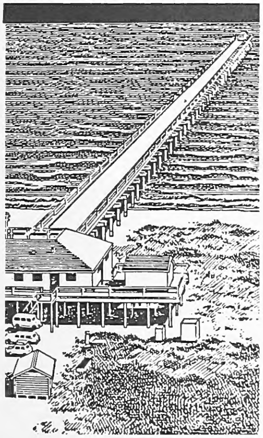
US Army Corps of Engineers



TECHNICAL REPORT CERC-86-1

DETACHED BREAKWATERS FOR SHORE PROTECTION

by

William R. Dally, Joan Pope

Coastal Engineering Research Center

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
PO Box 631, Vicksburg, Mississippi 39180-0631

January 1986

Final Report

Approved For Public Release; Distribution Unlimited

Prepared for DEPARTMENT OF THE ARMY
US Army Corps of Engineers
Washington, DC 20314-1000

Under Civil Works Research Work Unit 31232

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
1. REPORT NUMBER Technical Report CERC-86-1	2. GOVT ACCESSION NO.	3. RECIPIENT'S CATALOG NUMBER
4. TITLE (and Subtitle) DETACHED BREAKWATERS FOR SHORE PROTECTION		5. TYPE OF REPORT & PERIOD COVERED Final report
		6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) William R. Dally Joan Pope		8. CONTRACT OR GRANT NUMBER(s)
9. PERFORMING ORGANIZATION NAME AND ADDRESS US Army Engineer Waterways Experiment Station Coastal Engineering Research Center PO Box 631, Vicksburg, Mississippi 39180-0631		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS Civil Works Research Work Unit 31232
11. CONTROLLING OFFICE NAME AND ADDRESS DEPARTMENT OF THE ARMY US Army Corps of Engineers Washington, DC 20314-1000		12. REPORT DATE January 1986
		13. NUMBER OF PAGES 88
14. MONITORING AGENCY NAME & ADDRESS(if different from Controlling Office)		15. SECURITY CLASS. (of this report) Unclassified
		15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

Approved for public release; distribution unlimited.

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES

Available from National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161.

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

Breakwaters
Shore protection

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This report discusses the application of detached single and segmented breakwaters for shore protection and beach stabilization. Four single detached breakwaters and six segmented breakwater systems in the United States are reviewed. Based on this review, the performance of breakwaters in other countries, and general knowledge of coastal processes, the implementation and design of breakwaters are discussed. General guidance is presented for the design of detached breakwaters and several design examples are presented.

DD FORM 1 JAN 73 1473 EDITION OF 1 NOV 65 IS OBSOLETE

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

PREFACE

This report is published to provide coastal engineers with information and guidance on the use of detached breakwaters for shore protection and beach stabilization. The material is based on an extensive literature review and the analysis of existing projects constructed for these purposes. The work was begun under the Shore Protection and Restoration Research Program and completed under the Evaluation of Navigation and Shore Protection Structures Civil Works Research Work Unit 31232 at the US Army Engineer Waterways Experiment Station (WES), Coastal Engineering Research Center (CERC). This study was sponsored by the Office of the Chief of Engineers; Project Monitor was Mr. John H. Lockhart.

The report was prepared by Mr. William R. Dally, former Hydraulic Engineer, and Ms. Joan Pope, Research Physical Scientist, under the general supervision of Dr. Robert M. Sorensen, former Chief, Coastal Processes and Structural Branch; Mr. R. P. Savage, former Chief, Research Division; Mr. Thomas W. Richardson, Chief, Coastal Structures and Evaluation Branch; Dr. William L. Wood, former Chief, Engineering Development Division, and Dr. James R. Houston, Chief, CERC. The material on physical model testing of detached breakwaters was prepared by Mr. William C. Seabergh, Research Hydraulic Engineer, and Mr. Robert R. Bottin, Jr., Civil Engineer, Wave Dynamics Division. The manuscript was typed by Ms. Mary M. Logan, and the figures were prepared by Mr. Darryl D. Bishop, Civil Engineering Technician. A critical review was performed by Mr. Thomas W. Richardson and Dr. Nicholas C. Kraus.

COL Allen F. Grum, USA, was Director of WES. Dr. Robert W. Whalin was Technical Director.

CONTENTS

LIST OF FIGURES

LIST OF FIGURES (Continued)

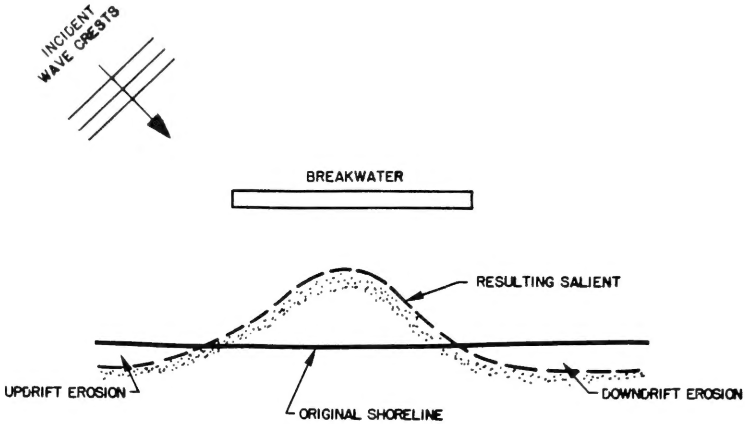
DETACHED BREAKWATERS FOR SHORE PROTECTION

PART I: INTRODUCTION

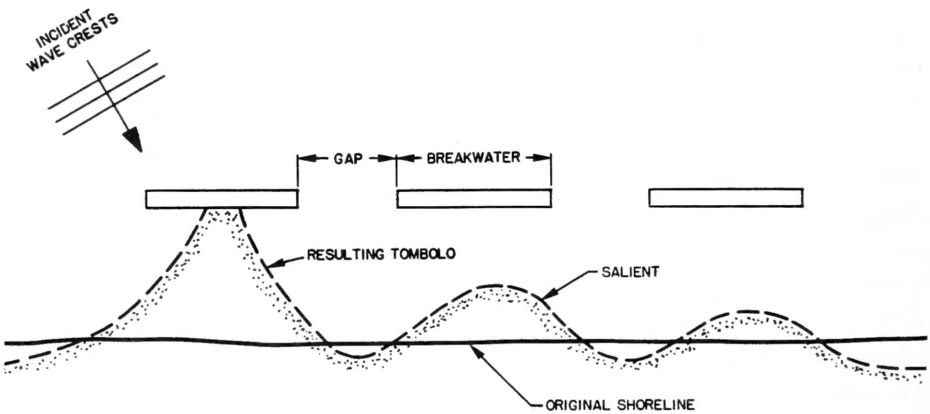
1. Detached breakwaters protect the shore by modifying wave action, thereby promoting sediment deposition shoreward of the structure and resulting in the development of a beach salient. The concept of detached breakwaters combines the wave energy dissipation of a natural shore-parallel sandbar or reef with the wave diffraction effects of a nearshore island. Detached breakwaters locally reduce incident wave energy and alter wave direction to create a "shadow zone" where sediment transported alongshore or placed as beach-fill is retained. This sediment will typically appear as a bulge in the beach planform. If this depositional feature or salient becomes connected to the structure, it is called a tombolo. This report examines the conditions which cause different degrees of sediment buildup, the philosophy and history behind the use of detached breakwaters for shore protection, and the advantages and disadvantages of detached breakwaters.

2. Detached breakwaters can be constructed as a single structure for very localized shore protection or as a segmented breakwater to protect a longer section of beach. A detached breakwater is not connected to shore by any type of sand-retaining structure, and is usually built approximately parallel to shore or to the predominant wave train. A segmented detached breakwater consists of two or more relatively short breakwater segments separated by gaps. Both the segment and gap length are usually regular. Figure 1 illustrates the general characteristics of detached single and segmented breakwaters.

3. Segmented detached breakwaters have many advantages over other, more conventional forms of shore protection. Unlike groins, segmented breakwaters can be designed to provide substantial protection without becoming a complete barrier to littoral transport, nor promoting offshore losses. Unlike revetments, bulkheads, and seawalls, they aid in the retention of the beach. If the breakwater system is properly sited and designed, and beach-fill is included as an item of construction, the impact to neighboring shores should be minimal. The main disadvantages of segmented detached breakwaters are that they are more expensive to construct than land-based structures and there are



a. Single detached breakwater



b. Segmented detached breakwater (three segments)

Figure 1. Definition of detached breakwater terminology

no standardized design criteria. Although segmented detached breakwaters have previously been implemented in Japan, Italy, Israel, Australia, and other countries, experience in the United States has been very limited (Pope in press). These structures have been built in Massachusetts, Ohio, Pennsylvania, and Virginia, and plans exist for their use in other states.

4. The literature on detached breakwaters yields little guidance for determining the optimum configurations of these structures for shore protection and illustrates that their shoreline response is a complex phenomenon which often cannot be predicted with great accuracy. The material presented herein is based on a review of detached breakwaters built for shore stabilization, with special emphasis on ten United States breakwater projects. The reader is referred to Toyoshima (1972) and Lesnick (1979) for additional background material. Guidance for developing a breakwater cross section is available in the Shore Protection Manual (SPM 1984).

5. The design concepts presented are those required for developing a breakwater plan and configuration. This guidance is intended to help the coastal engineer or scientist predict sediment response to a detached breakwater. The design and application of the detached breakwater concept at any particular site must be based on an evaluation of the local wave climate and littoral transport regime, plus a review of the lessons learned from previously constructed projects. The site specific nature of detached breakwater design and the habitual lack of coastal processes data for any particular site introduces a level of complexity into detached breakwater design which can be dealt with in this report in a qualitative sense only. Each project possesses its own specific problems which cannot be anticipated in any general guidance. Therefore, this report not only stresses design, philosophy, and theory, but also the prototype experiences which are currently available to assist in developing a detached breakwater plan. Although there are a limited number of numerical and physical modeling procedures which may be used, the eventual design must rely heavily on engineering judgment and experience on the local coast.

PART II: GENERAL OPERATION OF DETACHED BREAKWATERS

6. As previously mentioned, detached breakwaters protect a zone of the beach from direct wave action and transform the incoming waves. The area immediately behind the breakwater is sheltered because wave energy is dissipated on or reflected off of the structure. Wave energy is also reduced as waves diffract around the breakwater ends, resulting in a lateral spread of wave energy and a reduction in energy reaching the shore at any given point. The net effect is to reduce the capacity of waves to entrain and transport sediment in the breakwater shadow and to drive sand into the sheltered area immediately behind the breakwater where it is deposited, thus causing a bulge or salient to develop along the shore (Figure 2). Littoral drift entering the breakwater zone of influence may be permanently trapped or removed at a later date when transport conditions change. If wave conditions are relatively constant, a state of dynamic equilibrium may be attained.

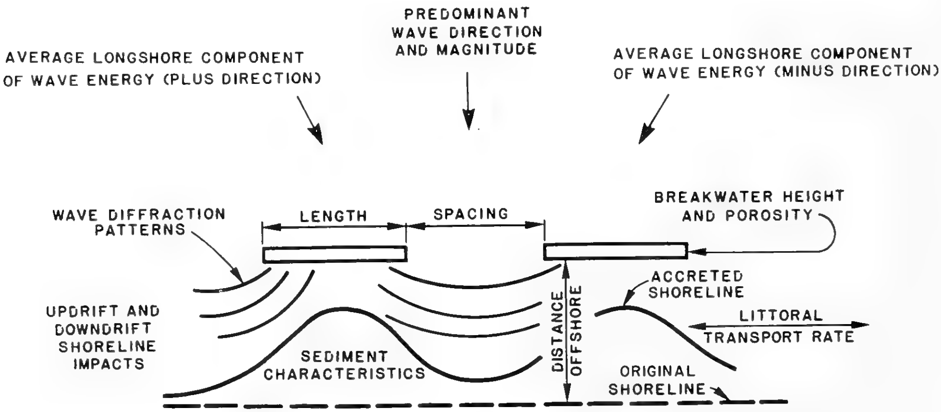


Figure 2. Segmented detached breakwater design considerations

7. The functioning of a detached breakwater is best understood by comparison with a more traditional sand-accreting shoreline structure, the groin. Both groins and detached breakwaters are methods of beach erosion control which can involve the use of a single structure or a group of structures designed as a system. Groins are generally built perpendicular to shore, while detached breakwaters are generally built parallel to the shore. Groins do not appreciably reduce the wave energy striking the shore and they

tend to compartmentalize the shore and the long-shore current system. Sediment moving alongshore is forced into deeper water in order to move around the structure ends, thereby increasing offshore losses. Frequently, the presence of a groin field will displace the nearshore bar system seaward. If detached breakwaters are designed properly, sediment will continue to move alongshore at a reduced rate behind the structures. The degree of reduction for given transport conditions is a function of the design. Sediment may be trapped temporarily and then removed when conditions change. Breakwaters have not been observed to increase offshore losses of sediment (unless tombolos form) and, in fact, are capable of decreasing the offshore transport rate. Therefore, unlike groins, breakwaters (a) seldom promote offshore sediment losses, (b) do reduce the potential for longshore movement, and (c) can allow regional littoral transport patterns to continue.

8. There are several disadvantages to the use of detached breakwaters. They are expensive to construct, often involving the use of marine-based equipment. Also, available design experience and guidance is limited. The parameters which control the complex interaction of sediments and structures are poorly understood, setting the stage for potential judgmental errors. Probably the greatest disadvantage is a perceived one. The scarcity of functioning examples especially for segmented detached breakwaters in the United States reduces the public and even the technical confidence level. People are reluctant to support a project if they cannot see a similar plan that is working.

9. Of primary interest to the coastal engineer when designing a detached breakwater are the equilibrium size and shape of the salient and its stability. Accurate prediction of the eventual shoreline configuration is beyond the present state of knowledge; however, the conditions necessary for equilibrium can be deduced. If the length of a structure is great enough in relation to its distance offshore, the salient may connect to the structure forming a tombolo. If the incoming breaking waves are normally incident to the original shoreline and there is no predominant longshore transport direction, the diffracted waves will transport sand from the shore adjacent to the structure into the structure's shadow. This process will continue until the shoreline is so aligned that the waves break parallel to the shoreline and the longshore transport again becomes zero along the entire coast. Figure 3 is an example of such a case, where the tombolo's equilibrium condition is such that

Figure 3. Breakwater and nearly symmetric tombolo (incident waves normal to shore), Tel-Aviv, Israel

it is symmetrical and concave on either side. However, if a salient forms, it will have a more rounded, convex shape. The diffracted waves may not be exactly normal in the vicinity of the salient apex, but in this region forces resulting from the waves diffracted by each of the breakwater heads will be in balance. No longshore current will be driven, and the longshore transport is zero.

10. Waves that arrive at an oblique angle will generate a net longshore transport rate that is maintained some distance updrift and downdrift of the breakwater. The shoreline near the structure will tend to adjust so that the same transport rate is achieved everywhere and dynamic equilibrium is again attained. To do so, smaller waves behind the structure must transport as much sediment alongshore as the larger waves adjacent to the structure. This situation will occur when the beach planform behind the breakwater has adjusted through the formation of a salient. The bathymetry of the salient causes the smaller, diffracted waves behind the structure to break at a more oblique angle than those waves outside the shelter of the structure. After tombolo

development sand may be transported seaward of the structure, possibly restoring the longshore transport rate. However, this usually involves deflecting sediment into deeper water where it may be lost to the littoral system. Even if transport is restored, the downdrift beach usually erodes due to the temporary deficiency of longshore-transported sand and the long-term increase in offshore losses. In addition, tombolo development will tend to promote offshore losses. The bulge in the shoreline resulting from oblique wave attack can be expected to be asymmetric, with its shape depending on the following: (a) structure length and distance offshore, (b) nearshore wave conditions, and (c) sediment characteristics. Figures 4 and 5 are examples of connected and nonconnected formations (i.e., tombolos and salients) that developed in response to oblique wave angles. In general, tombolos are pointed and have concave sides (Figure 4), while salients are more rounded (Figure 5).

11. Any structure that causes local accretion of sand may also cause damage to downdrift beaches if it removes material from the longshore system. Adding beach fill to the project site is a means of avoiding or minimizing this effect. A sufficient amount of sand should be placed to equal the amount which would otherwise be removed from the littoral system by the breakwater. Designing a detached breakwater requires prediction of the resultant equilibrium beach and the additional amount of sand necessary to maintain that stable shoreline. By artificially adding an equal amount of fill, there should, in principle, be no net adverse impacts on the neighboring shores. This is a general principle which should be rigorously examined for any proposed project. Even with the initial placement of beach fill, short-term variations in the wave climate at a site can result in unacceptable erosion on neighboring shores.

12. Accurate prediction of shoreline response to detached breakwaters is beyond the present state of the art, but these predictions must be approximated as a fundamental requirement for structure design. To do this, the designer needs to take advantage of many different tools to gain insight into the interrelationship of project and processes. This should involve studying existing breakwater projects and applying a basic understanding of coastal processes. The next two parts present a review of relevant breakwater projects constructed in the United States and discuss the parameters to be considered in breakwater design.

