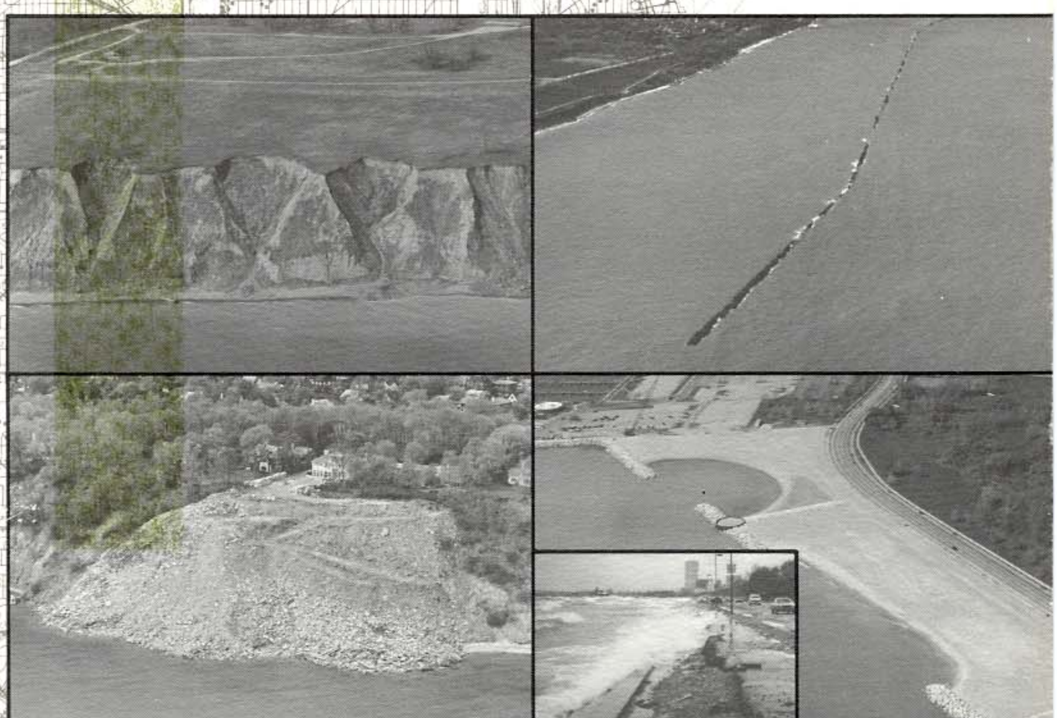
COMMUNITY ASSISTANCE
PLANNING REPORT NO. 163

A LAKE MICHIGAN SHORELINE EROSION MANAGEMENT PLAN FOR MILWAUKEE COUNTY WISCONSIN



SOUTHEASTERN WISCONSIN REGIONAL PLANNING COMMISSION

Front Cover Photographs:

Upper Left: Bender Park, April 1987.
Upper Right: South Shore Breakwater, April 1987.
Lower Left: Village of Shorewood, May 1986.
Lower Right: McKinley Beach, April 1988.
Insert: Lincoln Memorial Drive near McKinley Beach, December 1986.

Upper left, lower left, and lower right photographs by Robert T. McCoy.
Upper right and insert photographs by SEWRPC.

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Prepared by the

Southeastern Wisconsin Regional Planning Commission
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In Cooperation with the
WISCONSIN COASTAL
MANAGEMENT PROGRAM

The preparation of this report was financed in part by the Wisconsin Coastal Management Program under the Coastal Zone Management Act of 1972, administered by the Federal Office of Coastal Zone Management, National Oceanic and Atmospheric Administration.

October 1989

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SOUTHEASTERN WISCONSIN REGIONAL PLANNING COMMISSION

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October 9, 1989

Mr. F. Thomas Ament, Chairman
Milwaukee County Board, and
County Board of Supervisors
901 N. 9th Street
Milwaukee, Wisconsin 53233

Mr. David F. Schulz
County Executive
Milwaukee County Courthouse
901 N. 9th Street
Milwaukee, Wisconsin 53233

Gentlemen:

Protecting the Lake Michigan shoreline of Milwaukee County against wave and ice erosion has been a continuing, long-term problem for both lakefront property owners and the communities affected. These problems were exacerbated during the high-water period of the mid-1980's—with record-high water levels being reached in 1986. During that period, it became apparent that most shore protection structures were in need of major modification or repair to provide an adequate level of protection. It also became apparent that some structures were increasing the erosion of adjacent shoreline areas, that shore protection structures were being constructed with little coordination or control, and that insufficient guidance was being provided to both public and private lakefront property owners who needed to install protection against shoreline erosion.

Responding to the expressed need for more definitive information and for proper guidelines and procedures to assist public and private lakefront property owners, Milwaukee County in 1986 asked the Regional Planning Commission to prepare a shoreline erosion management plan. The planning effort was funded in part by Milwaukee County, and in part by the Wisconsin Coastal Management Program. Assisting the Commission in the work were Professors Tuncer B. Edil, PE, Theodore Green III, David M. Mickelson, and Kwang K. Lee, PE, of the University of Wisconsin system with bluff stability analyses and wave action simulation; W. F. Baird & Associates, Ltd., Oregon, Wisconsin, with evaluation of existing shoreline structure conditions; Wisconsin Testing Laboratories, Menomonee Falls, Wisconsin, with soil borings; Aero-metric Engineering, Inc., with large-scale topographic mapping and control surveys; and National Survey & Engineering, Inc., with land surveys and survey monumentation. The planning effort was carried out under the guidance of an Intergovernmental Coordinating and Technical Advisory Committee composed of representatives of the affected local communities, Milwaukee County, the Wisconsin Department of Natural Resources, the University of Wisconsin Sea Grant Institute, the University of Wisconsin-Milwaukee, the Milwaukee Metropolitan Sewerage District, the Audubon Society, the South Shore Yacht Club, and concerned and knowledgeable citizens.

For the approximately 30 miles of Lake Michigan shoreline within Milwaukee County, the studies on which the requested plan is based provide information useful to local governmental agencies and private property owners on existing shoreline conditions, and guidance on which measures can best protect against wave and ice action and stabilize the bluff slopes on a long-term basis. The adequacy of existing major shore protection structures to protect against various Lake Michigan water level and wave conditions is evaluated. The studies also identify those measures that are needed and are economically feasible, those measures that would not have a significant adverse impact either on adjacent shoreline areas or on the offshore coastal environment, and those measures that would, where practical, provide a shoreline usable for recreational activities. Alternative shoreline erosion control measures are evaluated, and a recommended plan is presented. An implementation program is also recommended to carry out the plan.

This report is being released during a period when Lake Michigan water levels are near long-term average stages and problems related to high water levels are diminishing. As a result, there is declining public interest in such problems. This turn of events should be viewed by public officials not as a basis for simply filing the report for possible future reference, but rather as an opportunity to begin what necessarily will be a long-term program of lakeshore improvements. Given the long lead times necessary for designing, funding, and constructing such improvements, the commonweal will best be served by steady progress toward plan implementation so that when lake levels again begin to rise—as historical experience indicates they will—lakefront property owners—public and private—will be well prepared.

The Regional Planning Commission is pleased to have been able to be of assistance in the preparation of this shoreline erosion management plan for Milwaukee County. The Commission stands ready, upon request, to assist Milwaukee County and the affected communities in presenting the information and recommendations to the public, and in adopting and implementing the recommendations contained in this report.

Sincerely,

Kurt W. Bauer
Executive Director

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Chapter I

INTRODUCTION

BACKGROUND

In January 1986, Milwaukee County requested that the Regional Planning Commission assist the County in defining and seeking solutions to the severe and costly shoreline erosion, bluff recession, and storm damage problems existing along the 30-mile reach of Lake Michigan shoreline within the County. Subsequently, the Commission applied for and obtained on behalf of Milwaukee County a grant under the Wisconsin Coastal Management Program in partial support of the conduct of a shoreline erosion management study for the entire Lake Michigan shoreline of Milwaukee County.

Work on the requested Milwaukee County study was initiated in May 1987, and completed in September 1989. The study was carried out under guidance of an Advisory Committee created by the Regional Planning Commission. The Committee consisted of representatives of each of the nine municipalities concerned, Milwaukee County, the Milwaukee Metropolitan Sewerage District, the Wisconsin Department of Natural Resources, the University of Wisconsin Sea Grant Institute, the University of Wisconsin-Milwaukee, the Milwaukee Audubon Society, concerned citizens, and the Regional Planning Commission. The full membership of the Advisory Committee is listed on the inside front cover of this report. The functions of the Committee were to help define the scope and content of the study, as well as to guide the development of a recommended shoreline erosion management plan for the Lake Michigan shoreline of Milwaukee County. The study included an inventory and analysis of the existing shoreline erosion, bluff recession, and storm damage conditions; an inventory and analysis of existing shore protection measures; an evaluation of alternative shoreline erosion, bluff recession, and storm damage control measures; selection of a recommended shoreline erosion management plan; and identification of the means for implementing the recommended plan.

DEFINITION OF SHORELINE EROSION, BLUFF RECESSION, AND STORM DAMAGE MANAGEMENT

For the purposes of this study, shoreline erosion, bluff recession, and storm damage management was defined as a coordinated set of measures designed to abate shoreline erosion, bluff recession, and storm damage, and thereby reduce attendant property losses, undesirable aesthetic impacts, and risks to human safety. Management measures include both onshore and offshore structural measures—such as revetments, bulkheads, groins, breakwaters, peninsulas, and islands—and nonstructural measures—such as land use regulations which prohibit certain types of development and land use activities in erosion-prone shoreland areas. The broad goal of shoreline erosion, bluff recession, and storm damage management is to effectively reduce the costs associated with such erosion, recession, and damage; and to enhance the overall quality of life of the residents of the area through the selective protection of those recreational, aesthetic, ecological, and cultural values normally assoicated with, and found concentrated in, coastal areas, and in a manner sensitive to and respectful of the historical development, traditional values, and uniqueness of certain shoreline areas.

NEED FOR A SHORELINE EROSION MANAGEMENT STUDY

There are two major adverse impacts of coastal processes on the Milwaukee County shoreline. The first is the erosion, and subsequent recession, of coastal terraces, bluffs, and beaches which threaten residential areas, parklands, public roadways, and industrial sites. From 1963 through 1985, average annual bluff and terrace recession rates ranged up to almost 13 feet. Over this 22-year period of record, this erosion resulted in an average annual loss of nearly 330,000 cubic yards of shore material and about 2.7 acres of land. It should be noted that shoreline erosion tends to be episodic rather than continuous. Erosion and recession rates may thus vary widely from year to year. Figure 1 shows a house in the City of Oak Creek near Bender Park threatened by bluff recession in 1973. As noted in the caption to the photograph, the bluff receded 63 feet over a 15-month period. Average annual recession rates in the Bender park area over the period 1963 through 1985, however, ranged from 3 to about 12.5 feet.

The second major impact of coastal processes is their effect on the various types of shore protection measures which have in the past been constructed to protect both public and private property from erosion and bluff recession damage. Some of these shore protection measures may have been ineffective, some subsequently damaged by wave action, and some perceived to be unsightly; and some may have accelerated erosion and bluff recession in adjacent shoreline areas. Significant concern was expressed by elected officials and citizens about the effects of high lake levels, such as those which occurred in 1986, on existing shore protection measures, harbor facilities, and lakefront buildings and facilities. Therefore, there was a need in the study to critically re-examine the approaches taken to protecting the shoreline, and to attempt to find more cost-effective approaches to shore protection.

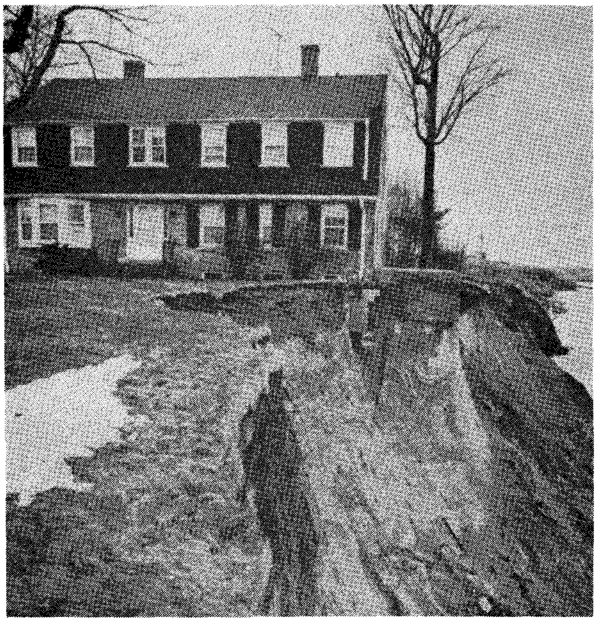
These two major adverse impacts of coastal erosion and bluff recession processes are accompanied by storm damage, not only to shorelines and adjacent land use, but to commercial and recreational vessels seeking refuge in the Milwaukee outer harbor.

Several primary needs for additional information were identified by the Advisory Committee and addressed in the study. These included:

- The need for more adequate knowledge about specific conditions and processes which contribute to shoreline erosion and bluff recession in Milwaukee County;
- The need for more adequate knowledge about the effectiveness of existing shore protection structures and harbor facilities in preventing shoreline erosion, bluff recession, and storm damage under various storm wave and water level conditions in Milwaukee County;

Figure 1

CITY OF OAK CREEK RESIDENCE THREATENED BY BLUFF RECESSION: 1973



This residence near Bender Park was severely threatened by bluff recession in January 1973. By April 1974, the bluff had eroded an additional 63 feet, and the residence was lost.

Photo courtesy of Oak Creek Pictorial.

- The need for more adequate knowledge about the adverse as well as beneficial effects of the various nonstructural and structural shore protection measures which can be used to protect private property as well as public facilities and parkland; and
- The need to better define the proper role of the County and the local units of government with respect to: the development and enforcement of shore protection design and construction standards and regulations; the coordination of the installation of large structures within entire physiographic reaches of shoreline; the development of financing arrangements for needed measures to protect private as well as public property; public education; and the control of shoreline erosion and bluff recession on public property.

The significant data base provided by this study provides an opportunity for affected private property owners, as well as public officials, to attain a better understanding of the severity and causes of the shoreline erosion, bluff recession, and storm damage problems existing in Milwaukee County. Accordingly, this report is intended to serve as a source of pertinent information on site conditions and design criteria which can help property owners, design engineers, the County, and the local units of government in the assessment of specific shoreline erosion, bluff recession, and storm damage problems and the formulation of solutions thereto.

REVIEW OF PREVIOUS STUDIES

An important element of the study was the collation and analysis of the findings and recommendations of previous studies relating to shoreline erosion, bluff recession, and storm damage in Milwaukee County. The following section identifies and briefly describes the major shore erosion studies heretofore conducted within Milwaukee County. The findings and recommendations of these studies are reflected, as appropriate, in Chapter II, "Inventory Findings"; Chapter III, "Evaluation of Coastal Erosion Problems and Damages"; and Chapter IV, "Alternative Shoreline Erosion Control Measures and a Recommended Shoreline Erosion Management Plan for Milwaukee County," of this report.

1. Proposed Extension of Lincoln Memorial Drive from Lake Park to Green Tree Road, T. Lindberg, Milwaukee County Regional Planning Department, 1934.

   This 1934 study by the Milwaukee County Regional Planning Department recommended that a series of offshore islands be constructed from Lake Park in the City of Milwaukee to E. Green Tree Road in the Village of Fox Point, as shown in Figure 2. The proposed islands were to be designed to provide protection against wave erosion, create additional public lake frontage, allow extension of Lincoln Memorial Drive to E. Green Tree Road, and provide protection for small boating activities. The study found that construction of offshore islands within the study area would be technically feasible. The study recommendation was not implemented.

2. "Stabilizing a Lake Michigan Bluff," C. S. Whitney, Civil Engineering, Vol. 6, No. 5, May 1936, pp. 309-313.

Figure 2

PROPOSED EXTENSION OF LINCOLN MEMORIAL DRIVE: 1934

The Milwaukee County Regional Planning Department recommended in 1934 that a series of islands be constructed which would allow the extension of Lincoln Memorial Drive from Lake Park in the City of Milwaukee to E. Green Tree Road in the Village of Fox Point. This recommendation was reaffirmed by Milwaukee County in 1945, and by the U. S. Army Corps of Engineers in 1945 and in 1975. Funds were never provided to implement the recommendation.

Illustration by T. Lindberg.

From the 1880's to approximately 1915, the Whitefish Bay Resort, located on the Lake Michigan shoreline between E. Henry Clay Street and E. Silver Spring Drive in the Village of Whitefish Bay, was a popular site for recreational and social activities, as shown in Figure 3. The resort was subsequently sold and the lot was subdivided for residential development. In the late 1920's and early 1930's, major bluff failures began to occur. An investigation of the causes of the bluff failures was completed in 1936 by Charles S. Whitney, a consulting engineer retained by the private property owners. The study presented information on the characteristics of the beach and bluff, and on the topography and groundwater conditions within the Lake Michigan near-shore area. In addition, the study described an erosion control method used to minimize further bluff failure. The method included the use of drainage tunnels to reduce the groundwater level and to relieve the hydrostatic pressure within a 530-foot reach of

Figure 3

WHITEFISH BAY RESORT: 1900

At the turn of the century, the Whitefish Bay Resort, located on the Lake Michigan shore just north of E. Henry Clay Street, was a popular gathering spot for area residents. However, business declined in the early 1900's, and the resort was sold in about 1915 and the property developed for residential use. In the late 1920's and early 1930's, bluff slope failure occurred in the area once occupied by this resort.

Photo from Whitefish Bay Resort, by Miriam Bird, courtesy of the Whitefish Bay Public Library.

shoreline. The drainage system was implemented in 1932 and continued to effectively discharge water until about 1960. The drainage of the groundwater may have reduced further slippage of the bluff slope. However, some slope movement apparently continued to occur, and the drainage system ceased to operate around 1960 when the outlet was damaged, perhaps by slope failure. In the 1970's, concrete rubble and soil were placed on the bluff slopes to stabilize them.

3. Beach Erosion Study, Lake Michigan Shore Line of Milwaukee County, Wis., U. S. Army Corps of Engineers, 1945.

In 1945, the U. S. Army Corps of Engineers completed a study to recommend methods of preventing beach erosion and of restoring and creating new beaches along the entire Milwaukee County Lake Michigan shoreline. Under the study, information was compiled on the geologic conditions of the area; wind and weather; the near-shore bathymetry; the sources and movement of the beach material; and the effects of lake levels and wave and ice action on the shoreline, and on shore protection structures.

The study recommended that the shoreline from the Milwaukee County-Racine County line northward to the mouth of Oak Creek be protected either by riprap revetments, or by a lakefront fill having beaches at intervals; that the groins at the mouth of Oak Creek, initially constructed in 1891, be restored; that riprap revetments and groundwater drainage be provided from the mouth of Oak Creek northward to the harbor breakwaters; and that restoration and artificial nourishment of the groin systems at Grant Park and Sheridan Park be provided. North of the harbor breakwaters to the City of Milwaukee Linnwood Avenue water treatment plant, an artificially nourished groin system was recommended. The shoreline from the Linnwood Avenue water treatment plant northward to E. Green Tree Road was recommended to be protected by an extension of Lincoln Memorial Drive along a lakefront fill having beaches at intervals, including at Atwater Park and Big Bay Park. It was recommended that the remainder of the county shoreline north of E. Green Tree Road be protected by a groin system artificially nourished with sand. The study also concluded that the federal government should not provide funds for the implementation of shore protection measures in Milwaukee County.

Since the publication of this study, most of the county shoreline south of the mouth of Oak Creek has not been protected and continues to erode. The groin system recommended to be constructed north of the harbor breakwaters to the City of Milwaukee Linnwood Avenue water treatment plant was not installed, with revetments instead being used to protect the shoreline between McKinley Beach and the water treatment plant. Both McKinley Beach and portions of Lincoln Memorial Drive have recently been threatened by high lake levels and the attendant increased erosion of the shoreline, as shown in Figure 4. The groin systems at

Figure 4

LAKE MICHIGAN SHORELINE NEAR LAKE PARK: 1908 AND 1986

LAKE PARK: CIRCA 1908

LINCOLN MEMORIAL DRIVE: 1986

The 1908 photograph (top view) shows the stable, well-vegetated bluffs of Lake Park at the Lake Michigan shoreline. Fill was later placed in the lake to create additional land, and Lincoln Memorial Drive was constructed on the fill. The land lakeward of Lincoln Memorial Drive eroded, allowing wave action during intense storms to wash debris onto the roadway, as shown in the December 1986 photograph (bottom view). The McKinley Beach Restoration Project, begun in 1987, is intended to protect this roadway (see Figure 9).

Photo (top) by E. C. Kropp Publications.
Photo (bottom) by SEWRPC.

Grant Park and Sheridan Park have not been nourished with sand; nevertheless, they still contain beaches and provide marginal protection of the bluffs. These groins were periodically repaired and maintained in the 1940's and 1950's, but have not been recently maintained and are in need of substantial repair. Figure 5 shows the Sheridan Park groins in 1945 and in 1987. The recommendation concerning the extension of Lincoln Memorial Drive has not been implemented. North of E. Green Tree Road relatively few groin systems have been installed and none have been nourished with sand. The entire county shoreline north of the Linnwood Avenue water treatment plant is partially protected by revetments, bulkheads, and a few groins. In summary, the shore protection measures recommended by the U. S. Army Corps of Engineers have been only partially implemented, with much of the county shoreline remaining inadequately protected against storm wave action.

4. Lake Michigan Shore Erosion, Milwaukee County, Wisconsin, Report of the Milwaukee County Committee on Lake Michigan Shore Erosion, 1945.

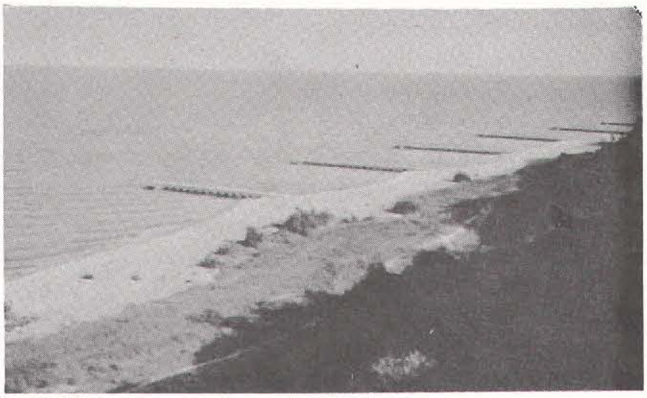
This study, authorized by the Milwaukee County Board of Supervisors on December 7, 1943, was conducted in order to evaluate the causes of shoreline erosion and to recommend control measures for the erosion conditions in Milwaukee County. The study, conducted cooperatively and concurrently with the U. S. Army Corps of Engineers' Beach Erosion Study described above, presented information on Lake Michigan water levels, geologic conditions, the extent of shore erosion, shoreline recession rates, existing shore protection structures, and beach conditions. To protect public property, it was recommended that groins be built at Doctors Park and that additional groins be built at Grant Park; that the beach be widened from the harbor breakwater northward to the City of Milwaukee Linnwood Avenue water treatment plant; and that maintenance work be done on the revetment at South Shore Park, and on the South Shore breakwater. An early photograph of the South Shore breakwater is shown in Figure 6, along with a 1987 photograph which illustrates the effects of overtopping and a lack of maintenance of the breakwater.

For private property, it was recommended that the type of erosion control structure installed be selected by individual property owners, but consist of riprap revetments or concrete bulkheads, along with groins for

Figure 5

SHERIDAN PARK GROINS: 1945 AND 1987

1945



1987

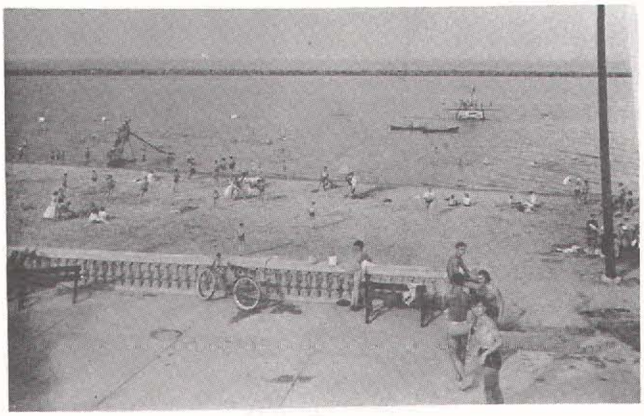
The Sheridan Park groins, constructed in 1933, have built a beach which has protected the bluff slopes for over 50 years. The accretion of the beach occurred rapidly; within 10 years after construction, over three acres of beach had been formed. The 11 permeable groins are constructed of precast concrete beams and sills arranged in an overlapping criss-cross fashion with a cover of solid concrete slabs. Because of a lack of maintenance, however, the groins have deteriorated and are now subject to overtopping and material loss. Increased erosion of the beach threatens the stability of the bluff slopes.

Photo (left) by Milwaukee County.
Photo (right) by SEWRPC.

Figure 6

MILWAUKEE SOUTH SHORE BREAKWATER

UNDATED



1987

The 12,500-foot-long Milwaukee South Shore breakwater was constructed in segments between 1913 and 1931. Most of the segments were constructed by the City of Milwaukee, although the southernmost 600 feet of the breakwater was built by The Milwaukee Electric Railway & Light Company, the predecessor company to the Wisconsin Electric Power Company. As shown in the early photograph, the breakwater, located about 1,000 feet offshore in an approximate water depth of 20 feet, provided substantial protection of the shoreline from the Milwaukee Harbor southward to the City of St. Francis. In 1950, Milwaukee County assumed responsibility for maintaining the breakwater, but little maintenance has actually been performed. As shown in the 1987 photograph taken near the South Shore Marina, the breakwater has deteriorated and is overtopped by waves during storms.

Photo (left) courtesy of the Milwaukee Public Library.
Photo (right) by Port of Milwaukee.

those areas where a beach was desired. The study committee noted that an effective solution to erosion problems in the northern portion of the County would be to extend Lincoln Memorial Drive on fill placed at the base of the bluff northward to the Village of Fox Point. The report stated that such an alternative would not only provide shore protection, but also provide improved public access to the lake shore. The Committee recommended that some form of coordinated government regulation of the design, construction, and maintenance of shore protection structures be established. As set forth in Chapter II of this report, revetments, bulkheads, and groins, as recommended in this 1945 study, have continued to be used to provide privately funded shore protection. A groin system was constructed in the 1950's at Doctors Park. Again, no action was taken on the proposed northerly extension of Lincoln Memorial Drive.

5. <u>Master Plan-Milwaukee County Marina Development</u>, Ralph H. Burke, Inc., Engineers-Architects, Chicago, Illinois, November 1958.

   At the request of the Milwaukee County Park Commission, a study was conducted in 1958 to evaluate 12 potential marina sites within the County. Seven of these sites were located on Lake Michigan, while the remaining five sites were located on the Milwaukee and Kinnickinnic Rivers, and on man-made lakes located inland near Oakwood Road and Brown Deer Road. A brief description of each of the proposed marina sites was presented in the study report, which also included information on the specific facility requirements, capacity, feasibility, and construction costs at each site. The plan recommended that a large marina be built at McKinley Beach; that the South Shore Yacht Club be expanded; and that new marinas be constructed at Grant Park, the north side of Lake Park, Sheridan Park, Ryan Road, and Doctors Park. The study set forth an implementation plan for Milwaukee County to develop the additional boating facilities in stages, in order that various sites could be developed individually or simultaneously. Some, but not all, of these recommendations were acted upon. Permanent docking facilities in addition to the existing open moorings were installed at the McKinley Marina and South Shore Yacht Club.

6. <u>Problems of Great Lakes Shore Erosion</u>, W. T. Painter, a paper presented at First World Congress on Water Resources, Chicago, Illinois, September 1973.

   This paper presented the findings of investigations of the causes of shoreline erosion and major bluff failures that occurred along the Lake Michigan shoreline, including one major slide that occurred in April 1973 at 5270 N. Lake Drive in the Village of Whitefish Bay. This property is located within the reach of shoreline that had been previously studied by Whitney (1936). The investigation, which was conducted between April and July 1973, collected information on the characteristics of the bluff and subsoil conditions within this Lake Michigan near-shore area. A combination of erosion control methods was used to minimize further bluff slope failure, including groundwater drainage facilities, fill, and a riprap revetment for toe protection. A stability analysis using the final slope configuration indicated the weight of the fill should prevent any future deep rotational slides. Since 1973, the fill has successfully stabilized the bluff slope, although maintenance of the toe protection may be required, as shown in Figure 7.

7. <u>A Geological Reconnaissance of Bender Park, Milwaukee County, Wisconsin</u>, David W. Hadley, 1974.

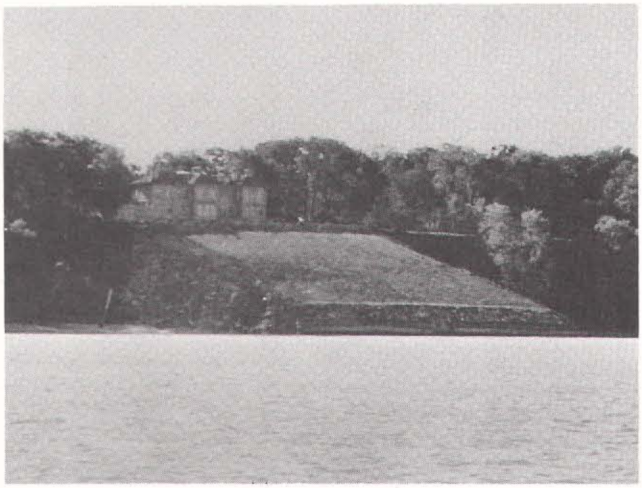
   A study of shoreline conditions at Bender Park, funded by the City of Oak Creek, was conducted to provide information on the geologic conditions of the bluff slopes. The study results were used by the City to evaluate the potential of the park for the location of a marina. The inventory of shoreline conditions within the park, which was conducted in September 1974, provided data on beach, bluff, and geologic characteristics.

   Although no specific recommendations were made, the report concluded that in its undeveloped state, the shoreline of the

Figure 7

BLUFF SLOPE FILL PROJECT AT 5270 N. LAKE DRIVE, VILLAGE OF WHITEFISH BAY: 1976 AND 1986

1976



1986

A major bluff slippage occurred at 5270 N. Lake Drive in April 1973, following a severe rainfall. Later that year, concrete rubble and soil were placed on the bluff slope to increase its stability. The fill material has effectively stabilized the slope, although erosion at the toe of the bluff is occurring, as shown in the 1986 photograph.

Photo (left) by James D. Rosenbaum.
Photo (right) by SEWRPC.

park was of limited recreational value. Most of the shoreline lacked beaches, and the use of the limited existing beaches was hazardous because of the threat of slides and slumps. The report concluded further that protection of the shoreline would be costly and require regrading of the bluffs; a lowering of the elevation of the ground-water table; and the construction of protective structures at the toe of the bluffs. The study was submitted to the City of Oak Creek for use in the City's Bender Park Marina justification described below.

8. Bender Park Boat Marina Justification, City of Oak Creek, April 1975.

In 1975, the City of Oak Creek undertook a study intended to promote the construction of a boat launch facility at Bender Park. The report was prepared by a study committee of city citizens and included information on the bluff characteristics, bluff recession, and the economic value of the parkland. The study found that because of the nature of the soils comprising the bluffs and the presence of protective structures which extend out into the lake on both sides of the park, erosion was progressing at a much greater than normal rate, with bluff recession rates of up to 20 feet per year being recorded during the period 1961 through 1974. The benefits of placing the marina at Bender Park, as determined by the study committee, included ready access to the shoreline for residents of the area; minimal potential adverse impacts on adjacent shorelines; minimal dredging requirements to maintain a marina; proximity to prime fishing areas; no disturbance to other park functions; and the opportunity to preserve and develop a valuable recreational area. The proposal was not implemented, and Bender Park remains in an undeveloped state.

9. Lake Michigan Shoreline, Milwaukee County, Wisconsin, U. S. Army Corps of Engineers, 1975a.

In 1975, the U. S. Army Corps of Engineers undertook a study intended to investigate the severity of the shoreline erosion problems in Milwaukee County, and to develop and evaluate alternative solutions to the problems along the publicly owned shorelands in Milwaukee County. The scope of the study was subsequently expanded to include a preliminary study of the earlier proposals to extend Lincoln Memorial Drive on land along, or offshore of, the Lake Michigan shoreline from the City of Milwaukee Linnwood Avenue water treatment plant to E. Green Tree Road. The study report presented data on climate, population, income, transportation facilities, recreational resources and demands, shore erosion damages to land and structures, and environmental impacts of alternative erosion control measures. The alternative shore protection measures that were evaluated included revetments, groins, breakwaters, and offshore islands. A proposed marina at Bender Park was also evaluated. The study did not recommend specific shore protection measures for the individual sections of shoreline considered. Generally, this study concurred with the 1945 recommendation by the Corps of Engineers that no federal funds be used for the protection of the shoreline in Milwaukee County. However, it was recommended that the potential for a recreational boat harbor at Bender Park be further investigated in the Corps study of harbors between Kenosha and Kewaunee, Wisconsin.

10. Harbors Between Kenosha and Kewaunee, Wisconsin, Preliminary Feasibility Report, U. S. Army Corps of Engineers, 1975b.

    In 1975, the U. S. Army Corps of Engineers undertook a study to investigate the need for additional recreational boating facilities between the Illinois-Wisconsin state line and the Kewaunee-Door County line. The study proposed that the forecast need for increased recreational boating facilities be met by constructing additional facilities at several existing federal harbors, and by constructing several entirely new harbors. Of the 10 new sites considered, three were in Milwaukee County—Doctors Park, the mouth of Oak Creek, and Bender Park.

    A brief description of each of the proposed harbor sites was presented in the study, including information on the design and capacity of the harbors; the economic, social, and environmental impacts of the proposed sites; and construction costs. Information was also compiled on longshore transport rates, wind and wave conditions, and Lake Michigan water levels.

    The study recommended that a detailed investigation be conducted to assess the degree of local support at the various potential harbor sites, and the need or urgency for improvements at specific sites to meet projected demand in a timely manner. The study further recommended that the federal government provide the final design and prepare plans and specifications for authorized projects. Although the study showed that there was a demand for additional harbor facilities and that construction of harbors at the proposed sites within Milwaukee County may be technically feasible, none of the proposed projects were implemented.

11. Shore Erosion Study, Technical Report, Appendix Three, Milwaukee County, D. M. Mickelson, R. Klauk, L. Acomb, T. Edil, and B. Haas, Wisconsin Coastal Management Program, 1977.

    An inventory of shoreline conditions within Milwaukee County was conducted in 1977 under the Wisconsin Coastal Management Program as part of a study of shore erosion along the Lake Michigan and Lake Superior shorelines of Wisconsin. The county shoreline was divided into four reaches, each reach having similar physical- and erosion-related characteristics. The study estimated that long-term—110-year—bluff recession rates ranged from one to three feet per year for the County. The study presented data on beach, bluff, and geologic characteristics, and analyzed shore damages and shore protection structures. Forty-eight bluff slope stability analyses and four soil borings were conducted under the study within the County. The study did not recommend specific types of shore protection measures.

12. Great Lakes Shoreland Damage Study, U. S. Army Corps of Engineers, February 1979.

In 1979, the U. S. Army Corps of Engineers completed a study to estimate the cost of erosion damage to the U. S. Great Lakes shorelands over the period 1972 through 1976. High water levels occurred during 1973 and 1974. The shoreline was divided into a series of reaches, each reach having similar land use, coastal processes, and shore form. A representative sample of property owners within each reach was contacted and asked to supply information on the assessed value of their property, the date of assessment, the legal description of their property, and the current status of their property and its land use. Property owners were also asked to provide a description, including cost, of any shoreline protection measures implemented. Shoreland damage costs were assessed by comparing the cost of damages incurred to the cost of protection. Based on 1973 property values, the estimated shoreland damage incurred along the Wisconsin shoreline of Lake Michigan during the 1972 through 1976 period was $20 million, compared to about $10 million spent by property owners on shoreland protection.

13. Lake Michigan Shoreline Study 1979-1980—Grant Park to Bender Park, Nelson & Associates, Inc., Foundation Engineering, Inc., and American Appraisal Company, 1980.

In 1980 a study funded in part by the Milwaukee County Park Commission and in part by the Wisconsin Coastal Management Program was undertaken to evaluate the geologic conditions, extent and rate of shoreline recession, causes of shoreline erosion, and existing land use and ownership within the 3.5-mile shoreline located between Grant Park and Bender Park. Two bluff profiles and four soil borings were taken under the study, with piezometers installed at two of the boring sites to observe groundwater fluctuations. Alternative types of shore protection measures were reviewed and three alternative shoreline stabilization plans were developed. All three plans utilized the fill method for slope stabilization, but the plans varied with respect to the quantity of fill required, the acquisition of riparian rights from private land owners, and the recreational opportunities provided. Under all three alternative plans, revetments or bulkheads would be constructed at the toe of the fill for protection against wave damage. The study concluded that the alternative methods of erosion control were technically feasible for the southern Milwaukee County shoreline, and that the estimated quantities of construction material were within reasonable limits so that construction could be undertaken over a 6- to 12-year period. The study also noted that the ability of the County to implement the plan would be enhanced by the availability of tunnel boring machine spoils to be produced by the Milwaukee Metropolitan Sewerage District water pollution abatement project, which would thereby reduce construction costs. No action was taken on this plan.

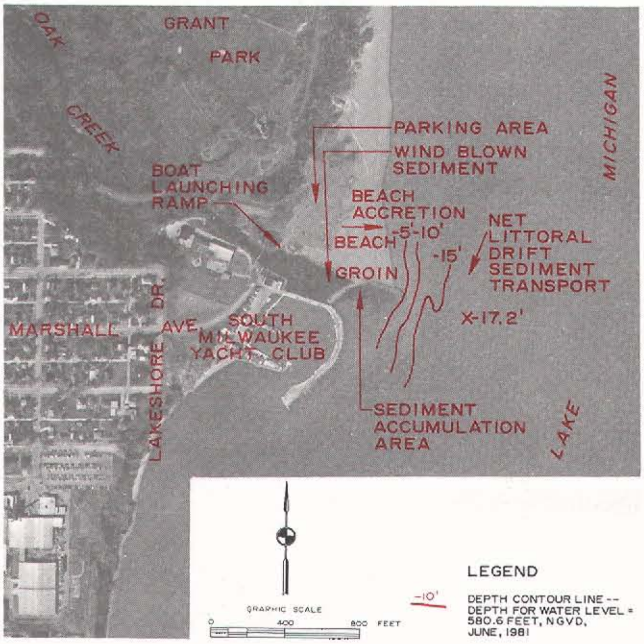
14. Geological Study, Lake Michigan at Mouth of Oak Creek, Edith M. McKee, Consulting Geologist, Winnetka, Illinois, October 1983.

In 1983, the Milwaukee County Department of Parks, Recreation and Culture undertook a study to determine the best means of maintaining a navigable channel for small boats at the mouth of Oak Creek. As shown in Figure 8, the mouth of Oak Creek is frequently obstructed by a sandbar. The study concluded that the cause of the problem was sand carried by wind from the Grant Park beach into the channel, with smaller amounts of sand being carried in by longshore currents and waves. This study presented information on beach and bluff characteristics, offshore bathymetry, and sources and movement of the beach material in the shoreline areas near the mouth of Oak Creek.

The study recommended that the Milwaukee County Department of Parks, Recreation and Culture install an offshore, artificial sandbar at the Grant Park beach, which would slow the longshore current, dissipate wave energy, and ultimately accrete additional sandbars and a beach, thereby preventing large amounts of sand from washing around the groin and into the channel. It was also recommended that a double hedgerow be planted parallel to

Figure 8

THE MOUTH OF OAK CREEK
AT LAKE MICHIGAN: 1985



The use of a recreational boat-launching ramp at the mouth of Oak Creek is periodically hampered by the formation of a sandbar at the mouth of Oak Creek between the boat launch and Lake Michigan. This sandbar formation began after the construction of two rubble-mound jetties at the mouth of Oak Creek in 1891.

Source: SEWRPC.

the groin on the north side of the Oak Creek channel in order to reduce the amount of sand blown into the channel. In 1984, a windbreak was constructed on the beach north of the creek in the form of a mound upon which shrubbery was planted. Although significant storage of sand was apparent on the north side of this structure, the shoaling problem at the mouth of Oak Creek remained severe.

15. A Lake Michigan Coastal Erosion and Related Land Use Management Study for the City of St. Francis, Wisconsin, Southeastern Wisconsin Regional Planning Commission, Community Assistance Planning Report No. 110, August 1984.

    At the request of the City of St. Francis, the Regional Planning Commission conducted a Lake Michigan shoreline erosion and related land use management study for the City's shoreline in 1984. The shoreline borders the location of the former Wisconsin Electric Power Company Lakeside electric power generating facility, which ceased operation in 1983. The study was funded in part by a federal grant made through the Wisconsin Coastal Management Program, and in part by funds provided by the Wisconsin Electric Power Company and the City itself. The study presented data on existing land use and zoning; beach, bluff, and geologic characteristics; existing regulations pertaining to shoreland development; shore protection structures; coastal erosion problems; and bluff recession rates. The study evaluated alternative structural shore protection measures and identified shoreline erosion risk distances and setback distances for buildings and facilities along shoreline reaches.

    A recommended set of regulations which could be incorporated into the existing city zoning and subdivision ordinances to protect proposed new urban development within those shoreland areas susceptible to erosion and bluff recession was provided. Plan maps were presented which showed those areas which could be utilized for urban development. Following the closing of the Lakeside facility, the site was sold to the St. Francis Lakeside Group, a land development organization. That organization proposed placing a landfill out into the lake to protect the shoreline, stabilize the bluff slopes, and create additional land suitable for urban development. As of 1988, some fill had been placed on the bluff slopes. The City did not adopt the regulations called for in the plan, but instead has used the plan as a guide in reviewing development proposals for the site.

16. Preliminary Site Investigation for Proposed Development in St. Francis, Wisconsin, Pittsburgh Testing Laboratory, Milwaukee, Wisconsin, July 1985.

    In 1985, a study was undertaken by the St. Francis Lakeside Group to provide information on subsurface conditions of lands adjacent to the Lake Michigan shoreline within the City of St. Francis, and to make general recommendations for the construction and design of building foundations for a proposed development. As part of the study, 45 soil borings were taken. Three alternative bluff slope stabilization plans were developed to protect the proposed development against shoreline erosion and recession. The first alternative plan consisted of regrading the bluff slope to a stable slope angle. The second alternative plan consisted of terracing the face of the bluff and providing a bulkhead at the toe of the bluff to protect against wave action. The third alternative plan consisted of the placement of fill material out into the lake and providing a bulkhead at the base of the fill to protect against wave action. This third alternative plan was selected in concept, and construction was initiated in 1986. As noted in the above discussion of the 1984 study by the Regional Planning Commission, as of 1988 a concrete rubble and soil landfill was being placed on the bluff to protect the shoreline, stabilize the bluff slope, and provide additional land for urban development, in accordance with the study recommendations.

17. A Comprehensive Plan for the Oak Creek Watershed, Southeastern Wisconsin Regional Planning Commission, Planning Report No. 36, August 1986.

    At the request of the Milwaukee Metropolitan Sewerage District and the City of South Milwaukee, the Regional Planning Commission conducted a comprehensive study of the severe flooding, water pollution, and related land use problems of the Oak Creek watershed. The study was completed in 1986. With respect to Lake Michigan shoreline conditions, the study presented information on existing beach characteristics, summarized the sources and movement of the beach material along the shoreline near the mouth of Oak Creek, and evaluated previous proposed remedies to the shoaling problem at the mouth of Oak Creek.

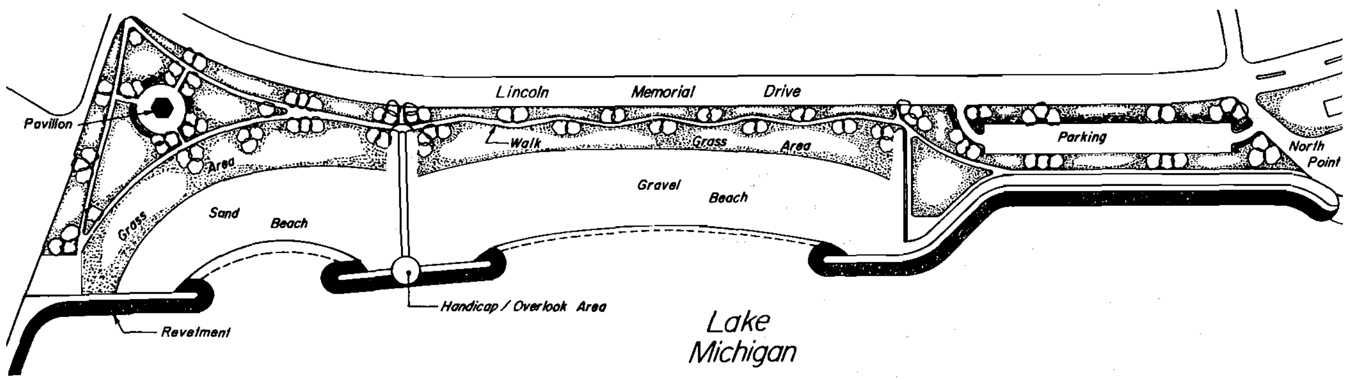
    The study developed four new alternatives designed to abate the shoaling problem. All of the alternatives involved flushing sand from the mouth of the creek using either the natural streamflow, or temporarily stored flow which would be periodically released. To help flush sand from the mouth of Oak Creek, it was recommended that a narrower channel be constructed. The existing jetty on the north side of the creek would serve as one channel boundary, and a new parallel bulkhead would be installed 20 feet to the south of the jetty. The west end of the new bulkhead would be connected to the jetty on the south side of the current channel. The plan recommended that diffusers be placed along the navigation channel to help scour the sand from the channel. To complement this effort, it was recommended that the sand level on the beach just north of the channel be lowered to provide for windblown sand storage behind the groin, and that minimal dredging be performed in the navigation channel in order to maintain a water depth of four feet. The plan recommended that the Department of Parks, Recreation and Culture be responsible for the construction of the bulkhead and the dredging of the new navigation channel. The Department has taken steps to implement the recommended plan. In February 1988, the detailed design of the recommended plan was in preparation.

18. McKinley Beach Restoration Project, Warzyn Engineering, Inc., 1986.

    In 1986 at the request of the Milwaukee County Department of Parks, Recreation and Culture, Warzyn Engineering, Inc., undertook a study of alternative measures to provide shoreline erosion protection for the 1,400 lineal feet of shoreline along the N. Lincoln Memorial Drive extension from the north dockwall of the McKinley Marina to the south end of the North Point parking area. The recomended project, as shown in Figure 9, uses a headland/beach system to protect the shoreline. Revetment protection provided at the north and south end of the project help to contain two beaches in the middle—one composed of sand and one composed of pebbles. The project not only provides shoreline protection, but also adds over 12 acres of parkland to this highly used recreational area. A total of 350,000 cubic yards of crushed limestone from the Milwaukee Metropoli-

Figure 9

PROPOSED MCKINLEY BEACH RESTORATION PROJECT



Source: Milwaukee County Department of Parks, Recreation and Culture.

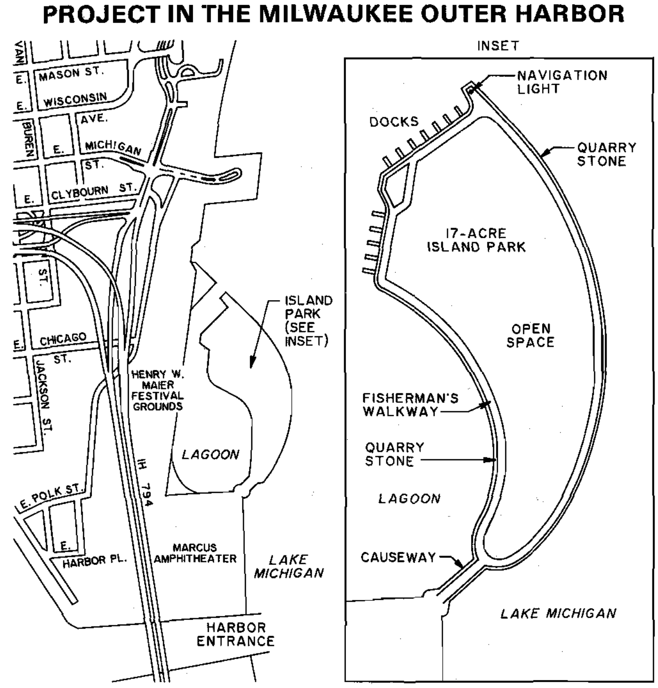
tan Sewerage District water pollution abatement project was used in the project. The total cost of the McKinley shoreline protection portion of the project was estimated at $3.9 million. Additional expenditures will be required for landscaping and park facilities. The construction of the project began in the summer of 1987 and is expected to be completed in the fall of 1989.

19. Island Park Project, Milwaukee Metropolitan Sewerage District, 1987.

A recreational island was proposed to be constructed offshore of the Henry Maier Festival grounds in the Milwaukee outer harbor. As shown in Figure 10, the island was under construction in 1988. The project consists of a 17-acre island connected to the northeastern corner of the Marcus Amphitheater grounds by a concrete causeway. A dock space will be provided on the northwest end of the island for small boats. Approximately 117,000 tons of revetment stone will be needed to protect the island from wave erosion, It was estimated that 650,000 cubic yards of crushed limestone from the Milwaukee Metropolitan Sewerage District water pollution abatement project will be used for construction of the island. Approximately 300- to 6,000-pound armor stone will be used to contain the island. The total cost of the island project is $4.4 million.

Figure 10

MILWAUKEE METROPOLITAN SEWERAGE DISTRICT ISLAND PARK PROJECT IN THE MILWAUKEE OUTER HARBOR



Source: Milwaukee Metropolitan Sewerage District.

The construction of the island, which began in the spring of 1988, is expected to be completed late in 1989.

20. Milwaukee County Shoreline Reconnaissance: Milwaukee, Wisconsin, Warzyn Engineering, Inc., Milwaukee, Wisconsin;

and Johnson, Johnson and Roy, Inc., Ann Arbor, Michigan, June 1987.

In 1987 the Milwaukee County Department of Public Works undertook a study to assess the overall general condition of the existing Milwaukee County shoreline, and to identify shoreline areas which might benefit from the use of 2.3 million cubic yards of tunnel boring machine spoils being produced under the Milwaukee Metropolitan Sewerage District water pollution abatement project. Several alternative methods of containing the tunnel spoils for shore protection were evaluated, including revetments, offshore breakwaters, armored headlands, groins, and seawalls. The cost of containing the tunnel spoils was estimated to vary from $350 to $1,500 per lineal foot of shoreline. Publicly owned facilities and land, particularly county parklands, were selected as potential sites for the lakefill projects. Nineteen individual projects which could use the tunnel spoils to enhance shore protection were identified. A brief description of each of the proposed lakefill projects was presented, along with the existing shoreline conditions, bluff stabilization and shore protection needs, and estimated quantity of fill material required. The study also set forth guidelines and a schedule to be used in the planning, permitting, design, and construction of the lakefill projects. In 1987 and 1988, tunnel boring machine spoils were being utilized for two shoreline protection projects: the McKinley Beach erosion control project in the City of Milwaukee and the Klode Park erosion control project in the Village of Whitefish Bay.

21. Conceptual Plans, Milwaukee Shoreline Protection, Milwaukee, Wisconsin, STS Consultants, Ltd., August 1987.

At the request of the Milwaukee County Department of Parks, Recreation and Culture, a study was conducted in 1987 to develop conceptual shoreline improvement plans for the Lake Michigan shoreline area adjacent to North Lincoln Memorial Drive in the City of Milwaukee extending from Bradford Beach northward to the Linnwood Avenue water treatment plant. As part of the inventory work, a bathymetric survey, consisting of five transects, each 2,000 feet long, was conducted. The physical characteristics of the shoreline, including approximate slopes, existing shore protection structures, and soil types, were described. Coastal engineering analysis was performed to quantify critical coastal parameters which affect shoreline erosion and shore protection measures. This analysis included such parameters as water level-frequency relationships, storm surge and wave conditions, and regional littoral drift patterns. Stone source investigations identified available materials for shoreline improvements. Alternative types of shore protection measures were reviewed and three alternative shoreline protection concepts were developed: 1) a "do nothing" alternative; 2) a strategic shore protection plan to improve critical areas in a prioritized fashion; and 3) a lake fill plan which includes an area of reclaimed shoreline. A prioritization and implementation scheme was also developed to provide the County with the ability to implement shoreline improvements in phases.

22. A Water Resources Management Plan for the Milwaukee Harbor Estuary, Southeastern Wisconsin Regional Planning Commission, Planning Report No. 37, Volume Two, December 1987.

In 1982 the Regional Planning Commission undertook a comprehensive study of the water pollution, flooding, storm damage, and dredging problems of the Milwaukee Harbor estuary area. The study was funded by the Milwaukee Metropolitan Sewerage District, the U. S. Environmental Protection Agency through the Wisconsin Department of Natural Resources, and the U. S. Department of the Interior, Geological Survey. One of the objectives of the study was to design control measures to abate storm damage problems in the Milwaukee Harbor, including shoreline protection measures, in order to ensure safe navigation and anchorage facilities. The study presented data on existing land use, climate, topography, near-shore bathymetry, Lake Michigan water levels, existing shore protection structures, the effects of wind and wave action on structures located within the harbor area, and the effects of ice action on the McKinley Marina.

The study concluded that except for routine maintenance, the outer harbor breakwaters do not need to be substantially modified at this time. It was recommended that revetments, bulkheads, and dockwalls continue to be constructed and maintained in order to protect facilities within the outer harbor. With regard to the McKinley Marina, two alternatives were considered to abate ice damage problems: 1) melting of ice in the entire anchorage area by diffused compressed air; and 2) melting of ice by diffused compressed air near the piers, with retention of ice floes by an air screen. The study recommended that a pilot application of the diffused air system be constructed over a few winters to provide information for detailed design and construction. It was also recommended that a contingency plan to deal with flooding and high groundwater problems, should Lake Michigan be in a long-term rising trend, be prepared. In 1987 the Regional Planning Commission prepared a prospectus for such a study.

23. Plan of Study Concerning the Reference on Fluctuating Water Levels in the Great Lakes-St. Lawrence River Basin, International Joint Commission, March 15, 1988.

    The governments of the United States and Canada, in August 1986, requested that the International Joint Commission undertake a comprehensive study of methods of alleviating the adverse impacts of changing water levels, ranging from very high to very low levels, on the Great Lakes/St. Lawrence River Basin. The study involves two phases. The first phase of the study, scheduled for completion in May 1989, considers short-term alternatives—not involving major structural improvements—to minimize the adverse impacts of fluctuating water levels. The first phase is to include a characterization of water level fluctuations and their environmental, social, and economic consequences.

    The second phase, which is scheduled to be completed in September 1991, is to include a comprehensive evaluation of potential solutions, including structural improvements, land use planning, and other management activities. In this regard, it should be noted that the governors of the Great Lakes states, as members of the Council of Great Lakes Governors, in 1986 voiced support for avoiding the further diversion of water from the Great Lakes. This concern will have to considered in any study of the potential regulation of Lake Michigan.

24. A Lake Michigan Shoreline Erosion Management Plan for Northern Milwaukee County, Wisconsin, Southeastern Wisconsin Regional Planning Commission, Community Assistance Planning Report No. 155, December 1988.

    A comprehensive shoreline erosion and bluff recession management study was completed by the Regional Planning Commission in 1988 for that portion of Milwaukee County extending from the City of Milwaukee Linnwood Avenue water treatment plant northerly through the Villages of Shorewood, Whitefish Bay, and Fox Point to Doctors Park. As part of that study, large-scale topographic maps were prepared, and detailed field inventories and bluff stability analyses conducted in order to identify and recommend erosion control measures needed to stabilize the bluff slopes and protect the shoreline from wave and ice erosion on a long-term basis. Shoreline erosion problems were described and evaluated for the entire reach studied, including, as shown in Figure 11, for Atwater, Big Bay, Buckley, and Klode Parks. The findings and recommendations of the northern Milwaukee County study were fully incorporated into this study.

Summary of Previous Studies

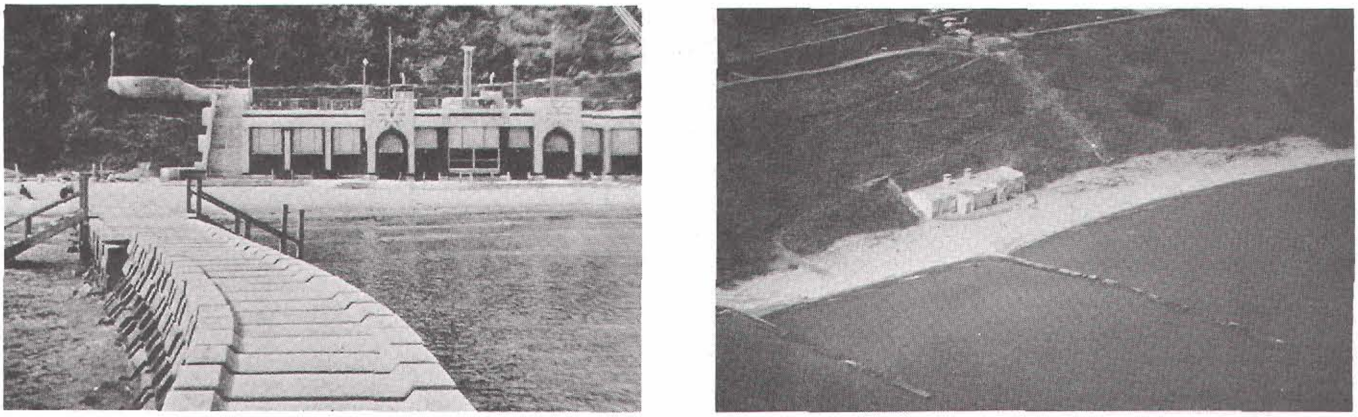
This chapter briefly summarized the results of 24 major shore erosion studies previously conducted within Milwaukee County. These studies, conducted between 1934 and 1988, have addressed the stratigraphy and groundwater conditions within the bluffs, bluff recession rates, the causes of shoreline erosion and bluff slope failure, wave conditions and littoral drift, and the adequacy of existing shore protection structures. Several studies presented recommendations for the protection and management of the County's Lake Michigan shoreline. However, many of the previous plan recommendations have not been implemented. In general, the bluff analyses and inventory efforts have not been conducted at a sufficient level of detail to

Figure 11

SHORELINE CONDITIONS AT ATWATER AND BIG BAY PARKS IN NORTHERN MILWAUKEE COUNTY

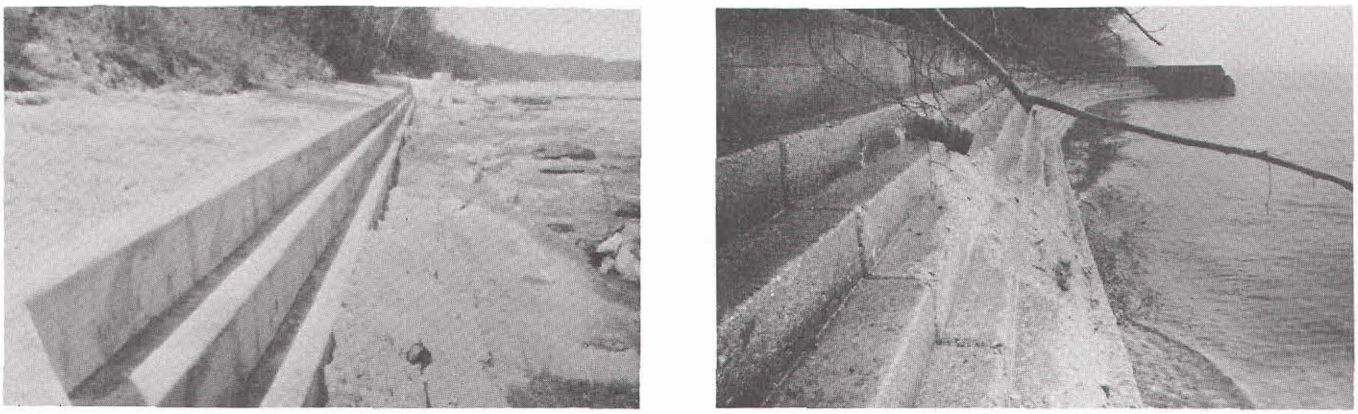
ATWATER PARK, VILLAGE OF SHOREWOOD

1939 | 1986



BIG BAY PARK, VILLAGE OF WHITEFISH BAY

1944 | 1986



These parks in northern Milwaukee County, which have been protected by shore protection structures since the 1930's, all were damaged by shoreline erosion during the high-water period of 1985 through 1987. The 400- to 500-foot-long permeable groins at Atwater Park, built in 1932, have deteriorated, resulting in severe erosion of the beach. The Atwater Park beach house was demolished in 1987. Concrete bulkheads at Buckley Park, Big Bay Park, and Klode Park, all built in 1943, were damaged by scouring and overtopping. The Big Bay Park bulkhead is subject to overtopping during high-lake-level periods, and the 1986 photograph shows the scouring effects frequently caused by waves deflecting off bulkheads. The scouring at the base of the structure has exposed at least three additional concrete steps which were previously buried by sand.

Photo (upper left) from American Guide Series—Shorewood, by the Village of Shorewood, 1939.
Photo (upper right) by SEWRPC.
Photo (lower left) from Erosion, by Miriam Bird, courtesy of the Whitefish Bay Public Library.
Photo (lower right) by SEWRPC.

identify those measures needed to stabilize the bluff in most areas. Insufficient data also exist on coastal wave conditions, on the effectiveness and long-range impacts of existing shore protection structures, and on appropriate methods of implementation and sources of funding.

SHORELINE EROSION MANAGEMENT STUDY AREA

The Milwaukee County study area consists of the 12.5 square miles of land adjoining the Lake Michigan shoreline from the Wisconsin Electric

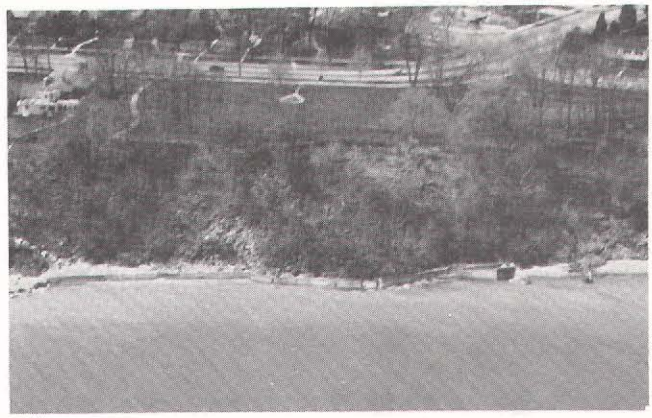
Figure 11 (continued)

SHORELINE CONDITIONS AT BUCKLEY AND KLODE PARKS IN NORTHERN MILWAUKEE COUNTY

BUCKLEY PARK, VILLAGE OF WHITEFISH BAY

1976

1987



KLODE PARK, VILLAGE OF WHITEFISH BAY

1944

1987

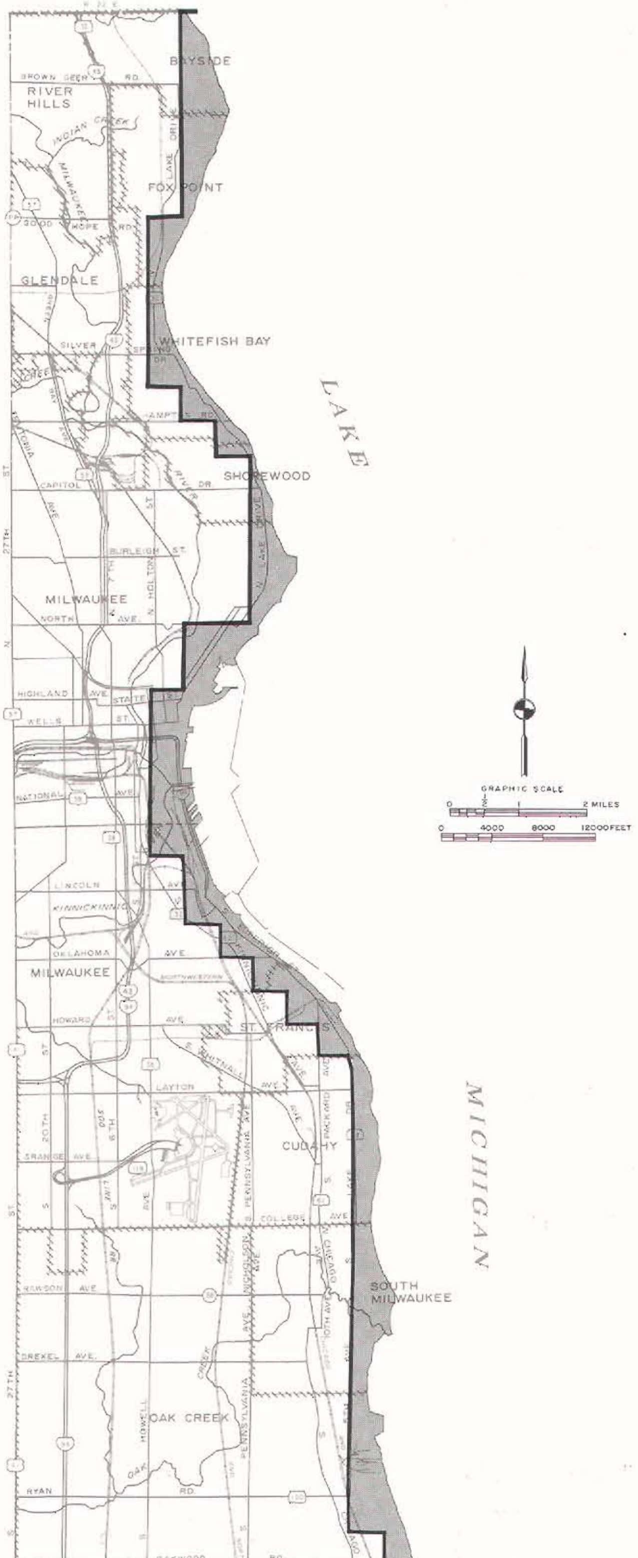
The Buckley Park bulkhead collapsed in November 1986, as the bluff slope above the bulkhead failed. A portion of the Klode Park bulkhead collapsed in December 1986, and the bluff behind the bulkhead then failed in April 1987, as shown in the May 1987 photograph.

Photo (top left) courtesy of the Wisconsin Coastal Management Program.
Photo (top right) by SEWRPC.
Photo (lower left) from Erosion, by Miriam Bird, courtesy of the Whitefish Bay Public Library.
Photo (lower right) by SEWRPC.

Power Company's Oak Creek electric power generating facility at the Milwaukee-Racine County line northerly through the Cities of Oak Creek, South Milwaukee, Cudahy, St. Francis, and Milwaukee; and the Villages of Shorewood, Whitefish Bay, Fox Point, and Bayside to the Milwaukee-Ozaukee County line, as shown on Map 1. The total study area contains about 30 miles of Lake Michigan shoreline. The study area thus consists of that portion of Milwaukee County that directly affects, or is directly affected by, shoreline erosion, bluff recession,

Map 1

LAKE MICHIGAN SHORELINE EROSION MANAGEMENT STUDY AREA FOR MILWAUKEE COUNTY



Source: SEWRPC.

and storm damage processes. Although this study focuses on a relatively narrow strip of land along the Lake Michigan shoreline, it is recognized that the Lake Michigan coastal area provides a unique setting for high-value development and recreational opportunities which attract users from throughout the greater Milwaukee area. Due consideration was given in this study to these and other important linkages between the study area and the balance of the greater Milwaukee area.

PURPOSE AND SCOPE

The primary purposes of the Milwaukee County shoreline erosion, bluff recession, and storm damage management study are to define the existing erosion problems and the risk of property damage along the Lake Michigan shoreline; to explore alternative and to recommend effective, economically feasible, and environmentally acceptable measures for erosion, bluff recession, and storm damage control; and to identify the implementation program and local regulations needed to successfully carry out the recommended plan. Important objectives of the study were to evaluate the impacts of erosion control measures on the natural resource base and on the erosion of adjacent shoreline areas, and to develop a recommended plan which minimizes any potential adverse impacts on the environment.

The degree of shoreline erosion and the effectiveness of erosion control measures are highly site-specific and may vary over time. Factors such as Lake Michigan water levels, nearby erosion control measures, and wind and wave characteristics contribute to, and complicate, this variability. The process used to prepare the shoreline management plan herein presented constitutes the first, or systems planning, phase of what may be regarded as a three-phase shore protection development process. Preliminary engineering is the second phase in this sequential process, with final design being the third and last phase. Systems planning concentrates on the definition and description of the erosion problems to be addressed; the development and evaluation of the types of alternative control measures needed for the resolution of those problems; and the provision of guidelines and general information which should be applied and followed in the subsequent preliminary engineering and final design of erosion control measures.

The following specific work elements were undertaken as part of the study:

1. The collation, interpretation, and presentation of all pertinent data relating to shoreline erosion, bluff recession, and storm damage in the study area and to the characteristics of the natural resource base which affect shoreline management.

2. The preparation of one inch equals 100 feet scale topographic maps, with attendant monumented horizontal and vertical survey control to provide essential topographic and cadastral data, and to help determine the need and the design parameters for both structural and nonstructural shore protection measures for the portion of the study area that lies within the Village of Bayside. Large-scale topographic maps prepared to Regional Planning Commission specifications were available for the remainder of the study area; however, an updating of the City of Oak Creek topographic maps, prepared in 1977, was provided under the study. These maps provide an invaluable, permanent base of benchmark information about the topography of the coastal area.

3. The identification of high erosion risk areas, and the determination of shoreline recession rates and areas of impact.

4. The assessment of the effectiveness of existing shore protection structures under various storm wave and lake level conditions, and of the stability of the bluff slopes.

5. The development and evaluation of alternative shore protection measures based upon the inventory and erosion hazard data, including both nonstructural and structural measures to reduce damages from shoreline erosion and bluff recession.

6. The recommendation of specific types of nonstructural and structural erosion, bluff recession, and storm damage control measures, as well as an implementation program to carry out the plan.

SUMMARY

In January 1986, Milwaukee County requested that the Regional Planning Commission assist the County in defining and seeking solutions to the severe and costly shoreline erosion, bluff recession, and storm damage problems existing along the 30-mile reach of Lake Michigan shoreline within the County. The requested study was undertaken with financial assistance obtained by the Commission from the Wisconsin Coastal Management Program. The study was conducted under the guidance of an intergovernmental and citizens advisory committee created by the Commission to assist in the work.

Shoreline erosion, bluff recession, and storm damage management is herein defined as a coordinated set of measures designed to abate shoreline erosion, bluff recession, and storm damage, reducing attendant property losses, aesthetic impacts, and risks to human safety. Average erosion and recession rates within the study area range up to almost 13 feet per year, with the loss of about 330,000 cubic yards of shore material per year. Information needs addressed in the study included the need for more adequate knowledge about specific conditions and processes which contribute to shoreline erosion and bluff recession; about the effectiveness of existing shore protection structures and harbor facilities in preventing shoreline erosion, bluff recession, and storm damage under various storm wave and water level conditions; about the effects of various types of shore protection measures on adjacent shoreline areas and on the offshore coastal environment; and about the roles of the County and the local units of government in protecting the shoreline. The primary purposes of the study are to define the risk of shoreline erosion, bluff recession, and storm damage; to evaluate the effectiveness of existing shore protection measures and the stability of the bluff slopes; to explore alternative erosion control and bluff recession measures; to recommend an erosion and bluff recession control plan; and to identify an appropriate implementation program to carry out the plan.

The study area consists of the entire Lake Michigan shoreline of Milwaukee County and the Milwaukee Outer Harbor area, including the shorelines in the Cities of Oak Creek, South Milwaukee, Cudahy, St. Francis, and Milwaukee and the Villages of Shorewood, Whitefish Bay, Fox Point, and Bayside. The study area encompasses about 30 miles of shoreline and about 12.5 square miles of land.

A number of studies of the Lake Michigan shoreline erosion and bluff recession problems have been made over the past half century. Many of these studies were initiated in response to high lake levels and associated increased erosion damages such as occurred in the mid-1970's, and more recently in 1985 and 1986. These studies have provided an historical record of changing shoreline conditions; insight into the effectiveness and life of certain shore protection structures; a perspective on the types of erosion control measures which have historically been desired; and an improved understanding of bluff and shoreline geological conditions and coastal processes.

The results and findings of 24 studies of shoreline erosion or bluff recession in Milwaukee County are summarized in this chapter. The most extensive inventories have been conducted on the stratigraphy and/or groundwater conditions within the coastal bluffs. Bluff conditions have been evaluated for two specific projects in the Village of Whitefish Bay (Whitney, 1936; Painter, 1973); for the reach of shoreline extending from Grant Park to Bender Park (Hadley, 1974; Nelson & Associates, Inc., et al., 1980); for the City of St. Francis shoreline (SEWRPC, 1984; Pittsburgh Testing Laboratory, 1985); for the northern Milwaukee County shoreline (SEWRPC, 1988); and for the entire county shoreline (Mickelson, et al., 1977). Bluff recession rates, which range from an average of less than one-half foot per year to a maximum of more than 60 feet per year, were presented in eight studies (Milwaukee County Committee on Lake Michigan Shore Erosion, 1945; U. S. Army Corps of Engineers, 1945 and 1975a; City of Oak Creek, 1975; Mickelson et al., 1977; Nelson & Associates, Inc., et al., 1980; SEWRPC, 1984 and 1988). These studies found that the bluffs are generally composed of relatively impermeable glacial tills. Permeable lake sediments are often located between these tills. Groundwater seepage within these lake sediments, as well as bluff toe erosion by wave action, is the major cause of bluff slope failure.

Relatively few studies (U. S. Army Corps of Engineers, 1945; McKee, 1983; Warzyn Engineering, Inc., 1986; Milwaukee Metropolitan Sewerage District, 1987; SEWRPC, 1988) have evaluated coastal processes such as wave conditions and littoral drift. For most of the shoreline, the anticipated wave conditions reaching the shore under various storms and water levels have not been quantified. Only rough estimates of the littoral drift rate exist, but it is generally agreed that the littoral drift rate in northern Milwaukee County is less than the drift rate in southern Milwaukee County, primarily because of the presence of exposed, rapidly eroding bluffs in southern Milwaukee County, which feed the littoral transport system. Five studies (Milwaukee County Committee on Lake Michigan Shore Erosion, 1945; U. S. Army Corps of Engineers, 1975a; Mickelson et al., 1977; Warzyn Engineering, Inc., et al., 1987; SEWRPC, 1988) have inventoried the effectiveness of existing shore protection structures and the types of structural failure occurring.

Several studies have presented recommended plans to protect the shoreline. The extension of Lincoln Memorial Drive northward from Lake Park to the Village of Fox Point on either an onshore or offshore landfill was recommended by four studies (Milwaukee County Regional Planning Department, 1934; Milwaukee County Committee on Lake Michigan Shore Erosion, 1945; U. S. Army Corps of Engineers, 1945 and 1975a). Six studies recommended using revetments to protect the shoreline (Milwaukee County Committee on Lake Michigan Shore Erosion, 1945; U. S. Army Corps of Engineers, 1945 and 1975a; Painter, 1973; Nelson & Associates, Inc., et al., 1980; SEWRPC, 1988). Groin systems were recommended to contain public beaches at Doctors Park, Grant Park, and/or Sheridan Park (Milwaukee County Committee on Lake Michigan Shore Erosion, 1945; U. S. Army Corps of Engineers, 1945 and 1975a) and to provide beaches along some residential areas, such as in the Village of Fox Point north of E. Green Tree Road (Milwaukee County Committee on Lake Michigan Shore Erosion, 1945; U. S. Army Corps of Engineers, 1945 and 1975a). Four studies (U. S. Army Corps of Engineers 1945 and 1975a; SEWRPC, 1984 and 1988) recognized that any new groin systems in Milwaukee County would likely need to be artificially nourished with sand or gravel. To meet an anticipated demand for new boating facilities, new harbors or marinas were recommended by three studies (Burke, 1958; City of Oak Creek, 1975; U. S. Army Corps of Engineers, 1975b). Three studies made recommendations to protect boating facilities or navigation channels (McKee, 1983; SEWRPC, 1986; SEWRPC, 1987).

In summary, the previously conducted studies provide useful information on bluff and shoreline conditions at specific sites within Milwaukee County over the past 50 years. However, while many studies described the bluff and shoreline conditions, the bluff inventory efforts and analyses have not been conducted at a sufficient level of detail to identify the measures needed to stabilize the bluffs for most sections of the County's shoreline. Insufficient data also exist on coastal wave conditions, on the effectiveness of existing shore protection structures, and on the long-range impacts of those structures on adjacent shoreline areas and on the offshore coastal environment. Implementation programs—consisting of designated implementing agencies and private entities, appropriate institutional mechanisms, and potential sources of funding—which are necessary to carry out coordinated shore protection projects for entire sections of shoreline have not been developed. In part because of these limitations, many of the previous plan recommendations have not been implemented, and there is a continued need for improved coordination of shore protection activities, and for guidelines and procedures to help lakefront property owners protect their shoreline.

During the past decade, coastal engineers and scientists have made a great deal of progress in better understanding the hydraulic and geological processes affecting the shoreline. Perhaps the most significant progress relates to a recognition of the potential for higher lake levels and the impact of those levels on the shoreline. Another important accomplishment is the realization of the beneficial and adverse impacts of shore protection measures on the coast, although further documentation and quantification of these impacts is needed. Locally, much experience and knowledge has been gained on the use of fill material to stabilize bluff slopes.

This progress in understanding the shoreline has generated a corresponding change in professional, and to lesser extent public, attitudes toward protecting the shore. There is a declining tendency to utilize large revetments and bulkheads to armor the shore, and a growing tendency to utilize beaches and associated beach containment systems which work in harmony with natural coastal processes, and which provide a usable, accessible shoreline for recreational activities under varying lake level conditions. Recent beach restoration projects include the McKinley Beach project developed by Milwaukee County and the Klode Park project implemented by the Village of Whitefish Bay. There is also a growing awareness that the effectiveness of shore protection measures can usually be enhanced by implementing projects within relatively large sections of shoreline.

Building on this increased understanding of shoreline processes and on the changing approaches to protecting the shoreline, this shoreline erosion management study includes the collection of shoreline-related inventory data on a systems level for designated sections of shoreline, and the application of state-of-the-art analytic techniques to properly evaluate the shoreline problems and to help identify the control measures needed to both stabilize the bluff slopes and protect the shoreline from wave erosion and storm damage. The recommended plan set forth in this report is intended to provide the means for obtaining long-term fundamental protection which conforms with natural coastal processes, which enhances the usability of the shoreline, and which lessens the damages and impacts of each storm, thereby reducing the need for short-term emergency responses. An implementation program is presented to carry out the plan in an efficient and orderly manner.

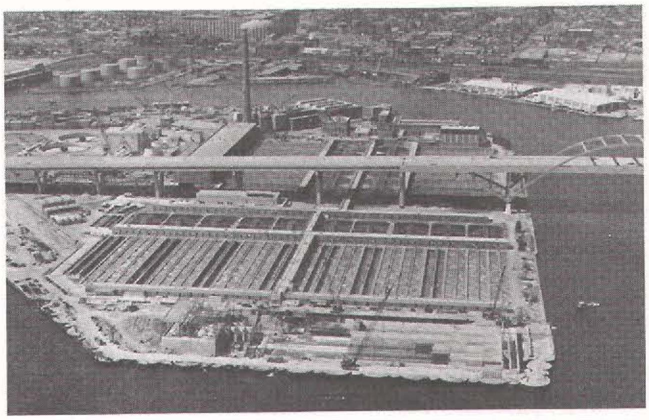
The Lake Michigan shoreline has an enormous value to the economy of, and to the quality of life within, Milwaukee County that warrants enhancement beyond simple repair and restoration. Since the turn of the century, the public and private lakefront property owners have carefully built and protected the lakefront, with shore protection structures currently covering about 65 percent of the county shoreline. Major public facilities such as the Jones Island sewage treatment plant, shown in Figure 12, have been built on the lakefront. About 12 lineal miles of lakefront park and open land, such as Juneau Park shown in Figure 13, are publicly owned, dedicated to public access and use, and available to the residents of the County. The beauty and amenities of the public park system along the shoreline have attracted people and businesses to the area along with tourists and conventions. The public beaches, lakefront facilities, and Milwaukee Harbor constitute a public recrea-

Figure 12

JONES ISLAND IN THE MILWAUKEE OUTER HARBOR: EARLY 1920'S AND 1987

EARLY 1920'S FISHING VILLAGE

1987—MILWAUKEE METROPOLITAN SEWERAGE DISTRICT SEWAGE TREATMENT PLANT



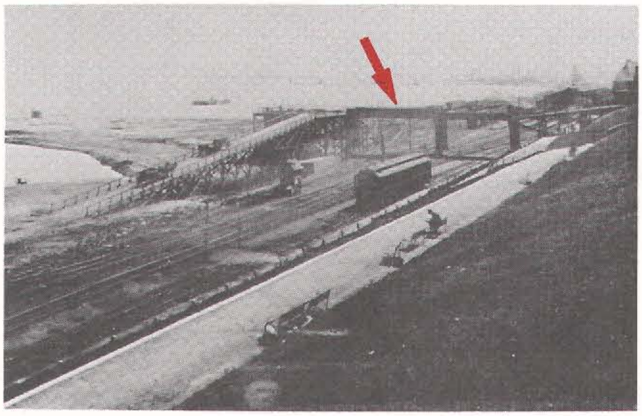
Prior to the construction of the Jones Island sewage treatment plant in 1925, Jones Island was a thriving community of fishermen and their families. The fishing village was removed from Jones Island in the early 1920's to allow construction of the treatment plant. The plant, the first in the United States to use the air-activated sludge process on a large-scale basis, underwent major expansions in 1935, 1952, and 1987, as shown in the photograph on the right.

Photo (left) courtesy of the Milwaukee County Historical Society.
Photo (right) by SEWRPC.

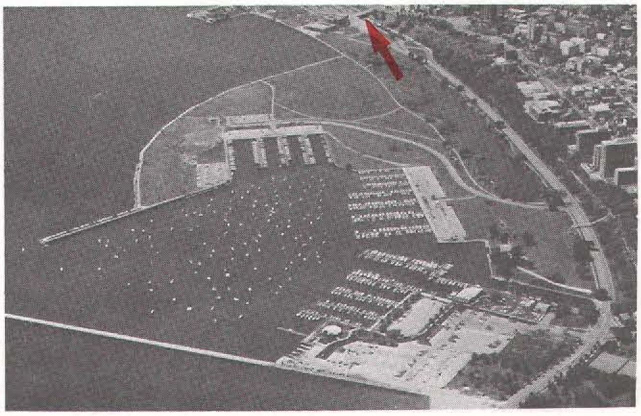
Figure 13

LANDFILL AT JUNEAU PARK: 1917 AND 1987

1917



1987



Landfills have been used to expand Juneau Park, to construct Lincoln Memorial Drive, and to develop the McKinley Marina, as shown in the 1987 photograph. The filling was in progress in 1917, with the ramp in the 1917 photograph being located at the present site of the War Memorial Bridge (see arrows). The fill material is protected and contained by concrete and steel sheet pile bulkheads.

Photo (left) courtesy of the Milwaukee Public Museum.
Photo (right) by SEWRPC.

tional resource for the nearly one million people in the Milwaukee area, enhancing their quality of life. The critical need to solve the erosion and bluff recession problems confronting the residents of Milwaukee County also provides the opportunity to build on past shore protection efforts and to develop a well-managed accessible and usable lakefront serving both lakefront property owners and the population of the County as a whole.

Chapter II

INVENTORY FINDINGS

INTRODUCTION

In order to identify and evaluate alternative structural and nonstructural shoreline protection measures, high-risk erosion areas must be identified, and careful consideration must be given to such factors as the existing land use pattern, the natural resource base, the coastal erosion processes and rates, and existing structural protection measures. Accordingly, this chapter describes the Lake Michigan shoreland study area, providing pertinent information on the elements of the natural resource base relevant to coastal erosion; on the existing land use and land use control patterns; and on the types, causes, and rates of shoreline erosion and bluff recession occurring within the coastal area of Milwaukee County.

The study area, as defined in Chapter I and shown on Map 1 of that chapter, includes that portion of Milwaukee County which most directly affects, and is most affected by, Lake Michigan shoreline erosion. Appendix A presents oblique aerial photographs of the Lake Michigan shoreline in Milwaukee County taken in April 1987. Certain of the data presented herein, including data on bluff characteristics, groundwater resources, types and causes of bluff erosion, and existing structural erosion control measures, were collected through special surveys conducted by consultants working under contract to the Regional Planning Commission. Other inventory data, such as data on the geology and climate of the area, were collated from Commission files. Detailed information on topographic and cultural features was provided by new 1 inch equals 100 feet scale, two-foot contour interval topographic maps prepared for the Village of Bayside shoreline, and by updated topographic maps prepared for the City of Oak Creek shoreline. These new and updated maps, which are based upon a monumented, high-precision, high-density horizontal and vertical control survey network, were prepared to Commission specifications by private photogrammetric engineers, working under contract to the Commission. Some of the inventory data, such as data on existing land use and soils, are presented for the entire study area. Other inventory data, particularly data on coastal erosion processes, rates, and problems and existing structural shore protection measures, are more site-specific, being for individual sections of the immediate shoreland area.

This chapter consists of seven sections. The first section describes the natural resource base pertinent to coastal erosion management. The second section describes the historical development of the shoreline and the existing land use pattern, including information on zoning and related regulations. The third section describes coastal erosion processes. The fourth section concerns existing regulations—other than zoning—relating to shoreland development. Existing shore protection structures are described in the fifth section, and the sixth section identifies the coastal erosion problems of the area. The seventh and final section presents data on historical shoreline recession rates.

NATURAL RESOURCE BASE

This section describes those aspects of the natural resource base which affect, or may be affected by, coastal erosion management. Data are presented on the bedrock geology and glacial deposits, soils, beach and bluff characteristics, groundwater resources, climate, and ecological resources of the shoreland and related areas.

Bedrock Geology and Glacial Deposits

The consolidated bedrock underlying Milwaukee County generally dips eastward at a rate of 25 to 30 feet per mile. Precambrian Age crystalline rock formations constitute the basement of the bedrock and are thousands of feet thick. Cambrian sandstone rock formations imbedded with siltstone and shale lie above the crystalline rock formations and are more than 800 feet thick. Above the Cambrian rock formations lie Ordovician sandstone, dolomite, and shale formations whose thickness approximates 700 feet. The uppermost bedrock throughout most of the County is Silurian Age dolomite, primarily Niagara dolomite, which lies above the Ordovician rock formations, and is approximately 300 feet thick. In northeastern Milwaukee County, the bedrock closest to the surface is composed of Devonian Age dolomite and shale of the Milwaukee Formation, which is approximately 100 feet thick in the northern portion of the study area. The Silurian and Devonian Formations are generally covered by glacial deposits whose thickness ranges from less than 50 feet along the shoreline in the Villages of Fox Point and Bayside, to more than 200 feet in the City of Milwaukee. The bedrock outcrops at the surface on the Lake Michigan beach in the 6800 block of N. Barnett Lane in the Village of Fox Point. Map 2 indicates the spatial variation of the thickness of the unconsolidated deposits overlying the bedrock in Milwaukee County.

Materials directly deposited by glacial ice are called till. The Milwaukee County study area is overlain by till believed to have been deposited by ice of the Lake Michigan lobe during the Wisconsinan stage of glaciation. Several layers of glacial debris can be identified in the study area. The Zenda Formation, whose maximum thickness is unknown at this time, is the oldest glacial deposit located above the lakebed within the study area. The Zenda Formation is believed to have been deposited between 18,000 and 20,000 years ago. The upper layer of the Zenda Formation is known as the Tiskilwa Member. Tiskilwa till is medium textured, slightly to moderately stony, and pink in color. Directly above the Zenda Formation lies a layer known as the New Berlin Formation, which ranges in thickness up to 70 feet and consists of a lower sand and gravel member and an upper till member. The gravel member is believed to have been deposited between 14,000 and 16,000 years ago as an outwash plain in front of and around the advancing Delavan sublobe of the Lake Michigan lobe. The New Berlin till is a coarser grained till, sandy in texture and dominated by pebbles, cobbles, and some boulders. The Oak Creek Formation, whose maximum thickness ranges up to 115 feet, lies above the New Berlin Formation. This formation is believed to have been deposited between 13,000 and 14,000 years ago, when the Lake Michigan lobe moved southwestward out of the current Lake Michigan basin. During brief periods of glacial recession, lacustrine sediment was deposited. The Oak Creek Formation is composed of a pebbly, silty clay loam till; lacustrine clay, silt, and sand; and glaciofluvial sand and gravel. The layer nearest the surface, and generally less than 100 feet thick, is known as the Ozaukee Member of the Kewaunee Formation. The Ozaukee Member is believed to have been deposited 12,500 to 13,000 years ago. The till of the Ozaukee Member is fine-grained, typically silty clay or silty clay loam, and red in color.

All four glacial formations are exposed by the bluffs within the study area. Within the exposed bluffs, the Zenda Formation ranges up to 45 feet in thickness, the New Berlin Formation ranges up to 10 feet in thickness, the Oak Creek Formation ranges up to 80 feet in thickness, and the Kewaunee Formation ranges up to 75 feet in thickness. The properties of these glacial deposits influence the resistance of the bluffs to processes such as wave erosion, and ultimately affect the severity and rate of bluff recession.

Soils

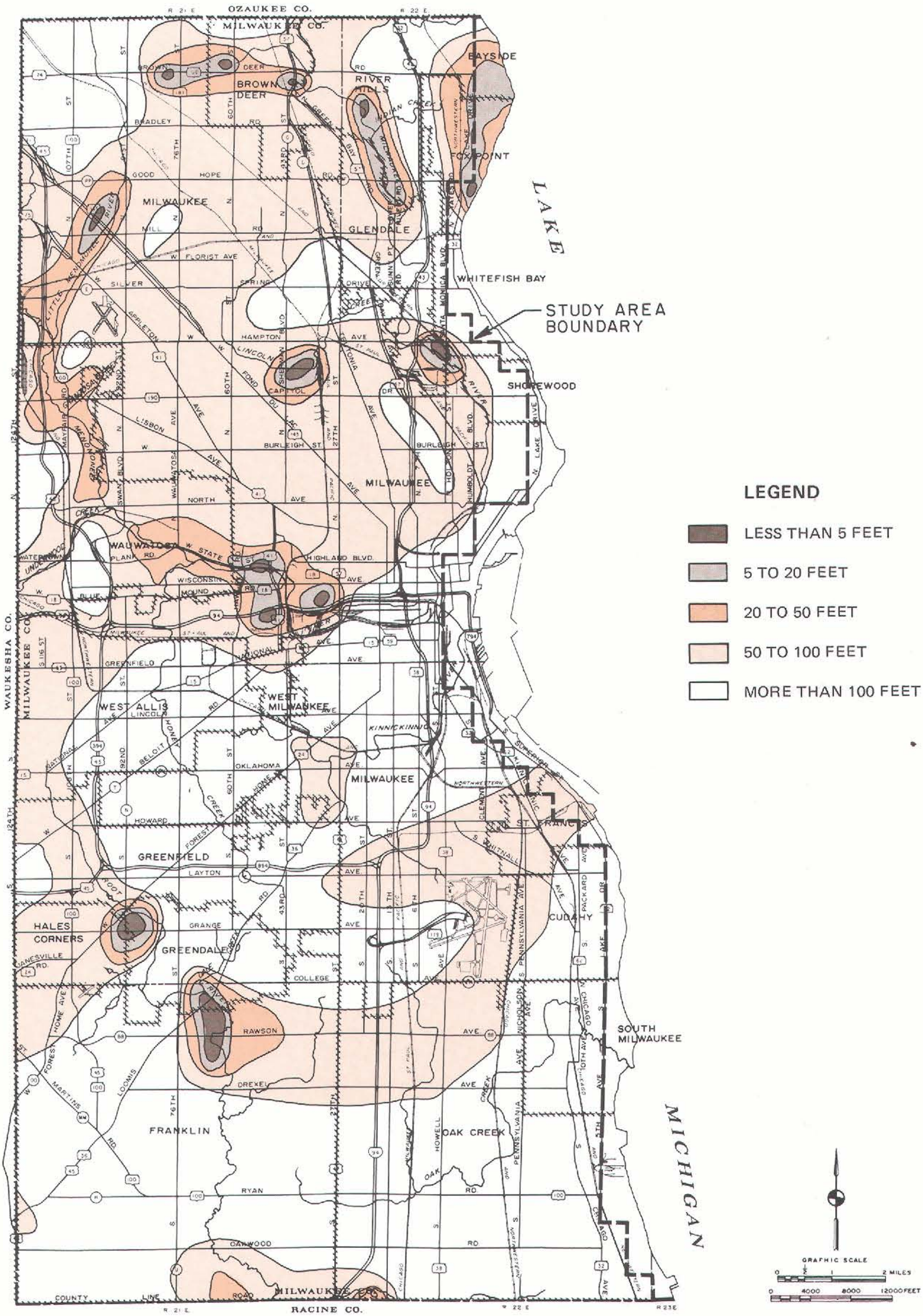
Soil properties influence the rate and amount of stormwater runoff, thereby affecting the severity of surface erosion on the face, and at the top, of the bluffs. Soil properties also are an important consideration in the evaluation of shallow groundwater seepage from the bluff area. The type of vegetative cover which can be supported along the shoreline is also greatly influenced by soil properties.

In order to assess the significance of the diverse soils found in southeastern Wisconsin, the Regional Planning Commission, in 1963, negotiated a cooperative agreement with the U. S. Soil Conservation Service under which detailed soil surveys were completed for the entire Planning Region except those areas intensively developed for urban use. The findings of the soil surveys have been published in SEWRPC Planning Report No. 8, Soils of Southeastern Wisconsin, 1966. The surveys provide data on the physical, chemical, and biological properties of the mapped soils; and, more importantly, provide interpretations of the soil properties for planning, engineering, agricultural, and resource conservation purposes.

As shown on Map 3, the detailed soils mapping for the study area was conducted within the then undeveloped areas of the Cities of Cudahy, Oak Creek, and South Milwaukee, and the Villages of Bayside and Fox Point. Detailed soils mapping was not conducted in the portion of the study area within the Cities of Milwaukee and St. Francis and within the Villages of Shorewood and Whitefish Bay because the portions of the study area within these communities were generally urbanized at the time of the soil surveys. In these areas the natural soils were

Map 2

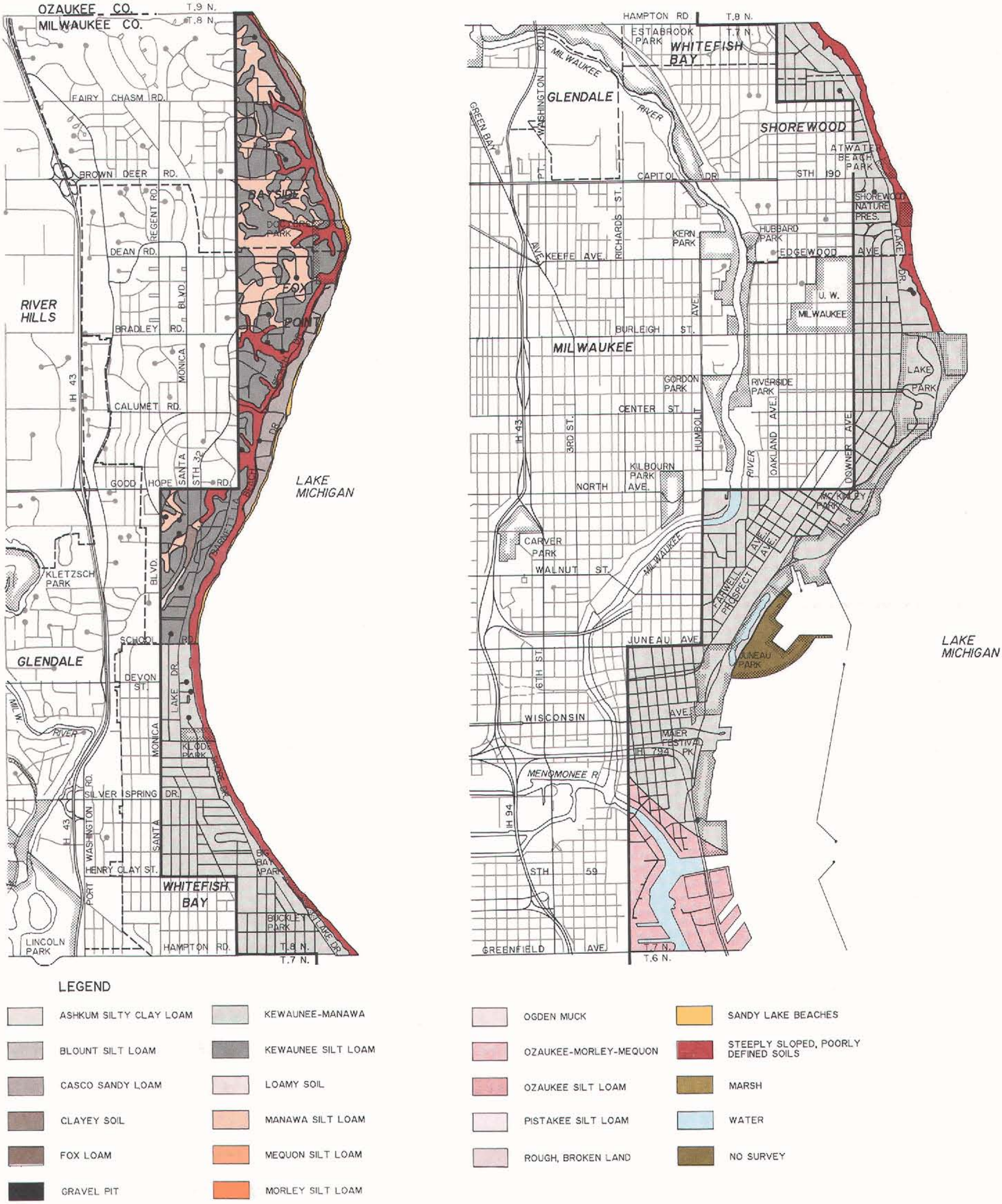
THICKNESS OF UNCONSOLIDATED MATERIALS IN MILWAUKEE COUNTY



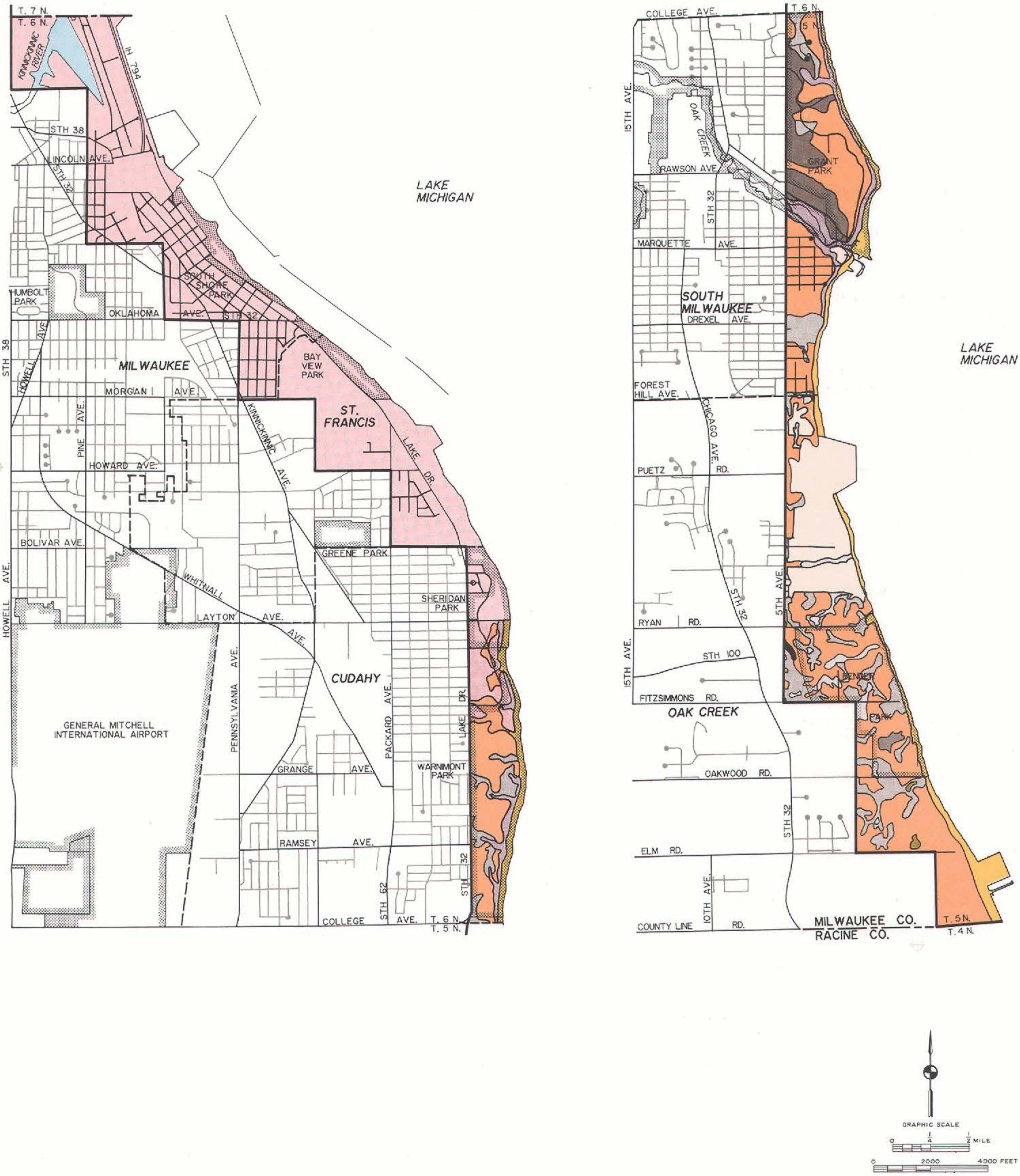
Source: U. S. Geological Survey and SEWRPC.

Map 3

SOILS WITHIN THE LAKE MICHIGAN SHORELINE OF MILWAUKEE COUNTY



Map 3 (continued)



Source: SEWRPC.

greatly disturbed and the soil boundaries generally could not be recognized and delineated. The general soil association group identified for these areas by the U. S. Department of Agriculture, Soil Conservation Service, must, therefore be used to evaluate soil conditions at the systems level of planning.

With respect to bluff erosion caused by surface stormwater runoff, the most significant soil interpretation is the categorization of soils into four hydrologic soil groups: A, B, C, and D. The characteristics of soils within a particular hydrologic soil group are often used by engineers to determine the amount and rate of stormwater runoff. For example, known hydrologic soil group characteristics are often used to calculate the coefficient of runoff for the Rational Method, which is one of the most common methods used in the Region to calculate the rate of stormwater runoff for storm sewer design. The coefficient of runoff is the ratio between the maximum rate of runoff and the average rate of rainfall on a given area during the time required for water to flow from the remotest part of the drainage area to the point being considered for design. The coefficient of runoff is primarily a function of the slope of the land surface, soil permeability, and land use. In terms of runoff characteristics, the four hydrologic soil groups are defined as follows:

1. Hydrologic Soil Group A: Very little runoff because of high filtration capacity, high permeability, and good drainage. In the Rational Method, a coefficient of runoff ranging from 0.09 to 0.14 is typically used for lawns and other pervious surfaces on Hydrologic Soil Group A soils with a slope of 2 to 6 percent.[1]

2. Hydrologic Soil Group B: Moderate amounts of runoff because of moderate infiltration capacity, moderate permeability, and good drainage. In the Rational Method, a coefficient of runoff ranging from 0.12 to 0.17 is typically used for lawns and other pervious surfaces on Hydrologic Soil Group B soils with a slope of 2 to 6 percent.

3. Hydrologic Soil Group C: Large amounts of runoff because of low infiltration capacity, low permeability, and poor drainage. In the Rational Method, a coefficient of runoff ranging from 0.16 to 0.21 is typically used for lawns and other pervious surfaces on Hydrologic Soil Group C soils with a slope of 2 to 6 percent.

4. Hydrologic Soil Group D: Very large amounts of runoff because of low infiltration capacity, low permeability, and poor drainage. In the Rational Method, a coefficient of runoff ranging from 0.20 to 0.25 is typically used for lawns and other pervious surfaces on Hydrologic Soil Group D soils with a slope of 2 to 6 percent.

As indicated in Table 1, 212 acres, or about 3 percent, of the study area were covered by Hydrologic Soil Group A soils; 1,363 acres, or 18 percent, were covered by Hydrologic Soil Group B soils; 1,593 acres, or 21 percent, were covered by Hydrologic Soil Group C soils; and 58 acres, or 1 percent, were covered by Hydrologic Soil Group D soils. Disturbed soils accounted for 4,064 acres, or 54 percent of the study area. The remaining 225 acres, or 3 percent of the study area, consisted of surface waters. The specific soil types within the study area are shown on Map 3.

The predominant soil type within the southern portion of the study area is Morely silt loam, which covers half of the surveyed area in the southern portion of the study area. Morely soils form in thin loess and silty clay glacial till on moraines, and are slowly permeable. The areas not surveyed in southern Milwaukee County—within the northern portion of the City of Cudahy, the City of St. Francis, and the southern portion of the City of Milwaukee—are covered by soils collectively referred to by the U. S. Soil Conservation Service as the Ozaukee-Morely-Mequon Association.

The predominant soil type within the northern portion of the study area is Kewaunee silt loam, which covers about one-half of the surveyed area in the northern portion of the study area. Kewaunee soils form in thin loess and silty clay glacial till on moraines and in depositional areas, and have a slow permeability. In general, in comparison to Morely soils, the Kewaunee soils form in material of finer texture. The areas not surveyed in northern Milwaukee County—

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[1] *Kurt W. Bauer, "Determination of Runoff for Urban Storm Water Drainage System Design," SEWRPC Technical Record, Vol. 2, No. 4, April-May 1965.*

Table 1

SOIL TYPES IN THE MILWAUKEE COUNTY SHORELINE MANAGEMENT STUDY AREA

	Civil Division																			
	City of Oak Creek		City of South Milwaukee		City of Cudahy		City of St. Francis		City of Milwaukee		Village of Shorewood		Village of Whitefish Bay		Village of Fox Point		Village of Bayside		Total Study Area	
Soil Type	Area (acres)	Percent of Total	Area (acres)	Percent of Total	Area (acres)	Percent of Total	Area (acres)	Percent of Total	Area (acres)	Percent of Total	Area (acres)	Percent of Total	Area (acres)	Percent of Total	Area (acres)	Percent of Total	Area (acres)	Percent of Total	Area (acres)	Percent of Total
Hydrologic Soil Group A																				
Sandy Lake Beaches . . .	87.2	8.0	52.4	8.4	45.7	10.2	- -	- -	- -	- -	- -	- -	- -	- -	12.8	1.9	13.4	2.7	. 211.5	2.8
Hydrologic Soil Group B																				
Casco Sandy Loam	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	80.0	12.0	26.7	5.3	106.7	1.4
Fox Loam	- -	- -	118.5	18.9	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	118.5	1.6
Kewaunee Silt Loam . . .	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	326.7	49.1	245.5	48.8	572.2	7.6
Rough Broken Land	- -	- -	70.5	11.3	31.8	7.1	- -	- -	- -	- -	- -	- -	- -	- -	138.0	20.8	71.5	14.2	311.8	-4.2
Loamy Sand	242.8	22.2	5.6	0.9	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	5.0	0.7	- -	- -	253.4	3.4
Hydrologic Soil Group C																				
Blount Silt Loam	152.3	13.9	56.7	9.0	37.1	8.3	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	246.1	3.3
Manawa Silt Loam	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	93.5	14.1	145.1	28.8	238.6	3.2
Mequon Silt Loam	- -	- -	- -	- -	31.6	7.1	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	31.6	1.4
Morley Silt Loam	527.6	48.1	314.7	50.2	154.1	34.4	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	996.4	13.3
Ozaukee Silt Loam	- -	- -	- -	- -	77.8	17.3	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	77.8	1.0
Pistakee Silt Loam	2.1	0.2	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	2.1	<0.1
Hydrologic Soil Group D																				
Askum Silty Clay	35.7	3.3	0.3	<0.1	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	36.0	0.5
Marsh	3.1	0.3	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	3.1	<0.1
Ogden Muck	2.1	0.2	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	2.1	<0.1
Clayey Land	8.0	0.7	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	9.2	1.4	- -	- -	17.2	0.2
Disturbed Soils	16.8	1.5	2.3	0.4	68.5	15.3	610.4	99.8	2,453.3	92.4	306.0	100.0	606.9	100.0	- -	- -	- -	- -	4,064.2	54.1
Gravel Pit	2.9	0.3	5.9	0.9	1.5	0.3	1.3	0.2	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	2.9	<0.1
Water	14.5	1.3	5.9	0.9	1.5	0.3	1.3	0.2	201.0	7.6	- -	- -	- -	- -	- -	- -	1.0	0.2	225.2	3.0
Total	1,095.1	100.0	626.9	100.0	448.1	100.0	611.7	100.0	2,654.3	100.0	306.0	100.0	606.9	100.0	665.2	100.0	503.2	100.0	7,517.4	100.0

Source: U. S. Soil Conservation Service and SEWRPC.

within the northern portion of the City of Milwaukee and the Villages of Shorewood and Whitefish Bay—are covered by soils collectively referred to by the U. S. Soil Conservation Service as the Kewaunee-Monowa Association.

Bluff Characteristics

The bluffs along the Milwaukee County shoreline of Lake Michigan exhibit a variety of height, slope, composition, vegetative cover, and groundwater conditions. These conditions affect the degree and rate of bluff recession along different sections of the study area. This section describes the physical characteristics—the height and composition—of the bluffs, as surveyed in 1986 and 1987. Bluff erosion processes and bluff recession rates are described in later sections of this chapter.

Table 2 summarizes the lengths of shoreline within various bluff height ranges. In the southern portion of the study area, within the Cities of Cudahy, Oak Creek, and South Milwaukee, the bluff heights vary considerably, but generally range from 70 to 100 feet. Northward through the City of St. Francis and the City of Milwaukee south of the Milwaukee Harbor, the bluff heights decrease somewhat, generally

Table 2

SUMMARY OF BLUFF HEIGHTS ALONG THE LAKE MICHIGAN SHORELINE OF MILWAUKEE COUNTY: 1987

Bluff Height (feet)	Length of Shoreline (feet)	Percent of Study Area Shoreline Length
1 - 10	50,980	32.0
11 - 20	0	0.0
21 - 30	2,480	1.6
31 - 40	3,700	2.3
41 - 50	5,940	3.7
51 - 60	8,380	5.3
61 - 70	8,890	5.6
71 - 80	14,060	8.9
81 - 90	19,725	12.4
91 - 100	23,145	14.5
101 - 110	12,210	7.7
111 - 120	8,170	5.1
121 - 130	1,430	0.9
Total	159,110	100.0

Source: SEWRPC.

ranging from 40 to 70 feet. The shoreline extending from the U. S. Coast Guard Station to the City of Milwaukee Linnwood Avenue water treatment plant does not have a natural bluff at the water's edge because of the extensive landfilling of the lakebed that has occurred in order to develop that land for industrial, commercial, navigational, and recreational uses. North of the water treatment plant through the Villages of Shorewood and Whitefish Bay, and up to Green Tree Road in the Village of Fox Point, the bluff heights again vary considerably, ranging from 75 to 130 feet. North of Green Tree Road, a relatively wide terrace exists in front of the bluffs, which extends to a maximum width of approximately 900 feet and ranges from 4 to 10 feet in height. Within Doctors Park, which lies at the boundary between the Villages of Fox Point and Bayside, the terrace disappears and bluff heights range from about 80 to 100 feet through the Village of Bayside. About 32 percent of the shoreline within the study area is located within the Milwaukee Harbor area and the terraced area within the Village of Fox Point, where there is no significant bluff at the water's edge. About 13 percent of the shoreline has bluffs ranging from 20 to 60 feet in height; about 41 percent of the shoreline has bluffs ranging from 61 to 100 feet in height; and about 14 percent of the shoreline has bluffs greater than 100 feet in height.

The natural bluffs of Milwaukee County are composed of a variety of glacial-deposited materials. Field surveys were conducted in October 1987 for that portion of the shoreline extending from the Milwaukee-Racine County line northward to the Milwaukee Harbor area, and in the Village of Bayside, to identify those materials exposed on the bluff faces. Field surveys were conducted in May 1986 for that portion of Milwaukee County extending from the City of Milwaukee Linnwood Avenue water treatment plant northward through the Villages of Shorewood, Whitefish Bay, and Fox Point to Doctors Park as part of the northern Milwaukee County study. In shoreline areas where the bluff face was covered with fill, debris, or vegetation, the underlying stratigraphy was determined using historical geologic records or soil boring data. Figure 14 shows locations where soil boring data were available prior to this study. Eight additional soil borings were taken in March 1988 by Wisconsin Testing Laboratories, Inc., under contract to the Regional Planning Commission, in areas for which no previous stratigraphic data were available and where identification of the types and locations of the materials within the bluff was considered critical to the evaluation of the stability of the bluff slopes. The locations where the additional soil borings were taken are also shown in Figure 14.

A graphic summary description of the composition of the bluffs, based on all of the above data, is shown on the longitudinal section in Figure 14. Table 3 indicates the relative predominance of the various materials on the face of the bluff. Oak Creek till was found to be the predominant bluff material, covering about 31 percent of the total bluff face surface area in a vertical plane within the study area. General lake sediments were found to be the second most common bluff material, covering about 10 percent of the total bluff face. Ozaukee till and sand and silt were each found to cover about 8 percent of the total bluff face, respectively. The material constituting about 14 percent of the bluff face was undetermined because no stratigraphic data were available and the slopes were considered to be stable and well vegetated. An outcrop of bedrock was also identified on the beach in the southern portion of the Village of Fox Point.

Laboratory analyses of the bluff materials collected in the field by grab samples in October 1987, and through the soil borings conducted in March 1988, were performed by the Department of Civil Engineering, University of Wisconsin-Madison. The laboratory analyses, the results of which are summarized in Table 4, evaluated those soil properties which determine the resistance of the soil to slope failure.

Two important soil properties are the liquid limit and the plastic limit. The liquid limit is defined as that water content of a soil, expressed in percent by weight, at which the soil begins to act as a viscous liquid. Measured liquid limits for soil samples collected within the study area ranged from 13 to 53 percent. The plastic limit is defined as the water content at which the soil begins to act as a plastic. The difference between the liquid limit and the plastic limit is known as the plasticity index, and represents the range in water content through which the soil acts as a plastic, and may move laterally under load. The plasticity index is related to the presence of clay in the soil and is an indicator of the behavior of the clay particles in the soil under load when moisture is present. Plasticity index values measured within the study area ranged from 0 to 30 percent.

The fraction of the soil which is composed of silt- and clay-size particles may affect the resistance of the soil materials to slope failure. Soils containing significant amounts of clay and silt are referred to as cohesive soils; whereas granular soils such as gravel and sand are referred to as cohesionless soils. Because of low permeability, cohesive soils are often poorly drained and exhibit excess pore pressure, which may reduce slope stability. The soils sampled within the study area exhibited a wide range in textures, with the silt and clay fraction ranging from 2 to 100 percent.

The effective friction angle of a soil is another important indicator of the ability of a soil to resist slope failure. The effective friction angle is defined as a coefficient related to the frictional resistance of the soil to shearing when placed under stress. For sand, the effective friction angle is that angle at which the soil would achieve a stable slope if no groundwater were present within the soil. Effective friction angles are generally higher for soils that have a higher density, well-graded particles, and angular grains than for soils that have a lower density, uniform-size particles, and rounded grains. Effective friction angles within the study area were found to be relatively uniform, ranging from 22 to 37 degrees.

Beach Characteristics

A beach may be defined as an area of unconsolidated material which extends landward from the ordinary low-water line to the line marking a distinct change in physiographic form, or the beginning of permanent terrestrial vegetation. The width of a beach and the size and character of the sediments found on beaches vary widely in response to the lake water level, the degree of wave action affecting the beach, the slope of the beach face and the near-shore lake bottom, the kinds of material available near the shore for the formation of beaches, and man-made structures. Beach materials are supplied by littoral drift transporting particles contributed to the lake by watershed drainage and up-current shoreline erosion and bluff recession. Beaches may also be formed by material or fill placed on the shoreline. These artificially nourished beaches must be periodically renourished in order to maintain an adequate width. Table 5 sets forth beach characteristics for the southern Milwaukee County and the Village of Bayside shoreline of Lake Michigan as surveyed in November 1987, and for northern Milwaukee County as surveyed in August 1986. As the lake levels declined from late 1986 through 1988, the beaches in the study area generally widened. The information presented in this report therefore represents beach conditions under water level conditions which existed during the survey period of 1986 and 1987.

The tables indicate that the beaches within the study area are composed primarily of sand, gravel, and cobbles; smaller particles like silt and clay do not usually remain on the beach as do the large-size materials, since clay and silt are more readily kept in suspension and carried out into the lake. These finer materials tend to ultimately settle out in calmer, deeper, offshore waters. In 1986 and 1987, about 57 percent of the Milwaukee County shoreline either exhibited no beach at all—the lake reaching the bluff toe or, in some cases, a shore protection structure—or a beach less than 10 feet in width. It should be noted that the beach widths within the northern Milwaukee County study area were measured when lake levels were about 2.2 feet above the lake level measured during the southern Milwaukee County beach survey, and therefore a slight beach may have developed in areas where no significant beach was apparent during the 1986 survey.

Map 4 shows the distribution of various beach materials along the shoreline. Sand and a combination of sand and gravel were predominant along the southern shoreline of Bender Park, the shoreline extending from the South Shore sewage treatment plant northward to the Sheridan Park groin system, Bay View Park, South Shore Park Beach, Bradford Beach, the shoreline north of the Linnwood Avenue water treatment plant, Atwater Park, the central portion of the Fox Point terrace, Doctors Park, and the shoreline along portions of the Village of Bayside. Beaches composed of larger materials such as gravel and cobbles were found along the northern shoreline of Bender Park northward to the South Shore wastewater treatment plant, portions of Sheridan Park, the shoreline along the City of St. Francis, Klode Park, and Big Bay Park. Nearly all of the remainder of the shoreline area contained little or no beach.

Table 5 and Map 4 also indicate the beach widths along the shoreline. Within the portion of the study area that includes southern Milwaukee County and the Village of Bayside, about 19 per-

Figure 14

LONGITUDINAL PROFILE OF BLUFF STRATIGRAPHY

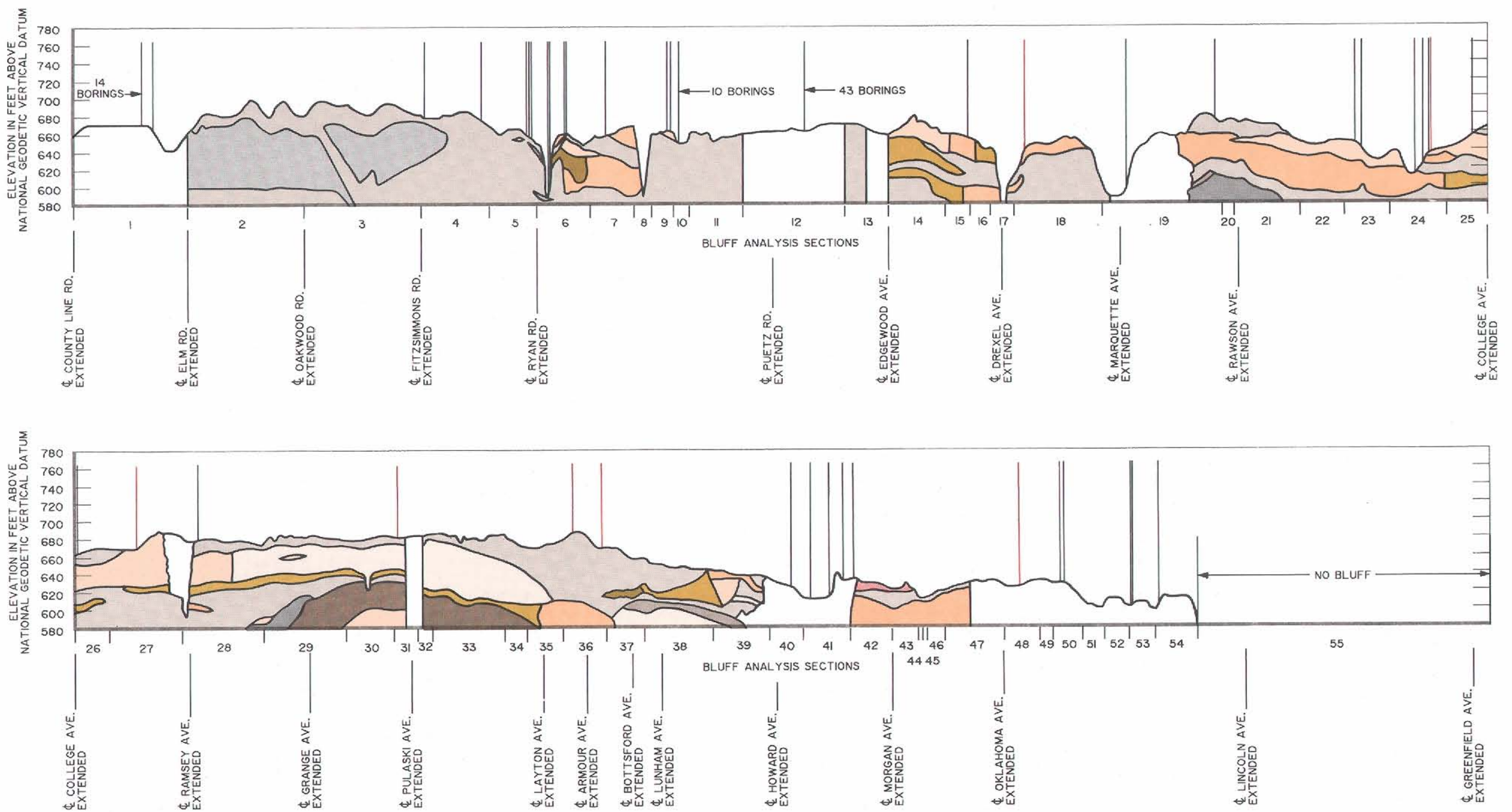
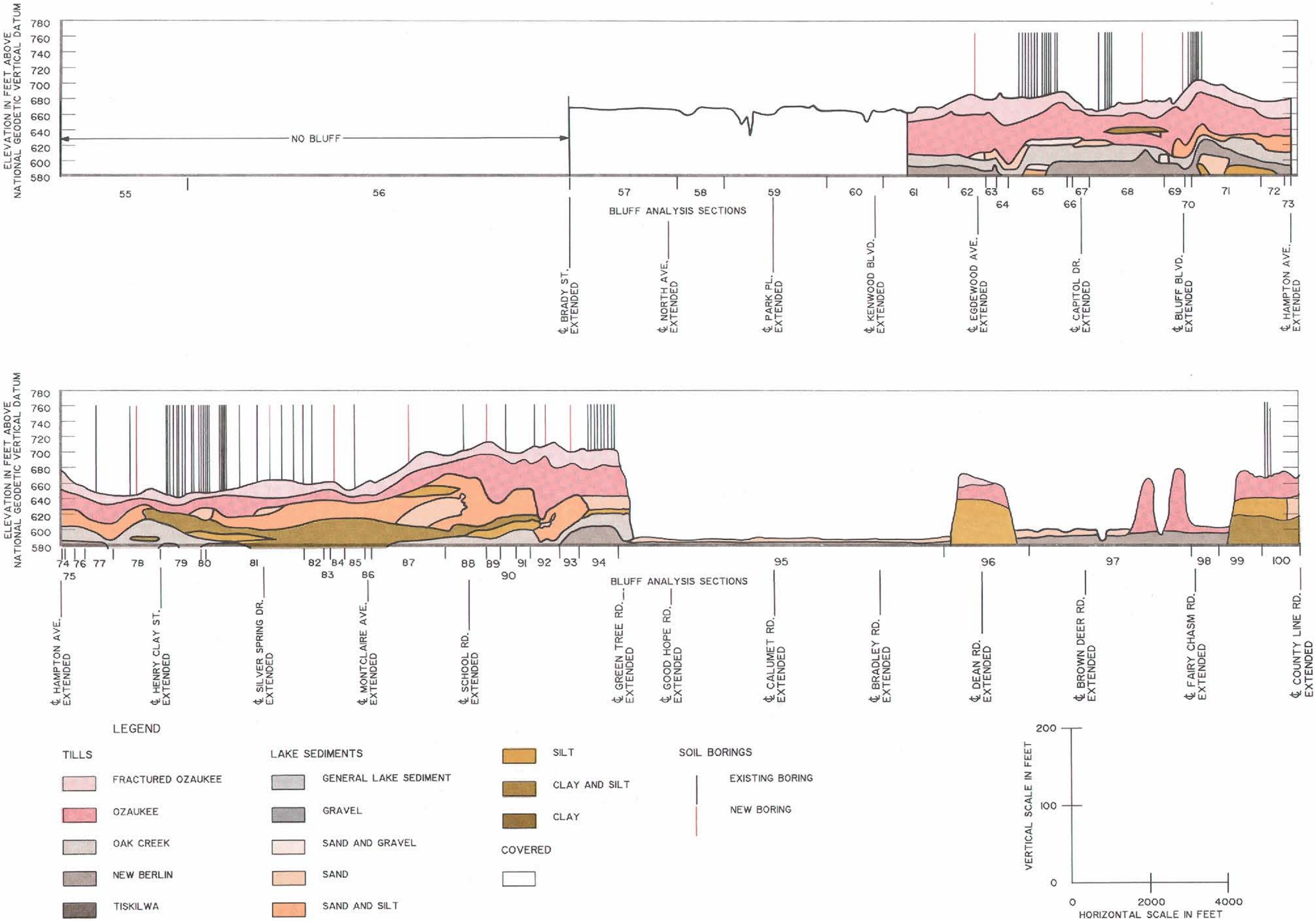


Figure 14 (continued)



Source: D. M. Mickelson and SEWRPC.

Table 3

BLUFF COMPOSITION ALONG THE LAKE MICHIGAN SHORELINE OF MILWAUKEE COUNTY: 1986-1987

Bluff Composition	Percent of Bluff Face Surface Area in the Vertical Plane
Tills	
Oak Creek	31
New Berlin	3
Tiskilwa	5
Ozaukee	8
Lake Sediment	
General Lake Sediment	10
Gravel	2
Sand and Gravel	6
Sand	7
Sand and Silt	8
Silt	4
Silt and Clay	2
Clay and Sand	<1
Undetermined	14

Source: D. M. Mickelson and SEWRPC.

cent of the total study area shoreline had a beach ranging in width from 11 to 50 feet, and about 12 percent had a beach ranging in width from 51 to 90 feet. Only about 4 percent of the shoreline, located north of the Oak Creek power plant, north of the South Shore sewage treatment plant, south of the South Milwaukee Yacht Club, at Grant Park, at South Shore Park Beach, and at Bradford Beach, had a beach over 90 feet wide during the 1987 survey. Within the northern Milwaukee County study area in 1986, about 5 percent of the total study area shoreline had a beach ranging in width from 11 to 50 feet, and about 2 percent had a beach ranging in width from 51 to 90 feet. Less than 1 percent of the study area shoreline located in northern Milwaukee County had a beach width greater than 90 feet during the 1986 survey.

The beach slopes of the Milwaukee County shoreline are also shown on Map 4. Generally, beach slopes ranged up to 10 degrees. However, steeper beach slopes ranging from 10 to 20 degrees were measured at the northern shoreline of Bender Park, the shoreline immediately south of the South Shore sewage treatment plant, a portion of the City of St. Francis shoreline, the southern portion of Big Bay Park, and the northern portion of Atwater Park. Table 6 summarizes the length of shoreline having various beach slopes. No beach slope determination was made for the approximately 57 percent of the total shoreline which at the time of the surveys had a beach width of 10 feet or less. Within the portion of the study area which includes southern Milwaukee County and the Village of Bayside, about 13 percent of the total study area shoreline had a beach slope ranging from 0 to 6 degrees, about 22 percent had a beach slope ranging from 7 to 12 degrees, and less than 1 percent had a beach slope greater than 12 degrees. In 1986, about 3 percent of the northern Milwaukee County study area shoreline had a beach slope ranging from 0 to 6 degrees, about 4 percent had a beach slope ranging from 7 to 12 degrees, and less than 1 percent had a beach slope greater than 12 degrees. Generally, the wider beaches tended to have slightly flatter slopes and were composed of finer grained materials, whereas the narrower beaches tended to have steeper slopes and were composed of coarser grained materials.

Near-shore Bathymetry

The near-shore bathymetry, or lake bottom elevation, influences the refraction and shoaling of waves; the absorption of wave energy; and the selection, design, and cost of both onshore and offshore protection structures. Generalized bathymetric data as of 1979 are available for the entire Milwaukee County Lake Michigan shoreline from the National Oceanic and Atmospheric Administration (NOAA). As presented in Table 7, along portions of the shoreline, the NOAA bathymetric data have been updated by various governmental and private sources.

The near-shore slopes are the most gentle along the shoreline behind the South Shore breakwater; off Bradford Beach and Lake Park; within the City of Milwaukee immediately north of the Linnwood Avenue water treatment plant; near the boundary between the Villages of Shorewood and Whitefish Bay; and in the vicinity of the Village of Fox Point terrace and Doctors Park. The near-shore slopes are steepest off the Oak Creek power plant in the City of Oak Creek; the old Lakeside power plant in the City of St. Francis; just north of Atwater Park; and at the northeast-facing shoreline in the Village of Whitefish Bay between Klode Park and Big Bay Park.

Table 4

SELECTED PROPERTIES OF BLUFF MATERIALS WITHIN THE LAKE MICHIGAN SHORELINE OF MILWAUKEE COUNTY: 1986-1988

Soil Type	Location[a] (bluff analysis section)	Sample Depth (feet)	Liquid Limit (percent)	Plastic Limit (percent)	Plasticity Index (percent)	Percent Gravel and Sand	Percent Silt and Clay	Cohesion Intercept (pounds per square foot)	Effective Friction Angle (degree)
Grab Samples									
Glacial Tills									
New Berlin	20	- -	20	20	0	43	57	- -	- -
	39	- -	13	12	1	64	36	- -	- -
	65	- -	15	12	3	56	44	- -	- -
	71	- -	20	15	5	41	59	- -	- -
	71	- -	20	14	6	44	56	- -	- -
	93	- -	14	13	1	56	44	- -	- -
	94	- -	16	14	2	34	66	- -	- -
	97	- -	16	14	2	39	61	- -	- -
	2	- -	26	14	12	6	94	- -	- -
	2	- -	28	14	14	1	99	- -	- -
Oak Creek	3	- -	26	15	11	11	89	- -	- -
	3	- -	22	14	8	18	82	- -	- -
	3	- -	26	15	11	8	92	- -	- -
	4	- -	28	16	12	0	100	- -	- -
	25	- -	33	17	16	8	92	- -	- -
	25	- -	35	17	18	6	94	- -	- -
	33	- -	43	20	23	5	95	- -	- -
	39	- -	38	17	21	7	93	- -	- -
	39	- -	38	17	21	9	91	- -	- -
	43	- -	29	15	14	15	85	- -	- -
	65	- -	30	16	14	13	87	- -	- -
	93	- -	34	17	17	10	90	- -	- -
Ozaukee	39	- -	32	17	15	23	77	- -	- -
	43	- -	39	17	22	22	78	- -	- -
	63	- -	33	17	16	10	90	- -	- -
	68	- -	32	16	16	9	91	- -	- -
	71	- -	32	16	16	7	93	- -	- -
	88	- -	38	18	20	11	89	- -	- -
Tiskilwa	29	- -	16	15	1	74	26	- -	- -
	30	- -	28	14	14	27	73	- -	- -
	30	- -	26	13	13	34	66	- -	- -
	33	- -	19	13	6	30	70	- -	- -
Lake Sediments									
Medium-Fine Sand	3	- -	- -	- -	- -	96	4	- -	- -
	20	- -	- -	- -	- -	98	2	- -	- -
	88	- -	- -	- -	- -	90	10	- -	- -
Sand and Gravel	29	- -	- -	- -	- -	98	2	- -	- -
Silt	2	- -	17	17	0	14	86	- -	- -
	3	- -	17	16	1	16	84	- -	- -
	5	- -	19	18	0	4	96	- -	- -
	14	- -	19	17	2	1	99	- -	- -
	28	- -	19	16	3	13	87	- -	- -
	39	- -	18	18	0	7	93	- -	- -
	65	- -	- -	18	- -	12	88	- -	- -
	88	- -	- -	21	- -	7	93	- -	- -
	100	- -	19	19	- -	4	96	- -	- -
Silt and Fine Sand	14	- -	17	14	3	14	86	- -	- -
	43	- -	14	14	0	40	60	- -	- -
Clay and Silt	2	- -	24	15	9	2	98	- -	- -
	2	- -	31	16	15	1	99	- -	- -
	16	- -	30	18	12	5	95	- -	- -
	- -	- -	31	17	14	9	91	- -	- -
	22	- -	23	16	7	1	99	- -	- -
	62	- -	48	22	26	3	97	- -	- -
	72	- -	35	17	18	0	100	- -	- -
	89	- -	34	11	23	0	100	- -	- -
	100	- -	42	23	19	0	100	- -	- -

Table 4 (continued)

Soil Type	Location[a] (bluff analysis section)	Sample Depth (feet)	Liquid Limit (percent)	Plastic Limit (percent)	Plasticity Index (percent)	Percent Gravel and Sand	Percent Silt and Clay	Cohesion Intercept (pounds per square foot)	Effective Friction Angle (degree)
Soil Borings									
Glacial Tills									
New Berlin	70	100	19	14	5	36	64	- -	- -
Oak Creek	62	80	21	16	5	28	72	540	32
	92	130	32	17	15	9	91	0	27
	93	110	32	17	15	9	91	- -	- -
Ozaukee	92	20	28	14	14	10	90	604	26
Tiskilwa	31	80	- -	- -	- -	26	74	350	27
Lake Sediments									
Medium-Fine Sand	92	85	- -	- -	- -	81	19	- -	- -
Silt	18	15	20	5	15	35	65	230	27
	18	15	27	18	9	6	94	- -	- -
	31	25	- -	- -	29	16	84	1,340	22
	84	35	18	18	<1	8	92	- -	- -
	84	40	48	- -	- -	0	100	4,820	31
	87	65	19	18	1	1	99	- -	- -
	90	112	18	17	1	2	98	- -	- -
Silt and Sand	24	40	- -	- -	- -	- -	- -	0	37
	68	45	- -	- -	- -	68	32	- -	- -
	87	70	- -	- -	- -	55	45	- -	- -
	87	85	- -	- -	- -	11	89	- -	- -
Clay and Silt	31	25	23	14	9	- -	- -	- -	- -
	78	25	27	- -	- -	1	99	- -	- -
	84	35	29	18	11	2	98	- -	- -
	84	55	30	18	12	0	100	- -	- -
	84	75	34	18	16	0	100	- -	- -
	90	110	22	13	9	24	76	- -	- -
	92	80	26	15	11	13	87	- -	- -
Clay	24	20	53	23	30	0	100	375	27
Fine Sand and Silt	68	40	23	14	9	26	74	- -	- -
	78	20	- -	18	- -	21	79	- -	- -
	84	30	- -	- -	- -	23	77	- -	- -
	92	30	- -	- -	- -	36	64	- -	- -
	92	70	- -	- -	- -	36	64	- -	- -

[a]*The locations of the bluff analysis sections are shown on Map 24.*

Source: T. B. Edil and D. M. Mickelson.

The near-shore bathymetry within the study area was previously surveyed in 1871, 1912, and 1944.[2] A review of these early data indicated that in 1871 and 1912, the near-shore slopes were more gentle than those recently surveyed. The bathymetric survey results in 1944, however, were similar to the existing conditions. High water levels, such as those that occurred in 1985 and 1986, and a decline in the availability of littoral drift as more shore protection structures are installed, would be expected to produce a near-shore zone somewhat steeper in the future, unless measures such as beach nourishment are implemented.

[2]*U. S. Army Corps of Engineers, Beach Erosion Study, Lake Michigan Shore Line of Milwaukee County, Wis., 1945.*

Table 5

BEACH CHARACTERISTICS OF THE LAKE MICHIGAN SHORELINE OF MILWAUKEE COUNTY

Beach Composition	Beach Width (feet): <10 Shoreline Length (feet)	<10 Percent of Total Shoreline Length	11-50 Shoreline Length (feet)	11-50 Percent of Total Shoreline Length	51-90 Shoreline Length (feet)	51-90 Percent of Total Shoreline Length	>90 Shoreline Length (feet)	>90 Percent of Total Shoreline Length	Total Shoreline Length (feet)	Percent of Total County Shoreline Length
	Northern Milwaukee County Study Area Shoreline[a]									
Sand	0	- -	1,540	1.0	720	0.5	920	0.5	3,180	2.0
Sand and Gravel	0	- -	4,560	2.9	2,080	1.3	80	0.1	6,720	4.3
Gravel	0	- -	1,920	1.2	60	<0.1	0	- -	1,980	1.2
Cobbles	0	- -	0	- -	200	0.1	0	- -	200	0.1
No Beach	26,690	16.8	- -	- -	- -	- -	- -	- -	26,690	16.8
Subtotal	26,690	16.8	8,020	5.1	3,060	1.9	1,000	0.6	38,770	24.4
	Remaining Shoreline of Milwaukee County[b]									
Sand	230	0.1	3,930	2.4	8,820	5.6	4,300	2.7	17,280	10.8
Sand and Gravel	0	- -	16,760	10.5	10,190	6.4	2,400	1.5	29,350	18.4
Gravel	0	- -	5,190	3.3	0	- -	0	- -	5,190	3.3
Cobbles	1,170	0.7	4,530	2.9	0	- -	0	- -	5,700	3.6
No Beach	62,820	39.5	- -	- -	- -	- -	- -	- -	62,820	39.5
Subtotal	64,220	40.3	30,410	19.1	19,010	12.0	6,700	4.2	120,340	75.6
Total	90,910	57.1	38,430	24.2	22,070	13.9	7,700	4.8	159,110	100.0

[a]*Based on field surveys conducted in August 1986. Includes the shoreline extending from the Linnwood Avenue water treatment plant in the City of Milwaukee northward through the Villages of Shorewood, Whitefish Bay, and Fox Point to Doctors Park.*

[b]*Based on field surveys conducted in November 1987. Includes the shoreline in the Cities of Oak Creek, South Milwaukee, Cudahy, St. Francis, and Milwaukee south of the Linnwood Avenue water treatment plant and in the Village of Bayside. The water level in November 1987 was about 2.2 feet lower than in August 1986.*

Source: SEWRPC.

Groundwater Resources

The occurrence, distribution, direction, and quantity of groundwater flow have important impacts on the stability of the bluff slopes. Along the Milwaukee County shoreline, groundwater generally flows toward the lake and discharges either at, or below, the base of the bluff into the lake, or seeps out of the bluff face at some elevation above lake level.

Groundwater flow rates into Lake Michigan have been estimated by Cherkauer and Hensel[3] under both natural and existing conditions. They reported that human activity—primarily groundwater pumping and reduced infiltration in urban areas—had reduced the natural flow of groundwater into Lake Michigan along the shoreline of southeastern Wisconsin by up to 15 percent. Under existing conditions—from about 1979 through 1983—the net inflow of groundwater into Lake Michigan was calculated to range from 12,700 to 19,300 cubic feet per day per mile of shoreline. This groundwater input represents 7 to 11 percent of the total river flow from the Wisconsin drainage basin of Lake Michigan. Thus, groundwater inflow represents a small but significant portion of the total water budget of the lake.

There are two major aquifers beneath the Milwaukee County study area. These aquifers are commonly called the "deep sandstone" aquifer and the "shallow limestone" aquifer. The aquifers differ widely in water yield capabilities and extend to great depths.

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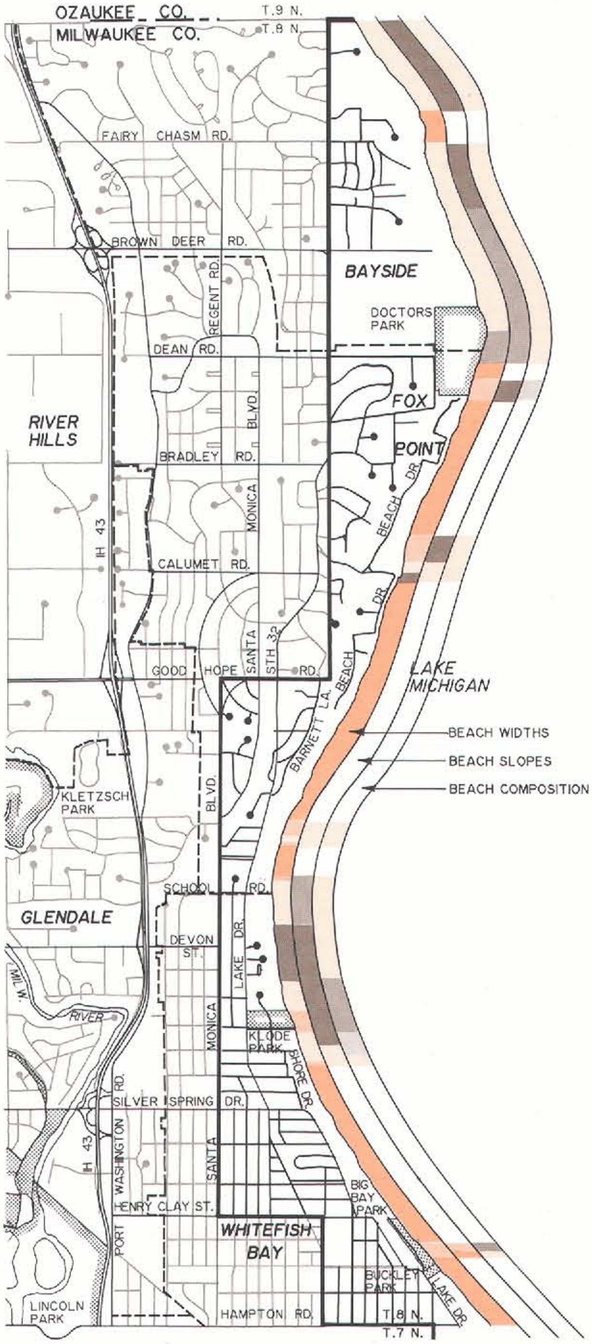
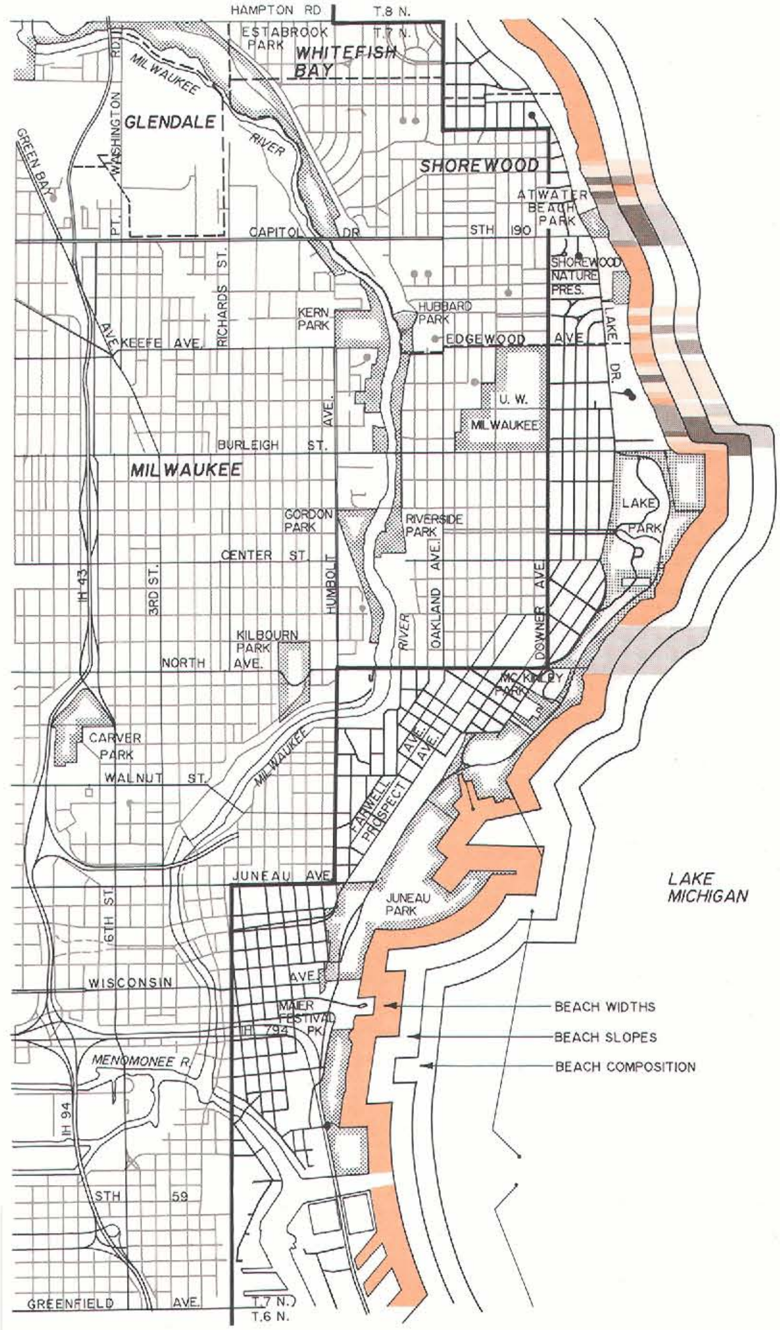
[3]*Douglas S. Cherkauer and Bruce R. Hensel, "Groundwater Flow into Lake Michigan from Wisconsin," Journal of Hydrology, 84 (1986), pp. 261-271.*

Map 4

EXISTING BEACH CONDITIONS ALONG THE MILWAUKEE COUNTY LAKE MICHIGAN SHORELINE





LEGEND

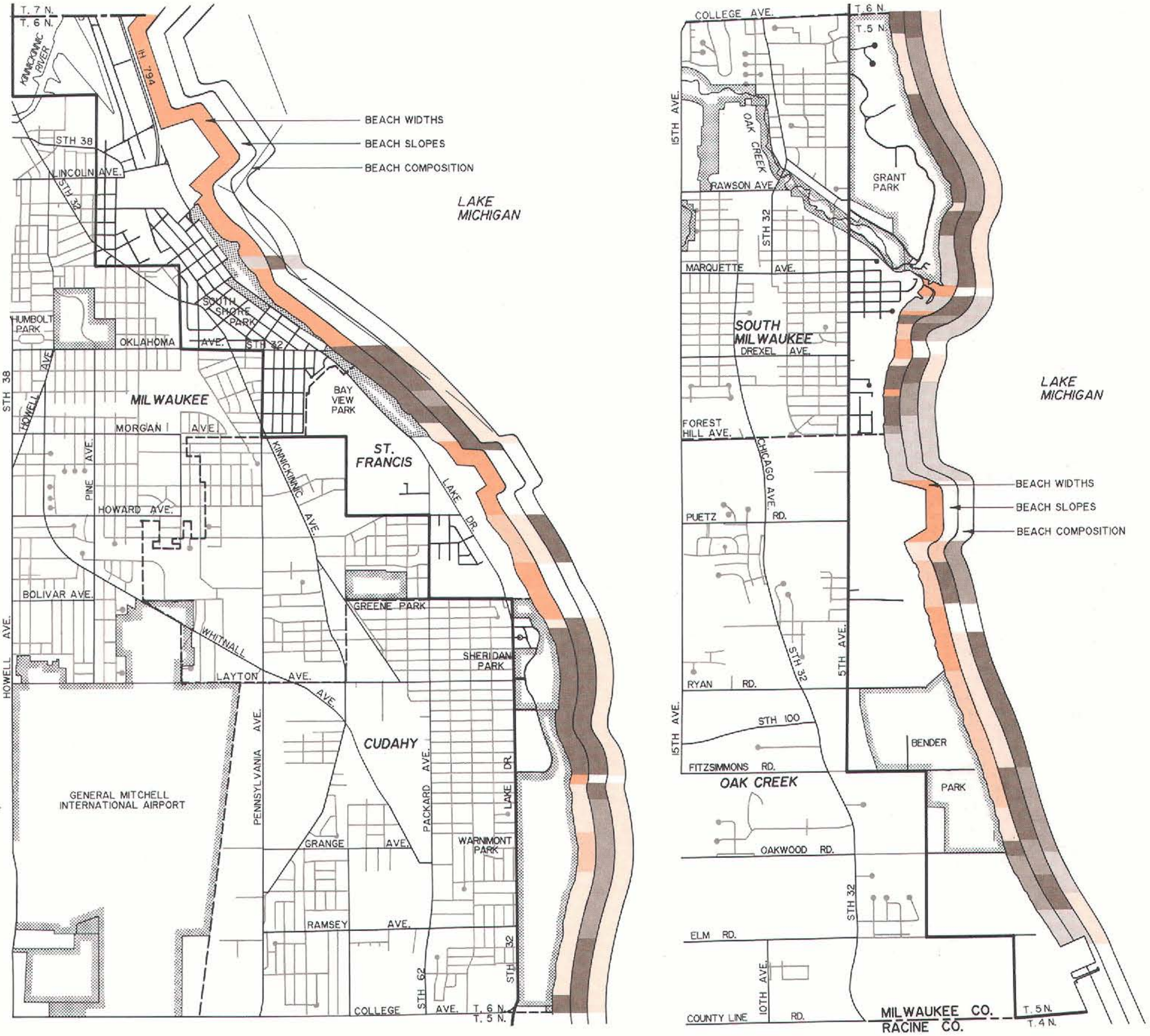
BEACH WIDTHS

- GREATER THAN 90 FEET
- 70-90 FEET
- 50-70 FEET
- 30-50 FEET
- 10-30 FEET
- LESS THAN 10 FEET

BEACH SLOPES

- 0-3 DEGREES
- 4-6 DEGREES
- 7-9 DEGREES
- 10-12 DEGREES
- 13-15 DEGREES
- GREATER THAN 15 DEGREES

Map 4 (continued)



BEACH COMPOSITION

- SAND
- GRAVEL
- COBBLES
- SAND AND GRAVEL
- SAND, GRAVEL, AND COBBLES

[a] BEACH CHARACTERISTICS FOR THE SHORELINE EXTENDING FROM THE LINNWOOD AVENUE WATER TREATMENT PLANT NORTHWARD TO DOCTOR'S PARK WERE SURVEYED IN 1986. BEACH CHARACTERISTICS FOR THE REST OF THE COUNTY WERE SURVEYED IN 1987.

Source: SEWRPC.

Table 6

BEACH SLOPES WITHIN THE LAKE MICHIGAN SHORELINE OF MILWAUKEE COUNTY: 1986-1987

Beach Slope (degrees)	Northern Milwaukee County Study Area Shoreline[a]		Remaining Shoreline of Milwaukee County[b]		Total Study Area Shoreline	
	Length of Shoreline (feet)	Percent of Shoreline	Length of Shoreline (feet)	Percent of Shoreline	Length of Shoreline (feet)	Percent of Shoreline
No Significant Beach	26,690	16.8	64,220	40.3	90,910	57.1
0 - 3	0	- -	5,260	3.3	5,260	3.3
4 - 6	4,080	2.6	16,080	10.1	20,160	12.7
7 - 9	2,730	1.7	29,600	18.6	32,330	20.3
10 - 12	4,440	2.8	4,530	2.9	8,970	5.7
13 - 15	620	0.4	650	0.4	1,270	0.4
> 15	210	0.1	0	- -	210	0.1
Total	38,770	24.4	120,340	75.6	159,110	100.0

[a]*Based on field surveys conducted in August 1986. Includes the shoreline extending from the Linnwood Avenue water treatment plant in the City of Milwaukee northward through the Villages of Shorewood, Whitefish Bay, and Fox Point to Doctors Park.*

[b]*Based on field surveys conducted in November 1987. The water level in November 1987 was about 2.2 feet lower than in August 1986. Includes the shoreline in the Cities of Oak Creek, South Milwaukee, Cudahy, St. Francis, and Milwaukee south of the Linnwood Avenue water treatment plant and in the Village of Bayside.*

Source: SEWRPC.

Table 7

SOURCES OF UPDATED BATHYMETRIC DATA

General Location	Source
Oak Creek Power Plant	Wisconsin Electric Power Company, 1972, 1973
South Shore Wastewater Treatment Plant	Milwaukee Metropolitan Sewerage District, South Shore Wastewater Treatment Plant Lakefill Site Development Site Key Plan, 1981
Bay View Park-South Shore Park	Foth & Van Dyke and Associates, Inc., "South Shore and Bay View Park Shoreline Restoration Concept Design and Costs," in preparation, 1988
Milwaukee Harbor Area	U. S. Army Corps of Engineers, Sounding Maps, Milwaukee Harbor, 1985 National Oceanic and Atmospheric Administration, Sounding Maps, Milwaukee Harbor, 1984
McKinley Beach	Warzyn Engineering, Inc., et al., Milwaukee County McKinley Beach Restoration Drawings, 1987
Lake Park	STS Consultants, Ltd., Conceptual Plans Milwaukee Shoreline Protection, Milwaukee, Wisconsin, 1987
Northern Milwaukee County	Warzyn Engineering, Inc., et al., A Future for the Milwaukee County North Shore, 1987

Source: SEWRPC.

The deep sandstone aquifer, which is more than 1,300 feet thick, underlies the entire County and is composed of Cambrian Age and Ordovician Age strata. The top of this aquifer lies about 600 feet below the surface of the study area. Most recharge of the sandstone aquifer is by lateral movement of water down the hydraulic gradient from west of the study area.

The shallow limestone aquifer, referred to as the Niagara aquifer, is actually composed of Silurian Age dolomite strata, and is about 300 feet thick. The top of this aquifer generally lies up to 100 feet below the surface of the study area. Recharge of this aquifer is by the downward seepage of precipitation which falls within, and west of, the study area. It is possible that some recharge may also be induced from Lake Michigan; however, if this does occur, the relatively impermeable layers of lake silt and glacial drift would make such recharge a very slow process.

Above the Niagara dolomite is a layer of unconsolidated glacial deposits composed primarily of till and sand and gravel. These deposits range in thickness to more than 200 feet over the study area. The sand and gravel layers may act as water-bearing units. The presence of groundwater in this glacial bluff material reduces the frictional resistance to stress forces, creates a seepage pressure in the direction of water flow, and adds weight to the bluff. All of these factors reduce bluff slope stability. For this reason, an attempt was made to define the elevation of the groundwater in the sediments and glacial tills within the Milwaukee County bluffs. Estimated groundwater levels for the study area were based on either field observations of seepage zones, soil borings, observation well measurements, electrical resistivity analyses, or the location of permeable soil strata.

As shown on Map 5 and in Table 8, there were 39 locations where the level of the water table was identified by observation of groundwater seepage in May 1986 within the northern Milwaukee County study area, and in October 1987 within the remaining portion of the County. As already noted, eight soil borings were taken in March 1988 as part of the study in areas where it was necessary to identify the stratigraphy of the bluff in order to more accurately evaluate the stability of the bluff slopes. At the time of the borings—in March 1988—and one or two days afterward, the depth to the water table was identified. The locations of the new soil boring sites are shown on Map 5. Nine soil borings were taken in October 1986 as part of the northern Milwaukee County study. At two of the northern Milwaukee County soil boring sites, groundwater observation wells were installed by the property owners. Electrical resistivity methods were used to measure the depths to the water tables at 10 locations along the northern Milwaukee County study area shoreline in October and November 1986. Such methods introduce electrical currents into the ground through a number of electrodes, and the resistivity of the subsurface materials is then measured. The resistivity of the materials can be related to the composition of the related materials, their porosity, the pore fluid conductivity, and the degree of saturation.

Based on the results of these data collection efforts, the main water table was identified within the Milwaukee County bluffs. The water table was generally located within a lake sediment layer lying between two glacial tills. The water table ranged in depth from 10 to 80 feet from the top of the bluff. Within northern Milwaukee County, an additional perched water table was usually found within the fractured Ozaukee till near the top of the bluff.

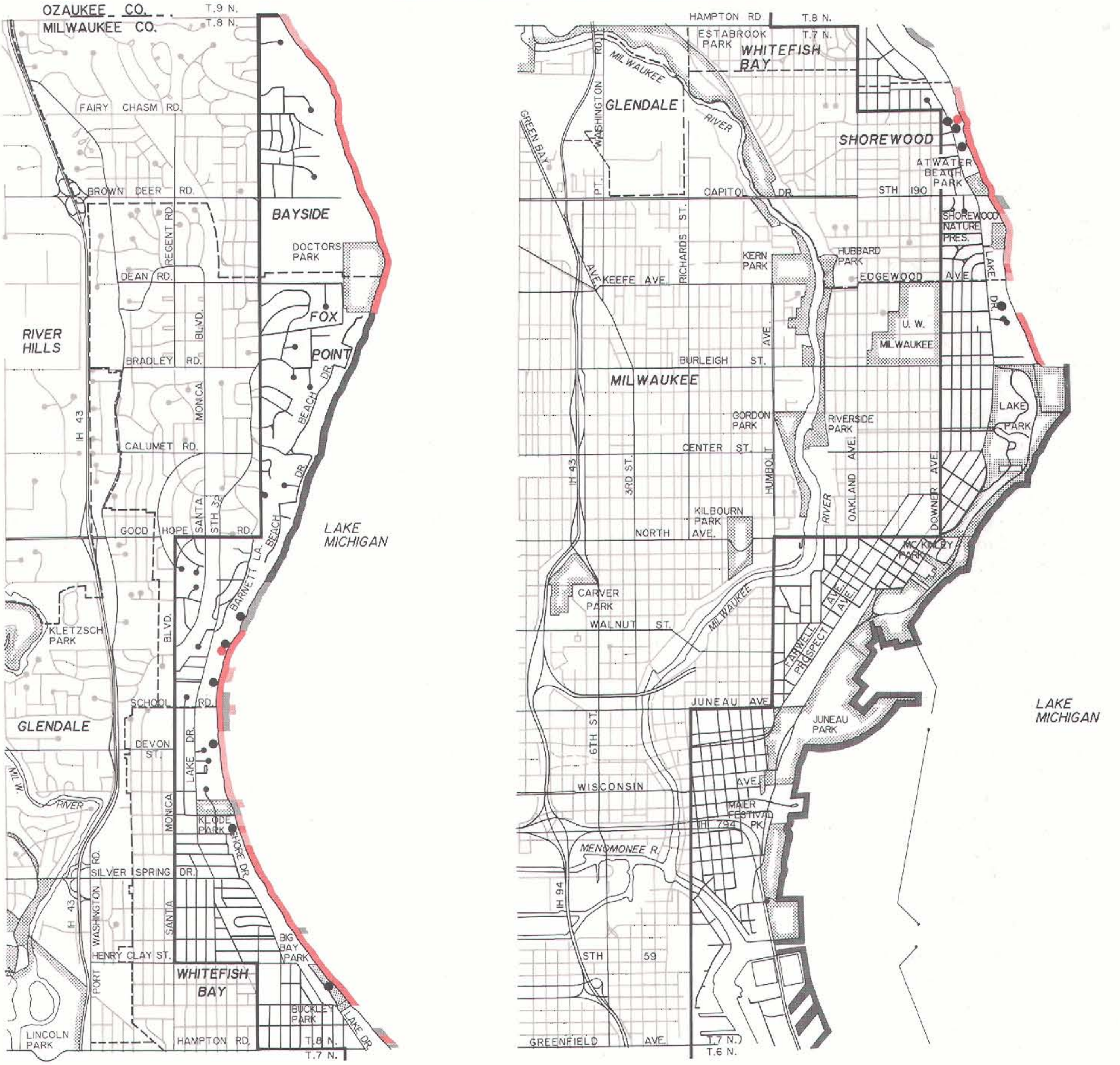
Climate

Air temperature and the type, intensity, and duration of precipitation events affect the degree and extent of shoreline erosion. Climatic impacts on shoreline erosion include freeze-thaw actions caused by water contained within the bluff material; high surface stormwater runoff from frozen soils in early spring; the reduction of wave action due to ice formation on the lake; and high levels of surface runoff and soil erosion during periods of heavy rainfall.

Air temperature impacts primarily include the formation of ice on the lake, the initiation of freeze-thaw actions on soils, and high stormwater runoff rates from frozen soils. Table 9 presents average monthly air temperature variations at the Milwaukee National Weather Service station for the 37-year period from 1951 through 1987. As shown in the table, winter temperatures, as measured by the monthly means for December, January, and February, range from 18.9 to 25.2°F. Summer temperatures, as measured by the monthly means for June, July, and August, range from 65.1 to 70.5°F.

Map 5

GROUNDWATER DATA SOURCES

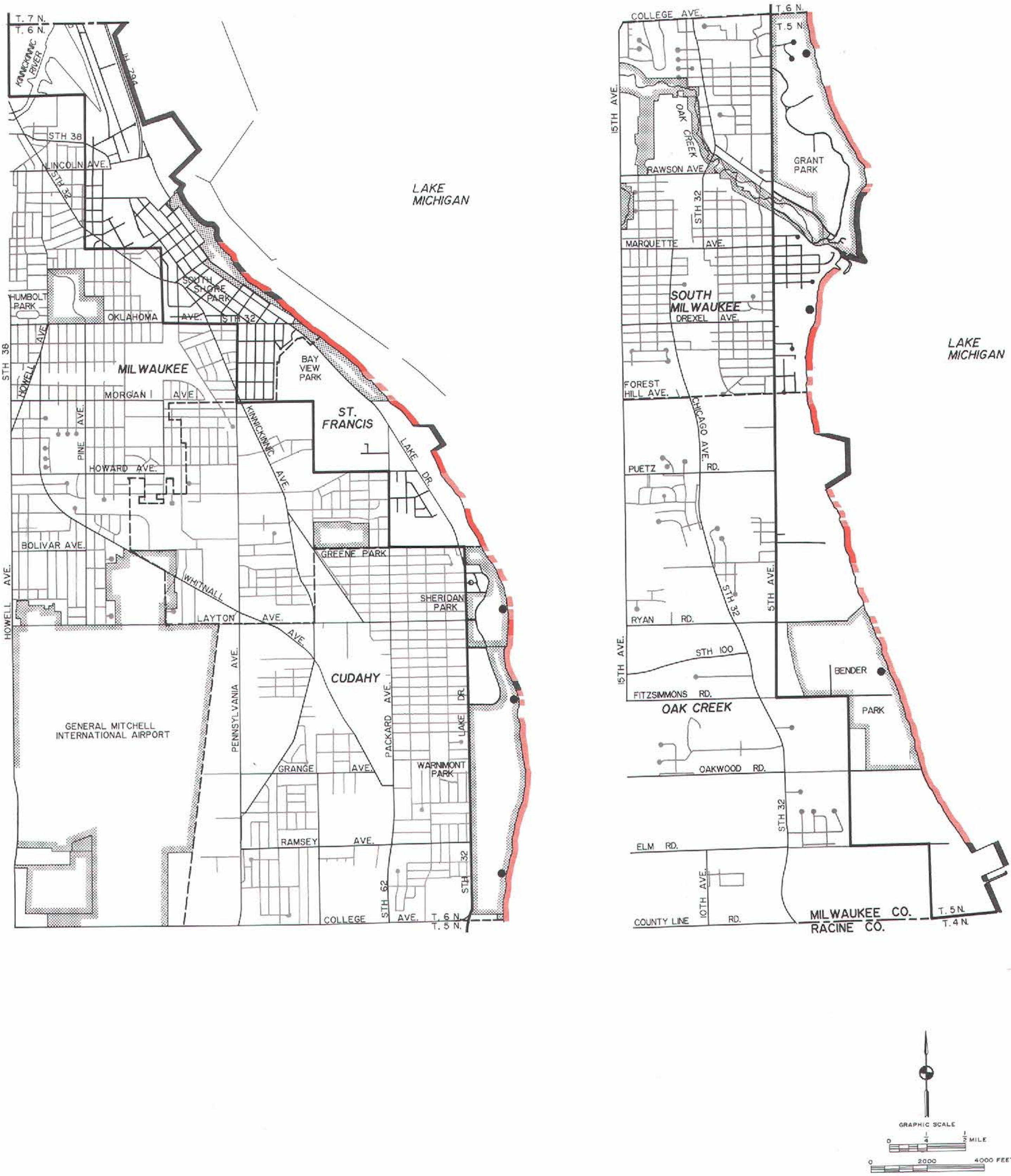


LEGEND

- OBSERVED GROUNDWATER SEEPAGE 1986-1987
- NEW SOIL BORING 1986 OR 1988
- NEW GROUNDWATER OBSERVATION WELL
- ELECTRICAL RESISTIVITY ANALYSIS
- LOCATION OF PERMEABLE SOIL STRATA
- GROUNDWATER NOT ESTIMATED

[a] GROUNDWATER ELEVATIONS FROM LINNWOOD AVENUE WATER TREATMENT PLANT TO THE SOUTH END OF DOCTOR'S PARK WERE ESTIMATED IN 1986. ALL OTHER ELEVATIONS WERE ESTIMATED IN 1987.

Map 5 (continued)



Source: SEWRPC.

Table 8

SOURCES OF IDENTIFIED GROUNDWATER LEVELS WITHIN THE MILWAUKEE COUNTY BLUFFS

Civil Division	Bluff Analysis Section[a]	Location	Observation of Seepage 1986-1987	Soil Boring 1986 and 1988	Observation Well Measurement	Estimate Electrical Resistivity Analysis	Based on Location of Permeable Soil Strata	Groundwater Not Estimated[b]
City of Oak Creek	1	WEPCo Oak Creek Power Plant	- -	- -	- -	- -	- -	X
	2	Elm Road- Oakwood Road	X	- -	- -	- -	- -	- -
	3	Bender Park	X	X	- -	- -	- -	- -
	4	Bender Park	X	- -	- -	- -	- -	- -
	5	Bender Park	X	- -	- -	- -	- -	- -
	6	9300 S. 5th Avenue	X	- -	- -	- -	- -	- -
	7	9180 S. 5th Avenue	X	- -	- -	- -	- -	- -
	8	9170 S. 5th Avenue	- -	- -	- -	- -	X	- -
	9	4301 E. Depot Road	- -	- -	- -	- -	X	- -
	10	9006 S. 5th Avenue	- -	- -	- -	- -	X	- -
	11	9006-8740 S. 5th Avenue	X	- -	- -	- -	- -	- -
	12	South Shore Treatment Plant	- -	- -	- -	- -	- -	X
	13	8400 S. 5th Avenue	- -	- -	- -	- -	X	- -
City of South Milwaukee	14	3817-3509 3rd Avenue	X	- -	- -	- -	- -	- -
	15	235 Lakeview Avenue- 3303 Marina Road	- -	- -	- -	- -	X	- -
	16	3303 Marina Road- 3333 5th Avenue	- -	- -	- -	- -	X	- -
	17	3333 5th Avenue	- -	- -	- -	- -	X	- -
	18	South Milwaukee Water Utility-Marshall Avenue	X	X	- -	- -	- -	- -
	19	South Milwaukee Yacht Club-Grant Park	- -	- -	- -	- -	- -	X
	20	Grant Park	X	- -	- -	- -	- -	- -
	21	Grant Park	X	- -	- -	- -	- -	- -
	22	Grant Park	X	- -	- -	- -	- -	- -
	23	Grant Park	X	- -	- -	- -	- -	- -
	24	Grant Park	X	X	- -	- -	- -	- -
	25	Grant Park	X	- -	- -	- -	- -	- -
City of Cudahy	26	College Avenue- Warnimont Park	X	- -	X	- -	- -	- -
	27	Warnimont Park	X	X	- -	- -	- -	- -
	28	Warnimont Park	X	- -	- -	- -	- -	- -
	29	Warnimont Park	X	- -	- -	- -	- -	- -
	30	Warnimont Park	X	- -	- -	- -	- -	- -
	31	Warnimont Park	X	X	- -	- -	- -	- -
	32	Cudahy Water Intake	- -	- -	- -	- -	- -	X
	33	Warnimont Park	X	- -	- -	- -	- -	- -
	34	Sheridan Park	- -	- -	- -	- -	X	- -
	35	Sheridan Park	X	X	- -	- -	- -	- -
	36	Sheridan Park	X	- -	- -	- -	- -	- -
	37	Sheridan Park	X	- -	- -	- -	- -	- -
City of St. Francis	38	Lunham Avenue- Denton Avenue	- -	- -	- -	- -	X	- -
	39	Denton Avenue-100's of Howard Avenue	X	- -	- -	- -	- -	- -
	40	100's of Howard Avenue- Power Plant Breakwater	- -	- -	- -	- -	X	- -
	41	WEPCo Lakeside Power Plant	- -	- -	- -	- -	- -	X
(continued)	42	Power Plant Breakwater- Packard Avenue Extended	- -	- -	- -	- -	X	- -
	43	Bay View Park	X	- -	- -	- -	- -	- -
	44	Bay View Park	X	- -	- -	- -	- -	- -
	45	Bay View Park	- -	- -	- -	- -	X	- -
	46	Bay View Park	- -	- -	- -	- -	X	- -
	47	Bay View Park	- -	- -	- -	- -	X	- -
City of Milwaukee	48	South Shore Park	- -	X	- -	- -	X	- -
	49	Texas Street Water Intake	- -	- -	- -	- -	- -	X
	50	South Shore Park	- -	- -	- -	- -	X	- -
	51	South Shore Park	- -	- -	- -	- -	- -	X
	52	South Shore Beach	- -	- -	- -	- -	X	- -

Table 8 (continued)

Civil Division	Bluff Analysis Section[a]	Location	Observation of Seepage 1986-1987	Soil Boring 1986 and 1988	Observation Well Measurement	Estimate Electrical Resistivity Analysis	Based on Location of Permeable Soil Strata	Groundwater Not Estimated[b]
City of Milwaukee (continued)	53	South Shore Yacht Club	--	--	--	--	--	X
	54	South Shore Park	--	--	--	--	--	X
	55	E. Russel Avenue-Jones Island Sewage Treatment Plant	--	--	--	--	--	X
	56	Marcus Amphitheater-McKinley Marina	--	--	--	--	--	X
	57	McKinley Beach-North Point	--	--	--	--	--	X
	58	Bradford Beach	--	--	--	--	--	X
	59	Lake Park	--	--	--	--	--	X
	60	Linnwood Water Treatment Plant	--	--	--	--	--	X
	61	UW Alumni Center-3052 Newport Court	--	--	--	--	X	--
	62	3378-3474 N. Lake Drive	--	X	--	--	--	--
Village of Shorewood	63	3510 N. Lake Drive	X	--	--	--	--	--
	64	3534 N. Lake Drive	--	--	--	--	X	--
	65	3550-3914 N. Lake Drive	X	--	--	--	--	--
	66	3426 N. Lake Drive	--	--	--	--	--	--
	67	3932-3966 N. Lake Drive	--	--	--	X	X	--
	68	Atwater Park-4300 N. Lake Drive	--	X	--	--	X	--
	69	4308-4320 N. Lake Drive	--	--	--	--	X	--
	70	4400-4408 N. Lake Drive	--	X	X	--	--	--
Village of Whitefish Bay	71	4424-4652 N. Lake Drive	X	--	--	--	--	--
	72	4668-4730 N. Lake Drive	--	--	--	X	--	--
	73	4744-4762 N. Lake Drive	--	--	--	--	X	--
	74	4780 N. Lake Drive	--	--	--	X	--	--
	75	4794-4800 N. Lake Drive	--	--	--	--	--	X
	76	4810-4840 N. Lake Drive	--	--	--	X	--	--
	77	4850 N. Lake Drive-Buckley Park	--	--	--	--	X	--
	78	Buckley Park-Big Bay Park	--	X	--	--	X	--
	79	Big Bay Park-5270 N. Lake Drive	--	--	--	--	X	--
	80	5290 N. Lake Drive	--	--	--	X	X	--
	81	5300 N. Lake Drive-808 Lakeview Avenue	--	--	--	--	X	--
	82	5722-5770 N. Lake Drive	X	--	--	--	--	--
	83	758 E. Day Avenue	--	--	--	--	X	--
	84	740 E. Day Avenue-5866 N. Shore Drive	X	X	--	--	--	--
	85	Klode Park	X	--	--	X	--	--
	86	5960 N. Shore Drive	X	--	--	--	--	--
	87	6000 N. Shore Drive-6260 N. Lake Drive	X	X	--	--	--	--
	88	6310-6424 N. Lake Drive	X	--	--	X	X	--
Village of Fox Point	89	6430-6448 N. Lake Drive	--	--	--	--	X	--
	90	6464-6516 N. Lake Drive	X	X	--	--	X	--
	91	6530-6620 N. Lake Drive	--	--	--	--	X	--
	92	6702 N. Lake Drive-6810 N. Barnett Lane	--	X	X	--	X	--
	93	6818-6840 N. Barnett Lane	--	X	--	X	--	--
	94	6868-6990 N. Barnett Lane	--	--	--	X	--	--
	95	7038-8130 N. Beach Drive	--	--	--	--	--	X
	96	Doctors Park	--	--	--	--	X	--
	97	Audubon Center-9360 N. Lake Drive	--	--	--	--	X	--
	98	1470-1434 E. Bay Point Road	--	--	--	--	X	--
	99	1430 E. Bay Point Road-9364 N. Lake Drive	--	--	--	--	X	--
	100	9400-9578 N. Lake Drive	X	--	--	--	--	--

[a] *As shown on Map 24.*

[b] *No bluff, or bluff obviously stable.*

Source: SEWRPC.

Table 9

AVERAGE MONTHLY AIR TEMPERATURE AT MILWAUKEE: 1951 THROUGH 1987

Month	Average Daily Maximum (°F)	Average Daily Minimum (°F)	Temperature (°F)
January	26.1	11.5	18.9
February	30.7	16.5	23.6
March	30.4	25.3	32.6
April	53.7	35.9	44.8
May	65.0	44.9	55.0
June	75.1	54.9	65.1
July	79.3	61.5	70.5
August	78.3	60.4	69.4
September . . .	71.1	52.8	62.0
October	59.7	41.9	50.8
November . . .	44.8	30.1	37.5
December . . .	32.0	18.3	25.2
Annual	53.8	37.8	46.3

Source: National Weather Service and SEWRPC.

Table 10

AVERAGE MONTHLY PRECIPITATION AT MILWAUKEE: 1951 THROUGH 1987

Month	Average Total Precipitation (inches)	Average Snow and Sleet (inches)
January	1.57	12.7
February	1.45	10.2
March	2.57	9.6
April	3.47	2.2
May	2.83	Trace
June	3.43	0.0
July	3.56	0.0
August	3.46	0.0
September	2.99	Trace
October	2.44	0.2
November	2.29	3.0
December	2.23	11.4
Year	32.29	49.3

Source: National Weather Service and SEWRPC.

The depth and duration of ground frost, or frozen ground, influences hydrologic and soil erosion processes, particularly freeze-thaw activity and the proportion of total rainfall or snowmelt that will run off the land. The amount of snow cover is an important determinant of frost depth. Since the thermal conductivity of snow cover is less than one-fifth that of moist soil, heat loss from the soil to the colder atmosphere is greatly inhibited by the insulating snow cover. Snow cover is most likely during the months of December, January, and February, during which there is at least a 40 percent probability of having one inch or more of snow cover, as measured at the Milwaukee weather station. Frozen ground is likely to exist throughout the study area for approximately four months each winter season, extending from late November through early March, with more than six inches of frost occurring in January, February, and the first half of March. Near-shore portions of Lake Michigan may begin to freeze in December, and ice breakup normally occurs in late March or early April.

Precipitation within the study area takes the form of rain, sleet, hail, and snow, and ranges from gentle showers of trace quantities to brief but intense and potentially destructive thunderstorms or major rainfall-snowmelt events causing severe bluff and beach erosion. Average monthly precipitation for the Milwaukee National Weather Service station is presented in Table 10. The average annual total precipitation in the Milwaukee area was 32.29 inches over the 37-year period from 1951 through 1987. Average total monthly precipitation for the Milwaukee area ranged from 1.45 inches in February to 3.56 inches in July.

Extreme precipitation events may result in massive shoreline losses due to high levels of erosion, seepage, and slumping, A one-hour storm with an expected average recurrence interval of once every two years may be expected to have a total rainfall of about 1.2 inches.[4] A one-hour, 10-year recurrence interval storm may be expected to have a total rainfall of about 1.8 inches; and a 24-hour, 10-year recurrence interval storm may be expected to have a total rainfall of about 3.7 inches. Extended wet periods may result in unusually high coastal losses. Over the period 1841 through 1987, the maximum annual amount of precipitation at Milwaukee was 50.36 inches in 1876, or 56 percent above the 1951 through 1987 annual average. The maximum monthly precipitation

[4]*Bauer, op. cit.*

amount was 10.83 inches, which occurred in June 1917. In late 1986, unusually high levels of precipitation occurring in Milwaukee and throughout the Lake Michigan drainage area resulted in a rapid rise in the level of the lake. A total of 16.08 inches of precipitation fell at Milwaukee during August and September 1986. This period included a rainfall event far more severe than any recorded in the 85 years for which precipitation data have been recorded in the Milwaukee area. On August 6, 1986, about 6.84 inches of rain fell in the 24-hour period.

The presence of Lake Michigan tends to moderate the climate of Milwaukee County. This is particularly true during those periods when the temperature differential between the lake water and the land air masses is the greatest. It is common, for example, for mid-day summer temperatures to be about 10°F lower in shoreline areas than in inland areas because of the cooling lake breezes. Lake Michigan does not have as pronounced an effect on precipitation as it does on temperature. A minor Lake Michigan effect is apparent in the late spring and summer, when there is about 0.5 inch less rainfall per month in coastal areas than in areas farther inland. This difference may be attributed to the cool lake waters maintaining a cooler lower atmosphere which inhibits convective precipitation. However, during the winter, Lake Michigan can serve as a source of moisture, resulting in slightly higher snowfalls near the lake.

Ecological Resources

The biological resources along the Lake Michigan shoreline affect the potential and desired uses of the shoreline, indicate the overall ecological health and stability of the near-shore Lake Michigan environment, and define those environmentally sensitive areas which should be preserved or enhanced when developing shore protection measures. This section describes the fishery resources in the Lake Michigan near-shore area—including the Milwaukee outer harbor; discusses toxic contamination of fish and other aquatic life; identifies important aquatic habitat areas; summarizes endangered resources; and discusses valuable wildlife habitats.

Fishery Resources: Prior to European settlement, the fish communities in Lake Michigan were comprised of native, diverse, and stable stocks of fish. These communities tended to be dominated by two large predators: the lake trout and the burbot. The predator fish were generally larger in size than those present today. Principal forage and prey fish species were ciscoes and white fishes. The appearance of the sea lamprey in the 1930's selectively reduced the already over-exploited stocks of lake trout and burbot to near extinction. The decline of the predators resulted in an explosion of various forage fishes and an unstable, ever-changing fish community.

Further complications arose with the introduction and invasion of two exotic forage species, the rainbow smelt and the alewife, and the introduction of pink, chinook, and coho salmon, and rainbow and brown trout. In spite of these changes, the total fish biomass at the present time is believed to be about the same as in the pre-settlement period. However, the fish populations are generally unstable and changing constantly in response to various stresses. The fish communities at this time are generally of a smaller size, comprised of species more dependent on the pelagic (open water) zone, lack large predators and benthic feeders, and are dominated by opportunistic invaders such as the alewife and rainbow smelt.

To control alewifes, Wisconsin and other states reintroduced predators to Lake Michigan by first controlling sea lamprey reproduction, then restocking native lake trout and species of Pacific salmon. Also, commercial fisherman were encouraged to harvest alewifes. Subsequently, the alewife populations have declined by about 85 percent since the mid-1970's, and the stocked trout and salmon have provided excellent sport fishing. The Department of Natural Resources is studying the abundance of Lake Michigan forage fish in order to assess the State's commercial fishing policy and to resolve conflicts between sport fishermen who favor maintaining high populations of alewife and commercial fisherman who wish to harvest the alewife. The study, to be completed in 1989, will also aid the Department in managing its trout and salmon stocking program.

Extensive fishery surveys were conducted in the outer harbor and near-shore zone of Lake Michigan by the Wisconsin Department of Natural Resources in 1983 as part of the Regional Planning Commission Milwaukee Harbor estuary comprehensive water resources management planning program. The tolerance level, type, and number of fish collected during these surveys are set forth in Tables 11 and 12. Thirty species of fish were captured within the Milwaukee outer

Table 11

TOLERANCE LEVEL, TYPE, AND NUMBER OF FISH COLLECTED DURING THE MILWAUKEE HARBOR ESTUARY FISH SURVEY IN THE OUTER HARBOR: 1983

Tolerance Level	Species	Number	Percent of Subtotal	Percent of Total
Intolerant	Bloater Chub	9	0.5	0.1
	Brook Trout	114	6.8	0.9
	Brown Trout	381	22.7	2.9
	Chinook Salmon	59	3.5	0.4
	Coho Salmon	21	1.2	0.1
	Lake Trout	230	13.7	1.7
	Lake Whitefish	48	2.8	0.4
	Longnose Dace	1	0.1	<0.1
	Rainbow Trout	573	34.1	4.4
	Redhorse	77	4.6	0.6
	Sculpin	66	3.9	0.5
	Spottail Shiner	23	1.4	0.2
	Trout-Perch	79	4.7	0.6
	Subtotal	1,681	100.0	12.8
Tolerant	Alewife	1,719	15.1	13.1
	Black Crappie	3	<0.1	<0.1
	Bluegill	1	<0.1	<0.1
	Gizzard Shad	11	0.1	0.1
	Golden Shiner	7	0.1	0.1
	Lake Chub	1	<0.1	<0.1
	Longnose Sucker	6	0.1	0.1
	Northern Pike	10	0.1	0.1
	Rainbow Smelt	494	4.3	3.8
	Rock Bass	3	<0.1	<0.1
	Walleye	1	<0.1	<0.1
	White Crappie	1	<0.1	<0.1
	White Sucker	3,435	30.1	26.2
	Yellow Perch	5,713	50.1	43.6
	Subtotal	11,405	100.0	87.1
Very Tolerant	Carp	11	68.8	0.1
	Goldfish	2	12.5	<0.1
	Green Sunfish	3	18.7	<0.1
	Subtotal	16	100.0	0.1
	Total	13,102	- -	100.0

Source: Wisconsin Department of Natural Resources.

harbor during the 1983 surveys, of which 13, or 43 percent, were rated as intolerant of pollution; 14, or 47 percent, were rated as tolerant of pollution; and 3, or 10 percent, were rated as very tolerant of pollution. The most abundant fish caught were yellow perch, followed by white sucker, alewife, rainbow trout, rainbow smelt, brown trout, and lake trout. Fish recapture studies indicated that there is little movement of fish between the outer harbor and the Milwaukee

Table 12

TOLERANCE LEVEL, TYPE, AND NUMBER OF FISH COLLECTED DURING THE MILWAUKEE HARBOR ESTUARY FISH SURVEY IN THE NEAR-SHORE ZONE OF LAKE MICHIGAN

Tolerance Level	Species	Number	Percent of Subtotal	Percent of Total
Intolerant	Bloater Chub	1	0.1	<0.1
	Brook Trout	31	2.6	0.2
	Brown Trout	179	14.8	1.4
	Chinook Salmon	71	5.9	0.5
	Coho Salmon	18	1.5	0.1
	Lake Trout	212	17.6	1.6
	Lake Whitefish	6	0.5	0.1
	Longnose Dace	17	1.4	0.1
	Rainbow Trout	284	23.6	2.1
	Redhorse	11	0.9	0.1
	Round Whitefish	1	0.1	<0.1
	Sculpin	335	27.8	2.5
	Spottail Shiner	15	1.2	0.1
	Trout-Perch	24	2.0	0.2
	Subtotal	1,205	100.0	9.1
Tolerant	Alewife	3,629	30.4	27.5
	Black Crappie	7	0.1	0.1
	Bluegill	2	<0.1	<0.1
	Creek Chub	1	<0.1	<0.1
	Fathead Minnow	6	<0.1	<0.1
	Gizzard Shad	30	0.3	0.2
	Golden Shiner	5	<0.1	<0.1
	Lake Chub	10	0.1	0.1
	Longnose Sucker	9	0.1	0.1
	Ninespine Stickleback	13	0.1	0.1
	Northern Pike	13	0.1	0.1
	Rainbow Smelt	410	3.4	3.1
	Rock Bass	17	0.1	0.1
	White Crappie	3	<0.1	<0.1
	White Sucker	1,432	12.0	10.8
	Yellow Perch	6,366	53.3	48.2
	Subtotal	11,953	100.0	90.4
Very Tolerant	Black Bullhead	2	3.3	0.1
	Carp	57	95.0	0.4
	Green Sunfish	1	1.7	<0.1
	Subtotal	60	100.0	0.5
	Total	13,218	--	--

Source: Wisconsin Department of Natural Resources.

River. Thirty-three species of fish were captured within the near-shore zone of Lake Michigan during the 1983 surveys, of which 14, or 42 percent, were rated as intolerant of pollution; 16, or 49 percent, were rated as tolerant of pollution; and 3, or 9 percent, were rated as very tolerant of pollution. The most abundant fish caught were yellow perch, followed by alewife, white sucker, rainbow smelt, sculpin, and rainbow trout.

Toxic Contamination: Environmental contamination by toxic organic substances and metals has become a widespread problem on the Great Lakes over the past 20 years, particularly near established urban areas such as Milwaukee. These toxic substances adversely affect the health of both fish and wildlife, and restrict human use of the aquatic resources. The extent of toxic substance distribution in the water, sediments, and fish of the Great Lakes is only now beginning to be understood.

In assessing the potential effects of toxic substances on the health of fish and other species, it is important to recognize that virtually all species have evolved systems for extracting and concentrating trace elements and compounds from their environment. Some toxic substances now present in the Great Lakes in trace amounts are concentrating and accumulating in the tissue of fish and other animals. Most threatening are those toxic substances which pose a health risk to humans who consume contaminated organisms.

Although more than 800 toxic contaminants have been identified within the Great Lakes,[5] water quality standards have been established only for those 126 substances referred to as "priority pollutants." The priority pollutants, designated by the U. S. Environmental Protection Agency, are in common use or are prevalent in the environment. Of these priority pollutants, a few substances have received the greatest attention with respect to fish consumption. U. S. Food and Drug Administration health standards for consumption of fish have been established for polychlorinated biphenyls (PCB's), DDT, toxaphene, chlordane, dieldrin, dioxin, and mercury.

With respect to human health, the greatest concerns have been related to the consumption of PCB-contaminated fish tissue. PCB's, which prior to 1976 were widely used in electrical equipment and other industrial applications, accumulate in bottom sediments and in the tissue of fish. The primary source of PCB's for Lake Michigan fish is their diet, through a process referred to as biomagnification. Fish can also uptake PCB's directly from the water as it passes over their gills—referred to as bioconcentration.

[5]*International Joint Commission, 1983 Report on Great Lakes Water Quality, Great Lakes Water Quality Board, 1983.*

Table 13

MEAN POLYCHLORINATED BIPHENYL CONCENTRATIONS IN SOUTHERN LAKE MICHIGAN SALMONIDS: 1985

Fish Species	Number of Samples	Mean PCB Concentration (ug/g)
Brook Trout	7	1.01
Rainbow Trout	20	0.61
Brown Trout	42	2.09
Lake Trout	82	4.03
Coho Salmon	58	0.88
Chinook Salmon	120	1.10

NOTE: The U. S. Food and Drug Administration's health standard for PCB's is 2.0 micrograms per gram (ug/g).

Source: Wisconsin Department of Natural Resources.

In 1985, the Wisconsin Department of Natural Resources analyzed the tissue of 791 individual fish of six species of salmonids for concentrations of PCB's.[6] The study results, summarized in Table 13, indicated that certain species exceeded the recommended health standards. The mean PCB concentration measured in lake trout and brown trout exceeded the U. S. Food and Drug Administration's health standard of 2.0 micrograms per gram (ug/g). Brook trout, coho salmon, and rainbow trout live only two growing seasons in Lake Michigan and rarely exceeded the health standard. The study concluded that the level of PCB contamination is a function of the size of the fish, their habitat, the fat content of the fish, and the season. Seasonal and spatial variations in PCB concentrations were observed.

PCB, dieldrin, and chlordane concentrations measured in the tissue of fish during 1986 and 1987 are summarized in Table 14.[7] Three chi-

[6]*Robert G. Masnado, Polychlorinated Biphenyl Concentrations of Eight Salmonid Species from the Wisconsin Waters of Lake Michigan: 1985, Wisconsin Department of Natural Resources Fish Management Report 132, February 1987.*

[7]*Wisconsin Department of Natural Resources, "Organics Data for Lake Michigan Since 1986," Unpublished Data, June 1988.*

Table 14

MEAN MEASURED CONCENTRATIONS OF PCB, DIELDRIN, AND CHLORDANE IN THE TISSUE OF LAKE MICHIGAN FISH: 1986-1987

Fish Species	PCB (ug/g)	Dieldrin (ug/g)	Chlordane (ug/g)
Alewife	0.64	0.07	0.01
Bloater Chub	0.82	0.13	0.01
Horned Sculpin	0.64	0.20	0.21
Whitefish	0.76	0.12	- -
Yellow Perch	0.27	- -	- -
Brown Trout	0.20	0.02	- -
Chinook Salmon	2.70	0.11	0.18
Lake Trout	1.62	0.12	0.29

NOTE: The FDA health standard for PCB's is 2.0 micrograms per gram (ug/g), for Dieldrin is 0.3 ug/g, and for Chlordane is 0.3 ug/g.

Source: Wisconsin Department of Natural Resources.

nook salmon were the only fish that exceeded the health standard of 2.0 ug/g for PCB's. No fish exceeded the health standards for dieldrin or for chlordane, the standard being 0.3 ug/g for both substances.

Based on PCB measurements in the tissue of fish, the Wisconsin Department of Natural Resources, along with the Wisconsin Division of Health, issued a health advisory in April 1988 for persons who consume fish caught in Wisconsin waters. The advisory recommended that, because of PCB contamination, no one eat very large lake trout, brown trout, or chinook salmon; carp; or catfish caught in Lake Michigan or the Milwaukee outer harbor. Furthermore, consumption of crappie, northern pike, redhorse, and smallmouth bass caught in the Milwaukee outer harbor was not recommended. The advisory also noted that small lake trout, coho salmon, and chinook salmon; brook and rainbow trout; pink salmon; rainbow smelt; and perch pose the lowest health risk.

Some studies have indicated that the concentrations of certain organic substances in the tissue of Lake Michigan fish have declined since the 1970's as the use of these substances has declined and the substances have been flushed from the lake or buried by cleaner sediments.[8,9] Figure 15 shows that coho salmon tissue concentrations of PCB, dieldrin, and DDT declined substantially over the period 1980 through 1984. Similarly, Figure 16 illustrates a decline in PCB, DDT, dieldrin, and oxychlordane concentrations in lake trout caught in Lake Michigan from the early 1970's through 1982. These declines have continued to occur.[10]

Toxic contaminants have also been measured in the tissue of fish caught in the Milwaukee outer harbor. Concentrations of 12 toxic organic substances and four metals were measured in the tissue of fish taken from the outer harbor in May 1970 and in August 1983. The results of the fish tissue toxic surveys are set forth in Table 15. PCB concentrations were generally higher than those found in Lake Michigan, and often exceeded the U. S. Food and Drug Administration's health standards. However, health standards for DDT, dieldrin, and mercury were not exceeded in the tissue of outer harbor fish.

An important source of toxic substances contained in the tissue of fish is believed to be contaminated bottom sediments. Toxic substances deposited in the bottom sediments may be released to the water column and may accumulate in the tissue of organisms which feed on the bottom substrate. Sediments composed of fine materials, such as silt and clay, have an affinity for adsorption and absorption of certain metals and organic substances. Presently there are few data available on the quality of the sediments within the near-shore zone of Lake Michigan in Milwaukee County. Limited investigations of the bottom characteristics of the near-shore zone indicate that the sediments are composed pri-

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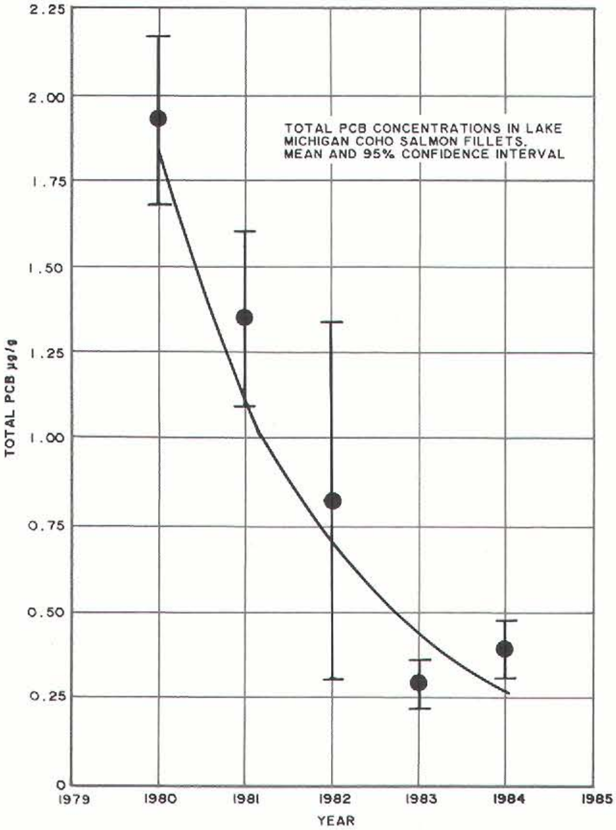
[8]*David S. DeVault, Wayne A. Willford, et al., "Contaminant Trends in Lake Trout (Salvelinus namaycush) from the Great Lakes," Archives of Environmental Contamination and Toxicology, Vol. 15, 1986, pp. 349-356.*

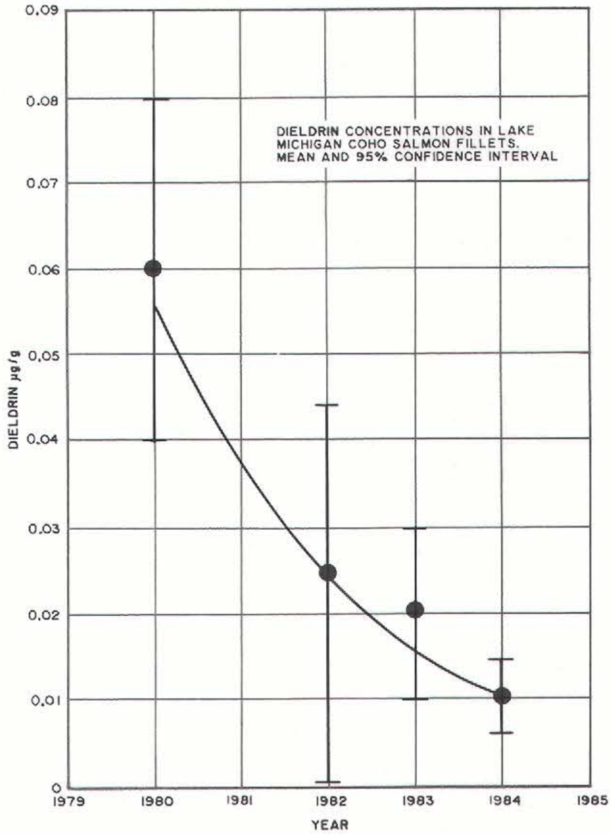
[9]*David S. DeVault, J. Milton Clark, et al., "Contaminants and Trends in Fall Run Coho Salmon," Journal of Great Lakes Research, Vol. 14, No. 1, 1988, pp. 23-33.*

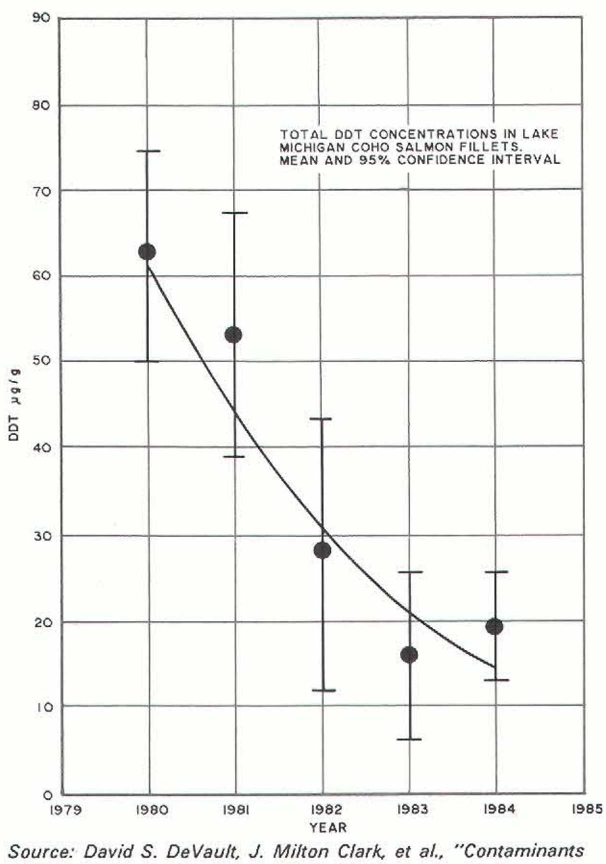
[10]*David S. DeVault, Personal Communication, June 8, 1988.*

Figure 15

CONTAMINANT TRENDS IN THE TISSUE OF LAKE MICHIGAN COHO SALMON: 1980-1984







Source: David S. DeVault, J. Milton Clark, et al., "Contaminants and Trends in Fall Run Coho Salmon," Journal of Great Lakes Reseach, Vol. 14, No. 1, 1988, pp. 23-33.

marily of sand, with localized patches of silty organic substrate. A study conducted in 1977 determined the surficial sediment distribution of the near-shore area of Lake Michigan adjacent to the Milwaukee Harbor and extending southward to the abandoned Wisconsin Electric Power Company's Lakeside power plant.[11] As shown on Map 6, the sediments were generally composed of sand and gravel, with areas of silt located at the harbor entrances.

The two major sources of sediment to the Lake Michigan coastal system are shoreline erosion and river loadings. In general, it is believed that sediments contributed to the lake by shoreline erosion do not contain significant amounts of toxic contaminants, since the majority of the eroded sediments are comprised of natural beach and bluff material. There is, however, some concern about the quality of the material which in the past has been used in landfilling projects for shoreline protection and land creation, and

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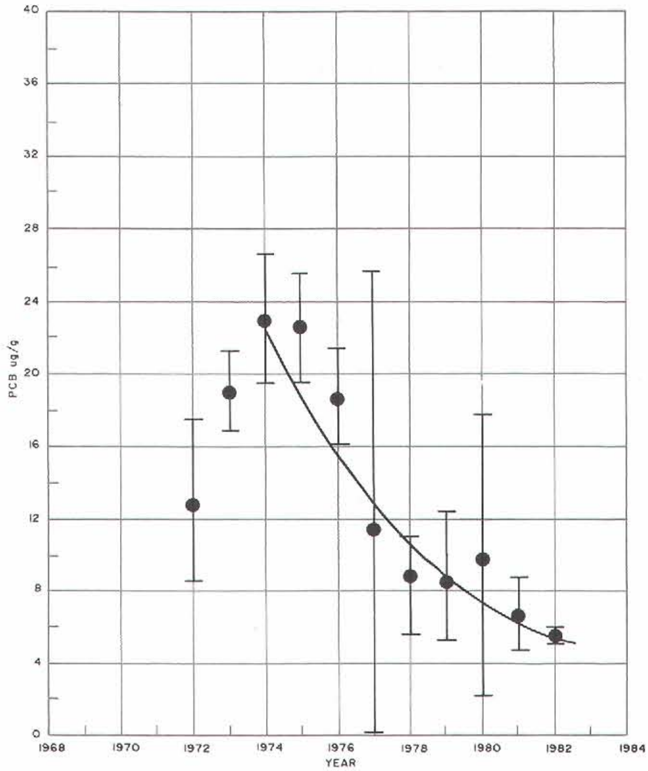
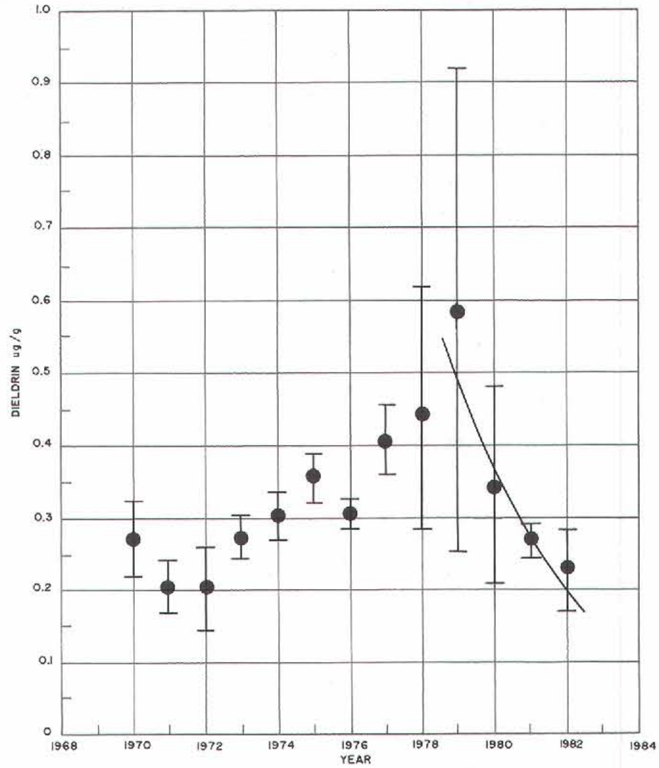
[11] *Carol J. Welkie, "Geophysical-Geological Exploration for Offshore Sand and Gravel, Western Lake Michigan," Doctor of Philosophy Dissertation, University of Wisconsin-Madison, 1980.*

Figure 16

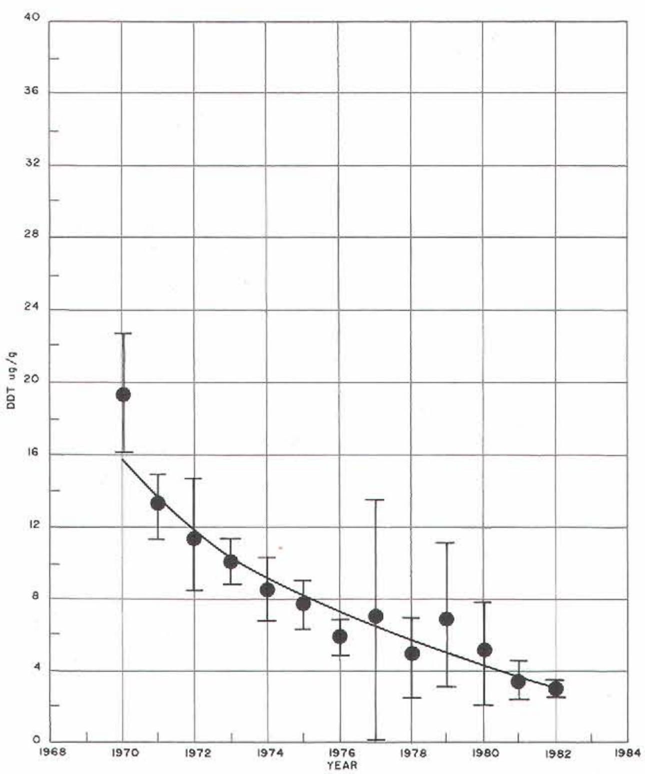
CONTAMINANT TRENDS IN THE TISSUE OF LAKE MICHIGAN LAKE TROUT: 1970-1982



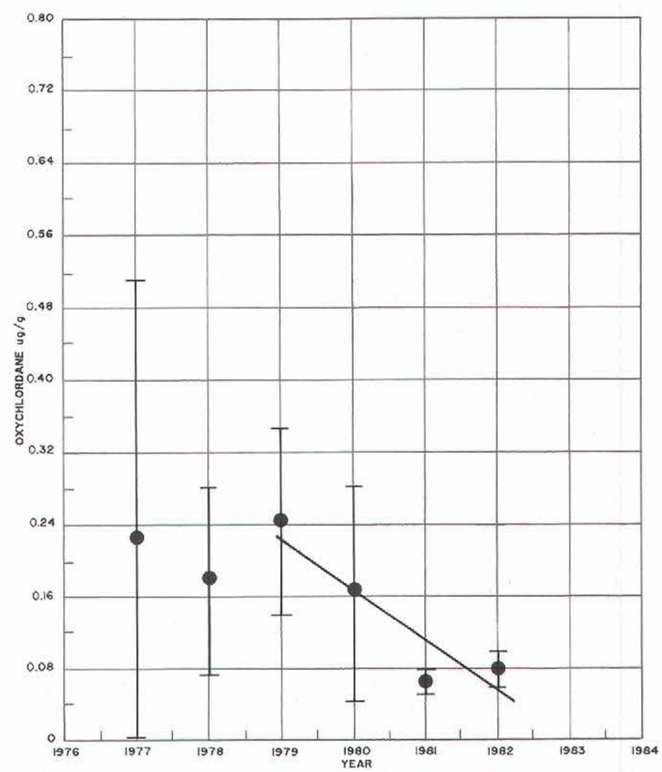
POLYCHLORINATED BIPHENYL (PCB) CONCENTRATIONS IN LAKE MICHIGAN LAKE TROUT. MEAN AND 95% CONFIDENCE INTERVAL.



DIELDRIN CONCENTRATIONS IN LAKE MICHIGAN LAKE TROUT. MEAN AND 95% CONFIDENCE INTERVAL.



TOTAL DDT CONCENTRATIONS IN LAKE MICHIGAN LAKE TROUT. MEAN AND 95% CONFIDENCE INTERVAL.



OXYCHLORDANE CONCENTRATIONS IN LAKE MICHIGAN LAKE TROUT. MEAN AND 95% CONFIDENCE INTERVAL.

Source: David S. DeVault, J. Milton Clark, et al., "Contaminants and Trends in Fall Run Coho Salmon," Journal of Great Lakes Research, Vol. 14, No. 1, 1988, pp. 23-33.

Table 15

CONCENTRATIONS OF TOXIC ORGANIC SUBSTANCES AND METALS IN THE TISSUE OF FISH IN THE OUTER HARBOR: 1970-1983

	Alewife			White Sucker			Yellow Perch			Brown Trout			Coho Salmon			Miscellaneous Species		
Toxic Substance	Number of Samples	Range	Mean	Number of Samples	Range	Mean	Number of Samples	Range	Mean	Number of Samples	Range	Mean	Number of Samples	Range	Mean	Number of Samples	Range	Mean
Dates of Sampling	August 2, 1983 to August 23, 1983			May 20, 1970 to August 23, 1983			August 2, 1983 to August 23, 1983			August 4, 1983 to August 24, 1983			May 20, 1970			May 20, 1970		
Organics																		
PCB's	2	1.1-2.9	2.0	2	3.4-4.2	3.8	2	2.2-2.7	2.4	2	2.7-3.8	3.2	--	--	--	--	--	--
Aldrin	2	<0.05	<0.05	2	<0.05	<0.05	2	<0.05	<0.05	2	<0.05	<0.05	--	--	--	--	--	--
Dieldrin	2	0.04-0.06	0.05	2	<0.02	<0.02	2	0.3-0.05	0.04	2	0.15-0.17	0.16	--	--	--	--	--	--
Endrin	2	<0.02	<0.02	2	<0.02	<0.02	2	<0.02	<0.02	2	<0.02	<0.02	--	--	--	--	--	--
DDT	2	0.17-0.29	0.23	2	0.17-0.18	0.18	2	0.21-0.28	0.24	2	0.70-0.84	0.77	--	--	--	--	--	--
Chlordane	2	<0.05-0.05	0.02	2	<0.05	<0.05	2	<0.05	<0.05	2	0.10-0.17	0.14	--	--	--	--	--	--
Hexachlorobenzene	2	<0.01	<0.01	2	<0.05	<0.05	2	<0.05	<0.05	2	<0.05	<0.05	--	--	--	--	--	--
Hexachlorocyclohexane	2	<0.01	<0.01	2	<0.01	<0.01	2	<0.01	<0.01	2	<0.01	<0.01	--	--	--	--	--	--
Heptachlor	2	<0.05	<0.05	2	<0.05	<0.05	2	<0.05	<0.05	2	<0.05	<0.05	--	--	--	--	--	--
Methoxychlor	2	<0.05	<0.05	2	<0.05	<0.05	2	<0.05	<0.05	2	<0.05	<0.05	--	--	--	--	--	--
Pentachloroanisol	2	<0.05	<0.05	2	<0.05	<0.05	2	<0.05	<0.05	2	<0.05	<0.05	--	--	--	--	--	--
Toxaphene	2	<1.0	<1.0	2	<1.0	<1.0	2	<1.0	<1.0	2	<1.0	<1.0	--	--	--	--	--	--
Metals																		
Chromium	2	<0.5	<0.5	3	<0.5-0.42	0.14	2	<0.5	<0.5	2	<0.5	<0.5	--	--	--	--	--	--
Copper	2	1.7-1.8	1.8	2	1.4-1.5	1.4	2	1.2-1.4	1.3	2	1.9	1.9	--	--	--	--	--	--
Mercury	2	0.03-0.05	0.04	2	0.03	0.03	2	0.06-0.08	0.07	2	0.08-0.12	0.10	--	--	--	3	0.05-0.22	0.13
Zinc	--	--	--	1	--	6.9	--	--	--	--	--	--	1	--	4.6	--	--	--

NOTE: All concentrations are in parts per million.

Source: Wisconsin Department of Natural Resources.

about material placed in waste disposal sites located close to the shoreline which may be seeping or eroding into the lake. Although currently only "clean" material—mostly soil and concrete rubble—is allowed to be used as fill, some metal and other foreign substances are occasionally included. Furthermore, even some of the soils used in the landfill projects may contain contaminants from urban land uses and atmospheric deposition such as accumulated lead from automobile exhaust. In addition, clean material was not used for some of the older landfill projects. Particularly near some of the industrial sites in the City of Oak Creek, industrial waste material and other unknown substances have been placed on the bluff slopes to help stabilize the slopes. Waste material also has been stored on top of the bluff near the shoreline in these areas. Some of this material may wash into the lake. The impact of these contaminants on the ecology of near-shore Lake Michigan has not been evaluated.

One such industrial site is the former Allis Chalmers property in the southwest one quarter of U. S. Public Land Survey Section 24, Township 5 North, Range 22 East, in the City of Oak Creek. The site formerly contained production plants owned by E. J. du Pont de Ne Mours and Company, and the Newport Chemical Company.[12] As shown in Figure 17, waste material and demolition debris have been buried on the property. Considerable work has been done on the site to treat soils and debris contaminated with an organic substance, napthylamine, and other pollutants. As shown in Figure 18, concrete rubble and other demolition debris has been placed on the bluff slope to protect against shoreline erosion. It is possible that some organic pollutants continue to erode or seep into the lake from this site. Adjacent to this industrial property are similar sites where industrial waste has been placed at the shoreline.

A second known landfill site which may contain toxic and hazardous materials includes a gully at E. Drexel Avenue extended and adjacent shoreline area in the northwest one quarter of

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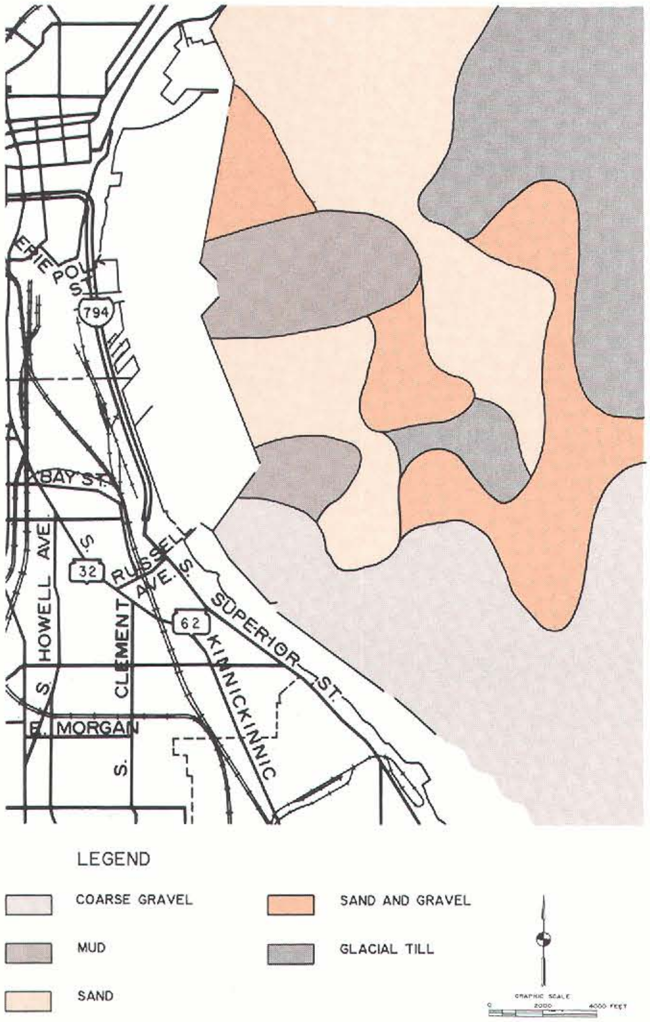
[12]*Wisconsin Department of Natural Resources, Active and Abandoned Landfills in Wisconsin, Bureau of Solid and Hazardous Waste, April 1988; and Southeast District file records.*

Map 6

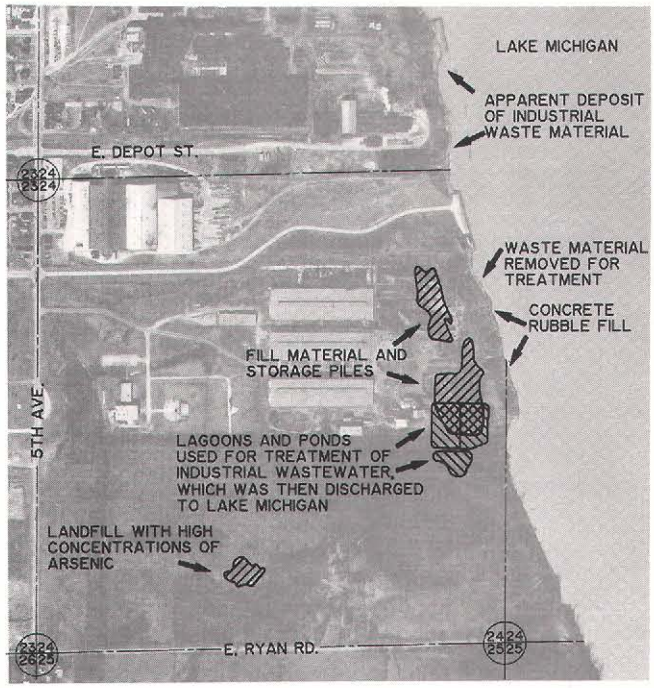
DISTRIBUTION OF BOTTOM SEDIMENT TEXTURE CLASSES IN THE NEAR-SHORE AREA OF LAKE MICHIGAN



Source: Carol J. Welkie, "Geophysical-Geological Exploration for Offshore Sand and Gravel, Western Lake Michigan," Doctor of Philosophy Thesis, University of Wisconsin-Madison, 1980.

Figure 17

KNOWN DEPOSITS OF WASTE MATERIAL WHICH MAY CONTAIN TOXIC SUBSTANCES NEAR THE SHORELINE OF SECTION 24, TOWNSHIP 5 NORTH, RANGE 22 EAST, CITY OF OAK CREEK



Source: Wisconsin Department of Natural Resources and SEWRPC.

U. S. Public Land Survey Section 13, Township 5 North, Range 22 East, in the City of South Milwaukee. This site is shown in Figures 18 and 19. Although it has been reported that barrels have been buried on the site, the origin, contents, and number of barrels is unknown. There has been no evidence that waste material at this site has actually eroded into the lake.

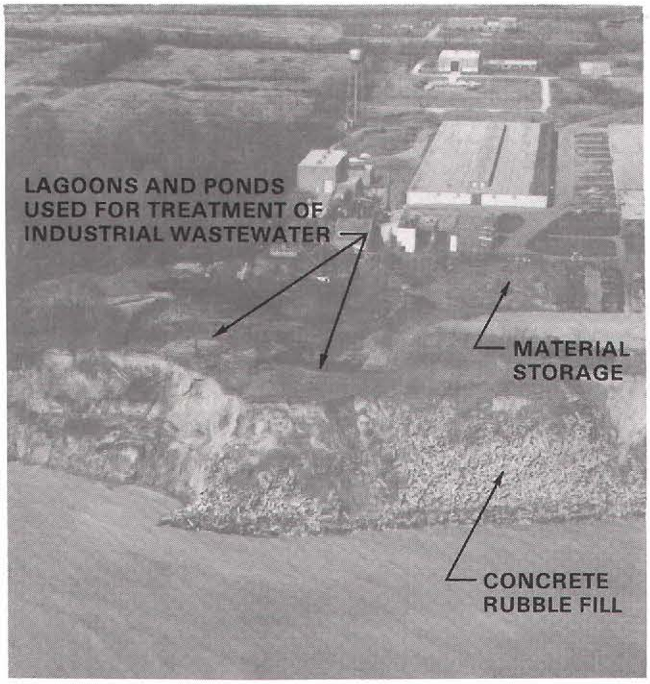
A third known landfill site which may contain toxic and hazardous materials is the Manke Dump, located on top of the bluff at Ramsey Avenue extended, in Warnimont Park, in the northwest one quarter of U. S. Public Land Survey Section 36, Township 6 North, Range 22 East, in the City of Cudahy. That landfill was in operation from at least 1950 until about 1963.[13] In some portions of the landfill, about 50 feet of waste material is covered by about 20 feet of soil. The landfill reportedly contains paints, dyes, resins, lacquers, foundry sand, castings, automobile bodies, empty barrels, broken concrete, vegetation, and construction debris. As shown in Figures 18 and 20, the bluff near the location of the former Manke Dump is actively eroding. There has been no evidence that the waste material at this site has actually eroded into the lake.

The quality of sediments discharged by inflowing rivers is different from most shoreline material. The finer grained particles contained in stormwater runoff tend to adsorb pollutants during transport and can contain high concentrations of nutrients, metals, and organic con-

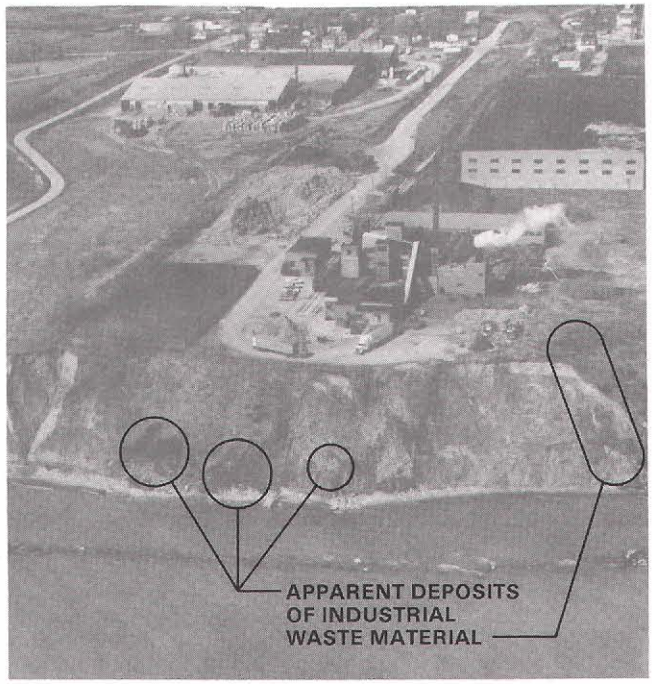
[13] *Ibid.*

Figure 18

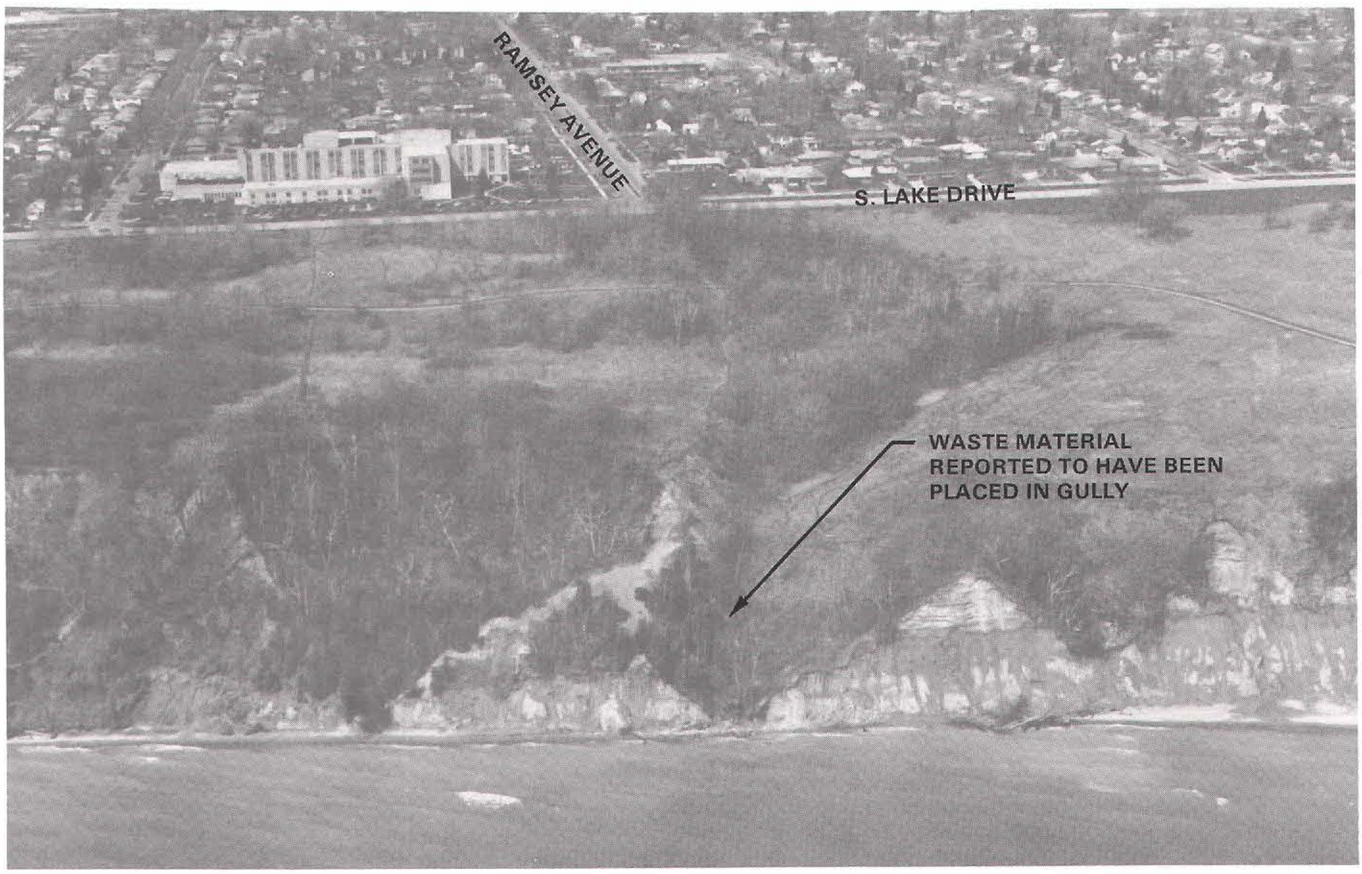
KNOWN LOCATIONS WHERE INDUSTRIAL WASTE MATERIAL WHICH MAY CONTAIN TOXIC AND HAZARDOUS SUBSTANCES MAY ERODE OR SEEP INTO LAKE MICHIGAN



FORMER ALLIS CHALMERS CO. PROPERTY, CITY OF OAK CREEK



INDUSTRIAL PROPERTY NEAR E. DEPOT ROAD EXTENDED, CITY OF OAK CREEK

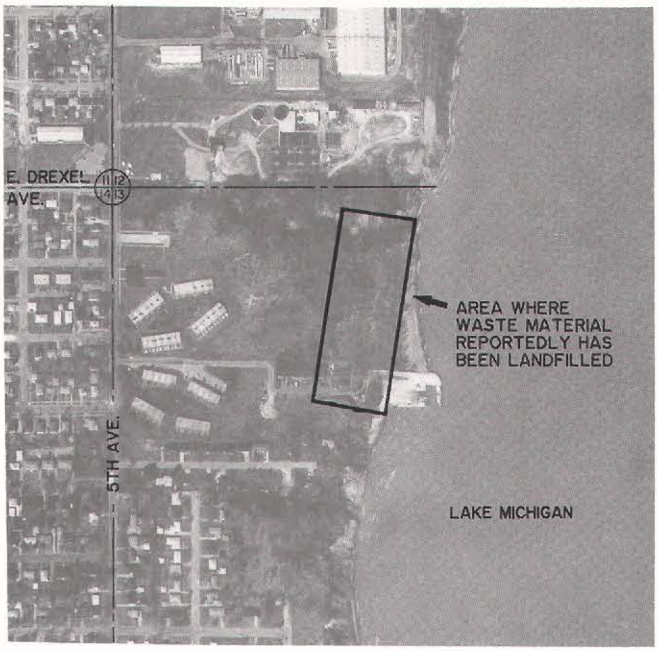


MANKE DUMP, WARNIMONT PARK AT RAMSEY AVENUE EXTENDED, CITY OF CUDAHY

Source: SEWRPC.

Figure 19

KNOWN DEPOSITS OF WASTE MATERIAL WHICH MAY CONTAIN TOXIC SUBSTANCES NEAR THE SHORELINE OF SECTION 13, TOWNSHIP 5 NORTH, RANGE 22 EAST, CITY OF SOUTH MILWAUKEE



Source: Wisconsin Department of Natural Resources and SEWRPC.

Figure 20

KNOWN DEPOSITS OF WASTE MATERIAL WHICH MAY CONTAIN TOXIC SUBSTANCES NEAR THE SHORELINE OF SECTION 36, TOWNSHIP 6 NORTH, RANGE 22 EAST, CITY OF CUDAHY

Source: Wisconsin Department of Natural Resources and SEWRPC.

taminants of both urban and rural origin. In Milwaukee County, the major rivers which flow into Lake Michigan are the Milwaukee, Menomonee, and Kinnickinnic Rivers, which discharge through the Milwaukee Harbor; and Oak Creek, which enters Lake Michigan at the southern end of Milwaukee County at Grant Park in the City of South Milwaukee. A large portion of the transported sediments is deposited near the mouth of the rivers. As discussed in SEWRPC Planning Report No. 37, A Water Resources Management Plan for the Milwaukee Harbor Estuary, 1987, the bottom sediments of the Milwaukee inner and outer harbors contain moderate to high levels of toxic contaminants. The impact of these contaminants on Lake Michigan is unknown at this time, but further studies have been proposed to define and address the problem, as may be found necessary.

Aquatic Habitat: The aquatic habitat is an important element of biological communities, and consists of both biotic—or organic—and abiotic—or inorganic—factors. Changes or stresses to the abiotic environment may actually be more damaging and enduring to the ecosystem than those to the biotic sector alone, since the biota may respond and recover more quickly. Aquatic habitats indicate the overall quality, or health, of the ecosystem, and constitute essential components of energy and material cycles. The most valuable habitats, located in the littoral zone, provide food and shelter for both vertebrates and invertebrates and spawning and nursery areas for many fish species. Although many factors affect the quality of the habitat, the type of bottom substrate and the extent of submergent and emergent vegetation are usually among the most important. A detailed inventory of the aquatic habitat within the near-shore zone of Lake Michigan has not been conducted.

The aquatic habitat in the Milwaukee outer harbor was evaluated by the Wisconsin Department of Natural Resources in 1984.[14] Overall, the Department evaluation concluded that while the water quality of the outer harbor varies substantially because of the high rate of exchange of water between the outer harbor and Lake Michigan, and the large loadings of pollutants from the inner harbor and the Jones Island wastewater treatment plant, habitat conditions are generally satisfactory to maintain propagation of warmwater fish and other aquatic life.

[14] *Wisconsin Department of Natural Resources, "Review of Water Quality Standards for the Outer Harbor at Milwaukee and the Nearshore Waters of Lake Michigan," 1984.*

The evaluation concluded that the substrate and habitat of the Milwaukee outer harbor are not conducive to the maintenance of self-sustaining salmonid—or coldwater—populations. Of the salmonid species indigenous to this part of Lake Michigan, only lake trout and brown trout have been documented as spawning successfully in Lake Michigan itself, and then only in the open lake environment on rocky, reef-like structures. Any other salmonid species that may be present naturally migrate up streams and require free-flowing areas with clean gravel substrates and cool water for successful reproduction. These required spawning areas are not present within the Milwaukee River.

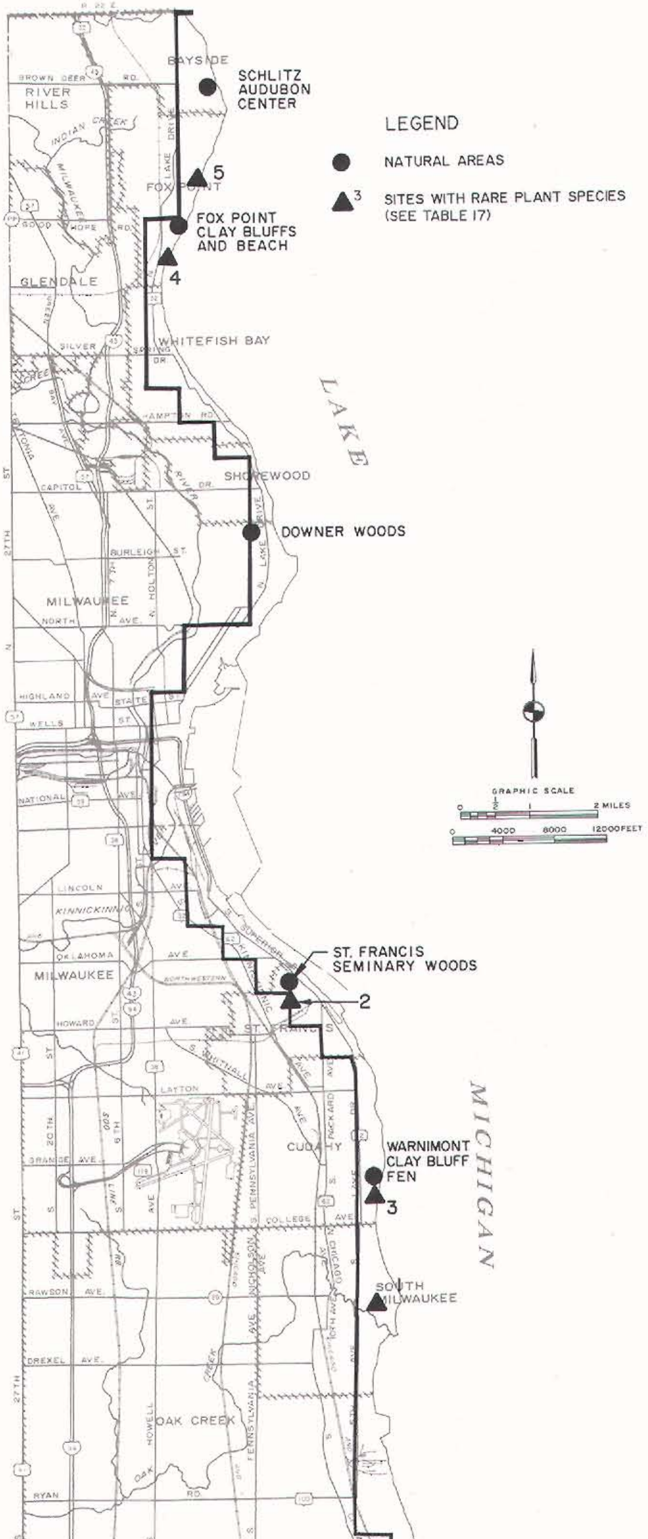
Substrate characteristics and the habitat in some portions of the outer harbor were, however, found to be conducive to the successful propagation of warmwater sport fish and a variety of indigenous forage species. Desirable substrate areas found were comprised of sand and rubble and of macrophyte beds, both of which provide spawning substrate and cover for a variety of fish species and food organisms. Bottom scouring is rare within the outer harbor, and some substrates do not have substantial accumulations of fine-grained organic material.

Endangered Resources: The Wisconsin Department of Natural Resources, Bureau of Endangered Resources, reviewed the study area and identified five natural areas and five sites where rare or endangered plant species have been identified. These natural areas and rare plant sites are shown on Map 7 and described in Tables 16 and 17.

Natural areas are defined by the Wisconsin Scientific Areas Preservation Council as tracts of land and water so little modified by human activities, or sufficiently recovered from such activities, that they contain native plant and animal communities believed to be representative of pre-settlement conditions. The five identified natural areas have a combined areal extent of about 369 acres. These official natural areas were selected by the Wisconsin Department of Natural Resources on the basis of the quality, uniqueness, diversity, size, and educational value of the natural resources. In addition to the official natural areas listed in Table 16, there are other sites of important natural vegetation along the shoreline, particularly in some of the less developed southern Milwaukee

Map 7

ENDANGERED RESOURCES



Source: Wisconsin Department of Natural Resources, Bureau of Endangered Resources.

Table 16

NATURAL AREAS ALONG THE LAKE MICHIGAN SHORELINE OF MILWAUKEE COUNTY: 1988

Area Name	U. S. Public Land Survey Location	Areal Extent (acres)	Features	Description
Downer Woods	T7N R22E Section 10	15	Southern dry mesic forest	This site is dominated by large, open-grown bur and white oaks which overtop a young forest of white ash, hawthorn, and basswood. Choke cherry, dogwoods, and several exotic species (notably honeysuckle) form a dense shrub layer
Fox Point Clay Bluffs and Beach	T8N R22E Sections 9, 16, 21, and 28	100	Bluffs, beach, and beach ridge	This stretch of Lake Michigan coast features a naturally nourished beach and offshore sand bars in Sections 21 and 28. A classic example of a terraced shoreline is present in Sections 9 and 16. The eroding clay banks above the terrace support several regionally uncommon plant species, including buffalo berry, bush honeysuckle, snowberry, white cedar, and yew
St. Francis Seminary Woods	T6N R22E Sections 14 and 15	50	Southern mesic forest	This southern mesic forest features old growth basswood, sugar maple, American beech, red oak, and paper birch. Cottonwood and willow trees grow along a stream which traverses the tract. The spring flora is fairly diverse. Disturbance factors include past cutting, a gravel road, and many exotic plantings. The site is notable for the presence of the state-endangered blue-stemmed goldenrod (Solidago caesia). The woods is separated from the Lake Michigan shore by STH 32
Schiltz Audubon Center	T8N R22E Sections 9 and 10	164	Prairie, bluffs, and lake terrace	This site contains a nature center, a prairie restoration tract, a wooded ravine, bluffs, and a lake terrace. Important plant species include yellow lady slipper, cream gentian, blue-stemmed goldenrod, buffalo berry, and big blue stem
Warnimont Clay Bluff Fen	T6N R22E Section 36	40	Fen, springs, and bluffs	This site features clay bluffs along Lake Michigan with spring seepages discharging from the base of the bluffs. Fen-line meadows support an unusual flora containing several rare or regionally uncommon plants, including buffalo berry, variegated scouring rush, purple false oats, and false asphodel, a threatened species in Wisconsin. Other plants occurring are Ohio goldenrod, grass of Parnassus, slender bog arrow-grass, small fringed gentian, northern bog orchid, and white cedar

Source: Wisconsin Department of Natural Resources, Bureau of Endangered Resources.

Table 17

SITES CONTAINING RARE PLANT SPECIES ALONG THE LAKE MICHIGAN SHORELINE OF MILWAUKEE COUNTY: 1988

Number on Map 7	U. S. Public Land Survey Location	Rare Plant Species		
		State Endangered	State Threatened	Of Special Concern
1	T5N R22E Sections 1 and 12	Solidago caesia (blue-stemmed goldenrod)	Tofieldia glutinosa (false asphodel)	- -
2	T6N R22E Section 14	Solidago caesia (blue-stemmed goldenrod)	- -	- -
3	T6N R22E Section 36	Solidago caesia (blue-stemmed goldenrod)	Tofieldia glutinosa (false asphodel)	Equisetum variegatum (variegated horsetail); Trisetum melicoides (purple false oats); Solidago ohioensis (Ohio goldenrod); and Triglochin palustre (slender bog arrow grass)
4	T8N R22E Section 21	- -	Tofieldia glutinosa (false asphodel)	- -
5	T8N R22E Section 16	- -	Trillium nivale (snow trillium)	- -

Source: Wisconsin Department of Natural Resources, Bureau of Endangered Resources.

County parks. However, an inventory of all natural areas in Milwaukee County has not been completed.

Endangered species are species whose existence in the State is in jeopardy. Threatened species are species which appear likely in the foreseeable future to become endangered in the State. Species of special concern are those about which some problem of abundance or distribution is suspected but not yet known. A total of one state endangered plant species—Solidago caesia, two state threatened plant species—Tofieldia glutinosa and Trillium nivale, and four plant species of special concern—Equisetum variegatum, Trisetum melicoides, Solidago ohioensis, and Triglochin palustre, have been identified in the study area.

Wildlife Habitat: Many of the shoreline bluffs, parks, and other open areas constitute significant wildlife habitat areas. Because of its location along the Mississippi flyway, the study area provides important habitat for migrating birds. A total of 900 acres of wildlife habitat, or 12 percent of the study area, have been identified within the study area and value rated, as shown on Map 8. Class I, or high-value, wildlife habitat areas encompass 165 acres, or 18 percent of the total wildlife habitat area. Class II, or medium-value, wildlife habitat areas cover 330 acres, or 37 percent of the total area; and Class III, or low-value, wildlife habitat areas cover the remaining 405 acres, or 45 percent of the total area. Of the total wildlife habitat area, about 17 acres, or 2 percent, consist of wetlands; 535 acres, or 59 percent, consist of upland forest; 180 acres, or 20 percent, consist of grass land; 108 acres, or 12 percent, consist of mixed vegetation; and 60 acres, or 7 percent, is open surface water.

MAN-MADE FEATURES

This section describes the historical development of the Lake Michigan shoreline in Milwaukee County. In addition, an understanding of the existing civil divisions, land use patterns, and zoning regulations is essential to the formation of practical shoreline management guidelines.

Historical Shoreline Development

The first permanent European settlement in the study area was a trading post established in 1795 on the east side of the Milwaukee River, just north of what is now Wisconsin Avenue. Urban development in the Milwaukee area was well underway by the 1830's. Initially, the Lake Michigan shoreline was primarily devoted to the handling of waterborne commerce, with later shoreline development being for boating facilities, residential use, industrial use, and park and open space. The recent emphasis on the impacts of fluctuating water levels and the benefits of certain types of shore protection measures has not obscured the fact that Milwaukee County residents have been attempting—with varying degrees of success—to protect the shoreline from the erosive effects of storm waves and ice action since the early 1800's. While large investments have been made to protect certain facilities and land uses, other shoreline areas have remained unprotected, or minimally protected.

This section addresses the historic urban growth pattern along the shoreline, historical places, the development of the lakefront park system, and shoreline uses. The development of the Milwaukee Harbor is also discussed.

Urban Growth: Completion of the U. S. Public Land Survey in southeastern Wisconsin by 1836 brought many settlers to the Milwaukee area. By 1850, much of what is now the downtown area of Milwaukee was developed. Map 9 illustrates the urban development of the shoreline area between 1850 and 1985. The areas developed are quantified in Table 18. The percent increase in urban development of the shoreline was highest between 1850 and 1880, between 1900 and 1920, and between 1920 and 1940. Relatively little new urban development has occurred since 1970.

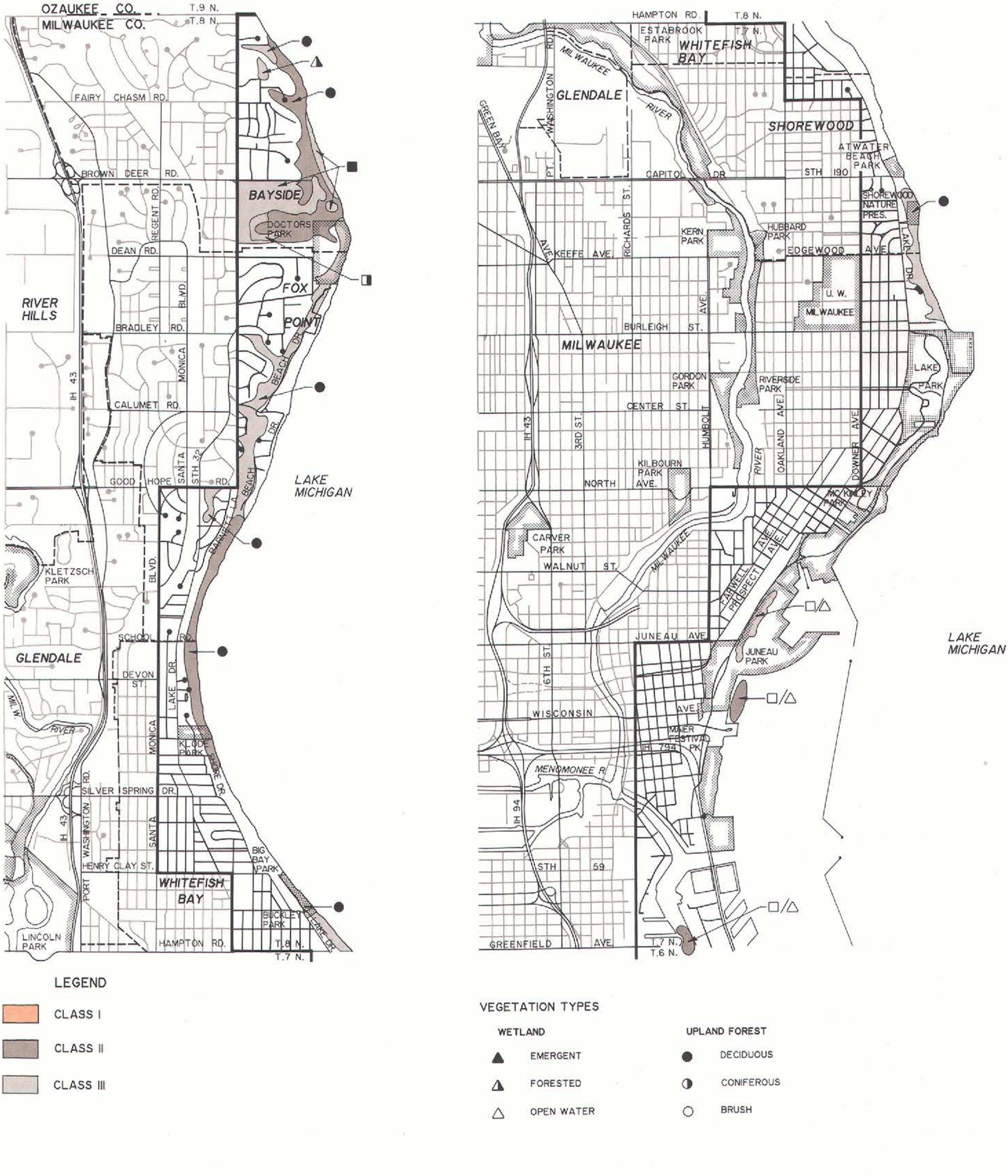
Although urban development has progressed both northward and southward from the initial Milwaukee settlement, the southern county shoreline has a much different character than the northern county shoreline. First, about 57 percent of the immediate shoreline south of the Milwaukee Harbor is parkland, while only 24 percent of the shoreline north of the harbor is parkland. Second, the southern portion has less land devoted to residential use than the northern portion, with 4 percent and 68 percent of the immediate shoreline length south and north of the harbor being in residential use, respectively. Third, the shoreline south of the harbor contains more major public or quasi-public facilities—the Wisconsin Electric Power Company Oak Creek power plant; the Oak Creek, Cudahy, and Milwaukee Texas Street water intake plants; and the South Shore and South Milwaukee sewage treatment plants—than the shoreline north of the harbor—which contains only the Milwaukee Linnwood Avenue water treatment plant and the North Shore Water Commission water intake plant. Overall, a smaller portion of the shoreline south of the harbor is protected by shore protection measures, compared to the shoreline north of the harbor.

