \beta n_{21} + \beta^2 n_{22} + \ldots \tag{16}$$

For small β we may hope that n_1, n_2 will be given sufficiently accurately by the first term of the power series.

Inserting the series into equations (13) and (14) and equating the coefficients of like powers of β we obtain as the equations to be satisfied by n_{10} and n_{20}:

$$n_{20}\, (n_{10}^3 - \alpha) \frac{dn_{10}}{dx'} = -\lambda \alpha K n_{10}^2\, (n_{10} + n_{20}) \tag{17}$$

$$n_{20} \frac{d}{dx'} (n_{10} + n_{20}) = 0 \tag{18}$$

If we take the origin $x = 0$ at the tip of the wedge, our boundary conditions are

$$n_{10}\ (0) = 1\ , \qquad\qquad n_{20}\ (0) = 0 \tag{19}$$

Equation (18) gives

$$n_{10} = 1 - n_{20} \tag{20}$$

and then (17) becomes

$$n_{20}\left[(1 - n_{20})^3 - \alpha\right] \frac{dn_{20}}{dx'} = \lambda K \alpha (1 - n_{20})^2 \tag{21}$$

Equation (21) can be integrated to give

$$\lambda \alpha K x' = \frac{1}{6}\, n_{20}^{\ 2}\ (3 - 2n_{20}) - \alpha \left[\frac{n_{20}}{1 - n_{20}} + \log\ (1 - n_{20})\right] \tag{22}$$

The left-hand side of equation (22) is $\alpha\, K\, \frac{x}{D_o}$. Hence, provided K can be found, and presuming α is given, equation (22) is an expression for the shape of the wedge. The value of K must be found by observing n_{20} for one or more experiments and calculating K from the equation (22) for these experiments.

So far we have no information concerning λ , the length of the wedge in fact this length is not yet defined. In the experimental set-up the channel leads to a reservoir which represents the ocean and the transition section from channel to reservoir determines the location of the wedge. We therefore define the length of the wedge to be the distance from the wedge tip to the reservoir. Experimentally the slope of the interface is found to be very large near the tip, to decrease with increasing x and the to increase again as the reservoir is approached. It is also observed tha $\frac{n_1^{\,3}}{\alpha}$, which equals $\frac{gD_1^{\,3}\beta}{q^2}$, approaches unity as one approaches the reservoir.

Equation (21) shows that the slope of the interface is infinite at the tip and again when $(1 - n_{20})^3 = \alpha$ or when

$$\frac{n_{10}^3}{\alpha} = 1. \tag{23}$$[3]

We cannot, of course, expect our equations to hold near these regions, but, in order to obtain an estimate of the length of the wedge we formally associate the point at which $\frac{n_{10}^3}{\alpha} = 1$ with the section of transition to the reservoir. Hence the depth of the wedge at the reservoir, say n_{20}^*, is given by

$$n_{20}\ (1) = n_{20}^* = 1 - \sqrt[3]{\alpha} \tag{24}$$

Substituting $n_{20} = n_{20}^*$ and $x' = 1$ into equation (22), we obtain a formula for λ in terms of n_{20}^*:

$$\lambda K \alpha = \frac{1}{6}\ (n_{20}^*)^2\ (3 - 2n_{20}^*) - \alpha \left[\frac{n_{20}^*}{1 - n_{20}^*} + \log\ (1 - n_{20}^*)\right] \tag{25}$$

Equation (25) together with equation (24) provides an alternate method of finding K by measuring λ for one or more experiments.

A convenient method of comparison between theory and experiment is afforded by plotting $\frac{n_{20}}{n_{20}^*}$ against x'. The theoretical curves are found to be practically independent of α for all permissible values of α, i.e., for α ranging between zero and one. They are approximately given by

$$x' = \left(\frac{n_{20}}{n_{20}^*}\right)^2 \left(3 - 2\,\frac{n_{20}}{n_{20}^*}\right) \tag{26}$$

If one wishes to apply the theory to actual estuaries where the values of U_0 and D_0 differ drastically from those in the experimental set-up, it is necessary to first obtain a value of K from observation. Inasmuch as K is a kind of measure of the turbulent shear at the interface its value will be different for various estuaries and, of course, will be different from that in the experimental set-up. It may be hoped, however,

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[3] For a separate discussion of this criterion at transitions see Stommel and Farmer, 1952.

that the value of K will remain reasonably constant for a given experimental set-up or estuary although U_o, D_o, β may vary.

EXPERIMENTATION

Figure 2 shows in outline form the experimental apparatus used in the salt wedge studies. The flume has plate glass side walls and a level bottom. The reservoir and flume are connected together through a six inch long elliptically shaped transition. The total depth of water in the reservoir and flume is controlled by the weir on the overflow side of the reservoir. The concrete reservoir, representing the ocean source in the apparatus, is constantly supplied with sea water. The salinity of the sea water remains at a satisfactorily constant 32 °/oo and the density is determined from its temperature and salinity. Fresh water is metered with a Rotometer before discharging into the upstream end of the flume. In order to vary the density difference of the two waters in the flume, sea water may be drawn from the concrete reservoir source, separately metered, and then mixed with the fresh water before discharging into the flume. The specific gravity of the fresh and salt water mixtures was taken by hydrometer, and these values were used as density. The wedge profiles were measured with point gauges, the level of the interface being taken in the troughs of the internal waves which were present at the interface.

A total of 45 different salt wedges was established in the flume, β being held a constant 0.025 for 27 runs, and in the remaining 18 it was varied from 0.0170 to 0.000684. A full account of the experimental data is given by Farmer, 1951.

DISCUSSION OF RESULTS

Figure 3 shows the approximate theoretical curve for the shape of the wedge as given by equation (26), as well as one set each of experimental data from the flume and observational results, Rhodes, 1950, from the Mississippi River South West Pass.

Figure 4 is a typical curve of dimensionless length $\lambda = \frac{L}{D_o}$ versus discharge q for a given density difference β and a fixed K and D_o, the values chosen being of the order of magnitude pertinent to the Mississippi River. The value K = .001 was determined for the salt wedge in South West Pass, D_o = 48 ft., q = 66 ft^2/sec., L = 10 miles and β = .0196. D_o was taken as the total depth at the tip of the wedge.

Figure 5 shows the variation of λ with β for a fixed value of $\frac{U_o^2}{gD_o}$ and K.

Comparison of theory with experimental results based on K = 0.006 gives fairly good agreement for the interface shape. Theoretical predictions of the length of the wedges agree with experiment to within approximately 15%.

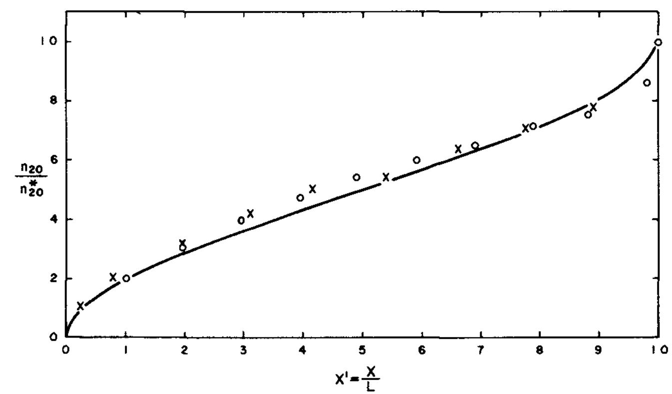

Fig. 3. Non-dimensional shape of salt wedge; the solid line is given by Equation (26), O's refer to Mississippi River South West Pass and x's refer to experimental run A-8.

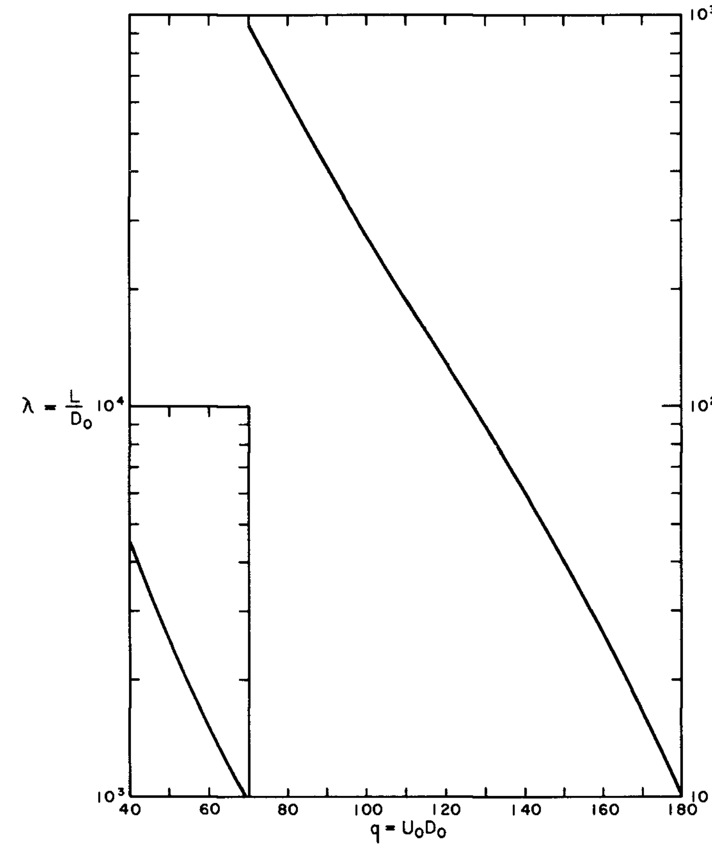

Fig. 4. Non-dimensional length L/D_o versus discharge per unit width q. D_o = 48 ft., β = .0196, and K = .001.

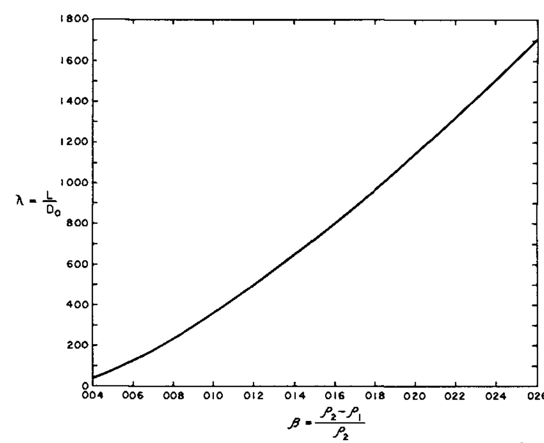

Fig. 5. Non-dimensional length L/D_o

Observational data were limited to a few isolated curves for the Mississippi River. In this connection it must be pointed out that in the range of α encountered in the Mississippi the length, as predicted by theory, is very sensitive to slight changes in river depth D_0. Since the bottom is not horizontal, the question of what value of D_0 to choose in attempting to apply the theory becomes a difficult one and will require further study.

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CHAPTER 6

CIRCULATION IN ESTUARIES[1]

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In discussing the circulation in estuaries, an apparent paradox must always be kept in mind. In the first place the problems of estuarine circulation are unique, being different from the problems of the open sea, and from the problems of river hydrology. Estuarine circulation and related problems consequently constitute a valid field for investigation. In contrast to this viewpoint is the tremendous range of conditions found in various estuaries - no two are alike. This inherent variability of estuaries discourages generalizations about the circulation.

The paradox thus lies in the recognized fact that estuaries are similar enough to constitute an integrated field of investigation, but at the same time estuaries are so different in details that generalizations are dangerous. The state of our knowledge is still so imperfect that one is never certain whether a general principle or a unique detail is being studied. Therefore it is difficult and sometimes impossible to apply methods which have been found useful in one estuary as the basis for predictions in others.

When asked to present a paper "summarizing the present state of the art and science" it seemed particularly important to attempt to classify our knowledge so as to indicate, on the one hand, principles which are generally applicable and tend to make estuaries similar, and, on the other hand, to contrast the circulation in various estuaries so as to indicate the wide variety of differences which may be encountered. Since our knowledge is meagre, this can be done only in a qualitative way.

Let us first consider what is meant by an estuary. Many definitions have been given, but one suggested by Pritchard (1952a) seems particularly applicable to studies of the circulation. "An estuary is a semi-enclosed coastal body of water having a free connection with the open sea and containing a measurable quantity of sea salt." The most common type of estuary is one in which the enclosed water is fresher than that of the open sea, since the increments of fresh water from precipitation and runoff exceed the loss of water by evaporation. Such locations have an estuarine type of circulation. In some enclosed bodies of water evaporation exceeds precipitation and runoff, and the water becomes more saline than that of the open sea. These may logically be included among estuaries, since the same processes must determine their circulation, even though everything is working in reverse. Redfield (1951) has called the

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[1] Contribution No. 642 from the Woods Hole Oceanographic Institution.

circulation in this type of embayment <u>anti-estuarine</u>. Little is known about anti-estuarine circulation (Collier and Hedgpeth, 1950) and the remainder of this paper will deal only with the positive type.

Among the positive type of estuaries in which the water within the estuary is fresher than pure sea water, three major types can be enumerated. The drowned river valley is the common type of estuary of the Atlantic and Gulf coasts. For example, the Delaware, Hudson and Chesapeake Bay systems are drowned river valleys. These estuaries are generally shallow relative to their width and have a tidal range which is substantial in proportion to the total depth. The sides of the estuary slope gradually in a shallow V shape and the surrounding coast is relatively low.

The Scandinavian coasts and the coasts of northern United States and of Canada are frequently indented by fjord type estuaries which present many contrasts. The depth is great relative to the width of the estuary, being not uncommonly equal to the width. There is frequently a sill at the entrance which isolates the deeper water within the estuary from coastal water at equal depths. The estuary is steep sided giving essentially a U shaped trench in contrast to the shallow V shape of the drowned river valley. The total depth of water is generally great relative to the range of tides.

Estuaries may also be formed by the development of an offshore barrier beach, having a narrow entrance to an enclosed bay. Such estuaries are found along the south shore of Long Island, and along our Southern Atlantic coast. The largest estuary of this sort on the Atlantic coast is the Pamlico Sound system in North Carolina. In general these estuaries are even more shallow relative to the width than are the drowned river valleys. However, the entrance to the sea is generally so narrow that the tidal wave is damped and consequently the range of tides in such estuaries is not as great in proportion to the total depth as it is in the drowned river valley. The wave may indeed be sufficiently damped so that within the embayment no predictions of tidal variations are possible.

In each of the above categories, estuaries of various sizes and shapes will be found. It is this variation which discourages generalizations about the circulation in estuaries. All of the positive type estuaries, however, have the following properties in common:

1. Fresh water additions exceed evaporation. The principle of the continuity of volume requires that the circulation remove the excess fresh water at a rate which, over a period of time, equals the rate of addition.

2. There is a gradual increase of salinity from the source of fresh water to the sea. As the salt content increases a greater volume of mixed water must move out in order to transport a unit

volume of fresh water seaward. The principle of continuity of salt requires that the circulation provide enough sea salt to maintain the salinity distribution and to balance the salt removed.

3. The volume of water within the estuary varies periodically with the stage of the tide.

4. The currents within the estuary vary periodically with the stage of the tide. These currents are directly related to the tidal change in volume, though in some locations the relationship may be obscure and unpredictable.

5. Net movements of water, or non-tidal drifts, result from inequalities in the direction, velocity or duration of flood and ebb tidal currents.

The circulation of water is the net result of two processes operating simultaneously and in three dimensions. These are advection and turbulent mixing. Advection is the mass transport of water, whereas turbulence involves random motions of water and the resultant intermixture of adjacent waters. Theoretically, at least, either one of these processes alone could produce a circulation in an estuary which would satisfy the requirements of the continuity of volume and the continuity of salt. Actually the two processes are combined in various ways to produce the diversity of circulation patterns which is so apparent and troublesome to students of estuaries.

To show the range of conditions which may be found in different estuaries the salinity variation with distance for four examples is plotted in Figure 1. In all cases the salinity increases along the length of the estuary, but the shape of the distribution curve is quite different. The ratio between the volumes of water entering the two ends of the estuary during each tidal cycle is informative in connection with this figure. In the Bay of Fundy, where the salinity increases so rapidly, the tidal flow exceeds mean river flow by a factor of about 900. This factor is about 300 for the Raritan system, 150 for the Delaware and the two volumes are nearly equal in the Connecticut. As this ratio decreases the change of salinity with distance becomes more gradual in the upper end of the estuary. It seems probable that the ratio of these two volumes will have a direct effect on the pattern of salinity distribution in all estuaries, though the circulation can be determined by so many combinations of the turbulent and advective processes that no simple relationship can be expected.

Let us consider first the advective processes, neglecting for the moment the horizontal transport resulting from turbulent mixing. In order to remove the fresh water increment there must be a net transport seaward. As the water moves seaward there is continuous admixture of more saline water, and the gross seaward transport of water must increase

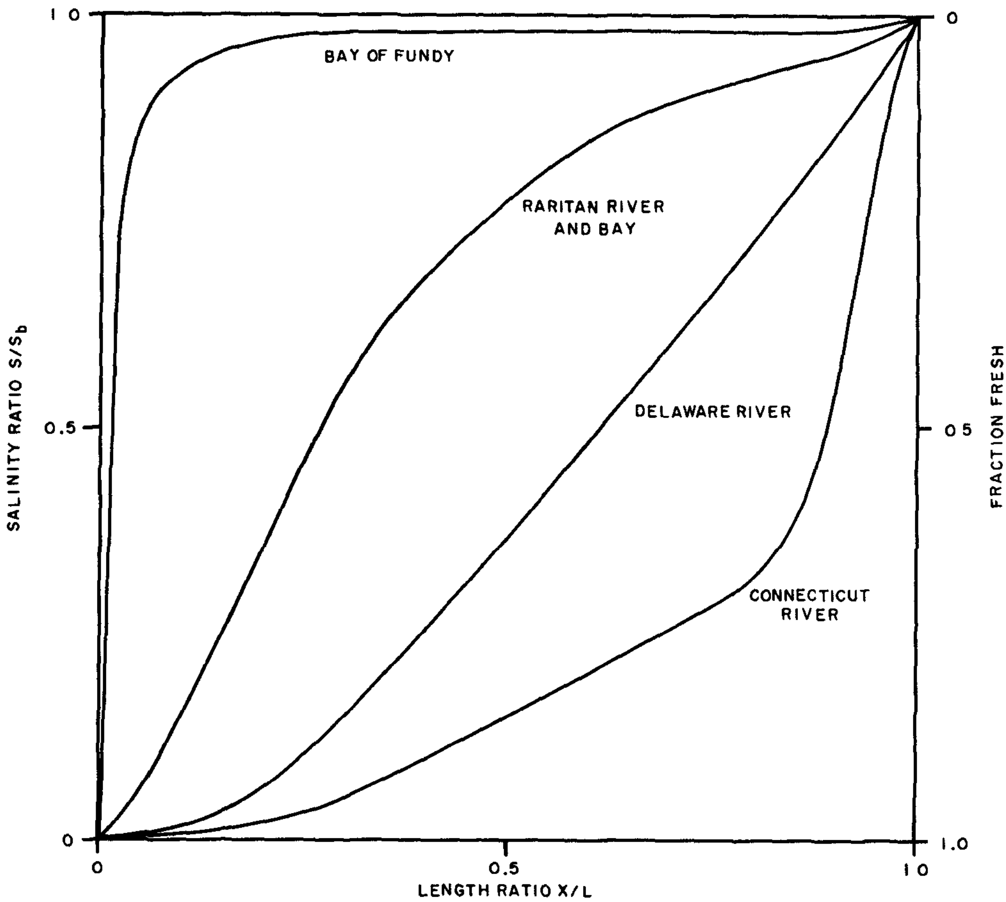

Fig. 1. The distribution of salinity along the length of four estuaries. Data for Delaware Bay from Mason and Pietsch (1940); for Connecticut River from Howard (1940).

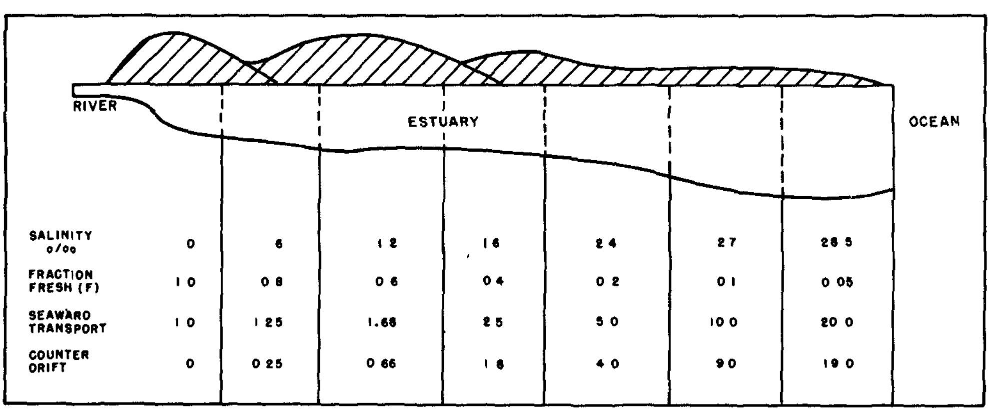

Fig. 2. The changes in mean salinity and in the volume of water which must be transported in order to maintain the steady state distribution in a hypothetical estuary.

directly in proportion to its salt content. At the same time a landward transport of salt must take place to satisfy the salt balance. For example, where the mixture contains half sea water and half fresh water two volumes must move seaward in order to carry one volume of fresh water seaward. A direct consequence of this simple relationship is that the total circulation in the estuary increases enormously in volume as the water moves from the river towards the sea. This is illustrated in a diagrammatic way in Figure 2 which shows the increase in seaward transport of mixed water and the compensating landward transport of salt, expressed in terms of the equivalent volume of pure sea water.

These seaward and landward advective transports may be separated vertically to produce two or three layered flow systems, or they may be horizontally separated. In still other cases there may not be a division into two distinct advective drifts, so that the horizontal transport in one direction must be due to rather large, horizontal mixing processes.

The relative importance of the advective and turbulent mixing processes has been assessed by Pritchard (1952) in the estuary of the James River. In this location the two most important processes were horizontal advection and vertical random mixing. Of secondary, but significant, importance was the vertical advection and lateral random mixing related to the change in width of the estuary with depth. Horizontal random mixing was negligible. The horizontal advection in this case was clearly separated into two layers, with a net seaward transport in the surface and a net landward transport in the deeper layer. Pritchard's measured tidal currents as a function of depth, and the resultant non-tidal drifts are shown in Figure 3.

As mentioned above there may be no separation of the system into two clear advective drifts such as those observed in the James River. As an example of such a case Table I presents the transport of water by the ebb and flood tidal currents in the mouth of the Raritan River. At this location both the surface and deeper water showed seaward non-tidal drifts, and no landward countercurrent was found. The seaward drift calculated from the river flow and the mean fraction of fresh water in the section indicates a volume of gross seaward transport intermediate in magnitude between the surface and deeper transports. In such a case turbulent mixing must be the mechanism which provides the salt required to maintain the salt balance.

The admixture of salt water into the system as the water moves seaward is clearly an important process. Two sets of evidence are available to indicate that the rate of admixture is correlated with the velocity of the tidal currents. These are given in Figure 4. The left-hand figure is from Tully (1949) who was describing the circulation in Alberni Inlet, a fjord type estuary where the deepest layer was essentially completely isolated from a two layered surface system. This shows that the total admixture of salt, whether it resulted from turbulence or from vertical advection, into the surface layers is directly

Table I

Tidal Excursions and Non-Tidal Drifts in Mouth of Raritan River
Negative values indicate seaward motion
(feet per tidal cycle)

Surface:-	Excursion on Flood	$+12.1 \times 10^3$
	Excursion on Ebb	-26.8×10^3
	Non-tidal drift	-14.7×10^3
Bottom:-	Excursion on Flood	$+16.5 \times 10^3$
	Excursion on Ebb	-22.1×10^3
	Non-tidal drift	-5.6×10^3

Average Non-Tidal Drift Calculated from Salinity:

$$NTD = \frac{-R}{FA} = \frac{-23 \times 10^6}{0.12 \times 27 \times 10^3} = -7.1 \times 10^3$$

Where R = River flow (cubic feet per tidal cycle),
F = Average fraction of fresh water,
A = Cross sectional area.

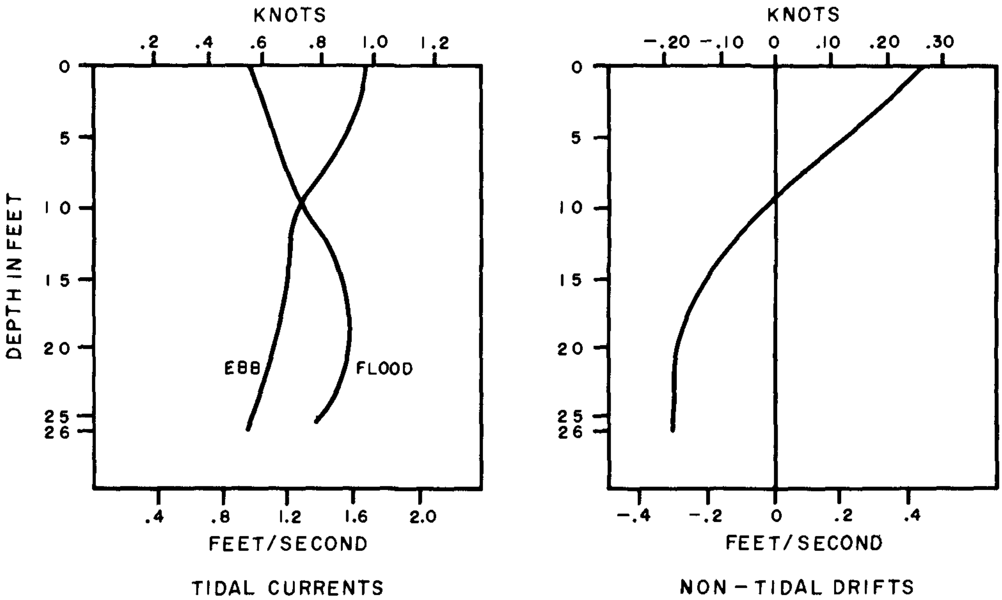

Fig. 3. Tidal currents and non-tidal drifts observed by Pritchard (1952) at various depths in the James River.

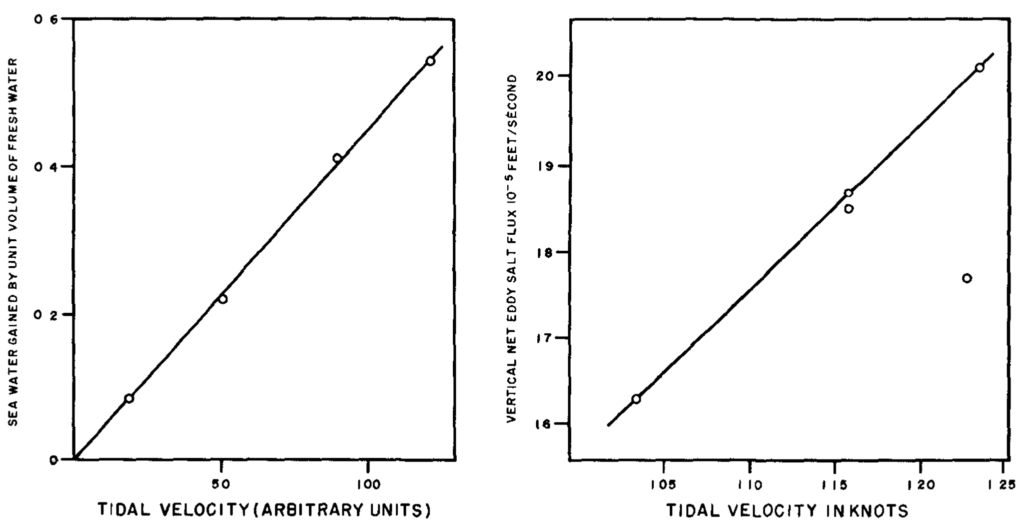

Fig. 4. The rate of admixture of salt into the freshened surface water in Alberni Inlet (at left, Tully, 1949) and in the James River (at right, Pritchard, 1952).

related to tidal velocity. The right-hand figure is from Pritchard's (1952) analysis of the James River, and shows that the random eddy flux, excluding the admixture of salt due to vertical advection, is a direct function of tidal velocity. These relationships between salt entrainment and tidal flow emphasize the importance of tidal currents in effecting the mixing between salt and fresh water.

As the estuary widens the seaward drift of freshened water may be horizontally separated from the landward counter drift. The distribution of salinity in Raritan Bay just off the mouth of the Raritan River illustrates this condition (see Figure 5). A similar effect is shown in Figure 6 for Mount Hope Bay. This effect is the result of the rotation of the earth, and is generally found in the wider parts of estuaries. Even in the narrow parts the effect may be present, but the change from one side to the other is so small that it frequently cannot be detected. In localities where additional river flow enters a bay along one side the distortion of the salinity distribution may be augmented, as it seems to be in the Chesapeake Bay (Pritchard, 1952) and in the Bay of Fundy (Ketchum and Keen, in press), or it may be decreased or completely obliterated, as seems to be the case in the lower Delaware Bay (Mason and Pietsch, 1940).

So far, we have been talking of estuaries in which mixing between salt and fresh water occurs throughout the length of the estuary. We have mentioned cases in which the landward counter drift and the seaward drift are separated vertically or horizontally, and in which only one advective drift can be identified. There are, of course, other possibilities. Tully (1949) clearly describes in Alberni Inlet, a case in which three layers are identifiable, the deepest being essentially isolated from the upper two. Farmer and Morgan (1953) will discuss in the next paper the salt wedge, which is a special case of two layer flow in which vertical mixing is very slight. There is evidence that in some estuaries intense mixing takes place in a certain region and that little additional mixing occurs as the freshened water flows seaward. Hachey (1934) has studied localized mixing phenomena in a small model and applied the results to the conditions in the upper end of Passamaquoddy Bay. There he finds a surface fresher layer mixing with a deep saline layer to produce an intermediate layer of intermediate properties.

In the Strait of Juan de Fuca - Strait of Georgia system in the Pacific Northwest, the San Juan islands, combined with rapid tidal flows, appear to provide a similar localized mixing. The resultant water masses and their distribution are shown in Figure 7. In this case water from mid depths of the Pacific Ocean enters through the Strait of Juan de Fuca unchanged until it is mixed with surface water from the Strait of Georgia in the San Juan channels. This mixed water then becomes the deep water of the Strait of Georgia and the surface water of the Strait of Juan de Fuca (Redfield, 1950). The volume of water flowing into the system from the Pacific Ocean in the countercurrent is about 15 times greater than the volume of fresh water introduced by the rivers. This

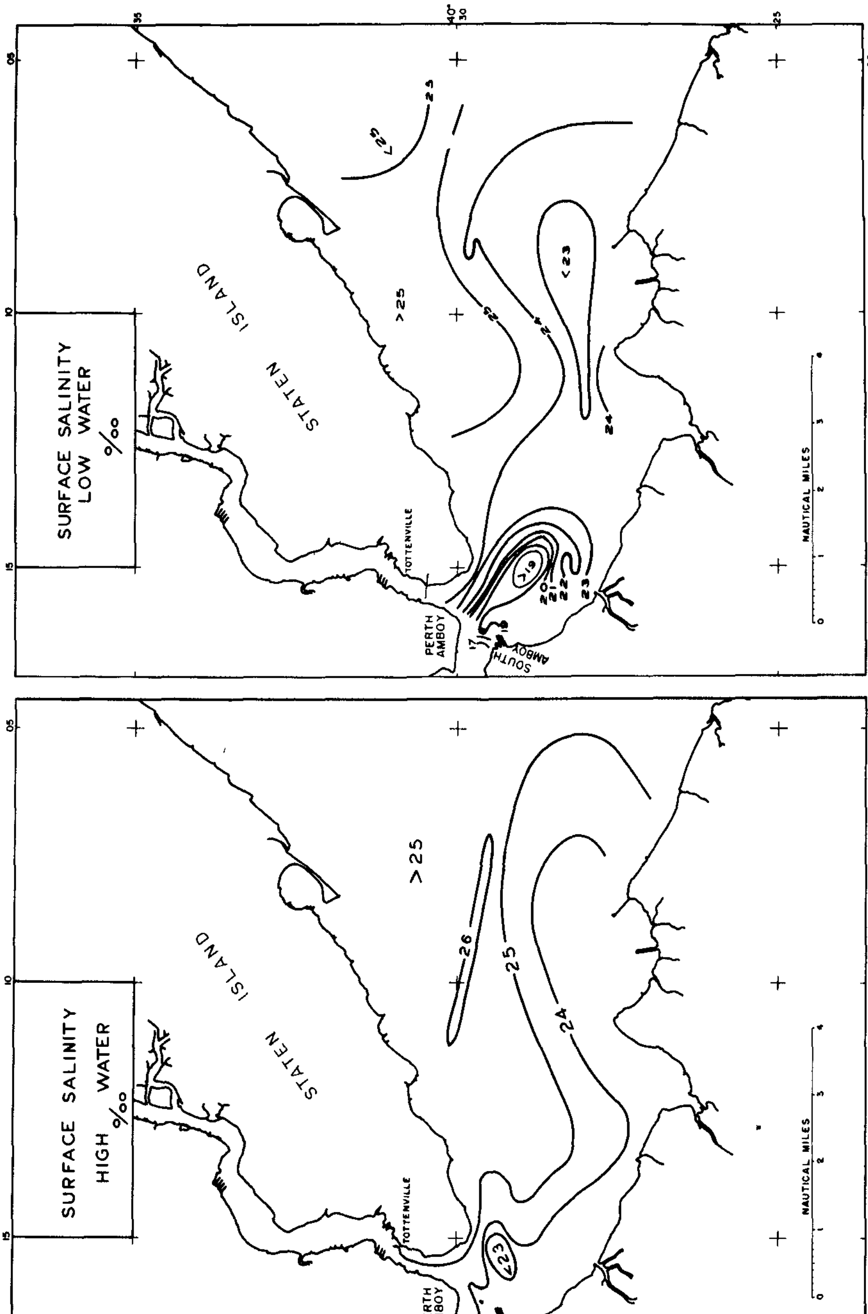

Fig. 5. Surface distribution of salinity at high and at low tide in Raritan Bay.

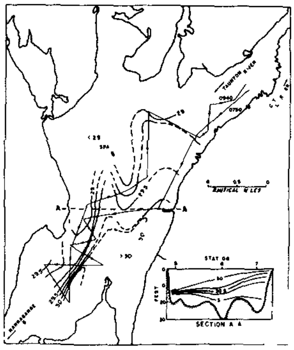

Fig. 6.

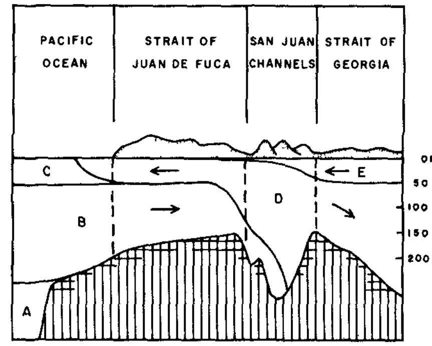

Fig. 7.

Fig. 6. Surface distribution of salinity in Mt. Hope Bay, and a cross section showing the slope of the isohaline surfaces.

Fig. 7. Diagrammatic chart of the distribution of various water masses in a section running from the Pacific Ocean through the Strait of Juan de Fuca to Georgia Strait (after Redfield, 1950).

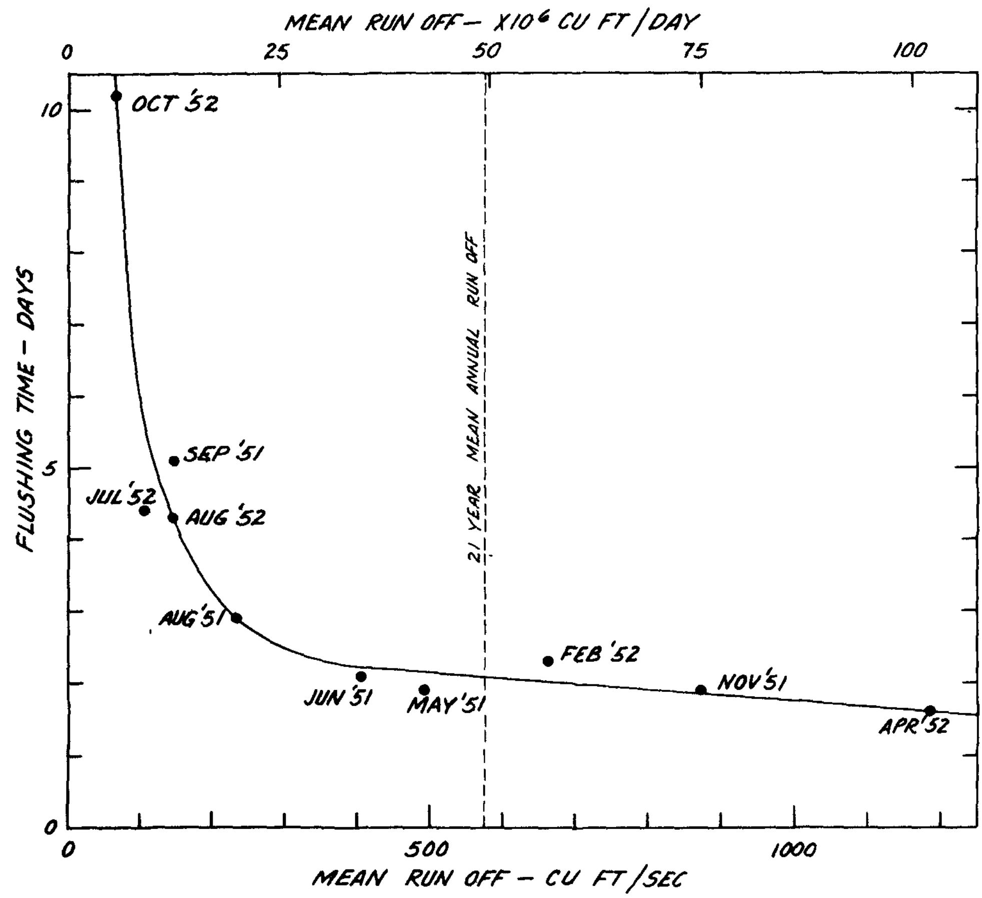

Fig. 8. The relationship between flushing time and river flow in Boston Inner Harbor (after Bumpus, 1952).

is a case of three layered flow in which all three layers have an active part in the circulation.

The importance of the tides as the effective force producing mixing has been emphasized. Empirical predictions of the distribution of fresh and salt water which are based upon the concept of tidal mixing have been shown to be applicable to a variety of estuaries (Arons and Stommel, 1951; Ketchum, 1951, 1951a; Ketchum and Keen, in press). What effect does variation in river flow have upon the circulation? Obviously increased river flow requires an increased transport of fresh water through each part of the estuary. This could be accomplished by increasing the fraction of fresh water in the mixed water which moves seaward, which would result in the movement downstream of the whole pattern of salinity distribution. It could also be accomplished by a more rapid circulation of the water with essentially no change in the salinity distribution. Once again both phenomena exert their influence. In the upper reaches of the estuary the rate of the circulation appears to increase as the river flow increases. Bumpus (1952) has determined the accumulation of fresh water in Boston Harbor and shows that the flushing time decreases from about ten days during low flow to about two days at the mean annual rate of flow. Further increases in runoff had only a slight additional effect on the rate of the circulation. His results are given in Figure 8.

As the water moves seaward, however, the fresh water is a smaller and smaller proportion of the mixture, and the effect of changes in fresh water volume is similarly decreased. In the offing of the Hudson River, for example, where the volume of sea water was about 100 times greater than the volume of fresh water in the mixture, variations in river flow had no measurable effect on the flushing time (Ketchum, Redfield and Ayers, 1951). It seems likely that river flow will, in all estuaries, have a direct effect on the rate of the circulation in the upper reaches, and a progressively smaller effect in the seaward region where the mixture moving seaward contains so much more sea water than fresh.

This paper has attempted to indicate the fundamental similarities in estuaries, and to point out the wide variety of detail in circulation which may be expected. It seems probable that the examples cited are isolated pieces of a continuous range of conditions extending from one extreme to the other. This is almost certainly true of the vertically stratified estuaries. In temperate latitudes, at least, the same geographical estuary can change from the vertically homogeneous type to the vertically stratified type at different seasons of the year as a result of changing runoff and temperature. As yet no unified theory is available which is comprehensive enough to permit application to a wide variety of conditions, though some success in interpreting quasi semi-state conditions in particular estuaries has been achieved. The recently expanded interest in estuarine circulation promises great advances in the art and science of such studies in the future.

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CHAPTER 7

NOTES ON THE GENERATION AND GROWTH OF OCEAN WAVES UNDER WIND ACTION

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The basic problem of forecasting wind-generated waves is the development of equations which express the energy budget between wind and waves, and the derivation of physical laws governing the growth of the component wave trains. The waves can grow only in the case where the supply of energy by wind exceeds the loss of energy by friction and turbulence. Thus any attempt to calculate the growth of ocean waves under wind action requires a knowledge of the energy supply and the energy dissipation in every phase of wave development.

Owing to the complexity of the actual wave motion it is exceedingly difficult, if not impossible at present, to follow this mechanism in detail. We are forced to concern ourselves with mean effects, and until now every approach to the problem has been based on certain empirical relationships and hypotheses. The purpose of the present paper is to call attention to a few recent advances and to outline briefly the hypotheses involved in the semi-theoretical treatment of the problem of generation and growth of ocean waves under the action of wind.

NOTES ON ENERGY TRANSFER FROM WIND TO WAVES

The first attempt to explain the generation of waves under wind action quantitatively has been made by Jeffreys (1925, 1926) in his "theory of sheltering," taking into account both the turbulent character of the wind and the viscosity of the water. By neglecting tangential action of the wind he considered only the power transmitted by the vertical component of the surface stress, τ_n , as the most important constituent in doing a net amount of work. The distribution of τ_n over a simple wave profile,

$$\eta = a \sin \kappa (x - \sigma t), \tag{1}$$

is according to Jeffreys

$$\tau_n = s\rho'(v - \sigma)^2 \partial\eta/\partial x \tag{2}$$

In these formulae a = wave amplitude, $\lambda = 2\pi/\kappa$ = wave length, σ = phase velocity of wave propagation, v = wind velocity, ρ' = density of the air. The unknown factor of proportionality, s, was called the "sheltering coefficient." With these assumptions concerning the wind pressure against different portions of the wave profile, the average energy supply from wind to waves per unit surface area is given by

$$A_n = \frac{1}{2} s\rho'(v - \sigma)^2 a^2 \kappa^2 \sigma \tag{3}$$

Difficulties arise in the determination of the factor s and of the "effective" wind velocity in the difference $v - \sigma$ at the "sea surface." Here the question of the reference level at the wavy sea surface has to be

*The results of this research have been sponsored by the Office of Naval Research, Washington, D.C.

answered. It would be possible to define an average wind velocity $\bar{v}$ for a given "anemometer height" and to determine empirically a factor $\bar{s}$ with reference to that particular height. By such a procedure we approach the problem on a semi-theoretical basis. But in any case it should be kept in mind that the determination of $\bar{s}$ (or s) as defined by (2), from simultaneous wind and wave measurements, for example, requires the elimination of effects of possible tangential stresses which may have been included in experimentally measured total stresses.

This has to be taken into consideration when the wavelets grow beyond their initial stage, that is when the wavelets attain their maximum steepness, and the unstable crests indicate small breaking processes. There is some evidence that such instability occurs at a wind velocity as small as $v = 125$ cm/sec and with wavelets of $\lambda = 11\text{-}12$ cm (Neumann, 1949). The sea surface in this state acts as an hydrodynamical "rough" surface, and tangential stresses are no longer to be considered as purely viscosity stress

The problem becomes even more complicated when considering the further development of waves to the stage of "turbulent sea." The first approach to this problem was made by Sverdrup and Munk (1947). They tried to take into account also the additional power transmitted by the horizontal wind stress component, but they applied the total surface stress instead of the tangential component and regarded it as constant over different portions of the wave profile.

The total wind force, τ, at any point of the wave profile may be split up into a component τ_n, acting normally to the rough wavy interface, and a component τ_t, acting in a tangential direction. Or, taking these forces with the negative sign, the total resistance of the rough surface may be considered as the vector sum of a "pressure resistance" and a "friction resistance." This has been done for practical reasons when considering the rough form of the wavy air-sea interface and approximating it by a "smoothed wave profile." The resistance of the rough superposition then will contribute to an effective frictional resistance, whereas the pressure resistance is determined by the pressure differences between the windward and the leeward slope of the general wave profile. These considerations form the basis of an approach to the problem recently made by the author (Neumann, 1950, 1952).

We may assume, that the distribution of τ_n and τ_t over different portions of the wave profile is given by expressions of the form

$$\begin{aligned} \tau_n &= \bar{\tau}_n + \tau_n' \cos \kappa (x - \sigma t) \\ \tau_t &= \bar{\tau}_t + \tau_t' \sin \kappa (x - \sigma t) \end{aligned} \qquad (4)$$

Here $\bar{\tau}_n$ and $\bar{\tau}_t$ represent constant stress components over the wave profil and $\bar{\tau}_t$ delivers some power to horizontal surface drift. The constant term $\bar{\tau}_n$ may be disregarded. τ_n' and τ_t' are the amplitudes of the effective normal and tangential components of the wind stress. Both of them will depend in a complicated way on the wind velocity v, or $v - \sigma$, and the surfac conditions (roughness). At present it seems exceedingly difficult to deter mine these effective values separately with the necessary degree of accurac

Assuming a wind distribution over the wave profile as given by the form

$$v = \overline{v}\,[1 + a\kappa \sin\kappa(x - \sigma t)\,], \tag{5}$$

we have, by putting $\tau_t = \rho' f_t (v - \partial\xi/\partial t)^2$, the effective tangential component

$$\tau_t = \rho' f_t\, \overline{v}^2 \left\{1 + 2a\kappa\left(1 - \frac{\sigma}{\overline{v}}\right) \sin\kappa(x - \sigma t) + \ldots\right\} \tag{6}$$

approximately, where $\xi = a\cos\kappa(x - \sigma t)$, the horizontal particle displacement, and f_t a dimensionless factor depending upon the roughness conditions of the wavy sea surface.

Similarly, retaining the plausible assumption of Jeffreys (formula (2)) and considering equation (1), the normal component may be written

$$\tau_n = \tau_n' \cos\kappa(x - \sigma t) = \rho' \overline{s}(\overline{v} - \sigma)^2\, a\kappa \cos\kappa(x - \sigma t) \tag{7}$$

The dimensionless coefficient $\overline{s}$ is now related to the average wind velocity $\overline{v}$ at "anemometer height."

With these assumptions the total amount of work done by the wind on the wave is given by

$$(A)_\lambda = (A_n + A_t)\lambda = -\int_0^\lambda \tau_n\,(\partial\eta/\partial t)\,dx + \int_0^\lambda \tau_t(\partial\xi/\partial t)\,dx$$

or, after carrying out the integration with assumptions (6) and (7),

$$A = \frac{1}{2}\left(a\kappa\tau_n' + a\kappa\tau_t'\right)\sigma \tag{8}$$

Both of the components do a net amount of work at the particle velocity, even in the case of the Airy theory of waves, as long as the wind is faster than the wave.*

Formula (8) is based on very special assumptions. In the case of actual ocean waves the distribution of the stress components may be much more complicated. But, even in the case where our assumptions hold, the application of this formula requires a knowledge of τ_n and τ_t over the wave profile, or at least a knowledge of τ_n' and τ_t' for rough ocean waves of different steepness, wave form, etc. At this point difficulties arise in our problem. It has not been possible hitherto to determine the wind stress components separately for actual wave motion at the sea surface.

To avoid the difficulties involved in the separate determination of τ_n and τ_t over the actual wave profile, the author (Neumann, 1949, 1950, 1952) used the total effective stress, τ_{eff}, in place of (4). This quantity is

*In the case of Stokes waves the average surface mass transport velocity has to be taken into account.

given as an integral effect of the wind force exerted at the sea surface, and related to the wind velocity $\overline{v}$ at "anemometer height," that is, about 10 m above the sea surface. The empirical relationship, as derived from special oceanographic observations (Neumann, 1948) is of the form

$$\tau_{eff} = \rho' C v^2 , \tag{9}$$

where the dimensionless quantity C has the meaning of a total resistance coefficient over the rough wavy sea surface. It is not a constant, as has been assumed before, but depends upon the roughness pattern of the sea surface, and therefore on the stage of wave development itself. C has been determined empirically

$$C = \left(\frac{1 \ {}^{m}/_{sec}}{v \ {}^{m}/_{sec}}\right)^{1/2} \cdot 10^{-2} \tag{10}$$

Now the hydrodynamical characteristics of the rough sea surface are implied in the dimensionless coefficient C. Because in the fully developed state the sea surface pattern is only a function of the wind velocity, C can also be expressed in terms of the wave characteristics, say of wave age $\beta = \sigma/v$.

If we consider the total effective pressure resistance over the entire wave profile, that is

$$\lambda(\tau_n)_{eff} = \int_0^{\lambda} \tau_n d\eta = \bar{s}\rho'(\bar{v}-\sigma)^2 \int_0^{\lambda} \left({}^{\partial\eta}/_{\partial x}\right)^2 dx$$

it follows that per unit surface area

$$(\tau_n)_{eff} = \frac{1}{2}\bar{s}\rho'(\bar{v}-\sigma)^2 a^2\kappa^2 = \frac{1}{2} a\kappa\tau_n' \tag{11}$$

Similarly we get for the total effective horizontal shearing stress per unit area

$$(\tau_t)_{eff} = \frac{1}{2} a\kappa\tau_t' \tag{12}$$

The effect of both $(\tau_n)_{eff}$ and $(\tau_t)_{eff}$ is in the same direction as long as the wave velocity $\sigma < v$. In other words, the wave generating tractions whether normal or tangential will tend to increase the wave energy to the point where the dissipation balances the work done by surface forces. Using the total effective stress we may write

$$A = \tau_{eff}\,\sigma \tag{13}$$

It is seen, from (6) and (7), that τ_{eff} can be written in the form

$$\tau_{eff} = \rho' C\, v^2 ,$$

where

$$C = f(\delta)(1-\beta)^2 + \gamma(\delta,\beta) \tag{14}$$

in the case of rough ocean waves is a complicated function of the surface conditions (wave age $\beta = \sigma/v$, wave steepness $\delta = 2a/\lambda$, roughness and form of the wave profile). The dimensionless parameter C can be determined only empirically.

NOTES ON ENERGY DISSIPATION WITH TURBULENT WAVE MOTION

This problem is one of the most difficult in Oceanography. In their paper on wave forecasting, Sverdrup and Munk (1947) neglected energy-dissipation in the energy balance.

In our case a probable relationship between wind energy, incidence of breakers, turbulence, and drift currents may be anticipated. It seems reasonable to assume that besides actual dissipation part of the wave energy released in breakers contributes to the energy of a semi-continuous forward motion of the upper water layers. Nevertheless this energy is lost by the wave motion and must be continuously replaced by energy from the wind field, if the average wave motion is to be maintained. This total "dissipated" energy has to be taken into account in the energy budget.

It has been proposed to allow for such turbulent processes by the introduction of eddy viscosity coefficients M in place of the molecular viscosity μ. A first attempt to determine such eddy viscosity-coefficients for wave motion has been made recently in a report on wind-generated ocean waves (Neumann, 1952). Here again it was necessary to abandon any attempt to follow the details of the phenomenon and to concern ourselves only with mean effects.

With the assumption that the energy dissipation D in turbulent wave motion is proportional to the square of the wave steepness δ, and replacing the molecular viscosity μ by M, we have according to Lamb (1932)

$$D = 2M\kappa^3\sigma^2a^2 = 2M\pi^2g\delta^2 \tag{15}$$

where M is a function of β, and in the fully developed state a function of the wind velocity v. In a fully arisen sea, where the longest wave present in the spectrum has a wave age $\beta = 1.37$ approximately, A equals D. From this it follows that

$$M = 0.1825\ 10^{-4}\ v^{5/2}\ (\mathrm{cm}^{-1}\mathrm{g\ sec}^{-1}) \tag{16}$$

It is remarkable that the eddy viscosity coefficients according to (16) agree fairly well with the values derived for wind-driven currents in the surface layer of the ocean.

SOME RESULTS ON THE GENERATION OF WAVELETS AND THE GROWTH OF TURBULENT SEA

If the wind starts to blow over an undisturbed water surface, primary wavelets are generated, originally beginning as minute disturbances which may always be present. The first attempt to explain the generation of initial waves was made by Jeffreys, but the question of the critical speed of the weakest wind which can raise any wave motion remained unanswered. Retaining Jeffreys' assumption, that in the stage of initial wave formatio the energy dissipation is determined only by molecular viscosity μ as long as the wavelets are not breaking, but taking the unit-area wave generating power equal to the product of effective wind stress and wave speed, the criterion can be written in the form

$$\frac{\rho' s}{8\pi} \frac{(v-\sigma)^2 \sigma}{\mu(g+\epsilon\kappa^2)} \geq \frac{a}{\lambda} \tag{17}$$

where $\sigma^2 = g/\kappa + \epsilon\kappa$ and $\epsilon = T/(\rho - \rho')$ (T = surface tension, ρ = density of the water). The effective wind stress is expressed in the form given by (9), (10) and (14) where in the case of initial waves, γ may be neglected and $f(\delta) = s\pi\delta$ by empirical evidence, with $s = 0.095$.

The equation shows that any wind with a velocity of more than 23.3 cm (minimum wave velocity) would be able to generate wavelets, but the steepness of these wavelets is very small at wind velocities below 60-70 cm/sec At a given wind velocity > 23.3 cm/sec the left hand side of (17) has a maximum value for a certain σ (or λ), and in the fully arisen state the ratio a/λ becomes a maximum for this wave. The wave heights, H_m, and wave lengths, λ_m, of the steepest waves generated by a given wind velocity are given in the following table, according to (17):

v cm/sec	40	50	60	70	80	90	100	120	125
λ_m(cm)	3.0	3.6	4.35	5.2	6.2	7.3	8.65	11.5	12.2
H_m(cm)	0.0065	0.022	0.056	0.118	0.21	0.38	0.63	1.49	1.75

Recently Roll (1951) studied the process of initial wave formation at different wind velocities by means of wave photographs. The first distinc waves appeared at wind velocities of about 70 cm/sec, but even at lower wind speeds wandering surface corrugations were observed. In an earlier paper the author (1949) mentioned similar effects as "vibrations of the wa surface, which can be noticed only in optical reflections." These exceedingly flat disturbances, which pass quickly over the mirror-like surface at the gentlest breeze, can be explained by extreme low wavelets as given in the table for wind velocities below 60-70 cm/sec.

With increasing wind the wavelets grow in height as well as in wave length and steepness. At a wind velocity of v = 100 cm/sec the theoretica height is 0.63 cm and the wave length 8.65 cm, which agrees with Jeffreys observations. But if the wind attains a velocity of 125 cm/sec the steepness of the initial waves with λ = 12.2 and H = 1.75 cm approaches the value 1/7, that is the maximum steepness according to Mitchell. In their further development these waves become unstable and break. Under these co ditions the assumptions made in the derivation of (17) no longer hold, because "turbulence" has to be taken into account, and the total wind stre over the wave profile must be regarded as the sum of normal plus tangentia components. That is, γ has to be taken into account in (14), because the tangential stress components may no longer be considered as pure viscosity stresses.

The growth of waves at higher wind velocities differs essentially from the growth of the wavelets in the initial stage. The initial waves increase in steepness, when growing in height and length, but turbulent ocean waves decrease in steepness while growing in height and length, as indicated by an empirical relationship between the wave steepness and the wave age β. This relationship was first used by Sverdrup and Munk (1947) and has been very useful in the semi-theoretical treatment of the problem.

Special observations made by the author (Neumann, 1952) in the Caribbean Sea in 1951, and recent measurements of Roll (1951) in waters near the German coast indicate a very rapid increase of wave steepness close to the value $\delta = 1/7$ for very "young sea" ($\beta \leq 1/3$). Thus, the original assumptions of Sverdrup and Munk (1947) and of the author in 1950 have to be modified at least for the earliest stages of wave development. In a preceding paper the author (Neumann, 1952) applied this empirical law, $\delta = f(\beta)$, by assuming that the steepness of turbulent ocean waves is approximately constant with the value $\delta = 0.124$ for $\beta \leq 1/3$.

The outstanding theoretical problem is the development of a method for forecasting the complete wind-generated wave spectrum, because energy is distributed over a range of wave periods as soon as instability and breaking occurs. But at present we are still far away from a complete understanding of the dynamical nature of this mechanism, which can hardly be other than complicated. Even the "state of the sea" has not yet been satisfactorily described.

To serve practical purposes, Sverdrup and Munk (1947) in their pioneer work introduced a statistical term, the "significant wave" as a first approach. But wind driven ocean waves are not characteristically long and flat like the "significant waves" defined by these authors, especially in the fully arisen state. The most impressive features of a wind-generated sea are more or less steep, breaking waves, and, in this turbulent wave pattern, fluctuations of both wave height and period, if we denote as "period" the time interval between succeeding crests at a fixed location.

A second approach, by the author (Neumann, 1952) is an attempt to forecast the significant variations of the wave periods and heights in the composite pattern of wind-driven sea at a fixed position. The method is based on the fact that at a given wind velocity the length and steepness of waves which are generated by direct wind action do not develop beyond particular maximum values which are functions of the wind velocity. They break from time to time and are continuously regenerated by energy supplied by the wind. Part of the energy released in breakers has been taken into account as an energy loss for the wave motion, but another part may be transferred to underlying longer waves which are not directly generated by the wind. There is some evidence that breakers thus play an important role in the energy supply from wind to longer waves which may proceed faster than the wind (Neumann, 1952). Thus, the process of wave generation in turbulent sea is considered as a discontinuous process, and in the fully developed state the characteristic wave pattern is not described only by a single "significant wave," but by a spectrum of possible wave periods with dominating waves in certain "bands" of the spectrum.

Some of the theoretical results may be mentioned briefly. The minimum fetches and minimum durations of wind action needed for generating fully developed sea increase rapidly with increasing wind velocity, but they are much smaller than the corresponding values according to the theory of Sverdrup-Munk.

At 5 m/sec wind velocity the sea would be fully developed over a fetch of only 13.8 km with a minimum duration of 2.25 hours. The range of significant periods in the composite wave pattern would be between 1.8 and 4.3 seconds with an average wave height of 0.5 meters.

At 10 m/sec wind velocity the waves would be fully developed over a fetch of 110 km with a minimum duration of 8.6 hours. The range of periods is between 4 and 8.6 seconds and the average wave height 2.3 meters. These results could describe, for example, the conditions in the trade wind region with a fresh breeze.

But for the development of fully arisen storm sea, fetches of more than 1000 km are needed.

Consider for example a storm of 24 m/sec wind velocity (strong to whole gale). For the development of fully arisen sea in this case a minimum fetch of 1800 km, or nearly 1000 nautical miles, would be needed. At the end of this fetch the characteristic wave pattern would show significant periods ranging between 12 and 23 seconds with optimum "bands" in the spectrum at periods of about 14, 16, and 20 seconds. The average wave height is between 13 and 14 meters.

At a limited fetch of 600 km, that is about the maximum fetch in the Black Sea, the significant periods in the considered case would range between 8 and 13 seconds, with wave heights between 6 and 13 meters, in the average about 11 meters. These wave dimensions are approximately those of the maximum waves in the Black Sea.

With this approach, it seems possible to forecast not only the range of significant periods, but also the frequency distribution of time intervals between succeeding wave crests as they pass a fixed position, and the average and maximum wave height.

Further work is being carried out at present at New York University to forecast power spectra for the wave pattern in the generating area at different wind velocities, fetches and durations of wind action. The second step in wave forecasting then is to determine how such a "wave package" or pattern spreads from the generating area into other regions of the sea until it reaches the coast. It may be mentioned at this point that eddy viscosity also has to be taken into account when considering the decay of waves (swell), if the waves travel through a region of turbulent sea. Preliminary results indicate that "swell dissipation" due to turbulence in storm regions is a very important decay factor. In some instances observations have shown that swell originating in storm regions of the North Atlantic Ocean was wholly obliterated while crossing other regions with moderate to heavy sea.

Since all attempts to calculate the growth of ocean waves under wind action have been largely based upon empirical evidence, it seems that our theoretical progress depends critically on progress in our phenomenological knowledge. An urgent need today is for adequate wave records from the open sea under different sea state conditions, and careful analysis of these records to separate swell from wind-driven sea. Efforts to advance in this direction are being made, and we hope that they will help to clarify questions about the composite pattern of wind-generated sea, and so to stimulate further theoretical work.

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CHAPTER 8

THE THEORY OF THE REFRACTION OF A SHORT CRESTED GAUSSIAN SEA SURFACE WITH APPLICATION TO THE NORTHERN NEW JERSEY COAST

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INTRODUCTION

WHAT ARE OCEAN WAVES?

The Thorndike Barnhart Dictionary (1951) defines a wave as a "moving ridge or swell of water." Almost everyone will agree to this definition. Milne-Thompson (1938) in Theoretical Hydrodynamics begins Chapter Fourteen on waves with the two paragraphs quoted in full below:

> "14·10 Wave motion. A wave motion of a liquid acted upon by gravity and having a free surface is a motion in which the elevation of the free surface above some chosen fixed horizontal plane varies with time.
>
> Taking the axis of x to be horizontal and the axis of z to be vertically upwards, a motion in which the vertical section of the free surface at time t is of the form
>
> $$z = a \sin(mx - nt) \tag{1}$$
>
> where a, m, n are constants, is called a simple harmonic progressive wave."

The definition of a wave as a moving ridge or swell of water does not say that all of the waves in a given wave system must have exactly the same amplitude, a, the same direction, toward positive x, the same angular frequency, n, the same wave number, m, and infinitely long crests in the y direction. In fact a wave system need not be a simple harmonic progressive wave at all.

On the open ocean or at a given coast, no man has ever seen a wave system of the form of equation (1). Such a system can only be approximated in a wave tank. Waves in nature, generated by the winds, do not have the properties of equation (1). No man will ever see a wave system on the open ocean like equation (1).

The moving ridges or swells of water on the surface of the ocean do not duplicate each other exactly in height or in the time intervals between successive crests. They do not extend to infinity along the crests. Our contention is that equation (1) is not an adequate re-

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presentation for actual ocean waves. Our contention is also that any physical quantities derived from the assumption that the sea surface is like equation (1) are invalid and inaccurate.

WAVE ANALYSIS

The usual practice in ocean wave analysis is to take a wave record or a pressure record, and count bumps. The maximum swing upward is measured, the following minimum swing is then found and the difference is the "height" of that particular "wave." The time interval between two successive crests is also called the "period" of that particular wave. Upon the completion of the <u>analysis</u> of the record, the result is a set of numbers for the wave "heights" and another set of numbers for the wave "periods."

However, at this point, a difficulty is encountered. We have a whole set of different "heights," and a whole set of different "periods." An inconsistency is evident here in that equation (1) only holds for <u>one</u> height and <u>one</u> period. The dilemma is usually evaded by averaging the height of the one third highest waves and calling the result the "significant" height. The time interval between successive crests of the one third highest waves is also averaged and the result is called the "significant" period.

The result is, lo and behold, <u>two</u> very nice simple numbers, and our troubles are all over. We have just enough numbers to fit equation (1). By the process of brute force, we have thrown away the irregularity of the original record, the short crestedness of the actual sea surface, and the difference between a "sea" wave condition and a "swell" wave condition.

FAULTS OF THE METHOD

One fault of the above method of analysis is that every time an analysis of simultaneous pressure and free surface records has been made, the result is that the "significant" height of the pressure record predicts a "significant" height of the free surface record (based upon the pressure record "significant" period) which is too low and which is in error by any where from 10 to 25 percent. This error has been explained by Pierson (1952) and Pierson and Marks (1952) and the error lies in the complete inadequacy of the "significant" height and period method of analysis.

It is our contention that the method of analysis described above is inadequate and inaccurate in connection with the entire process of ocean wave analysis, ocean wave forecasting, and ocean wave refraction. Pierson (1952) has treated the problem of wave forecasting and wave analysis in a more thorough way which shows that this is the case for wave forecasting and wave analysis.

PURPOSE OF PAPER

In this paper, theories of wave pattern analysis and of wave refraction as developed in electronic theory by Wiener (1949) and Tukey (1949) and as applied by Pierson (1952) will be applied to a model problem of wave refraction of points along the New Jersey coast. The result will be to show that the wave heights not only vary from point to point along the New Jersey coast but that also the "significant" period is not the same from point to point for the same wave system in deep water. Other features of interest will also be pointed out.

A SHORT CRESTED GAUSSIAN SEA SURFACE

DEFINITION

A formula which appears to yield all of the known properties of actual ocean waves except those due to non-linearity, is given by equation (2) for waves in infinitely deep water. η (x,y,t) is the free surface. The function, $[A_2(\mu, \theta)]^2$ is the power spectrum of the wave system. The variable, μ, is the spectral frequency, $(2\pi/T)$. The variable, θ, assigns directions to the crests. The function, $\psi(\mu, \theta)$, is a point set function chosen in random phase according to a rectangular probability function from zero to 2π. Equation (2) is not an integral which can be evaluated like those in the back of the calculus book. It is simply a schematic and idealistic way of thinking about a certain type of limiting process.

$$\eta(x,y,t) = \int_{-\pi}^{\pi}\int_{0}^{\infty} \cos\left[\frac{\mu^2}{g}(x\cos\theta + y\sin\theta) - \mu t + \psi(\mu,\theta)\right] \sqrt{[A_2(\mu,\theta)]^2\, d\mu\, d\theta} \quad (2)$$

The power spectrum is everywhere positive and it is defined over some area in the μ, θ polar coordinate system. The power spectrum has the dimension of cm^2-sec/radian. The form of the power spectrum determines whether the waves are "sea" waves or "swell" waves. If the power spectrum varies over a wide range of μ and θ, say, from $2\pi/15$ to $2\pi/1$ for μ and over a range of 45° for θ, the result is "sea" waves. If the power spectrum varies over a narrow range of μ and θ, say, from $2\pi/14$ to $2\pi/10$ for μ and over a range of 10° for θ, the result is "swell." Evidence for this statement will be cited later.

Equation (2) can be approximated to any desired degree of accuracy by a partial sum. The procedure is to divide the μ, θ polar coordinate system by picking values of μ at the points; μ_0, μ_1, μ_2 .... μ_{2n}, and values of θ at the points; $-\pi$, θ_1, θ_2, θ_3 .... θ_{2p-1}, π. Then the partial sum is given by equation (3) where the values of $\psi(\mu_{2r+1}, \theta_{2q+1})$ are picked at random between the values of zero and 2π.

$$\eta(x,y,t) = \lim_{\substack{r\to\infty \\ p\to\infty \\ \mu_{2r}\to\infty}} \sum_{q=0}^{p-1}\sum_{r=0}^{n-1} \cos\left[\frac{(\mu_{2r+1})^2}{g}\left(x\cos\theta_{2q+1} + y\sin\theta_{2q+1}\right) - \mu_{2r+1}t + \psi(\mu_{2r+1},\theta_{2q+1}\right. \cdot \sqrt{[A_2(\mu_{2r+1},\theta_{2q+1})]^2(\mu_{2r+2}-\mu_{2r})(\theta_{2q+2}-\theta_{2q})} \quad (3)$$

The integral given in equation (2) is the limit of equation (3) as the value of μ_{2r} approaches infinity and as the difference between successive μ's and θ's in the net approaches zero. Since $\psi(\mu, \theta)$ is picked at random, the limit is an infinite number of different forms for the sea surface, all with certain fundamental statistical characteristics for a given power spectrum.

PROPERTIES

Our claim is that equation (2) is a far better representation of actual ocean waves than is equation (1) fitted by the "significant" height and period method. Equation (2) is based upon the linear superposition of an infinite number of infinitesimally high sine waves with different directions and different periods. The result is an irregular pattern of short crested moving swells and ridges which appears to have all of the properties of waves on the ocean surface as they actually are except for non-linear effects.

THE EQUATION OF A WAVE RECORD MADE IN DEEP WATER

Equation (2) is a function of x, y, and t. When waves are observed as a function of time at any fixed point where equation (2) is valid, the result is that a function of the form of equation (4) is observed. Equation (4) can be defined by the limit of a partial sum in a way similar to the way equation (2) was defined above.

$$\eta(t) = \int_0^\infty \cos(\mu t + \psi(\mu)) \sqrt{[A(\mu)]^2 d\mu} \qquad (4)$$

It can also be proved that all of the equations given below are properties of the systems defined above when the waves are observed in deep water.

$$\lim_{\substack{\bar{T}\to\infty \\ \bar{y}\to\infty}} \int_{t^*}^{t^*+\bar{T}} \int_{y^*}^{y^*+\bar{y}} \frac{1}{\bar{T}} \frac{1}{\bar{y}} [\eta(x,y,t)]^2 dy\, dt = \frac{1}{2} \int_0^\infty \int_{-\pi}^{\pi} [A_2(\mu,\theta)]^2 d\theta\, d\mu \qquad (5)$$

$$\lim_{\bar{T}\to\infty} \frac{1}{\bar{T}} \int_{t^*}^{t^*+\bar{T}} [\eta(t)]^2 dt = \frac{1}{2} \int_0^\infty \left[\int_{-\pi}^{\pi} [A_2(\mu,\theta)]^2 d\theta \right] d\mu = \frac{1}{2} \int_0^\infty [A(\mu)]^2 d\mu \qquad (6)$$

$$\int_0^\infty \int_{-\pi}^{\pi} [A_2(\mu,\theta)]^2 d\theta\, d\mu = E \qquad (7)$$

The equations state that averages over an infinite distance and infinite time must be taken. Averages in reality over several kilometers or over twenty or thirty minutes are sufficiently long to provide extremely reliable values.

