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EFFECTIVE HEIGHT OF SEAWALLS

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I. Introduction

Both seawalls and bulkheads are structures placed parallel or nearly parallel to a shore line, generally separating a land area from a water area. The term seawall is used in this report since criteria for structural design of the two are essentially the same, although the structures differ functionally. Functionally a bulkhead is a structure whose primary purpose is to support the land behind it while separating this land from the water. A seawall's primary purpose is to protect the land area from erosion or damage due to wave action.

Though seawalls are one of the most frequently used, as well as one of the oldest means of coastal protection, their design has been dictated mainly by structural considerations. No general criteria have been established for determining their potential effectiveness in protecting the land behind them.

This report will deal with the determination of the efficiency of vertical face and curved re-entrant face seawalls in turning back damaging wave action. The results should not be applied to stepped face or sloping face walls. The controlling factor in this problem is the type of wave attack expected at the structure, which may be determined by observational data or if these are not available, by making use of hindcasting techniques; establishing, from historical weather charts, a deep water design wave; and with that, a design wave at the structure. Because the placement of a seawall is ordinarily determined by terrain or economic considerations, water depths and beach slopes at and before the wall's position will almost always be known. With these, wave characteristics at the structure may be determined through construction of refraction diagrams and the application of pertinent sections of this report.

The remainder of this report is divided into seven sections:

Section II deals with the types of water level fluctuation which may be expected at a seawall's location.

Section III establishes a criterion for a seawall to be totally effective in turning back damaging waves.

In Sections IV and V, the two problems of seawalls located in and seaward of the breaker zone are discussed. Each of these sections is divided into three parts; a general discussion of the means of determining wave characteristics at the wall if only deep water wave characteristics are known (though the construction and interpretation of refraction diagrams is not discussed): a determination of the height of wall which will be totally effective in terms of the height of an impinging wave; and a determination of the relative effectiveness of lower walls.

Section VI is a discussion of the problem of a seawall shoreward of the breaker zone.

Section VII is a summary of Sections IV, V, and VI.

Section VIII applies some of these results to an actual case.

II. Factors Involved in Water Depth Variations at a Seawall

As noted previously, if observational data are not sufficient, wave characteristics at a seawall's locations will have to be determined from deep water design waves through use of refraction diagrams. Construction of these, their interpretation, and ultimately, the type of wave attack at the wall's position, are all dependent on a knowledge of water depths at and before the wall. Therefore in this section we will discuss the major phenomena which may lead to a change of water depth; a seiche, a gravitational tide, a storm tide, and bottom scour.

A seiche consists of a periodic oscillation of water over any water area, determined by the inherent natural period of oscillation of the body of water. Seiches have been known to attain the heights of 6 feet in Lake Geneva,(1) and 15 feet in Lake Erie.(2) The causal phenomena may be wind, pressure difference over the water surface or even a gravitational tide.

Gravitational tides, caused by the attraction of sun and moon and by the earth's rotation, are the most familiar changes in water level, since all coasts and some lake shores experience them daily. The range of tide depends on local hydrographic features, and may vary from about 2 feet at points in the Gulf of Mexico to some 35 feet at Dutch Harbor, Alaska.

In many locations, the most important water level fluctuation to be dealt with is the so-called storm tide. When a severe storm strikes a coastal area, high winds accompanying the storm cause "pile-up" of water along the shore. Due to this, in narrowing inlets and bays, the water level may rise 15 to 30 feet. Though the range of the daily gravitational tide may be as large as that of a storm tide, the latter is unpredictable, and if in phase with the former, may well cause a water depth increase very much larger than that due to the local gravitational tide. For example, Galveston, Texas, where the normal height of high tide is less than 2 feet (MLW) experienced hurricanes which in 1900 caused a high of 15 feet (MLW) and in 1915 a high of 12.5 feet (MLW) (3). A hurricane in 1938 caused high water elevations as great as 14.7 feet (MLW) at certain points along the Massachusetts coast(3a). A storm on 5 October 1864 raised the water level at Calcutta 24 feet(1).

It should be noted that the danger of a storm tide lies not only in its range, which may or may not be excessive, but also in that, by its nature being caused by strong winds, the rise in water level is always accompanied by severe wave action.

A seawall is often located in an area of erosion for the purpose of preventing further loss of land landward of the wall's position. However the area's tendency toward erosion may continue to manifest itself by scouring the beach before the wall. Therefore, though water level fluctuation at a particular locale may be minor, depths before a wall may still increase.

III. Criterion for Total Effectiveness of Seawalls

Rough measures are available for the determination of the effectiveness under wave attack of seawalls whose crests are even with or below the maximum water level expected at the wall's location. It is necessary however, to establish a criterion for a seawall to be totally effective when undergoing wave attack. The standard which will be followed is this:

A seawall can be considered totally effective if its height is sufficient to prevent any solid water from passing over the wall with damaging horizontal momentum. This criterion will be considered satisfied if the height of the wall is equal to or greater than the height of an impinging wave.

It is ordinarily economically infeasible to design a wall high enough to prevent any overtopping under all wave conditions. However the primary purpose of a seawall is to prevent damage to the land or structures behind, and this damage will be caused by that water which overtops the wall with an appreciable horizontal momentum.

A seawall whose crest height is equal to or greater than that of the crest of an impinging wave will cause the wave to run up and overtop the wall. The amount of this overtopping is dependent on the shape of the wall, and on the characteristics of the waves at the wall. (e.g. the "clapotis" formed by a non-breaking wave at a vertical barrier). However the momentum of this water thus thrown above the wall will have been changed from a nearly horizontal one to one (depending on the wall shape) nearly vertical, and since the horizontal momentum is reduced considerably, the damaging power of the wave is similarly reduced.

It is true that the damaging effect of water falling in the immediate vicinity of the wall must be considered in the structural design of the wall itself, and of the embankment behind, which must be provided with pavement and drainage. Damage to the wall will reduce its effective protection of the land behind, but the prevention of this damage is a problem of structural design. It is not a consideration in determining the effective height of the wall.

IV. Maximum Conditions (Seawall in the Breaker Zone)

General - As the preceding discussion indicated, water depth at a structure may be so highly variable, especially under storm conditions, that it would be impossible to locate a structure outside the range of damaging wave action. It would be well therefore to discuss the effectiveness of a seawall under extreme wave conditions, that is when the wall is so placed that the impinging wave will be of maximum size. J. Larras has found that "When for a given swell, one set up the vertical wall at various points of the terminal slope, the position of the wall for which the breakers become most violent coincides with the position of the rollers on the same slope in the absence of the vertical wall. In other words, the waves break against walls in the same depths as they do upon slopes, ..." (4) (5).

If we can determine the characteristics of a breaking wave and the depth in which a wave may break in the absence of a wall, we can determine within broad limits, the effectiveness of a wall in repelling these waves.

The theoretical attack (by Munk)(6) on the problem of breakers has concentrated on the analogies between an oscillatory wave near breaking and a so-called solitary wave. "The application of the solitary wave theory was suggested...by an obvious resemblence between the theoretically derived wave profile and the observed profile in the region just outside the breaker zone." Actually, a solitary wave is a single plus whose length is infinite. However, most of its energy is concentraed about the crest, and in this manner resembles an oscillatory wave about to break. The assumption here, is that a breaking oscillatory wave is independent of following or preceding waves. Its wave length in the breaker zone is not a determining parameter for the wave's characteristics. There are two relations of importance derived from the application of solitary wave theory to oscillatory waves of finite length; that the relative height of a breaker is dependent only on the initial steepness of the incident wave,

$$\frac{H_b}{H_o} = \frac{1}{3.3\sqrt[3]{H_o/L_o}} \qquad (1)$$

and that the ratio of depth of breaking to breaker height is constant.

$$d_b/H_b = 1.28 \qquad (2)$$

therefore

$$d_b/H_o = d_b/H_b \times H_b/H_o = 1.28 \quad \frac{1}{3.3\sqrt[3]{H_o/L_o}} \qquad (3)$$

Graphs drawn from relationships (1) and (3) are shown on Figures 1 and 2. Their most noteworthy aspect is the dependence of breaker parameters on the initial steepness of the waves. The ratios between breaker height and deep water wave height and between breaker depth and deep water wave height increase with decrease in initial steepness. This may be interpreted to mean that on a given beach, a steep wave will break at a point before one less steep, and, before breaking, will have a smaller growth in proportion to its original height.

All observations, though with a large amount of scatter, have confirmed the existence of these general tendencies.

Two compilations of empirical data should be noted which have a bearing on this discussion of breakers. The first(7) is a plot of a great number of breaker observations made both in the laboratory and in the field. Through these points (which show a great deal of scatter) is drawn an average curve for d_b/H_o vs. H_o/L_o and H_b/L_o vs. H_o/L_o. (See Figures 1 and 2. Both curves lie fairly close to those developed

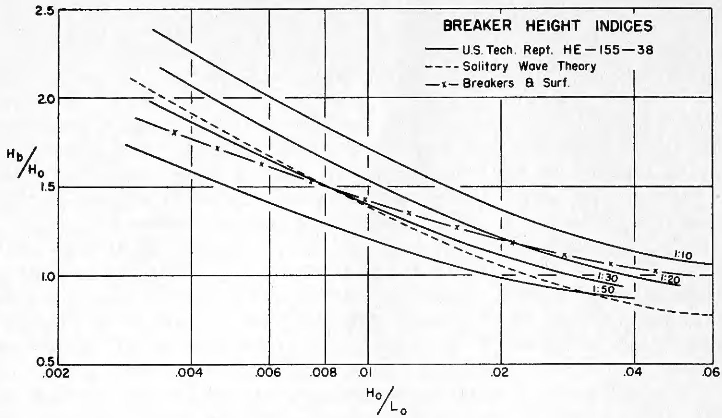


Figure 1.

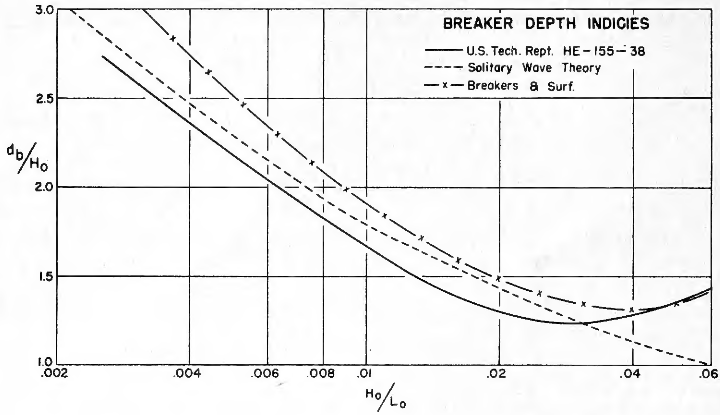


Figure 2

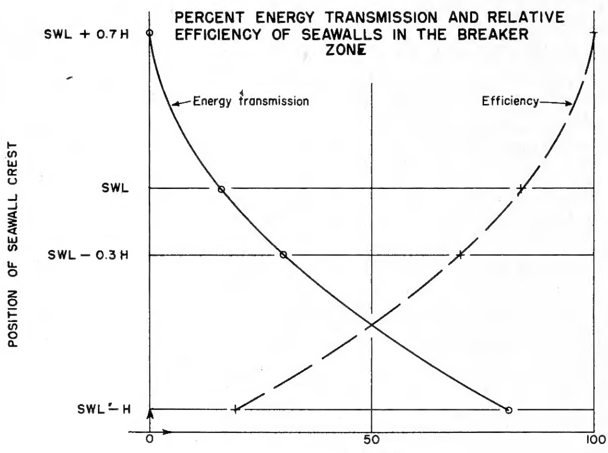


Figure 3

from the solitary wave theory, though the ratio d_b/H_b varies in range about 1.7 to about 1.2 rather than remaining constant.

The second compilation presents the results of an extensive laboratory investigation of breaker kinematics made at the University of California. During the tests made on various slope beaches, a correlation was noted between beach slope and relative breaker height. That is a wave with an initial steepness of (say) 0.01, on a 1:50 slope, will have a relative breaking height (H_b/H_o) of 1.4 but on a 1:10 slope H_b/H_o will be 1.7. The results still follow the general results derived from the solitary wave theory, i.e. that a steep wave will break before a shallow wave on a given beach, but instead of one curve, a family of curves is presented. It should be noted that the field verification of the solitary wave theory of breakers was conducted on the Scripps Institution of Oceanography beach which "...was of an average slope of approximately 1:30..."(8) Munk's curve for relative breaker heights vs. initial steepness follows quite closely the University of California curve for a 1:30 slope which would seem to further verify the laboratory results.

All in all, the University of California proposal that beach slope affects relative breaker height seems to be presented with enough data taken under controlled laboratory conditions to justify the use of its index curve.

To find either breaker heights or depths knowing a deep water height (H_o) and length (L_o), refraction diagrams must be constructed, and an equivalent deep water wave height H_o' determined from $H_o' = H_o \times K$ where K is the refraction coefficient found from the diagrams. The breaker heights (H_b) and depths (d_b) are then determined from Figures 1 and 2 by use of the ratio (H_o'/L_o).

<u>Height of Seawall to be Fully Effective</u> - If we assume then, that a seawall is placed at the point at which waves would ordinarily break in the absence of the wall, and that the wall has no effect on the magnitude of the breaker at that point (except as noted before, to divert part of the horizontal wave momentum <u>on</u> striking the wall) to be totally effective the wall must have a height equal to or greater than the crest height of the highest breaking wave expected. This height is composed of two parts: The water depth plus the wave's crest height above still wave level. The breaker height index (Figures 1 and 2) will give the maximum wave height to be expected at a certain beach location, provided that the deep water wave height and steepness are known. A review of the data presented by Reynolds(9) indicates that about 68% of the wave height on breaking is above still water level. Therefore, we may say, calling h_t the height of tide above some datum (MLW for example) that the wave's crest height on breaking will be $h_t + 0.7\ H_b$ above the chosen datum. This is equivalent to stating that a wall at which the maximum tide height above (say) MLW expected is h_t and which is founded in such depth that the maximum breaker height expected is H_b, will be totally effective if its crest

height h_c above MLW is

(4) $$h_c = h_t + 0.7\ H_b$$

<u>Comparative Effectiveness of Low Seawalls</u> - Though it is possible with the relationship just determined to design a seawall to be completely effective in turning back the highest tide and wave expected at its position, it is quite likely, due probably to economic considerations, that such a wall would not be feasible to construct. The question then arises of a wall's relative effectiveness when its crest height is below that level which would completely turn back a certain height of wave.

