A LAKE MICHIGAN SHORELINE
EROSION BLUFF RECESSION, AND
STORM DAMAGE CONTROL PLAN

FOR MILWAUKEE COUNTY, WI

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A LAKE MICHIGAN SHORELINE EROSION, BLUFF RECESSION, AND STORM DAMAGE CONTROL PLAN FOR MILWAUKEE COUNTY, WISCONSIN

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SEWRPC COMMUNITY ASSISTANCE PLANNING REPORT NO. 163

A LAKE MICHIGAN SHORELINE EROSION, BLUFF RECESSION, AND STORM DAMAGE CONTROL PLAN FOR MILWAUKEE COUNTY

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, bluff recession, and storm damage 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 June 1988. 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, the Port of Milwaukee, 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 U.S. Army Corps of Engineers, the Wisconsin Electric Power Company, concerned citizens, and the Regional Planning Commission. The full membership of the Advisory Committee is listed on the inside of the 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 the enhancement of the overall quality of life of the residents of the area through the selective protection of those environmental values--recreational, aesthetic, ecological, and cultural--normally associated with, and found concentrated in, coastal areas.

NEED FOR A SHORELINE EROSION, BLUFF RECESSION, AND STORM DAMAGE 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, parkland, a few public roadways, and some industrial sites. Bluff and terrace recession rates in the Milwaukee County study area range up to 12 feet per year, resulting in the loss of nearly 6.5 acres of land each year, and about 850,000 cubic yards of shore material. This annual amount of eroded material would fill over 5,200 railroad boxcars, which, if placed

end to end, would form a line 55 miles long. Figure I-1 shows a house in the City of Oak Creek threatened by bluff recession in 1973.

The second major impact of coastal processes is the 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; 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;

- The need for more adequate knowledge about the adverse as well as beneficial effects of the various nonstructural and structural

Figure I-1

CITY OF OAK CREEK RESIDENCE THREATENED BY BLUFF RECESSION: 1973

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

Photograph courtesy of Oak Creek Pictorial.

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 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, and the County and 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 Control Measures"; and Chapter IV, "Alternative Shoreline Erosion, Bluff Recession, and Storm Damage Control Plans and a Recommended Plan", 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 I-2. The proposed islands would 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.

   From the 1880s 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 I-3. The resort was subsequently sold and the lot was subdivided for residential development. In the late 1920s and early 1930s, 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 nearshore 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 about a 530-foot reach of 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

Figure I-3

WHITEFISH BAY RESORT: 1900

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

At the turn of the century, the Whitefish Bay Resort, located just north of E. Henry Clay Street, was a popular gathering spot for area residents. However, business declined in the early 1900s, reportedly in part due to increased use of the automobile. The resort was sold around 1915 and the property developed for residential use. In the late 1920s and early 1930s, bluff slope failure occurred.

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 1970s, concrete rubble and soil was placed on the bluff slopes to stabilize those slopes.

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; nearshore bathymetry; sources and movement of the beach material; 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 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 plan. 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 I-4. The groin systems at Grant Park and Sheridan Park have not been nourished with sand; but they have continued to contain beaches and provide marginal protection of the bluffs. These groins were periodically repaired and maintained in the 1940s and 1950s, but have not been recently maintained and are in need of substantial repair. Figure I-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

Figure I-4

LAKE MICHIGAN SHORELINE NEAR LAKE PARK: 1908 and 1986

Lake Park: circa 1908 Photograph by E. C. Kropp Publications.

The 1908 photograph 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 has eroded, allowing wave action during intense storms to wash debris onto the roadway, as shown in the December 1986 photograph.

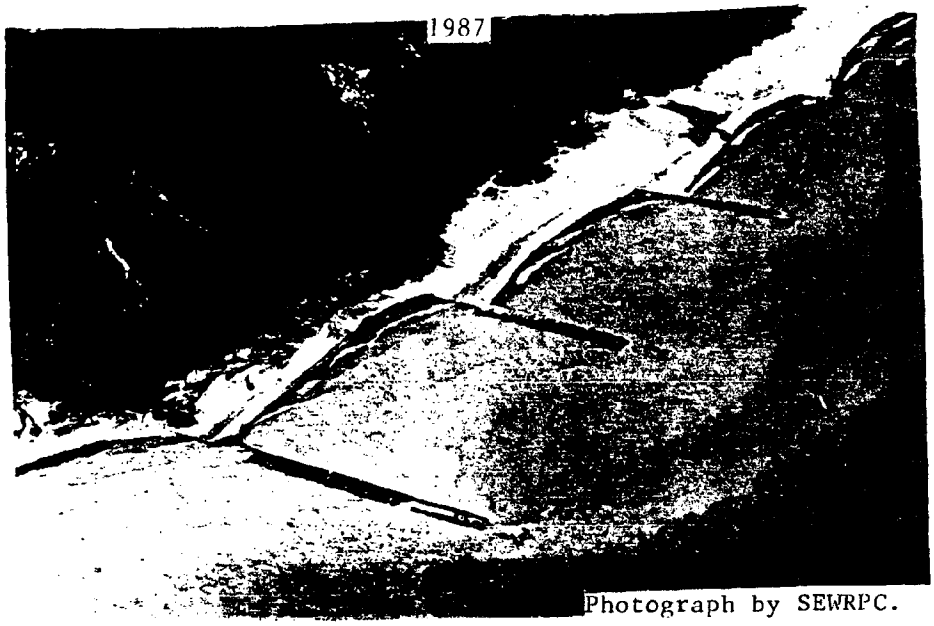
Lincoln Memorial Drive: 1986 Photograph by SEWRPC.

Figure I-5

SHERIDAN PARK GROINS: 1945 and 1987.

1945

Photograph by Milwaukee County.



Photograph by SEWRPC.

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, about 3.1 acres of beach had been formed. The eleven permeable groins are constructed of precast concrete beams and sills arranged in an overlapping crisscross fashion with a cover of solid concrete slabs. Due to 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.

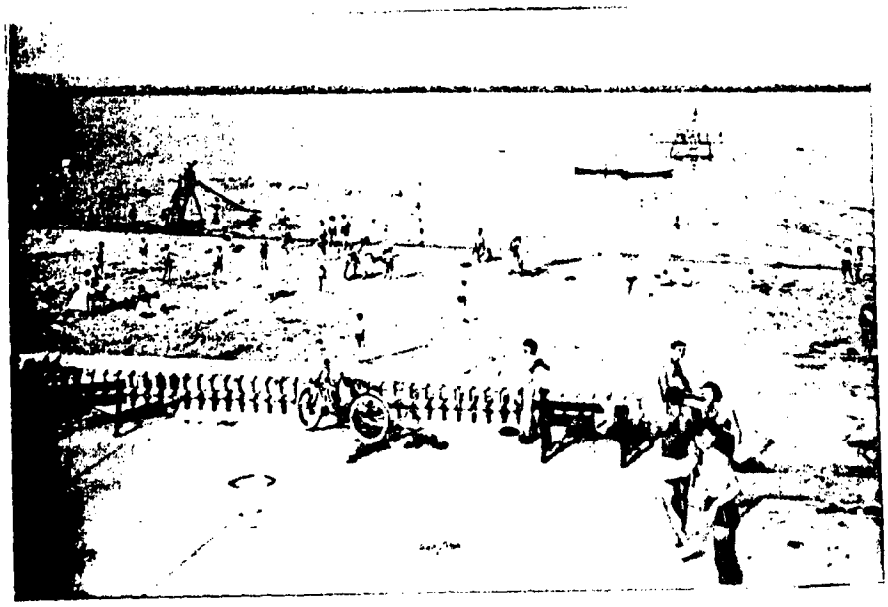
cooperatively and concurrently with the U.S. Army Corps of Engineers' Beach Erosion Study described above, presented information on Lake Michigan water levels, geologic conditions, the extent of shore erosion, shoreline recession rates, existing shore protection structures, and beach conditions. To protect 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 I-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 should consist of riprap revetments or concrete bulkheads, along with groins for 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 1950s at Doctors Park. Again, no action was taken on the proposed northerly extension of Lincoln Memorial Drive.

5. Master Plan-Milwaukee County Marina Development, Ralph H. Burke, Inc., Engineers-Architects, Chicago, IL, November 1958.

Figure I-6

MILWAUKEE SOUTH SHORE BREAKWATER.



Photograph courtesy of the Milwaukee Public Library. (undated)

1987

Photograph by SEWRPC.

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 Railway and 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 1948, Milwaukee County assumed responsibility for maintaining the breakwater, but little maintenance has actually been performed. As shown in the bottom 1987 photograph at the Texas Avenue water treatment plant, the breakwater has deteriorated and is being overtopped by waves.

At the request of the Milwaukee County Park Commission, a study was made 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 Avenue 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 by 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. Problems of Great Lakes Shore Erosion, W. T. Painter, a paper presented at First World Congress on Water Resources, Chicago, IL, September 1973.

This paper presented the findings of investigations of the causes of shoreline erosion and major bluff failures which occurred along the Lake Michigan shoreline, including one major slide which occurred in April 1973 at 5270 N. Lake Drive in the Village of Whitefish Bay. This property is located within the reach of shoreline 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 nearshore 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 I-7.

7. A Geological Reconnaissance of Bender Park, Milwaukee County, Wisconsin, 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 park was of limited recreational value. Most of the shoreline lacked beaches, and the use of the limited existing beaches was hazardous, due to the threat of slides and slumps. The report concluded further that protection of the shoreline would be costly and require regrading the bluffs; lowering the elevation of the groundwater table; and 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.

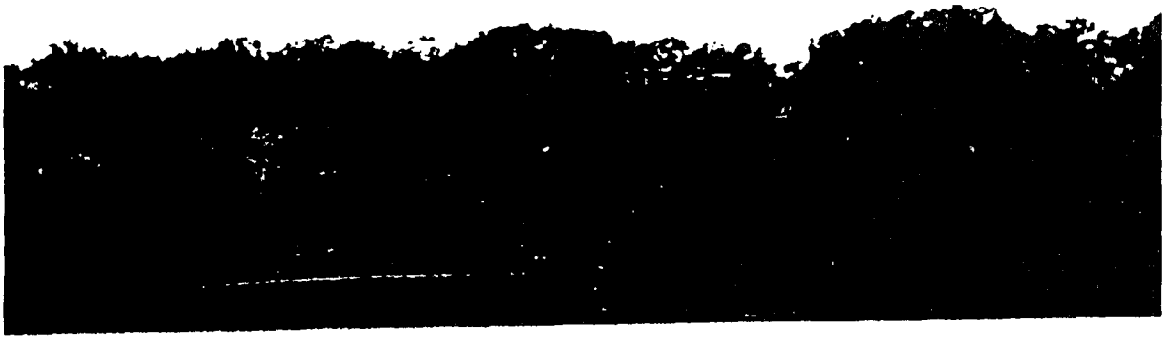
8. Bender Park Boat Marina Justification, City of Oak Creek, April 1975.

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 on the economic value of the park land. The study found that due to 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

Figure I-7

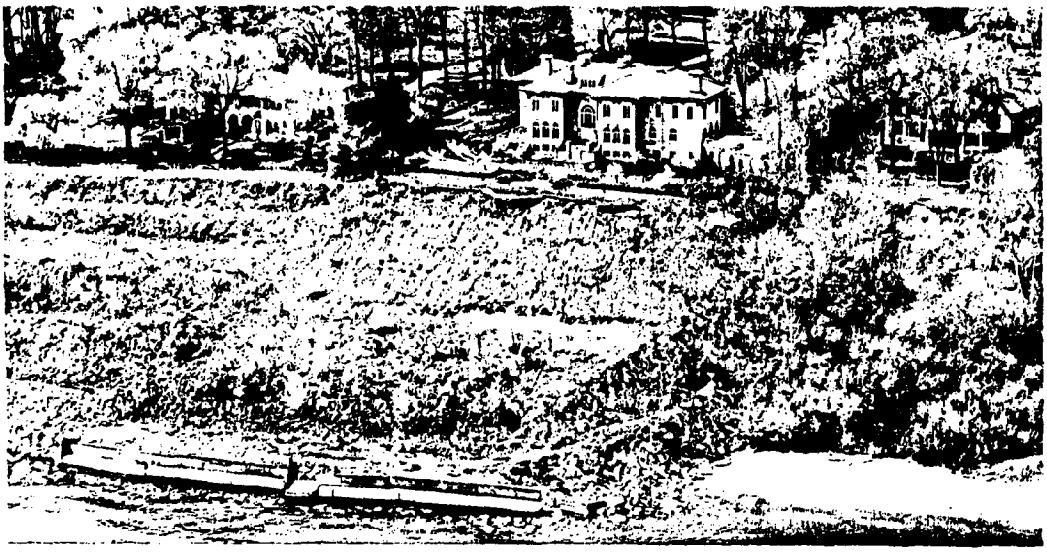
BLUFF SLOPE FILL PROJECT AT 5270 NORTH LAKE DRIVE, VILLAGE OF WHITEFISH BAY: 1976 and 1986

1976



Photograph by James D. Rosenbaum.

1986



Photograph by Robert T. McCoy.

A major bluff slippage occurred at 5270 N. Lake Drive in April, 1973, following a severe rainfall. Later that year, concrete rubble and soil was 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.

than normal rate, with bluff recession rates of up to 20 feet per year being recorded during the period of 1961 through 1974. 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. Alternative shore protection measures which 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 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 ten 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 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 long-term--110-year--bluff recession rates ranging 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. 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 funding 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 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 damages. 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 was within reasonable limits so that construction could be undertaken over a six to twelve-year period. The study also noted that the implementability of the plan by the County 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.

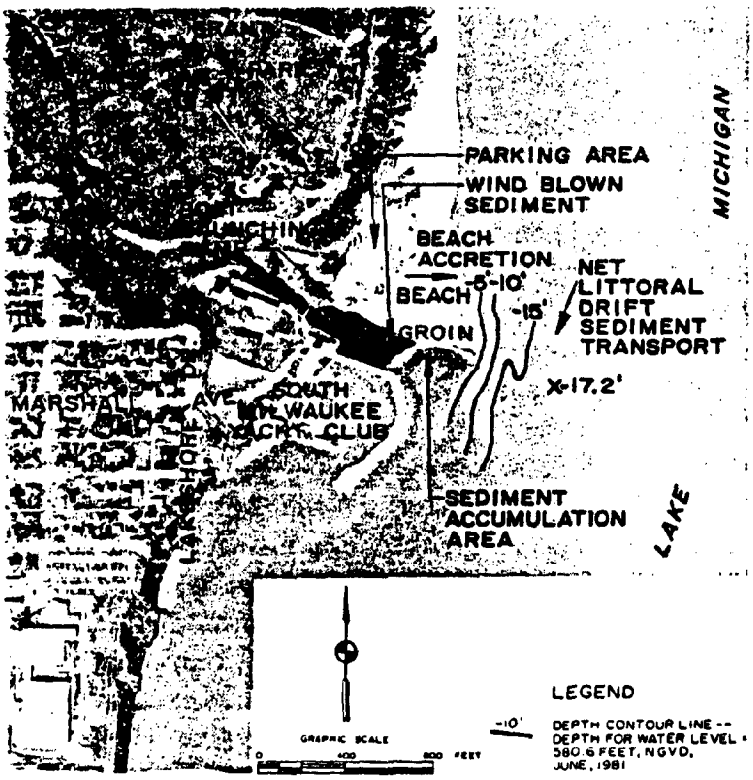
13. Geological Study, Lake Michigan at Mouth of Oak Creek, Edith M. McKee, Consulting Geologist, Winnetka, IL, October 1983.

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 I-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 existing 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 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.

Figure I-8

THE MOUTH OF OAK CREEK AT LAKE MICHIGAN: 1985



Photograph by SEWRPC.

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.

14. 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, existing coastal erosion problems; and bluff recession rates. The study evaluated alternative structural shore protection measures and identified shoreline erosion risk distances and associated recommended setback distances for buildings and facilities along shoreline reaches if proper structural shore protection measures would be provided, as well as if such measures would not be provided.

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. The development 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.

15. 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 under 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 future 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, a concrete rubble and soil landfill as of 1988 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.

16. 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 he 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 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 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 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.

17. 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 by a 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 costs 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 park lands, 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.

18. A Water Resources Management Plan for the Milwaukee Harbor Estuary, Southeastern Wisconsin Regional Planning Commission, Planning Report No. 37, Vol. Two, March 1988.

In 1982 the Regional Planning Commission undertook a comprehensive study of the water pollution, flooding, storm damage, an 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 an anchorage facilities. The study presented data on existing land use, climate, topography, nearshore 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.

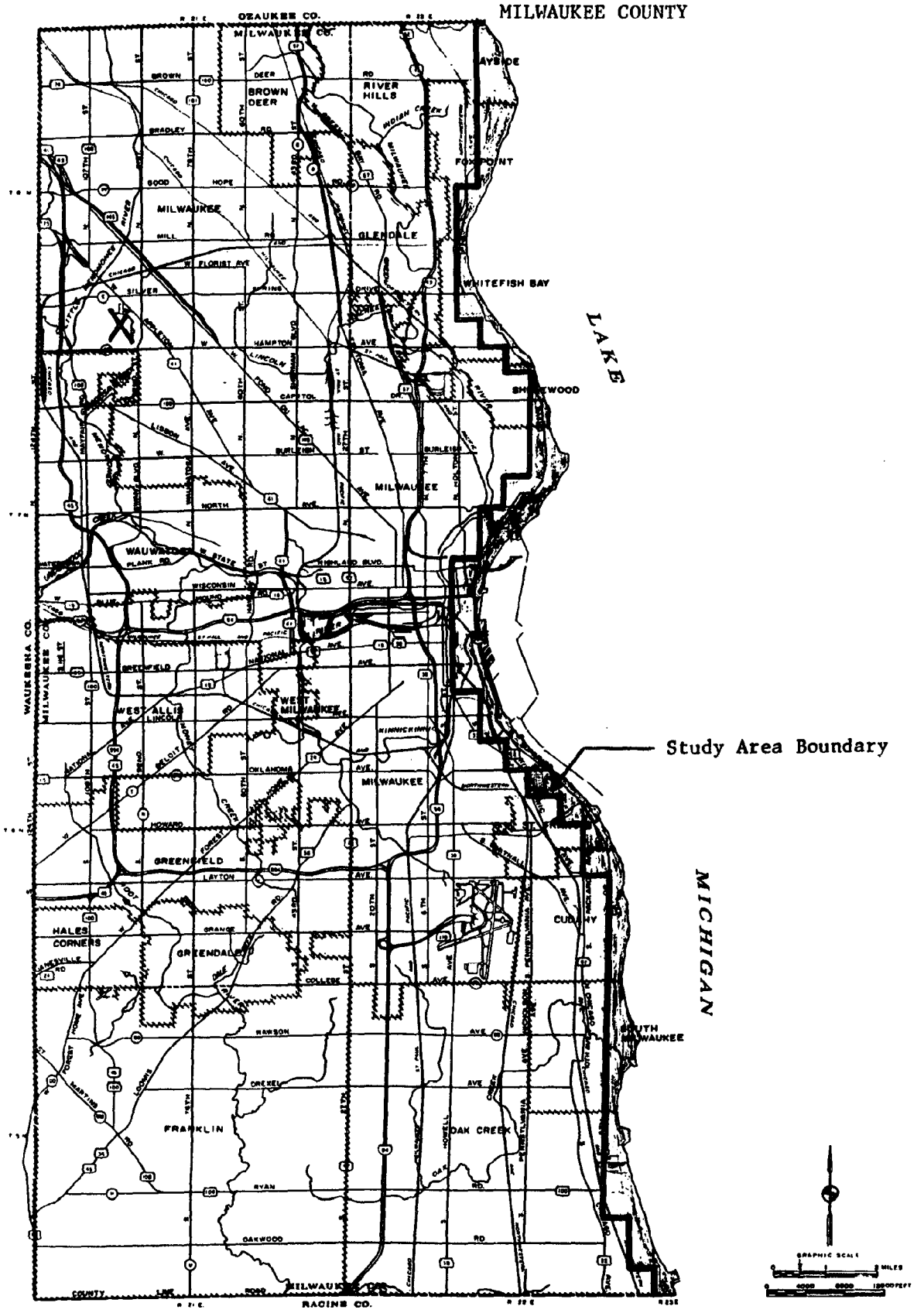
The study concluded that, except for routine maintenance, the outer harbor breakwaters do not need to be substantially modified at this time. It was recommended that revetments, bulkheads, and dockwalls continue to be constructed and maintained in order to protect facilities within the outer harbor. With regard to the McKinley Marina, three alternatives were considered to abate ice damage problems: melting of ice in the entire anchorage area by diffused compressed air; 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.