THE GAUSSIAN PROPERTY

There is one more important property of this method of representing ocean waves. As was first shown by Rudnick (1951), points picked from a wave record such as equation (4) are distributed according to a normal probability function with a second moment given by E/2 as stated by equation (8).

$$P(-\infty < \eta(t) < K) = \frac{1}{\sqrt{\pi E}} \int_{-\infty}^{K} e^{-\xi^2/E} \, d\xi \tag{8}$$

The above property has been verified by a number of different observations. For further details, see the references to Rudnick (1951), Pierson (1952), and Pierson and Marks (1952).

It should be noted that these representations for the wave system change slowly as a function of time and position and that a given power spectrum is only valid for twenty or thirty minutes and over a relatively small area.

WAVE REFRACTION THEORY

INTRODUCTORY REMARKS

Elementary wave refraction theory is developed on the tacit assumption that ocean waves have the form given by equation (1). One direction is taken for the waves, and the "significant" height and "significant" period are assigned to the equation. Then with a refraction diagram, the height and direction of the wave at the point of interest is found. It is assumed (we believe, erroneously) that the "significant" period does not change. If ocean waves were actually like equation (1), then first of all the concept of the "significant" height and period would not be needed at all. All waves would be exactly the same in height, the crests would be infinitely long, and every crest could follow exactly T seconds after its predecessor. Life would be very simple, and theory and observation probably would agree quite well. See, for example, a paper by Marks (1951) where pure sine waves are used in ripple tank studies, and see also all papers reporting on model studies in which pure sine waves were used. However ocean waves are like equation (2), and in current practice, especially for "sea" conditions, one picks out one period and direction from an infinite number of equally important periods and directions, refracts the wave system with just these two values, and then wonders why the process did not work. That it is practically impossible to verify wave refraction theory in actual wave systems for complicated refraction conditions was shown by Pierson (1951b) in a study of wave conditions at Long Branch, New Jersey.

Wave refraction theory as developed in studies of ocean waves is correct for a simple harmonic progressive wave. The papers by Eckart (1951), Johnson, O'Brien, and Isaacs (1948), Peters (1952), Sverdrup and Munk (1944), and Pierson (1951a) are all based upon the assumption that the wave are of the form of equation (1). A forthcoming paper by Arthur, Munk and Isaacs (1952) which will appear in the Transactions of the American Geophysical Union improves on the previous techniques of orthogonal construction as presented by Johnson, O'Brien and Isaacs (1948).

All of the above theoretical results carry over directly to actual ocean waves with all of their fundamental irregularity by virtue of the fact that the wave system is linear. All that we have to do is find out what happens to each term in equation (3), pass to a limit and compute what the new wave system looks like in a form analogous to equation (2) at the new point of observation in the refraction zone. Each term in equation (3) is a simple harmonic progressive wave and theoretically we know everything we need to know about simple harmonic progressive waves.

THE REFRACTION OF A PURE SINE WAVE

The first step, then, in the study of the refraction of a short crested Gaussian sea surface as in equation (2) is to study the refraction of a pure simple harmonic progressive wave as in equation (1). The most general simple harmonic progressive wave in deep water can be represented by an equation of the form of equation (1) where A_1 is the amplitude, θ is the direction toward which the wave is traveling with respect to some x', y' Cartesian coordinate system, μ_1 is a fixed frequency, and δ_1 is an arbitrary phase.

$$\eta_1(x',y',t) = A_1 \cos\left[\frac{\mu_1^2}{g}(x'\cos\theta_1 + y'\sin\theta_1) - \mu_1 t + \delta_1\right] \quad (9)$$

The angle, θ, is most easily associated with the x' axis of a coordinate system drawn with respect to a storm system out over the ocean. As the waves approach the New Jersey coast, it is convenient to define a coordinate system such that positive x points due west and y points to the south. Then the above angle considered with respect to a storm becomes a new angle considered with respect to the coast which will be called θ_F. Equation (9) for the wave still in deep water then becomes equation (10).

$$\eta_1(x,y,t) = A_1 \cos\left[\frac{\mu_1^2}{g}(x\cos\theta_F + y\sin\theta_F) - \mu_1 t + \delta_1)\right] \quad (10)$$

In general, we are interested in the wave system which will be present at some point in the shallower water at some fixed depth, H. We define a third coordinate system at this point with X_R pointing directly on shore, and θ_R measured with respect to the coordinate system.

A number of things happen to the wave system represented by equation (10) as the waves are refracted by the shallower water. These effects can be computed theoretically by constructing orthogonals by Snell's law and by considering the effect of the shoaling water. The net effect is that five things happen to the wave system. The wavelength of the wave shortens solely as a function of μ_1 and H. The direction toward which the crest is traveling changes due to the change in direction of the orthogonals. The crest becomes higher or lower due to the convergence or divergence of the orthogonals and the effect of shoaling. The crests have some phase difference with respect to the phase in deep water. Finally the wave crests become curves instead of straight lines.

All but this last effect can be represented by equation (11). The curvature of the crests is extremely difficult to represent analytically, and we limit this derivation by saying that equation (11) represents the crests in the vicinity of the point x_R, y_R equal to zero but that at large distances from the point, the derivation will not be satisfactory.

$$\eta_R(x_R, y_R, t) = A_{IR} \cos\left[\frac{I(\mu, H)\mu_I^2}{g}(x_R \cos\theta_R + y_R \sin\theta_R) - \mu_I t + \delta_I + \delta_{IR}\right] \quad (11)$$

The coefficient of the space variables, namely $I(\mu_I, H)\mu_I^2/g$ is equal to $2\pi/L_1$ where L_1 is the wave length of a wave with a period equal to $2\pi/\mu_1$ in water of depth H. It can be shown that a function, $I(\mu, H)$, can be found which easily yields the needed number by which μ_I^2/g (equal to $2\pi/L_{o1}$, where L_{o1} is the deep water wavelength) must be multiplied in order to obtain the value of $2\pi/L_1$.

The change in amplitude is a function of the deep water direction, θ_F, and of the deep water period (or frequency). A function of these variables can be found such that when A_1 is multiplied by it the result is A_{IR}, the amplitude after refraction. This function has been found as a function of period and direction for many places along the coasts of the United States. Examples are given of the forms it can take by Munk and Traylor (1947) and by Pierson (1951a) along with many others. This function can just as easily be plotted as a function of μ and θ_F, and the result will be a function defined as $K_H D(\mu, \theta_F)$ where the effects of refraction and shoaling are both included.

The angle, θ_R, is also a function of θ_F and μ, and it can be found by the same techniques that the above function was found. We define θ_R by equation (12)

$$\theta_R = \Theta(\mu, \theta_F) \quad (12)$$

In the refraction of a system like equation (2), a result will be that the phase change is unimportant although for precise treatment of any partial sum it should be theoretically known. We shall neglect the added refinement of considering δ_{IR} as a function of μ and θ_F.

THE REFRACTION OF A PARTIAL SUM

Under the above assumptions, the refraction of the sum of purely sinusoidal progressive waves as given by equation (3), is a straight forward procedure. The system is first referred to the μ, θ_F coordinate system. Then each term in the partial sum is treated by multiplying the amplitude of the term by the value of $K_H D(\mu, \theta_F)$ and by changing θ_F to θ_R with the aid of equation (12) for the appropriate direction and frequency. The wave length is changed to its new value for the shallower water.

The justification of such a procedure is that the system is linear and the total disturbance is the sum of all of the individual terms in the partial sum. For several hundred terms in the partial sum, the system would already have the appearance of actual ocean waves. The preponderance of the theoretical evidence and of the observational evidence is that the spectrum of ocean waves is continuous, and that an infinite number of terms must be considered, at least theoretically, in order to describe the sea surface properly.

THE REFRACTION OF A SHORT CRESTED GAUSSIAN SEA SURFACE

The final part of the theoretical derivation is to consider what happens to equation (2) when the wave system represented by it is refracted. One condition which must be preserved in the limit is that the average square of the refracted wave system both as represented by the partial sum and by the power integral is the same.

Definition of terms - Let the function, $[K_H D(\mu, \theta_F)]^2$ which is the square of the function mentioned before be defined to be the spectrum amplification function. Also let the function, $\theta_R = \Theta(\mu, \theta_F)$, be defined to be the direction function.

Now, if the direction function is a function of μ and θ_F, it can be inverted and θ_F can be expressed as a function of μ and θ_R. Theoretically, the inversion would involve a mathematical representation for the function and solving for θ_F in terms of θ_R and μ. Practically, it involves reading off the values of θ_F along a line on which θ_R is a constant in the direction function, plotting those values in a μ, θ_R polar coordinate system, and isoplething the lines for θ_F equal to a constant. The inverse direction function can then be defined by equation (13).

$$\theta_F = \Theta^*(\mu, \theta_R) \tag{13}$$

The Jacobian of the inverse direction function is also needed. The result is defined by equation (14). This function can be approximated to a considerable degree of accuracy by finite differences from an isoplethed drawing of equation (13).

$$\frac{\partial \theta_F}{\partial \theta_R} = \frac{\partial \Theta^*(\mu, \theta_R)}{\partial \theta_R} = \Gamma(\mu, \theta_R) \tag{14}$$

The power spectrum after refraction - After these definitions, our problem is to find the power spectrum which represents the waves at the point of interest after refraction. We just multiply the power spectrum of the waves by the spectrum amplification function. The result is still a function of μ and θ_F. The substitution of equation (13), the inverse direction function, then expresses the above product in terms of μ and θ_R. The result is squeezed together as a function of μ and θ_R for low values of μ and it must be properly amplified by multiplication by $\Gamma(\mu, \theta_R)$

in order to preserve the correct value of the average square of the record. The result is then the power spectrum of the waves at the point of observation in the refraction zone. This power spectrum is then given by equation (15).

$$[A_{2RH}(\mu,\theta_R)]^2 = [A_2(\mu,\Theta^*(\mu,\theta_R))]^2 \cdot [K_H D(\mu,\Theta^*(\mu,\theta_R))]^2 \cdot \Gamma(\mu,\theta_R) \tag{15}$$

The waves after refraction.- It then follows that the waves in the vicinity of the point of observation in the refraction zone are represented by a power integral over the power spectrum defined in equation (15). The phases are still to be picked at random from a rectangular probability distribution. This is why it was not necessary to treat the phase change when the refraction of a simple harmonic progressive wave was considered. The representation of the waves at the new point of interest is then given by equation (16).

$$\eta_R(x_R,y_R,t) = \int_{-\pi}^{\pi}\int_0^{\infty} \cos\left[\frac{\mu^2 I(\mu,H)}{g}(x_R\cos\theta_R + y_R\sin\theta_R) - \mu t + \psi(\mu,\theta_R)\right]\sqrt{[A_{2RH}(\mu,\theta_R)]^2 d\mu d\theta_R}$$

Equation (16) can be approximated by a partial sum just as equation (2) was approximated by a partial sum. For a large number of terms in the partial sum, it can be shown that the result is the same as the result of refracting the individual terms in the partial sum from deep water as was done in the section entitled, the refraction of a partial sum.

The equation of a wave record in the refraction zone - It can be shown that equation (16), if observed as a function of time at the point of interest, can be given by equation (17). Equation (17) would represent a wave record made with, say, a step resistance gage such as the one described by Caldwell (1948) in the refraction zone. A pressure record would have to have its power spectrum corrected for the effect of depth by a correct amplification factor point for point over the entire range of μ before it would represent the free surface power spectrum (see Pierson and Marks (1952)).

$$\eta_R(t) = \int_0^{\infty} \cos(\mu t + \psi(\mu t + \psi(\mu)) \sqrt{[A_{RH}(\mu)]^2 d\mu} \tag{17}$$

The function, $[A_{RH}(\mu)]^2$, can be found in either of the two ways defined by equations (18) and (19). Equation (19) shows that $\Gamma(\mu,\theta_R)$ need not be found if simply the wave record at one point as a function of time is needed.

$$[A_{2RH}(\mu)]^2 = \int_{-\pi}^{\pi} [A_{2RH}(\mu,\theta_R)]^2 d\theta_R \tag{18}$$

$$[A_{RH}(\mu)]^2 = \int_{-\pi}^{\pi} [A_2(\mu,\theta_F)]^2 [K_H D(\mu,\theta_F)]^2 d\theta_F \tag{19}$$

Additional properties - It can also be proved that the properties expressed by equations (20), (21), (22), (23) and (24) hold true at the point of observation in the refraction zone. The average over y_R can be restricted to only a few feet and the results would still be valid. The average square of the record as observed in the refraction zone is usually not the same as the average square of the record in deep water. These results only hold out beyond the point where non-linear effects become important, and for this reason, the wave record is still Gaussian as equation (24) states.

$$\lim_{\substack{\bar{T}\to\infty \\ \bar{y}_R\to\infty}} \frac{1}{\bar{T}\,\bar{y}_R} \int_{t^*}^{t^*+\bar{T}} \int_{y_R^*}^{y_R^*+\bar{y}_R} [\eta_R(x_R,y_R,t)]^2 dy_R dt = \frac{1}{2}\int_0^{\infty}\int_{-\pi}^{\pi} [A_{2RH}(\mu,\theta_R)]^2 d\theta_R d\mu \tag{20}$$

$$\lim_{\bar{T}\to\infty} \frac{1}{\bar{T}} \int_{t^*}^{t^*+\bar{T}} [\eta(t)]^2 dt = \frac{1}{2}\int_0^{\infty} [A_{RH}(\mu)]^2 d\mu \tag{21}$$

$$\int_0^{\infty}\int_{-\pi}^{\pi} [A_{2RH}(\mu,\theta_R)]^2 d\theta d\mu = E_R \tag{22}$$

$$E_R \neq E \tag{23}$$

$$P(-\infty < \eta_R(t) < K) = \frac{1}{\sqrt{\pi E_R}} \int_{-\infty}^{K} e^{-\xi^2/E_R} d\xi \tag{24}$$

SOME GENERAL COMMENTS

The above theoretical derivation is rather complicated. It suggests, at least, that ocean waves are far more complicated and far more intricate in their properties and construction than current theories and practices would admit. Waves are complicated, and oversimplifications at the start of a theoretical consideration of their properties must eventually lead to erroneous predicted results. The complete power spectrum, $[A_2(\mu,\theta)]^2$ of a wave system has never been determined. Its exact functional form is unknown. Arthur (1949) has shown that waves from a storm propagate out of the storm at angles to the direction of the wind such that they arrive at points they could not possibly reach if they traveled only in the direction of the average wind. Wave spectra as a function of μ alone determined electronically without an intensity scale have been reported by Klebba (1946), Deacon (1949), Rudnick (1951) and Barber and Ursell (1948). These spectra and a few determined by more precise techniques all show that μ can vary from $2\pi/20$ to $2\pi/6$ or over a range of periods from twenty seconds to six seconds, and that the various spectral components are all important. Since the records are pressure records in fairly deep water, there is even reason to believe that periods less than six seconds are also of importance.

If the available evidence suggests that "sea" waves have a power spectrum which varies over a wide range of μ and θ in a storm, and if the theory of refraction presented above is correct, then it is of interest to assume some functional form for the power spectrum and to find out the spectrum of the waves after refraction. The result will be that interesting features predicted by the model wave system will be obtained which will show that caution must be employed in the interpretation of wave records which are currently obtained along the coasts of the United States. In particular, the results will show that wave records obtained at Long Branch, New Jersey, do not represent wave conditions at nearby points on the New Jersey coast.

APPLICATION TO THE NORTHERN NEW JERSEY COAST

A MODEL STORM

In order to discover some of the consequences of the above theory and in order to provide an example of the techniques to be employed in forecasting waves according to the properties of their power spectra, a model storm was constructed over the Atlantic Ocean. Winds in the storm were assumed to be blowing from east to west over an area 566 km long and 550 km wide for a total duration of 24 hours. The center of the forward edge of the storm area was located 872 km due east of Cape Hatteras or at latitude 35°N and longitude 64°W. The time, t equal to zero, was referred to the start of the winds over the storm areas. The assumed functional form of the power spectrum was based upon the observations and results cited above. The center of the forward edge of the model storm was located 825 km from Long Branch, New Jersey.

Given these assumptions, the power spectrum at various times and places outside of the storm area can be forecasted according to the methods described by Pierson (1952). The wave conditions in deep water just offshore from Long Branch, New Jersey, were forecasted by these techniques and the different power spectra at this point were found.

The power spectrum of the waves off the New Jersey coast varies very, very slightly over distances comparable to the distance from Asbury Park to Sandy Hook which is 7 nautical miles. It can therefore be assumed to be the same in form for all points in deep water along this section of the coast.

THE THEORY OF THE REFRACTION OF A SHORT CRESTED GAUSSIAN SEA SURFACE WITH APPLICATION TO THE NORTHERN NEW JERSEY COAST

WAVE REFRACTION DATA

Source- Our interest in this particular paper is to find out the effects of refraction on these waves as they move from deep water to the coast. Data prepared for the northern New Jersey coast in a study by Pierson, Martineau, James and Pocinki (1951) were available and these data were worked up in more detail for three points along the coast for a depth offshore of 20 feet mean sea level.

The points which were chosen were at the base of Sandy Hook, near Ship Ahoy Inn, at latitude 40°22'N; at Long Branch, near the North End Beach Club, at latitude 40°18'N; and near Asbury Park*, at latitude 40°15'N. The point at Sandy Hook is four nautical miles north of Long Branch and the point near Asbury Park is three nautical miles south of Long Branch. These differences in distance are negligible compared to the scale of the wave forecasting problem.

The spectrum amplification function and the direction function

The effect of refraction at these three points is quite different. Figure 1 shows the spectrum amplification function for Ship Ahoy Inn, and Figure 2 shows the direction function. Figure 3 shows the spectrum amplification function for Long Branch, and Figure 4 shows the direction function. Figure 5 shows the spectrum amplification function for the point near Asbury Park, and Figure 6 shows the direction function.

The angular variables on these six figures are labeled in two different ways. One way shows the direction toward which the elemental components are traveling labeled in degrees from north. In such a system, the angle increases in a clockwise direction, and the notation in the derivation does not provide for such an angular system. The other way shows the angle, θ_F, where θ_F varies in a counterclockwise direction. The angle, θ_F, is zero for waves traveling from east to west. It is equal to ten degrees for waves traveling toward 260° (from north). These values for θ_F are shown in parenthesis on the figures. Since the coast runs very nearly north-south in the vicinity of Long Branch, the problem can be treated simply in terms of θ_F, but for coasts which are not north-south, sometimes another change of angular variable helps.

These six figures have features in common, and yet they are quite different. They show that it is practically impossible for spectral components with periods greater than 14 seconds to reach the northern corner of the state of New Jersey. The data have been analyzed by extrapolation for μ less than $2\pi/14$. At a depth of 20 feet, waves with a wavelength of 40 feet are unaffected by the bottom. Thus for μ greater than or equal to $2\pi/2.8$ the spectrum amplification function is essentially one everywhere. Even for μ equal to $2\pi/4$, the waves are affected by only a narrow strip of depths along the coast, and the spectrum amplification function for all three places is essentially the same. Between the values for μ equal to $2\pi/4$ and $2\pi/6$, all three spectrum amplification functions

*Actually the point is about two miles north of Asbury Park, proper.

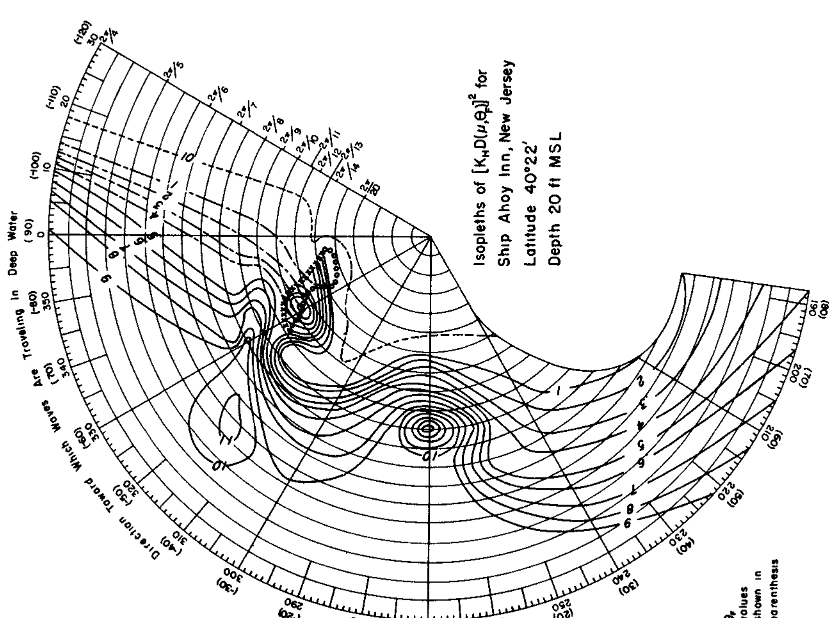

Fig. 1. Spectrum amplification function for Ship Ahoy Inn.

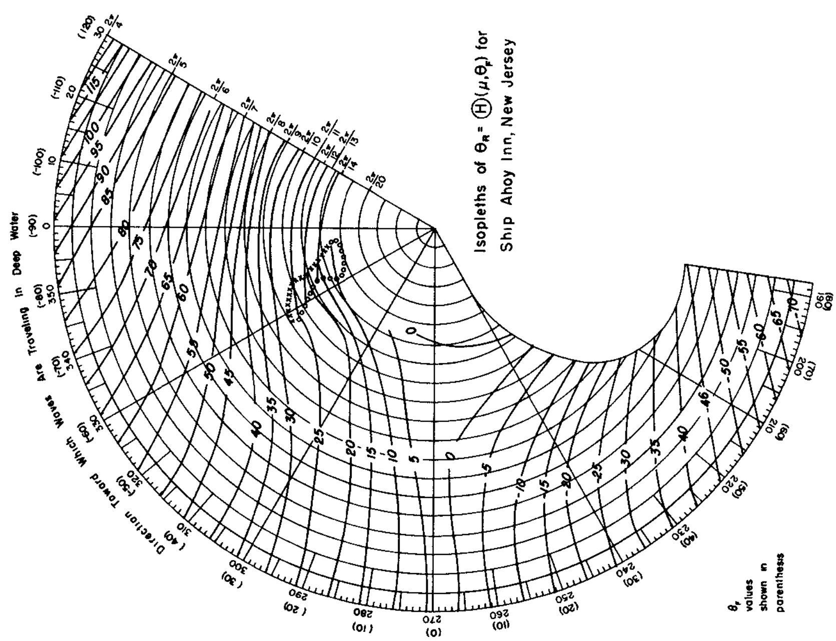

Fig. 2. Direction function for Ship Ahoy Inn.

THE THEORY OF THE REFRACTION OF A SHORT CRESTED GAUSSIAN SEA SURFACE WITH APPLICATION TO THE NORTHERN NEW JERSEY COAST

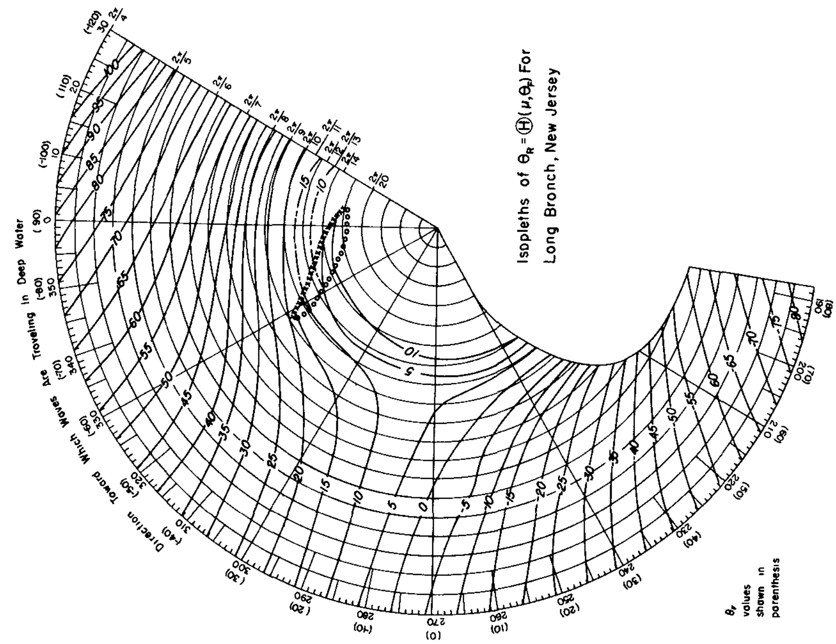

Fig. 4. Direction function for Long Branch.

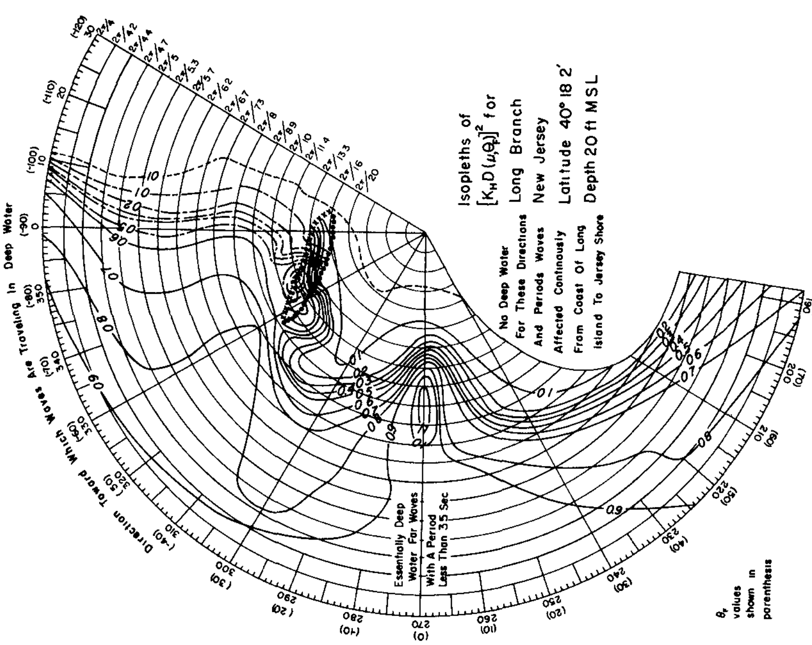

Fig. 3. Spectrum amplification function for Long Branch.

have values greater than 0.8 over a wide range of directions.

Each figure shows a narrow winding band of isopleths such that a small change in μ results in a large variation in the spectrum amplification function. The band is located at different places in the three figures. If the band is toward the center of the diagram, then low values of μ (corresponding to high periods) can be observed at that point if they come from those particular directions. If the band is toward high values of μ, then low frequencies do not show up at that point.

The little area bounded by circles and crosses enclosed an area where two heights and two directions exist for one sine wave in deep water. It indicates the presence of caustics in the orthogonal pattern. For further information, see Pierson (1951a).

In particular, compare the spectrum amplification function for Ship Ahoy Inn and Asbury Park for spectral components traveling toward directions between 300° and 310°. The 0.1 contour at Ship Ahoy Inn cuts down all spectral components for μ less than $2\pi/8$ for this range of angles. The 0.1 contour at Asbury Park affects only values of μ less than $2\pi/16$. In general over a wide range of θ and μ for waves traveling toward the northwest, the spectrum amplification function for Asbury Park amplifies low values of μ, (high period) far more than the spectrum amplification for Ship Ahoy Inn.

THE WAVE POWER SPECTRUM IN DEEP WATER AS FORECASTED FROM THE MODEL STORM

In the model which was constructed, it was possible to forecast the theoretical spectrum for deep water at six hour intervals. For example, at t equal to 54 hours, the spectrum was found to have the form shown in Figure 7 as a function of μ and θ. Figure 7 shows that the spectra consist of elemental frequencies which vary from $2\pi/18.2$ to (in this figure) $2\pi/5.3$. The sharp sides of the spectrum are due to the approximations used in the forecasting theory, and in actuality the edges and sides would be rounded. Later spectra included even lower values for the period. The waves in deep water would appear to be traveling toward approximately 300°. They would be quite short crested and the elemental spectral components would be present for all directions from 287° to 311°. The lowest frequency, $2\pi/20$ for some of the first spectra, was chosen to correspond with the observed maximum period found by Barber and Ursell (1918) in a storm with a wind velocity of 45 knots. The integral over θ of the deep water μ, θ power spectrum of course yields the power spectrum, $[A(\mu)]^2$, of the waves in deep water. Such a power spectraum could be evaluated from, say, a twenty-five minute record made with the spark plug type spar buoy wave recorder constructed by the Beach Erosion Board. The variation of $[A_2(\mu, \theta)]^2$ as a function of θ is much more difficult to determine in practice. If the deep water waves would have been picked to be traveling more toward 330°, the results which would have been obtained upon refraction would have been even more pronounced.

Some additional recent theoretical evidence

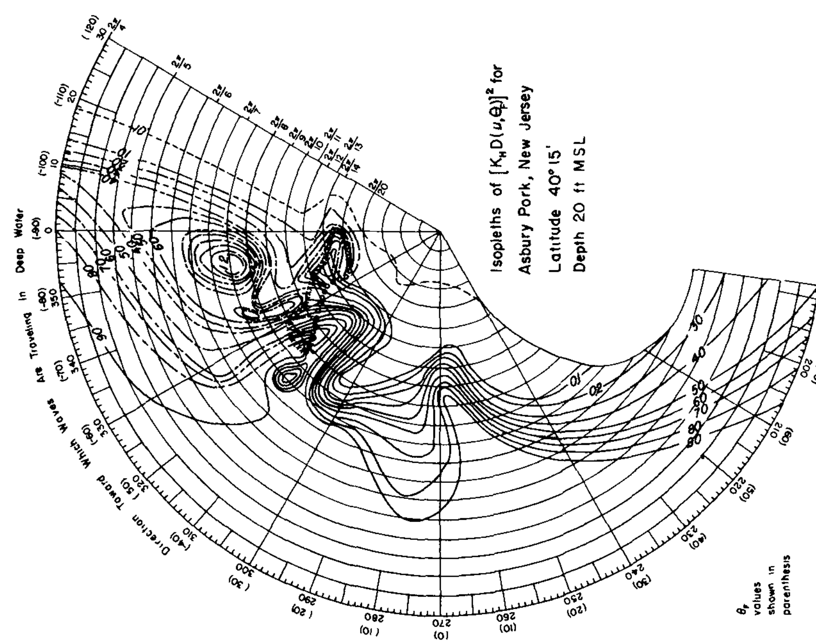

Fig. 5. Spectrum amplification function for Asbury Park.

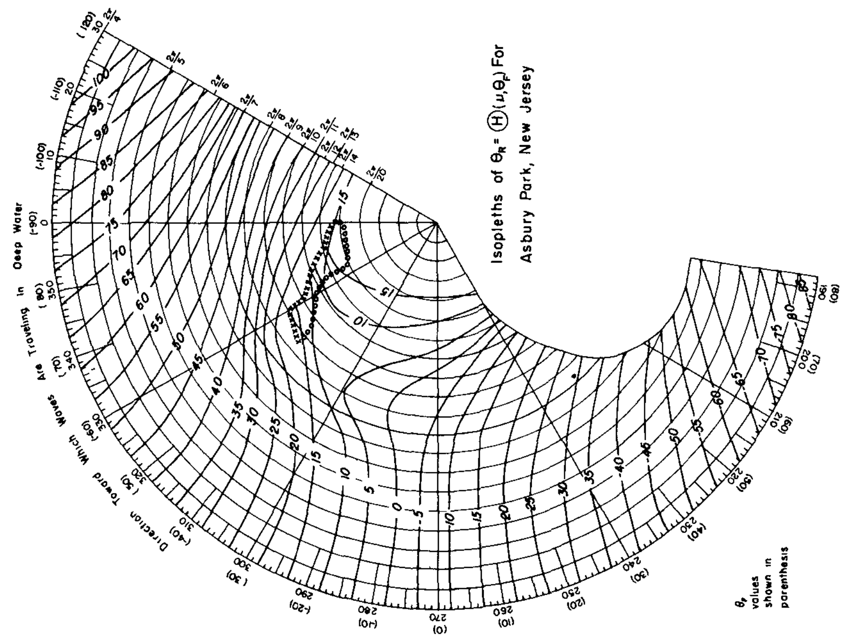

Fig. 6. Direction function for Asbury Park.

A certain functional form was assumed for the power spectrum of the waves at the edge of the storm. At the time when this investigation was undertaken, there was not too much evidence available as to the relative power associated with each frequency. Very recent results of Darbyshire show that the power spectra in deep water should be much higher for low values of μ and much lower for high values of μ than the ones assumed in this paper. The result would be even more striking in the effects on the refracted power spectra which would result. The results predicted by our mathematical model could easily be an underestimate of the actual effect.

THE WAVE POWER SPECTRA AT THE COAST

These forecasted power spectra in deep water were multiplied by the spectrum amplification function for each of the three points of interest. Then the result was integrated numerically over θ_F to find the function $[A_{RH}(\mu)]^2$, for that time and place. By virtue of equation (19), the transformations involved in equation (15) need not be made if only $[A_{RH}(\mu)]^2$ is desired.

The forecasted power spectra, $[A(\mu)]^2$ and $[A_{RH}(\mu)]^2$, for deep water and for Ship Ahoy Inn, Long Branch, and Asbury Park are shown in Figure 8. The values of E and E_R for each spectrum are shown below the spectrum and the range of variation over μ is also shown. Note that the origin of the μ axis is not shown and that it lies progressively more to the left for the later power spectra.

Free surface wave records, produced by the above power spectra would, of course, have some significant height and period. The significant period would correspond to some value of μ near the center of the spectrum. The significant height, crest to trough, would be approximately equal to $2.88\sqrt{E/2}$. These values are also shown below the different spectra. For some important recent results on the distribution of wave heights in a wave record, see a forthcoming paper by Longuet-Higgens(1952). The results of Longuet-Higgens, which are more accurate, yield a slightly higher value for the above factor.

The first discernible swell in deep water would be observed eighteen hours after the start of the storm. Its significant period would be about 17.4 seconds, and its significant height would be about 4.18 feet. At Ship Ahoy Inn, practically no waves would be observed whereas at Asbury Park, waves with a significant period of 17.3 seconds and a height of 1.56 feet would be evident.

Thirty hours after the start of the storm, waves with a significant height of 8.27 feet and a significant period of 12.5 seconds would be present in deep water. At Ship Ahoy Inn, the significant period would appear to be about 10.5 seconds and the significant height would be about 1.46 feet. At Asbury Park, the significant period would appear to be about 11.8 seconds and the significant height about 3.97 feet.

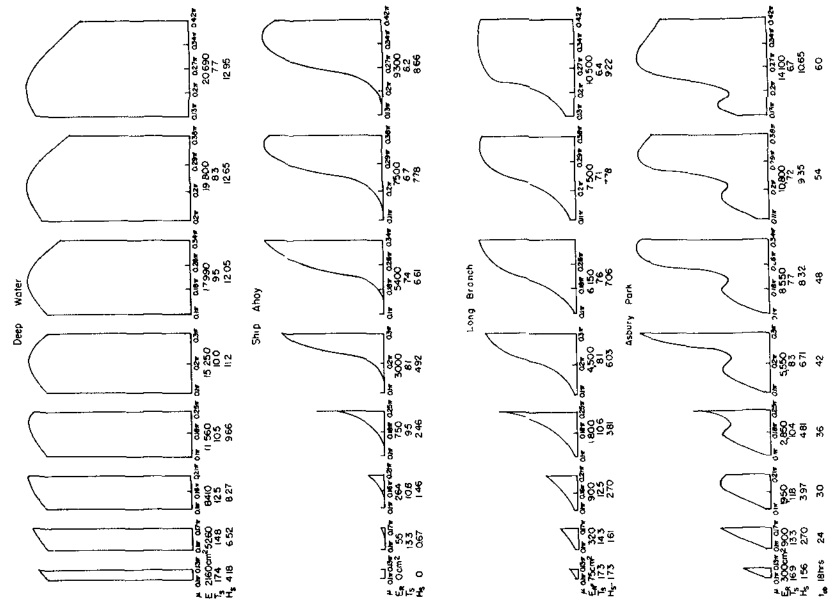

Fig. 8. Forecasted power spectra for deep water and the three points of observation.

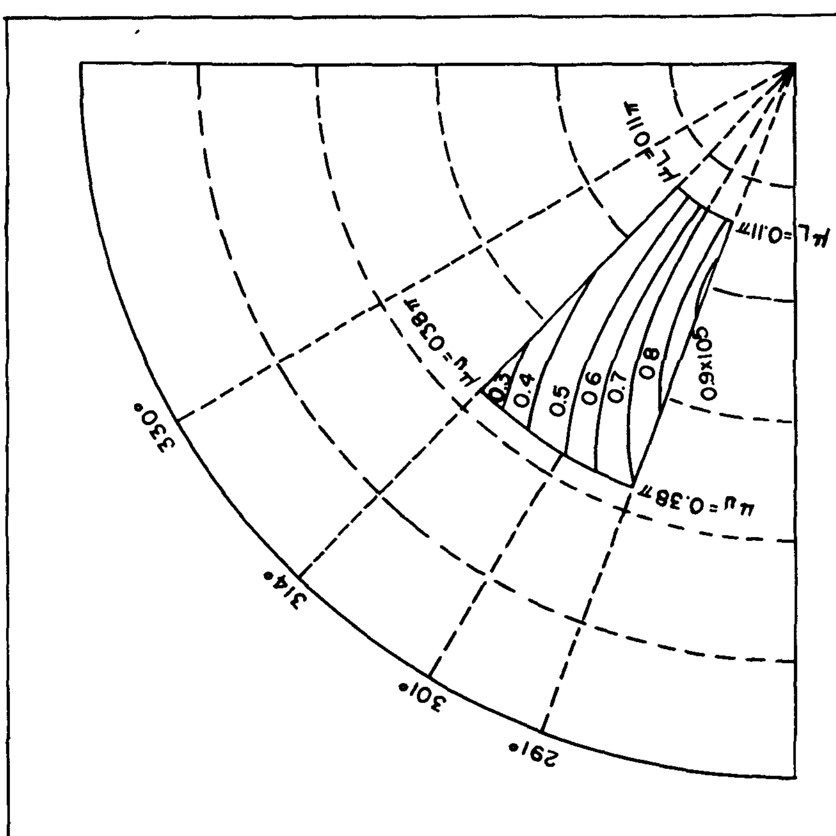

Fig. 7. Forecasted wave spectrum for a time 54 hours after the start of the storm (E equals 2,400 cm^2).

The spectra for the three points near shore are markedly different with low frequency components containing greater energy at Asbury Park than at the other two places. The waves present would differ in fundamental ways even for those cases in which the significant periods differ by only a few tenths of a second.

Pressure recorders located at the points of interest would not record these values, and a simple computation of the free surface values by means of the "significant" height and period of the pressure record would be incorrect. The pressure record would have a higher significant period than the free surface value, and the computed free surface significant height as based upon the pressure significant period would be too low.

INTERPRETATION OF RESULTS

The reason for the results is basically the effect of the Hudson Submarine Canyon. The elemental pure sine waves in the partial sum for high periods are focused at Asbury Park and very nearly obliterated at Ship Ahoy Inn. The lower period elemental waves still show up at Ship Ahoy Inn.

Consider the particular spectra for 18 hours after the start of the storm. The results show that for a storm to the southeast of the New Jersey coast, there could be conditions such that a significant period of 9.5 seconds and a significant height of 2.46 feet would be observed at Ship Ahoy Inn and simultaneously a significant period of 10.4 seconds and a height of 4.81 feet would be observed at Asbury Park. These two points are only seven nautical miles apart.

The waves in deep water would have a significant period of 10.5 seconds and a significant height of 9.66 feet. If a direction of 300° is assumed for the deep water waves, and if the waves from deep water are refracted according to their significant height and period, the result is a forecast of 10.5 seconds and 1.36 feet at Ship Ahoy Inn and 10.5 seconds and 3.05 feet at Asbury Park.

These values are compared in Table I. The significant height and period method when compared with the more accurate power spectrum method gives completely different results. Note that the significant period also changes from deep to shallow water in the power spectrum method of wave refraction. Of course, the computation of the significant height and period from the refracted power spectra is a step in the wrong direction because the power spectra tell us much more about the waves than these two numbers.

<u>The usefulness of coastal wave records</u>

Wave records are currently obtained at Long Branch and evaluated by the significant height and period method. If we take the significant height and period of these records and assume some one deep water wave direction, then the deep water significant height and period could be deduced from the refraction diagram. From these values, the significant

Table I. Comparison of the power spectrum method of wave refraction and the significant height and period method.

	Power Spectrum Method		Sig. Hgt. and Period Method	
	Sig. Hgt. feet	Sig. Period seconds	Sig. Hgt. feet	Sig. Period seconds
Deep Water	9.66	10.5	9.66	10.5
Ship Ahoy Inn	2.46	9.5	1.36	10.5
Asbury Park	4.81	10.4	3.05	10.5

height and period at the other two points could be forecasted. The results would be just as much in error as the refraction from deep water by the significant height and period. There is no assurance that the significant period as observed near the coast will be the same in deep water. It is necessary to conclude, therefore, if these theoretical results are correct and approximate true conditions, that wave records at Long Branch, New Jersey do not yield reliable information at nearby points along the coast or in deep water when interpreted by the significant height and period techniques.

THE EFFECT OF THE DIRECTION FUNCTION

The effect of the direction function is to make the waves in a partial sum from equation (16) travel in more nearly the same direction compared to those in equation (2) for low values of μ . This means that if the waves are relatively short crested in deep water, they will be longer crested in the shallower water after refraction. Such a phenomenon can be observed in many aerial photographs of waves undergoing refraction, and Pierson (1952) has discussed two such photographs. The complete evaluation and interpretation of this feature has not been worked out, and results of a continued study will be reported in the future.

VERIFICATION

The actual verification of these results quantitatively has not been accomplished. This paper has been written to demonstrate a theoretical example of the refraction of a wave system with properties similar to those known to be the properties of actual ocean waves. To verify the results completely, pressure wave recorders at the three points would be needed, and a method for determining the deep water conditions would be needed. Partial verification from three pressure recorders would be possible since completely different spectra are predicted for the three points for the same time.

A qualitative verification of these results based upon crude wave measurements and purely visual observations can be given. When the group at New York University first began to study waves a few years ago, a hurricane generated waves from a position roughly the same as the one assumed for the model storm in this paper. A field trip was organized to observe these waves and within a time interval of forty-five minutes or so the waves at the three points under study in this paper were observed. The result based upon these crude observations was that waves with a significant period of six to eight seconds with a significant height of three or four feet were observed at Ship Ahoy Inn. At Long Branch, the significant period was nine or ten seconds and the significant height was four or five feet. At Asbury Park, the significant period was about twelve seconds, and the waves had a significant height of six or seven feet.

The observations were doubted because it was thought that the significant period had to be the same at all points. The theory of ocean wave refraction was based solely on equation (1), and a change of period is not possible in such a theory. It was thought at the time that there was an error in the observation techniques and not in the theory.

Finally, for what it is worth, we report the experience of those who swim along the Northern New Jersey coast. Those who like to ride the breakers as they come up to the beach report that they prefer to swim at the points to the south along the coast. The rollers, they say, are higher and more regular at points to the south. Since waves are rarely of zero height at any point along the coast when waves are present at other points, this suggests that there is a difference (and a long time statistical difference at that, since otherwise it would not have been noted by swimmers simply out for pleasant recreation) in the character of the waves along the coast.

CONCLUSIONS

Wave refraction is a complex problem since actual ocean waves are not simple harmonic progressive waves. Theoretical results from model wave forecasts, and crude visual observations suggest that <u>both</u> different significant periods and significant heights can result at nearby points when a short crested Gaussian sea surface is refracted. Care must therefore be exercised in the extension of wave observations made at one point to nearby points.

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PART 2
COASTAL SEDIMENT PROBLEMS

Chapter 9

GEOLOGY IN SHORELINE ENGINEERING AND ITS APPLICATION TO MASSACHUSETTS BEACH PROBLEMS *

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At the outset I wish to make it clear that I am presenting this subject as a geologist, and not as a coastal engineer. I stand only on the fringe of that area of engineering science, and it would be presumptuous of me to discuss techniques of shoreline engineering. On the other hand, analysis of geologic processes that have molded and are now molding the shoreline furnishes basic terrane data of importance in the solution of coastal problems. As a geologist, then, perhaps I may properly point out the pertinency of geology to these problems, and indicate the kinds of appropriate data that are within the province of the geologist to explore and interpret. More or less an observer on the sidelines, I have for some time been impressed by the intricacies of the problems involved in coastal engineering projects. Such engineering is, of course, highly scientific and technological. But it seems to me that it is also somewhat of an art, for it is strongly tempered by experience, and the success of a calculated solution to a problem is often anxiously awaited by the engineer when the project is completed. There seems to be less of the slide-rule certainty that characterizes the planning and design of a bridge. Will the sea-wall, the jetty, or the offshore breakwater, for examples, accomplish the intended results? Sometimes they do not because of some unrealized factors. Such factors are often obscure geologic conditions - unrecognized because the geologic regimens along shores seem to be very delicately balanced with respect to several factors, and to be sensitive to even slight interferences, despite the massiveness of the natural forces that are at work. The geologic history of the coast, translated to the present, together with minutiae of existing geologic features may demonstrate such obscure factors. It behooves the engineer, therefore, to seek the offices of geologic sciences. Perhaps at this point I may be pardoned to related digression if - to employ the vernacular - I "get something off my chest".

It seems axiomatic, and yet strangely to need emphasis, that the engineer should be able to plan his work more effectively and with greater economy if he were provided with a knowledge of the compositions, properties, structures, and implications of the geologic formations and forces with which he perforce is obliged to deal so directly and so intimately. The strength, soundness, and secureness of a structure may depend largely upon these elements. This is patent whether he is dealing with highways, bridges, foundations, reservoirs, or works for shoreline control. Terrane intelligence is within the primary field of the geologist to provide and he is far better equipped than the engineer to obtain and interpret such

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basic data. He is, if well trained and experienced, aware of complex interrelations of phenomena that to the untrained may seem to have no tangible relation to each other.

To a degree this is an appeal for both geologists and engineers to recognize the fundamental correlations of their sciences. An engineer should know when to seek and how to use geologic data. His failure to do so more often is perhaps to be blamed more on the geologist than on himself, for in the past the geologist has too commonly seemed to be "long-haired", and to interest himself in the philosophic vistas of his science rather than in the pertinent and practical applications of it. The geologist should, of course, know how to select and present his data for engineering use. The fault, however, is to be charged in no small part to those engineering schools - and there are many - whose curricula and faculty make little or no provision for acquainting the engineer with the solid substance of geologic science, or perhaps make a perfunctory gesture in that direction. It is very gratifying, however, to note that engineers are taking an increasingly greater interest in geologic sciences, which, besides standard lithologic, structural, and morphologic studies, include such specialized geologic fields and laboratory techniques as geophysical studies of terranes, petrographic studies of materials, and "soil mechanics" tests. In recent years the fine work of the Beach Erosion Board and its promotion of correlative studies by geologic specialists is but one of several outstanding examples of coordination between engineering and geology. Nevertheless, there is a very common lack of awareness among engineers that geologic sciences can provide specialized basic data for their problems. Let the technical schools look to it!

What are the basic data for coastal studies that derive from geology? The science deals with processes and materials - processes that are ever striving to establish an equilibrium and rarely approach it closely over a large area, and materials that are locally varied in composition, properties, and structures. In the study of coastal processes, both the engineer and the geologist must often transgress the stricter limits of their fields and enter the domain of mathematical physics. They should both be prepared to do so. By virtue of his training, however, the geologist can do much physical analysis from observation of currently active processes, and an interpretation of the history and evolution of the land area as indicated by a study of the morphology and materials of the coastal belt. To him the marks of the past disclose the path to the present, and may predict the course of the future. He has the advantage of four dimensions with which to work, the fourth being the greatly expanded vista of time. Seemingly intangible philosophy can be transmuted into tangible facts.

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The morphology of the coastal belt reflects the recent history and evolution of the shore, and it further implies the future course of development under a continuing regimen that, through a routine cycle of geologic changes, culminates in ultimate planation, unless interfered with by uplift, downsinking, or tilting of the land area with respect to sea level. The changes are both constructive and destructive. The engineer seeks to control, interrupt, mold, or prevent some of these changes. He is dealing, then, with dynamic geologic processes and can accordingly benefit by increasing his knowledge of the regimen. Geology begins its contribution to engineering by indicating the morphologic stage of the cycle, and consequently the principal forces that are acting upon the coast. By the same token it indicates the normal path of future changes. These facts are fundamental in the analysis upon which control or remedial engineering measures are to be based.

The stratigraphy, structure, and lithology of the coastal zone subjected to wave erosion are closely involved in beach preservation and construction projects. Sea-walls are necessarily massive and heavy, and require firm foundations; the sub-surface materials and stratigraphy, therefore, must be determined and interpreted with relation to their soundness as foundation materials. Some beaches, for instance, are built on layers of peat or clay, over which they have naturally retrogressed during the normal geologic cycle. These layers, however, may be very close to the surface, so that, by depletion of the beach, they may quickly become exposed to wave action. The sea-walls then become unstable and may be undermined. Sea-walls are essentially defensive; they generally do little or nothing toward building up and preserving a beach. Indeed, they often tend to promote deterioration of the beach by sealing off natural sources of materials from replenishment of the strand. Their purpose is primarily to stabilize and preserve the backshore properties, but this objective is antipathetical to the geologic regimen, which seeks to establish a stable profile. Built generally near the normal high tide line, sea-walls are subjected to the severe buffeting of storm waves. Not only, then, must they be built of highly resistant materials, soundly integrated, but they must also rest firmly on sound foundations, and their bases must be well below any possible level of erosion. The instability of sea-walls is increased by the hydraulic surging effects of high storm tides acting upon porous, saturated, sub-surface layers.

The conditions just mentioned as affecting the design and maintenance of sea-walls are particularly critical with respect to some of the Massachusetts beaches, especially those within and north of the Boston Basin. Many of these are so-called "pocket beaches", essentially bayhead beaches, developed between either low, rocky headlands or projecting, partly submerged hills of relatively compact, bouldery, glacial till ("hard pan", and "boulder clay"). Because of the recent geologic history of the region, these beaches have been built variously upon bedrock, till, outwash sands and gravels, marine clays, and even peat. Moreover, this variability makes it impossible to closely predict the stratigraphy and structure of one beach from the next successive beach of the chain.

Each beach, then, becomes to some degree a coastal unit by itself, with its own distinctive personality, though the beaches may collectively bear the marks of family relationship. The first step, then, in the study of the coast for engineering projects is to determine the basic geologic facts of the coastal belt.

The natural sources of beach materials and the adequacy of such sources for future replenishment of the beaches are determined by this basic geologic study. Specifically, in this connection investigations are directed to details of coastal lithology, morphology of the deposits, rates of erosion of various materials (where indicated by geologic features and available statistical data), distribution of grain sizes in beach deposits (as indicative of redistribution by littoral currents and shore drift, sorting with respect to beach physiography, and variations in wave energy). In these areas of study geologic techniques are directly and highly contributory.

Natural replenishment of beach materials has become a critical factor for some of the Massachusetts beaches. The sealing off of supplies along the backshores and projecting headlands of till, necessitated by protection of properties, the retrogression of beaches over backshore marshes, and the loss of beach substance to the deeper offshore waters, have caused a permanent net loss. Rocky headlands contribute practically no materials, and even where till headlands are not fronted by sea-walls, the development of boulder pavements in front of these headlands somewhat retards and decreases the rate of supply by normal wave erosion. Such beaches may need to be artificially supplied, and to be protected against further erosion by such structures as offshore breakwaters and groins. It seems to be true that once sands are removed to the deeper offshore waters they are often forever lost, under the existing regimen of geologic forces. This is, of course, conditioned by such features as slope of the shore outward from the low tide line, the dimensions - height and length ratio - of the non-storm waves, and the consequent opportunity for the building of offshore bars. The Beach Erosion Board (1933) has made some investigation of these factors, and reached the conclusion that the offshore sea-bottom does not, in the absence of bars in the deeper waters, furnish a source of replenishment for the beach sands. In and north of the Boston Basin offshore bars are generally lacking, whereas along part of the South Shore, particularly within Cape Cod Bay, and off the Chatham area of Cape Cod, they are common.

For convenience the coastal belt of Massachusetts may be divided into three major segments that present contrasts in morphology, structure, and lithology. Consequently these segments present different problems in beach control. These three divisions of the coast are the North Shore, the Boston Basin, and the South Shore (including Cape Cod). (See index map, figure 1). Although these are essentially physiographic and geologic divisions, they are not to be completely separated, for some features of each are, of course, to be found in the others. In their broad aspects, however, the major differences are marked. I will consider only the geologic history, lithology, and natural sources of beach

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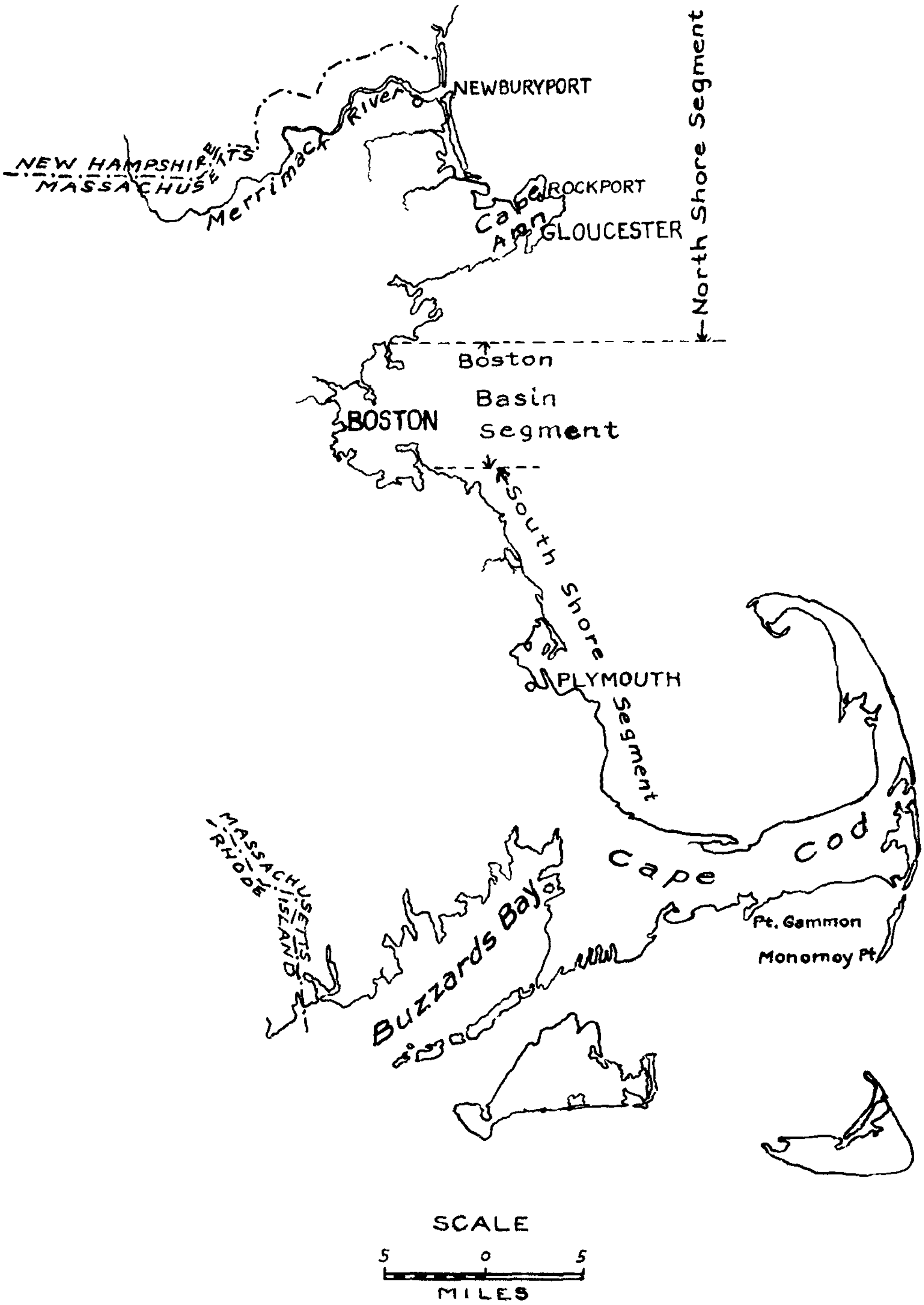

Fig. 1. Outline map of the coast of Massachusetts, indicating principal segments and features.

materials, but there are numerous other minor features that characterize individual beaches and are important to the engineer, and so need to be studied with relation to individual projects. A brief review of the geologic history will bring the beach characteristics into focus.

The coast of New England is one of submergence (Johnson, 1925), that is, the basic features have been determined primarily by a net relative depression of the coastal belt. Valleys that had been eroded in a maturely dissected upland composed chiefly of hard crystalline rocks were drowned by invasion of marine waters. A coast so developed is commonly irregular, in proportion to the degree of dissection and the magnitude of the depression, in contrast with an emerged or relatively uplifted coast, which is comparatively smooth and straight. Drowned coasts tend normally to have deeper offshore waters dotted with rocky islands; they are marked by estuarine embayments, rocky headlands, and, as in the area north of the Boston Basin, so-called "pocket beaches". If the geologic regimens are not interrupted such coasts ultimately become straightened by erosion of the headlands, and the development of sand spits, bay-mouth bars, long smooth beaches and, subsequently, offshore bars. Barrier beaches - long, straight, and smooth - form normally in the later stages, when the offshore waters become shallow enough by seaward deposition of land waste. In general, the New England shoreline is youthful; marks of the youthfulness of the Massachusetts coast, however, are somewhat more obvious in the North Shore and Boston Basin segments than in the South Shore segment (excepting the northern portion). The reasons for this are attributable to the accident of continental glaciation, which interrupted the geologic regimen of the pre-glacial coast, tilted the coastal belt differentially, and has masked or modified the rocky-coast features. For practical purposes, then, the coastal belt of Massachusetts is not to be classed quite so simply as a shoreline of submergence, for on it were superimposed the effects of continental glaciation. In the southern part especially these effects were so marked that much of the shoreline reflects the details of glacial processes and deposits more than the conventional features of a submerged coast. This has led to a great variety of forms that invest the Massachusetts coast not only with a visible charm, but also with a variety in beach structures and a complexity in local regimens. Clearly, the engineering problems are varied.

An ice sheet, such as traversed New England and came to rest far beyond the present shores, tends to abrade and round off the upland divides, and to deposit the abraded soil and rock materials over the entire area, but more thickly in the valleys. The deposits consist in part of till (ice-laid bouldery, heterogeneous, unsorted and unstratified debris) and in part of outwash deposits (gravel, sand, silt and clay, sorted and transported by the glacial melt waters) deposited broadly over the lowlands, partly or completely filling pre-glacial valleys. The sea came to rest against a glaciated land mass and began its work of erosion and redistribution of the thick, loose, glacial deposits. These deposits have been the principal sources of beach materials. It is true that rocky residuals of the pre-glacial land mass appear as prominent headland along part of the coast, but they are contributing very subordinately and slowly to the present strand in the North Shore segment, and not at all to the segment south of Plymouth and east of Buzzard's Bay. Where exposed, the rocks are mostly hard, crystalline varieties, and wave-cut terraces in them are conspicuously wanting. Hence, from an engineering standpoint, bedrock headlands are contributing practically nothing to the present beaches.

Appreciable natural replenishment of beach materials must therefore be derived from glacial deposits. The composition, distribution, and thickness of these deposits within the zone of wave action thus become an important part of the basic data to be provided by the geologist. It is in this connection that one of the major points of contrast between the three coastal segments mentioned above is brought out through a study of the recent geologic history.

The bevelled and deeply eroded pre-glacial land mass, with its sharply incised valleys and rounded divides, that underlies eastern Massachusetts slopes toward the southeast. In the general vicinity of Plymouth, about 30 miles south of the Boston Basin, it disappears completely below present sea level; southward and eastward from this point the old bedrock surface lies deeply buried beneath very thick deposits of glacial debris. The southernmost exposure of bedrock is in the vicinity of Plymouth; to the north, nearly to the southern limit of the Boston Basin, bedrock exposures are increasingly numerous, though small and low, as the old surface rises gradually to and above sea level. North of Boston Basin, the coast is truly "rock-bound". We have, then, a picture of the bedrock surface dominating the coast north of Boston, and gradually declining south of the Basin until it disappears below the zone of wave action to leave the soft and vulnerable glacial deposits unprotected and exposed to the full onslaught of the sea. Thus the coast has many aspects, and presents the engineers with a wide range of problems. The geologist can help materially in analyzing the natural forces currently at work and the probable future changes to be accomplished by them, through a study of the materials, land forms, morphology and slope of the beach, textural distribution of shore materials, and other features of the regimen.

The North Shore segment as far north as Rockport and Gloucester (the Cape Ann area) is clearly a drowned rocky coast that is only subordinately modified by glaciation. Along it, glacial deposits are everywhere present, in part as very thin deposits of till capping the rocky knolls and headlands, or as somewhat thicker, but local, deposits of outwash materials within the intervening valleys. Beds of silt and clay are irregularly and locally present, having been deposited postglacially within shallow estuaries. Small, poorly drained flat areas of marsh lands form the backlands of some of the characteristic "pocket beaches". Projecting rocky headlands afford some degree of protection to these beaches, but not enough to prevent some removal of materials to the deeper offshore waters. Sands that have been so removed seem to be permanently lost to the beaches; the headlands furnish no significant substance to them, and littoral currents for the most part cannot redistribute sands from distant sources, being unable to skirt the rocky points and thence to carry the materials landward. Where the backlands consist of low, marshy areas, or the shore properties are protected by sea-walls, as they necessarily must be in places, there is no adequate source for replenishment of beach sands by natural processes. The engineering problem then becomes one of preserving the present beaches, and preventing the seaward migration along the beach itself.

From the Cape Ann area north to the State line, the coastal strip presents some features that are in marked contrast with the rest of the North Shore segment. In this belt, bedrock exposures, though numerous, do not now form projecting headlands; rather, they appear a mile or more back of the present beaches. Extensive tidal marshlands, and groups of hills composed of glacial materials - both till and outwash - lie between the low rock-knolls and the chain of barrier beaches. This 15-mile stretch of the Massachusetts coast is, then, comparatively low and flat. Unlike the coast to the south, much of it has prograded. Nevertheless, it is entirely consistent with the interpretation of a submerged shoreline, for it represents the true ria type coastal features, in which wide and deep embayments mark the drowning of the mouths of major river valleys. Two major pre-glacial streams of eastern Massachusetts entered the sea along this part of the coast. These valleys were filled with glacial deposits and recent marine clays, and along the general site of one of the valleys the present Merrimack River flows eastward to meet the sea at Newburryport. Another valley to the south is marked by a concentration of drumlins and outwash forms. The gentle and regular seaward slope of the shore zone permitted offshore accumulations of sand and silt, and ultimately the formation of offshore bars, sand spits, and barrier beaches. The beaches have been developed and nourished partly from glacial deposits, including drumlins, but they have also been nourished from offshore sources, in part, at least, provided with materials moved southward along the coast by littoral currents. The Merrimack River is the only stream in Massachusetts that now furnishes material amounts of sediments to the offshore waters. By the same token it has necessitated corrective measures by the construction of long jetties at its mouth. It has played a constructive role in the development of the gentle seaward slope of the coast here, although, as pointed out by R. L. Nichols (Chute and Nichols, 1941) this slope was also in large measure determined by deposition of marine clays, and glacial outwash sands and gravels. Thus the geologic stratigraphy and history again furnish the key to an understanding of the processes that are operating along this unique portion of the New England coast. A detailed discussion may be found in the reference cited just above.

The Boston Basin consists of a down-faulted block bordered by higher areas of crystalline rocks both to the north and south. It embraces the major indentation that comprises Boston harbor. Containing many islands of glacial till hills (drumlins) and some small knob and islands of bedrock, Boston harbor presents a very irregular shoreline. Some of its prominent beaches have been developed in large measure from the drumlins.

Some of the drumlins have been entirely worn away; some are in different stages of removal and form projecting headlands with erosional cliffs fronting the sea. Erosion of the drumlins has left partly submerged boulder pavements marking the former extents and distribution of these land-forms. The beaches are, therefore, in part tied to drumlins, but bedrock headlands are lacking in general within this segment of the coast. Protection of shore properties has required the construction of sea-walls, in places even around the ends of the partly eroded drumlins, so that natural sources of beach sands have been removed from the regimen. In places beaches have retrogressed over back-shore marshes, and are consequently underlain by peat, clay, and other materials. Some of the beaches, also, are sand bars that tie rocky islands or till hills together as tombolos. There is thus a great variety of local shore forms within this segment, and the beach engineering problems involve various kinds of geologic data. Stratigraphy, sources of materials, and processes need to be analyzed with respect to each individual beach in order that pertinent background data may be furnished the engineer.

Along the South Shore - Cape Cod segment, where the pre-glacial bedrock surface declines below the zone of wave action, a markedly different geologic set-up prevails. Here bedrock is eliminated from consideration in shore problems. Thick glacial deposits provide the materials for extensive beaches, and materials are moved for long distances by littoral currents. For the most part there are no problems relating to adequate sources, but, rather, the problems are concerned with rapid erosion of the coast - as particularly the outer shore of Cape Cod - and the prevention of coastwise translation of beach materials. In many places short groins have been effective in locally tying down and preserving the beaches. Bay-mouth bars are prominent across some of the drowned margins of glacial outwash plains, and long sand spits project in places. Submarine profiles are varied and are particularly important in the analysis of the coastline for engineering purposes. The conditions and shore processes are too numerous and complex to epitomize adequately in this short paper, but the importance of geologic investigations can, indeed, be emphasized. For details the reader is referred to the earlier cited work by Johnson on the New England - Acadian shoreline, and to two papers by N. E. Chute (1939, 1946).

In 1938 the Commonwealth of Massachusetts, through its Department of Public Works, proposed and entered into a continuing cooperative program with the United States Geological Survey for geologic investigations in the State. The principal objective is to prepare a complete and detailed geologic map of the State by quadrangle units. It was with commendable foresight that the officials specified a broad objective, namely, that the work should provide geologic data for scientific, educational, and engineering uses, for mineral resources studies, and for general information. Thus the program is to furnish all-purpose geologic maps from which basic data can be obtained for various needs. Hence the work is not directed toward any particular technical use of geology. Necessarily the engineering application has been largely in connection with highway and bridge projects, and to studies of the occurrences and distributions of construction materials.

However, special studies of shoreline segments and certain beaches hav been made upon requests by engineers, and insofar as the fiscal limitations have permitted. These studies furnish background data for shoreline engine but make no attempt to discuss the strictly engineering aspects of the prob lems, or to recommend technical engineering procedures. So far under the cooperative program in Massachusetts we have made special studies of four l segments of the shoreline, and of six beaches. Two of the longer reports a primarily concerned with erosional effects of exceptional storms on the sou shore of Cape Cod (Chute, 1939, 1946). The third, an open-file report by N Chute, discusses the development of, and recent changes along, the South Sh between Nantasket Beach and Duxbury, and the fourth report (Chute, and Nich 1941) describes the development of the northeastern coast from Gloucester t the New Hampshire line, and important geologic changes that have recently occurred within that segment.

Perhaps the function of the Geological Survey should properly be to pr vide engineers and their technical consultants with the broader terrane dat that are pertinent to the preliminary stages of their projects, and that th are otherwise not likely to acquire. As a public agency we would not feel justified in entering the technical engineering field either as practicing consultant engineers - we have neither the qualifications nor the inclinati to do this. But the general scientific study of coastal forms and shore processes as they involve geologic principles and agents, both of the past present, is properly within our field of research. Such study in the field engineering geology, could well be expanded by the Geological Survey, and c dinated with the work of other agencies to fill in the fringe area between fields of geologic science and engineering that is apt to be neglected by t geologists and engineers engaged in local coastal projects.

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Chapter 10

ARTIFICIALLY NOURISHED AND CONSTRUCTED BEACHES

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INTRODUCTION

Construction, improvement and maintenance of beaches through the artificial deposition of sand on the shore is rapidly gaining prominance in the field of shore protection engineering. The trend toward this type of shore improvement has resulted from our changing economy, modes of transportation and recreational habits. As our mode of transportation improved and people had more time for recreation, beach resorts developed and grew to proportions typified by Atlantic City, N. J. Numerous factors control the growth of a resort of this size but undoubtedly all will agree that it is the beach which is the resorts' primary asset. This fact was recognized very early in resort development and every effort was made to preserve the beaches from the ravages of the sea. Unfortunately the science of shore protection lagged behind resort development and beaches soon became covered with a maze of structures which discouraged rather than encouraged their use. At this point something had to be done to restore the beaches to their original attractiveness. The obvious means for this improvement was to eliminate all structures as far as possible and to replace the beach material which had been removed. Was it possible that such plans could succeed? Careful study convinced a number of engineers that beach restoration employing artificial nourishment had possibilities and in some instances might be the most economical as well as best method of improvement. More importantly, there has developed a growing recognition of the fact that preventing erosion by means of protective structures is a dangerous practice, in the sense that in many cases such protection is secured at the expense of producing an ever expanding problem area. Artificial nourishment, on the other hand, benefits not only the shore upon which it is placed but adjoining shores as well. The economic merit of this type of treatment has often been difficult to evaluate because of uncertainties in prospective maintenance cost and in determination of the extent of shore which would be benefited. It is needless to say here that although the method has been employed without a complete understanding of all the factors controlling an ideal installation the results have been gratifying.