Theoretically the problem has been solved for surface waves of small amplitude (23) by considering the energy distribution of a wave in the vertical, and assuming that that portion of the energy which impinges on the submerged wall is not transmitted. (This criterion is an extension of the one adopted previously for total effectiveness of a wall). The results of this particular analysis cannot be extended to the case at hand for, by considering waves of small amplitude, the expression for the ratio of transmitted wave height to incident wave height to incident wave height becomes (in shallow water) $H_t/H_i \cong \sqrt[4]{1-h/d}$ where h and d are respectively the wall height and water depth before the wall. This indicates that when a wall is at the height of still water, nothing may be transmitted over it. Practically this if far from true.

Similarly we must reject, for our case, another attack on the problem made on the basis of shallow water wave theory (24) (25) (theory of tides). In this derivation, the expression for the transmission coefficient, (H_t/H_i) becomes 2 when h = d, but conservation of energy demands that this transmitted wave be propagated with zero velocity.

Since neither of the two theoretical treatments may be applied, we must take recourse to any observed or experimental work done on the problem. One study (26) made in an attempt to correlate wave parameters (especially length) to depth of water over a reef predicts (as would be expected from the theoretical treatments) a decrease in ratio of wave length over the reef to that before the reef, but unfortunately gives no information as to relative wave heights. Other studies (27,28) deal with underwater barriers of various cross-sections, all of which however are located seaward of the breaker zone.

The only study of which application may be made in the present case is one by J. Morison (29,30) on the damping effect of submerged rectangular barriers, some of which were located in the breaker zone. Even here, the application must be limited, for the problem at hand is essentially that of a nearly horizontal reef in shallow water, while Morison dealt with a rectangular barrier of finite width. However, broad relationships may be derived which deal with the amount of energy transmission over the barrier.

When the model was placed on a sloping beach at or near the surf zone, with its top one wave height below still water level, the transmitted wave height was approximately 90% of the incident height. When the model crest was placed at the level of the trough of the incident wave, this transmission coefficient was reduced to 55%, and when still further raised to still water level the coefficient became 40%. If we assume that the relative energy transmitted is proportional to the square of the relative transmitted wave height (this is not strictly true since energy is also a function of wave length, but another study(27) (See page 13) indicates that the relationship of heights squared is sufficiently accurate) the energy transmitted at these three barrier crest heights is approximately 80%, 30%, and 15% of the incident wave energy. Letting the energy transmitted over a wall be the measure of its effectiveness, we have four points through which a curve of wall efficiency versus its crest height relative to still water level may be drawn. (See Figure 3).

V. Seawall Seaward of the Breaker Zone

General - It is quite possible that a seawall must be placed on a slope in such a position that the depth of water at the wall would not be shallow enough to cause the maximum expected wave to break. That this may come about may be seen by referring to section II in which water depth variability is discussed. The wave attack at such a location will differ from that on a structure in the breaker zone, therefore a different approach must be used to find the maximum wave height expected at the wall's depth.

Theoretically, many approaches have been made to determine the change in wave parameters with decrease in depth. A few will be noted. For waves of finite height, Stokes(10,11) and Struik(12,13) found to a third approximation that the velocity of oscillatory waves is given by

$$(5) \qquad C^2 = \frac{gL}{2\pi} \tanh \frac{2\pi d}{L} \left[1 + \left\{ \frac{\cosh \frac{8\pi d}{L} + 2\cosh \frac{2\pi d}{L} + 6}{8\left(\sinh \frac{2\pi d}{L}\right)^4} \right\} \left(\frac{\pi h}{L} \right)^2 \right]$$

and the wave form by

$$(6) \qquad y = a\cos \frac{\pi x}{L} - \frac{2\pi a^2}{L} \left[\frac{\left(\cosh \frac{2\pi d}{L}\right)\left(\cosh \frac{4\pi d}{L} + 2\right)}{8\left(\sinh \frac{2\pi d}{L}\right)^3} \right] \cos \frac{4\pi x}{L} +$$

$$+ \frac{\pi^2 a^3}{L^2} \left[\frac{5\cosh \frac{12\pi d}{L} + 14\cosh \frac{8\pi d}{L} + 12\cosh \frac{4\pi d}{L} + 16}{32\left(\sinh 2\pi d/L\right)^6} \right] \cos \frac{6\pi x}{L}$$

Since seawalls will always be located in relatively shallow water, the solitary wave theory(6) may also apply. This gives for the velocity

$$(7) \qquad C^2 = g(d + H)$$

and for the profile

(8) $$y = \frac{H}{\sinh^2\left(\sqrt{\frac{3}{4}\frac{H}{d}}\left(\frac{x}{d}\right)\right)}$$

Recently J. J. Stoker[14] has extended the non-linear shallow water wave theory by means of methods derived for the study of unsteady flow in one dimension of a compressible gas. The theory is approximate, and application is lacking, but it is interesting to note that the procedure permits the analysis of unsteady motions and can perdict the wave form at all points up a beach to the breaker. The continuous wave form so derived becomes assymetrical as the breaker line is approached, with the wave front slope steeper than that behind the wave crest. All other theories however, approximate the unsteady motion up a beach by a series of different steady motions. The assumption is that at every depth on a slope, the wave will behave as if it were advancing over a horizontal bottom at that depth. The wave form then is predicted approximately by a series of still pictures, instead of a continuous record. Munk's theory in particular predicts a breaker which on the whole is symmetrical in shape, while Stoker's development predicts a marked steepening of the wave front and a very unsymmetrical shape for the waves at breaking.[15]

The theory most commonly used for the prediction of wave parameters is that of progressive oscillatory waves of small amplitude. This theory as with Stokes' second approximation for waves of finite amplitude gives for the wave velocity

(9) $$C^2 = \frac{gL}{2\pi} \tanh \frac{2\pi d}{L}$$

To obtain an expression for the change of wave height with depth, the assumption is made that the wave form approximates a sine curve[16] (or better that the effect of higher order terms may be ignored). That is

(10) $$y = \frac{h}{2} \cos \frac{2\pi x}{L}$$

The potential energy per unit surface area is given by

(11) $$E_p = \frac{\omega h^2}{16}$$

and the kinetic energy is numerically equal, therefore the total energy is

(12) $$E_t = \frac{\omega h^2}{8}$$

It has been shown for both deep water[18] and shallow water waves[19] that of this energy only a portion is transmitted forward with the wave form, and that this portion is given by the ratio of group velocity to the wave velocity[17]

(13) $$C_g/C = \frac{1}{2}\left(1 + \frac{4\pi d/L}{\sinh 4\pi d/L}\right) = n$$

which in deep water $= \frac{1}{2}$. The mean rate of transmission of energy per wave length and per unit crest width (the power) is $P = n\,E_t\,C$. This rate of transmission remains the same in both deep and shallow water ($P_o = P$) and equating the two we have, if there is no refraction $E_{to}\,n_o\,C_o = E_t\,N\,E$ or

(14) $$\frac{E_T}{E_{T_o}} = \frac{n_o C_o}{n C} = \frac{1}{2n}\,\frac{C_o}{C} = \left(\frac{H}{H_o'}\right)$$

With refraction[20] the assumption is made that the power transmitted between orthogonals to the wave crest remains unchanged, therefore calling the ratio of the distance between two orthogonals in deep and shallow water $\ell_o/\ell = K^2$.

(15) $$\frac{H}{H_o} = \left(\sqrt{\frac{1}{2}\cdot\frac{1}{n}\cdot\frac{C_o}{C}}\right)K = \left(\frac{H}{H_o'}\right)K$$

For these waves the wave lengths in shallow and deep water are related by $L/L_o = \tanh \frac{2\pi d}{L}$. This relationship permits the calculation of wave parameters in shallow water as functions of the deep water wave length, and as an aid in calculation, tables of these relationships have been compiled and published. Figure 4 is a curve of H_o/H_o' for various values of d/L and d/L_o.

Height of Seawall to be Fully Effective - With the aid of the relationships between shallow and deep water wave height (Figure 4), we can find the maximum wave to be expected at a seawall if it is so placed that these waves would not break on attaining this depth in the absence of the wall. To apply the established criterion for total effectiveness of the wall, i.e. that its crest height be at least as high as the crest height of the highest impinging wave we must find anew the percentage of wave height which lies above still water level. The paper by K. C. Reynolds[9], cited before, indicates that except in the immediate vicinity of the breaker zone, this percentage rarely exceeds 60%. If it is determined therefore, that a seawall must be placed on a slope so as to be open to attack by non-breaking waves, its crest height be above (say) MLW, for total effectiveness must be

(16) $h_t = h_t + 0.6\,H$

where h_t is the height above MLW of the greatest expected tide, and H is the greatest wave height expected.

H/H_o' FOR VARIOUS VALUES OF d/L AND d/L_o

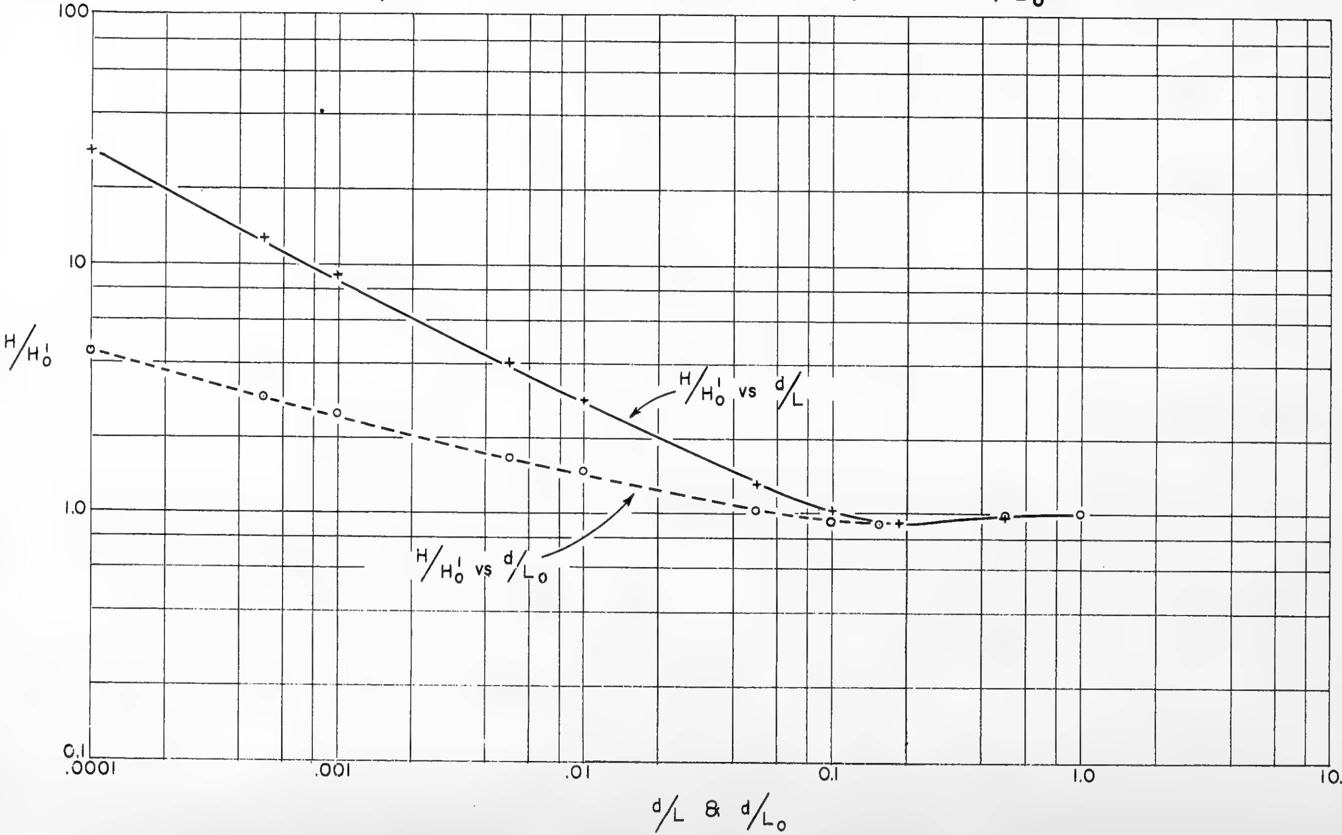


Figure 4

Comparative Effectiveness of Low Seawalls - By the same method used on page 7, we may determine the relative effectiveness of seawalls of a height not capable of completely turning back the expected wave attack. The primary source is the same paper by Morison[29] dealing with rectangular barriers, this time using his results for steep waves over an horizontal bottom. (The range of wave action at a seawall's probable location on a slope will resemble this model). The results for the heights of barrier reported on follow.

