A DETACHED BREAKWATER SYSTEM FOR BEACH PROTECTION

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SUMMARY

Three segmented, detached breakwaters were constructed in the fall of 1977 at Lakeview Park, Ohio, on Lake Erie, to protect a beachfill to be used for recreation and shore protection. This paper documents the design procedures which established the configuration of the breakwaters and the beachfill, and determined the need for a terminal groin. The beachfill has been monitored by aerial photography and bathymetric profiling. During the second year, a storm of near design intensity generated severe waves concurrently with high Lake levels and eroded the updrift beach; however, the initial beach configuration was partially restored by natural processes during the following summer season. The project has functioned well, with very little loss of sand from the system and without adverse impacts on the downdrift coast.

INTRODUCTION

Lakeview Park is a lakeside recreational area in Lorain, Ohio, on the southern shore of Lake Erie (figure 1). The Park provides 1,500 feet of beach serving the recreational needs of north central Ohio. Prior to implementation of this project, the beach had eroded due to high Lake levels and an inadequate supply of littoral drift.

Studies were conducted in 1974 to plan and design a beach-restoration project within the Park boundaries to satisfy the recreational needs of the tributary area. The study was undertaken at the request of the City of Lorain and was authorized by Congress. Alternative designs included groin fields and artificial headlands. An existing groin field at the project site had not worked well, although properly designed groins and heavy nourishment may have retained a beach. The potential for offshore loss of littoral drift with a groin field would result in innumerably greater annual costs for groin-field shore protection than for protection with offshore breakwaters. Artificial headlands (see Silvester, 1972) were a relatively new concept during the planning process. Although the headland plan would probably have functioned properly, it would have affected reaches of shoreline both up- and downdrift of the project. These up- and downdrift shorelines are privately owned. The Federal

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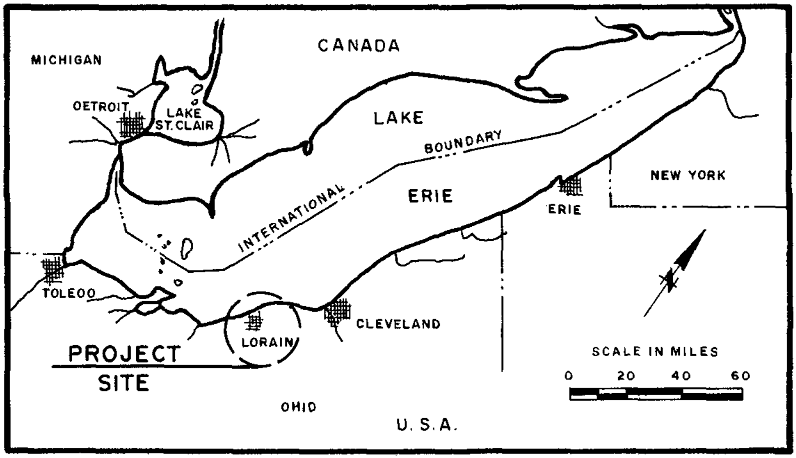

FIGURE 1: VICINITY MAP

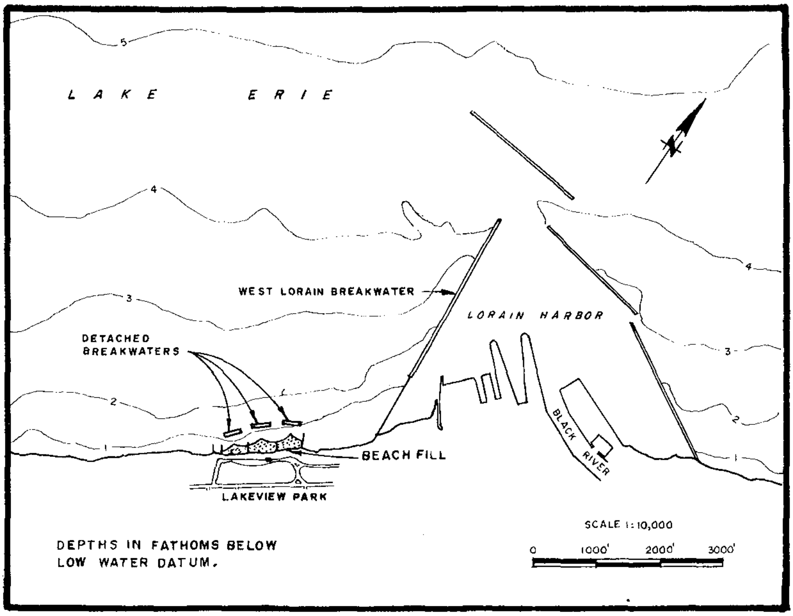

FIGURE 2: AREA MAP

Government does not cost-share in developing recreational beaches for protecting private property.

The design procedures presented herein are therefore state-of-the-art as of 1974 for construction of a small project. The design was done in less than a year without the use of mathematical or hydraulic models, which were not available during the design period. The project construction was completed in October 1977 and a post-construction monitoring program was initiated.

DESCRIPTION OF PROJECT SITE

Location

The project site is located in Lakeview Park in the city of Lorain, Ohio, which is 25 miles west of Cleveland. Lakeview Park is a lakeside recreational facility situated ½ mile west of Lorain Harbor as shown in figure 2. Lorain Harbor has several breakwaters that extend 5000 feet lakeward from shore. The shoreline in the vicinity of the project site comprises both heavily protected and eroding glacial till and shale bluffs on reaches to the west. The preproject site had six existing groins of various construction, ranging in length from 70 feet to 150 feet. The groins which did not represent a consistent design, were not long enough to trap the meager littoral transport in sufficient quantity to provide an adequate recreational beach, nor were they high enough to prevent sand from moving over them during extremely high Lake levels.