It is the purpose of this paper; first, to outline the criteria pertinent to the design of artificially nourished beaches and explain how each is derived and used; second, to present a brief history of five areas where the four types of artificial nourishment have been tried; namely the offshore dumping method, the stockpiling method, the continuous supply method, and the direct placement method; and third, to present a tabular record of a great number of artificially nourished and constructed beaches including factors relating to their placement and economic life.

DESIGN CRITERIA

At the present time although the design of artificially nourished and constructed beaches has not been firmly established on a scientific basis, advances have been made in the field of wave motion and the effect waves have on the shore which have established it on better than a rule of thumb basis. In the following paragraphs criteria for the design of artificially nourished and constructed beaches will be enumerated and their derivation and use will be explained.

The first task in approaching a design problem of this nature is to determine quantitatively the deficiency in material supply in the problem area. This is the rate of loss of beach material and is the rate at which the material supply must be increased to balance the transport capacity of littoral forces so that no net loss will occur. If there is no natural supply available, as may be the case on shores down drift from a major littoral barrier, the deficiency in supply will be equal to the full rate of littoral drift. If the problem area is part of a continuous and unobstructed sandy beach, it is likely that the deficiency will be relatively small compared with the drift rate. Comparison of surveys over a long period of time is the only accurate means of determining the rate of nourishment required to maintain stability of the shore. Since surveys in suitable detail for volumetric measurement are rarely available at problem areas, approximations computed from changes in the shore position determined by air photos or any other suitable records is often necessary. For such approximations a rule of thumb equation wherein one square foot of surface area equals one cubic yard of beach material appears to provide acceptable values on exposed seacoasts. For less exposed shores this ratio would probably result in volumetric estimates somewhat in excess of the true figure and would thus produce conservative values.

The next and equally important task is the determination of the predominant direction of littoral drift. This is most generally determined by studying the shore configuration at groins, jetties or other littoral barriers. The major accumulation of littoral material occurs on the updrift side of such barriers, however in the case of minor barriers such as short groins, seasonal variability or storm effects may obliterate the predominant trend. Care must be taken to avoid misinterpretation in such cases. Seasonal trends should be determined and evaluated where doubt exists on the basis of available evidence.

Unfortunately, or maybe fortunately, the engineer has not covered each sector of our entire shore line with structures whereby this determination of the rate of drift can be made. In the event that structures are not available on a sandy beach, or an area is to be improved that is devoid of littoral materials, another method of determining these factors must be employed. A rather long laborious method is available for use, which indicates the direction of the predominant littoral forces quite accurately, but indicates only the relative strength of the littoral forces along selected stretches of the shore.

This method of proceedure involves the use of the techniques of hindcasting wave data from synoptic weather charts to determine the wave climate over a period of years in a given area; the use of refraction diagrams to bring this wave budget into shallow water; and the use of vector diagrams to determine the resultant direction and magnitude of the wave energy which establishes the predominant direction and relative strength of the littoral movement. The predominant direction of the littoral drift is considered to coincide with the direction of the resultant of the flow of wave energy, and the relation between the strength of the littoral movement is determined to be the longshore component of the wave energy acting along its established direction toward the beach. In view of the lack of knowledge of the characteristics of the boundary conditions imposed by the surf zone it is not possible at the present time to actually relate the longshore component of the wave energy to a quantitative determination of littoral drift. In other words only the relative strengths of the littoral forces in the various related locations along a stretch of beach under study should be used.

Having established the direction and magnitude of the forces that will operate on a proposed fill the next problem to be encountered is that of selecting a suitable beach material. Unfortunately adequate criteria have not been established for evaluating the qualities of beach materials. However, a limited amount of information pertaining to the sorting of beach sands and the relation of grain size to beach slope are of value in selecting materials for artificial nourishment. When sand is deposited on a shore the waves operating in the area immediately start a sorting action on the surface layer of the fill moving the finer particles seaward leaving the coarser material shoreward of the plunge point. This sorting action continues until a layer of coarse particles compatible with the wave spectrum of the area armors the beach and renders it relatively stable. However, if the armor is broken due to a storm, the underlying material is again subjected to the sorting process. In view of this sorting process beach materials containing clay lenses or discolored particles may be used with the assurance that natural processes will clean the sand and make it an entirely suitable material for nourishment. Experience with the fills at Anaheim Bay, California, and Palm Beach, Florida, both of which contained foreign matter confirm this statement.

During the period of sorting, the beach slope is also adjusted until it becomes compatible with the grain size distribution of the sorted material. In view of this fact, a desired beach slope may be obtained by randomly placing material of a gradation that will assume the desired slope after sorting and slope adjustment. The selection of a material of the proper gradation to produce the desired slope as far as is known at the present time can only be determined by analyzing the sand taken from a beach in the surrounding area which has a similar orientation and is acted upon by the same wave forces. Sand selected for artificial nourishment should ideally contain the same gradation of materials as those found on the beach to be nourished if the original beach slope is to be maintained.

Material of coarser characteristics may be expected to produce a steeper than normal beach. Material finer than that occupying the natural beach will, when exposed on the surface, move seaward to a depth compatible with its size. Almost any source of borrow near the shore will produce some material of proper beach size. Since the source of artificial nourishment will control the cost to a major degree, evaluation of material characteristics is an important factor in economic design. At present such evaluation must be made largely on a basis of experience at other localities.

The beach crest height will be established ultimately by natural forces, that is, the cyclic changes in water level and the wave pattern. The foreshore and nearshore slopes will affect wave behavior and thus influence the natural beach crest height. If the beach fill is placed to an elevation lower than the natural crest height a ridge will subsequently develop along the crest. Concurrent high water stage and high waves will overtop the crest and cause ponding and temporary flooding of the backshore. Such flooding, if undersirable, may be avoided by fixing the berm height slightly above the natural beach crest height. If there is an existing beach at the site, the natural crest height can be determined therefrom. Otherwise determination must be made on a basis of comparison with other sites possessing similar exposure characteristics and beach material. There is at present no acceptable theoretical basis for predicting beach crest height.

Criteria for specifying berm width depends upon a number of factors. If the purpose of the fill is to restore an eroded beach damaged by a major storm, where inadequate natural nourishment is not a factor in the problem, the width may be determined by the protective width which experience has demonstrated to be required. Where the beach fill is to serve as a stockpile, the berm width should be sufficient to provide for expected recession during the intervals between artificial replenishment. It is generally considered that the toe of fill of a stockpile beach should not extend to such depth that transport of any material forming the surface of the fill would be retarded. There are no firm specifications for this limiting depth at present but available data indicate, that depths of twenty feet below low water datum on seacoasts and twelve feet on the Great Lakes may be used safely. It is obvious that the initial slope of any beach fill must be steeper than that of the natural shore area upon which it is placed. Subsequent behavior of the slope depends principally upon the characteristics of the fill material. Fills composed of material coarser than that found on the native beach will maintain a steeper than normal slope. Finer material tends to form a flatter slope. In ordinary practice the initial fill slope is designed paralled to the local or comparable natural beach slope above low water datum, and slopes of 1:20 to 1:30 from low water datum to intersection with the existing bottom. It is unnecessary to artificially grade beach slopes below the berm crest, for they will be naturally shaped by wave action.

The length of a stockpile beach may vary greatly depending upon local conditions. Lengths from a few hundred feet to a mile have been employed successfully. Since the updrift end of a stockpile beach will be depleted first, long stockpiles are usually most suitable where a bulkhead or seawall exists to protect the backshore as erosion progresses along the stockpile.

The foregoing general discussion of the derivation and use of the basic criteria pertaining to the artificial placement of sand to maintain, rehabilitate or construct a beach clearly indicates the lack of the present knowledge and consequently presents a challenge to investigators to direct their work toward this phase of shore protection work. The principal factors which appear to warrant detailed study in order to establish more rigorous design criteria are the relations between the characteristics of beach and nearshore materials and their modes of transport; the relations between beach materials, exposure, and the resulting geometry of naturally formed beaches; and more accurate methods of determining the deficiency in material supply on an eroding beach. In the present state of knowledge laboratory experimentation may be expected to contribute only to a limited degree to the solution of these problems. It is believed that emphasis must be placed on field investigation for this purpose, particularly in the form of follow-up studies of artificial beach fills.

TYPES OF ARTIFICIALLY NOURISHED AND CONSTRUCTED BEACHES

OFFSHORE DEPOSIT METHOD

This method of beach nourishment is constantly coming to the mind of the shore protection engineer since a large supply of beach material could be made available at comparatively low cost in connection with hopper dredge operations in coastal harbors.

A test of this type of nourishment was made in 1948 and 1949 by the Beach Erosion Board and the New York District, Corps of Engineers, Department of the Army at Long Branch, N. J. (reference 1) The city of Long Branch is located near the northern tip of New Jersey adjacent to the entrance to New York harbor. (Figure 1) It lies on a slight rise in the surrounding terrain, which slopes seaward to an elevation of 20 feet and terminates at the shore at the crest of a timber bulkhead retaining Ocean Avenue. The beach fronting the bulkhead is relatively steep and narrow and is intersected by numerous heavy rubble mound groins.

The history of the Long Branch area has been one of progressive erosion caused by the stabilization of updrift areas which formerly eroded and supplied abundant littoral material to down drift areas.

The purpose of this test was to determine the feasibility of restoring an adequate littoral drift to nourish the shore by employing natural forces to move material, dumped in relatively deep water, shoreward toward the beach.

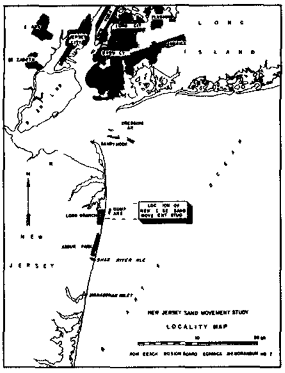

Fig. 1. Locality map, Long Branch, N.J.

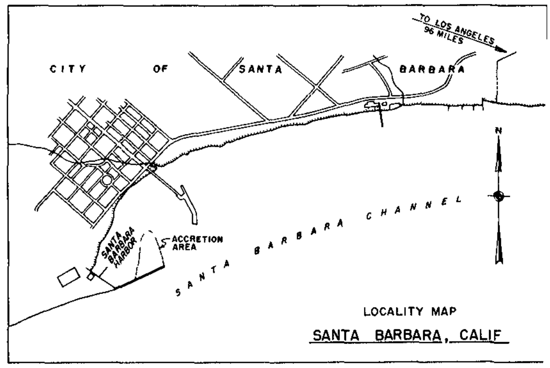

Fig. 3. Locality map, Santa Barbara, Cal

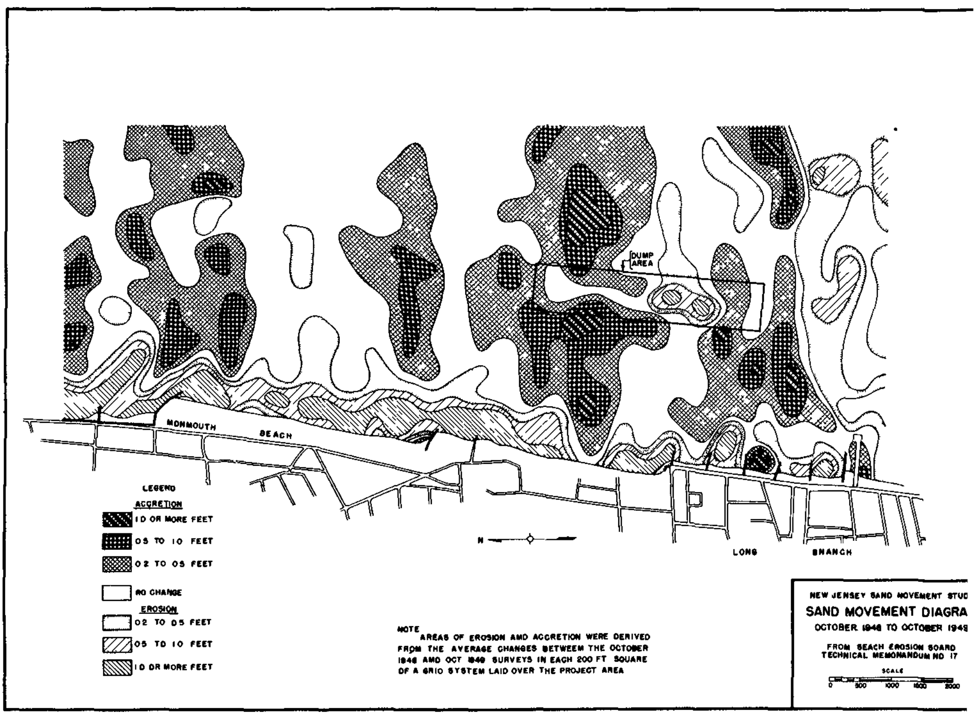

Fig. 2. Sand movement diagram, Long Branch, N.J.

The material dredged from New York Harbor entrance channels was placed in a ridge about 7 feet high, 3,700 feet long and 750 feet wide, lying about ½ mile from shore in a depth of 38 feet below mean low water, with its southerly limit on an east west line about 1500 feet north of the Long Branch Pier. Dumping at the site amounted to a total of 602,000 cu. yds. of sand. (Figure 1)

During the entire period of study, oceanographic forces effecting sand movement were recorded and the sand movement was traced by periodic hydrographic surveys covering the area from the Long Branch Fishing Pier northward to Monmouth Beach Coast Guard Station and from the bulkhead line seaward 6,000 feet to about the 42 foot depth contour (mean low water). An effort to trace sand movement through dissimilar minerals in the beach and dumped sand failed. At this point one may question the suitability of the Long Branch site for a study of this nature. Oceanographic and hydrographic data collected at the site proved its suitability since natural forces were found which were capable of moving material over the ocean floor in 35 to 40 feet of water and along the beach.

The result of the sand movement during the period October 1948 after all dumping had been completed to October 1949 are depicted by net bottom changes and are shown on Figure 2. The bottom changes show accretion to be general over the offshore area including the mound. An area of localized erosion developed near the center of the mound and erosion occurred over the shoal at the southern limit of the study area. Nearshore erosion has been extensive over the year. The general accretion over the mound coupled with the extensive erosion along the shore indicates that the deposited material, during the period of observation, has not benefited the beach. While observations over a longer period may indicate some benefit, it may be concluded from present evidence that this method will not provide nourishment at a suitable rate to justify its general use.

The conclusions reached in this study confirm the findings of two similar studies, one made at Santa Barbara, California where 202,000 cu. yds. of sand were deposited in 20 feet of water (mean lower low water) in September 1935, and the other at Atlantic City, New Jersey where 3,554,000 cu. yds. of sand were deposited off the beach in 18 to 20 feet of water (mean low water) during the period April 1935 - September 1943.

Although the results of this test to artificially nourish the beach at Long Branch, New Jersey, were negative, it is felt that they have a place in this paper to guide future work along these lines.

STOCKPILE METHOD

Probably the first shore protection project designed specifically for employment of this method was that developed at Santa Barbara, California (Figure 3). This project has been in successful operation since 1938. Details of the plan are contained in references 2 and 3 and only a summary will be presented herein.

The problem at Santa Barbara was created by construction of a breakwater, completed in 1929, which effectively blocked the movement of littoral drift. Material accumulated on the updrift side of the breakwater at a rate in the order of 300,000 cubic yards a year. By 1934 the impounding capacity above the breakwater was reached, and the zone of entrapment shifted to the protected waters within the harbor.

Meanwhile beaches downdrift from the harbor, being deprived of normal nourishment, were progressively eroding. By 1938 the erosion area had denuded the down drift beaches for a distance of ten miles, to a location where a large natural sand deposit served to maintain shores beyond. Offshore deposit of sand removed from the harbor by hopper dredge in 1935, described earlier, failed to aid the shore. Damages mounted and hastily built shore protection structures provided little relief.

In 1938 a cooperative project was developed on recommendation of the Beach Erosion Board providing for establishing a stockpile beach fill along 4000 feet of shore down drift from the harbor, to be initially filled and periodically maintained with material dredged from the harbor. The first fill was completed in July 1938 and replenishment has been accomplished at two or three year intervals since that date. The seventh repetitive nourishment operation is in progress at this time (October 1952).

By 1945, seven years after initiation of the project, stable conditions had been restored over the entire ten miles of previously eroding beach. No additional shore protection measures have been required since that date. The average rate of artificial nourishment in round figures, has been 300,000 cubic yards a year. The average cost is 21 cents a cubic yard. Harbor maintenance as well as shore protection is accomplished, and under the terms of the project the United States pays the cost of the former by the cheapest method (hopper dredging) at an established price of 13 cents a cubic yard. The work is accomplished with conventional pipe line dredging plant and equipment. Local interests contribute the added cost of depositing the material on the stockpile beach, an average of 8 cents a cubic yard or $24,000 a year. Considering the length of frontage receiving protection in this project, the average annual cost is about 50 cents a linear foot. This is a minor fraction of the cost experienced where defensive works are employed for shore protection.

A more recent example of stockpiling sand on a beach to be distributed along the down drift shore by the natural forces is the project undertaken several years ago at Palm Beach, Florida.

Palm Beach is located on the coastal lowlands of the east coast of Florida about 300 miles south of Jacksonville and 70 miles north of

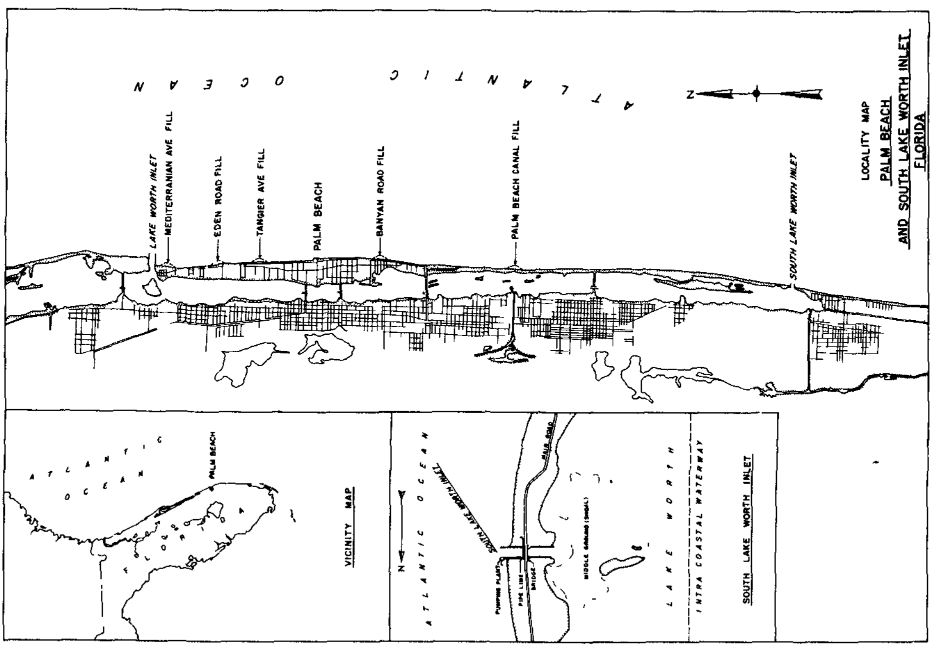

Fig. 4. Locality map, Palm Beach and South Lake Worth Inlet, Fla.

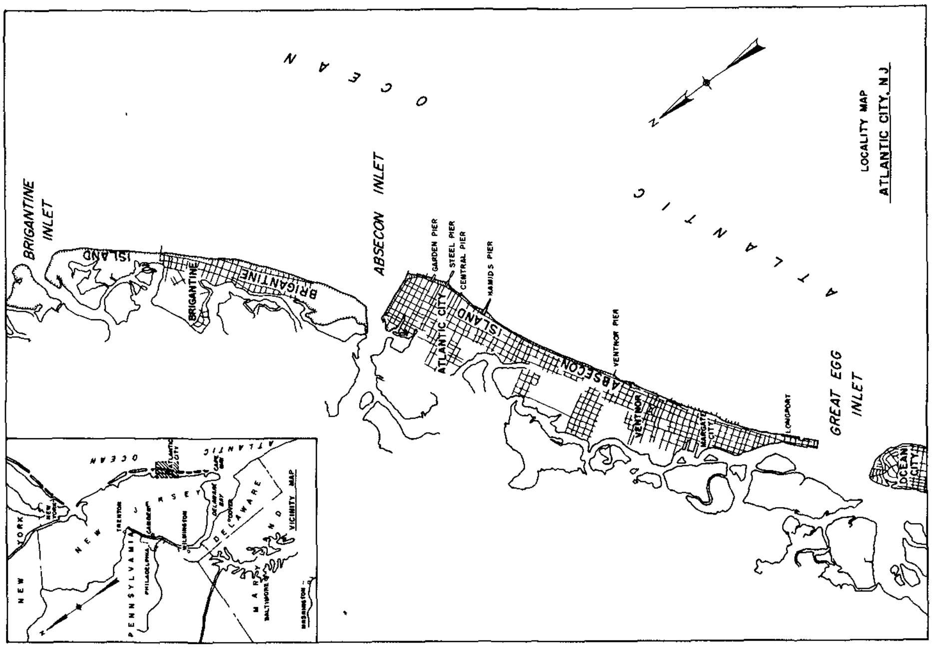

Fig. 5. Locality map, Atlantic City, N.J.

Miami Beach. (Figure 4) The barrier beach on which the town has been built separates Lake Worth from the Atlantic Ocean and is breached by two inlets, Lake Worth Inlet and South Lake Worth Inlet, about 15.5 miles apart. The barrier is composed principally of sand, part of which is artificial fill over former marsh areas. There are occasional outcroppings of coquina on the barrier and in the offshore area.

Lake Worth Inlet was dredged through the barrier and two protective jetties were constructed between 1918 and 1925. The construction of the jetties have caused changes in the adjacent shore lines similar to those at a number of other inlets along the east coast of Florida where jetties have been constructed; namely, accretion north of the north jetty and erosion south of the south jetty. An accurate estimate of the rate at which the littoral drift has been impounded by the north jetty cannot be made from available historical records but a number of rough estimates have been made utilizing available information. These estimates although rough, indicate the limits of the range between which the true value probably lies. They indicate that during the 14 years period immediately following completion of the inlet and jetties, material was impounded at a rate averaging 150,000 to 225,000 cubic yards per year and that during the next seven years the rate approximated 130,000 cubic yards per year.

The removal of this quantity of material from the littoral stream which formerly nourished the Palm Beach shores has resulted in continuous erosion. The rate of erosion has been retarded by the construction of a fairly uniformly spaced field of groins but in general the groins have not maintained as wide a beach as desired, primarily because of the lack of sufficient littoral drift.

Studies made by the Beach Erosion Board in cooperation with the Port of Palm Beach District to develop a plan or plans for the rehabilitation and future protection of Palm Beach resulted in the conclusion that because of the absence of an assured natural supply of beach material an artificial supply must be furnished. (reference 4) It was also concluded that the best method of nourishing this shore would be to pump sand from Lake Worth and place it in stockpiles along the beach. The decision to use this method of nourishment was due in part to a satisfactory test of stockpile nourishment made on the beach immediately south of Lake Worth Inlet in 1944.

The recommendations made by the Board were accepted by the cooperating agency and four stockpiles of sand were placed on the beach between May and November 1948. An additional stockpile of 100,000 cu. yds. of sand was placed on the beach opposite the West Palm Beach Canal by Palm Beach County in 1949. The quantity of material placed in each of the stockpiles together with previous and subsequent placements near the northern end of the beach and the locations of the piles are shown on Figure IV and in the following table.

Location	Date of Placement		No.Cu.Yds.	Cost per Cu.Yd.
Mediterranean Ave.	Aug.	1944	300,000	35.0 ¢
	May-Nov.	1948	215,690	32.2 ¢
	July	1949	380,000	
Eden Road	May-Nov.	1948	630,600	19.3 ¢
Tangier Ave.	May-Nov.	1948	454,640	19.3 ¢
Banyan Road	May-Nov.	1948	1,035,000	19.3 ¢
West Palm Beach Canal		1949	100,000	

The results obtained through the use of stockpiles to nourish the beach in the Palm Beach area can best be described by the following statements made by Mr. Norman C. Schmid, Engineer, Town of Palm Beach. "It is my opinion that artificial sand supply is the best method of beach protection that we have found in Palm Beach. The only trouble is that we have only supplied the beach with two and one half million yards and it is estimated that the project would require six million in order to bring the beach line to the 1928 location." Mr. Schmid further states that past experience shows that, "The northernmost stockpile should be replenished yearly, the others to the south every two or three years depending upon storm conditions". He concludes " - - - that the sand has moved as expected, also that the experiment even to the layman's eye has proven quite successful".

CONTINUOUS NOURISHMENT METHOD

One of the best examples of continuous nourishment to a beach down drift from an inlet is the sand bypassing plant at South Lake Worth Inlet, Florida. The factors pertinent to the installation of and the results obtained with this bypassing plant were thoroughly covered by Mr. Joseph M. Caldwell in the first Coastal Engineering Conference but since it is the intent of the writer to make this paper as complete as possible in the field of artificial nourishment the highlights of this installation will be briefly reviewed.

South Lake Worth Inlet is located on the east coast of Florida near the southern limit of Lake Worth which separates the mainland from the sand barrier on which the town of Palm Beach is located. (Figure 4)

This inlet was dredged through the barrier in 1927 by the South Lake Worth Inlet District to create a circulation of water in the southern end of the lake to relieve the stagnant condition of the waters. The inlet was fixed by two short jetties about 250 feet long. Due to the abundant littoral drift from north to south in this area the littoral reservoir formed by the north jetty was quickly filled and sand was carried around its outer end into the inlet where it dropped out of suspension forming a middle ground shoal.

Concurrently with the filling of the impounding area behind the north jetty and the formation of the shoal, the beach south of the inlet eroded. Property owners faced with the loss of valuable land and homes constructed numerous protective structures but due to the impounding of the natural supply by the inlet, these structures did not help in holding or building a protective beach. The failure of the structures to protect the area clearly indicated the necessity of rebuilding the beach as a protective barrier through the restoration of the littoral drift in the area. This was done by establishing a pumping plant on the north jetty to bypass the sand across the inlet to the eroding shore. The distribution down beach of this material was left to the action of natural forces. This method had the added advantage of reducing the sand available to be carried into the inlet to be deposited on the middle ground shoal.

The pumping plant was not designed to bypass the entire quantity of littoral drift but rather to supply the quantity of material required to restore the beaches to the south. During the first five years of operation prior to World War II about 250,000 cu. yds. of sand were supplied to the beach. The benefit derived from this operation was felt almost immediately and at the end of the five year period the beach south of the inlet was entirely restored. During this period shoaling decreased over the middle ground.

The cost of moving the sand including operation, maintenance, and depreciation was about 9 cents per cubic yard. Based on current prices the figure would still be well under the 19.3 cents to 35.0 cents per cubic yard cost of the stockpile nourishment placed on Palm Beach from Lake Worth.

It is recognized that although the sand has been moved economically with a fixed plant at South Lake Worth Inlet periodic nourishment using a floating plant may be more economical at other littoral barriers.

DIRECT PLACEMENT METHOD

It differs from the stockpile method in that the fill is completed at one time over the entire shore to be protected. In effect it may subsequently take the form of a stockpile project since it will serve as a supply source for the down drift shore, and future maintenance may be accomplished by artificial nourishment of those areas which first demonstrate supply deficiency by erosion.

This type of beach rehabilitation was used at Atlantic City, New Jersey in 1948 to quickly restore the ocean beach which was eroded to a point where it furnished little protection during fall and winter storms to the boardwalk and valuable real estate investments. (reference 5)

Atlantic City is located on the coast of New Jersey about 45 miles northeast of Cape May, the southern tip of the State at the entrance to Delaware Bay. (Figure 5) It comprises nearly one-half of the length of the barrier beach known as Absecon Island. Absecon Inlet is the northeastern boundary of the City and Island.

Because of its location near extensively developed and densely populated urban areas, being about 60 miles from Philadelphia and 125 miles from New York City, it has rapidly become the most popular resort of its kind in the country.

The ocean beach is generally wide and flat; supplied with material transported southward along Brigantine Island. The volume of sand moving along this shore cannot be accurately determined but dredging figures indicate that it may be about 400,000 cubic yards per year. The part of this quantity moved onto the Atlantic City beach by natural forces is not known. Studies show that the beach remained relatively stable prior to 1940 and then started to erode progressively for a distance of about 6,000 feet southwest of the inlet. In view of the natural condition extant in the area and the immediate need for a protective beach southwest of the inlet, the State of New Jersey and the City replenished the beaches with sand moved by hydraulic dredge and pipe line from the point of Brigantine Island across the inlet. Approximately 700,000 cubic yards of sand were deposited on the beach from the Oriental Avenue Jetty to a point about midway between Central and Hamid's Piers during the summer of 1948. This material was placed on the beach over its 6,000 foot length at a cost of 77 cents per cubic yard.

Immediately prior to placing the fill a stone jetty was constructed on the south side of the inlet to divert the channel eastward away from the beach.

Subsequent to placing the artificial fill, an existing groin was repaired and five others were constructed to retard the loss of sand from the beach. Replenishment of the material placed on the beach has not been made but will be made when necessary.

The results obtained through the direct placement of sand to the beach at Atlantic City has been as successful as the studies had indicated. Observations made at various intervals following the period of beach slope adjustment show the beach to be relatively stable. It is too early to determine maintenance requirements and costs, but indications to date are that maintenance by periodic nourishment will be both feasible and economically preferable.

In summary, it is believed that artificial nourishment is firmly established as a practicable and economic means of shore protection which must be considered and evaluated in comparison with alternative measures in the study of any erosion problem. The long term benefit of this method of protection with respect to very substantial lengths of shore is an important aspect to be considered. Extensive additional research is needed to establish proper design criteria and a more accurate basis for economic analysis of this method.

ARTIFICIALLY NOURISHED BEACHES IN THE UNITED STATES

The purpose of this section of the paper is to assemble in one document all information available in the records of the Beach Erosion Board including published references 6 to 12 pertaining to those beach areas of the United States which have been artificially nourished or constructed. The data is presented in three parts in tabular form; the first outlines basic information on the beach required for the design of its nourishment; the second outlines information pertaining to the material available for the nourishment; and the third outlines information pertaining to the stabilized beach. Although an effort has been made to include all of the known artificially nourished beaches in the United States in the table there are undoubtedly many that have been overlooked. In several of the cases listed the purpose of the beach fill was not shore nourishment but simply selection of a convenient disposal area for dredged material. Those have been included for possible future use of the data presented(see Appendix).

ACKNOWLEDGEMENT

The author is indebted to Mr. R. O. Eaton, Chief Technical Adviser to the President of the Beach Erosion Board for many helpful suggestions made during the preparation of this paper and to Mr. G. P. Magill of the Boards' staff who assisted in searching the literature and records.

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APPENDIX
TABULATED DATA ON ARTIFICIALLY NOURISHED AND CONSTRUCTED BEACHES

Table 1

ARTIFICIALLY NOURISHED AND CONSTRUCTED BEACHES

DESCRIPTION OF NATURAL CONDITIONS									
Coastal area	Pacific Ocean	Pacific Ocean	Pacific Ocean	Pacific Ocean	Pacific Ocean	Pacific Ocean	Pacific Ocean	Pacific Ocean	Pacific Ocean
Name	Morro Bay Beach Calif	Santa Barbara Calif	Santa Barbara Calif	Santa Barbara Calif	Santa Barbara Calif	Santa Barbara Calif	Santa Barbara Calif	Santa Barbara Calif	Santa Barbara Calif
Length of Protected area		10 0 miles	10 0 miles	10 0 miles	10 0 miles	10 0 miles	10 0 miles	10 0 miles	10 0 miles
Exposure	S thru W to NW Clear	SE thru S to SW Channel Islands to South	SE thru S to SW Channel Islands to South	SE thru S to SW Channel Islands to South	SE thru S to SW Channel Islands to South	SE thru S to SW Channel Islands to South	SE thru S to SW Channel Islands to South	SE thru S to SW Channel Islands to South	SE thru S to SW Channel Islands to South
Slope - Foreshore / Nearshore	1 on 10 1 on 15 - 1 on 100	1 on 15 1 on 40 - 1 on 75	1 on 15 1 on 40 - 1 on 75	1 on 15 1 on 40 - 1 on 75	1 on 15 1 on 40 - 1 on 75	1 on 15 1 on 40 - 1 on 75	1 on 15 1 on 40 - 1 on 75	1 on 15 1 on 40 - 1 on 75	1 on 15 1 on 40 - 1 on 75
Beach Material Characteristic	Coarse to fine sand	Medium to fine sand	Medium to fine sand	Medium to fine sand	Medium to fine sand	Medium to fine sand	Medium to fine sand	Medium to fine sand	Medium to fine sand
Rate of Littoral Transport per year		300 000 cu yds	300 000 cu yds	300 000 cu yds	300 000 cu yds	300 000 cu yds	300 000 cu yds	300 000 cu yds	300 000 cu yds
Dir of Littoral - Winter / Transport - Summer	South South	East East	East East	East East	East East	East East	East East	East East	East East
MATERIAL AVAILABLE FOR NOURISHMENT									
Location	Morro Bay Entrance Channel	Harbor	Harbor	Harbor	Harbor	Harbor	Harbor	Harbor	Harbor
Distance from Fill	0 1 to [illegible] 0 miles	2 0 miles	0 1 to 2 0 miles	0 1 to 2 0 miles	0 1 to 2 0 miles	0 1 to 2 0 miles	0 1 to 2 0 miles	0 1 to 2 0 miles	0 1 to 2 0 miles
Character of Material	Coarse to fine sand	Medium to fine sand and silt	Medium to fine sand and silt	Medium to fine sand and silt	Medium to fine sand and silt	Medium to fine sand and silt	Medium to fine sand and silt	Medium to fine sand and silt	Medium to fine sand and silt
Quantity Available	Ample	Ample	Ample	Ample	Ample	Ample	Ample	Ample	Ample
NOURISHMENT ACCOMPLISHED									
Type of Nourishment	Direct placement with hydraulic dredge	Offshore placement by a hopper dredge in 20' water	Stockpile by hydraulic dredge	Stockpile by hydraulic dredge	Stockpile by hydraulic dredge	Stockpile by hydraulic dredge	Stockpile by hydraulic dredge	Stockpile by hydraulic dredge	Stockpile by hydraulic dredge
Primary Purpose	Disposal of dredged material	Beach restoration	Shore nourishment	Shore nourishment	Shore nourishment	Shore nourishment	Shore nourishment	Shore nourishment	Shore nourishment
Slope of - (Before Adj) / Foreshore (After Adj)	1 on 5 1 on 10		1 on 10 1 on 15	1 on 10 1 on 15	1 on 10 1 on 15	1 on 10 1 on 15	1 on 10 1 on 15	1 on 10 1 on 15
Dimensions of Fill	0 4 Mi x 300' South 0 3 Mi x 500' North	0 4 Mi x 200'	[illegible] 0 Mi x 100'	1 0 Mi x 200'	1 0 Mi x 200'	1 0 Mi x 200'	1 0 Mi x 200	1 0 Mi x 200'
Total No of Cu Yds	[illegible]0 000	202 000	606 427	697 719	607 110	717 773	642 977	[illegible] 152
Cost per Cu Yd	[illegible] cents	13 cents	21 cents	17 5 cents	21 [illegible] cents	23 7 cents	17 cents	19 2 cents
Date Placed	[illegible] - 1943	September 1935	Mar-June 1938	June-July 1940	July-Aug 1942	June-Sept 1945	May-June 1947	May-June 1949
Estimated Life of Fill									

Table 2

ARTIFICIALLY NOURISHED AND CONSTRUCTED BEACHES								
DESCRIPTION OF NATURAL CONDITIONS								
Coastal Area	Pacific Ocean	Pacific Ocean	Pacific Ocean	Pacific Ocean	Pacific Ocean	Pacific Ocean	Pacific Ocean	Pacific Ocean
Name	Port Hueneme Calif	Port Hueneme Calif	Santa Monica Calif	Venice Calif	El Segundo Calif	El Segundo to Ocean Park Calif	Redondo Beach Calif	Cabrillo Beach Calif
Length of Problem Area		3 0 miles	13 0 miles	10 5 miles	6 5 miles	11 0 miles	1 0 mile	0 7 mile
Exposure	SE thru S to W, Channel Islands to SW Offshore Islands to S	SE thru S to W Channel Islands to SW Offshore Islands to S	S thru SW to W Offshore Islands to S	S thru SW to W Offshore Islands to S	SW to W Offshore Islands to S	SW to W Offshore Islands to S	SW to W Offshore Islands to S	SE thru S to SW Offshore Islands to S
Slope - Foreshore / Nearshore	1 on 15 / 1 on 80	1 on 15 / 1 on 80	1 on 30 / 1 on 240	1 on 30 / 1 on 210	1 on 20 / 1 on 230	1 on 20 / 1 on 230	1 on 10 / 1 on 45	1 on 25 / 1 on 100 - 1 on 150
Beach Material Characteristic	Medium to fine sand	Medium to fine sand	Medium to fine sand	Medium to fine sand	Medium to fine sand	Medium to fine sand	Medium to fine sand	Fine sand clay and silt
Rate of Littoral Transport per year	1 000 000 cu yds	1 000 000 cu yds	165,000 cu yds	165 000 cu yds	165 000 cu yds	165,000 cu yds		
Dir of Littoral-Winter / Transport Summer	South / South	South / South	Southeast / Southeast	Southeast / Southeast	Southeast / Southeast	Southeast / Southeast		Southeast / Southeast
MATERIAL AVAILABLE FOR NOURISHMENT								
Location	Harbor	Harbor	Harbor	Sand Dunes	Sand Dunes	Sand Dunes	Sand Dunes	Los Angeles Outer Harbor
Distance from Fill	0 2 to 1 0 mile	0 2 to 1 0 mile	0 06 to 0 5 mile	5 0 to 5 0 miles	2 0 miles	1 0 to 6 5 miles	3 0 miles	1 0 to 4 0 miles
Character of Material	Medium to fine sand & silt (Harbor spoil)	Medium to fine sand & silt (Harbor spoil)	Medium to fine sand with silt	Medium to fine sand	Medium to fine sand	Medium to fine sand	Medium to fine sand	Fine sand clay and silt
Quantity Available	Ample	Ample	Ample	Ample	Ample	Ample	Ample	Ample
NOURISHMENT ACCOMPLISHED								
Type of Nourishment	Direct placement with hydraulic dredge	Direct placement with hydraulic dredge	Direct placement and stockpile by hydraulic dredge	Direct placement by trucks	Direct placement sluicing from dunes	Direct placement by hydraulic means with eductor & dredge pumps	Direct placement by trucks	Direct placement with hydraulic dredge
Primary Purpose	Disposal of dredged material	Beach restoration and disposal of material	Shore nourishment	Beach restoration	Disposal of material and Beach restoration	Disposal of material and beach widening	Beach restoration	Beach restoration
Slope of - (Before Adj / Foreshore (After Adj	1 on 25 / 1 on 15	1 on 25 / 1 on 15	1 on 10 / 1 on 20	1 on 3 / 1 on 20	1 on 10 / 1 on 14	1 on 6 / 1 on 15	1 on 3 - 1 on 5 / 1 on 10	1 on 60 / 1 on 30
Dimensions of Fill	0 8 Mi x 440'	0 3 Mi x 200'	1 0 Mi x 300'	0 6 Mi x 75'	1 8 Mi x 200'	5 5 Mi x 600'	0 5 Mi x 50	0 2 Mi x 200
Total No of Cu Yds	1 445 000	175 000	1 000 000	150 000	1,800,000	14 000 000	47,000	500 000
Cost per Cu Yd	20 cents	20 cents	25 cents	67 cents		26 5 cents	$1 00	20 cents
Date Placed	1939	1943	1950	1945	1938	1948	1947	1927
Estimated Life of Fill			7 to 10 years	10 years		50 years		

Table 3

ARTIFICIALLY NOURISHED AND CONSTRUCTED BEACHES								
DESCRIPTION OF NATURAL CONDITIONS								
Coastal Area	Pacific Ocean	Pacific Ocean	Pacific Ocean	Pacific Ocean	Pacific Ocean	Pacific Ocean	Pacific Ocean	Pacific Ocean
Name	Cabrillo Beach Calif	Long Beach Calif	Anaheim Bay Calif	Anaheim Bay Calif	Newport Bay Calif	Mission Beach Calif	Ocean Beach Calif	Ocean Beach Calif
Length of Problem Area	0 7 mile	6 0 miles	15 5 miles	15 5 miles	5 0 miles		1 0 mile	1 0 mile
Exposure	SE thru S to SW Offshore Islands to S	Sheltered by Los Angeles - Long Beach breakwater	SE thru S to W Offshore Islands to SW	SE thru S to W Offshore Islands to SW	SE thru S to W Offshore Islands to SW	SW thru W to NW Channel Islands to W	SW thru W to NW Channel Islands to W	SW thru W to NW Channel Islands to W
Slope - Foreshore / Nearshore	1 on 25 / 1 on 100 - 1 on 150	1 on 25 / 1 on 150	1 on 7 / 1 on 70	1 on 7 / 1 on 70	1 on 25 / 1 on 35 - 1 on 75	1 on 7 / 1 on 50	1 on 7 / 1 on 50	1 on 7 / 1 on 50
Beach Material Characteristic	Fine sand clay and silt	Medium to fine sand	Fine to Med sand with small % of silt & clay	Fine to Med sand with small % of silt & clay	Coarse to fine sand	Medium to fine sand	Medium to fine sand	Medium to fine sand
Rate of Littoral Transport per year			300,000 cu yds	300,000 cu yds				
Dir of Littoral-Winter / Transport Summer	Southeast / Southeast	Nil / Nil	South - Predominate / North	South - Predominate / North	SE - Predominate / Northwest	South / South	South / South	South / South
MATERIAL AVAILABLE FOR NOURISHMENT								
Location	Los Angeles Outer Harbor	South of Los Angeles River	Anaheim Bay behind the Barrier Beach	Anaheim Bay behind the Barrier Beach	Newport Bay	Mission Bay	Mission Bay	Mission Bay
Distance from Fill	1 0 to 4 0 miles	5 0 miles	0 06 to 0 6 mile	0 06 to 0 6 mile	1 0 to 2 0 miles	2 0 to 3 0 miles	2 0 to 3 0 miles	2 0 to 3 0 miles
Character of Material	Fine sand clay and silt	Medium to fine sand	Fine to medium sand silt and clay	Fine to medium sand silt and clay	Coarse to fine sand and silt	Medium to fine sand	Medium to fine sand	Medium to fine sand
Quantity Available	Ample	Ample after each flood	Ample	Ample	Ample	Ample	Ample	Ample
NOURISHMENT ACCOMPLISHED								
Type of Nourishment	Direct placement with hydraulic dredge	Direct placement with hydraulic dredge	Direct placement and stockpile with hydraulic dredge	Direct placement and stockpile with hydraulic dredge	Direct placement with hydraulic dredge	Direct placement with hydraulic dredge	Direct placement with hydraulic dredge	Direct placement with hydraulic dredge
Primary Purpose	Beach restoration	Beach restoration	Shore nourishment	Shore nourishment	Disposal of material and beach restoration	Disposal of dredged material	Beach restoration	Beach restoration
Slope of - (Before Adj / Foreshore (After Adj	1 on 60 / 1 on 30	1 on 10 / 1 on 20	1 on 7 / 1 on 10	1 on 7 / 1 on 10	1 on 10 / 1 on 15	1 on 5 / 1 on 7	1 on 5 / 1 on 7	1 on 5 / 1 on 7
Dimensions of Fill	0 4 Mi x 500'	4 5 Mi x 300'		0 5 Mi x 1000' Wedge Type	2 0 Mi x 300'	1 0 Mi x 100'	0 8 Mi x 500'	0 8 Mi x 500'
Total No of Cu Yds	2,556,000	[illegible] 000	20 000	1 097 000	7,000 000	685,000	64 400	67,000
Cost per Cu Yd	40 cents	14 cents		23 cents	20 cents	21 cents	17 cents	17 cents
Date Placed	1943	1942 and 1940	1944	Oct 1947 to Jan 1948	1934 - 1935	September 1948	1949	May 1950
Estimated Life of Fill				3 to 5 years				

Table 4

ARTIFICIALLY NOURISHED AND CONSTRUCTED BEACHES								
DESCRIPTION OF NATURAL CONDITIONS								
Coastal Area	Pacific Ocean	Pacific Ocean	Pacific Ocean	Atlantic Ocean	Atlantic Ocean	Atlantic Ocean	Atlantic Ocean	Atlantic Ocean
Name	Coronado Beach Calif (North Island)	Coronado Beach Calif (Silver Strand)	Waikiki Beach Honolulu T H	Hampton Beach N H	Misquamicut Beach R I	West Haven Conn	Oak Neck & Center Is Causeway L I N Y	Orchard Beach N Y
Length of Problem Area	2 0 miles		2 0 miles	1 1 miles	0 4 mile	1 6 miles	2 1 miles	0 4 mile
Exposure	S to SW Point Loma to W Los Coronados Islands to S	S thru SW to W Los Coronados Islands to S	S thru SW to W Clear	N to SE Clear	E to SW Sheltered to Block Island Sound	SE thru S to SW sheltered to L I Sound	NE thru E to SE sheltered to L I Sound	N thru SE to S sheltered to L I Sound
Slope - Foreshore / Nearshore	1 on 20 / 1 on 75 - 1 on 200	1 on 20 / 1 on 75 - 1 on 200	1 on 7 - 1 on 10 / 1 on 70 - 1 on 160	1 on 7 / 1 on 60	1 on 5 / 1 on 20 - 1 on 50	1 on 7 / 1 on 300	1 on 5 / 1 on 90	1 on 5 / 1 on 60
Beach Material Characteristic	Medium to coarse sand and shell	Medium to fine sand and shell	Coral sand and shell	Sand and shell	Sand and shell	Medium to fine sand and shell	Sand and shell	Sand and shell
Rate of Littoral Transport per year			10 000 cu yds					
Dir of Littoral-Winter / Transport Summer	Northeast / Northeast	Northeast / Northeast	South / South	North / North	West / West		East / East	
MATERIAL AVAILABLE FOR NOURISHMENT								
Location	Channel to San Diego Bay	San Diego Bay	Bellows Field Kaneohe B A S	Hampton Harbor Inlet	Narragansett Bay Watch Hill Cove	New Haven Harbor Inlet	Long Island Sound	Sand barged E side L I pumped on beach
Distance from Fill	0 2 to 1 0 mile	2 0 to 3.0 miles	2 0 miles	0 1 to 1 0 mile	0 6 to 1 0 mile	1 8 miles	0 2 to 0 5 mile	0 ? to 0 6 mile
Character of Material	Medium to coarse sand and shell	Medium to fine sand and silt	Coral sand	Medium to fine sand silt and shell	Sand shell and silt	Medium to fine sand shell and silt	Sand shell and silt	Sand shell and silt
Quantity Available	Ample	Ample	Ample	Ample	Ample	Ample	Ample	
NOURISHMENT ACCOMPLISHED								
Type of Nourishment	Direct placement with hydraulic dredge	Direct placement with hydraulic dredge	Direct placement by trucks	Stockpile hydraulic dredge	Direct placement by hydraulic dredge	Direct placement by hydraulic dredge	Direct placement by hydraulic dredge	Direct placement by hydraulic dredge
Primary Purpose	Disposal of material and shore nourishment	Disposal of dredged material	Beach restoration	Shore nourishment	Beach restoration	Beach restoration	Beach restoration	Beach restoration
Slope of - (Before Ad) / Foreshore (After Ad)	1 on 15 / 1 on 25	1 on 40 / 1 on 25	1 on 3 / 1 on 5 - 1 on 7	1 on 5 / 1 on 7	1 on 7 / 1 on 5	1 on 7 / 1 on 5	1 on 19 / 1 on 10	1 on 20 / 1 on 10
Dimensions of Fill	1 0 Mi x 300'	3 0 Mi x 1500	0 5 Mi x 150'	0 3 x 0 4 x 0 3 Mi Triangle Shaped	0 4 Mi x 80'	1 5 Mi x 250'	0 7 Mi x 100'	
Total No of Cu Yds	3 500 000	19,000 000	110 000	535 000	170,000	1,000,000	507 000	78 313
Cost per Cu Yd	15 cents	12 5 cents		25 cents		75 cents	37 cents	69 cents
Date Placed	1938 - 1939	1940 - 1944	Dec 1951 to July 1952	1935	Oct 1948 - Feb 1949	July 1948 to Jan 1949	1947	1936 or 1937
Estimated Life of Fill								

Table 5

ARTIFICIALLY NOURISHED AND CONSTRUCTED BEACHES								
DESCRIPTION OF NATURAL CONDITIONS								
Coastal Area	Atlantic Ocean	Atlantic Ocean	Atlantic Ocean	Atlantic Ocean	Atlantic Ocean	Atlantic Ocean	Atlantic Ocean	Atlantic Ocean
Name	Jacob Riis Park Long Island N Y	Coney Island N Y	Long Branch N J	Atlantic City N J	Atlantic City N J	Atlantic City N J	Atlantic City N J	Atlantic City N J Absecon Inlet
Length of Problem Area	0 9 mile	0 7 mile	13 0 miles	Indefinite	Indefinite	Indefinite	Indefinite	0 5 miles
Exposure	E thru S to SW Clear	E thru S to SW Sheltered to Lower New York Harbor	N thru E to S	NE thru SE to SW Clear	NE thru SE to NE Clear	NE thru SE to SW Clear	NE thru SE to SW, Clear	NE to E
Slope - Foreshore / Nearshore	1 on 5 / 1 on 75	1 on 5 - 1 on 7 / 1 on 30 - 1 on 200	1 on 7 / 1 on 30 - 1 on 35	1 on 5 / 1 on 150	1 on 5 / 1 on 150	1 on 5 / 1 on 150	1 on 5 / 1 on 150	1 on 5 / 1 on 20
Beach Material Characteristic	Sand and shell	Medium to fine sand and shell	Medium to fine sand	Medium to fine sand and shell	Medium to fine sand and shell	Medium to fine sand and shell	Medium to fine sand and shell	Medium to fine sand and shell
Rate of Littoral Transport per year			177 000 cu yds	400 000 cu yds	400 000 cu yds	400 000 cu yds	400,000 cu yds	
Dir of Littoral-Winter / Transport Summer	West / West	East and West / East and West	North / North	Southwest / Southwest	Southwest / Southwest	Southwest / Southwest	Southwest / Southwest	
MATERIAL AVAILABLE FOR NOURISHMENT								
Location	Jamaica Bay N Y	New York Harbor	New York Harbor	Absecon Inlet	Absecon Inlet	Absecon Inlet	Absecon Inlet	Absecon Inlet
Distance from Fill	1 0 mile	0 3 to 1 0 mile	3 0 miles	1 0 to 2 0 miles	1 0 to 2 0 miles	1 0 to 2 0 miles	1 0 to 2 0 miles	0 1 to 0 5 mile
Character of Material	Sand shell and silt	Medium to fine sand shell and silt	Medium to fine sand and silt	Medium to fine sand and shell	Medium to fine sand and shell	Medium to fine sand and shell	Medium to fine sand and shell	Medium to fine sand shell and silt
Quantity Available	Ample	Ample		Ample	Ample	Ample	Ample	Ample
NOURISHMENT ACCOMPLISHED								
Type of Nourishment	Direct placement by hydraulic dredge	Direct placement by hydraulic dredge	Stockpile in water by hopper dredge	Offshore placement by a hopper dredge in 18' to 20' water	Offshore placement by a hopper dredge in 18' to 20' water	Offshore placement by a hopper dredge in 18' to 20' water	Offshore placement by a hopper dredge in 15' to 20' water	Direct placement by hydraulic dredge
Primary Purpose	Beach restoration	Beach restoration	Beach restoration and shore nourishment	Disposal of material and beach restoration	Disposal of material and beach restoration	Disposal of material and beach restoration	Disposal of material and beach restoration	Beach restoration
Slope of - (Before Adj / Foreshore (After Adj	1 on 7 / 1 on 5	1 on 5 - 1 on 10 / 1 on 4 - 1 on 7						1 on 7 / 1 on 5
Dimensions of Fill	0 9 Mi x 200'	0 7 Mi x 500'	0 7 Mi x 750'					0 5 Mi x 200'
Total No of Cu Yds	400 000	1 700 000	601 991	792,000	900 000	50? 000	1,362 000	500 000
Cost per Cu. Yd	15 cents	21 cents						77 cents
Date Placed	Sept 1939 - Dec 1939	Aug 1922 - May 1923	Apr 1948 - Oct 1948	Apr 1935 - Mar 1936	Feb 1937 - Sept 1937	Aug 1938 - Sept 1938	Aug 1942 - Sept 1943	July 1948
Estimated Life of Fill								

Table 6

ARTIFICIALLY NOURISHED AND CONSTRUCTED BEACHES									
DESCRIPTION OF NATURAL CONDITIONS									
Coastal Area	Atlantic Ocean	Atlantic Ocean	Atlantic Ocean	Atlantic Ocean	Atlantic Ocean	Atlantic Ocean	Atlantic Ocean	Atlantic Ocean	Atlantic Ocean
Name	Atlantic City N J	Ocean City N J Surf Road - 12th St	Virginia Beach Va	Wrightsville N C	Palm Beach Fla Garden & Ocean Roads	Palm Beach Fla Via Del Mar & Banyan Rd	Palm Beach Fla Tangier Ave	Palm Beach Fla Mediterranean Ave
Length of Problem Area	Indefinite	Indefinite	Indefinite	3 0 miles	Indefinite	Indefinite	Indefinite	Indefinite
Exposure	NE thru S to SW	NE thru E to SW Clear	NE thru E to SE Clear	E thru S to SW Clear	NE thru E to S Clear	NE thru E to S Clear	NE thru E to S Clear	NE thru E to S Clear
Slope - Foreshore Nearshore	1 on 5 1 on 150 - 1 on 200	1 on 5 1 on 50 - 1 on 300	1 on 10 1 on 60	1 on 7 1 or 1 - 1 on 20	1 on 10 1 on 50	1 on 10 1 on 50	1 on 10 1 on 50	1 on 10 1 on 50
Beach Material Characteristic	Medium to fine sand and shell	Medium to fine sand and shell	Medium to fine sand and shell	Medium to fine sand	Sand and shell	Sand and shell	Sand and shell	Sand and shell
Rate of Littoral Transport per year	400 000 cu yds	400,000 cu yds			225 000 cu yds	225 000 cu yds	225 000 cu yds	225 000 cu yds
Dir of Littoral - Winter / Transport Summer	South west South west	South west South west	South - Predominate North	South - Predominate North	South - Predominate North	South - Predominate North	South - Predominate North	South - Predominate North
MATERIAL AVAILABLE FOR NOURISHMENT								
Location	Absecon Inlet	Great Egg Inlet	Lake Rudee	Banks Channel	Lake Worth and Inland Waterway	Lake Worth and Inland Waterway	Lake Worth and Inland Waterway	Lake Worth and Inland Waterway
Distance from Fill	0 2 to 1 3 miles	0 6 to 2 5 miles	0 1 to 3 0 miles	0 0 to 3 0 miles	0 2 to 2 0 miles	0 2 to 2 0 miles	0 2 to 2 0 miles	0 2 to 2 0 miles
Character of Material	Medium to fine sand shell and silt	Medium to fine sand and shell	Medium to fine sand and silt and silt	Medium to fine sand or silt	Sand, shell and silt	Sand, shell and silt	Sand shell and silt	Sand shell and silt
Quantity Available	Ample	Ample	Ample	Ample	Ample	Ample	Ample	Ample
NOURISHMENT ACCOMPLISHED								
Type of Nourishment	Direct placement by hydraulic dredge	Direct placement by hydraulic dredge	Direct placement by hydraulic dredge	Direct placement by hydraulic dredge	Stockpile by hydraulic dredge	Stockpile by hydraulic dredge	Stockpile by hydraulic dredge	Stockpile by hydraulic dredge
Primary Purpose	Beach restoration	Beach restoration	Beach restoration	Beach restoration	Shore nourishment	Shore nourishment	Shore nourishment	Shore nourishment
Slope of - (Before Adj / Foreshore (After Adj	1 on 7 1 on 5	1 on 10 1 on 5	1 on 5 1 on 7	1 on 10 1 on 7	1 on 5 1 on 7 - 1 on 10	1 on 5 1 on 7 - 1 or 10	1 on 5 1 on 7 - 1 on 10	1 on 5 1 on 7 - 1 on 10
Dimensions of Fill	1 1 Mi x 200'	1 8 Mi x 300'	3 3 Mi x 300'		0 8 Mi x 400'	0 8 Mi x 400'	0 5 Mi x 400'	0 6 Mi x 390'
Total No of Cu Yds	700 000	1 590 000	1 078 890	600 000	630 000	1 035 000	454 640	300 000
Cost per Cu Yd	77 cents	75 cents	80 cents	14 cents	19 3 cents	19 3 cents	19 3 cents	35 cents
Date Placed	July 1948	1952	1952	1939	May 1948 - Nov 1948	May 1948 - Nov 1948	May 1948 - Nov 1948	Aug 1944
Estimated Life of Fill	5 years				5 years	5 years	5 years	5 years

Table 7

ARTIFICIALLY NOURISHED AND CONSTRUCTED BEACHES								
DESCRIPTION OF NATURAL CONDITIONS								
Coastal Area	Atlantic Ocean	Atlantic Ocean	Atlantic Ocean	Atlantic Ocean	Gulf of Mexico (Mississippi Sound)	Lake Michigan		
Name	Palm Beach Fla Mediterranean Ave	Palm Beach Fla Mediterranean Ave	Palm Beach Fla Opp W Palm Beach Canal	South Lake Worth Inlet Fla	Harrison County Miss	Chicago Park Ill North & Fullerton Aves		
Length of Problem Area	Indefinite	Indefinite	Indefinite	Indefinite	Indefinite	1 0 mile		
Exposure	NE thru E to S Clear	NE thru E to S Clear	NE thru E to S Clear	NE thru E to S Clear	SE thru S to SW Cat Is 1 3 Mi South Ship Is 2 3 Mi South	N thru E to SE Clear		
Slope - Foreshore Nearshore	1 on 10 1 on 50	1 on 10 1 on 50	1 on 10 1 on 50	1 on 5 1 on 25 - 1 on 50	1 on 10 1 on 150 - 1 on 700	1 on 7 1 on 50		
Beach Material Characteristic	Sand and shell	Sand and shell	Sand and shell	Medium to fine sand and shell	Medium to fine sand shell and clay	Sand and gravel		
Rate of Littoral Transport per year	225 000 cu yds	225 000 cu yds	225 000 cu yds	225 000 cu yds		12 600 cu yds		
Dir of Littoral - Winter / Transport Summer	South - Predominate North	South - Predominate North	South - Predominate North	South - Predominate North		South South		
MATERIAL AVAILABLE FOR NOURISHMENT								
Location	Lake Worth and Inland Waterway	Lake Worth and Inland Waterway	Lake Worth and Inland Waterway	North side of Inlet	Mississippi Sound	Lake Michigan		
Distance from Fill	0 2 to 2 0 miles	0.2 to 2.0 miles	0 2 mile	0 1 mile	0 3 mile	0 06 to 0 6 mile		
Character of Material	Sand shell and silt	Sand shell and silt	Sand shell and silt	Medium to fine sand and shell	Medium to fine sand, clay and silt	Medium to fine sand gravel and silt		
Quantity Available	Ample	Ample	Ample		Ample			
NOURISHMENT ACCOMPLISHED								
Type of Nourishment	Stockpile by hydraulic dredge	Stockpile by hydraulic dredge	Stockpile by hydraulic dredge	Continuous - Stockpile Bypassing Plant	Direct placement by hydraulic dredge	Direct placement by hydraulic dredge		
Primary Purpose	Shore nourishment	Shore nourishment	Shore nourishment	Shore nourishment	Beach restoration	Beach restoration		
Slope of - (Before Adj / Foreshore (After Adj	1 on 5 1 on 7 - 1 on 10	1 on 5 1 on 7 - 1 on 10	1 on 5 1 on 7 - 1 on 10	1 on 7 1 on 7	1 on 5 - 1 on 7 1 on 7 - 1 on 10	1 on 7 1 on 7		
Dimensions of Fill	0 6 Mi x 390'	0 6 Mi x 390'			26 0 Mi x 300'			
Total No of Cu Yds	71,690	380 000	100 000	50 000	6 000 000	250,000		
Cost per Cu Yd	32 2 cents			12 cents	12 6 to 14 6 cents			
Date Placed	May 1948 - Nov 1948	July 1949	1945	Each year	1951 - 1952	1938 - 1939		
Estimated Life of Fill	5 years	5 years	5 years					

CHAPTER 11

MEASURES AGAINST EROSION AT GROINS AND JETTIES

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Abstract. One of the difficult problems on a littoral drift coast is the erosion on the leeside of groins and jetties. This paper will deal with the problem giving special consideration to the conditions on the Danish North Sea coast where many interesting problems of littoral drift and coastal protection are found. They are discussed as an introduction to the main part of the paper which is principally concerned with leeside erosion and measures for its prevention.

A. THE DANISH NORTH SEA COAST

SITUATION

The Danish North Sea coast is the west coast of the peninsula Jutland, the Danish mainland. Fig. 1 shows the situation of this coast. From a point in the middle at Thyboroen lines are drawn NW, W and SW, and the distances to the respective coasts in England are indicated. It can be seen that there is free communication to the Atlantic Ocean from the northwest. The depths (metres) show that the North Sea is shallow; depths greater than 100 metres are found only in the northeast part of the Skager Rack, where there is a system of through-faults off the Norwegian coast.

GEOLOGY, WIND AND WAVES

The peninsula of Jutland is built of Pleistocene deposits, chiefly glacial drift originating from three different glaciations (boulder clay, meltwater-sand and clay), but in the northernmost part postglacial "marine-foreland" accumulations occur.

The west coast is shown in detail in Fig. 2. It is composed of weak material, mainly sand, and thus presents a smooth and plain shoreline, shaped by the action of the waves and currents. The most frequent and strongest winds are those from the western quadrant and they have consequently shaped the coast. Speaking of the southern part of the coast, a characteristic gale will start from the south-west with a force 4-6 Beaufort and in the course of 12 or 14 hours it will, with increasing force, 6-9 Beaufort, veer to the west or the northwest and continue there for two to three days or more. The water often begins to rise when the barometer falls, even before the gale has started because water from the English Channel is forced into the North Sea through the Straits of Dover. When the wind has turned to the north-west, the water falls. The tidal range at Esbjerg is about 4½-ft., at Hvide Sande (White Sands) about 2-ft., at Thyboroen about 16-in., and at the Scaw about 10-in. Heavy westerly gales may cause a high-water of 10 to 13-ft. at Esbjerg, 7 to 8-ft. at Hvide Sande, 5 to 6-ft. at Thyboroen and 3 to 4-ft. at the Scaw. The wave height is greatest at the North Sea coast,

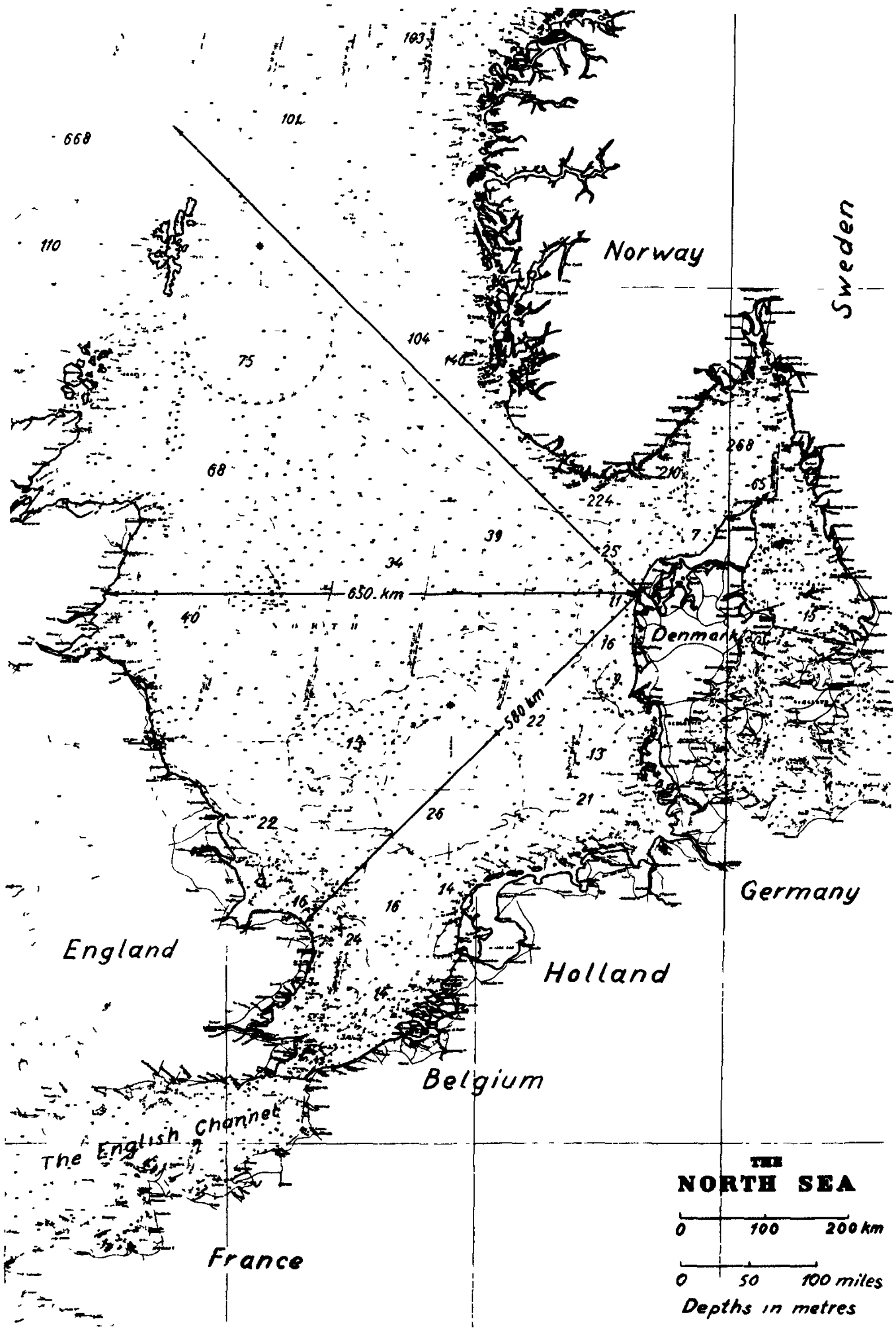


Fig. 1. The North Sea.

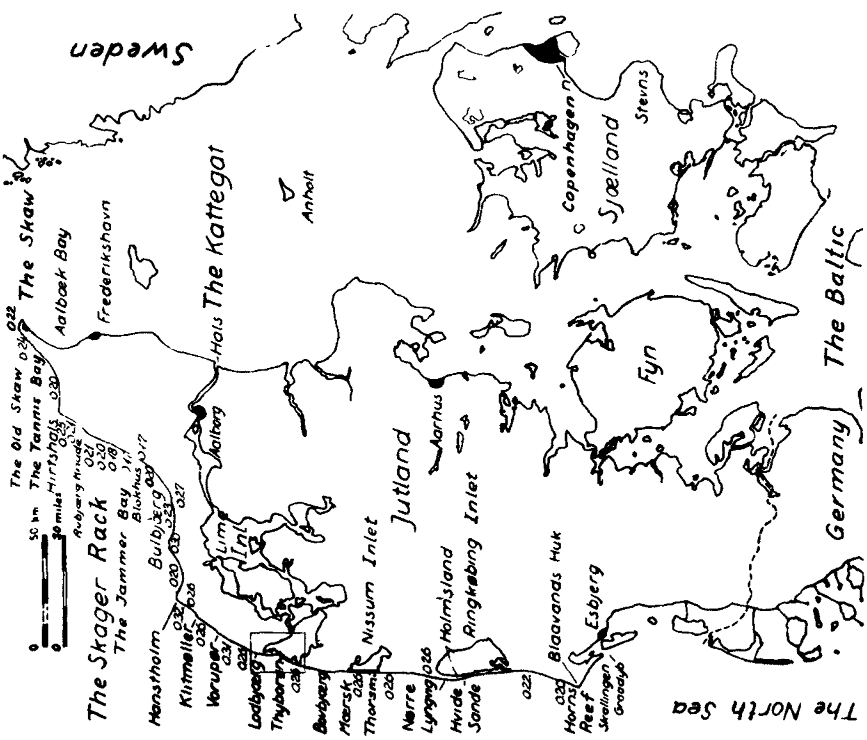


Fig. 2. Denmark.

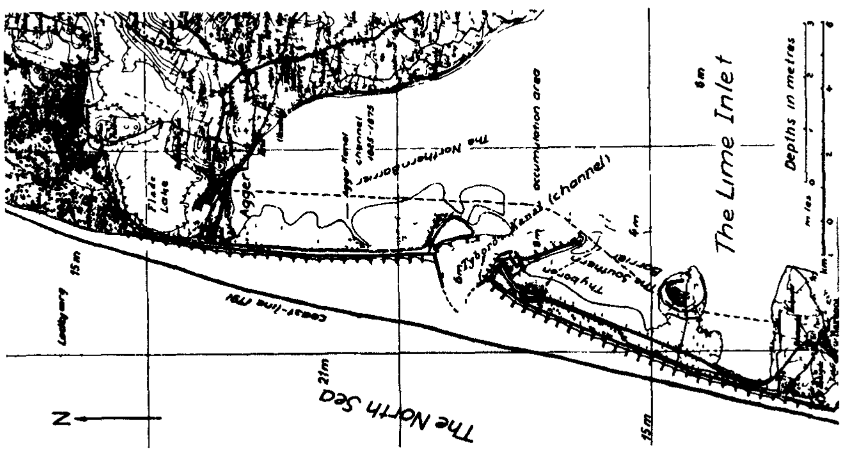


Fig. 3. The Lime Inlet Barriers.

about 10 to 12-ft. during heavy gales; the wave length is about 300-ft.