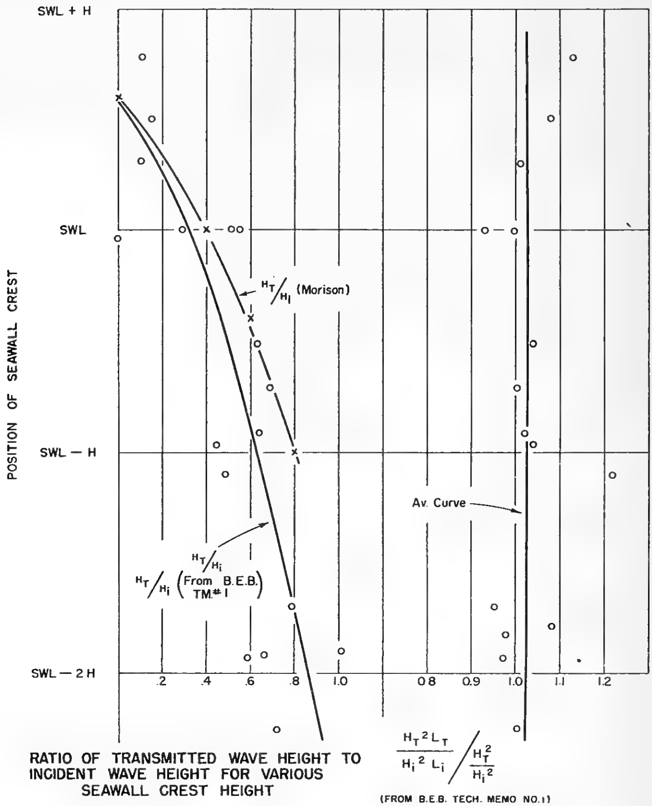
Depth of barrier crest below still water level (in terms of wave hgt.)	Ratio of transmitted to incident wave height	Ratio of transmitted "Energy" to that incident
H	0.8	0.64
0.4H	0.6	0.36
0	0.4	0.16

There is one other source which, in a broad way, confirms two important results of Morison's paper. One type of barrier tested by the Beach Erosion Board[27] was a vertical plane (e.g. a sheet pile bulkhead). If the results of this study are plotted as the ratio of depth over the barrier to incident wave height versus the relative height of transmission (Figure 5) a wide scatter of points is noted. However an average curve drawn through these points lies close to a curve drawn through points plotted from Morison's data. Actually Morison's points show higher transmission values, and therefore the use of his results should be conservative. If in addition a plot is made (Figure 6) of the ratio of relative energy (actual) transmitted $H_t^2 L_t / H_i^2 L_i$ to the square of the relative transmitted height (H_t^2/H_i^2), these points show little scatter, indicating that energy transmission may be approximated by (H_t^2/H_i^2), (the assumption made previously).

Therefore, the comparative effectiveness from an energy standpoint, of low barriers may be obtained from Figure 7, a plot of the ratio $\frac{\text{depth of water over the barrier}}{\text{incident wave height}}$ vs. $\left(\frac{H_t}{H_i}\right)^2$

VI. Seawall Shoreward of the Breaker Zone

If a seawall is placed far up a slope, and the water level fluctuation at its location is not great, damaging waves will break before reaching the structure. Indeed this is a common situation at many points on open sea coasts. Unfortunately, this problem is the most difficult of the three to analyze, for little has been done to find the dynamic characteristics of waves after breaking. Instead of undergoing predictable laminar orbital motion, the water is in a highly complex turbulent state which defies analytical breakdown.



RATIO OF TRANSMITTED WAVE HEIGHT TO INCIDENT WAVE HEIGHT FOR VARIOUS SEAWALL CREST HEIGHT

Figure 5

Figure 6

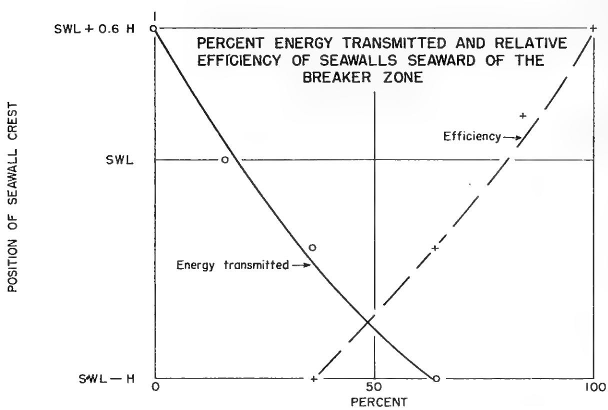


Figure 7

A suggestion for a theoretical approach was made in 1947 by Stoker,[14] who noted the similarity between broken waves and hydraulic jumps or shock waves. However this analogy has not been explored further.

Notwithstanding the paucity of information on the problem of broken waves and their characteristics at points landward of the breaker zone, some logical criterion should be established to determine how effective a seawall would be if so placed that the impinging waves are already broken.

At breaking, a wave reaches its maximum amplitude. Moving up a beach from the point of breaking, this amplitude must decrease, since energy is dissipated in the turbulent flow. However, this decrease has not been measured nor estimated and therefore no value may be placed on it. In order to insure conservative results for seawall height, the maximum wave amplitude instead of some lesser value should be employed.

The use of Figures 1 and 2 permits establishment of the depth and height of a breaking wave, and from these the maximum crest elevation may be determined. ($h_t + 0.7\ H_b$ MLW, see page 8). The criterion to be adopted for total effectiveness of a seawall follows:

If a horizontal line be projected from the point of the breaker crest shoreward to a seawall's proposed location, the seawall's crest should be at least as high as this line. That is, the absolute height of seawall crest should be equal to or greater than the absolute height of a breaker crest. This is essentially an extension of the analysis of a wall in the breaker zone.

To determine the relative effectiveness of seawalls lower than this height, the curves of Figure 3 may be employed. Actually such use is an extension of an approximate result, an an assumption of accuracy would be unwarranted.

VII. Summary

On the preceding pages general criteria have been established for absolute and relative effective heights of seawalls within, landward of, and seaward of, the breaker zone. For structures within and landward of this zone, to be totally effective their height h_c above some datum should be

$$h_c = h_t + 0.7\ H_b$$

where h_t is the maximum expected tidal height above this datum and H_b is the maximum expected breaker height at the wall's position. A basis for establishing relative effectiveness of lesser height walls is presented on Figure 3.

Similarly for seawalls located seaward of the breaker zone, for total effectiveness

$$h_c = h_t + 0.6 H_b$$

where here H is the maximum expected wave height at the wall's position. Relative effectiveness of lower walls is shown on Figure 7.

VIII. - An Actual Case - Galveston, Texas

Unfortunately little data are available on the type of storm attack experienced by those seawalls which have withstood such attack. Descriptions of coastal structures turning back damaging wave action are usually graphic, containing phrases such as "a huge wave" which are of little practical value. The report on Galveston's seawall[3] is an exception, though even here, a large portion of descriptive material is wholly subjective.

After a violent hurricane in 1900 which caused damage to most of the city, a seawall was constructed to a crest height of 17 feet, MLW. (the hurricane caused storm tide heights up to 15 feet). In 1915, another storm of comparable intensity accompanied by a storm tide of 12.5 feet struck the protected area. This tide height left about 10 feet or more water depth at the toe of the wall, and a wall free-board of only 4.5 feet. Though no description of the waves is given in the report, it is easy to guess that wave action accompanying the storm overtopped the wall. Portions of the report read, "Considerable quantities of water came over the wall, seriously damaging the embankment back of it in places....", "The volume of water passing over the wall was surprisingly large. One observer reports that at Sixth and Broadway, the water appeared to be coming over in a continuous sheet estimated to average 2 feet deep."

The distance from tide level to a point 2 feet above the seawall crest is about 6.5 feet. Therefore the wave height (equation 4) should have been about 6.5/0.7 = 9 feet. Other parts of the report estimate that waves "of any material frequency were about 5 feet higher than the wall;" in this case the wave height itself would have been 9.5/0.7 = 13.5 feet. Considering that the wind attained a maximum velocity of 93 miles per hour at Galveston, waves of these heights are not excessive.

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NOTE: The Bulletin will welcome comments or discussion of the foregoing or any other articles published in the Bulletin.

LABORATORY STUDY OF AN ELECTROMAGNETIC CURRENT METER

This article is a brief summary of some recent work done at the Beach Erosion Board's laboratory with a view to devising an instrument that would measure and record internal water velocities associated with wave motion. The work was accomplished by H. A. Taylor and C. M. Hare under the direction of J. M. Caldwell, Chief of the Research Division of the Beach Erosion Board.

In order to obtain more complete measurements of water wave characteristics, an instrument is desired which will accurately measure and record the orbital velocities of the water particles within a wave formation. It appeared that an all electrical instrument would eliminate bulkiness and have a high degree of flexibility, so the investigation was limited to an instrument utilizing the basic principle of electromagnetic induction. The motion of the water as associated with wave action would serve as a moving conductor in which an electro-motive force (EMF) would be induced in the presence of a magnetic field. A pair of electrodes, in the water connected to a suitable recording device would pick up and record the induced voltage, which if the field strength and electrode separation were maintained constant, would be directly related to the water velocity. The electrode alignment would be perpendicular to the field direction, and thus only the velocity component perpendicular to both field and electrode alignment would contribute to the voltage picked up by the electrodes. This suggests the possibility that two mutually perpendicular pairs of electrodes could be used to measure and record the components of a velocity both parallel and perpendicular to a given base line, and from these simultaneous values the magnitude and direction of the incident velocity could be computed.

After study of published works of other experimenters, it was decided to investigate the performance of an instrument similar to one proposed by Guelke and Vanneck.* Their instrument consisted of a toroidal coil to provide the magnetic field and pick-up electrodes in a plane parallel to the plane of the coil, suspended at any given distance along the coil axis. Guelke and Vanneck utilized alternating currents to energize the field coil as the use of direct current usually causes polarization of the electrodes. However, the use of alternating current appeared to have the following disadvantages: (1) an alternating field would induce an alternating voltage in any loops formed by the electrode leads, the magnitude of which could exceed that expected for

* The Measurement of Sea-Water Velocities by Electromagnetic Induction, R. W. Guelke, C. A. Schoute-Vanneck, Journal, Institution of Electrical Engineers, London, Vol. 94, Pt. 2, No. 37, February 1947.

the voltage induced by the water velocity; (2) the alternating field would cause an induced voltage in the water even though the water velocity were zero; (3) the alternating induced voltage would have to be rectified before it could be recorded in a direct current instrument; and (4) an alternating current power supply cannot be provided for a field installation as simply as a direct current supply such as portable storage batteries. For these reasons it was decided to employ direct current in the investigations conducted at the Beach Erosion Board's laboratory, with the expectation that the one disadvantage associated with its use, that of polarization, could be satisfactorily overcome.

Calculations indicated that an instrument using a toroidal exciting coil, energized by direct current, with two pairs of electrodes aligned on perpendicular axes to record simultaneously induced voltages on two General Electric Photoelectric Potentiometer Recorders, would be practicable. However, before assembling an instrument which would be practical for field tests, preliminary tests were made with a small laboratory model utilizing the same basic principles. For this purpose a small field coil and a storage battery were used. Copper wire electrodes coated with colloidal graphite were introduced into a specially built flume which provided a known water velocity, and the induced voltage was recorded on a portable d'Arsonval galvanometer. On the basis of the known water velocity and field strength, the anticipated induced voltage was computed to be approximately 0.17 millivolts. During these tests, deflections of approximately one unit were observed on the galvanometer whose sensitivity was estimated at 0.15 millivolts per unit. However, throughout the tests varying deflections of the galvanometer were noted which apparently were caused by a varying potential induced by some other source than the magnetic field and water velocity. This externally induced masking potential made observation of the smaller deflection caused by the water velocity induced potential very difficult. Qualitatively though, these preliminary tests indicated that the induced voltage was directly proportional to the velocity of the water and the separation of the electrodes.

Another series of tests was then initiated utilizing the same equipment described above with the exception that various types of electrodes were used, and the electrodes were connected to a General Electric Photoelectric Potentiometer Recorder, model 8CE5 DM5Y-1. The results of this series of tests were unsatisfactory in that the masking potential was still present. No noticeable change in reading was obtained from the recorder upon applying a voltage to the field coil, but a potential of greatly varying amplitude and varying polarity was present at all times. The induction of this troublesome masking potential into the measuring circuit was attributed to the chemical electrolysis between the water and the electrodes. To minimize the effect of this undesirable chemically induced voltage, another series of tests was made with a different coil providing a magnetic field of considerably greater strength. Calculations of induced voltage for different velocities of water flow were made for the new coil, which under some conditions exceeded the indicated value of the chemically produced voltage.

The several types of electrodes used for these tests are as follows:

a. 22 gauge bare copper wire
b. 1/4 inch bare copper tubing
c. 3/8 inch bare copper tubing
d. 1 inch copper discs, formed from capillary tubing
e. 22 gauge nichrome wire
f. 22 gauge bare copper sheet, exposed surface $1\frac{1}{2}$ inches by 1/4 inch
g. 22 gauge tantalum wire
h. 17 gauge titanium plate, exposed surface 1 inch by 1/4 inch

No satisfactory measurements could be made utilizing the 22 gauge bare copper or nichrome wire electrodes. A potential always existed across these wire electrodes which completely obscured any water velocity-induced voltage. The electrodes made up of copper tubing and those shaped as discs were discarded because they obstructed the flow of water and created turbulence which resulted in wildly fluctuating readings on the recorder. It was recognized that the wire electrodes had a terminal resistance much higher than the resistance value recommended to be connected with the recorder. The operating recorder requires a small current from the measured potential, and it is believed that this fact combined with the varying electrolytically-induced potential prevented the recorder from reaching a balance and recording the velocity-induced quantity. Electrodes fashioned from the highly corrosion resistant metals of tantalum and titanium were also unsatisfactory. The limited action between the water and electrodes resulted in a high circuit resistance and no satisfactory readings could be taken. Several adapter circuits developed for use between the electrodes and the recorder were tested, but proved unsatisfactory.

Efforts directed toward reducing the terminal resistance of the electrodes resulted in the use of the copper sheet electrodes, and with this type the velocity-induced voltage produced when the coil was energized, could be clearly observed on the recorder as superimposed upon the changing reading of the potential produced by electrolytic action. It was shown that in the presence of the magnetic field a voltage was induced in the flowing water that was a direct but non-linear function of the water velocity. The salinity of the water had no apparent effect on the relationship between velocity and the induced voltage.

Before a practical instrument utilizing the principle of electromagnetic induction can be developed to measure and record water velocities satisfactorily, it appears that a suitable means of eliminating or compensating the chemically induced voltage must be found. An instrument built for continuous operation must also overcome the basic deficiency of the direct current field, namely the polarization of the electrodes. With these basic deficiencies still unsolved after a fairly comprehensive investigation, the Research Division of the Beach Erosion Board has presently discontinued work on this project, although it still has a decided interest in encouraging the development of an instrument to measure the magnitude and direction of orbital wave currents whether the instrument utilizes electromagnetic induction or some other principle.