SHORELINE EROSION, BLUFF RECESSION, AND STORM DAMAGE 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 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 I-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, 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 unique recreational opportunities which attract users from throughout the greater Milwaukee area.

Map I-1

LAKE MICHIGAN SHORELINE EROSION, BLUFF RECESSION, AND STORM DAMAGE MANAGEMENT STUDY AREA FOR MILWAUKEE COUNTY



Source: SEWRPC

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 changing 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 design parameters for, both structural and nonstructural shore protection measures for the portion of the study area which 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.

It should be noted that a comprehensive shoreline erosion and bluff recession management study was completed by the Regional Planning Commission in March 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 were 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.[1] As shown in Figure I-9, shoreline erosion problems were described and evaluated for Atwater Park, Big Bay Park, Buckley Park, and Klode Park. The findings and recommendations of the northern Milwaukee County study were fully incorporated into this study. New study efforts, therefore, concentrated on that portion of the shoreline not previously studied by the Commission, consisting of southern Milwaukee County, the Milwaukee Harbor area, and the Village of Bayside.

SUMMARY

In January of 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 12 feet per year with the loss of about 850,000 cubic yards of shore material per year. Information needs

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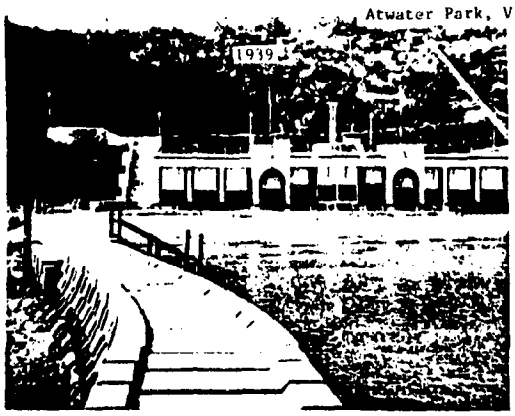
[1]SEWRPC Community Assistance Planning Report No. 155, Northern Milwaukee County Lake Michigan Shoreline Erosion Management Study, 1988.

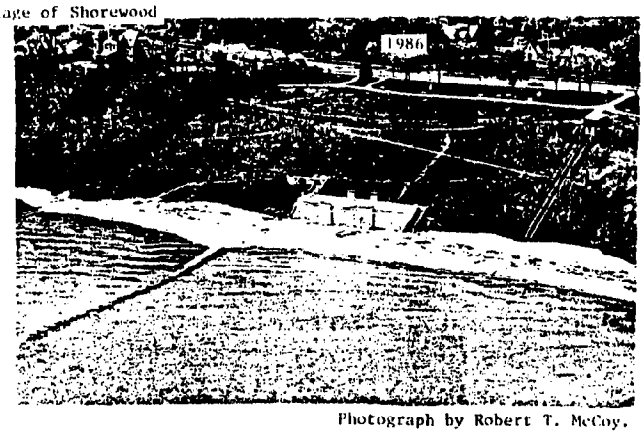
Figure I-9

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

Atwater Park, Village of Shorewood

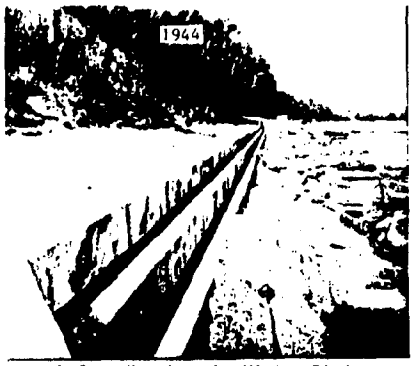


Photograph from American Guide Series-Shorewood, by the Village of Shorewood, 1939.

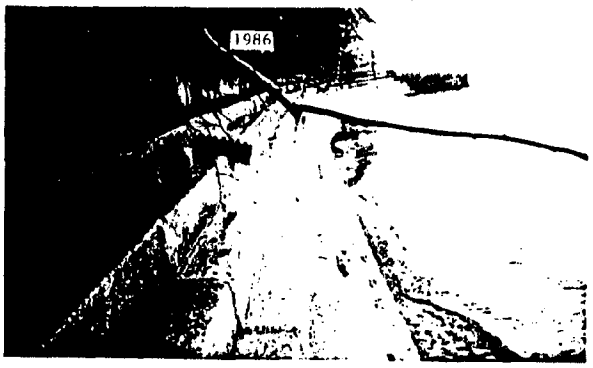


Photograph by Robert T. McCoy.

Big Bay Park, Village of Whitefish Bay



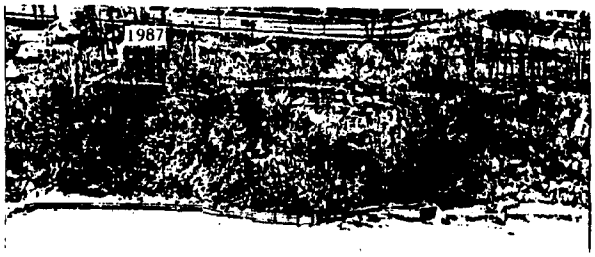
Photograph from Erosion, by Miriam Bird.
Photograph courtesy of the Whitefish Bay Public Library.



Photograph by SEWRPC.

Buckley Park, Village of Whitefish Bay

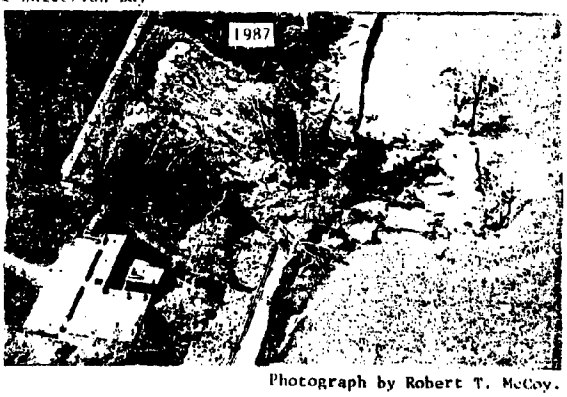
Photograph courtesy of the Wisconsin Coastal Management Program.



Photograph by Robert T. McCoy.

Klode Park, Village of Whitefish Bay

Photograph from Erosion, by Miriam Bird.
Photograph courtesy of the Whitefish Bay Public Library.



Photograph by Robert T. McCoy.

Figure I-9 caption

These parks in northern Milwaukee County, which have been protected by shore protection structures since at least the 1930s and 1940s, 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. 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.

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 local units of government with respect to 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, and to recommend an erosion and bluff recession control plan; and to identify an appropriate implementation program to carry out that plan.

The study area consists of 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 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-1970s, and more recently in 1985 through 1987. 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 eighteen 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; Pittsburg Testing Laboratory, 1985); and for the entire County shoreline (Mickelson, et al, 1977). Bluff recession rates, which range from am average of less than one foot per year to a maximum of more than sixty feet per year, were presented in six 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). These studies found that the bluffs are generally composed of relatively impermeable glacial tills. Permeable lake sediments are usually located between these tills. Groundwater seepage within these lake sediments, as well as bluff toe erosion by wave action, are the major causes of bluff slope failure.

Relatively few studies (U.S. Army Corps of Engineers, 1945; McKee, 1983) have evaluated coastal processes such as wave conditions and littoral drift for entire reaches of shoreline. 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 Commission, 1934; Milwaukee County Committee on Lake Michigan Shore Erosion, 1945; U.S.

Army Corps of Engineers, 1945 and 1975a). Five 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). 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). Three studies (U.S. Army Corps of Engineers 1945 and 1975a; SEWRPC, 1984) recognized that any new groin systems in Milwaukee County would likely need to be artificially nourished with sand. 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, 1988).

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 lake 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 respect and work in harmony with natural coastal processes, and which provide a useable, accessible shoreline for recreational activities under varying lake level conditions. Recent beach restoration projects include the McKinley Beach project being developed by Milwaukee County and the Klode Park project being 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, bluff recession, and storm damage control 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 useability 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 I-10, have been built on the lakefront. About 12 lineal miles of lakefront park and open land such as Juneau Park shown in Figure I-11, 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 recreational 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 useable lakefront serving both lakefront property owners and the population of the County as a whole.

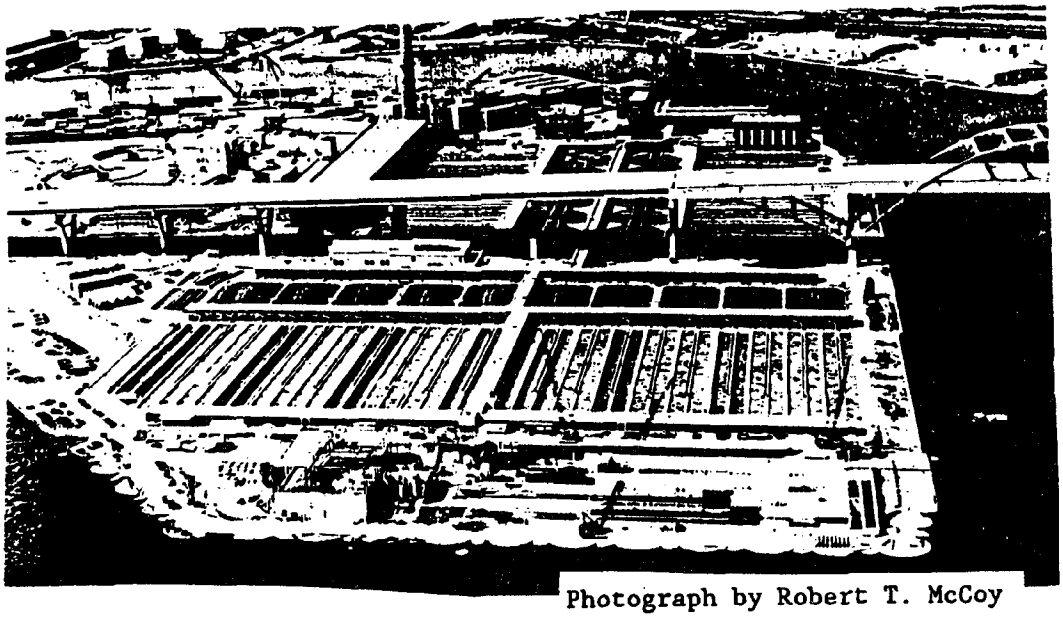
Figure I-10

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

Early 1920s Fishing Village

Photograph courtesy of the Milwaukee County Historical Society

1987 - Treatment Plant

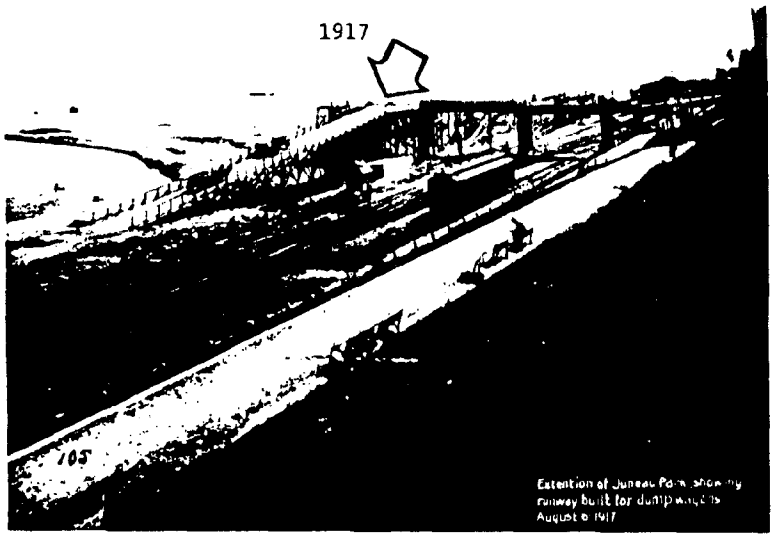


Photograph by Robert T. McCoy

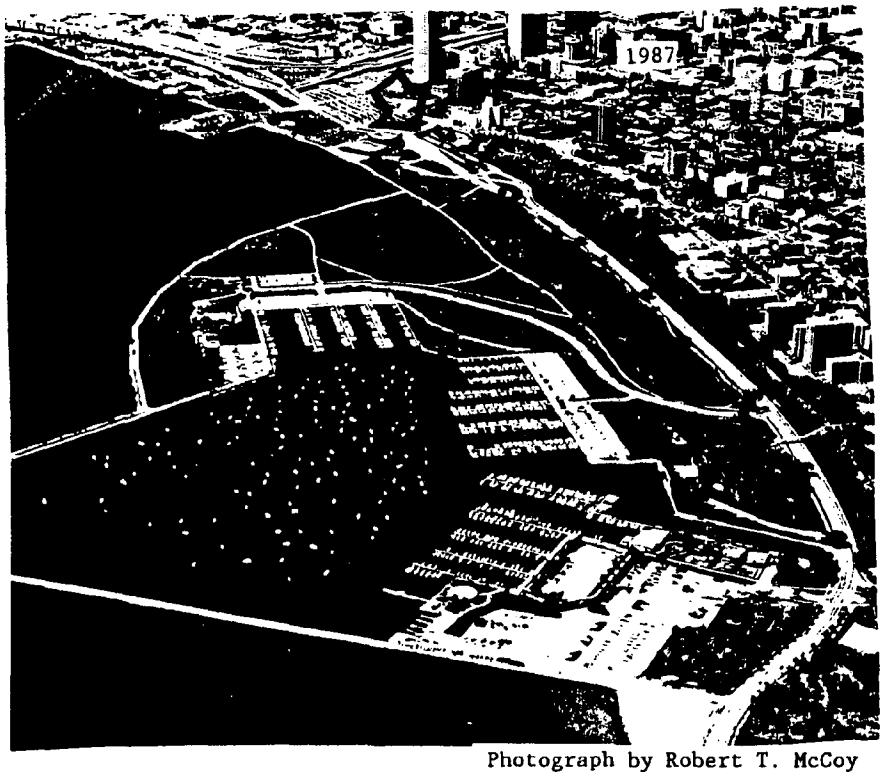
Prior to the construction of the Jones Island sewage treatment plant in 1925, Jones Island was a thriving community of fisherman and their families. The fishing village was removed from Jones Island in the early 1920s to allow construction of the treatment plant. The treatment plant, the first in the United States to use the air activated sludge process on a large-scale basis, underwent major expansions in 1935, 1952, and 1987-88, as shown in the above photograph.

-Figure I-11-

LANDFILL AT JUNEAU PARK: 1917 AND 1987



Photograph courtesy of the Milwaukee Public Museum.



Photograph by Robert T. McCoy

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.

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SEWRPC COMMUNITY ASSISTANCE PLANNING REPORT NO. 163

A Lake Michigan Shoreline Erosion, Bluff Recession, And Storm Damage Control Plan For Milwaukee County.

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, includes that portion of Milwaukee County which most directly affects, and is most affected by, Lake Michigan shoreline erosion. 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,2-foot contour interval, topographic maps prepared for the Village of Bayside Shoreline; and by up dated topographic maps prepared for the City of Oak Creek Shoreline. These new and up-dated 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 historic 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 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

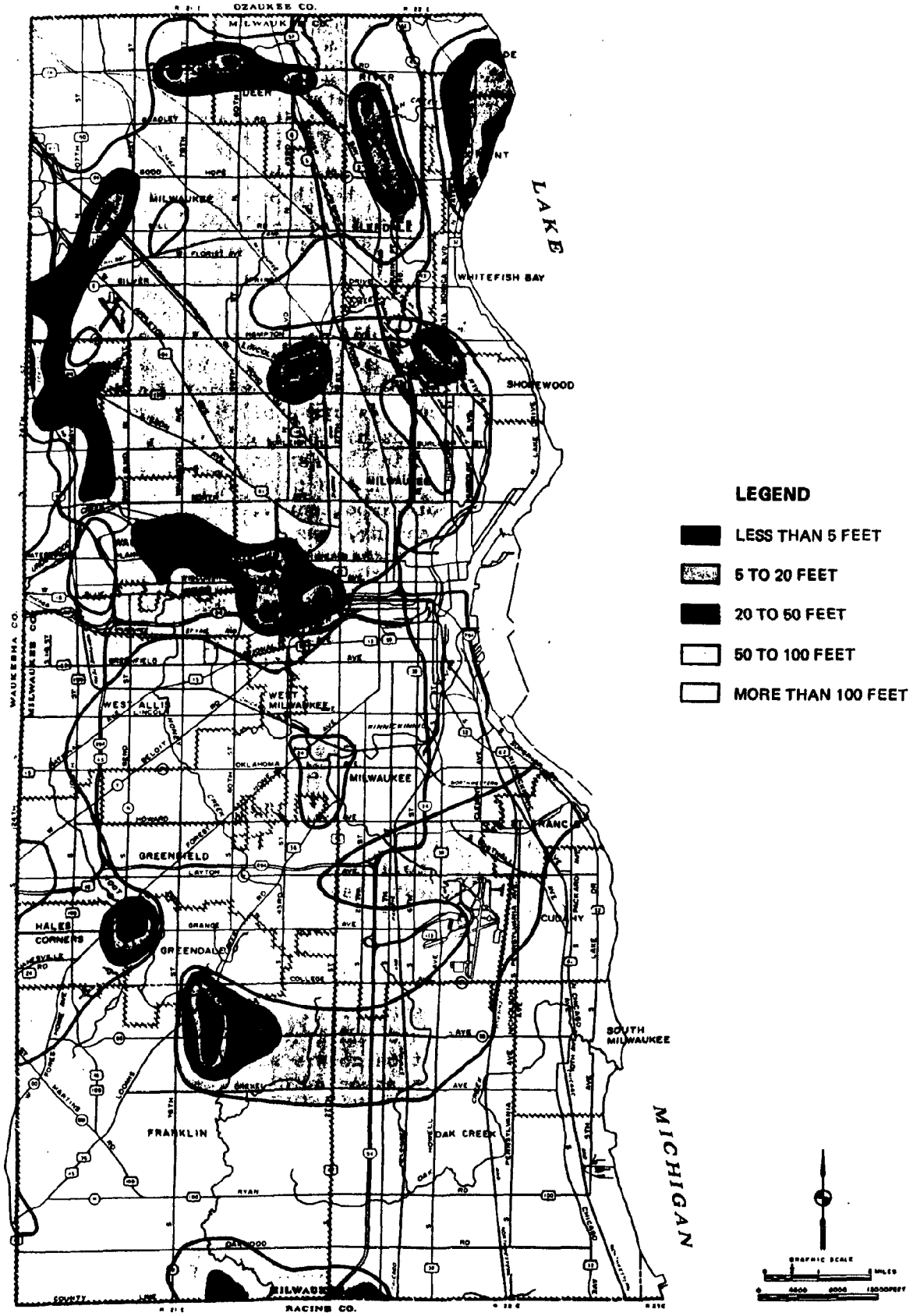
area. The Silurian and Devonian Formations are covered by glacial deposits whose thickness ranges from less than 50 feet along the shoreline in the Villages of Fox Point and Bayside, to more than 200 feet in the City of Milwaukee. Map II-1 indicates the spatial variation of the thickness of the unconsolidated 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 Wisconsin 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 by the Harvard sublobe of the Lake Michigan lobe 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

Map II-1

THICKNESS OF UNCONSOLIDATED MATERIALS IN MILWAUKEE COUNTY



Source: U. S. Geological Survey and SEWRPC.

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 storm water 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 for 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.

Detailed soils mapping was conducted within the Cities of Oak Creek, South Milwaukee, Cudahy, and St. Francis; and the Villages of Fox Point and Bayside. Detailed soils mapping was not conducted within the City of Milwaukee and Villages of Shorewood and Whitefish Bay because, due to the intensity of the urban development in these communities, the natural soils were greatly disturbed and the soil boundaries 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 storm water runoff, the most significant soil interpretation is the categorization of soils into four

hydrologic soil groups: A, B, C, and D. In terms of runoff characteristics, these four hydrologic soil groups are defined as follows:

1. Hydrologic Soil Group A: Very little runoff because of high infiltration capacity, high permeability, and good drainage.

2. Hydrologic Soil Group B: Moderate amounts of runoff because of moderate infiltration capacity, moderate permeability, and good drainage.

3. Hydrologic Soil Group C: Large amounts of runoff because of low infiltration capacity, low permeability and poor drainage.

4. Hydrologic Soil Group D: Very large amounts of runoff because of low infiltration capacity, low permeability, and poor drainage.