Geology

Lake bottom surveys indicated the bottom 300 feet offshore is 8 feet below low water datum (LWD) and comprises a 12- to 18-inch stratum of medium to coarse sand with gravel and traces of laminae of silty clay. Underlying this stratum is 2 to 12 feet of stiff-to-hard, gray, silty clay. Shale bedrock underlies these strata.

Lake Levels

Water levels on Lake Erie vary from year to year and from month to month, and fluctuate on an hourly basis. The yearly variations are due to long-term climatic cycles. The seasonal variations consist of high levels in the spring and early summer and low levels in the winter. Short-term hourly fluctuations are caused by local meterological disturbances. Astronomical tides are negligible. Low water datum, LWD=0, is 568.6 feet above the International Great Lakes Datum, 1955, (IGLD) based on mean water level at Fathers Point, Quebec. The highest recorded monthly mean Lake level was 573.5 feet and the lowest was 567.5 feet. The monthly mean was 570.4 feet from 1960 to 1970. The greatest annual fluctuation of monthly mean lake level was 2.75 feet. A 1.5-foot storm surge has a recurrence interval of 1 year.

The design water level with a statistical recurrence period of 20 years is 5.5 feet above LWD and comprises a 2.0-foot storm surge and 3.5-foot LWD monthly mean lake level.

Waves

The project site is directly exposed to fetches from the west clockwise through the north. The fetch length to the west is 36 miles and that to the north is 55 miles. The fetches in the Lake for directions east of north are 150 miles; however, the Lorain Harbor breakwaters shelter the project site from direct northeasterly wave attack. The wave climate used for the design of structural features was done by hindcasts that used a 24-year wind record taken at Cleveland. Calculation of littoral transport potential was derived from refraction, shoaling, and diffraction tranformations of a 3-year hindcast of waves on Lake Erie by Saville (1953). The published data were for a station 28 miles east of Lorain; therefore, appropriate corrections were made to reflect the differences in fetches between the two sites. The hindcast data were corrected for effective fetches and a 44-foot Lake depth over the generating area. The wave heights approaching the project site range up to 8 feet annually, and periods range up to 7 seconds. The wave climate for littoral transport calculation assumed waves arrived at the shore only during the typical ice-free period from April through November. The 20-year recurrance design wave was from the northwest with a height of 10 feet and a period of 8 seconds.

Littoral Transport Analysis

Analysis of the potential for littoral transport is a key element in designing beach-stabilizing structures. The analysis was conducted by reviewing historical aerial photographs, site observations, and calculations of the potential littoral transport rate by wave-energy flux methods.

The existing preproject beach at Lakeview Park comprises a medium sand with traces of gravel, some of which is debris derived from remnants of decomposing updrift to the west shore-protection structures. The beach slope is 1 on 12 above water and 1 on 18 below water. Immediately west of the project site, the shoreline comprises eroding shale bluffs. Glacial till and decomposing bulkheads on the bluffs are a source of littoral drift. The west Lorain Harbor breakwater, 1,500 feet east of the project site, has been accumulating a fillet of sand on its west side at a rate of between 5,000 and 8,000 cubic yards per year. The existing groins in the project site had trapped some sediment out of the littoral system, but material apparently was bypassing them to deposit in the fillet at the west Lorain Harbor breakwater.

The general trend of the shoreline is an orientation of a 58-degree relative to north. The analysis of aerial photographs taken in 1948, 1956, 1968, and 1974 indicates that the preproject groin field trapped fillets aligned on 50- to 55-degree azimuths. This indicates a west- to-east littoral transport.

The potential littoral transport rate was estimated by determining the longshore component of wave-energy flux. This was accomplished by transforming the incident waves to their breaking properties of height and direction of propagation. The energy flux is given by:

$$P = \frac{\rho g}{8} H_b^2 \, C \cos \alpha_b$$

where ρ is the density of water;

g is the acceleration due to gravity;

H_b is the breaking wave height;

α_b is the angle of the wave crest relative to depth contours at breaking; and

C is the wave celerity.

The incident deep water wave climate was transformed to the project site by diffraction about the Lorain Harbor breakwaters and by refraction and shoaling analyses. The sheltering effect of the Lorain Harbor breakwaters decreases with distance to the west. This is illustrated in figure 3, which plots the total and longshore components of energy flux as a function of distance from the west Lorain Harbor breakwater. The plot indicates a net eastward littoral transport at the project site; however, the net transport reverses to the west 1,500 feet west of the site. Furthermore, the total energy flux increases with an increase in distance from east to west. This clearly illustrates the influence of the Lorain Harbor breakwaters. The potential littoral transport rate is estimated by multiplying the longshore energy flux by an empirical coefficent described in the Shore Protection Manual (1974), where the potential longshore transport rate (Qs) is

$$Q_S = 7.5 \times 10^3 \, P_{ls}$$

where Q_S is in cubic yards per year and

$P_{ls} = P \sin \alpha_b$ (where P_{ls} is the longshore component of energy flux in foot-pounds per second per foot of beach.)