Historic Places: Historic sites and districts within the study area often have important recreational value, as well as educational and cultural values. Historic preservation helps retain those elements that give an area a distinctive identity, and may provide tangible benefits, such as stabilization of property values and encouragement of overall neighborhood improvement. Certain measures to protect the shoreline from wave erosion, storm damage, and bluff erosion, unless sensitively done, may adversely affect the aesthetic qualities, vistas, and shoreline uses historically and traditionally enjoyed by area residents.

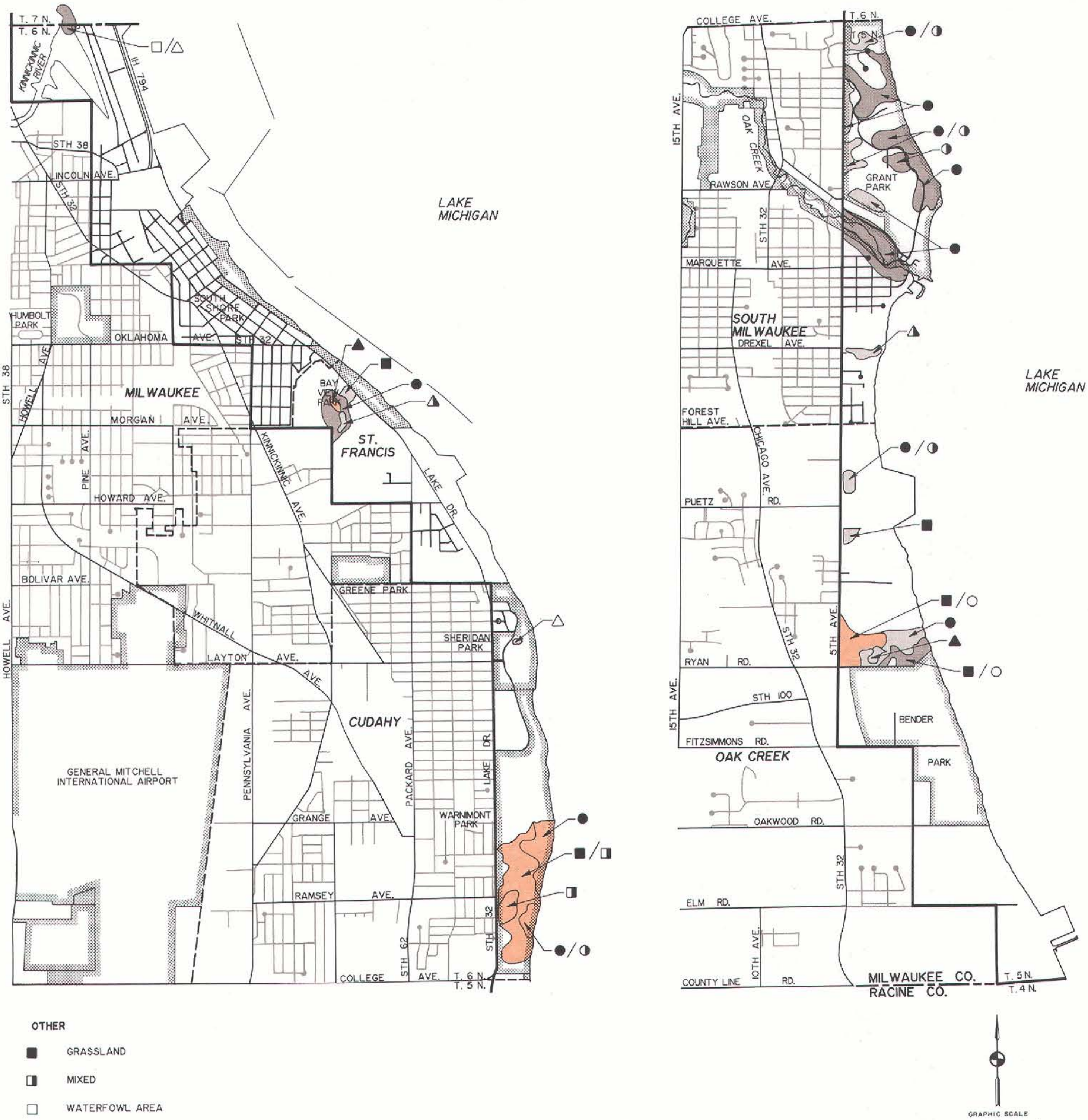
A variety of inventories and surveys of sites that possess architectural, cultural, and archaeological merit have been conducted by various units and agencies of government in Milwaukee County. The results of these inventories and

Map 8

WILDLIFE HABITAT ALONG THE LAKE MICHIGAN SHORELINE OF MILWAUKEE COUNTY



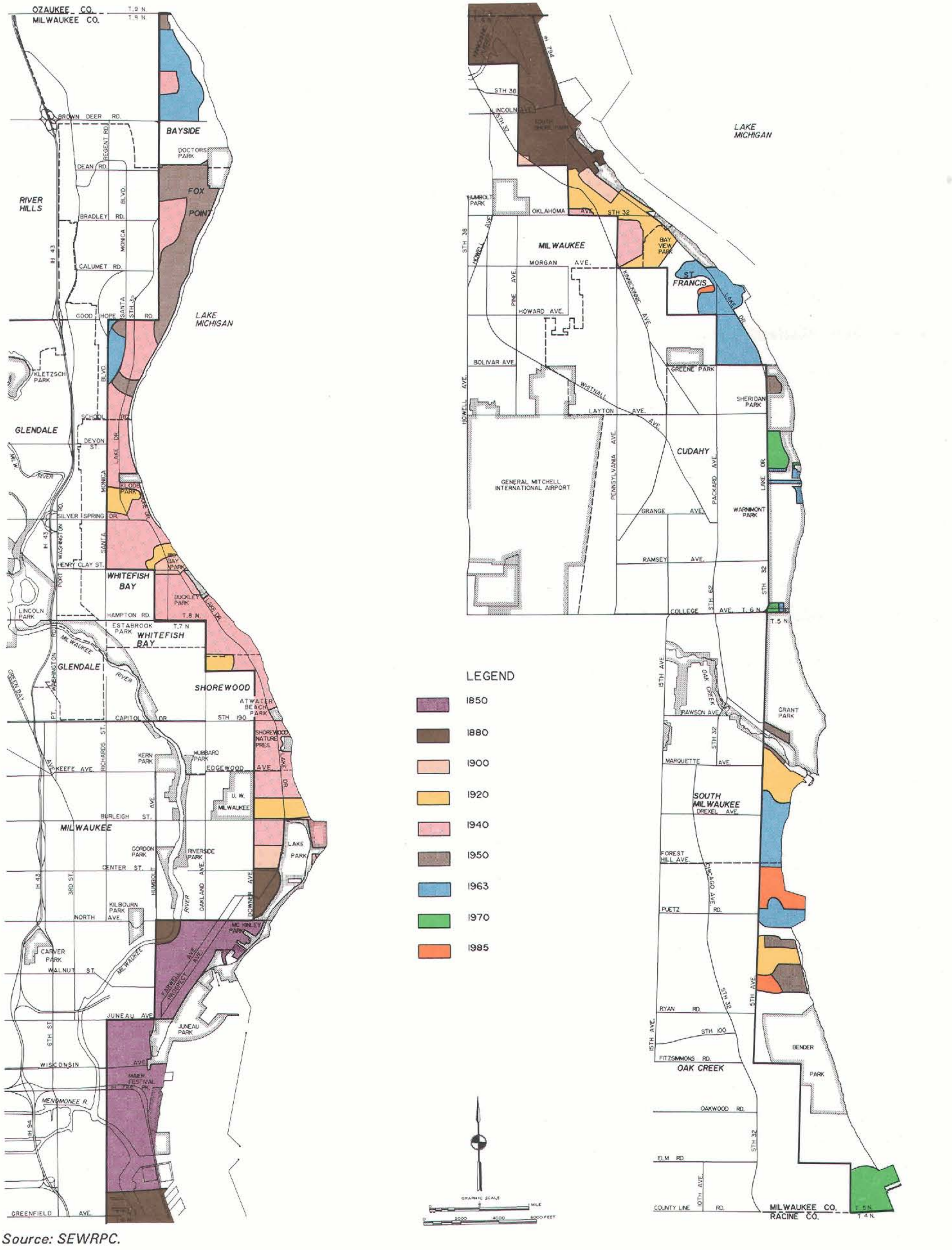
Map 8 (continued)



Source: SEWRPC.

Map 9

HISTORIC URBAN GROWTH ALONG THE LAKE MICHIGAN SHORELINE OF MILWAUKEE COUNTY: 1850-1985



Source: SEWRPC.

Table 18

URBAN GROWTH ALONG THE LAKE MICHIGAN SHORELINE OF MILWAUKEE COUNTY: 1850-1985

Year	Urban Land Area[a] (acres)	Incremental Area (acres)	Urban Growth Percent Increase
1850	849	--	--
1880	1,567	718	85
1900	1,647	80	5
1920	2,130	483	29
1940	3,267	1,137	53
1950	3,701	434	13
1963	4,286	585	16
1970	4,408	122	3
1985	4,531	123	3

[a]*For the purpose of this analysis of urban growth, urban land excludes rural, open, and park land.*

Source: SEWRPC.

surveys—on file at such agencies as the City of Milwaukee Historic Preservation Office and the Wisconsin State Historical Society—indicate that there are more than 10,000 historic sites in Milwaukee County. Certain sites of known historic significance in Milwaukee County are listed on the National Register of Historic Places. Property listed on the National Register has some degree of protection from the potentially adverse effect of federally funded or licensed activities. In 1988, there were 43 historic places in the study area listed on the National Register, including 37 individual sites and six historic districts. A historic site is a property that was the location of a significant event, activity, building, structure, or archaeological resource. A historic district is a geographically definable area possessing a significant concentration, linkage, or continuity of sites, buildings, or structures that are united by plan or by physical development.

A detailed list of historic sites in the study area on the National Register of Historic Places in 1988 is presented in Table 19. The location of these sites, as well as of the historic districts, is shown on Map 10. These sites and districts designated for preservation form a significant link to the past. It is important to note that additional sites in the study area could be identified as having historic significance and thus become eligible for listing on the National Register.

The Bay View historic district contains much of the old Village of Bay View, which was incorporated in 1878 and consolidated with the City of Milwaukee in 1887. Before consolidation, the Village of Bay View was an important industrial area in the Town of Lake. In addition to the Milwaukee Iron Company—a pioneer steel mill in the Milwaukee area constructed in 1868—Bay View contained workers' cottages, saloons, churches, and a yacht club. The South Shore Yacht Club, originally organized in 1913, established quarters on a sailing vessel and a barge. The present clubhouse was constructed in 1936. Early in its history, the club merged with the Steel Mill Yacht Club, which had been organized by the Illinois Steel Company for its employees. The Yacht Club and lakefront parks are integral elements of the Bay View historic district. The nomination form for the establishment of the district states that:

> The park, yacht club, and the lake are important foci of the district, and the open space is an historical characteristic of Bay View. The last unobstructed view of Lake Michigan in south Milwaukee is provided by the park. Bay View's location commands an excellent view of the upper shoreline and the bay, and from this vantage the name "Bay View" was derived. This open vista has long characterized Bay View and is a significant component of the historic district.[15]

The Third Ward historic district, much of which is constructed on filled marshland, was initially owned by Peter Juneau, Soloman Juneau's brother. Water Street—the first street to be graded in the City of Milwaukee—was a principal business thoroughfare and was lined with hotels and warehouses. Almost the entire Third Ward was destroyed in a disastrous fire which leveled 440 buildings in October 1892. Among the few survivors of the fire was the Cross Keys Hotel, which was constructed in 1853 and razed in 1980. The district now contains primarily commercial and institutional buildings.

The North Point South historic district, which forms the northeast corner of Milwaukee Bay, has always been a prime location, offering from

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[15]*U. S. Department of the Interior, National Park Service, National Register of Historic Places, Inventory-Nomination Form, Bay View Historic District.*

Table 19

HISTORIC SITES ALONG THE LAKE MICHIGAN SHORELINE OF MILWAUKEE COUNTY: 1988

Number on Map 10	Site Name	Civil Division	Public Land Survey Town, Range, and Section	Year Listed on the Register of Historic Places
1	Staile Mayer House	Fox Point	T8N R22E, Section 16	1985
2	Horace B. Hatch House	Whitefish Bay	T8N R22E, Section 28	1985
3	Barfield-Staples House	Whitefish Bay	T8N R22E, Section 33	1985
4	John F. McEwens House	Whitefish Bay	T8N R22E, Section 33	1985
5	Paul F. Grant House	Whitefish Bay	T8N R22E, Section 33	1985
6	Frank J. Williams House	Whitefish Bay	T8N R22E, Section 33	1985
7	Frederick Sperling House	Whitefish Bay	T8N R22E, Section 33	1985
8	Halbert D. Jenkins House	Whitefish Bay	T8N R22E, Section 33	1985
9	Herman Vihlen House	Whitefish Bay	T8N R22E, Section 33	1983
10	Harrison Hurdie House	Whitefish Bay	T7N R22E, Section 3	1985
11	George Goebel House	Whitefish Bay	T7N R22E, Section 3	1985
12	G. B. Van Devan House	Whitefish Bay	T7N R22E, Section 3	1985
13	Rufus Arndt House	Whitefish Bay	T7N R22E, Section 3	1985
14	Alfred M. Hoelz House	Milwaukee	T7N R22E, Section 3	1985
15	George E. Morgan House	Shorewood	T7N R22E, Section 3	1985
16	Henry A. Meyer House	Shorewood	T7N R22E, Section 10	1985
17	Milwaukee-Downer	Milwaukee	T7N R22E, Section 10	1974
18	North Point Lighthouse	Milwaukee	T7N R22E, Section 15	1984
19	Frederick C. Bogk House	Milwaukee	T7N R22E, Section 15	1972
20	Shorecrest Hotel	Milwaukee	T7N R22E, Section 22	1984
21	North Point Water Tower	Milwaukee	T7N R22E, Section 22	1973
22	Lloyd R. Smitt House	Milwaukee	T7N R22E, Section 28	1974
23	Charles Allis House	Milwaukee	T7N R22E, Section 21	1975
24	Presbyterian Church	Milwaukee	T7N R22E, Section 28	1974
25	Aslor on the Lake	Milwaukee	T7N R22E, Section 28	1984
26	German-English Academy	Milwaukee	T7N R22E, Section 28	1977
27	Office Building	Milwaukee	T7N R22E, Section 29	1983
28	Sixth Church of Christ, Scientist	Milwaukee	T7N R22E, Section 28	1980
29	Women's Club of Wisconsin	Milwaukee	T7N R22E, Section 28	1982
30	Abstract Association Building	Milwaukee	T7N R22E, Section 28	1982
31	St. John's Roman Catholic Cathedral	Milwaukee	T7N R22E, Section 28	1974
32	Old St. Mary's Church	Milwaukee	T7N R22E, Section 28	1973
33	The State Bank of Wisconsin/ Bank of Milwaukee	Milwaukee	T7N R22E, Section 28	1984
34	Northwestern Mutual Life Insurance Company, Home Office	Milwaukee	T7N R22E, Section 28	1973
35	Baumback Building	Milwaukee	T7N R22E, Section 28	1983
36	Saloon	Milwaukee	T7N R22E, Section 9	1977
37	Henni Hall	St. Francis	T6N R22E, Section 15	1974

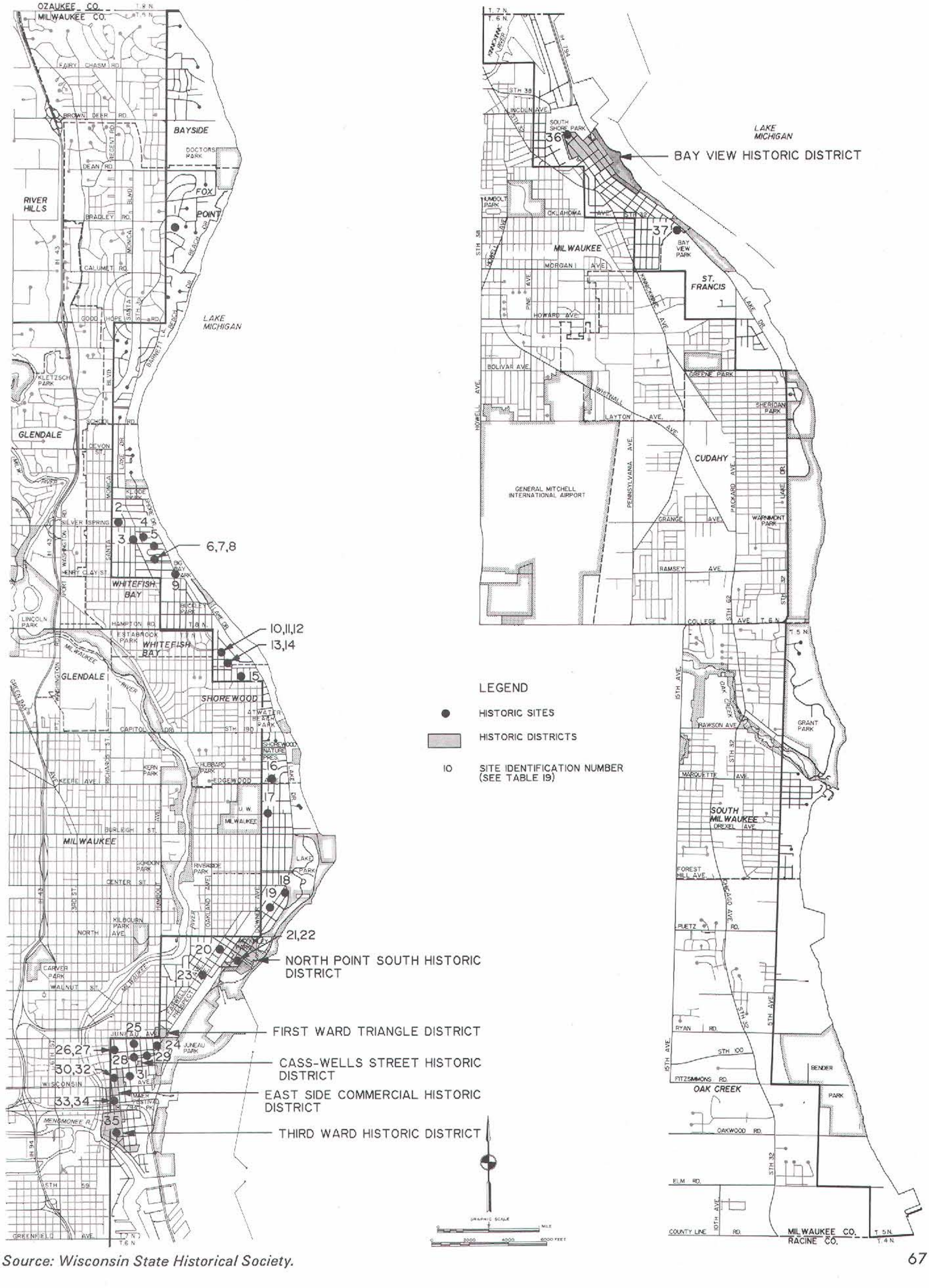
Source: Wisconsin State Historical Society.

its high bluff an excellent view of the bay and the eastern portion of the City of Milwaukee. The prestigious residential neighborhood which exists there was initially subdivided in 1854. The focus of the District is the 175-foot-high North Point Water Tower, a Victorian Gothic structure completed in 1873.

The First Ward historic district, which is located north of E. Juneau Avenue and South of E. Ogden Avenue on Prospect Avenue, represents one of the last intact groupings of high-style Victorian residential architecture in the down-town area. The District was developed before the Civil War as the City's first neighborhood for the

Map 10

HISTORIC SITES AND DISTRICTS ALONG THE LAKE MICHIGAN SHORELINE OF MILWAUKEE COUNTY



Source: Wisconsin State Historical Society.

wealthy and social elite. The District derives its name from the triangular park contained within its boundaries. In 1847, when James H. Robers platted this area, he set aside this land as the First Ward Triangle. It was later renamed Burns Triangle in 1909. Although the buildings in the District were all originally built as residences, the majority are now occupied as offices.

The Cass-Wells Street historic district, located in the former Yankee Hill neighborhood at the northwest corner of N. Cass and East Wells Streets, was once one of the City of Milwaukee's most exclusive residential districts. The neighborhood was developed from 1870 to 1914. The District constitutes one of the few remaining clusters of single-family residences that once covered this part of the City.

The East Side Commercial historic district contains seven blocks of the Milwaukee central business district east of the Milwaukee River. There are three periods of commercial development represented in the district. The buildings constructed from the earliest period, 1856 through 1875, were generally two- and three-story commercial structures with retail and service shops on the first floor, and offices and apartments above. During the second period of development, 1875 through 1900, four- to 10-story office buildings, wholesale blocks, and commission houses were built. The last period of development, 1900 through 1939, was characterized by 12- to 18-story high-rise office towers built to accommodate the increasing demand for office space in the central business district. The District is now comprised almost exclusively of mixed business uses, including retail shops, restaurants, wholesale houses, a variety of personal service firms, and numerous professional offices.

Park Development: Milwaukee County parks cover about 38 percent of the study area shoreline length, with local municipal parks covering an additional 3 percent of the shoreline length. These parks contain the best remaining elements of the natural resource base, including the best woodlands, wildlife habitat, undeveloped land, rugged terrain, and sites having special recreational and scientific value. These parks have ecological and aesthetic values and add to the unique natural beauty of the Milwaukee urban area, enhancing the quality of life for county residents.

The Milwaukee County park system was first proposed in 1923 by Charles B. Whitnall, a Milwaukee County Park Commissioner. That proposal envisioned a greenbelt of scenic drives generally following the major waterways and circling within the County. Rapid expansion of the county parkway system occurred soon afterward. In 1937, most parks within the County, regardless of municipal location, were transferred to county jurisdiction, and the City of Milwaukee Parks Board was disbanded. Some municipalities, however, continued to maintain jurisdiction over some local parks.

Table 20 summarizes the development of the parks along the Lake Michigan shoreline. Park acquisition began as early as 1872 and continued to 1979. As of 1988, there were about 1,666 acres of parks along the Lake Michigan shoreline, representing about 22 percent of the total study area. Of the total park acreage, about 1,573 acres are under the jurisdiction of Milwaukee County; 68 acres are under the jurisdiction of the City of Milwaukee; 12 acres are under the jurisdiction of the Village of Whitefish Bay; and 13 acres are under the jurisdiction of the Village of Shorewood.

Shoreline Uses: When developing shore protection plans and designing specific shore protection structures, it is essential to recognize and address the concerns and desires of both the nearby lakefront residents and the general public. Especially within certain residential areas, particular shoreline uses and characteristics have been traditionally preferred. For example, within the Bay View area, support has been expressed for maintaining the open land areas, the unobstructed views of the lake, the protected boat mooring areas, and the overall beauty of the "bay."

During a public hearing held on June 15, 1987, to discuss a Village of Whitefish Bay proposal to protect Klode Park and the North Shore Water Commission water intake plant from further bluff failure, several residents expressed a desire to maintain the relative privacy and solitude offered by a small village park. While some residents wanted enhanced recreational opportunities, many residents opposed any significant increase in the traffic and use of the park. In response to the comments made at the hearing, measures were taken by the Village to help limit any significant increase in use of the park.

Table 20

HISTORICAL DEVELOPMENT OF PARKS ALONG THE LAKE MICHIGAN SHORELINE OF MILWAUKEE COUNTY

Park	Date of Initial Land Acquisition	Date of Park Development	Initial Park Jurisdiction	1988 Park Jurisdiction	1988 Area Park Acres
Atwater	1916	1932	Village of Shorewood	Village of Shorewood	5
Bay View	1925	1963	City of Milwaukee	Milwaukee County	31
Bender	1967	Undeveloped	Milwaukee County	Milwaukee County	308
Big Bay	1937	1943	Milwaukee County	Milwauee County	8
Buckley	1931	1943	Village of Whitefish Bay	Village of Whitefish Bay	1
Doctors	1928	1930	Village of Fox Point[a]	Milwaukee County	49
Henry W. Maier Festival Park	Filled in early 1900's	1970	City of Milwaukee	City of Milwaukee	68
Grant	1910	1944	Milwaukee County	Milwaukee County	374
Juneau	1872	1920	City of Milwaukee[a]	Milwaukee County	64
Klode	1930's	1943	Village of Whitefish Bay	Village of Whitefish Bay	11
Lake	1890	1900	City of Milwaukee[a]	Milwaukee County	128
McKinley	1889	1936	City of Milwaukee[a]	Milwaukee County	183
Sheridan	1928	1928	Milwaukee County	Milwaukee County	78
Shorewood Nature Preserve	1979	Undeveloped	Village of Shorewood	Village of Shorewood	8
South Shore	1909	1920	City of Milwaukee[a]	Milwaukee County	48
Warnimont	1948	1958	Milwaukee County	Milwaukee County	302
Total	--	--	--	--	1,666

[a]*Transferred to Milwaukee County jurisdiction in January 1937.*

Source: Milwaukee County Park Commission, Milwaukee County Park System, Guide for Growth, 1978; and SEWRPC.

A similar concern for privacy was expressed at a public hearing on the northern Milwaukee County shoreline erosion management plan held by the Village of Fox Point on April 26, 1988. Residents of the Fox Point terrace along N. Beach Drive expressed strong support for limited access and use of the shoreline in order to prevent trespassing and use of the shoreline by nonlakefront residents. These residents also opposed any offshore protection measures which would interfere with their view of the lake.

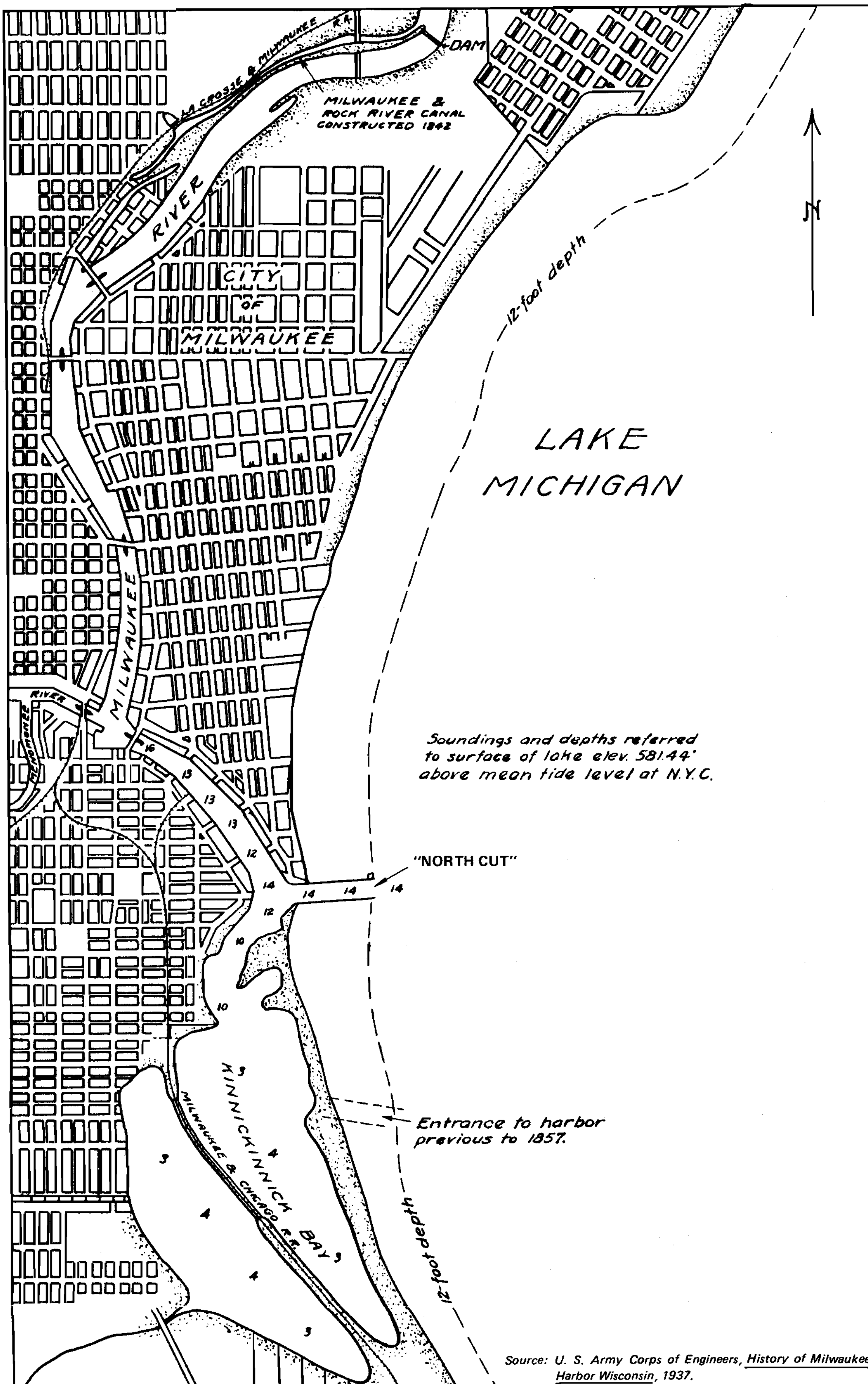
Milwaukee Harbor Development: Prior to the construction of the breakwater in Lake Michigan at Milwaukee, safe anchorage and dockage was found only in the inner harbor. The entrance to the inner harbor was at the natural mouth of the Milwaukee River at the south end of Jones Island. The location of the mouth had been fixed by the construction of jetties on both sides of the channel by the U. S. Army Corps of Engineers in 1843, as shown on Map 11. A new channel was excavated from the river to the lake in 1857, as also shown on the map. The new channel was constructed at the present location primarily to reduce the costs of maintenance dredging.

In 1877, the Chicago & North Western Railway Company built a breakwater about 100 feet offshore of North Point south to the inner harbor entrance channel to protect the railway line in its lakeside location. In 1889, the Corps of Engineers completed construction of a breakwater farther offshore to provide a harbor-of-refuge and to impede shoaling (sedimentation) in the inner harbor entrance channel. The protected water area of 540 acres was located north of the entrance channel and did not include Jones Island. The protected area was also used for temporary mooring when inner harbor traffic was heavy. By 1910, the breakwater had been extended south another 980 feet, as shown on Map 12.

In 1912, a City Harbor Commission was formed by the City of Milwaukee. A high priority of that Commission was planning for the construction

Map 11

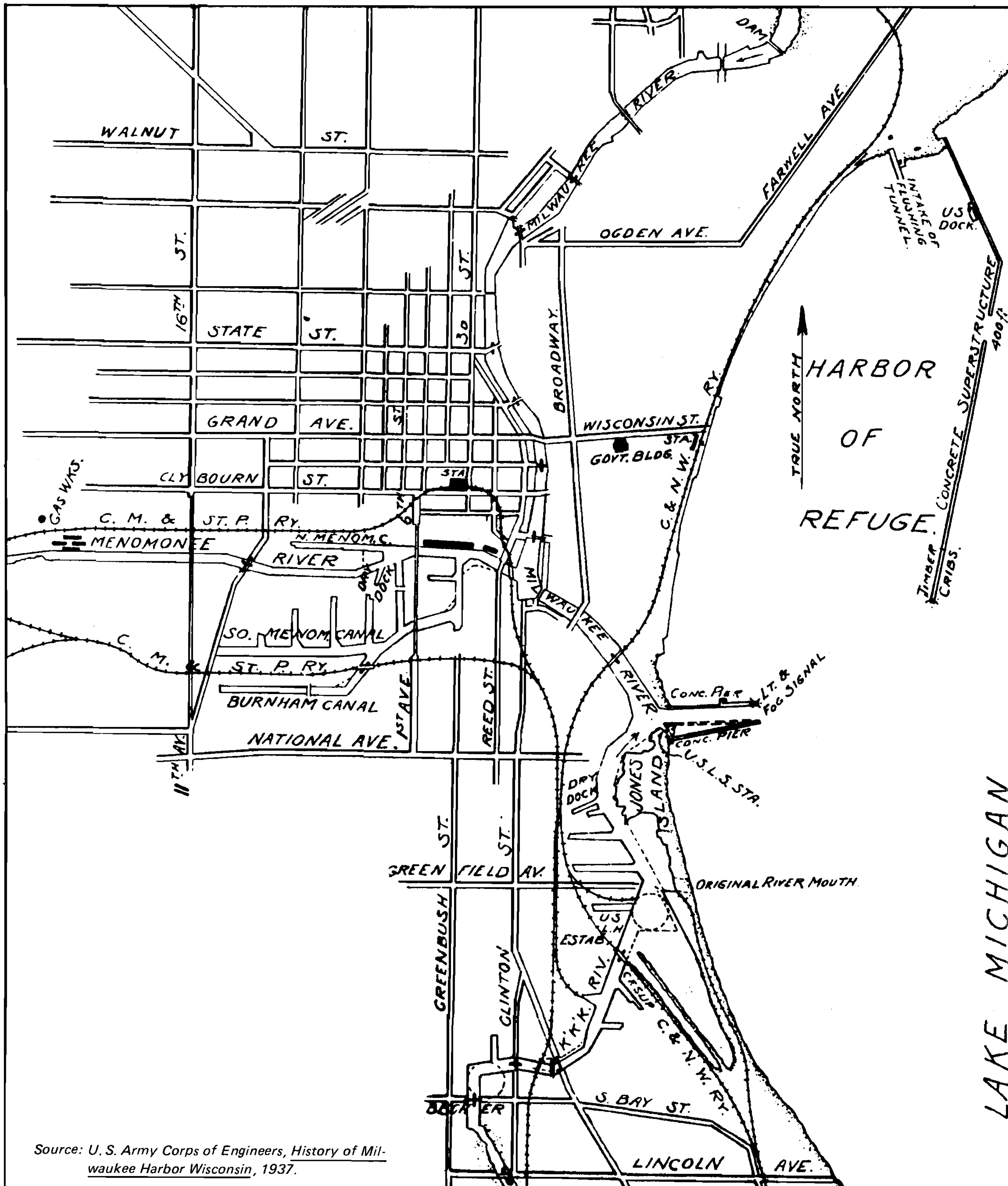
LOCATION OF THE MOUTH OF THE INNER HARBOR: 1867



Source: U. S. Army Corps of Engineers, History of Milwaukee Harbor Wisconsin, 1937.

Map 12

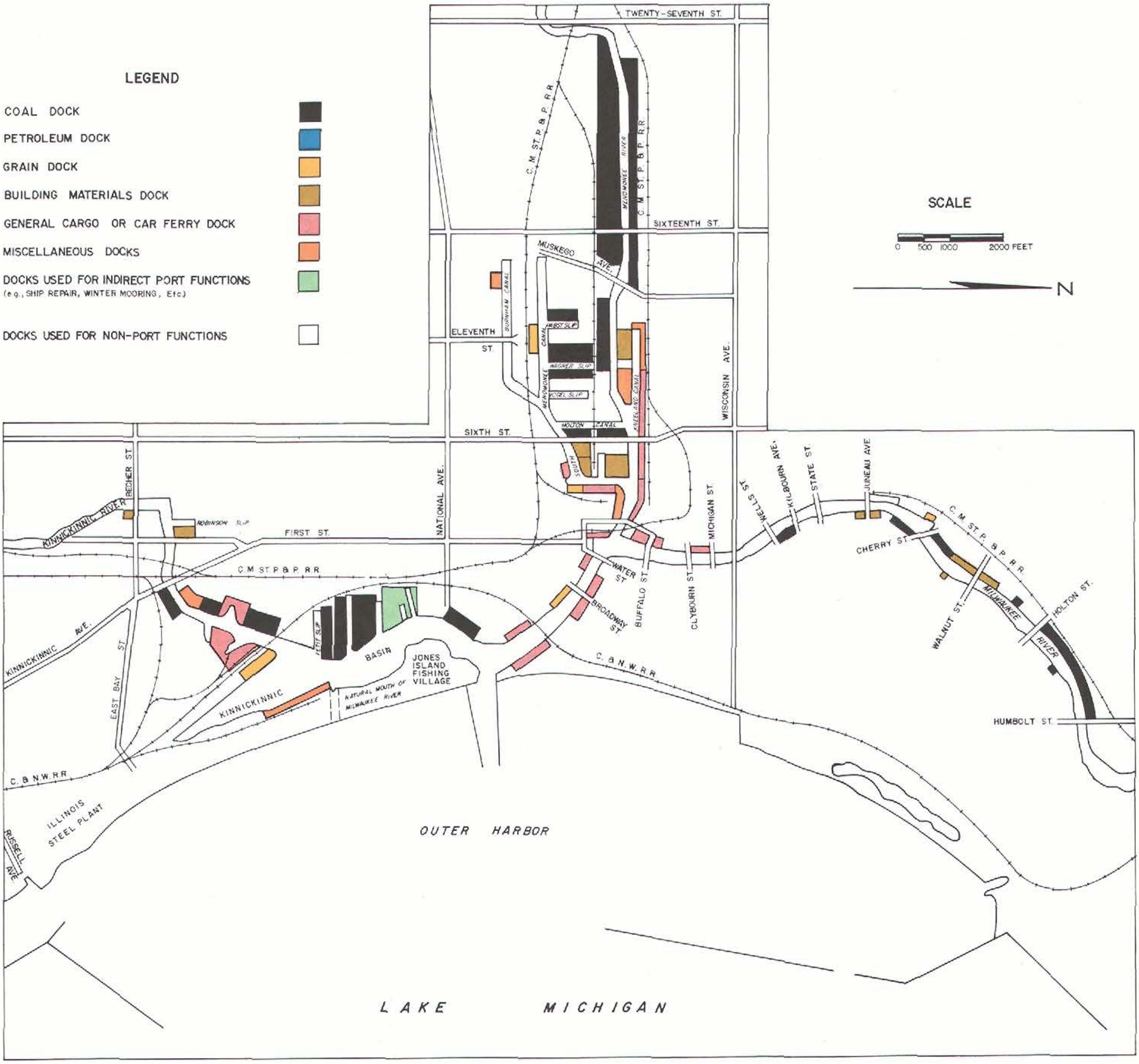
MILWAUKEE HARBOR FACILITIES: 1911



Source: U. S. Army Corps of Engineers, History of Milwaukee Harbor Wisconsin, 1937.

Map 13

HARBOR LAND USE, PORT OF MILWAUKEE: 1920



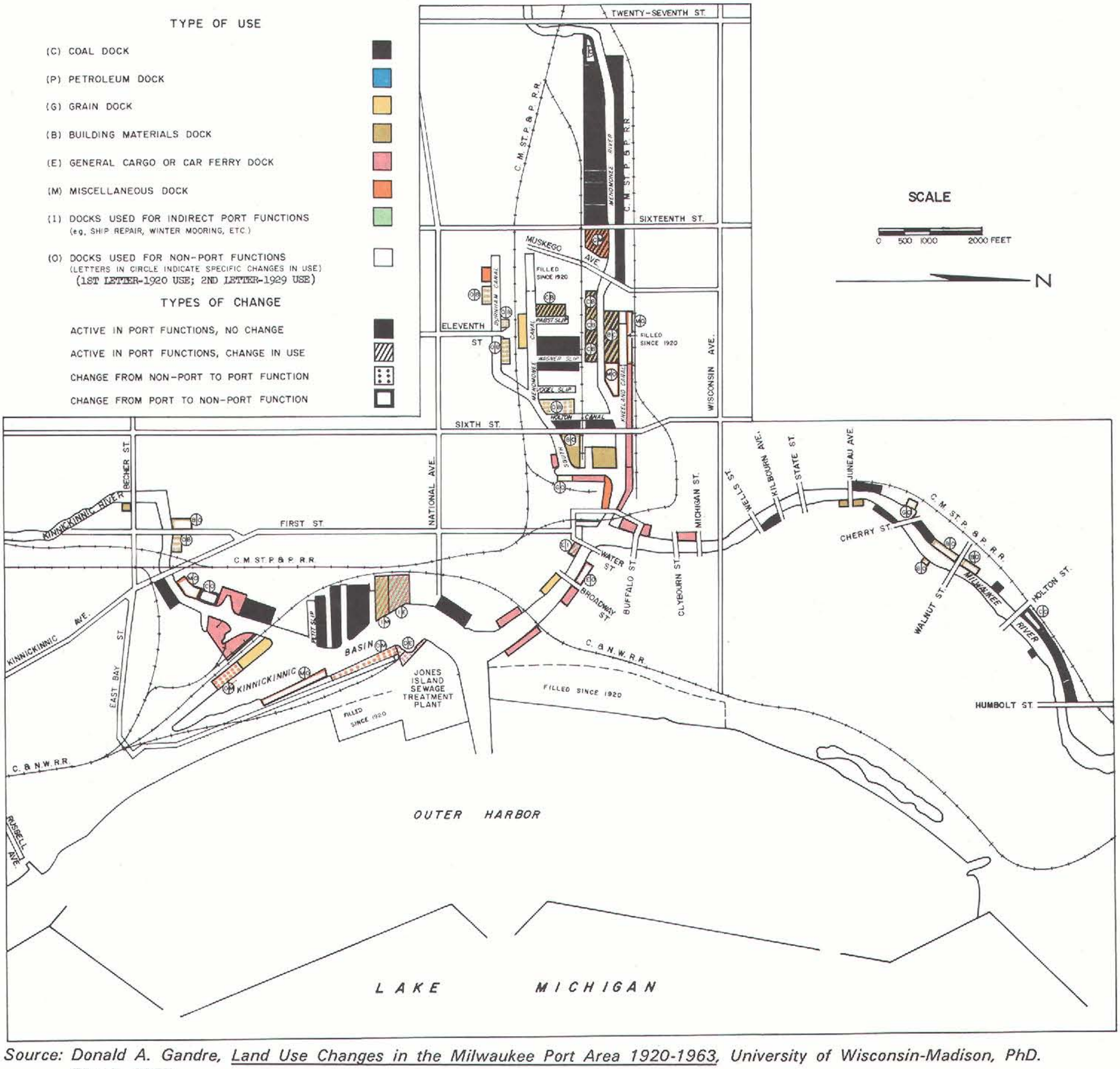
Source: Donald A. Gandre, Land Use Changes in the Milwaukee Port Area 1920-1963, University of Wisconsin-Madison, PhD. Thesis, 1965.

of outer harbor facilities and a longer breakwater to accommodate and protect larger ocean-going ships following completion of the proposed St. Lawrence Seaway. In 1929, the breakwater was completed by the U. S. Army Corps of Engineers at its present-day location.

The construction of the harbor breakwater coincided with the development of port facilities within both the inner harbor and outer harbor. Maps 13, 14, and 15 show the harbor land use changes that occurred in the Port of Milwaukee from 1920 through 1963. South Pier 1 in the outer harbor was completed at its present-day location in 1933. South Pier 2 was completed in 1961. The carferry pier was completed in 1960 at the present location of the Harbor Commission North Pier. The McKinley Park lakefill and marina were completed in 1964. The most recent major change in the outer harbor occurred in

Map 14

HARBOR LAND USE, PORT OF MILWAUKEE: CHANGES 1920-1929



Source: Donald A. Gandre, Land Use Changes in the Milwaukee Port Area 1920-1963, University of Wisconsin-Madison, PhD. Thesis, 1965.

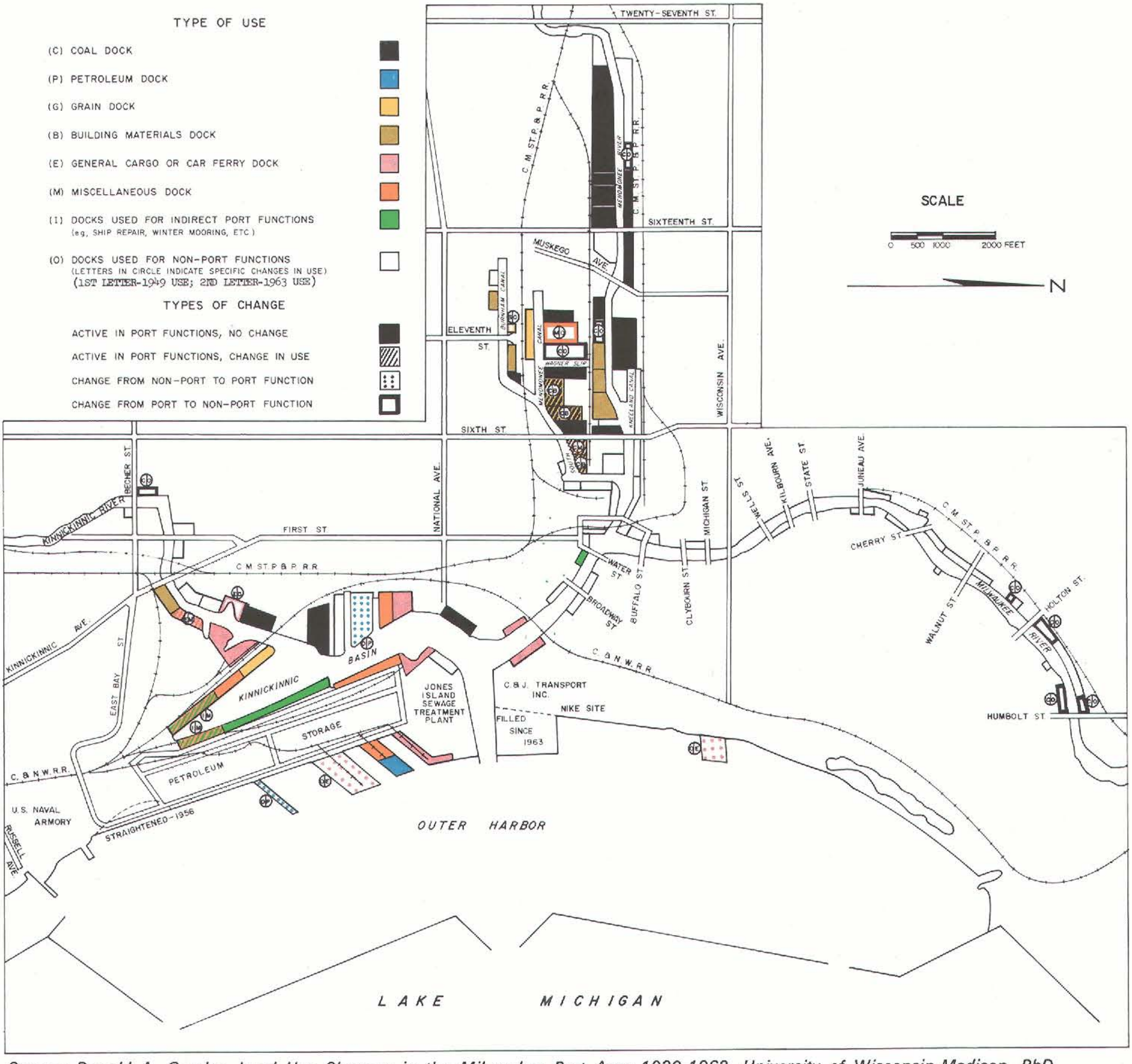
1976 with construction of the confined disposal facility for polluted dredge spoils by the Corps of Engineers, which is located on the south end of the outer harbor next to the U. S. Coast Guard Station. Existing Milwaukee Harbor facilities are shown on Map 16.

Civil Divisions

Local civil division boundaries within the study area are shown on Map 17. The study area contains portions of the Cities of Cudahy, Milwaukee, Oak Creek, St. Francis, and South Milwaukee, and the Villages of Bayside, Fox Point, Shorewood, and Whitefish Bay. The area and proportion of each municipality within the study area, as well as the length of Lake Michigan shoreline lying within the jurisdiction of each of these local units of government, are shown in Table 21. As indicated in the table, the City of Milwaukee contained 2,654 acres, or

Map 15

HARBOR LAND USE, PORT OF MILWAUKEE: CHANGES 1949-1963



Source: Donald A. Gandre, Land Use Changes in the Milwaukee Port Area 1920-1963, University of Wisconsin-Madison, PhD. Thesis, 1965.

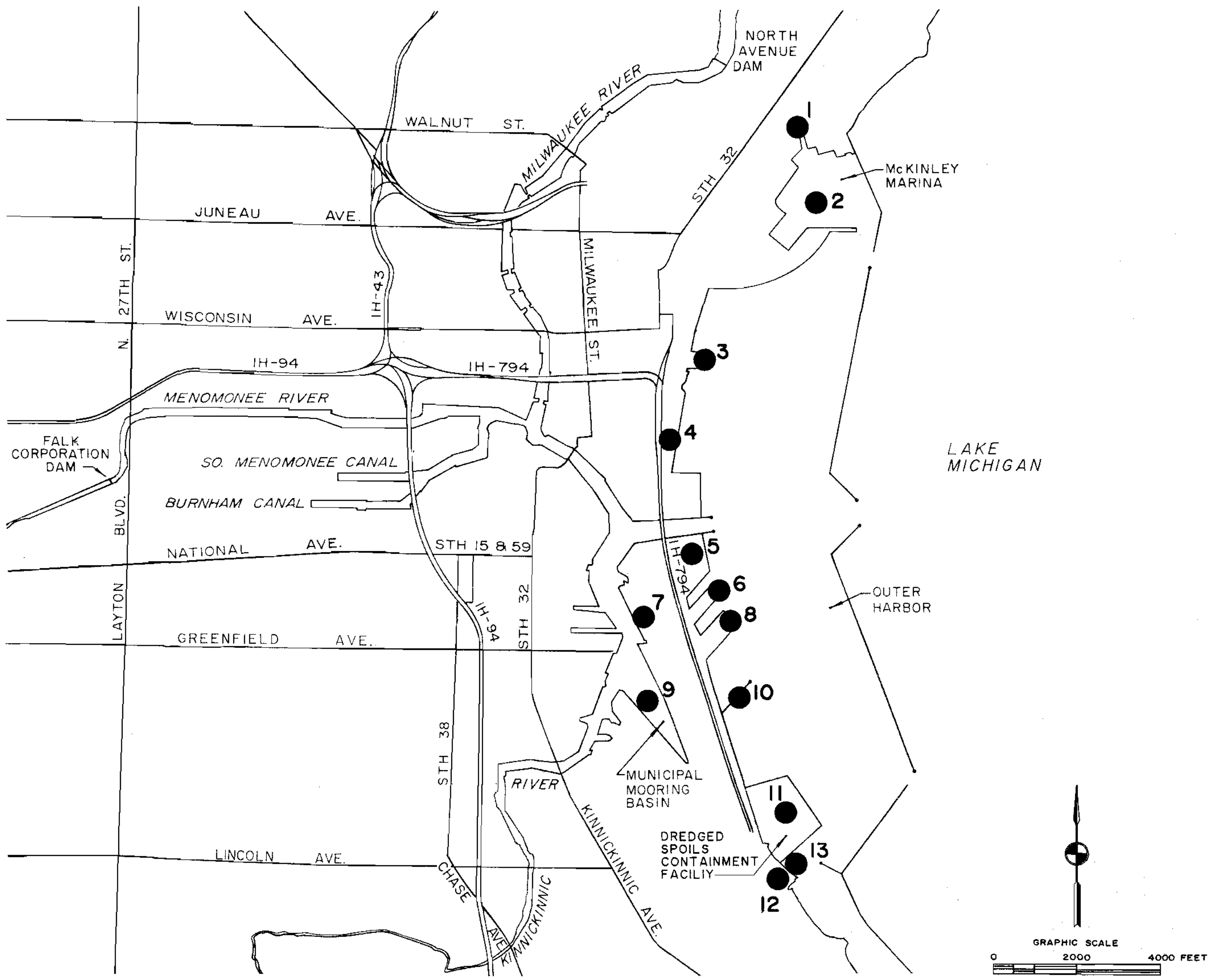
35 percent, of the study area, and accounted for 52,160 feet, or 33 percent, of the Lake Michigan shoreline within the study area.

Existing Land Use

The type and spatial distribution of the major categories of land use within the study area of Milwaukee County in 1985 are shown on Map 18. The areal extent of the various major categories of land use within the shoreland study area, which encompasses a total of 7,517 acres, is presented in Table 22. As shown on Map 18 and indicated in Table 22, the majority of the study area—4,443 acres, or 59 percent—was devoted to intensive urban uses in 1985, including residential, transportation and utility, industrial, governmental and institutional, and commercial uses. Of these urban land uses, residential uses

Map 16

MILWAUKEE HARBOR FACILITIES: 1988



LEGEND

1 MILWAUKEE RIVER FLUSHING TUNNEL INLET
2 McKINLEY MARINA
3 HARBOR COMMISSION NORTH PIER
4 HENRY W. MAIER FESTIVAL GROUNDS
5 JONES ISLAND SEWAGE TREATMENT PLANT
6 PIER NO. 1
7 HEAVY LIFT DOCK
8 PIER NO. 2
9 MUNICIPAL MOORING BASIN
10 LIQUID CARGO PIER
11 DREDGE SPOIL CONTAINMENT FACILITY
12 U.S. COAST GUARD STATION
13 KINNICKINNIC RIVER FLUSHING TUNNEL INLET

Source: SEWRPC.

and transportation and utility uses constituted the major proportions. A total of 1,942 acres, or 44 percent of the developed urban area, was residential. Transportation and utility uses accounted for 1,730 acres, or 39 percent of the urban area. Recreational uses constituted an additional 749 acres, or 10 percent of the total area. Undeveloped lands, including wetlands, woodlands, agricultural land, and unused urban land, encompassed 2,111 acres, or 28 percent of the total study area. Surface water accounted for the balance—215 acres, or 3 percent of the total study area.

Existing Zoning

In the absence of long-range land use plans, zoning ordinances and attendant zoning district maps provide an important expression of community land use development objectives. Zoning ordinances are presently in effect in each of the nine civil divisions that have jurisdiction in the Lake Michigan coastal erosion study area in Milwaukee County. Generalized zoning districts along the immediate shoreline are shown on Map 19. Table 23 summarizes the zoning categories shown on the map.

COASTAL EROSION PROCESSES

Erosion of the Lake Michigan shoreline is a natural process which can be accelerated—such as by increasing the rate and volume of stormwater runoff—or decelerated—such as by the construction of shore protection measures—by human activities. Shoreline erosion includes two processes: bluff erosion and beach erosion. Various factors which contribute to bluff and beach erosion include wave action, groundwater seepage, precipitation runoff, lake level elevation, freeze-thaw action, lake ice movement, the type of bluff material, and the type of vegetative cover.

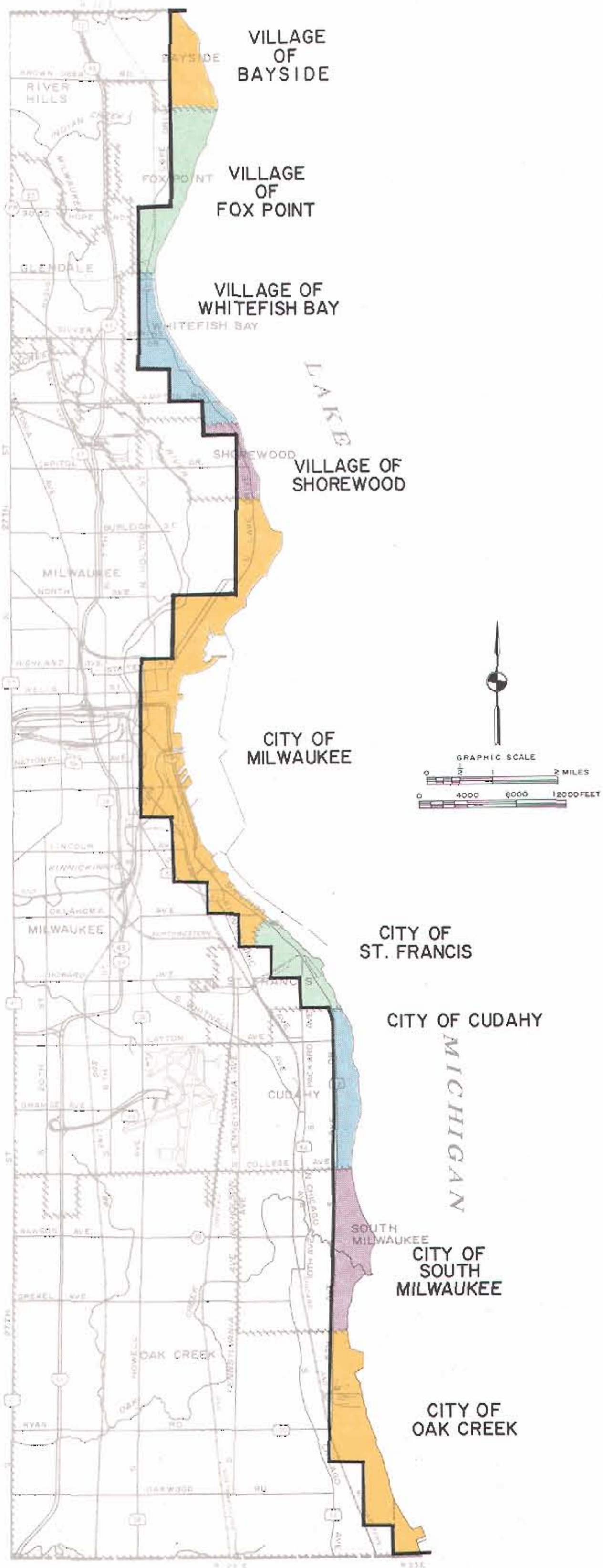
Bluff Erosion

Bluff erosion, occurring in the form of toe erosion, slumping, sliding, flow, surface erosion, and solifluction, results in the intermittent, sometimes massive, recession of the bluff. On all slopes gravity acts to move material on the slope to a lower elevation. On most slopes which are undisturbed by man, and where waves are not eroding the base of the slope, an equilibrium is established over a relatively long period of time between the forces acting to move material down the slope and the resistance of the materials in the slope to those forces. The shear stress forces acting on the materials in the bluffs are primarily determined by the weight of the soil and water mass in the bluff, water pressures in the bluff, and external loads such as buildings and vibrations. Bluff materials have a shear strength which, in stable slopes, is greater than the stresses. The shear strength depends on the properties of the soil and the moisture content, which is in part determined by soil drainage. Bluffs fail when either the shear stress is increased or the shear strength decreased, altering the balance of forces until the stresses exceed the resisting soil strength. Undercutting at the toe of the slope by waves steepens the bluff and increases the shear stress.

Types of Slope Failure: One major type of slope failure is sliding. In this type of failure, the material generally moves along a single slide plane. The two forms of slides common along the Milwaukee County shoreline are translational

Map 17

CIVIL DIVISIONS WITHIN THE LAKE MICHIGAN SHORELINE OF MILWAUKEE COUNTY: 1988



Source: SEWRPC.

Table 21

AREA AND SHORELAND LENGTH OF CIVIL DIVISIONS WITHIN THE MILWAUKEE COUNTY LAKE MICHIGAN COASTAL EROSION AREA: 1987

Civil Division	Area (acres)	Percent of Study Area	Lake Michigan Shoreline Length (feet)	Percent of Shoreline
City of Oak Creek	1,095.1	14.6	22,720	14.3
City of South Milwaukee	626.9	8.3	15,350	9.6
City of Cudahy	448.1	6.0	14,240	9.0
City of St. Francis	611.7	8.1	9,620	6.0
City of Milwaukee	2,654.3	35.3	52,160	32.8
Village of Shorewood	306.0	4.1	6,590	4.1
Village of Whitefish Bay	606.9	8.1	14,680	9.2
Village of Fox Point	665.2	8.8	14,580	9.2
Village of Bayside	503.2	6.7	9,170	5.8
Study Area Total	7,517.4	100.0	159,110	100.0

Source: SEWRPC.

slides and rotational slides, or slumps. Translational slides involve a surface layer several inches to a few feet thick, generally sliding parallel to the face of the slope. Translational slides can occur either rapidly or slowly. The term slump refers to the sliding of a fairly large mass along a curved surface. The slide mass rotates, and often the top of the slump block is tilted back toward the slope face. Slumps usually take place suddenly and can cause extensive damage since they can result in a large recession of the bluff.

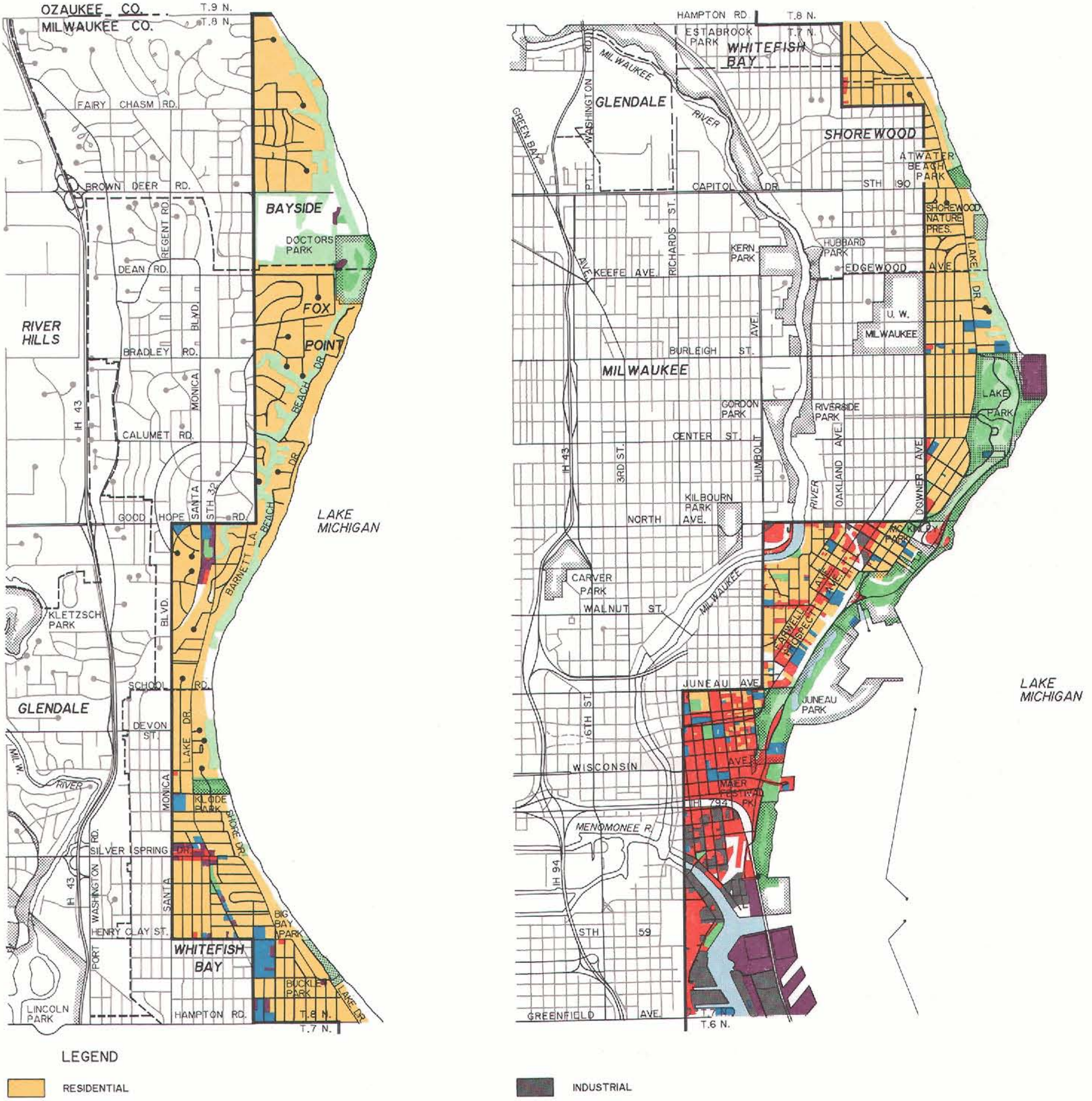
A second major type of slope failure is flow. With this kind of slope failure, large amounts of water are present and the soil mass actually liquifies and moves like a fluid. Some flow commonly occurs at the toe of slump blocks during and relatively soon after failure. Since slump blocks rotate such that the top of the block is often tilted back toward the bluff, surface water can accumulate in these depressions and saturate the underlying soil. Flows also occur when intense rains saturate the surface layer of soil or in the spring as intergranular ice melts near the soil surface. Flows can also occur where groundwater discharges along the bluff face through layers of silt or fine sand. If these more permeable soil layers are located between less permeable clay layers, removal of sediment by flow due to groundwater seepage—referred to as sapping—can occur, and cause undercutting which creates an unstable slope subject to slumping and sliding.

A third type of slope failure, related to flow, is solifluction. Solifluction is the slow, viscous downslope flow of water-saturated material over an impermeable base. Solifluction is often caused by freeze-thaw activity. During the thawing period, there is a buildup of excess pore pressure within the soil mass. Because of underlying impermeable frozen ground, the pore pressures cannot be dissipated and thus shear resistance decreases. Also, the growth of ice crystals within the soil during winter months weakens the structure of the soil. The amount of moisture in a soil prior to freezing will affect the shear strength after it has thawed; the higher the moisture content before freezing, the greater the reduction in shear strength after thawing. The net result is a shear resistance, or strength, which is less than the shear stress; therefore, even gentle slopes may fail. Solifluction can also occur in unconsolidated material which overlies impermeable bedrock.

A fourth type of slope failure is sheet wash and rill and gully erosion. Both sheet wash and rill and gully erosion result from surface water runoff flowing over the top of the bluff, and over the slope face itself. Sheet wash is the uncon-

Map 18

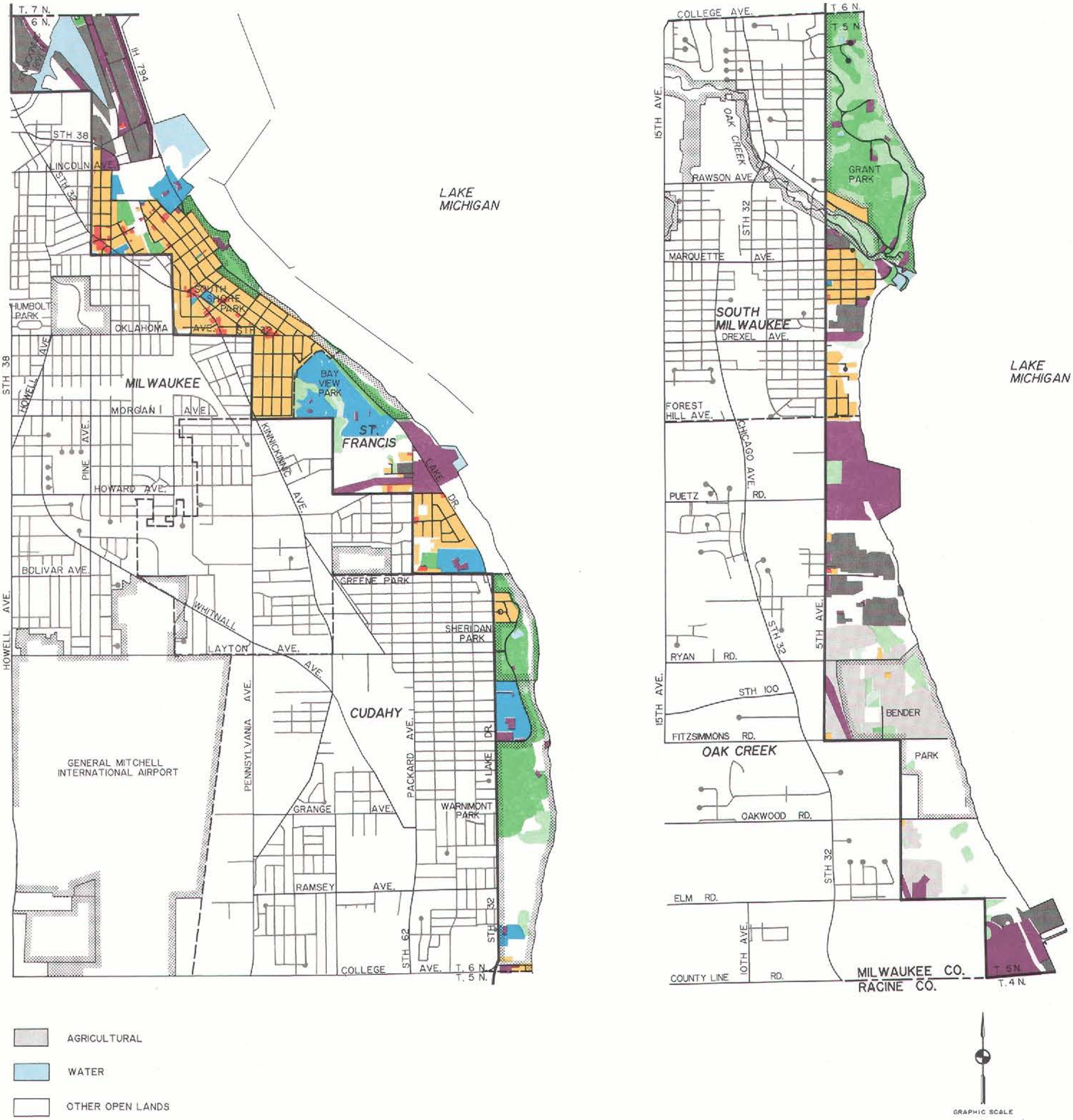
EXISTING LAND USE WITHIN THE MILWAUKEE COUNTY LAKE MICHIGAN SHORELINE STUDY AREA: 1985



LEGEND

- RESIDENTIAL
- COMMERCIAL
- TRANSPORTATION, COMMUNICATION AND UTILITIES
- GOVERNMENTAL AND INSTITUTIONAL
- INDUSTRIAL
- RECREATIONAL
- WOODLANDS
- WETLANDS

Map 18 (continued)



Source: SEWRPC.

Table 22

EXISTING LAND USE IN THE MILWAUKEE COUNTY SHORELINE MANAGEMENT STUDY AREA: 1985

	Civil Division																			
	City of Oak Creek		City of South Milwaukee		City of Cudahy		City of St. Francis		City of Milwaukee		Village of Shorewood		Village of Whitefish Bay		Village of Fox Point		Village of Bayside		Total Study Area	
Land Use Category	Area (acres)	Percent of Total	Area (acres)	Percent of Total	Area (acres)	Percent of Total	Area (acres)	Percent of Total	Area (acres)	Percent of Total	Area (acres)	Percent of Total	Area (acres)	Percent of Total	Area (acres)	Percent of Total	Area (acres)	Percent of Total	Area (acres)	Percent of Total
Residential	5.4	0.5	85.7	13.7	17.7	4.0	122.5	20.0	524.9	19.8	199.7	65.3	354.6	58.4	428.2	64.4	202.8	40.3	1,941.5	25.8
Commercial	0.4	<0.1	0.3	0.1	--	--	2.3	0.4	96.6	3.6	1.1	0.4	7.5	1.2	1.5	0.2	--	--	109.7	1.4
Industrial	117.0	10.7	24.1	3.8	--	--	4.2	0.7	240.5	9.1	--	--	--	--	--	--	--	--	358.8	5.1
Transportation, Communication, and Utilities	265.0	24.2	61.8	9.9	29.2	6.5	138.2	22.6	900.5	33.9	59.5	19.4	135.2	22.3	101.5	15.3	39.4	7.8	1,730.3	23.0
Governmental and Institutional	--	--	0.6	0.1	45.7	10.2	114.1	18.6	78.4	2.9	--	--	32.3	5.3	4.7	0.7	--	--	275.8	3.7
Recreational	1.2	0.1	212.5	33.9	167.8	37.4	29.1	4.8	294.3	11.1	6.5	2.1	13.9	2.3	12.1	1.8	11.6	2.3	749.0	10.0
Agricultural	239.0	21.8	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	239.0	3.2
Wetlands	10.5	1.0	5.5	0.9	--	--	6.8	1.1	--	--	--	--	--	--	--	--	--	--	22.8	0.3
Woodlands	57.2	5.2	172.1	27.4	25.0	6.0	29.4	4.8	74.8	2.8	22.3	7.3	22.7	3.8	100.6	15.1	145.3	28.9	649.4	8.6
Surface Water	--	--	5.9	0.9	2.2	0.1	4.7	0.8	201.1	7.6	--	--	--	--	--	--	0.6	0.1	214.5	2.9
Other Open Land	399.4	36.5	58.4	9.3	160.5	35.8	160.4	26.2	243.2	9.2	16.9	5.5	40.7	6.7	16.6	2.5	103.5	20.6	1,199.6	16.0
Total	1,095.1	100.0	626.9	100.0	448.1	100.0	611.7	100.0	2,654.3	100.0	306.0	100.0	606.9	100.0	665.2	100.0	503.2	100.0	7,517.4	100.0

Source: SEWRPC.

fined flow of water over the soil surface during and following a rainfall. Depths of flow are generally less than one-tenth of an inch. Raindrop impact is the dominant factor in the detachment of soil particles, and once the particles are detached, they are transported downslope at a rate determined by the water runoff rate, slope steepness, vegetative cover, and roughness of the surface, and by the transportability of the detached soil particles. Rills and gullies are formed by the concentrated, channelized flow of water on the surface. Rill and gully formation tends to follow zones of weakness established by desiccation, cracking, and differences in soil expansion due to freeze-thaw and wetting and drying. On the lake bluffs, the rills are generally destroyed over the winter months by freeze-thaw activity and solifluction, whereas gullies may exist for years.

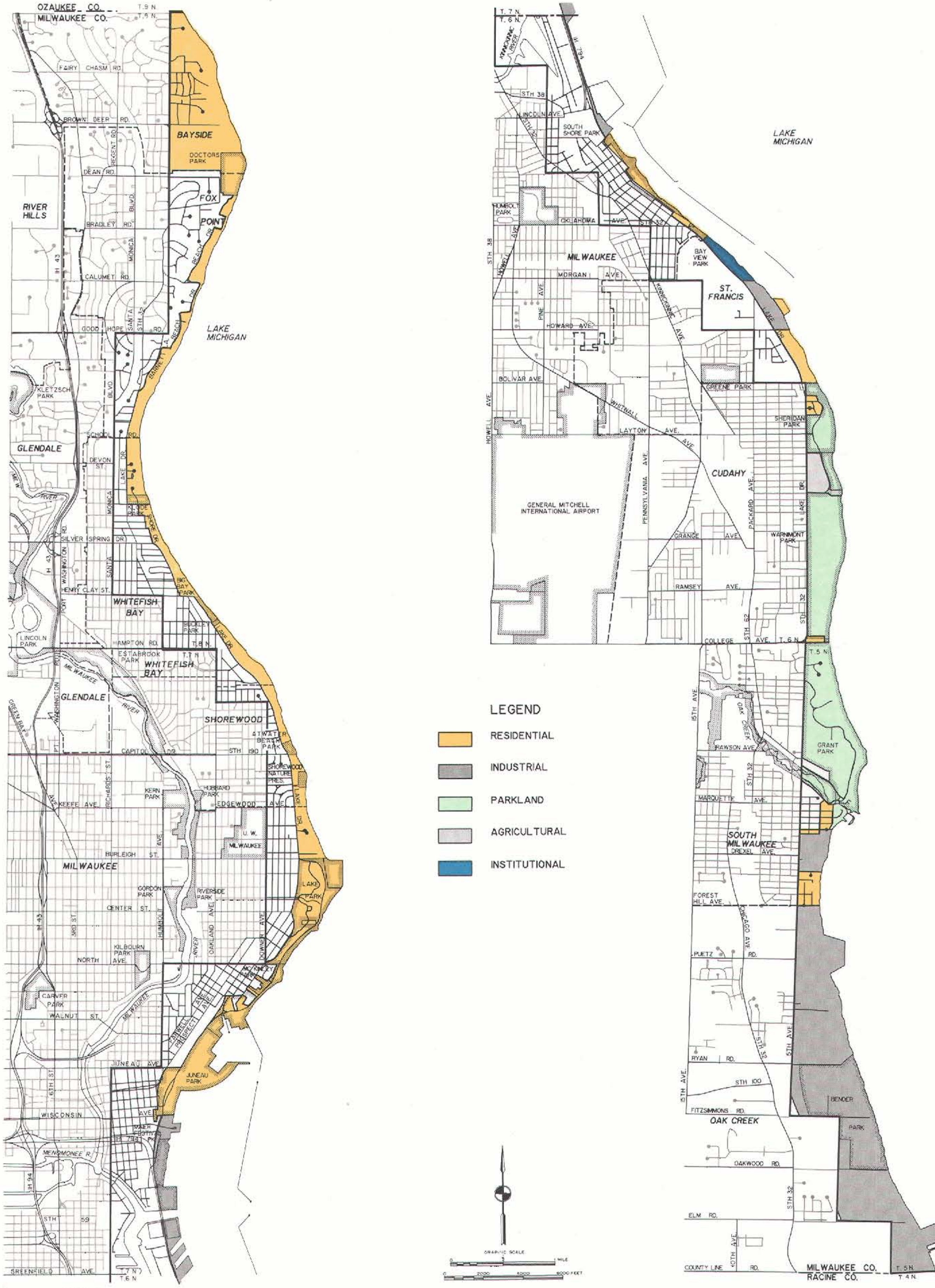
A fifth type of slope failure is rock or soil fall. This type of failure takes place when undercutting is extreme and near-vertical cliffs are produced. Even though some such segments of bluff are present along the Milwaukee County shoreline, these are generally small, and rock or soil fall from vertical faces plays only a small role in the overall shoreline erosion in the study area.

Because slope stability is influenced by dynamic factors, slope failure is a process that may occur in an unpredictable, abrupt fashion as opposed to a uniform, relatively continuous fashion. After each incremental slope failure, the soil masses tend to temporarily assume a stable configuration until the net effect of the many influencing factors once again decreases slope stability, thus precipitating another incremental failure.

Wave Action: Several factors affect the type of slope failure that occurs and the severity of that failure. The physical characteristics of the beach and bluff have a major influence on the resistance of the slope to failure. Numerous other factors affect the external stresses that are placed upon the slope, resulting in various types of failure. Among these factors is wave action, particularly during storms. When occurring concurrently with high lake levels, wave action can result in rapid and severe erosion of the toe of bluffs within the study area. This bluff toe erosion may cause instability of the entire bluff slope, and ultimately recession of the bluff. Wave action also affects the orientation, width, slope, and substrate of beaches. Figure 21 illustrates the pattern of breaking waves as they approach a beach. Wave action is also important because

Map 19

EXISTING ZONING DISTRICTS ALONG THE IMMEDIATE SHORELINE OF MILWAUKEE COUNTY: 1988



Source: SEWRPC.

Table 23

SUMMARY OF SELECTED EXISTING ZONING REGULATIONS FOR LANDS ADJACENT TO THE SHORELINE IN MILWAUKEE COUNTY

Civil Division	Zoning District	Permitted Uses	Special Uses
City of Oak Creek	Manufacturing	Agricultural buildings and uses; services; animal hospitals; self-service laundries; lodges; offices of labor organizations; manufacturing; public utilities and public services; railroad stations; police stations; sewage treatment plants; parks; playgrounds; restaurants; stadiums	Airports; bowling alleys; parking lots; motor vehicle sales; car washes; gas stations
City of South Milwaukee	Industrial	Limited food processing; manufacturing of products from textiles; glass; leather goods; plaster; paper; plastics and wood; electrical appliance manufacturing; laundries; dry cleaners; laboratories; printing and publishing; autobody shops	
	Residential	Single-family houses; churches; cemeteries; schools; public buildings; libraries; museums; police and fire stations; parks; playgrounds; professional offices; limited farming. In some areas: two- and multiple-family dwellings; boarding houses; convalescent homes; private clubs	
City of Cudahy	High-Rise Apartment	Multiple-story residential buildings	
	Residential	Single-family dwellings; churches; schools; colleges; public libraries; museums and art galleries; municipal buildings; professional offices	
	Parkland	Public recreation; recreational buildings; pools; playing fields; golf courses; ice skating and fishing ponds	
	Agricultural	Single-family dwellings; hospitals; general farming; public parks; playgrounds; community center buildings; municipal buildings; schools; drive-in theaters	

Table 23 (continued)

Civil Division	Zoning District	Permitted Uses	Special Uses
City of St. Francis (east of Lake Drive)	Industrial	Buildings or land may be used for any purpose except the following: residential; educational; religious; charitable or institutional uses. Manufacture of: acid; ammonia; chlorine; cement; lime; plaster-of-Paris; explosives; fertilizer; or asphalt. Explosives storage; garbage dumping; petroleum refining; stockyards; tanning; curing or storage of leather, hides, or skins; commercial animal raising or breeding; smelting of tin, lead, zinc, and iron ores	
	Institutional	Buildings used to house the offices of a public or a semi-public institution	
	Residential	Multiple-family dwellings; hotels; lodging houses; private clubs; and institutional and professional office buildings	
City of Milwaukee	Residential	Single-family dwellings; family day-care homes; convents; foster homes; churches; schools; colleges; governmental structures; public parks and playgrounds; nonretail agricultural uses In certain areas: two-family dwellings; multiple-family dwellings; dormitories; residential hotels; libraries; art galleries and museums; community centers; nursing homes; health clinics; hospitals	

of its potential for damaging shore protection structures such as revetments, bulkheads, breakwaters, and groins.

Lake Michigan Water Levels: Lake water-level fluctuations affect rates of wave-induced shoreline erosion. High water levels result in more rapid recession of the shoreline. When the water level is low, wave energy is expended as waves break along the beach. When water levels rise, waves can break directly on the toe of the bluff and erode the bluff material. The base of the slope is then undercut, creating unstable conditions in the slope above. This is eventually followed by slope failure and the movement of material down to the base of the bluff. As water levels decrease, the beach again widens and much of the wave energy is dissipated.

Table 23 (continued)

Civil Division	Zoning District	Permitted Uses	Special Uses
City of Milwaukee (continued)	Industrial	All uses allowed in residential districts PUBLIC AND QUASI-PUBLIC: Police and fire facilities, water treatment plants, sewage treatment plants RETAIL SALES MOTOR VEHICLE: Commercial parking, motor vehicle repair centers, car washes SERVICES: Medical and dental laboratories, research and testing laboratories, data processing centers, animal clinics, dry cleaning plants STORAGE AND WHOLESALE TRADE: Wholesale trade establishments, general storage, coal yards, storage of petroleum, storage of gas, junkyards TRANSPORTATION: Transportation passenger terminals, railroad switching and classification yards, terminals, ship terminals MANUFACTURING ANIMAL PRODUCTS: Leather finishing, meat, fish, poultry, fats and oils processing, tanning or tawing of hides, stockyards or slaughterhouses	OFFICES: Offices, banks, and other financial institutions MOTOR VEHICLE: Motor vehicle rental offices, motor vehicle service stations, car washes RETAIL SALES: General retail sales, general-purpose groceries, department stores, consumer services SERVICES: Funeral homes, photographic studios, dry cleaning and laundry stations, self-service laundries ENTERTAINMENT AND RECREATION: Recreation facilities, commercial hotels, restaurants, taverns, indoor theaters, convention centers and sports arenas STORAGE AND WHOLESALE TRADE TRANSPORTATION: Airports and heliports MANUFACTURING AND MINING

There is a time lag, however, between bluff recession rates and the decline in lake level because materials in the bluff take time to form a stable slope. Thus, even after water levels decline and wave erosion is decreased, bluff recession continues at a fairly high rate until the bluffs have reached a stable slope angle.

Figure 22 shows the annual mean water level for Lake Michigan, recorded at Milwaukee for the period 1860 through 1987. The historic low annual mean lake level at Milwaukee—577.06 feet above National Geodetic Vertical Datum (NGVD), also referred to as Mean Sea Level Datum—occurred in 1964. The historic high annual mean lake level—582.48 feet NGVD—occurred in 1986. The 1986 annual mean surpassed the previous record high annual mean of 582.24 feet NGVD set in 1886. The historic record low and record high annual mean lake levels at Milwaukee differ by 5.42 feet.

Table 23 (continued)

Civil Division	Zoning District	Permitted Uses	Special Uses
Village of Shorewood	Lake Drive Residential	Single-family dwellings; non-commercial greenhouses; nurseries and gardens; private garages	
Village of Whitefish Bay	Lake Shore Residential	Single-family dwellings; non-commercial greenhouses; nurseries and gardens; private garages	
	Churches, Public Buildings, and Grounds	Churches; public buildings and grounds; private and public schools; sewerage and water pumping stations and water storage tanks; parking; single-family dwellings and private garages	
Village of Fox Point	Residential	Residential dwellings; accessory uses	
Village of Bayside	Residential	Single-family dwellings. In some areas, schools and municipal buildings	

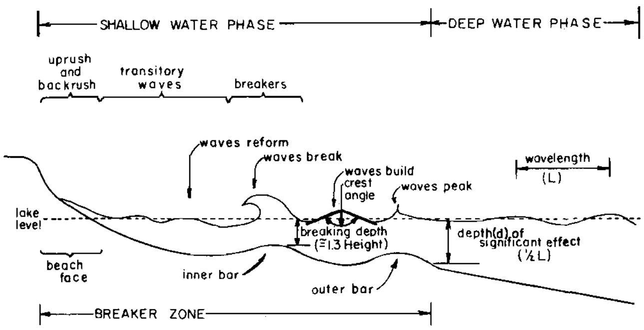
Source: SEWRPC.

The level of Lake Michigan is a function of inflow from Lake Superior, stormwater runoff from the tributary land surface, groundwater inflow and outflow, precipitation falling directly on the lake, outflow from Lake Michigan through the St. Claire River, evaporation from the lake surface, and resulting changes in the storage—volume of water—in the lake. The annual cycle in Lake Michigan water level elevations is shown in Figure 23. The highest water level elevations generally occur in June, July, and August, and the lowest water level elevations occur in January, February, and March. Generally, the lake levels rise from February through July and fall during the remainder of the year. The seasonal rise from February through July reflects the pattern of higher runoff and low evaporation during that period, in comparison to the remainder of the year. In a typical one-year period, the range in base lake levels may be expected to be about one foot. The historic range between maximum and minimum monthly mean water levels is about six feet for all months of the year. The highest maximum monthly lake levels recorded at Milwaukee were measured in 1838—584.3 feet NGVD—and in 1886—583.3 feet NGVD. However, these nineteenth century recorded water levels cannot be directly compared to recent water level measurements. Uncompensated channel improvements on the St. Claire River from 1933 and 1962 are reported to have reduced the levels of Lakes Michigan-Huron by about 1.2 feet. Therefore, the maximum monthly lake levels recorded in 1838 and 1886 are equivalent to water levels under existing channel conditions of about 583.1 feet NGVD and 582.1 feet NGVD, respectively. These nineteenth century maximum monthly water levels are generally equivalent to the monthly mean water level of 583.2 feet NGVD measured in October 1986.

There are five modest artificial diversions on the Great Lakes which change the natural supply of water to the lakes or which permit water to bypass a natural lake outlet, as shown on Map 20. These are the Long Lac, Ogoki, and Chicago diversions; the Welland Canal; and the

Figure 21

TYPICAL PATTERN OF WAVES APPROACHING A BEACH

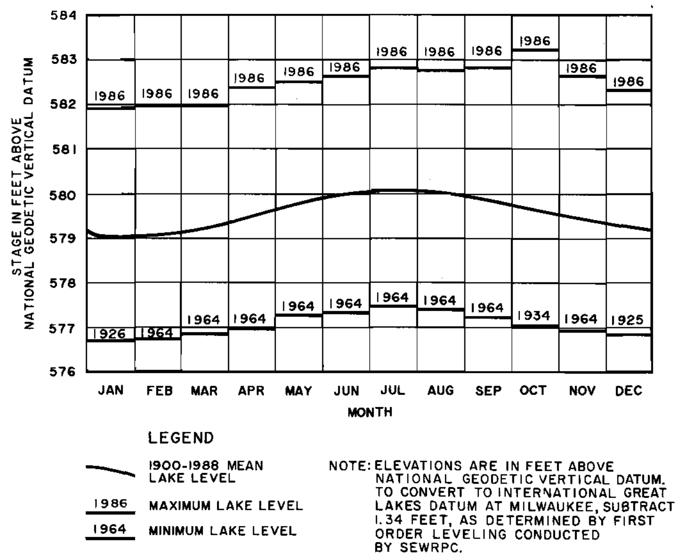


Note: ≅ denotes approximately

Source: S. N. Hanson, J. S. Perry, and W. Wallace, Great Lakes Shore Erosion Protection—A General Review with Case Studies, Wisconsin Coastal Management Program, 1977.

Figure 23

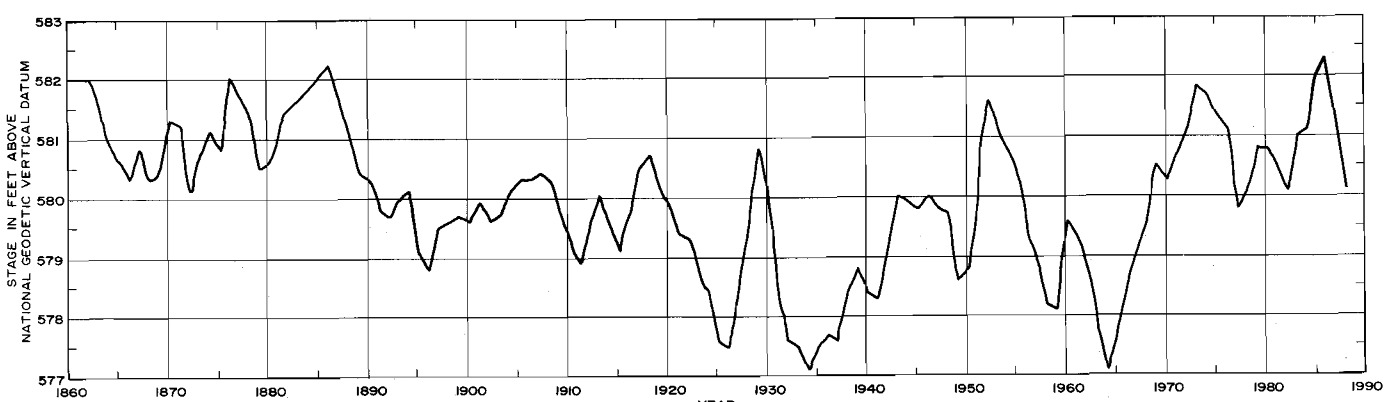
VARIATION IN MONTHLY MEAN WATER LEVELS FOR LAKE MICHIGAN AT MILWAUKEE: 1900-1987



Source: National Ocean Service and SEWRPC.

Figure 22

LAKE MICHIGAN ANNUAL MEAN WATER LEVELS AT MILWAUKEE: 1860-1987



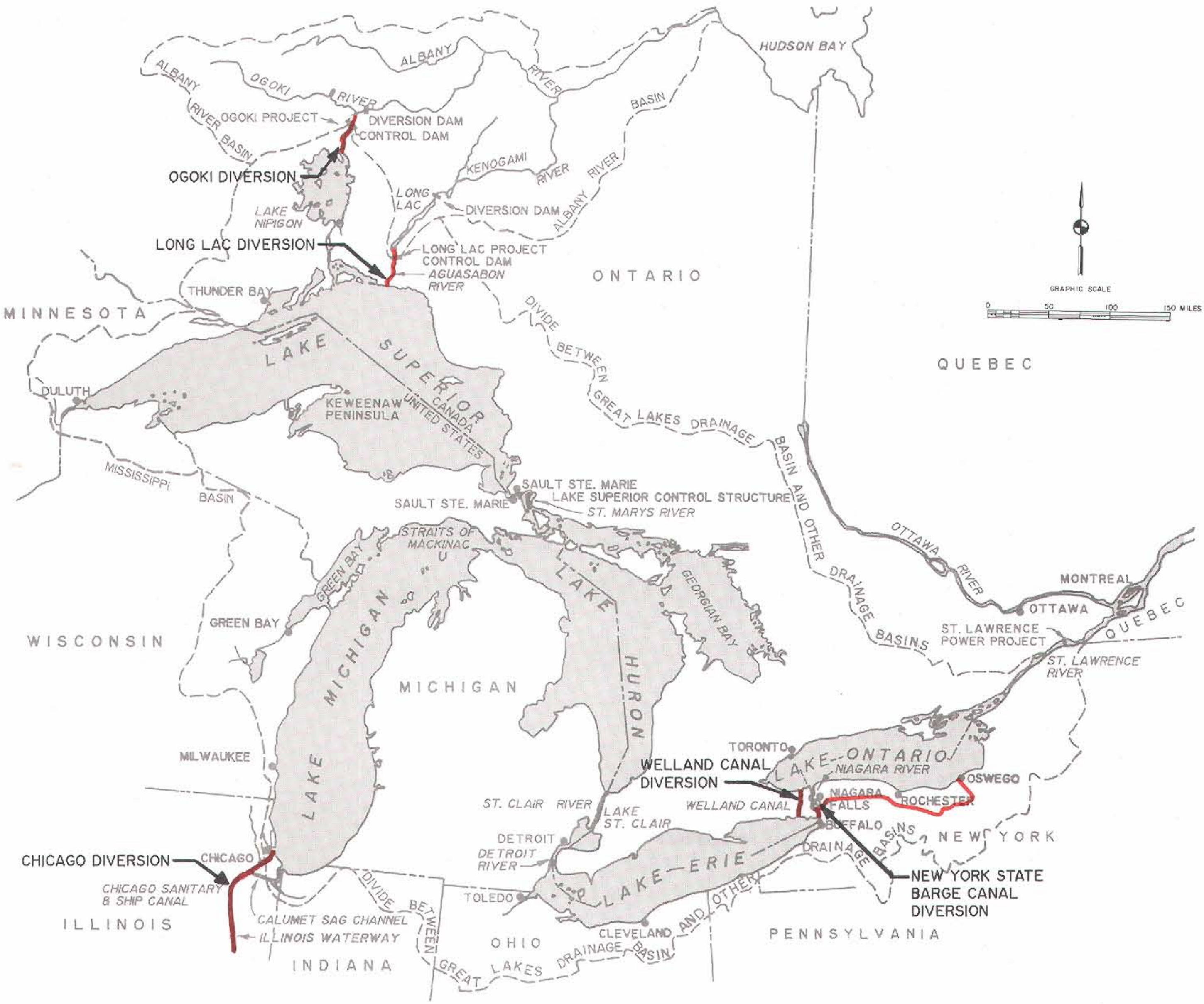
NOTE: ELEVATIONS ARE IN FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM. TO CONVERT TO INTERNATIONAL GREAT LAKES DATUM, SUBTRACT 1.34 FEET.

Source: National Ocean Service and SEWRPC.

New York State Barge Canal. Both the Ogoki and Long Lac diversions divert into Lake Superior water from the Albany River Basin which would otherwise drain to Hudson Bay. These two diversions were developed for the primary purpose of generating hydroelectric power. The Chicago diversion from Lake Michigan serves to dilute sewage effluent from the Chicago Sanitary District and divert the effluent from the lake. The diversion also facilitates navigation on the Chicago Sanitary and Ship Canal and hydroelectric power generation in Illinois. The Welland Canal diverts water from Lake Erie across the Niagara Peninsula to Lake Ontario, thereby bypassing the Niagara River and Niagara Falls, primarily for purposes of navigation and hydroelectric power generation. The New York State Barge Canal diverts water primarily for

Map 20

GREAT LAKES DRAINAGE BASIN



Source: U. S. Army Corps of Engineers.

navigation purposes from Niagara River at Tonawanda, New York, ultimately discharging it to Lake Ontario and the Hudson River.

Water levels in the Great Lakes can be partially regulated by means of artificial outlet control structures. Currently, two of the Great Lakes, Superior and Ontario, are regulated under plans approved by the International Joint Commission. The regulation of Lake Superior affects the entire Great Lakes system, whereas the regulation of Lake Ontario does not affect the other lakes because of the sheer drop in the water level at Niagara Falls. Additional regulation of water levels on Lakes Michigan, Huron, and Erie has been proposed as one method of alleviating shoreline erosion caused by high water levels. Increased regulation of the water levels could be accomplished by dredging to increase the hydraulic capacity of the lake outlet channels; by modifying existing diversions into and out of the lakes; and by constructing new diversions.

The governments of the United States and Canada, in August 1986, requested that the International Joint Commission undertake a comprehensive study of methods of alleviating the adverse impacts of changing water levels, ranging from very high to very low levels, on the Great Lakes/St. Lawrence River Basin.[16] The study involves two phases. The first phase of the study, scheduled for completion in May 1989, includes a characterization of water level fluctuations and their environmental, social, and economic consequences; and the identification and description of potential lake level management measures.

The second phase, which is scheduled to be completed in September 1991, is to include a comprehensive evaluation of potential solutions, including structural improvements, land use planning, and other management activities. In this regard, it should be noted that the governors of the Great Lakes states, as members of the Council of Great Lakes Governors, in 1986 voiced support for avoiding the further diversion of water from the Great Lakes. These concerns will have to be considered in any study of the potential regulation of Lake Michigan.

Century-record-high lake levels at Milwaukee were experienced in 1986. These high lake levels were caused by unusually large amounts of precipitation. As shown in Figure 23, record monthly highs were set for Lake Michigan at Milwaukee in 1986 for one year straight. There has been a significant decline in the level of Lake Michigan since the record high levels of October 1986. The mean level of Lake Michigan at Milwaukee for January 1988—580.13 NGVD—was 3.06 feet lower than the mean for January 1986. The recent decrease is attributable to persistently below-average precipitation.

It is important to note that despite the substantial decline since October 1986, severe storms could still result in flooding. During the storm of March 9, 1987, the level of Lake Michigan at Milwaukee rose to 584.3 feet NGVD, the same as the U. S. Army Corps of Engineers revised 100-year recurrence interval flood stage.[17] The lake level remained above 583.0 feet NGVD for much of that day, countering much of the previously observed lake level decline.

The recent period of below-average precipitation and declining lake levels does not necessarily indicate that Lake Michigan will continue to decline and remain at lower levels. In the future, lake levels may be expected to continue to fluctuate substantially in response to climatic variations, as has historically occurred. During the twentieth century, one similar period of lake level decline was followed by an extended period of low water levels, while another such decline was followed by an extended period of high water levels. A 2.1-foot decline in the seasonal high monthly mean level of Lake Michigan between 1930 and 1931 was followed by more than 10 years of low water levels. Conversely, a 1.9-foot decrease in the seasonal high monthly mean level of the lake between 1976 and 1977 was followed by one year of average water levels and, subsequently, by almost a decade of rising levels, reaching record high levels in 1986.[18]

Finally, it should be recognized that the period during which Great Lakes water levels have been systematically recorded—since 1860—is relatively short. Geological evidence is believed by some to indicate that within the last 1,500 years, there have been at least three episodes in which Lake Michigan water levels have substantially exceeded the 1986 record high annual mean lake level of 582.5 feet NGVD. Interpretation of such evidence is a complex and uncertain process given the crustal movement taking place in the Great Lakes area. High water levels are believed to have occurred sometime during the

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[16] *International Joint Commission, Plan of Study Concerning the Reference on Fluctuating Water Levels into the Great Lakes-St. Lawrence River Basin, March 15, 1988.*

[17] *U. S. Army Corps of Engineers, Revised Report on Great Lakes Open Coast Flood Levels, Detroit, Michigan, 1988.*

[18] *J. Philip Keillor, "Lake Level Update No. 22," Sea Grant Institute, University of Wisconsin-Madison, June 10, 1987. Lake level data in that document pertain to the master gage for Lakes Michigan-Huron located at Harbor Beach, Michigan.*

periods 480 to 610 AD, 1000 to 1150 AD, and 1580 to 1720 AD.[19]

The lake level estimates are based upon radiocarbon-dated stratigraphic studies of a beach ridge complex located along the southwestern shore of Lake Michigan, and indicate that maximum levels over the past 1,500 years may have historically ranged from one to nearly eight feet above the record high 1986 annual mean lake level. Other researchers have concluded, however, based upon historical archaeological and geo-botanical information generally more recent than Larsen's data, that the water levels of Lake Michigan during the seventeenth and eighteenth centuries and dating as far back as the 1640's were not significantly different from those recorded in the nineteenth and twentieth centuries. Bishop[20] concluded that the overall variation in the mean annual levels of Lake Michigan over the past 350 years has not differed substantially from the variation in such levels measured since 1860.

A recent study of historical summer—June and July—water supplies to the Great Lakes reconstructed from tree ring data concluded that variations in net basin supplies to the Great Lakes in the late eighteenth century and in the nineteenth century were similar to those which have been recorded in the twentieth century.[21] Although data presented in the study indicated that net basin supplies to some of the Great Lakes in the eighteenth and nineteenth centuries were at times greater than such supplies in the twentieth century, the Lake Michigan peak net basin supplies were similar in the eighteenth, nineteenth, and twentieth centuries. The study identified a strong correlation between the individual Great Lakes in net basin supplies—i.e., there was a tendency for all of the lakes to have high supplies at the same time.

Ice Formation: Ice formation tends to contribute to a seasonal cycle in bluff erosion. When stationary ice develops along the shore in winter, it may serve as a temporary protective barrier against wave action associated with winter storms, thereby reducing bluff erosion. When the ice is not stationary against the shore, however, floating ice chunks can scour the beaches and the bluff toe, thereby reducing the ability of the beach to dissipate wave energy and contributing to toe erosion. Floating ice fields, depending on wind conditions, may develop along the coast. Ice can also cause damage to structures that have been installed to protect the beach and bluff. Freeze-thaw activity may also increase slope failure by causing solifluction.

Groundwater Seepage: Groundwater seepage can also affect bluff stability in several ways. In most areas along the Milwaukee County shoreline, groundwater moves toward the lake and, in some places, discharges either at the toe of the bluff or from the bluff face. Saturated soil conditions decrease the grain-to-grain contact pressure in the soil and reduce the frictional resistance of the material to stress. Groundwater also adds weight to the bluff, further increasing stress on the slope. In addition, groundwater seepage creates a seepage pressure in the direction of water flow. This pressure is of particular importance in granular soils such as sands and silts and is of lesser importance when the clay content of the soils is fairly high. If groundwater actually discharges from the bluff face, some undercutting of materials may also occur. Removal of bluff materials by groundwater is especially important when sand layers either are interbedded with fine-grained materials or are present at the bluff top. When a layer of permeable sand is present on the top of the bluff, large amounts of water percolate through the sand until a less permeable material is reached, and the water then travels laterally toward the bluff face. Sapping of material may occur at the bottom of this permeable layer.

Vegetative Cover: Vegetation can also have an effect on bluff stability and erosion. The aboveground portion of vegetation physically inter-

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[19]*Curtis E. Larsen, Unpublished report distributed at the Colloquium on Great Lakes Levels, Water Science and Technology Board of the National Research Council, Chicago, Illinois, March 17-18, 1988.*

[20]*Craig T. Bishop, Great Lakes Water Levels: A Review for Coastal Engineering Design, National Water Research Institute Contribution 87-18, Environment Canada; Burlington, Ontario, 1987.*

[21]*W. A. R. Brinkmann, "Water Supplies to the Great Lakes-Reconstructed from Tree-Rings," Journal of Climate and Applied Meteorology, Vol. 26, No. 4, April 1987, pp. 530-538.*

cepts raindrops, thereby reducing their potential to loosen particles on the bluff face, reducing the impact of wind, and serving to trap windblown sediment. The underground portion of vegetation serves to bind the unconsolidated material in place, to prevent slippage between soil layers parallel to the bluff face, and to retard surface wash and filter out the sediment carried by that wash. Vegetative cover, therefore, may effectively reduce sheet and rill erosion and shallow translational sliding. Transpiration through vegetation can also help to remove groundwater from the bluff, and thereby contribute to its stability. Vegetation on the top of the bluff may serve to intercept and divert some surface runoff, thus preventing it from moving down the bluff face. The roots of vegetation, however, may induce infiltration by slowing runoff and providing infiltration passages into the bluff face, thereby possibly contributing to a decrease in bluff stability as a result of increased groundwater content and level. Probably one of the most significant aspects of the lack of vegetation on a bluff face is that it serves as an effective indicator of recent erosion.

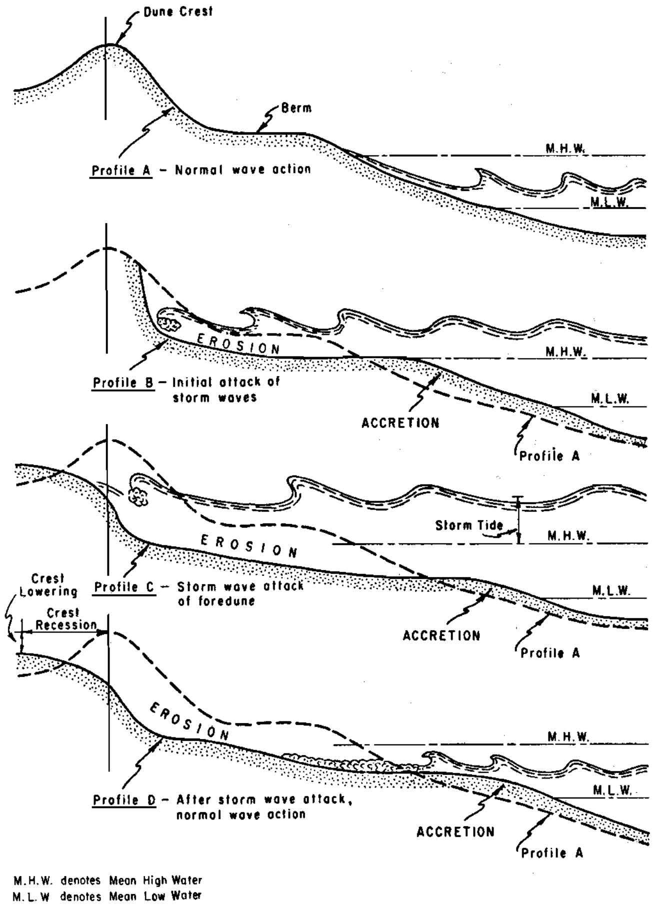
Beach Erosion

The features of a beach and the materials composing the beach are continuously in a state of flux as a result of the near-shore transport of sand and gravel, primarily in response to wave action. There is a constantly changing interaction between the forces that bring sand ashore and those that move it lakeward, with the position and configuration of the main mass of sand at any time serving as an index of the dominant forces. Large waves which often occur during storm events tend to erode beaches by removing material from them and transporting it in a lakeward direction. In contrast, the small waves—characteristic of periods between storm events—tend to build beaches up through a net landward transport of sediment. Thus, the beaches exhibit a continuous cyclic pattern of erosion and accretion in response to the nature of the waves impinging on the beach. Figure 24 shows the process of beach erosion in response to the impact of high, steep waves. A beach is said to be stable, even though subject to storm and seasonal changes, when the long-term—several years or more—rates of supply and loss of material are approximately equal.

Sediment is transported parallel to the shoreline along the beach by longshore currents. Longshore currents are currents in the breaker zone running generally parallel to the shoreline and usually caused by waves breaking at an angle to the shoreline. Longshore currents transport sediment, which is suspended in the current or bounced and rolled along the lake bottom, parallel to the shore. While the longshore currents within the coastal zone of Milwaukee County may move in either a northerly or southerly direction in response to the direction of the incident waves, the net sediment transport is to the south. Evidence of this fact is the tendency for beaches to exhibit accretion on the north side of groins, piers, and other structures while erosion occurs on the southerly side of such structures. The net southward transport

Figure 24

BEACH EROSION IN RESPONSE TO WAVE ACTION



Source: U. S. Army Corps of Engineers.

rate of littoral materials moving along the Milwaukee County shoreline is estimated to be on the order of 8,000 cubic yards annually.[22]

EXISTING REGULATIONS PERTAINING TO SHORELAND DEVELOPMENT

The State of Wisconsin and the federal government have long been involved in the protection of public rights on navigable waters, while more recently water quality has become an important management concern. Of particular concern for coastal erosion management are the means by which state and federal agencies regulate various activities affecting the protection of the Lake Michigan shoreline. In addition, Milwaukee County and the local communities have regulatory authority concerning certain types of shore protection and development measures within the study area shoreline.

The U. S. Army Corps of Engineers is the primary federal agency responsible for the regulation of structures and work related to surface waters. Initial Corps authority to regulate structures or work in, or affecting, navigable waters stems from the River and Harbor Act of 1899. Corps regulatory authority was expended with the passage of the Federal Water Pollution Control Act amendments in 1972. Section 404 of this act authorized the Corps to administer a permit program to regulate the deposition of dredged and fill materials into waters and related wetlands of the United States, as well as to regulate the construction of shore protection structures.

The State of Wisconsin, through the Department of Natural Resources (DNR), regulates shore protection-related activities under the provisions of Chapter 30 of the Wisconsin Statutes. State regulatory authority with respect to shore protection and erosion control projects is largely confined to projects initiated at or below the ordinary high-water mark. For example, Chapter 30 provides for the establishment of bulkhead lines by local units of government, which delineate an artificial shoreline and allow the deposit of materials or filling up to the bulkhead line if standards for the protection of fish, wildlife, and water quality are met. Under Chapter 30, the installation of riprap and shore protection structures on the bed and bank of the water—or the unbroken slope from the ordinary high-water mark—requires a DNR permit. DNR permits are also required to grade or otherwise remove soil from the bank of any navigable body of water where the area exposed would exceed 10,000 square feet; this provision, it should be noted, affects the grading of the bank below and above the ordinary high-water mark and underscores the importance of county and local management of shore protection activities.

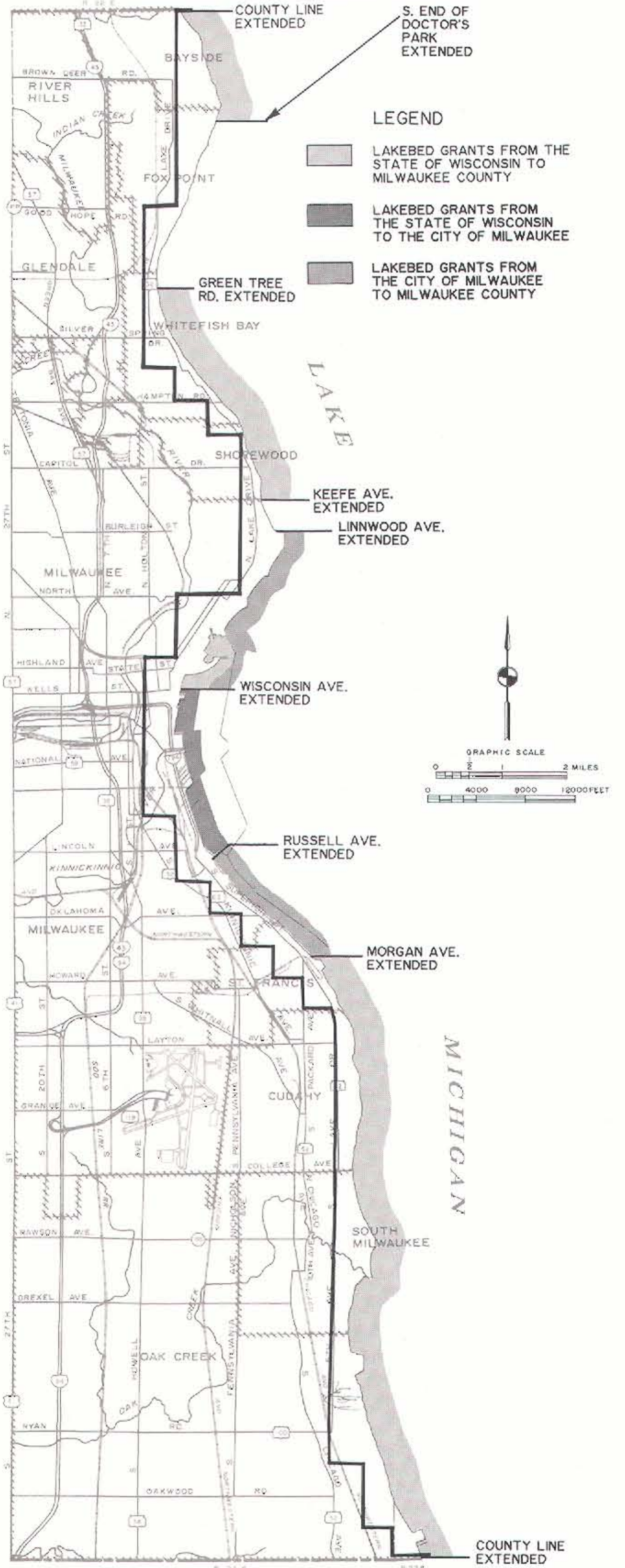
Although the Department of Natural Resources regulates shore protection-related activities throughout most of the Lake Michigan shoreline of the State, 93 percent of the immediate shoreline in Milwaukee County is regulated under lakebed grants made to the City of Milwaukee or to Milwaukee County between 1909 and 1973. The only two shoreline areas not regulated under lakebed grants are the 2,920-foot reach of shoreline just north of the City of Milwaukee Linnwood Avenue water treatment plant, and the 9,070-foot reach of shoreline along the Fox Point terrace near N. Beach Drive.

The lakebed grants made to the City of Milwaukee or to Milwaukee County govern submerged lands extending into Lake Michigan, and under the terms of the grants are to be held and used by the City or County for navigation or harbor facilities, public parks, or highway purposes. The shoreline areas included within the lakebed grants issued to the City of Milwaukee or to Milwaukee County are shown on Map 21. To protect the public interest within the County lakebed grant areas, the County administers a permit program for shore protection measures and dredge and fill activities which requires the submittal of a plan and that certain conditions established by the County be met. The City of Milwaukee, under Chapter 8 of the Code of Ordinances, requires that a city permit be obtained for the construction of dock improvements within the city lakebed grant areas. Along the entire shoreline of Lake Michigan within the State of Wisconsin, including the lakebed grant areas, the Wisconsin Department of Natural Resources has the authority under Section 401 of the Federal Water Pollution Control Act to review and grant water quality certification of

[22] *U. S. Army Corps of Engineers, Lake Michigan Shoreline, Milwaukee County, Wisconsin, March 1975.*

Map 21

SUBMERGED LAKEBED GRANTS WITHIN MILWAUKEE COUNTY



Source: Wisconsin Department of Natural Resources, Milwaukee, County, and SEWRPC.

federal actions which require a permit under Section 404 of the Act. This review, administered under Chapter NR 299 of the Wisconsin Administrative Code, is conducted to determine if the proposed activity will result in a discharge of wastes to surface waters, result in violations of applicable water quality standards, or interfere with public rights and the public interest. The State of Wisconsin does not regulate shore protection-related activities under Chapter 30 of the State Statutes within the lakebed grant area.

In summary, the construction of shore protection structures may therefore require permits from the U. S. Army Corps of Engineers, Wisconsin Department of Natural Resources, Milwaukee County, and the individual municipalities. A permit from the Corps of Engineers is required for all structures anywhere within the study area which extend below the ordinary high-water mark. However, many smaller structures—those involving the placement of less than one cubic yard of material per linear foot of shoreline for a shoreline length of less than 500 feet—are covered under what is referred to as a Nationwide permit, and the Corps must simply be notified of the proposed construction. Outside the Lake Bed Grant shoreline area, a permit is also required from the Wisconsin Department of Natural Resources for all structures extending below the ordinary high-water mark. Within the Lake Bed Grant shoreline area, water quality certification is required from the Department of Natural Resources, and a permit is required from the City of Milwaukee or Milwaukee County. Shore protection structures may also require building permits and special shore protection permits. In addition, some municipalities require that all trucks hauling fill for shore protection measures acquire a hauling permit. Maintenance of existing shore protection structures generally does not require a permit from the governmental agencies.

EXISTING STRUCTURAL EROSION CONTROL MEASURES

Shoreland structural erosion control measures are intended to reduce coastal erosion by providing an artificial protective barrier against direct wave and ice attacks on the beach and bluff toe, by increasing the extent of the beach to absorb wave energy before the water reaches the bluff, by dissipating wave energy, and/or by stabilizing bluff slopes. Structural protective measures installed by both public agencies and private shoreline property owners are costly and have had varying degrees of success. Some structures were not properly designed or constructed, and many have not been properly maintained, resulting in severe deterioration or disappearance within a period of time much shorter than the life of the facilities they were intended to protect.

Onshore protective structures include bulkheads, revetments, and groins constructed at or near the base of a bluff. Bulkheads, or seawalls, have two functions: 1) to serve primarily as bluff-retaining structures and support the bluff against gravity forces; and 2) to effectively absorb the force of impinging waves. A revetment is a flattened slope surface armored with erosion-resistive materials such as concrete or natural rock riprap, and underlaid by filter cloth or gravel. A groin, which is connected to and built perpendicular to the beach, is intended to partially obstruct the longshore current which results in the accumulation of transported sand on the beach up-current of a structure. Groins can also help contain an artificially nourished beach. The resulting beach absorbs wave energy and reduces toe erosion along the adjacent bluffs. The installation of groins—or any other structure which extends out into the lake—in the coastal system of southeastern Wisconsin can lead to erosion of the beach and bluff immediately downdrift of the structure if there is excessive interception of the littoral drift. All shore protection structures require periodic maintenance, extension, and sometimes replacement.

Breakwaters, islands, and peninsulas are protective structures built out from the shore into deeper water and generally parallel to the shore. They provide dissipation of wave energy, thus reducing bluff toe erosion while reducing the strength of the longshore current immediately landward of the structures. Like groins, however, offshore structures may accelerate beach and bluff erosion downdrift of the protected areas, as sediments settle in the sheltered water behind the structures.

Slope stabilization can be accomplished by using earth-moving equipment to regrade the face of the slope to a flatter, more stable profile, thus accelerating the natural stabilization process. This approach is practical only if sufficient vacant land is available at the top of the bluff to allow a cutback. Fill can also be placed on the face of the bluff to provide a stable slope. Another slope stabilization procedure involves the installation of internal drains to maintain a lowered water table within the bluff face and thus reduce the likelihood of slippage along bluff surfaces. Slope stabilization can also include maintenance of a protective cover of vegetation. Slope stabilization measures usually include a combination of these methods.

A review of the construction of shore protection measures over nearly the past 70 years helps ascertain the extent of protection provided, and the types of structures used. Shore protection structures that were in existence in 1920, 1945, 1975, and 1987 are shown on Map 22. The structures are identified on the maps as revetments, bulkheads, breakwaters, or groins. The lineal extent of each structure type at each time period is presented in Table 24 and illustrated in Figure 25.

In 1920, only 15 percent of the total county shoreline was protected by the structures.[23] The northern half of what is now the Milwaukee Harbor was protected, and a few groins and bulkheads had been placed along the north shore. Offshore breakwaters had been constructed off South Shore Park and along a small portion of the City of Oak Creek, and groins protected the mouth of Oak Creek.

By 1945, 35 percent of the county shoreline was protected.[24] Construction of the Milwaukee Harbor and South Shore breakwater had been completed, and several private property owners in the northern portion of the City of Milwaukee and in the Villages of Shorewood and Whitefish Bay had taken measures to protect their properties. The Lakeside power plant had been constructed in the City of St. Francis, and groin systems had been installed to protect portions of several parks in southern Milwaukee.

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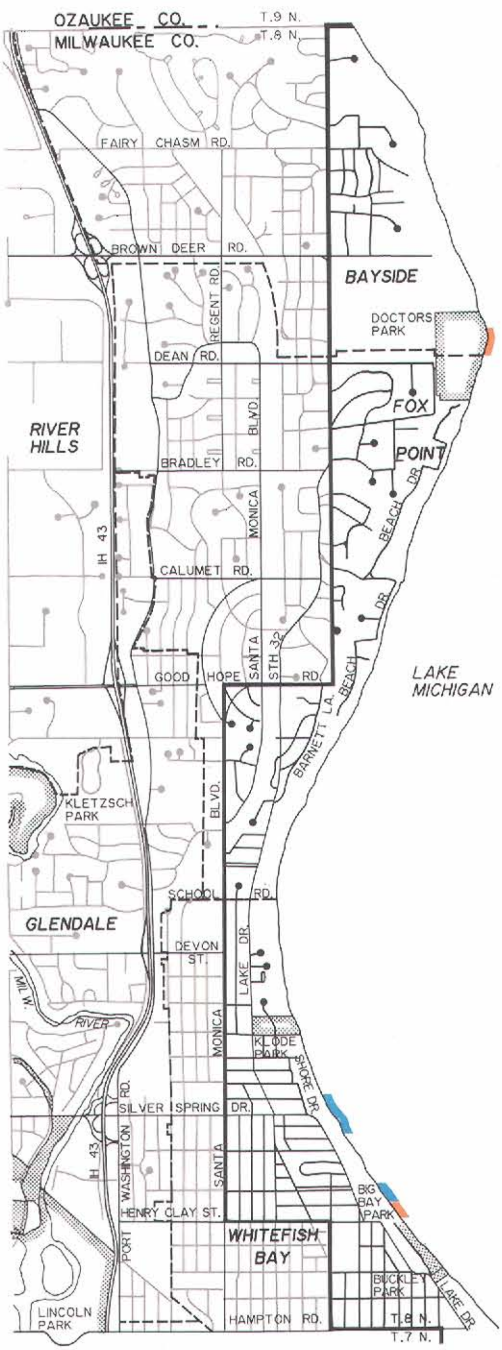
[23]*U. S. House of Representatives Document No. 526, "Beach Erosion Study, Lake Michigan Shoreline of Milwaukee County, Wisconsin," Letter from the Secretary of War, April 1946; and Milwaukee County Committee on Lake Michigan Shore Erosion, Lake Michigan Shore Erosion, Milwaukee County, Wisconsin, October 1945.*

[24]*Ibid.*

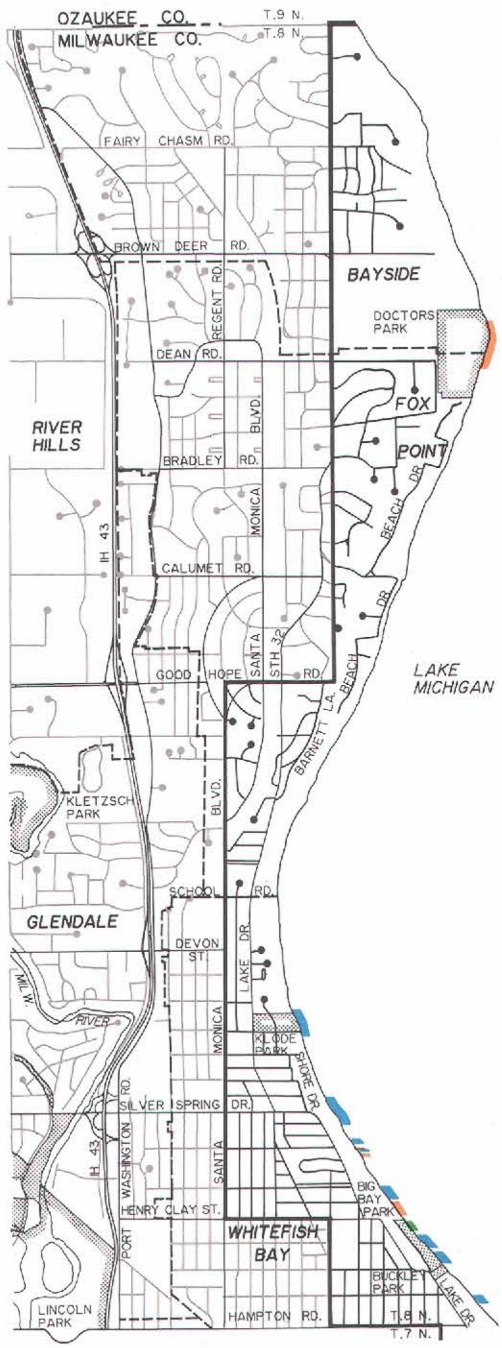
Map 22

HISTORIC EVOLUTION OF SHORE PROTECTION STRUCTURES ALONG THE LAKE MICHIGAN SHORELINE OF MILWAUKEE COUNTY: 1920, 1945, 1975, AND 1987

1920



1945

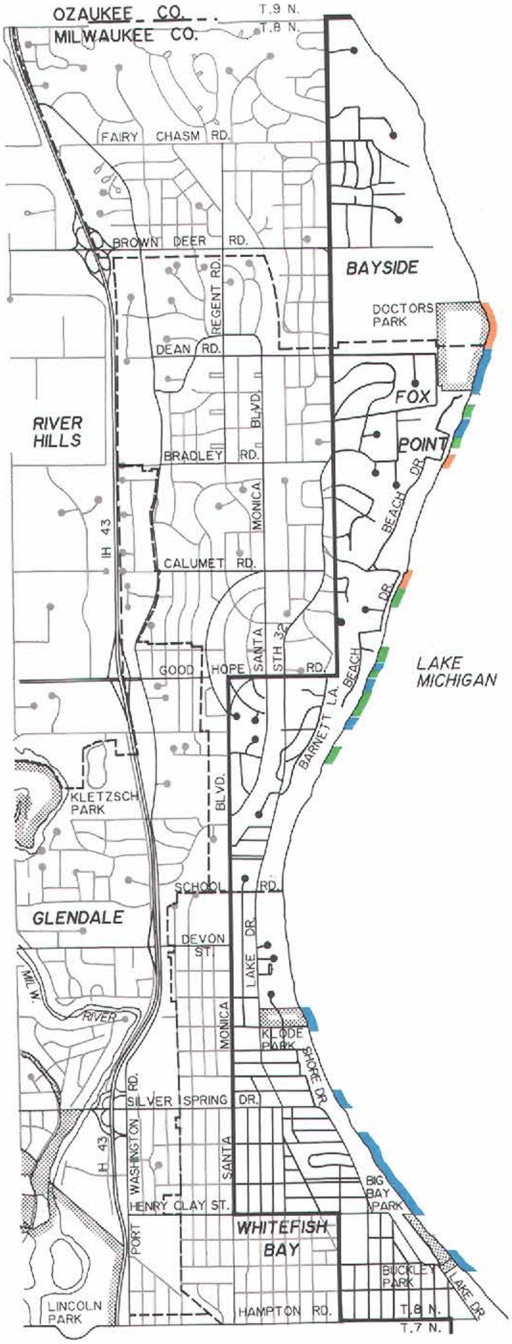


LEGEND

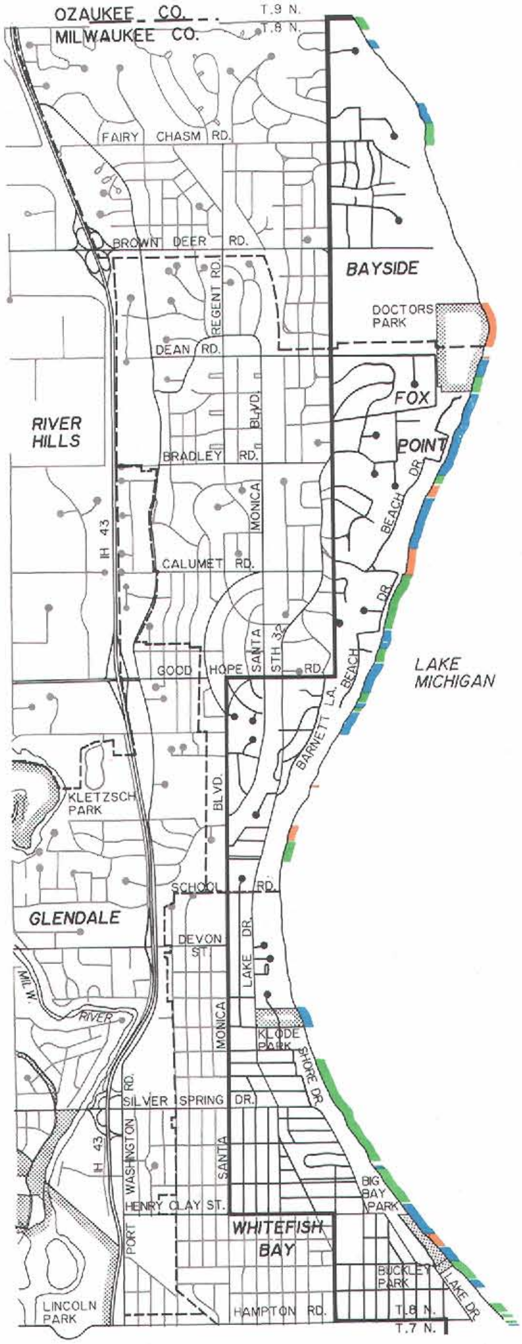
- GROIN
- BULKHEAD
- REVETMENT
- BREAKWATER

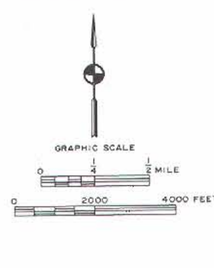
Map 22 (continued)

1975



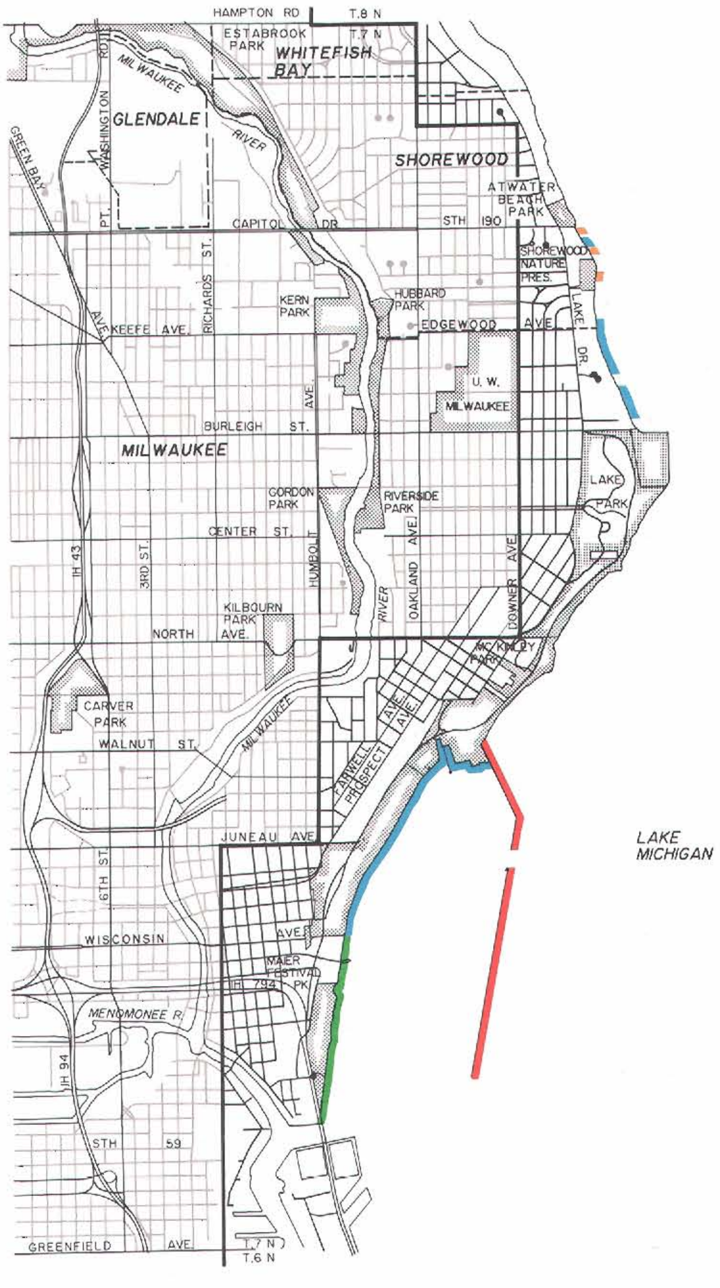
1987



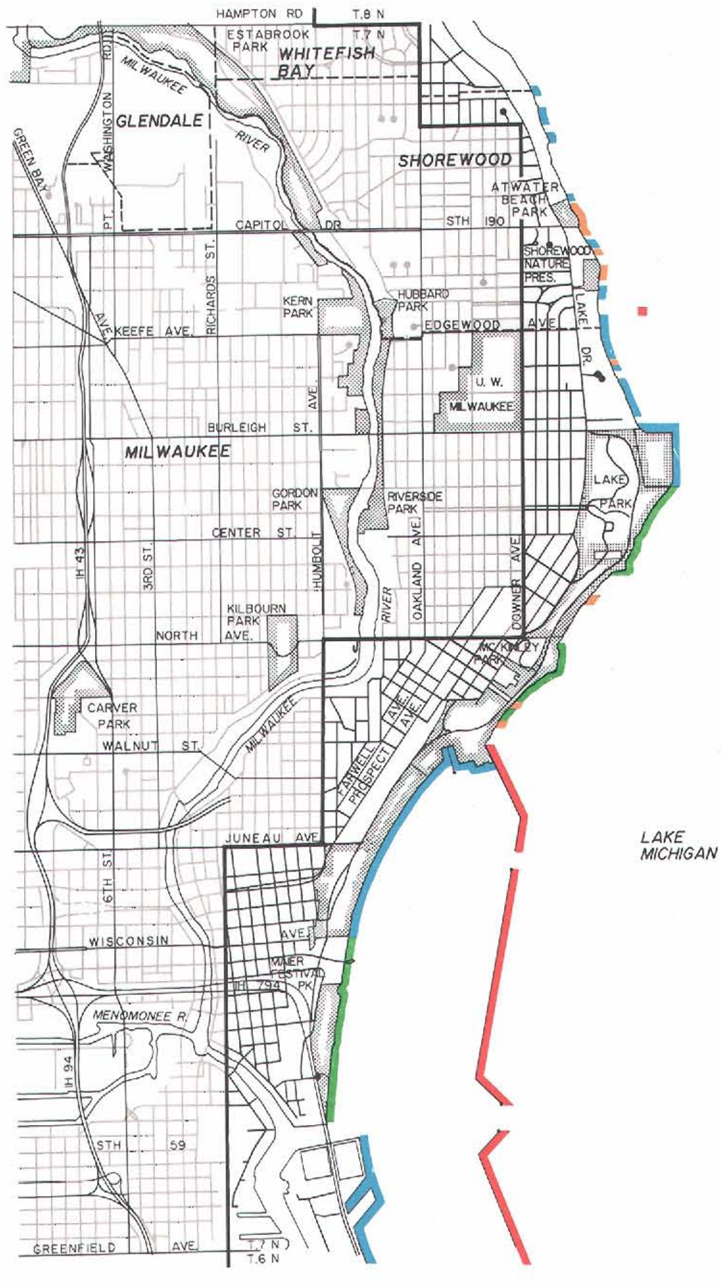


Map 22 (continued)

1920



1945



LEGEND

GROIN

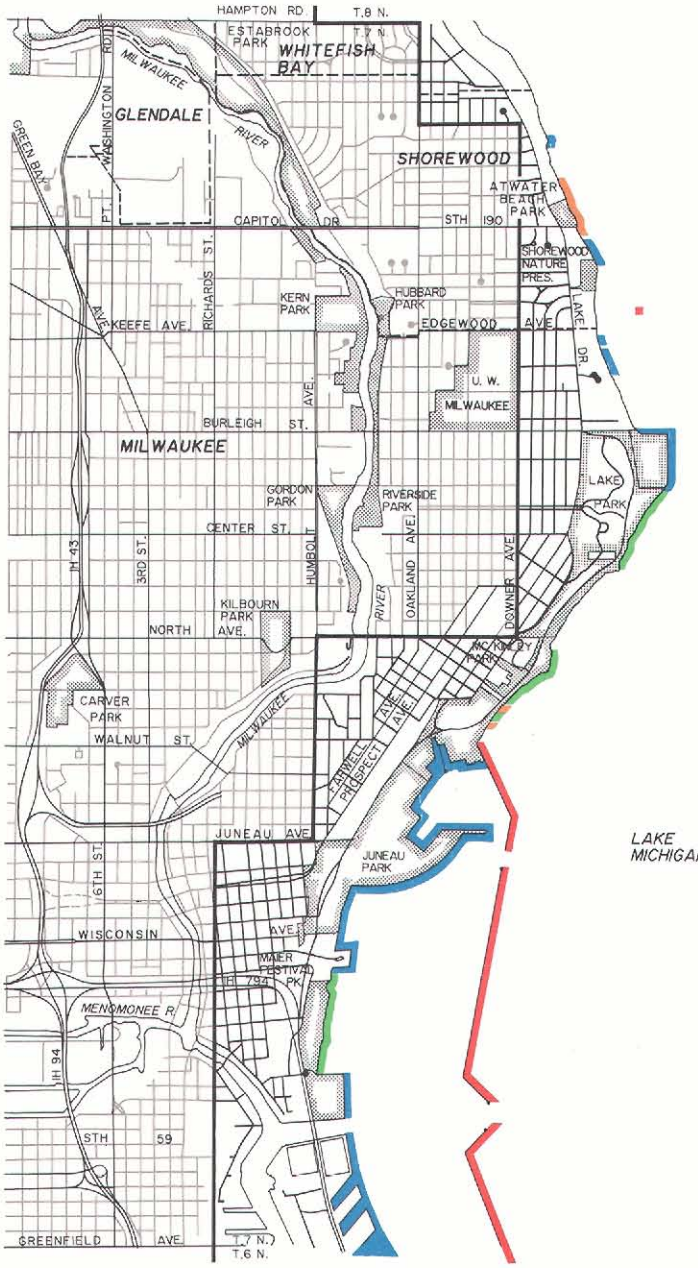
BULKHEAD

REVETMENT

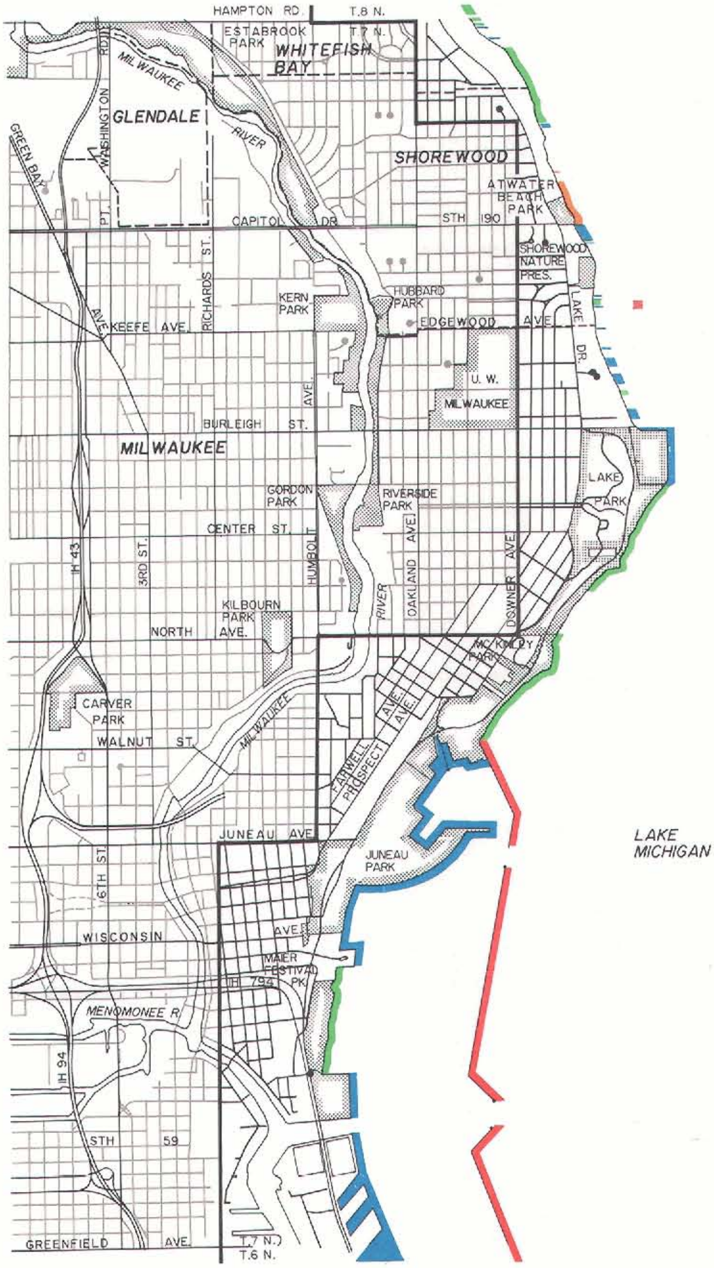
BREAKWATER

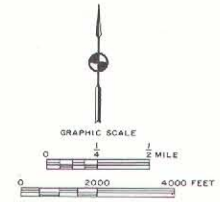
Map 22 (continued)

1975



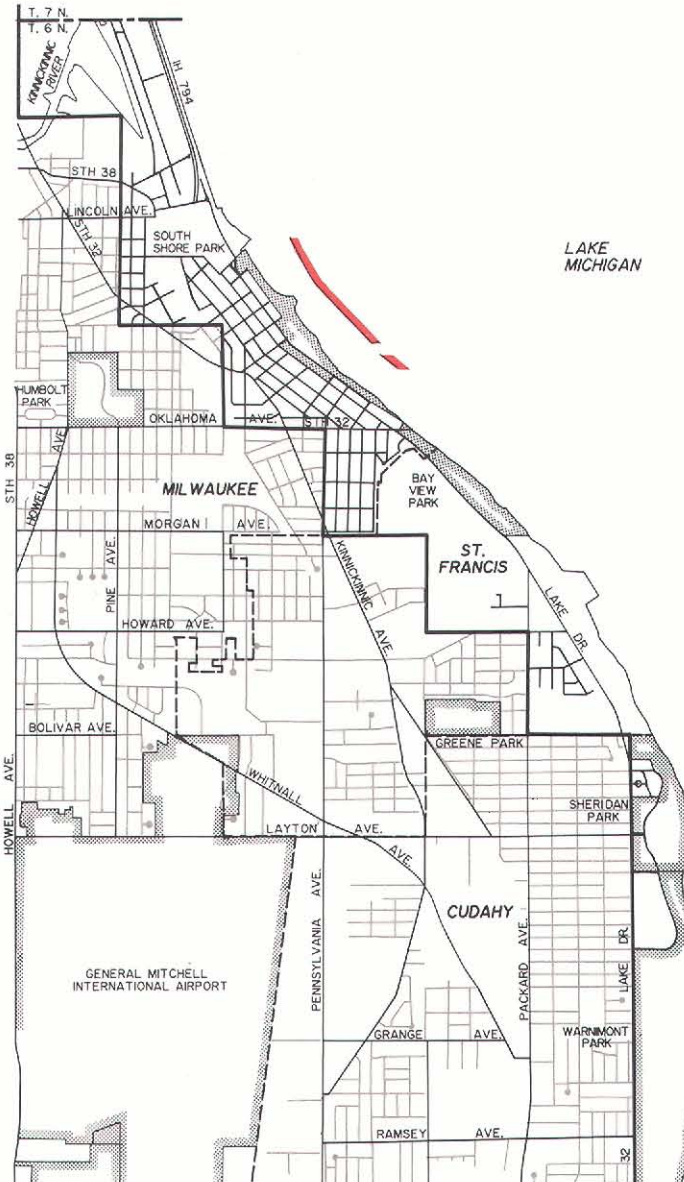
1987



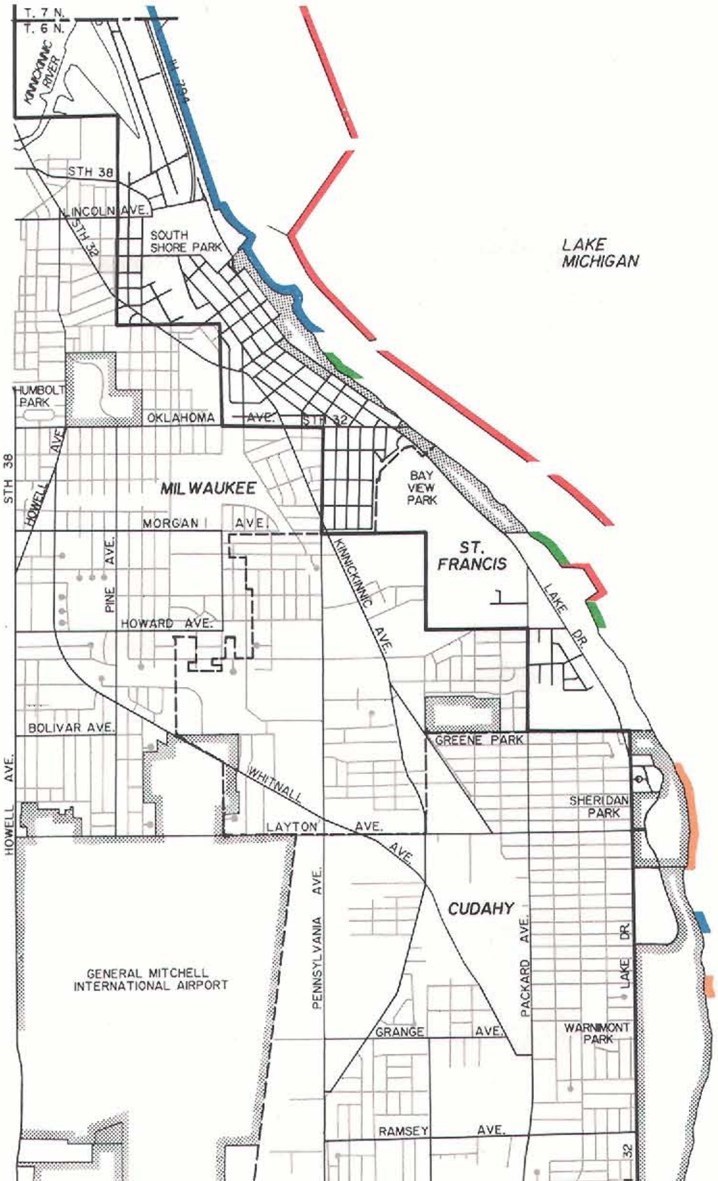


Map 22 (continued)

1920



1945



LEGEND

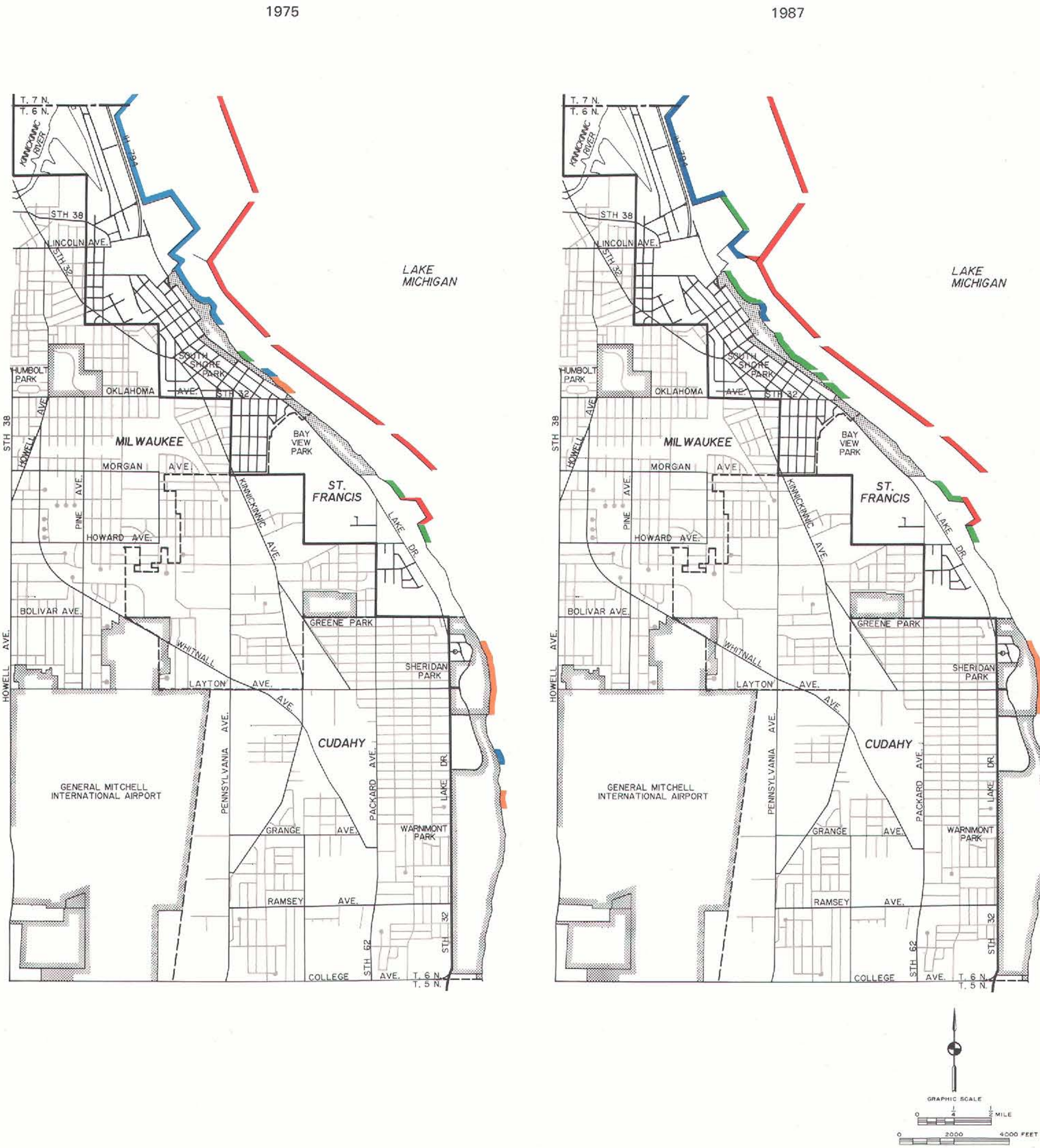
GROIN

BULKHEAD

REVETMENT

BREAKWATER

Map 22 (continued)



Map 22 (continued)

1920

1945

LEGEND

GROIN

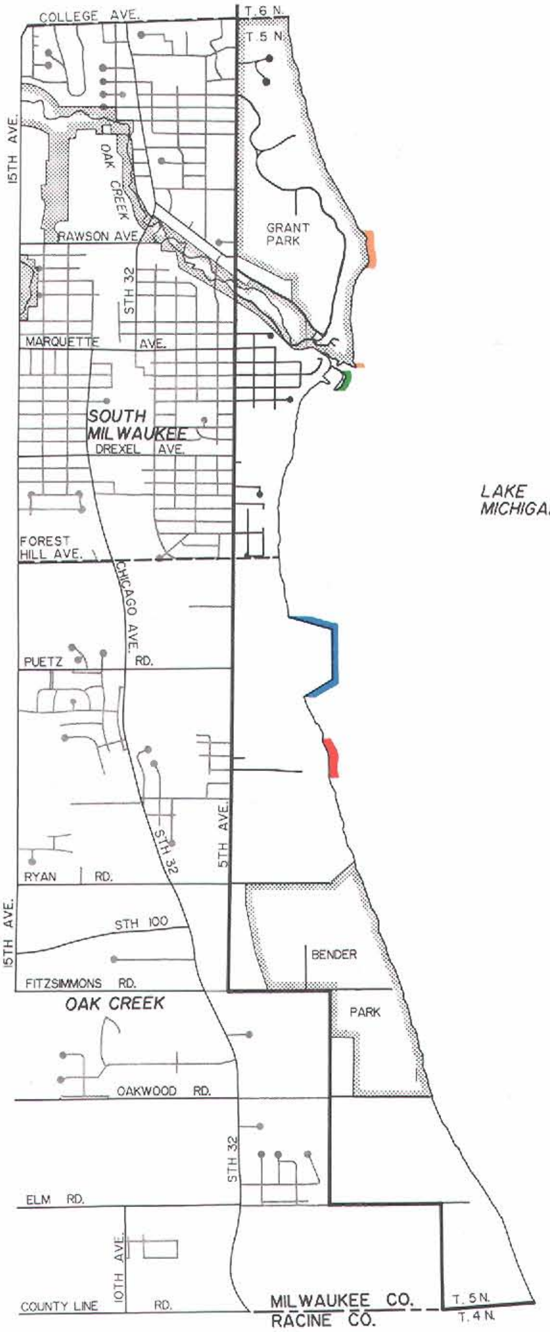
BULKHEAD

REVETMENT

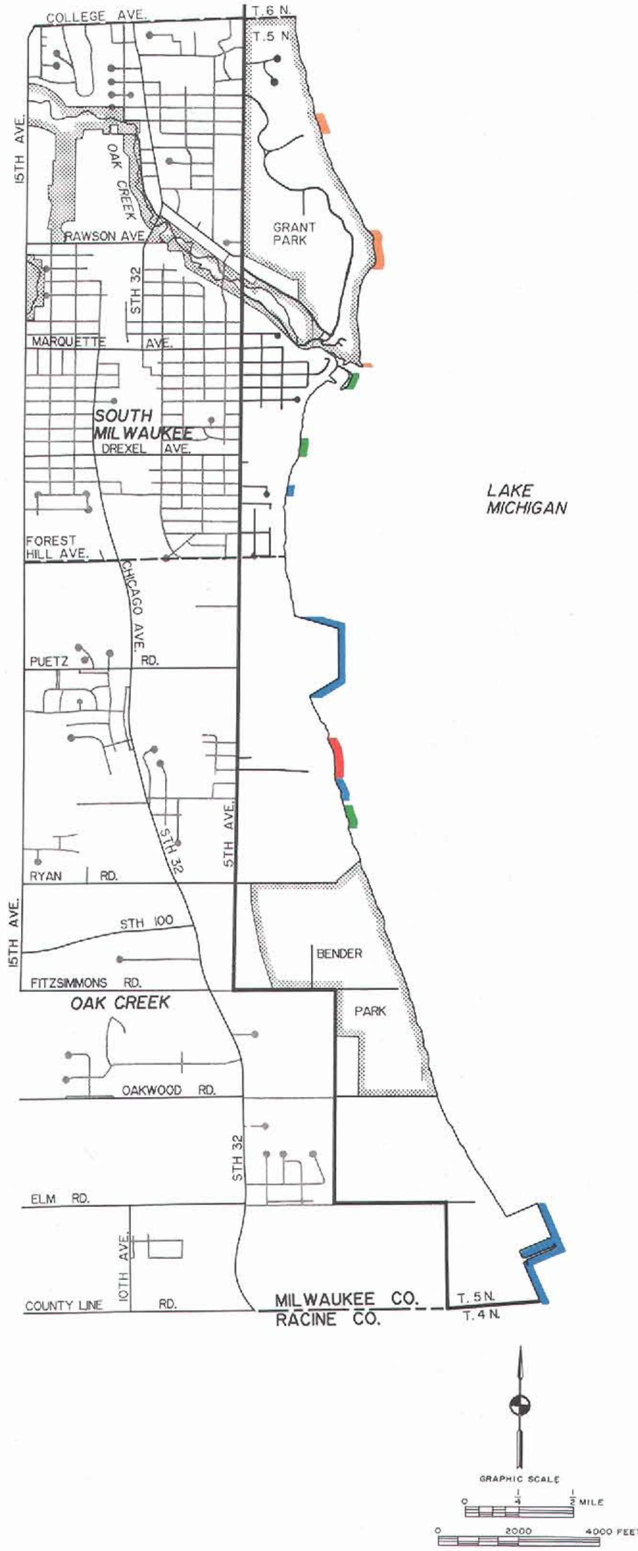
BREAKWATER

Map 22 (continued)

1975



1987



Source: SEWRPC.

Table 24

HISTORICAL DEVELOPMENT OF SHORE PROTECTION STRUCTURES ALONG THE LAKE MICHIGAN SHORELINE OF MILWAUKEE COUNTY: 1920-1987

Year	Length of County Shoreline Protected by Structures (feet)					Percent of Study Area Shoreline
	Revetments	Bulkheads	Groins	Breakwaters	Total Length[a]	
1920	4,000	11,000	3,000	10,000	24,000	15
1945	13,000	29,000	7,000	29,000	55,000	35
1975	12,000	45,000	13,000	29,000	79,000	50
1987	51,000	41,000	9,000	30,000	105,000	66

[a]*Represents the total shoreline protected. Some shoreline areas were protected by more than one type of structure.*

Source: U. S. Army Corps of Engineers, Milwaukee County, and SEWRPC.

About 50 percent of the Milwaukee County shoreline was protected in 1975.[25] The protected shoreline extended northward to include much of the Fox Point terrace off N. Beach Drive. Some structures present in 1945 were no longer in existence in 1975. The Oak Creek power plant and the South Shore sewage treatment plant were constructed in the southern portion of the County.

The surveys conducted under this study indicated that about 66 percent of the Milwaukee County shoreline was protected in 1987. However, as discussed below, many of these shore protection structures are in need of substantial repair, and are not providing adequate protection. Much of the southern portion of the County and the far northern end—in the Village of Bayside—remains unprotected. In general, relatively few new groins, breakwaters, or bulkheads have been constructed over the past decade, except for the installation of several new bulkheads along the Fox Point terrace. Most new structures installed are revetments.

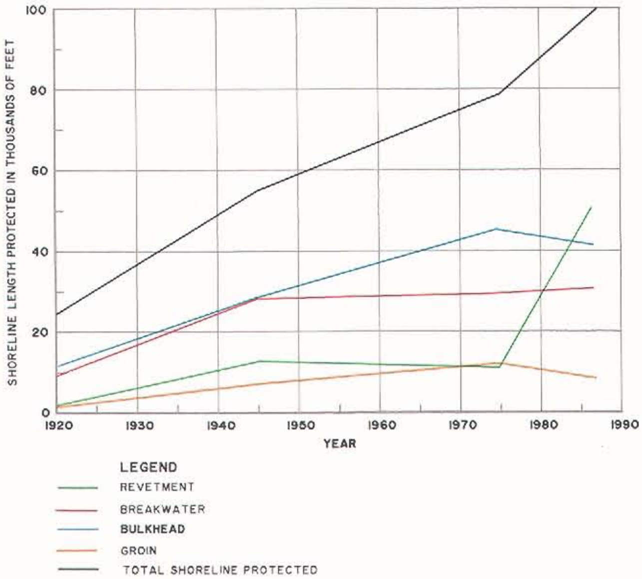
A total of 128 shoreline protection structures located within the study area were surveyed in 1986 and 1987. Of these 128 structures, 43, or 34 percent, were revetments; 61, or 48 percent, were bulkheads; 18, or 14 percent, were groins; and 6, or 4 percent, were breakwaters. Of the total, five, or 4 percent of the structures, were located in the City of Oak Creek; five, or 4 percent, were located within the City of South Milwaukee; three, or 2 percent, were located in the City of Cudahy; three, or 2 percent, were located in the City of St. Francis; 35, or 27 percent, were located in the City of Milwaukee; 12, or 10 percent, were located in the Village of Shorewood; 19, or 15 percent, were located in the Village of Whitefish Bay; 41, or 32 percent, were located in the Village of Fox Point; and five, or 4 percent, were located in the Village of Bayside. As shown on Map 23, approximately 105,000 feet, or 66 percent, of the Milwaukee County shoreline was protected by structures, although some of these structures were not providing adequate protection against shoreline erosion. Of the total protected shoreline, 31 structures covering 44 percent of the shoreline protected recreational and open land; 17 structures covering 34 percent of the shoreline protected land devoted to industrial, transportation, and utility use; 74 structures covering 19 percent of the shoreline protected residential land; and three structures covering 3 percent of the shoreline protected land devoted to commercial and governmental use.

The quality and effectiveness of shore protection structures vary considerably. An inventory of the condition of shore protection structures within the northern Milwaukee County study

[25]*U. S. Army Corps of Engineers, Preliminary Feasibility Report, Lake Michigan Shoreline, Milwaukee County, Wisconsin, March 1975.*

Figure 25

LENGTH OF LAKE MICHIGAN SHORELINE IN MILWAUKEE COUNTY PROTECTED BY SHORE PROTECTION STRUCTURES: 1920-1987



Source: U. S. Army Corps of Engineers, Milwaukee County, and SEWRPC.

area was conducted by the Regional Planning Commission staff in August 1986, and within the remaining Milwaukee County shoreline in November 1987. In addition, a more detailed structural analysis was conducted under contract to the Commission by W. F. Baird and Associates, Ltd., in April 1988 for 23 of the major structures in the County to determine overall structural integrity, identify any apparent signs of damage, describe needed repairs or modifications, and identify apparent problems in the design and/or construction of the structures based on field observation. To supplement this structural analysis, an underwater photographic survey was conducted also under contract to the Commission in May 1988 by Pro Photo, Inc., to assess the degree of toe scour or undercutting occurring at three structures.

The results of these field surveys are presented in Appendix B and summarized in Table 25. The table indicates that about 75 percent of the structures, including 58 percent of the revetments and 60 percent of the bulkheads, had observable failures of some type and at the time of the survey were in need of significant maintenance work. The remaining structures were found to be in good condition. Table 25 also summarizes the type of failures affecting these structures.

The predominant type of structural failure was overtopping, where the water level, or the wave heights, exceeded the top of the structure. Overtopping, which erodes material from behind revetments and bulkheads, and which reduces the effectiveness of groins and breakwaters, affected about 59 percent of the structures inventoried, including about 44 percent of the revetments, 78 percent of the groins, 62 percent of the bulkheads, and 83 percent of the breakwaters. This indicates that most structures either were not constructed high enough for the 1986 high lake levels, or had settled or partially collapsed. Overtopping can frequently result in the ultimate collapse of the structure foundations. Other failure types included flanking—where the sides of the structure are eroded; collapsing; material failure; and toe scour. Flanking affected 20 percent of the structures inventoried, including about 5 percent of the revetments, 36 percent of the bulkheads, and 6 percent of the groins. About 27 percent of the structures surveyed had at least partially collapsed, 20 percent had material failure, and 9 percent were undercut at the structure toe.

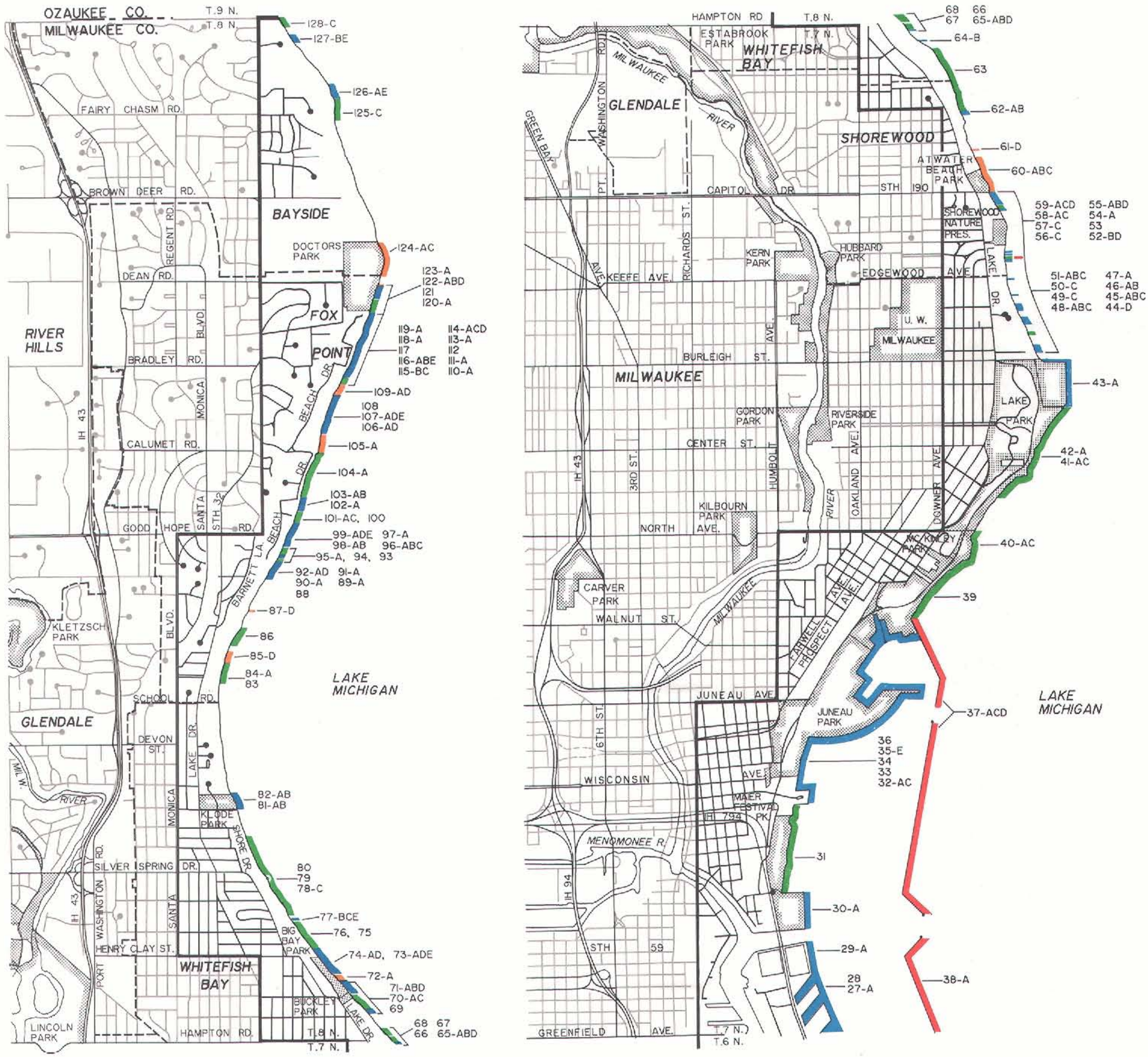
EXISTING SHORELINE EROSION PROBLEMS

Bluff recession results in the loss of extensive land areas; and the sometimes major, unexpected, and rapid slope failures caused by slumping and sliding may pose a threat to human safety. The erosion or accretion of the beaches is a related process in that the extent of the beach affects the degree of wave erosion at the bluff toe. As previously noted, other factors, some of them natural and some of them related to human activity, influence bluff stability either by altering the gravity-induced stresses which tend to cause bluff failure or by affecting the resisting strength factors which tend to maintain bluff stability.

The study area shoreline was divided into 100 sections, each with similar physical and erosion-related characteristics. The locations of these 100 bluff analysis sections are shown on Map 24. The boundaries of the sections are located on property boundary lines. Field surveys were conducted in May 1986 within the northern Milwaukee County study area, which extends from the City of Milwaukee Linnwood Avenue water treatment plant through the Village of Fox Point. Surveys were conducted in October

Map 23

CONDITION OF SHORE PROTECTION STRUCTURES IN THE MILWAUKEE COUNTY LAKE MICHIGAN STUDY AREA: 1986-1988



LEGEND

24 STRUCTURE NUMBER

SHORELINE PROTECTION STRUCTURES

TYPE OF STRUCTURE

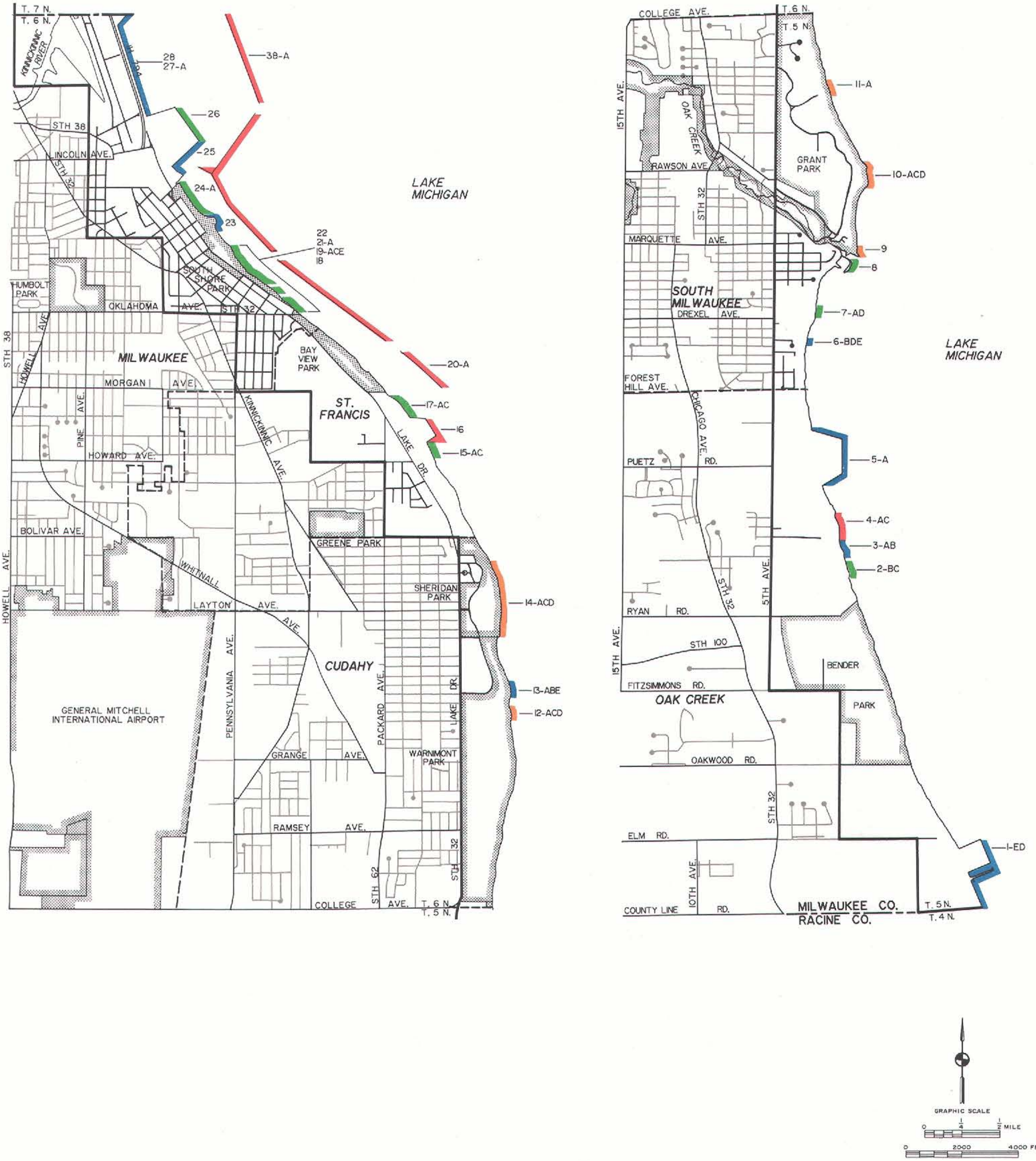
- GROIN
- BULKHEAD
- REVETMENT
- BREAKWATER

TYPE OF FAILURE

- A OVERTOPPING
- B FLANKING
- C COLLAPSE
- D MATERIAL FAILURE
- E TOE SCOUR

NOTE ABSENCE OF LETTER DENOTES NO OBSERVED FAILURE

Map 23 (continued)



Source: SEWRPC.

Table 25

SUMMARY OF MILWAUKEE COUNTY STRUCTURAL SHORE PROTECTION SURVEY: 1986-1987

Maintenance Required	Type of Structure									
	Revetment		Bulkhead		Groin		Breakwater		Total	
	Number	Percent	Number	Percent	Number	Percent	Number	Percent	Number	Percent
Yes	25	58	49	60	17	94	5	83	96	75
No	18	42	12	20	1	6	1	17	32	25
Total	43	100	61	100	18	100	6	100	128	100
Type of Failure[a]										
Toe Scour	1	2	10	16	0	0	0	0	11	9
Overtopping	19	44	38	62	14	78	5	83	76	59
Flanking	2	5	22	36	1	6	0	0	25	20
Collapse	15	35	11	18	6	33	2	33	34	27
Material Failure	0	0	16	26	9	50	1	17	26	20
None	18	42	12	20	1	6	1	17	32	25

[a]*More than one type of failure was observed on some structures.*

Source: SEWRPC.

1987 for the remainder of the County. During these surveys section boundaries were delineated, the physical characteristics of the sections were inventoried, and the causes and types of shoreline erosion and slope failure occurring were identified within each section. Table 26 summarizes the locations and the physical and erosion-related characteristics of each of the 100 bluff analysis sections.

City of Oak Creek

Approximately 22,720 feet of Lake Michigan shoreline, or about 14 percent of the total county shoreline, lies within the City of Oak Creek. The beach widths measured in the fall of 1987 ranged from 0 to greater than 100 feet, with approximately 43 percent of the shoreline having a beach width of less than 10 feet. The bluffs along the shoreline ranged in height from 60 to 120 feet, and generally were composed of Oak Creek till underlain by lake sediments, and then by another layer of Oak Creek till. About 54 percent of the shoreline had a fully vegetated bluff face, and an overall bluff slope of less than 30 degrees.

During the October 1987 field surveys, the City of Oak Creek shoreline was divided into 13 bluff analysis sections based on similar physical and erosion-related characteristics. Groundwater seepage was observed in seven of the 13 bluff analysis sections within the City, which included 12,300 feet, or 54 percent of the shoreline. Seven bluff analysis sections containing 9,400 feet, or 40 percent of the City of Oak Creek shoreline, were observed to have significant bluff toe erosion. Shoreline protection structures were in place within six sections, covering 9,500 feet, or 42 percent of the shoreline. Bluff slope failure observed within the City of Oak Creek was primarily caused by wave action and groundwater seepage. Bluff slope failures generally occurred as shallow slides and small slumps.

City of South Milwaukee

The City of South Milwaukee contains 15,350 feet of Lake Michigan shoreline, or about 10 percent of the total county shoreline. The beach widths measured in the fall of 1987 ranged from 0 to greater than 300 feet, with approximately

17 percent of the shoreline having a beach width of less than 10 feet. The bluffs along the shoreline ranged in height from 50 to 100 feet, and generally were composed of a layer of lake sediment underlain by Oak Creek till, then another layer of lake sediment, and then finally another layer of Oak Creek till. Within a portion of the shoreline area in Grant Park, a layer of New Berlin till is exposed beneath the lower Oak Creek till layer. Only 31 percent of the shoreline had a fully vegetated bluff face and an overall bluff slope of 30 degrees or less.

During the October 1987 field surveys, the City of South Milwaukee shoreline was divided into 12 bluff analysis sections based on similar physical and erosion-related characteristics. Groundwater seepage was observed in eight of the 12 bluff analysis sections within the City, which included 10,500 feet, or 68 percent of the shoreline. Ten bluff analysis sections containing 11,700 feet, or 76 percent of the City of South Milwaukee shoreline, were observed to have significant bluff toe erosion. Shoreline protection structures were present within five sections covering about 7,800 feet, or 51 percent of the shoreline. Bluff slope failure observed within the City of South Milwaukee was primarily caused by wave action and groundwater seepage. Bluff slope failures generally occurred as shallow slides; however, failure by slumping and sapping was also observed.

City of Cudahy

The City of Cudahy contains approximately 14,240 feet of Lake Michigan shoreline, or 9 percent of the total county shoreline. The beach widths measured in the fall of 1987 ranged from 0 to approximately 70 feet, with about 2 percent of the shoreline having a beach width of less than 10 feet. The bluffs along the shoreline ranged in height from 70 to 110 feet and generally were composed of Oak Creek till underlain by lake sediments, and then by another layer of Oak Creek till. A layer of Tiskilwa till is exposed beneath the lower Oak Creek till layer within portions of the shoreline located in Warnimont and Sheridan Parks. Approximately 15 percent of the shoreline had a vegetated bluff face and an overall bluff slope of 30 degrees or less.

During the October 1987 field surveys, the City of Cudahy's shoreline was divided into 12 bluff analysis sections based on similar physical and erosion-related characteristics. Groundwater seepage was observed in 10 of the 12 bluff analysis sections within the City, covering 12,100 feet, or 85 percent of the shoreline. Eight bluff analysis sections containing 11,000 feet, or 78 percent of the City of Cudahy shoreline, were observed to have bluff toe erosion. Shoreline protection structures were present within four sections, providing some protection to the bluff toe along 3,300 feet, or 23 percent of the shoreline. Bluff slope failure observed within the City of Cudahy was primarily caused by groundwater seepage and wave erosion. Bluff slope failures generally occurred as shallow slides; however failure by slumping, solifluction, and sapping was also observed.

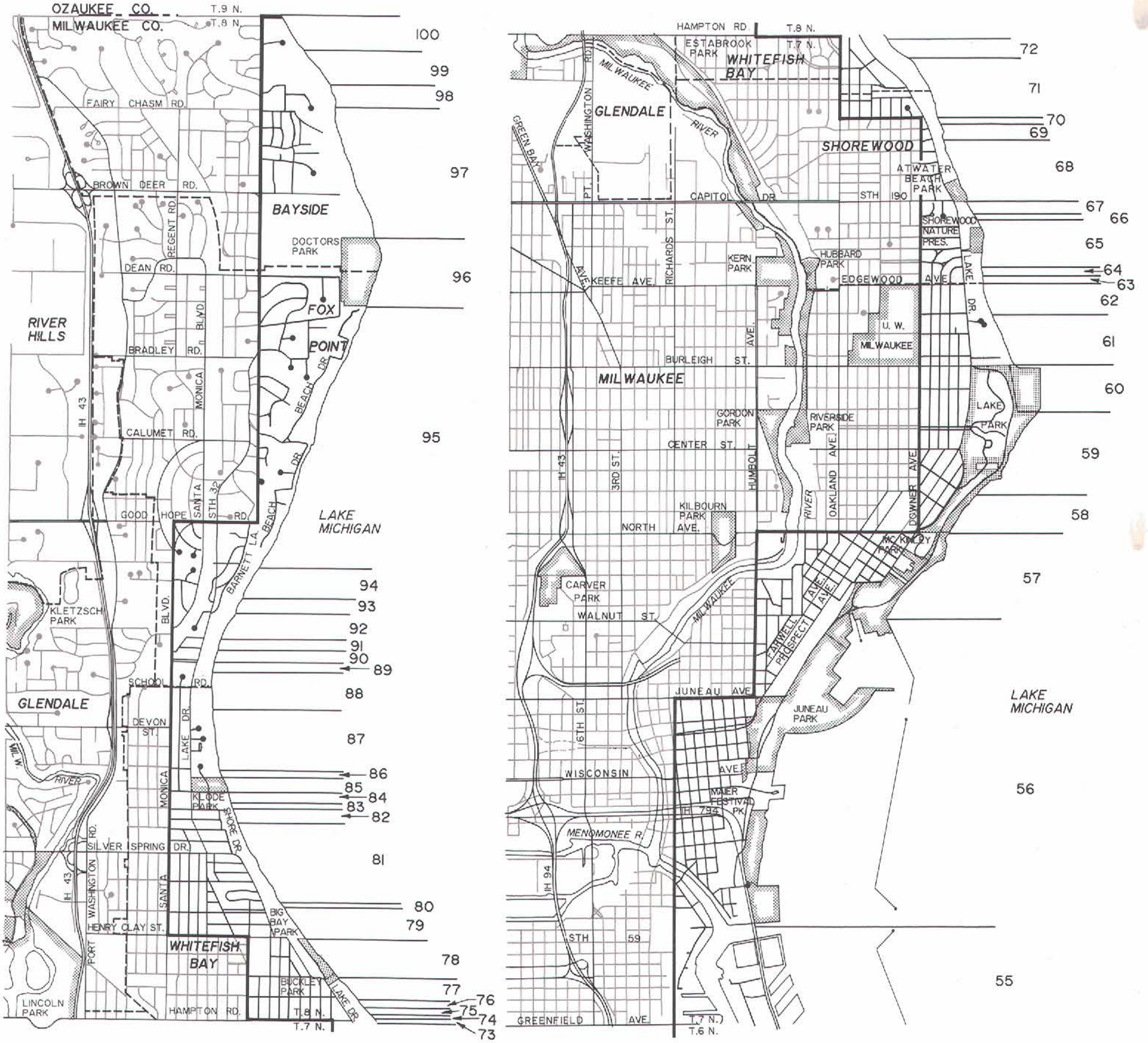
City of St. Francis

The City of St. Francis contains approximately 9,620 feet of Lake Michigan shoreline, or 6 percent of the total county shoreline. The beach widths measured in the fall of 1987 ranged from 0 to 50 feet, with approximately 49 percent of the shoreline having a beach width of less than 10 feet. The bluffs along the shoreline ranged in height from about 40 to 70 feet, and generally were composed of Oak Creek till at the top of the bluff, underlain by lake sediment and then by a second layer of Oak Creek till. Within a portion of the shoreline New Berlin till was exposed beneath the lower Oak Creek till layer. Ozaukee till was also exposed within a portion of the shoreline above the upper layer of Oak Creek till. Only 35 percent of the bluffs were well vegetated and had a bluff slope of 30 degrees or less.

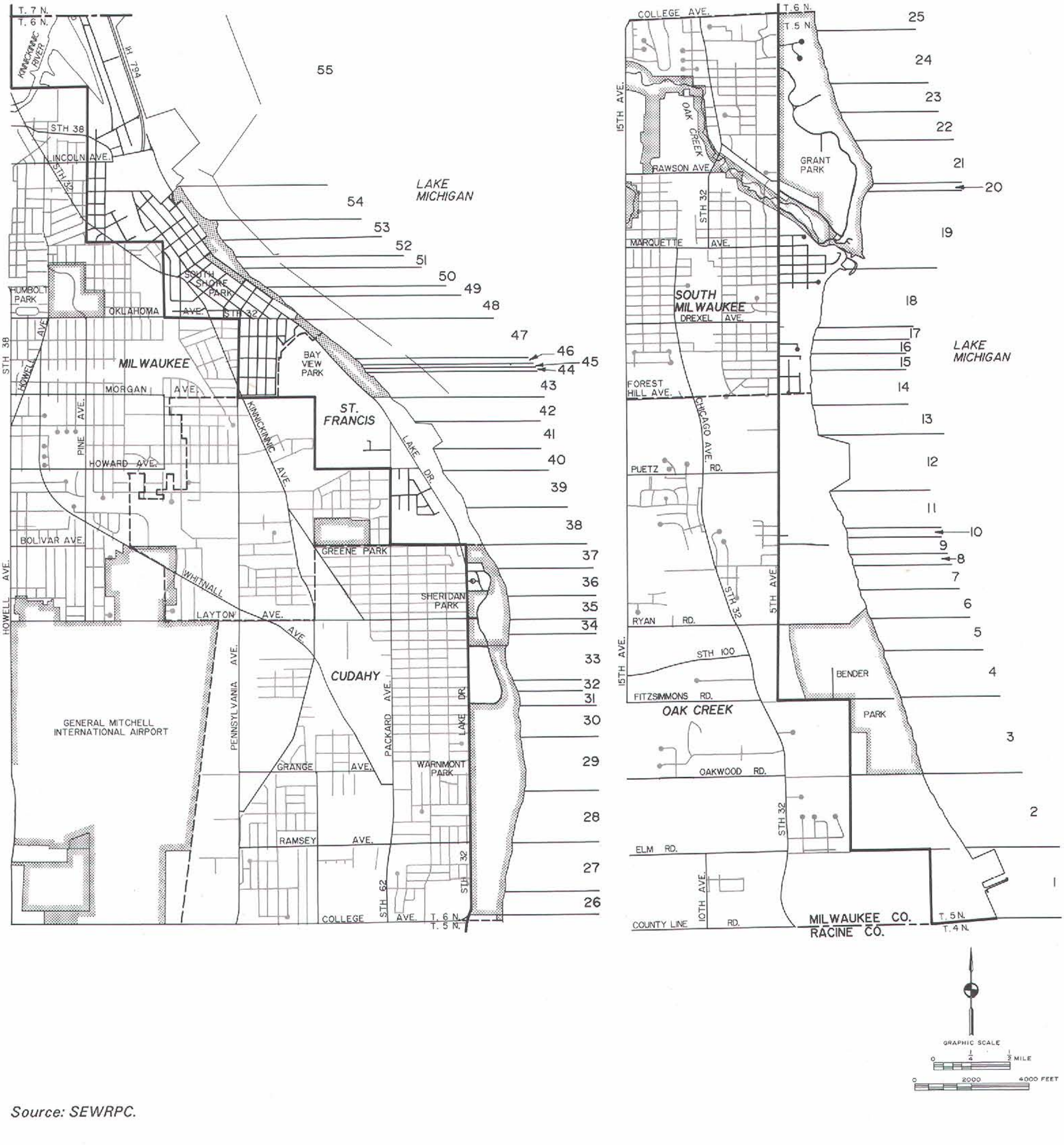
During the October 1987 field surveys, the City of St. Francis shoreline was divided into 10 bluff analysis sections based on similar physical and erosion-related characteristics. Groundwater seepage was observed in three of the 10 bluff analysis sections within the City, covering 3,000 feet, or 31 percent of the shoreline. Seven bluff analysis sections covering 5,800 feet, or 60 percent of the City of St. Francis shoreline, were observed to have toe erosion. Onshore shoreline protection structures were in place within three sections covering 3,400 feet, or 35 percent of the shoreline. The South Shore breakwater also provided some protection against wave action along 45 percent of the shoreline. Within the southernmost section of the shoreline, a concrete rubble and soil landfill was being placed on the bluff at the time of the field surveys to protect the shoreline. Bluff slope failure observed within the City of St. Francis was primarily caused by

Map 24

BLUFF ANALYSIS SECTIONS IN THE MILWAUKEE COUNTY SHORELINE STUDY AREA



Map 24 (continued)



Source: SEWRPC.

Table 26

PHYSICAL AND EROSION-RELATED CHARACTERISTICS OF BLUFF ANALYSIS SECTIONS: 1987

			Beach Characteristics				Bluff Characteristics							
Civil Division	Bluff Analysis Section	Location	Shoreline Length (feet)	Width (feet)	Slope (degree)	Composition	Height (feet)	Overall Slope (degree)	Vegetation (percent)	Composition	Groundwater Conditions	Shore Protection Structure	Significant Bluff Toe Erosion	Type of Bluff Slope Failure
City of Oak Creek	1	WEPCo Oak Creek Plant	4,470	0 - >90	0 - 3	Sand and gravel	60 - 96	25	100	Undetermined	No major groundwater seeps were noted	Steel sheet pile bulkhead protects southern 3,860 feet of section; beach developed within northern 610 feet of section from groin-like action of the power plant bulkhead	No	- -
	2	Elm Road—Oakwood Road	2,820	30 - >90	0 - 9	Sand and gravel	98 - 118	23	100	Oak Creek till at top bluff, underlain by clay and silt and Oak Creek till	Groundwater seeps occur at the top of the lower Oak Creek till layer	Beach formed by groin-like action of the power plant bulkhead	No	Shallow slides, sapping, solifluction, and gully erosion
	3	Bender Park	2,930	10 - 50	7 - 9	Gravel	100 - 114	45	0	Oak Creek till at top of bluff, underlain by lake sediment and Oak Creek till	Groundwater seeps occur at the top of the lower Oak Creek till layer	None	Yes	Slumping, shallow slides and sapping
	4	Bender Park	1,980	10 - 30	10 - 12	Cobble	82 - 104	40	0	Oak Creek till at top of bluff, underlain by lake sediment and Oak Creek till	Groundwater seeps occur at top of the lower Oak Creek till layer. Water accumulates between slump blocks on upper slope	None	Yes	Slumping and shallow slides
	5	Bender Park	1,070	10 - 30	10 - 12	Cobble	76 - 86	50	0	Oak Creek till at top of bluff, underlain by silt and Oak Creek till	Minor groundwater seepage. Most of bluff drained by ravine located behind bluff edge	None	Yes	Small slumps and shallow slides
	6	9300 S. 5th Avenue (Boerke Trust Company property)	1,170	40	10 - 12	Cobble	70 - 76	45	0	Oak Creek till at top of bluff, underlain by sand, sand and silt, and Oak Creek till	Minor groundwater seeps occur in the sand pods within the till or at the base of the sand layer at the north end	None	Yes	Shallow slides
	7	9180 S. 5th Avenue (former Allis Chalmers property)	1,000	0	- -	- -	76 - 90	40	20	Oak Creek till at top of bluff, underlain by sand, sand and silt, and Oak Creek till	Minor groundwater seeps occur in fine sandy silt layer	Concrete rubble fill	No	Sliding fill material and sapping in gullies
	8	9170 S. 5th Avenue (Oak Creek Water Intake Plant)	540	0	- -	- -	90	20	100	Undetermined	No major groundwater seeps were noted	Regraded bluff slope and steel sheet pile bulkhead with armor stone scour protection	No	- -
	9	4301 E. Depot Road (Hynite Corporation and Vulcan Material Company properties)	570	10 - 30	7 - 9	Gravel	82 - 84	45	20	Oak Creek till	No major groundwater seeps were noted	Granite rock breakwater	Yes	Shallow slides and solifluction
	10	9006 S. 5th Avenue (Peter Cooper plant property)	400	10 - 30	7 - 9	Gravel	82 - 84	35	90	Oak Creek till	No major groundwater seeps were noted	Regraded bluff slope and granite rock breakwater	Yes	Some creep and and solifluction
	11	9006-8740 S. 5th Avenue (Peter Cooper plant property)	1,290	30 - 50	8 - 14	Gravel	72 - 80	35	60	Oak Creek till	Minor groundwater seeps near base of bluff	None	Yes	Shallow slides
	12	South Shore Sewage Treatment Plant	3,160	0	- -	- -	76 - 90	18	100	Undetermined	No major groundwater seeps were noted	Steel sheet pile bulkhead with armor stone scour protection	No	- -
	13	8400 S. 5th Avenue	1,320	>90	0 - 3	Sand	80 - 90	20	100	Undetermined	No major groundwater seeps were noted	Beach formed by groin-like action of the treatment plant's bulkhead	No	- -
City of South Milwaukee	14	3817-3509 3rd Avenue	1,310	50 - 90	0 - 6	Sand	74 - 96	38	0	Sand at top of bluff, underlain by silt, Oak Creek till, silt and sand, and Oak Creek till	Minor groundwater seeps occur in till units	None	Yes	Shallow slides
	15	235 Lakeview Avenue—3303 Marina Road	790	0 - 70	4 - 6	Sand	72 - 74	35	50	Silt and sand at top of bluff, underlain by sand, Oak Creek till, silt, Oak Creek till, and silt and sand	No major groundwater seeps were noted	Southern 150 feet protected by concrete waste dumped over bluff top; northern 350 feet is protected by boat harbor and launch	Yes	Shallow slides
	16	3303 Marina Road—3333 5th Avenue	470	50 - 70	7 - 9	Sand and gravel	56 - 64	30	100	Clay and silt at top of bluff, underlain by Oak Creek till and silt and sand	No major groundwater seeps were noted	Beach formed by goin-like action of the boat launch structure	No	Some creep
	17	3333 5th Avenue	440	30 - 50	7 - 9	Sand and gravel	56 - 58	70	0	Clay and silt at top of bluff, underlain by Oak Creek till and silt and sand	No major groundwater seeps were noted. Bluff is drained by ravine located behind bluff edge	None	Yes	Some creep

Table 26 (continued)

Civil Division	Bluff Analysis Section	Location	Beach Characteristics: Shoreline Length (feet)	Beach Characteristics: Width (feet)	Beach Characteristics: Slope (degree)	Beach Characteristics: Composition	Bluff Characteristics: Height (feet)	Bluff Characteristics: Overall Slope (degree)	Bluff Characteristics: Vegetation (percent)	Bluff Characteristics: Composition	Bluff Characteristics: Groundwater Conditions	Shore Protection Structure	Significant Bluff Toe Erosion	Type of Bluff Slope Failure
City of South Milwaukee (continued)	18	South Milwaukee Sewage Treatment Plant—Marshall Avenue	1,880	0 - >90	4 - 9	Sand and gravel	58 - 76	45	80	Silt and sand at top of bluff, underlain by Oak Creek till, and silt and sand	Groundwater seeps occur in the northern half of the section low in the bluff, and at the base of the laminated sand	Concrete rubble fill	Yes	Sliding of fill material
	19	South Milwaukee Yacht Club—Grant Park Beach	3,180	50 - >90	7 - 9	Sand and gravel	56 - 88	25	100	Undetermined	No major groundwater seeps were noted	Granite rock groin with accumulated beach	No	- -
	20	Grant Park	1,280	30 - 50	4 - 6	Sand and gravel	88 - 100	40	100 (top) 0 (bottom)	Oak Creek till at top of bluff, underlain by silt and sand, Oak Creek till, New Berlin till, and sand	Minor groundwater seeps occur at the top of the Oak Creek till layer	Interlocking concrete block groin	Yes	Shallow slides
	21	Grant Park	1,060	50 - 70	4 - 6	Sand	84 - 94	30	100	Oak Creek till at top of bluff, underlain by silt and sand, Oak Creek till, New Berlin till, and sand and gravel	Minor groundwater seeps occur at the top of the Oak Creek till layer	Interlocking concrete block groin	Yes	Shallow slides
	22	Grant Park	950	50 - 70	4 - 6	Sand	66 - 84	40	100 (top) 0 (bottom)	Sand at top of bluff, underlain by silt and sand, clay and silt, and Oak Creek till	Groundwater seeps occur at the top of the Oak Creek till layer. Minor groundwater seeps occur at the top	None	Yes	Shallow slides
	23	Grant Park	1,200	50 - 70	4 - 6	Sand	52 - 78	35	0	Sand at top of bluff, underlain by silt and sand, clay and silt, and Oak Creek till	Groundwater seeps occur at the top of the silt layer	None	Yes	Slumps, shallow slides
	24	Grant Park	1,910	30 - 70	6 - 8	Sand	52 - 64	40	20	Sand at top of bluff, underlain by alternating layers of silt and sand, and silt and clay	Minor groundwater seeps occur at the top of the Oak Creek till layer within the northern one-third of section, drained by gully behind	Precast concrete groin at southern boundary of section	Yes	Shallow slides
	25	Grant Park	880	50 - 70	7 - 9	Sand	90 - 94	35	50	Oak Creek till at top of bluff, underlain by sand and gravel, and Oak Creek till	Minor groundwater seeps occur at top of the lower Oak Creek till layer	None	Yes	Shallow slides and solifluction
City of Cudahy	26	Lake Shore Tower Apartments	660	50 - 70	7 - 9	Sand	94 - 104	40	40	Oak Creek till at top of bluff, underlain by sand and gravel and Oak Creek till	Major groundwater seeps occur at the top of the lower Oak Creek till layer	None	Yes	Sapping, shallow slides and flows
	27	Warnimont Park	1,850	30 - 50	7 - 9	Sand	100 - 112	30	50	Oak Creek till at top of bluff, underlain by sand, silt and clay, Oak Creek till,and sand	Major groundwater seeps occur at the top of the lower Oak Creek till layer	None	Yes	Sapping
	28	Warnimont Park	2,050	30 - 50	4 - 6	Sand and gravel	94 - 102	40	20	Oak Creek till at top of bluff, underlain by sand, sand and gravel, silt, and Oak Creek till	Groundwater seeps occur within ravines. Minor seeps occur between ravines	None	Yes	Sapping, shallow slides and flows
	29	Warnimont Park	770	10 - 50	4 - C	Sand and gravel	102 - 104	40	0	Oak Creek till at top of bluff, underlain by sand, silt, Oak Creek till, gravel, and Tiskilwa till	Minor groundwater seeps occur at the top of the silt layer	None	Yes	Shallow slides and flows
	30	Warnimont Park	1,760	10 - 50	4 - 6	Sand and gravel	100 - 104	45	0	Oak Creek till at top of bluff, underlain by sand, silt, Oak Creek till, Tiskilwa till, and sand	Groundwater seeps occur at the top of the silt layer	None	Yes	Shallow slides and small slumps
	31	Warnimont Park	600	30 - 50	7 - 9	Sand and gravel	104	38	50	Fill material underlain by Oak Creek till, sand and gravel, Oak Creek till, and Tiskilwa till	Groundwater seeps occur at the base of the sand and gravel layer	Two interlocking concrete block groins	Yes	Shallow slides and solifluction
	32	Cudahy Water Intake Plant	340	0	- -	- -	100 - 106	22	100	Undetermined	No major groundwater seeps were noted	Regraded bluff slope, poured concrete bulkhead with granite rock scour protection	No	- -
	33	Warnimont Park	2,060	50 - 70	7 - 9	Sand and gravel	96 - 108	50	0	Oak Creek till at top of bluff, underlain by sand and silt	Groundwater seeps occur at the top of the Tiskilwa till layer	None	Yes	Shallow slides
	34	Sheridan Park	1,780	50 - 70	7 - 9	Sand and gravel	88 - 104	30	100	Oak Creek till at top of bluff, underlain by sand and gravel, silt, and Tiskilwa till	No major groundwater seeps were noted	Concrete block groin field	No	Creep, shallow solifluction, and small slumps

Table 26 (continued)

Civil Division	Bluff Analysis Section	Location	Beach Characteristics: Shoreline Length (feet)	Beach Characteristics: Width (feet)	Beach Characteristics: Slope (degree)	Beach Characteristics: Composition	Bluff Characteristics: Height (feet)	Bluff Characteristics: Overall Slope (degree)	Bluff Characteristics: Vegetation (percent)	Bluff Characteristics: Composition	Bluff Characteristics: Groundwater Conditions	Shore Protection Structure	Significant Bluff Toe Erosion	Type of Bluff Slope Failure
City of Cudahy (continued)	35	Sheridan Park	650	50 - 70	7 - 9	Sand and gravel	90 - 104	32	80	Oak Creek till at top of bluff, underlain by silt and sand	Groundwater seeps occur in the lower portion of the bluff	Concrete block groin field	No	Shallow slides and creep
	36	Sheridan Park	710	50 - 70	7 - 9	Sand and gravel	80 - 90	30	80	Oak Creek till at top of bluff, underlain by silt and clay, Oak Creek till, and silt and sand	Minor groundwater seeps within northern portion of the section on the Oak Creek and New Berlin till layers	Beach formed by groin system	No	Evidence of old slumps
	37	Sheridan Park	1,010	30 - 50	7 - 9	Sand and gravel	72 - 80	48	0	Oak Creek till at top of bluff, underlain by silt, Oak Creek till, New Berlin till, and sand and gravel	Groundwater seeps occur at the top of the New Berlin and upper Oak Creek till layers	None	Yes	Shallow slides and small slumps
City of St. Francis	38	Lunham Avenue—Denton Avenue	1,290	0	- -	- -	54 - 72	40	0	Oak Creek till at top of bluff, underlain by silt, Oak Creek till, New Berlin till, sand and gravel, and Tiskilwa till	No major groundwater seeps were noted	Concrete rubble fill in progress	Yes	- -
	39	Denton Avenue—100 feet south of Howard Avenue	1,480	10 - 30	10 - 12	Cobbles	46 - 54	35	0	Ozaukee till at top of bluff, underlain by silt, Oak Creek till, silty sand, Oak Creek till, and New Berlin till	Groundwater seeps occur at the top of the New Berlin till and upper Oak Creek till layers	None	Yes	Shallow slides and small slumps
	40	100 feet south of Howard Avenue—Lakeside Power Plant	820	0	- -	- -	46 - 48	22	100	Undetermined	No major groundwater seeps were noted	Regraded bluff slope. Dolomite block revetment	Yes	Small slumps
	41	Lakeside Power Plant	1,650	0	- -	- -	30 - 58	22	100	Undetermined	No major groundwater seeps were noted	Breakwater	No	- -
	42	Lakeside Power Plant, Packard Avenue	940	0	- -	- -	56 - 58	30	100	Ozaukee till at top of bluff, underlain by Oak Creek till and clay and silt	No major groundwater seeps were noted	Regraded bluff slope. Concrete block riprap revetment	No	Minor creep and sliding at top of bluff
	43	Bay View Park	1,370	30 - 50	7 - 9	Sand and gravel	42 - 56	45	20	Ozaukee till at top of bluff, underlain by Oak Creek till and silty fine sand	Minor groundwater seeps occur in the silty fine sand layer	South Shore breakwater	Yes	Small slumps and slides
	44	Bay View Park	140	30 - 50	4 - 6	Sand and gravel	40	30	20	Oak Creek till at top of bluff, underlain by silty fine sand	Minor groundwater seeps occur in the silty fine sand layer	South Shore breakwater	Yes	Shallow slides
	45	Bay View Park	80	30 - 50	4 - 6	Sand and gravel	38 - 40	35	0	Oak Creek till at top of bluff, underlain by silty fine sand	No major groundwater seeps were noted	Fill composed mostly of till material. South Shore breakwater	Yes	Shallow slides
	46	Bay View Park	360	50 - 70	4 - 6	Sand and gravel	38 - 40	30	0 - 80	Oak Creek till at top of bluff, underlain by silt and sand	No major groundwater seeps were noted	South Shore breakwater	Yes	Shallow slides
	47	Bay View Park—	2,470	50 - 70	7 - 9	Sand and gravel	40 - 52	18	100	Oak Creek till at top of bluff, underlain by silt and sand	No major groundwater seeps were noted	South Shore breakwater. Relatively wide beach	No	Minor creep
City of Milwaukee	48	Bay View Park—South Shore Park	1,420	0	- -	- -	46-50	30	100	Coarse sand at top of bluff, underlain by Oak Creek till and sand	No major groundwater seeps were noted	South Shore breakwater. Concrete cylinder groins, riprap revetment	No	Shallow slides and creep
	49	Texas Avenue Water Intake Plant	340	0	- -	- -	48 - 50	40	100	Undetermined	No major groundwater seeps were noted	South Shore breakwater. Limestone riprap revetment	No	- -
	50	South Shore Park	1,130	0	- -	- -	26 - 40	40	100	Undetermined	No major groundwater seeps were noted	South shore breakwater. Riprap revetment	No	Minor creep
	51	South Shore Park Pavilion	570	0	- -	- -	18 - 20	18	100	Undetermined	No major groundwater seeps were noted	South shore breakwater. Riprap revetment	Yes	- -
	52	South Shore Beach	450	>90	7 - 9	Sand	22 - 28	15	100	Undetermined	No major groundwater seeps were noted	Regraded bluff slope. South Shore breakwater. Relatively wide beach	No	- -
	53	South Shore Yacht Club	1,320	0	- -	- -	16 - 24	15	100	Undetermined	No major groundwater seeps were noted	Steel sheet pile bulkhead	No	- -
	54	South Shore Park	1,360	0	- -	- -	20 - 22	35	100	Undetermined	No major groundwater seeps were noted	South Shore breakwater. Riprap revetment	Yes	- -
	55	E. Russell Avenue—Jones Island Sewage Treatment Plant	14,750	0	- -	- -	- -	- -	- -	- -	- -	Steel sheet pile bulkheads and riprap revetments	No	- -
	56	Marcus Amphitheater—McKinley Marina	16,060	0	- -	- -	- -	- -	- -	- -	- -	Steel sheet pile bulkheads and riprap revetments	No	- -

Table 26 (continued)

Civil Division	Bluff Analysis Section	Location	Beach Characteristics: Shoreline Length (feet)	Width (feet)	Slope (degree)	Composition	Bluff Characteristics: Height (feet)	Overall Slope (degree)	Vegetation (percent)	Composition	Groundwater Conditions	Shore Protection Structure	Significant Bluff Toe Erosion	Type of Bluff Slope Failure
City of Milwaukee (continued)	57	McKinley Beach—North Point	3,210	0	- -	- -	- -	- -	- -	- -	- -	Headland/beach system and revetment	No	- -
	58	Bradford Beach	1,900	> 90	0 - 3	Sand	- -	- -	- -	- -	- -	None	No	- -
	59	Lake Park	3,540	0	- -	- -	- -	- -	- -	- -	- -	Revetment	Yes	- -
	60	Linnwood Avenue Water Treatment Plant	2,210	0	- -	- -	- -	- -	- -	- -	- -	Steel sheet pile bulkhead	No	- -
	61	UW-Alumni Center—3052 Newport Court	1,970	0 - 80	7 - 12	Sand and gravel	75 - 90	18	90	Undetermined; vegetated	No major groundwater seeps were noted	Three concrete bulkheads and one riprap revement	No	- -
	62	3378-3474 N. Lake Drive	950	60	10 - 12	Sand and gravel	90 - 100	25	85	Ozaukee till at top of bluff, underlain by Oak Creek till and New Berlin till	No major groundwater seeps were noted	Three concrete bulkheads	Yes	- -
Village of Shorewood	63	3510 N. Lake Drive	300	0 - 40	4 - 6	Sand and gravel	95 - 100	28	75	North end Ozaukee till at top of bluff, underlain by sand and gravel, sand, Oak Creek till, and New Berlin till. The rest is undetermined vegetated	Groundwater seeps occur at the lower two-thirds of the bluff	Concrete bulkhead	Yes	Shallow slides and deep-seated slumps
	64	3534 N. Lake Drive	290	< 10	- -	- -	95 - 100	24	20	Ozaukee till at top of bluff, underlain by sand, Oak Creek till, and New Berlin till	No major groundwater seeps were noted	Fill with revetment and breakwater	No	- -
	65	3550-3914 N. Lake Drive	1,710	0 - 60	10 - 12	Sand and gravel	100 - 110	20	100	Ozaukee till at top of bluff, underlain by sand and gravel, Oak Creek till, and New Berlin till	Minor groundwater seeps occur at the base of gravel on till surface	Three concrete bulkheads	Yes	- -
	66	3926 N. Lake Drive	170	< 10	- -	- -	110	20	100	Ozaukee till at top of bluff, underlain by sand and gravel, Oak Creek till, and New Berlin till	No major groundwater seeps were noted	Fill with concrete block revetment	Yes	- -
	67	3932-3966 N. Lake Drive	380	< 10	- -	- -	110	32	0	Ozaukee till at top of bluff, underlain by sand and gravel, sand, Oak Creek till, and New Berlin till	No major groundwater seeps were noted	Grout-filled bags	Yes	Sloughing and shallow slides
	68	Atwater Park—4216 N. Lake Drive	2,170	0 - 130	7 - 12	Sand, gravel, and cobbles	90 - 105	30	100	At southern end Ozaukee till at top of bluff, underlain by sand, Oak Creek till, and New Berlin till. At northern end is Nipissing terrace, with sand and gravel at top underlain by New Berlin till. The rest is undetermined vegetated	No major groundwater seeps were noted	Groin system at Atwater Park	No	- -
	69	4226-4320 N. Lake Drive	520	< 10	- -	- -	115	28	- -	Undetermined; vegetated	No major groundwater seeps were noted	None	Yes	- -
	70	4400-4408 N. Lake Drive	240	< 10	- -	- -	110-115	24	80	Ozaukee till at top of bluff, underlain by silt and sand, Oak Creek till, and New Berlin till	No major groundwater seeps were noted	Concrete bulkhead	Yes	Shallow slides and slumping
Village of Whitefish Bay	71	4424-4652 N. Lake Drive	2,370	< 10	- -	- -	95-115	30	0	Ozaukee till at top of bluff, underlain by silt and sand, Oak Creek till, New Berlin till, and silt and sand layers	Some groundwater seeps occur in the silt and sand layer	Fill with riprap revetment	Yes	- -
	72	4668-4730 N. Lake Drive	850	< 10	- -	- -	95	38	0	Ozaukee till at top of bluff, underlain by silt and sand, Oak Creek till, and New Berlin till	No major groundwater seeps were noted	Two concrete bulkheads	Yes	Surface sloughing, slumping, and shallow slides
	73	4744-4762 N. Lake Drive	190	< 10	- -	- -	95	33	0	Ozaukee till at top of bluff, underlain by silt and sand, Oak Creek till, and New Berlin till	No major groundwater seeps were noted	Fill with riprap revetment	Yes	- -
	74	4780 N. Lake Drive	160	< 10	- -	- -	95	36	0	Ozaukee till at top of bluff, underlain by silt and sand, Oak Creek till, and New Berlin till	No major groundwater seeps were noted	None	Yes	Surface sloughing, slumping and shallow slides
	75	4790-4800 N. Lake Drive	310	< 10	- -	- -	80-90	36	0	Ozaukee till at top of bluff, underlain by silt and sand, Oak Creek till, and New Berlin till	No major groundwater seeps were noted	Fill with riprap revetment	Yes	- -

Table 26 (continued)

Civil Division	Bluff Analysis Section	Location	Beach Characteristics: Shoreline Length (feet)	Beach Characteristics: Width (feet)	Beach Characteristics: Slope (degree)	Beach Characteristics: Composition	Bluff Characteristics: Height (feet)	Bluff Characteristics: Overall Slope (degree)	Bluff Characteristics: Vegetation (percent)	Bluff Characteristics: Composition	Bluff Characteristics: Groundwater Conditions	Shore Protection Structure	Significant Bluff Toe Erosion	Type of Bluff Slope Failure
Village of Whitefish Bay (continued)	76	4810-4840 N. Lake Drive	360	< 10	- -	- -	80	40	0	Ozaukee till at top of bluff, underlain by silt and sand, Oak Creek till, and New Berlin till	No major groundwater seeps were noted	None	Yes	Shallow slumps and slides
	77	4850-4940 N. Lake Drive	810	0 - 15	15 - 20	Sand and gravel	65 - 80	35	90	At southern end of section, Ozaukee till at top of bluff, underlain by silt and sand, Oak Creek till, and New Berlin till. The remainder of the bluff is undetermined vegetated	No major groundwater seeps were noted	Fill with riprap revetment	Yes	- -
	78	Buckley Park—southern portion Big Bay Park	1,660	0 - 25	15 - 20	Sand and gravel	65 - 80	24	90	Ozaukee till at top of bluff, underlain by layers of sand and silt, clay and silt, sand and silt, Oak Creek till, clay and silt, Oak Creek till, and New Berlin till	No major groundwater seeps were noted	Concrete stepped bulkhead	Yes	Slumping
	79	Northern portion Big Bay Park to 5270 N. Lake Drive	1,480	< 10	- -	- -	65 - 75	30	30 - 90	At southern end, Ozaukee till at top of bluff, underlain by silt and clay, Oak Creek till, and New Berlin till. At northern end, sand lies above silt and clay and silt lies above Oak Creek till	No major groundwater seeps were noted	Fill with riprap revetment	Yes	- -
	80	5290 N. Lake Drive	130	25	4 - 6	Sand	70	29	80	Ozaukee till at top of bluff, underlain by sand, silt and clay, laminated silt, Oak Creek till, and New Berlin till	No major groundwater seeps were noted	None	No	Shallow slides within top portion of bluff
	81	5300 N. Lake Drive—808 Lakeview Avenue	2,970	< 10	- -	- -	85	35	0 - 95	Ozaukee till at top of bluff, underlain by silt and sand, Oak Creek till, and New Berlin till	No major groundwater seeps were noted	Fill with riprap revetment	Yes	- -
	82	5722-5770 N. Lake Drive	490	0 - 50	7 - 9	Sand and gravel	80 - 85	30	90	Ozaukee till at top of bluff, underlain by silt and sand and Oak Creek till	Some groundwater seepage in the Ozaukee till layer	None	Yes	Slumping and shallow slides
	83	758 E. Day Avenue	140	35	7 - 9	Sand and gravel	85	35	90	Ozaukee till at top of bluff, underlain by silt and sand and Oak Creek till	No major groundwater seeps were noted	Fill	Yes	- -
	84	740 E. Day Avenue—5866 N. Shore Drive	430	25 - 35	7 - 9	Sand and gravel	80 - 85	27	90	Ozaukee till at top of bluff, underlain by silt and sand and Oak Creek till	Many groundwater seeps were noted	Concrete bulkhead northern 100 feet	Yes	Slumping along silt and sand layer
	85	Klode Park	480	20 - 35	7 - 9	Sand and gravel	75 - 80	22	90	Ozaukee till at top of bluff, underlain by silt and sand and Oak Creek till	Many groundwater seeps were noted in lower portion of the bluff	Regraded bluff slope and construction of breakwaters in progress in 1988	- -	- -
	86	5960 N. Shore Drive	170	35 - 40	7 - 9	Sand and gravel	80 - 90	32	80	Ozaukee till at top of bluff, underlain by silt and sand and Oak Creek till	Some groundwater seepage in the silt and sand layer	None	Yes	Slumping
	87	6000 N. Shore Drive—6260 N. Lake Drive	1,950	35 - 50	7 - 9	Sand and gravel	90 - 120	26	95	Ozaukee till at top of bluff, underlain by silt and sand, a layer of silt and sand, and Oak Creek till	Groundwater seeps are common from mid-height on bluff to base	None	No	Small slips
	88	6310-6424 N. Lake Drive	1,150	25 - 40	4 - 6	Sand and gravel	115 - 125	30	50	Ozaukee till at top of bluff, underlain by layers of silt and sand and Oak Creek till	Some groundwater seepage occurs in the silt and sand layer and the Oak Creek till	None	Yes	Surface sliding and slumping
Village of Fox Point	89	6430-6448 N. Lake Drive	320	< 10	- -	- -	120 - 125	25	0	Ozaukee till at top of bluff, underlain by silt and sand	No major groundwater seeps were noted	Fill with riprap revetment	No	- -
	90	6464-6530 N. Lake Drive	470	< 10	- -	- -	115 - 125	30	40	Ozaukee till at top of bluff, underlain by silt and sand	Groundwater seepage noted at the top of the silt layer	Riprap revement	Yes	Shallow slides and slumps
	91	6600-6702 N. Lake Drive	510	0 - 50	10 - 12	Sand and gravel	120 - 125	35	95	Ozaukee till at top of bluff, underlain by silt and sand	No major groundwater seeps were noted	Concrete groin field	Yes	Shallow slides and small slumps
	92	6720 N. Lake Drive—6818 N. Barnett Lane	770	0 - 15	10 - 12	Sand and gravel	115 - 125	38	60	Ozaukee till at top of bluff, underlain by silt and sand. At north end of section, sand and silt underlain by Oak Creek till and New Berlin till, and below New Berlin till lies bedrock	No major groundwater seeps were noted	Grout-filled bag revetment	Yes	Rapid surface sliding and small slumps

Table 26 (continued)

			Beach Characteristics				Bluff Characteristics							
Civil Division	Bluff Analysis Section	Location	Shoreline Length (feet)	Width (feet)	Slope (degree)	Composition	Height (feet)	Overall Slope (degree)	Vegetation (percent)	Composition	Groundwater Conditions	Shore Protection Structure	Significant Bluff Toe Erosion	Type of *Bluff Slope* Failure
Village of Fox Point (continued)	93	6820-6840 N. Barnett Lane	530	<10	- -	- -	115 - 120	30	90	Ozaukee till at top of bluff, underlain by silt and sand, Oak Creek till, and New Berlin till	No major groundwater seeps were noted	Fill with riprap revetment in progress in 1988	Yes	Slumping and shallow slides
	94	6868-6990 N. Barnett Lane	1,460	<10	- -	- -	115 - 120	45	70	Ozaukee till at top of bluff, underlain by sand, silt, Oak Creek till, and New Berlin till. At southern end of segment, silt and sand lie between sand and the silt	No major groundwater seeps were noted	Fill with riprap revetment in progress in 1988	Yes	Shallow slides and small slumps
	95	7000-8130 N. Beach Drive	9,070	0 - 64	4 - 9	Sand and gravel	4 - 10	- -	0	Sand at the top of the terrace, underlain by New Berlin till	No major groundwater seeps were noted	Revetments, bulkheads, and groin systems	Yes	- -
	96	Doctors Park	1,890	0 - 90	4 - 9	Sand, gravel, and cobbles	90 - 95	25	100	Undetermined; vegetated	No major groundwater seeps were noted	Bulkhead and concrete block groin field	No	- -
Village of Bayside	97	Audubon Center—9360 N. Lake Drive	4,660	30 - 50	4 - 9	Sand, gravel, and cobbles	80 - 90	20	100	Undetermined	No major groundwater seeps were noted	None	Yes	- -
	98	1470-1434 E. Bay Point Road	860	<10	- -	- -	80 - 90	20	100	Undetermined	No major groundwater seeps were noted	Riprap revement and concrete slab bulkhead	No	- -
	99	1430 E. Bay Point Road—9364 N. Lake Drive	1,280	30 - 50	7 - 9	Sand, gravel, and cobbles	85 - 90	20	100	Ozaukee till at top of bluff, underlain by silt, and clay and silt	No major groundwater seeps were noted	None	Yes	- -
	100	9400-9578 N. Lake Drive	1,320	30 - 50	7 - 9	Sand and gravel	90 - 95	42	10	Ozaukee till at top of bluff, underlain by silt, and clay and silt	Groundwater seepage occurs in the upper portion of the bluff	Concrete bulkhead and concrete block revetment	Yes	Shallow slides and small slumps

Source: SEWRPC.

wave erosion and groundwater seepage. Bluff slope failures generally occurred as shallow slides and small slumps.

City of Milwaukee

Approximately 52,160 feet of Lake Michigan shoreline, or 33 percent of the total county shoreline, lies within the City of Milwaukee. The beach widths measured in the fall of 1987 ranged from 0 to nearly 200 feet, with approximately 88 percent of the shoreline having a beach width of less than 10 feet. A bluff was present at the water's edge along the southern 7,600 feet and northern 2,900 feet of the city shoreline. Bluff heights within the southern portion of the shoreline generally ranged from about 40 to 50 feet, and within the northern portion of the shoreline from 75 to 110 feet. Nearly all of the bluffs within the City of Milwaukee were fully vegetated with a bluff slope of 30 degrees or less. During the field surveys, the City of Milwaukee shoreline was divided into 14 bluff analysis sections based on similar physical and erosion-related characteristics. There was no observed groundwater seepage from the bluffs during the field surveys. Wave erosion was observed within four analysis sections, covering about 6,400 feet, or 12 percent of the City of Milwaukee shoreline. However, the erosion did not affect the overall stability of the bluff slope in any of these areas. Onshore protection structures were in place within portions of 11 sections covering about 47,400 feet, or 91 percent of the shoreline. The South Shore breakwater and the Milwaukee Harbor breakwater provided additional protection against wave action within 74 percent of the City's shoreline. Minor bluff slope failure was observed within the City of Milwaukee, generally occurring as shallow slides and minor creeps.

Village of Shorewood

Approximately 6,590 feet of Lake Michigan shoreline, or 4 percent of the total county shoreline, lies within the Village of Shorewood. The beach widths measured in the summer of 1986 ranged from 0 to more than 100 feet, with approximately 40 percent of the shoreline having a beach width of less than 10 feet. The bluffs along the shoreline ranged in height from about 90 to 120 feet, and generally were composed of Ozaukee till underlain by sand and gravel or silt and sand, Oak Creek till, and New Berlin till. Approximately 60 percent of the shoreline had a fully vegetated bluff face, and an overall slope of less than 30 degrees.

During the May 1986 field surveys, the Village of Shorewood shoreline was divided into nine bluff analysis sections based on similar physical and erosion-related characteristics. Groundwater seepage was observed in three of the nine analysis sections within the Village, covering about 2,820 feet, or 43 percent of the shoreline. Seven bluff analysis sections containing about 3,600 feet, or 55 percent of the Village of Shorewood shoreline, were observed to have toe erosion. Shoreline protection structures were in place within portions of eight sections covering 4,000 feet, or 60 percent of the shoreline. Bluff slope failure observed within the Village of Shorewood was primarily caused by wave erosion and groundwater seepage. Bluff slope failures generally occurred as shallow slides and deep-seated slumps.

Village of Whitefish Bay

The Village of Whitefish Bay contains approximately 14,680 feet of Lake Michigan shoreline, or 9 percent of the total county shoreline. The beach widths measured in the summer of 1986 ranged from 0 to 50 feet, with approximately 70 percent of the shoreline having a beach width of less than 10 feet. The bluffs along the shoreline ranged in height from 65 to 125 feet, and generally were composed of Ozaukee till underlain by silt and sand or silt and clay, and by Oak Creek till. Within the southern portion of the Village, a layer of New Berlin till was exposed beneath the Oak Creek till layer. Approximately 6,400 feet, or 44 percent of the shoreline, was covered by fill material at the time of the field surveys in 1986. By 1988 an additional 2,600 feet of shoreline was in the process of being filled.

The Village of Whitefish Bay shoreline was divided into 18 bluff analysis sections based on similar physical and erosion-related characteristics. Groundwater seepage was observed in seven of the 18 analysis sections within the Village covering about 5,600 feet, or 38 percent of the shoreline. Portions of 16 bluff analysis sections containing 12,600 feet, or 86 percent of the shoreline of the Village of Whitefish Bay, were observed to have toe erosion. Shoreline protection structures were in place within portions of at least 11 sections covering 10,000 feet, or 70 percent of the shoreline. Bluff slope failure observed within the Village of Whitefish Bay was primarily caused by wave erosion and groundwater seepage. Bluff slope failures generally occurred as surface sloughing, slumping, and shallow slides.

Village of Fox Point

The Village of Fox Point contains approximately 14,580 feet of Lake Michigan shoreline, or 9 percent of the total county shoreline. The beach widths measured in the summer of 1986 ranged from 0 to 65 feet, with approximately 80 percent of the shoreline having a beach width of less than 10 feet. A bluff was present at the water's edge along the southern 4,670 feet and northern 840 feet of the village shoreline, and ranged in height from 90 to 125 feet. The bluff was generally composed of Ozaukee till underlain by silt and sand, Oak Creek till, and New Berlin till. Along the remainder of the village shoreline there was a relatively wide terrace in front of the bluffs, which extended to a maximum width of approximately 900 feet and ranged from 4 to 10 feet in height.

The Village of Fox Point shoreline was divided into nine bluff analysis sections based on similar physical and erosion-related characteristics. Groundwater seepage was observed in two of the nine analysis sections within the Village covering 1,100 feet, or 8 percent of the shoreline. Portions of seven sections containing 7,000 feet, or 48 percent of the shoreline of the Village of Fox Point, were observed to have toe erosion. Shoreline protection structures were in place within portions of at least six of the sections covering 10,200 feet, or 70 percent of the shoreline. Bluff slope failure observed within the Village of Fox Point was primarily caused by wave erosion and groundwater seepage. Bluff slope failures generally occurred as shallow slides and slumps.

Village of Bayside

Approximately 9,170 feet of Lake Michigan shoreline, or 6 percent of the total county shoreline, is located within the Village of Bayside. The beach widths measured in the fall of 1987 ranged from 0 to 70 feet, with approximately 10 percent of the shoreline having a beach width of less than 10 feet. The bluffs along the shoreline ranged in height from 90 to 100 feet, and generally were composed of Ozaukee till underlain by silt, and clay and silt.

During the October 1987 field surveys, the Village of Bayside shoreline was divided into five bluff analysis sections based on similar physical and erosion-related characteristics. Groundwater seepage was observed in only one of the analysis sections, accounting for 1,300 feet, or 14 percent of the village shoreline. Three bluff analysis sections containing 7,300 feet, or 80 percent of the Village of Bayside shoreline, were observed to have toe erosion. Shoreline protection structures were in place within portions of at least three sections covering 2,100 feet, or 23 percent of the shoreline. Bluff slope failure observed within the Village of Bayside was primarily caused by wave action and groundwater seepage. Bluff slope failures generally occurred as shallow slides and small slumps.

SHORELINE RECESSION RATES

The rate of shoreline recession may be estimated by measuring the change in location of a bluff edge—or shoreline where no bluff is present—over a specified time period. Shoreline recession rates for Milwaukee County were measured using Regional Planning Commission ratioed and rectified, one inch equals 400 feet scale aerial photographs taken in 1963 and in 1985; and Commission one inch equals 100 feet scale, two-foot contour interval topographic maps made from 1980 through 1987. All measurements on the aerial photographs and large-scale topographic maps were made parallel to the east-west U. S. Public Land Survey Section line which forms the southern boundary of the study area. The measurements were corrected for minor variations in aerial photograph scale and for the angle of the shoreline in order to represent recession perpendicular to the shoreline.

Shoreline recession was measured at intervals of 200 feet—the interval length being measured perpendicular to the section line—along the entire study area shoreline. These intervals define the boundaries of 638 shoreline recession reaches, which are shown on Map 25. The shoreline length of these reaches ranges from 200 feet to 980 feet, with a combined length of the shoreline recession reaches totaling 159,110 feet.

Appendix C presents the measured shoreline recession rates, as well as the volume of shoreline material lost, for the period 1963 through 1985, for each shoreline recession reach. The recession rates are also shown on Map 25. Shoreline length, bluff height, and the volume of bluff or shoreline material lost for each reach are also presented in Appendix C. The recession rates for the period 1963 through 1985 ranged from less than 0.5 foot per year to 12.5 feet per year. Those areas with a recession rate equal to or more than 0.5 foot per year had a shoreline length-weighted mean recession rate of about 1.9 feet per year. The highest recession rates were measured near Bender Park within the City of Oak Creek. It is important to note that these recession rates are averaged over the period of record. Erosion and recession rates vary widely from year to year.

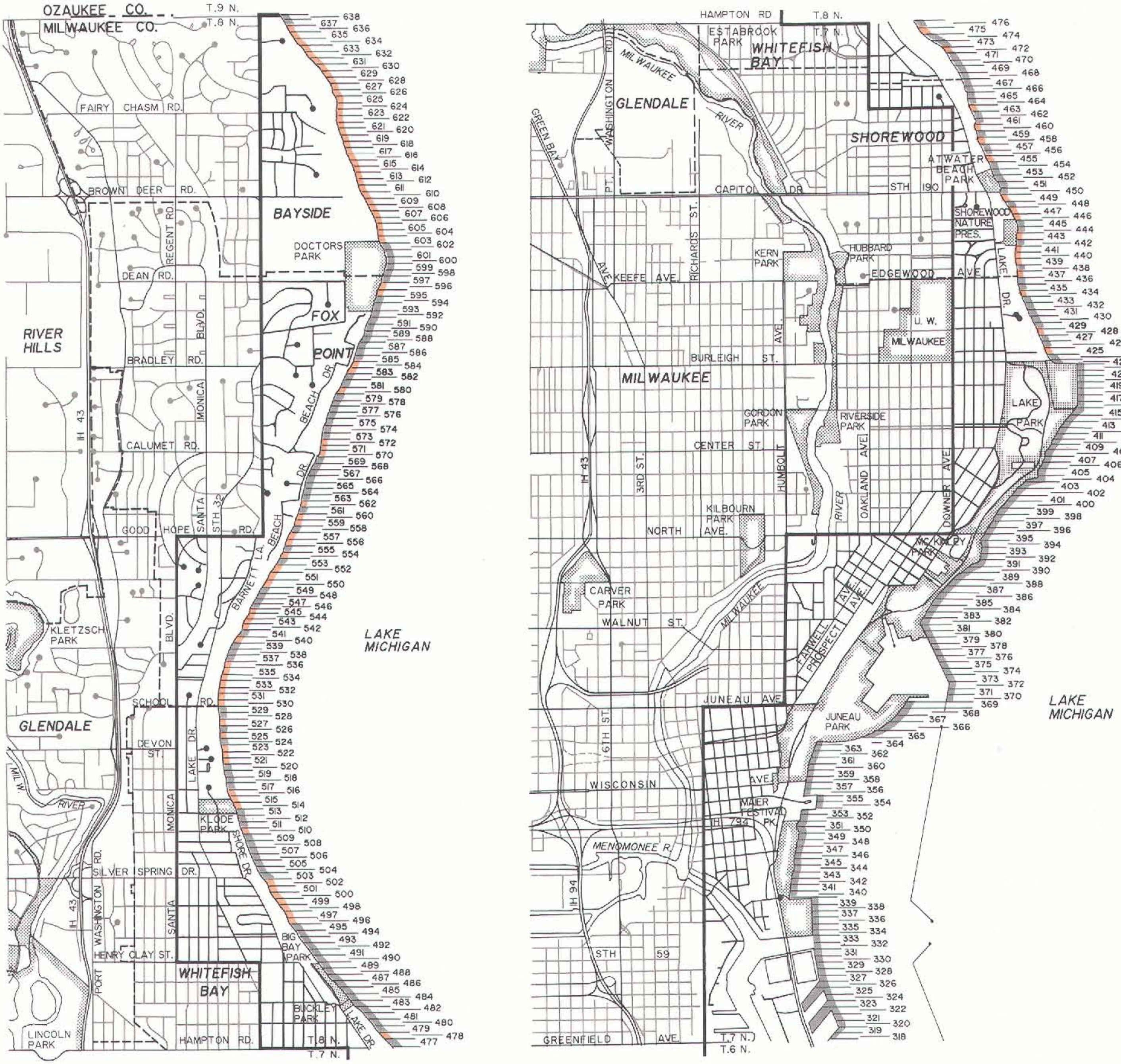
As noted in Chapter III of this report, slope failure and the attendant recession of the bluff is not a steady, uniform process. Slope failure often occurs sporadically, followed by extended periods of relative stability. For example, in the City of Oak Creek, a long-term recession rate of about 12 feet per year was estimated near Bender Park. However, as shown in Chapter I, one portion of the bluff receded a total of about 63 feet in 16 months. In 1986 and 1987, severe slope failures occurred in the Village of Whitefish Bay's Buckley Park and Klode Park in areas where the bluffs had not failed for decades.

A summary of estimated shoreline recession rates and associated shoreline lengths and the volume material loss to erosion is presented in Table 27. About 63 percent of the shoreline had an average annual recession rate of less than 0.5 foot. Shoreline recession, as measured from 1963 through 1985, resulted in the average annual loss of about 115,700 square feet, or about 2.7 acres, of land containing approximately 328,000 cubic yards of shore and bluff material. The 3 percent of the study area shoreline exhibiting a recession rate exceeding 4.0 feet per year accounted for nearly 36 percent of the total shore material loss in the study area.

For comparison purposes, long-term recession rates over the period 1836 through 1985 are also given in Appendix C. These long-term rates are based on the original U. S. Public Land Survey field notes made in 1836, the Commission one inch equals 100 feet scale, two-foot contour interval topographic maps made from 1980 through 1987, and the related control survey network which locates and monuments all U. S. Public Land Survey corners throughout the area map, and places those corners on the State Plane Coordinate System by high-precision field surveys. The long-term recession measurements were calculated at 19 U. S. Public Land Survey

Map 25

BLUFF RECESSION REACHES AND RECESSION RATES: 1963-1985



LEGEND

95 BLUFF RECESSION REACH NUMBER

BLUFF RECESSION REACH BOUNDARY

BLUFF RECESSION RATE (IN FEET PER YEAR)

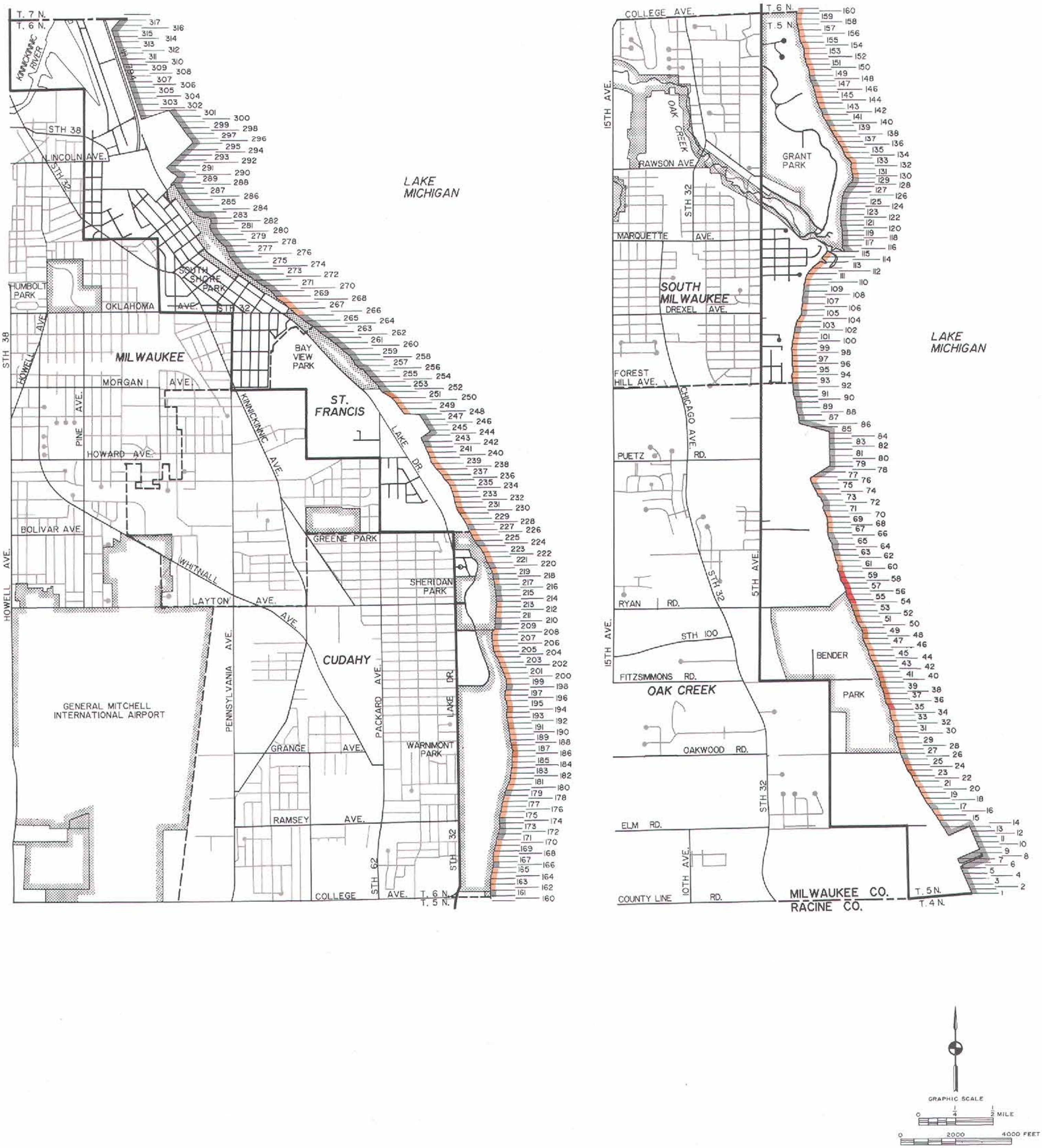
LESS THAN 0.5

0.5 - 4.4

4.5 - 8.4

8.5 OR GREATER

Map 25 (continued)



Source: SEWRPC.

Table 27

SUMMARY OF SHORELINE RECESSION RATES AND SHORE MATERIAL LOSS ALONG THE LAKE MICHIGAN SHORELINE OF MILWAUKEE COUNTY: 1963-1985

Bluff Recession Rate (feet/year)	Length (feet)	Percent of Total	Annual Volume of Shore Material Loss (cubic yards/year)	Percent of Total
<0.5	99,530	62.6	- -	- -
0.5 - 2.0	40,790	25.6	90,500	27.6
2.5 - 4.0	13,240	8.3	119,900	36.6
4.5 - 6.0	3,740	2.4	61,600	18.8
6.5 - 8.0	210	0.1	5,100	1.6
8.5 - 10.0	210	0.1	7,100	2.2
>10	1,390	0.9	43,400	13.2
Total	159,110	100.0	327,600	100.0

Source: SEWRPC.

Section lines which did not lie within areas where extensive filling had occurred since the original survey. The bluff recession over this long period of time can be accurately calculated because the original Public Land Survey corners set in 1836 have been perpetuated. The average long-term recession rate calculated was 1.6 feet per year. Eleven, or 58 percent, of the long-term recession rates were higher than the short-term recession rates; four, or 21 percent, of the long-term rates were lower than the short-term rates; and four, or 21 percent, were the same as the short-term rates. The long-term recession rates were generally higher than the short-term rates in those shoreline areas where shore protection structures had been installed. Three of the four sites where the long-term rates were lower than the the short-term rates were located downdrift of existing structures. Some types of structures have been known to increase erosion of downdrift shoreline areas.

SUMMARY

This chapter presents an inventory of certain elements of the natural resource base relevant to shoreline erosion and bluff recession; summarizes existing land use and zoning patterns; and sets forth the findings of an inventory and analysis of the types, causes, and rates of shoreline erosion and bluff recession occurring within Milwaukee County. This information is necessary for an assessment of the severity of erosion within various reaches of shoreline, and for the selection and evaluation of structural—both onshore and offshore—and nonstructural shoreline erosion management measures. Data on the geology and glacial deposits, soils, bluff and beach characteristics, groundwater resources, and climate of the study area are presented.

The Milwaukee County shoreline is underlain by Precambrian, Cambrian, Ordovician, and Silurian bedrock comprised primarily of dolomite, shale, sandstone, and crystalline rock. The bedrock is covered by unconsolidated glacial deposits which range up to more than 200 feet in thickness. Several layers of glacial debris, including the Kewaunee Formation, the Oak Creek Formation, the New Berlin Formation, and the Zenda Formation, can be identified on the eroding bluff faces along the County's Lake Michigan shoreline.

Soil properties influence the rate of stormwater runoff and the severity of surface erosion. About 21 percent of the study area shoreline is covered by soils which generate large amounts of stormwater runoff because of their low infiltration capacity, low permeability, and poor drainage. These soil properties result in substantial surface runoff being discharged over the top of the bluffs onto the bluff faces. About 22 percent of the study area is covered by well-drained or moderately drained soils which generate relatively small amounts of runoff. About 54 percent of the area is covered by disturbed soils, and the remaining 3 percent of the study area is covered by surface water.

Bluff heights along the shoreline range up to nearly 130 feet above beach levels. About one-half of the shoreline has bluffs greater than 70 feet in height. About 18 percent of the shoreline has bluffs ranging from 20 to 70 feet in height. The Milwaukee Harbor area and the terraced area within the Village of Fox Point, which lies up to 10 feet above the beach, together cover approximately 32 percent of the shoreline within the study area. The most dominant bluff material identified was the Oak Creek till, covering about 31 percent of the total bluff face surface within the study area. Other common bluff materials found were general lake sedimentation, silt and sand, and Ozaukee till. The composition of the bluff slopes along about 14 percent of the shoreline was undetermined because no stratigraphic data were available and the slopes were considered to be stable.

The most common beach materials found were sand, gravel, and cobbles. The most extensive beach, exceeding 300 feet in width, was found at Grant Park in the City of South Milwaukee, and was composed of sand. In 1987 within southern Milwaukee County and the Village of Bayside, about 19 percent of the shoreline had a beach width ranging from 11 through 50 feet; about 12 percent of the shoreline had a beach width ranging from 51 through 90 feet; and about 4 percent of the shoreline had a beach greater than 90 feet wide. About 65 percent of the shoreline contained either no beach—the lake reaches the bluff toe, or in some cases, a shore protection structure—or a beach less than 10 feet in width. In 1986 within northern Milwaukee County, about 20 percent of the shoreline had a beach width ranging from 11 through 50 feet, about 8 percent of the shoreline had a beach width ranging from 51 through 90 feet, and about 3 percent of the shoreline had a beach greater than 90 feet wide. The remaining 69 percent of the shoreline contained either no beach or a beach less than 10 feet in width. Beach slopes generally were less than 10 degrees.

Along the Milwaukee County shoreline, groundwater generally flows toward Lake Michigan. Two major aquifers underlie the coastal area: the deep sandstone aquifer and the Niagara dolomite aquifer. In addition, the sand and gravel glacial deposits that lie above the Niagara bedrock may act as water-bearing units. The presence of groundwater in this glacial bluff material reduces the frictional resistance to stress forces, creates a seepage pressure in the direction of water flow, and adds weight to the bluff.

Climate impacts on coastal erosion include freeze-thaw actions within bluff material, high surface runoff from frozen soils, lake ice effects, and high surface runoff and soil erosion during intense storm events. Frozen ground and snow cover may be expected throughout approximately four months each winter season. Lake ice formation begins in late November or December and ice breakup normally occurs in late March or early April.

The near-shore Lake Michigan area contains an established diverse coldwater fishery, with 33 species of fish being surveyed, while the Milwaukee outer harbor contains a warmwater fishery with 30 species of fish being surveyed. Some coldwater fish species are also present in the outer harbor, although these fish do not naturally reproduce within the harbor. The Lake Michigan fishery populations have been affected by the appearance of the sea lamprey in the 1930's and by the introduction of numerous exotic species.

The presence of toxic contaminants in the tissue of fish residing in Lake Michigan and in the Milwaukee outer harbor has been a widespread problem. Of greatest concern is the presence of polychlorinated biphenyls (PCB's) at levels exceeding U. S. Food and Drug Administration health standards.

Portions of the Lake Michigan littoral environment provide excellent habitat for fish and aquatic life. The Milwaukee outer harbor provides limited habitat areas, being suitable only for warmwater fish species.

The Lake Michigan shoreline contains over 900 acres of important wildlife habitat areas. The study area also contains five designated natural areas.

The study area has become highly urbanized since the mid-1800's. Historic places and traditions are highly valued in the Milwaukee area. Six historic districts and 37 historic sites in the study area were listed on the National Register of Historic Places in 1988. The Milwaukee Harbor was largely developed between the 1880's and the 1930's.

The study area, which lies entirely within Milwaukee County, contains portions of the Cities of Cudahy, Milwaukee, Oak Creek, St. Francis. and South Milwaukee, and the Villages of Bayside, Fox Point, Shorewood, and Whitefish Bay, and encompasses a total of 7,517 acres. About 4,443 acres, or 59 percent of the study area, was devoted to intensive urban uses in 1985. About 44 percent of the urban land area was in residential use. Zoning ordinances are important land use regulations which are in effect in each of the nine civil divisions within the study area. Amendments to existing zoning ordinances may be used to regulate land uses in relation to the risk of shoreline erosion and bluff recession. While local zoning ordinances regulate land uses within the shoreland area, they are generally devoid of provisions pertaining to Lake Michigan shoreline erosion hazards.

Bluff erosion is of particular concern in the study area because it results in property loss and may pose a threat to human safety. Bluff erosion may occur as toe erosion, slumping, sliding, flow, surface erosion, and solifluction. Slope failure is often an unpredictable, abrupt process which is constantly being altered by numerous factors. Factors affecting bluff erosion include the physical characteristics of the bluff and beach, wave action, lake level fluctuations, ice formation, groundwater seepage, surface runoff, and vegetative cover.

Shoreland development and activities are regulated by federal, state, and local units and agencies of government. The U. S. Army Corps of Engineers is the primary federal agency responsible for certain structures, dredging, and wetland protection structures. Although the Wisconsin Department of Natural Resources regulates shore protection-related activities throughout most of the Lake Michigan shoreline of the State, 93 percent of the shoreline within the study area is regulated under Lake Bed Grants issued to either Milwaukee County or the City of Milwaukee.

Inventories of shore protection structures conducted in June and July of 1986 and in November of 1987 indicated that a variety of structures, including bulkheads, revetments, groins, and breakwaters, had been installed along the Milwaukee County shoreline to provide an artificial protective barrier against direct wave and ice damage, to increase the extent of the beach, to dissipate offshore wave energy, and to stabilize bluff slopes. These costly measures, installed by both private shoreline property owners and public agencies, have had varying degrees of success. An inventory of all 128 shore protection structures in the study area indicated that only about 25 percent of the structures had no observable failure and at the time of the survey were not in need of any significant maintenance work. The remaining structures were observed to have some type of failure which included overtopping, where the water level, or waves, exceeded the top of the structure; flanking, where the sides of the structure were eroded; collapsing; and material failure.

A detailed inventory of the physical characteristics and erosion-related characteristics of the actively eroding bluffs was conducted in southern Milwaukee County and the Village of Bayside in October 1987 and in May 1986. The results of the inventory indicated that the primary cause of bluff recession in the study area was bluff toe erosion caused by wave action. Groundwater seepage also was a major cause of slope failure in some portions of the study area. Most slope failure was occurring as shallow slides, although many areas were experiencing deep-seated slumps.