Figure 4. Detached breakwaters and asymmetric tombolos (oblique wave climate), Singapore (See Silvester and Ho 1972)

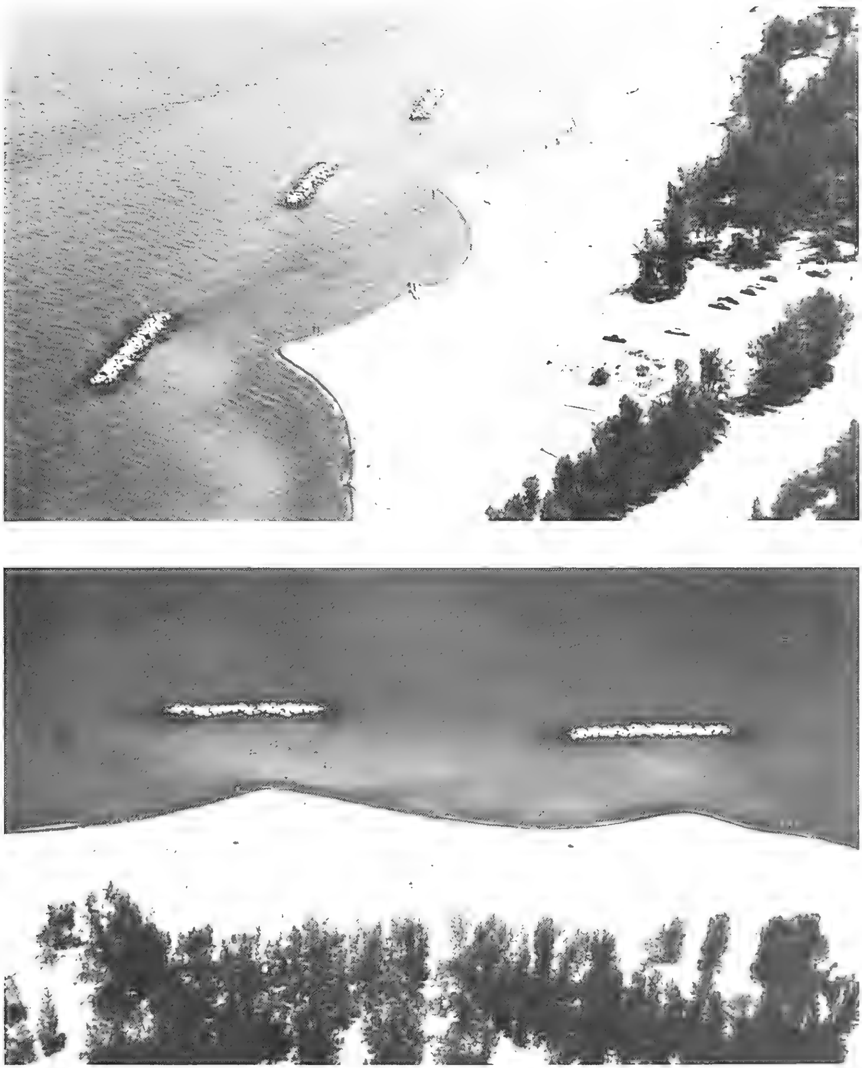


Figure 5. Two views of segmented detached breakwaters with asymmetric salients (slightly oblique wave climate), Presque Isle, Pennsylvania (1980)

PART III: REVIEW OF DETACHED BREAKWATER PROJECTS IN THE UNITED STATES

13. Although experience in the use of segmented detached breakwaters in the United States is limited, there have been a number of applications in other countries. In addition, single detached breakwaters have a long history along the shores of the United States. These structures range from low (frequently overtopped) structures located near the shoreline to high (deepwater) structures built in association with a harbor. Single detached breakwaters have been built by individual property owners and by all levels of Government, and thus exhibit different intents, designs, and construction.

14. A general impression of the performance of detached breakwaters as shore protection and some functional design guidance can be gained from a review of previous projects. In general, detached breakwaters in the United States are straight, shore-parallel, rubble-mound structures with project lengths ranging up to 600 m, distance offshore spanning roughly 46 to 600 m, crest elevations varying from +0.4 m above local low water datum to +5.5 m and the mean water depth at the structures ranging from 0.3 to 7.6 m below local low water datum. One must be especially careful in drawing general conclusions based on the shoreline response of any individual project, because history and configuration vary considerably. At least two single detached breakwaters (Santa Barbara and Venice, California) were eventually connected to shore by additional construction. The Lakeview Park, Ohio, segmented project has terminal groins. Three projects, Lakeshore Park, Ohio, Colonial Beach, Virginia, and East Harbor, Ohio, are new projects which are still evolving. A summary of these breakwater projects can be found in Table 1 on page 39.

Single Detached Breakwaters

15. A few major projects illustrate typical types of single detached breakwater applications (SPM 1984). One of the first was the Venice, California, rubble-mound breakwater. This structure was originally built to protect an amusement pier, and although the beach has been periodically eroded by storms, a tombolo has always returned. Another example project where a single detached breakwater was built for erosion control is Haleiwa Beach, Hawaii. Breakwaters built at Santa Barbara and Santa Monica, California, were originally intended to create harbors of refuge; however, both projects trapped

significant amounts of sediment causing formation of either a salient or a tombolo. An interesting multipurpose single detached breakwater project was constructed at Channel Islands, California. This structure overlaps the harbor entrance, simultaneously shielding boats from direct wave energy and trapping material adjacent to the entrance.

Venice, California

16. This rubble-mound breakwater (Figure 6) was constructed in 1905 to protect an amusement pier. It is 180 m long, has a crest elevation of +3.1 m

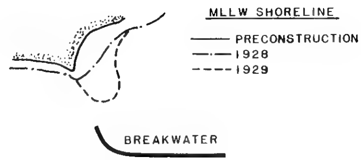
Figure 6. Detached breakwater at Venice, California (circa 1970)

mean sea level (MSL), and was placed approximately 370 m from the original mean high water line. The mean tide range in the region is 1.1 m. By 1910, the high water line salient behind the structure had advanced nearly 110 m, and by 1940 it had reached a position 170 m seaward of the preconstruction line. The pier was removed sometime between 1943 and 1948. There was a natural influx of sediment to the region during the 1940's, causing the neighboring shoreline to advance until the structure was only 210 m offshore. Subsequently, a tombolo developed behind the breakwater. A period of general shoreline erosion soon returned, and by 1963 the tombolo had eroded to form a salient positioned 120 m landward of the breakwater. Sometime shortly thereafter a low-crested timber groin was installed immediately behind the center of the breakwater which connected the breakwater to the shore and resulted in a "T" shaped structure. By April 1968, sand had accumulated and formed a nearly symmetric tombolo that completely buried the timber groin. Later in 1968, severe storms nearly removed the tombolo, exposing the groin. The beach has since recovered, with the present tombolo supported by the presence of the slightly exposed groin. The structure is relatively intact with very little wave transmission through or over the groin except during severe storms (Figure 6).

Santa Barbara, California

17. In early 1929 the original construction of the Santa Barbara rubble-mound breakwater (Figure 7) was completed, forming a protected harbor. The main portion of the structure is 430 m long, located 300 m offshore, and oriented at a slight angle to the shoreline. The water depth at the structure is approximately 7.6 m below mean lower low water (MLLW) and the structure crest is at +3.7 m MLLW. A shorter 120-m-long segment extended from the western end of the main portion toward the shore, leaving a 180-m gap with the shoreline. The structure is impermeable and infrequently overtopped. Even before initial construction was completed, the shoreline developed a bulge landward of the breakwater near its western end. By late 1929 this salient was very large and appeared to be well on its way to connecting to the structure (Figure 7c). However, the short breakwater segment was extended in 1930 to connect to the shore, and the harbor was dredged, precluding the development of a tombolo.

a. Preconstruction



c. Shoreline response

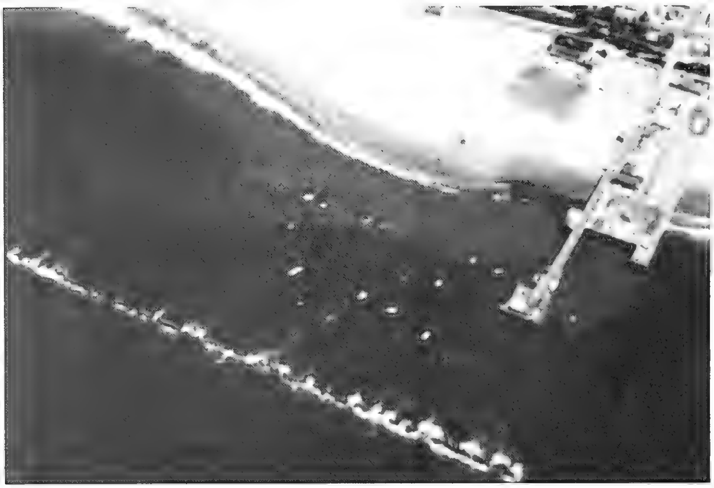
b. Initial breakwater construction almost complete

Figure 7. Shoreline response to initial construction of Santa Barbara breakwater

Santa Monica, California

18. The Santa Monica detached breakwater (Figure 8) is the largest of its kind in the United States. It is a 610-m-long rubble-mound structure

a. Salient with eroded downdrift (longshore drift is from bottom to top)



b. Deteriorated breakwater allowing wave transmission

Figure 8. Santa Monica breakwater and salient (circa 1967)

located 610 m offshore of the original shoreline. This shore-parallel structure is situated at a mean water depth of -8.4 m MSL and has a crest at +2.2 m MSL. The mean tide range is 1.1 m. Although the breakwater's original purpose was to provide a harbor of refuge for small craft, the shoreline response demonstrates the application of detached breakwaters for shore protection. It was completed in 1934, and by 1948 the shoreline behind the structure had accreted 240 m, while the downdrift shoreline experienced substantial erosion. The region shoreward of the structure was subsequently dredged and the material placed on the downdrift beach. The structure has lost crest elevation allowing some wave transmission (Figure 8b). The salient has never developed into a tombolo.

Haleiwa Beach, Hawaii

19. The Haleiwa Beach breakwater (Figure 9) was constructed in 1965 and protects a park facility and war memorial monument. It is a single impermeable, rubble-mound structure, 50 m long and approximately 90 m offshore of the original, eroding shore. Water depth at the structure is 2.1 m MSL, crest is

Figure 9. Breakwater at Haleiwa Beach, Hawaii (1966)

at +1.2 m MSL, and the tide range is 0.5 m. A 69,000-cu-m artificial fill placed shortly after construction moved the shoreline to within an estimated 50 m of the breakwater. A salient formed which nearly connected to the structure by August 1966. This condition is illustrated in Figure 9. In December 1969 a severe storm damaged the breakwater and beach fill but both were repaired by late 1970.

Segmented Detached Breakwaters

20. The concept of a segmented detached breakwater has been used extensively in other countries, creating a broad experience base (Lesnik 1979). One of the best documented projects is the series of shallow-water "artificial headlands" at Singapore (Figure 4), although other projects exist in Italy, France, Israel, and Denmark, to name just a few. Segmented offshore breakwaters have been used for almost 30 years in Japan (Toyoshima 1972). The Japanese have developed a construction and general configuration plan which is functioning successfully at more than 20 different sites. Typically, Japanese segmented detached breakwaters are built fairly close to shore, which would normally cause development of a tombolo. However, these breakwaters have no core and are fairly permeable; therefore, inhibiting tombolo formation.

Winthrop Beach, Massachusetts

21. The Winthrop Beach project (Figure 10) was the first segmented detached breakwater in the United States. The breakwater was built for shore protection; however, gaps were incorporated into the project plan to permit boat traffic. Completed in 1935, it consists of five 100-m-long impermeable rubble-mound segments separated by 30-m gaps. The total length of the breakwater is 625 m. The structure is shore-parallel and was constructed 300 m from a seawall at an average water depth of 3.8 m. The structure crest is at +4.1 m MSL. The region has a normal tidal range of 2.7 m and a spring range of 3.4 m. This large tidal range has important ramifications in the shoreline response. An irregularly shaped, multiple tombolo is exposed at low tide (Figure 10a), when the structure is only 150 m from the updrift shoreline alignment. However, at high tide (Figure 10b), only an unconnected bulge is visible above water. These features have been relatively persistent since 1937. Two possible scenarios could be used to explain the presence of

a. Low tide

b. High tide

Figure 10. Breakwater at Winthrop Beach, Massachusetts (1981)

distinctively different equilibrium beach planforms as a function of tide level. The significant tide range will result in different nearshore wave characteristics (i.e., steepness and angle are greatly influenced by water depth). In addition, the high tide shore is a much greater distance from the structures than the low tide shore. Both these factors will affect the diffracted wave angle at the shore and the resultant beach planform.

Lakeview Park, Lorain, Ohio

22. This project was completed in 1977 for the purpose of protecting a park facility and providing a recreational beach. It consists of an 84,000-cu-m beach fill bordered by two terminal groins and fronted by a rubble-mound breakwater divided into three segments (Figure 11). Each breakwater segment is 76 m long, and adjoining segments are separated by 50-m gaps. The segments were constructed in an average water depth of 3.5 m and are located 145 m from the original eroded shore with crest elevation at 1.8 m above the long-term average water level for Lake Erie. These segmented structures are relatively impermeable and overtopped infrequently. Beach fill displaced the shoreline to an average position of 76 m landward of the structure where the placed fill rapidly adjusted to a relatively stable morphology which has an undular shape due to the breakwater effects. The salients do not reach the structures, but the area behind them has shoaled substantially. In the first five years after construction, approximately 2,500 cu m of sand was gained annually. A substantial monitoring and evaluation program was initiated immediately after construction and continued for five years. A reduced level of monitoring still continues. Through this monitoring effort, much has been learned about the rate and nature of sediment/structure interaction, which will be the subject of a future report. A detailed description of the project and its performance is presently available in Pope and Rowen (1983).

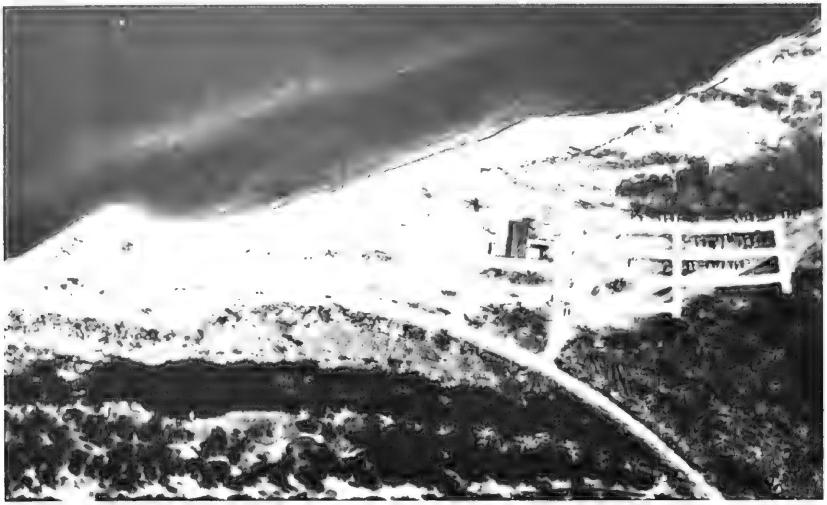
Presque Isle, Pennsylvania

23. At Presque Isle on Lake Erie, a segmented system of over 50 detached breakwaters is planned to protect the shoreline and create recreational beaches along an 11-km-long recurved sand spit. A test structure consisting of three rubble-mound segments was completed in 1978. The segments are each 40 m long, with one gap 60 m wide and the other 90 m wide. The crest elevation is approximately 1.2 m above the long-term average lake level and the water depth averages 1.0 m. Fill was trucked in and the shoreline was advanced to a location between 45 and 60 m from the structures. The subsequent shoreline evolution is displayed in Figures 12a-d. Note that the most westerly structure (left side of photo) dominates the entrapment of the longshore drift, which moves mostly from west (left) to east (right). This

a. April 1981

b. November 1979

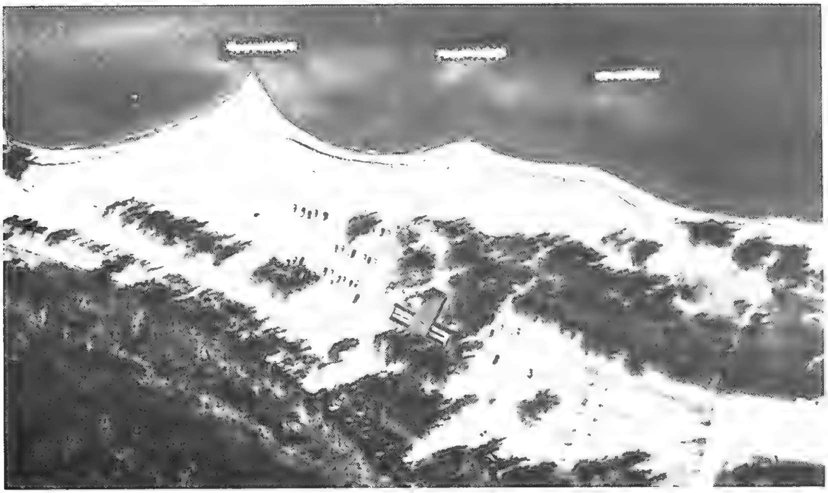
Figure 11. Spring (a) and fall (b) beach conditions and the detached breakwater and terminal groins at Lakeview Park, Lorain, Ohio



a. Preconstruction, May 1978

b. Immediately after construction, July 1978

Figure 12. Detached breakwater test at Presque Isle, Pennsylvania (Continued)



c. Nonuniform salient formation, September 1978
(longshore drift is from left to right)

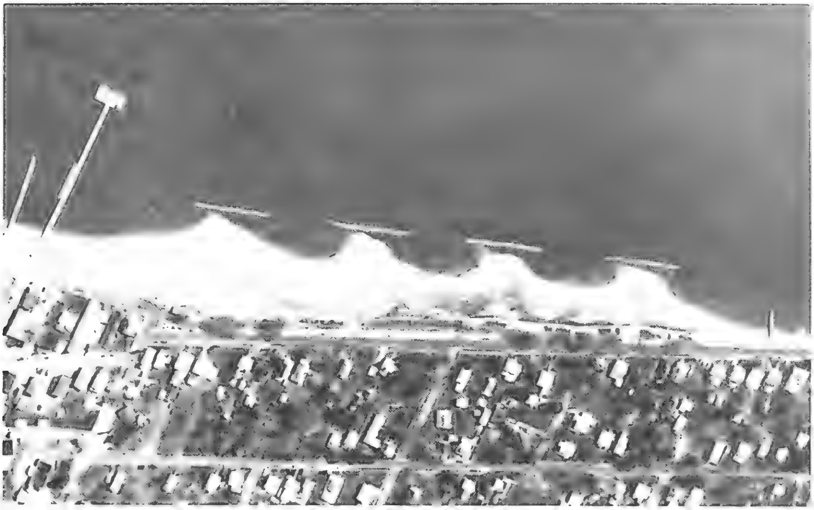
d. November 1978

Figure 12. (Concluded)

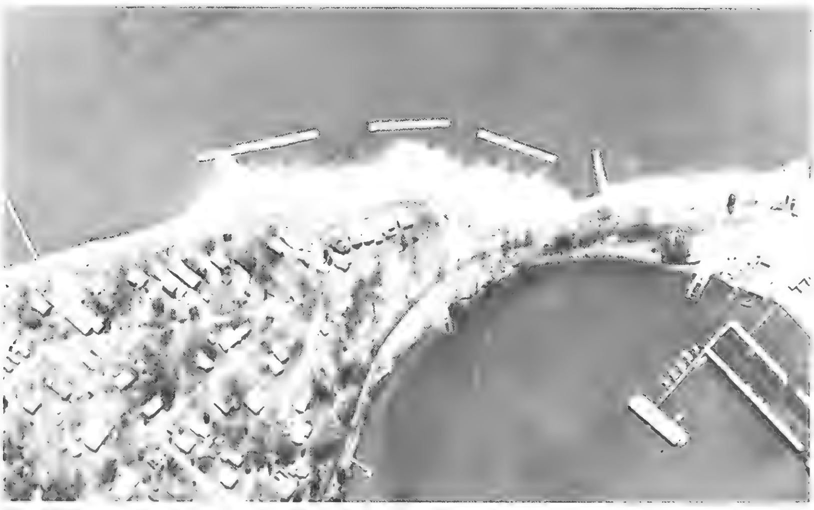
salient periodically connects to the structure, forming a tombolo. When this occurs, the shoreline resembles an undular crenulate bay. However, because of their relatively low crest elevation, these breakwaters are overtopped during storms, often causing the tombolo to become detached from the breakwater only to return with mild wave conditions. The test breakwater segments have been monitored, and the observed shoreline response has been used to assist in the development of a physical model study for the entire proposed project (Appendix C). In this way, field data are being used to improve a standard design tool prior to large-scale construction.