The potential longshore rate is 21,500 cubic yards per year to the east project site. This potential is not realized due to an inadequate supply of littoral drift. The 5,000 to 8,000 cubic yards per year that is accreting in the west Lorain Harbor-breakwater fillet is a more reliable indication of the actual transport rate. Figure 4 shows the distribution of total energy flux and the longshore component of energy flux at the project site. The distribution of wave-energy flux indicates a well-balanced potential for littoral

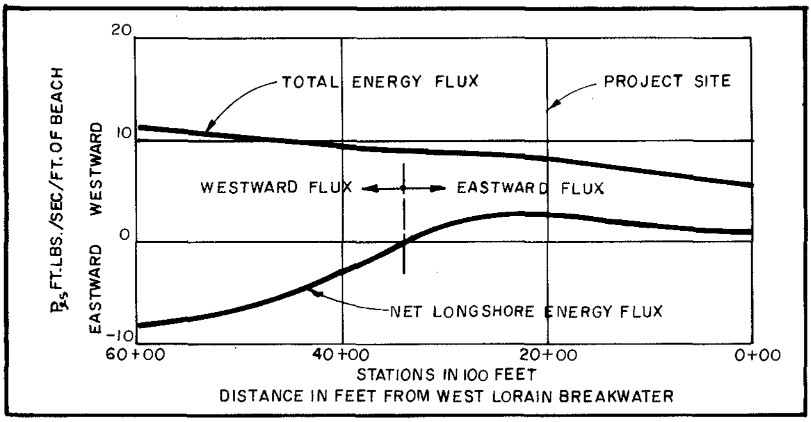

FIGURE 3: WAVE ENERGY FLUX ALONG COASTLINE

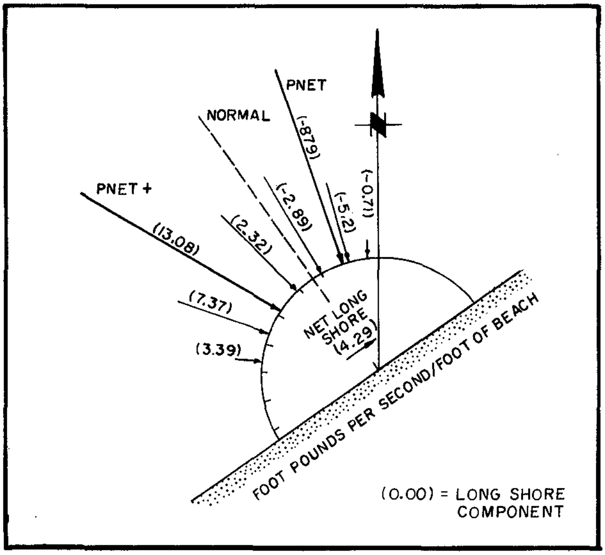

FIGURE 4: ENERGY–FLUX $P_{\ell s}$ AT THE PROJECT SITE

transport; i.e., the tendency for reversals in littoral transport due to waves from the east is nearly two-thirds of the potential transport for waves from the west. The sheltering effect of the Lorain breakwater therefore is partially offset by the greater fetches to the east.

PROJECT DESIGN

Alternative Studies

The primary purpose of the project is to provide a recreational beach and protect the Park from erosion. Beach-protection structures should not increase the erosion of up- and downdrift properties. This is a Federally sponsored project, which can protect only publicly owned lands. This ruled out the use of a massive beachfill or a large groin field extending to the Lorain breakwater. Artificial headlands would also affect a larger segment of shoreline than just the Park. Several groin fields of various lengths and spacing were considered; however, detached breakwaters were selected because they would decrease offshore sand movement. Each alternative considered would require imported sand to nourish and maintain the beach.

Plan Description

The detached-breakwater configuration is shown in figure 5. Three detached rubble-mound breakwaters, each 250 feet long, are arranged along a flat-arc alinement convex lakeward in water with a depth of -8 feet LWD. The breakwaters are spaced 160 feet apart. The west end of the west breakwater is 450 feet offshore and the east end of the east breakwater is 500 feet offshore. The breakwaters protect 1,500 feet of shoreline. An existing groin on the east side of the project site was increased in elevation and extended from 150 to 350 feet offshore. The crest of one of the groins on the west side of the park was also raised 2 feet to maintain the updrift beach profile during periods of high Lake levels. A medium-sand beachfill with an average of 200 feet, was placed behind the breakwaters. The initial beachfill of 110,000 cubic yards was placed on a 1 on 5 slope, with a berm elevation of +8 feet LWD. The project was completed in October 1977 at a cost of about $1,700,000. The Federal Government paid for 70 percent of the construction costs and the local government paid for 30 percent. The planning document also allowed for periodic nourishment, at a rate of 5,000 cubic yards per year, to be placed on the beach as nourishment, or on the downdrift beach if the project induces downdrift erosion.

Design Analysis

Several configurations of detached breakwaters were investigated. The primary function of the breakwater is to regulate incident waves and to reduce offshore littoral transport. For initial design, the breakwaters were assumed to be situated in constant water depth, to be impermeable, and to not be overtopped. The shoreline was assumed