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PROGRESS REPORTS ON RESEARCH CONTRACTS

It is proposed that future issues of the Bulletin include abstracts from progress reports on the several research contracts in force between universities or other institutions and the Beach Erosion Board. The following is based on progress reports from three such contracts.

I. <u>University of California, Status Report No. 4, 1 December 1951 through 31 January 1952.</u>

This report pertains mainly to the origin of sand upon beaches, particularly with reference to beaches of Southern California.

<u>Work Completed in Current Period</u>

1. Three trips were made to Santa Barbara:

   a. 12 to 15, December, immediately following a period of heavy rainfall to collect sand samples from principal streams entering the ocean, in order to make mineralogical studies of the sands with the object of determining the source of sand on the ocean beaches.

   b. 26 to 30, December, detailed survey of Santa Barbara Harbor and beaches at time of the year's lowest tides. The extremely low water permitted the detailed determination of slope of the underwater points of the sand island, which was found to range between 29° and 31°, averaging 30°. The feeder beach east of the harbor receded rapidly during the very high tides that preceded the very low tides.

   c. 17 to 20, January, photographic survey of stream and beach erosion immediately following the major floods of January 15 to 17. These floods were the most severe in 15 years. Sand samples for mineralogical analysis were taken at the same localities as in the December survey, and at other places as well.

2. Mechanical and mineralogical analyses of the samples collected during the two December surveys have been furnished.

3. The comprehensive report of the results of the current year's study is 75 per cent complete.

4. A summary report of progress is 98 per cent complete. The mineralogical studies have indicated that mineral composition of the sediments varies very little in the Santa Barbara area itself, whereas it varies significantly along the coast west and north of Santa Barbara.

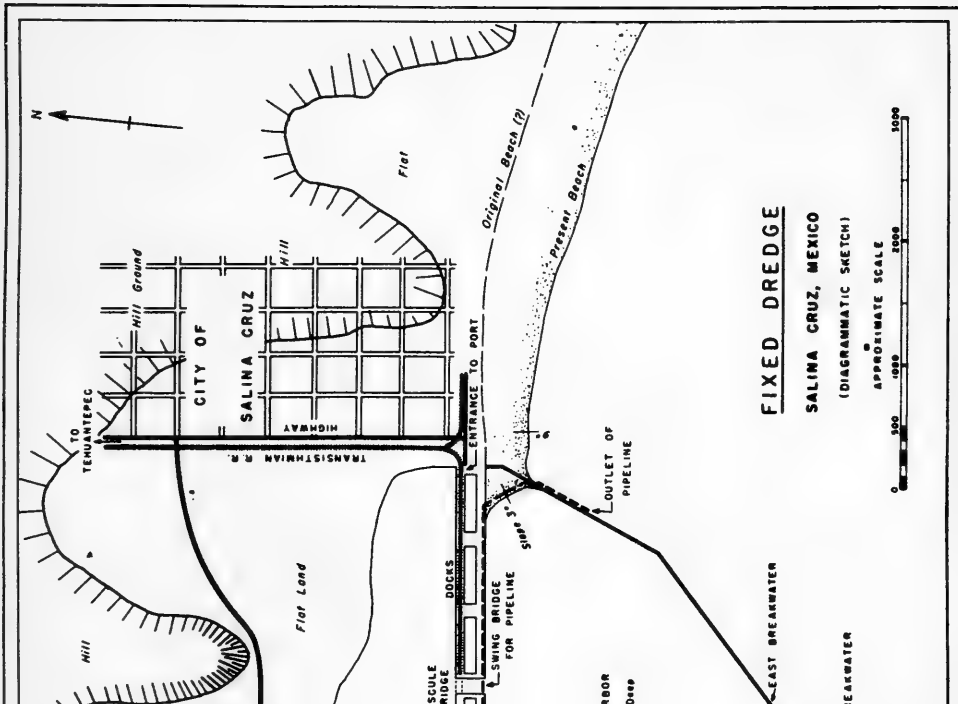
The difference in mineral content indicate that sand moves around Point Conception and Point Arguelo. As the streams have not been in major flood for a number of years, it follows that most of the 900 cubic yards of sand a day that is trapped in Santa Barbara comes from off shore areas or littoral drift from the north.

5. A report by Parker D. Trask entitled "Stationary Dredge for By-passing Sand at Salina Cruz Harbor, Isthmus of Tehuantepec, Mexico," was submitted to the Beach Erosion Board on 29 January. This report was based upon an inspection of the project in November 1951. The project was described in the Bulletin of 1 July 1951. The following abstract from Mr. Trask's report contains additional data on the operation of the project.

Stationary Dredge for By-Passing Sand at Salina Cruz Harbor, Isthmus of Tehuantepec, Mexico

Salina Cruz is located in a setting of great beauty on a small flat surrounded by hills on the Pacific coast of Southern Mexico. The harbor is entirely artificial, though a small lagoon evidently existed in the area at time of the construction. The two breakwaters that form the harbor are 4,000 feet in length, 700 feet apart at the entrance to the harbor and 3,500 feet apart at the rear of the harbor (Figure 1). These breakwaters are 18 feet wide on top and an equal height above mean low water. Massive granite riprap composed of blocks 8 or more feet in diameter effectively protects the breakwaters from the waves. A transverse dock supplied with roadways and railroad tracks divides the harbor into an outer and an inner port. Entrance to the inner port is made through a bascule drawbridge. The harbor is modern up to date in every respect, and is very efficiently managed.

The harbor and railroad across the Isthmus of Tehuantepec were constructed and placed in operation prior to the completion of the Panama Canal. In time sand, moving toward the east, filled the area between the breakwater and the headland to the west. Subsequent to 1910 the sand is reported to have drifted across the mouth of the harbor, effectively closing it to shipping. This sand was later removed, and the harbor dredged to a reported depth of 40 feet. As of November 1951 the harbor is said to have shoaled in places to 30 feet.



N
FIXED DREDGE
SALINA CRUZ, MEXICO
(DIAGRAMMATIC SKETCH)
APPROXIMATE SCALE
0
500
1000
2000
3000
CITY OF SALINA CRUZ
TO TEHUANTEPEC
TRANSISTHMIAN R.R.
HIGHWAY
ENTRANCE TO PORT
Hill Ground
Hill
Flat
Original Beach (?)
Present Beach
OUTLET OF PIPELINE
Slope 3°
DOCKS
SWING BRIDGE FOR PIPELINE
BASCULE BRIDGE
EAST BREAKWATER
OUTER HARBOR
30'-50'(?) Deep
Flat Land
Hill
ROAD
Flat
INNER HARBOR
DRYDOCK
DOCK
BOOSTER PUMP
WAREHOUSES
Fill
PIPELINE
Sand Island
Slope 5°
WINCH
POOL
DREDGE BLDG.
WEST BREAKWATER
PILE BULKHEADS
DRAG LINE
BUOY
Sand Mounds
Old Sand Fill
Granite Porphyry
Present Beach
BREAKWATER
LIGHTHOUSE
Hill 200'-300' high
TANKS
Flat
Original Beach (?)

FIGURE 1

At the present time the shore line is 75 feet landward from the end of the breakwater, as shown in Figure 1 and 2. The height of the sand fill is estimated to be 12 feet above mean low water. As the original depth of water at the present position of the shore line prior to the construction of the harbor was of the order of 40 feet, the average thickness of sand below mean low water is 20 feet and the average thickness of the entire fill is 32 feet. The area filled by sand is a triangle, extending 2000 feet along the shore and 3000 feet inland. The volume of fill, accordingly, is of the order of 3,500,000 cubic yards.

The sand on the beach is coarse, the average diameter is 0.4 to 0.6 millimeter (Table 1). The slope of the beach is 9° in front of the normal storm berm, and 12° in front of the artificial piles of sand near the fixed dredge. (Figure 2). Numerous rock fragments up to 6 inches in diameter, composed of granite, geniss, and porphyry, are washed along the beach by the waves. The water deepens rapidly off shore. The waves commonly are 4 to 6 feet high. The tide has a maximum rise and fall of 5 feet.

The area of sand fill ends 2200 feet west of the breakwater in a rocky point, composed of coarse-grained granitic rocks. A small rocky mass consisting of granite porphyry, lies 900 feet east of the rocky promontory (Figure 2). A jetty 400 feet in length has been built 300 feet east of the point, in order to trap sand that might otherwise settle on the beach and build it forward. This jetty had been in operation for 12 to 18 months prior to November 1951. During that time a bench had been built seaward 100 or more feet and up to about mean low water level on the west side of the jetty. The beach above this bench slopes 4°. The average grain size is 195 microns. High tide level is at the same position on the two sides of the jetty. In other words, sand that thus far has been trapped west of this jetty is much finer than sand deposited on the beach east of the jetty. Also, the sand has been laid down largely below mean low water. The jetty hence does not seem to be catching much of the material that moves along the shore, especially the coarse sand that forms the major part of the fill.

The beach along the ocean east of the harbor is 200 to 300 feet wide. It is composed of medium-grained sand, having an average diameter of 395 microns. The beach slopes 9° and is encroaching upon the breakwater at a very slow rate, if any.

A sand island is forming inside the harbor about 1000 feet inland from the end of the west breakwater. The beach facing the sea on this island slopes 5°. The sand is fine grained; the average diameter is 190 microns. A submerged beach is being built along the inside edge of the breakwater between the sand island and the end of the breakwater, as is attested by waves breaking along the breakwater as they move inland. Rocks up to 6 inches in length are washed along this submerged beach by waves. A small mass of fine sand having a median diameter of 170 microns has accumulated in the northeast corner of the outer harbor. The slope of the beach here is 3°.

It is interesting to note that according to the data presented in Table 1, the sand on the beach facing the open ocean is not particularly well-sorted for beach sand, as the coefficient of sorting ranges from 1.34 to 1.41. The sand on the beaches in the harbor and just west of the jetty, is well sorted, having a coefficient of sorting of 1.19 to 1.22. All samples have very little skewness, as the logarithm of skewness ranges between -0.011 and +0.014.

The fixed dredge is housed in a reinforced concrete building 200 feet long and 40 feet wide, with walls 5 feet thick. The base of this building lies 30 feet below mean low water to give the structure stability and protection against waves. The dredge is manned by six suction pipes 18 inches in diameter, which feed into a pipe line of equal dimensions, 7000 feet in length. This pipe line passes along the west side of the harbor and crosses the harbor along the central dock. A booster pump is located at the northwest corner of the harbor. A double swinging bridge carried the pipe across the entrance to the inner port. The pipe comes apart in three places to permit opening of the swinging bridge when ships enter the inner harbor. The level of the pipe is 10 or 12 feet above mean low water. The bends of the pipe at the corners of the harbor have a radius of curvature of 15 feet. The outlet of the pipe is 500 feet off shore on the east side of the east breakwater. It spills out on top of the riprap. No sand island has formed at the point of discharge. As of November 1951, the swinging bridge across the channel to the inner harbor was being maintained in an open position, and sand from the dredge was being pumped to low places west of the harbor.

A pond 100 feet in maximum width has been dug in front of the stationary dredge, but the sand from the ocean does not freely enter this pit so it can be dredged away. In order to cause the shore line to recede to a position where waves can wash sand into the dredging pit, a drag-line has been installed to pull sand into the dredging pit. An anchor buoy and winch are used for this purpose. Two lines of concrete piles about 20 feet apart have been constructed in order to facilitate the entrance of sand into the dredging pit. The drag-line and dredging pumps are said to operate four hours at each high tide. The sand-drag has to be operated continually, as sand soon fills the trough dredged by the drag-line, thus preventing the movement of sand into the dredging pit by natural beach processes. Sand is also scooped from the beach with the aid of bulldozers and piled on top of the storm berm just west of the dredging pit. (Figure 2).

As this dredging progresses, the beach is receding, as is attested by the steep little cliffs at the rear of the fore-slope. When the beach shall have receded to point A (Figure 2), it is planned to remove the line of piles from A to B, and those on the other side as well, so that sand more freely can enter the dredging pit. A temporary line of piles will then be driven along line BC. When the beach reaches B, this series of piles will be removed and a permanent

TABLE 1

Mechanical Analyses of Beach Sand Samples, Salina Cruz, Mexico

Location	Sample Number	Very Fine Sand	Fine Sand	Medium Sand	Coarse Sand	Very Coarse Sand & Fine Gravel	Medium D_{50}	Coef. Sorting	log. skewness	Beach Slope
		Microns						$\sqrt{\frac{Q_3}{Q_1}}$	$\log \sqrt{\frac{Q_3 \times Q_1}{D_{50}^2}}$	Degrees
		62–125	125–250	250–500	500–1000	1000–				
		Per Cent								
West Beach in area of sand piles	946	1.0	12.1	49.6	27.4	9.9	405	1.39	.014	12°
West Beach, normal storm berm area	947	0.3	6.2	35.2	47.0	11.3	545	1.41	-.011	9°
West Beach, west of jetty	948	7.4	72.1	20.5	0.1	-	195	1.22	.004	4°
East Beach, 300 feet east of breakwater	949	1.3	15.5	52.9	27.4	2.9	395	1.34	-.004	9°
Outer Harbor, south-east end	950	11.8	75.8	10.4	1.6	0.4	170	1.19	.010	3°
Outer Harbor, sand island	951	7.3	75.3	17.0	0.4	-	190	1.21	-.001	5°

See Figure 1 for location of samples. Samples taken two-thirds distance up the foreslope.
Sieve sizes used: 62, 125, 177, 250, 500 and 1000 microns.

pile bulkhead will be built from C to D to protect the beach and dredge from ocean waves. The drag-line has been in operation for about six months. The operators believe that one-fourth of the sand scheduled for removal has been taken out.