Colonial Beach, Virginia

24. Two of the most recently constructed detached segmented breakwaters were completed in 1982 at Colonial Beach, Virginia, on the lower Potomac River. The project consists of two sites approximately 1.5 km apart. The Central Beach Section consists of a breakwater with four segments fronting a beach fill (Figure 13a). The segments are each 60 m long with gap widths of 45 m, and are located approximately 65 m from the shoreline in 1.2 m (MSL) of water with a crest elevation of 0.4 m above MSL and a tide range of 0.5 m. The structures are shore-parallel. The Castlewood Park Beach Section (Figure 13b) consists of three segments and fill; two of the segments are 60 m long and separated by a gap of 26 m, and the third is 90 m long and 40 m from the adjoining segment. This structure is located approximately 45 m from the original shore. As shown in Figure 13b, the shoreline in this area is curved and the segments are slightly oblique. Crest elevation is 0.4 m above MSL and the water depth is 0.9 m MSL. Available fetch is small, with the area subject to significant wave attack only during local storms. The project is new and is being monitored jointly by the US Army Engineer District, Baltimore, and the US Army Engineer Coastal Engineering Research Center (CERC). These breakwaters are relatively close to the shoreline and tombolo formation is frequent. However, the elevations of both the structure crest and the placed beach berm are low. Apparently, a high tide accompanied by a storm surge can inundate the project, overtop the structures, and flood the beach. This postulated high overtopping component may contribute to making the tombolos unstable. More definitive information on the behavior of this project and its causes will result from the monitoring program.



a. Four segments at Central Beach



b. Three segments at Castlewood Park Beach

Figure 13. Breakwater project at Colonial Beach, Virginia

Lakeshore Park, Ashtabula, Ohio

25. Another project consisting of a segmented detached breakwater with three segments was completed in 1982 on Lake Erie at Lakeshore Park, Ashtabula, Ohio, for the purpose of retaining a recreational beach (Figure 14). The project included the placement of 27,000 cu m of beach fill with segments 40 m long, 60 m apart, and placed in an average water depth of 1.5 m. The structure is 120 m offshore of the original shoreline and constructed in a slightly arched configuration to provide better protection to the 244-m-long, 46-m-wide placed beach. Essentially, no natural littoral material enters the project due to the presence of Ashtabula Harbor to the west and a large water intake structure to the east. The beach has not yet attained a stable sinuous planform and its present width averages less than 30 m. The original postconstruction beach width was 46 m. In addition, beach fill appears to be moving out of the project to the west (to the right in Figure 14). The fine grain size of the placed material may contribute to this loss. Material eroding from the beach is causing significant shoaling in the vicinity of a boat launching area at the west end of the park. This project is being monitored jointly by CERC and the US Army Engineer District, Buffalo. Data collected to date include surveys, aerial photography, Littoral Environment Observations (LEO), and an extensive set of site visit observations. Approximately 800 cu m of material was dredged from the boat launching area both in 1983 and again in 1984 and placed back on the recreational beach.

East Harbor, Ohio

26. East Harbor State Park is located on a barrier beach near the west end of Lake Erie. It is approximately 3,600 m long and located between two structured harbor entrances. High lake levels in the 1970's and early 1980's stripped away the recreational beach and threatened the park facilities. In the spring of 1983 the state of Ohio built a segmented detached breakwater with four segments (Figure 15) as a test to gather prototype data in preparation for designing a 21-segment structure. These initial breakwater segments are 46 m long, and are separated by gap widths which range from 90 to 120 m. The segments were constructed approximately 180 m off the original shore in an average water depth of 1.5 m. No beach fill was placed as part of this project. Some planform sinuosity is slowly evolving; however, the lack of available sediment has greatly retarded the shoreline response. In spite of this,

Figure 14. Small recreational beach project at Lakeshore Park, Ashtabula, Ohio

Figure 15. Prototype test breakwater at East Harbor State Park, Ohio

sediment has built up behind the breakwater and the local bathymetry is beginning to show subaqueous evidence of salient development. The state of Ohio is currently monitoring this project.

PART IV: DESIGN CONSIDERATIONS

27. Discussed in this part are the parameters that affect shoreline response to detached breakwaters and so guide their design. The primary accretionary features associated with breakwaters, tombolos (structure-connected), and salients (nonconnected) are compared. Breakwater positioning and the significance of single versus segmented design are discussed, as well as available techniques used to control shoreline response.

Significant Parameters

28. Shoreline response to detached breakwaters is primarily controlled by wave diffraction (SPM 1984, Chapter 2, Section IV). Wave length, height, and angle and (in the case of segmented structures) the ratio of gap to wavelength affect the diffraction pattern and the wave height behind the breakwater (Figure 2). The shoreline tends to align itself parallel to the diffracted wave crests. The rate of shoreline response is governed predominantly by the wave energy and the incident angle of the diffracted waves as they approach the shore. Other important parameters are (a) the local water level range, (b) natural beach slope, (c) available supply of sediment, and (d) sediment grain size.

Wavelength

29. In general, the amount of wave energy diffracted into the region behind the breakwater increases with increasing wavelength. If diffraction theory using linear waves and a flat bottom are assumed, wavelength will not affect the pattern made by the crests, but will affect wave height at each location. Longer waves will provide more energy to the shadow zone behind the breakwater and might tend to prevent tombolo formation. The amount of energy that penetrates behind a detached breakwater can be computed by using the diffraction diagrams presented in Figures 2-28 to 2-39 of the SPM. An example of the use of diffraction analysis for designing a segmented detached breakwater is presented in Appendix A.

Breakwater gap size versus wavelength

30. The ratio of gap size to wavelength greatly affects the distribution of wave height behind segmented detached breakwaters. Stated simply, increasing the gap-to-wavelength ratio increases the amount of energy

transmitted past the segments while decreasing diffraction effects. Simple diffraction diagrams (i.e., Figures 2-42 to 2-52 of the SPM) can be used to compute the diffraction effect on a wave which passes through a gap. However, these simple diffraction diagrams do not account for wave shoaling or breaking. If the design wave breaks before passing the breakwater, isolines which predict the diffraction coefficient will predict values that are higher than should actually be expected. The effects of wavelength and the ratio of gap size to wavelength on the diffraction pattern are clearly demonstrated by comparing the wave patterns in aerial photographs taken at two different times at Lakeview Park (Figure 11). The shorter incident waves in Figure 11a are less distorted after passing through the gaps, and the shadow zones are relatively quieter than for the longer incident waves shown in Figure 11b. The shoreline tends to align itself with the waves; therefore, the salients are more pronounced with longer, more diffracted waves.

Wave angle

31. The orientation of the incident waves relative to shore and the breakwater affects both the degree of salient development and the equilibrium planform of the shoreline. Strongly oblique waves will drive a regional longshore current that may dominate the local effects of the breakwater, restricting the size of the salients and preventing connection to the structure. Accordingly, it is important when designing a breakwater to consider not only the predominant wave direction, but also the average annual wave angle distribution. Two regions might experience identical levels of wave energy, but if one region has a more diverse wave angle climate, this site may require a longer breakwater to suppress the effects of the more oblique waves. A terminal structure may also be required to reduce alongshore losses. The bimodal distribution in wave direction common to many coasts may be a particularly important consideration in breakwater design. A postulated criterion for salient development based on this factor is presented in Walker, Clark, and Pope (1981) and is reviewed in Appendix A. Shoreline planform is highly dependent on the directional characteristics of the wave climate. The bulge in the shoreline tends to align itself with the predominant wave direction. This is particularly noticeable for tombolos, which seem to point into the waves (Figure 4). Frequently, the feature's updrift side is filled, its apex is near the center of the structure, and the downdrift side is less filled or even eroded. However, if predominant waves are extremely oblique to the

shoreline, the tombolo's apex can be shifted downdrift. The tombolo's or salient's equilibrium position is dependent upon the strength of the predominant longshore current and the length of the structure. If there are significant seasonal variations in the predominant wave direction, the equilibrium position of the salient may readjust accordingly.

Wave height

32. The average, extremes, and seasonality of the wave height control the energy available for sediment transport. One of the greatest challenges in designing a detached breakwater system is not in determining the extreme wave conditions but in determining the average range of conditions which control the planform stability. Wave height also affects the pattern of diffracted wave crests. In shallow water, wave celerity is given by linear wave theory:

$$C = \sqrt{gd}$$

where

C = wave celerity

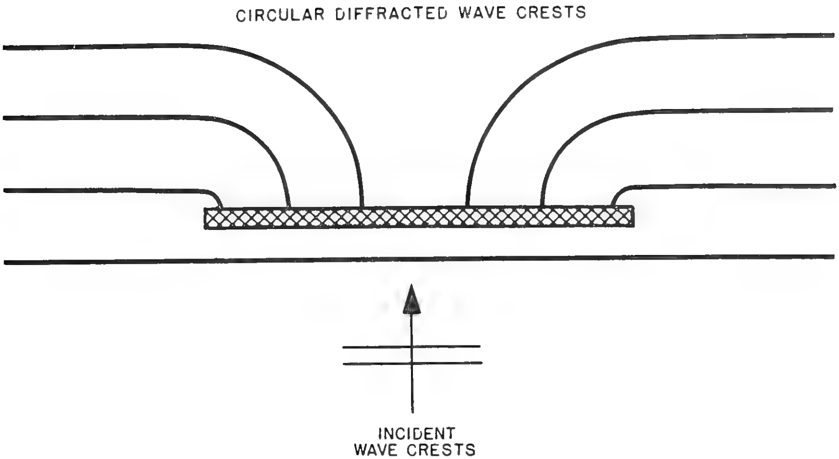
g = gravity acceleration constant

d = water depth

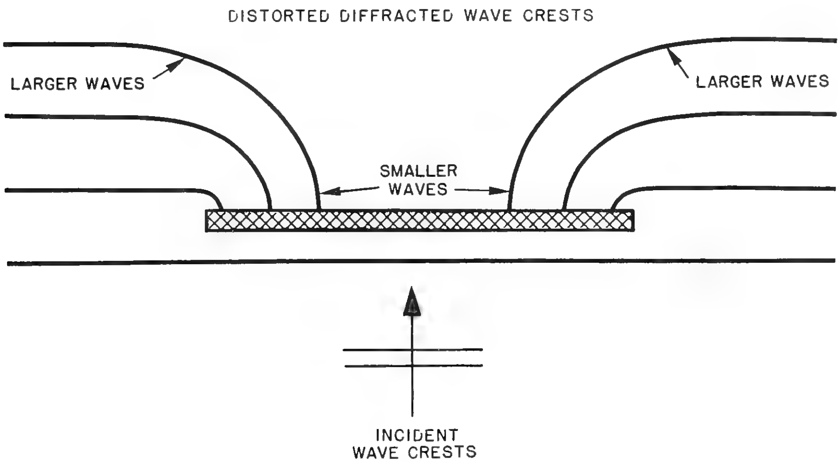
This relationship predicts that for water of constant depth, the crestline of the diffracted waves will be circular (Figure 16a). In this case the entire wave crest moves at a uniform and constant speed. This model is not quite accurate for very shallow water, where wave amplitude affects the wave speed (Weishar and Byrne 1978) and therefore affects diffraction characteristics. Wave celerity in very shallow water may be more accurately expressed as

$$C = \sqrt{g(d + H)}$$

In very shallow water, wave celerity decreases along the diffracted wave crest in relation to the decrease in wave height. The result is a distortion of the diffracted wave train from the circular pattern shown in Figure 16a to a series of arcs of decreasing radius (Figure 16b). The significance of this process may be seen in those shallow-water situations which enable the undiffracted portion of the wave adjacent to the structure to reach the shore before the waves diffracted around the breakwater ends intersect. This



a. Diffraction at a breakwater assuming linear wave theory ($C = \sqrt{gd}$)



b. Diffraction at a breakwater including the effects of amplitude dispersion ($C = \sqrt{g(d + H)}$)

Figure 16. Comparison of diffraction pattern theory

situation usually results in more readily formed tombolos. Colonial Beach may be an example of this process (Figure 13).

Local water level range

33. Prediction of the exact effect of a large water level range on shoreline response to detached breakwaters is extremely difficult. Generally speaking, a range over 1.5 m will tend to hinder permanent tombolo formation, especially if the structure is significantly overtopped during high water, and will certainly prevent the shoreline salient from attaining a smooth equilibrium shape. Winthrop Beach, with a tidal range of 2.7 m, is a good example of large tide range effects on shoreline response (Figure 10). In this case, two distinctive shoreline planforms developed, which were (a) a high-tide, low-sinuosity salient, and (b) five individual, low-tide sinuous tombolos. On the Great Lakes, variations in water level may cause seasonal or longer period changes in the stable beach planform. Lakeview Park exhibits very distinctive spring and fall shorelines which return each year (Figure 11), due, in part, to the approximately 0.5-m seasonal variation in water level. Storm-induced surges may cause significant, rapid changes in the beach planform which may include the loss of beach material.

Natural beach slope

34. The natural slope of the preconstruction beach may be an important consideration in selecting the appropriate distance offshore for the breakwater and in predicting its configuration. If the profile is gently sloping and the structure is to be placed outside the surf zone, the breakwater may have to be placed farther offshore and lengthened in order to be an effective sediment trap. A difficult design problem is the combination of a gently sloping beach and a large water level range. A large segment of the beach profile may be active over the range of water level changes. Choosing an optimum structure location under such conditions may be difficult.

Sediment supply

35. The development of a stable beach planform is dependent on there being sufficient sand to satisfy the equilibrium condition imposed by the breakwater and local wave climate. If the required sediment is not available from neighboring beaches (perhaps because of a reduced natural sediment supply or the presence of groins, seawalls, or rock intrusions), the salient(s) may develop very slowly, cause unacceptable levels of downdrift starvation, or may never reach equilibrium. Winthrop Beach and East Harbor are examples of

sediment-starved detached breakwater projects (Figures 10 and 15). In most cases, beach fill should be considered a necessary part of the project plan.

Sediment size

36. Sediment particle size and distribution affect longshore sediment transport rates and the characteristic equilibrium beach profiles and, therefore, affect the shore planform and the rate of beach response. A breakwater built offshore from a coarse-sediment beach will probably be in deeper water than it would be for a finer sized sediment site because the coarse beach equilibrium profile will be steeper. Waves approaching the coarse beach tend to refract less because of the steeper offshore bathymetry; therefore, these waves may reach the project at more oblique angles. Usually, a fine beach will respond more quickly and attain an equilibrium shape sooner than a coarse beach. A tombolo may develop on a fine sand beach, but the same incident wave conditions might not cause tombolo development on a coarse sand beach. This is due to greater wave energy in the lee of the structure, and lower volumes of sediment transport on coarse, cobble beaches. Also, placed coarse sand will probably not be lost from the project as rapidly as the native fine sand.

Tombolos versus Salients

37. Of great concern when designing a detached breakwater for shore protection is whether or not the resulting shoreline should be or will be connected to the structure. There are very different sediment transport patterns associated with a tombolo than there are for a salient, and advantages and disadvantages to each.

38. Although the formation of either tombolos or salients may cause erosion of neighboring shores, salients are usually preferred. Tombolo formation is more appropriate where the longshore transport regime is approximately balanced, or where sediment loss from adjacent shores is not a concern. If a detached breakwater is positioned seaward of the surf zone, or is long with respect to its distance offshore, it can effectively shut off the prevailing nearshore sediment transport, especially if a tombolo develops. Although longshore transport may continue offshore of the breakwater, this sediment is usually not immediately available to adjacent shores. The downdrift shoreline may undergo drastic erosion until the longshore rate is restored. When a tombolo forms, the large quantity of sediment which is impounded does little

to provide storm erosion protection beyond that already supplied by the breakwaters. Periodic tombolo formation, such as that exhibited by the Presque Isle project, may temporarily store sediment, which is then released to the downdrift during a storm. Salients have advantages over tombolos on coasts that are subject to seasonal changes in wave direction. With a salient, sediment does not have to be transported seaward of the breakwater to restore longshore transport. When the transport is reversed, the salient undergoes less drastic changes and the neighboring shores experience less seasonal erosion or accretion. Tombolos function as T groins. Longshore sediment transport is blocked and offshore losses are promoted. Mass transport from breaking waves can create a hydraulic head in the pocket between two adjacent tombolos. Water piled up between the tombolos may then return through the breakwater gap as a localized rip current, which can carry sediment offshore. This process has been observed at Colonial Beach. With salients the hydraulic head drives currents which can be dissipated alongshore as well as offshore. Segmented detached breakwaters do not provide uniform erosion protection along the entire project. This is especially true when one or a series of tombolos develops. Legal problems could occur if the region involved is owned by several different parties and storm damage is inflicted with partiality. Accordingly, the engineer is often required to design a project that will advance the shoreline in as spatially uniform a manner as possible.