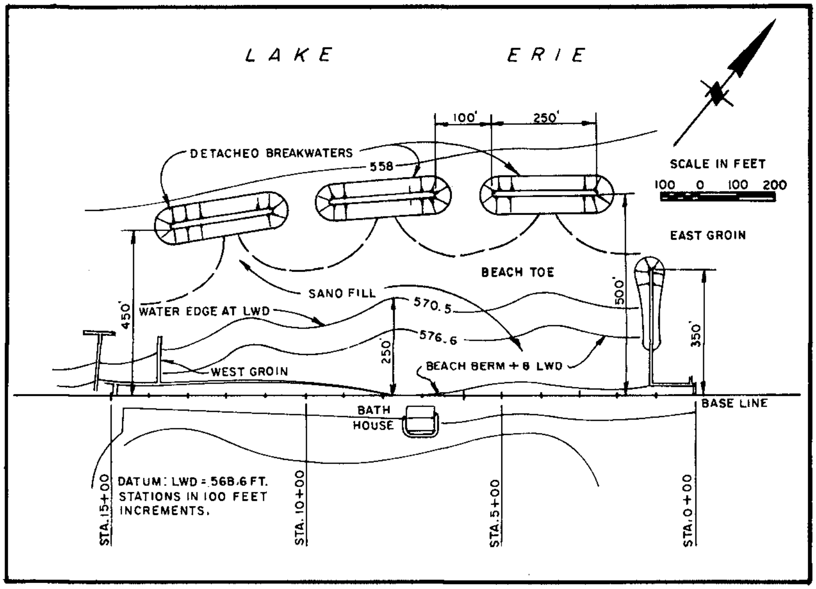

FIGURE 5: PLAN OF IMPROVEMENTS

to adjust to equilibrium in response to the predominant waves. The shoreline planform was assumed to be in equilibrium when the shoreline is coincident with the transformed wave crests; otherwise, waves arriving at an angle to the shore would induce longshore transport. The predominant wave was selected as being the wave from the direction associated with the net direction of energy flux. Figure 6 shows one of the simple diffraction analyses used in the design analysis. The heavy lines of variable thickness are wave crests that are weighted according to the diffraction coefficient. The resultant shoreline is shown to generally parallel the incident wave crests; however, the salients are more accentuated to account for refraction effects. A salient is located near the center of the geometric shadow of each breakwater. The apex of the salient shifts in response to incident waves. The updrift shoreline (to the west) must have continuity with the existing shoreline. The downdrift shoreline, similarly, must have continuity with the project shoreline. This was not feasible; therefore, the existing east groin was extended in length from 150 feet to 350 feet. The toe of the beach at the east groin had to intersect the existing Lake bottom at the groin head as shown in figure 7 to stabilize the project beach with a desired 200-foot width.

The salient in the geometric shadow of each breakwater is formed by littoral drift being transported into the sheltered region by diffracted wave crests. If the breakwaters are too close to shore, or are too long, the shadow region becomes too effective a barrier, and insufficient wave energy is left for littoral transport. The salient continues to grow and eventually becomes a tombolo. Once a tombolo is formed, natural nourishment of the downdrift beaches is denied, increasing the tendency of these beaches to erode. Because this is naturally an eroding reach of shoreline with a limited supply of littoral drift, a considerable period of downdrift starvation would be experienced before tombolo formation and before natural bypassing would continue. Therefore, formation of a tombolo was not considered desirable.

A tombolo can be prevented from forming by designing the breakwater lengths and distances offshore such that variation in directions of incident waves will allow sufficient energy to enter the shadow to transport littoral drift. The variations in wave attack were incorporated into the design by considering the net westerly and net easterly directions of energy flux, as shown in figure 4. The net easterly and westerly components each represent a weighted, long-term average potential to transport littoral drift. The predominant waves move sand toward the salients. If the variations in wave energy from various directions are adequate to move appreciable material past the salients, no tombolos should form.