It is too early to predict the general effectiveness of this fixed dredge installation. The beach most certainly was receding as of November 1, 1951. As shown on Figure 2, the shore line has gone back 30 or 40 feet from the position as shown on the design drawings. The positionof the shore line as shown in a photograph in the Beach Erosion Report (Bulletin, 1 July 1951) taken a short time prior to November 1950 is essentially the same as the position in November 1951. If the position in November 1951 is the same as that of 12 months or more earlier and if the beach is now receding, it would seem as if the beach had advanced and then retreated during the year. The question then arises as to whether the beach is now receding mainly as a result of the dredging action of the drag-line operations or is receding seasonally because of higher waves or higher tides, as is the custom of many beaches. Theoretically it would seem as if the drag line operations were removing sand from the beach, thus facilitating the recession of the shore line. If so, then in time the beach should attain some position whereby sand could progress directly into the dredging pit and be removed. It would be desirable to re-examine the beach in 6 months or a year to determine the rate of recession.

As shown in Figure 2, a permanent pile bulkhead is ultimately planned along line CD to protect the beach and dredge from the waves. This bulkhead is convex seaward, whereas most stable beaches between points of obstruction are concave seaward. It will be interesting to see if the beach does attain a stable convex shape, while at the same time supplying the dredging pit with sand. An alternative that might be considered is to allow or cause the beach to achieve a concave profile extending from the granite porphyry rock to the dredge (Figure 2). However the radius of curvature of such a beach that would be necessary to cause effective natural transmission of sand to the dredging pit might be too short for the beach to remain in equilibrium with the result that the beach would build seaward to an extent that sand could not enter the dredging pit. In such an event the rocky mass of porphyry 1/4 mile west of the dredge, might be removed in order to provide a greater recession of the shore line and a longer and perhaps more stable radius of curvature of the beach. One compilation of such a configuration of shore line would be direct approach of waves to the fixed dredge, which in time of storm might cause serious problems.

Even though a fixed dredge, such as the one at Salina Cruz should prove to be an effective means of combating surplus sand in harbors, the comparative cost of operation and amortization relative to the cost of periodic removal of sand from the harbor by floating dredge is also a factor to be considered. It would seem as if a satisfactory dredge could never trap all the sand that moves along the beach and in

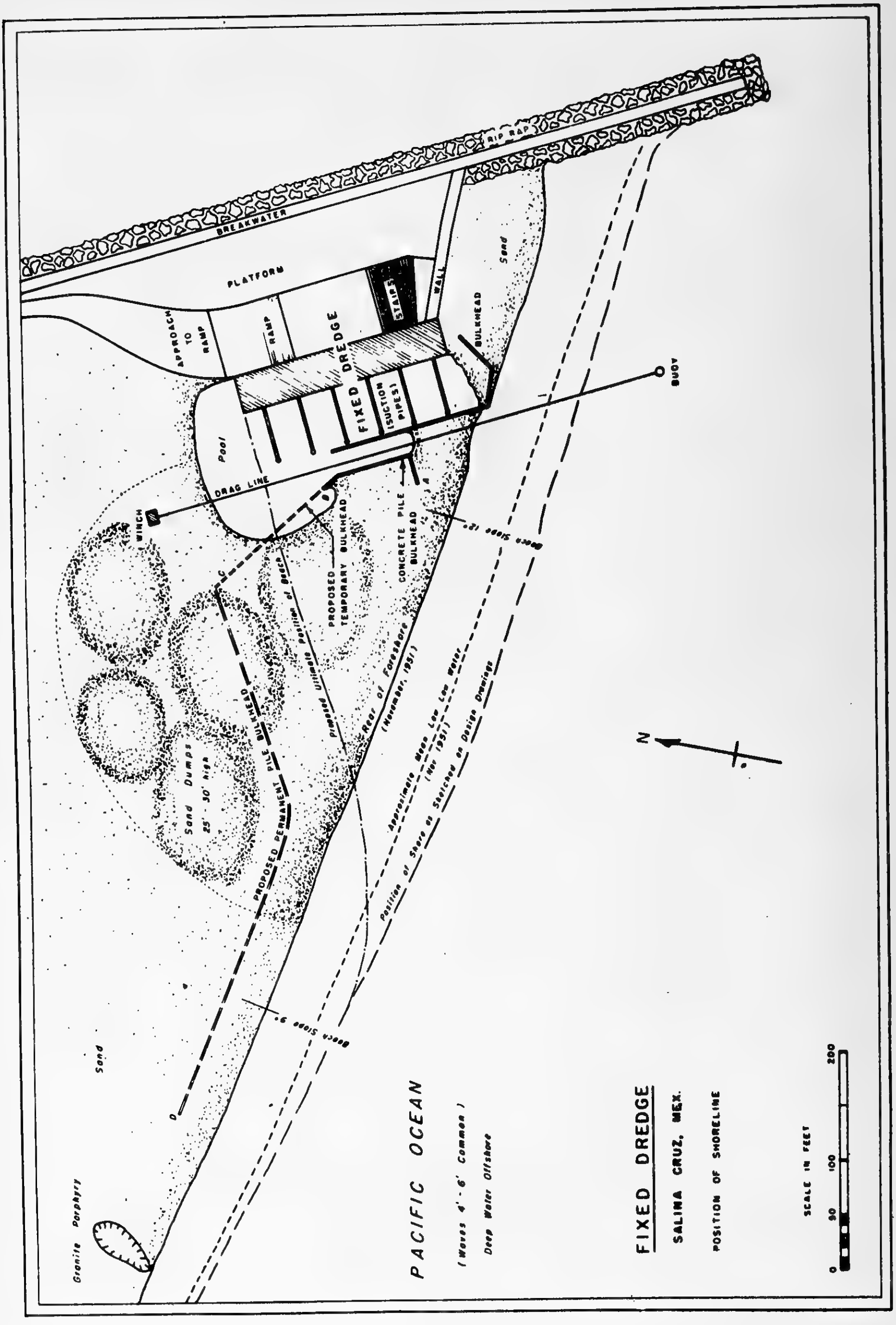

BREAKWATER
RIP RAP
PLATFORM
APPROACH TO RAMP
RAMP
FIXED DREDGE
STAIRS
WALL
Sand
BULKHEAD
(SUCTION PIPES)
BUOY
Pool
DRAG LINE
WINCH
CONCRETE PILE BULKHEAD
PROPOSED TEMPORARY BULKHEAD
Proposed Ultimate Position of Beach
Reef of Foreshore (November, 1951)
Approximate Mean Low Low Water (Nov 1951)
Position of Shore as Sketched on Design Drawings
PROPOSED PERMANENT PILE BULKHEAD
Sand Dumps 25'-30' high
Sand
Granite Porphyry
Beach Slope 9°
Beach Slope 12°
N
PACIFIC OCEAN
(Waves 4'-6' Common)
Deep Water Offshore
FIXED DREDGE
SALINA CRUZ, MEX.
POSITION OF SHORELINE
SCALE IN FEET
0
50
100
200

FIGURE 2

the water offshore, with the result that some sand would enter the harbor and perhaps ultimately lead to dredging. The amount of such sand seemingly would be a factor in deciding whether a fixed dredge would be more feasible than periodic use of a floating dredge.

The fact that the sand now accumulating on the island in the harbor is only one-half as coarse as the sand on the beach west of the harbor, suggests that the sand in the harbor is derived largely from sediment transported in water a short distance offshore, where wave and current action perhaps is weaker than in the very shallow water immediately adjacent to the beach. If so, an appreciable amount of such sand might fail to come within reach of the dredge and would enter the harbor.

At any rate the stationary dredge conceived by Sr. Rolland and his associates is an inspired innovation in harbor engineering. Drifting sand is a serious problem at Salina Cruz. A large quantity of sand, perhaps 500,000 or more cubic yards a year, moves along this beach. Unless this sand is effectively prevented from entering the harbor, the maintenance of the port becomes a serious problem. If the stationary dredge does achieve this objective, it will be a rewarding engineering achievement, for which the rest of the world will heartily thank our pioneering Mexican friends for providing a new procedure for coping with the serious problems of shifting beach sand.

II. Scripps Institution of Oceanography Quarterly Progress Report No. 10, October-December 1951

SUBMARINE GEOLOGY

Survey of Mission Bay Channel

As a result of the numerous recent drownings caused by small boats capsizing on the bar at the entrance to Mission Bay, a joint survey was made of the channel on 14 December 1951 by Scripps Institution of Oceanography and the Corps of Engineers (see Figure 1).

In May 1950, an 8 foot deep channel was dredged between the Middle and North jetties connecting Mission Bay with the open ocean. Following the opening of the new channel, surveys were made by the Beach Erosion Board in June, September, and November 1950 and April 1951.

The initial channel was dredged along the center line between the two jetties. Study of the first three surveys shows a progressive deepening of the channel on the bay side, and shoaling on the seaward end of the channel, where a bar formed. The location of the channel (midway between the jetties) was little changed during this period. The April 1951 survey showed that the relatively straight channel of previous surveys had become somewhat sinuous. Also there was appreciable shoaling along the seaward end of the Middle Jetty, and a bar extended from the shoal area toward the end of the North Jetty.

Comparison of the April 1951 survey with the survey of December 1951 shows that the channel has increased in sinuosity, now having an inverted "S" shape. On the bay end of the inlet the main channel runs along the Middle Jetty while on the seaward end it is along the North Jetty (see Figure 1). Where the channel runs next to the jetties it is narrow and deep, while the portion between jetties is broad and shallow, having a silt depth of about 7½ feet below MLLW. The shoaling along the seaward end of the Middle Jetty has continued, and a 10½-foot deep bar extends across the inlet from the end of the Middle Jetty to within 100 feet of the end of the North Jetty. The bar moved 500 feet seaward between the April and December surveys. It seems probable that the capsizing of small boats in the inlet resulted from a combination of minus tides, strong ebb currents, large waves breaking over the bar, and lack of local acquaintance with breaking entrances.

Statistical Study of Currents in the Surf Zone

The statistical study of the variability and prediction of longshore currents mentioned in previous progress reports has been completed. It will receive a limited initial distribution as SIO Submarine Geology Report No. 23.

The study showed that the variability of the longshore component as measured by its standard deviation is equal to or larger than the mean velocity. In order to obtain current velocities that are representative of the beach as a whole, it is necessary to take as many measurements and at as many different stations along a beach as possible.

The momentum approach to the prediction of longshore currents by Putnam, Munk and Traylor leads to useful forecasts provided the beach friction coefficient k is permitted to vary with the longshore velocity, V. The indicated relation is $k \sim V^{3/2}$. As an aid in computing longshore currents, three alignment charts have been prepared incorporating the above relation. Two are for natural beaches, with slopes ranging up to 3 per cent, in one case, and up to $10\frac{1}{2}$ per cent in the other. The third chart is for use on model beaches with slopes ranging from 0 to $10\frac{1}{2}$ degrees and breaker heights from 0.1 to 0.5 feet. Copies of these charts are available upon request.

Marine Beaches of the United States

Further study has been made of the large suite of samples collected in a series of trips along the beaches of the United States. Figures 2 and 3 show respectively the relation of the foreshore slope to grain size and the relation of sorting to grain size. These are a compilation of all the samples. Figure 3 differentiates between the samples from the west coast, the Florida and Gulf of Mexico beaches, and New England beaches. Some of the variation from the average curve showing increase of slope with increase in grain size (figure 2) appears to be related to protection of the area from the violence of wave attack. The protected areas show steeper slopes for corresponding grain sizes. The grain size of the sands, on the other hand, shows little relation to the exposure to wave attack, but is decidedly related to the source material. The reason that many pocket and cove beaches have coarse sand is that only coarse material is available to make these sands. The typical fine sand of long beaches is in many cases due to the derivation from a larger river which transports predominantly fine sand.

Changes in Submarine Canyon Heads

Continued soundings along the accurate range lines at the heads of Scripps Canyon during the past three months have revealed an interesting series of depth changes. The canyon heads had been filling in during the previous three-month period. Observations on 11 December, directly after a series of high waves, showed continued fill, amounting to as much as 5 feet. An earthquake of moderate intensity was felt on 25 December in the San Diego area. A survey on the following day showed a deepening of as much as 7 feet in one canyon head and of 2 or 3 feet in the adjacent head. The roiled condition of the water over the canyon heads during the survey was in marked contrast to the clear water on either side, indicating that the sediment had not yet settled in the 16-hour period which intervened between the earthquake and the survey. It was estimated that