39. A tombolo allows access to the breakwater structure; however, this has both advantages and disadvantages. Monitoring of the structure and maintenance or repairs are facilitated. However, beach users may be inclined to climb on the structure or swim immediately adjacent to it, which can be dangerous. One often overlooked advantage to a detached breakwater salient is that there is quiet water immediately behind the structure available for recreational use. (This sheltered water is ideal for very young beach users or those who otherwise do not like to challenge waves.) The protection of the breakwaters caused the beaches at both Presque Isle and Lakeview Park to evolve into very popular family-use beaches not long after construction.

Techniques for Controlling Shoreline Response

40. After deciding how much beach area is needed, choosing the preferred general planform of the shoreline, and if tombolos or salients are

desired, the next tasks are to determine to what extent the incident wave energy must be reduced or modified to stimulate formation of the desired shoreline, and the best possible means for accomplishing the required changes. Average seasonal shoreline variations give an indication of how the shore responds to different wave climates. Comparing the local winter and summer wave climates and observing how much the beach progrades during the summer months may provide insight into the degree of incident wave modification required to obtain the desired shoreline. From this comparison, an assumption can be made regarding the degree of energy reduction required to advance the shoreline the desired distance and the resultant sinuosity of the salients. Techniques available for reducing incident wave energy and controlling shoreline response are separated into two categories, those used for assuring tombolo development and those used for developing salients only.

Assuring tombolo development

41. Tombolos usually develop if the two diffracted wave crests behind a structure do not intersect before the undisturbed portions of the wave reach the shoreline. Figure 17 shows a highly idealized case and the resulting diffraction pattern. This single detached breakwater has a length ℓ and offshore distance x, and is parallel to the shoreline. Table 1 summarizes these and other parameters for various United States detached breakwater projects. The incident waves are assumed to be shore-normal and to obey linear wave theory, and the water landward of the structure is assumed to be uniform in depth. Diffracted waves will have a circular planform and the arcs will not intersect before the rest of the wave reaches the shore if the single breakwater length or segment length for a segmented breakwater is greater than two times the distance offshore, that is:

$\ell > 2x$	for a single detached breakwater
$\ell_s > 2x$	for a segmented detached breakwater (where ℓ_s is the individual segment length)

For oblique waves, the above condition also ensures no intersection of the arcs. However, the longshore current generated on the updrift side may be enough to penetrate the shadow zone and keep the tombolo from connecting. This is a general relationship and there are instances of tombolo connection where the single breakwater or segment length is equal to or less than

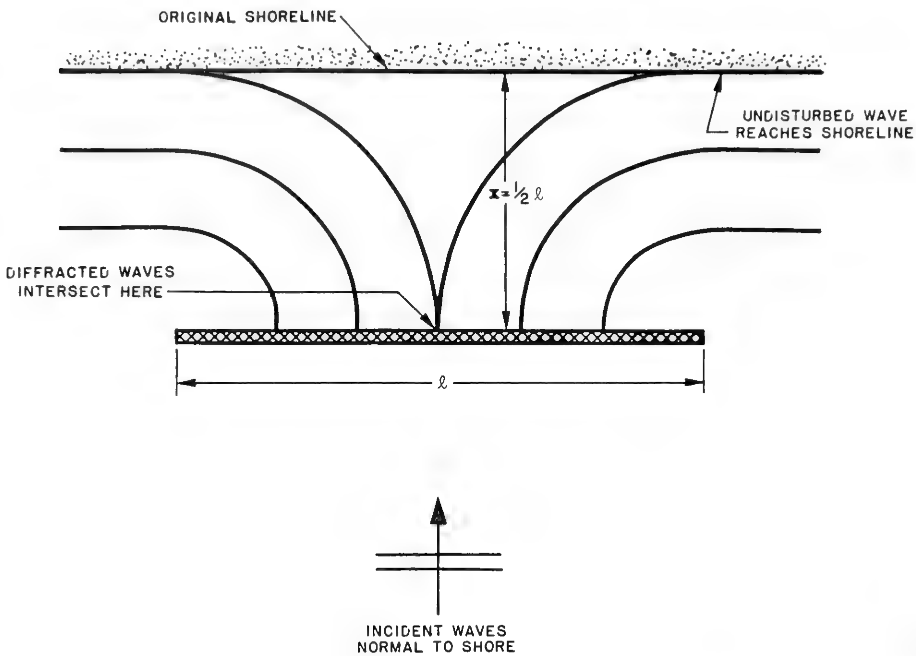


Figure 17. Condition ensuring tombolo development assuming linear wave theory

the distance offshore. A sufficient supply of sediment is necessary in order to form the tombolo. Refraction, shoaling, breaking, and the previously discussed higher order effects on celerity are not included, but this relationship is probably conservative. When comparing the ratio of length to distance offshore versus shoreline response, it appears that tombolos can occur when the ratio approaches 1.0 (Table 1). Bishop (1982) also reached this conclusion. The recommended criterion to assure tombolo development (assuming sufficient sand supply) may be a compromise. For example,

$\ell \geq 1.5x$ for a single detached breakwater

$\ell_s \geq 1.5x$ for a segmented detached breakwater

42. Lengthening the structure or reducing its distance offshore beyond the above condition will increase the size of the tombolo and may eventually induce the formation of double tombolos, with trapped water in between. This trend is suggested by the double beach ridge tendency at Colonial Beach

Table 1

Summary of Characteristics of United States Detached Breakwaters

Project	Type of Breakwater*	When Built	Total Length of Breakwater Project (ℓ)	Number of Segments	Length of Segments (ℓ_s)	Gap Width (ℓ_g)	Distance Offshore (x)	Water** Depth (d)	Crest** Elevation	Tombolo (T) or Salient (S)	Comments	Fill Placed	ℓ_s/x	ℓ/x
Venice, Calif.	Si	1905	183 m	N/A	N/A	N/A	366 m / 213	1.8 m	3.7 m	S / T	Pre 1940's / 1940's - 1960's groin installed early 1960's	No	N/A	0.5 / 0.9
Santa Barbara, Calif.	Si	1929	434 m	N/A	N/A	N/A	305 m	7.6	3.7	T	Project dredged and shoreward connection added to prevent tombolo formation	No	N/A	1.4
Santa Monica, Calif.	Si	1934	610 m	N/A	N/A	N/A	610 m	7.6	3.0	S	Periodic dredging prevents tombolo formation	No	N/A	1.0
Haleiwa Beach, Hawaii	Si	1965	49 m	N/A	N/A	N/A	91 original / 49 w/fill	2.4	1.5	S / Near T		No / Yes	N/A	0.5 / 1.0
Winthrop Beach, Mass.	Se	1935	625 m	5	91 m	30 m	305 m	3 MLW / 5.7 MHW	5.5 MLW / 2.8 MHW	T / S	Two beach planforms as a result of 2.7-m tidal range	No	0.3 / 0.3	2.0
Lakeview Park, Ohio	Se	1977	403 m	3	62 m	49 m	137 original / 76 w/fill	3.0	2.4	S	Terminal groins at both ends	No / Yes	0.5 / 0.8	2.9 / 5.3
Presque Isle, Penn.	Se	1978	440 m	3	38 m	60, 91 m	46 m	0.3	1.8	T/S	Tombolos form during low wave energy condition, removed by storms	Yes	0.8	9.6
Colonial Beach, Va. (Central)	Se	1982	427 m	4	61 m	45 m	64 m	1.2	0.4	T/S	Tombolos behind some breakwaters, salients behind others	Yes	0.9	6.7
(Castlewood)	Se	1982	335 m	3	61, 93 m	26, 40 m	46 m	1.2	0.4	T/S			1.3, 2.0	7.3
Lakeshore Park, Ohio	Se	1982	244 m	3	38 m	60 m	120 original / 75 w/fill	1.5	2.0	S	Still adjusting, fine size fill being lost longshore	Yes	0.3 / 0.5	2.0 / 3.3
East Harbor, Ohio	Se	1983	550 m	4	46 m	90, 105, 120 m	180 m	1.5	2.4	S	Still adjusting, low sand supply	No	0.3	3.1

* Si = single, Se = segment.
** Datum used is local MLW (unless otherwise stated) or Low Water Datum (LWD) for the Great Lakes.

(Figure 13). A large seasonal change in wave direction increases the probability of this occurring. Shore-connected double tombolos are generally an undesirable condition due to the large pool of stagnant water and associated beach use problems.

43. If the shoreline to be protected is very long, a segmented detached breakwater utilized to generate a series of small tombolos may be appropriate. A single long breakwater would need to be placed farther offshore in deep water to reduce the quantity of trapped sediment and prevent double tombolo formation. Such a plan is usually uneconomical. Generally, the condition necessary for developing a segmented breakwater project is one where the shore to be protected is approximately five or more times longer than the chosen distance offshore for breakwater construction (Table 1). Each segment should be between one and two times as long as the distance offshore if tombolos are desired, and the gaps should be sized according to the desired shoreline position opposite each gap. Unless the gap-to-incident wavelength ratio is very small, there will be little reduction in wave height at the shoreline directly opposite each gap. Without a substantial sediment supply, the shoreline will probably not accrete and may even erode in these areas, including into the placed fill (Figure 18). If this formation is not acceptable and a more uniform shoreline advance is desired, a perched beach plan might be appropriate.

44. To ensure a tombolo, the solid portion of the breakwater should be constructed to prevent wave transmission over or through it, thus minimizing wave activity in its lee. The structure should therefore have low permeability, with a crest height and slope sufficient to minimize overtopping by storm waves.

Prevention of tombolos

45. For many situations it is desirable to design a breakwater project to prevent tombolo development and encourage only salients. With the desired shoreline advance and sinuosity determined, the breakwater should be located such that the design shoreline would be less than halfway between the original shoreline and the breakwater. Tombolos are prevented by allowing sufficient wave energy to enter the protected region through one or more of the following techniques.

46. Structure length versus distance offshore. From Table 1 and an overview of international breakwater projects, it appears that a tombolo can

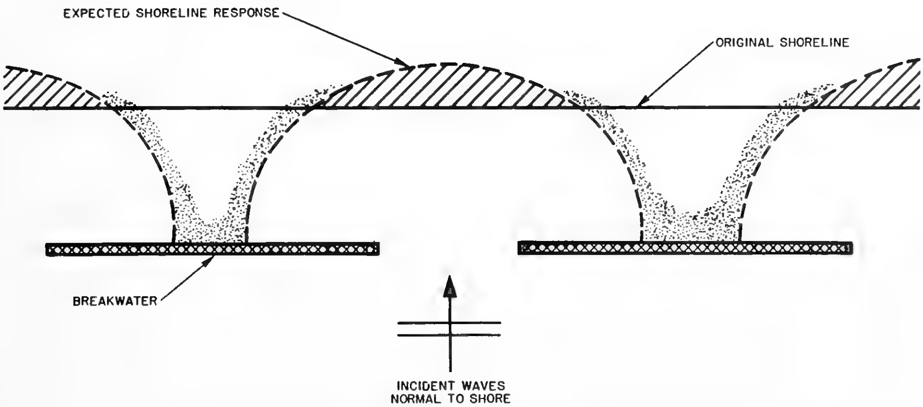


Figure 18. Possible shore response to a segmented detached breakwater with incident waves normal to shore

be prevented if the single breakwater or segment length is equal to or less than one-half the distance offshore (see also Bishop 1982), or simply

$$\ell \leq \frac{1}{2} x \qquad \text{for single detached breakwater}$$

$$\ell_s \leq \frac{1}{2} x \qquad \text{for segmented detached breakwater}$$

This configuration permits intersection of the diffracted wave crests well before the undiffracted portions reach the shoreline, thereby impeding tombolo development. If the predominant wave direction is nearly normal to the shore, the apex of the salient will be located approximately where the diffracted wave crests intersect at the moment the undiffracted portions reach the shoreline. The design theory employed for Lakeview Park postulated that salients would develop where the diffraction coefficient, K_d, equals 0.3 isolines crossed (Walker, Clark, and Pope 1981). The average stable salient configuration was estimated by examining the location of the 0.3 isoline for each predominant wave condition. This method is described in Appendix A.

47. Detached breakwaters designed for protection along an open coast are commonly placed in an average water depth between 1.0 and 8.0 m. If economic or other considerations preclude satisfying the ℓ or $\ell_s \leq 1/2\ x$

criterion, the following means are available for increasing energy flux into the protected region.

48. Wave overtopping. The structure cross-section can be designed to allow wave transmission over the top. The transmitted wave energy prevents connection of the salients to the structure and the salient tends to accrete in a more spatially uniform manner. However, reformed overtopped waves have a higher frequency and irregularity than the incident waves. Water level, wave height and period, and structure slope and roughness all affect, in a nonlinear manner, the amount and form of energy transmitted by overtopping. These parameters are rarely constant; therefore, the rate of overtopping is quite variable. The amount of energy which will pass the structure due to overtopping can be estimated using the procedures of Section (7.23) of the SPM (1984), Seelig (1980) or Douglass (in press). The structure cross-section can be altered so that sufficient energy is transmitted by overtopping to prevent tombolo or large salient formation. An existing structure that is not performing as required could conceivably have its crest raised or lowered, but this is often costly and impractical.

49. Breakwater permeability. Another means of transmitting wave energy is to make the structure permeable. Energy is transmitted at the incident frequency and is generally more predictable and regular than overtopping transmission. Also, wave energy that would be transmitted through a structure is generally more uniform than diffracted wave energy, resulting in a more uniform shoreline. However, transmission quantities are highly dependent on water level and wave period. Design permeability of the structure can be selected using the procedure contained in Chapter 7 of the SPM (1984) or Seelig (1979) to influence the degree of wave transmission through the structure, if the water level and wave period are predictable within a limited range. It is nearly impossible to economically adjust the permeability of an existing structure as a way of modifying transmitted wave energy except for sealing a permeable one to render it impermeable.

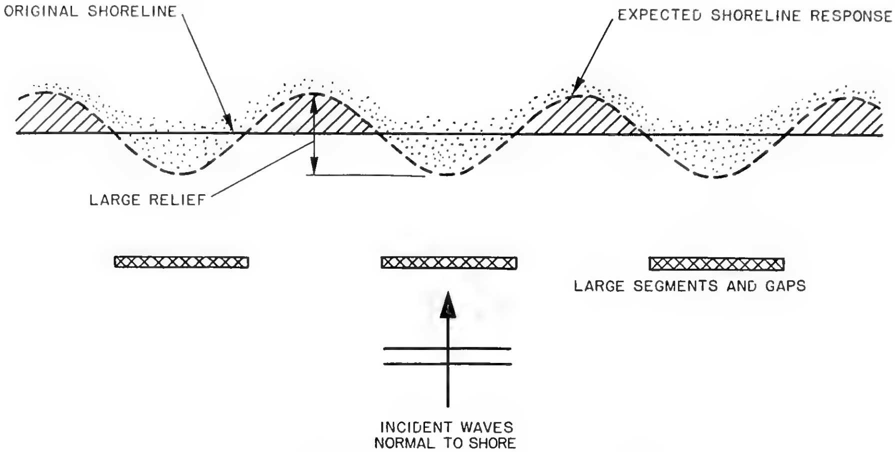
50. Segmentation. The most predictable method at present for influencing transmitted wave energy is by utilizing a segmented design. Segmented detached breakwaters are especially useful when protecting a long section of shore and tombolo development is not desired. Segmenting permits a predictable proportion of wave energy to enter the breakwater's lee and at the same time allows the structure to be built in an economical water depth. Waves in

the lee of the structure will have the same period as the incident waves. A properly designed segmented detached breakwater provides very effective storm protection. The protected beach can accrete sufficiently to survive storm-generated erosion while maintaining the natural longshore transport rate during normal wave conditions.

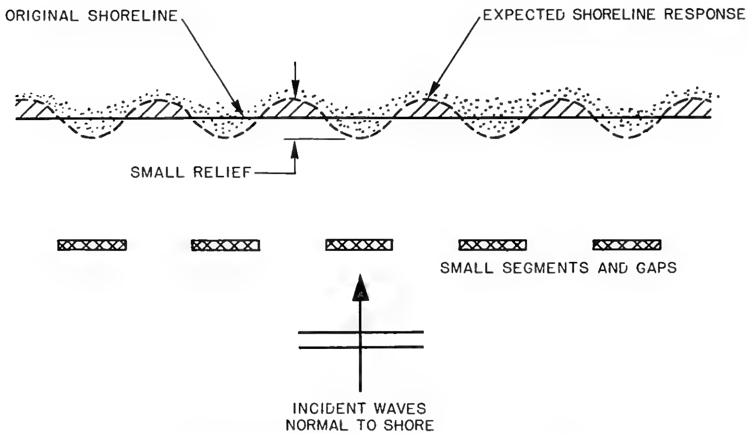
51. The amount of wave energy reaching the lee of a segmented structure is controlled by gap size and diffraction around the breakwater ends. The length of each structure segment should be on the order of one-half the distance to shore. After selecting the ratio of total segment length to total project length, it is necessary to design the number and lengths of the gaps. Gaps should be at least two wavelengths wide relative to those waves which cause the average, nonstorm sediment transport. Wide gaps will cause the shoreline to respond with various spaced salients and embayments. The planform relief (i.e., the distance perpendicular to shore from the salient tip to the shoreward-most point on the eroded embayment) will be great, and therefore will not provide uniform storm protection along the project. Increasing the number of gaps and shortening the length of each will promote a shoreline planform of less relief, and thus provide more uniform protection (Figure 19). Figure 20 shows a segmented breakwater constructed near Rome, Italy. Note the more uniform accretion behind the shorter segments. The gaps between the longer segments are probably too small.

52. Only under certain circumstances will the shoreline directly behind a gap stabilize in a position seaward of its original or filled location. The gap size and the distance offshore must be such that there is significant reduction in wave energy opposite the gap. If the volumetric retreat behind the gaps is less than accretion behind the segments, the project will cause a net increase in the local quantity of littoral material. Unless fill is placed, this increase will come from the existing littoral system and may damage adjacent shores.

