DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS



# THE ANNUAL BULLETIN

OF THE

BEACH EROSION BOARD

OFFICE, CHIEF OF ENGINEERS
WASHINGTON, D.C.





VOLUME 17 JULY 1963

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# A GEOLOGICAL PROCESS-RESPONSE MODEL FOR ANALYSIS OF BEACH PHENOMENA

by

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This paper is based on material presented before the ASCE Water Resources Engineering Conference, Milwaukee, Wisconsin, May 15, 1963. As presented in Milwaukee, this was the first of two papers that considered the topic of beach engineering within the geological framework of shore processes. The second paper, by J. V. Hall, Jr., immediately follows this in publication sequence. Research on which this paper is based was supported in part by the Beach Erosion Board, Corps of Engineers, and by the Office of Naval Research under Contract Number Nonr-1228(26), Task Number NR 389-135.

## SYNOPSIS

This paper describes beach processes and deposits in terms of a conceptual process-response model that considers the processes and deposits as separate though closely related aspects of shoreline phenomena. The model provides a formal framework for analysis of natural beaches as they may be modified or controlled by the beach engineer.

## INTRODUCTION

Engineers and geologists look at beaches from somewhat different points of view. To the geologist the beach is the result of natural forces operating on natural materials within the geometric framework of a particular stretch of shore. To the engineer the beach is a buffer zone that protects private or public property along the shore from erosive action by waves, currents, and tides. The engineer is thus concerned with the stability of the beach, and even with its enlargement, in terms either of protective action, or of the use of the beach for recreational purposes. This latter aspect of beach engineering has grown rapidly in the past several decades.

The beach engineer must have a clear understanding of natural processes along shorelines in order to develop enlarged beaches or to design protective shore structures. The geologist needs to understand these same processes in order to evaluate relations among source areas of natural

beach material, movement of this material by waves and currents, and its ultimate accumulation as a beach deposit. The balance between erosion and deposition, and the natural forces that control dominance of one or the other, are also of concern to the geologist.

In his study of beaches the geologist relies mainly on field observations, complicated as they may be by multitudinous events that take place simultaneously along the shore. These include the interplay among waves, currents, and tides, and the corresponding responses shown by beach deposits under these controlling conditions. The geologist evaluates this complex situation in terms of relatively long spans of time, viewing the phenomena of a given instant as the momentary end product of a succession of events that start with the inception of a beach deposit, and end with a mature beach that represents some balanced state between erosion and deposition.

The engineer is concerned with those aspects of shore processes that control erosion and deposition at a given time and place. He seeks for principles that permit an enlargement of beaches and design of shore protective structures appropriate to relatively well-defined needs. The engineer thus supplements the largely qualitative field observations of the geologist with quantitative study of shore processes in controlled wave tank experimentation. These experiments yield much basic data on factors that control beach slopes, movement of sand by waves and currents, formation of offshore bars, and other specific aspects of shore processes.

Wave tank experimentation involves scale model theory, and requires simplification of natural shore processes to those features that can ultimately be expressed as a physico-mathematical model. Extension of these experimental findings to their natural scale, combined with field measurement of wave forces on coastal structures, furnish data necessary for installing protective structures or enlarged beaches which are themselves not destroyed or removed by the very forces that they are designed to control.

In recent decades the geologist has increasingly quantified his field observations, and the beach engineer has increasingly taken the framework of geologic observation into account in his own work. Moreover, advent of the high-speed computer has made readily available a variety of ways in which the complexly interlocked variables of beach processes and deposits can be analyzed in greater detail than was feasible by hand calculation. The way has thus been opened for more fundamental field studies on the one hand, and for more comprehensive wave tank experimentation on the other, as the variety of models applicable to the study of shore processes and deposits is enlarged.

The purpose of this paper is to examine the geological framework of beach processes and deposits in which the beach engineer operates. The approach is to set up a conceptual model of the beach deposit as a

response to waves and currents acting within the geometric form of a given shoreline. This conceptual model is first considered in terms of unmodified natural beach phenomena, and is then examined to discern those features of the model that are of particular interest to the beach engineer.

## DEVELOPMENT OF A CONCEPTUAL BEACH MODEL

Figure 1 shows a cross-section of a beach with its foreshore rising to the berm, a relatively flat backshore, and dunes in the hinterland. The beach deposit is a three-dimensional body of sediment that lies on some foundation, indicated as bedrock in the figure. There may also be some interfingering of beach and dune sand, as well as of beach sand and nearshore bottom silts, as indicated by the dashed beach limits in Figure 1.

The elements from which the conceptual beach model is constructed are shown in the cross-section of Figure 2. The hinterland is arbitrarily separated from the beach proper, and may consist of dunes, a cliff or bank, or a lagoon such as occurs behind barrier beaches. The backshore portion of the beach is composed of dry sand at its surface. This sand is shifted about by blowing winds, and under suitable conditions the blown sand accumulates as dunes piled up in part on and behind the backshore, as shown in Figure 1. Dune sand is generally finer than beach sand, and has somewhat rounder particles, as a result of selective wind transportation in terms of grain size, shape, and density. Thus the dune sand is a derivative deposit from the beach sand, and is properly considered as part of the hinterland.

The foreshore and nearshore bottom in Figure 2 represent zones of active hydraulic forces along the shore. The crest of the berm marks the limit of maximum uprush of waves during any given state of the sea. More than one berm height may be preserved, some representing seasonal or storm uprush levels. The uprush-backwash zone, essentially coincident with the foreshore, lies between the berm and the plunge point where the waves break. This zone is subjected to a succession of highly turbulent uprushing tongues of water that come momentarily to rest on the foreshore, and then return downslope as backwash less turbulent in its flow. The effect of this continuous to-and-fro motion is to sort out and arrange the foreshore material selectively according to its particle size, shape, and density; and to produce the geometrical form of the foreshore as expressed by slope, width, and height of berm.

The plunge point is a relatively narrow zone of considerable activity, in which the breaking waves generate a high degree of turbulence that carries bottom material into suspension. Part of this material is swept beachward by the turbulent surge, and part of it, usually the finer material, is moved seaward by turbulence diffusion. Normally the plunge zone is marked by a shallow trough in the nearshore bottom, composed of somewhat

coarser material than that on either side. This is a lag deposit of material large enough in particle size so that, if lifted by turbulent forces, it drops back into the trough without being carried beachward by the uprushing tongue of water, or seaward by diffusion.

The cross-section of Figure 2 is properly supplemented by the beach map of Figure 3, in which waves approach a straight shore at an angle. The refracted wave crests, the surf zone, and the momentary water edge are shown schematically. The breaking waves, with their resulting uprush surge, give rise to swash marks that indicate limits of uprush, normally on the seaward side of the berm crest. For completeness, a backshore of moderate width, with dunes in the hinterland, is included in this map.

Waves that approach at an angle to the shore develop a longshore current as indicated by the arrow in Figure 3. This current normally transports sand, and introduces a component of movement parallel to the shoreline. On the other hand, the uprush-backwash zone represents a component of movement essentially normal to the shoreline. Thus two phenomena occur simultaneously: a downbeach movement of material from left to right on the map, and a crossbeach movement of material between the plunge point and the berm. For any stretch of shore some miles in length, the downbeach movement results in a gradual but systematic change in the average size, shape, and density of the beach particles. For any one cross-section normal to the shore, the nearly continuous uprush and backwash tends to develop a pattern of relatively rapid change in beach particle properties across the foreshore. Thus, the rates of change in beach particle properties are much greater along a traverse normal to the shoreline than in a direction parallel to the shoreline.

The tide, in essence, moves the water's edge to and fro across the foreshore, allowing the uprush to extend to greater or lesser distances from the normal position of the berm. The plunge point, for a given state of the waves, also moves with the water edge, resulting in considerable reworking of the foreshore material as the tide rises and falls. The tide also contributes a component to the shore current that may act with or in opposition to the shore current generated by the waves.

Where the amount of sand moving along a shore is relatively large, the foreshore may be aggraded, with the result that the berm moves seaward. This movement of the berm produces a wider backshore, and thus adds sand to the backshore "storage bin", free from subsequent wave work as long as the beach foreshore remains stable, or continues to aggrade. The backshore sand, removed from the zone of hydraulic forces, becomes subject to the geologic work of wind. The combination of shifting wind directions, and the selective transport of materials from backshore to dunes, tends to blur any gradients or trends that may have been inherited from the hydraulic conditions operative as the berm moves seaward.

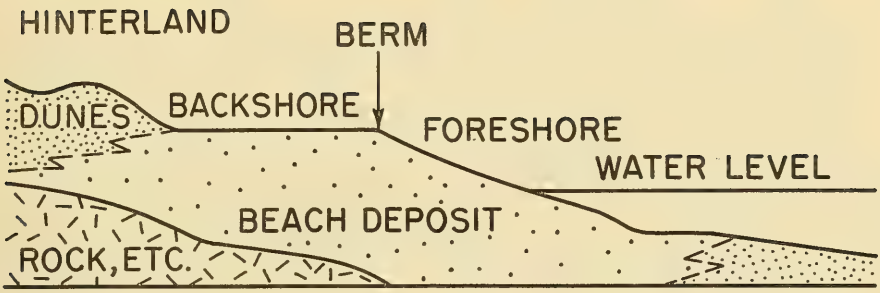


FIGURE 1. SKETCH OF BEACH CROSS-SECTION, SHOWING GENERAL GEOMETRY OF BEACH DEPOSIT

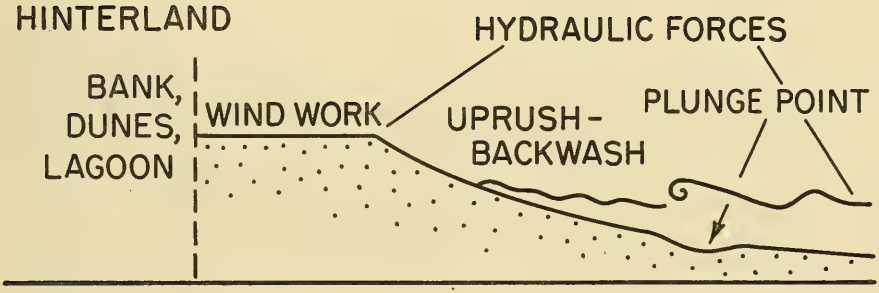


FIGURE 2. SKETCH OF BEACH CROSS-SECTION, SHOWING AREAS OF HYDRAULIC AND WIND FORCES. COMPARE WITH FIGURE 1.

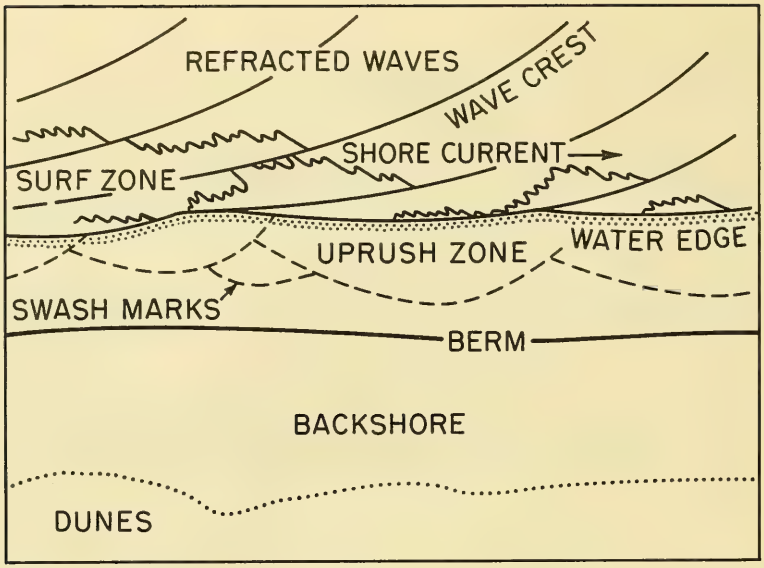


FIGURE 3. SKETCH MAP OF BEACH WITH WAVES APPROACHING AT AN ANGLE, TO ILLUSTRATE SIMULTANEOUS CROSS-BEACH AND ALONG-BEACH COMPONENTS OF MOTION

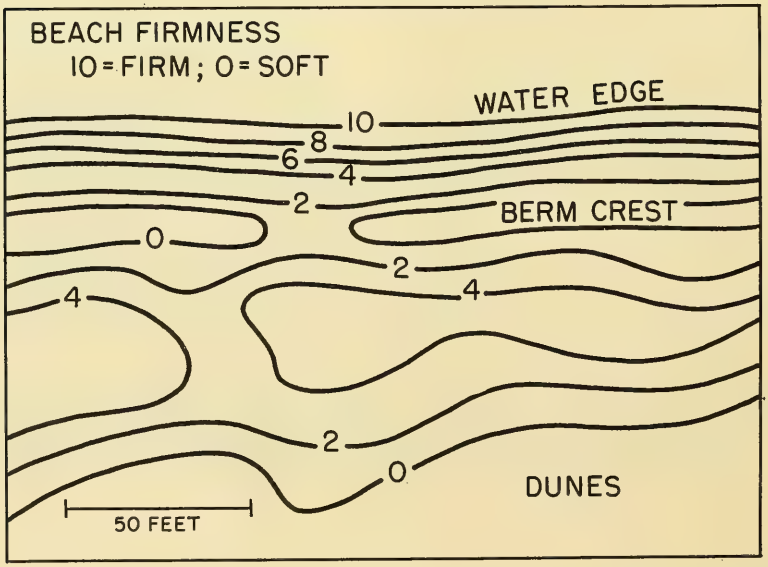


FIGURE 4. GENERALIZED CONTOUR-TYPE MAP OF BEACH FIRMNESS, SHOWING SYSTEMATIC CHANGES FROM SOFT TO FIRM SAND

It may be anticipated that if various properties of the beach are measured at a number of points on the beach, the measurements should show an areal pattern that reflects the processes taking place. Some beach properties, such as average grain size, sand firmness, and others, show fairly strong gradients between the plunge point and the berm, and a less marked but still systematic gradient on the backshore, with a transition to the hinterland dunes.

Figure 4 is a schematic map of beach firmness constructed by measuring the sand penetrability at a number of points, and drawing contours through the resulting field of numbers. The softest part of the beach is commonly along the berm crest, and firmness increases rapidly across the foreshore toward the water edge. On aggrading beaches a reversal in firmness may occur near the water line, though the situation shown in Figure 4 is not uncommon on sand beaches along the western shore of Lake Michigan. On the backshore the pattern of contours becomes less regular, though some zonation may be preserved. The dry, relatively fine wind-blown sand may show a decrease in firmness in the transition zone between backshore and dunes.

## A BEACH PROCESS-RESPONSE MODEL

Consideration of Figure 2, 3, and 4 furnishes a basis for constructing a conceptual process-response model of a natural beach. Figure 5 shows this model schematically, with the process elements on the left, and an arrow pointing to the response elements on the right.

The process elements include three major items. The first is the energy factor, expressed in terms of wave height, period, and angle of approach; by tidal range and related features; by the velocity and direction of the shore current; and by the velocity and direction of winds that operate on the backshore. The material factor on the process side includes all natural materials - pebbles, sand, silt, shells, etc. - which initially are present in the beach area and are available for movement or shifting about by the energy factors. The last element on the process side is the overall geometry of the shoreline. This represents the "boundary conditions" in which the beach processes operate. Thus, the distribution of wave energy along the shore is related to the form of the shoreline and to the nearshore bottom slope, which influence the pattern of wave refraction.

The response elements in the model include two main items as shown on the right-hand side of Figure 5. The first is the geometry of the beach deposit, which includes foreshore slope and width, height of berm, and width of backshore. In a three-dimensional sense, the geometric element also includes the volume and shape of the beach deposit as a whole.

The properties of the beach materials on the response side of the model are controlled by the kinds of material originally available at the

beach site or brought in by currents and tides. The average grain diameter of the foreshore sand, for example, is controlled by the particular combination of process elements that have recently occurred or are going on at some given time.

Although the same attributes of the shore and beach materials are listed on the process and response sides of the model in Figure 5, those on the left represent initial properties, whereas those on the right are response properties. Thus, by winnowing action on the original material, or by inflow of new materials along the shore, the response properties of the beach material may be significantly different from the initial materials.

A feature common to all elements in the conceptual model, though not explicitly stated in Figure 5, is the pattern of areal variation shown by the grain size, the angle of foreshore slope, and by other attributes, over the beach area as a whole, as was developed in Figure 4. Beach firmness, selected as an example for that map, represents a complexly interlocked response related to grain size, moisture content, and porosity or degree of packing of the grains.

Until relatively few years ago, limitations both in instrumentation and in methods of data analysis have hindered quantitative expression of conceptual beach models. As more quantitative data become available, additional implications of these models can be examined in terms of data interlock and feedback on natural beaches. As previous paragraphs indicate, close relations occur among the process elements themselves, in that the geometry of the beach site as expressed by nearshore bottom slope, influences the pattern of energy distribution on the shore. This is an example of interlock in that these two process elements are not wholly independent. A beach deposit, formed by wave and current energy, may involve changes in the configuration of the bottom slope with time. These changes in turn modify the pattern of wave approach, so that a response element may exert a feedback control on one or more process elements. These complexities are relatively common in process-response models, and a generalized feedback arrow is included in Figure 5.

Figure 6 shows a few data-interlock and feedback relationships that occur in natural beach processes. These have a strong bearing on engineering design. For example, a shore structure may produce unexpected results, if data interlock and feedback are not taken into consideration. As Figure 6 shows, wave energy exerts a controlling influence on the grain size of particles on the beach; it influences the foreshore slope and nearshore bottom slope; and (if waves approach at an angle) it controls in part the velocity and direction of the shore current. The shore current influences average grain size, and may also modify the nearshore bottom slope. Hence, the dashed arrow along the top of Figure 6, from bottom slope to wave energy, represents feedback.

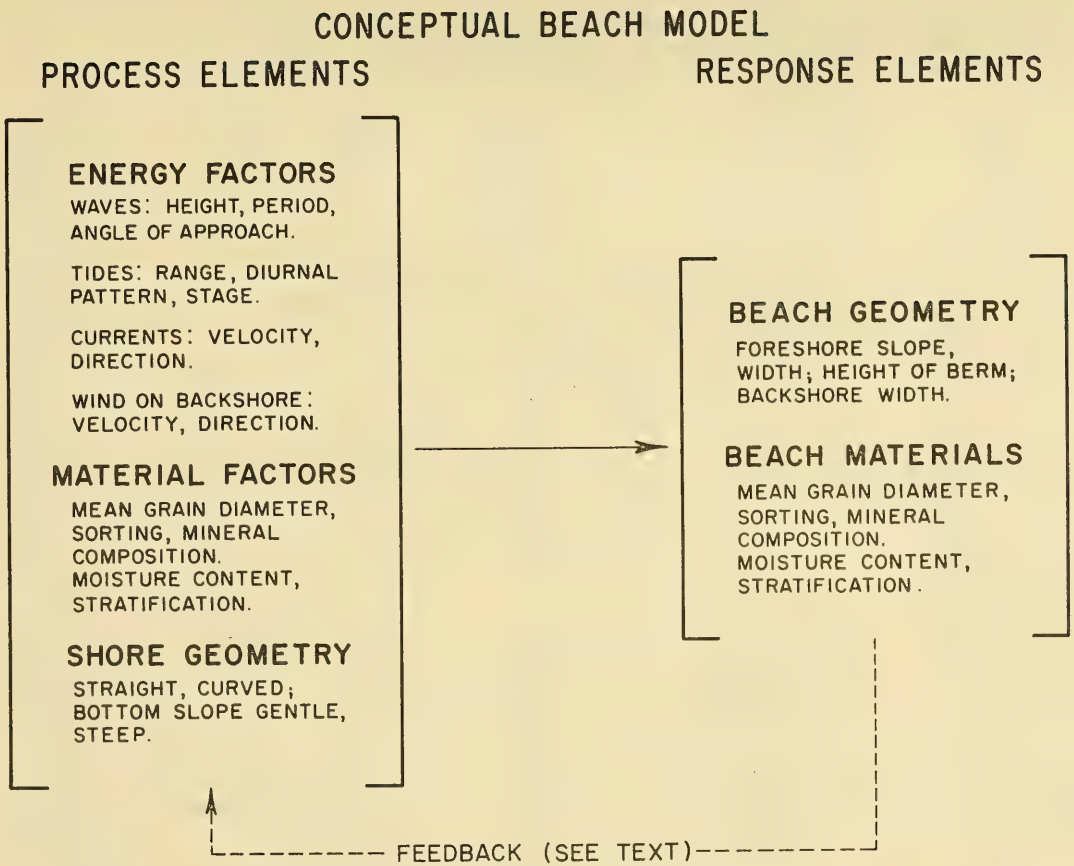


FIGURE 5. PROCESS-RESPONSE MODEL OF A BEACH

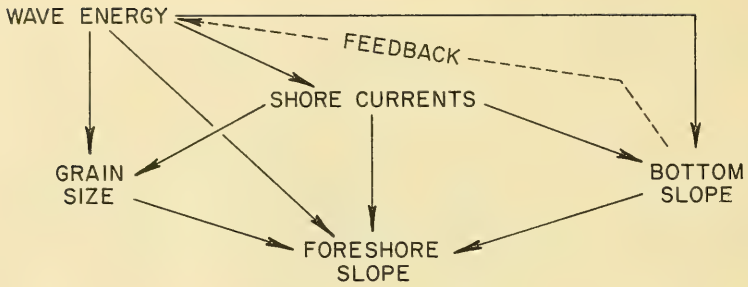


FIGURE 6. DIAGRAM ILLUSTRATING SOME INTER-RELATIONS AMONG PROCESS AND RESPONSE ELEMENTS OF THE MODEL IN FIGURE 5. SEE TEXT FOR DETAILS

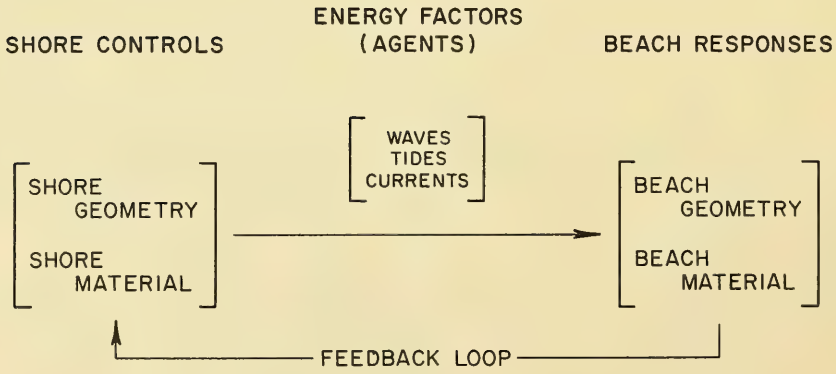


FIGURE 7. PROCESS-RESPONSE MODEL OF BEACH FORESHORE, ADAPTED FROM FIGURE 5 TO SHOW ENERGY FACTORS SEPARATE FROM GEOMETRY AND MATERIALS FACTORS

In terms of foreshore slope as a response element, the diagram indicates that this is influenced by wave energy, by shore currents, by the average size of the grains present on the foreshore, and very likely by the nearshore bottom slope. A response produced by the simultaneous effect of several factors (some of which may themselves be response elements), raises an important question in the evaluation of the relative effects played by each of the contributing factors. The engineer who replenishes a beach with imported sand needs to ask himself whether the mean grain size of the imported sand is the most important slope control in the area concerned, or whether other factors such as wave energy and shore currents are more important.

Until relatively recently, quantitative field data for analysis of these interlocking situations were not available; moreover, until the high-speed computer became generally available these complex regression problems could not be handled without a severe drain on man-hours available for hand computation.

It is evident from earlier remarks that the continued refinement of quantitative beach models involves analysis of field interrelations between process and response elements, as well as scale model experiments involving these same elements treated perhaps on a more analytical level. These interrelations can be studied in terms of a large variety of conceptual, physical, mathematical, and statistical models of beach behavior. In part the identification of an optimum model requires a search for underlying principles of data interlock and feedback, such that these may ultimately be expressed as functional relationships. The concept of energy acting upon material furnishes a basic approach, as is illustrated in the following paragraphs.

Figure 7 represents a rearrangement of elements in the process-response model of Figure 5, to indicate an alternative way of expressing the model as it applies to the foreshore, with explicit treatment of the energy factor. The model is now divided into three parts. On the left are two elements that may be called shore controls, which include the shore geometry and shore material as shown on the left-hand side of Figure 5. The right-hand block, showing the beach responses, contains the elements of foreshore beach geometry and beach material from the right-hand side of Figure 5. The central block in Figure 7 contains the energy factors - the operative geological agents - that produce the foreshore effects indicated by the main arrow pointing to the right.

In this variant of the model the energy factors - waves, tides, and currents - are those acting on the foreshore. A corresponding model for the backshore and dunes would have the geometry and materials of the backshore in the left-hand block, with a new central energy block having the term "wind work", and with a new response block on the right representing the geometric form and material composition of the hinterland dunes.

The form of the model in Figure 7 is more flexible than that in Figure 5, in that it can separate the influence of different energy sources that may operate successively on essentially the same materials. Thus, the response elements of the foreshore processes produce a set of backshore controls whose responses under wind work are sand dunes.

In all models of the form shown in Figure 7 at least one major feedback loop is present, although additional feedback loops extend from the response block to the central energy block.