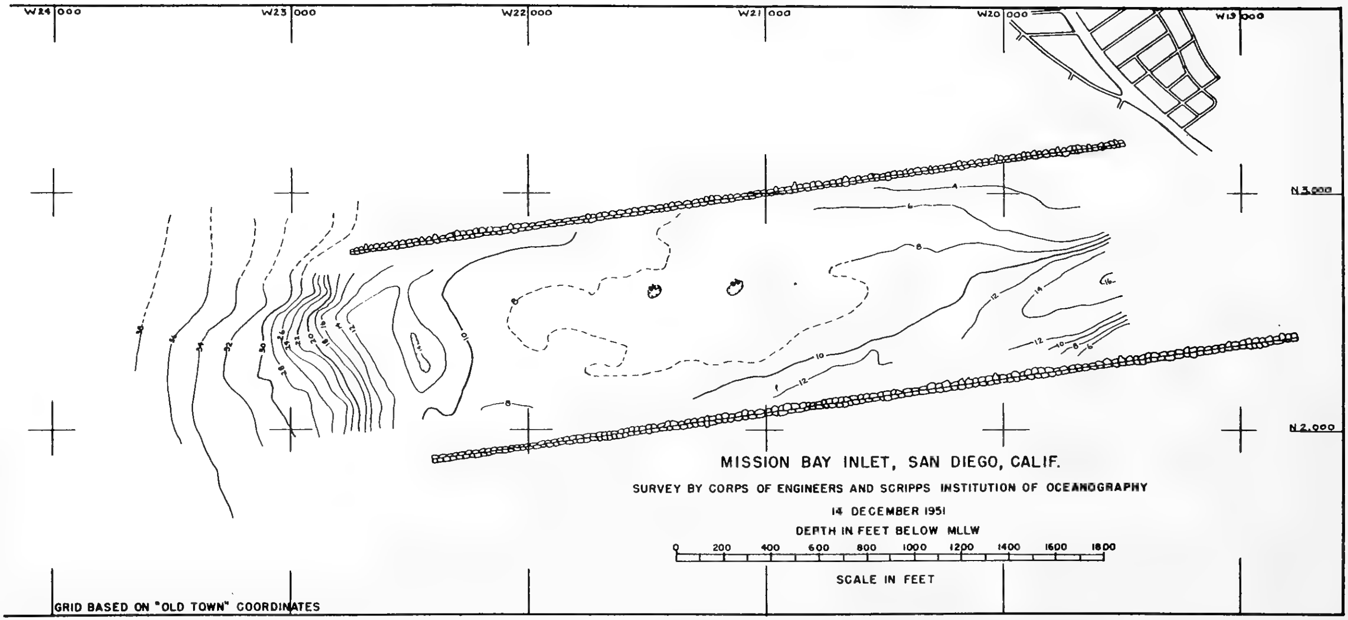
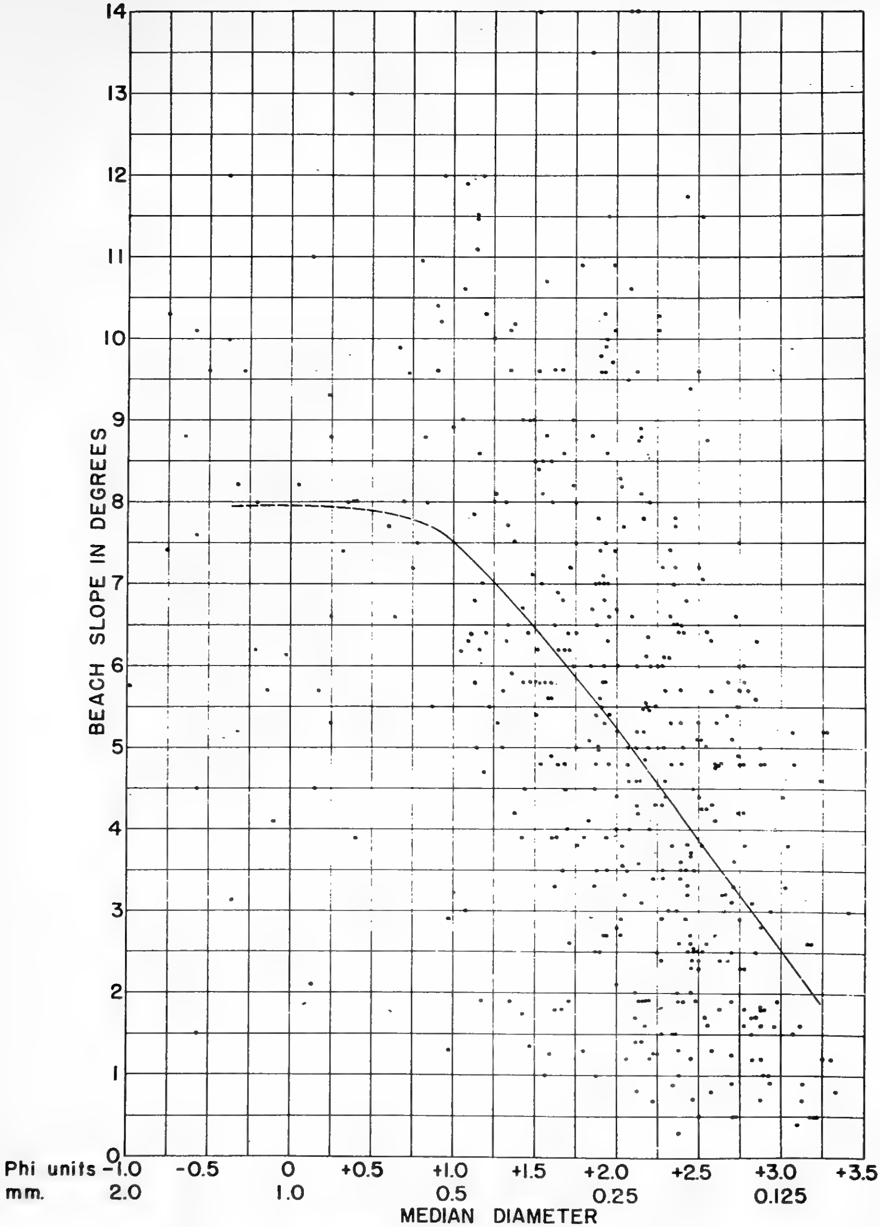
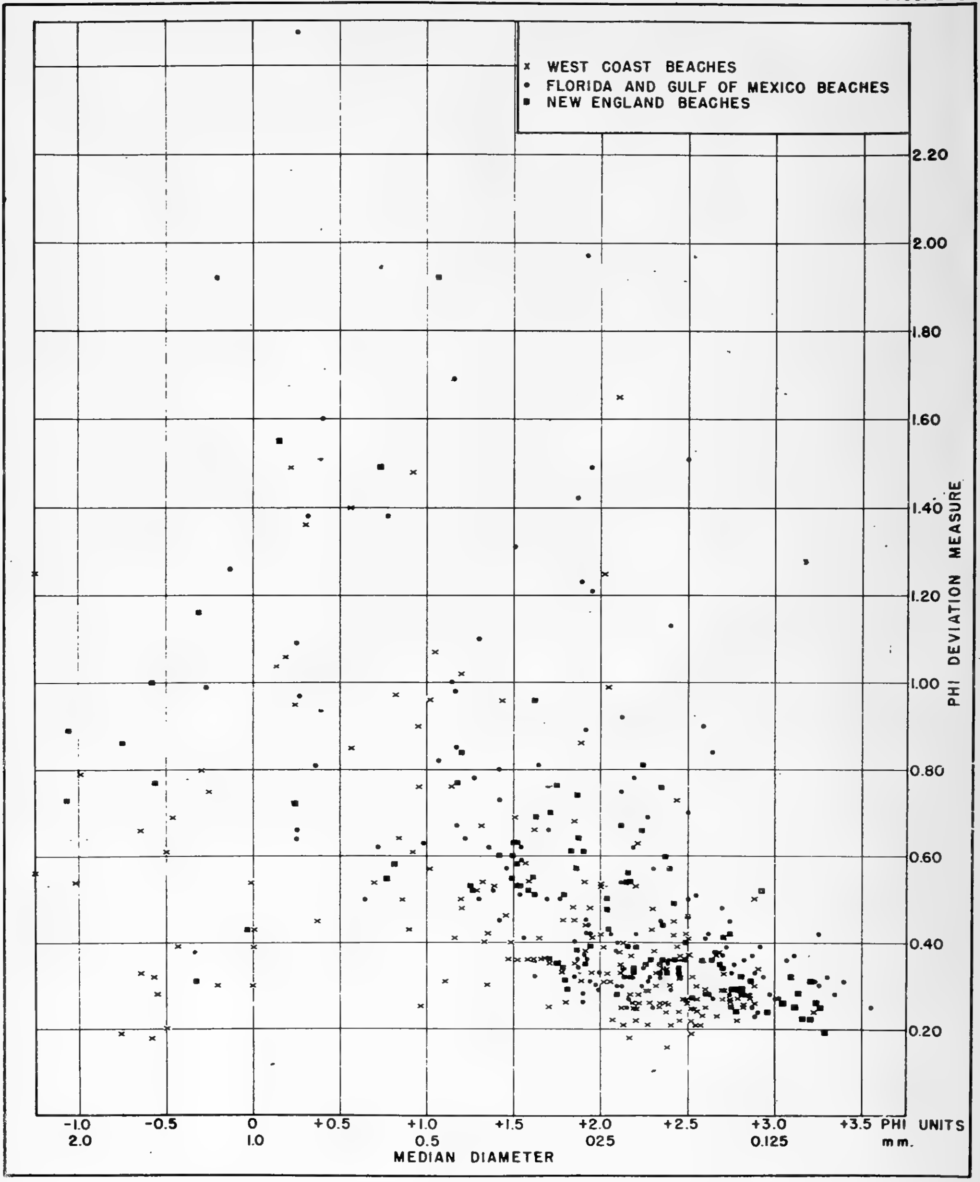

W24 000
W23 000
W22 000
W21 000
W20 000
W19 000
N 3,000
N 2,000
MISSION BAY INLET, SAN DIEGO, CALIF.
SURVEY BY CORPS OF ENGINEERS AND SCRIPPS INSTITUTION OF OCEANOGRAPHY
14 DECEMBER 1951
DEPTH IN FEET BELOW MLLW
0 200 400 600 800 1000 1200 1400 1600 1800
SCALE IN FEET
GRID BASED ON "OLD TOWN" COORDINATES

Figure 1

FIGURE 2



GRAIN SIZE COMPARED TO FORESHORE SLOPE FOR BEACH SAMPLES FROM ALL PARTS OF THE UNITED STATES



PHI DEVIATION COMPARED TO GRAIN SIZE. PHI DEVIATION IS ANALOGOUS TO STANDARD DEVIATION WITH THE MOST UNIFORM SEDIMENTS HAVING THE SMALLEST VALUES.

approximately a half million cubic feet of sand was dislodged as a result of this earthquake. An unusual occurrence was the shoaling of the ridge between the two deepened canyon heads. This shoaling had a maximum of 3 feet along one sounding line. This increase is decidedly above the possible error of surveying. The explanation for this shoaling is unknown but it is hoped in the near future to have a diving operation which may throw some light on the subject. A survey a few days after the slide had been detected, showed that the intersection, where the water is only about 15 feet deep, had filled to the extent of 2 feet. At this point the canyon depth below its surroundings was 5 feet and became 2 feet. However, at greater depths only a very slight fill was indicated during this same period. At the time of the landslide no change occurred in the valley which had been opened up, apparently by an earthquake, in 1949. This had not filled previous to the recent earthquake.

Multi-Sock Sediment Trap

Sand Movement - Observations of sand movement at different elevations above the bottom in La Jolla Bay were made using the multi-sock sediment trap described in Progress Report No. 2. As before, the trap was lowered and picked up from a DUKW. Most of the stations were made at 40- to 85-foot depths. Swimmers equipped with Aqua-lungs oriented the trap on the bottom, took photographs, and checked for sand movement, ripple marks, and bottom-dwelling organisms. Repeated observations under different wave conditions have been made now at 4- to 5-foot depth (near breaking), about 20 feet-40 feet, 60 feet, and 85 feet on fairly level sand bottom. In addition a series of four observations was made in the sloping, sand-covered, bowl-like head of South Branch of Scripps submarine canyon.

The samples collected were analyzed by the settling tube method. Occurrence of micaceous minerals was especially noted. Mica (biotite and sericite) was found to be especially abundant in sediments from the submarine canyon head. Median diameter of the sediments in the trap from the stations on gently sloping shelf show a decrease away from shore. The sands caught by the trap in the canyon head have a decidedly higher mica content than those from like depths on the open shelf, under similar wave conditions. However, samples taken in the canyon head during and just after a rainstorm have a mica content very much like normal shelf deposits. No evidence was found of abnormal sediment transport during the rain nor was any mud found in the bags.

Wave records made by the fathometer on the DUKW were analyzed whenever available to determine wave characteristics. From these values and theoretical relations, wave orbital velocities at the bottom were calculated. Where such records were not available, as, for instance, near the surf zone, orbital velocities were calculated from observed breaker heights and periods. When weight of sediment caught per hour at a given bay height above the bottom is plotted against orbital velocity computed from wave theory, the envelope of the point distribution conforms to expectations. However, no quantitative relations have been worked out as yet.

New Sediment Trap - A modified verision of the multi-sock trap has been built and preliminary calibration begun. It differs from the older model chiefly in that the frame is demountable and the legs more widely spaced. Two more traps of this new design are being constructed. Several such traps may then be placed in operation to get simultaneous observations at different water depths.

Ripple Mark Observations - Ripple marks were observed in some detail by the divers in connection with sediment movement studies. The following observations apply to fine sand bottom outside the breaker line:

1. Ripples are the result of orbital motion of waves causing a current which near the bottom is resolved into a long forward and backward horizontal movement.

2. Such a movement can be considered as two separate currents.

3. These currents do not oscillate in the same sense that small waves oscillate in a lake or pond, because with each reversal a new set of ripples is developed.

4. Therefore, the ripples observed are essentially "current" ripples rather than "oscillation" ripples.

Life Cycle of a Ripple Mark: Starting from a condition of no motion, i.e., no current and a ripple mark or roughness of bottom in existence, the initial movement causes a transfer of sediment on the crest from the steep up-current side to the down-current side as a flap. This is called the initial flap motion; its result is to alter the steep side into the gently sloping side of the ripple. As the current increases, sand is carried past the crest and an eddy is set up in the trough of the ripple. The ripple now appears to be rolling along with the current. When the current reaches its peak velocity all recognition of the ripple mark as a distinct structure is gone; the sand is moving in long horizontal streamers or as a blanket here called a "sheet flow." When the current velocity lessens, the process is reversed. First, the sand appears to be rolling, then the ripples begin to form, and the final stage is the flap motion which represents the last movement of the current. For an instant between trough and crest velocity the ripple is stationary; then the process is repeated in a reverse direction. The return current and ripple movement complete one cycle of orbital motion, i.e., one wave period.

With rather constant wave conditions, and especially with high orbital velocities, the ripple patterns on the sea floor, down to depths of 60 feet, have a very even and symmetrical appearance, although the ripples are destroyed and reformed each time a wave passes. When wave action decreases these same well-developed ripples, which sometimes have unbroken crest lengths of 20 feet, begin to decay and become irregular. Concomitantly, organisms which previously could not affect the sea floor, due to the strong currents, begin to dig up the bottom and further confuse the ripple pattern.