Bluff recession rates for the Milwaukee County study area were measured using the original U. S. Public Land Survey notes and maps, Regional Planning Commission aerial photographs taken in 1963 and 1985, and Commission large-scale topographic maps made from 1980 through 1987. For the period 1963 through 1985, about 63 percent of the study area shoreline exhibited bluff recession rates of less than 0.5 foot per year. About 26 percent of the shoreline exhibited a bluff recession rate ranging from 0.5 foot to 2.0 feet per year, and about 12 percent exhibited a bluff recession rate exceeding 2.0 feet per year. Those areas with a recession rate equal to or more than 0.5 foot per year had a shoreline length-weighted mean of about 1.9 feet per year. The highest recession rate measured from 1963 through 1985 was 12.5 feet per year, which occurred near Bender Park within the City of Oak Creek. Shoreline recession, as measured from 1963 through 1985, resulted in the average annual loss of about 115,700 square feet of land, containing approximately 328,000 cubic yards of shore material. Long-term bluff recession rates, calculated for the period 1836 to 1985, ranged from 0.5 foot to 4.5 feet per year, with an average rate of 1.6 feet per year.

Chapter III

EVALUATION OF COASTAL EROSION PROBLEMS AND DAMAGES

INTRODUCTION

The identification of those shoreland areas that are affected by shoreline erosion, bluff recession, and storm damage is essential to the evaluation of alternative structural and nonstructural shoreline erosion control measures. The purposes of this chapter are to describe those factors contributing to shoreline erosion and bluff recession; to assess the effectiveness of existing shore protection structures under various storm wave and water level conditions; to describe the reaches of the Lake Michigan shoreline of Milwaukee County experiencing bluff toe erosion and having the potential for bluff slope failure; to generally identify the types of shoreline protection measures necessary to control probable future property losses within each of the bluff analysis sections described in Chapter II; and to identify potential property and other economic losses which may result from continued shoreline erosion and bluff recession. This information is intended to enable public officials and other concerned and affected interests to better assess the risk of potential erosion damages and to demonstrate the need for those erosion control measures recommended in Chapter IV of this report.

The results of this chapter are based on the systems level analyses which were conducted to evaluate the condition and needs of each bluff analysis section. The evaluation of individual lakeshore properties and the detailed design of shore protection measures will require further site-specific analyses by a professional geotechnical or coastal engineer. It is intended that this report provide guidance and direction to property owners on what types of shore protection measures may be needed and should be investigated further. The information presented in this report is also intended to be used to help coordinate shore protection efforts of adjacent property owners, and thereby facilitate the design and construction of more effective measures, and help minimize any adverse impacts on nearby shoreline areas.

The Lake Michigan shoreline erosion problems of primary concern are storm damages to major harbor and lakefront structures, beaches, and facilities; the erosion of the toe, or base, of the bluff slope; and the failure of the bluff slope, resulting in the subsequent recession of the top of the bluff. The effectiveness of existing major shore protection structures and beaches to protect major harbor and lakefront facilities against storm damage was determined by a coastal engineering wave analysis, combined with observations made during field surveys conducted in 1987 and 1988. The wave analysis was conducted to estimate wave runup elevations on revetments and beaches, and wave overtopping rates on bulkheads, under selected lake level and storm wave conditions. The analyses identified the potential for damage of shore protection structures and beaches by storm wave overtopping under each of the lake level and storm wave conditions evaluated. The results of the analyses were verified by comparison to field observations made of the structures and beaches, and to videotapes taken during major storm events along the Milwaukee shoreline.

The extent and severity of bluff toe erosion was determined by aerial photograph interpretation and by observations made during field surveys conducted in 1987 for southern Milwaukee County and the Village of Bayside; and in 1986 for the remainder of northern Milwaukee County. The stability of the bluff slopes was evaluated using geotechnical engineering models which calculate the risk of bluff slope failure. Based on the results of both the bluff toe erosion analyses and the slope stability analyses, an assessment of the degree to which toe erosion was contributing to the slope failure was made. In some shoreline areas, erosion by wave action at the toe of the bluff was found to be the primary cause of bluff slope failure, while other areas experiencing toe erosion exhibited relatively stable bluff slopes. An assessment of the effect of toe erosion on slope stability was therefore needed to properly design and develop effective shoreline protection measures.

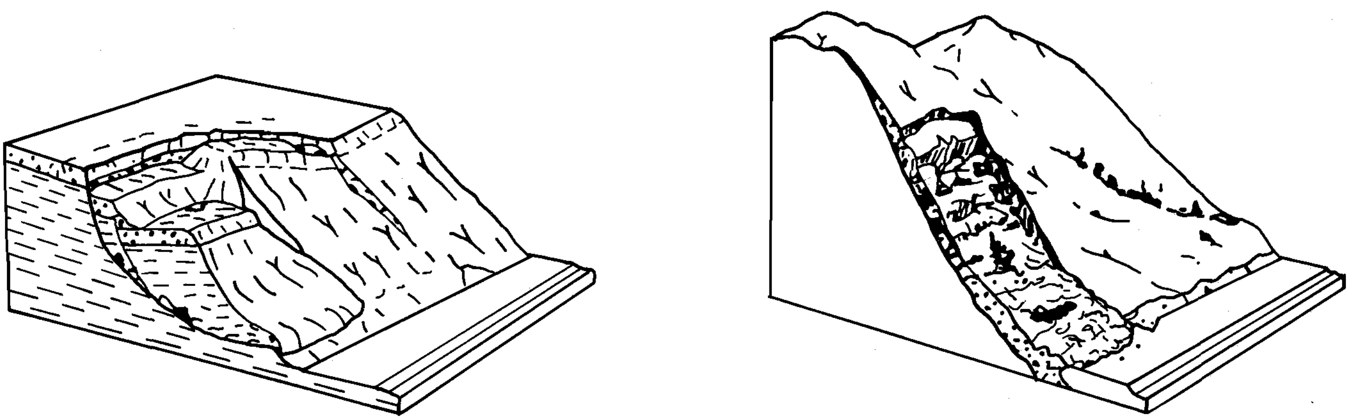
The bluff slope stability analyses were conducted to determine the likelihood of bluff slope failure within the various bluff analysis sections; to determine whether the most likely failures would be deep-seated slumps or shallow slides; to relate slope failures to bluff strata and

Figure 26

COMMON TYPES OF SLOPE FAILURES IN LAKE MICHIGAN COASTAL BLUFFS

ROTATIONAL SLIDING

TRANSLATIONAL SLIDING



Source: David J. Varnes, "Slope Movement and Types and Processes," Landslides: Analysis and Control, Transportation Research Board, National Academy of Sciences, Washington, D. C., Special Report 175, Chapter 2, 1978.

groundwater conditions; and to determine stable slope angles for the bluffs. These analyses utilized geotechnical engineering techniques to quantify and evaluate the strength and stress factors determining bluff slope stability.

The bluff slope stability analyses conducted under this study evaluated the potential for the occurrence of the two types of slope failures most common along the Milwaukee County shoreline: rotational slides and translational slides. Rotational sliding involves failure along a curved, or "spoon shaped," surface. As the slide mass rotates, the top of the slump block often tilts back toward the slope face. Translational sliding involves the failure of a shallow layer along a surface or plane lying generally parallel to the slope face. Figure 26 illustrates the two types of slope failure. The distinction between rotational and translational slides is useful in the planning and design of control measures. As shown in Figure 27, a rotational slide may restore equilibrium in the unstable mass by creating a more stable slope geometry, which decreases the driving momentum, and stops movement of the slide. Thus, bluff slopes undergoing rotational sliding may experience a period of relative stability following the slope failure. Translational sliding, however, may progress continuously if the slope surface is sufficiently inclined, and fallen material is removed from the base of the slope by wave action or some other means.

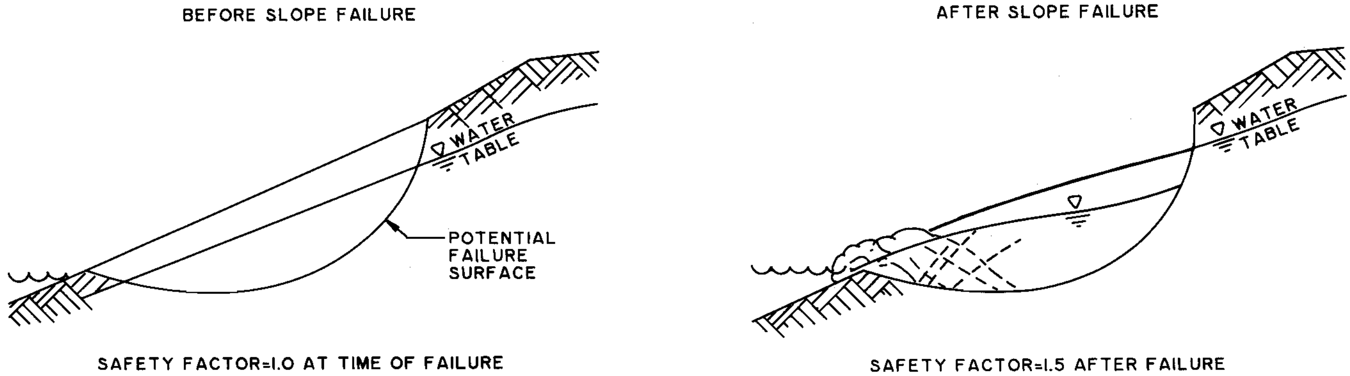
The first section of this chapter following this introduction describes the analytic procedures and coastal and geotechnical engineering techniques used to evaluate existing major shore protection structures and beaches, both within the harbor and on the open coast, and existing shore erosion and bluff recession problems within Milwaukee County. The second section presents the results of these evaluations, and identifies needed control measures for each of the 100 bluff analysis sections. The third section assesses the damages which may result from shoreline erosion, including the extent and economic value of the land and facilities adjacent to the shoreline which may be affected by erosion and bluff recession. A fourth and final section summarizes the coastal erosion problems and damages within Milwaukee County.

METHODS OF ANALYSIS

The effectiveness of existing and proposed shore protection structures is determined by the type of structure, the material the structure is made of, the top elevation and slope of the structure, the water depth at the toe of the structure, the offshore slope, and the incident wave conditions. The degree of erosion occurring at the toe of a bluff is determined by the offshore slope, the wave conditions, the beach width and slope, the type of material in the bluff, and the presence

Figure 27

EFFECT OF ROTATIONAL SLIDING ON SLOPE STABILITY



Source: J. David Rogers, "Slope Stability Evaluations of Various Geologic Situations," Choice of Input Parameters for Slope Stability Analysis, 1986.

of shore protection structures. Factors affecting the stability of the bluff slopes are highly variable and include slope geometry, stratigraphy, soil properties, and groundwater conditions. The following section describes the methods used to evaluate these factors and their effects upon shoreline erosion and bluff recession within the study area.

Water Level and Wave Impacts on Shore Protection Structures and Beaches

The evaluation of the adequacy of existing shore protection structures and beaches requires careful consideration of lake levels and storm wave conditions. This section discusses the range of lake levels and storm wave conditions which may be expected to occur, selects recommended water levels and wave conditions for the evaluation of existing structures and beaches, and describes the procedures used to evaluate the structures and beaches.

An assessment of the potential for wave overtopping damage under various water level and wave height conditions was conducted for 35 major shore protection structures and beaches within Milwaukee County. Thirteen of these structures and beaches lie within the Milwaukee outer harbor and the South Shore breakwater; the remaining 22 structures and beaches are located on the open coast. This analysis supplements the historical review of shore protection structures and the site surveys and inspections for all 128 structures within the County presented in Chapter II.

Recommended Water Levels and Wave Conditions: Statistical analyses of systematically recorded actual water levels represent a sound basis for developing water level projections. However, since the historical record is relatively short—extending in the Milwaukee area back to 1819, with systematic records extending back only to 1860—geological and archaeological information should also be considered in the use of projections based upon historical monitoring records. Such geological and archaeological data are herein presented for verification and comparison purposes. Supplementing the statistical analyses with a review of recorded data and geological and archaeological evidence provides the most comprehensive evaluation of potential water levels.

The available long-term water level records for Lake Michigan at Milwaukee have been summarized by the Regional Planning Commission,[1]

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[1]*SEWRPC Planning Report No. 37, A Water Resources Management Plan for the Milwaukee Harbor Estuary, Volume One, Inventory Findings; Volume Two, Alternative and Recommended Plans, 1987.*

and statistical analyses conducted of the annual, quarterly, monthly, daily, and instantaneous maximum, minimum, and mean water levels. Stage-frequency analyses of the Lake Michigan water level records collected at Milwaukee were also conducted by the Commission. Similar stage-frequency analyses have been conducted by the U. S. Army Corps of Engineers.[2] Prehistoric water levels based on geological information have been estimated by Larsen.[3] Past water levels based upon a review and interpretation of information compiled from several historical, archaeological, climatic, and geologic sources were also estimated by Bishop.[4] Bishop suggested a potential variation in Great Lakes water levels over the next 50 years. Potential water level changes under various climatic and water supply scenarios were examined by Hartmann[5] and Quinn[6] using a hydrologic response model. Using these primary data sources, the range of water levels that may be expected to occur on Lake Michigan was estimated. A more detailed discussion of Lake Michigan water levels is presented in the SEWRPC Staff Memorandum, Review and Analysis of Lake Michigan Water Levels to be Considered in the Preparation of the Lake Michigan Shoreline Erosion, Bluff Recession, and Storm Damage Control Plan for Milwaukee County, May 1988.

The earliest available measurements of Lake Michigan water levels at Milwaukee were recorded in 1819. From 1819 through 1859, the water level records were intermittent and irregular. For the period 1860 through 1905, daily water levels were measured at Milwaukee by the predecessor agencies of the National Ocean Service, National Oceanic and Atmospheric Administration. From 1906 through 1987, hourly instantaneous water level data were recorded at Milwaukee. A summary of the instantaneous maximum and minimum, daily mean, monthly mean, and annual mean water levels recorded over the period 1906 through 1987 is presented in Table 28. The highest monthly mean water level on record—584.3 feet National Geodetic Vertical Datum (NGVD), which is the same as the instantaneous maximum water level shown in Table 28 for March 9, 1987—was measured in 1838.[7]

The water level data collected at Milwaukee subsequent to 1914 have been subjected to several frequency analyses by both the Corps of Engineers and the Regional Planning Commission. In the frequency analyses, the recorded water levels were adjusted as necessary to represent existing diversion, Lake Superior outflow, and outlet channel and structure conditions. Data collected over the period 1906 through 1914 were not used in the frequency analysis because the U. S. Army Corps of Engineers has not adjusted these data to existing diversion and outlet conditions. The adjustment factors were derived by routing the 1900 through 1974 net basin supplies through the Great Lakes under existing diversion and outlet conditions. Under the routing procedures utilized by the Corps, the lakes did not fully respond to the outlet and diversion changes until 1915. Thus, pre-1915 data were not adjusted to existing conditions. The statistical procedures and

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[2]*U. S. Army Corps of Engineers, Report on Great Lakes Open-Coast Flood Levels, Detroit, Michigan, 1977; and Revised Report on Great Lakes Open-Coast Flood Levels, Phase I, Detroit, Michigan, 1988.*

[3]*Curtis E. Larsen, Report presented at the Colloquium on Great Lakes Levels, Water Science and Technology Board of the National Research Council, Chicago, Illinois, March 17-18, 1988.*

[4]*Craig T. Bishop, Great Lakes Water Levels: A Review for Coastal Engineering Design, National Water Research Institute Contribution 87-18, Environment Canada, Burlington, Ontario, 1987.*

[5]*Holly C. Hartmann, Potential Variation of Great Lakes Water Levels: A Hydrologic Response Analysis, Great Lakes Environmental Research Laboratory, Ann Arbor, Michigan, 1987.*

[6]*Frank H. Quinn, Likely Effects of Climate Changes on Water Levels in the Great Lakes, National Oceanic and Atmospheric Administration, Great Lakes Environmental Research Laboratory. Presented at the First North American Conference on Preparing for Climate Change: A Cooperative Approach, Washington D. C., October 27-29, 1987.*

[7]*Bishop, op. cit., 1987.*

Table 28

MAXIMUM AND MINIMUM LAKE MICHIGAN WATER LEVELS AT MILWAUKEE, WISCONSIN: 1906-1987

Water Level	Maximum (feet)			Minimum (feet)		
	Date	IGLD[a]	NGVD[a]	Date	IGLD[a]	NGVD[a]
Instantaneous	March 9, 1987	583.0	584.3	January 23, 1926	574.2	575.5
Daily Mean	October 4, 1986	582.2	583.6	January 27, 1964	575.0	576.4
Monthly Mean	October 1986	581.9	583.2	February 1964	575.4	576.8
Annual Mean	1986	581.2	582.5	1964	575.8	577.1

[a]IGLD - International Great Lakes Datum (1955)
NGVD - National Geodetic Vertical Datum (1929)
At Milwaukee, NGVD = IGLD + 1.34, as determined by first order leveling by SEWRPC.

Source: National Oceanic and Atmospheric Administration and SEWRPC.

adjustments used for the frequency analyses are described in Volume One of SEWRPC Planning Report No. 37.

The U. S. Army Corps of Engineers presented standardized frequency curves for use in determining design water levels for the Great Lakes.[8] As part of the analyses, water-level rise frequency computations were made using data collected at Milwaukee on an annual, quarterly, and monthly basis. Log Pearson Type III frequency analyses were conducted for the period 1915 through 1974. Recorded water levels were first adjusted to reflect existing diversions, outlets, and regulation schedules. This analysis resulted in a 100-year recurrence interval instantaneous maximum Lake Michigan level of 583.7 feet NGVD.

The water level data collected at Milwaukee were subjected to several stage-frequency analyses by the Regional Planning Commission staff to estimate the probability of different lake levels occurring. These analyses utilized a Log Pearson Type III frequency analysis of data taken at four-year intervals in order to avoid autocorrelation effects. A prerequisite to the use of normal or log normal probability theory in the development of stage-frequency analyses is that the annual data be independent of one another. Autocorrelation of water levels measures the tendency of a level to be similar to the previous year's—or subsequent year's—level. Autocorrelation analyses of the annual stage series for Lake Michigan at Milwaukee found strong correlations between water levels in adjacent years, and in two-year lags. A four-year lag was found to produce little autocorrelation. The stage-frequency analyses were based on a period of record of 1915 through 1985, with the levels being adjusted for present diversion and outlet conditions.

The 10-, 50-, 100-, and 500-year recurrence interval instantaneous maximum water levels for Lake Michigan at Milwaukee estimated by the Regional Planning Commission in SEWRPC Planning Report No. 37 are set forth in Table 29. A 100-year recurrence interval instantaneous maximum lake level of 584.5 feet NGVD was calculated. This level is 0.8 foot higher than the level determined by the Corps in its 1977 analyses. The higher level may be attributed to the inclusion in the Commission analyses of water years in the 1970's and 1980's when the Lake Michigan levels were higher than normal.

In 1988, the Corps of Engineers revised its 10-, 50-, 100-, and 500-year recurrence interval flood levels for the Great Lakes for the Federal

[8]*U. S. Army Corps of Engineers, Standardized Frequency Curves for Design Water Level Determinations on the Great Lakes, Detroit District, 1977.*

Table 29

SOUTHEASTERN WISCONSIN REGIONAL PLANNING COMMISSION INSTANTANEOUS MAXIMUM WATER LEVELS FOR VARIOUS RECURRENCE INTERVALS FOR LAKE MICHIGAN AT MILWAUKEE, WISCONSIN[a]

Recurrence Interval (years)	Instantaneous Maximum Water Level (feet)	
	IGLD[b]	NGVD[b]
10	581.6	582.9
50	582.8	584.1
100	583.2	584.5
500	584.0	585.3

[a]*Based on water level records for the period 1915-1985.*

[b]*IGLD - International Great Lakes Datum (1955)*
NGVD - National Geodetic Vertical Datum (1929)
At Milwaukee, NGVD = IGLD + 1.34, as determined by first order leveling by SEWRPC.

Source: SEWRPC Planning Report No. 37, A Water Resources Management Plan for the Milwaukee Harbor Estuary, 1987.

Emergency Management Agency. The analysis represented an update of the 1977 analysis described above, and used hourly instantaneous water level data from 1915 through 1986 collected at Milwaukee. The water levels were adjusted for present diversion and outlet conditions. The original Corps flood levels were revised upward by 0.3 foot for a 10-year flood, 0.5 foot for a 50-year flood, 0.6 foot for a 100-year flood, and 0.9 foot for a 500-year flood.

The Corps statistical analyses addressed frequency distributions, autocorrelation of the data, and regional skew values. The Pearson Type III frequency distribution was used for the analysis. Skew measures the distribution of the magnitude of the water levels. The Corps performed an extensive analysis of regional skew characteristics of the data and recommended that a skew of 0.2 foot be used for Lakes Michigan-Huron. This positive skew results in a greater frequency of extreme high water levels than if a skew of zero is used. The use of the recommended skew in the frequency analyses resulted in the 100-year recurrence interval flood levels being 0.1 to 0.2 foot higher than if a skew of zero had been used. Lakes Michigan-Huron showed the greatest autocorrelation in the yearly data of all of the Great Lakes. However, the Corps concluded that the effect of autocorrelation on the frequency distributions was insignificant because frequency distributions of even-year data were similar to the frequency distributions of odd-year data.

The new Corps of Engineers flood levels for the sections of the Lake Michigan shoreline in southeastern Wisconsin are set forth in Table 30. The flood levels, or instantaneous maximum levels developed by the Corps, are essentially the same—within 0.2 foot—as the levels developed by the Regional Planning Commission using data from 1915 through 1985 for all recurrence intervals. As shown by comparison of Tables 29 and 30, the 10- and 500-year recurrence interval instantaneous maximum levels are within 0.1 foot of the levels estimated by the Regional Planning Commission. The 50-year and 100-year recurrence interval levels determined by the Corps are within 0.2 foot of the levels estimated by the Regional Planning Commission. The new Corps of Engineers values thus essentially confirm those developed by the Commission as published in 1987.

The Regional Planning Commission performed a frequency analysis of instantaneous minimum water levels using the same procedures used by the Corps of Engineers for the maximum water level analysis. Table 31 presents 10-, 50-, 100-, and 500-year recurrence interval instantaneous minimum water levels for Lake Michigan at Milwaukee based on a period of record of 1915 through 1986. Ninety percent confidence intervals are also presented in the table.

Mathematical simulation models may be used to estimate the potential for water level variations in response to a range of climatic conditions. These models have been used by researchers to simulate the water level impacts on the Great Lakes of several hydrometeorological and water management scenarios, including changes in the net basin water supplies, increased outflows from Lake Superior, modifications to diversions in the Great Lakes system, an increase in the flow capacity of certain lake outlets, and climate

Table 30

U. S. ARMY CORPS OF ENGINEERS FLOOD LEVELS FOR THE LAKE MICHIGAN SHORELINE OF SOUTHEASTERN WISCONSIN: 1988

Section[a]	General Location	Instantaneous Maximum Water Levels (feet NGVD)							
		10-Year		50-Year		100-Year		500-Year	
		Level	90 Percent Confidence Interval	Level	90 Percent Confidence Interval	Level	90 Percent Confidence Interval	Level	90 Percent Confidence Interval
1[b]	Kenosha-Racine	583.1	582.8-583.5	584.2	583.8-584.7	584.6	584.1-585.2	585.6	585.0-586.3
2	Milwaukee-Port Washington	582.8	582.5-583.2	583.9	583.5-584.4	584.3	583.8-584.9	585.2	584.6-585.9

[a]*Section 1 extends from the Wisconsin-Illinois State line north to Wind Point in Racine County; Section 2 extends from Wind Point north to the Ozaukee-Sheboygan County line.*

[b]*Confidence intervals for Section 1 were not estimated by the U. S. Army Corps of Engineers because no water level gaging stations were located within the section. Therefore, the confidence intervals shown for Section 1 were interpolated by the Regional Planning Commission staff using confidence intervals calculated for the Milwaukee (Wisconsin) and Calumet Harbor (Illinois) gaging stations.*

Source: U. S. Army Corps of Engineers Revised Report on Great Lakes Open-Coast Flood Levels, Phase One, Detroit, Michigan, April 1988, and SEWRPC.

Table 31

INSTANTANEOUS MINIMUM WATER LEVELS FOR LAKE MICHIGAN AT MILWAUKEE[a]

Instantaneous Minimum Water Levels (feet NGVD)							
10-Year		50-Year		100-Year		500-Year	
Level	90 Percent Confidence Interval	Level	90 Percent Confidence Interval	Level	90 Percent Confidence Interval	Level	90 Percent Confidence Interval
576.2	575.9-576.6	575.1	574.7-575.6	574.9	574.4-575.5	574.3	573.8-575.1

[a]*Based on a period of record of 1915 through 1986.*

Source: SEWRPC Planning Report No. 37, A Water Resources Management Plan for the Milwaukee Harbor Estuary, 1987.

changes. The models were used to estimate the water levels that may be expected to occur under each of the scenarios considered. Thus, the model results are not used to actually predict water levels. Rather, the model results help identify those conditions that would produce relatively high or low water levels.

The hydrologic response model developed by the Great Lakes Environmental Research Laboratory of the National Oceanic and Atmospheric Administration was used by the Research Laboratory to examine the potential lake level response to continued high water supplies for a 20-year period under four different scenarios: 1) a continuation of the recorded maximum monthly net basin supplies; 2) a 75 percent increase in the 1900 through 1985 mean net basin supplies; 3) a 50 percent increase in the 1900 through 1985 mean net basin supplies; and 4) a 25 percent increase in the 1900 through 1985 mean net basin supplies.[9] This 20-year period used for the modeling allowed the lakes to reach equilibrium and fully respond to the net basin supplies.

The lake levels estimated by applying the hydrologic response model are set forth in Table 32. The study noted that in order to raise Lake Michigan about three feet above its October 1986 record monthly level, net basin supplies, including Lake Superior outflows, would need to be increased by 50 percent above the long-term—1900 to 1985—average. Bishop concluded that, based on Hartmann's modeling results, an elevation of 583.7 NGVD could be considered a realistic maximum, or upper bound, monthly level of Lake Michigan over the next 50 years.[10] Assuming an increase in elevation of 2.0 feet for wind setup and seiche, this would result in an instantaneous maximum level of about 585.7 feet NGVD.

A 50 percent increase in net basin supplies could occur, at least for a short period of time. The combined net basin supplies for Lakes Michigan-Huron in 1985 was 53 percent above the long-term average. Furthermore, even a short-term increase in net basin supplies may substantially increase water levels. The hydrologic response model runs showed that about 90 percent of the water level rise occurs in the first six years. An independent analysis of Hartmann's data shows that the Lakes Michigan-Huron Basins had combined net basin supplies 25 percent above average for the six-year period of 1981 through 1986.[11] A 50 percent increase in net basin supplies over a long term is very unlikely, although it is a possibility. Such conditions have been seen only in the Lake Erie Basin. The maximum six-year average net basin supplies for the Lake Erie Basin, which occurred from 1972 through 1977, were 60 percent higher than the twentieth century average net basin supplies to that lake.

Table 32

LAKE MICHIGAN MAXIMUM MONTHLY MEAN WATER LEVELS ESTIMATED WITH A HYDROLOGIC RESPONSE MODEL ASSUMING INCREASED NET BASIN WATER SUPPLIES

Net Basin Supply Scenario	Water Level (feet) IGLD[a]	NGVD[a]
1. A continuation of the recorded maximum monthly net basin supplies	593.4	594.7
2. An increase in the 1900 through 1985 mean net basin supplies by 75 percent	587.3	588.6
3. An increase in the 1900 through 1985 mean net basin supplies by 50 percent	584.6	585.9
4. An increase in the 1900 through 1985 mean net basin supplies by 25 percent	581.7	583.0

[a]IGLD - International Great Lakes Datum (1955)
NGVD - National Geodetic Vertical Datum (1929)

Source: Holly C. Hartmann, Potential Variation of Great Lakes Water Levels: A Hydrologic Response Analysis, Great Lakes Environmental Research Laboratory, Ann Arbor, Michigan, 1987.

Researchers at the National Oceanic and Atmospheric Administration have expressed concern that a climatic warming resulting from an increase in carbon dioxide and other gases in the atmosphere could result in a 15 to 30 percent decrease in the average net water basin supplies to the Great Lakes.[12] This climate warming is generally referred to as the "greenhouse effect." The researchers estimated that this decrease in net water basin supplies could result in a 2.5- to 5.0-foot decline in the average twentieth century Lake Michigan water levels. If this occurred, the now record-low water levels would become common. It was also reported that the annual variability of water levels could decline by 4 to 11 percent. Since the variability, and thus the range, of water levels could decrease, extreme minimum water levels could decline by less than the 2.5- to 5.0-foot decline provided for overall water levels, while extreme maximum water levels could decline by more than 2.5 to 5.0 feet. More recent studies by the U. S. Environmental Protection Agency have indicated that the

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[9]*Hartmann, op. cit.*

[10]*Bishop, op. cit.*

[11]*J. Philip Keillor, "Caught by Surprise: The Great Lakes Water Level Crisis of 1985-1987," 1988. Draft.*

[12]*Quinn, op. cit.*

climatic warming resulting from the greenhouse effect may lower the Lake Michigan waters by as much as three to eight feet.[13]

Therefore, the 100-year recurrence interval instantaneous minimum water level of 574.9 feet NGVD presented in Table 31 may be expected to decline by less than 2.5 to 5.0 feet. Further analysis of the "greenhouse" effect should more clearly define the climatic warming that may be expected to occur over relatively small areas such as the Great Lakes drainage basin. It is also likely that any warming will be small initially, then escalate later in the twenty-first century. Thus, several periods of extreme high lake levels could occur even if long-term average lake levels were declining. The "greenhouse" effect modeling analyses that have been conducted to date are based on somewhat arbitrary and artificial conditions. Thus, these results should not be interpreted to represent actual future conditions. Rather, the study results should be considered to be an evaluation of the potential lake level response to a set of assumed meteorological conditions, connected channel sizes, and diversions.

Recorded water levels, and water level estimates based on statistical frequency analyses, geologic and archaeologic evidence as presented in Chapter II, and simulation modeling of climatic changes, are graphically illustrated in Figure 28. The lake level estimates presented herein suggest that lake levels up to six feet above the recently estimated 100-year recurrence interval level of 584.3 feet NGVD may be possible. However, the net basin water supplies would need to be substantially increased for a long period of time—at least five or six years—to produce lake levels significantly higher than the 100-year recurrence interval water levels determined by the Regional Planning Commission in 1987 and the U. S. Army Corps of Engineers in 1988.

The lake water levels selected for the evaluation of existing major shore protection structures and beaches are graphically shown in Figure 29. The storm of March 9, 1987, was used to help verify the results of wave modeling conducted in the Milwaukee Harbor by comparing the results to observed wave conditions as shown in records and photographs taken by the Port of Milwaukee staff and in video tapes obtained from Milwaukee television stations and from the U. S. Army Corps of Engineers. A water level of 584.3 feet NGVD was recorded on March 9, 1987, which included a seiche and wind setup of 2.5 feet. This is the same level as the new U. S. Army Corps of Engineers 100-year flood stage, and essentially the same as the recommended regulatory 100-year recurrence interval instantaneous maximum stage developed by the Regional Planning Commission in 1987 of 584.5 feet NGVD. Thus, the March 9 model run may also be assumed to represent 100-year recurrence interval conditions with respect to lake level. Within this report, this first water level—584.3 feet NGVD—is hereinafter referred to as the 100-year water level.

The second water level used in the analyses, 585.9 feet NGVD, represents an upper bound in potential high lake levels. For this level, the upper 90 percent confidence limit of the 500-year recurrence interval instantaneous water level estimated by the U. S. Army Corps of Engineers in 1988 was selected. This second water level, evaluated as a worst case condition, also may be used to assess the possibility that flood levels may be revised upward in the future if above-average water levels recur for an extended period of time. This second water level is hereinafter referred to as the 500-year water level.

The third water level used in the analyses is the U. S. Army Corps of Engineers 10-year recurrence interval instantaneous maximum water level of 582.8 feet NGVD estimated in 1988, which is essentially the same—within 0.1 foot—as the revised 10-year recurrence interval instantaneous maximum lake level developed by the Regional Planning Commission in 1987. This lower lake level may be appropriate for evaluating shore protection facilities that are not protecting major facilities or public works improvements. This third water level is hereinafter referred to as the 10-year water level.

In addition to the three instantaneous maximum water levels described above, consideration was given to two potential low water elevations. The first low water level considered was the 100-year recurrence interval minimum monthly mean

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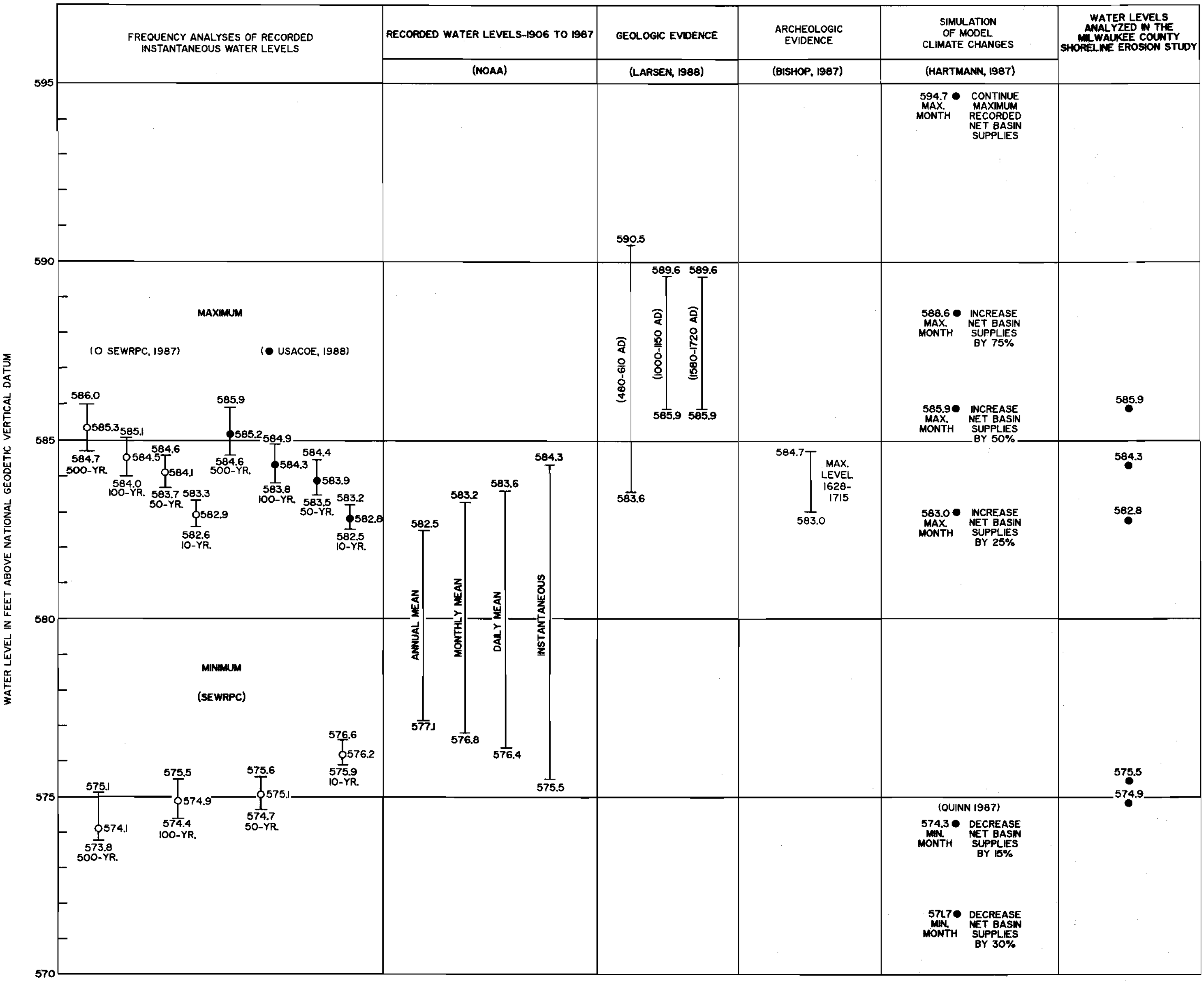
[13] *U. S. Environmental Protection Agency, "The Potential Effects of Global Climate Change on the United States, Executive Summary," Draft Report to Congress, October 1988.*

Figure 28

LAKE MICHIGAN WATER LEVELS DEVELOPED BY VARIOUS SOURCES



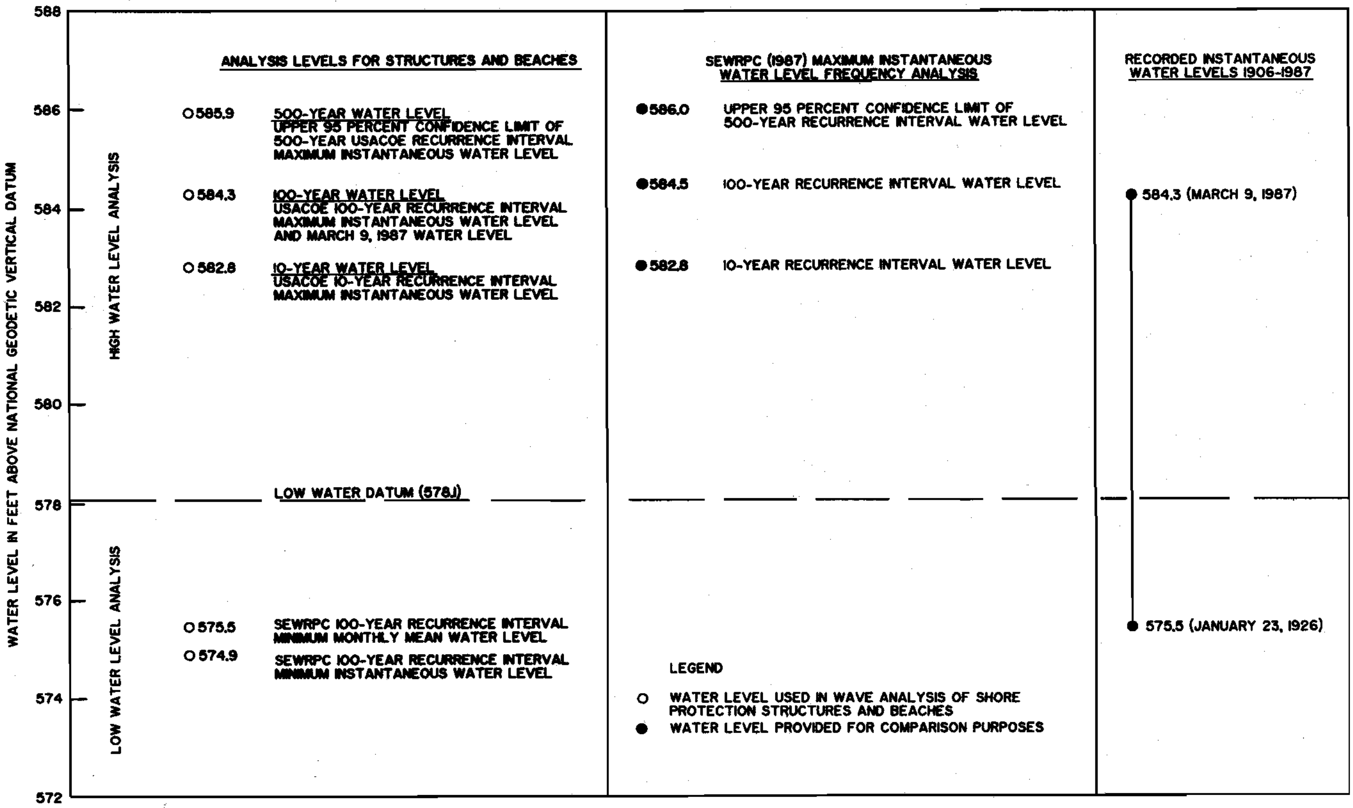
Source: SEWRPC.

water level of 575.5 feet NGVD as developed by the Regional Planning Commission. A monthly low water level was selected since the impacts on structures due to exposure of normally submerged components, such as timber pilings, would be more severe under longer term periods. This monthly mean level represents a reduction of approximately 5.0 feet from the 1987 minimum monthly mean level. In this respect, it is consistent with the 1987 work done by the National Oceanic and Atmospheric Administration, which indicated that a 2.5- to 5.0-foot decline in overall Lake Michigan water levels could occur as a result of the "greenhouse effect," and with the 1988 work done by the U. S. Environmental Protection Agency, which estimated a potential decline of 3.0 to 8.0 feet.

A second low water level considered in this analysis is the 100-year recurrence interval instantaneous minimum level of 574.9 feet NGVD calculated by the Commission staff. This

Figure 29

LAKE MICHIGAN WATER LEVELS USED FOR THE EVALUATION OF SHORE PROTECTION STRUCTURES AND BEACHES



Source: SEWRPC.

level is appropriate for the consideration of impacts, such as toe scour, which could be aggravated by low levels. This level is within 1.1 feet of the lower limit of the 90 percent confidence level for the 500-year recurrence interval instantaneous minimum level, and thus can be considered to be near the "worst case" low water level condition.

Wave Impacts on Major Shore Protection Structures and Beaches: The impact of waves on structures and beaches is dependent upon the wave condition and wave climate. The wave condition is the particular combination of wave heights, wave periods, and wave directions at a given time. Wave climate is the temporal distribution of wave conditions over a period of years. Wave conditions offshore depend on the wind velocity, wind duration, and fetch. Deep-water waves as high as 25 feet have been reported on the Great Lakes. On the night of November 10, 1975, vessels in the vicinity of the stricken vessel the Edmund Fitzgerald reported waves 16 to 25 feet in height and winds of 50 to 67 miles per hour.[14]

As storm waves travel from deep water into shallow water, they begin to feel and be impacted by the bottom of the lake and usually change height and direction. These changes may be attributed to shoaling, refraction, and bottom friction. Waves approaching a shoreline typically feel bottom when the water depth is

[14]*Paul Trimble, Vice Admiral, U. S. Coast Guard (ret.) and President, Lake Carrier's Association, September 16, 1977, letter to Mr. Webster B. Todd, Jr., Chairman, National Transportation Safety Board.*

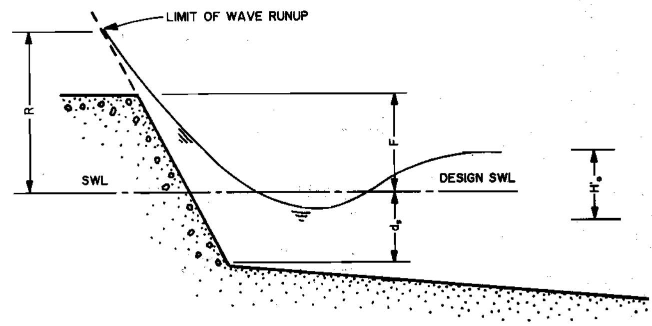
approximately one-half the wave length.[15] As a result of the decrease in water depth, each wave slows down and other waves crowd in behind it, causing the wave length to shorten and the wave height to increase until the wave breaks. This process is known as shoaling. Refraction is the bending of wave crests due to the slowing down of that part of the wave crest that is in shallower water.

Waves break when they reach a limiting water depth that is proportional to their height, and at that condition, the wave height is greatly reduced. Waves normally break in water depths of about 1.3 times the wave height.[16] Therefore, by the time waves reach the shoreline, the largest waves have broken. Once a wave breaks, it reforms into smaller waves, although the height of these newly formed waves is limited by the available water depth. Near-shore wave heights are typically limited to 55 to 65 percent of the water depth.[17] This is an important factor concerning shore protection, since the amount of wave energy that is expended against the shoreline is proportional to the wave height squared. Rising water levels create deeper water, which allows larger waves to break closer to the shore. Water levels thus have a substantial effect on shoreline recession rates and damages to coastal structures.

The stability of a shore protection structure may be adversely affected by high lake levels and severe storms. Large waves reaching the shoreline can cause considerable damage to structures by overtopping, flanking, toe scouring, or material failure. Of these various types of failure, overtopping is the most common. As discussed in Chapter II, more than twice as many structures are affected by overtopping than by any other type of failure. Evidence of wave overtop-

Figure 30

SCHEMATIC DIAGRAM OF WAVE RUNUP ON A SHORE PROTECTION STRUCTURE



LEGEND

R = WAVE RUNUP HEIGHT ON STRUCTURE
F = STRUCTURE HEIGHT ABOVE STILL WATER LEVEL
SWL = STILL WATER LEVEL
d_s = WATER DEPTH AT TOE OF STRUCTURE
H'_o = DEEPWATER WAVE HEIGHT

Source: S. L. Douglas, Review and Comparison of Methods for Estimating Irregular Wave Overtopping Rates, U. S. Army Water Ways Experiment Station, Coastal Engineering Research Center, December 1986.

ping was observed for nearly 60 percent of all structures surveyed. Of those structures that were in need of repair, about 80 percent were being damaged by overtopping. Particularly during periods of high lake levels and severe storms, wave overtopping damage will be much more common than other types of damage. Therefore, the impact of the selected lake levels and storm wave conditions on major shore protection structures was determined by estimating the wave runup, or overtopping rates, for major revetments, bulkheads, and beaches. It is recognized that other types of damage—flanking, toe scour, and material failure—are also more likely to occur under high water levels than under lower water levels. The wave runup and overtopping analysis was thus intended to be representative of the overall impact on the stability of the structures. Low-water impacts on structural stability were also evaluated; in fact, low water levels can exacerbate toe scouring of some structures. It is assumed that new structures will be constructed in conformance with recommended design criteria to adequately protect against these various types of failure.

Wave runup is defined as the maximum distance water will rise when washing up on a beach or shore protection structure, as shown in Fig-

[15] *U. S. Army Corps of Engineers, Shore Protection Manual, Vol. I, Coastal Engineering Research Center, 1984.*

[16] *Ibid.*

[17] *S. A. Hughes, The TMA Shallow-Water Spectrum Description and Applications, U. S. Army Corps of Engineers Coastal Engineering Research Center Technical Report CERC-84-7, 1984.*

ure 30. This distance depends on wave conditions, as well as the characteristics of the beach or structure. Generally, structures that are more porous, rough, and gently sloping have lower runups than those that are impervious, smooth, or steeply sloped, as illustrated in Figure 31.

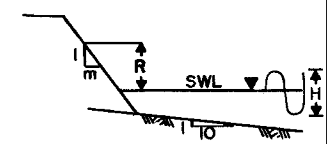
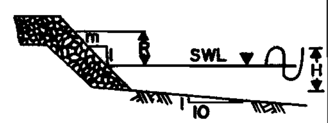
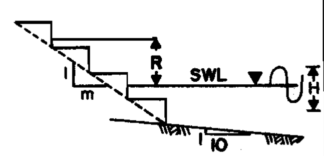
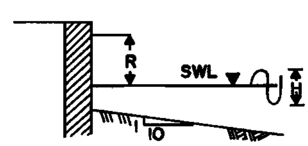
As waves hit a coastal structure, the water rushes up and sometimes over the structure. Thus, wave runup and wave overtopping are closely related phenomena. Several different aspects of overtopping are important to evaluating the adequacy of existing coastal structures. For structures such as revetments or bulkheads which lie at the shoreline, the amount of water that flows over the structure is important because of backside flooding. Erosion of the bluff also occurs above and behind the structure. The amount of wave energy transmitted over a breakwater is a function of the wave height which overtops the breakwater. In addition, the stability of the backside of a rubble mound breakwater may be affected by overtopping.

Wave runup on a revetment depends on the shape, slope, and roughness of the revetment; the water depth at the toe; the bottom slope in front of the revetment; and the incident wave conditions. Revetments may contain splash aprons and drainage systems to help reduce damages caused by overtopping. For a given site, wave runup heights on revetments, because of their porosity, would be generally lower than would the runup heights on either beaches or bulkheads.

An accurate estimate of runup on a beach is difficult because beaches often have complex profiles which are constantly being reshaped by the waves. Storm waves steepen beaches, causing the runup distance to increase. During the interim periods between storms, small waves—along with deposited littoral drift material—create a gentler beach profile, resulting in relatively less runup. This interaction of beach slope and wave conditions means that maximum wave runup estimates for beaches are best made in spring or fall when the beach is most likely to be at its steepest slope. Beach slopes used in this report to estimate beach runup were measured in the fall of 1987. Wave runup on a beach also depends on the time between successive waves, and the grouping of waves approaching the beach. Maximum wave runup occurs when the preceding backwash of water is small and the large incoming wave can run unhindered up the beach slope.

Figure 31

RELATIVE WAVE RUNUP ON DIFFERENT TYPES OF SHORE PROTECTION STRUCTURES LOCATED WITHIN A PROTECTED HARBOR AREA

 SMOOTH FACE	m 1.5 2.5 4.0	R 2.25H 1.75H 1.50H
 ROUGH FACE	m 1.5 2.5 4.0	R 1.25H 1.00H 0.75H
 STEPPED FACE	m 1.5	R 2.00H
 VERTICAL FACE	m --	R 2.00H

Source: U. S. Army Corps of Engineers, Low Cost Shore Protection. . . A Guide for Engineers and Contractors, 1981.

The approach used to evaluate the adequacy of a bulkhead in terms of wave runup is different from the approach used for beaches or revetments. The required height of bulkheads to prevent all wave overtopping can be estimated, but the bulkheads would need to be very high because of the splash of the waves. A rule of thumb is that the vertical splash distance is three to seven times the height of the approaching wave.[18] Instead, adequate bulkhead elevations are estimated for "acceptable" overtopping

[18]*J. P. Ahrens, Presentation at the December 6, 1988, Conference on Coastal Engineering for the Great Lakes, University of Wisconsin, Madison, Wisconsin.*

rates of water that can be drained away without jeopardizing the stability of the bulkhead. This approach recognizes that if the overtopping rates are not too great, provisions can be made to drain away water without erosion of the bank or bluff behind the wall.

The analysis of wave runup and overtopping was conducted for each of the three instantaneous maximum water levels presented in Figure 29. For each major structure or beach and for each water level evaluated, runup was determined for 20-year and 50-year recurrence interval storm waves. A 20-year storm wave condition has a 5 percent chance in any one year that its wave height will be exceeded. A 50-year storm wave condition has a 2 percent chance in any one year that its wave height will be exceeded. These conditions were selected as severe storm conditions that could occur at the same time as the three selected maximum lake water levels. Thus, six alternative high water level-wave conditions were considered for each structure or beach.

This analysis gives a relative indication of potentially significant and serious overtopping of shore structures and beaches under a given combination of stormwater level and wave conditions. For each structure or beach under each stormwater level and wave condition, the potential for wave overtopping damage was classified as insignificant, low, moderate, or high. These classifications were made for convenience in prioritizing structures and beaches with potential overtopping problems. All structures and beaches should be periodically monitored and inspected for indications of overtopping damage after major storm events. A site-specific engineering analysis is needed to determine whether an individual structure or beach would, in fact, be damaged by a certain level of overtopping, and to select an appropriate design height for a structure or beach.

A set of deepwater design wave conditions was required to evaluate both the open coast and the harbor structures and beaches. Deep-water wave heights and periods for the Great Lakes are set forth in Design Wave Information for the Great Lakes by D. T. Resio and C. L. Vincent (1976). These deepwater waves are based on hindcasts using an elaborate numerical model together with actual storm-wind data as recorded at coastal airports. The wave conditions are far enough offshore to prevent any shoaling effects.

It should be noted that the wave heights given by Resio and Vincent are "significant wave heights"—the average of the highest one-third of all the waves at a point. Thus, a substantial portion of the waves would be higher than these values. However, significant wave heights are commonly used in coastal engineering for both design and evaluation purposes.

For the Lake Michigan shoreline of Milwaukee County, the following design wave heights and wave periods were calculated using the Resio and Vincent procedures:

Storm Recurrence Interval (years)	Wave Height (feet)	Wave Period (seconds)
20	21.0	10.9
50	24.8	12.0

To evaluate the open coast structures and beaches, these deep-water waves were moved onshore using equations and procedures described later in this chapter. To evaluate those structures located within the outer harbor or South Shore breakwater, the deep-water waves were used as input to a numerical harbor wave and oscillation simulation model modified and applied under this study.

Open Coast Structures and Beaches: There are 22 major structures and beaches located along the Milwaukee County shoreline outside the Milwaukee outer harbor and South Shore breakwater. These structures and beaches, particularly those located in areas with steep offshore slopes, are subject to direct wave attack as the waves break against the shoreline, rather than on offshore structures.

Revetments: The eight revetments located on the open coast evaluated in this section are constructed of either quarried rock—generally limestone or granite—or concrete rubble. Therefore, procedures developed to estimate the wave runup on a riprap—or randomly placed rock—revetment were used in this study. These procedures, developed by the U. S. Army Corps of Engineers and referred to herein as the Ahrens method,[19] were used to calculate the maximum

[19]*J. P. Ahrens and M. S. Heimbaugh, Approximate Upper Limit of Irregular Wave Runup on Riprap, U. S. Army Coastal Engineering Research Center, Technical Report CERC-CO-88-5, May 1988.*

wave runup on riprap revetments caused by irregular wave action. The estimate of runup of irregular waves is a substantial improvement over more conventional engineering methods which evaluate the runup of regular waves. Conventional methods based on regular waves assume that waves move in an orderly fashion, have the same size, and travel at uniform spacing. The irregular wave approach herein applied in the Ahrens method more accurately represents actual lake wave conditions. The formulas used to calculate the wave runup were derived from laboratory tests of riprap revetments exposed to irregular waves conducted by the U. S. Army Corps of Engineers Coastal Engineering Research Center, Vicksburg, Mississippi.

The calculation of wave runup on revetments using the Ahrens method consists of three parts. The first part is the estimate of near-shore wave height at the toe of the structure. The second part is the determination of the surf parameter, which is a function of the slope of the structure, the near-shore wave height, and the wave length. The surf parameter used in the Ahrens method is a measure of the ratio of wave steepness to structure steepness. The third part is the calculation of the runup on the revetment.

The near-shore wave height at the toe of a structure is dependent on the deep-water wave conditions and the wave shoaling and breaking that occurs in the near-shore zone. Ahrens and Heimbaugh[20] recommended that three different procedures be used to estimate the near-shore wave height. However, in calculating the near-shore wave heights for revetments in Milwaukee County, it was found that the depth-limited wave height was always lower than the wave height estimated by the other two procedures. Therefore, the depth-limited wave height is the limiting condition that determines wave height for all of the wave and water level conditions used in this study. The depth-limited wave height is calculated by the following formula given by Ahrens and Heimbaugh:

[20]*Ibid.*

$$Hmo = \left[0.10 \tanh\left(\frac{2\pi\, ds}{Lo}\right)\right] Lo$$

where:

Hmo = significant depth-limited wave height in feet (average of the one-third largest wave heights)

ds = water depth at toe of structure in feet

Lo = deep-water wave length, in feet, calculated by:

$$Lo = \frac{gT^2}{2\pi}$$

where:

T = deep-water wave period, in seconds

g = acceleration of gravity, or 32.2 feet per second^2

The surf parameter for irregular waves, which is the second part of the Ahrens method, is defined as:

$$S = \frac{\tan\theta}{(Hmo/Lo)^{\frac{1}{2}}}$$

where:

S = surf parameter, which is dimensionless
θ = angle between the structure slope and the horizontal, in degrees

Wave runup on a revetment, which is the third part of the Ahrens method, is calculated as follows:

$$R = Hmo\left[\frac{a\,S}{1.0+bS}\right]$$

where:

R = wave runup height above the still-water level on structure, in feet

a and b = dimensionless runup coefficients determined by regression analyses of laboratory data on irregular waves:

a = 1.154
b = 0.202

Most revetments in Milwaukee County are not designed to withstand damage if a significant amount of runup exceeds the top of the structure. Splash guards and drainage systems are seldom incorporated into the designs. Thus, any significant runup that exceeds the height of a revetment may damage the structure. The calculated runup levels represent the maximum vertical excursion of "green" water near the middle of the structure. Spray or splash from the waves is not considered runup for the purposes of these calculations. If the calculated wave runup did not exceed the top of the revetment, an insignificant potential for overtopping damage was indicated. A revetment was considered to have a low potential for overtopping damage if the calculated wave runup exceeded the top of the revetment by less than 1.0 foot. The potential for overtopping damage was considered moderate if the wave runup exceeded the structure height by 1.1 to 5.0 feet, and high if the wave runup exceeded the structure height by more than 5.0 feet.

Beaches: During storms, damages in beach areas may occur owing to the erosion of beach sediment, and to wave runup which overtops the beach and erodes or floods land or facilities located behind the beach. The loss of beach sediment depends on wave characteristics, the size of the beach material, and the beach slope. An assessment of the long-term impacts of storm waves on beach sediment would require detailed surveys of beach profiles, particle-size distributions, and littoral drift transport rates. Such an analysis is beyond the scope of this systems level plan.

Since the existing major beach sites evaluated in this section have extensive sand deposits, it was assumed, for the purposes of this plan, that beach sediment would continue to be retained if the beach containment structures—such as groins or breakwaters—were properly designed to protect against the high lake levels being considered. However, it is also recognized that higher lake levels will not only inundate a portion of the beaches, but also allow larger waves to reach the shoreline, thereby somewhat increasing beach erosion. Thus, if high lake levels persist for several years, additional beach nourishment may be required to protect the shoreline, even if the beach containment structures are designed for those levels.

The wave runup on seven existing beaches on the open coast was estimated under each of the lake level and storm wave conditions evaluated. Wave runup on beaches is a complicated process because the beach slope can be reshaped by the waves and because some waves break far offshore owing to the gentle offshore slope. A wave approaching a beach may refract and will shoal and break. After breaking, the water mass, or "swash," is driven up the beach slope by inertia. After reaching the highest point of runup, the water flows back again by gravity until the backflow is eliminated by the following wave.

A method developed by Holman[21] was used to estimate the runup on beaches. This method uses near-shore wave conditions just outside the breaker zone—rather than deep-water wave conditions—to predict runup. Holman's method was derived from field measurements made during two storm events at the U. S. Army Corps of Engineers Coastal Engineering Research Center's Field Research Facility at Duck, North Carolina, where the deep-water wave heights ranged from 1.3 to 13 feet, the wave periods from 6 to 16 seconds, and the beach slopes from 7 to 10 percent. The following runup relationship equation, which is a modification of Hunt's equation,[22] was suggested by Holman:

$$R/Hs = \hat{S}$$

where:

R = maximum wave runup on a beach above the still-water level, in feet

Hs = near-shore wave height, in feet, which may be calculated for a water depth of 20 feet using the Ahrens and Heimbaugh formula for depth-limited waves

$\hat{S}$ = a dimensionless surf parameter, similar to that discussed above for the Ahrens method, but based on beach slope and on the wave height at a mean water depth of 20 feet

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[21] *R. A. Holman, "Extreme Value Statistics for Wave Run-Up on a Natural Beach,"* *Coastal Engineering, Vol. 9, 1986, pp. 527-544.*

[22] *I. A. Hunt, "Design of Seawalls and Breakwaters,"* *Proceedings of the American Society of Civil Engineers, Vol. 85, 1959, pp. 123-152.*

The surf parameter is strongly affected by nearshore dynamics. Surf parameters above a certain minimum indicate that the incident waves will not break before reaching the beach. Below this minimum, breaking occurs to reduce wave height such that the minimum surf parameter is maintained on the shore face. Unfortunately for erodible beaches which change their shape or texture during storms, the surf parameter cannot be determined a priori because the beach slope is itself a function of the incident wave characteristics. In other words, the beach slope will change substantially during a major storm event, which will affect the runup distance. For storms, Holman suggested that the surf parameter be estimated using the following equation:

$$\hat{S} = 6.3 \tan B$$

where:

B = beach slope before the storm, in degrees

Thus, runup on a beach may be calculated as follows:

$$R = (6.3 \tan B)Hs$$

For each beach evaluated, if the calculated runup height exceeded the elevation of the top of the beach, the beach was considered overtopped. Facilities and bluffs located behind a beach are usually not protected, and erosion or flooding damages can readily occur. The classification of the potential for overtopping damage to beaches was the same as that discussed above for revetments. If the calculated wave runup did not exceed the top of the beach, an insignificant potential for overtopping damage was indicated. A low potential for overtopping damage was indicated if the calculated wave runup exceeded the maximum beach elevation by less than 1.0 foot. The potential for overtopping damage was considered moderate if the wave runup exceeded the beach elevation by 1.1 to 5.0 feet, and high if the wave runup exceeded the beach elevation by more than 5.0 feet.

Bulkheads: The evaluation of wave runup on bulkheads is based on the overtopping rate—in cubic feet per second per lineal foot of bulkhead (cfs per foot)—rather than simply on the runup height, because bulkhead heights that equal runup distances would be unreasonably high, and because provisions to drain water that overtops the structure can be readily incorporated into the design of most bulkheads. Therefore, acceptance of some level of overtopping is a compromise with the cost of construction or modification of a bulkhead. Overtopping rates of up to 0.01 cfs per foot should not cause major damage to a bulkhead regardless of whether a drainage system is in place.[23] Overtopping rates ranging from 0.01 to 0.1 cfs per foot may be safely drained by a well-designed drainage system. However, rates in this range could damage bulkheads that do not include an adequate drainage system. Overtopping rates exceeding 0.1 cfs per foot could damage even major bulkheads that contain a drainage system. This larger rate is used as a general guideline for dockwalls in Japan which include large drainage systems.[24]

Even well-designed bulkheads with large drainage systems may undergo structural failure if they are exposed to heavy wave overtopping for several hours. Such failure may include leakage, cracks, and breakage of the bulkhead wall and toe protection; erosion of the soil behind the structure; and total collapse. The susceptibility to overtopping damage is in part dependent on the materials used; a bulkhead with concrete on the front face and soil on the crown (top) and backslope is 10 times more susceptible to damage than a bulkhead of equal height that has concrete on the front face, crown, and backslope.[25]

For each of the seven bulkheads on the open coast evaluated in this section, the overtopping rate was estimated for each water level and storm wave condition using graphs formulated by Goda[26] and shown in Figures 32 through 35. The graphs were derived from irregular wave hydraulic laboratory tests and the calculation of wave deformation in the surf zone. The symbol *Ho* in the figures represents the deep-water wave height; *h* the water depth at the toe of the structure; *hc* the crest elevation of the bulkhead

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[23] *Y. Goda, Random Seas and Design of Maritime Structures, 1985.*

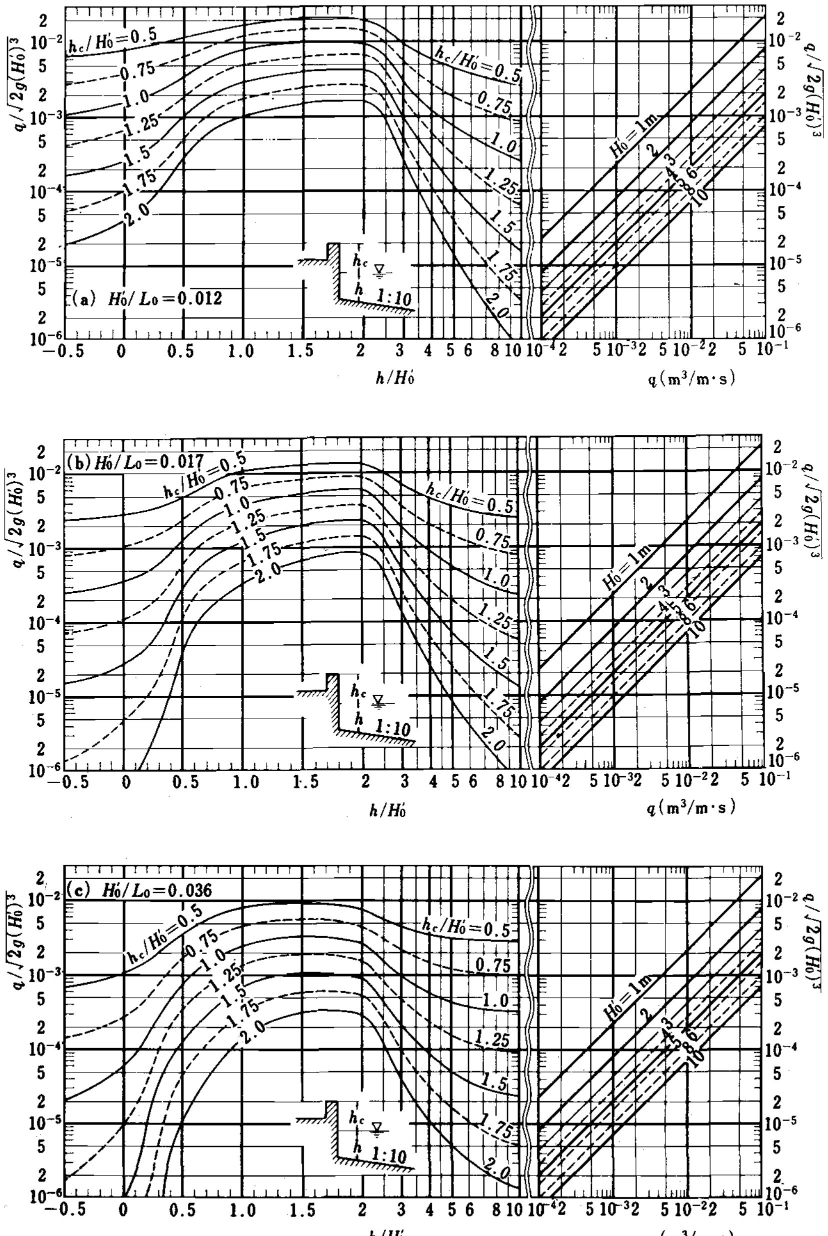
[24] *Ibid.*

[25] *Ibid.*

[26] *Ibid.*

Figure 32

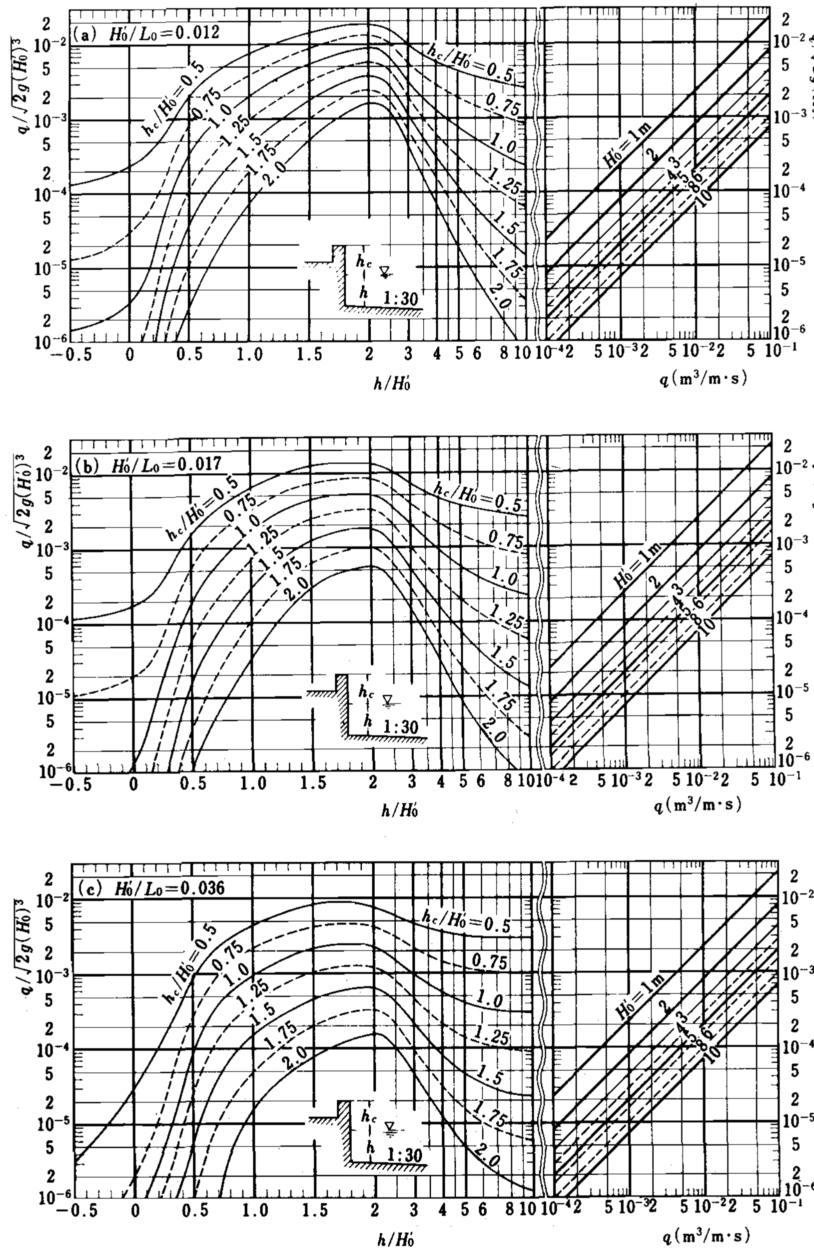
DIAGRAMS USED TO CALCULATE THE WAVE OVERTOPPING RATE FOR BULKHEADS WITHOUT QUARRY STONE TOE PROTECTION AND WITH AN OFFSHORE LAKEBED SLOPE OF 1:10



Source: Y. Goda, Random Seas and Design of Maritime Structures, 1985.

Figure 33

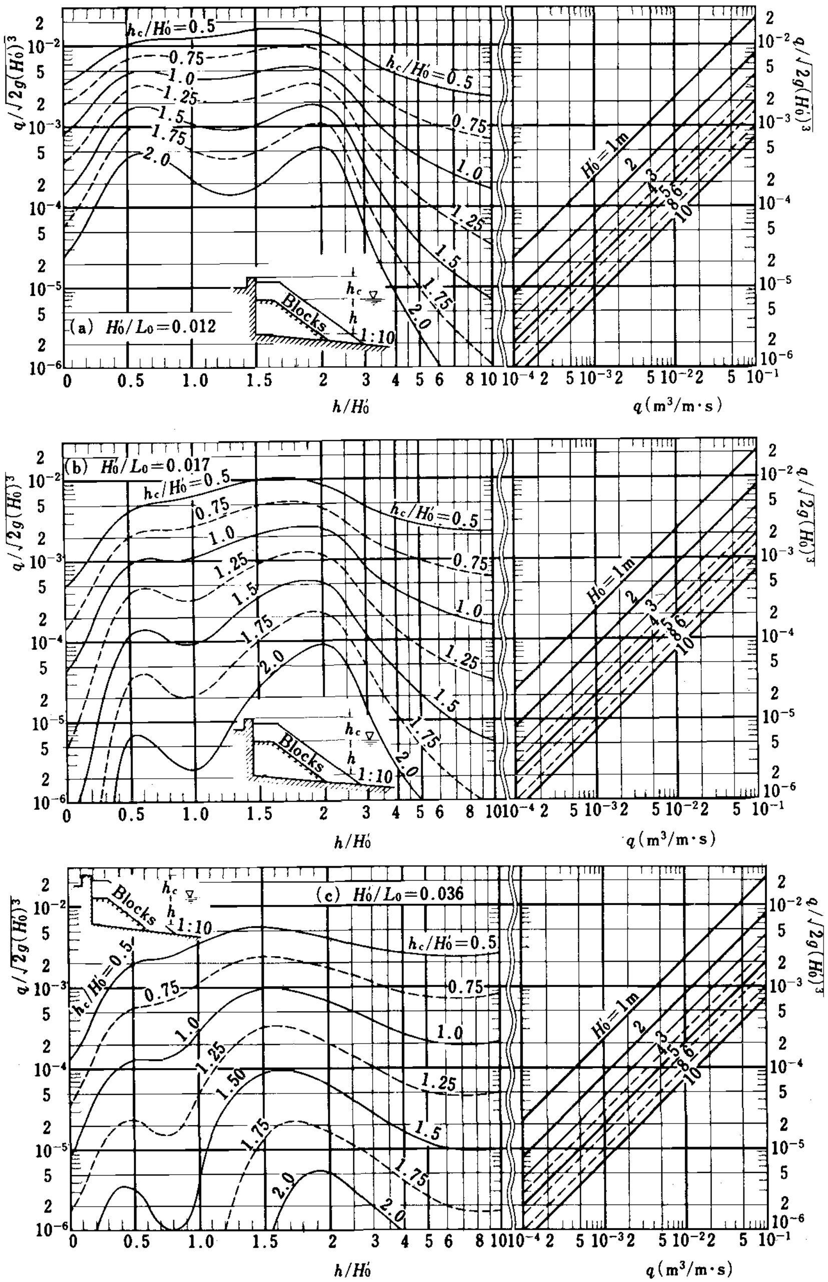
DIAGRAMS USED TO CALCULATE THE WAVE OVERTOPPING RATE FOR BULKHEADS WITHOUT QUARRY STONE TOE PROTECTION AND WITH AN OFFSHORE LAKEBED SLOPE OF 1:30



Source: Y. Goda, Random Seas and Design of Maritime Structures, 1985.

Figure 34

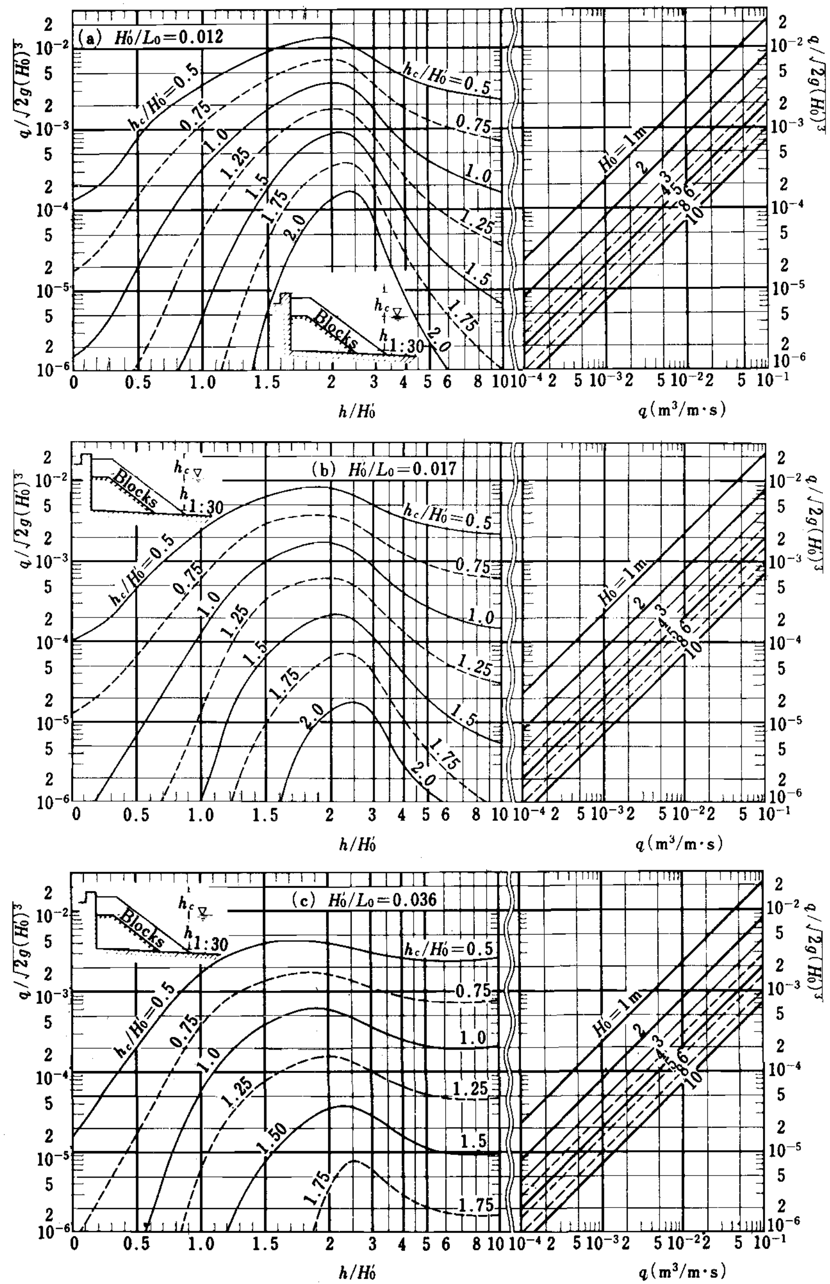
DIAGRAMS USED TO CALCULATE THE WAVE OVERTOPPING RATE FOR BULKHEADS WITH SUBSTANTIAL QUARRY STONE TOE PROTECTION AND WITH AN OFFSHORE LAKEBED SLOPE OF 1:10



Source: Y. Goda, Random Seas and Design of Maritime Structures, 1985.

Figure 35

DIAGRAMS USED TO CALCULATE THE WAVE OVERTOPPING RATE FOR BULKHEADS WITH SUBSTANTIAL QUARRY STONE TOE PROTECTION AND WITH AN OFFSHORE LAKEBED SLOPE OF 1:30



Source: Y. Goda, Random Seas and Design of Maritime Structures, 1985.

above the still-water level; *Lo* the wave length; *g* the acceleration of gravity (32.2 feet per second^2); and *q* the wave overtopping rate.

For each figure, overtopping is shown for three different levels of wave steepness, which is the deep-water wave height divided by the deep-water wave length *(Ho/Lo)*. Figure 32 is used to estimate the overtopping rate for a bulkhead without substantial toe protection and an offshore slope of 1 in 10. Figure 33 shows the overtopping rate for a bulkhead without substantial toe protection and an offshore slope of 1 in 30. Figures 34 and 35 present the overtopping rates for bulkheads with substantial toe protection and offshore slopes of 1 in 10 and 1 in 30, respectively. Figures 34 and 35 were also used to evaluate multiple bulkheads comprised of two walls, one behind the other, with the landward wall being higher than the lakeward wall. The waves break upon the lakeward wall, with some wave energy being dissipated on a splash apron that lies between the two walls. For a wave steepness or offshore slope different from those shown in the figures, the overtopping rates were interpolated or extrapolated.

For each bulkhead evaluated, the potential for damage from overtopping was considered insignificant if the calculated overtopping rate was less than 0.005 cfs per foot. A low potential for overtopping damage was indicated if the overtopping rate ranged from 0.005 to 0.01 cfs per foot. If the calculated overtopping rate ranged from 0.01 to 0.1 cfs per foot, a moderate potential for overtopping damage was noted. As mentioned above, a well-designed drainage system can help prevent damage from overtopping rates less than 0.1 cfs per foot. Bulkheads with a calculated overtopping rate exceeding 0.1 cfs per foot were considered to have a high potential for damage by overtopping.

Harbor Structures and Beaches: Thirteen major structures and beaches are located within the Milwaukee outer harbor and South Shore breakwater. Wave conditions within harbors are different from those in the open lake. Waves propagating from the open lake into a harbor experience shoaling, refraction, and diffraction (or scattering). They may encounter reflected waves from vertical or near-vertical surfaces along the boundaries of the harbor. These effects may even induce wave amplification if certain effects coincide with the natural frequencies in the harbor. The trapped wave energy may lead to wave resonant oscillations in the harbor, which may cause docking and navigation hazards and damage vessels, mooring systems, docking facilities, and shore structures.

When waves strike a breakwater, wave energy is either reflected from, dissipated on, or transmitted through or over the structure. The distribution of the wave energy depends on the wave characteristics and the type and shape of the breakwater. Ideally, harbor breakwaters should reflect or dissipate as much wave energy as possible. Transmission of wave energy over or through a breakwater should be minimized to prevent damaging waves within the harbor.

Because of the complexity of wave propagation within the Milwaukee outer harbor and the South Shore breakwater, a numerical simulation model, referred to as the Milwaukee Harbor Model, was modified and applied under the study to calculate wave and oscillation conditions.[27] The model consists of a hybrid finite element submodel for the outer boundary of the harbor, and a finite element submodel for the harbor and breakwater area itself. The detailed formulation of the Milwaukee Harbor Model is presented in Lee and Chen (1985).[28] A schematic diagram of the procedures used to evaluate wave conditions and runup and overtopping rates within the outer harbor and South Shore breakwater is shown in Figure 36.

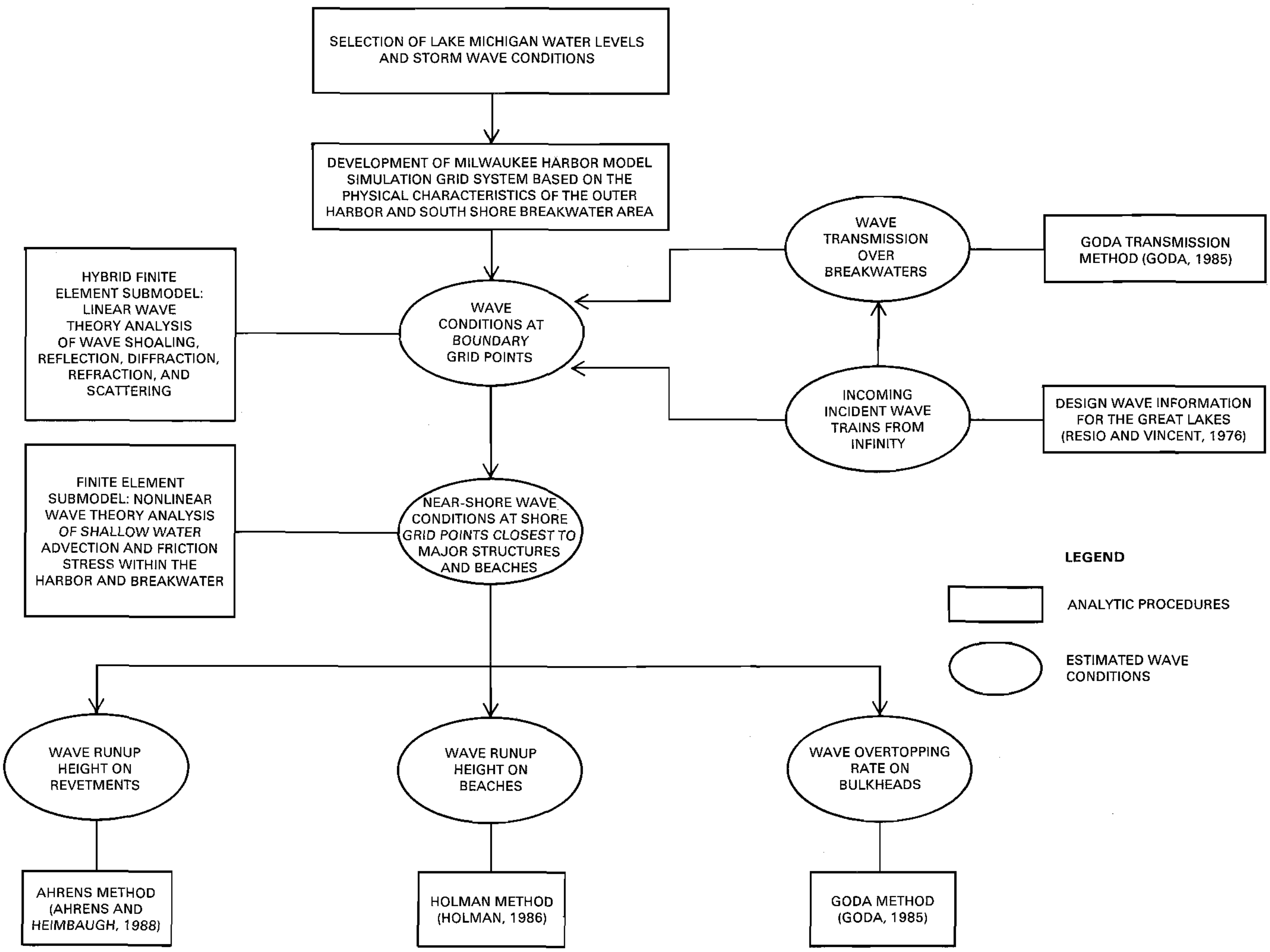
The hybrid finite element submodel combines the use of the boundary integral method, which provides convenient data preparation and numerical operation, with the finite difference method, which is powerful in solving the partial differential hydrodynamic equations. For a given deep-water wave, the hybrid finite element submodel estimates the wave oscillations at the

[27] *Kwang K. Lee, Computation of Wave Heights Along the Milwaukee County Shoreline and in the Milwaukee Harbor and South Shore Break Wall, Prepared for the Southeastern Wisconsin Regional Planning Commission, 1988.*

[28] *Kwang K. Lee and Ching-Lin Chen, A Shallow Water Storm Wave and Oscillation Model, Research Report, Department of Civil Engineering, University of Wisconsin-Milwaukee, Milwaukee, Wisconsin, 1985.*

Figure 36

SCHEMATIC DIAGRAM OF WAVE ANALYSIS OF SHORE PROTECTION STRUCTURES AND BEACHES WITHIN THE MILWAUKEE OUTER HARBOR AND SOUTH SHORE BREAKWATER



Source: Kwang K. Lee, Computation of Wave Heights along the Milwaukee County Shoreline and in the Milwaukee Harbor and South Shore Break Wall, Prepared for the Southeastern Wisconsin Regional Planning Commission, 1988.

harbor entrances based on the shoreline configuration, bathymetry, bottom friction, and shore reflection characteristics. The most recent data available were used to define shoreline and bathymetric conditions. The effects of varying water depths, wave radiation at the harbor entrances, and wave reflection and diffusion by coastal structures are taken into account in these model simulations. The wave trains computed by using the hybrid finite element submodel were imposed at the harbor entrances within the outer harbor and South Shore breakwaters.

Wave transmission over a breakwater results primarily from the regeneration of waves in the lee of the breakwater formed by the impact of the overtopping water mass. Thus, the wave transmission coefficient is primarily determined by the ratio of the crest elevation of the breakwater to the incident wave height. Wave transmission over the outer harbor and South Shore breakwaters was estimated using a procedure based on hydraulic model tests and formulated by Goda,[29]

[29]*Goda, op. cit.*

as shown in Figure 37. The 20- and 50-year deep-water waves discussed above were shoaled linearly to the water depths just outside various points along the breakwaters to find the incident wave height (Hi). The transmitted wave height (H_t) was then calculated using Figure 37, and used as a boundary condition for the Milwaukee Harbor Model.

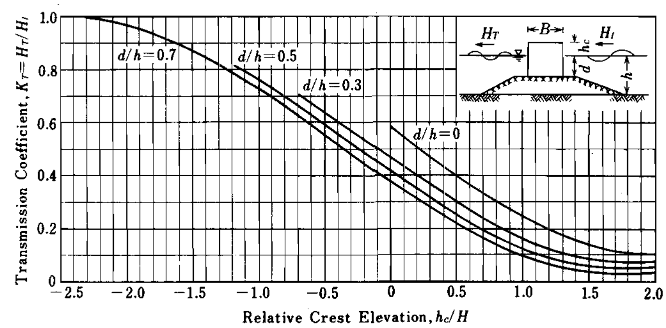
The finite element submodel for the outer harbor and South Shore breakwater then was used to calculate wave propagation along an extensive finite element grid system consisting of linear triangular elements. The elements are approximately 300 feet long parallel to the shoreline. A Chezy-Manning type friction formula was used, and full reflection at all solid boundaries was implied. The assumed full reflection of wave energy may result in some wave heights being overestimated because, in reality, some wave energy overtops the structure and is thus not reflected back into the harbor. Therefore, the simulated wave heights approximate the conditions that would exist if the harbor facilities were modified to prevent overtopping.

The approach used in applying the Milwaukee Harbor Model was as follows. First, the grid system, based on the shape, bathymetry, and other physical characteristics of the harbor and near-shore Lake Michigan, was assembled for the submodels. Second, the designated design wave trains, including significant wave heights, frequencies or periods, and direction, were imposed onto the model. The incoming waves were assumed to be incident from infinity. Utilizing linear wave theory, the hybrid finite element submodel was used to estimate the corresponding wave heights at all boundary grid points, including those at the breakwaters and at the harbor entrances. The hybrid finite element submodel takes into account the effects of reflection, diffraction, and scattering on the incident wave. Third, with the given wave conditions at the harbor entrances and the transmitted waves over the breakwaters, the finite element submodel for the outer harbor and South Shore breakwater was employed to solve the nonlinear shallow water equations which take into account advection and friction stress in the analysis of wave oscillation inside the harbor.

The maximum wave heights simulated by the model at those shore grid points closest to the major structures and beaches were used to

Figure 37

WAVE TRANSMISSION COEFFICIENTS FOR A VERTICAL BREAKWATER



Source: Y. Goda, Random Seas and Design of Maritime Structures, 1985.

evaluate wave impacts on the structures and beaches. As presented in Table 33 under the selected three maximum water levels, the water depths at these near-shore grid points ranged from 14.6 to 39.8 feet. The equations and procedures used to evaluate structures and beaches on the open coast were then used to bring these simulated waves onshore and calculate wave runup heights and overtopping rates for structures and beaches located within the harbor.

For the four revetments located within the harbor, the model-simulated wave height at the near-shore grid point was used as the wave height at the toe of the structure (Hmo) in the Ahrens method. For the South Shore beach, which is the only beach located within the harbor, the model-simulated wave height was used to represent the near-shore wave height (Hs) in the Holman method. The use of the model-simulated wave heights to evaluate the eight bulkheads within the harbor was somewhat more complicated because equivalent offshore wave heights had to be recalculated to represent waves within the harbor. The model-simulated wave heights were assumed to represent the near-shore wave heights. Based on the near-shore wave height, the water depth at the structure toe, and the wave length, an equivalent offshore wave height was computed using relationships set forth in Appendix C, Miscellaneous Tables and Plates, of the U. S. Army Corps of Engineers Shore Protection Manual, Volume II, (1984). The Goda figures presented in Figures 32 through 35 were then used to calculate wave

Table 33

LAKE MICHIGAN WATER DEPTHS AT MILWAUKEE HARBOR MODEL SIMULATION OUTPUT GRID POINTS WITHIN THE MILWAUKEE OUTER HARBOR AND SOUTH SHORE BREAKWATER

Major Shore Protection Structure or Beach	Breakwater	Water Depth (in feet) at Simulation Output Grid Point Used to Evaluate Structure or Beach		
		10-Year Water Level (582.8 feet NGVD)	100-Year Water Level (584.3 feet NGVD)	500-Year Water Level (585.9 feet NGVD)
1. Milwaukee County South Shore Park-South Revetment	South Shore	14.6	16.1	17.7
2. Milwaukee County South Shore Park Beach	South Shore	14.6	16.1	17.7
3. Milwaukee County South Shore Yacht Club Bulkhead	South Shore	19.8	21.3	22.9
4. Milwaukee County South Shore Park-North Revetment	South Shore	19.8	21.3	22.9
5. U. S. Army Corps of Engineers Dredge Spoils Confined Disposal Facility Revetment	Outer Harbor	29.2	30.7	32.3
6. South Lincoln Memorial Drive Bulkhead	Outer Harbor	15.8	17.3	18.9
7. Port of Milwaukee Bulkhead Slips	Outer Harbor	36.6	38.1	39.7
8. MMSD Jones Island Wastewater Treatment Plant Bulkhead	Outer Harbor	36.7	38.2	39.8
9. Marcus Amphitheatre Bulkhead	Outer Harbor	33.4	34.9	36.5
10. Henry W. Maier Festival Grounds Revetment	Outer Harbor	21.6	23.1	24.7
11. Milwaukee Harbor Commission Municipal Pier Bulkhead	Outer Harbor	22.6	24.1	25.7
12. Milwaukee County War Memorial Center Bulkhead	Outer Harbor	14.6	16.1	17.7
13. Milwaukee County Juneau Park Landfill Bulkhead	Outer Harbor	24.3	25.8	27.4

Source: Kwang K. Lee, Computation of Wave Heights Along the Milwaukee County Shoreline and in the Milwaukee Harbor and South Shore Break Wall, Prepared for the Southeastern Wisconsin Regional Planning Commission, 1988.

overtopping rates for each bulkhead. The potential for wave overtopping damage was classified based on the same criteria used to determine such damage for the structures and beaches located on the open coast.

Low-Water Impacts: In addition to the above evaluation of high-water impacts, the structural damages that could result from low water levels were evaluated. A potential for structural damage was considered to exist if normally submerged timber components of a structure would be exposed by the 100-year recurrence interval minimum monthly mean water level of 575.5 feet NGVD. Exposure of timber components to the air may accelerate the decomposition of the wood by processes such as dry rot. Furthermore, there was considered to be an increased risk of toe scouring if the bottom of a structure was located above the 100-year recurrence interval instantaneous minimum level of 574.9 feet NGVD, since the bottom of the structure would be exposed to wave attack.

Shoreline Erosion

The extent and severity of shoreline erosion problems in Milwaukee County became apparent during the record high lake levels of 1986, during which waves were breaking directly at the base of the bluff along many shoreline areas. This bluff toe erosion occurs to some degree in nearly all shoreline areas not protected by adequate shore protection structures or large beaches. Erosion at the toe of the bluff initiates changes in slope geometry, which in turn trigger slope failures on the bluff slope. Shoreline erosion also affects the low terraces, such as those in the Villages of Bayside and Fox Point.

During the 1987 field surveys conducted in southern Milwaukee County and the Village of Bayside, and the 1986 field surveys conducted in the remainder of northern Milwaukee County, those portions of the study area shoreline that were experiencing wave erosion at the toe of the bluff were identified and mapped. Those affected shoreline reaches included areas where waves were observed to be attacking an unprotected bluff; where there was noticeable evidence of recent toe erosion; or where existing shore protection structures were failing and exposing the shoreline to active wave erosion. Shoreline reaches experiencing erosion within the study area were also identified on color, oblique aerial photographs taken under this study in April 1987. Detailed measurements of the geometry of the bluff slope, which were conducted at 104 sites, as presented in Table 34 and shown on Map 26, provided site-specific assessments of the severity of toe erosion at these selected locations. The results of the slope stability analyses conducted at these sites were used to evaluate the impact of bluff toe erosion on the overall stability of the bluff slope.

Using these analytical methods, the presence of shoreline erosion and the impact of bluff toe erosion on the overall stability of the bluff slope were determined for each bluff analysis section. The bluff analysis sections were classified into three categories of shoreline erosion. Category I, defined as having slight toe erosion, includes shoreline areas that had little or no evidence of erosion. Category II, defined as having moderate shoreline erosion, includes shoreline areas where evidence of erosion was observed, but where such erosion did not appear to be affecting the overall stability of the bluff slope, often because of the presence of a terrace at the base of the bluff. Category III, defined as having severe shoreline erosion, includes shoreline areas where erosion was threatening the overall stability of the bluff slope.

Bluff Slope Instability by Rotational Sliding

Rotational slides are characterized by rotation of the top of the sliding mass backward toward the slope face. Deep-seated slips may occur, involving a massive amount of bluff material and the loss of up to 10 feet or more of land at the top of the bluff. Slope stability analyses for rotational slides provide not only an indication of the likelihood of circular slips, but also an overall indication of the resistance of the slopes to all types of massive slope failures. In reality, massive slope failure surfaces are rarely truly circular; most are more "spoon-shaped" with a steeper upper portion at the rupture surface, and with a progressively decreasing slope angle. Slope stability analyses were performed for the bluffs using surveyed geometric profiles of the bluffs; estimated stratigraphic and groundwater conditions; laboratory analyses of the bluff material properties; and modified versions of the computer program STABL.[30] STABL was devel-

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[30]*R. A. Siegel, STABL User Manual, Joint Highway Research Project, Purdue University and the Indiana State Highway Commission, JHRP-75-9, June 1975.*

Table 34

LOCATION OF PROFILE SITES

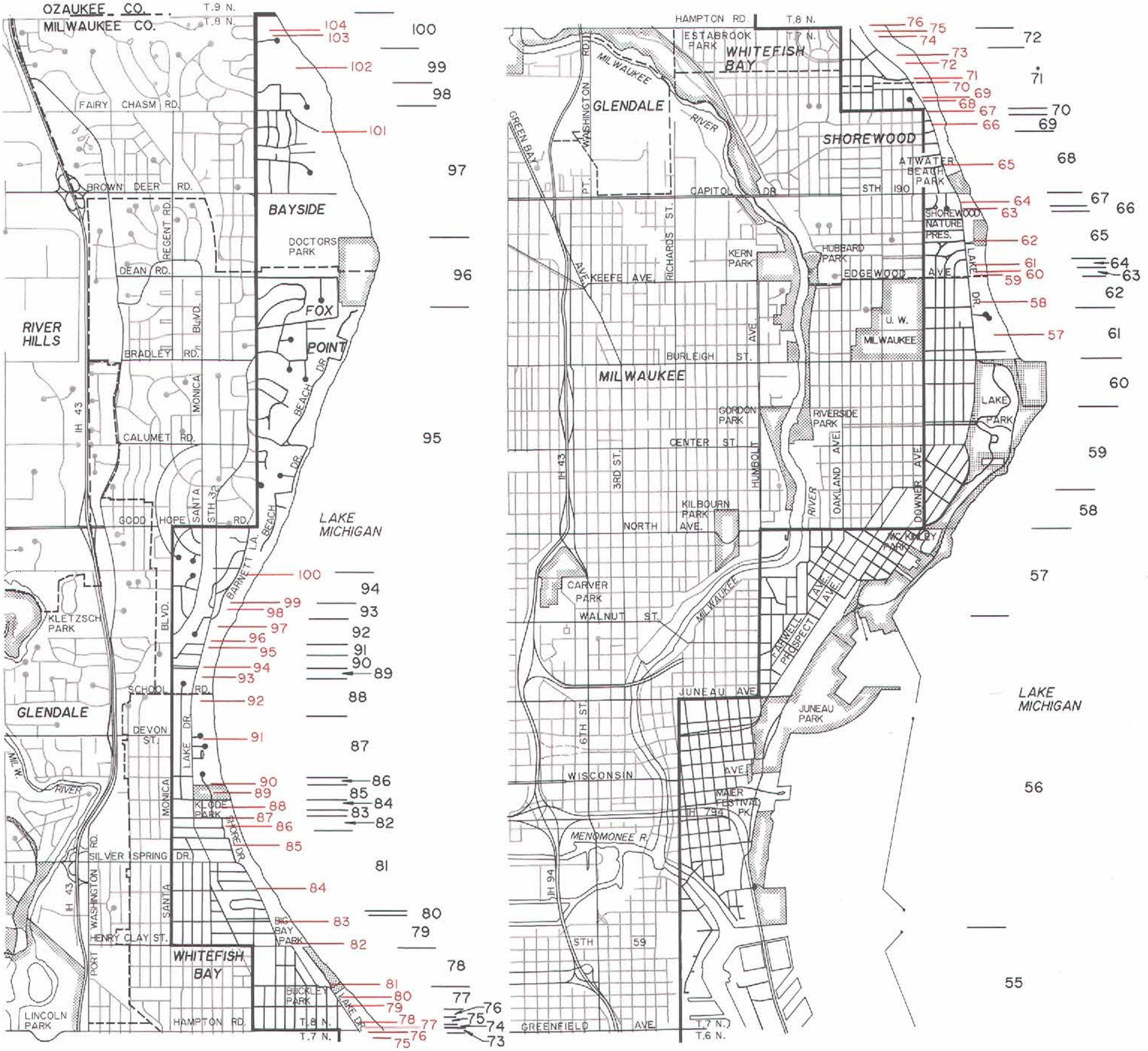
Civil Division	Bluff Analysis Section	Profile Number	Location
City of Oak Creek	2	1	4750 E. Elm Road
		2	4750 E. Elm Road
		3	4750 E. Elm Road
	3	4	Bender Park
		5	Bender Park
		6	Bender Park
	4	7	Bender Park
		8	Bender Park
	5	9	Bender Park
	6	10	9300 S. 5th Avenue
	7	11	9180 S. 5th Avenue
	8	12	4301 E. Depot Road
	10	13	9006 S. 5th Avenue
	11	14	9006 S. 5th Avenue
	13	15	8400 S. 5th Avenue
City of South Milwaukee	14	16	3817 S. 3rd Avenue
		17	3613 S. 3rd Avenue
	15	18	3333 S. 5th Avenue
	16	19	3333 S. 5th Avenue
	17	20	3333 S. 5th Avenue
	18	21	3015 S. 5th Avenue
		22	315 Marion Avenue
	20	23	Grant Park
	21	24	Grant Park
	22	25	Grant Park
	23	26	Grant Park
	24	27	Grant Park
	25	28	Grant Park
City of Cudahy	26	29	6260 S. Lake Drive
	27	30	Warnimont Park
		31	Warnimont Park
	28	32	Warnimont Park
	29	33	Warnimont Park
	30	34	Warnimont Park
	31	35	Warnimont Park
	33	36	Sheridan Park
	34	37	Sheridan Park
		38	Sheridan Park
	35	39	Sheridan Park
	36	40	Sheridan Park
	37	41	Sheridan Park
		42	Sheridan Park
City of St. Francis	38	43	4158 S. Lake Drive
	39	44	4158 S. Lake Drive
		45	4158 S. Lake Drive
	40	46	4158 S. Lake Drive
	42	47	4158 S. Lake Drive
	43	48	Bay View Park
		49	Bay View Park
		50	Bay View Park
	44	51	Bay View Park
	45	52	Bay View Park
	46	53	Bay View Park
	47	54	Bay View Park
City of Milwaukee	48	55	South Shore Park
	50	56	South Shore Park
	61	57	3252 N. Lake Drive
	62	58	100 feet north of E. Newport Avenue
Village of Shorewood	63	59	3510 N. Lake Drive
		60	3510 N. Lake Drive
	64	61	3534 N. Lake Drive
	65	62	3704 N. Lake Drive
	66	63	3926 N. Lake Drive
	67	64	3932 N. Lake Drive
	68	65	4098 N. Lake Drive
	69	66	4308 N. Lake Drive
	70	67	4408 N. Lake Drive
	71	68	4460 N. Lake Drive
Village of Whitefish Bay	71	69	4500 N. Lake Drive
		70	4620 N. Lake Drive
	72	71	4652 N. Lake Drive
		72	4730 N. Lake Drive
	73	73	4762 N. Lake Drive
	74	74	4780 N. Lake Drive
	75	75	4794 N. Lake Drive
	76	76	4810 N. Lake Drive
	77	77	4890 N. Lake Drive
		78	4930 N. Lake Drive
	78	79	Big Bay Park
	79	80	Henry Clay Street
	80	81	5290 N. Lake Drive
	81	82	5486 N. Lake Drive
		83	5674 N. Shore Drive
	82	84	5738 N. Shore Drive
	83	85	758 Day Street
	84	86	5822 N. Shore Drive
	85	87	Klode Park
	86	88	5960 N. Shore Drive
	87	89	614 E. Lake Hill Court
Village of Fox Point	88	90	6330 N. Lake Drive
		91	6424 N. Lake Drive
	89	92	6448 N. Lake Drive
	90	93	6530 N. Lake Drive
	91	94	6610 N. Lake Drive
	92	95	6720 N. Lake Drive
		96	6818 N. Barnett Lane
	93	97	6840 N. Barnett Lane
	94	98	6960 N. Barnett Lane
	96	99	Doctors Park
		100	Doctors Park
Village of Bayside	97	101	9360 N. Lake Drive
	99	102	9364 N. Lake Drive
	100	103	1240 E. Donges Court
		104	9560 N. Lake Drive

Source: SEWRPC.

oped in 1975 by the Joint Highway Research Project, conducted by Purdue University and the Indiana State Highway Commission. The program can generate circular failure surfaces, sliding block surfaces, and irregularly shaped surfaces. It is capable of evaluating the effects of different soil and groundwater conditions, earthquakes, and surcharge loadings. Bluff slope data used as input to the program include the geometry of the slope, bluff stratigraphy interfaces, soil properties, and groundwater elevations. The program has been modified by Associate

Map 26

BLUFF PROFILE SITES IN MILWAUKEE COUNTY



LEGEND

12 BLUFF ANALYSIS SECTIONS

— 24 PROFILE SITE AND NUMBER

Map 26 (continued)

Source: SEWRPC.

Professor Peter J. Bosscher of the University of Wisconsin-Madison for personal computer use, and for data enhancement purposes.

Using shear strengths and stresses, factors of safety were calculated for potential failure surfaces within the bluff. A safety factor is defined as the ratio of the forces resisting shear to the forces promoting shear along the failure surface. Thus, a safety factor less than or equal to 1.0 indicates that the forces promoting failure are greater than or equal to the forces resisting failure.

The particular method of analysis for calculating safety factors used in this study was the Modified Bishop method, which is applicable to circular-shaped failure surfaces. For each potential failure surface, the resisting forces or strength parameters, such as soil cohesion and friction, and the driving forces, such as the soil mass along the failure surface, were determined and a corresponding safety factor calculated. The program randomly generates and evaluates potential failure surfaces in order to identify the most critical—and the most likely—failure surface. The Modified Bishop method is a "method of slices" procedure, i.e., the program divides a potential sliding mass into vertical sections. The forces acting upon a typical section are shown in Figure 38. The forces exerted in a vertical direction are taken into account, while the difference between the horizontal forces across a section—or between sections—are ignored. The resulting equation for calculating the safety factor is:

$$SF = \frac{\Sigma \left\{[c'\,b+(W-ub)\tan\phi']\right\} \quad 1/m_{\alpha}}{\Sigma\, W \sin\alpha}$$

where:

SF = safety factor, dimensionless

$m_{\alpha} = \cos\alpha\,[1 + (\tan\alpha \tan \frac{\phi'}{SF})]$, a dimensionless coefficient

W = weight of individual section, in pounds

b = width of slice, in feet

α = slope angle, in degrees

u = pore water pressure, in pounds per square foot

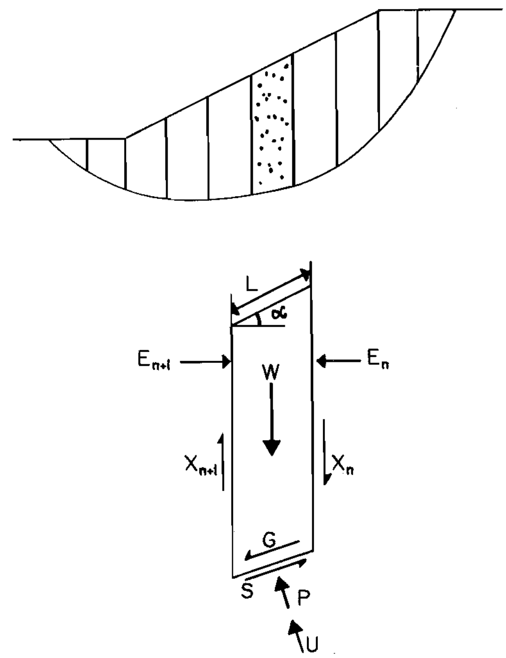
c' = cohesion intercept, in pounds per square foot

ϕ' = internal friction angle, in degrees

The equation is solved in an iterative manner, and is repeated for several trial failure surfaces to determine the lowest safety factor.

Figure 38

FORCES ACTING ON A TYPICAL SECTION IN THE MODIFIED BISHOP METHOD OF ROTATIONAL SLOPE STABILITY ANALYSIS



WHERE:

L = LENGTH OF FAILURE SURFACE, IN FEET

α = ANGLE OF INCLINATION, IN DEGREES

W = WEIGHT OF SLICE, IN POUNDS PER FOOT

G = GRAVITY, IN POUNDS PER FOOT

S = SHEAR STRENGTH FORCES (COHESION AND FRICTION), IN POUNDS PER SQUARE FOOT

P = NORMAL FORCE, IN POUNDS PER FOOT

U = WATER FORCE, IN POUNDS PER SQUARE FOOT

E_n, E_{n+1} = NORMAL SIDE FORCES ON SLICE, IN POUNDS PER FOOT

X_n, X_{n+1} = TANGENTIAL SIDE FORCES ON SLICE, IN POUNDS PER FOOT

Source: T. B. Edil.

Distinction Between Deterministic and Probabilistic Slope Stability Analyses: Two separate versions of the STABL program were used in the slope stability analysis for the Milwaukee County shoreline.[31] The first version utilized a deterministic approach in which site-specific data collected at the profile sites were used to compute potential failure surfaces at the given

[31] *P. J. Bosscher, T. B. Edil, and D. M. Mickelson, "Evaluation of Risks of Slope Instability Along a Coastal Reach," Proceedings of the Vth International Symposium on Landslides, 1988, Lausanne, Switzerland, 1988.*

location. The second version utilized a probabilistic approach which allowed the input data to vary randomly within specified dispersions.[32] The intent of the probabilistic analysis was to provide a general assessment of the stability of the bluff slopes within an entire bluff analysis section, where the bluff characteristics vary, rather than only at the specific profile sites with known characteristics. A probabilistic analysis also helped improve the evaluation of those profile sites where some of the bluff characteristics were not well defined. Thus, the probabilistic analysis quantified the risk of slope failure where some of the analysis factors could not be accurately determined. More detailed descriptions of each of the two types of analyses are presented below.

Deterministic Slope Stability Analysis: A total of 104 bluff profiles prepared during the field surveys conducted in the fall of 1987 for southern Milwaukee County and the Village of Bayside, and in the summer of 1986 for the remainder of northern Milwaukee County, were used in the deterministic slope stability analysis. The locations of the profile sites, which are presented in Table 34 and shown on Map 26, were selected to be representative of bluff areas with different physical characteristics and different causes and types of slope failure. For 82 of the bluff analysis sections, from one to three profiles were prepared. The 18 remaining bluff analysis sections include shoreline areas either where the bluff has been regraded to a stable slope angle and is adequately protected by a major shore protection structure or beach, or where a natural bluff is not located in proximity to the shoreline. No slope stability analysis was conducted within these 18 sections.

Soil properties used as input to the program include the cohesion intercept, the internal friction angle, and the unit weight of both saturated and unsaturated soil. The relative importance of each of these soil properties for stability is influenced by the physical characteristics of the bluff and by the groundwater conditions. In general, the cohesion intercept is the most important soil property when the bluff height is less than 80 feet, while the internal friction angle is the most important in bluffs greater than 80 feet high.[33] The angle at which a slope will become relatively stable is primarily a function of the internal friction angle and the level of the groundwater. The unit weight of the soil influences slope stability differently depending upon the level of the groundwater. For low groundwater levels, soils with a lower unit weight are more stable, whereas for high groundwater levels, soils with a higher unit weight are more stable.

The rotational slope stability analyses utilized in this study provide the locations of potential failure surfaces and the attendant safety factors based upon drained soil strength parameters and calculated pore water pressures. An "effective stress analysis" for long-term stability, rather than a "total stress analysis" for short-term stability, was conducted. For the effective stress analysis, "worst case" groundwater conditions were utilized. Late winter and early spring have been found to be the most critical periods for the stability of Lake Michigan coastal bluffs for both deep-seated and shallow slides.[34] During this period, groundwater levels and flows generally rise, but the surface is still frozen, which decreases its permeability and prevents groundwater discharge from the slope face. This creates an inclined artesian effect, resulting in increased pore pressures and reduced slope stability. The elevation of the water table is affected by many of the same factors which result in fluctuations of the level of Lake Michigan. In some bluffs, the groundwater may be hydraulically connected to the lake; thus, the elevation of the water table would be directly related to the lake level. In most bluffs within the study area, however, the water table is at a higher elevation than the lake level. High precipitation and cool air temperature conditions, which contribute to high lake levels, would also tend to increase the elevation of the water

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[32]*The term "dispersion" refers to the variability of data from a mean value.*

[33]*T. B. Edil and L. E. Vallejo, "Mechanics of Coastal Landslides and the Influence of Slope Parameters," Engineering Geology, Vol. 16, 1980, pp. 83-96.*

[34]*L. E. Vallejo and T. B. Edil, "Design Charts for Development and Stability of Evolving Slopes," Journal of Civil Engineering Design, Vol. 1, No. 3, 1979, pp. 231-252.*

table. Therefore, at least in some bluffs, the elevation of the water table may have been relatively high in 1986, when the lake levels were also high. Fluctuations in groundwater elevations may be even greater than the fluctuations in lake levels, because the groundwater is contained only within the soil pores, and because the contributing recharge area for a groundwater system would be much smaller than the total tributary drainage area to Lake Michigan, and therefore more sensitive to local climatic variations.

Interpreting the stability of coastal slopes is a problem complicated by the dynamic nature of slope geometry. There are forces constantly seeking to achieve slope equilibrium and other forces constantly initiating new slope failures. Since the geometry of the slope changes in response to bluff toe erosion and face stabilization processes, the safety factor—especially for deep rotational slides—varies with time. Slope failure over time is referred to as the evolution of the slope. Along the Lake Michigan shoreline, bluff slopes generally evolve in one of two ways.

The first common type of slope evolution involves a successive series of shallow slumps retrogressing from the toe to the top of the bluff. Typically, this first type of evolution occurs in bluff slopes with an angle of less than about 30 degrees, and in bluffs which contain layers of cohesive silt and clay.[35] In the evaluation of the stability of this type of slope, the failure surface having the lowest safety factor is the most important, even if that failure surface would affect a small portion of the bluff slope.

The second common type of slope evolution involves the retreat of the bluff generally parallel to the existing face. Large, deep rotational slips may also occur. This type of slope evolution typically occurs in bluffs with a steep slope—greater than 30 degrees—and in bluffs composed of noncohesive glacial tills and sand. The evaluation of the stability of this second type of slope involves the consideration of all failure surfaces with a safety factor of less than one. Thus, the interpretation of the slope stability analysis considers the potential for failure throughout a zone delineated by the largest failure surface with a safety factor of less than one.

The soil stratigraphy at each profile site is critical to the evaluation of the stability of the bluff slopes. As indicated in Chapter II, the stratigraphy was identified on the basis of field surveys conducted in the fall of 1987 for southern Milwaukee County and the Village of Bayside, and in the summer of 1986 for the remainder of northern Milwaukee County; on historical geologic records of soil boring data; and on new soil boring data. The determination of the stratigraphy at each of the profile sites was based on the sources of data set forth in Table 35. The reliability of the slope stability evaluations was greater at some profile sites than at others because the quantity and precision of available inventory data varied substantially between sites.

The results of laboratory analyses of the properties of soils identified in the study area were summarized in Chapter II. The soil property summaries were based on historical data and on the geotechnical laboratory analyses of grab samples collected in May 1986 for northern Milwaukee County and in October 1987 for southern Milwaukee County and the Village of Bayside; and of soil boring samples collected in October and November 1986 for northern Milwaukee County and in March 1988 for southern Milwaukee County. These soil properties were used to estimate the ability of the soil materials to resist slope failure. The soil properties of the bluff materials used in the deterministic slope stability analyses are presented in Table 36.

The groundwater elevations used in the deterministic slope stability analysis at each profile site were based on observed groundwater seepage, soil boring data, groundwater observation wells, and electrical resistivity analyses. Where no specific groundwater data were available, the elevation of the groundwater was estimated based on the depth of permeable soil layers. The elevation of the groundwater within each of the bluff analysis sections was determined based on the sources of data set forth in Table 8 in Chapter II.

For each profile site, the deterministic version of STABL was used to generate 100 potential failure surfaces and to calculate the corresponding safety factors. The 10 failure surfaces with

[35]*Ibid.*

Table 35

SOURCES OF STRATIGRAPHIC DATA USED FOR THE SLOPE STABILITY ANALYSIS OF PROFILE SITES

Civil Division	Bluff Analysis Section	Profile Number	Profile Location	Field Observation of Exposed Bluff Face Within Section 1986-1987[a]	Soil Boring Within Section			Field Observation of Exposed Bluff Face Within Adjacent Sections[a]	Soil Boring Within Adjacent Sections
					Pre-1986	1986	1987		
City of Oak Creek	1	--	--	--	--	--	--	--	--
	2	1	4750 E. Elm Road	X	--	--	--	--	X
		2	4750 E. Elm Road	X	--	--	--	--	X
		3	4750 E. Elm Road	X	--	--	--	--	X
	3	4	Bender Park	--	X	--	--	--	--
		5	Bender Park	--	X	--	--	--	--
		6	Bender Park	--	X	--	--	--	--
	4	7	Bender Park	--	--	--	X	--	--
		8	Bender Park	--	--	--	X	--	--
	5	9	Bender Park	X	X	--	--	--	--
	6	10	9300 S. 5th Avenue	X	--	--	--	--	--
	7	11	9180 S. 5th Avenue	--	X	--	--	--	--
	8	12	4301 E. Depot Road	X	--	--	--	--	--
	9	--	--	--	--	--	--	--	--
	10	13	9006 S. 5th Avenue	--	--	--	--	--	X
	11	14	9006 S. 5th Avenue	--	X	--	--	--	--
	12	--	--	--	--	--	--	--	--
	13	15	8400 S. 5th Avenue	--	--	--	--	--	--
City of South Milwaukee	14	16	3817 S. 3rd Avenue	X	--	--	--	--	--
		17	3613 S. 3rd Avenue	X	--	--	--	--	--
	15	18	3333 S. 5th Avenue	--	X	--	--	--	--
	16	19	3333 S. 5th Avenue	--	--	--	--	X	--
	17	20	3333 S. 5th Avenue	X	--	--	--	--	--
	18	21	3015 S. 5th Avenue	--	--	--	X	--	--
		22	315 Marion Avenue	--	--	--	X	--	--
	19	--	--	--	--	--	--	--	--
	20	23	Grant Park	X	--	--	--	--	--
	21	24	Grant Park	--	X	--	--	--	--
	22	25	Grant Park	X	--	--	--	--	--
	23	26	Grant Park	X	X	--	--	--	--
	24	27	Grant Park	X	--	--	X	--	--
	25	28	Grant Park	X	--	--	--	--	--
City of Cudahy	26	29	6260 S. Lake Drive	X	X	--	--	--	--
	27	30	Warnimont Park	--	--	--	X	--	--
		31	Warnimont Park	--	--	--	X	--	--
	28	32	Warnimont Park	X	--	--	--	--	--
	29	33	Warnimont Park	X	--	--	--	--	--
	30	34	Warnimont Park	X	--	--	--	--	--
	31	35	Warnimont Park	--	--	--	X	--	--
	32	--	--	--	--	--	--	--	--
	33	36	Sheridan Park	X	--	--	--	--	--
	34	37	Sheridan Park	--	--	--	--	--	X
		38	Sheridan Park	--	--	--	--	--	X
	35	39	Sheridan Park	--	--	--	X	--	--
	36	40	Sheridan Park	--	--	--	X	--	--
	37	41	Sheridan Park	X	X	--	--	--	--
		42	Sheridan Park	X	X	--	--	--	--
City of St. Francis	38	43	4158 S. Lake Drive	--	X	--	--	--	--
	39	44	4158 S. Lake Drive	X	X	--	--	--	--
		45	4158 S. Lake Drive	X	X	--	--	--	--
	40	46	4158 S. Lake Drive	--	X	--	--	--	--
	41	--	--	--	--	--	--	--	--
	42	47	4158 S. Lake Drive	--	X	--	--	--	--
	43	48	Bay View Park	X	--	--	--	--	--
		49	Bay View Park	X	--	--	--	--	--
		50	Bay View Park	X	--	--	--	--	--
	44	51	Bay View Park	X	--	--	--	--	--
	45	52	Bay View Park	--	--	--	--	X	--
	46	53	Bay View Park	X	--	--	--	--	--
	47	54	Bay View Park	--	--	--	--	X	--

Table 35 (continued)

Civil Division	Bluff Analysis Section	Profile Number	Profile Location	Field Observation of Exposed Bluff Face Within Section 1986-1987[a]	Soil Boring Within Section			Field Observation of Exposed Bluff Face Within Adjacent Sections[a]	Soil Boring Within Adjacent Sections
					Pre-1986	1986	1987		
City of Milwaukee	48	55	South Shore Park	--	--	--	X	--	--
	49	--	--	--	--	--	--	--	--
	50	56	South Shore Park	--	--	--	--	--	X
	51	--	--	--	--	--	--	--	--
	52	--	--	--	--	--	--	--	--
	53	--	--	--	--	--	--	--	--
	54	--	--	--	--	--	--	--	--
	55	--	--	--	--	--	--	--	--
	56	--	--	--	--	--	--	--	--
	57	--	--	--	--	--	--	--	--
	58	--	--	--	--	--	--	--	--
	59	--	--	--	--	--	--	--	--
	60	--	--	--	--	--	--	--	--
	61	57	3252 N. Lake Drive	--	--	--	--	--	X
	62	58	100 feet north of E. Newport Avenue	--	--	X	--	--	--
Village of Shorewood	63	59	3510 N. Lake Drive	--	--	--	--	--	X
		60	3510 N. Lake Drive	--	--	--	--	--	X
	64	61	3534 N. Lake Drive	--	--	--	--	--	X
	65	62	3704 N. Lake Drive	--	X	--	--	--	--
	66	63	3926 N. Lake Drive	--	--	--	--	X	--
	67	64	3932 N. Lake Drive	X	--	--	--	--	--
	68	65	4098 N. Lake Drive	--	--	X	--	--	--
	69	66	4308 N. Lake Drive	--	--	--	--	--	X
	70	67	4408 N. Lake Drive	--	--	X	--	--	--
	71	68	4460 N. Lake Drive	X	--	--	--	--	--
Village of Whitefish Bay	71	69	4500 N. Lake Drive	X	--	--	--	--	--
		70	4620 N. Lake Drive	X	--	--	--	--	--
	72	71	4652 N. Lake Drive	X	--	--	--	--	--
		72	4730 N. Lake Drive	X	--	--	--	--	--
	73	73	4762 N. Lake Drive	X	--	--	--	--	--
	74	74	4780 N. Lake Drive	X	--	--	--	--	--
	75	75	4794 N. Lake Drive	X	--	--	--	--	--
	76	76	4810 N. Lake Drive	X	--	--	--	--	--
	77	77	4890 N. Lake Drive	--	--	--	--	X	--
		78	4930 N. Lake Drive	--	X	--	--	--	--
	78	79	Big Bay Park	--	--	X	--	--	--
	79	80	Henry Clay Street	--	X	--	--	--	--
	80	81	5290 N. Lake Drive	--	X	--	--	--	--
	81	82	5486 N. Lake Drive	--	X	--	--	--	--
		83	5674 N. Shore Drive	--	X	--	--	--	--
	82	84	5738 N. Shore Drive	--	X	--	--	--	--
	83	85	758 Day Street	--	--	--	--	--	X
	84	86	5822 N. Shore Drive	--	--	X	--	--	--
	85	87	Klode Park	--	--	--	--	X	--
	86	88	5960 N. Shore Drive	X	X	--	--	--	--
	87	89	614 E. Lake Hill Court	--	--	X	--	--	--
Village of Fox Point	88	90	6330 N. Lake Drive	--	X	--	--	--	--
		91	6424 N. Lake Drive	--	X	--	--	--	--
	89	92	6448 N. Lake Drive	--	--	--	--	--	X
	90	93	6530 N. Lake Drive	--	--	X	--	--	--
	91	94	6610 N. Lake Drive	--	--	--	--	--	X
	92	95	6720 N. Lake Drive	--	--	X	--	--	--
		96	6818 N. Barnett Lane	--	--	--	--	--	X
	93	97	6840 N. Barnett Lane	--	--	X	--	--	--
	94	98	6960 N. Barnett Lane	X	X	--	--	--	--
	95	--	--	--	--	--	--	--	--
	96	99	Doctors Park	X[b]	--	--	--	--	--
		100	Doctors Park	X[b]	--	--	--	--	--

Table 35 (continued)

Civil Division	Bluff Analysis Section	Profile Number	Profile Location	Field Observation of Exposed Bluff Face Within Section 1986-1987[a]	Soil Boring Within Section			Field Observation of Exposed Bluff Face Within Adjacent Sections[a]	Soil Boring Within Adjacent Sections
					Pre-1986	1986	1987		
Village of Bayside	97	101	9360 N. Lake Drive	--	--	--	--	--	X
	98	--	--	--	--	--	--	--	--
	99	102	9364 N. Lake Drive	--	--	--	--	--	X
	100	103	1240 E. Donges Court	X	X	--	--	--	--
		104	9560 N. Lake Drive	X	X	--	--	--	--

[a]*X - Denotes that at least a portion of the bluff face was unvegetated and exposed during the field surveys, allowing determination of the stratigraphy.*

[b]*Estimated in Mickelson, et al., Shore Erosion Study, Technical Report, Appendix 3, "Milwaukee County," 1977.*

Source: SEWRPC.

Table 36

SOIL PROPERTIES USED IN THE DETERMINISTIC SLOPE STABILITY ANALYSIS FOR ROTATIONAL SLIDING

Soil Type	Unit Weight (pounds per cubic foot)	Saturated Unit Weight (pounds per cubic foot)	Effective Cohesion Intercept (pounds per square foot)	Internal Fraction Angle (degrees)
Tills				
Fractured Ozaukee	134	134	10	30
Ozaukee	134	134	150	30
Oak Creek	135	135	10-100	27-31
New Berlin	138	138	10	34-36
Tiskilwa	10	130	350	27
Lake Sediments				
Sand and Gravel	120	120	0-1,340	22-33
Coarse Sand	120	120	0	33
Medium Fine Sand	120	120	0	33-43
Fine Sand and Silt	110-125	110-125	100	33
Silt and Fine Sand	110	110	0-10	31-43
Silt	130	130	200-4,000	27-32
Clay and Silt	130	130	150-1,340	22-27
General Lake Sediment	125	125	0-100	26-31
Fill				
Concrete Rubble and Soil	130	130	0	33-35

Source: T. B. Edil and SEWRPC.

the lowest safety factors were identified. The three lowest safety factors are presented in this report for each profile site.

Probabilistic Slope Stability Analysis: A probabilistic version of STABL was developed for use in this study by Associate Professor Peter J. Bosscher and Professor Tuncer B. Edil of the University of Wisconsin-Madison. The probabilistic model was intended to verify the results of the deterministic slope stability analyses, particularly for those profile sites where the bluff conditions were not well defined, and to provide an assessment of overall slope stability within bluff analysis sections, rather than just for the specific profile sites. The probabilistic model uses the Monte Carlo method to generate random values within specified dispersions of the position of the soil interface lines, soil properties, and groundwater elevations. The slope height and slope angle were not varied during the probabilistic analysis. It was assumed that the measured profiles within a bluff analysis section were representative of the geometry of the bluffs within that section. The Monte Carlo method is particularly useful when there are complex interrelationships between the uncertain bluff parameters. A probabilistic analysis was conducted at 63 of the 104 profile sites that were analyzed using the deterministic slope stability analysis method. The remaining 41 profile sites were sites where, based on the deterministic analysis and field observations, the slope stability within the entire bluff analysis section was clearly stable or unstable; or where fill had been placed on the face of the bluff. The probabilistic method was not suitable for evaluating the stability of fill sites.

The bluff conditions assumed for the deterministic analysis were used to establish the mean conditions for the probabilistic analysis. The magnitude of the soil interface lines, soil properties, and groundwater elevations were then randomly varied within a distribution determined based upon a review of observed conditions within each bluff analysis section, and other available data. The allowed dispersion of data was specified for each profile site by assigning a standard deviation of those bluff parameters that were allowed to vary randomly.

The data dispersions used for the probabilistic analyses were selected by Professor Tuncer B. Edil. Computer-generated distributions of data were based on specified means and standard deviations. Combinations of data for each stability analysis were then selected randomly from the data distributions. The dispersions used for the soil properties—the cohesion intercept and the internal friction angle—were assigned using all available analyses of the soil types identified within the study area. Generally, from 3 to 10 test results were available for each soil type. The dispersions were assigned by examining the dispersion of the available test data and the nature of the soil. These soil property data are presented in Chapter II. The dispersions used for the elevation of the groundwater and the elevation and inclination of the soil interface lines were not specifically calculated, rather being estimated based upon a review of the range of variability of these characteristics within each bluff analysis section. Thus, considerable judgment was used in establishing the range of variation of bluff characteristics for the probabilistic analysis. It must be recognized that because of the nature of the probabilistic analysis, there is substantial uncertainty that the bluff conditions randomly selected actually exist. Indeed, extremely low safety factors—some less than 0.3—are sometimes calculated. In actuality, a slope would fail long before such a low safety factor would be reached. However, numerous repetitions of the analysis, each corresponding to a combination of the variable parameters randomly selected within their dispersions, help assess the likelihood of slope failure associated with variable bluff conditions.

The location of the soil interface lines on the bluff face, as well as the angle of inclination of these lines as they proceed into the bluff, was varied. The degree of variability differed at each profile site, but in general, the variation of the elevation of the soil interface lines ranged from 0 to 30 feet from the mean, and the variation of the angle of inclination ranged from 0 to 6 degrees from the mean. The lowest variability of soil interface lines was selected for those sites where the strata were well defined and the represented bluff analysis section was small—sometimes including only one residential property.

The dispersion of soil properties from the means used in the probabilistic analyses is set forth in Table 37. The elevation of the groundwater was varied based on available water data and on the type and thickness of the lake sediment layers within each profile site. In general, the variation

Table 37

VARIATION IN SOIL PROPERTIES USED IN THE PROBABILISTIC SLOPE STABILITY ANALYSIS

Soil Type	Effective Cohesion Intercept (pounds per square foot)				Internal Friction Angle (degrees)			
	Minimum	Maximum	Mean	Standard Deviation	Minimum	Maximum	Mean	Standard Deviation
Tills								
Fractured Ozaukee	- -	- -	- -	5	- -	- -	- -	3
Ozaukee	50	300	150	100	27	34	30	3
Oak Creek	0	200	100	75	26-28	32-34	30-30.5	2-3
New Berlin	0	20	10	5	28	36	34	3
Tiskilwa	0	20	10	5	28	36	34	3
Lake Sediments								
Sand and Gravel	0	11	5	5	29	36	33	2
Coarse Sand	0	20	10	5	30	36	33	2
Medium Fine Sand	0	11	5	5	30	36	33	2
Silt and Fine Sand	0	21	10	10	29-31	34-45	31-35	2-4
Silt	0-500	250-3,000	100-2,000	75-1,000	20-21	33-34	27	3-5
Clay and Silt	100	850	450	350	25	30	27	2
Clay	- -	- -	375	150	- -	- -	27	2
General Lake Sediment	0-100	21-850	10-450	10-350	21-29	30-37	27-33	2-3

Source: T. B. Edil and SEWRPC.

of the elevation of the main water table ranged from 2 to 30 feet from the mean, and the variation of the elevation of the perched water table, located in the fractured Ozaukee layer within the bluffs in the northern Milwaukee County communities, ranged from 2 to 7.5 feet from the mean. An illustration of the variability of bluff parameters and the resultant effects on the safety factors calculated with the probabilistic analysis is shown in Figure 39. Compared to the deterministic analysis, the probabilistic analysis yields both higher and lower safety factors.

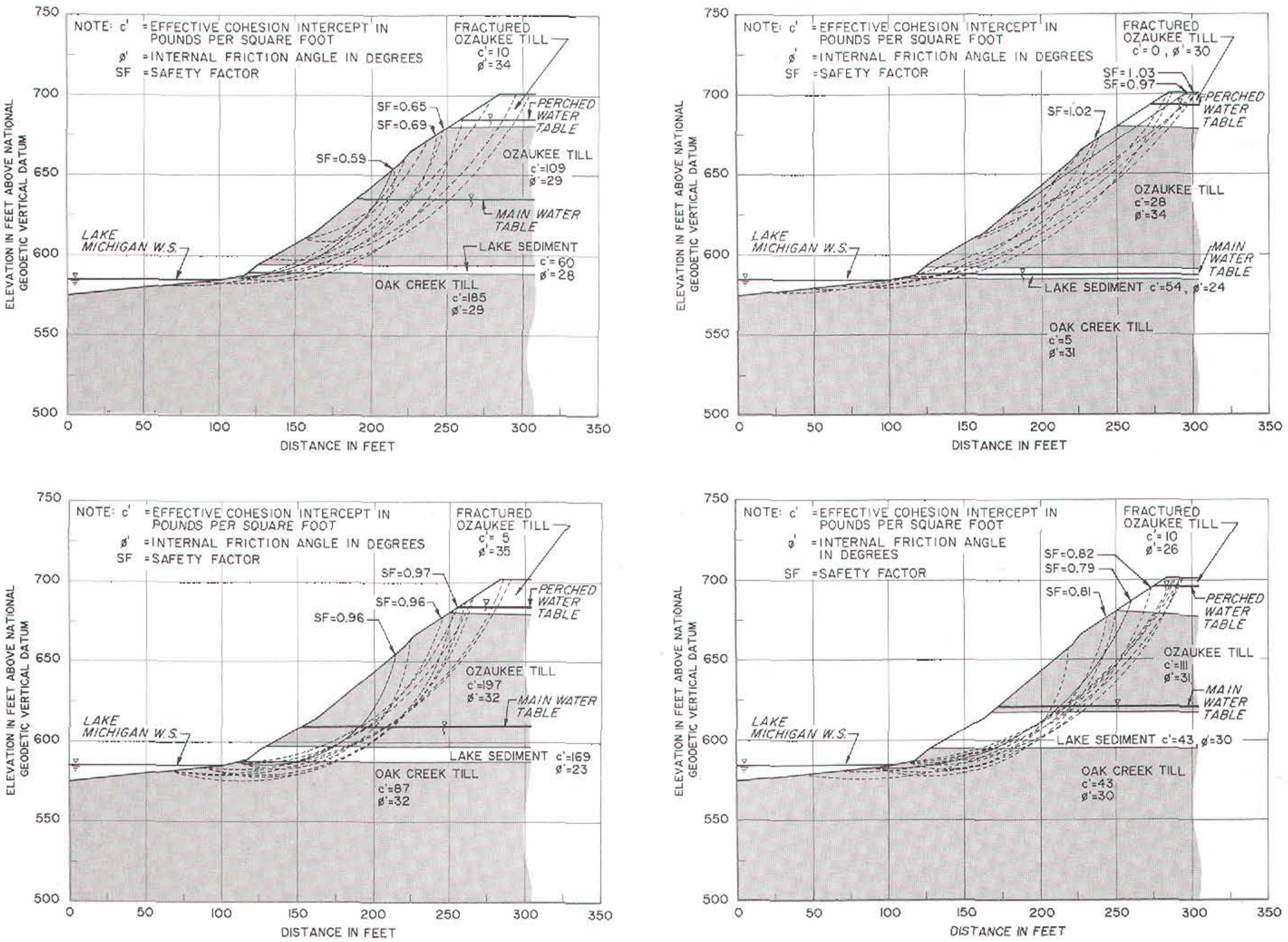
For each of the 63 profile sites evaluated with the probabilistic analysis, a minimum of 20 stability analyses were performed using a random combination of variable bluff parameters. Each stability analysis involved the generation of 100 potential failure surfaces and the calculation of the corresponding safety factors. Thus, a total of 2,000 potential failure surfaces were analyzed for each profile site with the probabilistic analysis. For each analysis, the 10 failure surfaces with the lowest safety factors were identified.

Interpretation of Rotational Slope Stability Analysis Results: The stability of the bluff slopes with respect to rotational sliding was determined for each bluff analysis section. The bluff slopes within each section were classified as stable, marginal, or unstable. The stability classifications were based on a combined interpretation of the deterministic slope stability analyses, the probabilistic slope stability analyses, observed slope conditions in the fall of 1987 for southern Milwaukee County and the Village of Bayside and in the summer of 1986 for the remainder of northern Milwaukee County, and historical records of slope failures.

A set of general guidelines developed to classify the bluff slopes on the basis of the deterministic and probabilistic slope stability analysis results is presented in Table 38. These guidelines were used to provide a general indication of slope stability—the final classifications set forth in this chapter being determined by a review of all available data. In interpreting the results of the deterministic and probabilistic stability analyses, both the lowest safety factor and the 10 lowest safety factors were considered.

Figure 39

SAMPLE VARIATION OF BLUFF CONDITIONS AND THE RESULTANT SAFETY FACTORS CALCULATED BY THE PROBABILISTIC SLOPE STABILITY ANALYSIS



Source: T. B. Edil, D. M. Mickelson, and SEWRPC.

<u>Bluff Slope Instability by Translational Sliding</u>

Translational slides, which involve slope failure along a planar surface generally parallel to the slope face, have little of the rotational movement or backward tilting characteristics discussed above for rotational slides. The stability of translational failure surfaces within the Milwaukee County shoreline was analyzed with the computer program INSLOPE (Infinite Slope Analysis). INSLOPE was developed by Professor Donald H. Gray at the University of Michigan. The program calculates the safety factors of slopes where the thickness of failed material is small in comparison to the height of the slope and where the failure surface is parallel to the slope surface. The concept of the infinite slope stability analysis for translational sliding is illustrated in Figure 40. In the analysis the resisting forces are due to cohesion and to friction. The primary driving force is the weight parallel to the failure surface. The safety factor is therefore defined as the ratio of the resisting force due to the shear strength of the soil along the failure surface to the driving force due to the weight of the sliding mass.

The safety factor for translational sliding based on the infinite slope analysis is calculated with the following equation:

Table 38

GUIDELINES FOR CLASSIFICATION OF BLUFF SLOPES FOR ROTATIONAL SLIDING

Stability Classification	Deterministic Slope Stability Analysis		Probabilistic Slope Stability Analysis	
	Lowest Safety Factor	Number of 10 Lowest Safety Factors Less than 1.0	Percent of Lowest Safety Factors Less than 1.0	Percent of 10 Lowest Safety Factors per Model Run Less than 1.0
Stable	> 1.0	0	< 25	< 10
Marginal	0.9-1.0	1-5	25-75	10-50
Unstable	< 0.9	6-10	> 75	> 50

NOTE: These guidelines are presented for general classification purposes only. The final slope stability classifications set forth in this chapter were based on the estimated safety factors, the size and location of the predicted failure surfaces, the observed slope conditions, and historical records of previous slope failure. Using the above guidelines, different stability classifications could be identified for a given bluff site, depending upon which modeling analysis and safety factors were considered. In those cases, a final classification was determined based on the judgment of the Commission staff and its consultants.

Source: T. B. Edil and SEWRPC.

$$SF = \frac{\left[\frac{c'}{\cos^2\alpha \tan\phi'} + (q_o + \gamma H) + (\gamma_{BUOY} - \gamma)H_w\right]\frac{\tan\phi'}{\tan\alpha}}{[(q_o + \gamma H) + (\gamma_{SATD} - \gamma)H_w]}$$

where:

SF = safety factor, dimensionless

ϕ' = internal friction angle, in degrees

c' = cohesion intercept, in pounds per square foot

α = slope angle, in degrees

γ = moist density of soil, in pounds per cubic foot

$\gamma SATD$ = saturated density of soil, in pounds per cubic foot

$\gamma BUOY$ = buoyant density of soil, in pounds per cubic foot ($\gamma_{BUOY} = \gamma_{SATD} - \gamma_w$)

γw = density of water, in pounds per cubic foot

H = vertical thickness of sliding mass, in feet

Hw = piezometric height above sliding surface, in feet

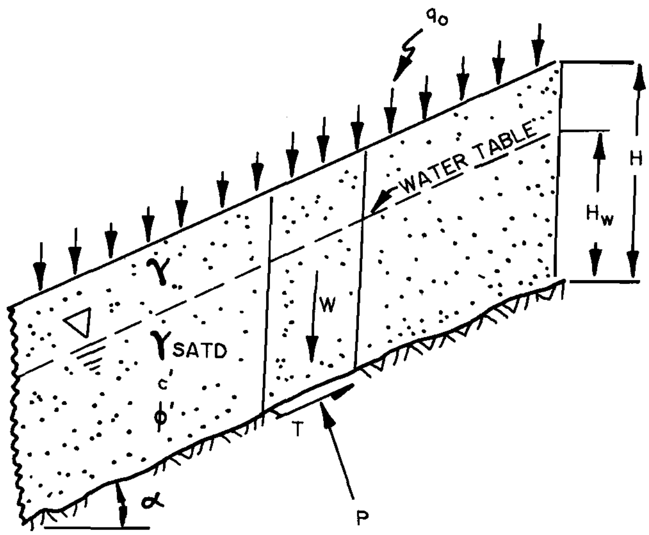
q_o = uniform vertical surcharge stress on slope, in pounds per square foot

The analysis was conducted under those bluff slope conditions commonly found within the study area to determine the conditions under which translational sliding may be expected to occur. The results were then applied to the specific bluff slope characteristics identified within each bluff analysis section. Bluff slope data used in the program included the thickness of the sliding mass, the slope angle, the soil properties, the hydrologic conditions, and the vegetative cover.

For the purposes of the translational sliding analysis, the thickness of the sliding mass was assumed to be three feet. This thickness is typical of shallow sliding masses along the Lake Michigan shoreline. A depth of three feet also

Figure 40

CONCEPT OF THE INFINITE SLOPE ANALYSIS FOR TRANSLATIONAL SLIDING



WHERE:
q_o = VERTICAL SURCHARGE, IN POUNDS PER SQUARE FOOT
W = WEIGHT OF SOIL MASS, IN POUNDS
P = NORMAL FORCE, IN POUNDS
γ = UNSATURATED DENSITY OF SOIL, IN POUNDS PER CUBIC FOOT
γ_{SATD} = SATURATED DENSITY OF SOIL, IN POUNDS PER CUBIC FOOT
c' = COHESION INTERCEPT, IN POUNDS PER SQUARE FOOT
ϕ' = INTERNAL FRICTION ANGLE, IN DEGREES
H = VERTICAL THICKNESS OF SLIDING MASS, IN FEET
Hw = PIEZOMETRIC HEIGHT ABOVE SLIDING SURFACE, IN FEET
T = TENSILE STRENGTH OF VEGETATION ROOTS, IN POUNDS PER SQUARE FOOT
α = SLOPE ANGLE, IN DEGREES

Source: D. H. Gray and A. T. Leiser, Biotechnical Slope Protection and Erosion Control, 1982.

approximates the average maximum depth of penetration by the roots of vegetation on the bluff face. Vegetative cover can minimize or prevent shallow mass movement in bluff slopes. The slope angles used in the analysis ranged from 10 to 40 degrees. The likelihood of translational sliding in slopes at an angle of less than 10 degrees was assumed to be minimal and therefore not evaluated. The effects of translational sliding at slope angles greater than 40 degrees were assumed to be modest compared to the effects of rotational sliding, and therefore were also not evaluated. The soil properties assumed in the analysis were the same as those used in the rotational slope stability analysis set forth in Table 36.

The effect of groundwater was evaluated under three conditions. The first condition assumed the soil to be unsaturated, which would produce the most stable slope. The second condition considered movement of groundwater parallel to the bluff face. The third condition considered the effects of groundwater emerging from the bluff face, which would produce the least stable slope.

Vegetation has an important influence on both surficial erosion and shallow mass movement. The presence of vegetation on a bluff slope can minimize many of the factors and conditions causing shallow slope failure by increasing the soil shear strength by root reinforcement and by decreasing soil moisture by evapotranspiration. However, vegetation can also reduce slope stability by adding a surcharge, or loading, to the bluff slope. The contribution and significance of vegetation to the stability of slopes was evaluated in this analysis by increasing the cohesion of the soil by a factor of 200 pounds per square foot (psf) and by adding a vertical surcharge of 25 psf.

The safety factors calculated with INSLOPE were grouped into three categories of potential for translational sliding. Conditions where safety factors were less than 1.0 were assumed to indicate a likelihood of failure, and such bluffs were classified as unstable. Bluff slopes with safety factors ranging from 1.0 to 1.5 were classified as marginal. Bluff slopes with safety factors greater than 1.5 were classified as stable. Table 39 presents the results of the translational stability analysis for the bluff slope conditions modeled. For each bluff analysis section, the potential for slope failure by translational sliding was determined on the basis of the observed slope, soil, hydrologic, and vegetation conditions at each profile site, and of the INSLOPE modeling results set forth in Table 39.

RESULTS

The results of the wave analysis of major shore protection structures and beaches; the evaluation of the stability of the bluff slopes within each bluff analysis section; and an assessment of shoreline erosion are presented below. The results of these analyses were used to determine the shoreline protection needs in the study area. Those major structures and beaches that may be damaged by wave overtopping were identified under six different water level and storm wave

Table 39

BLUFF STABILITY CLASSIFICATION BASED ON THE POTENTIAL FOR TRANSLATIONAL SLIDING UNDER BLUFF CONDITIONS FOUND IN MILWAUKEE COUNTY

Soil Type	Vegetated Bluff Face[a]					Unvegetated Bluff Face[a]				
	Groundwater Condition in Bluff	Slope Angle				Groundwater Condition in Bluff	Slope Angle			
		10°	20°	30°	40°		10°	20°	30°	40°
Tills										
Ozaukee	Unsaturated	S	S	S	S	Unsaturated	S	S	S	M
	Seepage parallel to face	S	S	S	S	Seepage parallel to face	S	S	M	M
	Seepage emerging from face	S	S	S	U	Seepage emerging from face	S	M	U	U
Oak Creek	Unsaturated	S	S	S	S	Unsaturated	S	S	S	M
	Seepage parallel to face	S	S	S	S	Seepage parallel to face	S	S	M	U
	Seepage emerging from face	S	S	S	M	Seepage emerging from face	S	U	U	U
New Berlin	Unsaturated	S	S	S	S	Unsaturated	S	S	M	U
	Seepage parallel to face	S	S	S	M	Seepage parallel to face	S	M	U	U
	Seepage emerging from face	S	S	M	U	Seepage emerging from face	U	U	U	U
Tiskilwa	Unsaturated	S	S	S	S	Unsaturated	S	S	S	S
	Seepage parallel to face	S	S	S	S	Seepage parallel to face	S	S	S	S
	Seepage emerging from face	S	S	S	S	Seepage emerging from face	S	S	S	S
Lake Sediments										
Medium Fine Sand	Unsaturated	S	S	S	S	Unsaturated	S	S	M	U
	Seepage parallel to face	S	S	S	M	Seepage parallel to face	S	U	U	U
	Seepage emerging from face	S	S	M	U	Seepage emerging from face	U	U	U	U
Coarse Sand	Unsaturated	S	S	S	S	Unsaturated	S	S	M	U
	Seepage parallel to face	S	S	S	M	Seepage parallel to face	S	U	U	U
	Seepage emerging from face	S	S	M	U	Seepage emerging from face	U	U	U	U
Sand and Gravel	Unsaturated	S	S	S	S	Unsaturated	S	S	M	U
	Seepage parallel to face	S	S	S	M	Seepage parallel to face	S	U	U	U
	Seepage emerging from face	S	S	M	S	Seepage emerging from face	U	U	U	U
Silt and Fine Sand	Unsaturated	S	S	S	S	Unsaturated	S	S	M	U
	Seepage parallel to face	S	S	S	S	Seepage parallel to face	S	U	U	U
	Seepage emerging from face	S	S	M	U	Seepage emerging from face	U	U	U	U
Fine Sand and Silt	Unsaturated	S	S	S	S	Unsaturated	S	S	S	M
	Seepage parallel to face	S	S	S	S	Seepage parallel to face	S	S	M	U
	Seepage emerging from face	S	S	S	M	Seepage emerging from face	S	U	U	U
Clay and Silt	Unsaturated	S	S	S	S	Unsaturated	S	S	S	S
	Seepage parallel to face	S	S	S	S	Seepage parallel to face	S	S	S	S
	Seepage emerging from face	S	S	S	S	Seepage emerging from face	S	S	S	S
Silt	Unsaturated	S	S	S	S	Unsaturated	S	S	S	S
	Seepage parallel to face	S	S	S	S	Seepage parallel to face	S	S	S	S
	Seepage emerging from face	S	S	S	S	Seepage emerging from face	S	S	S	S
General Lake Sediment	Unsaturated	S	S	S	S	Unsaturated	S	S	M	M
	Seepage parallel to face	S	S	S	M	Seepage parallel to face	S	M	U	U
	Seepage emerging from face	S	S	S	M	Seepage emerging from face	S	U	U	U

[a] *Bluff stability classification based on the potential for translational sliding:* *S - Stable Bluff Slope*
M - Marginal Bluff Slope
U - Unstable Bluff Slope

Source: SEWRPC.

conditions. For each bluff analysis section the severity of the shoreline erosion problem is identified, and the types of measures needed to fully stabilize the bluff slope presented. Effective stabilization of a bluff slope may require a combination of bluff toe protection, surface water and groundwater drainage control, revegetation of the bluff face, and modification of the bluff slope either by filling or by cutting back the slope. In order to maintain the natural aesthetic properties and natural drainage characteristics of the bluffs, modification of a bluff slope by filling or by cutting back the slope was recommended only where other control measures—which would maintain or reestablish these natural characteristics—would not effectively stabilize the slope. In this respect, however, it is recognized that filling could effectively be used to stabilize many slopes in lieu of other types of slope stabilization measures. Chapter IV describes and evaluates the specific alternative shore protection measures available.

The results set forth in this report are based on systems level, generalized analyses of major shore protection structures and bluff analysis sections. The evaluation of individual lakeshore properties and the detailed design of shore protection structures will require site-specific analyses by professional geotechnical or coastal engineers. Such analyses should, however, be conducted within the systems plan provided by this report.

<u>Evaluation of Major Shore Protection Structures and Beaches</u>

The potential for wave runup overtopping damage was evaluated for 35 major shore protection structures and beaches under each of the six alternative high water level-wave conditions. These evaluations were based upon systems level wave analyses and general structure conditions. A detailed analysis, including a site-specific bathymetric survey and a structural engineering inspection, will be required to provide a more definitive estimate of the potential for actual damage. The major structures and beaches herein evaluated, which cover about 12.8 miles of shoreline, or 43 percent of the total shoreline of Milwaukee County, constitute the most well-protected portion of the shoreline. Thus, the remaining 57 percent of the county shoreline would be expected to suffer damage more frequently than would the shoreline protected by these major structures and beaches.

As discussed in the "Methods and Analysis" section, the potential for damage to revetments and beaches is determined by the estimated wave runup height. The runup heights that would be expected to cause significant damage were determined on the basis of judgment exercised by the Commission staff and consultants. For bulkheads, the potential for damage was based upon the calculated wave overtopping rate. The overtopping rates that would be expected to cause damage were derived from guidelines set forth by Goda.[36]

The estimates of the potential for overtopping damage to the Port of Milwaukee bulkhead slips within the outer harbor, the Milwaukee County McKinley Beach/revetment, and the Village of Whitefish Bay Klode Park breakwater/beach were not based solely on the wave runup calculations. These structures were previously evaluated using hydraulic physical models.[37] The physical models were judged to provide a more accurate depiction of wave conditions than the mathematical models, and the results of the physical modeling were therefore considered in the classification of the potential for overtopping damage to these structures.

The McKinley Marina bulkheads and the outer harbor and South Shore breakwaters themselves were not evaluated for overtopping damage. The McKinley Marina has not historically suffered wave damage because the Marina is nearly enclosed, and because the northernmost portion of the outer harbor breakwater—which protects the Marina—is quite high. During periods of high water, however, the Marina has been prone

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[36]*Goda, op. cit.*

[37]*Physical modeling results for the McKinley Beach/revetment and the Klode Park breakwater/beach were discussed with personnel of Warzyn Engineering, Inc., who designed the structures. Physical modeling results for the Port of Milwaukee bulkhead slips are set forth in: E. F. Brater and L. D. Stair, <u>Model Study of Milwaukee Outer Harbor</u>, University of Michigan Engineering Research Institute, August 1952; and STS Consultants, Ltd., <u>Final Report, Milwaukee Harbor Hydraulic Model Study</u>, Prepared for the Milwaukee Water Pollution Abatement Program, September 1982.*

to be damaged by ice effects. A description of the ice damage that has occurred in the Marina is provided in Volume Two of SEWRPC *Planning* Report No. 37, A Water Resources Management Plan for the Milwaukee Harbor Estuary, 1987. Recommended methods to abate this damage are presented in Chapter VI of that report.

The outer harbor and South Shore breakwaters are severely overtopped under all water level and storm wave conditions considered. These structures are not necessarily damaged by this overtopping—although portions of the breakwaters are in need of repair. The damages caused by waves overtopping the breakwaters are primarily suffered by the onshore structures which are impacted by the transmitted waves. As already noted, wave transmission over the breakwaters was considered in the evaluation of those onshore structures which lie within the harbor. Possible modifications to the breakwaters to reduce this wave transmission are discussed in Chapter IV of this report.

The results of the wave runup and overtopping analyses are summarized in Table 40. For each structure and beach under each of the high water level-wave conditions, the potential for overtopping damage is classified as insignificant, low, moderate, or high. The results of the analyses under each of the high water level-wave conditions are graphically illustrated on Maps 27 through 32.

Under a 10-year recurrence interval water level with a 20-year recurrence interval storm wave, as shown on Map 27, seven, or 58 percent, of the revetments; seven, or 88 percent, of the beaches; and four, or 27 percent, of the bulkheads may be expected to exhibit an insignificant or low potential for wave overtopping damage. The remaining five, or 42 percent, of the revetments; one, or 12 percent, of the beaches; and 11, or 73 percent, of the bulkheads may be expected to exhibit a moderate or high potential for overtopping damage. Of the total 12.8 miles of Milwaukee County shoreline protected by major structures and beaches, about 37 percent would have an insignificant potential, 11 percent a low potential, 22 percent a moderate potential, and 30 percent a high potential for overtopping damage.

Under a 100-year recurrence interval water level with a 20-year recurrence interval storm wave, as shown on Map 28, only one, or 8 percent, of the revetments; seven or 88 percent, of the beaches; and two, or 13 percent, of the bulkheads may be expected to exhibit an insignificant or low potential for wave overtopping damage. The remaining 11, or 92 percent, of the revetments; one, or 12 percent, of the beaches; and 13, or 87 percent, of the bulkheads may be expected to exhibit a moderate or high potential for overtopping damage. Of the total 12.8 miles of Milwaukee County shoreline protected by major structures and beaches, about 17 percent may be expected to exhibit an insignificant potential, 7 percent a low potential, 17 percent a moderate potential, and 59 percent a high potential for overtopping damage.

Under a 500-year recurrence interval water level with a 20-year recurrence interval storm wave, as shown on Map 29, only one, or 8 percent, of the revetments; five, or 62 percent, of the beaches; and one, or 7 percent, of the bulkheads may be expected to exhibit an insignificant or low potential for wave overtopping damage. The remaining 11, or 92 percent, of the revetments; three, or 38 percent, of the beaches; and 14, or 93 percent, of the bulkheads may be expected to exhibit a moderate or high potential for overtopping damage. Of the total 12.8 miles of Milwaukee County shoreline protected by major structures and beaches, about 7 percent may be expected to exhibit an insignificant potential, 8 percent a low potential, 11 percent a moderate potential, and 74 percent a high potential for overtopping damage.

Under a 10-year recurrence interval water level with a 50-year recurrence interval storm wave, as shown on Map 30, six, or 50 percent, of the revetments; seven, or 88 percent, of the beaches; and two, or 13 percent, of the bulkheads may be expected to exhibit an insignificant or low potential for wave overtopping damage. The remaining six, or 50 percent, of the revetments; one, or 12 percent, of the beaches; and 13, or 87 percent, of the bulkheads may be expected to exhibit a moderate or high potential for overtopping damage. Of the total 12.8 miles of Milwaukee County shoreline protected by major structures and beaches, about 33 percent may be expected to exhibit an insignificant potential, 3 percent a low potential, 18 percent a moderate potential, and 46 percent a high potential for overtopping damage.

Table 40

POTENTIAL FOR WAVE OVERTOPPING DAMAGE TO MAJOR SHORE PROTECTION STRUCTURES AND BEACHES IN MILWAUKEE COUNTY UNDER VARIOUS WATER LEVEL-STORM WAVE CONDITIONS

Shore Protection Structure or Beach	Potential for Damage by Wave Overtopping[a]					
	20-Year Storm Wave			50-Year Storm Wave		
	10-Year Water Level (582.8 feet NGVD)	100-Year Water Level (584.3 feet NGVD)	500-Year Water Level (585.9 feet NGVD)	10-Year Water Level (582.8 feet NGVD)	100-Year Water Level (584.3 feet NGVD)	500-Year Water Level (585.9 feet NGVD)
1. Village of Bayside Beach	M	M	H	M	M	H
2. Milwaukee County Doctors Park Beach	I	L	M	I	M	M
3. Village of Fox Point Beach Drive-North Revetment	L	M	H	L	M	H
4. Village of Fox Point Beach Drive-South Revetment	I	M	H	I	M	H
5. Village of Whitefish Bay Klode Park Breakwater/Beach	I	I	L	I	I	L
6. Milwaukee County Big Bay Park Bulkhead	M	H	H	H	H	H
7. Village of Whitefish Bay Buckley Park Bulkhead	H	H	H	H	H	H
8. Village of Shorewood Atwater Park Beach	I	I	L	I	I	M
9. City of Milwaukee Linnwood Avenue Water Treatment Plant Bulkhead	H	H	H	H	H	H
10. Milwaukee County Lake Park-North Revetment	I	M	M	I	M	H
11. Milwaukee County Lake Park-South Revetment	I	M	M	I	M	H
12. Milwaukee County Bradford Beach	I	I	L	I	I	M
13. Milwaukee County McKinley Beach/ Revetment	I	L	L	I	L	L
14. Milwaukee County Juneau Park Landfill Bulkhead	L	H	H	H	H	H
15. Milwaukee County War Memorial Center Bulkhead	I	I	M	L	M	M
16. Milwaukee Harbor Commission Municipal Pier Bulkhead	L	H	H	H	H	H
17. Henry W. Maier Festival Grounds Revetment	M	H	H	H	H	H
18. Marcus Amphitheatre Bulkhead	M	H	H	M	H	H
19. MMSD Jones Island Wastewater Treatment Plant Bulkhead	I	I	I	I	I	M
20. Port of Milwaukee Bulkhead Slips	H	H	H	H	H	H
21. South Lincoln Memorial Drive Bulkhead	M	H	H	M	H	H

Table 40 (continued)

Shore Protection Structure or Beach	Potential for Damage by Wave Overtopping[a]					
	20-Year Storm Wave			50-Year Storm Wave		
	10-Year Water Level (582.8 feet NGVD)	100-Year Water Level (584.3 feet NGVD)	500-Year Water Level (585.9 feet NGVD)	10-Year Water Level (582.8 feet NGVD)	100-Year Water Level (584.3 feet NGVD)	500-Year Water Level (585.9 feet NGVD)
22. U. S. Army Corps of Engineers Dredge Spoils Confined Disposal Facility Revetment	H	H	H	H	H	H
23. Milwaukee County South Shore Park-North Revetment	M	H	H	M	H	H
24. Milwaukee County South Shore Park Marina Bulkhead	M	H	H	M	H	H
25. Milwaukee County South Shore Park Beach	I	I	I	I	I	I
26. Milwaukee County South Shore Park-South Revetment	I	M	H	M	M	H
27. Former WEPCo Lakeside Power Plant Site Revetment	M	H	H	M	H	H
28. Milwaukee County Sheridan Park Beach	I	I	M	I	L	M
29. City of Cudahy Water Intake Bulkhead	M	H	H	H	H	H
30. Milwaukee County Grant Park Beach	I	I	I	I	I	L
31. South Milwaukee Yacht Club Revetment	I	M	M	I	M	M
32. South Milwaukee Wastewater Treatment Plant Revetment	M	M	H	M	M	H
33. MMSD South Shore Wastewater Treatment Plant Bulkhead	H	H	H	H	H	H
34. City of Oak Creek Water Intake Bulkhead	M	M	H	M	H	H
35. WEPCo Oak Creek Power Plant Bulkhead	H	H	H	H	H	H

[a]*Potential for wave overtopping damage is classified as follows:*

<u>Revetments and Beaches</u> *(runup height)*

I - Insignificant; wave runup generally does not exceed top of structure or beach.
L - Low; wave runup exceeds top of structure by 0 to 1.0 foot.
M - Moderate; wave runup exceeds top of structure by 1.1 to 5.0 feet.
H - High; wave runup exceeds top of structure by greater than 5.0 feet.

<u>Bulkheads</u> *(overtopping rate)*

I - Insignificant; wave overtopping is less than 0.005 cfs/foot.
L - Low; wave overtopping ranges from 0.005 to 0.01 cfs/foot.
M - Moderate; wave overtopping ranges from 0.01 to 0.1 cfs/foot.
H - High; wave overtopping is greater than 0.1 cfs/foot.

Source: SEWRPC.

Map 27

POTENTIAL FOR WAVE OVERTOPPING DAMAGE TO MAJOR STRUCTURES AND BEACHES WITH A 10-YEAR LAKE MICHIGAN WATER LEVEL OF 582.8 FEET NGVD AND A 20-YEAR STORM WAVE

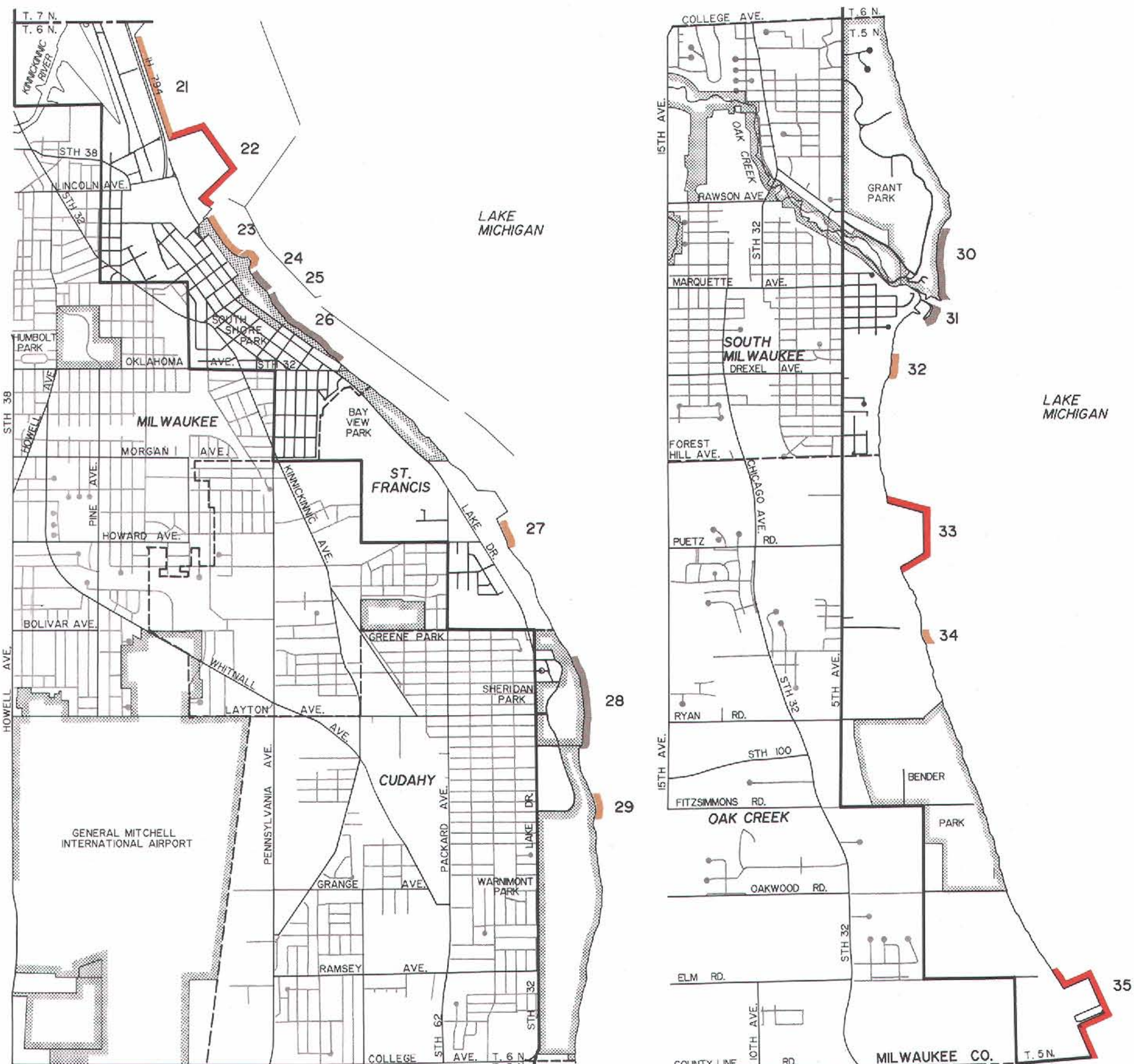
LEGEND

POTENTIAL FOR WAVE OVERTOPPING DAMAGE

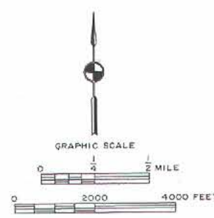
- INSIGNIFICANT
- LOW
- MODERATE
- HIGH

1 BAYSIDE BEACH
2 DOCTORS PARK BEACH
3 BEACH DRIVE-NORTH REVETMENT
4 BEACH DRIVE-SOUTH REVETMENT
5 KLODE PARK BREAKWATER/BEACH
6 BIG BAY PARK BULKHEAD
7 BUCKLEY PARK BULKHEAD
8 ATWATER PARK BEACH
9 LINNWOOD AVENUE WATER TREATMENT PLANT BULKHEAD
10 LAKE PARK-NORTH REVETMENT
11 LAKE PARK-SOUTH REVETMENT
12 BRADFORD BEACH
13 MC KINLEY BEACH/REVETMENT
14 JUNEAU PARK LANDFILL BULKHEAD
15 WAR MEMORIAL CENTER BULKHEAD
16 MUNICIPAL PIER BULKHEAD
17 HENRY W. MAIER FESTIVAL GROUNDS REVETMENT
18 MARCUS AMPHITHEATER BULKHEAD
19 MMSD JONES ISLAND WASTEWATER TREATMENT PLANT BULKHEAD
20 PORT OF MILWAUKEE BULKHEAD SLIPS

Map 27 (continued)



21 SOUTH LINCOLN MEMORIAL DRIVE BULKHEAD
22 USACOE DREDGE SPOILS CONFINED DISPOSAL FACILITY REVETMENT
23 SOUTH SHORE PARK-NORTH REVETMENT
24 SOUTH SHORE YACHT CLUB BULKHEAD
25 SOUTH SHORE PARK BEACH
26 SOUTH SHORE PARK-SOUTH REVETMENT
27 FORMER WEPCO LAKESIDE POWER PLANT SITE REVETMENT
28 SHERIDAN PARK BEACH
29 CUDAHY WATER INTAKE BULKHEAD
30 GRANT PARK BEACH
31 SOUTH MILWAUKEE YACHT CLUB REVETMENT
32 SOUTH MILWAUKEE WASTEWATER TREATMENT PLANT REVETMENT
33 MMSD SOUTH SHORE WASTEWATER TREATMENT PLANT BULKHEAD
34 OAK CREEK WATER INTAKE BULKHEAD
35 WEPCO OAK CREEK POWER PLANT BULKHEAD



Source: SEWRPC.

Map 28

POTENTIAL FOR WAVE OVERTOPPING DAMAGE TO MAJOR STRUCTURES AND BEACHES WITH A 100-YEAR LAKE MICHIGAN WATER LEVEL OF 584.3 FEET NGVD AND A 20-YEAR STORM WAVE

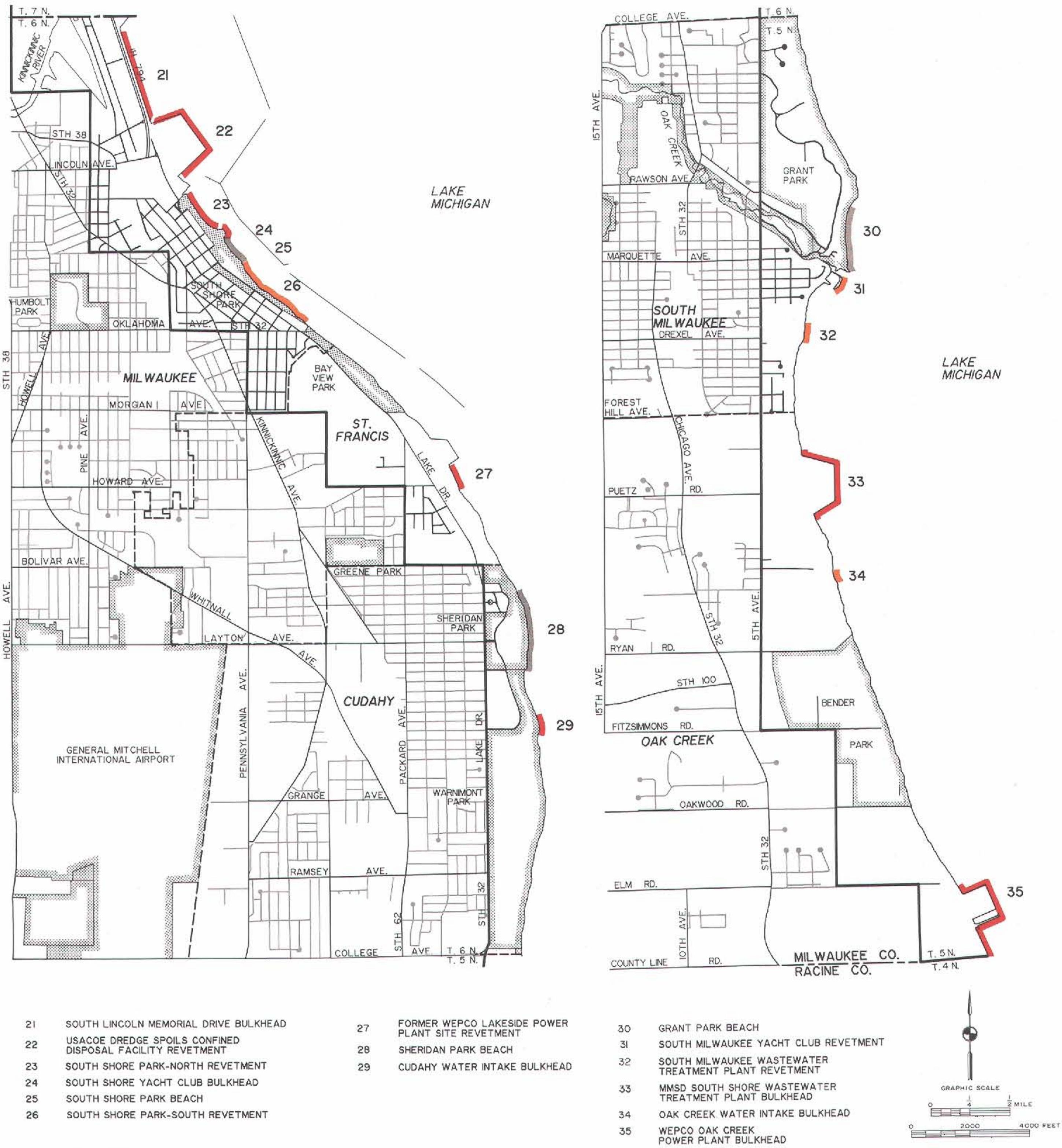
LEGEND

POTENTIAL FOR WAVE OVERTOPPING DAMAGE

- INSIGNIFICANT
- LOW
- MODERATE
- HIGH

1 BAYSIDE BEACH
2 DOCTORS PARK BEACH
3 BEACH DRIVE-NORTH REVETMENT
4 BEACH DRIVE-SOUTH REVETMENT
5 KLODE PARK BREAKWATER/BEACH
6 BIG BAY PARK BULKHEAD
7 BUCKLEY PARK BULKHEAD
8 ATWATER PARK BEACH
9 LINNWOOD AVENUE WATER TREATMENT PLANT BULKHEAD
10 LAKE PARK-NORTH REVETMENT
11 LAKE PARK-SOUTH REVETMENT
12 BRADFORD BEACH
13 MC KINLEY BEACH/REVETMENT
14 JUNEAU PARK LANDFILL BULKHEAD
15 WAR MEMORIAL CENTER BULKHEAD
16 MUNICIPAL PIER BULKHEAD
17 HENRY W. MAIER FESTIVAL GROUNDS REVETMENT
18 MARCUS AMPHITHEATER BULKHEAD
19 MMSD JONES ISLAND WASTEWATER TREATMENT PLANT BULKHEAD
20 PORT OF MILWAUKEE BULKHEAD SLIPS

Map 28 (continued)

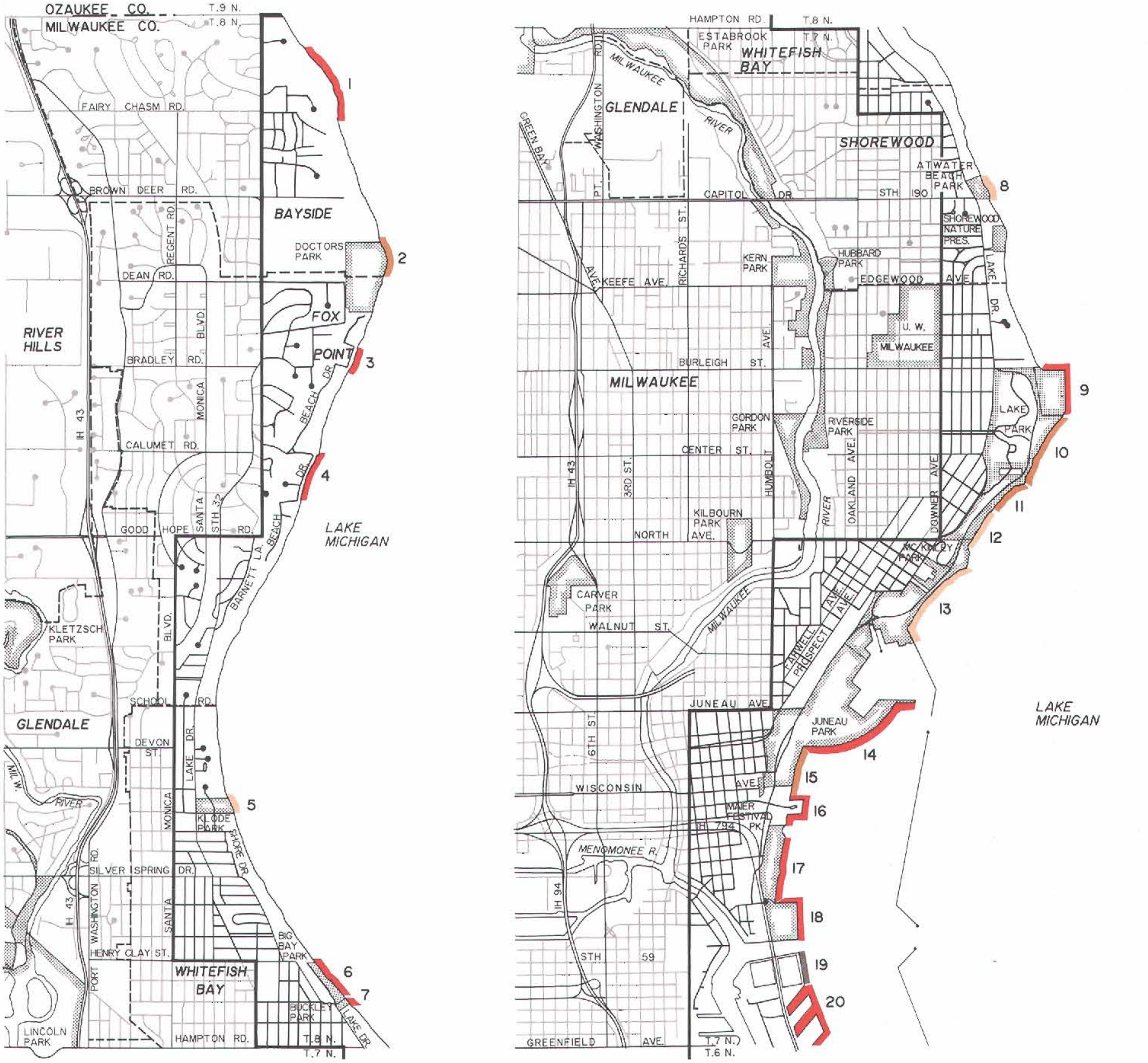


21 SOUTH LINCOLN MEMORIAL DRIVE BULKHEAD
22 USACOE DREDGE SPOILS CONFINED DISPOSAL FACILITY REVETMENT
23 SOUTH SHORE PARK-NORTH REVETMENT
24 SOUTH SHORE YACHT CLUB BULKHEAD
25 SOUTH SHORE PARK BEACH
26 SOUTH SHORE PARK-SOUTH REVETMENT
27 FORMER WEPCO LAKESIDE POWER PLANT SITE REVETMENT
28 SHERIDAN PARK BEACH
29 CUDAHY WATER INTAKE BULKHEAD
30 GRANT PARK BEACH
31 SOUTH MILWAUKEE YACHT CLUB REVETMENT
32 SOUTH MILWAUKEE WASTEWATER TREATMENT PLANT REVETMENT
33 MMSD SOUTH SHORE WASTEWATER TREATMENT PLANT BULKHEAD
34 OAK CREEK WATER INTAKE BULKHEAD
35 WEPCO OAK CREEK POWER PLANT BULKHEAD

Source: SEWRPC.

Map 29

POTENTIAL FOR WAVE OVERTOPPING DAMAGE TO MAJOR STRUCTURES AND BEACHES WITH A 500-YEAR LAKE MICHIGAN WATER LEVEL OF 585.9 FEET NGVD AND A 20-YEAR STORM WAVE



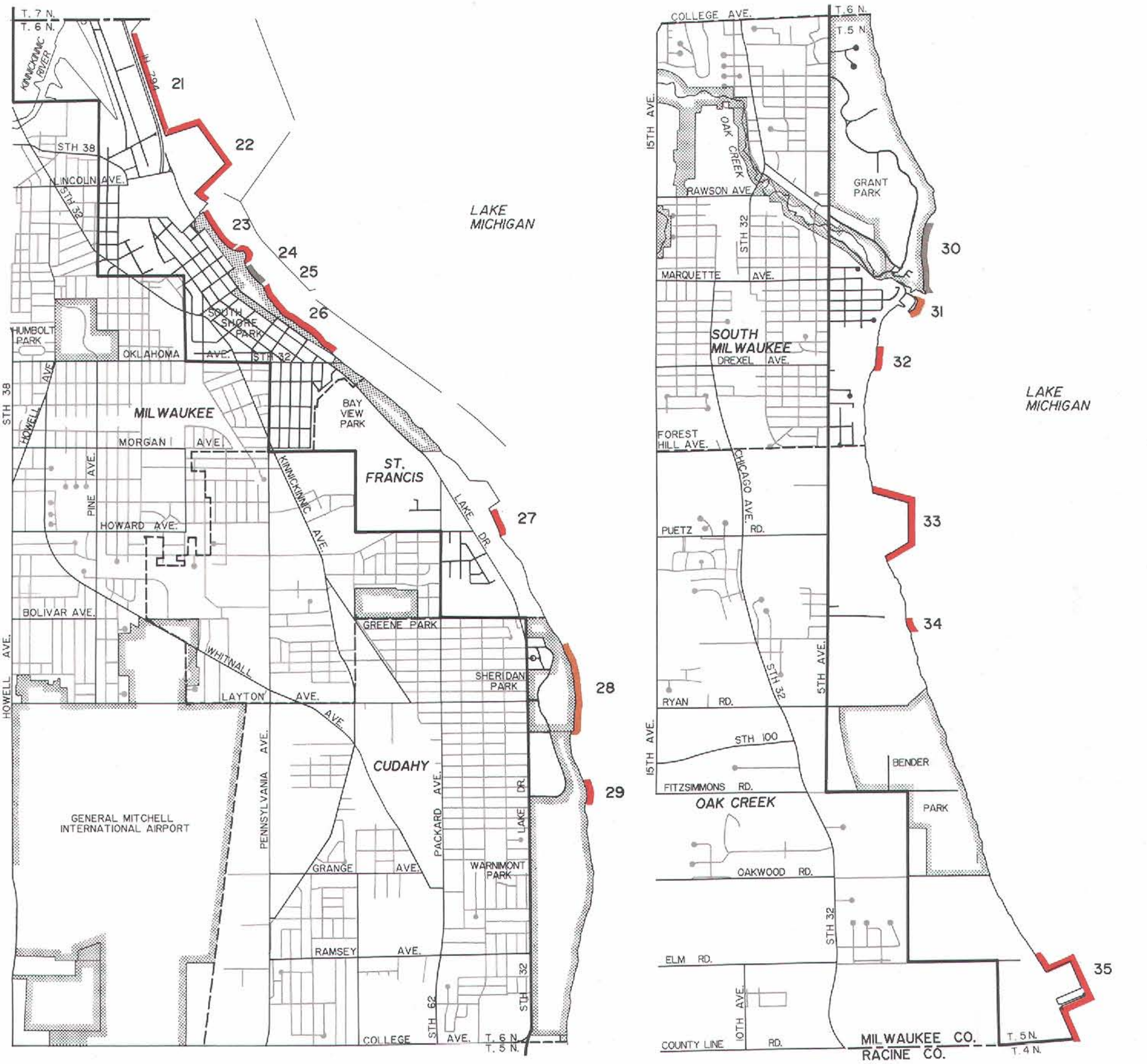
LEGEND

POTENTIAL FOR WAVE OVERTOPPING DAMAGE

- INSIGNIFICANT
- LOW
- MODERATE
- HIGH

1 BAYSIDE BEACH
2 DOCTORS PARK BEACH
3 BEACH DRIVE-NORTH REVETMENT
4 BEACH DRIVE-SOUTH REVETMENT
5 KLODE PARK BREAKWATER/BEACH
6 BIG BAY PARK BULKHEAD
7 BUCKLEY PARK BULKHEAD
8 ATWATER PARK BEACH
9 LINNWOOD AVENUE WATER TREATMENT PLANT BULKHEAD
10 LAKE PARK-NORTH REVETMENT
11 LAKE PARK-SOUTH REVETMENT
12 BRADFORD BEACH
13 MC KINLEY BEACH/REVETMENT
14 JUNEAU PARK LANDFILL BULKHEAD
15 WAR MEMORIAL CENTER BULKHEAD
16 MUNICIPAL PIER BULKHEAD
17 HENRY W. MAIER FESTIVAL GROUNDS REVETMENT
18 MARCUS AMPHITHEATER BULKHEAD
19 MMSD JONES ISLAND WASTEWATER TREATMENT PLANT BULKHEAD
20 PORT OF MILWAUKEE BULKHEAD SLIPS

Map 29 (continued)



21 SOUTH LINCOLN MEMORIAL DRIVE BULKHEAD
22 USACOE DREDGE SPOILS CONFINED DISPOSAL FACILITY REVETMENT
23 SOUTH SHORE PARK-NORTH REVETMENT
24 SOUTH SHORE YACHT CLUB BULKHEAD
25 SOUTH SHORE PARK BEACH
26 SOUTH SHORE PARK-SOUTH REVETMENT
27 FORMER WEPCO LAKESIDE POWER PLANT SITE REVETMENT
28 SHERIDAN PARK BEACH
29 CUDAHY WATER INTAKE BULKHEAD
30 GRANT PARK BEACH
31 SOUTH MILWAUKEE YACHT CLUB REVETMENT
32 SOUTH MILWAUKEE WASTEWATER TREATMENT PLANT REVETMENT
33 MMSD SOUTH SHORE WASTEWATER TREATMENT PLANT BULKHEAD
34 OAK CREEK WATER INTAKE BULKHEAD
35 WEPCO OAK CREEK POWER PLANT BULKHEAD

GRAPHIC SCALE
0 1/4 1/2 MILE
0 2000 4000 FEET

Source: SEWRPC.

Map 30

POTENTIAL FOR WAVE OVERTOPPING DAMAGE TO MAJOR STRUCTURES AND BEACHES WITH A 10-YEAR LAKE MICHIGAN WATER LEVEL OF 582.8 FEET NGVD AND A 50-YEAR STORM WAVE

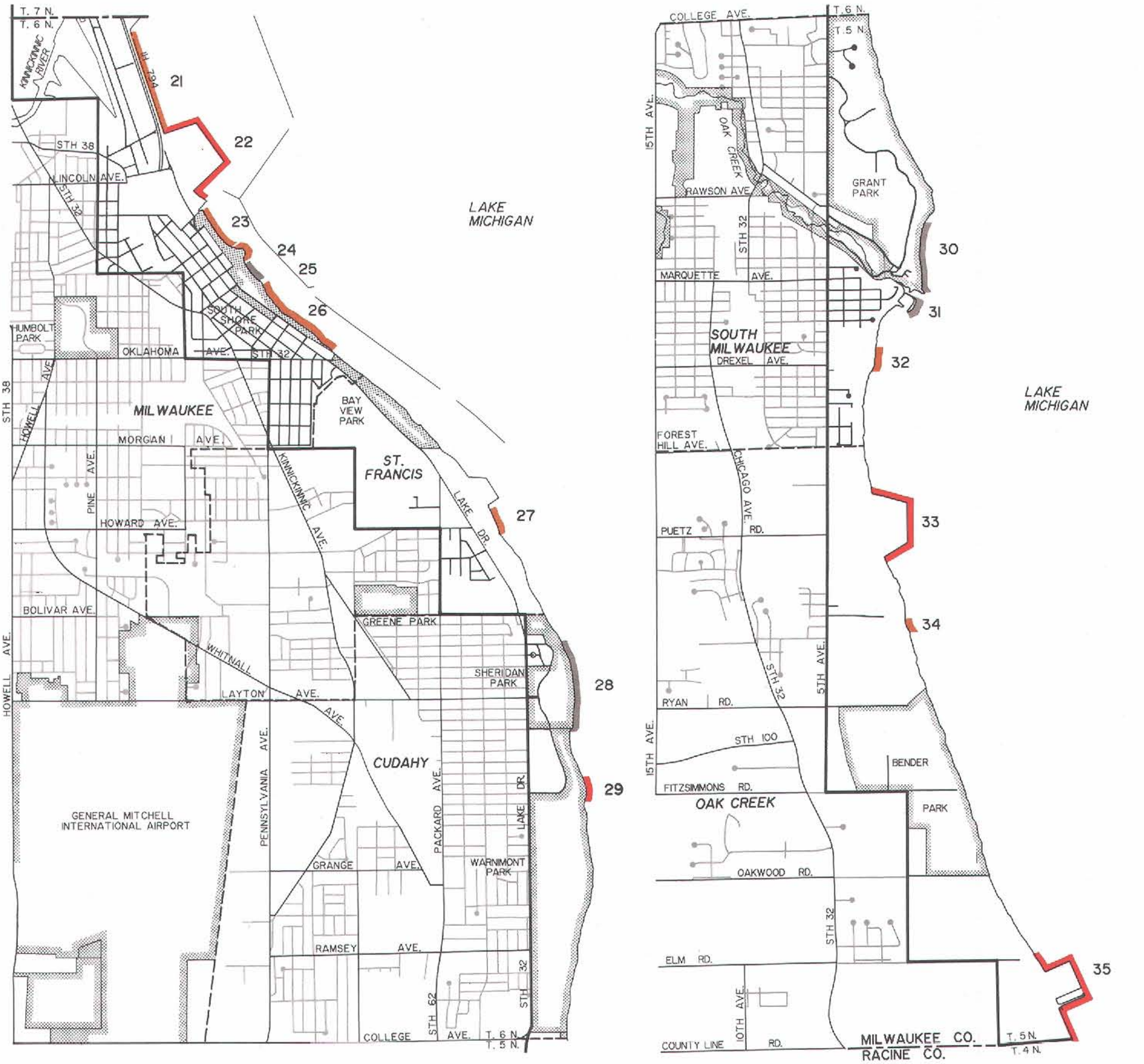
LEGEND

POTENTIAL FOR WAVE OVERTOPPING DAMAGE

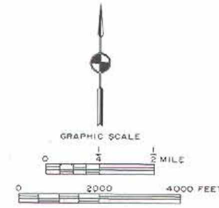
- INSIGNIFICANT
- LOW
- MODERATE
- HIGH

1 BAYSIDE BEACH
2 DOCTORS PARK BEACH
3 BEACH DRIVE-NORTH REVETMENT
4 BEACH DRIVE-SOUTH REVETMENT
5 KLODE PARK BREAKWATER/BEACH
6 BIG BAY PARK BULKHEAD
7 BUCKLEY PARK BULKHEAD
8 ATWATER PARK BEACH
9 LINNWOOD AVENUE WATER TREATMENT PLANT BULKHEAD
10 LAKE PARK-NORTH REVETMENT
11 LAKE PARK-SOUTH REVETMENT
12 BRADFORD BEACH
13 MC KINLEY BEACH/REVETMENT
14 JUNEAU PARK LANDFILL BULKHEAD
15 WAR MEMORIAL CENTER BULKHEAD
16 MUNICIPAL PIER BULKHEAD
17 HENRY W. MAIER FESTIVAL GROUNDS REVETMENT
18 MARCUS AMPHITHEATER BULKHEAD
19 MMSD JONES ISLAND WASTEWATER TREATMENT PLANT BULKHEAD
20 PORT OF MILWAUKEE BULKHEAD SLIPS

Map 30 (continued)



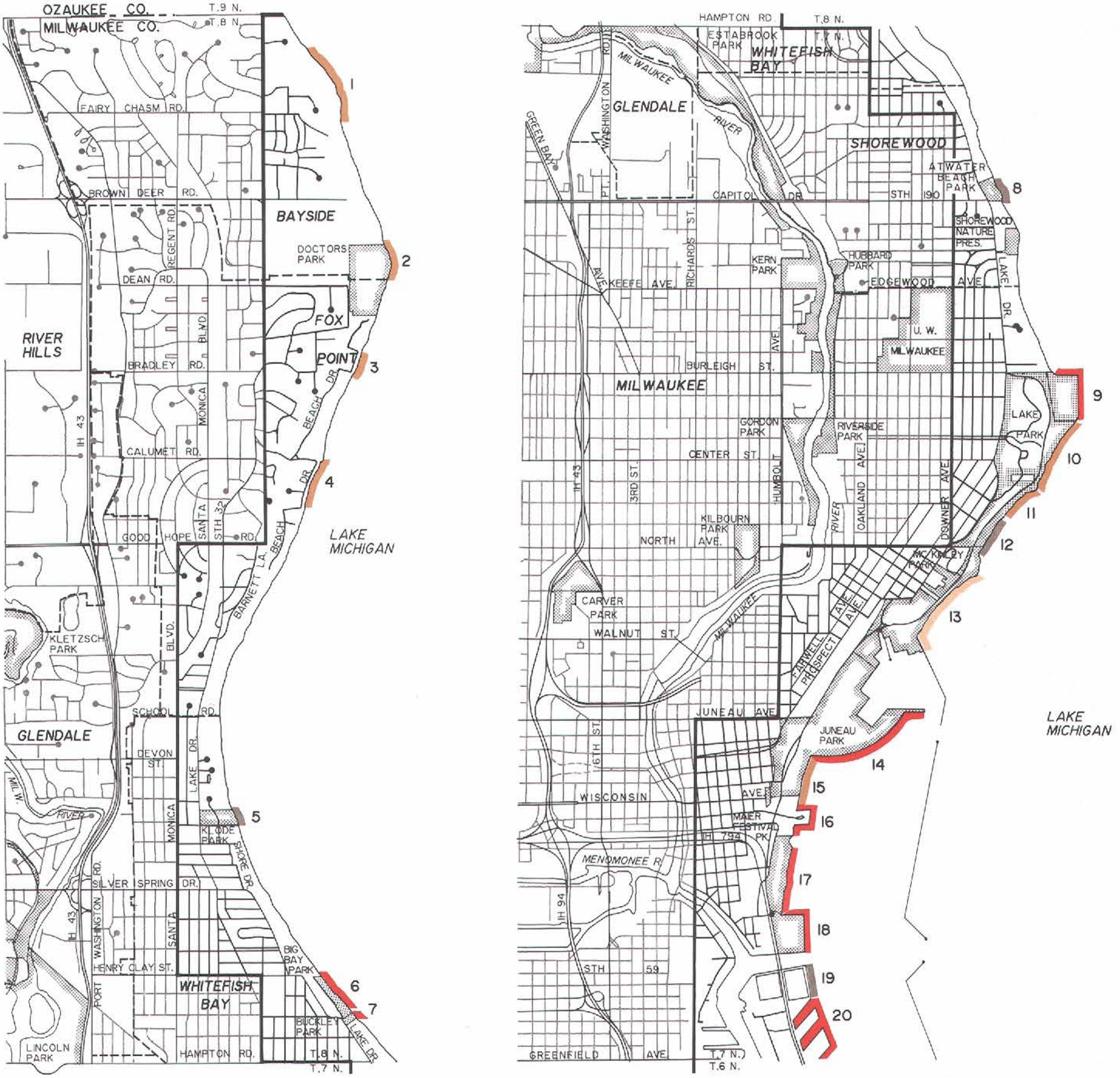
21	SOUTH LINCOLN MEMORIAL DRIVE BULKHEAD
22	USACOE DREDGE SPOILS CONFINED DISPOSAL FACILITY REVETMENT
23	SOUTH SHORE PARK-NORTH REVETMENT
24	SOUTH SHORE YACHT CLUB BULKHEAD
25	SOUTH SHORE PARK BEACH
26	SOUTH SHORE PARK-SOUTH REVETMENT
27	FORMER WEPCO LAKESIDE POWER PLANT SITE REVETMENT
28	SHERIDAN PARK BEACH
29	CUDAHY WATER INTAKE BULKHEAD
30	GRANT PARK BEACH
31	SOUTH MILWAUKEE YACHT CLUB REVETMENT
32	SOUTH MILWAUKEE WASTEWATER TREATMENT PLANT REVETMENT
33	MMSD SOUTH SHORE WASTEWATER TREATMENT PLANT BULKHEAD
34	OAK CREEK WATER INTAKE BULKHEAD
35	WEPCO OAK CREEK POWER PLANT BULKHEAD



Source: SEWRPC.

Map 31

POTENTIAL FOR WAVE OVERTOPPING DAMAGE TO MAJOR STRUCTURES AND BEACHES WITH A 100-YEAR LAKE MICHIGAN WATER LEVEL OF 584.3 FEET NGVD AND A 50-YEAR STORM WAVE



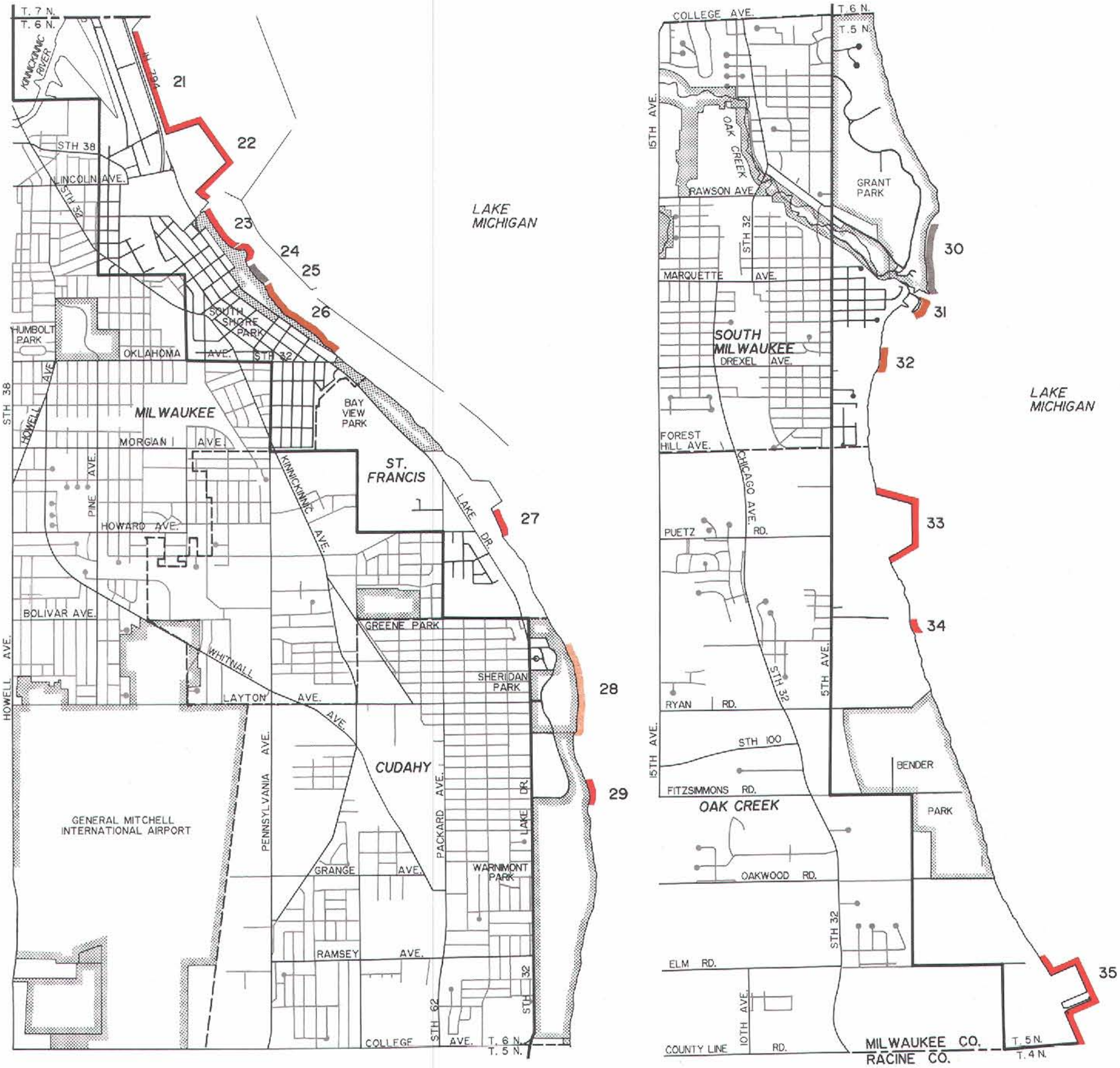
LEGEND

POTENTIAL FOR WAVE OVERTOPPING DAMAGE

- INSIGNIFICANT
- LOW
- MODERATE
- HIGH

1	BAYSIDE BEACH	11	LAKE PARK-SOUTH REVETMENT
2	DOCTORS PARK BEACH	12	BRADFORD BEACH
3	BEACH DRIVE-NORTH REVETMENT	13	MC KINLEY BEACH/REVETMENT
4	BEACH DRIVE-SOUTH REVETMENT	14	JUNEAU PARK LANDFILL BULKHEAD
5	KLODE PARK BREAKWATER/BEACH	15	WAR MEMORIAL CENTER BULKHEAD
6	BIG BAY PARK BULKHEAD	16	MUNICIPAL PIER BULKHEAD
7	BUCKLEY PARK BULKHEAD	17	HENRY W. MAIER FESTIVAL GROUNDS REVETMENT
8	ATWATER PARK BEACH	18	MARCUS AMPHITHEATER BULKHEAD
9	LINNWOOD AVENUE WATER TREATMENT PLANT BULKHEAD	19	MMSD JONES ISLAND WASTEWATER TREATMENT PLANT BULKHEAD
10	LAKE PARK-NORTH REVETMENT	20	PORT OF MILWAUKEE BULKHEAD SLIPS

Map 31 (continued)



21	SOUTH LINCOLN MEMORIAL DRIVE BULKHEAD	27	FORMER WEPCO LAKESIDE POWER PLANT SITE REVETMENT	30	GRANT PARK BEACH
22	USACOE DREDGE SPOILS CONFINED DISPOSAL FACILITY REVETMENT	28	SHERIDAN PARK BEACH	31	SOUTH MILWAUKEE YACHT CLUB REVETMENT
23	SOUTH SHORE PARK-NORTH REVETMENT	29	CUDAHY WATER INTAKE BULKHEAD	32	SOUTH MILWAUKEE WASTEWATER TREATMENT PLANT REVETMENT
24	SOUTH SHORE YACHT CLUB BULKHEAD			33	MMSD SOUTH SHORE WASTEWATER TREATMENT PLANT BULKHEAD
25	SOUTH SHORE PARK BEACH			34	OAK CREEK WATER INTAKE BULKHEAD
26	SOUTH SHORE PARK-SOUTH REVETMENT			35	WEPCO OAK CREEK POWER PLANT BULKHEAD

GRAPHIC SCALE
0 1/4 1/2 MILE
0 2000 4000 FEET

Source: SEWRPC.

Map 32

POTENTIAL FOR WAVE OVERTOPPING DAMAGE TO MAJOR STRUCTURES AND BEACHES WITH A 500-YEAR LAKE MICHIGAN WATER LEVEL OF 585.9 FEET NGVD AND A 50-YEAR STORM WAVE

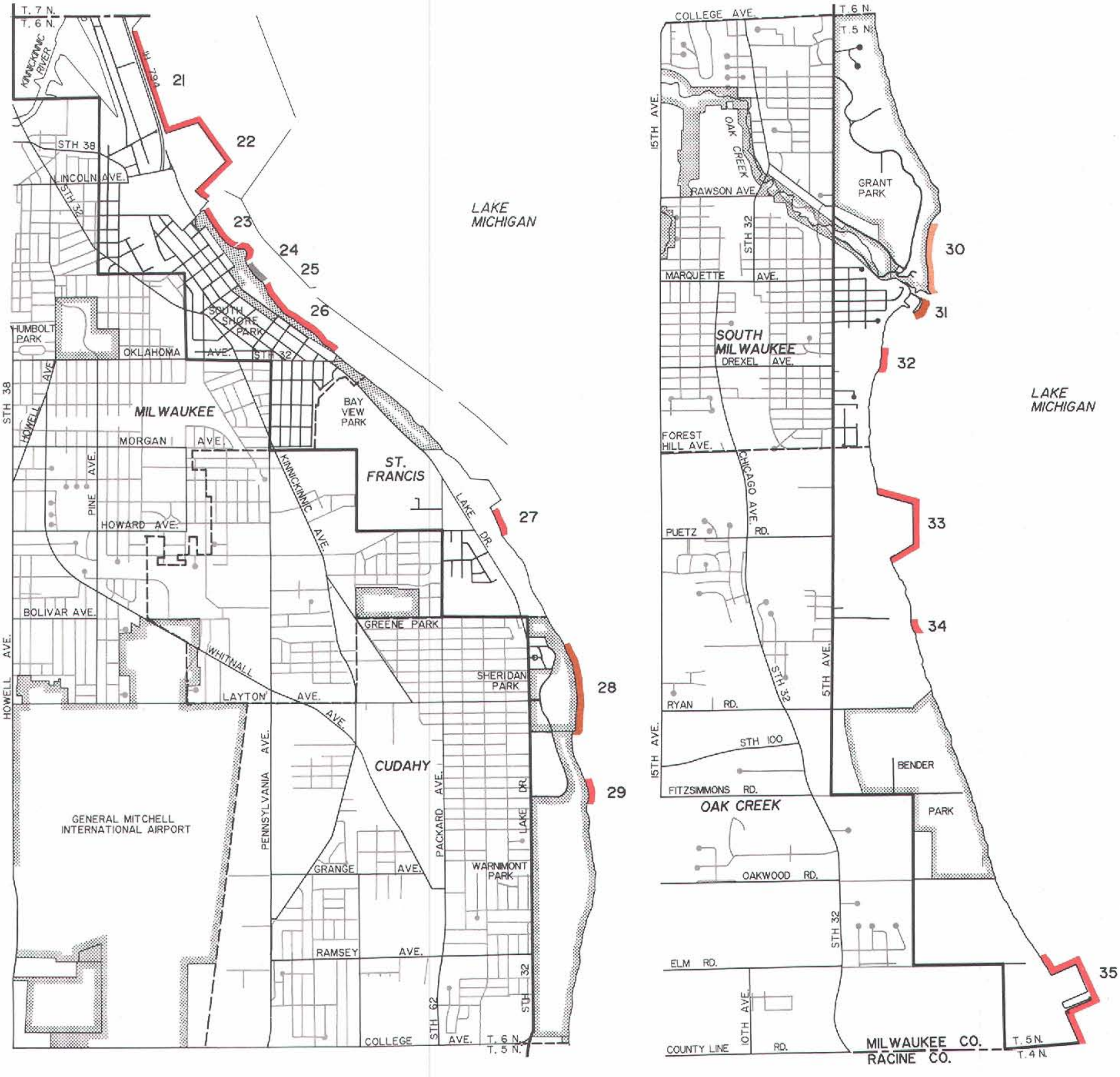
LEGEND

POTENTIAL FOR WAVE OVERTOPPING DAMAGE

- INSIGNIFICANT
- LOW
- MODERATE
- HIGH

1 BAYSIDE BEACH
2 DOCTORS PARK BEACH
3 BEACH DRIVE-NORTH REVETMENT
4 BEACH DRIVE-SOUTH REVETMENT
5 KLODE PARK BREAKWATER/BEACH
6 BIG BAY PARK BULKHEAD
7 BUCKLEY PARK BULKHEAD
8 ATWATER PARK BEACH
9 LINNWOOD AVENUE WATER TREATMENT PLANT BULKHEAD
10 LAKE PARK-NORTH REVETMENT
11 LAKE PARK-SOUTH REVETMENT
12 BRADFORD BEACH
13 MC KINLEY BEACH/REVETMENT
14 JUNEAU PARK LANDFILL BULKHEAD
15 WAR MEMORIAL CENTER BULKHEAD
16 MUNICIPAL PIER BULKHEAD
17 HENRY W. MAIER FESTIVAL GROUNDS REVETMENT
18 MARCUS AMPHITHEATER BULKHEAD
19 MMSD JONES ISLAND WASTEWATER TREATMENT PLANT BULKHEAD
20 PORT OF MILWAUKEE BULKHEAD SLIPS

Map 32 (continued)



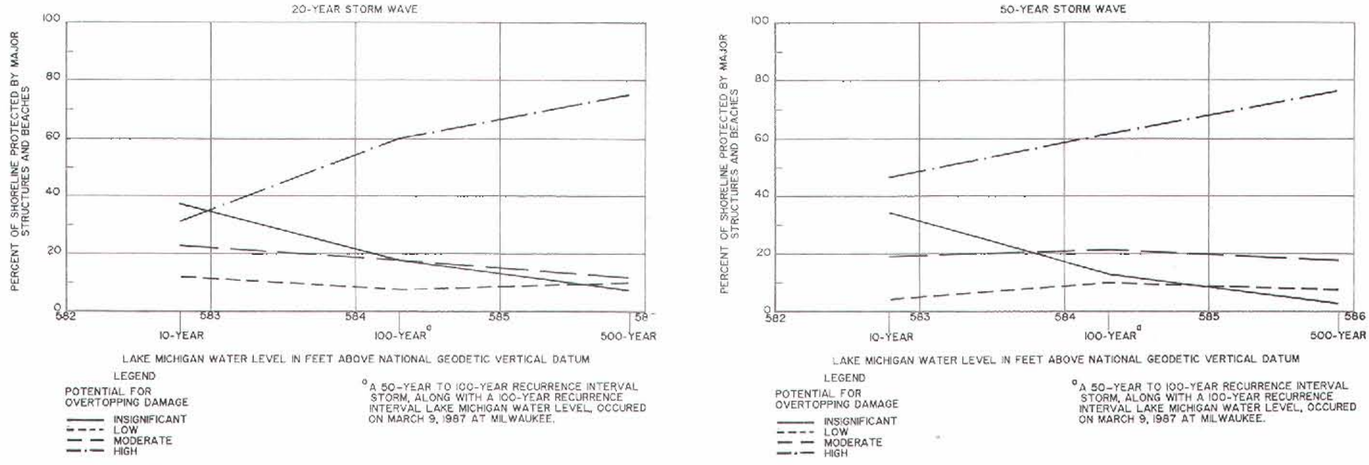
21	SOUTH LINCOLN MEMORIAL DRIVE BULKHEAD
22	USACOE DREDGE SPOILS CONFINED DISPOSAL FACILITY REVETMENT
23	SOUTH SHORE PARK-NORTH REVETMENT
24	SOUTH SHORE YACHT CLUB BULKHEAD
25	SOUTH SHORE PARK BEACH
26	SOUTH SHORE PARK-SOUTH REVETMENT
27	FORMER WEPCO LAKESIDE POWER PLANT SITE REVETMENT
28	SHERIDAN PARK BEACH
29	CUDAHY WATER INTAKE BULKHEAD
30	GRANT PARK BEACH
31	SOUTH MILWAUKEE YACHT CLUB REVETMENT
32	SOUTH MILWAUKEE WASTEWATER TREATMENT PLANT REVETMENT
33	MMSD SOUTH SHORE WASTEWATER TREATMENT PLANT BULKHEAD
34	OAK CREEK WATER INTAKE BULKHEAD
35	WEPCO OAK CREEK POWER PLANT BULKHEAD

GRAPHIC SCALE
0 ¼ ½ MILE
0 2000 4000 FEET

Source: SEWRPC.

Figure 41

PERCENT OF SHORELINE PROTECTED BY MAJOR STRUCTURES AND BEACHES SUBJECT TO POTENTIAL OVERTOPPING DAMAGE UNDER VARIOUS LAKE MICHIGAN WATER LEVEL AND STORM WAVE CONDITIONS



Source: SEWRPC.

Under a 100-year recurrence interval water level with a 50-year recurrence interval storm wave, as shown on Map 31, only one, or 8 percent, of the revetments; six, or 75 percent, of the beaches; and one, or 7 percent, of the bulkheads may be expected to exhibit an insignificant or low potential for wave overtopping damage. The remaining 11, or 92 percent, of the revetments; two, or 25 percent, of the beaches; and 14, or 93 percent, of the bulkheads may be expected to exhibit a moderate or high potential for overtopping damage. Of the total 12.8 miles of Milwaukee County shoreline protected by major structures and beaches, about 12 percent may be expected to exhibit an insignificant potential, 8 percent a low potential, 20 percent a moderate potential, and 60 percent a high potential for overtopping damage.

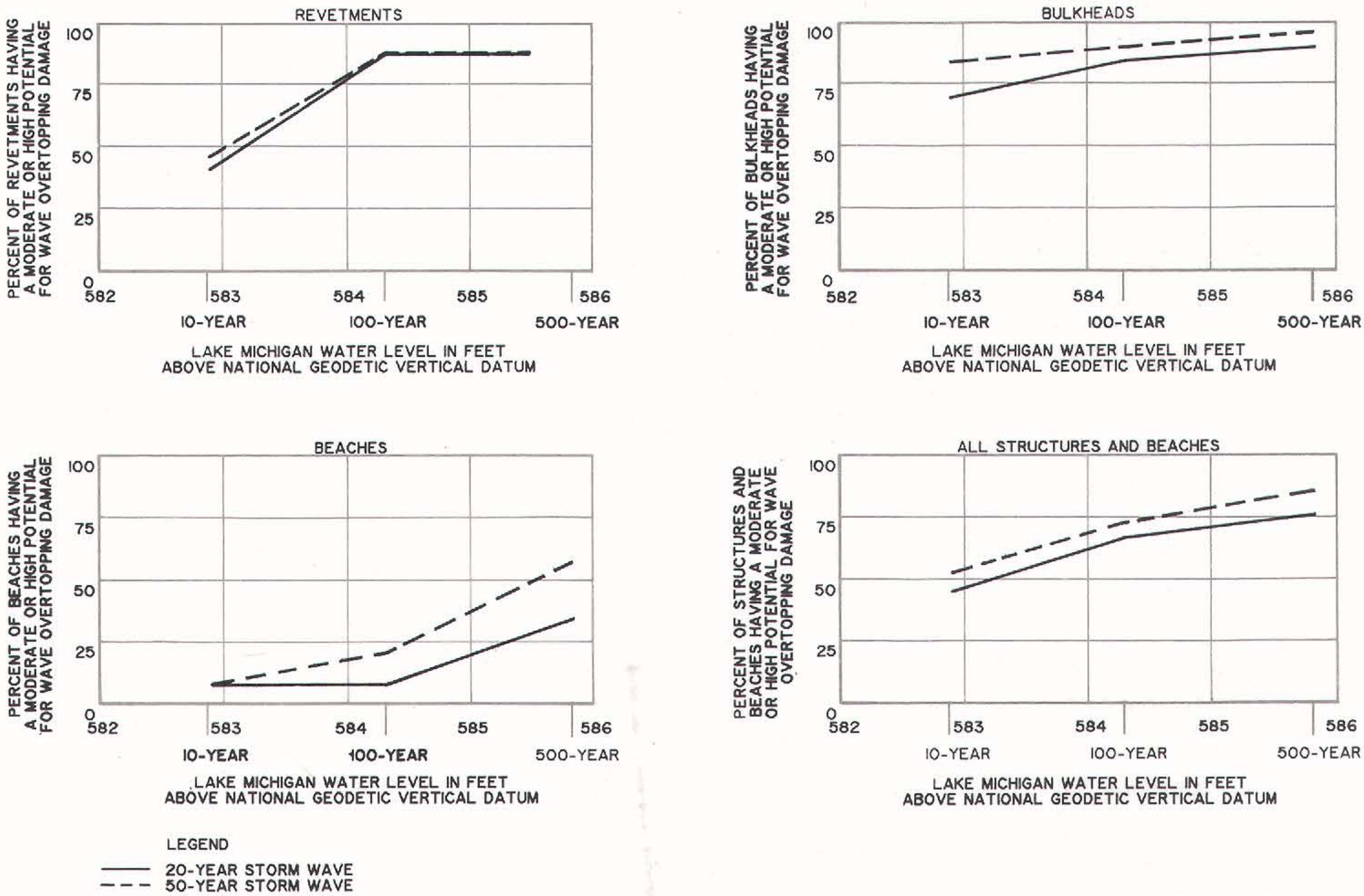
Under a 500-year recurrence interval water level with a 50-year recurrence interval storm wave, as shown on Map 32, only one, or 8 percent, of the revetments; three, or 38 percent, of the beaches; and none of the bulkheads may be expected to exhibit an insignificant or low potential for wave overtopping damage. The remaining 11, or 92 percent, of the revetments; five, or 62 percent, of the beaches; and all 15 of the bulkheads may be expected to exhibit a moderate or high potential for overtopping damage. Of the total 12.8 miles of Milwaukee County shoreline protected by major structures and beaches, only 1 percent may be expected to exhibit an insignificant potential, 7 percent a low potential, 17 percent a moderate potential, and 75 percent a high potential for overtopping damage.

The percent of the Lake Michigan shoreline protected by the major structures and beaches that could be damaged by wave overtopping is graphically illustrated in Figure 41. The figure shows, as may be expected, that the degree of overtopping would be greater under a 50-year storm wave than under a 20-year storm wave. The height of the deep-water storm wave, however, was found to be of somewhat limited importance because the near-shore waves are often water depth-limited. The water levels were found to have a greater impact on overtopping than the height of the deep-water storm waves.

Figure 42 summarizes the percent of the major structures and beaches having a high or moderate potential for wave overtopping damage. Beaches are expected to suffer the least overtopping damage, with bulkheads most prone to such damage. In general, most bulkheads may be expected to be damaged under even a 10-year water level; overtopping damage may be expected to affect most revetments under a 100-year water level; and substantial overtopping damage to beaches should not occur until a 500-year water level is reached. Overall, from 49 to

Figure 42

PERCENT OF MAJOR SHORE PROTECTION STRUCTURES AND BEACHES HAVING A MODERATE OR HIGH POTENTIAL FOR WAVE OVERTOPPING DAMAGE



Source: SEWRPC.

57 percent of the structures and beaches may be expected to exhibit a high or moderate potential for overtopping damage under a 10-year water level, from 71 to 77 percent under a 100-year water level, and from 80 to 89 percent under a 500-year water level.

Based on the wave analyses, it appears that those structures currently offering the greatest protection against wave action are the Milwaukee County Grant Park and South Shore beaches, and the newly constructed Milwaukee County McKinley Beach/revetment and Village of Whitefish Bay Klode Park breakwater/beach. The latter two structures were designed and constructed after the 1986 high water levels with the aid of physical hydraulic models. Other beaches—most notably Milwaukee County Bradford beach and the Village of Shorewood Atwater Park beach—also offer excellent protection, but may be damaged by extremely high—500-year—water levels. The bulkhead offering the greatest protection appears to be the newly constructed Milwaukee Metropolitan Sewerage District Jones Island wastewater treatment plant bulkhead, also designed with the aid of a physical hydraulic model.

Although the probability of a very large storm occurring simultaneously with a very high lake level is extremely low, such conditions do indeed occur, and in fact have been recorded. Large storms which produce high waves may also create a large storm seiche and wind setup, which temporarily increases the water level. Such a condition occurred in Milwaukee on March 9, 1987. A severe storm produced a seiche and wind setup of 2.5 feet, increasing the lake level to a record high of 584.3 feet NGVD, which is the 100-year recurrence interval instantaneous maximum water level. A hindcast wave analysis indicated that the waves produced by this storm had a recurrence interval of between 50 and 100 years.[38]

Structural damages that could result from low Lake Michigan water levels were also evaluated. An underwater inspection conducted under this study indicated that the Milwaukee County War Memorial Center bulkhead rests on timber pilings which may be exposed to the atmosphere at the 100-year recurrence interval minimum monthly mean water level of 575.5 feet NGVD. Exposure of the timber could accelerate its decomposition. No other major structures in the County have known timber pilings which would be exposed by the minimum monthly mean water level. However, about 40 percent of the major structures in the County could be affected by toe erosion damage and bottom scouring under extremely low water levels. Increased toe scourings of the 14 structures listed in Table 41 and shown on Map 33 could occur under low water conditions as the waves break at the base of structures that are normally submerged. These structures include nine revetments, or 75 percent of all of the revetments in the County; and five bulkheads, or 33 percent of all of the bulkheads in the County. The bases or bottoms of these structures are higher than the 100-year recurrence interval instantaneous minimum water level of 574.9 feet NGVD. Thus, under extreme low-water-level conditions wave action could undercut these structures.

Table 41

MAJOR SHORE PROTECTION STRUCTURES IN MILWAUKEE COUNTY WHICH MAY BE DAMAGED BY INCREASED TOE EROSION AND BOTTOM SCOURING UNDER EXTREMELY LOW LAKE MICHIGAN WATER LEVELS

Structure
1. Village of Fox Point Beach Drive-North Revetment
2. Village of Fox Point Beach Drive-South Revetment
3. Milwaukee County Big Bay Park Bulkhead
4. Village of Whitefish Bay Buckley Park Bulkhead
5. Milwaukee County Lake Park-North Revetment
6. Milwaukee County Lake Park-South Revetment
7. Milwaukee County McKinley Beach/Revetment
8. Milwaukee County War Memorial Center Bulkhead
9. Milwaukee County South Shore Park-North Revetment
10. Milwaukee County South Shore Park-South Revetment
11. City of Cudahy Water Intake Bulkhead
12. South Milwaukee Yacht Club Revetment
13. South Milwaukee Wastewater Treatment Plant Revetment
14. City of Oak Creek Water Intake Bulkhead

NOTE: Wave action could erode the bases or bottoms of these structures, which lie above the 100-year recurrence interval instantaneous minimum water level of 574.9 feet NGVD.

Source: SEWRPC.

Evaluation of Bluff Analysis Sections

The following section describes for each bluff analysis section the general condition of the bluff slope and of any major shore protection structures, and the degree of shoreline erosion observed. The condition of the bluff slope was indicated by the results of the deterministic and probabilistic slope stability analyses and field observations made in 1986 and 1987. For sections with unstable or marginal bluff slopes, those measures needed to fully stabilize the slopes are identified.

Bluff Analysis Section 1: Bluff Analysis Sections 1 through 13 lie within the City of Oak Creek, as shown in Figure 43. The entire shoreline of Section 1 is located on the Wisconsin Electric Power Company Oak Creek plant site in the City of Oak Creek. The natural bluff, which is set back approximately 300 feet from the water's edge, has been regraded to a stable slope. Slope stability analyses were not conducted for this section because the stable bluff is located behind the power plant facilities.

[38]*J. Philip Keillor, Coastal Engineer, Sea Grant Institute, University of Wisconsin-Madison, letter to Mr. David B. Kendziorski, Southeastern Wisconsin Regional Planning Commission, May 5, 1989.*

The shoreline of the power plant is protected by two steel sheet pile bulkheads with riprap toe protection. The lakeward bulkhead was observed to be bowing in both directions during the field survey conducted in the spring of 1988, although there was no evidence that major structure failure was imminent. Scour at the base of the riprap which fronts about 80 percent of the bulkhead was also observed during the field survey.

No measures are needed to prevent rotational or translational sliding within Section 1. It is recommended that a site-specific analysis be conducted to determine the structural integrity of the bulkhead. Adequate toe erosion control measures should be provided to protect the plant and prevent erosion from wave and ice action.

Bluff Analysis Section 2: The stability of the bluff slope within Section 2, located within the undeveloped portion of the Wisconsin Electric Power Company property north of the plant, and extending from Elm Road to Oakwood Road in the City of Oak Creek, was characterized by the use of Profile Nos. 1, 2, and 3.

The results of the deterministic slope stability analyses, shown in Figure 44 for Profile Nos. 1, 2, and 3, indicate that portions of the bluff slope within Section 2 are just barely stable with respect to rotational sliding. The lowest failure surface at Profile No. 1 had a safety factor of 1.0, and was located within the upper two-thirds of the bluff. The next nine lowest safety factors ranged from 1.02 to 1.16. The lowest failure surface calculated at Profile No. 2 had a safety factor of 1.43, and included the entire bluff. The next nine lowest safety factors ranged from 1.43 to 1.58. The lowest failure surface calculated at Profile No. 3 had a safety factor of 1.18, and was located within the mid-section of the bluff. The next nine lowest safety factors ranged from 1.25 to 1.34.

The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 1 ranged from 0.73 to 1.37, with 11, or 55 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated at Profile No. 1, 87, or 43 percent, had safety factors of less than 1.0. The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 2 ranged from 0.79 to 1.36, with 9, or 45 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated at Profile No. 2, 64, or 32 percent, had safety factors of less than 1.0. The lowest safety factors indicated by the 20 probabilistic analyses conducted for Profile No. 3 ranged from 0.89 to 1.62, with two, or 10 percent, having safety factors of less than 1.0. Of the 200 failure surfaces evaluated at Profile No. 3, two, or 1 percent, had safety factors of less than 1.0.

In field surveys conducted in the fall of 1987, shallow slides and solifluction were observed in the southern end of Section 2. In the northern end of the section, the edge of the bluff was scalloped and interrupted by several ravines, indicating past bluff slope failures and channeling of surface water runoff lakeward. Based on a review of deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 2 was considered to have a marginal bluff slope with respect to rotational sliding.

Section 2 was considered to have an unstable slope with respect to translational sliding. In the southernmost end of the section, steep slope angles were considered to be the major factor causing translational slope failure. In the northern two-thirds of the section, the lower segment of the bluff slope was unstable, despite relatively good vegetative cover. Groundwater seepage observed during the 1987 field survey was considered to be the major cause of this slope failure.

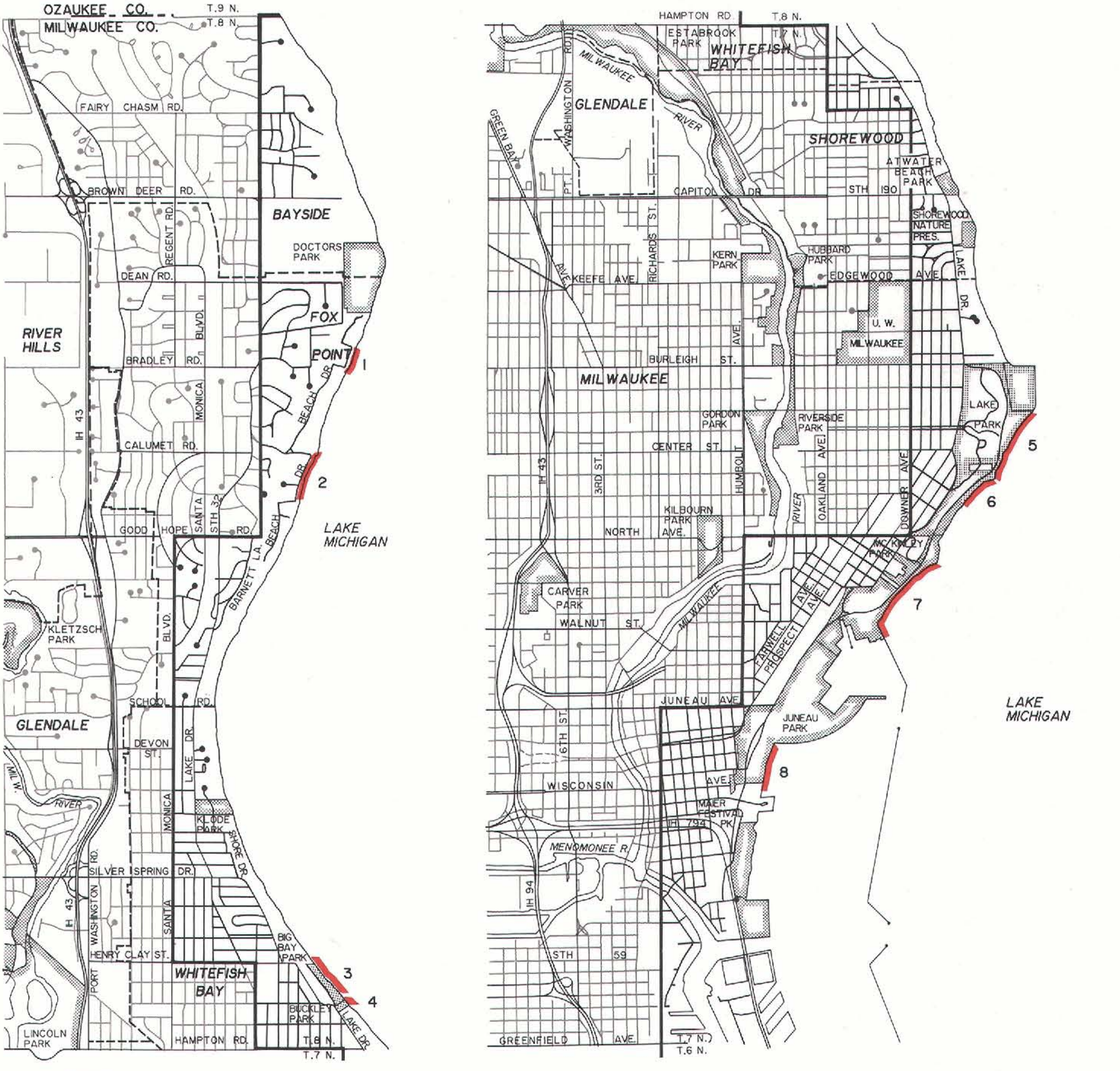
No significant toe erosion was observed in the southern two-thirds of this section during the 1987 field survey. The toe of the bluff was protected by a terrace and by a relatively wide sand beach which had accumulated to the north of the WEPCo Oak Creek power plant bulkhead. In the northern one-third of the section, toe erosion was observed, but such erosion did not appear to be threatening the stability of the overall bluff slope. No shore protection structures were located within this section in 1987.

To prevent rotational and translational sliding, it is recommended that a groundwater drainage system be installed and that surface water runoff be controlled. It does not appear necessary at this time to provide additional protection against wave and ice action at the toe of the bluff.

Bluff Analysis Section 3: The stability of the bluff slope within Section 3, located in Bender Park in the City of Oak Creek (between Oak-

Map 33

SHORE PROTECTION STRUCTURES WHICH MAY BE DAMAGED UNDER EXTREMELY LOW LAKE MICHIGAN WATER LEVELS



LEGEND

1	BEACH DRIVE-NORTH REVETMENT
2	BEACH DRIVE-SOUTH REVETMENT
3	BIG BAY PARK BULKHEAD
4	BUCKLEY PARK BULKHEAD
5	LAKE PARK-NORTH REVETMENT
6	LAKE PARK-SOUTH REVETMENT
7	MC KINLEY BEACH/REVETMENT
8	WAR MEMORIAL CENTER BULKHEAD

Map 33 (continued)

9 SOUTH SHORE PARK-NORTH REVETMENT

10 SOUTH SHORE PARK-SOUTH REVETMENT

11 CUDAHY WATER INTAKE BULKHEAD

12 SOUTH MILWAUKEE YACHT CLUB REVETMENT

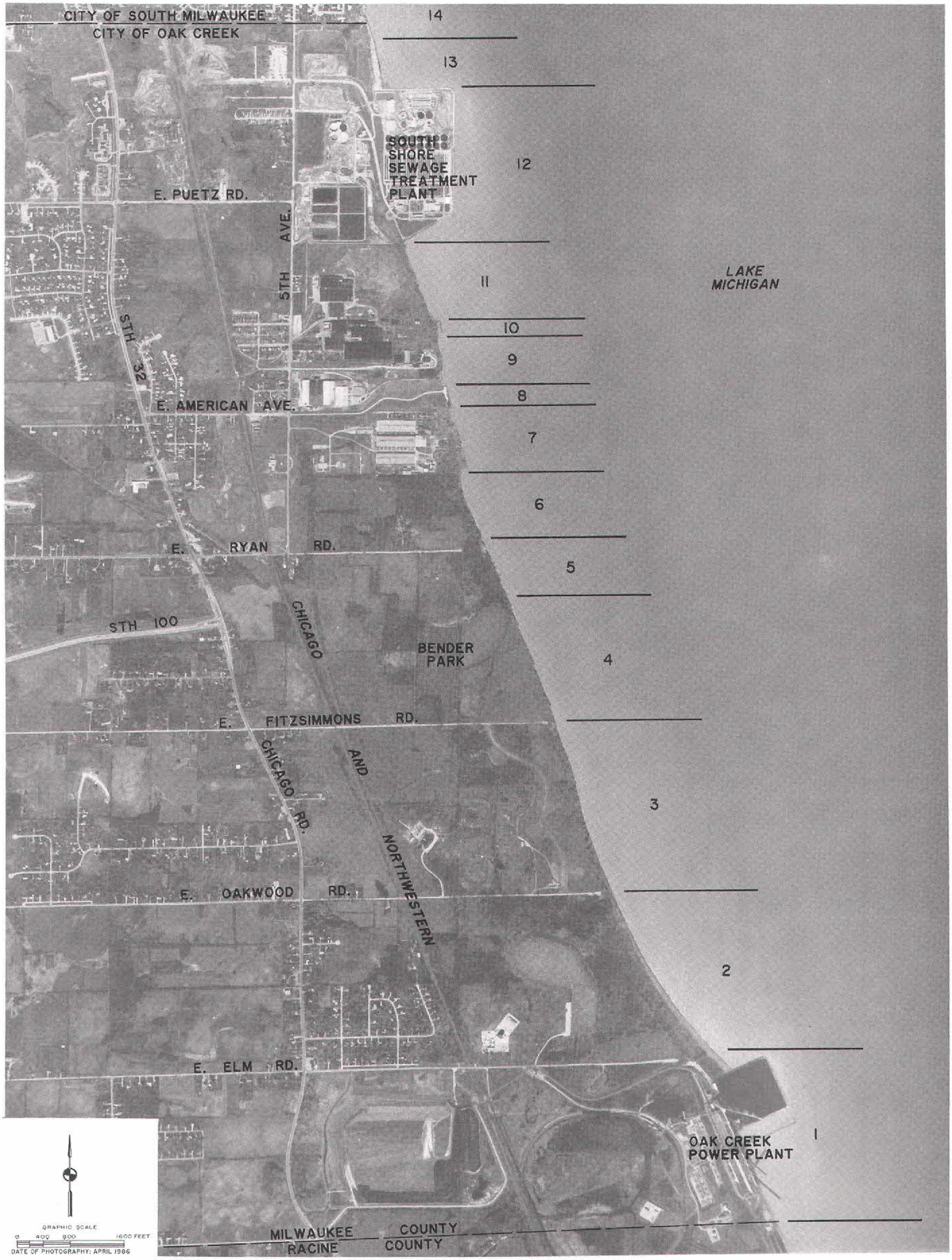
13 SOUTH MILWAUKEE WASTEWATER TREATMENT PLANT REVETMENT

14 OAK CREEK WATER INTAKE BULKHEAD

Source: SEWRPC.

Figure 43

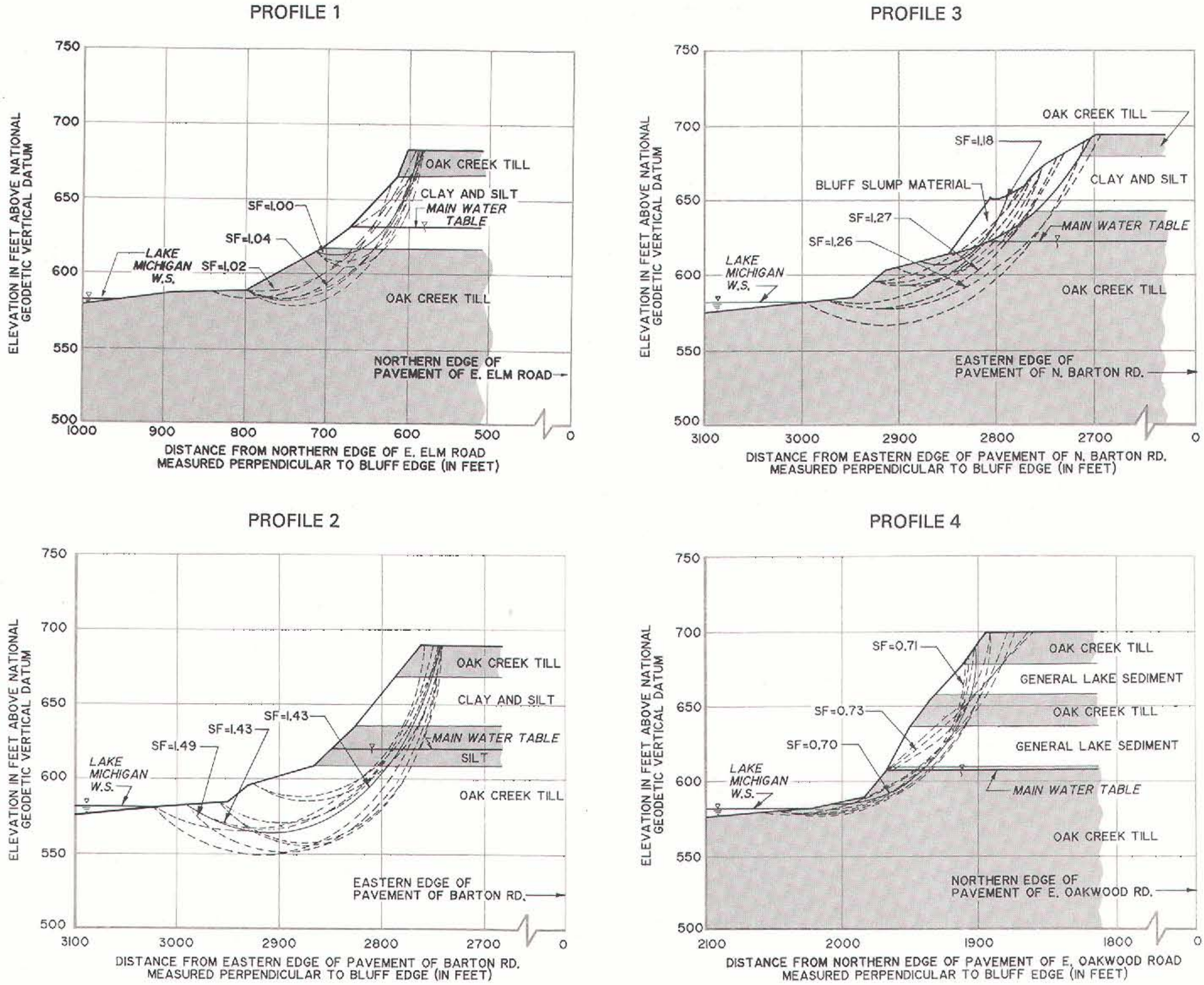
BLUFF ANALYSIS SECTIONS WITHIN THE CITY OF OAK CREEK



Source: SEWRPC.

Figure 44

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSES FOR PROFILES 1 THROUGH 4



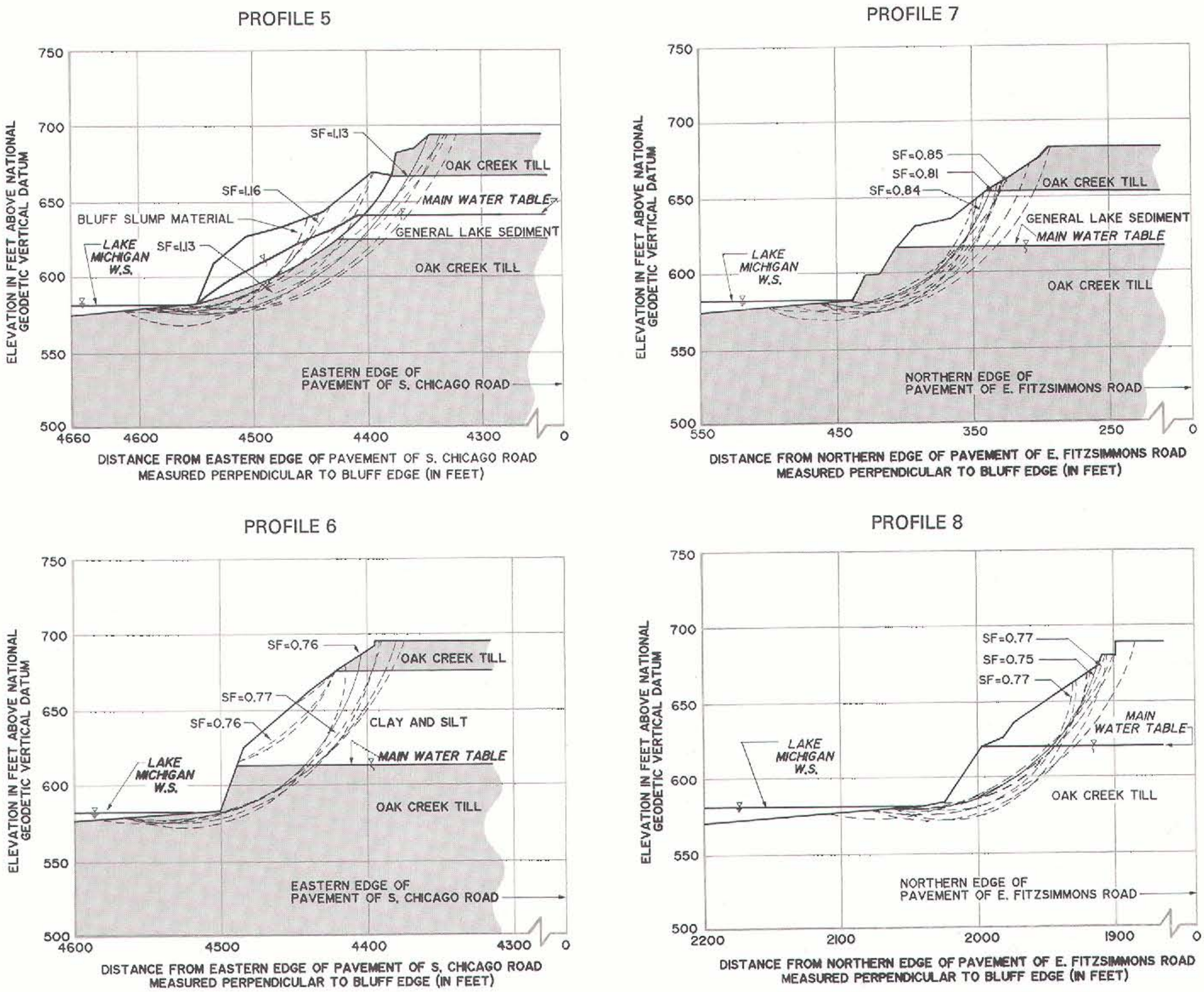
Source: T. B. Edil, D. M. Mickelson, and SEWRPC.

wood Road and Fitzsimmons Road extended), was characterized by the use of Profile Nos. 4, 5, and 6.

The results of the deterministic slope stability analyses, shown in Figures 44 and 45 for Profile Nos. 4 and 6, respectively, indicate that the bluff slope in Section 3 is unstable with respect to rotational sliding. The lowest failure surface calculated at Profile No. 4 had a safety factor of 0.70 and included the entire bluff slope. The next nine lowest safety factors ranged from 0.71 to 0.75. The lowest failure surface calculated at Profile No. 6 had a safety factor of 0.76 and was located within the lower portion of the bluff slope. The next nine lowest safety factors ranged from 0.76 to 0.83. The results of the deterministic slope stability analyses for Profile No. 5, shown in Figure 45, a site of recent slumping, indicated that the slope was stable with respect to rotational sliding. The lowest failure surface calculated at Profile No. 5 had a safety factor of 1.13 and included the entire bluff face. The next nine lowest safety factors ranged from 1.13 to 1.19. A probabilistic slope stability analysis was not conducted for this section because the bluff slope

Figure 45

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSES FOR PROFILES 5-8



Source: T. B. Edil, D. M. Mickelson, and SEWRPC.

was considered to be obviously unstable based on the field observations and the results of the deterministic slope stability analyses.

Overall, Section 3 was also considered to have an unstable bluff slope with respect to translational sliding. This may be attributed to the lack of vegetative cover on the bluff face, the steepness of the bluff slope, and the accumulation of stormwater runoff. Groundwater seepage in the lower segment of the slope also contributes to the overall instability.

Toe erosion contributing to the instability of the bluff slope was observed along the entire shoreline of Section 3 during the field surveys conducted during the fall of 1987. No shore protection structures were located within this section in 1987.

In order to fully stabilize the bluff in this section, it is recommended that the bluff slope be regraded to a stable slope angle and revegetated. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 4: The stability of the bluff slope within Section 4, located in Bender Park in the City of Oak Creek (north of Fitzsimmons Road extended), was characterized by the use of Profile Nos. 7 and 8.

The results of the deterministic slope stability analyses, shown in Figure 45 for Profile Nos. 7 and 8, indicate that the bluff slope in Section 4 is unstable with respect to rotational sliding. The lowest failure surface calculated at Profile No. 7 had a safety factor of 0.81 and included the lower two-thirds of the bluff. The next nine lowest safety factors ranged from 0.84 to 0.93. The lowest failure surface calculated at Profile No. 8 had a safety factor of 0.75 and also included the lower portion of the bluff. The next nine lowest safety factors ranged from 0.77 to 0.84. A probabilistic slope stability analysis was not conducted for this section because the bluff slope was considered to be obviously unstable based on the field observations and the results of the deterministic slope stability analyses.

Overall, Section 4 was also considered to have an unstable slope with respect to translational sliding. Lack of vegetation on the bluff slope, steep slope angles, and groundwater seepage in the lower portion all contributed to slope instability.

Bluff toe erosion was observed along the entire shoreline of Section 4 during the field surveys conducted during the fall of 1987. This erosion was contributing significantly to the instability of the bluff slope. No shore protection structures were located within the section in 1987.

In order to fully stabilize the bluff slope in this section, it is recommended that the bluff slope be regraded to a stable slope angle and revegetated. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 5: The stability of the bluff slope within Section 5, located in Bender Park in the City of Oak Creek (south of Ryan Road extended), was characterized by the use of Profile No. 9.

The results of the deterministic slope stability analysis, shown in Figure 46, indicate that Profile No. 9 had an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.79, and was located within the upper two-thirds of the bluff slope. The next nine lowest safety factors ranged from 0.89 to 1.03.

The lowest safety factors indicted by the 20 probabilistic stability analyses conducted were all less than 1.0. Of the 200 failure surfaces evaluated, 177 surfaces, or 88 percent, had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 5 was considered to have an unstable bluff slope with respect to rotational sliding.

Section 5 was also considered to have an unstable slope with respect to translational sliding. This was due in part to the lack of vegetative cover on most of the bluff slope, and in part to the relatively steep angle of the bluff slope.

Bluff toe erosion was observed within the entire shoreline of Section 5 during the 1987 field survey. This erosion was affecting the stability of the slope. As of 1987, no shore protection structures were located within this section.

In order to fully stabilize the bluff slope in this section, it is recommended that the bluff slope be regraded to a stable slope angle and revegetated. Bluff toe protection is recommended to prevent erosion from wave and ice action.