Two implications of the model in Figure 7 deserve comment. The first is that the blocks can be expressed as matrices of numbers, and the model may be manipulated by the procedures of matrix algebra, conveniently treated with high-speed computers. For this sort of analysis the central matrix is a transformation matrix, as pointed out by Griffiths (1962) in his discussion of a general petrogenetic model.

A second feature of the conceptual beach model is that it permits examination of those aspects of beach processes that can be controlled by the engineer. The engineer enters from outside the natural beach model, and thus introduces an element different either from data interlock or feedback.

## SHORE PROCESSES AND THE BEACH ENGINEER

The role of the engineer may be seen by referring back to the process-response models in Figures 5 and 7. Though the engineer cannot directly control the height, period, or angle of approach of the waves, he can design structures such as offshore breakwaters to modify the pattern of energy flow to the beach, as is illustrated by Hall (1963) - Figures 13 and 15.

The engineer can exert direct control on the material factor of the process elements. This may be accomplished by introducing imported sand onto a beach in the form of stock piles or a sand fill placed along the foreshore. By this introduction of selected material, the engineer strongly influences such response elements as the foreshore slope and average grain size of the resulting modified beach deposit, as well as the width of the backshore.

Another important way in which the beach engineer can modify the response side of the beach model is by building structures that introduce special boundary conditions into the geometry of the shoreline. Among the commonest of these is the groin, built normal to the shoreline. Introduction of a groin produces an accumulation of sand (if beach drift is present) on the upbeach side. Entrapment of sand on the upbeach side of the groin lessens the amount of sand moving downbeach from the groin. During this time of downbeach impoverishment, erosion may occur immediately downbeach of the groin. If this is severe, the structure may be flanked.

Obviously, groin design is important here, as is also developed by Hall (1963).

Erosion and deposition along shorelines play an important part in the selection and effectiveness of beach engineering structures. This can be illustrated by the two diagrams in Figure 8. As mentioned earlier, aggradation along the foreshore moves the berm seaward, and adds sand to the natural "storage bin" represented by the backshore. With adequate backshore width, the bank in the upper cross-section of Figure 8 is effectively protected from wave action. Thus, in terms of the model of Figure 5, the geometric response of the beach with aggradation is such that a natural buffer zone is developed between wave uprush and the bank in the hinterland.

If shore drift is so lean that waves and currents strip material from the foreshore, the landward movement of the berm narrows the protective backshore, resulting in the loss of sand from the natural storage bin. In the extreme case shown in the lower diagram of Figure 8 erosion has continued until the bin is empty, and the foreshore extends directly to the bank. In these circumstances the bank is subject to nearly continuous wave action, and the final erosional response of beach geometry has been that no backshore remains.

The beach engineer, faced with the problem in the lower diagram of Figure 8, has several choices for remedial action. One may be to build groins along the shore, perhaps with a fill of imported sand, to establish a berm some distance from the bank. If erosion is very severe, and if costs of beach restoration are prohibitive, the engineer may recommend a seawall to protect the bank, even at the risk of losing the remaining sand. Decisions of this sort require not only judgment regarding the kinds of structures to use, but involve cost factors as well. Thus, the beach engineer, in taking the conceptual beach model into account, superimposes upon it a cost factor, and accordingly introduces man-made components into the model. By introduction of the economic factor and other restraints, the conceptual model becomes adaptable to various kinds of operational research studies.

## CONCLUDING REMARKS

The treatment of shore processes in this paper is expository in terms of a conceptual beach model, and it does not include information on rates of erosion, quantities of shore drift, or related items. Much data are available on these quantitative aspects, and for applied use of the conceptual model such information is used to convert the expository treatment into a more formal quantitative treatment for analysis of data matrices, as was mentioned in connection with Figure 7.

There seems little doubt that improvements in instrumentation and better understanding of interrelations between process and response elements have made it possible to design field experiments on beaches

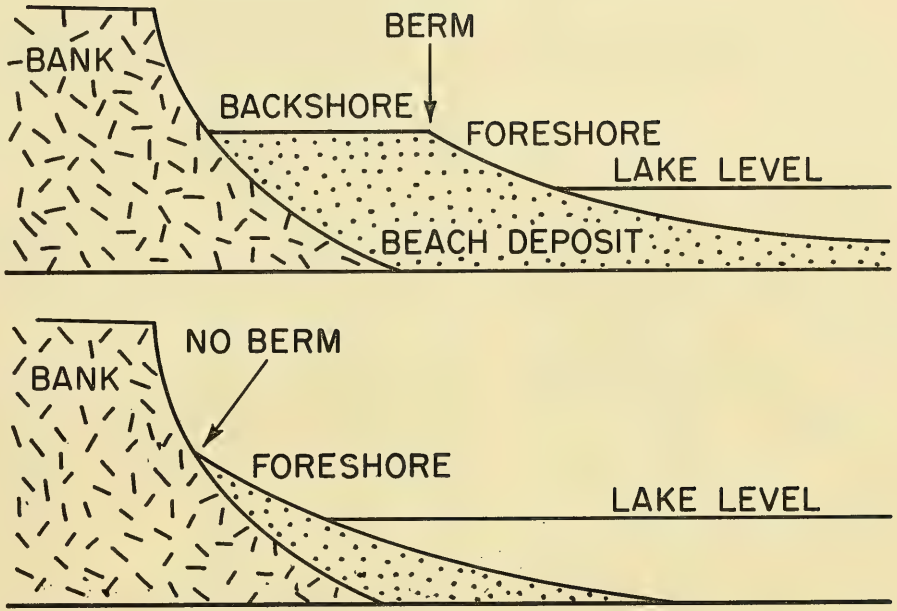


FIGURE 8. UPPER CROSS-SECTION SHOWS HINTERLAND BANK PROTECTED BY WIDE BACKSHORE; LOWER SECTION SHOWS EXTREME EROSIVE CONDITION WITH NO BACKSHORE REMAINING

that parallel the kinds of experiments carried on in wave tanks. In both cases it is possible to work with many more variables than has formerly been the rule. As long as computational facilities are limited to those of a desk calculator, the treatment of multivariate problems is very tedious, and indeed is largely impractical. The advent of the high-speed computer has changed this picture markedly.

During this decade both the geologist and the engineer can gain greater insight into beach processes and responses than has been possible heretofore. The more closely that experimentation and field work are coordinated, the greater will be the opportunity for discerning those aspects of beach process and response that are of basic importance in the design of protective shore structures.

The concept of a beach model, in the degree that it provides a framework for this closer understanding, can become the point of entry for development of an enlarged and in some respects a more fundamental approach to the problem of protecting shorelines from severe erosion.

## SELECTED REFERENCES

Three of the following papers are pertinent to geological models, and they are offered as an introduction to the growing literature on the subject.

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# COASTAL ENGINEERING STRUCTURES

by

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## SYNOPSIS

This paper describes the physical characteristics of basic coastal engineering structures in general use; the behavior of individual structures and their behavior when grouped as a system. Also described is a typical example of planning for coastal engineering works.

## INTRODUCTION

The foregoing paper by Krumbein has covered beach processes and deposits in terms of a conceptual process-response model. This paper parallels Krumbein's presentation with emphasis on how these natural forces are controlled or modified to suit man's purposes by means of engineering structures.

During the early years of this nation, shore protection was a relatively unimportant problem since the shores were sparsely settled. As our economy developed and the value of shore front property increased a line was established beyond which for economic reasons the shoreline could not be permitted to erode. The science of shore protection, like others, developed first as an art as the need arose and continued on that plane until research and development placed it on a scientific basis. Although some work has been done throughout the years, the first major concerted effort to increase the knowledge started shortly after World War I. The next great advance was made in World War II, due to its amphibious nature.

The science of shore protection covers numerous fields of endeavor in which geology has and is playing an important role in developing a fuller understanding of beach processes, so essential to the solution of shore protection problems. In other words, it is the accumulation of this knowledge that has enabled the engineer to design structures, which modify or control the natural forces to suit man's purposes.

Numerous types of structures have been developed, each having its mission in shore protection work. These are the working tools of the engineer and he employs them after a complete engineering study to provide protection at the least cost.

## BASIC TYPES OF STRUCTURES

Numerous types of structures are seen in the coastal areas but all are variations of a few basic types of which the seawall is possibly the oldest type used today. It is defined as a structure separating land and water areas, primarily designed to prevent erosion of the backshore and other damage due to wave action. Seawalls are generally massive and expensive, and are used only where economics dictate their use. Most seawalls are built of masonry or reinforced concrete (Figure 1). They are normally designed as gravity structures to resist a frontal attack by the waves. Their dimensions, height and length, are generally dictated by the natural features of the area to be protected. In some instances the top is set at a superelevation to prevent overtopping and flooding of inland areas due to storm wave run-up on a superelevated tide. Except in special cases the seawall is rarely recommended for use since more economical means of protection have been devised.

A bulkhead is defined as a structure which separates land and water areas, primarily designed to resist earth pressures (Figure 2). By definition it is not designed for dissipating or absorbing high wave energy. It is generally built high on the beach as a delimiting boundary between the shore area and the developed upland. Like a seawall the dimensions of the bulkhead are determined by the natural features of the area to be protected. In some instances, the bulkhead, as a secondary line of defense to the beach, is superelevated to prevent overtopping and flooding during storm conditions. However, if the beach erodes, the waterline approaches the bulkhead with the result that the bulkhead must act as a seawall.

Revetments are structures related to seawalls or bulkheads. They are defined as a facing of stone or concrete shapes built to protect a scarp, embankment, or shore structure against erosion or damage by wave action or currents (Figure 3). Revetments have evolved from the simple rubble facing to the more esthetic structure built of interlocking blocks. The structure in a sense replaces the bulkhead and its height is designed in a manner similar to that of the bulkhead, but due to its ability to absorb wave energy, a lesser run-up is experienced, thus resulting in a lower design height. The cover or armor stone of the rubble type revetment is designed to be stable under prevailing wave action.

Unlike the preceding structures, which serve essentially as barriers between land and sea, other structures have been developed and are used to furnish protection by modifying the natural regimen of the area. A jetty is classed as a structure extending from the shore into a body of water and designed to prevent shoaling of a channel by littoral materials, and to direct and confine the stream or tidal flow (Figure 4). It is usually built with a trapezoidal section of heavy rubble designed to be stable when subjected to wave forces extant in the area. The structure may vary in length from a few hundred to several thousand feet, depending of the distance from the shore to a water depth equivalent to the depth of channel

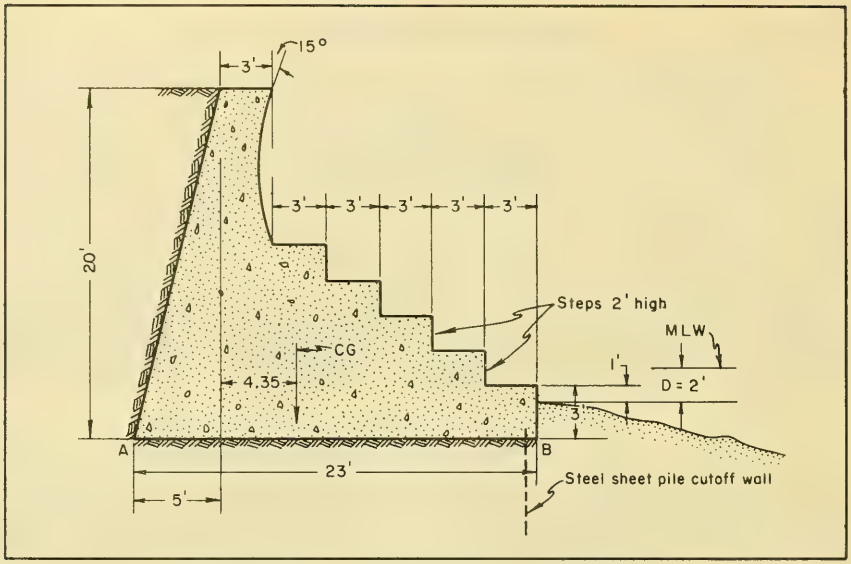



FIGURE 1. CONCRETE SEAWALL

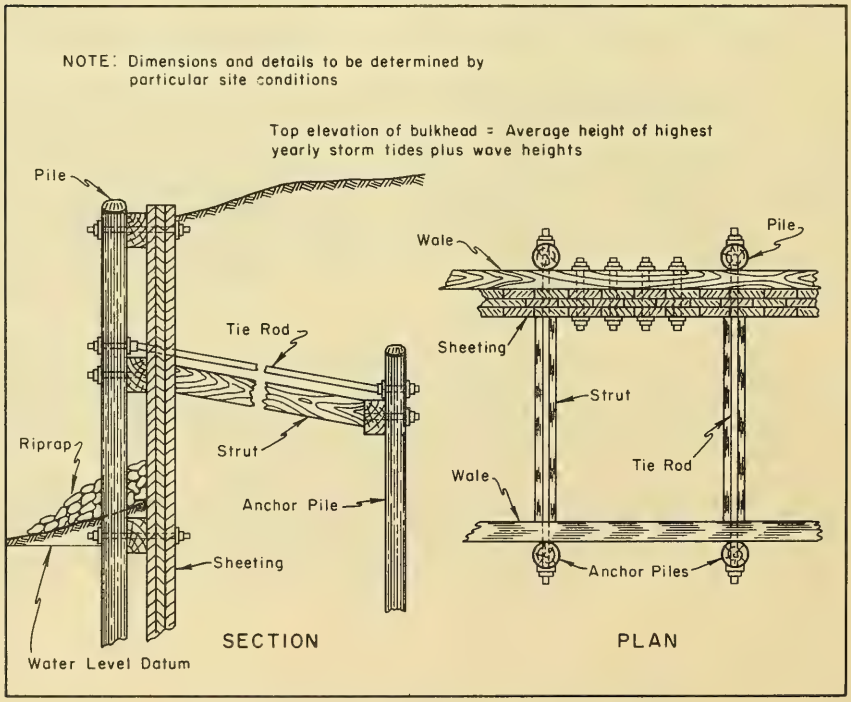



FIGURE 2. TIMBER BULKHEAD

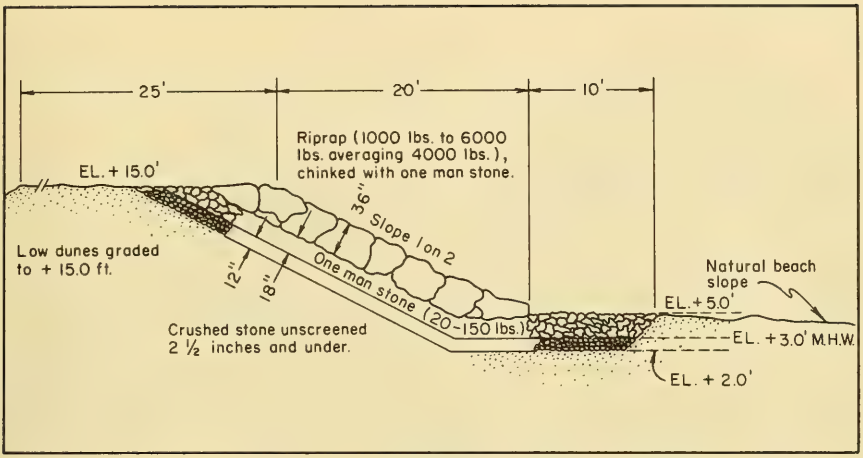
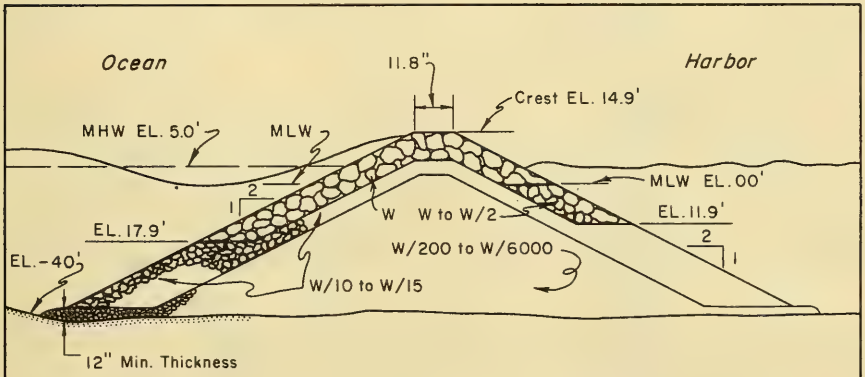



FIGURE 3. RUBBLE REVETMENT




| Stone | Weight | Total Layer Thickness |
|---|---|---|
| W | 4.7 tons plus | 7.8 ft. |
| W to W/2 | 2. to 5 tons | 7.8 ft. |
| W/10 to W/15 | 600 to 2000 lbs. | 4.0 ft. (min.) (larger stone on outer surface) |
| W/200 to W/6000 | 600 lbs. to chips | — (larger stone on outer surface) |

FIGURE 4. RUBBLE MOUND JETTY OR BREAKWATER

to be maintained. A jetty is normally a high structure to insure that sand will not be carried over it into the navigation channel.

Possibly the most common shore protection structure is the groin. It is built usually perpendicular to the shoreline to trap littoral drift or to retard erosion of the shore (Figure 5). It is narrow in width (measured parallel to the shoreline) and its length may vary from less than one hundred to several hundred feet (extending from a point landward of the shoreline out into the water). It is constructed of timber, concrete, steel or rubble. Groins may be classified as permeable or impermeable. An impermeable groin is a solid or nearly solid structure, whereas a permeable groin has openings through it of sufficient size to permit passage of appreciable quantities of littoral drift. The top profile of groins generally parallels the slope of the beach out to about 1 foot above MLW, and thence is horizontal to the outer end. An impermeable groin is normally designed with low profile to build a beach by impounding the drift to its top elevation, and then permitting the drift to pass over its top to nourish the downdrift beaches. A high groin is normally used only as a barrier at the ends of littoral zones, such as limiting groins of a bayhead beach or at a harbor entrance.

Breakwaters may be either shore connected or detached (offshore). Both are used to modify the littoral processes as well as provide shelter for shipping.

The shore-connected breakwater usually leaves the shore as a jetty but then changes alignment to roughly parallel to shore. It is generally built of heavy rubble, and designed to be stable under attack of storm waves extant in the area. Other construction materials are also used for breakwater construction, such as precast concrete shapes and, when sited on hard bottom, concrete, timber or steel caissons loaded with ballast. The length of the structure is dependent upon the required harbor area to be protected. Shore-connected breakwaters may be used singly or in pairs to totally enclose a harbor area.

An offshore breakwater is normally set parallel to the shore. It is generally built with a trapezoidal section of heavy rubble similar to the jetty, and designed to be stable when attacked by the waves extant in the area. The length of the structure is approximately equivalent to the distance the structure is sited from the shore. From an engineering point of view, the length is determined by the wave diffraction patterns around the ends of the structure. In other words, the length of the structure is so designed that a calm area of required size is created shoreward opposite the center of the structure.

Undoubtedly the most widely used shore protection device at present, is the artificially constructed and nourished beach. It is included here

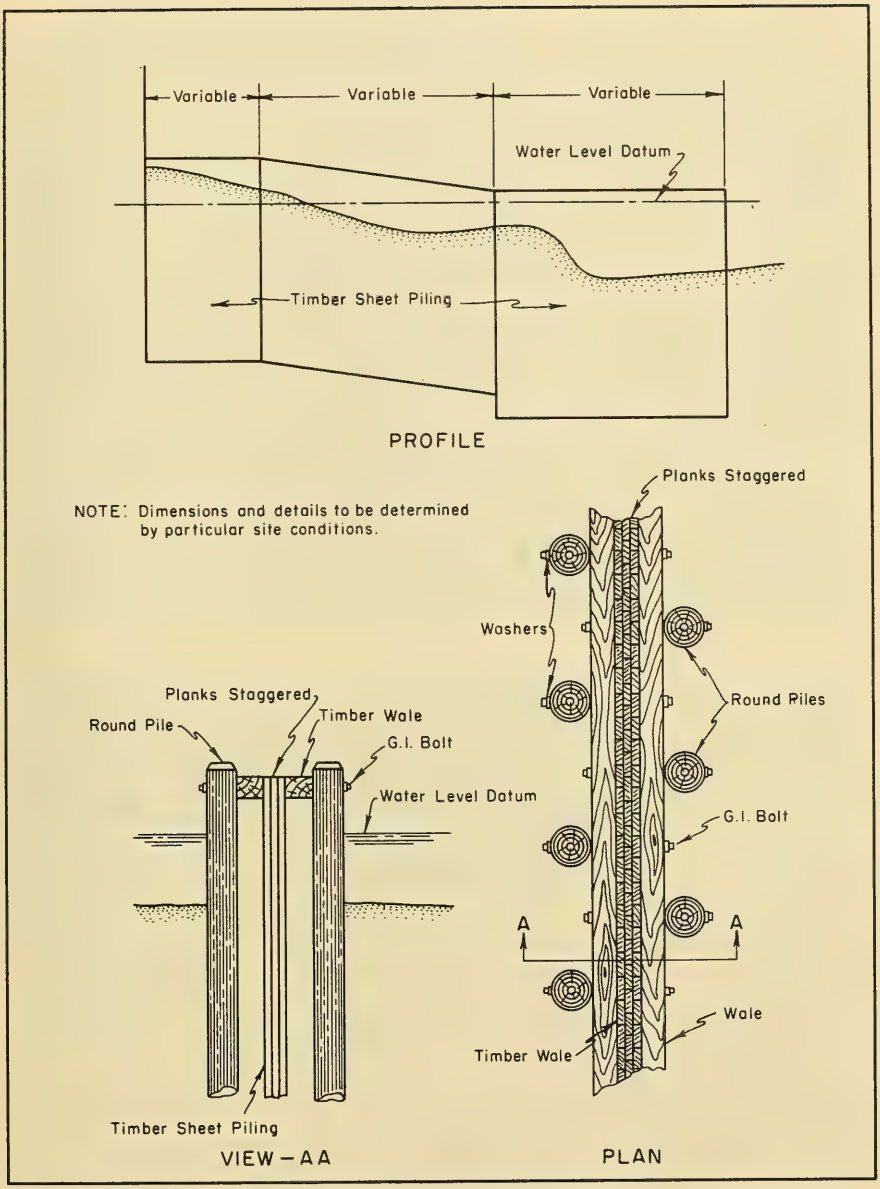



FIGURE 5. TIMBER GROIN

as a structure because it is built by man. It has long been recognized that a beach of proper proportions, including an adequate berm backed by dunes, is man's and nature's best solution to the erosion problem. Where possible, an eroded natural beach is rebuilt to the exact characteristics of the original beach. In other words, the borrow material is selected to conform as nearly as possible with material comprising the natural beach. The dimensions of the beach, such as slope, height, and breadth of berm are adhered to as closely as possible where similar beach materials are available. Where the characteristics of the borrow and native beach materials are not the same, the constructed beach is placed to such dimensions that the material available will be stable under prevailing coastal processes as is illustrated by Krumbein's Figures 5 and 7 (foregoing paper).

The widespread use of constructed beaches has to a large degree been brought about by the increasing construction costs of other types of structures and the need of beaches for recreational use. The beach at Harrison County, Mississippi, illustrated in Figure 6, is a classic example of this type of structure.

## BEHAVIOR OF INDIVIDUAL STRUCTURES

The structures enumerated all have specific functions to perform and all react differently in use. The mission of the seawall is to protect the upland area and is not expected to protect and preserve the beach (Figure 7). It is a massive structure designed to take the full impact of the wave and is not designed to modify the littoral forces in the area to preserve or enlarge the beach. When a wave breaks on a vertical seawall the energy which throws water high into the air also permits it to move downward with an equal force, thus eroding the sand at the toe of the wall and carrying it seaward. Erosion usually continues until the bottom material ceases to be affected by the downward velocities. A great deal of effort has been expended in seawall modification to reduce this erosive feature associated with it. Modification has helped to some extent, but a seawall should never be used alone as protection if the retention of the beach is desirable.

Bulkheads behave in the same manner as seawalls but are not used directly as a bulwark against the sea (Figure 8). They are usually placed well back of the waterline to retain low bluffs or dunes and to serve as a secondary line of defense during storms - the first line of defense is the beach. However, if the beach erodes to a point where the wave strikes the structure the bulkhead becomes a seawall and rapid toe erosion normally takes place. Since the bulkhead is not designed as a bulwark against the sea, the structure generally fails by falling seaward after damaging scour or erosion at the base of the structure has occurred.

A revetment is an alternative to seawalls and bulkheads. Like seawalls and bulkheads, revetments protect the shore area from direct attack but they do not modify the littoral regimen of the area to protect or preserve the beach (Figure 9). Revetments are generally sloping permeable

FIGURE 6. BEFORE AND AFTER BEACH FILL - HARRISON COUNTY, MISSISSIPPI

FIGURE 7. CONCRETE SEAWALL - WINTROP BEACH, MASSACHUSETTS

FIGURE 8. TIMBER BULKHEAD AND GROINS - LONG BRANCH, NEW JERSEY

FIGURE 9. STONE REVETMENT - FORT STORY, VIRGINIA

structures constructed of rubble stone or concrete shapes. When a wave impinges on the structure, its energy is absorbed in the interstices of the surface units, resulting in a greatly reduced run-up and backwash. The downward eroding effect of vertical seawalls and bulkheads is greatly reduced by use of this type of structure.