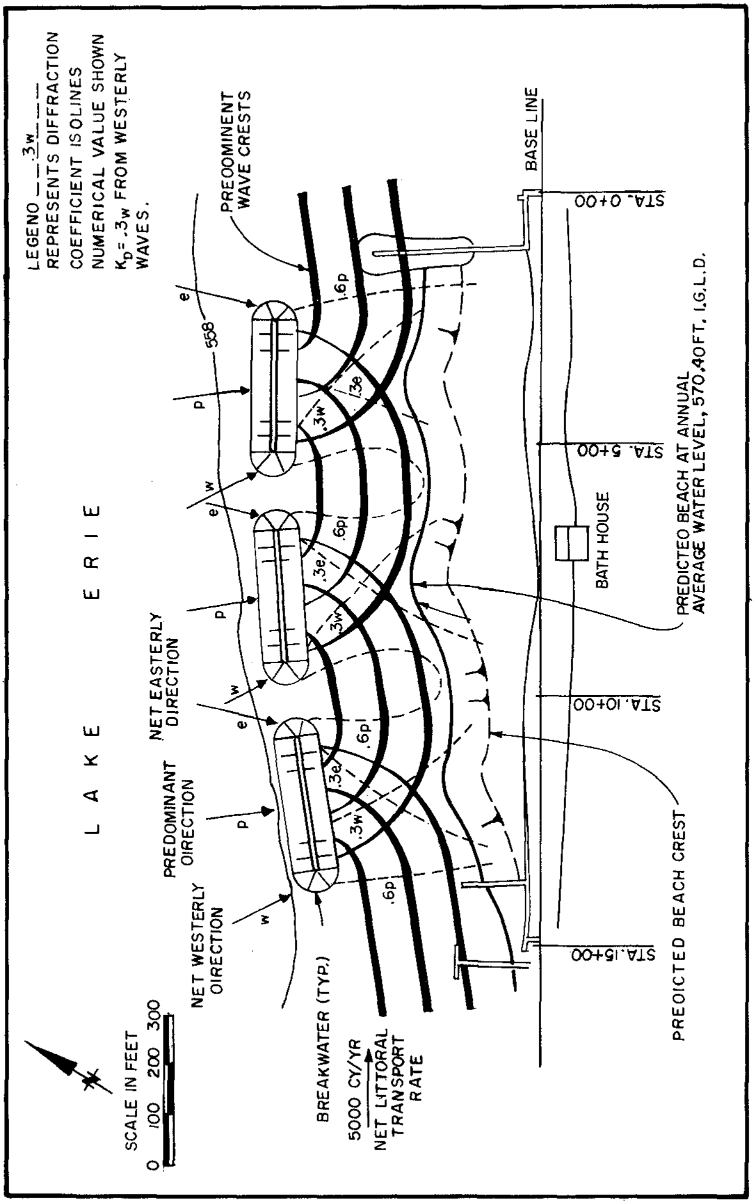

FIGURE 6: DIFFRACTION ANALYSIS

The amount of wave energy required to prevent tombolo formation is not accurately known; however, observations of several detached breakwaters in southern California by James Dunham[1/], led to the hypothesis that if the shoreline were landward of the intersection of the diffraction coefficient (K_D) isolines where $K_D = 0.3$ for the easterly and westerly wave components derived from the energy-flux analysis, a tombolo would not form. A case-in-point is the Venice, California, detached breakwater. This breakwater, built about 1902, is 600 feet long and was then 1,200 feet offshore. A small salient formed in the lee of the breakwater; however, the salient never advanced to more than 200 feet from the original shoreline. A 14 million-cubic yard sandfill, placed along 5 miles of the local beach in 1947 and 1948, advanced the shoreline about 500 feet, and a tombolo soon formed in the lee of the breakwater. Dunham discovered that the intersection of $K_D = 0.3$ isolines of waves diffracting about the heads of the breakwater was just shoreward of the advanced shoreline when the fill was first placed. This finding roughly checked with beach buil-outs behind other breakwaters in southern California, and the principle was used successfully in siting two new breakwaters in the next few years so as to prevent unwanted tombolo formations. As a result of this experience, the same principle was adopted for design in Lake Erie. Apparently, the rule works because the wave energy is nearly an order of magnitude less than the incident wave energy where $K_D = 0.3$. Storm waves generally have an order of magnitude more energy than the daily predominant waves. Therefore, during storm conditions, there would be as much wave energy reaching the apex of the salient as the prevailing waves bring to unshielded segments of the beach. This energy is adequate to prevent tombolo formation.

The intersections of the $K_D = 0.3$ isolines for waves from directions representative of the net easterly and westerly component of the energy-flux waves vectors are shown in figure 6. The intersections are well offshore, thus ensuring that, even at low Lake levels, a tombolo should not form. The $K_D = 0.6$ isolines from the predominant wave direction are also shown for reference. The primary function of the breakwaters is to regulate the direction of wave approach. The lengths of the breakwaters, the distance offshore and the spacing between the breakwaters dictate the shape of the beach. The updrift and downdrift shorelines also control the lakeward extent of the shoreline behind the breakwaters.

The resultant beach was designed to be normal to the predominant wave approach. The flat-arc configuration of the breakwaters resulted from the design process of matching adjacent shoreline boundary conditions in the project area. In order to maintain natural transport to downdrift beaches, the net transport behind the breakwater had to

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[1/] Senior Coastal Engineer, Moffatt & Nichol, Engineers, Long Beach, California; formerly with the U.S. Army Corps of Engineers be slowly eastward. With the east end of the project shoreline designed to be 300 feet lakeward of the natural shoreline, blending of adjacent shore segments as was done at the west end was not feasible, therefore, a terminal groin at the east end was designed to maintain the advanced position of the beach in that area. This groin was designed to be impermable to sand, but short enough to allow the littoral drift that enters the system from the updrift shoreline to bypass around its head. This would prevent any acceleration of erosion of the downdrift beaches due to the project structures.

Figure 7 illustrates the design of the east groin. The pre-project concrete-sheet-pile groin was raised 2 feet to prevent sand from overtopping it during high Lake levels, and it was extended lakeward sufficiently to retain the new beach in its design configuration. An important feature of this groin is the impermeable diaphragm along its centerline. This membrane is of steel-sheet-pile construction; however, concrete poured in place between the voids in armor rock could also have been used.

The typical breakwater cross section is shown in figure 8. The crest elevation, at +8 feet LWD is designed to allow minor wave overtopping during storm conditions. A transmitted height of 1 foot was allowed. A lower crest elevation would have resulted in greater wave overtopping, which would induce a higher wave runup on the lee beach and generate a wave setup in the water. The setup would induce rip currents between the gaps and around the ends of the two outboard breakwaters, thereby increasing offshore loss of littoral drift. The rip-current scour might also undermine the breakwater ends. This actually occurred in model studies of an offshore breakwater system designed and tested by the Corps of Engineers in conjunction with erosion-prevention planning for Imperial Beach, California (U.S. Army Corps of Engineers, 1978).

MONITORINC PROCRAM

Description of Program

The Corps of Engineers initiated a 5-year monitoring program immediately after construction to document littoral transport and the wave climate in the vicinity of the project site. The purpose of the monitoring program is to provide information on required operations and maintenance, such as beach-nourishment needs, to assist in developing design criteria for detached breakwater systems, and to provide data that will lead to a better understanding of nearshore processes in the Creat Lakes.

The monitoring program comprises aerial photography, bathymetric and topographic surveys, sand samplings, Littoral Environment Observations (LEO), site inspections, and installation of a wave gage. Color aerial photographs are taken three times a year over a 6-mile reach of shore to document morphological changes. The photographs are at a scale of 1:4800 and have 60 percent overlap. Surveys are