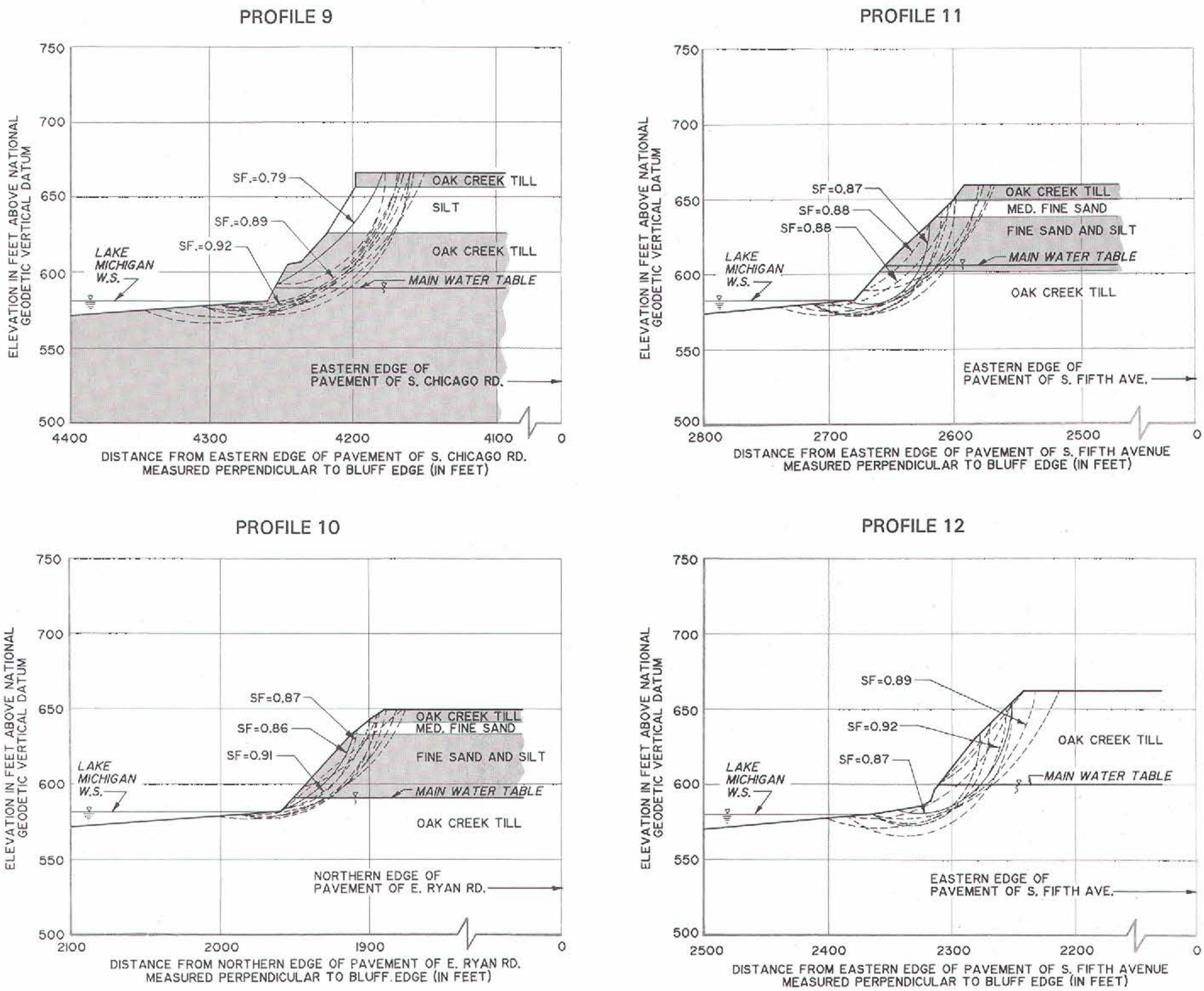
Bluff Analysis Section 6: The stability of the bluff slope within Section 6, located at 9300 S. 5th Avenue in the City of Oak Creek (north of Ryan Road extended), was characterized by the use of Profile No. 10.

The results of the deterministic slope stability analysis, shown in Figure 46, indicate that Profile No. 10 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.86, and was located within the lower two-thirds of the bluff slope. The next nine lowest safety factors ranged from 0.87 to 0.99.

The lowest safety factors indicated by the 20 probabilistic stability analyses ranged from 0.65 to 1.03, with 19 failure surfaces, or 95 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated, 159, or 79 percent of the surfaces, had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 6 was considered to have an unstable bluff slope with respect to rotational sliding.

Figure 46

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSES FOR PROFILES 9-12



Source: T. B. Edil, D. M. Mickelson, and SEWRPC.

Section 6 was also considered to have an unstable slope with respect to translational sliding. During the field surveys conducted during the fall of 1987, numerous shallow slides were observed. The bluff slope was steep and mostly unvegetated. These conditions contributed significantly to the instability of the bluff in this section.

Bluff toe erosion was observed in the entire shoreline of Section 6 and was identified as a primary cause of bluff slope failure. As of 1987, no shore protection structures were located within this section.

In order to fully stabilize the bluff in Section 6, it is recommended that the bluff slope be regraded to a stable slope angle and revegetated. Investigations may be needed to determine whether toxic substances are present in the bluff and whether bluff slope stabilization measures could have an adverse effect on the ecology of near-shore Lake Michigan. Bluff toe protection is recommended to prevent erosion from wave and ice action.

<u>Bluff Analysis Section 7</u>: The stability of the bluff slope underlying the concrete slab fill

within Section 11, located at 9180 S. 5th Avenue in the City of Oak Creek (south of Dexter Avenue extended), was characterized by Profile No. 11.

The results of the deterministic slope stability analyses, shown in Figure 46, indicate that Profile No. 11 has an unstable slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.87. This surface was located within the lower two-thirds of the slope. The next nine lowest safety factors ranged from 0.88 to 1.03.

All of the 200 failure surfaces evaluated with the probabilistic model had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses and on the observed bluff conditions, Section 7 was considered to have an unstable bluff slope with respect to rotational sliding.

Section 7 was also considered unstable with respect to translational sliding. Sliding was observed at the top of the fill area during the field survey conducted in the fall of 1987. Bluff slopes were generally steep throughout both the filled and natural slope areas. These conditions, coupled with the lack of vegetation on the bluff slope, contributed to the instability of the bluff.

No significant toe erosion was observed in Section 7 during the field survey. Toe protection along the entire shoreline of this section was provided by a concrete slab revetment.

Several small ponds and settling basins were located on top of the bluff near the southern end of Section 7. The effect of the ponds on the hydrogeology of the bluff cannot be determined without more detailed, site-specific inventories and analyses. Because the elevation of the water table is a critical factor in assessing bluff stability and because surface water bodies can affect the position of the water table, a detailed groundwater investigation of this site is recommended.

To prevent rotational and translational sliding in Section 7, it is recommended that the bluff slope be regraded to a stable slope angle and revegetated. Investigations may be needed to determine whether toxic substances are present in the bluff and whether bluff slope stabilization measures could have an adverse effect on the ecology of near-shore Lake Michigan. Adequate toe erosion control measures should be provided to prevent erosion from wave and ice action.

<u>Bluff Analysis Section 8</u>: The shoreline of Section 8 is located at 9170 S. 5th Avenue, and includes the City of Oak Creek water intake. The natural bluff has been regraded to provide an access road to the intake plant. No erosion of the bluff was observed during the field survey conducted during the fall of 1987. The slope stability analyses were not conducted for this section because the bluff slope was considered to be stable based upon the field observations.

The shoreline of the water intake plant is protected by a bulkhead constructed of steel sheet piling reinforced with a concrete wall and quarrystone. At the time of the field survey, the structure appeared to be well maintained.

No measures are needed to prevent rotational or translational sliding within Section 8. Adequate toe erosion control measures should be provided to protect the facility and prevent erosion from wave and ice action.

<u>Bluff Analysis Section 9</u>: The stability of the bluff slope within Section 9, located at 4301 E. Depot Road in the City of Oak Creek, was characterized by the use of Profile No. 12.

The results of the deterministic slope stability analysis, shown in Figure 46, indicate that Profile No. 12 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.87, and was located within the lower two-thirds of the bluff slope. The next nine lowest safety factors ranged from 0.94 to 1.07.

All of the lowest safety factors indicated by the 20 probabilistic stability analyses were less than 1.0. Of the 200 safety factors evaluated, 170, or 85 percent were less than 1.0. Based on both the probabilistic slope stability analyses and on the observed bluff conditions, Section 9 was considered to have an unstable bluff slope with respect to rotational sliding.

Section 9 was considered to have a marginal bluff slope with respect to translational sliding. In the northern part of the section, solifluction and numerous shallow slides were observed during the field survey conducted during the fall of 1987. Lack of vegetative cover and moderately steep slopes were considered to be the primary influences on slope failure by translational sliding in this section.

Significant toe erosion was observed within the entire shoreline of Section 9 during the field survey. The Peter Cooper Company breakwater, a low, quarrystone structure built in the early 1900's—approximately 950 feet in length and located approximately 200 feet offshore of Sections 9 and 10—was submerged throughout Section 9. A rubble revetment was also located in the southern third of this section. While these structures provided some protection, there was continued erosion by waves washing over the top of the structures.

In order to fully stabilize the bluff in this section, the bluff slope should be regraded to a stable slope angle and revegetated. Investigations may be needed to determine whether toxic substances are present in the bluff and whether stabilization measures could have an adverse effect on the ecology of near-shore Lake Michigan. Bluff toe protection should be provided to prevent erosion from wave and ice action.

Bluff Analysis Section 10: The stability of the bluff slope within Section 10, located at 9006 S. 5th Avenue (north of E. Lakeside Street extended), was characterized by the use of Profile No. 13.

The results of the deterministic slope stability analysis, shown in Figure 47 for Profile No. 13, indicate a threat of bluff slope failure with respect to rotational sliding. The lowest failure surface calculated had a safety factor of 0.92, and occurred within the lower two-thirds of the slope. The next nine lowest safety factors ranged from 0.98 to 1.06.

The lowest safety factors indicated by the 20 probabilistic stability analyses ranged from 0.70 to 1.14, with 14 of the failure surfaces, or 70 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated, 91, or 45 percent, had safety factors of less than 1.0.

During the field surveys conducted in the fall of 1987, several exposed soil areas on an otherwise well-vegetated slope were observed, especially in the northern end of Section 10. In the southern end of the section, the slope had been regraded and partially covered with industrial debris and waste material. Based on both the deterministic and probabilistic slope stability analyses and on the observed bluff conditions, Section 10 was considered to have a marginal bluff slope with respect to rotational sliding.

Section 10 was also considered to have a marginal slope with respect to translational sliding. The base of the bluff had good vegetative cover and a gentle to moderate slope angle. Soil creep and solifluction were active in the upper portion of the bluff slope. Numerous disturbed soil areas were observed and the slope angle was generally greater than 40 degrees. Therefore, the potential for translational sliding was far greater in the upper portion of the bluff slope than in the lower bluff slope.

Bluff toe erosion observed within the entire section during the 1987 field survey was considered a significant threat to bluff stability. The Peter Cooper Company breakwater, a low quarrystone structure—approximately 950 feet in length and located about 200 feet offshore of Sections 9 and 10—was ineffective in preventing shoreline erosion in the southern end of Section 10, where the breakwater was submerged. Where the structure was emergent at the northern end of the section, the breakwater was marginally effective in preventing shoreline erosion.

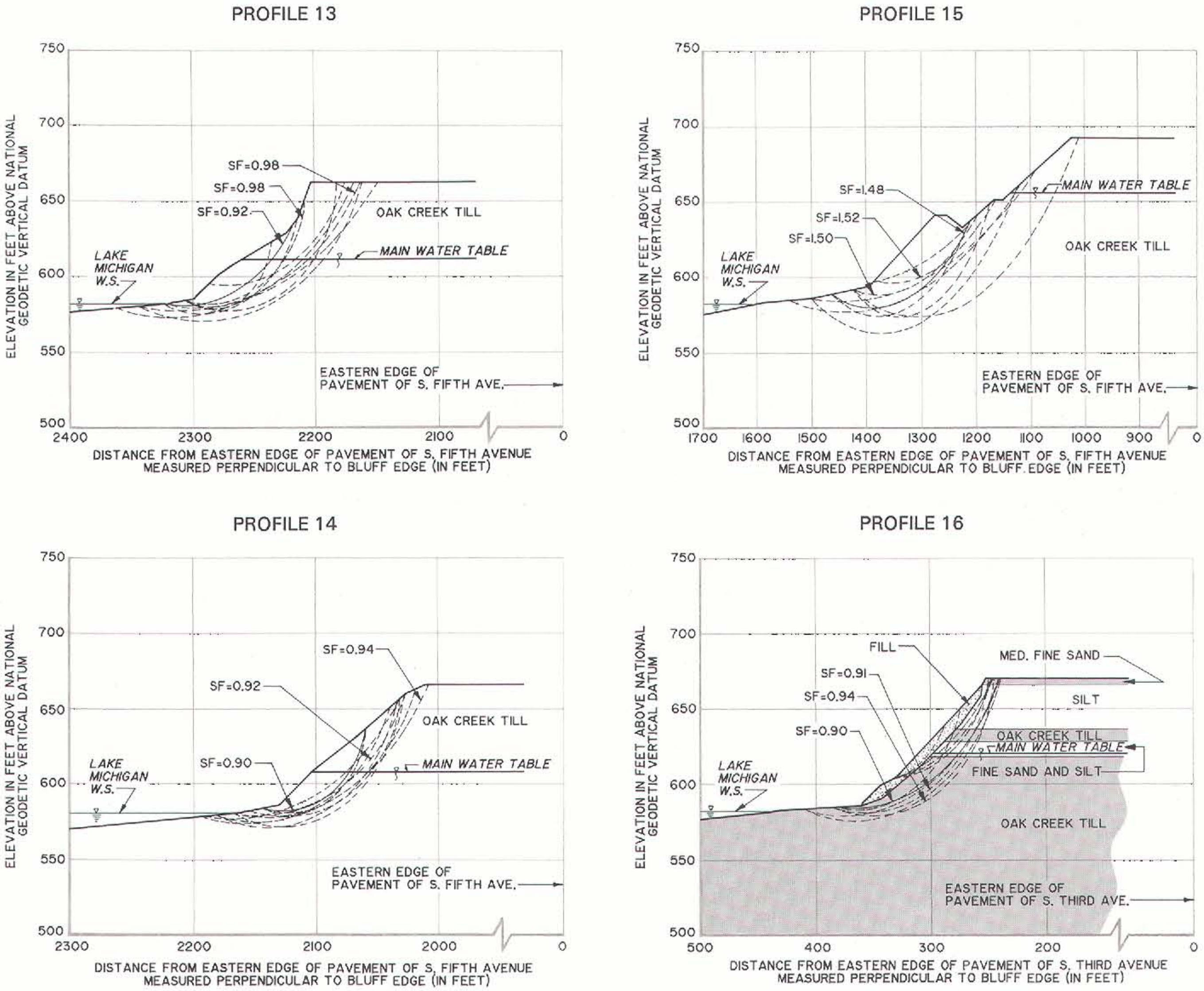
Regrading of the bluff slope and revegetation is recommended to help stabilize the slope. Investigations may be needed to determine whether toxic substances are present in the bluff and whether bluff slope stabilization measures could have an adverse effect on the ecology of near-shore Lake Michigan. Bluff toe protection is also recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 11: The stability of the bluff slope within Section 11, which extends from 9006 to 8740 S. 5th Avenue (south of Puetz Road extended), was characterized by the use of Profile No. 14.

The results of the deterministic slope stability analysis, shown in Figure 47, indicate that Profile No. 14 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.90, and was located within the lower two-thirds of the bluff slope. The next nine lowest safety factors ranged from 0.92 to 1.02.

Figure 47

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSES FOR PROFILES 13-16



Source: T. B. Edil, D. M. Mickelson, and SEWRPC.

The lowest safety factors indicated by the 20 probabilistic stability analyses ranged from 0.51 to 1.01, with 18 of the failure surfaces, or 90 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated, 157 surfaces, or 78 percent, had safety factors of less than 1.0.

During the field survey conducted during the fall of 1987, recent shallow slides were observed throughout the section. Based on a review of the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 11 was considered to have an unstable bluff slope with respect to rotational sliding.

Section 11 was considered to have a marginal bluff slope with respect to translational sliding. The base of the bluff was partially vegetated with a slope angle of about 40 degrees. The middle and upper portions of the bluff were moderately vegetated, and had a slope angle of about 25 degrees. The potential for translational sliding is therefore greater in the lower portion of the slope, as field observations verified.

Bluff toe erosion was observed in the entire shoreline of Section 11 and was identified as a contributing factor to bluff slope instability. No shore protection structures were located in this section as of 1986.

In order to fully stabilize the bluff slope in this section, it is recommended that the bluff slope be regraded to a stable slope angle and revegetated. Investigations may be needed to determine whether toxic substances are present in the bluff and whether bluff slope stabilization measures could have an adverse effect on the ecology of near-shore Lake Michigan. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 12: The entire shoreline of Section 12 is located at the Milwaukee Metropolitan Sewerage District South Shore wastewater treatment plant site in the City of Oak Creek. The natural bluff, which is set back behind the lower level plant facilities, has been regraded. Slope stability analyses were not conducted for this section because the bluff is stable and is protected by a major lakefront facility.

The shoreline is protected by multiple steel sheet pile bulkheads comprised of two walls with riprap toe protection. Although the bulkhead was reported to be in relatively good condition during a 1988 field inspection, evidence of overtopping and toe erosion was noted.

No measures are needed to prevent rotational or translational sliding within Section 12. Adequate toe erosion control measures should be maintained to protect the plant facilities and prevent erosion from wave and ice action.

Bluff Analysis Section 13: The stability of the bluff slope within Section 13, located at 8400 S. 5th Avenue in the City of Oak Creek (south of Edgewood Avenue extended), was characterized by the use of Profile No. 15.

The results of the deterministic slope stability analysis, shown in Figure 47, indicate that Profile No. 15 has a stable slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 1.48 and was located within the lower two-thirds of the bluff slope. The next nine lowest safety factors ranged from 1.50 to 1.75. A probabilistic slope stability analysis was not conducted for this section because the bluff slope was considered to be stable based on field observations and the results of the deterministic slope stability analyses.

Section 13 was also considered to have a stable slope with respect to translational sliding. The bluff slope is vegetated and terraced, with an average slope angle of 20 degrees.

No toe erosion was observed in Section 13 during the field survey conducted during the fall of 1987. The toe of the bluff was protected by a relatively wide sand beach which had accumulated to the north of the Milwaukee Metropolitan Sewerage District South Shore wastewater treatment plant bulkhead. No shore protection structures were located in Section 13 as of 1986.

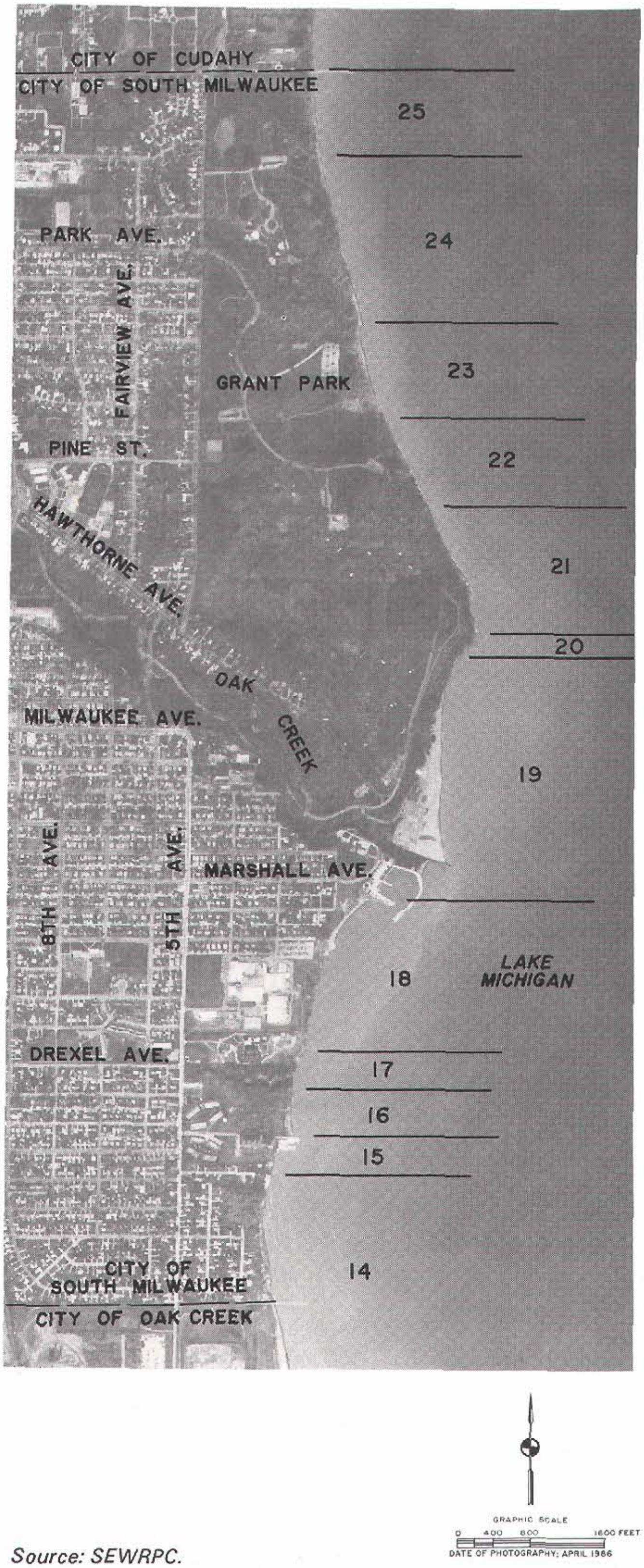
No measures are needed to prevent rotational or translational sliding within Bluff Analysis Section 13 other than continued maintenance of the vegetative cover. It does not appear necessary at this time to provide additional protection against wave and ice action at the toe of the bluff.

Bluff Analysis Section 14: Bluff Analysis Sections 14 through 25 lie within the City of South Milwaukee, as shown in Figure 48. The stability of the bluff slope in Section 14, which extends from 3817 to 3509 3rd Avenue in the City of South Milwaukee (north of Edgewood Avenue extended), was characterized by the use of Profile Nos. 16 and 17.

The results of the deterministic slope stability analyses are shown in Figures 47 and 49 for Profile Nos. 16 and 17, respectively. Profile No. 16 was taken in a portion of the section covered with concrete rubble. The lowest failure surface calculated at this site had a safety factor of 0.90, and included the entire bluff slope. The next nine lowest safety factors ranged from 0.91 to 0.97. Profile No. 17 represents the natural bluff slope. The lowest failure surface calculated at this site had a safety factor of 0.74, and was located within the lower two-thirds of the bluff slope. The next nine lowest safety factors ranged from 0.75 to 0.88. These results indicate an unstable bluff slope with respect to rotational sliding.

Figure 48

BLUFF ANALYSIS SECTIONS WITHIN THE CITY OF SOUTH MILWAUKEE



Source: SEWRPC.

Of the 200 failure surfaces evaluated for Profile No. 16 with the probabilistic model, all had safety factors lower than 1.0. A probabilistic stability analysis was not conducted for Profile No. 17, because the bluff slope at that site was obviously unstable. Based on both the deterministic and probabilistic slope stability analyses and on the observed bluff conditions, Section 14 was considered to have an unstable slope with respect to rotational sliding.

Overall, Section 14 was considered to have a marginal slope with respect to translational sliding. During the field survey conducted during the fall of 1987, evidence of recent shallow slides in the upper portion of the natural bluff slope was noted. Here, the slope angle was greater than 40 degrees and the vegetative cover was sparse. In the lower portion of the slope, the angle was more gentle and there was better vegetative cover, thus decreasing the potential for translational sliding.

Toe erosion, which appeared to be contributing to the instablity of the bluff slope, was observed along the entire shoreline of Section 14 during the fall of 1987 field survey. No shore protection structures were located within this section in 1987.

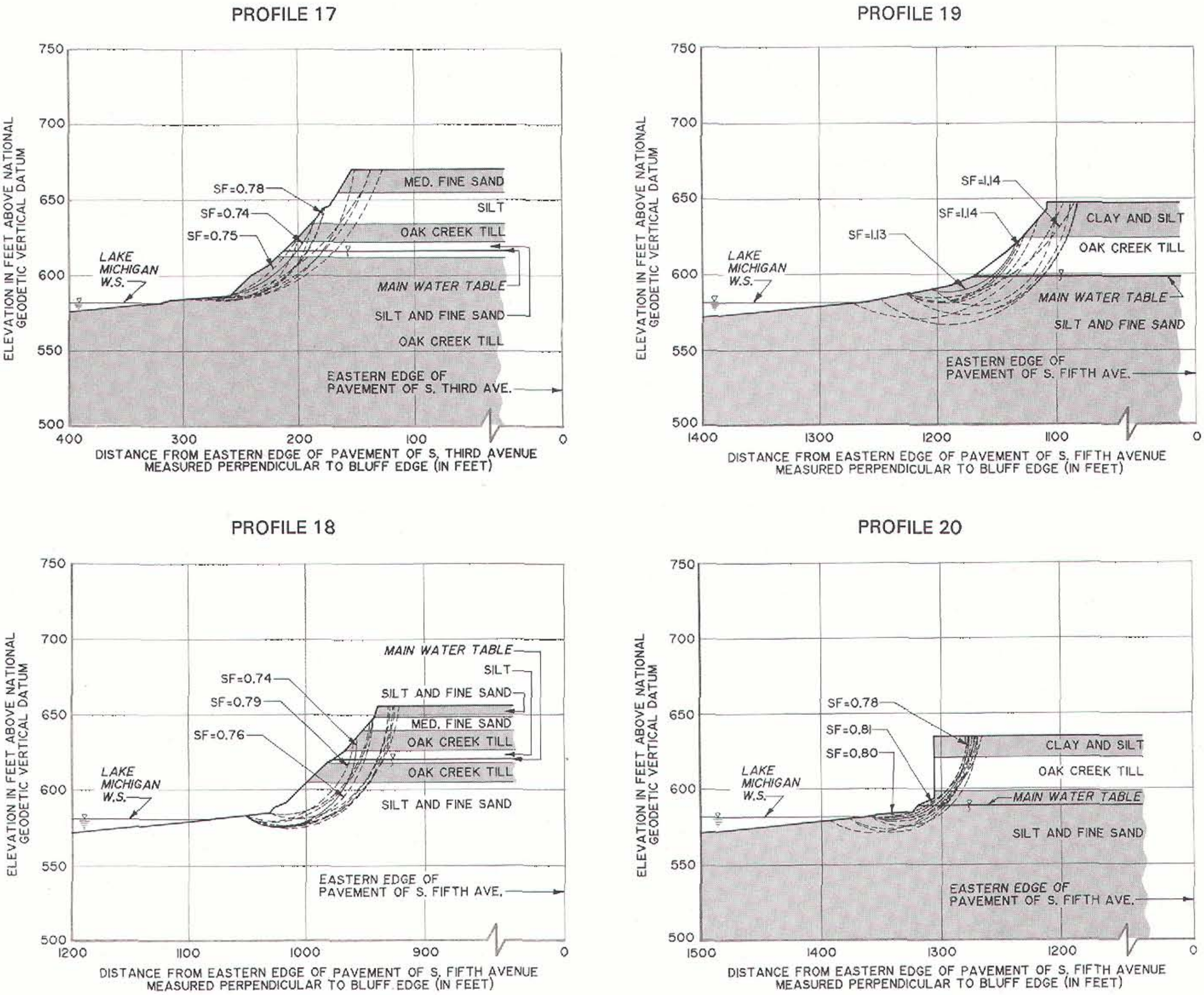
Soil and concrete rubble fill has been placed on the bluff in portions of this section. The placement of fill at Profile No. 16 has modestly improved bluff stability. However, the fill at the base of the bluff is susceptible to toe erosion. In order to fully stabilize the bluff in this section, the bluff slopes should be regraded further to a stable slope angle. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 15: The stability of the bluff slope within Section 15, which extends from 235 Lakeview Avenue to 3303 Marina Road in the City of South Milwaukee, was characterized by the use of Profile 18. A small private marina is located at the northern end of this section.

The results of the deterministic slope stability analysis, shown in Figure 49, indicate that Profile No. 18 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.74, and was located within the lower two-thirds of the bluff slope. The next nine lowest safety factors ranged from 0.76 to 0.83.

Figure 49

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSES FOR PROFILES 17-20



Source: T. B. Edil, D. M. Mickelson, and SEWRPC.

The lowest safety factors indicated by the 20 probabilistic stability analyses ranged from 0.70 to 1.10, with 19 of the critical surfaces, or 95 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated, 188, or 94 percent, had safety factors of less than 1.0.

During the field survey conducted during the fall of 1987, evidence of recent slope failure by shallow sliding was observed in the northern portion of this section. The southern portion of the section, however, was covered with concrete waste which had been dumped over the top of the bluff. This action appears to have increased the stability of that portion of the bluff. Based on both the deterministic and probabilistic slope stability analyses, and on observed bluff conditions, Section 15 was overall considered to have an unstable slope with respect to rotational sliding.

Overall, Section 15 was also considered to have an unstable slope with respect to translational sliding. Just north of the concrete fill, the slope was steeper than 40 degrees, and was barren of vegetation. Farther north, behind the private marina boat-launching facility, the slope was partially vegetated and had been graded to a more gentle angle, decreasing the potential for failure.

No significant bluff toe erosion was observed during the 1987 field survey. A small sand beach had accumulated north of the concrete fill. In the northern end of the section, the private marina boat-launching facility protected the bluff from toe erosion. The concrete structure itself, however, was being undercut at its base by wave action.

To prevent rotational and translational sliding, primarily in the northern portion of the section, it is recommended that the bluff slope be regraded to a stable slope angle and revegetated. Investigations may be needed to determine whether toxic substances are present in the bluff and whether bluff slope stabilization measures could have an adverse effect on the ecology of near-shore Lake Michigan. Within that portion of the section outside the private marina, bluff toe protection is recommended to prevent erosion from wave and ice action. Maintenance of the marina structures is needed to ensure continued bluff toe protection in the northernmost end of the section.

Bluff Analysis Section 16: The stability of the bluff slope within Section 16, which extends from 3303 Marina Road to 3333 5th Avenue in the City of South Milwaukee (between the private marina and the gully that lies just south of the South Milwaukee wastewater treatment plant), was characterized by the use of Profile No. 19.

The results of the deterministic slope stability analysis, shown in Figure 49, indicate that Profile No. 19 has a stable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 1.13, and was located within the middle third of the bluff slope. The next nine lowest safety factors ranged from 1.14 to 1.16.

A probabilistic slope stability analysis, under which bluff conditions at the profile site were varied, was conducted for Profile No. 19. The lowest safety factors indicated by the 20 probabilistic slope stability analyses ranged from 0.74 to 1.49, with 30 percent of the failure surfaces having safety factors of less than 1.0. Of the 200 failure surfaces evaluated, 36, or 18 percent, had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses and on the observed bluff conditions, Section 16 was considered to have a marginal bluff slope with respect to rotational sliding.

Section 16 was considered to have a stable bluff slope with respect to translational sliding. The bluff slope was generally well vegetated, with only small areas of exposed soil.

The toe of the bluff was protected by a wide sand beach which had accumulated to the north of the private marina boat-launching facility located in Section 15. The beach narrows in the northern end of the section, exposing the bluff to minor toe erosion. No shore protection structures were located in Section 16 in 1987.

Although Section 16 has a marginal bluff slope, no measures are recommended at this time to prevent rotational or translational slope failure. It is believed that the bluff is in the final stages of natural stabilization, and it is expected that a good vegetative cover will become established on the slope. It does not appear necessary at this time to provide additional protection against wave and ice action. It is essential, however, that the beach be maintained in order for the bluff stabilization to continue.

Bluff Analysis Section 17: The stability of the bluff slope within Section 17, located at 3333 5th Avenue in the City of South Milwaukee (the gully just south of the South Milwaukee wastewater treatment plant), was characterized by the use of Profile No. 20.

The results of the deterministic slope stability analysis, shown in Figure 49, indicate Profile 20 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface had a safety factor of 0.78, and included the entire bluff slope. The next nine lowest safety factors ranged from 0.80 to 0.92. A probabilistic slope stability analysis was not conducted for this section because the bluff slope was considered obviously unstable based on the field observations and the results of the deterministic slope stability analyses.

Section 17 was also considered to have an unstable bluff slope with respect to translational sliding. This was in part due to the very steep slope angles in the upper portion of the bluff and the absence of vegetative cover on the entire bluff slope throughout the section. Evidence of recent shallow slides was noted during the field survey conducted during the fall of 1987.

Bluff toe erosion contributing to the instability of the bluff slope was observed along the entire shoreline of Section 17 during the 1987 field survey. No shore protection structures were located in this section in 1987.

In order to fully stabilize the bluff in this section, it is recommended that the bluff slope be regraded to a stable slope angle and revegetated. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 18: The stability of the bluff slope within Section 18, which extends from the southern boundary of the South Milwaukee wastewater treatment plant northward to Marshall Avenue in the City of South Milwaukee, was characterized by the use of Profile No. 21 and Profile No. 22. Portions of the slope in this section have been covered with fill material composed of soil and concrete rubble.

The results of the deterministic slope stability analyses are shown in Figure 50 for Profile Nos. 21 and 22. Profile 21 was located on a large slump block in the southern end of the section. The lowest failure surface calculated at this profile site had a safety factor of 1.25 and was located in the upper two-thirds of the bluff slope. The next nine lowest safety factors ranged from 1.43 to 1.63. Bluff slopes often become temporarily stable immediately following a major slope failure. Profile No. 22 was located on an unfailed portion of the bluff to the north of Profile No. 21. The lowest failure surface calculated at this profile site had a safety factor of 0.87, and included the entire bluff slope. The next nine lowest safety factors ranged from 0.91 to 1.04. The results indicate the bluff slope of the recently slumped area is stable with respect to rotational sliding. The bluff slope in the unfailed area, which represents the majority of this section, showed a risk of rotational slope failure.

A probabilistic slope stability analysis, under which bluff conditions at the profile site were varied, was conducted for Profile No. 22. The lowest safety factors indicated by the 20 probabilistic slope stability analyses ranged from 0.51 to 0.84. Of the 200 failure surfaces evaluated, 195, or 97 percent, had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 18 was considered to have an unstable bluff slope with respect to rotational sliding.

Section 18 was also considered to have an unstable slope with respect to translational sliding. The potential for failure was greatest in the unvegetated previously failed areas. The remaining portion of the section was sparsely vegetated.

Bluff toe erosion is a major factor contributing to bluff slope failure within Section 18. In the northernmost end, a sand beach partially protects the toe of the bluff from wave and ice action. Concrete rubble provided minimal toe protection for this section in 1987.

In order to fully stabilize the bluff slope within Section 18, it is recommended that the bluff slope be regraded to a stable slope angle and revegetated. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 19: The stability of the bluff slope within Section 19, which extends from the South Milwaukee Yacht Club through the Grant Park beach in the City of South Milwaukee, was based on observed bluff conditions. The slope stability analyses were not conducted within this section because the bluff slope was found to be stable based upon field observations made in the fall of 1987.

In the southern end of Section 19, the bluff slope had been graded to a stable angle. The toe of the bluff was protected from erosion by the South Milwaukee Yacht Club breakwater. No erosion of the bluff was observed during the field survey conducted during the fall of 1987.

The mouth of Oak Creek is located just north of the Yacht Club. A rubble mound groin exists along the north bank of the mouth of Oak Creek at the southern end of Grant Park and has trapped a wide sand beach on the updrift side.[39] The bluff slope was well protected from

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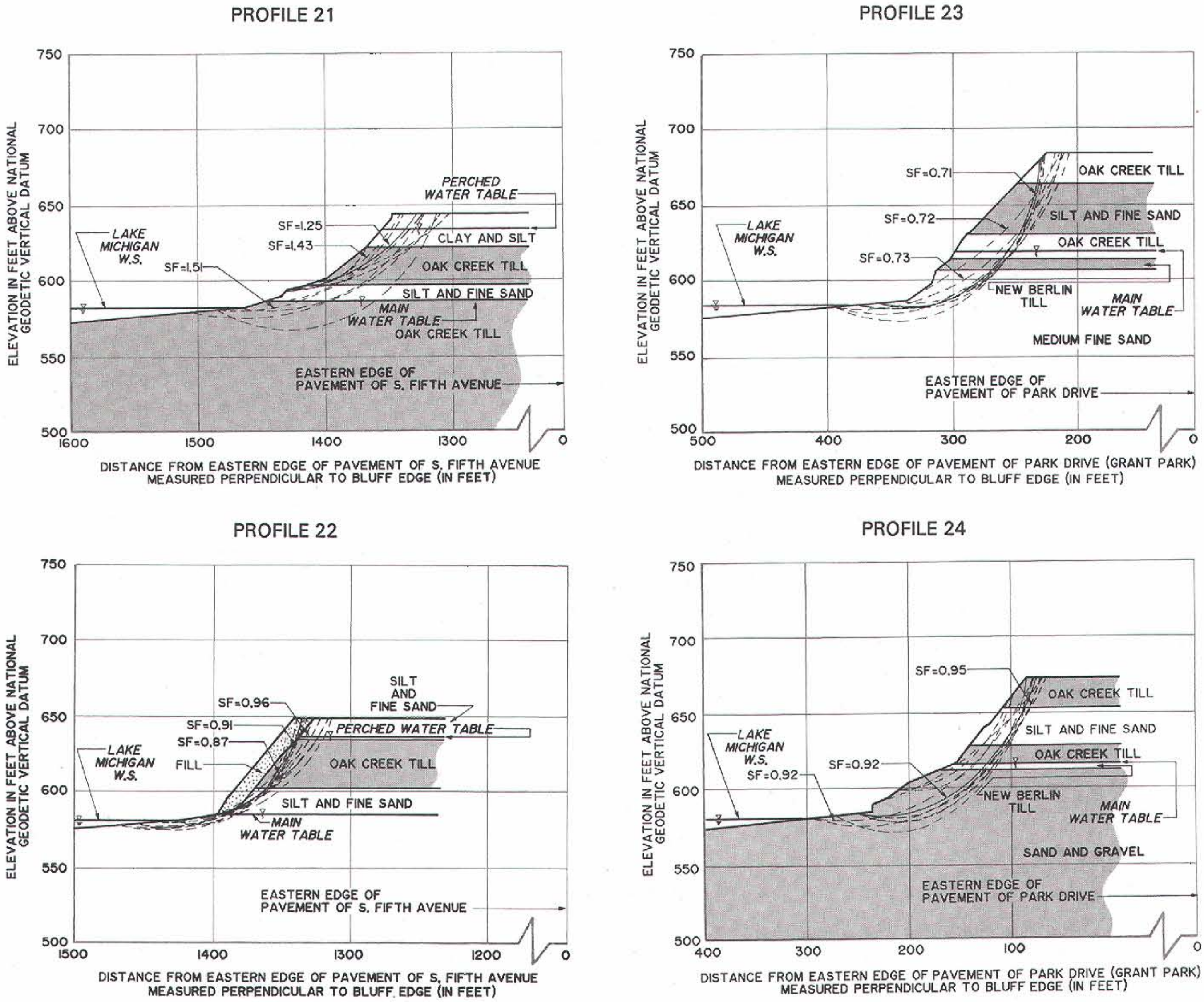
[39] *The mouth of Oak Creek is frequently obstructed by a sandbar, which at times impedes navigation into Oak Creek. Some of this sand is deposited by the littoral drift, and some is blown into the channel from the Grant Park beach, which lies immediately north of the channel. The shoaling problem in Oak Creek was addressed in SEWRPC Planning Report No. 36,*

(Footnote continued on Page 197)

Figure 50

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSES FOR PROFILES 21-24



Source: T. B. Edil, D. M. Mickelson, and SEWRPC.

(Footnote continued from Page 196)

<u>A Comprehensive Plan for the Oak Creek Watershed</u>, 1986. Four alternatives to abate the problem were developed and evaluated. All of the alternatives involved flushing sand from the mouth of the creek using either the natural streamflow, or temporarily stored flow which would be periodically released. To help flush sand from the mouth of Oak Creek, it was recommended that a narrower channel be constructed. The existing jetty on the north side of the creek would serve as one channel boundary, and a new parallel bulkhead would be installed 20 feet to the south of the jetty. The west end of the new bulkhead would be connected to the jetty on the south side of the current channel. The plan recommended that diffusers be placed along the navigation channel to help scour the sand from the channel. To complement this effort, it was recommended that the sand level on the beach just north of the channel be lowered to provide for wind-blown sand storage behind the groin, and that minimal dredging be performed in the navigation channel in order to maintain a water depth of four feet. The plan recommended that the Milwaukee County Department of Parks, Recreation and Culture be responsible for the construction of the bulkhead and the dredging of the new navigation channel. In 1988, the detailed design of the recommended plan was underway by the Department.

toe erosion by the wide beach. Overall, the bluff slope was considered stable with respect to rotational and translational sliding. The bluff slope was well vegetated, with a slope angle of about 25 degrees. There was evidence, however, of minor soil creep.

No measures are needed to prevent rotational or translational sliding within Section 19. It does not appear necessary at this time to construct additional bluff toe protection against wave and ice action other than the maintenance of the existing sand beach.

Bluff Analysis Section 20: The stability of the bluff slope within Section 20, located at Grant Park in the City of South Milwaukee (south of Rawson Avenue extended), was characterized by the use of Profile No. 23.

The results of the deterministic slope stability analysis, shown in Figure 50, indicate that Profile No. 23 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.71, and included the entire bluff slope. The next nine lowest safety factors ranged from 0.71 to 0.78. A probabilistic slope stability analysis was not conducted for this section because the bluff slope was considered to be unstable based on observations in the field within the entire section, and the results of the deterministic analyses.

Section 20 was also considered to have an unstable slope with respect to translational sliding. Bluff slope angles were generally greater than 30 degrees, and the majority of the bluff face was unvegetated.

Bluff toe erosion was considered to be the major cause of slope failure in most of this section. During the field survey conducted in the fall of 1987, undercutting was observed at the base of the bluff. A partially submerged groin located in the middle of Section 20 had trapped a narrow sand beach to the north, which did not provide sufficient toe protection against wave action. In order to fully stabilize the bluff in this section, it is recommended that the lower portion of the bluff slope be regraded to a stable slope angle and revegetated. Additional bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 21: The stability of the bluff slope within Section 21, located within Grant Park in the City of South Milwaukee (near Rawson Avenue extended), was characterized by the use of Profile No. 24.

The results of the deterministic slope stability analyses, shown in Figure 50 for Profile No. 24, indicate a risk of rotational slope failure. The lowest failure surface calculated had a safety factor of 0.92, and included the entire bluff slope. The next nine lowest safety factors ranged from 0.92 to 1.03.

A probabilistic slope stability analysis, under which the bluff conditions at the profile site were varied, was conducted to help characterize the stability of the bluff within the entire section. The lowest safety factors indicated by the 20 probabilistic stability analyses ranged from 0.82 to 1.07, with 15, or 75 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated, 98 surfaces, or 49 percent, had safety factors of less than 1.0. Groundwater seepage observed during the field survey conducted during the fall of 1987 was considered to be a major cause of bluff slope failure. Based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 21 was considered to have a marginal slope with respect to rotational sliding.

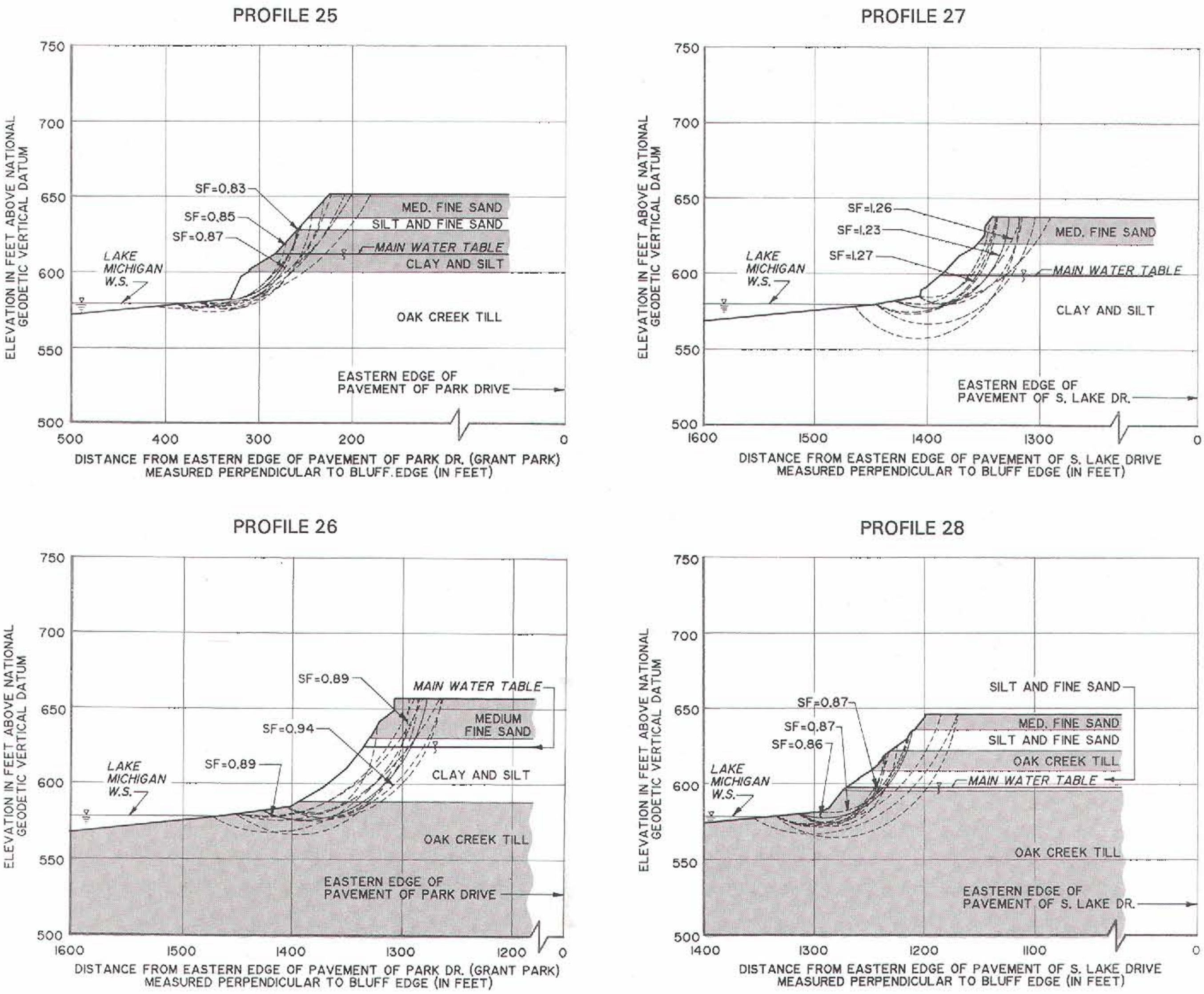
Section 21 was considered to have stable slope with respect to translational sliding. This was due to the good vegetative cover on most of the bluff slope and the relatively gentle angle of the bluff slope.

Bluff toe erosion in the northern end of Section 21 was reported during the 1987 field survey. A groin in the southern portion of the section had trapped a sand beach approximately 100 feet wide on the updrift or north side of the structure. This beach protected the toe of the bluff from erosion by wave action. Toward the northern end, the beach narrowed to approximately 25 feet. In order to fully stabilize the bluff in this section, it is recommended that a groundwater drainage system be installed to reduce the potential for failure by rotational sliding. Additional bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 22: The stability of the bluff slope within Section 22, located in Grant Park in the City of South Milwaukee (near Pine

Figure 51

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSES FOR PROFILES 25-28



Source: T. B. Edil, D. M. Mickelson, and SEWRPC.

Street extended), was characterized by the use of Profile No. 25.

The results of the deterministic slope stability analysis, shown in Figure 51, indicate that Profile No. 25 has an unstable slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.83, and was located in the lower two-thirds of the bluff slope. The next nine lowest safety factors ranged from 0.85 to 0.98.

The lowest safety factors indicated by the 20 probabilistic slope stability analyses ranged from 0.65 to 1.06, with 16, or 80 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated at Profile No. 25, 129, or 64 percent, had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 22 was considered to have an unstable slope with respect to rotational sliding.

Overall, Section 22 was considered to have a marginal bluff slope with respect to translational sliding. This was due to the lack of vegetative cover on the lower portion of the bluff slope. Numerous groundwater seeps were observed at the base of the bluff during the field survey conducted in the fall of 1987. The potential for translational slope failure was considered to be the greatest in areas where vegetation was sparse and groundwater seepage was present.

Bluff toe erosion was observed throughout Section 22 during the 1987 field survey, and was considered to have a significant impact on bluff stability. No shore protection structures were located in this section in 1987.

To fully stabilize the bluff in Section 22, it is recommended that the bluff be regraded to a stable slope angle and revegetated. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 23: The stability of the bluff slope in Section 23, located in Grant Park in the City of South Milwaukee (south of Sycamore Avenue extended), was characterized by the use of Profile No. 26.

The results of the deterministic slope stability analysis, shown in Figure 51, indicate that Profile No. 26 has an unstable slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.89, and included the entire bluff slope. The next nine lowest safety factors ranged from 0.89 to 1.02.

A probabilistic slope stability analysis, under which the bluff conditions at the profile site were varied, was conducted to help characterize the stability of the bluff slope within the entire section. All of the lowest safety factors indicated by the 20 probabilistic stability analyses were less than 1.0. Of the 200 failure surfaces evaluated, 180, or 90 percent, had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 23 was considered to have an unstable slope with respect to rotational sliding.

Section 23 was also considered to have an unstable slope with respect to translational sliding. During the field survey conducted in the fall of 1987, groundwater seepage was observed in the middle portion of the bluff face. Evidence of sapping and shallow sliding was also noted. The entire bluff face in this section was unvegetated, with a bluff slope angle of approximately 35 degrees.

Bluff toe erosion, observed throughout the section, also contributed to bluff instability. In 1987, no shore protection structures were located in Section 23.

It is recommended that the bluff slope be regraded to a stable slope angle and revegetated to fully stabilize the bluff. Bluff toe protection is also recommended to prevent erosion caused by wave and ice action.

Bluff Analysis Section 24: The stability of the bluff slope within Section 24, located in Grant Park in the City of South Milwaukee (from Sycamore Avenue extended northward to Badger Avenue extended), was characterized by the use of Profile No. 27.

The results of the deterministic slope stability analysis, shown in Figure 51, indicate a stable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 1.23, and included the entire bluff face. The next nine lowest safety factors ranged from 1.26 to 1.43.

A probabilistic slope stability analysis, under which conditions at the profile site were varied, was conducted to help characterize the stability of the bluff slope within the entire section, and to help determine whether, under certain conditions, the bluff slope would be unstable. The lowest safety factors indicated by the 20 probabilistic stability analyses ranged from 0.71 to 1.19, with 14, or 70 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated, 65, or 32 percent, had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses and on the observed bluff conditions, Section 24 was considered to have a marginal bluff slope with respect to rotational sliding.

Section 24 was considered to have an unstable slope with respect to translational sliding. This was due, in part, to the steep angle of the bluff slope and the lack of vegetative cover on the bluff face.

Bluff toe erosion, observed throughout the section during the fall of 1987, was considered to be the major cause of bluff slope instability in Section 24. At the time of the field survey, a groin located in the southern end of the section had trapped a beach approximately 70 feet wide on the updrift side of the structure. The beach narrowed to a width of approximately 20 feet at the northern end of the section.

To prevent rotational and translational sliding within Section 24, it is recommended that the bluff be regraded to a stable slope angle and revegetated. Bluff toe protection is recommended to prevent erosion caused by wave and ice action.

Bluff Analysis Section 25: The stability of the bluff slope in Section 25, located in Grant Park in the City of South Milwaukee (south of College Avenue extended), was characterized by the use of Profile No. 28.

The results of the deterministic slope stability analysis, shown in Figure 51, indicate that Profile No. 28 has an unstable slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.86, and was located in the lower two-thirds of the bluff slope. The next nine lowest safety factors ranged from 0.87 to 1.09. A probabilistic slope stability analysis was not conducted for this section because the bluff slope was considered to be unstable based on the field observations and the results of the deterministic analysis.

Section 25 was also considered to have an unstable slope with respect to translational sliding. This was due primarily to the steepness of the bluff slope angle, and to the sparsely vegetated bluff face. During the field survey conducted in the fall of 1987, evidence of solifluction and shallow slides was observed.

Due primarily to the relatively wide beach built up in Section 25, only minor bluff toe erosion was observed during the 1987 field survey. Thus under the existing shoreline and lake level conditions, wave action did not appear to substantially affect the toe of the bluff. However, if lake levels increased, the potential for toe erosion and subsequent bluff slope failure would also increase in this already unstable section. No shore protection structures were present in this section in 1987.

Regrading the bluff slope to a stable angle and revegetation is recommended to fully stabilize the bluff in Section 25. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 26: Bluff Analysis Section 26 through 37 lie within the City of Cudahy, as shown in Figure 52. The stability of the bluff slope in Section 26, which extends from College Avenue extended to the southern portion of Warnimont Park in the City of Cudahy, was characterized by the use of Profile No. 29.

The results of the deterministic slope stability analysis, shown in Figure 53, indicate that Profile No. 29 has an unstable slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.79 and included the entire bluff slope. The next nine lowest safety factors ranged from 0.83 to 0.97. A probabilistic slope stability analysis was not conducted for this section because the bluff slope was considered to be unstable based on the field observations and the results of the deterministic analysis.

Section 26 was also considered to have an unstable slope with respect to translational sliding. The upper slope was generally well vegetated, but had disturbed soil areas in steeply sloping areas. Numerous large seeps at the base of the bluff were noted during the field survey conducted during the fall of 1987. The steep slopes and the groundwater seepage were primary factors causing translational sliding.

Bluff toe erosion was observed throughout Section 26 and was considered to be a primary cause of bluff slope failure. No shore protection structures were present in Section 26 in 1987.

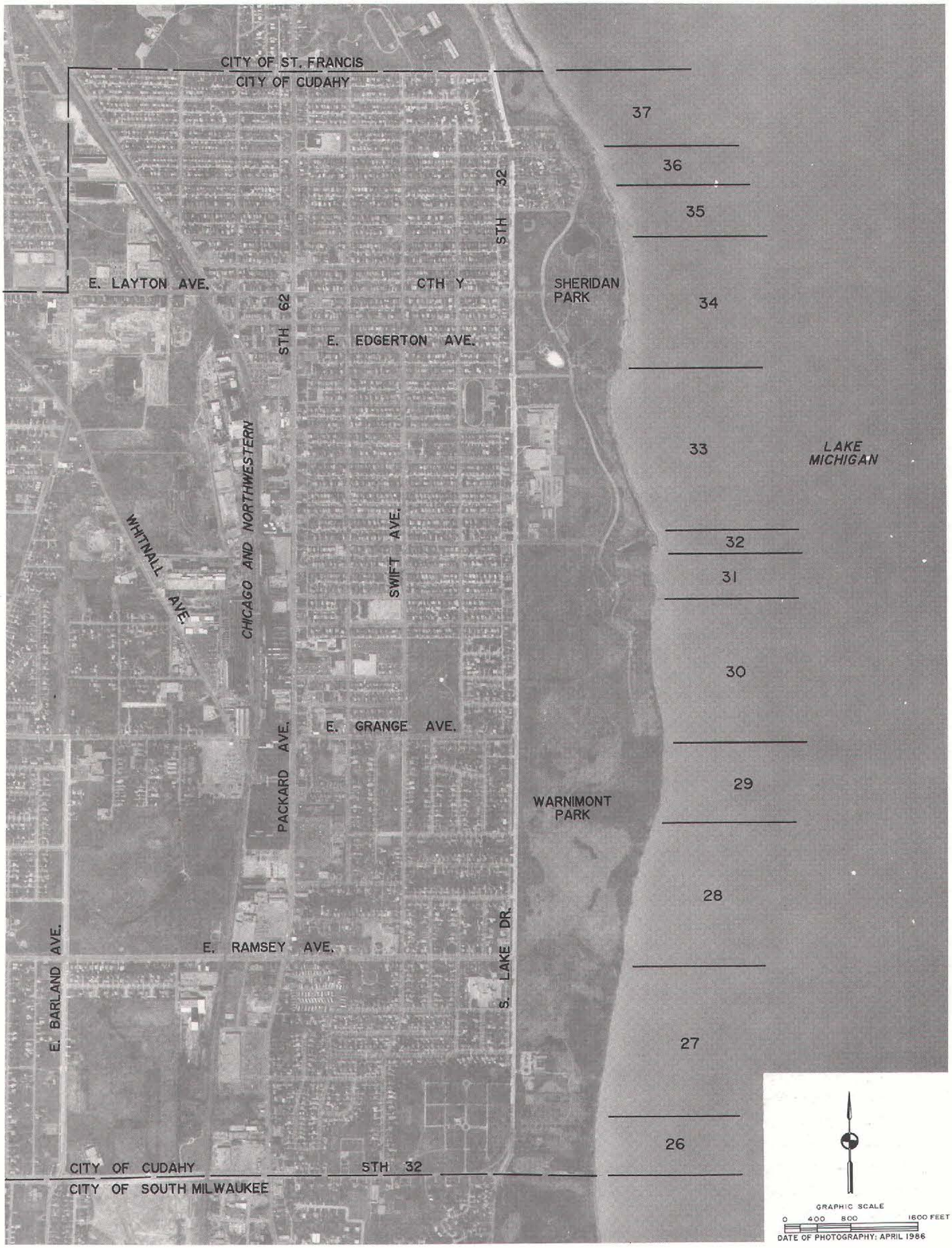
Bluff slope stabilization within Section 26 would require bluff slope regrading and revegetation. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 27: The stability of the bluff slope in Section 27, located in Warnimont Park in the City of Cudahy (south of Ramsey Avenue extended), was characterized by the use of Profile Nos. 30 and 31.

The results of the deterministic slope stability analyses are shown in Figure 53 for Profile Nos. 30 and 31. The results of the deterministic

Figure 52

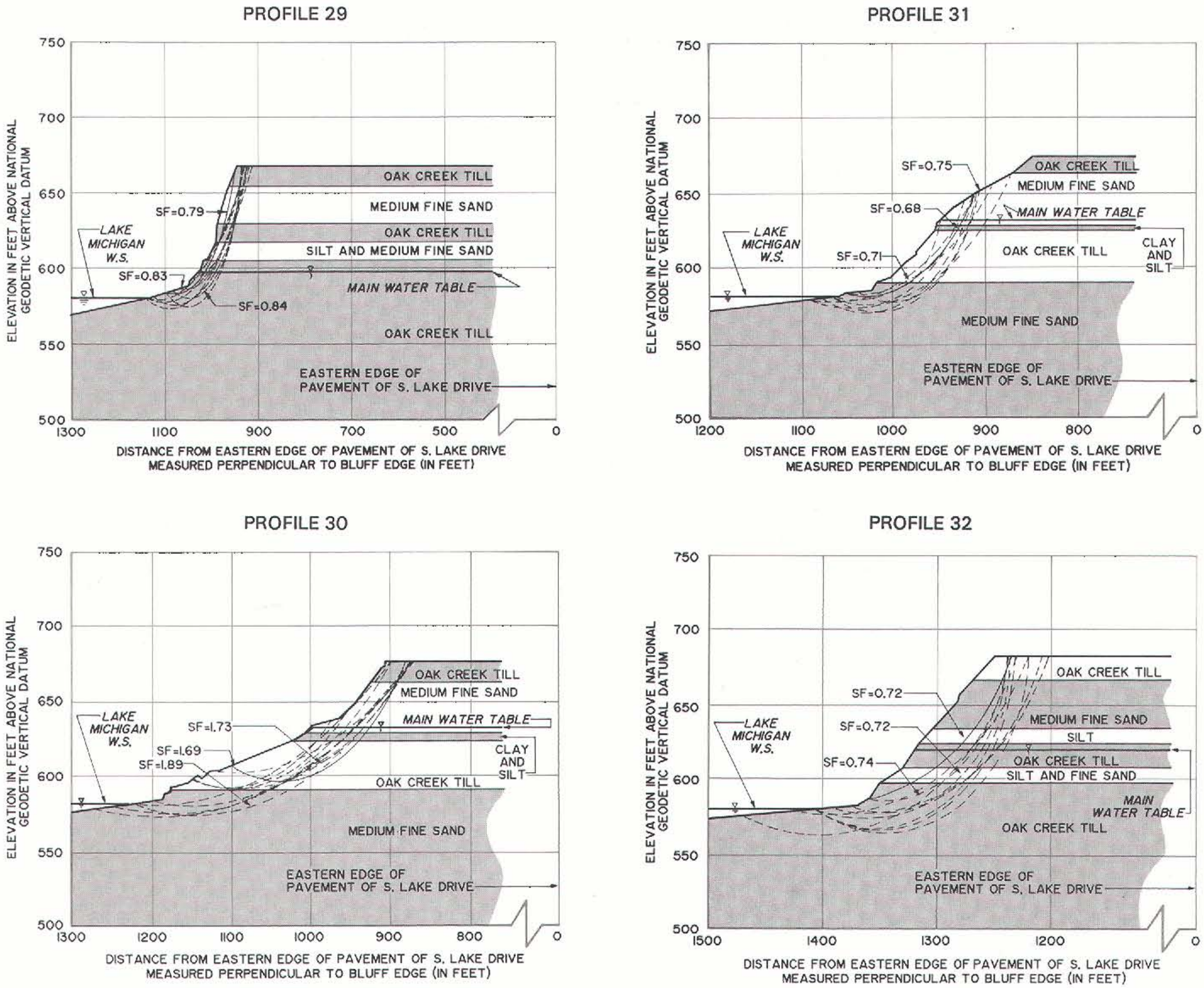
BLUFF ANALYSIS SECTIONS WITHIN THE CITY OF CUDAHY



Source: SEWRPC.

Figure 53

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSES FOR PROFILES 29-32



Source: T. B. Edil, D. M. Mickelson, and SEWRPC.

slope analysis for Profile 30, taken on a recently slumped area in the southern end of the section, indicate that the slope is stable with respect to rotational sliding. The lowest failure surface calculated had a safety factor of 1.69, and was located in the upper two-thirds of the bluff slope. The next nine lowest safety factors ranged from 1.73 to 1.95. Bluff slopes often become temporarily stable immediately following a major slope failure. Profile 31 was taken just north of Profile 30, on a section of unfailed bluff slope. The lowest failure surface calculated had a safety factor of 0.68, and was located in the upper two-thirds of the bluff slope. The next nine lowest safety factors ranged from 0.71 to 0.98.

Probabilistic stability analyses were conducted for Profile Nos. 30 and 31. The lowest safety factors indicated by the 20 analyses conducted for Profile No. 30 ranged from 1.09 to 1.79. Of the 20 analyses conducted for Profile No. 31, the lowest safety factors ranged from 0.46 to 1.11, with 18, or 90 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated at Profile No. 31, 165, or 82 percent, had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses and on the observed bluff conditions, Section 27 was considered to have an unstable bluff slope with respect to rotational sliding.

Section 27 was considered to have a marginal bluff slope with respect to translational sliding. Translational sliding was observed primarily in the lower portion of the bluff slope, and was believed to be associated with groundwater seepage.

Bluff toe erosion was observed throughout the section during the field survey conducted during the fall of 1987. It was severe enough to affect the stability of the bluff at certain locations within the section. No shore protection structures were present in Section 27 in 1987.

In order to fully stabilize the bluff slope in Section 27, bluff slope regrading to a stable slope angle and revegetation is recommended. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 28: The stability of the bluff slope within Section 28, which is located in Warnimont Park in the City of Cudahy (between Ramsey Avenue extended and about Grange Avenue extended), was characterized by the use of Profile No. 32.

The results of the deterministic slope stability analysis shown in Figure 53 indicate that Profile No. 32 has an unstable slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.72, and included the entire bluff slope. The next nine lowest safety factors ranged from 0.72 to 0.83. A probabilistic slope stability analysis was not conducted for this section because the bluff slope was considered unstable based on the filed observations and the results of the deterministic analysis.

Section 28 was considered to have a marginal bluff slope with respect to translational sliding. This may be attributed to the lack of vegetative cover on most of the bluff slope, to the relatively steep angle of the slope, and to the presence of groundwater seepage at the base of the bluff.

Bluff toe erosion contributing to the instability of the bluff slope was observed along the entire shoreline of Section 28 during the field survey of 1987. No shore protection structures were located within this section in 1987.

To prevent rotational and translational sliding within Section 28, it is recommended that the bluff slope be regraded to a stable slope angle and revegetated. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 29: The stability of the bluff slope within Section 29, located in Warnimont Park in the City of Cudahy (north of Grange Avenue extended), was characterized by the use of Profile No. 33.

The results of the deterministic slope stability analysis, shown in Figure 54, indicate that Profile No. 33 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.65, and was located within the lower two-thirds of the bluff slope. The next nine lowest safety factors ranged from 0.66 to 0.84. A probabilistic slope stability analysis was not conducted for this section because the bluff slope was considered to be obviously unstable based on the field observations and the results of the deterministic analysis.

Section 29 was also considered to have an unstable slope with respect to translational sliding. Numerous shallow slides and flows were reported during the field survey conducted in the fall of 1987. Lack of vegetative cover and the steepness of the slope were considered the major causes for failure by translational motion.

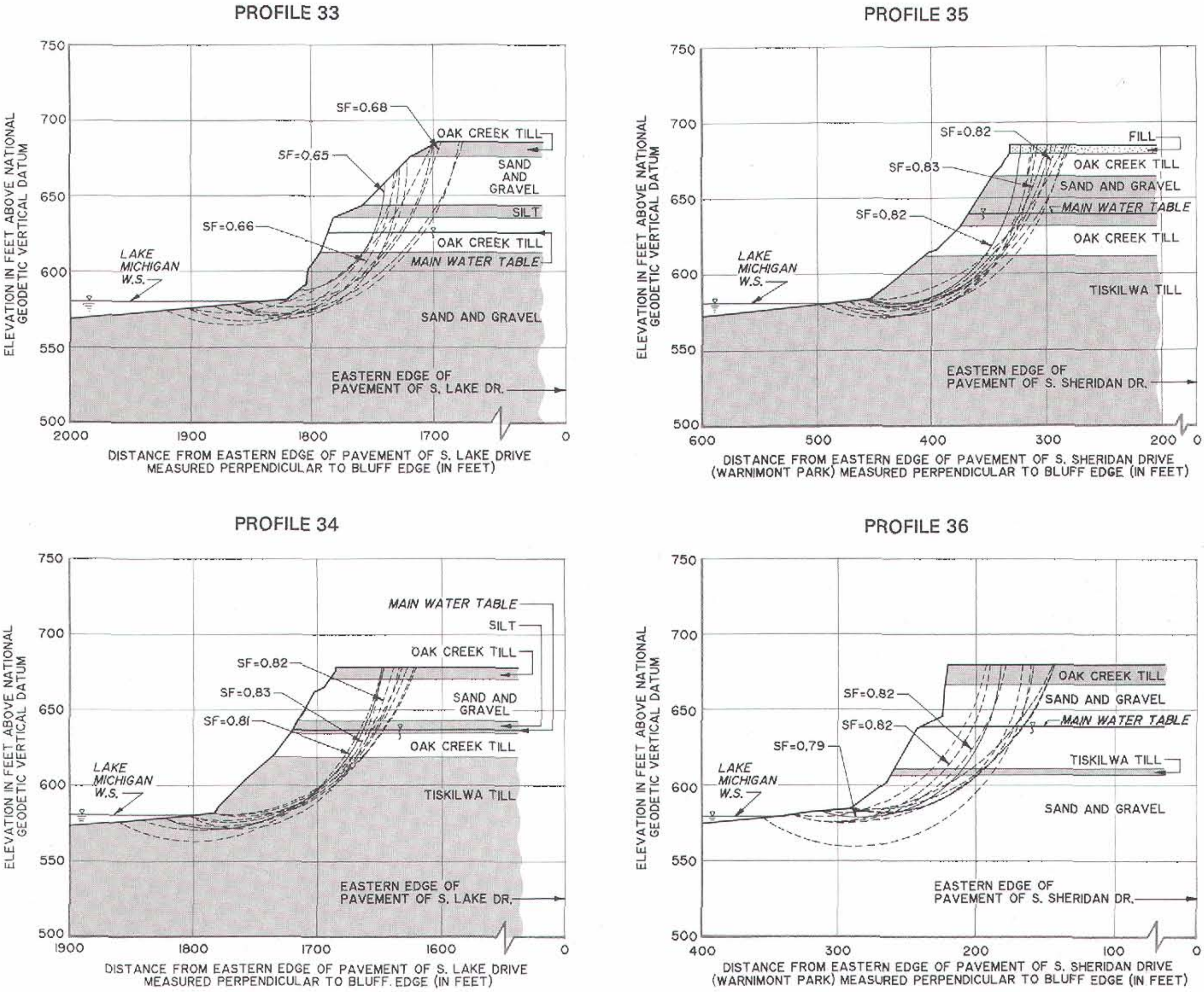
Bluff toe erosion contributing to the instability of the bluff slope was observed along the entire shoreline of Section 29 during the field survey of 1987. No shore protection structures were located within this section in 1987.

To prevent rotational and translational sliding within Section 29, it is recommended that the bluff slope be regraded to a stable slope angle and revegetated. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 30: The stability of the bluff slope within Section 30, located in Warnimont Park in the City of Cudahy (between north of Grange Avenue extended to Morris Avenue extended), was characterized by the use of Profile No. 34.

Figure 54

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSES FOR PROFILES 33-36



Source: T. B. Edil, D. M. Mickelson, and SEWRPC.

The results of the deterministic slope stability analysis, shown in Figure 54, indicate that Profile No. 34 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.81, and included the entire bluff slope. The next nine lowest safety factors ranged from 0.82 to 0.90. A probabilistic slope stability analysis was not conducted for this section because the bluff slope was found to unstable based on the field observations and the results of the deterministic analysis.

Section 30 was also considered unstable with respect to translational sliding. The slope was unvegetated, with average slope angles greater than 40 degrees. Shallow slides on the lower portion of the bluff were observed during the field survey conducted during the fall of 1987.

Toe erosion contributing to the instability of the bluff slope along the entire shoreline of Section 30 was observed during the 1987 field survey. No shore protection structures were located within this section in 1987.

To prevent rotational and translational sliding within Section 30, it is recommended that the bluff slope be regraded to a stable slope angle and revegetated. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 31: The stability of the bluff slope within Section 31, which is located in Warnimont Park in the City of Cudahy (near Edgerton Avenue extended), was characterized by the use of Profile No. 35.

The results of the deterministic slope stability analysis, shown in Figure 54, indicate that Profile No. 35 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.81, and was included in the entire bluff slope. The next nine lowest safety factors ranged from 0.82 to 0.87.

The lowest safety factors indicated by the 20 probabilistic stability analyses ranged from 0.53 to 0.89. Of the 200 failure surfaces evaluated, all had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses and on the observed bluff conditions, Section 31 was considered to have an unstable bluff slope with respect to rotational sliding.

Section 31 was also considered to have an unstable bluff slope with respect to translational sliding, due in part to the lack of vegetative cover and in part to the relatively steep angle of the bluff slope. Numerous shallow slides were observed during the field survey conducted during the fall of 1987.

Bluff toe erosion affecting the stability of the bluff slope was observed along the entire shoreline of Section 31 during the 1987 field survey, and was identified as a major cause of bluff slope failure. Shore protection structures present in the section in the fall of 1987 included two permeable groins, which had collapsed and were overtopped, and in need of maintenance.

In order to fully stabilize the bluff in this section, it is recommended that the bluff slope be regraded to a stable slope angle and revegetated. Additional bluff toe protection should be provided to prevent erosion from wave and ice action.

Bluff Analysis Section 32: The shoreline of Section 32 is located on the City of Cudahy water intake site. The natural bluff has been regraded to provide an access road to the station. No erosion of the bluff was observed during the field survey conducted during the fall of 1987. The slope stability analyses were not conducted for this section because the bluff slope was considered to be stable based on the field observations.

The shoreline of the water intake plant is protected by a concrete bulkhead with riprap toe protection. Evidence of overtopping of the bulkhead was observed during a field inspection conducted during the spring of 1988. Also noted were several locations along the structure where the riprap had settled away from the bulkhead, exposing steel reinforcement rods.

No measures are needed to prevent rotational or translational sliding within Section 32. Maintenance of the existing structure is recommended to ensure continued bluff stability in this section.

Bluff Analysis Section 33: The stability of the bluff slope within Section 33, which is located in Sheridan Park in the City of Cudahy (between the City of Cudahy water intake facility and Barnard Avenue extended), was characterized by the use of Profile No. 36.

The results of the deterministic slope stability analysis, shown in Figure 54, indicate that Profile No. 36 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.79, and included the entire bluff slope. The next nine lowest safety factors ranged from 0.82 to 0.92. A probabilistic slope stability analysis was not conducted for this section because the bluff slope was considered obviously unstable based on the field observations and the results of the deterministic analysis.

Section 33 was also considered to have an unstable slope with respect to translational sliding. This was due to the lack of vegetative cover on the bluff face and the steepness of the bluff slope. Evidence of shallow slides was observed during the field survey conducted during the fall of 1987.

Bluff toe erosion was observed along the entire shoreline of Section 33 during the 1987 field survey, and contributes to the instability of the bluff slope. No shore protection structures were present in this section in 1987.

It is recommended that the bluff slope be regraded to a stable slope angle and revegetated to prevent rotational and translational sliding in Section 33. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 34: The stability of the bluff within Section 34, located at Sheridan Park in the City of Cudahy (south of Layton Avenue extended), was characterized by the use of Profile Nos. 37 and 38.

The results of the deterministic slope stability analyses, shown in Figure 55 for Profile Nos. 37 and 38, indicate that portions of the bluff slope within Section 34 are just barely stable with respect to rotational sliding. The lowest failure surface calculated at Profile No. 37 had a safety factor of 1.21 and included the entire bluff slope. The next nine lowest safety factors ranged from 1.25 to 1.34. The lowest failure surface calculated at Profile No. 38 had a safety factor of 1.02 and included the lower two-thirds of the bluff slope. The next nine lowest safety factors ranged from 1.13 to 1.29.

Probabilistic slope stability analyses, under which the bluff conditions at each profile site were varied, were conducted to help characterize the stability of the bluff slope within the entire section, and to help determine whether, under certain conditions, the bluff slope would be unstable. The lowest safety factors indicated by the 20 probabilistic stability analyses conducted at Profile No. 37 ranged from 0.73 to 1.25, with 13 failure surfaces, or 65 percent, having safety factors of less than 1.0. Of the 200 failure surfaces evaluated at this site, 40, or 20 percent, had safety factors of less than 1.0. The lowest safety factors indicated by the 20 probabilistic stability analyses conducted at Profile No. 38 ranged from 0.71 to 1.25, with 11 failure surfaces, or 55 percent, having safety factors of less than 1.0. Of the 200 failure surfaces evaluated at this site, 72, or 36 percent, had safety factors of less than 1.0.

In the field survey conducted in the fall of 1987, some creeping and solifluction and minor slumping at the top of the bluff were observed in Section 34. Therefore, based on the probabilistic slope stability analyses and on the observed bluff conditions, Section 34 was considered to have a marginal bluff slope with respect to rotational sliding, depending on specific conditions in the bluff.

Section 34 was considered to have a stable bluff slope with respect to translational sliding. The bluff slope, overall, was well vegetated and had an average slope angle of less than 30 degrees.

Bluff toe erosion was considered to be slight throughout Section 34. This was due to the protection provided by a groin field at Sheridan Park. A beach up to 70 feet wide had been trapped on the updrift side of each groin.

To prevent rotational sliding, as well as to provide protection against wave and ice action at the toe of the bluff, it is recommended that measures be undertaken to maintain the beach at Sheridan Park. It is also recommended that a good vegetative cover be maintained on the bluff slope to prevent translational sliding.

Bluff Analysis Section 35: The stability of the bluff slope within Section 35, located at Sheridan Park in the City of Cudahy (north of Layton Avenue extended), was characterized by the use of Profile No. 39.

The results of the deterministic slope stability analysis, shown in Figure 55, indicate that Profile No. 39 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.74, and was located within the lower two-thirds of the bluff slope. The next nine lowest safety factors ranged from 0.77 to 0.89.

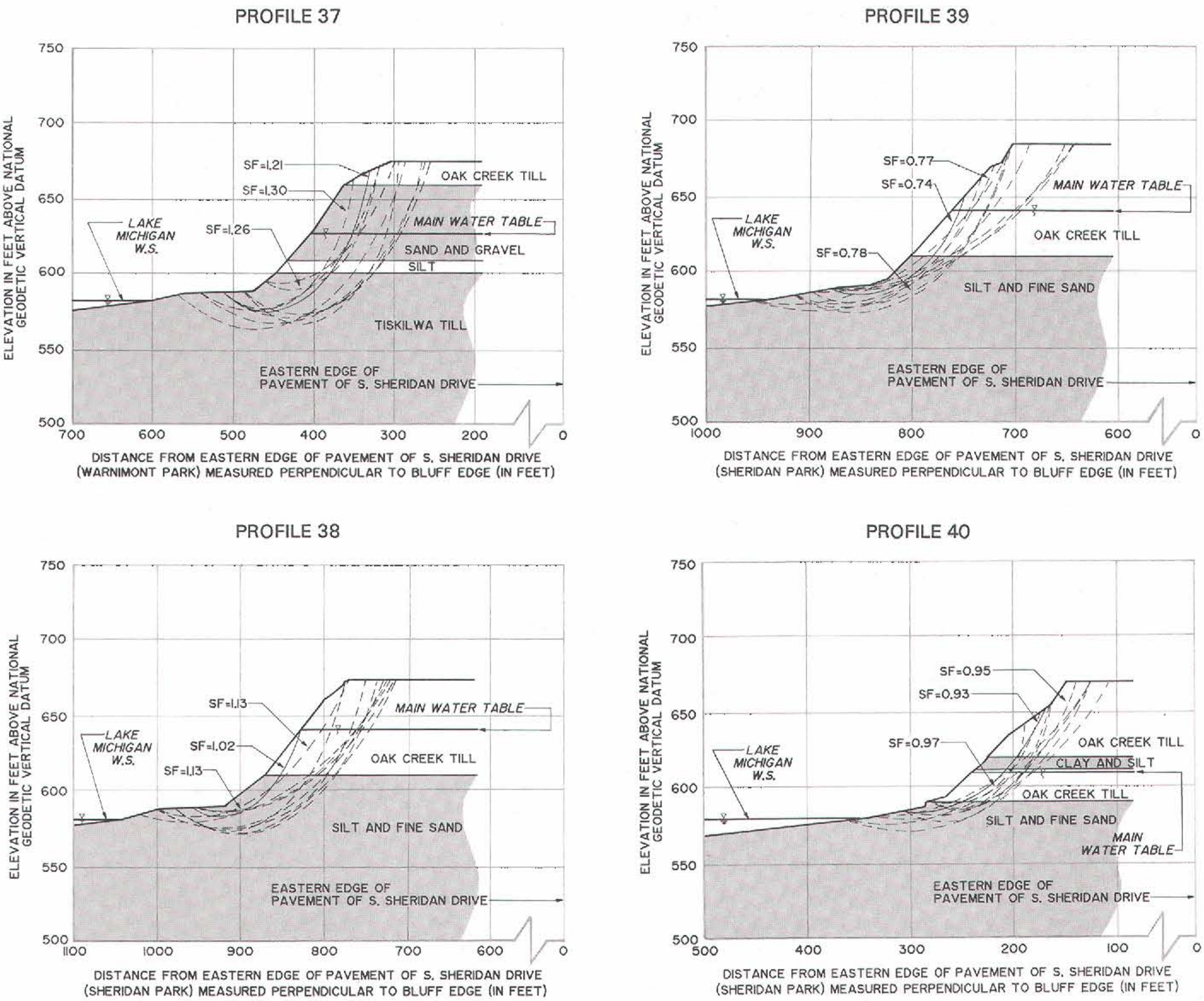
The lowest safety factors indicated by the 20 probabilistic stability analyses ranged from 0.58 to 0.94. Of the 200 failure surfaces evaluated, 152 surfaces, or 76 percent, had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses and on the observed bluff conditions, Section 35 was considered to have an unstable bluff slope with respect to rotational sliding.

Section 35 was considered to have a marginal bluff slope with respect to translational sliding. This was due in part to the relatively steep angle of the slope, and in part to the sparse vegetative cover on the bluff slope. Groundwater seepage observed during the field survey conducted in the fall of 1987 was also considered to be a major influence on bluff slope failure in this section.

Slight bluff toe erosion was observed throughout Section 35. The toe of the bluff was protected by a relatively wide beach built up by the Sheridan

Figure 55

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSES FOR PROFILES 37-40



Source: T. B. Edil, D. M. Mickelson, and SEWRPC.

Park groin field, which extends from the southern end of Section 34 through Section 35.

In order to prevent rotational sliding in Section 35, it is recommended that a detailed groundwater study be conducted to determine the impact of the man-made pond in Sheridan Park on the groundwater elevation, and whether installation of a groundwater drainage system would be desirable to lower the groundwater elevation. It is also recommended that a good vegetative cover be maintained on the bluff slope to prevent translational sliding. Measures should be undertaken to maintain the beach at Sheridan Park.

Bluff Analysis Section 36: The stability of the bluff slope in Section 36, located at Sheridan Park in the City of Cudahy (between Cudahy Avenue extended and Allerton Avenue extended), was characterized by the use of Profile No. 40.

The results of the deterministic slope stability analysis, shown in Figure 55, indicate that Profile No. 40 has a marginal slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.93 and was located within the lower two-thirds of the bluff slope. The next nine lowest safety factors ranged from 0.95 to 1.09.

The lowest safety factors indicated by the 20 probabilistic stability analyses ranged from 1.05 to 1.45. Although the probabilistic analyses indicated that the bluff was stable, observations reported during the field survey conducted during the fall of 1987, and the deterministic analysis, indicated a potential for bluff slope failure by rotational sliding in this section. The bluff slope was therefore considered to have marginal stability.

Section 36 was considered to be stable with respect to translational sliding. This was due to the good vegetative cover and relatively gentle bluff slope angle. However, some disturbed soil areas in the upper section of the slope, where translational sliding may have occurred, were noted during the fall 1987 field survey. These small isolated slides, however, did not appear to be threatening the stability of the overall bluff slope.

Toe erosion was slight in Section 36 because of the presence of a single concrete groin. The up to 70-foot-wide beach which has accumulated on the updrift side of this structure was protecting the toe of the bluff. Due primarily to this relatively wide beach, only minor erosion was observed—generally in the southern portion of the section—during the field survey conducted in 1987.

To prevent rotational sliding, as well as to provide protection against wave and ice action at the toe of the bluff, it is recommended that additional bluff toe protection be provided, which may include protection of the existing beach. It is also recommended that a good vegetative cover be maintained on the bluff slope to help prevent slope failure.

Bluff Analysis Section 37: The stability of the bluff slope within Section 37, located at Sheridan Park in the City of Cudahy (south of Lunham Avenue extended), was characterized by the use of Profile Nos. 41 and 42.

The results of the deterministic slope stability analyses, shown in Figure 56 for Profile Nos. 41 and 42, indicate that Section 37 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at Profile No. 41 had a safety factor of 0.81, and was located in the lower two-thirds of the bluff slope. The next nine lowest safety factors ranged from 0.83 to 0.95. The lowest failure surface calculated at Profile No. 42 had a safety factor of 0.86, and was also located in the lower two-thirds of the bluff slope. The next nine lowest safety factors ranged from 0.89 to 0.99. A probabilistic slope stability analysis was not conducted for this section because the bluff slope was considered to be unstable based on the field observations and the results of the deterministic analysis.

Section 37 was also considered to be unstable with respect to translational sliding. This was due to the steep angle of the bluff slope, and to the lack of good vegetative cover. Many shallow slides and small slumps were observed in this section during the field survey conducted in the fall of 1987.

Toe erosion threatening the stability of the bluff was observed throughout the entire length of shoreline in Section 37. No shore protection structures were located in this section in 1987.

In order to fully stabilize the bluff in this section, it is recommended that the bluff slope be regraded to a stable slope angle and revegetated. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 38: Bluff Analysis Sections 38 through 47 lie within the City of St. Francis, as shown in Figure 57. Bluff Analysis Section 38 was a fill project under construction in the fall of 1987.[40] The stability of the fill and

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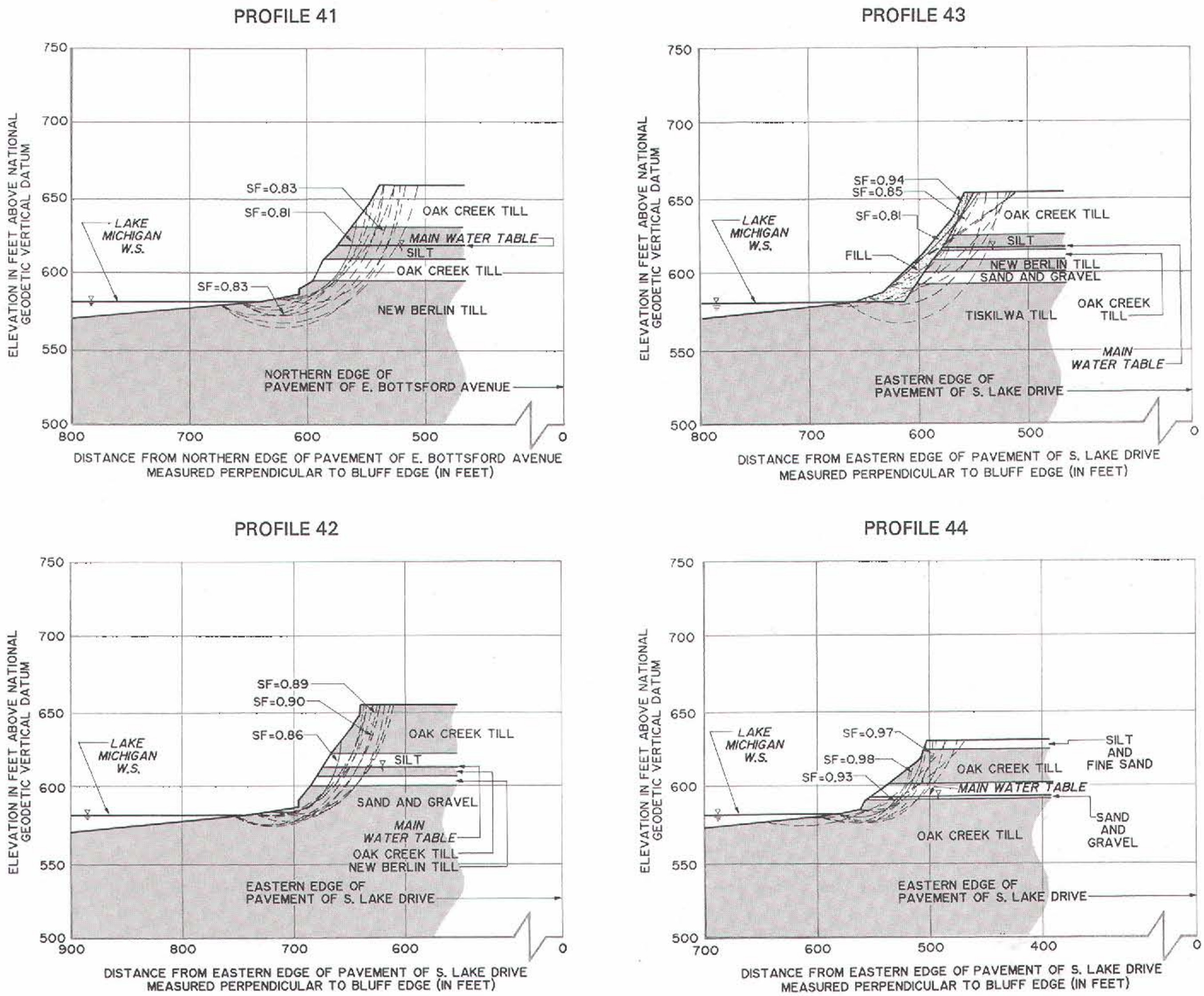
[40]*Shoreline erosion and bluff stability within Bluff Analysis Sections 38 through 47 were previously evaluated in SEWRPC Community Assistance Planning Report No. 110, A Lake Michigan Coastal Erosion and Related Land Use Management Study for the City of St. Francis, Wisconsin, 1984. That study, conducted prior to the initiation of any ongoing fill projects, found the bluff slopes within Bluff Analysis Sections 38, 39, 43, 44, 45, and 46 to be marginal or unstable, and indicated that the bluff was receding at a rate of up to six feet per year. The study evaluated alternative structural shore protection measures, identified shoreline erosion risk distances and associated setback distances for new urban development, and recommended a set of regulations which may be incorporated into the existing city zoning and subdivision ordinances to protect new urban development from shoreline erosion and bluff failure. As of 1988, the City was using the report as a guideline for reviewing proposed developments for the former Lakeside power plant site.*

Figure 56

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSES FOR PROFILES 41-44



Source: T. B. Edil, D. M. Mickelson, and SEWRPC.

underlying bluff slope within Section 38, which extends from Lunham Avenue to Denton Avenue in the City of St. Francis, was characterized by the use of Profile No. 43.

The results of the deterministic slope stability analysis, shown in Figure 56, indicate a threat of rotational bluff slope failure during the construction of the fill. The lowest failure surface calculated at this profile site had a safety factor of 0.81 and was located in the upper two-thirds of the bluff slope within the fill layer. The next nine lowest safety factors ranged from 0.85 to 1.16. A probabilistic slope stability analysis was not conducted for this profile because it is a fill site.

Although translational sliding within fill areas was generally considered unlikely, the potential for sliding was evaluated within this section because of the thin layer of fill placed on the natural bluff slope. Overall, Section 38 was considered unstable with respect to translational sliding. This was due in part to the lack of vegetative cover on the bluff slope, and in part to the steep angle of the bluff slope.

Bluff toe erosion was observed within the entire shoreline of Section 38 during the field survey conducted in the fall of 1987. However, because of the large amount of fill material placed at the base of the bluff, this toe erosion had only a modest effect on the stability of the bluff slope. No shore protection structures were located within this section in 1987, although the fill project was still in progress at the time of the field survey.

To abate the severe potential for both rotational and translational sliding, it is recommended that the bluff slope be regraded to a stable slope angle and revegetated. It is recommended that adequate toe protection be provided at the base of the fill, when complete, to prevent erosion from wave and ice action.

Bluff Analysis Section 39: The stability of the bluff slope within Section 39, which extends from Denton Avenue to 100 feet south of Howard Avenue in the City of St. Francis, was characterized by the use of Profile Nos. 44 and 45. At the time of the field surveys in the summer of 1988, fill had not been placed on the bluff slope within Section 39.

The results of the deterministic slope stability analyses, shown in Figure 56 and Figure 58 for Profile No. 44 and Profile No. 45, respectively, indicate that the bluff slope in Section 39 is unstable. The lowest failure surface calculated at Profile No. 44 had a safety factor of 0.93, and included the entire bluff slope. The next nine lowest safety factors ranged from 0.97 to 1.45. The lowest failure surface calculated at Profile No. 45 had a safety factor of 0.72 and was located in the lower two-thirds of the bluff slope. The next nine lowest safety factors ranged from 0.79 to 0.97.

The lowest safety factors indicated by the 20 probabilistic stability analyses conducted at Profile No. 44 ranged from 0.73 to 1.28, with six surfaces, or 30 percent, having safety factors of less than 1.0. Of the 200 failure surfaces evaluated at this profile site, 15, or 7 percent, had safety factors of less than 1.0. The lowest safety factors indicated by the 20 probabilistic stability analyses conducted at Profile No. 45 ranged from 0.72 to 1.14, with 13 surfaces, or 65 percent, having safety factors of less than 1.0. Of the 200 failure surfaces evaluated at this profile site, 62, or 31 percent, had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses and on the observed bluff conditions, Section 39 was considered to have an unstable bluff slope with respect to rotational sliding.

Section 39 was also considered to have an unstable bluff slope with respect to translational sliding. This was due in part to the steep angle of the bluff slope, and in part to the lack of vegetation.

Bluff toe erosion was observed along the entire shoreline of Section 39 during the field survey conducted in the fall of 1987, and was identified as a major cause of bluff slope failure. No shore protection structures were located within this section in 1987.

In order to fully stabilize the bluff in this section, it is recommended that the bluff slope be regraded to a stable slope angle and revegetated. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 40: The stability of the bluff slope within Section 40, which lies just south of the former Wisconsin Electric Power Company Lakeside power plant in the City of St. Francis, was characterized by the use of Profile No. 46. Within the section, the bluff had been regraded to help stabilize the slope.

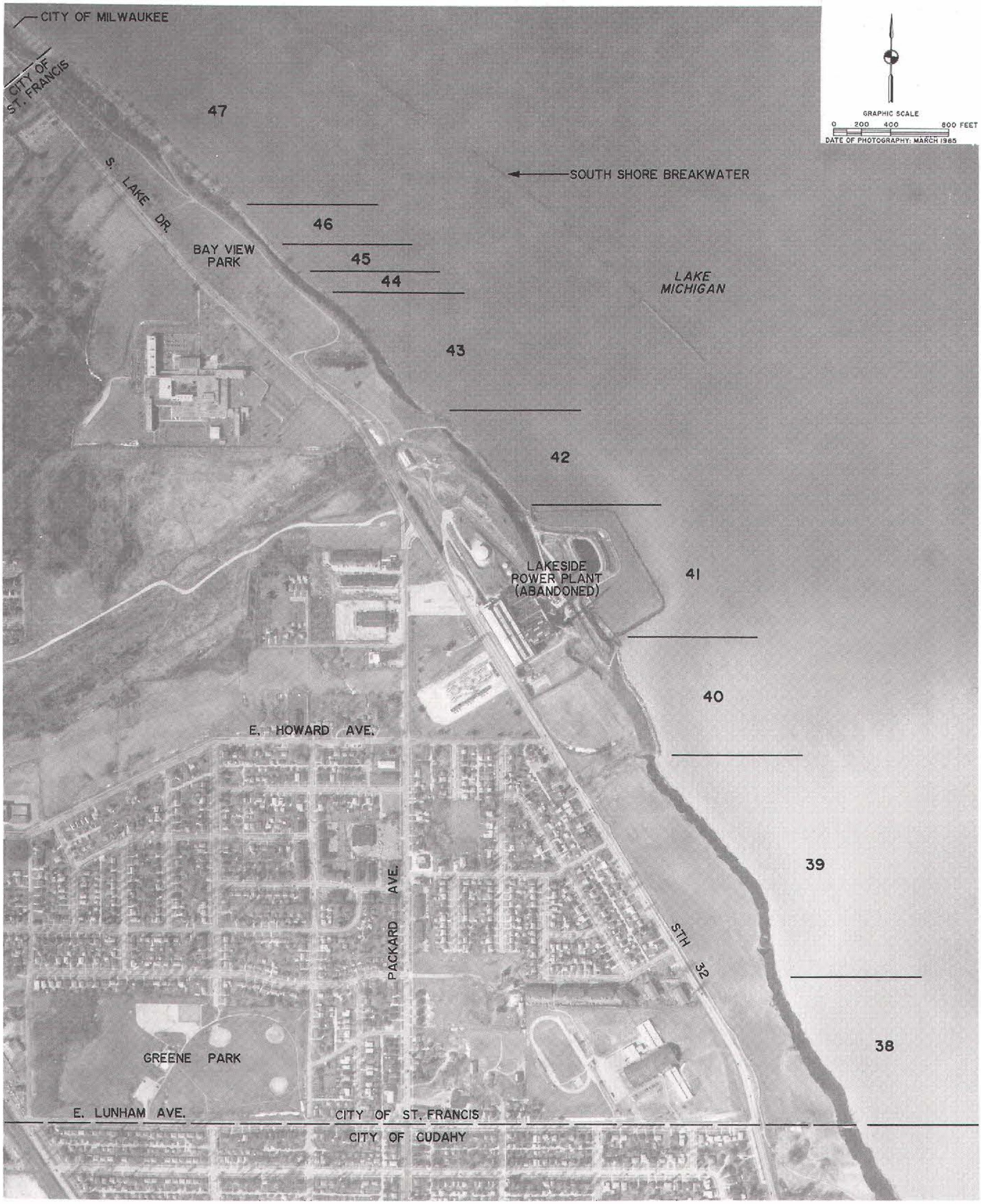
The results of the deterministic slope stability analysis, shown in Figure 58, indicate that Profile No. 46 has a stable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 1.17, and was located within the lower two-thirds of the bluff slope. The next nine lowest safety factors ranged from 1.24 to 1.45. A probabilistic slope stability analysis was not conducted for this site because the bluff slope has been regraded and partially filled. Based on both the deterministic slope stability analysis and on observed bluff conditions, Section 40 was considered to have a stable bluff slope with respect to rotational sliding.

Section 40 was also considered stable with respect to translational sliding. The regraded bluff slope was well vegetated and had an average slope angle of 22 degrees.

During the field survey conducted in the fall of 1987, bluff toe erosion was observed along the entire shoreline of Section 40. A riprap revet-

Figure 57

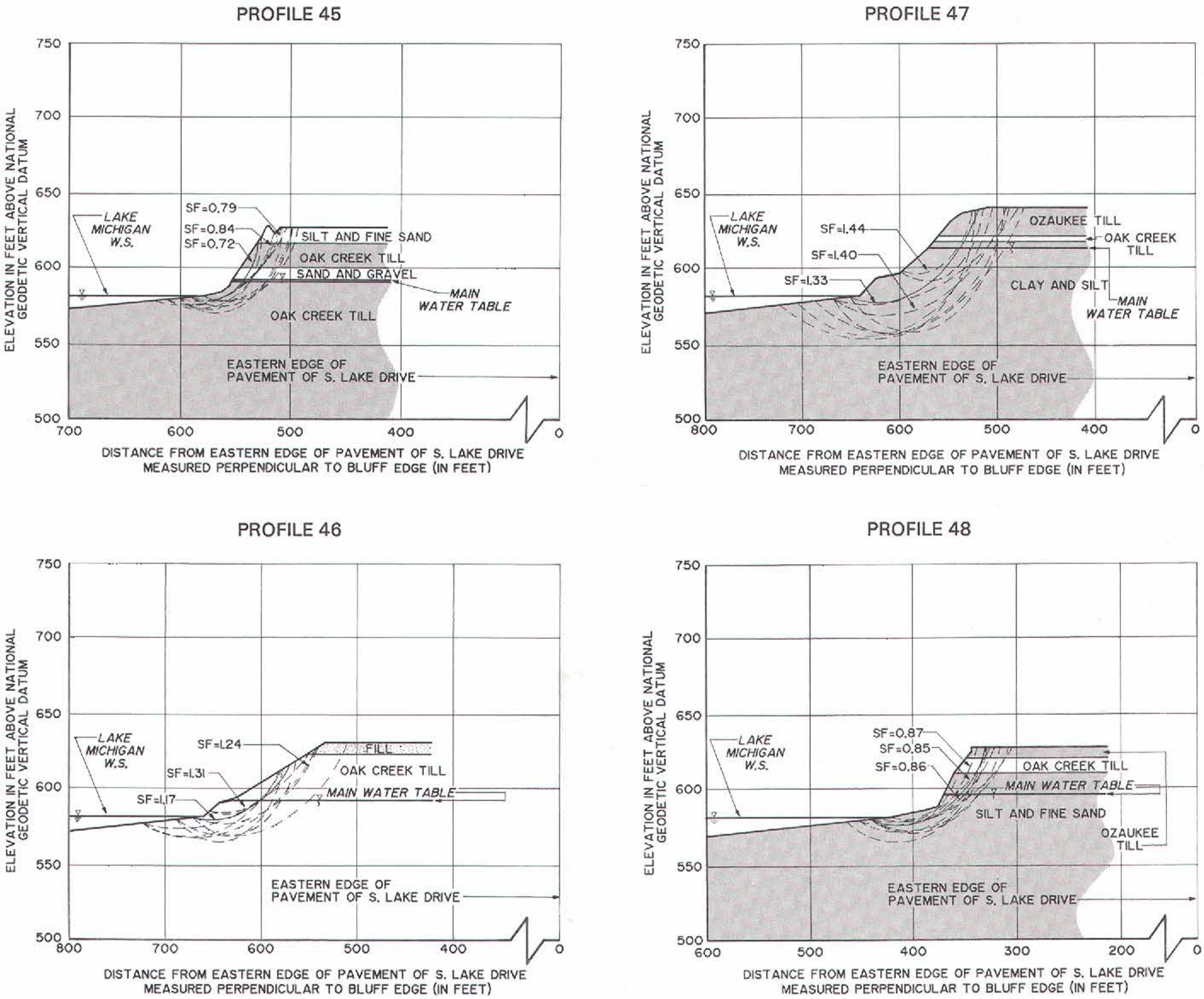
BLUFF ANALYSIS SECTIONS WITHIN THE CITY OF ST. FRANCIS



Source: SEWRPC.

Figure 58

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSES FOR PROFILES 45-48



Source: T. B. Edil, D. M. Mickelson, and SEWRPC.

ment composed of dolomite blocks provided some protection of the shoreline. However, at the time of the field survey, portions of the structure had collapsed and were subject to overtopping.

No measures are needed to prevent rotational or translational sliding within Section 40. Adequate toe erosion control measures should be maintained at the base of the regraded bluff slope to prevent erosion from wave and ice action.

Bluff Analysis Section 41: Section 41 includes the former Wisconsin Electric Power Company Lakeside power plant in the City of St. Francis. The natural bluff on the lakeward side of the abandoned electric power generation plant—which was in the process of being razed in 1988—has been regraded. No erosion of the bluff was observed during the field survey conducted during the fall of 1987. The slope stability analyses were not conducted for this section because no signs of slope failure of the regraded bluff slope were observed during the field survey.

The shoreline is protected by a breakwater, or dike, which encloses a pond formerly used as a cooling facility for the plant. A field inspection of the structure was made in the spring of 1988.

The outer breakwater was found to be in good condition, indicating that no significant damage resulted from the overtopping caused by the recent period of high lake levels. The adjacent shoreline is protected by a pile-supported wall, which also appeared to be in good condition.

No measures are needed to prevent rotational or translational sliding within Section 41. Adequate toe erosion control measures should be maintained at the former power plant site to prevent erosion from wave and ice action.

Bluff Analysis Section 42: The stability of the bluff within Section 42, located just north of the former Wisconsin Electric Power Company Lakeside power plant in the City of St. Francis, was characterized by the use of Profile No. 47.

The results of the deterministic slope stability analysis, shown in Figure 58, indicate that Profile No. 47 has a stable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 1.33, and included the entire bluff slope. The next nine lowest safety factors ranged from 1.40 to 1.59. A probabilistic slope stability analysis was not conducted for this section because the bluff slope was considered to be stable based on field observations and the results of the deterministic analysis.

Section 42 was considered to have a marginal slope with respect to translational sliding. The majority of the slope had been regraded to a gentle angle and had been vegetated. There were, however, small disturbed soil areas on the steeper sections of bluff slope, especially in the southern end of this section. Minor creep and sliding was observed during the field survey conducted in 1987.

Only minor bluff toe erosion was occurring in Section 42. A concrete rubble revetment extended from the Power Company breakwater to the south through the northern end of the section. Portions of the revetment had collapsed, allowing overtopping and subsequent toe erosion to occur. The South Shore breakwater provided additional protection to the shoreline within this section, although the southern end of the breakwater was seriously deteriorated and subject to severe overtopping.

No measures are needed to prevent rotational slope failure within Section 42. Revegetation of scattered disturbed soil areas is recommended to prevent the occurrence of translational sliding. Adequate toe erosion control measures should be maintained to prevent erosion from wave and ice action.

Bluff Analysis Section 43: The stability of the bluff within Section 43, located at the southern end of Bay View Park in the City of St. Francis, was characterized by the use of Profile Nos. 48, 49, and 50.

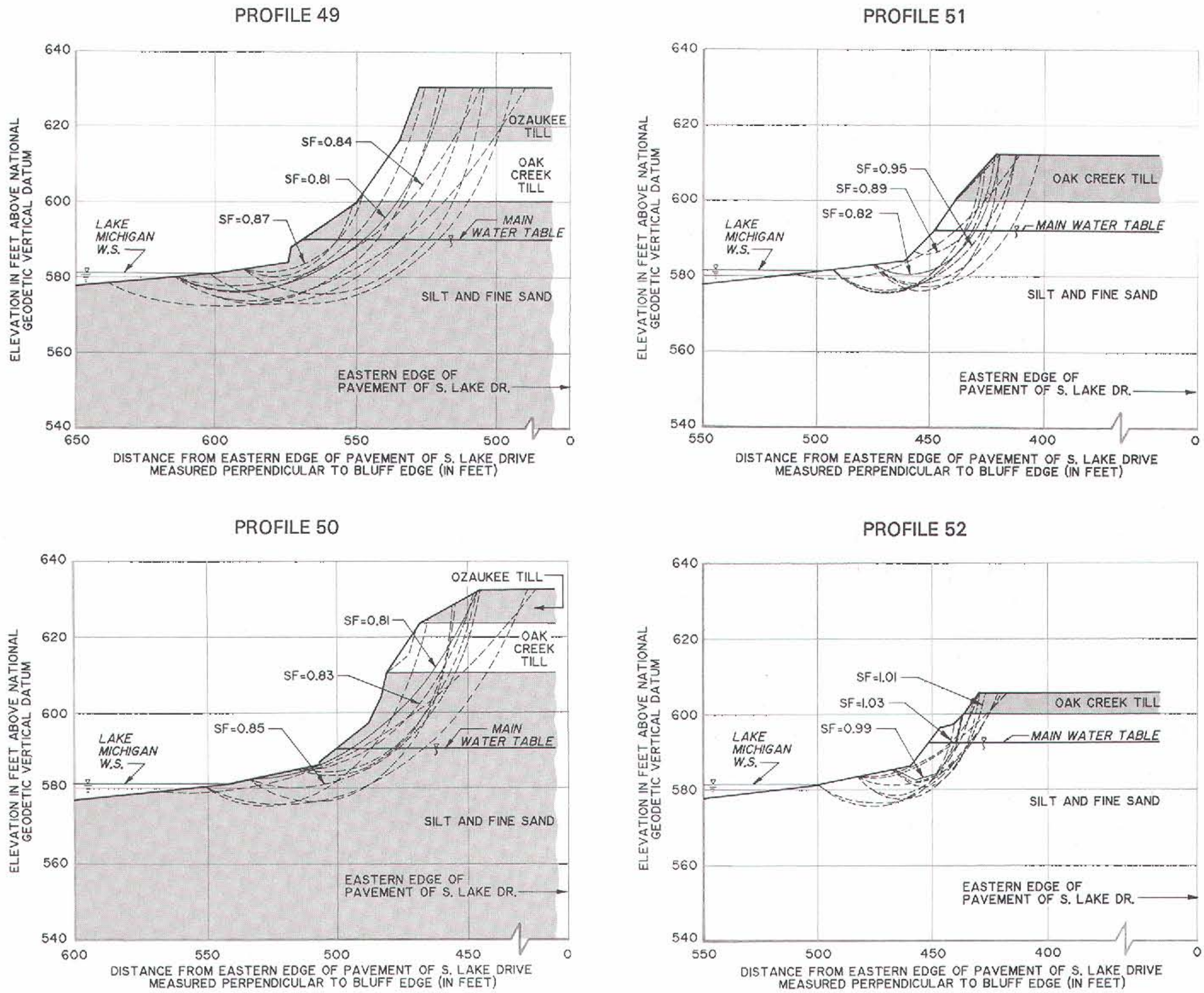
The results of the deterministic stability analyses, shown in Figures 58 and 59 for Profile Nos. 48, 49, and 50, indicate that Section 43 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at Profile No. 48 had a safety factor of 0.85 and included the entire bluff slope. The next nine lowest safety factors ranged from 0.86 to 1.09. The lowest failure surface calculated at Profile No. 49 had a safety factor of 0.81 and also included the entire bluff slope. The next nine lowest safety factors ranged from 0.84 to 1.04. At Profile No. 50, the lowest safety factor calculated had a safety factor of 0.81. The next nine lowest safety factors ranged from 0.83 to 1.07.

The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 48 ranged from 0.74 to 1.15, with 13, or 65 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated at this profile site, 47, or 23 percent, had safety factors of less than 1.0. Of the 20 probabilistic stability analyses conducted for Profile No. 49, the lowest safety factors ranged from 0.62 to 1.15, with 19, or 95 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated at Profile No. 49, 90, or 45 percent, had safety factors of less than 1.0. The lowest safety factors indicated by the 20 probabilistic stability analyses conducted at Profile No. 50 ranged from 0.66 to 1.04, with 16, or 80 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated at this profile site, 88, or 44 percent, had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses and on the observed bluff conditions, Section 43 was considered to have an unstable bluff slope with respect to rotational sliding.

Section 43 was also considered to have an unstable bluff slope with respect to translational sliding. This was due to the steep angle of the

Figure 59

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSES FOR PROFILES 49-52



Source: T. B. Edil, D. M. Mickelson, and SEWRPC.

bluff slope and to the lack of vegetation on the bluff face. Evidence of numerous shallow slides was reported during the field survey conducted during the fall of 1987.

Severe bluff toe erosion was observed throughout the section during the fall of 1987. This appeared to be a significant factor contributing to the overall instability of the bluff. In 1987, no onshore protection structures were present in this section. The South Shore breakwater, though deteriorated, provided some protection to the shoreline.

In order to fully stabilize the bluff in Section 43, it is recommended that the bluff slope be regraded to a stable slope angle and revegetated. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 44: The stability of the bluff in Section 44, located in Bay View Park in the City of St. Francis, was characterized by the use of Profile No. 51.

The results of the deterministic slope stability analysis, shown in Figure 59, indicate that Profile No. 51 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.82 and included the entire bluff slope. The next nine lowest safety factors ranged from 0.89 to 1.11. A probabilistic slope stability analysis was not conducted for this section because the bluff slope was considered unstable based on the field observations and the results of the deterministic analysis.

Section 44 was also considered to have an unstable slope with respect to translational sliding. Groundwater seepage was observed in the lower portion of the bluff slope, which also lacked a good vegetative cover and had a relatively steep slope. These factors contribute to the instability of the bluff slope caused by translational sliding in this section.

Bluff toe erosion was observed throughout Section 44 during the field survey conducted during the fall of 1987. No onshore protection structures were present in this section in 1987. The South Shore breakwater, though deteriorated, provided some protection against wave action for the shoreline within this section.

In order to fully stabilize the bluff slope in Section 44, it is recommended that the bluff slope be regraded to a stable slope angle and revegetated. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 45: The stability of the bluff in Section 45, located in Bay View Park in the City of St. Francis, was characterized by the use of Profile No. 52.

The results of the deterministic analysis, shown in Figure 59, indicate that Profile No. 52 has a marginal slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.99, and was located in the middle third of the bluff slope. The next nine lowest safety factors ranged from 1.01 to 1.23.

The lowest safety factors indicated by the 20 probabilistic slope stability analyses conducted for Profile No. 52 ranged from 0.53 to 1.64, with 10, or 50 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated, 27, or 13 percent, had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 45 was considered to have a marginal bluff slope with respect to rotational sliding.

Section 45 was also considered to have a marginal slope with respect to translational sliding. During the field survey conducted in the fall of 1987, evidence of numerous shallow slides in the thin layer of fill which covered the bluff slope was observed. The steepness of the bluff slope and the lack of vegetative cover were considered the major causes of bluff failure by translational sliding in this section.

Bluff toe erosion was observed in Section 45 during the 1987 field survey. No shoreline protection structures were present in Section 45 in 1987. The South Shore breakwater, however, provided some protection against wave action in this section. A beach approximately 30 to 50 feet wide provided additional protection to the bluff toe.

In order to fully stabilize the bluff slope in Section 45, it is recommended that the bluff slope be regraded to a stable slope angle and revegetated. Bluff toe protection is recommended to prevent erosion from wave and ice action.

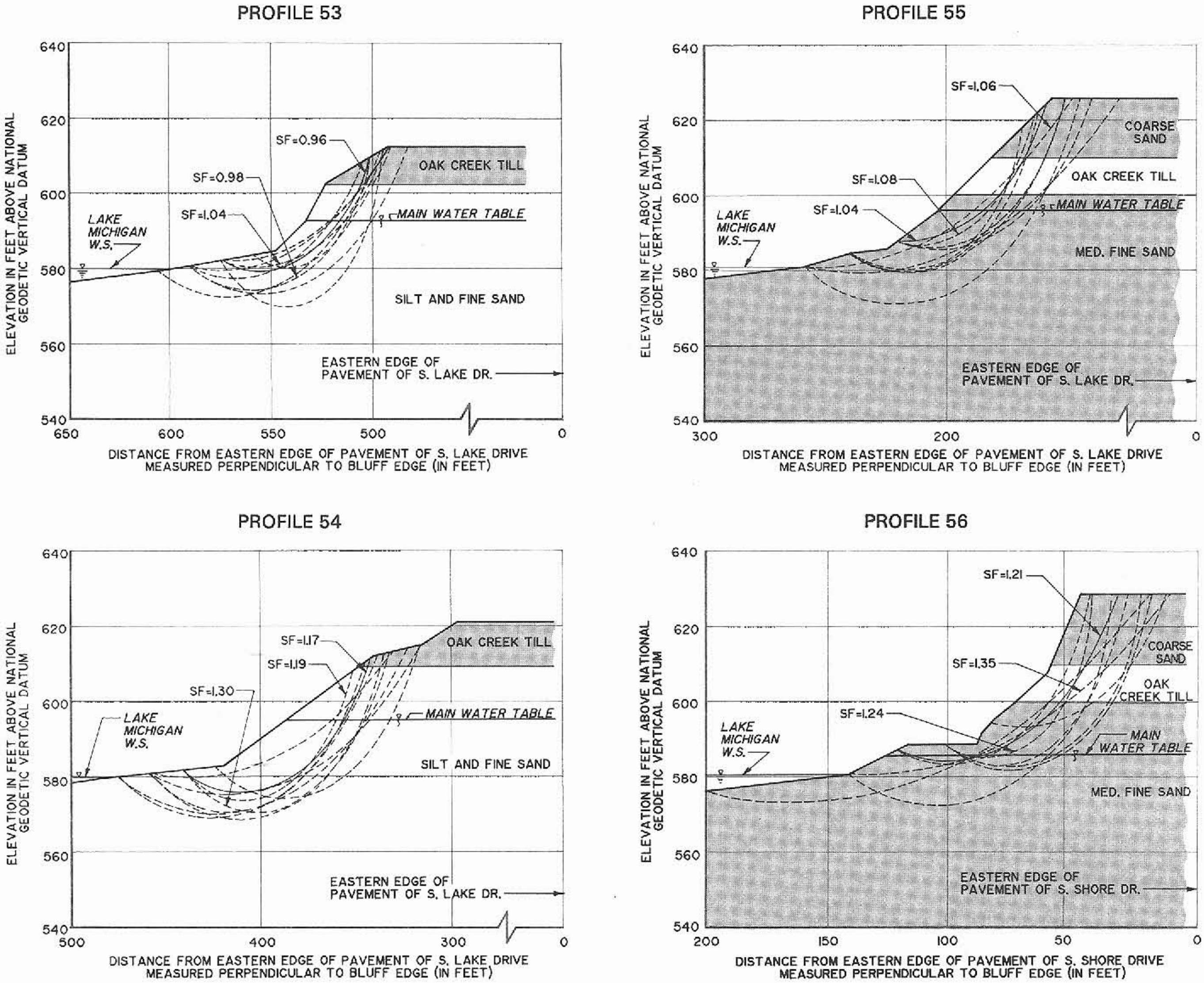
Bluff Analysis Section 46: The stability of the bluff in Section 46, located in Bay View Park in the City of St. Francis, was characterized by the use of Profile No. 53.

The results of the deterministic slope stability analysis, shown in Figure 60, indicate that Profile No. 53 has a marginal bluff slope with respect to rotational sliding. The lowest failure surface calculated at this site had a safety factor of 0.96, and included the entire bluff slope. The next nine lowest safety factors ranged from 0.98 to 1.26.

The lowest safety factors indicated by the 20 probabilistic stability analyses ranged from 0.75 to 1.48, with six, or 30 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated, nine, or 4 percent, had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses, and on observed bluff conditions, Section 46 was considered to have a marginal bluff slope with respect to rotational sliding.

Figure 60

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSES FOR PROFILES 53-56



Source: T. B. Edil, D. M. Mickelson, and SEWRPC.

Section 46 was also considered to have a marginal bluff slope with respect to translational sliding. This was due to the relatively steep angle of the bluff slope and the sparse vegetation.

Bluff toe erosion was observed in Section 46 during the survey conducted in the fall of 1987. No onshore protection structures were present in Section 46 at the time of the survey. The South Shore breakwater provided some protection against wave action in this section. A beach approximately 50 to 70 feet wide provided additional protection to the bluff toe.

In order to fully stabilize the bluff slope in Section 46, it is recommended that the bluff slope be regraded to a stable slope angle and revegetated. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 47: The stability of the bluff in Section 47, located at the northernmost end of Bay View Park and the southernmost end of South Shore Park, was characterized by the use of Profile No. 54.

The results of the deterministic slope stability analysis, shown in Figure 60, indicate that Profile No. 54 has a stable bluff slope with respect to rotational sliding. The lowest failure surface at this profile site had a safety factor of 1.17, and was located in the lower two-thirds of the bluff slope. The next nine lowest safety factors ranged from 1.19 to 1.43. A probabilistic slope stability analysis was not conducted for this section because the bluff slope was considered to be stable based on the field observations and the results of the deterministic analysis.

Section 47 was considered to have a marginal bluff slope with respect to translational sliding. Although the bluff had a gentle slope and generally a good vegetative cover, there were some small disturbed areas where translational sliding may have occurred. These small isolated slides, however, did not appear to be threatening the overall stability of the bluff slope.

Due primarily to the relatively wide sand beach built up in Section 47, only minor bluff toe erosion was observed during the field survey conducted in the fall of 1987. This toe erosion, however, has damaged a bicycle path located at the base of the bluff. No onshore protection structures were present in this section at the time of the surveys; however, the beach did receive some protection from the South Shore breakwater.

No measures are needed to prevent rotational or translational sliding within Section 47. Measures should be taken to maintain the beach at Bay View Park.

Bluff Analysis Section 48: Bluff Analysis Sections 48 through 63 lie within the City of Milwaukee, as shown in Figure 61. The stability of the bluff in Section 48, located in South Shore Park in the City of Milwaukee (north of Oklahoma Avenue extended), was characterized by the use of Profile No. 55.

The results of the deterministic slope stability analysis, shown in Figure 60, indicate that Profile No. 55 has a stable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 1.04 and was located in the upper two-thirds of the bluff slope. The next nine lowest safety factors ranged from 1.06 to 1.32.

The lowest safety factors indicated by the 20 probabilistic stability analyses ranged from 1.04 to 1.56. Based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 48 was considered to have a stable bluff with respect to rotational sliding.

Section 48 had a marginal slope with respect to translational sliding. Although the bluff slope was well vegetated and had a gentle slope angle of approximately 25 degrees, there were some small disturbed areas where translational sliding may have occurred. These small slides, however, did not appear to be threatening the overall stability of the bluff slope.

Minor bluff toe erosion was observed within Section 48 during the fall of 1987 field survey. The entire length of shoreline in this section was protected by a riprap revetment, which was overtopped in some places. The section lies within the South Shore breakwater, which provides additional shoreline protection.

No measures are needed to prevent rotational or translational sliding within this section. Adequate toe erosion control measures should be maintained to prevent erosion from wave and ice action.

Bluff Analysis Section 49: The entire shoreline of Section 49 is located at the City of Milwaukee Texas Street water intake site. The natural bluff has been regraded and is retained by the outer walls of the water intake building. No erosion of the bluff was observed during the field survey conducted during the fall of 1987. The slope stability analyses were not conducted for this section because the bluff appeared to be stable.

The shoreline of the water intake plant is protected by an armor stone riprap revetment with a toe of steel sheet pile. During the field survey conducted in the spring of 1988, some damage to the revetment on the north side of the structure was observed. The shoreline receives additional shore protection from the South Shore breakwater.

No measures are needed to prevent rotational or translational sliding within this section. Adequate toe erosion control measures should be maintained to protect the plant and prevent erosion from wave and ice action.

Bluff Analysis Section 50: The stability of the bluff slope within Section 50, located in South Shore Park just north of the Texas Street water intake plant within the City of Milwaukee, was characterized by the use of Profile No. 56.

The results of the deterministic slope stability analysis, shown in Figure 60, indicate that Profile No. 50 has a stable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 1.21, and was located within the upper two-thirds of the bluff slope. The next nine lowest safety factors ranged from 1.24 to 1.53.

A probabilistic slope stability analysis, under which the bluff conditions at the profile site were varied, was conducted to help characterize the stability of the bluff slope within the entire section, and to help determine whether, under certain conditions, the bluff slope would be unstable. The lowest safety factors indicated by the 20 probabilistic stability analyses ranged from 0.78 to 1.49, with five failure surfaces, or 25 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated, six, or 3 percent, had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses and on the observed bluff conditions, Section 50 was considered to have a stable bluff slope with respect to rotational sliding.

Overall, Section 50 was considered to have a marginal bluff slope with respect to translational sliding. Because of the moderately steep slopes, there is some risk of shallow slides.

No toe erosion of the bluff in this section was observed either in the spring of 1988 survey or in a field survey conducted during the fall of 1987. The entire shoreline of Section 50 was protected by a concrete rubble riprap revetment. No obvious signs of damage to the structure were noted in the spring of 1988. The shoreline receives additional shore protection from the South Shore breakwater.

No measures are needed to prevent rotational or translational sliding within Section 50 other than the continued maintenance of a good vegetative cover on the entire bluff slope. Adequate toe erosion control measures should be maintained to prevent erosion from wave and ice action.

Bluff Analysis Section 51: Section 51 includes the South Shore Park Pavilion in the City of Milwaukee. The natural bluff has been graded to an approximate slope angle of 18 degrees. No slope failures were observed during the field survey conducted in the fall of 1987. Slope stability analyses were not conducted for this section because the bluff appeared to be stable.

During a field survey conducted during the spring of 1988, moderate erosion at the toe of the bluff was observed. The shoreline is protected by a revetment which was subject to overtopping during the 1986 high lake levels. The shoreline received additional shore protection from the South Shore breakwater.

No measures are needed to prevent rotational or translational sliding within Section 51. Adequate toe erosion control measures should be maintained to protect the pavilion and prevent erosion from wave and ice action.

Bluff Analysis Section 52: Section 52 includes the South Shore Park Beach in the City of Milwaukee. The natural bluff has been graded to a slope angle of approximately 15 degrees. No slope failures were observed during the field survey conducted during the fall of 1987. Slope stability analyses were not conducted for this section because the bluff appeared to be stable.

Due primarily to the relatively wide beach built up in Section 52, no significant bluff toe erosion was observed during the field survey conducted in the fall of 1987. No onshore protection structures were present in this section at the time of the field survey; however, the beach received some protection from the South Shore breakwater.

No measures are needed to prevent rotational or translational sliding within Section 52. Measures should be taken to maintain the beach at South Shore Park.

Bluff Analysis Section 53: The entire shoreline of Section 53 is located at the South Shore Yacht Club in the City of Milwaukee. The natural bluff had been graded to a very gentle angle of

Figure 61

BLUFF ANALYSIS SECTIONS WITHIN THE CITY OF MILWAUKEE

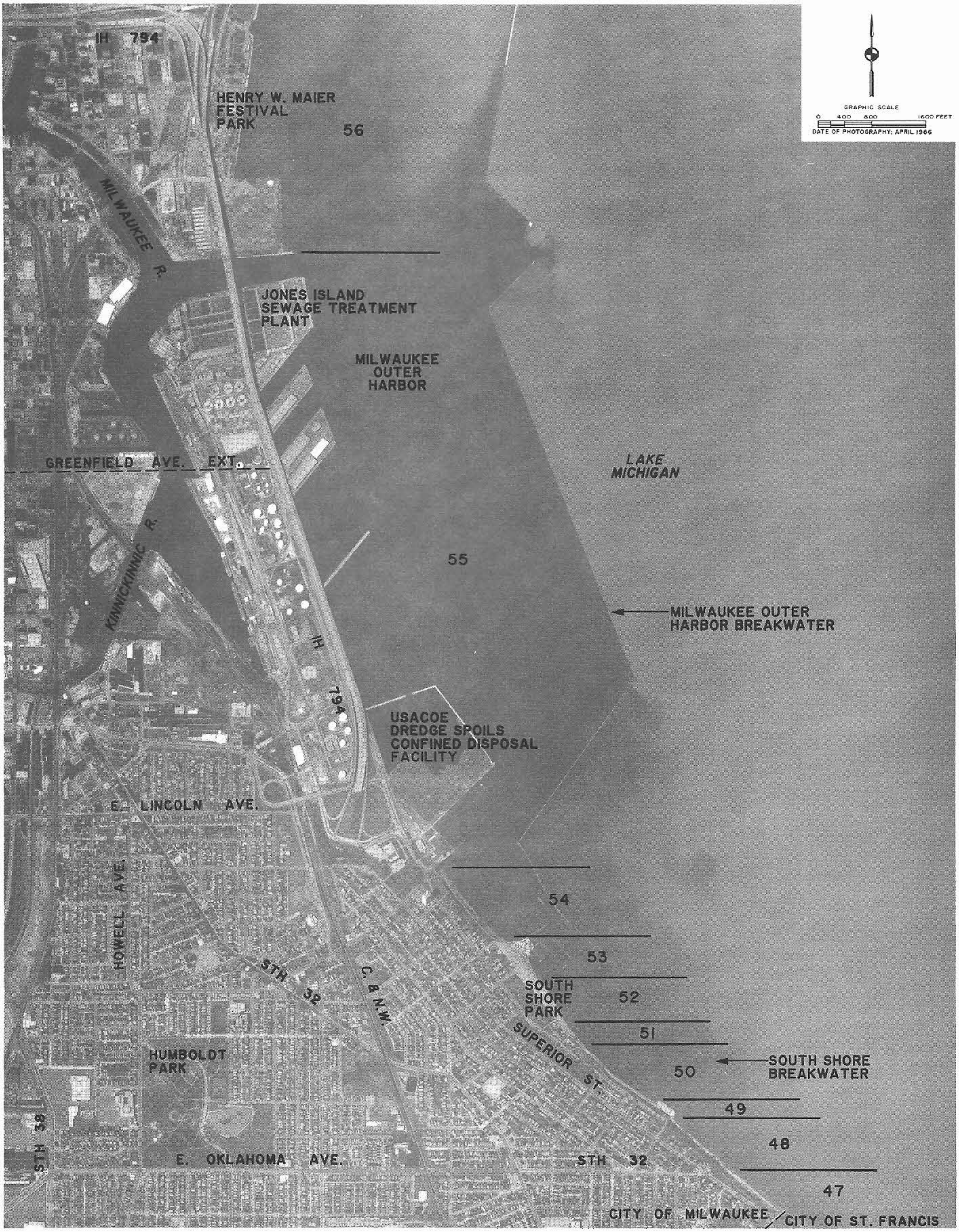
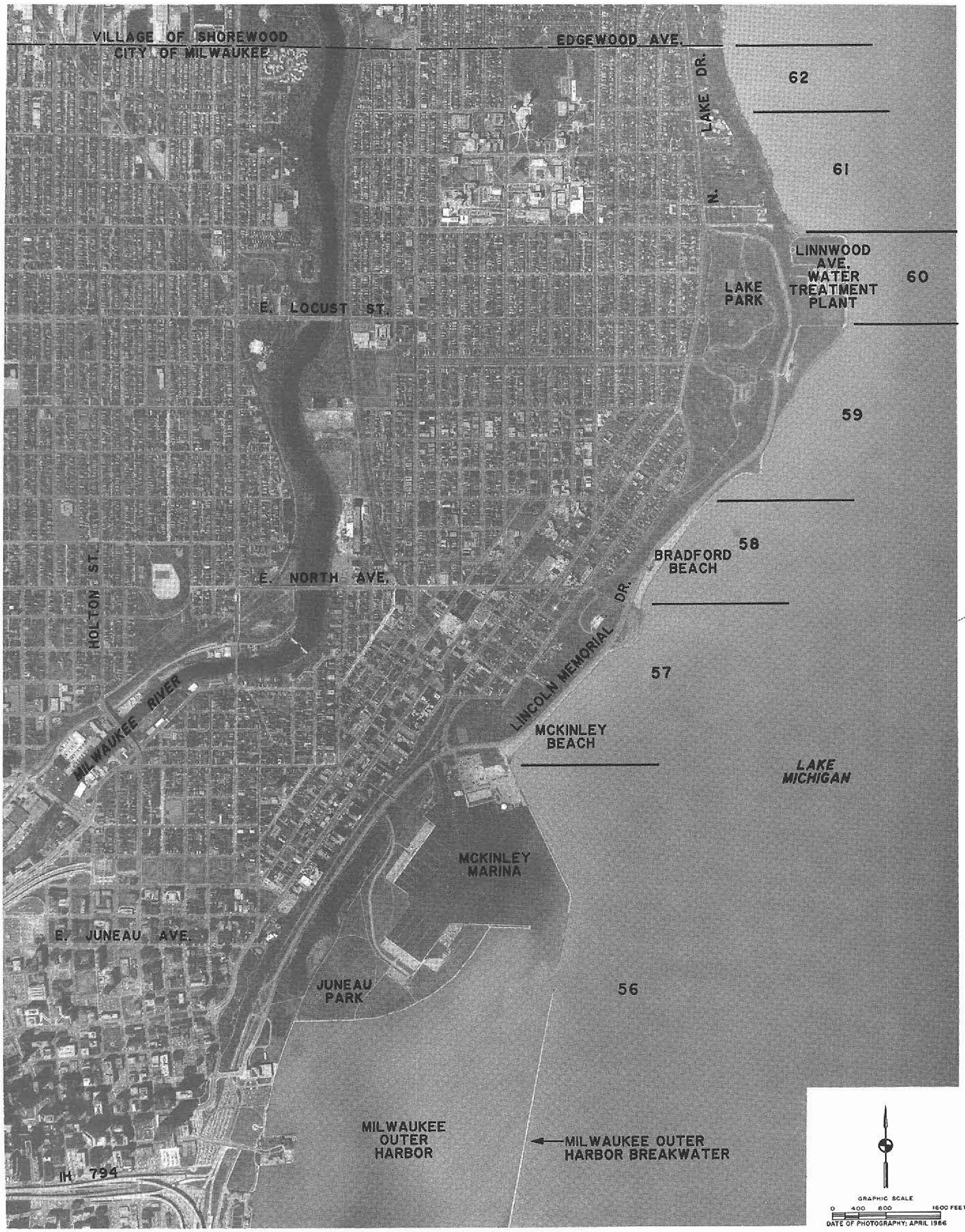


Figure 61 (continued)



Source: SEWRPC.

approximately 15 degrees. No slope failures were observed during the field survey conducted during the fall of 1987. Slope stability analyses were not conducted for this section because the bluff appeared to be stable. The shoreline in the section is protected by a steel sheet pile bulkhead. No apparent signs of structure failure were observed during the field survey.

No measures are needed to prevent rotational or translational sliding within Section 53. Adequate shoreline protection should continue to be provided to protect the yacht club and prevent erosion from wave and ice action.

Bluff Analysis Section 54: Section 54 includes the northern portion of South Shore Park in the City of Milwaukee. The natural bluff slope has an overall angle of approximately 35 degrees. No slope failures were observed during the field survey conducted during the fall of 1987. Slope stability analyses were not conducted for this section because the bluff appeared to be stable.

At the time of the field surveys conducted in 1987, the shoreline in this section was protected by a riprap revetment which was being overtopped. The South Shore breakwater provided additional protection to the shoreline.

No measures are needed to prevent rotational or translational sliding within Section 54. Adequate shoreline protection should continue to be provided to prevent erosion from wave and ice action.

Bluff Analysis Section 55: Section 55, which extends from E. Russell Avenue to the Milwaukee Metropolitan Sewerage District Jones Island wastewater treatment plant and inner harbor entrance, is located within the southern portion of the Milwaukee outer harbor breakwater within the City of Milwaukee. There is no natural bluff at the water's edge within this section. A variety of structures line the shoreline within Section 55. The U. S. Army Corps of Engineers dredged material confined disposal site revetment covers 30 percent of the shoreline. Approximately 25 percent of the shoreline is protected by the S. Lincoln Memorial Drive steel sheet pile bulkhead. Port of Milwaukee harbor slips protect 30 percent of the shoreline, and the Jones Island wastewater treatment plant bulkhead protects the remaining 15 percent. Continued maintenance of these structures is recommended to protect major lakefront facilities.

Bluff Analysis Section 56: Section 56, which extends from the Marcus Amphitheatre to the McKinley Marina, is located within the northern portion of the Milwaukee outer harbor breakwater in the City of Milwaukee. There is no natural bluff at the water's edge within this section. A number of structures are located along the shoreline within this section. The Marcus Amphitheatre bulkhead and Summerfest revetment protect 15 and 20 percent of the shoreline, respectively. About 10 percent is protected by the Milwaukee Harbor Commission bulkhead. The Milwaukee County War Memorial Center bulkhead protects about 10 percent of the shoreline, and the Juneau Park landfill bulkhead about 20 percent. The McKinley Marina bulkhead protects the remaining 25 percent of the shoreline. Continued maintenance of these structures is recommended to protect major lakefront facilities. Shortly after the field surveys were conducted, construction of a recreational island began offshore of the Henry Maier festival grounds. The project consists of a 17-acre island connected to the northeastern corner of the Marcus Amphitheatre grounds by a concrete causeway. The island was being constructed of about 650,000 cubic yards of crushed limestone from the Milwaukee Metropolitan Sewerage District deep tunnel project. Approximately 117,000 tons of 300- to 6,000-pound armor stone will be used to contain the island. Construction of the island is expected to be completed in 1990.

Bluff Analysis Section 57: The shoreline of Section 57 extends from the McKinley Beach/revetment to North Point in the City of Milwaukee. Because the natural bluff is located west of Lincoln Memorial Drive, and approximately 200 feet from the shoreline, it was not evaluated under this study. The McKinley Beach/revetment, a revetment and pocket beach system, protects 80 percent of this section; the North Point revetment, which suffered overtopping damage as a result of the 1986 high water levels, protects the remaining 20 percent. The recently constructed McKinley Beach/revetment uses a headland/beach system to protect the shoreline. Revetment protection provided at the north and south end of the project helps to contain two beaches in the middle—one composed of sand and one composed of pebbles. The project not only provided shoreline protection, but also added over 12 acres fo parkland to this highly used recreational area. A total of 350,000 cubic yards of crushed limestone from the Milwaukee Metropolitan Sewerage District deep tunnel separate and combined sewer overflow abatement project was used to construct the system. Adequate shoreline erosion control measures should be maintained to protect N. Lincoln Memorial Drive.

Bluff Analysis Section 58: Section 58 includes Bradford Beach in the City of Milwaukee. Because the natural bluff is located west of N. Lincoln Memorial Drive and over 200 feet from the shoreline, it was not evaluated under this study. The entire shoreline within Section 58 is protected by the sand beach, which is nearly 200 feet wide. Maintenance of Bradford Beach is recommended to preserve this important recreational facility and protect N. Lincoln Memorial Drive.

Bluff Analysis Section 59: The entire shoreline of Section 59 is located in Lake Park in the City of Milwaukee. Because the natural bluff is located west of N. Lincoln Memorial Drive and approximately 200 feet from the shoreline, it was not evaluated under this study. The shoreline in Section 59 is protected by a revetment. Evidence of overtopping by waves was noted during the field survey conducted during the spring of 1988. Adequate toe erosion control measures should be maintained to protect N. Lincoln Memorial Drive and prevent further overtopping damage.

Bluff Analysis Section 60: Section 60 includes the City of Milwaukee Linnwood Avenue water treatment plant site. The bluff was not evaluated within this section because it is located behind the plant facilities, and more than 400 feet from the shoreline. A field inspection of the concrete and steel sheet pile bulkhead protecting the plant was conducted during the spring of 1988. Damage by wave overtopping was noted.

Adequate shoreline erosion control measures should be maintained to protect N. Lincoln Memorial Drive, and prevent erosion from wave and ice action.

Bluff Analysis Section 61: The stability of the bluff slope within Section 61, which extends from the City of Milwaukee Linnwood Avenue water treatment plant to 3052 E. Newport Court, was characterized by the use of Profile No. 57.

The results of the deterministic slope stability analysis, shown in Figure 62, indicate that Profile No. 57 has a stable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 1.46, and was located within the lower two-thirds of the bluff slope. The next nine lowest safety factors ranged from 1.48 to 1.61.

A probabilistic slope stability analysis, under which the bluff conditions at the profile site were varied, was conducted to help characterize the stability of the bluff slope within the entire section, and to help determine whether, under certain conditions, the bluff slope would be unstable. Of the 20 probabilistic stability analyses conducted, the lowest safety factors ranged from 0.98 to 1.60, with only one failure surface, or 5 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated, only one surface had a safety factor of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses and on the observed bluff conditions, Section 61 was considered to have a stable bluff slope with respect to rotational sliding.

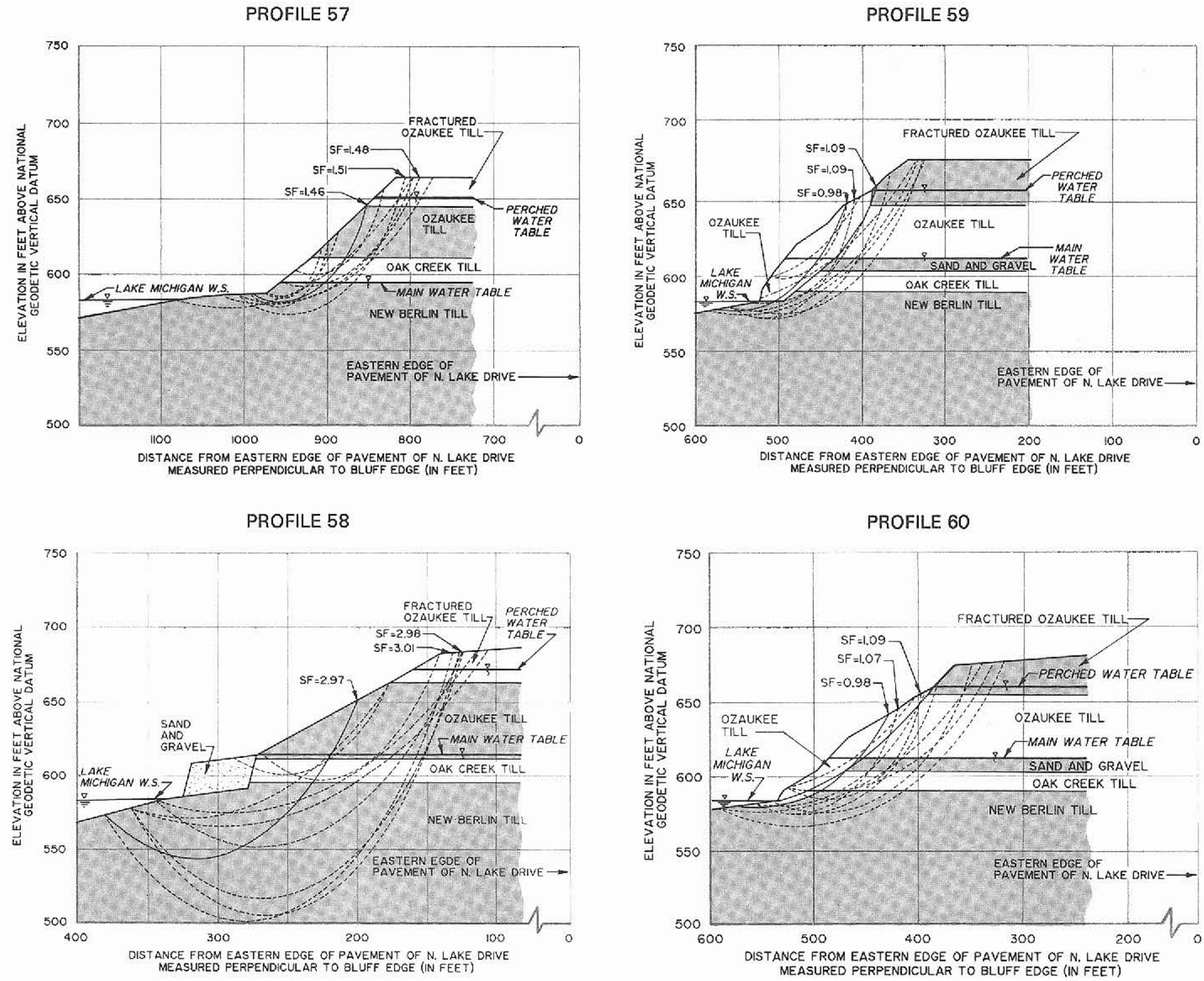
Overall, Section 61 was also considered to have a stable bluff slope with respect to translational sliding. This was due in part to the gentle angle of the bluff slope, and in part to the good vegetative cover on the entire bluff face. There were, however, small disturbed soil areas observed on the upper portion of the bluff slope where translational sliding may have occurred. These small isolated slides, however, did not appear to be threatening the stability of the overall bluff slope.

Due primarily to the relatively wide beach built up in Section 61, no significant bluff toe erosion was observed during the field survey conducted in the summer of 1986. Shore protection structures consisting of three bulkheads and one revetment provide additional toe protection for 65 percent of the shoreline. Thus, under existing shoreline and lake level conditions, wave action did not appear to substantially affect the toe of the bluff. However, during the study period, the beaches were eroding rapidly. Should beach erosion continue or the lake levels remain relatively high, the potential for toe erosion will increase, primarily in the northern portion of the section.

No measures are needed to prevent rotational sliding within Bluff Analysis Section 61. Revegetation of the scattered disturbed soil areas

Figure 62

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSES FOR PROFILES 57-60



Source: T. B. Edil, D. M. Mickelson, and SEWRPC.

within the upper portion of the bluff slope is recommended to prevent the occurrence of translational sliding. Additional toe protection measures are recommended within the northern portion of the section to prevent erosion from wave and ice action.

Bluff Analysis Section 62: The stability of the bluff slope within Section 62, which extends from 3378 to 3474 N. Lake Drive (north of Newport Avenue extended) in the City of Milwaukee, was characterized by the use of Profile No. 58.

The results of the deterministic slope stability analysis, shown in Figure 62, indicate that Profile No. 58 has a stable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 2.97, and was located within the lower two-thirds of the bluff slope. The next nine lowest safety factors ranged from 2.98 to 3.13.

The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 58 were all well above 1.0, with values ranging from 2.01 to 2.89. Based on both the deterministic and probabilistic slope stability analyses and on the observed bluff conditions, Section 62 was considered to have a stable bluff slope with respect to rotational sliding.

Overall, Section 62 was also considered to have a stable bluff slope with respect to translational sliding. This was due in part to the gentle angle of the bluff slope and in part to the good vegetative cover on the entire bluff face. However, portions of the vegetative cover on a ravine located just south of 3432 N. Lake Drive had been cleared, which may increase the risk of translational sliding. These slides, however, would probably not threaten the stability of the overall bluff slope.

During the field survey conducted in the summer of 1986, 25 percent of the shoreline within the section was partially protected by a collapsed concrete bulkhead. The alluvial fan located at the base of the ravine had experienced significant erosion due to wave action. However, because of the width of that fan, the resulting toe erosion should not affect the stability of the bluff slope. Should erosion of the fan continue, the attendant risk of the toe erosion affecting the overall stability of the bluff would increase.

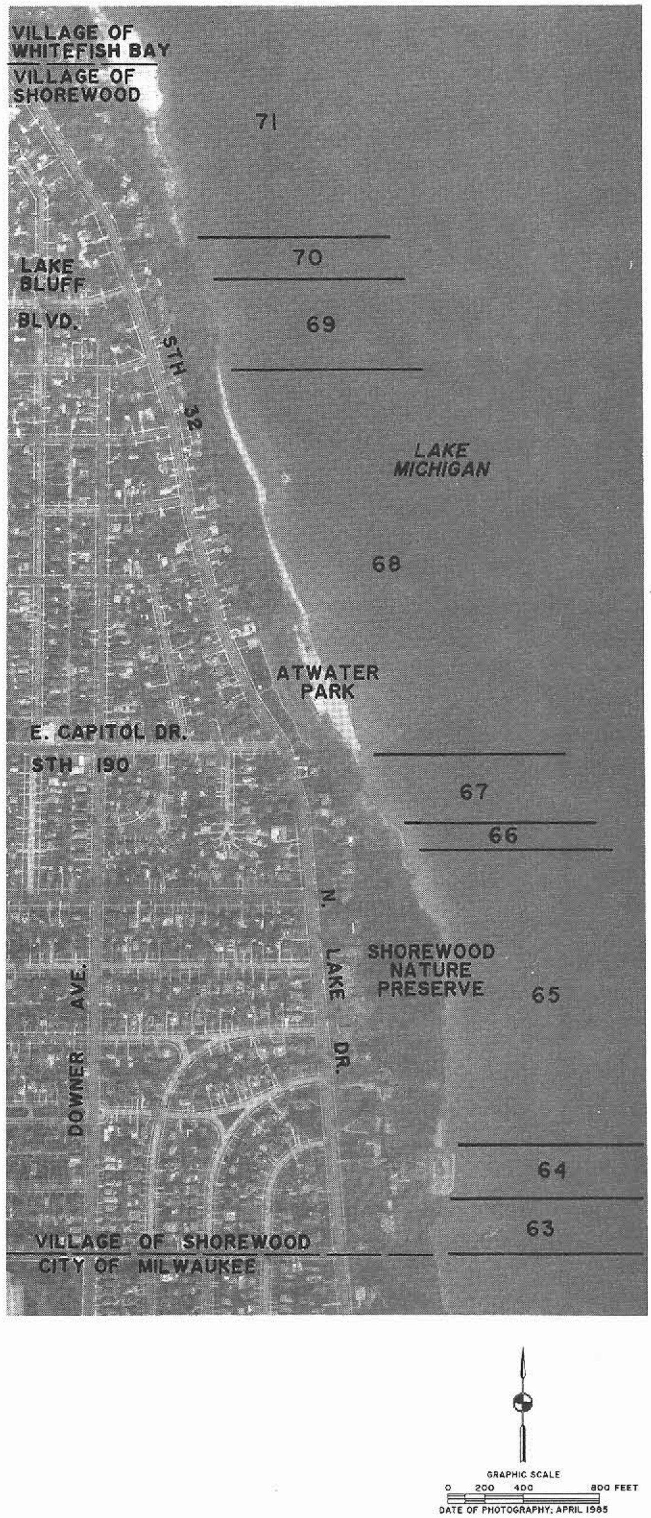
No measures are needed to prevent rotational sliding within Bluff Analysis Section 62. Surface runoff control and the establishment of a good vegetative cover on the land that was cleared is recommended within this section, especially on the steep ravine slopes, to prevent the occurrence of translational sliding. Bluff toe protection is recommended to prevent erosion by wave and ice action.

Bluff Analysis Section 63: Bluff Analysis Sections 63 through a portion of 71 lie within the Village of Shorewood, as shown in Figure 63. The stability of the bluff slopes within Section 63, which is located at 3510 N. Lake Drive (near Edgewood Avenue extended) in the Village of Shorewood, was characterized by the use of Profile No. 59 and Profile No. 60.

The results of the deterministic slope stability analyses, shown in Figure 62 for Profile No. 59 and Profile No. 60, indicate a threat of bluff slope failure with respect to rotational sliding. The lowest failure surface calculated at Profile No. 59 had a safety factor of 0.98, and was located within the lower two-thirds of the bluff

Figure 63

BLUFF ANALYSIS SECTIONS WITHIN THE VILLAGE OF SHOREWOOD



Source: SEWRPC.

slope within old slump block material. The next nine lowest safety factors ranged from 1.09 to 1.38. The lowest failure surface calculated at Profile No. 60 had a safety factor of 0.98, and was also located within the lower two-thirds of the bluff slope. The next nine lowest safety factors ranged from 1.07 to 1.38.

The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 59 ranged from 0.62 to 1.08, with 13, or 65 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated at Profile No. 59, 63, or 32 percent, had safety factors of less than 1.0. The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 60 ranged from 0.81 to 1.15, with 11, or 55 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated at Profile No. 60, 29, or 15 percent, had safety factors of less than 1.0.

During the field survey conducted in the summer of 1986, the slump block located on the lower portion of the bluff slope was experiencing some slope failure. Thus, there was some indication of sliding at the bottom of the bluff slope, as predicted by the slope stability analyses. Based on both the deterministic and probabilistic slope stability analyses and on the observed bluff conditions, Section 63 was considered to have a marginal bluff slope with respect to rotational sliding.

Overall, Section 63 was also considered to have a marginal bluff slope with respect to translational sliding. There was vegetative cover on most of the slump block and on the remaining bluff slope. However, in some areas the vegetative cover was sparse, and there was an increased potential for translational sliding because of the relatively steep angle of the bluff slope. The potential for translational sliding was further enhanced within the lower two-thirds of the bluff slope, where groundwater seepage was noted during the field surveys.

Bluff toe erosion was observed in portions of Section 63 during the field survey conducted in the summer of 1986. Bluff toe erosion within this section may be threatening the stability of the bluff slope, especially within the slump block which covers the lower portion of the slope. Shore protection structures present in the section in 1986 included one concrete bulkhead covering about 150 feet, or 50 percent of shoreline within the section. This structure was in need of major maintenance or reconstruction at the time of the survey.

To prevent rotational sliding, as well as to provide protection against wave and ice erosion at the toe of the bluff, it is recommended that actions be taken to prevent further failure of the slump block which lies at the base of the slope in the northern part of Section 63. It is recommended that the base of the slump be regraded to a stable slope angle and revegetated; that toe protection be provided at the base of the slump block; and that surface runoff control be utilized to prevent the accumulation of water on the top of the slump block. The toe protection measure selected should be flexible so that the structure will not be damaged by slight movement of the slump block. Toe protection should be provided along the entire shoreline of the section. Maintenance of a good vegetative cover on the entire bluff slope is recommended to prevent the occurrence of translational sliding.

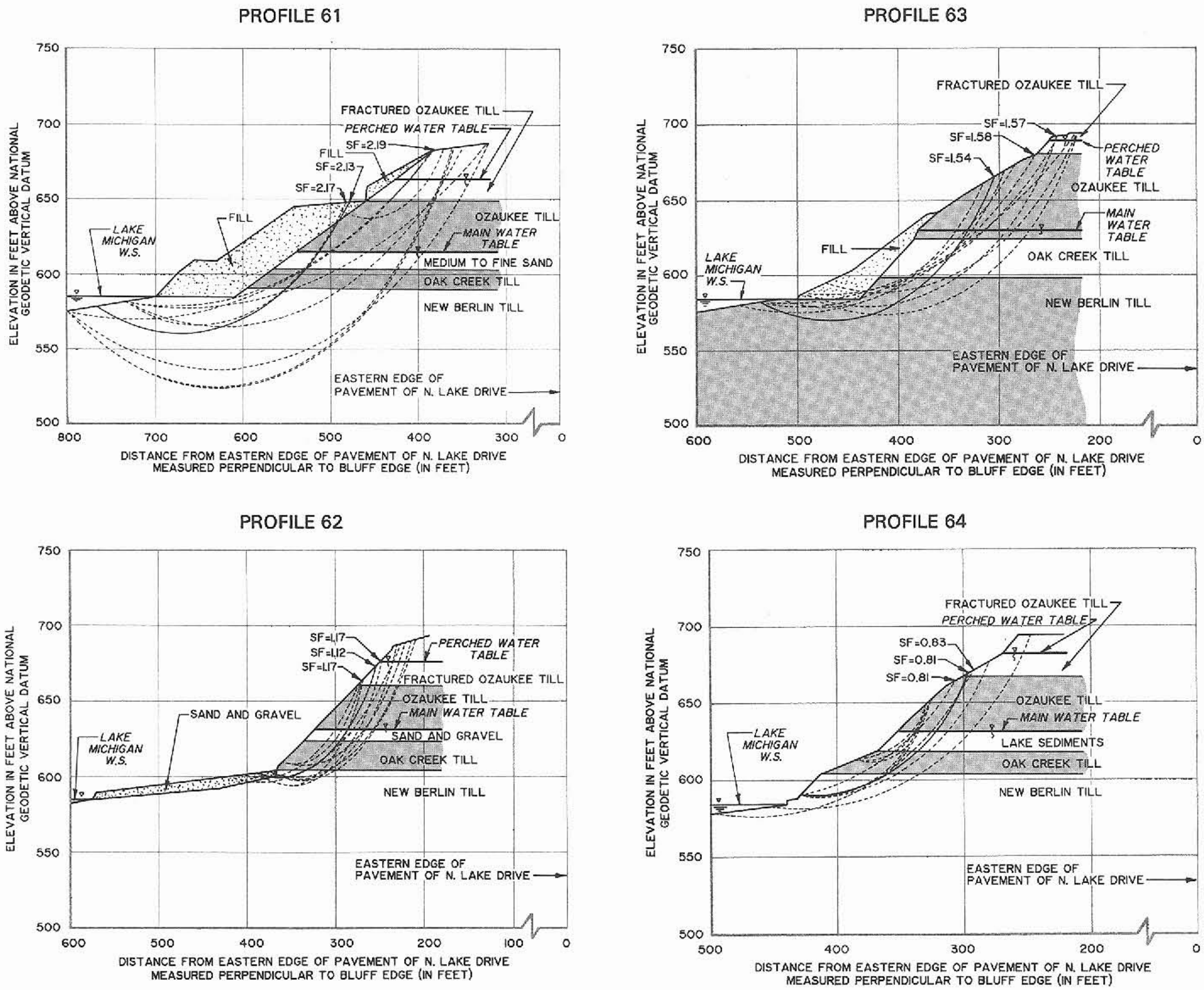
Bluff Analysis Section 64: The stability of the fill and the underlying bluff slope within Section 64, which is located at 3534 N. Lake Drive south of Shepard Avenue extended) in the Village of Shorewood, was characterized by the use of Profile No. 61.

The results of the deterministic slope stability analysis for Profile No. 61 are shown in Figure 64. The lowest failure surface calculated at this profile site had a safety factor of 2.13, and was located on the lower portion of the bluff slope beneath the fill layer. The next nine lowest safety factors ranged from 2.17 to 2.31. A probabilistic slope stability analysis was not conducted for this section because it is a fill site. Based on the deterministic slope stability analysis and on the observed bluff conditions, Section 64 was considered to have a stable bluff slope with respect to rotational sliding.

Section 64 was also considered to have a stable bluff slope with respect to translational sliding. In general, translational sliding within sites covered with concrete rubble and soil fill was considered unlikely because of the ability of the fill material to maintain a relatively steep slope, and because of the benefits realized by loading the base of the slope. A large amount of fill material had been placed at the base of the natural bluff slope within Section 64.

Figure 64

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSES FOR PROFILES 61-64



Source: T. B. Edil, D. M. Mickelson, and SEWRPC.

Primarily due to the effectiveness of the rock and rubble revetment placed at the toe of the fill in Section 64, as well as an offshore breakwater, no significant bluff toe erosion was observed during the field survey conducted in the summer of 1986. Should maintenance of the revetment not be provided as necessary, the potential for erosion at the toe of the fill would increase. Although the risk of rotational sliding was slight, it is recommended that the top of the terraced fill be regraded to allow surface water to flow toward the lake, rather than accumulating on top of the fill. For aesthetic purposes, it is also recommended that the fill be covered with a two-foot-thick layer of soil and revegetated. Toe erosion control measures are not needed, other than maintenance of the existing rock and concrete rubble revetment.

Bluff Analysis Section 65: The stability of the bluff slope within Section 65, which extends from 3550 to 3914 N. Lake Drive (between Shepard Avenue extended and about Shorewood Avenue extended) in the Village of Shorewood, was characterized by the use of Profile No. 62.

The results of the deterministic slope stability analysis, shown in Figure 64, indicate that Profile No. 62 has a stable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 1.12. The next nine lowest safety factors ranged from 1.17 to 1.25.

The lowest safety factors indicated by the 20 probabilistic stability analyses conducted ranged from 0.86 to 1.23, with four critical surfaces, or 20 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated, 17 surfaces, or 8 percent, had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 65 was considered to have a stable bluff slope with respect to rotational sliding. However, the probabilistic analysis did indicate a slight potential for slope failure depending upon the specific conditions within the bluff.

Overall, Section 65 was also considered to have a stable bluff slope with respect to translational sliding. This was due primarily to the good vegetative growth which covered the entire bluff face, and also to the relatively low bluff slope angle of about 25 degrees. Since there were no disturbed soil areas observed during the 1986 field survey within this section, the potential for translational sliding appeared to be minimal.

The Nipissing terrace present at the base of the bluff had experienced significant erosion because of inadequate protection against wave and ice action, and because the material the terrace is composed of is easily eroded. Because, however, the terrace was approximately 300 feet wide, the resulting toe erosion was not affecting the stability of the overall bluff slope.

No measures are needed to prevent rotational or translational sliding within Bluff Analysis Section 65. Bluff toe protection is recommended to protect the terraced portion of the section, which includes the Shorewood Nature Preserve.

<u>Bluff Analysis Section 66</u>: The stability of the fill and the underlying bluff slope within Section 66, which is located at 3926 N. Lake Drive in the Village of Shorewood, was characterized by the use of Profile No. 63.

The results of the deterministic slope stability analysis for Profile No. 63 are shown in Figure 64. The lowest failure surface calculated at this profile site had a safety factor of 1.54. The next nine lowest safety factors ranged from 1.57 to 1.61. A probabilistic slope stability analysis was not conducted for this section because it is a fill site. Based on the deterministic slope stability analysis and on the observed bluff conditions, Section 66 was considered to have a stable bluff slope with respect to rotational sliding.

Section 66 was also considered to have a stable bluff slope with respect to translational sliding. In general, translational sliding within sites covered with concrete rubble and soil fill was considered unlikely because of the ability of the fill material to maintain a relatively steep slope, and because of the benefits realized by loading the base of the slope. A large amount of fill material had been placed at the base of the natural bluff slope, which reduced the overall slope angle. Furthermore, a good vegetative cover had been established on the fill.

Although the toe of the bluff was protected by a rubble and concrete block revetment, it had experienced erosion due to wave action. However, because of the large amount of fill material at the base of the bluff, the resulting toe erosion was not affecting the stability of the bluff slope.

No measures are needed to prevent rotational or translational sliding within Bluff Analysis Section 66. Additional bluff toe protection is recommended to prevent erosion from wave and ice action.

<u>Bluff Analysis Section 67</u>: The stability of the bluff slope within Section 67, which extends from 3932 to 3966 N. Lake Drive (just south of Capitol Drive extended) in the Village of Shorewood, was characterized by the use of Profile No. 64.

The results of the deterministic slope stability analysis, shown in Figure 64, indicate that Profile No. 64 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.81, and was located within the middle portion of the bluff slope. The next nine lowest safety factors ranged from 0.81 to 0.88.

Of the 20 probabilistic stability analyses conducted, the lowest safety factors ranged from 0.51 to 0.90. Of the 200 failure surfaces evaluated, 193 surfaces, or 96 percent, had safety factors of less than 1.0. Four houses were located within 50 feet of the edge of the bluff. Based on both the deterministic and probabilistic slope stability analyses and on the observed bluff conditions, Section 67 was considered to have an unstable bluff slope with respect to rotational sliding.

Section 67 was also considered to have an unstable bluff slope with respect to translational sliding. This was due in part to the lack of vegetative cover on most of the bluff slope, and in part to the relatively steep angle of the bluff slope. The potential for translational sliding was further enhanced by surface stormwater runoff and by broken drainage tiles which were leaking onto the bluff face.

Bluff toe erosion was observed within the entire shoreline of Section 67 during the field survey conducted in the summer of 1986, and was identified as a primary cause of bluff slope failure. Shore protection structures present in the section in the summer of 1986 included a 400-foot concrete bulkhead backfilled with rubble. In the southern portion of the section, two layers of grout-filled bags were placed behind the bulkhead. These shore protection structures were not providing adequate protection against wave action.

To abate the severe potential for both rotational and translational sliding, it is recommended that the bluff slope be regraded to a stable slope angle and revegetated. This action may require filling, since cutting back the top of the slope may not be feasible because some houses at the top of the bluff are as close as 20 feet from the bluff edge. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 68: The stability of the bluff slope within Section 68, which extends from Atwater Park to 4216 N. Lake Drive (north of Capitol Drive extended) in the Village of Shorewood, was characterized by the use of Profile No. 65.

The results of the deterministic slope stability analysis, shown in Figure 65 for Profile No. 65, indicate a threat of bluff slope failure with respect to rotational sliding. The lowest failure surface calculated at Profile No. 65 had a safety factor of 0.99, and was located on the lower two-thirds of the bluff. The next nine lowest safety factors ranged from 1.05 to 1.15.

The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 65 ranged from 0.66 to 1.17, with 12 failure surfaces, or 60 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated at Profile No. 65, 90, or 45 percent, had safety factors of less than 1.0.

During the 1986 field survey, the southern portion of the section, which includes Atwater Park, was terraced, with no signs of slope failure. Evidence of past slope surface movement was observed north of the park. Therefore, this section was divided into two parts. Based on field observations, the southern portion of the section, consisting of Atwater Park, was considered stable with respect to rotational sliding. The portion of the section north of the park was considered to have a marginal bluff slope with respect to rotational sliding, based on both the deterministic and probabilistic slope stability analyses and on the observed bluff conditions.

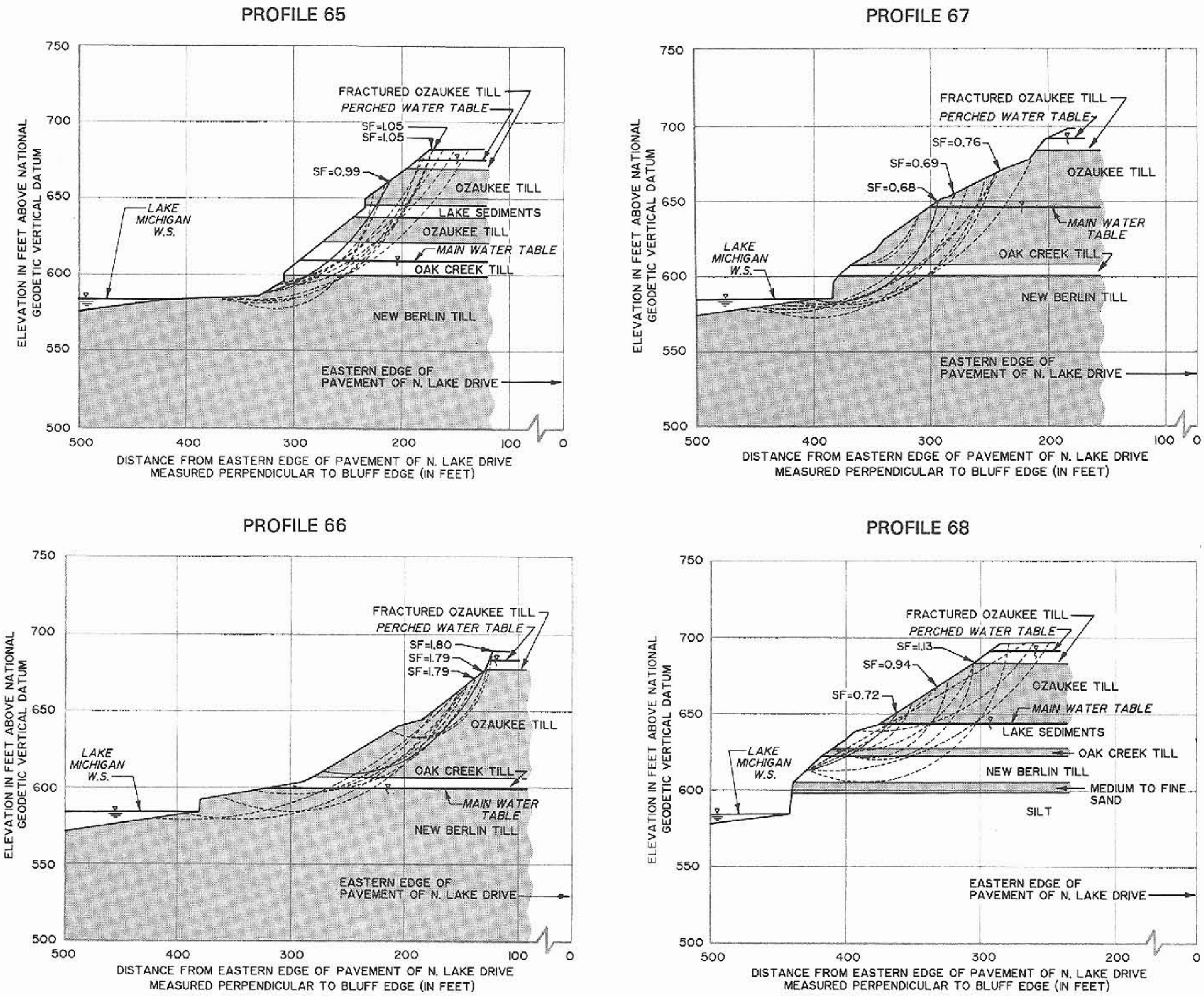
Section 68 was considered to have a stable bluff slope with respect to translational sliding. This was due to the good vegetative growth that covered the entire bluff face. No major disturbed soil areas were observed during the field surveys conducted within this section.

Due primarily to the relatively wide beach built up in Section 68, only minor bluff toe erosion was observed—and only in the northern portion of the section—during the field survey conducted in the summer of 1986. Thus, under existing shoreline and lake level conditions, wave action did not appear to substantially affect the toe of the bluff. However, during the study period, the beaches were eroding rapidly. Should beach erosion continue or the lake levels remain relatively high, the potential for toe erosion would increase, thereby increasing the potential for slope failure in the marginally unstable portion of the section.

No measures are needed to prevent rotational sliding with the southern portion of Bluff Analysis Section 68, which includes Atwater Park. Measures should be undertaken to maintain the beach at Atwater Park. In order to prevent rotational sliding in the northern portion of the section, it is recommended that a detailed groundwater study be conducted to determine whether a groundwater drainage system needs to be installed to lower the ground-

Figure 65

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSES FOR PROFILES 65-68



Source: T. B. Edil, D. M. Mickelson, and SEWRPC.

water elevation. Bluff toe protection is recommended within the northern 1,380 of Section 8 to prevent erosion from wave and ice action.

Bluff Analysis Section 69: The stability of the bluff slope within Section 69, which extends from 4226 to 4320 N. Lake Drive (south of Lake Bluff Boulevard extended) in the Village of Shorewood, was characterized by the use of Profile No. 66.

The results of the deterministic slope stability analysis, shown in Figure 65, indicate that Profile No. 66 has a stable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 1.79. The next nine lowest safety factors ranged from 1.79 to 1.90. The Nipissing terrace present at the base of the bluff helped improve the stability of the bluff slope.

The lowest safety factors indicated by the 20 probabilistic stability analyses conducted ranged from 0.74 to 1.99, with only one failure surface, or 5 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated, only four surfaces, or 2 percent, had a safety factor of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 69 was considered to have a stable bluff slope with respect to rotational sliding.

Overall, Section 69 was also considered to have a stable bluff slope with respect to translational sliding. This was due to the good vegetative growth covering most of the bluff face. The potential for translational sliding was slightly higher in the upper portion of the bluff, where the slope was steeper and the vegetative cover relatively sparse.

The Nipissing terrace had experienced significant erosion by wave action. As of 1986 there were no shore protection structures located within this section. Where the terrace ranged from about 30 to 100 feet in width, the erosion would not be expected to affect the stability of the bluff slope. However, the terrace is much narrower at the northern end of the section. Further toe erosion in this shoreline area may begin to affect the stability of the bluff slope.

No measures are needed to prevent rotational or translational sliding within Bluff Analysis Section 69. Bluff toe protection is recommended to protect the terraced portion of the section, especially within the northern 150 feet of the section where the terrace narrows.

<u>Bluff Analysis Section 70</u>: The stability of the bluff slope within Section 70, which extends from 4400 to 4408 N. Lake Drive (north of Lake Bluff Boulevard extended) in the Village of Shorewood, was characterized by the use of Profile No. 67.

The results of the deterministic slope stability analysis, shown in Figure 65 for Profile No. 67, indicate that the bluff slope is potentially unstable with respect to rotational sliding. The lowest failure surface calculated at Profile No. 67 had a safety factor of 0.68, and was located on the lower half of the bluff. The next nine lowest safety factors ranged from 0.69 to 0.88.

The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 67 ranged from 0.61 to 0.97. Of the 200 failure surfaces evaluated at Profile No. 67, 160, or 80 percent, had safety factors of less than 1.0.

When the field survey was conducted in the summer of 1986, the overall bluff slope within Section 70 was well vegetated, although some slope movement had occurred, and some soil areas were exposed. The elevation of the groundwater shown in Figure 65 was measured in an observation well installed in 1986 at 4408 N. Lake Drive. The slope stability analyses indicated that some slope failures may be expected to occur on the slump block lying on the lower portion of the slope. Both houses within this section were located within 50 feet of the top edge of the bluff. A bulkhead present at the base of the slope was modified in 1985 by a local contractor to help buttress the slope and prevent further slope failure. The contractor has indicated that the bulkhead was structurally intact. The probability that the bulkhead will be able to effectively hold the slope and prevent a major failure cannot be evaluated at the systems planning level. It is therefore recommended that a site-specific analysis be conducted to properly evaluate the effect of the bulkhead on the stability of the bluff slope. The bluff slope would be classified as unstable if it is shown in this site-specific analysis that the bulkhead is not providing suitable protection.

Overall, Section 70 exhibited a slight potential for translational sliding. This was due to the good vegetative growth covering most of the bluff slope. However, in areas where there was little vegetation, there would be a moderate potential for translational sliding because of the relatively steep angle of the bluff slope and the relatively high elevation of the groundwater.

Bluff toe erosion was observed in Section 70 during the field survey conducted in the summer of 1986. The toe of the bluff was protected by a 200-foot-long concrete bulkhead which was being overtopped, especially at the southern end. While the bulkhead offered some protection, there was severe erosion from waves washing over the top of the structure. This toe erosion was contributing to the instability of the bluff slope.

To prevent rotational sliding, as well as to provide protection against wave and ice action at the toe of the bluff, it is recommended that adequate bluff toe protection be provided within Section 70. It is also recommended that exposed soil areas be revegetated. As noted above, a site-specific analysis of the effect of the bulkhead on the stability of the slope should be conducted.

Bluff Analysis Section 71: A portion of Bluff Analysis Section 71 lies within the Village of Shorewood, and a portion lies within the Village of Whitefish Bay. The bluff analysis sections that are located within the Village of Whitefish Bay are shown in Figure 66. Bluff Analysis Section 71 was a fill project under construction during the summer of 1986. The stability of the fill and the underlying bluff slope within Section 71, which extends from 4424 to 4652 N. Lake Drive, was characterized by the use of three profile sites, which illustrate the section prior to, and during construction of, the fill project. Profile No. 68 was used to represent the bluff slope conditions of the section prior to the fill project, because filling had not yet occurred at that profile site at the time the profile was prepared. Profile No. 69 and Profile No. 70 represent the bluff slope conditions in the summer of 1986 during the construction of the fill project.

The results of the deterministic slope stability analysis for the prefill conditions, shown in Figure 65, indicate that Profile No. 68 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.72. The next nine lowest safety factors ranged from 0.94 to 1.38. The results of the deterministic slope stability analyses for Profile No. 69 and Profile No. 70, shown in Figure 67, indicate that the bluff slope was stable during the construction of the fill. The lowest failure surface calculated at Profile No. 69 had a safety factor of 1.44, and was located within the fill material. The next nine lowest safety factors ranged from 1.82 to 2.48. The lowest failure surface calculated at Profile No. 70 had a safety factor of 2.11, and was located within the fill material. The next nine lowest safety factors ranged from 2.14 to 3.37. A probabilistic slope stability analysis was not conducted for this section because it is a fill site. Based on the deterministic slope stability analyses at Profile Nos. 69 and 70, on the observed bluff conditions, and on the anticipated geometry of the fill project when completed, Section 71 was considered to have a stable bluff slope with respect to rotational sliding. It should be noted that at the southern and northern ends of the section, fill was being placed only on the lower portion of the bluff slope. Shoreline areas where fill is placed only at the toe of the bluff may not be as stable as the bluffs shown in Profile Nos. 69 and 70.

Section 71 was also considered to have a stable bluff slope with respect to translational sliding. In general, translational sliding within sites covered with concrete rubble and soil fill was considered unlikely because of the ability of the fill material to maintain a relatively steep slope, and because of the benefits realized by loading the base of the slope. It was anticipated that a large amount of fill material would be placed at the base of the natural bluff slope within Section 71.

Erosion at the toe of the bluff was not evaluated in this section because construction of the fill was still in progress at the time of the field surveys. Toe erosion may be expected to occur if adequate toe protection is not provided at the base of the bluff following completion of the fill project.

No measures are needed to prevent rotational or translational sliding within Bluff Analysis Section 71. It is recommended that adequate toe protection be provided at the base of the fill, when completed, to prevent erosion by wave and ice action.

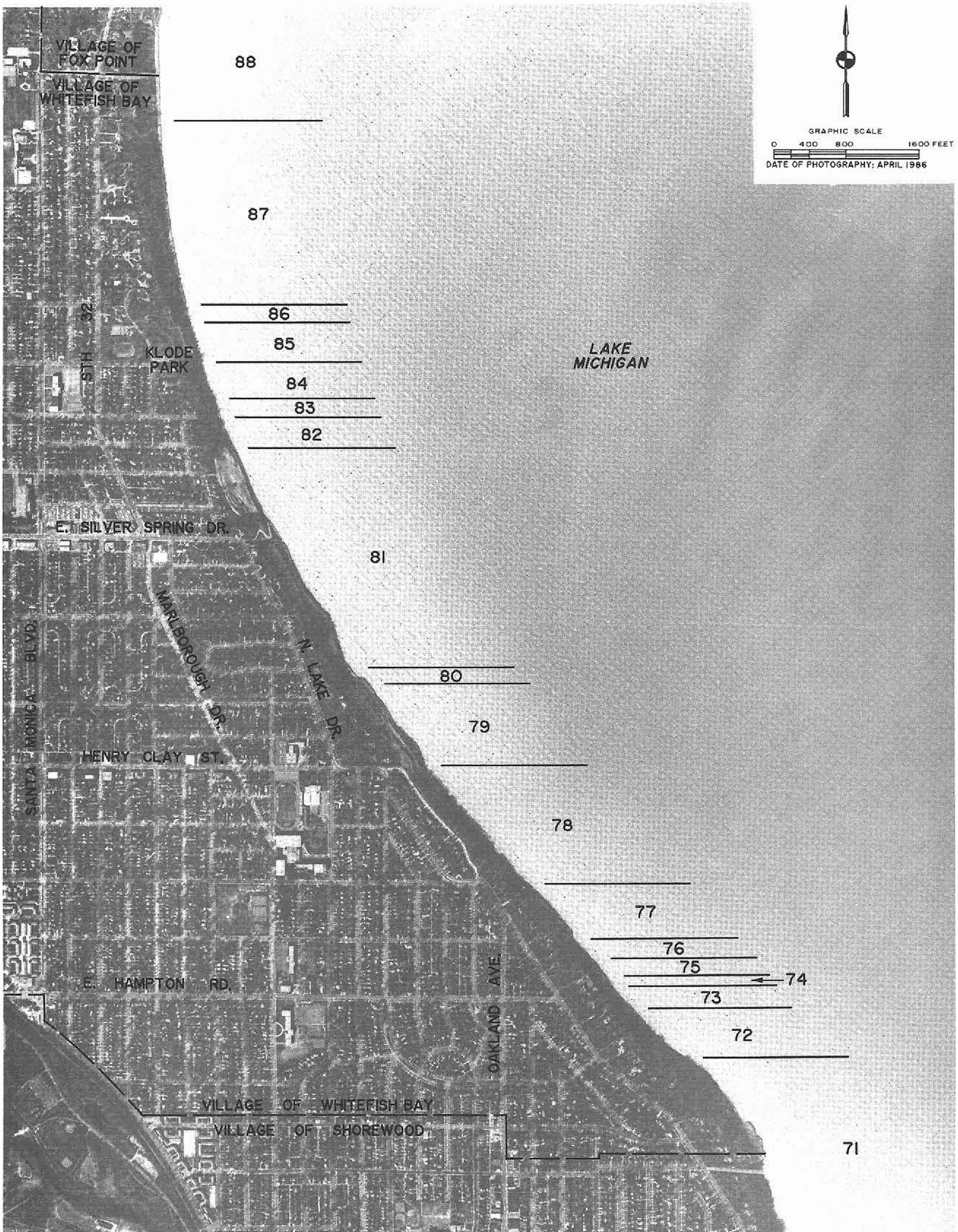
Bluff Analysis Section 72: The stability of the bluff slope within Section 72, which extends from 4668 to 4730 N. Lake Drive (south of Hampton Avenue extended) in the Village of Whitefish Bay, was characterized by the use of Profile No. 71 and Profile No. 72.

The results of the deterministic slope stability analyses, shown in Figure 67 for Profiles No. 71 and No. 72, indicate that the bluff slope is unstable with respect to rotational sliding. The lowest failure surface calculated at Profile No. 71 had a safety factor of 0.64 and included the entire bluff slope. The next nine lowest safety factors ranged from 0.78 to 0.94. The lowest failure surface calculated at Profile No. 72 had a safety factor of 0.66, and also included the entire bluff slope. The next nine lowest safety factors ranged from 0.72 to 0.83.

The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 71 ranged from 0.50 to 0.97. Of the 200 failure surfaces evaluated at Profile No. 15, 185, or 92 percent, had safety factors of less than 1.0. The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 72 ranged from 0.52 to 0.81. Of the 200 failure surfaces evaluated at Profile No. 72,

Figure 66

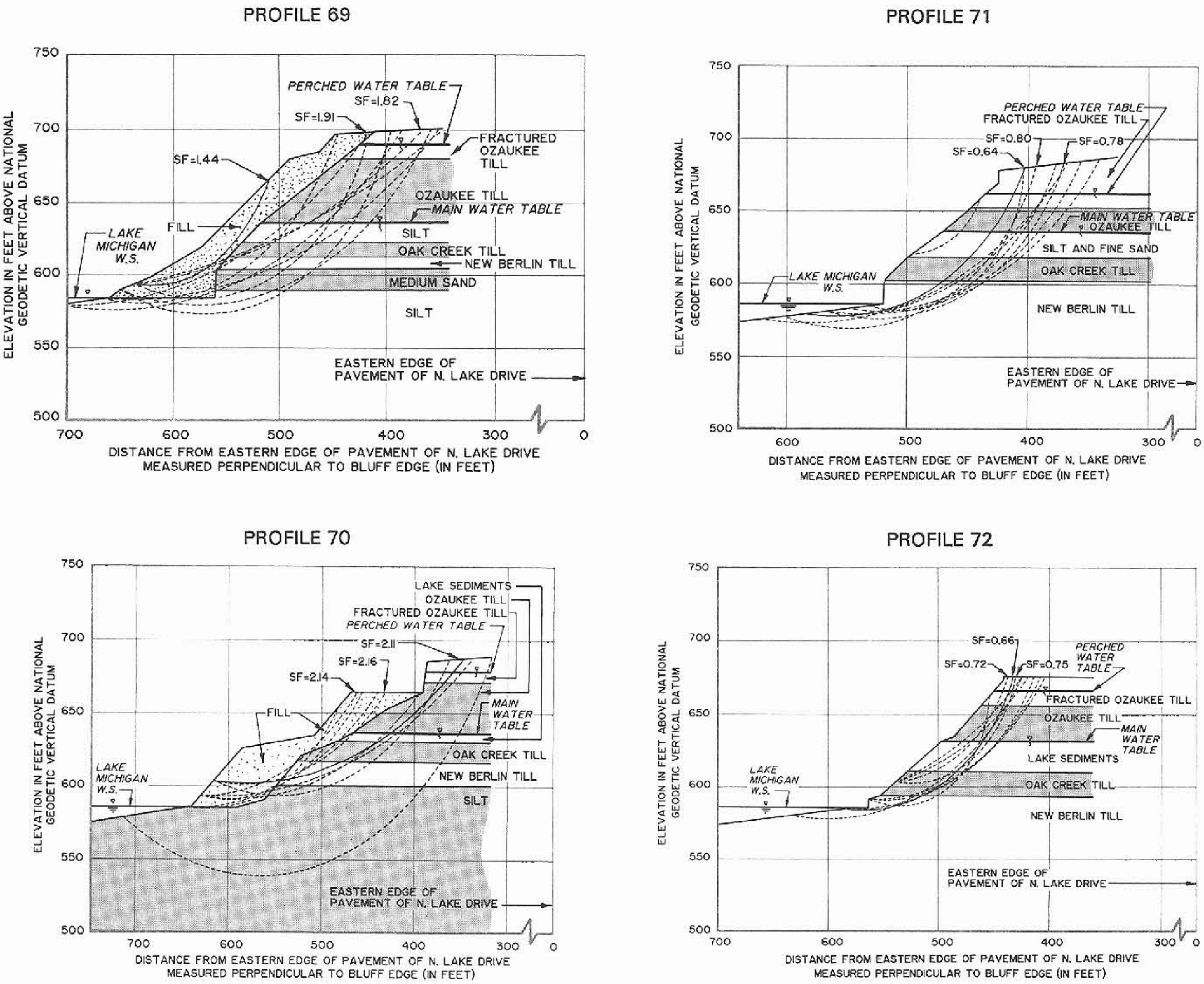
BLUFF ANALYSIS SECTIONS WITHIN THE VILLAGE OF WHITEFISH BAY



Source: SEWRPC.

Figure 67

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSES FOR PROFILES 69-72



Source: T. B. Edil, D. M. Mickelson, and SEWRPC.

186, or 93 percent, had safety factors of less than 1.0. Two houses were located within 50 feet of the top edge of the bluff. Based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 72 was considered to have an unstable bluff slope with respect to rotational sliding.

Section 72 was also considered to have an unstable bluff slope with respect to translational sliding. This was due in part to the lack of vegetative cover on most of the bluff slope, and in part to the relatively steep angle of the bluff slope.

Bluff toe erosion was observed along the entire shoreline of Section 72 during the field survey conducted in the summer of 1986, and was identified as a primary cause of bluff slope failure. Shore protection structures present in the section in the summer of 1986 included two concrete bulkheads, each 100 feet in length, which were being overtopped and flanked and were in need of maintenance at the time of the survey, and a 100-foot-long revetment still under construction, composed of limestone rock and grout-filled bags. The remaining shoreline within the section was not protected by shore protection structures at the time of the survey.

To abate the severe potential for both rotational and translational sliding, it is recommended that the bluff slope be regraded to a stable slope angle and revegetated. This action may require filling, since cutting back the top of the slope may not be feasible because houses at the top of the bluff are as close as 40 feet from the bluff edge. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 73: Bluff Analysis Section 73 was a fill project under construction during the summer of 1986. The stability of the fill and the underlying bluff slope within Section 73, which extends from 4744 to 4762 N. Lake Drive (near Hampton Avenue extended) in the Village of Whitefish Bay, was characterized by the use of Profile No. 73.

The results of the deterministic slope stability analysis, shown in Figure 68, indicate that Profile No. 73 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.61 and was located beneath the layer of fill material, within the natural bluff. The next nine lowest safety factors ranged from 0.67 to 0.86. A probabilistic slope stability analysis was not conducted for this section because it is a fill site. One house was located within 50 feet of the top edge of the bluff. Section 73 was considered to have an unstable bluff slope with respect to rotational sliding.

Although translational sliding within fill areas was generally considered unlikely, the potential for sliding was evaluated within this section because of the thin layer of fill placed on the natural bluff slope. Overall, Section 73 was considered to have an unstable bluff slope with respect to translational sliding. This was due in part to the lack of vegetative cover on the bluff slope, and in part to the steep angle of the slope.

Bluff toe erosion was observed within the entire shoreline of Section 73 during the field survey conducted in the summer of 1986. This toe erosion was contributing to the instability of the bluff slope. No shore protection structures were located within this section in 1986.

To abate the severe potential for both rotational and translational sliding, it is recommended that the bluff slope be regraded to a stable slope angle. This action may require filling, since cutting back the top of the slope may not be feasible because houses at the top of the bluff are as close as 40 feet from the bluff edge. Bluff toe protection is recommended to prevent erosion from wave and ice action. As previously noted, the evaluation for Section 73 was based on the conditions of the bluff slope as of the summer of 1986, at which time a fill project was in progress, and therefore does not reflect the condition of the completed fill.

Bluff Analysis Section 74: The stability of the bluff slope within Section 74, located at 4780 N. Lake Drive (close to Hampton Avenue extended) in the Village of Whitefish Bay, was characterized by the use of Profile No. 74.

The results of the deterministic slope stability analysis, shown in Figure 68, indicate that Profile No. 74 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.80, and included the entire bluff slope. The next nine lowest safety factors ranged from 0.81 to 0.97.

The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 74 ranged from 0.55 to 0.82. Of the 200 failure surfaces evaluated at Profile No. 74, all had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 74 was considered to have an unstable bluff slope with respect to rotational sliding.

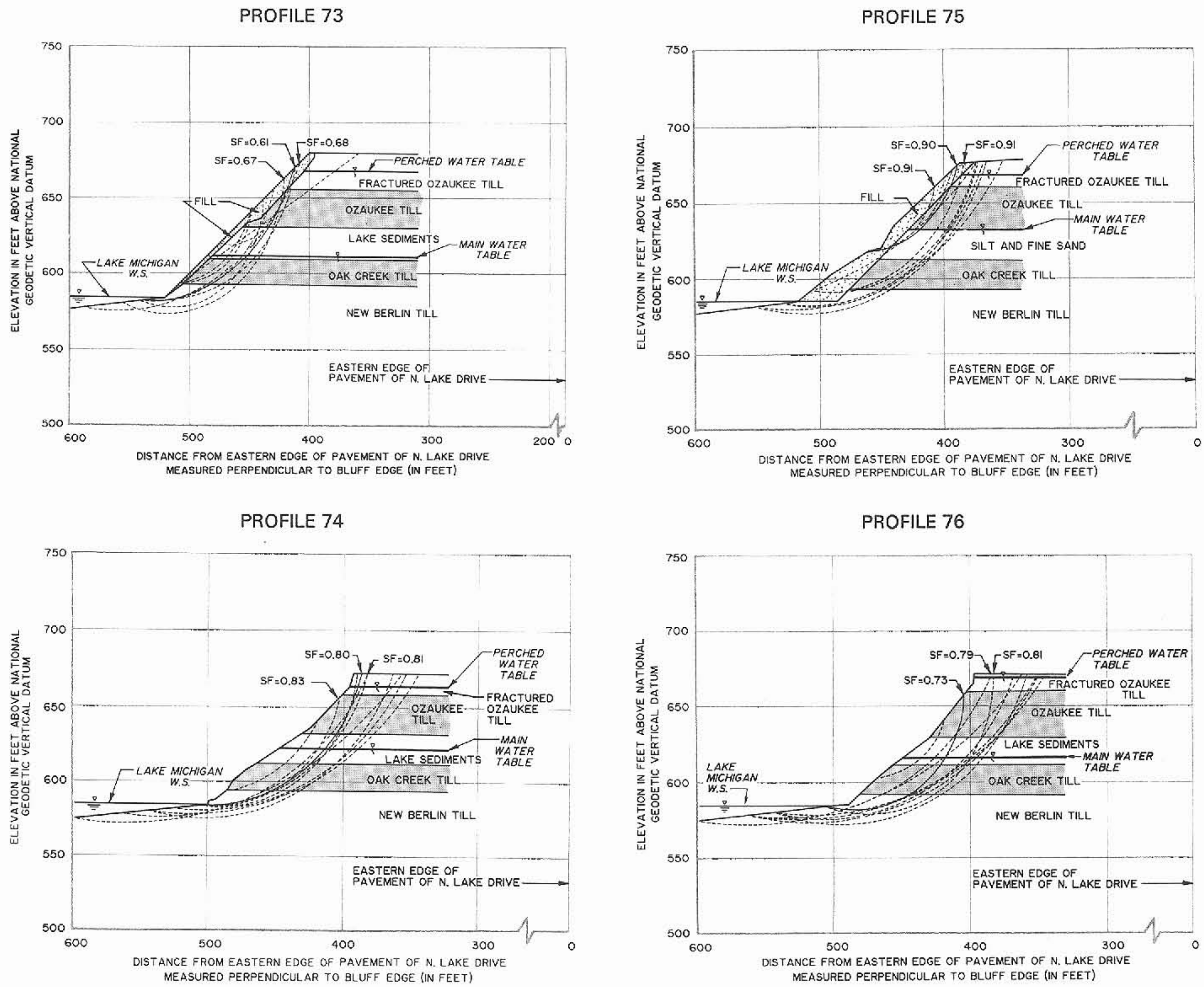
Section 74 was also considered to have an unstable bluff slope with respect to translational sliding. This was due in part to the lack of vegetative cover on most of the bluff slope, and in part to the relatively steep angle of the bluff slope.

Bluff toe erosion was observed within Section 74 during the field survey conducted in the summer of 1986. This toe erosion was affecting the stability of the bluff slope. As of 1986, no shore protection structures were located within this section.

To abate the severe potential for both rotational and translational sliding, it is recommended that the bluff slope be regraded to a stable slope angle and revegetated. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Figure 68

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSES FOR PROFILES 73-76



Source: T. B. Edil, D. M. Mickelson, and SEWRPC.

Bluff Analysis Section 75: Bluff Analysis Section 75 was a fill project under construction during the summer of 1986. The stability of the fill and the underlying bluff slope within Section 75, which extends from 4790 to 4800 N. Lake Drive (north of Hampton Avenue extended) in the Village of Whitefish Bay, was characterized by the use of Profile No. 75.

The results of the deterministic slope stability analysis, shown in Figure 68, indicate that Profile No. 75 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.90, and was located within the upper portion of the fill material. The next nine lowest safety factors ranged from 0.91 to 0.96. A probabilistic slope stability analysis was not conducted for this section because it is a fill site. One house was located within 50 feet of the top edge of the bluff. Section 75 was considered to have an unstable bluff slope with respect to rotational sliding.

Although translational sliding within fill areas was generally considered unlikely, the potential for slope failure by translational slides was evaluated within this section because of the relatively thin layer of fill placed on the natural bluff slope. Overall, Section 75 was considered to have an unstable bluff slope with respect to translational sliding, and some sliding of the fill material itself was observed.

Bluff toe erosion was observed within Section 75 during the field survey conducted in May 1986. This erosion was contributing to the instability of the bluff slope. During the summer of 1986, a revetment, 300 feet in length, composed of stone blocks and grout-filled bags was under construction. The effectiveness of this structure was not evaluated.

To abate the severe potential for both rotational and translational sliding, it is recommended that the bluff slope be regraded to a stable slope angle and revegetated. This action may require filling, since cutting back the top of the slope may not be feasible because houses at the top of the bluff are as close as 25 feet from the bluff edge. Bluff toe protection is recommended to prevent erosion from wave and ice action. As previously noted, the evaluation for Section 75 was based on the conditions of the bluff slope as of the summer of 1986, at which time a fill project was in progress, and therefore does not reflect the condition of the completed fill.

<u>Bluff Analysis Section 76</u>: The stability of the bluff slope within Section 76, which extends from 4810 to 4840 N. Lake Drive (south of Chateau Place extended) in the Village of Whitefish Bay, was characterized by the use of Profile No. 76.

The results of the deterministic slope stability analysis, shown in Figure 68, indicate that Profile No. 76 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.73, and was located within the lower portion of the bluff slope. The next nine lowest safety factors ranged from 0.79 to 0.94.

The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 76 ranged from 0.53 to 0.83. Of the 200 failure surfaces evaluated at Profile No. 76, 190, or 95 percent, had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 76 was considered to have an unstable bluff slope with respect to rotational sliding.

Section 76 was also considered to have an unstable bluff slope with respect to translational sliding. This was due in part to the lack of vegetative cover on most of the bluff slope, and in part to the relatively steep angle of the bluff slope.

Bluff toe erosion was observed within Section 76 during the field survey conducted in the summer of 1986, and was identified as a primary cause of bluff slope failure. As of 1986, no shore protection structures were located within this section.

To prevent rotational and translational sliding, as well as to provide protection against wave and ice action at the toe of the bluff, it is recommended that the bluff slope be regraded to a stable slope angle and revegetated, and that bluff toe protection be provided within Section 76.

<u>Bluff Analysis Section 77</u>: The stability of the fill and the underlying bluff slope within Section 77, which extends from 4850 N. Lake Drive to the southern portion of Buckley Park in the Village of Whitefish Bay, was characterized by the use of Profile No. 77 and Profile No. 78.

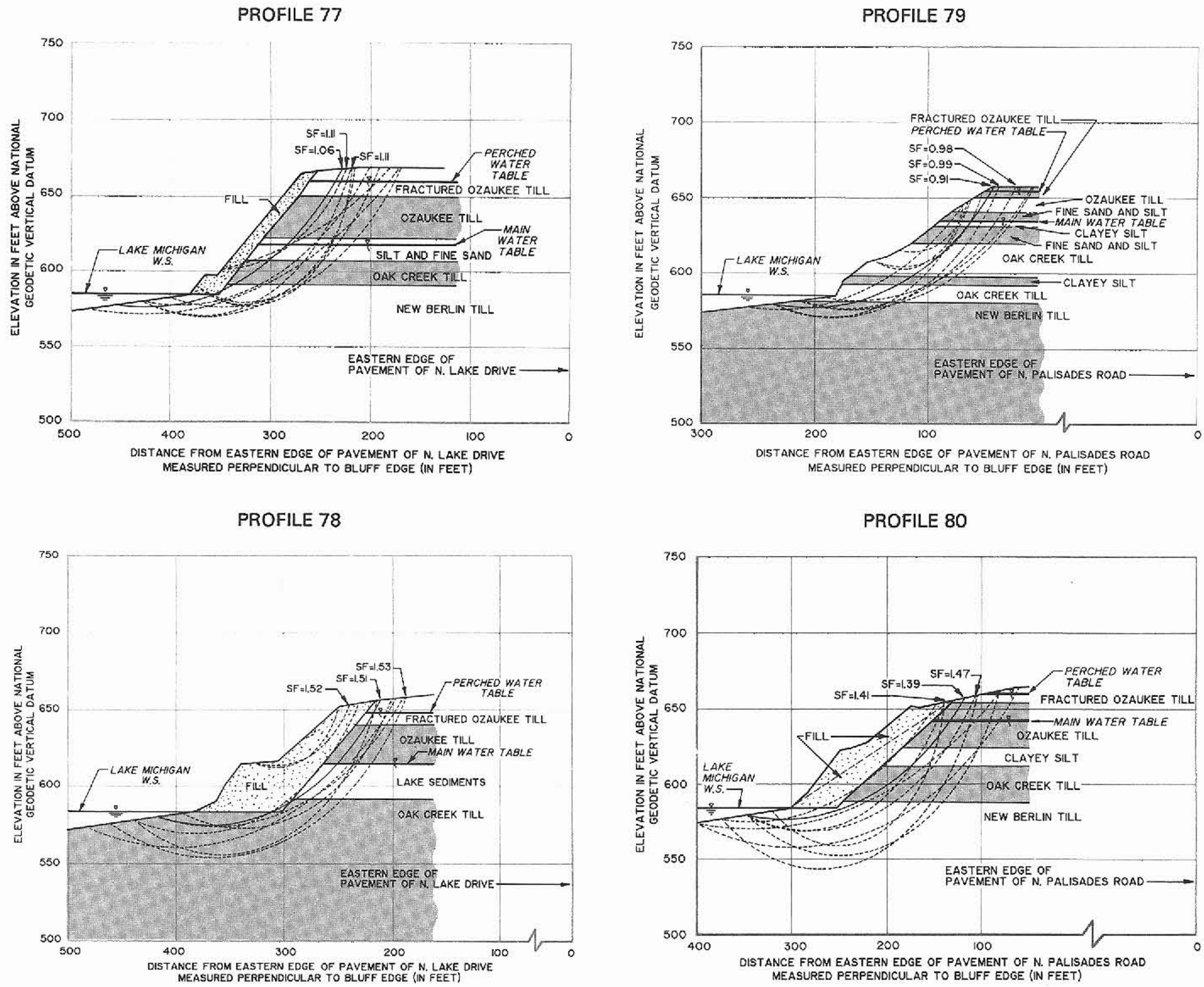
The results of the deterministic slope stability analyses, shown in Figure 69 for Profiles No. 77 and No. 78, indicate stable bluff slopes with respect to rotational sliding. The lowest failure surface calculated at Profile No. 77 had a safety factor of 1.06, and was located beneath the fill layer. The next nine lowest safety factors ranged from 1.11 to 1.44. The lowest failure surface calculated at Profile No. 78 had a safety factor of 1.51, and was also located beneath the fill layer. The next nine lowest safety factors ranged from 1.52 to 1.59.

A probabilistic slope stability analysis was not conducted for this section because it is a fill site. Therefore, based on the deterministic slope stability analysis and on the observed bluff conditions, Section 77 was considered to have a stable bluff slope with respect to rotational sliding.

Section 77 was also considered to have a stable bluff slope with respect to translational sliding. In general, translational sliding within fill areas was considered unlikely because of the ability of the fill material to maintain a relatively steep slope, and because of the benefits realized by loading the base of the slope. A large amount of fill material had been placed at the base of the natural bluff slope within Section 77.

Figure 69

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSES FOR PROFILES 77-80



Source: T. B. Edil, D. M. Mickelson, and SEWRPC.

Bluff toe erosion was observed within the northern portion of Section 77 during the field survey conducted in the summer of 1986. However, because of the large amount of fill material at the base of the bluff, this erosion was not affecting the stability of the bluff slope. Within the southern portion of the section, where the fill project was still under construction in 1986, a revetment composed of rubble and concrete slabs was being placed at the toe of the fill for protection against wave action. Because the structure had not been not completed as of time of the field survey, the degree of bluff toe protection provided could not be determined.

No measures are needed to prevent rotational or translational sliding within Bluff Analysis Section 77. Adequate toe erosion control measures should be maintained at the base of the fill to prevent erosion from wave and ice action.

Bluff Analysis Section 78: The stability of the bluff slope within Section 78, which includes the Village of Whitefish Bay Buckley Park and the southern portion of Milwaukee County Big Bay Park, was characterized by the use of Profile No. 79.

The results of the deterministic slope stability analysis, shown in Figure 69 for Profile No. 79, indicate a threat of bluff slope failure with respect to rotational sliding. The lowest failure surface calculated at Profile No. 79 had a safety factor of 0.91, and included the entire bluff slope. The next nine lowest safety factors ranged from 0.98 to 1.07.

The lowest safety factors indicated by the 20 probabilistic stability analyses ranged from 0.54 to 1.06, with 17 failure surfaces, or 70 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated, 155, or 78 percent, had safety factors of less than 1.0.

In the 1986 summer field survey, the bluff appeared quite stable, although some dislocation of trees was noted. However, in November 1986, a very large slump occurred at the southern end of this section in Buckley Park. Based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 78, overall, was considered to have a marginal bluff slope with respect to rotational sliding.

Section 78 was considered to have a stable bluff slope with respect to translational sliding. This was due to the good vegetative growth that covered most of the bluff face. There were, however, small disturbed soil areas observed on portions of the bluff slope, especially within the recent slope failure, where there was a moderate potential for translational sliding.

In the summer of 1986, the toe of the bluff was protected by a concrete bulkhead. While the bulkhead offered some protection, there was erosion of the bluff toe by waves washing over the top of the structure. This erosion was contributing to the instability of the bluff slope. A portion of this bulkhead collapsed when the bluff slope failed in November 1986.

To prevent rotational sliding within Section 78, it is recommended that a groundwater drainage system be installed to lower the groundwater elevation. Within the southern 600 feet of the section, which includes Buckley Park, it is recommended that the bluff slope be graded to a stable slope angle and revegetated. Also, additional toe erosion control measures are recommended along the entire section to prevent erosion from wave and ice action.

Bluff Analysis Section 79: The stability of the fill and the underlying bluff slope within Section 79, which extends from the northern portion of Big Bay Park to 5270 N. Lake Drive in the Village of Whitefish Bay, was characterized by the use of Profile No. 80.

The results of the deterministic slope stability analysis, shown in Figure 69, indicate that Profile No. 80 has a stable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 1.39, and was located beneath the fill. The next nine lowest safety factors ranged from 1.41 to 1.71. A probabilistic slope stability analysis was not conducted for this section because it is a fill area.

Section 79 was also considered to have a stable bluff slope with respect to translational sliding. In general, translational sliding within fill areas was considered unlikely because of the ability of the fill material to maintain a relatively steep slope, and because of the benefits realized by loading the base of the slope. A large amount of fill material had been placed at the base of the natural bluff slope within Section 79.

A small amount of bluff toe erosion was observed in Section 79 during the field survey conducted in the summer of 1986. A revetment composed of rock and concrete rubble and a concrete bulkhead located within the section were not providing adequate toe protection against wave and ice action. Because of the large amount of fill material placed at the base of the bluff, the observed toe erosion was not affecting the stability of the bluff.

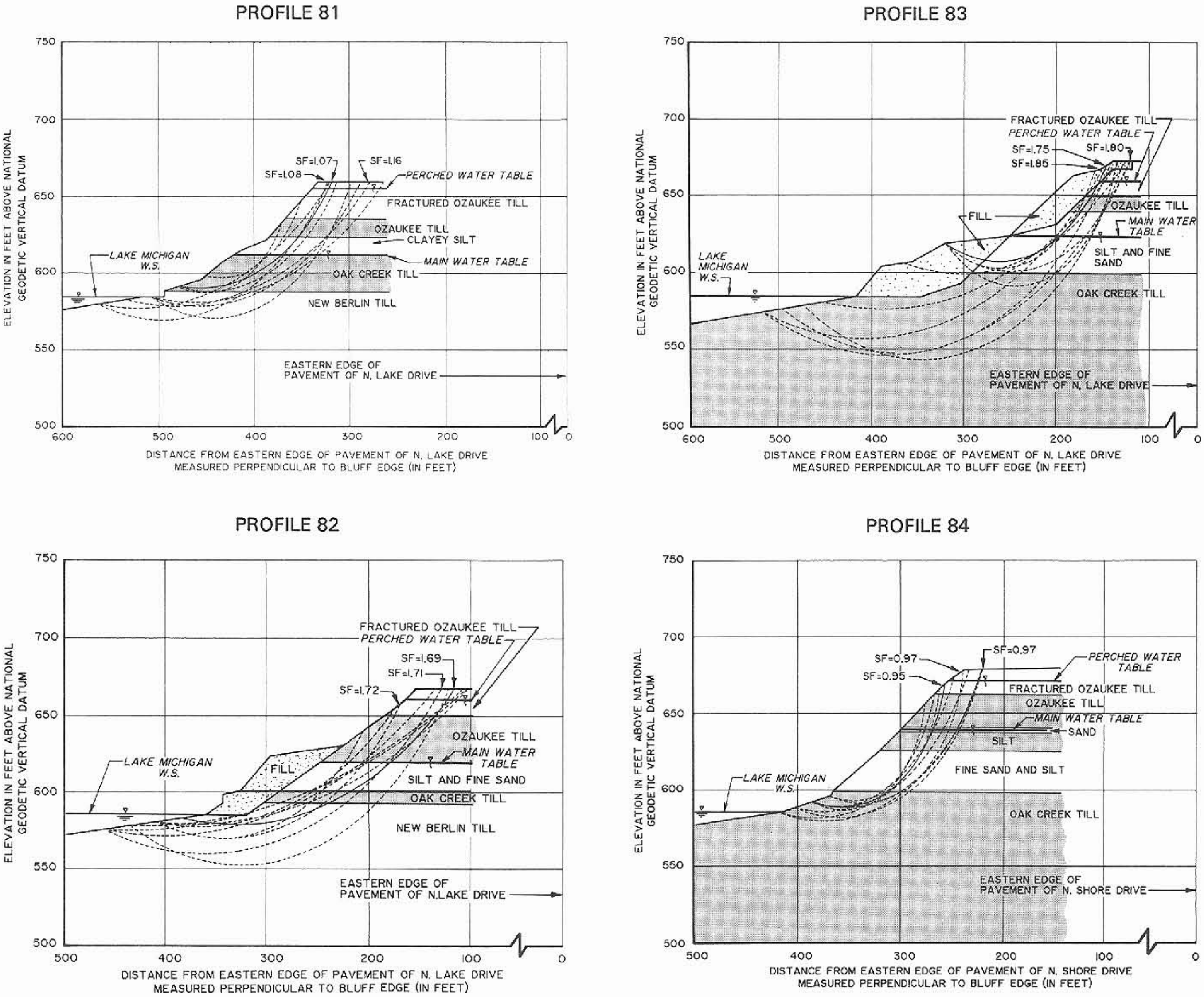
No measures are needed to prevent rotational or translational sliding within Bluff Analysis Section 79. Additional bluff toe protection is recommended at the base of the fill to prevent erosion from wave and ice action.

Bluff Analysis Section 80: The stability of the bluff slope within Section 80, located at 5290 N. Lake Drive in the Village of Whitefish Bay, was characterized by the use of Profile No. 81.

The results of the deterministic slope stability analysis, shown in Figure 70 for Profile No. 81,

Figure 70

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSES FOR PROFILES 81-84



Source: T. B. Edil, D. M. Mickelson, and SEWRPC.

indicate that the bluff slope is just barely stable with respect to rotational sliding. The lowest failure surface calculated had a safety factor of 1.07, and was located mainly within the upper portion of the bluff slope. The next nine lowest safety factors ranged from 1.08 to 1.33.

The lowest safety factors indicated by the 20 probabilistic stability analyses conducted ranged from 0.76 to 1.44, with 15 of the failure surfaces, or 25 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated, 26, or 13 percent, had safety factors of less than 1.0.

In the field survey conducted in the summer of 1986, the overall bluff slope appeared to be stable. However, the upper portion of the slope showed signs of past failure.

Based upon a review of the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 80 was considered to have a marginal bluff slope with respect to rotational sliding.

Section 80 was also considered to have a marginal bluff slope with respect to translational sliding. The base of the bluff had good vegetative cover, with a relatively gentle slope angle of approximately 20 degrees. The upper portion of the bluff slope contained disturbed soil areas, with a much steeper slope of approximately 35 degrees. Therefore, the potential for translational sliding was far greater on the upper portion of the bluff slope than on the lower bluff slope.

Due primarily to the relatively wide beach built up in Section 80, no significant bluff toe erosion was observed during the field survey conducted in the summer of 1986. Thus, under existing shoreline and lake level conditions, wave action did not appear to substantially affect the toe of the bluff. However, during the study period, the beaches were eroding rapidly. Should beach erosion continue or the lake levels remain relatively high, the potential for toe erosion would increase.

It is recommended that the upper portion of the bluff slope be regraded to a stable slope angle and revegetated. It does not appear necessary at this time to provide additional protection against wave and ice action at the toe of the bluff.

Bluff Analysis Section 81: The stability of the fill and underlying bluff slope within Section 81, which extends from 5300 N. Lake Drive to 808 Lakeview Avenue (near Silver Spring Drive extended) in the Village of Whitefish Bay, was characterized by the use of Profile No. 82 and Profile No. 83.

The results of the deterministic slope stability analyses, shown in Figure 70 for Profiles No. 82 and No. 83, indicate stable bluff slopes with respect to rotational sliding. The lowest failure surface calculated at Profile No. 82 had a safety factor of 1.69, and was located beneath the fill layer. The next nine lowest safety factors ranged from 1.71 to 1.81. The lowest failure surface calculated at Profile No. 83 had a safety factor of 1.75, and was located beneath the top portion of the fill layer. The next nine lowest safety factors ranged from 1.80 to 2.02. A probabilistic slope stability analysis was not conducted for this section because it is a fill area. Therefore, based on the deterministic slope stability analysis and on observed bluff conditions, Section 81 was considered to have a stable bluff slope with respect to rotational sliding.

Section 81 was also considered to have a stable bluff slope with respect to translational sliding. In general, translational sliding within fill areas was considered unlikely because of the ability of the fill material to maintain a relatively steep slope, and because of the benefits realized by loading the base of the bluff. Within the northern portion of Section 81, translation sliding was considered to unlikely to occur because of the large amount of fill material that had been placed on nearly the entire natural bluff slope. In the southern portion of the section, however, fill material had been placed only on the lower portion of the bluff slope. The upper portion of the bluff slope therefore had an increased potential for translational sliding.

Bluff toe erosion was observed within the southern portion of Section 81, south of Silver Spring Drive, during the field survey conducted in the summer of 1986. However, because of the large amount of fill material at the base of the bluff, the observed toe erosion was not affecting the stability of the bluff slope. North of Silver Spring Drive, where the fill project was still under construction in 1986, a rock revetment was being placed at the toe of the fill for protection. Because the structure had not been completed as of the time of the field surveys, an evaluation of the degree of bluff toe protection provided was not conducted.

No measures are needed to prevent rotational sliding within Bluff Analysis Section 81. Only minimal translational sliding may be expected to occur—primarily on the upper bluff slope in the southern portion of the section. Additional toe erosion control should be provided along the southern 1,700 feet of the section south of Silver Spring Drive, and adequate toe protection is recommended north of Silver Spring Drive when the fill project is completed in that area to prevent erosion by wave and ice action.

Bluff Analysis Section 82: The stability of the bluff slope within Section 82, which extends from 5722 to 5770 N. Shore Drive (south of Day Avenue) in the Village of Whitefish Bay, was characterized by the use of Profile No. 84.

The results of the deterministic slope stability analysis, shown in Figure 70, indicate that Profile No. 84 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated had a safety factor of 0.95, and was located within the middle portion of the bluff slope. The next nine lowest safety factors ranged from 0.97 to 0.99.

The lowest safety factors indicated by the 20 probabilistic stability analyses conducted ranged from 0.47 to 1.12, with 17 of the failure surfaces, or 85 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated, 159, or 80 percent, had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 82 was considered to have an unstable bluff slope with respect to rotational sliding.

Section 82 was considered to have a marginal bluff slope with respect to translational sliding. This was due to the relatively steep slope of the bluff, and the abundance of disturbed soil areas located throughout the section. The potential for translational sliding was greater on the lower portion of the bluff where groundwater seepage was noted during the 1986 field survey.

Minor erosion of the toe of the bluff due to wave action was observed. Should it continue, this toe erosion may affect the stability of the bluff slope. In the summer of 1986, the toe of the bluff was protected by a relatively wide beach. However, during the study period, the beaches were eroding rapidly.

To prevent rotational sliding within Section 82, it is recommended that the bluff slope be regraded to a stable slope angle and revegetated. Since the completion of the stability analyses, a fill project was initiated in this section. This fill project was in progress in 1988. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 83: The stability of the fill and the underlying bluff slope within Section 83, which is located at 758 E. Day Avenue in the Village of Whitefish Bay, was characterized by the use of Profile No. 85.

The results of the deterministic slope stability analysis, shown in Figure 71, indicate that Profile No. 85 has a stable bluff with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 1.14, and was located beneath the fill. The next nine lowest safety factors ranged from 1.14 to 1.18. A probabilistic slope stability analysis was not conducted for this section because it is a fill area.

Section 83 was also considered to have a stable bluff slope with respect to translational sliding. In general, translational sliding within fill areas was considered unlikely because of the ability of the fill material to maintain a relatively steep slope, and because of the benefits realized by loading the base of the slope. The fill material placed on the natural bluff slope, especially within the lower portion of the slope, should minimize the potential for translational sliding.

Bluff toe erosion was observed within Section 83 during the field survey conducted in the summer of 1986. This toe erosion may affect the stability of the bluff slope. During the study period, the beaches were eroding rapidly. Should beach erosion continue or the lake levels remain relatively high, the potential for toe erosion and subsequent bluff slope failure will increase.

Although the bluff slope was considered stable in 1986, a fill project was in progress in 1988 to reinforce the previous fill material and further improve the stability of the slope. Bluff toe protection was also being installed.

Bluff Analysis Section 84: The stability of the bluff slope within Section 84, which extends from 740 E. Day Avenue to 5866 N. Shore Drive in the Village of Whitefish Bay, was characterized by the use of Profile No. 86.

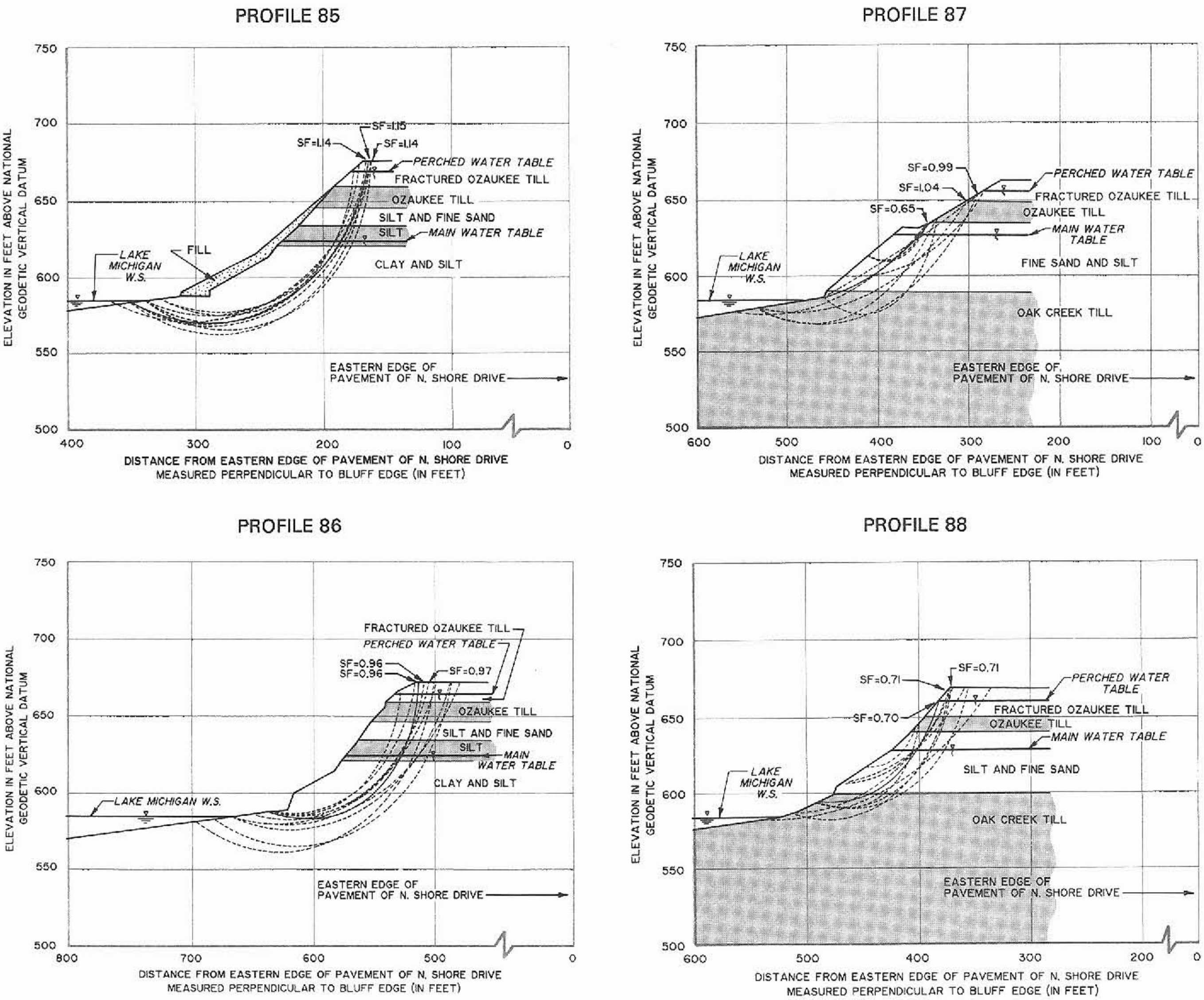
The results of the deterministic slope stability analysis, shown in Figure 71 for Profile No. 86, indicate a threat of bluff slope failure with respect to rotational sliding. The lowest failure surface calculated at Profile No. 86 had a safety factor of 0.96, and included the entire bluff slope. The next nine lowest safety factors ranged from 0.96 to 1.03.

The lowest safety factors indicated by the 20 probabilistic stability analyses conducted ranged from 0.54 to 1.06, with 18 of the failure surfaces, or 90 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated, 184, or 64 percent, had safety factors of less than 1.0.

In the 1986 field survey, small slips and slumps were noted throughout the section. Based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 84 was considered to have an unstable bluff slope with respect to rotational sliding.

Figure 71

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSES FOR PROFILES 85-88



Source: T. B. Edil, D. M. Mickelson, and SEWRPC.

Section 84 was considered to have a marginal bluff slope with respect to translational sliding. This was due to the relatively steep slope of the bluff, and the abundance of disturbed soil areas located throughout the section. The potential for translational sliding was greater on the lower portion of the bluff, where groundwater seepage was noted during the 1986 field survey.

Erosion of the toe of the bluff due to wave and ice action was observed in 1986. Continued bluff toe erosion within this section would affect the stability of the bluff slope. In the summer of 1986, the toe of the bluff was partially protected by a relatively wide beach. However, during the study period, the beaches were eroding rapidly. Should beach erosion continue, or the lake levels remain relatively high, the resulting erosion will increase the potential for slope failure.

To prevent rotational and translational sliding within Section 84, it is recommended that the bluff slope be regraded to a stable slope angle and revegetated. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 85: The stability of the bluff slope within Section 85, which is located at Klode Park, was characterized by the use of Profile No. 87.

The results of the deterministic slope stability analysis, shown in Figure 71 for Profile No. 87, indicate a threat of bluff slope failure with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.65, and was located within the lower portion of the bluff slope. The next nine lowest safety factors were much higher, ranging from 0.99 to 1.19.

The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 87 ranged from 0.53 to 1.13, with 14 of the failure surfaces, or 70 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated at Profile No. 87, 70, or 35 percent, had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions in the summer of 1986, Section 85 was considered to have an unstable bluff slope with respect to rotational sliding.

Overall, Section 85 was considered to have a stable bluff slope with respect to translational sliding. This was due to the good vegetative growth which covered most of the bluff face. There were, however, small disturbed soil areas observed on portions of the bluff slope, especially within the recent slope failure, where there was an increased potential for translational sliding.

In December 1986, a concrete bulkhead protecting the northern portion of Section 25 north of the North Shore Water Commission pumping station collapsed. During the same storm event, a slope failure involving the lower 10 to 15 feet of the bluff along the entire section occurred. To protect the pumping station, approximately 4.5 tons per lineal foot of 500- to 2,000-pound rock riprap and fill were placed behind the remaining bulkhead in the southern portion of the section in January 1987. Following the placement of the riprap and fill, a slope stability analysis conducted by Warzyn Engineering, Inc., indicated that the safety factor for the southern reach at the pumping station was approximately 1.25.

During April 1987, a large rotational slide occurred at Klode Park north of the pumping station and reinforced bulkhead, changing the configuration of the slope. The failure occurred along the critical failure surface indicated for Profile No. 87 in Figure 68, which had a safety factor of about 0.65, indicating a high potential for failure. A water-bearing sand layer was exposed on the bluff failure surface, near the top of the slide. Two soil borings conducted by Warzyn Engineering, Inc., at the beach level in Klode Park indicated two and one-half to six feet of sand and gravel, underlain by Oak Creek till.

To protect the bluff slope at Klode Park and the North Shore Water Commission pumping station, a shore protection project was undertaken by the Village of Whitefish Bay in 1987 and 1988. The project included the installation of a groundwater drainage system and the regrading of the bluff to provide a stable slope. In addition, three offshore breakwaters with steel sheet pile groins were constructed to contain a sand and gravel beach, which provides excellent protection of the toe of the newly regraded bluff. A second level of protection for the pumping station was provided by a riprap revetment which was constructed above the beach directly in front of the pumping station. To assure the performance of the design, the proposed shore protection structures were physically modeled at a scale of 1 to 20 in the Canadian National Research Laboratory's Hydraulic Offshore Wave Basin in Ottawa, Canada, in June and July of 1987. Construction of the project began in the fall of 1987 and was completed in the summer of 1988. No further measures are required to stabilize the bluff slope or protect the toe of the bluff in Section 85, other than continued maintenance of the project.

Bluff Analysis Section 86: The stability of the bluff slope within Section 86, which is located at 5960 N. Shore Drive (just north of Klode Park), was characterized by Profile No. 88.

The results of the deterministic slope stability analysis, shown in Figure 71, indicate that Profile No. 88 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.70, and was located within the middle portion of the bluff slope. The next nine lowest safety factors ranged from 0.71 to 0.88.

The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 88 ranged from 0.52 to 1.10, with 18 of the failure surfaces, or 90 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated at Profile No. 32, 163, or 82 percent, had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 86 was considered to have an unstable bluff slope with respect to rotational sliding.

Section 86 was also considered to have an unstable bluff slope with respect to translational sliding. This was due in part to the lack of vegetative cover on most of the bluff slope, and in part to the relatively steep angle of the slope. The potential for translational sliding was further increased by groundwater seepage from the face of the bluff.

In the summer of 1986, the toe of the bluff was protected by a relatively wide beach. However, during the study period, the beaches were eroding rapidly, and slight erosion of the toe was observed in the fall of 1986. Continued erosion of the toe would reduce the stability of the bluff slope.

To prevent rotational and translational sliding within Section 86, it is recommended that the bluff slope be regraded to a stable slope angle and revegetated. Bluff toe protection was provided to Section 86 in 1987 and 1988 by the northward extension of the newly constructed Klode Park beach.

Bluff Analysis Section 87: The stability of the bluff slope within Section 87, which extends from 6000 N. Shore Drive to 6260 N. Lake Drive (from just north of Klode Park to just south of School Road), was characterized by the use of Profile No. 89.

The results of the deterministic slope stability analysis, shown in Figure 72 for Profile No. 89, indicate a threat of bluff slope failure with respect to rotational sliding. The lowest failure surface calculated at Profile No. 89 had a safety factor of 0.91, and was located on the lower portion of the bluff slope. The next nine lowest safety factors ranged from 0.92 to 1.03.

The lowest safety factors indicated by the 20 probabilistic stability analyses conducted ranged from 0.62 to 1.60, with 13 of the failure surfaces, or 65 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated, 96, or 48 percent, had safety factors of less than 1.0.

In the 1986 field survey, the overall bluff slope within Section 87 appeared to be stable. However, small slips and slumps were noted throughout the section, especially on the lower portion of the bluff slope. Because of the steep bluff slope, and the groundwater seepage present within Section 87, there was a potential for deep-seated failures. Therefore, based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 87 was considered to have a marginal bluff slope with respect to rotational sliding.

Overall, Section 87 was considered to have a stable bluff slope with respect to translational sliding. This was due to the good vegetative growth that covered most of the bluff face. There were, however, small disturbed soil areas observed on portions of the bluff slope, especially the lower bluff slope where groundwater seepage was noted. Translational sliding may be expected to occur in these disturbed areas.

In the summer of 1986, the toe of the bluff was protected by a relatively wide beach. However, during the study period, beaches were eroding rapidly. The toe of the bluff had experienced slight erosion due to wave action. Continued bluff toe erosion within the section would reduce the stability of the bluff slope.

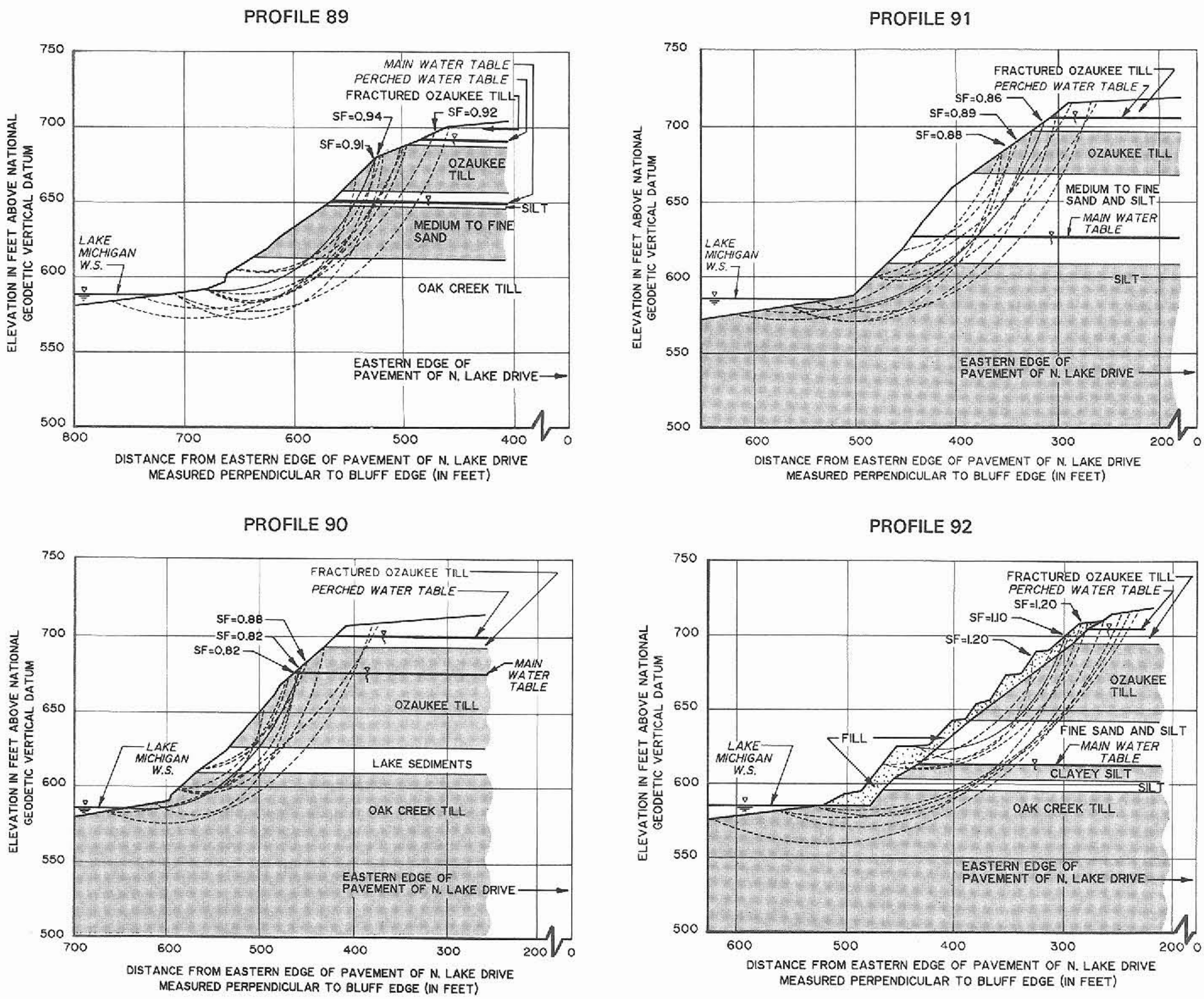
To prevent rotational sliding in Section 87, it is recommended that a groundwater drainage system be installed to lower the groundwater elevation. To prevent translational sliding, it is recommended that a good vegetative cover be maintained on the bluff slope. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 88: Bluff Analysis Sections 88 through 96 lie within the Village of Fox Point, as shown in Figure 73. The stability of the bluff slope within Section 88, which extends from 6310 to 6424 N. Lake Drive (generally north of School Road extended), was characterized by the use of Profile No. 90 and Profile No. 91.

The results of the deterministic slope stability analyses, shown in Figure 72 for Profiles No. 90 and No. 91, indicate the bluff slope is unstable with respect to rotational sliding. The lowest

Figure 72

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSES FOR PROFILES 89-92



Source: T. B. Edil, D. M. Mickelson, and SEWRPC.

failure surface calculated at Profile No. 90 had a safety factor of 0.82, and was located within the lower two-thirds of the bluff slope. The next nine lowest safety factors ranged from 0.82 to 0.95. The lowest failure surface calculated at Profile No. 91 had a safety factor of 0.86, and was also located within the lower portion of the bluff slope. The next nine lowest safety factors evaluated at Profile No. 91 ranged from 0.88 to 1.02.

The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 90 ranged from 0.63 to 1.00, with 19 of the failure surfaces, or 95 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated at Profile No. 90, 137, or 68 percent, had safety factors of less than 1.0. The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 91 ranged from 0.51 to 1.03, with 18 of the failure surfaces, or 90 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated at Profile No. 91, 167, or 84 percent, had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 88 was considered to have an unstable bluff slope with respect to rotational sliding.

Section 88 was also considered to have an unstable bluff slope with respect to translational sliding. This was due in part to the lack of vegetative cover on most of the bluff face, and in part to the relatively steep angle of the bluff slope. Within the lower portion of the bluff slope, the potential for translational sliding was increased by the groundwater seepage occurring in the silt and sand layers.

Bluff toe erosion was observed within the entire shoreline of Section 88 during the field survey conducted in the summer of 1986, and was identified as a major cause of bluff slope failure. There were no shore protection structures present within the section during the field survey; however, a beach did offer some protection against wave action. During the study period the beaches were eroding rapidly. Should beach erosion continue, or the lake levels remain relatively high, the resulting erosion would increase the potential for slope failure.

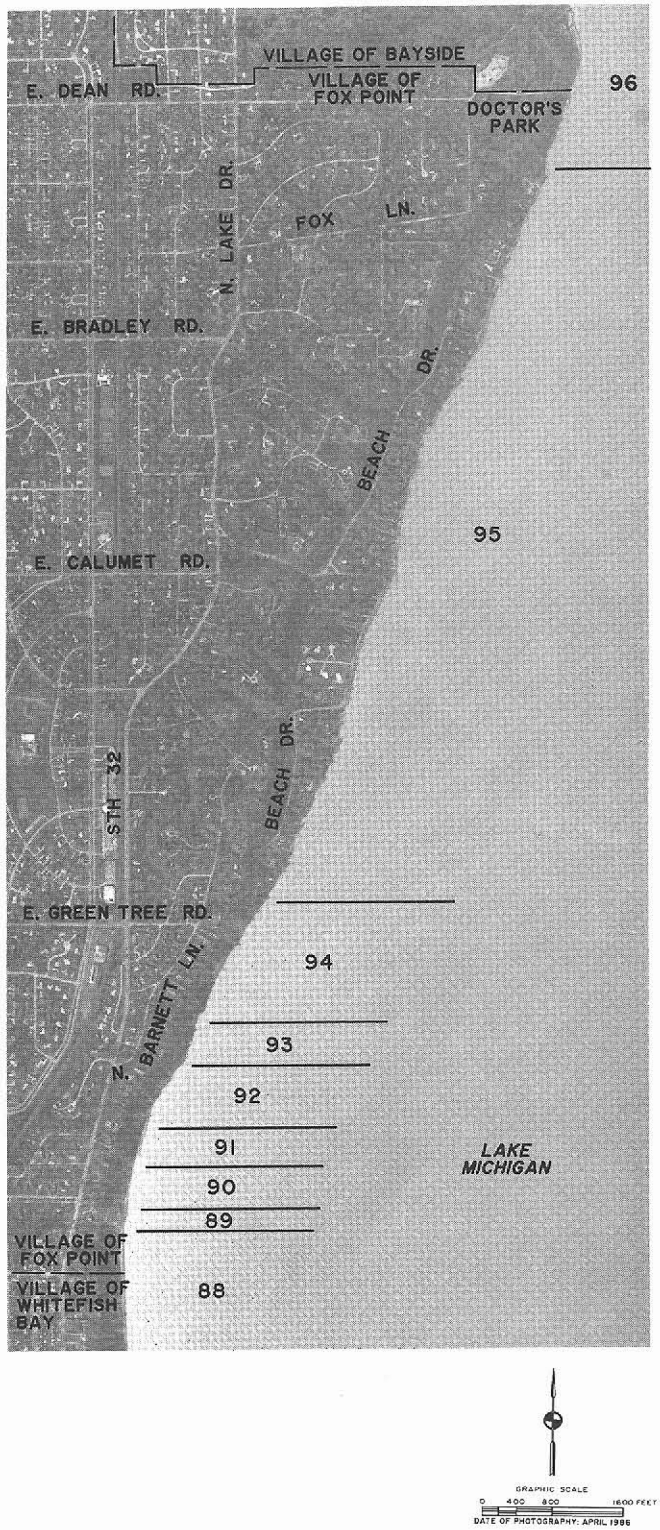
To abate the potential for both rotational and translational sliding within Section 88, it is recommended that the bluff slope be regraded to a stable slope angle and revegetated. In 1988, a fill project was in progress at the northern end of this section. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 89: The stability of the fill and the underlying bluff slope within Section 89, which extends from 6430 to 6448 N. Lake Drive (north of Acacia Road extended) in the Village of Fox Point, was characterized by Profile No. 92.

The results of the deterministic slope stability analysis, shown in Figure 72, indicate that Profile No. 92 has a stable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile had a safety factor of 1.10, and was located beneath the middle portion of the fill. The next nine lowest safety factors ranged from 1.20 to 1.24. A probabilistic slope stability analysis was not conducted for this section because it is a fill area.

Figure 73

BLUFF ANALYSIS SECTIONS WITHIN THE VILLAGE OF FOX POINT



Source: SEWRPC.

Section 89 was also considered to have a stable bluff slope with respect to translational sliding. In general, translational sliding within fill areas was considered unlikely because of the ability of the fill material to maintain a relatively steep slope, and because of the benefits realized by loading the base of the slope. A large amount of fill material had been placed at the base of natural bluff slope within Section 89.

Erosion at the toe of the bluff was not evaluated in this section because the fill was still under construction in 1986. A revetment composed of large concrete blocks and slabs was being placed at the toe of the fill during the 1986 field survey.

No measures are needed to prevent rotational or translational sliding within Bluff Analysis Section 89, other than the completion of the fill project and revegetation of the bluff. It is recommended that adequate toe protection be provided at the base of the fill when the project is completed to prevent erosion from wave and ice action.

Bluff Analysis Section 90: The stability of the bluff slope within Section 90, which extends from 6464 to 6530 N. Lake Drive (near Apple Tree Road extended) in the Village of Fox Point, was characterized by the use of Profile No. 93.

The results of the deterministic slope stability analysis, shown in Figure 74, indicate that Profile No. 93 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.91, and was located within the middle portion of the bluff slope. The next nine lowest safety factors ranged from 0.92 to 0.98.

The lowest safety factors indicated by the 20 probabilistic stability analyses conducted ranged from 0.41 to 0.90. Of the 200 failure surfaces evaluated, all had safety factors of less than 1.0. Two houses were located within 50 feet of the edge of the bluff. Based on both the deterministic and probabilistic slope stability analyses and on the observed bluff conditions, Section 90 was considered to have an unstable bluff slope with respect to rotational sliding.

Section 90 was also considered to have an unstable bluff slope with respect to translational sliding. This was due in part to the lack of vegetative cover on the lower portion of the bluff slope, and in part to the relatively steep angle of the bluff slope. Within the lower portion of the bluff slope, the potential for translational sliding was increased by groundwater seepage at the top of the silt and sand layer.

Bluff toe erosion was observed within the entire shoreline of Section 90 during the field survey conducted in 1986, and was identified as a major cause of bluff slope failure. In the summer of 1986, the toe of the bluff was protected by a revetment composed of rock and concrete rubble. While the revetment offered some protection, there was continued erosion by waves washing over the top of the structure.

To prevent rotational and translational sliding, it is recommended that the bluff slope be regraded to a stable slope angle and revegetated. This action may require filling, since cutting back the top of the slope may not be feasible because houses at the top of the bluff are as close as 10 feet from the bluff edge. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 91: The stability of the bluff slope within Section 91, which extends from 6600 to 6702 N. Lake Drive (near Daphne Road extended) in the Village of Fox Point, was characterized by the use of Profile No. 94.

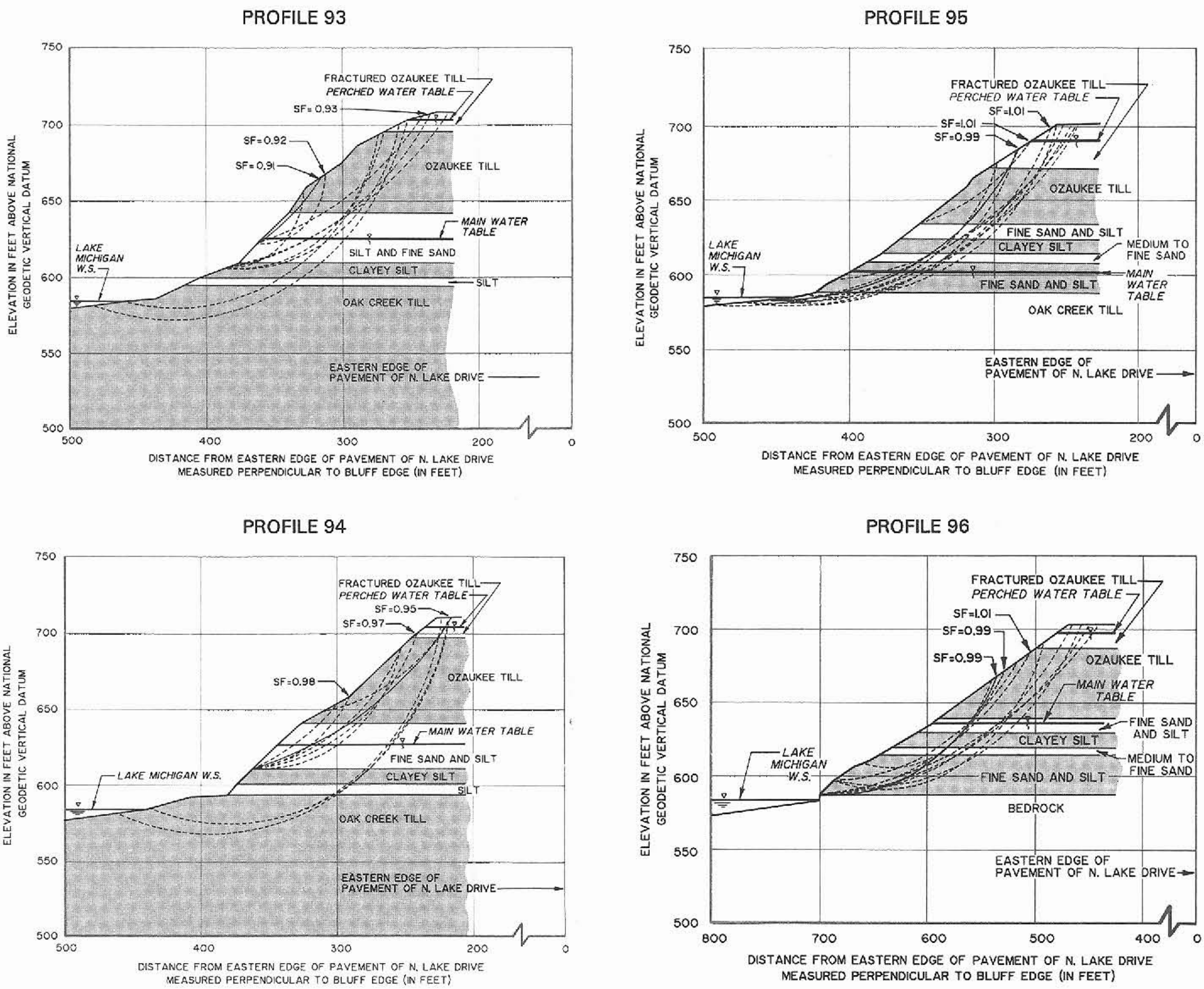
The results of the deterministic slope stability analysis, shown in Figure 74, indicate that Profile No. 94 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.95, and was located within the middle portion of the bluff slope. The next nine lowest safety factors ranged from 0.97 to 1.07.

The lowest safety factors indicated by the 20 probabilistic stability analyses conducted ranged from 0.45 to 1.03, with 18 of the failure surfaces, or 90 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated, 180, or 90 percent, had safety factors of less than 1.0. Three houses were located within 50 feet of the top edge of the bluff. Based on both the deterministic and probabilistic slope stability analyses and on the observed bluff conditions, Section 91 was considered to have a marginally unstable bluff slope with respect to rotational sliding.

Overall, Section 91 was considered to have a stable bluff slope with respect to translational

Figure 74

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSES FOR PROFILES 93-96



Source: T. B. Edil, D. M. Mickelson, and SEWRPC.

sliding. This was due to good vegetative growth covering the entire bluff face. There were, however, small disturbed soil areas observed on the bluff slope where translational sliding may occur. These small isolated slides, however, did not appear to be threatening the stability of the overall bluff slope.

Due primarily to the relatively wide beach built up by a small groin system in Section 91, only slight bluff toe erosion was observed during the field survey conducted in the summer of 1986. However, during the study period, beaches were eroding rapidly. Should beach erosion continue or the lake levels remain relatively high, the resulting erosion would increase the potential for slope failure.

To prevent rotational and translational sliding, it is recommended that a groundwater drainage system be installed and that a good vegetative cover be maintained on the bluff slope. Good bluff toe protection should be maintained to prevent erosion from wave and ice action.

Bluff Analysis Section 92: The stability of the bluff slope within Section 92, which extends from 6720 N. Lake Drive to 6818 N. Barnett Lane in the Village of Fox Point, was characterized by the use of Profile No. 95 and Profile No. 96.

The results of the deterministic slope stability analyses, shown in Figure 74 for Profiles No. 95 and No. 96, indicate a threat of bluff slope failure with respect to rotational sliding. The lowest failure surface calculated at Profile No. 95 had a safety factor of 0.99, and included the entire bluff slope. The next nine lowest safety factors ranged from 1.01 to 1.06. The lowest failure surface calculated at Profile No. 96 had a safety factor of 0.99, and was located on the lower portion of the bluff slope. The next nine lowest safety factors ranged from 0.99 to 1.14.

The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 95 ranged from 0.74 to 1.10, with 14 of the failure surfaces, or 70 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated, 123, or 62 percent, had safety factors of less than 1.0. The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 96 ranged from 0.55 to 1.54, with three failure surfaces, or 15 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated, 22, or 11 percent, had safety factors of less than 1.0. Profile No. 96 was significantly more stable than Profile No. 95 because bedrock was present at the base of the bluff in Profile No. 96. This bedrock minimized the potential for slope failures within the lower portion of the bluff slope. Based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 92 was considered to have a marginal bluff slope.

Section 92 was also considered to have a marginal bluff slope with respect to translational sliding. The upper portion of the bluff slope had good vegetative cover on a gentle slope, while the lower portion of the bluff slope had disturbed soil areas on a steeper slope. Therefore, the potential for translational sliding was greater on the lower bluff slope than on the upper bluff slope.

Bluff toe erosion was observed along the entire shoreline of Section 92 during the field survey conducted in the summer of 1986. There were no shore protection structures present within the section during the summer field survey. However, in the fall of 1986, grout-filled bags were placed at the base of the bluff along a portion of the shoreline. The bags, which were placed to a height of about 10 feet, are intended to minimize the further erosion of the toe.

To prevent rotational and translational sliding within Bluff Analysis Section 92, it is recommended that a groundwater drainage system be considered and that a good vegetative cover be maintained on the bluff slope. The bluff toe protection measures installed in the fall of 1986 should be maintained to prevent erosion from wave and ice action.