It is to be emphasized that the above-mentioned structures are used only to establish a delimiting line between land and sea, and they do not modify the littoral regimen either to preserve or build a beach. Consequently material transported in the littoral zone is not obstructed, and so moves past the seawall, bulkhead, or revetment.

Jetties are used at harbor entrances to maintain the channel through the littoral zone (Figure 10). In addition to providing shelter to the channel from incident waves they also provide protection by preventing sand carried alongshore as littoral drift from entering the channel. In order to serve this latter function they must be impermeable and high. Since they are high they present a complete obstruction to the littoral drift, and thus deprive the downdrift shore of sand supply. This undesirable feature has long been recognized, but only in the last decade has it been possible to develop a satisfactory means of transporting the impounded material downdrift to re-establish the natural processes. At the present time systems have been devised whereby material is moved across harbor entrances by a fixed pumping plant mounted on the updrift jetty. This system, while helpful, has not been shown to be entirely satisfactory. Another type now being used will be discussed in subsequent paragraphs.

A groin is the most important shore structure for the enlargement or stabilization of beaches (Figure 11). It is almost always built perpendicular to the shore to impound the available littoral drift or to reduce the rate of loss of sand from the area by littoral forces. This structure has been developed in a variety of forms. It may be long or short; high or low; and permeable or impermeable. The length of an impermeable groin is usually determined by measuring the distance from the inner edge of the berm to the 6-foot depth, on the usual assumption that most of the material movement along the beach takes place shoreward of the 6-foot depth. The height of the groin depends on its function. If the structure is designed to impound all available drift except that which moves around its outer end, it is built high. It is pointed out, however, that complete obstruction of the drift creates serious erosion downdrift and may cause flanking of the structure on the inner end of the downdrift side.

If a groin is required to hold or build a beach at its original profile, a low groin is used, with its top coinciding with the existing or desired beach profile. The advantage of this type of groin is that when full, it permits material to pass over its top and nourish downdrift beaches, thus eliminating shore starvation and possible flanking at the inner end.

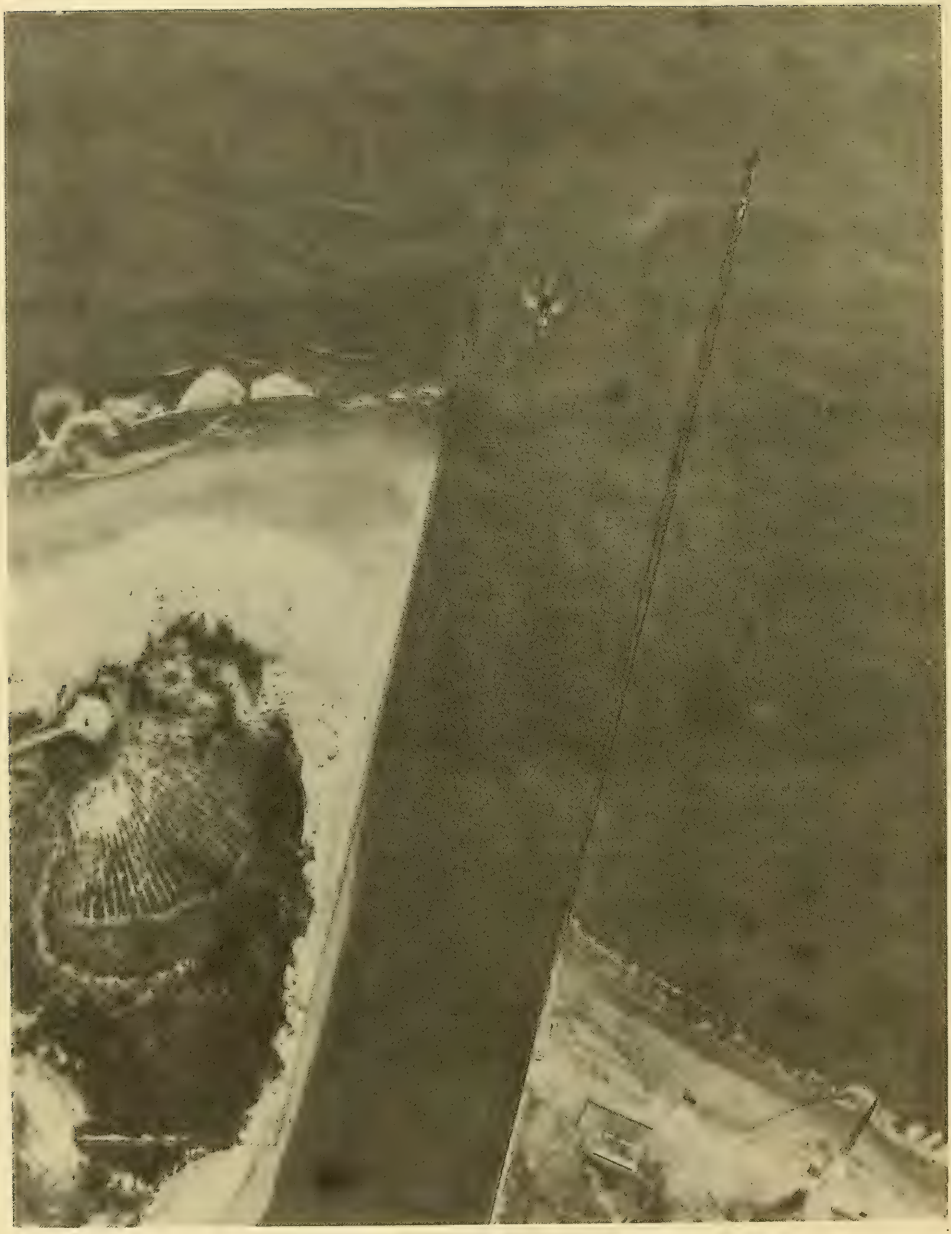


FIGURE 10. RUBBLE MOUND JETTIES-COLD SPRING INLET, NEW JERSEY

FIGURE 11. GROIN SYSTEM - WILLOUGHBY SPIT, VIRGINIA

Permeable groins are usually built high, the top serving as a recreation facility such as a fishing platform or sun deck. This kind of groin has openings through it from top to bottom over its entire length to permit beach material to pass through. The effectiveness of this structure compared to the low impermeable groin has not been fully demonstrated to the satisfaction of most coastal engineers, and it is interesting to note that the Beach Erosion Board has never recommended construction of permeable groins in its 33-year history.

Breakwaters both shore-connected and detached (offshore) are used to provide shelter for anchorages and harbor entrances.

The inner end of the shore-connected breakwater acts in the same manner as a jetty (Figure 12). It impounds the littoral material on the updrift side and thus excludes these materials from the harbor area. A structure of this nature, high and sand-tight, is a total littoral barrier, and its construction will result in accretion updrift and erosion downdrift. Eventually the impounded material will move along the seaward face of the structure and accumulate in the harbor area. Through harbor maintenance by pipeline dredge the material can be spoiled downdrift to nourish the eroding shore, effecting a sand-bypassing system. The seaward portion of the structure or that part roughly paralleling the shore acts essentially as an offshore breakwater absorbing the wave energy to create a calm anchorage or harbor area in its lee.

Offshore breakwaters have been designed more recently for the dual purposes of providing shelter to a harbor entrance and to provide a protected littoral reservoir from which sand-bypassing operations can be conducted (Figure 13). In this application the offshore breakwater is installed seaward and updrift of the harbor entrance to intercept wave action which would normally strike the shore in the area. This arrangement provides a relatively calm approach for ships entering the harbor. By intercepting the wave action, it also results in arresting the flow of littoral material which impounds in the form of a bulge on the shore opposite the center of the structure. Thus it can be seen that this structural arrangement improves conditions by stilling the sea and preventing littoral material from moving into the navigation channel.

It has long been recognized that littoral barriers are detrimental to the downdrift shores and that where they exist some sort of sand-bypassing system should be used. The floating hydraulic pipeline dredge is ideal equipment for moving sand from one location to another, but it cannot be safely used near shore in an exposed location. In other words it would be impracticable to dredge material impounded by a jetty. Dredging behind an offshore breakwater is another matter. The shelter afforded by such a structure is conducive to bypassing by pipeline or hopper dredge. It is considered that this type of bypassing system is superior to others now in service.

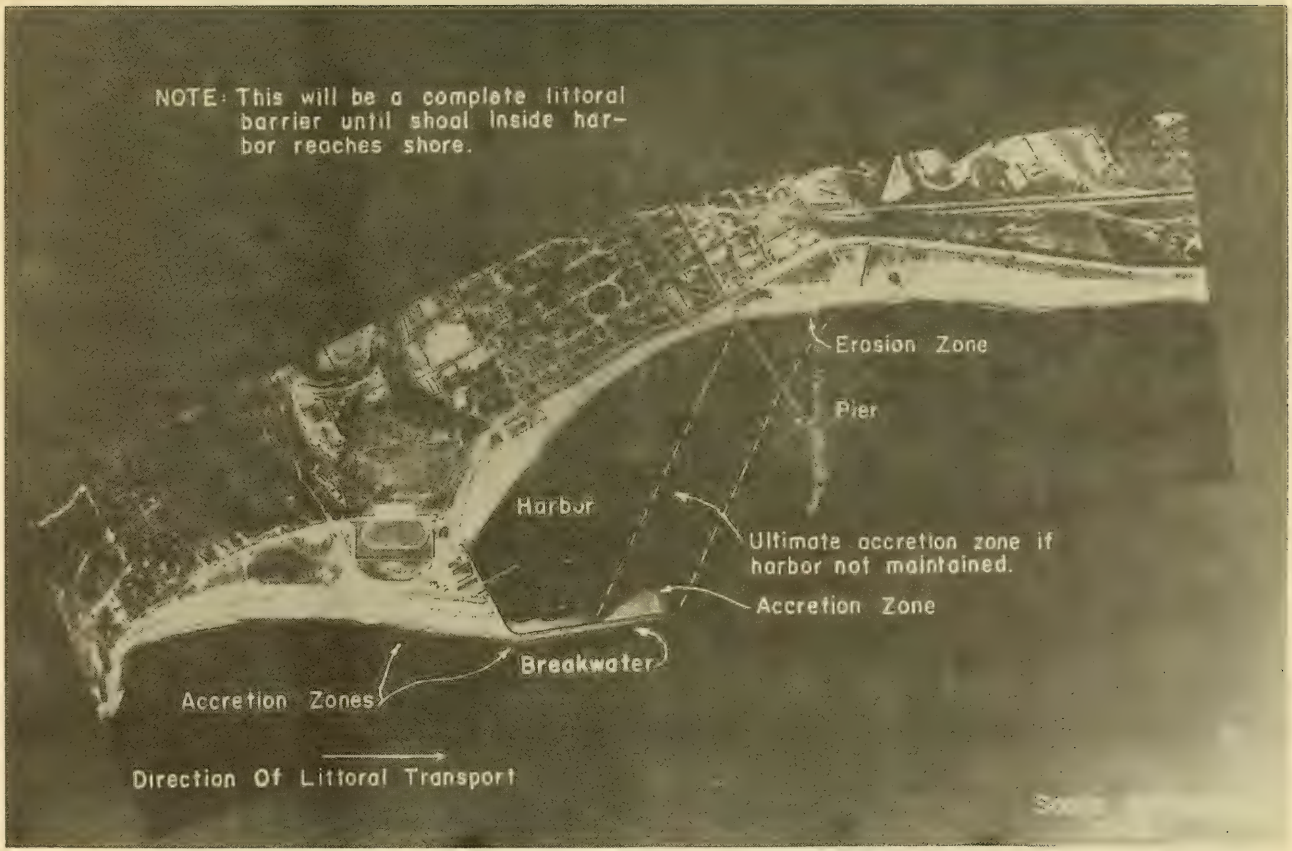



FIGURE 12. SHORE CONNECTED BREAKWATER - SANTA BARBARA, CALIFORNIA

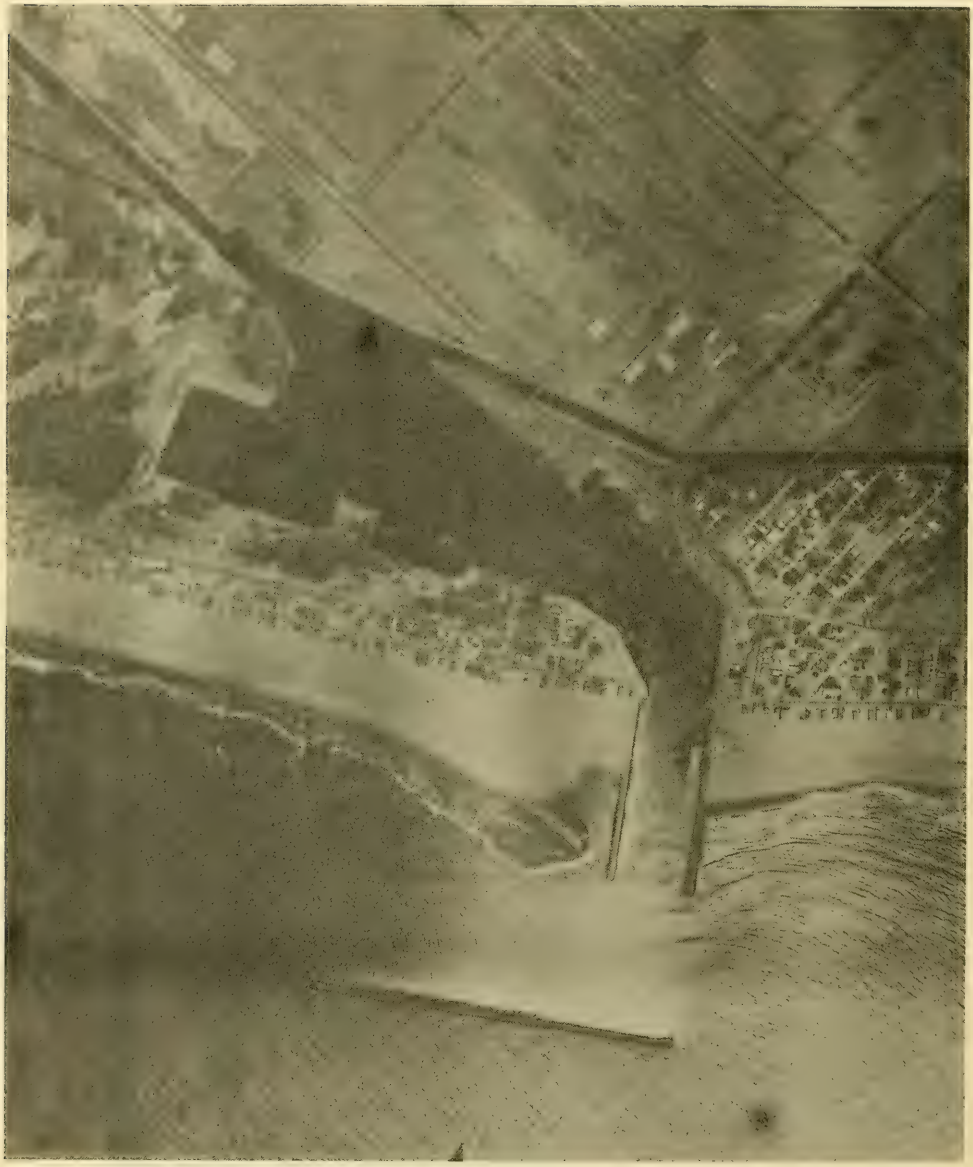


FIGURE 13. OFFSHORE BREAKWATER - VENTURA COUNTY HARBOR, CALIFORNIA

FIGURE 14. BEFORE AND AFTER BEACH FILL - OCEAN CITY, NEW JERSEY

Artificially nourished and constructed beaches are used extensively at present to restore required conditions suitable for recreational areas (Figure 14). The installation of this type of structure involves replacing a beach using the criteria derived from the natural beach. That is, the grain size of the material and its gradation; beach slopes (both foreshore and backshore); and elevation of berm are adhered to as strictly as possible. In the event that a different beach material, such as sand of different grain size, need be used, the reconstructed beach will possess different characteristics even when nourishment is supplied at a rate equal to past natural losses. This type of constructed beach, however, behaves as did the natural beach with respect to adjacent shores and is normally beneficial to the downdrift shores.

## BEHAVIOR OF SYSTEMS OF STRUCTURES

Structures used singly or in systems have a marked effect on the land forms in the immediate and contiguous areas through accretion and erosion. Groin fields are designed to build and hold a beach usually of appreciable size. Numerous factors must be considered in design, namely, the magnitude and direction of littoral drift and the resultant direction and magnitude of natural forces in the area. In an area of abundant littoral drift, or where natural sand supply approaches the capacity of littoral forces to transport it, a system of groins can be designed to increase the dimensions of the natural beaches (Figure 11). In constructing a groin field, construction should proceed updrift. In other words, the downdrift groin should be constructed first so that the adjacent updrift embayments can be filling as construction proceeds updrift. Occasionally extended periods of time are required between the construction of each groin to achieve the best results. If the groins are built to a low profile, with top at the elevation of the natural beach, the littoral material passes over their tops, nourishing the entire system. The process continues until all groins are full. An undesirable feature of this procedure may be, that while the groins are filling, the beach downdrift from the groin system may be eroded. Almost without exception, groin systems are now filled artificially to allow immediate movement of the littoral material through the groin field.

For beaches in areas of little or no drift (or relatively little natural supply of beach sand), two or more high groins in combination with beach fill can be used to accomplish this same purpose. In this type of arrangement, the high groins are used at the downdrift ends of the compartmented areas to form pockets or bayheads that restrain the material from being moved out of the area by the littoral forces. Since there was little or no natural drift in such an area to start with, accelerated erosion of the downdrift shore should not be induced in important degree.

In locations where a field of low groins is installed adjacent to the end of a littoral compartment with appreciable natural drift, terminating

at a submarine canyon or deep water at the foot of a headland, a high terminal groin is used on the downdrift end to prevent the loss of sand from the littoral zone. Such a system is beneficial in conserving sand under these circumstances since an erosion problem immediately downdrift would not exist.

Groin systems are the most universally used structures for beach retention and enlargement. Frequently it is found desirable, when building and maintaining a protective beach, to supply additional protection to highly valuable property behind the beach berm and dune line. To accomplish this, a seawall, bulkhead or revetment is installed to protect the backshore during storm periods (Figure 8). Although an adequately wide beach affords excellent protection, it may be overtopped during major storms, resulting in dune deterioration and major interior flooding. Occasionally a breach through the beach may occur forming an inlet from the ocean. In this case the secondary defense structures prevent severe changes in the land forms.

Harbor jetties, either alone or with an offshore breakwater, accomplish the purpose of protecting an entrance through a barrier beach, but will result in a considerable change in the land form unless a sand bypassing operation is effected. The offshore breakwater creates a calm area shoreward of its center, resulting in the deposition of littoral material at that point. As the point builds, it acts as a natural groin and a great expanse of beach is built updrift. Further, as this wide expanse of sand dries, it is frequently moved by the wind into a substantial dune system. This change in land form can create other problems. Instances are recorded where sand movement by wind from impounded areas has built dunes around and completely over shore installations, including buildings. When this occurs, beach or dune stabilization measures such as sand fences and vegetal plantings are indicated. The storage of such large quantities of littoral material upsets the natural regimen of the area by removing the downdrift shores from the system of natural nourishment. Erosion is thus accelerated in this downdrift area. In order to reduce or eliminate this type of change in land form, the material can be moved downdrift by a hydraulic dredge working in the lee of the breakwater to re-establish natural conditions. To date sand-bypassing operations at jettied entrances which do not include an offshore breakwater have not been completely satisfactory.

Where natural features permit, it is frequently more economical to supply sand to the beach than to build groins. The artificially nourished beach is the most satisfactory shore protection system (Figure 14). It is the one system that maintains the balance of nature, and is relatively free of the undesirable features of other systems.

## ENGINEERING PLANNING AND DESIGN

In planning a shore protection project all factors pertinent to the problem must be considered to produce criteria necessary for a sound design. These factors are:

1. Geomorphology of the beach area.
2. Littoral materials.
3. Littoral forces.
4. Shoreline history

Study of these factors permits further analysis as follows:

1. Shore processes involved in the problem.

a. The direction, average rate, and variability of littoral transport.

b. Present and prospective rates of supply and loss, and quantitative deficiency or surplus in cubic yards.

c. Manner of movement of littoral materials that produced the problem conditions.

d. Predicted or estimated future shore conditions if no remedial measures are undertaken.

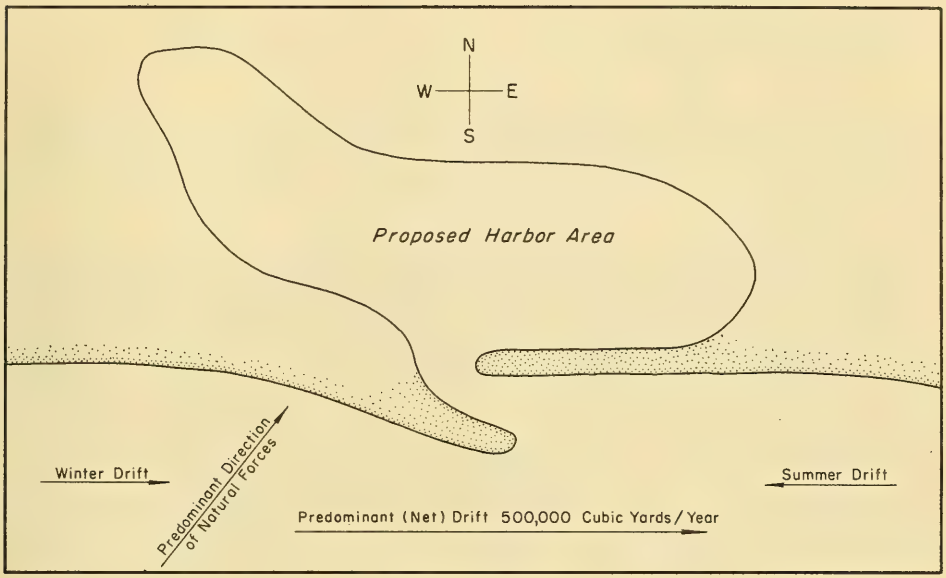
2. Methods of correcting problem conditions. This includes the selection of structures by which the objectives can be attained, and considers their effects within the problem area and adjoining shore.

3. Selection of design criteria for structures under consideration.

As an example; consider the following proposed harbor area (Figure 15).

1. Examination of the land form and a study of wind, wave, tides, and current directions indicate a seasonal reversal in drift, but a net dominance of the eastward component. Further, the resultant of the natural forces is found to strike the shore from a southwesterly direction.

2. Present supply and loss are in balance, with a net volume of about 500,000 cubic yards per year passing the area in the dominant direction of transport (eastward). In the recent past the flow of material has been uninterrupted eastward past the inlet. The problem arises from fixation of the inlet, which will cause accretion on the updrift side and equivalent erosion downdrift.



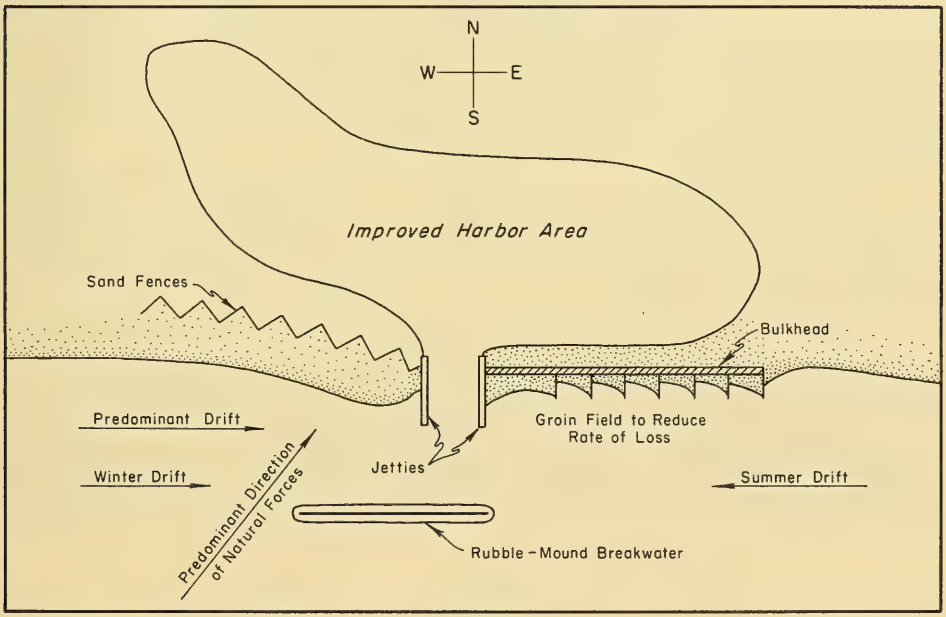




FIGURE 15. SHORE PROTECTION-HARBOR STABILIZATION

3. If remedial measures are not taken following fixation of the inlet, erosion downdrift thereof will be serious with a loss of about 500,000 cubic yards per year.

4. In order to correct the conditions created by fixation of the inlet, all facets of the problem must be considered and a properly designed system selected to cover all foreseeable events. In other words, a structure system must be selected to combat the adverse forces created by the proposed improvement.