On that part of the coast between Blaavands Huk and Lodbjerg, which takes an almost north-south direction, we find the inlets closed by barriers, (the Ringkoebing Inlet, the Nissum Inlet and the Lime Inlet, cf. the New Jersey coast. Between Lodbjerg and the Scaw the coast is, as it were, suspended from hooks formed by headlands: Hanstholm (chalk and moraine, 160-ft), Bulbjerg (chalk, 100-ft), Hirtshals (Moraine, 100-ft.), and a few smaller headlands. Between these the coast is smooth and plain.

The height of the west coast is generally 10 to 20-ft. above mean water level, but the height of the barriers is only 3 to 6-ft., and, therefore, they must be protected from the sea either by dunes or by artificial dikes. Besides the headlands already mentioned considerable moraines are found at Bovbjerg (130-ft.), Lodbjerg (100-ft.), and Rubjerg (230-ft.).

It looks as if the near-shore drift along the coast, a result of the action of waves and currents, takes a northward direction from a point near Lodbjerg; whereas to the south of Lodbjerg, except for a short distance along the Southern Lime Inlet Barrier (see later), it takes a southward direction, i.e. towards Blaavands Huk. Lodbjerg is a neutral point for the near-shore littoral drift.

In Fig. 2 the average size of the sand for each locality is shown. The size is the average computed from 20 characteristic surface-samples, 10 of which were taken about May 1, 1951 and 10 about June 1, 1951 at the reference point (see (2)).

These investigations showed:

a. The average size of the sand ($d_{50\%}$) is greatest between the headlands I hjerg and Hanstholm. This is the part of the coast exposed to the most pow ful waves (see Fig. 1) and also the part nearest the neutral point. Becaus of refraction local maxima are found on the headlands Bulbjerg, Hirtshals and Hanstholm. The sand is largest where the coast retrogrades, or where the beach profile commonly is steepest.

b. The average size of the sand is smallest in the extreme areas, the Scaw and the Blaavands Huk, and at the slightly exposed coasts including the bays, and where progradation takes place, or where the beach profile is flat. The smallest grains appear at the bottom of the Jammer Bay (0.17 mm) This minimum size is also found on the beach of the Pacific Ocean (see (2) p. 869).

c. The size of the sand decreases southward from the point, supposed neutral, near Lodbjerg. The size also decreases northward, but not regular ly as local minima appear in the bays. It can be assumed, however, that this decrease can be attributed less to the wear of the material, than to a sorting out due to transportation (see (9) pp. 67-71).

d. The coefficient of uniformity ($d_{60\%}/d_{10\%}$) fluctuates between 1.2

and 1.9, and is generally highest for the largest average size.

COASTAL MORPHOLOGY

As mentioned the material is carried along the coast to the north from a point near Lodbjerg. By far the greater quantity is probably deposited north of the Scaw on the long reef which projects 2.5 miles into the sea. This reef consists of fine white sand. Consequently the spit is annually shifted 4 to 5 yards northward and about 1 mill. cubic yards are deposited there every year.

Between Hanstholm and the Scaw there are generally two off-shore bars; at the Scaw where the littoral drift is at a maximum there are even three or four bars.

With the exception of certain short stretches, especially to the north, the coast between Lodbjerg and the Scaw is fairly stable because the headlands lessen the possibility of changes in the coastline. As a result of the action to which they have been exposed, the headlands are almost identical in shape and position. Since the angle made by the shoreline is everywhere between 100 and 120 degrees the shape of the coast has a mature appearance.

We have seen earlier that the littoral drift south of Lodbjerg takes a southward direction, towards Blaavands Huk.

North of Blaavands Huk there are generally two off-shore bars, but between the Nissum Inlet and Hanstholm only one, and this is poorly developed along the Lime Inlet Barriers. It looks as if the number of bars increases with the intensity of the littoral drift, but the number is not indicative of the quantity of the littoral drift.

The beach profile is steepest at Lodbjerg where the 20-ft. depth contour is situated about 1000 feet from the shoreline, the 35-ft. depth contour about 2000 feet from the shoreline, and the 60-ft. depth contour about 9000 feet from the shoreline.

The shoreline has retired very considerably at the Lime Inlet Barriers about Thyboroen, where special conditions connected with the breach of the barrier prevail. Before the groins were built at Bovbjerg, the shoreline annually retired about 2 yards on an average, calculated over a considerable number of years. The erosion decreases towards the south until at the barriers by Ringkoebing Inlet it is zero. From this point towards Blaavands Huk, we find a growing tendency to accretion which makes the coastline turn slowly to the south-west. During the period 1870-1950, Blaavands Huk has thus been shifted about 1.200 yeads to NNW, or 15 yards a year. The movement of the coastline here must be considered from a geological point of view, according to which the 25 miles of Horns Reef, probably the remnants of a marginal moraine, must be supposed to play a great part as a sheltering and material-stopping wall (tombolo-development). Thus it looks as if great quantities of the sand eroded on the southern part of the North Sea coast are deposited at Horns Reef. Various circumstances, however, indicate that a considerable quantity of small grained sand is carried over the reef from which it drifts further to the south along the peninsula of Skallingen.

Here it contributes to the great difficulties of maintaining the fairway of Graadyb, which has a depth of 27 feet leading to the harbor of Esbjerg, the important outport for agricultural produce to England. About 1 million cubic yards are dredged every year.

Nothing is known about the littoral drift at greater depths, caused by the action of waves and especially by currents, but, because of the westerly winds, a drift of small-sized material is probably taking place from the North Sea to the deep water of the Skager Rack, where the material settles.

Beach profiles which retrograde and have only one bar follow roughly the equation $y^{3/2} = p \cdot x$ with $p = 0.03-0.05$ (y = the water-depth, x = distance from the shoreline to the point with the depth y) between 16-ft. depth and 45-ft. depth, or a distance of about 3-4000-ft. Yet near the shore and out over the 45-ft. depth contour the profile follows a 2° - parabola. More stable and flat profiles will follow the equation $y^2 = px$, p = about 0.1. Provided storm-waves with L_0 = 300-ft., and constant shearing stress, i.e. that the average velocity of the oscillating wave-motion at the bottom is constant, and dE/dx = constant, where E is the transported wave energy, a theoretical approach gives the equation $y^{3/2} = px$ valid for depth outside of the bar, 16-ft. to about 45-ft.

COASTAL PROTECTION WORKS AND HARBORS

In 1949 six groins were built at the Old Scaw, cf. Fig. 1 and Fig. 18, 170-ft. in length and 270-ft. apart, consisting of a wall of wooden piling with rubble heaped along the sides, combined with a lateral work built of rubble and supported by wooden piling.

At Hirtshals a harbor, covering a water area of 46 acres, was finished in 1932. The harbor serves as a fishing-port and as terminal ferry-harbor to Norway (Kristianssand). The ferries require a depth of about 23-ft. at the entrance, but the maintenance of the depths has involved great difficulties owing to the strong north-easterly littoral drift, and it seems that these difficulties are likely to continue with the present dimensions of the jetties. At the headland of Hanstholm the construction of a harbor has been started. The western jetty has been built, but has caused great deposits on the leeside due to diffraction, cf. section D, the angular groin.

Very considerable works of coastal protection have been carried out from Lodbjerg to a point about 13 miles to the south, except for a distance of about 1/2 mile at Thyboroen Channel. Fig. 3 shows the cut indicated in Fig. 2. The figure comprises principally the two barriers (1/2 to 1 mile in width) which separate the North Sea from the Lime Inlet. The barriers are built up of sand to a level of about 5-ft., below this is a weak, inlet deposited, Litorina clay (level + 19-ft.). On the point of the Southern Barrier lies a fishing harbor, Thyboroen (about 2,000 inhabitants). In Fig. 3 is shown the position of the shoreline in 1791, when the barrier was unbroken, and that of the shoreline of to-day. The existing open channel was formed by a barrier-breach in 1862. Several channels existed before that time, f. inst. the Agger Channel, cf. Fig. 3, breached

1825, closed by nature in 1875. Historical accounts suggest that in the 11th century the Lime Inlet had an open and good connection with the sea, through which the Viking raids on England passed. On account of trading interests, fishing and shipping, the State has maintained the channel by extensive protective and regulating works, groins and dikes, cf. Fig. 3. The latest measures in this respect are embodied in the Act of August, 1946, which provided for the construction of two big jetties, one on each side of the channel, and a new solid dike about 1 1/2 miles from the sea. The dike will later be built across the channel as a dam with sluices. This new project, just commenced, is indicated by dotted lines in Fig. 3.

Immediately after the breach the barriers began curving inwards towards the channel, so that to-day the shoreline at Thyboroen is situated about 1 1/4 miles farther landwards than in 1791. The curving of the barriers is caused by the erosion in connection with the difference in water level (up to about 5-ft.) between the sea and the inlet prevailing during westerly gales, the result of which is that the water with its contents of suspended material is sucked into the inlet, where the solids are deposited in large shoals, cf. Fig. 3. At present about 1 million cubic yards of sand are deposited annually, whereas the annual average erosion of the barriers is 1 1/2 to 2 yards on the Southern Barrier and 2 to 3 yards on the Northern Barrier. During strong westerly gales the current in the channel may reach a speed of about 10-ft. per second. The mean depth of the channel has been stated in metres in Fig. 3.

Until now the coastal protection works, which are to be described in the following, have comprised the construction of dikes and groins. During the period 1875-1910, thirty groins were built on the Southern and twenty-five on the Northern Barrier, besides the big dikes (shown in Fig. 3) and the groins and dikes constructed along the channel. All these works, as the harhor at Thyboroen, are financed, built and maintained by the Danish State (the Ministry of Works), acting through The Board of Marine Works.

The dikes are to protect the barriers from wash-outs, and the groins are to check the littoral drift, which for a distance of 5 to 7 miles on both sides of the channel goes in the direction of the latter. The space between the groins is about 1.250 feet, and the length of the groins is 700 to 1.300 feet, generally increasing towards the channel. The last groins on both sides of the channel are reinforced and built out as jetties of 1.400 feet and 3.000 feet respectively, cf. Fig. 3. The groins are not maintained to their full length any more, for the continued erosion made them gradually become too expensive, and 200 to 400 feet of the end farthest from the coast have been abandoned.

The construction of the groins and the dikes has been gradually developed and improved. Fig. 4 shows a section of the costal protection of the Southern Barrier, which, like the Northern Barrier, has a "sea dike" and a "reserve dike" of the dimensions stated in the figure. The sand dikes are planted with marram-grass (16 tufts of three to four plants per square yard). They are built by excavators and bulldozers. The new dike is built by suction-dredgers. The figure shows a longitudinal section of a groin. The height of the outer end (4-ft.) has been chosen for constructional reasons.

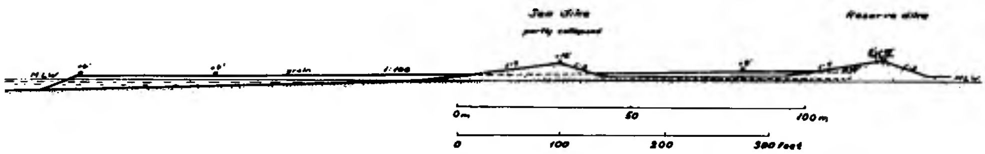


Fig. 4. Coastal protection, the Lime Inlet Barriers.

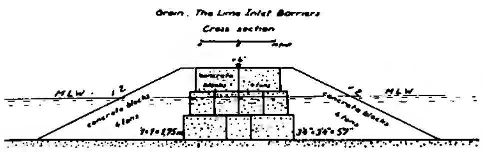


Fig. 5. Cross section of groin, the Lime Inlet Barriers.

Fig. 6. The crane at work, the Lime Inlet Barriers, 1948.

Fig. 7. Groin K, leeside-erosion, 1949.

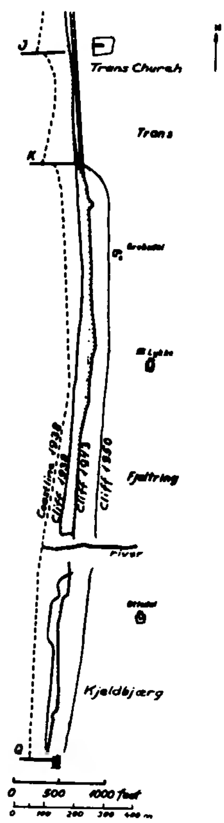


Fig. 8. The coast between groins J-K and Q.

Fig. 9. Erosion on the leeside of groin I, Bovbjerg, 1895.

Fig. 5 shows a cross section. The groin is built of a core of piled-up concrete blocks - the "crown" - with concrete blocks or rubble heaped along the sides. The weight of a block is 4 tons, the volume 62 cubic feet, and dimensions 3-ft. 4-in. x 3-ft. 4-in. x 5-ft. 9-in. Each block contains 8-cwt. of cement, 53 cubic feet of gravel (taken from the foreshore), 28 cubic feet of sand (from the foreshore or screened from a gravel pit), a filler, and 50 gallons of water. The blocks are cast by means of a concrete-mixer in a working place nearby and take 6 to 8 man-hours. They are rammed or vibrated carefully and are kept wet after the casting. The block is cast with a hooked iron bar in it, which is used for slinging when the block is being placed in the groin. The cement used is a sea-water cement specially produced by the Danish cement factories. This "sea-water cement" is a type of Portland cement specially manufactured to withstand the influences to which it is exposed in sea-water, particularly the deteriorating effect of sulphates. Experiments on air-entrainment and asphalt-compounds for surface-treatment are being carried out.

The 4 tons blocks are taken to the groin on trolleys and put in place by means of specially designed self-propelled cranes running on 100 lb. rails laid out on sleepers attached to the block-hooks. The cranes are designed to carry 10 tons, so that they may also be used for handling 8 tons blocks, which are used only in the exposed places of the groin. Fig. 6 shows the crane in function. The 'land-ends' of the groins consist of three 'crown-blocks' only, and the block is often cast in the groin itself.

The type of groin used is called an 'after-filling groin'. The material required for the construction of a groin 700-ft. long, about half of which is situated within the coastline, consists of approximately 3,000 4 tons blocks, and at the present time costs 150,000 dollars. The weakness of this type of groin is that it gradually sinks and must be re-heightened, generally about every 10 years. This consists in redressing the crown and supplementing the concrete blocks or the rubble along the sides.

A stepped revetment of reinforced concrete, slope 1 in 2, has been built on the outer slope of the dike north of Agger (the Northern Barrier, cf. Fig. 3) for a distance of about 1 mile off Flade lake and for a distance of about 1200-ft. at Thyboroen (the Southern Barrier). In places the design has not proved very durable, partly because the sheet-piling was not tight, and partly because the waves swept over the construction and washed away sand behind and under the revetment. A flexible type would have been better, but to a coastline subject to continual erosion like the Lime Inlet Barriers this design offers an adequate protection (see (11) pp. 163-167).

The groin construction described has also been used at Bovbjerg where 23 groins 500 to 700-ft. long, spaced about 1250-ft. apart, were built during the period 1875-1937. The coast erosion, which immediately before the groin-building at Bovbjerg was about 10-ft. a year, is now less than 3-ft. The soil is moraine clay and is classed with the best arable land in Denmark. On the leeside of the group of groins (the South side) a severe erosion, which will be described later, has taken place in the 30 to 40-ft. high, rather sandy coast.

South of Bovbjerg are two small groups of five groins each. Here a block-construction with a central wooden sheet pile wall has been tried, but so far this type does not seem to be an improvement on the pure block-construction.

To protect the sand barrier in front of Nissum Inlet from overtopping, a dike has been constructed 300 yards from the shoreline which retrogrades nearly 2 yards a year. The upper level and width are 16-ft. 6-in. and 6-ft. 6-in., the slope towards the sea is 1 in 7, and the slope towards the land 1 in 3. The littoral drift is to the south. At Thorsminde the outflow from the inlet is covered by two jetties built of concrete blocks, 500-ft. and 700-ft. in length respectively, and by four groins, two of which (650-ft. long) are situated north of the outflow and two (350-ft. long) south of the outflow. The outflow is provided with a sluice, first built in 1868-1870 during an attempt to dry up part of the inlet. There is also a fishing harbor navigable for boats up to 10 G.R. tons.

At Husby, south of the Nissum Inlet, there is a group of 12 smaller groins built of concrete blocks. The sand barrier in front of Ringkoebing Inlet is unprotected, as the coastline seems to be in a state of equilibrium. The outflow at Hvide Sande is protected by two jetties, 850-ft. long, built of concrete blocks. The outflow is provided with sluices, and there is a fishing harbor for boats of 10-20 G.R. tons.

Between Hvide Sande and Blaavands Huk there is no coastal protection work as the coast is either in a state of equilibrium or advancing. To the south-east of Blaavands Huk on the peninsula of Skallingen, where the shoreline is retiring, some smaller groins of wooden sheet piling have been built.

South of Blaavands Huk are the islands of Fanoe, Manoe and Roemoe; between them and the mainland is the Danish marsh-sea of a nature similar to that of the adjacent German and Friesian coasts.

B. EROSION ON THE LEESIDE OF JETTIES AND GROUPS OF GROINS

THE PROBLEM EXPLAINED WITH AN EXAMPLE

We have heard earlier of the very severe erosion on the leeside of the groins at Bovbjerg, cf. Fig. 2. Erosion on the leeside of jetties and groins is without doubt due to the fact that the beach drift on the luffside changes, past the groin, into suspended-load and bed-load, while at the same time the ratio H_0/L_0 is lowered in consequence of the wave diffraction, by which the beach drift on the leeside is possibly increased (see (28) pp. 558-562).

Since the groin breaks off the beach drift from the up-drift side, the result must be erosion on the leeside. In Fig. 7 is shown the groin K, photographed from the leeside. This groin is the last on the leeside of a group consisting of 5 groins constructed in 1933-1937 south of the already existing groins at Bovbjerg, to protect among other things the old church at Trans, a typical Danish village church in Roman style, built of granite probably in the 12th century, though the tower is not so old, cf. Fig. 7. The tower is now situated 270-ft. from the top of the cliff. Immediately after the construction of groin K (1937) a very severe erosion started on the leeside. Fig. 8 shows the rather sandy coast, 30 to 40-ft. high, between the groins J-K and Q, the former at Trans church, the latter at the clay-hill of Kjeldbjerg, about 1 mile to the south. The upper edge of the slope in 1938-1943-1950 and the shoreline in 1938 is shown. The average width of the beach is about 170-ft. - more in the summer and less in the winter.

The shoreline has retired 20 to 25-ft. each year for the past 10 years. It has, therefore, been constantly necessary to lengthen the land-end of groin K. The groin was in 1937 465-ft. long, in 1950 650-ft., which means it has had to be lengthened 185-ft. in only 13 years.

The beach profile is about 1300 feet from the shoreline to the 20-ft. depth contour and 2600 feet to the 30-ft. depth contour. The erosion on the leeside has in this, as well as in many other cases, especially effected a retrogradation of the shoreline, but not corresponding retrogradations of the depth contours in deeper water, because the 0 to 20-ft. area has been widened both inward and outward, the latter probably by deposits of eroded material from the cliff.

The development at groin K is probably the most severe in Denmark. A corresponding development has taken place at Maersk, cf. Fig. 10, and at the Old Scaw, cf. Fig. 18. In technical litterature "the leeside erosion" has been described several times, f. inst. in (3) chapter 6/35, (4), (5), (21), and (22) pp. 103-110 and pp. 192-212.

In (5) the chapter called "Federal Responsibilities in Shore Protection" deals with the problem from a legal point of view. Neither has any court in Denmark up to 1952 awarded the injured on the leeside any compensation for loss of area from such groin-building and the consequent leeside-erosion.

C. MEASURES AGAINST EROSION ON THE LEESIDE OF JETTIES AND GROUPS OF GROINS: OLDER PRINCIPLES

A LATERAL WORK ON THE LEESIDE

This measure is the most primitive, but it is often used, and it is effective when the coast retrogradation is not so strong that the shoreline moves quickly behind the end of the lateral work. Fig. 9 shows the lateral work of concrete blocks at groin 1 at Bovbjerg, photographed in 1895.

The method is expensive and not correspondingly effective. As a rule one may, therefore, say that lateral work on the leeside is a rather passive arrangement, as the work in many cases will, sooner or later, be cut off at the end of the lateral work, cf. Fig. 19. The lateral work ought under all circumstances to be built so that it gives as little reflection as possible, because the littoral drift as bed-load seems to depend on the shearing stress raised to the 2.5 power (see (27) p. 799) and, as a consequence of this, is approximately proportional to the average water-velocity, raised to the 5 power.

CORNER-GROINS

Fig. 10 shows the coastal area at groin V in Maersk north of the Nissum Inlet. Groin V is the most southerly groin in the group I - V, built in 1931-1932. Included in the figure are the shorelines in 1938 and 1948, and one can see that the shoreline during these 10 years has retired about 20-ft. a year. Actually, the retrogradation has mainly taken place in the period 1946-1948. The groin is the end-point to the reserve dike on the North Thorsminde barrier, which was built 1941-1949 and carried out to the groin in a great circle. It became clear, when in 1946-1947 the erosion south of the groin group began seriously, cf. Fig. 11, that the dike ought to have been connected to a groin further north or should have been carried due north into the high-lying land north of Maersk. The construction of the dike-section in the circle was, however, very much advanced, and it was, therefore, decided to secure the corner at groin V, first building a corner-groin, and then a dike-reinforcement, between groin IV and groin V. In order to increase the effect of the corner-groin, the land-end of groin V was enlarged, and the groin was built as shown in Fig. 10. Fig. 12 shows the groin in action during the severe gale in October 1948. Only a little of the old sea dike was left after the storm, cf. Fig. 13. Some German pill-boxes, built in 1944 into the dike, can still be seen far out on the beach. The January gale in 1949 demolished the rest of the dike (Fig. 14), but the reserve dike, which was nearly completed, stood. Now the reserve dike is extended northward into the high-lying land north of Maersk.

The corner-groin hindered the erosion on the leeside from running in directly along the main-groin; this method was expensive, however, and the effect was limited to a very small area. Also an unfortunate effect was that the uprush and the wind were pressed together between the main groin and the corner-groin. Corner-groins are especially not a good measure when the coast is under continuous erosion.

THE SHORTENING OF GROINS ON THE LEESIDE

I do not know when the principle of shortening was used for the first time, but Kressner's interesting thesis was published in 1928 (see (21)). At the Technical University of Danzig Kressner carried out model-experiments, concerning the space between groins and the problem of the leeside-end of a group of groins. A short summary of Kressner's investigations is given below.

The investigations, concerning the space between groins, were first carried out as sheer current experiments with a velocity of 0.25 m/sec. These investigations proved that the largest space allowed between groins was 5 x the groin-length outside the shoreline; if more, there was at once a decrease of material from the area between the groins, as this no longer

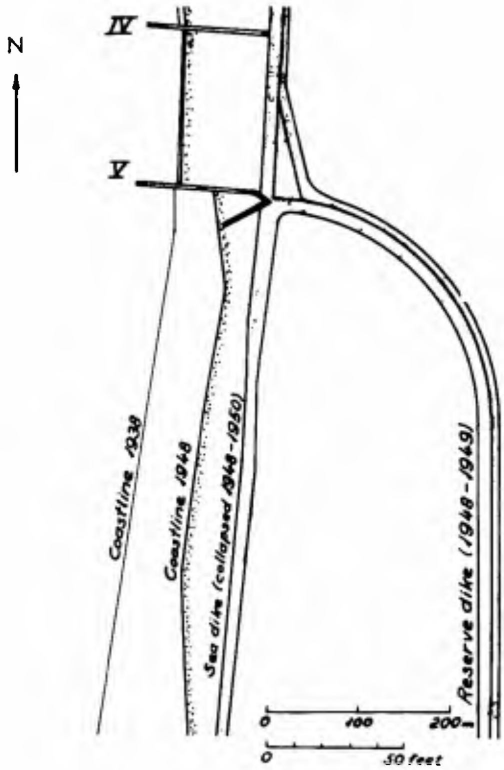


Fig. 10. The coast at Maersk.

Fig. 11. Groin V, April 1948.

Fig. 12. Groin V, October 1948.

Fig. 13. Groin V, October 1948.

Fig. 14. The coast south of groin V, January 1949.

was filled with eddies, and the longshore current penetrated the area between the groins and removed material.

A similar result was achieved in experiments with spurs in rivers, carried out in Poona (Pakistan). The experiments are described in (23). The spurs were supplied with a short T-formed head. The erosion between the groins started when the space between the groins was between 5 and 8 x their length in the water.

Later on Kressner carried out experiments with waves about 1 1/2 in. high, and a current 1/2-ft. per sec. These investigations proved that the shoreline between the groins could only be kept in a straight position, when the space between the groins was not more than 2-3 x the length in the water. If the space were greater, the shore would get a saw-tooth form as material was still accumulated on the up-drift side.

After this Kressner carried out experiments concerning the erosion on the leeside.

The experiments showed that erosion could be avoided by shortening the groins on the leeside; the angle of shortening being about 6 degrees, cf. Fig. 15. By doing this the longshore current ran smoothly along the shore and the groins, i.e. the drifting material was not pushed away from the beach.

The principle of shortening which is also mentioned in (29) pp.11-12 is now extensively used. Fig. 16, taken from Thierry's and van der Burgt's report to the XVIIth International Navigation Congress in Lissabon, 1949 (see (30) pp. 135-156), shows schematically this principle as regards the common Dutch 'Strandhoofden' (or beach groins) and the so-called V-groins; the latter being small headlands which probably can be built with double the normal space (see (30) pp. 154-156). Besides, these V-groins may prevent leeside-erosion better. Fig. 17 shows the groins at Scheveningen, Holland, where the principle has been successfully employed. In accordance with the shortening the space should be decreased so that the normal ratio between the groin-length and the space is maintained.

The shortening is generally carried out at about 6 degrees. In the successful case at Scheveningen the angle is about 5 degrees.

The principle of shortening has not been employed on the Danish North Sea coast, but has on the coast of Zeeland (Sjaelland), the largest of the Danish islands. The results here are not very convincing, however, but the shortening-angle is $<$ 10-15 degrees, i.e. very steep. I believe that the shortening should begin far in the group. If the group consists only of a few groins the shortening should start after the first groin on the up-drift side. If the group consists of several groins the shortening should probably be carried out on the up-drift side also, to ensure a smooth passage to the unprotected coast, cf. Fig. 35.

D. MEASURES AGAINST EROSION ON THE LEESIDE: NEWER PRINCIPLES

THE BREAKWATER GROIN

In a report to the XVth International Navigation Congress in 1931, Pala and d'Arrigo write about a so-called 'Bayonet-groin' (see (24) pp. 6-7) as follows:

"There is a special type of groins which must be particularly taken into consideration, especially when there is not a large flow of matter towards the beach: it is what we might call the "bayonet" type. It is formed by two successive broken stretches, the second stretch being practically at about right angles to the direction of the prevailing waves. In the construction of a certain number of these groins for the protection of a long stretch of coast, the head of each groin must mask for a certain distance, in the direction of the prevailing waves, the point of juncture between the first and second stretches of the neighbouring groin downstream.

What distinguishes this second type of groin is that the second section has as its function the breaking of the most violent waves, which it thus prevents from flinging themselves on to the shore and causing erosion.

In addition, seeing that in rough weather the waves which pass over the tops of the groins carry, more or less, materials torn up from the bottom of the sea, it happens that such sediment, poured into the space protected by the groin, deposits there and thus forms the reconstitution of the coast.

This phenomenon also occurs when the filling-materials do not arrive on the beach in sufficient quantity. It is for this reason that this kind of groins may be efficacious where the others, transversal and aligned end to end, would give no result or only have a limited result."

I am, however, inclined to believe that the explanation given of the effect of the Bayonet-groin is not quite satisfactory, and I believe that the groin probably worked as an angular groin too, see below.

Use of breakwaters as coastal protection is fairly common in Italy, see f.inst. (33).

The Italian breakwaters are often joined to the coast, among other things for maintenance-reasons, but this practice has often been necessary to achieve a successful effect from the total construction (see (33) p. 123).

Vera, Isla and Acena write in (32) p. 27 that "l'erosion adjacente" can be avoided partly by joining the groin-group to solid cliff, and partly by building a breakwater in connection with the groin, i.e. some sort of a "Bayonet-groin". No explanation, however, has been given of the effect of these breakwaters. In a new passage in the same report, called "Etude Theorique Des Transports De Sable Causes Par La Houle" ((32) pp. 36-44) computations are made of the coastal current outside the breaker-zone, on the basis of Iribarren's theory which only considers the static pressure differences arising from the refraction etc. The currents are particularly strong in places where the coastline bends or where there is a headland.

I shall not go further into Iribarren's theory nor into the descripti given by Vera, Isla and Acena, concerning the development of the shoreline behind a curved jetty which, according to the figure, gives an erosion abo 10 x as big as the deposit. For this one reason the theory cannot be satis factory. Iribarren does not go into wave diffraction which gives the real explanation of the current on the leeside of the jetty, cf. (15).

THE T-GROIN

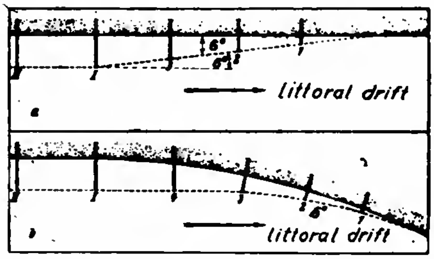
In his report to the XVIIth International Navigation Congress in 1949, Col. Frech writes about the T-groins, built at Asbury Park in New Jersey as follows (see (14) p. 52): "Several of these groins have been experimentally supplemented by breakwater members extending 100 to 150 feet at right angles at their outer ends forming a T. It is too early to judge the results produced by these structures, but the following observations may be made: certainly, further erosion of the bluff has been stopped; the groins not having the breakwater feature at their outer ends have not accumulated much sand, if any, but those with the oreakwater addition (known now as T-groins) have had a paradoxical effect, sand having accumulated along both sides of the stem of the T, but with deposits on the down-drift side exceeding considerably the accumulation on the up-drif side".

The T-groin has, however, been used before. Matthews wrote in 1934 (see (22) p. 218): "If, on the other hand, there is a sufficient travel of material, the protection of the coast can be assured by groynes alone. If the supply is insufficient, or there is a reason to fear that storm-waves may strike the coast in a direction parallel with that of the groyne so producing an excessive thinning of the beach, the protective works should comprise in addition to groynes a longitudinal structure. Special types of groynes, in a bent line, T-shaped or crusiform, designed for this purpose, have not yet received the sanction of experience".

The successful effect of the T-groin is explained by the theory of tl angular groin (see below).

INCLINED GROINS

Fig. 18 shows the groin-group at the Old Skaw, cf. Fig. 2. It is buil mainly in 1949, and the groins are constructed with slanting landends, particularly advantageous at the last groin in the group, because, when it is so built, the erosion on the leeside does not move close to the lee-side, but the construction of the last groin as a short angular groin probably would have been more advantageous and will now be carried out (sh with dotted lines).



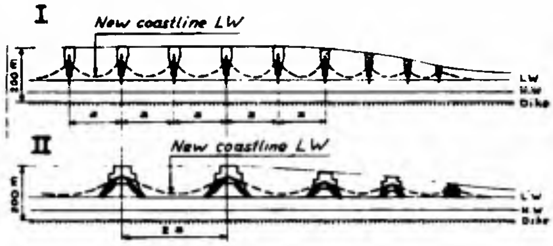


Fig. 15. The principle of shortening: Kressner.

Fig. 16. The principle of shortening: Thierry and van der Burgt.

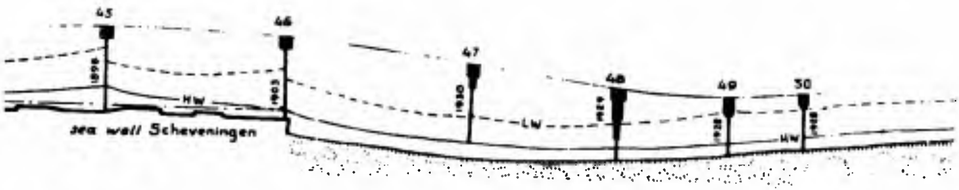


Fig. 17. The principle of shortening, Scheveningen, Holland: Thierry and van der Burgt.

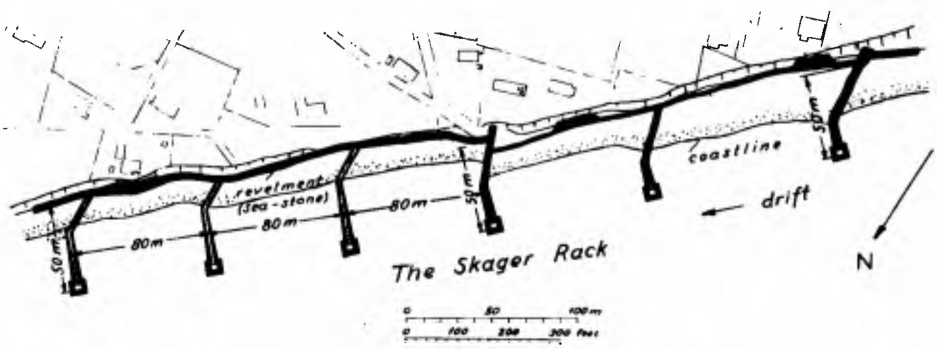


Fig. 18. The groins at the Old Scaw.

Fig. 19. The leeside-groin at the Old Scaw.

THE ANGULAR GROIN

In order that deposits may take place on the side of a groin, there must be:

a. Littoral drift towards the groin, and
b. the material must be thrown ashore.

a. Since the littoral drift moves towards the groin on the up-drift side and off the groin on the down-drift, it is, if deposits shall take place on the leeside, necessary to turn the littoral drift. This might take place in a limited area on the leeside, if the groin is equipped with a short lateral work, an "angle".

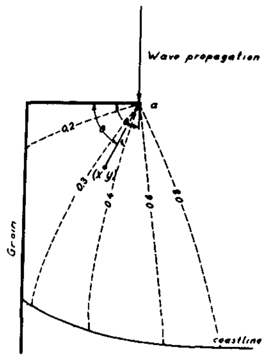
Fig. 20 shows an angular groin. The wave propagation is as in the figure. A cylinder-wave is caused by the diffraction at the corner a. The wave-action at an arbitrary point x.y can be calculated (see (19) and (25)). In the figure dotted lines are drawn through points with the same diffraction-coefficient provided there is a constant depth behind the angle, complete absorption at the stem of the groin (length 2 x L where L = wave length) and no friction-loss at the bottom.

As a consequence of the diffraction a littoral drift is produced from the leeside, towards the groin.

b. The mere transport of material towards the groin is, however, not sufficient. The strong gales certainly supply the leeside of the groin with considerable material, but investigations with beach profiles at the laboratory (see (10) Beach Erosion Board, (17) Waters, and (28) Thorndike Saville, and Fig. 22, experiments in Copenhagen 1949), as well as experiments with actual beach profiles (see (7)) and investigations on the Danish West coast have proved that waves with a large steepness-ratio will erode the beach making the profile flatter, cf. Fig. 22 B, b, bar profile, whereas waves with a small steepness-ratio build up the beach and make the profile steeper, cf. Fig. 22 D, d, and C, c, beach ridge profiles. (The small letters indicate similar results with tests in scale 1:2).

Storm-waves will often strike the stem of the groin at a greater angle, in consequence of which the steepness-ratio is increased along the stem, and, therefore, erosion will occur. This is particularly so, if the stem gives rise to a strong reflection, or in some other way causes an increase of the turbulence, cf. Fig. 23, which shows seasonal fluctuations of the shoreline of the up-drift side of a groin on the Lime Inlet Barriers. Even if the littoral drift towards the groin is much stronger in the winter than in the summer the beach will be eroded in the winter. The steepness-ratio is, however, lowered behind the angle. The lowering is considerable (Fig. 20), and consequently the waves will quickly change into a constructive type; cf. Fig. 22 D, d, and C, c.

The total effect of the circumstances mentioned in a and b is that a beach is built up behind the angle. Provided that no reflection from the stem of the groin takes place, the shoreline will, as a consequence of diffraction-currents, bend further out rather than follow that circle centered at the end of the angle.



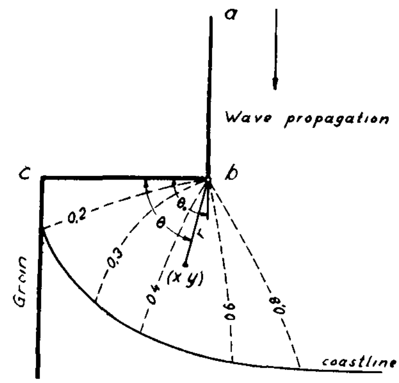


Fig. 20. An angular groin, diffraction.

Fig. 21. A Z-groin, diffraction.

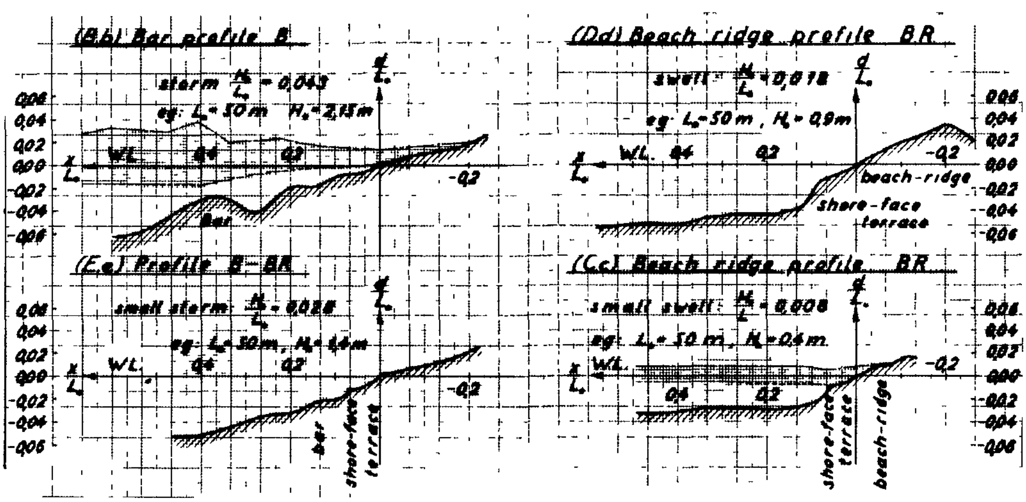


Fig. 22. Beach profiles. Laboratory experiments, Copenhagen 1949.

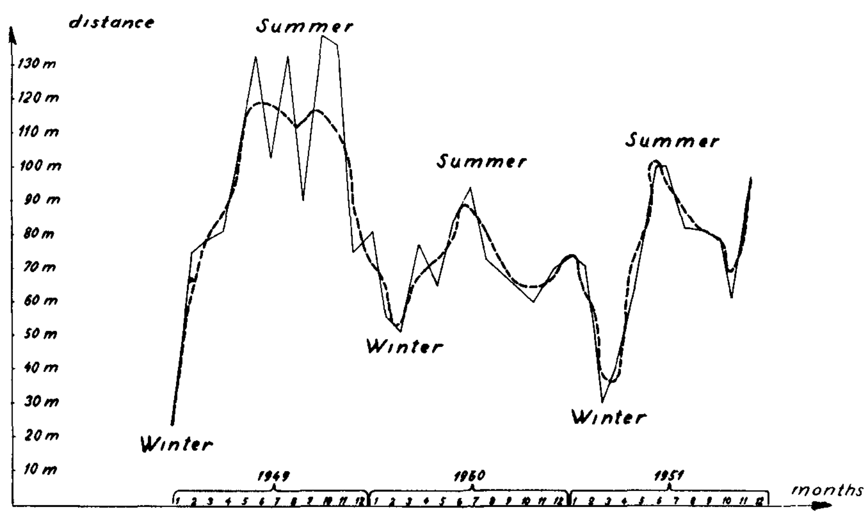


Fig. 23. Seasonal fluctuations of the shoreline on the updrift side of a groin on the Lime Inlet Barriers.

It is, therefore, of great importance that the energy-absorption of the stem is increased.

Laboratory investigations with an angular groin. - The investigations were carried out in the Hydraulic Laboratories in Copenhagen (1951). The length of the tank was 44-ft., water depth = 1-ft., wave length L_o = 5 1/2-ft., wave height H_o = 3-in., i.e. H_o/L_o = 0.045 (storm-wave) and wave period = 1.05 sec. Sand with an average size of 0.22 mm, representing shingle, was used. The coefficient of uniformity ($d60\%/d10\%$) was 2.

In the tank an experimental groin consisting of plates was erected. The angle between the wave direction and that of the groin was 15°. The length of the groin outside the shoreline was 4-ft. Before the tests a beach profile with a slope of 1:10 was constructed.

Fig. 24 shows the normal influence of a groin, accumulation on the up-drift side and erosion on the down-drift side. In Fig. 25 the groin is provided with an angle of 2-ft. as well as a rubble mound, and it can be seen that the shoreline has prograded on the leeside. Fig. 26 shows the groin with a 6-in. angle, and here the shoreline has almost the same position on both sides of the groin.

Development of the angular groin in detail.- There was, however, a decided defect in the structure, as considerable erosion repeatedly appeared at the bottom of the end of the angle. The erosion indicated that there was an increase in the shearing stress, which is of course disadvantageous, and the erosion so influenced the beach profile that its stability was reduced and the shoreline retrograded. Furthermore, the erosion is undesirable for the construction, since it may cause the rubble mound or other constructive elements to give way, which will make its maintenance expensive.

The erosion is due to a submergence of trajectories, caused by their bending around the end of the angle, while at the same time the velocity of the water particles is greater at the surface than at the bottom. This is explained in detail by Professor Tison in Gent, Belgium, who has worked out a theory about erosion at bridge-piers (see (31) pp. 3-5).

The erosion is avoided by turning the angle-end outward, so that the trajectories actually move upwards, cf. Fig. 27. It is also important that reflection from the outside of the angle is reduced as much as possible.

Fig. 28 shows a groin with its angle-end turned outward, radius 5-in.; when the angle-end is made in this way, no erosion will occur at the bottom. But if the angle is turned so that the end bends inward, erosion will start at once.

There was another weak point. A strong retrogradation of the shoreline appeared on the up-drift side right along the stem of the groin. This is due to a concentration of wave-energy along the stem, because the waves arrive slantwise, and, therefore, the wave height and the steepness-ratio is increased which causes the beach to be lowered. This is also known

Fig. 24. Laboratory experiment, normal groin, erosion on the lee-side.

Fig. 25. Laboratory experiment, angular groin, 24-in. angle.

Fig. 26. Laboratory experiment, angular groin, 6-in. angle.

Fig. 27. Laboratory experiment, angular groin, inward-turned groin-head.

Fig. 28. Laboratory experiment, angular groin, the angle-end turned inward.

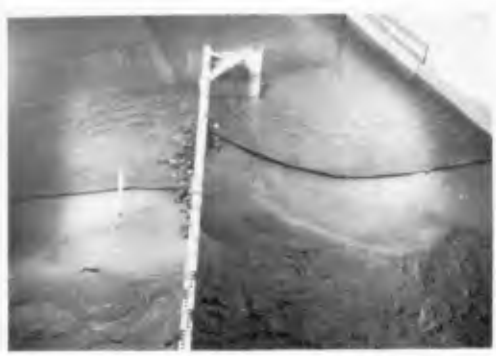


Fig. 29. Laboratory experiment, angular groin with a rubble mound along the stem.

in practice where a smooth stem often has bad results, while a stem roughened with inclined piles works better.

In order to neutralize the tendency of retrogradation along the stem of the groin, the up-drift side and part of the leeside, cf. Fig. 29, are supplied with a rubble mound 1-2 in., which is especially satisfactory on the up-drift side.

From investigations concerning the construction of lateral works it appeared that horizontal sleepers provided particularly well the uneveness needed to reduce the wave-energy (see (13)). When the interstices in a rubble mound are filled with water, it works by its roughness alone and is in this respect not very suitable. An uneveness achieved by piles will work in the same way. In a supplementary investigation this was shown in the following manner:

I built a canal 5-ft. wide, which in a distance of 12-ft. was equipped on both sides with 3/4 x 2-in. piles and at intervals of 16-in. The water depth was 10-in., and a wave was lead through the canal, (H_o = 4-in., L_o = 10-ft., H_o/L_o = 0.033 representing a moderate storm-wave). It was found by measuring the wave height before and after the passage past the piles that the height was reduced 30%, i.e. a 50% loss of energy. This is probably due to the eddies between the piles. A rubble mound 1-ft. tall, with a slope of 1 in 1 on both sides of the canal, was thrown up in the same section as the piles, and the rubble mound of 1 to 2-in. shingle rose 2-in. above the water level. The experiment was repeated and showed that the energy-loss was about 50%, although the decrease of the cross-section of the canal in this case was greater than with the piles.

The energy-loss mainly caused by eddies between the piles is comparable with the friction-loss on a sand bottom corrugated by ripple marks, cf. Bagnolds experiments (see (1) and (18)).

THE Z-GROIN

The Z-groin is an angular groin where the angle is a part of the stem giving the groin a Z-form. It has all the properties of the angular groin. In Fig. 21 a Z-groin is shown schematically. One will see that the groin is produced by shifting the angle into the middle of the stem, which means that the Z-groin for that reason alone is going to be cheaper than the corresponding angular groin. The wave action at an arbitrary point, x,y, can be calculated in the same way as with the angular groin. In the figure lines are drawn through points with the same diffraction-coefficient, with the assumption that the depth is constant, that there is complete absorption at the groin (the length of the stem between the initial coastline and the coast-parallel part of the groin is L), and that there is no friction-loss at the bottom. As one will see a "double" diffraction will occur at the groin, as soon as the waves arrive slantwise from the up-drift side, first at point a and later at point b, although the former is fairly unimportant. The form of the Z-groin will cause the shoreline to be drawn further out on its leeside than will the corresponding angular groin; thus the land-end is better protected. With the Z-groin there is, furthermore, a possibility of making the section a-b of the stem rough so that the effectiveness of the groin, for that reason also, is increased.

At the same time the section b-c is very much effected, but if the waves do not break exactly at b-c, the force cannot be much different from the force in the angular groin, as the angle that the wave crest makes with the stem of the groin hardly ever will be less than about 60°.

The Z-groin is, as the angular groin, rounded at the angle-end b in order to reduce erosion of the bottom. The corner c ought to be sharp, or, if not, bent outward in the same way as b with the stem of the groin moved some distance towards the middle of the groin.

The rubble mound at b-c decreases the reflection, and it is not necessary that the wall be absolutely tight; but the waves passing through the structure must not interfere with the diffracted waves so that waves with a steepness-ratio greater than that of the constructive type are produced.

Laboratory-investigations with a Z-groin. - Fig. 30 shows an experiment with a Z-groin. Similar experiments with the corresponding angular groin proved that the Z-groin in conformity with the diffraction accumulates better than the angular groin on the leeside, and the danger of erosion on the up-drift side is less than with a smooth-stemmed angular groin, as part of the wave-energy otherwise running along the groin, is now destroyed by the part parallel with the coast. In Fig. 31 the Z-groin is made rough by the use of piles; (1 x 1-in., space 4-in) this proved to be effective.

SUMMARY, LABORATORY EXPERIMENTS

The experiments proved:

a. that the angular groin caused the shoreline to prograde both on the up-drift as well as the down-drift side. This fact allows an increase of space between the groins. Consequently, the head of the groins must be placed on the leeside, where the ice from the windward cannot be destructive;

b. that erosion at the angle-end can be limited by suitable rounding;

c. that a rubble mound probably may be replaced by uneveness along the stem, but in that case there ought to be a bottom protection;

d. that the Z-groin accumulates better right along the land-end than the corresponding angular groin.

EXPERIMENTS IN NATURE

Fig. 32 shows a small angular groin built by the Board of Marine Works in the Nissum Inlet, cf. Fig. 2. This groin is 50-ft. long, the length of the angle 7-ft. including the rounding at the angle-end, radius 1-ft. The wave height and wave length in heavy gales are about 1-ft. and 9-ft. respectively. The up-drift side of the groin is to the left, and

Fig. 30. Laboratory experiment, Z-groin, smooth.

Fig. 31. Laboratory experiment, Z-groin, rough.

Fig. 32. Practical experiment, angular groin, Thorsminde.

Fig. 33. Practical experiment, groin with an angle, the Scaw.

Fig. 34. Practical experiment, angular groin at Belle-Vue Beach, Copenhagen.

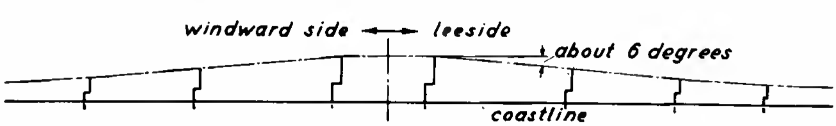


Fig. 35. Schematic group of Z-groins, arranged on the principle of shortening.

materials (sand 0.2 mm) are accumulated on the down-drift side quite as in the laboratory. The picture shows the groin at low water, but levellings on both sides of the groin show the greatest deposit on the leeside. Fig. 33 shows an angle on a groin at the Scaw, cf. Fig. 2.

Fig. 34 shows an angular groin constructed at Belle-Vue Beach, the famous seaside-resort near Copenhagen. The stem of the groin is 80-ft., the length of the angle 33-ft. It was built by Gentofte municipality.

One can observe many instances of effects similar to those of this groin, just discussed, f.inst. the deposits behind the Santa Monica breakwater in California (see (8) p. 5 and (20) p. 258).

PRACTICAL CONSTRUCTION OF ANGULAR AND Z-GROINS

Only a few practical experiments are known, f.inst. the breakwater groins in Italy and Spain and the T-groins in New Jersey.

The length of the angle should correspond to the dimensions of the groin. The ratio between the length of the angle and the length of the stem, provided that the gales are normal, cannot be given in general, as it depends on the intensity of the littoral drift, the angle of the waves with the shoreline, the beach profile, the size of the material, and the construction of the groin. Before more data are available for fixing a suitable ratio, the ratio 1/3-1/4 is practicable for most sand-beaches. In cases of ample littoral drift, the ratio may be reduced; for the contrary it ought to be increased, especially for places where an ordinary retrogradation of the shoreline takes place.

The angle may be a wall of sheet piling, supported by sloping piles on the inside and provided with a rubble mound in front, cf. Fig. 25; it may also consist of two rows of scattered piles with rubble. In the latter case one must make sure, by computation or experiment (see (6)) that the waves passing through the construction do not interfere with the diffracted waves so that they will be of the destructive type. The steepness ratio, therefore, must be $<$ about 0.02. In the usual groin-heads, built of piles with rubble, the loss of energy is, however, generally so great that one may completely disregard the wave-energy penetrating the construction.

The angle-end ought to be rounded outwards in order to decrease the possibility of erosion of the bottom at the angle-end. Practical experiences, as f.inst. an experimental construction at the coast-section concerned, should be used as a guide. The stem of the groin ought to be made uneven, f.inst. with the use of rammed coupled piles. In this case also, practical experience, on the basis of experimental constructions, should be the deciding factor in the type of construction.

CRITICISM

It could be maintained that the angular groin is generally more expensive than an ordinary groin. When T-groins and breakwater groins have, nevertheless, been built, it is because a beach must be produced.

Angular and Z-groins are particularly euited to the leeside-end of a group of groins and they will, therefore, probably be much cheaper than ordinary groins. All over the world one will find numeroue instances of the necessity for making expensive extensions of the land-ends becauee of eroeion on the leeside, ae we saw in Fig. 8. Theee lengtheninge of a land-end can be avoided by the use of angular groins or Z-groine.

It might be maintained that the angular groin producee merely a different distribution of the material in the group of groins. Thie is incorrect. When a groin does not accumulate any more material on the updrift side, it is among other things due to the fact that the eteepnese ratio of the waves is so great that the material is drawn out beyond the outer end of the groin and ie thus lost for the coaet. On the other hand material can accumulate on the leeside, if the groin is either an angular or a Z-groin.

CONCLUSION

On the basis of these hydraulic and practical experiments I believe that the most satisfactory group of groins ought to be conetructed as is shown schematically in Fig. 35 which illuetrates the uee of the principle of shortening with Z-groins. The coast is in thie way divided into small headlands.

The height of the groins outside the ehoreline must be fixed according to practical conditions (see (20) pp. 246-253); but the land-end must be as low as possible so that it starte functioning only under extraordinary depletions of the beach (eee (30) p. 155).

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CHAPTER 12

INTER-RELATIONS BETWEEN JET BEHAVIOR AND HYDRAULIC PROCESSES OBSERVED AT DELTAIC RIVER MOUTHS AND TIDAL INLETS*

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Price (1951) has stated:

> "(In) geological oceanography as practiced today there is constant emphasis on quantitative measurement and quantitative theoretical development."

As a demonstration of this geophysical approach to geological problems the authors have studied the theory of processes involved when sediment-laden water flows into a currentless, tideless, wave-free basin. The theoretical concepts developed have then been tested against the actual occurrences in nature. The result is that it is possible to propose a comprehensive theory of delta formation which may explain many of the features observed near mouths of rivers. According to this theory, there are three distinct and basic types of river inflow into a still basin, as shown in Figure 1:

a. "Hyperpycnal" inflow (inflow more dense) which results in a turbidity (density) current forming a submarine delta far below the water's surface at the foot of reservoirs and submarine canyons.

b. "Homopycnal" inflow (inflow equally dense) under appreciable hydraulic head. (Result is that a vertical frontal boundary forms between river and basin water and slowly spreads radially from the orifice, thereby causing an extremely rapid decrease in issuing velocity and the deposition of most of the sedimentary load within a distance of one diameter from the orifice. This depositional pattern results in the classical delta of geological textbooks with its well developed

*Contribution from the Department of Oceanography of the Agricultural and Mechanical College of Texas, No. 21. This paper is based upon research for the doctor's dissertation of the principal author (Bates). The opinions expressed in this paper are those of the authors as individuals and not to be construed as necessarily reflecting the official views of the Navy Department

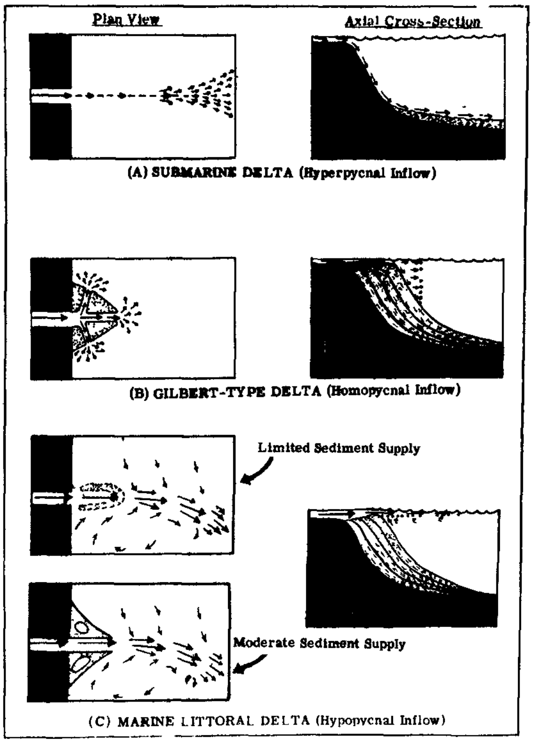

Fig. 1. Three basic methods by which sediment-laden water enters basins filled with water of differing densities.

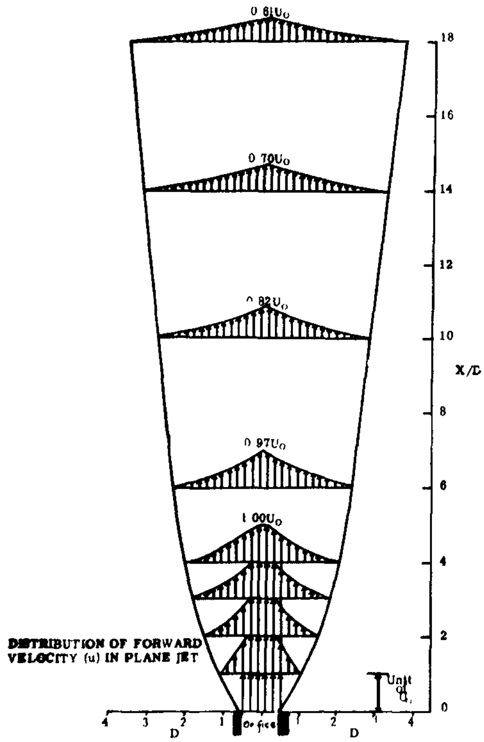

Fig. 2. Plan view of forward velocity vectors in a plane jet of the Tollmien type.

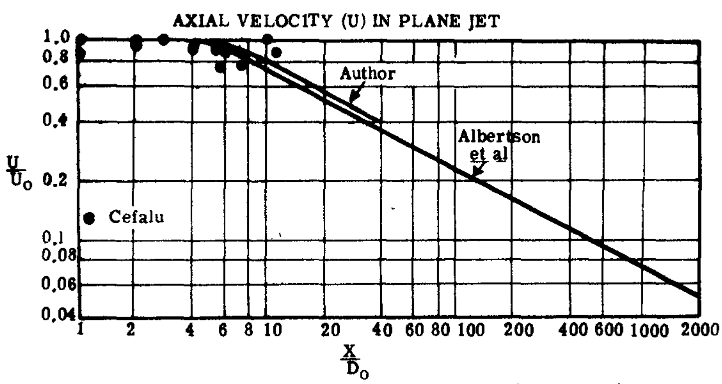

Fig. 3. Distribution of axial velocity in a plane jet according to theory and observation.

top-set, fore-set, and bottom-set beds. This frontal theory also explains the flow of Mississippi River water through Lake Ponchartrain when the Bonnet Carre Spillway is opened just above New Orleans).

c. "Hypopycnal" inflow (inflow less dense) with sediment-laden water flowing into a basin filled with water of greater density, such as a river emptying into an ocean.

The first two phases of this theory will be fully discussed in a forthcoming paper entitled, "A Rational, Quantitative Theory of Delta Formation". However, because of the interest of the coastal engineer in hypopycnal inflow, the general behavior of this type of inflow will be described in some detail here. In this particular case, a tongue of outflowing brackish or fresh water entering the ocean is, as it loses its hydraulic head, continually buoyed up by the salt-water below so that issuing velocity is maintained for appreciable distances beyond the orifice. This flow pattern strongly suggests that of the plane jet. Tollmien (1926) has developed a theory for the behavior of a turbulent plane jet moving solely under its own inertia and entering and entraining homogeneous fluid identical with that composing the jet.

By assuming that mixing length was a function only of distance away from the orifice, that profile similarity* existed throughout the jet, and that an expression could be applied from the Prandtl theory for turbulent shear stress, Tollmien obtained a theoretical streamline pattern for two-dimensional flow. Förthmann (1934), Peters and Bicknell (Rossby, 1936), and Albertson, Dai, Jensen and Rouse (1948) have found for plane jets that there is satisfactory agreement between the results of this theory and actual experiment. Even more extensive and just as successful experimentation has been done with the three-dimensional, axial jet by such aerodynamicists as Ruden (1933), Kuethe (1935), and Corrsin and Uberoi (1949), by chemical engineers (Taylor, 1950), and heating engineers (Nottage, Slaby, and Gojsza, 1952), to name but a few. All of the above workers assumed that the cross-sectional area of the jet must continue to increase continually downstream; however, Goldstein (1938) was able to prove that in the plane jet, the lateral streamlines eventually became nearly parallel downstream so that the boundary of this two-dimensional jet is parabolic. Such parabolic tongues are readily visible on aerial photographs of the outflow patterns of the Mississippi River and off tidal inlets of the Louisiana Coast. A typical velocity pattern is shown in Figure 2 based on the plane jet theory. The boundary of this jet is based on the parabolic margin of flow issuing from South Pass of the Mississippi River on October 14, 1940, during very low river stage when hydraulic head effects are at a minimum.

*Similarity in the curve velocity $\times \sqrt{\text{distance from orifice}}$.

The general equation for this parabolic boundary is:

$$y^2 = \frac{3}{4} x$$

In this jet, a zone of strong lateral shear forms a fraction of a diameter away from the orifice and extends outward to a distance of at least four diameters from the orifice. Leighly (1934) has pointed out that sediment is deposited parallel to these threads of high shear; thus, natural levee formation can be expected to begin a short distance out from the orifice and to parallel the core of high-speed flow. Theory and experiment both indicate that up to a distance variously reported as falling between 4.0 and 5.0 diameters from the orifice, the center-line velocity remains the same as the issuing speed and hence there is no pronounced deposition of sediment. Beyond this critical region, however, lateral turbulence from the flanks of the jet penetrates to the very core of the jet and the center-line velocity thereafter decreases exponentially. It is interesting to note that as long as the flow is turbulent, this fall-off rate is essentially independent of Reynold's number and it makes no difference whether the jet and entraining material is a gas or a liquid if the material's viscosity is low. Of the many possible sets of available distribution of axial velocity data for this type of velocity distribution in plane and axial jets, Figure 3 shows the findings of Albertson et al (1948) for a two-dimensional slot compared to values obtained by converting the three-dimensional values of Corrsin and Uberoi (1949) to the two-dimensional case, as well as to observations by Cefalu (1918) made with floats at the mouth of Southwest Pass of the Mississippi River. The conversion of the three- to the two-dimensional case was accomplished by changing the curve of U/U_0 from a function of D/x to a function of $D/\sqrt{x}$ in accordance with Rossby's (1936) discussion of this general subject. The terminology in this particular case is is that "U" represents the center-line velocity at any point, "U_0" the issuing center-line velocity, "D" the width expressed in diameters of a circular orifice or the width of a slot, and "x" the distance expressed in forms of D and measured along the jet's axis, the point of origin being the mouth of the orifice or slot. Geologically speaking, the agreement is good between these two distinctly different approaches to the same problem. Thus, deposition of suspended material may be theoretically expected to begin on the center-line of the jet somewhere in the zone of four to five diameters out from the slot, thereby creating a horizontal bar which links the two natural levees forming along the flanks of the jet. Such a pattern of deposition gives the familiar lunate bar observed in nature.

Interestingly enough, the principle that deposition in the path of the jet becomes important only when the above distance is reached out from the channel opening appears to operate in practically all cases of well-defined channels. Figure 4 illustrates only a few of the supporting examples contained in the files of the authors. In the case of the 4-foot road culvert near Bryan Beach, Texas, the scour-pit extended about 4.5 dismeters out from the culvert; the Golden Gate has the crest of its famous bar 3.8 diameters out; and according to a survey made in 1871 before man constructed jetties,

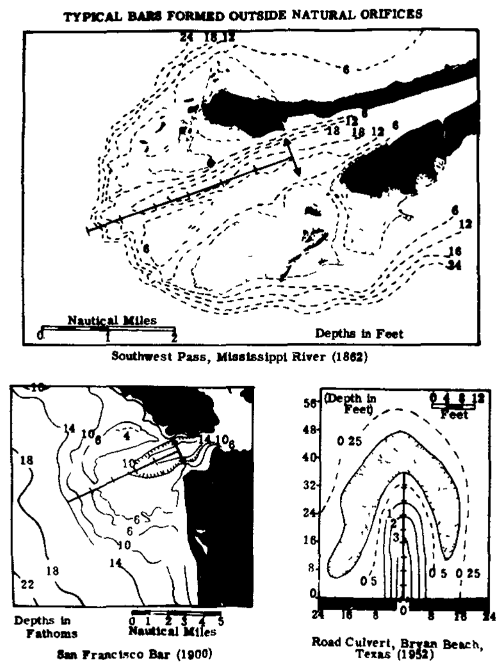

Fig. 4.

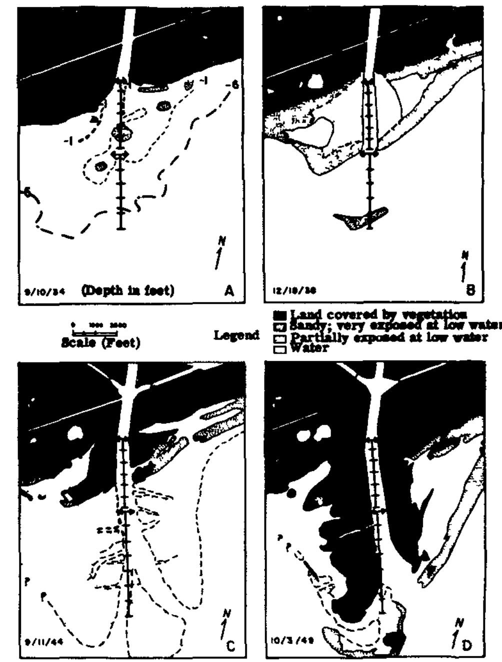

Fig. 5. Four stages of delta formation at the mouth of the Brazos River, Texas, following dredging of a new channel to the Gulf of Mexico in 1929.

the bar of the Mississippi's Southwest Pass formed about 3.5 diameters out from a line drawn between the seaward ends of high-water-marks on opposing river banks. In the latter case, it is most surprising that such a high-water mark is an important feature in defining the morphology of a river's mouth, but it may well be that the end of the high-water mark represents the orifice of a confined channel.

However, the formation of Δ-shaped deltas without distributaries, i.e., the cuspate delta described by Johnson (1919, p. 409), can likewise be shown to be a direct outgrowth of the depositional pattern associated with the Tollmien jet. The history of the rapidly-building delta of the new Brazos River near Freeport, Texas, is an excellent demonstration of the operation of this principle. The new channel was cut in 1929 and a hydrographic survey by the U. S. Corps of Engineers in July, 1931, shows a well-developed submerged bar whose crest formed about 5 diameters out from the river's mouth. Waves and high meteorological tides from hurricanes markedly affected this region in August, 1932, September, 1933, and July, 1934. The protruding mass of deltaic sediments was subjected to a marked concentration of wave energy because of wave refraction effects, so that by October, 1934, the bar had become roughly triangular in shape, linked to the land, and various portions stood above low water (Figure 5-A). In fact, the bar thereafter became sufficiently developed to form a barrier beach and the channel through which the river crossed this beach became the new orifice, probably after the occurrence of a nearby hurricane during June, 1936. Sketches based on a series of excellent aerial mosaics by Jack Ammann, Photogrammetric Engineers, Inc., permit the reconstruction of the subsequent history. By December, 1938, (Figure5-B) a new bar had formed blocking the channel at a distance of about 4.5 diameters in front of the original bar. A period of relative stability followed, and the site of this new bar soon became the point of channel bifurcation, while wave refraction, particularly active during a hurricane which passed over the area in August, 1942, molded the entire complex into even more of a triangle, as shown by photography in September, 1944 (Figure 5-C). Hurricanes crossing inland in this immediate region during August, 1945, and October, 1949, created barrier beaches bordering this deposit, so by late October, 1949, (Figure 5-D) there is again only one channel bisecting the cuspate delta. It should be noted that bays have formed behind the barrier beaches. On larger deltas, such embayments might be termed "delta-flank depressions", as in the case of Barataria Bay on the west flank of the present Mississippi delta, but their history shows that they are obviously the result of a combination of natural levee and barrier beach formation. Thus, the formation of a delta such as that of the new Brazos is obviously the result of a balance of forces between bar formation with resulting channel bifurcation <u>versus</u> wave action concentrated by wave refraction because of unusual bottom conditions.

INTER-RELATIONS BETWEEN JET BEHAVIOR AND HYDRAULIC PROCESSES OBSERVED AT DELTAIC RIVER MOUTHS AND TIDAL INLETS

At the present time, our research has not been pursued far enough to adequately develop a method for predicting the depth of channel over the bars deposited according to the above theory. However, there are several promising approaches, and by the next Coastal Institute, it is hoped that this problem will have been solved.

For almost a century the importance of the effect of the earth's rotation on stream flow has been a debatable question. This apparent deflecting force, appropriately named the "Coriolis force" after the French mathematician Coriolis, has been believed by some to be responsible for anomalous stream-meandering, a phenomenon considered to occur in accordance with "Baer's Law". The favored opinion has been that while the effect existed, it was so slight that other environmental factors always masked it. This opinion still prevails, as we have found by polling some of the world leaders in hydraulics and geomorphology, as well as by reviewing current literature in the fields of hydraulics and limnology. However, detailed oceanographic studies of estuaries of the United States during the past four years have indicated again and again that Coriolis force is strong enough to cause seaward flowing water to be deflected to the right-hand side of the estuary as it flows downstream. While aware of this world-wide effect as early as 1949, it was only during the summer of 1952 that an example has been located by the authors from the Southern Hemisphere. Figure 6 shows the cross-sectional profile of isohalines across the Hawkesbury River just north of Sydney, Australia, at two stages of the tide on July 14-15, 1952. These data have been kindly furnished by Dr. D. J. Rochford of the Australian Marine Biological Laboratory, and are believed to be the first to clearly illustrate the manner in which outflowing fresh water is shunted to the left-hand side of an estuary in the Southern Hemisphere. The opposite effect is known to occur in the Northern Hemisphere, as in the case of the James River where Pritchard (1952a) finds the fresh-water to move seaward along the right-hand bank. The theory of such asymmetry has already been given in detail by Pritchard (1952b); the importance of this phenomenon in harbor flushing has been discussed by Elliott, Tressler, and Myers (1953).