Bluff Analysis Section 93: The stability of the bluff slope within Section 93, which extends from 6820 to 6840 N. Barnett Lane in the Village of Fox Point, was characterized by the use of Profile No. 97.

The results of the deterministic slope stability analysis, shown in Figure 75 for Profile No. 97, indicate a threat of bluff slope failure with respect to rotational sliding. The lowest failure surface evaluated had a safety factor of 0.96, and was located within the lower portion of the bluff slope. The next nine lowest safety factors ranged from 1.01 to 1.21.

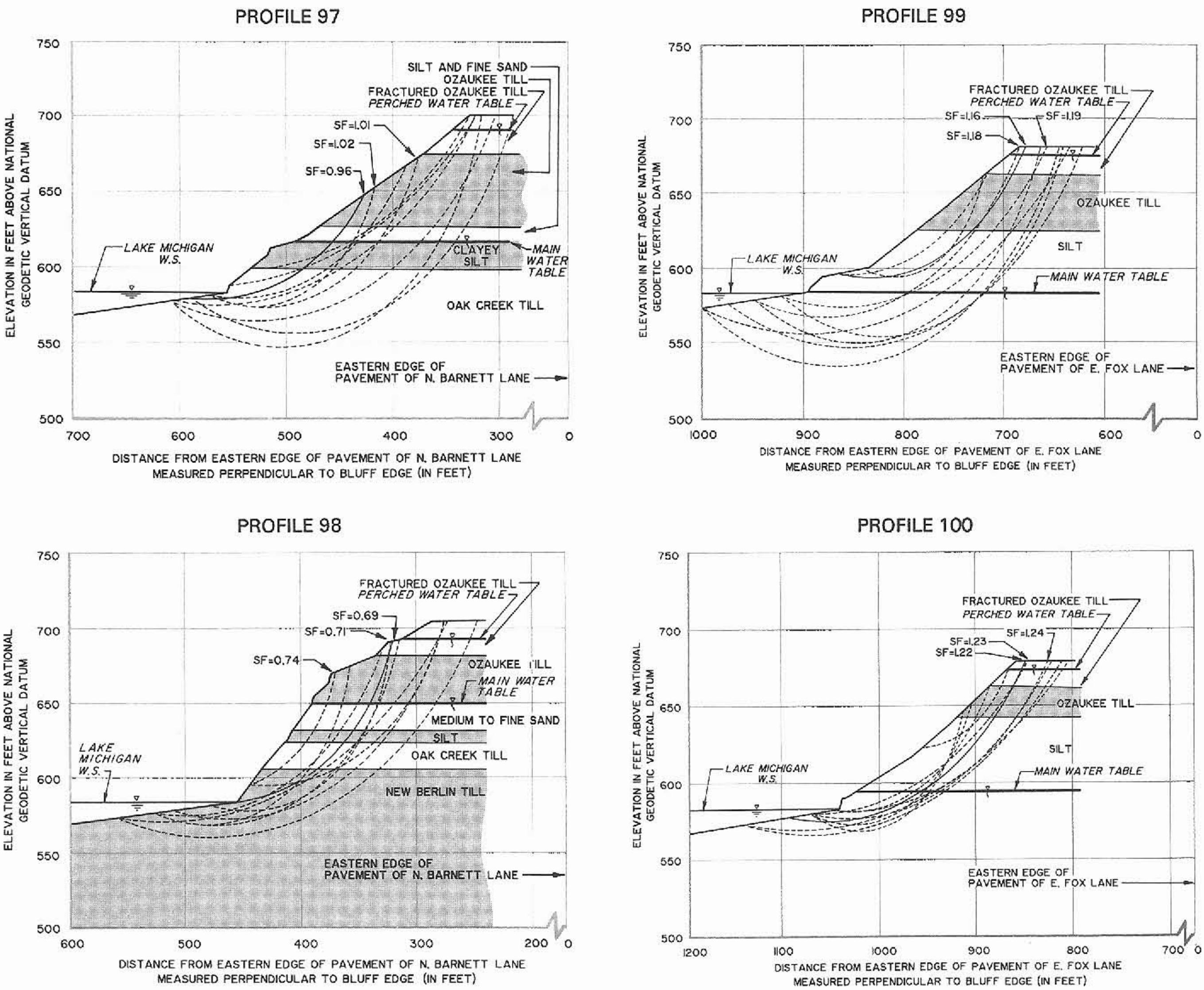
The lowest safety factors indicated by the 20 probabilistic stability analyses conducted ranged from 0.60 to 1.35, with eight of the failure surfaces, or 40 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated, 51, or 26 percent, had safety factors of less than 1.0.

During the field surveys, while the overall bluff slope appeared to be stable, some slumps and shallow slides were observed, especially on the lower portion of the bluff slope. Therefore, based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 93 was considered to have a marginal bluff slope with respect to rotational sliding.

Overall, Section 93 was considered to have a stable bluff slope with respect to translational sliding. A good vegetative growth generally covered most of the bluff face. There were, however, small disturbed soil areas observed on the lower portion of the bluff slope where the potential for translational sliding would be greater.

Figure 75

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSES FOR PROFILES 97-100



Source: T. B. Edil, D. M. Mickelson, and SEWRPC.

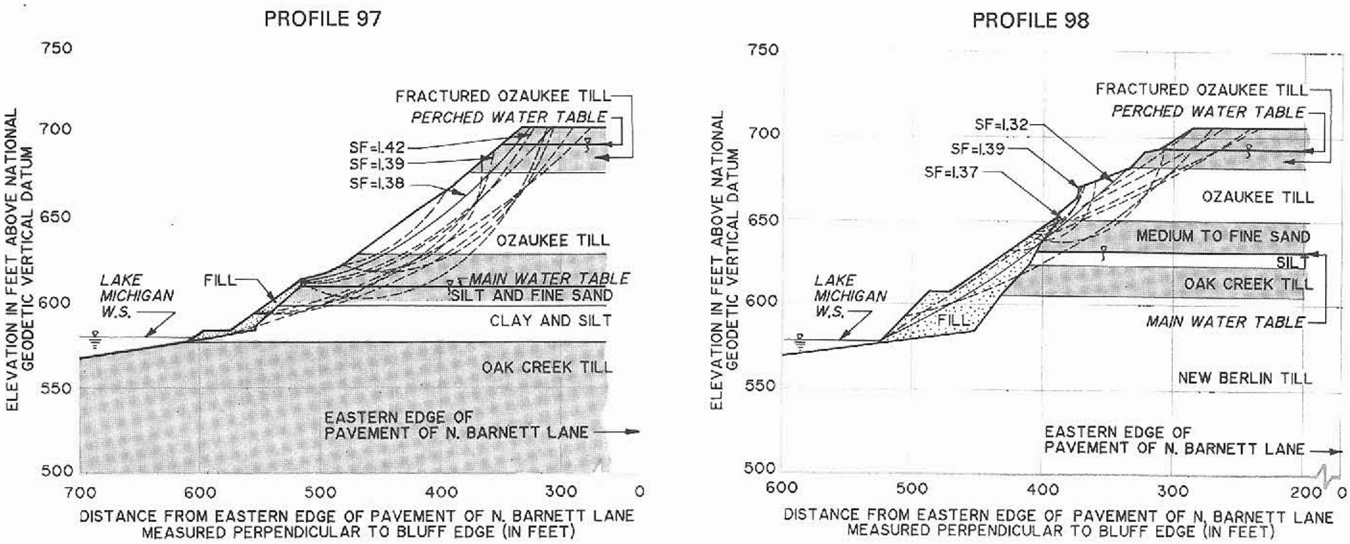
Erosion of the toe of the bluff due to wave action was observed. Continued bluff toe erosion within this section would affect the stability of the bluff slope. In the summer of 1986, the toe of the bluff was receiving partial protection from the pilings of an old mining railroad system.

A fill project was initiated in Sections 93 and 94 in 1987. The project included the installation of a surface water and groundwater drainage system, the placement of fill on the lower portion of the bluff slope, and the construction of a riprap revetment to protect the toe of the fill. A revised deterministic slope stability analysis, shown in Figure 75 for the completed fill project, indicates that the regraded bluff slope is stable, with the lowest safety factor at Profile No. 97 being 1.38.

Bluff Analysis Section 94: The stability of the bluff slope within Section 94, which extends from 6868 to 7004 N. Barnett Lane in the Village of Fox Point, was characterized by the use of Profile No. 98.

Figure 76

REVISED DETERMINISTIC SLOPE STABILITY ANALYSIS FOR PROFILE NOS. 97 AND 98 IN ANALYSIS SECTION 93 AND 94: COMPLETION OF THE FILL PROJECT IN 1988



Source: SEWRPC.

The results of the deterministic slope stability analysis, shown in Figure 75, indicate that Profile No. 98 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.69, and was located within the lower portion of the bluff slope. The next nine lowest safety factors ranged from 0.71 to 1.00.

The lowest safety factors indicated by the 20 probabilistic stability analyses conducted ranged from 0.51 to 0.73. Of the 200 failure surfaces evaluated, 196, or 98 percent, had safety factors of less than 1.0. Three houses were located within 50 feet of the top edge of the bluff. Based on both the deterministic and probabilistic slope stability analyses and on the observed bluff conditions, Section 94 was considered to have an unstable bluff slope with respect to rotational sliding.

Section 94 was also considered to have an unstable bluff slope with respect to translational sliding. This was due in part to the lack of vegetative cover on the lower portion of the bluff slope, and in part to the steep angle of the bluff slope.

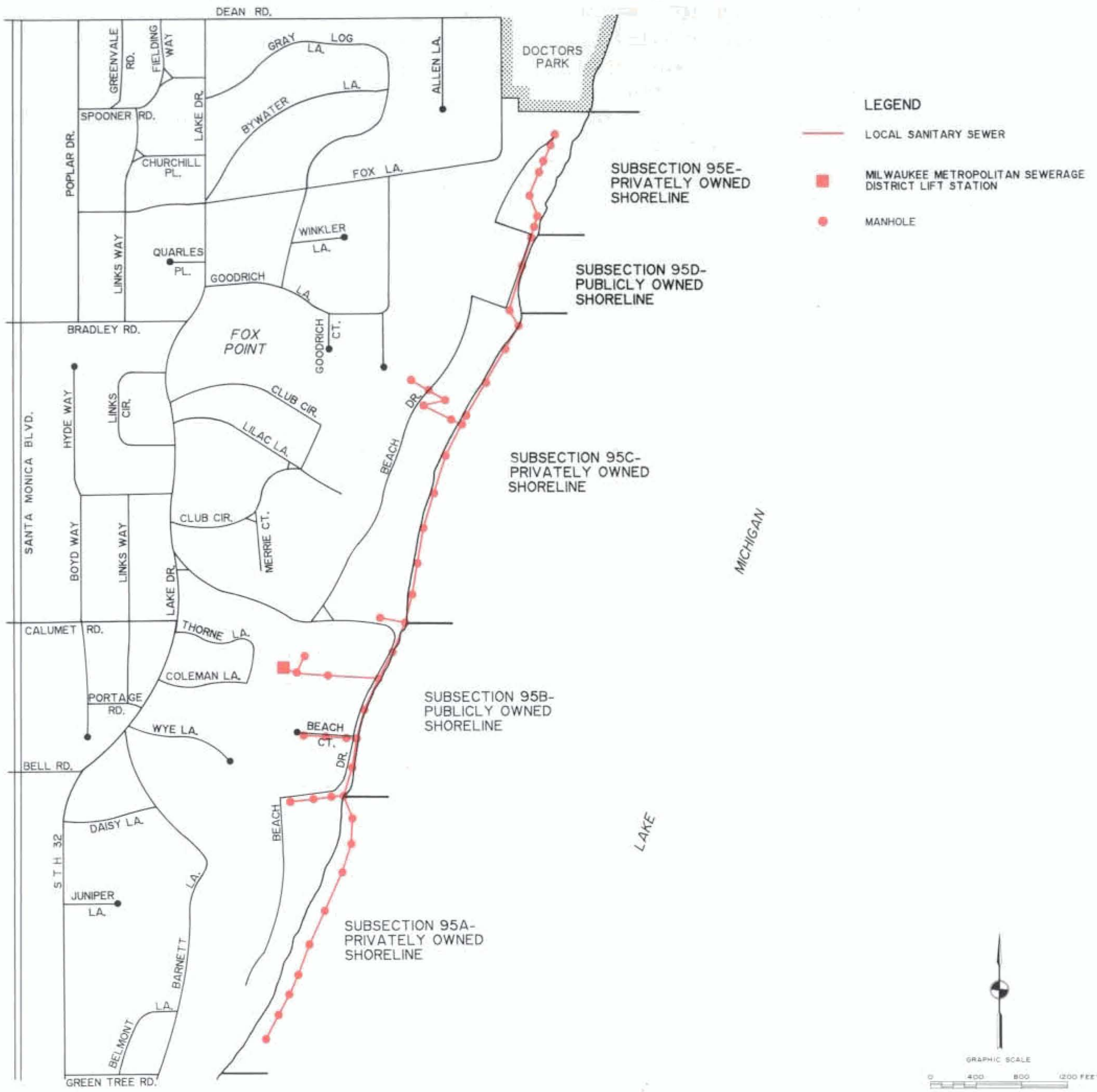
Bluff toe erosion was observed along the entire shoreline of Section 94 during the field survey conducted in 1986, and was identified as a major cause of bluff slope failure. Aside from a collapsed groin, no shore protection structures were present within this section.

As discussed above, a fill project was initiated in Sections 93 and 94 in 1987. A revised deterministic slope stability analysis conducted at the completion of the fill project, as shown in Figure 76, indicated that the regraded bluff slope is stable. The lowest safety factor at Profile No. 98 was 1.32. A riprap revetment was constructed at the base of fill to provide protection against wave and ice action.

Bluff Analysis Section 95: The evaluation of Section 95 differs from that for other analysis sections because it is comprised of a 9,310-foot-long terrace, extending from 7038 to 8130 N. Beach Drive in the Village of Fox Point. Special consideration was given to this section in the evaluation of the erosion problems because of the vulnerable location of the Beach Drive sanitary sewer in 1986, which extended along the Lake Michigan shoreline, as shown on Map 34. For the purposes of this analysis, the section was divided into five subsections based on ownership. As shown on Map 34, three of the subsections, which include about 6,750 feet, or about 74 percent of the total shoreline within the section, are in private ownership. The two remaining subsections, containing about 2,320

Map 34

BEACH DRIVE SANITARY SEWER AND BLUFF ANALYSIS SECTION 95 SUBSECTIONS: 1986



Source: Donohue Engineers & Architects and SEWRPC.

feet, or about 26 percent of the total shoreline, are comprised of public land, the immediate shoreline being owned by the Village of Fox Point.

Analysis Subsection 95A: Subsection 95A extends from 7038 to 7328 N. Beach Drive in the Village of Fox Point and includes 2,390 feet, or 26 percent of the total shoreline, within Section 95. All the shoreline within this subsection is privately owned.

A variety of shoreline protection structures have been installed by private property owners along the shoreline to reduce the erosion of the terrace by wave action. In 1986, approximately 2,150 feet, or 90 percent of the shoreline, was protected by onshore structures, such as bulkheads, revetments, and groins. However, only 840 feet, or 35 percent, of this subsection was protected by structures which had no observable failures, or were not in need of any significant maintenance work. About 240 feet, or 10 percent of the shoreline, was not protected by any onshore structures and was eroding.

Although originally built on land near the shoreline, the portion of the sanitary sewer included within this subsection was located within the lake in 1986. The manholes within this subsection were just slightly above the lake level and were extremely vulnerable to wave and ice action. Within the southern portion of this subsection, continued erosion could expose the sewer pipe which is laid only one to two feet below the lake bottom.

<u>Analysis Subsection 95B</u>: Subsection 95B includes the shoreline area east of the southern portion of N. Beach Drive in the Village of Fox Point which lies adjacent to the lake. It includes about 1,600 feet, or 18 percent, of the total shoreline within the Section 95. The shoreline is owned by the Village of Fox Point.

The terrace within this entire subsection contained a revetment composed of concrete blocks and rubble. In the summer of 1986, the revetment was being overtopped, allowing erosion to occur behind the structure. This erosion posed a threat to N. Beach Drive, which was located as close as 25 feet from the edge of the terrace. At the southern end of this subsection, at the turnaround point of N. Beach Drive, lies a bulkhead composed of concrete slabs and cut stone slabs. Located at the northern end of this subsection is a concrete groin, extending approximately 140 feet in length, and which has built up a beach for the properties to the north of it.

The portion of sanitary sewer included within this subsection was located partially within the lake, and partially on land immediately adjacent to the lake. The southernmost manhole within this subsection was located one-and-one-half feet below lake level, making it vulnerable to damage from wave and ice action.

<u>Analysis Subsection 95C</u>: Subsection 95C extends from 7540 to 7966 N. Beach Drive in the Village of Fox Point, and includes 3,000 feet, or 32 percent, of the total shoreline within Section 95. All the shoreline within this subsection is privately owned.

A variety of shoreline protection structures have been installed by private property owners along the shoreline to reduce erosion of the terrace by wave action. In 1986, approximately 2,640 feet, or 88 percent of the shoreline, was protected by onshore structures. However, only 550 feet, or 19 percent of this subsection, was protected by structures which had no observable failures, or were not in need of any significant maintenance work. About 360 feet, or 12 percent of the shoreline, was not protected by onshore structures and was eroding.

Within Subsection 95C, a beach was present along most of the shoreline in the summer of 1986. The portion of the sanitary sewer included within this subsection was buried beneath that beach. However, during 1986, beaches were eroding rapidly. Continued beach erosion could expose the manholes and sewer to wave and ice attack.

<u>Analysis Subsection 95D</u>: Subsection 95D includes the shoreline area east of the northern portion of N. Beach Drive in the Village of Fox Point which lies adjacent to the lake. It includes about 720 feet, or 8 percent, of the total shoreline within Section 95. The shoreline is owned by the Village of Fox Point.

The terrace within this subsection contained a revetment composed of blocks and concrete rubble. In the field survey conducted in the summer of 1986, the revetment was being overtopped, allowing erosion to occur behind the structure. The attendant erosion posed a threat to N. Beach Drive, which was located as close as 10 feet from the edge of the terrace. In the fall of 1986, concrete blocks were placed approximately 10 feet offshore of the terrace and parallel to the shoreline to help reduce wave action.

The portion of sanitary sewer included within Subsection 95D was located along the east side of N. Beach Drive. The sewer was not being damaged by wave or ice action in 1986, but the erosion did pose a threat to the sewer.

Analysis Subsection 95E: Subsection 95E extends from 8035 to 8130 N. Beach Drive in the Village of Fox Point, and includes 1,360 feet, or 15 percent, of the total shoreline within Section 95. All the shoreline within this subsection is privately owned.

A variety of shoreline protection structures have been installed by private property owners along the shoreline to reduce erosion of the terrace by wave action. In 1986 approximately 1,140 feet, or 84 percent of the shoreline, was protected by onshore structures. However, only 240 feet, or 18 percent of this subsection, was protected by structures which had no observable failures, or were not in need of any significant maintenance work. About 220 feet, or 16 percent of the shoreline, was not protected by onshore structures and was eroding.

The portion of the sanitary sewer included within Subsection 95E was located 100 to 350 feet inland from the Lake Michigan shoreline. The sewer was not being damaged by wave or ice action in 1986.

Recommendations: Adequate shoreline protection is recommended to be provided along the entire shoreline of Bluff Analysis Section 95. Such protection may require the maintenance of existing shore protection structures, the reconstruction of existing structures, or the construction of new structures. Approximately 18 percent of the shoreline within the section was protected by structures that did not require maintenance, about 71 percent of the shoreline was protected by structures that were in need of maintenance, and 11 percent of the shoreline was not protected by structures. It is recommended that the shore protection structures selected be coordinated with measures needed to resolve the Beach Drive sanitary sewer problem.

Bluff Analysis Section 96: A portion of Bluff Analysis Section 96 lies within the Village of Fox Point, and a portion lies with the Village of Bayside. The bluff analysis sections that are located within the Village of Bayside are shown in Figure 77. The stability of the bluff slope within Section 96, which is located within Doctors Park, was characterized by the use of Profile No. 99 and Profile No. 100.

The results of the deterministic slope stability analyses, shown in Figure 75 for Profiles No. 99 and No. 100, indicate stable bluff slopes with respect to rotational sliding. The lowest failure surface calculated at Profile No. 99 had a safety factor of 1.16, and was located within the upper two-thirds of the bluff slope. The next nine lowest safety factors ranged from 1.18 to 1.24. The lowest failure surface calculated at Profile No. 100 had a safety factor of 1.22, and included the entire bluff slope. The remaining failure surfaces had safety factors ranging from 1.23 to 1.37.

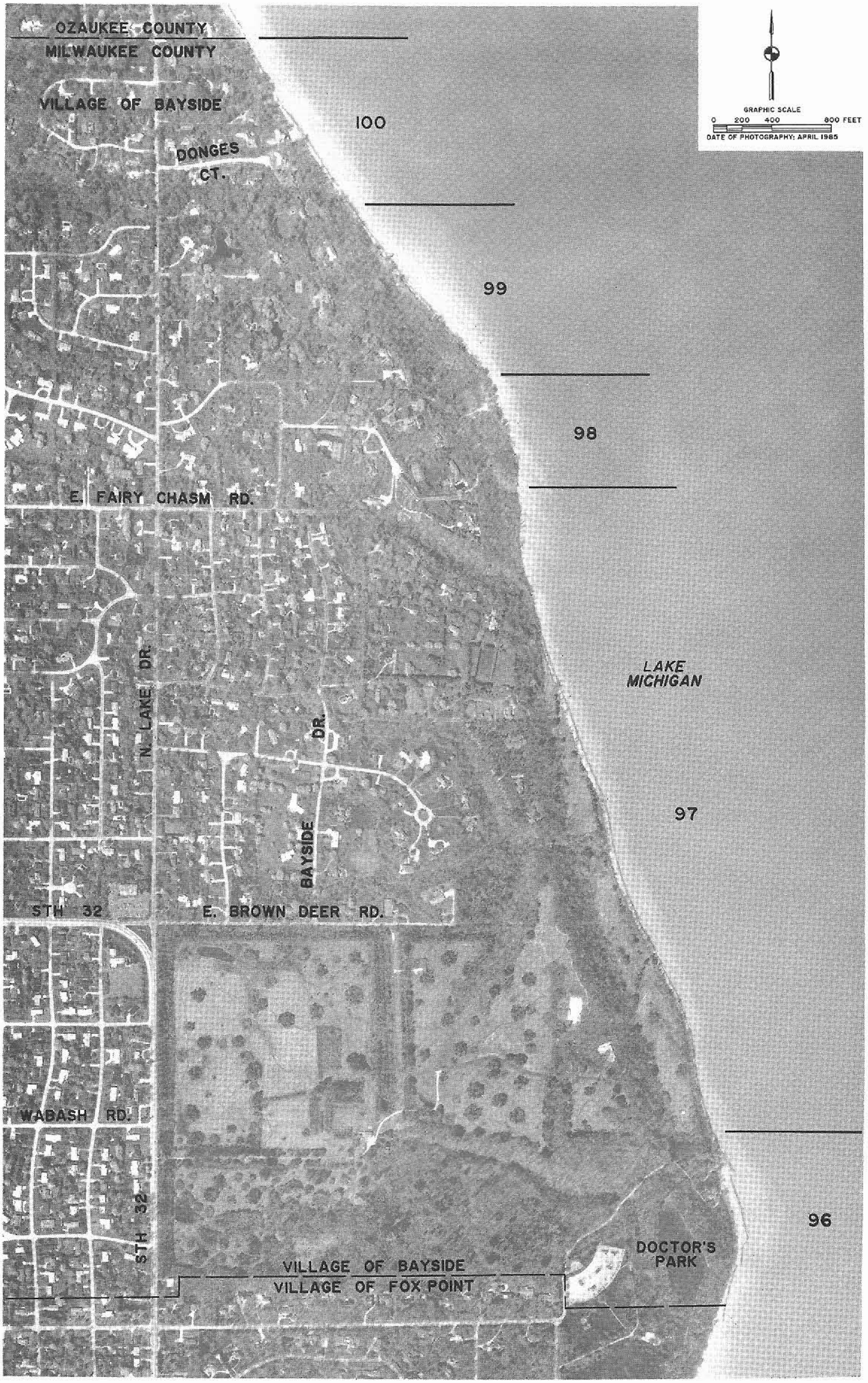
The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 99 ranged from 0.95 to 1.38, with three failure surfaces, or 15 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated, 12, or 6 percent, had safety factors of less than 1.0. The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 100 ranged from 0.79 to 1.42, with three failure surfaces, or 15 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated, 26, or 13 percent, had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses and on the observed bluff conditions, Section 96 was considered to have a stable bluff slope with respect to rotational sliding. However, the probabilistic analysis indicated that under certain conditions, there was a slight risk of slope failure. Section 96 was also considered to have a stable bluff slope with respect to translational sliding. This was due in part to the gentle angle of the bluff slope, and in part to the good vegetative growth covering the entire bluff face.

Within Section 96, the bluff slope was protected by a concrete bulkhead in the southern portion of the section, and a groin system within the northern portion of the section. While the bulkhead offered some protection, there was erosion of the bluff toe from waves washing over the top of the structure. This erosion was not affecting the overall stability of the bluff slope.

No measures are needed to prevent rotational or translational sliding within Bluff Analysis Section 96. Toe protection in addition to the existing concrete bulkhead is recommended to prevent wave overtopping in the southern portion of the section.

Figure 77

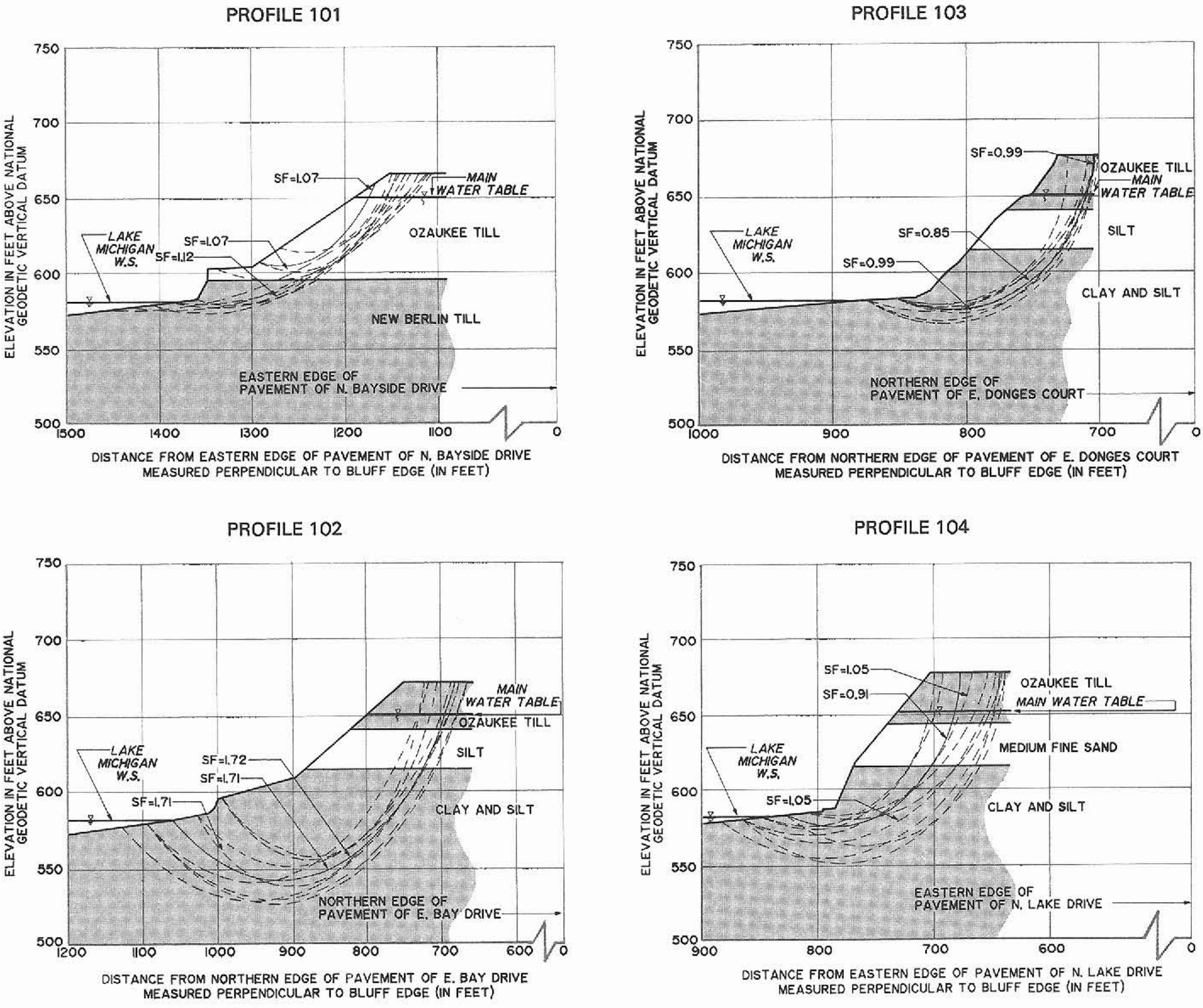
BLUFF ANALYSIS SECTIONS WITHIN THE VILLAGE OF BAYSIDE



Source: SEWRPC.

Figure 78

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSES FOR PROFILES 101-104



Source: T. B. Edil, D. M. Mickelson, and SEWRPC.

Bluff Analysis Section 97: The stability of the bluff within Section 97, which extends from the Schlitz Audubon Center to 9360 N. Lake Drive in the Village of Bayside, was characterized by the use of Profile No. 101.

The results of the deterministic slope stability analysis, shown in Figure 78, indicate that the bluff slope at Profile No. 101 is just barely stable with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 1.07, and was located in the upper two-thirds of the bluff slope. The next nine lowest safety factors ranged from 1.12 to 1.23.

A probabilistic stability analysis, under which the bluff conditions at the profile site were varied, was conducted to help characterize the stability of the bluff slope within the entire section, and to help determine whether, under certain conditions, the bluff slope would be unstable. The lowest safety factors indicated by the 20 probabilistic stability analyses conducted ranged from 0.70 to 1.68, with six, or 30 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated, 23 surfaces, or 11 percent, had safety factors of less than 1.0. The results of the probabilistic analyses thus indicate that the bluff could fail under certain conditions. In spite of this slight risk of slope failure, Section 97 was considered to have a stable bluff slope with respect to rotational sliding based on the slope stability analyses and on observed bluff conditions.

Section 97 was also considered to have a stable slope with respect to translational sliding. The bluff slope is well vegetated and protected by a wide terrace.

Erosion of the base of the terrace was observed during the field survey conducted in the fall of 1987. Although this erosion did not affect the stability of the bluff slope, it poses a threat to future bluff slope stability, particularly in the northern end of the section where the terrace is narrower. No measures are needed to prevent rotational or translational sliding within Bluff Analysis Section 97. Protection of the base of the terrace is necessary to ensure continued bluff slope stability in this section.

Bluff Analysis Section 98: The shoreline of Section 98 extends from 1470 to 1434 E. Bay Point Road in the Village of Bayside. The natural bluff has an overall bluff slope angle of approximately 20 degrees, and is protected by a terrace which ranges up to 200 feet in width. No slope failures were observed during the field survey conducted during the fall of 1987. Slope stability analyses were not conducted for this section because the bluff appeared to be stable.

The shoreline in this section was protected by a riprap revetment and a bulkhead. These structures appeared to be providing adequate protection of the base of the terrace. During the field survey conducted in the fall of 1987, the terrace was not being eroded.

No measures are needed to prevent rotational or translational sliding within Section 98. To ensure continued shoreline protection in Section 98, measures should be taken to maintain the shore protection structures within this section.

Bluff Analysis Section 99: The stability of the bluff in Section 99, which extends from 1430 E. Bay Point Road to 9364 N. Lake Drive in the Village of Bayside, was characterized by the use of Profile No. 102.

The results of the deterministic slope stability analysis, as shown in Figure 78, indicate that Profile No. 102 has a stable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 1.71, and included the entire bluff slope. The next nine lowest safety factors ranged from 1.71 to 1.79. A probabilistic slope stability analysis was not conducted for this section because the bluff slope was considered to be stable based on the field observations and the results of the deterministic analysis.

Section 99 was also considered to have a stable bluff slope with respect to translational sliding. This was due to the good vegetative cover and gentle angle of the bluff slope.

Moderate shoreline erosion was observed in Section 99 during the field survey conducted in the fall of 1987. This erosion was affecting the wide terrace at the base of the bluff.

No measures are needed to prevent rotational or translational sliding within Bluff Analysis Section 99. It is recommended that shore protection measures be installed to prevent erosion of the terrace by wave and ice action.

Bluff Analysis Section 100: The stability of the bluff in Section 100, which extends from 9400 to 9578 N. Lake Drive (just south of County Line Road extended) in the Village of Bayside, was characterized by the use of Profiles No. 103 and No. 104.

The results of the deterministic slope stability analyses, shown in Figure 78 for Profiles No. 103 and No. 104, indicate that Section 100 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at Profile No. 103 had a safety factor of 0.85, and included the entire bluff slope. The next nine lowest safety factors ranged from 0.99 to 1.03. The lowest failure surface calculated at Profile No. 104 had a safety factor of 0.91 and also included the entire bluff slope. The next nine lowest safety factors ranged from 1.05 to 1.13.

Probabilistic slope stability analyses were also conducted for each profile site. The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 103 ranged from 0.49 to 1.12, with 19, or 95 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated for Profile No. 103, 142, or 71 percent, had safety factors of less than 1.0. The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 104 ranged from 0.59 to 1.10, with 17, or 85 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated at Profile No. 104, 127, or 63 percent, had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses and on the observed bluff conditions, Section 100 was considered to have an unstable bluff slope with respect to rotational sliding.

Section 100 also was considered to have an unstable bluff slope with respect to translational sliding. This was due to the lack of vegetative cover and the steep angle of the bluff slope. Groundwater seepage was observed during the field survey conducted in the fall of 1987, and was considered to contribute to the translational sliding.

Severe toe erosion which affected the overall stability of the bluff slope was observed within most of Section 100. Shore protection structures present in the fall of 1987 included a concrete bulkhead which showed evidence of overtopping and flanking, and a revetment composed of concrete blocks which had collapsed. The remaining shoreline within the section was not protected by shore protection structures in 1987.

To abate the severe potential for both rotational and translational sliding, it is recommended that the bluff slope be regraded to a stable slope angle and revegetated. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Summary of the Evaluation of Bluff Analysis Sections

The analyses of each of the 100 bluff analysis sections were conducted to better quantify shoreline erosion and the risks of bluff slope failure by rotational sliding and translational sliding. A summary of the deterministic and probabilistic slope stability analysis results for each profile site is set forth in Table 42. The evaluations of the bluff conditions are presented in Table 43 and shown on Map 35. While these discussions presented the study findings for each bluff analysis section, it must be recognized that the bluff conditions within any given section can vary substantially.

With respect to rotational sliding, 36 bluff analysis sections, which cover 51,130 feet, or 32 percent of the total study area shoreline, were found to have stable bluff slopes. A total of 18 bluff analysis sections, comprising 17,820 feet, or 11 percent of the total study area shoreline, were found to have marginal bluff slopes. A total of 39 bluff analysis sections, comprising 39,420 feet, or 25 percent of the total study area shoreline, were found to have unstable slopes. Bluff slope stability was not evaluated for the remaining seven sections located within 1) the shoreline protected by the Milwaukee Harbor breakwater; 2) the terrace directly north of the harbor extending to the City of Milwaukee Linnwood Avenue water treatment plan; and 3) the Fox Point terrace. Together these seven sections comprised the remaining 50,740 feet, or 32 percent, of the total Milwaukee County shoreline.

With respect to translational sliding, 36 bluff analysis sections, comprising 52,230 feet, or 33 percent of the total study area shoreline, were considered to have stable bluff slopes. Twenty-three bluff analysis sections, comprising 18,980 feet, or 12 percent of the total study area shoreline, were considered to have marginal bluff slopes. Thirty-four bluff analysis sections, covering 37,160 feet, or 23 percent of the total study area shoreline, were considered to have unstable bluff slopes.

Thirty-seven bluff analysis sections, comprising about 77,870 feet, or 49 percent of the total study area shoreline, were found to be exhibiting insignificant or slight shoreline or bluff toe erosion. The remaining 63 bluff analysis sections, comprising 81,240 feet, or 51 percent of the total study area shoreline, were exhibiting substantial erosion of the shoreline or bluff toe. The erosion occurring within 43 bluff analysis sections, comprising 42,850 feet, or 53 percent of the eroding shoreline, was considered to be affecting the overall stability of the bluff slopes.

The measures needed to stabilize the bluff slopes were identified for each of the 100 bluff analysis sections. The needed types of bluff stabilization measures are listed in Table 44 and shown on Map 36. Those indicated measures include regrading the bluff slope to a stable angle; the installation of a groundwater drainage system to lower the elevation of the groundwater; the construction of surface water runoff control measures; and revegetation of the bluff slopes. The extent of the shoreline within each munici-

Table 42

SUMMARY OF DETERMINISTIC AND PROBABILISTIC SLOPE STABILITY ANALYSIS RESULTS FOR ROTATIONAL SLIDING

Civil Division	Bluff Analysis Section[b]	Profile Site	Deterministic Analysis[a]		Probabilistic Analysis[a]			Model-Indicated Stability Classification of Section[c]
			Lowest Safety Factor	Percent of 10 Lowest Safety Factors < 1.0	Range of Lowest Safety Factors	Percent of Lowest Safety Factors < 1.0	Percent of 10 Lowest Safety Factors per Model Run < 1.0	
City of Oak Creek	1	- -	- -	- -	- -	- -	- -	- -
	2	1	1.00	0	0.73-1.37	55	43	M
		2	1.43	0	0.79-1.36	45	32	- -
		3	1.18	0	0.89-1.62	10	1	- -
	3	4	0.70	100	- -	- -	- -	U
		5	1.13	0	- -	- -	- -	- -
		6	0.76	100	- -	- -	- -	- -
	4	7	0.81	100	- -	- -	- -	U
		8	0.75	100	- -	- -	- -	- -
	5	9	0.79	70	0.49-0.83	100	88	U
	6	10	0.86	100	0.65-1.03	95	79	U
	7	11	0.87	80	0.40-0.83	100	100	U
	8	- -	- -	- -	- -	- -	- -	- -
	9	12	0.87	70	0.40-0.92	100	85	U
	10	13	0.92	30	0.70-1.14	70	45	M
	11	14	0.90	70	0.51-1.01	90	78	U
	12	- -	- -	- -	- -	- -	- -	- -
	13	15	1.48	0	- -	- -	- -	S
City of South Milwaukee	14	16	0.90	100	0.42-0.84	100	100	U
		17	0.74	100	- -	- -	- -	- -
	15	18	0.74	100	0.70-1.10	95	94	U
	16	19	1.13	0	0.74-1.49	30	18	M
	17	20	0.78	100	- -	- -	- -	U
	18	21	1.25	0	- -	- -	- -	- -
		22	0.87	30	0.51-0.84	100	97	U
	19	- -	- -	- -	- -	- -	- -	- -
	20	23	0.71	100	- -	- -	- -	U
	21	24	0.92	70	0.82-1.07	75	49	M
	22	25	0.83	100	0.65-1.06	80	64	U
	23	26	0.89	80	0.44-0.94	100	90	U
	24	27	1.23	0	0.71-1.19	70	32	M
	25	28	0.86	80	- -	- -	- -	U
City of Cudahy	26	29	0.79	100	- -	- -	- -	U
	27	30	1.69	0	1.09-1.79	0	0	- -
		31	0.68	100	0.46-1.11	90	82	U
	28	32	0.72	100	- -	- -	- -	U
	29	33	0.65	100	- -	- -	- -	U
	30	34	0.81	100	- -	- -	- -	U
	31	35	0.81	100	0.53-0.89	100	100	U
	32	- -	- -	- -	- -	- -	- -	- -
	33	36	0.79	100	- -	- -	- -	U
	34	37	1.21	0	0.73-1.25	65	20	M
		38	1.02	0	0.71-1.25	55	36	- -
	35	39	0.74	100	0.58-0.94	100	76	U
	36	40	0.93	50	1.05-1.45	0	0	M
	37	41	0.81	100	- -	- -	- -	U
		42	0.86	100	- -	- -	- -	- -
City of St. Francis	38	43	0.81	30	- -	- -	- -	M
	39	44	0.93	50	0.73-1.28	30	7	U
		45	0.72	100	0.72-1.14	65	31	- -
	40	46	1.17	0	- -	- -	- -	S
	41	- -	- -	- -	- -	- -	- -	- -
	42	47	1.33	0	- -	- -	- -	S

Table 42 (continued)

Civil Division	Bluff Analysis Section[b]	Profile Site	Deterministic Analysis[a]: Lowest Safety Factor	Deterministic Analysis[a]: Percent of 10 Lowest Safety Factors < 1.0	Probabilistic Analysis[a]: Range of Lowest Safety Factors	Probabilistic Analysis[a]: Percent of Lowest Safety Factors < 1.0	Probabilistic Analysis[a]: Percent of 10 Lowest Safety Factors per Model Run < 1.0	Model-Indicated Stability Classification of Section[c]
City of St. Francis (continued)	43	48	0.85	70	0.74-1.15	65	23	U
		49	0.81	70	0.62-1.15	95	45	- -
		50	0.81	80	0.66-1.04	80	44	- -
	44	51	0.82	60	- -	- -	- -	U
	45	52	0.99	10	0.53-1.64	50	13	M
	46	53	0.96	20	0.75-1.48	30	4	M
	47	54	1.17	0	- -	- -	- -	S
City of Milwaukee	48	55	1.04	0	1.04-1.56	0	0	S
	49	- -	- -	- -	- -	- -	- -	- -
	50	56	1.21	0	0.78-1.49	25	3	S
	51	- -	- -	- -	- -	- -	- -	- -
	52	- -	- -	- -	- -	- -	- -	- -
	53	- -	- -	- -	- -	- -	- -	- -
	54	- -	- -	- -	- -	- -	- -	- -
	55	- -	- -	- -	- -	- -	- -	- -
	56	- -	- -	- -	- -	- -	- -	- -
	57	- -	- -	- -	- -	- -	- -	- -
	58	- -	- -	- -	- -	- -	- -	- -
	59	- -	- -	- -	- -	- -	- -	- -
	60	- -	- -	- -	- -	- -	- -	- -
	61	57	1.46	0	0.98-1.60	5	1	S
	62	58	2.97	0	2.01-2.89	0	0	S
Village of Shorewood	63	59	0.98	10	0.62-1.08	65	32	M
		60	0.98	10	0.81-1.15	55	15	- -
	64	61	2.13	0	- -	- -	- -	S
	65	62	1.12	0	0.86-1.23	20	8	S
	66	63	1.54	0	- -	- -	- -	S
	67	64	0.81	100	0.51-0.90	100	96	U
	68	65	0.99	10	0.66-1.17	60	45	M
	69	66	1.79	0	0.74-1.99	5	2	S
	70	67	0.68	100	0.61-0.97	100	80	M
	71	68	0.72	20	- -	- -	- -	S
Village of Whitefish Bay	71	69	1.44	0	- -	- -	- -	- -
		70	2.11	0	- -	- -	- -	- -
	72	71	0.64	100	0.50-0.97	100	92	U
		72	0.66	100	0.52-0.81	100	93	- -
	73	73	0.61	100	- -	- -	- -	U
	74	74	0.80	100	0.55-0.82	100	100	U
	75	75	0.90	100	- -	- -	- -	U
	76	76	0.73	100	0.53-0.83	100	95	U
	77	77	1.06	0	- -	- -	- -	S
		78	1.51	0	- -	- -	- -	- -
	78	79	0.91	30	0.54-1.06	70	78	M
	79	80	1.39	0	- -	- -	- -	S
	80	81	1.07	0	0.76-1.44	25	13	M
	81	82	1.69	0	- -	- -	- -	S
		83	1.75	0	- -	- -	- -	- -
	82	84	0.95	100	0.47-1.12	85	80	U
	83	85	1.14	0	- -	- -	- -	S
	84	86	0.96	70	0.54-1.06	90	64	U
	85	87	0.65	20	0.53-1.13	70	35	U
	86	88	0.70	100	0.52-1.10	90	82	U
	87	89	0.91	50	0.62-1.60	65	48	M

Table 42 (continued)

Civil Division	Bluff Analysis Section[b]	Profile Site	Deterministic Analysis[a] Lowest Safety Factor	Deterministic Analysis[a] Percent of 10 Lowest Safety Factors < 1.0	Probabilistic Analysis[a] Range of Lowest Safety Factors	Probabilistic Analysis[a] Percent of Lowest Safety Factors < 1.0	Probabilistic Analysis[a] Percent of 10 Lowest Safety Factors per Model Run < 1.0	Model-Indicated Stability Classification of Section[c]
Village of Fox Point	88	90	0.82	100	0.63-1.00	95	68	U
		91	0.86	90	0.51-1.03	90	84	- -
	89	92	1.10	0	- -	- -	- -	S
	90	93	0.91	100	0.41-0.90	100	100	U
	91	94	0.95	40	0.45-1.03	90	90	M
	92	95	0.99	10	0.74-1.10	70	62	M
		96	0.99	20	0.55-1.54	15	11	- -
	93	97	0.96	10	0.60-1.35	40	26	M
		97(rev)[d]	1.38	0	- -	- -	- -	S
	94	98	0.69	90	0.51-0.73	100	98	U
		98(rev)[d]	1.32	0	- -	- -	- -	S
	95	- -	- -	- -	- -	- -	- -	- -
	96	99	1.16	0	0.95-1.38	15	6	S
		100	1.22	0	0.79-1.42	15	13	- -
Village of Bayside	97	101	1.07	0	0.70-1.68	30	11	S
	98	- -	- -	- -	- -	- -	- -	- -
	99	102	1.71	0	- -	- -	- -	S
	100	103	0.85	40	0.49-1.12	95	71	U
		104	0.91	10	0.59-1.10	85	63	- -

[a]*The deterministic slope stability analysis utilizes site-specific data collected at individual profile sites to compute potential slope failure surfaces. The probabilistic slope stability analysis evaluates slope stability as soil properties, stratigraphy, and groundwater conditions vary randomly within specified ranges. The intent of the probabilistic analysis is to provide a general assessment of the stability of bluff slopes within entire bluff analysis sections, rather than at specific profile sites. The probabilistic analysis helps improve the evaluation of those profile sites where some bluff characteristics are not well-defined.*

[b]*Based on field observations, the bluffs in Bluff Analysis Sections 1, 8, 12, 19, 32, 41, 49, 51, 52, 53, 54, and 98 appeared stable, and therefore no stability analyses were conducted. Bluff Analysis Sections 55, 56, 57, 58, 59, 60, and 95 did not contain a bluff at the shoreline.*

[c]*M - Marginal*
U - Unstable
S - Stable

[d]*Stability analyses were revised following the completion of the fill project in Bluff Analysis Sections 93 and 94 in 1988.*

Source: SEWRPC.

pality associated with each of the indicated bluff stabilization measures is summarized in Table 45.

Regrading the bluff slopes to a stable angle, either by placing fill on the bluff slope or by cutting back the top of the bluff and then revegetating the bluff, was indicated for 48 bluff analysis sections, which include about 44,270 feet, or 28 percent, of the study area shoreline. Groundwater drainage systems were indicated for eight bluff analysis sections, covering about 10,200 feet, or 6 percent of the shoreline. Detailed studies of the groundwater systems should be conducted within these eight sections to determine the feasibility of lowering the elevation of the groundwater. If such detailed studies find that draining the groundwater is not feasible, it is likely that regrading of at least a portion of the bluff slope would then be necessary. Control of surface water runoff was indicated for five bluff analysis sections comprising about 6,830 feet, or 4 percent, of the shoreline. Revegetation of at least a portion of the bluff face, without regrading, was indicated for 15 bluff analysis sections, comprising 16,080 feet, or 10 percent, of the shoreline.

EVALUATION OF COASTAL EROSION DAMAGES

The damages that may be expected to result from continued shoreline erosion and bluff recession can best be expressed in terms of actual property loss and associated economic loss. A major concern is the erosion, and subsequent recession, of coastal terraces, bluffs, and beaches which threaten residential areas, parkland, a few public roadways, and some industrial sites. The recession of the bluff and terrace can be a sporadic process dependent upon the degree of shoreline erosion and the evolution of the bluff slope. In order to approximate the extent and economic value of the land and buildings subject to a risk of erosion damage, the distance the top of the bluff may be expected to recede over a 25-year and 50-year period was calculated for existing marginal or unstable bluff slopes and the Fox Point terrace. These distances were determined by multiplying the average annual recession rates established for the period 1963 through 1985 by 25 years and 50 years. The potential property loss was estimated by multiplying the estimated shoreline erosion and bluff recession distances over periods of 25 and 50 years by the shoreline length, assuming no further erosion control measures were implemented. Because of the sporadic nature of bluff recession, and the numerous factors affecting the recession rates, such rates should not be used to predict the "life" of a particular building or facility on a site-specific basis. Those shoreline areas herein identified as subject to potential erosion damages would be protected if adequate bluff toe protection and slope stabilization measures were provided.

Potential Property Loss

The Milwaukee County shoreline erosion management study focuses on a relatively narrow strip of land which comprises a small portion of the total area of the communities along the Lake Michigan shoreline. Table 46 sets forth—for each local unit of government—the area within the entire study area, the area directly adjacent to the Lake Michigan shoreline, and the area potentially subject to shoreline erosion—that is, lying within marginal or unstable bluff analysis sections and directly adjacent to the shoreline. The term "directly adjacent" was defined as the area consisting of the first tier of real property ownerships along the lake shoreline, generally extending to the first public street paralleling the shoreline. As presented in Table 46, the study area encompasses about 8 percent of the combined total areal extent of those communities that abut Lake Michigan. About one-half of the study area—or about 3.9 percent of the combined total areal extent of the communities—lies directly adjacent to the Lake Michigan shoreline. About one-half of this directly adjacent area—or about 2.0 percent of the combined total areal extent of the communities—is potentially subject to shoreline erosion because the property has a marginal or unstable bluff.

Of the total land area of 1,889 acres directly adjacent to Lake Michigan and potentially subject to shoreline erosion, approximately 844 acres, or 45 percent, is privately owned, while the remaining 1,045 acres, or 55 percent, is publicly owned. This narrow strip and relatively small area of land is, however, an extremely valuable resource, providing a unique setting for high-value residential development and recreational opportunities. These shoreline areas attract users from well inland. It is therefore important to identify those shoreland areas subject to damages by shoreline erosion and bluff recession in order to identify the need for shore protection measures which would provide a desirable and usable shoreline for property owners as well as for the general public.

The property potentially at hazard was delineated for the bluff analysis sections that were determined to have marginal or unstable bluff slopes. Approximately 58,030 feet, or 36 percent, of the study area shoreline were found to be within bluff analysis sections determined to be marginal or unstable. Potential erosion hazard areas were also delineated for the 9,070 feet, or 6 percent, of the shoreline located within Bluff Analysis Section 95, which includes the Fox Point terrace. The land and facilities lying within the calculated 50-year recession distance from the edge of the existing bluff or terrace were considered to be at some risk of erosion damage. The land and facilities lying within the calculated 25-year recession distance from the edge would have the greatest risk of erosion damage.

Table 43

SUMMARY OF EVALUATIONS OF LAKE MICHIGAN BLUFF CONDITIONS IN MILWAUKEE COUNTY: 1986-1987

Civil Division	Public Park	Bluff Analysis Section	Profile Number	Shoreline Length (feet)	Potential for Rotational Sliding	Potential for Translational Sliding	Existing Shoreline or Bluff Toe Erosion
City of Oak Creek	- -	1	- -	4,470	Stable	Stable	Slight
	- -	2	1,2,3	2,820	Marginal	Unstable	Slight
	Bender	3	4,5,6	2,930	Unstable	Unstable	Severe
	Bender	4	7,8	1,980	Unstable	Unstable	Severe
	Bender	5	9	1,070	Unstable	Unstable	Severe
	- -	6	10	1,170	Unstable	Unstable	Severe
	- -	7	11	1,000	Unstable	Unstable	Slight
	- -	8	- -	540	Stable	Stable	Slight
	- -	9	12	570	Unstable	Marginal	Severe
	- -	10	13	400	Marginal	Marginal	Severe
	- -	11	14	1,290	Unstable	Marginal	Severe
	- -	12	- -	3,160	Stable	Stable	Slight
	- -	13	15	1,320	Stable	Stable	Slight
City of South Milwaukee	- -	14	16,17	1,310	Unstable	Unstable	Severe
	- -	15	18	790	Unstable	Unstable	Slight
	- -	16	19	470	Marginal	Stable	Slight
	- -	17	20	440	Unstable	Unstable	Severe
	- -	18	21,22	1,880	Unstable	Unstable	Severe
	Grant	19	- -	3,180	Stable	Stable	Slight
	Grant	20	23	1,280	Unstable	Unstable	Severe
	Grant	21	24	1,060	Marginal	Stable	Severe
	Grant	22	25	950	Unstable	Marginal	Severe
	Grant	23	26	1,200	Unstable	Unstable	Severe
	Grant	24	27	1,910	Marginal	Unstable	Severe
	Grant	25	28	880	Unstable	Unstable	Severe
City of Cudahy	- -	26	29	660	Unstable	Unstable	Severe
	Warnimont	27	30,31	1,850	Unstable	Marginal	Severe
	Warnimont	28	32	2,050	Unstable	Marginal	Severe
	Warnimont	29	33	770	Unstable	Unstable	Severe
	Sheridan	30	34	1,760	Unstable	Unstable	Severe
	Sheridan	31	35	600	Unstable	Unstable	Severe
	Sheridan	32	- -	340	Stable	Stable	Slight
	Sheridan	33	36	2,060	Unstable	Unstable	Severe
	Sheridan	34	37,38	1,780	Marginal	Stable	Slight
	Sheridan	35	39	650	Unstable	Marginal	Slight
	Sheridan	36	40	710	Marginal	Stable	Slight
	Sheridan	37	41,42	1,010	Unstable	Unstable	Severe
City of St. Francis	- -	38	43	1,290	Marginal	Unstable	Moderate
	- -	39	44,45	1,480	Unstable	Unstable	Severe
	- -	40	46	820	Stable	Stable	Moderate
	- -	41	- -	1,650	Stable	Stable	Slight
	- -	42	47	940	Stable	Marginal	Slight
	Bay View	43	48,49,50	1,370	Unstable	Unstable	Severe
	Bay View	44	51	140	Unstable	Unstable	Severe
	Bay View	45	52	80	Marginal	Marginal	Severe
	Bay View	46	53	360	Marginal	Marginal	Severe
	Bay View	47	54	2,470	Stable	Marginal	Moderate
City of Milwaukee	South Shore	48	55	1,420	Stable	Marginal	Slight
	South Shore	49	- -	340	Stable	Stable	Slight
	South Shore	50	56	1,130	Stable	Marginal	Slight
	South Shore	51	- -	570	Stable	Stable	Moderate

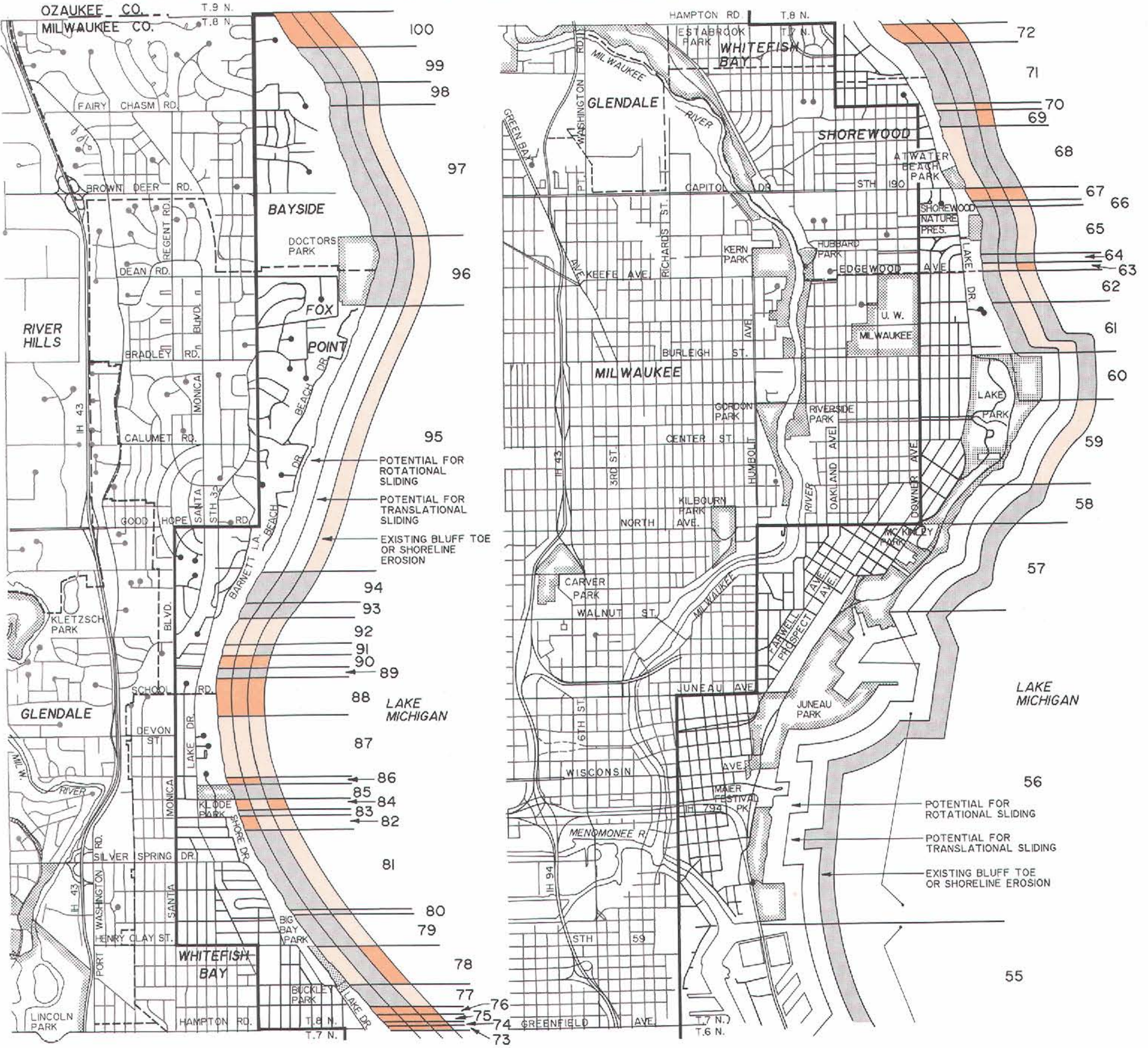
Table 43 (continued)

Civil Division	Public Park	Bluff Analysis Section	Profile Number	Shoreline Length (feet)	Potential for Rotational Sliding	Potential for Translational Sliding	Existing Shoreline or Bluff Toe Erosion
City of Milwaukee (continued)	South Shore	52	- -	450	Stable	Stable	Slight
	South Shore	53	- -	1,320	Stable	Stable	Slight
	South Shore	54	- -	1,360	Stable	Stable	Moderate
	- -	55	- -	14,750	- -	- -	Slight
	Juneau	56	- -	16,060	- -	- -	Slight
	McKinley	57	- -	3,210	- -	- -	Slight
	Bradford	58	- -	1,900	- -	- -	Slight
	Lake	59	- -	3,540	- -	- -	Moderate
	- -	60	- -	2,210	- -	- -	Slight
	- -	61	57	1,970	Stable	Marginal	Slight
	- -	62	58	950	Stable	Marginal	Moderate
Village of Shorewood	- -	63	59,60	300	Marginal	Marginal	Severe
	- -	64	61	290	Stable (fill)	Stable	Slight
	Shorewood Nature Preserve	65	62	1,710	Stable	Stable	Moderate
	- -	66	63	170	Stable (fill)	Stable	Moderate
	- -	67	64	380	Unstable	Unstable	Severe
	Atwater	68	65	2,170	Marginal	Stable	Slight
	- -	69	66	520	Stable	Stable	Severe
	- -	70	67	240	Marginal	Marginal	Severe
	- -	71	68,69,70	2,370	Stable (fill)	Stable	Slight
Village of Whitefish Bay	- -	72	71,72	850	Unstable	Unstable	Severe
	- -	73	73	190	Unstable (fill)	Unstable	Severe
	- -	74	74	160	Unstable	Unstable	Severe
	- -	75	75	310	Unstable (fill)	Unstable	Severe
	- -	76	76	360	Unstable	Unstable	Severe
	- -	77	77,78	810	Stable (fill)	Stable	Moderate
	Buckley, Big Bay	78	79	1,660	Marginal	Stable	Severe
	- -	79	80	1,480	Stable (fill)	Stable	Moderate
	- -	80	81	130	Marginal	Marginal	Slight
	- -	81	82,83	2,970	Stable (fill)	Stable	Moderate
	- -	82	84	490	Unstable	Marginal	Moderate
	- -	83	85	140	Stable (fill)	Stable	Moderate
	- -	84	86	430	Unstable	Marginal	Severe
	Klode	85	87	480	Stable	Stable	Slight
	- -	86	88	170	Unstable	Unstable	Slight
	- -	87	89	1,950	Marginal	Marginal	Moderate
Village of Fox Point	- -	88	90,91	1,150	Unstable	Unstable	Severe
	- -	89	92	320	Stable (fill)	Stable	Slight
	- -	90	93	470	Unstable	Unstable	Severe
	- -	91	94	510	Marginal	Marginal	Slight
	- -	92	95,96	770	Marginal	Marginal	Moderate
	- -	93	97	530	Stable	Stable	Slight
	- -	94	98	1,460	Stable	Stable	Slight
	- -	95	- -	9,070	- -	- -	Moderate
	Doctors	96	99,100	1,890	Stable	Stable	Moderate
Village of Bayside	- -	97	101	4,660	Stable	Stable	Moderate
	- -	98	- -	860	Stable	Stable	Slight
	- -	99	102	1,280	Stable	Stable	Moderate
	- -	100	103,104	1,320	Unstable	Unstable	Severe

Source: SEWRPC.

Map 35

BLUFF CONDITIONS ALONG THE LAKE MICHIGAN SHORELINE OF MILWAUKEE COUNTY: 1986-1987



LEGEND

50 BLUFF ANALYSIS SECTIONS

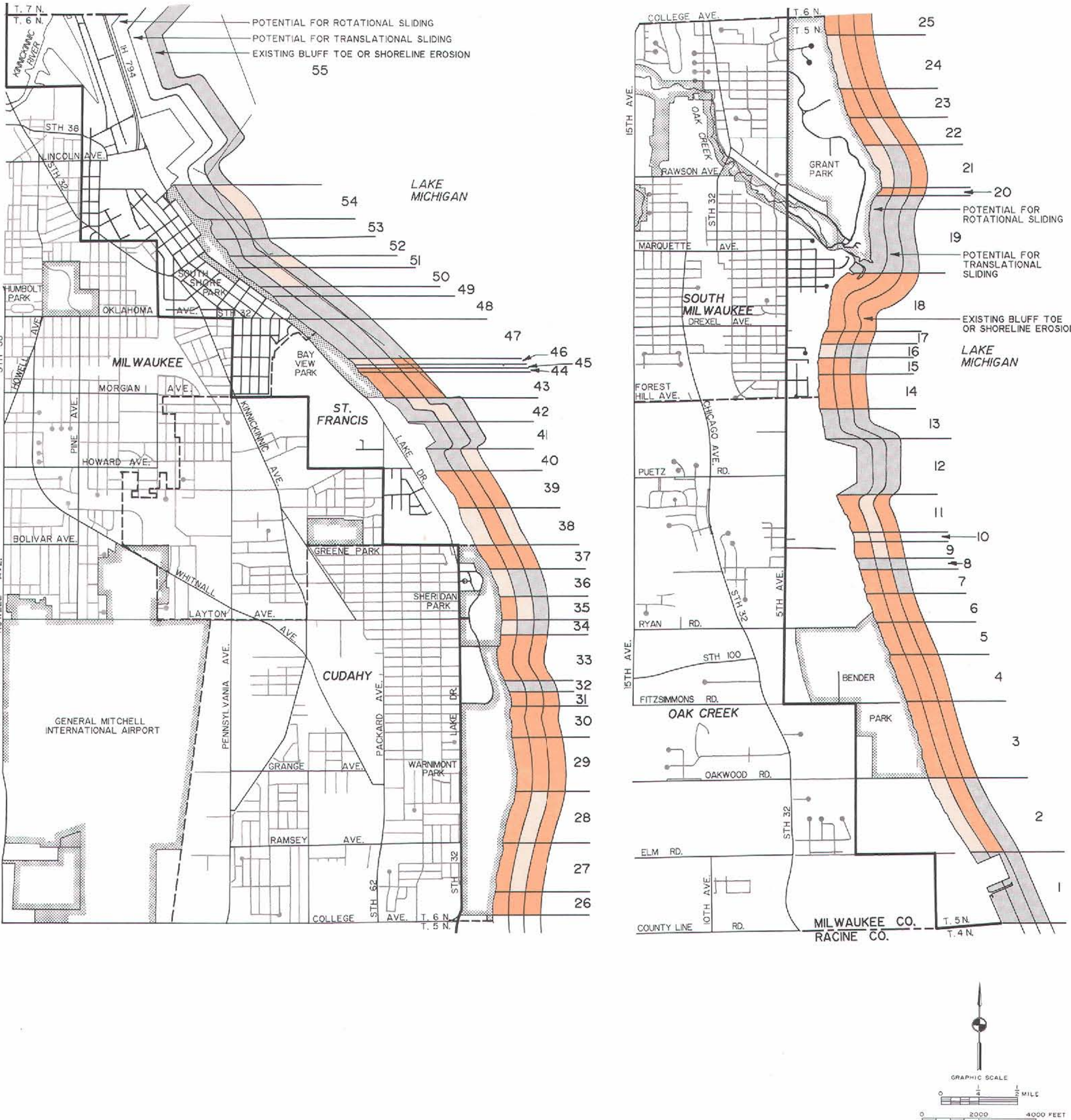
ROTATIONAL OR TRANSLATIONAL SLIDING

- STABLE
- MARGINAL
- UNSTABLE

BLUFF TOE OR SHORELINE EROSION

- SLIGHT
- MODERATE
- SEVERE

Map 35 (continued)



Source: SEWRPC.

Table 44

INDICATED MEASURES TO STABILIZE THE BLUFF SLOPES ALONG THE LAKE MICHIGAN SHORELINE OF MILWAUKEE COUNTY: 1986-1987

Civil Division	Bluff Analysis Section	Bluff Slope Revegetation Without Regrading	Surface Water Runoff Control	Groundwater Drainage	Bluff Slope Regrading and Revegetation[a]
City of Oak Creek	1	- -	- -	- -	- -
	2	- -	X	X	- -
	3	- -	- -	- -	X
	4	- -	- -	- -	X
	5	- -	- -	- -	X
	6	- -	- -	- -	X
	7	- -	- -	- -	X
	8	- -	- -	- -	- -
	9	- -	- -	- -	X
	10	- -	- -	- -	X
	11	- -	- -	- -	X
	12	- -	- -	- -	- -
	13	- -	- -	- -	- -
City of South Milwaukee	14	- -	- -	- -	X
	15	- -	- -	- -	X
	16	- -	- -	- -	- -
	17	- -	- -	- -	X
	18	- -	- -	- -	X
	19	- -	- -	- -	- -
	20	- -	- -	- -	X
	21	- -	- -	X	- -
	22	- -	- -	- -	X
	23	- -	- -	- -	X
	24	- -	- -	- -	X
	25	- -	- -	- -	X
City of Cudahy	26	- -	- -	- -	X
	27	- -	- -	- -	X
	28	- -	- -	- -	X
	29	- -	- -	- -	X
	30	- -	- -	- -	X
	31	- -	- -	- -	X
	32	- -	- -	- -	- -
	33	- -	- -	- -	X
	34	X	- -	- -	- -
	35	X	- -	X	- -
	36	X	- -	- -	- -
	37	- -	- -	- -	X
City of St. Francis	38	- -	- -	- -	X
	39	- -	- -	- -	X
	40	- -	- -	- -	- -
	41	- -	- -	- -	- -
	42	X	- -	- -	- -
	43	- -	- -	- -	X
	44	- -	- -	- -	X
	45	- -	- -	- -	X
	46	- -	- -	- -	X
	47	X	X	- -	- -
City of Milwaukee	48	- -	- -	- -	- -
	49	- -	- -	- -	- -
	50	- -	- -	- -	- -
	51	- -	- -	- -	- -

Table 44 (continued)

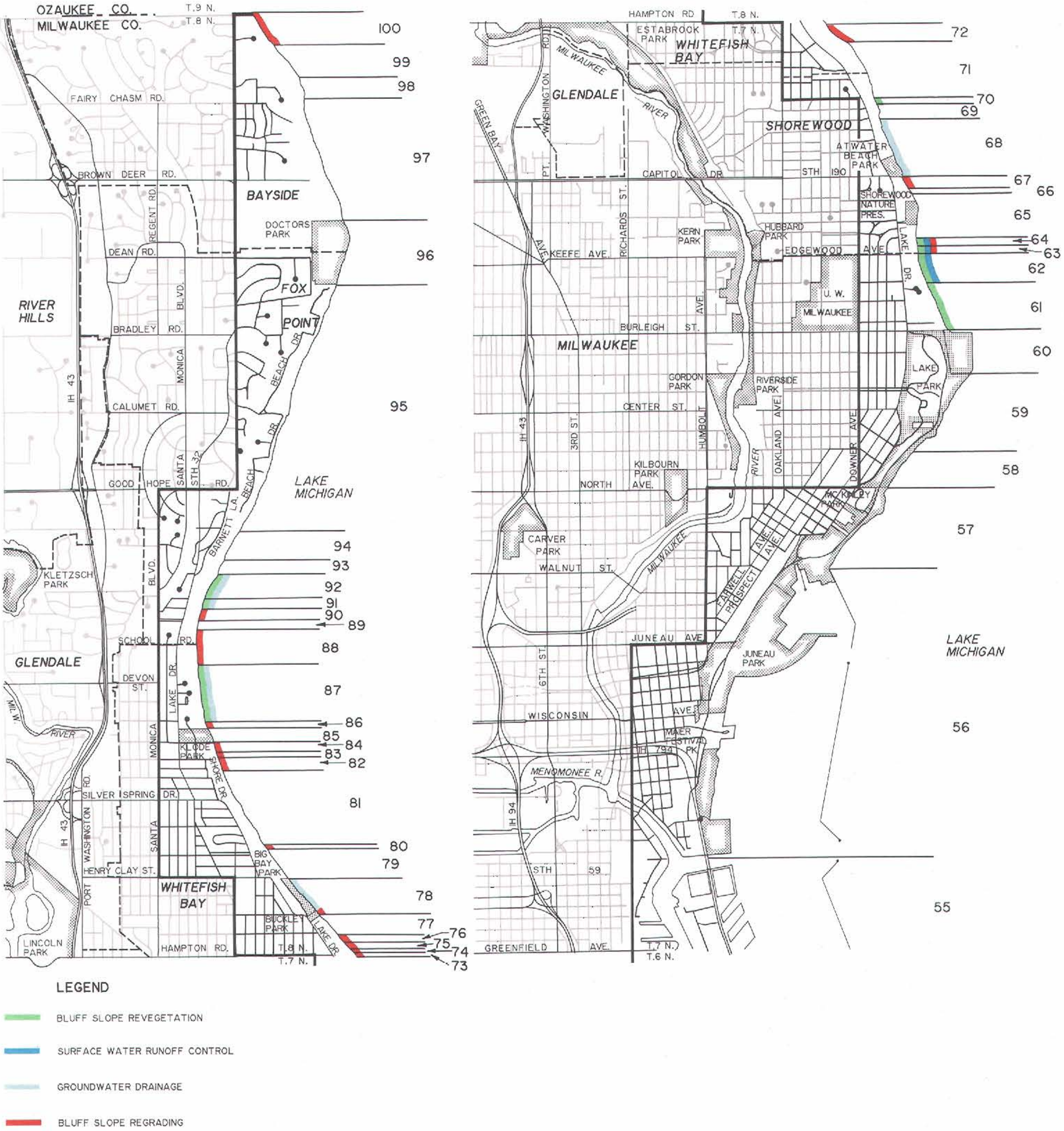
Civil Division	Bluff Analysis Section	Bluff Slope Revegetation Without Regrading	Surface Water Runoff Control	Groundwater Drainage	Bluff Slope Regrading and Revegetation[a]
City of Milwaukee (continued)	52	--	--	--	--
	53	--	--	--	--
	54	--	--	--	--
	55	--	--	--	--
	56	--	--	--	--
	57	--	--	--	--
	58	--	--	--	--
	59	--	--	--	--
	60	--	--	--	--
	61	X	--	--	--
	62	X	X	--	--
Village of Shorewood	63	X	X	--	X
	64	X	X	--	X
	65	--	--	--	--
	66	--	--	--	--
	67	--	--	--	X
	68	--	--	X	--
	69	--	--	--	--
	70	X	--	--	--
	71	--	--	--	--
Village of Whitefish Bay	72	--	--	--	X
	73	--	--	--	X
	74	--	--	--	X
	75	--	--	--	X
	76	--	--	--	X
	77	--	--	--	--
	78	--	--	X	X
	99	--	--	--	--
	80	--	--	--	X
	81	--	--	--	--
	82	--	--	--	X
	83	--	--	--	X
	84	--	--	--	X
	85	--	--	--	--
	86	--	--	--	X
	87	X	--	X	--
Village of Fox Point	88	--	--	--	X
	89	--	--	--	--
	90	--	--	--	X
	91	X	--	X	--
	92	X	--	X	--
	93	--	--	--	--
	94	--	--	--	--
	95	--	--	--	--
	96	--	--	--	--
Village of Bayside	97	--	--	--	--
	98	--	--	--	--
	99	--	--	--	--
	100	--	--	--	X

[a]*Bluff slope regrading may include cutting back the top of the slope and/or placing fill material on the bluff slope.*

Source: SEWRPC.

Map 36

BLUFF STABILIZATION NEEDS FOR THE LAKE MICHIGAN SHORELINE OF MILWAUKEE COUNTY: 1986-1987



Map 36 (continued)

Source: SEWRPC.

Table 45

EXTENT OF INDICATED BLUFF STABILIZATION MEASURES FOR THE MILWAUKEE COUNTY CIVIL DIVISIONS ALONG THE LAKE MICHIGAN SHORELINE 1986-1987

Civil Division	Bluff Slope Revegetation Without Regrading		Surface Water Runoff Control		Groundwater Drainage		Bluff Slope Regrading and Revegetation	
	Shoreline Length (feet)	Percent of Shoreline in Civil Division	Shoreline Length (feet)	Percent of Shoreline in Civil Division	Shoreline Length (feet)	Percent of Shoreline in Civil Division	Shoreline Length (feet)	Percent of Shoreline in Civil Division
City of Oak Creek	0	0	2,820	12	2,820	12	10,410	46
City of South Milwaukee	0	0	0	0	1,060	7	10,640	69
City of Cudahy	3,140	22	0	0	650	5	10,760	8
City of St. Francis . . .	2,430	25	1,490	15	0	0	4,720	49
City of Milwaukee	6,450	12	1,930	4	0	0	0	0
Village of Shorewood	830	13	590	9	1,380	21	970	15
Village of Whitefish Bay	1,950	13	0	0	3,010	21	4,370	30
Village of Fox Point . . .	1,280	9	0	0	1,280	9	1,080	7
Village of Bayside	0	0	0	0	0	0	1,320	14
Total Study Area	16,080	10	6,830	4	10,200	6	44,270	28

Source: SEWRPC.

Table 46

AREAL EXTENT OF STUDY AREA, AREA DIRECTLY ADJACENT TO THE LAKE MICHIGAN SHORELINE, AND AREA POTENTIALLY SUBJECT TO SHORELINE EROSION WITHIN EACH CIVIL DIVISION

Civil Division	Total Areal Extent of Civil Division (acres)	Study Area		Area Directly Adjacent to Lake Michigan Shoreline		Area Potentially Subject to Shoreline Erosion[a]	
		Areal Extent (acres)	Percent of Civil Division	Areal Extent (acres)	Percent of Civil Division	Areal Extent (acres)	Percent of Civil Division
City of Oak Creek	18,180	1,095	6.0	1,039	5.7	829	4.6
City of South Milwaukee . . .	3,100	627	20.2	472	15.2	392	12.6
City of Cudahy	3,030	448	14.8	395	13.0	388	12.8
City of St. Francis	1,640	612	37.3	150	9.1	80	4.9
City of Milwaukee	61,840	2,654	4.3	638	1.0	0	0.0
Village of Shorewood	1,088	306	28.1	69	6.3	19	1.7
Village of Whitefish Bay	1,363	607	44.5	146	10.7	82	6.0
Village of Fox Point	1,843	665	36.1	121	6.6	93	5.0
Village of Bayside	1,416	503	35.5	135	9.5	6	0.4
Total	93,500	7,517	8.0	3,615	3.9	1,889	2.0

[a] *The area potentially subject to shoreline erosion is defined as that land lying within a marginal or unstable bluff analysis section—including the Fox Point terrace—and directly adjacent to the shoreline.*

Source: SEWRPC.

The loss of land and facilities may result from continued shoreline erosion and the parallel retreat of the bluff, or from additional slope failure which would provide a slope more gentle than under existing conditions. It cannot be assumed that the bluff face would remain at its existing angle, and the potential exists for the bluff slope to rapidly, and sometimes catastrophically, recede to a more gentle, and stable, slope angle. The existing bluffs within Milwaukee County could recede to a slope angle as gentle as one on two and one-half, or about 22 degrees, although many existing stable bluff slopes have angles steeper than 22 degrees.

A slope angle of one on two and one-half is similar to the average angle of stable bluff slopes along the Lake Michigan shoreline reported by Edil and Vallejo.[41] Another report by Vallejo and Edil[42] noted that, given certain physical characteristics of the soils, the slope angle at which a bluff becomes stable may be expected to vary in relation to the ratio of the height of the groundwater level—measured from the base of the bluff—to the height of the bluff. As shown in Table 47, the angle at which a bluff slope may become stable ranges from a minimum of 16 degrees if the height of the groundwater is three-fourths or more of the height of the bluff, to a maximum of 31 degrees if no groundwater is contained within the bluff. However, the effect of groundwater on the angle at which a bluff slope may become stable is difficult to determine because:

1. Groundwater levels, and specifically seepage zones, are highly variable on a seasonal and annual basis;

2. Surveys of groundwater seepage zones were conducted during only limited time periods; and

3. Groundwater conditions can change significantly as the bluff recedes and strata of permeable bluff materials are eroded, covered, or disturbed.

Table 47

TYPICAL BLUFF STABLE SLOPES AS A FUNCTION OF THE HEIGHT OF THE GROUNDWATER IN THE BLUFF

Maximum Height of Groundwater in Bluff (H = bluff height)	Ultimate Stable Slope	
	Ratio (horizontal:vertical)	Angle (degrees)
0	1.7:1	31
¼ H	1.8:1	29
½ H	3.0:1	18
¾ H	3.5:1	16
Unknown	2.5:1	22

Source: L. E. Vallejo and T. B. Edil, "Design Charts for Development and Stability of Evolving Slopes," Journal of Civil Engineering Design, Vol. 1, No. 3, 1979, pp. 231-252.

When concrete rubble and soil fill are placed on the face of a bluff, a steeper slope angle can generally be maintained. Fill sites with stable bluff slopes within study area often had slope angles of approximately 35 degrees. Most fill sites were terraced, having broken, or compound, slopes which enhanced the stability of the slopes.

Table 48 summarizes the potential erosion damages within each bluff analysis section, while Tables 49 and 50 summarize potential damages by civil division.

As set forth in Table 49, approximately 62.9 acres of land lie within the 25-year bluff recession distance of existing marginal or unstable bluff analysis sections. Of this total area, about 27.5 acres, or 44 percent, are located within the City of Oak Creek; about 13.0 acres, or 21 percent, are located with the City of Cudahy; about 9.0 acres, or 14 percent, are located within the City of South Milwaukee; and the remaining 13.3 acres, or 21 percent, are located within the Cities of St. Francis and Milwaukee, and the Villages of Fox Point, Whitefish Bay, Shorewood, and Bayside. Of the land at hazard, about 23.9 acres, or 38 percent, is in private ownership, and 39.0 acres, or 62 percent, in public ownership. A total of 24 buildings lie, in whole or in part, within the 25-year bluff recession distance.

Approximately 126.5 acres of land lie within the 50-year bluff recession distance of existing marginal or unstable bluff analysis sections, as

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[41] *T. B. Edil and L. E. Vallejo, "Mechanics of Coastal Landslides and the Influence of Slope Parameters," Engineering Geology, Vol. 16, 1980, pp. 83-96.*

[42] *L. E. Vallejo and T. B. Edil, "Design Charts for Development and Stability of Evolving Slopes," Journal of Civil Engineering Design, Vol. 1, No. 3, 1979, pp. 231-252.*

Table 48

SUMMARY OF POTENTIAL EROSION DAMAGES WITHIN THE MILWAUKEE COUNTY SHORELINE OF LAKE MICHIGAN

Civil Division	Bluff Analysis Section[a]	Shoreline Length (feet)	Average Bluff Recession[b] (feet/year)	25-Year Recession Distance (feet)	25-Year Total Area (feet²)	50-Year Recession Distance (feet)	50-Year Total Area (feet²)
City of Oak Creek	2	2,820	2.4	60	169,200	120	338,400
	3	2,930	3.5	88	257,840	175	512,750
	4	1,980	3.5	88	174,240	175	346,500
	5	1,070	6.4	160	171,200	320	342,400
	6	1,170	11.7	292	341,640	585	684,450
	7	1,000	1.9	48	48,000	95	95,000
	9	570	0.5	12	6,840	25	14,250
	10	400	0.8	20	8,000	40	16,000
	11	1,290	0.7	18	23,220	35	45,150
Subtotal	- -	13,230	- -	- -	1,200,180	- -	2,394,900
City of South Milwaukee	14	1,310	1.7	42	55,020	85	111,350
	15	790	1.5	38	30,020	75	59,250
	16	470	1.0	25	11,750	50	23,500
	17	440	1.3	32	14,080	65	28,600
	18	1,880	1.1	28	52,640	55	103,400
	20	1,280	0.5	12	15,360	25	32,000
	21	1,060	0.7	18	19,080	35	37,100
	22	950	0.5	12	11,400	25	23,750
	23	1,200	0.8	20	24,000	40	48,000
	24	1,910	1.7	42	80,220	85	162,350
	25	880	3.5	88	77,440	175	154,000
Subtotal	- -	12,170	- -	- -	391,010	- -	783,300
City of Cudahy	26	660	1.1	28	18,480	55	36,300
	27	1,850	1.1	28	51,800	55	101,750
	28	2,050	0.9	22	45,100	45	92,250
	29	770	2.2	55	42,350	110	84,700
	30	1,760	4.0	100	176,000	200	352,000
	31	600	2.1	52	31,200	105	63,000
	33	2,060	2.5	62	127,720	125	257,500
	34	1,780	0.9	22	39,160	45	80,100
	35	650	0.7	18	11,700	35	22,750
	36	710	0.5	12	8,520	25	17,750
	37	1,010	0.5	12	12,120	25	25,250
Subtotal	- -	13,900	- -	- -	564,150	- -	1,133,350
City of St. Francis	38	1,290	2.9	72	92,880	145	187,050
	39	1,480	2.8	70	103,600	140	207,200
	43	1,370	1.2	30	41,100	60	82,200
	44	140	0.5	12	1,680	25	3,500
	45	80	0.5	12	960	25	2,000
	46	360	0.5	12	4,320	25	9,000
	47 (part)	1,490	0.5	12	17,880	25	37,250
Subtotal	- -	6,210	- -	- -	262,420	- -	528,200

Table 48 (continued)

Civil Division	Bluff Analysis Section[a]	Shoreline Length (feet)	Average Bluff Recession[b] (feet/year)	25-Year Recession Distance (feet)	25-Year Total Area ($feet^2$)	50-Year Recession Distance (feet)	50-Year Total Area ($feet^2$)
City of Milwaukee	47 (part)	980	0.5	12	11,760	25	24,500
	48	1,420	0.5	12	17,040	25	35,500
	50	1,130	0.5	12	13,560	25	28,250
Subtotal	- -	3,530	- -	- -	42,360	- -	88,250
Village of Shorewood	63	300	0.5	12	3,600	25	7,500
	67	380	0.5	12	4,560	25	9,500
	68	1,380	0.5	12	16,560	25	34,500
	70	240	0.5	12	2,880	25	6,000
Subtotal	- -	2,300	- -	- -	27,600	- -	57,500
Village of Whitefish Bay	72	850	0.5	12	10,200	25	21,250
	73	190	0.5	12	2,280	25	4,750
	74	160	0.5	12	1,920	25	4,000
	75	310	0.5	12	3,720	25	7,750
	76	360	0.5	12	4,320	25	9,000
	78	1,660	0.5	12	19,920	25	41,500
	80	130	0.5	12	1,560	25	3,250
	82	490	0.5	12	5,880	25	12,250
	84	430	0.5	12	5,160	25	10,750
	86	170	0.5	12	2,040	25	4,250
	87	1,950	0.5	12	23,400	25	48,750
	88	540	0.5	12	6,480	25	13,500
Subtotal	- -	7,240	- -	- -	86,880	- -	181,000
Village of Fox Point	88	610	0.5	12	7,320	25	15,250
	90	470	0.5	12	5,640	25	11,750
	91	510	0.5	12	6,120	25	12,750
	92	770	0.5	12	9,240	25	19,250
	95	9,070	0.5	12	108,840	25	226,750
Subtotal	- -	11,430	- -	- -	137,160	- -	285,750
Village of Bayside	100	1,320	0.5	22	29,040	45	59,400
Total Study Area	- -	71,330	- -	- -	2,740,800	- -	5,511,650

[a]*Includes only marginal or unstable bluff analysis sections.*

[b]*Average annual bluff recession rates are based on average recession rates calculated over the period 1963 through 1985.*

Source: SEWRPC.

Table 49

ECONOMIC VALUE OF LAND AND BUILDINGS LYING WITHIN THE 25-YEAR BLUFF RECESSION DISTANCE OF THE EDGE OF MARGINAL OR UNSTABLE BLUFFS OR TERRACES WITHIN THE LAKE MICHIGAN SHORELINE OF MILWAUKEE COUNTY

Civil Division	Land			Buildings			Total	
	Extent (acres)	Economic Value[a]	Percent of Total Economic Value	Number	Economic Value[a]	Percent of Total Economic Value	Economic Value[a]	Percent of Total Economic Value
City of Oak Creek	27.5	$ 78,000	1.6	1	$ 50,000	1.1	$ 128,000	2.7
City of South Milwaukee	9.0	127,700	2.6	0	- -	- -	123,400	2.6
City of Cudahy	13.0	42,700	0.9	0	- -	- -	42,700	0.9
City of St. Francis	6.0	136,800	2.9	0	- -	- -	136,800	2.9
City of Milwaukee	1.0	17,000	0.4	0	- -	- -	17,000	0.4
Village of Shorewood	0.6	37,300	0.8	10	1,668,100	35.2	1,705,400	36.0
Village of Whitefish Bay	2.0	155,600	3.3	4	633,600	13.4	789,200	16.7
Village of Fox Point	3.1	242,400	5.1	7	1,081,500	22.9	1,323,900	28.0
Village of Bayside	0.7	54,600	1.2	2	409,700	8.6	464,300	9.8
Total	62.9	$ 892,200	18.8	24	$3,842,900	81.2	$4,735,100	100.0

[a]*Economic values are in 1986 dollars.*

Source: SEWRPC.

presented in Table 50. Of this total area, about 55 acres, or 43 percent, are located within the City of Oak Creek; about 26 acres, or 21 percent, are located within the City of Cudahy; about 18 acres, or 14 percent, are located within the City of South Milwaukee; and the remaining 27.5 acres, or 20 percent, are located within the Cities of St. Francis and Milwaukee, and the Villages of Fox Point, Whitefish Bay, Shorewood, and Bayside. Of the land at hazard, about 48.2 acres, or 38 percent, is in private ownership, and 78.3 acres, or 62 percent, is in public ownership. A total of 44 buildings lie, in whole or in part, within the 50-year bluff recession distance.

Potential Economic Loss

A measure of the potential economic losses resulting from continued bluff recession was made by determining the economic value of the land and facilities located within the 25-year and 50-year recession distance from the edge of existing bluffs and terraces, and thus considered to be at some risk of recession and erosion damage. The values of land and facilities in those areas were estimated based upon the 1986 equalized assessed valuations—which by law should approximate market value—as presented in the Milwaukee County statistical report prepared for tax equalization purposes according to State law. Map 37 and Tables 49 and 50 summarize the approximate economic value of the land and facilities contained within the 25-year and 50-year bluff recession distances from the edge of a marginal or unstable bluff or terrace for each civil division in the study area. The economic values presented in the table do not include the value of public utilities and improvements such as streets and sewers; nor do

Table 50

ECONOMIC VALUE OF LAND AND BUILDINGS LYING WITHIN THE 50-YEAR BLUFF RECESSION DISTANCE OF THE EDGE OF MARGINAL OR UNSTABLE BLUFFS OR TERRACES WITHIN THE LAKE MICHIGAN SHORELINE OF MILWAUKEE COUNTY

Civil Division	Land			Buildings			Total	
	Extent (acres)	Economic Value[a]	Percent of Total Economic Value	Number	Economic Value[a]	Percent of Total Economic Value	Economic Value[a]	Percent of Total Economic Value
City of Oak Creek	55.0	$ 154,400	1.8	1	$ 50,000	0.6	$ 204,400	2.4
City of South Milwaukee	18.0	254,200	2.9	4	138,000	1.6	392,200	4.5
City of Cudahy	26.0	85,400	1.0	0	- -	- -	85,400	1.0
City of St. Francis	12.2	276,200	3.2	0	- -	- -	276,200	3.2
City of Milwaukee	2.0	34,200	0.4	0	- -	- -	34,200	0.4
Village of Shorewood	1.3	92,400	1.1	17	2,710,700	31.1	2,803,100	32.2
Village of Whitefish Bay	4.1	318,900	3.7	9	1,434,200	16.5	1,753,100	20.2
Village of Fox Point	6.5	578,500	6.7	11	2,040,500	23.5	2,619,000	30.2
Village of Bayside	1.4	106,400	1.2	2	409,700	4.7	516,100	5.9
Total	126.5	$1,900,600	22.0	44	$6,783,100	78.0	$8,683,700	100.0

[a]*Economic values are in 1986 dollars.*

Source: SEWRPC.

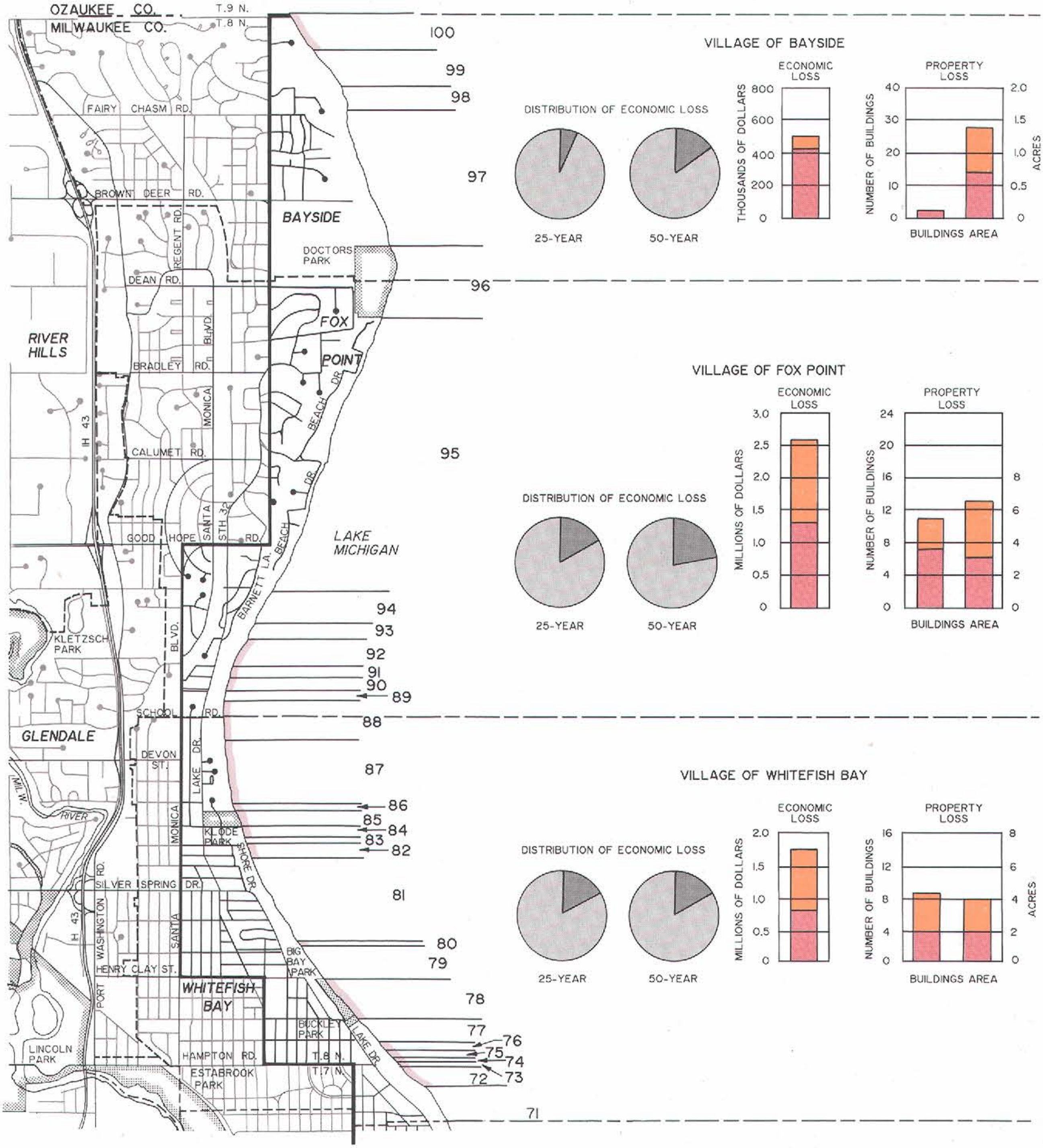
they include the value of lakeshore protection structures, beaches, or facilities. The recommended plan presented in Chapter IV includes the cost of those measures needed to protect the shoreline, including reconstruction or maintenance of some existing structures.

The total economic value of land and buildings lying within the 25-year bluff recession distance of the edge of marginal or unstable bluffs or terraces is approximately $4.7 million, of which about $0.9 million, or 19 percent, represents the value of the land, and about $3.8 million, or 81 percent, represents the value of the buildings. Of the total value, $1.7 million, or 36 percent, represents property in the Village of Shorewood; about $1.3 million, or 28 percent, represents property in the Village of Fox Point; about $0.8 million, or 17 percent, represents property in the Village of Whitefish Bay; and $0.5 million, or 10 percent, represents property in the Village of Bayside. The remaining $0.4 million, or about 9 percent, represents property in the Cities of Cudahy, Oak Creek, South Milwaukee, St. Francis, and Milwaukee.

The total economic value of land and buildings lying within the 50-year bluff recession distance of the edge of marginal or unstable bluffs or terraces is approximately $8.7 million, of which about $1.9 million, or 22 percent, represents the value of the land; and about $6.8 million, or 78 percent, represents the value of the buildings. Of the total value, about $2.8 million, or 32 percent, represents property in the Village of Shorewood; about $1.8 million, or 20 percent, represents property in the Village of Whitefish Bay; about $2.6 million, or 30 percent, represents

Map 37

ECONOMIC VALUE OF LAND AND BUILDINGS LYING WITHIN THE 25- AND 50- YEAR BLUFF RECESSION DISTANCE WITHIN MARGINAL AND UNSTABLE BLUFFS IN MILWAUKEE COUNTY



LEGEND

50 BLUFF ANALYSIS SECTION

MARGINAL OR UNSTABLE BLUFF ANALYSIS SECTION (ROTATIONAL SLIDING)

POTENTIAL EROSION DAMAGES

PROPERTY LOCATED WITHIN 25-YEAR BLUFF RECESSION DISTANCE

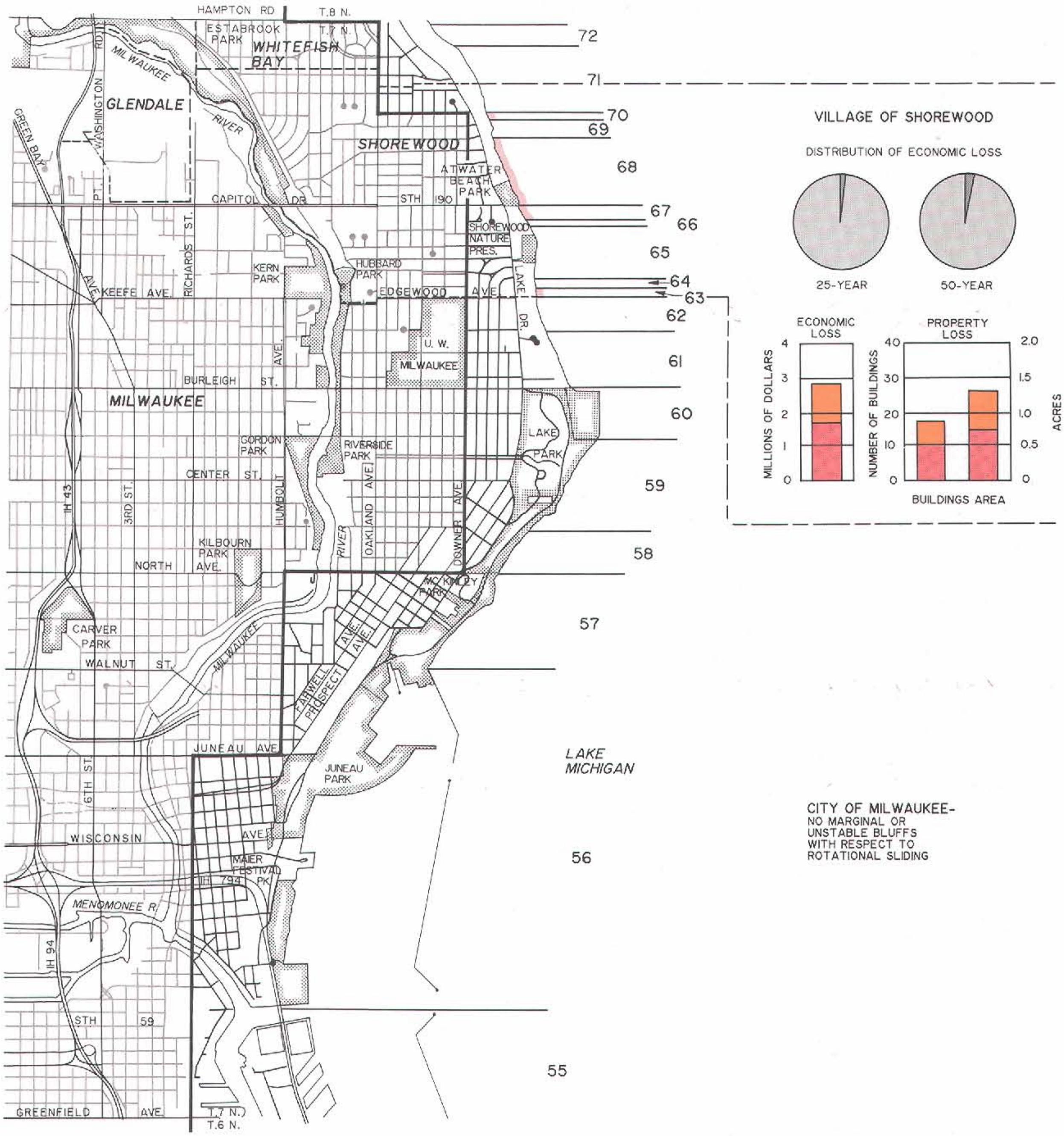
PROPERTY LOCATED WITHIN 50-YEAR BLUFF RECESSION DISTANCE

DISTRIBUTION OF POTENTIAL ECONOMIC LOSS

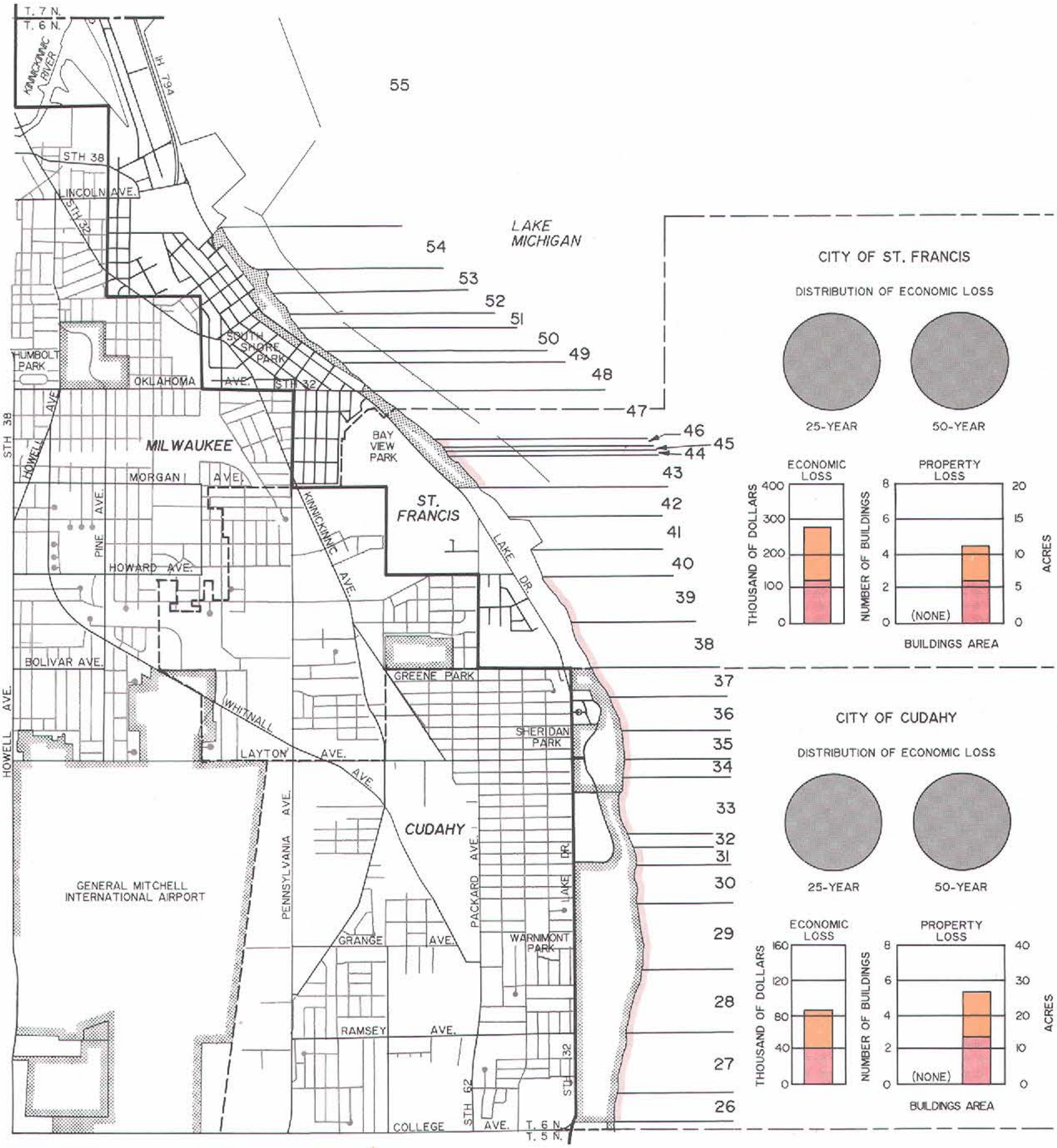
LAND

IMPROVEMENTS

Map 37 (continued)



Map 37 (continued)



LEGEND

50 BLUFF ANALYSIS SECTION

MARGINAL OR UNSTABLE BLUFF ANALYSIS SECTION (ROTATIONAL SLIDING)

POTENTIAL EROSION DAMAGES

PROPERTY LOCATED WITHIN 25-YEAR BLUFF RECESSION DISTANCE

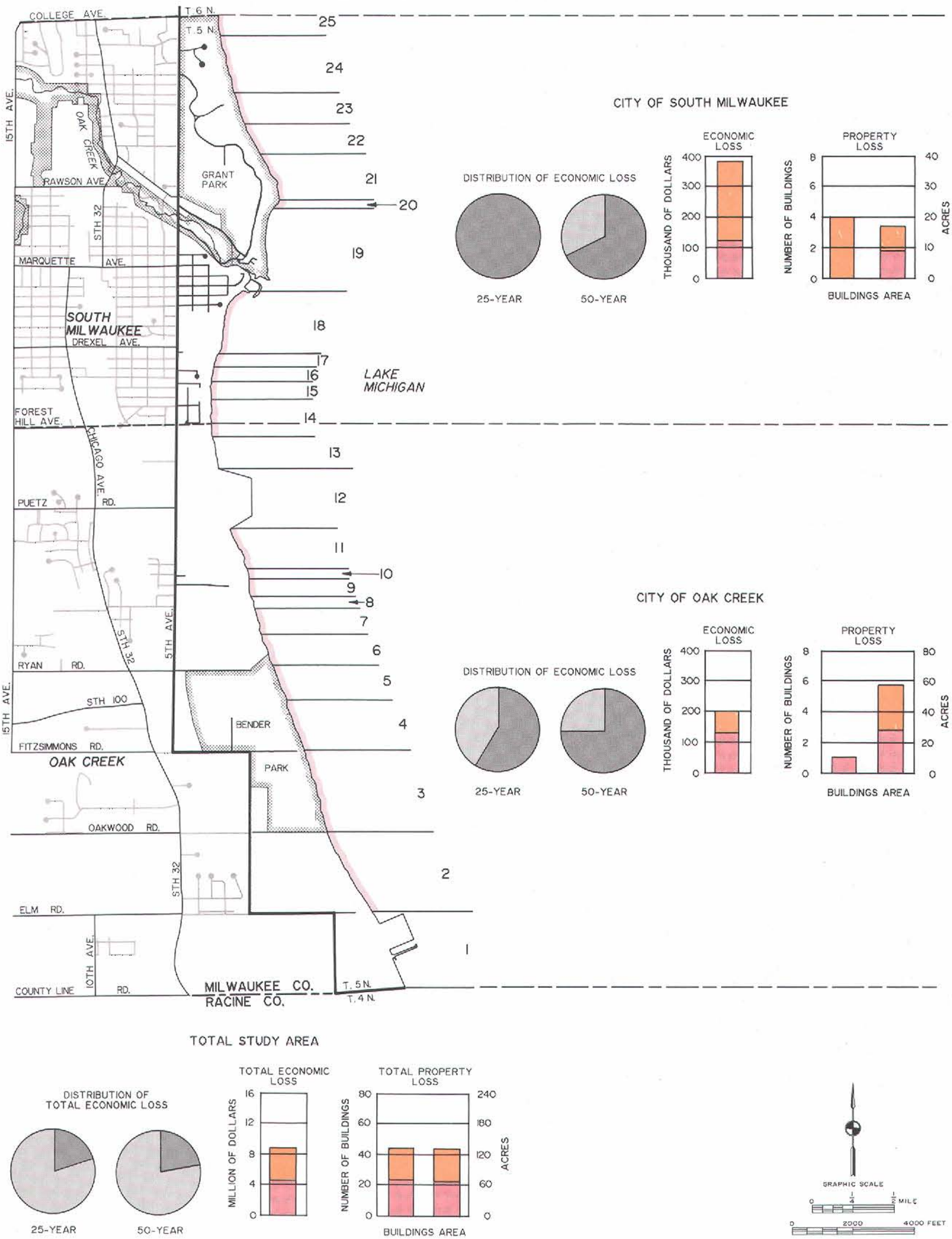
PROPERTY LOCATED WITHIN 50-YEAR BLUFF RECESSION DISTANCE

DISTRIBUTION OF POTENTIAL ECONOMIC LOSS

LAND

IMPROVEMENTS

Map 37 (continued)



Source: SEWRPC.

property in the Village of Fox Point; about \$0.5 million, or 6 percent, represents property in the Village of Bayside; and about \$0.4 million, or 5 percent, represents property in the City of South Milwaukee. The remaining \$0.6 million, or about 7 percent, represents property in the Cities of Cudahy, Oak Creek, St. Francis, and Milwaukee.

SUMMARY

This chapter has evaluated the shoreline erosion and bluff recession occurring within the study area; evaluated the stability of the bluff slopes; identified the factors causing the erosion and attendant bluff recession; identified the types of control measures needed to stabilize the bluff slopes; assessed the potential for wave overtopping damage to major shore protection structures and beaches; and estimated the property and economic losses that may result if shoreline protection is not implemented. The identification of the shoreland areas that may be expected to continue to be affected by shoreline erosion, bluff recession, and storm damage enables public officials and private property owners to better assess potential erosion losses and evaluate and select appropriate control measures to abate those problems.

Analytic procedures and geotechnical engineering techniques were used to evaluate the existing and potential coastal erosion problems within each of 100 bluff analysis sections. The evaluation included a determination of the stability of the bluff slope with respect to rotational sliding and translational sliding, and an assessment of the severity of bluff toe erosion.

With respect to rotational sliding, 32 percent of the total study area shoreline was determined to have stable bluff slopes, 11 percent to have marginal bluff slopes, and 25 percent to have unstable bluff slopes. Bluff slope stability was not evaluated for the remaining 32 percent of the shoreline, consisting of the shoreline protected by the Milwaukee Harbor breakwater, the terrace directly north of the harbor to the City of Milwaukee Linnwood Avenue water treatment plant, and the Fox Point terrace.

With respect to translational sliding, 33 percent of the total study area shoreline was determined to have stable slopes, 12 percent to have marginal bluff slopes, and 23 percent to have unstable bluff slopes.

With respect to bluff toe erosion, 49 percent of the total study area shoreline was observed to have little or no shoreline or bluff toe erosion in the field surveys conducted in 1986 and 1987. About 24 percent of the shoreline was found to be exhibiting erosion at the toe of the bluff, but the erosion did not appear to affect the overall stability of the bluff slope. The remaining 27 percent of the shoreline was found to be exhibiting toe erosion which was threatening the overall stability of the bluff slope.

The measures needed to stabilize the bluff slopes within each bluff analysis section were identified. It was recommended that the bluff slopes within about 28 percent of the study area shoreline be regraded to a stable slope angle and revegetated; that groundwater drainage systems be installed to lower the elevation of the groundwater along about 6 percent of the shoreline; that surface water runoff control measures be implemented along about 4 percent of the shoreline; and that revegetation of the bluff slope be provided for about 10 percent of the shoreline.

The performance of 35 major shore protection structures and beaches was evaluated under six different Lake Michigan maximum water level and storm wave conditions. Three maximum water level conditions were selected based on statistical analyses of recorded water level data, and on a review of recorded data, hydrologic response modeling results, and geological and archaeological evidence. The three maximum water levels evaluated included the 10-year recurrence interval instantaneous maximum water level—elevation 582.8 feet National Geodetic Vertical Datum (NGVD); the 100-year recurrence interval instantaneous maximum water level—elevation 584.3 feet NGVD; and the upper 95 percent confidence limit of the 500-year recurrence interval instantaneous maximum water level—elevation 585.9 feet NGVD. Each of these three water levels was used with a 20-year recurrence interval storm wave, which has an approximate height of 21.0 feet in deep water, and with a 50-year recurrence interval storm wave, which has an approximate height of 24.8 feet in deep water, to evaluate the potential for wave overtopping damage to major structures and beaches. In addition, two minimum water levels were selected to evaluate damages which could occur under extreme low-water-level conditions: the 100-year recurrence interval minimum monthly level—elevation 575.5 feet NGVD; and the 100-year recurrence interval instantaneous minimum level—elevation 574.9 feet NGVD.

For each structure or beach, the potential for wave overtopping damage was classified as insignificant, low, moderate, or high. The wave analysis indicated that the extent and degree of wave overtopping damage was heavily influenced by the water level. The height of the deep-water storm wave is frequently of secondary importance because the near-shore wave heights are often limited by the near-shore water depths. Beaches are the least threatened by wave overtopping damage, followed by revetments. Bulkheads were found to be the most susceptible to wave overtopping damage. Most bulkheads may be expected to be damaged under even a 10-year water level, whereas substantial damage to most revetments may not occur until a 100-year water level is reached. Most beaches in Milwaukee County would be overtopped only under the most extreme maximum water level evaluated—the 90 percent upper confidence limit of the 500-year water level. Overall, from 49 to 57 percent of the structures and beaches would have a moderate or high potential for overtopping damage under a 10-year water level, compared to 71 to 77 percent of the structures and beaches for a 100-year water level, and 80 to 89 percent for a 500-year water level.

An analysis of damages that could result from extremely low Lake Michigan water levels indicated that only one structure—the Milwaukee County War Memorial Center bulkhead—may be expected to suffer structural damage owing to the exposure to the atmosphere of timber pilings, with attendant rotting of the timber, which is already subject to toe erosion. In addition, toe erosion of 14 structures could increase under very low water levels because the toes of the structures would be exposed to direct wave attack. These structures include 75 percent of the major revetments in the County, and 33 percent of the major bulkheads.

The land area lying within the 25-year and 50-year bluff recession distance of a marginal or unstable bluff or terrace was delineated on large-scale topographic maps. The area lying within the 25-year bluff recession distance of the marginal or unstable bluffs and terraces was found to encompass about 62.9 acres of land and 24 buildings. The economic value of the land and buildings was estimated at $4.7 million. About 39.0 acres, or 62 percent of the land within the 25-year bluff recession distance, were found to be in public ownership, while the remaining 23.9 acres, or 38 percent, were found to be in private ownership. About 126.5 acres of land were found to lie within the 50-year bluff recession distance of the marginal or unstable bluffs and terraces. The economic value of the land and buildings was estimated at $8.7 million. About 78.3 acres, or 62 percent of the land within the 50-year bluff recession distance, were found to be in public ownership, while the remaining 48.2 acres, or 38 percent, were found to be in private ownership. The areas identified as subject to potential erosion damages would be protected if adequate bluff toe protection and slope stabilization measures were implemented.

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Chapter IV

ALTERNATIVE SHORELINE EROSION CONTROL MEASURES AND A RECOMMENDED SHORELINE EROSION MANAGEMENT PLAN FOR MILWAUKEE COUNTY

INTRODUCTION

Alternative measures to protect the shoreline and stabilize the bluff slopes within Milwaukee County were identified to resolve the erosion, storm damage, and bluff slope stability problems described in Chapter II and evaluated in Chapter III. This chapter describes those alternative measures, and presents an evaluation of the costs and effects of those measures as the basis for the selection of a recommended shoreline erosion, bluff recession, and storm damage control plan for Milwaukee County. The alternative shoreline erosion control and bluff stabilization measures presented in this chapter include both structural measures such as bluff toe protection, surface and groundwater drainage control, and bluff slope stabilization, and nonstructural measures such as zoning and land use management.

The alternative erosion control and bluff stabilization measures presented herein were evaluated with respect to technical effectiveness, economic feasibility, environmental impacts, and implementability. Various methods of financing and implementing the erosion control measures were considered, and an implementation program proposed as part of the recommended plan. The recommended plan reflects the concerns and preferences of the local lakefront communities, as expressed through the guidance provided by the Intergovernmental Coordinating and Technical Advisory Committee for the study.

The first section of this chapter following the introduction presents design criteria and analytic procedures used in the development and evaluation of the alternative control measures. The second section describes the conceptual measures that could be utilized within the study area. The third section describes alternative shore protection plans. The fourth section describes the recommended shoreline erosion, bluff recession, and storm damage control plan for Milwaukee County; the fifth section describes the recommended implementation program; and the sixth and final section summarizes the findings and recommendations of the chapter.

PLAN DESIGN AND ANALYSIS

An understanding of the planning process applied and the level of analysis used in the development of the shoreline plan herein presented is essential to a proper understanding of the plan itself and the steps required for its proper implementation. Importantly, those steps include additional site-specific evaluations in the preliminary engineering phase and final design phase of the measures to be carried out. The systems level planning, which is the focus of this study, entails the application of analytical procedures and design criteria that are intended to ensure a suitable level of shore protection and a consistent basis for comparing alternative protection measures.

Planning Process

The planning process used to prepare this shoreline control plan constitutes the first, or systems planning, phase of what may be regarded as a three-phase shore protection development process. Preliminary engineering is the second phase in this sequential process, with final design being the third and last phase. The systems planning is comprehensive and area-wide, covering the entire section of shoreline concerned. The preliminary engineering and the final design phases combined are more site-specific, focusing on selected subsections of the shoreline, and on individual real property ownerships.

The systems planning phase concentrates on the definition and description of the erosion problems to be addressed, and on the development and evaluation of alternative measures for resolution of those problems. Systems planning is intended to permit the selection of the most effective and desirable measures to resolve the identified problems. Each alternative plan is developed in sufficient detail to permit a sound, consistent comparison of the technical and economic aspects of the plans. Properly conducted, systems planning takes into consideration the pertinent characteristics of the entire coastal environment, the effects of shore protection on adjacent shoreline areas, and the full spectrum of potential shore protection measures. The key to efficient systems planning is not examining each of the many possible alternative measures, but rather examining alternatives that are truly representative of the full range of available measures. Systems planning is not carried out in sufficient detail to permit immediate implementation of the recommended measures.

Implementation of the recommended systems level plan requires that the technical, economic, environmental, and other features of the plan elements be examined in greater depth and detail. The second, or preliminary engineering, phase of the shore protection development process is properly carried out by the implementing units of government and private property owners. The preliminary engineering phase, which should be conducted for individual bluff analysis sections, is no longer comprehensive. It concentrates on the solution identified in the recommended systems plan, and will usually involve the collection and analysis of more detailed geotechnical and coastal engineering data. The preliminary engineering phase, using more detailed site-specific data, either reaffirms or revises the solution set forth in the recommended plan, and determines the best way to carry out the recommended solution.

The third phase, or final design, should also be carried out by the implementing units of government and private property owners. The final design phase consists of the development of construction plans and specifications needed to completely implement the needed shore protection measures. The final design should include layout drawings, construction details, materials specifications, a schedule for construction, and access arrangements. The final design plan should also include the existing and proposed profile of the bluff slope, the quantity of materials to be used, material placement instructions, and an inspection and quality assurance program to ensure compliance with plans.

For many reasons, the planning process for shore protection often does not proceed in the simple three-step process described above. In some cases, an iterative process occurs whereby a reexamination of an earlier phase is required. This frequently occurs where additional data are collected and analyzed. Changes in federal and state regulations can also disrupt the planning process. In planning for shore protection, there is a tendency to circumvent critical steps in the planning process—sometimes in an attempt to minimize costs, and sometimes in response to intense concern and controversy over a particularly severe problem. This approach may achieve short-term benefits in that it leads to a prompt resolution of the pressing problem. Unfortunately, however, circumvention of key planning steps often leads to long-term problems as a result of the failure to fully define the problem concerned, and to determine the best and most cost-effective long-term solution to that problem.

Analytical Procedures and Design Criteria

The lack of consistent analytical procedures and design criteria has sometimes limited the effectiveness of shore protection projects. Chapter II demonstrated that the existing shore protection measures in Milwaukee County have had varying degrees of success, with about 75 percent of the structures in need of repair or maintenance, and exhibiting some type of damage. Proposals for new shore protection measures have generally not included an analysis of potential adverse impacts on adjacent shoreline areas. In some cases, shore protection measures have been designed and constructed without a thorough understanding of the coastal processes and hydrogeologic features affecting the site concerned, or of the interaction of that site with adjacent shoreline reaches.

The site-specific analytical procedures and design criteria for shore protection presented herein represent a consistent set of guidelines which can and should be applied not only in the systems level phase, but also in the preliminary engineering and final design phases of the shore protection development process. These procedures and criteria are intended to promote a better understanding of the data collection and analysis efforts needed prior to plan implementation. The design criteria were used to design the alternative plans set forth in this systems level planning report, to help test and evaluate their technical feasibility, and to ensure the comparability of those plans.

Recommended analytical procedures and design criteria for bluff toe erosion control, bluff slope regrading and revegetation, groundwater drainage, and surface water management are set forth in Table 51. These procedures and criteria provide the means for sizing and thereby ensuring the performance of shore protection measures, thus providing a uniform and consistent base of reference for use in project development and design. Because of the variability of coastal and hydrogeologic conditions along the shoreline, step-by-step instructions to properly analyze or design a shore protection project cannot be specified. Table 51 lists those issues that should be addressed in site-specific analyses, recognizing that the actual analyses may have to be varied depending on the site characteristics.

Total shore protection at a site will often involve the implementation of more than one specific management measure. The application of these recommended procedures and criteria alone will not assure that the total shore protection project is properly integrated, or that the project is fully consistent with adjacent shore protection projects. Thus, additional planning and engineering efforts will be needed to test, with adjustments made as necessary, the performance of the proposed total project. Furthermore, certain design elements may be in conflict and require resolution through compromise, such compromise being an essential part of any design effort. It should also be noted that these recommendations are minimum procedures and criteria; some sites will require additional analyses or more stringent performance criteria.

Two of the recommended criteria—the design water level and the design recurrence interval storm wave—deserve further discussion. It is recommended that major shore protection structures be designed to prevent severe damage under at least the 100-year recurrence interval instantaneous maximum lake level of 584.3 feet National Geodetic Vertical Datum (NGVD). The 100-year lake level is recommended because most major public or quasi-public buildings and facilities, along with their attendant infrastructures, may be reasonably assumed to have an economic life of at least 50 years. There is a 40 percent probability that a 100-year water level will occur during an average 50-year period. Thus, there is a reasonable probability that a 100-year water level will occur at least once within the economic life of a typical major lakefront building or facility. Furthermore, in addition to their economic value, many lakefront buildings and facilities provide essential benefits related to public safety and health.

In addition, structures should be designed to perform well under a range of stillwater conditions, as opposed to one design level. It is recommended that the performance of a major structure under lake levels ranging from 574.9 to 585.9 feet NGVD be considered in the design of new major structures. While it may not be feasible to prevent significant damage to a structure under extreme water level conditions, design provisions should be made to ensure that the damage is readily repairable, that the structure does not collapse, and that the building or facility being protected is not seriously threatened. For example, a structure designed to prevent serious damage only under a 100-year water level would likely be damaged by a major storm that occurred during a 500-year water level. However, if the structure were properly designed in accordance with the proposed criteria, that damage would be modest—such as scouring of backfill material by waves which overtopped the structure, and the structure could be repaired without total reconstruction being necessary. In this example, the structure would be designed to remain structurally intact even if backfill material was temporarily scoured away.

It is recognized that it may not be economically feasible for many residential lakefront property owners to construct shore protection structures that are designed to prevent damage during a major storm with a 100-year water level. It is therefore recommended that shore structures protecting single-family residential dwellings be designed to prevent major damage during a major storm with at least a 10-year recurrence interval water level—or 582.8 feet NGVD. A greater level of protection—where possible as high as a 100-year water level design—should also be considered for residential structures, with the selected design water level being dependent upon the financial resources available to the property owner, the risk of property loss, the threat to human safety, and the value of the property. All structures should be designed to prevent total failure and collapse during a 100-year water level storm.

At the design lake levels specified above, it is recommended that major shore protection structures be designed to prevent severe damage by the 20-year recurrence interval wave height, which in deep water approximates 21.0 feet. This level of protection is appropriate for residential property, public parkland, and limited use roadways. For major public or quasi-public facilities where shoreline damages could have catastrophic effects, it is recommended that consideration be given to designing structures to

Table 51

RECOMMENDED SITE-SPECIFIC INVENTORIES, ANALYSES, AND DESIGN CRITERIA FOR SHORE PROTECTION MEASURES

Shoreline Problem	Potentially Applicable Shore Protection Measures	Site Specific Inventories and Analyses	Design Criteria
Bluff Toe Erosion	Revetments, bulkheads, onshore and nearshore beach systems, offshore breakwaters, offshore islands, and peninsulas	1. Determine lake bottom profiles offshore of proposed measure, and 300 feet on both sides of proposed structure, from the shoreline out to a minimum water depth of 12 feet. Extend lake bottom profiles far enough offshore to include primary and secondary sand bars, if present 2. Calculate the anticipated wave height and runup at the structure under the design water level and storm wave conditions, and under an appropriate range of water level and wave conditions 3. Evaluate the impacts on adjacent shoreline areas of wave reflection or interruption of the littoral drift. Estimate the amount of beach material which is expected to be removed from the drift zone by the proposed shore protection measure. Evaluate the lakeward limit of significant sand transport and estimate littoral drift rates at the site 4. Determine the capability of the lake-bed materials to a suitable depth to provide an adequate foundation to support the proposed structure 5. Identify available access sites for construction and maintenance activities, and the cost and availability of suitable construction materials	1. Major structures should be designed to prevent severe damage and operate well under the 100-year recurrence interval instantaneous maximum Lake Michigan level—which includes seiche effects and wind setup during storms—of 584.3 feet NGVD (583.0 feet IGLD). Structures should also be designed to perform well under a wide range of water levels, rather than a single design level. The design of structures should consider performance under various lake levels, ranging from a low of the 100-year recurrence interval instantaneous minimum water level of 574.9 feet NGVD (573.6 feet IGLD) to the upper 95 percent confidence limit of the 500-year recurrence interval maximum instantaneous water level of 585.9 feet NGVD (587.2 feet IGLD). Higher water levels may be used to design offshore structures, and structures which protect major public facilities where storm damage would have catastrophic impacts. Structures protecting single-family residential property should be designed to prevent severe damage and operate well under at least the 10-year recurrence interval instantaneous maximum water level of 582.8 feet NGVD (584.1 feet IGLD) 2. Major structures should be designed to prevent severe damage and operate well, at the design lake level, under the 20-year recurrence interval storm wave height. Consideration should be given to using a 50-year recurrence interval design storm wave for offshore structures and for structures which protect major public facilities where storm damage would have catastrophic impacts 3. Structures should be designed to prevent severe damage from undercutting, flanking, or overtopping during the design storm. Positive drainage for water which overtops the structure and for groundwater which seeps toward the structure should be provided, and filter cloth and stone bedding layers should be properly applied 4. Structures should be designed to resist earth pressures and to protect against excessive hydrostatic pressures behind the structures 5. Bluff toe protection structures should be uniformly implemented over extensive segments of shoreline, and should not increase erosion of adjacent shoreline areas. Bulkheads should be used only where necessary to accommodate important or essential shoreline uses, and then measures should be taken to minimize wave reflection and adverse impacts on adjacent

Table 51 (continued)

Shoreline Problem	Potentially Applicable Shore Protection Measures	Site Specific Inventories and Analyses	Design Criteria
Bluff Toe Erosion (continued)			shoreline areas. Groins and other beach-containing structures should be artificially nourished with beach material as needed. Groin construction should begin at the downdrift end of the shoreline segment, and the beach fill should be placed promptly following completion of the groins 6. Inflexible gravity structures should not be installed on sand and gravel or soft clay deposits. Glacial till containing boulders is generally acceptable for gravity structures, but is often difficult for pile driving 7. Suitable measures should be incorporated into the site plan to allow ready access by heavy construction equipment, as needed, to maintain the structure on a long-term basis
Bluff Slope Instability	Regrading of bluff slope utilizing cutback, filling, and/ or terracing	1. Survey the bluff geometry and groundwater conditions and conduct at least three soil borings to identify the stratigraphy, unless adequate borings were previously conducted. Install at least one groundwater observation well, or piezometer, unless a suitable well was previously installed, and monitor seasonal fluctuations in the water table elevation. Conduct soil tests as necessary 2. Conduct a detailed slope stability analysis of the existing bluff slope. Conduct additional stability analyses where the bluff profile, stratigraphy, or groundwater conditions vary substantially 3. Conduct a slope stability analysis of the bluff slope anticipated to exist at the completion of regrading	1. In some locations where damage to property or risk to public safety are not involved, the bluff may be allowed to achieve its equilibrium slope naturally 2. Where sufficient land exists at the top of the bluff to maintain a 50-foot buffer for existing residential buildings, the bluff edge can be cut back to provide a maximum slope angle of 22°, or 1 on 2½, unless a detailed slope stability analysis indicates that a steeper slope angle would be stable. Filling only the lower portion of the slope, cutting back the top of slope and filling the lower portion of the slope, or terraces, may also be utilized in those areas with at least a 50-foot buffer 3. Filling may be utilized only to provide reasonable shore protection and stabilize the bluff slope. Filling should not be used to reclaim land previously lost to shoreline erosion except where important buildings or facilities are located less than 50 feet from the bluff edge. Fill should be placed only on the lower portion of the bluff slope, unless additional fill is required to stabilize the slope or to provide access to the lower portion. Fill may be used to construct a safe roadway, suitable for haul trucks, down the face of the bluff. Where an access roadway must be constructed from the top down, the fill material should be distributed along the face of the bluff to avoid large accumulations of fill material on top of the bluff. The amount of fill used, and the extension of the fill, if any, into Lake Michigan should be the minimum needed to adequately stabilize the bluff slope

Table 51 (continued)

Shoreline Problem	Potentially Applicable Shore Protection Measures	Site Specific Inventories and Analyses	Design Criteria
Bluff Slope Instability (continued)			or to provide a configuration aligned with the adjacent shoreline 4. Where fine-grained material is used as fill, a coarse gravel drainage layer with a suitable outlet should be provided beneath the fill. This drainage system must be maintained on a long-term basis to freely drain the fill layer at all times 5. The slope stability analyses and observed angles of similar fill slopes should be used to specify the stable slope angle for fills composed of mixtures of soil, concrete rubble, rock, and similar materials 6. Fill material may include granular soil, broken concrete, rock, and other clean material. Lumber, metal, asphalt, tires, clay soils, easily corroded material, and litter should not be used for fill 7. The fill material should be deposited at the base of the bluff first, and then filled upward 8. Granular fill material should be covered with a two-foot layer of finer-grained silt or loam soil to allow rapid revegetation of the bluff slope. Impermeable clay soils should not be used to cover fill material. No rocks or broken concrete should be visible on the completed surface 9. Bluff toe protection and surface water and groundwater drainage control should be incorporated into a fill project in accordance with the guidelines provided in this table. Provision should be made for drainage of groundwater where the presence of water-bearing strata or groundwater seepage is observed or monitored
Groundwater Seepage from Face of Bluff Which Threatens Stability of the Bluff Slope	Groundwater drainage systems: trench drains, horizontal drains, or vertical well pumping systems	1. Conduct a thorough site analysis of the hydrogeology of the area. Identify the stratigraphy and the position, inclination, and extent of permeable soil layers. Estimate or measure the shear strength, plasticity, and density of the soil materials. Evaluate water-bearing strata, seepage quantities and patterns, and the accumulation of water in joints and seams. Note artesian groundwater conditions. Measure hydraulic properties and hydrostatic pressures. Install bore-holes, well nests, and piezometers as needed, run pump tests, and determine horizontal and vertical heads and gradients. Note possible leakage from water or sewer mains or from swimming pools	1. The pore spaces in drains and filters should be small enough to prevent soil particles from washing through them, yet large enough to impart sufficient permeability to provide adequate capacities to remove seepage quickly without inducing high seepage forces or excessive hydrostatic pressures. The drainage system should be resistent to clogging 2. Strict adherence should be made to using proper aggregate which provides adequate permeability for drainage 3. The drainage system should be flexible with respect to discharge capacity, and have sufficient capacity for extended wet-weather periods 4. The collected water should be discharged to an adequate surface water drainage system, or to the base of the bluff

Table 51 (continued)

Shoreline Problem	Potentially Applicable Shore Protection Measures	Site Specific Inventories and Analyses	Design Criteria
Groundwater Seepage (continued)		2. Identify seasonal fluctuations in groundwater levels and seepage rates 3. Conduct a detailed slope stability analysis of the existing bluff conditions and the anticipated bluff conditions following groundwater drainage 4. Estimate the magnitude of the drainage system, identifying the area needed to be drained, the probable rate of water inflow, and the drawdown needed to stabilize the bluff slope	5. Groundwater observation wells and/or piezometer monitoring systems should be installed to verify the effectiveness of the drainage systems under seasonal conditions, and to help avoid failures due to unknown groundwater conditions
Excessive Surface Water Runoff and Soil Erosion	Channels, diversions, culverts, energy dissipators, outlet structures, drop structures, slope drains, erosion control measures	1. Review condition of existing gullies and channels. Identify eroded or scoured waterways, areas of sheet and rill erosion, and poorly drained areas 2. Identify sources of surface water runoff and evaluate condition and capacity of outlets. Identify discharge sites for rooftop and driveway runoff 3. Estimate peak flow discharges and flow velocities in critical channels and gullies	1. Stormwater drainage systems should be designed to utilize, to the fullest extent practicable, the natural drainage system, and to provide the most economical installation of gravity flow systems. A primary objective of stormwater management is the maintenance of a good vegetative cover on drainageways and on the bluff slope, and the prevention of soil erosion 2. Stormwater drainage outlets should be located and designed to avoid discharging surface runoff over the top of the bluff, unless suitable conveyance facilities are provided to accommodate the flow without causing soil erosion or reducing the stability of the bluff slope 3. To prevent excessive scouring of open drainage channels, flow velocities during a 10-year recurrence interval design storm should be limited to a maximum of six feet per second for turf-lined channels which, if necessary, contain a concrete cunette; and to a maximum of 10 feet per second for riprap-lined channels. Where practicable, grade control structures should be provided as necessary to reduce the channel gradient and obtain flow velocities within accepted limits. Turf-lined side slopes should be limited to a maximum of one on two 4. The use of measures to enhance infiltration of stormwater which would increase groundwater levels or seepage rates should be avoided 5. Water should not be allowed to accumulate or pond at the top of the bluff, on terraced bluff slopes, or on top of slump blocks 6. Stormwater discharge outlets at the base of the bluff should be designed to prevent scouring or erosion

Table 51 (continued)

Shoreline Problem	Potentially Applicable Shore Protection Measures	Site Specific Inventories and Analyses	Design Criteria
Poorly Vegetated Bluff Slope Which Allows Surface Erosion or Shallow Sliding		1. Prior to undertaking a revegetation project, ensure that the bluff slope is not subject to deep-seated sliding. Evaluate the potential for shallow sliding 2. Conduct a thorough site analysis of climate, soils, slope, and water availability. Identify specific needs of carefully selected plant species with respect control of surface water and groundwater, slope shaping, and soil management 3. Survey the existing vegetation to identify what vegetation currently exists and effectively controls erosion on the slope 4. Identify aesthetic and functional preferences	1. Where revegetation is indicated, the bluff may be allowed to re-establish a vegetative cover naturally if the threat of massive shallow sliding is minimal 2. Where practicable, native plant species, including wild flowers, should be used to revegetate bluff slopes. The vegetation should be planted to give the slope a natural and undisturbed appearance and character 3. Some shaping and terracing of the slope may be needed to provide a suitable slope angle and eliminate drainage problems. Groundwater and surface water drainge systems should be installed, as needed, prior to planting 4. Initial grass or pioneer species should be used to establish a good ground cover first, then trees and shrubs may be planted at three- to six-foot spacings. Plantings should be conducted in spring or fall 5. Maintenance-free deep-rooting plant species which are suitable for the physical site conditions should be selected 6. Mulch should be applied after seeding. Drilling or hydroseeding may be necessary to successfully establish herbaceous plants on steep slopes 7. Watering and fertilization after planting should be limited to the minimum needed for successful establishment of the vegetation 8. All revegetation projects should have provisions for follow-up inspection, care, and maintenance

Source: SEWRPC.

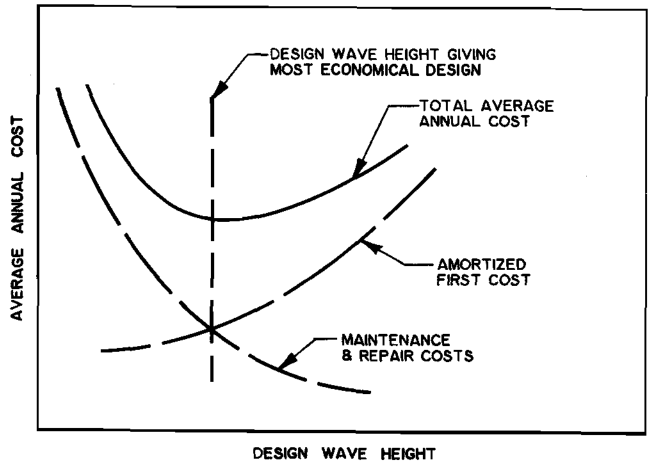
prevent severe damage by the 50-year recurrence interval wave height, which is about 24.8 feet in deep water. As noted in Chapter III, because relatively high water levels may be expected to persist for extended periods of time, generally several years, there is a slight but realistic potential for a severe storm with a recurrence interval of 20 to 50 years to occur concurrently with a high water level.

Historically in Milwaukee County, and on the Great Lakes in general, the design of a structure is often not determined by a coastal engineering analysis of the height needed to protect against a certain recurrence interval water level and wave height. The most economical design of a structure—including construction, maintenance, and repair costs, as illustrated in Figure 79—may or may not coincide with the design specifications needed to protect against a high water level combined with a 20-year or 50-year storm wave. Furthermore, there is a tendency for many public and private lakefront property owners to provide the minimum, lowest cost protection, often using whatever materials are readily available at the time. Such low-cost measures frequently provide only partial short-term benefits.

As shown in Chapter III, the use of a 100-year water level and a 20-year storm wave to design shore protection structures would provide a greater level of protection than is currently provided by most structures. The U. S. Army Corps of Engineers normally selects a structure

Figure 79

RELATIONSHIP OF DESIGN WAVE HEIGHT TO AVERAGE ANNUAL COST OF SHORE PROTECTION STRUCTURES



Source: U. S. Army Corps of Engineers, Shore Protection Manual, Volume II, 1984.

height based upon a 20-year water level and a 10-year storm wave on the Great Lakes.[1] However, most structures designed by the Corps—primarily breakwaters and revetments—are constructed to withstand a certain degree of overtopping. The Corps will also use more stringent design criteria when the facility being protected is particularly important and vulnerable. For structures located in deeper water which must protect against nonbreaking waves, the Corps recommends that the average of the highest 1 to 10 percent of all waves be used, which are 27 to 67 percent higher than the significant wave height—the average of the highest 33 percent of all waves.[2] The significant wave height, conventionally used in coastal engineering design and analysis, is based upon the understanding of waves as a random process. Although the significant wave is expressed in terms of a single wave height and wave period, it does not represent waves of constant height and period. Recent advances in wave analysis have focused on the importance of wave irregularity and its relevance to engineering applications.

As presented in Chapter III, some large beaches, along with certain major structures designed since the 1986 record high water levels, such as the Klode Park breakwater/beach, the McKinley Park armored headland/pocket beach system, and the new Jones Island wastewater treatment plant bulkhead, appear to provide a level of protection consistent with that recommended herein. The only other existing structure that would not have a moderate to high potential for wave overtopping damage under the recommended design water level and storm wave condition is the Milwaukee County War Memorial Center bulkhead, located within the outer harbor.

CONCEPTUAL SHORE PROTECTION MEASURES

The analysis of the need for, and the selection of, potential shore protection measures should first identify the causes of shoreline erosion and bluff recession. The probable causes of these problems in each of the 100 bluff analysis sections were identified in Chapter II. Measures suitable for the protection of the shoreline and for the stabilization of the bluff slopes within each of the bluff analysis sections were then identified in Chapter III. The indicated measures included protection of the toe of the bluff against wave and ice action; regrading the bluff slope to a stable angle; the installation of a groundwater drainage system to lower the elevation of the groundwater; the construction of surface water runoff control measures; and the revegetation of the bluff slopes. A description of alternative structural measures, along with conceptual designs and estimated costs, is presented for those protection measures that should be considered for installation within the study area. The alternative structure designs and associated costs presented in this chapter represent typical structural designs for Lake Michigan shoreline areas. All costs are presented in 1988 dollars.

Shoreline Protection

Shoreline areas exhibiting erosion were identified in Chapter III of this report and include approximately 78,770 feet, or 50 percent, of the county shoreline. Alternative shoreline protec-

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[1] *David Rollig, Personal Communication, U. S. Army Corps of Engineers, Chicago, Illinois, December 22, 1988.*

[2] *U. S. Army Corps of Engineers, Shore Protection Manual, Volume II, 1984.*

tion measures evaluated for the Milwaukee County study area included both onshore and offshore structures. Onshore structures include revetments, bulkheads, and groins; offshore structures include breakwaters, barrier reefs, and peninsulas and islands. A general comparison of selected characteristics of shoreline protection measures is provided in Table 52. The table presents certain requirements for successful application of the structures, lists the advantages and disadvantages of each type of structure, and notes the compatibility of the structure with alternative shoreline uses. These data serve as the basis for determining which structures should be evaluated for individual bluff analysis sections. There is no single type of structure that should be used in all cases; consideration of the specific characteristics of each section to be protected is essential in the planning and design of bluff toe protection measures. In addition, maintenance, modification, or reconstruction of existing shore protection structures are all viable alternatives for several bluff analysis sections.

The following sections describe common structural shoreline protection measures currently used in the Great Lakes, and provides guidelines for the application of these measures. The guidelines and general design criteria described relate only to the preliminary design and sizing of the structures; detailed design criteria for structures are set forth in engineering reports such as the U. S. Army Corps of Engineers Shore Protection Manual (1984).

Revetment: Various types of revetments are commonly used to provide shoreline protection within Milwaukee County. Revetments contain a flattened slope at the shoreline armored with material resistant to wave erosion and ice damage, and usually underlaid by filter cloth and gravel or cobble bedstone. The armor layer may consist of natural rock, quarry stone, concrete rubble, or precast or cast-in-place concrete materials. The armor layer resists the wave and ice action and provides structural stability. The gravel bedstone and filter cloth support the armor layer against settlement, provide drainage through the revetment, and prevent underlying soil from being washed through the armor layers by waves or groundwater seepage.

Described below are three alternative revetment designs—a riprap revetment, a grout-filled bag revetment, and an interlocking concrete block revetment.

Riprap: As shown in Figure 80, a riprap revetment utilizes rock or quarry stone as its armor layer. To be durable, the armor stone should be free of laminations and cracks. The stone should be angular, with the greatest dimension no greater than three times the least dimension. Slab-shaped stones are not desirable for an armor layer. Riprap revetments providing three levels of protection are illustrated in Figure 80. A light revetment may require two to three tons of stone per lineal foot of shoreline; a medium revetment, three to five tons of stone per foot; and a heavy revetment, five to 10 tons of stone per foot. The size of the armor stones needed to provide adequate protection is dependent on the wave height, the specific gravity and quality of the stone, the slope of the structure, and the degree of interlocking of the individual stones. An alternative riprap revetment design, known as a berm revetment, utilizes a thick layer of variable-size armor stone with an average weight typically less than one-half the weight of the stone required by conventional design methods. Wave action shapes the thick armor layer into a berm which dissipates the wave energy.

The advantages of a riprap revetment are that it is relatively easy to construct and maintain; it is flexible, and can therefore withstand some movement or displacement without total failure; and it provides a relatively natural appearance to the shoreline.

The primary disadvantages of a riprap revetment are that the structure generally makes use of the immediate shoreline area for recreational activities difficult, and access to the water may be precluded. A riprap revetment is generally poorly suited to use for swimming, boating, and fishing, although recreational facilities such as walkways and piers may be incorporated into the design. Riprap revetments, particularly steep structures, reflect wave energy, although less than would most bulkheads. This reflected energy may scour offshore lakebed material, especially immediately in front of the structure. A steeper offshore slope would allow larger waves to reach the shoreline.

The life of a riprap revetment depends on the durability of the rock used for construction and on the degree of maintenance performed. Riprap revetments may be affected by settling and

Table 52

COMPARISON OF SHORELINE PROTECTION MEASURES

Shoreline Protection Measure	Type	Advantages	Disadvantages	Compatibility with Alternative Shoreline Uses: Walking	Swimming	Fishing	Boating	Aesthetics	Capital Cost ($/lineal foot of shoreline)[a]	Annual Maintenance Cost ($/lineal foot of shoreline)[a]
Revetment	Riprap	Easy to construct and maintain Flexible, durable	Limits access to shoreline Heavy equipment required for installation May reflect wave energy	Fair	Poor	Fair	Poor	Good	200-700	5-20
	Grout-filled bags	Constructed where access limited Adaptable to add-on construction	Limits access to shoreline Relatively inflexible Not as durable as quarry stone	Fair	Poor	Poor	Poor	Fair	200-250	10-20
	Manufactured concrete systems	Provides uniform appearance Adaptable to add-on construction Concrete units interlock for stability	Relatively inflexible Generally not as durable as quarry stone Heavy equipment required for installation	Fair	Fair	Fair	Fair	Fair	150-450	15-20
Bulkhead	Concrete cantilevered	Uniform appearance Infrequent maintenance requirements Durable	Loss of beach may be intensified Relatively inflexible Maintenance, when required, is difficult and costly Reflects wave energy	Good	Fair	Good	Fair	Fair	400	10-15
	Steel sheet piling	Uniform appearance Infrequent maintenance requirements Durable	Loss of beach may be intensified Relatively inflexible Maintenance, when required, is difficult and expensive Special pile-driving equipment required to install Reflects wave energy	Good	Fair	Good	Fair	Fair	650	5-10
	Concrete-stepped	Provides uniform appearance and usable shoreline Infrequent maintenance requirements Durable	Relatively inflexible Loss of beach may be intensified Maintenance, when required, is difficult and costly Reflects wave energy	Good	Fair	Good	Good	Fair	1,300	5-10
Onshore or Near-shore Beach Systems	Quarry stone groins with nourished gravel beach	Provides usable shoreline Flexible Absorbs wave energy Feeds littoral transport system Adjusts to variable water levels	The beach would need to be periodically re-nourished Trapping sand supply in littoral drift may reduce the available sand for down-current beach areas	Good	Good	Good	Good	Good	300-1,000	10-40
	Armored headland-pocket beach system	Flexible, durable Provides usable shoreline Pocket beaches absorb wave energy Adjusts to variable water levels	May require large amount of fill to construct Beach would need to be periodically re-nourished to maintain sand or fine gravel Trapping sand supply in littoral drift may reduce the available sand for down-current beach areas Armored headlands may reflect wave energy	Good	Good	Good	Good	Good	600-1,200	10-40
	Near-shore reefs with nourished gravel beaches	Flexible Provides uniform appearance and continuous usable shoreline Feeds littoral transport system Adjusts to variable water levels	The beach would need to be periodically re-nourished Trapping sand supply in littoral drift may reduce the available sand for down-current beach areas Reefs are subject to large wave attack and thus more susceptible to damage than are onshore structures Limits view of horizon	Good	Good	Good	Fair	Fair	450-1,100	15-50
	Perched cobble beach without covering of sand or gravel	Cobbles absorb considerable wave energy without causing scouring from wave reflection Adjusts to variable water levels	Limits use of shoreline	Poor	Poor	Poor	Poor	Fair	350-400	20

Table 52 (continued)

Shoreline Protection Measure	Type	Advantages	Disadvantages	Compatibility with Alternative Shoreline Uses: Walking	Swimming	Fishing	Boating	Aesthetics	Capital Cost ($/lineal foot of shoreline)[a]	Annual Maintenance Cost ($/lineal foot of shoreline)[a]
Beach Systems (continued)	Near-shore pervious concrete sill	Reduces wave attack by tripping and slowing waves Enhances sediment accretion near shore	Requires some beach, gentle offshore slope, and low-wave-energy environment Interferes with small boat navigation near shore	Good	Good	Fair	Poor	Good	350-750	10
	Manufactured concrete systems nourished with sand or gravel	Provides partially usable shoreline	Limits access to water Blocks may settle or move out of alignment	Good	Fair	Fair	Fair	Fair	200-300	15-50
Offshore Breakwater with Nourished Sand Beach	Rubble mound	Provides substantial protection Use of shoreline not restricted Provides large sand beach	Heavy equipment mounted on barges may be required for installation and maintenance Trapping sand supply in littoral drift may reduce the available sand for down-current beaches	Good	Good	Good	Good	Good	1,000-2,000	20-50
Offshore Island or Peninsula	--	Additional land created for recreational use Provides substantial protection Use of shoreline not restricted	Large amount of fill material required to construct Degree of protection needed on lakeward side of island Heavy equipment mounted on barges may be required for installation and maintenance	Good	Good	Good	Good	Good	800-1,500	20-40

[a]The costs shown are estimates of the likely costs entailed where these measures could effectively be used. Because, at any one site, the different structures would not all offer the same level of protection, and because the structure unit costs are site specific, a direct comparison of the costs for the different structures may not be appropriate.

Source: SEWRPC.

displacement. If armor stones are moved by wave action, the entire structure may be weakened if not maintained. Riprap revetments placed on sand without proper filter material and those utilizing undersized armor stone are particularly prone to failure.

The cost of riprap revetments is influenced by design water level and depth, wave environment, accessibility, material cost, and other site-specific factors. In general, the capital costs may range from \$200 to \$700 per lineal foot of shoreline. Average annual maintenance costs for a riprap revetment range from \$5.00 to \$20 per lineal foot.

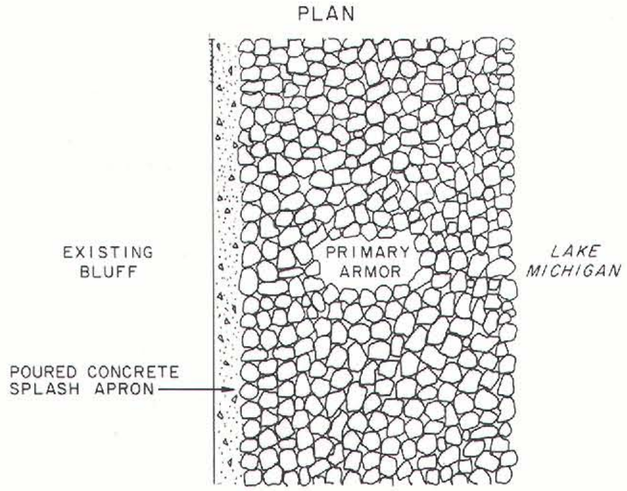
Grout-Filled Bags: Large grout-filled bags have been placed at the toe of bluffs to form revetments within the study area. These bags are typically six feet deep by two and one-half feet high, and up to 20 feet long. The 20-foot-long bags weigh about 14 tons each. As shown in Figure 81, the bags should be placed parallel to the shore, with reinforcing bars installed both vertically and horizontally to hold the bags together. A filter cloth and a gravel bed should be placed beneath the bags to provide drainage and prevent the underlying soil from being washed away by waves or groundwater seepage. The bags are the most appropriate for low- to moderate-wave-energy environments.

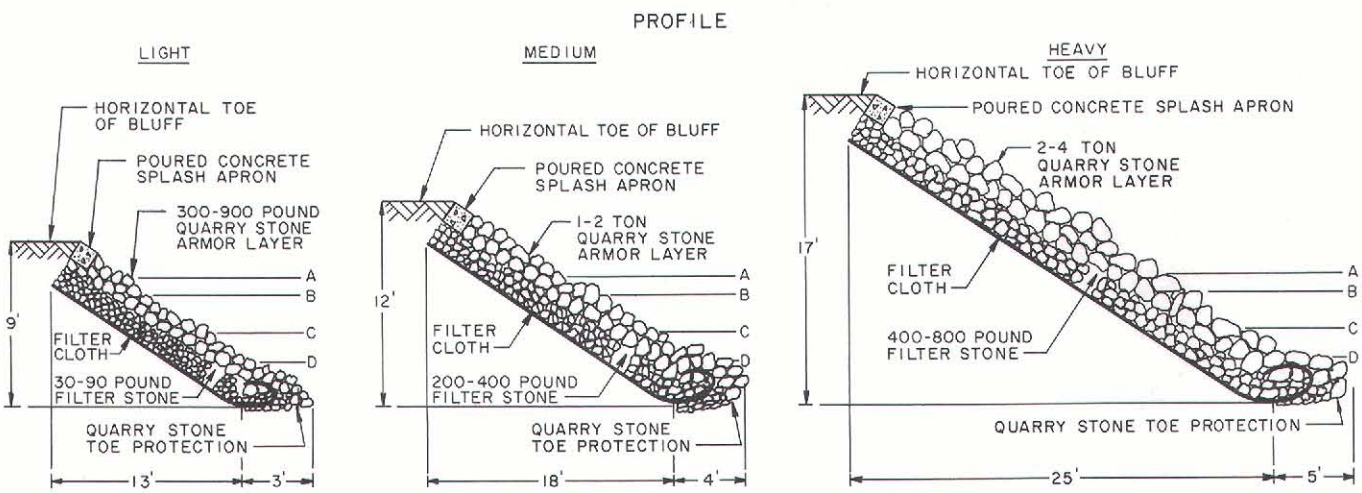
The primary advantage of a grout-filled bag revetment is that it can be constructed where access is limited. A grout pump which can be operated from the top of a bluff is used to fill the bags. In addition, the structure is readily adaptable to add-on construction if additional structure height is necessary. The bags are rounded, providing limited access to the shoreline.

The primary disadvantage of a grout-filled bag revetment is that it is relatively inflexible, and is therefore more vulnerable to wave forces than is an equivalent riprap revetment. Because of this relative inflexibility, it is particularly important to provide a sound foundation for the bags. The bags may not be as durable as quarry

Figure 80

TYPICAL RIPRAP REVETMENT





LEGEND

LAKE MICHIGAN WATER LEVELS

A 100-YEAR RECURRENCE INTERVAL MAXIMUM INSTANTANEOUS WATER LEVEL 584.3 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

B 10-YEAR RECURRENCE INTERVAL MAXIMUM INSTANTANEOUS WATER LEVEL 582.8 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

C 1900 TO 1987 ANNUAL MEAN WATER LEVEL 579.6 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

D LOW WATER DATUM 578.1 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

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Source: SEWRPC.

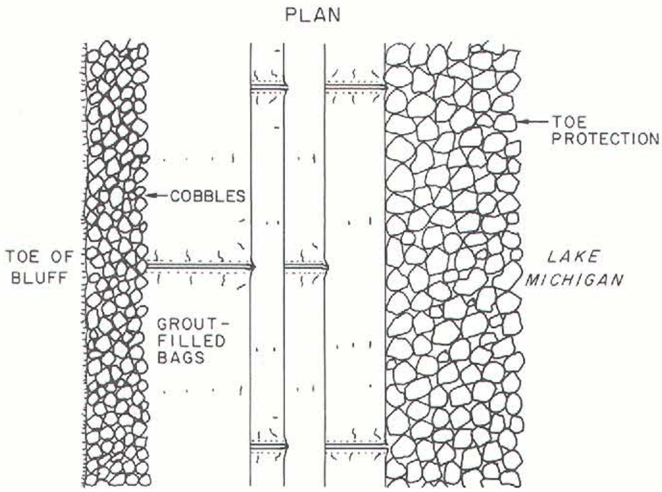
stone in some applications, and may be susceptible to bottom scouring. Since concrete is not as dense as natural rock, a larger volume of concrete is required to provide the same weight and therefore protection as natural rock.

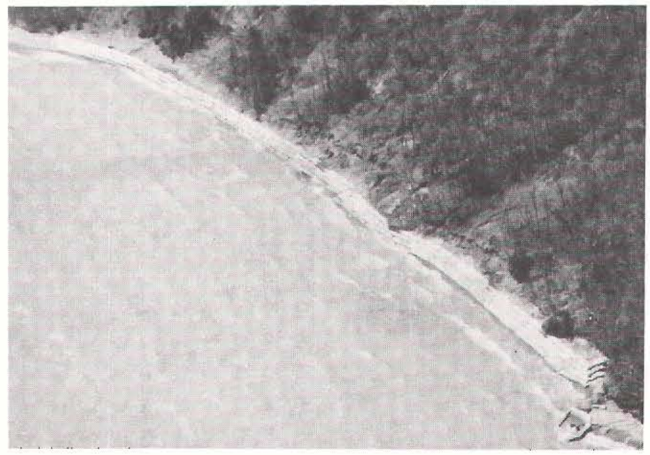
The capital cost of a grout-filled bag revetment is influenced by design water level and depth, wave environment, material cost, and other site-specific factors, but in general ranges from $200 to $250 per lineal foot of shoreline. Average annual maintenance costs may range from $10 to $20 per lineal foot.

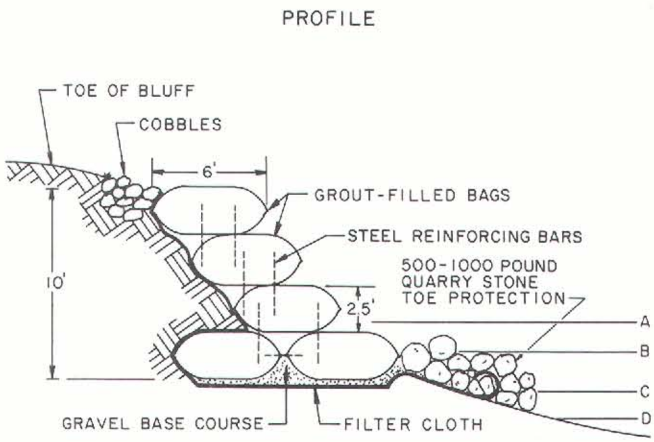
<u>Concrete Structures</u>: Several different types of manufactured concrete structures are commer-

Figure 81

TYPICAL GROUT-FILLED BAG REVETMENT







LEGEND

LAKE MICHIGAN WATER LEVELS

A 100-YEAR RECURRENCE INTERVAL MAXIMUM INSTANTANEOUS WATER LEVEL 584.3 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

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Source: U. S. Army Corps of Engineers and SEWRPC.

cially available. The interlocking concrete blocks or slabs fit together to form a revetment. These blocks or slabs typically weigh up to 1,000 pounds each. As shown in Figure 82, the blocks or slabs are usually perforated with holes or slots to neutralize pressure from changing water levels and to absorb energy from wave action. A filter cloth should be placed beneath the concrete units to prevent the underlying soil from being washed away by waves or groundwater seepage, and stone should be placed at the toe of the revetment to prevent scouring.

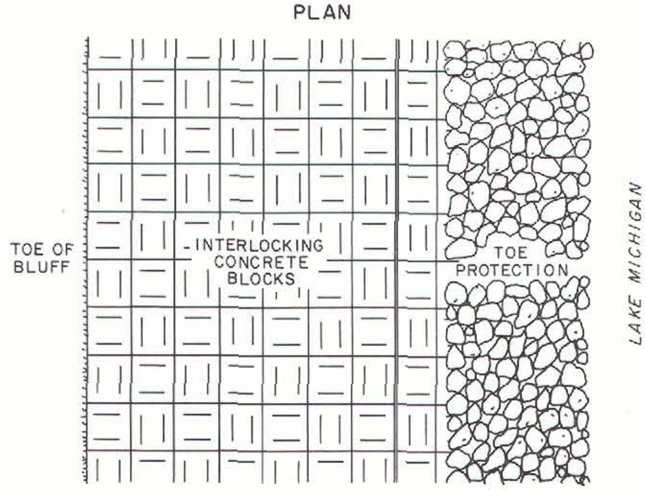
The advantage of an interlocking concrete system is that it provides a uniform appearance and a usable shoreline which may be suitable for some recreational activities. In addition, the system is readily adaptable to add-on construction.

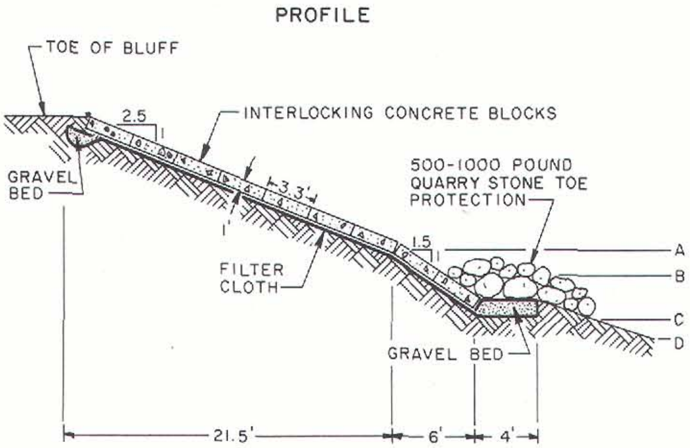
A major disadvantage of interlocking concrete blocks in general is that the failure of one block can lead to rapid failure of adjacent blocks. In some applications, the blocks may not be as durable as a riprap revetment. Failure of the subgrade will quickly result in excessive movement of the blocks. The interlocking concrete block systems are most appropriate for relatively low-wave-energy environments.

The capital cost of an interlocking concrete block system, depending on site characteristics, approximates \$150 to \$450 per lineal foot of

Figure 82

TYPICAL INTERLOCKING CONCRETE BLOCK REVETMENT





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Source: Spancrete, Inc., and SEWRPC.

shoreline. The average annual operation and maintenance cost would approximate \$15 to \$20 per lineal foot.

Larger concrete units, usually cast in place, which do not specifically interlock are also commercially available. These units can be placed along the shoreline to create a revetment. The units, an example of which is shown in Figure 83, vary in size, often ranging from two to three tons each. Heavy construction equipment is usually required to install the structures. These structures are most appropriate for relatively low-wave-energy environments. The capital cost of a revetment constructed of larger cast-in-place concrete units ranges from \$100 to \$200 per lineal foot of shoreline. The average annual operation and maintenance cost would range from \$10 to \$20 per lineal foot.

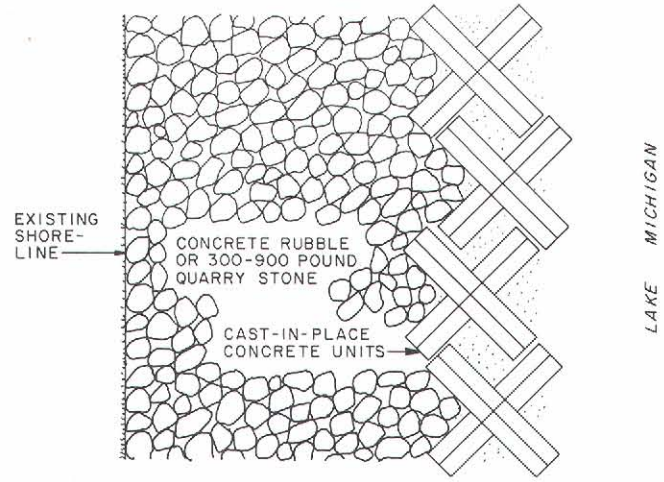
Bulkheads: Bulkheads are vertical retaining walls constructed of concrete, steel sheet piling, or timber which support the base of the bluff and provide protection against wave and ice action. Prior to the 1980's, bulkheads were the most commonly used shore protection structure in Milwaukee County, with most being constructed of concrete.

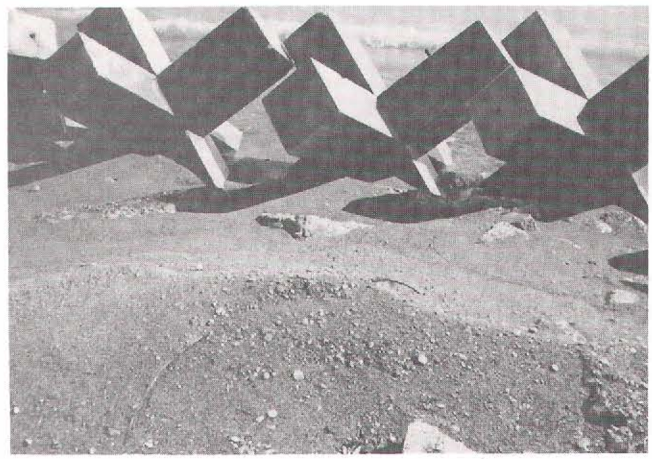
One advantage of bulkhead is that they can be constructed to a height of 10 to 15 feet above the existing beach and can be placed lakeward of

Figure 83

TYPICAL CAST-IN-PLACE CONCRETE UNITS

PLAN





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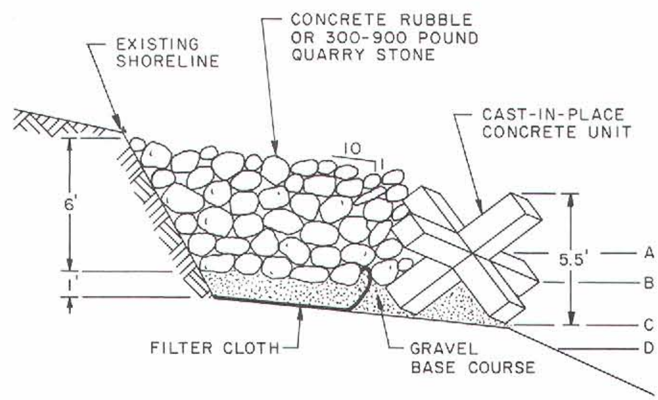
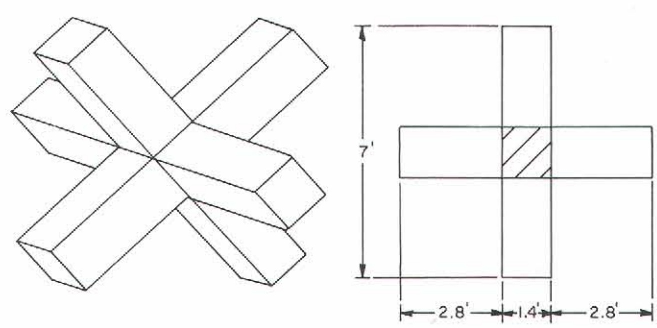


TYPICAL CONCRETE UNIT



LEGEND

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Source: Dan Libecki Grading and SEWRPC.

the existing bluff toe. Fill can be placed behind the bulkhead, and the bluff slope can be regraded from the top of the bulkhead rather than from the existing bluff toe. This effectively reduces the required bluff top regrading distance to achieve a stable bluff slope, as shown in Figure 84. Thus, the necessary cutting back of the top of the bluff to form a stable slope could be significantly reduced if a bulkhead is constructed. Another advantage of a bulkhead is that it provides a uniform appearance and may be suited for recreational facilities such as walkways, piers, and boat slips which may enhance the use of the shoreline.

The disadvantages of bulkheads are that they are inflexible, and that maintenance, when required, is difficult and costly. Bulkheads are less suitable during periods of widely fluctuating water levels than are most other protection structures. A high bulkhead may also limit direct access to the lake water, and uses such as swimming may be precluded. A bulkhead also deflects the wave energy both upward and downward, often leading to overtopping and severe scouring at the base of the structure. It is therefore likely that existing beach areas in front of the bulkhead would be eroded by the wave action.

Described below are three alternative bulkhead designs—a concrete cantilevered bulkhead, a steel sheet piling bulkhead, and a concrete-stepped bulkhead.

Concrete Cantilevered Bulkhead: A cantilevered, cast-in-place, reinforced concrete bulkhead, as illustrated in Figure 85, consists of a concrete base with a cantilevered wall. The wall is constructed with weep holes and backfilled with coarse granular material to prevent hydrostatic pressure buildup and frost heave. Riprap toe protection should be provided. A cantilevered bulkhead derives its support solely from ground penetration, so sufficient embedment is required.

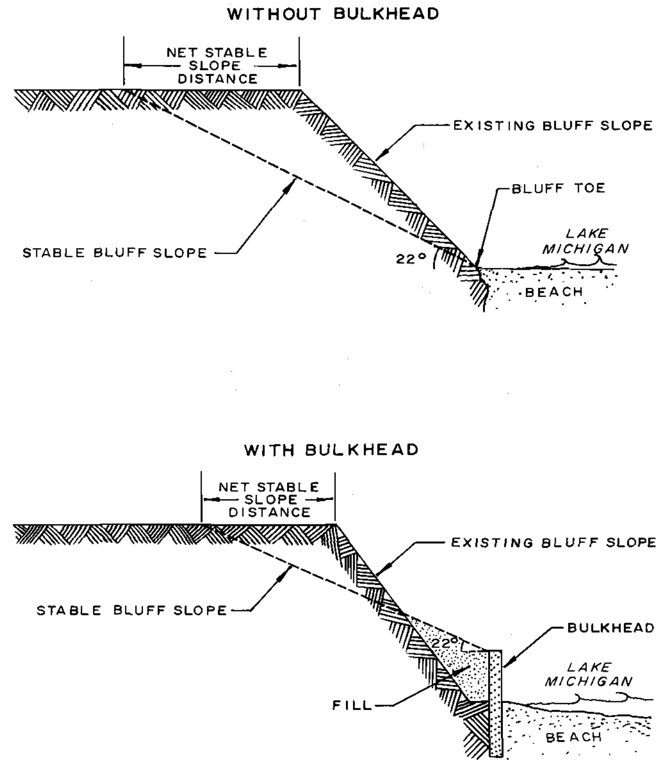
Construction of a concrete cantilevered bulkhead along the Lake Michigan shoreline of Milwaukee County would entail a capital cost of approximately $400 per lineal foot of shoreline. Average annual maintenance costs would range from $10 to $15 per lineal foot.

Steel Sheet Piling Bulkhead: A steel sheet piling bulkhead, as shown in Figure 86, is deeply embedded beneath the beach surface, and includes the construction of piling with adequate walers to provide rigidity. As an alternative design, the sheet piling can be anchored with tie backs, as also shown in Figure 86. Riprap toe protection and weep holes for drainage should be provided. The structure should be backfilled with coarse granular material. Special pile-driving equipment is required to install the structure.

Construction of a steel sheet piling bulkhead along the Lake Michigan shoreline of Milwaukee County would require a capital cost of approximately $650 per lineal foot of shoreline. Average annual maintenance costs would range from $5.00 to $10 per lineal foot.

Figure 84

EFFECT OF A BULKHEAD ON THE BLUFF TOP CUTBACK DISTANCE REQUIRED TO ACHIEVE A STABLE BLUFF SLOPE



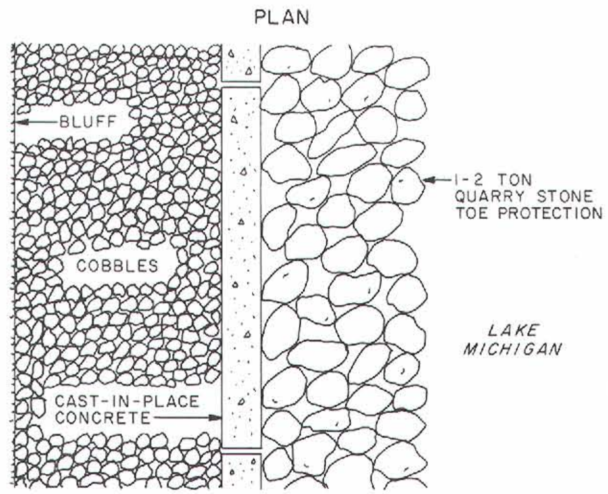
Source: SEWRPC.

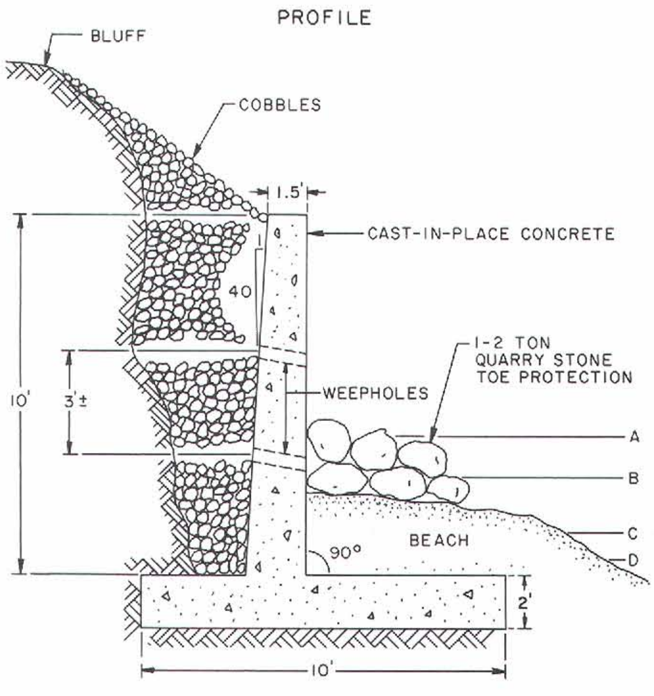
Concrete-Stepped Bulkhead: A third type of bulkhead is a cast-in-place, concrete-stepped bulkhead, as shown in Figure 87. The bulkhead, cast as a massive, gravity-held structure to resist overturning by wave action or soil pressures, should include a splash apron along its crest to prevent erosion caused by wave action overtopping the structure. As shown in the figure, the face of the bulkhead is stepped toward the lake. The concrete-stepped bulkhead does not require deep embedment or piles beneath the beach, and the steps provide access to the lake water. The structure is, therefore, more suitable for uses such as swimming and wading than most revetments or other types of bulkheads.

Construction of a concrete-stepped bulkhead along the Lake Michigan shoreline of Milwaukee County would entail a capital cost of approximately $1,300 per lineal foot of shoreline. Average annual maintenance costs would range from $5.00 to $10 per lineal foot.

Figure 85

TYPICAL CONCRETE CANTILEVERED BULKHEAD





LEGEND

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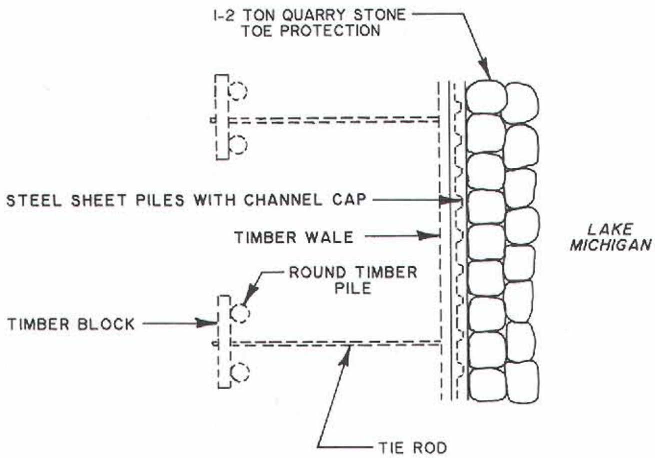
Source: SEWRPC.

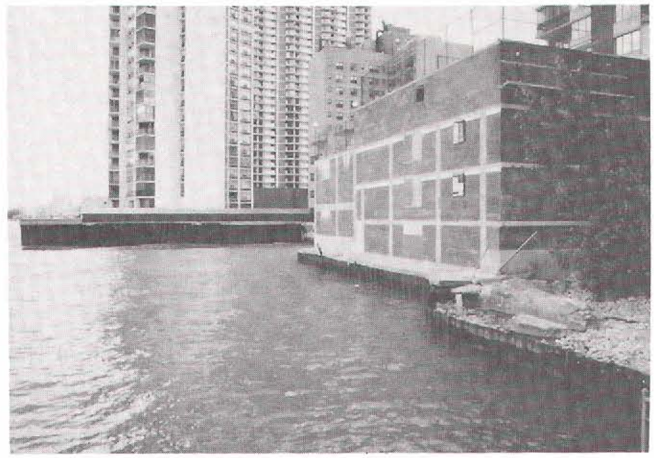
<u>Onshore or Near-shore Beach Systems</u>: There are several onshore or near-shore protection structures which may support a beach and in turn protect the bluff toe against wave action, while providing opportunities for the pursuit of recreational activities such as walking, swimming, and boating. Beach systems require structures that are built out from the shoreline, or placed in the lake in shallow water. The structures are intended to prevent wave action from eroding a natural or artificially nourished beach. Because the supply of sand in the littoral drift is limited, especially in the northern portion of the County and in the Milwaukee Harbor, it is often necessary to artificially nourish the beaches with coarse-grained material, usually coarse sand or gravel. The beaches need to be occasionally renourished. Generally, the coarser

Figure 86

TYPICAL STEEL SHEET PILING BULKHEAD

PLAN





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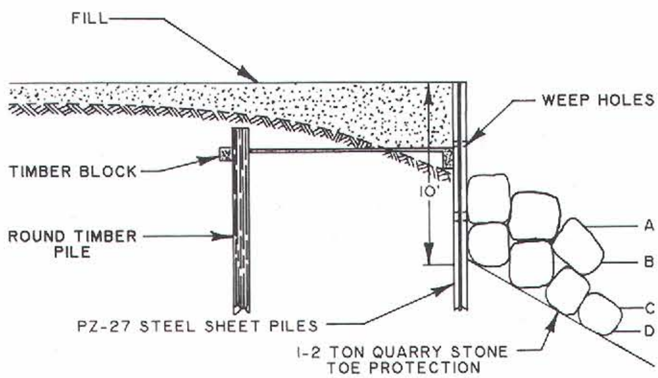

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Source: U. S. Army Corps of Engineers and SEWRPC.

the beach material, the steeper the beach that would form. Table 53 lists the beach slopes expected to form on different sized beach material. As shown in the table, while sand beaches would generally have a slope of less than 5 degrees, gravel beaches may frequently have slopes approximating 10 degrees.

The major advantage of an onshore beach system is that the extended beach provided to protect the bluff toe against wave action also allows access to, and use of, the shoreline for walking, swimming, and fishing.

The disadvantages of beach systems include the potential for increasing downdrift erosion if the littoral drift is obstructed to form the beach, and the considerable maintenance that may be required to keep the extended beach intact. Also, insufficient bluff toe protection may be provided by the beach during large storm events, especially during high lake levels.

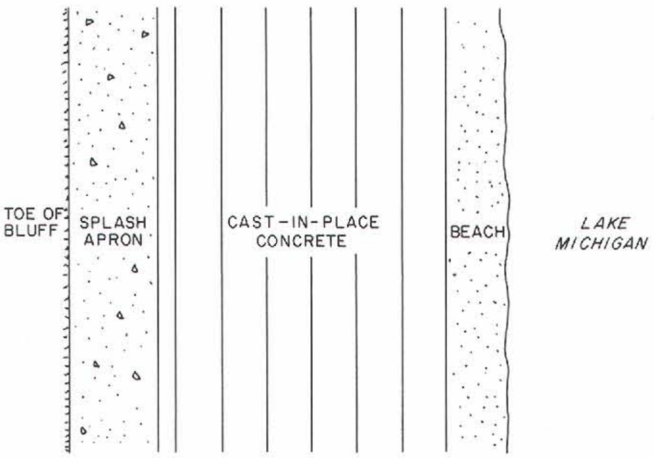
Described below are several types of onshore and near-shore beach system designs: groins, an armored headland-pocket beach system, near-shore reefs, and manufactured concrete systems.

Groins: Groins are the most common type of structure used to create beaches. Groins can be constructed of quarry stone, concrete, steel sheet pile, or timber. Groins extend out into the lake perpendicular to the shoreline. They are intended to hold beach material and to partially

Figure 87

TYPICAL CONCRETE-STEPPED BULKHEAD

PLAN



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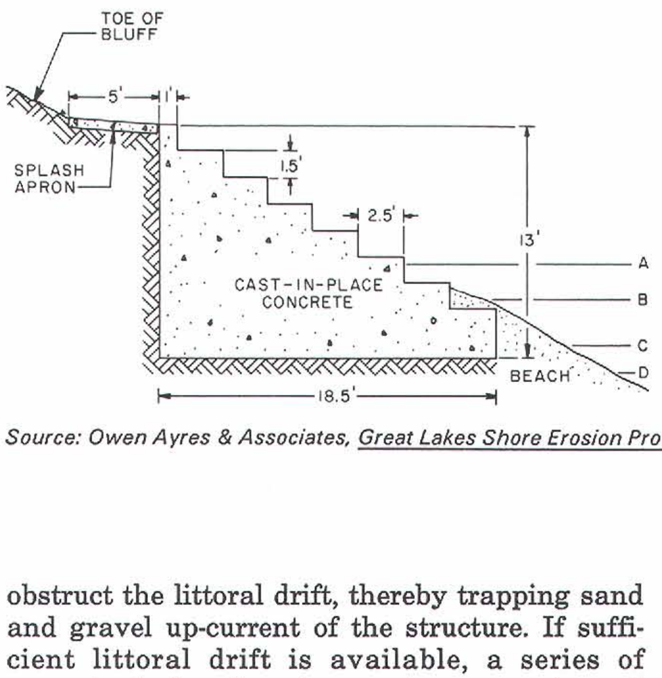

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Source: Owen Ayres & Associates, Great Lakes Shore Erosion Protection, Structural Design Examples, 1978: and SEWRPC.

obstruct the littoral drift, thereby trapping sand and gravel up-current of the structure. If sufficient littoral drift is available, a series of properly designed groins can trap enough sand and gravel to build a beach which absorbs wave energy and protects the bluff toe. Because the supply of sand and gravel in the littoral drift along most of the Milwaukee County shoreline appears to be quite limited, it is unlikely that new groin systems would trap enough material to form an adequate beach. Rather, the groins would be designed to hold an artificially nourished beach composed of coarse sand and gravel. The groins themselves do not appreciably reduce the wave energy striking the shore, and sediment moving along shore may be forced into deeper water to move around the structure ends. Thus, groins may displace near-shore sandbar systems lakeward.

Figures 88 and 89 show examples of quarry stone and sheet pile groin systems designed to maintain a beach composed of gravel. The onshore portion of the groins would be constructed with a top elevation about seven feet above the existing beach level to retain the beach fill. The orientation and spacing of a groin system is highly dependent on the site-specific

Table 53

ESTIMATED BEACH SLOPES THAT WOULD FORM ON VARIOUS BEACH FILL MATERIALS

Breaking Wave Height	Very Coarse Sand 0.06 inch (1.5mm)	Very Fine Gravel 0.12 inch (3mm)	Fine Gravel 0.24 inch (6mm)	Medium Gravel 0.48 inch (12mm)	Coarse Gravel 0.96 inch (24mm)
3 feet (0.9m)	4°	6°	8°	12°	16°
6 feet (1.8m)	3°	4°	6°	8°	12°
9 feet (2.7m)	2°	3°	5°	7°	10°
12 feet (3.7m)	2°	3°	4°	6°	8°

NOTE: Calculated using the following formula from J. W. Kamphuis, M. H. Davies, R. B. Nairn, and O. J. Sayao: "Calculation of Littoral Sand Transport Rates," Coastal Engineering, Vol. 10, pp. 1-21, 1986:

$$m = \tan^{-1}\left[1.8\left(\sqrt{\frac{H}{D}}\right)^{-1}\right]$$

where: m = beach slope (degrees)
H = breaking wave height (m)
D = beach particle diameter (m)

Source: SEWRPC.

details of the project location, but spacing between groins should generally be equal to about one and one-half to twice the groin length. The groins should be of sufficient height to prevent excessive overtopping. Periodic replenishment of the beach material will usually be required.

The height, orientation, and shape of groins may be modified, depending on the site characteristics, to either maximize beach containment or minimize trapping of the littoral drift. For example, the lakeward end of groins may be sloped downward to reduce downdrift impacts. The capital cost of a groin system ranges from $200 to $500 per lineal foot of shoreline, with an additional cost ranging up to $115 per lineal foot of shoreline to artificially nourish the beach by shore, or an additional cost of up to $500 per lineal foot of shoreline to artificially nourish the beach by barge. Annual maintenance costs depend upon the need for additional fill material, and range from $10 to $40 per lineal foot.

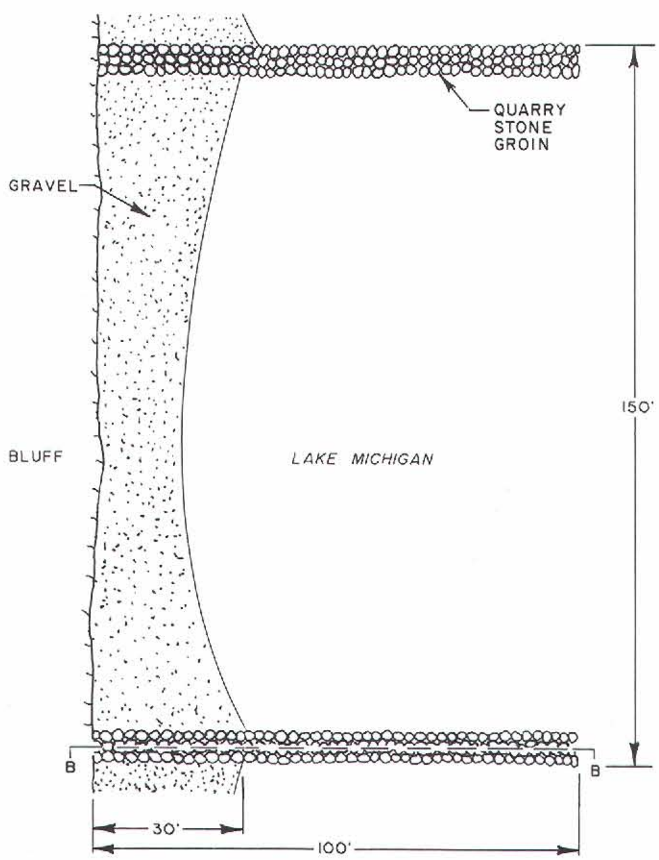
Armored Headland-Pocket Beach System: An armored headland and pocket beach system acts similarly to a groin system in that the headland is connected to and extends out from the shoreline. Coarse beach material is trapped or held within the pocket areas of the structure, as shown in Figure 90. The headlands are usually protected with an armor stone revetment. A headland beach system may create a relatively large amount of land for recreational use. Design considerations for the headlands are similar to those for a revetment.

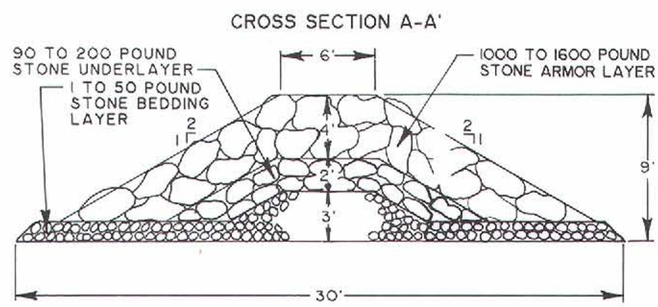
The capital cost of a headland and pocket beach system would range from $600 to $1,200 per lineal foot of shoreline. Average annual maintenance costs would range from $10 to $40 per lineal foot.

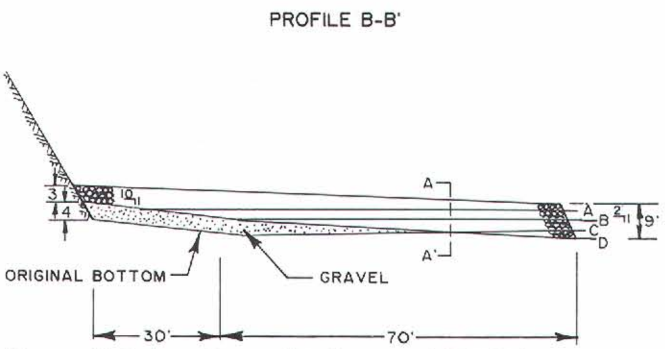
Near-shore Reefs: Near-shore reefs are constructed of stone and placed generally parallel to the shoreline in a water depth of four to five feet. Such reefs are generally located less than 100

Figure 88

TYPICAL QUARRY STONE GROIN SYSTEM WITH ARTIFICIALLY NOURISHED GRAVEL BEACH







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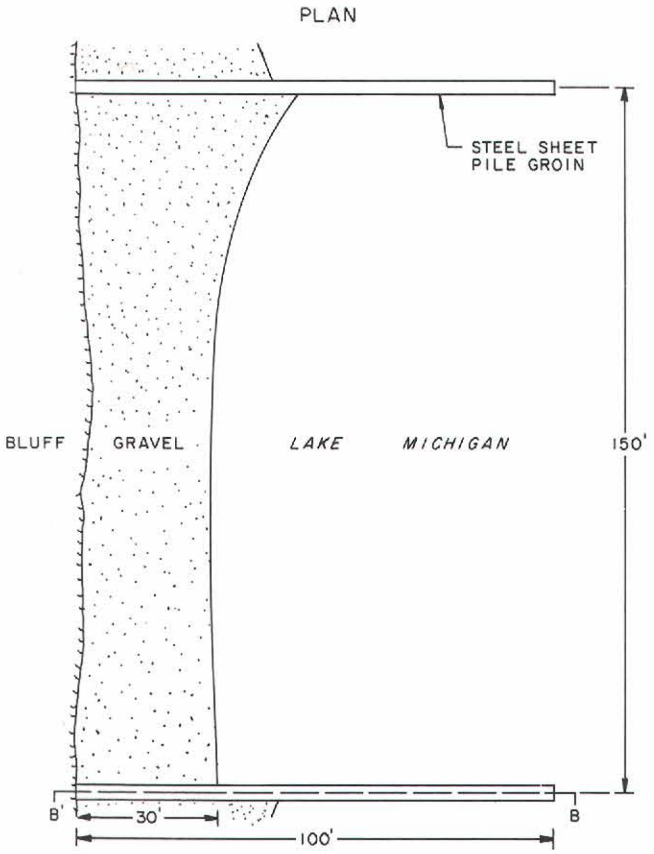
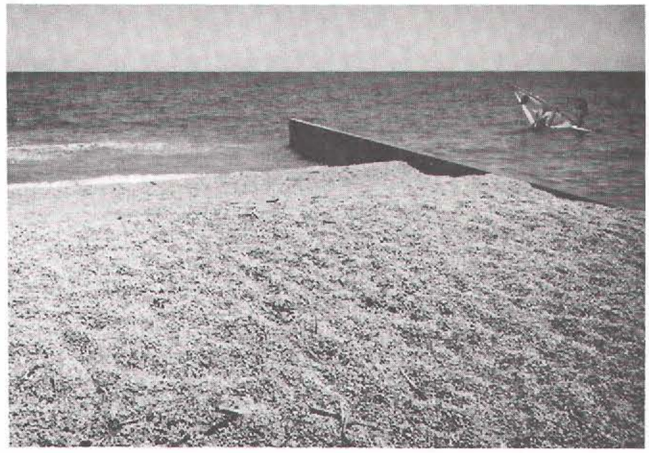
Source: U. S. Army Corps of Engineers and SEWRPC.

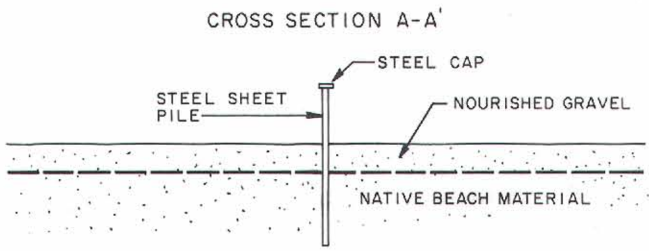
feet from the shoreline, as shown in Figure 91. In some applications, the reefs may curve into the shoreline, or the system may be supplemented by groins. In a typical installation, a filter cloth would be placed on the lake bottom, covered with 5- to 90-pound stone, and then by 300- to 900-pound stone. An armor layer, consisting of 3- to 5-ton stone, would then be placed. The reefs would extend to a height about two feet above the design maximum instantaneous water level. A beach nourished with coarse sand or gravel would be maintained behind the reefs. As with the other beach systems, periodic addition of beach fill would likely be required.

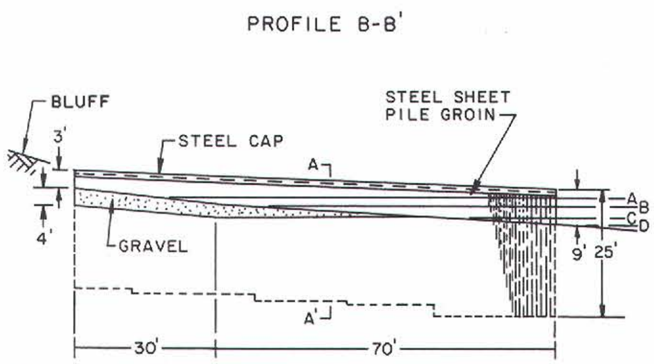
Figure 89

TYPICAL STEEL SHEET PILE GROIN SYSTEM WITH ARTIFICIALLY NOURISHED BEACH









LEGEND

LAKE MICHIGAN WATER LEVELS

A 100-YEAR RECURRENCE INTERVAL MAXIMUM INSTANTANEOUS WATER LEVEL 584.3 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

B 10-YEAR RECURRENCE INTERVAL MAXIMUM INSTANTANEOUS WATER LEVEL 582.8 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

C 1900 TO 1987 ANNUAL MEAN WATER LEVEL 579.6 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

D LOW WATER DATUM 578.1 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

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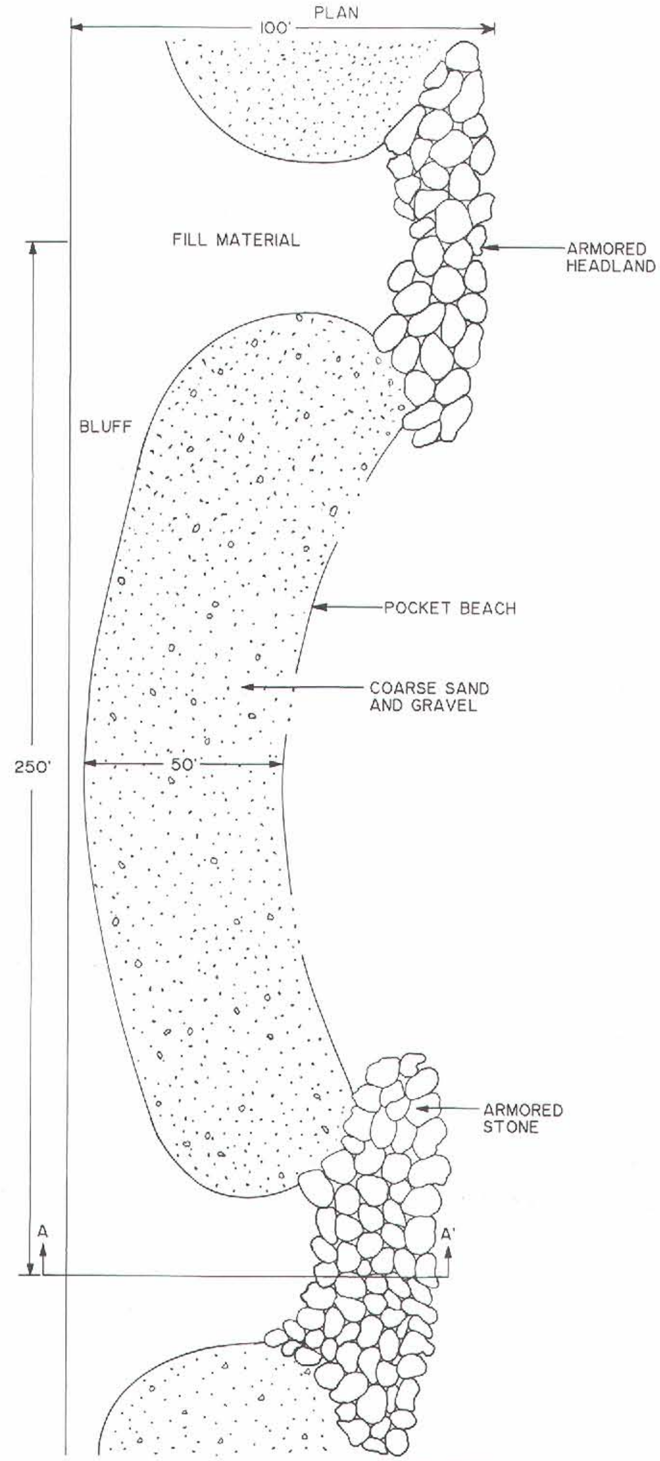
Source: Warzyn Engineering, Inc., and SEWRPC.

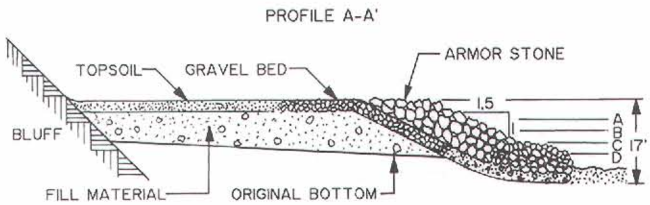
The capital cost of a near-shore reef ranges from \$350 to \$600 per lineal foot of shoreline, with an additional cost ranging up to \$115 per lineal foot of shoreline to artificially nourish the beach by shore, or an additional cost of up to approximately \$500 per lineal foot of shoreline to artificially nourish the beach by barge. Annual operation and maintenance costs would depend upon the need for periodic renourishment of the beach material, and are estimated to range from \$15 to \$50 per lineal foot.

<u>*Perched Cobble Beach System*</u>: Perched beaches constructed of cobbles would serve as wave-

Figure 90

TYPICAL ARMORED HEADLAND AND POCKET BEACH SYSTEM





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LAKE MICHIGAN WATER LEVELS

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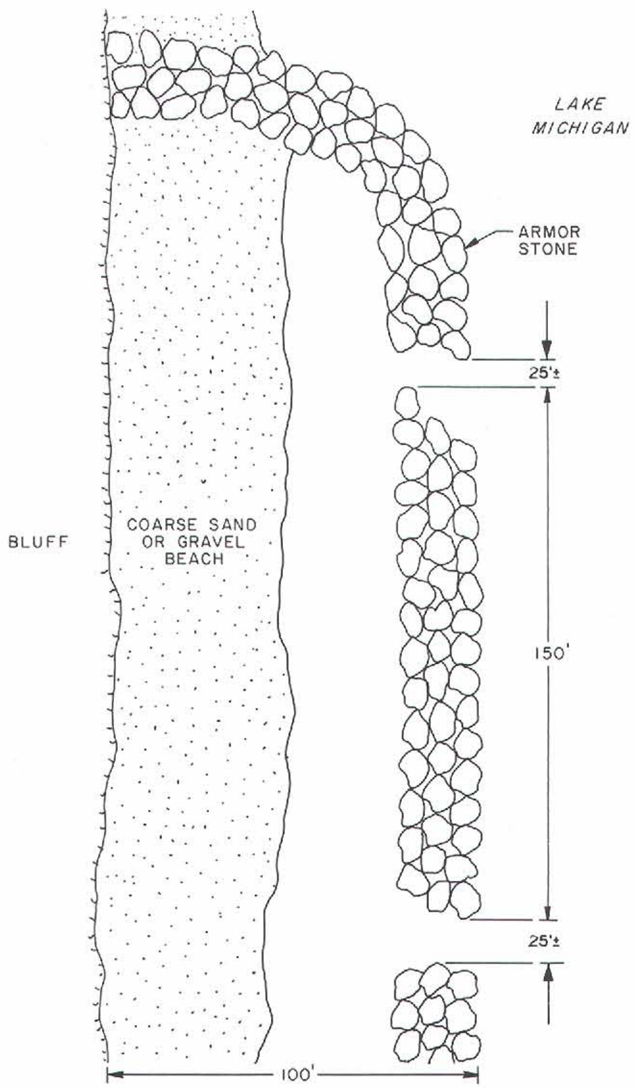
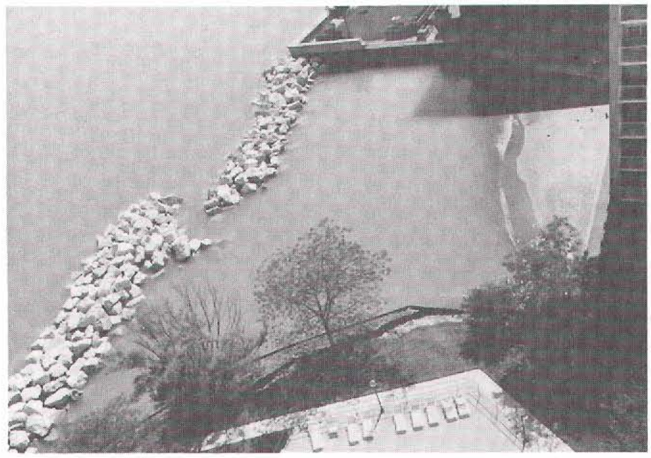
Source: Warzyn Engineering, Inc., and SEWRPC.

Figure 91

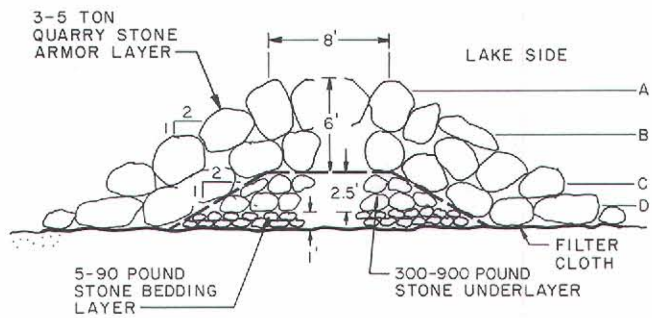
TYPICAL NEAR-SHORE STONE REEF WITH NOURISHED COARSE SAND AND GRAVEL BEACH

PLAN





PROFILE DETAIL



LEGEND

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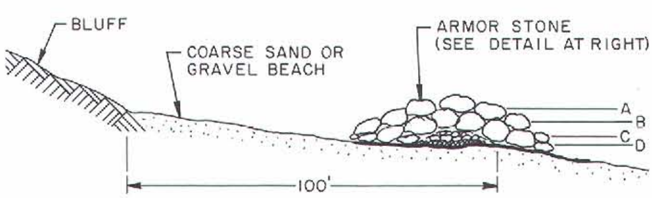
PROFILE



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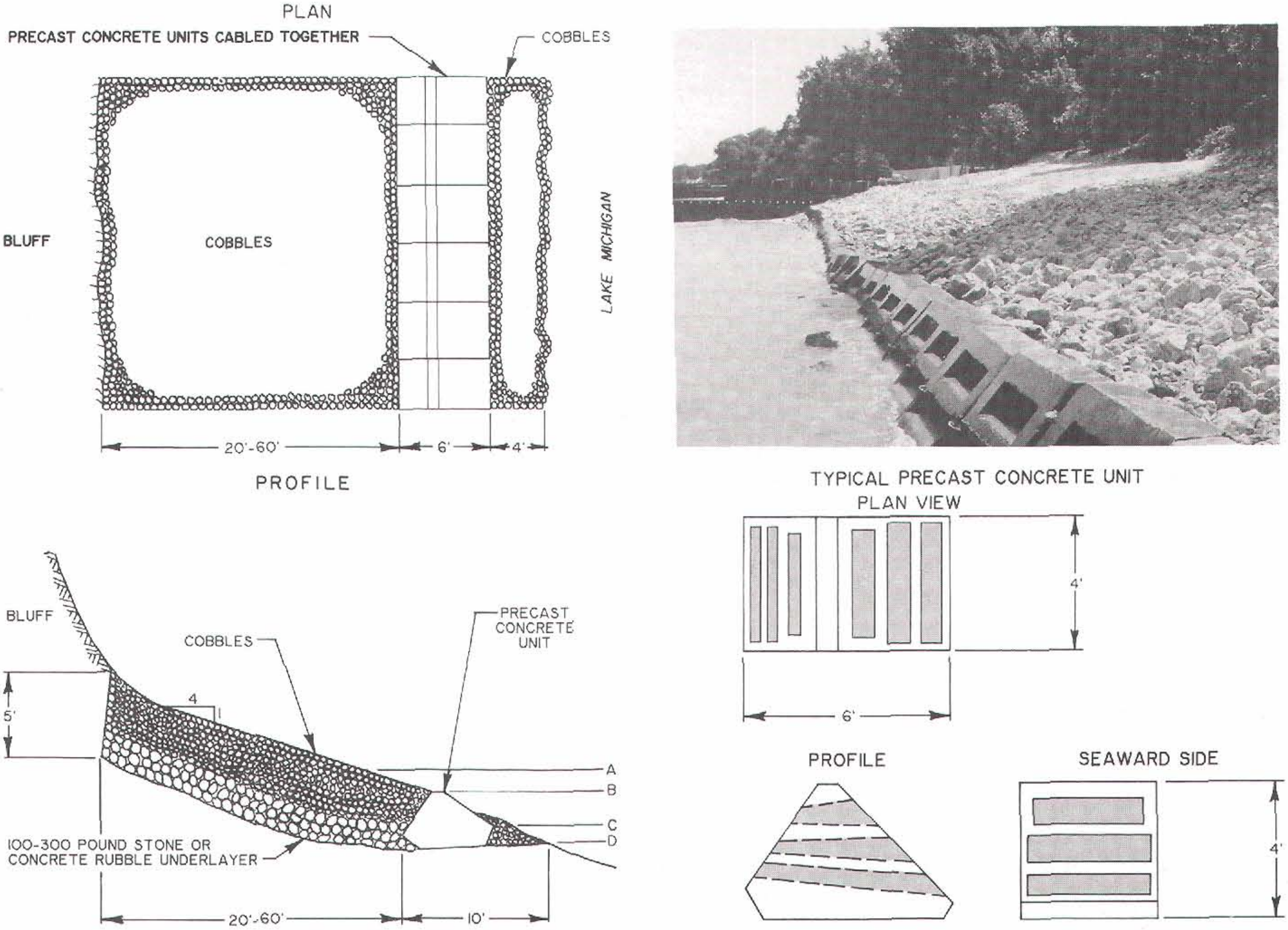
Source: STS Consultants, Ltd., and SEWRPC.

Figure 92

TYPICAL PERCHED COBBLE BEACH SYSTEM



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Source: Great Lakes Environmental Marine, Ltd., and SEWRPC.

absorbing structures, particularly suitable where the shoreline water is deep. A beach constructed of cobble stones ranging from 3 to 12 inches in diameter, as shown in Figure 92, would be able to absorb considerable wave energy, while staying intact better than do beaches composed of sand and gravel. The reduced wave reflection would help prevent scouring of the lakebed. The cobbles are typically swept by the storm surge to form raised ridges on the backshore, adding protection to bluffs. Lateral migration of the cobbles can be controlled by constructing barriers similar to groins on the downdrift sides. The disadvantage of a cobble beach system is that the use of the shoreline and access to the water may be severely limited, depending on the size and shape of the cobbles. The usability of cobble beaches installed primarily for erosion control can be enhanced by the placement of a one- to two-foot layer of gravel on top of the cobbles. Although the gravel layer would need renourishment, the stability of the cobble base and the perched beach design would reduce the need for replacement material.

To increase the effectiveness of the cobble beach and prevent the migration of the cobbles, a sill would be placed lakeward of the original shoreline to create a perched beach and a new shoreline. The sills could be constructed of precast concrete units, quarry stone, or steel sheet pile. As shown in Figure 92, the permeable, precast, steel fiber-reinforced concrete units typically weigh two tons each, and measure approximately four feet high, four feet wide, and six feet deep. They are usually set adjacent to each other in water typically three to six feet deep. The structures are secured to each other with steel cables. The manufacturers of concrete units report that the sloped front and back profile and tapered openings temper wave energy but allow enough energy transmission to avoid scouring of the lakebed from wave reflection. Accretion of beach material has been reported both lakeward and landward of some pervious sills. Heavy construction equipment is required to install the structures. The capital cost of precast concrete units is approximately $250 per lineal foot of shoreline. A sill constructed of quarry stone would have a capital cost of about $250 per lineal foot of shoreline, and a sill constructed of sheet pile would have a capital cost of about $600 per lineal foot of shoreline.

The cobble beach system, including a concrete unit or quarry stone sill, would entail a total capital cost of $350 to $400 per lineal foot of shoreline. Annual maintenance costs, which depend upon the need to renourish the supply of cobbles as well as to maintain the sill, would approximate $20 per foot.

Near-shore Pervious Concrete Sill: Where an existing natural beach of sand or coarser material is present, the erosion of the beach can be at least partially controlled by the installation of a pervious concrete sill in the surf zone, as shown in Figure 93. Precast concrete units similar to those described above for perched cobble beaches are usually placed parallel to the shore and connected with steel cables, typically 40 to 60 feet offshore in water two to six feet deep.

Wave attack on the shore and bluff is reduced by the sill's attenuating effects on the waves when they are still offshore. The sill trips and slows the waves. In addition to tempering storm surge and backwash, the sill system enhances the deposition of sediment from the littoral drift along shore. Accretion of sediment may occur both landward and lakeward of the sill. The sill would be the most effective in a shallow, low-wave-energy environment which contains a substantial amount of littoral drift material. Heavy construction equipment is required to install the structures. A disadvantage of a sill is that it interferes with small boat navigation near the shore.

The near-shore pervious concrete sill would entail a total capital cost of $200 to $300 per lineal foot of shoreline. Annual maintenance costs would approximate $10 per foot.

Manufactured Concrete Beach Containment Systems: Different types of manufactured concrete structures can be used to contain a beach area. Large, steel-reinforced concrete blocks, being about six feet high and weighing about six or seven tons each, can be placed offshore side by side in water three to four feet deep, as shown in Figure 94. The blocks can also be placed along the side of the contained beach. The structure is intended to allow waves to run up along the face and over the top, trapping the coarser, water-borne particles behind the blocks. The beach could also be artificially nourished. Toe protection and a filter layer could help to prevent scouring and the uneven settling of the blocks.

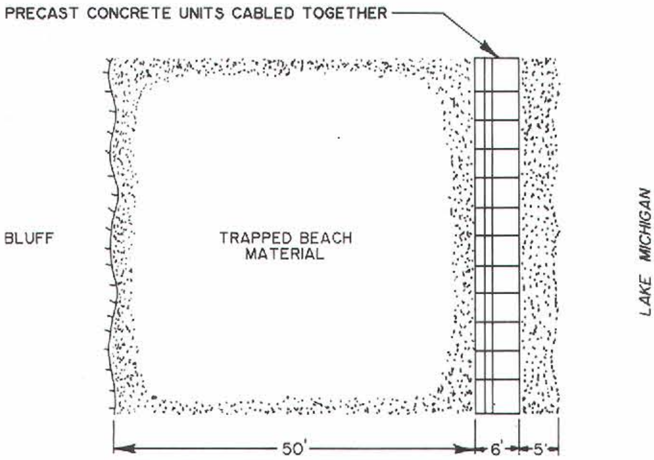
The advantages of the concrete beach containment system are that the structures directly protect the beach material from wave action, and they may—because of the height of the structures—allow the development of a beach in deeper water environments. A disadvantage is the poor suitability of the beaches for swimming or wading because of the presence of the concrete structures. The concrete structures may also be subject to displacement, which would result in the loss of beach material.

The capital cost of a concrete beach containment system would be approximately $250 per lineal foot of shoreline, with an additional cost ranging

Figure 93

TYPICAL PERVIOUS CONCRETE SILL

PLAN



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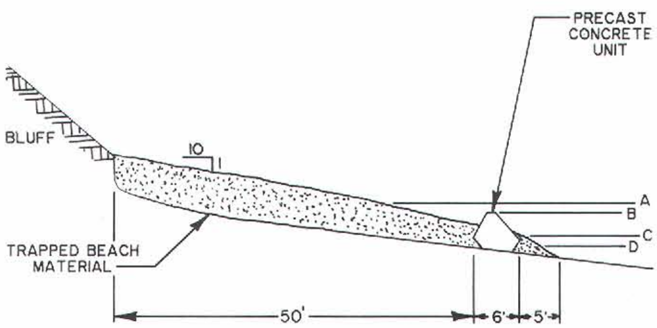
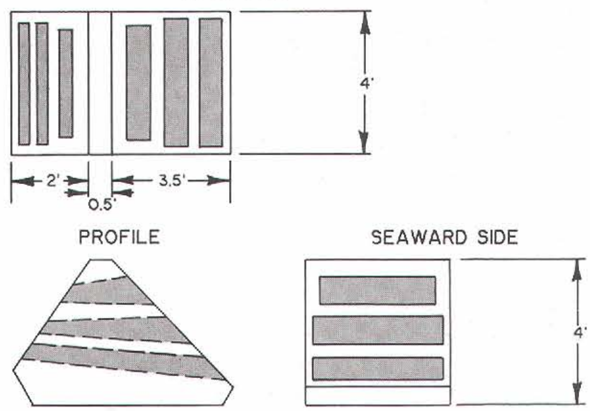


TYPICAL PRECAST CONCRETE UNIT

PLAN VIEW



LEGEND

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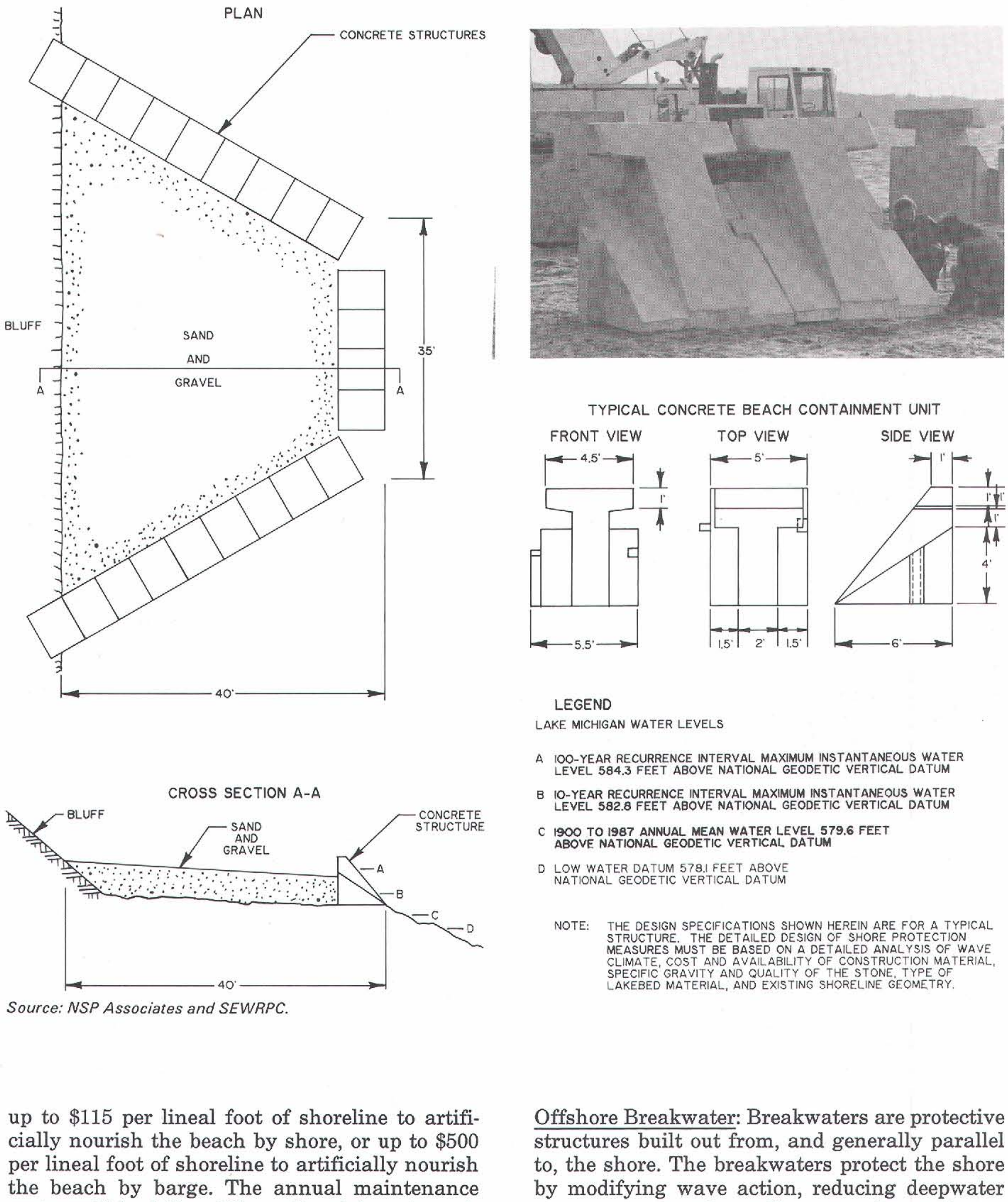
Source: Great Lakes Environmental Marine, Ltd., and SEWRPC.

Figure 94

TYPICAL CONCRETE BEACH CONTAINMENT SYSTEM WITH NOURISHED COARSE SAND AND GRAVEL BEACH



Source: NSP Associates and SEWRPC.

up to $115 per lineal foot of shoreline to artificially nourish the beach by shore, or up to $500 per lineal foot of shoreline to artificially nourish the beach by barge. The annual maintenance cost would be $15 to $50 per lineal foot, depending primarily upon the need for periodic renourishment of the beach material.

Offshore Breakwater: Breakwaters are protective structures built out from, and generally parallel to, the shore. The breakwaters protect the shore by modifying wave action, reducing deepwater wave energy, and usually promoting sediment deposition or maintenance of existing sediment shoreward of the structure. Breakwater systems can be used to contain large, nourished sand beaches. The structures are generally constructed of stone, although some designs use rock-filled concrete caissons, cellular sheet piles, timber cribs, and floating devices. One advantage of any near-shore, or offshore, protection system is that the structures are positioned off the existing shoreline, thereby providing recreational benefits while protecting the shore from erosion. Breakwaters, if properly designed, provide effective protection during periods of widely fluctuating water levels. Breakwaters can be designed to provide substantial protection without becoming complete barriers to littoral transport. A major disadvantage of a breakwater is that a large quantity of material must be deposited in relatively deep water. Heavy equipment mounted on barges is normally required for installation and continued maintenance. Because breakwaters may extend well above the water, they may interfere with the scenic view of the horizon for beach users.

Construction of an offshore breakwater along the Lake Michigan shoreline of Milwaukee County would entail a capital cost of $1,000 to $2,000 per lineal foot of shoreline. Average annual maintenance costs would approximate $20 to $50 per lineal foot.

Described below are five alternative breakwater designs: a rubblemound breakwater, a caisson breakwater, a sheet pile breakwater, a timber crib breakwater, and a floating breakwater.

Rubblemound Breakwater: A rubblemound breakwater is the most common type of breakwater in the Great Lakes. The structure, as shown in Figure 95, is usually constructed of several layers of quarry stone, rubble, or concrete units. In a typical rubblemound breakwater, the core of the breakwater is constructed of small-size stone, each weighing 1 to 50 pounds. Armor stone forms the outer layer of the breakwater. An intermediate layer acts as a filter layer to prevent the inner core materials from being washed out through the larger armor stone. Depending on the water depth and on the subsurface conditions in the area of the breakwater structure, a filter cloth is sometimes used to prevent bottom scouring and settlement of the structure.

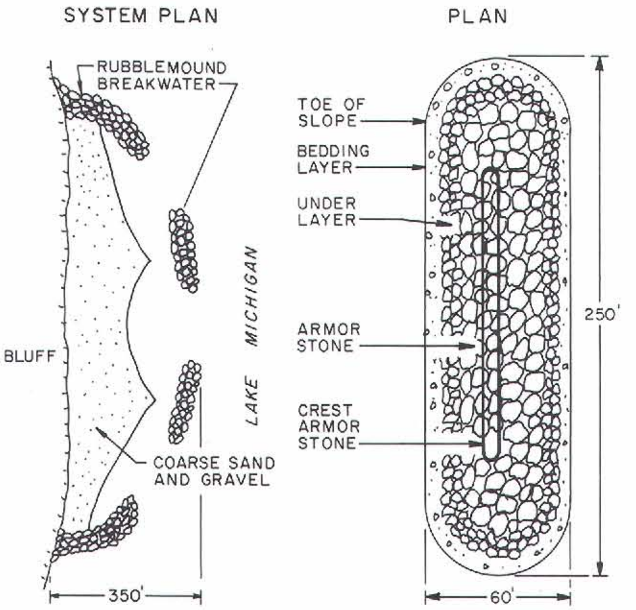
The rubblemound breakwater is intended to prevent or reduce the transmission of wave energy behind it by absorbing much of the energy and reflecting some of the remaining energy back to the main water body. If rubblemound breakwaters are too porous, they allow a high percentage of longer period wave energy to pass through, causing excessive wave action behind the structure.

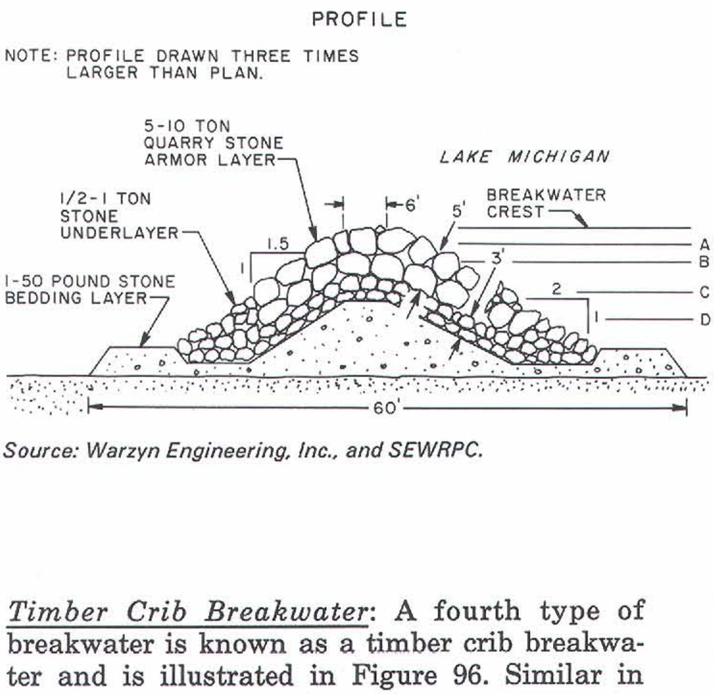
Caisson Breakwater: A caisson breakwater, as shown in Figure 96, consists of reinforced concrete boxes which are floated into position, settled on a prepared foundation, filled with stone or rubble for stability, and capped with concrete slabs or large stones. Riprap protection is then placed along the toe of the structure to prevent tilting or overturning due to scour. Caisson breakwaters were used extensively in the Great Lakes, including at the Port of Milwaukee, during the early 1900's for construction of commercial harbors. At that time, the caissons provided distinct construction advantages in deepwater situations, as the total amount of construction material used could be held to a minimum, and the labor-intensive construction costs were not excessive. Presently, caisson breakwaters are rarely considered because of the relatively shallow water in which newer breakwaters are usually located, and because of the excessive cost of construction. In addition, when the caisson structures are not properly tied into the lakebed, the rectangular shape of the structures makes them subject to overturning or sliding in severe wave climates.

Sheet Pile Breakwater: Breakwaters can also be constructed of steel sheet piles. Many variations are found in the design of sheet pile breakwaters. One design provides a series of circular cells constructed of steel sheet piling and filled with either stone or rubble and capped with concrete, as shown in Figure 96. Single steel sheet pile cells are often used at the end of rubblemound structures to clearly define the safe water area of the entrance to the harbor. The cells also provide a solid base for the installation of navigation lights. Riprap toe protection is required along the base of all sheet pile breakwaters to prevent scouring. Sheet pile breakwater structures provide navigable water up to their edge. In addition to a high initial cost, a disadvantage of the steel sheet pile breakwater is that the face of the structure does not absorb wave energy and, if improperly located, may cause severe reflected wave conditions.

Figure 95

TYPICAL SEGMENTED RUBBLEMOUND BREAKWATER SYSTEM





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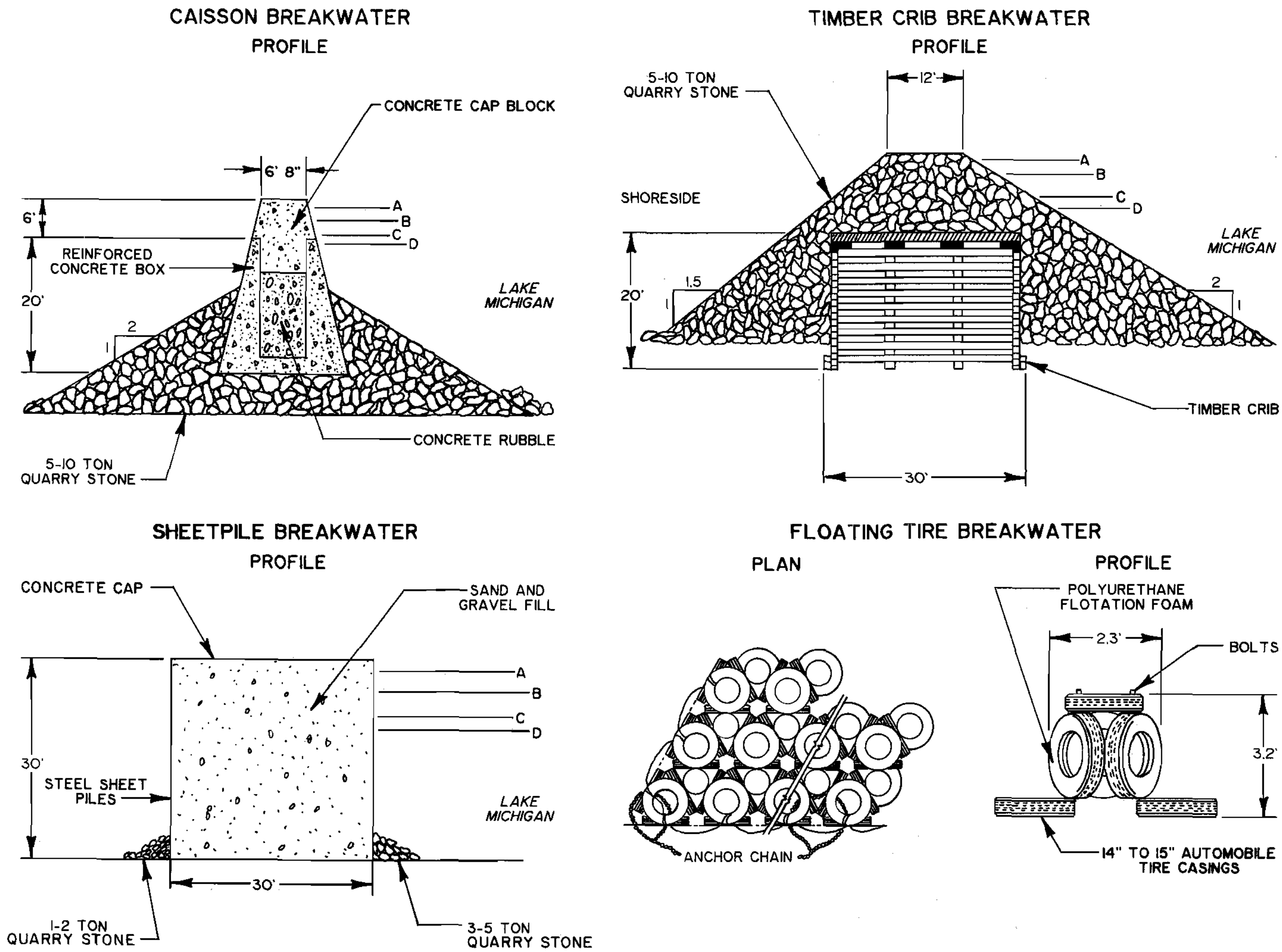
Source: Warzyn Engineering, Inc., and SEWRPC.

Timber Crib Breakwater: A fourth type of breakwater is known as a timber crib breakwater and is illustrated in Figure 96. Similar in construction to the caisson breakwater, the timber cribs are floated into position and settled on a prepared foundation by filling the compartments with stone. The toe of the structure is protected by riprap placed at the base of the structure. In the early 1900's, timber cribs were frequently used for the construction of harbors, including for the Milwaukee Harbor. The advantage of using timber cribs for construction of a breakwater is the reduced need for large armor stone, since the cribs may be filled with smaller sized material. The major disadvantage of the timber cribs is the limited durability of wood compared to other materials, as exposed timbers are subject to decay. Timber cribs should be designed to remain submerged, and covered by armor stone, as shown in Figure 96.

Floating Breakwater: Floating breakwaters, as shown in Figure 96, are constructed of buoyant materials such as logs, hollow concrete boxes,

Figure 96

MISCELLANEOUS ALTERNATIVE TYPES OF BREAKWATERS



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Source: U. S. Army Corps of Engineers, Milwaukee County Park Commission, and SEWRPC.

and rubber tires. Floating breakwaters have not been able to effectively and economically dissipate deepwater wave energy in the open Lake Michigan environment. However, in areas of partially protected waters, such as behind rubblemound breakwaters and islands, some designs of floating structures may reduce moderate waves. Floating breakwaters are advantageous where offshore slopes are steep and fixed breakwaters would be too expensive because of water depths. However, since floating breakwaters are effective only against small to moderate, short-period waves, they could be used only as supplementary protection in Milwaukee

County. Most floating breakwaters would need to be removed during the winter to prevent ice damage to the structure.

Offshore Islands and Peninsulas: Islands and peninsulas lying 250 to 1,500 feet offshore could be constructed to provide substantial protection from wave action while creating additional recreational land. The islands or peninsulas, as shown in Figure 97, would be constructed of fill material consisting of rubble, soil, or tunnel construction debris. The fill material should be protected from wave action by the use of a revetment or an armored headland and pocket beach system. The offshore islands or peninsulas, like offshore breakwaters, dissipate deepwater wave energy before it reaches the shoreline. However, the islands and peninsulas should be far enough offshore to prevent the accumulation of significant amounts of sediment landward of the islands.

A major advantage of islands and peninsulas is the additional land created for recreational use. A relatively protected waterway may also be created adjacent to the existing shoreline.

The major disadvantages of islands and peninsulas are the large amount of material required for construction, and the need to protect the lakeward side against deepwater wave energy. A reduced level of armor protection can be provided along the landward side of the island or peninsula. The cost of offshore islands and peninsulas varies greatly, depending primarily on the type and cost of fill material available for the internal core of the structure, the armor protection cost, and the method of construction. Construction of offshore islands would entail a capital cost of $800 to $1,500 per lineal foot of shoreline, assuming that fill material is available at a minimal cost. Average annual maintenance costs would approximate $20 to $40 per lineal foot.

Bluff Slope Stabilization

In Chapter III of this report, 57 bluff analysis sections, covering 57,240 feet, or 36 percent, of the total study area shoreline, were classified as having marginal or unstable bluff slopes with respect to rotational sliding. Potential bluff slope stabilization measures include regrading the bluff slope to a stable angle, installing groundwater drainage systems to lower the elevation of the groundwater and prevent groundwater seepage from the face of the bluff, constructing surface water control measures, and revegetating the bluff slope.

Bluff Slope Regrading: Regrading the bluff slope to a stable angle was indicated for 48 bluff analysis sections covering 44,270 feet, or 28 percent of the study area shoreline. Bluff analysis sections identified as needing bluff slope regrading were those in which other economically feasible measures would not effectively stabilize the slopes. A primary advantage of bluff slope regrading is that further bluff recession is prevented—if bluff toe protection, surface and groundwater drainage, and slope revegetation are also provided where needed. Slope regrading will also provide structural stability to the bluff toe protection measures, preventing them from being buried by bluff material.

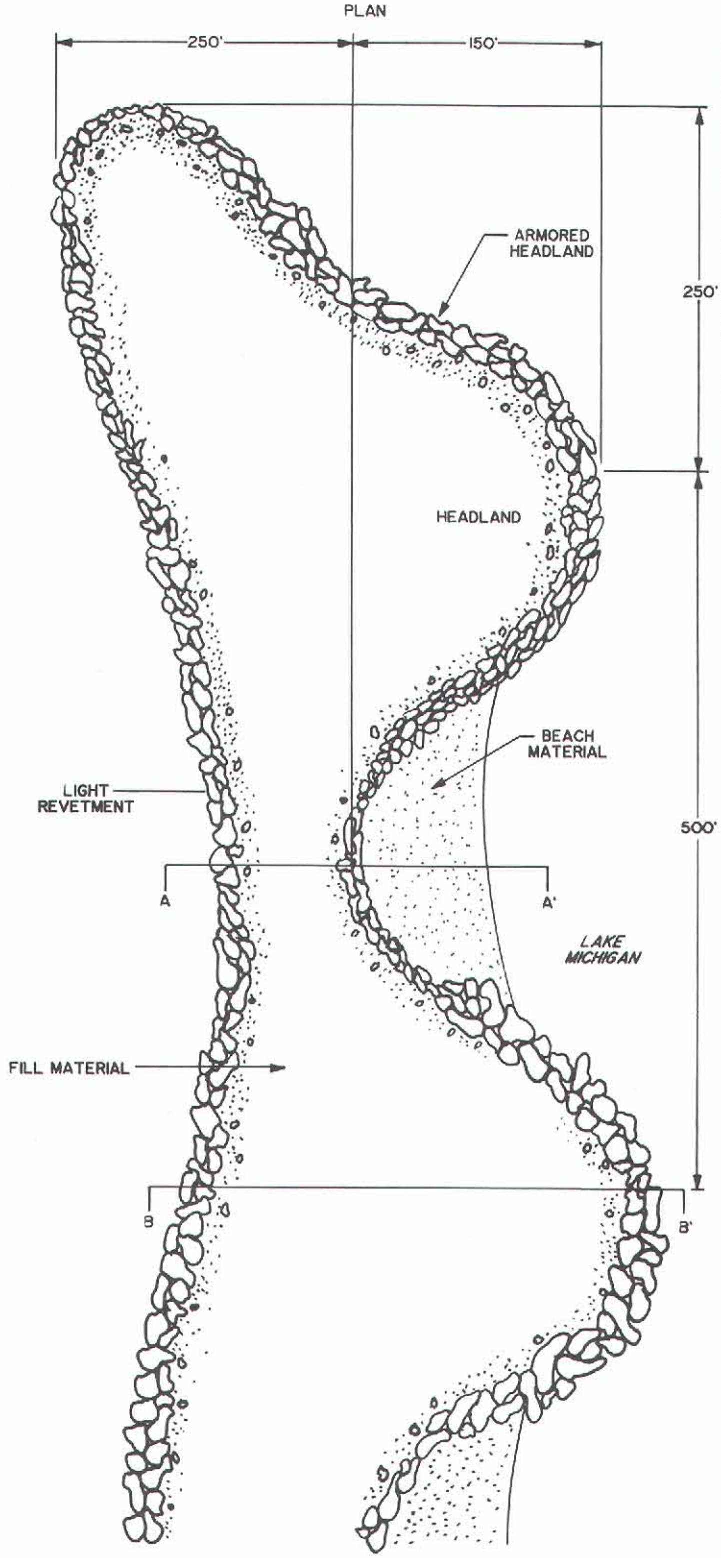
The disadvantage of bluff slope stabilization is that the natural aesthetic properties and drainage characteristics of the bluff are disrupted. In addition, there are problems, albeit temporary, related to the truck and heavy equipment traffic moving to and from the site, as well as to the impacts of the dust and noise at the construction site itself.

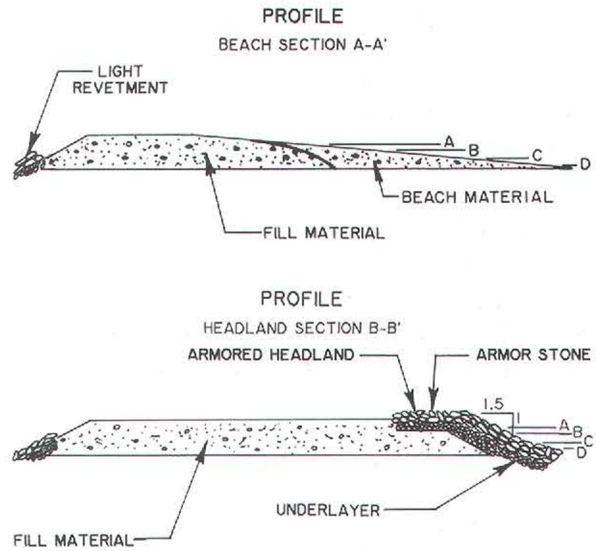
Four alternative methods for bluff slope regrading, as shown in Figure 98 and described below, include the cutback method, the fill method, the cut and fill method, and the terracing method. All four methods involve regrading at least a portion of the bluff slope to a flatter angle.

Cutback Method: Bluff slope regrading can be accomplished by using earth-moving equipment to regrade the face of the slope to a flatter, more stable profile, as shown in Figure 98. As already noted, a bluff slope of one on two and one-half will usually provide a stable bluff slope in the study area. The cutback method can be used only in areas where the houses concerned are located a sufficient distance from the edge of the bluff. Topsoil placement, seeding, and mulching would be required to develop a protective vegetative cover. Where needed, adequate toe protection, as well as drainage of surface- and groundwater, would have to be provided to maintain the regraded bluff slope. The cutback method eliminates, or reduces, the need for the placement of fill on the bluff face. The disadvantage of the cutback method for bluff slope regrading is that land at the top of the bluff is lost.

Figure 97

TYPICAL OFFSHORE ISLAND OR PENINSULA





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C 1900 TO 1987 ANNUAL MEAN WATER LEVEL 579.6 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

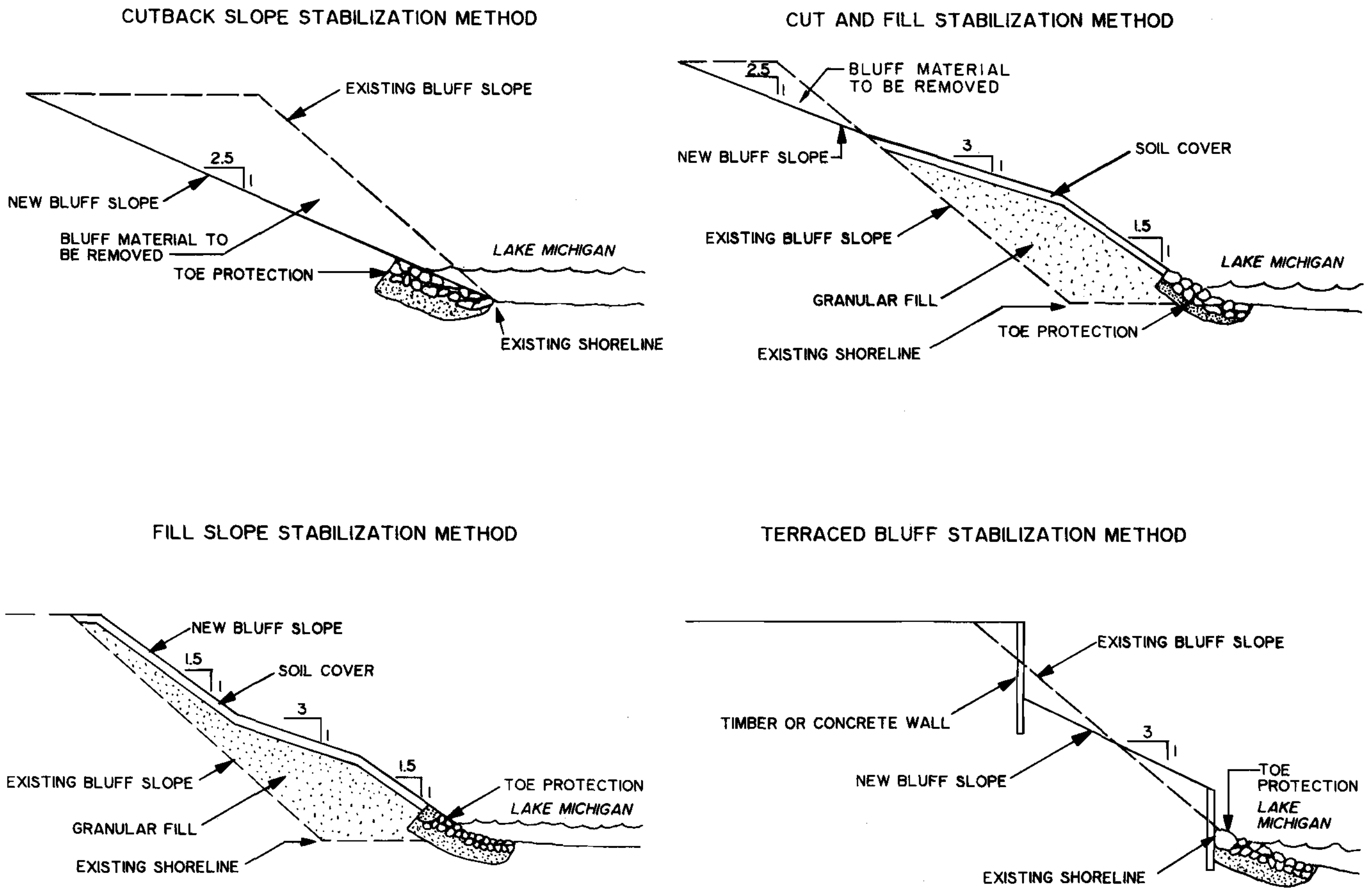
D LOW WATER DATUM 578.1 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

NOTE: THE DESIGN SPECIFICATIONS SHOWN HEREIN ARE FOR A TYPICAL STRUCTURE. THE DETAILED DESIGN OF SHORE PROTECTION MEASURES MUST BE BASED ON A DETAILED ANALYSIS OF WAVE CLIMATE, COST AND AVAILABILITY OF CONSTRUCTION MATERIAL, SPECIFIC GRAVITY AND QUALITY OF THE STONE, TYPE OF LAKEBED MATERIAL, AND EXISTING SHORELINE GEOMETRY.

Source: Warzyn Engineering, Inc., and SEWRPC.

Figure 98

ALTERNATIVE METHODS OF BLUFF SLOPE STABILIZATION



Source: SEWRPC.

Bluff slope regrading using the cutback method would entail a capital cost of $100 to $150 per lineal foot of shoreline. Maintenance costs are assumed to be about $15 per lineal foot during the first three years following bluff slope regrading, primarily for the maintenance of a new vegetative cover.

Fill Method: Bluff slope regrading can also be accomplished by transporting soil, concrete rubble, and other clean fill from an outside source and placing it on the face of the bluff to provide a more stable profile. Filling will likely be required for those bluff analysis sections where the houses are located close to the edge of the bluff. The fill materials, as shown in Figure 98, should be granular. Fine-grained, clay-type materials are not suitable for fill material in those areas susceptible to groundwater drainage problems. Depending on the type of material used for filling, a slightly steeper angle—often approximating 35 degrees—may be utilized for portions of the regraded bluff slopes. Slopes constructed of fill material are normally terraced, or contain compound slopes. Filling should begin at the slope bottom, and some bluffs may need to be filled only along the lower portions of the slope. Soil placement, seeding, and mulching would be required to develop a vegetative cover. Adequate toe protection would also be provided to maintain and protect the fill material.

The primary benefit of using the fill method is that land at the top of the bluff is not removed, which is particularly advantageous in areas where houses are located close to—within 50 feet of—an unstable the bluff edge. An adverse impact of using fill is the necessity to sometimes fill into the lake in order to provide a stable slope. Other disadvantages include the trucking and aesthetic impacts associated with filling.

Bluff slope regrading using the fill method would entail a capital cost of $150 to $250 per lineal foot of shoreline. Maintenance costs are assumed to be $10 to $15 per lineal foot during the first three years following bluff slope regrading, primarily for the maintenance of a new vegetative cover.

Cut and Fill Method: A combination of cutting the upper unstable portion of the bluff slope, and placing that material—along with additional fill material, if necessary—at the base of the bluff slope can also provide a stable bluff slope. The cut and fill method is also shown in Figure 98. The cut and fill method should be limited in use to those areas in which houses are located at least 50 feet from the edge of the bluff slope. Soil placement, seeding, and mulching are required to develop a protective vegetative cover; and adequate toe protection should be provided to maintain the regraded bluff slope.

The advantage of using the cut and fill method over the cutback method is that less land is lost at the top of the bluff slope. The majority of the material needed for filling is already at the site, and, compared to the total fill method, less fill material would extend out into the lake.