An important phase of present day design is that the natural flow of littoral material across an inlet should be restored after construction of jetties. A long updrift jetty preserves the channel to the limit of its impounding capacity; when it is filled the material will move around its end. A second disadvantage of this type of structure is that the material in the impounded area is exposed to wave action and thus difficult to transport downdrift by mechanical means. In view of these factors, it appears that the most satisfactory method of inlet fixation consists of two short jetties and an offshore breakwater. With this arrangement jetty length is not a major problem since the impounded area forms a short distance updrift from the structure. A further advantage lies in the shelter afforded by the breakwater, which facilitates sand bypassing by hydraulic pipeline dredge.

The resulting land form includes impounded material updrift from the jetties in the lee of the breakwater which is adjacent to the improved harbor area. It thus appears that some form of sand fixation in the dry area will be required since the prevailing winds and the direction of natural forces will undoubtedly carry some sand from the impounded area into the harbor. Fixation by vegetation and plantings is not entirely satisfactory since the impounded area will be removed periodically by dredging. In view of the above, it is considered that sand fences will be more satisfactory for the purpose.

On the downdrift side of the jetties it is estimated that a loss rate of 500,000 cubic yards per year will occur. Since bypassing will be accomplished every other year, with consequent removal of 1,000,000 cubic yards in that interval, some provision may be desirable to protect the downdrift shores during this period.

Downdrift the land form is a low narrow sand spit separating the ocean from the harbor area. Since the loss rate from this area will be about 500,000 cubic yards per year, groins will be required to reduce this rate of loss if found to be more economical than advance nourishment or periodic bypassing. Further, since the spit is low and could be breached during storm tide periods, the need for a second line of defense is apparent if groins are employed. A bulkhead or revetment, whichever has the least cost, may be used. The extent of the groin system is determined by the worth of

the land to be protected. It must certainly cover the low spit area and extend to a point where some shore recession can be tolerated between sand bypassing or groin filling operations. The groin field should be of the low profile type. In the event that a beach is not necessary for recreational purposes in the area, the bulkhead and groins may be replaced with a substantial seawall, preferably a rubble mound to reduce wave run-up and overtopping. In the event that large rock is not available in the area, concrete shapes, tetrapods, quadripods or others may be used.

Structural design today is a straightforward process. Having selected a design wave (usually defined as the average of the highest one-third of the waves in the spectrum), together with a design tide (taken as the astronomical tide plus selected storm surge), construction material characteristics and foundation conditions, the final design becomes a matter of adjusting natural forces to man's need, with recognition that these forces are to be controlled by structures compatible with the natural situation.

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# REVIEW OF GERMAN EXPERIENCE ON COASTAL PROTECTION BY GROINS

by

Dr. Marcus Petersen

Dr. Marcus Petersen, renowned German coastal engineer and scientist, Director of Water Management and Economy of Schleswig-Holstein in Kiel, Germany, published in the periodical "Die Kuste" of 1961, under the title "Das Deutsche Schrifttum uber Seebuhnen en Sandigen Kusten", a survey on experience with groin construction. The 57-page article evaluates the experience accumulated by 120 prominent German coastal engineers and scientists as published in 210 bibliographic items which extend over a period of 140 years. It offers an interesting insight on the opinion and attitude of German engineers toward the historically controversial and highly disputed subject of coastal protection by means of groins, in which not only engineering and scientific, but to a great extent economic elements, have an important role. The paper by Dr. Petersen has six sections: I - Introduction; II - Collection and Evaluation of German Documentation on Groins; III - Chronologic Survey of German Documentation on Groins; IV - Experience on Seagroin Effect; V - Conclusions; and VI - Bibliography. Essentially the entire article was translated by Otakar W. Kabelac, Engineering Division, Beach Erosion Board. The following is an abstract, prepared by Mr. Kabelac, of each section of the article.

## I - INTRODUCTION

The function of sea groins, as a means of protecting sandy shores, remains even to the present time a controversial problem. The coasts of the earth are perpetually changing, the rocky coasts changing to a lesser degree than sandy coasts. Natural changes fluctuate in short time intervals around a certain base, and vary with location and time. Coastal regions exist, which when viewed with regard to long-term prospects, have a tendency to silt up, while others tend toward erosion. At the time of observation, the tendency for long-term coastal recession may coincide with an equally directed tendency for temporary fluctuation (as for example, the conditions following storm tides); then it is difficult to forecast what development of the beach will take place in a long period of time.

The engineers responsible for coastal construction are confronted with this problem more and more. Human interference with the labile equilibrium of nature, sandy coasts in particular, became more extensive because of the growing demands for coastal developments and protection. Often it cannot

be determined, whether the gain or loss of land in a coastal region is due to:

a. natural, cyclical and local fluctuation from a basic average position,

b. natural long-term trends, or

c. human interference.

It is therefore quite understandable that opinions in professional circles on the effect of sea groins have always been controversial and even at present are widely disputed.

In view of the considerable cost of coastal construction it was found necessary to examine the German literature on the subject to determine which publications on the effect of sea groins are of importance and also which results can be accepted without reservation. The survey is limited to papers which are generally easy to obtain and can be perused, examined and evaluated. Unpublished papers in official or private archives are not considered. The very extensive foreign literature such as Abecasis (Portugal), Bruun (U.S.A. and Denmark), Minikin (England), De Rouville (France), Schijf (Holland), Zenkovich (U.S.S.R.) is not used and only occasionally referred to. Even in foreign countries opinions vary considerably on this subject.

## II - COLLECTION AND EVALUATION OF GERMAN DOCUMENTATION ON GROINS

The first sea groins in Germany were constructed in the period 1818 to 1821. The first publications dealing with them are about one hundred years old. They may be found in manuals and in approximately forty different periodicals. In addition to engineering papers, there are numerous publications by geographers, geologists, oceanographers and local historians, in which are described certain conditions and events useful for the groin investigation, or in which pertinent relationships and developments are revealed.

All of these papers, together with the treatises of those German engineers engaged in coastal research, offer material for a new science of coastal morphology. Coastal engineering and coastal morphology are therefore in close relationship. Remarkable surveys on morphological studies are given in the book by Valentin (1952) "The Coasts of the Earth". However, only a few of the essays in that book deal with coastal engineering problems and groins.

Petersen's classification of German documentation on groins is divided according to coastal subjects, regions involved, and also chronologically.

The geographic location of Germany accounts for the regional distinction necessitated by dissimilar conditions on the Baltic Sea (a tideless inland sea), and the North Sea (a marginal sea characterized by its high and low tides). In the Baltic Sea the coastal currents generally are unimportant, while the tidal currents of the North Sea create a periodic movement of water masses. In addition, the currents there are augmented, when the filling and emptying of tidal flat areas (Wattenraum) occurs through submarine canyons (Strom gaten), tidal flat streams and river outlets.

Distinctions exist also between beaches with strong tidal or drift currents parallel to the coast with or without surge and those beaches with surge but without strong tide or drift currents. The surge itself often produces "a surge current" (Brandungstromung) parallel to the coast and of an intensity exceeding considerably that of the tide or drift current.

Prior to 1860 only a few reporters were writing on the construction and effect of sea groins because not many such structures were in existence then, and consequently only a few observations were made and data collected.

Gothilf Hagen was the first German coastal engineer to write on his experimentations with the sea groins on the Baltic Sea (1863). He traveled extensively in Holland, Belgium, England and other countries and presented his observations and the results of his investigations and experiences in a manual on coastal structures. A substantial part of his manual is valid up to this day.

At the XVth International Navigation Congress of 1931 in Venice, considerable attention was given to the defense against the sea on shores with and without major sediment movement. German papers, by Coen Cagli, R. Schmidt and H. Heiser, presented there, played an important role during the discussion. German authors found then that a thorough and systematic investigation of the coastal zone is paramount for coastal protection and coastal structural planning. They found that up to this day no generally applied rule for sea groin construction is apparent, indicating the difficulties associated with the clarification of this complex problem.

Often the most informative documents were simple published statements describing how the groin or group of groins were built, the construction material used, the dimensions selected and the experience gained from the construction. Occasionally we find that these data indicate the purpose and reason for the construction and also give information on the effectiveness of the installation. Of course, data on the effectiveness of groins are of limited value if collected immediately or soon after the completion of the structure. Usually a minimum period of 10 to 20 years is required for an objective evaluation of the groin effectiveness. Valuable information and experience was accumulated by manufacturers of building material; the bituminous material industry in particular. Recommendation regarding groin effectiveness made without a sufficient period of observation should be considered with some reservation.

A description of structural coastal protection is considerably more valuable if morphological data regarding coastal changes above and below sea level, sea stages, storm tides, wind observations and other information are added. Such descriptions are of greater value if based on data and measurements, measurements in nature as well as on models, systematically arranged, carried out, and presented with a view to the entire regional development.

A positive contribution to the knowledge of the functions and effectiveness of groins can be made only after a long period of observation. The unexpected malfunctions of groin systems sometimes are concealed to avoid adverse publicity and to protect the designer. However, we can learn from failures, and the sooner they are made known the sooner they can be eliminated by structural changes. The survey of available German publications on groins indicates that great attention was given to coastal morphology as the structures were analyzed.

In general, considerable effort was made to achieve close cooperation between the engineers and natural scientists.

## III - CHRONOLOGIC SURVEY OF GERMAN DOCUMENTATION ON GROINS

Contributions to the knowledge of groin construction (starting in 1818) which were based on significant events in the German coastal zone and led to their use and development, are discussed in this part of the review by Dr. Petersen. The chronologic progress of German groin construction with regard to distinct scientific, engineering and economic trends is surveyed in five steps.

a. Construction period prior to 1900 - By 1900 several basic papers on coastal morphology, coastal engineering and a treatise on the big Baltic Sea storm tide of 1872 were already published in which the authors tried to find the causes and effects of coastal changes. They were good observers of natural phenomena on beaches. The groins constructed during this period have a special experimental value, and for this reason researchers in subsequent years draw often on the ideas of older authors. They compare the old construction methods with the observations of a later period to see how the older design ideas materialized relative to their effectiveness.

b. Construction Period 1900 to 1920 - In this period a distinction between sea groins and stream or river groins can be noted. However, full definition relative to functional limitation and structural forms involved is not made clear.

c. Construction Period 1920 to 1930 - The German Harbor Engineering Society entered into the discussion on questions dealing with groin construction. Of great significance to the German coastal problems was

the XVth International Navigation Congress of 1931 in Venice. The recommendation made at the Congress by Coen Cagli clarifies the German attitude as follows: "Each plan for coastal protection must be preceded by a thorough study of the locality and all factors acting on the formation of the coast and how much it is exposed; the action set up by waves, currents of various kinds, atmospheric precipitation, and by floating ice; the origin and nature of material constituting beaches and the regimen of river deltas; the situation and regimen of the aquiferous sheets of fresh water flowing toward the sea; the influence on the sea by new scructures when they project from the coast." It was further stated at the Congress that the causes of the effect of the sea on coasts should be substantiated. The German model experiments during this period were successful, and their continuance under natural conditions was recommended.

d. Construction Period 1931 to 1945 - A general reversal of coastal construction method took place during this period as nearly all of the groins on the North Sea as well as the Baltic Sea were built of steel sheet piling. The economic life of this type of groin structure is very short (10 to 20 years), particularly when single steel sheet piling is considered. The result of a series of investigations in coastal morphology carried out in this period opened wide vistas into coastal zone phenomena and brought out that the knowledge of forces involved in those processes is very limited. The demands for more intensive and systematic investigations were thus substantiated. Differences were increasing between the practical builders, who believed in progressive success of groins, and the research engineers and natural scientists. Also, contradictory experiences and opinions were noticeable among the practicing professionals.

e. Construction Period 1945 to 1960 - Dr. Petersen's survey covers both parts of Germany, East and West, with considerable reference to East German scientists and engineers, including governmental agencies and institutes of higher learning. Following World War II, the incoming information was at first very scanty, consisting mostly of reports on the use of asphalt for construction of groins. Ideas on the role of sea groins continued to be very controversial. They have to serve for current deflection, beach protection and sand collection. It is confirmed by several authors that the groins fulfilled their role as coastal current deflectors. The groins failed as beach protectors and sand collectors, particularly on coasts with a deficient balance of sand. Following systematic experimental observation, it was found that the sand deficit could be balanced by artificial sand replenishment.

## IV - EXPERIENCE ON SEAGROIN EFFECT

The overall survey of experiences on the effectiveness of sea groins in Germany with special regard to engineering and economics is evaluated, indicating that controversy on the value of groins continues. Great progress in the structural strength of groins relative to stability and material does not prove their functional effectiveness which requires long periods of time for proper evaluation.

a. Motive for Seagroin Construction - In most cases, seagroin construction was motivated by damages caused by storm tides and the dynamic forces of the surge. Damages appearing at the foot of dunes and seawalls, as well as on longitudinal structures and on groins themselves, resulted from lee-erosion and the effect of currents moving progressively beachward.

b. Current Groins and Beach Groins - Seagroins are of two types; current groins and beach groins - each type having a different function. Current groins are current-deflecting structures and it has been proved that they are capable of fulfilling this role. Beach groins are expected to maintain the surge-exposed beach and improve it as much as possible. On certain beaches it is considered, with some reservation, that groins do retard the shore recession. The opinion on the role of beach groins varies from that on the unverified extent of the reduced recession rate (success = 1% to 99%), to that on maintenance (success = 100%), up to that on improvement of the beach (success > 100%).

On transitional reaches which are influenced by rivers and inlets from the sea, and on surge beaches, groins may serve both the functions of current deflection and beach protection. Here the success can be assessed only when the beach groin performance is clarified. The German technical literature does not yet provide a satisfactory clarification. Current-deflecting structures were constructed to an equal degree with those which would serve for beach conservation and improvement.

c. Groin Types - Beach groins have been tested in nature in an unusual number of cases. From the original procedure on constructing groins as permeable structures, in the form of single or double-row pile groins or stone cribs and fascine groins, progress was made toward impermeable massive stone groins, sheet pile groins of steel or reinforced concrete, and recently toward flat stone pitch asphalt grouted structures. Permeable and impermeable groins are found side by side in the same coastal reaches. The design of steel sheet-pile and reinforced concrete pile structures was apparently governed almost entirely by limitations imposed by structural engineering practices.

d. Groin Length - Initially the length of the groins was determined by structural engineering limitations. Only after the introduction of the steam pile driver did the groins advance into deep water. Already Germelman in 1881 (mentioned by Franzius) had used the jetting procedure, which was found very suitable for long piles (also sheet piling). The water jet was applied by appropriate equipment. Current-deflecting groins were constructed with good results, seaward up to and beyond the MTLW*, and also through stream channels.

*Mean Tide Low Water - Symbol in use in German Hydrographic Service.

With regard to beach groins, opinions as to their proper length were far apart. Groins of limited length were considered adequate on dry as well as on wet beaches, while groins reaching beyond the first sand reef were proposed when sand movement was the main problem involved. In the Baltic Sea this reef is located approximately 100 meters, and on the North Sea (Sylt Island) 300 meters, from the shore line. Only economic reasons account for the fact that such proposals regarding groin length were not, or only in part, accomplished. The construction as well as maintenance cost increased considerably with the length of the structure.

e. Elevation of Groin Crest - With regard to the most efficient elevation of the groin crest, a variety of ideas were found to exist. Some authors had the attitude that a horizontal position of the crest was correct, others held that a certain minimum margin above the beach elevation was necessary; still others constructed groins high above the beach, to the final elevation which they desired to achieve when the sand-catching capacity of the structure would be reached. New proposals point toward structures built so that they would follow the profile of the beach. This is possible only in the downward direction. During periods of heavy sand movement these groins would be buried.

f. Groin Groups - Lee Erosion and Groin Spacing - Wherever groins were erected, it was found that single groins never met the requirements and that more of them were always needed to provide protection or maintenance of beaches or to retard shore degradation. Always in the lee of surf currents (adjacent downdrift areas) new damages were found, requiring the expansion of groin groups. This development logically led to a "Totality System", meaning that the entire coast had to be protected by groins. However, Hansen (1938), considering the natural conditions of sand behavior, believed such a limitless procedure should be rejected not only for functional but also for financial reasons.

In a series of investigations regarding methods to alleviate the lee erosion problem, no satisfactory solution had yet been found. This problem played a considerable role in deciding the terminal limits of a group of current groins as well as for all beach groins in general.

Within a group of groins, spacing of the structures was determined in many ways; it varied from one to three times the length of the groins, without any distinct recognized system. As during the course of development, lengths of the groins were considerably increased in comparison to original concepts, a definite length-spacing relationship could not be derived on the basis of available engineering experience.

g. Direction of Attack (Groin Alignment) - A number of authors have held it appropriate to place the heads (outer ends) of a group of groins in a line conforming to the direction of current flow. In dealing with current-deflecting groins, this attitude is unamimously accepted as

correct. However, in observing the underwater groins between Borkum and Norderney, a difference in this attitude can be noticed which has not created disadvantages of a functional nature.

Some modern authors consider the line of attack in design of groins; some attribute no meaning to the line of attack while other authors do not present the subject at all.

The reason that a pliant line of attack is required in river engineering is the effort to achieve a regulated flow, particularly for the purpose of navigation. On beaches we deal with one shore only. The currents change direction and intensity, and even though of low velocity, they develop, in combination with the surf, a considerable transport capacity, as the surf action provides the suspended sand.

The importance of the line of attack is doubtful with regard to short groins which do not reach the offshore sand bars where the sand transport usually takes place.

h. Construction Material - A variety of construction materials used in building seagroins is listed below:

Wood: piles, spars, timber of available beach types, and brushwork (endangered by marine borers).

Natural stone.

Steel: single or multiple wall groins built of steel sheet piling; connecting elements of structural parts (endangered by rusting and corrosion).

Concrete: used experimentally at the beginning of the 20th Century, since then systematically.

Asphalt: for several years used for seagroin construction.

When one construction material is being replaced progressively by another, it is usually because of its economic life. This is influenced by changes of wetness, solar radiation, temperatures, surge impact (wave action), surge currents, elevation changes of the shore, sand abrasion, ice drift and sea bottom composition.

The economic life of steel and reinforced concrete for seagroins is generally only 10 to 20 years.

Wood can be destroyed in two to three years, when used in regions subject to marine borers or other wood-damaging phenomena. Without this deteriorating effect, the durability can extend several centuries.

Stone has an unlimited durability. It is being used as loading element in connection with brushworks or for bracing of steep banks or slopes.

As to the use of asphalt for coastal groins it is still too early to express a conclusive opinion in view of existing experience and disappointments with other construction materials.

The economic life of a structure fixes its maintenance cost.

i. Artificial Sand Addition - The artificial addition of sand in regions with a deficient balance of natural sand supply, is the most recent method of coastal protection, despite the fact that several decades earlier dredged material was discharged for such purpose on beaches of the Baltic and North Seas with good results. The time is too short to report on the results of this method of protection. The basic cause of the natural sand deficiency will not be alleviated by abundant sand importation, and this operation would have to be periodically repeated. However, sand has the advantage when compared with other construction materals because of its natural behavior in places where sand was located before. It is a question of economics whether or not artificial importation of sand should be recommended. The volume required and the transportation distances figure decisively in such calculations.

j. Costs - We find only one article on the cost of construction and maintenance of coastal structures in the entire German literature. This was written by Hibben (1935) and deals with the installation at Borkum. Up to 1916 the cost for the maintenance of protective installation and for repair of storm damages amounted to 42% of the total expenditures for new and additional construction. However, it is pointed out that "the latter mentioned additional construction has to be included in maintenance cost". This proved to be right, as according to another unpublished statement, the ratio of maintenance to new construction costs up to 1957 was 5 to 4. The costs for Borkum cannot be generalized, yet it offers an insight into the range of costs which may apply also to seagroins.

Initial construction cost for a beach groin is so high that the importation of sand by pipeline dredging proves to be entirely competitive. Comparison of costs requires the consideration of the limits of restoration. Fulscher, as a responsible referent in the Central Administration, proposed as early as 1905 to analyze the problem of cost.

## V. CONCLUSION

Up to the present, it has not been possible, from available information, to clarify the complex problems occurring on sandy beaches and to interpret all cases properly. It is generally admitted, in view of the fact that the literature on the effect of beach groins is encumbered with so many statements and hypotheses, that it is necessary to either substantiate or invalidate those statements analytically by measurements

and figures. In view of the high cost of construction and maintenance, this procedure should be given primary importance. It has become obvious that only through such methodical measurement and systematic investigation, patterned after G. Hagen's ideas, would conclusive judgment be possible to some degree. Besides the necessary measurements in nature, theoretical hydrodynamic investigations, measurements and observations on model tests should be promoted. New measuring instruments will have to be developed and a good start has already been made.

Systematic observation of coastal development trends, carried out over a period of decades, may help to avoid obviously unsound investments. It was shown in some cases that seawalls and other protective structures should not have been built because the pattern of natural behavior of the coast indicates that they will be destroyed. Also the construction of beach protective installations on the East Friesian Islands should have been postponed, as the current observations would have indicated a natural tendency toward ultimate silting of protective structures. Generally, disappointments can be avoided at the construction stage if reports are critically drafted and failures made known.

Observing development outside of Germany, we can verify that in general the same problems are encountered in places where surge exists in front of a sandy beach, and that all pertinent coastal countries are working on the development of a satisfactory procedure for coastal protection. Opinions on the effectiveness of seagroins also differ considerably in other countries, indicated by the new Dutch studies which attribute beach stabilizing effect to the groins on one side while denying them any effect on the other. The trend toward artificial beach nourishment is increasing.

Close and substantial cooperation between scientists and practical men seems to be needed more than ever before in order to avoid duplication of work and to recognize problems and the proper approach to their solution. The Coastal Commissions of the Baltic Sea and North Sea represent an established arrangement which offers the possibility for exchange of ideas for the purpose of promoting coordinated investigation.

Furthermore, it should be required that all available literature on coastal regions, both domestic and foreign, be collected at the Coastal Commissions of the Baltic Sea and North Sea and be publicized. A researcher of a coastal problem, in most cases, due to lack of time to undertake a survey of information sources, has not sufficient knowledge of previous achievements in a particular field.

The present status of coastal research and coastal engineering make combined action of all participating disciplines mandatory; only then can the physical, technological and economic fundamentals be found for creating functionally correct and structurally acceptable coastal protective structures.

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# MODEL STUDY OF OFFSHORE WAVE TRIPPER

by

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The Galveston District of the U. S. Army Corps of Engineers recently requested that the Beach Erosion Board's staff perform a series of small-scale model tests on a proposed offshore structure. This structure would act as a wave tripper, causing offshore breaking of larger waves, thus preventing waves of height greater than a particular value from reaching the shoreline structures. The system of a shoreline structure combined with an offshore structure was to be considered for providing protection during hurricanes to the shoreline of Texas City, Texas.

The study was carried out at the Beach Erosion Board laboratory in a wave tank approximately 74 feet long, 1.5 feet wide, and 2 feet deep. Waves were generated by a vertical-bladed, pusher-type wave generator, operated by an arm attached eccentrically to a drive wheel. A bottom profile, generalized from a hydrographic survey sheet provided by the Galveston District, was modeled at a 1:50 scale in the wave tank. A comparison between the bottom profile, as taken from the hydrographic survey sheet, and the generalized profile used in the model, is shown in Figure 1. A prototype storm surge level of 15 feet above mean sea level was used for all tests, as was a prototype wave period of 6 seconds. These conditions remained constant throughout the duration of the test series.

Two types of offshore structures were included in the test program. One type simulated a breakwater of rubble-mound construction, and the other a vertical-face structure. Two structure elevations were tested; one of 6 feet and the other 2 feet above mean sea level. In conjunction with the offshore structure variations, two types of shoreline conditions were also tested. One consisted of a smooth concrete slope to an elevation of 15 feet above mean sea level, with a rubble absorber landward of that point, permitting wave reflection to only a negligible extent. The other shore condition tested consisted of a rubble shore structure that extended from mean sea level to an elevation of 23 feet above mean sea level, and which permitted considerable reflection of wave energy.

Wave heights were measured at a number of locations in the wave tank without an offshore structure present, and both with and without a structure present on shore, in order to determine the characteristics of the incident waves in undisturbed form. Then, wave height measurements were made with each type of offshore structure in place, each with two elevations, and with different shoreline conditions. The locations of wave gages relative to the bottom profile may be seen in Figure 1. Figure 2 shows the wave gage locations with respect to the various test conditions.

Wave heights were measured by means of parallel wire resistance probes suspended in the tank at various locations. These were calibrated

regularly in order to maintain linearity in their recording. The wave heights thus measured were recorded on paper tape by means of a Brush recorder. The heights of ten waves for each eccentric setting were read directly from the recording paper, averaged, and converted to prototype heights. (For convenience, prototype dimensions are used throughout this report.) Whenever it was noted that reflection would distort the dimensions of the recorded waves, less than ten waves (but no less than five) were taken as being more truly representative.

In order to determine the effect of the offshore structure on the waves transmitted over it, the wave gage was moved from its initial location on the centerline of the offshore structure, without the structure in place, to a point slightly more than one wave length shoreward from the structure in place. The depth of the bottom profile at this point was the same as that at the offshore breakwater location so that any noted changes in wave characteristics could be directly attributed to the presence of the structure. This location was approximately 195 feet (prototype) shoreward of the breakwater. As both the rubble-mound and vertical-face types of breakwater fitted quite snugly against the sides of the wave tank, the leakage of water at effective velocities around the barrier was very small; hence this factor is considered negligible as a source of error in the test series.