Coriolis force is also important at the mouths of at least major rivers such as the Mississippi. For example, Figure 7 shows that at the time of the first complete hydrographic survey (1859-1871) of the eight major outlets in the lower Mississippi delta, seven of the passes or bayous had their right-hand banks extending further seaward than did the left-hand banks; while the eighth pass (Northwest Pass) was questionable for it had only an unusual cluster of islands beyond the tip of the right-hand shore, which bank did not extend as far as the left-hand bank. Studies by the U. S. Corps of Engineers (1919) have also shown that this anomalous depositional pattern appears in isopach studies of sediment volumes deposited off the passes; for example, an area on the left-hand tip of Southwest Pass received but 72% of the volume of sediment deposited between 1867 and 1905 in an area of comparable size and location off the right-hand tip of the

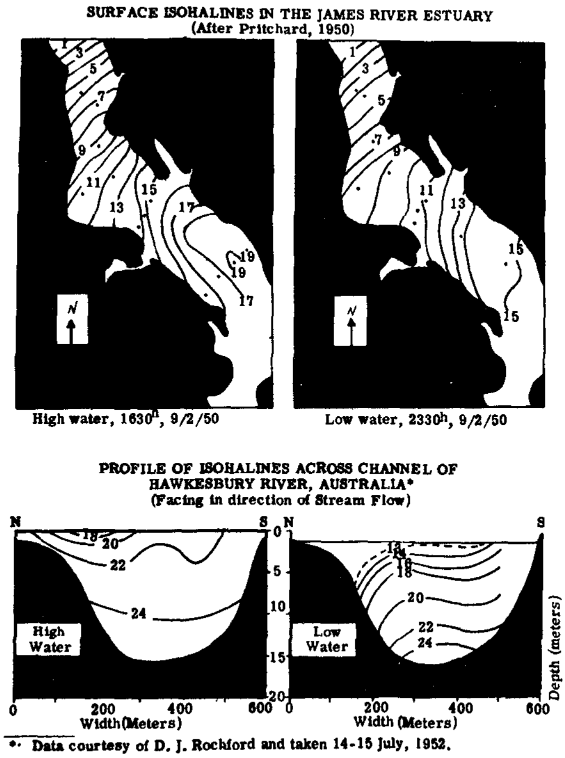

Fig. 6. Asymmetric distribution of salinity in estuaries of the northern and southern hemispheres caused by Coriolis force.

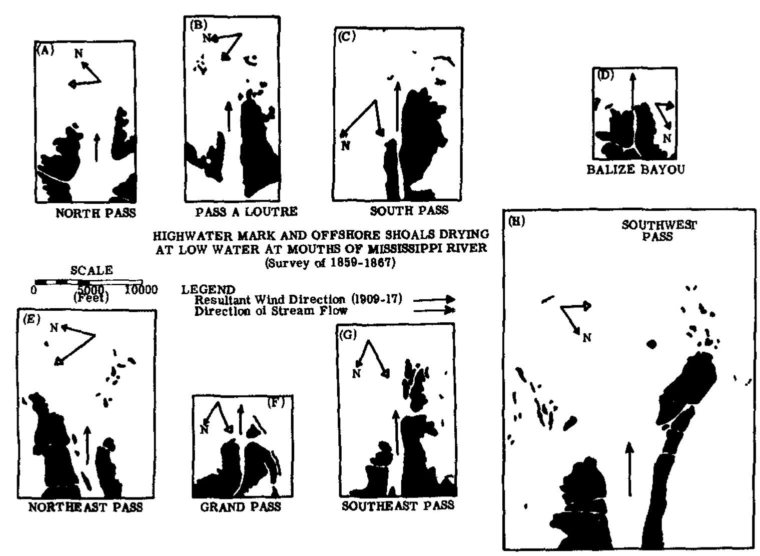

Fig. 7. Asymmetric formation of river banks at important mouths of the Mississippi River, the major causative factor probably having been Coriolis force.

pass. Such anomalies have been ascribed by the Corps of Engineers to a littoral current flowing from east to west created by the prevailing wind. Inasmuch as the resultant wind in this area is from 115° at a speed of about 3.2 statute miles per hour, it is obviously incapable of creating the littoral current reported. In the forthcoming geological paper on deltas, it will be demonstrated that the right-bank extensions are the result of two manifestation of Coriolis force, namely, a cross-channel slope and a deflection of the outflow as it leaves the distributary mouths.

This marked deflection of rivers to the right upon entering a basin was reported by Fehlman (1948) for Swiss lakes but he did not ascribe the effect to any particular cause. Study of this effect at the mouths of the Mississippi River indicates that the phenomenon is caused by Coriolis force and that it is widespread geographically, occurring at the mouths of such rivers as the Po, Danube, Volga, and Nile. Off the Mississippi and during periods of low winds, the turning appears to be at a more rapid rate than that calculated by using the arc of an inertial circle based on issuing velocity; on the other hand, the turning is at a slower rate than would be expected if the flow were a combination of littoral currents and jet flow. The turning is powerful enough, however, for a relatively fixed boundary to form between river and oceanic water five to ten miles off the eastern half of the Mississippi delta during flood stage. The main discharge of the distributaries presumably remains inside this boundary, the flow along this boundary being directed first to the southeast and then veering slowly to the west off the southern portion of the delta, so that this source contributes an important, if not major, portion of the water making up the east-west current present. However, aerial photographs taken by the U. S. Corps of engineers of the outflow from South to Southwest Passes, today's main navigational channels, show that both discharges take the shape of parabolic jets similar to the one shown in Figure 2. Both jets turn to the right, although their turning is obviously more limited in extent--that is, by a smaller angle than that observed off the passes to the north and northeast where the trajectory of the flow approaches an 180° arc. As soon as enough water has been banked against the coast to permit downslope gravitational force to balance Coriolis force and density patterns created by the flow of fresh water over saline water, the flow from South and Southwest Passes tends, at best, to only parallel the main shoreline (90° or a little more); frequently, the turning is not that great. Strong winds during "Northers", of course, deflect this surface flow and may temporarily direct it towards the south or southeast.

To conclude, it has been proposed that on the basis of modified jet theory it is possible to explain rationally and quantitatively much that occurs at the mouths of rivers emptying into the sea. These explanations are based on physical principles which have been checked in the laboratory and supported by field observations made throughout the world. Because the approach is quantitative, it may permit objective forecasting of future conditions at natural and artificial outlets that are of interest to the hydraulic engineer.

ACKNOWLEDGMENTS

The authors wish to aoknowledge various oourtesies extended during the preparation of this paper, partioularly those by the firm of Jaok Ammann, Photogrammetrio Engineers, Ino. of San Antonio, Texas, for supplying aerial mosaios of the new Brazos River delta; by Mr. John Lyman, Direotor, Division of Oceanography, U. S. Navy Hydrographio Offioe, who suggested the terms for the basio types of inflow; by Dr. D. J. Roohford of the Marine Biologioal Laboratory at Cronulla, New South Wales, who kindly supplied raw salinity data obtained from Hawkesbury River in July, 1952; by the New Orleans Distriot Office, U.S. Corps of Engineers whioh has made available large amounts of data; and by Dr. W. A. Prioe and Mr. R. O. Reid of the Department of Oceanography, Agrioultural and Meohanioal College of Texas, who disoussed in detail oerta. of the oonoepts proposed in this paper.

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Part 3

DESIGN OF COASTAL WORKS

Chapter 13
SPANISH PRACTICE IN HARBOR DESIGN

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Highway, Waterway and Port Engineers
Madrid, Spain

Up to a relatively short time ago maritime works, especially costly works of harbor protection, were designed merely by intuition or by misleading comparisons with other works seemingly identical, but lacking the application of appropriate scientific analysis.

Just as in what might be termed "land" design there were solutions, or methods for arriving at reasonably approximate calculations for the majority of practical cases, in "maritime" design all, or almost all, was yet to be developed. Most of the existing texts dealing with the subject were simple summaries with comments upon works previously constructed. According to the good or bad fortune experienced, efforts were made to explain the reasons for the success of some, or the failure of others, but there was a lack of the required methods for calculation according to engineering techniques.

It must be stated that following the development of the original formula in 1933 for the calculation of rock fill dikes by Castro and Briones, the Spanish contribution to maritime design was continued with the initial application of the system of wave front pattern to the port of Motrico in the designs composed by Iribarren in 1932 (1)* and in 1936 (2), where the reflections were also taken into account.

This fundamental method of wave-front pattern**, later called "refraction" by others, by means of which the direction and characteristics of waves may be determined with sufficient practical approximation in any area whose depth contours are known, was first published by Iribarren, January 1941 (3)***. In this article a study also was made of the lateral wave expansion**, later called "diffraction" by some engineers.

*Numbers refer to references. A complete set of these papers was provided at the Third Conference on Coastal Engineering and are on file for loan purposes at the Hydrodynamic Laboratory, Massachusetts Institute of Technology, Cambridge, Mass. (Editor's Note: Since considerable editorial changes have been made in the author's original manuscript, the reader is encouraged to consult the original paper when certain statements are challenged.

**We prefer to continue using our own definitions regarding the undulating movement of gravity-waves, in which we are highly interested, to those dealing with other vibratory movements such as those of light and sound. For although there is indeed some similarity, they also differ with the waves.

***The English translation of the title is "Harbor protection works - wave front pattern". This article was published in English by "The Dock and Harbour Authority" November and December 1942 (4). In French by "Annales des Ponts et Chaussees", September - October 1945. In Portuguese by "Technica", 1945. It is necessary to point out that the date of publication of an article is that of the original and not of the translation in other countries.

The Spanish report by Iribarren and Nogales presented at the XVII International Congress of Navigation in 1949 (5), constitutes a more recent, more up-to-date study of the subject. It is recommended that this report be read, whether in French or in English, by all interested in the subject. Sections of this publication are presented as follows:

In Section (a) Wave characteristics, a discussion is presented on the determination of the "maximum" wave. This wave height is somewhat higher than the one which may occur in any sea or ocean as a function of the fetch, and, if a closer approximation is desired, of the wind pattern. The results obtained in this presentation coincide, with a suitable margin of safety, with those published later by the U.S. Navy Hydrographic Office in "Wind Waves and Swell: Principles in Forecasting".

In Section (b), Wave front pattern, the method of calculating the wave front pattern of the maximum wave is presented. By this method the direction of the waves can be determined for any point where construction work is planned.

Section (c), Height of the wave, deals specifically with the determination, as a first approximation, of the maximum possible wave height at any point in the wave front pattern.

Section (d), Special cases, lateral expansion, deals with the expansion or diffraction of waves. This phenomenon is of utmost interest, especially in the determination of the extent of harbor protection. This procedure in the second degree of approximation was presented in a later publication (6). It is of interest to note that an extensive test is being conducted, and it is expected that the results will be available in the near future. It appears that the author's method of considering diffraction and the resulting change of wave height along the crest is an accurate method.

Section (e) is devoted to the application of the methods to the Port of Palma in Majorca, and section (f) treats of the problem of diffraction at a breakwater gap. Section (g), deals with the reflection of waves, and section (h) deals with this same problem, in the first approximation, but with the breaking of the waves. In addition it treats the problem of the critical slope between incipient breaking and reflection of waves which also is published elsewhere (7,8).

Chapter II of Reference 5 deals with the variation in the height of the wave on the hypothesis of conservation of energy. It proves the acceptability of the first approximation in studies involving the wave front patterns.

Chapter III of Reference 5 starts with the study, in a preliminary manner pending further work, of the wave front patterns in the second approximation. The problem recently has been presented under the title

"Wave front patterns in the second degree application" (6)*. These studies in second approximation are now being revised and will be published shortly.

Chapter III of Reference (5) also deals with the different ways in which swell is produced in both deep water and shallow water. Also awaiting early publication is a study which confirms the increase of the period of the swell as it advances from the sea toward the shore. This increase of period is generally of little importance in regard to the wave front patterns studied, particularly for relative shallow waters where harbor works are projected. This work shows that the swell produced by the wind in deep water is composed mainly of two complex wave trains whose bisector of their directions is the wind direction, as well as that of the advance of the protuberances, or short crested waves.

The wave front patterns in the first approximation, at a reduced margin of safety, constitute in effect a simple but proportionate and orderly plan of analysis. It does not consider the actual complex movement of the sea, but considers the characteristics and direction of advance of the maximum protuberance, which are of interest for the practical calculation of maritime works.

If it is so desired, one may determine separately the wave patterns from the two mentioned wave trains whose directions of advance tend to coincide with the line of maximum slope of the bottom as the depths in the direction of propagation are reduced. The length of the crest, or protuberances, formed by the coexistence of both wave trains, consequently is increased. This may have some relation with the lateral expansion or diffraction of the waves, thus increasing the angle of penetration of the waves towards the inside of the harbor in an identical form to that of the first approximation, which is generally and but for some exceptional cases, sufficient for practical applications.

In Chapter IV of Reference (5) a study of the currents and oscillations of the surge action inside of harbors is presented. This problem also is discussed in Reference (9). These surge actions were defined as the movements which were produced in the sea during storms of much longer period than the waves. The surges might even exceed a period of three or four minutes. After determining that these movements may be amplified, impairing the usefulness of the inside of the harbors, due to resonance, and after proving in a manner that can be calculated, that the period of oscillation pertaining to a dock or zone within the harbor is a function mainly of its length and depth, it is therefore concluded that by varying the length when planning the work of a harbor, or the depth generally by means of dredging after it is constructed, the period of the oscillation

*Regarding this question the article by Fernando Rodriguez Perez published in the Revista de Obras Publicas, July 1946 also should be consulted.

or surge action in the sea can be controlled, thus reducing greatly the dangerous currents or surge action. This has been done successfully in several Spanish ports. By means of calculations it is shown also that the apparently smooth slopes of the beaches may permit reflections of the surge action. An explanation is presented on how these waves might originate from the beats of the wave groups to which their periods approximate.

Chapter V of Reference (5) is concerned with the study of the conditions of access to the harbors. This factor usually is contrary to that of its protection, since to improve the former means making the latter worse, or vice versa. It is necessary to study them jointly in such a way as to make both conditions, that is, protection and access to the harbor, if not perfect at least acceptable.

After calculating the maximum possible wave characteristics relating to the section of the coast under consideration, it is necessary to calculate upon this basis, the cross-sections of the dikes of protection. This protection can be constituted by means of the reflection and consequent dissipation of the waves and their energy to the sea, or by means of their breaking. Publications have been made relative to the calculation of both types of dikes. One is concerned with calculations for vertical walls (10, 11), and the other is concerned with rock-fill dikes (12, 13). Both were published in 1938 by Iribarren by the Bermejillo Press, Pasajes, Spain.

In the first of these which concerns vertical dikes, the diagram of the pressure of wave reflection is determined, which is a basis for the calculation of the stability of the structure. This diagram, whose approximation to real values has been proven in practice, is unique in that it is only one that, in accordance with experience*, as the steepness increases, the amplitude of movement on the wall also increases, becoming indeed much larger than double the incident wave height.

In the second of the above publications, which concerns the formula for the calculation of rock-fill dikes, a method is presented for determining the weight of natural stones or artificial blocks, which are necessary for the stability of the structure. This weight is a function of the height of the wave, the side slope of the dike, and the density of the material which constitutes the stones or blocks. A generalization of the formula for the calculation of rock-fill dikes and the verification of its coefficients was published in 1950 (15, 16, 17). Besides presenting a method of calculation for generalizing the formula to apply to the submerged zones of the structure, verification data are presented from observations on the breakwater at the port of Argel. A new statement of this formula appears in the following chapter.

*See the illustrations 13 and 21 in Reference (14)

For some years past in the School for Highway, Waterway and Port Engineering in Madrid, there has been taught the method of determining the height of the protecting works or of their parapets, their width at the top, the thickness of the protective covering, and the grading of the sizes of the stones. Also considered are the influence of the angle or direction at which the waves approach the breakwater and other construction details of such important and generally costly work of protection of the harbors*, as well as of wharves.

All that has been discussed above, and much more that cannot be given here in detail, will be included in a publication entitled "Tratado de Obras Maritimas" (Treatise of Maritime Works) whose first volume is now being printed.

The sand deposits carried by currents constitute one of the gravest dangers to many harbors. This interesting problem has been studied especially in regard to swell, whose action was less known than that of sand moving forces of other origin. As a result of this study an article on currents and the sweeping of sand due to swell has been published (18, 19, 20). Although the main or direct purpose of these studies was to determine the form of the work for harbor protection in order to avoid or diminish the danger of sand deposits, they also may be applied in the opposite sense; that is, to produce sand at the beaches or shores (21). This study demonstrates that, as the bottom configuration affects the form of propagation and characteristics of the waves that may be determined from the wave front patterns, the waves exercise in turn a decisive influence upon the shape of a movable sandy bottom. The form of the bottom may be modified in certain cases by changes in the wave characteristics which are induced by proper location of proposed structures.

Another study in progress, based mainly upon the deformation of waves in shallow water, deals with the subject of sand deposits. The results of this study, though nearer to the physical reality of this phenomenon, differ but little, in general, from those obtained by means of the process mentioned above.**

We consider the application of small scale models to maritime design problems as highly delicate, primarily because of their complexity, but also because of the excessive scale reduction which is necessary in the majority of cases. In the complex model, however, certain details can be treated separately with models of larger scale. Of interest in connection with model studies is the wave generating apparatus of the Harbor Laboratory of the School for Highway, Waterway, and Harbor Engineers (22).

*The article "Diques de abrigo en puertos" by M. Martinez Catena published in Revista de Obras Publicas, July to October 1941, should be consulted.
**The following articles should be consulted: Una comprobación de la utilidad de los planos de oleaje en el proyecto de obras para la regeneración de Playas" - Revista de Obras Públicas - September 1947 and "Notas para el proyecto de las obras para defensa y regeneración de costas-Comprobaciones en el litoral N.E. de la Península" Revista de Obras Públicas - August and September 1948, whose author is Aurelio Conzalez Isla.

This mechanism permits the wave period to be continually changed as well as the motion of the upper and lower "throw" of the wave flap, by means of which may be produced in a continuous manner and without interruption all kind of waves - from those of pure oscillation to those of translatior

Recently a study of the violent and accidental pressures produced by waves breaking on a structure was submitted to the "International Committee for the Study of Waves". It was shown that the causes of these pressures, as well as their importance, are due to their relatively small duration (23). Usually structures must be designed for ordinary pressures which are more prevalent, though less violent, by the method presented at the School for Highway, Waterway and Harbor Engineers in Madrid. Diagrams for ordinary pressures of breaking waves are to be included in the first volume of "Tratado de Obras Maritimas" mentioned above.

It should be noted that all the above mentioned studies are of highly practical character, and due mainly to the complexity and initial variability of the real waves, no pretension is made to a utopian theoretical precision, but instead to plain practical approximations. Besides the practical studies discussed above, mention is also made of a more speculative phenomenon - the oscillatory-centrifugal theory of tides (24).

We are grateful for this invitation to summarize the Spanish practice in harbor design. Although this summary is quite short, it is hoped that it will help to,establish fruitful collaboration with our colleagues in the United States. In regard to these complex subjects one cannot, nor must, improvise anything. The collaborations that tend to revise or suppl ment methods, like those which confirm the ideas discussed above, are very important. During the short time at our disposition it has been possible for us to prepare a paper on a new confirmation of the formula for calculation of rock-fill dikes. This paper constitutes the following chapter. It is interesting to note that starting from the study made by Kaplan (25) we have fully reconfirmed our formula which dates from the year 1938.

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Chapter 14

NEW CONFIRMATION OF THE FORMULA FOR THE CALCULATION OF ROCK FILL DIKES

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In the paper entitled "Una fórmula para el cálculo de los diques de escollera" (A Formula for the Calculation of Rock Fill Dikes) published July 1938* there was presented the expression,

$$P = \frac{NA^3 d}{(\cos\alpha - \sin\alpha)^3 \ (d-1)^3}$$

where

P = weight of the individual stones or blocks in kilograms
N = 15 for dikes of natural rock fill
N = 19 for dikes of artificial block fill
A = $2h$ = total height of the wave that breaks on the dike, measured in meters
d = specific weight of material of the stones in tons (metric) per cubic meter
α = angle of the dike's side slope with the horizontal

Before the preliminary determination of those tentative values of N, each based only on a single observed case - the natural rock fill dike of Orio and the artificial rock fill dike of San Juan de Luz, - the publication mentioned above stated that:

"It only remains now to determine the coefficient N and verify if it is sensibly constant, as seems inferred from the material submitted, or varies with the other elements of the formula.

"In the worst case, a coefficient similar to the classic and variable coefficient C of the formula of uniform flow, $V = C\sqrt{RS}$, will be considered."

In spite of the fourteen years intervening, during which the reasoning followed for the deduction of the formula has been refined, in a manner that might have been advantageously taken into account by the translators of the paper, the coefficients 15 and 19 still stand, due to the satisfactory results always obtained in numerous cases of practical application.

*Translated and published in the January 1949 Bulletin of the Beach Erosion Board, Office Chief of Engineers, Department of the Army, Wash. D.C. The coefficient K of that publication is denoted by N in this paper to avoid confusion with $K = \coth \pi \frac{H}{L}$, and similarly, the angle "a" we here use α.

The formula is actually derived for the upper slope of the dikes, and a generalization for all the depths of the structure was indicated only tentatively at the end of the paper. In this connection substantially the following was stated:

"This generalization of the formula assumes a certain margin of security, but it is not logical to apply on the sea the strict results obtained from theoretical formulas when on land it is usual to multiply them by ample safety factors.

"I should be most grateful to my colleagues who, acquainted in detail with concrete practical cases, would kindly furnish me information for refining the coefficients."

As a result of the ideas presented in the first publication of 1938 it is possible to refine somewhat the coefficients in the article entitled "Generalización de la fórmula para el cálculo de los diques de escollera y comprobación de sus coeficientes", (Generalization of the Formula for the Calculation of Rock-fill Dikes and Determination of its Coefficients) published in May 1950* in the Revista de Obras Publicas. On comparing them with results with the established experience obtained from the Argel dikes, the degree of approximation sufficient for practical application was definitely confirmed.

Besides the practical confirmation of the coefficients mentioned in the article of the Revista de Obras Publicas, the composition of the formula was verified. In this connection it was substantially stated:

As was a matter of record, in the XVII International Congress of Navigation held in Lisbon, it is satisfying that the formula deduced in the American report of Epstein and Tyrrel for reflecting rock fill dikes, starting from our expression for pressures of reflection $P = \frac{CV^{**}}{g}$, is similar to that which was obtained in 1938 for breakwaters.

In effect, the American formula is:

$$P_t = R_t \frac{s\,H^3}{(s-1)^3\,(\mu - r)^3}$$

and ours:

$$P = \frac{NA^3 d}{(\cos\alpha - \sin\alpha)^3\,(d-1)^3}$$

*This article was translated and published in the January 1951 issue of The Bulletin of The Beach Erosion Board, Office Chief of Engineers, Department of the U.S. Army, and by the Waterways Experiment Station; Translation No. 51-2.

**See the publication "Cálculo de diques verticales" (Calculation of Vertical Dikes) published 1938, translated and published in French and in English in the "Bulletin de l'Association Internationale Permanente des Congrés de Navigation", No. 23, July 1939.

in which

$P_t = P$ = weight of the individual stones
$H = A = 2h$ = wave height
$s = d$ = relative density of the material
μ = natural batter of the rock fill $\simeq 1$
$r = \tan\alpha$ = slope of the rock fill
R_t and N are coefficients.

Expressing the American formula in our notation, it becomes:

$$P = R_t \cos^3\alpha \quad \frac{A^3 d}{(\cos\alpha - \sin\alpha)^3 (d-1)^3}$$

which is ours, except only that it includes the factor $\cos^3\alpha$ in the coefficient.

It is a matter of record that on establishing our formula it was indicated that the coefficient should vary with the data of the problem. Practically the angle α, which varies most for the upper part of the dike, will not vary much, for from $\cot\alpha_1 \simeq 3$, corresponding to present rock fill dikes, it cannot get much steeper than $\cot\alpha_2 \simeq 2$, even in the reflecting dikes, because of the enormous weight of the stones this requires. Between those maximum limits the relation is:

$$\frac{\cos^3\alpha_1}{\cos^3\alpha_2} \simeq 1,2$$

which would represent only a small difference in the weight of the stones, and even less in their size, whose relation would be $\sqrt[3]{1.2} = 1.06$. Only direct observation can determine properly N or R_t, including cases of very steep slopes.

Another interesting confirmation of our formula is given now in the article entitled "Notes on Determination of Stable Underwater Breakwater Slopes" published by Kaplan in the Bulletin of the Beach Erosion Board for July of the present year.* In this article, starting from Blanchet's formula for the destruction of stone piles by a current**

$$v = K_1 \sqrt{2g \frac{w_1}{w}} \sqrt{D} \sqrt{\sin(\alpha_0 - \alpha)}$$

in which

v = velocity limit for the disintegration of the stone pile
K_1 = a dimensional coefficient which should be constant for stones of a given shape

*Kenneth Kaplan, "Notes on Determination of Stable Underwater Breakwater Slopes", Bulletin of the Beach Erosion Board, Vol. 6, No. 3, p. 20.
**Ch. Blanchet, "Formation et destruction par un courant d'eau des massifs en pierres", in "La Houille Blanche", March 1946.

D = lineal dimension of the stones
w = specific weight of the water
w_1 = specific weight of the stones
α = angle of slope with the horizontal
α_0 = angle with the horizontal of the stones' natural slope

Calling W the weight limit of the individual stones, "a" the major axis of the corresponding orbitary ellipse and T the wave period, Kaplan gets the formula:

$$\sin(\alpha - \alpha_0) = \frac{Ka}{W^{1/6}\ T}$$

where the value of the coefficient K is a function of the acceleration of gravity "g" and the specific weights of the water and of the stones.

Actually the case which is of most interest is that concerning submerged stones, or stones entirely enveloped by water. In this case it is necessary to take into account, besides the hydrodynamic thrust, the vertical hydrostatic force. It is desirable to substitute the specific weight w_1 of the stones, without deviating from Blanchet's formula, by $w_1 - w$ corresponding to these submerged stones. Therefore the formula becomes:

$$v = K_1 \sqrt{2g\,\frac{w_1 - w}{w}}\ \sqrt{D}\ \sqrt{\sin(\alpha_0 - \alpha)} \qquad (1)$$

Following an identical procedure of our observations of 1938 and 1950, the weight of the stones is expressed as

$$W = K_2 w_1 D^3 \qquad (2)$$

as well as the maximum orbital velocity of the water

$$v' = \frac{\pi a}{T}$$

which definitely is supposed to admit also, with a certain degree of practical approximation for such complicated problems, the application of the trochoidal theory, even in the case of steep slopes.

It is important to remember that in our first report of 1938, besides deducing the formula for the direct action of the wave breaking upon the breakwater, the same formula was also deduced, thus confirming its generalization, upon the basis of the water's descent on the slope at a reduced velocity due mainly to the roughness and permeability of the rock fill. Hence, following our own previous deduction it may be stated that

$$v = K_3\ v' = K_3 \cdot \frac{\pi a}{T} \qquad (3)$$

Thus from the equations (1), (2), and (3) the formula at once is obtained, this being duly corrected, in which

$$\sqrt{\sin(\alpha_0 - \alpha)} = \frac{K_a}{W^{1/6}\ T} \tag{4}$$

where the coefficient K, also duly corrected, becomes:

$$K = K' \left(\frac{w}{g\ (w_1 - w)} \right)^{1/2} w_1^{1/6} \tag{5}$$

in which K' is dimensional.

As was deduced from our publication of 1938, the maximum velocity of the wave breaking over steep slopes, being theoretically equal to its velocity, the following is obtained

$$C = v' = \sqrt{g \frac{A}{2}} = \frac{\pi a}{T} \tag{6}^*$$

From the expressions (4) (5) and (6), in connection with our definitions $W = P$; $w_1 = d$; $w = d_1$ and the angle of the natural slope being approximately $\alpha_0 = \frac{\pi}{4}$ the following expression is readily obtained

$$P = K'' \frac{A^3}{(\cos\alpha - \sin\alpha)^3} \cdot d \left(\frac{d_1}{d - d_1} \right)^3 \tag{7}$$

which is definitely our formula; since this expression is obtained likewise readily and without introducing previous simplifications, for any one of the processes followed in our report of 1938, in which K" is likewise dimensional.

If, in order to simplify the applications, we define the specific weights in tons per cubic meter, with which they are reduced to the relative densities, and that of the water, salt water included, being practically $d_1 = 1$ the formula is reduced to

$$P = \frac{NA^3 d}{(\cos\alpha - \sin\alpha)^3 (d-1)^3}$$

which is our plain, convenient practical formula once more confirmed in an interesting way; whose degree of approximation is more than sufficient in such complex problems, where only a slight variation in the height of the wave might give rise to greater variations than those resulting from this degree of approximation; which might also be increased by simply refining the coefficient in the manner already demonstrated in the first report of 1938.

*Actually this result does not apply to calculations in a contrary sense for the generalization of the formula for submerged slopes.

CHAPTER 15

SOME ASPECTS OF SHORE PROTECTION IN BOSTON HARBOR

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INTRODUCTION

At the end of World War II, the newly created Port of Boston Authority was faced with embarking upon an extensive belated shore protection program. The funds for the projects were approved by the State Legislature based on pre-war estimates and with the proviso that the affected town or city must pay an equal amount towards the total cost of such work. In the face of rising costs of labor and materials, it was obvious that appropriations were inadequate to permit construction of the proposed seawalls unless a more economical type could be found. This was primarily the reason, along with a natural desire to gain more knowledge and obtain basic design criteria, that the Authority made a comprehensive study of existing shore structures along the Atlantic coast.

I personally became so wrapped up in the subject that I spent one whole summer traveling around on my own time during week-ends. It is a very fascinating subject, I assure you, because of its endless and unknown problems which tax one's ingenuity and knowledge for a solution.

The survey and study indicated a wide range of thinking as to basic concepts of shore protection engineering, design and construction materials. I do not wish to be critical, but the survey did indicate a need for more dissemination of the latest engineering concepts and more thought to the individual problem. There was a noticeable lack of periodic beach condition surveys after construction of shore protective measures. Without such surveys, it appears impossible to evaluate the protective maintenance action. The following aspects were noticeably lacking in consideration:

(1) Measures for stabilization of beach.
(2) Coordination and correlation of sectional measures into the over-all picture.
(3) Selection of materials for economy and proper function.
(4) Over-topping of seawalls by impinging waves.
(5) Attrition of wave energy as much as possible before meeting a more or less vertical barrier.

"PERMEABLE" SEAWALL

GENERAL CONSIDERATIONS

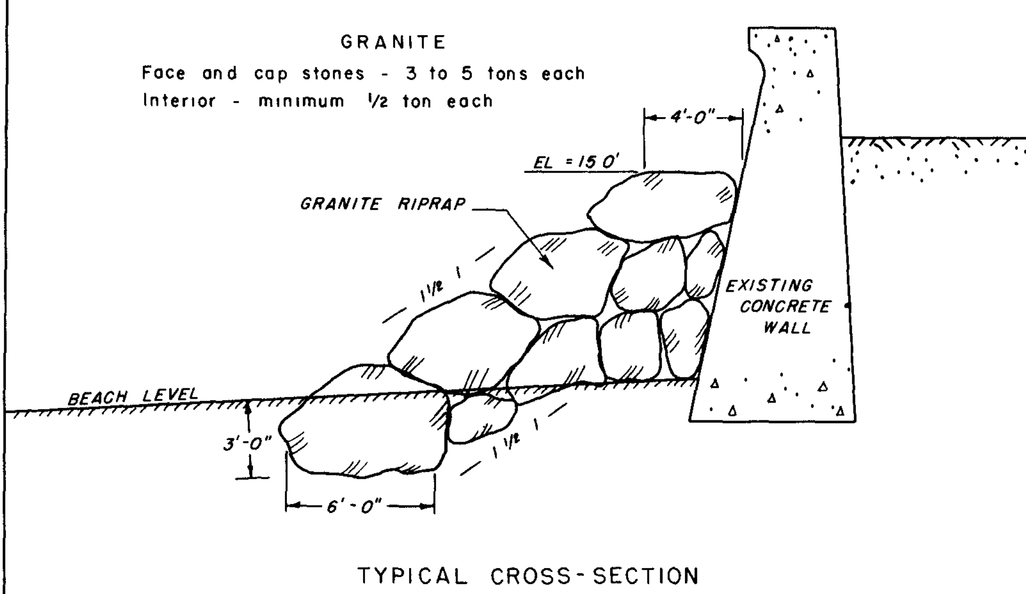
As the economic justification for shore protection is often dubious, it becomes necessary to design a seawall at the lowest cost possible, consistent with satisfactory performance. In our attempt to find a suitable type of seawall to be followed in the program of shore protection in Boston Harbor, the following standards were established for evaluation:

(1) Low cost
(2) Minimum maintenance
(3) Pleasing appearance
(4) Flexibility
(5) Low wave over-topping consistent with low wall crest

The so-called permeable type of seawall satisfied all these requirements. The low cost is achieved by the use of rip-rap stone in the base. This is plentiful and very reasonable, varying from $3.50 to $5.00 per cubic yard measured in place. The cost of placing the rip-rap is very low, as the stone is placed entirely by crane with a skilled operator. The crane also accomplishes the excavation immediately ahead of the wall, therefore eliminating costly cofferdams and the pumping involved with a rigid type of seawall regardless of the tidal conditions. It also permits the performance of the work in the winter months when construction activity is very small and during inclement weather, a factor which is reflected in the competitive bids received on the work. The plain concrete cap can be either precast or cast in place, at the discretion of the contractor. The quality of the concrete is normally much better, as it is poured under more satisfactory conditions and does not come in contact with the seawaters in any way during the curing period. The cost of the concrete is about $25.00 per cubic yard in place. Excavation generally costs about $1.00 per cubic yard.

Flexibility is achieved because of the fact that the wall is made up of many individual components, and therefore it would not be adversely affected by differential settlement and horizontal movement as would a rigid concrete seawall. One very interesting and unusual attribute of this type of wall is in connection with shores having poor foundation soil condition, such as peat and silt underlying granular beach material. Since the structural stability and integrity of the wall are unimpaired by reasonable settlement or movement, it has been used under such conditions satisfactorily. The only differences in construction are a 12 inch layer of quarry chips as a blanket over the entire foundation of the wall before placing the rip-rap and the excavation of the foundation is sloped uniformly from toe to heel. A typical cross-section of the seawall is shown on Figure 1.

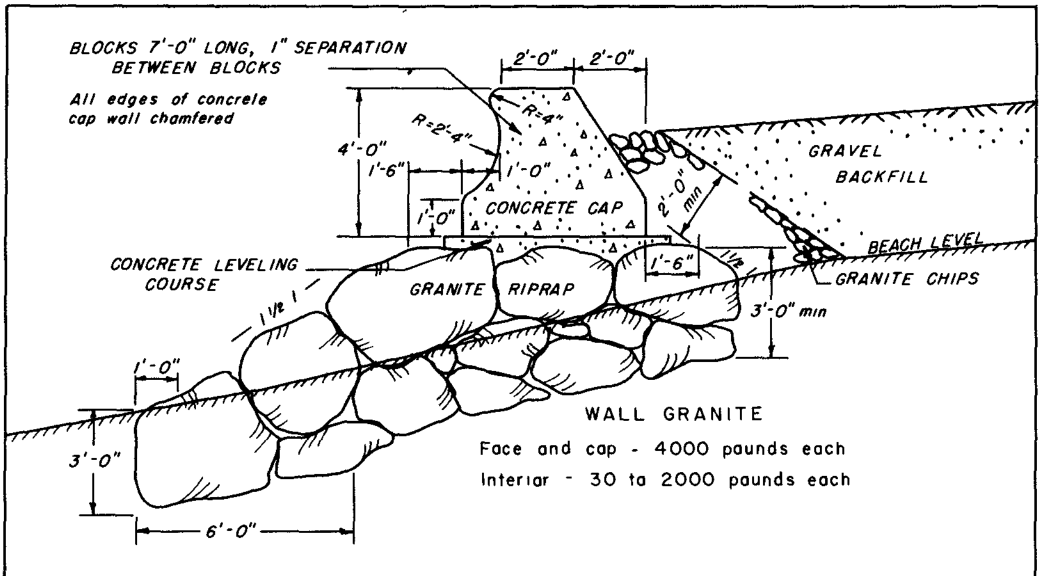
The two-tone color combination of the rip-rap base and the concrete cap, along with the pleasing lines of the cap, makes the wall appear very attractive as shown on Figures 3, 4, and 5. The maintenance is low, as the abrasion of the wall by the coarse beach material carried by the impinging waves is best resisted by the granite rip-rap. The concrete which is not so resistant is above the abrasion zones. The wall with the slots in the cap and voids in the base permits seepage of ground water behind the wall, thus eliminating any heaving of the wall as result of freezing and thawing of entrapped water.



TYPICAL CROSS-SECTION

SEA WALL REINFORCEMENT IN BOSTON HARBOR

FIGURE 1



TYPICAL CROSS-SECTION

PERMEABLE SEA WALL IN BOSTON HARBOR

FIGURE 2

Fig. 3.

Fig. 4.

Views of "Permeable Seawall" constructed at North Weymouth, Massachusetts. Note groins on beach.

Fig. 5. "Permeable Seawall".

Two conditions which made the problem difficult were the great water depth variation at the barrier and the necessity to locate the structure on geographical and economic considerations. The main tidal variation is $9\frac{1}{2}$ feet, with Spring tides of about 12 feet. In the layout the shore barrier is placed as high on the beach as possible and still permit a smooth and easy shore configuration coordinated in the comprehensive long range protection program.

A great deal of thought was given to mitigating overtopping of the seawall consistent with low wall crest height for economical and effective coastal protection. There are a number of factors that were taken into consideration in attaining a more effective design, namely, the use of a rough sloping barrier base for changing the horizontal wave momentum into a vertical component in order that attrition may result from opposing gravitational pull, and a curved re-entrant face of the cap to turn back the diminished remaining force of the wave. Another factor which has been observed to cause attrition of wave energy is a rough surface and voids in the rip-rap face. The wave, as it moves up the slope, encounters flow resistance from the rough surface and from the gushing in and out of water in the voids. A system of groins will also reduce wave energy by setting up turbulence in the wave flow, besides maintaining beach slopes.

DESIGN

Unfortunately, at the time of the design of this type of seawall we did not have the intensely interesting and helpful article of Eduardo de Castro entitled "Rock Fill Dams and Dikes" which was translated by Mr. D. Heinrich of the University of California, edited by Dr. M. A. Mason of the Beach Erosion Board, and published by the Board Bulletin in Volume 3, dated January 1, 1949.

The effective height of the wall was determined basically to cut down the over-topping during wave attack to non-damaging proportions. However, a minimum satisfactory wall crest was very desirable as it greatly affected the economics and determined whether or not criticism would be forthcoming from owners of developed shore areas on account of blocked seaward view. The first section of wall constructed was designed in accordance with the "Cycloidal" theory and compared with observations made of the performance of existing structures in the Harbor. After this section was completed and evaluated for performances, it became the criterion for all subsequent sections in the harbor. The height of the wall for any particular location would be referred to the test section and varied to take into consideration differences of fetch, direction and magnitude of the expected most severe storms, distance of wall up on beach slope from point of breakers and steepness of beach slope. With the great variation in water level from both gravitation and storms, the wave analysis becomes very difficult. In all cases the normal maximum conditions are used, such as a 12 foot maximum

Fig. 6.

Fig. 7.

Views of typical seawall reinforcement with granite rip-rap. Note groins on beach.

Fig. 8. Typical rip rap beach groin.

tide, instead of the highest recorded water level of 15 feet above mean low water in Boston Harbor.

The toe at the slope of the rip rap base is placed at least 3 feet below the beach level in order to prevent failure of the wall from erosion and to anchor it securely from displacement caused by breaking waves. The slope of the rip rap face has been found to be most satisfactory between the limits of $1\frac{1}{2}$ and 2 horizontal to 1 vertical and tangent to the curved re-entrant face of the concrete cap. The exterior stones of the rip rap base should be as large as possible and securely interlocked against movement. The interior mass should be compact, consisting of stones large enough to block the interstices in the face.

The fill at the back of the wall should have a vertical layer consisting of a well assorted mixture of quarry chips or grout to develop passive pressure capable of resisting horizontal and overturning forces besides providing excellent drainage. No evidence of damage from wave impact in water filled interstices has been observed.

A necessary adjunct of this seawall is a system of rip rap groins. These groins project about $3\frac{1}{2}$ feet above the beach, and are relatively short in length, about 100 feet. The cost is about $5.00 per linear foot.

REINFORCING VERTICAL CONCRETE SEAWALLS

There were a number of old plain cement concrete seawalls with a vertical face which showed evidence of bad scour at the base from the abrasive action of coarse beach material carried along with impinging waves. This condition was so bad that failure of the wall was imminent unless immediate repairs or corrective measures were taken. Under severe wave attack considerable over-topping of the wall was observed. There was also another condition which had to be taken into consideration - evidence of lowering of the beach from erosion.

The corrective action consisted of a rip rap base in front of the wall on a $1\frac{1}{2}$ to 1 slope extending up to the level of maximum high water, with a system of rip rap groins to stabilize the beach (Figures 2, 6, 7, and 8). The rip rap base reinforcement not only was more abrasion-resistant, but it presented a more energy absorbing face along with less base turbulence. The reinforced wall observed now undergoing severe wave attack shows considerably less over-topping.

CONCLUSION

Under conditions such as we have in Boston Harbor the "permeable" type of seawall has been found satisfactory in every respect. About 4 miles have been completed since 1947, and a commitment has been made for the construction of 8000 feet next year. Continuous observations are being made to uncover unanticipated functional characteristics and to improve upon our present design criteria. The economic approach in the engineering design for the use of low cost local construction materials is well illustrated.

CHAPTER 16

SUBSTRUCTURE DESIGN OF THE NEW MYSTIC PIER NO. 1, BOSTON

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In order to provide adequate modern terminal facilities to handle the anticipated commerce requirements, the Port of Boston Authority, in 1947, prepared a master plan for the future development of the port.* The second step of this plan was the reconstruction, on modern standards, of the old Mystic Pier No. 1. Planning for this work was started in 1949 and the new pier was opened to commerce in September 1952.

The Mystic Pier is located in the Charlestown section of Boston, adjacent to the freight yards of the Boston and Maine Railroad Company, and extends out into the Mystic River. Its position in relation to the City of Boston and other port developments is shown in Fig. 1.

The new pier is shown in Fig. 2. It has an over-all width of 468 ft, with about 600 ft of berthing space along the northern edge and about 900 ft along the southern edge. The greater part of the pier is covered by a transit shed, 418 ft in width and 538 ft long. The shed has a steel framework, the walls consist of concrete extending up to 6 ft above the floor with transite siding above, and the roof is of precast concrete plank with a generous allotment of skylights. Office space is provided at the west end of the shed and on the landward side of the shed are two covered loading platforms, each 200 ft in length, and a battery charging building.

Outside the transit shed there is a working apron 20 ft wide at the east end of the pier and 25 ft wide on both the north and south sides. Five railroad tracks extend practically the full length of the pier, one along the northern apron, one along the southern apron and three are located in a depressed trackwell along the center of the transit shed.

The general locations of these facilities are shown on the site plan in Fig. 3.

Site and Subsoil Conditions

The new Mystic Pier was constructed at the site of the old pier which had existed, in various forms, at this location for about 85 years. This old pier, which is shown in Fig. 4, consisted essentially

*See Chapter 22 by G. L. Wey

of a fill which was held in place by masonry retaining walls supported by timber piles, and surrounded on three sides by a wooden platform supported by timber pile bents. The greater part of the pier area was covered by two timber freight houses which were founded on spread footings resting on the fill. It was estimated that the dead load plus live load storage capacity of the freight houses was equivalent to a maximum area load of 430 lb/sq ft. The bottom of the channel around the pier was approximately at El. -35.0 at the east end of the pier and at El. -30.0 along the north and south sides.

The subsoil conditions underlying the pier were investigated by twenty-five exploratory borings and two undisturbed sample borings. These borings showed that the soil strata underlying the pier are relatively consistent. The elevation of the ground surface of the old pier was approximately El. 13.5. Beneath the upper layer of miscellaneous fill are strata of fine silty sand and coarse sandy gravel, the lower boundry of the sandy gravel varying between El. -21.0 and El. -26.0. Below the sandy gravel is a deep layer of soft blue clay, which, over most of the area, has a thickness of about 85 to 90 ft. The clay is underlain by compact sand and gravel, hardpan and bedrock.

A typical section through the east end of the old pier showing the subsoil conditions is presented in Fig. 5.

Stability of Embankments

An important consideration in the design or construction of any pier or embankment resting on a thick layer of soft clay is the possibility of shear failure occurring in the clay resulting in a slide of the embankment toward the toe of the slope. A number of such slides in piers, cuts and embankments have been reported in engineering literature. Perhaps the most famous of these in the slide which occurred during the construction of a quay-well at Goteborg, Sweden in 1916 and from an examination of which our present method of investigating the stability of slopes was developed.

Examination of a large number of slides has shown that failure of an embankment constructed in a clay soil or underlain by soft clay occurs by the sliding of a mass of earth along a surface which, in section, corresponds approximately to the arc of a circle, as shown in Fig. 6. Thus the entire sliding surface corresponds roughly to a section of the surface of a cylinder. This cylindrical sliding surface may pass above, through or below the toe of the slope depending on the extent and character of the clay.

Theoretically, whether or not such a slide will occur along any particular sliding surface in a relatively homogeneous clay may readily be determined. The weight of the soil mass located above the potential sliding surface tends to cause the mass to rotate about the axis of the cylindrical surface. This rotation is resisted by the shear resistance of the soil along the potential sliding surface. If the overturning moment due to the weight of the mass exceeds the maximum resisting

Fig. 1. Boston Harbor.

Fig. 2. New Mystic Pier No. 1.

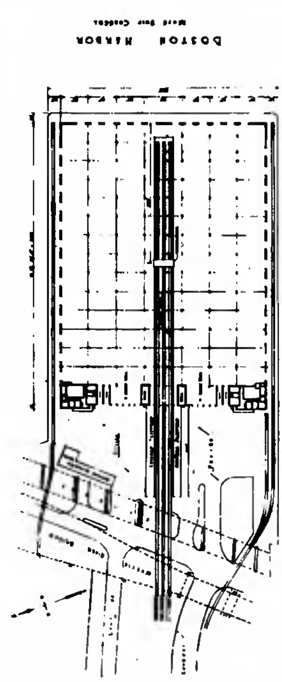

Fig. 3. Site plan of new Mystic Pier No. 1.

Fig. 4. Site of Mystic Pier No. 1.

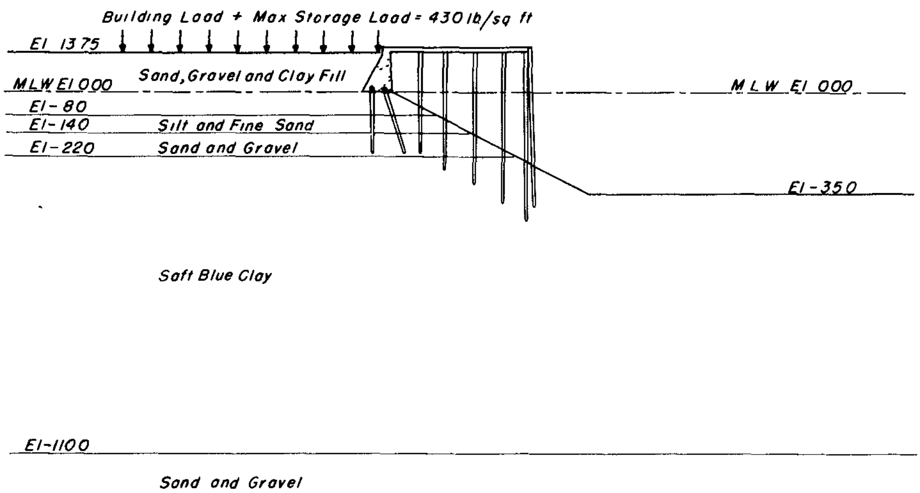

Fig. 5. Section through East end of old Mystic Pier.

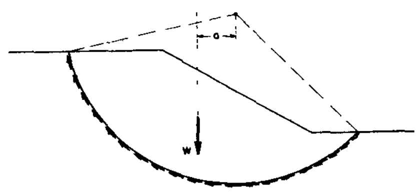

Fig. 6. Cylindrical sliding surface in clay soil.

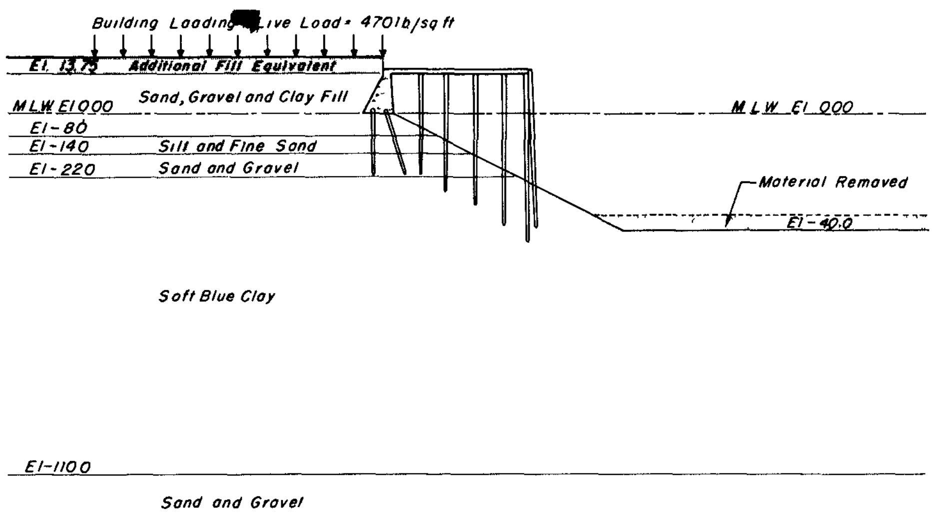

Fig. 7. Section through East end of pier showing effect of proposed changes on load distribution.

moment when the full strength of the soil is mobilized at all sections of the potential sliding surface, then failure will occur. The factor of safety against sliding along any particular surface is determined by the ratio of the maximum resisting moment to the overturning moment.

The main problem in analyzing the stability of a slope is that of determining the shear strength of the soil. A variety of test methods are available for this purpose, such as direct shear tests, unconfined compression tests, triaxial compression tests and field tests, and a variety of test procedures may be used. Unfortunately, these different tests methods and procedures give different results for the strength of a clay. However, experience obtained by the analysis of actual slides has shown that for slides in fairly homogeneous clay, the average shearing resistance along the sliding surface is roughly equal to one half of the unconfined compressive strength of the clay. Shear strength values determined in this way can therefore be used with some confidence for estimating the danger of a slide occurring in a fairly homogeneous clay.

Design Standards for New Pier

The design standards which were established by the Port of Boston Authority and which affected the selection of a suitable substructure for the new pier were as follows:

1. The ground elevation for the new pier should be raised to El. 17.0.
2. The deck of the transit shed should be designed to support a floor load of 600 lb/sq ft.
3. The channel around the pier should be dredged to El. -40.0 at the east end of the pier and to El. -35.0 along the north and south sides.

In itself, each of these requirements involves a relatively small change from the conditions existing at the old Mystic Pier. However, each of these changes tends to increase the possibility of a shear slide in the blue clay underlying the pier. The net effect of the changes corresponds to loading the entire area of the old pier with about 6 ft of compacted fill in addition to the freight houses and their storage loads and simultaneously increasing the channel depth by 5 ft, as shown on Fig. 7. The effect of such a change in load distribution on the stability of the pier is approximately equal to that produced by increasing the channel depth of the old pier by about 20 ft, as shown in Fig. 8. Such an amount of dredging has been known, in past experience, to cause failure of a previously stable embankment and thus considerable importance was attached to the problem of stability in the design of the new pier.

Stability Analysis of Old Mystic Pier

In order to ascertain the severity of the effects of the proposed changes, it was first necessary to make a stability analysis of the old Mystic Pier.

To determine the strength characteristics of the soils, two undisturbed sample borings were made close to the east end of the pier where it was thought the stability conditions would be most critical. A total of 20 samples of the blue clay, each about 2 feet in length, were taken in thin-wall seamless steel tubes using a piston type sampler. The samples were cut into sections which were tested to determine their natural water contents, liquid and plastic limits and unconfined compression strengths.

In general the samples were found to consist of grey-green slightly silty clay with some partings of silt and fine sand. The clay had a sensitive structure, characteristic of much of the clay in the Boston area; in the undisturbed state it was medium stiff and brittle but it became soft and sticky when remoulded at the natural water content.

A total of 49 unconfined compression tests were made to determine the variation of the shear strength of the clay with depth. Although there was considerable scatter of the results, the over-all average of the results for both borings showed a general increase in strength from about 0.6 kg/sq cm at El. -25 to about 1.2 kg/sq cm at El. -110.

On the basis of these results, a large number of trials were made to determine the surface along which the factor of safety against sliding was a minimum. This analysis, for the east end of the pier, showed the most critical surface to be located as shown in Fig. 9 with a computed factor of safety against sliding of about 1.15. For the north and south sides, where the channel depth was not so great, the computed factor of safety was slightly higher.

Such values for the factor of safety against sliding are somewhat lower than is generally considered desirable for earth banks of this type. However, in any such analysis a number of assumptions must necessarily be made and the computed result, while indicating the probable order of magnitude of the factor of safety, cannot be considered a precise determination. Furthermore, the old pier had been standing for many years and although its condition left much to be desired, it showed no signs of distress due to shear failure in the clay. Thus in spite of its low value, the computed factor of safety was evidently perfectly adequate. On the other hand, since the analysis was carried out using a method proved by experience to be relatively reliable, the computed result could not be considered to be too far removed from the actual conditions. A review of the assumptions made in the analysis showed that the computed factor of safety was unlikely to be too low by more than about 10% due to the use of assumed values; and even if the factor were 1.25 rather than 1.15, it was still as low as would normally be considered desirable for the design of a stable slope. Consequently, the conclusion drawn from this analysis was that although a factor of safety of 1.15, computed on the basis of the selected assumptions, was perfectly adequate, this value was also the lowest which could reasonably be adopted as a basis for the design of the new pier.

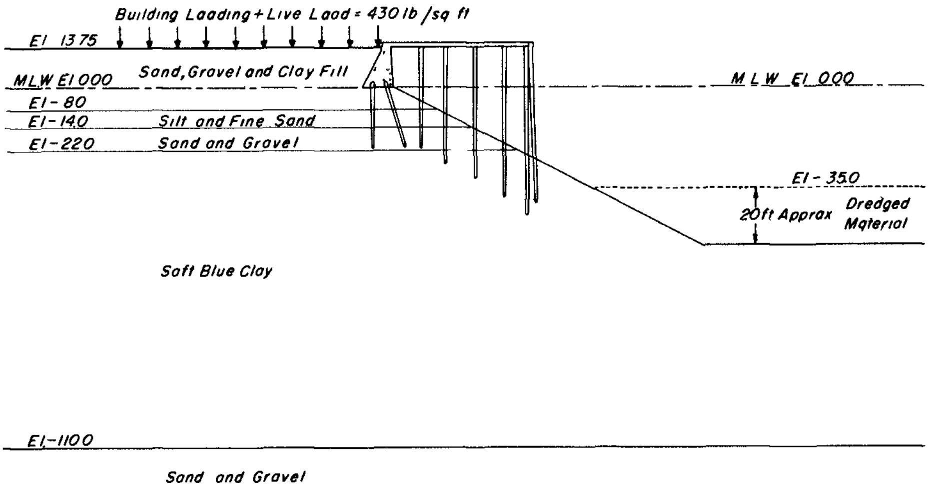

Fig. 8. Seotion through East end of old pier showing depth of dredging produoing same ohange in stability as that oaused by requirod additional load.

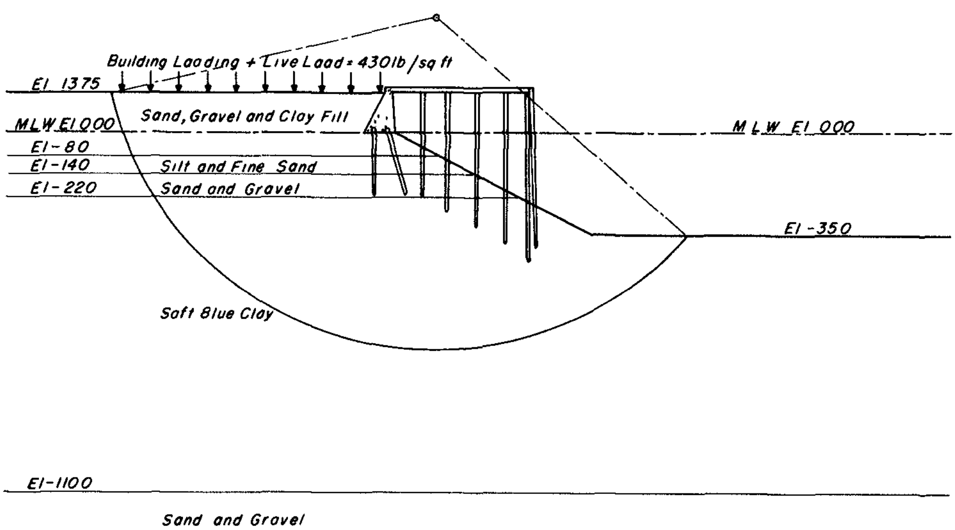

Fig. 9. Seotion through East end of old Mystic Pier showing most oritical sliding surface.

Many factors were taken into account before this conclusion was finally reached. For example, it was known that in Europe the shear strengths of clays, in situ, had been measured by a vane shear apparatus and found, in the case of sensitive clays, to be considerably higher than the values determined by one half of their unconfined compression strengths. Therefore, it might be argued, the actual resisting moment was likely to be considerably greater than that used in the analysis and the actual factor of safety would thereby be considerably greater than the computed value.

To offset this were the results of recent research at Harvard University, where it had been found that under a sustained load, a sample of a sensitive clay would fail under a stress considerably lower than the normal compressive strength. Again, the development of progressive failure in a clay will cause the average shear resistance at failure to be somewhat less than that which would be produced if the full strength were mobilized simultaneously at all points in the mass. Thus even if the in situ strength of a clay were greater than the unconfined compressive strength, the effects of progressive failure and the application of sustained stress, as would occur in an embankment, would cause failure to take place at a stress lower than the in situ strength and therefore, at an average stress which might (1) be slightly greater than the unconfined compressive strength, (2) correspond closely to the unconfined compressive strength, or (3) even be slightly less than the unconfined compression strength. In the last event, the analysis would give too high a factor of safety rather than the conservative value indicated by consideration of vane shear effects alone.

However, the fact that in so many cases the average shear strength of the clay when a slide has occurred has been closely equal to one half the unconfined compression strength would seem to indicate that in general the combination of progressive failure and sustained load, and perhaps some other effect yet unknown, is just sufficient to offset the difference in strength between that of the clay in situ and that of a sample in the laboratory. It is because of this experience with actual slides that the method of determining slope stability using an average shear strength in clay equal to one half the unconfined compression strength can be used with a reasonable degree of confidence.

Effect of Proposed Changes on Stability of Pier

It has already been explained that the reconstruction of the new pier along the lines of the old one, and also in conformity with the design standards, would correspond to increasing the storage load of the old freight houses by about 650 lb/sq ft and increasing the depth of the channel by 5 ft. It was found that these changes would reduce the computed factor of safety against shear failure in the clay to about 1.03 at the east end of the pier and about 1.08 along the north and south sides. Since those values were below the minimum of 1.15 established on the basis of the stability analysis of the old pier, it became necessary to devise some alternative method of reconstruction which would not cause any reduction in the stability of the pier.

Selection of Substructure for the New Mystic Pier

The simplest method of relieving the load on the clay and yet meeting the design conditions was evidently to support the entire floor of the transit shed on a pile foundation. By using long piles driven through the clay, the weight of the structure and its storage load could be transmitted to the underlying layer of compact sand and gravel and the shear stresses in the clay would not exceed their original values. However, this method of construction would have required the use of about 1000 piles, each about 130 ft long, in addition to those used to support the deck of the pier,and it was quickly realized that the cost of such a substructure would be prohibitive.

Another method of relieving the load on the clay was to dispense with the fill required to bring the floor grade to El. 17.0 and instead, construct a concrete slab floor at this elevation, supported by beams and spread footings resting on the fill. However, the cost of such a floor slab to support a live load of 600 lb/sq ft was quite high and furthermore, there was considerable objection to leaving a space between the floor slab and the top of the fill on the grounds that it would provide a breeding place for vermin and therefore be unhygienic.

The design finally selected made use of a combination of methods for preventing any increase in shear stresses in the blue clay. First, the structure of the transit shed, but not the floor, was supported by long piles driven through the blue clay. This was actually a direct consequence of the pile support required for the concrete apron surrounding the transit shed. The piles used to provide this support were required to act, above the bottom of the channel, as columns and in some cases had an unsupported length of about 50 ft. Consequently, it was necessary to select a type of pile which would act also as a column and this requirement narrowed the choice to either steel H-piles or concrete-filled steel cylinders. Of these types of pile, steel H-piles were found to be best suited to the design conditions and also in this case, the more economical.

Another feature of the piles supporting the concrete apron was the high loads which they were required to carry. This resulted from the fact that their positions were determined to some extent by the locations of the numerous timber piles of the old pier. Some idea of the large number of timber piles, all of which were subsequently cut off below the water level, may be obtained from Fig. 10. In the new design, a careful survey was made of the locations of all these timber piles and, in order to facilitate construction, the new piles were located so as to minimize the danger of obstruction by the old ones during driving. It was felt that this careful procedure was rewarded during construction but it complicated the design by necessitating the placing of piles at different positions along the various bents so that a single standard design could not be adopted and by resulting in unusually high pile loads

Fig. 10. Timber piles in old pier.

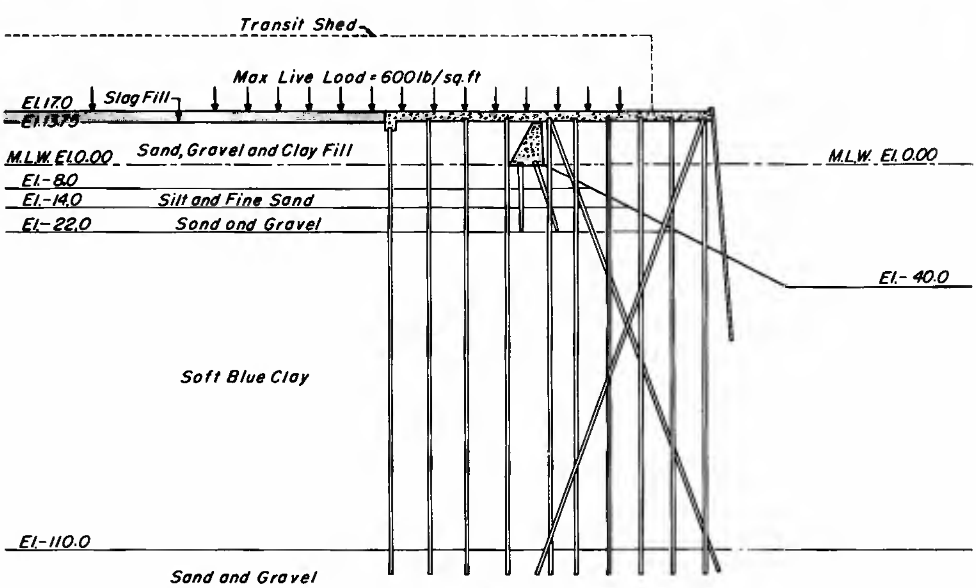

Fig. 11. Section through East end of new Mystic Pier No. 1.

where the positions of the old piles caused the new ones to be placed relatively far apart.

Because of their high loads, the piles supporting the deck had to be driven to a hard statum below the blue clay and since these piles supported also the edges of the transit shed structure, it was necessary to provide an equally firm foundation for the remainder of the structure. These long piles, then, served to prevent the weight of the structure itself from causing any increase in shear stress in the underlying clay.

The second method adopted to minimize the stress increase in the blue clay was that of using a light-weight slag fill to raise the ground level of the existing pier to the required elevation. By this means the weight of fill required was reduced by about 35 percent.

Since the two procedures described above were not sufficient in themselves to prevent a reduction in the factor of safety against shear failure in the clay under the new loading conditions, a further relief of stress in the clay was provided by extending the pile-supported concrete deck for a suitable distance behind the sea-wall. By this means the live-load at the edges of the pier was transferred to a bearing stratum below the clay. In addition, it was considered that the piles themselves would provide restraint against shear failure or plastic deformation of the clay. Thus the final design was similar to that shown in Fig. 11.

Analyses were made to determine what additional width of concrete deck should be supported on a pile foundation in order to maintain the computed factor of safety at its lowest permissible value of 1.15. In these analyses the chief problem was to evaluate the restraining effect of the piles. Since the piles are effectively restrained at their upper and lower ends, any tendency of the clay to deform will develop a bending action in the piles and, provided the clay can arch between adjacent piles without developing pressures exceeding the bearing capacity of the clay, the magnitude of the resistance provided will depend on the bending strength of the piles. Thus in order to determine this resistance it is necessary to make decisions on such problems as the lateral pressure distribution against the piles, the degree to which arching can develop in a clay soil, and the maximum permissible stress in the piles. Many hours of thought were given to these problems and the opinions of a number of leading engineers were carefully considered before the final decisions and analyses were made.

These analyses showed that it would be necessary to provide a pile-supported floor at the east end of the pier for a distance of about 50 ft behind the sea wall and along the sides, for a distance of about 20 ft behind the sea wall in order that the stability of the new pier should be essentially the same as that of the old one. The new pier was designed accordingly. It is believed that for the required conditions, the design provides adequate safety consistent with reasonable economy.

Perhaps the most interesting feature of this design is the fact that the major problem was not one of a really technical nature. When an engineer uses an accepted method of analysis to determine the stability of an earth bank which has been standing for fifty years or more, and he finds the factor of safety to be lower than or as low as is normally considered desirable, what should be his attitude towards making changes which will affect this factor of safety? It would probably be generally agreed that a factor of safety which has been adequate for fifty years will continue to be adequate for the next fifty years. But differences of opinion would exist as to whether any reduction in the factor of safety might be permissible. Some engineers would appear to believe that a long period of stability is indicative of a high degree of stability and that the height of a long-standing earth bank can be increased by, say, 20 percent without any undue risk of failure. On the other hand, there is the point of view that an earth bank which has existed for many years may be only just stable and may fail if the loading conditions are only slightly increased; experience shows this to have happened in many cases. Between these two extremes there is a wide variety of schools of thought.

Another important factor to be considered in such a problem is the cost of the measures required to prevent any reduction in the factor of safety. If the cost of the measures is excessive, it may be decided to take a calculated risk. Yet, even in this case, there must be a minimum permissible value for the factor of safety, and when this value is reached, the question of cost will become secondary since adequate safety must always be the primary consideration.

In the design of the new Mystic Pier No. 1 the designers were confronted with the dilemma of whether to place more faith in the intuitive opinions of reputable engineers or in the results obtained by a method of analysis which is necessarily subject to considerable limitations but which is based on previous experience of failures. It was finally decided that although the factor of safety was entirely adequate it was yet sufficiently low that no reduction should be allowed in designing the new pier. Fortunately, by a combination of measures, this was achieved at a reasonably small additional cost. It is believed that this approach, based on the technical advances in the field of soil mechanics in the past twenty-five years, was in good accord with the principles of sound engineering practice.

Acknowledgements:

The new Mystic Pier No. 1 was designed by Thomas Worcester Inc., Consulting Engineers, Boston for whom Mr. F. K. Perkins was chief engineer and Mr. M. Ja was project manager. Mr. George Wey, Chief engineer, represented the Port of Boston Authority on Technical matters pertaining to the design of the pier. The cooperation of these gentlemen in assembling the material presented in this paper is gratefully acknowledged. Thanks are also due to Mr. C. L. Monismith and Mr. L. C. Reese who assisted in making the stability computations and to Mr. G. Dierking who prepared the figures.

PART 4

FACTORS AFFECTING THE LIFE OF COASTAL STRUCTURES

CHAPTER 17

LIFE OF STEEL SHEET PILE STRUCTURES IN ATLANTIC COASTAL STATES

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As a basis for more accurate estimates of the useful life of steel sheet piling, in 1936 the Beach Erosion Board, in cooperation with a number of District offices of the Corps of Engineers, began a survey of the rate of deterioration of steel sheet piling. The plan of operations was developed by and the first inspection was made under the supervision of Mr. Ralph F. Rhodes, Engineer of the Savannah District. Subsequent inspections were under the supervision of Mr. Jay V. Hall, Jr. and most of the compilation and analysis of data were done by Mr. C. W. Ross, both of the Beach Erosion Board staff. The Corps of Engineers is indebted to the many owners of structures who contributed to the study by permitting drilling and measurement of piling.

The survey included examination of ninety-four structures located along the Atlantic Coast of the United States and the Gulf coast of Florida. The structures were selected to include a variety of conditions of exposure and treatment. Most were exposed to normal sea water or sea water only moderately diluted with fresh water. A few were exposed to fresh water with only occasional intrusions of sea water.

Measurements were made of the thickness of the webs of 153 groups of piles. In most cases, each group comprised 5 piles having similar environment and three sets of measurements were made covering the period of approximately 10 years from 1936 to 1946. Where practicable, measurements were taken at several elevations on each pile to include both tidal and atmospheric exposures. The removal of piling from structures at two locations permitted measurements of the entire length of each pile to obtain data on rates of loss under water and in earth.