Bluff slope regrading using the cut and fill method would entail a capital cost of $100 to $200 per lineal foot of shoreline. Maintenance costs would range from $10 to $15 per lineal foot during the first three years following bluff slope regrading, primarily for the maintenance of a vegetative cover.

Terracing Method: Slope stabilization can also be provided by the placement of a series of vertical retaining walls within the regraded bluff slope, as shown in Figure 98. The retaining walls may be constructed of stone, timber, interlocking concrete blocks, steel sheet pile, or gabions. The bluff slope between the retaining walls is regraded to a slope of one on three or flatter, and vegetated. The terracing method can provide improved access to the shoreline if a suitable walkway is provided. Depending upon the design of the terrace system, less bluff material may need to be removed at the top of the bluff than under the cutback method, or under the cut and fill method.

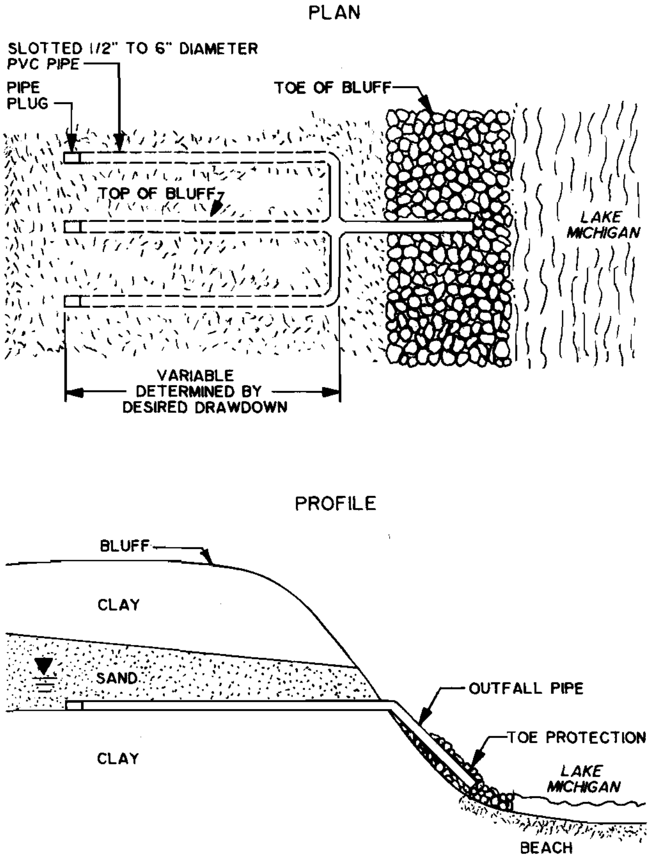
The primary disadvantages of the terracing method are its relatively high cost, and construction difficulty. Construction of a bluff slope that is entirely terraced may entail a capital cost of $1,000 to $3,500 per lineal foot of shoreline. Annual maintenance costs would be $10 to $15 per lineal foot during the first three years following bluff slope regrading, primarily for the maintenance of a new vegetative cover. Because of this relatively high cost, terraces are often constructed on only a portion, such as, the upper one-third, of the bluff slope.

Groundwater Drainage: Groundwater drainage was indicated to enhance slope stability in eight bluff analysis sections covering 10,200 feet, or 6 percent, of the study area shoreline. The groundwater conditions and stratigraphy assumed within these marginal or unstable sections was such that lowering the level of the water table may be expected to significantly help stabilize the bluff slopes. Detailed, site-specific analyses of the groundwater conditions must be conducted at the preliminary engineering phase to affirm the feasibility of groundwater drainage systems. Groundwater drainage is also recommended to be considered during, and following, the construction of fill projects to prevent excess hydrostatic pressures caused by the compression of saturated soils by the weight of the fill material and the blocking of seepage paths. Drainage systems require relatively minor maintenance and should not limit the use of the shoreline. A groundwater drainage system would also not disturb the vegetative cover on the bluff slope, nor require changing the slope geometry. A limitation of groundwater drainage as a slope stabilization control measure is that drainage is usually economically feasible only in granular layers. The removal of water within clay glacial till layers is usually too costly and difficult. Three alternative groundwater drainage systems are described below: horizontal drains, vertical drains, and trench drains.

Horizontal Drains: A horizontal drain is a small-diameter boring drilled into the face of the bluff slope on a 5 to 10 percent grade and fitted with a perforated pipe. As shown in Figure 99, a system of collector pipes or ditches is provided to carry the collected water to the base of the bluff or to a suitable outlet. A horizontal drainage system is the most effective in layers of granular material containing sand and gravel. Drains are usually spaced across the face of the

Figure 99

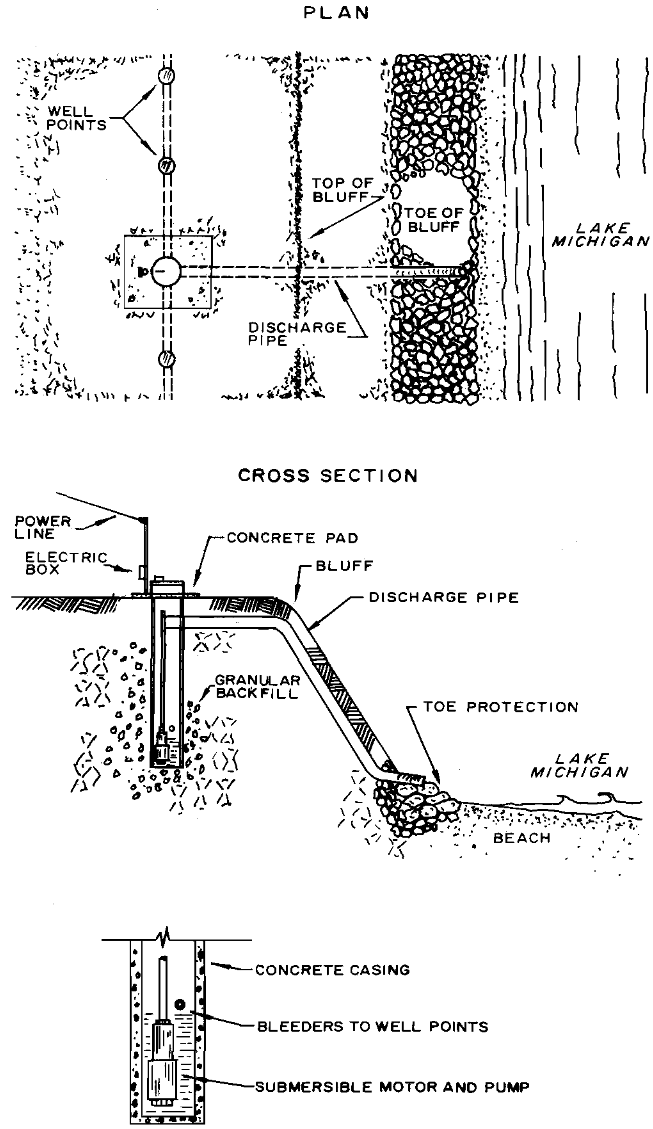
HORIZONTAL DRAINAGE SYSTEM



Source: Owen Ayres & Associates, Great Lakes Shore Erosion Protection, Structural Design Examples, 1978; and SEWRPC.

Figure 100

VERTICAL DRAINAGE SYSTEM



Source: SEWRPC.

bluff slope at suitable intervals based on the anticipated flow rates and soil permeability.

The advantages of a horizontal drain system are that the system drains by gravity, and requires relatively little maintenance. The primary disadvantage of the system is that access to the base of the bluff to install the drains is often difficult.

Construction of a horizontal drain system to lower the level of groundwater would entail a capital cost of $30 to $75 per lineal foot of shoreline. The annual operation and maintenance cost would range from $5.00 to $10 per lineal foot.

Vertical Drains: A vertical drain, or well, usually consists of an 18- to 36-inch-diameter boring drilled vertically from the top of the bluff into the water-bearing strata. Water can be either pumped from the well, or tapped with a gravity outlet, as shown in Figure 100. Gravity-drained vertical wells can be connected to horizontal drains which carry the collected water out of the bluff to a safe point of disposal. Water pumped from a vertical well can be discharged to the base of the bluff or to a suitable surface water outlet. Unlike most horizontal drains, vertical drains can be designed to drain several water-bearing strata separated by impermeable layers. Detailed geotechnical analyses should be conducted in the preliminary engineering phase to determine the necessary location, spacing, depth, and pumping rate of the well points. Under favorable conditions, relatively large amounts of water can be pumped from the wells to lower the groundwater table. In addition, access to install the drains is generally not a problem because vertical drains are installed from the top of the bluff. The disadvantages of this system are that the wells must be pumped continuously to maintain the lower water table, and substantial maintenance of the wells and pumps may be required.

Construction of a vertical drain system would entail a capital cost of $50 to $150 per lineal foot of shoreline. The annual maintenance cost would range up to $20 per lineal foot.

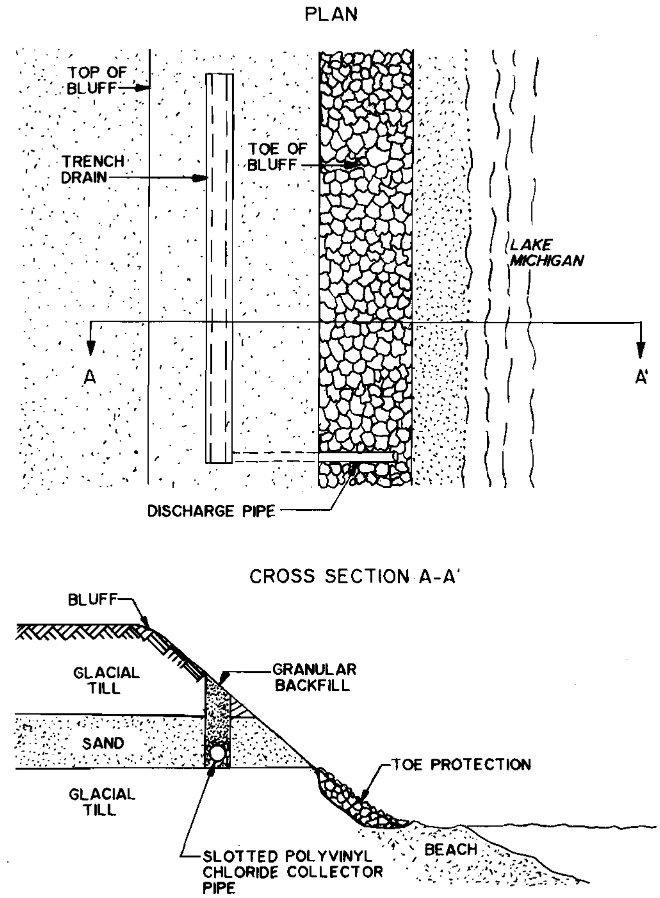
Trench Drains: The purpose of a trench drain is to intercept and divert shallow seepage. A typical design consists of a narrow trench, dug parallel to the edge of the bluff, in which a perforated collector pipe is installed. The pipe is connected to a discharge outlet and the trench backfilled with granular material, as shown in Figure 101. Drainage trenches are typically two to six feet deep, and 18 to 24 inches wide. A trench drain is relatively inexpensive and easy to install, and drains by gravity. The disadvantage of this system is that it is limited to areas of shallow seepage, although deeper water-bearing strata can sometimes be drained by constructing the trench on the face of the bluff.

Construction of a trench drain may entail a capital cost of $20 to $80 per lineal foot of shoreline, with an annual maintenance cost of up to $5.00 per lineal foot.

Surface Water Drainage: Uncontrolled storm runoff can pond water at the top of the bluff, on top of slump blocks, and behind shore protection structures, as well as form gullies on bluff slopes. Surface water drainage control is particularly indicated for four bluff analysis sections covering 4,360 feet, or 3 percent, of the study area shoreline. Specific drainage problems which reduced the stability of the bluff slopes were identified within each of these sections. Surface water drainage measures include various types of structures intended to prevent the ponding of water, to reduce surface flows over the top of the bluff, to prevent scouring and erosion of drainage channels and gullies, and to prevent excessive infiltration into the bluff. An example of a stormwater drainage system to prevent excessive runoff over the top of the bluff is shown in Figure 102. Surface water drainage systems have a relatively low cost, require little maintenance, and should not limit the recreational use of the shoreline.

Figure 101

TRENCH DRAINS



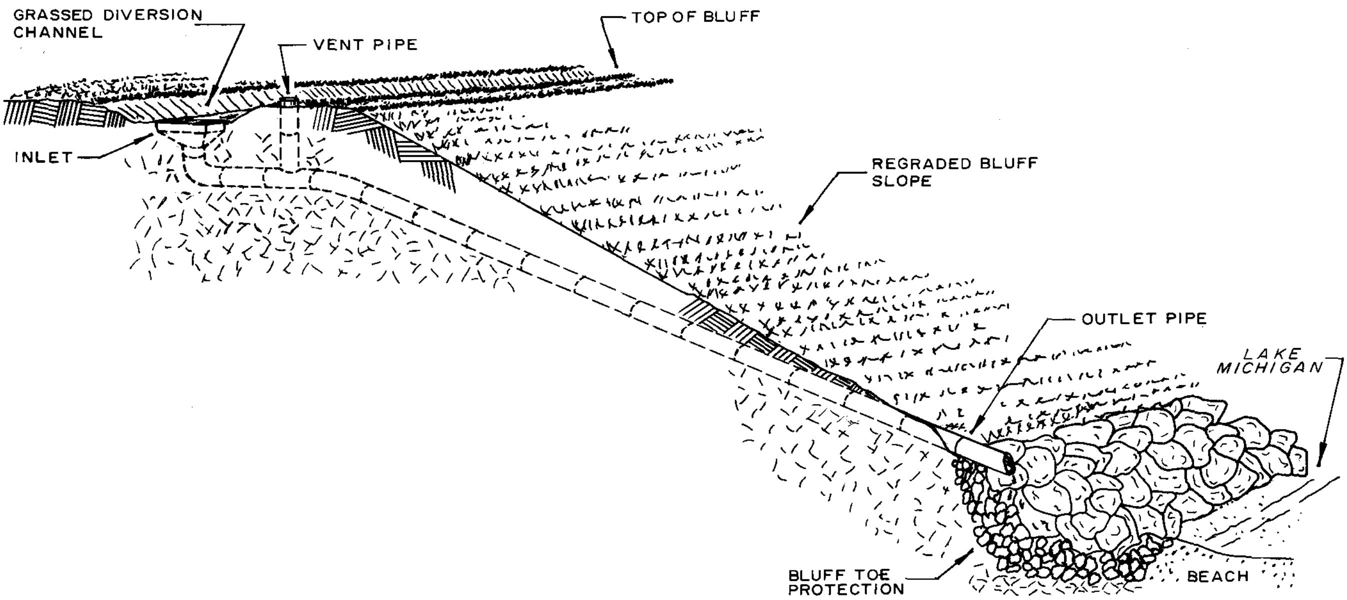
Source: SEWRPC.

A drainage system would entail a capital cost of $15 to $150 per lineal foot of shoreline, with an annual maintenance cost of up to $5.00 per lineal foot.

Revegetation: Revegetation of the bluff slope as a means to enhance slope stability was indicated for portions of 12 bluff analysis sections. In addition, revegetation will be required for all bluffs where extensive regrading occurs. Revegetation can improve slope stability by preventing translational sliding, trapping sediment, and controlling surface runoff. In addition, a well-vegetated bluff slope is aesthetically pleasing,

Figure 102

STORMWATER DRAINAGE SYSTEM TO PREVENT EXCESSIVE STORM RUNOFF OVER THE TOP OF THE BLUFF



Source: SEWRPC.

improves access to the shoreline, and provides habitat for wildlife. The establishment of a vegetative cover has a modest cost and requires minimal maintenance. Two alternative methods of revegetating bluff slopes include seeding and transplanting.

Seeding: Grass and other herbaceous plant mixtures can be seeded by scattering the seed on the bluff face by hand; by hydroseeding, which distributes the seed in a mixture of water, fertilizer, and mulch; or by drilling, in which a seed and fertilizer are inserted into the soil and covered. Hydroseeding and drilling, which are best suited for large-scale planting and for planting on steep slopes, are labor- and equipment-intensive and therefore more expensive methods of seeding. With hand broadcast seeding, fertilizer is applied as needed, and mulch is used to prevent erosion of the seed, to control weeds, and to reduce moisture loss. Straw and hay are the most suitable mulching materials; however, wood fiber mulches applied by hydroseeding have also given good results.

Spot seeding is an effective method of establishing many of the woody plants. This method enhances the successful germination of the seeds, although it does require more intensive preparation and care of each seeding spot. Seeds are typically placed in holes approximately four inches deep with controlled-release fertilizers. Mulching would again be used, but special care would be needed to prevent the mulch from interfering with seedling emergence or growth.

The cost of revegetating a bluff slope by seeding would range from $20 per 1,000 square feet if scattered by hand, to $40 per 1,000 square feet if hydroseeding or drilling is used. Annual maintenance costs for the first three years following seeding would approximate $5.00 per 1,000 square feet for hand scattering, and $10 per 1,000 square feet for hydroseeding or drilling.

Transplanting: Transplanting may be necessary to revegetate difficult sites, and can be used for establishing grasses, shrubs, and trees. Typically conducted by hand, transplanting would require careful attention to excavation of the holes, placement of the plants, fertilization, and watering. Transplanting provides the benefits of an immediate vegetative cover and allows the individual plants to be arranged as desired. It is, however, highly labor-intensive.

The capital cost of revegetating a bluff slope by transplanting would range from $200 to $500 per 1,000 square feet. Annual maintenance costs would range from $40 to $100 per 1,000 square feet for the first three years following planting.

Setback Requirements for New Urban Development

Setback requirements for new urban development directly related to erosion hazards can be incorporated into existing city and village zoning ordinances. These requirements are intended to prevent the placement of new urban development in areas with a substantial risk of erosion damage over the economic life of the facilities. Setback distances would be comprised of two components: an erosion risk distance and a minimum facility setback distance. Erosion risk distances would consist of the distance from the existing bluff edge which could be affected by recession of the bluff over time, and by the regrading of the bluff slope as required to achieve a stable slope angle. The minimum facility setback distance would provide an additional safety factor intended to prevent facilities from being placed too close to the bluff edge, and to provide an open space area which can be effectively utilized for surface water and groundwater drainage control. Setback distances from the existing bluff edge for new urban development would be calculated under both nonstructural—that is, without shore protection—and structural—that is, with shore protection—alternatives.

Currently, under the State shoreland zoning legislation, which applies to unincorporated areas, structures must be set back a minimum of 75 feet from the ordinary high-water line. The setback distances are intended to protect the State's public waters from pollution and to safeguard other environmental, aesthetic, and recreational values of the shoreline. Counties are directed to adopt ordinances that regulate lot sizes, establish building and structural setbacks, and restrict the cutting of shoreland vegetative cover. Presently, five Wisconsin counties—Douglas, Manitowoc, Sheboygan, Ozaukee, and Racine—have adopted more stringent shoreline setback ordinances which take into account Lake Michigan coastal erosion rates. The county setback distances generally consist of a stable slope component based on a stable bluff slope of one on two and one-half, plus the recession of the bluff which may be expected to occur over a period of approximately 50 years. The setback distances within these counties generally range from 100 to 200 feet from the edge of the calculated bluff slope.

Nonstructural Setback Distance: The procedure developed for delineating setback distances from the bluff edge where inadequate structural shore protection is provided is illustrated in Figure 103. Nonstructural setback distances for new buildings and facilities would consist of the sum of the nonstructural erosion risk distance and a minimum facility setback distance.

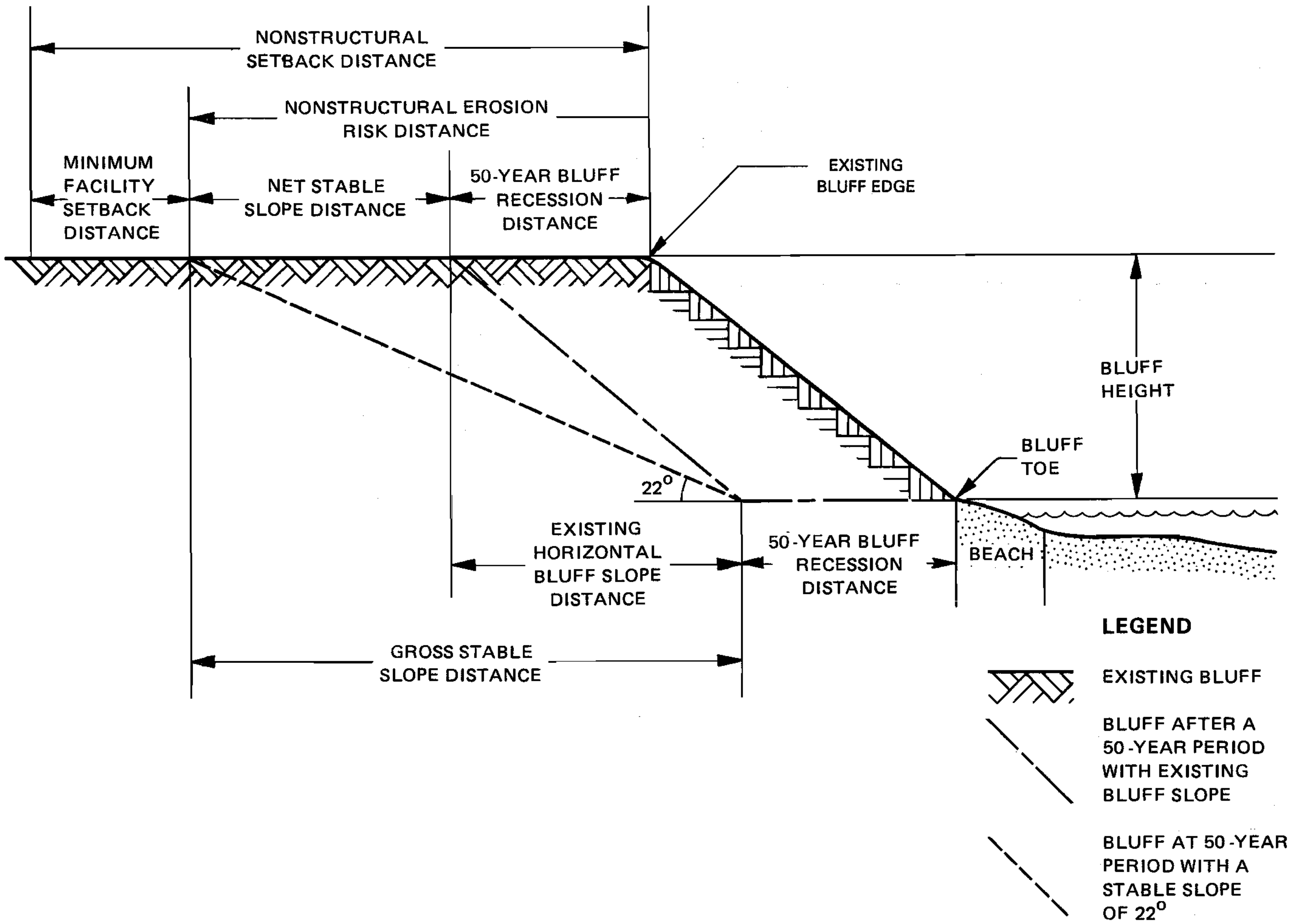
Nonstructural erosion risk distances are comprised of a bluff recession distance over a given time period, plus the distance required to grade the bluff face to a stable slope. Erosion risk distances are recommended to be delineated for a 50-year period of continued bluff recession. This distance is calculated in Table 39 in Chapter III of this report for the marginal and unstable bluff slopes along the Lake Michigan shoreline of Milwaukee County. The distance required to grade the face of the bluffs to an assumed stable slope of approximately one on two and one-half, or about 22 degrees, as discussed in Chapter III of this report, is included in the erosion risk distance for two reasons. First, the stable slope distance serves as a safety factor. It cannot be assumed that the bluff face will remain at its existing slope, and the potential exists for the bluff slope to rapidly, and sometimes catastrophically, recede to a more stable slope. Second, for shoreline reaches currently unprotected by shore protection structures, the stable slope distance allows the opportunity to properly construct an adequate shore protection structure which would include bluff slope stabilization.

Minimum facility setback distances are recommended because future bluff recession rates could differ substantially from the historical bluff recession rates. A minimum facility setback distance of 50 feet is recommended for public utilities and public recreation facilities, and a 100-foot minimum facility setback distance is recommended for all other permanent buildings and facilities.

Structural Setback Distance: The procedure developed for delineating setback distances from the bluff edge where adequate structural shore protection is provided is illustrated in Figure 104. Structural setback distances consist of the sum of the structural erosion risk distance and a minimum facility setback distance. Structural setback distances would also apply to those

Figure 103

PROCEDURE UTILIZED TO ESTIMATE NONSTRUCTURAL EROSION RISK DISTANCE AND NONSTRUCTURAL SETBACK DISTANCE



NONSTRUCTURAL EROSION RISK DISTANCE = NET STABLE SLOPE DISTANCE + 50-YEAR BLUFF RECESSION DISTANCE

NONSTRUCTURAL SETBACK DISTANCE=NONSTRUCTURAL EROSION RISK DISTANCE+MINIMUM FACILITY SETBACK DISTANCE

WHERE: NET STABLE SLOPE DISTANCE = GROSS STABLE SLOPE DISTANCE - EXISTING HORIZONTAL BLUFF SLOPE DISTANCE

$$\text{GROSS STABLE SLOPE DISTANCE} = \frac{\text{BLUFF HEIGHT}}{\text{TAN } 22^{\circ}} = \frac{\text{BLUFF HEIGHT}}{0.4}$$

MINIMUM FACILITY SETBACK DISTANCE: INTENDED TO PROVIDE A SAFETY ZONE, PROVIDE AESTHETIC BENEFITS, AND ALLOW PROVISION OF FUTURE SURFACE WATER AND GROUNDWATER DRAINAGE SYSTEMS

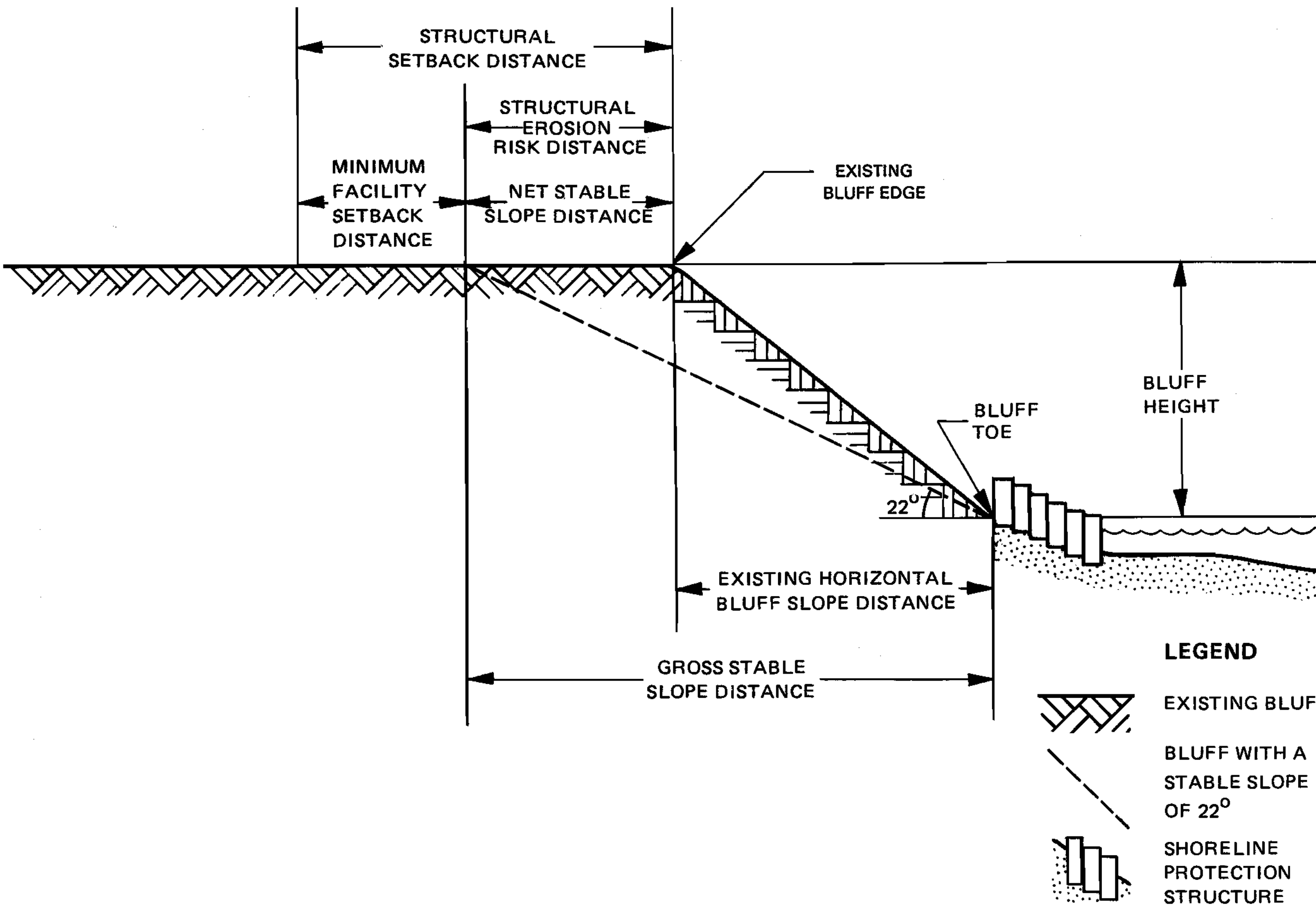
Source: SEWRPC.

portions of the Lake Michigan shoreline that are currently stabilized, even if no shore protection structure is in place.

The rate of bluff recession would be assumed to be zero once the structural measures were in place, the bluff toe protected, and the bluff slope stabilized. A structural erosion risk distance would therefore consist of that distance required to form a stable bluff slope of one on two and one-half, or about 22 degrees. As discussed in Chapter III of this report, where fill is used to regrade bluff slopes, a steeper slope angle can generally be maintained. Fill sites with stable bluff slopes within the study area have slope angles as steep as 35 degrees. In shoreline areas

Figure 104

PROCEDURE UTILIZED TO ESTIMATE STRUCTURAL EROSION RISK DISTANCE AND STRUCTURAL SETBACK DISTANCE



STRUCTURAL EROSION RISK DISTANCE=NET STABLE SLOPE DISTANCE

STRUCTURAL SETBACK DISTANCE=STRUCTURAL EROSION RISK DISTANCE+MINIMUM FACILITY SETBACK DISTANCE

WHERE: NET STABLE SLOPE DISTANCE=GROSS STABLE SLOPE DISTANCE-EXISTING HORIZONTAL BLUFF SLOPE DISTANCE

$$\text{GROSS STABLE SLOPE DISTANCE} = \frac{\text{BLUFF HEIGHT}}{\text{TAN } 22^\circ} = \frac{\text{BLUFF HEIGHT}}{0.4}$$

MINIMUM FACILITY SETBACK DISTANCE: INTENDED TO PROVIDE A SAFETY ZONE, PROVIDE AESTHETIC BENEFITS, AND ALLOW PROVISION OF FUTURE SURFACE WATER AND GROUNDWATER DRAINAGE SYSTEMS

Source: SEWRPC.

where a fill project is used to stabilize the bluff slope, a slope stability analysis should be conducted to determine the stable slope angle. A minimum facility setback distance of 50 feet would be recommended for all permanent buildings and facilities.

In addition to setbacks for new urban development and redevelopment, recommendations—and regulations where appropriate—may address shoreline erosion problems related to urban land under construction and to stormwater management. The following provisions would aid in controlling soil erosion and excessive stormwater runoff within the study area during construction of urban development:

1. Development within the study area would be limited to outside the identified structural or nonstructural setback distances.

2. Plans for development would be prepared on 1 inch equals 100 feet scale topographic maps to identify the existing ground surface; to identify areas with steep slopes; to propose and estimate street grades and profiles; to aid in the design of gutters, storm sewers, open drainage channels, water diversions, drainage easements, and soil erosion control practices; and to show the type and location of shoreline erosion control measures.

3. Plans for development would indicate the suitability of soils for development and identify areas covered by highly erodible soils.

4. Provisions would be made to effectively accommodate the stormwater runoff under the changed soil and surface conditions during construction which may aggravate shoreline erosion problems.

5. During construction, the smallest practicable area of soil would be exposed at any given time.

6. Such soil exposure during construction would be kept to as short a duration of time as is practicable.

7. Temporary vegetation, mulching, or other cover would be used to protect critical areas, and permanent vegetation would be installed as soon as practicable.

8. Adequate provisions would be taken to minimize the tracking or dropping of dirt or other materials from the site onto any public or private street.

The following provisions would aid in controlling stormwater runoff within the study area following completion of the development:

1. Stormwater drainage systems would consist of both a "minor" system and a "major" system. The minor stormwater drainage system would consist of engineered paths for the stormwater runoff during a more frequent storm event—one with a recurrence interval of up to 10 years. Minor stormwater drainage components include storm sewers and drainage ditches.

   The major stormwater drainage system would be designed for conveyance of stormwater runoff during a very infrequent storm event—one with a recurrence interval of up to 100 years. Major stormwater drainage components include streets and drainageways.

2. Provisions would be made to prevent surface stormwater runoff from being discharged uncontrolled over the top of the bluff, and to prevent runoff from damaging bluff toe protection measures by eroding soil behind the structures or by creating excessive hydrostatic pressures behind the structures.

3. The stormwater drainage systems would be carefully adjusted to the topography of the land in order to minimize grading and drainage problems, although modifications may be needed to prevent surface stormwater runoff from being discharged over the top of the bluff.

4. Provisions would be made to accommodate effectively the increased peak flows and volumes of stormwater runoff resulting from the addition of impervious surfaces to the study area.

5. Stormwater storage measures such as detention ponds and parking lot or rooftop storage devices—which could cause increased infiltration and groundwater seepage and add excessive weight too close to the top of the bluff—would not be utilized if such measures could threaten the stability of the bluff slope.

<u>Regulation of Lake Michigan Water Levels</u>

Regulation of Great Lakes water levels has been proposed as one method to alleviate increased shoreline erosion caused by high water levels. The regulation could be accomplished by increased dredging of the Lake Michigan outlet channels, by modification of existing diversions into and out of Lakes Michigan and Superior, and by construction of new diversions.

There are five major artificial diversions on the Great Lakes, which change the natural supply of water to the lake or which permit water to bypass a natural lake outlet. These are the Long Lac, Ogoki, and Chicago diversions, the Welland Canal, and the New York State Barge Canal.

Although they are separate diversions, the Ogoki and Long Lac diversions are frequently considered together because they both divert into Lake Superior water from the Albany River Basin that would otherwise drain to Hudson Bay. Completed in 1941, the Long Lac diversion connects the headwaters of the Kenogami River with the Aguasabon River, which flows into Lake Superior. Completed in 1943, the Ogoki diversion diverts water from the Ogoki River to Nipigon Lake, which is located in the Lake Superior Basin. These diversions were developed for the purpose of generating hydroelectric power. The Long Lac diversion was also developed to help transport pulpwood logs southward.

The combined average flow for the Long Lac and Ogoki diversions is about 5,600 cubic feet per second (cfs). This diversion can be compared with the annual average outflow from Lake Superior of 76,000 cfs for the period 1900 to 1986.

It should be noted that the diversion of water from the Ogoki River was temporarily reduced or stopped during the high-water periods of 1951 through 1953 and 1972 through 1974, and, most recently, in 1985. The 1985 reduction is estimated to have caused about a 0.03 foot reduction in the level of Lake Superior,[3] and an insignificant reduction in the water level of Lake Michigan.

Water has been diverted from Lake Michigan through the Chicago diversion since 1848. This diversion serves to dilute sewage effluent from the Chicago Sanitary District and divert the effluent from Lake Michigan. The diversion also facilitates navigation on the Chicago Sanitary and Ship Canal and hydroelectric power generation in Illinois. The rate of flow is subject to the jurisdiction of the U. S. Supreme Court, the current average authorized flow being 3,200 cfs.

The Welland Canal diverts water from Lake Erie across the Niagara Peninsula to Lake Ontario, thereby bypassing the Niagara River and Niagara Falls, primarily for navigation and hydroelectric power generation. The canal was originally built in 1829 and has been modified and realigned several times. The rate of flow through the canal is about 9,200 cfs.

The New York State Barge Canal diverts water primarily for navigation purposes from the Niagara River at Tonawanda, New York, ultimately discharging it to Lake Ontario. The rate of flow varies seasonally; the average rate is estimated to be 700 cfs and the maximum rate during the navigation season is estimated to be 1,100 cfs.

The effects of these diversions, other than the New York State Barge Canal, on Great Lakes water levels—as estimated by the International Great Lakes Diversions and Consumptive Uses Study Board of the International Joint Commission—is indicated in Table 54. The New York State Barge Canal, it should be noted, has a negligible effect on the water levels of the Great Lakes.

Water levels in the Great Lakes can be partially regulated by means of artificial outlet control structures. Currently, two of the Great Lakes, Superior and Ontario, are partially regulated under plans approved by the International Joint Commission. The regulation of Lake Superior affects the entire Great Lakes system, whereas the regulation of Lake Ontario does not affect the other lakes because of the sheer drop in water level at Niagara Falls. The outflow from Lake Superior is currently governed by Regulation Plan 1977. The basic objective of that plan is to balance the levels of Lake Superior and Lakes Michigan-Huron, maximizing benefits for riparian, navigation, and power generation interests.

Any reduction in high lake levels would help reduce the degree and severity of shoreline erosion. However, the diversion or outlet modifications needed to achieve a significant decline in lake levels would be very expensive, and there would be concerns that the increased outflow of water from Lake Michigan and Lake Huron could adversely affect the shipping and hydroelectric industries and could lead to increased flooding downstream of some of the diversions.

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[3]*Great Lakes Commission, Water Level Changes—Factors Influencing the Great Lakes, 1986.*

Table 54

ESTIMATED EFFECT OF EXISTING DIVERSION RATES ON GREAT LAKES WATER LEVELS

Diversion	Rate (cfs)	Effect on Mean Water Level (feet)			
		Lake Superior	Lakes Michigan-Huron	Lake Erie	Lake Ontario
Long Lac/Ogoki	5,600	0.21	0.37	0.25	0.22
Lake Michigan at Chicago	3,200	-0.07	-0.21	-0.14	-0.10
Welland Canal	9,400[a]	-0.06	-0.18	-0.44	0

[a]*The effects on lake levels were evaluated for a rate of 9,400 cfs, slightly higher than the current rate of 9,200 cfs. An evaluation based upon the current rate would yield similar results.*

Source: International Great Lakes Diversions and Consumptive Uses Study Board of the International Joint Commission.

A related study focusing on measures to alleviate the high water level crisis existing in 1985 and 1986 was initiated in 1986, and completed in October 1988 by a task force composed of International Joint Commission staff and specialists.[4] The report was limited to those measures which could be evaluated and implemented within approximately two years and would not require significant new structural works. The measures evaluated to reduce Lake Michigan water levels included increasing the storage of Lake Superior; modifying river diversions, such as closing the Long Lac and Ogoki diversions, and increasing the rate of the Chicago and Welland Canal diversions; increasing Lake Erie outflows; modifying flows in the St. Clair and Detroit Rivers; and improving flows under the ice cover in the St. Clair River. The report estimated that by implementing all of the potential measures investigated, Lake Michigan water levels would decrease by only about 1.2 feet after two years, and by about 1.5 feet after five years. These results were confirmed by a study conducted by the U. S. National Oceanic and Atmospheric Administration Great Lakes Environmental Research Laboratory, which noted that eliminating the Long Lac and Ogoki diversions and increasing the Chicago and Welland Canal diversions could be expected to reduce Lake Michigan water levels by only 0.8 foot after eight years, with half of that lowering occurring within two to three years.[5]

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[4]*International Joint Commission, Interim Report on 1985-86 High Water Levels in the Great Lakes-St. Lawrence River Basin, 1988. Any diversion of water into or out of the Lake Michigan basin presents a complex issue having social, economic, environmental, political, and legal, as well as technical, ramifications. The International Joint Commission is currently engaged in a major study of the diversion issue in the Great Lakes basin. It is anticipated that this ongoing study will address these issues.*

[5]*Holly C. Hartmann, Potential Variation of Great Lakes Water Levels: A Hydrologic Response Analysis, National Oceanic and Atmospheric Administration, Great Lakes Environmental Research Laboratory, Ann Arbor, Michigan, 1987.*

ALTERNATIVE SHORELINE EROSION MANAGEMENT PLANS

Alternatives were developed for two plan elements: a bluff slope stabilization element and a shoreline protection plan element. Each of the alternative plans, described in more detail below, was designed to protect the entire shoreline of Milwaukee County. An estimate of the total capital cost and annual maintenance cost of each plan was developed. To facilitate the comparison of the alternative plans on an economic basis, the present worth and the equivalent annual cost—or the equivalent present worth of a series of future expenditures—were also developed. An economic analysis period of 50 years and an interest rate of 6 percent were used in the economic analyses.

Similar alternative bluff slope stabilization and shoreline protection plans—as well as a recommended plan—for northern Milwaukee County were set forth in SEWRPC Community Assistance Planning Report No. 155.[6] In order to develop conceptually consistent alternatives for the entire county shoreline, the alternatives previously developed for northern Milwaukee County have been incorporated into the alternative plans presented herein. Since these alternatives were thoroughly evaluated in SEWRPC Community Assistance Planning Report No. 155, the recommended plan set forth in that report for northern Milwaukee County has been incorporated into the recommended plan presented in this report.

Bluff Slope Stabilization Plan Element

The preliminary bluff slope stabilization plan identifies those measures needed to fully stabilize the bluff slopes along the entire shoreline of the County. Measures needed may include regrading and revegetating the bluff face, controlling surface water runoff, and reducing groundwater seepage from the bluff face. The preliminary bluff slope stabilization plan element is illustrated on Map 38.

This plan element, as a systems level plan, identifies those factors that need to be controlled to stabilize the bluff slopes. How best to control these factors, and the specific types of control measure to be used, however, require a site-specific preliminary engineering analysis. Accordingly, the study did not evaluate alternative methods of controlling the factors contributing to bluff slope instability.

Bluff slopes may be regraded by cutting back the top of the bluff, or by placing fill at the toe of the bluff and on the bluff face. A combination of cut and fill may also be used. Cutting back the bluff may help minimize the amount of fill required to stabilize the slope, and reduce the disruption of the natural aesthetic properties and drainage characteristics of the bluff slope where bluff regrading is indicated as a control measure. The appropriate method for regrading the bluff slope within a particular bluff analysis section should be selected based on more site-specific data and analyses. The distance from the existing houses and other structures to the edge of the bluff, and the alignment of adjacent shoreline areas should be considered when selecting and designing a bluff slope regrading project. In order to maintain the regraded bluff slope, surface water or groundwater drainage may be needed, as well as topsoil placement, seeding, and mulching to develop a protective vegetative cover.

Typical examples of stabilizing a bluff slope by using alternative methods of bluff slope regrading are graphically illustrated in Figure 105. Alternative methods of bluff regrading—fill, cutback, and cut and fill—are shown for the bluff slopes at Profile Site No. 90, in Bluff Analysis Section 88, within the Village of Fox Point, and at Profile Site No. 17, in Bluff Analysis Section 14, within the City of South Milwaukee. The lowest safety factor calculated for the existing bluff slope at Profile Site No. 90 was 0.82, and the lowest safety factor calculated at Profile Site No. 17 was 0.74, indicating unstable slope conditions at both sites. To help ensure stabilization of the bluff slopes, surface water and groundwater drainage systems could be installed, and the bluff slopes revegetated.

The criteria used to select the bluff stabilization measures and the estimated cost of each stabilization measure are set forth in Table 55. Bluff slopes would be regraded to a stable angle and

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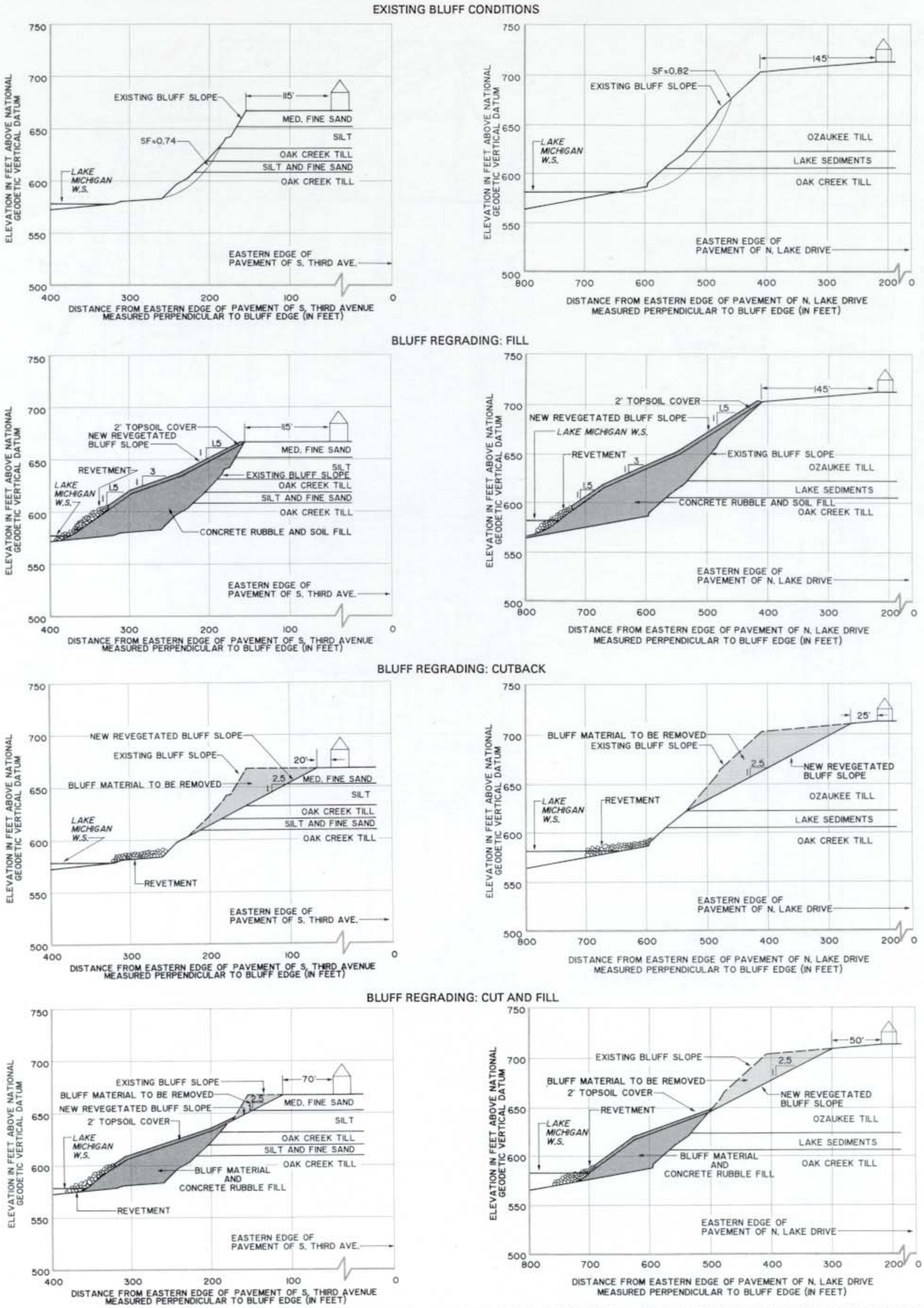
[6] *SEWRPC Community Assistance Planning Report No. 155, A Lake Michigan Shoreline Erosion Management Plan for Northern Milwaukee County, Wisconsin, December 1988.*

Figure 105

EXAMPLES OF ALTERNATIVE BLUFF SLOPE REGRADING TECHNIQUES

PROFILE 17, BLUFF ANALYSIS SECTION 14,
CITY OF SOUTH MILWAUKEE

PROFILE 90, BLUFF ANALYSIS SECTION 88,
VILLAGE OF FOX POINT

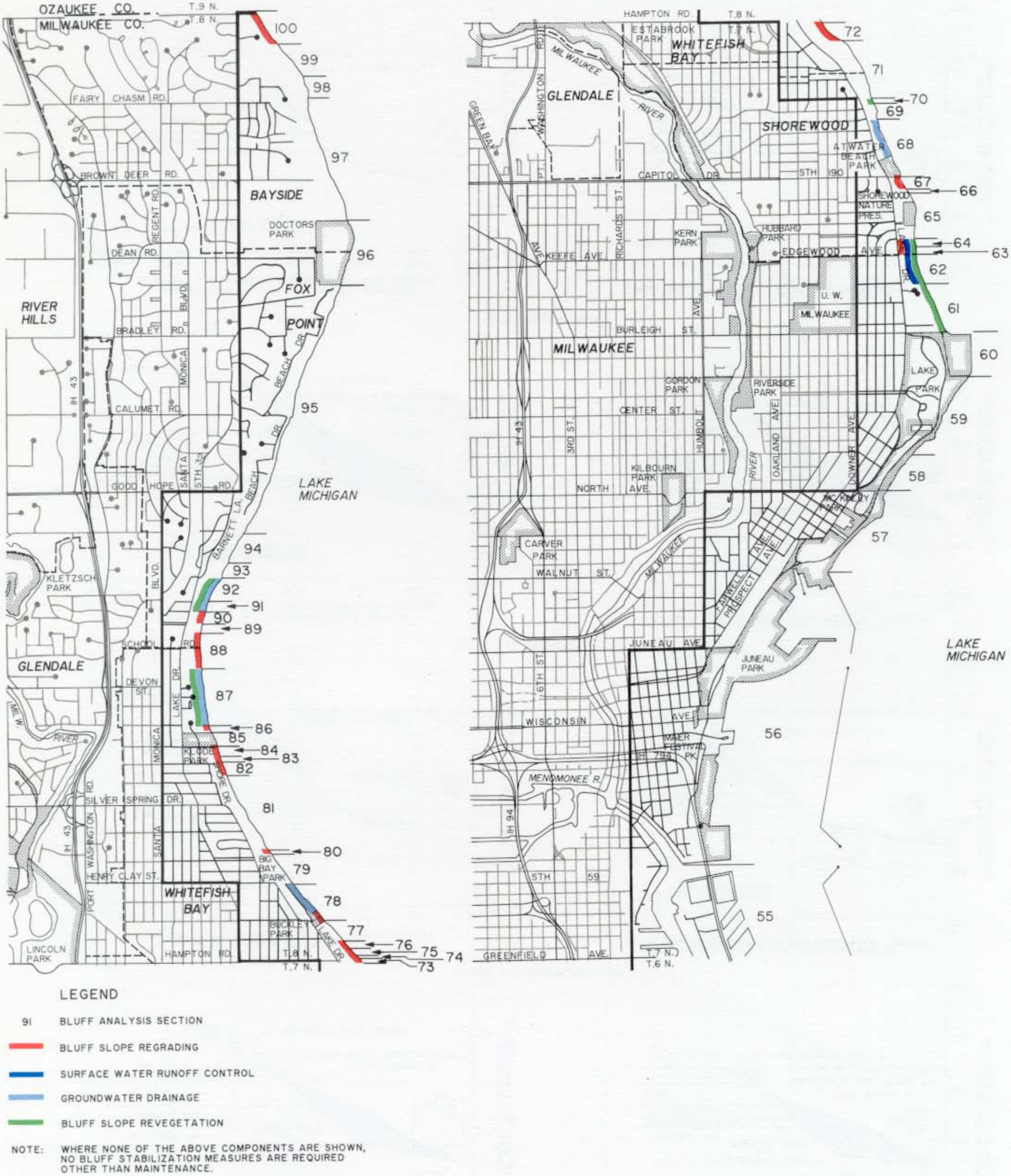


NOTE: SURFACE WATER RUNOFF CONTROL, GROUNDWATER DRAINAGE, AND BLUFF SLOPE REVEGETATION WOULD ALSO HELP ENSURE THE STABILITY OF THE REGRADED BLUFF SLOPE.

Source: SEWRPC.

Map 38

PRELIMINARY BLUFF STABILIZATION PLAN ELEMENT FOR MILWAUKEE COUNTY



LEGEND

91 BLUFF ANALYSIS SECTION

BLUFF SLOPE REGRADING

SURFACE WATER RUNOFF CONTROL

GROUNDWATER DRAINAGE

BLUFF SLOPE REVEGETATION

NOTE: WHERE NONE OF THE ABOVE COMPONENTS ARE SHOWN, NO BLUFF STABILIZATION MEASURES ARE REQUIRED OTHER THAN MAINTENANCE.

Map 38 (continued)

Source: SEWRPC.

Table 55

SELECTION CRITERIA AND TYPICAL CAPITAL AND MAINTENANCE UNIT COSTS OF BLUFF STABILIZATION PLAN COMPONENTS

Plan Component	Criteria for Selection	Typical Unit Cost ($/lineal foot of shoreline unless otherwise indicated)	
		Total Capital	Annual Maintenance
Groundwater Drainage	Areas where lowering the elevation of the groundwater may be expected to significantly help stabilize the bluff slopes	50	10
Surface Water Runoff Control	Areas where specific surface water drainage problems were identified which significantly affected slope stability	10	2
Revegetation	Areas where lack of vegetation could cause translational sliding	350/1,000 ft^2	10/1,00 ft^2[a]
Regrading Bluff	Areas were slope regrading needed to stabilize slope. Often, groundwater and surface water drainage and bluff slope revegation are also required when a slope is regraded	150	15[a]

[a]*Annual maintenance costs would apply only for the first three years following construction of the bluff slope stabilization method.*

Source: SEWRPC.

revegetated along about 44,270 feet of shoreline, or about 28 percent of the total county shoreline. Detailed studies to determine the feasibility of installing groundwater drainage systems along about 10,200 feet of shoreline, or about 6 percent of the total county shoreline, would be conducted. Surface water runoff control would be provided along about 4,360 feet of shoreline, or about 3 percent of the county shoreline. Revegetation of at least a portion of the bluff face without bluff slope regrading would be provided along about 11,060 feet of shoreline, or about 7 percent of the county shoreline.

The preliminary recommended plan components and estimated cost of bluff slope stabilization are listed for each bluff analysis section in Table 56. The bluff stabilization plan element would have a capital cost of about $7.4 million, and an average annual maintenance cost of about $0.8 million. About 86 percent of the maintenance cost, however, would be required only during the first three years following construction.

In addition to the structural control measures recommended for bluff slope stabilization, communities where new urban development and redevelopment is expected to occur would protect the development from excessive shoreline erosion and bluff recession by identifying the distance from the existing bluff edge that is subject to erosion damages, and by specifying a setback distance which restricts or prohibits the location of buildings and other land uses that are vulnerable to damages or destruction from erosion. These regulations can be readily incorporated into existing municipal zoning ordinances which regulate the use of land, the area and dimensions of lots, and the location of buildings and facilities on such lots. Zoning can also control grading, filling, vegetation removal, and certain other land management practices. To be constitutionally valid, however, regulation of the land use within the setback distances must serve valid public objectives, leave the property owner with some reasonable use of the property, and provide sufficient standards to prevent arbitrary decision-making.

New urban development along the Lake Michigan shoreline may be expected to occur within the Cities of Oak Creek, South Milwaukee, and St. Francis. Under the bluff stabilization plan element, amendments would be incorporated into the existing zoning ordinances of these communities which would, in the public interest, regulate land uses, activities, and facility locations within the specified setback distances. The amendments would include provisions defining pertinent terms, designating the lands to be regulated, specifying the necessary regulation of land use and facility location, specifying the regulation of certain land disturbance activities, and describing procedures for modifying the location and extent of the designated setback distances. It is further recommended that these communities establish construction erosion control ordinances addressing land development activities related to the control of erosion and stormwater runoff. Suggested provisions were set forth in the previous section of this chapter. A suggested model ordinance is presented in "Construction Site Erosion Control Model Ordinance," prepared jointly by the League of Wisconsin Municipalities and the Wisconsin Department of Natural Resources in 1987. The Regional Planning Commission would, upon request, assist these communities in incorporating into the zoning and subdivision ordinance provisions related to erosion risk, associated setback distances, and land development activities along the Lake Michigan shoreline.

Significant shoreline development or redevelopment is not expected to occur within the Cities of Cudahy and Milwaukee and the Villages of Bayside, Fox Point, Shorewood, and Whitefish Bay. Therefore, it is not necessary for these communities to develop regulations related to setback distances or land development activities. Instead, under the bluff stabilization plan element, these communities would consider the setback distances described in Figures 103 and 104 and the land development activity recommendations as advisory in the administration of their zoning and subdivision control ordinances. It is further recommended under this plan element that Milwaukee County consider the setback and land development provisions as advisory in the management of county parkland and other lakefront properties, and furthermore, that the County abide by any local regulations in effect.

The costs of administering the proposed regulations and guidelines are not included in the plan costs. It was assumed that such costs would be borne as part of the normal municipal operations. It is not anticipated that additional local staff will be required to administer these regulations and guidelines.

Shoreline Protection Plan Element

The shoreline protection plan represents the second element of the recommended plan and of alternatives thereto. Three conceptual alternative plans were developed to protect the county shoreline from wave and ice erosion.

The first conceptual alternative would utilize revetments wherever practicable to protect the shoreline. For systems level planning purposes, it was assumed that the revetments would be constructed of quarry stone, although other types of revetments could also be used. The revetment alternative would have a relatively low cost.

The second conceptual alternative for protection of the county shoreline would provide, wherever practicable, artificially nourished beach systems with either onshore or near-shore structures being used to help maintain the beaches. The beach alternative would provide a usable beach, in most instances composed of gravel, for a large portion of the study area shoreline. Sand beaches would be provided where public swimming is desired, and in some locations where a limited sand beach now exists. For the purposes of the systems level planning, it was assumed that short groins constructed of quarry stone would be used to help contain the beach material along most of the beach areas, but other structures—notably steel sheet pile groins, armored headlands, and near-shore stone reefs—could also be used. The beach alternative would have a relatively moderate cost.

The third conceptual alternative for protection of the county shoreline would utilize offshore peninsulas, islands, and breakwaters—along

Table 56

PRELIMINARY BLUFF STABILIZATION PLAN FOR MILWAUKEE COUNTY

Bluff Analysis Section	Shoreline Length (feet)	Plan Component	Cost per Lineal Foot		Total Cost			
			Capital	Annual Maintenance	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual Cost
1	4,470	None	$ - -	$ - -	$ - -	$ - -	$ - -	$ - -
2	2,820	Groundwater drainage, surface water runoff control	60	12	169,000	34,000	703,000	44,000
3	2,930	Bluff slope regrading[a]	150	15	440,000	44,000[b]	557,000	35,000
4	1,980	Bluff slope regrading	150	15	297,000	30,000[b]	376,000	22,000
5	1,070	Bluff slope regrading	150	15	161,000	16,000[b]	204,000	13,000
6	1,170	Bluff slope regrading	150	15	176,000	18,000[b]	223,000	14,000
7	1,000	Bluff slope regrading	150	15	150,000	15,000[b]	190,000	12,000
8	540	None	- -	- -	- -	- -	- -	- -
9	570	Bluff slope regrading	150	15	84,000	9,000[b]	109,000	7,000
10	400	Bluff slope regrading	150	15	60,000	6,000[b]	76,000	5,000
11	1,290	Bluff slope regrading	150	15	194,000	19,000[b]	245,000	16,000
12	3,160	None	- -	- -	- -	- -	- -	- -
13	1,320	None	- -	- -	- -	- -	- -	- -
14	1,310	Bluff slope regrading	150	15	197,000	20,000[b]	249,000	16,000
15	790	Bluff slope regrading	150	15	119,000	12,000[b]	150,000	10,000
16	470	None	- -	- -	- -	- -	- -	- -
17	440	Bluff slope regrading	150	15	66,000	7,000[b]	84,000	5,000
18	1,880[a]	Bluff slope regrading	150	15	282,000	28,000[b]	357,000	23,000
19	1,500	None	- -	- -	- -	- -	- -	- -
20	1,280	Bluff slope regrading	150	15	192,000	19,000[b]	243,000	15,000
21	1,060	Groundwater drainage	50	10	53,000	11,000	220,000	14,000
22	950	Bluff slope regrading	150	15	143,000	14,000[b]	181,000	12,000
23	1,200	Bluff slope regrading	150	15	180,000	18,000[b]	228,000	15,000
24	1,910	Bluff slope regrading	150	15	287,000	29,000[b]	363,000	23,000
25	880	Bluff slope regrading	150	15	132,000	13,000[b]	167,000	11,000
26	660	Bluff slope regrading	150	15	99,000	10,000[b]	126,000	8,000
27	1,850	Bluff slope regrading	150	15	278,000	28,000[b]	352,000	22,000
28	2,050	Bluff slope regrading	150	15	308,000	31,000[b]	390,000	25,000
29	770	Bluff slope regrading	150	15	116,000	12,000[b]	147,000	9,000
30	1,760	Bluff slope regrading	150	15	264,000	26,000[b]	335,000	21,000
31	600	Bluff slope regrading	150	15	90,000	9,000[b]	114,000	7,000
32	340	None	- -	- -	- -	- -	- -	- -
33	2,060	Bluff slope regrading	150	15	309,000	36,000[b]	392,000	25,000
34	1,780	Bluff slope revegetation	15	3	27,000	5,000[b]	41,000	3,000
35	650	Groundwater drainage, bluff slope revegetation	65	13	43,000	9,000[c]	150,000	10,000
36	710	Bluff slope revegetation	15	3	11,000	2,000[b]	16,000	1,000
37	1,010	Bluff slope regrading	150	15	152,000	15,000[b]	192,000	12,000
38	1,290	Bluff slope regrading	150	15	194,000	19,000[b]	245,000	16,000
39	1,480	Bluff slope regrading	150	15	222,000	22,000[b]	281,000	18,000
40	820	None	- -	- -	- -	- -	- -	- -
41	1,650	None	- -	- -	- -	- -	- -	- -
42	940	Bluff slope revegetation	15	3	14,000	3,000[b]	22,000	1,000
43	1,370	Bluff slope regrading	150	15	206,000	21,000[b]	261,000	17,000
44	140	Bluff slope regrading	150	15	21,000	2,000[b]	27,000	2,000
45	80	Bluff slope regrading	150	15	12,000	1,000[b]	15,000	1,000
46	360	Bluff slope regrading	150	15	54,000	5,000[b]	68,000	4,000
47	2,470	Surface water runoff control, revegetation	25	5	66,000	12,000[c]	160,000	10,000
48	1,420	Revegetation	15	3	21,000	4,000[b]	33,000	2,000
49	340	None	- -	- -	- -	- -	- -	- -
50	1,130	Revegetation	15	3	17,000	3,000[b]	26,000	2,000
51	570	None	- -	- -	- -	- -	- -	- -
52	450	None	- -	- -	- -	- -	- -	- -
53	1,320	None	- -	- -	- -	- -	- -	- -
54	1,360	None	- -	- -	- -	- -	- -	- -
55	14,750	None	- -	- -	- -	- -	- -	- -
56	16,060	None	- -	- -	- -	- -	- -	- -
57	3,210	None	- -	- -	- -	- -	- -	- -
58	1,900	None	- -	- -	- -	- -	- -	- -
59	3,540	None	- -	- -	- -	- -	- -	- -
60	2,210	None	- -	- -	- -	- -	- -	- -
61	1,970	Bluff slope revegetation	10	2	20,000	4,000[b]	30,000	2,000

Table 56 (continued)

Bluff Analysis Section	Shoreline Length (feet)	Plan Component	Cost per Lineal Foot		Total Cost			
			Capital	Annual Maintenance	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual Cost
62	950	Bluff slope revegetation, surface water runoff control	$ 15	$ 3	$ 14,000	$ 3,000[b]	$ 22,000	$ 1,000
63	300	Bluff slope revegetation, surface water runoff control, bluff slope regrading	100	10	30,000	3,000[b]	38,000	2,000
64	290	Bluff slope revegetation, surface water runoff control, bluff slope regrading	80	10	23,000	2,000[b]	31,000	2,000
65	1,710	None	- -	- -	- -	- -	- -	- -
66	170	None	- -	- -	- -	- -	- -	- -
67	380	Bluff slope regrading	150	15	57,000	6,000[b]	72,000	5,000
68	790	None	- -	- -	- -	- -	- -	- -
68	1,380	Groundwater drainage	50	10	69,000	14,000	287,000	18,000
69	520	None	- -	- -	- -	- -	- -	- -
70	240	Bluff slope revegetation	15	3	4,000	1,000[b]	6,000	1,000
71	2,370	None	- -	- -	- -	- -	- -	- -
72	850	Bluff slope regrading	150	15	128,000	13,000[b]	162,000	10,000
73	190	Bluff slope regrading	150	15	29,000	3,000[b]	36,000	2,000
74	160	Bluff slope regrading	150	15	24,000	2,000[b]	30,000	2,000
75	310	Bluff slope regrading	150	15	47,000	5,000[b]	59,000	4,000
76	360	Bluff slope regrading	150	15	54,000	5,000[b]	68,000	4,000
77	810	None	- -	- -	- -	- -	- -	- -
78	600	Bluff slope regrading	150	15	90,000	9,000[b]	114,000	7,000
	1,060	Groundwater drainage	50	10	53,000	11,000	220,000	14,000
79	1,480	None	- -	- -	- -	- -	- -	- -
80	130	Bluff slope regrading	100	10	13,000	1,000[b]	16,000	1,000
81	2,970	None	- -	- -	- -	- -	- -	- -
82	490	Bluff slope regrading	150	15	74,000	7,000[b]	93,000	7,000
83	140	Bluff slope regrading	150	15	21,000	2,000[b]	26,600	2,000
84	430	Bluff slope regrading	150	15	65,000	7,000[b]	82,000	5,000
85	480	None	- -	- -	- -	- -	- -	- -
86	170	Bluff slope regrading	150	15	26,000	3,000[b]	32,000	2,000
87	1,950	Groundwater drainage, bluff slope revegetation	60	12	117,000	23,000[c]	435,000	28,000
88	1,150	Bluff slope regrading	150	15	173,000	17,000	219,000	14,000
89	320	None	- -	- -	- -	- -	- -	- -
90	470	Bluff slope regrading	150	15	71,000	7,000[b]	90,000	6,000
91	510	Groundwater drainage, bluff slope revegetation	65	12	33,200	6,000[c]	116,000	7,000
92	770	Groundwater drainage bluff slope revegetation	80	16	62,000	12,000[c]	195,000	12,000
93	530	None	- -	- -	- -	- -	- -	- -
94	1,460	None	- -	- -	- -	- -	- -	- -
95	9,070	None	- -	- -	- -	- -	- -	- -
96	1,890	None	- -	- -	- -	- -	- -	- -
97	4,660	None	- -	- -	- -	- -	- -	- -
98	860	None	- -	- -	- -	- -	- -	- -
99	1,280	None	- -	- -	- -	- -	- -	- -
100	1,320	Bluff slope regrading	150	15	198,000	20,000[b]	251,000	16,000
Total	159,110	- -	- -	- -	$7,401,000	$823,000[d]	$11,048,000	$702,000

[a]*Bluff slope regrading includes the placement of a soil cover and revegetation of the slope. Depending on the site-specific conditions, surface water and/or groundwater drainage control may also be required.*

[b]*Annual maintenance costs would apply for first three years following bluff slope regrading or revegetation.*

[c]*Of the total maintenance cost of $12,400 for Bluff Analysis Section 47, $7,410, or 60 percent, would be required only for the first three years following revegetation. Of the total maintenance cost of $23,400 for Bluff Analysis Section 87, $3,900, or 17 percent would be required only for the first three years following revegetation of the total maintenance cost of $6,100 for Bluff Analysis Section 91, $1,000, or 16 percent, would be required only for the first three years following revegetation of the total maintenance cost of $12,300 for Bluff Analysis Section 92, $4,600, or 37 percent, would be required only for the first three years following revegetation.*

[d]*About $706,000, or 86 percent, of the total maintenance cost would be required only for the first three years following bluff slope regrading or revegetation.*

Source: SEWRPC.

with some onshore structures—to protect the shoreline. The existing harbor and South Shore breakwaters would be replaced by these new offshore structures. This alternative would create more than 1,300 acres of new land for recreational uses. The offshore alternative would have a relatively high cost.

The three conceptual alternative plans to protect the entire Milwaukee County shoreline include recommendations as necessary and appropriate to modify existing major shore protection structures to abate wave overtopping damage. For those major structures that are expected to have a moderate or high potential for wave overtopping damage under a 100-year recurrence interval instantaneous maximum water level with a 20-year recurrence interval storm wave, and that are not recommended to be replaced by new structures, the available methods of abating overtopping damage set forth in Table 57 were reviewed, and an appropriate method was selected and used to estimate costs.

In addition to the alternative plans considered for the entire county shoreline, five separate alternatives were developed for the South Shore breakwater. These alternatives include various combinations of reconstructing, relocating, and demolishing the breakwater. For the purposes of the countywide plan, it was assumed under the revetment and beach alternative plans that the entire South Shore breakwater would be reconstructed to a crest elevation of 588.6 feet NGVD, which is the approximate crest elevation at the far northern end of the breakwater. The selection of this crest elevation is discussed later in this chapter. Under the offshore alternative plan, it was assumed that the South Shore breakwater would be replaced by islands and peninsulas.

Four separate alternatives were developed for the Milwaukee outer harbor breakwater. The costs and benefits of increasing the elevation of the Milwaukee outer harbor breakwater were investigated in Volume Two of SEWRPC Planning Report No. 37, A Water Resources Management Plan for the Milwaukee Harbor Estuary, 1987. The findings of this 1987 Commission study are summarized herein. Under the revetment and beach alternative plans, it was assumed that the outer harbor breakwater would be maintained at its existing elevation. Under the offshore alternative plan, it was assumed that peninsulas and islands would be constructed where the outer harbor breakwater now lies.

In the development of the alternative shoreline protection plans, a number of important assumptions were made concerning local preferences and priorities. It was assumed that the sand beaches would be desired at lakefront parks which historically have contained beaches, and that additional sand beaches would be desired in some areas to provide greater opportunity for swimming. It was further assumed that lakeshore residents of low terrace areas—primarily Bluff Analysis Section 95 in the Village of Fox Point—would oppose any structures that would obstruct the view of the lake from the residences. Finally, it was assumed that most lakeshore and other county residents would desire a usable shoreline—though not necessarily requiring a sand beach.

The potential shore protection measures previously described in this chapter were then screened to determine which types of measures should be specifically included in the alternative plans. Based upon that screening, it was concluded that the construction of new bulkheads should be discouraged except where shoreline uses such as docking facilities require such structures. Bulkheads are generally difficult and costly to maintain; often reflect wave energy which may cause scouring of the lakebed; and generally do not provide an attractive, natural appearance to the shoreline. All of the alternative plans, however, recommend the continued maintenance or modification of some existing bulkheads, especially within the Milwaukee outer harbor. The alternative shoreline protection plans primarily considered the use of quarry stone revetments; gravel beach systems with short groins; sand beaches with long groins or offshore breakwaters; and offshore islands and peninsulas. For the purposes of the systems level planning, it was assumed that these structures would be constructed of stone, sand and gravel, and natural soil and concrete rubble fill material. The maintenance, reconstruction, or demolition of existing shore protection structures is also addressed in the alternative plans.

A variety of shore protection materials and products are commercially available, and some of these systems have been described in this chapter. When properly designed and constructed, these systems may be useful in certain situations. In general, however, structures composed of natural stone material are preferred, being usually more effective, durable, easy to maintain, and aesthetically attractive. Structures constructed of rubber tires or tubes, timber, "plastic

Table 57

ALTERNATIVE METHODS OF ABATING OVERTOPPING DAMAGE TO EXISTING SHORE PROTECTION STRUCTURES

Structure Type	Methods of Abating Overtopping Damage	Advantages	Disadvantages	Capital Cost ($/lineal foot of shoreline)
Revetment	1. Increase height of revetment	High degree of protection provided; relatively easy to construct	Sometimes impractical to build high enough to protect shoreline; in low-lying areas, a high revetment may obstruct the scenic view of the lake	100-400
	2. Increase thickness of armor layer	Relatively easy to construct; more suited for low-lying areas	Structure may extend out into lake; large volume of armor stone needed	200-500
	3. Construct riprap berm in front of revetment	Helps reduce scouring of lakebed caused by wave reflection; increased availability of stone, which usually can be smaller than typical armor stone	More frequent maintenance may be needed because smaller sized stone may be used; structure may extend into lake	250-500
	4. Construct splash apron above revetment	Prevents erosion of the soil above revetment	Additional measures may be needed to prevent structural and shoreline damages within areas severely overtopped	50
	5. Construct properly designed and sized drainage system to safely remove water which overtops the revetment	Prevents erosion, as well as accumulation of water behind the revetment, which could lead to structural failure	Additional measures may be needed to prevent structural and shoreline damages within areas severely overtopped	50
Bulkhead	1. Increase height of bulkhead	Cost of construction is modest; provides a high degree of protection	Usually impractical to increase overall height by more than five feet; obstructs access to, and view of, lake	150-500
	2. Construct riprap berm in front of bulkhead	Riprap also serves as toe protection; relatively easy to construct and maintain	Limits access to shoreline, and navigation near the structure	200-500
	3. Construct recurved concrete wall on top of bulkhead to deflect waves	Controls wind-blown overtopping; more effective than a vertical bulkhead of same height	Ineffective when crest of wave nears, or exceeds, elevation of the of the recurved wall; high cost	500-1,000
	4. Construct splash apron above bulkhead	Prevents erosion of the soil above bulkhead	Additional measures may be needed to prevent structural and shoreline damages within areas severely overtopped	50
	5. Construct properly designed and sized drainage system to safely remove water which overtops the bulkhead	Prevents erosion, and accumulation of water, behind the bulkhead	Additional measures may be needed to prevent structural and shoreline damages within areas severely overtopped	50
Beach	1. Increase width by beach nourishment or by modification of beach containment structures	High degree of protection provided; usable shoreline created; effective against fluctuating water levels	High cost of construction and maintenance	250-1,000
	2. Increase slope and height of beach by nourishing with larger sized particles	Less beach material needed initially and lower maintenance requirements than for a comparable sand beach	Less desirable and usable beach than a sand beach	100-500
	3. Construct splash apron or similar protection above or behind beach	Prevents erosion of the soil above the beach	Additional measures may be needed to prevent structural and shoreline damages within areas severely overtopped	50
	4. Construct properly designed and sized drainage systems to safely remove water which overtops the beach	Prevents erosion, and accumulation of water, above the beach	Additional measures may be needed to prevent structural and shoreline damages within areas severely overtopped	50
Breakwater	1. Increase height of breakwater	High degree of protection provided	High cost; limits view of horizon from shoreline	500-1,500
	2. Increase width of breakwater	Moderate degree of protection provided; will not further limit view of horizon from shoreline	High cost	400-1,000
	3. Construct berm or layer of more permeable armor material	Lower construction cost; will not further limit view of horizon from shoreline	Higher maintenance cost; restricts navigation near structure	250-1,000

Source: SEWRPC.

seaweed," sand bags, small precast concrete units, or gabions do not provide long-term protection and should not be used along the Lake Michigan shoreline. Steel sheet piling is durable, but reflects wave energy which tends to increase bottom scouring. Large interlocking concrete units, concrete blocks, and grout-filled bags are generally not as durable as high-quality quarry stone, but can be used to provide effective shore protection at certain locations.

Geotextile filter cloths are required at the base of most quarry stone shore protection structures to protect against undermining, except where structures are constructed offshore in a water depth greater than three times the maximum wave height, where the anticipated current velocities are too weak to move the average size bed material, or where a structure is constructed directly on bedrock.[7] The nonwoven types made of synthetic fiber mats or machine-punched sheets tend to tear or otherwise lose their filtering capability when placed under stress.[8] Woven filter cloths are usually composed of polypropylene or polyvinylidene chloride. The cloth made of polyvinylidene chloride—usually dark green—is heavier than water and should be used when constructing below the water surface. Polypropylene cloth—usually dark brown—is lighter than water and stiffer, stronger, and less costly than polyvinylidene chloride cloth. Polypropylene cloth should be used for construction above the water surface. Filter cloth with very small pore sizes should not be used. This grade of fabric is almost impermeable to hydraulic transients, and the wave energy causes considerable uplift pressures.[9] Rather, large pore-size filter fabric is preferred. With this grade of fabric, a layer of sand and gravel must be placed over underlying silt or clay soil prior to placement of the fabric.

All structures contained within the alternative shoreline protection plans are identified as either publicly or privately owned. As noted earlier in this chapter, major shore protection structures—most of which are publicly owned—should be designed for a higher level of protection than may be feasible for many private lakefront property owners. The public sector generally has greater financial resources available to construct protection measures than does the private sector, and the public sector is generally more willing and able to provide long-term protection and carry out a long-range maintenance program. For these reasons, the cost estimates shown for the alternative plans are usually slightly higher for the publicly owned structures than for the privately owned structures. However, the costs were varied to reflect known problems and physical conditions, such as the value of the property, the risk of property loss, the threat to human safety, and the condition of the existing structure.

Revetment Alternative Plan: An alternative shoreline protection plan utilizing quarry stone revetments wherever practicable represents a relatively low-cost, basic protection plan. It is recognized that under this plan, the revetments in some locations could be constructed of material other than quarry stone. The revetment alternative plan, as graphically illustrated on Map 39, would include construction or reconstruction of quarry stone revetments for about 87,070 feet—or 16.5 miles—of shoreline, or about 54 percent of the total county shoreline. The size and associated cost of a revetment required to provide adequate protection for a particular bluff analysis section is dependent upon the degree of toe erosion occurring, the existing beach width and near-shore slope, the anticipated wave heights during storms, and the location and value of the facility or building being protected. For systems level cost estimation, it was assumed that new construction of revetments would require about three to five tons of stone per lineal foot of shoreline, and cost \$300 to \$500 per foot of shoreline. Annual maintenance costs were assumed to range from \$10 to \$15 per foot. Lesser amounts of stone would be required for reconstruction of existing revetments because some stone would already be present.

The criteria used in the selection of a revetment alternative plan component, along with the estimated unit cost of each component, are set forth in Table 58. A new revetment would be

[7] *U. S. Army Corps of Engineers, 1984, op. cit.*

[8] *U. S. Army Corps of Engineers, Low Cost Protection, Final Report on the Shoreline Erosion Control Demonstration (Section 54) Program, 1981.*

[9] *Charles Johnson, Coastal Engineer, U. S. Army Corps of Engineers, Chicago, Illinois, Personal Communication, July 27, 1987.*

Table 58

SELECTION CRITERIA AND TYPICAL CAPITAL AND MAINTENANCE UNIT COSTS OF REVETMENT ALTERNATIVE PLAN COMPONENTS

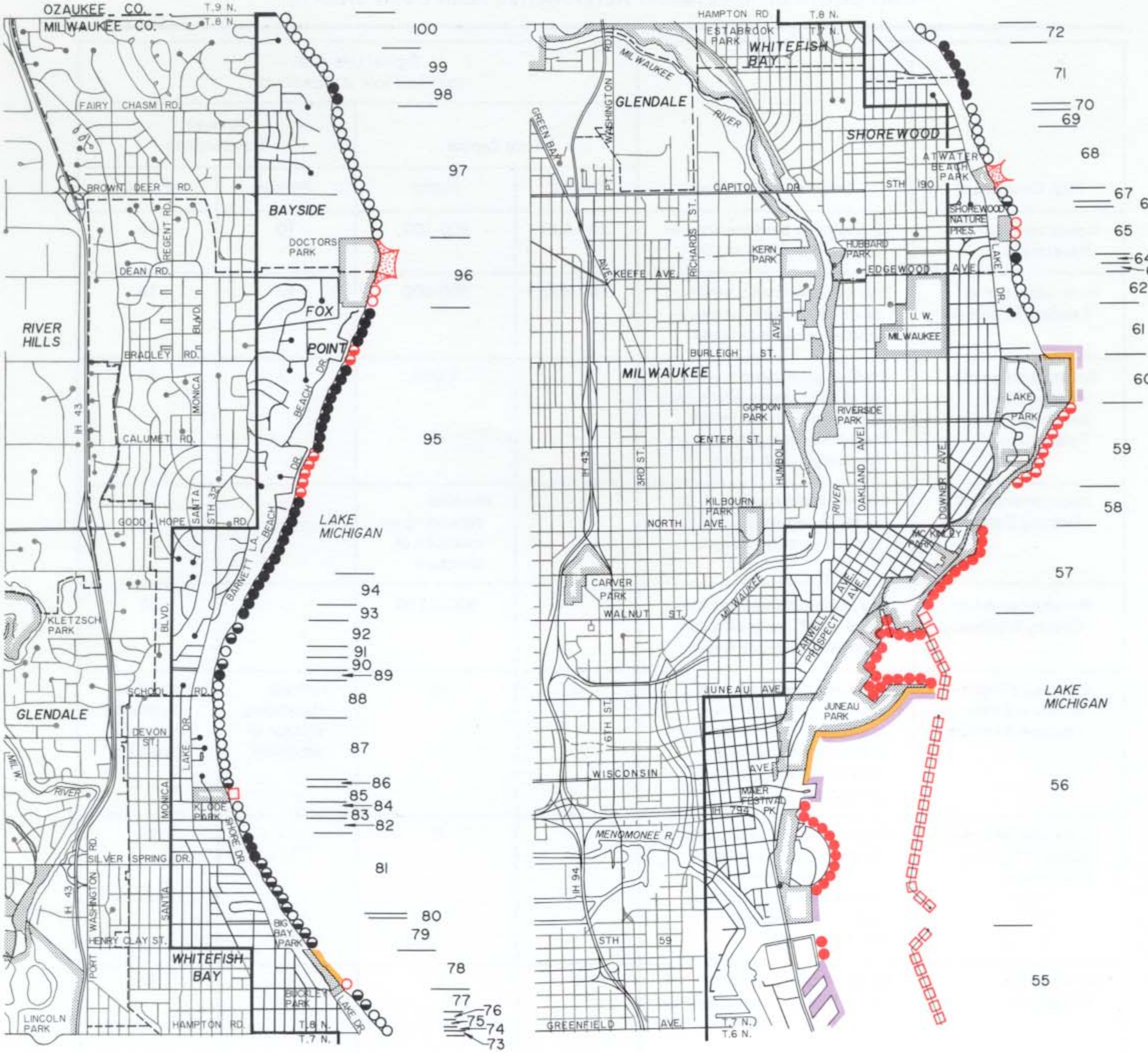
		Typical Unit Cost ($/lineal foot of shoreline)[a]			
		Total Capital		Annual Maintenance	
Plan Component	Criteria for Selection	Private	Public	Private	Public
Construction of New Revetment	Shoreline or bluff toe erosion observed in 1986 or 1987	300-400	400-500	10	15
Reconstruction of Existing Revetment	Existing revetments which, as of 1987, required a substantial amount of repair	200-300	300-400	10	15
Reconstruction of Existing Sand Beach with Groin System	Existing groin system which, as of 1987, required a substantial amount of repair Probable community support for a large sand beach	- -	1,000	- -	30
Reconstruction of Existing Bulkhead	Existing bulkhead which, as of 1987, required a substantial amount of repair	- -	Variable, depending on condition of structure	- -	15
Reconstruction of Existing Breakwater	Existing breakwater which, as of 1987, required a substantial amount of repair	- -	800-1,250	- -	35
Continued Maintenance of Existing Onshore Structure	Onshore structure which was protecting against erosion in 1987, and which, if maintained, could provide continued effective protection	0	0	Variable, depending on type of structure	Variable, depending on type of structure
Continued Maintenance of Existing Breakwater	Breakwater which was protecting against erosion in 1987, and which, if maintained, could provide continued effective protection	- -	0	- -	45
No Shoreline Protection	No significant shoreline or bluff toe erosion observed in 1987 and none expected to occur	0	0	0	0

[a]*Typical unit costs are presented herein. However, costs applicable to individual bluff bluff analysis sections were varied to reflect known problems and physical conditions.*

Source: SEWRPC.

Map 39

REVETMENT ALTERNATIVE PLAN



LEGEND

91 BLUFF ANALYSIS SECTION

CONSTRUCT NEW STRUCTURES

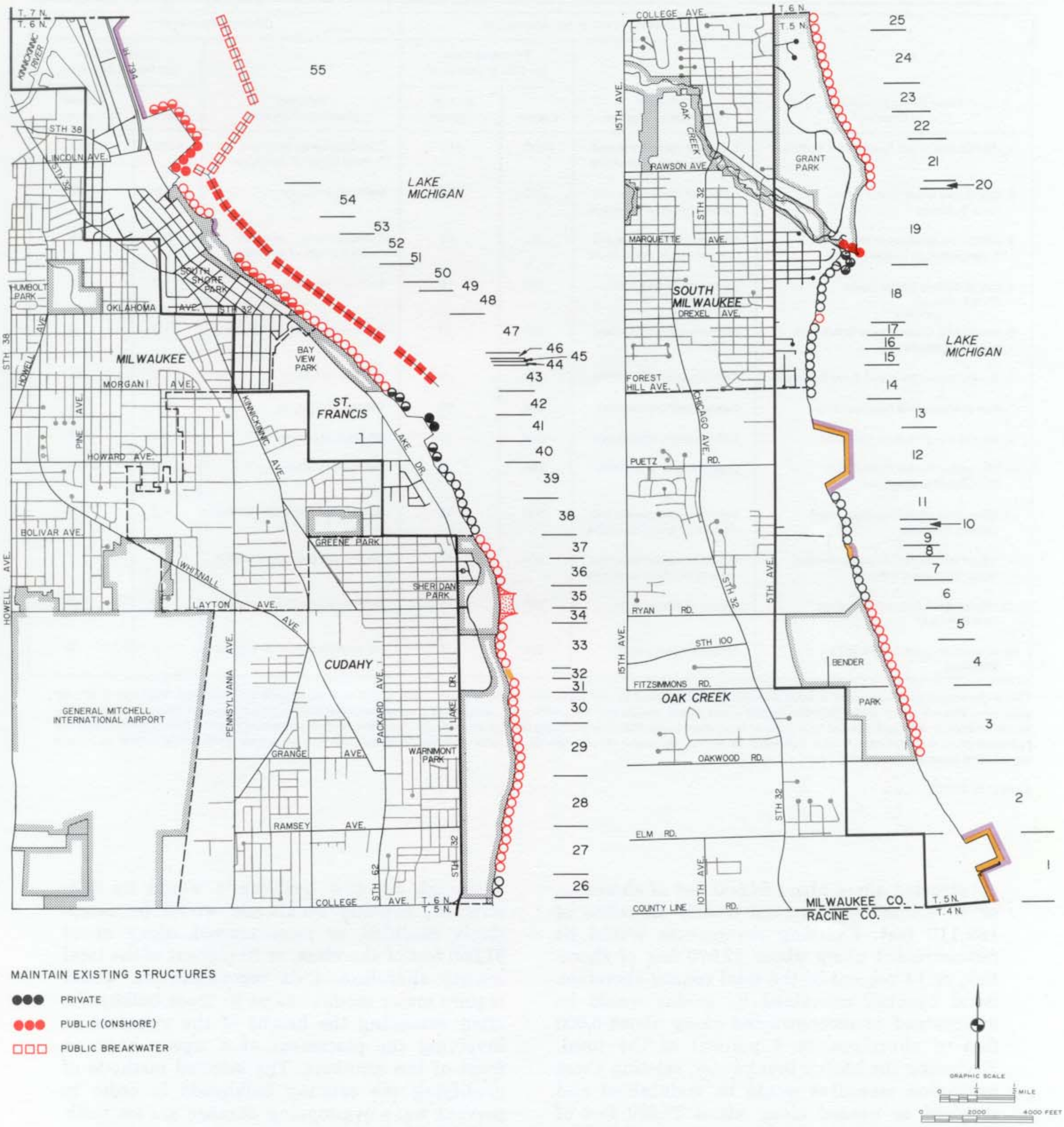
- PRIVATE REVETMENT
- PUBLIC REVETMENT

RECONSTRUCT EXISTING STRUCTURES

- PRIVATE REVETMENT
- PUBLIC REVETMENT

- PUBLIC GROIN SYSTEM WITH SAND BEACH
- PUBLIC BULKHEAD
  - RIP-RAP BERM
  - HEIGHT EXTENSION
  - BERM AND EXTENSION
- PUBLIC BREAKWATER

Map 39 (continued)



Source: SEWRPC.

Table 59

SELECTED METHODS OF MODIFYING EXISTING BULKHEADS TO PREVENT WAVE OVERTOPPING DAMAGE

Major Existing Bulkhead to be Modified[a]	Revetment and Beach Alternatives			Offshore Alternative		
	Selected Modification Method	Estimated Cost per Foot of Shoreline		Selected Modification Method	Estimated Cost per Foot of Shoreline	
		Capital	Annual Maintenance		Capital	Annual Maintenance
1. WEPCo Oak Creek Power Plant Bulkhead	Construct riprap berm and extend height of structure	$800	$15	Construct riprap berm and extend height of structure	$800	$15
2. City of Oak Creek Water Intake Plant Bulkhead	Construct riprap berm and extend height of structure	150	15	Maintain structure	0	10
3. MMSD South Shore Wastewater Treatment Plant Bulkhead	Construct riprap berm and extend height of structure	750	15	Construct riprap berm and extend height of structure	750	15
4. City of Cudahy Water Intake Plant Bulkhead	Construct riprap berm	150	15	Maintain structure	0	10
5. Milwaukee County South Shore Park Marina Bulkhead	Extend height of structure	150	10	No onshore protection needed	0	0
6. South Lincoln Memorial Drive Bulkhead	Extend height of structure	300	15	Maintain structure	0	15
7. Port of Milwaukee Bulkhead Slips	Extend height of structure	500	15	Maintain structure	0	15
8. Marcus Amphitheatre Bulkhead	Extend height of structure	200	15	Maintain structure	0	15
9. Milwaukee Harbor Commission Municipal Pier Bulkhead	Extend height of structure	200	15	Maintain structure	0	15
10. Milwaukee County Juneau Park Landfill Bulkhead	Construct riprap berm and extend height of structure	500	15	No onshore protection needed	0	0
11. City of Milwaukee Linnwood Avenue Water Treatment Plant	Construct riprap berm and extend height of structure	500	15	No onshore protection needed	0	0
12. Village of Whitefish Bay Buckley Park Bulkhead	Construct riprap berm	200	15	No onshore protection needed	0	0
13. Milwaukee County Big Bay Park Bulkhead	Construct riprap berm	200	15	No onshore protection needed	0	0

[a]These bulkheads were estimated to have a moderate to high potential for wave overtopping damage under a 100-year recurrence interval water level with a 20-year recurrence interval storm wave. Only the MMSD Jones Island wastewater treatment plant, Milwaukee County McKinley Marina, and Milwaukee County War Memorial Center bulkheads were estimated to have a low or insignificant potential for overtopping damage under these conditions. It is recommended that the Jones Island wastewater treatment plant and McKinley Marina bulkheads be maintained under all alternatives, and that the War Memorial bulkhead be further protected by a berm to prevent toe scouring under all alternatives.

Source: SEWRPC.

constructed along about 64,500 feet of shoreline, or 40 percent of the total county shoreline of 159,110 feet. Existing revetments would be reconstructed along about 22,570 feet of shoreline, or 14 percent of the total county shoreline. Sand beaches contained by groins would be maintained or reconstructed along about 6,000 feet of shoreline, or 4 percent of the total. Excluding the harbor breakwater, existing shore protection measures would be maintained and repaired as needed along about 27,620 feet of shoreline, or 17 percent of the county shoreline.

Although no new bulkheads would be constructed, existing bulkheads would be extensively modified, or reconstructed, along about 31,050 feet of shoreline, or 20 percent of the total county shoreline. This reconstruction would require major modifications in these bulkheads, often extending the height of the structure or involving the placement of a riprap berm in front of the structure. The selected methods of modifying the existing bulkheads in order to prevent wave overtopping damage are set forth in Table 59.

The Milwaukee outer harbor breakwater would be maintained at its existing elevation. The South Shore breakwater would be reconstructed to an elevation of 588.6 feet NGVD. Alternatives for the Milwaukee outer harbor and South Shore breakwaters are discussed later in this chapter.

About 7,370 feet of shoreline, or 5 percent, was not eroding in 1987 and would not require shoreline protection or maintenance under this alternative. The selected plan component and estimated cost of bluff toe protection are listed for each bluff analysis section in Table 60. The revetment alternative plan would have a total capital cost of about $57 million, and an annual maintenance cost of about $3.3 million.

The major advantages of the revetment alternative plan are its relatively low cost, ease of construction and maintenance, and good implementability. The proposed shore protection measures would represent an essential continuation of the existing approach to shore protection, although the proposed structures would be better designed, maintained, and coordinated than most existing structures. The plan could be readily implemented by individual property owners, or, preferably, by groups of property owners, and by municipalities.

A major disadvantage of the revetment alternative plan is the lack of a usable shoreline. In some sections, revetments would have an adverse effect on the littoral environment, which could, in the long term, increase wave action against the shoreline. Revetments tend to reflect wave energy—although less so than bulkheads—and do not feed the littoral transport system. Over time, the near-shore slopes of areas with erodible offshore sand deposits would become somewhat steeper, which would increase the maximum wave height that could reach the shore. Where offshore sand deposits are shallow and the erosion-resistant clay hardpan lies close to the surface of the lake bottom, wave reflection from revetments would probably not significantly steepen the offshore slopes.

<u>Beach Alternative Plan</u>: The beach alternative plan would include the construction or reconstruction of about 70,690 feet, or 13 miles, of usable beach composed of sand or gravel, which is about 44 percent of the Milwaukee County shoreline. The beach alternative plan is shown on Map 40.

The criteria used in the selection of a beach alternative plan component, along with the estimated unit cost of each component, are set forth in Table 61. Nourished coarse sand or gravel beaches contained by short groins would be created along about 61,450 feet, or 38 percent, of the Milwaukee County shoreline. These beaches could also be contained by armored headlands or near-shore reefs constructed of quarry stone. New or reconstructed revetments would lie along 27,800 feet of shoreline, or 17 percent of the County total. New or reconstructed sand beaches would cover 9,240 feet of shoreline, or 6 percent. About 22,200 feet of existing structures would be maintained. Bulkheads would be modified or reconstructed along about 31,050 feet of shoreline, or 20 percent of the total. The Milwaukee outer harbor breakwater would be maintained at its existing elevation, and the South Shore breakwater would be reconstructed to an elevation of 588.6 feet NGVD, as assumed under the revetment alternative plan. No shoreline protection would be required along 7,370 feet of shoreline, or 5 percent of the county shoreline. The selected plan component and estimated cost of bluff toe protection are listed for each bluff analysis section in Table 62. The beach alternative plan would have a total capital cost of about $69.0 million, and an average annual maintenance cost of about $3.6 million.

Even under the beach alternative plan, revetments are recommended to protect the bluff toe of many sections recommended for bluff slope regrading, especially those bluffs where existing or proposed fill projects would help stabilize the bluffs. Although in most cases a sand or gravel beach technically could be constructed to protect the toe of a fill project, a beach was not proposed under this alternative to protect the toe of most fill sites for two major reasons. First, the lakebed bathymetry offshore of most fill projects tends to be relatively steep, and most of the bluff slopes that are filled or proposed to be filled face easterly or northeasterly. Hence, most fill areas will be subjected to some of the largest storm waves attacking the Milwaukee County shoreline. It would be difficult—and costly—to maintain a beach on a long-term basis in such a high-wave-energy environment. Second, since the fill projects generally require the placement of fill toward, and often into, the lake, the additional construction of a nourished beach, and the attendant containment structures, from

Table 60

REVETMENT ALTERNATIVE PLAN FOR MILWAUKEE COUNTY

Bluff Analysis Section	Shoreline Length (feet)	Plan Component	Cost per Lineal Foot: Capital	Cost per Lineal Foot: Annual Maintenance	Total Cost: Capital	Total Cost: Annual Maintenance	Total Cost: 50-Year Present Worth	Total Cost: Equivalent Annual Cost
1	4,470	Reconstruct existing public bulkhead	$ 800	$15	$ 3,576,000	$ 67,000	$ 4,633,000	$ 294,000
2	2,820	No additional shore protection required	0	0	0	0	0	0
3	2,930	Construct new public revetment	400	15	1,172,000	44,000	1,866,000	118,000
4	1,980	Construct new public revetment	400	15	792,000	30,000	1,260,000	80,000
5	1,070	Construct new public revetment	400	15	428,000	16,000	682,000	43,000
6	1,170	Construct new private revetment	400	15	468,000	18,000	745,000	47,000
7	1,000	Construct new private revetment	400	15	400,000	15,000	636,000	40,000
8	540	Reconstruct existing public bulkhead	150	15	81,000	8,000	209,000	13,000
9	570	Construct new private revetment	400	15	228,000	9,000	364,000	23,000
10	400	Construct new private revetment	400	15	160,000	6,000	255,000	16,000
11	1,290	Construct new private revetment	400	15	516,000	19,000	822,000	52,000
12	3,160	Reconstruct existing public bulkhead	750	15	2,370,000	48,000	3,117,000	198,000
13	1,320	No additional shore protection required	0	0	0	0	0	0
14	1,310	Construct new private revetment	300	10	393,000	13,000	600,000	38,000
15	790	Construct new private revetment	300	10	237,000	8,000	362,000	23,000
16	470	Construct new private revetment	300	10	141,000	5,000	215,000	14,000
17	440	Construct new private revetment	300	10	132,000	4,000	201,000	13,000
18	220	Construct new public revetment	500	15	110,000	3,000	162,000	10,000
	1,660	Construct new private revetment	400	15	664,000	25,000	1,057,000	67,000
19	2,480	Maintain existing public structures (onshore)	0	15	0	12,000	189,000	12,000
	700	Reconstruct existing private revetment	200	10	140,000	7,000	250,000	16,000
20	1,280	Construct new public revetment	400	15	512,000	190,000	815,000	52,000
21	1,060	Construct new public revetment	400	15	424,000	16,000	675,000	43,000
22	950	Construct new public revetment	400	15	380,000	14,000	605,000	38,000
23	1,200	Construct new public revetment	400	15	480,000	18,000	764,000	48,000
24	1,910	Construct new public revetment	400	15	764,000	29,000	1,216,000	77,000
25	880	Construct new public revetment	400	15	352,000	13,000	560,000	36,000
26	660	Construct new private revetment	500	15	330,000	10,000	486,000	31,000
27	1,850	Construct new public revetment	400	15	740,000	30,000	1,178,000	75,000
28	2,050	Construct new public revetment	400	15	820,000	31,000	1,306,000	83,000
29	770	Construct new public revetment	400	15	308,000	12,000	491,000	31,000
30	1,760	Construct new public revetment	400	15	704,000	26,000	1,120,000	71,000
31	600	Construct new public revetment	400	15	240,000	9,000	382,000	24,000
32	340	Reconstruct existing public bulkhead	150	10	51,000	3,000	105,000	7,000
33	2,060	Construct new public revetment	400	15	824,000	31,000	1,311,000	83,000
34	1,780	Reconstruct existing public groin with sand beach	1,000	30	1,780,000	53,000	2,615,000	166,000
35	650	Reconstruct existing public groin with sand beach	1,000	30	650,000	20,000	957,000	61,000
36	710	Construct new public revetment	400	15	284,000	11,000	453,000	29,000
37	1,010	Construct new public revetment	400	15	404,000	15,000	644,000	41,000
38	1,290	Construct new private revetment	300	10	387,000	13,000	590,000	37,000
39	1,480	Construct new private revetment	300	10	444,000	15,000	677,000	43,000
40	820	Reconstruct existing private revetment	400	15	328,000	12,000	522,000	33,000
41	1,650	Maintain existing private structures	0	15	0	25,000	391,000	25,000
42	940	Reconstruct existing private revetment	400	15	376,000	14,000	598,000	38,000
43	1,370	Reconstruct existing public breakwater	800	35	1,096,000	48,000	1,853,000	118,000
	1,370	Construct new public revetment	200	10	274,000	14,000	490,000	31,000
44	140	Reconstruct existing public breakwater	1,240	35	174,000	5,000	251,000	16,000
	140	Construct new public revetment	200	10	28,000	1,000	50,000	3,000
45	80	Reconstruct existing public breakwater	1,240	35	99,000	3,000	143,000	9,000
	80	Construct new public revetment	200	10	16,000	1,000	290,000	2,000
46	360	Reconstruct existing public breakwater	1,010	35	364,000	13,000	562,000	36,000
	360	Construct new public revetment	200	10	72,000	4,000	129,000	8,000
47	2,470	Reconstruct existing public breakwater	1,160	35	2,865,000	87,000	4,229,000	268,000
	2,470	Construct new public revetment	200	10	494,000	25,000	883,000	56,000
48	1,420	Reconstruct existing public breakwater	1,240	35	1,761,000	50,000	2,544,000	161,000
	1,420	Reconstruct existing public revetment	150	10	213,000	14,000	437,000	28,000
49	340	Reconstruct existing public breakwater	880	35	299,000	12,000	487,000	310,000
	340	Reconstruct existing public revetment	300	10	102,000	3,000	156,000	10,000
50	1,130	Reconstruct existing public breakwater	700	35	791,000	40,000	1,415,000	90,000
	1,130	Reconstruct existing public revetment	150	10	170,000	11,000	348,000	22,000
51	570	Reconstruct existing public breakwater	690	35	393,000	20,000	709,000	45,000
	570	Reconstruct existing public revetment	150	10	86,000	6,000	175,000	11,000
52	450	Reconstruct existing public breakwater	820	35	369,000	16,000	618,000	39,000
	450	No additional onshore protection required	0	0	0	0	0	0
53	1,320	Reconstruct existing public breakwater	410	35	541,000	46,000	1,269,000	81,000
	1,320	Reconstruct existing public bulkhead	150	15	198,000	20,000	510,000	32,000
54	1,360	Reconstruct existing public breakwater	410	35	558,000	48,000	1,308,000	83,000
	1,360	Construct new public revetment	200	10	272,000	14,000	486,000	31,000
55	9,600	Maintain existing public breakwater	0	45	0	432,000	6,809,000	432,000
	4,600	U. S. Army Corps of Engineers dredge spoils confined disposal facility—reconstruct existing revetment	400	10	1,840,000	46,000	2,565,000	163,000
	3,400	South Lincoln Memorial Drive—reconstruct existing bulkhead	300	10	1,020,000	34,000	1,556,000	99,000
	5,650	Port of Milwaukee slips—reconstruct existing bulkhead	500	10	2,825,000	56,000	3,708,000	235,000
	1,100	MMSD Jones Island wastewater treatment plant—maintain existing bulkhead	0	10	0	11,000	173,000	11,000

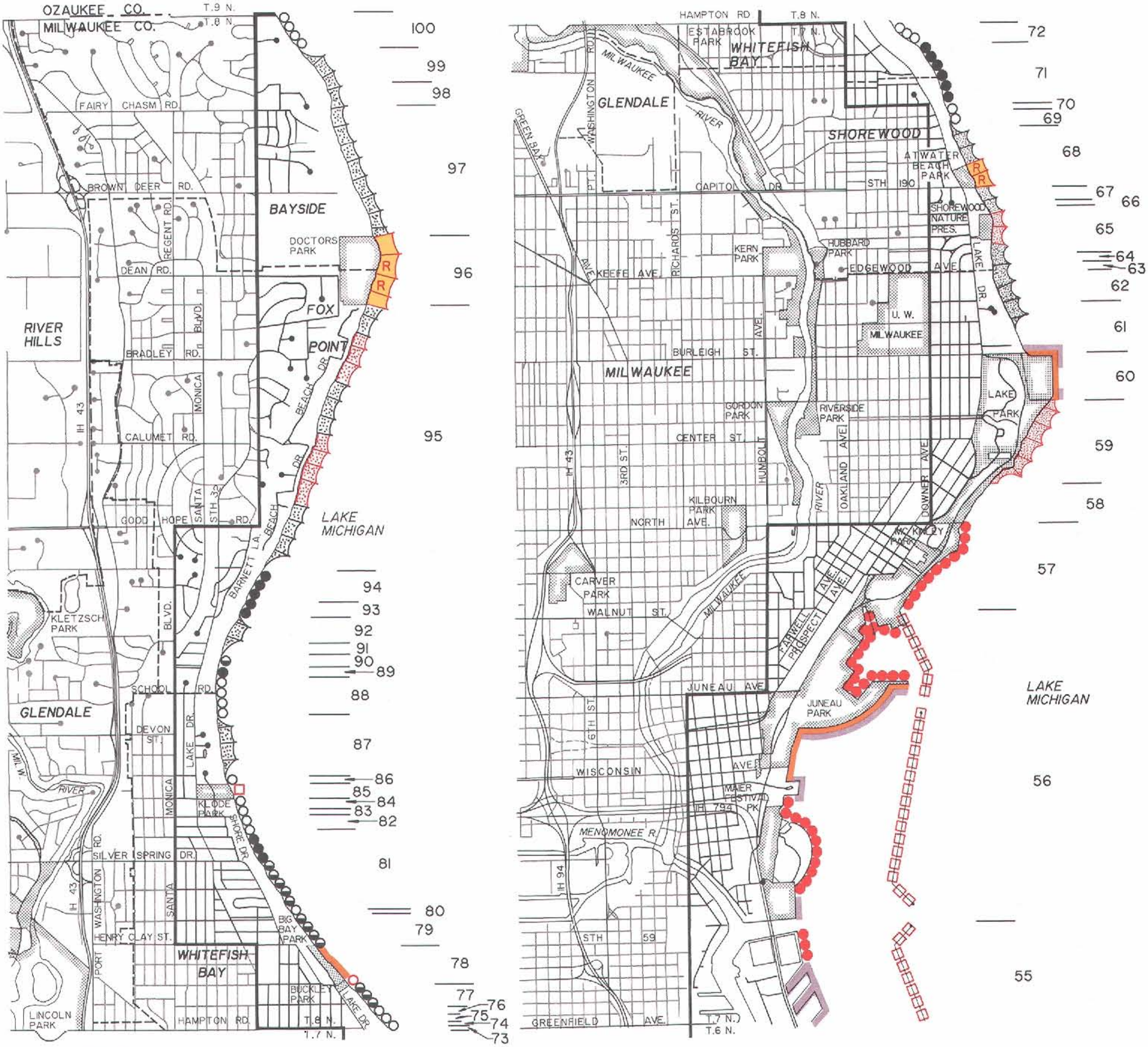
Table 60 (continued)

Bluff Analysis Section	Shoreline Length (feet)	Plan Component	Cost per Lineal Foot		Total Cost			
			Capital	Annual Maintenance	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual Cost
56	9,500	Maintain existing public breakwater	$ 0	$45	$ 0	$ 428,000	$ 6,738,000	$ 428,000
	1,900	Marcus Amphitheatre—reconstruct existing bulkhead	200	10	380,000	19,000	679,000	43,000
	2,900	Henry W. Maier festival grounds—maintain existing island and revetment	0	10	0	29,000	457,000	29,000
	1,400	Milwaukee Harbor Commission municipal pier—reconstruct existing bulkhead	200	10	280,000	14,000	501,000	32,000
	1,700	Milwaukee County War Memorial Center—reconstruct existing bulkhead	200	10	340,000	17,000	608,000	39,000
	3,900	Milwaukee County Juneau Park landfill—reconstruct existing bulkhead	600	10	2,340,000	39,000	2,955,000	187,000
	4,260	McKinley Marina—maintain existing public structures (onshore)	0	10	0	43,000	678,000	43,000
57	3,210	Maintain existing public structures (onshore)	0	30	0	46,000	1,518,000	96,000
58	1,900	No additional shore protection required	0	0	0	0	0	0
59	3,540	Reconstruct existing public revetment	300	15	1,062,000	53,000	1,899,000	121,000
60	2,210	Reconstruct existing public bulkhead	500	15	1,105,000	33,000	1,628,000	103,000
61	880	No additional shore protection required	0	0	0	0	0	0
	1,090	Construct new private revetment	300	10	327,000	11,000	499,000	32,000
62	950	Construct new private revetment	300	10	285,000	10,000	435,000	28,000
63	300	Construct new private revetment	300	10	90,000	3,000	137,000	9,000
64	290	Maintain existing private structures	0	10	0	3,000	46,000	3,000
65	1,710	Construct new public revetment	400	15	684,000	26,000	1,098,000	69,000
66	170	Reconstruct existing private revetment	200	10	34,000	2,000	61,000	4,000
67	300	Construct new private revetment	300	10	114,000	4,000	174,000	11,000
68	790	Reconstruct existing public groin system with sand beach	1,000	30	790,000	24,000	1,164,000	74,000
	1,380	Construct new private revetment	300	10	414,000	14,000	632,000	40,000
69	520	Construct new private revetment	300	10	156,000	5,000	238,000	15,000
70	240	Construct new private revetment	300	10	72,000	2,000	110,000	7,000
71	2,370	Maintain existing private structures	0	10	0	24,000	374,000	24,000
72	850	Construct new private revetment	300	10	255,000	9,000	389,000	25,000
73	190	Construct new private revetment	300	10	57,000	2,000	87,000	6,000
74	160	Construct new private revetment	300	10	48,000	2,000	73,000	5,000
75	310	Construct new private revetment	300	10	93,000	3,000	142,000	9,000
76	360	Construct new private revetment	300	10	108,000	4,000	165,000	10,000
77	810	Reconstruct existing private revetment	200	10	162,000	8,000	290,000	18,000
78	600	Construct new public revetment	400	15	240,000	9,000	382,000	24,000
	1,060	Reconstruct existing public bulkhead	200	15	212,000	16,000	463,000	29,000
79	1,480	Reconstruct existing private revetment	200	10	296,000	15,000	529,000	34,000
80	130	Construct new private revetment	300	10	39,000	1,000	60,000	4,000
81	1,700	Reconstruct existing private revetment	200	10	340,000	17,000	608,000	39,000
	1,270	Maintain existing private structures	0	10	0	13,000	200,000	13,000
82	490	Construct new private revetment	300	10	147,000	5,000	224,000	14,000
83	140	Construct new private revetment	300	10	42,000	1,000	64,000	4,000
84	430	Construct new private revetment	300	10	129,000	4,000	197,000	13,000
85	480	Maintain existing public breakwater	0	30	0	14,000	227,000	14,000
86	170	Maintain existing private structures	0	10	0	2,000	27,000	2,000
87	1,950	Construct new private revetment	300	10	585,000	20,000	307,000	20,000
88	1,150	Construct new private revetment	300	10	345,000	12,000	526,000	33,000
89	320	Maintain existing private structures	0	10	0	3,000	50,000	3,000
90	470	Reconstruct existing private revetment	200	10	94,000	5,000	168,000	11,000
91	510	Construct new private revetment	300	10	153,000	5,000	233,000	15,000
92	770	Reconstruct existing private revetment	200	10	154,000	8,000	275,000	18,000
93	530	Maintain existing private structures	0	10	0	5,000	84,000	5,000
94	1,460	Maintain existing private structures	0	10	0	15,000	230,000	15,000
95A	2,390	Maintain existing private structures	0	10	0	24,000	377,000	24,000
95B	1,600	Reconstruct existing public revetment	400	15	640,000	24,000	1,018,000	65,000
95C	3,000	Maintain existing private structures	0	10	0	30,000	473,000	30,000
95D	720	Reconstruct existing public revetment	400	15	288,000	11,000	458,000	29,000
95E	1,360	Maintain existing private structures	0	10	0	14,000	214,000	14,000
96	800	Construct new public revetment	400	15	320,000	12,000	509,000	32,000
	1,090	Reconstruct existing public groin system with sand beach	1,000	30	1,090,000	33,000	1,605,000	108,000
97	4,660	Construct new private revetment	300	10	1,398,000	47,000	2,133,000	135,000
98	860	Maintain existing private structures	0	10	0	9,000	136,000	9,000
99	1,280	Construct new private revetment	300	10	384,000	13,000	586,000	37,000
100	1,320	Construct new private revetment	300	10	396,000	13,000	604,000	38,000
Total	159,110	- -	$ 304	$17	$56,998,000	$3,324,000	$107,107,000	$7,060,000

Source: SEWRPC.

Map 40

BEACH ALTERNATIVE PLAN



LEGEND

91 BLUFF ANALYSIS SECTION

CONSTRUCT NEW STRUCTURES

- PRIVATE GROIN SYSTEM WITH COARSE SAND OR GRAVEL BEACH
- PUBLIC GROIN SYSTEM WITH COARSE SAND OR GRAVEL BEACH
- PUBLIC GROIN SYSTEM WITH SAND BEACH
- PUBLIC SAND BEACH
- PRIVATE REVETMENT
- PUBLIC REVETMENT

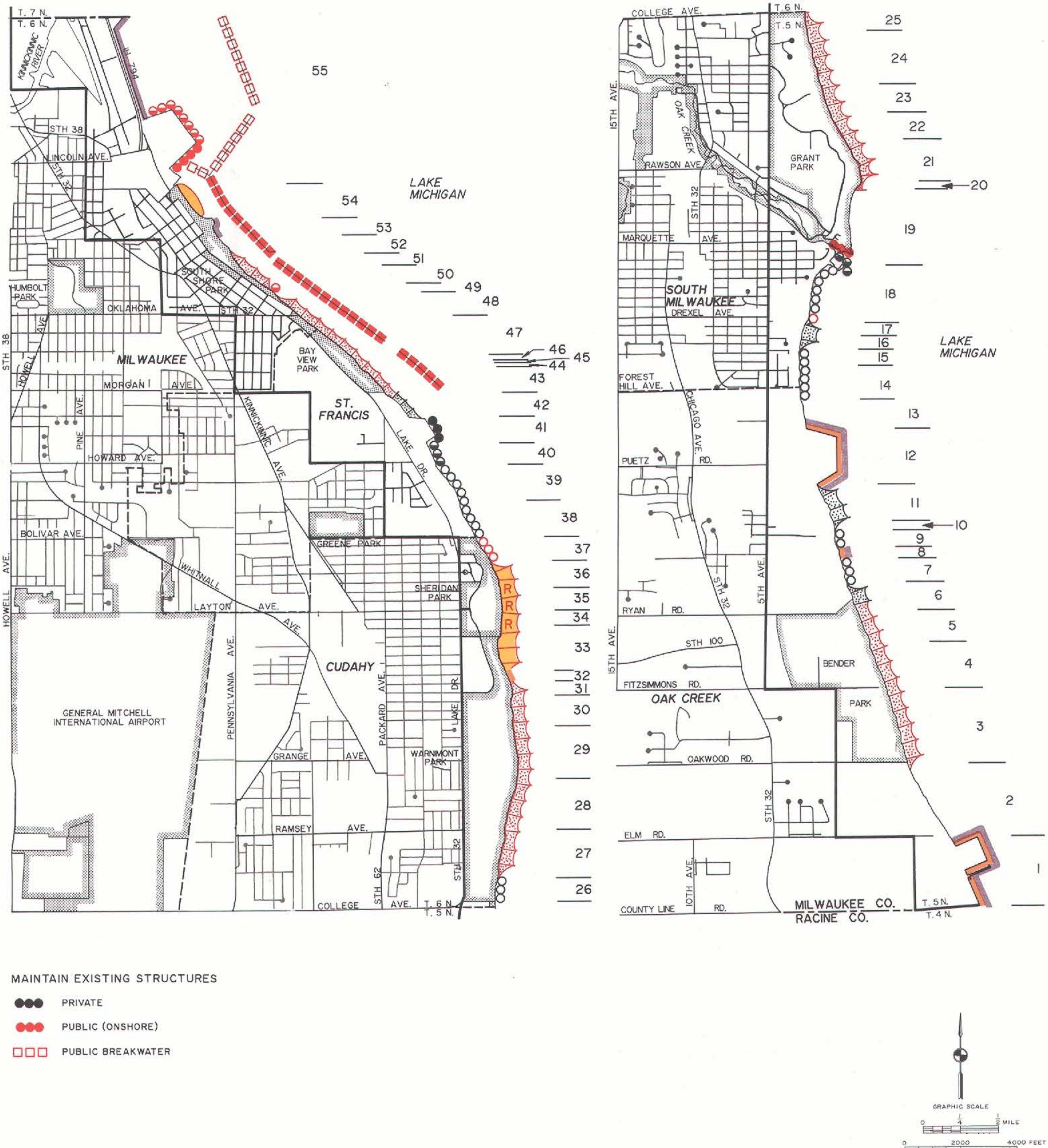
RECONSTRUCT EXISTING STRUCTURES

- R PUBLIC GROIN SYSTEM WITH SAND BEACH
- PRIVATE REVETMENT
- PUBLIC REVETMENT

PUBLIC BULKHEAD

- RIP-RAP BERM
- HEIGHT EXTENSION
- BERM AND EXTENSION
- PUBLIC BREAKWATER

Map 40 (continued)



Source: SEWRPC.

Table 61

SELECTION CRITERIA AND TYPICAL CAPITAL AND MAINTENANCE UNIT COSTS OF BEACH ALTERNATIVE PLAN COMPONENTS

		Typical Unit Cost ($/lineal foot of shoreline)[a]			
		Total Capital		Annual Maintenance	
Plan Component	Criteria for Selection	Private	Public	Private	Public
Construction of a Nourished Gravel Beach System with Short Groins	Shoreline or bluff erosion observed in 1986 or 1987	400	500	20	20
Construction of Sand Beach and, Where Needed, a Groin System	Probable community support for a large sand beach	- -	800-1,200	- -	30-50
Construction of Revetment	Bluff slope regrading filling is required to stabilize the bluff slope Beach system is not needed to provide consistent shoreline Probable community support for a revetment	300-400	400-500	10	15
Reconstruction of Existing Nourished Sand Beach with Groin System	Existing groin system which, as of 1987, required a substantial amount of repair Probable community support for retaining a large sand beach	- -	1,000	- -	30
Reconstruction of Existing Revetment	Bluff slope regrading requiring filling has previously been conducted Existing revetment which, as of 1987, required a substantial amount of repair	200-300	300-400	10	15
Reconstruction of Existing Bulkhead	Existing bulkhead which, as of 1987, required a substantial amount of repair	- -	Variable, depending on condition of structure	- -	15
Reconstruction of Existing Breakwater	Existing breakwater which, as of 1987, required a substantial amount of repair	- -	800-1,250	- -	35
Continued Maintenance of Existing Onshore Structure	Onshore structure which was protecting against erosion in 1987 and which, if maintained, could provide continued effective protection	0	0	Variable, depending on type of structure	Variable, depending on type of structure
Continued Maintenance of Existing Breakwater	Breakwater which was protecting against erosion in 1987 and which, if maintained, could provide continued effective protection	- -	0	- -	45
No Shoreline Protection	No significant shoreline or bluff toe erosion observed in 1987 and none expected to occur	0	0	0	0

[a] *Typical unit costs are presented herein. However, costs applicable to individual bluff analysis sections were varied to reflect known problems and physical conditions.*

Source: SEWRPC.

a fill site would often have to extend a considerable distance out into the lake. Beaches extending too far into the lake would again be difficult to maintain, and the required containment structures could adversely affect downdrift shoreline areas. Nourished beaches should be constructed in reasonable alignment in order to prevent massive beach material accumulations in some areas, and scarce accumulations in others. However, despite these limitations, beaches are recommended to protect the toe of selected fill projects where a beach is needed to provide a consistent, uniform shoreline; where offshore slopes are not excessive; or where only a minimal amount of fill is thought to be needed to adequately stabilize the slope.

The major advantage of the beach alternative plan is the provision of a more usable shoreline. The sand or gravel beaches would not only offer access and recreational opportunities while protecting the shoreline from erosion, but also reduce wave reflection and, to a limited extent, feed the littoral transport system, thereby reducing adverse effects on the littoral environment. The beach alternative plan could be implemented by groups of property owners as well as by municipalities.

A disadvantage of the beach alternative plan is the increased maintenance and periodic beach nourishment required. To successfully implement the plan, all property owners within the specified beach sections would have to participate in both the construction and maintenance of the beach systems—the systems should not be implemented in a piecemeal manner. In some privately owned shoreline areas, the provision of coarse sand or gravel beaches could lead to increased trespassing on the shoreline, which may be opposed by some private property owners who desire access restrictions and privacy.

Offshore Alternative Plan: The offshore alternative plan would provide a series of offshore islands, peninsulas, and breakwaters for about 131,620 feet, or 24.9 miles, of shoreline, or 83 percent of the total county shoreline. The offshore alternative plan is illustrated on Map 41.

The islands and peninsulas, likely composed largely of concrete rubble, soil, and other clean fill material from construction or demolition projects, would be protected on the lakeward side by either a revetment or an armored headland-pocket beach system, and on the landward side by a smaller revetment. The islands and peninsulas, which would be constructed with land-based equipment, would usually be located 300 to 1,000 feet offshore at an approximate water depth of 10 to 12 feet, although those in the Milwaukee outer harbor would lie in about 30 feet of water. The publicly owned islands and peninsulas in the northern portion of the County, and in the Cities of St. Francis, Cudahy, and South Milwaukee, could be utilized for recreational uses such as small boating, hiking, fishing, and nature study; while offshore facilities at Juneau Park, Lake Park, South Shore Park, Bay View Park, Bender Park, and the Milwaukee outer harbor could be utilized for more intensive recreational uses, such as swimming, bicycling, baseball, picnicking, and playground activities. New offshore breakwaters with sand beaches would be constructed at the Village of Shorewood Atwater Park, the southern portion of North Beach Drive in the Village of Fox Point which lies directly adjacent to the lake, and Milwaukee County's Doctors Park, Bay View Park, and Sheridan Park.

A similar proposal for the creation of offshore islands to protect the entire Milwaukee County shoreline was set forth by the University of Wisconsin-Milwaukee School of Architecture in 1974.[10] That proposal, as graphically illustrated on Map 42, was prepared at the request of the Lakefront Recreational Development Task Force, created by the Mayor of the City of Milwaukee in 1973.[11] Except for the Milwaukee outer harbor, the School of Architecture proposal generally included larger islands which would have been located in deeper water than those envisioned in the offshore alternative plan set forth in this report. The proposed island configurations presented by the School of Architecture for most of the Milwaukee outer harbor area

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[10]*University of Wisconsin-Milwaukee School of Architecture, Offshore Island Parklands Project, Architecture 430-Environmental Systems, November 1974.*

[11]*Lakefront Recreational Development Task Force, Milwaukee's Lakefront, A Precious Heritage, A Vital Resource, Prepared for the City of Milwaukee, March 1978.*

Table 62

BEACH ALTERNATIVE PLAN FOR MILWAUKEE COUNTY

Bluff Analysis Section	Shoreline Length (feet)	Plan Component	Cost per Lineal Foot		Total Cost			
			Capital	Annual Maintenance	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual Cost
1	4,470	Reconstruct existing public bulkhead	$ 800	$ 15	$ 3,576,000	$ 67,000	$ 4,633,000	$ 294,000
2	2,820	No additional shore protection required	0	0	0	0	0	0
3	2,930	Construct new public groin system with coarse sand or gravel beach	500	20	1,465,000	59,000	2,389,000	152,000
4	1,980	Construct new public groin system with coarse sand or gravel beach	500	20	990,000	40,000	1,614,000	102,000
5	1,070	Construct new public groin system with coarse sand or gravel beach	500	20	535,000	21,000	872,000	55,000
6	1,170	Construct new private groin system with coarse sand or gravel beach	400	20	468,000	23,000	835,000	53,000
7	1,000	Construct new private revetment	400	15	400,000	15,000	636,000	40,000
8	540	Reconstruct existing public bulkhead	150	15	81,000	8,000	209,000	13,000
9	570	Construct new private revetment	400	15	228,000	9,000	364,000	23,000
10	400	Construct new private groin system with coarse sand or gravel beach	400	20	160,000	8,000	286,000	18,000
11	1,290	Construct new private groin system with coarse sand or gravel beach	400	20	516,000	26,000	923,000	59,000
12	3,160	Reconstruct existing public bulkhead	750	15	2,370,000	47,000	3,117,000	198,000
13	1,320	No additional shore protection required	0	0	0	0	0	0
14	1,310	Construct new private revetment	300	10	393,000	13,000	600,000	38,000
15	790	Construct new private revetment	300	10	237,000	8,000	362,000	23,000
16	470	Construct new private groin system with coarse sand or gravel beach	400	20	188,000	9,000	336,000	21,000
17	440	Construct new private groin system with coarse sand or gravel beach	400	20	176,000	9,000	315,000	20,000
18	220	Construct new public revetment	500	15	110,000	3,000	162,000	10,000
	1,660	Construct new private revetment	400	15	664,000	25,000	1,057,000	67,000
19	2,480	Maintain existing public structures (onshore)	0	15	0	12,000	189,000	12,000
	700	Reconstruct existing private revetment	200	10	140,000	7,000	250,000	16,000
20	1,280	Construct new public groin system with coarse sand or gravel beach	500	20	640,000	26,000	1,044,000	66,200
21	1,060	Construct new public groin system with coarse sand or gravel beach	500	20	530,000	21,000	864,000	55,000
22	950	Construct new public groin system with coarse sand or gravel beach	500	20	475,000	19,000	775,000	49,000
23	1,200	Construct new public groin system with coarse sand or gravel beach	500	20	600,000	24,000	978,000	62,000
24	1,910	Construct new public groin system with coarse sand or gravel beach	500	20	955,000	38,200	1,557,100	99,000
25	880	Construct new public groin system with coarse sand or gravel beach	500	20	440,000	18,000	717,000	46,000
26	660	Construct new private revetment	500	15	330,000	10,000	486,000	31,000
27	1,850	Construct new public groin system with coarse sand or gravel beach	500	20	925,000	37,000	1,508,000	96,000
28	2,050	Construct new public groin system with coarse sand or gravel beach	500	20	1,025,000	41,000	1,671,000	106,000
29	770	Construct new public groin system with coarse sand or gravel beach	500	20	385,000	15,000	628,000	40,000
30	1,760	Construct new public groin system with coarse sand or gravel beach	500	20	880,000	35,000	1,435,000	91,000
31	600	Construct new public groin system with coarse sand or gravel beach	500	20	300,000	12,000	489,000	31,000
32	340	Reconstruct existing public bulkhead	150	15	51,000	5,000	131,000	8,000
33	2,060	Construct new public groin system with sand beach	1,200	30	2,472,000	62,000	3,449,000	219,000
34	1,780	Reconstruct existing public groin system with sand beach	1,000	30	1,780,000	53,000	2,615,000	166,000
35	650	Reconstruct existing public groin system with sand beach	1,000	30	650,000	20,000	957,000	61,000
36	710	Construct new public groin system with coarse sand or gravel beach	1,200	30	852,000	21,000	1,188,000	75,000
37	1,010	Construct new public revetment	400	15	404,000	15,000	644,000	41,000
38	1,290	Construct new private revetment	300	10	387,000	13,000	590,000	37,000
39	1,480	Construct new private revetment	300	10	444,000	15,000	677,000	43,000
40	820	Reconstruct existing private revetment	400	15	328,000	12,000	522,000	33,000
41	1,650	Maintain existing private structures	0	10	0	17,000	260,000	17,000
42	940	Construct new private groin system with coarse sand or gravel beach	400	20	376,000	19,000	672,000	43,000
43	1,370	Reconstruct existing public breakwater	800	35	1,096,000	48,000	1,853,000	118,000
	1,370	Construct new public groin system with coarse sand or gravel beach	300	10	411,000	13,700	627,000	40,000
44	140	Reconstruct existing public breakwater	1,240	35	174,000	5,000	251,000	16,000
	140	Construct new public groin system with coarse sand or gravel beach	300	10	42,000	1,000	64,000	4,000
45	80	Reconstruct existing public breakwater	1,240	35	99,000	3,000	143,000	9,000
	80	Construct new public groin system with coarse sand or gravel beach	300	10	24,000	1,000	37,000	2,000
46	360	Reconstruct existing public breakwater	1,010	35	364,000	13,000	562,000	36,000
	360	Construct new public groin system with coarse sand or gravel beach	300	10	108,000	4,000	165,000	10,000
47	2,470	Reconstruct existing public breakwater	1,160	35	2,865,000	87,000	4,229,000	268,000
	2,470	Construct new public groin system with coarse sand or gravel beach	300	10	741,000	25,000	1,130,000	72,000
48	1,420	Reconstruct existing public breakwater	1,240	35	1,761,000	50,000	2,544,000	161,000
	1,420	Construct new public groin system with coarse sand or gravel beach	300	10	426,000	14,000	650,000	41,000
49	340	Reconstruct existing public breakwater	880	35	299,000	12,000	487,000	31,000
	340	Reconstruct existing public revetment	300	10	102,000	3,000	156,000	10,000
50	1,130	Reconstruct existing public breakwater	700	35	791,000	40,000	1,415,000	90,000
	1,130	Construct new public groin system with coarse sand or gravel beach	300	10	339,000	11,000	517,000	33,000
51	570	Reconstruct existing public breakwater	690	35	393,000	20,000	709,000	45,000
	570	Construct new public groin system with coarse sand or gravel beach	300	10	171,000	6,000	261,000	17,000
52	450	Reconstruct existing public breakwater	820	35	369,000	16,000	618,000	39,000
	450	No additional on-shore protection required	0	0	0	0	0	0
53	1,320	Reconstruct existing public breakwater	410	35	541,000	46,000	1,269,000	81,000
	1,320	Reconstruct existing public bulkhead	150	10	198,000	13,000	406,000	26,000
54	1,360	Reconstruct existing public breakwater	410	35	558,000	47,600	1,308,000	83,000
	1,360	Construct new public sand beach	800	30	1,088,000	41,000	1,731,000	110,000

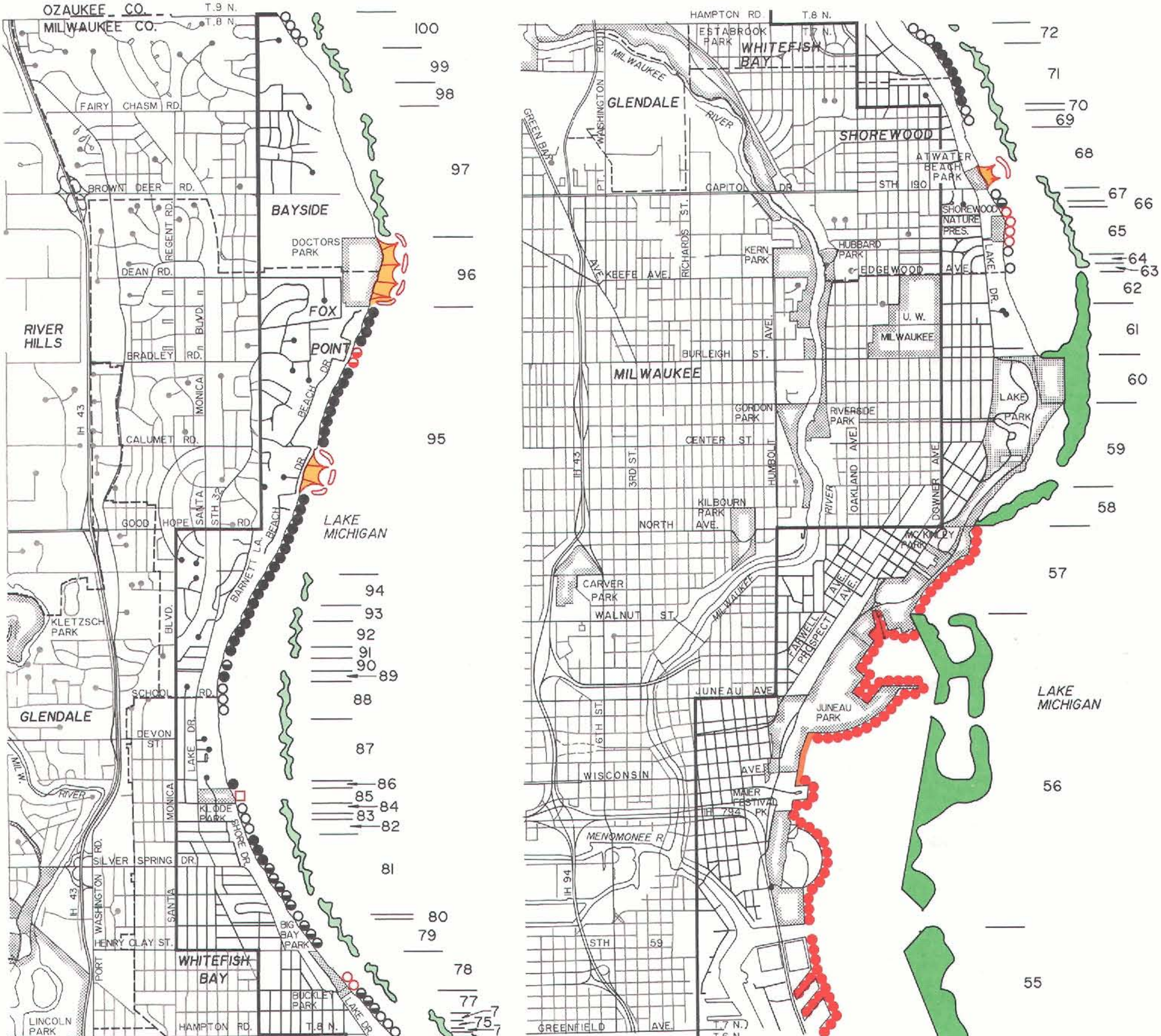
Table 62 (continued)

Bluff Analysis Section	Shoreline Length (feet)	Plan Component	Cost per Lineal Foot		Total Cost			
			Capital	Annual Maintenance	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual Cost
55	9,600	Maintain existing public breakwater	$ 0	$ 45	$ 0	$ 432,000	$ 6,809,000	$ 432,000
	4,600	U. S. Army Corps of Engineers dredge spoils confined disposal facility—reconstruct existing revetment	400	10	1,840,000	46,000	2,565,000	163,000
	3,400	South Lincoln Memorial Drive—reconstruct existing bulkhead	300	10	1,020,000	34,000	1,556,000	99,000
	5,650	Port of Milwaukee slips—reconstruct existing bulkhead	500	10	2,825,000	56,000	3,708,000	235,000
	1,100	MMSD Jones Island wastewater treatment plant—maintain existing bulkhead	0	10	0	11,000	173,000	11,000
56	9,500	Maintain existing public breakwater	0	45	0	428,000	6,738,000	428,000
	1,900	Marcus Amphitheatre—reconstruct existing bulkhead	200	10	380,000	19,000	679,000	43,000
	2,900	Henry W. Maier festival grounds—maintain existing island and revetment	0	10	0	29,000	457,000	29,000
	1,400	Milwaukee Harbor Commission Municipal pier—reconstruct existing bulkhead	200	10	280,000	14,000	501,000	32,000
	1,700	Milwaukee County War Memorial Center—reconstruct existing bulkhead	200	10	340,000	17,000	608,000	39,000
	3,900	Milwaukee County Juneau Park landfill—reconstruct existing bulkhead	600	10	2,340,000	39,000	2,955,000	187,000
	4,260	McKinley Marina—maintain existing bulkhead	0	10	0	43,000	678,000	43,000
57	3,210	Maintain existing public structures (onshore)	0	30	0	96,000	1,518,000	96,000
58	1,900	No additional shore protection required	0	0	0	0	0	0
59	3,540	Construct new public groin system with coarse sand or gravel beach	500	20	1,770,000	70,800	2,886,000	183,000
60	600	Reconstruct existing public bulkhead	200	15	120,000	9,000	262,000	17,000
	1,610	Reconstruct existing public bulkhead	450	15	725,000	24,000	1,106,000	70,000
61	880	No additional shore protection required	0	0	0	0	0	0
	1,090	Construct new private groin system with coarse sand or gravel beach	400	20	436,000	22,000	780,000	50,000
62	950	Construct new private groin system with coarse sand or gravel beach	400	20	380,000	19,000	680,000	44,000
63	300	Construct new private groin system with coarse sand or gravel beach	400	20	120,000	6,000	215,000	14,000
64	290	Construct new private groin system with coarse sand or gravel beach	400	20	116,000	6,000	207,000	13,000
65	1,710	Construct new public and private groin system with coarse sand or gravel beach	500	20	855,000	34,000	1,391,000	88,000
66	170	Construct new private groin system with coarse sand or gravel beach	400	20	68,000	3,000	122,000	8,000
67	380	Construct new private groin system with coarse sand or gravel beach	400	20	152,000	8,000	272,000	17,000
68	790	Reconstruct existing public groin system with sand beach	1,000	30	790,000	24,000	1,164,000	74,000
	1,380	Construct new private groin system with coarse sand or gravel beach	400	20	552,000	28,000	987,000	63,000
69	520	Construct new private revetment	300	10	156,000	5,000	238,000	15,000
70	240	Construct new private revetment	300	10	72,000	2,000	109,800	7,000
71	2,370	Maintain existing private structures	0	10	0	24,000	374,000	24,000
72	850	Construct new private revetment	300	10	255,000	9,000	389,000	25,000
73	190	Construct new private revetment	300	10	57,000	2,000	87,000	6,000
74	160	Construct new private revetment	300	10	48,000	2,000	73,000	5,000
75	310	Construct new private revetment	300	10	93,000	3,000	142,000	9,000
76	360	Construct new private revetment	300	10	108,000	4,000	165,000	10,000
77	810	Reconstruct existing private revetment	200	10	162,000	8,000	290,000	18,000
78	600	Construct new public revetment	400	15	240,000	9,000	382,000	24,000
	1,060	Reconstruct existing public bulkhead	200	15	212,000	16,000	463,000	29,000
79	1,480	Reconstruct existing private revetment	200	10	296,000	15,000	529,000	33,600
80	130	Construct new private revetment	300	10	39,000	1,000	60,000	4,000
81	1,700	Reconstruct existing private revetment	200	10	340,000	17,000	608,000	39,000
	1,270	Maintain existing private structures	0	10	0	13,000	200,000	13,000
82	490	Construct new private revetment	300	10	147,000	5,000	224,000	14,000
83	140	Construct new private revetment	300	10	42,000	1,000	64,000	4,000
84	430	Construct new private revetment	300	10	129,000	4,000	197,000	13,000
85	480	Maintain existing public breakwater	0	30	0	14,000	227,000	14,000
86	170	Maintain existing private structures	0	10	0	2,000	27,000	2,000
87	1,950	Construct new private groin system with coarse sand or gravel beach	400	20	780,000	39,000	1,395,000	89,000
88	1,150	Construct new private revetment	300	10	345,000	12,000	526,000	33,000
89	320	Maintain existing private structures	0	10	0	3,000	50,000	3,000
90	470	Reconstruct existing private revetment	200	10	94,000	4,700	168,000	11,000
91	510	Construct new private groin system with coarse sand or gravel beach	400	20	204,000	10,000	365,000	23,000
92	770	Construct new private groin system with coarse sand or gravel beach	400	20	308,000	15,000	551,000	35,000
93	530	Maintain existing private structures	0	10	0	5,000	83,000	5,000
94	1,460	Maintain existing private structures	0	10	0	15,000	230,000	15,000
95A	2,390	Construct new private groin system with coarse sand or gravel beach	400	200	956,000	48,000	1,709,000	108,000
95B	1,600	Construct new public groin system with coarse sand or gravel beach	500	20	800,000	32,000	1,304,000	83,000
95C	3,000	Construct new private groin system with coarse sand or gravel beach	400	20	1,200,000	60,000	2,146,000	136,000
95D	720	Construct new public groin system with coarse sand or gravel beach	500	20	360,000	14,000	587,000	37,000
95E	1,360	Construct new private groin system with coarse sand or gravel beach	400	20	544,000	27,000	973,000	62,000
96	1,890	Reconstruct existing public groin system with sand beach	1,000	30	1,890,000	57,000	2,784,000	177,000
97	4,660	Construct new private groin system with coarse sand or gravel beach	400	20	1,864,000	93,000	3,333,000	211,000
98	860	Construct new private groin system with coarse sand or gravel beach	400	20	344,000	17,000	615,000	39,000
99	1,280	Construct new private groin system with coarse sand or gravel beach	400	20	512,000	26,000	916,000	58,000
100	1,320	Construct new private revetment	300	10	396,000	13,000	604,000	38,000
Total	159,110	- -	$ 368	$ 19	$68,988,000	$3,641,000	$126,342,000	$8,020,000

Source: SEWRPC.

Map 41

OFFSHORE ALTERNATIVE PLAN



LEGEND

91 BLUFF ANALYSIS SECTION

CONSTRUCT NEW STRUCTURES

PUBLIC OFFSHORE ISLAND OR PENINSULA

PASSIVE USE

INTENSIVE USE

PUBLIC OFFSHORE BREAKWATERS WITH SAND BEACH

PRIVATE MARINA

PUBLIC MARINA

PRIVATE REVETMENT

PUBLIC REVETMENT

RECONSTRUCT EXISTING STRUCTURES

PRIVATE REVETMENT

PUBLIC REVETMENT

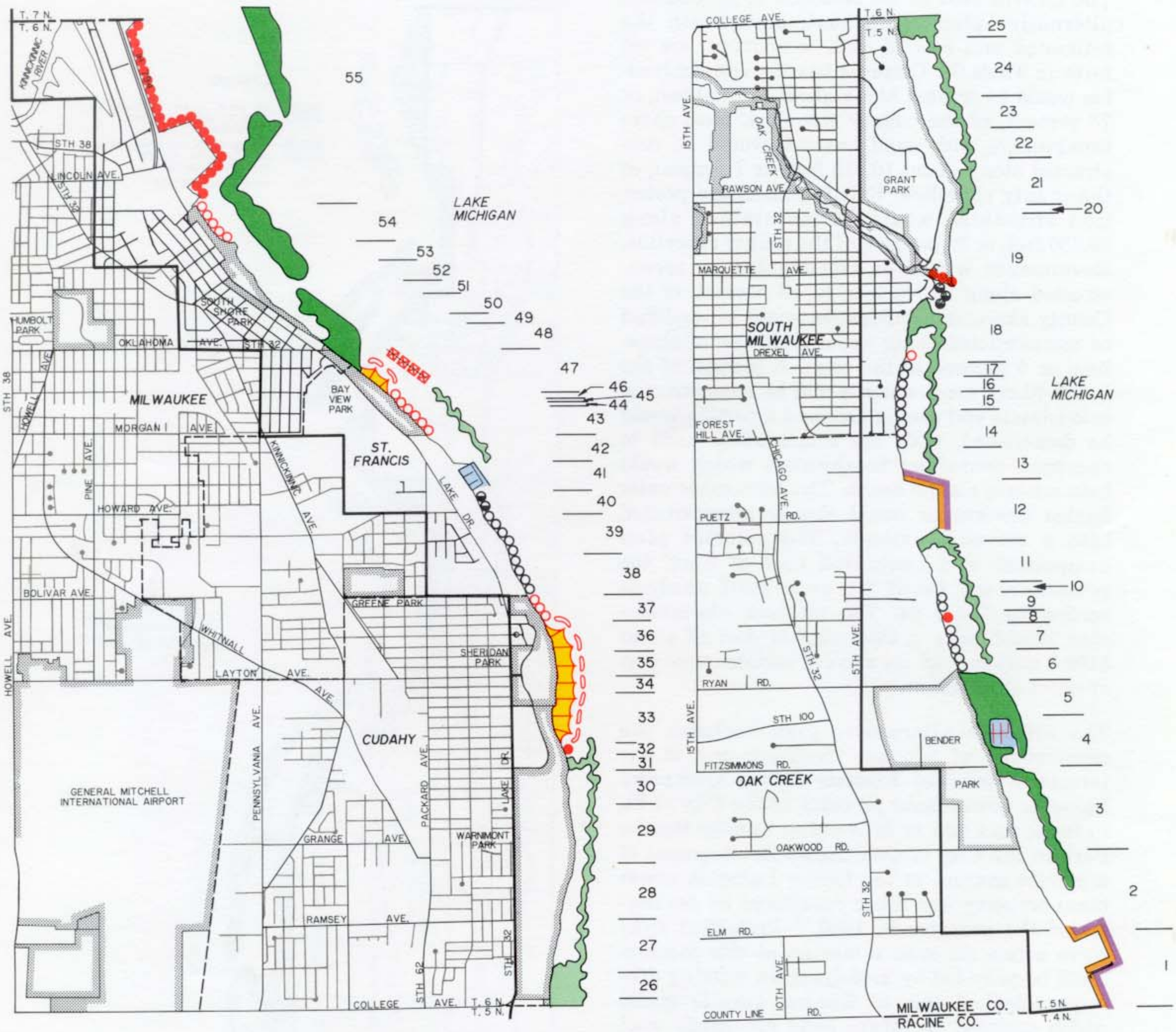
PUBLIC BULKHEAD

RIP-RAP BERM

HEIGHT EXTENSION

BERM AND EXTENSION

Map 41 (continued)



MAINTAIN EXISTING STRUCTURES

PRIVATE

PUBLIC (ONSHORE)

PUBLIC BREAKWATER

DEMOLISH EXISTING STRUCTURES

PUBLIC BREAKWATER

Source: SEWRPC.

have been incorporated into this report's offshore alternative plan.

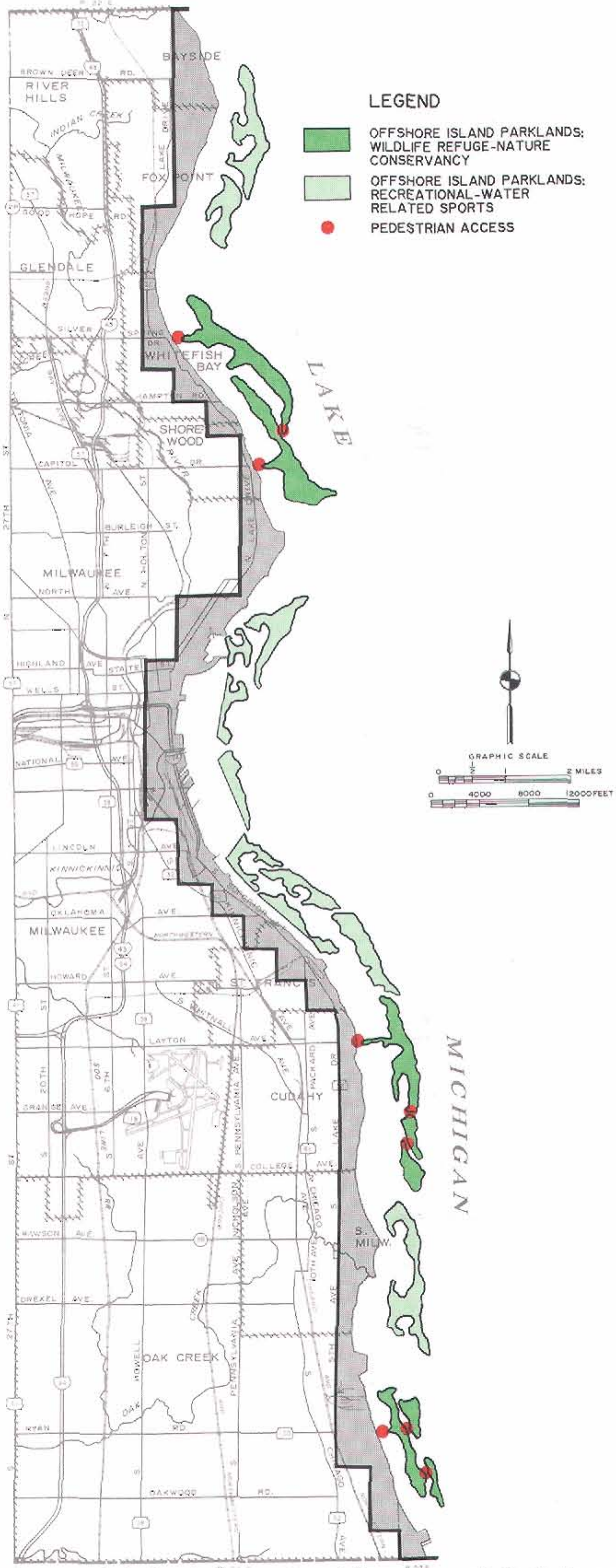
The criteria used in the selection of an offshore alternative plan component, along with the estimated unit cost of each component, are set forth in Table 63. Offshore islands and peninsulas would be created along about 120,710 feet, or 76 percent, of the county shoreline. Near-shore breakwaters with sand beaches would be constructed along about 10,915 feet, or 7 percent, of the county shoreline. Existing shoreline protection structures would be maintained along 49,830 feet, or 32 percent, of the county shoreline. Revetments would be constructed or reconstructed along 28,740 feet, or 18 percent, of the County shoreline. Bulkheads would be modified or reconstructed along about 9,800 feet of shoreline, or 6 percent of the total. A portion of the South Shore breakwater would be reconstructed into islands and peninsulas, and a portion would be demolished, with the armor stone used to construct near-shore breakwaters which would help contain a sand beach. The Milwaukee outer harbor breakwater would also be reconstructed into a series of islands. The selected plan component and estimated cost of bluff toe protection are listed for each bluff analysis section in Table 64. The offshore alternative plan would have a total capital cost of about $199.8 million, and an annual maintenance cost of about $3.4 million.

The offshore alternative plan includes the construction of two new marinas: one at the former Wisconsin Electric Power Company Lakeside power plant property in the City of St. Francis, and one at Milwaukee County Bender Park in the City of Oak Creek. Development of a private marina at the former Lakeside power plant property was being considered by developers of the property in 1989.[12] Protection from wave action for such a marina at this location could be provided by modifying an existing dike constructed of one- to five-ton granite stone, which encloses an intake pond previously used for the power plant. Preliminary plans and designs for a public marina at Bender Park have been prepared by Warzyn Engineering, Inc., for

[12]*Ralph J. Voltner, Jr., City of St. Francis Administrator, Personal Communication, January 1989.*

Map 42

OFFSHORE ISLAND PROPOSAL PREPARED BY THE UNIVERSITY OF WISCONSIN-MILWAUKEE SCHOOL OF ARCHITECTURE IN 1974



Source: Lakefront Recreational Development Task Force, Milwaukee's Lakefront, A Precious Heritage, A Vital Resource, 1978.

Table 63

SELECTION CRITERIA AND TYPICAL CAPITAL AND MAINTENANCE UNIT COST OF OFFSHORE ALTERNATIVE PLAN COMPONENTS

Plan Component	Criteria for Selection	Typical Unit Cost ($/lineal foot of shoreline)[a] Total Capital Private	Total Capital Public	Annual Maintenance Private	Annual Maintenance Public
Construction of Island or Peninsula	Entire shoreline, except: 1. Where breakwaters are proposed to maintain a sand beach; 2. Where an unobstructed view of the horizon from a beach or low terrace is desired by property owners; or 3. Where other existing or proposed structures make use of offshore islands or peninsulas impracticable	- -	1,200-3,000	- -	20-30
Construction of Breakwater System with Sand Beach	Existing public sand or fine gravel beaches Probable community support for a public sand beach Desire to provide additional public access and usable beach to public shoreline areas	- -	1,500	- -	30
Construction of New Marina	Probable community support for a marina	2,500	4,000	30	30
Construction of a New Revetment	Shoreline areas which require substantial bluff slope regrading, and fill projects under construction in 1987 Areas exhibiting moderate or severe shoreline or bluff toe erosion in 1987 and which would require additional protection beyond that provided by the offshore structures	200-300	300-400	5	10
Reconstruction of an Existing Revetment	Existing revetments which, as of 1987, required a substantial amount of repair in order to provide additional protection beyond that provided by the offshore structure	100-200	300	10	5
Reconstruction of Existing Bulkhead	Existing bulkhead which, as of 1987, required a substantial amount of repair	- -	Variable, depending on condition of structure	- -	15

Table 63 (continued)

Plan Component	Criteria for Selection	Typical Unit Cost ($/lineal foot of shoreline)[a] Total Capital Private	Total Capital Public	Annual Maintenance Private	Annual Maintenance Public
Continued Maintenance of Existing Onshore Structures	Onshore structure which was protecting against shoreline erosion in 1987 and which should be maintained to provide continued shore protection in combination with offshore structures	0	0	Variable, depending on type of structure	Variable, depending on type of structure
Demolish Existing Breakwater	Breakwater poses a potential hazard Opportunity to use stone to construct new structure which provides for recreational activities	--	800	--	0
No Shoreline Protection	No significant shoreline or bluff toe erosion observed in 1987 and none expected to occur	0	0	0	0

[a]Typical unit costs are presented herein. However, costs applicable to individual bluff analysis sections were varied to reflect known problems and physical conditions.

Source: SEWRPC.

Milwaukee County,[13] and for the Milwaukee Metropolitan Sewerage District.[14] The proposed 200-slip marina for Bender Park is illustrated on Map 43.

The major advantages of the offshore alternative plan would be the creation of approximately 1,300 acres of new public lakeshore parkland, of which 700 acres, or 54 percent, would be intended for intensive recreational use, and 600 acres, or 46 percent, would be intended for passive use; the creation of over 41 miles of new shoreline; the provision of protected surface water for safer and more enjoyable boating and swimming; the expansion of large public sand beaches; the provision of two new marinas; and the creation of new wildlife and fishery habitat. The plan would minimize the disruption associated with protecting the immediate shoreline, instead moving that construction offshore. As designed, the offshore structure would be constructed with land-based equipment, resulting in significant savings over marine (barge) construction techniques. The concept of an offshore plan offers an opportunity to utilize public funds to create new public parkland while helping to protect both public and private property.

The primary disadvantage of the offshore alternative plan, in addition to its high cost, is the need for over 42 million cubic yards of fill material for construction of the islands and peninsulas. Two potential major sources of fill

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[13]*Warzyn Engineering, Inc., Bender Park Waterfront Development Master Plan, Milwaukee County, Wisconsin, Prepared for Milwaukee County, August 1985.*

[14]*Warzyn Engineering, Inc., Technical Memorandum, Task No. 850, Preliminary Waterfront Development Plan for Bender Park, Prepared for the Milwaukee Metropolitan Sewerage District, August 1988.*

Table 64

OFFSHORE ALTERNATIVE PLAN

		Plan Components and Costs						Section Costs			
			Cost per Lineal Foot			Cost per Lineal Foot					
Bluff Analysis Section	Shoreline Length (feet)	Onshore Components	Capital	Annual Maintenance	Offshore Components	Capital	Annual Maintenance	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual Cost
1	4,470	Reconstruct existing public bulkhead	$800	$15	- -	$ - -	$- -	$ 3,576,000	$ 67,000	$ 4,633,000	$ 291,000
2	1,410	No protection needed	- -	- -	- -	- -	- -	- -	- -	- -	- -
	1,410	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,200	20	1,692,000	28,000	2,137,000	136,000
3	2,930	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,200	20	3,516,000	59,000	4,440,000	282,000
4	1,980	No protection needed	- -	- -	Construct new public marina and landfill	4,000	30	7,920,000	59,000	9,442,000	599,000
5	1,070	No protection needed	- -	- -	Construct new public marina and landfill	4,000	30	4,280,000	32,000	4,784,000	304,000
6	1,170	Construct new private revetment	300	10	Construct new public offshore island or peninsula	1,200	20	1,755,000	35,000	2,308,000	147,000
7	1,000	Construct new private revetment	300	10	Construct new public offshore island or peninsula	1,200	20	1,500,000	30,000	1,973,000	125,000
8	540	Maintain existing public structures	0	10	Construct new public offshore island or peninsula	1,200	20	648,000	16,000	903,000	57,000
9	570	Construct new private revetment	300	10	Construct new public offshore island or peninsula	1,200	20	855,000	17,000	1,125,000	72,000
10	400	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,200	20	480,000	8,000	606,000	39,000
11	1,290	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,200	20	1,548,000	26,000	2,000,000	127,000
12	3,160	Reconstruct existing bulkhead	750	15	Construct new public offshore island or peninsula	1,200	- -	2,370,000	47,000	3,117,000	198,000
13	1,320	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,200	20	1,584,000	26,000	2,000,000	127,000
14	1,310	Construct new private revetment	200	5	Construct new public offshore island or peninsula	1,200	20	1,834,000	33,000	2,351,000	150,000
15	790	Construct new private revetment	200	5	Construct new public offshore island or peninsula	1,200	20	1,106,000	20,000	1,418,000	90,000
16	470	Construct new private revetment	200	5	Construct new public offshore island or peninsula	1,200	20	658,000	11,000	844,000	53,000
17	440	Construct new private revetment	200	5	Construct new public offshore island or peninsula	1,200	20	616,000	11,000	790,000	50,000
18	220	Construct new public revetment	400	10	Construct new public offshore island or peninsula	1,200	20	359,000	7,000	465,000	30,000
	1,660	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,200	20	1,985,000	33,000	2,507,000	159,000
19	700	Reconstruct existing private revetment	200	10	- -	- -	- -	140,000	7,000	250,000	16,000
	2,480	Maintain existing public structures	0	15	- -	- -	- -	0	12,000	189,000	12,000
20	1,280	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,200	20	1,536,000	26,000	1,949,000	123,000
21	1,060	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,200	20	1,272,000	21,000	1,606,000	102,000
22	950	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,200	20	1,140,000	19,000	1,440,000	91,000
23	1,200	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,200	20	1,440,000	24,000	1,818,000	115,000
24	1,910	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,200	20	2,292,000	38,000	2,894,000	186,000
25	880	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,200	20	1,056,000	18,000	1,340,000	85,000
26	660	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,200	20	792,000	13,000	1,000,000	63,000
27	1,850	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,200	20	2,220,000	37,000	2,803,000	178,000
28	2,050	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,200	20	2,460,000	41,000	3,106,000	197,000
29	770	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,200	20	924,000	15,000	1,167,000	74,000
30	1,760	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,200	20	2,112,000	35,000	2,667,000	169,000
31	660	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,200	20	720,000	12,000	909,000	58,000
32	340	Maintain existing public structures	- -	- -	Construct new public offshore island or peninsula	1,200	20	408,000	7,000	515,000	33,000
33	2,060	No protection needed (except beach)	- -	- -	Construct new public offshore breakwater with sand beach	1,500	30	3,090,000	62,000	4,067,000	258,000
34	1,780	No protection needed (except beach)	- -	- -	Construct new public offshore breakwater with sand beach	1,500	30	2,670,000	53,000	3,505,000	222,000
35	650	No protection needed (except beach)	- -	- -	Construct new public offshore breakwater with sand beach	1,500	30	975,000	20,000	1,290,000	82,000
36	710	No protection needed (except beach)	- -	- -	Construct new public offshore breakwater with sand beach	1,500	30	1,065,000	21,000	1,396,000	89,000
37	1,010	Construct new public revetment	400	15	- -	- -	- -	404,000	15,000	644,000	41,000
38	1,290	Construct new private revetment	300	10	- -	- -	- -	387,000	13,000	590,000	37,000
39	1,480	Construct new private revetment	300	10	- -	- -	- -	444,000	15,000	677,000	43,000

Table 64 (continued)

Bluff Analysis Section	Shoreline Length (feet)	Onshore Components	Onshore Cost per Lineal Foot: Capital	Onshore Cost per Lineal Foot: Annual Maintenance	Offshore Components	Offshore Cost per Lineal Foot: Capital	Offshore Cost per Lineal Foot: Annual Maintenance	Section Costs: Capital	Section Costs: Annual Maintenance	Section Costs: 50-Year Present Worth	Section Costs: Equivalent Annual Cost
40	820	Reconstruct existing private revetment	$400	$15	- -	$ - -	$- -	$ 328,000	$ 12,000	$ 522,000	$ 33,000
41	1,650	No protection needed	- -	- -	Construct new private marina	2,500	30	4,125,000	50,000	4,905,000	311,000
42	940	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,000	20	940,000	19,000	1,236,000	78,000
43	1,370	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,000	20	1,137,000	27,000	1,563,000	99,000
44	140	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,000	20	140,000	3,000	187,000	12,000
45	80	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,000	20	80,000	2,000	112,000	7,000
46	360	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,000	20	360,000	7,000	470,000	30,000
47	2,470	No protection needed (except beach)	- -	- -	Demolish existing South Shore breakwater; construct new near-shore breakwater with sand beach	1,000	20	2,025,000	87,000	3,389,000	215,000
48	1,420	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,000	20	1,420,000	28,000	1,868,000	119,000
49	340	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,000	20	340,000	7,000	447,000	28,000
50	1,130	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,000	20	1,130,000	23,000	1,486,000	94,000
51	570	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,000	20	570,000	11,000	250,000	48,000
52	450	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,000	20	450,000	9,000	592,000	38,000
53	1,320	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,000	20	1,320,000	26,000	1,736,000	110,000
54	1,360	Construct new public revetment	200	10	Construct new public offshore island or peninsula	1,000	20	1,632,000	41,000	2,282,000	145,000
55	14,760	Maintain existing public structures	0	5	Construct new public offshore island or peninsula	3,000	30	28,800,000	362,000	34,507,000	2,189,000
56	14,360	Maintain existing public structures	0	5	Construct new public offshore island or peninsula	3,000	30	25,650,000	329,000	30,828,000	1,956,000
	1,700	Reconstruct existing public bulkhead	200	5	Construct new public offshore island or peninsula	3,000	30	3,190,000	36,000	3,765,000	239,000
57	3,210	Maintain existing public structures	0	15	- -	- -	- -	0	48,000	760,000	48,000
58	1,900	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,200	20	2,280,000	38,000	2,879,000	183,000
59	3,540	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,200	20	4,248,000	71,000	5,364,000	340,000
60	2,210	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,200	20	2,652,000	44,000	3,349,000	212,000
61	1,970	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,200	20	2,364,000	39,000	2,985,000	189,000
62	950	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,200	20	1,140,000	19,000	1,440,000	91,000
63	300	Construct new private revetment	200	5	Construct new public offshore island or peninsula	1,200	20	420,000	8,000	539,000	34,000
64	290	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,200	20	348,000	6,000	439,000	28,000
65	1,710	Construct new public revetment	300	10	Construct new public offshore island or peninsula	1,200	20	2,545,000	51,000	3,374,000	214,000
66	170	Reconstruct existing private revetment	100	5	Construct new public offshore island or peninsula	1,200	20	221,000	4,000	289,000	18,000
67	3380	Construct new private revetment	200	5	Construct new public offshore island or peninsula	1,200	20	532,000	10,000	682,000	44,000
68	790	No protection needed (except beach)	- -	- -	Construct new public offshore breakwater with sand beach	1,500	30	1,185,000	24,000	1,563,000	99,000
	1,380	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,200	20	1,656,000	28,000	2,091,000	133,000
69	520	Construct new private revetment	200	5	Construct new public offshore island or peninsula	1,200	20	728,000	13,000	933,000	59,000
70	240	Construct new private revetment	200	5	Construct new public offshore island or peninsula	1,200	20	336,000	6,000	431,000	27,000
71	2,370	Maintain existing private structures	0	5	Construct new public offshore island or peninsula	1,200	20	2,844,000	59,000	3,779,000	240,000
72	850	Construct new private revetment	200	5	Construct new public offshore island or peninsula	1,200	20	1,190,000	21,000	1,526,000	97,000
73	190	Construct new private revetment	200	5	Construct new public offshore island or peninsula	1,200	20	266,000	5,000	342,000	21,000

Table 64 (continued)

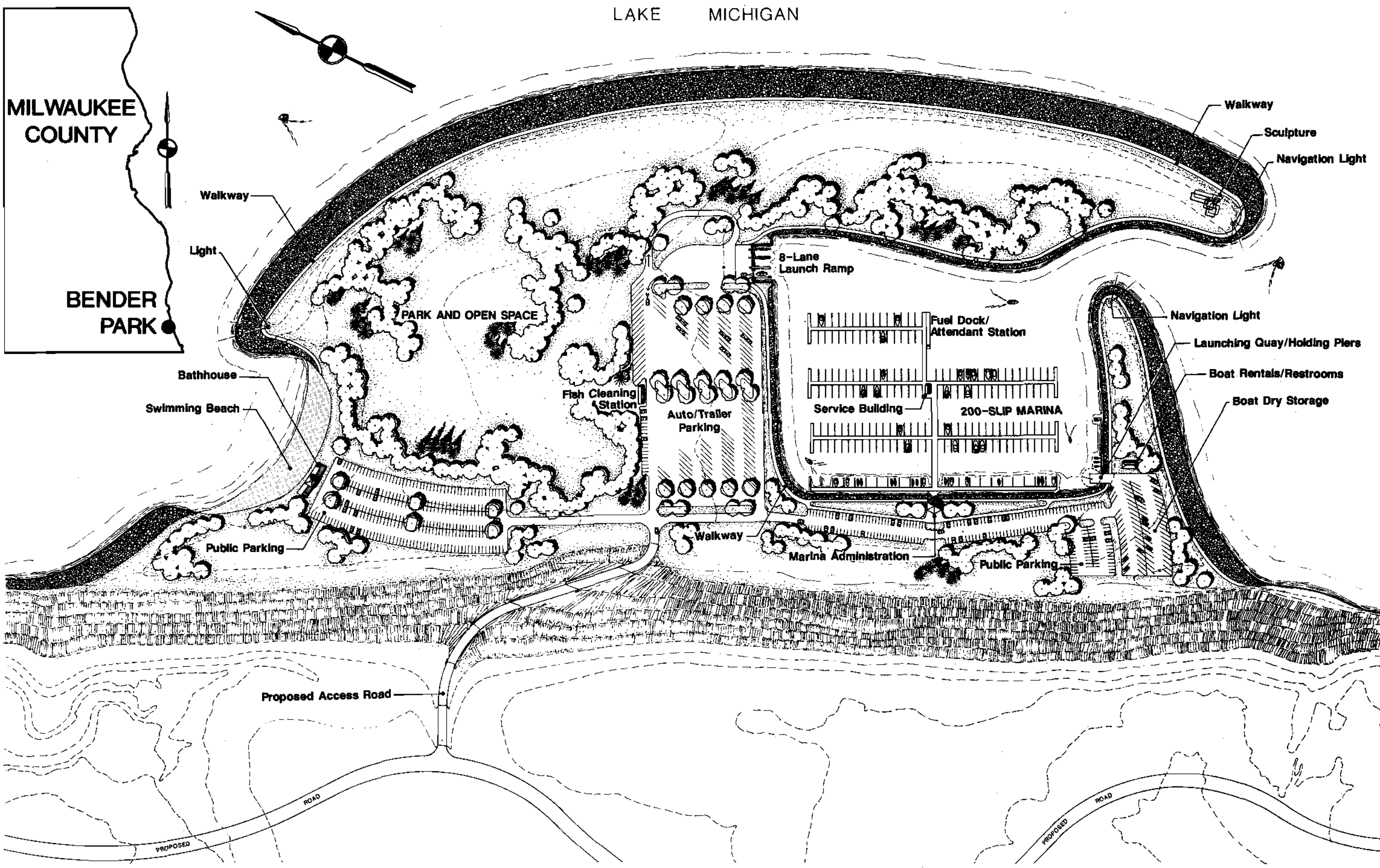
Bluff Analysis Section	Shoreline Length (feet)	Plan Components and Costs: Onshore Components	Cost per Lineal Foot: Capital	Cost per Lineal Foot: Annual Maintenance	Offshore Components	Cost per Lineal Foot: Capital	Cost per Lineal Foot: Annual Maintenance	Section Costs: Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual Cost
74	160	Construct new private revetment	$200	$ 5	Construct new public offshore island or peninsula	$1,200	$20	$ 224,000	$ 4,000	$ 287,000	$ 18,000
75	310	Construct new private revetment	200	5	Construct new public offshore island or peninsula	1,200	20	434,000	8,000	557,000	42,000
76	360	Construct new private revetment	200	5	Construct new public offshore island or peninsula	1,200	20	504,000	9,000	646,000	41,000
77	810	Reconstruct existing private revetment	100	5	Construct new public offshore island or peninsula	1,200	20	1,053,000	20,000	1,423,000	27,000
78	600	Construct new public revetment	300	10	Construct new public offshore island or peninsula	1,200	20	640,000	14,000	855,000	54,000
	1,060	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,200	20	812,000	13,000	1,026,000	65,000
79	1,480	Reconstruct existing private revetment	100	5	Construct new public offshore island or peninsula	1,200	20	1,924,000	37,000	2,508,000	159,000
80	130	Construct new private revetment	200	5	Construct new public offshore island or peninsula	1,200	20	182,000	4,000	229,000	15,000
81	1,700	Reconstruct existing private revetment	100	5	Construct new public offshore island or peninsula	1,200	20	2,210,000	43,000	2,880,000	183,000
	1,270	Maintain existing private revetment	0	5	Construct new public offshore island or peninsula	1,200	20	1,354,000	31,000	2,024,000	128,000
82	490	Construct new private revetment	200	5	Construct new public offshore island or peninsula	1,200	20	686,000	13,000	280,000	56,000
83	140	Construct new private revetment	200	5	Construct new public offshore island or peninsula	1,200	20	196,000	4,000	251,000	17,000
84	430	Construct new private revetment	200	5	Construct new public offshore island or peninsula	1,200	20	602,000	11,000	773,000	49,000
85	480	No protection needed (except beach)	- -	- -	Maintain existing public breakwater and beach	0	30	0	14,000	227,000	14,000
86	170	Maintain existing private structures	0	5	Construct new public offshore island or peninsula	1,200	20	204,000	4,000	272,000	17,000
87	1,950	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,200	20	2,340,000	39,000	2,955,000	187,000
88	1,150	Construct new private revetment	200	5	Construct new public offshore island or peninsula	1,200	20	1,610,000	29,000	2,064,000	131,000
89	320	Maintain existing private structures	0	5	Construct new public offshore island or peninsula	1,200	20	384,000	8,000	510,000	33,000
90	470	Reconstruct existing private revetment	100	5	Construct new public offshore island or peninsula	1,200	20	611,000	11,000	797,000	50,000
91	510	Maintain existing private structures	0	5	Construct new public offshore island or peninsula	1,200	20	612,000	13,000	814,000	52,000
92	770	Maintain existing private structures	0	5	Construct new public offshore island or peninsula	1,200	20	924,000	19,000	1,229,000	78,000
93	530	Maintain existing private structures	0	5	Construct new public offshore island or peninsula	1,200	20	636,000	14,000	846,000	54,000
94	1,460	Maintain existing private structures	0	5	Construct new public offshore island or peninsula	1,200	20	1,752,000	36,000	2,327,000	147,000
95A	2,390	Maintain existing private structures	0	10	- -	- -	- -	0	24,000	377,000	24,000
95B	1,600	No protection needed (except beach)	- -	- -	Construct new public offshore breakwater with sand beach	1,500	30	2,400,000	80,000	3,661,000	232,000
95C	3,000	Maintain existing private structures	0	10	- -	- -	- -	0	30,000	473,000	30,000
95D	720	Reconstruct existing public bulkhead	400	15	- -	- -	- -	288,000	11,000	458,000	29,000
95E	1,360	Maintain existing private structures	0	10	- -	- -	- -	0	14,000	214,000	14,000
96	1,890	No protection needed (except beach)	- -	- -	Construct new public offshore breakwater with sand beach	1,500	30	2,835,000	57,000	3,733,000	237,000
97	4,660	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,200	20	5,592,000	93,000	7,061,000	448,000
98	860	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,200	20	1,032,000	17,000	1,303,000	83,000
99	1,280	No protection needed	- -	- -	Construct new public offshore island or peninsula	1,200	20	1,536,000	26,000	1,940,000	123,000
100	1,320	Construct new private revetment	200	5	Construct new public offshore island or peninsula	1,200	20	1,848,000	33,000	2,368,000	150,000
Total	159,110	- -	$ - -	$- -	- -	$ - -	$- -	$199,846,000[a]	$3,445,000[a]	$253,943,000	$16,125,000

[a]*The total offshore alternative plan would have a capital cost per lineal foot of about $1,256 and an annual maintenance cost per lineal foot of about $22.*

Source: SEWRPC.

Map 43

PROPOSED MARINA AT BENDER PARK, CITY OF OAK CREEK



Source: Warzyn Engineering, Inc.

material would be spoil from the Milwaukee Metropolitan Sewerage District deep tunnel project, and construction and demolition debris. It was estimated that 1.8 million cubic yards of spoil were expected to be removed from the tunnels by the end of 1989.[15] An additional 500,000 cubic yards is expected to be removed during 1990 and 1991. Coast-Tec Construction Company, Ltd., a major lakefill contractor in the Milwaukee area, reports that an average of 35,000 to 50,000 cubic yards of construction and demolition debris per year is generally available for the construction of lakefill projects in Milwaukee County.[16] This fill material is from certain types of construction and demolition projects—often street reconstruction and sewer work—which usually are located relatively close to the landfill site. At present, it is not economically feasible to haul construction debris long distances for disposal. This existing fill material is considered to be available at little or not cost. Based on the large needed volume of fill material, and the relatively small amounts available, it is impractical to implement even major portions of the offshore alternative plan. Addi-

[15]*Warzyn Engineer, Inc., and Johnson, Johnson, and Roy, Inc., Milwaukee County Shoreline Reconnaissance, Milwaukee, Wisconsin, Report Project No. 26021, Prepared for the Milwaukee County Department of Public Works, June 1987.*

[16]*William T. Painter, President, Coast-Tec Construction Company, Ltd., Personal Communication, April 14, 1989.*

tional volumes of fill could be purchased from more outlying areas, but, at a cost of up to $10 per cubic yard, the already high cost of the offshore alternative plan could be doubled or tripled. Even if a sufficient amount of fill were available, the plan probably could not be implemented by groups of private property owners; implementation would have to be carried out by a public agency or agencies. Although a high degree of shore protection would be provided, a usable beach would not be provided along most of the existing shoreline. Thus, easy access to the water in most shoreline areas would continue to be limited.

Milwaukee Outer Harbor Breakwater Alternative Plans: The development of the Milwaukee Harbor was discussed in Chapter II of this report. Construction of the breakwater began in 1877, and the 3.9-mile-long breakwater was completed by 1929 by the U. S. Army Corps of Engineers. Although completion of the breakwater in 1929 provided a much safer harbor than previously existed, the breakwater did not entirely eliminate damage and danger in the outer harbor. The breakwater has been repaired and modified periodically by the Army Corps of Engineers. At the present time, storm waves frequently overtop the structure and occasionally damage port facilities and shore protection structures. During storms, hazardous conditions exist for small craft even within the confines of the breakwater, and even in the McKinley Park small craft anchorage area. Wind storms over Lake Michigan periodically cause damage to the shoreline within the outer harbor due, primarily, to large waves incoming through the breakwater gaps, and to wave energy transmitted over the breakwater. A storm on April 9, 1973, caused about $280,000 in damage in the outer harbor, and provided evidence that additional protective measures may be needed. Severe damages to the harbor shoreline and Port of Milwaukee facilities also occurred during the March 9, 1987, storm event, when a 100-year recurrence interval instantaneous maximum water level of 584.3 feet NGVD was reached. Some of these damages are shown in Figure 106. Protection of the shoreline and riparian facilities behind the breakwater forming the outer harbor of Milwaukee has become increasingly important as the number of facilities has increased and as the level of Lake Michigan has risen above previous record highs for the twentieth century.

Larger commercial vessels can safely moor in the open water of the outer harbor during large storms, but berthing at the municipal piers during very severe storms can be hazardous. Storm waves moving unimpeded through the main harbor entrance into the slip adjacent to South Pier 1, shown on Map 16 in Chapter II, reflect off the vertical dockwall and thus cause the development of standing waves having about twice the height of the incoming waves. Waves as high as 13 feet were reported in this slip during the storm of April 9, 1973. Such standing waves generate very strong reversing horizontal currents. These currents, combined with the violent vertical motion of the water surface, severely tax mooring lines and repeatedly push moored vessels into the pier walls, causing damage to both vessels and walls. During a severe storm on December 26, 1979, the vessel E. M. Ford, owned by the Huron Cement Company, sank at berth in Slip 1 after repeated collisions with the pier. Similar but less severe problems occur in Slip 2.

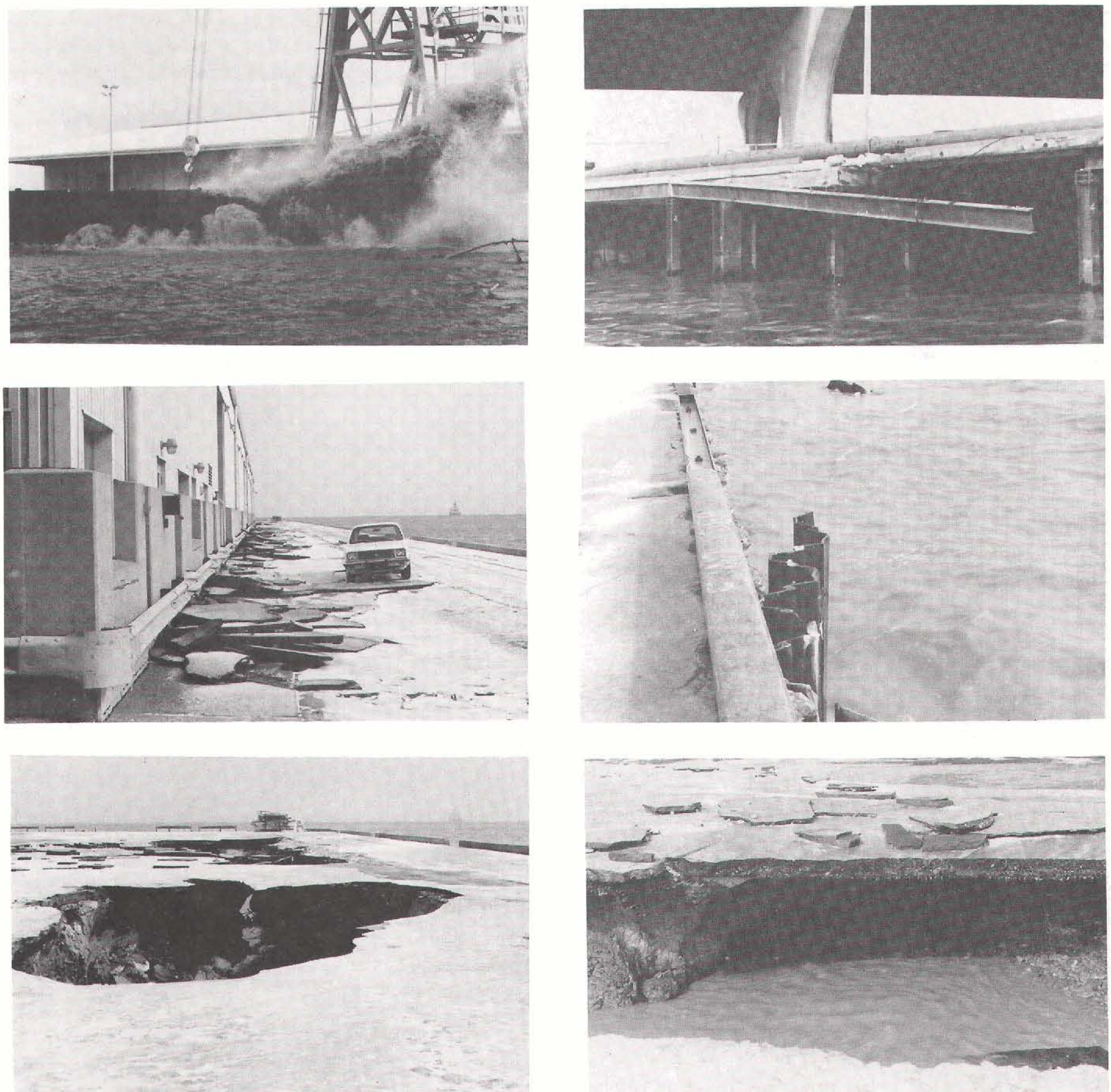
The large standing waves generated by storms from the northeast and occurring at the dock walls of the municipal piers not only create hazardous berthing conditions in the slips, but also have caused flooding on Jones Island. Crests of these waves can peak higher than the top of the dock walls. Strong onshore winds cause these waves to break over the top of the walls and into adjacent buildings. The storm of April 9, 1973, which overtopped the north breakwater, caused standing waves which overtopped the dock wall and crashed into adjacent buildings, staving in doors and creating standing water and water damage. Flooding also occurred during the March 9, 1987, storm event.

In 1977, the U. S. Army Corps of Engineers constructed a parapet on top of the north breakwater from the shoreline to the mouth of the McKinley Park anchorage area to protect pedestrians on the pier during flight from rising storms on the lake. The parapet, constructed to an elevation of 589.4 feet NGVD, also increased the effective height of the breakwater by about three feet, and significantly reduced the frequency and severity of wave conditions in the outer harbor.

In the McKinley Marina anchorage area, wave heights are generally smaller than in the main outer harbor because the area is better protected

Figure 106

DAMAGES TO THE HARBOR SHORELINE AND PORT OF MILWAUKEE FACILITIES CAUSED BY THE MARCH 9, 1987, STORM EVENT



Source: Port of Milwaukee.

from northeast winds, which generally produce the largest waves in the Milwaukee coastal zone. Significant overtopping of the north breakwater can occur, however. Waves transmitted over the breakwater are reflected off the vertical walls bordering nearly all of the anchorage area. Waves are reflected in many directions and could create damaging conditions. However, damage to recreational craft and marina facilities from Lake Michigan storms has not been a major problem during the small craft boating season.

The design height of the breakwater was set by the U. S. Army Corps of Engineers at a time when little long-term water-level data were available for use in determining a design lake level. Also, at that time wave characteristics were yet to be accurately and systematically measured so that a realistic design wave could be selected. Consequently, the existing breakwater height is inadequate to protect fully against the design lake levels set forth in Table 51. Therefore, additional onshore protection measures are needed to protect against erosion. Moreover, as noted above, severe storms can at times damage vessels berthed at the municipal piers. As described in Chapter III, most of the onshore protection structures within the outer harbor are overtopped during severe storms. It should, however, be noted that the Army Corps of Engineers believes that the outer harbor breakwaters are adequately performing their primary intended purpose—that is, to provide safe harbor access for commercial navigation.[17] The Corps has concluded that it is more cost-effective to repair the storm damage that occasionally occurs than it is to substantially modify the breakwaters.

Four alternatives were developed and evaluated to protect the Milwaukee Harbor from severe and damaging wave action. To properly estimate the total cost of protection, costs of both the offshore and onshore measures were calculated. Thus, the trade-off between providing offshore protection and onshore protection was addressed. The first alternative, continued maintenance of the breakwater, was incorporated into the revetment and beach alternative plans for the entire county shoreline. The second and third alternatives would involve increasing the height of the breakwater. The fourth alternative, which would involve the construction of islands and peninsulas, was incorporated into the offshore alternative plan for the County. A description of each outer harbor breakwater alternative, along with the advantages and disadvantages and the costs of each alternative, are summarized in Table 65 and discussed below.

<u>*Alternative No. 1—Continued Maintenance of Existing Outer Harbor Breakwater*</u>: Alternative No. 1, shown on Map 44, would involve the continued maintenance of the breakwater at its existing elevation. Of the alternatives considered, this alternative provides the least offshore protection, and therefore requires the most onshore protection. This alternative is also the only option which would not substantially further restrict the view of the open lake from the shoreline. Alternative No. 1 would entail a capital cost of approximately $9.0 million (all for onshore measures), an annual maintenance cost of about $1.2 million, and an equivalent annual cost of about $1.7 million.

<u>*Alternative No. 2—Reconstruct Outer Harbor Breakwater to Raise Elevation by 8.7 Feet with Construction of Poured Concrete Wall*</u>: Alternative No. 2, shown on Map 45, would include construction of an eight-foot-wide, 8.7-foot-high poured concrete wall to enhance offshore protection, as shown in Figure 107. Wave energy would continue to enter the harbor through the breakwater openings. A structural engineering analysis would be required to determine whether the existing breakwater would be able to support the new wall. The only onshore measure that would need to be reconstructed is the bulkhead protecting the Milwaukee County War Memorial Center, which has a toe scouring problem. Onshore storm damages could be reduced by as much as 80 percent.[18] Waves would reflect off the new breakwater wall, which could increase wave oscillation within the harbor and make navigation difficult close to the wall and in the harbor openings. Waves reflecting off vertical waves can create standing waves twice the height of the incoming waves. Such standing waves generate strong horizontal currents, which, combined with the violent vertical motion of the water surface, make navigation hazardous. This alternative would severely restrict the view of the open lake from the shoreline. Alternative No. 2 would entail a capital cost of approximately $30.3 million, an annual maintenance cost of about $1.1 million, and an equivalent annual cost of about $3.0 million.

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[17] *Mark S. Grazioli, Chief, Construction-Operations Division, Detroit District, U. S. Army Corps of Engineers, Letter to Mr. Kevin Conlon, City of Milwaukee Fiscal Liaison Office, September 9, 1987.*

[18] *SEWRPC Planning Report No. 37, <u>A Water Resources Management Plan for the Milwaukee Harbor Estuary</u>, Volume Two, <u>Alternative and Recommended Plans</u>, December 1987.*

Table 65

COMPARISON OF MILWAUKEE OUTER HARBOR BREAKWATER ALTERNATIVES

				Cost							
				Onshore Protection		Offshore Protection		Total			
Alternative Number	Description	Advantages	Disadvantages	Capital	Annual Maintenance	Capital	Annual Maintenance	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual
1	Continued maintenance of existing breakwater	1. Low cost 2. Maintenance of lake view from the shoreline	1. Damage from a 100-year recurrence interval storm could total $600,000 2. Moderate risk of damage to commercial and recreational vessels	$9,025,000	$308,000	$ --	$860,000	$ 9,025,000	$1,168,000	$27,436,000	$1,740,000
2	Reconstruct breakwater to raise elevation by 8.7 feet with construction of 8-foot-wide poured concrete wall	1. Average annual storm damages would be reduced by up to 80 percent 2. Reduced maintenance of onshore measures 3. Reduced risk of vessel damage, and safer harborage	1. High cost 2. Concrete wall would reflect wave energy both inside and outside harbor, creating high standing waves and making navigation hazardous at times 3. Wall would restrict view of open lake from shoreline	340,000	216,000	30,000,000	860,000	30,340,000	1,076,000	47,300,000	3,001,000
3	Reconstruct breakwater to raise elevation by 8.7 feet by enclosing the existing breakwater within a new rubblemound breakwater	1. Average annual storm damages would be reduced by up to 80 percent 2. Reduced maintenance of onshore measures 3. Reduced risk of vessel damage, and safer harborage 4. Less wave reflection than under Alternative No. 2	1. Very high cost 2. Breakwater would restrict view of open lake from shoreline	340,000	216,000	65,000,000	860,000	65,340,000	1,076,000	82,300,000	5,221,000
4	Reconstruct breakwater to form islands and peninsulas	1. Average annual storm damages could be reduced by up to 80 percent 2. Reduced maintenance of onshore measures 3. Reduced risk of vessel damage, and safer harborage 4. Less wave reflection than under Alternative No. 2 5. Creation of about ___ acres of new lakefront parkland and ___ miles of new shoreline	1. Very high cost 2. Islands and further peninsulas would restrict view of open lake from shoreline 3. Over 24 million cubic yards of fill would be required. It appears impractical to acquire this large volume of fill at little or no cost. Purchasing the fill, at a cost of up to $10 per cubic yard, would make the cost of this already expensive alternative prohibitive	340,000	154,000	57,300,000	573,000	57,640,000	727,000	69,099,000	4,384,000

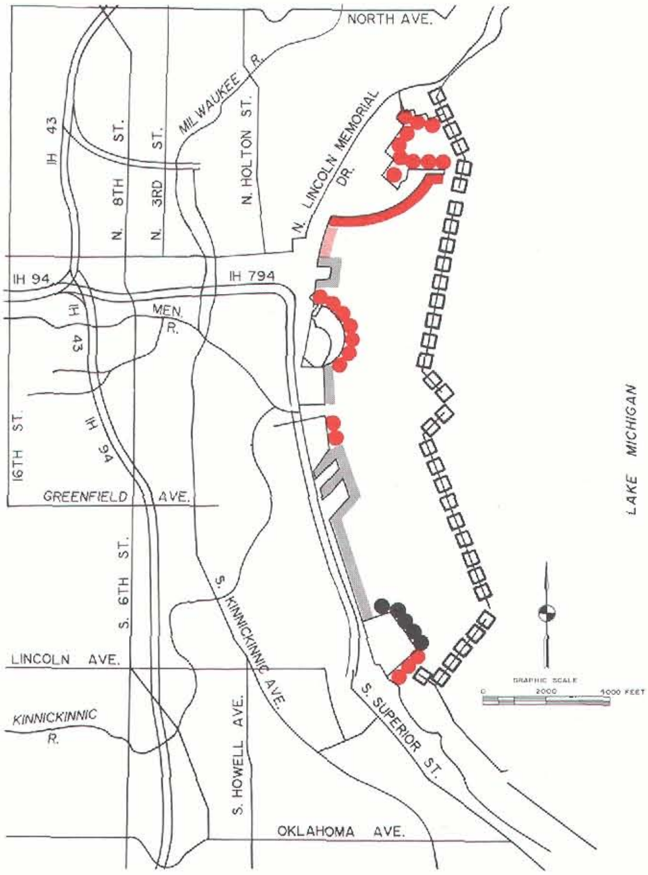
Source: SEWRPC.

Alternative No. 3—Reconstruct Outer Harbor Breakwater to Raise Elevation by 8.7 Feet by Enclosing Within a New Rubblemound Breakwater: Under this alternative, as shown on Map 46, the breakwater would be enclosed within a new rubblemound breakwater, increasing the height of the structure by about 8.7 feet, as shown in Figure 108. The onshore benefits would be similar to those of Alternative No. 2. However, navigation would be less hazardous near the harbor openings, and waves within the harbor may be expected to be slightly lower than under Alternative No. 2 because the rubblemound would reflect less wave energy than would a concrete wall. This alternative would severely restrict the view of the open lake from the shoreline. Alternative No. 3 would entail a capital cost of approximately $65.3 million, an annual maintenance cost of about $1.1 million, and an equivalent annual cost of about $5.2 million.

Alternative No. 4—Reconstruct Outer Harbor Breakwater to Form Islands and Peninsulas: Alternative No. 4, shown on Map 47, would provide the most offshore protection and requires the least onshore protection. Only the bulkhead protecting the Milwaukee County War Memorial Center, which has a toe scouring problem, would need to be reconstructed. Annual

Map 44

MILWAUKEE OUTER HARBOR BREAKWATER ALTERNATIVE NO. 1—CONTINUED MAINTENANCE



LEGEND

OFFSHORE PROTECTION MEASURE

MAINTAIN EXISTING PUBLIC BREAKWATER

ONSHORE PROTECTION MEASURE

MAINTAIN EXISTING PUBLIC STRUCTURE

RECONSTRUCT EXISTING BULKHEAD WITH RIP-RAP BERM

RECONSTRUCT EXISTING BULKHEAD WITH HEIGHT EXTENSION

RECONSTRUCT EXISTING BULKHEAD WITH BERM AND EXTENSION

RECONSTRUCT EXISTING PUBLIC REVETMENT

Source: SEWRPC.

Map 45

MILWAUKEE OUTER HARBOR BREAKWATER ALTERNATIVE NO. 2 *RECONSTRUCT TO RAISE ELEVATION BY* 8.7 FEET WITH POURED CONCRETE WALL

LEGEND

OFFSHORE PROTECTION MEASURE

CONSTRUCT POURED CONCRETE WALL

ONSHORE PROTECTION MEASURE

MAINTAIN EXISTING PUBLIC STRUCTURE

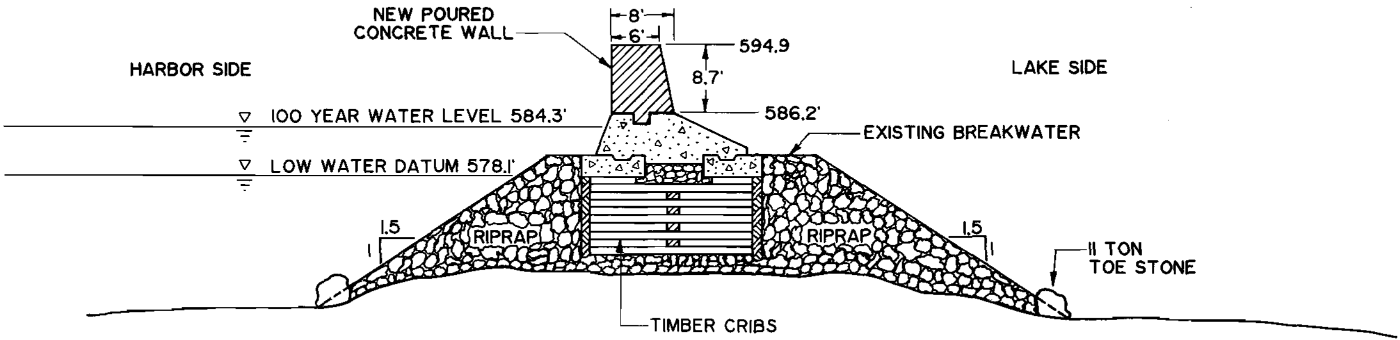
RECONSTRUCT EXISTING BULKHEAD WITH RIP-RAP BERM

Source: SEWRPC.

maintenance of onshore measures may be expected to be even lower than under Alternatives No. 2 or No. 3. The islands would reflect little wave energy and virtually eliminate wave overtopping damages. Under this alternative, about 390 acres of lakefront parkland and 10 miles of shoreline would be created, offering enhanced recreational opportunities in proximity to the central portion of the City of Milwaukee. This alternative would further restrict the view of the open lake from the existing shoreline. A large amount of fill material—about 24 million cubic yards—would be required for the construction of the islands and peninsulas. It was assumed, for costing purposes, that this fill material—either spoil from the deep tunnel

Figure 107

MILWAUKEE OUTER HARBOR ALTERNATIVE NO. 2—TYPICAL CROSS-SECTION OF BREAKWATER WITH NEW 8.7-FOOT-HIGH CONCRETE WALL

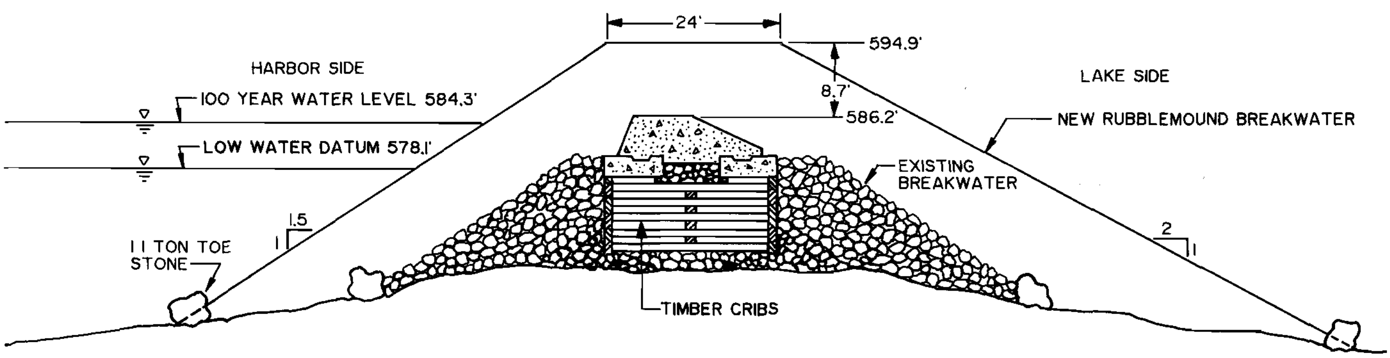


NOTE: ALL ELEVATIONS ARE IN FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM.

Source: SEWRPC.

Figure 108

MILWAUKEE OUTER HARBOR ALTERNATIVE NO. 3—TYPICAL CROSS-SECTION OF NEW RUBBLEMOUND BREAKWATER



NOTE: ALL ELEVATIONS ARE IN FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM.

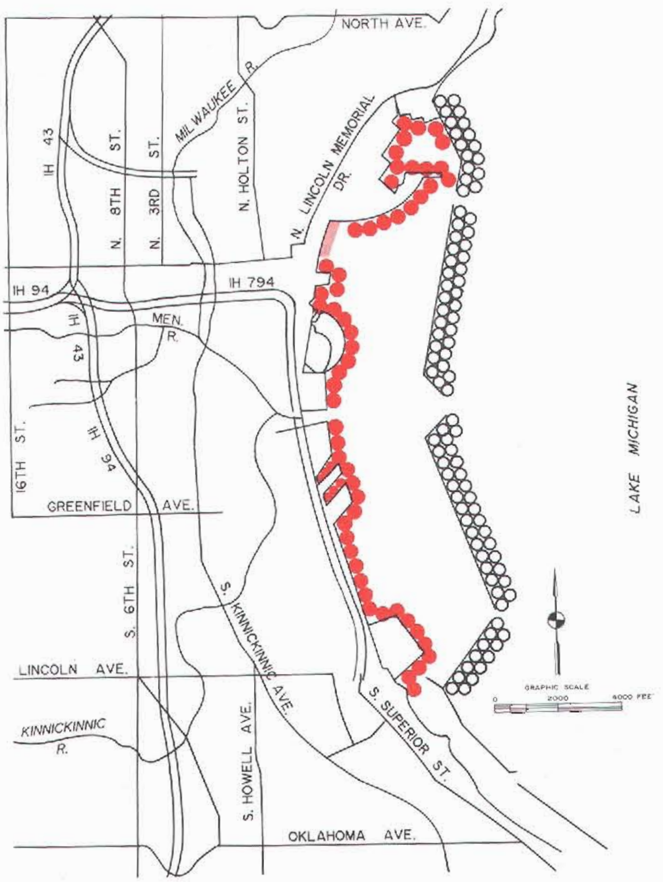
Source: SEWRPC.

project or construction and demolition debris—could be obtained at little or no cost. As mentioned in the discussion of the offshore alternative plan, however, it does not appear that this large volume of fill material would be available in the Milwaukee area at little or no cost. Purchasing the fill material would make the cost prohibitive. Thus, this alternative does not appear feasible at this time. Under federal law, constructing and maintaining the Milwaukee outer harbor breakwater is the responsibility of the U. S. Army Corps of Engineers. Since this alternative would provide substantial benefits besides protection of navigation, it is likely that an act of the United States Congress would be required to relinquish this responsibility to a local unit of government. If a sufficient amount of fill material were available at little or no cost, Alternative No. 4 would entail a capital cost of approximately $57.6 million, an annual maintenance cost of about $0.7 million, and an equivalent annual cost of about $4.4 million. If fill material had to be purchased, the capital cost could increase by up to four times this amount.

South Shore Breakwater Alternative Plans: The 12,500-foot-long South Shore breakwater was constructed in segments between 1913 and 1936, as shown on Map 48. Most of the segments were constructed by the City of Milwaukee, although

Map 46

MILWAUKEE OUTER HARBOR BREAKWATER ALTERNATIVE NO. 3 RECONSTRUCT TO RAISE ELEVATION BY 8.7 FEET WITH NEW RUBBLEMOUND BREAKWATER



LEGEND

OFFSHORE PROTECTION MEASURE

CONSTRUCT NEW RUBBLEMOUND BREAKWATER

ONSHORE PROTECTION MEASURE

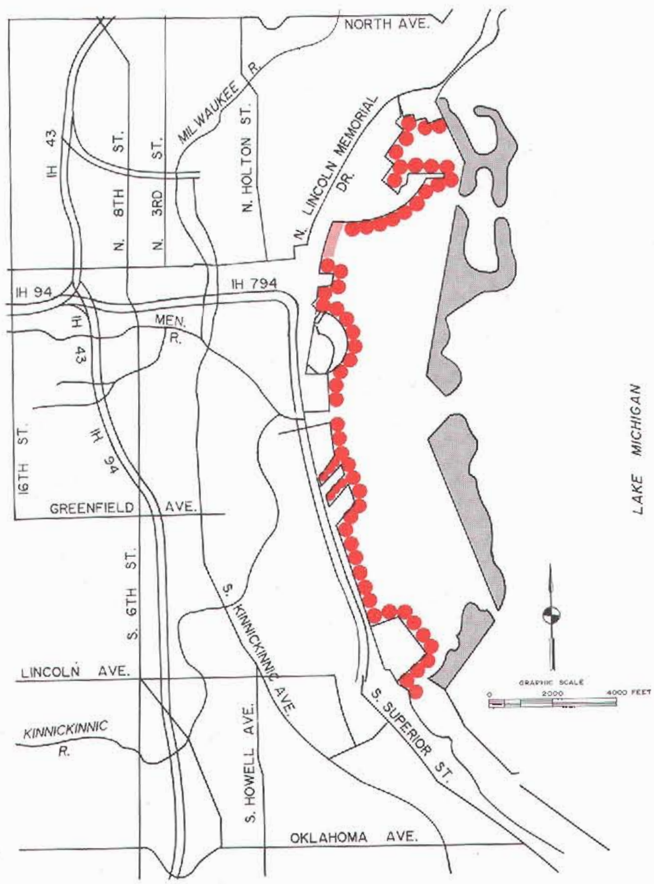
MAINTAIN EXISTING PUBLIC STRUCTURE

RECONSTRUCT EXISTING BULKHEAD WITH RIP-RAP BERM

Source: SEWRPC.

Map 47

MILWAUKEE OUTER HARBOR BREAKWATER ALTERNATIVE NO. 4—ISLANDS AND PENINSULAS



LEGEND

OFFSHORE PROTECTION MEASURE

CONSTRUCT ISLANDS AND PENINSULAS

ONSHORE PROTECTION MEASURE

MAINTAIN EXISTING PUBLIC STRUCTURE

RECONSTRUCT EXISTING BULKHEAD WITH RIP-RAP BERM

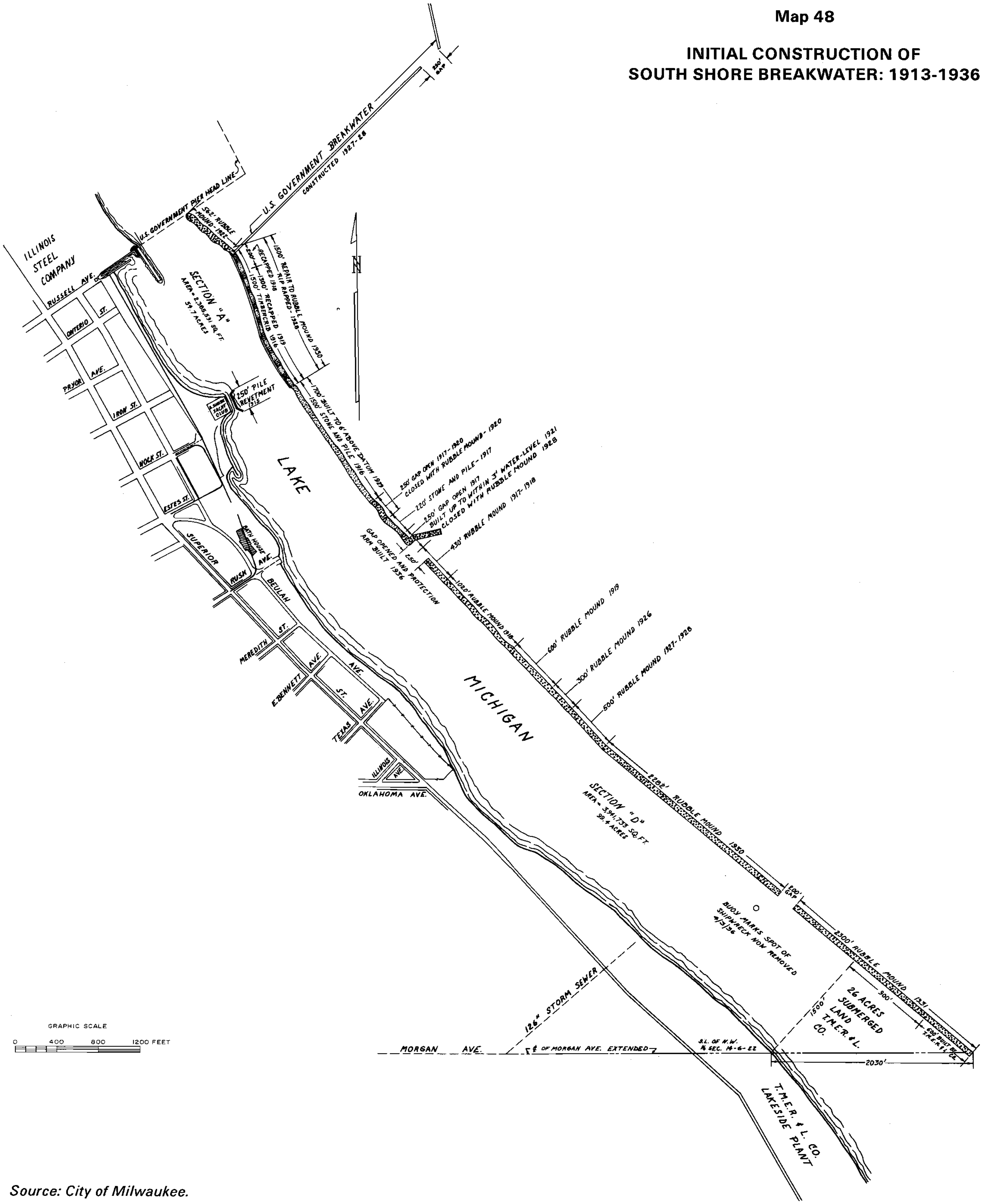
Source: SEWRPC.

the southernmost 600 feet of the breakwater was built by The Milwaukee Electric Railway & Light Company, the predecessor company to the Wisconsin Electric Power Company. The breakwater was essentially completed by 1931. However, in 1936, to improve circulation of water behind the breakwater and to allow improved access, a 250-foot opening was made in the breakwater at E. Bennett Avenue extended. The removed stone was used to build a short angular protection arm on the north side of the new opening. In 1943, the total cost of constructing the South Shore breakwater was estimated at \$2.35 million.[19]

[19] *A. Riemenschneider, City of Milwaukee Memorandum to E. A. Howard, August 2, 1943.*

Map 48

INITIAL CONSTRUCTION OF SOUTH SHORE BREAKWATER: 1913-1936



Source: City of Milwaukee.

As shown on Map 48, the first segments completed, at the northern end of the breakwater, were constructed of rock-filled timber cribs and steel sheet piling. Sheet piling soon became impractical because the hard clay substrate was too difficult to penetrate adequately. These sections were subsequently covered with stone. The remaining sections—south of E. Bennett Avenue—are rubblemound breakwaters.

To build the breakwater, the City of Milwaukee acquired ownership of the immediate shoreline and the associated riparian rights from private lakeshore property owners. The shoreline property and riparian rights were first acquired near E. Russell Avenue at the far northern end of the breakwater in 1913. Deed provisions between the property owners—or the grantors—and the City of Milwaukee enabled the City to construct the breakwater in return for the property owners' riparian rights and the ownership of the immediate shoreline. The deed provisions stipulated that the breakwater would be constructed at the outer line of the lakebed grant from the State of Wisconsin.[20] Ownership of the land generally east of the bluff was transferred to the City. The provisions further stated that if the City found that the breakwater provided insufficient protection to prevent erosion of the grantor's land by wave action, the City would construct such shore protection as deemed necessary. The private property owners agreed not to hold the City liable for any future damages to their land or rights related to any shore protection measures constructed by the City.

In 1927, the City of Milwaukee Parks Board was disbanded, and most city parks were transferred to county jurisdiction. The ownership and responsibility for the South Shore breakwater and adjacent parkland, however, apparently remained in question for some time. There were several discrepancies, errors, and omissions made in the transfer which affected the South Shore area.[21] The matter was finally clarified in 1950, when the breakwater, submerged lakebed, and adjacent parkland were formally transferred to the County for the total sum of $9,000.[22] The County thus assumed responsibility for maintenance of the breakwater. Although isolated repairs have been made, little overall maintenance of the breakwater has been performed since 1950. Clearly, the County is responsible for protecting the shoreline behind the existing breakwater. It is unclear, however, whether the deed provisions with the private property owners require that the current breakwater be maintained, or whether the County is free to select alternative methods of shore protection. According to the deed provisions, the County would apparently be able to select alternative protection measures if the breakwater offered inadequate protection. While the breakwater has in the past provided a high degree of protection, some shoreline erosion has continued to occur. For the purposes of this report, it was assumed that alternative methods of protecting the shoreline behind the breakwater could indeed be considered.

Due in part to the overall lack of maintenance, and in part to the relatively high lake levels that have occurred during much of the 1970's and 1980's, portions of the breakwater have collapsed or have washed away. North of the South Shore Park beach, the crest of the breakwater is at about elevation 589 feet NGVD. The crest elevation of the remaining portions of the breakwater ranges from 583 to 586 feet NGVD. The deterioration of the breakwater occurs in two forms. The first form of deterioration is the erosion and steepening of the side slopes of the breakwater. As these side slopes become steeper than about one on one and one-half, they become more susceptible to collapse. The second form of deterioration is the erosion and collapse of the crest, or top, of the breakwater, which reduces both the height and width of the crest. During high-water periods, portions of the breakwater lie just barely above—and in some cases below—the water surface, creating a potential navigation hazard.

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[20] *Warranty Deed Provision between the City of Milwaukee and Christian G. Goelz and Marguite Goelz, July 17, 1918.*

[21] *Letter from the Supervising Engineer for the Milwaukee County Regional Planning Department to J. C. Dretzka, Milwaukee County Park Commission, March 11, 1938.*

[22] *Quit Claim Deed Dated June 13, 1950, recorded in the Milwaukee County Register of Deeds office, in Volume 3024 of the Deeds, page 611, Document No. 3115993, City of Milwaukee to County of Milwaukee.*

Figure 109

WAVES OVERTOPPING THE SOUTH SHORE BREAKWATER ON MARCH 9, 1987

Source: Port of Milwaukee.

Furthermore, as the breakwater deteriorates, more wave energy is transmitted over the breakwater. During a 100-year recurrence interval lake level with a 20-year recurrence interval storm wave, as much as 60 percent of the incoming storm wave height is estimated to be transmitted over the breakwater. Figure 109 shows waves overtopping the breakwater near the South Shore Marina during the March 9, 1987, storm event, when a 100-year recurrence interval maximum instantaneous water level was recorded. The portion of the breakwater shown in the figure is the highest breakwater section. Other breakwater sections were totally submerged during that storm. Wave transmission, and the resulting shoreline erosion behind the breakwater, may be expected to increase as the breakwater further deteriorates. Proper rehabilitation or reconstruction of the breakwater would require rebuilding the side slopes to a stable angle and increasing the elevation, and sometimes the width, of the breakwater crest. In 1979, Milwaukee County estimated that reconstructing the South Shore breakwater to a crest elevation of 588.6 feet NGVD—or eight feet above City of Milwaukee datum—would entail a capital cost of about $8,250,000.[23] Funds for the reconstruction were never allocated by the County.

Six alternatives were developed and evaluated to adequately protect Bluff Analysis Sections 42 through 54, which lie behind, and are protected by, the South Shore breakwater. Five of the alternatives involve various combinations of reconstructing, relocating, and demolishing portions of the breakwater. The sixth alternative would include the construction of islands, peninsulas, and near-shore breakwaters to replace the present breakwater. To properly estimate the total cost of protection, costs of both the offshore and onshore measures were calculated. Thus, the trade-off between offshore measures and onshore measures was addressed; providing less protection offshore would increase the level of needed onshore protection. However, the cost of the onshore measures may actually be reduced in those cases where stone from the breakwater can be used to construct onshore measures. The first five alternatives considered could be incorporated into either the revetment or the beach alternative plans for the entire county shoreline. To provide for a consistent evaluation, it was assumed that revetments would be used to provide onshore protection. Thus, the type of onshore protection provided would be the same for all of the first five alternatives. However, the

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[23]*Thomas Borgwardt, Supervising Engineer, County of Milwaukee Inter-office Communication, South Shore Stone Breakwater Repair, to Irving Heipel, Milwaukee County Landscape Architect, December 21, 1979.*

stone size, design height, and volume of additional stone required—as well as the attendant costs—would vary substantially, depending on the level of offshore protection provided. If beaches, instead of revetments, were utilized to provide the onshore protection, the onshore capital costs would increase by 60 to over 180 percent, depending on the alternative selected, and the onshore annual maintenance costs would increase by 15 to 45 percent. The sixth alternative is the same as shown in the offshore alternative plan. The breakwater would be replaced by islands, peninsulas, and near-shore breakwaters which would contain a new sand beach. A description of each South Shore breakwater alternative, along with the advantages and disadvantages and costs of each alternative, are summarized in Table 66 and discussed below.

Alternative No. 1—Reconstruct Entire Breakwater to 588.6 Feet NGVD: This alternative, as shown on Map 49, would provide a high level of offshore protection and preserve the appearance and use of the shoreline and near-shore area. An elevation of 588.6 feet NGVD was selected because this elevation—eight feet above City of Milwaukee datum—was used by Milwaukee County to estimate the cost of reconstructing the breakwater,[24] and because the northern portion of the breakwater, which offers the greatest protection, has been constructed to about this elevation. This elevation is 4.3 feet above the 100-year recurrence interval instantaneous maximum water level of 584.3 feet NGVD, and thus the breakwater would extend above the waterline and not pose a navigation hazard under even high-water-level conditions. This elevation is also similar to the elevation of the highest portion of the outer harbor breakwater, the north breakwater with the newly constructed parapet having an elevation of about 589.4 feet NGVD. This elevation would be about the same as the current height on the north end, north of the South Shore Park beach. However, the top width would be increased and the side stabilized. In the other areas, the top height would be increased from three to six feet in order to reach elevation 588.6 feet NGVD, as shown in Figures 110 through 120. Wave overtopping would continue to occur during severe storm events; however, only about 38 percent of the wave height would be transmitted over the breakwater, compared to up to 60 percent under existing conditions. Overall, a crest elevation of 588.6 feet was considered to be the maximum elevation that would be practical and economically feasible.

Cross-section drawings of the breakwater, presented in Figures 110 through 120 show the existing breakwater profile, and the proposed profile of the breakwater reconstructed to a crest elevation of 588.6 feet NGVD. The side slopes on the shore side of the reconstructed breakwater would be one on one and one-half, while the lake side would have a slope of one on two. A total of about 116,000 cubic yards of new stone would be required to reconstruct the breakwater. Because the breakwater would be relatively high, relatively low-cost onshore structures would be required to supplement the offshore protection. Alternative No. 1 would entail a capital cost of about $11.4 million and an annual maintenance cost of about $500,000. The equivalent annual cost is $1,224,000. This alternative was used to estimate the costs of the revetment and beach alternative plans in Tables 60 and 62.

Alternative No. 2—Relocate Breakwater South of E. Bennett Avenue Extended to 300 Feet Offshore, and Reconstruct the Entire Breakwater to 588.6 Feet NGVD: Alternative No. 2, shown on Map 50, would provide the same high level of offshore protection as Alternative No. 1, except that the southern two-thirds of the existing breakwater would be relocated closer to shore in shallower water. A smaller volume of stone could then be used to construct a breakwater high enough to provide adequate protection. A total of 41,000 cubic yards of new stone would be required to reconstruct the breakwater under this alternative. However, the shoreline would appear more "enclosed," and the protected surface water area would be reduced by about 50 percent. Onshore protection measures would be the same as those utilized in Alternative No. 1. Alternative No. 2 would entail a capital cost of about $11.0 million, an annual maintenance cost of about $500,000, and an equivalent annual cost of approximately $1,196,000. This alternative has a slightly lower cost than Alternative No. 1 because less new stone would be required to reconstruct the portion of the breakwater that would be placed in shallower

[24]*Ibid.*

Table 66

COMPARISON OF SOUTH SHORE BREAKWATER ALTERNATIVES

Alternative Number	Description	Advantages	Disadvantages	Cost							
				Onshore Protection		Offshore Protection		Total			
				Capital	Annual Maintenance	Capital	Annual Maintenance	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual
1	Reconstruct entire breakwater to 588.6 feet NGVD	1. High degree of offshore protection 2. Minimum need for further onshore protection 3. Appearance and use of shoreline same as existing conditions, except breakwater would be higher 4. Maximum protected surface water area	1. High cost 2. High maintenance requirement for breakwater	$2,112,000	$115,000	$ 9,300,000	$385,000	$11,412,000	$500,000	$19,293,000	$1,224,000
2	Relocate breakwater south of E. Bennett Avenue extended to 300 feet offshore Reconstruct entire breakwater to 588.6 feet NGVD	1. High degree of offshore protection 2. Minimum need for further onshore protection	1. Shoreline would appear more enclosed where breakwater is nearer to shore 2. Protected surface water area would be reduced by 50 percent 3. High cost 4. High maintenance requirement for breakwater	2,112,000	115,000	8,853,000	385,000	10,965,000	500,000	18,846,000	1,196,000
3	Reconstruct entire breakwater to 585.0 feet NGVD	1. Appearance and use of shoreline same as existing conditions 2. Maximum protected surface water area	1. Breakwater would be severely overtopped during severe storms 2. High degree of onshore protection required	3,089,000	169,000	3,318,000	385,000	6,407,000	554,000	15,139,000	960,000
4	Demolish breakwater south of E. Bennett Avenue extended Reconstruct breakwater north of E. Bennett Avenue extended to 588.6 feet NGVD	1. Reduced maintenance requirement for breakwater 2. Low overall maintenance cost	1. Appearance and use of shoreline and near-shore waters would change substantially 2. Protected surface water area would be reduced by 72 percent 3. High degree of onshore protection required	$1,005,000	$159,000	$ 5,962,000	$110,000	$ 6,967,000	$269,000	$11,207,000	$ 711,000
5	Demolish breakwater south of E. Oklahoma Avenue extended Reconstruct breakwater north of E. Oklahoma Avenue extended to 588.6 feet NGVD	1. Only modest change in appearance and use of shoreline 2. Reduced maintenance requirement for breakwater 3. Low cost	1. Protected surface water area would be reduced by 40 percent 2. High degree of onshore protection required south of E. Oklahoma Avenue extended	1,402,000	142,000	3,648,000	231,000	5,050,000	373,000	10,929,000	693,000
6	Replace breakwater with islands, peninsulas and near-shore breakwaters	1. Maximum degree of offshore protection 2. Minimum onshore measures required 3. Creation of a new sand beach, about 115 acres of new public lakefront land and 4 miles of additional shoreline	1. Appearance and use of shoreline and near-shore area would change substantially 2. High cost	272,000	14,000	11,505,000	276,000	11,777,000	290,000	16,348,000	1,037,000

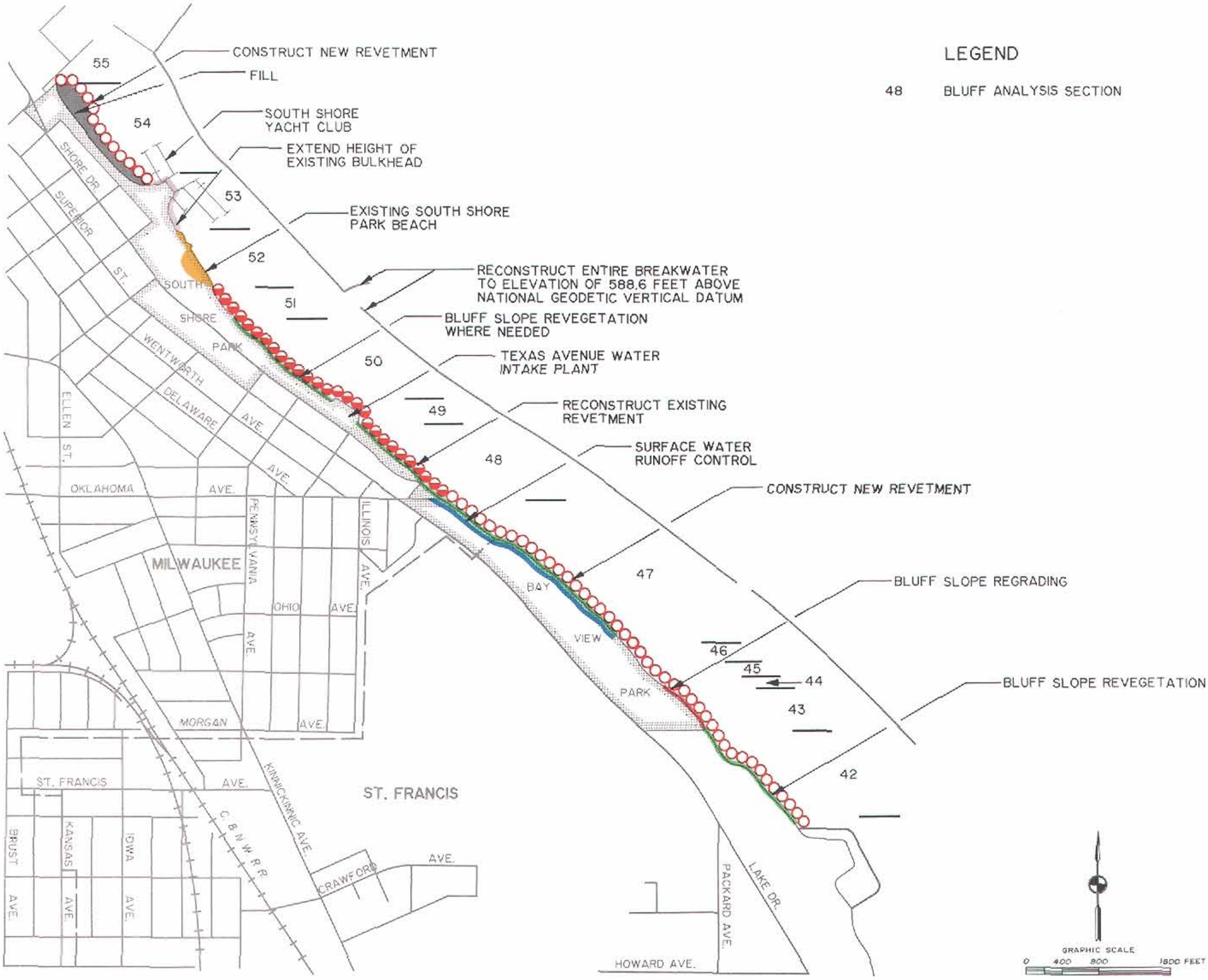
Source: SEWRPC.

water. However, this cost reduction is somewhat offset by the costs entailed in moving this portion of the breakwater closer to shore.

<u>Alternative No. 3—Reconstruct Entire Breakwater to 585.0 Feet NGVD</u>: Under this alternative, which is shown on Map 51, the existing breakwater would be reconstructed to form a stable structure, but the crest elevation would be set at only 585.0 feet NGVD. This crest height would require no increase in the areas north of South Shore Beach and increases in the current height of up to three feet in other areas, as shown in Figures 110 through 120. A crest elevation of 585.0 feet NGVD was selected as the minimal height which would not pose a significant navigation hazard under extended high-water conditions. This design height is about 1.8

Map 49

SOUTH SHORE BREAKWATER ALTERNATIVE NO. 1—RECONSTRUCT BREAKWATER TO 588.6 FEET NGVD

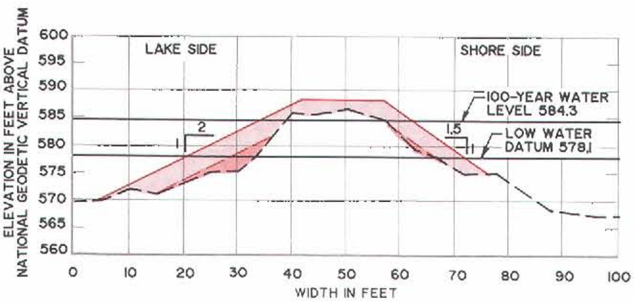


Source: SEWRPC.

feet above the maximum monthly mean water level of 583.2 feet NGVD recorded in October 1986, but only 0.7 foot above the 100-year recurrence interval instantaneous maximum water level of 584.3 feet NGVD. Those portions of the breakwater that are higher than 585.0 feet would remain at their current height. All portions of the breakwater would require the placement of some additional stone to form stable side slopes. Cross-section drawings of the breakwater reconstructed to a crest elevation of at least 585.0 feet NGVD are set forth in Figures 110 through 120. A total of about 31,000 cubic yards of new stone would be required to reconstruct the breakwater. This alternative would preserve the appearance and use of the shoreline and nearshore area. However, compared with Alternatives No. 1 and No. 2, a higher level of onshore protection would be required to compensate for the reduced offshore protection. Alternative No. 3 would entail a capital cost of approximately $6.4 million, an annual maintenance cost of about $554,000, and an equivalent annual cost of about $960,000. The annual maintenance cost is higher than under Alternatives No. 1 and No. 2 because the onshore maintenance costs

Figure 110

SOUTH SHORE BREAKWATER CROSS-SECTION OFFSHORE OF BLUFF ANALYSIS SECTION 43: 1979



LEGEND

— — EXISTING BREAKWATER

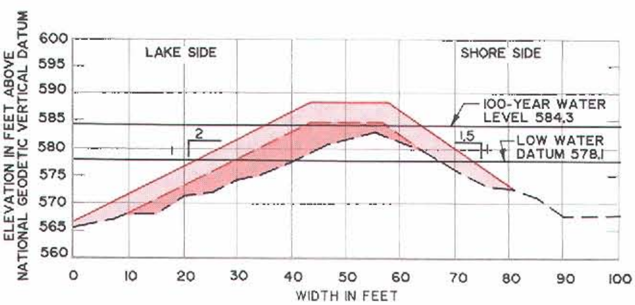
BREAKWATER RECONSTRUCTED TO AN ELEVATION OF 588.6 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM (ALT. 1)

BREAKWATER RECONSTRUCTED TO AN ELEVATION OF 585.0 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM (ALT. 3)

Source: Milwaukee County and SEWRPC.

Figure 111

SOUTH SHORE BREAKWATER CROSS-SECTION OFFSHORE OF BLUFF ANALYSIS SECTIONS 44 AND 45: 1979



LEGEND

— — EXISTING BREAKWATER

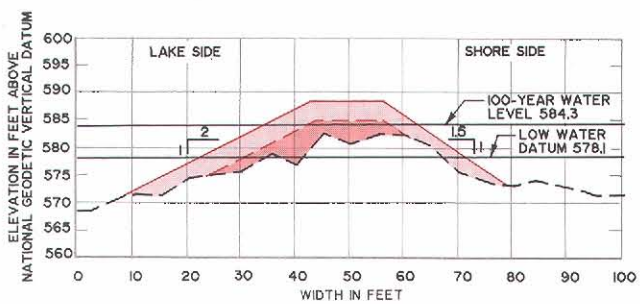
BREAKWATER RECONSTRUCTED TO AN ELEVATION OF 588.6 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM (ALT. 1)

BREAKWATER RECONSTRUCTED TO AN ELEVATION OF 585.0 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM (ALT. 3)

Source: Milwaukee County and SEWRPC.

Figure 112

SOUTH SHORE BREAKWATER CROSS-SECTION OFFSHORE OF BLUFF ANALYSIS SECTION 46: 1979



LEGEND

— — EXISTING BREAKWATER

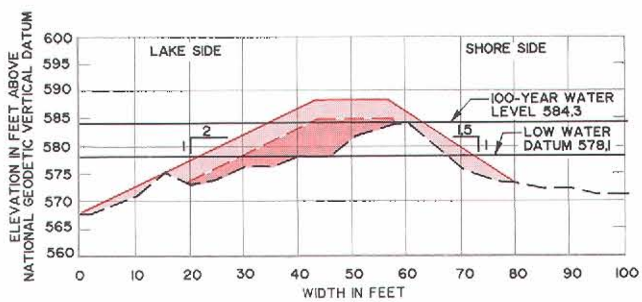
BREAKWATER RECONSTRUCTED TO AN ELEVATION OF 588.6 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM (ALT. 1)

BREAKWATER RECONSTRUCTED TO AN ELEVATION OF 585.0 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM (ALT. 3)

Source: Milwaukee County and SEWRPC.

Figure 113

SOUTH SHORE BREAKWATER CROSS-SECTION OFFSHORE OF BLUFF ANALYSIS SECTION 47: 1979



LEGEND

— — EXISTING BREAKWATER

BREAKWATER RECONSTRUCTED TO AN ELEVATION OF 588.6 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM (ALT. 1)

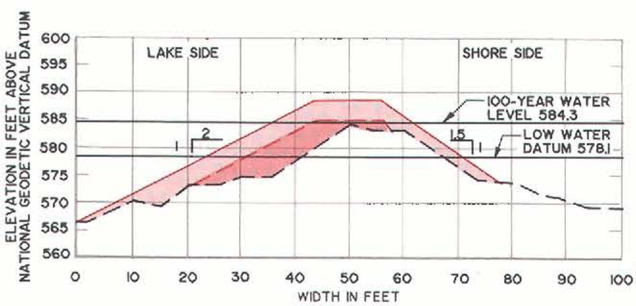
BREAKWATER RECONSTRUCTED TO AN ELEVATION OF 585.0 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM (ALT. 3)

Source: Milwaukee County and SEWRPC.

would increase by about $54,000 per year. Offshore maintenance costs would remain about the same, it being just as costly to maintain a breakwater at a height of 585.0 feet as it is to maintain one at 588.6 feet because the lower breakwater—though containing less stone—would be overtopped more frequently, which could cause more damage and require more frequent repair. During severe storms, about 44 percent of the wave height would be expected to be transmitted over the breakwater, as compared to about 38 percent under Alternative No. 1, and up to 60 percent under existing conditions.

Figure 114

SOUTH SHORE BREAKWATER CROSS-SECTION OFFSHORE OF BLUFF ANALYSIS SECTION 48: 1979



LEGEND

EXISTING BREAKWATER

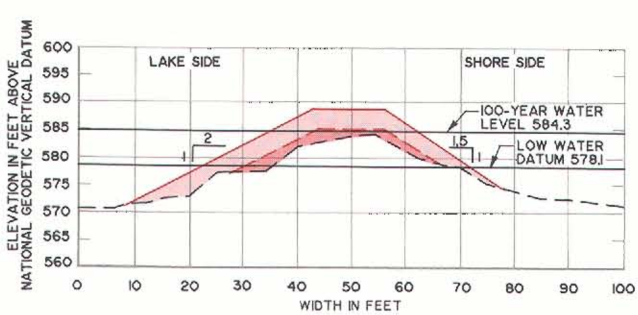
BREAKWATER RECONSTRUCTED TO AN ELEVATION OF 588.6 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM (ALT. 1)

BREAKWATER RECONSTRUCTED TO AN ELEVATION OF 585.0 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM (ALT. 3)

Source: Milwaukee County and SEWRPC.

Figure 115

SOUTH SHORE BREAKWATER CROSS-SECTION OFFSHORE OF BLUFF ANALYSIS SECTION 49: 1979



LEGEND

EXISTING BREAKWATER

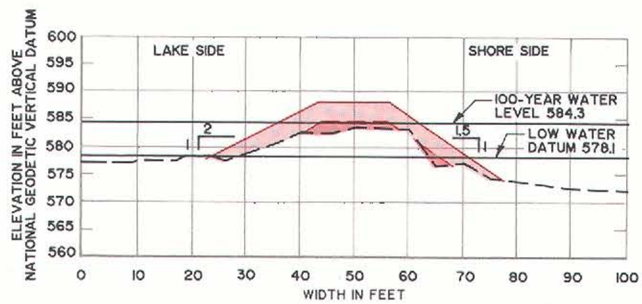
BREAKWATER RECONSTRUCTED TO AN ELEVATION OF 588.6 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM (ALT. 1)

BREAKWATER RECONSTRUCTED TO AN ELEVATION OF 585.0 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM (ALT. 3)

Source: Milwaukee County and SEWRPC.

Figure 116

SOUTH SHORE BREAKWATER CROSS-SECTION OFFSHORE OF BLUFF ANALYSIS SECTION 50: 1979



LEGEND

EXISTING BREAKWATER

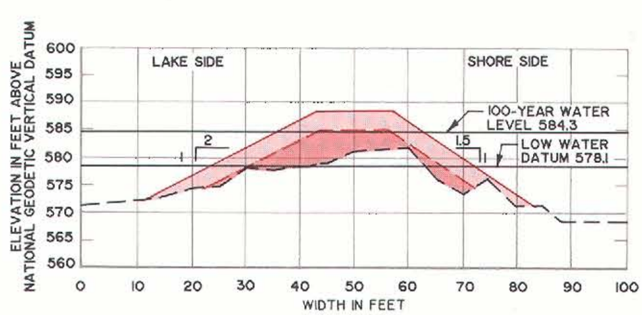
BREAKWATER RECONSTRUCTED TO AN ELEVATION OF 588.6 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM (ALT. 1)

BREAKWATER RECONSTRUCTED TO AN ELEVATION OF 585.0 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM (ALT. 3)

Source: Milwaukee County and SEWRPC.

Figure 117

SOUTH SHORE BREAKWATER CROSS-SECTION OFFSHORE OF BLUFF ANALYSIS SECTION 51: 1979



LEGEND

EXISTING BREAKWATER

BREAKWATER RECONSTRUCTED TO AN ELEVATION OF 588.6 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM (ALT. 1)

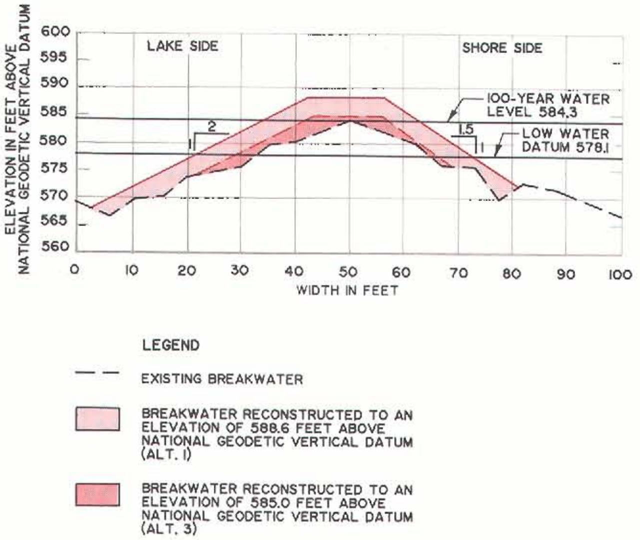
BREAKWATER RECONSTRUCTED TO AN ELEVATION OF 585.0 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM (ALT. 3)

Source: Milwaukee County and SEWRPC.

Alternative No. 4—Demolish Breakwater South of E. Bennett Avenue Extended and Reconstruct Breakwater North of E. Bennett Avenue Extended to 588.6 Feet NGVD: Alternative No. 4, shown on Map 52, would involve demolishing the breakwater south of the South Shore Park beach, with the salvaged breakwater stone being used to construct onshore protection measures, and to reconstruct the remaining portion of the breakwater to provide a high level of protection. The breakwater in Bluff Analysis Section 52 would be relocated to a southerly direction to protect the South Shore Park beach from waves produced by southeasterly winds.

Figure 118

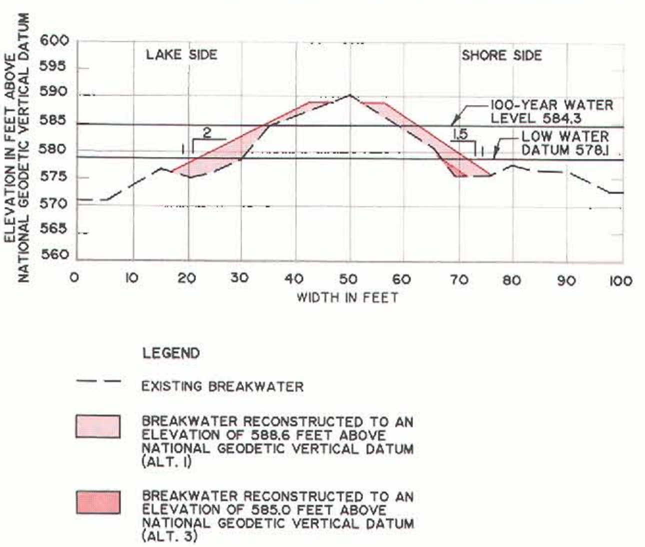
SOUTH SHORE BREAKWATER CROSS-SECTION OFFSHORE OF BLUFF ANALYSIS SECTION 52: 1979



Source: Milwaukee County and SEWRPC.

Figure 119

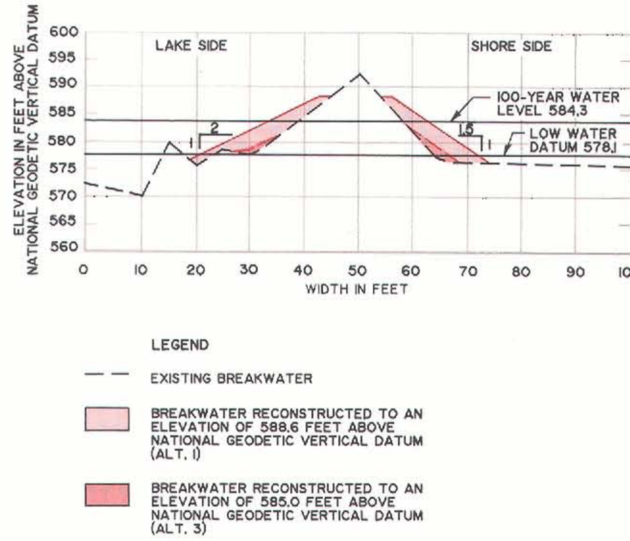
SOUTH SHORE BREAKWATER CROSS-SECTION OFFSHORE OF BLUFF ANALYSIS SECTION 53: 1979



Source: Milwaukee County and SEWRPC.

Figure 120

SOUTH SHORE BREAKWATER CROSS-SECTION OFFSHORE OF BLUFF ANALYSIS SECTION 54: 1979



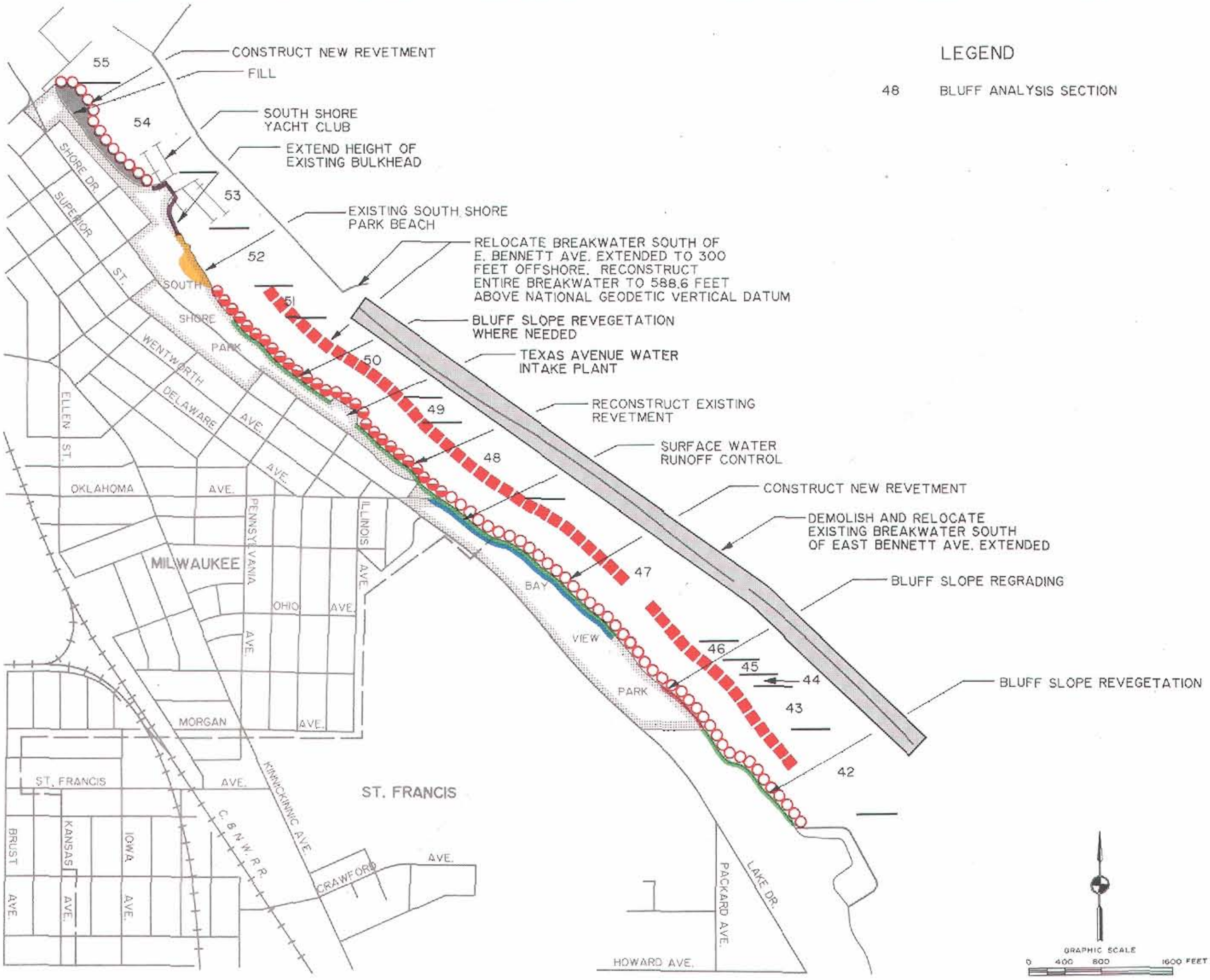
Source: Milwaukee County and SEWRPC.

The stone from the demolished breakwater would be used for all reconstruction and new construction; no new stone would be required. This alternative would substantially change the appearance and use of the shoreline and nearshore area. About 72 percent of the protected surface water area would be eliminated. Alternative No. 4 would entail a capital cost of approximately $7.0 million, an annual maintenance cost of about $269,000, and an equivalent annual cost of about $711,000. Compared to Alternative No. 3, Alternative No. 4 would have a much smaller maintenance cost because of the greatly reduced offshore structures.

Alternative No. 5—Demolish Breakwater South of E. Oklahoma Avenue Extended, and Reconstruct Breakwater North of E. Oklahoma Avenue Extended to 588.6 Feet NGVD: Alternative No. 5, as shown on Map 53, would retain the breakwater north of E. Oklahoma Avenue extended, where residential property is located at the top of the bluff. The breakwater would be demolished south of E. Oklahoma Avenue extended, where more open land lies at the bluff top. This alternative is a compromise between Alternative No. 1 which would involve reconstruction of the entire breakwater, and Alternative No. 4 which would involve demolition of most of the breakwater. Implementation of this alternative would result in only a modest change in the appearance and use of the shoreline and about a 40 percent reduction in the protected surface water area. Alternative No. 5 would entail a capital cost of approximately $5.0 million, an annual maintenance cost of about $373,000, and an equivalent annual cost of about

Map 50

SOUTH SHORE BREAKWATER ALTERNATIVE NO. 2—RELOCATE BREAKWATER SOUTH OF E. BENNETT AVENUE EXTENDED, AND RECONSTRUCT BREAKWATER TO 588.6 FEET NGVD



Source: SEWRPC.