Figure 3 shows the results of the wave height measurements made 195 feet shoreward of the offshore breakwater, without a shore structure in the model. The full heavy line at 45-degree slope on the graph in Figure 3, of course, represents the line of no effect; i.e., in the zone above or to the left of this line, the offshore breakwater had no effect relative to the dissipation of wave energy. The data points, connected by the dashed curved line in Figure 4 represent actual measurements of wave heights at the intersection of the bottom profile of the model with mean sea level, the measurements taken relative to the undisturbed incident wave height, with neither the shore structure nor the offshore breakwater in place. Other curves in Figure 4 show the measured wave heights, relative to the undisturbed incident wave height taken for each of four conditions for the offshore breakwater in place. Although reflection at this point was considered negligible due to the presence of an effective absorber beach landward from the intersection of the bottom profile with the test water level (15 feet above mean sea level), it is quite possible that some slight amount of reflection from the bottom slope was responsible for the crossing of the curves in Figure 4.

The main results of the study are shown in Figures 3 and 4. Figure 3 shows the wave height in the protected area immediately shoreward of the proposed offshore tripper which would be observed for various incident wave heights and structure conditions. For example, for a water level of +15 feet MSL, an incident 10-foot wave would be reduced to approximately a 9-foot wave just shoreward of the tripper if the crest of the wave tripper is at +2 feet MSL; and to a 7.6-foot wave if the crest of the offshore tripper is at +6 feet above MSL. It may be noted that essentially

the same reduction in wave height is observed whether the offshore tripper is of rubble-mound construction or of vertical-face, sheet-pile construction.

Figure 4 shows the wave height which would be incident at the shore (measured at the location of the MSL contour) for various incident wave conditions. For example, for a 10-foot wave incident to the general area, the offshore wave tripper would reduce the wave incident on the shore structure from about 11 feet to between 7.3 and 7.9 feet, depending on the characteristics of the tripper itself. Again it may be noted that very little difference was observed between the rubble and sheet-pile structures.

Some additional data were obtained from the testing program relative to reflection conditions existing seaward of the proposed offshore structure with that structure in place. These data, summarized in Figure 5, showed no measurable effect on wave heights seaward of the proposed structure.

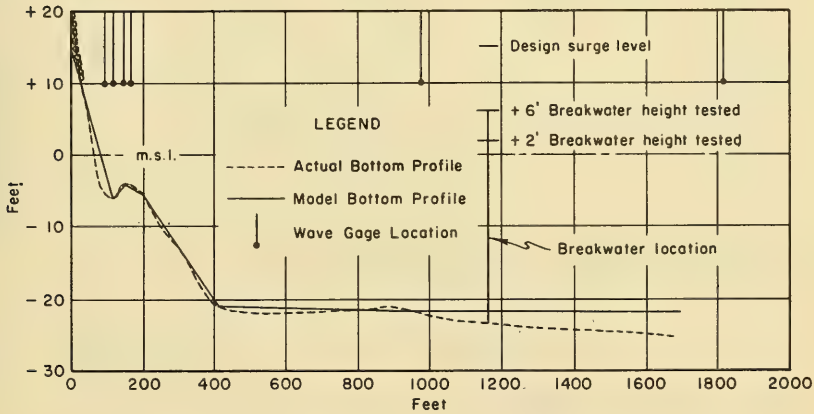


FIGURE I. GENERAL TESTING ARRANGEMENT

| BREAKWATER TYPE & HEIGHT (in feet above m.s.l.) | WAVE GAGE LOCATIONS: Intersection of Bottom Profile with m.s.l. | 1050' Shoreward from Offshore Structure Centerline. L/4 from Shore Structure. | 1022.5' Shoreward of Offshore Structure Centerline. | 1004' Shoreward of Offshore Structure Centerline. L/2 from Shore Structure. | 195' Shoreward of Offshore Structure Centerline. | Offshore Structure Centerline. | 650' Seaward of Offshore Structure Centerline. |
|---|---|---|---|---|---|---|---|
| Rubble Mound, +6 | | | | | | | |
| Rubble Mound, +2 | | | | | | | |
| Vertical Face, +6 | | | | | | | |
| Vertical Face, +2 | | | | | | | |
| Without Structure | | | | | | | |

KEY

- Waves measured at this location with smooth, concrete shore facing.
- Waves measured at this location with rubble shore facing.
- Waves measured at this location with both types of shore facing.

FIGURE 2. WAVE HEIGHT MEASUREMENT PROGRAM

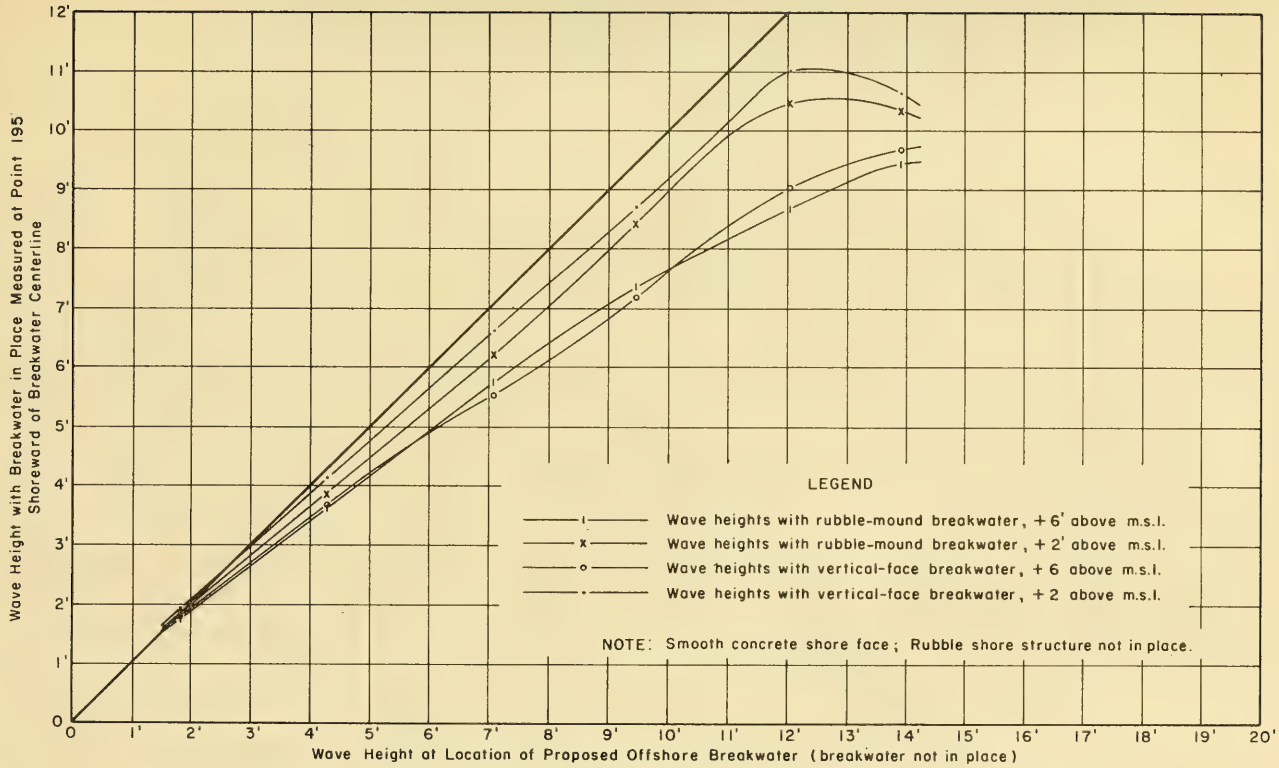


FIGURE 3. EFFECT OF WAVE TRIPPER IMMEDIATELY IN ITS LEE

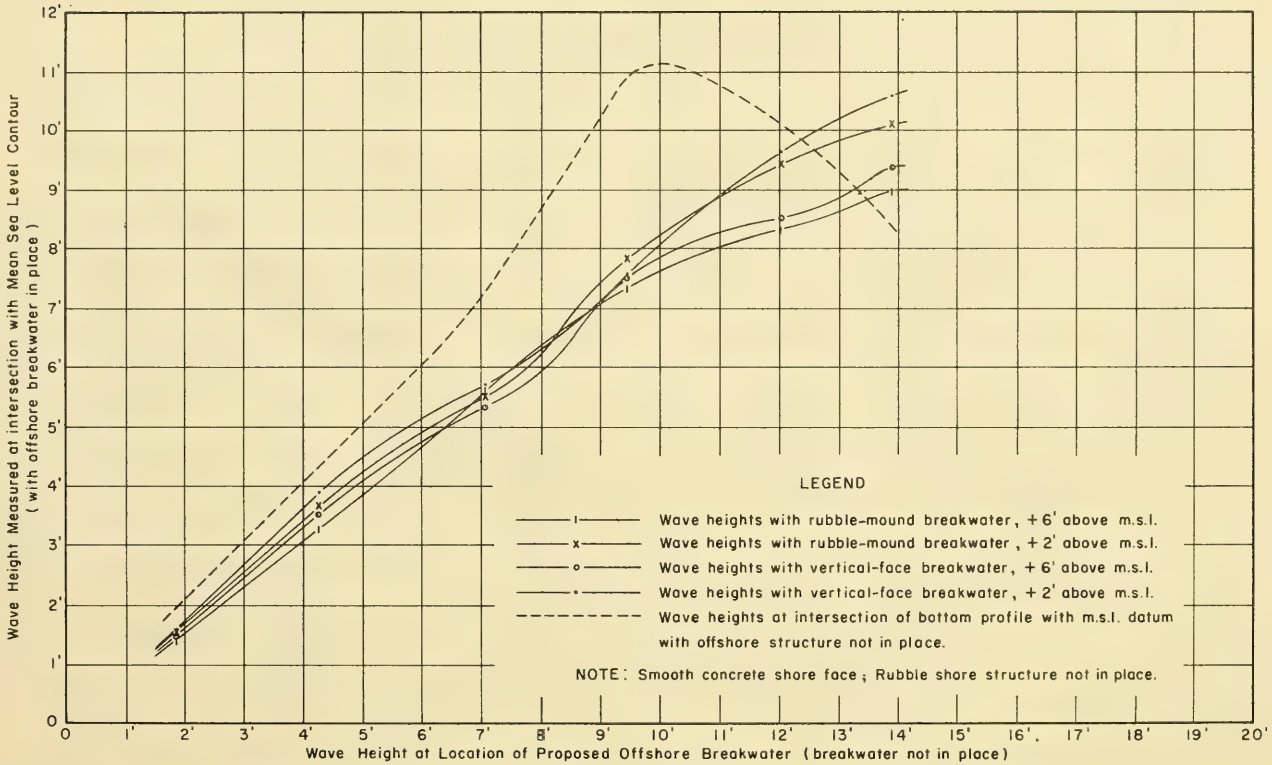


FIGURE 4. EFFECT OF WAVE TRIPPER AT M.S.L. SHORE LINE

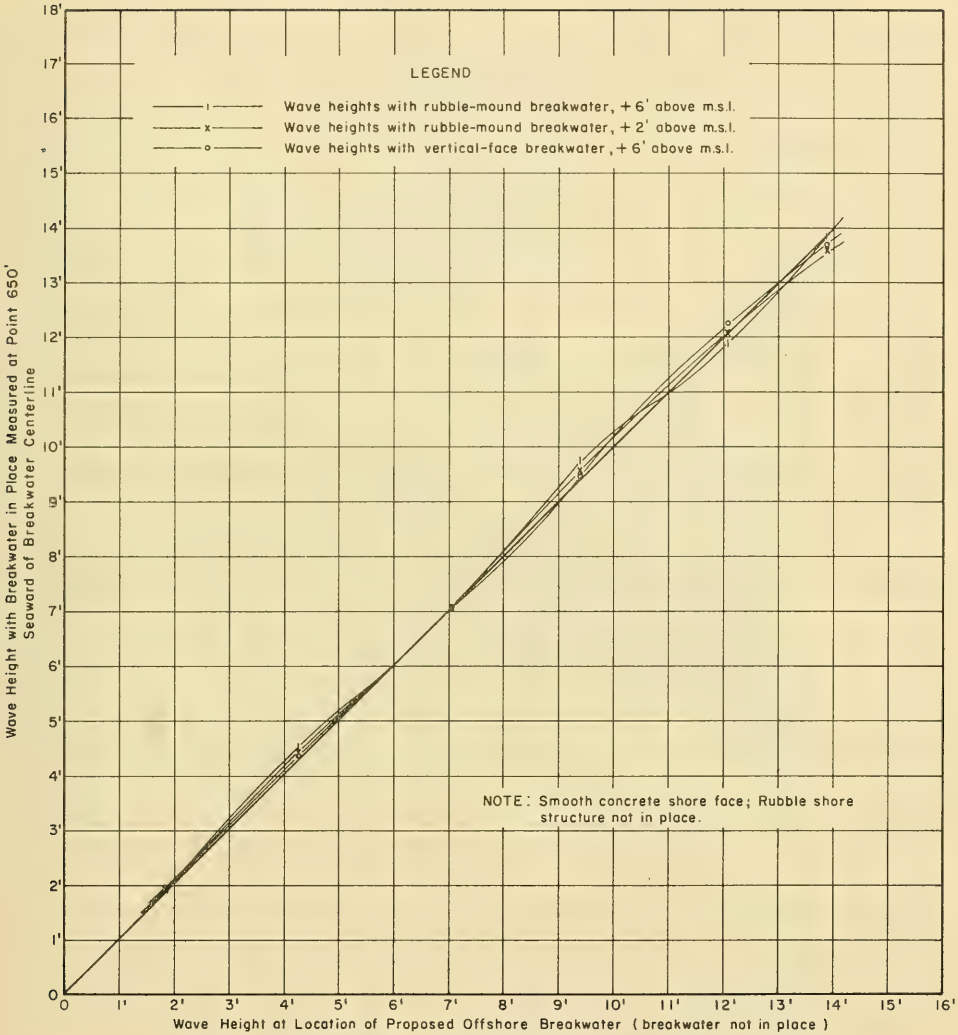


FIGURE 5. EFFECT OF WAVE TRIPPER ON WAVE HEIGHTS SEAWARD OF THAT STRUCTURE

# NOMOGRAPH FOR DETERMINING PRODUCT OF SHOALING AND REFRACTION COEFFICIENTS FOR USE IN WAVE ANALYSIS

by

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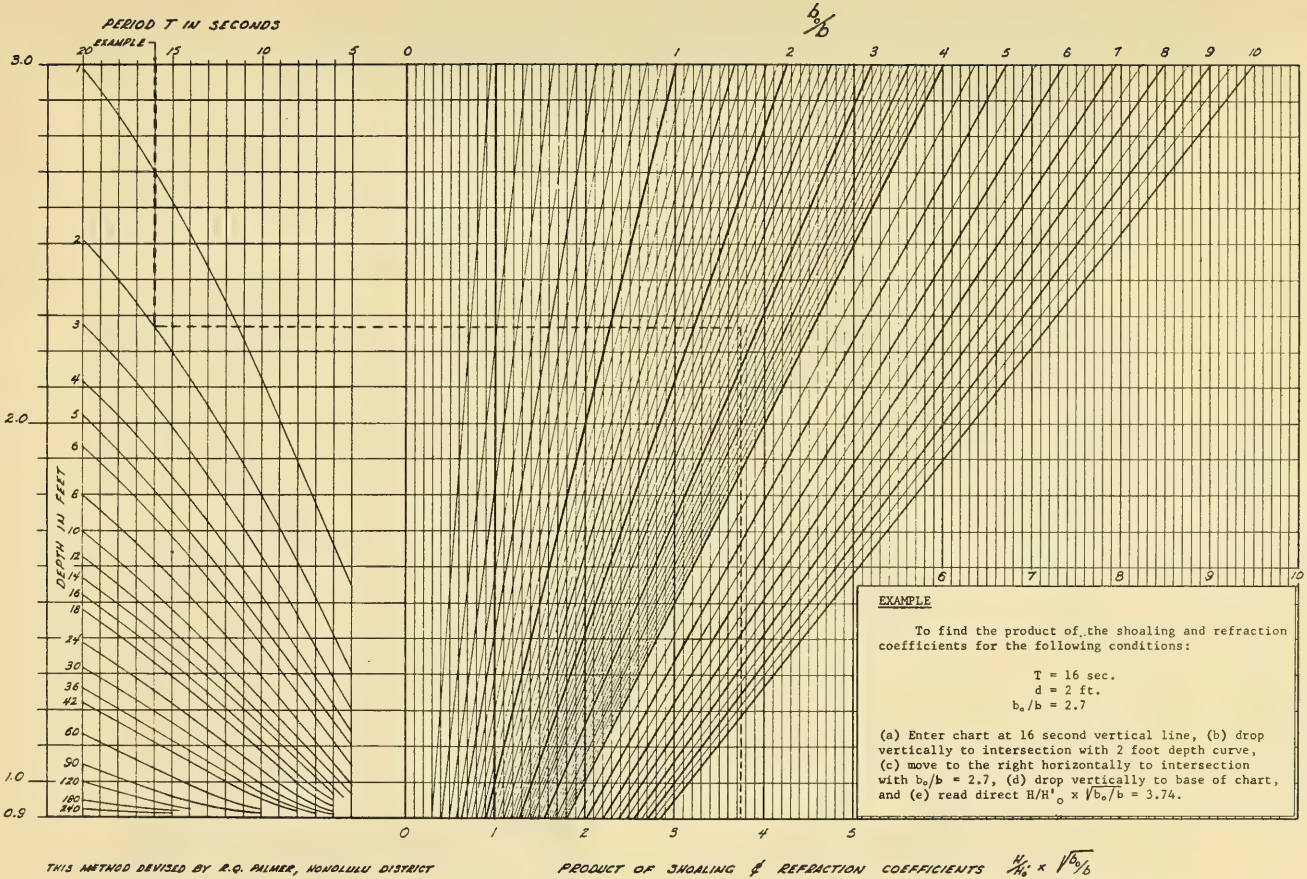
## Description of the Nomograph

The nomograph shown on the adjacent page may be used to readily calculate the combined effects of shoaling and refraction for a given wave period and depth. The shoaling coefficient alone may be obtained by using the left third of the chart. Likewise, the square root of the $b_o/b$ value may be obtained alone by using the right portion of the chart. However, the chart is most helpful when used, as shown by the example, to obtain the product of the shoaling and refraction coefficients. This value is useful in determining wave height values at a specific location in shallow water when the deep water height ($H_o$) is available from other means, as by statistical hindcasting.




EXAMPLE

To find the product of the shoaling and refraction coefficients for the following conditions:

T = 16 sec.
d = 2 ft.
$b_o/b$ = 2.7

(a) Enter chart at 16 second vertical line, (b) drop vertically to intersection with 2 foot depth curve, (c) move to the right horizontally to intersection with $b_o/b$ = 2.7, (d) drop vertically to base of chart, and (e) read direct $H/H'_o \times \sqrt{b_o/b}$ = 3.74.

THIS METHOD DEVISED BY R.Q. PALMER, HONOLULU DISTRICT

PRODUCT OF SHOALING & REFRACTION COEFFICIENTS $H/H_o' \times \sqrt{b_o/b}$

NOMOGRAPH YIELDING PRODUCT OF SHOALING AND REFRACTION COEFFICIENTS

# PROGRESS REPORTS ON RESEARCH SPONSORED BY THE BEACH EROSION BOARD

Compiled by Thorndike Saville, Jr., Research Division
Beach Erosion Board

Summaries of progress made during fiscal year 1963 (i.e. to June 30, 1963) on the several research contracts in force between universities or other institutions, and the Beach Erosion Board, together with brief statements as to the status of some research projects being prosecuted in the laboratory of the Beach Erosion Board, are presented below. These summaries supplement and continue those contained in prior issues of the Bulletin.

## I. University of California, Contract DA-49-055-eng-8. Sources of Beach Sand.

This contract was terminated in September 1962 with distribution of the report, "Beaches in Northwestern California", which was given limited distribution by the University of California as Institute of Engineering Research Technical Report, Series 14, Issue 25. This report describes conditions at the various beaches sampled north of the Russian River, and indicates differences between them. The report indicates that although groups of stations having sands that are not significantly different usually fall within boundaries of geographic units, the sands at any given locality can seldom be correlated with existing wave exposure; consequently it is concluded that beach character must reflect other causes as well, such as material source, location, and quantity.

## II. Massachusetts Institute of Technology, Contract DA-49-055-civ-eng-62-9, Study of Beach Processes in the Inshore and Foreshore Zones. (Old Contract DA-49-055-eng-16, Sorting of Beach Sand by Waves).

Work previously carried out under Contract DA-49-055-eng-16 was continued under a new contract, Contract DA-49-055-civ-eng-62-9, with the new title "Study of Beach Processes in the Inshore and Foreshore Zones". Development of an adequate orbital velocity probe, utilizing a thermistor, for use in the study was continued and the probe and its electronic circuity is described in MIT Hydrodynamics Laboratory Report No. 61, "A Thermistor Probe for Measuring Particle Orbital Speed in Water Waves", which has been given limited distribution by MIT. Publication of this report by the Beach Erosion Board is planned. The thermistor used has a bead diameter of 0.014 inches, and is mounted at the tip of a probe of 0.15-inch tubing. Both static and dynamic calibrations have been performed, and compare favorable with each other. If carefully constructed, the instrument also shows relative direction insensitivity.

Theoretical studies aimed at the description of the geometry and kinematics of the shoaling wave were continued.

## III. University of California Contract DA-49-055-civ-eng-63-4, Transport Of Coastal Sediments. (Old Contract DA-49-055-eng-17, "Fundamental Mechanics of Sand Movement by Waves")

Work previously carried out under Contract DA-49-055-eng-17 was continued under a new contract, Contract DA-49-055-civ-eng-63-4, "Transport of Coastal Sediments".

A report "Sand Movement by Wind" was given limited distribution by the University of California as Institute of Engineering Research Technical Report Series 72, Issue 7, and is now being published as a Beach Erosion Board Technical Memorandum. This report discusses earlier experimental results in wind tunnels of sand movement by wind, and describes and compares results obtained from new wind tunnel tests. Findings of previous investigators with respect to rate of sand transport were generally reaffirmed, but average flying distance of sand particles was found to be much greater, possibly due to the method of calculation. Of particular interest are tests on the influence of moisture content on sand movement. The experimental data clearly demonstrate that, as the sand moisture content increases, the value of the threshold shear velocity of sand movement may also materially increase. A quantitative expression of this effect is obtained. A further report, "Sand Transport by Wind - Studies with 0.145 Millimeter Sand", given limited distribution by the University of California as Institute of Engineering Research Technical Report HEL 2-5, is also included as an appendix. This appendix extends the investigation of sand movement by wind to smaller sand particle size ranges, and indicates that threshold velocity is best determined by experiment rather than by formula when the sand grain size is less than about 0.2 millimeters.

Another report, "Beach Profile as Affected by Vertical Walls", was published as Beach Erosion Board Technical Memorandum No. 134. This report presents the results of a laboratory model study to investigate the equilibrium beach profile resulting when vertical walls of various top elevations above or below the water surface elevation were located in the beach zone and subjected to wave action. As might be expected, walls of highest relative crest elevation, by allowing less energy to pass over the wall, resulted in greater scour in front of the wall; lower walls resulted in increased scour areas behind the wall. Effects of wave steepness and grain size of beach material were also investigated. As appreciable scale effect may be involved in such a study, care must be exercised in interpretation of the data for prototype use; nevertheless the data is considered of value in considering practical problems involving vertical face walls.

A further report "Investigation on a Two-Phase Problem in Closed Pipes" was given limited distribution by the University of California as Institute of Engineering Research Technical Report HEL 2-2. This report concerns transportation of sediment in pipelines, and particularly discusses two systems for measurement of concentration of the water-sediment mixture, and hence the transportation characteristics. Two measuring devices were tested, a loop-system consisting of two identical vertical

pipe sections with opposite flow direction, from which head-loss readings could be obtained and correlated with flow rate and sediment concentration; and a Venturi meter, in which the pressure drop was again measured and correlated with sediment concentration.

A further report "Sand Movement on Coastal Dunes" was given limited distribution by the University as Institute of Engineering Research Technical Report HEL-2-3, and is also being published in the Proceedings of the Federal Interagency Sedimentation Conference in Jackson, Mississippi, 28 January - 1 February 1963. This report summarizes the results of laboratory studies of sand movement by wind, discussed more fully in the report, "Sand Movement by Wind", and its appendix discussed above.

A further report, "Transportation of Bed Material Due to Wave Action" was given limited distribution by the University as Institute of Engineering Research Technical Report HEL-2-4, and is now being published as a Beach Erosion Board Technical Memorandum. This report discusses the mechanism of sediment transport in a layer immediately adjacent to the ocean floor for long waves of small amplitude in relatively deep water. The fundamental principle of the supporting theory is that at equilibrium the submerged weight of the solid particle is balanced by the vertical component of the resultant hydrodynamic force exerted on the particle by the flow above. Effects of both the unsteady mean flow velocity and the turbulent fluctuations are taken into account. The distribution of the lift forces associated with the former was determined experimentally, while a statistical approach based on experience with the same phase of the problem in a steady mean flow was used to determine the latter.