TYPES OF STRUCTURES INVESTIGATED

The piling groups studied are classified by type of structure as harbor bulkheads, beach bulkheads, and groins. Harbor bulkheads are found in wharves, piers, slips, and retaining walls, all of which are protected to some extent from wave action. Backfill generally protects one surface of the piling, but the top portion of the piles may be exposed to the atmosphere on both sides. The portion between the levels of high and low water is exposed to alternate wetting or drying. The third portion is always under water, and the lowest portion is buried in the harbor bottom. Beach bulkheads are located along the shore to protect the land from storm waves. Those located back of the high water line are not subject to frequent wetting, except by salt spray from waves. The backfill and the beach materials are usually sand, but occasionally consist of coarser materials. Groins are structures built along the beach generally normal to the shore line to retard littoral currents near the shore with a view to accumulating or retaining beach material in a specific area. Some part of the groin surface is normally subject to rapid movement of beach material caused primarily by wave action. The exposure of jetties is generally similar to that of groins. Several pile groups in jetties have been classified with and analyzed with those in groins.

LOCATIONS OF STRUCTURES

In order to have distinct differences in climatic conditions, structures were selected by regions northward from Point Pleasant, New Jersey, and southward from Wilmington, North Carolina. A number of groins located at Cape May and Cape May Point, New Jersey, were also observed during the investigation, but were not measured. As no suitable beach bulkheads or groins were found north of Long Island, New York, only harbor bulkheads were observed in New England.

The various locations at which steel sheet piling were measured or observed by States are as follows:

Florida - Clearwater Beach, Tampa, Sarasota, Fort Myers, Miami, Miami Beach, Bakers Haulover, Hollywood Beach, Palm Beach, Fort Pierce and Jacksonville;
Georgia - Savannah Beach;
South Carolina - Charleston;
North Carolina - Wilmington and Kure Beach;
New Jersey - Cape May, Point Pleasant, Manasquan, Belmar, Deal, Sea Bright, Long Branch and Harrison;
New York - Harlem River, Brooklyn, Queens, Long Island City, East River, Island Park, Roslyn, Glen Cove, Oyster Bay, Asharoken Beach, Shinnecock Canal, and Southampton;
Connecticut - Greenwich, Stamford, Norwalk, Bridgeport, New Haven, Branford, Middletown and New London;
Rhode Island - Point Judith and Newport;
Massachusetts - New Bedford, Woods Hole, Boston, Chelsea, Charleston, Lynn and Salem;
Maine - Portland.

MEASUREMENTS OF THICKNESS OF PILING

The thickness of the web of each pile was determined by 8 or 10 measurements at each inspection. Rust, paint, and scale were carefully cleaned from both surfaces of the web before the measurements were made. Where practicable, 10 measurements were made near the top of the pile by means of a large striding micrometer. Where it was impossible to use the striding micrometer, a 3/4-inch hole was drilled through the web and 8 equally spaced measurements were made around this hole with a small hub micrometer. Where there was backfill which could not be excavated, a small amount of the backfill material was removed through the hole, paper was forced into the cavity to form a lining and the inner surface of the steel was cleaned with special scrapers operated through the hole.

Where practicable, measurements were made of the thickness of sound metal in the web of each pile at four zones defined relative to tidal planes, namely approximately at mean low water, mean tide level, mean high water and above high tide. After the measurements were completed the holes were filled with soft iron or lead plugs. New holes several inches from the old holes were drilled for the measurements at subsequent inspections. Averages of measured thicknesses by groups for each inspection were tabulated, classified by zones relative to tidal planes. The tabulated values were usually averages of at least 40 measurements, as in general each group comprised 5 piles and 8 or 10 measurements were made of each pile in each zone. Data relating to geographical locations, date of installation, condition of piling when installed (new or used) and nominal thickness of piling were also tabulated, classified by type of structure.

OTHER DATA ASSEMBLED

Other data recorded as far as practicable in connection with the investigation were as follows:

a. Manufacturer
b. Section of piling
c. Copper content, if any
d. Distance from nearest breakers
e. Exposure to direct wave action
f. Estimated intensity of currents
g. Salinity (estimated average condition)
h. Pollution
i. Tidal range
j. Nature of adjacent materials
k. Type of capping, if any
l. Condition of paint at each inspection
m. Amount of rusting and pitting
n. Oil or grease scum
o. Marine life

However, data on the foregoing factors did not indicate that any one was of major importance in determining the durability of steel piling.

RELIABILITY OF MEASUREMENTS AND COMPUTED RATES OF LOSS

The web thickness of the piles was measured with micrometers reading to 0.001 inch. The micrometers were checked and adjusted frequently. Although the measurements were at times made under difficult conditions, the number of measurements is considered adequate to render the effects of random errors and pitting unimportant.

As the measurements were made by a number of inspectors, it is possible that the surfaces were not always cleaned to the same degree. However, it is unlikely that errors from this source are serious, as this type of error is compensating.

Since the second and third sets of measurements were made at holes drilled several inches from the old ones, an error was introduced in the computed rate of loss of steel in cases where the original web thickness was not uniform. The webs of most new piles are of nearly uniform thickness, but in certain types the thickness of the webs increases from the center towards the edge. In known cases of these latter types, the new holes were drilled either vertically above or below the old holes. However, there are certain groups in which the change in thickness was less noticeable and in a few cases an increase in measured thickness resulted, presumably because of the change in the location of the holes.

The nominal thickness, as well as the measurements of the webs, was generally used in computing changes in thickness. Variations in thickness of new piling result from wear or lack of adjustment of the rollers. A weight tolerance of 2½ per cent either way from listed weights is standard permissible mill variation. The first measurements were frequently greater than the nominal thickness by more than 2½ per cent, indicating that the mills tended to produce piles which were overthick. When the web thickness determined from the first measurements exceeded the nominal thickness, the former has been taken as the initial thickness in computing losses.

The effect of errors in the determination of the thickness on the annual rate of loss decreases with time. The error of the rate of loss is probably seldom larger than 0.002 inch per year.

The computed losses of thickness are losses from all causes. The two causes which are believed to be principally responsible are: (a) corrosion, defined herein as loss due to chemical reaction with the environment, and (b) abrasion, loss due to wearing away by friction of moving materials. The latter appears to be an important factor in the case of groins, although the removal of rust and exposure of bare steel by abrasion may accelerate the corrosive processes.

PRINCIPAL FACTORS AFFECTING THE RATE OF DETERIORATION

All data assembled in connection with the investigation, as mentioned previously, have been considered to determine their significance relative to the rate of deterioration of piling. The number of variables considered which may have an effect on the rate of loss of steel is so large that even the number of measurements made in this survey is inadequate to cover all conditions thoroughly. Data secured appeared adequate to warrant general conclusions by comparison of rates only under the following categories:

a. Type of structure
b. Geographical locations
c. Zone relative to tidal planes
d. Sand, earth or other cover
e. Exposure to salt spray
f. Paint protection

Comparison of rates of loss in the several categories gave the following results.

LIFE OF STEEL SHEET PILE STRUCTURES IN ATLANTIC COASTAL STATES

TYPE OF STRUCTURE - Based on all measurements the average rate of loss of thickness for harbor bulkheads was 0.0033 inch per year. By comparison the average rates for beach bulkheads, groins and jetties were 0.016 inch per year, indicating a durability of steel sheet piling in harbor bulkheads about 5 times that in other shore structures.

Observation of perforation of groins at Cape May and Cape May Point, New Jersey, provided additional data on rates of loss. All of these groins were built of piling having nominal web and flange thicknesses of 0.281 and 0.250 inch respectively. Some holes were observed after 4 years, indicating a maximum annual rate of loss of about 0.06 or 0.07 inch. The average time in which holes first appeared was about 7 years, indicating a rate of loss of about 0.04 inch per year. As may have been expected, the rates indicated by these observations are larger than average rates derived from measured holes, since the perforations naturally occur at points of worst exposure. Natural holes observed in measured piling provide examples of similar rates of loss. A previous study of deterioration of steel piling in groins at Palm Beach, Florida, revealed perforations with rates of loss from 0.04 to 0.09 inch per year.

GEOGRAPHICAL LOCATION - The average rates of loss of thickness for northern and southern regions for the three types of structures were as follows:

Region	Harbor Bulkheads Inch/year	Beach Bulkheads Inch/year	Groins and Jetties Inch/year
Southern	0.0062	0.017	0.018
Northern	0.0023	0.0075	0.011

Comparison of these rates indicates a durability of steel piling in the Northern Region averaging about double that in the Southern Region. However, the number of measurements of groin piling in the Northern Region was relatively small. Observations of natural perforations in unmeasured piles in New Jersey indicate high rates of loss, similar to those experienced in the South. A general conclusion that steel piling in groins has materially greater durability in the North than in the South appears to be unwarranted.

ZONE RELATIVE TO TIDAL PLANES - The average rates of loss of thickness for the several zones relative to tidal planes were as follows:

Zone	Harbor Bulkheads Inch/year	Beach Bulkheads Inch/year	Groins and Jetties Inch/year
8' above MHW	0.0049	0.020	0.010
5' to 8' above MHW		0.022	
2' to 5' above MHW		0.0081	

Mean high water	0.0027	0.0074	0.0055
Mean tide level	0.0024	0.001	0.024
Mean low water	0.0035	0.002	0.028

Comparison of rates for harbor bulkheads indicates little difference in the tidal zone from mean low water to mean high water. Above mean high water a rate about 70 per cent greater than the average rate in the tidal zone is indicated. For beach bulkheads, the rates for the area more than 5 feet above mean high water were somewhat more than double the rates for the area between, and 5 feet above, mean high water. In each of the two locations where piling was measured below mean high water, sand covered the zone at least part of the time and the rates of loss were negligible. For groins and jetties, rates for areas at and above mean high water averaged about one-third of the rates for the areas exposed to tidal and wave action.

Measurements of pulled steel piling provide data on deterioration by zones below mean low water. From mean low water to the ground line, the zone continuously submerged, the rate of loss averaged 0.0035 inch per year, the highest rates being within a few feet below the mean low water line. In the Miami groups lesser peaks in the rate also appeared just above the ground line. Below the ground line the average rate of loss was slightly under 0.002 inch per year.

SAND, EARTH OR OTHER COVER - The average rates of loss of thickness for several conditions of cover were as follows:

Cover	Harbor Bulkheads Inch/year	Beach Bulkheads Inch/year	Groins and Jetties Inch/year
No cover on either surface of pile	0.0075	0.027	0.019
One surface never covered, other covered part time	0.0076	0.020	0.014
One surface only covered	0.0026	0.0094	0.020
One surface always covered, other covered part time		0.0065	0.0057
Both surfaces covered part time			0.017
Both surfaces always covered		0.0017	0.0026

Comparison of rates for harbor bulkheads indicates that lack of backfill for all or part of the time greatly increases the rate of loss. For beach bulkheads the rate of loss rapidly decreased as the cover increased. For groins and jetties the rates of loss were uniformly high except for those covered on both sides all or part of the time.

EXPOSURE TO SALT SPRAY - The average rates of loss of thickness of harbor bulkheads for heavy, moderate and light salt spray were respectively 0.0083, 0.0041, and 0.0024 inch per year. All beach bulkheads, groins and jetties are considered to be subject to heavy spray. The average rate of loss for these structures was 0.016 inch per year. You will note the much higher rates of loss where piling is subject to salt spray.

PAINT PROTECTION - The average rates of loss of thickness without painting and with painting on one or more occasions were as follows:

Painting	Harbor Bulkheads Inch/year	Beach Bulkheads Inch/year	Groins and Jetties Inch/year
None	0.0045	0.018	0.020
At least once	0.0027	0.011	0.010

Comparison indicates a substantially lower rate of loss for structures that had been painted at least once. Data on composition of paints and the manner of application were too limited to permit study of these factors. Few of the structures studied were painted regularly. The fact that occasional painting reduced the rate of loss substantially indicates that regular painting would result in greater reduction. Determination of the economic justification of regular painting would require study of painting costs. As rates of loss for beach bulkheads are high and as painting of these structures is less costly than complete painting of harbor bulkheads, regular painting of beach bulkheads may be justified. The same is true of the zone above mean high water of harbor bulkheads. Painting of groins and jetties in the tidal zone, subject to abrasive action, probably could not be justified.

CONCLUSIONS

The mean rate of loss of thickness of steel sheet piling based on a total of 451 weighted averages was about 0.008 inch per year. The rates of loss vary materially under different environmental conditions. Consequently, different types of structures and the several portions of the same structure have different rates of loss. In addition to specific rates of loss under various conditions, just discussed, the data are adequate to warrant the following general conclusions:

a. The rates of loss are much lower for harbor bulkheads than for other shore structures studied;

b. The rates of loss for harbor and beach bulkheads are lower in northern than in southern States;

c. The rates of loss for harbor and beach bulkheads are materially higher for surfaces above mean high water than for surfaces within the tidal range. The rates of loss for groins and jetties are much greater at mean tide and mean low water levels than at higher elevations;

d. Sand or earth cover materially decreases the rate of loss, the rates of loss for all practical purposes being negligible for piling always covered on both sides;

e. Exposure to salt spray greatly increases the rate of loss;

f. Painting, either initially or at irregular intervals, materially reduces the rate of loss.

The useful life of steel sheet pile structures depends on the original thickness of the steel, the rate of loss of thickness, and the thickness of the piling when the structure is no longer useful because of loss of thickness. The data obtained will enable the designer to estimate the probable life of a structure, giving due consideration to environmental factors, and to determine the justification of protective coverings such as paint, concrete or wood sheathing to reduce the rate of loss in the more vulnerable portions of the structure.

CHAPTER 18

EXPOSURE RESEARCH ON CONCRETE IN SEA WATER

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INTRODUCTION

Many coastal installations are constructed entirely of concrete or contain concrete in a large part of the structure, particularly those portions which are submerged, and those portions between low and high water. It is the latter locations which are submitted to the most severe exposure.

The work done by the Corps of Engineers, on investigations of the durability of concrete in sea water, which is now being carried on by the Waterways Experiment Station, was essentially initiated by the Concrete Laboratory of the Passamoquoddy Tidal Power Project, Eastport, Maine, in 1935. While this project was never completed, a very extensive investigation of materials and the properties of various concrete mixtures was performed, including the exposure of concrete specimens to natural weathering in sea water.

Between 1935 and 1952 the Waterways Experiment Station through its Concrete Research Division and its predecessor concrete laboratories, has exposed 2556 concrete specimens to natural weathering. The principal exposure station, and the only one at which specimens have been continuously exposed since 1935, is at Treat Island (Eastport), Me. A second exposure station, which has been in use since 1940, is at St. Augustine, Fla. Other exposure stations, not now in service, have been maintained at Buzzards Bay, Mass. and at West Point and Mount Vernon, N. Y. At all but the two stations in New York, the specimens were subjected to alternate exposure to air and immersion in sea water as governed by tidal action. The geographical location of the two stations now in use are shown in Fig. 1. Detailed results of the various programs of investigation are given in laboratory reports indicated in the list of references.

The purpose of this paper is to review the results of the exposure of concrete specimens from the several essentially unrelated investigations and to summarize the major information revealed thereby as related to the durability of concrete.

DESCRIPTION OF EXPOSURE STATIONS

TREAT ISLAND EXPOSURE STATION

The exposure station at Treat Island, Me. is by far the largest and most important of the stations that have been used. The exposure rack is located at mean or half-tide elevation on a Government wharf at Treat Island, situated in Cobscook Bay at the mouth of the Bay of Fundy about midway between Eastport and Lubec, Me. The normal range in tide is 18 ft with a maximum of 26 ft and a minimum of 13 ft. The volume of tidal water is such that its temperature is remarkably uniform throughout the year; varying from a low of 34 F in April to a high of 40 F in early September. During the months of December through March the air temperature averages about 14 F with a normal low of -10 F and a normal high of 36 F.

The combination of air and water temperatures during the winter months is such that the specimens are thawed in water to a temperature of about 37 F when covered at high tide and are frozen in air to a temperature of between -10 and 28 F when out of the water at low tide. The change from the frozen to the thawed condition is very rapid, thus increasing the severity of the exposure.

The principal weathering influences to which the specimens are subjected are freezing and thawing in the winter and wetting and drying in the summer. The influence of the sulfates in the sea water on the specimens is apparently insignificant probably because the uniformly low temperature of the water is not conducive to chemical action. Further evidence of the lack of chemical action as a factor in the failure of specimens at Treat Island is the complete absence of any indication of deterioration of many comparable specimens installed in warm sea water at St. Augustine, Fla.

The original installation of specimens on the Treat Island rack consisted of 5 by 5 by 60-in. columns made in connection with concrete studies for the proposed Passamoquoddy Tidal Power Project. The specimens were originally installed in a vertical position. During the summer of 1940 an entirely new rack was constructed providing for the installation of the specimens in a horizontal position. Since 1940 all specimens at all stations have been installed horizontally.

ST. AUGUSTINE EXPOSURE STATION

The effects of mild marine weathering are evaluated at Salt Run, off Anastasia Island, near St. Augustine, Fla. The principal agent of attack at this installation is warm sea-water. The mean water temperature is about 70 F. The average tide range is 4.5 ft, with a maximum of 5.3 ft and a minimum of 3.7 ft. This station affords information on the effects of sea-water on concrete specimens, apart from the effects of freezing and thawing. Companion specimens to those exposed at Treat Island in connection with a major investigation on portland cements have been exposed here since 1940.

BUZZARDS BAY EXPOSURE STATION

The exposure station at Buzzards Bay was operated from 1938

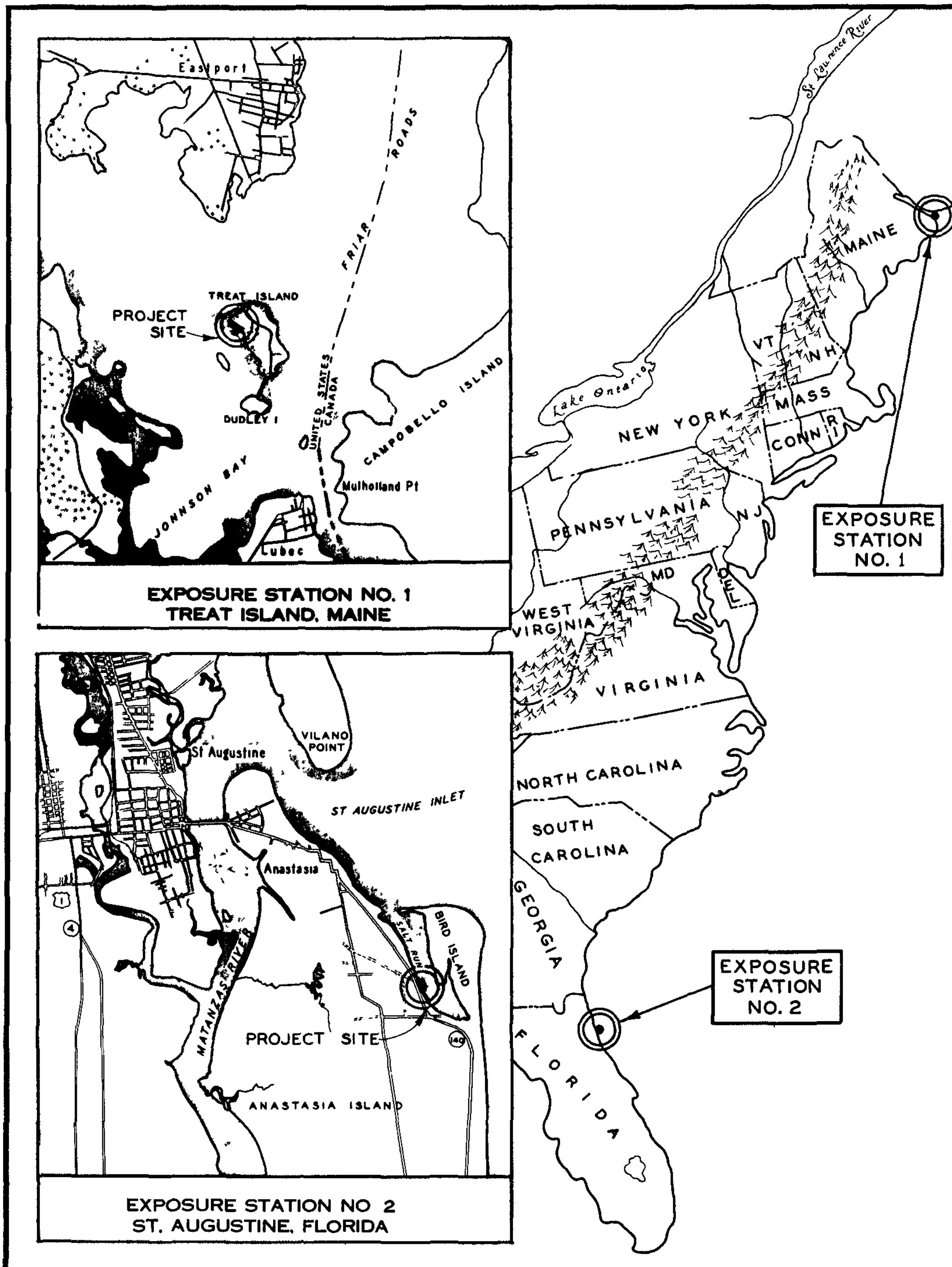

FIGURE 1. LOCATION AND VICINITY MAPS OF EXPOSURE STATIONS NOS. 1 AND 2.

through 1942. A total of 19 column specimens was exposed during this period in an investigation of blends of portland cement with certain admixtures. The exposure rack was located in the Cape Cod Canal near the shore opposite the Sandwich Coast Guard Station. The average tidal fluctuation is 9 ft. The Portland Cement Association has experimental piling exposed here also for their long-time cement studies. Because of the presence of floating ice in the Cape Cod Canal during a considerable portion of the winter season and because of the fact that the water temperature drops as low as 29 F during the late winter, this station has been abandoned for installation of concrete specimens in favor of Treat Island.

NEW YORK STATE EXPOSURE

The moderate-weathering exposure installation, not involving immersion in water, used in connection with two investigations was located out-of-doors adjacent to the Central Concrete Laboratory, at the U. S. Military Academy, West Point, N. Y., between 1940 and 1942 and in Mount Vernon, N. Y., from 1942 to 1946. Companion specimens subjected to "no-weathering" were located inside the laboratory buildings at the respective locations during the respective periods.

NUMBER OF SPECIMENS INSTALLED AT EXPOSURE STATIONS

Station	Year	Number Installed	Cumulative Total
Treat Island	1936	43	43
Buzzards Bay	1938	19	62
Treat Island	1940	271	333
St. Augustine	1940	152	485
New York (outdoors)	1940	156	641
New York (indoors)	1940	52	693
Treat Island	1941	210	903
New York (outdoors)	1941	12	915
Treat Island	1942	182	1097
Treat Island	1943	767	1864
Treat Island	1944	108	1972
Treat Island	1946	82	2054
Treat Island	1947	6	2060
Treat Island	1948	300	2360
Treat Island	1949	66	2426
Treat Island	1951	130	2556

MEASUREMENT OF EXPOSURE AND TESTING OF SPECIMENS

The basic factor in the three tidal exposures is the alternation of environment around the specimens produced by rise and fall of the tide. If the air and water temperatures at Treat Island were constant at below and above the freezing point of concrete saturated with sea water, then the only unit required for measurement of exposure would be "cycles of freezing and thawing". Since this not the case, it is necessary, in making comparisons between specimens installed at different times, to introduce a correction for variable intensity of

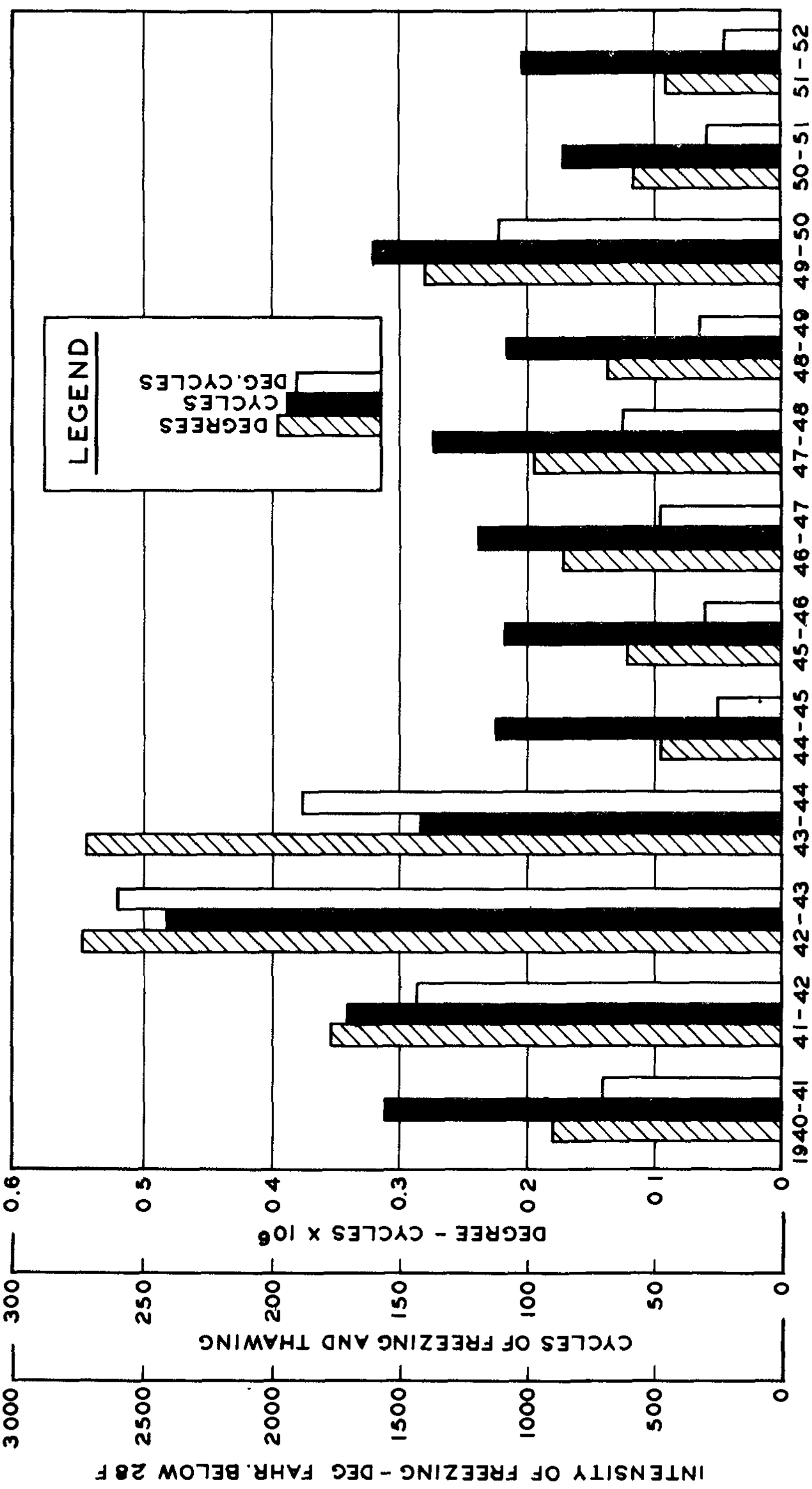

FIG. 2 ANNUAL VARIATION IN EXPOSURE OF CONCRETE SPECIMENS SUBJECTED TO FREEZING AND THAWING AT TREAT ISLAND, ME.

freezing. Fig. 2 shows a graph on which, for each winter since 1940-41, are plotted the number of freezing and thawing cycles, the cumulative total number of degrees below 28 F measured at the center of a 6 by 6 by 36-in. concrete prism at the lowest temperature reached in each freezing cycle, and the number of "degree-cycles", a value obtained by multiplying the total number of cycles by the number of degrees below 28 F. The 28 F temperature was chosen as the approximate freezing point of concrete saturated with sea water.

In all cases where the size and shape of the specimens permit its use, the method, A.S.T.M. (1952), of evaluating the effects of the exposure involves successive determinations of fundamental transverse frequency. Since this property varies directly with Young's modulus of elasticity, the evaluation is calculated as percentage change in Young's modulus. Briefly, the fundamental transverse frequency of a specimen is determined by causing it to vibrate by the action of a variable frequency driver placed in contact with the center of the surface of one side. For each specimen, the fundamental flexural frequency is that at which the amplitude of vibration is greatest. A microphone or pick-up, placed against the side of the specimen near the end, and connected to a suitable voltmeter, ammeter, or oscilloscope, is used to indicate relative amplitude of vibration. The frequency at which the maximum amplitude is indicated is noted and recorded. The actual value of Young's modulus of elasticity, or, in actual practice, the value of relative modulus expressed as a percentage of the modulus at the time of installation of the specimen are calculated as follows:

$$E = CWn^2$$

where:

E = Young's modulus of elasticity in pounds per square inch,

C and W are constants for a specimen of given dimensions and weight, and

n = fundamental transverse frequency in cycles per second.

hence:

$$P_c = \frac{n_1^2}{n^2} \times 100$$

where:

P_c = relative modulus of elasticity, per cent, after c cycles of freezing and thawing,

n = fundamental transverse frequency at 0 cycles of freezing and thawing, and

n_1 = fundamental transverse frequency after c cycles of freezing and thawing.

EXPOSURE RESEARCH ON CONCRETE IN SEA WATER

PROGRAMS OF INVESTIGATION

Concrete specimens have been exposed to natural weathering for the major programs of investigation described briefly below. Results indicated "to-date" are as of the spring of 1951. Results of the 1952 inspections were not available in time for inclusion in this paper. The results of certain programs conducted by or for other organizations are not included herein, notably 66 beams installed in December 1941 for the Portland Cement Association in connection with its long-time study, PCA (1948); 82 reinforced concrete beams for the Reinforced Concrete Research Council; and 48 beams from a joint study by the National Sand and Gravel Association and the Public Roads Administration. The latter two groups were installed in December 1951.

CEMENT DURABILITY PROGRAM

The largest single study was an investigation started in 1939 to develop data to permit the preparation of specifications for portland cement which would insure a greater durability in concrete exposed to severe weathering than provided by the existing specifications. At that time the use of air entrainment was essentially in the discovery stage. A total of 52 samples of cement and clinker from 47 mills distributed throughout the United States was included in the study and only six of these were "treated cements". It is interesting to note that so little was known about the effect of the "treated cements" that the air content of the concrete used in the exposure specimens was not determined and those engaged in the investigation believed that about every known test for cement and concrete had been included. Hence, the only data available on air content are based on unit weight tests and microscopic examination of the hardened concrete. The fact that, of the original column specimens exposed at Treat Island in 1940, the 32 containing the treated cements were the only ones showing no deterioration in 1942 was outstanding evidence that the "treatment", or as it later developed, air entrainment, contributed marked durability to portland-cement concrete. In 1951, after eleven winters' exposure (1472 cycles of freezing and thawing) 19 of the specimens remain and all are in excellent condition with values for dynamic modulus of elasticity of more than 100 per cent, except for one which is 94 per cent.

After eleven years of exposure at St. Augustine, Fla. only eleven of the 152 specimens from this same series have failed and, of these, eight contain cements with calculated tricalcium aluminate contents in excess of 12 per cent.

The specimens exposed to moderate and no-weathering in New York developed no data of significance except that the cement showing poorest durability at Treat Island showed map-cracking after 5-1/2 years of moderate weathering.

NATURAL CEMENT PROGRAM

This investigation compared the relative durability of concrete containing blends of natural and portland cement, with similar concrete containing portland cement, both with and without entrained air. Ninety-four columns representing the various test conditions were installed at Treat Island in October 1942. The results of the program to date indicate that if the natural cements contain appreciable quantities of an air-entraining addition their blending with portland cements greatly improves the durability of concrete.

AIR-ENTRAINMENT PROGRAM

It was known from the results of the cement durability program previously described, that the use of cements containing air-entraining additions materially increased the durability of concrete. The principal purpose of the air-entrainment program was to determine whether this increase in durability was due to the air-entraining addition, as such, or was due to the entrained air. In October of 1943, 182 concrete columns were installed at Treat Island. The specimens represented five plain portland cements, five portland cements with 0.03 per cent flake Vinsol resin, five portland cements with 0.01 per cent neutralized Vinsol resin, and five portland cements with 0.02 per cent neutralized Vinsol resin. All of the Vinsol resin, both flake and neutralized, was interground with the cement at the mill. Two types of coarse aggregate were used; rounded siliceous gravel, and crushed trap rock. The fine aggregate for all concrete was natural siliceous sand. Except for the concrete containing neutralized Vinsol resin, one-half of the series was mixed under normal air pressure and one-half in a vacuum of 60 mm of mercury. The purpose of the vacuum mixing was to preserve the effect of the air-entraining agent, if such effect was a factor, but to remove the entrained air. After seven winters of exposure 142 of the 182 specimens remain in sound condition. The results indicate the superior durability of concrete containing entrained air, and of concrete mixed in air, compared with that mixed in a vacuum.

CURING PROGRAM

In February 1943, 300 concrete specimens were prepared to study the effect of the method of curing on the durability of the concrete. The specimens (3-1/2 by 4-1/2 by 20 in.) were sawed from laboratory-cast slabs, at the end of the curing period. After sawing, the specimens were installed at Treat Island in 100 boxes (3 per box) in such a manner that only the finished or formed surface was exposed. The test variables were ten different liquid membrane-forming compounds, water, waterproof paper, and one integral curing material. After an exposure of four and a half winters only one of the specimens had failed. In May 1947, to intensify the exposure, the specimens were removed from the insulating boxes and exposed to weathering effects on all surfaces. Failure of specimens has not progressed to the point where any conclusive results can be drawn.

DURABILITY OF HORIZONTAL JOINTS IN MASS CONCRETE

The durability of construction joints in mass concrete as affected by the method of consolidating the upper surface of the lower lift, and the method of cleaning the top of the lower lift; the use of grout between the lifts has also been studied. Cores extracted from four projects were subjected to detailed examination in the laboratory and were exposed at Treat Island:

Project	Number of Cores Exposed	Date Installed
John Martin Dam	10	October 1942
Bluestone Dam	40	December 1943
Dale Hollow Dam	23	December 1943
Norfolk Dam	4	March 1943

While the interpretation of the results was complicated in some cases by difficulty in locating the joint, inadequate area of joint plane, or failure of the adjacent concrete coincidental or prior to joint failure, the following general conclusions were drawn:

1. The quality of the joint is governed largely by the quality of the concrete immediately below it.

2. Adequate development of the influence of type of clean-up, use of grout, and type of surface-curing was not obtained principally because the size of the cores that could practically be taken from the structures were too small in proportion to the area of the joint surface to secure true representation.

DURABILITY OF MASS CONCRETE

Several specimens of mass concrete have been exposed at Treat Island in recent years for durability testing as follows:

a. Sixty-eight 6-in. diameter cores drilled from laboratory-made mass concrete (3-1/2 and 4 bags per cu yd cement factor, coarse aggregate graded to 4 in. in size), were exposed in June, 1946. Surfaces were formed against oiled wood, and against absorptive form lining. Each core was encased, except for the formed face, in a 2-in. thickness of mortar. All of the cores are sound to-date, the surfaces formed against absorptive form-lining being generally in better condition than those formed against wood forms.

b. Six 8 cu ft concrete cubes representing concrete with 2.0, 3.0, and 4.0 bags of cement per cu yd, Type II and Type IV cement, entrained air, and aggregate proposed for use in the Pine Flat Dam, were exposed in September, 1947. All of the cubes are in good condition showing no deterioration except for minor raveling of the edges and some spalling. Three 10-in. cores from concrete of similar cement factors, all containing Type IV cement and the aggregate actually being used on the Pine Flat project were exposed in the fall of 1949. In the spring of 1951 all of the cores had relative dynamic moduli of elasticity of more than 100 per cent.

c. In the fall of 1949, nine 10-in. diameter cores taken from a 70-cu-yd test block of Prepakt concrete were exposed at Treat Island. All have relative E values in excess of 100 per cent.

d. Thirty-six 10-in. diameter cores and ten 8-in. diameter cores representing mass concrete with vacuum-treated surfaces were installed at Treat Island in the fall of 1949. These cores also are still in excellent condition.

e. Eleven 10-in. diameter cores drilled from mass concrete in the Mt. Morris Dam were installed at Treat Island in the fall of 1949. These also had relative E values in excess of 100 per cent in 1951. Exposure of all of the above specimens has not been sufficient to permit the drawing of any comparisons or particular observation except that it is interesting to note that the blocks containing only 2.0 bags of cement per cubic yard, exposed in 1947, are still in good condition.

PASSAMOQUODDY TIDAL POWER PROJECT

The purpose of this installation was to find a cement-aggregate combination which would give the greatest assurance of durability for the proposed concrete structures. In connection with the study 43 concrete columns were installed in 1935. After approximately 600 cycles of freezing and thawing, six of the most durable specimens were installed on the new rack in October, 1940. Three of these columns contained plain portland cement similar to the present Type II but manufactured by a mill which permitted introduction of crusher oil by leakage. The other three specimens were made with aluminous cement. After approximately 1200 cycles of freezing and thawing the three plain concrete specimens were removed due to severe deterioration. The three columns containing aluminous cement are still sound after more than 1850 cycles of freezing and thawing.

ST. LAWRENCE SEAWAY

The purpose of this investigation was to find an aggregate that would give the greatest assurance of durability in concrete for the proposed structures. Twelve columns representing twelve test conditions involving four aggregates were exposed at Treat Island in October 1941. The results of the exposure indicated the relative durabilities of the aggregates in question and indicated that neither the presence of entrained air nor the use of absorptive form-lining are sufficient to protect concrete containing definitely unsound aggregate even though the use of such precautions greatly improves the durability of concrete made with sound aggregate. The specimens were discarded in April 1946 to provide needed space on the rack.

ADMIXTURE PROGRAMS

A series of investigations was made to determine the relative effect of several commercial admixtures on the durability of concrete. The first series of 12 columns exposed at Treat Island in March, 1942, studied the effect of a non-air-entraining admixture when used in concrete made with plain and with air-entraining portland cement. The second series (specimens made by the National Bureau of Standards) consisted of 116 columns exposed in October, 1943, represented another commercial admixture, not basically air-entraining in nature, with 13 cements. The third series, represented by 90 columns exposed in November, 1944, included specimens representing eight commercial admixtures, seven of which were air-entraining in nature, and plain cement. The general indications are that air-entraining admixtures greatly improve the durability of concrete. Non-air-entraining admixtures tested in general showed little improvement in durability over plain portland-cement concrete, but did not adversely affect the improvement in durability shown by air-entraining admixtures when used in conjunction with them.

COMPARATIVE FIELD AND LABORATORY DURABILITY PROGRAM

This investigation will attempt to correlate the field exposure condition at Treat Island with laboratory freezing and thawing in the standard accelerated freezing-and-thawing equipment used in the laboratory, Wuerpel and Cook (1945).

For this program 300 specimens were installed at Treat Island in December, 1948. Half the specimens were 3-1/2 by 4-1/2 by 16-in. beams and half were 6 by 6 by 30-in. beams duplicating the smaller specimens. One-hundred and fifty 3-1/2 by 4-1/2 by 16-in. companion beams were frozen and thawed in the laboratory. Six coarse and eight fine aggregates were used.

It is felt that at least one more winter of exposure is necessary before definite trends can be developed and any positive correlation established. It may well be that no definite correlation exists and if so it also should be understood that any correlation with this particular exposure condition does not imply any correlation with any other natural weathering. However, a few general observations of indications to date may be of interest. Twenty-six aggregate combinations in the smaller beams and 35 aggregate combinations in the larger beams had relative moduli of over 90 per cent as of May, 1951, at Treat Island. The larger specimens appear to stand the exposure somewhat better than the smaller ones. Comparing the two exposures on a cycle basis, the laboratory test appears to be more severe, however, on a degree-cycle basis the Treat Island exposure appears more severe. Nine of the 48 aggregate combinations in the smaller specimens, of which six were with quartzite coarse and three with non-chert gravel coarse aggregates, showed greater deterioration at Treat Island after 355 cycles than did their companions after 300 cycles in the laboratory apparatus. A statistical analysis is now being made to determine the degree of correlation that may exist.

MISCELLANEOUS PROJECTS

Beam specimens from the Rome Air Depot, Rome, New York, and the Syracuse Air Depot, Syracuse, New York, with and without air-entrainment were exposed at Treat Island in 1941 and 1942. In both instances the plain cement concrete failed during the first winter and the air-entrained specimens are in sound condition. Some of the sound specimens have been removed to make room for additional specimens on the rack. Also, 42 specimens from the John Martin Dam, Caddoa, Colorado, were installed on the Treat Island rack in 1941. Twenty-four were columns made from wet-screened job concrete and 24 were cores from the downstream face of the dam. The purpose of the installation was to observe the influence of method of surface preparation on the durability of concrete. The columns were cast against oiled wood forms, and against two different types of absorptive form-lining. The cores represented surfaces cast against oiled wood forms, cast against one type of absorptive form-lining, and produced by screeding. The cores were exposed in boxes and were surrounded by fine gravel, only the formed or screeded surfaces being exposed to weathering. All specimens had either failed or were discontinued after two winters. The results indicated that the surfaces cast against absorptive form-linings tested were of appreciably greater durability than those obtained by the use of oiled wood forms or by screeding.

CONCLUSIONS

The results obtained from the exposure of more than 2500 concrete specimens to natural weathering over a 16-yr period are the basis for the following statements:

1. The entrainment of properly regulated quantities of air is the most important factor in the improvement of the durability of concrete under severe weathering conditions that has been developed by these investigations. At Treat Island well-made concrete of good quality materials will not ordinarily withstand the exposure for more than one winter unless the concrete contains the proper amount of entrained air.

2. The use of various non-air-entraining admixtures did not appear to be of material benefit in increasing the durability of plain concrete but were not harmful in that they did not appear to decrease the durability of air-entrained concrete.

3. The use of air entrainment does not protect concrete which contains unsound aggregate.

4. The blending of natural cement with plain portland cement greatly improves the durability of concrete if by so doing the proper amount of entrained air is produced in the concrete.

5. No definite trends in the effect of curing conditions on

durability have been revealed.

6. Aluminous cement produced highly durable concrete.

7. The use of absorptive form-lining improves the durability of concrete surfaces.

8. The quality of horizontal construction joints appears to be governed primarily by the quality of the concrete at the top of the lower lift.

9. The use of cement with a tricalcium aluminate content in excess of 12 per cent has resulted in concrete that is non-durable in warm sea water. The use of Type II cement with a tricalcium aluminate content less than 8 per cent appears warranted for such exposure.

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CHAPTER 19

CORROSION STUDIES OF STEEL PILING IN SEA WATER IN BOSTON HARBOR

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INTRODUCTION

A vital problem which has resulted from our present economic justification of the use of steel piles in all of the Port's waterfront terminals is the corrosion of the piles in sea water. The selection of some type of steel piles was not made on the basis of greater resistance to fire and marine organisms, but for purely economic reasons. Timber piling has its marine borer problem, and steel has its corrosion. In this article there are three thoughts which I hope to convey to every engineer engaged in coastal engineering, namely:

(1) Each project's corrosion consideration should be treated as a distinct and separate study in designing a pile-supported structure, based on investigation of adjacent local conditions.
(2) There is nothing universal about the pattern or rate of corrosion to be expected; it is subject to many variables.
(3) Provisions or measures to mitigate or eliminate corrosion should have economic justification consistent with the planned life expectancy of the study.

With the greater use of steel piles we naturally become more conscious of this vital problem of corrosion. In order to keep informed as to the correctness of our design criteria and to evaluate protective coatings for the mitigation of corrosion, the Authority has started a regular program for the periodic examination of steel pile structures in Boston Harbor. At the present time we have utilized outside marine divers in collaboration with the William F. Clapp Laboratories, Inc., for our own structures. It is expected that we shall very shortly revive our own diving unit, which prior to World War II made examinations of all timber structures in the Harbor. The diver would have to undergo an extensive course in corrosion pattern recognition, location of possible accelerated activity, measuring procedures and description of observations; otherwise the value of the work would be questionable. Whenever a steel pile is removed, regardless of by whom, we attempt to get all the corrosion information and data before the pile is disposed of. We also have been most fortunate in obtaining the cooperation of other interests in the Harbor who have steel structures, in making similar examinations for their own information as to the existing condition of the piles and for our compilation of corrosion data.

Investigation and studies made of existing structures in Boston Harbor indicated a great range of pattern and rates of corrosion which refute widespread concepts accepted by many design engineers, such as that:

Fig. 1. Corrosion pattern on unencased pile.

Fig. 2. Corrosion pattern on encased pile.

(1) Corrosion rate is uniform throughout the entire length of pile, and that a 10 per cent additional allowance of steel in the section of the member would provide a life expectancy of 40 to 50 years. The 10 per cent has been increased by many to 20 per cent.
(2) All types of steel piles have the same corrosion pattern and rate.

In this short article it is impossible to cover the subject with any degree of thoroughness. Some of the findings described herein are no doubt startling to many. These are just a few of the actual conditions, to emphasize the need for better understanding of the problem, and not with any intention of being spectacular.

TYPICAL EXAMPLES

At one location two steel "H" bearing piles were removed from sea water after 11 years of service. These two were about 12 feet apart. One had concrete encasement to low water, and the other no protection of any kind. The corrosion pattern was entirely different in each case. The encased pile had a more or less uniform corrosion rate consisting mainly of grooves as shown on Figure 1. The plain pile had a severe attack at low water with a pattern consisting of pits as shown on Figure 2. The maximum loss of metal in the cross-section of the encased pile was about 3 per cent, whereas the other had a loss of 10.4 per cent occurring at approximately the extreme low water line, with an insignificant loss for the remainder of the length. The loss of 10.4 per cent of the plain pile does not appear alarming until you analyze it from a structural standpoint. The loss occurred at the edges of the flanges, which would normally be expected in galvanic action resulting from the differential of oxygen concentration. Structurally the loss takes place in the worst part of the member, the weak axis.

As an example of the seriousness of this loss, take a 14 inch 73# bearing pile with an unsupported length of 48 feet. A reduction in the radius of gyration from a 10 per cent loss of metal at the ends of the flanges results in a 44 per cent decrease in the allowable load on the pile. Under this degree of corrosion attack it would be impractical and uneconomical to provide a sufficient cross-section of metal to sustain such a loss over a great period of time.

Since this accelerated attack took place below the normal water level and the pile was covered with a thick growth of marine organisms, discovery of the accelerated attack is not likely by visual observation. Unless periodic examinations by a competent diver are made to discover such critical conditions, collapse of the structure would be imminent.

Pursuing this investigation and study further along, the owners of a wharf made a random inspection of 8 piles in place, selected as representative of the areas in which each was located. This was done in order to verify the following finding:

Low water accelerated attack does not occur on the encased piles, and grooving with a more or less uniform corrosion loss is also a characteristic of the encased piles.

This finding was found to be correct. It was also found that the seaward most piles had evidence of greater corrosion rate. The reason for the latter is rather difficult to explain, since it occurred not uniformly, but somewhere between the lower zone of a breaking wave and the location of the propeller of a deep-draft vessel. In attempting to ascertain the cause of the grooving corrosion pattern, a number of experts were consulted. It was the consensus of opinion that this peculiar pattern was caused by stress weaknesses in the mill scale as a result of the rolling, storage, transportation or handling process. The breaks in the mill scale become anodic and the scale cathodic in the galvanic cell. This is evidenced by the calcareous coating on the mill scale surrounding the corroded areas. The rather difficult observation to explain is the presence of no grooving on the plain pile, which had a pit pattern. The question remains as to whether this is a coincidence or whether there is a definite relationship.

At the location of our new Hoosac Pier No. 1 the former terminal had a steel sheet pile bulkhead which was installed in 1935 and removed in 1943. Thickness measurements taken at the time of removal showed tremendous loss in a short period of time. At this point the salinity of the water is reduced by an inflow of fresh water above this location, and is also polluted from a sewerage overflow discharge line. Four random piles were measured, but there was no similarity of attack common to all. The record contains no apparent cause for this abnormal severe attack, which contains very little localized pitting. The legs or flanges of the DP 1 Section had a greater rate of corrosion than the web. The more or less uniform attack from the mud line to the top of the pile in the splash zone is difficult to understand in view of well-established concepts as to the areas of accelerated attack, namely, just below mean low water, and the splash zone above mean high water. At this location in 1948 the steel sheet pile bulkhead type of quay wall was constructed for the new Hoosac Pier No. 1. Before driving the sheeting that portion above 3 feet below mean low water was flame cleaned of mill scale and rust, and then coated with bitumastic enamel. Annually the sheeting is examined by a diver for accelerated corrosion. To date nothing had developed requiring remedial action.

Also involved in the submarine examination of steel piles is our desire for information on grooving adjacent to welded splices caused by galvanic action between the bare metal at the weld, which is anodic, and the cathodic mill scale. To date we have found little evidence of such phenomena.

Cylindrical steel piles and caissons have also been examined, and found to have a good performance as regards corrosion. The absence of edges prevents galvanic current concentrations, especially at the low water area. The caisson examination indicated a very puzzling observation. The caissons on the opposite side of the pier are not coated. Because of the very large diameter, and the fact that they are filled with concrete, an accurate verification would be difficult. There is very little pitting, indicating a uniform pattern.

ECONOMIC JUSTIFICATION OF PROTECTION

There are many protective systems to mitigate or completely eliminate corrosion for a given period. The anticipated life of the new piers of the Port of Boston Authority is between 40 and 50 years, after which the piers become obsolete, and should be reconstructed to fit the needs of that time. Other structures may have different life expectancies; therefore, the measures to attain the required life should be justified economically consistent with the planned life, meaning that one with a 15-year life should not necessarily have the same protective measures as one with a 40-year life.

The greatest boost given to overcoming sea water corrosion is cathodic protection, for it has proven positive, it can be installed at any time after completion of a structure, and it protects the most difficult section, which is below the surface of the water. Although cathodic protection is the only practical means of prohibiting corrosion below the surface of the water, there are many protective measures for the zone above, such as metal jackets, organic coatings, metal coatings, greases and wrapped plastic coats. In spite of the number of methods for the tidal and atmospheric zone, there is still a need for improvement - some of the methods are too expensive, and other have too short a life for economic justification. There are many factors to be considered in protective coatings. The degree of preparation of the steel surface required; setting time, especially for recoat jobs in the tidal zone; the number of applications for a complete system; resistance to abrasion; the size of the project; and the anticipated protective life. In sandblasting steel there are 3 classes which indicates the wide range one can expect in cost and quality:

(1) Primary, which removes all loose scale and rust;
(2) Intermediate, which removes all scale to gray coating;
(3) Complete, all scale taken off to bright metal.

An average good coating will have an effective life of about 5 or 6 years. Poor coating, including poor workmanship, will fail in less than a year, with the best going to an 8 to 9 year life.

This is an example of the economic analysis that has been made on insuring a pier life expectancy of 40 to 50-years as it affects the corrosion of piles. The recommended protective system in this case consists of cathodic protection below the water and organic coatings above. The cost of this system of protection for the period starting upon completion of the structure is as follows:

Cathodic Protection	\$360,000.
Protective Coatings, using re-coating every 5 years	\$480,000.
	\$840,000.

The steel sheet piles could be replaced in 20 years to stand up for another 20-year period, at a cost of \$560,000. This is the plan accepted to insure the investment for the anticipated life.

The logical action to take in many cases, if it is determined that protective measures will be needed immediately upon completion of a large waterfront structure, is to use a more costly type of substructure which will not require a continuous protective program. However, in cases where accelerated corrosion is discovered after completion of a structure, the only alternative is the system using cathodic protection for below water, and coatings or jackets above.

CONCLUSION

This article is intended to stimulate the need for giving more thought and analysis to each waterfront corrosion problem, for continual observation of existing structures, and research for more economical protective measures. The findings and discussion herein precented are not meant to be taken as any criteria for the corrosion pattern, rate, and means of mitigating corrosion. It is impossible to cover adequately and properly in this short space the subject of marine corrosion of steel piling.

ACKNOWLEDGMENTS

In connection with the compilation of corrosion data of steel piles in Boston Harbor, I wish to take this opportunity to express my appreciation for the cooperation and assistance given me by Mr. A. P. Richards, Director of the William F. Clapp Laboratories, Inc., Duxbury, Massachusetts; Commander A. C. Husband, Public Works Officer, Boston Naval Shipyard; and Commander F. A. Tinsler, District Engineer, First Coast Guard District.

CHAPTER 20

PREVENTION OF DETERIORATION IN WATERFRONT STRUCTURES

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The Bureau of Yards and Docks is greatly interested in prevention of deterioration and the preservation of marine structures such as wharves, piers, quay walls, mooring dolphins and various other structures which form a part of all of our coastal Naval stations.

The Bureau is what may be called the "Public Works" agency for the Navy. It is responsible for the design and construction of structures that comprise the Naval Shore Establishments which are necessary to support our vast fleet. The marine structures are a large item in this great system of bases. When you consider the number of installations scattered all over the world you will realize that the engineers charged with the design, construction and maintenance of these port facilities are faced with great responsibilities. These installations present many problems — not only due to their vastness — but also because they are installed and must be maintained under all types of climactic conditions ranging from frigid to tropical.

The protection of timber structures against deterioration and destruction by marine borers has always been of major concern to builders of marine structures and to maintenance engineers.

In the early 1880's a rather novel and original experiment in the control of marine borers was tried out by a New York inventor. This inventor owned a string of scows which he used in his business. They were attacked by Teredo. To prevent damage from the marine borer he is said to have smeared the bottom and sides with coal tar and sprinkled over it an even layer of Copenhagen Snuff. Being a hater of tobacco he believed that this would be the most disagreeable thing he could do to the Teredo. Unfortunately, however, for the experiment, the coal tar rubbed off in spots and the Teredo got in. There is no proof that the snuff was of any value in preventing Teredo attack. More scientific methods are now being used in the evaluation of protective measures against the attack of these animals.

The marine borers are found to some degree in all oceans. In other words, no place in the world where salt water occurs seems to be entirely safe from their attack. They are even known to exist in fresh water.

Some waters are highly infested while other waters appear to be only slightly infested, or even free of the marine borers. Those waters which are highly infested sometimes become less infested. The marine borers apparently disappear. In waters which have shown no infestation, the marine borer may suddenly appear in large quantities and endanger the structures in that area.

The migration of borers into harbors is being watched carefully through maintenance of control panels in various locations. A systematic replacement of these panels gives an indication of the seasons of attack, breeding periods in harbors and infestation of harbors which, up to now, have been free of marine borers.

These indicators also serve as a warning that attack may be occurring and gives maintenance engineers sufficient time to take remedial measures to prevent complete collapse of such structures. In some instances where such indicators have not been maintained, certain structures have suddenly collapsed without warning due to the inroads made on the structures by these organisms.

Short-time tests by no means guarantee that the harbor water in a certain area will continue to be free from any marine borer attacks. Therefore, the engineers and maintenance crews at no time can be sure of complete or continuing safety. Continuing and vigilant defense against the attack of marine borers must be provided for the timber harbor structures or they may be destroyed.

Harbor conditions may change from time to time due to change in sewer discharge and disposal of industrial wastes in a given area. For example, at a Navy pier on the West Coast, it was felt that there would be no need for any treatment of the piles in that area, inasmuch as other structures using untreated wood had been in place for many years. No consideration was given to the fact that adjacent to this area a pulp mill was in operation and its waste effluent was dumped into the harbor close in line with the structures. The harbor authorities and Fish and Wild Life Service forced the pulp mill to extend their effluent line to the deeper portion of the harbor in order to prevent pollution which was injurious to fish life in that area. Within a short time marine borer attack progressed to such an extent that a large appropriation of funds was required to replace piling which had been in place over the previous ten or fifteen years with no damage during the time.

Since the point of attack of marine borers lies between the water level and mudline on submerged timber, a good many precautionary steps can be taken in the design of wooden structures in marine locations. However, the precautions in design which may be taken should not give the builder or the maintenance engineer a sense of security against marine borer attack, for certain types of borers, known as the family Teredine or Teredinidae, enter the timbers by drilling tiny holes, then growing within the timber, and destroying the timber during their growth. The Teredinidae attack is generally more severe at the mudline portion of the pile, but can occur at any point that is submerged. Surface inspections often fail to detect such attack.

Of course, the Crustacean type of borer, known as the Limnoria family, can ordinarily be seen more readily on examination of piling and on timbers in that they burrow very close to the surface of the timber and the burrow shell cuts off, leaving a gradual surface erosion. One of the best ways to determine whether timber structures are being attacked by the Limnoria is the hour-glass appearance of the piling — mostly appearing at the waterline, but also may occur at the mudline.

From numerous inspections of structures and the search of literature reporting on failures of marine structures it has been concluded that important and frequent causes of short service life of pilings may be listed somewhat in this order:

1. The use of timbers which do not lend themselves to acceptable treatment.

2. The use of a poor grade of preservative which does not show acceptable service records, although it may conform to standard specifications now in use.

3. Improper treating.

4. Improper plant procedures in which the temperatures, pressures and time of treatment may be so drastic that the timbers may be seriously damaged and the value of the preservative lost.

5. The insufficient penetration and retention of the preservative for the specified desired treatment.

6. Improper handling of timbers after treatment and during construction.

7. Inadequate inspection.

Realizing that adequate inspection will lessen some of the causes I mentioned, the Bureau is now in the process of publishing an inspection and preservation manual for engineers in the construction and maintenance of marine structures. This manual will be for Navy use and is expected to contain in one volume all the necessary data for use of Navy designers, constructors and maintenance engineers.

One of the headaches of the engineer is the selection of the material to be used in a marine structure. For the most part, the problem of deterioration and prevention of attack by marine organisms on marine structures falls into two classes:

1. Those relating to the choice of the type of material and kind of piling for installation in new structures.

2. Those relating to the deterioration or destruction of piling already in place.

Problems of the first type; that is, the choice of the most suitable type of piling for a particular installation, perhaps gives the average harbor expert more difficulty than any other of his numerous worries. It is well understood that each type of piling which may be used has its merits or advantages. Among the major considerations in selection of the type of piling for a given location are:

1. The type of structure to be erected -- open wharf, closed wharf, finger pier, marginal wharf, open pier, closed pier, quay or mooring dolphin.

2. An estimate of the maximum loads to be used for design purposes.

3. The unit cost of piles based on the estimated life of the structure, including the original cost.

4. The length of piles which may be used.

5. Investigation of the possibility of piles being subjected to excessive current flow, wave action and the abrasive action of water-borne sand.

6. A study of harbor conditions, including corrosiveness of water to concrete and steel, as well as the possibility of infestation by marine borers.

7. The action of the soil in which the pilings are to be driven.

The selection of the materials going into a structure is of utmost importance. So, too, is the design of structures to prevent deterioration.

A review of the work which has been done in the past on design of structures for protection has changed the Navy's concept of conventional bracing design of structures. As a result of this change, bracing is now placed in such a way as to avoid its coming into contact with the water.

In designing timber structures exposed to marine borer attack, piling and underwater timbers should be thoroughly pressure-treated with creosote. They should be handled and driven under very close supervision for protection against any damage to the treated surface. Specifications for the protection of the piling after treatment should prohibit the use of cant hooks, dogs, chain slings, or any other mechanical handling device which might break the sealed surface of the treated area, thereby setting up a danger point for attack.

The designer should so design the structure as to prevent any boring of holes or tapping of piles within the water area. All such connections should be kept above the high-water mark where possible. Designers should also use great care in their designs to prevent any untreated timbers from coming into contact with treated timbers, especially where these parts may be submerged in salt water at any time. Specifications should also include provisions for bolt hole treatment and repair of damaged surfaces.

The protection of structures against attack after the timbers have been in place presents a problem to the engineer. There are methods which may be used, such as encasement in concrete, the replacement with new treated piles, and in some instances, the encasement in metal armor. If metal armor is used the problem of corrosion immediately enters the picture. The losses to the Navy have been great in the past. They have caused the engineer responsible for the design and maintenance of these types of structures a great deal of concern.

At this time I would like to discuss the research work that the Navy Department is sponsoring on methods of obtaining more positive means of prevention of deterioration caused by marine borers and the destruction the piling through decay.

The Navy has consistently sponsored a program of inspection and study which includes practically every U.S. Naval installation in the world. This work is being carried out by the William F. Clapp Laboratories of Duxbury, Massachusetts. It consists essentially of the replacement of test panels in waters of the harbors with periodic examination of these panels to determine whether infestation is occurring, the types of borers prevalent in that area, and the breeding season of the borers, as well as the intensity of attack at various seasons of the year.

This work has been carried out over many years and the results have been compiled in a harbor report.

In addition to the ecological studies being carried on with the Clapp Laboratories, the Bureau is also sponsoring a series of studies at the

University of Miami Marine Laboratory, Coral Gables, Florida. These studies cover the physiological aspects of the borers. From these studies we hope to find better ways of protecting our wooden structures from marine borer attack, through a deeper knowledge of their general structure, feeding habits and their reaction to treatments of various types. The information which has been gathered so far by various approaches shows promise that the research will be more than worth the expenditure which is being made on these studies.

In addition to the physiological studies, the University of Miami is also cooperating with the Naval Research Laboratory by making bioassays of various elements extracted from creosote. This effort is directed at devising a more positive method of determining when creosote is of a grade which would be effective in protection against the inroads of the borers. These studies may result in changes in the present specifications for creosote materials.

The Naval Research Laboratory is making studies of creosotes and analyzing and extracting various groups of elements.

The Bureau is also sponsoring research at the Naval Civil Engineering Research and Evaluation Laboratory at Port Hueneme using the spectroscopic method of evaluation of the purity of the creosote. These show promise of being very valuable in determining whether creosotes have been diluted or whether, in their distillation, certain important elements have been inadvertantly excluded from the distilled product. The Bureau feeling is that adequate specifications and inspection of the timbers which will be treated for use in marine structures are highly important.

In order to devise methods of prolonging the life of the structures, the continuing of long-range research program on the constituent parts of creosotes and their effect is absolutely necessary.

The Navy has been and still is greatly concerned in endeavoring to improve the present specifications for creosoting materials. Accordingly, a good deal of research is being done at the present time in order to try to obtain a more positive specification for the treatment of waterfront structures which will insure the selection of materials to give the longest service.

We have come a long way since the days when the ravages of marine borers remained unchecked due to lack of scientific knowledge.

We have made great strides in devising scientific techniques to cope with marine borer attacks. Despite these advances, there are still further problems to be solved for improving our present methods and devising new methods to protect our structures.

Part 5
CASE HISTORIES OF COASTAL PROJECTS

CHAPTER 21

CASE HISTORY OF THE CAPE COD CANAL

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No more interesting or appealing subject than the Cape Cod Canal could be assigned to one who is engaged in the study and development of navigation in New England. This sea-level canal, located 50 miles south of Boston at the narrow neck of land joining Cape Cod to the mainland, principally serves coastwise shipping to and from Boston and Northern New England. While it was only completed in 1940, no one should entertain the thought that it is of recent origin.

Construction of the Cape Cod Canal was first considered and explored in 1623 by Captain Myles Standish of the original Plymouth Colony. He traversed the route by boat and portage to reach the south shore of the Cape. There a trading post had been set up to encourage commerce with Dutch merchantmen sailing to and from New Amsterdam or New York. Undoubtedly, Captain Standish had the typical soldier's aversion to walking if you could ride.

The idea of a canal is immediately suggested by the geography of the area. A low swale of alluvial formation, nowhere more than 30 feet above mean sea level, crosses the neck of land where Cape Cod joins the mainland. The Monument River drained this valley to the south, and Scusset River to the north, and their headwaters were not more than three-fourths of a mile apart. Surveys, investigations, and studies recommending construction of a canal followed in steady procession for three centuries. It has been said that, every grain of sand along the proposed route has been made the victim of detailed study. The only obstacle to its construction was that of "where to find the funds with which to build it." New England, as many of you appreciate, has never enjoyed the reputation for hastily adopting new improvements, particularly when money was involved.

To avoid the problem of strong tidal currents all proposals made up to 1862 included plans for a lock canal, but in that year the earliest known suggestion for a sea-level waterway was advanced. Generally, the proposals all followed the same route, although 4 other routes were seriously considered. Three proposals were based on a canal about 18 miles to the east from Hyannis to Barnstable; and one even further east, where the Cape hooks to the north at Orleans. (See Figure 1.) Construction of a sea-level canal from Buzzards Bay to Sandwich was finally undertaken by a private company in 1909 and completed in 1916. This Canal, 25 feet deep and 100 feet wide, cost $13,000,000. The southerly approach through the shoal water of upper Buzzards Bay bent to the east of Hog Island and Mashnee Island, closely following the shore of the

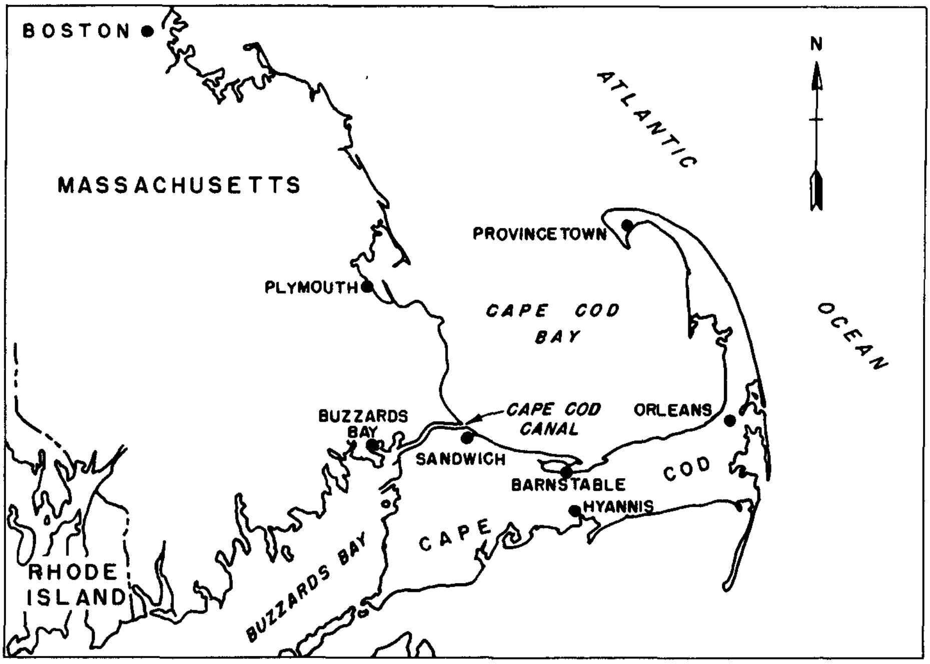

Fig. 1. Vicinity map - Cape Cod, Massachusetts.

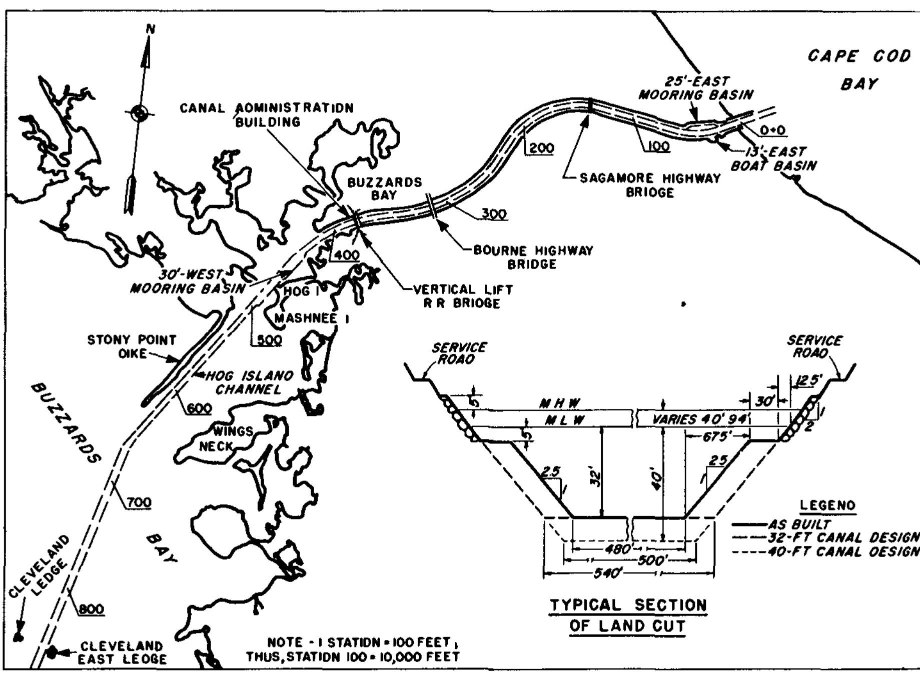

Fig. 2. Cape Cod Canal - plan and section.

Cape. (See Figure 2.) This tortuous channel required navigating a zig-zag course, frequently veering off on 50 degree tangents.

To make matters worse, currents in the Canal ranged from 4 to 5 knots. These currents were due principally to the variation between the 4-foot tide at Buzzards Bay and the 9.4-foot range at Cape Cod Bay. In addition, the Buzzards Bay tide precedes the Cape Cod Bay tide by about 3 hours, further complicating the problem. Due to these strong tidal currents and the limited channel width, several serious accidents to shipping occurred. Within a few months of opening of the Canal, two separate ship sinkings occurred, each occasioning a loss of $275,000. One ship blocked the channel for three months. From 1915 to 1930, 75 accidents were reported for a total loss of $800,000. The narrow width limited the Canal use to one-way traffic, with resultant navigation delays and inconveniences. It did not take long for the Canal to develop a bad reputation with mariners, and shipping and tolls failed to meet expectations.

The owners, having what then proved to be an uneconomical investment, sold out to the Federal Government for $11,500,000 in 1928 after eleven years of negotiations. Under the particular circumstances, it is not surprising that the local interests relaxed their traditional opposition to Federal intervention and endorsed Federal assumption of a waterway project of doubtful merit. The purchase of the Canal by the United States was made with the full knowledge that it would require considerable improvement to be a practicable waterway.