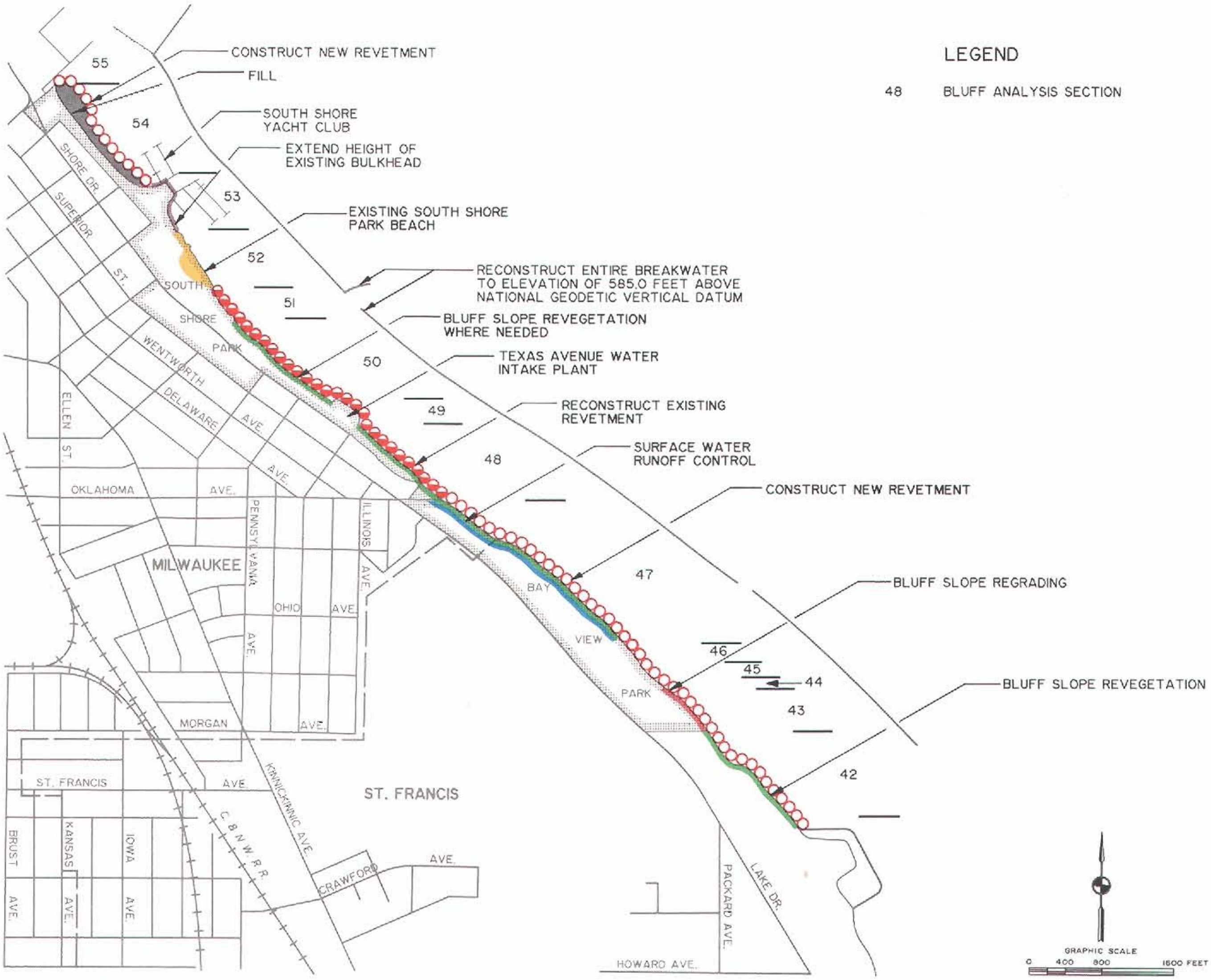
$693,000. This alternative thus represents the lowest total cost alternative considered. Compared to Alternative No. 4, Alternative No. 5 has a lower capital cost because less of the breakwater would be demolished, but a higher maintenance cost because of the greater cost of maintaining the longer breakwater.

Alternative No. 6—Replace Breakwater with Islands, Peninsulas, and Near-shore Breakwaters: Alternative No. 6, as shown on Map 54, was incorporated into the offshore alternative plan for Milwaukee County. Landfill would be placed in the lake to create islands and a peninsula. In Bluff Analysis Section 47, the South Shore breakwater would be demolished, with the stone being used to construct smaller breakwaters closer to shore which would help contain a new sand beach. Only minimum onshore control measures would be required under this alternative.

This alternative would provide a maximum level of offshore protection since the islands and peninsulas would essentially eliminate wave overtopping. About 115 acres of public lakefront

Map 51

SOUTH SHORE BREAKWATER ALTERNATIVE NO. 3—RECONSTRUCT BREAKWATER TO 585.0 FEET NGVD

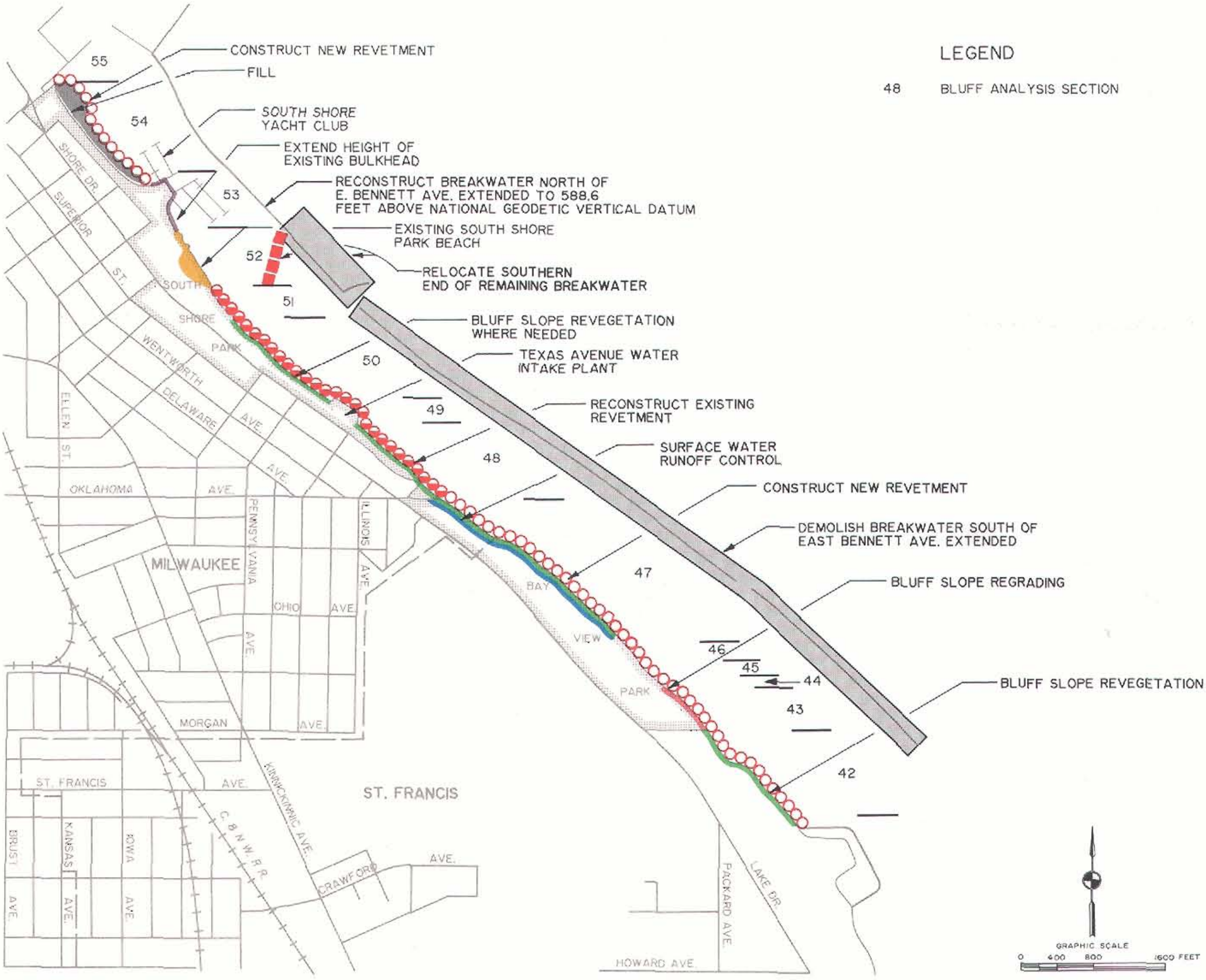


Source: SEWRPC.

land, about four miles of shoreline, and a sand beach would be created. This alternative would substantially change the appearance and use of the shoreline and near-shore area. A large amount of fill material—about 2.5 million cubic yards—would be required for the construction of the islands and peninsula. It is possible that the needed amount of fill material could be made available—over an extended time period of about 20 years—to construct the offshore islands and peninsulas to replace the South Shore breakwater on an incremental basis, especially if other major projects requiring large amounts of fill were not undertaken at the same time. Alternative No. 6 would entail a capital cost of approximately $11.8 million, an annual maintenance cost of about $290,000, and an equivalent annual cost of about $1,037,000. In estimating the cost of this alternative, it was assumed that the fill material for construction of the islands and peninsulas—either spoil from the deep tunnel project or construction and demolition debris—could be obtained at little or no cost. If all of the fill material needed to be purchased at a cost of

Map 52

SOUTH SHORE BREAKWATER ALTERNATIVE NO. 4—DEMOLISH BREAKWATER SOUTH OF E. BENNETT AVENUE EXTENDED, AND RECONSTRUCT BREAKWATER NORTH OF E. BENNETT AVENUE EXTENDED TO 588.6 FEET NGVD



Source: SEWRPC.

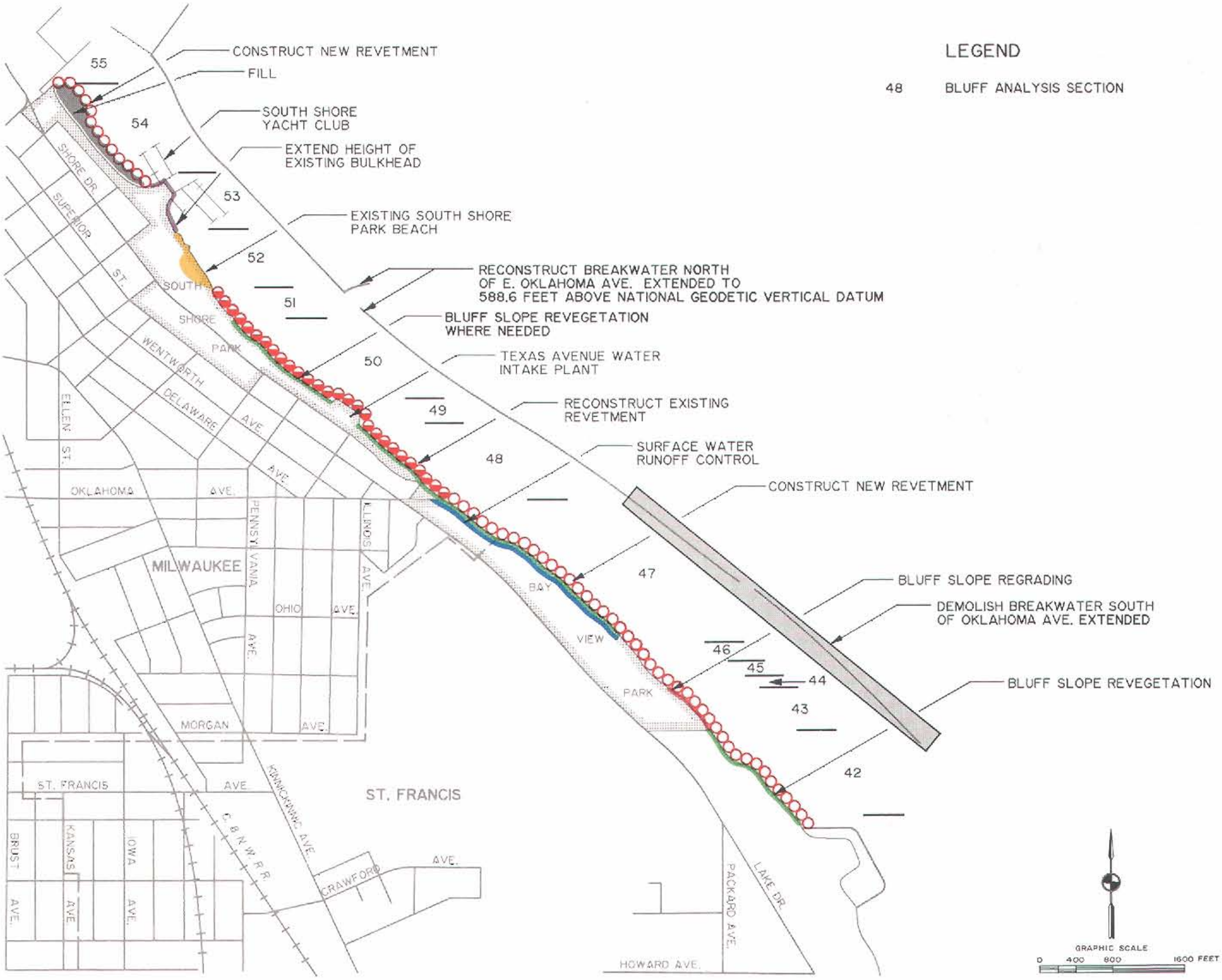
up to $10 per cubic yard, the capital cost could triple. It was also assumed that the stone from the existing breakwater would be used to protect the lake side of the new islands and peninsulas.

RECOMMENDED SHORELINE EROSION MANAGEMENT PLAN

Based upon careful consideration of the alternatives, the Intergovernmental Coordinating and Technical Advisory Committee selected a recommended shoreline erosion management plan for Milwaukee County. The recommended plan consists of a bluff stabilization element and a shoreline protection element. The recommended plan incorporates those shore protection measures which, within each section of shoreline, would most effectively abate the bluff recession and shoreline erosion problems; would recognize the preferences and priorities of the local units of government and lakefront private property owners; would be economically feasible and implementable; and would provide—where

Map 53

SOUTH SHORE BREAKWATER ALTERNATIVE NO. 5—DEMOLISH BREAKWATER SOUTH OF E. OKLAHOMA AVENUE EXTENDED, AND RECONSTRUCT BREAKWATER NORTH OF E. OKLAHOMA AVENUE EXTENDED TO 588.6 FEET NGVD



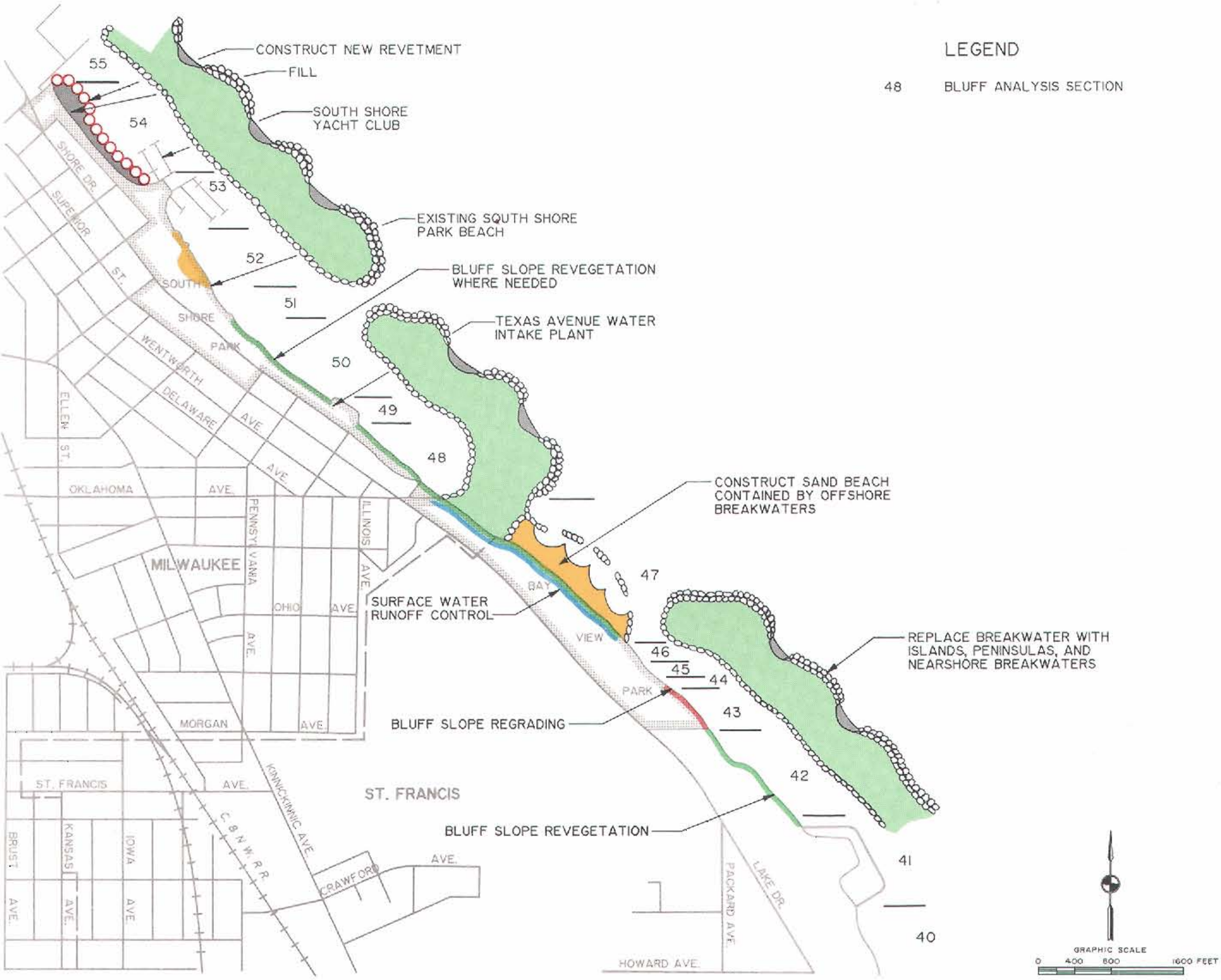
Source: SEWRPC.

practicable—a usable shoreline to be enjoyed by the general public as well as by lakefront property owners. The recommended plan is illustrated on Map 55. The recommended measures are listed for each bluff analysis section in Table 67.

The scope of the plan extends beyond the selection of individual shore protection measures. Coastal processes and the anticipated impacts of the various types of shore protection measures were thoroughly investigated. The plan recognizes that environmental trade-offs must at times be made—particularly when shore protection is not undertaken until a severe erosion problem has developed and real property is threatened. The plan attempts to minimize these environmental trade-offs, as well as potential adverse impacts on adjacent shoreline areas, by trying to foresee problems and carefully selecting those protection measures which are needed and most appropriate for different coastal environments within the study area. The plan seeks to ensure that the recommended measures will not have long-term harmful effects on the overall coastal environment—including the offshore bathymetry, sediments, and ecosystem. Cost was a major consideration in the

Map 54

SOUTH SHORE BREAKWATER ALTERNATIVE NO. 6—REPLACE BREAKWATER WITH ISLANDS, PENINSULAS, AND NEAR-SHORE BREAKWATERS



Source: SEWRPC.

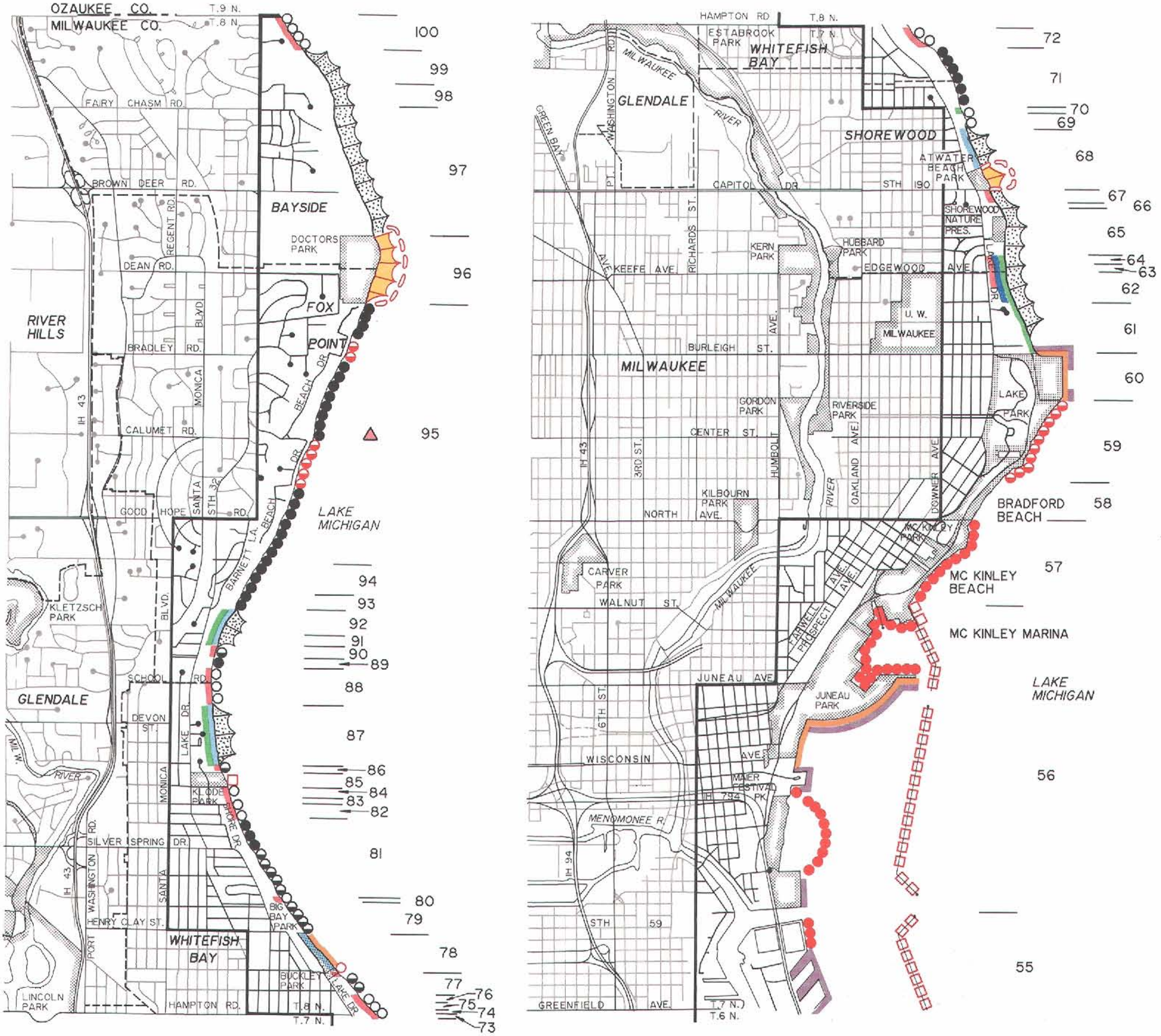
selection of the individual plan components. Other factors, such as the level of protection provided, recreational benefits, local preferences, existing adjacent shore protection measures, and environmental impacts, were also considered in the selection of the recommended plan.

The recommended plan provides protection for the entire Milwaukee County shoreline that is currently actively eroding, or where erosion may be expected to occur within the foreseeable future. It is recognized that Milwaukee County and local units of government concerned may place a very low priority on protecting certain reaches of shoreline. In fact, the County and local units of government may conclude that some reaches should be left to erode, in that the investment required to install and maintain the protective structures would not be worth the benefits received. Such judgments should be made locally and be based upon factors such as budget constraints, recreational use demands, the value of the facilities or land to be protected, and property owner preferences.

(Continued on page 398)

Map 55

RECOMMENDED SHORELINE EROSION MANAGEMENT PLAN FOR MILWAUKEE COUNTY



LEGEND

91 BLUFF ANALYSIS SECTION

BLUFF STABILIZATION PLAN ELEMENT

- BLUFF SLOPE REGRADING WITH REVEGETATION
- SURFACE WATER RUNOFF CONTROL
- GROUNDWATER DRAINAGE
- BLUFF SLOPE REVEGETATION

SHORELINE PROTECTION PLAN ELEMENT

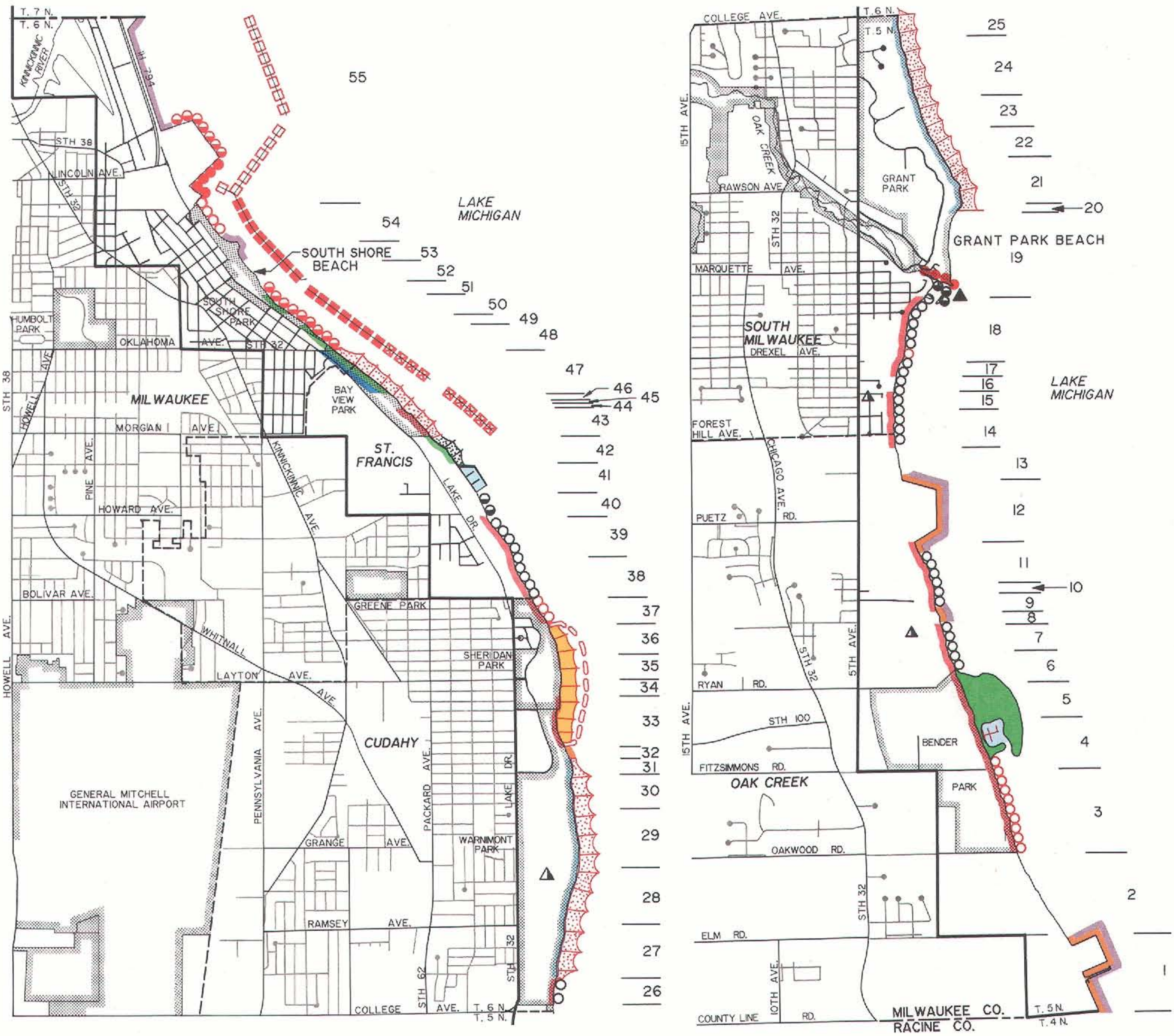
CONSTRUCT NEW STRUCTURES

- PRIVATE REVETMENT
- PUBLIC REVETMENT
- PRIVATE GROIN SYSTEM WITH COARSE SAND OR GRAVEL BEACH
- PUBLIC GROIN SYSTEM WITH COARSE SAND OR GRAVEL BEACH
- PUBLIC OFFSHORE BREAKWATERS WITH SAND BEACH
- PRIVATE MARINA
- PUBLIC MARINA

RECONSTRUCT EXISTING STRUCTURES

- PRIVATE REVETMENT
- PUBLIC REVETMENT

Map 55 (continued)



PUBLIC BULKHEAD	MAINTAIN EXISTING STRUCTURES
RIP-RAP BERM	PRIVATE
HEIGHT EXTENSION	PUBLIC (ONSHORE)
BERM AND EXTENSION	PUBLIC BREAKWATER AT EXISTING ELEVATION
PUBLIC BREAKWATER TO 588.6 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM	DEMOLISH EXISTING STRUCTURES
	PUBLIC BREAKWATER

AUXILIARY PLAN RECOMMENDATIONS

- ABATEMENT OF OAK CREEK SHOALING PROBLEM
- ASSESSMENT OF POTENTIAL TOXIC SUBSTANCES IN INDUSTRIAL WASTE SITES ON OR NEAR COASTAL BLUFFS
- MONITOR COASTAL ENVIRONMENT OFFSHORE OF FOX POINT TERRACE

GRAPHIC SCALE 0 1/4 1/2 MILE; 0 2000 4000 FEET

Source: SEWRPC.

Table 67

ESTIMATED COST OF THE RECOMMENDED SHORELINE EROSION MANAGEMENT PLAN FOR MILWAUKEE COUNTY

BLUFF SLOPE STABILIZATION							
Civil Division	Bluff Analysis Section	Shoreline Length (feet)	Plan Component	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual Cost
City of Oak Creek	1	4,470	- -	$ - -	$ - -	$ - -	$ - -
	2	2,820	- -	- -	- -	- -	- -
	3	2,930	Bluff slope regrading	440,000	44,000[a]	557,000	35,000
	4	1,980	Bluff slope regrading	297,000	30,000[a]	376,000	22,000
	5	1,070	Bluff slope regrading	161,000	16,000[a]	204,000	13,000
	6	1,170	Bluff slope regrading	176,000	18,000[a]	223,000	14,000
	7	1,000	Bluff slope regrading; toxic substance analysis	160,000	15,000[a]	200,000	13,000
	8	540	- -	- -	- -	- -	- -
	9	570	Bluff slope regrading	84,000	9,000[a]	109,000	7,000
	10	400	Bluff slope regrading	60,000	6,000[a]	76,000	5,000
	11	1,290	Bluff slope regrading	194,000	19,000[a]	245,000	16,000
	12	3,160	- -	- -	- -	- -	- -
	13	1,320	- -	- -	- -	- -	- -
City of South Milwaukee	14	1,310	Bluff slope regrading	197,000	20,000[a]	249,000	16,000
	15	790	Bluff slope regrading; toxic substance analysis	129,000	12,000[a]	160,000	10,000
	16	470	- -	- -	- -	- -	- -
	17	440	Bluff slope regrading	66,000	7,000[a]	84,000	5,000
	18	220	Bluff slope regrading	34,000	3,000[a]	43,000	3,000
		1,660	Bluff slope regrading	248,000	25,000[a]	314,000	20,000
	19	700	- -	- -	- -	- -	- -
		2,480	- -	- -	- -	- -	- -
	20	1,280	Groundwater drainage	64,000	13,000	269,000	17,000
	21	1,060	Groundwater drainage	53,000	11,000	226,000	14,000
	22	950	Groundwater drainage	48,000	10,000	206,000	13,000
	23	1,200	Groundwater drainage	60,000	12,000	249,000	16,000
	24	1,910	Groundwater drainage	96,000	19,000	295,000	25,000
	25	880	Groundwater drainage	44,000	9,000	186,000	12,000
City of Cudahy	26	660	Bluff slope regrading	99,000	10,000[a]	126,000	8,000
	27	1,850	Groundwater drainage	92,000	18,000	376,000	24,000
	28	2,050	Groundwater drainage; toxic substance analysis	112,000	20,000	427,000	27,000
	29	770	Groundwater drainage	38,000	8,000	164,000	10,000
	30	1,760	Groundwater drainage	88,000	18,000	372,000	24,000
	31	600	Groundwater drainage	30,000	6,000	125,000	8,000
	32	340	- -	- -	- -	- -	- -
	33	2,060	Bluff slope regrading	309,000	36,000	392,000	25,000
	34	1,780	- -	- -	- -	- -	- -
	35	650	- -	- -	- -	- -	- -
	36	710	- -	- -	- -	- -	- -
	37	1,010	Bluff slope regrading	152,000	15,000[a]	192,000	12,000

Table 67 (continued)

BLUFF SLOPE STABILIZATION							
Civil Division	Bluff Analysis Section	Shoreline Length (feet)	Plan Component	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual Cost
City of St. Francis	38	1,290	Bluff slope regrading	$ 194,000	$ 19,000[a]	$ 245,000	$ 16,000
	39	1,480	Bluff slope regrading	222,000	22,000[a]	281,000	18,000
	40	820	- -	- -	- -	- -	- -
	41	1,650	- -	- -	- -	- -	- -
	42	940	Revegetation	14,000	3,000[a]	22,000	1,000
	43	1,370	Bluff slope regrading	206,000	21,000[a]	261,000	17,000
	44	140	- -	- -	- -	- -	- -
	45	80	- -	- -	- -	- -	- -
	46	360	- -	- -	- -	- -	- -
	47	2,470	Surface control; revegetation	66,000	12,000[b]	160,000	10,000
City of Milwaukee	48	1,420	Revegetation	21,000	4,000[a]	33,000	2,000
	49	340	- -	- -	- -	- -	- -
	50	1,130	Revegetation	17,000	3,000[a]	26,000	2,000
	51	570	- -	- -	- -	- -	- -
	52	450	- -	- -	- -	- -	- -
	53	1,320	- -	- -	- -	- -	- -
	54	1,360	- -	- -	- -	- -	- -
	55	9,600	- -	- -	- -	- -	- -
		4,600	- -	- -	- -	- -	- -
		3,400	- -	- -	- -	- -	- -
		5,650	- -	- -	- -	- -	- -
		1,100	- -	- -	- -	- -	- -
	56	9,500	- -	- -	- -	- -	- -
		1,900	- -	- -	- -	- -	- -
		2,900	- -	- -	- -	- -	- -
		1,400	- -	- -	- -	- -	- -
		1,700	- -	- -	- -	- -	- -
		3,900	- -	- -	- -	- -	- -
		4,260	- -	- -	- -	- -	- -
	57	3,210	- -	- -	- -	- -	- -
	58	1,900	- -	- -	- -	- -	- -
	59	3,540	- -	- -	- -	- -	- -
	60	2,210	- -	- -	- -	- -	- -
	61	880	- -	- -	- -	- -	- -
		1,090	Revegetation	20,000	4,000[a]	30,000	2,000
	62	950	Surface water control; revegetation	14,000	3,000[a]	22,000	1,000
	63	300	Surface water control; revegetation; bluff slope regrading	30,000	3,000[a]	38,000	2,000
Village of Shorewood	64	290	Surface water control; revegetation; bluff slope regrading	23,000	2,000[a]	31,000	2,000
	65	1,710	- -	- -	- -	- -	- -
	66	170	- -	- -	- -	- -	- -
	67	380	Bluff slope regrading	57,000	6,000	72,000	5,000
	68	790	- -	- -	- -	- -	- -
		1,380	Groundwater drainage	69,000	14,000	287,000	18,000
	69	520	- -	- -	- -	- -	- -
	70	240	Revegetation	4,000	1,000[a]	6,000	1,000

Table 67 (continued)

BLUFF SLOPE STABILIZATION							
Civil Division	Bluff Analysis Section	Shoreline Length (feet)	Plan Component	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual Cost
Village of Whitefish Bay	71	2,370	--	$ --	$ --	$ --	$ --
	72	850	Bluff slope regrading	128,000	13,000[a]	162,000	10,000
	73	190	Bluff slope regrading	29,000	3,000[a]	36,000	2,000
	74	160	Bluff slope regrading	24,000	2,000[a]	30,000	2,000
	75	310	Bluff slope regrading	47,000	5,000[a]	59,000	4,000
	76	360	Bluff slope regrading	54,000	5,000[a]	68,000	4,000
	77	810	--	--	--	--	--
	78	600	Bluff slope regrading	90,000	9,000[a]	114,000	7,000
		1,060	Groundwater drainage	53,000	11,000	220,000	14,000
	79	1,480	--	--	--	--	--
	80	130	Bluff slope regrading	13,000	1,000[a]	16,000	1,000
	81	1,700	--	--	--	--	--
		1,270	--	--	--	--	--
	82	490	Bluff slope regrading	74,000	7,000[a]	93,000	7,000
	83	140	Bluff slope regrading	21,000	2,000[a]	26,000	2,000
	84	430	Bluff slope regrading	65,000	7,000[a]	82,000	5,000
	85	480	--	--	--	--	--
	86	170	Bluff slope regrading	26,000	3,000[a]	32,000	2,000
	87	1,950	Revegetation, groundwater drainage	117,000	23,000[b]	435,000	28,000
Village of Fox Point	88	1,150	Bluff slope regrading	173,000	17,000[a]	219,000	14,000
	89	320	--	--	--	--	--
	90	470	Bluff slope regrading	71,000	7,000[a]	90,000	6,000
	91	510	Revegetation, groundwater drainage	33,000	6,000[b]	116,000	7,000
	92	770	Revegetation, groundwater drainage	62,000	12,000[b]	195,000	12,000
	93	530	--	--	--	--	--
	94	1,460	--	--	--	--	--
	95A	2,390	--	--	--	--	--
	95B	1,600	--	--	--	--	--
	95C	3,000	--	--	--	--	--
	95D	720	--	--	--	--	--
	95E	1,360	--	--	--	--	--
	95	9,070	--	--	--	--	--
Village of Bayside	96	1,890	--	--	--	--	--
	97	4,660	--	--	--	--	--
	98	860	--	--	--	--	--
	99	1,280	--	--	--	--	--
	100	1,320	Bluff slope regrading	198,000	20,000[a]	251,000	16,000
Bluff Slope Stabilization Total	--	159,110	--	$ 5,766,000	$ 699,000[c]	$10,203,000	$ 654,000

Table 67 (continued)

BLUFF TOE PROTECTION							
Civil Division	Bluff Analysis Section	Shoreline Length (feet)	Plan Component	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual Cost
City of Oak Creek	1	4,470	Reconstruct existing public bulkhead	$ 3,576,000	$ 67,000	$ 4,633,000	$ 294,000
	2	2,820	- -	- -	- -	- -	- -
	3	2,930	Construct new public revetment	1,172,000	44,000	1,866,000	118,000
	4	1,980	Construct new public marina and landfill	7,920,000	59,000	9,442,000	599,000
	5	1,070	Construct new public marina and landfill	4,280,000	32,000	4,784,000	304,000
	6	1,170	Construct new private revetment	468,000	18,000	745,000	47,000
	7	1,000	Construct new private revetment	400,000	15,000	636,000	40,000
	8	540	Reconstruct existing public bulkhead	81,000	8,000	209,000	13,000
	9	570	Construct new private revetment	228,000	9,000	364,000	23,000
	10	400	Construct new private revetment	160,000	6,000	255,000	16,000
	11	1,290	Construct new private revetment	516,000	19,000	822,000	52,000
	12	3,160	Reconstruct existing public bulkhead	2,370,000	48,000	3,117,000	198,000
	13	1,320	- -	- -	- -	- -	- -
City of South Milwaukee	14	1,310	Construct new private revetment	393,000	13,000	600,000	38,000
	15	790	Construct new private revetment	237,000	8,000	362,000	23,000
	16	470	Construct new private revetment	141,000	5,000	215,000	14,000
	17	440	Construct new private revetment	132,000	4,000	201,000	13,000
	18	220	Construct new public revetment	110,000	3,000	162,000	10,000
		1,660	Construct new private revetment	664,000	25,000	1,057,000	67,000
	19	700	Reconstruct existing private revetment	140,000	7,000	250,000	16,000
		2,480	Maintain existing public structures	0	12,000	189,000	12,000
		- -	Abate Oak Creek shoaling problem	145,000	5,000	224,000	14,000
	20	1,280	Construct public groin system with coarse sand or gravel beach	640,000	26,000	1,044,000	66,000
	21	1,060	Construct public groin system with coarse sand or gravel beach	530,000	21,000	864,000	55,000
	22	950	Construct public groin system with coarse sand or gravel beach	475,000	19,000	775,000	49,000
	23	1,200	Construct public groin system with coarse sand or gravel beach	600,000	24,000	978,000	62,000
	24	1,910	Construct public groin system with coarse sand or gravel beach	955,000	38,000	1,557,000	99,000
	25	880	Construct public groin system with coarse sand or gravel beach	440,000	18,000	717,000	46,000

Table 67 (continued)

BLUFF TOE PROTECTION							
Civil Division	Bluff Analysis Section	Shoreline Length (feet)	Plan Component	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual Cost
City of Cudahy	26	660	Construct new private revetment	$ 330,000	$ 10,000	$ 486,000	$ 31,000
	27	1,850	Construct public groin system with coarse sand or gravel beach	925,000	37,000	1,508,000	96,000
	28	2,050	Construct public groin system with coarse sand or gravel beach	1,025,000	41,000	1,671,000	106,000
	29	770	Construct public groin system with coarse sand or gravel beach	385,000	15,000	628,000	40,000
	30	1,760	Construct public groin system with coarse sand or gravel beach	880,000	35,000	1,433,000	91,000
	31	600	Construct public groin system with coarse sand or gravel beach	300,000	12,000	489,000	31,000
	32	340	Reconstruct existing public bulkhead	51,000	5,000	131,000	8,000
	33	2,060	Construct new public offshore breakwater with sand beach	3,090,000	62,000	4,067,000	258,000
	34	1,780	Construct new public offshore breakwater with sand beach	2,670,000	53,000	3,505,000	222,000
	35	650	Construct new public offshore breakwater with sand beach	975,000	20,000	1,290,000	82,000
	36	710	Construct new public offshore breakwater with sand beach	1,065,000	21,000	1,396,000	89,000
	37	1,010	Construct new public revetment	404,000	15,000	644,000	41,000
City of St. Francis	38	1,290	Construct new private revetment	387,000	13,000	590,000	37,000
	39	1,480	Construct new private revetment	444,000	15,000	677,000	43,000
	40	820	Reconstruct existing private revetment	328,000	12,000	522,000	33,000
	41	1,650	Construct new private marina	4,125,000	50,000	4,905,000	311,000
	42	940	Construct private groin system with coarse sand or gravel beach	282,000	19,000	581,000	37,000
	43	1,370	Demolish South Shore breakwater; construct public groin system with coarse sand or gravel beach	893,000	27,000	1,319,000	84,000
	44	140	Demolish South Shore breakwater; construct public groin system with coarse sand or gravel beach	91,000	3,000	138,000	9,000
	45	80	Demolish South Shore breakwater; construct public groin system with coarse sand or gravel beach	52,000	2,000	84,000	5,000
	46	360	Demolish South Shore breakwater; construct public groin system with coarse sand or gravel beach	235,000	7,000	345,000	22,000

Table 67 (continued)

BLUFF TOE PROTECTION							
Civil Division	Bluff Analysis Section	Shoreline Length (feet)	Plan Component	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual Cost
City of St. Francis (continued)	47	2,470	Demolish South Shore breakwater; construct public groin system with coarse sand or gravel beach	$ 1,610,000	$ 49,000	$ 2,382,000	$ 115,000
City of Milwaukee	48	1,420	Reconstruct South Shore breakwater to 588.6 feet NGVD; reconstruct existing public revetment	993,000	64,000	2,002,000	127,000
	49	340	Reconstruct South Shore breakwater to 588.6 feet NGVD; reconstruct existing public revetment	234,000	15,000	470,000	30,000
	50	1,130	Reconstruct South Shore breakwater to 588.6 feet NGVD; reconstruct existing public revetment	520,000	51,000	1,324,000	84,000
	51	570	Reconstruct South Shore breakwater to 588.6 feet NGVD; reconstruct existing public revetment	259,000	26,000	669,000	42,000
	52	450	Reconstruct South Shore breakwater to 588.6 feet NGVD	166,000	16,000	418,000	27,000
	53	1,320	Reconstruct South Shore breakwater to 588.6 feet NGVD; reconstruct existing public marine bulkheads	437,000	59,000	1,367,000	87,000
	54	1,360	Reconstruct South Shore breakwater to 588.6 feet NGVD; construct new public revetment	523,000	61,000	1,484,000	94,000
	55	9,600	Maintain existing public outer harbor breakwater	0	432,000	6,809,000	432,000
		4,600	U. S. Army Corps of Engineers dredge spoils confined disposal facility —reconstruct existing revetment	1,840,000	46,000	2,565,000	163,000
		3,400	South Lincoln Memorial Drive—reconstruct existing public bulkhead	1,020,000	34,000	1,556,000	99,000
		5,650	Port of Milwaukee ships—reconstruct existing public bulkhead	2,825,000	56,000	3,708,000	235,000
		1,100	MMSD Jones Island wastewater treatment plant—maintain existing public bulkhead	0	11,000	173,000	11,000
	56	9,500	Maintain existing public outer harbor breakwater	0	428,000	6,738,000	428,000
		1,900	Marcus Amphitheatre—reconstruct existing public bulkhead	380,000	19,000	679,000	43,000

Table 67 (continued)

BLUFF TOE PROTECTION							
Civil Division	Bluff Analysis Section	Shoreline Length (feet)	Plan Component	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual Cost
City of Milwaukee (continued)		2,900	Henry W. Maier festival grounds—maintain existing public island and revetment	$ 0	$ 29,000	$ 457,000	$ 29,000
		1,400	Milwaukee Harbor Commission municipal pier—reconstruct existing bulkhead	280,000	14,000	501,000	32,000
		1,700	Milwaukee County War Memorial Center—reconstruct existing bulkhead	340,000	17,000	608,000	39,000
		3,900	Milwaukee County Juneau Park landfill—reconstruct existing bulkhead	2,340,000	39,000	2,955,000	187,000
		4,260	McKinley Marina—maintain existing public structures	0	43,000	678,000	43,000
	57	3,210	Maintain existing public structure	0	96,000	1,518,000	96,000
	58	1,900	- -	- -	- -	- -	- -
	59	3,540	Reconstruct existing public revetment	1,062,000	53,000	1,899,000	121,000
	60	2,210	Reconstruct existing public bulkhead	1,105,000	33,000	1,628,000	103,000
	61	880	- -	- -	- -	- -	- -
		1,090	Construct private groin system with coarse sand or gravel beach	436,000	22,000	780,000	50,000
	62	950	Construct private groin system with coarse sand or gravel beach	380,000	19,000	680,000	44,000
	63	300	Construct private groin system with coarse sand or gravel beach	120,000	6,000	215,000	14,000
Village of Shorewood	64	290	Construct private groin system with coarse sand or gravel beach	116,000	6,000	207,000	13,000
	65	1,710	Construct public/private system with coarse sand or gravel beach	855,000	34,000	1,391,000	88,000
	66	170	Construct private groin system with coarse sand or gravel beach	68,000	3,000	122,000	8,000
	67	380	Construct private groin system with coarse sand or gravel beach	152,000	8,000	272,000	17,000
	68	790	Construct new offshore breakwater with sand beach	1,185,000	24,000	1,563,000	99,000
		1,380	Construct private groin system with coarse sand or gravel beach	552,000	28,000	987,000	63,000
	69	520	Construct new private revetment	156,000	5,000	238,000	15,000
	70	240	Construct new private revetment	73,000	2,000	110,000	7,000

Table 67 (continued)

BLUFF TOE PROTECTION							
Civil Division	Bluff Analysis Section	Shoreline Length (feet)	Plan Component	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual Cost
Village of Whitefish Bay	71	2,370	Maintain existing private structures	$ 0	$ 24,000	$ 374,000	$ 24,000
	72	850	Construct new private revetment	255,000	9,000	389,000	25,000
	73	190	Construct new private revetment	57,000	2,000	87,000	6,000
	74	160	Construct new private revetment	48,000	2,000	73,000	5,000
	75	310	Construct new private revetment	93,000	3,000	142,000	9,000
	76	360	Construct new private revetment	108,000	4,000	165,000	10,000
	77	810	Reconstruct existing private revetment	162,000	8,000	290,000	18,000
	78	600	Construct new public revetment	240,000	9,000	382,000	24,000
		1,060	Reconstruct existing public bulkhead	212,000	16,000	463,000	29,000
	79	1,480	Reconstruct existing private revetment	296,000	15,000	529,000	34,000
	80	130	Construct new private revetment	39,000	1,000	60,000	4,000
	81	1,700	Reconstruct existing private revetment	340,000	17,000	608,000	39,000
		1,270	Maintain existing private structures	0	13,000	200,000	13,000
	82	490	Construct new private revetment	147,000	5,000	224,000	14,000
	83	140	Construct new private revetment	42,000	1,000	64,000	4,000
	84	430	Construct new private revetment	129,000	4,000	197,000	13,000
	85	480	Maintain existing public breakwater	0	14,000	227,000	14,000
	86	170	Maintain existing private structures	0	2,000	27,000	2,000
	87	1,950	Construct private groin system with coarse sand or gravel beach	780,000	39,000	1,395,000	89,000
Village of Fox Point	88	1,150	Construct new private revetment	345,000	12,000	526,000	33,000
	89	320	Maintain existing private structures	0	3,000	50,000	3,000
	90	470	Reconstruct existing private revetment	94,000	5,000	168,000	11,000
	91	510	Construct private groin system with coarse sand or gravel beach	204,000	10,000	365,000	23,000

Table 67 (continued)

BLUFF TOE PROTECTION							
Civil Division	Bluff Analysis Section	Shoreline Length (feet)	Plan Component	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual Cost
Village of Fox Point (continued)	92	770	Construct private groin system with coarse sand or gravel beach	$ 308,000	$ 15,000	$ 551,000	$ 35,000
	93	530	Maintain existing private structures	0	5,000	84,000	5,000
	94	1,460	Maintain existing private structures	0	15,000	230,000	15,000
	95A	2,390	Maintain existing private structures	0	24,000	377,000	24,000
	95B	1,600	Reconstruct existing public revetment	640,000	24,000	1,018,000	65,000
	95C	3,000	Maintain existing private structures	0	30,000	473,000	30,000
	95D	720	Reconstruct existing public revetment	288,000	11,000	458,000	29,000
	95E	1,360	Maintain existing private structures	0	14,000	214,000	14,000
	95	9,070	Monitor coastal environment offshore of Fox Point Terrace	0	5,000	79,000	5,000
Village of Bayside	96	1,890	Construct new public offshore breakwater with sand beach	2,835,000	57,000	3,733,000	237,000
	97	4,660	Construct private groin system with coarse sand or gravel beach	1,864,000	93,000	3,333,000	211,000
	98	860	Construct private groin system with coarse sand or gravel beach	344,000	17,000	615,000	39,000
	99	1,280	Construct private groin system with coarse sand or gravel beach	512,000	26,000	916,000	58,000
	100	1,320	Construct new private revetment	396,000	13,000	604,000	38,000
Bluff Toe Protection Total	- -	159,110	- -	$76,515,000	$3,442,000	$131,320,000	$8,336,000

Table 67 (continued)

RECOMMENDED TOTAL PLAN						
Civil Division	Bluff Analysis Section	Shoreline Length (feet)	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual Cost
City of Oak Creek	1	4,470	$ 3,576,000	$ 67,000	$ 4,633,000	$ 294,000
	2	2,820	- -	- -	- -	- -
	3	2,930	1,612,000	88,000	2,423,000	153,000
	4	1,980	8,217,000	89,000	9,818,000	621,000
	5	1,070	4,441,000	48,000	4,988,000	317,000
	6	1,170	644,000	36,000	968,000	61,000
	7	1,000	560,000	30,000	836,000	53,000
	8	540	81,000	8,000	209,000	13,000
	9	570	312,000	18,000	473,000	30,000
	10	400	220,000	12,000	331,000	21,000
	11	1,290	710,000	38,000	1,067,000	68,000
	12	3,160	2,370,000	48,000	3,117,000	198,000
	13	1,320	- -	- -	- -	- -
City of South Milwaukee	14	1,310	590,000	33,000	849,000	54,000
	15	790	366,000	20,000	522,000	33,000
	16	470	141,000	5,000	215,000	14,000
	17	440	198,000	11,000	285,000	18,000
	18	220	144,000	6,000	205,000	13,000
		1,660	912,000	50,000	1,371,000	87,000
	19	700	140,000	7,000	250,000	16,000
		2,480	0	12,000	189,000	12,000
		- -	145,000	5,000	224,000	14,000
	20	1,280	704,000	39,000	1,313,000	83,000
	21	1,060	583,000	32,000	1,090,000	69,000
	22	950	523,000	29,000	981,000	62,000
	23	1,200	660,000	36,000	1,227,000	78,000
	24	1,910	1,051,000	57,000	1,952,000	124,000
	25	880	484,000	27,000	903,000	58,000
City of Cudahy	26	660	429,000	20,000	612,000	39,000
	27	1,850	1,017,000	55,000	1,884,000	120,000
	28	2,050	1,137,000	61,000	2,098,000	133,000
	29	770	423,000	23,000	792,000	50,000
	30	1,760	968,000	53,000	1,805,000	115,000
	31	600	330,000	18,000	614,000	39,000
	32	340	51,000	5,000	131,000	8,000
	33	2,060	3,399,000	98,000	4,459,000	283,000
	34	1,780	2,670,000	53,000	3,505,000	222,000
	35	650	975,000	20,000	1,290,000	82,000
	36	710	1,065,000	21,000	1,396,000	89,000
	37	1,010	556,000	30,000	836,000	53,000

Table 67 (continued)

RECOMMENDED TOTAL PLAN						
Civil Division	Bluff Analysis Section	Shoreline Length (feet)	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual Cost
City of St. Francis	38	1,290	$ 581,000	$ 32,000	$ 835,000	$ 53,000
	39	1,480	666,000	37,000	958,000	61,000
	40	820	328,000	12,000	522,000	33,000
	41	1,650	4,125,000	50,000	4,905,000	311,000
	42	940	296,000	22,000	603,000	38,000
	43	1,370	1,099,000	48,000	1,580,000	101,000
	44	140	91,000	3,000	138,000	9,000
	45	80	52,000	2,000	84,000	5,000
	46	360	235,000	7,000	345,000	22,000
	47	2,470	1,676,000	61,000	2,542,000	161,000
City of Milwaukee	48	1,420	1,014,000	68,000	2,035,000	129,000
	49	340	234,000	15,000	470,000	30,000
	50	1,130	537,000	54,000	1,350,000	86,000
	51	570	259,000	26,000	669,000	42,000
	52	450	166,000	16,000	418,000	27,000
	53	1,320	437,000	59,000	1,367,000	87,000
	54	1,360	523,000	61,000	1,484,000	94,000
	55	9,600	0	432,000	6,809,000	432,000
		4,600	1,840,000	46,000	2,565,000	163,000
		3,400	1,020,000	34,000	1,556,000	99,000
		5,650	2,825,000	56,000	3,708,000	235,000
		1,100	0	11,000	173,000	11,000
	56	9,500	0	428,000	6,738,000	428,000
		1,900	380,000	19,000	679,000	43,000
		2,900	0	29,000	457,000	29,000
		1,400	280,000	14,000	501,000	32,000
		1,700	340,000	17,000	608,000	39,000
		3,900	2,340,000	39,000	2,955,000	187,000
		4,260	0	43,000	678,000	43,000
	57	3,210	0	96,000	1,518,000	96,000
	58	1,900	- -	- -	- -	- -
	59	3,540	1,062,000	53,000	1,899,000	121,000
	60	2,210	1,105,000	33,000	1,628,000	103,000
	61	880	- -	- -	- -	- -
		1,090	456,000	26,000	810,000	52,000
	62	950	394,000	22,000	702,000	45,000
	63	300	150,000	9,000	253,000	16,000
Village of Shorewood	64	290	139,000	8,000	238,000	15,000
	65	1,710	855,000	34,000	1,391,000	88,000
	66	170	68,000	3,000	122,000	8,000
	67	380	209,000	14,000	344,000	22,000
	68	790	1,185,000	24,000	1,563,000	99,000
		1,380	621,000	42,000	1,274,000	81,000
	69	520	156,000	5,000	238,000	15,000
	70	240	77,000	3,000	116,000	8,000

Table 67 (continued)

RECOMMENDED TOTAL PLAN						
Civil Division	Bluff Analysis Section	Shoreline Length (feet)	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual Cost
Village of Whitefish Bay	71	2,370	$ 0	$ 24,000	$ 374,000	$ 24,000
	72	850	383,000	22,000	551,000	35,000
	73	190	86,000	5,000	123,000	8,000
	74	160	72,000	4,000	103,000	7,000
	75	310	140,000	8,000	201,000	13,000
	76	360	162,000	9,000	233,000	14,000
	77	810	162,000	8,000	290,000	18,000
	78	600	330,000	18,000	496,000	31,000
		1,060	265,000	27,000	683,000	43,000
	79	1,480	296,000	15,000	529,000	34,000
	80	130	52,000	2,000	76,000	5,000
	81	1,700	340,000	17,000	608,000	39,000
		1,270	0	13,000	200,000	13,000
	82	490	221,000	12,000	317,000	21,000
	83	140	63,000	3,000	90,000	6,000
	84	430	194,000	11,000	279,000	18,000
	85	480	0	14,000	227,000	14,000
	86	170	26,000	5,000	59,000	4,000
	87	1,950	897,000	62,000	1,830,000	117,000
Village of Fox Point	88	1,150	518,000	29,000	745,000	47,000
	89	320	0	3,000	50,000	3,000
	90	470	165,000	12,000	258,000	17,000
	91	510	237,000	16,000	481,000	30,000
	92	770	370,000	27,000	746,000	47,000
	93	530	0	5,000	84,000	5,000
	94	1,460	0	15,000	230,000	15,000
	95A	2,390	0	24,000	377,000	24,000
	95B	1,600	640,000	24,000	1,018,000	65,000
	95C	3,000	0	30,000	473,000	30,000
	95D	720	288,000	11,000	458,000	29,000
	95E	1,360	0	14,000	214,000	14,000
	95	9,070	0	5,000	79,000	5,000
Village of Bayside	96	1,890	2,835,000	57,000	3,733,000	237,000
	97	4,660	1,864,000	93,000	3,333,000	211,000
	98	860	344,000	17,000	615,000	39,000
	99	1,280	512,000	26,000	916,000	58,000
	100	1,320	594,000	33,000	855,000	54,000
Total	- -	159,110	$82,281,000	$4,141,000	$141,623,000	$8,990,000

[a]*Annual maintenance costs would apply only for the first three years following bluff slope regrading and revegetation.*

[b]*Of the total maintenance cost of $12,000 for Bluff Analysis Section 47, $7,400, or 60 percent, would be required only for the first three years following revegetation. Of the total maintenance cost of $23,000 for Bluff Analysis Section 87, $3,900, or 17 percent, would be required only for the first three years following revegetation. Of the total maintenance cost of $6,000 for Bluff Analysis Section 91, $1,000, or 6 percent, would be required only for the first three years following revegetation. Of the total maintenance cost of $12,000 for Bluff Analysis Section 92, $4,600, or 37 percent, would be required only for the first three years following revegetation.*

[c]*About $485,000, or 69 percent, of the annual maintenance cost for the bluff slope stabilization plan element, or 12 percent of the annual maintenance cost for the total plan, would be required for the first three years following bluff slope regrading or revegetation.*

Source: SEWRPC.

The bluff stabilization element of the recommended plan envisions that marginal or unstable bluff slopes will be stabilized by regrading the slopes, revegetating the slopes, and constructing groundwater and surface water drainage systems. However, the recommended bluff slope stabilization measures have a lower cost than the previously described bluff stability plan element because it is recommended that some bluff slopes be left to regrade back to a stable slope naturally. Those bluffs that could recede naturally to a stable slope—if adequate toe protection was provided—without endangering shoreline buildings and improvements cover about 19,790 feet, or 12 percent of the total county shoreline. They include the northern portion of the WEPCo Oak Creek Power Plant property, and portions of Milwaukee County's Grant, Warnimont, Sheridan, and Bay View Parks.

Bluff slopes would be regraded to a stable angle and revegetated along about 30,440 feet of shoreline, or 19 percent of the total county shoreline. Detailed studies to determine the feasibility of installing groundwater drainage systems along about 19,980 feet of shoreline, or 13 percent of the total county shoreline, would be conducted. Surface water runoff control would be provided along about 4,010 feet of shoreline, or 3 percent of the county shoreline. Revegetation of at least a portion of the bluff face, in addition to that required for slope regrading projects, would be provided along about 12,060 feet of shoreline, or 8 percent of the county shoreline. No bluff slope stabilization measures would be required along about 100,450 feet, or 63 percent, of the county shoreline. The recommended bluff stabilization measures would entail a capital cost of about $5.8 million and an annual maintenance cost of about $699,000, although 69 percent of the maintenance cost would be required for only a three-year period after construction.

The recommended plan includes the construction of two new marinas. A public marina would be constructed in Milwaukee County Bender Park in the City of Oak Creek. The Bender Park marina and adjacent landfill is intended to meet recreational needs in the far southern portion of the county shoreline. This marina would satisfy a recommendation of the regional park and open space plan,[25] which suggests that additional boat launching and mooring facilities and a harbor-of-refuge be provided between the mouth of Oak Creek in the City of South Milwaukee and the Racine harbor. A private marina would be constructed in the City of St. Francis on the former Wisconsin Electric Power Company Lakeside power plant property. This privately developed marina would enhance the water-based recreational activities that would result from the proposed development of the abandoned power plant property. The construction of the two new marinas would entail a capital cost of about $16.3 million, and an annual maintenance cost of about $141,000. Marina operating expenses not associated with shoreline protection are not included in the cost estimates because these costs would be covered by the user fees and other revenue produced by the operation.

Under the recommended plan, sand beaches contained by offshore breakwaters would be constructed at the Village of Shorewood's Atwater Park, and Milwaukee County's Doctors Park and Sheridan Park. All of these parks have eroded beaches which offer limited sunbathing and swimming opportunities during high-water periods. The recommended facilities would increase the size of the existing beaches, provide increased protection against erosion of the beaches, and provide calmer water areas to increase the enjoyment of swimming. A small sand beach would also be included within the proposed marina development at Milwaukee County's Bender Park. The 7,880 feet of public sand beach, covering about 5 percent of the total county shoreline, and providing about 37 acres of beach area, would substantially increase recreational opportunities for sunbathing and swimming within the County. The recommended sand beach systems would entail a capital cost of about $11.8 million and an annual maintenance cost of about $237,000.

The recommended plan envisions that nourished coarse sand and/or gravel beach systems contained by short groins will be located along about 36,850 feet, or 23 percent, of the county shoreline. The beach systems were recommended primarily for the following reasons:

1. Compared to revetments and bulkheads, less wave energy is reflected by beaches, thereby reducing associated damages to

[25]*SEWRPC Planning Report No. 27, A Regional Park and Open Space Plan for Southeastern Wisconsin: 2000, November 1977.*

adjacent shoreline reaches, to the littoral drift, and to the offshore coastal environment;

2. Properly designed beaches are flexible, energy absorptive, and durable, adjusting and remolding in response to storm and water level conditions;

3. In the long term, wave heights approaching beaches are more apt to remain stable, or possibly to even decrease over time, whereas scouring in front of revetments and bulkheads may be expected to increase the heights of the approaching waves;

4. The beach system would create a usable shoreline; and

5. While beach renourishment may be required following a highly erosive storm, massive structural failure is unlikely.

The coarse sand and gravel beach systems would entail a capital cost of about $15.5 million and an annual maintenance cost of about $719,000.

The recommended plan proposes that about 44,840 feet of quarry stone revetments, or 28 percent of the county shoreline, be constructed or reconstructed, usually to protect existing or proposed bluff slope fill projects. Beaches generally are not recommended for the fill projects because they would be subject to high wave energy, which would make them difficult and costly to maintain, and because the beaches would have to extend too far out into the lake, with potential damage to downdrift shoreline areas. The revetments would entail a capital cost of about $14.3 million and an annual maintenance cost of about $540,000.

Under the recommended plan, about 31,050 feet of concrete or steel sheet pile bulkheads, covering about 19 percent of the county shoreline, would be reconstructed in order to reduce wave overtopping damage. Bulkheads recommended to be reconstructed include those located at Milwaukee County Big Bay Park, the Village of Whitefish Bay Buckley Park, the City of Milwaukee Linnwood Avenue water treatment plant, the Milwaukee County Juneau Park landfill, the Milwaukee Harbor Commission municipal pier, the Marcus Amphitheater, the Port of Milwaukee slips, South Lincoln Memorial Drive, the Milwaukee County South Shore Park Marina, the City of Cudahy water intake plant, the Milwaukee Metropolitan Sewerage District South Shore wastewater treatment plant, the City of Oak Creek water intake plant, and the WEPCo Oak Creek power plant. Reconstruction could include placement of a riprap berm in front of the structure and/or increasing the height of the bulkhead with a sheet pile extension or a concrete cap. While new bulkheads are generally not recommended because they reflect wave energy, scour offshore sand deposits, and limit use of the shoreline, it was concluded that the existing bulkheads are necessary to accommodate uses such as boat docking, navigation, and industrial or utility facilities, and should therefore be retained. Reconstruction of the existing bulkheads would entail a capital cost of approximately $14.8 million and an annual maintenance cost of about $369,000.

Existing onshore protection structures along about 27,300 feet of shoreline, or 17 percent of the county total, would be maintained under the recommended plan. Maintenance of existing onshore structures would entail an annual cost of about $335,000. No major capital expenditures would be required.

For the 6,920 feet of shoreline, or 3 percent of the county total, where shoreline erosion is not a significant threat, no shore protection measures are recommended. Because of natural conditions or the impacts of adjacent structures, these areas have developed stable shorelines. Extensive construction or maintenance is not expected to be needed within these areas in the foreseeable future.

Recommendations for the Milwaukee Outer Harbor Breakwater

Alternatives considered for the Milwaukee outer harbor breakwater included continued maintenance of the structure; reconstruction to an elevation of about 595 feet NGVD, or about 8.7 feet higher than the existing breakwater, using either a poured concrete wall or a new rubblemound breakwater; and the construction of islands and peninsulas. The capital cost of reconstructing the breakwater would be high—ranging from $30 to $65 million. The breakwater provides adequate protection under normal water level and storm conditions. Under high-water-level and severe storm conditions, however, the current breakwater does not provide an adequate level of protection for safe harborage, or against shoreline erosion. However, it was concluded that damages to port facilities and shoreline structures are not extensive enough to justify further offshore protection measures in that it is less expensive to repair the damages as they occur, or to modify the onshore structures, than to construct a higher breakwater or islands. It is therefore recommended that the breakwater not be substantially modified at this time, but rather that facilities in the outer harbor be protected by the reconstruction or repair of onshore structures and facilities, including revetments, bulkheads, dock-wall improvements, and other floodproofing measures. Alternative No. 1, continued maintenance of the existing breakwater, is thus recommended to control shoreline erosion and provide safe navigation within the outer harbor. Maintaining the outer harbor breakwater would entail an annual cost of about $860,000.

The recommendations for the outer harbor should not be construed as implying that further development of the shoreline concerned is unwise or unwarranted. In fact, further development of portions of the shoreline may be desirable. For example, previous studies have indicated that the shoreline between the inner harbor entrance and the Milwaukee County War Memorial Center—often referred to as the North Harbor Tract—is underutilized and contains some inappropriate uses such as automobile parking and the long-term lease for a physically and conceptually isolated restaurant facility.[26] The location of this land area—between the Milwaukee River and Lake Michigan, two of the central City's most valuable natural assets, and adjacent to downtown commercial activity—provides a unique setting which offers an unequalled opportunity for recreational use.

The proper recreational development of this site could provide economic benefits, stimulate further water-based recreational development, and generally enhance the attractiveness of the central business district of Milwaukee. However, any development proposals should include shore protection measures which provide, to the extent practicable, a usable shoreline, and which are properly designed to provide adequate protection against the recommended design water level and storm wave conditions. Small-scale offshore projects, such as the island being constructed offshore of the Henry W. Maier festival grounds, are appropriate if justified on the basis of the costs and benefits entailed.

Recommendations for the South Shore Breakwater

Alternatives considered for the South Shore breakwater included reconstructing, relocating, and demolishing the breakwater, and constructing new islands and peninsulas. Factors considered in the evaluation of alternatives included the level of offshore and onshore protection provided, cost, the existing heavy recreational use of South Shore and Bay View Parks, the desire for protected surface water area, the need to maintain the South Shore Marina and beach, and the location of the parks within a long-established residential community with expressed historical and traditional values.

Several important conclusions were reached during the analyses of the alternatives. First, because of the large volume of stone required, it would be very expensive to reconstruct the entire breakwater to an elevation that would provide adequate protection against the design water level and storm wave condition. Second, it would also be very expensive to relocate a portion of the breakwater closer to shore because of the large volume of stone that would have to be moved. Third, the lowest cost alternatives would involve demolishing a portion of the breakwater and using the stone to reconstruct the remaining portion of the breakwater and to construct new onshore protection measures. Fourth, it was concluded that the recreational facilities and attendant opportunities could be enhanced at a relatively modest cost considering the benefits received.

In developing a recommendation for the South Shore breakwater, it was understood and fully recognized by the Intergovernmental Coordinating and Technical Advisory Committee that many residents in the South Shore and Bay View Park area strongly desire that the appearance and use of the shoreline and near-shore area remain relatively unchanged. To many residents, the aesthetic features of the shoreline and lake views have important historical significance. Indeed, South Shore and Bay View Parks provide some of the most unique and beautiful vistas of the Milwaukee lakefront.

[26]*Lakefront Recreational Development Task Force, op. cit.*

With those qualities in mind, the Committee recommended a modification of Alternative No. 5, the lowest cost alternative, which would include demolishing the breakwater south of E. Oklahoma Avenue extended, and using this stone to reconstruct the remaining portion of the breakwater and to build necessary onshore protection measures. This alternative would essentially preserve the appearance of the existing lakefront within the City of Milwaukee, where residential development adjoins the parkland. However, the Committee determined that beaches, rather than the revetments shown on Map 53 for Alternative No. 5, should be provided south of E. Oklahoma Avenue extended, to create a more accessible and usable shoreline. The coarse sand and gravel beaches would also reflect less wave energy than would revetments, and adjust and respond better to fluctuating water level conditions. The recommendations for South Shore and Bay View Parks would preserve the existing type of onshore protection: The revetments that currently lie between the South Shore Park beach and E. Oklahoma Avenue extended would be reconstructed; and the existing 20- to 50-foot-wide sand and gravel beaches that lie south of E. Oklahoma Avenue extended would be nourished with additional gravel, and groins would be constructed using stone from the demolished breakwater to contain the beach material. Map 56 illustrates the plan recommendations for the Bay View and South Shore Park areas. Demolishing the breakwater south of E. Oklahoma Avenue would entail a capital cost of about $1.6 million. Reconstructing and maintaining the remaining portion of the South Shore breakwater—which lies north of E. Oklahoma Avenue—would entail a capital cost of about $2.1 million, and an annual maintenance cost of about $231,000.

Auxiliary Plan Recommendations

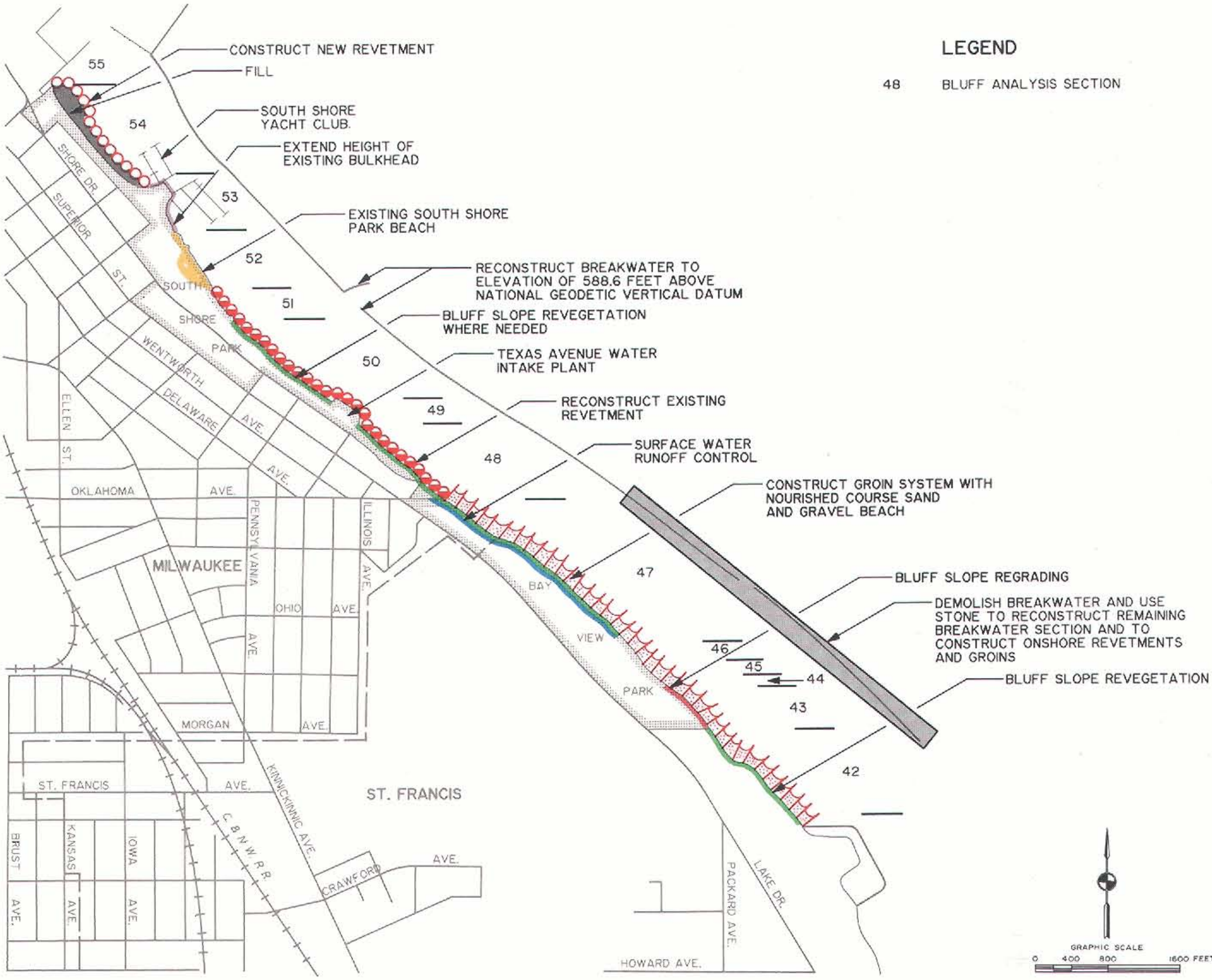
The foregoing recommendations address structural measures to protect the shoreline and stabilize the bluff slopes of Milwaukee County. There are a number of additional recommendations auxiliary to the plan recommendations which are related to the use of the shoreline. These auxiliary recommendations include an assessment of toxic substances contained in industrial wastes which have been placed on or near the bluffs; a coastal monitoring program for the Village of Fox Point terrace; and maintenance of navigation at the mouth of Oak Creek.

Assessment of Toxic Substances: Contamination of the water, bottom sediments, and certain biota by toxic substances has been of increasing concern on the Great Lakes, particularly near established urban areas such as Milwaukee. A potential source of toxic contamination is the erosion or seepage of industrial waste materials which have been dumped, buried, or stockpiled near the shoreline. Some of these industrial wastes may contain substances which are toxic to humans and to fish and other aquatic life. As presented in Chapter II of this report, three such areas along the Milwaukee County Lake Michigan shoreline lying outside the Milwaukee Harbor drainage area were identified as containing industrial waste materials which could result in the runoff or seepage of toxic substances into the lake. Important industrial material storage sites within the direct drainage area to the Milwaukee Harbor which may contribute contaminated stormwater runoff to the harbor were identified in SEWRPC Planning Report No. 37, A Water Resources Management Plan for the Milwaukee Harbor Estuary, 1987.

It is recommended that appropriately designed studies be conducted to ascertain the risk of toxic contamination associated with the industrial waste sites. The studies would be designed to document and characterize the toxic substance problems in specific areas along the Lake Michigan shoreline that have been identified as potentially containing industrial wastes, and which are to be considered for bluff slope stabilization. The studies, using currently available data and collecting additional data as may be needed, would identify and quantify the toxic substances of concern; evaluate the potential erosion, leakage, or seepage of the toxic substances into the lake; and develop and evaluate alternative methods of stabilizing the slope, abating the toxic problems, and protecting water quality and desired aquatic life. These studies would cost an estimated $10,000 for each waste storage site, or about $30,000 for the three currently known sites containing industrial wastes and which are recommended to be considered for bluff stabilization. These currently known sites are located in Section 24 within the City of Oak Creek, south of the City of Oak Creek water intake plant; in Warnimont Park within the City of Cudahy north of E. Ramsey Avenue extended; and in Section 13 within the City of South Milwaukee south of E. Drexel Avenue extended. The cost of these studies is

Map 56

RECOMMENDATIONS FOR THE SOUTH SHORE BREAKWATER AREA



Source: SEWRPC.

included in the recommended plan cost because they would be expended in conjunction with bluff stabilization projects.

<u>Village of Fox Point Coastal Monitoring Program</u>: The preliminary shoreline erosion management plan for northern Milwaukee County, set forth in SEWRPC Community Assistance Planning Report No. 155, recommended nourished gravel beaches for the Village of Fox Point terrace, which lies within Bluff Analysis Section 95. Objection to that recommendation was, however, expressed by both the Village of Fox Point and the lakefront property owners along N. Beach Drive, which lies within the terrace. These objections were related in part to the problem of increased traffic and parking on public lands, and to trespassing and vandalism on private lands which could result from increased use of the lakefront by the public. It was feared that the construction of a beach system along the terrace would attract too many inland users. The beaches were also objected to because some property owners along the terrace had already installed shore protection measures different from those recommended in the preliminary plan.

In response to the concerns raised by the Village Board and residents of Fox Point, the Advisory Committee for the northern Milwaukee County study concluded that the plan should be changed, and instead recommended the continued maintenance of existing structures along the privately owned shoreline of the Fox Point terrace and the reconstruction of the revetments along the village-owned shoreline adjacent to two portions of N. Beach Drive.

The final recommendations for the Fox Point terrace were made reluctantly by the Advisory Committee. The Committee was concerned about the long-term adverse effects on adjacent shoreline areas and on the offshore coastal environment that could be caused by wave energy reflected from the existing bulkheads and revetments. These adverse effects are most likely to occur in coastal areas with deep sand deposits and gentle offshore slopes, such as the Fox Point terrace area. The coastal area offshore of the Fox Point terrace is thus more susceptible to these effects than are most other locations along the Milwaukee county shoreline, and the low terrace is more susceptible to damage from increased wave attack than are other locations.

To help avoid the occurrence of serious, irreversible adverse effects on the Village of Fox Point terrace, it is recommended that a long-term, continuing coastal monitoring program be implemented by the Village of Fox Point along the 9,070-foot-long Fox Point terrace. This monitoring program would be intended to detect the early stages of any significant adverse effects caused by the existing—or any new—shore protection structures along the terrace. If significant effects are detected, corrective action—such as revising the design of some structures, structure modification, and localized beach nourishment—could be undertaken. If serious impacts should occur, further revisions to the final recommended plan should be considered. It is recommended that the coastal monitoring program include periodic bathymetric surveys, characterization of the composition of the coastal sediments, and observation of damage to structures. The bathymetric profiles should be prepared at approximately 1,000-foot intervals to a water depth of at least 12 feet below low water datum. Grab samples of the sediment may be used to characterize sediment composition. The structure evaluations would require onsite field inspections. It is recommended that the monitoring program initially be conducted at two- to five-year intervals. It is estimated that the recommended coastal monitoring program for the Fox Point terrace would entail a cost, distributed on an average annual basis, of $5,000.

Maintenance of Navigation at the Mouth of Oak Creek: The use of the recreational boat launch ramp located at the mouth of Oak Creek in Milwaukee County Grant Park is periodically denied by the formation of a sandbar at the mouth of the creek between the ramp and Lake Michigan. In order to alleviate this problem, it was recommended in the Regional Planning Commission Oak Creek watershed study[27] that a navigation channel be constructed at the mouth of Oak Creek and that this channel be maintained by the flushing of accumulated sand from it. This plan would be implemented by the provision of an approximately 20-foot-wide by four-foot-deep navigation channel at the mouth of Oak Creek through the construction of a jetty parallel to the north shore of the creek; by lowering of the sand level on the beach north of the channel to an elevation about two feet below the top of the jetty located on the north side of the Oak Creek channel; and by the performance of such minimal dredging of the navigation channel as may be necessary to maintain four feet of depth in the channel.

If the above measures do not yield adequate results, then it is recommended that a system of diffuser-type structures be constructed in the channel to resuspend the sand and help flush it from the creek. The shore protection measures set forth in this recommended plan would not be expected to significantly decrease or increase the shoaling problem. Construction of the proposed 20-foot-wide navigation channel may be expected to entail a capital cost, in 1988 dollars, of about $145,000, and an annual maintenance cost of about 5,000. If needed, the placement of a diffuser system may be expected to entail an additional capital cost of about $40,000. The plan recommends continued maintenance of the shore protection structures that lie adjacent to

[27]*SEWRPC Planning Report No. 36, A Comprehensive Plan for the Oak Creek Watershed, August 1986.*

the Oak Creek channel. These structures may need to be modified to accommodate the measures needed to resolve the shoaling problem.

Plan Cost Summary

The estimated cost of the recommended shoreline erosion management plan is presented in Table 67. The plan has an estimated capital cost of $82.3 million, an annual maintenance cost of about $4.1 million, and an equivalent annual cost of about $9.0 million. Public sector and private sector plan costs within each municipality are summarized in Table 68. Of the total plan equivalent annual cost, about $6.7 million, or 75 percent, would be financed by the public sector, and $2.3 million, or 25 percent, would be financed by the private sector. Milwaukee County would be responsible for about $4.3 million of the equivalent annual cost, or 48 percent of the plan total. Table 69 presents the recommended plan costs of each of the county and municipal parks located along the shoreline. A capital cost of about $45.8 million, or 56 percent of the total plan capital cost, and an annual maintenance cost of about $1.7 million, or about 41 percent of the total plan maintenance cost, would be required to protect parkland and facilities. The distribution of the plan costs, by type of control measure provided, is shown in Figure 121.

The recommended plan costs are best estimates at the systems planning level. Depending on site-specific characteristics, individual projects may cost substantially more or less than herein estimated. Where new construction or reconstruction is recommended, it was assumed that some of the material currently protecting the shoreline—primarily quarry stone—would be reused. It was also assumed that as the recommended structures are constructed over time, the design costs would eventually decrease as engineers and contractors became more familiar with the structure designs which are successful. It was further assumed that some economy-of-scale could be achieved by constructing measures to protect relatively long reaches of shoreline.

In addition to these specific plan recommendations, it is recommended that low-cost general shoreline management practices be followed by both public and private lakefront property owners, and that such owners consider the impact of land use or disturbance activities on the stability of the bluff slopes and the protection of the shoreline. Property owners should avoid the placement of heavy structures—such as swimming pools or garages—close to the bluff edge. Basic stormwater management should be practiced to reduce the amount of water infiltrating into, or discharging over, the bluffs. For example, rooftop downspouts should not be allowed to discharge to the lawns near the bluff edge. Lawn sprinkling should be minimized, and runoff from large impervious areas such as driveways should be diverted away from the bluff edge if possible. Finally, and perhaps most importantly, all lakefront property owners should practice sound vegetation management, maintaining a good vegetative cover of deep-rooting plants both on the bluff face and on the top of the bluff.

PLAN IMPLEMENTATION

The recommended shoreline erosion, bluff recession, and storm damage control plan for the Lake Michigan shoreline of Milwaukee County can be best implemented throughout entire reaches of shoreline having similar physiographic characteristics. The recommended control measures cannot be properly implemented on a piecemeal basis. To ensure proper design and maintenance, and to minimize construction impacts, these measures should be implemented within the entire implementation segments shown on Map 57. The shoreline length and location of each segment, along with the existing shoreline property owners, are provided in Table 70. There are 43 implementation segments along the county shoreline, with shoreline lengths ranging from 340 to 11,010 feet. The shoreline contained within each segment would, under the recommended plan, have a relatively uniform type of shore protection, and implementation of a project within an entire specified segment would not be expected to have an adverse effect on adjacent segments.

To assist in the implementation of the plan in the northern residential portion of Milwaukee County, SEWRPC Community Assistance Planning Report No. 155, A Lake Michigan Shoreline Erosion Management Plan for Northern Milwaukee County, Wisconsin, December 1988, proposed general locations for nine permanent access sites which would be used for the construction and continued maintenance of the recommended shore protection measures. Each of the sites

Table 68

DISTRIBUTION OF THE ESTIMATED COST OF THE RECOMMENDED SHORELINE EROSION MANAGEMENT PLAN

		Capital		Annual Maintenance		50-Year Present Worth		Equivalent Annual Cost	
Civil Division	Public or Private Sector	Cost	Percent of Total	Cost	Percent of Total	Cost	Percent of Total	Cost	Percent of Total
City of Oak Creek	Public Wisconsin Electric Power Company	$ 3,576,000	4.3	$ 67,000	1.6	$ 4,633,000	3.3	$ 294,000	3.3
	Milwaukee Metropolitan Sewerage District	2,370,000	2.9	48,000	1.2	3,117,000	2.2	198,000	2.2
	City of Oak Creek	81,000	0.1	8,000	0.2	209,000	0.1	13,000	0.1
	Milwaukee County	14,270,000	17.4	225,000	5.4	17,229,000	12.2	1,091,000	12.2
	Public Subtotal	$20,297,000	24.7	$ 348,000	8.4	$ 25,188,000	17.8	$1,596,000	17.8
	Private	$ 2,446,000	3.0	$ 134,000	3.2	$ 3,675,000	2.6	$ 233,000	2.6
	Total	$22,743,000	27.7	$ 482,000	11.6	$ 28,863,000	20.4	$1,829,000	20.4
City of South Milwaukee	Public City of South Milwaukee	$ 144,000	0.2	$ 6,000	0.2	$ 205,000	0.1	$ 13,000	0.1
	Milwaukee County	4,150,000	5.0	237,000	5.7	7,879,000	5.6	500,000	5.6
	Public Subtotal	$ 4,294,000	5.2	$ 243,000	5.9	$ 8,084,000	5.7	$ 513,000	5.7
	Private	$ 2,347,000	2.9	$ 126,000	3.0	$ 3,492,000	2.5	$ 222,000	2.5
	Total	$ 6,641,000	8.1	$ 369,000	8.9	$ 11,576,000	8.2	$ 735,000	8.2
City of Cudahy	Public City of Cudahy	$ 51,000	0.1	$ 5,000	0.1	$ 131,000	0.1	$ 8,000	0.1
	Milwaukee County	12,540,000	15.2	432,000	10.4	18,679,000	13.2	1,186,000	13.2
	Public Subtotal	$12,591,000	15.3	$ 437,000	10.5	$ 18,810,000	13.3	$1,194,000	13.3
	Private	$ 429,000	0.5	$ 20,000	0.5	$ 612,000	0.4	$ 39,000	0.4
	Total	$13,020,000	15.8	$ 457,000	11.0	$ 19,422,000	13.7	$1,233,000	13.7
City of St. Francis	Public Milwaukee County	$ 3,153,000	3.8	$ 121,000	3.0	$ 4,689,000	3.3	$ 298,000	3.3
	Private	$ 5,996,000	7.3	$ 153,000	3.6	$ 7,823,000	5.5	$ 496,000	5.5
	Total	$ 9,149,000	11.1	$ 274,000	6.6	$ 12,512,000	8.8	$ 794,000	8.8
City of Milwaukee	Public City of Milwaukee	$ 5,844,000	7.1	$ 200,000	4.8	$ 8,999,000	6.4	$ 571,000	6.4
	Milwaukee Metropolitan Sewerage District	0	- -	11,000	0.2	173,000	0.1	11,000	0.1
	Milwaukee County	6,678,000	8.1	532,000	12.9	14,981,000	10.5	951,000	10.5
	United States	1,840,000	2.2	906,000	21.9	16,112,000	11.4	1,023,000	11.4
	Public Subtotal	$14,362,000	17.4	$1,649,000	39.8	$ 40,265,000	28.4	$2,556,000	28.4
	Private	$ 1,000,000	1.2	$ 57,000	1.4	$ 1,765,000	1.2	$ 113,000	1.3
	Total	$15,362,000	18.6	$1,706,000	41.2	$ 42,030,000	29.6	$2,669,000	29.7
Village of Shorewood	Public Village of Shorewood	$ 1,613,000	2.0	$ 41,000	1.0	$ 2,259,000	1.6	$ 143,000	1.6
	Private	$ 1,697,000	2.1	$ 104,000	2.5	$ 3,214,000	2.3	$ 205,000	2.3
	Total	$ 3,310,000	4.1	$ 145,000	3.5	$ 5,473,000	3.9	$ 348,000	3.9
Village of Whitefish Bay	Public Village of Whitefish Bay	$ 330,000	0.4	$ 32,000	0.8	$ 723,000	0.5	$ 45,000	0.5
	Milwaukee County	265,000	0.3	27,000	0.7	683,000	0.5	43,000	0.5
	Public Subtotal	$ 595,000	0.7	$ 59,000	1.5	$ 1,406,000	1.0	$ 88,000	1.0
	Private	$ 3,094,000	3.8	$ 208,000	5.0	$ 5,676,000	4.0	$ 364,000	4.0
	Total	$ 3,689,000	4.5	$ 267,000	6.5	$ 7,082,000	5.0	$ 452,000	5.0
Village of Fox Point	Public Village of Fox Point	$ 928,000	1.1	$ 40,000	1.0	$ 1,555,000	1.1	$ 99,000	1.1
	Milwaukee County	1,418,000	1.7	29,000	0.7	1,867,000	1.3	119,000	1.3
	Public Subtotal	$ 2,346,000	2.8	$ 69,000	1.7	$ 3,422,000	2.4	$ 218,000	2.4
	Private	$ 1,290,000	1.6	$ 175,000	4.2	$ 3,658,000	2.6	$ 232,000	2.6
	Total	$ 3,636,000	4.4	$ 244,000	5.9	$ 7,080,000	5.0	$ 450,000	5.0

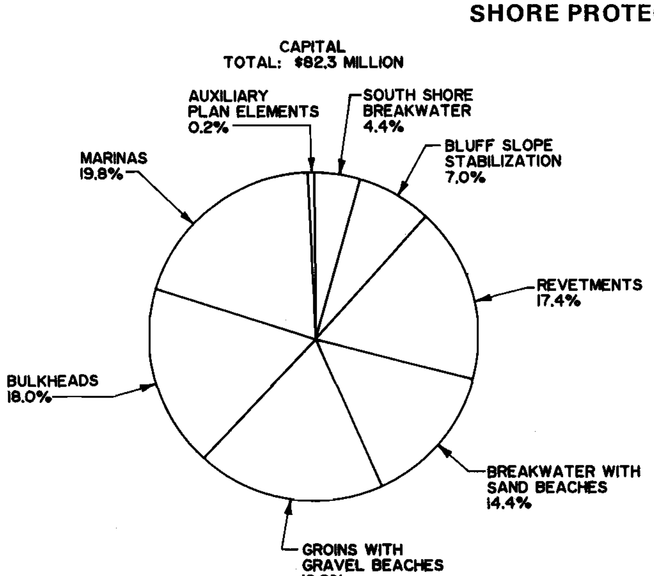
Table 68 (continued)

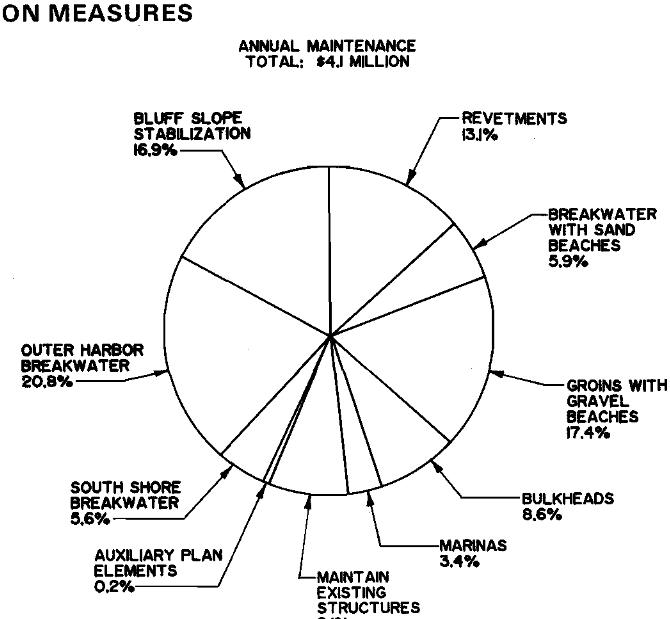
Civil Division	Public or Private Sector	Capital Cost	Capital Percent of Total	Annual Maintenance Cost	Annual Maintenance Percent of Total	50-Year Present Worth Cost	50-Year Present Worth Percent of Total	Equivalent Annual Cost Cost	Equivalent Annual Cost Percent of Total
Village of Bayside	Public Milwaukee County	$ 1,417,000	1.7	$ 28,000	0.7	$ 1,866,000	1.3	$ 118,000	1.3
	Private	$ 3,314,000	4.0	$ 169,000	4.1	$ 5,719,000	4.0	$ 362,000	4.0
	Total	$ 4,731,000	5.7	$ 197,000	4.8	$ 7,585,000	5.4	$ 480,000	5.3
Milwaukee County Total	Public Wisconsin Electric Power Company	$ 3,576,000	4.3	$ 67,000	1.6	$ 4,633,000	3.3	$ 294,000	3.3
	Milwaukee Metropolitan Sewerage District	2,370,000	2.9	59,000	1.4	3,290,000	2.3	209,000	2.3
	Village of Fox Point	928,000	1.1	40,000	1.0	1,555,000	1.1	99,000	1.1
	Village of Shorewood	1,613,000	2.0	41,000	1.0	2,259,000	1.6	143,000	1.6
	Village of Whitefish Bay	330,000	0.4	32,000	0.8	723,000	0.5	45,000	0.5
	City of Cudahy	51,000	0.1	5,000	0.1	131,000	0.1	8,000	0.1
	City of Milwaukee	5,844,000	7.1	200,000	4.8	8,999,000	6.4	571,000	6.4
	City of Oak Creek	81,000	0.1	8,000	0.2	209,000	0.1	13,000	0.1
	City of South Milwaukee	144,000	0.2	6,000	0.2	205,000	0.1	13,000	0.1
	Milwaukee County	43,891,000	53.3	1,631,000	39.4	67,873,000	47.9	4,306,000	47.9
	United States	1,840,000	2.2	906,000	21.9	16,112,000	11.4	1,023,000	11.4
	Public Subtotal	$60,668,000	73.7	$2,995,000	72.4	$105,989,000	74.8	$6,724,000	74.8
	Private	$21,613,000	26.3	$1,146,000	27.6	$ 35,634,000	25.2	$2,266,000	25.2
	Total	$82,281,000	100.0	$4,141,000	100.0	$141,623,000	100.0	$8,990,000	100.0

Source: SEWRPC.

Figure 121

DISTRIBUTION OF CAPITAL AND ANNUAL MAINTENANCE COSTS OF RECOMMENDED SHORE PROTECTION MEASURES





NOTE: OF THE TOTAL BLUFF SLOPE STABILIZATION ANNUAL MAINTENANCE COST, 69 PERCENT WOULD BE REQUIRED ONLY FOR THE FIRST THREE YEARS FOLLOWING COMPLETION OF A PROJECT.

Source: SEWRPC.

Table 69

RECOMMENDED SHORELINE EROSION MANAGEMENT PLAN COSTS FOR PUBLIC PARKS

Park	Location	Capital Cost	Annual Maintenance Cost
Milwaukee County Parks			
Bay View[a]	Cities of St. Francis and Milwaukee	$ 3,153,000	$ 121,000
Bender	City of Oak Creek	14,270,000	225,000
Big Bay	Village of Whitefish Bay	265,000	27,000
Doctors	Villages of Bayside and Fox Point	2,835,000	57,000
Grant	City of South Milwaukee	4,150,000	237,000
Juneau (includes McKinley Marina and the War Memorial Center)	City of Milwaukee	2,680,000	99,000
Lake (includes Bradford Beach)	City of Milwaukee	1,062,000	53,000
McKinley	City of Milwaukee	0	96,000
Sheridan	City of Cudahy	9,963,000	293,000
South Shore[a]	City of Milwaukee	2,936,000	284,000
Warnimont	City of Cudahy	2,577,000	139,000
Subtotal	- -	$43,891,000	$1,631,000
Municipal Parks			
Atwater	Village of Shorewood	1,185,000	24,000
Buckley	Village of Whitefish Bay	330,000	18,000
Klode	Village of Whitefish Bay	0	14,000
Nature Preserve	Village of Shorewood	428,000	17,000
Subtotal	- -	$ 1,943,000	$ 73,000
Public Parks Total	- -	$45,834,000	$1,704,000

[a]*Includes cost to carry out recommendations for the South Shore breakwater.*

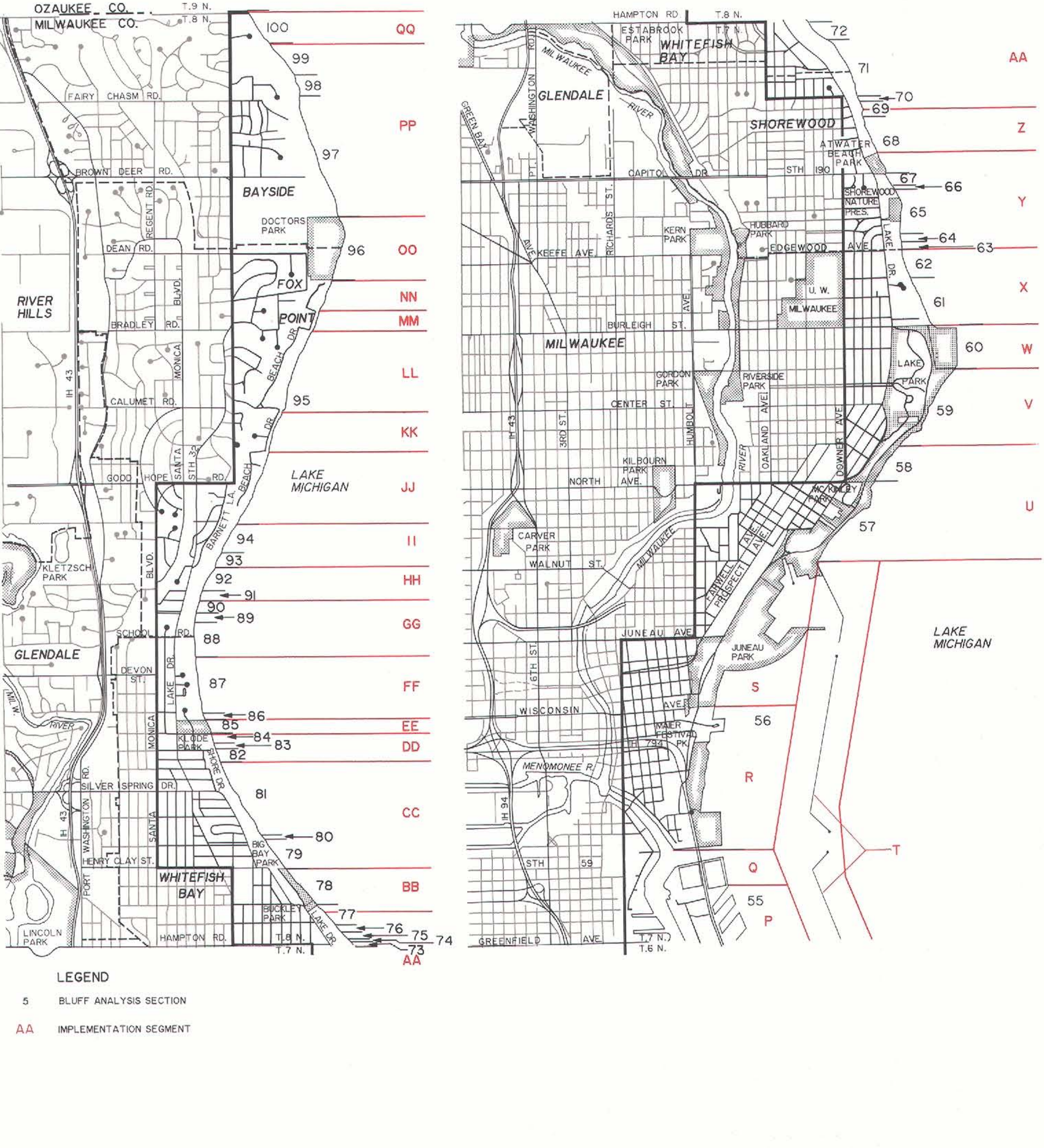
Source: SEWRPC.

could contain a permanent roadway, suitable for trucks and heavy construction equipment, extending down to the shoreline. The implementation segments that would be served by each of the proposed access sites were also designated in that plan. The provision of the permanent access sites would help centralize and thereby reduce the areawide impacts in residential areas of the movement of heavy equipment and large volumes of material. However, for the remaining portion of the county shoreline, access for construction and maintenance purposes generally should not present a problem, although some regrading of steep unstable bluffs may be required to allow trucks and equipment to work at the toe of the bluffs. Therefore, it was not believed necessary to designate additional permanent access sites in the recommended plan.

Because of variations in shoreline ownership patterns, the implementation program is discussed separately for three sections of the Milwaukee county shoreline. The northern Milwaukee County section includes the shoreline north of the City of Milwaukee Linnwood Avenue water treatment plant, and is primarily in residential use. The central Milwaukee County section includes the shoreline extending from the Linnwood Avenue water treatment plant southward to the southern edge of Bay View Park in

Map 57

IMPLEMENTATION SEGMENTS FOR THE RECOMMENDED PLAN

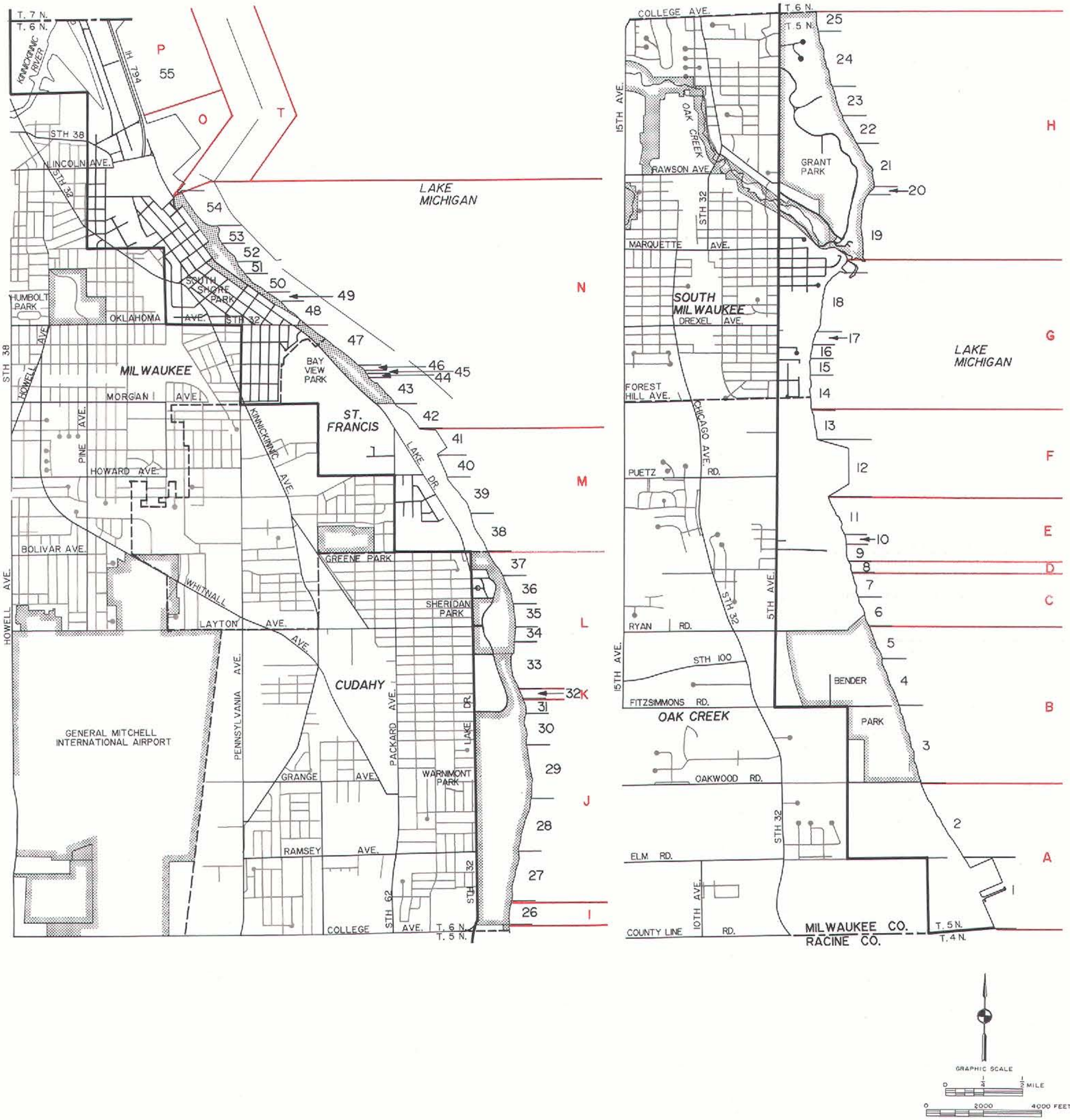


LEGEND

5 BLUFF ANALYSIS SECTION

AA IMPLEMENTATION SEGMENT

Map 57 (continued)



Source: SEWRPC.

Table 70

RECOMMENDED IMPLEMENTATION SEGMENTS FOR MILWAUKEE COUNTY

Implementation Segment	Shoreline Length (feet)	Address	Civil Division	Bluff Analysis Sections	Shoreline Property Owner
A	7,290	WEPCo Oak Creek Plant, Oakwood Road	Oak Creek	1-2	Wisconsin Electric Power Company
B	5,980	Bender Park	Oak Creek	3-5	Milwaukee County
C	2,170	9300-9180 S. 5th Avenue	Oak Creek	6-7	Private—industrial, open land
D	540	Oak Creek water intake	Oak Creek	8	City of Oak Creek
E	2,260	4301 E. Depot Road-8740 S. 5th Avenue	Oak Creek	9-11	Private—industrial
F	4,480	South Shore treatment plant—8400 S. 5th Avenue	Oak Creek	12-13	Milwaukee Metropolitan Sewerage District
G	5,590	3817 3rd Avenue—South Milwaukee Yacht Club	South Milwaukee	14-19	Private—residential, South Milwaukee Yacht Club, industrial, open land South Milwaukee sewage treatment plant
H	9,760	Grant Park	South Milwaukee	19-25	Milwaukee County
I	660	Lake Shore Tower Apartments	Cudahy	26	Private—apartments
J	7,030	Warnimont Park	Cudahy	27-31	Milwaukee County
K	340	Cudahy water intake	Cudahy	32	City of Cudahy
L	6,210	Warnimont Park-Sheridan Park	Cudahy	33-37	Milwaukee County
M	6,180	Lunham Avenue-Packard Avenue	St. Francis	38-42	Private—under development
N	11,010	Bay View Park-South Shore Park	St. Francis-Milwaukee	43-54	Milwaukee County
O	4,600	U. S. Army Corps of Engineers dredged spoils confined disposal facility and U. S. Coast Guard Station	Milwaukee	55	United States
P	9,050	South Lincoln Memorial Drive—Port of Milwaukee slips	Milwaukee	55	City of Milwaukee
Q	1,100	MMSD Jones Island wastewater treatment plant	Milwaukee	55	Milwaukee Metropolitan Sewerage District
R	6,200	Marcus Amphitheatre—Milwaukee Harbor Commission municipal pier	Milwaukee	56	City of Milwaukee
S	9,860	Milwaukee County War Memorial Center—McKinley Marina	Milwaukee	56	Milwaukee County
T	- -	Milwaukee outer harbor breakwater	Milwaukee	55-56	United States
U	5,110	McKinley Beach-Bradford Beach	Milwaukee	57-58	Milwaukee County
V	3,540	Lake Park	Milwaukee	59	Milwaukee County
W	2,210	Linnwood Avenue water treatment plant	Milwaukee	60	City of Milwaukee
X	2,920	UW Alumni Center—3473 N. Lake Drive	Milwaukee	61-62	Private—residential
Y	3,640	3510 N.Lake Drive-Atwater Park	Shorewood	63-68	Private—residential Village of Shorewood
Z	1,660	4060-4240 N. Lake Drive	Shorewood	68	Private—residential
AA	5,530	4300-4940 N.Lake Drive	Shorewood-Whitefish Bay	69-77	Private—residential
BB	1,660	Buckley Park-Big Bay Park	Whitefish Bay	78	Village of Whitefish Bay, Milwaukee County
CC	4,580	Big Bay Park-808 Lakeview Avenue	Whitefish Bay	79-81	Private—residential Village of Whitefish Bay, Milwaukee County
DD	1,060	5722-5866 N. Shore Drive	Whitefish Bay	82-84	Private—residential
EE	480	Klode Park	Whitefish Bay	85	Village of Whitefish Bay
FF	2,120	5960 N. Shore Drive-6260 N. Lake Drive	Whitefish Bay	86-87	Private—residential
GG	1,940	6310-6530 N. Lake Drive	Whitefish Bay-Fox Point	88-90	Private—residential

Table 70 (continued)

Implementation Segment	Shoreline Length (feet)	Address	Civil Division	Bluff Analysis Sections	Shoreline Property Owner
HH	1,280	6600 N. Lake Drive-6818 N. Barnett Lane	Fox Point	91-92	Private—residential
II	1,990	6820-7010 N. Barnett Lane	Fox Point	93-94	Private—residential
JJ	2,390	7038-7828 N. Beach Drive	Fox Point	95A	Private—residential
KK	1,600	7405-7535 N. Beach Drive	Fox Point	95B	Village of Fox Point
LL	3,000	7540-7966 N. Beach Drive	Fox Point	95C	Private—residential
MM	720	8005-8035 N. Beach Drive	Fox Point	95D	Village of Fox Point
NN	1,360	8040-8135 N. Beach Drive	Fox Point	95E	Private—residential
OO	1,890	Doctors Park	Fox Point-Bayside	96	Milwaukee County
PP	6,800	Schlitz Audubon Center-9364 N. Lake Drive	Bayside	97-99	Private—residential, Schlitz Audubon Center
QQ	1,320	9400-9578 N. Lake Drive	Bayside	100	Private—residential

Source: SEWRPC.

the City of St. Francis. The shoreline within this central section is owned by the United States, the Milwaukee Metropolitan Sewerage District, Milwaukee County, and the City of Milwaukee, and is used for dredged spoils disposal, sewage and water treatment facilities, port and marina facilities, and park and recreation activities. The southern Milwaukee County section includes the shoreline south of Bay View Park. The majority of this southern shoreline is owned by Milwaukee County and used for park purposes, although there are several isolated shoreline reaches used for residential, industrial, and utility purposes.

Northern Milwaukee County

For the reach of shoreline extending from the Linnwood Avenue water treatment plant to Doctors Park in the Village of Fox Point, three alternative implementation programs were presented and evaluated in SEWRPC Community Assistance Planning Report No. 155. That plan, however, did not address shoreline protection within the Village of Bayside, which was not included in that study.

The first alternative implementation program considered was the use the existing institutional structure and having Milwaukee County coordinate the implementation activities. Such implementation would rely upon the voluntary cooperation of all of the property owners within an implementation segment. Individual property owners would be able to effectively thwart proper plan implementation. Thus, this alternative method of implementation was not recommended.

The second alternative implementation program considered was the creation of a new lakeshore management district whose specific purpose would be to stabilize the bluff slopes and protect the shoreline. State legislation would be sought to enable the County and the local units of government concerned to cooperatively create a lakeshore management district. Upon an appropriate petition from the property owners within a designated implementation segment, the district would construct and maintain the recommended bluff stabilization and shore protection measures. In order to finance the improvements, the district governing body would have to be empowered to levy a tax upon all real property within the district, to make special assessments to benefiting parties for specific projects, and to contract and discharge debt. This approach would not have to rely upon the voluntary cooperation of all property owners within an implementation segment. All lakebed grants would be made to the lakeshore management district. This approach was not recommended because of anticipated opposition to the creation of a new taxing body, and because elected officials of the general-purpose units of government desired a more active role in the plan implementation process.

The third alternative implementation program considered would place primary responsibility with the municipalities. The municipalities could continue to issue permits for new shore protection measures; enter into contracts; levy taxes or special assessments; condemn property if necessary; and monitor compliance with the plan. To

Table 71

PLAN IMPLEMENTATION AUTHORITIES RECOMMENDED IN THE SHORELINE EROSION MANAGEMENT PLAN FOR NORTHERN MILWAUKEE COUNTY

Plan Implementation Authority	Implementation Agency Options
Administration, Coordination, and Information	Cooperative Contract Commission
Lakebed Grant Designee or Delegate	Municipalities
Review Proposed Projects	Cooperative Contract Commission and Municipalities
Issue Construction Permit for Shore Protection Structures	Municipalities or Cooperative Contract Commission
Contract to Construct and Maintain Shore Protection Structures	Cooperative Contract Commission, Municipalities, or Private Property Owners
Levy Taxes or Special Assessments	Municipalities
Condemn Property if Necessary	Municipalities
Monitor Plan Compliance and Maintenance	Cooperative Contract Commission and Municipalities

Source: SEWRPC.

enhance the efficiency of these functions, the municipalities could, under Section 66.30 of the Wisconsin Statutes, form a cooperative contract commission. This commission could exercise all of these functions with the following exceptions: It could not levy taxes or special assessments and it could not condemn property without the approval of the municipality.

This third approach to plan implementation was recommended in SEWRPC Community Assistance Planning Report No. 155, and is included in this plan. Because the powers of such a commission are limited, the municipalities would have to exercise their municipal authorities to help implement the plan—especially with respect to the financing of the projects and the condemnation of property.

During the preparation of SEWRPC Community Assistance Planning Report No. 155, it was initially considered preferable to have the specific plan implementation functions to be carried out by the proposed commission negotiated among the municipalities concerned. However, in response to requests made at public hearings on that plan, recommendations on the duties and functions of each implementation agency concerned were made in the final plan. The recommended implementation program calls for the creation of a municipal-cooperative contract commission and for shared implementation responsibilities between that commission and the local municipalities, as summarized in Table 71.

The contract commission, which would be created jointly by the municipalities, would have authority to provide information and administer and coordinate shore protection projects. The contract commission would also share responsibility with the municipalities for issuing permits and providing contracts for construction and maintenance of shore protection structures, and for monitoring plan compliance and the maintenance of shore protection measures. The commission would have no tax levy powers. Only the municipalities would have authority to levy taxes or special assessments to help finance projects. Under the recommended implementation program, projects could be initiated only after a petition by a majority of the property owners within an implementation segment had been submitted to the respective municipality. Project costs should be distributed based on the benefits received.

State legislative action would be sought to secure new lakebed grants to the municipalities in northern Milwaukee County except offshore of county parkland. Under the recommended implementation approach, Milwaukee County would retain ownership of the lakebed off county parkland, and would remain responsible for protecting the county-owned shoreline, without the need to obtain approval or permits from the municipalities or from a newly formed commission. However, it was recommended that the County cooperate with other property owners to implement projects within entire implementation segments.

Under the recommended implementation program, property owners under certain circumstances could be required to comply with the plan by the municipality. It was recommended that a municipality consider requiring a property owner to comply with the plan only where a petition for a project in conformance with the final recommended plan is submitted to the municipality by a majority of the property owners concerned; where plan compliance is necessary for the successful construction or maintenance of the project; and where failure to comply with the plan could result in an increased risk of damage or a significantly increased cost to other properties within the segment. The fact that a particular property may not require the same degree of added protection as do other properties in the segment would not, in itself, be considered a valid reason for not complying with the plan.

It is recommended that the Village of Bayside participate in the implementation program recommended for the remainder of the northern Milwaukee county shoreline, as documented in SEWRPC Community Assistance Planning Report No. 155. This participation would include joining the Villages of Fox Point, Shorewood, and Whitefish Bay and the City of Milwaukee in the creation of a cooperative contract commission, and carrying out the implementation activities identified for municipalities in Community Assistance Planning Report No. 155, as summarized in Table 71. The shoreline conditions and erosion problems within that Village, and the recommended shore protection measures, are similar to those found in the Villages of Shorewood, Whitefish Bay, and Fox Point.

This recommended implementation approach for northern Milwaukee County would ensure local control and management of the lakeshore; require local governmental approval for decisions related to condemnation of property and the financing of projects; and provide a new agency to assist the municipalities whose sole purpose is protecting the shoreline. This approach would provide an efficient and consistent mechanism for regulating shore protection measures and for ensuring the proper design, construction, and maintenance of such measures.

Central Milwaukee County

The institutional arrangements recommended above for northern Milwaukee County are needed to ensure proper design and coordination of shore protection projects undertaken by several adjacent property owners. Such an arrangement, however, is not required in central Milwaukee County because each project would be undertaken by a single property owner—either the City of Milwaukee, Milwaukee County, the Milwaukee Metropolitan Sewerage District, or the United States. Thus, an individual protection approach is recommended to carry out the plan recommendations in central Milwaukee County.

It is recommended that the City of Milwaukee construct and maintain the shoreline protection measures recommended for the Linnwood Avenue water treatment plant, and the shoreline within the Milwaukee outer harbor extending from the Milwaukee Harbor Commission Municipal Pier to the U. S. Army Corps of Engineers confined disposal facility. This shoreline area includes the Henry W. Maier festival grounds and the Port of Milwaukee facilities.

It is recommended that Milwaukee County construct and maintain the recommended control measures for the shoreline extending from the War Memorial Center northward through Lake Park, and for the shoreline in South Shore and Bay View Parks. It is also recommended that the County reconstruct and maintain the portion of the South Shore breakwater proposed to remain in place. The southern portion of the South Shore breakwater—south of E. Oklahoma

Avenue—would be demolished, with the stone used to reconstruct the remaining breakwater and onshore structures.

It is recommended that the Milwaukee Metropolitan Sewerage District continue to maintain the Jones Island sewage treatment plant bulkhead. The treatment plant was expanded eastward and a new bulkhead constructed in 1986 and 1987. Special measures were undertaken to reduce wave reflection impacts on the adjacent Port of Milwaukee ship-docking slip.

It is recommended that the U. S. Army Corps of Engineers continue to maintain the Milwaukee outer harbor breakwater, the confined disposal facility—until filled, and the U. S. Coast Guard Station bulkhead. When the existing confined disposal facility is filled, which is expected to occur in the mid-1990's, the area is intended to be regraded, landscaped, and converted to public recreational use.

It is recommended that all major shore protection projects in the central Milwaukee County section be constructed or modified within the entire implementation segments shown on Map 57. It is further recommended that all structures be designed and constructed in accordance with the design criteria set forth in Table 51. Projects would be financed by the responsible implementing agencies.

Southern Milwaukee County

Within the southern Milwaukee County section, Milwaukee County would be the lead governmental organization in coordinating plan implementation efforts. Municipality-based coordination, which is recommended for northern Milwaukee County, would not be effective because the vast majority of the shoreline is owned by the County and by other governmental agencies. The regulatory structure already put in place by Milwaukee County in carrying out its duties and responsibilities under the lakebed grants from the State of Wisconsin would provide the basic mechanism for coordinating the individual projects that may be proposed over time by the lakeshore property owners. Milwaukee County is the lakebed grant designee throughout the entire southern Milwaukee County section.

In order for Milwaukee County to properly coordinate and regulate the installation of structural measures along the southern county lakeshore, it would be necessary to relate the existing regulatory authority directly to the plan recommendations. It will be necessary for Milwaukee County to amend its existing ordinance to require that all permits henceforth issued for proposed structural measures along the southern portion of the county Lake Michigan shoreline be consistent with the plan recommendations. Such an implementation strategy will require a strong long-term commitment to the plan on the part of the County. Prior to issuance of any permit for a proposed structure, then, Milwaukee County would have to make a finding that the project is designed in accordance with the criteria set forth in the plan and that the geographic scope of the project is fully in accordance with the plan. This would mean that in some instances, permits for proposed shore protection structures would not be approved.

The main deficiency attendant to this approach to plan implementation lies in the fact that a particular proposal for which a permit is sought may fail to encompass an entire implementation segment, thus requiring denial of the permit. There is no readily apparent way under this recommended approach to plan implementation to require appropriate groups of property owners to act collectively in implementing the plan. However, since only three of the 13 implementation segments in the southern Milwaukee County section contain more than one property owner, this problem should occur infrequently. It is proposed that projects in these few sections be carried out through the voluntary, cooperative action of the property owners concerned.

Review of Implementation Program

Following a careful evaluation of the alternative implementation approaches available to carry out the recommended plan, it became apparent that no single approach would be effective throughout the entire Milwaukee county shoreline. Therefore, a municipality-based coordination approach was recommended for the northern Milwaukee County section to enhance local government control and to encourage the cooperation of residential property owners. For the central Milwaukee County section, it was concluded that each of the responsible governmental agencies could individually implement its protection structures without the need for an institutional arrangement to ensure coordination and cooperation. For the southern Milwaukee County section, county-based coordination was recommended to ensure the proper design and coordination of projects that usually would lie within, or adjacent to, county parkland.

It is important that the plan be adopted by the Milwaukee County Board of Supervisors and by the governing bodies of each of the shoreline municipalities. In addition, the plan should be endorsed or approved by the Wisconsin Department of Natural Resources and by the U. S. Department of the Army, Corps of Engineers. This overall policy level agreement with the plan recommendations is required to achieve the necessary intergovernmental coordination needed to implement shore protection projects in an effective and timely manner.

In addition to the implementation program described above, there would always remain in effect other existing institutional requirements that could affect the implementability of the plan. There would remain, for example, the need in every case to obtain a U. S. Army Corps of Engineers permit for any new shore protection structure. Presumably, if the Corps of Engineers endorsed the plan recommendations, and if the proposed structures were found to be in accordance with the plan recommendations, Corps of Engineers issuance of a permit would be routine. There would remain in effect zoning, erosion control, hauling and filling, and possibly other regulatory ordinances administered by the local governments concerned, the provisions of which would have to be met. Any local ordinance that requires a permit for the hauling of fill, for example, would constitute a possible constraint on a given project. Again, however, if the local governments endorsed the plan and if a particular proposed shoreline structure or bluff stabilization project along the shoreline were found to be in conformance with the plan, then issuance of any necessary local permits should be routine.

The recommended plan implementation program is summarized on Map 58. The specific implementation activities assigned to each governmental agency are listed in Table 72.

The successful implementation of the plan will require substantial capital expenditures and a commitment to carrying out long-term maintenance programs by those responsible for implementing the plan. As a systems level plan, this plan serves as a point of departure for the necessary preliminary engineering and site-specific analyses. Adoption and implementation of the recommended plan should ensure the provision of a high-quality, well-managed coastal environment for Milwaukee County.

SUMMARY

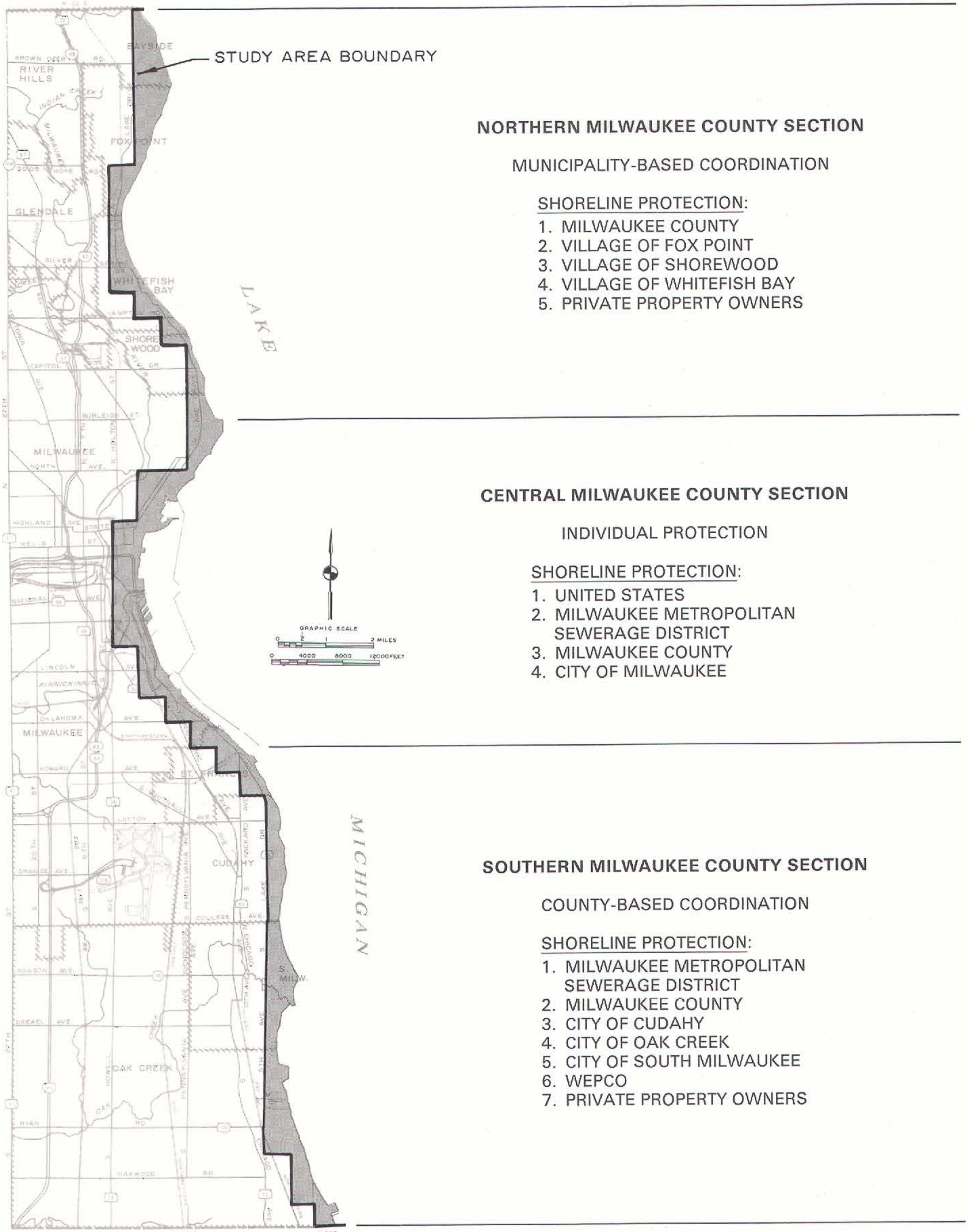
This chapter describes alternative structural and nonstructural methods of controlling, or reducing the damages from, shoreline erosion and bluff recession, and presents an evaluation of the costs and benefits of those alternative measures, broadly defined, as the basis for the selection of a recommended shoreline management plan for Milwaukee County. Various methods of implementing the recommended plan were considered, and an implementation program was proposed as part of the recommended plan. The recommended plan reflects the concerns and preferences of the local units of government and private lakefront property owners concerned.

This study is intended to constitute the first, or systems planning, phase of what may be regarded as a three-phase shore protection development process. Preliminary engineering is the second phase in this sequential process, with final design being the third and final phase. Analytical procedure and design criteria were presented to ensure a consistent basis for comparing alternative protection measures, and the characteristics, advantages, and disadvantages of the alternative measures were described. These procedures and criteria should also be helpful in the preliminary engineering and detailed design of shore protection measures.

Available types of shore protection measure designs were described. A combination of shoreline protection, bluff stabilization, surface water and groundwater drainage control, and revegetation will be required to adequately prevent bluff recession. Shoreline protection measures described included four types of revetments, three types of bulkheads, five types of onshore or near-shore beach systems, and six types of offshore structures. The capital costs of these structures were estimated to range from $150 to $2,000 per lineal foot of shoreline, with annual maintenance costs ranging from $5.00 to $50 per lineal foot. Bluff slope stabilization could be accomplished by cutting back, filling, cutting and filling, or terracing the bluff slope with retaining walls, with capital costs ranging from $100 to $3,500 per lineal foot of shoreline, and

Map 58

RECOMMENDED IMPLEMENTATION PROGRAM



Source: SEWRPC.

Table 72

RECOMMENDED GOVERNMENTAL PLAN IMPLEMENTATION RESPONSIBILITIES

Governmental Unit	General				Auxiliary		
	Adopt Plan	Approve or Endorse Plan	Develop Nonstructural Ordinance Provisions	Develop Nonstructural Advisory Guidelines	Study Toxic Substances in Bluffs	Protect Navigation in Oak Creek	Monitor Coastline in Fox Point Terrace
Villages							
Bayside	X	--	--	X	--	--	--
Fox Point	X	--	--	X	--	--	X
Shorewood	X	--	--	X	--	--	--
Whitefish Bay	X	--	--	X	--	--	--
Cities							
Cudahy	X	--	--	X	--	--	--
Milwaukee	X	--	--	X	--	--	--
Oak Creek	X	--	X	--	--	--	--
St. Francis	X	--	X	--	--	--	--
South Milwaukee	X	--	X	--	--	--	--
Other							
Proposed Northern Milwaukee County Cooperative Contract Commission	X	--	--	--	--	--	--
Milwaukee County	X	--	--	X	X	X	--
Milwaukee Metropolitan Sewerage District	--	--	--	--	--	--	--
State of Wisconsin	--	X	--	--	--	--	--
United States	--	X	--	--	--	--	--

Governmental Unit	Northern Milwaukee County Section								
	Protect Shoreline Property	Form Cooperative Contract Commission	Administration, Coordination, and Information	Lakebed Grant Designee	Review Proposed Project	Issue Permits for Structures	Levy Taxes or Special Assessments	Condemn Property if Necessary	Monitor Plan Compliance
Villages									
Bayside	--	X	--	X	X	X	X	X	X
Fox Point	X	X	--	X	X	X	X	X	X
Shorewood	X	X	--	X	X	X	X	X	X
Whitefish Bay	X	X	--	X	X	X	X	X	X
Cities									
Cudahy	--	--	--	--	--	--	--	--	--
Milwaukee	--	X	--	X	X	X	X	X	X
Oak Creek	--	--	--	--	--	--	--	--	--
St. Francis	--	--	--	--	--	--	--	--	--
South Milwaukee	--	--	--	--	--	--	--	--	--
Other									
Proposed Northern Milwaukee County Cooperative Contract Commission	--	--	X	--	X	X	--	--	X
Milwaukee County	X	--	--	--	--	--	--	--	--
Milwaukee Metropolitan Sewerage District	--	--	--	--	--	--	--	--	--
State of Wisconsin	--	--	--	--	--	--	--	--	--
United States	--	--	--	--	--	--	--	--	--

Governmental Unit	Central Milwaukee County Section		Southern Milwaukee County Section				
	Protect Shoreline Property	Develop Ordinance to Coordinate Plan Implementation	Protect Shoreline Property	Administration, Coordination, and Information	Review Proposal Projects	Issue Permits for Structures	Monitor Plan Compliance
Villages							
Bayside	--	--	--	--	--	--	--
Fox Point	--	--	--	--	--	--	--
Shorewood	--	--	--	--	--	--	--
Whitefish Bay	--	--	--	--	--	--	--
Cities							
Cudahy	--	--	X	--	--	--	--
Milwaukee	X	--	--	--	--	--	--
Oak Creek	--	--	X	--	--	--	--
St. Francis	--	--	--	--	--	--	--
South Milwaukee	--	--	X	--	--	--	--
Other							
Proposed Northern Milwaukee County Cooperative Contract Commission	--	--	--	--	--	--	--
Milwaukee County	X	X	X	X	X	X	X
Milwaukee Metropolitan Sewerage District	X	--	X	--	--	--	--
State of Wisconsin	--	--	--	--	--	--	--
United States	X	--	--	--	--	--	--

Source: SEWRPC.

an average annual maintenance cost of $5.00 to $15 per lineal foot for the first three years after construction. Groundwater drainage could be provided at a capital cost of $20 to $150 per lineal foot of shoreline, with an average annual maintenance cost of $5.00 to $20 per lineal foot. Surface water drainage control could be provided at a capital cost of $10 to $150 per lineal foot, with annual maintenance costs of up to $5.00 per lineal foot. Revegetating the bluff slope could be accomplished at a capital cost of $20 to $500 per 1,000 square feet, with an average annual maintenance cost of up to $15 per 1,000 square feet for three years. Procedures for delineating both nonstructural and structural setback distances for new buildings and facilities were also developed.

Alternative shore protection plans were presented for the entire Milwaukee county shoreline. The shoreline erosion management plan consists of two elements: a bluff stabilization element and a shoreline protection element. The bluff stabilization element specifies the measures needed to regrade or revegetate the slope and control groundwater or surface water flow. The capital cost of the preliminary bluff stabilization element is estimated at $7.4 million, the average annual maintenance cost at $823,000, and the equivalent annual cost over a 50-year period at $702,000.

Three alternative shoreline protection plans were developed. The first alternative plan assumed the use of revetments wherever practicable to protect the shoreline. The revetment alternative plan would have an estimated capital cost of $57.0 million, an annual maintenance cost of $3.3 million, and an equivalent annual cost over a 50-year period of $7.0 million. The second alternative plan for shoreline protection would provide, wherever practicable, gravel or sand beach systems. The beach alternative plan would have an estimated capital cost of $69.0 million, an average annual maintenance cost of about $3.6 million, and an equivalent annual cost over a 50-year period of $8.0 million. The third alternative plan would utilize offshore islands, peninsulas, and breakwaters to protect the shoreline and provide additional sand beaches, creating about 200 acres of new lakefront parkland. The offshore alternative plan would have an estimated capital cost of $199.8 million, an average annual maintenance cost of $3.5 million, and an equivalent annual cost over a 50-year period of $16.1 million.

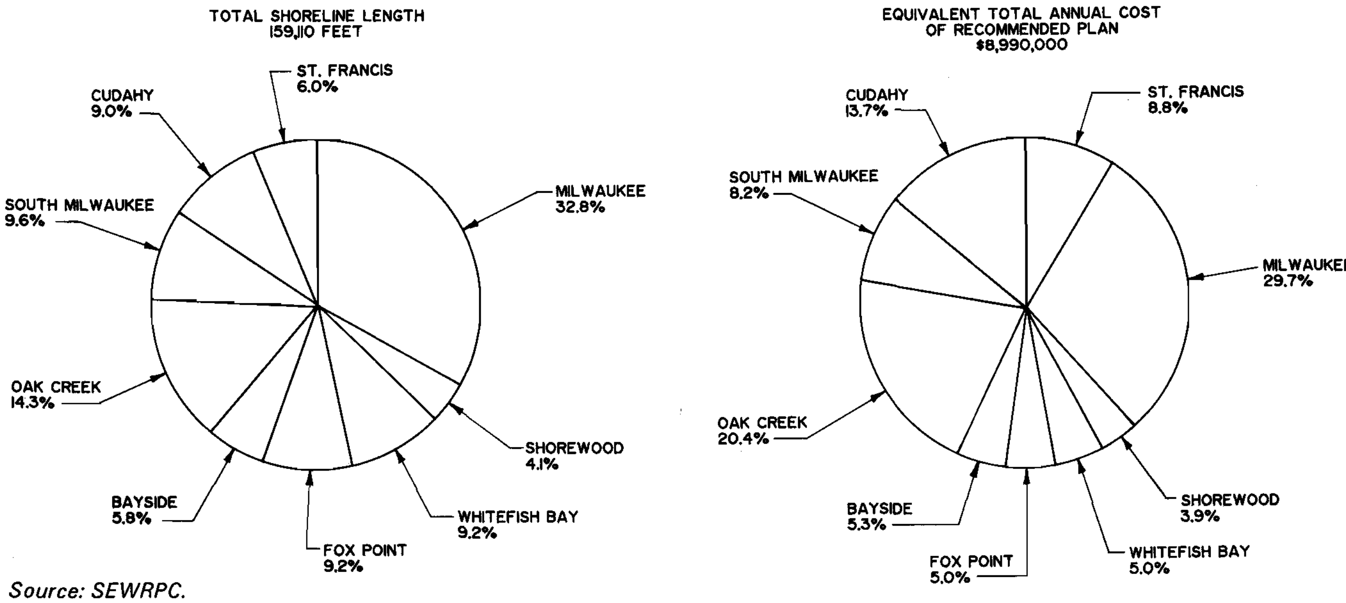
The recommended shoreline erosion, bluff recession, and storm damage control plan for Milwaukee County identifies those shore protection measures that would effectively abate the erosion problems within each section of shoreline; would recognize the preferences and priorities of the local units of government and lakefront private property owners; would be economically feasible and implementable; and would provide a usable shoreline to be enjoyed by those property owners as well as by the general public. To meet these needs, the recommended plan consists of a bluff stabilization plan element, and carefully selected components of all three alternative shoreline protection plans.

The recommended shoreline erosion management plan envisions that the bluff slopes would be stabilized by regrading and revegetating the bluff slopes along about 30,440 feet of shoreline, or 19 percent of the total county shoreline. It is recommended that studies be conducted to determine the feasibility of installing groundwater drainage systems along about 19,980 feet of shoreline, or 13 percent of the total county shoreline. Surface water drainage systems should be installed along about 4,010 feet of shoreline, or 3 percent of the total county shoreline. Under the recommended plan, about 12,060 feet of shoreline bluffs would be revegetated, covering about 8 percent of the total county shoreline. It is recommended that about 19,790 lineal feet of bluffs, or 12 percent of the county shoreline, be left to stabilize naturally. The bluff slope stabilization element of the recommended plan would entail a capital cost of about $5.8 million, an annual maintenance cost of about $699,000, and an equivalent annual cost of approximately $654,000.

The recommended plan includes the construction of two new marinas. A public marina would be constructed in Milwaukee County Bender Park in the City of Oak Creek, and a private marina would be constructed in the City of St. Francis on the former Wisconsin Electric Power Company Lakeside power plant property. Nourished sand beaches contained by offshore breakwaters would be constructed at the Village of Shorewood Atwater Park, at Milwaukee County Doctors Park, and at Sheridan Park. These breakwaters would contain a total of about 7,080 lineal feet, or 5 percent of the total county shoreline, and about 37 acres of public sand beach. Nourished gravel beaches contained by

Figure 122

DISTRIBUTION OF LAKE MICHIGAN SHORELINE
LENGTH AND RECOMMENDED PLAN COSTS
AMONG CIVIL DIVISIONS IN MILWAUKEE COUNTY



Source: SEWRPC.

rock groins would be located along about 36,850 feet, or 23 percent, of the total county shoreline. Riprap revetments would be constructed or reconstructed to protect about 44,840 feet, or 27 percent of the total county shoreline, including nearly all existing or proposed bluff fill projects.

About 31,050 feet of concrete or steel sheet pile bulkheads, covering about 19 percent of the county shoreline, would be reconstructed in order to reduce wave overtopping damage. It is also recommended that existing structures be maintained along 27,300 feet of shoreline, or 17 percent of the total county shoreline. In addition, the Milwaukee outer harbor breakwater would be maintained at its existing elevation. It is recommended that the portion of the South Shore breakwater located north of E. Oklahoma Avenue be reconstructed to an elevation of 588.6 feet National Geodetic Vertical Datum (NGVD), and that the portion of the breakwater south of E. Oklahoma Avenue be demolished, with the stone being used to reconstruct the northern portion of the breakwater, and to construct onshore protection measures. For 6,920 feet of shoreline, or 4 percent of the county total, shoreline erosion is not a significant threat, and no shore protection measures are recommended. The recommended onshore and offshore measures to protect the immediate shoreline from wave action would entail a capital cost of about $76.5 million, an annual maintenance cost of about $3.4 million, and an equivalent annual cost of approximately $8.3 million.

There are a number of additional recommendations auxiliary to the primary plan recommendations which are related to the use of the shoreline. These auxiliary recommendations include an assessment of toxic substances from industrial waste sites located on or near the bluffs, at a cost of approximately $30,000; maintenance of navigation at the mouth of Oak Creek, which would entail a capital cost of about $145,000 and an annual maintenance cost of about $5,000; and a coastal monitoring program for the Fox Point terrace, which would require an average annual cost of about $5,000.

The total capital cost of the recommended shoreline erosion management plan is about $82.3 million, the annual maintenance cost about $4.1 million, and the equivalent annual cost about $9.0 million. Of the total plan cost, about 25 percent would be financed by the private sector and 75 percent by the public sector. The distribution of the plan costs by municipality is shown in Figure 122. It is expected that the plan would be phased in by implementation segment as the need for additional shore protection arises, thus making the plan more economically feasible. However, steps should be undertaken to ensure that plan implementation activities do not come to a standstill when water levels are lower and there is reduced public interest in shoreline erosion problems. This plan should be viewed as an opportunity to begin what necessarily will be a long-term program of lakeshore improvements. Given the long lead times necessary for designing, funding, and constructing such improvements, the citizens of Milwaukee County will best be served by steady progress toward plan implementation, so that when lake levels do rise to high levels, the public and private shoreline property owners will be well prepared.

Following a careful review of the alternative implementation approaches available to carry out the recommended plan, it became apparent that no single approach would be effective throughout the entire Milwaukee county shoreline. Therefore, a municipality-based coordination approach was recommended for the northern Milwaukee County section to enhance local government control and to encourage the cooperation of residential property owners. For the central Milwaukee County section, it was concluded that each of the responsible governmental agencies could individually implement its protection structures without the need for an institutional arrangement to ensure coordination and cooperation. For the southern Milwaukee County section, county-based coordination was recommended to ensure the proper design and coordination of projects which usually would lie within, or adjacent to, county parkland.

It is important that the plan be adopted by the Milwaukee County Board of Supervisors and by the governing bodies of each of the shoreline municipalities. In addition, the plan should be endorsed or approved by the Wisconsin Department of Natural Resources and by the U. S. Department of the Army, Corps of Engineers. This overall policy level agreement with the plan recommendations is required to achieve the necessary intergovernmental coordination needed to implement shore protection projects in an effective and timely manner.

Chapter V

SUMMARY

INTRODUCTION

The erosion and recession of shorelines and bluffs constitutes one of the more difficult and costly problems facing private property owners and local governments along the Lake Michigan coastline. Average annual shoreline and bluff recession rates in Milwaukee County range up to 12.5 feet, while episodic rates can range up to 100 feet during major storms, as occurred in Klode Park in April of 1987. This recession results in an average annual loss of nearly 2.7 acres of land surface and nearly 330,000 cubic yards of shore material.

In the past, to protect both private and public property from erosion damage, various types of shore protection and bluff stabilization measures were constructed along the shore. While some lakefront properties, buildings, and facilities are well protected by these measures, other measures are ineffective. Some measures have been damaged by wave action; some interfere with the use of the shoreline and are perceived to be unsightly; and some may have accelerated the erosion of adjacent shoreline areas. Significant concern was expressed by elected officials and citizens about the effects of high lake levels, such as those which occurred in 1986, on existing shore protection measures, harbor facilities, and lakefront buildings and facilities. Therefore, a need developed to critically reexamine the approaches taken to protect the shoreline and to develop more cost-effective approaches to shore protection. Responding to the need for information and for proper guidelines and procedures to help lakefront property owners, Milwaukee County in 1986 asked the Southeastern Wisconsin Regional Planning Commission to prepared a shoreline erosion management plan for the County.

PURPOSE AND SCOPE OF STUDY

The Milwaukee County shoreline erosion management plan is intended to define the risk of erosion and bluff recession damage along the Lake Michigan shoreline; to explore alternative measures, and subsequently to recommend effective, economically feasible, and environmentally acceptable measures for shoreline protection; and to identify implementation mechanisms needed to carry out the recommended plan. To achieve these purposes, the study consisted of an inventory of erosion- and recession-related characteristics of the shoreline area; the identification of erosion risk areas and shoreline recession rates; an assessment of the effectiveness of existing shore protection structures under various storm wave and lake level conditions; an assessment of the stability of bluff slopes; the development and evaluation of alternative shore protection bluff recession control measures; and the preparation of a recommended plan.

The study was carried out under the guidance of an Intergovernmental Coordinating and Technical Advisory Committee created by the Regional Planning Commission and composed of representatives of each of the nine municipalities concerned, Milwaukee County, the Milwaukee Metropolitan Sewerage District, the Wisconsin Department of Natural Resources, the University of Wisconsin Sea Grant Institute, the University of Wisconsin-Milwaukee, the Milwaukee Audubon Society, and concerned and knowledgeable citizens. The study itself was conducted by the staff of the Regional Planning Commission with the assistance of consultants. The consultants included Professors Tuncer B. Edil and David M. Mickelson from the University of Wisconsin-Madison, who assisted in the bluff slope stability analyses; Professor Theodore Green III from the University of Wisconsin-Madison and Professor Kwang K. Lee from the University of Wisconsin-Milwaukee, who assisted in the wave analysis of existing shore protection structures; Mr. David J. Warren of W. F. Baird & Associates, Ltd., who conducted onsite inspections of shore protection structures; and Wisconsin Testing Laboratories, Inc., which conducted the soil borings for the study.

INVENTORY FINDINGS

The Milwaukee County shoreline erosion, bluff recession, and storm damage management study area was defined as the entire Lake Michigan shoreline of Milwaukee County and the Milwaukee Harbor area, including the shorelines in the

Cities of Oak Creek, South Milwaukee, Cudahy, St. Francis, and Milwaukee; and the Villages of Shorewood, Whitefish Bay, Fox Point, and Bayside. The study area is comprised of those lands that are most directly affected by Lake Michigan erosion processes and encompasses about 30 miles of shoreline and 12.5 square miles of land.

Those elements of the natural resource base within the study area pertinent to an understanding of coastal erosion and bluff recession processes were inventoried, including the bedrock geology and glacial deposits; soils; the height, slope, vegetative cover, stratigraphy, and stability of bluffs; the composition, slope, and width of beaches; the groundwater conditions; and the climate. The study area is underlain by Silurian, Ordovician, Cambrian, and Precambrian bedrock. Up to 200 feet of unconsolidated glacial deposits cover the bedrock, and include layers of the Kewaunee Formation, the Oak Creek Formation, the New Berlin Formation, and the Zenda Formation. The soils covering the upland portions of the study area generally have low infiltration capacity, low permeability, and poor drainage. The sandy soils which cover the terrace within the Villages of Fox Point and Bayside have moderate infiltration capacity, moderate permeability, and good drainage.

The bluffs along the Milwaukee County shoreline range up to nearly 130 feet in height above beach levels, with about one-half of the length of shoreline within the study area having bluffs greater than 70 feet in height. The two largest shoreline areas without lakeshore bluffs—the Milwaukee Harbor area and the terraced area within the Village of Fox Point—cover approximately 32 percent of the study area. The bluffs are generally comprised of glacial till and lacustrine sediments. About 65 percent of the shoreline surveyed in southern Milwaukee County and the Village of Bayside in 1987, and 69 percent of the shoreline surveyed in northern Milwaukee County in 1986, had a beach width of less than 10 feet.

Along the Milwaukee County shoreline, groundwater generally flows toward Lake Michigan. Two major aquifers underlie the coastal area: the deep sandstone aquifer and the Niagara dolomite aquifer. In addition, the sand and gravel glacial deposits that lie above the Niagara bedrock may act as water-bearing units. The presence of groundwater in this glacial bluff material reduces the frictional resistance to stress forces, creates a seepage pressure in the direction of water flow, and adds weight to the bluff.

Climate impacts on coastal erosion and bluff recession include freeze-thaw actions within bluff material; high surface runoff from frozen soils; lake ice effects; and high surface runoff and soil erosion during intense storms. Frozen ground and snow cover may be expected for approximately four months each winter season. About 15 percent of the average annual precipitation of 32.29 inches occurs as snowfall and sleet. Lake ice formation begins in late November or early December, and ice breakup normally occurs in late March or early April.

The type, degree, and extent of shore erosion and bluff recession damage is determined by the interrelationship of the natural and man-made features of the study area. In 1985 about 4,443 acres of the study area, or 59 percent of the total study area, was devoted to intensive urban uses. About 44 percent of the area devoted to intensive urban uses consisted of residential uses.

Shoreland development and activities are regulated by federal, state, and local units and agencies of government. The U. S. Army Corps of Engineers is the primary federal agency responsible for regulating certain structures, dredging, and wetland protection. Although the Wisconsin Department of Natural Resources regulates shore protection-related activities throughout most of the Lake Michigan shoreline of the State, 93 percent of the length of shoreline within the study area is regulated under lakebed grants made by the State Legislature to either Milwaukee County or the City of Milwaukee. Local zoning ordinances are currently in effect in each of the nine municipalities within the study area, but are generally devoid of provisions pertaining to Lake Michigan shoreline erosion and bluff recession hazards.

Numerous types of shore protection structures exist along the Milwaukee County shoreline. The effectiveness of these structures—which include groins, bulkheads, revetments, and breakwaters—has varied. A field inspection of all 128 shore protection structures in the study area conducted in 1986 and 1987 indicated that 75 percent exhibited some type of damage and required repair. Very little maintenance is performed on most structures.

A survey was conducted under the study in southern Milwaukee County and the Village of Bayside in October of 1987, and in northern Milwaukee county in May of 1986, to evaluate the physical and erosion-related characteristics of the bluffs. The results of the inventory indicated that the primary cause of bluff recession in the study area was bluff toe erosion by wave action. Groundwater seepage was also a major cause of slope failure in some portions of the study area. Shallow sliding was the most common type of slope failure, although many areas had deep-seated slumps. From 1963 through 1985, the bluff recession rate along 63 percent of the study area shoreline was less than 0.5 foot per year. Those areas with a recession rate equal to or more than 0.5 foot per year had a shoreline length-weighted mean of about 1.9 feet per year. The highest recession rate measured from 1963 through 1985 was 12.5 feet per year, which occurred near Bender Park within the City of Oak Creek.

EVALUATION OF COASTAL EROSION PROBLEMS AND DAMAGES

The identification of the shoreland areas that are expected to be affected by shoreline erosion and bluff recession enables public officials and concerned and affected private property owners to better assess potential erosion losses and to evaluate alternative shoreline erosion control measures. Therefore, the study included a determination of the stability of the bluff slope with respect to both rotational and translational sliding, and an assessment of the severity of bluff toe erosion.

The evaluation of the adequacy of existing shore protection structures and beaches, and the design of new structures, requires careful consideration of lake water levels. Public concerns about water levels were intensified by the high water levels that occurred in 1986. The annual mean lake level in 1986, and the monthly mean level that occurred in October of that year, set twentieth century record highs. A record high instantaneous maximum water level of 584.3 feet National Geodetic Vertical Datum (NGVD) occurred on March 9, 1987, during a severe storm that generated a 2.5-foot seiche and wind setup. Damages to shore protection structures are more severe during high-water periods than during low-water periods.

Statistical analyses of systematically recorded water levels, along with historical records and geological and archaeological evidence, and the results of hydrologic mathematical simulation modeling by the Great Lakes Environmental Research Laboratory of the National Oceanic and Atmospheric Administration and by the U. S. Army Corps of Engineers were used to select three maximum water levels to be used in the evaluation of the adequacy of existing structures. Those three maximum water levels were the 10-year recurrence interval instantaneous maximum water level of 582.8 feet NGVD; the 100-year recurrence interval instantaneous maximum water level of 584.3 feet NGVD; and the upper 90 percent confidence limit of the 500-year recurrence interval instantaneous maximum water level of 585.9 feet NGVD. In addition, two minimum water levels were selected to help evaluate the impacts of low water levels on existing structures; the 100-year recurrence interval minimum monthly mean water level of 575.5 feet NGVD; and the 100-year recurrence interval instantaneous minimum water level of 574.9 feet NGVD.

In order to provide an adequate level of protection against severe storms which occur during high water levels, it was recommended that major shore protection structures be designed to prevent severe damage under at least the 100-year recurrence interval instantaneous maximum water level of 584.3 feet NGVD. Since it may not be economically feasible for many residential lakefront property owners to construct shore protection structures designed to prevent damage during a 100-year recurrence interval water level, it was recommended that shore protection structures protecting single-family residential dwellings be designed to prevent damage during a major storm with at least a 10-year recurrence interval water level of 582.8 feet NGVD. It was recommended that all structures be designed to prevent severe damage during a 20-year recurrence interval wave height, which in deep water approximates 21.0 feet. In addition, it was recommended that all structures be designed to perform well under a range of water level and storm wave conditions.

With respect to rotational sliding, or slumping, 32 percent of the total length of the study area shoreline was determined to have stable bluff slopes; 11 percent marginal bluff slopes; and 25 percent unstable bluff slopes, as shown on

Map 35 in Chapter III. Bluff slope stability was not evaluated for the remaining 32 percent of the shoreline, consisting of the shoreline protected by the Milwaukee Harbor breakwater, the terrace directly north of the harbor which extends to the City of Milwaukee Linnwood Avenue water treatment plant, and the Fox Point terrace.

With respect to shallow translational sliding, 35 percent of the length of shoreline in the study area was determined to have stable bluff slopes, 10 percent marginal bluff slopes, and 23 percent unstable bluff slopes.

With respect to shoreline erosion, 50 percent of the length of shoreline within the study area was observed to exhibit little or no evidence of significant shoreline erosion resulting in the retreat of the shoreline in 1986. About 23 percent of the length of shoreline was found to be exhibiting erosion at the shoreline, but the erosion did not appear to affect the overall stability of the bluff slope. The remaining 27 percent of the shoreline length was observed to exhibit toe erosion which was threatening the overall stability of the bluff slope.

The shore protection needs for each of 100 bluff analysis sections within the study area were identified. It was indicated that in order to stabilize the bluff slopes, about 28 percent of the length of shoreline within the study area would need to be regraded to a stable slope angle and revegetated; groundwater drainage systems would need to be installed to lower the evaluation of the groundwater along about 6 percent of the length of shoreline; surface water runoff control measures would need to be implemented along about 3 percent of the length of shoreline; and the bluff slope along about 7 percent of the length of shoreline would need be revegetated.

The performance of 35 major shore protection structures and beaches was evaluated under six different Lake Michigan maximum water level and storm wave conditions. For each structure or beach, the potential for wave overtopping damage was classified as insignificant, low, moderate, or high. Overall, from 49 to 57 percent of the structures and beaches would have a moderate or high potential for overtopping damage under a 10-year water level, compared to 71 to 77 percent of the structures and beaches for a 100-year water level and 80 to 89 percent for a 500-year water level. In addition, toe erosion at 13 structures could increase under very low water levels because the toes, or bases, of the structures would be exposed to direct wave attack.

The land area lying within 25-year and 50-year bluff recession distance of a marginal or unstable bluff or terrace was delineated for the entire study area shoreline. The area lying within the 25-year bluff recession distance of the marginal or unstable bluffs and terraces was found to total nearly 63 acres of land and to contain 24 buildings, with the land and buildings having a 1986 economic market value of about $4.7 million. The area lying within the 50-year bluff recession distance of the marginal or unstable bluffs and terraces was found to total about 126 acres of land and to contain 44 buildings, with the land and buildings having a 1986 economic market value of about $8.7 million.

ALTERNATIVE SHORELINE EROSION MANAGEMENT MEASURES

Specific structural shore protection measures required at any particular site can be properly determined only on the basis of a detailed engineering analysis of the physical characteristics of the site, the causes of erosion on the site, the degree of erosion expected, and property values. Bluff toe protection measures evaluated for Milwaukee County included four types of revetments, three types of bulkheads, five types of onshore or near-shore beach systems, and six types of offshore structures. The installation of the bluff toe protection structures would entail a capital investment of $150 to about $2,000 per lineal foot of shoreline, with average annual maintenance costs ranging from $5.00 to $50 per lineal foot. Bluff slope stabilization could be accomplished by cutting back, filling, cutting and filling, or terracing the bluff slope with retaining walls at a capital cost ranging from $100 to $3,500 per lineal foot of shoreline, and an average annual maintenance cost of $5.00 to $15 per lineal foot for the first three years after construction. Improved groundwater drainage could be provided at a capital cost of $20 to $150 per lineal foot of shoreline, with an average annual maintenance cost ranging from $5.00 to $10 per lineal foot. Improved surface water drainage control could be provided at a capital cost of $10 to $150 per lineal foot of shoreline, with average annual maintenance costs of up to $5.00 per lineal foot. Revegetating the bluff slope could be accomplished at a capital cost of $20 to $500 per 1,000 square feet, with an average annual maintenance cost of up to $15 per 1,000 square feet for three years.

Alternative erosion management plans for the study area shoreline consisted of two elements: a bluff slope stabilization plan element and a shoreline protection plan element. A single bluff slope stabilization plan was presented, along with three alternative shoreline protection plans. The preliminary bluff slope stabilization plan, which specifies the measures needed to regrade or revegetate the slope and control groundwater or surface water flow, and which should be implemented regardless of the toe protection measures selected, would entail a capital cost of approximately $7.4 million, an average annual maintenance cost of about $823,000, and an equivalent annual cost over a 50-year period of $702,000. The bluff slope stabilization plan is shown on Map 38 in Chapter IV.

The revetment alternative shoreline protection plan, shown on Map 39 in Chapter IV, which proposes the use of riprap revetments wherever practicable to protect the shoreline, would entail a capital cost of about $57.0 million, an average annual maintenance cost of about $3.3 million, and an equivalent annual cost over a 50-year period of about $7.1 million. The beach alternative shoreline protection plan shown on Map 40 in Chapter IV, which would provide wherever practicable artificially nourished beach systems, would entail a capital cost of about $69.0 million, an average annual maintenance cost of about $3.6 million, and an equivalent annual cost over a 50-year period of about $8.0 million. The offshore alternative shoreline protection plan shown on Map 41 in Chapter IV, which would utilize offshore islands and breakwaters to protect the shoreline, would entail a capital cost of approximately $199.8 million, an average annual maintenance cost of about $3.4 million, and an equivalent annual cost over a 50-year period of $16.1 million.

In addition to the alternative shoreline protection plans, several alternatives were considered for the South Shore and Milwaukee outer harbor breakwaters. Under the South Shore breakwater alternatives, which included various combinations of reconstructing, relocating, and demolishing the breakwater, as well as constructing needed onshore measures, the capital cost ranged from $5.0 to $11.8 million, with annual maintenance costs ranging from $269,000 to $554,000. Under the Milwaukee outer harbor breakwater alternatives, the capital cost of breakwater modifications and needed onshore measures ranged from $9.0 to $65.3 million, with annual maintenance costs ranging from $0.7 to $1.2 million.

RECOMMENDED SHORELINE EROSION MANAGEMENT PLAN

Upon careful consideration of the alternatives, the Intergovernmental Coordinating and Technical Advisory Committee selected a recommended plan which both fully stabilizes the bluff slopes and protects the immediate shoreline from wave and ice erosion on a long-term basis. This plan, which consists of a bluff slope stabilization element, and a combination of the best components of each of the alternative shoreline protection plans considered, sought those shore protection measures which, when applied on a reach-by-reach basis, would effectively abate the erosion problems, would recognize the preferences and priorities of the local units of government and lakefront property owners concerned, would be economically feasible and implementable, and would provide—where practicable—a usable shoreline to be enjoyed by the general public, as well as by lakefront property owners. The recommended plan is shown on Map 55 in Chapter IV.

Under the recommended shoreline erosion management plan, the bluff slopes would be stabilized by regrading and revegetating the bluff slopes along about 30,440 feet of shoreline, or 19 percent of the total county shoreline. It is recommended that studies be conducted to determine the feasibility of installing groundwater drainage systems along about 19,980 feet of shoreline, or 13 percent of the total county shoreline. Surface water drainage would be installed along about 4,010 feet of shoreline, or 3 percent of the total county shoreline. Under the recommended plan, about 12,060 feet of shoreline bluffs would be revegetated without being regraded, covering about 8 percent of the total county shoreline. The bluff slope stabilization element of the recommended plan would entail a capital cost of about $5.8 million, an annual maintenance cost of about $699,000, and an equivalent annual cost of approximately $654,000. This cost is somewhat less than the cost of the preliminary bluff slope stabilization plan because the recommended plan proposes that about 19,790 lineal feet of bluffs, or 12 percent of the county shoreline—comprising about one-third of all of the marginal or unstable bluffs—be left to stabilize naturally.

The recommended plan includes the construction of two new marinas: a public marina to be constructed in Milwaukee County Bender Park in the City of Oak Creek, and a private marina to be constructed in the City of St. Francis on the former Wisconsin Electric Power Company Lakeside power plant property. A total of 37 acres of nourished sand beaches contained by offshore breakwaters would be constructed at the Village of Shorewood Atwater Park, and Milwaukee County Doctors Park and Sheridan Park. These breakwaters would contain a total of about 7,880 lineal feet, or 5 percent, of the total county shoreline. Nourished gravel beaches contained by rock groins would be located along about 36,850 feet, or 23 percent, of the total county shoreline. Quarry stone revetments would be constructed or reconstructed to protect about 44,840 feet, or 28 percent, of the total county shoreline, including nearly all existing or proposed bluff fill projects.

Although no new bulkheads would be constructed, about 31,050 feet of existing concrete or steel sheet pile bulkheads, covering about 19 percent of the county shoreline, would be reconstructed by increasing the height of the structure or by placing a riprap berm in front of the bulkhead in order to reduce the potential for wave overtopping damage. Existing structures would be maintained along 27,300 feet of shoreline, or 17 percent of the total county shoreline.

Under the recommended plan, the Milwaukee outer harbor breakwater would be maintained at its existing elevation. It is much less expensive to modify or reconstruct threatened onshore structures than it is to substantially modify the outer harbor breakwater. It is recommended that the portion of the South Shore breakwater located north of E. Oklahoma Avenue be reconstructed to an elevation of 588.6 feet NGVD, and that the portion of the breakwater south of E. Oklahoma Avenue be demolished, with the stone being used to reconstruct the northern portion of the breakwater, and to construct onshore protection measures. For only 6,920 feet of shoreline, or 4 percent of the county total, shoreline erosion is not a significant threat, and no shore protection measures are recommended. The onshore and offshore measures recommended to protect the immediate shoreline from wave action would entail a capital cost of about $76.5 million, an annual maintenance cost of about $3.4 million, and an equivalent annual cost of approximately $8.3 million.

Additional recommendations auxiliary to the primary plan recommendations include an assessment of toxic substances from industrial waste sites located on or near the bluffs, at a cost of approximately $30,000; maintenance of navigation at the mouth of Oak Creek, which would entail a capital cost of about $145,000 and an annual maintenance cost of about $5,000; and a coastal monitoring program for the Fox Point terrace, which would require an average annual cost of about $5,000.

The total capital cost of the recommended shoreline erosion management plan is about $82.3 million, the annual maintenance cost is about $4.1 million, and the equivalent annual cost about $9.0 million. Of the total plan cost, about 25 percent would be financed by the private sector and 75 percent by the public sector.

It is expected that the plan would be phased in as the need for additional shore protection arises, thus making the plan more economically feasible. However, steps should be undertaken to ensure that plan implementation activities do not come to a standstill when water levels are lower and there is reduced public interest in shoreline erosion problems.

PLAN IMPLEMENTATION

The recommended plan can be best implemented within 43 specified reaches of shoreline referred to as implementation segments. The delineated implementation segments are shown on Map 57 in Chapter IV. Following a careful review of alternative implementation approaches available to carry out the recommended plan, it became apparent that no single approach would be effective throughout the entire Milwaukee County shoreline. Therefore, a municipality-based coordination approach was recommended for the northern Milwaukee County section—north of the City of Milwaukee Linnwood Avenue water treatment plant—to enhance local government control and to encourage the cooperation of residential property owners. For the central Milwaukee County section—from Bay View Park northward to the Linnwood Avenue water treatment plant—it was concluded that each of the responsible governmental agencies could individually implement its protection structures without the need for an institutional arrangement to ensure coordination and cooperation. Those agencies responsible for shoreline protection in the central section include the City of Milwaukee, Milwaukee County, the Milwaukee Metropolitan Sewerage District, and the U. S. Army Corps of Engineers. For the southern Milwaukee County section—south of Bay View Park—county-based coordination was recommended to ensure the proper design and coordination of projects which usually would lie within, or adjacent to, county parkland.

Historically, much of the Milwaukee County shoreline has been lined with shore structures which do not provide an adequate level of protection, and which were constructed of whatever materials happened to be readily—and economically—available at the time. Far too little maintenance of shore protection structures has been performed by both the public and private sectors. This approach to protecting the shoreline has led to the severe erosion and bluff recession problems described in Chapter II, and to the creation of miles of inaccessible and unusable shoreline. Continuation of this approach would undoubtedly lead to an increase in erosion damages, and a decrease in the length of accessible and usable shoreline as more structures built over the last five decades collapse.

To a limited extent, there has been an increasing trend on the part of local governments to design and construct more expensive structures which do provide the needed level of protection. These recent projects include the McKinley armored headland/pocket beach system, the Klode Park breakwater and beach system, and the island being constructed offshore of the Henry W. Maier festival grounds. These projects were, in large part, generated in response to the high water levels in 1985 and 1986 and because of the availability of deep tunnel spoils for fill material. Since 1985, shore protection designs have also been commissioned by Milwaukee County for Lake, South Shore, Bay View, Big Bay, and Bender Parks.

However, there remain three primary barriers to implementation, all interrelated, that must be overcome by local units of government. The first primary barrier often cited by government officials and informed citizens is complacency. Public demand and support for governmental action obviously peaks when the threat to life or property is imminent. As the threat lessens, the interest in governmental action recedes, and, in the case of shoreline protection, complacency sets in. Hopefully, this report as well as subsequent educational efforts will help sustain public support and demonstrate that attempts to protect the shoreline in the midst of a crisis often lead to poorly thought out and inadequate solutions.

A second primary barrier to implementation is a reluctance to make the necessary public investment in capital improvement and maintenance programs. The value of the Lake Michigan shoreline on the quality of life in Milwaukee County should be recognized, and public investments made accordingly. Of course, investments should first be made to protect threatened vital public facilities. But financial support will also be needed for other shore protection projects which are not a response to a crisis situation.

A third primary barrier to implementation is the fear of making large public investments in a project which may literally be "high and dry" when lake levels are receding, or in a project that in the future may require modification or substantial maintenance. Such projects, unfortunately, are erroneously often considered wasteful spending. Again, educational efforts are needed to inform public officials, and the general public, that lake level predictions are an inexact science, that most structures should be designed to protect against a storm which occurs very infrequently, and that the nature of shoreline protection often requires continued modification and maintenance.

The successful implementation of the final recommended plan will require not only a substantial capital investment, but a stable, long-term commitment to carrying out the recommended capital improvement projects and related maintenance programs. The Milwaukee County shoreline of Lake Michigan is beautiful, rugged, often tranquil, and rich in habitat value and in the diversity and quality of its natural resources. The shoreline offers unique and invaluable recreation opportunities which are a source of great pleasure for thousands of Milwaukee County residents. Adoption and implementation of this plan should ensure that the scenic and natural resource characteristics, and potential recreational amenities, of the Lake

Michigan shoreline are enhanced, protected, managed, and passed onto future generations.

PUBLIC REACTION TO THE RECOMMENDED PLAN

A formal public hearing on the recommended shoreline erosion management plan was held on Wednesday, September 27, 1989, at 7:00 p.m. at the South Shore Park Pavilion, Milwaukee. The purpose of the hearing was to present the findings and recommendations of the plan for review and comment by interested citizens, lakefront property owners, and public officials. The hearing was announced through news releases sent to the local media and through the distribution of a SEWRPC Newsletter which summarized the plan.[1] Copies of selected newspaper articles dealing with the plan are presented in Appendix D.

Committee Chairman Daniel Cupertino opened the hearing with a statement of purpose. A summary of the findings and recommendations of the planning effort was then presented by the Commission staff. The summary statement focused on the alternative shore erosion management plans considered, and on the recommended plan as proposed by the Advisory Committee for public hearing.

The following summarizes the comments received at the hearing and the staff and Intergovernmental Coordinating and Technical Advisory Committee responses thereto:

1. The Administrator of the City of St. Francis, Mr. Ralph J. Voltner, Jr., entered into the record a letter from Mayor Milton Vretenar commenting on the plan. A copy of the letter is included in Appendix E. Mr. Voltner summarized the contents of the letter, noting that the City of St. Francis opposed the recommendation to demolish the portion of the South Shore breakwater lying south of E. Oklahoma Avenue extended and which protects a portion of the City of St. Francis shoreline.

   Mr. Voltner indicated that the City was concerned that if the breakwater were demolished, there would be no assurance that proper onshore protection measures would be constructed and maintained. Because the South Shore breakwater, along with much of the shoreline, is owned by Milwaukee County, the City would be unable to insure that the plan would be implemented in its entirety. Mr. Voltner noted that the City of St. Francis recommended that Alternative No. 1 for the South Shore breakwater, under which the entire existing breakwater would be reconstructed to a crest elevation of 588.6 feet NGVD, be included in the recommended plan. Mr. Voltner added that Alternative No. 1 does not differentiate between community boundaries, and under this alternative Milwaukee County would assume its rightful responsibility to protect uniformly the existing lakefront behind the breakwater.

2. Mr. John Ebersol, a Bay View resident and a representative of Save Our Shores, a community organization interested in preserving the existing shoreline area, stated that that organization believes that removal of the breakwater would not be in the best interest of the community and that the protected water area for boaters would be significantly decreased. Mr. Ebersol stated that the organization favored maintaining the entire South Shore breakwater, rebuilding it as necessary to stabilize the structure.

3. Mr. John Sternkopf, a resident of S. Superior Street, commented that the shoreline and shoreline uses would be best served by maintaining and improving the entire existing South Shore breakwater. Mr. Sternkopf cited the removal of the water intake crib in the offshore of McKinley Beach as a project which should have been reconsidered since that structure may have been useful as a start of a shore protection system. He noted that once a structure is removed, its potential future usefulness is lost. Mr. Sternkopf also stated that the community uses, including aesthetics, would be aided by repairing the breakwater at its current location. He therefore opposed demolishment of any

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[1] *SEWRPC Newsletter, Vol. 29, No. 3, May-June 1989, "Lake Michigan Shoreline Erosion Management Plan Prepared for Milwaukee County."*

portion of the breakwater and supported its repair and reconstruction.

4. Mr. Robert Quincy, a resident of S. Superior Street, also favored reconstructing the existing breakwater to a higher elevation, which he said would provide for better lake uses for the South Shore and Bay View area.

5. Christine B. Bastian, Mayor of the City of Oak Creek, supported the plan. Mayor Bastian appealed to Milwaukee County to consider seriously the proposals and analyses in the plan particularly as related to the protection of the Bender Park shoreline. While recognizing that construction of a major project such as the proposed marina at Bender Park would have to be implemented over a period of time, the Mayor requested that the County begin the process by committing funds for the initial stages of such a project.

6. Mr. J. Gerald Schlosser, a resident of Santa Monica Boulevard, asked about the function of the private marina proposed to be constructed at the old Lakeside electric power generation plant site, and how that marina would help improve the shoreline of the area. The Commission staff responded that a marina at that location would be compatible with adjacent shoreline protection measures and uses, such as the nourished gravel beach which lies north of the proposed private marina and the other facilities recommended for Bay View and South Shore Parks.

7. Milwaukee County Supervisor John St. John commented that Lake Michigan shoreline erosion is an enormous problem which presents several lifetimes of challenges, and will require intergovernmental consensus and cooperative decision-making over a long period of time. He indicated that he preferred those shore protection measures which serve dual purposes, not only protecting the shoreline but also providing some economic benefits to offset the cost of the structures. In this regard, he noted that the Bender Park marina which has been proposed may be constructed with a combination of public and private funding. Supervisor St. John also noted another dual purpose use concept which should be considered in selecting alternatives. That concept would provide both erosion control and protected calm water areas—both of which can be achieved with offshore breakwater or island systems. Supervisor St. John closed by indicating he was very pleased with and enthusiastic about the work carried out to date. However, he cautioned that currently tax monies for public improvement projects was very difficult to obtain. Thus, he again noted the importance of alternatives which had an economic value in order to make it possible to utilize sources of funding other than taxes.

In addition to the personal comments made at the public hearing, two written comments were received, one from the Village of Fox Point and one from the Great Lakes Coalition. Copies of this correspondence and the responses thereto are included in Appendix E. Mr. F. R. Dingle, the Fox Point Village President, stated in his letter that the Fox Point Village Board opposed any new taxing entity to pay for shoreline projects. He also stated the Village's preference that lakeshore property owners work either individually or in voluntary groups to address their erosion problems and finance the projects. As noted in the Commission staff response to the Village also enclosed in Appendix E, the preliminary plan recommendations are fully consistent with the concerns raised by the Village of Fox Point in that no new taxing entity is proposed in the plan and in that the projects recommended in Fox Point provided for maintenance of existing structures which could be done individually or cooperatively by the residents, with no proposals being advanced for a countywide project to protect private properties.

The written comments from the Great Lakes Coalition noted that the water levels of Lake Superior and Lake Ontario have been regulated for many years; that the U. S. Army Corps of Engineers has stated that Lakes Michigan, Huron, and Erie can also be regulated; and that the International Joint Commission is currently studying the possibility of such lake level regulation. The Coalition letter stated that if water levels were regulated to prevent severe fluctuations, structures would not need to be designed to protect against high lake levels, and, therefore, the recommended plan cost could be substantially reduced. The Coalition requested the Committee to encourage the Wisconsin Governor and the federal representatives to urge the International Joint Commission to complete its lake level reference studies promptly, with a recommendation for the construction of measures to regulate the levels of Lakes Michigan and Huron.

Response to Public Hearing Comments

There were two issues raised at the public hearing which required further consideration by the Advisory Committee: 1) the proposal to remove a portion of the South Shore breakwater; and 2) the substitution of the regulation of levels on the Great Lakes for the protective measures proposed in the recommended plan.

Three of the four citizens who spoke at the hearing opposed the proposal to demolish the portion of the South Shore breakwater lying south of E. Oklahoma Avenue extended. No citizens spoke in favor of that recommendation. In addition, the City of St. Francis appeared in opposition to that proposal. The Intergovernmental Coordinating and Technical Advisory Committee accordingly reconsidered its initial plan recommendation in this respect.

The preliminary plan, which called for demolition of a portion of the breakwater, was recommended because the shoreline south of E. Oklahoma Avenue extended would receive better protection than under existing conditions; because the cost of protecting the shoreline could be substantially reduced since the costs would be partially offset by the use of the stone salvaged from the demolished breakwater; and because the recommended plan would provide a more usable shoreline.

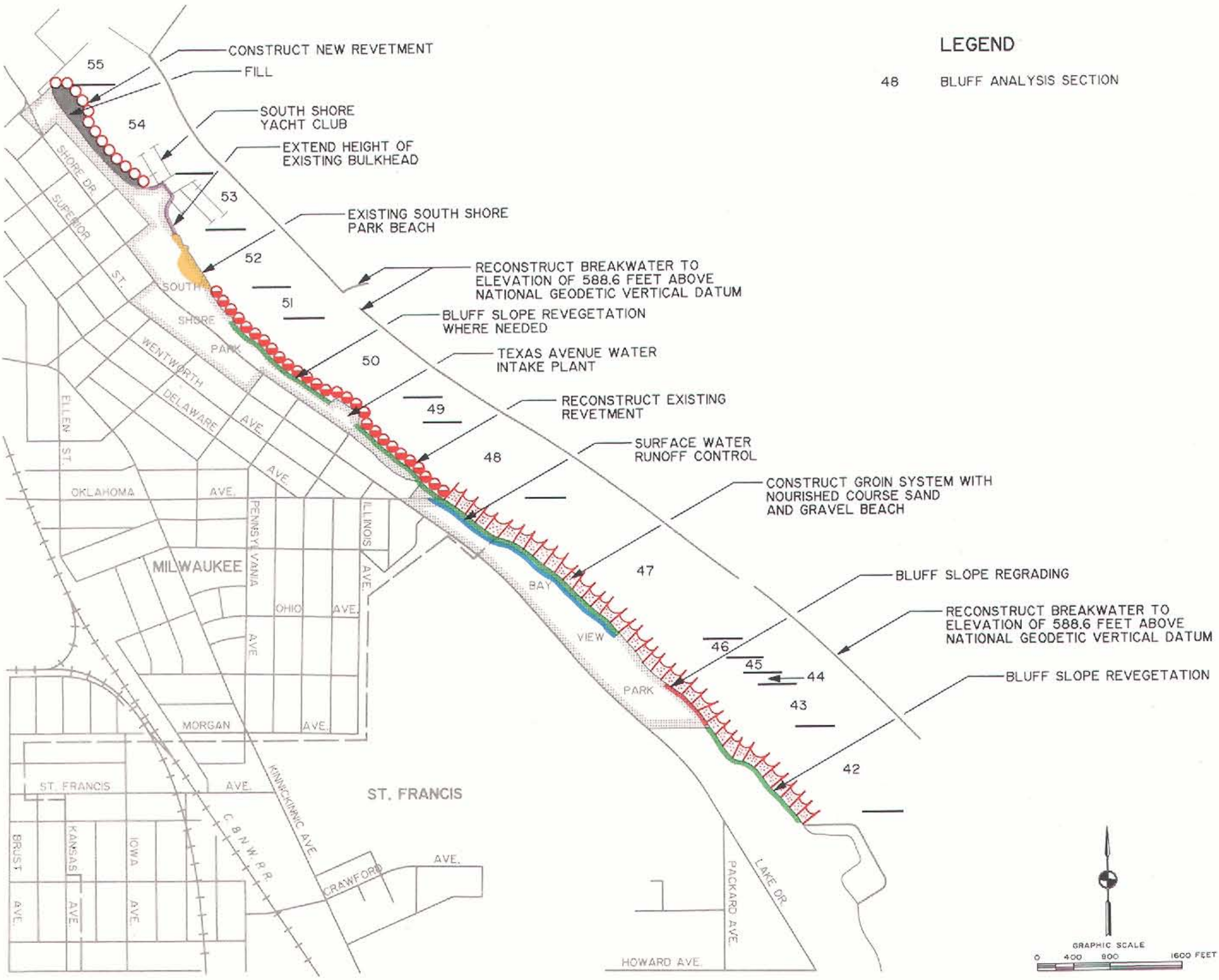
Alternative No. 1 for the South Shore breakwater, which calls for reconstructing the entire breakwater to an elevation of 588.6 feet NGVD, would also provide a high level of protection for the shoreline and provide a usable shoreline, if nourished gravel beaches were constructed along the shoreline south of E. Oklahoma Avenue extended, as proposed in the preliminary recommended plan. This alternative has the advantage of maintaining a relatively large area of protected water for boating and other uses. The only disadvantage of Alternative No. 1, compared to the preliminary recommended alternative, is the higher cost.

Because opposition to the preliminary recommendations for the South Shore breakwater area—which called for demolition of the breakwater south of E. Oklahoma Avenue—was expressed at the hearing by both a municipal corporation and concerned private citizens, and because no public body or private citizen appeared to express support for demolishing the portion of the breakwater concerned, the Committee concluded that the plan should be changed to recommend that the entire South Shore breakwater be maintained and reconstructed to an elevation of 588.6 feet NGVD. This recommendation increases the capital cost of the plan by approximately $5.7 million, and the total annual maintenance cost of the plan by approximately $112,000. The final recommendations for the South Shore breakwater area are shown on Map 59, and the final plan costs for Milwaukee County are summarized in Table 73. The final recommended plan as shown on Map 60, would entail a capital cost of approximately $88.1 million, an annual maintenance cost of about $4.3 million, a 50-year present worth of about $149.2 million, and an equivalent annual cost of about $9.5 million. Of the final plan equivalent annual cost, about $7.2 million, or 76 percent, would be funded by the public sector and the remaining $2.3 million, or 24 percent, would be funded by the private sector.

The Advisory Committee also considered the Great Lakes Coalition proposal to encourage further regulation of Great Lakes water levels in lieu of the construction of protective facilities. In this regard, the Advisory Committee concluded that the plan should not be changed because of the uncertainties related to the construction of the massive and costly facilities which would be needed to regulate Lakes Michigan and Huron and because of the complex institutional structure which would be required in an international setting to fairly carry out such regulation. Rather, the Committee concluded that the responsible action would be to provide a plan of action for resolving the problem at the local level. It clearly appears that there is great uncertainty concerning any attainment of regulation of Lakes Michigan and Huron, as indicated by the

Map 59

FINAL RECOMMENDED PLAN FOR THE SOUTH SHORE BREAKWATER AREA



Source: SEWRPC.

project team for the International Joint Commission in its Phase 1 report which was published in July 1989. Quoting from that report:[2]

> . . .there seems no reason to modify the conclusions presented in previous studies in regard to the likelihood of full regulation being implemented. The current understanding of the technical merit, socio-economic rationale and government policy support for full regulation all make the implementation of such a proposal unlikely in the foreseeable future. The conclusion, that full regulation is not the preferred course of action at this time, does not arise because of lack of knowledge or investigation, but because of the realities of the present economic and political situation.

Thus, no changes in the plan were proposed in this regard by the Committee.

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[2] *International Joint Commission Project Management Team, Living With the Lakes: Challenges and Opportunities, July 1989.*

Table 73

ESTIMATED COST OF FINAL RECOMMENDED SHORELINE EROSION MANAGEMENT PLAN FOR MILWAUKEE COUNTY

Civil Division	Bluff Analysis Section	Shoreline Length (feet)	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual Cost
City of Oak Creek	1	4,470	$ 3,576,000	$ 67,000	$ 4,633,000	$ 294,000
	2	2,820	- -	- -	- -	- -
	3	2,930	1,612,000	88,000	2,423,000	153,000
	4	1,980	8,217,000	89,000	9,818,000	621,000
	5	1,070	4,441,000	48,000	4,988,000	317,000
	6	1,170	644,000	36,000	968,000	61,000
	7	1,000	560,000	30,000	836,000	53,000
	8	540	81,000	8,000	209,000	13,000
	9	570	312,000	18,000	473,000	30,000
	10	400	220,000	12,000	331,000	21,000
	11	1,290	710,000	38,000	1,067,000	68,000
	12	3,160	2,370,000	48,000	3,117,000	198,000
	13	1,320	- -	- -	- -	- -
City of South Milwaukee	14	1,310	590,000	33,000	849,000	54,000
	15	790	366,000	20,000	522,000	33,000
	16	470	141,000	5,000	215,000	14,000
	17	440	198,000	11,000	285,000	18,000
	18	220	144,000	6,000	205,000	13,000
		1,660	912,000	50,000	1,371,000	87,000
	19	700	140,000	7,000	250,000	16,000
		2,480	0	12,000	189,000	12,000
		- -	145,000	5,000	224,000	14,000
	20	1,280	704,000	39,000	1,313,000	83,000
	21	1,060	583,000	32,000	1,090,000	69,000
	22	950	523,000	29,000	981,000	62,000
	23	1,200	660,000	36,000	1,227,000	78,000
	24	1,910	1,051,000	57,000	1,952,000	124,000
	25	880	484,000	27,000	903,000	58,000
City of Cudahy	26	660	429,000	20,000	612,000	39,000
	27	1,850	1,017,000	55,000	1,884,000	120,000
	28	2,050	1,137,000	61,000	2,098,000	133,000
	29	770	423,000	23,000	792,000	50,000
	30	1,760	968,000	53,000	1,805,000	115,000
	31	600	330,000	18,000	614,000	39,000
	32	340	51,000	5,000	131,000	8,000
	33	2,060	3,399,000	98,000	4,459,000	283,000
	34	1,780	2,670,000	53,000	3,505,000	222,000
	35	650	975,000	20,000	1,290,000	82,000
	36	710	1,065,000	21,000	1,396,000	89,000
	37	1,010	556,000	30,000	836,000	53,000
City of St. Francis	38	1,290	581,000	32,000	835,000	53,000
	39	1,480	666,000	37,000	958,000	61,000
	40	820	328,000	12,000	522,000	33,000
	41	1,650	4,125,000	50,000	4,905,000	311,000
	42	940	484,000	22,000	791,000	50,000
	43	1,370	1,715,000	83,000	2,747,000	174,000
	44	140	216,000	6,000	311,000	20,000
	45	80	124,000	4,000	187,000	12,000
	46	360	474,000	17,000	742,000	47,000
	47	2,470	3,661,000	123,000	5,505,000	349,000
City of Milwaukee	48	1,420	1,989,000	68,000	3,010,000	191,000
	49	340	400,000	15,000	636,000	40,000
	50	1,130	976,000	54,000	1,789,000	113,000
	51	570	478,000	26,000	888,000	56,000
	52	450	373,000	16,000	625,000	40,000
	53	1,320	735,000	59,000	1,665,000	106,000
	54	1,360	837,000	61,000	1,814,000	115,000
	55	9,600	0	432,000	6,809,000	432,000
		4,600	1,840,000	46,000	2,565,000	163,000
		3,400	2,010,000	34,000	1,556,000	99,000
		5,650	2,825,000	56,000	3,708,000	235,000
		1,100	0	11,000	173,000	11,000

Table 73 (continued)

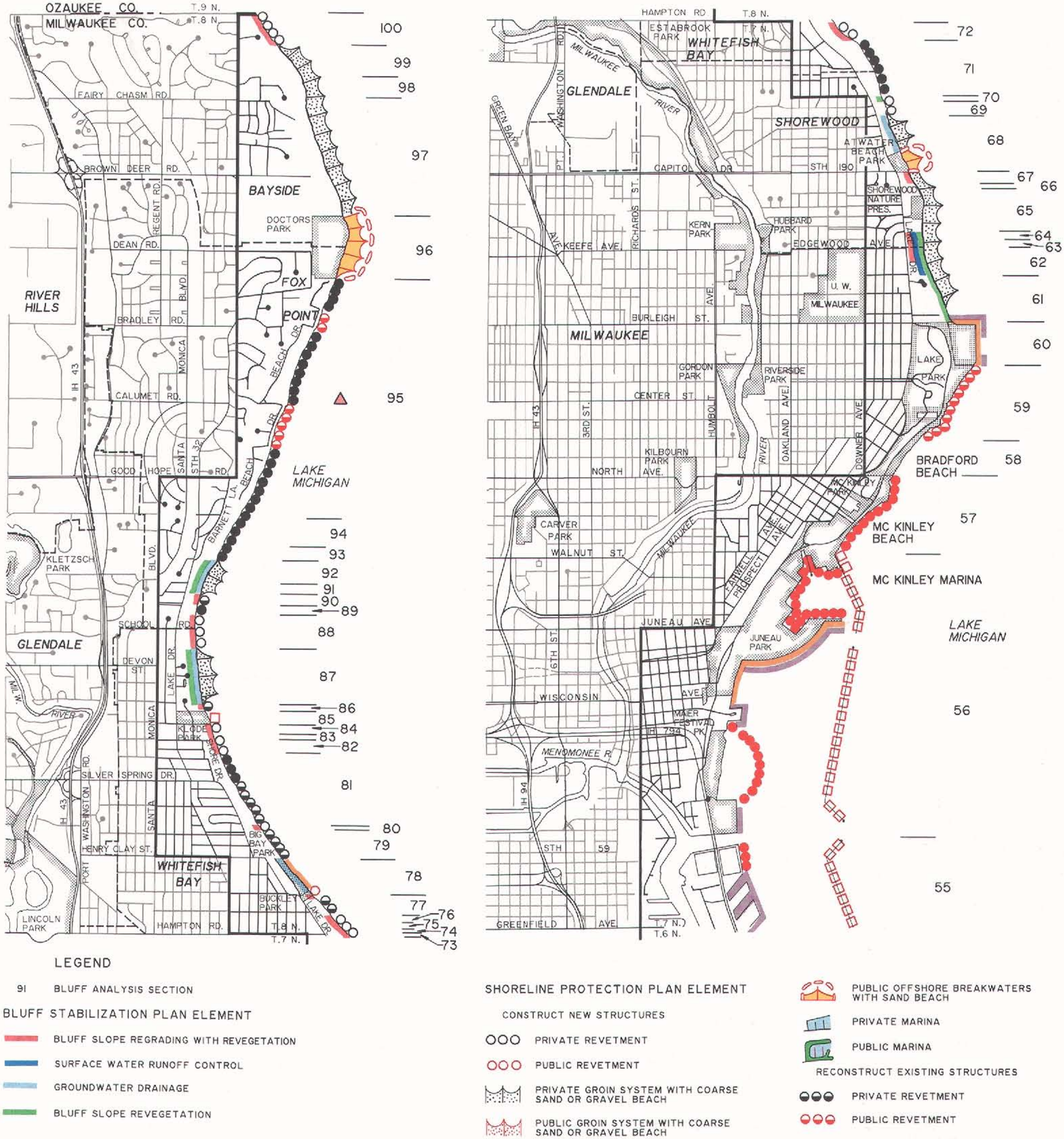
Civil Division	Bluff Analysis Section	Shoreline Length (feet)	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual Cost
City of Milwaukee (continued)	56	9,500	$ 0	$ 428,000	$ 6,738,000	$ 428,000
		1,900	380,000	19,000	679,000	43,000
		2,900	0	29,000	457,000	29,000
		1,400	280,000	14,000	501,000	32,000
		1,700	340,000	17,000	608,000	39,000
		3,900	2,340,000	39,000	2,955,000	187,000
		4,260	0	43,000	678,000	43,000
	57	3,210	0	96,000	1,518,000	96,000
	58	1,900	- -	- -	- -	- -
	59	3,540	1,062,000	53,000	1,899,000	121,000
	60	2,210	1,105,000	33,000	1,628,000	103,000
	61	880	- -	- -	- -	- -
		1,090	456,000	26,000	810,000	52,000
	62	950	394,000	22,000	702,000	45,000
	63	300	150,000	9,000	253,000	16,000
Village of Shorewood	64	290	139,000	8,000	238,000	15,000
	65	1,710	855,000	34,000	1,391,000	88,000
	66	170	68,000	3,000	122,000	8,000
	67	380	209,000	14,000	344,000	22,000
	68	790	1,185,000	24,000	1,563,000	99,000
		1,380	621,000	42,000	1,274,000	81,000
	69	520	156,000	5,000	238,000	15,000
	70	240	77,000	3,000	116,000	8,000
Village of Whitefish Bay	71	2,370	0	24,000	374,000	24,000
	72	850	383,000	22,000	551,000	35,000
	73	190	86,000	5,000	123,000	8,000
	74	160	72,000	4,000	103,000	7,000
	75	310	140,000	8,000	201,000	13,000
	76	360	162,000	9,000	233,000	14,000
	77	810	162,000	8,000	290,000	18,000
	78	600	330,000	18,000	496,000	31,000
		1,060	265,000	27,000	683,000	43,000
	79	1,480	296,000	15,000	529,000	34,000
	80	130	52,000	2,000	76,000	5,000
	81	1,700	340,000	17,000	608,000	39,000
		1,270	0	13,000	200,000	13,000
	82	490	221,000	12,000	317,000	21,000
	83	140	63,000	3,000	90,000	6,000
	84	430	194,000	11,000	279,000	18,000
	85	480	0	14,000	227,000	14,000
	86	170	26,000	5,000	59,000	4,000
	87	1,950	897,000	62,000	1,830,000	117,000
Village of Fox Point	88	1,150	518,000	29,000	745,000	47,000
	89	320	0	3,000	50,000	3,000
	90	470	165,000	12,000	258,000	17,000
	91	510	237,000	16,000	481,000	30,000
	92	770	370,000	27,000	746,000	47,000
	93	530	0	5,000	84,000	5,000
	94	1,460	0	15,000	230,000	15,000
	95A	2,390	0	24,000	377,000	24,000
	95B	1,600	640,000	24,000	1,018,000	65,000
	95C	3,000	0	30,000	473,000	30,000
	95D	720	288,000	11,000	458,000	29,000
	95E	1,360	0	14,000	214,000	14,000
	95	9,070	0	5,000	79,000	5,000
Village of Bayside	96	1,890	2,835,000	57,000	3,733,000	237,000
	97	4,660	1,864,000	93,000	3,333,000	211,000
	98	860	344,000	17,000	615,000	39,000
	99	1,280	512,000	26,000	916,000	58,000
	100	1,320	594,000	33,000	855,000	54,000
Total	- -	159,110	$88,124,000	$4,253,000[a]	$149,243,000	$9,472,000

[a]*About $485,000, or 11 percent of the annual maintenance cost, would be required only for the first three years following bluff slope regrading or revegetation.*

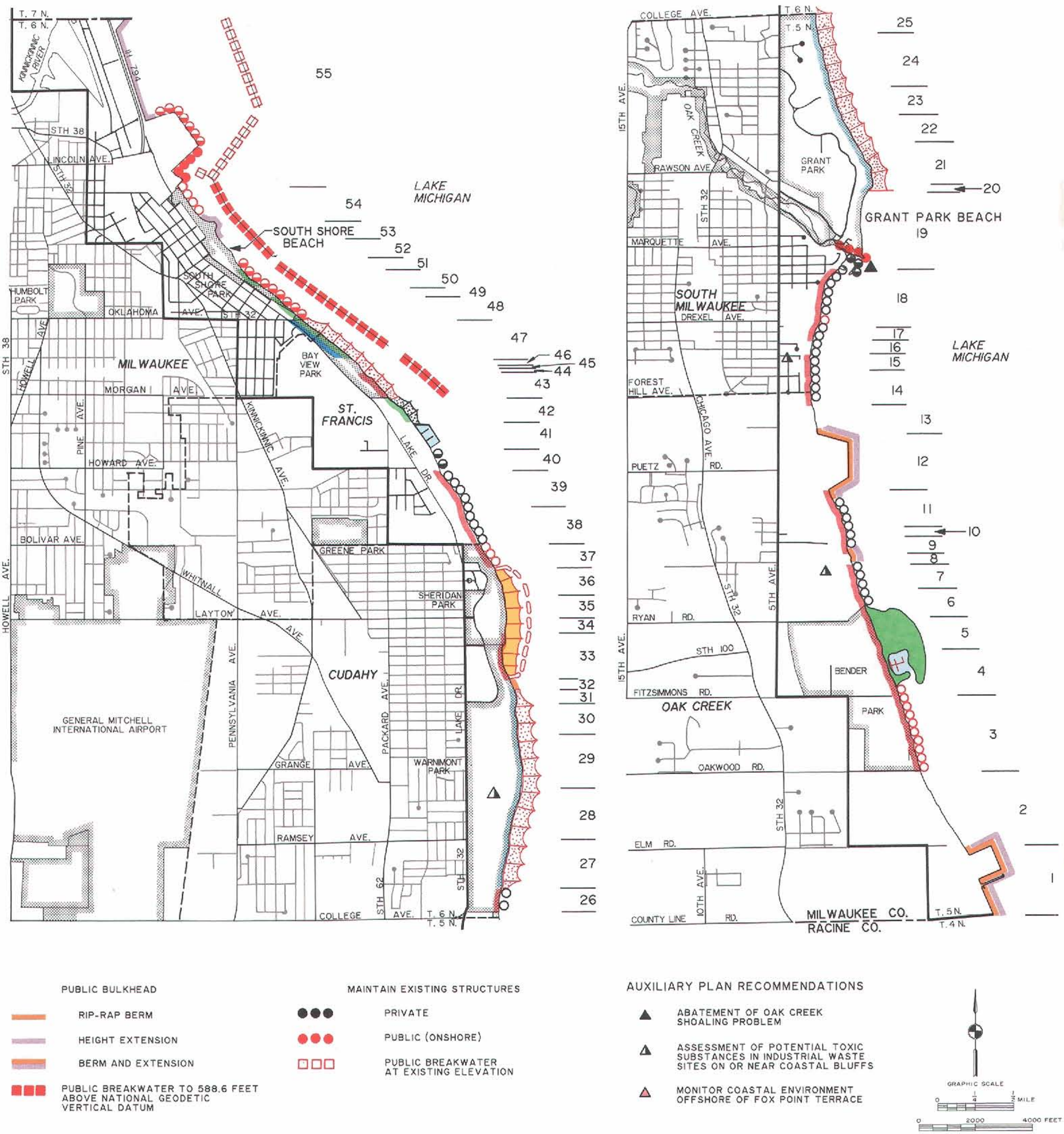
Source: SEWRPC.

Map 60

FINAL RECOMMENDED LAKE MICHIGAN SHORELINE EROSION MANAGEMENT PLAN FOR MILWAUKEE COUNTY



Map 60 (continued)



Source: SEWRPC.

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APPENDICES

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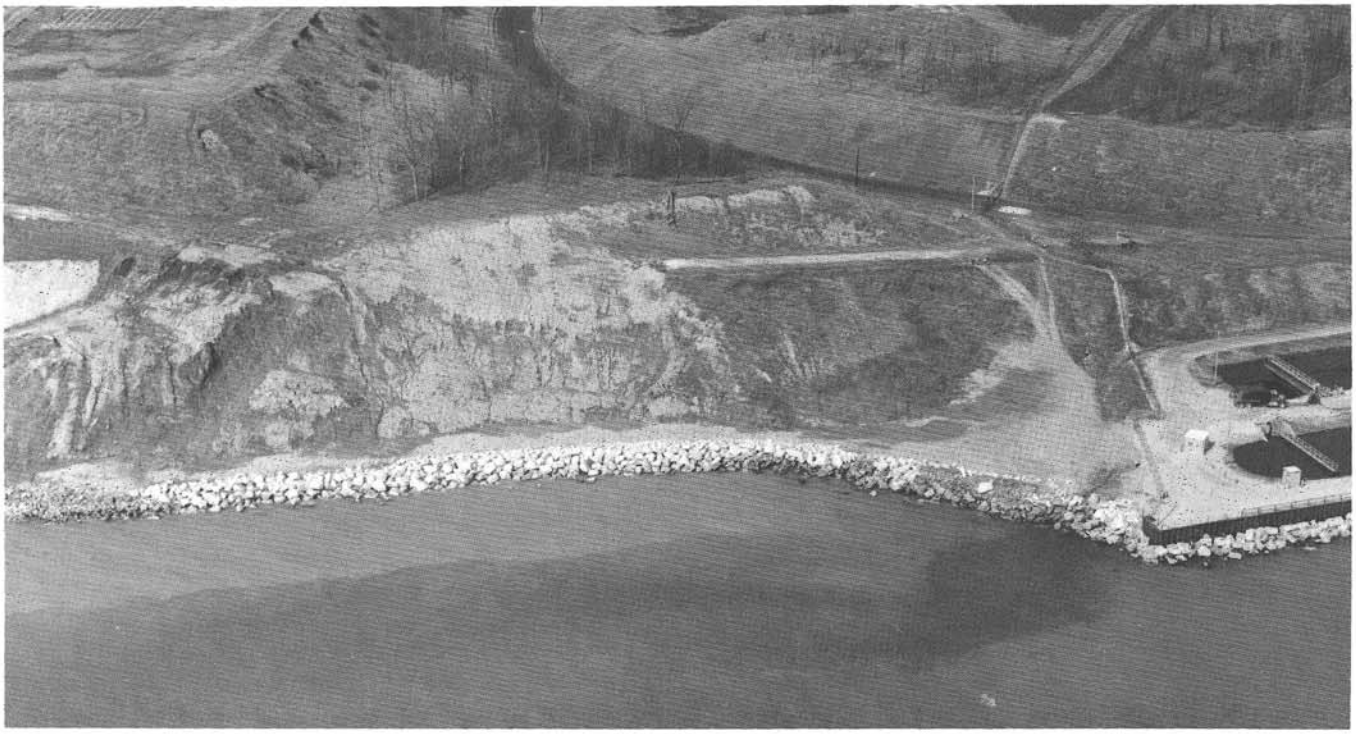
Appendix A

AERIAL PHOTOGRAPHS OF THE LAKE MICHIGAN SHORELINE OF MILWAUKEE COUNTY: APRIL 1987[a]

Figure A-1

SOUTH END OF WISCONSIN ELECTRIC POWER COMPANY OAK CREEK POWER PLANT, CITY OF OAK CREEK



Source: SEWRPC.

Figure A-2

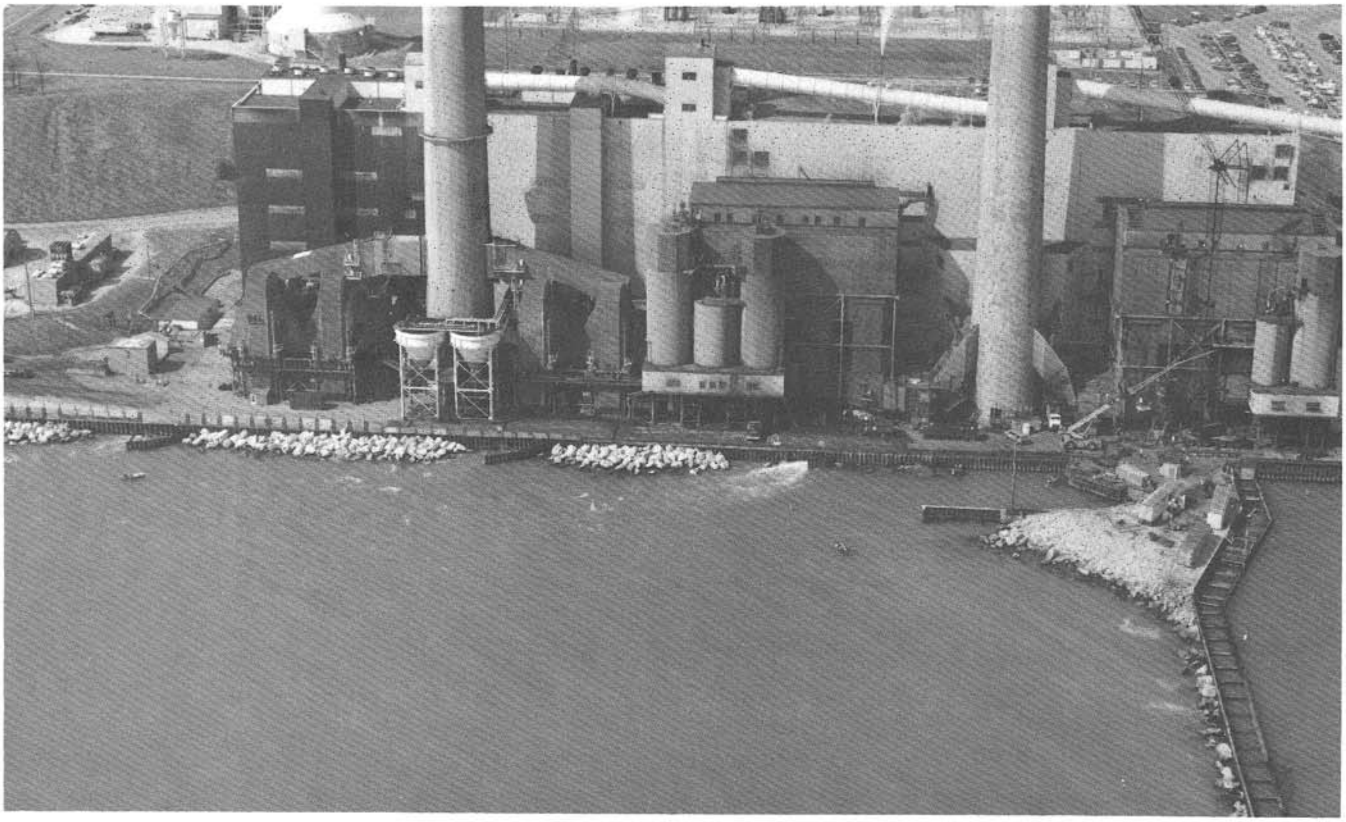
WISCONSIN ELECTRIC POWER COMPANY OAK CREEK POWER PLANT, CITY OF OAK CREEK

Source: SEWRPC.

[a]*All photographs were taken in April 1987, unless noted otherwise.*

Figure A-3

WISCONSIN ELECTRIC POWER COMPANY OAK CREEK POWER PLANT, CITY OF OAK CREEK



Source: SEWRPC.

Figure A-4

WISCONSIN ELECTRIC POWER COMPANY OAK CREEK POWER PLANT, CITY OF OAK CREEK

Source: SEWRPC.

Figure A-5

JUST NORTH OF WISCONSIN ELECTRIC POWER COMPANY OAK CREEK POWER PLANT, CITY OF OAK CREEK

Source: SEWRPC.

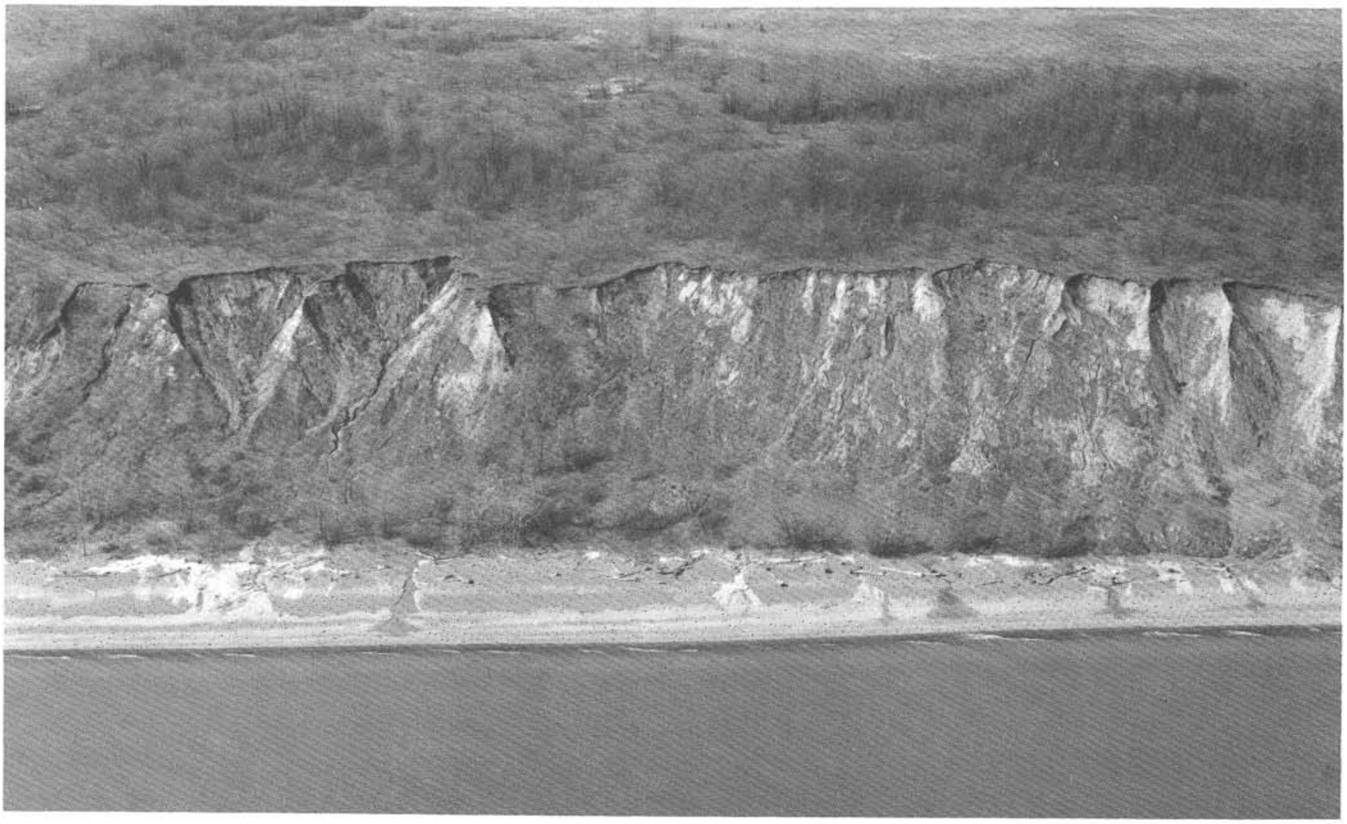
Figure A-6

ELM ROAD, CITY OF OAK CREEK

Source: SEWRPC.

Figure A-7

BETWEEN ELM ROAD AND OAKWOOD ROAD, CITY OF OAK CREEK



Source: SEWRPC.

Figure A-8

OAKWOOD ROAD, CITY OF OAK CREEK

Source: SEWRPC.

Figure A-9

BETWEEN OAKWOOD ROAD AND FITZSIMMONS ROAD, CITY OF OAK CREEK

Source: SEWRPC.

Figure A-10

FITZSIMMONS ROAD, CITY OF OAK CREEK

Source: SEWRPC.

Figure A-11

SOUTHERN END OF BENDER PARK, CITY OF OAK CREEK

Source: SEWRPC.

Figure A-12

BENDER PARK, CITY OF OAK CREEK

Source: SEWRPC.

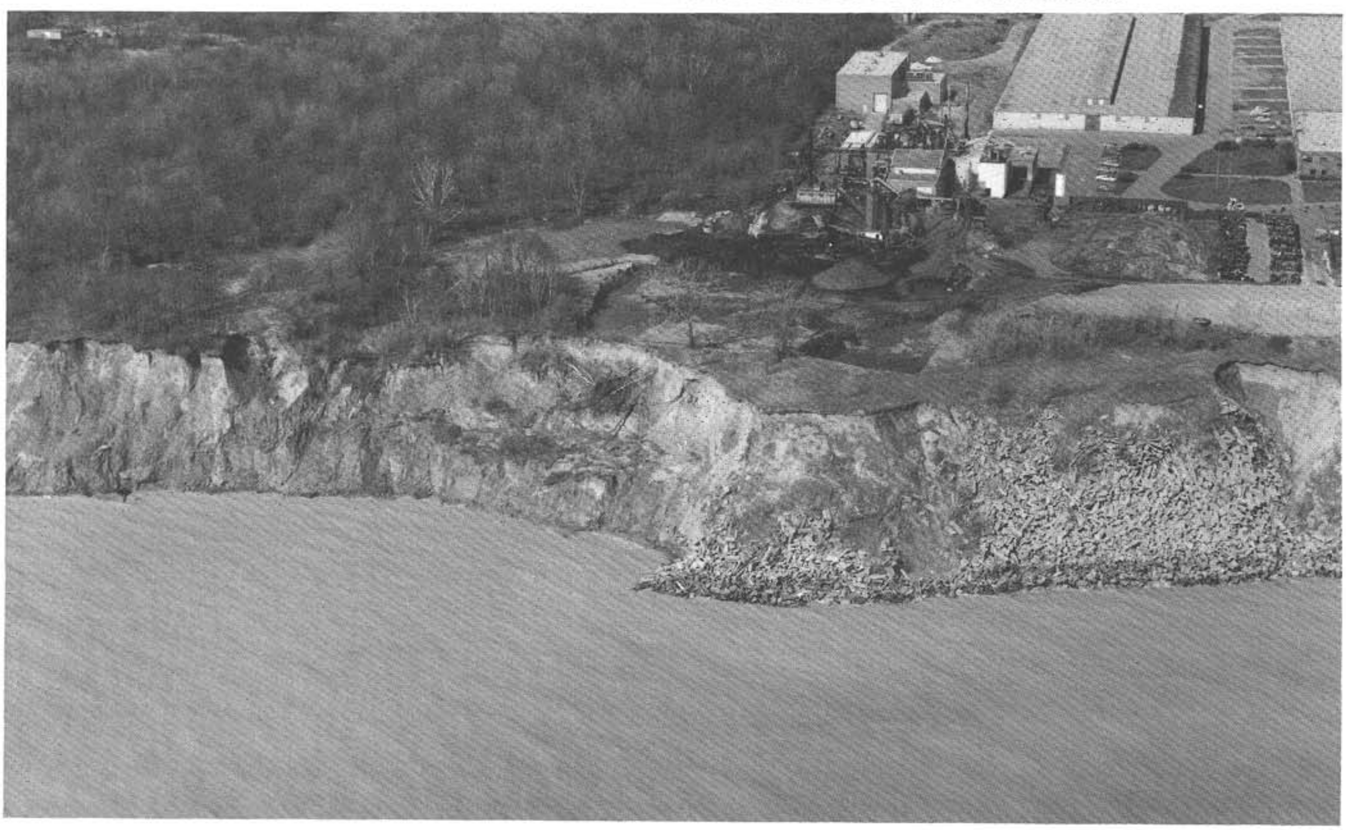
Figure A-13

NORTH OF RYAN ROAD, CITY OF OAK CREEK

Source: SEWRPC.

Figure A-14

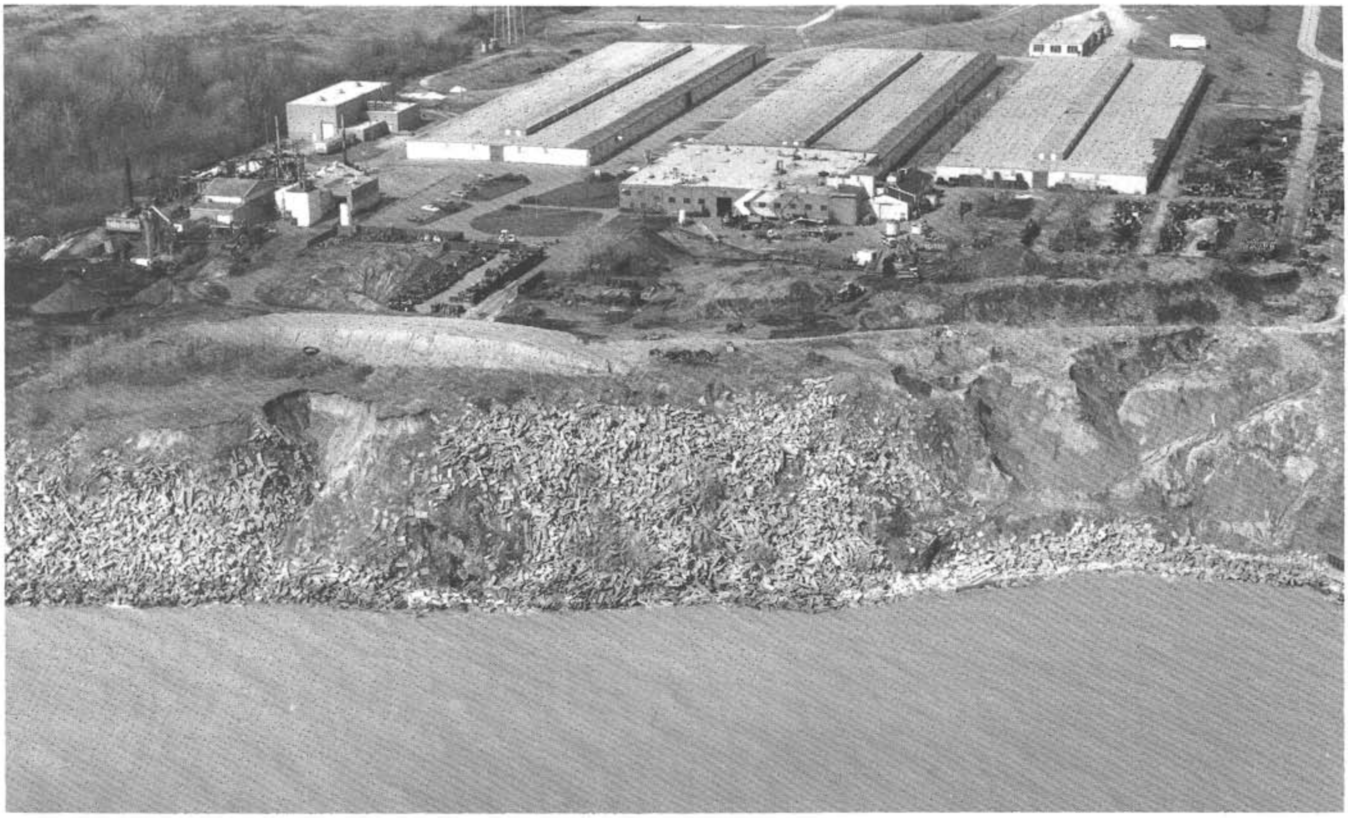
BETWEEN RYAN ROAD AND AMERICAN AVENUE, CITY OF OAK CREEK



Source: SEWRPC.

Figure A-15

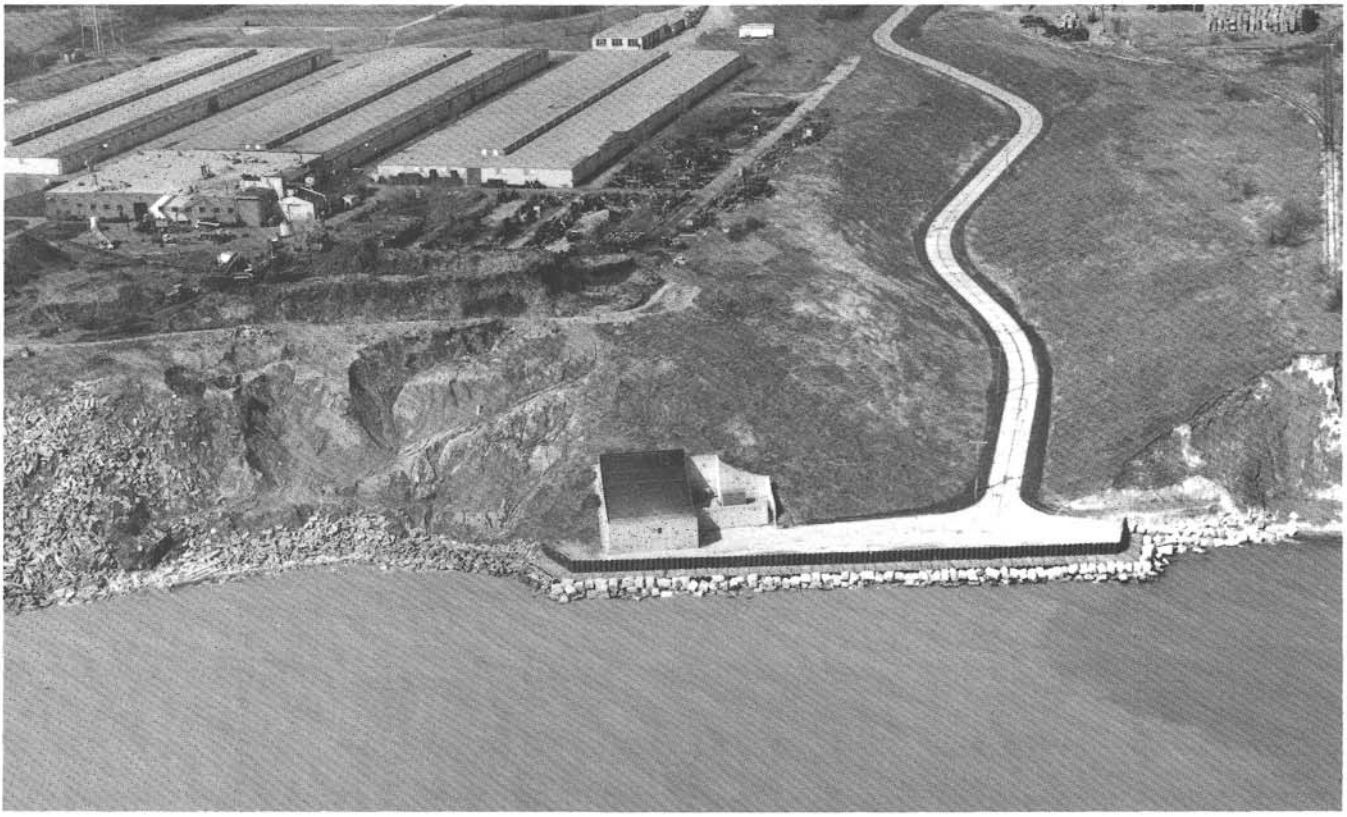
JUST SOUTH OF DEPOT ROAD, CITY OF OAK CREEK



Source: SEWRPC.

Figure A-16

OAK CREEK WATER INTAKE PLANT, CITY OF OAK CREEK



Source: SEWRPC.

Figure A-17

BETWEEN DEPOT ROAD AND LAKESIDE AVENUE, CITY OF OAK CREEK

Source: SEWRPC.

Figure A-18

BETWEEN LAKESIDE AVENUE AND PUETZ ROAD, CITY OF OAK CREEK

Source: SEWRPC.

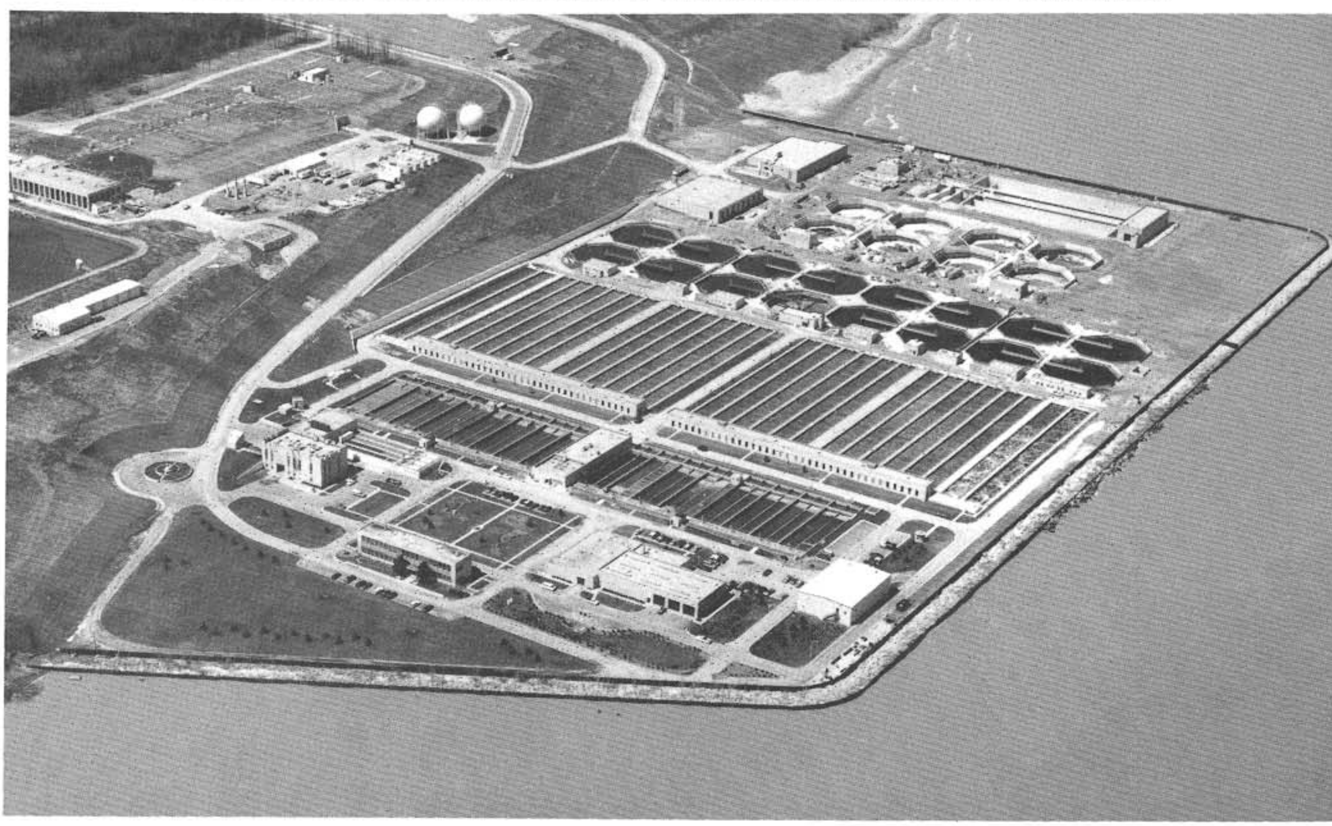
Figure A-19

SOUTHERN END OF MMSD SOUTH SHORE WASTEWATER TREATMENT PLANT, CITY OF OAK CREEK

Source: SEWRPC.

Figure A-20

MMSD SOUTH SHORE WASTEWATER TREATMENT PLANT, CITY OF OAK CREEK



Source: SEWRPC.

Figure A-21

JUST NORTH OF MMSD SOUTH SHORE WASTEWATER TREATMENT PLANT, CITY OF OAK CREEK

Source: SEWRPC.

Figure A-22

EDGEWOOD AVENUE, CITY OF SOUTH MILWAUKEE

Source: SEWRPC.

Figure A-23

BETWEEN EDGEWOOD AVENUE AND WILLIAMS AVENUE, CITY OF SOUTH MILWAUKEE

Source: SEWRPC.

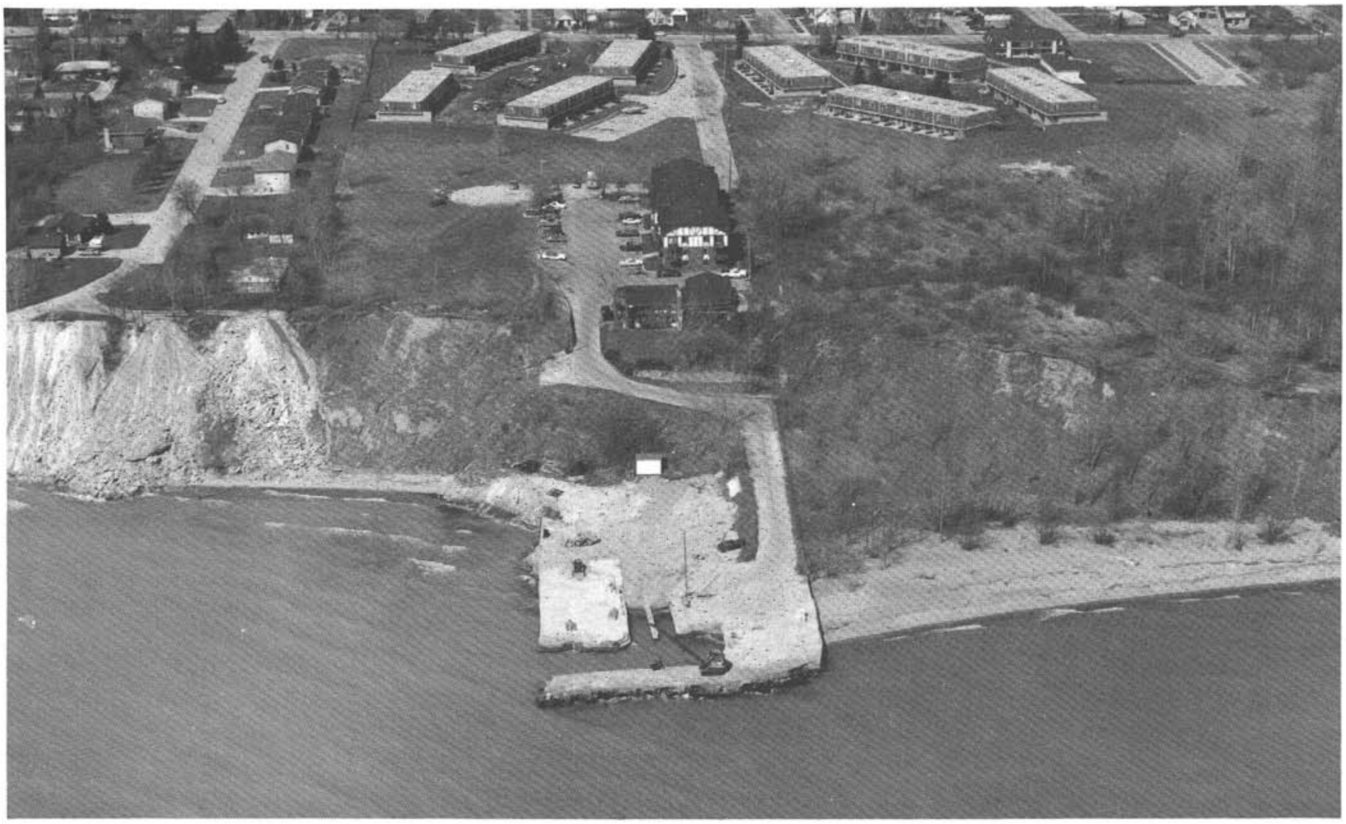
Figure A-24

LAKEVIEW AVENUE, CITY OF SOUTH MILWAUKEE

Source: SEWRPC.

Figure A-25

MARINA CLIFFS, CITY OF SOUTH MILWAUKEE



Source: SEWRPC.

Figure A-26

JUST SOUTH OF SOUTH MILWAUKEE WASTEWATER TREATMENT PLANT, CITY OF SOUTH MILWAUKEE

Source: SEWRPC.

Figure A-27

SOUTH MILWAUKEE WASTEWATER TREATMENT PLANT, CITY OF SOUTH MILWAUKEE

Source: SEWRPC.

Figure A-28

SOUTH OF MARION AVENUE, CITY OF SOUTH MILWAUKEE

Source: SEWRPC.

Figure A-29

SOUTH OF SOUTH MILWAUKEE YACHT CLUB, CITY OF SOUTH MILWAUKEE

Source: SEWRPC.

Figure A-30

SOUTH MILWAUKEE YACHT CLUB AND MOUTH OF OAK CREEK, CITY OF SOUTH MILWAUKEE

Source: SEWRPC.

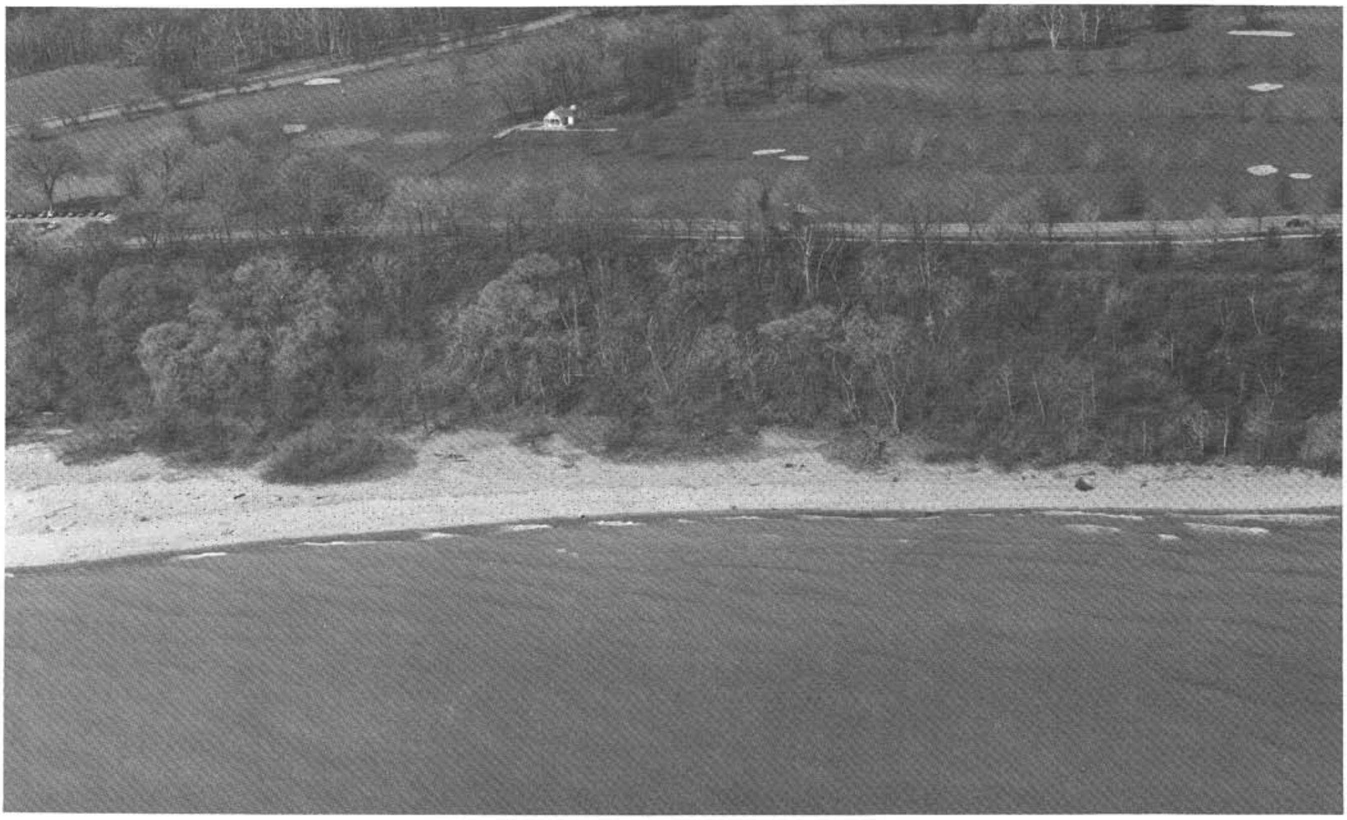
Figure A-31

GRANT PARK BEACH, CITY OF SOUTH MILWAUKEE

Source: SEWRPC.

Figure A-32

GRANT PARK, CITY OF SOUTH MILWAUKEE



Source: SEWRPC.

Figure A-33

GRANT PARK, CITY OF SOUTH MILWAUKEE

Source: SEWRPC.

Figure A-34

GRANT PARK, CITY OF SOUTH MILWAUKEE

Source: SEWRPC.

Figure A-35

S. LAKE DRIVE AT GRANT PARK, CITY OF SOUTH MILWAUKEE

Source: SEWRPC.

Figure A-36

GRANT PARK, CITY OF SOUTH MILWAUKEE

Source: SEWRPC.

Figure A-37

GRANT PARK, CITY OF SOUTH MILWAUKEE

Source: SEWRPC.

Figure A-38

COLLEGE AVENUE, CITY OF CUDAHY

Source: SEWRPC.

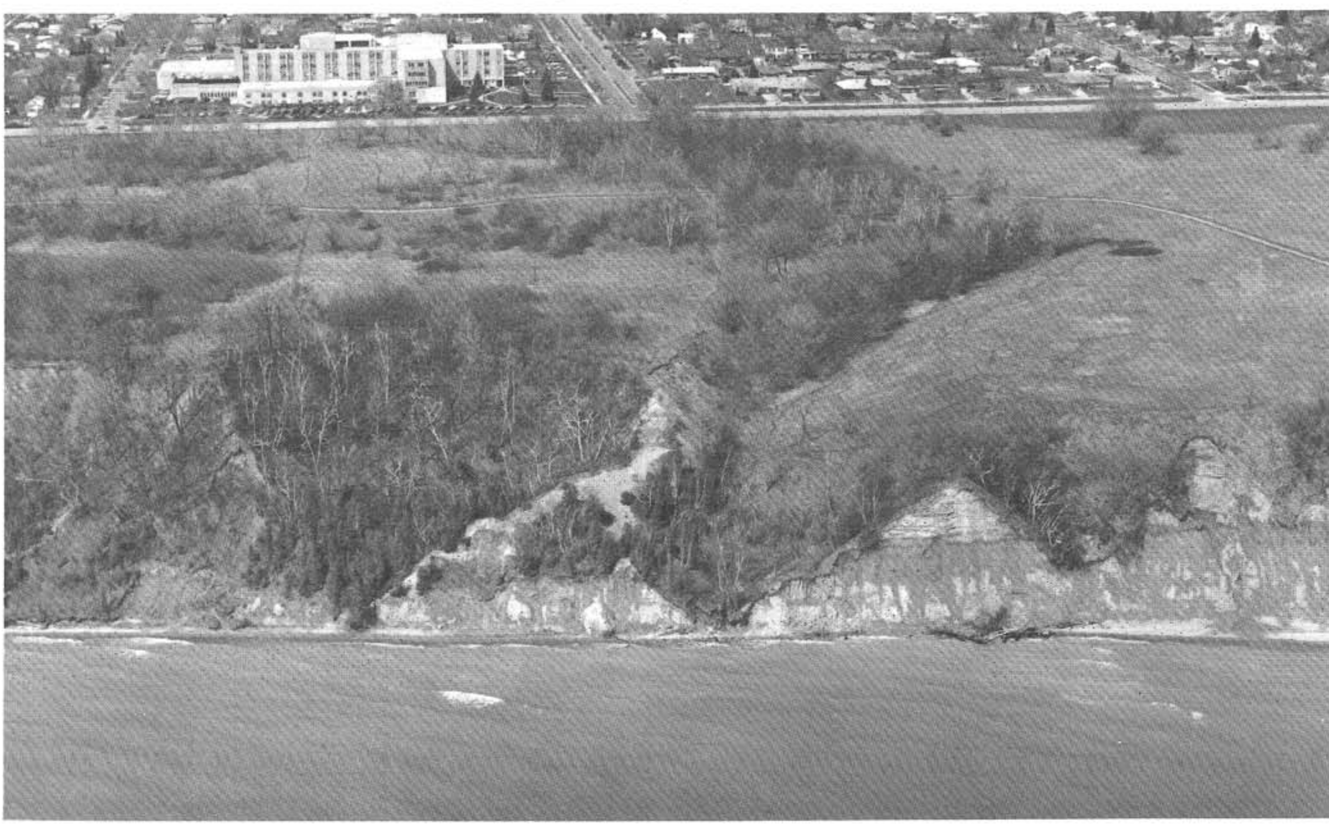
Figure A-39

KLIEFORTH AVENUE, CITY OF CUDAHY

Source: SEWRPC.

Figure A-40

WARNIMONT PARK, CITY OF CUDAHY



Source: SEWRPC.

Figure A-41

WARNIMONT PARK, CITY OF CUDAHY

Source: SEWRPC.

Figure A-42

WARNIMONT PARK, CITY OF CUDAHY

Source: SEWRPC.

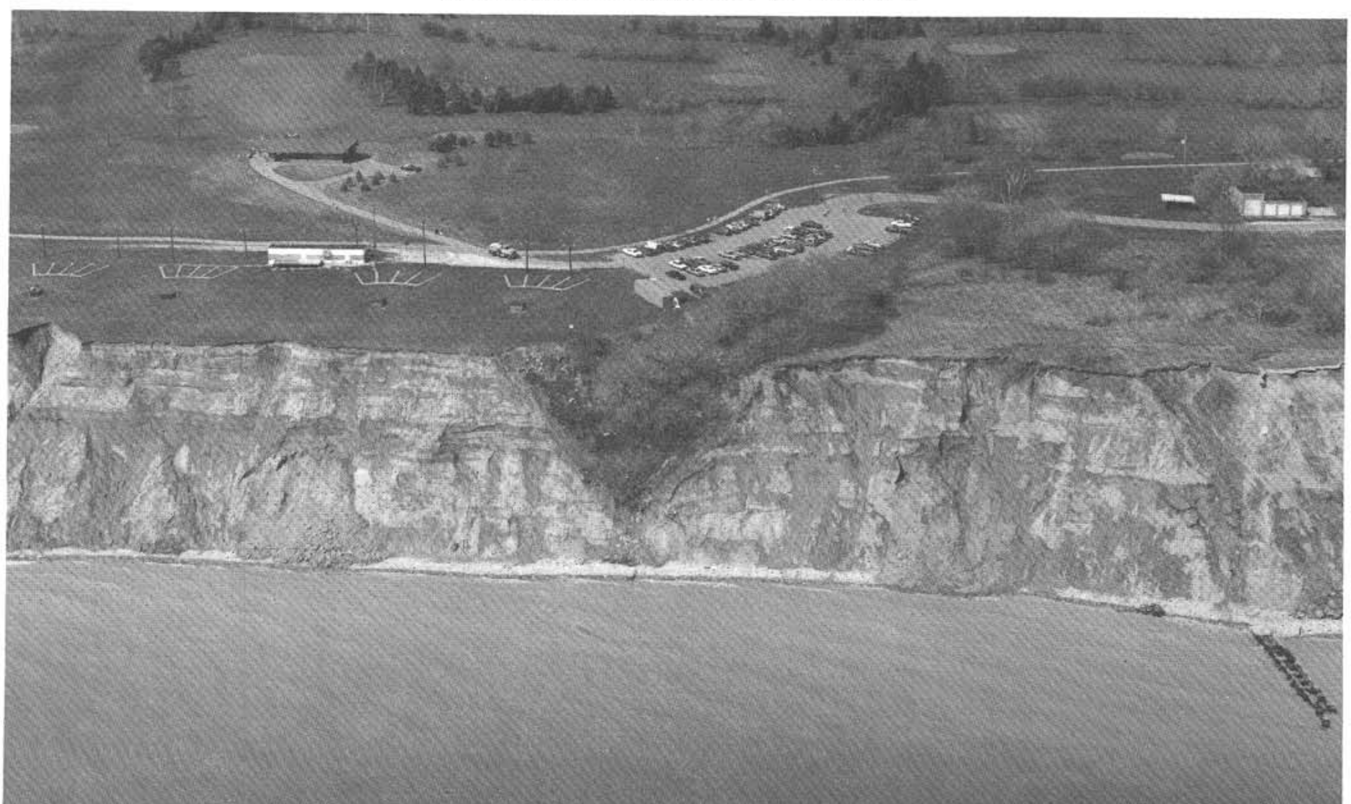
Figure A-43

WARNIMONT PARK, CITY OF CUDAHY

Source: SEWRPC.

Figure A-44

WARNIMONT PARK, CITY OF CUDAHY



Source: SEWRPC.

Figure A-45

CUDAHY WATER INTAKE PLANT, CITY OF CUDAHY

Source: SEWRPC.

Figure A-46

SHERIDAN PARK AND CUDAHY HIGH SCHOOL, CITY OF CUDAHY

Source: SEWRPC.

Figure A-47

SHERIDAN PARK, CITY OF CUDAHY

Source: SEWRPC.

Figure A-48

SHERIDAN PARK, ARMOUR AVENUE, CITY OF CUDAHY

Source: SEWRPC.

Figure A-49

SHERIDAN DRIVE, CITY OF CUDAHY

Source: SEWRPC.

Figure A-50

SHERIDAN DRIVE, CITY OF CUDAHY

Source: SEWRPC.

Figure A-51

FORMER WEPCO LAKESIDE POWER PLANT SITE, CITY OF ST. FRANCIS

Source: SEWRPC.

Figure A-52

FORMER WEPCO LAKESIDE POWER PLANT SITE, CITY OF ST. FRANCIS

Source: SEWRPC.

Figure A-53

FORMER WEPCO LAKESIDE POWER PLANT SITE, CITY OF ST. FRANCIS

Source: SEWRPC.

Figure A-54

JUST SOUTH OF FORMER WEPCO LAKESIDE POWER PLANT SITE, CITY OF ST. FRANCIS

Source: SEWRPC.

Figure A-55

FORMER WEPCO LAKESIDE POWER PLANT, CITY OF ST. FRANCIS

Source: SEWRPC.

Figure A-56

FORMER WEPCO LAKESIDE POWER PLANT, CITY OF ST. FRANCIS

Source: SEWRPC.

Figure A-57

FORMER WEPCO LAKESIDE POWER PLANT DIKE, CITY OF ST. FRANCIS

Source: SEWRPC.

Figure A-58

JUST NORTH OF FORMER WEPCO LAKESIDE POWER PLANT DIKE, CITY OF ST. FRANCIS

Source: SEWRPC.

Figure A-59

S. LAKE DRIVE AT PACKARD AVENUE, CITY OF ST. FRANCIS

Source: SEWRPC.

Figure A-60

S. LAKE DRIVE AT DESALLES SEMINARY, CITY OF ST. FRANCIS

Source: SEWRPC.