WAVES AND CURRENTS

Wave Refraction

A manuscript which describes a new method for the direct construction of way rays (orthogonals) has been completed in preliminary form. The problem is considered using as a starting point the differential equations which govern the ray path. The results permit a determination of the error in the approximate formula:

$$\Delta\alpha = \frac{\Delta L}{L_{av}} \tan \alpha$$

which is derived by Johnson, O'Brien, and Isaacs (Graphic Construction of Wave Refraction Diagrams, H. O. Publ. No. 605, equation (2), p. 19). Suggestions are made for improving the accuracy and ease of construction of rays.

Tsunami Recorder

Mr. Fulk has taken the place of Mr. Osborn. Some improvements in instrumentation have been carried out. During this period several heavy storms were experienced. These storms were preceded by 15-20 minute wave activity on Scripps and Oceanside recorders. A nearby earthquake, off San Clemente Island, did not cause any detectable tsunamis, even though our instruments are capable of recording amplitudes down to 0.1 inches. Some progress has been made in recording on magnetic tape moving at very slow speeds for the purpose of frequency analysis. The electric filter components of the seaborne tsunami recorder have been completed.

This research has been chiefly supported by the Office of Naval Research.

High-Frequency Wave Recorder

A very stable high-frequency, beat-frequency oscillator operating at 3 to 4 megacycles has been constructed for detecting and measuring ripples produced in the shallow water in the laboratory. It is adequate for working with waves 0.10 mm high, and tests are under way on capacitance type pickup elements. A modification of this instrument for use at sea is being planned.

Work on a ripple generator for the laboratory tank has commenced. The generator will operate over a wide range of periods from about 5 seconds to 0.01 seconds.

This work has been chiefly supported by the Air Force.

Measurements of Wind Stress

Records of water slope were obtained for a number of days, including one series of continuous records for 24 hours on 5 December, when wind speeds reached 30 mph. During that day, detergent was added on five occasions to still waves. At the higher speeds the effect of detergent is to reduce the slope (and stress) by one third. Measurements of wind speed are now made at three elevations for each five-minute period. Much of the data have been reduced. Work is in progress to reduce the temperature "noise" in the measurements, which is now the limiting factor in measuring slope at low wind speeds.

SPECIAL DEVELOPMENTS

Underwater Camera

An underwater camera, used by the Division of Submarine Geology, was rebuilt to permit increased load on gear train required for unorthodox requirements of external controls. Modifications were accomplished on the external case which included replacement of worn parts and the addition of new control glands. A flash synchronizer was added to the camera components, which require modification of the camera for accommodation and at the same time additional leads through the case for the electrical connections.

Miscellaneous Shop Work

Continued assistance was rendered in performing miscellaneous repairs to Aqua-lungs during this quarter.

PUBLICATIONS

Articles Published

Arthur, Robert S., Wave Forecasting and Hindcasting, Proc. of First Conf. on Coastal Engineering, Long Beach, California, Oct 1950, Ch. 8, SIO Wave Report No. 98, reprints distributed as SIO Reference 51-56, 15 December 1951.

Munk, Walter H., Origin and Generation of Waves, Proc. of First Conf. on Coastal Engineering, Long Beach, Calif., Oct 1950, Chapter 1. SIO Wave Report No. 99, reprints distrubuted as SIO Reference 51-57, 15 Dec 1951.

Shepard, Francis P., Sand Movement on the Shallow Intercanyon Shelf at La Jolla, California, BEB Tech. Memo. 26, Nov 1951, SIO Submarine Geology Report No. 21.

Shepard, Francis P., Transportation of Sand into Deep Water, Soc. Ec. Pal. & Min., Sp. Publ. No. 2, November 1951

Shepard, Francis P., and D. L. Inman, Nearshore Circulation, Proc. of First Conf. on Coastal Engineering, Long Beach, Calif., Oct. 1950. Chapter 5, SIO Submarine Geology Report No. 14, reprints distributed as SIO Reference 51-53, 15 December 1951.

Article Submitted for Publication

Inman, D. L., Measures for Describing the Size Distribution of Sediments, SIO Submarine Geology Report No. 15, (revised), Journ. Sed. Petr.

Mimeographed Report

Inman, D. L., Measures for Describing the Size Distribution of Sediments, SIO Submarine Geology Report No. 15, (revised), SIO Reference 51-46, 7 November 1951.

III. New York University Bi-monthly Progress Report, 4 March 1952

1. A paper entitled "A Unified Mathematical Theory for the Analysis Propagation, and Refraction of Storm Generated Ocean Surface Waves," Part 1, has been prepared. Since the paper is sponsored jointly by the Office of Naval Research and the Beach Erosion Board, requests from interested persons outside the United States should be sent to New York University. Upon approval by the Office of Naval Research, copies will then be forwarded to them.

The paper applies superior methods of wave analysis to the problem. It permits quantitative values of the power spectrum and quantitative forecasts of sea and swell properties.

Part 1 covers only the theory of wave analysis and wave forecasting. The theory of wave refraction and some examples will be given in Part 2 which is in preparation.

2. It is planned to present some of the results obtained at the spring meetings of the American Geophysical Union in Washington, D. C.

BEACH EROSION STUDIES

The principal types of beach erosion control studies of specific localities are the following:

a. Cooperative studies (authorization by the Chief of Engineers in accordance with section 2, River and Harbor Act approved 3 July 1930).

b. Preliminary examination and surveys (Congressional authorization by reference to locality by name.)

c. Reports on shore line changes which may result from improvements of the entrances at the mouths of rivers and inlets (Section 5, Public Law No. 409, 74th Congress).

d. Reports on shore protection of Federal property (authorization by the Chief of Engineers).

Of these types of studies, cooperative beach erosion studies are the type most frequently made when a community desires investigation of its particular problem. As these studies have greater general interest, information concerning studies of specific localities contained in these quarterly bulletins will be confined to cooperative studies. Information about other types of studies can be obtained upon inquiry to this office.

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

SUMMARIES OF REPORTS TRANSMITTED TO CONGRESS

PAWLEYS ISLAND, EDISTO BEACH AND HUNTING ISLAND, SOUTH CAROLINA

The areas studied are located on the ocean shore of South Carolina. Pawleys Island is in Georgetown County on the northeastern portion of the State's coast line. Edisto Beach and Hunting Island are respectively in Charleston and Beaufort Counties on either side of St. Helena Sound on the southwest shore of the State. Pawleys Island is a summer resort community with a summer population reported to be about 6,000.

It is a narrow sandy barrier beach island about 3.5 miles long between Midway and Pawleys Inlets. The entire island is privately owned. Erosion has caused recession of the shore line. High sand dunes in front of the houses have been washed away. Many of the houses have been moved to the rear of their lots and cannot be moved farther back because of the proximity of the marshes in back of the island. Edisto Beach is also a summer resort community. Its summer population is estimated at 2,000. It includes the ocean frontage of a narrow, sandy barrier beach island about 4.4 miles long between Jeremy Inlet and South Edisto River. About 1.4 miles of the shore frontage at the north end of the island are included in the Edisto Beach State Park. The remainder of the shore is privately owned. Erosion has caused recession of the shore line except at the southern end of the island which is an area of marked accretion. Many cottages were damaged or destroyed during the 1940 hurricane. The State Highway along this shore has been damaged by erosion in recent years. Hunting Island is a State Park. The island is about 0.7 mile wide and has an ocean frontage of about 4.3 miles. The beach is wide and flat, backed by a series of high sand dunes. Although the entire shore is publicly owned, building lots have been leased for the construction of private cottages, only one of which has been built. The public bath house, the only building close to the shore, is not at present seriously threatened by the recession of the shore line which has been rapid in the past few years.

The immediate sources of material reaching the problem areas are the adjacent sections of the shore north of the respective areas. The beaches are composed generally of fine to medium sand. The tides in the area are semidiurnal. The mean ranges of tide are 4.5 feet at Pawleys Island, 6.1 feet at Edisto Beach and 6.2 feet at Hunting Island. Spring ranges are 5.3, 7.2, and 7.3 feet respectively. Waves approach the shore from the north and northeast during the fall and winter and from the southeast during the spring and summer. The predominant direction of littoral drift is toward the southwest.

The district engineer, considered the desires of the cooperating agency, determined the sources and movement of beach material, the changes in the shore line and offshore bottom, the effects of winds, waves and storms, the effects of experimental groins, and developed a plan for protecting and improving the shores of the study areas. He concluded that complete systems of timber groins properly placed will arrest or retard the drifting material to the extent necessary to protect the endangered beaches at Pawleys Island and Edisto Beach, and that groins supplemented by artificial nourishment will be required to protect the beach at Hunting Island. He also found that the permanent type pile and timber groins are suitable to produce the desired results with a minimum of maintenance. He recommended that, if immediate prevention of further erosion of the localities studied is desired, 71 creosoted pile and timber groins 300 to 375 feet long and 600 feet apart be constructed, supplemented by artificial nourishment to be placed between the groins at Hunting Island. The division engineer concurred in the program of remedial measures recommended by the district engineer for accomplishment by the State of South Carolina.

The Beach Erosion Board carefully considered the reports of the district and division engineers. The comments and conclusions of the Board are contained in the following paragraphs.

At Pawleys Island, the supply of new beach material is derived from erosion of beaches to the north. The bar across Midway Inlet is indicative of passage of sand across this inlet. The material is moved alongshore by wave action. Southward migration of Midway and Pawleys inlets indicates the predominance of southward littoral drift. The recent erosion of the dunes at Pawleys Island indicates a somewhat smaller rate of supply than of loss. The overall rate of supply cannot be increased except by artificial replenishment. The four groins constructed in 1948 and 1949 have not materially widened the beach to the north but they have caused accelerated erosion downcoast therefrom. Although the groin system proposed by the district engineer may be effective to some degree in widening and stabilizing the beach at Pawleys Island, the Board was of the opinion that the proposed groins are too short and that the stability alignment with longer groins would be such that fewer groins would be required to stabilize the shore. The Board concluded that the best method of protection comprises initial construction of one groin at the south end of the developed area, extending to the 5-foot depth contour (mean low water), and extension of one existing groin near the middle of the island to the same depth contour, the latter to be deferred until its need has been demonstrated.

At Edisto Beach, as at Pawleys Island, the sources of supply of beach material are the beaches to the north. Except at the accretion area at the south end of Edisto Beach, the rate of loss exceeds the rate of supply, with resultant recession of the shore line. The material is moved southward to the area of accretion by beach drifting. The four groins completed in 1948 and 1949 have caused widening of the beach in their immediate vicinity and north thereof, but have caused accelerated erosion of the shore to the south. In order to prevent this adverse initial effect, the groin system could be filled by artificial placement of beach material in an estimated volume of 120,000 cubic yards, or the groin system could be built starting beyond the south end of the present problem area and progressing northward at a rate of not to exceed 4 groins the first year and 2 groins a year thereafter. After the groin system has been filled by natural or artificial means, the natural supply may be sufficient to maintain the stability of the shore with little or no artificial placement of additional material. Based on the behavior of the existing groins, the Board believed that groins spaced 1,200 feet apart will be satisfactory at Edisto Beach. The Board concluded that the best method of protection would require construction of 8 additional groins on a spacing of 1,200 feet and immediate artificial placement of fill in their impounding areas, that the most suitable alternative method would comprise construction of those groins immediately needed on the frontage experiencing damaging erosion and artificial placement of fill in their impounding areas, and that, if no fill is to be placed, the plan should consist of 10 additional groins spaced 1,200 feet apart, constructing 4 groins the first year and two each year thereafter.

At Hunting Island the possible sources of supply of beach material are Harbor Island to the north, and the offshore bottom. Some material crossing St. Helena Sound via the numerous shoals may reach Hunting Island, but it appears that little new material reaches the problem area. The north end of the island supplies material to the south end and erosion of the dunes supplies sand to the beach. The rate of loss exceeds the rate of supply for almost the entire ocean frontage, the greatest rate of loss being at the north end of the island where about 1.5 miles of the north end have been lost as shown by shore line changes over the period of record. The predominant direction of littoral drift is southward, as indicated by minor southward migration of Fripp Inlet. Material is moved by beach drifting, but strong tidal currents at the entrance to St. Helena Sound probably cause frequent reversals in direction. The four groins built from 1948 to 1950 have caused minor accumulation of material in their immediate vicinity without adverse effects in adjacent areas, but the evidence is not conclusive that they will hold sufficient material to result in permanent stabilization or widening of the beach; in fact, erosion of the offshore bottom above elevation -9 feet mean low water, a condition which cannot be remedied economically by the construction of groins, leads to the conclusion that groins alone will not accomplish the desired stabilization. If wider beaches are needed, or if prevention of erosion of the dunes becomes essential, material must be provided by artificial means. The Board concluded that probably the most economical method of complete protection is by artificial placement alone. The estimated material requirements, on an annual basis, are 275,000 cubic yards. However, the Board believed, that at this locality, consideration should be given to planning development and use of the shore in such manner that the detrimental effect of continued recession would be minimized.

The Board pointed out that the dunes constitute valuable protection and that they should be preserved wherever feasible. Excavation of material in connection with construction of buildings on or the cutting of paths through the dunes materially lessens their protective value and should not be permitted. Building should be permitted only in back of the dunes.

The Board noted that the groin details shown in the district engineer's report included a triple thickness of 3-inch timber sheet piling. A double thickness of sheet piling is ordinarily adequate for groin construction. Although the triple thickness would theoretically have a longer life than a double thickness of sheet piling, other parts of the structure may limit the useful life of the entire groin, so that no value will be secured from the extra thickness of sheeting. The Board was of the opinion that modification of the structural design of groins in this respect would be permissible. The Board also believed that all piling and timber in the proposed groins should be of creosoted timber or other timber equally resistant to the action of marine borers. The Board emphasized the importance of anchoring the groins into the dunes to prevent outflanking. The Board concluded that palmetto log groins of the design tested are less effective and more costly on an annual basis than treated timber sheet-pile groins.

The cooperating agency did not desire an economic analysis of plans for protection and improvement. As the Board had not studied the economic justification of protective measures for the beaches of South Carolina, it could not make recommendations for construction of such measures. However, the Board was of the opinion that: a) the public interest in the work is not sufficient to warrant Federal assistance at this time; b) adoption of Federal projects for the localities is inadvisable; and c) no share of the expense of the measures should be borne by the United States.