IV. University of California, Contract DA-49-055-civ-eng-63-5. Wave Diffraction and Refraction Studies. (Old Contract DA-49-055-eng-44, "Laboratory Study of Wave Refraction")

Work previously carried out under Contract DA-49-055-eng-44 was continued under a new contract, Contract DA-49-055-civ-eng-63-5, "Wave Diffraction and Refraction Studies". A report, "Higher Approximation to Non-Linear Water Waves and the Limiting Heights of Cnoidal, Solitary, and Stokes' Waves", was supported partially under this contract and published as Technical Memorandum 133 of the Board. This report presents a mathematical description of some of the higher approximations to non-linear water waves and is particularly important in indicating limiting heights. First and second approximations to solitary and cnoidal waves are obtained by carrying the shallow water expansion method of Friedrichs and Keller to the fourth order. The rigorous first approximation to these finite amplitude waves of permanent form is identical to earlier solutions. The second approximation, however, results in new expressions for predicting behavior of long waves in shallow water. The limiting height for solitary waves is found to be 8/11ths of the free water depth. The third approximation to Stokes' waves in water of finite depth is verified by use of classical small-perturbation expansion methods. For finite amplitude waves the series expansion is found to be in terms of a parameter that is most

suitable for wave lengths shorter than eight times the depth. Rather severe restrictions inherent in the analogy between non-linear shallow water flow and two-dimensional perfect gas flow are also pointed out.

A report, "Effect of Bottom Slope on Wave Diffraction" was given limited distribution by the University as Institute of Engineering Research Technical Report Series 89, Issue 8. The report discusses a laboratory study of the combined problem of wave diffraction and refraction in a harbor. Several different bottom configurations were used in the protected area behind the breakwater, including both continuous slopes and abrupt changes. The resulting data are compared graphically with data for a horizontal slope in the sheltered area, such that refraction and shoaling would have no effect.

Work was continued on study of the Mach-stem phenomenon, and a report "Diffraction of Periodic Waves Along a Vertical Breakwater for Small Angles of Incidence" was given limited distribution by the University as Institute of Engineering Research Technical Report HEL-1-2. This report discusses the laboratory investigation of the reflection of periodic gravity waves as they advance along a straight vertical breakwater for the case of initial angle between the breakwater and the direction of incident wave advance of less than 20 degrees. The results were similar to phenomena observed for solitary waves, in that the waves were not regularly reflected from the breakwater, but a wave energy concentration was found next to the breakwater. Apparently this was a non-deep water phenomenon.

Theoretical work was carried out on the problem of two-dimensional spectra of wind-generated waves, and preliminary tests were made in the newly modified 12-foot wide wind wave tank.

V. Dr. W. C. Krumbein (Consultant). Study of Beach Phenomena.

A report, "Geological Process - Response Model for Analysis of Beach Phenomena" has been prepared, and is published elsewhere in this Bulletin. This report describes beach processes and deposits in terms of a conceptual process-response model that considers the processes and deposits as separate though closely related aspects of shoreline phenomena. The model provides a formal framework for analysis of natural beaches as they may be modified or controlled by the coastal engineer. Further study is being made of the application of computing machine methods to the study of factors influencing beach characteristics and stability.

A pilot study of sand sample data collected near the mouth of the Cape Fear River, North Carolina, has been subjected to analysis by computer methods and results are being analyzed.

VI. Virginia Institute of Marine Science. Contract DA-49-055-civ-eng-63-6 Study of Beach Response Relationships.

A new contract was negotiated with the Virginia Institute of Marine Science to study beach and bottom processes and response elements in an

attempt to better determine their interrelationships. The investigation involves the simultaneous observation of the various elements, with subsequent data analysis using statistical and high-speed computer techniques. Progress so far has involved establishment of horizontal and vertical control along three profile lines in the Virginia Beach area. Simultaneous observations of wave, tide, and current data have been made on several occasions, together with obtention of bottom sediment samples and profile data. Computer analysis for the development of trend surface data is underway.

VII. University of California Contract DA-49-055-civ-eng-63-8. Wave Forces On Coastal Structures

A new contract was negotiated with the University of California in Berkeley for the laboratory study of wave forces on coastal structures. In particular, information will be obtained on the vertical forces exerted by water waves on structural members of various types of coastal structures; and the details of eddy formation by waves moving past vertical cylinders will be investigated. Laboratory work on study of vertical forces induced by waves on a dock has been initiated, as has theoretical work on the distribution functions of wave forces on a circular pile.

VIII. Texas A & M Contract DA-49-055-civ-eng-63-9. Modification of Two-Dimensional Long Waves over Variable Bottom topography.

A new contract was negotiated with the Agricultural and Mechanical College of Texas to investigate the modification of free gravity waves in variable depth, including reflection and transmission aspects, especially considering those long wave phenomena where simple refraction theory is inadequate. The study will have particular application to propagation of tsunami wave packets and solitary surges over the continental shelf. Initial phase of the work has been to establish a modified long wave equation which can properly take into account the dispersive character of quasi-long waves.

IX. Scripps Institution of Oceanography Contract DA-49-055-civ-eng-63-10 Mechanics of Sediment Transport by Waves and Currents.

A new contract was negotiated with the Scripps Institution of Oceanography to study the mechanics of sediment transport by waves and currents in shallow water. Major objectives of the study are to obtain quantitative measurements of the sediment movement associated with the motion of waves and currents and to compare and relate this transport to measurements of the basic properties of the fluid inducing the transport. Particular attention is to be given the critical boundary of the sediment-water interface. Initial work under this contract will be to design a data acquisition system to give analog records permitting later computer processing for power spectra and cross spectral analysis.

X. Professor Frank D. Masch (Consultant). Wave Characteristics in Shoaling Water

A consultant contract has been negotiated with Professor Masch at the University of Texas for study of wave characteristics in shoaling water, with particular attention to cnoidal wave theory. The basic purpose is to present a workable and convenient method for computing water wave characteristics in shoaling water utilizing the non-linear cnoidal wave theory. Basically the method involves calculating power transmission for a wave train in shallow water from cnoidal theory and equating this to the wave power in deep water. Computation has been made of the average cnoidal wave power by integrating the product of the pressure in the horizontal component of water particle velocity over the distance from the bottom of the fluctuating water surface and over one wave period. This integration has involved determination of the various integrals of the powers of the cnoidal functions in terms of the wave characteristics and the complete integrals.

XI. University of Miami (Marine Laboratory). Contract DA-49-055-civ-eng-63-12. The Role of Shell Material in the Natural Sand Replenishment Cycle of the Beach and Nearshore Area.

A new contract has been negotiated with the University of Miami to investigate the role of natural shell replenishment as a contributing element in the nourishment of beach material along the beach and nearshore area. Particular attention will be paid to the area lying between Lake Worth Inlet and the Miami Ship Channel on the Atlantic Coast of Florida. Study will be made of the kind and amount of shell being contributed to this area, its source, its disposition, and its duration as a component of the beach materials.

XII. The University of Southern California. Contract DA-49-055-civ-eng-63-13. Shelf Sediment Transport.

A new contract has been negotiated with the University of Southern California (Department of Geology) to study the pattern of sediment movement in the nearshore shelf area, and in particular of sediment transport in submarine canyons. The investigation will utilize fluorescent tracers, and is pointed toward determination of quantitative estimates of material movement down submarine canyons from the littoral zone near the shore.

XIII. University of Southern California. Contract DA-49-055-civ-eng-63-14. Surf Zone Transport

A new contract was negotiated with the University of Southern California (Department of Geology) to study the movement of sediment within the surf zone, and its relation to the distribution of water particle velocity in the surf area. A telemetering dynamometer will be used to obtain velocity measurements, and fluorescent dyed sand grains will be used as tracers.

## XIV. Beach Erosion Board Laboratory.

### (a) Wave Forces on Structures

Analysis was continued on the wave force data obtained in the large wave tank on a vertical 12-inch diameter pile. A report presenting some of this data was prepared in draft form.

In connection with another study concerning transmission of wave energy over low-crested breakwaters, a small, though useful, amount of information was obtained on stability of rubble breakwaters under heavy overtopping conditions.

### (b) Wave Run-up.

The joint distribution of wave height and wave period for a fully developed sea, as developed by Bretschneider's method, has been used to obtain the distribution of relative wave run-ups in a sea condition, and also the distribution of actual run-up. This latter distribution turns out to be essentially identical with that for wave height as developed by Puttz, Longuet-Higgins, and others (at least for conditions examined). As such, it appears that about 13% of the run-ups exceed the run-up of the significant wave, about 1% of run-ups will be 1.5 times ($R/H_{1/3}$), and about 1 in 1,000 will be about 2 times ($R/H_{1/3}$). This analysis has been made only for the case of relatively deep water fronting a structure ($d/H_o > 3$) and for a fully developed sea, though it may be extended to include normal generating areas. A report, " An Approximation of the Wave Run-up Frequency Distribution", by Thorndike Saville, Jr. was published in the Proceedings of the 8th Conference on Coastal Engineering, Council on Wave Research, Engineering Foundation (1963).

A graphical method was developed for checking the design height of protective dunes for beach fills which are to be constructed along an existing shoreline. This design height is dependent on the maximum run-up which could be expected on the structure. The method involves computing a "critical profile" which, when plotted on transparent paper, can be overlayed on field profile plots to determine if run-up from expected waves will exceed the structure height. A report, "A Graphical Method for Checking the Design Height of Structures Subjected to Wave Run-up", by R. P. Savage was published in the Bulletin of the Beach Erosion Board, Volume 16, July 1962.

Wave run-up testing was carried out at small scale on some stepped seawall designs. The tests reaffirmed previous conclusions that artificial roughening of sloped seawalls by steps materially decreased wave run-up below that which would be expected on a smooth slope. Tests were made for both 1 on 2 and 1 on 3 slopes. Tests were made for vertical steps and also for a re-entrant type step. Essentially the same run-up was obtained for both types of step, although there is an indication that slightly lesser run-ups may occur with the re-entrant type step. This

indication is masked within the general scatter of data for both types. Results have been given limited distribution by the Board in an interim report, "Interim Report on Interlocking Precast Concrete Block Seawall Study", by R. A. Jachowski and J. R. Byerly, May 1963. Later formal publication is planned.

(c) Study of Sand Bypassing Operations.

Efforts were continued to collect all available data on sand bypassing operations (past, present, or planned) for correlation and study. Analysis continued of the hydrographic survey data obtained in the Port Hueneme area in June 1959, and comparison of this data with previous survey data. Coordination with Palm Beach County officials was continued for compiling data information on the operation of the sand transfer plant at Lake Worth Inlet, Florida. A field observation program was continued in the vicinity of Ventura County Harbor, California, in which an offshore breakwater (parallel to the shore) forms a protected area serving as a sand trap. Use of 3 wave gages at this location, each with different degrees of sheltering, permits some degree of evaluation of wave direction. A summary report was initiated which will summarize pertinent details of all operations of sand bypassing at coastal inlets in the United States.

Award of contract was made to install in the Beach Erosion Board Laboratory a mass flow density meter using a radioactive source to measure flow of sand through a 3-inch pipe. Delivery of the component parts is being made separately, and some of these are considerably delayed. Complete installation of the gage is expected sometime in August 1963, at which time it will be tested for acceptance.

(d) Laboratory Study on Relation of Littoral Drift Rate to Incident Waves.

Yearly summaries of the littoral transport tests for 1961 and 1962 were prepared. These summaries are in draft form, but are planned for limited distribution about the end of the calendar year.

Considerable effort was expended in attempts to secure a more adequate measurement of littoral current velocities, and the variations thereof. A sonic current meter developed by Westinghouse has shown considerable promise, but still appears to have certain drawbacks. In the meantime estimates of current velocities by dye tracers are being continued.

Testing was continued on determination of the relationship between incident wave characteristics and the amount of littoral transport. Measurement and analysis of waves has continued in an attempt to determine the cause of apparent resonance phenomena in the model basin, and the meaning of these phenomena. Of particular concern is the fact that this phenomenon results in a change in wave characteristics, not only with time-duration in a particular test run, but also for the same time-durations of otherwise identical test runs. Some of the effect is caused by inconsistency

in the machine operation, and may be eliminated by, for example, always starting the machine with the generating blade in the same position. In addition, baffles have been attached to the wave-generating bulkheads and are being studied for effectiveness in reducing parasitic cross-waves. Consideration is now being given to the possibility that some of the effect may result from slightly changed reflection patterns caused by slight changes in the beach face due to the incident wave action. In the meantime, accurate determination of the incident wave conditions for the littoral tests, or rather, interpretation of the wave records obtained during these tests, remains a problem. In general, incident wave values are obtained from a 1 to 2 scale model of the generating system in a narrow wave flume, in which reflection-caused resonance phenomena are not a problem.

Two of the tests conducted this past fiscal year were used as the vehicle for radioactive and luminescent tracer studies. The primary purpose of these tests was to develop procedural techniques, although limited research objectives were also explored. In both tests, glass grains containing an irradiated element, having the same specific gravity and size distribution as the beach sand, were injected onto the beach. Following injection, waves were generated at an angle to the beach, moving material alongshore. The first test involved a wave of constant height and period, and the second a wave of varying characteristics. The beach and bottom were periodically sampled to allow counts of luminescent and radioactive grains, and a radiological survey was made with a scintillation counter. The tracer used in the first test was sodium 24 (with a half-life of 15 hours), and the tracer in the second test was lanthanum 40 (with a half-life of 40 hours). The techniques used and the data collected in the first test are presented in a report, "Laboratory Applications of Radioisotopic Tracers to Follow Beach Sediments", by N. E. Taney in the Proceedings of the 8th Conference on Coastal Engineering, Council on Wave Research, Engineering Foundation (1963). Particle velocity in the second test was as great as 15 feet per minute. Maximum depth of burial of tracer particles, determined from a relatively small number of cores, was 0.1 foot. Analysis of the data gathered from the fluorescent samples is still underway with particular attention now being applied to determination of size of sample and number of samples required in such tests to give statistical validity to the resulting grain count contours.

(e) Measurement of Suspended Material in Laboratory Wave Tanks.

Additional suspended samples were obtained under wave action using a pump-type sampler in a wave flume testing crushed coal rather than sand. It is hoped that these measurements may aid in defining scale relations by comparison with prototype measurements. An eductor type sampler was tested in conjunction with other tests in the large tank, and found satisfactory. It provides certain advantages and convenience of use over the previously used centrifugal pump-type sampler.

(f) Equilibrium Profile, Beach Deformation, and Model Scale Effect Studies.

Testing was continued in a small tank using low specific gravity crushed coal to study the effect of scale on movable bed models under

wave action. Some of the tests were repeated with a shorter length of tank, to correspond to the prototype tank length for tests involving a 0.4-millimeter sand. Use of this length will, by comparison with results using a previous tank length, permit an evaluation of the comparability of tests in the prototype tank with 0.2 and 0.4-millimeter sand where different tank lengths were used for each sand. Comparison of the results of the tests in the big tank with the two sands of different size was made to enable rapid estimates of the greater width of beach fill required for smaller grain sands to provide protection equivalent to that for coarser sands. Application was necessary for emergency protection following the March 1962 Atlantic coast storm. More complete analysis of the data is now underway.

(g) Wave Measurements and Analysis.

The cooperative visual surf observation program was continued by the Research Division of the Board. This program consisted originally of 27 observation stations distributed along the coasts of the continental United States which were operated in cooperation with the U. S. Coast Guard. Visual observations of surf characteristics are made at 4-hour intervals by the Coast Guard and sent to the Beach Erosion Board weekly. The program was initiated in 1954 and has operated continuously since, although the number of stations has varied from time to time.

The instrumented wave gaging program was also continued and wave gage records were taken as continuously as possible at all field gage locations by the Board. The wave gage at Atlantic City, destroyed in the March 1962 storm, was reinstalled. The primary installation was the normal BEB relay-type step resistance gage, but several other gages were installed to afford comparison. Comparison is being made of pen-and-ink records, and also by spectral analysis of records obtained on magnetic tape. Of particular interest will be comparison of spectrum analyses for a pressure type gage installed near the sea bottom with those of the surface type step resistance gage.

A programming device has been developed and installed with certain reservoir gages. It permits continuous sampling of the waves, and when specific pre-set height is increased, the recorder is turned on for a pre-set duration. Sampling continues after this recording, but the recorder will not be turned on unless a second pre-set height is measured. If after a pre-set duration the sampled height has not reached this value, the recorder will be turned on if the height exceeds the first pre-set height. Any number of such height settings can be incorporated in the programmer to enable recording of all desired waves, but eliminate recording during periods of no interest.

A 45-foot gage has been installed on the Coast Guard tower at Buzzards Bay. This gage is of particular interest in that it may record either over the bottom 15 feet of the gage with recording plugs spaced

at 0.2-foot intervals, or over the entire 45-foot length of gage with recording plugs spaced at 1-foot intervals. It thus gives finer detail for the normal smaller waves, but also affords measurement of the very large waves associated with higher water levels during severe storms.

(h) Regional Studies.

Additional compilations are underway on geomorphological and littoral material data for the coastal sector from Cape Henlopen, Delaware, to Cape Charles, Virginia. The report on the geomorphology of this area is under preparation.

A comprehensive program was initiated late in FY 1962 to develop data on offshore deposits of material along the Atlantic coast that could be utilized in beach fill and nourishment projects. The study of the offshore deposits was intended to encompass all factors pertinent to the use of such deposits in the beach zone. A pilot study for the offshore program was completed in the Miami Beach - Palm Beach, Florida area during August 1962. Comparative profiles and geophysical exploration of the shallow sub-bottom strata were studied to locate potential offshore borrow sites. Surface samples of the bottom sediments were obtained by the dredge ESSAYONS in the vicinity of Hillsboro Inlet and Bakers Haulover. A further pilot study involving sediment core sampling as well as geophysical exploration of the bottom is planned early in FY 1964, probably for an area off the middle Atlantic coast.

(i) Re-examination of Beach Protection Projects.

A continuing program is being carried out on the re-examination of beach fill and periodic nourishment projects to determine the effectiveness of the fill material within the beach zones, and to better establish the factors upon which the desired characteristics of fill material are based. Continuing studies of other projects constructed following Beach Erosion Control studies are also underway to determine effectiveness of various structure components. In particular data have been obtained on behavior of fill placed at Seaside Park and Sherwood Island State Park, Connecticut, and at Key West, Florida. These data are being compiled and a report is under preparation.

Over the past 25 years many ground level photographs have been taken of beaches on the New Jersey coasts. This photographic data has been brought up to date as of 1962, and a pictorial history type report is essentially completed.

Data collection program has also been carried out at the Port Mansfield, Texas, entrance channel. The navigation project at Port Mansfield involves a jettied entrance with a channel cut through the offshore barrier (Padre Island) into Laguna Madre and on to Port Mansfield. The data collection program involves both shoaling and tidal data. Tidal elevations

are recorded at several points in the channel and also in the lagoon. Some small amount of tidal current data has also been obtained.

(j) Experimental Studies, Effectiveness of Sand Fences.

In cooperation with the State of North Carolina and the Wilmington District of the Corps of Engineers, a study has been underway on Core Bank, a part of the Outer Banks of North Carolina, on the effectiveness of various types of sand fence in building and stabilizing dunes. The portable wind station in the fence area has been operating more or less continuously, and has recorded wind data through two hurricanes. The drive system for the recorder has been converted from a spring-would system to an electric motor powered by a gas thermo-couple generator; a battery system has also been installed as standby in case the gas generator stops operating. Installation of tide gages on both the sound and sea side of the Banks has been considered, but not yet carried out. With the filling of the first fences installed, an additional 2,000 feet of fence was installed to give further information on the rate of dune growth, and the effectiveness of various types of fencing. This fencing was installed on top of the dune generated by the earlier fences. Roughly half of this new fencing was destroyed during a heavy storm in the spring of 1963, but this has since been replaced. Periodic profiles were again taken of the accumulation area of the fences to indicate effectiveness of the various types. A report "Experimental Study of Dune Building with Sand Fences" by R. P. Savage was published in the Proceedings of the 8th Conference on Coastal Engineering, Council on Wave Research, Engineering Foundation (1963).

(k) Study of Proposed Los Angeles-Santa Monica Causeway.

Several proposals have been made to the State of California for construction of an offshore causeway across a portion of Santa Monica Bay. One of these proposals involves an offshore sand barrier which would serve not only as a causeway, but also provide a large recreational beach frontage on the ocean side, and a large area of marina facilities on the shore side. One of the proposals for such a sand barrier also involves a submerged rock barrier seaward of the sand barrier itself to hold the seaward end of the barrier. The rock barrier would be submerged to perhaps 15 feet to prevent damage to swimmers and small craft. This rock barrier thus would eliminate the large quantity of sand otherwise necessary to form the toe of the sand barrier. This perched beach type of construction is virtually unknown, and an abbreviated set of model tests was carried out to give an indication of the effect of the submerged rock barrier in inducing scour adjacent to and landward of it, and also some indication of the proportion of the scoured material which might be expected to move seaward across the barrier and thus be lost to the shore regime. The model was to a 1 to 2 scale, and it involved waves of 3 to 6-foot model height. The test series was quite abbreviated, and a much fuller set of data would be desirable should such a proposal ever reach design stage.

However, the tests indicated that the amount of scour depends quite critically on wave height, and that the direction of movement of the scour material depends primarily on sand size (being greater offshore with finer sands) and wave period (being greater offshore with shorter periods).

(l) Wave Transmission Over Low-Crested Breakwaters.

A study was made for the Navy Bureau of Yards and Docks of wave transmission over certain proposed low-crested breakwaters which would permit considerable overtopping of storm waves. Tests involved only rubble-mound breakwaters, but both permeable and impermeable breakwaters were tested, thus affording a comparison of the amount of energy transmitted through the breakwater as opposed to that transmitted over the crest of the breakwater. A report discussing the tests for the Navy is under preparation, and it is hoped that it can be published in 1964. It is planned to follow these tests with additional small scale tests on submerged breakwaters of both rubble and vertical pile types.

A small test of similar type was also carried out for the Galveston District of the Corps of Engineers and is reported in an article, "Model Study of Offshore Wave Tripper" by F. F. Monroe.

(m) Beach Vulnerability as Related to Storm Wave and Water Level Conditions.

Following Congressional discussion of the March 1962 Atlantic storm, investigations have been initiated by various Government agencies on the improvement of storm warning data. Responsibility for issuance of storm warnings is that of the Weather Bureau, but the Weather Bureau and the Corps of Engineers have initiated a study to provide data on the vulnerability of shore areas for particular storm waves and tides which might occur from a particular storm. It is recognized that the Weather Bureau may be able to forecast relatively accurately the water levels and waves which may occur from a particular storm, but the damage which may result from these water levels and waves will then depend primarily on the condition of the elevation and width of the beach area on which it impinges. The beach condition, and hence the shore vulnerability, is different for different locations and also varies from time to time at any given location. Study has now been initiated on several beaches in the New Jersey, New York, and New England area in an attempt to relate observed beach changes and/or damage to observed water level and wave conditions, with the aim of accumulating data which may be used to prepare vulnerability charts for different types of beaches in other areas.

(n) Interlocking Precast Concrete Block Seawall Study.

A study is being made of various types of interlocking precast concrete blocks for use as seawall protection. It is felt that such type walls might afford relatively inexpensive backshore protection for severe storms, although they may not be expected to substitute for normal protection in the immediate normal wave action zone. Field installations

of various types are being examined, with an eye to their effect on the fronting and adjacent shores, and their stability. A particular type of block resulting in a stepped seawall has been designed and is being investigated at the Board Laboratory. Run-up tests have been made, and are discussed earlier in this report (see paragraph b), and stability tests are now underway. Stability tests are being carried out for two beach conditions, one a normal condition with a beach fronting the seawall and the wave breaking somewhat offshore, and the other an extreme storm condition at which considerable scour is hypothesized to have taken place in front of the seawall, and the wave breaks directly on the wall. A preliminary report discussing the data to date "Interim Report on Interlocking Precast Concrete Block Seawall Study" by R. A. Jachowski and J. R. Byerly has been given rather limited distribution. Testing is continuing, and a more comprehensive report will be prepared when more conclusive data are available.

(o) Propagation of Mechanically Generated Waves.

A short series of tests have been made on the propagation down a wave tank of the first waves generated by a wave generator. Normally used methods predict each wave to move down the tank at a constant speed dependent on the period of the generator and the water depth. In each wave length of travel, each wave leaves behind it a particular portion of its energy (again dependent only on the wave period and depth), transmitting the rest forward with the wave form. Such has not been found the case. The waves are more dispersive in character, the first wave having a longer period than that of the generator, and travelling at a greater velocity. The velocity of each successive wave decreases until finally the velocity appropriate to the period of the wave generator is reached, after which both wave period and speed remain constant. The wave height at a point increases with each successive wave until its full height is reached, after which time it also remains constant. The rate increase is faster than would be predicted in the way discussed above, and the energy front arrives earlier. When the wave generator is stopped abruptly (so that no extraneous longer period waves are generated by "coasting" of the generator blade), the waves do not die down gradually, but cease relatively abruptly.

## XV Publications

Technical Memoranda published by the Board during fiscal year 1963 are listed below. Copies can be furnished on request to persons within the United States to the extent of a limited printing.

| T. M. No. | Title and Date |
|---|---|
| 131* | Littoral Studies Near San Francisco Using Tracer Techniques, November 1962. |
| 132 | Waves in Inland Reservoirs (Summary Report on Civil Works Investigation Projects CW-164 and CW-165) November 1962. |

*This number is already out of print.

133 Higher Approximation to Nonlinear Water Waves and the Limiting Heights of Cnoidal, Solitary, and Stokes' Waves, February 1963.