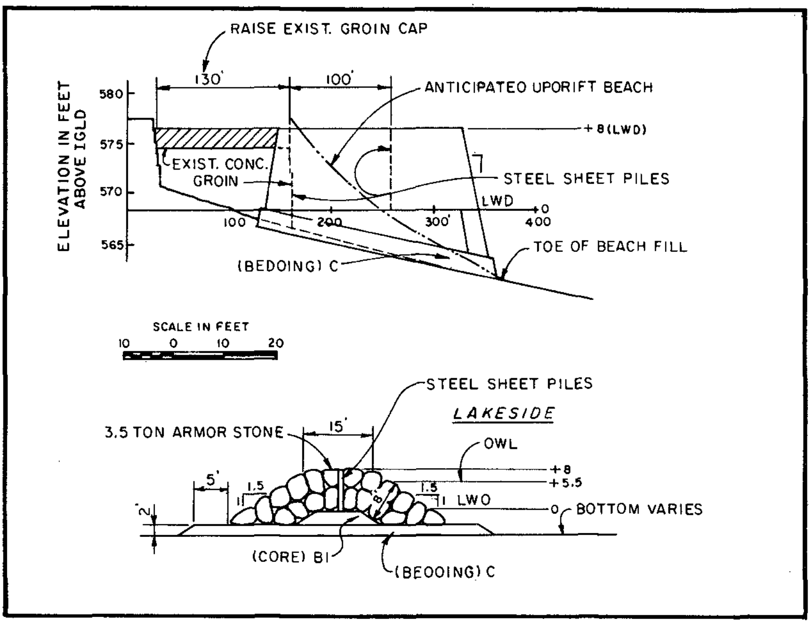

FIGURE 7: EAST GROIN PLAN

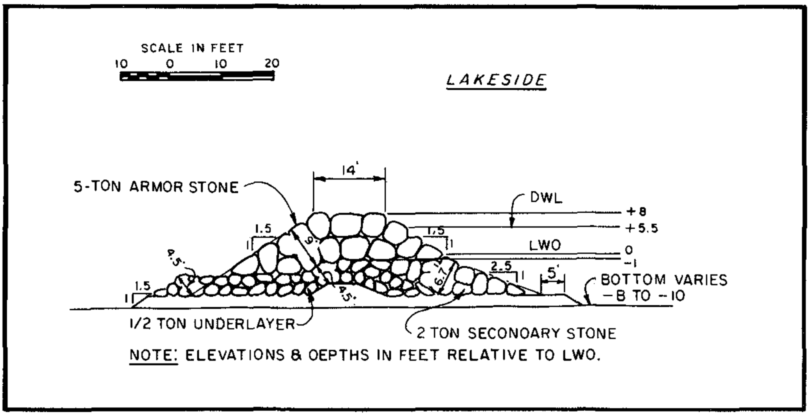

FIGURE 8: DETACHED BREAKWATER PLAN

taken twice a year, once in the spring and once in the fall. The survey area extends 2,000 feet east and 3,000 feet west of the project site, and range lines are at 100-foot intervals within the project site. Each line is surveyed from the backshore out to a depth of -20 feet LWD. Surface sediment grab samples are taken once a year, in the fall.

A local observer documents daily wave and beach conditions under the LEO program. The LEO data have not yet been reduced nor analyzed at the time of this writing. Site inspections are made three times a year to describe beach morphology and conditions. A wave gage will be installed in 1981 and records kept for a period of 1 year.

FINDINGS

The project construction was completed in October 1977. Photograph 1 shows that the planform quickly adjusted to the shape predicted by the design. Salients soon formed in the lee of each of each breakwater, and the beach slopes flattened. Photograph 2 shows the configuration of the beach in May 1980. Figures 9 and 10 show the morphology of two representative beach and offshore profiles. Figure 9 was a section taken in the gap between the east and central breakwater and figure 10 was a section taken behind the west breakwater. The profiles in these figures show that the offshore slope during the first 6 months adjusted to 1 on 20 behind the breakwaters (figure 10), and to 1 on 15 in the gaps (figure 9). The foreshore slopes in both cases are 1 on 12. These slopes remained essentially the same, although the most recent survey (in November 1979) showed that the west end of the beach had eroded to within 20 feet of a seawall on the east side of the west groin at station 13 + 75. (Stations are at 100-foot incremental distances west of the east end of the Park, as shown in figure 5).

The project was subjected to a storm from the west in May 1979. The storm generated waves of near design conditions at near record high Lake levels. The west end of the beach eroded severely; however, normal wave action restored much of the beach later in the year. Since that storm, other storm waves from the west have eroded the west end, but, again, the normal wave activity partially restored the lost sand.

A volumetric analysis was made to determine the changes in quantities of sand within the project boundaries. October 1977 was the base year. The initial beachfill comprised 110,000 cubic yards of medium sand. The volume of sand was measured by topographic and bathymetric surveys along the range lines shown in figure 5. Volumes were computed by the average-end-area method. The downdrift shoreline did not experience noticeable erosion. The total project area behind the breakwaters has accumulated sand. Figure 11 summarizes the volumetric changes behind each of the breakwaters as a function of time. The changes indicate that the project rapidly gained 4,000 to 6,000 cubic yards of sand, but that the volume of sand retained