As indicated in Table II-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 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, the Kewaunee soils form in finer texture material than Morely soils. The area not surveyed in northern Milwaukee County--within

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Table II-1

SOIL TYPES IN THE MILWAUKEE COUNTY SHORELINE MANAGEMENT STUDY AREA

Soil Type	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	
	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	<.1
HYDROLOGIC SOIL GROUP D																				
Askum Silty Clay Loam	35.7	3.3	0.3	<.1	--	--	--	--	--	--	--	--	--	--	--	--	--	--	36.0	0.5
Marsh	3.1	0.3	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	3.1	<.1
Ogden Muck	2.1	0.2	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	2.1	<.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	606.9	100	--	--	--	--	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	<.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.

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 II-2 summarizes the lengths of shoreline within various bluff height ranges. In the southern portion of the study area, within the Cities of Oak Creek, South Milwaukee, and Cudahy, 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 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, 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 terrace 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

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Table II-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	32.1
11- 20	-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

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 Village of 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, determination of the underlying stratigraphy was made using historical geologic records or soil boring data. Eight additional soil borings were taken in March 1988 by Wisconsin Testing Laboratories, Inc., under contract to the Regional Planning Commission, in areas where 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.

Table II-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 was performed by the Department of Civil Engineering, University of Wisconsin-Madison. The laboratory analyses, the results of which are summarized in Table

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Table II-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
General Lake Sediment:	
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.

II-4, evaluated those of 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-sized 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 was 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

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Table II-4

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

Soil Type	Location[a] (Bluff Analysis Section)	Depth (Ft)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Percent Gravel and Sand	Percent Silt and Clay	Cohesion Intercept C' (PSF)	Effectiv Friction Angle Ø (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	--	--

Table II-4 (cont'd)

Soil Type	Location[a] (Bluff Analysis Section)	Depth (Ft)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Percent Gravel and Sand	Percent Silt and Clay	Cohesion Intercept C' (PSF)	Effectiv Friction Angle Ø (Degree)
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	0	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	--	--
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

Table II-4 (cont'd)

Soil Type	Location[a] (Bluff Analysis Section)	Depth (Ft)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Percent Gravel and Sand	Percent Silt and Clay	Cohesion Intercept C' (PSF)	Effectiv Friction Angle Ø (Degree)
				Soil Borings					
Lake Sediments									
Medium-	92	85	--	--	--	81	19	--	--
Fine	18	15(top)	20	5	15	35	65	230	27
Sand		15	27	18	9	6	94	--	--
Silt		(bottom)							
	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	24	40	--	--	--	--	--	0	37
Sand	68	45	--	--	--	68	32	--	--
	87	70	--	--	--	55	45	--	--
	87	85	--	--	--	11	89	--	--
Clay and	31	25	23	14	9	--	--	--	--
Silt	78	25	27	--	--	1	99	--	--
	84	35	29	18	11	2	98	--	--
		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	68	40	23	14	9	26	74	--	--
Sand and	78	20	--	18	--	21	79	--	--
Silt	84	30	--	--	--	23	77	--	--
	92	30	--	--	--	36	64	--	--
	92	70	--	--	--	36	64	--	--

[a]The location of the Bluff Analysis Sections are shown on Map II-11.

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

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. Table II-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.

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. During the periods surveyed, 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 occurring 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.

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

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Table II-5

BEACH CHARACTERISTICS OF THE LAKE MICHIGAN
SHORELINE OF MILWAUKEE COUNTY

	Beach Width (Feet)									
	<10		11-50		51-90		>90			
Beach Composition	Shoreline Length (feet)	Percent of Total Shoreline Length	Shoreline Length (feet)	Percent of Total Shoreline Length	Shoreline Length (feet)	Percent of Total Shoreline Length	Shoreline Length (feet)	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 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 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

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 II-5 also indicates the beach widths along the shoreline. Within the portion of the study area which includes southern Milwaukee County and the Village of Bayside, about 19 percent 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, South Shore Park Beach, and 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 II-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 II-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. Within the northern Milwaukee County study area in 1986, about 3 percent of the total 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

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Table II-6

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

	Northern Milwaukee County Study Area Shoreline[a]		Remaining Shoreline of Milwaukee County[b]		Total Study Area Shoreline	
Beach Slope (Degrees)	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
715	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**

and were composed of finer-grained materials; whereas the narrower beaches tended to have steeper slopes and were composed of coarser-grained materials.

Nearshore Bathymetry

The nearshore 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 is available for the entire Milwaukee County Lake Michigan shoreline from the National Oceanic and Atmospheric Administration (NOAA). As presented in Table II-7, along portions of the shoreline, the NOAA bathymetric data have been updated by various governmental and private sources.

The nearshore slopes are 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 nearshore 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 the northeast-facing shoreline in the Village of Whitefish Bay between Klode Park and Big Bay Park.

The nearshore bathymetry within the study area was previously surveyed in 1871, 1912, and 1944.[1] A review of these early data indicated that in 1871 and 1912 the nearshore slopes were more gentle than those presently survey. The bathymetric survey results in 1944 were similar to the existing conditions. High water levels, such as those which 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 nearshore zone somewhat steeper in the future, unless measures such as beach nourishment are implemented.

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

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Table II-7

SOURCES OF UPDATED BATHYMETRIC DATA

General Location	Source
Oak Creek Power Plant	Wisconsin Electric Power Company
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 Assoicates Inc., South Shore and Bay View Park Shoreline Restoration Concepted Design and Costs, in preparation, 1988.
Milwaukee Harbor Area	U. S. Army Corps of Engineers Sounding Maps, 1985 National Oceanic and Atmospheric Administration, Sounding Maps-Milwaukee Harbor, 1984
McKinley Beach	Warzyn Engineering, 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, et al, A Future for the Milwaukee County North Shore, 1987

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.

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.

The deep sandstone aquifer, which is more than 1,300 feet thick, underlies the entire County and is composed of Cambrian- 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 presented in Table II-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. Nine soil borings were previously 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. The technique introduces 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, its 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 till layers, and usually ranged in depth from 10 to 80 feet from the surface. Within northern Milwaukee County, an additional perched water table was usually found within the fractured Ozaukee till near the top of the bluffs.

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 storm water 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.

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Table II-8

SOURCES OF IDENTIFIED GROUNDWATER LEVELS WITHIN THE MILWAUKEE COUNTY BLUFFS

Civil Division	Bluff Analysis Section	Location	Observation of Seepage 1986-1987	Soil Boring 1986 & 1988	Observation Well Measurement	Estimate Electrical Resistivity Analysis	Based on Location of Permeable Soil Strata	Groundwater Not Estimated[a]
City of Oak Creek	1	WEPCO Oak Creek Power Plant	--	--	--	--	--	x
	2	Elm Road-Oakwood Rd	x	--	--	--	--	--
	3	Bender Park	x	--	--	--	--	--
	4	Bender Park	x	x	--	--	--	--
	5	Bender Park	x	--	--	--	--	--
	6	9300 S. 5th Ave.	x	--	--	--	--	--
	7	9180 S. 5th Ave.	x	--	--	--	--	--
	8	9170 S. 5th Ave.	--	--	--	--	x	--
	9	4301 E. Depot Rd.	--	--	--	--	x	--
	10	9006 S. 5th Ave.	--	--	--	--	x	--
	11	9006-8740 S. 5th Ave.	x	--	--	--	--	--
	12	South Shore Treatment Plant	--	--	--	--	--	x
	13	8400 S. 5th Ave.	--	--	--	--	x	--
City of South Milwaukee	14	3817-3509 3rd Ave.	x	--	--	--	--	--
	15	235 Lakeview Ave.-3303 Marina Road	--	--	--	--	x	--
	16	3303 Marina Road - 3333 5th Ave.	--	--	--	--	x	--
	17	3333 5th Ave.	--	--	--	--	x	--
	18	South Milwaukee Water Utility-Marshall Ave.	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 Ave.-Warnimont Park	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 Ave.-100's of Howard Ave.	x	--	--	--	--	--
	40	100's of Howard Ave.-Power Plant Breakwater	--	--	--	--	x	--
	41	WEPCO Lakeside Power Plant	--	--	--	--	--	x

Table VII-8 (cont'd)

Civil Division	Bluff Analysis Section	Location	Observation of Seepage 1986-1987	Soil Boring 1986 & 1988	Observation Well Measurement	Estimate Electrical Resistivity Analysis	Based on Location of Permeable Soil Strata	Groundwater Not Estimated[a]
City of Milwaukee	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-South Shore Park	--	--	--	--	x	--
	48	South Shore Park	--	x	--	--	x	--
	49	Texas St. Water Intake	--	--	--	--	--	x
	50	South Shore Park	--	--	--	--	x	--
	51	South Shore Park	--	--	--	--	--	x
	52	South Shore Beach	--	--	--	--	x	--
	53	South Shore Yacht Club	--	--	--	--	--	x
	54	South Shore Park	--	--	--	--	--	x
	55	E. Russel Ave.-Jones Island STP	--	--	--	--	--	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 Ct.	--	--	--	--	x	--
	62	3378-3474 N. Lake Drive	--	x	--	--	--	--
Village of Shorewood	63	3510 N. Lake Dr.	x	--	--	--	--	--
	64	3534 N. Lake Dr.	--	--	--	--	x	--
	65	3550-3914 N. Lake Drive	x	--	--	--	--	--
	66	3426 N. Lake Dr.	--	--	--	--	--	--
	67	3932-3966 N. Lake Drive	--	--	--	x	x	--
	68	Atwater Park-4300 N. Lake Dr.	--	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 Dr.	--	--	--	x	--	--
	75	4794-4800 N. Lake Drive	--	--	--	--	x	--
	76	4810-4840 N. Lake Drive	--	--	--	x	--	--
	77	4850 N. Lake Dr.-Buckley Park	--	--	--	--	x	--

Table VII-8 (cont'd)

Civil Division	Bluff Analysis Section	Location	Observation of Seepage 1986-1987	Soil Boring 1986 & 1988	Observation Well Measurement	Estimate Electrical Resistivity Analysis	Based on Location of Permeable Soil Strata	Groundwater Not Estimated[a]
Village of Whitefish Bay (cont'd)	78	Buckley Park-Big Bay Park	--	x	--	--	x	--
	79	Big Bay Park-5270 N. Lake Dr.	--	--	--	--	x	--
	80	5290 N. Lake Dr.	--	--	--	x	x	--
	81	5300 N. Lake Dr.-808 Lakeview Avenue	--	--	--	--	x	--
	82	5722-5770 N. Lake Drive	x	--	--	--	--	--
	83	758 E. Day Ave.	--	--	--	--	x	--
	84	740 E. Day Ave.-5866 N. Shore Dr.	x	x	--	--	--	--
	85	Klode Park	x	--	--	x	--	--
	86	5960 N. Shore Dr.	x	--	--	--	--	--
	87	6000 N. Shore Dr.-6260 N. Lake Dr.	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 Dr.-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 Dr.	--	--	--	--	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]No bluff, or bluff obviously stable.

Source: SEWRPC

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 II-9 presents average monthly air temperature variations at the Milwaukee National Weather Service Station for the 35-year period from 1951 through 1985. As shown in the table, winter temperatures, as measured by the monthly means for December, January, and February, range from 18.6 to 24.9° F. Summer temperatures, as measured by the monthly means for June, July, and August, range from 64.9° to 70.3° F.

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. Nearshore 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 and annual total precipitation and snowfall for the Milwaukee National Weather Service Station are presented in Table II-10. The average annual total precipitation in the Milwaukee area was 31.81 inches over the 35-year period from 1951 through 1985. Average total monthly precipitation for the Milwaukee area ranged from 1.39 inches in February to 3.49 inches in April. The average annual snowfall and sleet, measured as snow and sleet, over the 35-year period was 50.2 inches.

Table II-9

AVERAGE MONTHLY AIR TEMPERATURE
AT MILWAUKEE 1951 THROUGH 1985

Month	Average Daily Maximum (°F)	Average Daily Minimum (°F)	Mean (°F)
January	25.9	11.2	18.6
February	30.5	16.2	23.4
March	39.5	25.1	32.3
April	53.5	35.7	44.6
May	64.8	44.7	54.8
June	74.9	54.8	64.9
July	79.2	61.3	70.3
August	78.4	60.4	69.4
September	71.1	52.6	61.9
October	59.8	42.0	50.9
November	44.8	30.0	37.4
December	31.8	17.9	24.9
Annual	54.5	34.7	46.1

Source: National Weather Service and SEWRPC.

Table II-10

AVERAGE MONTHLY PRECIPITATION AND SNOW AND SLEET AT MILWAUKEE: 1951 THROUGH 1985

Month	Average Total Precipitation (inches)	Average Snow and Sleet (inches)
January	1.60	12.8
February	1.39	10.4
March	2.61	10.0
April	3.49	2.3
May	2.81	Trace
June	3.43	0.0
July	3.47	0.0
August	3.15	0.0
September	2.89	Trace
October	2.48	0.2
November	2.32	3.1
December	2.17	11.4
Year	31.81	50.2

Source: National Weather Service and SEWRPC.

Assuming that 10 inches of measured snowfall and sleet are equivalent to one inch of water, the average annual snowfall of 50.2 inches is equivalent to 5.02 inches of water. Therefore, about 16 percent of the average annual total precipitation occurred as snowfall and sleet. The principal snowfall months are December, January, February, and March, during which 89 percent of the average annual snowfall may be expected to occur. 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.[2] 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 from 1841 through 1986, the maximum annual amount of precipitation at Milwaukee was 50.36 inches in 1876, or 58 percent above the 1951 through 1985 annual average. The maximum monthly precipitation 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

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

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 nearshore 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 nearshore 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 1930s 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. Commercial fisherman were also encouraged to harvest alewifes. Subsequently, the alewife populations have declined by about 85 percent since the mid-1970s, 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 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 Table II-11. Thirty species of fish were captured within the Milwaukee outer 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 three, 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 River.

Toxic Contamination

Environmental contamination by toxic organic substances and metals has become a widespread problem on the Great Lakes over the past twenty 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

Table II-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.

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 potential health risk to humans who consume contaminated organisms.

Although over 800 toxic contaminants have been identified within the Great Lakes,[3] 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 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), 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. PCBs, 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 PCBs for Lake Michigan fish is their diet, through a process referred to as biomagnification. Fish can also uptake PCBs directly from the water as it passes over their gills--referred to as bioconcentration.

In 1985, the Wisconsin Department of Natural Resources analyzed the tissue of 791 individual fish of six species of salmonids for concentrations of PCBs.[4] The study results, summarized in Table II-12, indicated the 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 ug/g. Brook trout, coho salmon, and rainbow

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

[4]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.

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Table II-12

MEAN POLYCHLORINATED BIPHENYL CONCENTRATIONS
IN SOUTHERN LAKE MICHIGAN SALMONIDS: 1985

Fish Species	No. 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 health standard for PCBs is 2.0 ug/g

Source: Wisconsin Department of Natural Resources

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 II-13.[5] Three chinook salmon and one lake trout were the only fish which exceeded the health standard of 2.0 ug/g for PCBs. No fish exceeded the health standards for chlordane or dieldrin--that standard being 0.3 ug/g for both substances.

Based on PCB measurement 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 should 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 1970s as the use of these substances has declined and the substances flushed from the lake or buried by cleaner sediments.[6,7] Figure II-1 shows that coho salmon

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

[6]David S. DeVault, Wayne A. Willford, Robert J. Hesselberg, David A. Nortrupt, Eric G.S. Rundberg, Alwan K. Alwan, and Cecilia Bautista, "Contaminant Trends in Lake Trout (Salvelinus namaycush) From the Great Lakes," Arch. Environ. Contam. Toxicol., Vol. 15, 349-356, 1986.

[7]David S. DeVault, J. Milton Clark, Garet Lahris, and Joseph Weishaar,
(Footnote Continued)

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Table II-13

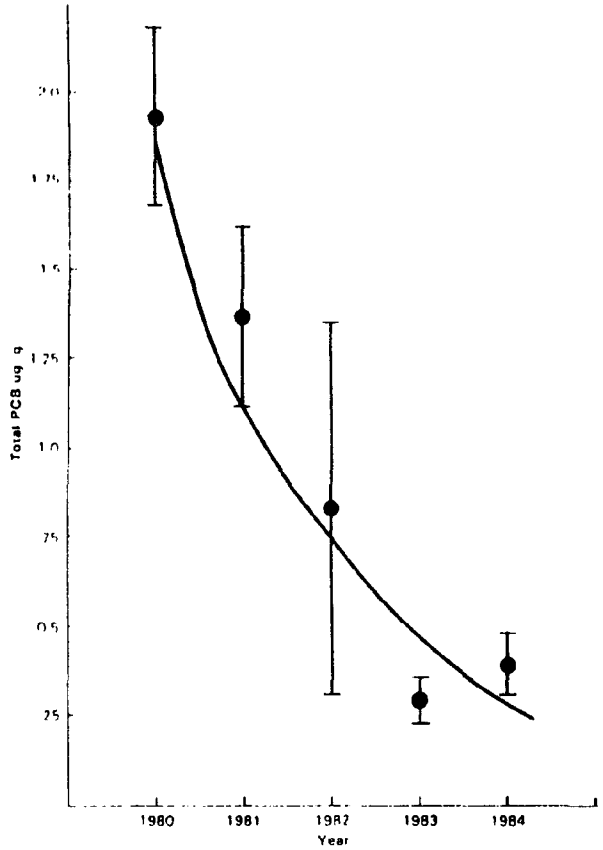
MEAN MEASURED CONCENTRATIONS OF PCB, DIELDRIN, AND CHLORDANE IN THE TISSUE OF LAKE MICHIGAN FISH: 1986-1987

Fish Species	PCB (ug/g)	Dieldrin (ug/g)	Total 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

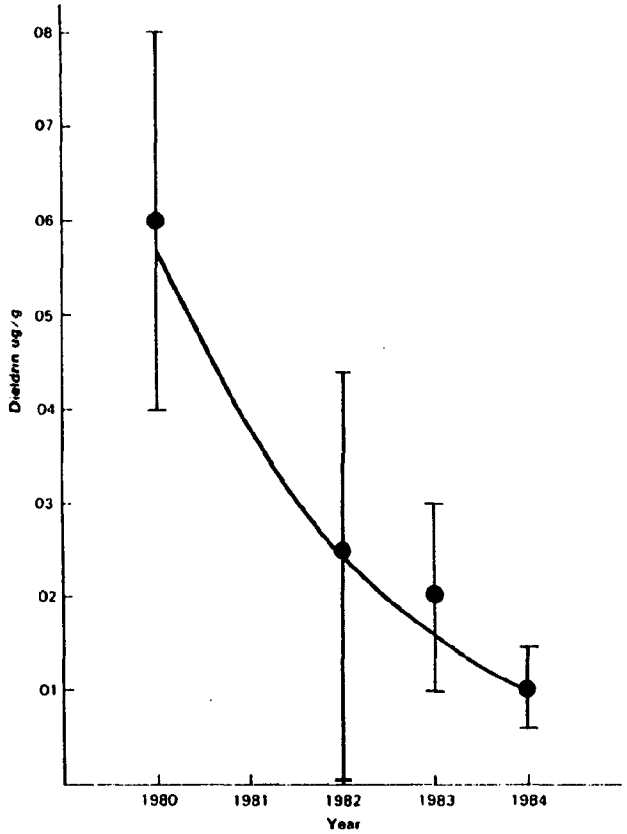
Source: Wisconsin Department of Natural Resources

Figure II - 1

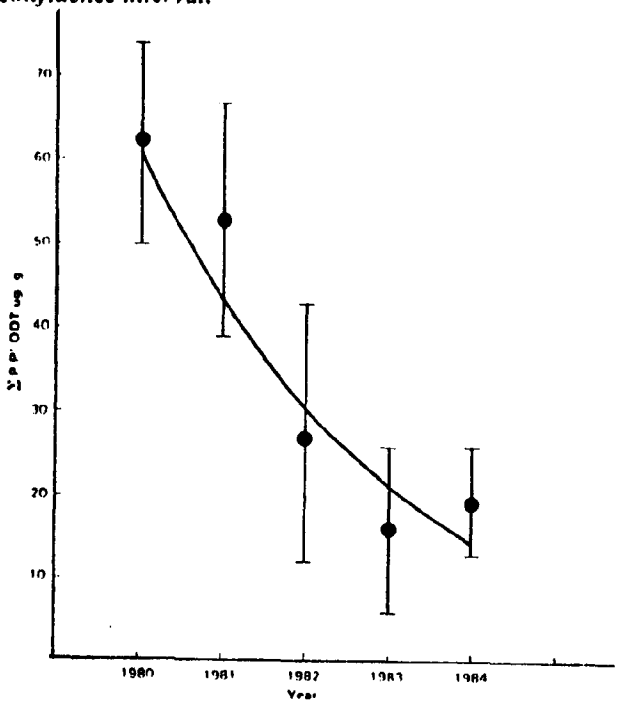
CONTAMINANT TRENDS IN THE TISSUE OF LAKE MICHIGAN COHO SALMON: 1980 - 1984



Total PCB concentrations in Lake Michigan coho salmon fillets. Mean and 95% confidence interval.