Two alternative plans of improvement of the waterway by the Government were considered and model studies made of each. The plan initially recommended in 1931 provided for its modification to a lock canal 32 feet deep, generally 250 feet wide. The lock itself would have the same dimensions as those in the Panama Canal, 40 feet deep, 110 feet wide, and 1,000 feet long. The lock was designed with both middle and end gates for ships not requiring the full length. Tests were made for lifts from 3 feet to 7 feet, the ordinary differences in tidal elevations at any one time between Cape Cod Bay and Buzzards Bay. Tests were also made for a lift of 12 feet, which is greater than any known instantaneous storm head.

The test ship was a model similar to the standard type of cargo vessel, but 600 feet long, 75 feet wide and loaded to 35,000 tons displacement. These dimensions are larger than those of most cargo vessels in use today. Tests were made on a scale of 1 to 40. It was found that the lock could be filled in 7 minutes for the 12-foot lift with resultant stern hawser forces of 15 tons. These results were considered entirely satisfactory. Construction of a lock would make the Canal practically currentless, enabling safe two-way traffic in a 250-foot width.

Experience gained in operating the Canal from 1928 to 1934 and the low costs incurred in widening the channel to 170 feet by that date, led to a re-examination of the relative merits of a lock canal as compared to a sea-level canal. It was found on this re-analysis that a sea-level waterway would be more economical and would possess certain advantages over a lock canal. The design of the sea-level canal provided for a depth of 32 feet and a width of 540 feet. This width was primarily established to allow a future Canal width of 500 feet at a depth of 40 feet. (See Figure 2.) A width of 500 feet was greater than the length of ships ever expected to use the Canal. In the event a ship ever got caught in the current, it could not swing sideways and ground out at both ends, damming the Canal. This proposed width was considered sufficient to afford safe two-way traffic, despite the strong tidal currents involved. The layout of the Canal was such that the sharpest curves were of a radius of curvature of a mile and a half.

The model study for the sea-level waterway was made in 1935 by the Massachusetts Institute of Technology under contract with the Corps of Engineers. A model about 111 feet long and 34 feet wide was constructed, reproducing the waterway at a horizontal scale ratio of 1 to 600 and a vertical scale ratio of 1 to 60. Three different channel sections were studied as follows:

1. The 170-foot by 25-foot Canal section then existing.
2. A 500-foot by 40-foot Canal.
3. A 540-foot by 32-foot Canal.

The study of the 170-foot by 25-foot Canal section was made with a tide homologous to that observed December 21, 1934. Observations at 9 points along the length of the model were correlated with observations taken at identical stations along the prototype. The then existing bridges, with draw openings of 140 feet, and approaches on closely-spaced pile bents, were simulated by sills in the model, causing equivalent loss in head. Boulders of 1-1/2 cubic yards or more in size, scattered along the bottom of the Canal at locations known from dredging records, were simulated in the model by stones of 3/4 inch to 2-1/2 inches in size.

Comparison of high and low water profiles of the model and prototype and the profiles at times of maximum head indicated an average difference in water surface elevations of less than 0.1 foot with a maximum difference of less than 0.5 foot. Upon completion of the initial tests, studies were then made on the 500 by 40-foot section and the 540 by 32-foot section. These studies of the enlarged sections were made with a straight approach channel through upper Buzzards Bay, rather than the previous zig-zag approach. The studies were made both with and without dikes alongside the proposed Buzzards Bay straight approach channel.

It was found that the 40-foot Canal would have a low-water profile up to 0.5 foot higher in the easterly 4 miles of the Canal, and up to 0.3 foot lower from there to the State Pier at the west end of the land cut of the Canal. However, the 32-foot Canal would have a low-water profile generally about 0.3 foot lower than in the existing 25-foot Canal. The velocities for mean tides would range from 2 to 3 knots in both enlarged Canals, a reduction of 20 to 40 percent from the velocities in the 170 by 25-foot Canal, resulting from the observed tide of December 21, 1934. Unfortunately, no measurements were made in the model of the 170 by 25-foot Canal based on mean tides, so it is not possible to state exactly how much of this decrease in velocity is due to the difference between the tide of December 21, 1934 and mean tides, and how much is due to the change in Canal section. The tide of December 21, 1934 was roughly ten percent greater than a mean tide.

It was determined that the greater part of the reduction in velocities was due to construction of dikes alongside the approach channel in upper Buzzards Bay. These dikes in effect extend the length of the Canal proper about 1-1/8 miles, or about 15 to 20 percent, thereby reducing the slope and velocities an equivalent amount. However, the dikes were built primarily to eliminate hazardous cross-currents in the approach channel. These cross-currents resulted from a large part of the tidal flow following the old zig-zag approach channel bed.

The model tests proved conclusively that an enlarged sea-level Canal was entirely practicable, and construction was initiated.

Experience gained in construction of the original Canal demonstrated that excavation in the dry was more economical than dredging. This procedure was followed insofar as it was practical. Examination of the original waterway indicated that it would be necessary to revet the banks 5 feet above mean high water to 5 feet below mean low water to prevent erosion of the banks and consequent shoaling of the Canal. The design found most suitable was 18 inches of riprap, the stones being run of the quarry ranging from 50 to 300 pounds each, laid on a 6-inch blanket of crushed stone. In areas where excavation in the dry below low water was impracticable due to the proximity of the existing Canal, the riprap was hand-placed down to mean low water, and then a sufficient volume of stone was dumped against the bank. The theory was that gradual erosion of the bank below low water in that area would cause the stone to settle into place, stopping further erosion.

Approximately 54,000,000 cubic yards of excavation were involved in the construction of the Canal, including 15,000,000 cubic yards in the original channel. This is about one-fourth of that required in the construction of the Panama Canal. Approximately 15 percent of the material was excavated in the dry, 30 percent by hydraulic dredges, and 55 percent by dipper dredges. The construction by the Government was commenced in 1932 and completed in 1940. The final width obtained was 480 feet, making the Cape Cod Canal the widest in the world.

Two high level fixed highway bridges and a vertical lift railroad bridge were constructed over the Canal at the time it was widened. These bridges were designed to permit a clear channel width of 500 feet, and are 135 feet above mean high water. Protective riprap about the bridge piers has decreased the horizontal clearances at the channel bottom to

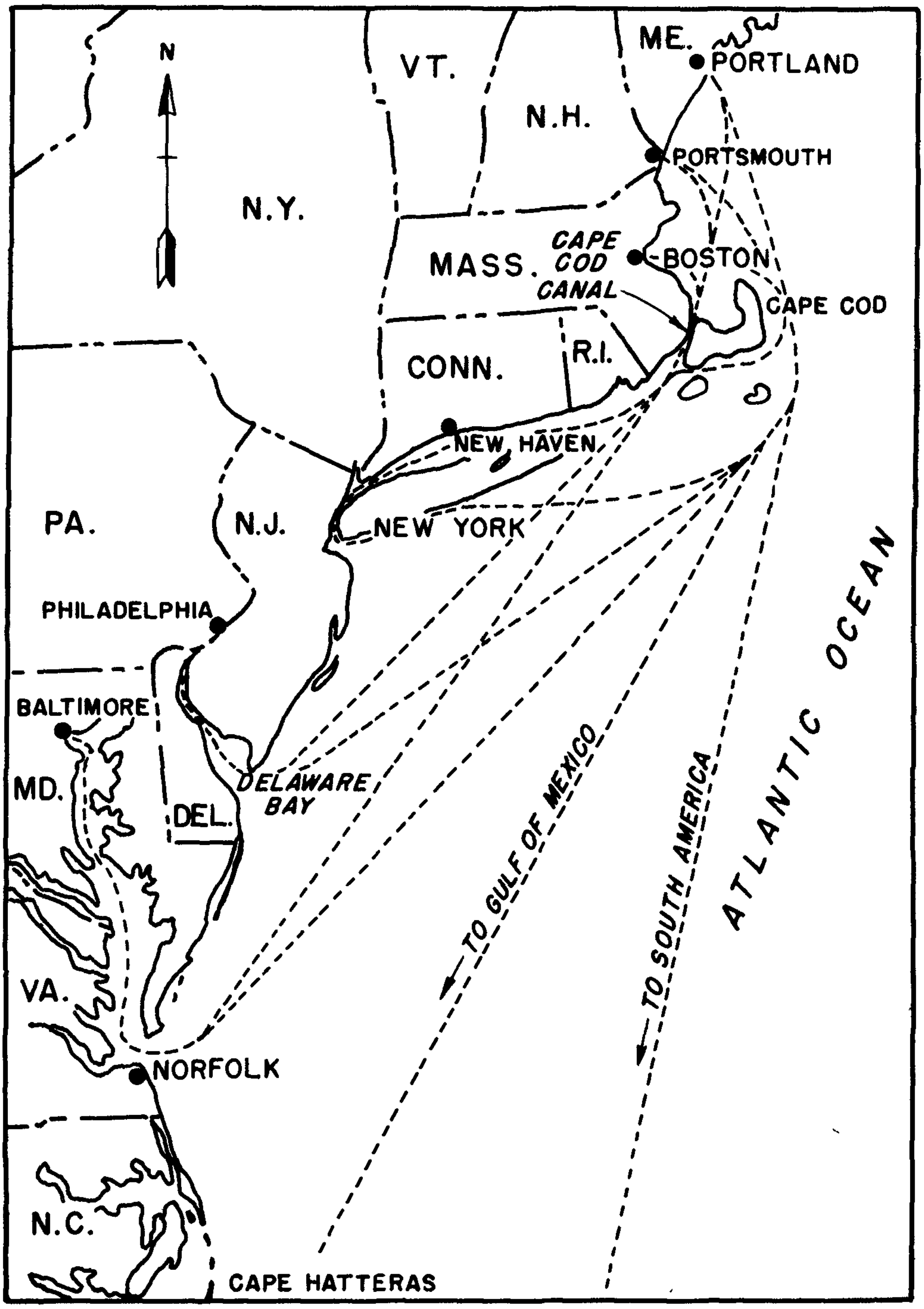

Fig. 3. Sea lanes - traffic passing Cape Cod.

465 and 480 feet. The 500-foot lift span of the railroad bridge is the longest such span in the world.

Having reviewed the history leading up to the building of the Canal, and the design and construction of the Canal, our interest now turns to the benefits realized from this waterway project.

Since the purchase by the Government in 1928, the annual tonnage shipped through the Canal has increased about 9 times, or from 1,500,000 tons to 13,500,000 tons. The present volume is about half of that carried by the Panama Canal and about equal to that carried by the Welland Canal in Canada. The number of ship transits per year in the Cape Cod Canal have increased since 1928 about 50 percent, from 9,500 to 13,500. It is obvious that the size of the ships using the Canal has noticeably increased, the average tonnage per ship being 6 times that formerly carried. It is interesting to note that the growth of total volume of commerce has far exceeded the population and industrial growth of New England, indicating a shift to the Canal by commerce formerly travelling other routes. This expanding use is the best proof of the economic value of the Canal.

The increase in the average size of the ships reflects the transition from the moderate-sized barge and package freight steamer to the bulk carrier, and the continuous growth in size of these bulk carriers. The design of the Canal was evidently successful beyond original anticipation, as it is now daily used by vessels larger and more heavily laden than contemplated in the construction of the Canal to its present dimensions.

The waterway saves from 65 to 150 miles in navigation distances, depending upon the particular route that might otherwise have been used. (See Figure 3.) This saving in distance is a significant percentage of the total shipping distances to the Atlantic coast and Gulf ports. If an average saving in distance of 100 miles is assumed, there is a total annual saving for the deeper draft ships using the Canal of well over 500,000 miles. Although exact figures of savings could not be obtained without a laborious compilation of statistics, it is conservatively estimated that over $3,000,000 a year is saved in shipping costs due to time savings alone. This figure can be substantiated by relatively simple computations. Let us assume that the entire 13,000,000 tons of commerce passing through the Canal is carried in the larger ships known to be the more economical to operate per ton mile. There are 5,000 ships per year of greater than 15-foot draft using the Canal. That would indicate an average cargo per ship of 2,600 tons which would appear reasonable when it is considered that a good deal of the commerce to New England is one-way commerce such as coal and oil, and the ships return empty. The class of vessels concerned has an average speed of about 12 knots. Therefore, the 100-mile saving in distance would amount to about 8-1/3 hours' time. Operating costs of these ships averages about $75 per hour. Each ship, therefore, would save $625, or the 5,000 ships would save $3,125,000.

The Federal Government has expended about $43,000,000 to date on Cape Cod Canal as follows:

Purchase price	$ 11,500,000
Improvement of the Canal	20,000,000
Maintenance since 1928	5,700,000
Operation since 1928	5,800,000

Therefore, the average total annual costs of the Canal, including interest and amortization of the Federal investment and annual maintenance and operation costs, amount to less than $1,500,000, or less than half of the value of the annual saving in shipping time. Of even more importance, however, is the benefit due to avoidance of the hazardous route around Cape Cod. For the 20 years preceding the original construction of the Canal, an average of 50 ships a year were wrecked rounding Cape Cod, with an annual loss of life of 15 persons, and an annual loss of vessels and cargoes of well over $500,000.

It is true that the Canal itself has not been without accidents, particularly before its purchase and widening by the Government. However, the annual number of accidents or wrecks in the Canal up to the time of its purchase by the Government was 6 and the average annual value of losses or damages suffered was about $75,000. No loss of life was recorded. Since completion of the Canal widening in 1940, there have been an average of less than 3 accidents or wrecks per year, mostly minor in nature, for an annual loss suffered of $50,000. This figure is exclusive of the sinking of the Arizona Sword in 1951 at a net loss of about $500,000. The contributing factors in the sinking of that vessel are presently being investigated and there is an element of doubt as to whether this loss can be fairly attributed to navigation conditions in the Canal.

You will appreciate that the Canal is particularly valuable in time of war affording a protected route safe from submarine attack. It might be well to state at this time that the tonnage figures previously given do not include military and naval vessels.

Congress has directed the Corps of Engineers to ascertain whether or not the existing project should be modified. You will recall that I stated that the width of the waterway was determined in 1935, and was based on the expectation that ships of greater than 500-foot length would not use the Canal. The standard deep-draft vessel of that period drew 25 feet. However, ships built since 1935 generally have been of lengths greater than 500 feet and drafts of 30 feet or more. As the older ships are replaced, the merchant fleet is rapidly becoming predominantly characterized by ships beyond the design capacity of the Canal.

It has been claimed that only 40 percent of the deep-draft ships which might use the Canal do so. It is known that fully loaded deep-draft tankers usually avoid the Canal. This is borne out by the fact that about 60 percent of the coastwise commerce in the Port of Boston is in petroleum as compared to 15 percent of the Canal commerce. It will be recalled that two large tankers, the Fort Mercer and the Pendleton, broke in half off Cape Cod in the same storm in February 1952. Both tankers were fully loaded with oil from Louisiana. One tanker was headed for Boston, one for Portland, Maine. Over 50 lives were lost on this one occasion. It should be recognized that all the traffic passing Cape Cod will probably never go through the Canal. In certain cases, such as ships arriving from some South American ports, the direct route may be east of the Cape. However, it is probable that the present depth is a factor in the limited use now made of the Canal. Therefore, in view of presently changing conditions, re-study of the Canal design at this time is highly desirable. As a basis for this study of possible future deepening or enlarging of the Canal, a more detailed review of the previous design, construction experience, and actual resulting conditions should prove of value.

The Canal was dredged to a width of 480 feet rather than the 540 feet originally contemplated. (See Figure 2.) Berms 30 feet wide, 5 feet below mean low water, were left on both sides of the Canal to protect the revetment until additional experience could be gained as to bank erosion that might occur. The revetment was generally placed as originally planned, from 5 feet below mean low water to 5 feet above mean high water. The Canal banks at mean low water are 700 feet apart, the width designed to permit a 500-foot channel width at a depth of 40 feet, with banks sloping one vertical on 2-1/2 horizontal.

The maximum velocities experienced in the Canal range from 3.5 to 4.0 knots, or roughly about 10 percent less than in the original waterway, and roughly about 10 percent more than indicated by the model tests for a Canal 32 feet deep and 540 feet wide.

The annual maintenance of the Canal has averaged about $225,000 since its purchase by the Government in 1928. This figure has remained uniform throughout the period of Federal ownership of the Canal despite decline of the dollar value. This $225,000 includes maintenance of the bridges, roads, and buildings as well as maintenance of the Canal itself, but excludes an annual cost of $250,000 for purely operational purposes. Maintenance of the channel depths over the past ten years has required dredging an average of 350,000 cubic yards of material a year at an annual cost of $90,000. Maintenance of the 100-foot Canal from 1916 to 1928 had required an average annual dredging of 250,000 cubic yards. Thus, it is seen that the increase in channel maintenance has been proportionately much less than the increase in the channel width. Comparison of channel surveys of different years indicates an average annual erosion, or scouring, of the channel bottom of 370,000 cubic yards. However, this rate of erosion is apparently diminishing, the volume in the 4 years 1944 to 1948 being about half that in the first 4 years of the completed Canal, 1940-1944. Although the Canal was only dredged to a depth of 32 feet, holes 20 to 30 feet below the channel bottom have scoured out at several locations. There are at least 15 different locations where this scour occurs. One of these holes is more than a mile long. The reasons for the scour at the particular localities is not readily apparent. At some of the locations there is an irregularity in the Canal section, such as at the east end of the Canal at the jetties, and where the Canal widens at the mooring basins. Some of the eroded areas are in the vicinity of structures built within the Canal prism, such as at the bridge piers and the bulkhead at the State Pier. However, at about half of the locations there is no evident reason for erosion. No detailed study has been made of the character of the bottom materials at the various locations along the channel, which might explain the areas of erosion. It would appear from comparison of the volumes of channel scour and channel shoaling that the material eroded from the channel bottom remains in the channel, depositing at various points to form shoals.

Despite the berms and revetment protecting the Canal banks, some erosion of these banks has occurred. This erosion has principally occurred on the outer bank at the Canal curves. Bank restoration and stabilization costs have averaged about $40,000 a year over the past 8 years. It is not considered that the erosion is due in any large part to the wash from ships. Regulations have been established to permit restriction of vessel speed to 9 knots at slack water, or 6 knots against the current and 12 knots with the current. However, it has not generally been found necessary to exercise these restrictions and within reason, the ship master is permitted to use his own judgment as to proper ship speed for safe navigation.

What can be learned from this project review? It is dangerous to attempt to draw conclusions of general applicability from the history of one waterway project. Local conditions and numerous other factors make each project a unique study in itself. However, experience can and should serve as a guide, if viewed carefully as to particular conditions affecting the project. For instance, the Cape Cod Canal project was studied and was the subject of proposed construction over a period of some three hundred years. The volume of coastwise commerce, the size of ships in this traffic, and their propelling power and navigating characteristics, changed considerably over this period. It is fairly obvious why a sea-level Canal with its attendant tidal currents was not even considered during the era of sailing craft prior to 1860. The disadvantages of the navigation conditions of a tidal Canal steadily diminished in importance as the merchant fleet became characterized by the more powerful and more easily handled steamers and motor vessels. However, even up to the purchase of the Canal by the Government in 1928, opinion leaned toward a lock canal as preferable, perhaps partly because of the unsatisfactory experience with the narrow sea-level Canal since 1916. The unusual circumstance of the depression of the early nineteen-thirties, with funds made available for work projects that could be immediately undertaken, and the low excavation prices then obtainable, changed the economics of the picture. Largely because of these economics, but also with other factors in mind, the Canal design was changed from a lock canal to the sea-level canal.

The experience of the original 25-foot Canal, 100 feet wide, indicates that a project of scant dimensions is not necessarily an economy, and may in fact be wasteful and fail to develop the potential benefits.

Earlier studies of the Canal concluded that currents would be increased in a seaway canal of enlarged cross-section. However, these earlier studies did not take into account the modification of tidal head that would result from the Canal enlargement. The model study, undertaken as the only means of determining the total effect of the Canal enlargement, indicated currents in the enlarged Canal would be of the same order of magnitude as in the existing Canal. The model tests further indicated the necessity of extending the waterway by dikes to eliminate hazardous cross-currents at its original mouth. The model tests revealed that this extension of the Canal would decrease the currents 15 to 20 percent. The report on the model tests cautioned that the velocities be considered qualitative rather than quantitative.

It is not yet definite whether the Canal has reached a stable condition, and whether the existing erosion and shoaling will continue. However, with the results of the previous studies, and with the 10-year experience of the 32-foot Canal, the present study of further enlargement of the Canal can proceed on a sound basis. It might be said then, on the basis of this case history, that modification of this Canal, or construction of any similar navigation project, rests in order of importance upon the following:

First: - Economics, or the weighing of costs and benefits.

Second: - Timeliness, or the urgency of need to adapt to changing times.

Third: - Adequate design, based on past experience and tests, with a reasonable allowance for future trends.

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CHAPTER 22

DEVELOPMENT OF MODERN PORT FACILITIES IN THE PORT OF BOSTON

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INTRODUCTION

Boston is known throughout the world for its soul stirring part in the history of this great nation, its famed "Clipper Ships", its educational institutions, even its baked beans, and, last but not least, its port which has played such a strong role in world trade from the days of the country's first settlement.

Historically speaking, the Port of Boston was the first major port in these United States, and had its beginning early in the seventeenth century, shortly after the arrival of the Puritans. It was only logical that the city became a center of waterborne commerce. Its storm-sheltered harbor and deep waterways, with very little current, practically free of estuary sedimentary deposits; its closeness to the open ocean; and its excellent hinterland accessibility—all contributed to the development of the port. Boston Harbor comprises a tidewater area of about forty-seven square miles, with a shore line of more than one hundred miles. The maximum current velocity is less than one knot per hour. It is only seven miles from the open ocean to the center of the waterfront terminal area.

Originally all the commerce of Massachusetts Bay was with England. In 1641, when the Civil War there disrupted normal trade, Boston quickly sought other markets, and thus began its foreign trade, with salt fish as its principal export product. This port was the principal point of entry when the vast immigration commenced, with the arrival of fifteen hundred colonists in 1630. During the days of the Clipper Ships, the finest vessels were built in Boston's yards, and carried the name and trade of the city to every corner of the globe. Then came the days of steam transportation, and, with the arrival of the BRITANNIA in 1840, the establishment of regular passenger service by the Cunard Line between Great Britain and the United States. The Port of Boston prospered, and has continued to this day to maintain its strong place in world maritime trade. Boston had the first Naval shipyard in the country, where some of our early warships were constructed.

In the history of the Port, there has always been a strong influence of progressiveness, which has numerous times set the pace for the rest of the nation to follow. For example, in the early part of the twentieth century, the State of Massachusetts constructed the largest drydock in the world in Boston; it is still in excellent condition, and is in use by the Navy. The Commonwealth Pier No. 5, built in 1912, was an ultra-modern pier far in advance of its day, for both general cargo and the handling of passengers. It was the forerunner of our present-day modern-design waterfront terminal, and is still considered to be one of the finest piers in the United States. At about this time also, the Boston Fish Pier was constructed by the State for the fishing industry, which had previously used the picturesque but inadequate small piers along the old Atlantic Avenue section of the waterfront. Boston's fishing industry set the pattern for the world in the efficient processing of fish on a large scale.

FORMULATION OF MASTER PLAN

In 1941 the Commonwealth of Massachusetts realized the necessity of embarking on a program of modernizing the terminal facilities for the handling of general cargo. Authorization and funds were provided for the construction of the first new pier at the site of the former Mystic Piers Nos. 46 and 47, but World War II caused a postponement of the commencement of construction until after the termination of hostilities. When conditions permitted the starting of the project in 1945, construction costs had risen so much that previous funds allocated were insufficient. Along with the need to re-estimate the project, it was also found that the functional layout did not ideally fit the post-war requirements of efficient interchange of cargo between land and water transportation. For this reason and because of changed conditions brought about by the war involving increased size and capacity of general cargo vessels, palletizing of goods, greater use of mechanized material handling equipment, increase in cargo movement by truck transportation, and anticipated large increase of trade, the Commonwealth decided that a comprehensive study of the entire port should be made and a broad plan for maintaining and improving our competitive position in the future formulated. In line with this thought, the Port of Boston Authority was empowered to embark on a Master Plan for providing adequate modern terminal facilities, for which purpose an initial capital outlay of $19,700,000. was approved by the Legislature. (Fig. 1).

The survey and study involved every aspect of the entire port with its correlating influence and relationship to all activities essential to waterways, land transportation, labor, industry, and the hinterland served by the port. Certain basic facts were derived from this study as follows:

(1) Sectional development priority for chronological sequence planning.

(2) Certain existing obsolete piers were strategically located for maximum efficiency in the interchange of waterborne cargo by rail and highway transportation, meeting the following fundamental essential requirements for port development: close to deep approach roadsteads and main highway arteries, easily accessible by waterfront labor, maximum construction economy, and minimum disruption of normal business activity of the area.

DEVELOPMENT OF MODERN PORT FACILITIES IN THE PORT OF BOSTON

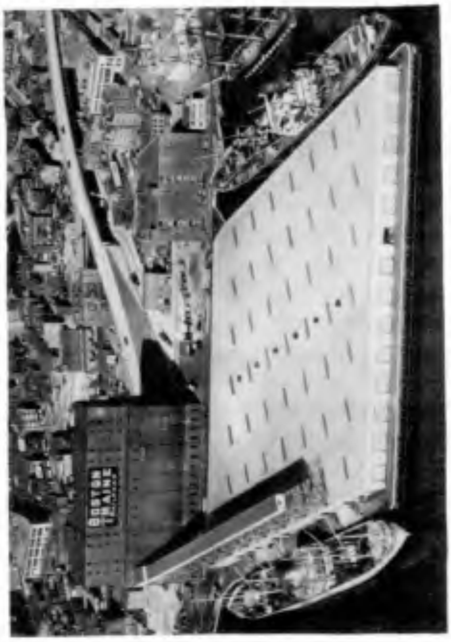

Fig. 2. Aerial photograph of Hoosac Pier No. 1 - the first step in the Master Plan. Completed in 1950.

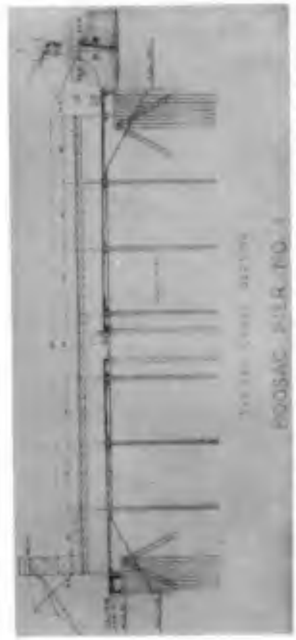

Fig. 3. Typical cross section of Hoosac Pier No. 1.

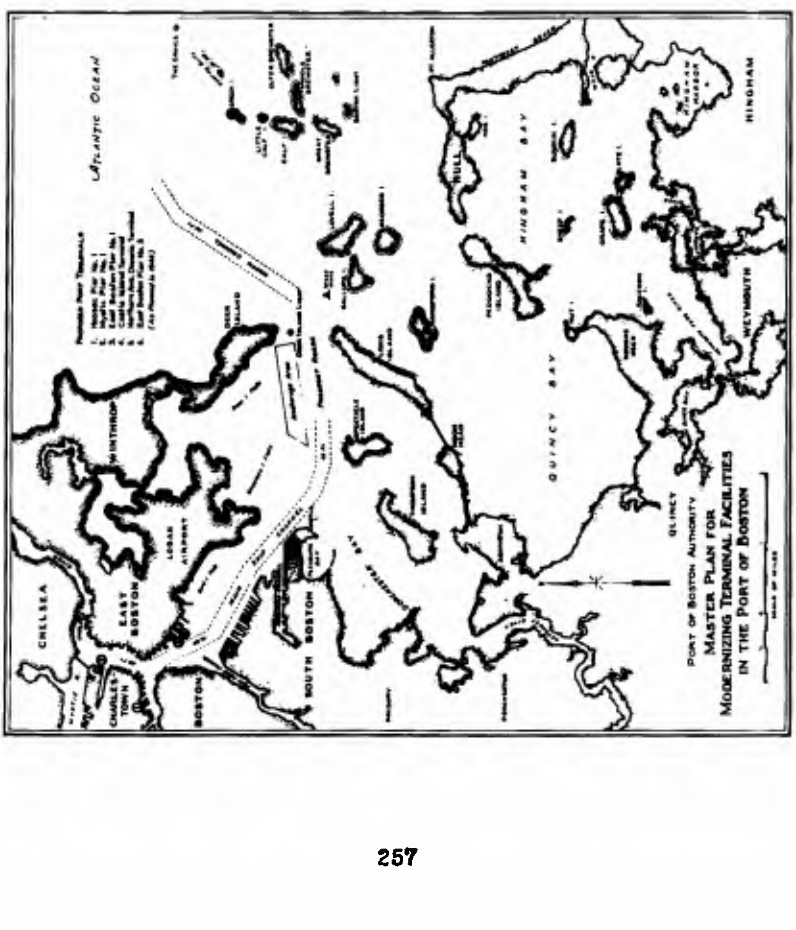

Fig. 1. Plan of Boston Harbor showing location of proposed port terminal facilities in the Master Plan.

Terminals constructed on new sites hitherto unused for such purposes not only were found to be more costly, but would not have a satisfactory operational layout of supporting facilities.

Further study indicated that the necessary additional margin of capacity to handle anticipated future expansion of waterborne commerce could be provided by a definite increase in the efficiency of the cargo handling process of the new terminal facilities without utilizing greater waterfront area.

To gain this efficiency in the cargo handling process, it was proposed that an improved functional layout would attain the objective. The functional layout considerations will be discussed later under design aspects.

In the formulation of the Master Plan it was decided to locate the projects strategically so as not to favor any one particular section of the port or any one railroad. In this manner the benefits to the city and waterfront would be more evenly divided, and traffic congestion on land and water minimized. From the port survey and study it appeared that the first two piers should be constructed in the Charlestown section of the city, the third in East Boston, and the fourth in South Boston.

Our plans for the various projects are coordinated in a long-range plan so that future development may not be prejudiced because of lack of vision, and may be accomplished with as little as possible disruption or displacement of waterfront business and establishments.

Because of existing commerce demands, it was not possible to proceed with the construction of more than one project at a time without jeopardizing the normal flow of business in the port. One pier at a time has been scheduled for construction, with the displaced business of that pier being absorbed by other terminals in the port. Each new pier being constructed will provide that necessary margin to handle the business of the pier being replaced while reconstruction is going on, as well as to take care of such new business as may be acquired after completion of each step in the overall development.

In establishing the modernization program for the Charlestown sectior of the port, the Hoosac Tunnel Docks were designated as the first project, to be called Hoosac Pier No. 1, as this facility was unusable. The second to be chosen was the site of Mystic Piers Nos. 46 and 47, to be called Mystic Pier No. 1, which would be commenced after the completion of the first step. Mystic Piers Nos. 46 and 47 were in use, and their business would be transferred to the new Hoosac Pier without creating a hardship. By the time the second step was completed there would always be a margin i business expansion. We have found our predictions more or less correct ir this regard (See Figures 2 and 3).

The third step was the location of two piers in East Boston, namely, Commonwealth Pier No. 1, which was very inefficient, and Grand Junction Pier No. 2, which was badly in need of major structural rehabilitation.

The fourth step is the revision of a military facility constructed early in World War II into a modern, efficient commercial terminal at Castle Island in South Boston.

Every year the needs of our waterborne commerce are re-studied with the object of reflecting current forecasts. In 1950 two additional projects were added to the Master Plan, the modernization of passenger facilities at East Boston Pier No. 3, and the Northern Avenue Oceanic Shipping Center in South Boston. The completion of the latter two was expected to maintain our competitive position until 1960. Because of increasing port commerce, consideration is presently being given to adding another step for materialization in 1959.

Another phase of port commerce which has been of much concern was the modernization of our 50-year-old export grain handling facilities. A study of this problem resulted in a decision that only 2 of the 3 grain terminals warranted modernization. From this decision Hoosac in the Charlestown District and Grand Junction in the East Boston District were selected for improvement at a cost consistent with the benefits to be derived on a competitive level with grain facilities of other North Atlantic ports. The third grain facility, the Mystic Elevator in Charlestown, was judged to be obsolete and scheduled for demolition.

DESIGN CONSIDERATIONS

In the planning of these piers, stress is placed on the economic aspect of functional layout, design and construction consistent with maximum efficiency of operation and low maintenance. Since the enabling legislation requires a 20-year lease with amortization of total construction costs at the rate of 3 per cent per year, every bit of Yankee thriftiness and ingenuity has to be used to meet the demands of lessees. There is nothing ornamental or monumental in the pier design - it is all purely functional with only such architectural treatment as may be given without increasing the cost. The appearance of our transit sheds is attractive, a result which has been attained by skillful use of simple lines, and the careful arrangement and use of low cost materials.

The first aspect of the basic layout to be studied for maximum efficiency of the cargo handling process was protection of labor and cargo against foul weather. It must be borne in mind that during the winter, we do have some snowstorms and cold rains. A working wharf apron width of 25' was selected as giving maximum protection against damage of walls of transit shed from cargo handling by ship's gear, minimum exposure of longshoremen to the elements, adequate clearances for rail tracks and highway vehicles for direct cargo interchange. A spacious transit shed to enclose the entire transfer operation once the cargo reaches the shed permitted a more expeditious and efficient movement of cargo. Such an arrangement not only has a better psychological effect on the worker, but involves no delays or slow-downs regardless of weather. When a snowstorm occurs it is only necessary to plow the snow into the sea and open a passage-way to a close-by main artery. All new transit sheds will have a minimum of 100,000 square feet per main berth. This exceptionally large covered area permits maximum use of high equipment speeds in both assembly and dispatch of goods and also low stacking of cargo. In most cases the new project has the same or less berths than the terminal being replaced, but the increased efficiency of the layout plus larger covered storage area permits the accommodation of more ships with a shorter turn-around time.

Since most of the trucking companies in the port having business c the pier find it more advantageous for their vehicles to go into the shed to pick up or deliver goods at the point of assembly, our basic layout has a double ramp on each side of the trackwell for truck entrance into the shed. This arrangement permits complete circulation on either half or the whole pier. For those trucking companies which desire to load at platform height, truck docks are provided on the shore end of the shed. The wide column spacing and overhead clearance permit uninhibited freedom of movement of trucks and material handling equipment.

Because of wide variance in the percentage of cargo moving by rail and truck at the various piers throughout the port, a basic layout was decided upon which was equally advantageous to both modes of transportatic The wide finger pier with a minimum of two rail tracks down the center for flush deck loading and one on the work apron not only met this condition, but minimized interference with the other transportation. The traffic pattern in the layout was given considerable thought with the view of eliminating congestion and interference with the efficient movement of vehicles and cargo.

It was also believed that an increased efficiency would be attain by improving the working conditions of pier labor, namely, a light-colore interior to enhance good artificial and natural lighting, excellent wash-room facilities, and a special forming or shaping hall for use during inclement weather.

In layout, all the new terminals follow the basic pattern of a finger pier with one-story transit sheds, except for the Castle Island Te minal, which is a marginal wharf about 4000 feet long. The piers have wo ing aprons twenty-five feet wide, with flush railroad tracks, ship water supply and fire-fighting hose connections for each berth, and adequate il mination with non-glare lights for night work and security. The terminals have one-story transit sheds of fire-resistant materials, with a twenty-foot overhead clearance which is adequate for maximum stacking of cargo; at least two railroad tracks in a depressed well in the center of the shed for flush loading of railroad cars; truck docks for platform-height loading of trucks; ramps for the entrance of trucks into the sheds good natural lighting for daylight operation and excellent artificial lighting for night work; the pre-action type of sprinkler system for fire protection, with an automatic and manual fire alarm system; power-operate trackwell bridges for flexible movement of cargo from one side of the she to the other; warm rooms for perishable cargo; and offices for steamship and terminal-operating interests. An added adjunct which is entirely new in ports is the supporting utility building adjacent to the pier, housing a forming or shaping hall for longshoremen, and the repair shops for cargo-handling equipment. The plans provide for adequate open storage and parking areas, with good flood lighting. The entire terminals are enclosed with an eight-foot high chain link type of fence to provide maximum security at the least cost. These new piers each have large railroad classification yards adjacent to the piers, and are close to main arteries that serve the port's hinterland.

Incidentally, recently for the first time I received a complaint that we were doing our job too well - a longshoreman acquaintance reported workers are not happy about the new projects, pilfering is awful tough!

Some of the seemingly insignificant phases of waterfront terminal design are often neglected, although they greatly affect the operation and efficiency of the cargo handling processes. A small matter such as the proper location of Customs control would have a direct bearing on whether there is truck congestion on a pier, or whether there is fast movement of motor vehicles on to and off a pier.

Another item is the layout of the terminal to control pilfering. This aspect of pier design has been taken into consideration by us in our plans, and the effort has proven successful. It is estimated that pilfering on our new Mystic Pier has been practically eliminated. This is a very noteworthy accomplishment when one understands the problem of the shipping companies in large ports. In some places it has been assumed that nothing can be done to curb this illegal practice which has a direct bearing on the cost of handling goods at a port.

The 9½ foot mean tide differential and a considerable snow load in the winter have placed certain difficulties and restrictions on our terminal designs. The live load criteria set up for the design are as follows:

Snow 30 p.s.f.

Storage 600 p.s.f.

Wheel H 20-S16-44 + 30 per cent Impact

R. R. Tracks Cooper E60 + 20 per cent Impact

The cargo doors are of the rolling steel shutter type and have an alternate pattern. The doors are at least 16 feet high by 18 feet wide.

Our standard fire protection system for the new piers is a departure from the conventional criteria of the past. The entire shed area is uninhibited by firewalls and the uncertain value of stand pipes and hose racks. The fire protection consists of three more or less interconnected systems: (a) manually-operated fire alarm stations, strategically located throughout the pier and specially distinguished by lights and color to permit quick location in an emergency, are connected to the City Fire Department alarm system; (b) central supervisory fire alarm which is

Fig. 4. Interior of typical waterfront transit shed showing excellent natural lighting. Note hydraulic operated trackwell bridge.

Fig. 5. Brilliant apron lighting at Hoosac Pier No. 1 permits efficient cargo handling both night and day.

Fig. 6. Interior of transit shed at the new Hoosac Pier No. 1. Excellent lighting permits transfer of cargo between ship, rail, and highway carriers at night.

connected to the sprinkler system; (c) pre-action type of sprinkler system, with adequate draft curtains, covers every part of the transit cargo shed and offices. Also located around the pier are some hand extinguishers for ambient fires.

The alarm system, when actuated, sets off a large horn which not only alerts everyone on the pier as to a fire but also indicates the exact location. This eliminates any loss of time by the fire-fighting units in locating the fire upon their arrival on the pier. This heretofore new approach to the fire protection problem results from a study of pier fires consistent with a good functional layout and cargo operations. The primary move of the discovery of a fire is to actuate an alarm which will bring trained fire fighting units immediately to the scene, to supplement the protection of the sprinkler system.

The sprinkler system used is the pre-action dry pipe system actuated by rate of rise of ambient temperatures. Hose stations are costly to maintain, and without organized personnel they become of little use in fighting a fire. Because of the unsteady employment of longhsoremen and freight loaders, an organized fire-fighting unit would not be practical.

The lighting of the piers has been given special study with the view of providing the best illumination as a means of increasing the efficiency of the cargo handling process. There are skylights for daylight hours and incandescent electric lights for night work. The artificial lighting has been designed for low cost installation and operation consistent with the following requirements (See Figs. 4, 5, and 6):

1. maximum utilization and efficiency of lighting energy
2. low cost and ease of maintenance of the system
3. uniform distribution of illumination
4. minimum depreciation of light from dust and dirt accumulation on lighting units.

Since the interiors of the shed, including structural frame, will be painted with a semi-gloss protective coating, the appearance of the structure and the lighting efficiency are greatly enhanced. The lighting units used in the building are of the heavy duty industrial type, dust and weathertight, with wide symmetric distribution refractor lenses.

On the working aprons, adequate non-glare lighting is provided for the safe operation of ship's cargo-handling gear and cargo handling on the wharf. These lighting units are flush mounted on the wall to eliminate damage from, and any interference with, the swinging of cargo between ship and shore. Also, these heavy duty units have high stressed or tempered prismatic refractor lenses for proper light distribution and impact resistance.

The plan for modernizing the Hoosac and East Boston grain facilities consists of increasing the ship-loading capacity from 10,000 to 30,000 bushels per hour; provision for loading a minimum of four ship hatches simultaneously; installation of remote-control power-driven winches for operation of the shiploading spouts; an up to date communication and interlocking conveyor control system; and adequate power outlets for any arrangement of new grain-trimming machines. The modernization of the Grand Junction grain facilities in East Boston was completed by December, 1951, and the Hoosac Pier in Charlestown in April, 1951. The shiploading time of grain cargo has been reduced to less than one-half that required before.

DEVELOPMENT PROGRESS

1. Hoosac Cargo and Grain Terminal in the Charlestown District, the first step in the Master Plan, was completed on August 7, 1950, and immediately placed in operation. This large three-berth cargo terminal took almost three years to construct at a cost, including the acquisition of the site and grain handling facilities, of approximately five million dollars. The Hoosac Terminal becomes the first of the large comprehensive covered type of cargo piers in the world today. This terminal replaces the former Hoosac Piers known as Nos. 40, 41, 42, 43 and 44. Not only is the cargo handling capacity greater than the former facilities, but the pier has proven to be much more efficient, and with lower security, administration and maintenance costs (Figs. 2 and 3).

This terminal has a skew-type finger pier about 550 feet long by 515 feet wide, having a fireproof sub-structure, consisting of steel sheet pile bulkhead enclosure with a concrete relieving platform supported on timber piles, a fire-resistant shed, which covers an area of approximately four and one-half acres; a structural steel frame, bituminous concrete floor on earth fill, insulated flat galvanized steel roof deck with a built-up tar and gravel roof covering, and corrugated cement asbestos exterior walls; an adjacent battery-charging building; the supporting 1,000,000-bushel export grain elevator, and an automatic vehicle weighing scale at the entrance of the terminal for expeditious weighing of commodities passing over the pier.

Some of the distinguishing features peculiar to this pier are a vertical lift trackwell bridge, flat skylights, and ship water supply outlets in underground chambers on the wharf aprons. When the trackwell bridge is in use, a blinking red light both at the bridge and front of the building warns trainmen of the obstruction across the tracks within the building.

The grain handling facilities have been modernized by increasing the shiploading capacity to thirty thousand bushels per hour using two conveyor belts with five simultaneous points of tripping grain providing the latest portable equipment for the bagging of grain at the rate of about forty-five tons per hour in the transit shed, motorizing the movement of the ship loading spouts. A vacuum cleaning system was also installed to keep the elevator clean and free from dust, as a precaution against explosion, and to insure better working conditions. The vacuum cleaning system has outlets on all floors of the elevator to which portable suction cleaning devices may be connected. The dust and dirt are collected on the outside of the building, and provision is made to load this refuse into freight cars for disposal. With the use of the new belt trimming machines, the trimming of grain in the ship hold has been increased from three thousand bushels to almost fifteen thousand per hour.

There is no congestion of trucks that are waiting to load or take on cargo, such as is usually found at other piers. Trucks can be loaded simultaneously at truck docks at the front of the building and in the transit shed, where a two-way traffic pattern can be attained.

The cost of the pier including transit shed, trackwork, outside paving, utility building, grain gallery, utilities, fire protection system, and dredging and flood lighting, is about $12.30 per square foot of area.

2. Mystic Pier No. 1 Project, the second step in the Master Plan, was completed in July of this year at a cost of approximately $5,600,000., including the purchase of the site, and replaces Mystic Piers Nos. 46 and 47. It took approximately twenty-two months to complete from commencement of demolition of old structures. The general layout and features of the pier are similar to the Hoosac Pier. It is approximately 900 feet long by 468 feet wide with 25-foot working aprons on the side berths and a 20-foot apron on the outboard berth. The transit shed is a one-story building occupying a floor area of 246,000 square feet. This pier has a berthing capacity of three ships at one time, supported by a transit shed and one open berth for tie-up or bulk cargo operations. The characteristics and features which differ from Hoosac Pier are: the greater column spacing; three tracks in the depressed well in the center of the building; the hydraulically operated trackwell bridge which disappears into the track bed when not in use; two long canopied loading platforms connecting to the transit building, one on each side of the well tracks, and ship water supply outlets on the exterior walls of the building instead of in a chamber below the deck of the working apron. The roof deck is precast lightweight concrete slabs and the skylights are of the gable type. Part of the deck is bituminous concrete on fill and the rest reinforced concrete. (See Chapter 16).

The construction cost of the Mystic Pier is about $9.60 per square foot, which is considerably less than the Hoosac Pier. The decrease in unit cost results from maximum utilization of existing site conditions. The berths of the new pier have been dredged to thirty-five feet at mean low water on the two sides and forty feet at the outboard end.

The sub-structure, which is a wide reinforced concrete apron supported on long steel H bearing piles around three sides of an earth mole, is extremely interesting from both engineering and construction aspects. Because of the existing extremely thick underlying stratum of soft blue clay, the earth mole of the old pier was found to be too sensitive to support the much greater loads required to be imposed by the new struc-

Fig. 7. Aerial photograph of site of the Third Step in the Master Plan - proposed East Boston Pier No. 1.

Fig. 8. Sketch showing proposed East Boston Pier No. 1.

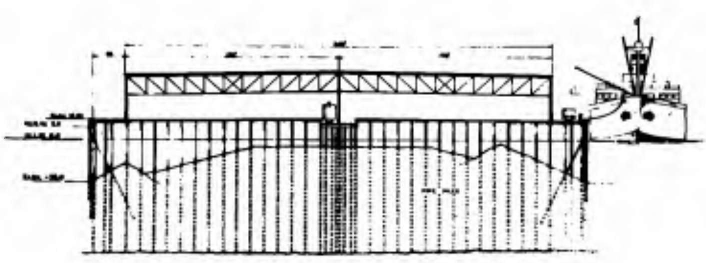

Fig. 9. Typical cross section of East Boston Pier No. 1.

ture. In order to eliminate the effect of differential settlement on the wharf building and create a greater margin of stability of the banks of the earth mole, it was found advisable to drive the piles to rock, a length of about 160 feet. In addition, 15,000 cubic yards of local low-cost lightweight aggregate were placed in raising the floor of the mole area about three feet. This expedient minimized the effect on the foundation soil from the additional depth of fill. In a period of eight months, about 70 miles of timber and steel piles have been driven, indicating the magnitude of the project and speed with which the work was accomplished.

3. Proposed East Boston Pier No. 1 in the East Boston District is the third step in the Master Plan. Besides a new pier facility involving a cost of nine million dollars, the project includes the modernization of the ship-loading grain facilities on Pier No. 4 at a cost of about one-half million dollars (Figs. 7, 8, and 9).

The plans for the project were completed in 1950, at which time contract bids for its construction were taken. However, waterborne commerce requirements at that time would not permit the withdrawal of the two existing piers, No. 1 and No. 2, from port operations. Therefore, because of this condition and the Korean crisis, construction of the pier was postponed until after completion of Step 2, but the grain facilities modernization proceeded without further delay and was completed in December, 1951.

The proposed pier has not been changed in basic layout. It is, briefly, 600 feet long and 390 feet wide, with 25-foot working aprons on the side berths and 20-foot aprons on the outboard end; a transit shed with 20-foot overhead clearance, covering an area of approximately 196,000 square feet; four sets of tracks, one flush with the deck on each side apron, and two depressed in a well at the center of the shed; the side berths to have a piping arrangement for the transfer of bulk liquid cargo from ship to tank cars; ramps for vehicular access into the shed; offices, warm rooms, and truck docks will be provided at the inshore end of the building; more parking and open storage area will be provided; a separate two-story utility building will be built which will house pier cargo-handling equipment, repair shops, a gasoline station, and a large hall for the shaping of longshoremen for the work on the pier. A chain link type of fence will be constructed around the entire property to provide the necessary security for cargo and terminal facilities. The cargo working area of the transit shed will be entirely devoid of interior columns, making it the ultimate in modern and efficient operating layouts. The pier will be entirely supported on steel pipe piles which will have a length exceeding 100 feet.

A contract for dredging, demolition and filling has been awarded. It is expected work will commence by October 31, 1952. The remaining contracts will be awarded within three months. The project is scheduled for completion in 1955.

4. The Castle Island Terminal was constructed by the United States

Army as a port of embarkation during World War II. It comprises an area of approximately 101 acres, and has potentialities for development into an ideal commercial port terminal. It has a marginal wharf which is 4200 feet long, with a controlling depth of 35 feet at mean low water at the berths. There are two existing transit sheds, each 840 feet long, by 180 feet wide, one of which is a more or less permanent structure, but too far removed from the caplog for maximum efficiency and flexibility of operation. The other is a temporary wood structure having a close column spacing which prohibits efficient cargo handling and stowage. There are also many small temporary buildings which constitute a fire hazard and will be removed in the first-stage development of the terminal. The roads of the terminal are practically non-existent, as they were originally of a temporary nature. The tremendous classification yard has a total capacity of approximately 650 rail cars. The entire terminal is lighted for night operations by banks of flood lights in structural steel towers located strategically throughout the terminal.

The first-stage development plan to convert this terminal into a modern, efficient commercial facility consists of the following improvements and alterations which will involve an expenditure of $1,200,000.00:

(1) Replacement of about 1000 untreated timber piles in the wharf which are undergoing a severe marine borer attack.

(2) The demolition of Transit Shed No. 2 and the construction of a new one-story transit shed of fire-resistant materials, approximately 500 feet long by 200 feet wide.

(3) Transit Shed No. 1 will be extended toward the caplog, a distance of about 60 feet, and increased in length about 180 feet toward Shed No. 2, in order to provide a better working apron and adequate covered transit storage area for the handling of two ships at one time. Offices will be constructed at each outboard corner of the building for steamship companies and Customs.

(4) The removal of all unnecessary trackage and the revision of the existing layout for efficient movement in classification of rail cars consistent with low maintenance. The existing holding yard capacity of approximately 650 cars will be cut down to about 80 cars. The shipside tracks along Transit Shed No. 1 will have to be moved closer to the caplog, in order that ship's gear can handle cargo directly from cars to the hold of the ship.

(5) Replacement of the entire underground water supply system, which has been found to be in very bad condition due to electrolytic and chemical action. This work is necessary to provide an adequate source of water supply for fire protection of the terminal, and for servicing the requirements of ships while at the docks.

(6) The existing temporary roads are in very poor condition and require reconstruction on a permanent basis. The entire existing layout of roads will be revised to permit a more desirable traffic pattern, security, and maximum use of the area comprising the terminal.

(7) A single-story storage building will be constructed in the rear of Transit Shed No. 1 for use as a supporting storage facility in connection with waterborne commerce. This shed will be of one story, approximately 120 feet wide by 300 feet long, with ramps for truck entrance into the sheds, and tracks in a depressed area in the rear of the building to permit floor-level loading of freight cars.

After completion of the first-stage development, consideration will have to be given to further development after complete study is made of the commerce requirements and the economic benefits to be derived from the proposed improvements.

The contract plans have been completed and the work is scheduled for commencement about March of 1953.

5. The Proposed Northern Avenue Oceanic Shipping Center is the fifth step in our Master Plan for the modernization of terminal facilities in Boston Harbor. Until World War II, the so-called New York, New Haven & Hartford Railroad Piers Nos. 1 to 4, inclusive, were used for intra-coastal trade. During the war and since, these facilities have been abandoned and allowed to deteriorate to such an extent that rehabilitation is impractical. This location is ideal for development as a combined passenger and cargo terminal, since it is situated on the main ship channel with a depth of 40 feet at mean low water; it is sheltered against rough water; the area encompassed would allow an extensive open storage and parking area for cars and trucks; close by is a large railroad classification yard, near the main arteries leading to and from the city and close to the business district of the city and to rail, air, and bus transportation. The development of the area would not disrupt any strongly-rooted businesses or require the taking of sound, usable structures. The existing structures are dilapidated and present a very unsightly appearance.

The project would consist of one combined passenger and general cargo terminal having a two-story building approximately 200 feet wide by 500 feet long; one general cargo terminal having a transit shed 200 feet wide by 500 feet long; another general cargo terminal having a transit shed 200 feet wide by 600 feet long; a three-story industrial center building approximately 200 feet wide by 600 feet long; a vehicular ramp to the second floor of the passenger terminal; roads, open storage, and a parking area.

The industrial center would be a three-story fireproof building having ramps, truck and rail loading platforms, elevators, and such other appurtenances and features as would be necessary for efficient and flexible use of the building for commercial, industrial, and warehousing operations relative to the maritime trade. An international trade center is planned to be located on the top floor.

The first part of this project which is scheduled for commencement about 1955 is the passenger and general cargo terminal. The passenger service would be exclusively on the upper deck, while the lower deck would be used primarily for the handling of general cargo. The second deck would also include the general offices of the Port of Boston Authority; offices for Customs and steamship lines; a restaurant; waiting rooms; and other conveniences and services pertinent to the passenger trade. There would be a freight elevator for transferring baggage and cargo between the first and second decks. Special attention would be given in the layout to the elimination of the confusion that exists in most passenger terminals because of the intermingling of passengers and visitors and the processing operations of disembarkation and embarkation. The motor vehicular and pedestrian ramp to the passenger terminal, which is to be on the second deck of the proposed new structure, would enable passenger and other operations of the second floor to be carried on independently without interference from the first floor general cargo activities. The large parking area that would be available would permit visitors and passengers to arrive in their private cars. This is an aspect that is sadly lacking at most piers throughout the country.

The economic study made, indicates that the revenue derived from the rental of the actual passenger terminal, the offices, and concessions, the parking, and the general cargo function would be more than adequate to amortize the cost of the project.

In order to permit expansion of the passenger service facilities when future conditions warrant, the foundations of the transit shed adjacent to the passenger terminal have been designed to carry an 80-foot wide passenger gallery with an observation mezzanine on the roof. The gallery would be connected to the main terminal by a 100-foot long second-floor bridge. The maximum length of the passenger terminal with gallery would be 1100 feet, permitting the accommodation of the largest passenger ships.

The plans have been completed for the sub-structure of the project. It is estimated that the total cost of this first step in the overall development will be about 15 million dollars.

The proposed ultimate development will involve an additional 12 million dollars.

6. Modernization of Pier No. 3 in East Boston--The passenger facilities on the second deck of Pier No. 3 of the so-called Grand Junction Docks in East Boston, now owned by the Commonwealth, should be modernized and placed in service again.

This area, since Colonial times, has been one of the principal locations in the Port for the handling of waterborne cargo and passengers. Near the turn of the nineteenth century, the existing Pier No. 3 was conalong the New Jersey Shore, the more significant environmental factors will be discussed without elaboration and the present status of shore preservation summarized with pertinent comment of general interest.

ENVIRONMENTAL FACTORS

LOCATION

The coastline of New Jersey boldly faces the Atlantic Ocean between New York Harbor and Delaware Bay at the northerly end of the eastern seaboard coastal plain. It extends 124 miles from the tip of Sandy Hook Peninsula to the rounded bluffs of Cape May. From about mid-point at Barnegat Inlet, the shoreline runs generally north upcoast and trends southwesterly downcoast.

The coastal frontage is varied in physical form. The mainland coastal plain directly meets the ocean in the northerly frontage creating about 18 miles of marine cliff headland. A narrow, 11 mile long barrier beach including Sandy Hook Peninsula extends northerly from this headland frontage. South of the headland to Cape May are 95 miles of barrier beach broken by inlets and backed by rearward bays, waterways, and salt marshes. The mainland touches the ocean in the form of marine cliffs for a short distance at the rounded tip of Cape May.

In addition to use as location references, the political divisions of the New Jersey Shore have further significance. By New Jersey Law, the state government arranges directly with each municipality for the establishment and execution of cooperative programs within the municipality's borders. While the state functions as a coordinating agency between neighboring municipalities, the New Jersey Shore is not a unit conservation district per se.

The New Jersey Shore lies within four of the 21 counties of the state. These four counties in order downcoast are Monmouth, Ocean, Atlantic and Cape May. The coastline is divided politically at the present time between 45 municipalities and two federal reservations making a total of 47 separate units. The northern mainland 18 mile frontage is in Monmouth County and usually its marine cliffs are termed the Monmouth County Headland. The 11 mile barrier beach to the north is also in Monmouth County. The 95 miles of barrier beaches south from the headland to Cape May are located within the other three Counties.

The 29 mile Monmouth County frontage is divided among 14 municipalities and one federal reservation, Fort Hancock. The Fort and two municipalities, Sea Bright and Monmouth Beach are on the 11 mile northerly barrier beach. The other 12 municipalities closely occupy the headland from Long Branch to Manasquan Beach. The Ocean County 42 miles of barrier beach includes 15 municipalities and Atlantic County's 20 mile barrier beach is divided among 6 municipalities. Nine Municipalities are on the 33 mile Cape May County barrier beach with one, Cape May Point, on the mainland frontage. Cape May Coast Guard base, a federal reservation, is on the barrier beach north of Cape May City.

HISTORIC SHORELINE CHANGES

The New Jersey coast is composed of sand, gravel and clay deposited in ancient times. Wells drilled as deep as 2,300 feet into the earth along the coast do not strike underlying rock. The erodible nature of the mainland soil has led to the commonly held opinion that the marine cliffs of the Monmouth County headland yielded the materials from which the barrier beaches were formed in pre-historic times.

Recession of the New Jersey coastline has been a continuing phenomen of historic times. It can be accepted that the marine cliffs of the Monmouth County headland have been subject to continuous wave-cutting and soil-loss until protected in recent times. It can be taken that the barrier beach shorelines have been formed and reformed by the restless ocean waves and currents and the vital energies of the inlets.

Evidence supporting these conclusions is delineated on special composite maps of shoreline changes along the New Jersey coast which were prepared by the Chart Division of the United States Coast and Geodetic Survey in connection with a report issued in 1922 by the New Jersey Board of Commerce and Navigation. These maps were published in that report and were based on surveys of the New Jersey coast line made during three periods, 1835 to 1842, 1865 to 1885, and 1899 to 1915, by the Survey and a State Board survey made in 1920.

Examination of the comparative shorelines plotted on these maps reveals that measurable shoreline changes did occur at all parts of the 124 mile coastline with recessions overshadowing accretions. The State Board stated in its 1922 Report that comparison of the first survey, 1835 to 1842, with the 1920 survey showed an accretion of 3,025 acres and erosion of 5,220 acres or a net loss of 2,195 acres in about 80 years. The State Board pointed out that this was equivalent to an average recession of two feet per year along the entire 124 mile frontage.

These general observations included mixed accretion and erosion at the inlets. The State Board pointed out that, on the unbroken coastline, recessions varied from 100 feet to 1,000 feet with median of 500 feet. The latter, on the basis of an 80 year period, indicates an average recession of 6 feet per year.

Further evidence of general recession along the entire coastline is that in the last thirty years protective structures have been built in 45 of the 47 political units along the coast embracing 111 miles or 90% of the 124 mile frontage. The exceptions are the 10 mile undeveloped frontage of Island Beach in Ocean County and the 3 miles of isolated island frontages in the vicinity of the Ocean-Atlantic county line.

There is no present evidence of shoreline changes due to subsidence of the coastal plain in New Jersey. The only change in vertical relationship between land and sea, is the slow progressive rise of sea level along the Atlantic Coast, since 1930, which is reported by the United States Coast and Geodetic Survey as approximately one-third of a foot to date.

THE INLETS

The historic surveys previously cited and more recent observations reveal the major influence of ocean inlets on adjacent shorelines. In the past, a number of inlets were closed to obliterate unwelcome features of the shoreline. The policy, today, is to maintain all existing inlets in the interests of navigation. This requires that the objectives of both navigation and shore protection must be integrated in planning and offers great opportunities to achieve maximum benefits in both fields. As will be noted later by direct reference, the improvement of inlets for navigation in some instances has created difficult shore preservation problems.

Formerly 19 rivers and streams cut through the Monmouth County headland and emptied into the ocean. Two rivers, the Shrewsbury and the Navesink, had inlets through the now unbroken northerly barrier beach. Today, all but two streams have either been diverted or confined to seaward drainage through conduits. Only Shark River and Manasquan River, important seaports, have jetty-controlled inlets. There is some thought that the closing of historic headland inlets diminished sand supply along parts of that frontage. More material is the fact that these land-locked streams are available as borrow areas of sand to augment natural supplies.

The barrier beaches of Ocean, Atlantic and Cape May Counties were cut through formerly by at least 17 inlets. By combination of natural and artificial means, seven of these have been closed. The ten remaining inlets, gateways to valuable seaports, represent particular opportunities to serve both navigation and shore preservation.

From time to time, it is suggested that the two great inlets at either end of the New Jersey Coast, New York Harbor and Delaware Bay, may greatly influence the entire intervening coast line. The idea usually springs from the observation that the dominant littoral drift in the north half is upcoast to New York Harbor and in the south half is downcoast to Delaware Bay. The nodal point has never been fixed being usually considered as just north of Barnegat Inlet. This suggested theory has not progressed beyond the conjecture stage.

OCEAN STORMS

Ocean storms have had and will continue to have a leading role in the field of coastal engineering. These awesome, always dramatic outbursts of natural forces underline the need for coastal engineering and provide full-scale tests of protective works. Every storm has its importance, but in particular, two widely separated storms of extreme intensity greatly influenced the practice of coastal engineering in New Jersey.

The three successive severe storms of Winter 1913-1914 provided critical tests of protective works existing at that time. The first struck at Christmas, the second at New Year, and the third in mid-February. Mountainous seas driven by gales exceeding 100 MPH lashed the entire coast line. The wide spread land loss, property damage, and destruction of protective structures during these storms made clear the need for community action in the common defense and led to greater municipal participation in coastal engineering. The interest of the State at large was also awakened. Engineering evaluation of the protective structure failures led to new concepts of design and planning which many consider the start of modern coastal engineering in New Jersey.

The hurricane of September 14, 1944, is of more recent memory. It roared the full length of the coast. Shorefront damage ran into millions of dollars. Ocean piers were destroyed. Boardwalks and adjoining buildings were torn apart. The barrier beaches were flooded. Homes were destroyed. Lives were lost. The Governor ordered the State Police to take custody of many stricken communities. The New Jersey Shore was termed a disaster area. This critical testing of the New Jersey Shore's ocean defenses, aroused concerted popular support. Sample activity was the formation of an Emergency Erosion Committee in 1944 by Atlantic City's civic and business organizations. Later, in 1948, the State created the State Beach Erosion Commission to advise the Governor and the Legislature on this major public problem. Such broader support permitted enlarged planning concepts which advanced the maturity of coastal engineering practice in New Jersey. This coming of age was featured by the initiation of continuing State-Municipal programs which are in progress today.

PRESENT STATUS OF SHORE PRESERVATION

Evaluation today of the New Jersey Shore's defenses against the ocean requires ratings from critical to reasonably secure. The degree of protection varies from locality to locality and remains to be improved as funds for further work are made available principally by the state and local governments.

State law in New Jersey requires each municipality to finance one-half the cost of local state-municipal programs so that the completion rate of such programs depends first, on the financial ability of the muncipality and secondly, on the ability of the State to match available municipal funds. This leads to extended construction periods of several years to complete planned work, and in some instances, to the inability of the state and municipality to execute any work due to the latter's financial inability.

Starting at Sandy Hook and proceeding downcoast, a review of the present status of shore preservation supplemented by pertinent references to future work and special problems will permit a comprehensive understanding of coastal engineering in New Jersey.

MONMOUTH COUNTY

The Monmouth County barrier beach north of the headland has attained a high degree of security against direct frontal assult by the construction of heavy seawalls and bulkheads marginal to the oceanfront. In Sea Bright alone, two miles of stone seawall were construoted in 1946 and another quarter mile was completed this year. Groin protection of the barrier beach is minimum and the need is urgent.

The Monmouth County headland frontage, occupied by 12 closely grouped municipalities, provides a variety of conditions. Extensive marine cliff frontage is secure behind seawalls and bulkheads, but there is equally long frontage exposed at this time due to deterioration of former structures and their destruction by storms. It is the universal policy among these highly developed resorts to hold the present margin of the land. Further anticipated construction includes bulkheading of the now open bluff frontages.

Groin construction along this frontage is well advanced. The groin patterns, while representing local conditions in each municipality, collectively present a coordinated grouping for the whole frontage. The task for the future is to fill in the missing units so as to complete the grouping. In the majority, the existing groins were constructed originally of a minimum length to provide emergency protection with the available construction funds. The extension seaward and maintenance of such structures are being carried out as rapidly as funds will permit. Lateral broadening of the groin field will permit establishment of wider beaches more generally than now exist. In this connection there is some question as to the adequacy of natural sand supply to create and continue beaches of desirable width. Natural sand movement has produced many admirable sections of beach along this frontage, but the question of artificial supply is being given serious study. The headland rivers are being weighed as prime sources of material by hydraulic pipe line transportation. Offshore dumping by hooper dredges from New York Harbor is also being considered.

The construction of navigation jetties at Shark River and Manasquan River inlets created special problems. Navigation jetties at Shark River Inlet were completed first in 1918 under a state project. Shore erosion developed both north and south of the Inlet. The situation was corrected in Belmar on the windward side by extension of the south inlet jetty in 1923. A fine broadbeach was created. The beach recession in Avon by-the-Sea, leeward of the Inlet, became very serious and required bulkheading of the shorefront and the construction of a series of groins. In 1948, the Inlet north jetty was redesigned and constructed to complement the groin field. Recently, the groins were extended to further the relationship and the results have been very favorable.

The completion of the Manasquan Inlet navigation jetties by the Federal Government in 1933 provided a very wide beach along the windward frontage at Point Pleasant Beach. This security permitted the development of this locality which previously had been retarded by extreme exposure to storm damage. On the leeward side of the Inlet, the beaches of Manasquan began to disappear. Remedial action was taken immediately by the Federal Government adjacent to the Inlet, but the municipal frontage generally receded until a timber groin field was constructed by Manasquan in 1939. The sea ends of a number of these groins were exposed and destroyed by currents of a deep, longshore slough which featured this frontage. This situation was corrected by construction of a heavy stone-portected groin across the slough so as to reduce its effectiveness and lead to its ultimate shoaling and closure. Future similar reconstruction of several groins to safeguard present gains is anticipated.

OCEAN COUNTY

The erosion tendency along the Ocean County barrier beaches has been restrained only at the older settled locations such as Bayhead and Beach Haven. It is anticipated that control will be extended to more localities and longer frontages as a result of the unprecedented boom in home building since World War II. Concentrated occupation and ownership diversity traditionally have created demand for shoreline stability and protective works. The Ocean County oceanfront includes, also, the untouched, natural 10 miles of barrier beach just north of Barnegat Inlet which is the site of a proposed state seashore park. The Legislature has appropriated funds for purchase and negotiations with the private owner are in progress.

There are several isolated oceanfront islands in the vicinity of the Ocean-Atlantic county line. This region is dominated by the large and powerful Little Egg Inlet with complementary action by the lesser Beach Haven Inlet to the north. The most exposed of these islands, Tucker Beach, formerly was sparsely occupied. This island and surrounding waters have been the scene of natural forced operating without restraint. There has been drastic erosion and reshaping in process for many years. These islands presently are not valued and corrective work is not required. The situation has not gone unremarked, however, since it affords opportunity to study a critical erosion location within a highly valuable coastal area.

ATLANTIC COUNTY

Directly south is Brigantine Island, the large oceanfront island that forms the northerly part of Atlantic County's barrier beach. The changing shoreline has been controlled by groins at only a few locations. A general program would be of great value, but lack of funds has prevented action beyond the planning stage.

Absecon Island comprises the southerly part of the Atlantic County shorefront between two major inlets, Absecon Inlet to the north and Great Egg Inlet to the south. The four municipalities which occupy the island represent the greatest concentration of values on the New Jersey Coast. Atlantic City occupies the north end of the island with Ventnor, Margate and Longport in order to the south.

Practically from the founding of Atlantic City in 1854, erosion has been a problem on Absecon Island. At this time, the entire shoreline is protected either by bulkheads and seawalls or by groin controlled beaches. These structures represent very large investments in protective works and provide a large measure of security.

The Atlantic City shores lie within the influence of Absecon Inlet, one of the largest on the New Jersey coast. The city has beaches along the Inlet channel as well as on the oceanfront. These beaches have been maintained for many years by judicious use of groins. Unfavorable relationships between the Inlet and the City's beaches resulted in shoreline recessions of alarming proportions about 1944. The hurricane of that year, cited previously, served to accentuate the situation.

A previously designed major program of protective work was set in motion. This program has been considered overly ambitious in some quarters but now received universal support indicating a maturity in planning for the future. The program was in two parts:-First, to provide direct local protection to the City's beaches including restoration of recreational areas; Second, to divert the Inlet channel away from the City's beaches.

A one thousand foot long groin was constructed to separate the north end of the oceanfront beach from the Inlet with a group of shorter groins on both sides in support. It was originally intended to construct a complete groin field along the oceanfront and inlet beaches before depositing artificial beachfill, but due to the necessity of establishing recreation areas, the order was reversed. Favorable changes in the Inlet's behavior gave support to this decision.

In a ten week period in the Spring of 1948, 1,500,000 yards of sand were placed on the City's beaches to provide berms 300 to 400 feet wide outside the Boardwalk. The sand was dredged inside the Inlet and distributed through pipelines. Erosion losses were considerable on the open beaches thus provided, so that a series of timber groins were constructed in 1950 along the oceanfront. Further groin construction was deferred by decision in 1951 to proceed immediately with the second part of the program.

The second phase consists of constructing a 4,200 foot long jetty on the northerly ocean bar of Absecon Inlet parallel to the Inlet channel and the City's beaches. The jetty is located 2,000 feet from these beaches and is intended to shelter them from open ocean exposure. Its further purpose is to act as an inlet training jetty and permit diversion of the Inlet channel away from the City's Inlet beaches. At present the Inlet channel, about 25 feet deep, undercuts these Inlet beaches requiring difficult and expensive protection. Construction of this jetty is in progress with 25% completed to date and the halfway mark projected for late in 1953.