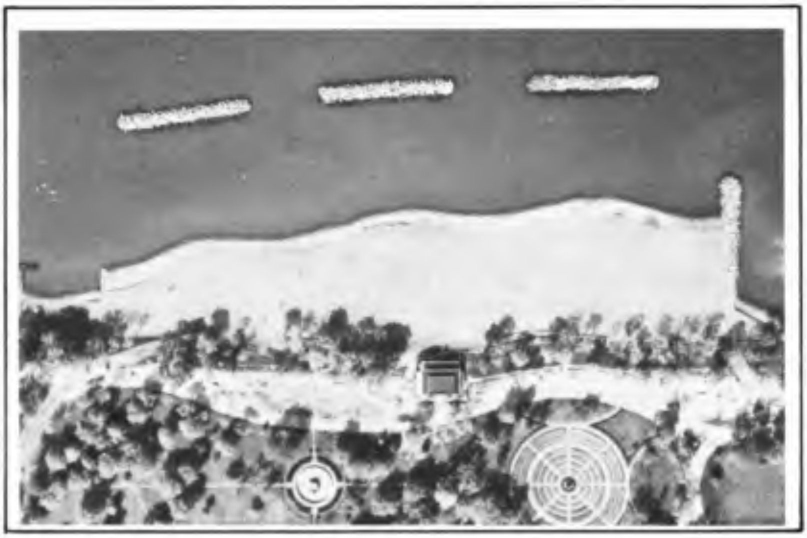

PHOTO 1: IMMEDIATELY AFTER CONSTRUCTION, OCTOBER 1977

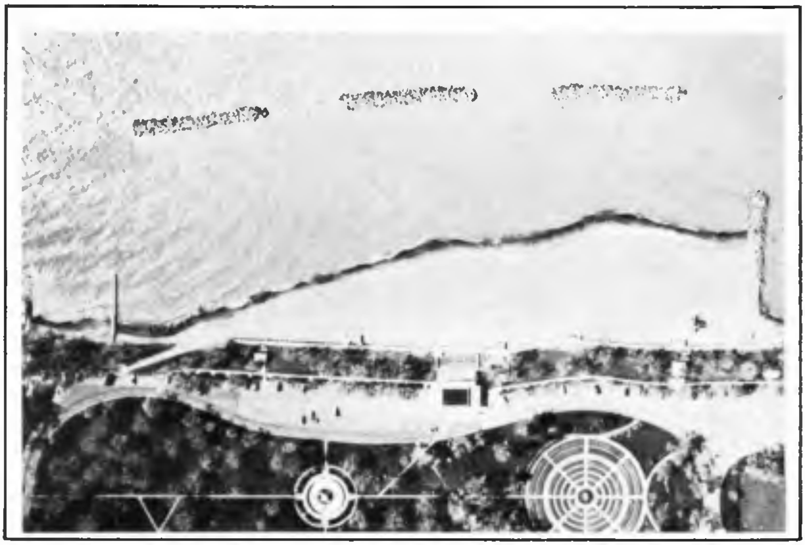

PHOTO 2: TWO AND A HALF YEARS AFTER CONSTRUCTION, MAY 1980

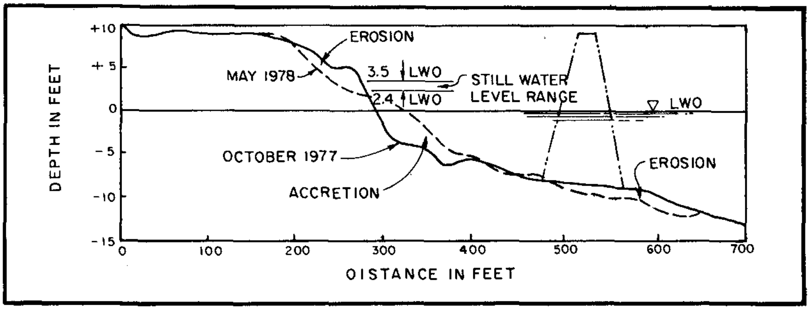

FIGURE 9: COMPARISON OF BEACH PROFILES
STATION 6+00W
(GAP BETWEEN EAST AND CENTRAL BREAKWATERS)

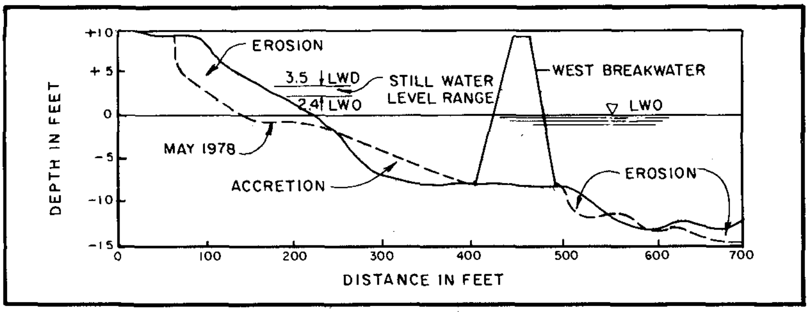

FIGURE 10: COMPARISON OF BEACH PROFILES
STATION 13+00W
(IN LEE OF WEST BREAKWATER)

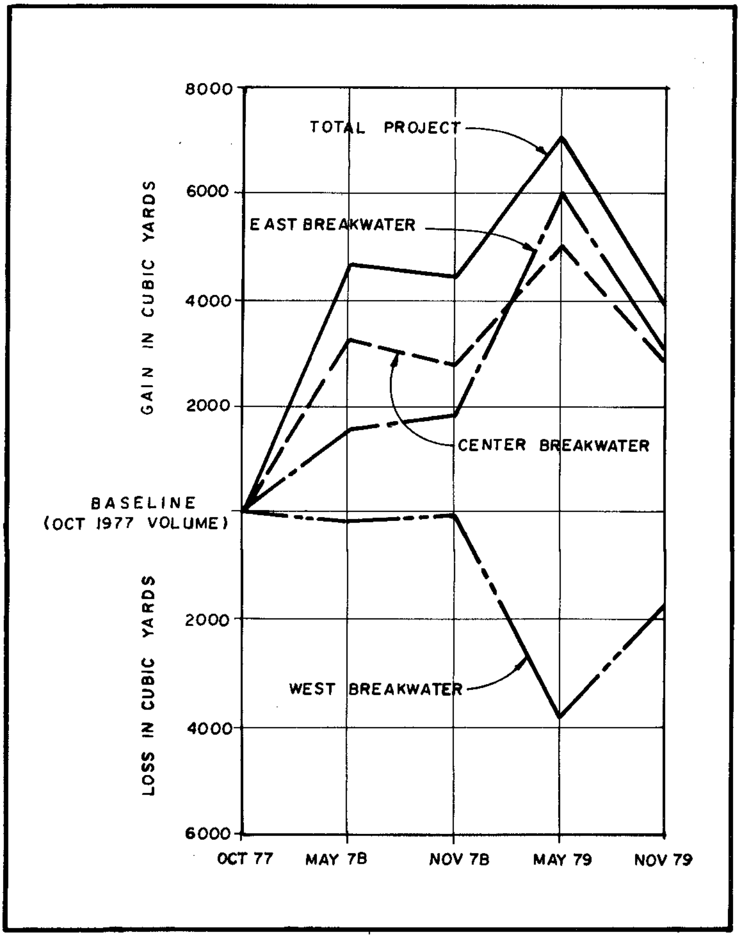

FIGURE 11: CHANGES IN LEE OF BREAKWATERS

behind the structures fluctuates. Both the central and eastern breakwaters now retain more material than was initially installed; however, the west breakwater has lost material. The areas of gain and loss can be seen in figure 12 as a function of survey period.