Dieldrin concentrations in Lake Michigan coho salmon fillets. Mean and 95% confidence interval.



Total p,p' DDT concentrations in Lake Michigan coho salmon fillets. Mean and 95% confidence interval.

Source : DeVault et al, 1988.

tissue concentrations of PCB, dieldrin, and DDT declined substantially over the period of 1980 through 1984. Similarly, Figure II-2 illustrates a decline in PCB, DDT, dieldrin, and oxychlordane concentrations in lake trout caught in Lake Michigan from the early 1970s through 1982. These declines have continued to occur.[8]

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 II-14. 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.

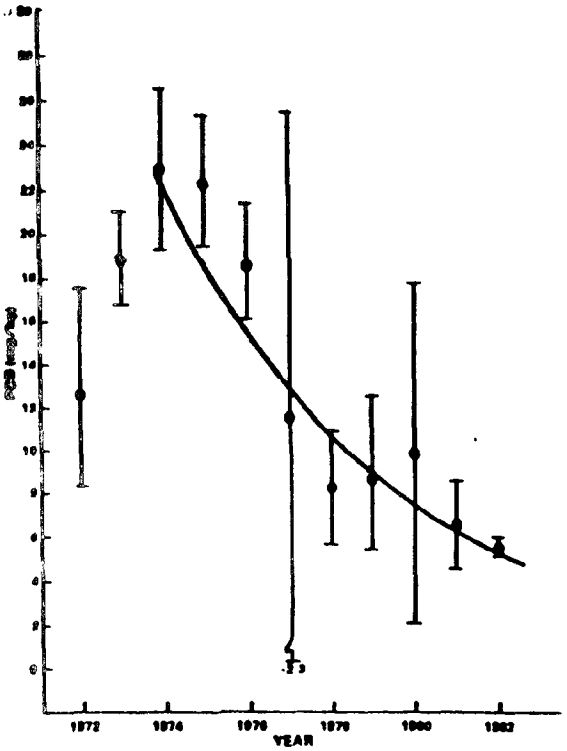
Aquatic Habitat: The aquatic habitat is an important element of biological communities, and consists of both biotic--or organic, and abiotic--or inorganic factors. Changes or stresses to the abiotic environment may actually be more damaging and enduring to the ecosystem than those to the biotic sector alone, since the biotia may respond and recover more quickly. Aquatic habitats indicate the overall quality, or health, or 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.

(Footnote Continued)
"Contaminants and Trends in Fall Run Coho Salmon," J. Great Lakes Res., Vol. 14, No. 1, 23-33, 1988.

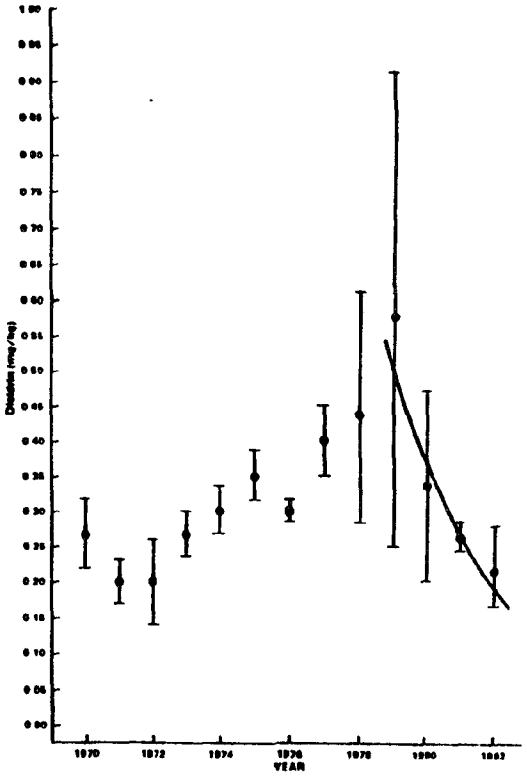
[8]David S. DeVault, Personal Communication, June 8, 1988.

Figure II - 2

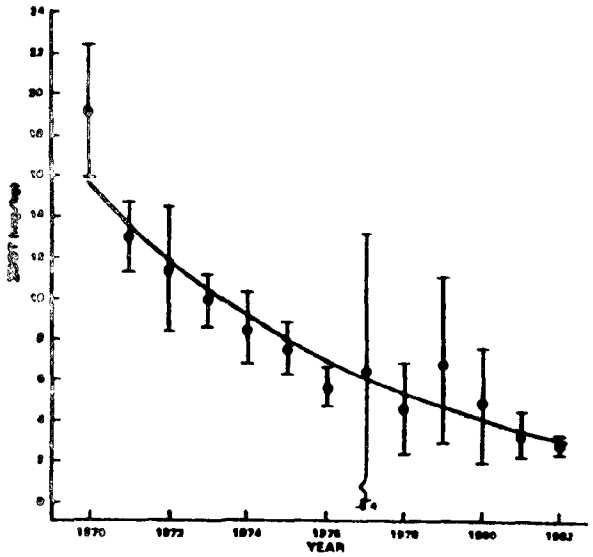
CONTAMINANT TRENDS IN THE TISSUE OF LAKE MICHIGAN LAKE TROUT: 1972 - 1982



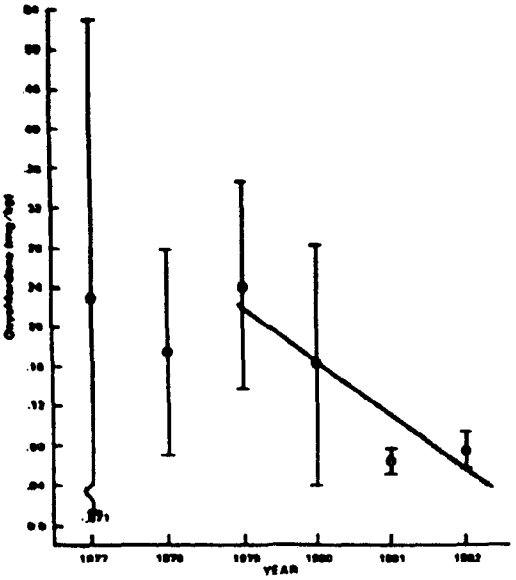
Polychlorobiphenyl (PCB) concentrations in Lake Michigan trout. Mean and 95% confidence interval



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 : DeVault et al, 1986.

Table II-14

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

Toxic Substance	Alewife			White Sucker			Yellow Perch			Brown Trout			Coho Salmon			Miscellaneous Species		
	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.

The aquatic habitat in the Milwaukee outer harbor was evaluated by the Wisconsin Department of Natural Resources in 1984.[9] 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.

The evaluation concluded that the substrate and habitat of the Milwaukee outer harbor are not conducive to the maintenance of self-sustaining salmonid 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 which 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.

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

These natural areas and rare plant sites are described in Tables II-15 and II-16.

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 that they contain native plant and animal communities believed to be representative of presettlement conditions. The five identified natural areas have a combined aerial extent of about 369 acres.

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 species, two State threatened species, and four species of special concern 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. 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, are 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.

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Table II-15

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

Area Name	U. S. Public Land Survey Location	Aerial Extent (Acres)	Features	Description
Downer Woods	T7N R22E Sec. 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 Secs. 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 Secs. 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 small 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).

Table II-15 (cont'd)

Area Name	U. S. Public Land Survey Location	Aerial Extent (Acres)	Features	Description
Schlitz Audubon Center	T8N R22E Secs. 9 and 10	164	Prairie, Bluffs, and Lake Terrace	This site contains a nature center, a prairie restoration tract, ornamental, a wooded ravine, bluffs, and a lake terrace.
Warnimont Clay Bluff Fen	T6N R22E Sec. 36	40	Fen, Springs, and Bluffs	This site features clay bluffs along Lake Michigan with and 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, Trisetum Melicoides, and False Asphodel, a threatened species in Wisconsin. Other plants occurring here are Ohio Goldenrod, Grass of Parnassus, Slender Bog Arrow-Grass, Small Fringed Gentian, Northern Bog Orchid and White Cedar, here near the natural southern edge of its range in Wisconsin.

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

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Table II-16

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

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

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

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 1830s. 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 1800s. 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. The urban development of the shoreline area between 1850 and 1985 is quantified in Table II-17. 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;

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Table II-17

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

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 the 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, as well as educational and cultural, values. Historic preservation helps retain these 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 diversely 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 archeological merit have been conducted by various units and agencies of government in Milwaukee County. The results of these inventories and surveys--on file at such agencies as the City of Milwaukee Historic Preservation Office and the Wisconsin State Historical Society--indicate that there are over 10,000 historic sites in Milwaukee County. Certain sites of known historic significance in Milwaukee County are listed in the National Register of Historic Places. Property listed on the National Register has some degree of protection from the potential adverse effect of federally funded or licensed activities. In 1988, there were 43 historic places 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 archeological 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 II-18. These important sites

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Table II-18

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

Site Number	Site Name	Civil Division	Public Land Survey Town Range & Section	Year Listed on The Register of Historic Places
1	Staile Meyer House	Fox Point	T8N, R22E, Sec. 16	1985
2	Horace B. Hatch House	Whitefish Bay	T8N, R22E, Sec. 28	1985
3	Barfield-Staples House	Whitefish Bay	T8N, R22E, Sec. 33	1985
4	John F. McEwens House	Whitefish Bay	T8N, R22E, Sec. 33	1985
5	Paul F. Grant House	Whitefish Bay	T8N, R22E, Sec. 33	1985
6	Frank J. Williams House	Whitefish Bay	T8N, R22E, Sec. 33	1985
7	Frederick Sperling House	Whitefish Bay	T8N, R22E, Sec. 33	1985
8	Halbert D. Jenkins House	Whitefish Bay	T8N, R22E, Sec. 33	1985
9	Herman Vihlen House	Whitefish Bay	T8N, R22E, Sec. 33	1983
10	Harrison Hurdie House	Whitefish Bay	T7N, R22E, Sec. 3	1985
11	George Goebel House	Whitefish Bay	T7N, R22E, Sec. 3	1985
12	G. B. Van Devan House	Whitefish Bay	T7N, R22E, Sec. 3	1985
13	Rufus Arndt House	Whitefish Bay	T7N, R22E, Sec. 3	1985
14	Alfred M. Hoelz House	Milwaukee	T7N, R22E, Sec. 3	1985
15	George E. Morgan House	Shorewood	T7N, R22E, Sec. 3	1985
16	Henry A. Meyer House	Shorewood	T7N, R22E, Sec. 10	1985
17	Milwaukee-Downer	Milwaukee	T7N, R22E, Sec. 10	1974
18	North Point Lighthouse	Milwaukee	T7N, R22E, Sec. 15	1984
19	Frederic C. Bogk	Milwaukee	T7N, R22E, Sec. 15	1972

Table II-18 (cont'd)

Site Number	Site Name	Civil Division	Public Land Survey Town Range & Section	Year Listed on The Register of Historic Places
20	Shorecrest Hotel	Milwaukee	T7N, R22E, Sec. 22	1984
21	North Point Watertower	Milwaukee	T7N, R22E,Sec. 22	1973
22	Lloyd R. Smitt House	Milwaukee	T7N, R22E, Sec. 28	1974
23	Charles Allis House	Milwaukee	T7N, R22E, Sec. 21	1975
24	Immanuel Presbyterian Church	Milwaukee	T7N, R22E, Sec. 28	1974
25	Aslor on the Lake	Milwaukee	T7N, R22E, Sec. 28	1984
26	German-English Academy	Milwaukee	T7N, R22E, Sec. 28	1977
27	Office Building	Milwaukee	T7N, R22E, Sec. 29	1983
28	Sixth Church of Christ Scientist	Milwaukee	T7N, R22E, Sec. 28	1980
29	Womens Club of Wisconsin	Milwaukee	T7N, R22E, Sec. 28	1982
30	Abstract Association Building	Milwaukee	T7N, R22E, Sec. 28	1982
31	St. John's Roman Catholic Cathedral	Milwaukee	T7N, R22E, Sec. 28	1974
32	Old St. Mary's Church	Milwaukee	T7N, R22E, Sec. 28	1973
33	The State Bank of Wiscons/Bank of Milwaukee	Milwaukee	T7N, R22E, Sec. 28	1984
34	Northwestern Mutual Life Insurance Company, Home Office	Milwaukee	T7N, R22E, Sec. 28	1973
35	Baumback Building	Milwaukee	T7N, R22E, Sec. 28	1983
36	Saloon	Milwaukee	T7N, R22E, Sec. 9	1977
37	Henni Hall	St. Francis	T6N, R22E, Sec. 15	1974

Source: Wisconsin State Historical Society

and districts designated for preservation form a significant link to the past. It is important to note that the potential exists for the identification of additional sites of historic significance which would be eligible for listing on the National Register and which, therefore, should be preserved.

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."[10]

The Third Ward historic district, much of which is constructed on filled marshland, was initially owned by Peter Juneau, Soloman'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 of 1892. Among the few survivors of the fire was the

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

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 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 downtown area. The District was developed before the Civil War as the City's first neighborhood for the 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 North Cass and East Wells streets, was once one of the City of Milwaukee's most exclusive residential districts. The neighborhood which exists there 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 ten-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 by Charles B. Whitnall, a Milwaukee County Park Commissioner, in 1923. 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 II-19 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 are about 1,666 acres of parks along the Lake Michigan shoreline, which represent 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

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Table II-19

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 Milw	Milw County	31
Bender	1967	Undeveloped	Milw County	Milw County	308
Big Bay	1937	1943	Milw County	Milw County	8
Buckley	1931	1943	Village of Whitefish Bay	Village of Whitefish Bay	1
Doctors	1928	1930	Village of Fox Point[a]	Milw County	49
Henry W. Maier Festival Park	Filled in early 1900s	1970	City of Milw	City of Milw	68
Grant	1910	1944	Milw County	Milw County	374
Juneau	1872	1920	City of Milwaukee[a]	Milw County	64
Klode		1943	Village of Whitefish Bay	Village of Whitefish Bay	11
Lake	1890	1900	City of Milwaukee[a]	Milw County	128
McKinley	1889	1936	City of Milwaukee[a]	Milw County	183
Sheridan	1928	1928	Milw County	Milw County	78
Shorewood Nature Preserve	1979	Undeveloped	Village of Shorewood	Village of Shorewood	8
South Shore	1909	1920	City of Milwaukee[a]	Milw County	48
Warnimont	1948	1958	Milw County	Milw 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.

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.

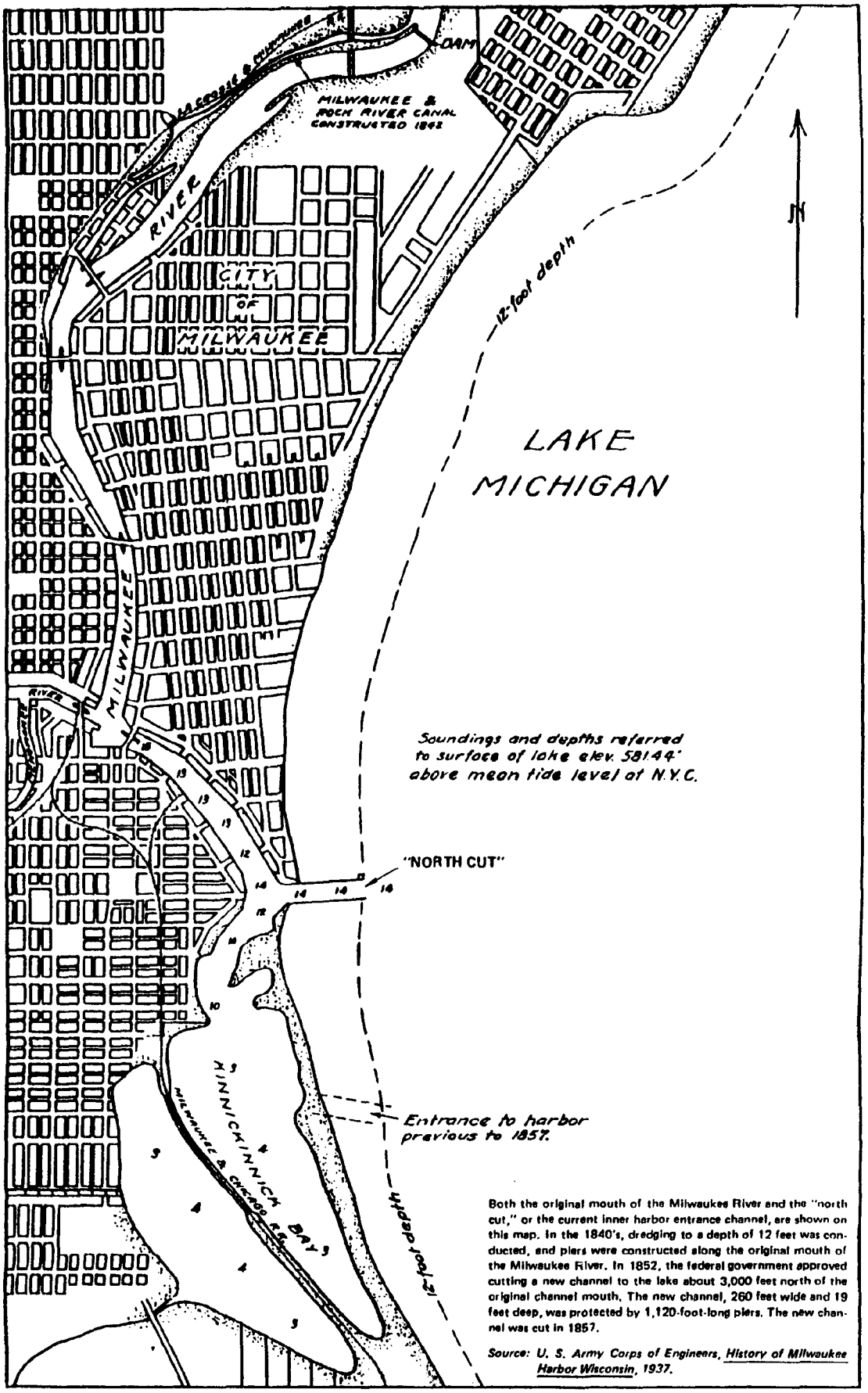
A similar concern for privacy was expressed at a public hearing on the Northern Milwaukee County Shoreline Erosion Management Plan held by the Village of Fox Point on April 26, 1988. Residents of the Fox Point terrace along N. Beach Drive expressed strong support for limited access and use of the shoreline in order to prevent trespassing and use of the shoreline by non-lakefront 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 II-2. 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 costs associated with 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

Map II-2

LOCATION OF THE MOUTH OF THE INNER HARBOR: 1867



Both the original mouth of the Milwaukee River and the "north cut," or the current inner harbor entrance channel, are shown on this map. In the 1840's, dredging to a depth of 12 feet was conducted, and piers were constructed along the original mouth of the Milwaukee River. In 1852, the federal government approved cutting a new channel to the lake about 3,000 feet north of the original channel mouth. The new channel, 260 feet wide and 19 feet deep, was protected by 1,120-foot-long piers. The new channel was cut in 1857.

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

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 II-3.

In 1912, a City Harbor Commission was formed by the City of Milwaukee. A high priority of that Commission was planning for the construction 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 in 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 II-4, 5, and 6 show the harbor land use changes which 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 car ferry 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 1976 with construction of the confined disposal facility for polluted dredge spoils by the U. S. Army 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 II-7.