53. If a uniform level of shoreline advance is necessary, segmented breakwaters separated by large gaps should not be used. A single long structure designed to permit wave transmission by either overtopping or permeability, or a segmented structure with numerous small gaps, possibly including underwater sills, should be considered to achieve a more uniform shoreline advancement and formation of a perched beach.



a. Large-amplitude shoreline sinuosity



b. Small-amplitude shoreline sinuosity

Figure 19. Change in shoreline response due to reducing segment length and increasing gap number

Figure 20. Segmented detached breakwaters showing the effects of varying the segment length and the number of gaps; near Rome, Italy (circa 1971)

54. Combined response control techniques. The designer may use a combination of the previous techniques to prohibit tombolo formation. The amount and spatial variation of energy reaching the zone behind the breakwater is the sum of transmission from (a) the ratio of structure length to distance offshore, (b) wave transmission through or over the structure, and (c) diffraction of wave energy through breakwater gaps.

Location with respect to breaker line

55. Placement of the breakwater landward of the normal breaker zone will advance the shoreline and may cause tombolo development. When the breakwater is positioned well inside the breakers, a large percentage of the total

longshore transport will pass seaward of the structure and effects on the neighboring shoreline will be less drastic. However, structural integrity may become a problem due to scour at the structure toe. Figure 21 shows a segmented low-crested, permeable breakwater that was placed inside the breaker zone. Substantial accretion occurred but there is still access to the water. The structure was built at a shallow depth, permitting more economical construction.

Figure 21. Permeable, overtopped, segmented breakwater located landward of breaker zone; Kakuda-Hama, Japan (1976)

Structure orientation

56. Orientation of the breakwater with respect to both the predominant wave direction and the original shoreline can affect the size and shape of the resulting salient(s). A change in structure orientation changes the diffraction pattern at the shoreline and subsequently the equilibrium planform configuration. The general shape of a salient with incident waves normal to the shoreline can be estimated by determining the configuration of the diffracted wave crests (Chapter 2 of the SPM). It may be advisable to orient the structure parallel to the incoming wave crests when the local predominant incident waves are oblique to shore. A longer stretch of shoreline will be protected per length of structure, and toe scour at the breakwater head sections will be reduced. The quantity of material required to build the breakwater may increase if one end is located in deeper water instead of being oriented parallel to the bottom contours. Figure 22 illustrates the effect of structure orientation on shoreline configuration.

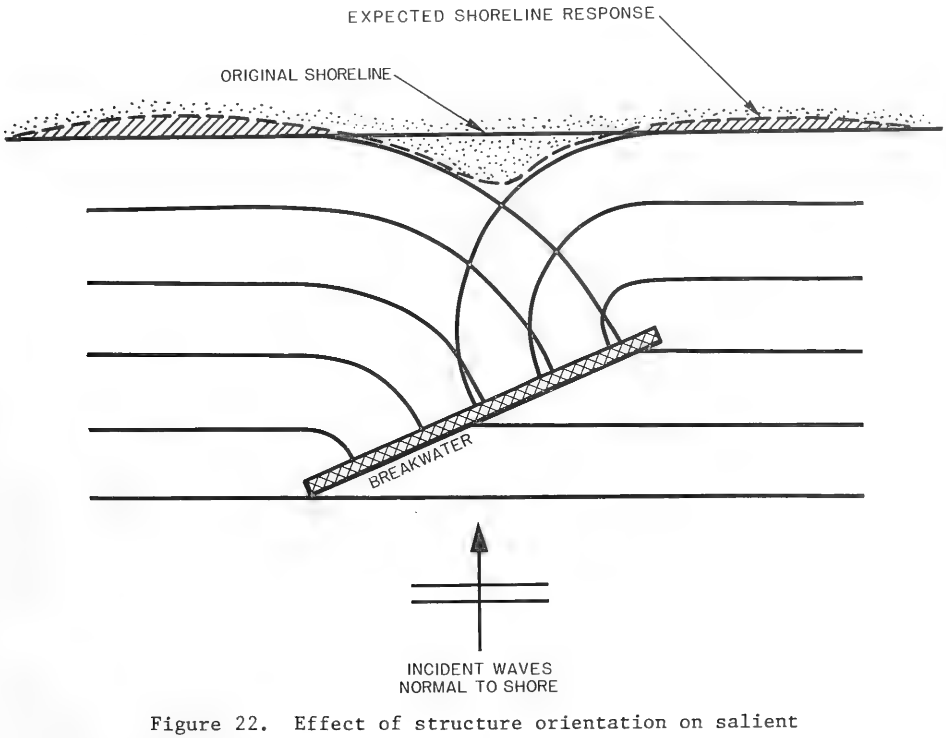


Figure 22. Effect of structure orientation on salient

Other Considerations

57. There are several other factors which can affect detached breakwater design. These considerations include (a) ecology, (b) aesthetics, (c) return flow through breakwater gaps, (d) beach user safety, (e) navigation, and (f) construction. Structural aspects such as foundation design, scour protection, cross-section shape, and stone sizing and placement are not discussed in this report. Information on these design aspects can be found in Chapter 7 of the SPM.

Ecology

58. The design analysis of detached breakwaters should include an appraisal of the environmental impact of the project. Rounsefell (1972) discusses the ecological effects of offshore construction and Thompson (1973) examines the ecological effects of offshore dredging and beach nourishment. While these studies suggest that detached breakwaters generally should not cause long-term undesirable ecological changes, each proposed project site is unique and must be examined for possible negative impact to the ecological system.

59. An ecological concern that has been associated with detached breakwaters in countries such as France and Israel is the effect of reduced circulation on water quality. If wave transmission over or through the structure is too limited, the exchange of water in the embayments can be reduced significantly, raising the possibility of the entrapped water becoming stagnant and unhealthy. Regions where water level variations are small may be especially susceptible to this problem. Making the breakwater gaps larger or more numerous may increase the water circulation in segmented structures.

Aesthetics

60. If a breakwater is to be constructed to protect a recreational beach, aesthetics should be considered. Bathers sitting on the beach like to see the horizon, and the height of the breakwater may need to be reduced accordingly. This can influence the structural design by increasing wave overtopping. Reducing permeability, increasing structure length, and reducing gap size for segmented breakwaters are ways to compensate for the increase in energy transmission due to overtopping. Some areas may have a particular aesthetic appeal because of high surf. Breakwaters reduce that aesthetic value, which may be perceived as a disadvantage.

Return flow through breakwater gaps

61. Return currents through the gaps of segmented breakwaters can increase offshore sand losses and possibly cause scour at the segment heads. These currents may pose a hazard to swimmers who venture too close to the structure's gaps. The currents usually occur when the structure is nearly impermeable and low crested so that the water transmitted by overtopping can return only through the gaps or around the ends of the structure. This problem may only arise during storm conditions when significant overtopping occurs. The return currents can become particularly strong if the breakwater is long and has only a few gaps. A method for estimating the magnitude of these currents is presented in Seelig and Walton (1980). Return currents can be reduced by raising the breakwater crest elevation, enlarging the gaps between segments, or increasing structure permeability.

Beach user safety

62. Coastal structures can present hazards to beach users who may be inclined to swim too close to the structures or climb on them. A tombolo allows easy access to the breakwater structure and invites such activity. The number of people inclined to wade or swim out to the breakwater can be reduced by placing it farther offshore. A more reasonable treatment of the problem is proper beach supervision. A beach which is protected by a breakwater cannot be used safely by surfers unless the breakwater is segmented and the gaps are quite wide.

Navigation

63. If boats are commonly launched from a beach that is to be protected by a long detached breakwater, gaps may be necessary to provide access to the ocean, as was the case at Winthrop Beach. Detached breakwaters may need to be marked with navigation aids if their location or distance offshore poses a potential hazard to boaters. This is especially true if the structures are low crested.

Construction

64. Because of the difficulties associated with predicting shoreline changes caused by segmented detached breakwaters, it may be prudent to first build a small prototype test breakwater, or the entire breakwater using segments with large gaps. In subsequent years, the structure could then be adjusted by partially closing the gaps. The expected shoreline change behind the structure should be compensated for by placing a volume of sand equal to

that which the new beach will require. This will reduce starvation of downdrift beaches. If a number of breakwaters or a long segmented structure is to be built, construction should begin at the downdrift end of the project and proceed updrift to promote a more spatially uniform accretion of the shoreline. Constructing the most downdrift portion first reduces construction-induced erosion of the project beach.

65. Construction limitations may play a major role in determining the water depth in which the breakwater is to be placed. There is a zone where construction often is impractical without highly specialized equipment. Its landward boundary is the maximum depth at which land-based machinery can operate (say 1 to 1.5 m) and its seaward boundary is defined by the draft of floating construction vessels (say 2 to 3 m). Wave activity and tidal range greatly affect the boundaries of this zone. Most large-scale detached breakwater projects require sea-based construction. However, if wave activity during construction will be slight, sand access roads to the breakwater location can be constructed with fill material. In this way, land-based machinery can work farther offshore than is normally accessible. The access roads should be removed and/or the fill material redistributed along the shore after construction, to prevent the project from functioning as a T-groin. This construction procedure was employed at Colonial Beach. Conversely, floating construction equipment can work closer to shore than normal by dredging a channel to the nearshore. This technique was employed at Presque Isle.

PART V: DESIGN PROCEDURES

66. This part recommends a procedure for designing detached breakwaters for shore protection and beach stabilization. There are three major steps in the procedure. The first step is the initial desk-top design, which uses the material presented in Part IV in conjunction with experience with the local coast and general scientific judgment. The second step is to refine the design using either physical or numerical model tests, or both. Finally, field tests may be performed to verify and adjust the design. The scope and scale of the project will influence the degree to which each of these steps is utilized.

Initial Design

67. The first step in designing a detached breakwater project is to select the proper design conditions. The average range of wave-climate and sediment-transport conditions at the project site, as well as the extreme conditions, will control the eventual stable beach planform. The breakwater configuration must be based on a number of design conditions, including (a) extreme storms, (b) average seasonal patterns, (c) periods of unusual quiescence, and (d) the factors controlling the bulk of sediment transport. Selection of design conditions is perhaps the most difficult and critical task in designing structures for beach erosion control. Although a number of years of directional wave data are needed to properly select the design conditions, such data are rarely available. Usually the design conditions must be assumed based on hindcast climatology, LEO, and the historic shoreline response. A coastal geology study of the historic nearshore condition can reveal much about the seasonality, extremes, and long-term evolution of the project site. Aerial photography, historic maps, surveys and sediment sampling programs, and subsurface data may be used to provide information on bar formation and migration, seasonal and storm profiles, shoreline response to existing structures, and variations in littoral transport. Most of the following design conditions discussed will be those associated with the average sediment transport climate.

Project length

68. Determine the length of shoreline to be protected. Neighboring

beaches, especially downdrift, may be subject to erosion and the project length may be adjusted to tie into littoral nodal areas, hard points, or other structures so that there is minimal disruption of adjacent shores. As project length is usually limited by legal and economic factors, downdrift mitigation may be necessary.

Desired protection and shoreline advance

69. Determine how far seaward the average shoreline should be advanced. If the purpose of the project is to provide and stabilize a recreational beach, ascertain the required area and beach frontage from beach use studies. Surge and erosion analyses are needed if additional beach width is provided for storm protection. The average shoreline advance probably should be no more than approximately 25 percent of the project length, although the maximum can be substantially more, especially if tombolos are generated. In general, regulations prohibit extending the shoreline beyond the documented historical shore for authorized Federal projects. If the desired average additional beach width is greater than approximately 20 percent of the project length, additional structures such as terminal groins should be considered.

Salient shape

70. Resolve which of the three general shoreline configurations is acceptable (single salient, multiple salient, or uniform shoreline). From the project length and the required average shoreline advance, determine which configuration(s) are desirable and feasible. Also, determine if a tombolo or salient is preferred.

Sediment supply

71. Determine if the regional longshore drift will supply all the sand to be retained by the structure, or if fill will be added. If the region is sediment-starved, terminal structures may be needed to help contain the fill. Determine what the effect will be on neighboring beaches from sediment trapped by the structures or from beach nourishment sand leaking out of the project.

Structure type and planform geometry

72. The desired and feasible shoreline configuration(s) determine the type of breakwater to be designed, and pose limits on planform geometry. A simple diffraction analysis is a very valuable tool for estimating shoreline response to the proposed structure plan and should be part of the initial design. General design guidance follows for various beach planforms:

a. For nonuniform protection over short distances use a single detached breakwater.

(1) Tombolo. The structure should be at least as long as the shoreline to be protected and perhaps longer to ensure enough shoreline advance along the entire project. The breakwater should be placed offshore a distance between two-thirds and one-half times its length. If the water depth at this location is too great, move the structure landward, keeping in mind the possibility of double tombolo formation or of the structure ultimately acting as a seawall. Otherwise, a different shoreline configuration should be pursued. Design the breakwater to be impermeable and of sufficient crest elevation to minimize overtopping during storm events.

(2) Salient. As in the design for a tombolo, the structure should be at least as long as the project. Make the distance offshore between one-and-one-half and two times the length of the breakwater and design for low permeability and infrequent overtopping. If the water depth at this location is too great, move the structure landward and increase wave transmission by increasing permeability and/or overtopping.

b. For nonuniform protection over longer distances use a segmented breakwater.

(1) Tombolo. The approximate size of the tombolo which will provide the required average beach width will dictate the distance offshore. The segment length should be roughly one-and-one-half times this distance. Gap width will depend on the design shoreline position opposite the gap. If the region has a substantial longshore transport rate, or beach fill is to be added, it may be possible to widen the beach opposite the gap. Make the gap at least one wavelength wide, but no greater than the segment length. If the predominant wave direction is directly onshore, the gaps will have to be very narrow for the opposite shoreline to be advanced substantially. Use diffraction diagrams to determine the reduction in wave height opposite each gap. Alter the gap size, keeping in mind the possible effects of a large water-level range and overtopping driving currents through the gaps, or a small water-level range causing stagnation in the embayment. If the water depth at the structure is too great for feasible and economical construction, the desired tombolo size, segment length, distance offshore, and gap width may need to be scaled down. Determine the number of segments (and gaps) required to cover the length of the project. Slight adjustment of the segments and gap length or perhaps just the end segments may be necessary to cover the required length of shore. Design each segment to be impermeable and overtopped infrequently.

(2) Salient. Select a size for the salient that will provide the desired average beach width without unacceptable levels of recession behind the gaps. The ratio of segment length to distance offshore should be between 2:3 and 1:2 for impermeable, nonovertopped structures. If the water depth is too great, move the segments landward and increase their permeability or overtopping characteristics. Size the gaps according to the desired beach width opposite each gap and the regional longshore drift.

c. For nearly uniform protection over a long distance, use a highly permeable, partially submerged, or frequently segmented structure.

(1) Connected shoreline. Uniform shoreline advance that connects to the structure is not recommended because it would block the nearshore sediment transport.

(2) Unconnected shoreline. The structure length will be slightly longer than the shoreline to be protected. Compare the seasonal wave climate and shoreline positions to obtain a gross estimate of the energy reduction required to stabilize the shoreline in the desired position. Uniform shoreline advance is not possible if too much incident energy is blocked by the structure. This is particularly true for areas where the waves are seldom normal to the shore and where there is significant gross longshore transport. Roughly 60 percent or more of the wave energy should pass behind the structure. If the wave direction is predominantly shore-normal and sediment movement at the site is generally on/offshore, a greater percentage of wave energy can be blocked while still permiting a uniform shoreline advance. The breakwater should be located well outside the normal surf zone. There are three methods that can be utilized to transmit wave energy in this case. Wave overtopping is the least predictable and least manageable method of wave transmission and should not be used unless economic constraints deem it necessary. Wave transmission due to structure permeability is more feasible if artificial armor units are used. The most predictable and practical method of uniform wave transmission is to build a highly segmented breakwater. This is accomplished by making the segments and gaps numerous and very short. The segments should be impermeable and overtopped infrequently. The distance offshore should be greater than eight times the segment length to provide sufficient distance for the diffracted waves to reorient themselves via refraction before reaching the shoreline. Combining two or all three of the wave transmission techniques is possible but highly complex when used in a design.

Structure orientation

73. In most situations, detached breakwaters should be oriented parallel to the preproject shoreline. However, if the predominant wave direction at the structure is very oblique to the shoreline (>30°) the breakwater may be oriented parallel to the waves to block the incident energy. Waves from other directions will be blocked less efficiently. A diffraction analysis should be performed to identify any important changes in the wave pattern or distribution in wave height due to the structure orientation.

Modeling

Physical modeling

74. The initial design(s) for a major detached breakwater project should be tested and refined using physical model experiments. Physical hydraulic models have proven to be a practical tool for the functional design of detached breakwaters. This section will discuss how these model studies are conducted and how they can be used to assist in developing a design. Four previous model studies performed at the US Army Engineer Waterways Experiment Station (WES) are briefly examined in Appendix C.

75. As seen in Part IV, many variables must be considered in designing detached breakwaters. A physical model can be used to great advantage for such situations, since parameters such as wave height, wave period, wave spectrum, wave angle, water level, structure height, segment length, distance offshore, etc., can be varied within model limits and the effects on wave height and wave-generated currents determined. Sediment movement trends can be qualitatively reproduced.

76. Since short-period wind waves are usually the primary force responsible for nearshore sediment transport, a physical model for detached breakwater design is based on Froude's Law and is constructed geometrically similar to the prototype. Previous detached breakwater studies have varied in scale ratios from 1:50 to 1:100. Larger scale ratios minimize wave attenuation by surface tension, internal friction, and friction in the bottom boundary layer. Proper scaling of the equivalent hydraulic size for the sediment used in movable-bed models and bottom friction effects create significant problems with modeling sediment transport. However, in large-scale models (i.e., 1:50

to 1:75) these factors may be adjusted for analytically. Friction reduces transmitted wave energy, but previous model tests have provided scaling relationships for selecting the unit size used in the model to achieve the correct wave transmission. Details on scale effects are discussed by Hudson et al. (1979). Models used to reproduce short-period wave effects are normally undistorted. Therefore, wave patterns due to refraction (governed by the vertical scale) and diffraction (governed by the horizontal scale) can be produced simultaneously.

77. The types of physical models which may be used to determine the effect of detached breakwaters on the nearshore environment are:

a. Fixed-bed models.

b. Fixed-bed models with sediment tracers.

c. Movable-bed models.