Longport at the south end of Absecon Inlet has been waging a running conflict for years with Great Egg Inlet. It is estimated that the Inlet cut away 4,000 feet of barrier beach prior to 1920. Efforts by individual owners to half the encroachment proved ineffective and it remained for the municipality to make a stand in the common defense. The investment by the municipality about 1920 in a major groin and heavy seawall stopped the land loss but had serious effect on the town's financial history. Only recently, has it been possible for this small community to undertake further necessary work with state aid. As pointed out previously, state-municipal projects can be executed only when the municipality can finance one-half the cost. The value of the Inlet to navigation may lead to an inlet improvement project including features beneficial to the adjoining Longport coastline.

CAPE MAY COUNTY

The Cape May County barrier beaches are in the form of a series of long narrow islands separated by inlets of which only the most southerly one is improved. Localized protection has been installed in every municipality to prevent shoreline recessions and to cope with unfavorable inlet effects. Such work has been extensive at some locations and minimum at others. Present indications are that increased general use of the County's oceanfront will provide the opportunity to meet the evident demand for remedial work on a much larger scale. Inlet improvements for navigation may also be a source of aid in the future by inclusion of protective features beneficial to adjacent communities.

In recent times, the principal work has been at Ocean City and Cape May City. Some construction also has been performed at Sea Isle City and Stone Harbor. Serious situations at Strathmere, Avalon, and North Wildwood exist without correction due to financial difficulties. Wildwood and Wildwood Crest enjoy ample protective beaches and do not have a current problem.

Ocean City, the most northerly municipality of Cape May County, falls within the influence of Great Egg Inlet. The northerly City shorefront adjacent to the Inlet about 1932 began a period of recession in contrast to favorable accretion during the previous 30 years. As the situation became marked, a groin system was planned to control the northerly two miles of oceanfront. This program was started at the south end in 1939 and was completed in major part by spring of this year.

It has been planned to complete the groin field before pumping in artificial beach fill. The extended construction period, however, had permitted beach depletion, despite favorable action by the groin field, to such an extent that the proposed beach fill became mandatory and further groin construction was deferred in its favor. Between April 7, and August 4, of this year, 2,500,000 cubic yards of sand was dredged from the rearward bay, transported 4,000 feet across the island and distributed through pipeline along the southerly 1.5 miles of the groin field. Beach berms of 200 to 300 feet in width were created outshore of the Boardwalk. Completion of the groin field is being undertaken immediately in order to assure maintenance of the newly created beach.

Both Cape May City and Cape May Point are located leeward of the Cold Spring Inlet Jetties. These jetties were completed in 1909 and extended 4,500 feet seaward of the shoreline. In the early 1900's, Cape May City's beach was the Daytona Beach of that time. Early automobile makers, including Henry Ford, raced their new mechanical marvels on the City's wide open beach.

Sand diversion by the Inlet Jetties set in motion serious recession of the shorelines of both municipalities. Very heavy investiments in protective structures during the 1920's and 1930's provided temporary security. Since those times deterioration of structures and lack of funds for maintenance and new construction have diminished defensive values. In 1946 Cape May City initiated a groin construction program which it hopes to continue to completion and supplement with artificial beach fill. A second portion of the groin program was completed this year, and further work is planned for 1953. Financial difficulties have prevented any recent work at Cape May Point.

OTHER SHOREFRONTS

It should be stated parenthetically at this point that shore preservation is also extremely important at locations other than the coastline. Erosion problems exist along Raritan and Sandy Hook Bays, Delaware Bay and River, and inside the ocean inlets. These regions have equal status with the coastline in all particulars and are omitted only to avoid extended exposition.

THE STRUCTURES

In the foregoing status summary, reference has been made to the several classes of structures employed in coastal engineering. It is of interest to describe these structures in some detail and to point to changes and evolution in designs and use dictated by growing experience.

BULKHEADS AND SEAWALLS

In the New Jersey, bulkheads and seawalls have been used to barricade the upland face and mark the line of defense against the ocean. Since the cited 1913-14 winter storms, the emphasis in design has been on great strength and durability. Bulkheads of timber or steel predominate although in recent times steel has not been used where sand abrasion exists. At difficult locations, such as the Monmouth County headland, stone embankment is provided as frontal defense of such bulkheads.

Reinforced concrete seawalls were used in the 1920's along the Absecon Island frontage, but are rare elsewhere. Massive seawalls constructed of large rough stones have received wide preference. The usual design is frequently described as rubble-mound construction. Emphasis is placed on dense, compact construction by individual placement of stones having a density of 165 to 185 pounds per cubic foot. Stone sizes range from two to twelve tons. Top berm widths are usually 10 to 12 feet at elevations of 6 to 13 feet above high water.

GROINS

In earlier times groins were essentially timber baffles, either open or sand-tight, constructed normal to the shoreline. The cited winter storms of 1913-14 illustrated the need for permanent designs. This led to the first construction of stone groins which today are the most poplar. Between 1921 and 1928, stone groins were built at Sea Bright, Monmouth Beach, Long Branch, Allenhurst, and Asbury Park, all in northern Monmouth County, and at the south end of Ocean City in Cape May County. These first stone groins were of rubble-mound construction and were placed at an acute angle of 30 to 60 degrees from the littoral windward shoreline. This permitted the leeward groin face to act as a breakwater defense of the beach entrapped within the acute opening. The arrangement provided satisfactory results in some instances, but later general practice has been to construct groins normal to the beach to minimize the leeward-groin effect.

Until the early 1940's, the stone groins were essentially sand-tight timber or steel walls supported by stone embankment. In many cases, the quantity of stone used was limited for the purpose of economy so that the corewall was exposed to direct sea action. Deterioriation of the groin core-walls was followed by loss in effective action and by need for maintenance. In the usual design to-day, the core-wall feature has been replaced by a compact center mass of small stones contained within an enveloping large stone cover and side supports thus producing an all stone groin. Where timber groins are considered appropriate, stone embankment is used only to support and secure the seaward end. It has been found that timber groins in exposed locations are subject to extensive damage without such stone protection.

Stone groins are constructed with a flat berm top, 14 to 18 feet wide, with elevation 2 to 4 feet above high water. These dimensions permit use of the completed portion of the groin as a working platform and facilitate organization of equipment to individually handle and securely place the large, rough quarry stones which weight from two tons each to in excess of eight tons each and form the bulk of the groin. Location of operations directly at the working face assures the close attention required to obtain a dense, compact structure and provides the best opportunity to judge and meet the changing conditions and emergencies, which are recurrent and characterize this class of construction.

ARTIFICIAL BEACHFILL

Although placement of artificial beachfill was executed largely without groins at Atlantic City in 1948 and with a partial groin field at Ocean City this year, it is felt that groins and beachfill are complementary. The groin field provides the necessary pattern and physical change within the subject area to receive the artificial beachfill and to assure maximum benefits. The attitude of Ocean City at this time may be cited as in point. Dispite the fact that the beachfill was not completed until mid-summer, the City reports the largest beach populations in its history and a 25% better resort income than last year. The City attributes these gains directly to its investment in the newly created beaches and considers it good judgement to conserve its new asset by completion of the groin field. Atlantic City in similar vein constructed groins to protect its 1948 beachfill.

The expanding development of the New Jersey Shore has created a wider interest in the augmentation of natural sand supply along all parts of the coastline. As mentioned previously, the headland rivers are being considered as supplementary sources for the Monmouth County frontage. Along the barrier beaches fringing the inland waterways, the interest is two-fold. Navigation on the inland waterways is an important feature of the New Jersey Shore. Channel and harbor developments beneficial to navigation are being considered as sources of beachfill material. In general the justification for beachfill goes beyond the basic need for security and defense into the field of extremely valuable recreational and economic benefits.

SUMMARY REMARKS

In summation, the historic conflict between seashore development and ocean encroachment along the New Jersey coast is being resolved in favor of the New Jersey Shore by unceasing vigilance and the will to hold the sea frontiers.

While security is the keynote, it is recognized that enlarged economic and recreational benefits also have important inter-relationship with direct protective values. Rising curves of population growth, new highways unfolding shoreward, unprecedented investment in summer homes, all portend unabated shore development and expanding concepts for coastal engineering in New Jersey.

Once the problem of the individual property owner along, shore protection in general has become large scale public works executed cooperatively by the state and the municipalities in the common interests. The need for shore preservation receives popular understanding and support, not only among shore residents, but also within the growing ranks of summer visitors and home owners.

While the problem of shore preservation in New Jersey still remains large in scope, the record of accomplishments permits a strong sense of achievement and furnishes confidence for the road ahead.

CHAPTER 25

CASE HISTORY OF ST. JOHNS RIVER AND JACKSONVILLE HARBOR, FLORIDA

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INTRODUCTION

It seems particularly appropriate that this year should be chosen. for the presentation of a case history of St. Johns River, since it is this year that marks the centennial of the first effort to secure a navigable channel across its bar for ocean-bourne vessels. That effort led to a congressional appropriation of $10,000, a sizable sum in those days, for planning and definite work toward that end. Work was completed only last year, which provided a 34-foot channel from Jacksonville to the ocean, complete with various protective works, and a cut-off channel which eliminates three hazardous bends and reduces the distance to be navigated. Thus, the accomplishments of a century of efforts can be seen. It is those efforts which comprise the substance of this paper. A map of the river is shown as figure 1.

PURPOSE

The purpose of this presentation is to give an insight to conditions which made improvement of St. Johns River desirable, some smattering of early history, and a brief account of how the present improvement was accomplished. Since the beginning of work at St. Johns River bar was one of the earliest attempts in this country to conquer the mouth of a major tidal river, the history of that conquest forms an excellent index to the advance of knowledge in the field of coastal engineering.

HISTORICAL

An early book on Jacksonville begins by paraphrasing a statement attributed to Herodatus, in which he stated in summarizing the economy of an empire: "Egypt is a gift of the Nile." The paraphrase continues: "In a somewhat similar sense, Jacksonville is a gift of the commerce-bearing St. Johns." In a true sense it may be said that Jacksonville owes its existence, its early development, and much of its present commercial status to the river which passes through the city and provides its outlet to the Atlantic Ocean. The earliest Indian village to occupy the site of Jacksonville, and the small town which succeeded it--both chose that location because it was the narrowest point on the lower 80 miles of the river and thus was easier to cross. This geographical advantage placed the early settlement on the highroad of commerce where it has since remained. As the area developed, the village became the key port for the commerce of the region and grew in stride with that development. The river valley attracted the first

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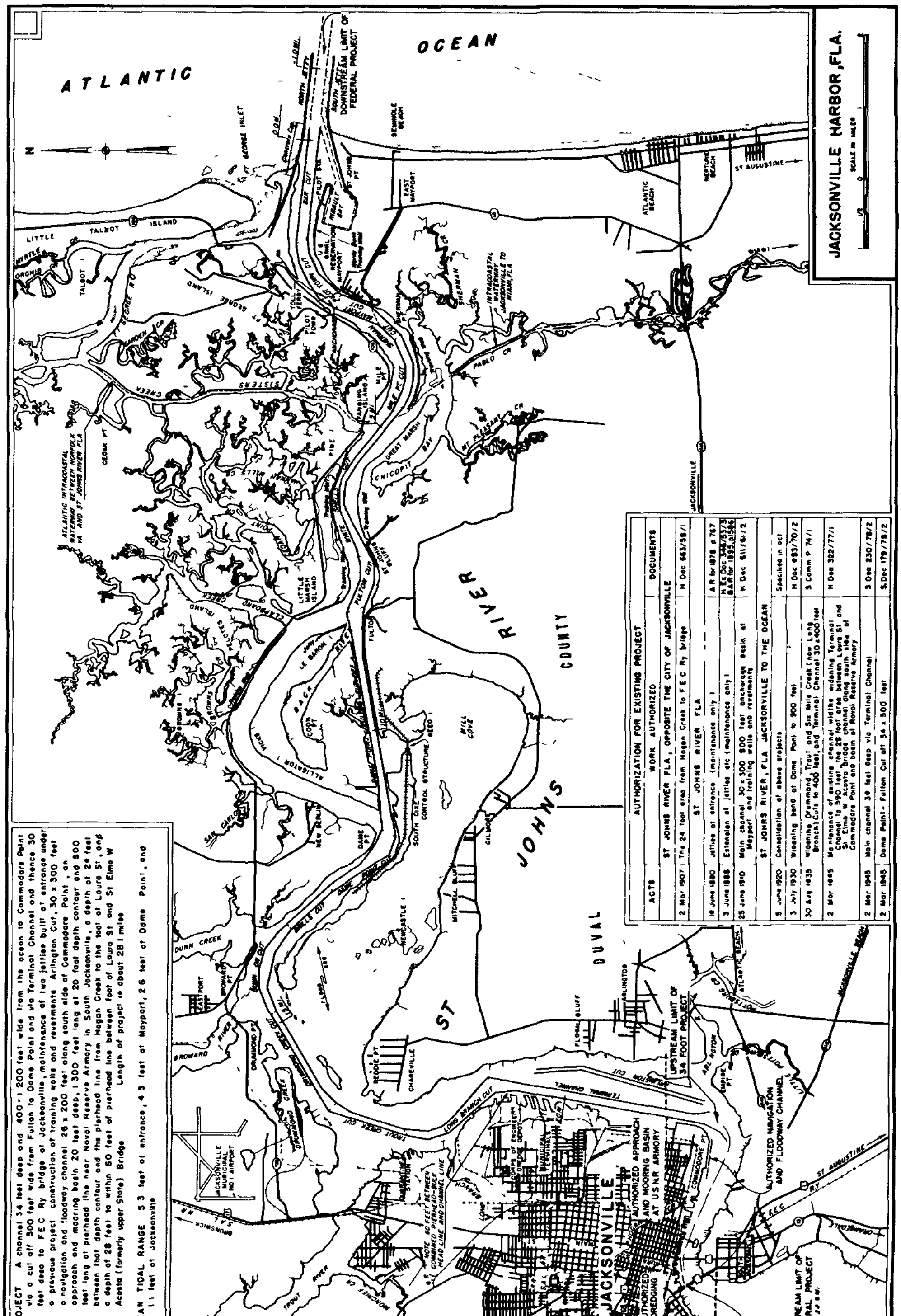

Fig. 1, St. Johns River, Jacksonville, Florida, to the ocean important Florida tourist trade, saw the beginning of the Florida citrus industry, and poured forth a flood of lumber used in the construction of our rising nation. The river transported those tourists, oranges, and timbers--and Jacksonville bloomed in proportion as that economic life-blood flowed through its vital artery, the St. Johns River.

EARLY HISTORY

Recorded history credits Jean Ribault, the French Hugenot soldier and seaman, with the discovery of the St. Johns River. He anchored at the mouth on May 1, 1562, naming the stream the River of May. Ribault's account of that day is quoted as follows:

> The night now approaching, we returned to our ships, for we durst not hazard our ship because of the bar of sand that was at the mouth of the river, notwithstanding, at full tide there were at least two fathoms and a half of water, and it was but a leap over a surge to pass this bar, not exceeding two cables (1,200') in length, and then afterwards there are six or seven fathoms of water everywhere; *** a ship of four to six hundred tons may enter therein at all tides, yea, of a far greater burden if there were pilots.

On his second voyage to the mouth of the St. Johns, in 1565, however, Ribault found water depths at the bar too shallow to allow entrance of his four largest vessels. He was forced to anchor them outside. This fact very nearly brought him to grief almost immediately, since a Spanish fleet of five ships under Menendez attacked the anchored ships. Their skeleton crews were barely able to escape with the vessels as the Spanish gave futile chase. The fugitive ships later found their way back to join Ribault. In the main, this successful escape only delayed the evil day for Ribault's venture, but the examples cited show the very early need for improvement of navigation facilities at St. Johns Bar, and the shifting nature of its channel.

NEED FOR IMPROVED CHANNEL

Long before improvement of the river was a seriously considered possibility, the seeds for a profitable river-borne commerce were planted near Tallahassee. There, in the 1820's and '30's, a large cotton area was brought into production. Much of the crops were moved by oxcarts to the St. Johns River upstream from Jacksonville, and loaded on sailing vessels bound for Savannah and Charleston for transshipment. The increase in cotton shipments from plantations west of the lower river and the increasing needs for imported consumer goods in Jacksonville and other St. Johns River ports led to the establishment of a weekly steamer schedule from Savannah to the St. Johns River in 1835. As freight and passenger demands increased, better-type vessels were gradually substitued for the earlier crude carriers. After the close of the Seminole War in the early 1840's, commercial traffic began to increase daily in volume and regularity. The variety of shipments grew to include lumber, cotton, oranges, and barreled fish. Those commodities were generally shipped to Savannah in the shallow-draft side-wheel steamers of that era. The railroads had not yet reached the region and the river was the only outlet for volume shipments, overland routes being poorly developed. Most of the shallow-draft steamers were comparatively small ships which chose the protected inland water route to Savannah, which is now a part of the Atlantic Intracoastal Waterway system. That waterway afforded depths up to 6 to 10 feet and threaded its way through the tidal estuaries along the coast, where the waters were protected from the ocean by a continuous string of islands. There was some risk connected with shipping across St. Johns bar to the ocean, since craft of shallow enough draft to cross the bar were too light to be entirely seaworthy, especially during the fall season of greatest incidence of tropical storms.

Those larger steamers or deeper-draft schooners taking the open ocean route were often required to stand off the bar for days or even weeks, awaiting the capriciousness of the shifting channel at the bar. When those ships did enter the river they had no further difficulty in the main river where depths exceeded those at the bar. After loading with timber or other commodities at Jacksonville or other river ports, the real difficulty was encountered in getting the loaded vessel over the bar with its deeper draft. The delay and economic loss incurred by such limitations made Jacksonville a rather ill-favored port city as compared to Savannah, and the even closer port of Fernandina, 30 miles northward. The excellent natural harbor at Fernandina easily afforded berth for hundreds of trading vessels which could and did make use of its excellent facilities. It was those limitations on shipping to Jacksonville and the great competitive advantage of its neighboring coastal ports which led Jacksonville citizens to first dream of a deep-water channel over the bar and its continuation up to the town.

First attempts to secure improvement - In 1852, the first step in securing deep water at the bar was made, when Dr. Abel Seymour Baldwin, local physician and civic leader, was sent to Washington to secure an appropriation for planning and definite work toward obtaining deep water at the bar. Dr. Baldwin's trip to Washington came as a result of his very early interest in the river. The birth of the idea, in a practical form, is directly attributed to his foresight. As a physician, he was often forced to call on his patients by boat over many miles up and down the river. During such trips to Fort George Island and the mouth of the river, Dr. Baldwin became convinced that a small appropriation from Congress to close Fort George Inlet would tend to channelize the waters of the St. Johns River, giving them freer discharge to the ocean, thus forcing a deeper channel for the passage of ships over the bar. While not an engineer, Dr. Baldwin had a scientific turn of mind, which caused him to note and interpret natural phenomena. He observed the currents in the river, and kept the meteorological record for Jacksonville for many years.

First appropriation - While in Washington, he was successful in securing a congressional appropriation of $10,000 for St. Johns bar. That appropriation provided funds for a survey and for project planning by the Army Engineers, which had the responsibility, then as now, for the development of river and harbor projects. In 1853, Lt. H. G. Wright was sent to Jacksonville to make the survey and experimental dredging. Wright reported that one pier or jetty on the north side of the main channel at the bar would largely overcome the difficulties. Lt. Wright's recommendation, however, was not acted upon and did not result in further appropriation for actual construction work. Withholding of the appropriation was said to have been caused when powerful influence was exerted in Washington on behalf of those interested in Fernandina harbor. Those interests apparently did not want their investments in Fernandina jeopardized by permitting water-borne traffic to be weaned away by the upstart port which was developing at Jacksonville despite the natural restrictions at the bar. Thus, work was delayed until the War between the States broke upon the country, presenting further delays. Progress was definitely arrested from 1853 to 1877, a period of 24 years, with no improvement for St. Johns River.

CONDITIONS AT THE BAR

Before resuming the history of improvement, it might be well to digress and see just what conditions were like at the bar. The earliest known map showing the bar is that of de Brahm, dated 1769, which is in the British Museum. It shows the south bank of the river extending into the ocean a considerable distance beyond the north bank. The mouth was divided into two distinct channels by a large marshy island designated on the map as "Middle Ground." That region was also shown on charts of the United States Coast and Geodetic Survey as early as 1856, and it was again surveyed by the Army Engineers in 1868. Before improvement there were two channels in the bar, which provided varying depths of from 6 to 8 feet at mean low water. The middle ground between the channels was a shifting sandy shoal called Pelican Island or Pelican Shoals. In 1878, this middle ground was found to be 2,000 feet south southeast of its position in 1868. It was then understood that the channel performed a cycle of operations during a period of about 25 years. The channel would work its way southeast, such movement being caused by the littoral drift (southward in this area), until it had progressed to a point sufficiently southward to extend the channel oceanward by deposition of sand carried by the stream. At such point the natural slope of the channel became flattened and inefficient, and a new, more efficient channel would break out over the shorter distance to tidewater on the north. Then, the cycle would begin to repeat itself southward. Sometimes there was only one channel, but usually there were two, one being deeper than the other, but subject to rapid changes due to storms or excessively high tides. The sailing directions varied from year to year, and often from day to day. No shipper or pilot would dare take his ship over the bar without sounding the channel by small boat.

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DEVELOPMENT IN THE TRIBUTARY AREA

By the 1870's the citrus industry was becoming one of Florida's largest commercial activities. The banks of the St. Johns River were especially favored sites for orange groves because the soil gave a fine flavor to the fruit, and the broad river ameliorated freezing winter temperatures to a marked degree. Then, too, the St. Johns provided virtually the only means of volume transportation of the fruit to the northern markets. Thus, the real beginning of the commercial citrus industry in Florida occurred along the banks of the St. Johns River. The larger groves had their own packing houses with long wharfs extending into the stream for direct loading to river steamers. Smaller groves were served by a "floating packing house." An old steamer was renovated for such use; it would pick up bulk fruit all along the river and pack it en route to Jacksonville where it was transshipped to the northern markets. By the early 1880's, orange raising on the St. Johns was a $10,000,000 investment with an annual yield of 75,000,000 oranges selling at $15 per 1,000, representing an annual gross of one and one-quarter million dollars.

In the importance of the freight steamers, passengers were not neglected. During winter months the tourist industry began to vie with citrus as the leading cash crop of the St. Johns valley. In the 1880's, Jacksonville had less than 15,000 population, but its 40 hotels and even more boarding houses played host to some 75,000 tourists annually. Those tourists were influential people, controlling much of the wealth of the nation. Many remained in the area, and others invested in the industry of the region. Such visitors contributed greatly to Florida's progress.

Long before the tourist era, New England shipbuilders discovered that the St. Johns River was a unique region, in that the two main timbers used in their craft grew together down to the banks of the stream. Live oak and long leaf yellow pine grew in such profusion that logging crews were sent to fell and collect the timbers during the late fall and winter months. In summer, the shipbuilders sent schooners which loaded up logs and loggers all along the river and returned them to New England.

An enormous lumber industry also began in the St. Johns valley in the early 1850's, and has continued to the present day. The early rapid growth of Jacksonville began as this industry rose and flourished. Palatka became a supply point for millions of board feet of cypress annually. Cross ties, shingles, and construction lumber of oak, pine, cypress, and other woods found their way out to the markets of the nation through the St. Johns River.

The above indications of the industrial and financial development of the area are mentioned merely to show that in spite of serious restrictions on navigation at the mouth of the river, a high economic structure was rising in the St. Johns valley, and particularly in

Jacksonville. That the restriction was severe is indicated by the faot that, according to the secretary of the Board of Trade in 1895, the average tonnage of vessels entering Jacksonville in 1870, before any improvement was made on the river, was 338 tons including steamers. Sailing vessels averaged below 200 tons, and it was a rare schooner that reaohed as high as 250 tons. By 1894, channel improvement had increased the average tonnage of visiting vessels to 1,060 tons, with sailing vessels up to 450 tons being quite common. Even with those limitations, however, Jacksonville was described in 1895 as "the most important orange market in the world and the greatest winter resort in America." When Henry M. Flagler and Henry B. Plant pushed the tourist and citrus industries farther south by the advent of their railroads, other industries rose to take their places, and the need continued for ever-increasing improvement of the river to facilitate the growing water transportation of the area.

EARLIER PROJECTS

About 1877, Dr. Baldwin, who had never lost his dream of river improvement, was able to put his plans into action again. Early in 1878 he went to New Orleans to confer with Capt. James B. Eads, who had been one of the engineering minds behind the design and construction of the Mississippi River jetties. Capt. Eads agreed to come as a consultant for local interests and make a survey and report on the St. Johns Bar for $1,000. Eads was not a member of the Corps of Engineers, but was a well-known engineer of that era. Dr. Baldwin returned and raised the money by popular subscription. Eads arrived in March 1878 and reported that there was no doubt of the success of a jetty system. He recommended two converging jetties from the mainland across the bar to deep water. He recommended a high-level jetty rising above the water surfaoe. He envisioned the resulting possibility of a 20-foot channel and estimated the cost at about $1,700,000. His report was approved by the local citizens' committee, which prepared a brief requesting a Federal appropriation of $1,700,000 for the work. The brief stated that from 1866 to 1878 the loss of vessels and cargoes from Cape Canaveral to Brunswick was $1,500,000, and that in 1872 alone such loss was $570,000, much of which might have been saved by a deep-water entrance at the mouth of the St. Johns.

The brief was effective, because in 1878 Capt. George Daubigny was sent to make an exhaustive survey of the river. Capt. Daubigny worked under the direction of Lt. Col. Q. A. Gillmore, in charge of the office of the Corps of Engineers in New York. On Capt. Daubigny's report, Col. Gillmore recommended a jetty system such as Capt. Eads had done, but he favored a low or submerged jetty. Gillmore's plan was submitted to Congress on June 30, 1879 and adopted.

ORIGINAL PROJECT

It is interesting to note the theory behind the Gillmore recommendation. The north jetty was to be 9,400 feet long and the south jetty

6,800 feet. A channel 15 feet deep at mean low water was envisioned. The outer 2,000 feet of each jetty was to be built up to half-tide level, but the inner portions were to be submerged to permit the waters of the ebb flow to escape at periods of high velocity on the theory that a reduction in such velocity was desirable to prevent undue channel erosion. It was assumed that the jetties would result in a channel with central depth of 15 feet, gradually sloping to each inner side of the jetties. The jetties were to consist of rip-rap stones resting on a mattress of logs and brush to prevent settlement. The mattress would be 18 to 38 inches thick. The jetty stones were to be placed with a slope of 1 foot vertical to 4 or 5 feet horizontal, making a rather wide structure at the base. Congress appropriated $125,000 in 1880 to begin work on the jetties. Meanwhile, a Government dredge boat, the "Henry Burden" was engaged in experimental dredging at the bar, together with other contract dredges, from 1870 to 1873, expending a congressional appropriation of $50,000. The dredging was not expected to produce permanent results, but it did show that maintenance of a channel deeper than 6 to 8 feet was an expensive job of hide-and-seek in the shifting channels, and that excavated areas quickly refilled. Thus, jetties became a necessity.

Early construction work - Under the jetty appropriation, contracts were awarded in 1880 to Lara, Ross and Company of Staunton, Virginia, for a solid raft of logs covered with brush compressed to a thickness of about 20 inches and topped with stone to make a total height of about 3 feet. This was to be the foundation for the south jetty, and 2,785 linear feet was laid in 1880. Mr. R. G. Ross, of the above-mentioned company, has related that logs used in that foundation work cost 2-1/2 to 5 cents each. The logs were cut on the banks of the river. Only the longest and straighest timbers were used, some without a limb up to 70 feet. By the middle of 1881 the entire foundation seaward of the low-water shore line had become covered by sand, which was considered a good thing, since it was thus protected from the toredo, or marine borer. The immediate result of the work was to arrest the southerly progress of the north channel and hold it more stationary. A secondary result was that the shoal to the north of the north channel (Ward's Bank Shoal) began to have its southern tip scoured off. Vessels continued to use the south channel, passing right over the jetty foundation.

In March 1881, Congress appropriated $100,000 more, and a contract was awarded to J. H. Durkee of Jacksonville for work similar to that already described. Work continued under both contracts as additional layers were placed on the south jetty, and foundations were placed for the north jetty. As the height of the south jetties rose, a gap 300 feet wide was left to permit passage of vessels at that point, since the scouring action had not been sufficient to open up the north channel as expected. Perversely, the water rushed along the south jetty and caused minor undermining until several spur jetties were constructed north of, and perpendicular to, the main south jetty. Serious erosion also occurred along the south line of the south jetty, resulting from water falling over the low jetty when the river was higher than the ocean level during ebb tide flow. It was found necessary to raise the jetty above mean low water to prevent this. As rock was piled higher, appreciable settlement began.

Difficulties encountered - In August 1882, Congress again appropriated $150,000. A new contractor was added to the force, but an epidemic and an unusually severe winter in 1882-83 resulted in little progress. Channel erosion at once became so severe along the south jetty that the safety of the entire project was jeopardized. At places the jetty sank 15 feet. In a single night one portion sank 6 feet. By the middle of 1883, there was 2 feet less water over the bar than during the preceding year. The District Engineer, in reporting to the Chief of Engineers, expressed great doubt that Congress would appropriate money fast enough to permit continuation of contracts to raise the jetties and save them from certain destruction.

Work was accelerated, however, and sunken places were filled with more layers of mattress and stone. At length, the structures were found to be too narrow to place additional material, and work was begun in widening the base and building up to the level already reached. Unequal settlement of the foundation, the uneven bottom, and the shifting sands caused engineers and contractors unending hardships.

It was later discovered that the log mattress was not flexible enough to conform to the bottom surface sufficiently to prevent undermining scour action. One of the contractors, devised a facine mattress of bound brush instead of logs, which was cheaper and gave superior results. It was found also that two dikes of stones could be laid along the edges of the foundation and filled with an oyster shell core, giving excellent results, and speeding construction. The shells were later capped with stone. Appropriations continued, but often at such wide intervals as to require large expenditure of the new appropriation in repairing damages to the work accomplished in the earlier contracts.

Much of the earlier rock dumped on the jetties was gneiss and granite from New York, which was hauled in by seagoing barges in tow. Often the barges would arrive in rough weather, and the greatest efforts were expended in dumping their cargoes in place to prevent excessive demurrage. The New York product was expensive as compared to Florida limerock. While not as dense as the granite, the limerock had a natural affinity for marine growth such as oysters and other shell fish, which quickly cemented the jetty into a solid mass. Later, the New York product was entirely abandoned in favor of Florida limerock.

About 1890, the work was showing favorable results in that the north channel was beginning to scour out. In the report of the Chief of Engineers for that year, the original estimate of $1,306,000 was increased to $1,471,000, with the recommendation that the north jetties be extended 2,260 feet to a point 11,250 feet from shore, and raised to full height for the shoreward 6,700 feet; and that the south jetty be extended nearly 2,000 feet, to a point 8,500 feet from shore with full height for the shoreward 4,400 feet.

Other appropriations were made, usually from $100,000 to $200,000 at a time; other contractors were brought in for new work. Concrete blocks weighing 6 to 10 tons were used in place of natural rock. In 1887, however, it was found that some of those 10-ton blocks were removed by gales on north jetty at a point about a mile out to sea. While that was unusual, it showed that no small stones could be used above mean low water, even as a support for larger stones. In a contract in 1891, it was specified that stone for topping must be at least 1 ton in weight and at least 50 percent over 4 tons each.

Beneficial results begin - By June 1892, definite results were noted from the work. About 4,500,000 cubic yards of soft material had been scoured out of the channel, which, if divided by the cost of the jetties, amounted to only 21 cents a cubic yard for removal--less than half the unit cost of the original dredging operations. The jetties were even then an unqualified success, and quite an engineering feat.

Original project completed - On July 13, 1892, and March 3, 1893, Congress appropriated $397,000 with which to complete the original project. The south jetty was completed to project dimensions during 1894-1895. During 1894 the bar channel deepened itself to 15 feet, where there had been only 10.7 feet the year before. At the completion of the project in 1895 there was 15 feet of navigable water over the bar, and 18 feet in the channel up to Jacksonville. This 18-foot channel throughout the river was due largely to a bond issue floated by Duval County. This bond issue made $300,000 available to the government as a local contribution. It was used in removing shoals and improving the channel near Dames Point, where existed the minimum channel dimensions between Jacksonville and the ocean. The money was expended under the direction of the Corps of Engineers in dredging in the river, and in the construction of training walls along the river to retain the flow down the permanent navigation channel, so that sufficient velocities would be maintained to keep the channel scoured out and free of sand. At the completion of the original work $1,785,000 had been spent, including the $300,000 contributed by the county.

Erosion along the river - As the jetties gradually became effective, changes took place farther up the river. Water began to flow past St. Johns Bluff at dangerously eroding velocities. The same thing happened at Dames Point. Banks began to cave in, and hundreds of thousands of yards of sand fell into the channel, which had to be dredged out later at great expense. It was not until retaining walls were built and exposed banks were rip-rapped with stone that the condition was corrected.

The jetties then stood completed to project dimensions, the foundations permanently settled and welded together by oysters and other sea growth. The south jetty was about 2-1/2 miles long, the north jetty was 3 miles long, and the two were about 1,600 feet apart at the outer ends. The final cap was made of huge pieces of South Carolina granite, which is about all that is visible today. It is impressive to note that only about 10 to 15 percent of the jetties can be seen, the remaining portions being below the low water level. From time to time the jetty topping has been replaced or added to as needed.

24 AND 30-FOOT PROJECTS

The original project was hardly completed when the need for deeper water became apparent. After several reports by the Corps of Engineers, and considerable backing by the Jacksonville Board of Trade, Congress made an additional appropriation of $350,000 in 1902 to start a 24-foot channel, 300 feet wide, from Jacksonville to the ocean. Two powerful dredges were built for the work, the "St. Johns" and the "Jacksonville." Within 4 years the 24-foot channel became a reality. By 1916, another project addition resulted in a 30-foot channel from Jacksonville to the ocean. The latter channel improvements were quite expensive, requiring the removal of tens of millions of cubic yards of material. The total cost of the project to June 30, 1946, was $19,540,000, including maintenance costs.

PRESENT PROJECT

Work was completed in 1951, providing a 34-foot channel from Jacksonville to the ocean. Minimum width is 400 feet, with greater widths up to 1,200 feet at critical banks. Existing protective works along the river include 7 miles of training walls, and revetments at five localities totaling some 3.5 miles of bank protection. Upstream from the 34-foot reach, project depths are 30 feet for about 2.5 miles past other piers and wharfs bordering the principal business area. The upstream limit of the project is 26.1 miles from the outer end of the jetties. above that project, but of lesser interest in this discussion, another project begins on St. Johns River and extends upstream 161 miles to Lake Harney. Dimensions of the latter waterway range from 13 feet deep by 200 feet wide at Jacksonville, to 5 feet deep by 100 feet wide in the upper reaches.

Costs of the Jacksonville to the ocean project from its inception to date have totaled $14,970,300 for new work and $11,222,600 for maintenance, a total of $26,192,900.

THE CUT-OFF CHANNEL

An interesting feature of the new work is the cut-off channel from Fulton to Dames Point, which removes three critical bends, and shortens the navigation distance by 1.9 miles. That work also entails training dikes and additional revetments to channelize the flow, prevent erosion, and to reduce maintenance costs. As a part of the cut-off channel, an extensive dike was provided parallel and south thereof to prevent shoaling in the channel. An access-way was provided in the dike for the passage of small craft, and to assist in reducing any tendenoy toward stagnation in Mill Cove, the large water expanse south of the cut-off.

Model study.--Design of the features of the 34-foot project, inoluding the cut-off channel and various training walls, was based on an extensive model study conducted by the Waterways Experiment Station in Vicksburg, Miss. A scale model was constructed of the lower 110 miles of the river. Scales employed were 1:1,000 horizontally and 1:100 vertically. Tides and flows were mechanically reproduced in the model and interpreted in an exhaustive study. Since completion of construction, velocities, tidal effects, and other hydraulic phenomena have been measured on the prototype and oompared to model results with surprising accuracy. It becomes quite obvious that if present model techniques had been available at the inception of work on St. Johns River, many expensive trial and error methods could have been eliminated.

JACKSONVILLE HARBOR

The port of Jacksonville has 81 wharves and piers, affording more than 8 miles of berthing space for vessels. It is served by four major railroads. Adequate warehouse and cold-storage facilities are available in connection with the port. Repair facilities are adequate for marine needs and include floating dry-dook installations.

The port is an important distributing center for petroleum and fertilizer products, oyster shell, paper, creosote, automobiles and machinery, coffee, sugar, and other items from both foreign and domestic sources. It also ranks high as a naval-stores port, and enjoys a large commerce in lumber, fruits, and vegetables. Port facilities are adequate for existing oommerce, with ample space for expansion to accommodate future needs. For Jacksonville Harbor in 1951, foreign waterbourne oommerce totaled 755,432 tons imported and 214,072 tons exported. Domestic coastwise receipts totaled 2,480,225 tons, with shipments totaling 59,562 tons. Internal receipts totaled 268,433 tons and shipments amounted to 489,464 tons. Local commerce accounted for 144,514 tons, or a grand total of 4,411,702 tons for the year.

CONCLUSION

Throughout this case history, the latest principles of coastal and hydraulic engineering have been adopted as they become known. From the earliest beginnings of dragging heavy chains across the bar for deepening the channel, to the present efficient excavation performed by modern pipe-line and hopper dredges, the best in scientific knowledge has been employed. From the earliest "cut and try" procedures to the latest techniques in model studies, the St. Johns River has received the best that our field of engineering has had to offer. The records show that for every increase in depth of the river there has been a corresponding increase in population and commercial growth in Jacksonville. In a hundred years, and at a cost of $26,000,000, the bar of a tidal river has been conquered, and a great deep-water port has developed at a distance of some 25 miles from the sea. Surely the expenditure of Federal funds for the betterment of St. Johns River and its ocean bar have resulted in great and wide-spread national benefits.

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Chapter 26

CASE HISTORY OF SHORE PROTECTION AT PRESQUE ISLE PENINSULA, PA.

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INTRODUCTION

The beach protection aspects rather than the importance to navigation of Presque Isle Peninsula is stressed in this paper. The history of the locality is so extensive that only that which bears on beach protection and the portion deemed pertinent to provide background to the early importance of the site will be related. The three main reasons for protecting Presque Isle Peninsula are as follows: for protection of the natural harbor at Erie, Pennsylvania; for the preservation of the beaches which provide Pennsylvania's only public lakeshore recreational area on Lake Erie; and to prevent destruction of the only land access to the facilities on the peninsula. It is reported that in 1947 over 1,500,000 people visited the 3,200 acre Presque Isle State Park. There are also located on the peninsula installations of the Erie Water Works, the United States Coast Guard and the Pennsylvania State Department of Fisheries.

GENERAL DESCRIPTION

Presque Isle Peninsula is located at Erie on the south shore of Lake Erie, 78 miles southwest of Buffalo, New York, and 102 miles east-northeast of Cleveland, Ohio. Lake Erie, one of the Great Lakes chain, is an elongated body of fresh water, 9,940 square miles in area, lying between Buffalo and Toledo, Ohio. Its maximum length and width are 241 and 57 miles, respectively, with its major axis lying in an east-northeast direction. The lake is comparatively shallow with a maximum depth of 210 feet and an average depth of about 58 feet. The 30-foot depth curve along the south shore of the lake is approximately one mile offshore. Except at the extreme ends, the lake is free of shoals and islands. Presque Isle Peninsula and Long Point, Canada, a similar peninsula to the north project from opposite shores and constrict the width of the lake at this point to 26 miles (Fig. 1).

Presque Isle Peninsula is a compound recurved sandspit joined to the mainland at the southerly end by a narrow section called "the neck". The peninsula extends in a northeasterly direction for a distance of 6-1/4 miles and broadens from a minimum width of approximately 250 feet at the neck to a maximum width of 1-1/4 miles near the distal end. The northern end of the peninsula terminates in several low, flat, cuspate bars. The north pier of Erie Harbor entrance channel is joined to the distal end. The south pier of the entrance channel is joined to the mainland by a tombolo at a point 5 miles eastward of the root of the peninsula.

The general elevation of the spit is low, rising to a height of about 5 feet above average lake level. There are four major and several minor beach ridges or dunes, extending across the spit in an east-and-west direction, which rise to an average elevation of about 20 feet above lake level. The wide part of the spit incorporates several shallow lagoons and marshes.

The lakeward shore is, in general, a regular flat sandy beach ranging in width from zero at sections where the neck is protected by sea walls to 250 feet at the distal end. Its regularity and continuity are broken only at points where protective works have altered the natural alignment of the shore line. The bay shore is characterized by narrow beaches, flat offshore slopes, and numerous small bays, coves and inlets. Except on the beach areas, the peninsula sustains a thick growth of almost every variety of trees and shrubs common to these latitudes.

The peninsula is exposed to storm winds from southwest through north to northeast. The minimum effective fetch of 26 miles is from the north, the maximum fetch, of 140 miles, is from the west-southwest. A study of wind records shows that the maximum velocity recorded for a 4 hour period was a Beaufort force of 10 (56 to 65 mph), and was observed 12 times during the 17 year period of record - twice from the southeast, once from the south, six times from the southwest, twice from the west and once from the northwest. Therefore, during the period of record 50% of the maximum velocity storms, each such storm occurring on an average of once in about three years, came from the direction of longest fetch. Neglecting the development of waves occurring prior to such a 4 hour portion of the storm, wind of this velocity and duration acting over the 140 mile fetch would develope a deepwater wave on the order of 16 feet in height. Further inspection of the records indicate that 27% of the total wind duration and 30% of the total wind movement is from the southwest. The shallow depth of the lake causes the larger waves to break appreciable distances from shore resulting in complex wave patterns. However, dead roll or swell sometimes runs as long as 24 hours after a severe storm of long duration.

Wave refraction studies of the area fail to show appreciable convergence or divergence in orthogonals prior to the wave breaking point which indicate a more or less constant beach slope. Waves from the northwest appear to have a smaller energy loss due to refraction than from other directions.

Wind data show that predominent direction of wind movement is from the southwest. Since the longest fetch is also to the southwest, it then follows that predominent direction of littoral current and drift is from the west to east in this portion of Lake Erie. Conclusions as to the predominent direction of drift are substantiated by the fact that the larger quantity of accretion occurs to the west of impounding structures. Harbor structures at Conneaut, Ohio prevent any significant amount of material of beach building size from passing that point, therefore, the source of beach material arriving at Presque Isle then comes principally from bluff and beach erosion between Conneaut and Presque Isle.

Levels of the Great Lakes fluctuate from year to year and from month to month during each year depending upon the volume of water in the lakes. There are two fluctuations of this type occurring simultaneously. One is a long term variation with a period in terms of years, which follows gener-

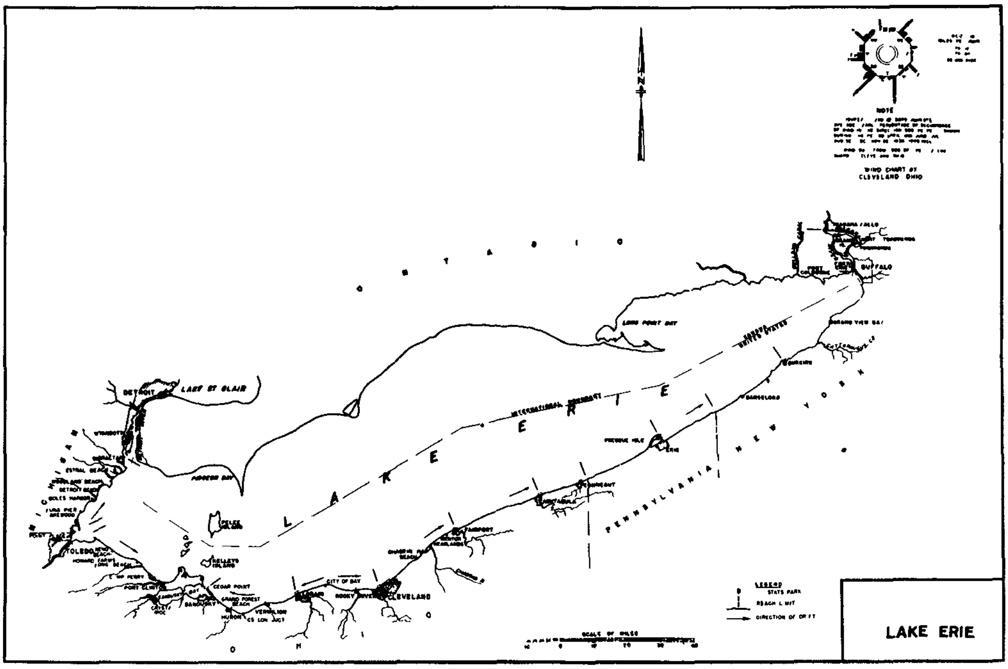
LAKE ERIE
CLEVELAND
BUFFALO
DETROIT
TOLEDO

Fig. 1.

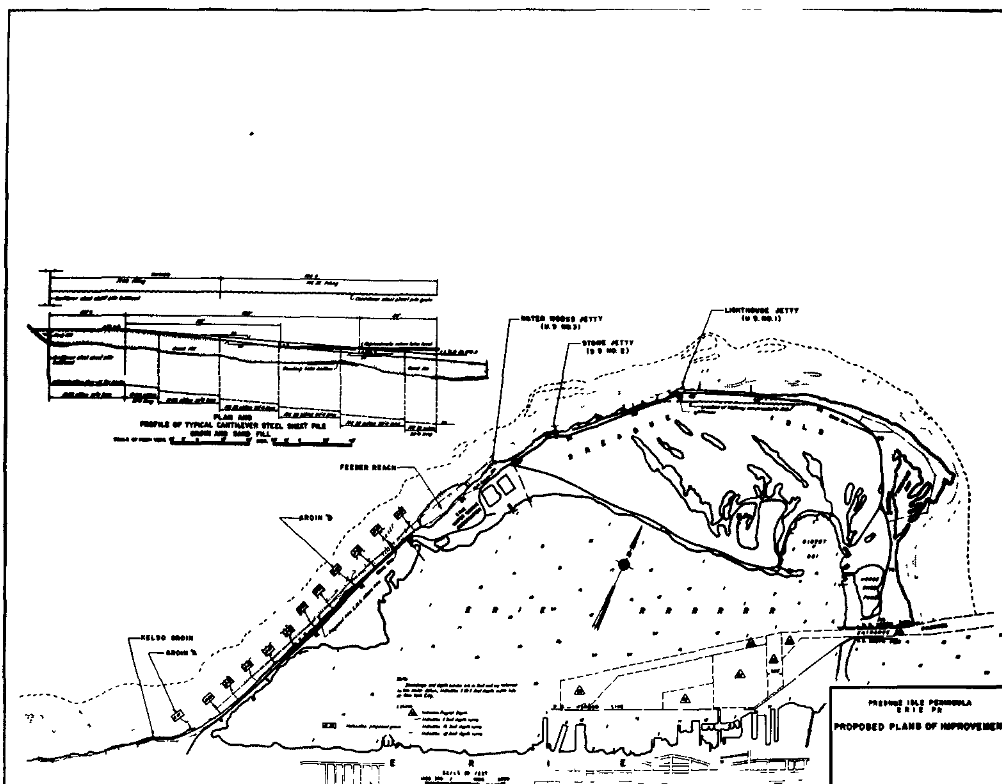
LIGHTHOUSE JETTY
FEEDER REACH
KELSO GROIN
PROPOSED PLANS OF IMPROVEMENT

Fig. 2.

ally the trend of precipitation; and the second is a seasonal fluctuation which follows generally the seasonal pattern of precipitation runoff. In addition, the stages at any given locality fluctuate due to unbalance or tilting of the lake surface, caused chiefly by wind and differential barometric pressures. These short period fluctuations vary in size from a few inches to many feet, according to location and cause, and are superimposed on the seasonal and long range fluctuations.

The effect of the level of Lake Erie on the degree of erosion has been demonstrated by the breaching of the neck. Each of the breaches, which occurred in 1828, 1833, 1874 and 1917, was preceded by several years of relatively high lake level. Pertinent data on levels of Lake Erie are shown in Table 1.

TABLE 1

WATER LEVEL FLUCTUATIONS

Item	High Stage, Feet(1)	Date	Low Stage, Feet(1)	Date	Variation, Feet	Elevation (2)
Extreme stage	+4.20	May 1952	-1.07	Feb.1936	5.27	-
Extreme stage, instantaneous 1900-1949	+5.25	Sep.1947	-2.85	Feb. 1936	8.10	-
Maximum seasonal variation in calendar year	+3.83	June 1947	+1.08	Mar.1947	2.75	-
Minimum seasonal variation in calendar year	+1.08	June 1895	+0.21	Nov.1895	0.87	-
Mean lake level	From January 1860 to December 1951, incl.					572.
Mean lake level	From January 1900 to December 1951, incl.					572.
Average lake level	April to Nov., incl., 1900-1949, incl.					572.

(1) Referred to low water datum, 570.5 feet above mean tide at New York City. All stages computed from monthly means, except instantaneous high and low.

(2) Referred to mean tide at New York City.

Ice action is another factor to be considered in the design of shore protective measures. Ice forms along the shore of Lake Erie in years of average winter temperatures, but in more severe winters the entire lake surface freezes over. Generally, the ice begins to form in shallow water along the shore in early December and eventually attains an average thickness of over 12 inches. Usually by early March the ice becomes honeycombe breaks up and forms into moving fields.

Ice may be beneficial or harmful in its effect on the shore or shore structures. The harmful effects are principally from the battering of structures by floating ice, particularly during heavy ice floes following the spring breakup, and from uplift pressures on structures. Beneficial effects generally occur from the following ice conditions which provide protection from wave action.

a. Usually in early winter ice forms on shore and shore structures from spray, at times the ice reaching a height of 12 to 15 feet above lake level.

b. Storms usually break up the ice fields in early winter and build windrows offshore. These, at times, attain a height of 10 to 15 feet as far out as one-half mile from shore.

c. In the middle of winter ice forms a solid covering for several miles offshore.

At Presque Isle it is considered that the benefits derived from ice more than balances the adverse effects. These beneficial effects were illustrated by a severe storm on March 25, 1947 when the east end of Lake Erie, including Presque Isle, was protected by ice and remained undamaged while the Cleveland area and the area further west, where the lake was free of ice, was seriously damaged.

SHORE PROTECTIVE MEASURES PRIOR TO 1939

In 1679, LaSalle, the French explorer, built and launched the first vessel to sail on Lake Erie at a point on the Niagara River about six miles above the Falls. As early as 1669 the Hudson Bay Company transported its goods and pelts in batteaux on these and western waters. In 1753 during the struggle with the English for possession of the Great Lakes Region, the French built a fort at Presqu'ile, meaning "nearly an island", and garrisoned it with 100 men. Then, as now, the Bay of Presqu'ile furnished one of the best natural harbors on Lake Erie. After the fall of Quebec in 1759 the French lost their grip in the Lakes Region, and Fort Presqu'ile was abandoned in 1760. The harbor at Presque Isle had been considered by both the French and English as an important point of communication and defense, and as a base for supplies between Pittsburgh, Niagara and Detroit.

Following the War of Independence the United States came into ownership of this portion of the country, and in 1792 Pennsylvania acquired the peninsula by purchase from the U. S. The title to the Peninsula was destined to change hands again, however, for in 1871 the Commonwealth of Pennsylvania conveyed the title to the U. S. for the purpose of national defense. In November 1922, the U. S. by Act of Congress, reconveyed the title to Pennsylvania for use as a park. The first permanent American settlement on the site of Erie was established in 1795 by the Population Land Company who laid out a town facing the harbor, and in conformity with an act of the General Assembly provided for a survey. The name Erie was given to the community (Fig. 2).

The peninsula was breached for the first time of record in the fall of 1828, however, the breach closed naturally in the winter of 1828 - 29. In 1833 a breach occurred in the neck of the peninsula immediately southwest of the area now occupied by the Water Works Reservation. After examination and study, it was suggested that a second entrance to the harbor from the west be provided through the breach. However, it was recommended that the effect of the breach be studied for a year or two before any plan was decided upon. By 1835 the breach had widened to the extent that where trees were thick in 1824, there was now an opening nearly a mile wide. The opening appeared to be increasing continually and to threaten the whole peninsula. A plan was submitted in 1835 to close the breach by stone fille cribs, leaving a channel 400 feet wide, thereby making entrances at both ends of the harbor.

In 1836 construction of the cribs began and by 1839 about 4,500 feet were completed. From 1840 to 1844 no appropriations were made by Congress and during this period a portion of the incompleted cribwork was destroyed leaving a breach of about 3,000 feet in width, the northeast end of the peninsula virtually remaining an island. By 1852 practically all the cribwork had been destroyed. In 1853 a project of revetting the shore with brush and stone was begun. By 1856 the prospect of restoring the original shore line was in sight. An evident economy move by Congress ensued and for a time no appropriations for the project were made. However, by 1864 the breach was entirely closed by natural shore processes.

In 1870 a Board of Engineers, Corps of Engineers convened at Erie to consider the condition of the peninsula. They were of the opinion that the harbor was in no immediate danger from the action of lake waves, but suggested that as a precaution against possible damage due to a succession of years of high water accompanied by severe storms that the narrower portions be reinforced. They advised the planting of silver poplar or beec trees where the vegetation was sparse. In 1871 and 1872, 350 loads of brus and 187 cords of stone were used in revetting the shore, and 50,000 young trees and slips were planted for protection of the peninsula by vegetal cov

In November 1874 a heavy gale again breached the neck. Repairs began in 1875, the plan being the construction of a bulkhead of timber piles and plank built to a height of about six feet, riprapped on both sides with stone. Following the breach it was recognized that the experiment of relying upon trees alone to protect the low portions of the peninsula had failed. Construction of the bulkhead was completed in 1877.

The first groins, eight of which were located along the neck and one on the outer section of the peninsula, were constructed in 1881. The groin consisting of closely spaced timber piles, were spaced 200 feet apart at right angles to the shore and extended to the six-foot depth. In 1883 and 1884 additional timber pile jetties and a timber pile bulkhead were built, but by 1887 they were all in ruins. The early destruction of all timber structures prompted the District Engineer to state that any timber structur placed to protect the neck must be replaced every six or seven years.

CASE HISTORY OF SHORE PROTECTION AT PRESQUE ISLE PENINSULA, PA.

In 1885 the Light House Jetty, the oldest structure now existing on the peninsula, was constructed by the United States near the center of the outer section of Presque Isle. It is a stone filled timber crib groin with a concrete cap on the outer end. In 1936 the inner end was replaced with steel sheet piles. At present the structure is in fair condition and an extensive beach has accreted to the west.

In 1889 a project to construct an offshore breakwater along the neck was begun. The proposed structure was to consist of 6,000 feet of timber pile and wood sheeting constructed alongshore to 3 feet above mean lake level, and 100 feet lakeward of the shore line. In 1890, after 4,500 feet of the breakwater had been completed, approximately 3,200 feet was destroyed during a storm. The contract was then terminated and the project abandoned.

During the periods of 1896-98 and 1916-17 approximately 16,500 willow, cottonwood, poplar, scotch pine and yellow locust trees; eight bushels of blue grass, shrubs, orchard grass and clover; and 2,725 feet of hedge were planted along the neck. Also during a part of this period, 1896 to 1904, two timber crib groins about 300 feet long were constructed by the United States. The first, located about 2,000 feet north of the Water Works, was completed in 1900 and the second, fronting the Water Works was completed in 1903. Except for the addition of concrete caps to each and concrete curtain wall adjacent to the northerly groin to decrease permeability, these groins have required only minor repairs and are now in fair to good condition. Both now have substantial beaches to the west.

Abetted by high water, a storm occurring on October 12 - 13, 1917 caused a breach in the neck of the peninsula less than 2,000 feet from the mainland. In April 1920, following the storms of December 1917, November 1918 and January 1919, the breach had increased to 1,470 feet. An unsuccessful attempt was made in 1918 to close the breach with a timber pile bulkhead. However, during 1920 - 21 closure was made by the United States with a rubble mound 1,700 feet long placed in a trench excavated to or near bedrock. In 1923 the rubble mound closure structure was extended 2,320 feet northward and about 5,000 linear feet of sand backfill was placed on the harbor side behind the rubble mound. To stabilize the fill rye was sown and poplar and willow trees planted. The wall remained in fairly good condition but was too low (1 or 2 feet above mean lake level) to provide effective protection during storms at average or high lake levels. The wall was raised and extended as a part of the emergency work performed by the Commonwealth during the fall of 1952.

In 1930 following several years of high water an area immediately south of Erie Water Works Reservation threatened to breach. To avert the breach the United States constructed 3,320 feet of steel sheet pile bulkhead with rubble mound facing. However, offshore erosion continued and about 2,600 feet of the southerly section partially failed in 1943 due to settlement of the stone. Realignment and repair of this 2,600 feet was completed in 1944. In 1947 the northerly 740 feet of the bulkhead was realigned and repaired.

In 1929 Lake Erie attained the highest monthly average level reached, up to that time, since 1876. This high lake level, with the attending increased erosion, was evidently responsible for the increased constructio of protective works by the Commonwealth of Pennsylvania between 1927 and 1939. For ease of explanation, this period of time is treated as a unit and the construction by the state is covered geographically, from south to north, in the following subparagraphs:

a. In 1927, near the center of the neck, four patented concret sand traps were constructed. These were groin-like structures about 50 feet long with a "T" on the lakeward end. The traps had little effect on the shore line and erosion continued. In 1930 the United States construct behind the traps, 1,960 feet of steel sheet pile bulkhead with stone facin A 400 foot section of this stone facing was repaired by the United States during the summer of 1951 and the bulkhead is now in good condition.

b. In 1931 the state constructed 1,232 feet of steel sheet pil bulkhead adjoining to the north bulkhead described immediately above and i 1939 extended it an additional 1,385 linear feet northward. A rubble moun facing was added to this latter section of bulkhead by the United States. This bulkhead is still in good condition. At the junction of these two sections of bulkhead a steel sheet pile groin 220 feet long, with the oute 100 feet curved to the north, was constructed in 1931. The outer 100 feet was destroyed by wave action shortly after construction (about 1939) but the inner 120 feet is in fair condition and maintains a beach from 100 fee to 50 feet wide to the west.

c. In 1927 - 1932 a precast concrete sandtrap, as described in subparagraph a, and 9 steel sheet pile groins were constructed on the 6,00 feet of shore immediately north and east of the waterworks reservation. The outer ends of several of the sheet pile groins were curved northward. It is of note that none of the curved groins have retained a beach lakewar of the groin bend. Practically all of these groins have failed at one tim or another and have required repair. The failures have been attributed to insufficient penetration of the piles, as failures have generally occurred near the center of the groins where shorter piles were used. Practically all of these groins constructed normal to the shore line have retained capacity beaches. The sand trap is still in fair condition and completely filled. However, its length and height are so small that little change in the shore line results therefrom.

d. Along the northeasterly 3,500 feet of the area covered in the preceding subparagraph and just southwest of the Light House Jetty, a steel sheet pile bulkhead was constructed in 1929 behind the groins. This bulkhead is now buried in sand accretion. In 1932 the state constructed 1,500 feet of similar bulkhead northeast of the Light House Jetty. In 193 an 850 foot extension was added. Erosion of the offshore and backfill continued and in 1946 the bulkhead failed and a large section of the highw was destroyed. In 1947 - 48 the state repaired and realigned the bulkhead added a 250 foot extension northward, relocated the highway, and construct 2 groins in this section of the bulkhead. The following winter the groins were destroyed. Since that time the bulkhead has failed in numerous places and has been outflanked. The present shore line of the peninsula is landward of the bulkhead.

INTERIM BEACH EROSION CONTROL COOPERATIVE STUDY

In April 1939 the Commonwealth of Pennsylvania applied to the Corps of Engineers for a cooperative beach erosion control study, under authority of Section 2 of the River and Harbor Act approved July 3, 1930, for the purpose of developing plans to prevent further erosion and to stabilize the beaches. A physical survey of the peninsula was made in 1939 and existing data on the area were compiled for analysis. However, due to the limited amount of basic data available and to the lack of knowledge regarding the characteristic action of beach building phenomena along the peninsula the Beach Erosion Board in an interim report dated April 1942 recommended the following action:

a. Extension of the bulkhead along the neck of the peninsula;

b. The construction of two experimental rubble mound groins extending lakeward 300 feet from the bulkhead line;

c. That upon the completion of the bulkhead and groins, four semi-annual surveys be made to determine the effectiveness of groins of this design;

d. That a topographic and hydrographic survey of the peninsula be made prior to construction to serve as a base for comparison; and

e. That formulation of a complete plan of improvement be deferred until the results of the surveys described above are available.

During July and August 1943, the Office of the District Engineer, Buffalo undertook the following:

a. Beach profiles for the entire peninsula. The profiles extended from the baseline of the 1939 survey to the 18-foot depth contour;

b. Vertical aerial photographs of the peninsula;

c. Ground photographs of conditions along the lake shore of the peninsula.

Comparison of the 1939 and 1943 profiles indicated that more or less continuous erosion had occurred in the Western section (between the mainland and the Water Works Jetty). The volumetric change was a loss of about 202,000 cubic yards, of which about 80% occurred above the low water datum contour. The eastern section (from Water Works Jetty around the distal end to the Harbor Jetty) comprised a generally prograding zone. The total accretion in this area amounted to about 1,050,000 cubic yards, of which about 90% occurred off shore.

The above information was compiled by the District Engineer in an interim report of September 1943. The report also outlined emergency protective works and test structures to be constructed at this time. The works consisted of:

a. A rubble mound wall 2,750 feet long extending northeasterly from the Kelso groin at the root of the peninsula. Near the center of the wall groin "A" was built perpendicular to the wall and extended 300 feet lakeward.

b. Realignment and repair of the steel sheet pile bulkhead constructed by the state in 1939, addition of rubblemound facing lakeward thereof, placement of sand fill behind the bulkhead, and construction of a groin "B" similar to groin "A" at the downdrift end of the bulkhead.

c. Construction of a rubble mound facing on a stone blanket lakeward of the steel sheet pile bulkhead constructed by the United States in 1930 southward of the water works. Also realignment and repair of 800 feet of the bulkhead and placement of sand fill behind the bulkhead.

The first post-construction survey was made by the Buffalo District Engineer in April 1944. At the time of this survey the construction of bulkheads was not yet complete although the groins had been completed the previous fall. The results of the survey indicated accretion ranging from 0.0 at about 1,000 feet southwest of the experimental groins "A" and "B" to about a 1 foot thickness at the groins. The accretion at the groin extended about 500 to 600 feet off shore. There was also evidence of a leveling of off-shore bars and valleys in the area. The remainder of the peninsula continued to erode slightly except at the distal end where accretion was evident. There was but one major storm between the surveys of 1943 and 1944. This occurred in December 1943 and breached the bulkhead at the northerly end of the neck. This report recommended that the remainder of the post-construction surveys be made annually, in late summer, rather than semi-annually.

The second annual post-construction survey was made in August-September 1945. Between the first and second surveys the Commonwealth of Pennsylvania constructed two permeable groins near the north-easterly end of the peninsula. The location of the state groins were such that impoundment at the experimental groins were not affected by their presence.

Conditions during the three years prior to this survey were somewhat more severe than typical years as lake levels were high. The high levels allow damaging waves to progress further inland with consequent increased erosion of the shore. The second survey indicated that (see Fig. 2):

a. The permeable groins installed by the state had settled 2.5 to 3.0 feet. Inspection indicated that some accretion occurred with easterly winds but erosion occurred with more frequent westerly winds;

b. General erosion had occurred in the unprotected areas except at the distal end of the peninsula;

c. No appreciable changes had occurred in the sections where protective works (groins and bulkheads) were constructed, except at groin "B" where a three foot deep accumulation of sand occurred adjacent to the updrift side. It was concluded that groin "A" was providing protection although the amount of onshore accretion was not as large as at groin "B".

The third post-construction survey was made in July - August 1946. The report on this survey recommended that the fourth post-construction survey include a topographic and hydrographic survey of the entire peninsula. The third post-construction survey revealed that over the period studied the protective works were at least partially successful. Groin "A" collected a beach to the west although erosion occurred to the eastward for a distance of about 1,000 feet. The groin was in good condition and the rubble mound bulkhead backing the area showed no sign of damage or deterioration. Groin "B" also accreted a substantial amount of material since construction, amounting to a layer four to six feet deep immediately west of the groin. Erosion was evident east of the groin. The bulkhead backing this area remained in good condition. Bulkheads in areas not protected by groins for the most part were in need of repair. Unprotected areas had eroded generally and in the area of the waterworks a length of paved highway was destroyed.

COOPERATIVE COMPREHENSIVE BEACH EROSION CONTROL REPORT

A Cooperative Beach Erosion Control Report was prepared by the United States in cooperation with the Commonwealth of Pennsylvania, acting through the Department of Forests and Waters and the State Park and Harbor Commission of Erie, Pennsylvania. The elements considered in the design analysis and the plan of improvement recommended in the report are discussed in the following paragraphs.

In considering the history briefly covered in this paper, it may be noted that the natural tendency of the peninsula is to migrate eastward. This is shown by the constant erosion along the neck and accretion at the distal end; by the persistent breaching of the neck; and, prior to the installation of structures which altered the natural shore processes, by natural closing of the breaches with littoral drift. The essence of the problem is the stabilization of the peninsula under existing conditions. This may be accomplished by protecting the neck and assuring that a sufficient quantity of littoral drift is available in the accreting areas.

A plan consisting of revetment and bulkhead alone was considered for protection of the peninsula. However, this method was eliminated in the final selection of plans because of advantages offered by a spending beach. Spending beaches furnish protection and also provide recreational area, a principal function of the peninsula. A comprehensive plan using this method was recommended in the report. After reviewing the recommendations of the District and Division Engineers, the Beach Erosion Board recommended the comprehensive plan described in the following paragraphs (see Fig. 2).

The neck, which consists of the shoreline from the root to the Erie Waterworks Reservation, has been subject to large losses of material during closely spaced series of severe storms such as preceded the 1917 breach, and by single severe storms. For this reason the plan for the neck included groins, artificial fill, and a bulkhead as a last line of defense. The groins are to be spaced 1,000 to 1,300 feet apart, are 290 to 320 feet long, and will incorporate experimental groins "A" and "B" without change, and state groin 8 modified to the recommended design. The height of the inner end of the groins, and consequently the design beach berm, is 8 feet above low water datum, or 6 feet above mean lake level, and the groins extend lakeward horizontally for 60 feet then on a 1 on 20 slope to mean lake level. The complete plan consists of construction of two cellular steel sheet pile groins and 7 cantilever steel sheet pile groins, alteration of one steel sheet pile groin, raising of 1,700 feet of stone seawall, construction of 1,120 feet of cantilever steel sheet pile bulkhead, and placement of 1,079,000 cubic yards of sand.

The plan for the remainder is composed of a stockpile of 1,000,000 cubic yards of sand at the southerly extreme of the Water Works Reservation, an artificial beach extending from that point to the about center of the reservation and the removal of the outflanked steel sheet pile bulkhead near the center of the outer section of the peninsula (constructed in 1932). The area north of the Water Works Reservation has been fronted by wide beaches in the past and is assumed to be a natural accreting zone when abundant littoral drift is available. Therefore, it is assumed that protection will be formed by the natural drift in conjunction with erosion of the stockpile and the updrift beach fill. It is believed that this general plan is the most practicable and least expensive method to provide the protection needed.

The Cooperative Beach Erosion Control Report recommended that a project be adopted by the United States authorizing Federal participation by the contribution of Federal funds in amount of one-third of the construction cost of the comprehensive plan described in the two preceding paragraphs. This is the maximum Federal participation permitted by the policy established by Public Law 727, 79th Congress, which permits Federal participation in beach protection projects. The total cost of the proposed plan is $5,259,000 of which the United States share would be $1,753,000. Maintenance of the project will be the responsibility of local interests.

During the high water of 1952 the condition of the southernmost 2,000 feet of the neck again became critical and the Commonwealth of Pennsylvania appropriated approximately $600,000 for emergency work. The Commonwealth desired to construct emergency work that would also become an integral part of the comprehensive plan. During the late summer of 1952 2,800 feet of rubblemound bulkhead was raised to about 6 feet above average lake level. It is understood that the Commonwealth plans to also construct six groins in this area during summer of 1953.

CASE HISTORY OF SHORE PROTECTION AT PRESQUE ISLE PENINSULA, PA.

SUMMARY

In the 117 years in which protective works have been applied to Presque Isle Peninsula there have been many types of structures and many methods of protection attempted. The large forces involved proved too great for vegetal growth alone to be effective in the porous material which forms the peninsula. Timber structures other than the rock-filled crib groins have had short life. Bulkheads exposed to the direct action of the waves have required frequent and expensive repairs. As demonstrated by the impoundment at the privately owned Kelso groin at the junction of the peninsula and the mainland, and at experimental groins "A" and "B", a spending beach gives best results. The problem is then the method to be used in forming the beach.

Erosion appears greatest at the root of the peninsula, decreasing in extent to about the Water Works Jetty, beyond this point accretion is predominent when sufficient material is available. Persistent breaching indicates that the peninsula is subject to rapid losses with high lake levels and severe storms. Based on this analysis of characteristic action, sand fill alone would not suffice at the neck and retaining structures are indicated. From the mainland northward along the neck and fronting the Water Works Reservation an artificial beach with a stockpile at the Water Works Reservation to provide littoral drift and groins to hold the beach along the neck appears logical. The shore easterly of the Water Works appears to be a natural accreting zone, therefore with the natural amount of littoral drift restored and the additional material available from the stockpile it is believed that this zone will form the needed protective beach by natural processes. The comprehensive plan should be followed for best results as experience has shown that piecemeal construction has not been effective.

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