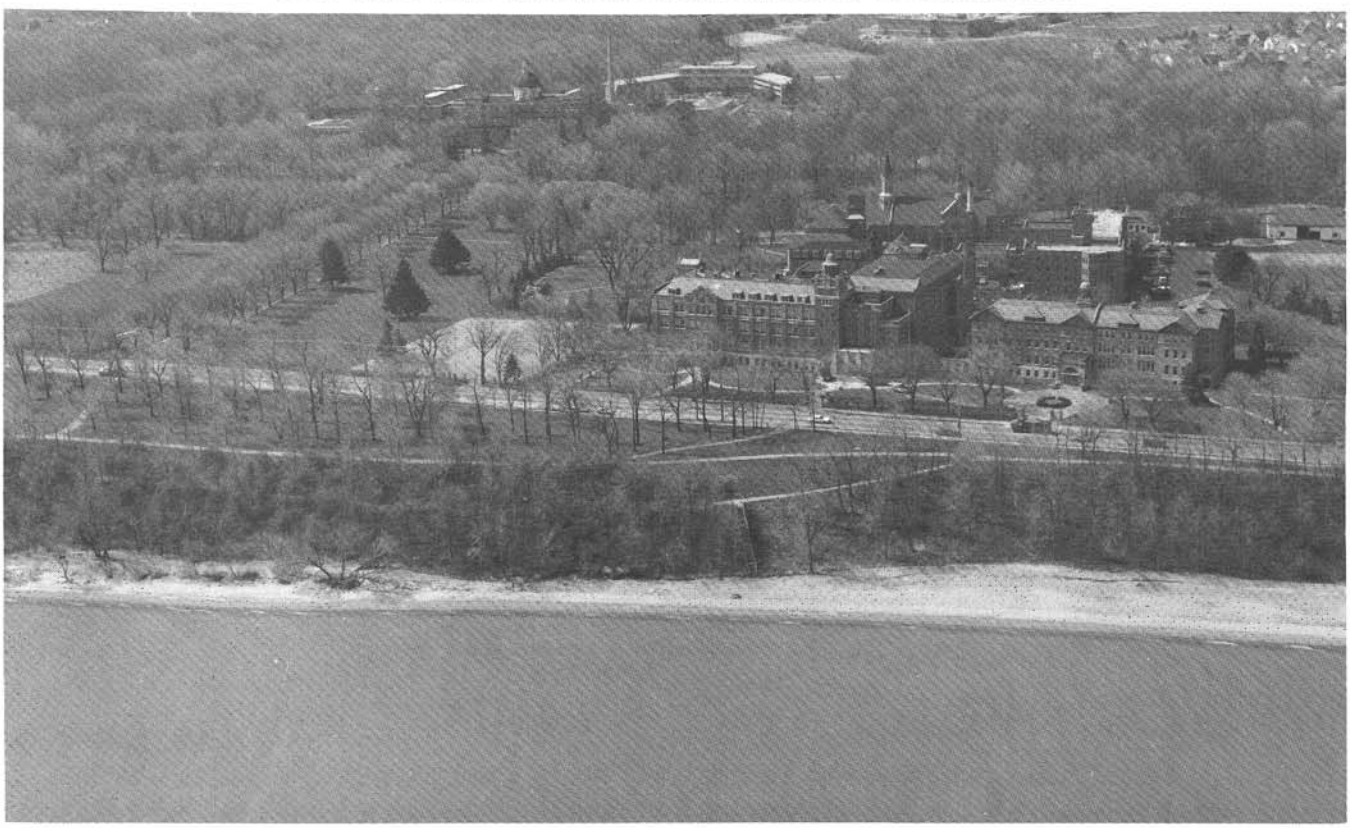
Figure A-61

BAY VIEW PARK, CITY OF ST. FRANCIS

Source: SEWRPC.

Figure A-62

BAY VIEW PARK, ST. MARY'S ACADEMY, CITY OF ST. FRANCIS



Source: SEWRPC.

Figure A-63

BAY VIEW PARK AT E. OKLAHOMA AVENUE, CITY OF MILWAUKEE

Source: SEWRPC.

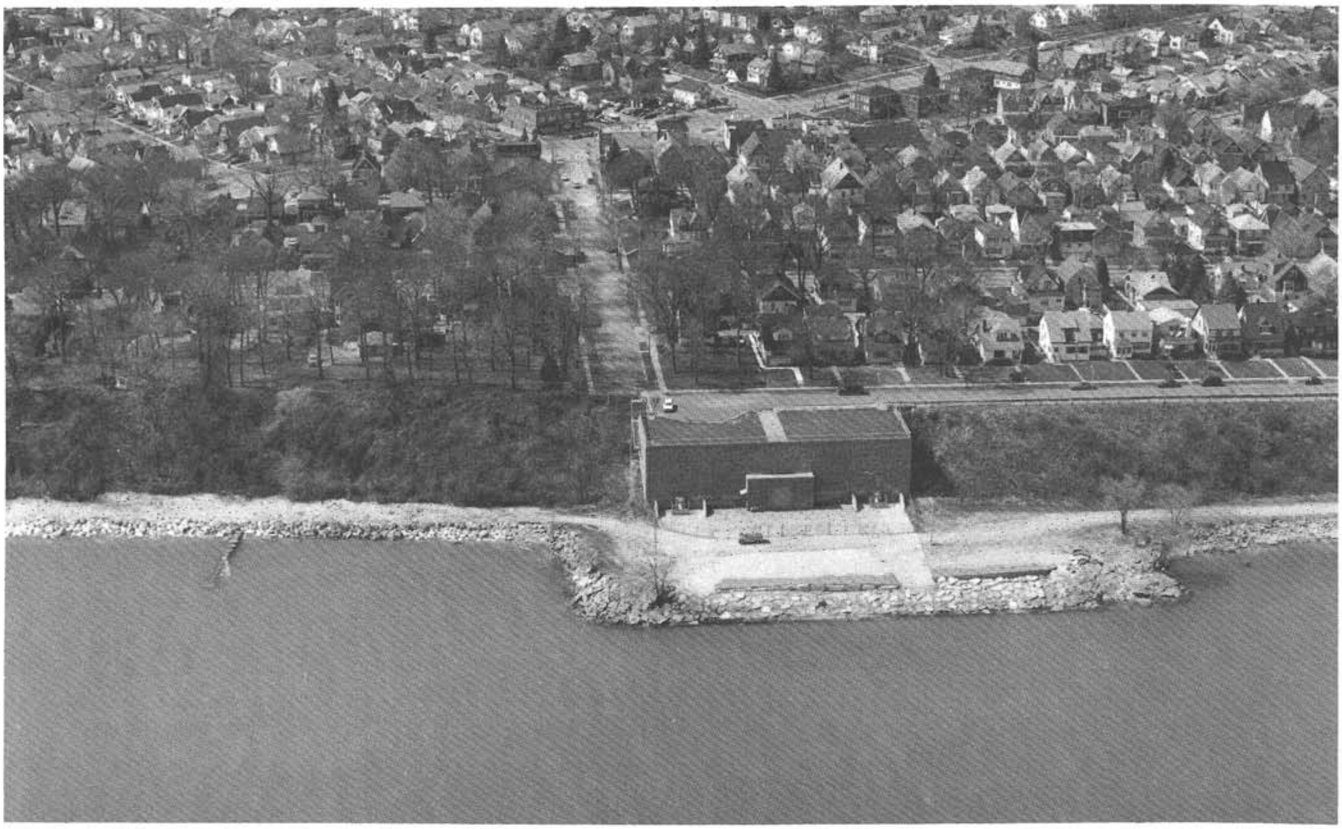
Figure A-64

SOUTH SHORE PARK, SUPERIOR STREET, CITY OF MILWAUKEE

Source: SEWRPC.

Figure A-65

TEXAS STREET WATER INTAKE PLANT, CITY OF MILWAUKEE



Source: SEWRPC.

Figure A-66

SOUTH SHORE PARK, PENNSYLVANIA AVENUE TO MEREDITH STREET, CITY OF MILWAUKEE

Source: SEWRPC.

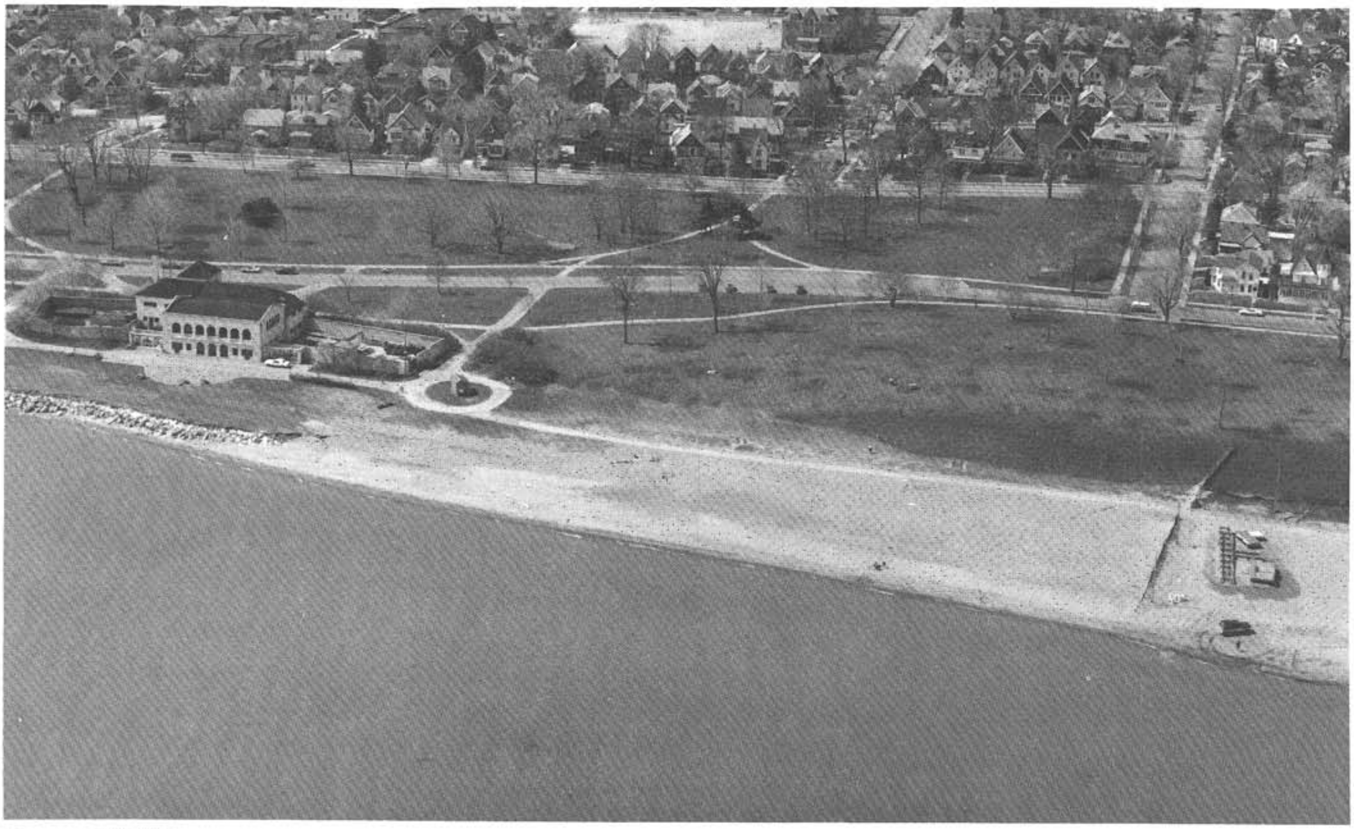
Figure A-67

SOUTH SHORE PARK PAVILION, CITY OF MILWAUKEE

Source: SEWRPC.

Figure A-68

SOUTH SHORE PARK BEACH, CITY OF MILWAUKEE



Source: SEWRPC.

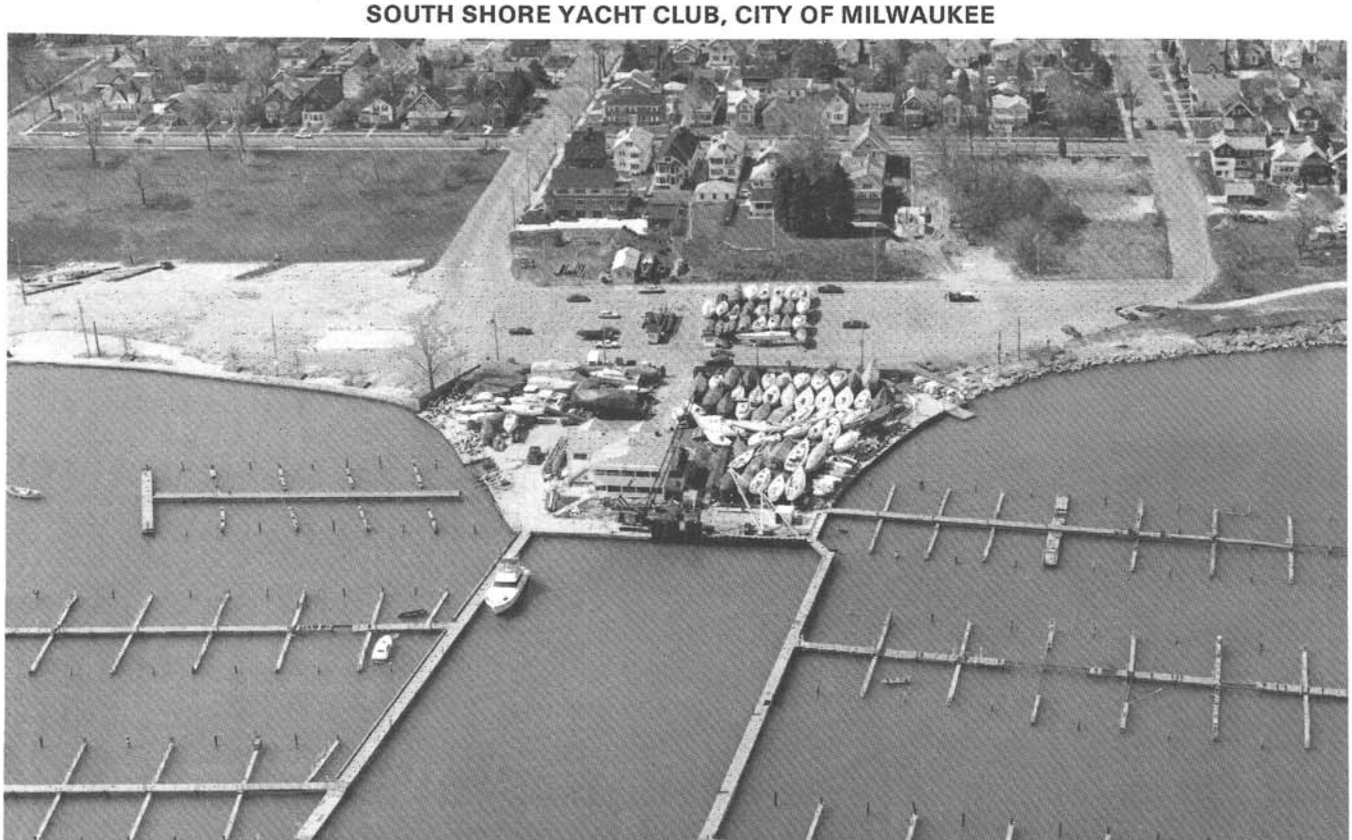
Figure A-69

SOUTH SHORE YACHT CLUB, CITY OF MILWAUKEE

Source: SEWRPC.

Figure A-70

SOUTH SHORE YACHT CLUB, CITY OF MILWAUKEE



Source: SEWRPC.

Figure A-71

E. IRON STREET TO PRYOR AVENUE, CITY OF MILWAUKEE

Source: SEWRPC.

Figure A-72

U. S. COAST GUARD STATION, OUTER HARBOR, CITY OF MILWAUKEE

Source: SEWRPC.

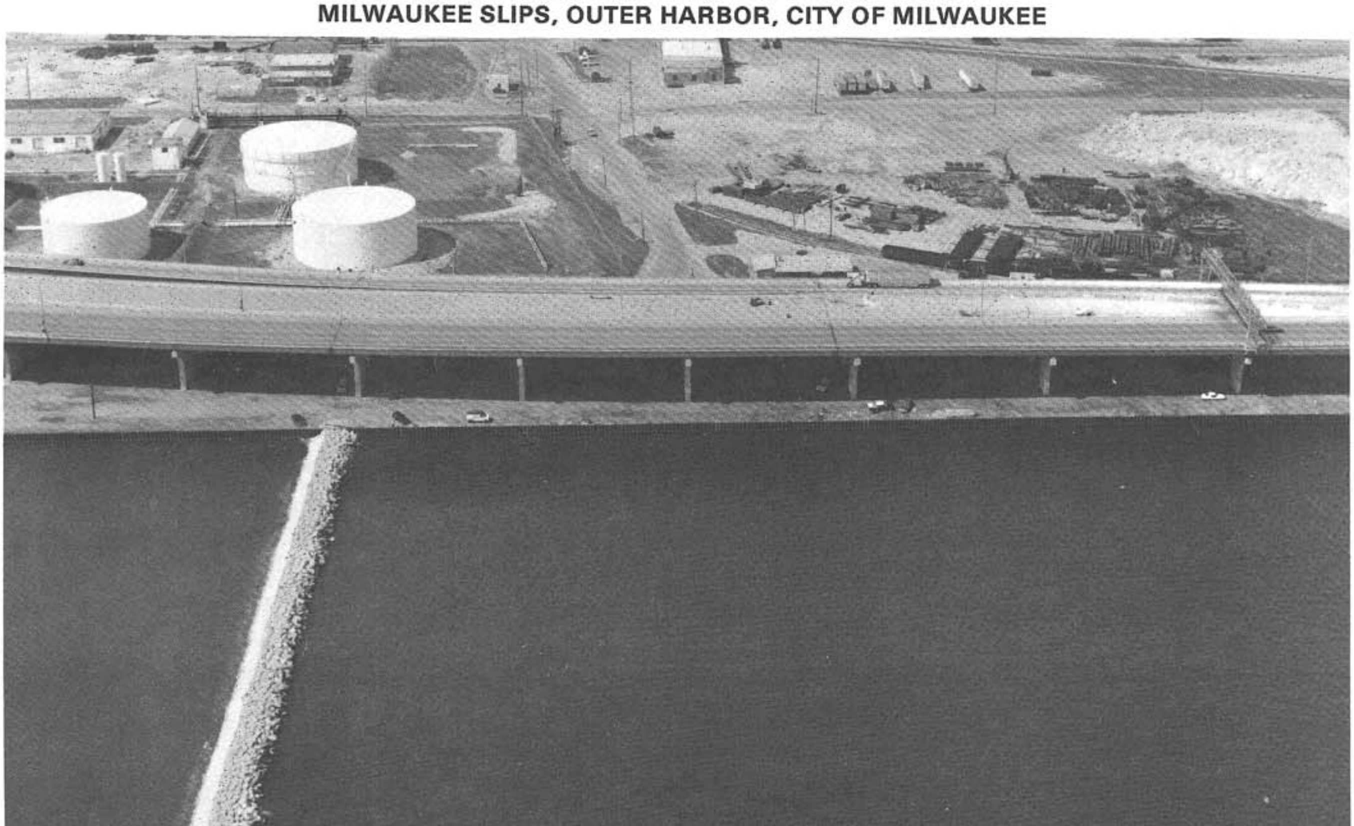
Figure A-73

U. S. ARMY CORPS OF ENGINEERS DREDGE SPOILS
CONFINED DISPOSAL FACILITY, OUTER HARBOR, CITY OF MILWAUKEE

Source: SEWRPC.

Figure A-74

S. LINCOLN MEMORIAL DRIVE, SOUTH OF PORT OF
MILWAUKEE SLIPS, OUTER HARBOR, CITY OF MILWAUKEE



Source: SEWRPC.

Figure A-75

LIQUID CARGO PIER, OUTER HARBOR, CITY OF MILWAUKEE

Source: SEWRPC.

Figure A-76

PORT OF MILWAUKEE SLIPS, OUTER HARBOR, CITY OF MILWAUKEE

Source: SEWRPC.

Figure A-77

PORT OF MILWAUKEE SLIPS, OUTER HARBOR, CITY OF MILWAUKEE

Source: SEWRPC.

Figure A-78

MMSD JONES ISLAND WASTEWATER TREATMENT PLANT, OUTER HARBOR, CITY OF MILWAUKEE

Source: SEWRPC.

Figure A-79

INNER HARBOR MOUTH, MARCUS AMPHITHEATER, OUTER HARBOR, CITY OF MILWAUKEE

Source: SEWRPC.

Figure A-80

HENRY W. MAIER FESTIVAL PROPERTY, OUTER HARBOR, CITY OF MILWAUKEE

Source: SEWRPC.

Figure A-81

HOAN BRIDGE AT HENRY W. MAIER FESTIVAL GROUNDS, CITY OF MILWAUKEE

Source: SEWRPC.

Figure A-82

HENRY W. MAIER FESTIVAL PROPERTY, OUTER HARBOR, CITY OF MILWAUKEE

Source: SEWRPC.

Figure A-83

WAR MEMORIAL CENTER, OUTER HARBOR, CITY OF MILWAUKEE

Source: SEWRPC.

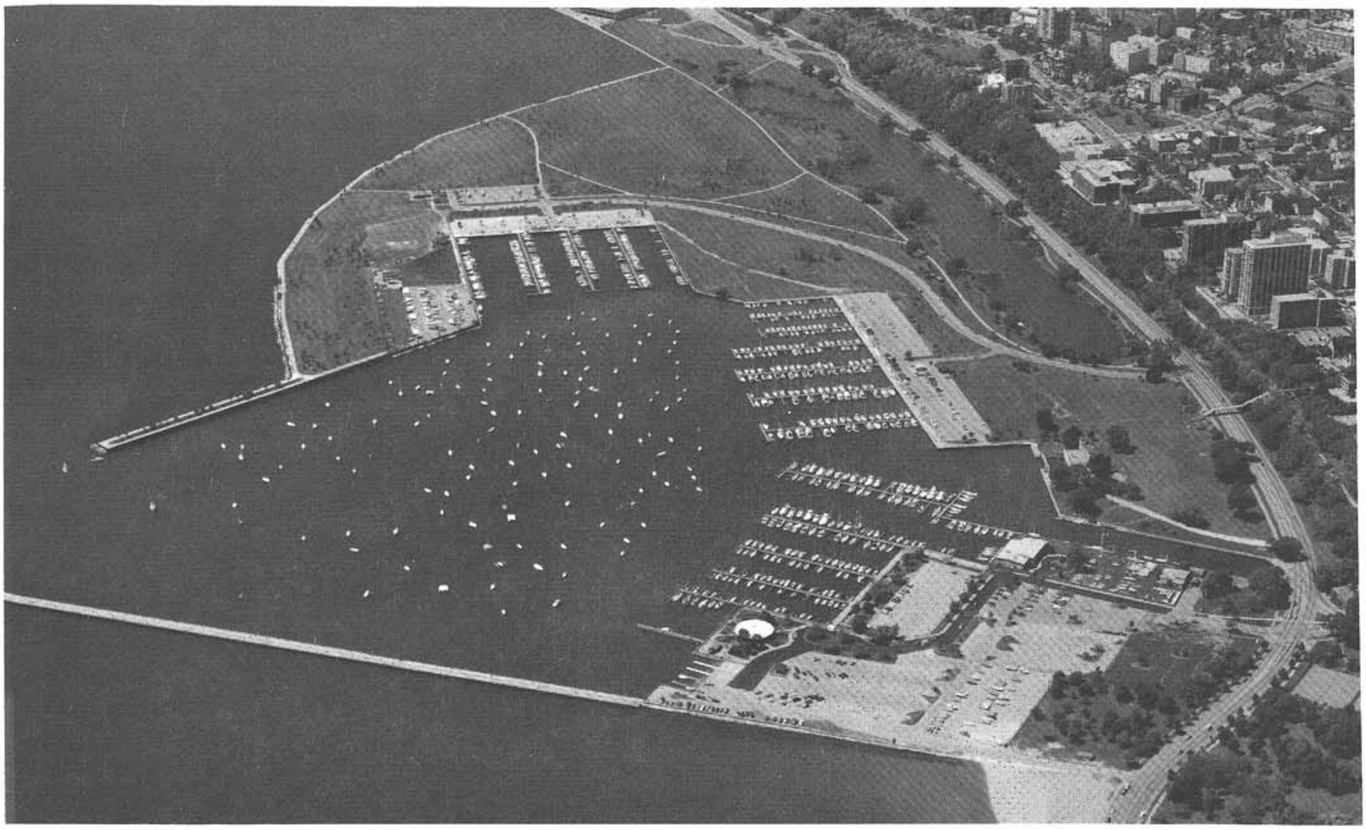
Figure A-84

JUNEAU PARK LANDFILL, OUTER HARBOR, CITY OF MILWAUKEE

Source: SEWRPC.

Figure A-85

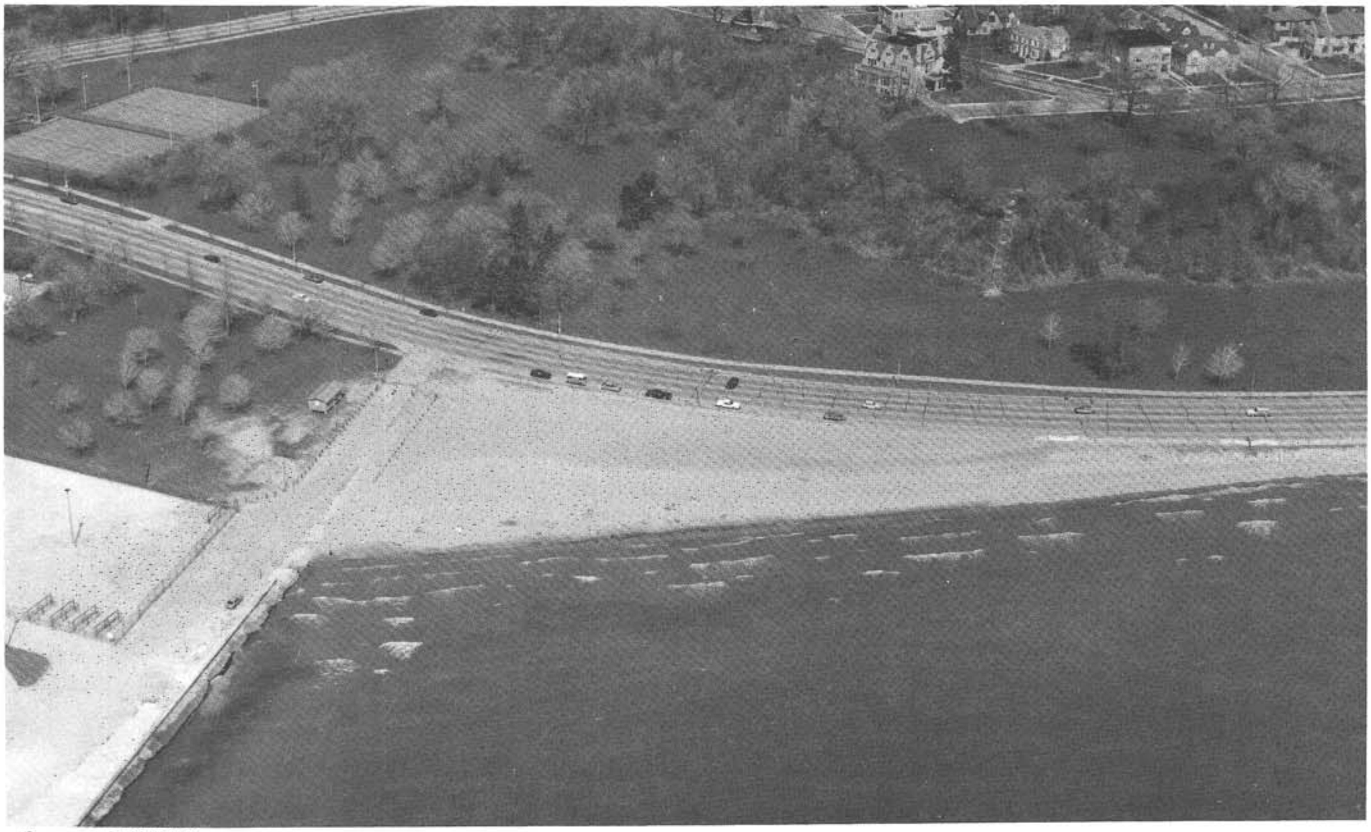
MCKINLEY MARINA, OUTER HARBOR, CITY OF MILWAUKEE



Source: SEWRPC.

Figure A-86

MCKINLEY BEACH, CITY OF MILWAUKEE: APRIL 1987



Source: SEWRPC.

Figure A-87

MCKINLEY BEACH, CITY OF MILWAUKEE: APRIL 1988

Source: SEWRPC.

Figure A-88

LINCOLN MEMORIAL DRIVE, CITY OF MILWAUKEE

Source: SEWRPC.

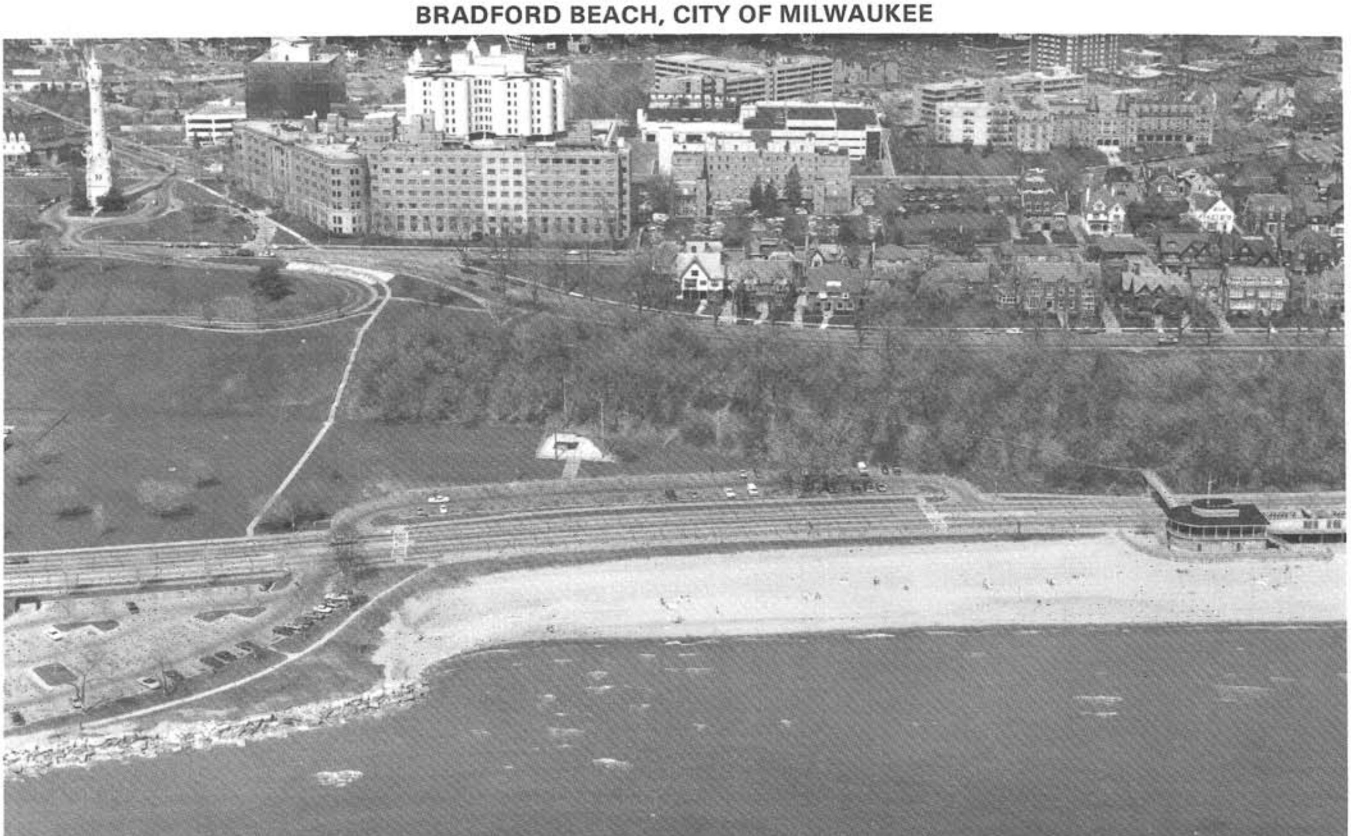
Figure A-89

LINCOLN MEMORIAL DRIVE AT NORTH POINT, CITY OF MILWAUKEE

Source: SEWRPC.

Figure A-90

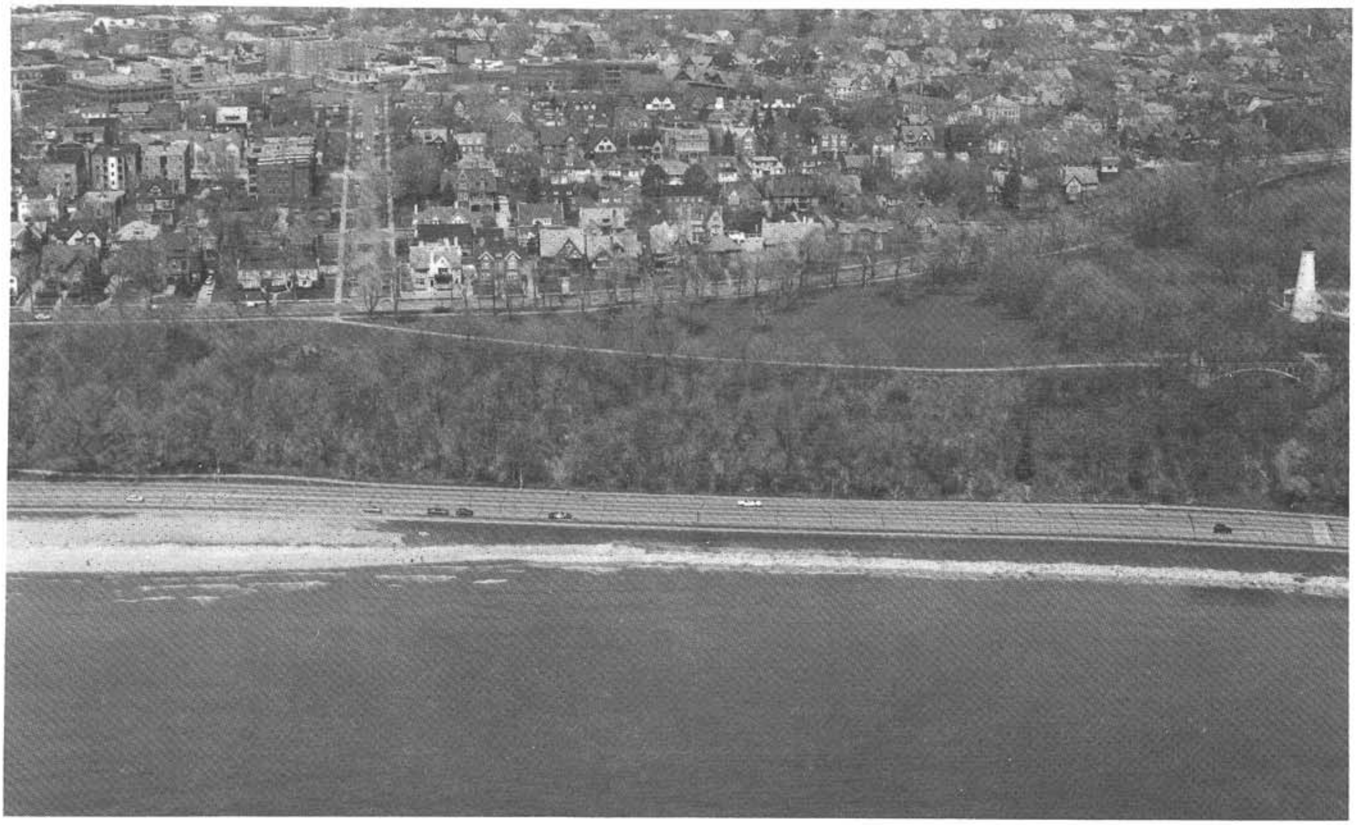
BRADFORD BEACH, CITY OF MILWAUKEE



Source: SEWRPC.

Figure A-91

LAKE PARK, LINCOLN MEMORIAL DRIVE, CITY OF MILWAUKEE



Source: SEWRPC.

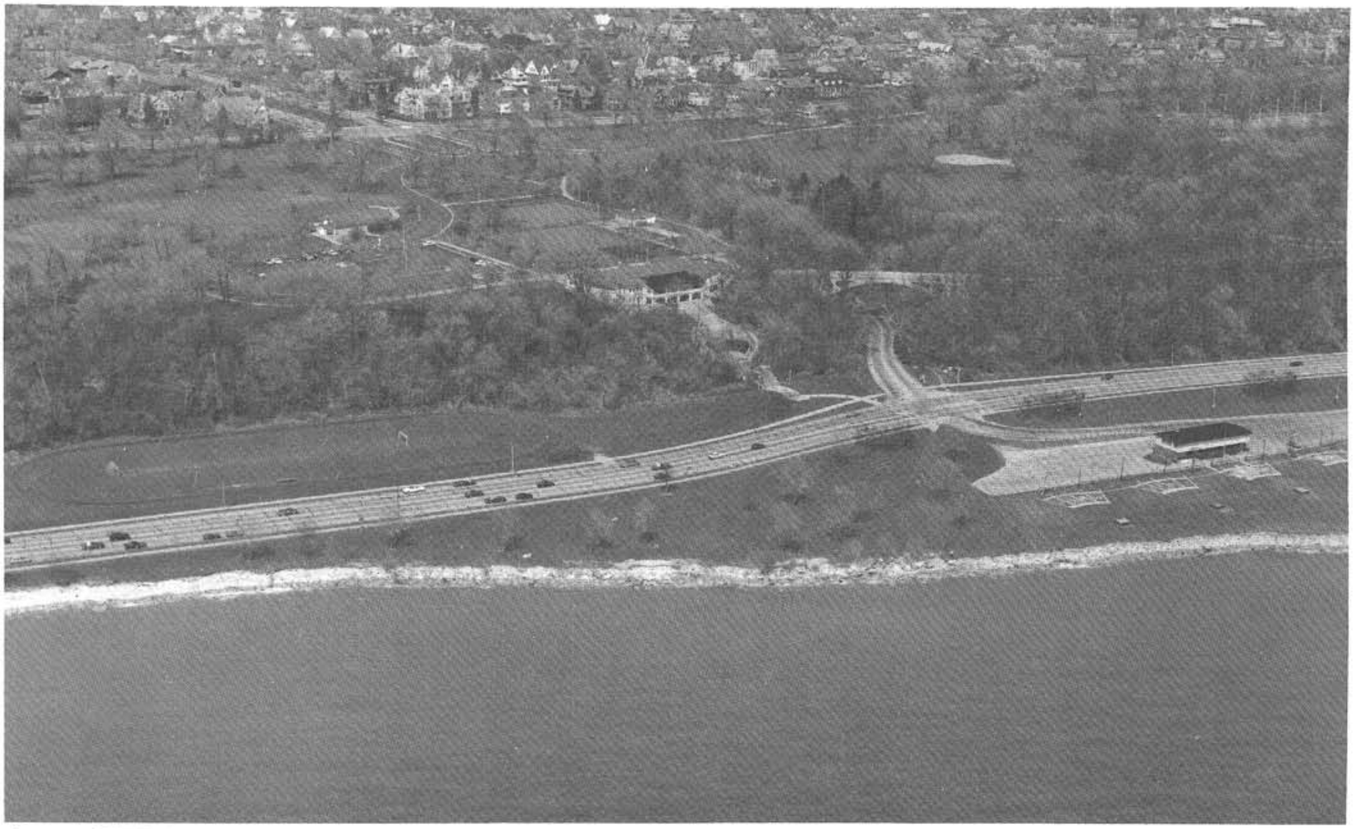
Figure A-92

LAKE PARK, LINCOLN MEMORIAL DRIVE, CITY OF MILWAUKEE

Source: SEWRPC.

Figure A-93

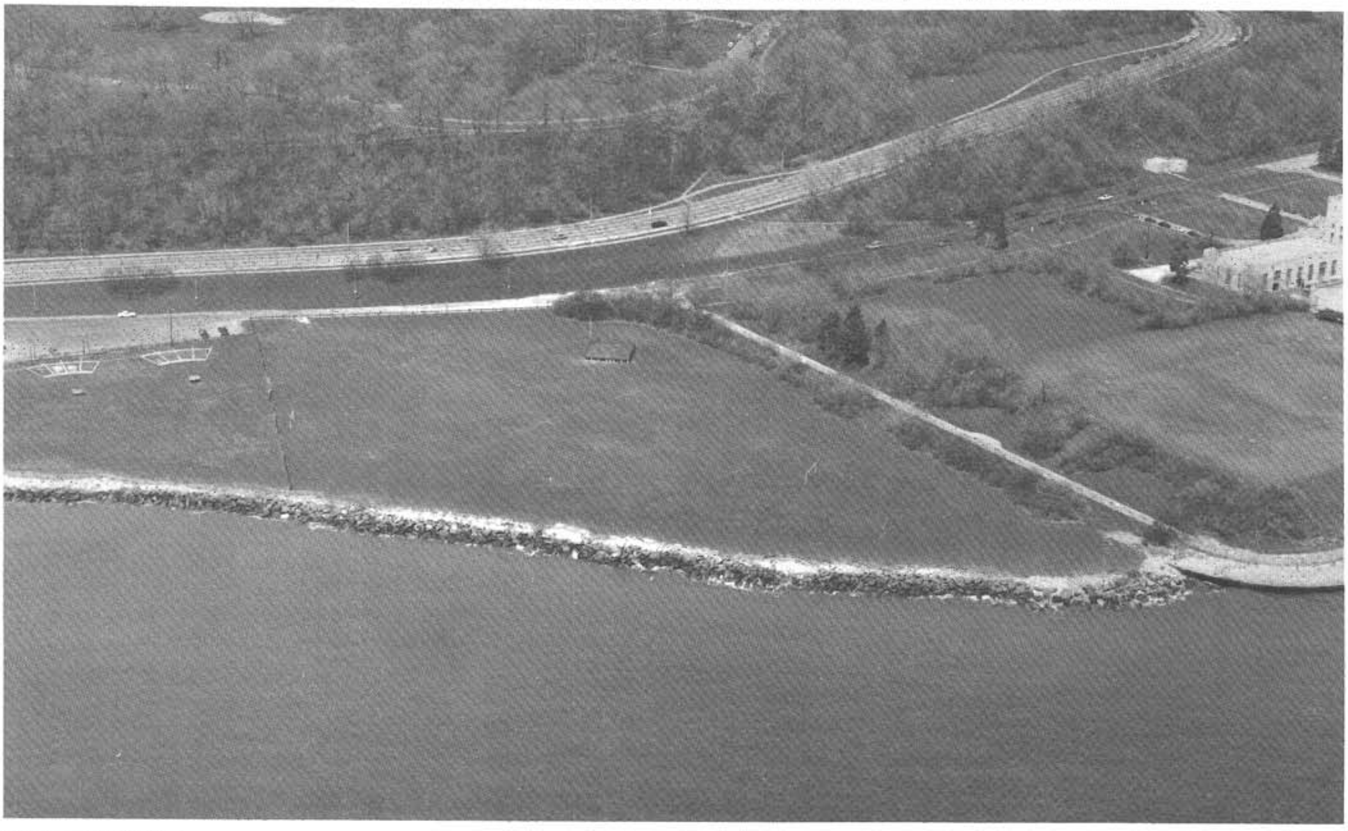
LAKE PARK, LINCOLN MEMORIAL DRIVE AT GUN CLUB, CITY OF MILWAUKEE



Source: SEWRPC.

Figure A-94

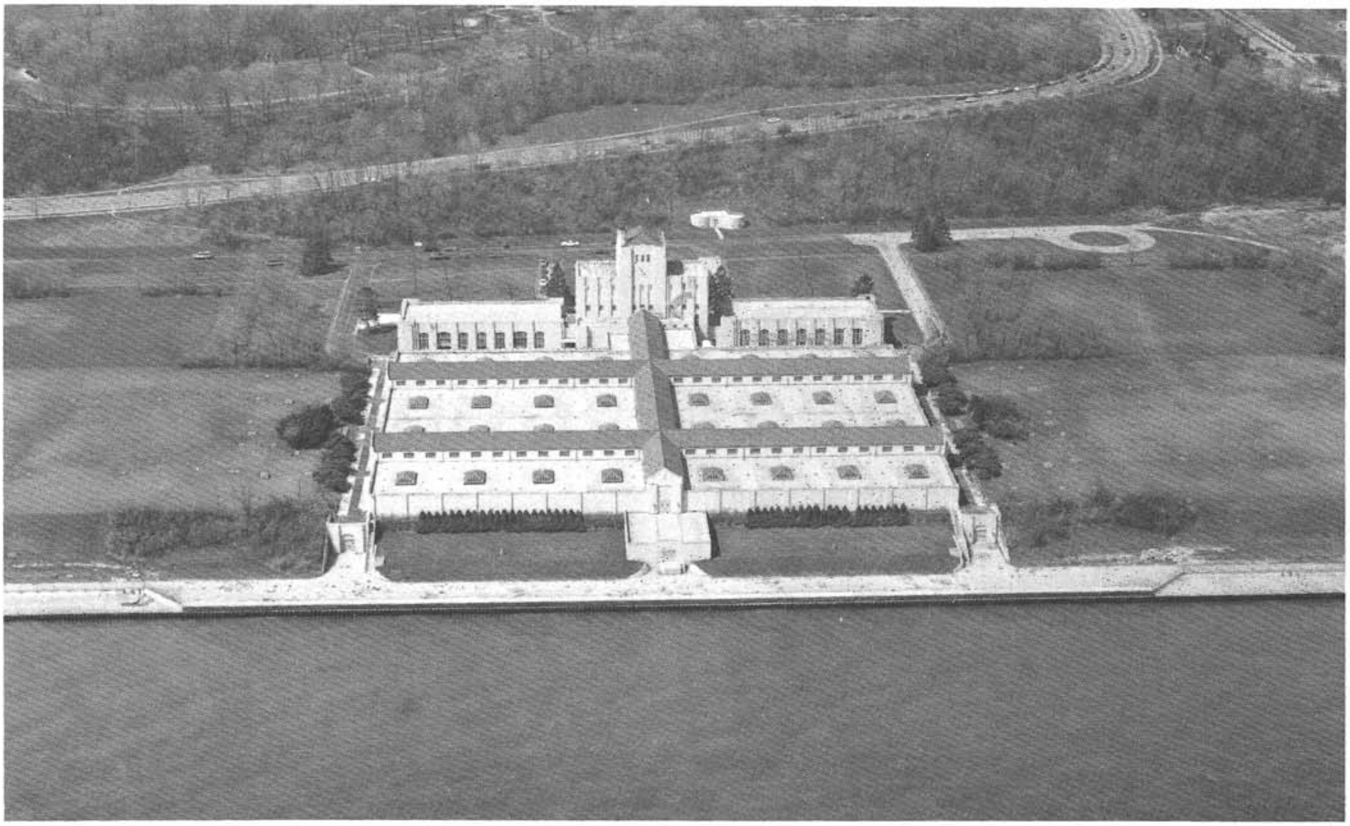
LINCOLN MEMORIAL DRIVE BETWEEN GUN CLUB AND LINNWOOD AVENUE WATER TREATMENT PLANT, CITY OF MILWAUKEE



Source: SEWRPC.

Figure A-95

LINNWOOD AVENUE WATER TREATMENT PLANT, CITY OF MILWAUKEE



Source: SEWRPC.

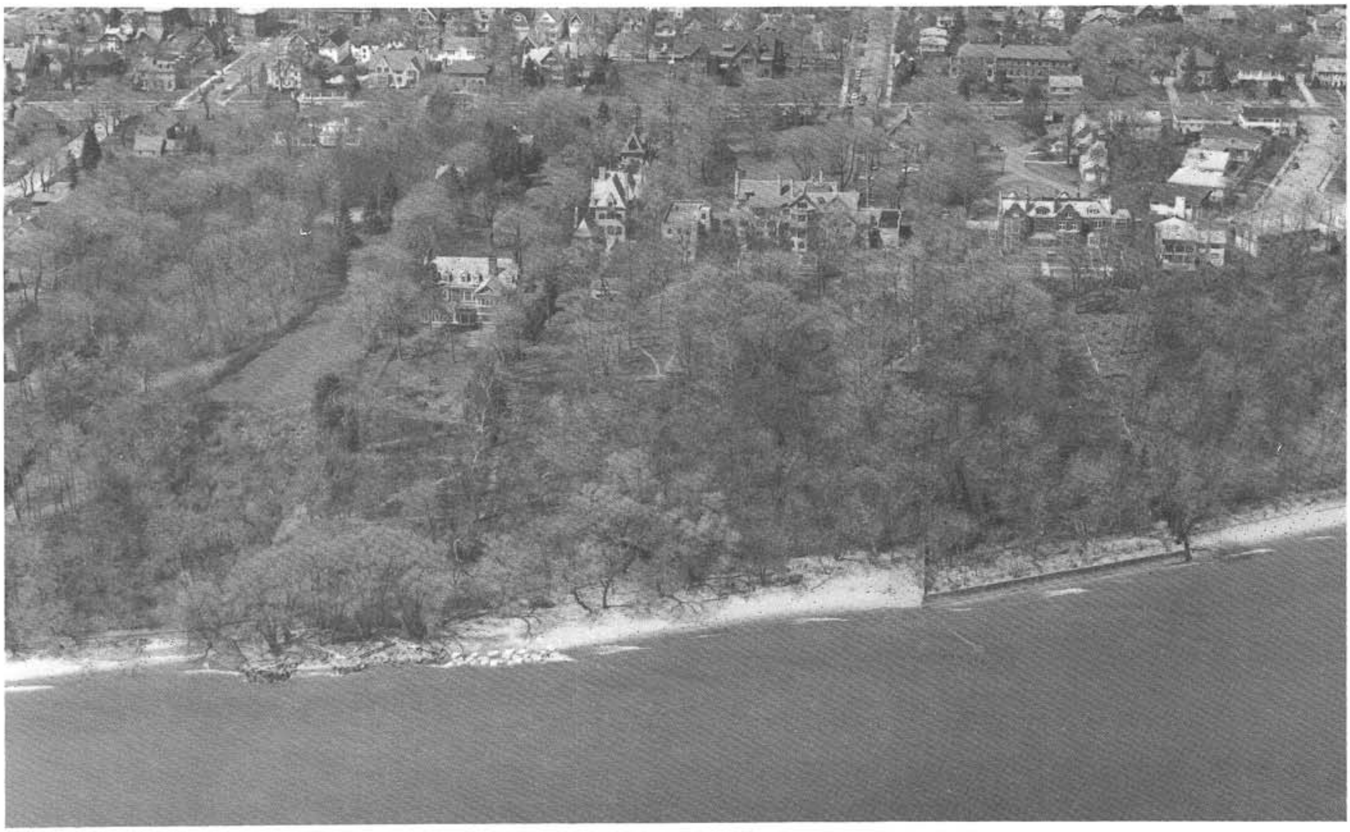
Figure A-96

KENWOOD BOULEVARD, CITY OF MILWAUKEE

Source: SEWRPC.

Figure A-97

LAKE DRIVE AT HARTFORD AVENUE-NEWPORT COURT, CITY OF MILWAUKEE



Source: SEWRPC.

Figure A-98

LAKE DRIVE AT NEWPORT COURT, CITY OF MILWAUKEE

Source: SEWRPC.

Figure A-99

EDGEWOOD AVENUE, CITY OF MILWAUKEE AND VILLAGE OF SHOREWOOD

Source: SEWRPC.

Figure A-100

3510-3704 N. LAKE DRIVE, VILLAGE OF SHOREWOOD

Source: SEWRPC.

Figure A-101

3704-3944 N. LAKE DRIVE, VILLAGE OF SHOREWOOD

Source: SEWRPC.

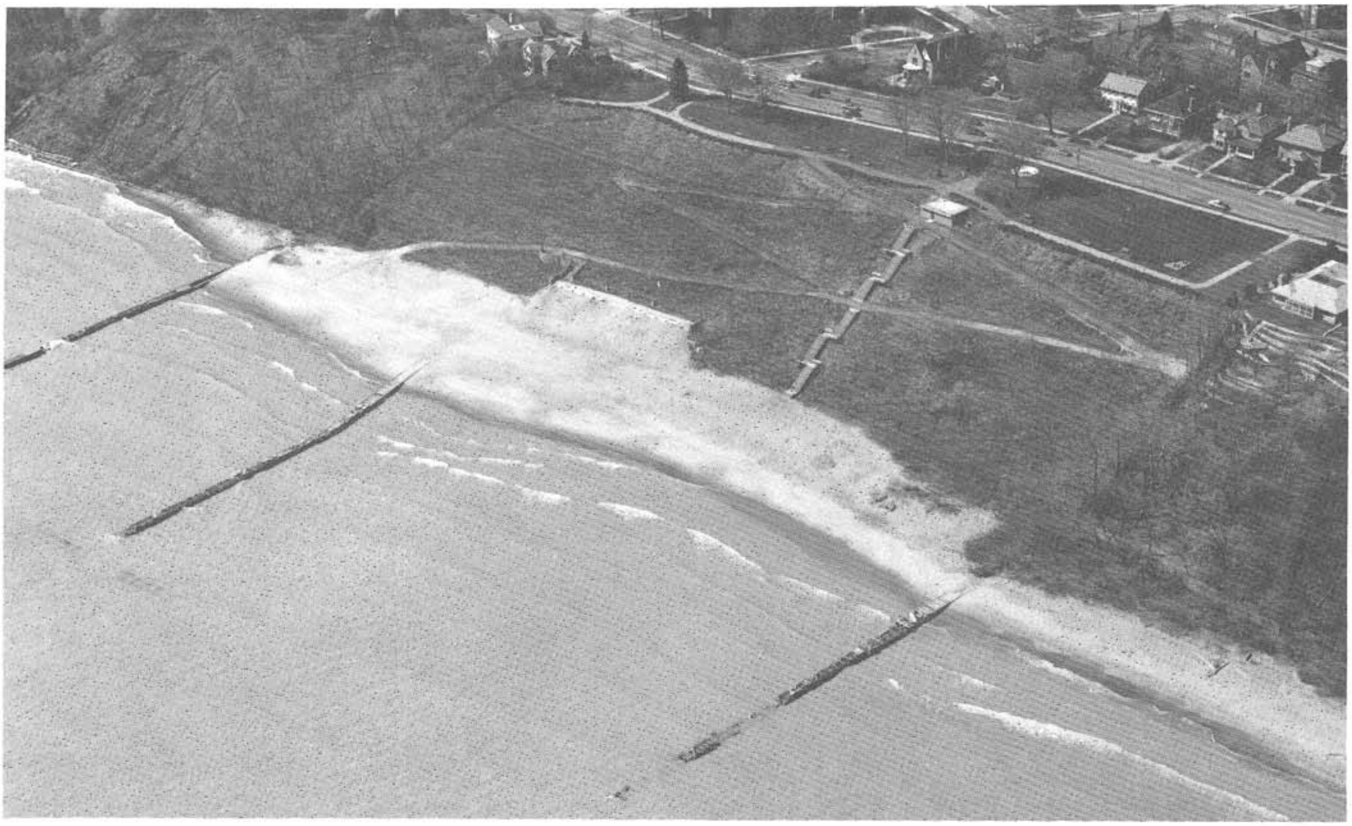
Figure A-102

ATWATER PARK-N. LAKE DRIVE AT E. CAPITOL DRIVE, VILLAGE OF SHOREWOOD: APRIL 1987

Source: SEWRPC.

Figure A-103

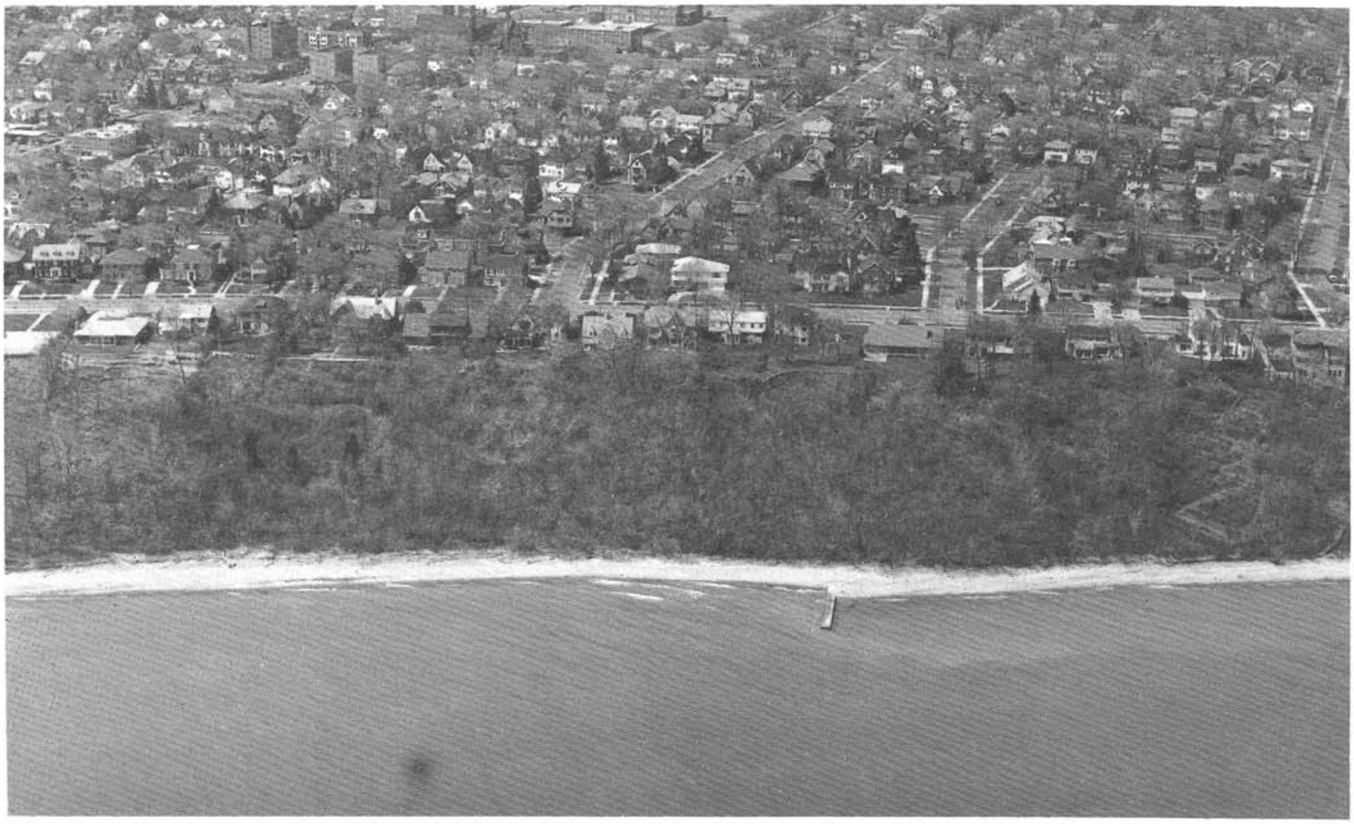
ATWATER PARK-N. LAKE DRIVE AT E. CAPITOL DRIVE, VILLAGE OF SHOREWOOD: APRIL 1988



Source: SEWRPC.

Figure A-104

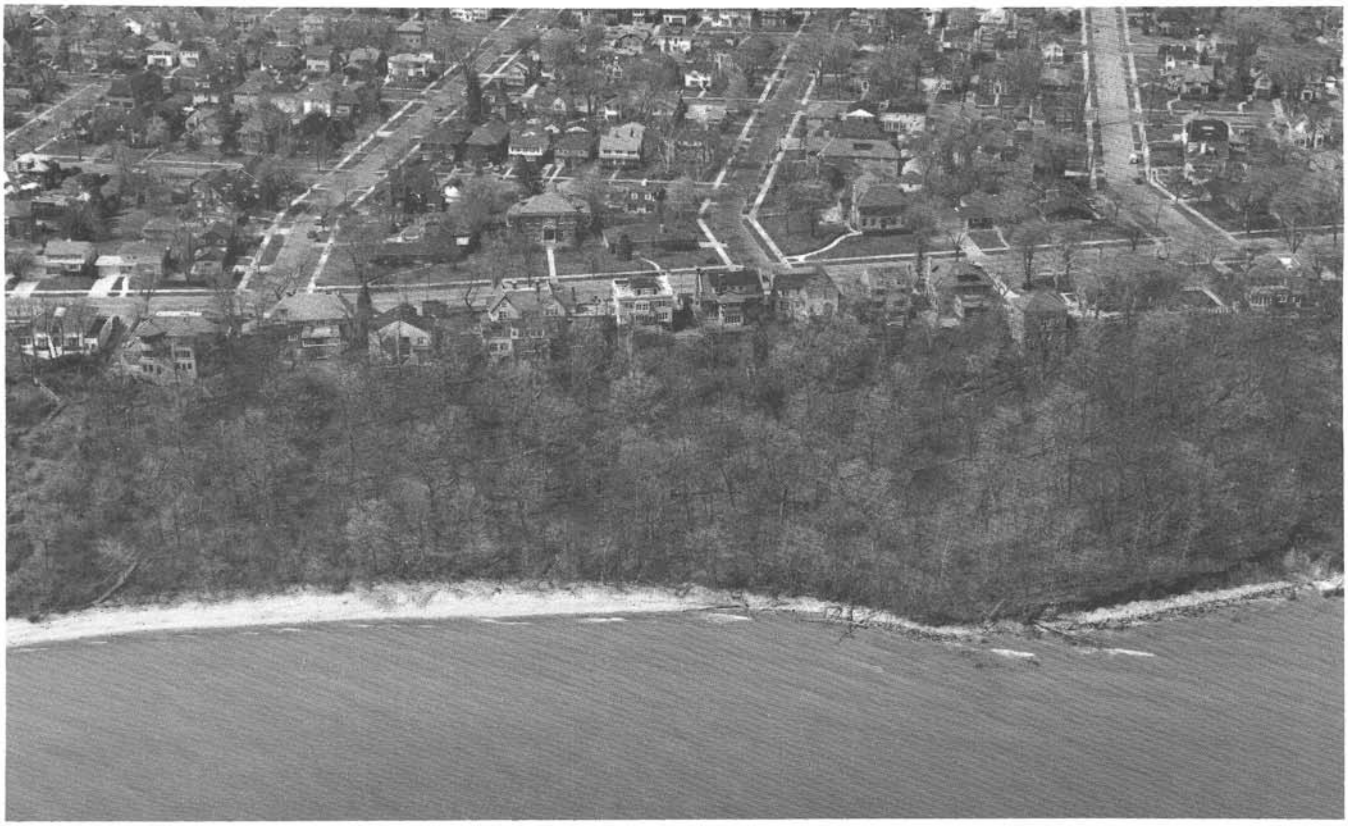
4060-4162 N. LAKE DRIVE, VILLAGE OF SHOREWOOD



Source: SEWRPC.

Figure A-105

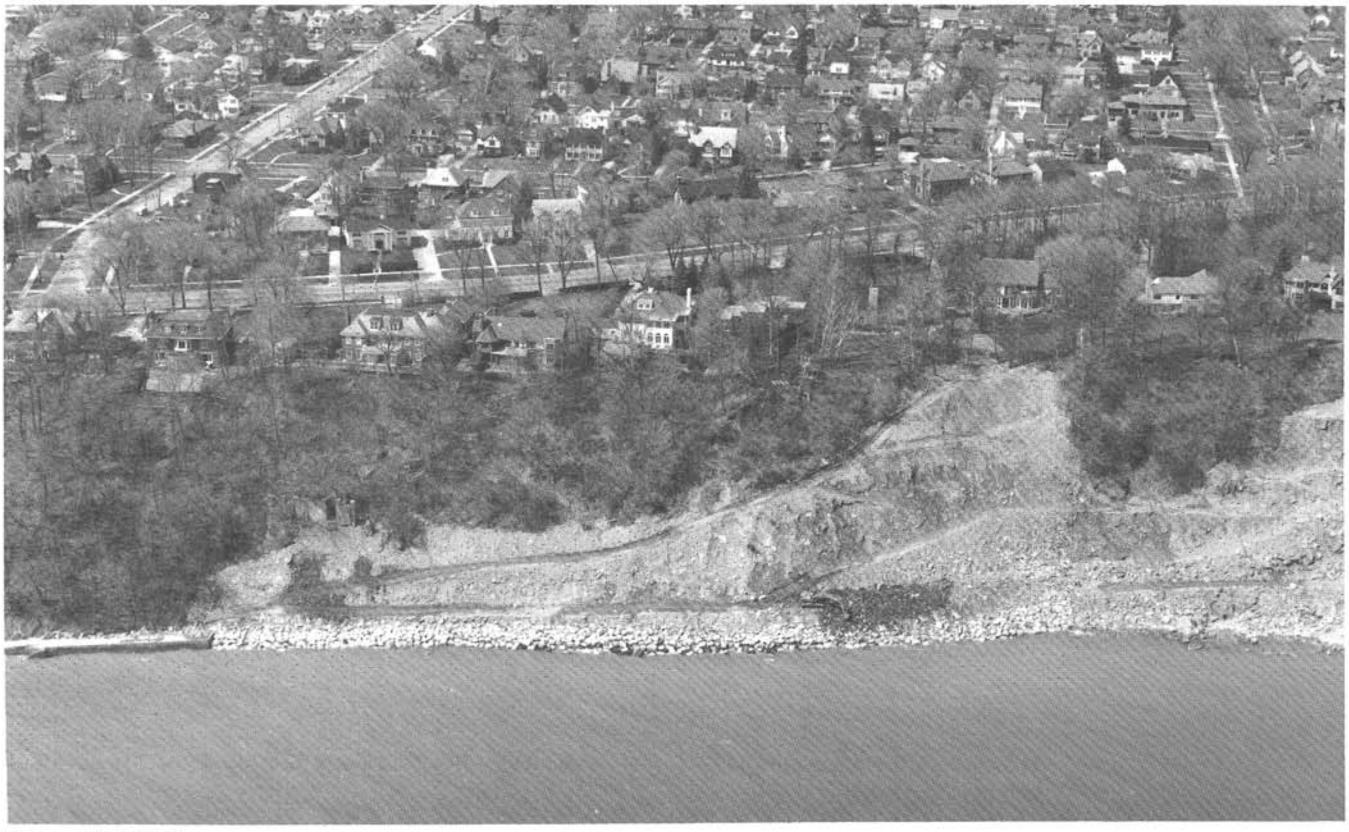
4154-4400 N. LAKE DRIVE, VILLAGE OF SHOREWOOD



Source: SEWRPC.

Figure A-106

4408-4496 N. LAKE DRIVE, VILLAGE OF SHOREWOOD



Source: SEWRPC.

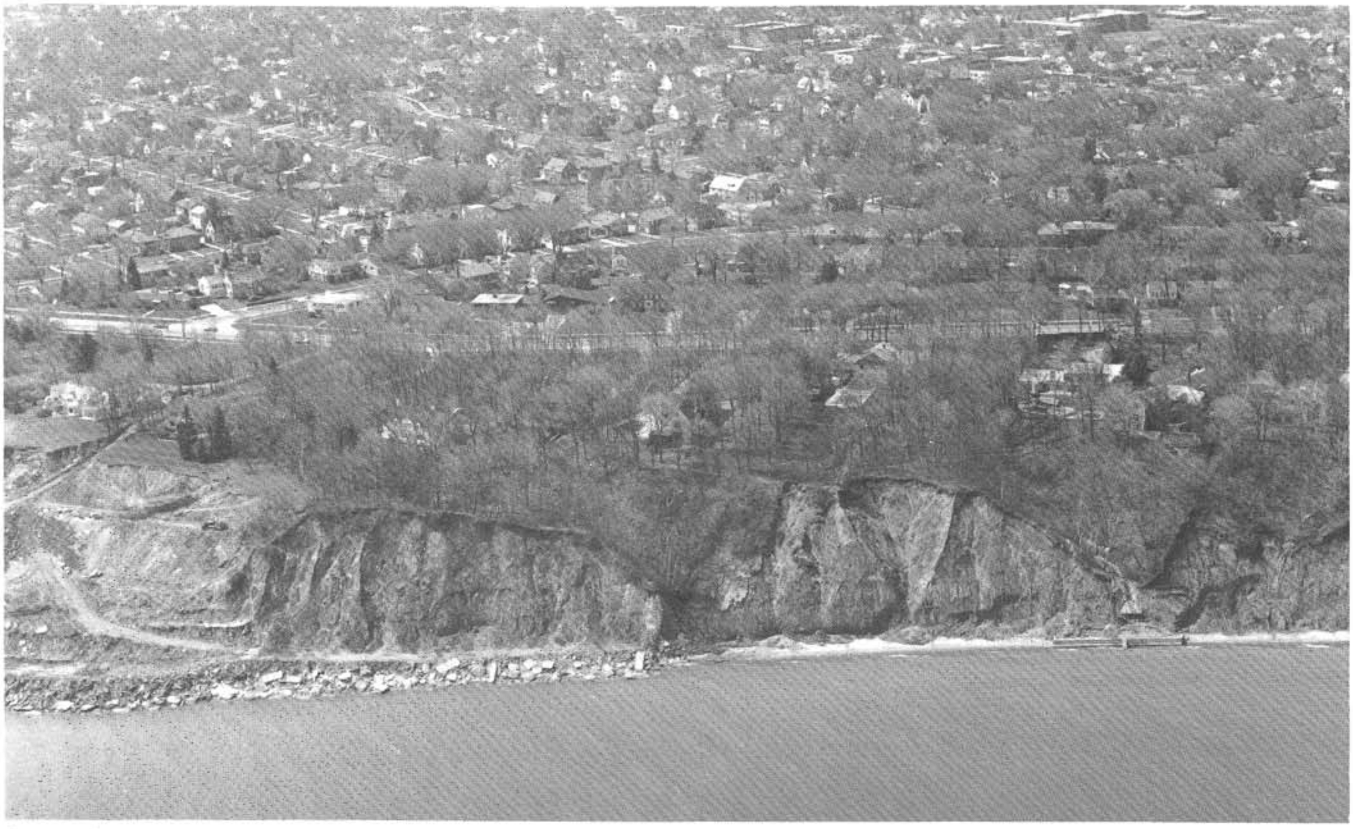
Figure A-107

4480-4646 N. LAKE DRIVE, VILLAGES OF SHOREWOOD AND WHITEFISH BAY

Source: SEWRPC.

Figure A-108

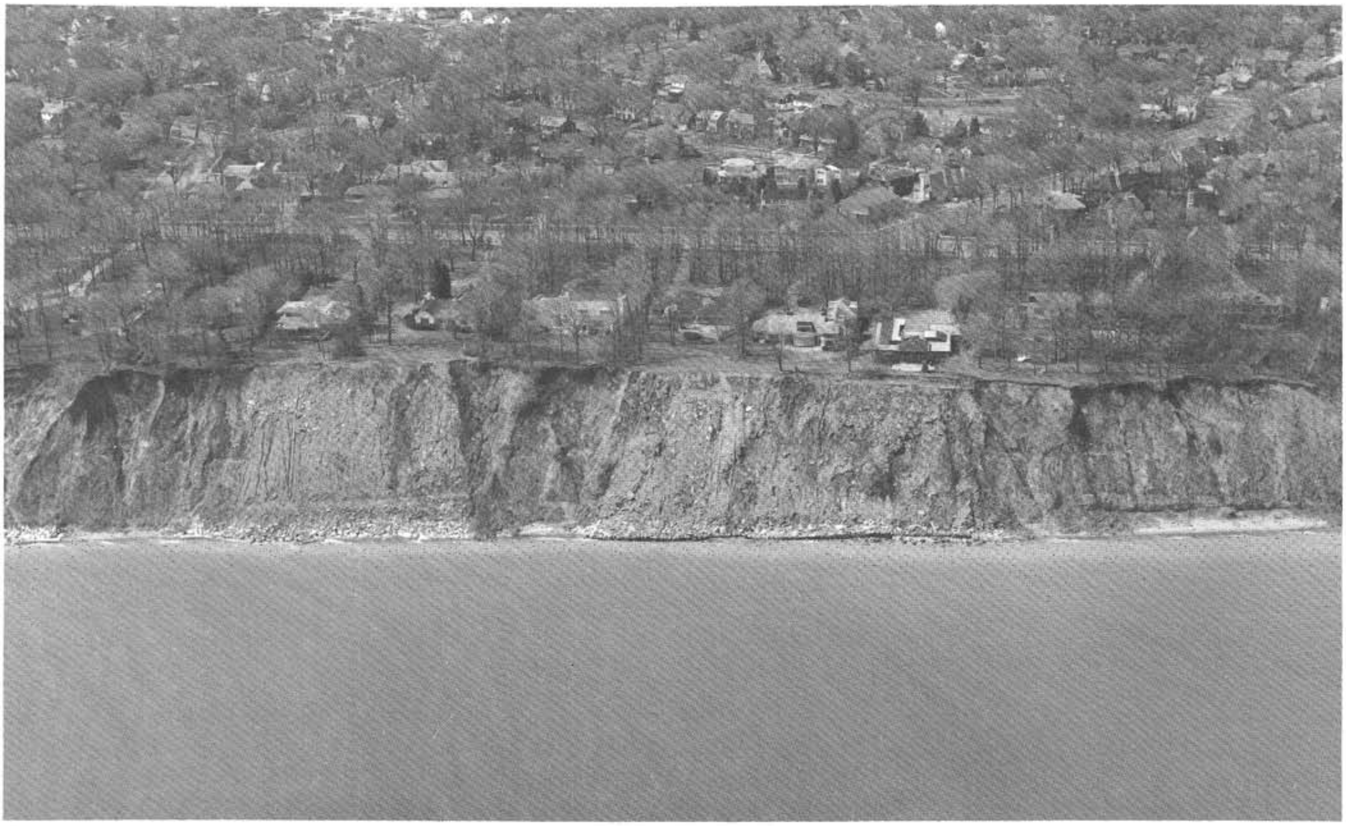
4614-4686 N. LAKE DRIVE, VILLAGE OF WHITEFISH BAY



Source: SEWRPC.

Figure A-109

4700-4840 N. LAKE DRIVE, VILLAGE OF WHITEFISH BAY



Source: SEWRPC.

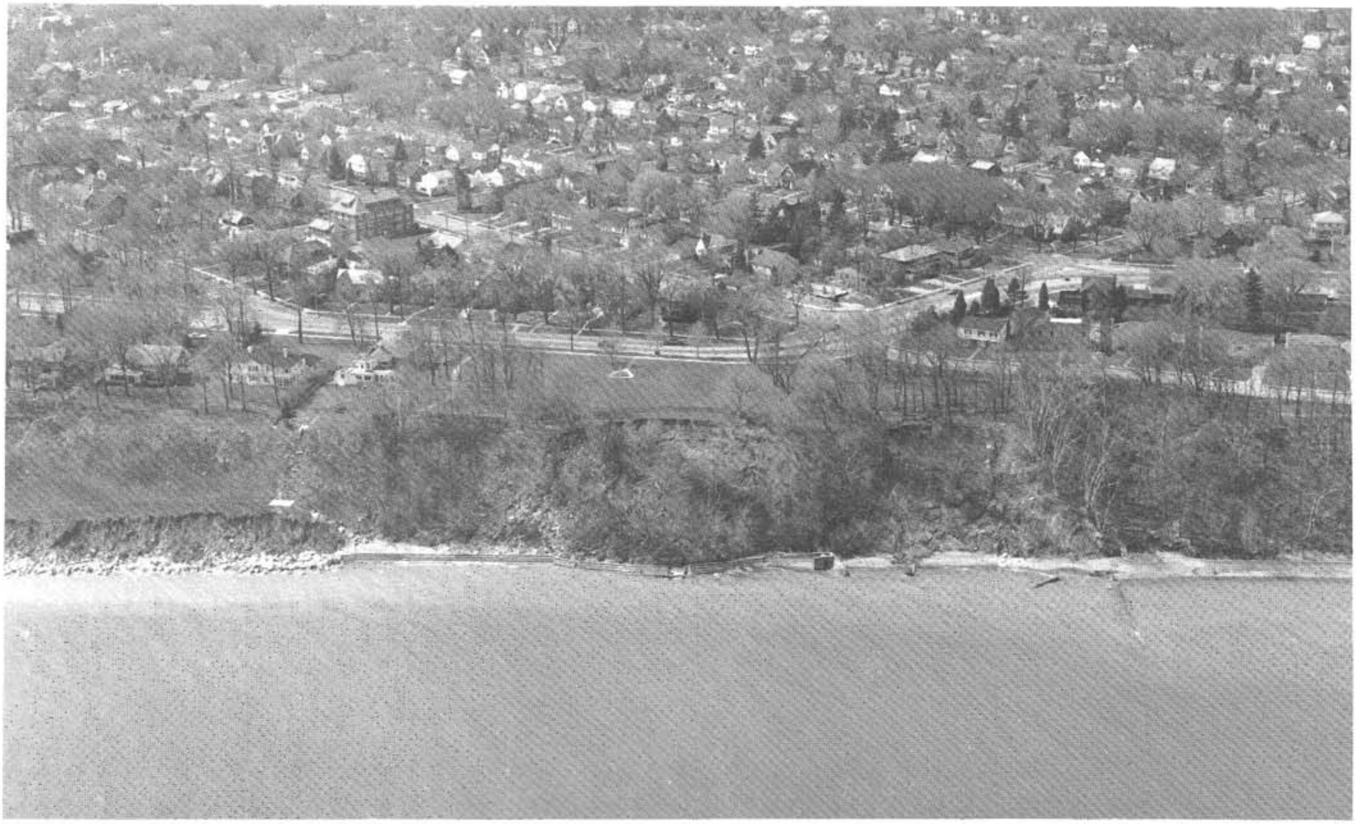
Figure A-110

4830-4940 N. LAKE DRIVE, VILLAGE OF WHITEFISH BAY

Source: SEWRPC.

Figure A-111

BUCKLEY PARK, VILLAGE OF WHITEFISH BAY



Source: SEWRPC.

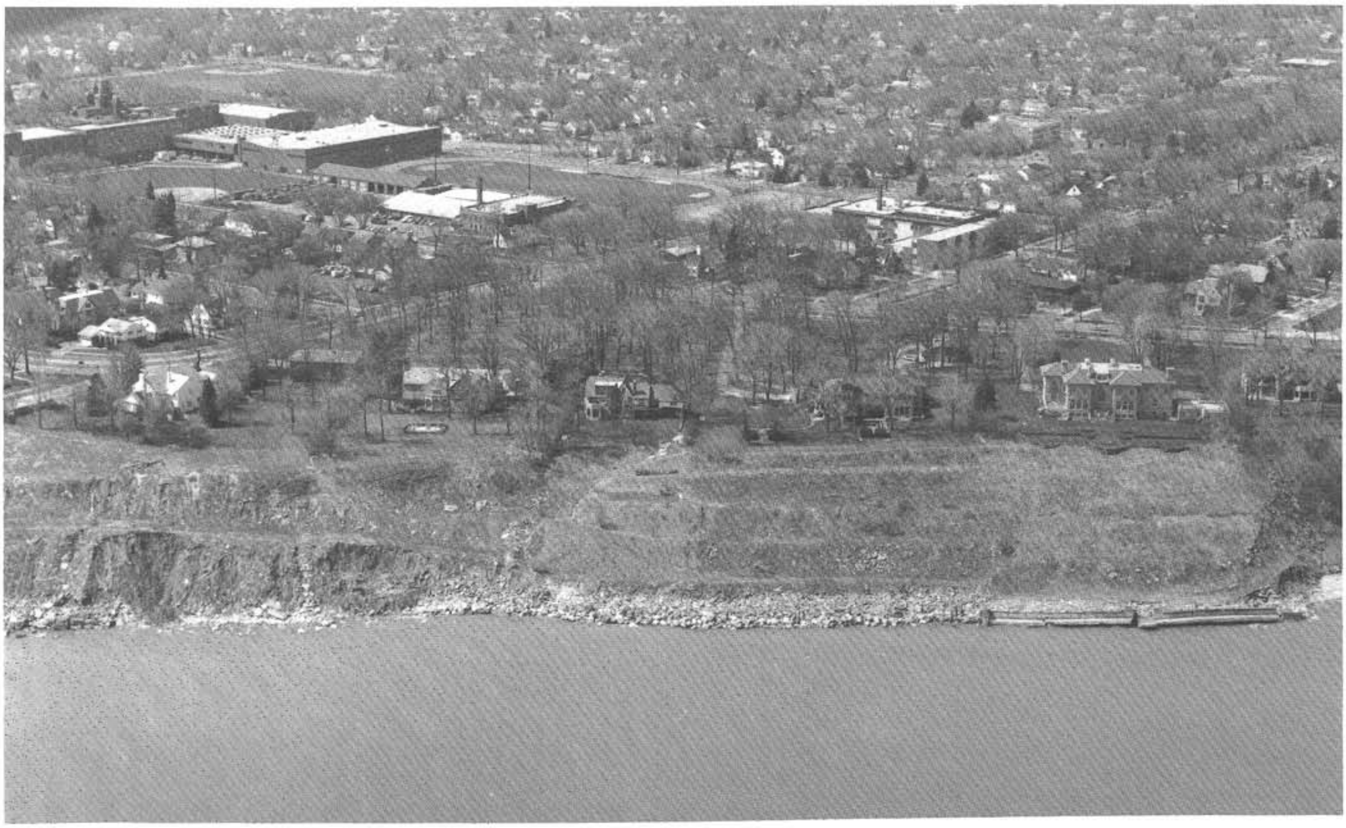
Figure A-112

BIG BAY PARK, PALISADES DRIVE, VILLAGE OF WHITEFISH BAY

Source: SEWRPC.

Figure A-113

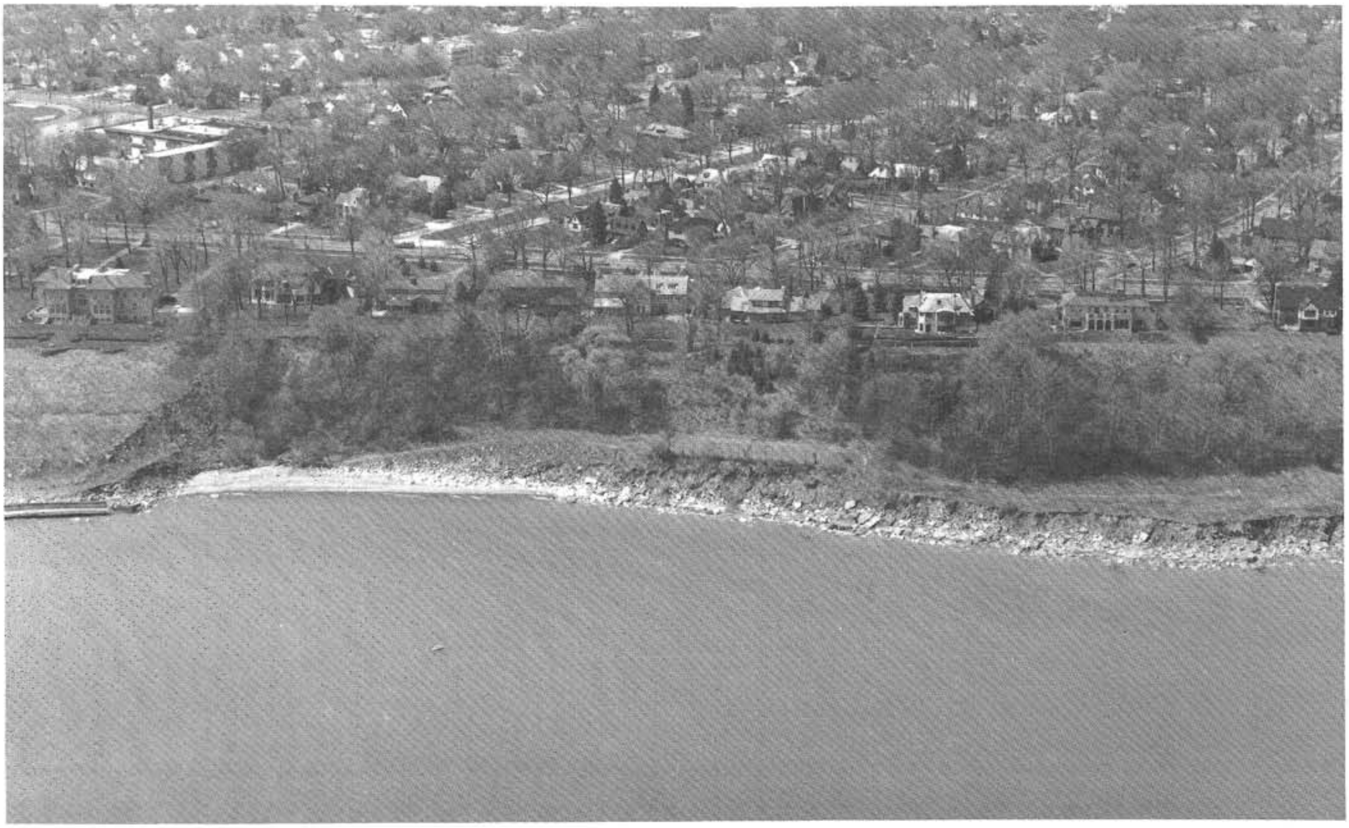
1500 E. HENRY CLAY STREET-5290 N. LAKE DRIVE, VILLAGE OF WHITEFISH BAY



Source: SEWRPC.

Figure A-114

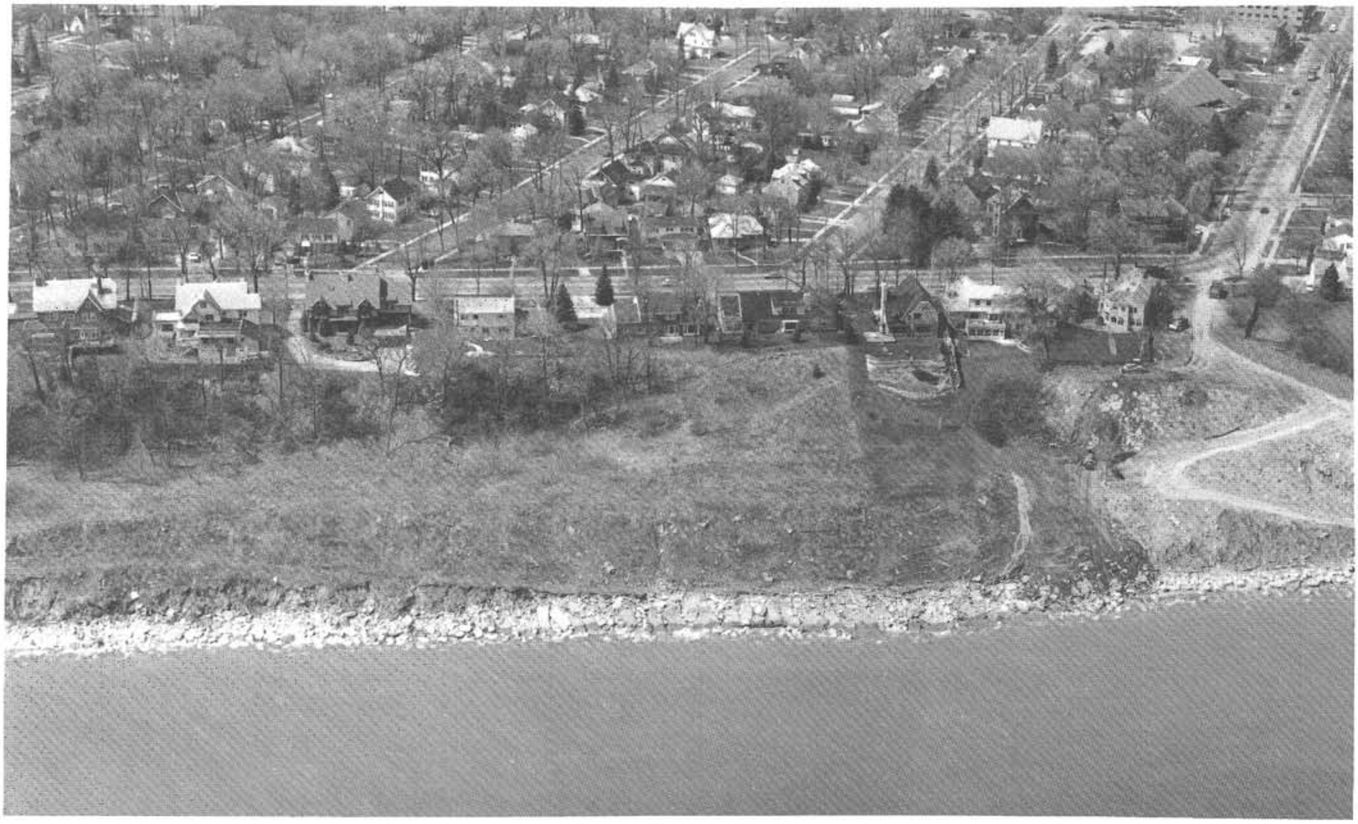
5270-5418 N. LAKE DRIVE, VILLAGE OF WHITEFISH BAY



Source: SEWRPC.

Figure A-115

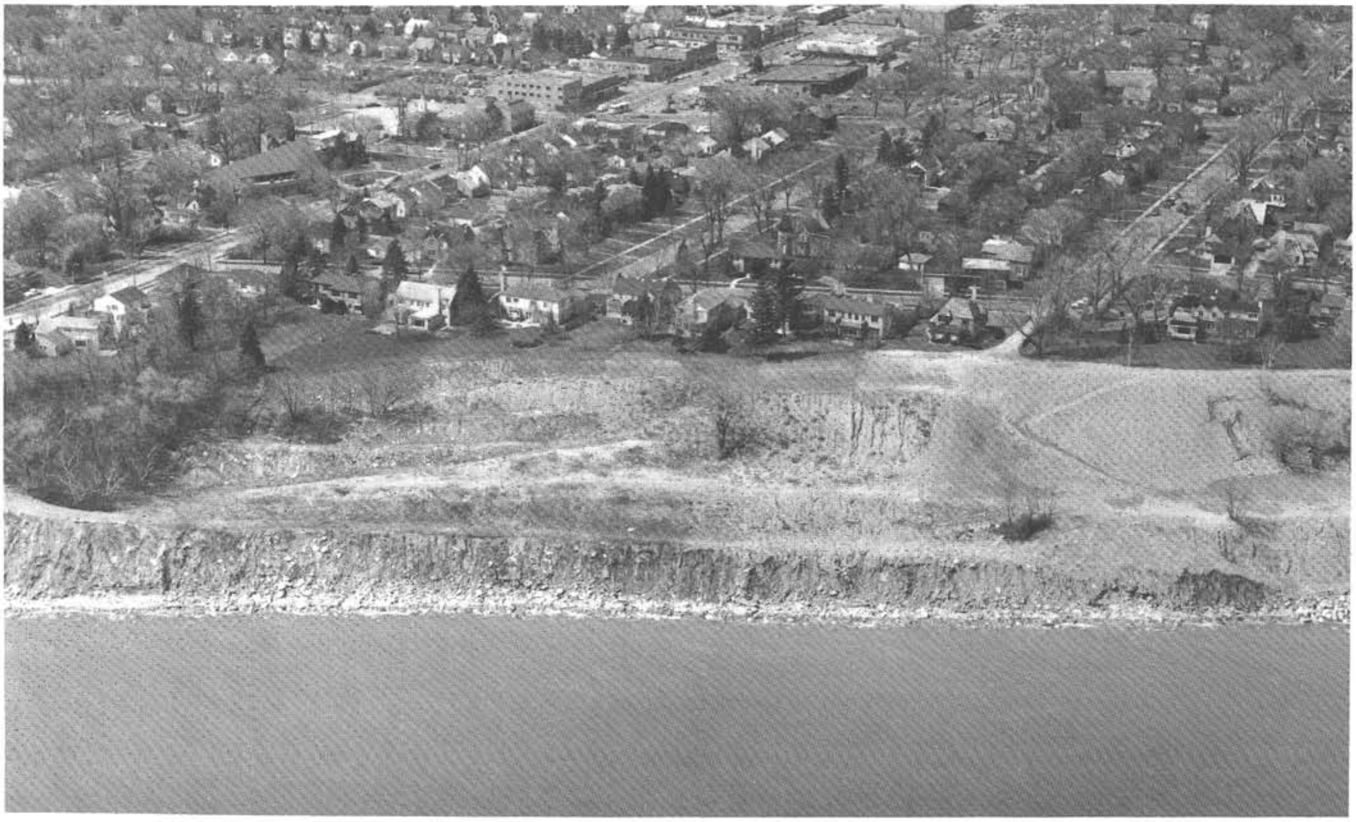
5460-5570 N. LAKE DRIVE-SILVER SPRING DRIVE, VILLAGE OF WHITEFISH BAY



Source: SEWRPC.

Figure A-116

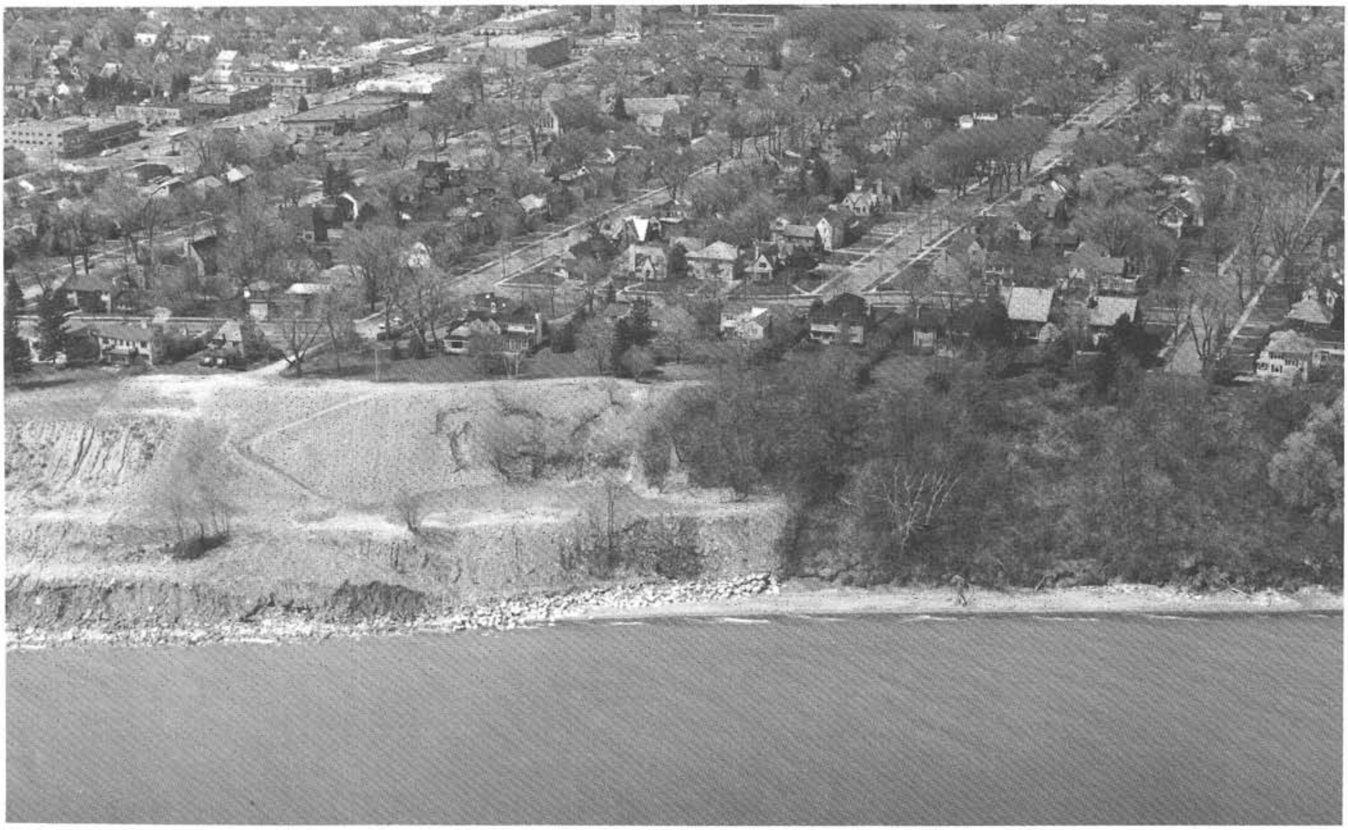
SILVER SPRING DRIVE-5722 N. SHORE DRIVE, VILLAGE OF WHITEFISH BAY



Source: SEWRPC.

Figure A-117

5664 N. SHORE DRIVE-E. DAY AVENUE, VILLAGE OF WHITEFISH BAY



Source: SEWRPC.

Figure A-118

5752 N. SHORE DRIVE-KLODE PARK, VILLAGE OF WHITEFISH BAY

Source: SEWRPC.

Figure A-119

KLODE PARK, VILLAGE OF WHITEFISH BAY: APRIL 1987

Source: SEWRPC.

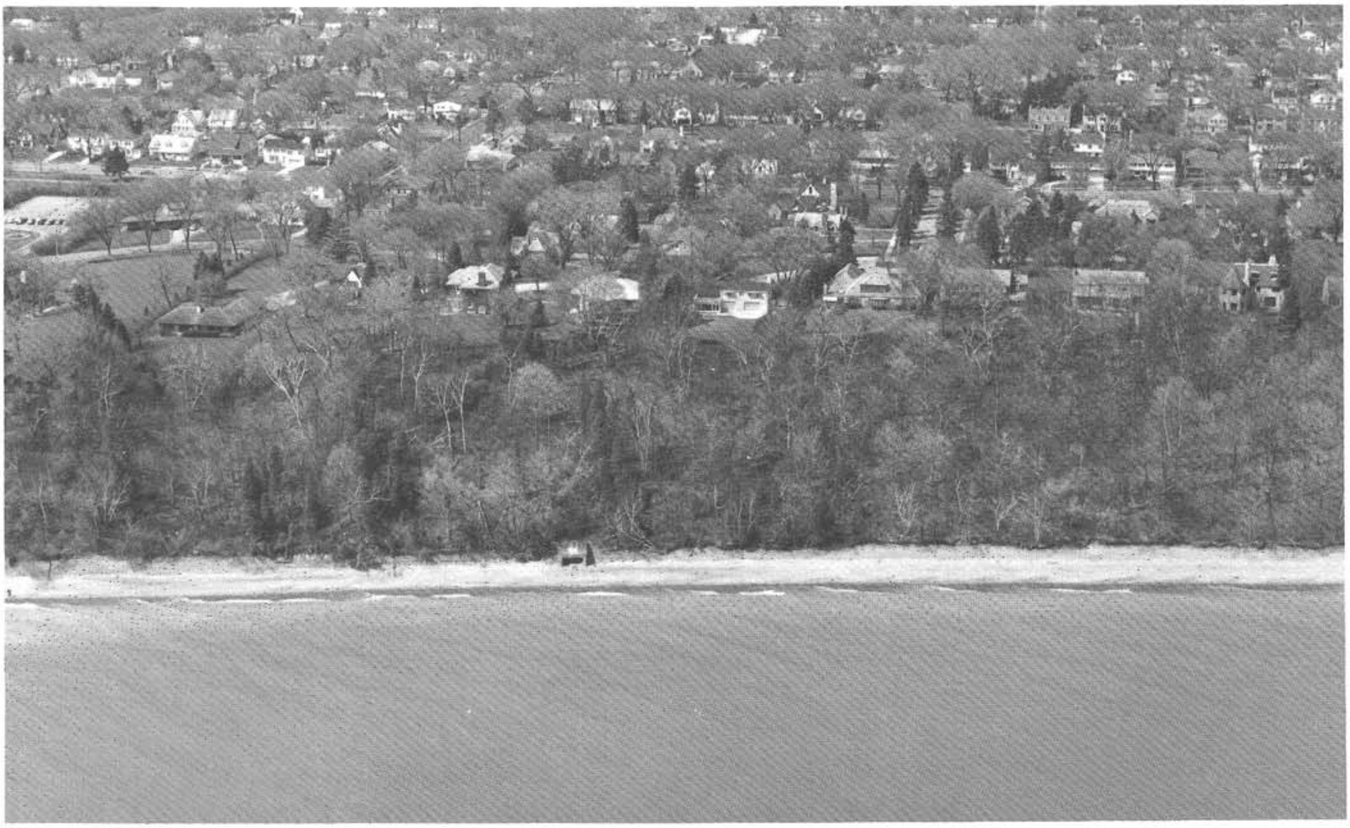
Figure A-120

KLODE PARK, VILLAGE OF WHITEFISH BAY: APRIL 1988

Source: SEWRPC.

Figure A-121

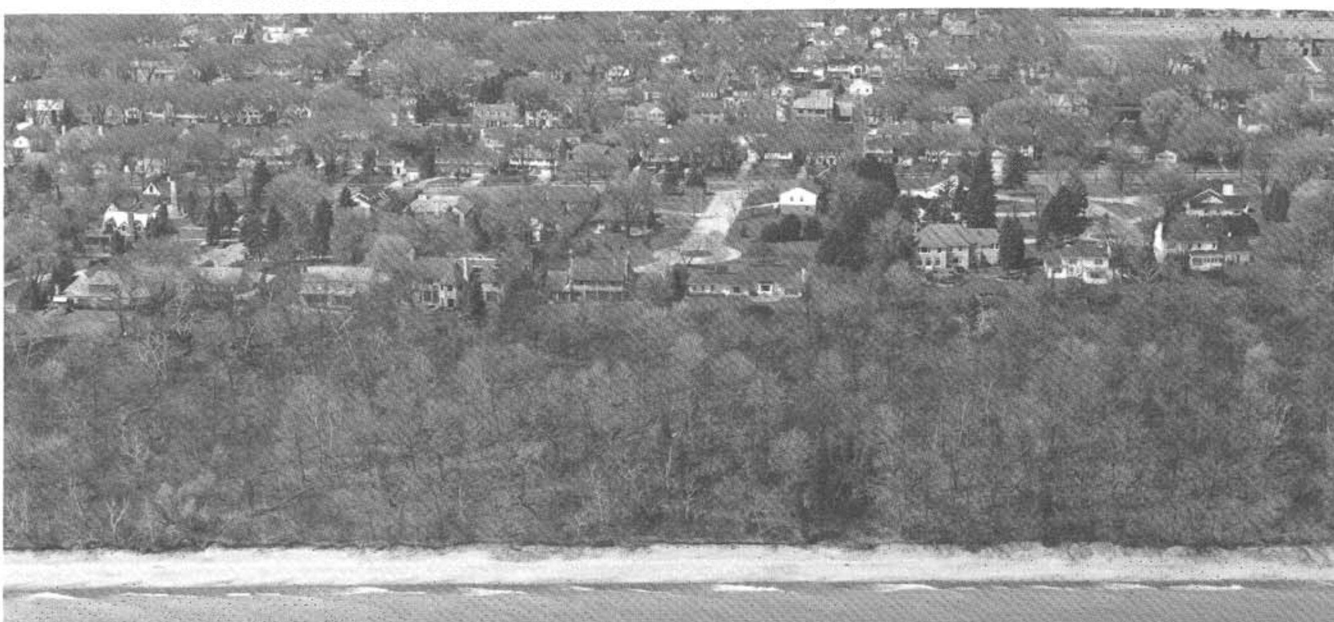
KLODE PARK-6130 N. LAKE DRIVE COURT, VILLAGE OF WHITEFISH BAY



Source: SEWRPC.

Figure A-122

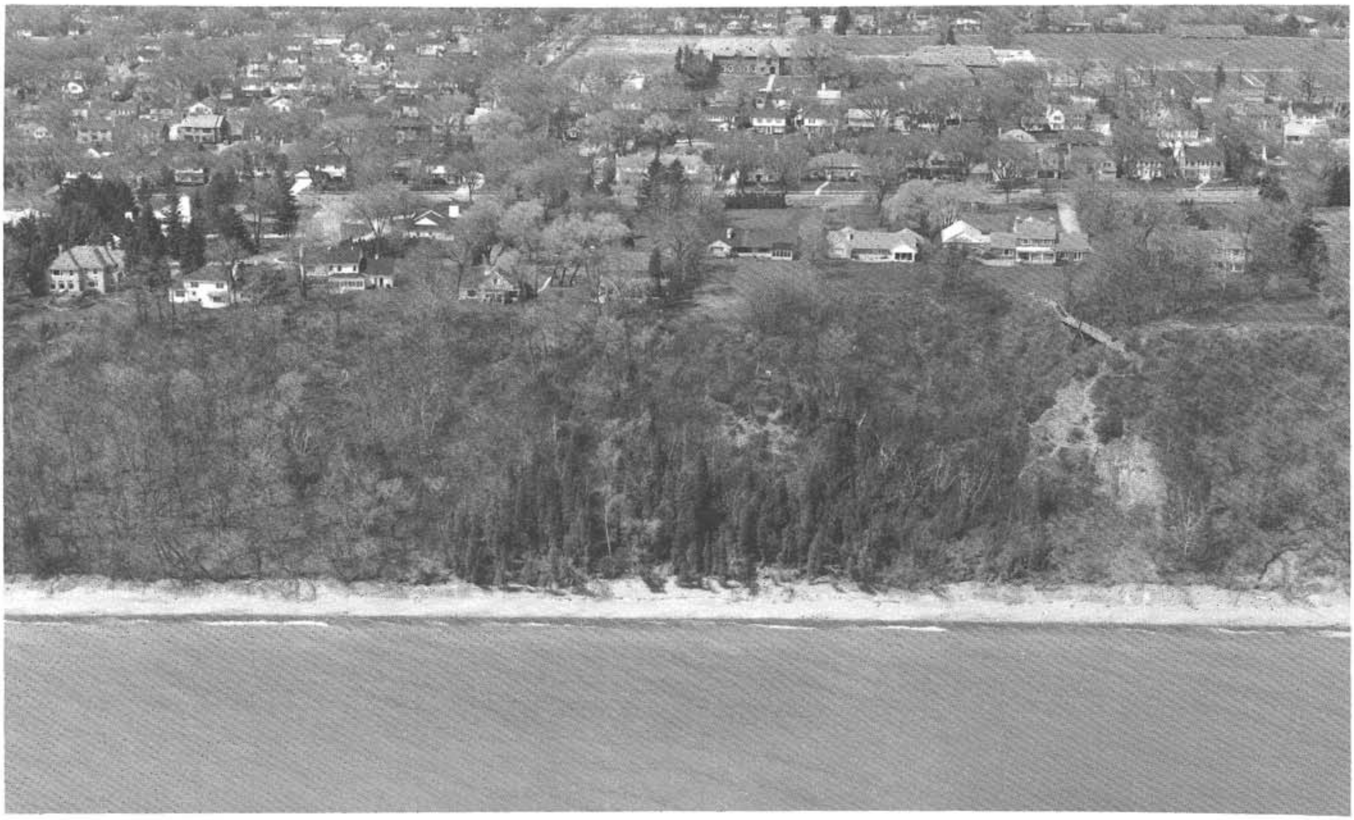
6100 N. LAKE DRIVE COURT-610 E. LAKE HILL COURT, VILLAGE OF WHITEFISH BAY



Source: SEWRPC.

Figure A-123

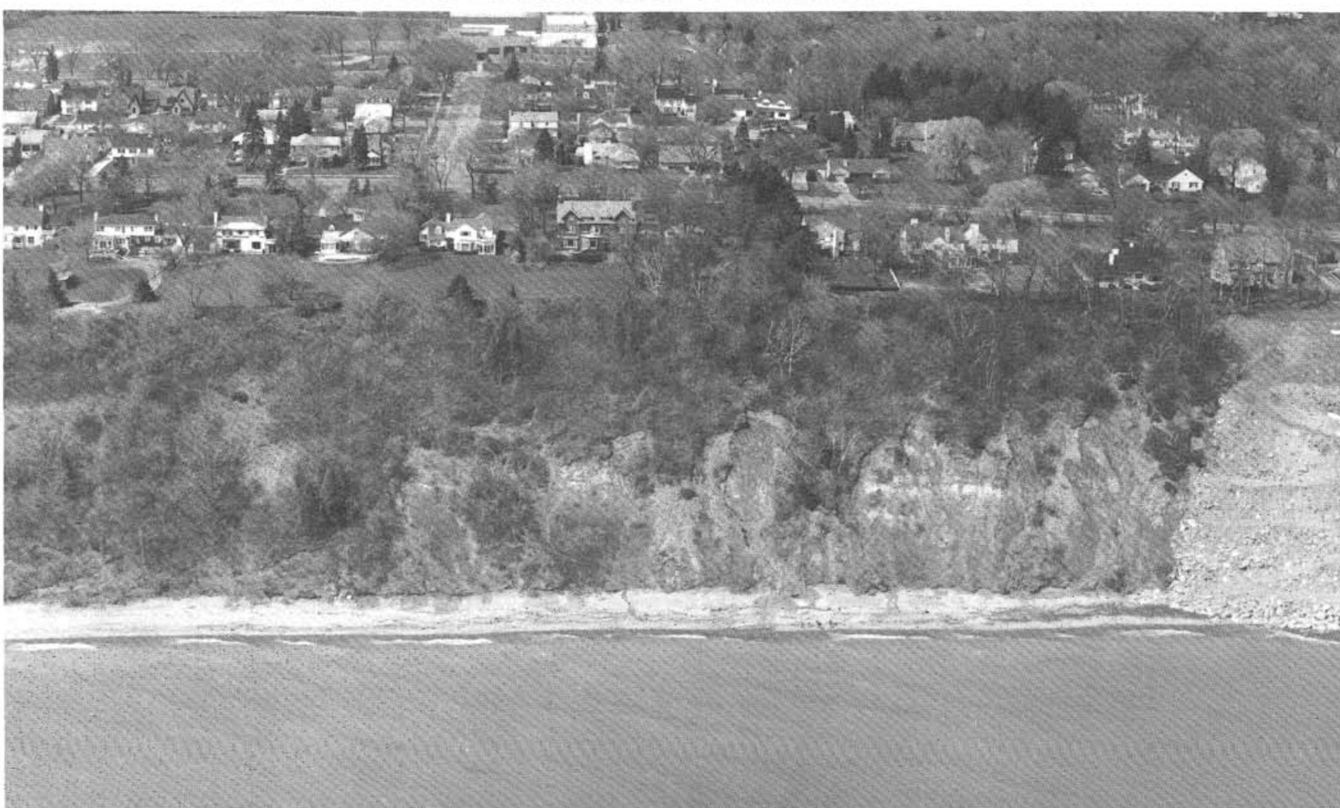
611 E. LAKE HILL COURT-6310 N. LAKE DRIVE, VILLAGE OF WHITEFISH BAY



Source: SEWRPC.

Figure A-124

6340-6440 N. LAKE DRIVE, VILLAGES OF WHITEFISH BAY AND FOX POINT



Source: SEWRPC.

Figure A-125

6448-6620 N. LAKE DRIVE, VILLAGE OF FOX POINT

Source: SEWRPC.

Figure A-126

6510-6750 N. LAKE DRIVE, VILLAGE OF FOX POINT

Source: SEWRPC.

Figure A-127

6720 N. LAKE DRIVE-6820 N. BARNETT LANE, VILLAGE OF FOX POINT

Source: SEWRPC.

Figure A-128

6828-6942 N. BARNETT LANE, VILLAGE OF FOX POINT

Source: SEWRPC.

Figure A-129

6928-7010 N. BARNETT LANE, VILLAGE OF FOX POINT

Source: SEWRPC.

Figure A-130

7004 N. BARNETT LANE-7106 N. BEACH DRIVE, VILLAGE OF FOX POINT

Source: SEWRPC.

Figure A-131

7124-7210 N. BEACH DRIVE, VILLAGE OF FOX POINT

Source: SEWRPC.

Figure A-132

7234-7415 N. BEACH DRIVE, VILLAGE OF FOX POINT

Source: SEWRPC.

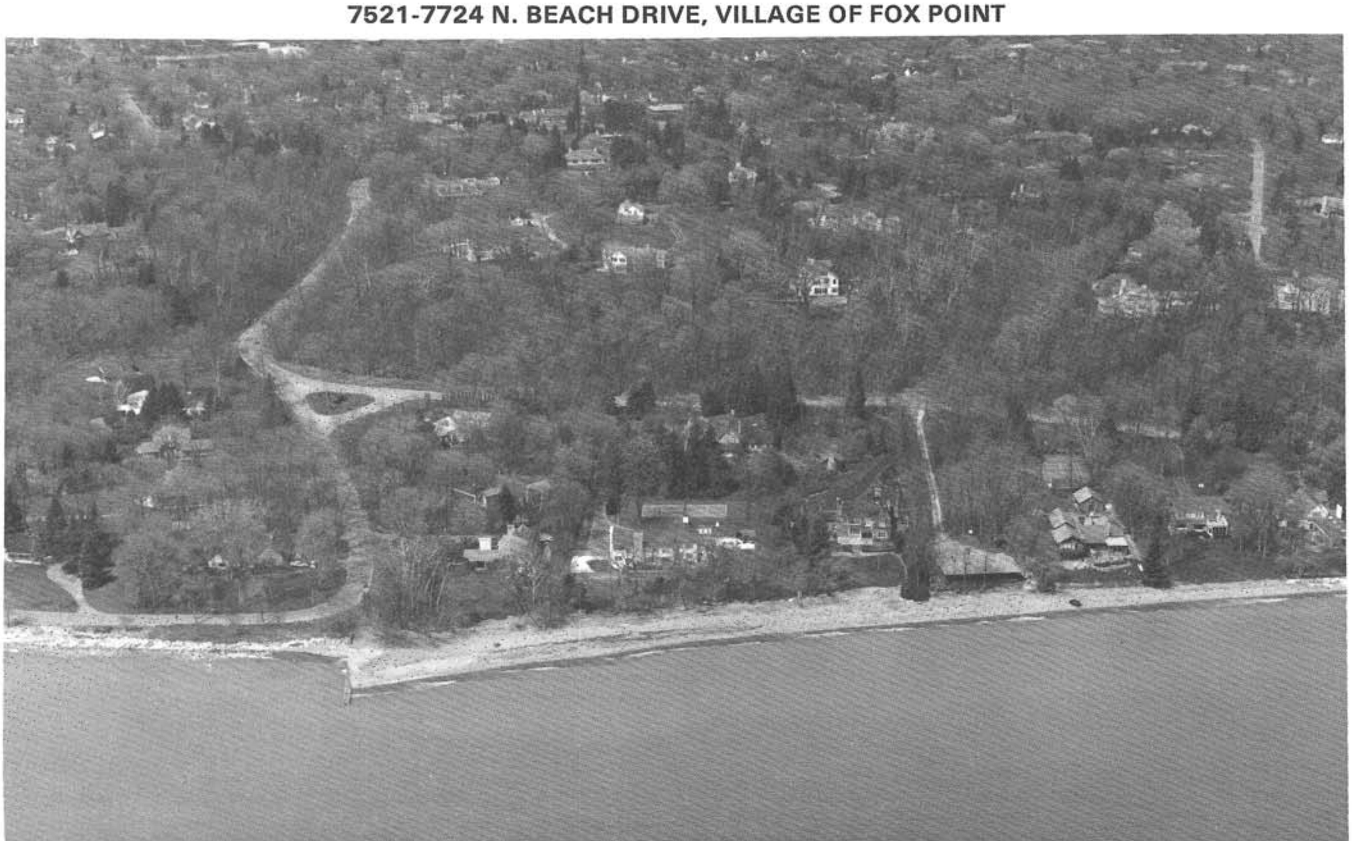
Figure A-133

7258-7481 N. BEACH DRIVE, VILLAGE OF FOX POINT

Source: SEWRPC.

Figure A-134

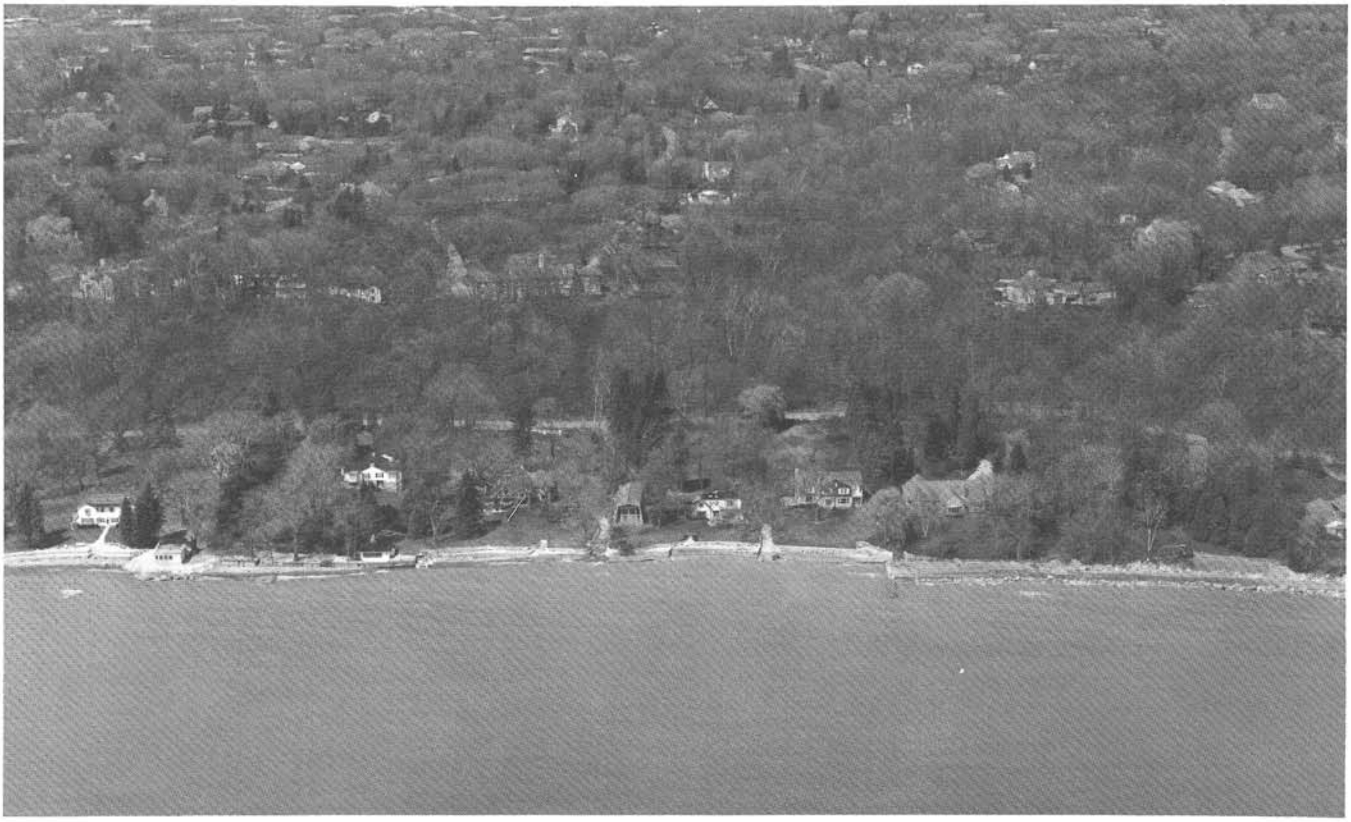
7521-7724 N. BEACH DRIVE, VILLAGE OF FOX POINT



Source: SEWRPC.

Figure A-135

7736-7930 N. BEACH DRIVE, VILLAGE OF FOX POINT



Source: SEWRPC.

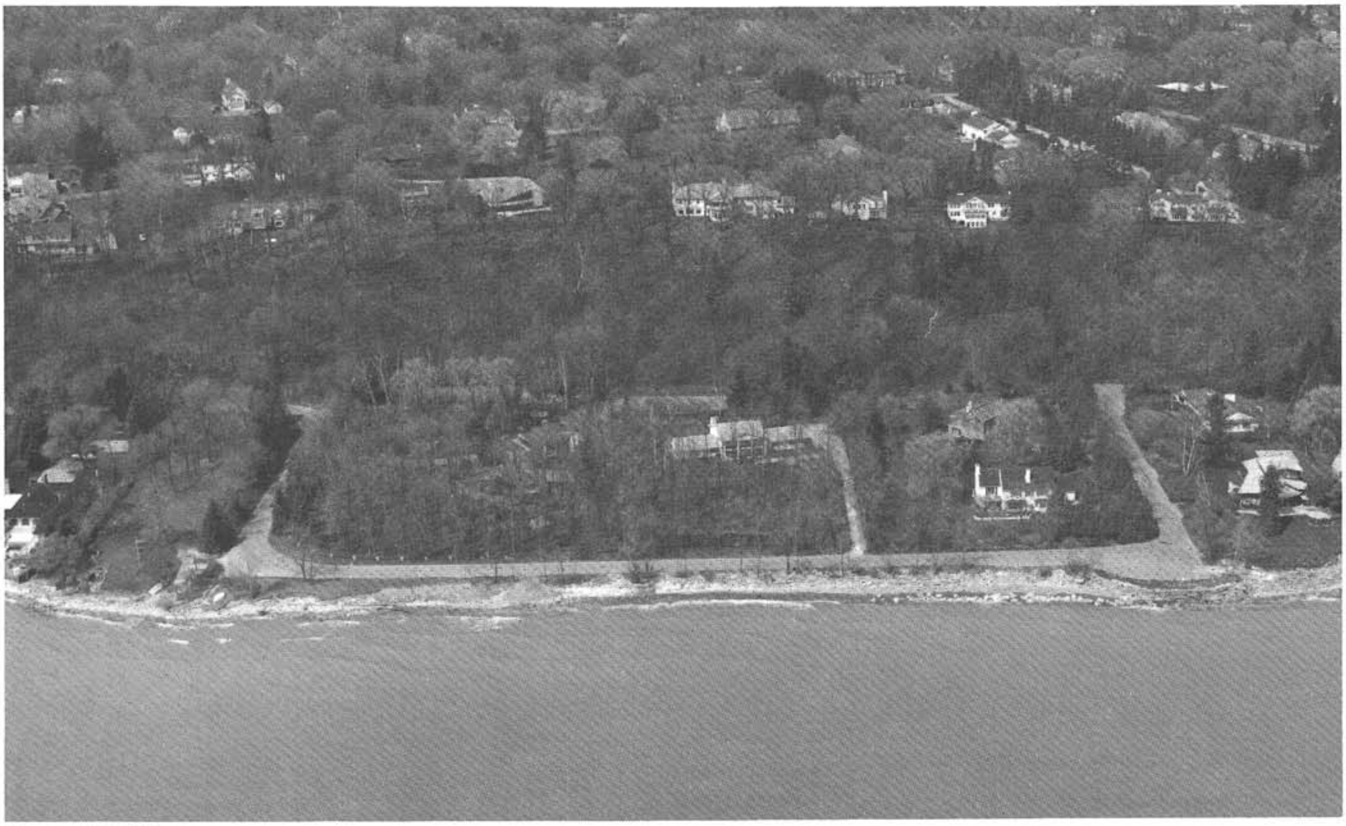
Figure A-136

7828-7954 N. BEACH DRIVE, VILLAGE OF FOX POINT

Source: SEWRPC.

Figure A-137

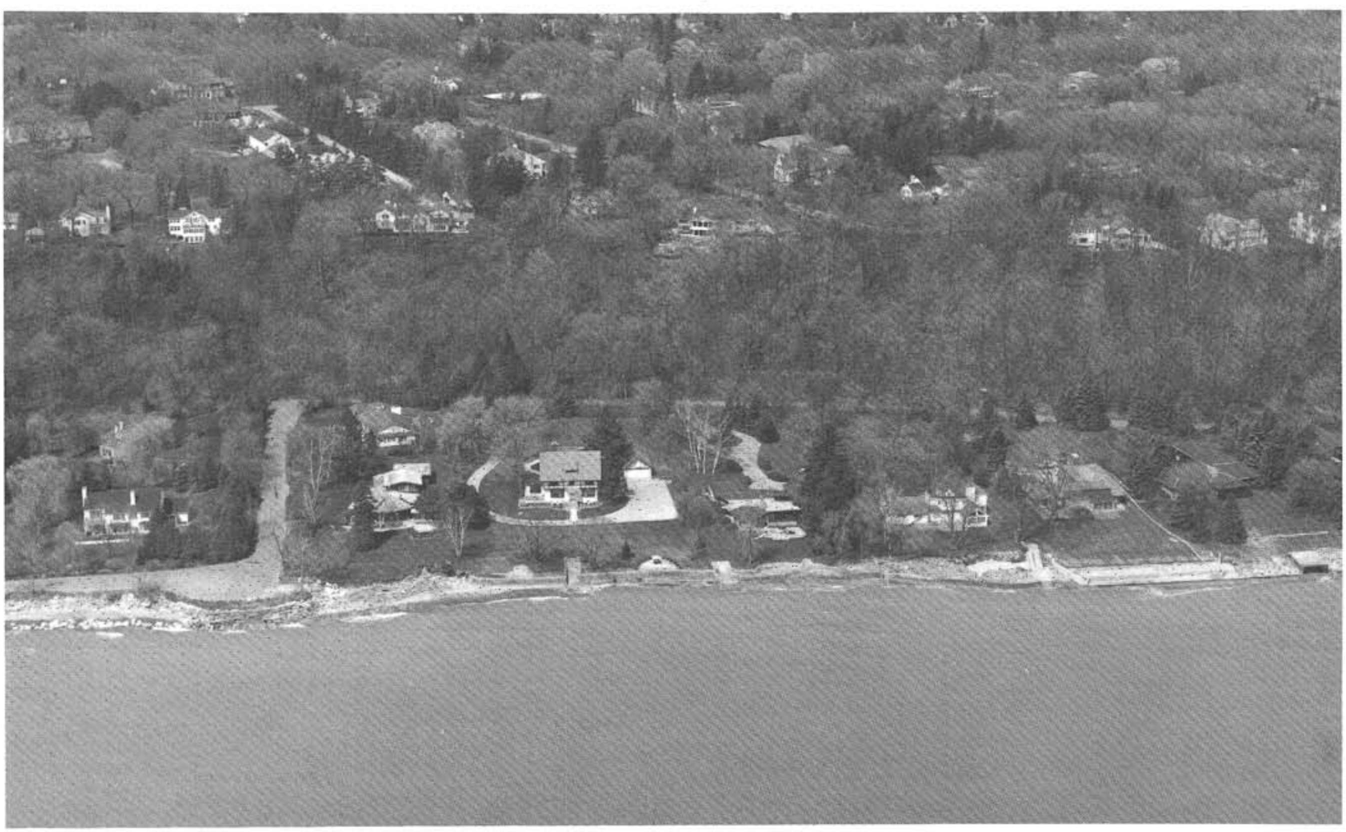
7966-8040 N. BEACH DRIVE, VILLAGE OF FOX POINT



Source: SEWRPC.

Figure A-138

8035-8110 N. BEACH DRIVE, VILLAGE OF FOX POINT



Source: SEWRPC.

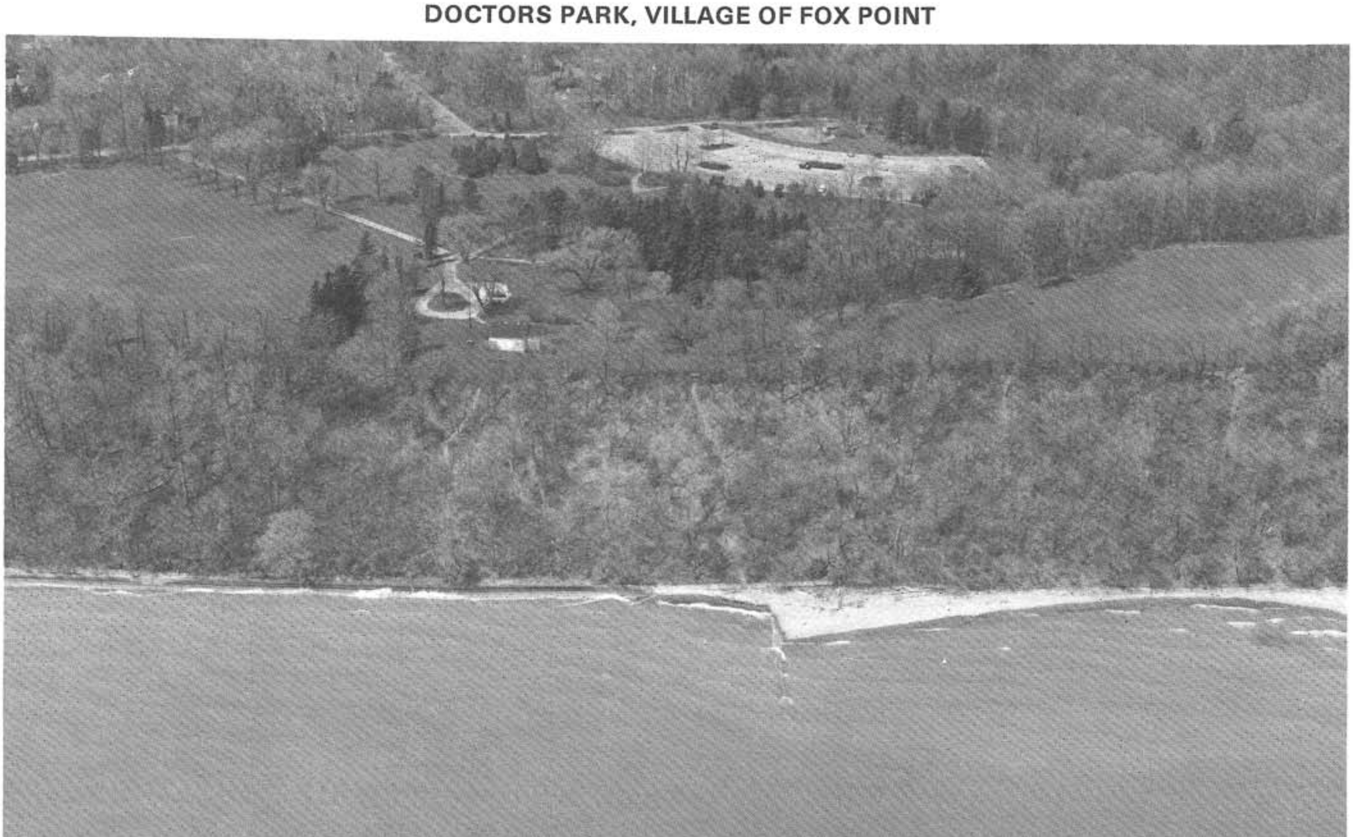
Figure A-139

8106-8130 N. BEACH DRIVE, VILLAGE OF FOX POINT

Source: SEWRPC.

Figure A-140

DOCTORS PARK, VILLAGE OF FOX POINT



Source: SEWRPC.

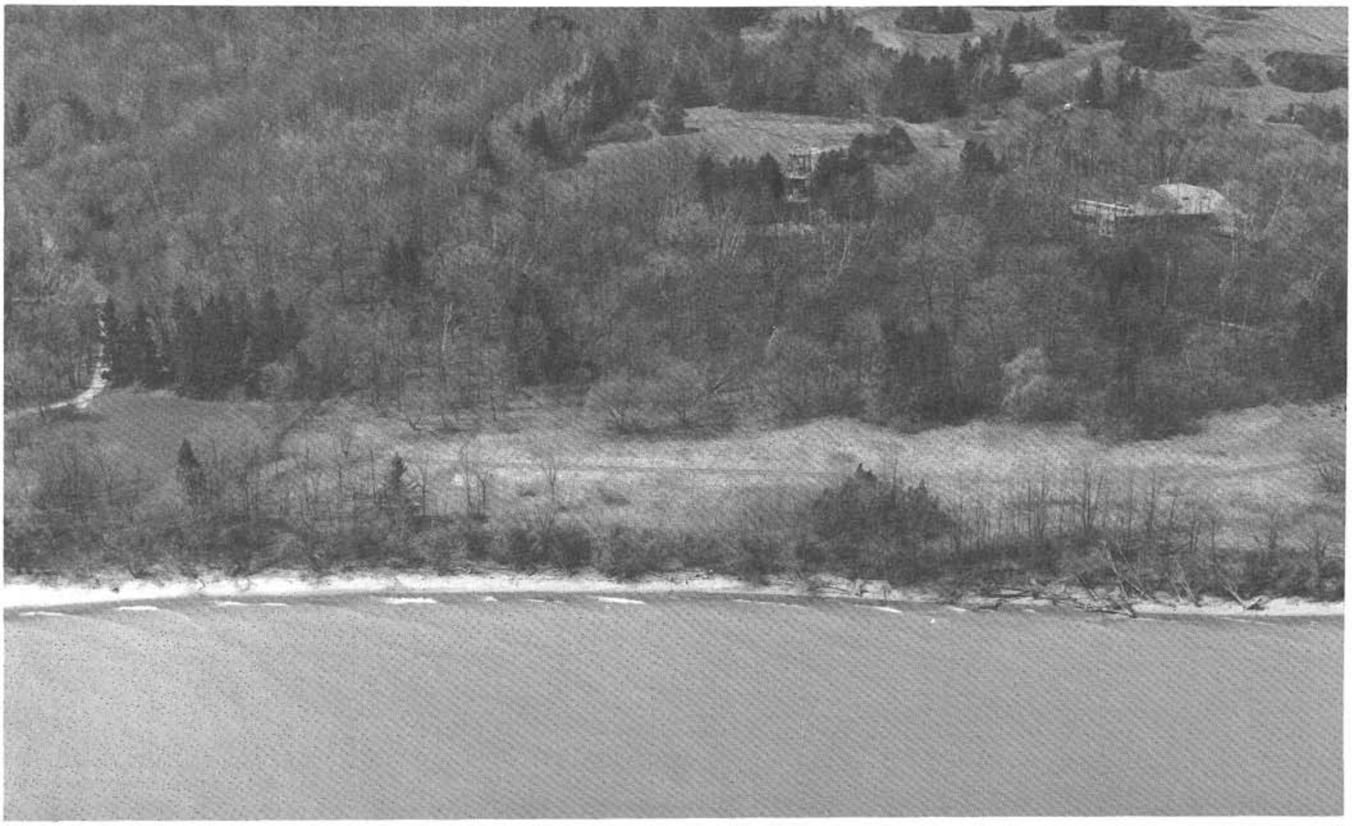
Figure A-141

DOCTORS PARK, VILLAGE OF BAYSIDE

Source: SEWRPC.

Figure A-142

DOCTORS PARK, SCHLITZ AUDUBON CENTER, VILLAGE OF BAYSIDE



Source: SEWRPC.

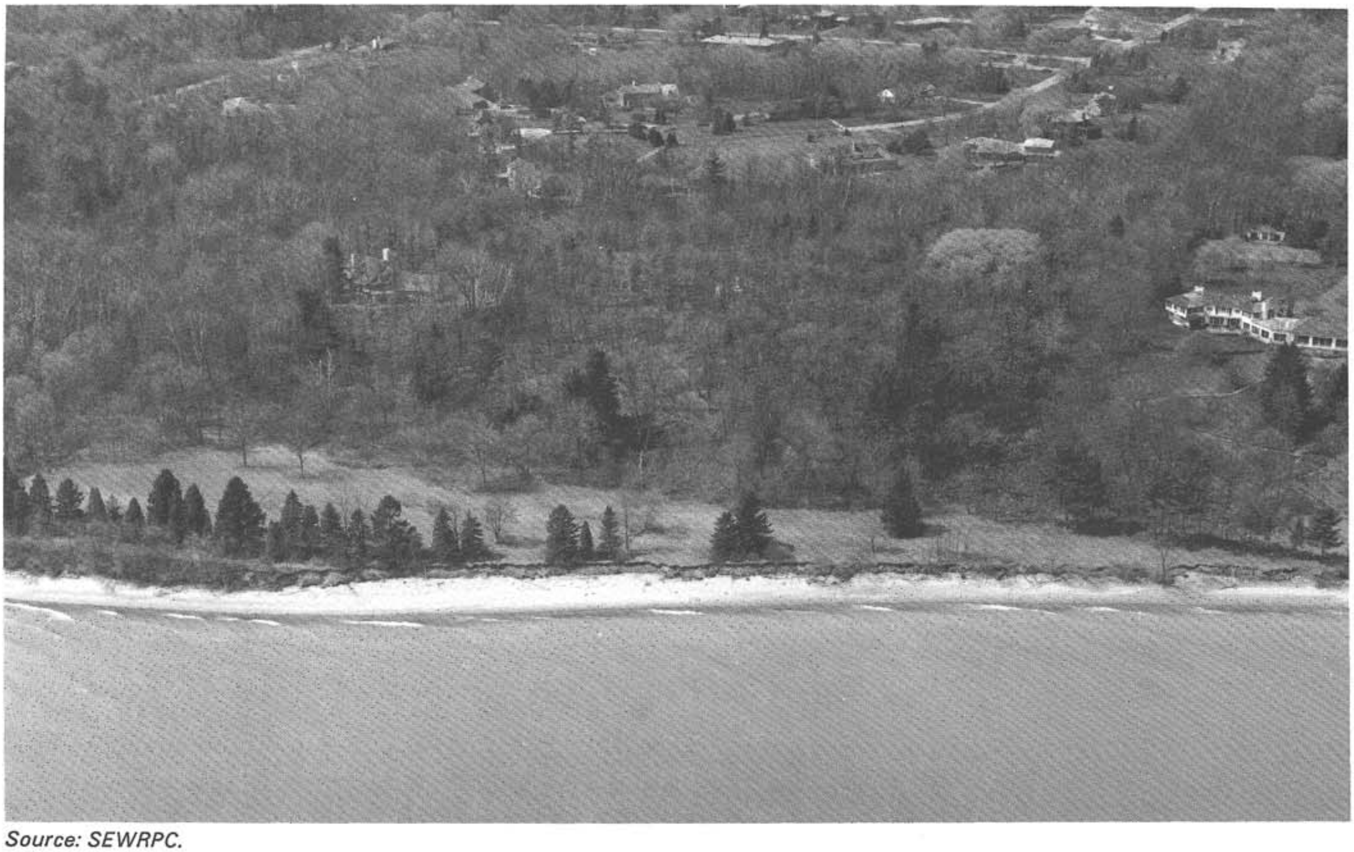
Figure A-143

SCHLITZ AUDUBON CENTER, VILLAGE OF BAYSIDE

Source: SEWRPC.

Figure A-144

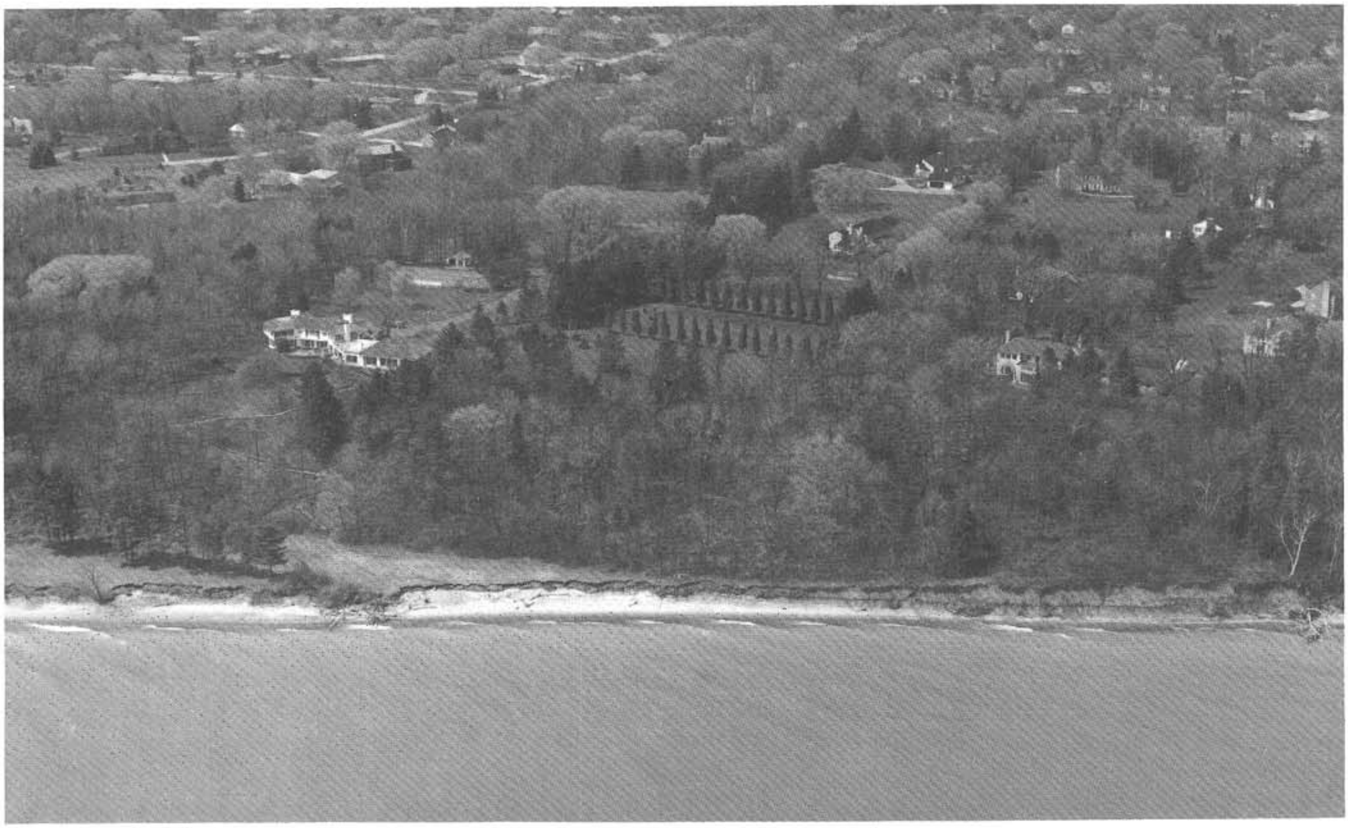
9008-9040 N. BAYSIDE DRIVE, VILLAGE OF BAYSIDE



Source: SEWRPC.

Figure A-145

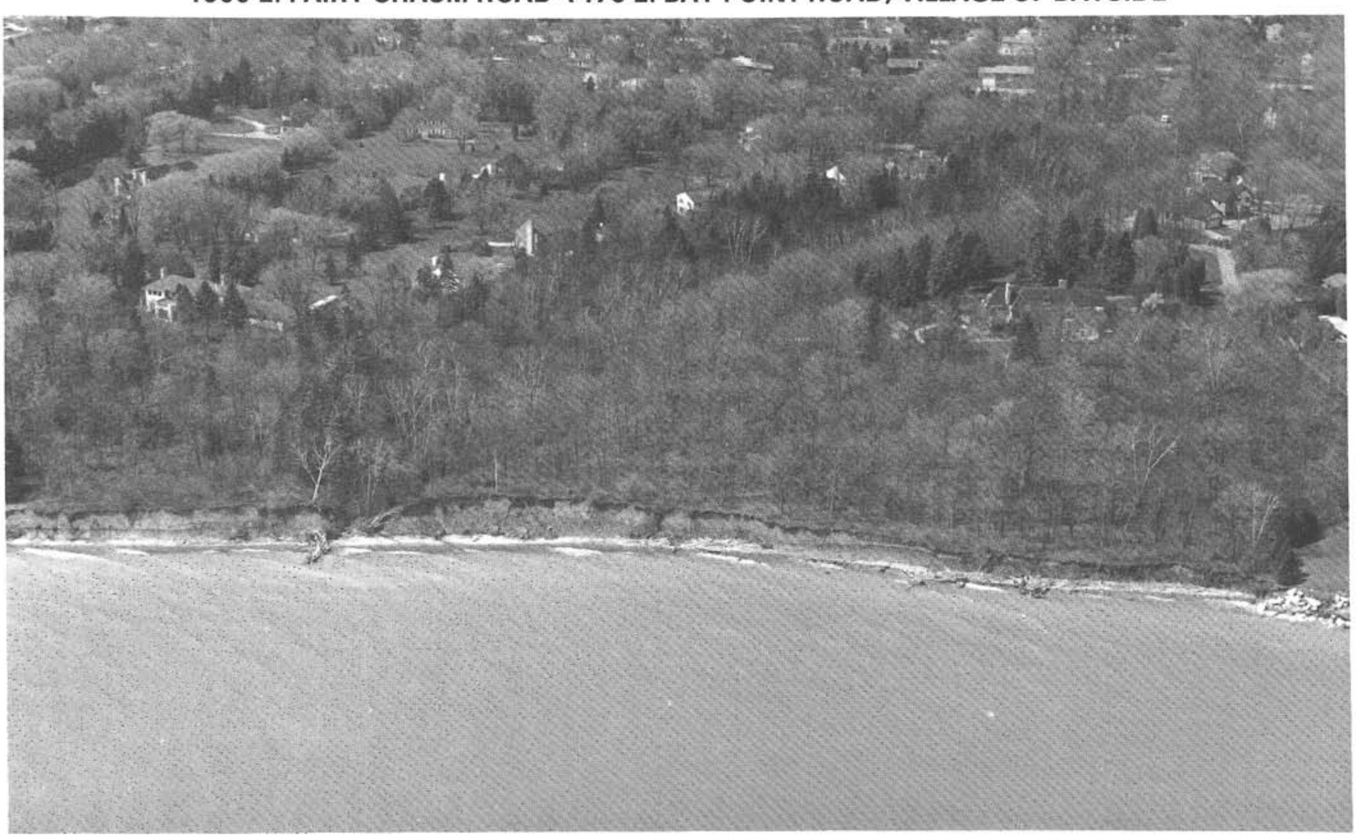
9040 N. BAYSIDE DRIVE-1500 E. FAIRY CHASM ROAD, VILLAGE OF BAYSIDE



Source: SEWRPC.

Figure A-146

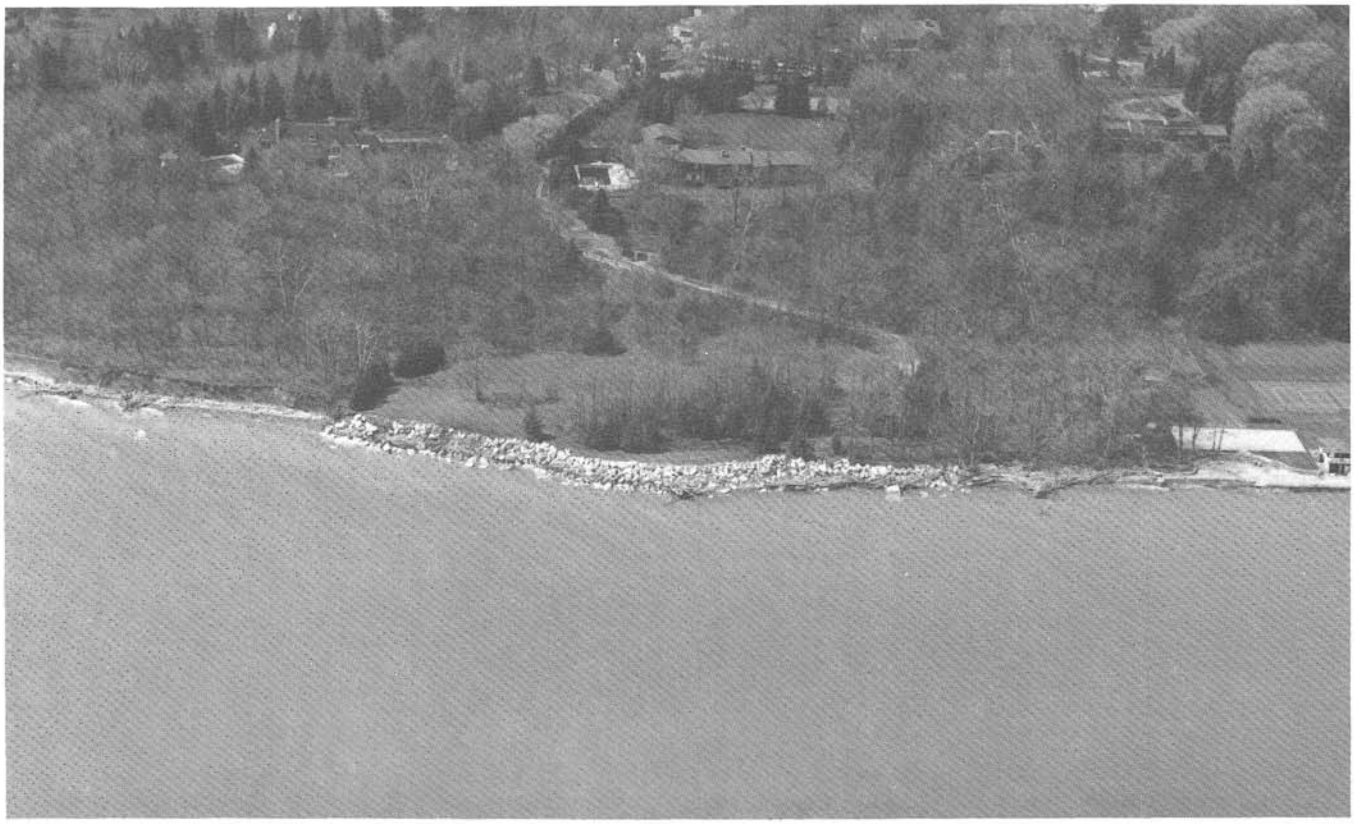
1500 E. FAIRY CHASM ROAD-1476 E. BAY POINT ROAD, VILLAGE OF BAYSIDE



Source: SEWRPC.

Figure A-147

1476-1434 E. BAY POINT ROAD, VILLAGE OF BAYSIDE



Source: SEWRPC.

Figure A-148

1470-1400 E. BAY POINT ROAD, VILLAGE OF BAYSIDE

Source: SEWRPC.

Figure A-149

9364-9400 N. LAKE DRIVE-EXTENDED, VILLAGE OF BAYSIDE

Source: SEWRPC.

Figure A-150

9364 N. LAKE DRIVE-EXTENDED-9550 N. LAKE DRIVE, VILLAGE OF BAYSIDE

Source: SEWRPC.

Figure A-151

1260 E. DONGES COURT-9578 N. LAKE DRIVE (COUNTY LINE), VILLAGE OF BAYSIDE

Source: SEWRPC.

Appendix B

INVENTORY OF SHORE PROTECTION STRUCTURES IN MILWAUKEE COUNTY: 1986-1987

Structure Number	Address	U. S. Public Land Survey Location			Structure Type	Physical Setting				Length of Structure (feet)	Material Composition of Structure	Maintenance Required	Types of Failure	Date of Construction
		Township	Range	Section		Bluff Height (feet)	Bluff Slope (degrees)	Vegetation[a]	Beach Width (feet)					
1	WEPCo Oak Creek Power Plant	5	23	31	Bulkhead	80	25	C	0	4,000	Sheet pile	Yes	Toe scour, material failure	N/A
2	9180 S. 5th Avenue	5	22	24	Revetment	76	40	NC	0	980	Concrete curb and gutter	Yes	Flanking, collapse	N/A
3	Oak Creek Water Intake Plant	5	22	24	Bulkhead	80	20	C	0	500	Sheet pile	Yes	Overtopping, flanking	N/A
4	4301 E. Depot Road-9006 S. 5th Avenue	5	22	24	Breakwater	80	45	PC	25	950	Stone	Yes	Overtopping, collapse	N/A
5	MMSD South Shore Sewage Treatment Plant	5	22	13	Bulkhead	80-90	18	C	0	3,000	Sheet pile	Yes	Overtopping	N/A
6	3303 Marina Drive	5	22	13	Bulkhead	74	22	C	0	140	Poured concrete	Yes	Toe scour, flanking, material failure	N/A
7	South Milwaukee Sewage Treatment Plant	5	22	12	Revetment	58	36	PC	0	400	Stone	Yes	Overtopping, material failure	N/A
8	South Milwaukee Yacht Club	5	22	12	Revetment	55	15	C	0	1,150	Concrete slabs	No	- -	N/A
9	Grant Park	5	22	12	Groin	48	24	C	300	60	Stone	No	- -	1952
10	Grant Park	5	22	12	Groin	97	35	PC	60	225	Precast concrete	Yes	Overtopping, collapse, material failure	1934
11	Grant Park	5	22	1	Groin	93	30	PC	60	60	Precast concrete	Yes	Overtopping	N/A
12	Warnimont Park	6	22	25	Groin	104	38	PC	60	80	Precast concrete	Yes	Overtopping, collapse, material failure	1944
13	Cudahy Water Intake Plant	6	22	25	Bulkhead	100	22	C	0	320	Poured concrete	Yes	Overtopping, flanking, toe scour	1974
14	Sheridan Park	6	22	25	Groin	93	30	C	100	80	Precast concrete	Yes	Overtopping, collapse, material failure	1933-34
15	Former WEPCo Lakeside Power Plant	6	22	14	Revetment	50	22	C	0	800	Stone	Yes	Overtopping, collapse	Mid-1930's
16	Former WEPCo Lakeside Power Plant	6	22	14	Breakwater	45	22	C	0	1,750	Stone	No	- -	1927
17	Former WEPCo Lakeside Power Plant	6	22	14	Revetment	57	25	C	0	900	Stone	Yes	Overtopping, collapse	Mid-1930's
18	South Shore Park	6	22	10	Revetment	45	30	C	0	1,000	Stone	No	- -	1956
19	Texas Street Water Intake Plant	6	22	10	Revetment	50	22	C	0	350	Stone	Yes	Overtopping, collapse, toe scour	1956
20	South Shore Breakwater	6	22	10/14	Breakwater	- -	- -	- -	- -	11,500	Stone and timber crib	Yes	Overtopping	1913-1936
21	South Shore Park	6	22	10	Revetment	48	40	C	0	1,300	Stone	Yes	Overtopping	N/A
22	South Shore Park	6	22	10	Revetment	15-30	25	C	0	500	Stone	No	- -	1987
23	South Shore Yacht Club	6	22	10	Bulkhead	30	25	C	0	850	Sheet pile	No	- -	N/A
24	South Shore Park	6	22	10	Revetment	30	30	C	0	150	Stone	Yes	Overtopping	N/A
25	U. S. Coast Guard Station	6	22	10	Bulkhead	- -	- -	- -	0	900	Sheet pile	No	- -	N/A
26	U. S. Army Corps of Engineers Dredged Spoils Confined Disposal Facility	6	22	3/4	Revetment	- -	- -	- -	0	3,500	Stone	No	- -	N/A
27	S. Lincoln Memorial Drive	6	22	4	Bulkhead	- -	- -	- -	0	3,300	Sheet pile	Yes	Overtopping	N/A
28	Port of Milwaukee Slips	7	22	33	Bulkhead	- -	- -	- -	0	6,800	Sheet pile	No	- -	N/A
29	MMSD Jones Island Sewage Treatment Plant	7	22	33	Bulkhead	- -	- -	- -	0	2,000	Sheet pile	Yes	Overtopping	N/A

Appendix B (continued)

Structure Number	Address	U. S. Public Land Survey Location			Structure Type	Physical Setting				Length of Structure (feet)	Material Composition of Structure	Maintenance Required	Types of Failure	Date of Construction
		Township	Range	Section		Bluff Height (feet)	Bluff Slope (degrees)	Vegetation[a]	Beach Width (feet)					
30	Marcus Amphitheatre	7	22	33	Bulkhead	--	--	--	0	1,850	Steel sheet pile	Yes	Overtopping	N/A
31	Henry W. Maier Festival Property	7	22	33	Revetment	--	--	--	0	2,400	Stone	No	--	N/A
32	Henry W. Maier Festival Property	7	22	28	Bulkhead	--	--	--	0	900	Stone	Yes	Overtopping, collapse	N/A
33	Municipal Pier	7	22	28	Bulkhead	--	--	--	0	1,400	Steel sheet pile	No	--	N/A
34	South of War Memorial Center	7	22	28	Bulkhead	--	--	--	0	500	Stone	No	--	N/A
35	War Memorial Center	7	22	28	Bulkhead	--	--	--	0	1,300	Concrete	Yes	Toe scour	N/A
36	Juneau Park and McKinley Marina	7	22	28	Bulkhead	--	--	--	0	5,300	Steel sheet pile	No	--	N/A
37	Outer Harbor Breakwater North of North Entrance	7	22	22	Breakwater	--	--	--	0	3,000	Concrete	Yes	Overtopping, collapse, material failure	N/A
38	Outer Harbor Breakwater	7	22	27/34	Breakwater	--	--	--	0	16,000	Stone	Yes	Overtopping	1877-1929
39	McKinley Beach	7	22	22	Revetment	--	--	--	0	2,500	Stone	No	--	1987-in progress
40	McKinley Park	7	22	22	Revetment	--	--	--	0	500	Stone	Yes	Overtopping, collapse	1929
41	Lake Park	7	22	14/15	Revetment	--	--	--	0	3,500	Stone	Yes	Overtopping, collapse	1929
42	Lake Park	7	22	14	Revetment	--	--	--	0	800	Stone	Yes	Overtopping	1929
43	Linnwood Avenue Water Treatment Plant	7	22	14	Bulkhead	--	--	--	0	2,000	Steel sheet pile	Yes	Overtopping	1934
44	3224 E. Hampshire Street	7	22	10	Bulkhead	74	17	C	60	370	Poured concrete	Yes	Material failure	Pre-1945
45	3252 N. Lake Drive	7	22	10	Revetment	78	20	C	<5	160	Stone	Yes	Overtopping, flanking, collapse	N/A
46	3318-3322 N. Lake Drive	7	22	10	Bulkhead	82	16	C	<5	200	Poured concrete	Yes	Overtopping, flanking	Pre-1945
47	3318-3322 N. Lake Drive	7	22	10	Groin	82	16	C	<5	85	Concrete	Yes	Overtopping	1929
48	3063 E. Newport Court	7	22	10	Bulkhead	88	18	C	10	350	Precast concrete	Yes	Overtopping, flanking, collapse	Pre-1945
49	3432 N. Lake Drive	7	22	10	Bulkhead	100	15	C	<5	75	Concrete	Yes	Collapse	Pre-1945
50	3444 N. Lake Drive	7	22	10	Bulkhead	102	26	C	<5	50	Concrete	Yes	Collapse	Pre-1945
51	3474 N. Lake Drive	7	22	10	Bulkhead	103	16	C	<5	275	Concrete	Yes	Overtopping, flanking, collapse	Pre-1945
52	3510 N. Lake Drive	7	22	10	Bulkhead	98	29	C	<5	150	Concrete	Yes	Flanking, material failure	Pre-1945
53	3534 N. Lake Drive	7	22	10	Revetment	96	19	PC	<5	250	Concrete	No	--	N/A
54	3534 N. Lake Drive	7	22	10	Breakwater	96	19	PC	<5	225	Stone	Yes	Overtopping	1929
55	3562 N. Lake Drive	7	22	10	Bulkhead	106	20	C	<5	80	Precast concrete	Yes	Overtopping, flanking, material failure	1920
56	3580 N. Lake Drive	7	22	10	Bulkhead	105	16	C	<5	85	Concrete	Yes	Collapse	1920
57	3900 N. Lake Drive	7	22	10	Bulkhead	106	19	C	15	200	Poured concrete	Yes	Collapse	1920
58	3926 N. Lake Drive	7	22	10	Revetment	100	24	C	<5	125	Precast concrete	Yes	Overtopping, collapse	1977
59	3932-3966 N. Lake Drive	7	22	10	Bulkhead	107	29	C	<5	400	Concrete/grout-filled bags	Yes	Overtopping, material failure, collapse	N/A
60	3966-4060 N. Lake Drive	7	22	3	Groin	88	26	C	100	1,150	Concrete	Yes	Collapse, overtopping, material failure	1933

Appendix B (continued)

Structure Number	Address	U. S. Public Land Survey Location			Structure Type	Physical Setting				Length of Structure (feet)	Material Composition of Structure	Maintenance Required	Types of Failure	Date of Construction
		Township	Range	Section		Bluff Height (feet)	Bluff Slope (degrees)	Vegetation[a]	Beach Width (feet)					
61	4120-4130 N. Lake Drive	7	22	3	Groin	103	27	C	60	95	Sheet pile/concrete	Yes	Material failure	Pre-1975
62	4400-4408 N. Lake Drive	7	22	3	Bulkhead	111	29	C	<5	200	Concrete/grout-filled bags	Yes	Overtopping, flanking	Pre-1945
63	4442-4668 N. Lake Drive	7	22	3	Revetment	115	33	PC	<5	2,200	Stone	No	- -	1986
64	4676 N. Lake Drive	7	22	3	Bulkhead	102	25	PC	<5	100	Poured concrete/timber	Yes	Flanking	1981
65	4700 N. Lake Drive	7	22	3	Bulkhead	94	38	PC	<5	100	Poured concrete	Yes	Overtopping, flanking, material failure	1981
66	4720 N. Lake Drive	7	22	3	Revetment	97	35	NC	<5	100	Limestone/grout-filled bags	No	- -	1986
67	4762 N. Lake Drive	8	22	34	Revetment	96	33	NC	<5	100	Limestone/grout-filled bags	No	- -	1986
68	4790-4800 N. Lake Drive	8	22	34	Revetment	93	36	NC	<5	300	Stone/grout-filled bags	No	- -	1985
69	4850-4870 N. Lake Drive	8	22	34	Bulkhead	83	31	NC	<5	210	Concrete	No	- -	1986
70	4890-4940 N. Lake Drive	8	22	34	Revetment	71	31	C	<5	600	Concrete/grout-filled bags	Yes	Overtopping, collapse	1976
71	Buckley Park	8	22	34	Bulkhead	66	31	C	<5	350	Concrete	Yes	Overtopping, flanking, material failure	1946
72	Big Bay Park	8	22	33	Groin	73	19	C	40	400	Precast concrete	Yes	Overtopping	1956
73	Big Bay Park	8	22	33	Bulkhead	64	26	C	<5	350	Precast concrete	Yes	Toe scour, overtopping material failure	1943
74	Big Bay Park	8	22	33	Bulkhead	70	25	C	<5	750	Precast concrete	Yes	Overtopping, material failure	1954
75	1400-1500 E. Henry Clay	8	22	33	Revetment	76	29	PC	<5	655	Concrete	No	- -	1982
76	5220-5240 N. Lake Drive	8	22	33	Revetment	72	19	PC	<5	350	Concrete/stone	No	- -	1981
77	5270 N. Lake Drive	8	22	33	Bulkhead	68	17	C	<5	220	Concrete	Yes	Toe scour, flanking, collapse	Pre-1975
78	5312-5570 N. Lake Drive	8	22	33	Revetment	82	19	C	<5	1,700	Concrete/stone	Yes	Collapse	1978
79	5570-5616 N. Lake Drive	8	22	28	Revetment	82	22	NC	<5	370	Concrete/stone	No	- -	1981
80	5626 N. Lake Drive-808 E. Lakeview Drive	8	22	28	Revetment	88	17	NC	10	840	Concrete	No	- -	1986
81	5866 N. Lake Drive	8	22	28	Bulkhead	84	29	C	15	50	Precast concrete	Yes	Overtopping, flanking	1943
82	Klode Park	8	22	28	Bulkhead	74	20	C	20	480	Precast concrete	Yes	Overtopping, flanking	1943
83	6430-6448 N. Lake Drive	8	22	21	Revetment	130	28	NC	<5	240	Concrete	No	- -	1986
84	6530 N. Lake Drive	8	22	21	Revetment	130	31	C	<5	400	Stone	Yes	Overtopping	1972
85	6530-6620 N. Lake Drive	8	22	21	Groin	126	32	C	30	425	Precast concrete	Yes	Material failure	1972
86	6720-6818 N. Barnett Lane	8	22	21	Revetment	120	30	PC	5	700	Grout-filled bags	No	- -	1972
87	6880 N. Barnett Lane	8	22	21	Groin	108	26	C	10	35	Precast concrete	Yes	Material failure	Pre-1975
88	7000 N. Beach Drive	8	22	21	Bulkhead	- -	- -	- -	<5	80	Stone	No	- -	1986
89	7000 N. Beach Drive	8	22	21	Groin	- -	- -	- -	<5	50	Concrete blocks	Yes	Overtopping	N/A
90	7000 N. Beach Drive	8	22	21	Revetment	- -	- -	- -	<5	25	Stone	Yes	Overtopping	1986
91	7038 N. Beach Drive	8	22	21	Bulkhead	- -	- -	- -	<5	160	Concrete	Yes	Overtopping	1982
92	7106 N. Beach Drive	8	22	21	Bulkhead	- -	- -	- -	<5	220	Poured concrete	Yes	Overtopping, material failure	1976
93	7120 N. Beach Drive	8	22	21	Revetment	- -	- -	- -	<5	90	Concrete slabs	No	- -	1986
94	7124 N. Beach Drive	8	22	21	Bulkhead	- -	- -	- -	<5	125	Concrete blocks	No	- -	1986
95	7134 N. Beach Drive	8	22	21	Revetment	- -	- -	- -	<5	100	Concrete slabs and blocks	Yes	Overtopping	Pre-1975
96	7152-7200 N. Beach Drive	8	22	21	Bulkhead	- -	- -	- -	<5	270	Concrete blocks, stone	Yes	Overtopping, flanking, collapse	1974

Appendix B (continued)

Structure Number	Address	U. S. Public Land Survey Location			Structure Type	Physical Setting				Length of Structure (feet)	Material Composition of Structure	Maintenance Required	Types of Failure	Date of Construction
		Township	Range	Section		Bluff Height (feet)	Bluff Slope (degrees)	Vegetation[a]	Beach Width (feet)					
97	7210 N. Beach Drive	8	22	16	Bulkhead	- -	- -	- -	<5	170	Rubble-filled steel crib	Yes	Overtopping	1973
98	7210 N. Beach Drive	8	22	16	Groin	- -	- -	- -	<5	175	Poured concrete/steel	Yes	Overtopping, flanking	N/A
99	7228 N. Beach Drive	8	22	16	Bulkhead	- -	- -	- -	<5	230	Poured concrete	Yes	Material failure, toe scour, overtopping	1975
100	7234-7240 N. Beach Drive	8	22	16	Revetment	- -	- -	- -	<5	165	Stone	No	- -	Pre-1975
101	7242-7250 N. Beach Drive	8	22	16	Revetment	- -	- -	- -	<5	200	Concrete	Yes	Overtopping, collapse	Pre-1975
102	7254-7328 N. Beach Drive	8	22	16	Bulkhead	- -	- -	- -	<5	200	Concrete block	Yes	Overtopping	1985
103	North of 7328 N. Beach Drive	8	22	16	Bulkhead	- -	- -	- -	<5	65	Concrete slabs, cut stone	Yes	Overtopping, flanking	1974
104	7400-7535 N. Beach Drive	8	22	16	Revetment	- -	- -	- -	<5	1,450	Concrete block and slabs	Yes	Overtopping	1950
105	7540-7710 N. Beach Drive	8	22	16	Groin	- -	- -	- -	<5	560	Concrete	Yes	Overtopping	1945
106	7718-7736 N. Beach Drive	8	22	16	Bulkhead	- -	- -	- -	<5	800	Timber	Yes	Overtopping, material failure	N/A
107	7810 N. Beach Drive	8	22	16	Bulkhead	- -	- -	- -	<5	160	Timber/concrete	Yes	Toe scour, overtopping, material failure	1972
108	7818-7834 N. Beach Drive	8	22	16	Bulkhead	- -	- -	- -	10	400	Stone slabs cemented	No	- -	1982
109	7900-7912 N. Beach Drive	8	22	16	Groin	- -	- -	- -	30	240	Concrete	Yes	Overtopping, material failure	N/A
110	7930 N. Beach Drive	8	22	16	Revetment	- -	- -	- -	<5	175	Concrete slabs/stone	Yes	Overtopping	1971
111	7938 N. Beach Drive	8	22	16	Bulkhead	- -	- -	- -	<5	150	Concrete	Yes	Overtopping	1970
112	7944 N. Beach Drive	8	22	16	Bulkhead	- -	- -	- -	<5	110	Concrete	No	- -	N/A
113	7954 N. Beach Drive	8	22	16	Bulkhead	- -	- -	- -	<5	145	Stone slabs	Yes	Overtopping	1980
114	7966-8035 N. Beach Drive	8	22	16	Revetment/ Bulkhead	- -	- -	- -	<5	720	Cut stone, rubble material failure	Yes	Overtopping, collapse	N/A
115	North of 3035 N. Beach Drive	8	22	10	Bulkhead	- -	- -	- -	<5	105	Concrete slabs, stone	Yes	Flanking, collapse	1941
116	8064 N. Beach Drive	8	22	10	Bulkhead	- -	- -	- -	<5	190	Concrete	Yes	Toe scour, overtopping, flanking	N/A
117	8030 N. Beach Drive	8	22	10	Bulkhead	- -	- -	- -	<5	110	Stone slabs	No	- -	1973
118	8090 N. Beach Drive	8	22	10	Bulkhead	- -	- -	- -	<5	130	Concrete	Yes	Overtopping	1972
119	8100-8120 N. Beach Drive	8	22	10	Bulkhead	- -	- -	- -	<5	250	Concrete	Yes	Overtopping	1975-1985
120	8120 N. Beach Drive	8	22	10	Revetment	- -	- -	- -	<5	90	Cut stone	Yes	Overtopping	1986
121	8130 N. Beach Drive	8	22	10	Revetment	- -	- -	- -	<5	350	Cut stone	No	- -	1986
122	Doctors Park	8	22	10	Bulkhead	92	25	C	<5	570	Concrete	Yes	Overtopping, flanking, material failure	Pre-1975
123	Doctors Park	8	22	10	Groin	92	25	C	20	40	Concrete	Yes	Overtopping	1964
124	Doctors Park	8	22	10	Groin	94	29	C	70	70	Precast concrete	Yes	Overtopping, collapse	1915
125	1470 E. Bay Point Road	8	22	4	Revetment	83	13	C	0	450	Stone	Yes	Collapse	N/A
126	1434 E. Bay Point Road	8	22	4	Bulkhead	89	12	C	0	250	Concrete slabs	Yes	Overtopping, toe scour	N/A
127	1240 E. Donges Court	8	22	4	Bulkhead	74	37	NC	20	100	Poured concrete	Yes	Flanking, toe scour	1978
128	9560 N. Lake Drive	8	22	4	Revetment	87	34	PC	20	100	Precast concrete blocks	Yes	Collapse	N/A

NOTE: N/A indicates data not available.

[a]*C - Bluff face covered with vegetation.*
PC - Bluff face partly covered with vegetation.
NC - Bluff face not covered with vegetation.

Source: SEWRPC.

Appendix C

SHORELINE RECESSION RATES ALONG THE LAKE MICHIGAN SHORELINE OF MILWAUKEE

Civil Division	Bluff Analysis Sections	Shoreline Recession Reaches	Shoreline Length (feet)	Bluff Height (feet)	Annual Recession Rates (feet/year)		Annual Volume of Shore Material Loss (cubic feet per year)
					Long Term (1936-1985)	Short Term (1963-1985)	
City of Oak Creek	1	1	220	95	2.0	<0.5	- -
		2	220	96	- -	<0.5	- -
		3	240	94	- -	<0.5	- -
		4	220	94	- -	<0.5	- -
		5	220	94	- -	<0.5	- -
		6	220	94	- -	<0.5	- -
		7	210	94	- -	<0.5	- -
		8	480	94	- -	<0.5	- -
		9	450	94	- -	<0.5	- -
		10	220	80	- -	<0.5	- -
		11	220	62	- -	<0.5	- -
		12	220	62	- -	<0.5	- -
		13	980	62	- -	<0.5	- -
		14	350	86	- -	<0.5	- -
	2	15	260	98	- -	1.0	25,500
		16	280	106	- -	1.0	29,700
		17	240	104	- -	<0.5	- -
		18	250	104	- -	2.5	65,000
		19	250	106	- -	3.0	79,500
		20	230	116	- -	2.0	53,400
		21	220	118	- -	3.0	77,900
		22	220	110	- -	2.5	60,500
		23	210	104	- -	4.5	98,300
		24	210	110	- -	4.5	103,900
		25	240	110	- -	4.5	118,800
		26	210	102	- -	<0.5	- -
	3	27	220	104	4.0	1.0	22,900
		28	220	112	- -	1.0	24,600
		29	210	112	- -	2.5	58,800
		30	220	112	- -	4.0	98,600
		31	210	112	- -	2.0	47,000
		32	210	114	- -	3.0	71,800
		33	210	104	- -	3.0	65,500
		34	210	100	- -	3.0	63,000
		35	210	108	- -	8.5	192,800
		36	210	112	- -	4.5	105,800
		37	210	114	- -	3.0	71,800
		38	220	110	- -	4.0	96,800
		39	220	104	- -	5.5	125,800
		40	210	102	- -	5.0	107,100
	4	41	210	100	- -	3.5	73,500
		42	220	100	- -	1.5	33,000
		43	220	102	- -	3.0	67,300
		44	210	104	- -	2.5	54,600
		45	210	104	- -	2.5	54,600
		46	210	98	- -	2.5	51,400
		47	220	94	- -	3.5	72,400
		48	210	90	- -	4.5	85,000
		49	210	82	- -	8.0	137,800

Appendix C (continued)

Civil Division	Bluff Analysis Sections	Shoreline Recession Reaches	Shoreline Length (feet)	Bluff Height (feet)	Annual Recession Rates (feet/year) Long Term (1936-1985)	Annual Recession Rates (feet/year) Short Term (1963-1985)	Annual Volume of Shore Material Loss (cubic feet per year)
City of Oak Creek (continued)	5	50	220	86	- -	3.0	56,800
		51	210	86	- -	5.0	90,300
		52	210	82	- -	8.0	51,700
		53	210	76	4.5	5.5	87,800
		54	220	72	- -	10.5	166,300
	6	55	220	74	- -	11.0	179,100
		56	230	72	- -	11.0	182,200
		57	240	76	- -	12.5	228,000
		58	220	72	- -	11.5	182,200
		59	260	72	- -	12.5	234,000
	7	60	220	76	- -	5.0	83,600
		61	210	76	- -	1.5	23,900
		62	230	86	- -	1.5	29,700
		63	230	88	- -	<0.5	- -
		64	220	90	- -	<0.5	- -
	8	65	210	80	- -	<0.5	- -
		66	250	80	- -	<0.5	- -
	9	67	220	82	- -	0.5	9,000
		68	220	84	- -	0.5	9,200
		69	220	84	- -	0.5	9,200
	10	70	210	84	- -	1.0	17,600
		71	250	82	- -	<0.5	- -
	11	72	220	80	- -	<0.5	- -
		73	240	72	- -	<0.5	- -
		74	220	76	- -	0.5	8,400
		75	220	74	- -	<0.5	- -
		76	210	78	- -	1.5	24,600
	12	77	310	76	- -	<0.5	- -
		78	410	80	- -	<0.5	- -
		79	250	80	- -	<0.5	- -
		80	200	80	1.0	<0.5	- -
		81	200	82	- -	<0.5	- -
		82	200	84	- -	<0.5	- -
		83	200	82	- -	<0.5	- -
		84	200	82	- -	<0.5	- -
		85	480	82	- -	<0.5	- -
		86	680	86	- -	<0.5	- -
		87	200	90	- -	<0.5	- -
	13	88	200	110	- -	<0.5	- -
		89	200	110	- -	<0.5	- -
		90	200	110	- -	<0.5	- -
		91	200	90	- -	<0.5	- -
		92	200	80	- -	<0.5	- -
City of South Milwaukee	14	93	210	80	- -	<0.5	- -
		94	210	82	- -	1.0	17,200
		95	210	92	- -	1.0	19,300
		96	210	96	- -	1.0	20,200
		97	210	88	- -	3.0	55,400

Civil Division	Bluff Analysis Sections	Shoreline Recession Reaches	Shoreline Length (feet)	Bluff Height (feet)	Annual Recession Rates (feet/year) Long Term (1936-1985)	Annual Recession Rates (feet/year) Short Term (1963-1985)	Annual Volume of Shore Material Loss (cubic feet per year)
City of South Milwaukee (continued)	14	98	210	84	- -	3.5	61,700
		99	220	74	- -	1.0	16,300
	15	100	210	72	- -	1.5	22,700
		101	400	74	- -	2.0	59,200
		102	230	64	- -	<0.5	- -
	16	103	200	64	- -	1.0	12,800
		104	200	56	- -	1.0	11,200
	17	105	200	58	- -	<0.5	- -
		106	240	56	2.0	2.0	26,900
	18	107	210	58	- -	<0.5	- -
		108	200	64	- -	<0.5	- -
		109	210	70	- -	<0.5	- -
		110	210	68	- -	1.0	14,300
		111	210	72	- -	1.5	22,700
		112	210	72	- -	2.5	37,800
		113	220	76	- -	0.5	8,400
		114	260	68	- -	2.0	35,400
		115	920	62	- -	1.0	57,000
	19	116	220	60	- -	<0.5	- -
		117	320	54	- -	<0.5	- -
		118	220	N/A	- -	<0.5	- -
		119	210	N/A	- -	<0.5	- -
		120	200	64	- -	<0.5	- -
		121	200	56	- -	<0.5	- -
		122	200	64	- -	<0.5	- -
		123	200	76	- -	<0.5	- -
		124	200	78	- -	<0.5	- -
		125	200	76	- -	<0.5	- -
		126	200	76	- -	<0.5	- -
		127	200	88	- -	<0.5	- -
	20	128	250	100	- -	<0.5	- -
		129	270	100	- -	<0.5	- -
		130	210	96	- -	0.5	10,100
		131	200	94	- -	0.5	9,400
		132	200	94	1.0	0.5	9,400
	21	133	250	92	- -	0.5	11,500
		134	220	92	- -	0.5	10,100
		135	220	92	- -	1.5	30,400
		136	220	88	- -	0.5	9,700
		137	240	84	- -	0.5	10,100
	22	138	220	74	- -	0.5	8,100
		139	220	70	- -	0.5	7,700
		140	220	66	- -	<0.5	- -
		141	220	66	- -	0.5	7,300
	23	142	230	72	- -	<0.5	- -
		143	210	78	- -	0.5	8,200
		144	210	76	- -	0.5	8,000
		145	210	68	- -	1.0	14,300
		146	210	58	- -	2.0	24,000
		147	250	52	- -	<0.5	- -

Appendix C (continued)

Civil Division	Bluff Analysis Sections	Shoreline Recession Reaches	Shoreline Length (feet)	Bluff Height (feet)	Annual Recession Rates (feet/year) Long Term (1936-1985)	Annual Recession Rates (feet/year) Short Term (1963-1985)	Annual Volume of Shore Material Loss (cubic feet per year)
City of South Milwaukee (continued)	24	148	220	52	- -	<0.5	- -
		149	210	52	- -	0.5	5,500
		150	220	58	- -	0.5	6,400
		151	210	54	- -	1.0	11,300
		152	210	60	- -	3.0	37,800
		153	210	60	- -	2.5	31,500
		154	210	62	- -	3.0	39,100
		155	210	64	- -	3.0	40,300
		156	200	64	- -	3.0	38,400
	25	157	200	68	- -	3.0	40,800
		158	200	70	- -	3.5	49,000
		159	200	80	- -	4.0	64,000
City of Cudahy	26	160	200	86	1.0	4.0	68,800
		161	200	90	- -	<0.5	- -
		162	200	92	- -	1.0	18,400
		163	200	94	- -	1.0	18,800
	27	164	200	94	- -	0.5	9,400
		165	200	94	- -	1.0	18,800
		166	210	94	- -	<0.5	- -
		167	200	94	- -	2.5	47,000
		168	200	98	- -	2.0	39,200
		169	200	104	- -	1.0	20,800
		170	200	106	- -	2.0	42,400
		171	200	112	- -	<0.5	- -
		172	200	112	- -	<0.5	- -
		173	200	100	- -	<0.5	- -
	28	174	200	100	- -	<0.5	- -
		175	200	100	- -	1.0	20,000
		176	210	100	- -	2.5	52,500
		177	200	100	- -	1.0	20,000
		178	240	98	- -	1.0	23,500
		179	210	100	- -	<0.5	- -
		180	210	98	- -	0.5	10,300
		181	210	94	- -	0.5	9,900
		182	210	92	- -	0.5	9,700
	29	183	210	100	- -	2.5	52,500
		184	200	102	- -	1.0	20,400
		185	200	100	- -	1.0	20,000
		186	200	102	- -	5.0	102,900
	30	187	200	100	1.0	5.0	100,000
		188	210	100	- -	4.5	94,500
		189	210	100	- -	3.0	63,000
		190	200	98	- -	4.0	78,400
		191	210	98	- -	5.0	102,900
		192	200	100	- -	3.5	70,000
		193	210	98	- -	2.5	51,400
		194	210	100	- -	5.0	105,000
		195	240	102	- -	2.5	61,200
	31	196	220	102	- -	1.5	33,700
		197	210	102	- -	2.5	53,500

Civil Division	Bluff Analysis Sections	Shoreline Recession Reaches	Shoreline Length (feet)	Bluff Height (feet)	Annual Recession Rates (feet/year) Long Term (1936-1985)	Annual Recession Rates (feet/year) Short Term (1963-1985)	Annual Volume of Shore Material Loss (cubic feet per year)
City of Cudahy (continued)	32	198	220	102	- -	<0.5	- -
		199	230	106	- -	1.5	36,600
	33	200	230	104	- -	1.0	23,900
		201	220	108	- -	2.5	59,400
		202	220	106	- -	3.5	81,600
		203	220	100	- -	3.0	66,000
		204	220	100	- -	2.5	55,000
		205	210	100	- -	4.0	84,000
		206	210	96	- -	4.0	80,600
		207	200	100	- -	2.0	40,000
		208	220	106	- -	1.0	23,300
	34	209	240	104	- -	<0.5	- -
		210	220	96	- -	<0.5	- -
		211	220	92	- -	1.0	20,206
		212	220	88	- -	3.0	58,100
		213	220	88	- -	0.5	9,700
		214	220	92	0.5	<0.5	- -
		215	220	92	- -	<0.5	- -
		216	220	104	- -	<0.5	- -
	35	217	220	94	- -	1.0	22,900
		218	210	102	- -	<0.5	- -
		219	220	90	- -	0.5	9,900
	36	220	240	86	- -	<0.5	- -
		221	210	82	- -	0.5	8,600
		222	260	80	- -	0.5	10,400
	37	223	240	80	- -	0.5	9,600
		224	260	76	- -	<0.5	- -
		225	260	74	- -	0.5	9,600
		226	220	72	- -	0.5	7,900
		227	220	72	- -	<0.5	- -
City of St. Francis	38	228	230	74	- -	1.5	25,500
		229	230	64	- -	4.0	58,900
		230	200	56	- -	5.5	61,600
		231	210	56	- -	2.5	29,400
		232	200	54	- -	3.5	37,800
		233	200	54	- -	1.5	32,400
	39	234	210	54	- -	1.5	17,000
		235	240	52	- -	2.5	31,200
		236	240	48	- -	3.0	34,600
		237	240	48	- -	3.5	40,300
		238	260	46	- -	3.5	41,900
		239	220	46	- -	1.0	10,100
	40	240	200	46	2.0	2.5	23,000
		241	210	46	- -	1.5	14,500
		242	219	48	- -	<0.5	- -
		243	240	30	- -	<0.5	- -
	41	244	310	32	- -	<0.5	- -
		245	210	32	- -	<0.5	- -

Appendix C (continued)

Civil Division	Bluff Analysis Sections	Shoreline Recession Reaches	Shoreline Length (feet)	Bluff Height (feet)	Annual Recession Rates (feet/year) Long Term (1936-1985)	Annual Recession Rates (feet/year) Short Term (1963-1985)	Annual Volume of Shore Material Loss (cubic feet per year)
City of St. Francis (continued)	41	246	220	56	- -	<0.5	- -
		247	220	58	- -	<0.5	- -
		248	860	58	- -	1.5	74,800
	42	249	250	56	- -	1.5	21,000
		250	250	56	- -	1.5	21,000
		251	360	56	- -	4.5	90,700
	43	252	270	46	- -	<0.5	- -
		253	220	44	- -	<0.5	- -
		254	270	48	- -	<0.5	- -
		255	320	42	- -	<0.5	- -
	44	256	290	40	- -	<0.5	- -
	45	257	260	38	- -	<0.5	- -
	46	258	270	40	- -	<0.5	- -
	47	259	310	40	- -	<0.5	- -
		260	270	46	- -	<0.5	- -
		261	300	50	- -	<0.5	- -
		262	350	52	- -	<0.5	- -
City of Milwaukee	47	263	330	52	- -	<0.5	- -
		264	320	52	- -	<0.5	- -
		265	350	48	1.5	<0.5	- -
		266	310	48	- -	1.0	14,900
	48	267	340	46	- -	0.5	7,800
		268	330	48	- -	1.0	15,800
		269	260	50	- -	<0.5	- -
		270	290	50	- -	<0.5	- -
	49	271	510	48	- -	<0.5	- -
	50	272	370	44	- -	<0.5	- -
		273	290	34	- -	<0.5	- -
		274	300	24	- -	<0.5	- -
	51	275	280	22	- -	<0.5	- -
		276	240	20	- -	<0.5	- -
		277	220	32	- -	<0.5	- -
	52	278	260	30	- -	<0.5	- -
		279	240	30	- -	<0.5	- -
	53	280	280	30	- -	<0.5	- -
		281	210	26	- -	<0.5	- -
		282	420	26	- -	<0.5	- -
		283	510	28	- -	<0.5	- -
	54	284	270	34	- -	<0.5	- -
		285	260	34	- -	<0.5	- -
		286	260	32	- -	<0.5	- -
		287	250	32	- -	<0.5	- -
	55 and 56	288-382	49, 910	N/A	- -	<0.5	- -

Appendix C (continued)

Civil Division	Bluff Analysis Sections	Shoreline Recession Reaches	Shoreline Length (feet)	Bluff Height (feet)	Annual Recession Rates (feet/year) Long Term (1936-1985)	Annual Recession Rates (feet/year) Short Term (1963-1985)	Annual Volume of Shore Material Loss (cubic feet per year)
City of Milwaukee (continued)	57	383	220	N/A	--	<0.5	--
		384	230	N/A	--	<0.5	--
		385	280	N/A	--	<0.5	--
		386	260	52	--	<0.5	--
		387	260	52	--	<0.5	--
		388	280	58	--	<0.5	--
		389	270	60	--	<0.5	--
		390	240	68	--	<0.5	--
		391	400	68	--	<0.5	--
		392	350	80	--	<0.5	--
		393	210	82	--	<0.5	--
		394	210	82	--	<0.5	--
	58	395	220	80	--	<0.5	--
		396	220	80	--	<0.5	--
		397	230	76	--	<0.5	--
		398	230	78	--	<0.5	--
		399	250	82	--	<0.5	--
		400	270	86	--	<0.5	--
		401	240	88	--	<0.5	--
		402	240	86	--	<0.5	--
	59	403	300	84	--	<0.5	--
		404	500	82	--	<0.5	--
		405	350	88	--	<0.5	--
		406	210	80	--	<0.5	--
		407	210	78	--	<0.5	--
		408	210	78	--	<0.5	--
		409	210	78	--	<0.5	--
		410	240	N/A	--	<0.5	--
		411	220	N/A	--	<0.5	--
		412	250	N/A	--	<0.5	--
		413	290	N/A	--	<0.5	--
		414	300	N/A	--	<0.5	--
		415	250	N/A	--	<0.5	--
	60	416	220	N/A	--	<0.5	--
		417	200	N/A	--	<0.5	--
		418	200	N/A	--	<0.5	--
		419	200	N/A	--	<0.5	--
		420	200	N/A	--	<0.5	--
		421	200	N/A	--	<0.5	--
		422	210	N/A	--	<0.5	--
		423	780	N/A	--	<0.5	--
Village of Shorewood	61	424	210	75	--	<0.5	--
		425	210	80	--	0.5	8,400
		426	230	75	--	<0.5	--
		427	200	80	--	<0.5	--
		428	200	80	--	0.5	8,000
		429	200	90	--	<0.5	--
		430	230	90	--	<0.5	--
		431	240	90	--	<0.5	--
		432	320	90	--	<0.5	--
	62	433	200	100	--	<0.5	--
		434	200	100	--	0.5	10,000

Appendix C (continued)

Civil Division	Bluff Analysis Sections	Shoreline Recession Reaches	Shoreline Length (feet)	Bluff Height (feet)	Annual Recession Rates (feet/year) Long Term (1936-1985)	Annual Recession Rates (feet/year) Short Term (1963-1985)	Annual Volume of Shore Material Loss (cubic feet per year)
Village of Shorewood (continued)	62	435	210	100	- -	<0.5	- -
		436	210	95	- -	<0.5	- -
		437	200	95	- -	0.5	9,500
	63	438	210	95	- -	<0.5	- -
	64	439	200	100	- -	0.5	10,000
		440	210	100	- -	0.5	10,500
	65	441	200	105	- -	0.5	10,500
		442	200	100	- -	<0.5	- -
		443	200	100	- -	0.5	10,000
		444	200	100	- -	<0.5	- -
		445	200	100	- -	<0.5	- -
		446	250	105	- -	0.5	13,100
		447	290	110	- -	0.5	16,000
	66	448	270	110	- -	<0.5	- -
	67	449	210	110	- -	0.5	11,600
	68	450	240	105	1.0	<0.5	- -
		451	200	90	- -	<0.5	- -
		452	250	90	- -	<0.5	- -
		453	200	99	- -	<0.5	- -
		454	210	90	- -	<0.5	- -
		455	210	95	- -	0.5	10,000
		456	210	95	- -	<0.5	- -
		457	220	95	- -	<0.5	- -
		458	200	100	- -	0.5	10,000
		459	200	100	- -	<0.5	- -
		460	200	100	- -	0.5	10,000
	69	461	200	90	- -	<0.5	- -
		462	220	115	- -	<0.5	- -
	70	463	200	115	- -	0.5	11,500
		464	200	110	- -	<0.5	- -
	71	465	200	110	- -	<0.5	- -
		466	200	115	- -	<0.5	- -
		467	200	115	- -	<0.5	- -
		468	230	110	- -	<0.5	- -
Village of Whitefish Bay	71	469	350	110	- -	<0.5	- -
		470	250	100	- -	<0.5	- -
		471	240	100	- -	<0.5	- -
		472	260	95	- -	1.0	24,700
		473	350	95	- -	<0.5	- -
	72	474	290	95	- -	<0.5	- -
		475	260	95	- -	0.5	12,400
	73	476	260	95	2.0	<0.5	- -
		477	250	95	- -	<0.5	- -
	74	478	270	90	- -	0.5	12,200

Civil Division	Bluff Analysis Sections	Shoreline Recession Reaches	Shoreline Length (feet)	Bluff Height (feet)	Annual Recession Rates (feet/year) Long Term (1936-1985)	Annual Recession Rates (feet/year) Short Term (1963-1985)	Annual Volume of Shore Material Loss (cubic feet per year)
Village of Whitefish Bay (continued)	75	479	280	80	- -	<0.5	- -
	76	480	260	80	- -	<0.5	- -
	77	481	250	80	- -	<0.5	- -
		482	250	70	- -	<0.5	- -
		483	300	70	- -	<0.5	- -
		484	250	70	- -	<0.5	- -
		485	270	70	- -	<0.5	- -
		486	250	65	- -	<0.5	- -
	78	487	270	70	- -	<0.5	- -
		488	250	70	- -	<0.5	- -
		489	260	75	- -	<0.5	- -
		490	250	70	- -	<0.5	- -
	79	491	240	70	- -	<0.5	- -
		492	300	65	- -	<0.5	- -
		493	250	70	- -	<0.5	- -
		494	250	75	- -	<0.5	- -
		495	300	70	- -	<0.5	- -
	80	496	240	70	- -	0.5	8,400
	81	497	240	70	- -	0.5	8,400
		498	210	80	- -	1.0	16,800
		499	260	80	- -	1.0	20,800
		500	240	80	- -	0.5	9,600
		501	230	75	- -	1.0	17,200
		502	250	80	- -	0.5	10,000
		503	250	80	1.5	<0.5	- -
		504	250	85	- -	<0.5	- -
		505	250	85	- -	<0.5	- -
		506	210	85	- -	<0.5	- -
		507	210	85	- -	<0.5	- -
		508	230	80	- -	<0.5	- -
	82	509	210	80	- -	<0.5	- -
		510	200	85	- -	0.5	8,500
	83	511	210	85	- -	<0.5	- -
	84	512	210	85	- -	<0.5	- -
		513	200	80	- -	<0.5	- -
	85	514	200	75	- -	<0.5	- -
		515	200	75	- -	0.5	7,500
		516	200	80	- -	0.5	8,000
	86	517	200	90	- -	<0.5	- -
	87	518	200	95	- -	<0.5	- -
		519	200	90	- -	<0.5	- -
		520	200	90	- -	<0.5	- -
		521	200	90	- -	0.5	9,000
		522	200	105	- -	0.5	10,500
		523	200	115	- -	0.5	11,500

Appendix C (continued)

Civil Division	Bluff Analysis Sections	Shoreline Recession Reaches	Shoreline Length (feet)	Bluff Height (feet)	Annual Recession Rates (feet/year)		Annual Volume of Shore Material Loss (cubic feet per year)
					Long Term (1936-1985)	Short Term (1963-1985)	
Village of Whitefish Bay (continued)	87	524	200	120	- -	<0.5	- -
		525	200	120	- -	0.5	12,000
		526	200	115	- -	<0.5	- -
	88	527	200	115	- -	0.5	11,500
		528	200	120	- -	<0.5	- -
		529	200	115	- -	<0.5	- -
		530	200	125	1.5	0.5	12,500
		531	200	125	- -	0.5	12,500
		532	200	125	- -	0.5	12,500
	89	533	200	120	- -	0.5	12,500
		534	220	120	- -	0.5	13,200
	90	535	210	115	- -	0.5	12,100
		536	210	125	- -	0.5	13,100
	91	537	200	120	- -	<0.5	- -
		538	200	125	- -	<0.5	- -
		539	200	125	- -	<0.5	- -
	92	540	250	120	- -	0.5	15,000
		541	250	120	- -	<0.5	- -
		542	240	115	- -	0.5	13,800
	93	543	200	120	- -	0.5	12,000
		544	220	120	- -	0.5	13,200
	94	545	220	115	- -	0.5	12,600
		546	210	115	- -	<0.5	- -
		547	240	120	- -	<0.5	- -
		548	240	120	- -	<0.5	- -
		549	240	120	- -	<0.5	- -
		550	240	120	- -	0.5	14,400
		551	220	25	- -	1.0	5,500
	95	552	230	10	- -	1.0	2,300
		553	250	5	- -	<0.5	- -
		554	200	5	- -	1.5	1,500
		555	210	5	- -	<0.5	- -
		556	220	5	- -	0.5	600
		557	220	5	0.5	0.5	600
		558	200	5	- -	<0.5	- -
		559	200	5	- -	0.5	500
		560	200	5	- -	1.0	1,000
		561	200	5	- -	<0.5	- -
		562	200	5	- -	1.5	1,500
		563	200	5	- -	<0.5	- -
		564	200	5	- -	<0.5	- -
		565	200	5	- -	<0.5	- -
		566	200	5	- -	<0.5	- -
		567	210	5	- -	<0.5	- -
		568	210	5	- -	<0.5	- -
		569	290	5	- -	<0.5	- -
		570	230	5	- -	0.5	600
		571	200	5	- -	0.5	500
		572	200	5	- -	0.5	500
		573	200	5	- -	<0.5	- -

Appendix C (continued)

Civil Division	Bluff Analysis Sections	Shoreline Recession Reaches	Shoreline Length (feet)	Bluff Height (feet)	Annual Recession Rates (feet/year) Long Term (1936-1985)	Annual Recession Rates (feet/year) Short Term (1963-1985)	Annual Volume of Shore Material Loss (cubic feet per year)
Village of Whitefish Bay (continued)	95	574	200	5	- -	< 0.5	- -
		575	210	5	- -	< 0.5	- -
		576	200	5	- -	< 0.5	- -
		577	210	5	- -	< 0.5	- -
		578	230	5	- -	< 0.5	- -
		579	200	5	- -	< 0.5	- -
		580	240	5	- -	0.5	600
		581	230	5	- -	< 0.5	- -
		582	240	5	- -	< 0.5	- -
		583	270	5	- -	0.5	700
		584	200	5	0.5	0.5	500
		585	200	5	- -	< 0.5	- -
		586	210	5	- -	< 0.5	- -
		587	220	5	- -	< 0.5	- -
		588	210	10	- -	< 0.5	- -
		589	220	10	- -	< 0.5	- -
		590	220	10	- -	< 0.5	- -
		591	250	10	- -	< 0.5	- -
		592	200	5	- -	< 0.5	- -
		593	200	10	- -	< 0.5	- -
	96	594	200	90	- -	< 0.5	- -
		595	210	90	- -	0.5	9,500
		596	270	90	- -	0.5	12,200
Village of Bayside	96	597	200	93	- -	< 0.5	- -
		598	220	95	- -	< 0.5	- -
		599	200	91	- -	< 0.5	- -
		600	210	89	- -	< 0.5	- -
		601	210	15	- -	< 0.5	- -
		602	210	15	- -	< 0.5	- -
	97	603	210	17	- -	0.5	1,800
		604	210	15	- -	1.0	3,200
		605	210	11	- -	1.0	2,300
		606	210	9	- -	2.5	4,700
		607	220	9	- -	1.5	3,000
		608	220	7	- -	2.0	3,100
		609	220	7	- -	2.0	3,100
		610	220	9	- -	1.0	2,000
		611	220	9	- -	1.5	3,000
		612	210	9	- -	1.5	2,800
		613	200	15	- -	1.5	4,500
		614	220	11	- -	1.0	2,400
		615	220	9	- -	1.0	2,000
		616	210	9	- -	1.0	1,900
		617	210	7	- -	2.0	2,900
		618	220	7	- -	2.0	3,100
		619	200	11	- -	2.0	4,400
		620	210	17	- -	2.0	7,100
		621	210	21	- -	1.0	4,400
		622	200	19	- -	1.5	5,100
		623	200	13	- -	2.5	6,500
	98	624	210	11	- -	1.0	2,300
		625	210	9	- -	1.5	2,800
		626	220	9	- -	1.5	3,000
		627	220	9	- -	1.0	2,000

Appendix C (continued)

Civil Division	Bluff Analysis Sections	Shoreline Recession Reaches	Shoreline Length (feet)	Bluff Height (feet)	Annual Recession Rates (feet/year)		Annual Volume of Shore Material Loss (cubic feet per year)
					Long Term (1936-1985)	Short Term (1963-1985)	
Village of Bayside (continued)	99	628	230	9	- -	1.5	3,100
		629	270	9	- -	3.0	7,300
		630	260	11	- -	3.0	8,600
		631	260	11	- -	1.0	2,900
		632	260	13	- -	3.0	10,100
		633	250	93	- -	1.5	34,900
	100	634	250	93	- -	1.5	34,900
		635	240	93	- -	0.5	11,200
		636	240	93	- -	0.5	11,200
		637	240	91	- -	< 0.5	- -
		638	100	91	1.0	< 0.5	- -
Total	- -	- -	159,110	- -	- -	- -	8,860,900
Percent of total shoreline length with recession > 0.5 feet/year							37.4
Mean recession rate of shoreline > 0.5 feet/year							1.9 feet per year

NOTE: N/A indicates no shoreline bluff.

Source: SEWRPC.

Appendix D

NEWSPAPER ARTICLES PERTAINING TO THE LAKE MICHIGAN SHORELINE EROSION MANAGEMENT PLAN FOR MILWAUKEE COUNTY

SOUTHEASTERN WISCONSIN REGIONAL PLANNING COMMISSION

916 N. EAST AVENUE • P.O. BOX 1607 • WAUKESHA, WISCONSIN 53187-1607 • TELEPHONE (414) 547-6721

Serving the Counties of: KENOSHA MILWAUKEE OZAUKEE RACINE WALWORTH WASHINGTON WAUKESHA

News Release

FOR IMMEDIATE RELEASE

September 6, 1989
Release No. 89-2

PUBLIC HEARING SET FOR COUNTY SHORELINE EROSION CONTROL PLAN

A public hearing on a Lake Michigan Shoreline Erosion Control Plan for Milwaukee County has been scheduled for Wednesday, September 27, 1989, at 7:00 p.m., at the South Shore Park Pavilion, 2900 South Shore Drive.

The plan was prepared by Milwaukee County with the technical assistance of the Southeastern Wisconsin Regional Planning Commission. The preparation of the plan was guided by an Intergovernmental Coordinating and Technical Advisory Committee created by the County for this purpose. This Advisory Committee was chaired by County Supervisor Daniel Cupertino, Jr. and consisted of state and local public officials and concerned citizens. The full membership of the Committee is listed on the attachment to this news release. Milwaukee County undertook the preparation of the plan in order to provide state and local elected and appointed officials and property owners with the information required to make sound decisions concerning the development, redevelopment, and use of the Lake Michigan shoreline.

The hearing was announced by the Advisory Committee Chairman, County Supervisor Cupertino. Despite recent declines in lake levels and the resultant reduction in shoreline erosion problems, Supervisor Cupertino urged affected parties to avoid complacency in dealing with potential shoreline erosion problems. He urged attendance at the public hearing by affected and interested citizens. "It's essential that public and private lakefront property owners realize that based on historic experience, the Lake Michigan water levels can be expected to again rise." said Supervisor Cupertino. "Proper planning can ensure that sufficient lead times are available for adequate design, funding, and construction of needed shore protection measures. Shoreline erosion is not the type of problem which can be addressed only when lake levels are near record highs." he said.

The plan which will be presented at the public hearing was selected after an extensive review of available alternatives. Plan recommendations include:

1. Measures to stabilize the bluff slopes, including regrading or reshaping the bluff slopes along 19 percent of the 30-mile County shoreline; revegetating the bluff slopes along 8 percent of the shoreline; controlling groundwater seepage from about 13 percent of the shoreline; and abating surface erosion caused by stormwater runoff along 3 percent of the shoreline.

2. The construction of two new marinas: a public marina at Bender Park in the City of Oak Creek, and a private marina in the City of St. Francis on the former Wisconsin Electric Power Company Lakeside power plant property.

3. The construction of nourished sand beaches protected by offshore breakwaters at Atwater, Doctors, and Sheridan Parks, providing about 37 acres of public sand beach for swimming and sunbathing.

4. The construction of about seven miles of new gravel beaches, which would provide for easy shoreline access and use.

5. The construction of about eight and one-half miles of stone revetments.

6. The continued maintenance and repair of nearly six miles of existing bulkheads, or vertical concrete or steel walls.

7. The provision of appropriate setbacks for new urban development in order to adequately protect new structures and to preserve the shoreline character.

The plan also includes recommendations for the Milwaukee outer harbor breakwater--often referred to as the Government Pier--and for the South Shore breakwater, which lies offshore of South Shore and Bay View Parks. The plan concludes that it would not be cost effective to increase the height of the outer harbor breakwater, and instead recommends that this breakwater be maintained at its existing elevation.

The plan recommends that the portion of the South Shore breakwater--which is rapidly deteriorating--located north of E. Oklahoma Avenue be reconstructed to increase its elevation by up to about four feet, and that the side slopes of the breakwater be stabilized. This new elevation would provide adequate protection against lake levels that may be expected to occur an average of once every 100 years, and a storm wave expected to occur an average of once every 20 years. The portion of the South Shore breakwater located south of E. Oklahoma Avenue would be demolished under the plan, with the stone--a red granite excellent for building structures--being salvaged and used to reconstruct the northern portion of the breakwater, and to construct needed onshore measures for the reach of shoreline which currently receives some protection from the existing breakwater.

The accompanying map presents in graphic summary form the recommended shoreline erosion control plan as it is to be presented at the public hearing.

(Editors Note: See Map 6 on pages 24 and 25 of attached SEWRPC Newsletter)

Excluding the cost of the two marinas, the total capital cost of the recommended plan is about $66.0 million, or about $440 per lineal foot of shoreline. The plan would also entail an average annual maintenance cost of about $4.0 million upon full implementation. About 75 percent of the plan cost would be entailed in projects protecting publicly-owned lands and facilities. The remaining 25 percent would be entailed in protecting privately-owned lands and facilities. Public funds would not be expended to protect privately-owned shoreline property. The marinas would have a capital cost of about $16.4 million, which cost would be funded under recreational programs utilizing public and private funds and user fees.

The plan could be carried out over a 20 to 30-year period of time as the need for additional shore protection measures arises. If implemented over a 25-year period, the plan would entail an average annual capital cost of about $2.0 million for the protection of public lands and facilities. The preparation of the plan allows implementation to proceed on a reach by reach basis as needed in a manner which will result in an integrated and coordinated shoreline management program for the entire County upon full implementation.

In the northern part of the County, the plan recommends that the individual municipalities take the lead in implementing the plan, while in the southern part of the County, the County would coordinate plan implementation activities. It is recommended that the plan be implemented within 43 specified reaches of shoreline referred to as implementation segments. Additional shore protection measures would be provided for a segment only when the shoreline property owners within that segment concluded that additional protection was needed. The plan is intended to serve as a basis for encouraging coordination between adjacent property owners in constructing and maintaining shore protection measures.

Public attendance and comment at the hearing are encouraged by the Advisory Committee. For persons unable to attend the hearing, information on the plan can be obtained from the offices of the Regional Planning Commission at 916 N. East Avenue, Waukesha, or by calling 547-6721. Written comments on the plan are welcome and may be submitted through Wednesday, October 4, 1989, to the above address. The Advisory Committee will carefully review all comments received and recommend adoption of a final plan later in October.

* * * *

Public Hearing Set For Shore Line Erosion Control Plan

SOUTH SIDE SPIRIT
September 10-16, 1989

A public hearing on a Lake Michigan Shoreline Erosion Control Plan for Milwaukee County has been scheduled for **Wednesday, September 17**, at 7 p.m. at the South Shore Park Pavilion, 2900 South Shore Drive.

The plan was prepared by Milwaukee County with the technical assistance of the Southeastern Wisconsin Regional Planning Commission.

The hearing was announced by the Advisory Committee Chairman, County Supervisor Dan Cupertino, Jr. Despite recent declines in lake levels and the resultant reduction in shoreline erosion problems, Supervisor Cupertino urged affected parties to avoid complacency in dealing with potential shoreline erosion problems. He urged attendance at the public hearing by affected and interested citizens. "It's essential that public and private lakefront property owners realize that based on historic experience, the Lake Michigan water levels can be expected to rise again," said Supervisor Cupertino. "Proper planning can ensure that sufficient lead times are available for adequate design, funding, and construction of needed shore protection measures. Shoreline erosion is not the type of problems which can be addressed only when lake levels are near record highs," he said.

The plan which will be presented at the public hearing was selected after an extensive review of available alternatives. Plan recommendations include:

1. Measures to stabilize bluff slopes, including regarding or reshaping the bluff slopes along 19 percent of the 30-miles County shoreline; revegetating the bluff slopes along 8 percent of the shoreline; controlling groundwater seepage from about 13 percent of the shoreline; and abating surface erosion caused by stormwater runoff along 3 percent of the shoreline.
2. The construction of two new marinas: a public marina at Bender Park in the City of Oak Creek, and a private marina in the City of St. Francis on the former Wisconsin Electric Power Company Lakeside power plant property.
3. The construction of nourished and sand beaches protected by offshore breakwaters at Atwater, Doctors, and Sheridan Parks, providing about 37 acres of public sand beach for swimming and sunbathing.
4. The construction of about seven miles of new gravel beaches, which would provide for easy shoreline access and use.
5. The construction of about eight and one-half miles of stone revetments.
6. The continued maintenance and repair of nearly six miles of existing bulkheads, or vertical concrete or steel walls.
7. The provision of appropriate setbacks for new urban development in order to adequately protect new structures and to preserve the shoreline character.

The plan also includes recommendations for the Milwaukee outer harbor breakwater—often referred to as the Government Pier—and for the South Shore breakwater, which lies offshore of South Shore and Bay View Parks. The plan concludes that it would not be cost effective to increase the height of the outer harbor breakwater, and instead recommends that this breakwater be maintained at its existing elevation.

The plan recommends that the portion of the South Shore breakwater—which is rapidly deteriorating—located north of East Oklahoma Avenue be reconstructed to increase its elevation by up to four feet, and that the side slopes of the breakwater be stabilized. This new elevation would provide adequate protection against lake levels that may be expected to occur an average of once every 100 years, and a storm wave expected to occur an average of once every 20 years. The portion of the South Shore breakwater located south of East Oklahoma Avenue would be demolished under the plan, with the stone—a red granite excellent for building structures—being salvaged and used to construct the northern portion of the breakwater, and to construct needed onshore measures for the reach of shoreline which currently receives some protection from the existing breakwater.

Excluding the cost of the two marinas, the total capital cost of the recommended plan is about $66.0 million, or about $440 per lineal foot of shoreline. The plan would also entail an average annual maintenance cost of about $4.0 million upon full implementation. About 75 percent of the plan cost would be entailed in projects protecting publicly-owned lands and facilities. The remaining 25 percent would be entailed in protecting privately-owned shoreline property. Public funds would not be expended to protect privately-owned shoreline property. The marinas would have a capital cost of about $16.4 million, which would be funded under recreational programs utilizing public and private funds and user fees.

The plan could be carried out over a 20 to 30-year period of time as the need for additional shore protection measures arises. If implemented over a 25-year period, the plan would entail an average capital cost of about $2.0 million for the protection of public lands and facilities. The preparation of the plan allows implementation to proceed on a reach by reach basis as needed in a manner which will result in an integrated and coordinated shoreline management program for the entire County upon full implementation.

In the northern part of the County, the plan recommends that the individual municipalities take the lead in implementing the plan, while in the southern part of the County, the County would coordinate plan implementation activities. It is recommended that the plan be implemented with 43 specified stretches of shoreline referred to an implementation segments. Additional shore protection measures would be provided for a segment only when the shoreline property owners within that segment concluded that additional protection was needed. The plan is intended to serve as a basis for encouraging coordination between adjacent property owners in constructing and maintaining shore protection measures.

Public attendance and comment at the hearing are encouraged by the Advisory Committee. For persons unable to attend the hearing, information on the plan can be obtained from the offices of the Regional Planning Comission at 916 N. East Avenue, Waukesha. or by calling 547-6721. Written comments on the plan are welcome and may be submitted through Wednesday, October 4, to the above address. The Advisory Committee will carefully review all comments received and recommend adoption of a final plan later in October.

Plan offered to control shore erosion

By DON BEHM
Journal environment reporter

Milwaukee County should protect its Lake Michigan shoreline with stone and steel today, before resurgent lake levels catch us by surprise again, members of a lakeshore advisory committee say.

Today's below-normal water levels on Lake Michigan provide an opportunity to protect the shoreline against future erosion and storm damage caused by rising water levels, the committee concludes in a report to be released at a public hearing next week.

The $80 million price tag for all of the rock embankments, sea walls, new marinas and beaches that the committee recommends could be spread out over a 20-year period to reduce the burden on property-tax payers, the report says.

Since 1986, Great Lakes water levels have risen and fallen in an unbroken rhythm.

If Lake Michigan seems to be shrinking this year, and its beaches getting wider, it is only because there has been less rain than usual in the region.

The lake's surface is 4 inches lower than its September average, according to the US Army Corps of Engineers. At 578.3 feet above sea level, it is nearly 3 feet below the record-high September level set in 1986. Lake Michigan's unusually high water levels recorded in 1986 and 1987 were fed by prolonged heavy rains.

In the same way, the lake could rise again in the near future to challenge our hold on its shoreline, warns County Supervisor Daniel Cupertino Jr., chairman of the lakeshore advisory committee.

Water levels on the Great Lakes could rise or fall under the so-called greenhouse effect, in which higher temperatures are forecast for the upper Midwest. As pollutants accumulate in the Earth's upper atmosphere, they will block an increasing amount of the heat that now radiates off the surface and escapes into the atmosphere. That will cause average annual temperatures to rise.

Under this theory, it is not clear whether the Great Lakes region will be hit by a drought. There could be reduced rain and snowfall in some areas of the country, such as the Plains states. But higher temperatures will also cause more evaporation from oceans and lakes.

The Earth is a closed environmental system, much like the radiator of an automobile. The water that evaporates from the oceans eventually will return to the Earth's surface as precipitation. Rain or snow anywhere on the land that drains into Lakes Superior, Huron and Michigan will sustain water levels at Milwaukee's lakefront, because Superior drains into Huron, and Huron and Michigan are connected.

Since water levels may rise, the time to start building protective measures is now, Cupertino said.

"Shoreline erosion is not the type of problem which can be addressed only when lake levels are near record highs," he said in a statement prepared for the upcoming hearing, scheduled for 7 p.m. Wednesday at the South Shore Park pavilion, 2900 South Shore Dr.

To prevent problems in the future, the Southeastern Wisconsin Regional Planning Commission and the advisory committee have both approved a long list of protective measures that include:

- Building a new public marina at Bender Park in Oak Creek and a new private marina at the former Wisconsin Electric Power Co. plant in St. Francis.
- Raising the existing breakwater between South Shore Park and the north boundary of Bay View Park.
- Demolishing the section of the shoreline breakwater that extends south of E. Oklahoma Ave. at Bay View Park.
- Adding sand beaches, protected by new breakwaters, at Atwater Park in Shorewood and two county parks: Doctors Park in Fox Point and Sheridan Park in Cudahy.
- Adding gravel beaches, protected by large rock walls, along 35,970 feet of shoreline — nearly 7 miles.
- Building rock walls, or riprap, along another 8 miles of shoreline.
- Building concrete or steel walls along 5.6 miles of the shoreline.

MILWAUKEE JOURNAL
September 22, 1989

Milwaukee County urged to protect lake shoreline

MILWAUKEE (AP) — Milwaukee County should use current conditions to protect its Lake Michigan shoreline with stone and steel before lake levels rise again, an advisory committee has recommended.

The current below-normal water levels on Lake Michigan provide an opportunity to protect the shoreline against future erosion and storm damage caused by rising water levels, the committee concludes in its report.

The $80 million price tag for all of the rock embankments, sea walls, new marinas and beaches that the committee recommends could be spread out over a 20-year period to reduce the burden on property-tax payers, the report says.

Since 1986, Great Lakes water levels have risen and fallen in an unbroken rhythm.

The Lake Michigan surface is 4 inches lower than its September average, according to the U.S. Army Corps of Engineers. At 578.3 feet above sea level, it is nearly 3 feet below the record-high September level set in 1986.

Lake Michigan's unusually high water levels recorded in 1986 and 1987 were fed by prolonged heavy rains.

WAUKESHA COUNTY FREEMAN
September 23, 1989

Committee says act now to stop lake erosion

JOURNAL TIMES
September 23, 1989

MILWAUKEE (AP) — Milwaukee County should use current conditions to protect its Lake Michigan shoreline with stone and steel before lake levels rise again, an advisory committee has recommended.

The current below-normal water levels on Lake Michigan provide an opportunity to protect the shoreline against future erosion and storm damage caused by rising water levels, the committee concludes in its report.

The $80 million price tag for all of the rock embankments, sea walls, new marinas and beaches that the committee recommends could be spread out over a 20-year period to reduce the burden on property-tax payers, the report says.

Since 1986, Great Lakes water levels have risen and fallen in an unbroken rhythm.

The Lake Michigan surface is 4 inches lower than its September average, according to the U.S. Army Corps of Engineers. At 578.3 feet above sea level, it is nearly 3 feet below the record-high September level set in 1986.

Lake Michigan's unusually high water levels recorded in 1986 and 1987 were fed by prolonged heavy rains.

In the same way, the lake could rise again in the near future to challenge our hold on its shoreline, said Milwaukee County Supervisor Daniel Cupertino Jr., chairman of the lakeshore advisory committee.

Since water levels may rise, the time to start building protective measures is now, Cupertino said.

"Shoreline erosion is not the type of problem which can be addressed only when lake levels are near record highs," he said in a statement prepared for a hearing slated next week.

To prevent problems in the future, the Southeastern Wisconsin Regional Planning Commission and the advisory committee have both approved a long list of protective measures that include:

—Building a new public marina at Bender Park in Oak Creek and a new private marina at the former Wisconsin Electric Power Co. plant in St. Francis.

—Raising the existing breakwater between South Shore Park and the north boundary of Bay View Park.

—Demolishing a section of the shoreline breakwater on the city's south side.

—Adding sand beaches, protected by new breakwaters, at Atwater Park in Shorewood and two county parks: Doctors Park in Fox Point and Sheridan Park in Cudahy.

—Adding gravel beaches, protected by large rock walls, along 35,970 feet of shoreline.

—Building rock walls along another 8 miles of shoreline.

—Building concrete or steel walls along 5.6 miles of the shoreline.

St. Francis critical of erosion plan

By LEONARD SYKES JR.
of The Journal staff

St. Francis was the only municipality critical of a $66 million plan to control erosion along the Lake Michigan shoreline when the proposal was discussed at a public hearing Wednesday.

The plan, prepared by the Erosion Control Management Study Committee, with technical assistance from the Southeastern Wisconsin Regional Planning Commission, was shown to the public Wednesday during a presentation at the South Shore Park pavilion, 2900 S. Shore Dr.

The erosion committee includes representatives of the regional planning commission, Milwaukee County, the Milwaukee Metropolitan Sewerage District, the State Department of Natural Resources and communities along the Lake Michigan shoreline.

In a letter, St. Francis Mayor Milton Vretenar pointed out that the committee recommended demolition of the breakwater that now protects shoreline property within his city.

"There is no assurance that if the city went on record in favor of this recommended design that politically this plan would come to pass in its entirety," the letter said. "Once the breakwater is gone, it is gone forever and the political reality is that the City of St. Francis may not have the clout to ensure the construction of on-shore revetments and groins in our community as is recommended."

Robert Biebel, chief environmental planner for the regional commission, said that despite recent drops in lake levels, erosion along the shoreline would continue without a plan to control it.

The proposal calls for measures to stabilize the bluff slopes along 19% of the 30-mile shoreline in Milwaukee County. Under that plan, bluff slopes along 8% of the shoreline would be planted with various species that once grew there, and measures to control groundwater seepage would be taken, Biebel said. About 75% of the money spent on the plan would go toward protecting public lands and facilities.

Other recommendations in the plan included:

■ Construction of two marinas: a public marina at Bender Park in Oak Creek, and a private marina on the former Wisconsin Electric Power Co. lakeside power plant property in St. Francis. The total price for the two marinas would be an additional $16 million.

■ Construction of sand beaches protected by offshore breakwaters at Atwater, Doctors and Sheridan Parks. Biebel said the beaches would provide about 37 acres of public sand beach for swimming and sunbathing in areas where beaches previously existed before erosion.

■ Construction of about seven miles of new gravel beaches, which would provide for easy shoreline access and use.

■ Construction of about 8½ miles of stone embankments along the shoreline.

MILWAUKEE JOURNAL
September 28, 1989

Public hearing set for county shoreline erosion control plan

A public hearing on the Lake Michigan Shoreline Erosion Control Plan for Milwaukee County has been scheduled for Wednesday, Sept 27 at 7 p.m., at the South Shore Park Pavilion, 2900 South Shore Drive.

The plan was prepared by Milwaukee County with the technical assistance of the Southeastern Wisconsin Regional Planning Commission. County Supervisor Daniel Cupertino, Jr., chaired an Intergovernment Coordinating and Technical Advisory Committee, which was chosen to guide the preparation of the plan. The committee consisted of state and local public officials as well as a group of concerned citizens.

The preparation of the plan was undertaken to provide state and local elected and appointed officials and property owners with the information required to make sound decisions concerning the development, redevelopment, and use of the Lake Michigan shoreline.

Despite recent declines in lake levels and the resultant reduction in shoreline erosion problems, Supervisor Cupertino has urged effective parties to avoid complacency in dealing with potential shoreline erosion problems. Cupertino urges attendence at the public hearing by affected and interested citizens. "It's essential that public and private lakefront property owners realize that based on historic experience, the Lake Michigan water levels can be expected to rise again," stated Cupertino. "Proper planning can ensure that sufficient lead times are available for adequate design, funding and construction of needed shore protection measures. Shoreline erosion is not the type of problem which can be addressed only when lake levels are near record highs," he claimed.

The plan recommendations, which will be presented at the public hearing, include:

• Measures to stabilize the bluff slopes, including regrading or reshaping the bluff slopes along 19% of the 30-mile County shoreline; revegetating the bluff slopes along 8% of the shoreline; controlling groundwater seepage from about 13% of the shoreline; and abating surface erosion caused by storm water runoff along 3% of the shoreline.

• The construction of two new marinas: a public marina at Bender Park in Oak Creek, and a private marina in the City of St. Francis on the former Wisconsin Electric Power Company Lakeside power plant property.

• The construction of nourished sand beaches protected by offshore breakwaters at Atwater, Doctors, and Sheridan Parks, providing about 37 acres of public sand beach for swimming and sunbathing.

• The construction of about seven miles of new gravel beaches, which would provide for easy shoreline access and use.

• The construction of about 8-1/2 miles of stone revetments.

• The continued maintenance and repair of nearly six miles of existing bulkheads, or vertical concrete or steel walls.

• The provision of appropriate setbacks for new urban development in order to adequately protect new structures and to preserve the shoreline character.

The plan will also outline recommendations for the Milwaukee outer harbor breakwater – often referred to as the Government Pier – and for the South Shore breakwater, which lies offshore of South Shore and Bay View parks. The plan concludes that it would not be cost effective to increase the height of the outer harbor breakwater and instead recommends that this breakwater be maintained at its existing elevation.

Included is a recommendation that a portion of the South Shore breakwater, which is rapidly deteriorating, located north of E. Oklahoma Ave., be reconstructed to increase its elevation by up to about four feet and that the side slopes of the breakwater be stabilized. The new elevation is expected to provide adequate protection against lake levels that may be expected to occur an average of once every 100 years and a storm wave expected to occur an average of once every 20 years.

Excluding the cost of the two marinas, the total capital cost of the recommended plan is about $66 million. The plan also entails an average annual maintenance cost of about $4 million upon full implementation. About 75% of the plan's cost would be entailed in projects protecting publicly-owned lands and facilities. The remaining 25% would be entailed in protecting privately-owned lands and facilities. Public funds would not be expended to protect privately-owned shoreline property. The marinas would have a capital cost of about $16.4 million, and would be funded under recreational programs utilizing public and private funds and user fees.

If implemented the plan could be carried out over a 20 to 30-year period of time as the need for additional shore protection arises. If implemented over a 25 year period, the plan would entail an average annual capital cost of about $2 million for the protection of public lands and facilities.

SHOPPER COMMUNITY NEWS
September 19-October 2, 1989

Commission announces lakefront erosion plan

By John Scott Lewinski

In a final effort to answer erosion problems confronting Milwaukee County's lakefront, the Southeastern Wisconsin Regional Planning Commission announced its Lake Michigan Shoreline Erosion Management Plan.

Unveiled at a Sept. 27 public hearing at the South Shore Yacht Club, 2900 South Shore Drive, the recommended plan, to be carried out during a 25-to 30-year span, would cost a total of $82.3 million.

Seventy-five percent of that cost would be paid by state and federal funding programs designed to protect public lands and shoreline facilities. The remaining 25 percent of the cost would be absorbed by the private sector.

The plan calls for the construction of a new public marina at Oak Creek's Bender Park and a private facility near the former Wisconsin Electric Power Company Lakeside plant property in St. Francis.

In addition, throughout the Milwaukee County coastline, portions of deteriorating shore would be regraded, reshaped, revegetated and rebuilt to control groundwater erosion. Miles of revetments also would be installed.

Atwater, Sheridan and Doctors Parks would see the installation of sand beaches, protected by breakwaters, providing swimming facilities. Approximately seven miles of gravel beaches would be implemented to allow easy lake access.

In the Bay View area, extensive public revetment and breakwater implementation would be used.

An estimated 29.7 percent of the project's $9 million-per-year price tag would be shouldered by Milwaukee. The Milwaukee and Bay View area work would be supervised and conducted jointly by the city of Milwaukee, Milwaukee County, the Milwaukee Metropolitan Sewerage District and the U.S. Army Corps of Engineers.

The shoreline erosion strategy was worked out by the specially-formed Intergovernmental Coordinating and Technical Advisory Committee, with Daniel Cupertino, Milwaukee County supervisor, as chairman.

The rest of the 28-member committee was comprised of local and state government officials, concerned citizens and other public figures. Included in the group were Milwaukee Alderman Christopher Krajniak; Jan Marsh, president of the Audubon Society of America's Milwaukee chapter; John Erickson, city of Milwaukee engineer; Dennis Noble of the South Shore Yacht Club and officials of the Whitefish Bay, Shorewood, Oak Creek, Bayside, South Milwaukee and St. Francis municipal governments.

The 43-page report prepared by the committee and the Regional Planning Commission stated the plan was devised to answer continuing concerns about Lake Michigan's eroding shoreline.

The report explains public concern about shoreline erosion peaked in 1986-87 when lake water levels were at the highest point of the 20th century. Those high levels awoke disquietude about the lake's decreasing shoreline stability.

The shoreline erosion study was called for in 1986, when the Regional Planning Commission and the advisory committee took up the project. The reported cost for the study itself was $200,000.

The pamphlet, explaining the study's results, states, "Giving the long lead times necessary for designing funding and constructing shoreline improvements, Milwaukee County residents would be best served by steady progress toward protecting the shorelines. . ."

The study came up with several different plan options to protect the lakefront. All plans were intended to provide alternatives in shoreline protection, bluff recession and storm damage controls.

Two types of plans were developed; a bluff slope stabilization element and a shoreline protection element.

One bluff slope plan and three shoreline protection plans were designed.

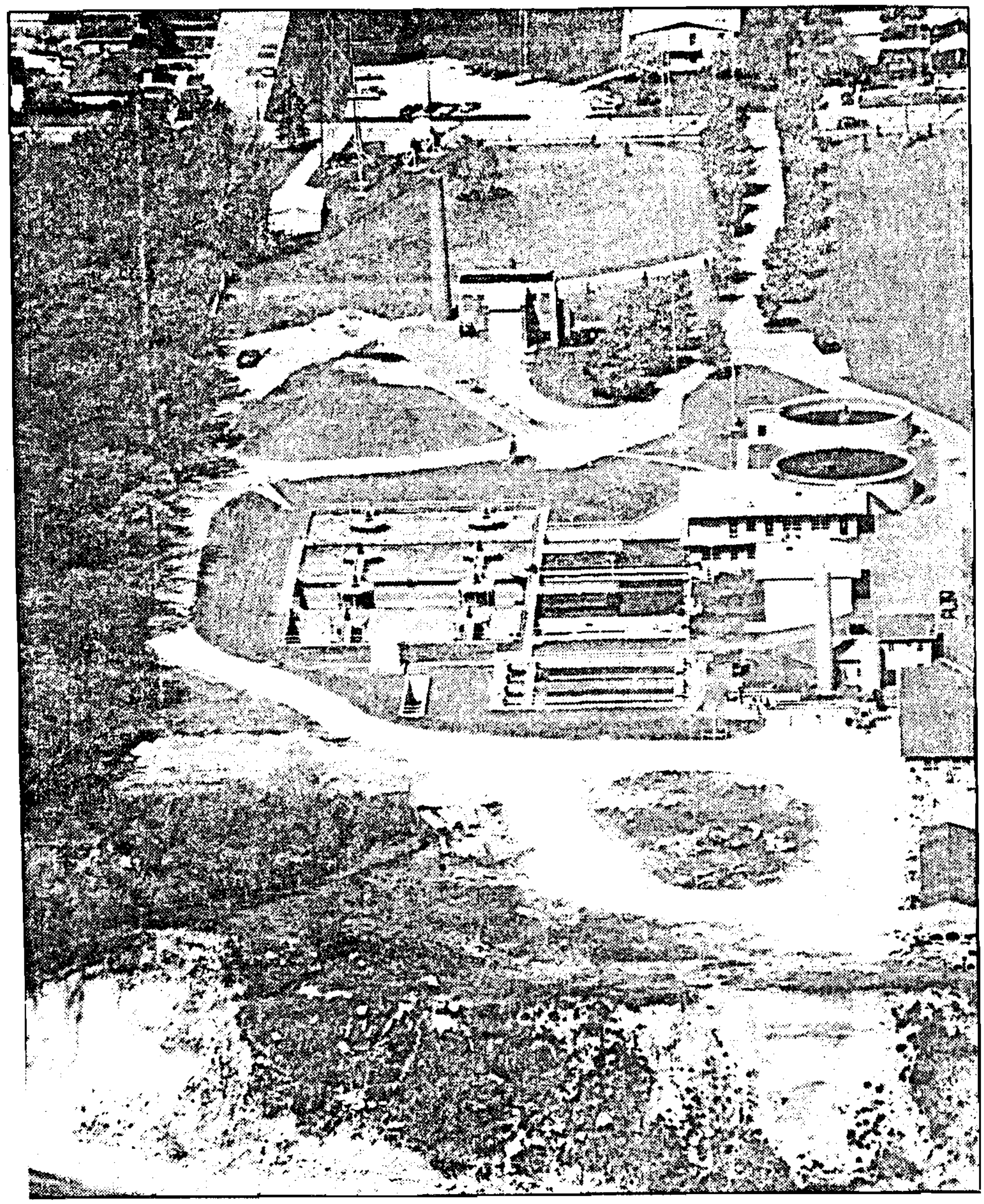
The bluff plan identifies the measures needed to regrade and replant the slope to control surface water flow at a cost of about $7.4 million. The plan explains that, in the Bay View area of the coast line, bluff slope regrading, surface water runoff control, groundwater drainage designs and bluff slope revegitation would be executed.

The first of the shoreline protection alternatives would use revetment (the use of masonary to protect embankments) techniques. Public funds would be utilized to construct new protection structures and to rebuild old ones at a total cost of $57 million.

The second plan, termed the Beach Alternative, would install coarse sand or gravel beaches along the shore to strengthen and to allow easy access to lake facilities at a cost of $69 million.

The final design, the Offshore Alternative, would use islands and extended breakwaters to protect the shore at a cost of $199.8 million. The materials needed to build the structures could come from the Milwaukee Metropolitan Sewerage District's Deep Tunnel project debris and by removing breakwaters in the St. Francis coastal area.

SOUTH MILWAUKEE VOICE GRAPHIC
October 5, 1989



THE NEAT PATTERNS made by the facilities at the South Milwaukee Waste Treatment Plant are a contrast to the eroded cliff at the Lake Michigan shoreline in this aerial photo looking west.
(Staff photo by Jack Plale)

SOUTH MILWAUKEE VOICE GRAPHIC
October 5, 1989

Face-lift for fallen shoreline in the planning stages

A great place on a great lake...the line to describe metro Milwaukee and its fabulous shoreline. But over the last several years that fabulous shoreline has been crumbling beneath the pounding of the waves that beat against it.

Last May when the Southeastern Wisconsin Regional Planning Commission highlighted some of their plans, the one controversial part concerned the South Shore breakwater portion, south of Oklahoma Ave. The plan was to demolish that breakwater and use the rock from that breakwater to reconstruct a breakwater north of Oklahoma and provide a different kind of protection for the shoreline.

Months down the road, that still remains the controversial part of the plan and St. Francis officials aren't too happy about it.

St. Francis administrator Ralph Voltner acted as spokesman for the municipality at the Sept. 27 hearing held at the South Shore Yacht Club, 2900 South Shore Drive. The hearing was held to outline the plan which would span 25 to 30 years.

The original plan explained last May was estimated to cost 482.4 million and the estimated maintenance cost was $4.1 million per year. Twenty-five percent of the cost was slated to be covered by private property owners and the remaining 75% would be paid for by the public sector which included state and federal funding designed to public lands and shorelines.

The latest figure is now $82.3 million.

The newly unveiled plan calls for the construction of a new marina in Oak Creek at Bender Park and another in St. Francis near the Wisconsin Electric Power Company site.

An effort to stabilize the bluff slopes by regrading them, controlling ground water drainage and surface water drainage and revegetating the ~~land is one of the elements contained in~~ the plan.

Part of the $7.4 million bluff slop plan would take place in the Bay View area.

On shore, new sand beaches are ~~slated for~~ Sheridan, Atwater and Doctors Parks, contained by off-shore breakwaters and in other areas new gravel beaches, contained by rock groins.

Revetments and breakwaters would be constructed around the Bay View area.

In addition, another alternative to be used in this plan to protect the shoreline is the use of off shore islands and extended breakwaters built from the Milwaukee Metropolitan Sewerage district debris and from the removal of the breakwaters from the St. Francis coastline.

It is the removal of these breakwaters from the St. Francis area that has city officials up in arms.

In a letter read at the hearing, Mayor Milton Vretenar expressed his concern that once removed, the protection for the St. Francis area may never be replaced and that a city the size of St. Francis may not "have the clout to ensure construction of on-shore revetments" as detailed in the plan.

The fear expressed by an opposing group called Save Our Shores (SOS) was that once the breakwaters were removed the damage from the waves would eventually take their roll and that land enforcements may not be sufficient.

Milwaukee County Supervisor, Daniel Cupertino, who acted as chairman for the Intergovernmental Coordinating and Technical Advisory Committee that worked on the shoreline erosion strategy, stated he was satisfied with the plan as it was designed.

Along with Cupertino were 28 other members of the committee who worked on the shoreline erosion strategy. Together with the Southeastern Wisconsin Regional Planning Commission a 43-page report was prepared on lake shore erosion. The cost of the study was around $200,000.

SHOPPER COMMUNITY NEWS
October 16-30, 1989

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Appendix E

CORRESPONDENCE PERTAINING TO PUBLIC HEARINGS

VILLAGE OF FOX POINT
MILWAUKEE COUNTY
WISCONSIN

VILLAGE HALL
7200 N. SANTA MONICA BLVD.
FOX POINT 53217
414-351-8900

September 27, 1989

Mr. Daniel Cupertino, Jr.
County Supervisor
Courthouse, Room 201
901 North Ninth Street
Milwaukee, Wisconsin 53233

Dear Supervisor Cupertino:

I read in the September 22 Milwaukee Journal that a committee is proposing extensive and costly shoreline construction.

Please be advised that Fox Point has participated in lengthy erosion studies. The Village Board has unanimously gone on record (a) opposing any taxing entity to pay for work on the shoreline; and (b) after a public hearing attended by many residents, the Village stated they did not choose to participate in joint shoreline projects.

The residents of our Village who live on the lake have joined with one another to deal with their erosion and have individually paid for the cost. Therefore, the residents have made it clear they do not want to be required to take part in a county-wide project.

Sincerely,

F. R. Dengel

F. R. Dengel
Village President

FRD/jm

COPY

SOUTHEASTERN WISCONSIN REGIONAL PLANNING COMMISSION

916 N. EAST AVENUE • P.O. BOX 1607 • WAUKESHA, WISCONSIN 53187-1607 • TELEPHONE (414) 547-6721
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WAUKESHA

Mr. F. R. Dingle, President
Village of Fox Point
7200 N. Santa Monica Blvd.
Fox Point, Wisconsin 53217

September 29, 1989

Dear Mr. Dingle:

We are in receipt of a copy of your letter dated September 27, 1989, to Supervisor Daniel Cupertino, Jr., Chairman of the Technical Advisory Committee for the Milwaukee County Lake Michigan Shoreline Erosion Plan. In your letter you indicate that the Village Board has unanimously gone on record as: 1) opposing the creation of any tax entity to pay for work on the shoreline and 2) opposing the participation by the Village in joint shoreline projects with other communities. You further indicate that residents of the Village of Fox Point who live on the Lake have joined with one another to deal with their erosion and have individually paid for the cost of their shore protection improvements, and that those residents do not want to be required to take part in any countywide project. While your letter will be made a part of the record of the public hearing held on the proposed County shore erosion control plan, the letter and the Village Board position appear to reflect a misunderstanding of the proposed plan.

With regard to the Village Board's first concern, please be advised that the proposed shore erosion control plan for Milwaukee County does not recommend the creation of any new taxing entity to pay for work for shore protection projects. It is recommended--as was also the case for the north shore erosion control plan--that the municipalities on the north shore create a cooperative commission which would help coordinate plan implementation activities. That Commission would not represent a new taxing authority. Thus, in this respect, the plan is consistent with the Village Board's position.

With regard to the second concern of the Village Board, the draft plan for the Fox Point area recommends the continued maintenance of existing structures along the privately owned shoreline and the reconstruction of revetments along the Village-owned shoreline. These projects would be carried out by the property owners themselves either independently or in cooperation with adjoining property owners as is the Village's preference. Furthermore, the plan does not recommend that the residents of the Village of Fox Point participate in the construction of any joint shoreline projects of a countywide nature with other communities. Thus, the recommendations set forth in the plan are fully consistent with the position you have indicated that the Village Board has taken.

We trust this information will help to correct any misunderstandings that exist concerning the proposed County plan and will help to allay the concerns raised in your September 27 letter. Should you, however, have any further questions regarding the plan, please do not hesitate to call.

Sincerely,

Kurt W. Bauer
Executive Director

KWB/ib

cc: Supervisor Daniel Cupertino
Mr. Ralph Knoernschild

international great lakes
COALITION
wisconsin lake michigan shoreline chapter, inc.

9/16/89

To: All MEMBERS OF THE INTERGOVERNMENTAL COORDINATING AND TECHNICAL ADVISORY COMMITTEE FOR THE MILWAUKEE COUNTY LAKE MICHIGAN SHORELINE EROSION MANAGEMENT PLAN

Dear Member:

We have read with interest the SOUTHEASTERN WISCONSIN REGIONAL PLANNING COMMISSION Newsletter dated May-June 1989, concerning the LAKE MICHIGAN SHORELINE EROSION MANAGEMENT PLAN PREPARED FOR MILWAUKEE COUNTY.

While it is obvious you have done your homework, there are three facts that are not mentioned in your recommendation. And...when you realize their full impact, may make a considerable difference in your planned dollar expenditure. $82.3 million may be much too much to accomplish the task.

> FACT ONE: The water levels of Lake Superior and Lake Ontario have been regulated for many years.
>
> FACT TWO: The Army Corps of Engineers is on record stating that the three middle lakes (Mighigan/Huron and Erie) can also be regulated.
>
> FACT THREE: As a result of a reference received by the INTERNATIONAL JOINT COMMISSION from the Canadian and U.S. Governments, the IJC is currently studying the possibility of such regulation. Phase I of that study has just been released (9/5/89).

Should the final report (due September of 1991) suggest to both governments that regulation of Lake Michigan be between 578' and 580' it could make a big difference in your thinking, and...save the major portion of the $83.2 million.

That regulation would mean that even with an instantaneous wind set-up of 2.5 feet, the maximum height would be 582.5 feet, considerably lower than the 100 year high for which you have planned, and lower than the 1986 record high.

p.o. box 1325 • sheboygan wisconsin 53082-1325

It's a solution that is entirely possible.

The Great Lakes Coalition (a Great Lakes Basinwide [U.S. and Canada] Coalition with over 25,000 members) is not endorsing any water diversion from the Great Lakes Basin beyond the natural outlets that now exist, but the Coalition feels that regulation would go a long way toward solving the erosion problems caused by high water levels.

For instance, we know through experience and research, that without regulation, each time the lake levels have risen, they have exceeded the previous high. What does that mean for future maintenance costs of breakwaters and revetments? and what does that mean for shoreline zoning? Consider the ecological factors without regulation, wildlife habitat will continue to flood and dry; and the political/social factors; marinas continually repositioning finger piers, the re-establishment of lakeshore roadways and beaches, the protection and reprotection of the waste water treatment plant, etc.

Then consider what low water levels could do to our beaches, marinas, power plants, etc. And without regulation low water is also possible, 1964 tells us that.

Shoreline zoning is on the minds of many governmental planners. But, without water level regulation, where will the shoreline be? Planning for a moveable shoreline does not seem feasible.

The Wisconsin Shoreline Chapter of the Great Lakes Coalition humbly requests the following: Please contact Governor Thompson (his <u>Water Level Task Force</u> has completed its studies and published the final report) and request that he continue his efforts within the Governor's Conference to urge the IJC to complete its studies promptly. Additionally, the GLC would like you to contact your federal representatives and urge them to enhort the International Joint Commission to complete the water level studies. The problem of water level regulation is political, not physical, and requires a political solution.

All of us concerned with future water levels must join together to force the IJC to complete its water level regulation studies. The present apathy can destroy all the efforts thus far. Without the completion of those studies, even the possibility of water level regulation is out of the question. And, without a solution recommended by the International Joint Commission, when the water levels rise again, we will be right back to square one.

Please consider our suggestion. You need the Great Lakes Coalition, we have achieved great strides in our efforts to accomplish water level regulation in the three "middle" lakes. We know its possible, we know the solution is political, it's a matter of getting enough voices to sing the same tune.

Sincerely,

William J. Wiesmueller
William Wiesmueller, Chairman

Alan C. Chase
Alan Chase, Vice-Chair

Pamela Ellis
Pamela Ellis, Secretary

Board of Directors
Great Lakes Coalition Wisconsin Lake Michigan Shoreline Chapter

PS: We would appreciate the opportunity to present our information to you in person. Please contact our Public Relations Office at (414) 564-2737, they will be glad to schedule a meeting.

Robert Ozaune, Director
Walter Lindeman, Director
Carl L. Parker, Director
Donn Powell Director

COPY

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September 25, 1989

Chairman and Members of the
Board of Directors
Great Lakes Coalition
Wisconsin Lake Michigan
Shoreline Chapter, Inc.
P. O. Box 1325
Sheboygan, Wisconsin 53082-1325

Dear Chairman and Board Members:

The Chairman of the Intergovernmental Coordinating and Technical Advisory Committee for the Milwaukee County Lake Michigan Shoreline Erosion Management Plan has asked that the Commission staff formally acknowledge your letter of September 16, 1989, concerning the preliminary Lake Michigan shoreline erosion management plan for Milwaukee County. The thrust of your comments is that if Lakes Michigan-Huron were to be regulated, such regulation could have a significant impact upon the specific plan recommendations and attendant costs with respect to protecting the Lake Michigan shoreline in Milwaukee County. Accordingly, you urge the Committee to contact the Governor and federal legislators and seek their support for completion of the current International Joint Commission study attendant to such regulation.

Please be assured that your letter will be entered into the record of the public hearing on the Milwaukee County Lake Michigan shoreline erosion management plan scheduled to be held on Wednesday, September 27, 1989. Following that hearing, the Committee will meet to consider all of the comments and suggestions received, and determine an appropriate course of action with respect to the recommendations to be contained in the final plan.

Thank you for your interest in this important matter.

Sincerely,

Kurt W. Bauer
Executive Director

KWB/rj
A043
cc: Daniel Cupertino, Chairman, Intergovernmental Coordinating and Technical Advisory Committee for the Milwaukee County Lake Michigan Shoreline Erosion Management Plan

City of St. Francis
OFFICE OF THE MAYOR

4235 South Nicholson Avenue
St. Francis, Wisconsin 53207
(414) 481-2300

Milton Vretenar
Mayor
Res: 744-7218

September 28, 1989

Supervisor Daniel Cupertino, Chairman
Erosion Control Management Study Committee
South Shore Pavilion

RE: PROPOSED MILWAUKEE COUNTY'S LAKE MICHIGAN SHORELINE EROSION PLAN

Dear Supervisor Cupertino:

I would like to take this opportunity, for the record, to extend our thanks to Kurt Bauer, Executive Director, and the staff of the SEWRPC, for their work on the proposed erosion management plan for Milwaukee County. We have viewed shoreline erosion as a serious concern for our community's future and have over the years supported both studies and actual erosion control projects which will help save this unique and invaluable natural resource for future generations.

However, as Mayor of the City of St. Francis, I have severe reservations and can not support the plan the Committee has recommended. The Committee has recommended the demolition of the breakwater that now protects the shoreline property which lies within the City of St. Francis. Furthermore, the Committee has recommended that the salvaged stone be used to help protect the shoreline area currently protected by the breakwater which would be placed between East Oklahoma Avenue extended and the former lakeside power plant dike. This would protect both public and private shoreline now protected by the south shore breakwater.

There is no assurance that if the City went on record in favor of this recommended design that politically this plan would come to pass in its entirety. Once the breakwater is gone, it is gone forever and the political reality is that the City of St. Francis may not have the clout to insure the construction of on-shore revetments and groins in our community as is recommended. Thus, a partial use of this report's recommendations may adequately protect the shoreline of our sister city to the north, who may have the political pull to protect their shoreline to the detriment of the City of St. Francis. This possibility leaves us with no recourse but to oppose the plan.

We strongly recommend that Alternative No. 1, which would reconstruct the entire breakwater to 588.6 feet above NGVD, as the only acceptable alternative. Alternative No. 1 treats all affected property owners in Milwaukee County, whether in St. Francis or Milwaukee, equally. It does not differentiate between community boundaries and the County Board in adopting this alternative would assume its rightful responsibility to protect uniformly the existing lakefront within Milwaukee County.

I, as Mayor of the City of St. Francis, a member of the Technical Advisory Committee for Milwaukee County Lake Michigan Shoreline Erosion Plan, strongly recommend that Alternative No. 1 be adopted by this Committee and the concerns I make in this letter be made a part of the public hearing record.

Sincerely,

Milton Vretenar
Mayor

MV/kh

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Special acknowledgement is due the private consultants, contractors, and lakefront property owners who supplied valuable information for the study.