78. Fixed-bed models are usually molded out of concrete to accurately reproduce the preproject bathymetry. The model is then used to examine the initial response of wave-generated current patterns to the breakwater as superimposed on the preproject bathymetry. This is especially useful when the preproject condition contains other structures such as groins. The interaction of the existing structures and the planned breakwater on wave-generated currents can then be examined. The breakwater location may be adjusted to reduce the possibility of strong rip currents and other undesirable effects which may, in nature, cause local scour around the structures, offshore transport of littoral material, and hazards to swimmers. Fixed-bed models also may be used to determine wave heights both at the seaward toe of the structure where it can be used for computing the structure cross-section and also behind the structure where the wave attenuation characteristics can be compared for various wave conditions, water level, breakwater lengths, and distances offshore.

79. Fixed-bed models with sediment tracers are an extension of fixed-bed testing. A thin layer of tracer material is introduced on the fixed-bed surface and its movement observed. Previous studies and analytical work indicate which materials tend to best simulate prototype sediment movement (Hudson et al. 1979). Also, information from the prototype may aid in determining an appropriate material to simulate sediment movement. Field studies such as tracer tests may aid in determining how sediment transported either alongshore or cross-shore enters and leaves a project region and where it is distributed.

These studies may be used to develop the model tracer tests to determine the optimum distance offshore for the breakwater relative to the longshore sediment movement. Model tracer tests can be used to qualitatively duplicate tombolo development behind a breakwater, illustrating the points of tombolo attachment for different wave angles and orientations of the breakwater. Since the offshore contours do not adjust in a fixed-bed model, extrapolation of the bathymetry change through time may not be valid, and a movable-bed model may be desired.

80. Movable-bed models may be used to determine beach planform and bathymetry response to a detached breakwater and to simulate beachfill readjustment shoreward of the breakwater. Ideally, a movable-bed model requires verification of past beach changes in the study area. This in turn requires significant amounts of prototype data. Movable-bed models require longer running time per test to allow the development of an equilibrium beach condition. In addition, the model results must be considered as qualitative due to the difficulties associated with reconciling the different scaling requirements of waves and sediment transport. For any particular study one or all three modeling approaches may be used.

81. For any physical model study, prototype data are necessary. Fixed-bed studies require up-to-date bathymetry and historical aerial photography to ensure that the average beach planform is selected. Nearshore current patterns (including locations of rip currents) are useful in ascertaining the model's accuracy. Beach profile studies are useful in understanding the beach dynamics and for planning the structure location. Good wave and water-level data are necessary to select the appropriate test conditions. Detailed sediment sampling also is desirable. If a movable-bed model study is undertaken, more field data are required. Periodic seasonal bathymetric surveys of the study area along with a detailed record of the waves, water levels, and winds which occurred during the survey period are very valuable. A minimum of two years of data should be collected.

Numerical modeling

82. Numerical models are destined to become a valuable and practical design tool for predicting shoreline response to various structure configurations. Simple models designed to predict zones of scour and erosion in the fluvial environment have been available for many years from the Hydrologic Engineering Center (HEC 1975) and recent site-specific models have been

developed for the littoral environment (e.g., Kraus 1983). However, generally applicable models are only just becoming available.

83. The simplest form of a numerical model is a diffraction analysis with resultant energy flux calculations along the shoreline. A model which computes shoreline response to wave conditions through time as the waves diffract, refract, and shoal over a homogeneous offshore slope and around structures is called a "1-line" model (Kraus 1983). A "multi-line" or "N-line" model can account for a downward-sloping, heterogeneous offshore bathymetry to transform the deepwater wave and modify the bathymetry to develop an equilibrium profile and a stable beach planform configuration (Perlin and Dean 1983). Field verification of the basic model theory is necessary before widespread design application.

84. Any model, physical or numerical, is only valid if the input parameters, particularly the wave conditions, are appropriate for the project site. A few degrees of variation in approach angle, incorrect wave periods or heights, or an improper distribution of the relative directional wave dominance can all result in erroneous predictions. In addition, the assumptions and limitations inherent in any model can greatly affect its results and how they should be applied. The designer must understand how the model works in order to develop the correct input and correctly interpret the model predictions. There needs to be significant interaction between the designer and the modeler to assure reliable site-specific adaptation of the model and a valid interpretation of the model predictions.

85. In summary, neither a numerical model nor a physical model should be used as the only mechanism for determining a breakwater configuration. At present, no single model can account for all the factors which might affect a specific project site. Sediment transport on bars and bar migration, regional and local current patterns, water-level changes, variability in the sediment supply, wave reflection, rip currents, the interaction of long period swells and local sea state, etc., can all significantly affect the actual sediment response, but may be difficult or impossible to include in a single model.

Field Tests of Breakwater Design

86. Even with meticulous initial design and extensive physical or numerical model tests, shoreline response to a detached breakwater project is

difficult to estimate. Whenever possible, on-site field tests of the proposed structure(s) should be performed. Because of the large costs associated with breakwater construction and even greater cost of modification, field tests may prove to be cost-effective, especially for larger projects. A test structure could be built or material could be added later to the prototype structure to adjust the design through staged construction.

87. For a single breakwater project, adjustment of the design could be approached by constructing the breakwater so it is shorter than initially designed and perhaps has a lower crest elevation to allow more overtopping transmission. Shoreline response should be monitored for a suitable length of time, at least two years. If the beach accretion is insufficient, the design of the structure may be adjusted by lengthening the structure and/or raising its crest elevation. This process may require several repetitions before a suitable shoreline configuration is reached. The total construction cost for the project probably would be significantly increased by the practice of field modification; therefore, it may not always be practical. It is usually impractical to alter the orientation or increase the permeability of an existing breakwater.

88. If a long stretch of shoreline is to be protected by a segmented breakwater, several test segments should be built, starting at the downdrift end of the project or the area most in need of protection. The segments might vary slightly in length, distance offshore, gap size, orientation, or even crest elevation and permeability. The segment with the most satisfactory shoreline response can then be used as the final breakwater design. When judging the shoreline response to each segment, be wary of the effects of segment interaction, such as an updrift segment blocking the longshore sand supply to downdrift segments. Presque Isle, Pennsylvania, and East Harbor, Ohio, are the only sites in the United States where breakwater field tests are presently being conducted in anticipation of a future, larger project.

PART VI: SUMMARY AND CONCLUSIONS

89. Detached breakwaters, especially segmented ones, are a viable and cost-effective alternative for many shoreline erosion and beach stabilization problems. However, no means presently exists for quantitatively predicting beach response to these structures. The functional design of a detached breakwater is, not unlike other coastal engineering design problems, an empirical process. Review of the literature and examination of existing breakwater projects provide substantial insight and some qualitative design guidance. The natural parameters which are most important are those which affect wave diffraction (wave length, height, direction, and the gap width-to-wavelength ratio for segmented breakwaters), natural beach slope, water-level range, native sediment size, and available supply of sediment. The techniques available for controlling beach response include: (a) variation of the ratio of breakwater length to distance offshore, (b) location of the structure with respect to the breaker line, (c) orientation of the structure with respect to the original shoreline and the predominant wave direction, and (d) the degree of wave transmission by overtopping, permeability, or by segmenting the breakwater. Other topics which should be considered when designing detached breakwaters include ecology, safety, navigational aspects, aesthetics, and currents through breakwater gaps.

90. A three-phase approach is recommended for the functional design of detached breakwaters. First, based on the material presented in this report, one or more initial designs should be developed and the shoreline response under various conditions predicted. Physical and/or numerical model tests should then be performed to find and improve upon the best plan. Finally, field tests could then be conducted to arrive at a final configuration.

REFERENCES

Bishop, C. T. 1982. "A Review of Shore Protection by Headland Control," Study No. 352, National Water Research Institute, Canada Center for Inland Waters.

Bottin, R. R., Jr. 1982. "Lakeview Park Beach Erosion Study, Ohio," Letter Report, US Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, Miss.

Curren, C. R., and Chatham, C. E., Jr. 1977. "Imperial Beach, California, Design of Structures for Beach Erosion Control; Hydraulic Model Investigation," CERC Technical Report H-77-15, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

__________. 1980. "Oceanside Harbor and Beach, California, Design of Structures for Harbor Improvement and Beach Erosion Control; Hydraulic Model Investigation," CERC Technical Report HL-80-10, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

Douglass, S. L. In press. Review and Comparison of Methods for Estimating Irregular Wave Overtopping Rates, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

HEC. 1975. "Scour and Deposition in Rivers and Reservoirs," US Army Engineer Hydrologic Engineering Center, Davis, Calif.

Hudson, R. Y., et al. 1979. "Coastal Hydraulic Models," CERC Special Report No. 5, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

Kraus, N. C. 1983. "Applications of a Shoreline Prediction Model," Proceedings of Coastal Structures '83, American Society of Civil Engineers, pp 632-645.

Lesnick, J. R. 1979. "An Annotated Bibliography on Detached Breakwaters and Artificial Headlands," CERC Miscellaneous Report 79-1, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

Perlin, M., and Dean, R. G. 1983. "A Numerical Model to Simulate Sediment Transport in the Vicinity of Coastal Structures," Miscellaneous Report 83-10, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

Pope, J. In press. "Segmented Offshore Breakwaters - An Alternative for Beach Erosion Control," Proceedings of the Coastal Society Ninth Annual Conference, Atlantic City, N. J.

Pope, J., and Rowen, D. D. 1983. "Breakwaters for Beach Protection at Lorain, Ohio," Proceedings of Coastal Structures '83, American Society of Civil Engineers, pp 753-768.

Rounsefell, G. A. 1972. "Ecological Effects of Offshore Construction," Marine Science, Marine Science Institute, Bayou LaBatre, Ala., Vol 2, No. 1.

Seabergh, W. C. 1983. "Design for Prevention of Beach Erosion at Presque Isle, Erie, Pennsylvania," CERC Technical Report HL-83-15, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

Seelig, W. N. 1979. "Estimation of Wave Transmission Coefficients for Permeable Breakwaters," CETA 79-6, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

__________. 1980. "Two-Dimensional Tests for Wave Transmission and Reflection Characteristics of Laboratory Breakwaters," CERC Technical Report 80-1, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

Seelig, W. N., and Walton, T. L., Jr. 1980. "Estimation of Flow Through Offshore Breakwater Gaps Generated by Wave Overtopping," CETA 80-8, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

Shore Protection Manual. 1984. 4th ed., 2 vols, US Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, US Government Printing Office, Washington, DC.

Silvester, R., and Ho, S. 1972. "Use of Crenulate Shaped Bays to Stabilize Coasts," Proceedings to the Thirteenth Conference on Coastal Engineering, American Society of Civil Engineers, Vol 1, pp 1347-1365.

Thompson, J. R. 1973. "Ecological Effects of Offshore Dredging and Beach Nourishment," CERC Miscellaneous Paper 1-73, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

Toyoshima, O. 1972. Coastal Engineering for Practicing Engineers - Beach Erosion, Morikita Publishing Co., Tokyo, Japan, 320 pp. (English translation available through the Coastal Engineering Research Center for Chapter 8 on "Offshore Breakwaters," pp 227-317). US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

US Army Engineer District, Buffalo. 1975. Cooperative Beach Erosion Control Project at Lakeview Park, Lorain, Ohio, General Design Memorandum, Phase II - Project Design, Buffalo, N.Y.

Walker, J. R., Clark, D., and Pope, J. 1981. "A Detached Breakwater System for Beach Protection," Proceedings of the Seventeenth Coastal Engineering Conference, American Society of Civil Engineers, Vol 2, pp 1968-1987.

Weishar, L. L., and Byrne, R. J. 1978. "Field Study of Breaking Wave Characteristics," Proceedings of the Sixteenth Coastal Engineering Conference, American Society of Civil Engineers, Vol 1, pp 487-506

APPENDIX A: SCHEME OF INITIAL DESIGN USED AT LAKEVIEW PARK

1. Presented in this Appendix is the scheme used for designing a segmented detached breakwater and an artificially placed recreational beach at Lakeview Park, Ohio, on Lake Erie (Figure 11, main text). A complete description of the history, design, construction, and performance of the project is given by Walker, Clark, and Pope (1981) and Pope and Rowen (1983).

2. The designers wanted to prevent tombolo formation and avoid starvation of adjacent beaches. Based on a review of existing single breakwater projects, they postulated a criterion based on the diffraction coefficient of waves from the net easterly and westerly directions (Figure A1). Using diffraction diagrams such as Figures 2-28 to 2-58 of the SPM, the 0.3 diffraction coefficient (Kd) isolines for waves from the west at the western end of each segment and waves from the east at the eastern end of each segment were drawn on a chart (Figure A1). If the wave conditions and breakwater configuration were such that these isolines crossed lakeward of the original shoreline, then it was postulated that enough energy penetrated the shadow zone to prevent tombolo development. This has proven to be the case at Lakeview Park. The $Kd = 0.3$ isolines crossed lakeward of the filled beach and the shoreline salients have not connected to the structure. The shape of the equilibrium shoreline is approximated by drawing in the diffracted wave crests. At Lakeview Park, the general position and orientation of the shoreline is determined by the length of the terminal groin at the east end of the project, the predominant wave direction, and the amount of fill placed. The structure did not cause accretion of a large expanse of beach, although the project has entrapped slightly more material than was artificially placed. The shoreline response reflects not only the effects of the breakwater, but also the terminal groins and the placed beachfill.

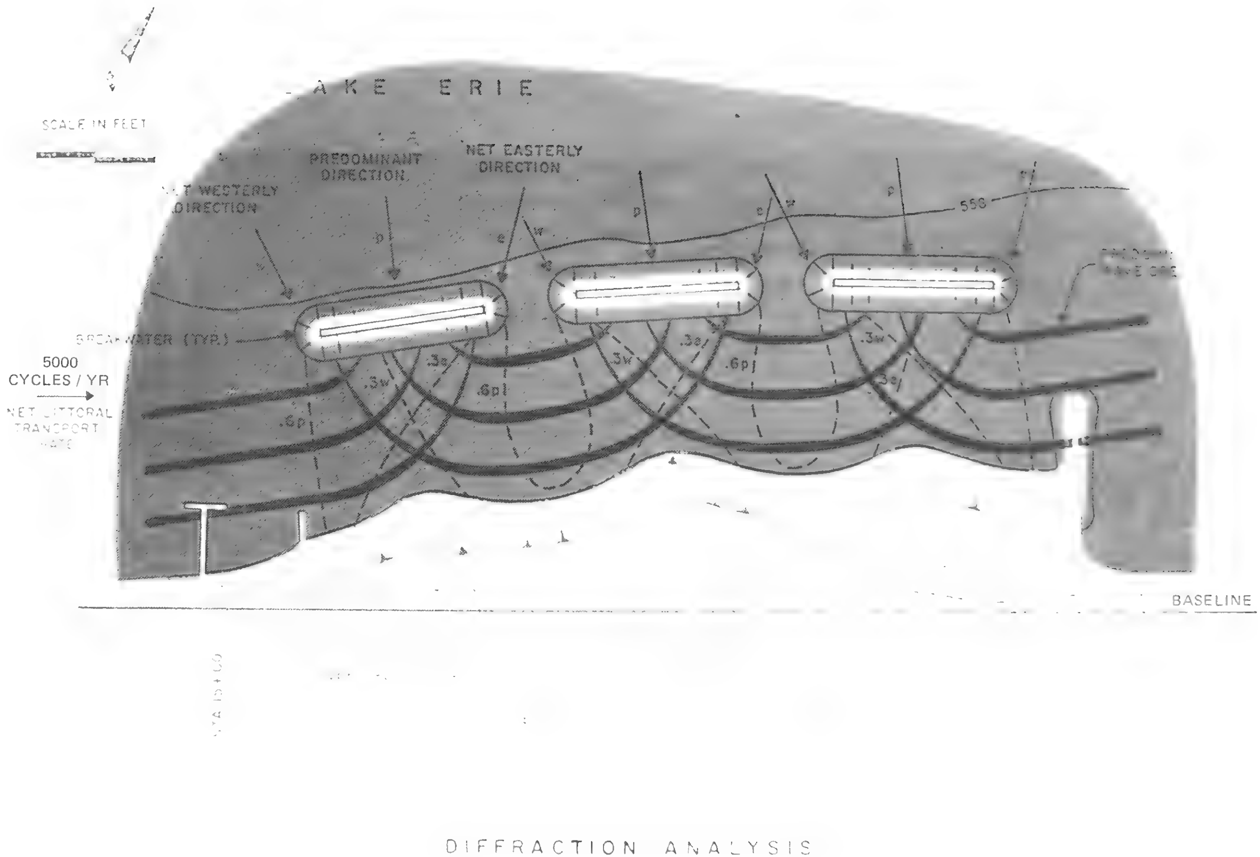


Figure A1. Lakeview Park design scheme (US Army Engineer District, Buffalo 1975). (To convert feet to metres, multiply by 0.3048)

APPENDIX B: EXAMPLE DESIGN PROBLEMS

1. The example problems contained in this Appendix illustrate several types of evaluations which may be appropriate for designing a detached breakwater project. Although the scenarios are slightly contrived, they emphasize the importance of understanding the local wave climate, local sediment transport history, and the project's intent. General breakwater design guidance can be derived from examining existing projects. Selection of a design configuration for any particular project is ultimately a qualitative art. The plans presented in these example problems should subsequently be tested via a diffraction analysis and a numerical and/or physical model.

Problem 1: Design of a Single Tombolo

Problem

2. An important building is threatened by erosion because of an updrift harbor blocking the predominant longshore transport and offshore losses associated with frequent storms. The owners of the building own large segments of beach in each direction and are not concerned with any effects the protection may have on neighboring shores. The area to be protected is 100 m long and is presently only 30 m from the normal high tide line. The predominant breaking wave climate has a period of 7 sec, a height of 1.2 m, and produces a slight longshore transport from left to right, as shown in Figure B1. The average nearshore slope is 1 on 50 and the diurnal tide range is 1.0 m. The design storm surge is approximately 2.0 m and the design breaker height is 4.0 m. A protection scheme can be designed by using a detached breakwater (Figure B1).