The overall project has gained about 5,000 cubic yards of material. The beach has become alined in a slightly more westward-facing orientation than predicted. This is evident by erosion of the west end. Although the project has been in place for 2 ½ years, no maintenance has been performed. Nourishment is planned for 1980 to replenish the west end of the project with 6,000 cubic yards of sand.

The project has been exposed to several severe storms from the west during periods of high Lake levels, but no damage to the breakwaters or groins has resulted.

Proper analysis of the project will require several years, in order to include observations during periods of low Lake levels. The intensity and frequency of storms from the west during the early years of the project may not be representative of the relative frequency of long-term, typical wave conditions. Furthermore, the winter of 1979-1980 was very mild, and Lake ice which normally protects the shoreline from severe winter storms did not form. Therefore, the project was subjected to severe winter storm waves had not been taken into account in the design analysis.

CONCLUSIONS

The beach-restoration project performed well despite being subjected to atypical severe westerly storms, high Lake levels, and a winter with no ice. The beach is the focal point of a popular recreational facility. The project has required little maintenance and has not induced erosion of the downdrift shoreline.

The simplified diffraction analysis using the predominant wave direction from the littoral transport analysis for prediction of beach planform appears to have worked reasonably well. Designing the beach landward of the $K_D = 0.3$ isoline intersections of diffraction diagrams drawn for the easterly and westerly resultants of wave-energy flux has precluded formation of a tombolo. The effects of wave attack due to severe individual storms may have a significant impact on localized areas such as the west end of the beach. Perhaps a greater safety margin should have been provided by making the initial beach wider or by extending the breakwater system farther westward.

Significant erosion has not been induced on the Lake bottom between the breakwater gaps. The height of the breakwater was sufficient to preclude wave overtopping from inducing a strong rip current that would scour the Lake bottom.

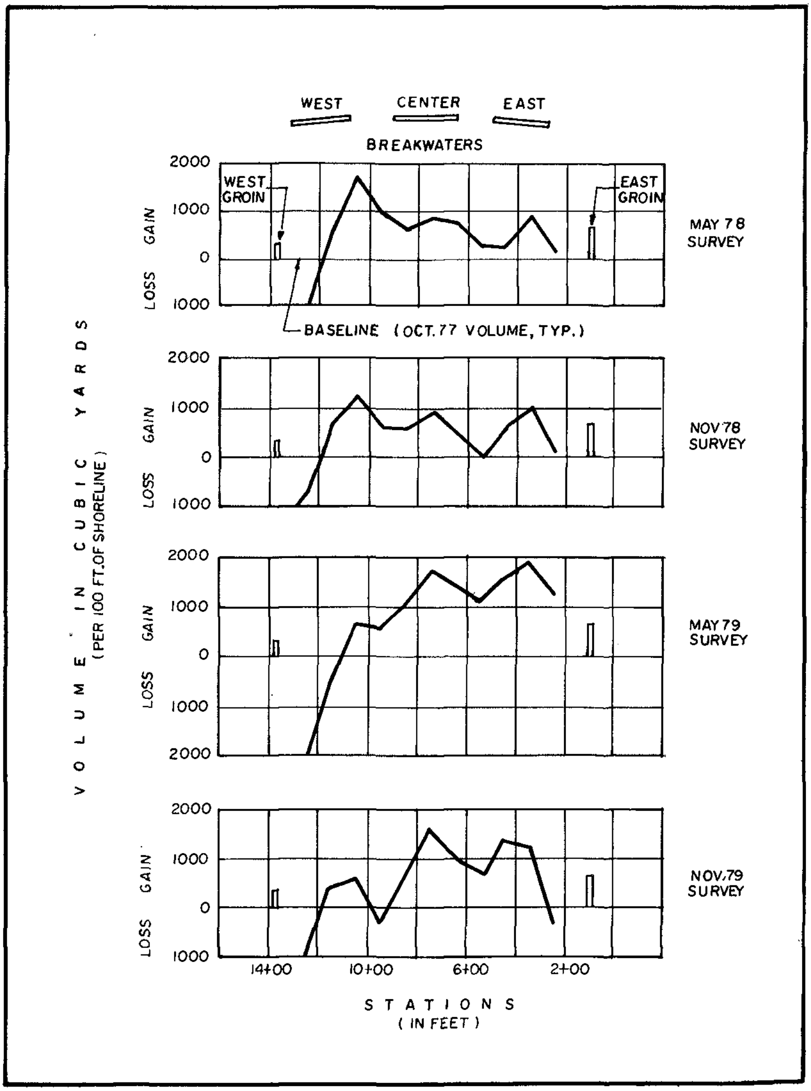

FIGURE 12: SAND VOLUME CHANGES RELATIVE TO OCTOBER 1977

RECOMMENDATIONS

The beach project at Lakeview provides a case history of a beach protection design using detached breakwaters. Insufficient funding was available to conduct a hydraulic model study or to develop a rigorous mathematical model prior to project implementation. Since the Lakeview design, Dean (1978) and others have developed shoreline evolution mathematical models that more accurately incorporate the effects of wave diffraction through segmented breakwaters. It would be useful to use a case study such as the Lakeview Park project to calibrate movable- or semimovable-bed hydraulic models and mathematical models.

Detached breakwaters have been shown to be a feasible method of protecting a beach. They should be considered where strict regulation of the beach planform is required. The gap spacings, the length and distance offshore, and the crest height should be designed by use of a rational procedure, giving adequate attention to sediment supply, the potential for littoral transport, variations in wave direction, the potential effects of single storms, and the consequences of variation in planform resulting therefrom.

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