Civil Divisions

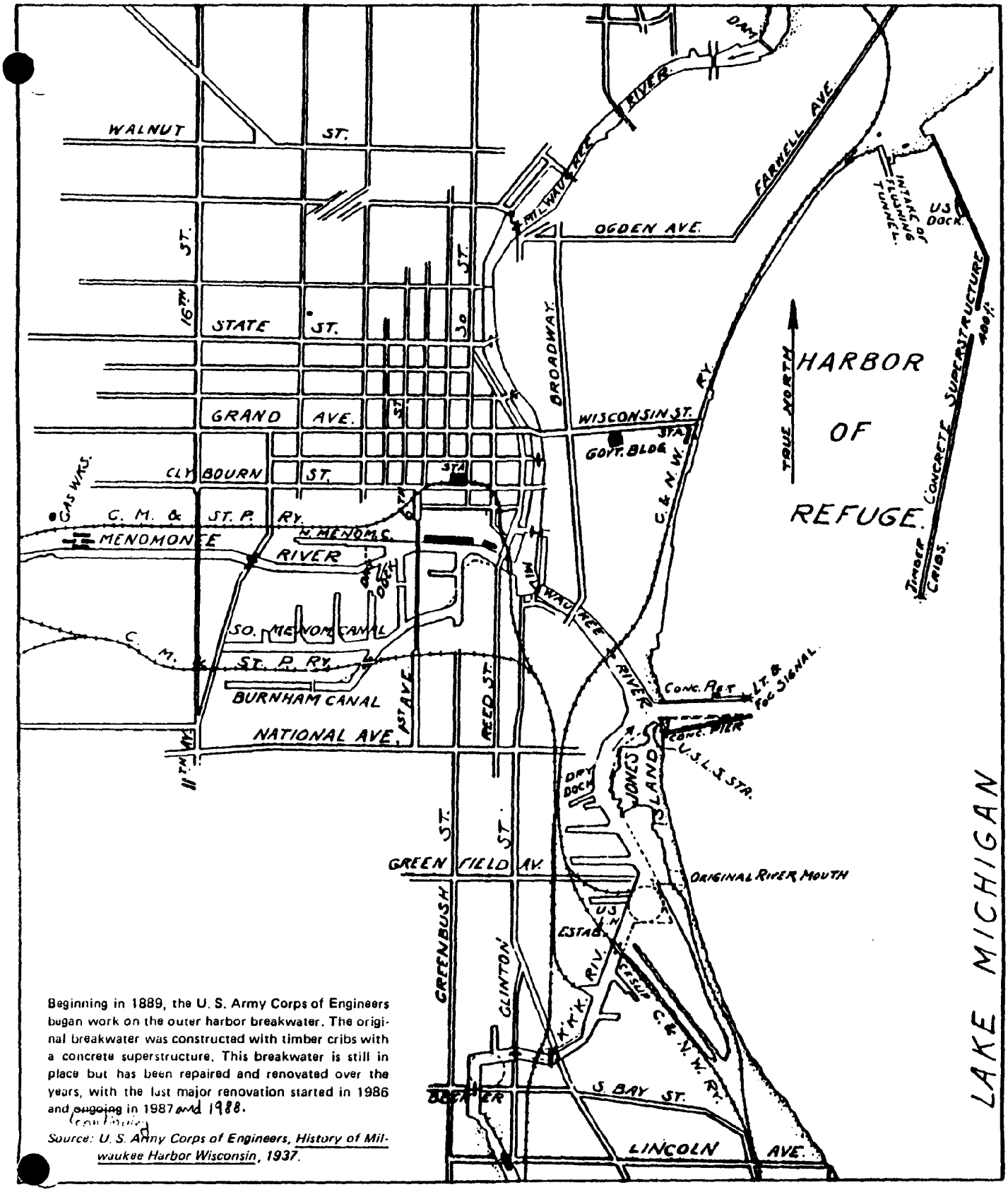
Local civil division boundaries within the study area are shown on Map II-8. The study area contains portions of the Cities of Oak Creek, South Milwaukee, Cudahy, St. Francis, and Milwaukee; and the Villages of Shorewood, Whitefish Bay, Fox Point, and Bayside. The area and proportion of each municipality 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 II-20. As indicated in the table, the City of Milwaukee contained 2,654 acres, or 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 existing within the study area of Milwaukee County in 1985. The areal extent of the

Map II-3

MILWAUKEE HARBOR FACILITIES: 1911

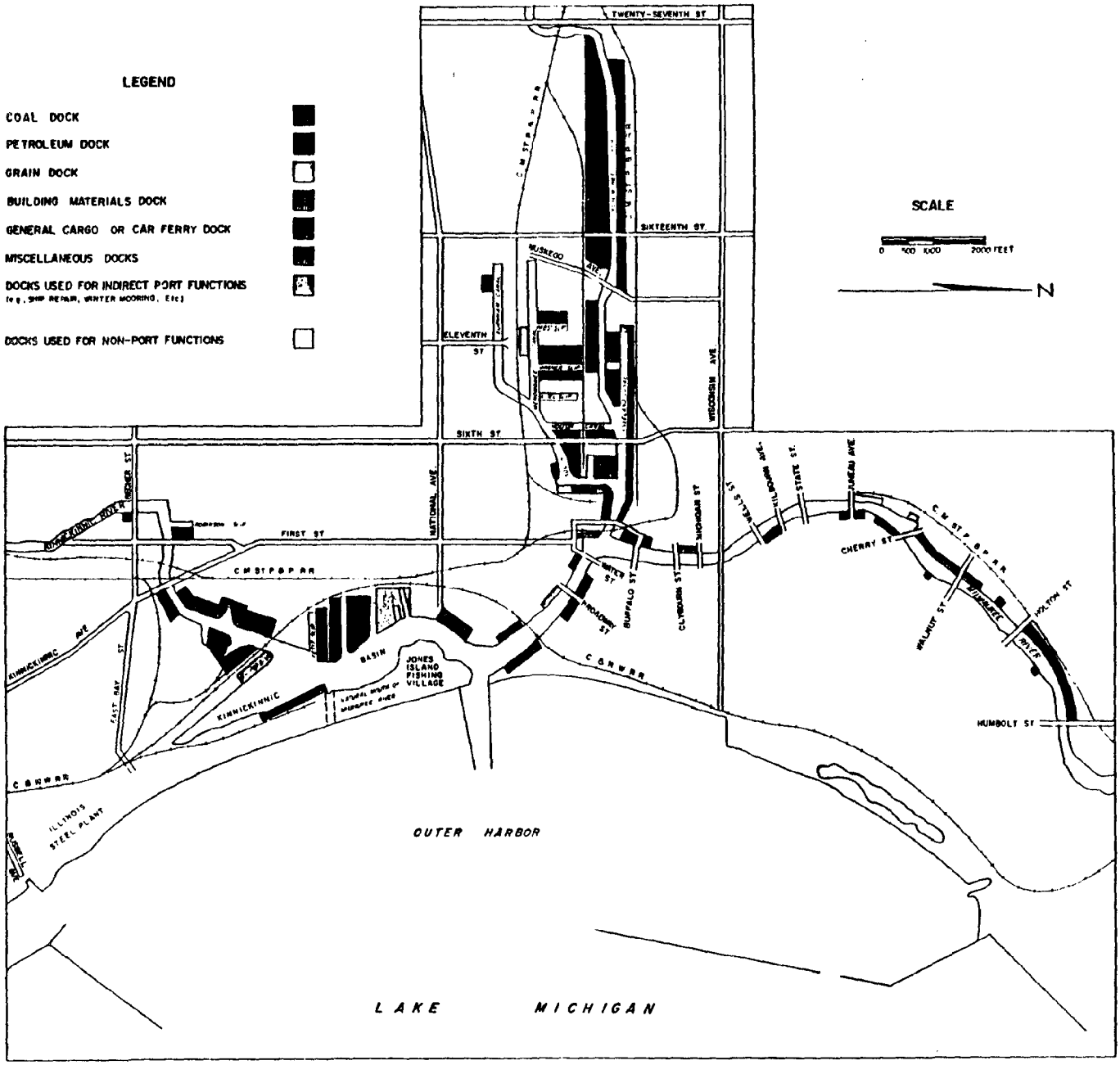


Beginning in 1889, the U. S. Army Corps of Engineers began work on the outer harbor breakwater. The original breakwater was constructed with timber cribs with a concrete superstructure. This breakwater is still in place but has been repaired and renovated over the years, with the last major renovation started in 1986 and ~~ongoing~~ continuing in 1987 and 1988.

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

Map II-4

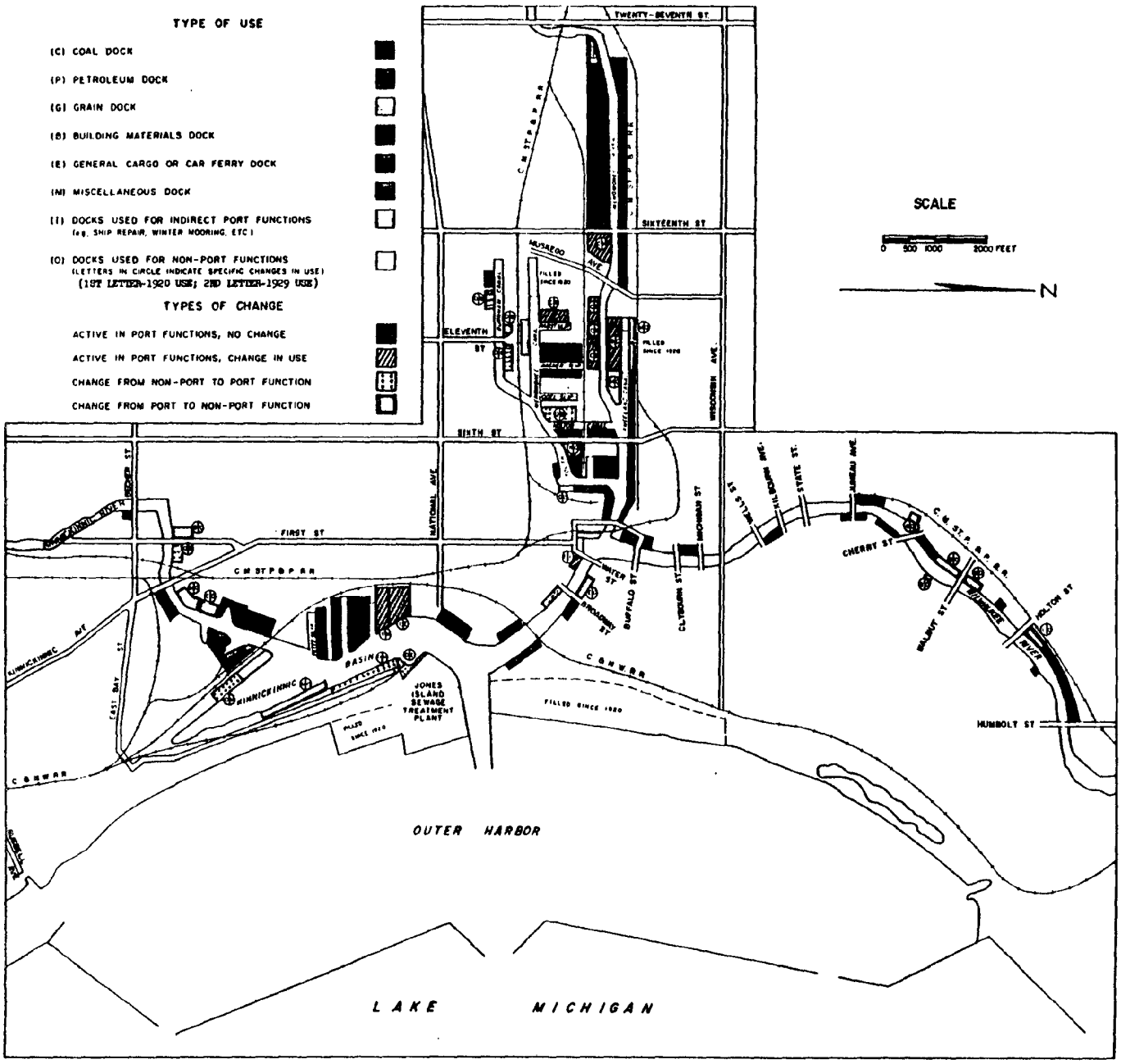
HARBOR LAND USE, PORT OF MILWAUKEE: 1920



Although the first warehouse was built on the inner harbor at E. Water Street in 1838, the true development of port facilities coincided with the arrival, in the early 1900's, of the first Great Lakes bulk carriers, and the growth of manufacturing. By 1920, wharves, warehouses, coalyards, lumberyards, tanneries, grain elevators, flour mills, maltsters, and breweries lined the inner harbor. The outer harbor breakwater was about one-half completed in 1920.

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

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

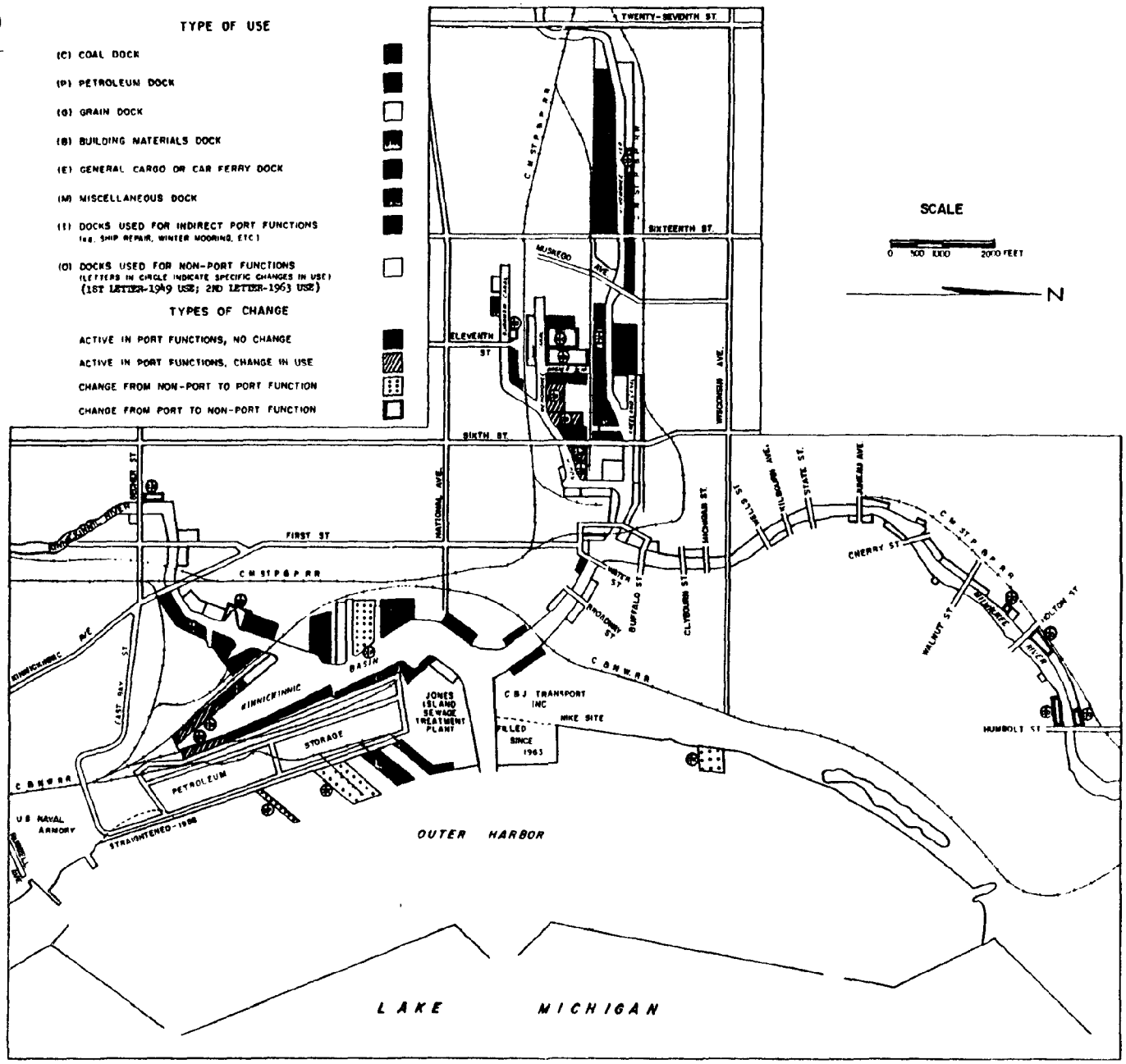


Development of the port facilities continued in the 1920's, with the completion of the outer harbor breakwater. The inner harbor area became a major manufacturing center, particularly for heavy pumping machinery, large gas engines, lubricating equipment, steam shovels, automobile parts, industrial machinery, hydroelectric units, and malt projects, including beer manufacturing. Milwaukee was one of the largest grain markets in the United States, and a major coal receiving port.

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

Map II-6

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

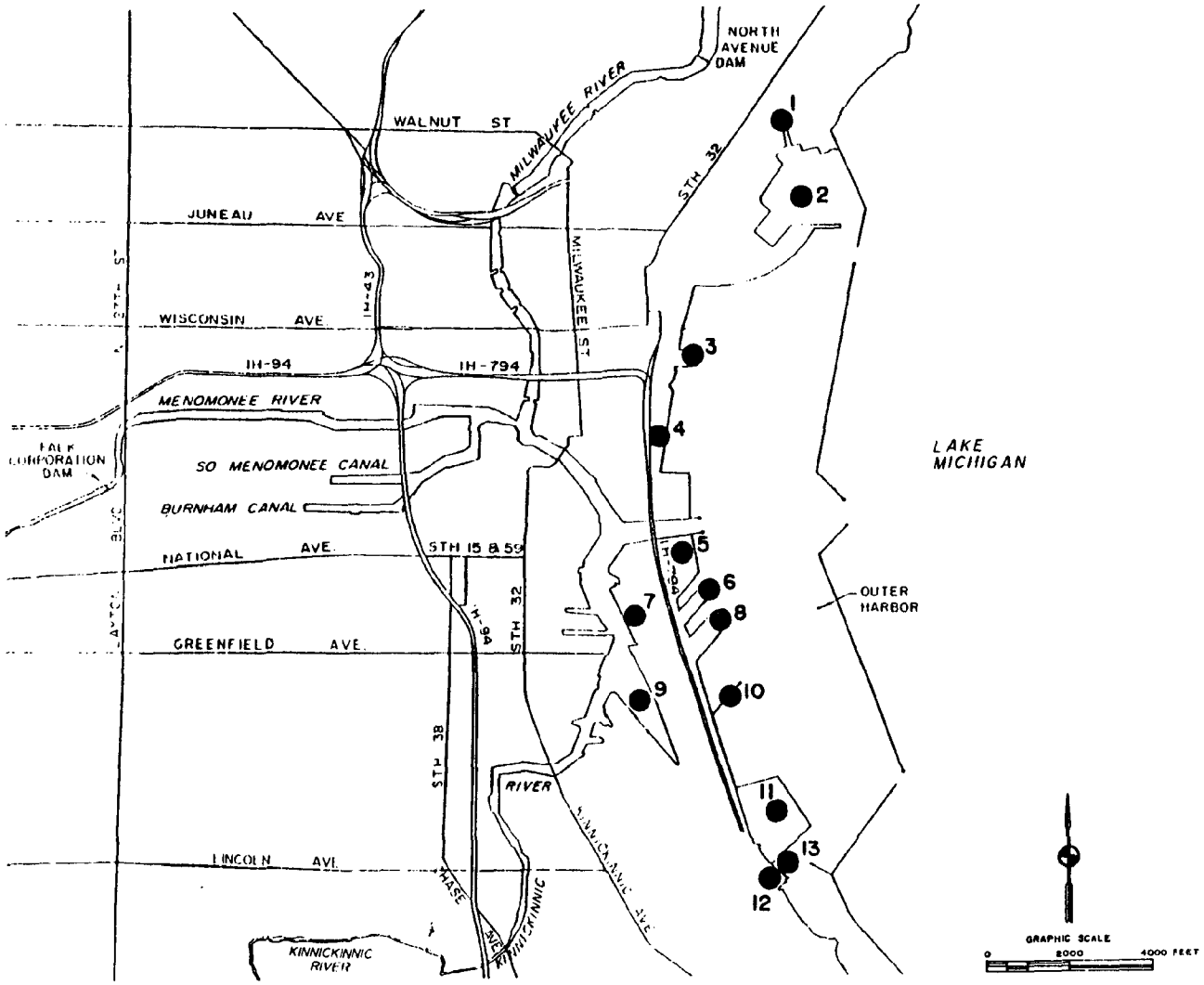


Changing lifestyles and land use patterns, along with the desire to eliminate bridge openings to accommodate automobile traffic, resulted in the end to commercial navigation on the Milwaukee River upstream of Buffalo Street in 1959. Emphasis shifted to the development of the land adjacent to the outer harbor. Fill was placed in the outer harbor to create new land and provide additional lakefront facilities.