Solution

3. Because the effects on the adjacent shoreline are not important, and the task is to provide maximum protection to a short segment of beach, the structure is designed to develop a single tombolo. The location of the normal breakwater line (for $H_i = 1.2$ m) is estimated to be approximately 75 m offshore. To trap as much sediment as possible while placing the structure in a reasonable water depth, it is suggested that the breakwater be located outside the breaker line, 100 m offshore at -2.0 m. To ensure tombolo formation a structure length of 250 m is selected, a little more than twice the distance offshore. This length will make the tombolo large enough to provide good

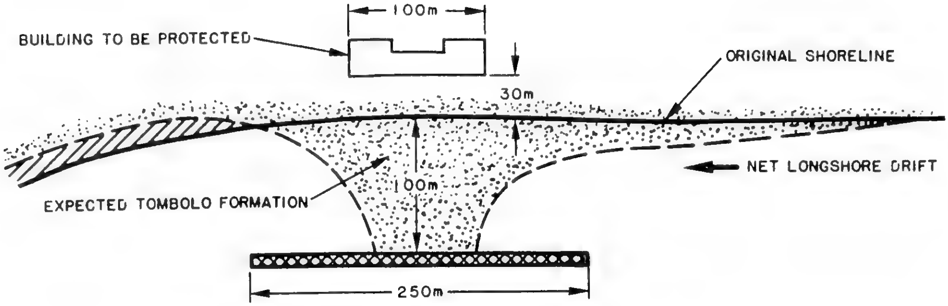


Figure B1. Layout of design for a single tombolo

protection along the entire 100 m of the site. Also to ensure tombolo formation and to promote the survival of the tombolo during storm events, the breakwater should be impermeable and have a crest high enough to prevent significant overtopping by storm waves. The crest elevation required to prevent overtopping during a significant storm with accompanying surge is determined for $H_i = 4.0$ m and $d_s = -4.0$ m MSL using Chapter 7 of the SPM. Figure B1 shows the site plan of the project and the expected adjusted shoreline.

Problem 2: Design of a Series of Unconnected Salients

Problem

4. A submerged rock intrusion partially blocks the predominant longshore transport to a beach which contains the remains of a Civil War fortification and is a National Historic Landmark and park facility (Figure B2). The shoreline is responding by assuming a spiral shape. Retreat downdrift of the rock outcrop has already inundated much of the battlements and the remaining ones are threatened. The beach is a popular recreation area. The shore requiring protection is approximately 750 m long. The wave climate has a distinctive seasonality. The affected shoreline accretes approximately 20 m beyond the normal winter beach during the summer when the waves are out of the southeast. It is desirable to avoid any adverse impact to adjacent beaches which are not owned by the park. If the shore fronting the historic site is restored to a general alignment with the updrift beach, sediment can be

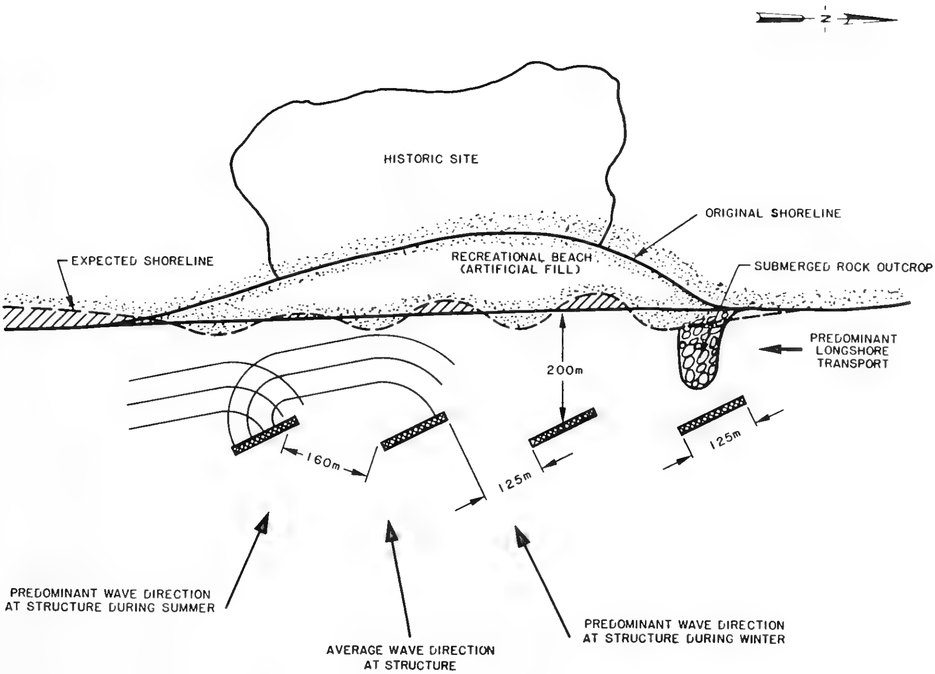


Figure B2. Layout of design for a series of unconnected salients

transported through the project area to avoid such problems. A protection scheme should be devised by the use of a detached breakwater that will halt the erosion of the historic site and provide a recreational beach without any adverse effects to the adjacent nonpark property.

Solution

5. Because the project is relatively long and the longshore drift during the summer is to be maintained as much as possible, a technique for transmitting sand past the structure must be used. One solution would be the use of a segmented structure designed to cause salients to develop. Because the erosion is caused by longshore transport and not offshore transport, the shoreline opposite the gaps can be stabilized in a significantly advanced position. Tombolos should not be allowed to form in order to minimize interruption of the longshore transport and to keep bathers away from the structure. A beachfill is placed, advancing the eroded shoreline to the position of the updrift beach. The gap size and orientation of the breakwater segments

should be such that waves from the northeast (the predominant winter wave direction) will be significantly blocked to prevent loss of the fill, and at the same time allow southerly wave-induced transport to continue during the summer, preventing project-induced erosion north of the rock outcrop. The structure configuration must also prevent tombolo formation when subjected to either seasonal wave condition. To facilitate marine-based construction and provide enough room for salients to form, the breakwater is positioned 200 m seaward of the alignment of the updrift beach and artificial fill, in 3 m of water. From a refraction analysis, the orientation of each of the predominant winter and summer wave crests at the segments is determined. The breakwater segments are aligned perpendicular to the predominant winter waves to provide maximum protection during the winter while allowing a slightly greater proportion of energy through during the summer. Each segment should have a length between two-thirds and one-half the distance offshore and 125 m is deemed appropriate. The percentage of predominant wave energy blocked during the winter months is estimated to be 50 percent and the projected gap size is approximately 125 m. If too much fill is lost the gaps can subsequently be narrowed. During summer months this configuration will have an effective width of 160 m. This should provide enough energy transmission to prevent tombolo formation and avoid problems to the northern beaches. The average orientation of each salient is formed by the predominant net wave direction with the apex of each salient located slightly downdrift of the midpoint for each segment. Although the precise size and shape of the expected salients for any instant of time cannot be determined, a projected average condition is shown in Figure B2. The shoreline south of the project is expected to retreat slightly in response to a reduction in the quantity of sediment normally supplied by erosion of the park frontage. Some placed fill will be lost to downdrift areas, thus reducing this adverse impact. The segments are designed to be impermeable and overtopped infrequently. Construction of the segments should progress from south to north.

Problem 3: Design for Uniform Shoreline Advance

Problem

6. Storm erosion protection is to be provided to a highly developed 2,000-m-long open coast. Protective beach fills have previously been constructed, but the material is continually lost to the offshore during winter

storms, and periodic nourishment has become too expensive. Very little erosion can be attributed to longshore transport because the predominant wave direction is approximately normal to the shore, and there are no local littoral barriers. During the summer, a change in wave conditions causes the shoreline to advance approximately 15 m. The project plan is to retain a beach with a shoreline 30 m seaward of its normal summer position, even during winter storm conditions.

Solution

7. A breakwater plan which creates large variations in the shoreline planform cannot be utilized because it will probably cause shoreline recession opposite the gaps, and will not provide uniform protection along the project. Therefore, a design for uniform shoreline advance will be investigated. An indication of the desired energy reduction is found by comparing the summer wave climate (found to have a predominant breaker height of 1.0 m and period of 8 sec) with the predominant winter waves (1.5 m breaker height and 7 sec period) and noting the summer shoreline advance. Thus, a reduction in wave height of 33 percent and a slight increase in period should advance the shoreline 15 m. It appears that greater reduction is needed (approximately 60 percent), to permit stabilization of the shoreline 30 m seaward of its present location. Because the region has only a slight longshore sediment drift a uniform shoreline advance of 30 m appears feasible. The structure is placed 300 m offshore in 5 m of water, well outside the normal winter breaker line. If the structure is designed to transmit only 40 percent of the wave energy during winter storms by overtopping, even less will be transmitted during summer conditions and tombolos may begin to form. Therefore, overtopping is not a viable option for controlling wave transmission. A highly segmented design with segments roughly 30 m long and gaps 20 m wide, requiring at least 44 segments to cover the project, could create a more uniform shoreline advance. Each segment should be impermeable and overtopped infrequently. A combination of increased structure permeability and overtopping to increase wave transmission is also a feasible option, but field tests of the structure's cross section and data on the wave climatology are needed. A sufficient volume of beachfill is required to advance the shoreline 30 m; plus an overfill allowance is required to replace initial losses. Terminal groins may be necessary if alongshore losses are anticipated. A schematic of the project planform and expected shoreline is illustrated in Figure B3.

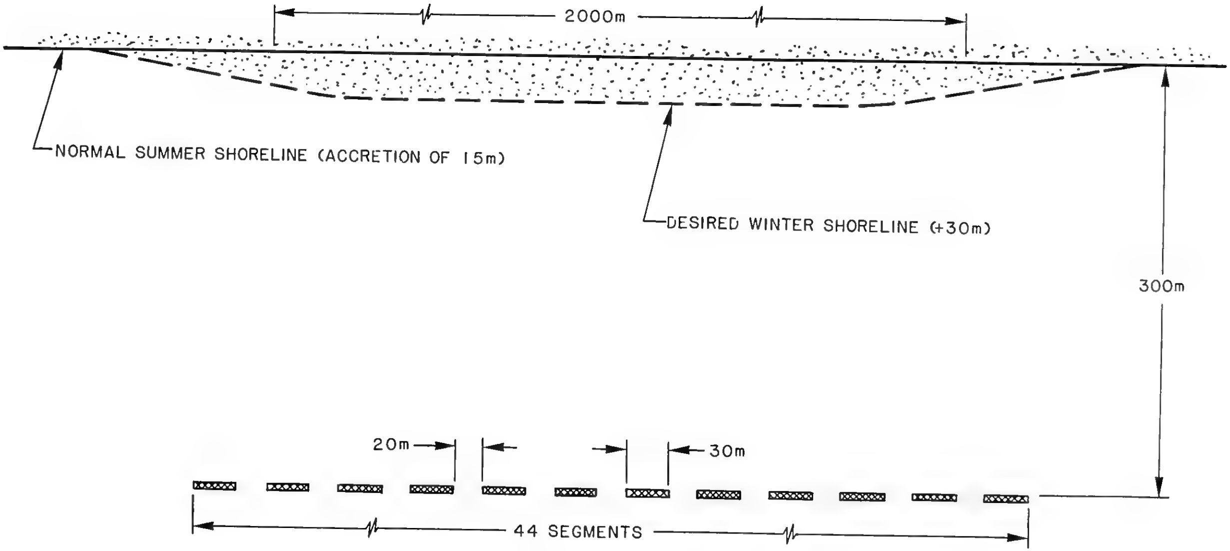


Figure B3. Layout of design for uniform shoreline advance

APPENDIX C: EXAMPLES OF PHYSICAL MODEL STUDIES USED IN DETACHED BREAKWATER DESIGN

1. This Appendix discusses four site-specific model studies performed at WES which involved detached breakwaters and illustrate the types of testing which may be performed to aid in their design.

Presque Isle Model Study

2. A model study was conducted by Seabergh (1983) on Presque Isle Peninsula, Erie, Pennsylvania, which is a recurved sandspit protecting Erie Harbor. The peninsula has a lakeward shoreline of approximately 11 km and serves as a state park with 11 recreational beaches. The landward connection of the spit has been severed several times during storm events, and beach erosion is the status quo as the spit migrates to the east. Previously constructed groin systems and beachfill projects have not halted the erosion. A 1:50-scale (undistorted) physical model was constructed to aid in evaluating the use of segmented detached breakwaters at the site. The model reproduced 2,865 m of shore, including a portion of the existing groin field, and a relatively unstructured section of shore. This permitted study of the interaction of the proposed breakwaters with two beach types. Of particular interest was the positioning of the breakwaters with respect to the existing groins. Figure C1 shows the extent of the shore and offshore which was modeled.

3. A prototype segmented breakwater with three segments was constructed on Presque Isle Peninsula in 1978. Monitoring the shoreline response to this field test provided enough data to verify a sediment movement model. A movable-bed section was constructed in the model test basin using a fine coal sediment (d_{50} = 0.5 mm , specific gravity = 1.35). A shoreline response similar to that observed in the prototype was experimentally duplicated. Figure C2 shows how the model and prototype compared after an accretionary period (Figures C2a,b), and then after the winter season when higher water levels and severe wave conditions cut back the previous tombolo development (Figures C2c,d).

4. Testing of the study area (Figure C1) followed for the existing (or base) conditions and included (a) measurement of the wave-generated current and water circulation patterns, (b) tracer tests (in which the coal sediment

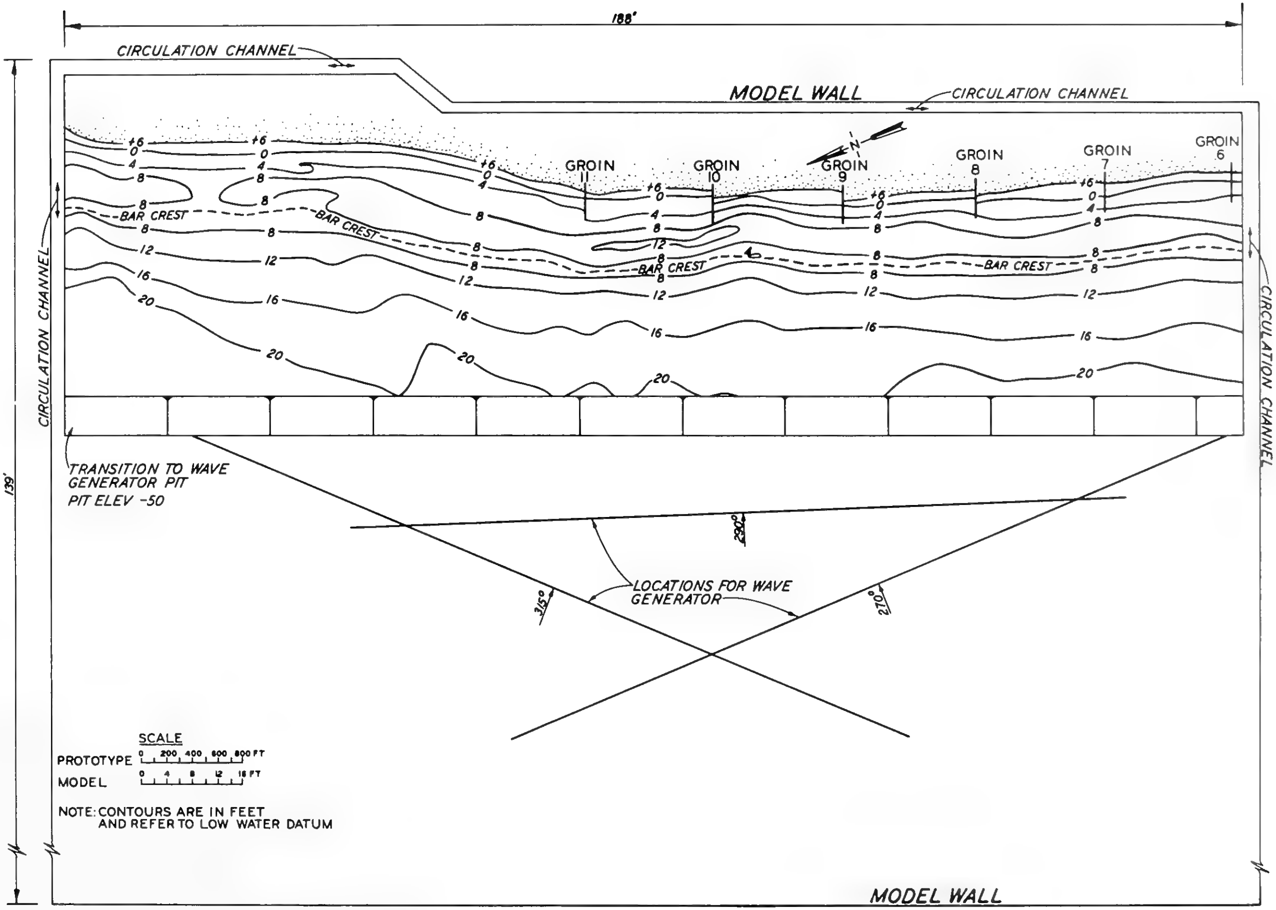


Figure C1. Presque Isle model layout. (To convert feet to metres, multiply by 0.3048)

a. MODEL (ACCRETION OF SHORELINE)

b. PROTOTYPE (16 NOVEMBER 1979)

c. MODEL (EROSION OF SHORELINE)

d. PROTOTYPE (17 APRIL 1980)

Figure C2. Model-prototype comparison of shoreline response to segmented detached breakwater; Presque Isle is placed in the model (Figure C3), and (c) beachfill tests (where a beachfill is simulated). After the base tests were completed for a large number of water levels, wave angles, and wave heights, the number of test conditions was reduced by grouping tests which produced similar results.

a. Pretest

b. Posttest

Figure C3. Sediment tracer placement in model; Presque Isle

5. Figure C4 is the model with an example plan installed. Figure C5 summarizes velocities for a specific test condition for the base condition and three breakwater plans. Figure C6 illustrates the shoreline response and the currents for a beachfill test of one of the plans. Testing indicated that a 107-m spacing between 46-m-long segments produced satisfactory results within the reach covering the groin field. It was best to place the breakwaters so

they were offshore of the ends of each groin. With the groin field removed, the segments could be placed closer to shore with reduced generation of offshore currents. Further details of the results can be found in Seabergh (1983).