STATE OF OHIO-FAIRPORT TO ASHTABULA

The area studied is located in Lake and Ashtabula Counties on the south shore of Lake Erie from about 30 to 57 miles east of Cleveland, Ohio. It extends from just east of the mouth of the Grand River to just east of the mouth of Ashtabula River, a distance of about 26.5 miles. Fairport and Ashtabula Harbors, which have been improved by the United States for navigation, are located at the mouths of these rivers. Lake and Ashtabula Counties had populations of about 50,000 and 69,000 respectively in 1940. The principal centers of population are the cities of Painesville and Ashtabula which had populations in 1940 of about 12,000 and 21,000 respectively. Except for industrial development in Painesville Township, the property along the shore line of the study area has been developed mainly for private residential and recreational purposes. The population of the area is increased considerably by summer visitors. Inland areas are devoted mainly to agricultural uses. The shore is publicly owned at parks in Painesville, Perry, Madison, Geneva, Saybrook and Ashtabula Townships and in the city of Ashtabula. All are used for recreational purposes. The park beaches are generally narrow except at Walnut Park just west of Ashtabula Harbor west breakwater.

The shore line of the study area consists principally of eroding bluffs averaging about 40 feet high of clay, silt, sand and gravel fronted by narrow beaches of sand and gravel. Analysis of samples of bluff material indicated that in general approximately 25 per cent of the material is suitable for beach building. Rapid erosion of the bluffs makes available a considerable volume of beach material. West of Ashtabula Harbor a wide beach has formed by accretion caused by the harbor structures. Miscellaneous groins and seawalls have been constructed in an attempt to prevent erosion of the shore. Short groins have generally caused accretion on their west sides and have reduced recession of the bluffs to some extent. The pronounced accretion west of short groins indicate a marked eastward predominance of littoral drift.

The mean level of Lake Erie in the study area is about 2 feet above the established low water datum. The highest stage recorded and the highest monthly mean are respectively about 5 and 4 feet above that datum. The greater fetch and movement of winds from the westerly quadrant account for the predominance of eastward littoral drift. Due to the limited size of Lake Erie, local storms are the sole cause of important wave action.

The district engineer considered the desires of the cooperating agency, determined the source and movement of beach material, the changes in the shore line and offshore bottom, the effects of winds, waves, ice and storms, the effects of existing structures, and developed plans for protecting and improving Perry Township Park, Geneva Township Park, and Lake Shore Park, and four general plans for protecting privately owned shores of the study area.

The district engineer concluded that Perry Township Park and Geneva Township Park are the only publicly owned sections of the shore where protection and improvement are warranted at this time. He recommended, subject to certain conditions, that projects be adopted by the United States authorizing Federal participation in the amount of 1/3 of the first cost of groin construction at Perry Township Park and at Geneva Township Park. The four general plans for protecting privately owned shores comprise: (1) Plan C which consists of grading and draining of the bluffs, revetment of the toe of the slope, and a cellular steel sheet pile seawall; (2) Plan D, similar to Plan C except that it provides, in lieu of the seawall, maintenance of relatively narrow beaches by means of short groins; (3) Plan E, which consists of groins to retain material eroded from the bluff; (4) Plan F, which consists of revetment of the toe of the bluff. The district engineer recommended that owners of private property adopt one of the four proposed plans of protection best suited to the physical characteristics and desired utilization of their shore front property. The division engineer concurred in the conclusions and recommendations of the district engineer.

The Beach Erosion Board carefully considered the reports of the district and division engineers. It concurred generally in their views and recommendations, subject to the comments contained in the following paragraphs.

The Board noted that the district engineer presented four methods for protecting the shore of privately owned property and recommended that owners adopt the plan best suited to the physical characteristics and desired utilization of their shore front property.

The Beach Erosion Board concurred in the foregoing methods of protection, and in the manner of selection of the type best suited to each particular section of the shore, as proposed by the district engineer. The Board also emphasized the desirability of coordinated action by owners within a section to protect a stretch of frontage under the plan of protection best suited for the privately owned shores in that section, and the necessity of adequately protecting the ends of the work to prevent flanking. The Board recommended that private owners adopt one of the plans of protection proposed by the district engineer, or a plan for slope revetment or dumped riprap seawall, selecting that most suitable to the physical characteristics and desired use of their shore frontage, consistent with the effect on adjacent shore sections. As existing Federal law includes no policy for Federal assistance in the cost of protecting privately owned shores, no Federal participation in the cost of any of the foregoing work was recommended.

The Board reviewed the prospective benefits for the projects for Perry Township Park and Geneva Township Park. It noted that the value of the park land subject to erosion is low. Because of the inconsequential protective benefits, the Board considered that the need for protection is insufficient to warrant Federal aid under the provisions of Public Law 727, 79th Congress. The Board concluded that adoption of Federal projects for these parks is inadvisable but that local benefits, other than those from prevention of damages, may warrant construction of the projects at local expense substantially in accordance with the plans proposed by the district engineer. The Board considered it advisable, however, for local interests to make independent evaluations of prospective benefits from these proposed projects in determining justification for construction at local expense.

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

a. It is inadvisable for the United States to adopt projects authorizing Federal participation in the cost of protecting and improving the Lake Erie shores of Ohio within the area studied;

b. Except for recreational benefits in connection with improvment of Perry Township Park and Geneva Township Park, the public interest involved in the proposed measures is small;

c. No share of the expense should be borne by the United States.

The Board recommended that no projects be adopted by the United States at this time for the protection of the shores of Lake Erie within the area covered by the report.

STATE OF OHIO-ASHTABULA TO THE PENNSYLVANIA STATE LINE

The area studied is located in Ashtabula County on the south shore of Lake Erie from 58 to 72 miles east of Cleveland, Ohio. It lies between a point about $1\frac{1}{4}$ miles east of the mouth of the Ashtabula River and the Pennsylvania State Line, a distance of about 14 miles. Ashtabula Harbor, located just west of the study area, and Conneaut Harbor, located near the east limit of the study area, have been improved by the United States for navigation. Ashtabula County had a population of about 69,000 in 1940. The principal centers of population are the cities of Ashtabula and Conneaut which had populations of about 21,400 and 9,400 respectively. The property along the shore line of the study area has been developed mainly for private residential and recreational purposes. The principal summer colonies are in the village of North Kingsville. The population of the shore area is increased somewhat by summer visitors. Inland areas are devoted mainly to agricultural uses. The shore is publicly owned at the Conneaut Water Works, Conneaut Township Park and Lake View Park. The latter lies within Conneaut Harbor. Its beach is not suitable for bathing. Conneaut Water Works and Lake View Park are not in need of additional protection at this time. Conneaut Township Park is used for recreational purposes. It has a wide beach for about the eastern half of its frontage. The remainder of the shore within the study area is privately owned except for a short stretch along the highway at Whitman Creek, which is adequately protected at present.

The shore line of the study area consists generally of eroding bluffs 40 to 80 feet high of clay, silt, sand and gravel fronted by narrow beaches of sand and gravel. The bluffs are founded on shale which varies in elevation from about 4 feet above to 4 feet below low water datum. The bluffs are the major source of beach material in the study area. Probably no material reaches the area from west of Ashtabula Harbor and little is supplied by tributary streams. Analysis of samples of bluff material indicated that in general approximately 13 per cent of the material is suitable for beach building in the western half of the study area and 27 per cent in the eastern half. Erosion of the bluffs thus makes available some beach material in the eastern half of the study area. West of Conneaut Water Works and Conneaut Harbor relatively wide beaches have formed by accretion caused by the structures extending into the lake. Miscellaneous groins and seawalls have been constructed in an attempt to prevent erosion of the shore. Short groins have generally caused minor accretion on their west sides and have reduced recession of the bluffs to some extent. The pronounced accretion west of the harbor structures and the accretion west of short groins indicate a marked eastward predominance of littoral drift.

The mean level of Lake Erie in the study area is about 2 feet above the established low water datum. The highest stage recorded and the highest monthly mean are respectively about 5 and 4 feet above that datum. The greater fetch and movement of winds from the westerly quadrant account for the predominance of eastward littoral drift. Due to the limited size of Lake Erie, local storms are the sole cause of important wave action.

The district engineer considered the desires of the cooperating agency, determined the sources and movement of beach material, the changes in the shore line and offshore bottom, the effects of winds, waves, ice and storms, the effects of existing structures, and developed a plan for protecting and improving Conneaut Township Park and five general plans for protecting and improving the privately owned shores of the study areas. He concluded that for the western half of the study area where an adequate supply of beach material is lacking, the most economical and practical general plan of protection consists in grading and draining the bluffs and armoring the toe of the slope. Three methods of armoring were presented for use under varying bluff conditions. For the eastern half of the study where a larger supply of beach material is available, he presented two plans of protection. One comprised grading and draining the bluff, armoring the toe of the slope and maintenance of a protective beach by means of short groins. Where the bluffs contain a considerable proportion of beach material and no structures are located so close to the top of the bluff as to necessitate positive protection against any further recession of the bluff, a less costly plan using high short groins may be used. Under this plan the slope would not be armored and erosion of the bluff would be permitted to fill the groin system. The groin system would operate to retard erosion of the beach and the beach might be expected to build up to protect the toe of the bluff. The district engineer recommended that owners of private property adopt one of the five proposed plans of improvement best suited to the physical characteristics and the desired utilization of their shore front property. He further concluded that Conneaut Township Park is the only publicly owned section of the shore line where additional protection or improvement is needed at this time and that the plan best suited to the needs and resources of the township consists of one cellular steel pile groin. He recommended, subject to certain conditions, that a project be adopted by the United States authorizing Federal participation to the extent of 31 per cent of the first cost of the groin construction at Conneaut Township Park. The division engineer concurred in the conclusions and recommendations of the district engineer.

The Beach Erosion Board carefully considered the reports of the district and division engineers. It concurred generally in their views and recommendations, subject to the comments contained in the following paragraphs.

The Board noted that the district engineer presented five methods for protecting the shores of privately owned property and recommended that owners adopt the plan best suited to the physical characteristics and desired utilization of their shore front property. The Beach Erosion Board concurred in these methods of protection and in the manner of selection of the type best suited to each particular section of shore, as proposed by the district engineer. It emphasized the desirability of coordinated action by owners within a section to protect a stretch of frontage under the plan of protection best suited for the privately owned shores in that section, and the necessity of adequately protecting the ends of the work to prevent flanking.

The Board recommended that private owners adopt one of the plans of protection proposed by the district engineer, selecting that most suitable to the physical characteristics and desired use of their shore frontage, consistent with the effect on adjacent shore sections.

The Board reviewed the prospective benefits for the project for Conneaut Township Park recommended by the district and division engineers. It noted that the value of the park land subject to erosion is low. Because of the inconsequential protective benefits, the Board considered that the need for protection insufficient to warrant Federal aid under the provisions of Public Law 727, 79th Congress. Moreover, the Board noted that improvement anticipated from the proposed work in a period of 7 years would develop as a result of natural processes in a period of 25 years. The total benefits warrant consideration of the construction of the proposed project at local expense, however, the Board recommended that local interests independently evaluate the urgency of need for added recreational facilities in determining justification for undertaking the work.

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

a. It is inadvisable for the United States to adopt a project authorizing Federal participation in the cost of protecting and improving the Lake Erie shores of Ohio within the area studied;

b. Except for recreational benefits in connection with improvement of Conneaut Township Park, the public interest in the proposed work is small;

c. Noshare of the expense should be borne by the United States.

The Board recommended that no project be adopted by the United States at this time for the protection or improvement of the shores of Lake Erie within the area covered by the report.

AUTHORIZED COOPERATIVE BEACH EROSION STUDIES

NEW HAMPSHIRE

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

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

MASSACHUSETTS

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

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

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

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

NEW YORK

JONES BEACH. Cooperating Agency. Long Island State Parks Commission

Problem: To determine behavior of the shore during a 12-month cycle, including study of littoral drift, wave refraction and movement of artificial sand supply between Fire Island and Jones Inlets.

NEW JERSEY

OCEAN CITY. Cooperating Agency: City of Ocean City

Problem: To determine the causes of erosion or accretion and the effect of previously constructed groins and structures, and to recommend remedial measures to prevent further erosion and to restore the beaches.

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

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

VIRGINIA

VIRGINIA BEACH. Cooperating Agency: Town of Virginia Beach.

Problem: To determine the methods for the improvement and protection of the beach and existing concrete sea wall.

FLORIDA

PINELLAS COUNTY. Cooperating Agency: Board of County Commissioners.

Problem: To determine the best methods of preventing further recession of the gulf shore line, stabilizing the gulf shores of certain passes, and widening certain beaches within the study area.

LOUISANA

LAKE PONTCHARTRAIN. Cooperating Agency: Board of Levee Commissioners, Orleans Levee District.

Problem: To determine the best method of effecting necessary repairs to the existing sea wall and the desirability of building an artificial beach to provide protection to the wall and also to provide additional recreational beach area.

TEXAS

GALVESTON COUNTY. Cooperating Agency: County Commissioners Court of Galveston County.

Problem: To determine the best method of providing a permanent beach and the necessity for further protection or extending the seawall within the area bounded by the Galveston South Jetty and Eight Mile Road.

To determine the most practicable and economical method of preventing or retarding bank recession on the shore of Galveston Bay between April Fool Point and Kemah.

CALIFORNIA

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

Problem: To conduct a study of the problems of beach erosion and shore protection along the entire coast of California. The current study covers the Santa Cruz area.

WISCONSIN

KENOSHA. Cooperating Agency: City of Kenosha.

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

OHIO

STATE OF OHIO. Cooperating Agency: State of Ohio (Acting through the Superintendent of Public Works).

Problem: To determine the best method of preventing further erosion of and stabilizing existing beaches, of restoring and creating new beaches, and appropriate locations for the development of recreational facilities by the State along the Lake Erie shore line. Sections of the coast are being studied in order of priority as requested by the cooperating agency until the entire coast has been included.

TERRITORY OF HAWAII

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

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