134 Beach Profile as Affected by Vertical Walls, June 1963.

135 The Relationship Between Watershed Geology and Beach Radioactivity, August 1963.

Material covered by Technical Memoranda listed above, excepting numbers 132 and 135, is briefly described in foregoing paragraphs II and IV of Research Progress, or in the section of Research Progress in Volume 16, July 1962, of the Annual Bulletin of the Beach Erosion Board. The work covered in Technical Memorandum No. 132 was carried out by several U. S. Army Corps of Engineers' installations, including the Beach Erosion Board, under the CWI program related to design, construction, and operation of flood control, navigation, and multiple purpose projects involving major reservoirs, levees and channel improvements. An abstract of its contents follows:

> The report summarizes wave observations in Fort Peck Reservoir and Lake Texoma, the latter formed by Denison Dam. It briefly reviews certain investigational programs and publications pertinent to wave study in inland reservoirs, and summarizes analytical studies made to adapt, modify or supplement procedures currently used in estimating wave characteristics corresponding to wind and related factors to conform with observations in the two reservoirs. It presents procedures for quantitatively determining wave characteristics, and briefly outlines additional investigations needed to further improve methods and criteria. Certain general guidance and approximate formulas are presented for interim use in estimating wind tide effects in deep reservoirs.

The work described in Technical Memorandum No. 135 was done at the University of California, supported by an Atomic Energy Commission grant, but because of a close relationship to the Beach Erosion Board's research program it was published in the Technical Memoranda series. An abstract of the contents follows:

> Correlation between watershed geology of the Ben Lomond Mountain area, north of Santa Cruz, California, and radioactivity of beaches receiving sediment from the watershed is attempted. Radioactivity of stream and littoral sediments are presented in terms of thorium concentration determined by gamma-ray spectroscopy. Results are inconclusive regarding watershed geology - beach radioactivity relationship. Radiometric determination for littoral samples are remarkably constant with low activity indicated, but considerable variation in thorium content of stream sediment is found which is not consistent with any known geological variation. It is concluded that studies of geological maps and petrographic descriptions are not sufficient to determine applicability of the radioactive tracer technique.

## BEACH EROSION STUDIES

Beach erosion control studies of specific localities in the United States and its territories are usually made by the Corps of Engineers under provisions of Section 2 of the River and Harbor Act approved 3 July 1930 and amended by Public Law 87-874 approved 23 October 1962. Prior to the 1962 amendments to the law, beach erosion control studies could be authorized for specific problem areas by the Chief of Engineers under authority of the Secretary of the Army and were made in cooperation with appropriate agencies of the various States. By executive ruling the costs of these cooperative studies were divided equally between the United States and the cooperating agencies. The law as presently amended provides that beach erosion control studies now be made as Federal surveys wholly at Federal expense and such surveys be authorized by Resolution of the Public Works Committee of either the U. S. Senate or House of Representatives. Those studies still in progress at the time of adoption of the amendments will be completed on the cooperative basis, but new studies are authorized by Resolution as Federal surveys. Information concerning the initiation of such studies may be obtained from any District or Division Engineer of the Corps of Engineers. After a report on a beach erosion control 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 lists of completed and authorized studies follow.

## SUMMARIES OF REPORTS TRANSMITTED TO CONGRESS

### ROCKPORT, MASSACHUSETTS

The purpose of the investigation was to determine the best method of restoring the beach and protecting the beach and cottage development. The study area, located on the Atlantic Ocean shore in Essex County, Massachusetts, about 30 miles northeast of Boston, comprises about 1.5 miles of the southeast shore of Cape Ann between Brier Neck and Lands End, about 500 feet of which is in the city of Gloucester and the remainder in the town of Rockport. In 1960 the permanent population of Essex County was about 569,000. The population of Rockport and Gloucester is about 30,000, but is greatly increased by summer residents. The study area included three barrier pocket beaches known as Long Beach, Cape Hedge Beach and Pebbly Beach. Most of the shore is in public ownership, but the backshore lots at Long Beach are leased for private use which limits the access to the public beach. Long Beach is developed with resort commercial enterprises along the private frontage in Gloucester and summer cottages along the remaining frontage. The developed areas in Rockport are protected by a continuous concrete seawall, portions of which were rebuilt in 1959 after severe damage in an April 1958 storm. A narrow tidal creek separates Long Beach from Cape Hedge Beach to the east. There is no development at Cape Hedge Beach, but there is some development consisting of inns and residences at the privately owned rocky promontory between this beach and

Pebbly Beach to the east. A natural barrier consisting of a high-peaked shingle ridge backs the western half of Cape Hedge Beach and gradually flattens to the east. An unpaved road paralleling Pebbly Beach is separated from the beach by a low shingle or cobble ridge and there is residential development at Lands End at the eastern end of this pocket beach.

The shores of the study area are directly exposed to waves from the Atlantic Ocean from the southeast quadrant but are sheltered in varying degrees from other directions. Tides are semi-diurnal, the mean and spring ranges being respectively about 8.6 and 10 feet. Maximum tides are estimated at about 13 feet above mean low water. There is little evidence of a predominant direction of littoral drift, but rather that principal material movement is offshore and onshore. However, such evidence as is available indicates the predominant direction of alongshore movement would be toward the northeast. Sources of beach material have been the eroding rocky headlands and glacial overburden. Depletion of this material has reduced the supply, but the pocket beaches are reasonably stable. Widening of the beaches may be accomplished by artificial placement of sand. The rate of loss of beach material is presently low, but would probably be greater from a widened beach.

The problem causing concern to the cooperating agency consisted of erosion of the beaches, particularly during storms, and damages to existing protective structures and development due to wave attack. Undermining or overtopping of beaches by wave run-up with accompanying deposition of beach material and debris on adjacent roads and developed areas also occurs along the unprotected frontage. At Long Beach a tidal drainage creek meanders across the east end of the beach. Reconstruction of the failed portions of the seawall at Long Beach and placement of a stone apron in front of its seaward face affords adequate protection to the Long Beach developed areas, but the cooperating agency desired a plan for improving the beach for possible future use.

The Division Engineer developed a plan for improvement of Long Beach consisting of widening the beach to a 150-foot width between the seawall and mean high water and construction of a training jetty at the mouth of the tidal creek, and found that overtopping of the beach and transport of beach material onto backshore improvements at Cape Hedge Beach and Pebbly Beach can be prevented or reduced by construction of barriers to landward movement. The Division Engineer and Beach Erosion Board concluded that the improvement and protective plans considered cannot be justified by evaluated benefits and recommended that no project be adopted at this time by the United States for the protection or improvement of Long, Cape Hedge or Pebbly Beaches at Rockport, Massachusetts. They further recommended that protective measures which may be undertaken by local interests, based upon their own determination of economic justification, be accomplished in accordance with plans and methods considered in this report. The Chief of Engineers concurred in the views and recommendations of the Beach Erosion Board.

## SALISBURY BEACH, MASSACHUSETTS

The purpose of the investigation was to determine the best method of restoring and protecting the beach, and protecting the beach development. Salisbury Beach is located in Essex County about 35 miles north of Boston. It is about 3.5 miles long, extending from the New Hampshire State line to the jettied mouth of Merrimack River. The shore area consists of a barrier beach ranging from 600 to 1,500 feet in width. Salisbury Beach is a popular summer resort with a permanent population of about 3,100. The estimated peak population on summer weekends is 12,000. The shore of the study area is publicly owned. The southerly 0.6 mile has been developed as a State beach reservation, but the remainder of the shore is backed by private development. The beaches are used for recreational purposes. The tides in the study area are semi-diurnal. The mean and spring ranges are respectively 8.3 and 9.5 feet. The estimated highest tides are 12.5 feet above mean low water. The shores of the study area are exposed to waves from the east with unlimited fetch. The fetch to the northeast and southeast are limited by the peninsulas of Nova Scotia and Cape Ann respectively. However, the greater energy of waves from the northeast quadrant cause a predominant southward littoral transport. The source of littoral materials is adjacent beaches to the north.

The Division Engineer studied the sources and movement of the beach material and the changes in the shore line and the offshore bottom, and found that accretion occurred to the shore line from 1953 to 1960, that existing beaches are generally adequate for protection of the beach development, but that, if recession of the shore occurs, restoration can be accomplished by direct placement of sand along the shores or in stockpiles to be distributed by wave action. He concluded that minor infrequent damages which have occurred do not warrant construction of protective works at this time, but that surveys should be made to determine trends of shore line and offshore depth changes and the need for beach restoration or protection. He recommended that no project be adopted by the United States for protection of Salisbury Beach and recommended further that future construction of buildings be limited to the area behind the general line of development and that protective measures which may be undertaken by local interests based on their determination of need be accomplished in accordance with methods discussed in his report. The Beach Erosion Board concurred in the conclusions and recommendations of the Division Engineer, and noted that minor damages to development features have resulted from their construction within the zone of seasonal or storm shore line changes, but in view of shore line accretion in recent years and the small amount of damages, no protective measures are warranted at this time. The Board noted further that the erosion which has deepened the nearshore and offshore zones since 1940 increases the vulnerability of the area to storm damages, and emphasized the importance of continued observation of profile changes to determine need of protective measures before that need becomes urgent. The Board also noted that periodic nourishment of Hampton Beach to the north will probably increase the supply of material to Salisbury

Beach, but provision for continuation of a supply to Seabrook and Salisbury Beaches must be included in case navigation improvements are constructed at Hampton Harbor entrance in the future.

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

## FORT MACON - ATLANTIC BEACH AND VICINITY, NORTH CAROLINA

The purpose of the investigation was to determine the best method of protecting the shores of the Fort Macon and Atlantic Ocean shore from further erosion. The study area comprises the Atlantic Ocean shore of Fort Macon and Atlantic Beach and vicinity for a distance of about 5 miles west from Beaufort Inlet. In 1960 the permanent population of the coastal area within 50 miles of the study area was about 202,000. About 9,250 feet of the shore frontage at Fort Macon is publicly owned, including the Beaufort Inlet shore; the remaining frontage of the study area, about 20,300 feet, is privately owned. The study area is located on a sandy barrier beach extending westward from Beaufort Inlet. Extensive dunes on the island have a maximum height of about 40 feet. Tides in the area are semi-diurnal with mean and spring ranges of 3.8 and 4.6 feet respectively. The highest water of record was 12.6 feet above mean low water, during Hurricane Donna in September 1960. Waves approach the shore from the southeast and southwest quadrants. The predominance of littoral drift appears to be westward, but reversals occur especially along the ocean shore of Fort Macon State Park due to tidal currents in Beaufort Inlet.

The District and Division Engineers and the Beach Erosion Board developed plans for restoring and protecting the shores of the area, and made economic analyses of proposed protective measures. They concluded that practicable plans for the restoration and stabilization of shores within the study area are as follows:

a. Fort Macon State Park. Restoring approximately 7,750 feet of beach by direct placement of sand fill, and constructing a groin, revetment and seawall;

b. Atlantic Beach and Vicinity. Restoring approximately 20,300 feet of beach by direct placement of sand fill.

Both plans include periodic nourishment to stabilize the restored beaches. The reporting officers and The Beach Erosion Board found that restoration and protection of the shores of Fort Macon State Park and Atlantic Beach and vicinity are justified by evaluated benefits. They further found that the nature and amount of benefits warrant Federal participation in protection of the shore of Fort Macon State Park and recommended adoption of a project by the United States authorizing, subject to certain conditions,

Federal participation by the contribution of Federal funds in amount of one-third of the first costs and periodic nourishment costs for a period of 10 years. As no local agency indicated ability to comply with requirements of local cooperation for protection of Atlantic Beach, they did not recommend a Federal project for that shore. The Chief of Engineers concurred in the views and recommendations of the Beach Erosion Board.

## VIRGINIA AND BISCAYNE KEYS, FLORIDA

The purpose of the investigation was to determine the best method of preventing further erosion and of maintaining and restoring the ocean beaches. The study area comprises the Atlantic Ocean shore between Government Cut, the jettied entrance to Miami Harbor, and Florida Point, the south end of Key Biscayne. The total length of shore frontage studied is about 8 miles. The problem area comprises the shores of Virginia and Biscayne Keys with a total length of about 6 miles. In 1960 the permanent population of Dade County was about 935,000. The entire shore frontage of Virginia Key and the northern half of the frontage of Key Biscayne are publicly owned. The study area is characterized by low sandy barrier beach islands. Tides in the area are semi-diurnal with mean and spring ranges of 2.5 and 3.0 feet respectively. A reported high water mark of 9.1 feet above mean low water occurred during a hurricane in 1926. Waves approach the shore from directions from northeast through east to southeast. The directions of waves are such as to produce a southward predominance of littoral drift during the winter and a northward predominance during the summer. The fetches in the wave generating area are limited by the islands of the Bahama group.

The District and Division Engineers developed plans for restoring and protecting the shores of the area, and made economic analyses of proposed protective measures. They concluded that practicable plans for the restoration and stabilization of shores within the study area are as follows:

a. Virginia Key - Restoring and widening approximately 1.8 miles of shore by direct placement of sand fill, three groins for deferred construction, if needed.

b. Key Biscayne - Restoring and widening approximately 4.3 miles of shore by direct placement of sand fill, two groins for deferred construction if needed.

The plan for Crandon Park, the county-owned shore of Key Biscayne, comprises beach restoration and widening along 1.9 miles of shore and one groin at its north end. This plan could be constructed separately or as part of the plan for the entire key. Both plans include periodic nourishment to stabilize the restored beaches. The District and Division Engineers found that restoration and protection of the shores of Virginia and Biscayne Keys are

justified by evaluated benefits, and further found the nature and amount of benefits warrant Federal participation. They recommended adoption of projects by the United States authorizing, subject to certain conditions, Federal participation by the contribution of Federal funds in an amount equal to one-third of the first costs and periodic nourishment costs for a period of 10 years of protecting the public shores of the two keys.

The Beach Erosion Board noted that the existing beaches at Virginia Key and Key Biscayne appear adequate in width to provide protection to backshore improvements under moderate storm conditions. Considering presently unused frontage and area landward of the beach, there appears to be sufficient area for recreational needs for many years. In view of these conditions the Board felt there is no justification for Federal participation in the cost of widening the beach in the near future. However, the Board believed that stabilization of the shores is desirable to prevent future beach losses, and that such stabilization can be accomplished by periodic nourishment with suitable sand as required. The Board recommended adoption of projects by the United States authorizing Federal participation by the contribution of Federal funds in amount of one-third of the costs of periodic nourishment of the shores at Virginia and Biscayne Keys, Florida. The plans comprise periodic nourishment of 1.8 miles of beach on Virginia Key and 1.9 miles of beach on Key Biscayne. The plans also include three groins on Virginia Key and one groin on Key Biscayne for deferred construction when experience indicates their justification. The Board recommended Federal participation in the costs of such periodic nourishment of the beach for an initial period of 10 years. The Board further recommended that protective measures which may be undertaken by local interests for the privately owned portion of the Key Biscayne shore, based on their own determination of economic justification, be accomplished generally in accordance with the method developed by the District Engineer.

The Chief of Engineers concurred generally in the findings of the Board and accordingly, recommended adoption of projects providing for Federal participation in the cost of periodic nourishment of 1.8 miles of beach on Virginia Key and 1.9 miles of beach on Key Biscayne for an initial period of 10 years, and in the cost of deferred construction of groins when experience indicates their justification, in accordance with the plans proposed by the Beach Erosion Board.

## SAN JUAN, PUERTO RICO

The purpose of the investigation was to determine methods of preventing further erosion and stabilizing or restoring the beach, with special emphasis on protecting existing upland properties and future recreational, industrial or residential areas, and on reclaiming some eroded land at various locations. The study area comprises the Atlantic Ocean shore of Puerto Rico between Punta Salinas and Punta Vacia Talega. The total length

of shore frontage studied was about 24 miles. In 1960 the permanent population of the coastal area was about 600,000. About 56 percent of the shore frontage of the area is publicly owned. The western part of the study area is characterized by rocky headlands between which are the deeply indented bays, Ensenada de Boca Vieja and Bahia de San Juan. The eastern part consists generally of low sandy beaches. Tides in the area are semi-diurnal with mean and spring ranges of 1.1 and 1.3 feet respectively. The recorded high water was 2.8 feet above mean low water, but a hurricane stage of 4 to 6 feet in September 1960 was reported. Waves approach the shore principally from directions from east through north-northeast, resulting in a general, but minor westward predominance of littoral drift east of the entrance to Bahia de San Juan. West thereof in Ensenada de Boca Vieja the direction of drift appears to alternate with little, if any, predominance in either direction. Deeply eroded embayments prevent normal westward drift, so that there appears to be a general deficiency in supply, except in the most easterly section.

The District and Division Engineers developed plans for restoring and protecting the shores of the area, and made economic analyses of proposed protective measures. They concluded that practicable plans for the restoration and stabilization of shores within the study area are as follows:

a. Ensenada de Boca Vieja - Restoring approximately 1 mile of beach by direct placement of sand fill;

b. Catano - Restoring approximately 1.4 miles of beach by direct placement of sand fill and protecting El Canuelo by rubble;

c. Condado and Ocean Park - Restoring approximately 0.9 mile of beach by direct placement of sand fill and construction of a breakwater 129 feet long at the west end of the reach;

d. Isla Verde and International Airport - Restoring approximately 1.3 miles of beach by direct placement of sand fill;

All plans include periodic nourishment to stabilize the restored beaches. The District and Division Engineers and the Beach Erosion Board found that restoration and protection of the Condado-Ocean Park shore are justified by evaluated benefits. They further found the nature and amount of benefits warrant Federal participation, and recommended adoption of a project by the United States authorizing, subject to certain conditions, Federal participation by the contribution of Federal funds in an amount equal to 8.2 percent of the first costs and periodic nourishment costs for a period of 10 years. The Beach Erosion Board emphasized the importance of sand supply to the stability of the shore of Playa de las Tres Palmitas and pointed out that continued removal of sand for commercial purposes from that shore and from the mouth of Loiza River will result in future shore recession in that reach. The Chief of Engineers concurred in the views and recommendations of the Beach Erosion Board.

## CLARK POINT, NEW BEDFORD, MASSACHUSETTS

The purpose of the investigation was to determine the best method of restoration and stabilization of the city beaches along Rodney French Boulevard on the east and west sides of the Clark Point Peninsula. New Bedford is located in Bristol County about 50 miles south of Boston. The shores studied are located on the east and west sides of Clark Point, a peninsula projecting into Buzzards Bay. The peninsula consists of glacial deposits. Beaches fronting the low banks are narrow. The total length of the study area is about 3 miles, including about 1 mile of frontage on the outer tip of the point occupied by Fort Rodman, a Federal military reservation. New Bedford is a residential and industrial community with a permanent population of about 102,000. The shore of the study area is publicly owned except for about 0.1 mile on the east side of the point. The beaches are used for recreational purposes. The tides in the study area are semi-diurnal. The mean and spring ranges are respectively 3.7 and 4.6 feet. The maximum tide of record, 14.2 feet above mean low water, occurred during the hurricane of September 1938. Tides in excess of 3 feet above mean high water occur about once in 2 years. The shores of the study area are exposed to waves up to about 6 feet high from the south generated in the limited fetch of Buzzards Bay. This portion of Buzzards Bay is cut off from the full fetch of the Atlantic Ocean by the Elizabeth Islands and Martha's Vineyard. Beach material has been supplied to the shore of the study area by northward littoral transport from erosion of Clark Point headland. Protection of the headland shore has reduced this supply with resultant erosion of the beaches.

The Division Engineer developed plans for restoration and stabilization of the beaches. He concluded that practicable plans for protection and improvement of the shores where recreational beaches are required comprise sand fills and groin construction, and that for shores where no fronting beach exists, stone revetment or rubble-mound wall construction is practicable. He developed two alternative plans for protecting and improving Rodney French Boulevard West Beach; alternative No. 1 comprises raising the inshore end of an existing groin, lengthening two existing groins, and widening the beach by direct placement of sand fill; alternative No. 2 comprises raising the inshore end of the same existing groin, constructing three new intermediate groins, and widening the beach by direct placement of sand fill. He also developed a plan comprising beach fill and two new groins for improving Rodney French Boulevard East Beach, and typical plans comprising stone revetment or rubble-mound wall for critical sections along Rodney French Boulevard West where no beaches exist. The Division Engineer concluded that work at the west beach is justified by evaluated benefits but that the public interest in projects for the east beach or other city-owned areas is insufficient to warrant Federal aid. He recommended a project providing for Federal participation in amount equal to one-third of the first costs of either alternative for the west beach, subject to certain conditions.

The Beach Erosion Board concurred generally in the conclusions of the Division Engineer that either the groin extension plan or the new groin plan would be a practicable plan of improvement for Rodney French Boulevard West Beach, and that either plan is amply justified by prospective benefits. However, the Board noted that the State expressed a preference for the groin extension plan. As that plan would provide greater beach area and additional benefits, making the greater cost of the plan incrementally justified, the Board concurred in the State's preference for that plan, and recommended adoption of a project by the United States authorizing Federal participation by the contribution of Federal funds in amount of one-third of the first costs of measures for the restoration and protection of the publicly owned shore at Rodney French Boulevard West Beach, New Bedford, Massachusetts, substantially in accordance with the groin extension plan of the Division Engineer, with such modifications thereof as may be considered advisable by the Chief of Engineers. The recommended plan comprises widening approximately 1,600 feet of beach to a minimum width of 100 feet by direct placement of suitable sand fill, raising the inshore end of the existing groin at Dudley Street, and extending two existing groins. The Chief of Engineers concurred in the views and recommendations of the Beach Erosion Board.

## HILLS BEACH, BIDDEFORD, MAINE

The purpose of the investigation was to determine the best method of restoration of protective and recreational beaches and protection of shore property. Biddeford is located on Saco River in the northeast part of York County, Maine. It has a permanent population of nearly 20,000 and a somewhat greater summer population. Saco River empties into the south end of Saco Bay, which is about 7 miles in length and has a maximum width of 3 miles. Hills Beach is privately owned. It is developed with about 200 cottages, some of which are used as year-round residences. Hills Beach lies immediately south of the jettied mouth of Saco River. The study area extends about 1½ miles in a general southeasterly direction from the mouth of Saco River to the entrance to The Pool. The area is a low sandy barrier beach from 200 to 1,000 feet wide between Saco Bay and The Pool. The tides at Hills Beach are semi-diurnal. At Wood Island Harbor, just east of the beach, the mean and spring ranges are respectively 8.7 and 9.9 feet. Waves affecting the Hills Beach shore approach from between east and northeast and cause littoral drift principally toward the northwest across the Saco River south jetty. The problem is erosion and loss of the protective beach thus exposing the development to storm wave attack. This erosion has resulted in extensive damages to seawalls and bulkheads previously constructed by local interests to protect upland property.

The Division Engineer developed plans for protecting the shore of the problem area. He concluded that widening approximately 5,400 feet of beach by direct placement of sand fill and raising 700 feet of the inshore end of

the Saco River south jetty is a practicable method of restoring and protecting Hills Beach, and that an alternative method of protecting individual properties consists of constructing stone revetments in front of existing seawalls or bluffs. The Division Engineer and Beach Erosion Board found that due to the private ownership of the shore and lack of public benefits, Hills Beach is not eligible for Federal assistance in construction of protective works. However, the Division Engineer indicated that placement of suitable sandy spoil on the beach may be possible in connection with future maintenance of Federal navigation projects for Saco River and The Pool at Biddeford. They recommended that no project be adopted at this time by the United States for the protection of the shore of Hills Beach. They further recommended that protective measures which may be undertaken by local interests, based upon their own determination of economic justification, by accomplished in accordance with plans and methods considered in this report. Local interests stated an opinion that the United States is responsible for a substantial portion of the erosion which has occurred at Hills Beach, especially since 1954, as a result of failure to maintain the existing south jetty at the entrance to Saco River, and also that the United States should undertake restoration of the jetty to its as-built condition and replacement of that amount of shore material determined to have been eroded as a result of the failure to maintain the south jetty, at no cost to local interests. The Beach Erosion Board believed that the study did not establish that the recent erosion of Hills Beach can be attributed to deterioration or lack of maintenance of the south jetty. It also believed that the United States has neither obligation nor authority to maintain the existing Saco River south jetty, except in the interest of navigation. The Board stated that in any event restoration of the jetty to its as-built elevation (5.5 feet above mean low water as far as can be determined from available records) would result in only minor accretion to the adjacent shore to the south and would thus not solve the Hills Beach erosion problem. The Board does not favor piecemeal protection of separate frontages by individual owners, but concurred in the plan for beach fill and jetty raising as the most suitable plan of restoring and protecting the entire beach. The Chief of Engineers concurred in the views and recommendations of the Beach Erosion Board.