Figure C4. Detached breakwater plan in Presque Isle model

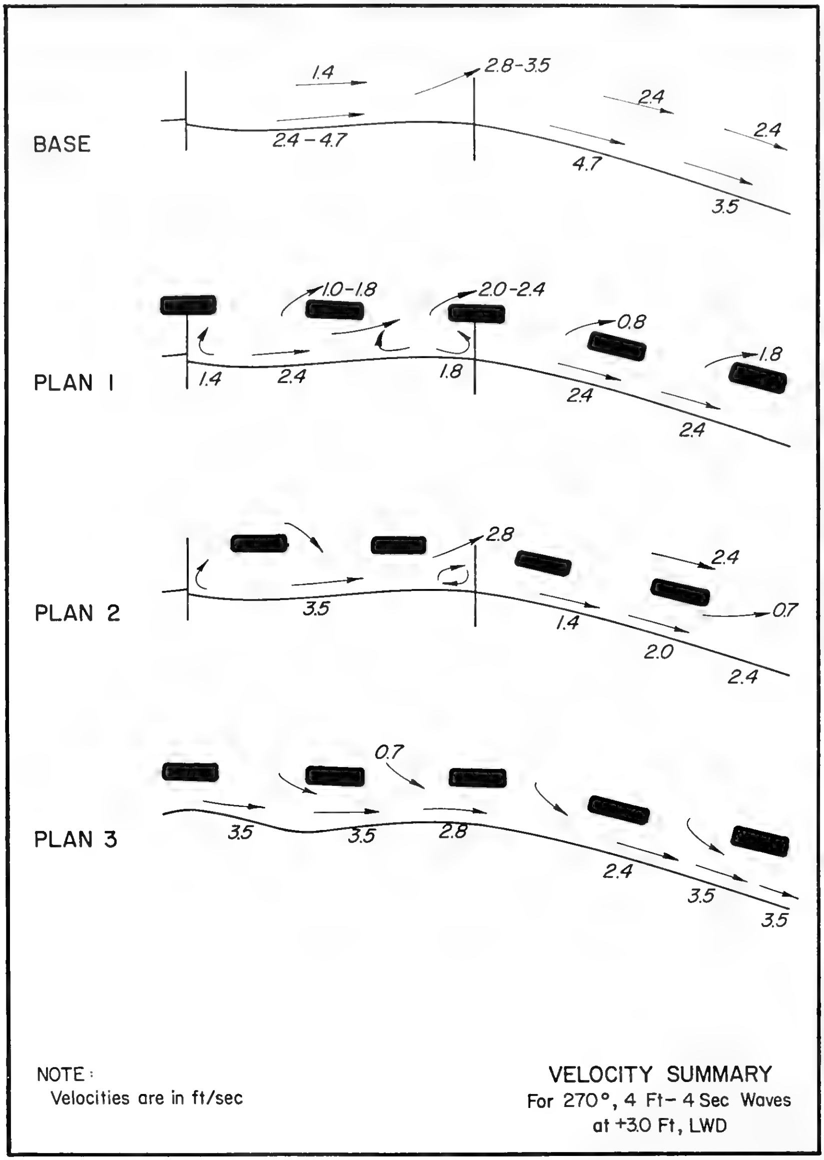


Figure C5. Velocity summary for Presque Isle model test
(To convert feet (or feet per second) to metres (or metres per second), multiply by 0.3048.)

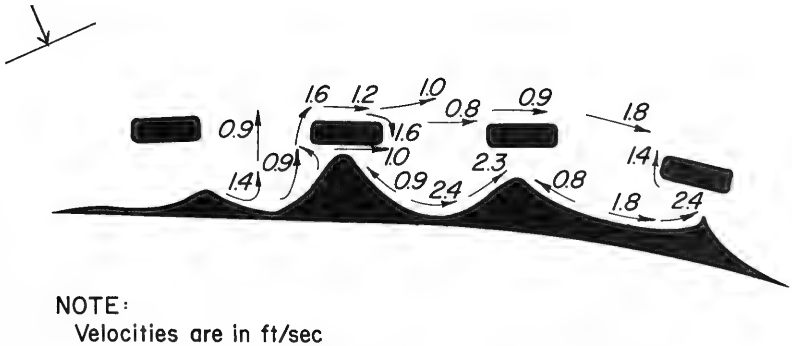


Figure C6. Currents generated by 4 ft-4 sec, 270-deg wave at +3.0-ft water level during beachfill test on Plan 3; Presque Isle

Lakeview Park Model Study

6. Bottin (1982) conducted a study on Lakeview Park, a recreational facility located in and owned by the city of Lorain, Ohio, along the southern shore of Lake Erie. The project consisted of a segmented detached breakwater with three 76-m-long rubble-mound segments; a 59-m-long rubble-mound extension of the east groin, an increased crest height for the landward 15 m of the preproject west groin; and initial placement of 84,106 cu m of beachfill. The detached breakwater and groin modifications were designed and installed to protect the beachfill and the preproject shore. After construction a tendency for localized erosion of the beachfill on the eastern side of the west groin was observed. The fill was replenished but again eroded in the same manner to form a stable but narrower than desired beach. An aerial photograph showing the typical condition of the beachfill east of the west groin is shown in Figure C7.

7. Movable-bed hydraulic model tests were conducted to qualify the degree of erosion at Lakeview Park for various improvement plans. Because of limited funds testing of the proposed modifications for Lakeview Park was conducted using a portion of the 1:50-scale model of Presque Isle, Pennsylvania. The Lakeview Park structures and immediate underwater contours were installed on a section of the Presque Isle model. A portion of the fixed-bed model was

Figure C7. Aerial view of Lakeview Park showing the typical condition of the beachfill east of the west groin replaced with crushed coal to create a movable bed which represented the Lakeview Park contours. Still-water levels (SWL's) were adjusted so that depths in the model were comparable to those in the area of Lakeview Park.

8. Model tests were initially conducted for the as-constructed project at Lakeview Park. After examining many combinations of wave height, period, direction, and SWL, test conditions were selected which produced a stabilized shore similar to that observed in the prototype (as evidenced by a series of aerial photographs). The shoreline configuration obtained in the model for the as-constructed plan is shown in Figure C8.

9. Test data were secured for rubble-mound extensions of the west groin and west breakwater, and various combinations of these modifications. The recommended plan, with respect to beach protection and economics, consisted of a 30.5-m-long extension of the west groin toward the western head of the west breakwater segment (Figure C9). This resulted in a smaller opening between the groin and breakwater; therefore, less wave energy penetrated the opening and only minor retreat of the west-end shoreline occurred. The test condition with the west groin totally removed resulted in a wider beach at the project's west end than that observed with the west groin in place. This illustrates the significance of the groin in causing the local erosion. This model test was a qualitative study and quantitative interpretations should not be made. However, relative comparisons for the various test plans using the same test conditions should be valid.

Oceanside Beach Model Study

10. Curren and Chatham (1980) conducted a model study on Oceanside Beach which is primarily a recreational beach located on the Pacific Ocean approximately 129 km southeast of Los Angeles and 48 km northwest of San Diego, California. Since construction of Del Mar Boat Basin in 1943, persistent erosion of Oceanside Beach has occurred, accompanied by accretion of sand in the harbor and entrance channel.

11. A 1:100-scale hydraulic model was constructed (representing 37.8 sq km in the prototype) and used to investigate the arrangement and design of proposed structures for the prevention of shoaling at Oceanside Harbor and the prevention of beach erosion at Oceanside Beach. Various groin

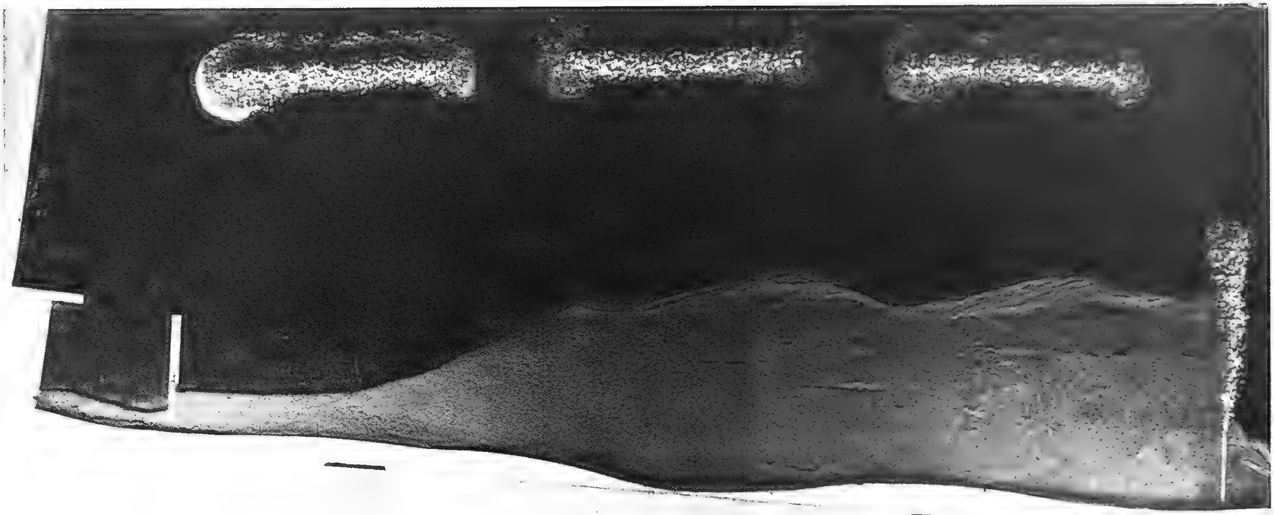


Figure C8. Shoreline erosion occurring east of the west groin for the as-constructed base test plan in the model of Lakeview Park

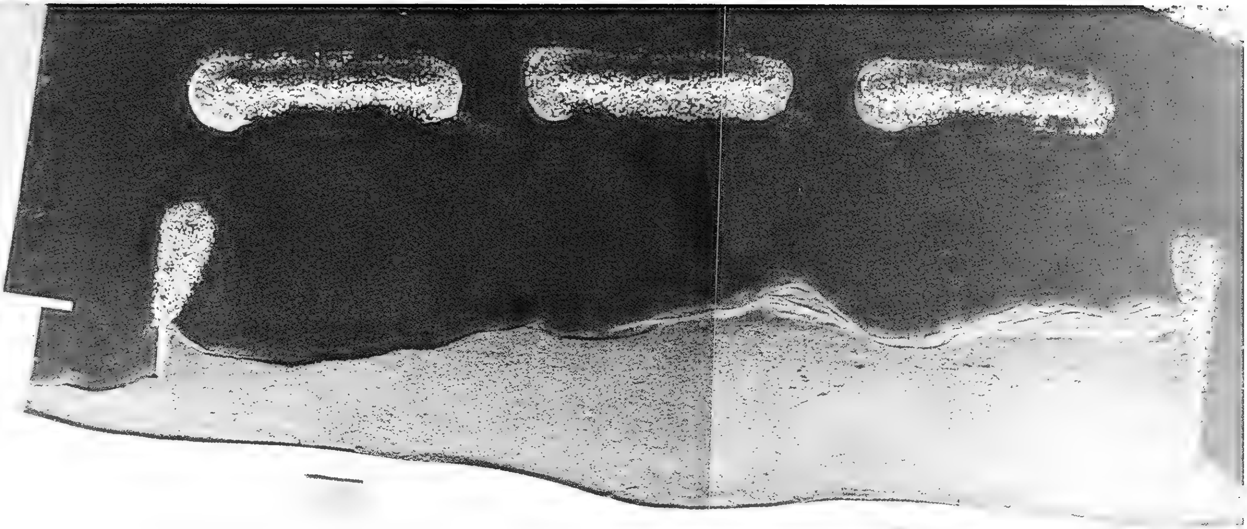


Figure C9. The recommended plan (30.5-m extension of the west groin toward the head of the west segment) at Lakeview Park and detached breakwater configurations were proposed. Only those tests involving detached breakwaters will be discussed here.

12. Model tests were conducted using a crushed-coal tracer material for existing conditions and the various improvement plans under different test wave characteristics. The following three types of tracer tests were used:

a. Fixed-bed tracer. Tracer material was placed on the fixed-bed model surface at selected locations and/or fed into the longshore current to determine the mechanisms of littoral movement in the study area.

b. Semimovable-bed. Tracer material was placed in a layer representing beachfill on the model surface to determine areas of accretion and erosion. The extent of erosion was limited by the fixed model surface.

c. Movable-bed section. The fixed-bed contours in a certain area were removed to a point well beyond the breaker zone and remolded entirely with crushed coal tracer. This type of test is the most reliable for determining areas of accretion and erosion and was used for each major beach protection plan.

13. Tracer tests for the existing condition produced an onshore movement of coal tracer for small waves of low steepness with longshore transport at the shoreline. For high-steepness waves, the coal tracer moved seaward forming a bar at the seawardmost breaker zone. This material migrated north or south depending on wave direction. The high-steepness waves reformed and broke a second time near the shoreline, resulting in a second nearshore zone of longshore transport. Detached breakwater plans tested included a single structure (1,494 m long) with varying crest elevations over 213-m sections and a segmented breakwater consisting of four 203-m-long segments with 203-m gaps. Each plan was tested both with and without groins at the northern and southern extremes of the project. Movable-bed model tests indicated that the test plans without groins would, generally, result in erosion of the shore on the updrift side of the test section and loss of material from the downdrift side; thus, these did not provide adequate protection to the beachfill. The installation of groins reduced the amount of coal leaving the study area, resulting in a fairly stable shore. Views of the model with a single structure and a segmented breakwater in place are shown in Figures C10 and C11.

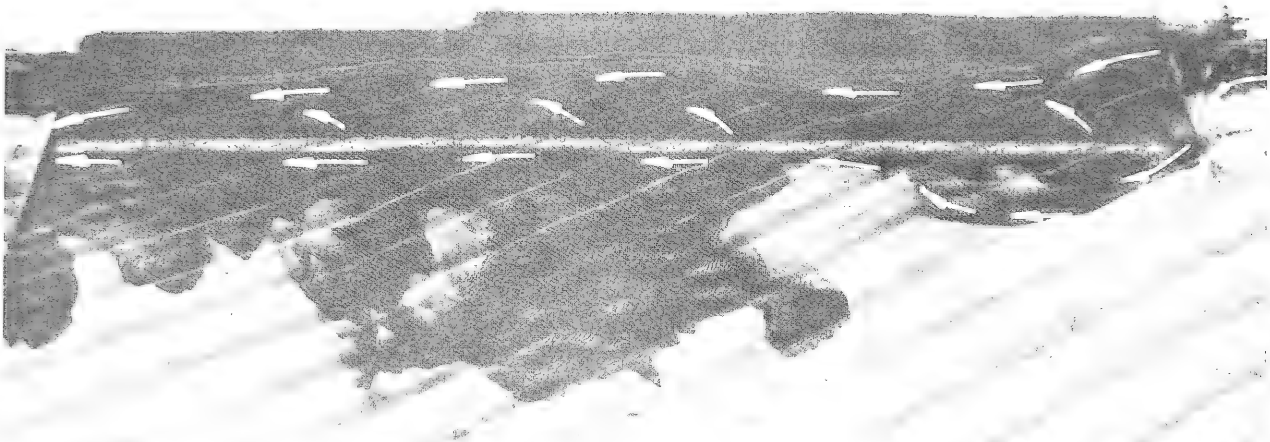


Figure C10. Single detached breakwater without groins in the Oceanside Beach model

Figure C11. Segmented breakwater with groins in the Oceanside Beach model

Imperial Beach Model Study

14. Curren and Chatham (1977) conducted a model study on Imperial Beach which is located on the Pacific Ocean 5.6 km north of the Mexican border and 17.7 km south of San Diego, California. It is primarily a recreational beach, with a 366-m-long fishing pier situated in the approximate center of the study area. Two groins, 226 m and 122 m long, are located 899 and 495 m north of the fishing pier, respectively. The Tijuana River is believed to be the main historical source of sediment for Imperial Beach. However, construction of the Morena and Barret Dams in Cottonwood Creek and the Rodriquez Dam in the Rio de las Palmas causes river sediments to be trapped behind the dams without ever reaching the coast. In addition, the lack of recent floods has caused a shortage of sediment reaching the mouth of the Tijuana River. Therefore, there is a decreased quantity of sediment available for longshore transport to Imperial Beach and beach erosion has increased. Two groins in the area, constructed between 1959 and 1963, have been ineffective in stabilizing the beach.

15. A 1:75-scale hydraulic model was constructed (representing about 13.5 sq km in the prototype) and used to investigate the arrangement and design of alternative proposed structures for preventing erosion of the Imperial Beach shore.

16. Model tests were conducted using a crushed-coal tracer material for existing conditions and the improvement plans under various wave conditions. The structures proposed for Imperial Beach consisted of (a) single detached breakwaters at the -4.6 and -3.0 m contours, (b) segmented breakwaters at the -4.6 and -1.5 m contours, (c) a single detached breakwater segmented by low sill sections at the -3.0 and -1.5 m contours, and (d) various groin plans.

17. Model test results for existing conditions indicated that both north- and south-directed longshore currents would be interrupted at regular intervals by strong rip currents. These rips transported significant quantities of sediment offshore where it was either (a) transported longshore on the bar, (b) lost in deep water, or (c) transported back shoreward by low-steepness waves. Rip currents occurring in the model were similar to those observed in the prototype. The five-groin plan tested in the model created strong rip currents for almost all incident wave conditions. The five groins were not only ineffective in trapping tracer material but contributed to

offshore movement. A series of nine groins was effective in trapping tracer material, but significant quantities of stone were required. Testing segmented breakwater plans at the -4.6 m depth indicated that shorter segments with shorter gaps produced weaker rip currents and appeared to retain most of the tracer material. However, a large volume of stone was required. Test results with submerged structures at the -3.0 m depth revealed that breaking waves piled water between the breakwater and shore. The seaward return of the water created strong rip currents, resulting in an offshore loss of tracer material. Low sills were placed in the breakwater gaps and were successful in retaining all but small quantities of tracer. Tests with segments located at the -1.5 m contour separated by gaps allowed too much wave transmission into the structure lee. The installation of low sills between breakwater segments appeared to be a viable way of reducing the total wave transmission and would have the least impact on longshore transport seaward of the structure. Figures C12 and C13 show views of the detached breakwaters at the -4.6 m and -1.5 m contours in the Imperial Beach model.

Figure C12. Typical wave and current patterns and current magnitudes for segmented detached breakwaters at the -4.6 m contour in the Imperial Beach model (velocities are in ft/sec) (To convert feet per second to metres per second, multiply by 0.3048)

Figure C13. Single detached breakwater with low sills at the −1.5 m contour in the Imperial Beach model