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

Map II- 7

MILWAUKEE HARBOR FACILITIES: 1988



LEGEND

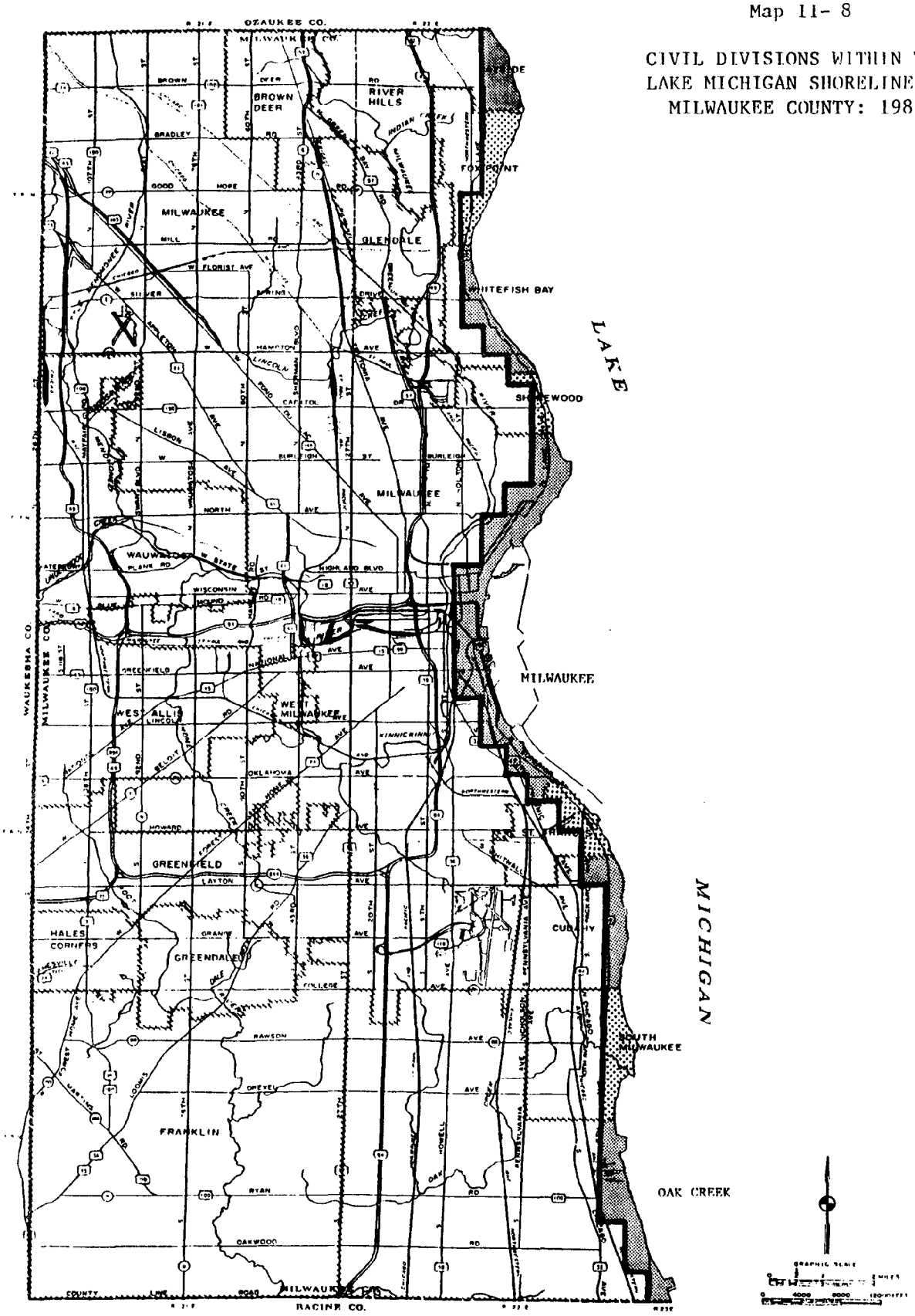
1 MILWAUKEE RIVER FLUSHING TUNNEL INLET
2 McKINLEY MARINA
3 HARBOR COMMISSION NORTH PIER
4 HENRY W. MAIER FESTIVAL GROUNDS
5 JONES ISLAND SEWAGE TREATMENT PLANT
6 PIER NO. 1
7 HEAVY LIFT DOCK
8 PIER NO. 2
9 MUNICIPAL MOORING BASIN
10 LIQUID CARGO PIER
11 DREDGE SPOIL CONTAINMENT FACILITY
12 U.S. COAST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

Major port development projects since the 1960's have included further development of Juneau Park and the McKinley Park anchorage area, the construction of the U. S. Army Corps of Engineers confined dredge spoils disposal area, and the ongoing eastward expansion of the Jones Island sewage treatment plant property. Port facilities designed to handle both manufactured goods and agricultural products include refrigerated terminals, building material wharves, grain elevators, petroleum terminals, bulk handling and storage facilities, and fixed and mobile derricks and cranes.

Source: SEWRPC.

Map 11- 8

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



Source: SEWRPC

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Table II-20

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

Civil Division	Area (Acres)	Percent of Study Area	Lake Michigan Shoreline Length (ft.)	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

various major categories of land use within the shoreland study area, which encompasses a total of 7,517 acres, is presented in Table II-21. As indicated in Table II-21, 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, transportation and utilities uses constituted the major proportions--1942 acres, or 44 percent of the developed urban area was residential. 1,730 or 39 percent of the urban area was used for transportation and utilities. Recreational uses constituted an additional 749 acres, or 10 percent of the total area. Undeveloped lands, including wetlands, woodlands, agricultural, and unused urban land, encompassed 2111 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 which have jurisdiction in the Lake Michigan coastal erosion study area in Milwaukee County. Table II-22 summarizes the zoning categories along the immediate shoreline.

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 this bluff and beach erosion, include wave action, groundwater seepage, precipitation runoff, lake level elevation, freeze-thaw action, lake ice movement, 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

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Table II-21

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	.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

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Table II-22

SUMMARY OF SELECTED EXISTING ZONING REGULATIONS
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, laundromats, 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, automobile laundries, 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,	

Table II-22, Continued

Civil Division	Zoning District	Permitted Uses	Special Uses
		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	Multi-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 II-22, 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, 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	Not specified.	Not specified
	Residential	Single-family dwellings, churches, schools, colleges, public libraries, museums and art galleries, municipal buildings, public recreational and community center buildings, professional offices, railroad rights-of-way and stations.	

Table II-22, Continued

Civil Division	Zoning District	Permitted Uses	Special Uses
City of Milwaukee	Residential	Single-family dwellings, family day-care homes, convents, foster homes, churches, schools, colleges, governmental structures, public parks and playgrounds, non-retail 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.	
	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.	OFFICES: Offices, banks and other financial institutions. RETAIL SALES: General retail sales, general purpose groceries, department stores, consumer services. MOTOR VEHICLE: Motor vehicle rental offices, motor vehicle service stations, car washes. SERVICES: Funeral homes, photographic studios, dry cleaning and laundry stations, laundromats.

Table II-22, Continued

Civil Division	Zoning District	Permitted Uses	Special Uses
		SERVICES: Medical and dental laboratories, research and testing laboratories, data processing centers, animal clinics, dry cleaning plants.	ENTERTAINMENT AND RECREATION: Recreation facilities, commercial hotels, restaurants, taverns, indoor theaters, convention centers and sports arenas.
		STORAGE AND WHOLESALE TRADE: Wholesale trade establishments, general storage, coal yards, storage of petroleum, storage of gas, junkyards.	STORAGE AND WHOLESALE TRADE
		TRANSPORTATION: Transportation passenger terminals, railroad switching and classification yards, terminals, ship terminals.	TRANSPORTATION: Airports and heliports.
		MANUFACTURING	MANUFACTURING AND MINING
		ANIMALS PRODUCTS: Leather finishing, meat, fish, poultry, fats and oils processing, tanning or tawing of hides, stock yards or slaughterhouses.	

Table II-22, Continued

Civil Division	Zoning District	Permitted Uses	Special Uses
Village of Bayside	Residential	Single-family dwellings. In some areas: schools and municipal buildings.	
Village of Shorewood	Lake Drive Residential District	Single-family dwellings; non-commercial greenhouses, nurseries and gardens; private garage.	
Village of Whitefish Bay	Lake Shore Residential District	Single-family dwellings; non-commercial greenhouses, nurseries and gardens; private garage.	
	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.	

Source: SEWRPC

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 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, or soil flow resulting from freeze-thaw activity occurring both in fall and spring, can reduce the stability of bluff slopes. 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 and, therefore, even gentle slopes may fail.

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 unconfined 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, roughness of the surface, and 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.

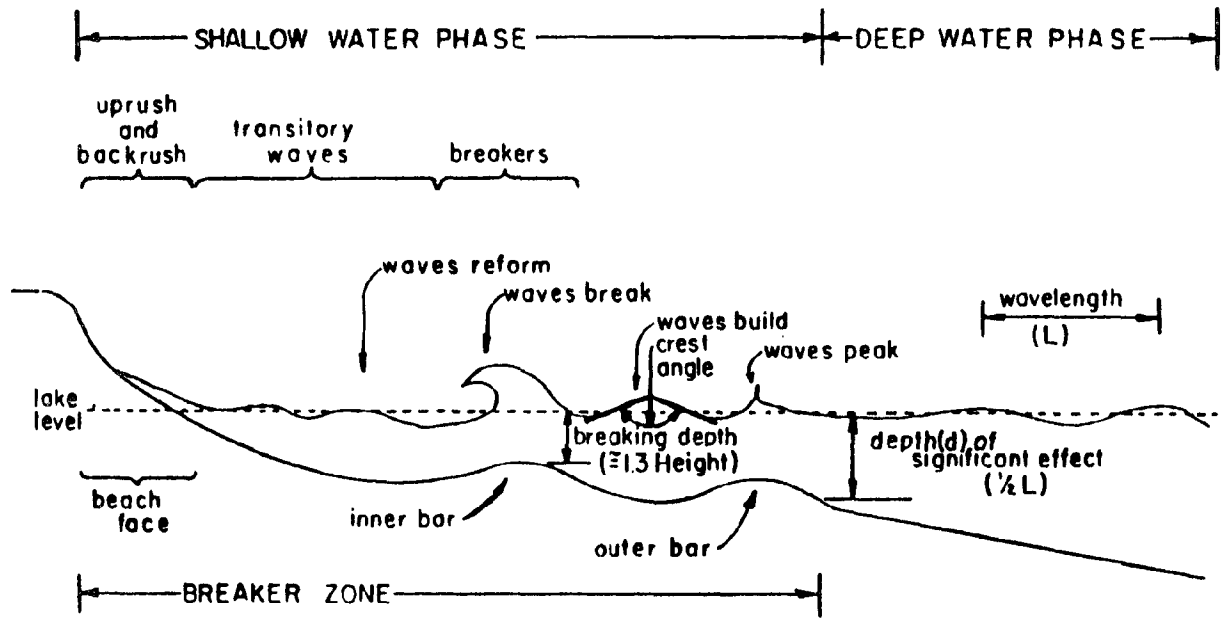
Wave Action: Several factors affect the type of slope failure which occurs and the severity of that failure. The physical characteristics of the beach and bluff have a major influence on the resistance of the slope to failure. Numerous other factors affect the external stresses which are placed upon 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 II-3 illustrates the pattern of breaking waves as they approach a beach. Wave action is also important because of its potential for damaging shore protection structures such as revetments, bulkheads, breakwaters, and groins.

Lake Michigan Water Level: 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.

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

Figure II-3

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.

recession continues at a fairly high rate until the bluffs have reached a stable slope angle. Peak bluff-top recession rates typically occur about four years after a high water level within this portion of the Lake Michigan shoreline.

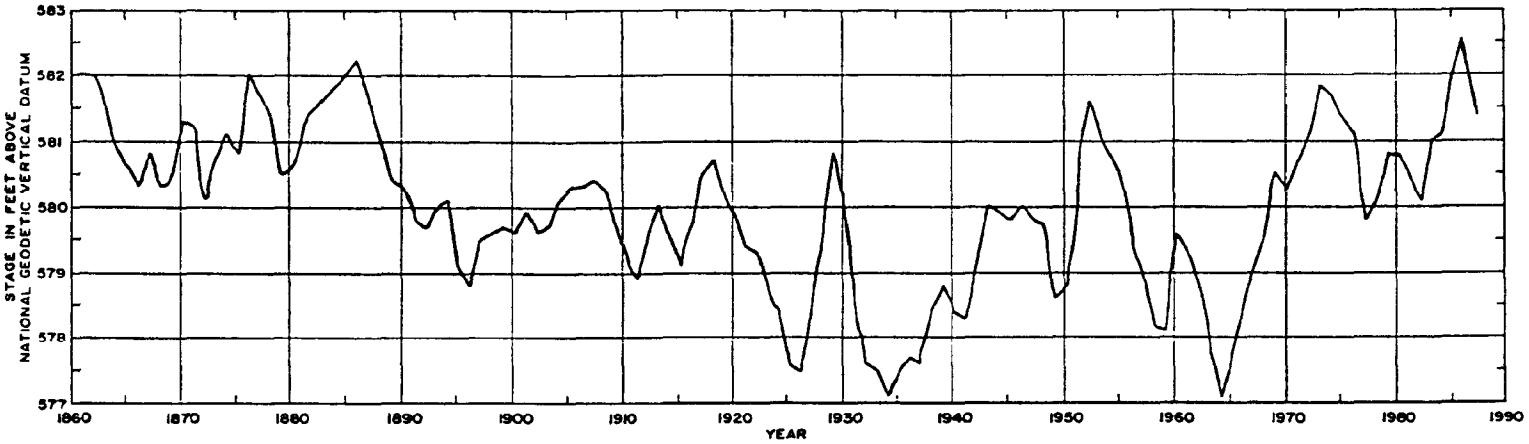
Figure II-4 shows the annual mean water level for Lake Michigan, recorded at Milwaukee for the period 1860 through 1987. The historic low annual mean lake level at Milwaukee--577.06 feet above National Geodetic Vertical Datum (NGVD), also referred to as Mean Sea Level Datum--occurred in 1964. The historic 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.

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 II-5. 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.

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 II-9. These are the Long Lac, Ogoki, and Chicago diversions; the Welland Canal; and the New York State Barge Canal. Both the Ogoki and Long Lac diversion 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

Figure II-4

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

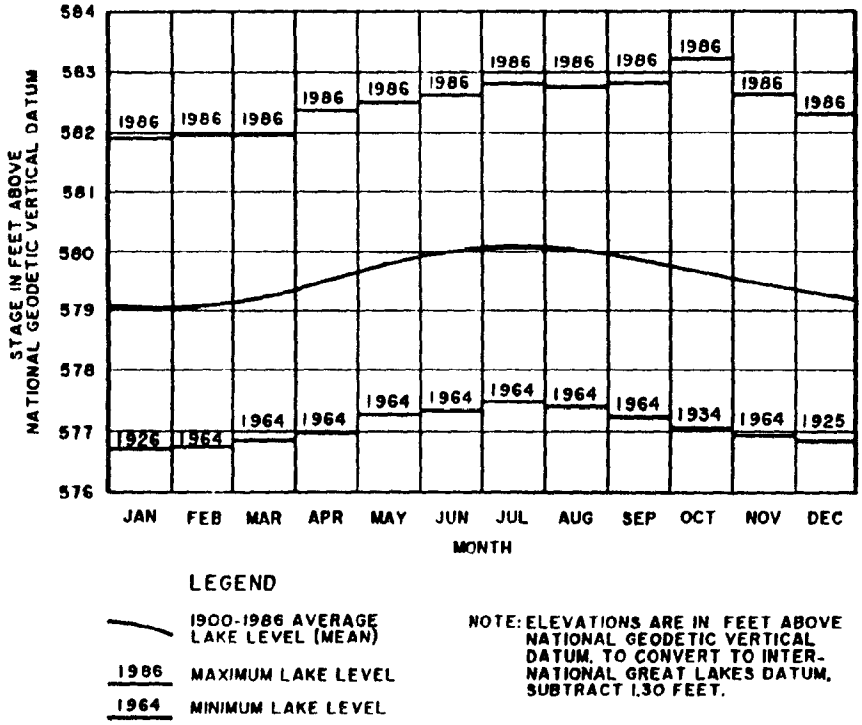


NOTE: ELEVATIONS ARE IN FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM. TO CONVERT TO INTERNATIONAL GREAT LAKES DATUM, SUBTRACT 1.30 FEET.

Source: National Ocean Service and SEWRPC.

Figure II-5

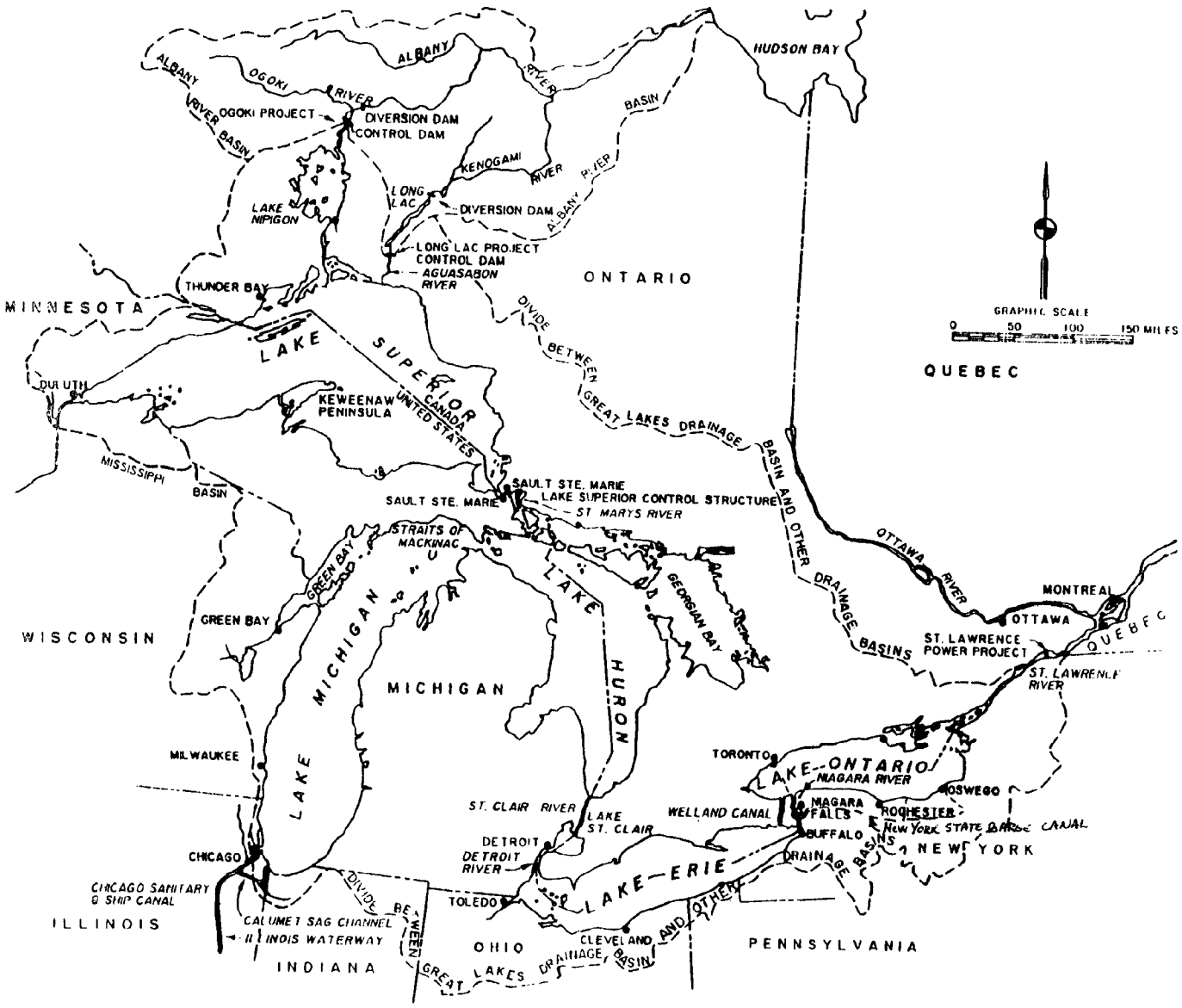
VARIATION IN MONTHLY MEAN WATER LEVELS FOR LAKE MICHIGAN AT MILWAUKEE: 1900-1987



Source: National Ocean Service and SEWRPC.

Map II- 9

GREAT LAKES DRAINAGE BASIN



Source: U. S. Army Corps of Engineers.

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 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. Presently, 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.[11] 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

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

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 II-5, 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.[12] 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. During the twentieth century, one similar period of

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

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.[13]

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 periods 480 to 610 AD, 1000 to 1150 AD, and 1580 to 1720 AD.[14] 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. However, other researchers have questioned the reliability of these estimates of prehistoric lake levels. Bishop[15] disagreed with Larsen's interpretation of the stratigraphic data and stated that archeological and geobotanical studies and findings conflict with the trends postulated by

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[13]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.