## SAN GABRIEL RIVER TO NEWPORT BAY, ORANGE COUNTY, CALIFORNIA

The purposes of the investigation were to determine the causes and most effective and economical methods of controlling erosion of the shore, and the extent of Federal aid which should be granted to the Anaheim Bay area in equity without regard to limitations of Federal law applicable to beach erosion control. Orange County is in Southern California immediately south of Los Angeles County. Its Pacific Ocean shore line, extending in a general northwest-southeast direction, is about 42 miles long, the portion of which covered by this report being about 17 miles long from the mouth of San Gabriel River to the entrance to Newport Bay. The coastal area

consists generally of sandy beaches backed by tidal lagoons and marshes, except at Seal Beach and Huntington Beach where plateaus 10 to 30 feet above sea level lie behind the beaches. The principal shore communities in the study area are Seal Beach, Huntington Beach and Newport Beach, but the population of Orange County and adjacent counties to which Orange County beaches are readily accessible is over 6,000,000. Of the shore frontage, 13.64 miles or 81.8 percent is publicly owned. The principal publicly owned sections are Bolsa Chica Beach and Huntington Beach State Parks and the municipal beaches in Huntington Beach and Newport Beach. The tides in the study area have a diurnal inequality, the mean and diurnal ranges being respectively about 3.7 and 5.3 feet. The maximum tide each year is about 7 feet above mean lower low water. Characteristic waves are swells generated in distant ocean areas. They have heights up to 10 feet and periods up to 20 seconds with the greater heights and shorter periods occurring in the winter. Winter waves generally approach the shore from upcoast of normal, summer waves frequently approach from downcoast of normal. As a result the predominant direction of littoral transport is generally toward the southeast. However, Seal Beach is protected from westerly waves by offshore breakwaters and the predominant direction of drift is now northwestward. Sand has been supplied to the shore by tributary streams, especially during storm runoff, but the supply has been greatly reduced by construction of dams on those streams.

The District and Division Engineers concluded that the most suitable plan of shore protection for the problem area from Surfside to Newport Beach comprises a protective beach in the vicinity of Surfside-Sunset Beach generally 500 feet wide and 9,200 feet long to be provided by artificial placement of approximately 3,000,000 cubic yards of suitable sand on the beach, construction of an offshore breakwater at Newport Beach and providing periodic artificial nourishment of the Surfside-Sunset Beach shore by transferring sand from the impoundment area of that breakwater. They made an economic analysis of the plan of protection for the shore from Surfside to Newport Beach and concluded that the plan is justified by prospective benefits, and that in equity Federal assistance to the extent of 61 percent of the costs of protection of these shores is warranted. The District and Division Engineers recommended modification of the existing project to authorize Federal participation, subject to certain conditions, by contribution of funds in amount of 61 percent of the first costs and periodic nourishment and maintenance costs of the project.

Residents of Newport Beach objected generally to construction of a breakwater off Newport Beach on the basis that there is presently no erosion problem at Newport Beach and that protective construction should preferably be located at Surfside, the site of the present erosion problem. Some also indicated that surf action at Newport Beach would be reduced and that use of the sheltered area behind the breakwater for navigational purposes would be objectionable.

The Beach Erosion Board considered the apportionment of costs in equity as derived by the District Engineer and considered that the Anaheim Bay jetties were of equal importance with the flood control and navigation improvements in contributing to erosion of the Orange County beaches. Accordingly the Board recomputed the extent of Federal aid in equity as 67 percent instead of 61 percent of the costs. The Board therefore recommended that in lieu of the existing project for Surfside, California (Anaheim Bay Harbor) a project be adopted by the United States authorizing Federal participation by the contribution of Federal funds in amount of 67 percent of the first costs and costs of periodic nourishment and maintenance of a plan comprising restoration of the beach in the Surfside-Sunset Beach area, a feeder beach for nourishing the shore from Surfside to Newport Beach, and an offshore breakwater at Newport Beach to provide an impounding area from which sand would be transferred periodically to the feeder beach at Surfside, substantially in accordance with the plan of the District Engineer, with such modification thereof as may be considered advisable by the Chief of Engineers. The Chief of Engineers concurred in the views and recommendations of the Beach Erosion Board.

## RECOMPUTATION OF FEDERAL SHARE OF CONSTRUCTION COSTS UNDER PROVISIONS OF PL 87-874

The 1962 amendments to beach erosion law increased the permissible Federal share of construction cost for regular beach erosion control projects from one-third to one-half and provided further that Federal participation in the cost of projects for restoration and protection of State, county, and other publicly owned parks and conservation areas may be as high as 70 percent of the total cost. These revised cost-sharing provisions were made applicable to existing authorized projects not substantially completed as of the date of approval of the amendments (23 October 1962) and the Chief of Engineers through the Beach Erosion Board was directed to recompute the extent of Federal participation in the costs of those projects accordingly. On this basis the level of Federal participation was recomputed and approved by the Chief of Engineers for projects authorized from two of the foregoing eight summarized reports as follows:

Fort Macon-Atlantic Beach and Vicinity, N. C. - From 1/3 to 70 percent.

Virginia and Biscayne Keys, Florida - From 1/3 to 70 percent.

In addition a pending recomputation of the level of Federal participation for the authorized project resulting from the report on San Juan, Puerto Rico should result in some increase in the Federal share.

## COMPLETED COOPERATIVE BEACH EROSION STUDIES

| Location | BEB Report Completed | Published in H. Doc. | Published in Cong. | Federal Projects Recommendation * | Federal Projects Authorized by Congress |
|---|---|---|---|---|---|
| **ALABAMA** | | | | | |
| Perdido Pass (Alabama Pt.) | 18 Jun 54 | 274 | 84 | Unfav. | |
| **CALIFORNIA** | | | | | |
| Santa Barbara - Initial | 15 Jan 38 | 552 | 75 | Unfav.* | |
| Suppl. | 18 Feb 42 | | | | |
| Final | 22 May 47 | 761 | 80 | Unfav. | |
| Ballona Creek & San Gabriel R. (Partial) | 11 May 38 | | | Unfav.* | |
| Orange County | 10 Jan 40 | 637 | 76 | Unfav.* | |
| Coronado Beach | 4 Apr 41 | 636 | 77 | Unfav.* | |
| Long Beach | 3 Apr 42 | | | Unfav.* | |
| Mission Beach | 4 Nov 42 | | | Unfav.* | |
| Pt. Mugu to San Pedro BW | 27 Jun 51 | 277 | 83 | Fav. | 3 Sep 54 |
| Carpinteria to Pt. Mugu | 4 Oct 51 | 29 | 83 | Fav. | 3 Sep 54 |
| Oceanside, Ocean Beach, Imperial Beach & Coronado, San Diego County | 26 Jul 55 | 399 | 84 | Fav. | 3 Jul 58 |
| Santa Cruz County | 13 Sep 56 | 179 | 85 | Fav. | 3 Jul 58 |
| Humboldt Bay (Buhne Pt.) | 29 Mar 57 | 282 | 85 | Fav. | 3 Jul 58 |
| Newport Bay to San Mateo Creek, Orange County | 3 Dec 59 | 398 | 86 | Fav. | 14 Jul 60 |
| San Diego County | 30 Jun 60 | 456 | 86 | Fav. | 29 Mar 61 |
| Ventura | 28 Dec 61 | 458 | 87 | Fav. | 23 Oct 62 |
| San Gabriel River to Newport Bay, Orange Co. | 20 Apr 62 | 602 | 87 | Fav. | 23 Oct 62 |
| **CONNECTICUT** | | | | | |
| Compo Beach, Westport | 18 Apr 35 | 239 | 74 | Unfav.* | |
| Hawk's Nest Beach, Old Lyme | 21 Jun 39 | | | Unfav.* | |
| Ash Crk. to Saugatuck R. | 29 Apr 49 | 454 | 81 | Fav. | 17 May 50 |
| Hammonasset R. to East R. | 29 Apr 49 | 474 | 81 | Fav. | 3 Sep 54 |
| New Haven Hbr. to Housatonic R. | 29 Jun 51 | 203 | 83 | Fav. | 3 Sep 54 |

* No established policy for Federal participation in construction of shore protection works existed prior to 1946.

| Location | BEB Report Completed | Published in H. Doc. | Cong. | Federal Projects Recommendation * | Authorized by Congress |
|---|---|---|---|---|---|
| **CONNECTICUT (Cont.)** | | | | | |
| Conn. R. to Hammonasset R. | 28 Dec 51 | 514 | 82 | Unfav. | |
| Pawcatuck R. to Thames R. | 31 Mar 52 | 31 | 83 | Unfav. | |
| Niantic Bay to Conn. R. | 11 Jul 52 | 84 | 83 | Unfav. | |
| Housatonic R. to Ash Creek | 12 Mar 53 | 248 | 83 | Fav. | 3 Sep 54 |
| East R. to New Haven Hbr. | 15 Nov 55 | 395 | 84 | Fav. | 3 Jul 58 |
| Saugatuck R. to Byram R. | 14 Nov 56 | 174 | 85 | Fav. | 3 Jul 58 |
| Thames R. to Niantic Bay | 17 Jun 57 | 334 | 85 | Unfav. | |
| **DELAWARE** | | | | | |
| Kitts Hummock to Fenwick Is. | 11 Feb 57 | 216 | 85 | Fav. | 3 Jul 58 |
| **FLORIDA** | | | | | |
| Blind Pass (Boca Ciega) | 1 Feb 37 | 187 | 75 | Unfav.* | |
| Miami Beach | 1 Feb 37 | 169 | 75 | Unfav.* | |
| Hollywood Beach | 28 Apr 37 | 253 | 75 | Unfav.* | |
| Daytona Beach | 15 Mar 38 | 571 | 75 | Unfav.* | |
| Bakers Haulover Inlet | 21 May 45 | 527 | 79 | Unfav.* | |
| Anna Maria & Longboat Keys | 12 Feb 47 | 760 | 80 | Unfav. | |
| Jupiter Island | 13 Feb 47 | 765 | 80 | Unfav. | |
| Palm Beach (1) | 13 Feb 47 | 772 | 80 | Fav. | 17 May 50 |
| Pinellas County | 22 Apr 53 | 380 | 83 | Fav. | 3 Sep 54 |
| Palm Beach County (Lk. Worth Inlet to S. Lake Worth I.) | 12 Jul 57 | 342 | 85 | Fav. | 3 Jul 58 |
| Key West | 10 Mar 58 | 413 | 85 | Fav. | 14 Jul 60 |
| Amelia Island | 16 Aug 60 | 200 | 87 | Unfav. | |
| Palm Beach County | 23 Aug 60 | 164 | 87 | Fav. | 23 Oct 62 |
| Virginia & Biscayne Keys | 6 Apr 62 | 561 | 87 | Fav. | 23 Oct 62 |
| Broward Co. & Hillsboro Inl. | 23 Apr 63 | | | Fav. | |
| **GEORGIA** | | | | | |
| St. Simon Island | 18 Mar 40 | 820 | 76 | Unfav.* | |

(1) A cooperative study of experimental steel sheet pile groins was also made, under which methods of improvement were recommended in an interim report dated 19 Sep 1940. Final report on experimental groins was published in 1948 as Technical Memorandum No. 10 of the Beach Erosion Board.

| Location | BEB Report Completed | Published in H.Doc. | Published in Cong. | Federal Projects Recommendation * | Federal Projects Authorized by Congress |
|---|---|---|---|---|---|
| **HAWAII** | | | | | |
| Waikiki Beach | 5 Aug 52 | 227 | 83 | Fav. | 3 Sep 54 |
| Waimea & Hanapepe Bay, Kauai | 17 Jan 56 | 432 | 84 | Fav. | 3 Jul 58 |
| Haleiwa Beach, Oahu | 28 Feb 63 | | | Fav. | |
| **ILLINOIS** | | | | | |
| State of Illinois | 8 Jun 50 | 28 | 83 | Fav. | 3 Sep 54 |
| **LOUISIANA** | | | | | |
| Grand Isle | 28 Jul 36 | 92 | 75 | Unfav.* | |
| Grand Isle | 28 Jun 54 | 132 | 84 | Unfav. | |
| Belle Pass to Raccoon Point | 13 Jun 61 | 338 | 87 | Unfav. | |
| **MAINE** | | | | | |
| Old Orchard Beach | 20 Sep 35 | | | Unfav.* | |
| Saco | 2 Mar 56 | 32 | 85 | Unfav. | |
| Hills Beach, Biddeford | 27 Nov 61 | 590 | 87 | Unfav. | |
| **MASSACHUSETTS** | | | | | |
| South Shore of Cape Cod (Pt. Gammon to Chatham) | 26 Aug 41 | | | Unfav.* | |
| Salisbury Beach | 26 Aug 41 | | | Unfav.* | |
| Winthrop Beach | 12 Sep 47 | 764 | 80 | Fav. | 17 May 50 |
| Lynn-Nahant Beach | 20 Jan 50 | 134 | 82 | Fav. | 3 Sep 54 |
| Revere Beach | 12 Jan 50 | 146 | 82 | Fav. | 3 Sep 54 |
| Nantasket Beach | 12 Jan 50 | | | Unfav. | |
| Quincy Shore | 2 May 50 | 145 | 82 | Fav. | 3 Sep 54 |
| Plum Island | 18 Nov 52 | 243 | 83 | Unfav. | |
| Chatham | 22 Oct 56 | 167 | 85 | Unfav. | |
| Pemberton Pt. to Cape Cod Canal | 13 Jan 59 | 272 | 86 | Fav. | 14 Jul 60 |
| Wessagussett Beach, Weymouth | 6 Jul 59 | 334 | 86 | Fav. | 14 Jul 60 |
| Cape Cod Canal to Provincetown | 5 Feb 60 | 404 | 86 | Fav. | 14 Jul 60 |
| Clark Point, New Bedford | 14 Aug 61 | 584 | 87 | Fav. | 23 Oct 62 |
| Rockport | 21 Nov 61 | 515 | 87 | Unfav. | |
| Salisbury Beach | 5 Dec 61 | 517 | 87 | Unfav. | |
| Falmouth | 28 Feb 63 | | | Unfav. | |

| Location | BEB Report Completed | Published in H. Doc. | Published in Cong. | Federal Projects Recommendation * | Federal Projects Authorized by Congress |
|---|---|---|---|---|---|
| **MICHIGAN** | | | | | |
| Berrien County (St. Joseph) | 17 Jun 57 | 336 | 85 | Fav. | 3 Jun 58 |
| **MISSISSIPPI** | | | | | |
| Hancock County | 3 Apr 42 | | | Unfav.* | |
| Harrison County - Initial | 15 Mar 44 | | | | |
| Harrison County - Suppl. | 16 Feb 48 | 682 | 80 | Fav. | 30 Jun 48 |
| **NEW HAMPSHIRE** | | | | | |
| Hampton Beach | 15 Jul 32 | | | Unfav.* | |
| Hampton Beach | 14 Sep 53 | 325 | 83 | Fav. | 3 Sep 54 |
| Atlantic Ocean shore (entire) | 30 Jun 61 | 416 | 87 | Fav. | 23 Oct 62 |
| **NEW JERSEY** | | | | | |
| Manasquan Inlet & Adjacent Beaches | 15 May 36 | 71 | 75 | Unfav.* | |
| Atlantic City | 11 Jul 49 | 538 | 81 | Fav. | 3 Sep 54 |
| Ocean City | 15 Apr 52 | 184 | 83 | Fav. | 3 Sep 54 |
| Sandy Hook to Barnegat Inlet | 24 Mar 54 | 361 | 84 | Fav. | |
| Review Report - Sandy Hook to Barnegat Inlet | 6 May 57 | 332 | 85 | Fav. | 3 Jul 58 |
| Barnegat Inlet to Delaware Bay Entrance to Cape May Canal | 22 Sep 58 | 208 | 86 | Fav. | 14 Jul 60 |
| Delaware Bay Shore - Cape May Canal to Maurice River | 10 Jun 60 | 196 | 87 | Unfav. | |
| Raritan & Sandy Hook Bays | 2 Nov 61 | 464 | 87 | Fav. | 23 Oct 62 |
| Atlantic City | 25 Mar 63 | | | Fav. | |
| **NEW YORK** | | | | | |
| Jacob Riis Park, Long Island | 16 Dec 35 | 397 | 74 | Unfav.* | |
| Orchard Beach, Pelham Bay, Bronx | 30 Aug 37 | 450 | 75 | Unfav.* | |
| Niagara County | 27 Jun 42 | 271 | 78 | Unfav.* | |
| South Shore of Long Island | 6 Aug 46 | | | Unfav. | |
| Selkirk Shores State Park | 21 Oct 53 | 343 | 83 | Fav. | 3 Sep 54 |
| Fair Haven Beach State Park | 18 Jun 54 | 134 | 84 | Fav. | 3 Jul 58 |
| Hamlin Beach State Park | 20 Sep 54 | 138 | 84 | Fav. | 3 Jul 58 |

| Location | BEB Report Completed | Published in H. Doc. | Cong. | Federal Projects Recommendation* | Authorized by Congress |
|---|---|---|---|---|---|
| **NEW YORK (Cont.)** | | | | | |
| Braddock Bay State Park | 15 Apr 55 | | | Unfav. | |
| Fire Island Inlet to Jones Inlet | 10 Feb 56 | 411 | 84 | Fav. | 3 Jul 58 |
| Fire Island Inlet to Montauk Pt. (combined coop. BEC and HUR) | 30 Jun 59 | 425 | 86 | Fav. | 14 Jul 60 |
| **NORTH CAROLINA** | | | | | |
| Fort Fisher | 10 Nov 31 | 204 | 72 | Unfav.* | |
| Wrightsville Beach | 2 Jan 34 | 218 | 73 | Unfav.* | |
| Kitty Hawk, Nags Head & Oregon Inlet | 1 Mar 35 | 155 | 74 | Unfav.* | |
| State of North Carolina | 22 May 47 | 763 | 80 | Unfav. | |
| Carolina Beach & Vicinity | 10 Mar 61 | 418 | 87 | Fav. | 23 Oct 62 |
| Fort Macon-Atlantic Beach | 30 Apr 62 | 555 | 87 | Fav. | 23 Oct 62 |
| **OHIO** | | | | | |
| Erie County - Vic. of Huron | 26 Aug 41 | 220 | 79 | Unfav.* | |
| Michigan Line to Marblehead | 30 Oct 44 | 177 | 79 | Unfav.* | |
| Cities of Cleveland & Lakewood | 22 Mar 48 | 502 | 81 | Fav. | 3 Sep 54 |
| Chagrin River to Fairport | 22 Nov 49 | 596 | 81 | Unfav. | |
| Vermilion to Sheffield Lake Village | 24 Jul 50 | 229 | 83 | Fav. | 3 Sep 54 |
| Fairport to Ashtabula | 1 Aug 51 | 351 | 82 | Unfav. | |
| Ashtabula to Penna. St. Line | 1 Aug 51 | 350 | 82 | Unfav. | |
| Sandusky to Vermilion | 7 Jul 52 | 32 | 83 | Unfav. | |
| Sandusky Bay | 31 Oct 52 | 126 | 83 | Unfav. | |
| Sheffield Lake Village to Rocky R. | 31 Oct 52 | 127 | 83 | Unfav. | |
| Euclid to Chagrin River | 25 Jun 53 | 324 | 83 | Unfav. | |
| Michigan Line to Marblehead (Review) | 14 Jun 60 | 63 | 87 | Fav. | 23 Oct 62 |
| Sheffield Lake Community Park | 13 Jun 61 | 414 | 87 | Fav. | 23 Oct 62 |

| Location | BEB Report Completed | Published in H. Doc. | Published in Cong. | Federal Projects Recommendation* | Federal Projects Authorized by Congress |
|---|---|---|---|---|---|
| **PENNSYLVANIA** | | | | | |
| Presque Isle Peninsula, Erie | | | | | |
| (Interim) | 3 Apr 42 | | | | |
| (Final | 23 Apr 52 | 231 | 83 | Fav. | 3 Sep 54 |
| (Review) | 21 Jan 60 | 397 | 86 | Fav. | 14 Jul 60 |
| **PUERTO RICO** | | | | | |
| Punta Las Marias, San Juan | 5 Aug 47 | 769 | 80 | Unfav. | |
| San Juan | 3 May 62 | 575 | 87 | Fav. | 23 Oct 62 |
| **RHODE ISLAND** | | | | | |
| South Shore (Towns of Narragansett, South Kingstown, Charlestown & Westerly) | 4 Dec 48 | 490 | 81 | Fav. | 3 Sep 54 |
| South Kingstown & Westerly | 27 Jan 58 | 30 | 86 | Fav. | 14 Jul 60 |
| **SOUTH CAROLINA** | | | | | |
| Folly Beach | 31 Jan 35 | 156 | 74 | Unfav.* | |
| Pawleys Island, Edisto Beach & Hunting Island | 24 Jul 51 | | | Unfav. | |
| Hunting Island Beach | 9 May 63 | | | Fav. | |
| **TEXAS** | | | | | |
| Galveston (Gulf Shore) | 10 May 34 | 400 | 73 | Unfav.* | |
| Galveston Bay, Harris County | 31 Jul 34 | 74 | 74 | Unfav.* | |
| Galveston (Gulf Shore) | 5 Feb 53 | 218 | 83 | Unfav. | |
| Galveston (Bay Shore) | 19 Jun 53 | 346 | 83 | Unfav. | |
| Bolivar Peninsula (Gulf Shore and Rollover Fish Pass) | 8 Jun 59 | 286 | 86 | Unfav. | |

| Location | BEB Report Completed | Published in H. Doc. | Cong. | Federal Projects Recommendation * | Authorized by Congress |
|---|---|---|---|---|---|
| | VIRGINIA | | | | |
| Willoughby Spit, Norfolk | 20 Nov 37 | 482 | 75 | Unfav.* | |
| Colonial Beach, Potomac R. | 24 Jan 49 | 333 | 81 | Fav. | 17 May 50 |
| Virginia Beach | 25 Jun 52 | 186 | 83 | Fav. | 3 Sep 54 |
| Virginia Beach (Review) | 13 Jun 61 | 382 | 87 | Fav. | 23 Oct 62 |
| | WISCONSIN | | | | |
| Milwaukee County | 21 May 45 | 526 | 79 | Unfav.* | |
| Racine County | 5 Mar 52 | 88 | 83 | Unfav. | |
| Kenosha | 16 Sep 54 | 273 | 84 | Unfav. | |
| Manitowoc County | 15 Apr 55 | 348 | 84 | Fav. | 3 Jul 58 |

## CURRENTLY AUTHORIZED BEACH EROSION STUDIES
## (Cooperative and Federal Surveys)

Those studies authorized and in progress prior to 23 October 1962, the date of enactment of Public Law 87-874, were authorized, and are being continued, on the cooperative basis. Those studies authorized since that date are full Federal surveys, authorized by Resolution of the Public Works Committee of either the U. S. Senate or House of Representatives, under provisions of the amended law. A listing by States of all currently authorized studies follows:

| Locality | Date Auth. | Study Type |
|---|---|---|
| CALIFORNIA | | |
| Point Conception to Mexican Boundary (General study) | 6/12/58 | Coop. |
| City of San Diego (Special Study) | 5/17/62 | Coop. |
| Alamitos Bay (Special Study) | 5/17/62 | Coop. |
| Emma Wood State Park & vicinity (Special Study) | 5/17/62 | Coop. |
| Point Delgada to Point Ano Nuevo | 3/18/59 | Coop. |
| Daly City (San Mateo County) | 5/2/63 | Fed. Survey |
| Pacifica (San Mateo County) | 6/19/63 | Fed. Survey |
| El Grenada Beach | 6/19/63 | Fed. Survey |
| DELAWARE | | |
| Kitts Hummock to Fenwick Island (Review) | 1/7/63 | Fed. Survey |
| FLORIDA | | |
| Bakers Haulover - Miami | 5/24/60 | Coop. |
| Fort Pierce | 12/30/60 | Coop. |
| Duval County | 1/7/63 | Fed. Survey |
| St. John's County | 1/7/63 | Fed. Survey |
| Jupiter Island | 6/19/63 | Fed. Survey |
| Mullet Key | 6/19/63 | Fed. Survey |
| Pinellas County (Review) | 6/19/63 | Fed. Survey |
| GEORGIA | | |
| Sea Island & St. Simons Island | 4/29/63 | Fed. Survey |
| Tybee Island | 4/29/63 | Fed. Survey |
| HAWAII | | |
| State of Hawaii (General Study) | 7/17/62 | Coop. |
| Waikiki Beach (Review) | 8/25/59 | Coop. |
| Kihei, Maui | 4/10/63 | Fed. Survey |
| Kapaa, Kauai | 4/10/63 | Fed. Survey |

| Locality | Date Auth. | Study Type |
|---|---|---|
| ILLINOIS | | |
| Evanston (Review) | 8/11/59 | Coop. |
| MARYLAND | | |
| Worcester County | 6/19/63 | Fed. Survey |
| MASSACHUSETTS | | |
| Marthas Vineyard | 2/14/61 | Coop. |
| Nantasket & Revere Beaches | 9/19/61 | Coop. |
| NEW JERSEY | | |
| Perth Amboy | 2/2/61 | Coop. |
| Coastal Inlets of New Jersey | 12/18/61 | Coop. |
| NEW YORK | | |
| Staten Island | 3/23/59 | Coop. |
| Fire Island Inlet to Jones Inlet (Review) | 9/15/60 | Coop. |
| East Rockaway Inlet to Norton Point | 3/29/61 | Coop. |
| Jones Inlet to East Rockaway Inlet | 3/20/63 | Fed. Survey |
| Suffolk County (North Shore) | 3/20/63 | Fed. Survey |
| NORTH CAROLINA | | |
| Ocracoke Island | 10/29/57 | Coop. |
| Ocracoke Inlet to Cape Lookout | 12/16/59 | Coop. |
| OHIO | | |
| Lake Erie Shore: (Ashtabula to Lake County line) | 6/10/63 | Fed.Survey |
| RHODE ISLAND | | |
| Newport | 2/14/61 | Coop. |
| WASHINGTON | | |
| Tokeland | 5/15/62 | Coop. |
| Titlow Beach | 6/19/63 | Fed. Survey |