[14]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.

[15]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.

larsen. Historical archeological and geobotanical information from several sources cited by Bishop indicates that the water levels of Lake Michigan during the 17th and 18th centuries and dating as far back as the 1640's were not significantly different than those recorded in the 19th and 20th centuries. Bishop concluded that the variation in the mean annual levels of Lake Michigan over the past 350 years has not differed substantially from those measured since 1860. A recent study of past summer water supplies reconstructed from tree-ring data also concluded that variations in water supplies to the Great Lakes during the summer growing season over the past 200 years were similar to those which occurred during this century.[16]

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 which have been installed to protect the beach and bluff.

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

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

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 intercepts 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, however, 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 nearshore 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 II-6 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 rate of littoral materials moving along the Milwaukee County shoreline is estimated to be on the order of 8,000 cubic yards annually.[17]

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.

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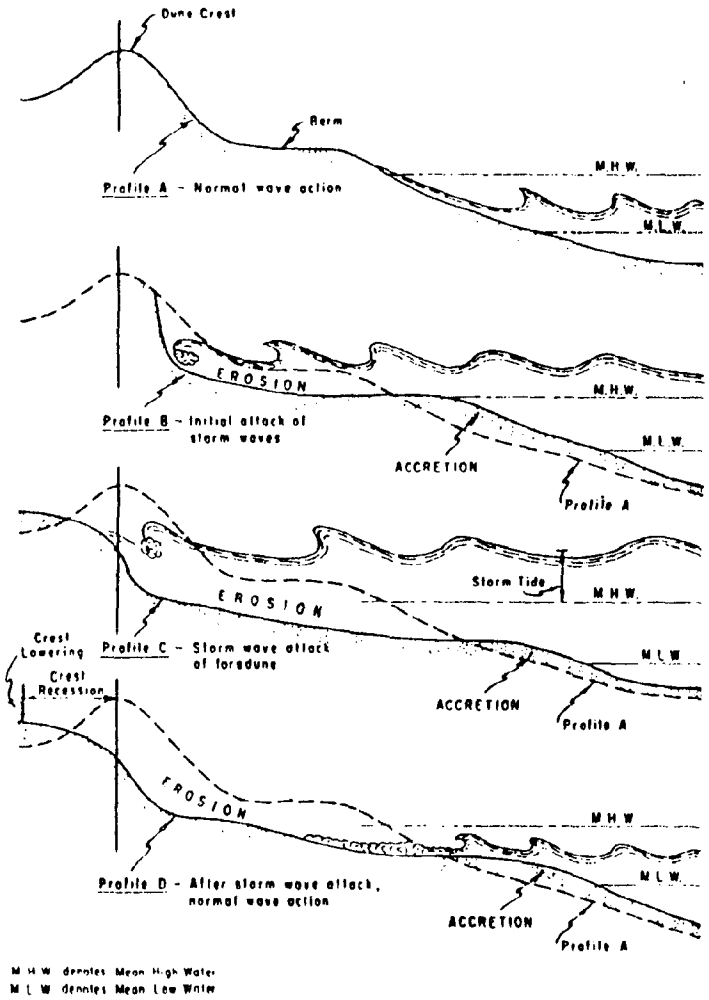
[17]U.S. Army Corps of Engineers, Lake Michigan Shoreline, Milwaukee County, Wisconsin, March 1975.

Figure II-6

BEACH EROSION IN RESPONSE TO WAVE ACTION



Source: U. S. Army Corps of Engineers.

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 delineates an artificial shoreline and allows 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.

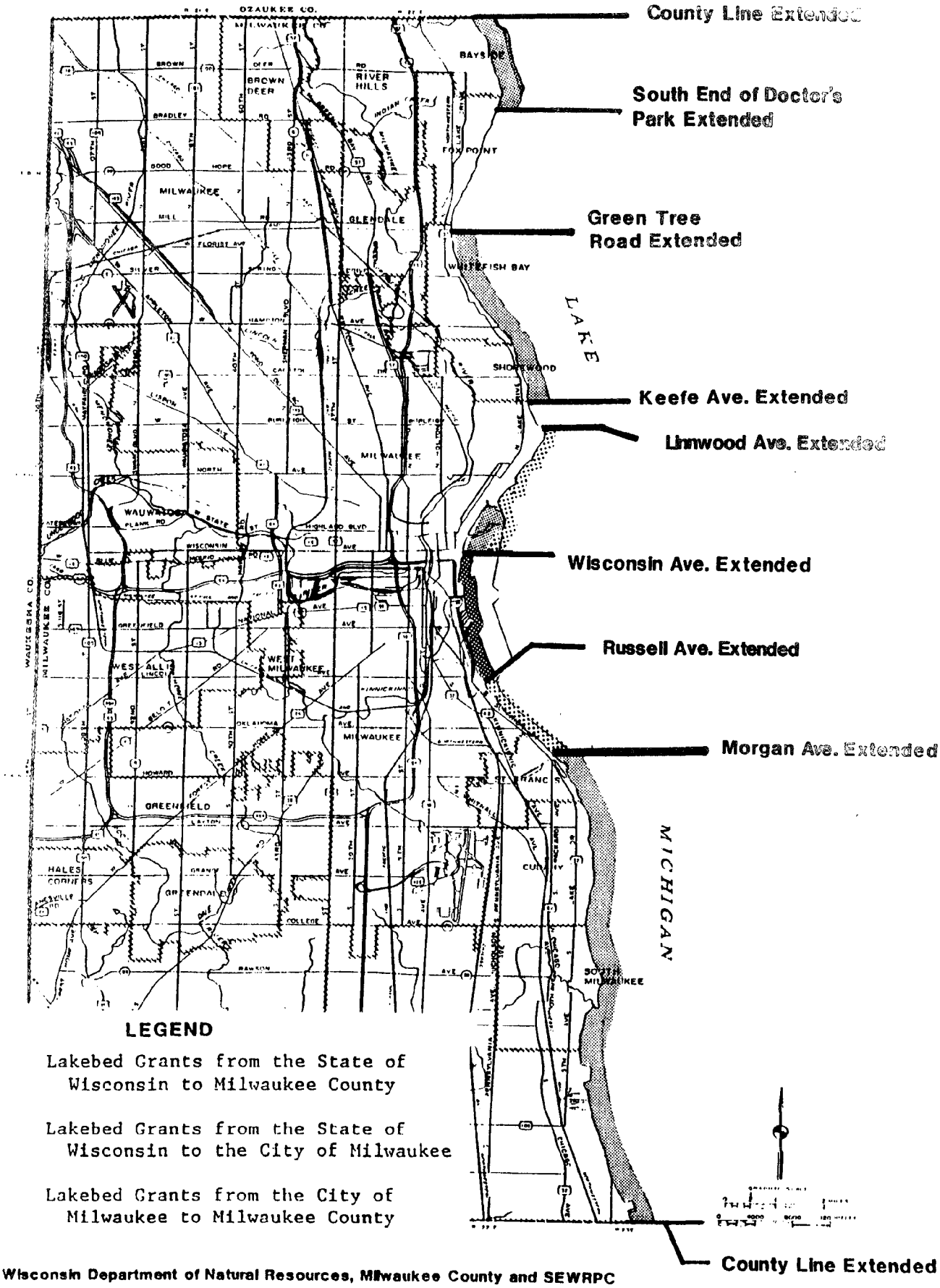
Although the Department of Natural Resources regulates shore protection-related activities throughout most of the Lake Michigan shoreline of the State, 93 percent of the immediate shoreline in Milwaukee County is regulated under lakebed grants made to the City of Milwaukee, or to Milwaukee County, between 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 grant are to be held and used by the City or County for navigation or harbor facilities, public park, 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 II-10. 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 requires 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 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.

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 of 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

Map LI-10

SUBMERGED LAKEBED GRANTS WITHIN MILWAUKEE COUNTY



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

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 by 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 indicate the extent of protection provided, and the types of structures used. The lineal extent of shore protection structures which were in existence in 1920, 1945, 1975, and 1987 are presented in Table II-23 and illustrated in Figure II-7.

In 1920, only 15 percent of the total County shoreline was protected by the structures.[18] 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.

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[18]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.

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Table II-23

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

	Length of County Shoreline Protected by Structures (feet)					
Year	Revetments	Bulkheads	Groins	Breakwaters	Total Length[a]	Percent of Study Area Shoreline
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

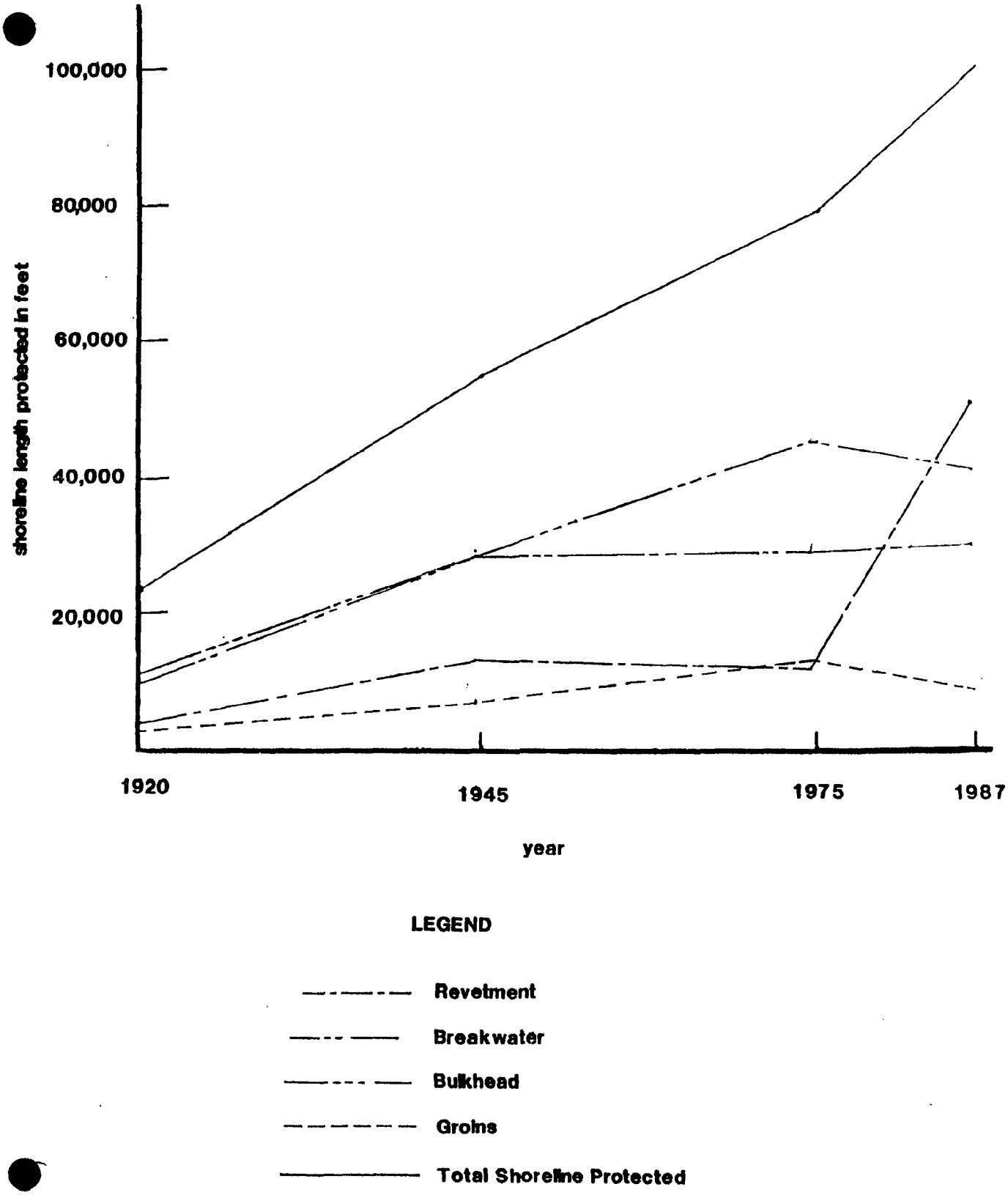
[a]Represents the total shoreline protected. Some shoreline areas were protected by more than one type of structure.

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

Figure II- 7

SHORE PROTECTION STRUCTURES ALONG THE

LAKE MICHIGAN SHORELINE OF MILWAUKEE COUNTY: 1920-1987



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

By 1945, 35 percent of the County shoreline was protected.[19] 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 protected 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.

About 50 percent of the Milwaukee County shoreline was protected in 1975.[20] 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 of the structures were located within the City of South Milwaukee; three or 2 percent of the structures were located in the City of Cudahy; three, or

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[19]Ibid.

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

2 percent of the structures were located in the City of St. Francis; 35, or 27 percent of the structures, were located in the City of Milwaukee; 12, or 10 percent of the structures, were located in the Village of Shorewood; 19, or 15 percent of the structures, were located in the Village of Whitefish Bay; 41, or 32 percent of the structures, were located in the Village of Fox Point; and five, or 4 percent of the structures, were located in the Village of Bayside. 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 feet or 19 percent of the shoreline protected residential land; and 3 structures covering 3 percent of the shoreline protected land devoted to commercial and governmental use.

The quality and effectiveness of shore protection structures varies considerably. An inventory of the condition of shore protection structures within the northern Milwaukee County study 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 A and summarized in Table II-24. The table indicates that about 75 percent of the structures, including 58 percent of revetments and 80 percent of 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 II-24 also summarizes the type of failures affecting these structures.

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Table II-24

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

Maintenance Required	Type of Structure: Revetment Number	Revetment Percent	Bulkhead Number	Bulkhead Percent	Groin Number	Groin Percent	Breakwater Number	Breakwater Percent	Total Number	Total 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

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 were either not constructed high enough for the 1986 high lake levels, or that the structures 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.

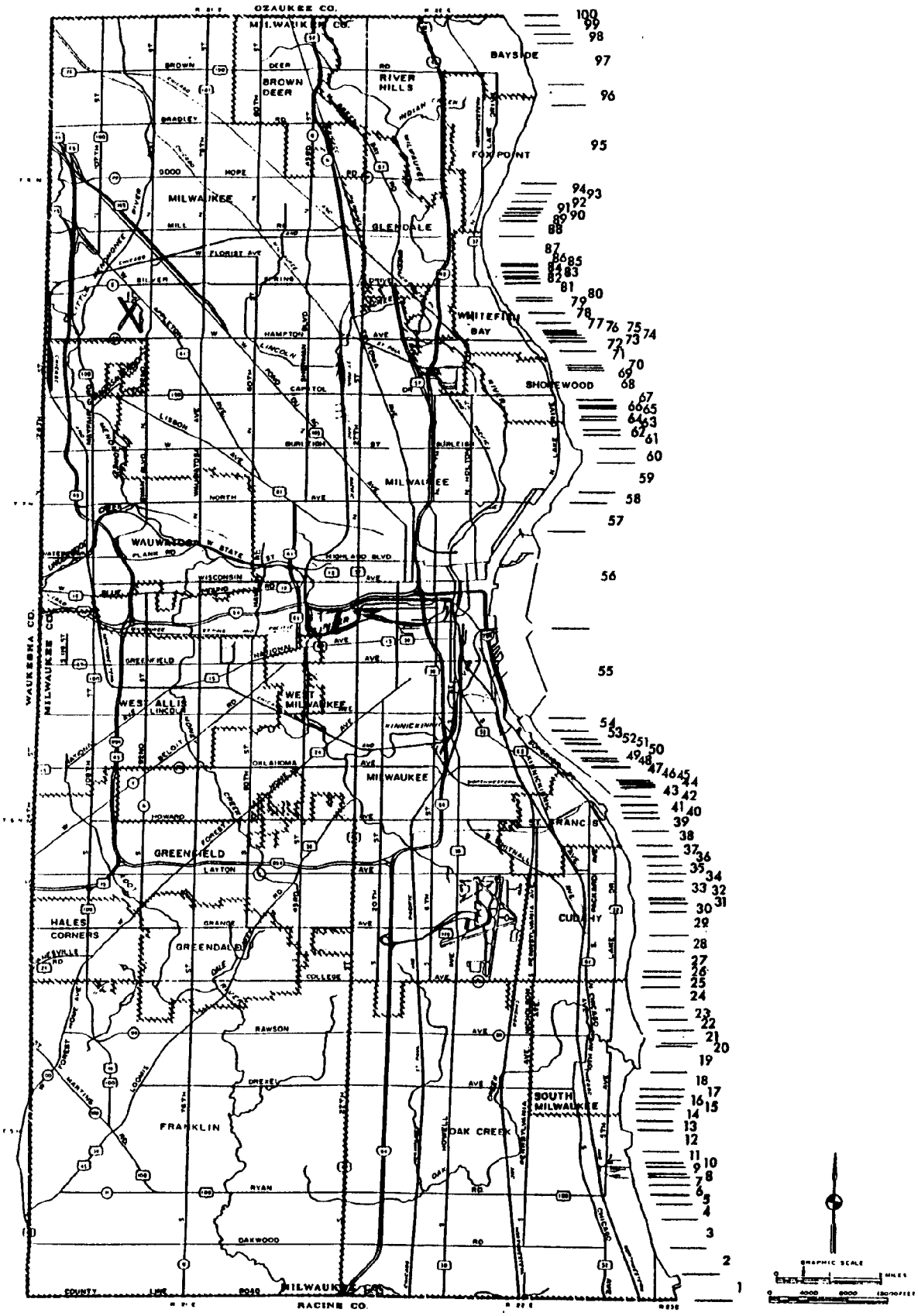
EXISTING SHORELINE EROSION PROBLEMS

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 location of these 100 bluff analysis sections is shown on Map II-11. The boundaries of the sections are located on property boundary lines. Field surveys 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 1987 for the remainder of the County. During these surveys section boundaries were delineated, the physical

Map II - 11

BLUFF ANALYSIS SECTIONS IN MILWAUKEE COUNTY



Source;SEWRPC.

characteristics of the sections were inventoried, and the causes and types of shoreline erosion and slope failure occurring were identified within each section. Table II-25 summarizes the locations, and the physical and erosion-related characteristics of each of the 100 bluff analyses 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 that ten 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 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

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Table II-25

PHYSICAL AND EROSION RELATED CHARACTERISTICS OF BLUFF ANALYSIS SECTIONS: 1987

Civil Division	Bluff Analysis Section	Location	Shoreline Length (feet)	Beach Characteristics: Width (feet)	Beach Characteristics: Slope (°)	Beach Characteristics: Composition	Bluff Characteristics: Height (feet)	Bluff Characteristics: Overall Slope (°)	Bluff Characteristics: Vegetation (%)	Bluff Characteristics: 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 ft 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's bulkhead	No	Shallow slides sapping, soli-fluction, 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. 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 Co. 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 (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)	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	--

Table II-25 (cont'd)

Civil Division	Bluff Analysis Section	Location	Shoreline Length (feet)	Beach Characteristics: Width (feet)	Beach Characteristics: Slope (°)	Beach Characteristics: Composition	Bluff Characteristics: Height (feet)	Bluff Characteristics: Overall Slope (°)	Bluff Characteristics: Vegetation (%)	Bluff Characteristics: Composition	Groundwater Conditions	Shore Protection Structure	Significant Bluff Toe Erosion	Type of Bluff Slope Failure
	9	4301 E. Depot Road (Hynite Corp. and Vulcan Material Co. 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 Ave. (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 Treatment Plant	3,160	0	--	--	76-90	18	100	Undermined	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	Undermined	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 Ave.	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 Ave.-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 II-25 (cont'd)

Civil Division	Bluff Analysis Section	Location	Shoreline Length (feet)	Beach Characteristics: Width (feet)	Beach Characteristics: Slope (°)	Beach Characteristics: Composition	Bluff Characteristics: Height (feet)	Bluff Characteristics: Overall Slope (°)	Bluff Characteristics: Vegetation (%)	Bluff Characteristics: Composition	Groundwater Conditions	Shore Protection Structure	Significant Bluff Toe Erosion	Type of Bluff Slope Failure
City of South Milwaukee (cont'd)	18	South Milwaukee Water Utility-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 anc
	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 and sand, and silt and clay	Minor groundwater seeps occur at the top of the Oak Creek till layer within the northern 3rd 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

Table II-25 (cont'd)

Civil Division	Bluff Analysis Section	Location	Shoreline Length (feet)	Beach Characteristics: Width (feet)	Beach Characteristics: Slope (°)	Beach Characteristics: Composition	Bluff Characteristics: Height (feet)	Bluff Characteristics: Overall Slope (°)	Bluff Characteristics: Vegetation (%)	Bluff Characteristics: Composition	Groundwater Conditions	Shore Protection Structure	Significant Bluff Toe Erosion	Type of Bluff Slope Failure
City of Cudahy	26	Lake Shore Tower Apts. Warnimont Park	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 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 and flows
	29	Warnimont Park	770	10-50	4-6	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 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 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	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 II-25 (cont'd)

Civil Division	Bluff Analysis Section	Location	Shoreline Length (feet)	Beach Characteristics: Width (feet)	Beach Characteristics: Slope (°)	Beach Characteristics: Composition	Bluff Characteristics: Height (feet)	Bluff Characteristics: Overall Slope (°)	Bluff Characteristics: Vegetation (%)	Bluff Characteristics: Composition	Groundwater Conditions	Shore Protection Structure	Significant Bluff Toe Erosion	Type of Bluff Slope Failure
City of Cudahy (cont'd)	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	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' 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' south of Howard Avenue-Power Plant breakwater	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	Power Plant breakwater 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

Table II-25 (cont'd)

Civil Division	Bluff Analysis Section	Location	Shoreline Length (feet)	Beach Characteristics: Width (feet)	Beach Characteristics: Slope (°)	Beach Characteristics: Composition	Bluff Characteristics: Height (feet)	Bluff Characteristics: Overall Slope (°)	Bluff Characteristics: Vegetation (%)	Bluff Characteristics: Composition	Groundwater Conditions	Shore Protection Structure	Significant Bluff Toe Erosion	Type of Bluff Slope Failure
	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-South Shore 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	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 Street Water Intake	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 Pavillon	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	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 STP	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 II-25 (cont'd)

Civil Division	Bluff Analysis Section	Location	Shoreline Length (feet)	Beach Characteristics: Width (feet)	Beach Characteristics: Slope (°)	Beach Characteristics: Composition	Bluff Characteristics: Height (feet)	Bluff Characteristics: Overall Slope (°)	Bluff Characteristics: Vegetation (%)	Bluff Characteristics: Composition	Groundwater Conditions	Shore Protection Structure	Significant Bluff Toe Erosion	Type of Bluff Slope Failure
City of Milwaukee	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 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	3 concrete bulkheads and 1 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	3 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	3 concrete bulkheads	Yes	--
	66	3926 N. Lake Drive 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

Table II-25 (cont'd)

Civil Division	Bluff Analysis Section	Location	Shoreline Length (feet)	Beach Characteristics: Width (feet)	Beach Characteristics: Slope (°)	Beach Characteristics: Composition	Bluff Characteristics: Height (feet)	Bluff Characteristics: Overall Slope (°)	Bluff Characteristics: Vegetation (%)	Bluff Characteristics: Composition	Groundwater Conditions	Shore Protection Structure	Significant Bluff Toe Erosion	Type of Bluff Slope Failure
Village of Shorewood (cont'd)	68	Atwater Park-4300 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	4308-4320 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, New Berlin till.	No major groundwater seeps were noted	2 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, New Berlin till.	No major groundwater seeps were noted	Fill with riprap revetment	Yes	--
	74	4780 N. Lake Drive Drive	160	<10	--	--	95	36	0	Ozaukee till at top of bluff underlain by silt and sand, Oak Creek till, New Berlin till.	No major groundwater seeps were noted	None	Yes	Surface sloughing, slumping and shallow slides
	75	4794-4800 N. Lake Drive	310	<10	--	--	80-90	36	0	Ozaukee till at top of bluff underlain by silt and sand, Oak Creek till, New Berlin till.	No major groundwater seeps were noted	Fill with riprap revetment	Yes	--

Table II-25 (cont'd)

Civil Division	Bluff Analysis Section	Location	Shoreline Length (feet)	Beach Characteristics: Width (feet)	Beach Characteristics: Slope (°)	Beach Characteristics: Composition	Bluff Characteristics: Height (feet)	Bluff Characteristics: Overall Slope (°)	Bluff Characteristics: Vegetation (%)	Bluff Characteristics: Composition	Groundwater Conditions	Shore Protection Structure	Significant Bluff Toe Erosion	Type of Bluff Slope Failure
Village of Whitefish Bay (cont'd)	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, New Berlin till.	No major groundwater seeps were noted	None	Yes	Shallow slumps and slides and shallow slides
	77	4850 N. Lake Drive-Southern portion of Buckley Park	1,410	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 majro groundwater seeps were noted	Fill with riprap revetment	Yes	--
	78	Northern portion Buckley Park-Southern portion Big Bay Park	1,060	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 and New Berlin till.	No major groundwater seeps were noted	Concrete stepped bulkhead	Yes	Slumping
	79	Northern portion Big Bay Park 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 Ave.	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

Table II-25 (cont'd)

Civil Division	Bluff Analysis Section	Location	Shoreline Length (feet)	Beach Characteristics: Width (feet)	Beach Characteristics: Slope (°)	Beach Characteristics: Composition	Bluff Characteristics: Height (feet)	Bluff Characteristics: Overall Slope (°)	Bluff Characteristics: Vegetation (%)	Bluff Characteristics: Composition	Groundwater Conditions	Shore Protection Structure	Significant Bluff Toe Erosion	Type of Bluff Slope Failure
Village of Whitefish Bay (cont'd)	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 Dr.	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 Dr.	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 Dr.-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	6464-6516 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	6530-6620 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	6702 N. Lake Drive -6810 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 lies bedrock.	No major groundwater seeps were noted	Grout-filled bag revetment	Yes	Rapid surface sliding and small slumps

Table II-25 (cont'd)

Civil Division	Bluff Analysis Section	Location	Beach Characteristics: Shoreline Length (feet)	Beach Characteristics: Width (feet)	Beach Characteristics: Slope (°)	Beach Characteristics: Composition	Bluff Characteristics: Height (feet)	Bluff Characteristics: Overall Slope (°)	Bluff Characteristics: Vegetation (%)	Bluff Characteristics: Composition	Groundwater Conditions	Shore Protection Structure	Significant Bluff Toe Erosion	Type of Bluff Slope Failure
Village of Fox Point (cont'd)	93	6818-6840 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	7038-8130 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	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 seeps occur in the upper portion of the bluff	Concrete bulkhead and concrete block revement	Yes	Shallow slides and small slumps

Source: SEWRPC

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 7800 feet, or 51 percent of the shoreline. Bluff slope failures 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 were 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 3300 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 subfluction, and sapping were 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 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 and 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 with a bluff slope of 30 degrees or less.

During the October 1987 field surveys, the City of St. Francis shoreline was divided into ten bluff analysis sections based on similar physical and erosion-related characteristics. Groundwater seepage was observed in three of the ten 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, which included 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 southern most 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 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 7600 feet and northern 2900 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 ranged from about 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 6400 feet or 12 percent of the City of Milwaukee shoreline. In all of these areas, however, the erosion does not affect the overall stability of the bluff slopes. 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 range 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 bluff analysis sections within the Village covering about 2820 feet or 43 percent of the shoreline. Seven bluff analysis sections containing

about 3600 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, which included 4000 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 is 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 2600 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 bluff analysis sections within the Village covering about 5600 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 at least portions of 11 sections, which included 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 waters edge along the southern 4670 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 a relatively wide terrace exists in front of the bluffs, which terrace extends to a maximum width of approximately 900 feet and ranges 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 bluff analysis sections within the Village, which included 1100 feet or 8 percent of the shoreline. Portions of seven sections containing 7000 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 at least portions of six of the sections, which included 10200 feet of 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 9170 feet of Lake Michigan shoreline, or 6 percent of the total County shoreline is included 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 bluff analysis sections, accounting for 1300 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 at least portions of three sections, which included 2100 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. 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 B presents the measured shoreline recession rates, as well as the volume of shoreline material lost, for the time period 1963 through 1985, for each shoreline recession reach. Shoreline length, bluff height, and the volume of bluff or shoreline material lost for each reach are also presented in the Appendix. 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.

A summary of estimated shoreline recession rates and associated shoreline lengths and the volume material loss to erosion is shown in Table II-26. About 63 percent of the shoreline had an average annual recession rate of less than 0.5 foot per year. 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 each year containing approximately 328,000 cubic yards of shore and bluff material. The 3 percent of the total 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 of 1836 through 1985 are also given in Appendix B. These long-term recession rates are based on the original U.S. Public Land Survey field notes made in 1836, and the Commission one inch equals 100 feet scale, two-foot contour interval topographic maps made from 1980 through 1987, and 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 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 have 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

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Table II-26

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

Bluff Recession Rate (Feet/Year)	Shoreline 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
>10.0	1,390	0.9	43,400	13.2
TOTAL	159,110	100.0	327,600	100.0

Source: SEWRPC

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 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 range 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, covers 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 i 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. About 17 percent of the annual precipitation occurs as snowfall and sleet. Lake ice formation begins in late November or December and ice breakup normally occurs in late March or early April.

The nearshore Lake Michigan area contains an established salmonid fishery, while the Milwaukee outer harbor contains a warmwater fishery with 30 species of fish being surveyed. 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 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 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, with several perch spawning areas being located offshore of Milwaukee County. 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 Oak Creek, South Milwaukee, St. Francis, and Milwaukee, and the Villages of Shorewood, Whitefish Bay, Fox Point, and Bayside, and encompasses a total of 7,517 acres. About 4,443 acres, or 59 percent of the total 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 presently 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 to 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, have 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. However, these costly measures, installed by both private shoreline property owners and by public agencies, have had varying degrees of success. An inventory of all 128 shore protection structures in the study area indicated that only abut 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 is October 1987 and in May of 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 to 2.0 feet per year, and about 12 percent of the shoreline exhibited a bluff recession rate exceeding 2.0 feet per year. Those areas with a recession rate of 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 each year, containing approximately 328,000 cubic yards of shore material. Long-term bluff recession rates, calculated for the period of 1836 to 1985, ranged from 0.5 to 4.5 feet per year, with an average rate of 1.6 feet per year.

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SEWRPC Community Assistance Planning Report No. 163

A LAKE MICHIGAN SHORELINE EROSION, BLUFF RECESSION, AND STORM DAMAGE CONTROL PLAN FOR MILWAUKEE COUNTY

Chapter III

EVALUATION OF COASTAL EROSION PROBLEMS AND DAMAGES

INTRODUCTION

The identification of those shoreland areas which 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 assess the effectiveness of existing shore protection structures under various storm wave and lake level conditions; to describe the reaches of the Lake Michigan shoreline through Milwaukee County experiencing bluff toe erosion and having the potential for bluff slope failure; to identify the potential property and economic losses which may result from continued shoreline erosion and bluff recession; to describe those factors contributing to that erosion and recession; and to generally identify the types of shoreland protection measures necessary to control future property loss within each of the bluff analysis sections described in Chapter II. This information is intended to enable public officials and other concerned and affected parties 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.

It must be recognized that the results of this chapter are based on systems level, somewhat generalized 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 requires a site specific analysis 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 should also be used to help coordinate shore protection efforts of adjacent

property owners, which should facilitate the design and construction of more effective measures, and help minimize any potential adverse impacts on nearby shoreline areas.

The first section of this chapter following the introduction describes the analytic procedures, and coastal and geotechnical engineering techniques used to evaluate existing shore protection measures, major harbor, and lake front facilities, and existing shore erosion problems within Milwaukee County; presents the results of this evaluation, and identifies needed control measures for each of the 100 bluff analysis sections. The second 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 third and final section summarizes the coastal erosion problems and damages within Milwaukee County.

EVALUATION OF COASTAL EROSION PROBLEMS

The Lake Michigan shoreline erosion problems of primary concern are storm damages to major harbor and lake front 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 shore protection measures to protect major harbor and lake front 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 runnup elevations on revetments, bulkheads, and beaches under selected lake levels and associated wave conditions; to identify those shore protection structures and facilities subject to overlapping, flanking, undercutting, and other types of damage from wave action; and to determine the measures needed to repair or rehabilitate existing shore protection structures. The analysis utilized coastal engineering techniques, including a hydrodynamic model to simulate wave conditions for the entire Milwaukee County shoreline.

The extent and severity of bluff toe erosion was determined by aerial photo interpretation and by observations made during field surveys conducted in 1987 for southern Milwaukee County and the Village of Bayside; and in 1986 for

property owners, which should facilitate the design and construction of more effective measures, and help minimize any potential adverse impacts on nearby shoreline areas.

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The extent and severity of bluff toe erosion was determined by aerial photo interpretation and by observations made during field surveys conducted in 1987 for southern Milwaukee County and the Village of Bayside; and in 1986 for

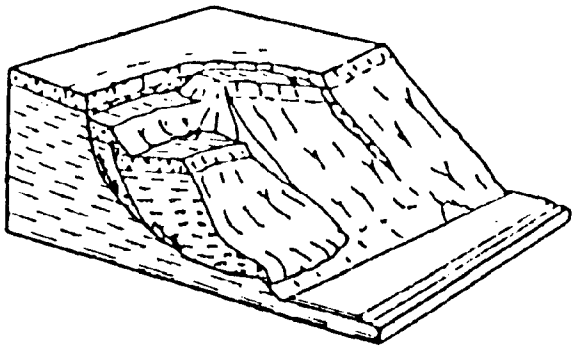
northern Milwaukee County. The stability of the bluff slopes was evaluated using geotechnical engineering models which calculate the risk of rotational and translational sliding. Based on the results of both the bluff toe erosion analyses and the slope stability analyses, an assessment of the degree 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 is the primary cause of bluff slope failure, while other areas experiencing toe erosion exhibit 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 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.

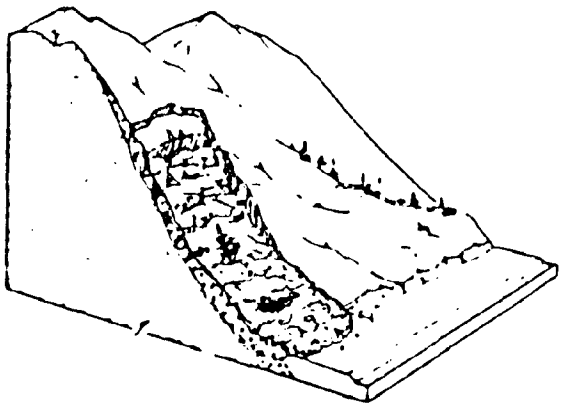
The bluff slope stability analyses conducted under this study evaluated the potential for the two types of slope failures most common along the Milwaukee County shoreline: rotational slides and translational slides. Rotational sliding involves failure along a curved, or "spoon shaped," surface. As the 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 III-1 illustrates the characteristics of rotational slides and translational slides. The distinction between rotational and translational slides is useful in the planning and design of control measures. As shown in Figure III-2, 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.

Figure III - 1

COMMON TYPES OF SLOPE FAILURES IN
LAKE MICHIGAN COASTAL BLUFFS



ROTATIONAL SLIDING

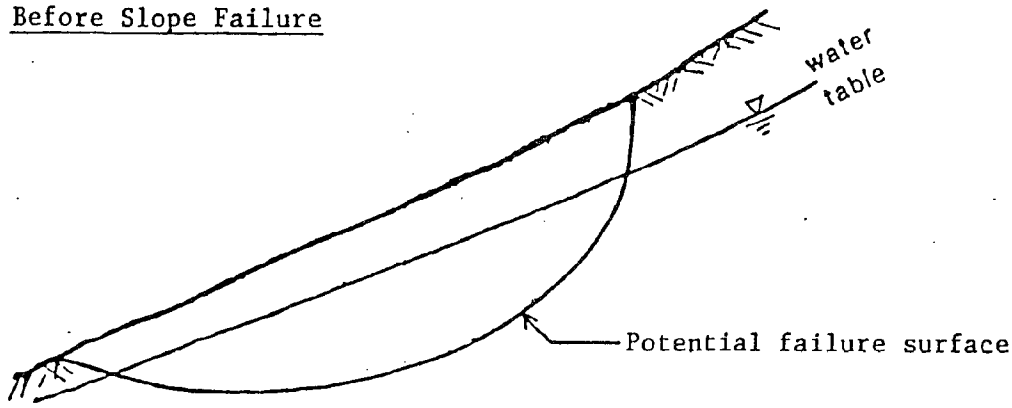


TRANSLATIONAL SLIDING

Source: David J. Varnes Slope Movement and Types and Processes. In *Landslides: Analysis and Control*, Transportation Research Board, National Academy of Sciences, Washington, D.C., Special Report 176, Chapter 2, 1978.

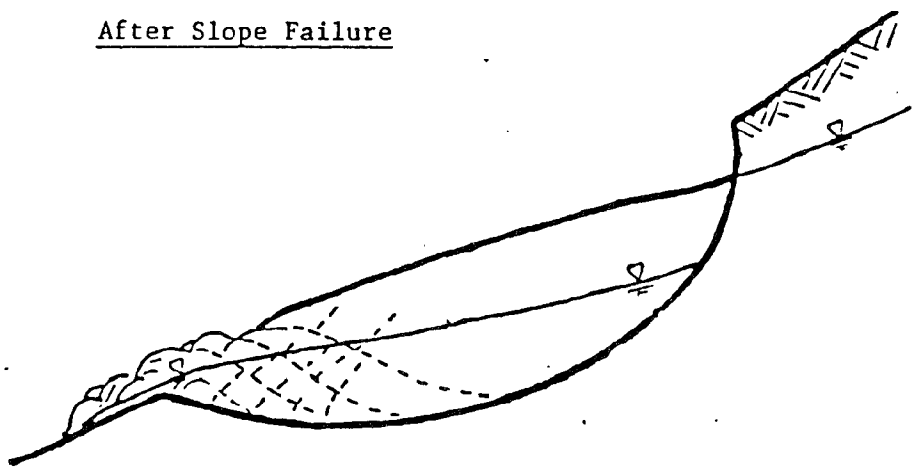
Figure III-2
EFFECT OF ROTATIONAL SLIDING ON SLOPE STABILITY

Before Slope Failure



Safety factor=1.0 at time of failure

After Slope Failure



Safety factor=1.5 after failure

SOURCE: J. David Rogers, Choice of Input Parameters for Slope Stability Analysis, Chapter 4, "Slope Stability Evaluations of Various Geologic Situations", 1986.

Definition of Safety Factor

Using shear strengths and stresses, factors of safety were calculated for potential failure surfaces within the bluff. This 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.

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 wave conditions. The degree of erosion occurring at the toe of a bluff is determined by the offshore slope, wave conditions, beach width and slope, type of material in the bluff, and the presence 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. It is important to note that the specific conditions present at any given shoreline site vary from the general conditions described herein. The following section describes the methods used to evaluate these factors and their effects upon shoreline erosion and bluff recession within the study area.

Bluff Toe Erosion: Bluff toe erosion is of particular concern to this study because of the record-breaking high water levels experienced in Lake Michigan in 1986, which, in many shoreline reaches, caused waves to break directly at the base of the bluff. This toe erosion occurs to some degree in nearly all shoreline areas not protected by adequate shore protection structures. Erosion at the toe of the bluff initiates changes in slope geometry, which in turn trigger slope failures on the upper bluff slope, and therefore must be considered in any bluff stability analysis. Toe erosion also affects the erosion and recession of the low terrace in the Villages of Fox Point and Bayside.

During the 1987 field surveys conducted in southern Milwaukee County and the Village of Bayside, and the 1986 field surveys conducted in northern Milwaukee County, those portions of the study area shoreline which 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, exposing the base of the bluff. Shoreline reaches experiencing bluff toe erosion within the study area were also identified on colored, oblique aerial photographs prepared under this study in 1987.

Detailed measurements of the geometry of the bluff slope, which were conducted at 104 sites, 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 toe erosion and the impact of toe erosion on the overall stability of the bluff slope, was determined for each bluff analysis section. The bluff analysis sections were classified into three categories of toe erosion. Category I, defined as having slight toe erosion, includes shoreline areas with little or no evidence of toe erosion. Category II, defined as having moderate toe erosion, includes shoreline areas where evidence of toe erosion was observed, but where such erosion did not appear to be affecting the overall stability of the bluff slope, generally because of the presence of a terrace at the base of the bluff. Category III, defined as having severe toe erosion, includes shoreline areas where bluff toe 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 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 planar with a steeper upper portion at the rupture surface, with a progressively decreasing slope angle. Slope stability analyses were performed for the bluffs using surveyed geometric profiles of the bluffs; laboratory analyses of the bluff

material properties; and modified versions of the computer program STABL.[1] STABL was developed 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 Professor Peter J. Bosscher of the University of Wisconsin-Madison for personal computer use, and for data enhancement purposes.

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 slices. The forces acting upon a typical slice are shown in Figure III-3. The forces exerted in a vertical direction are taken into account, while the horizontal forces across a slice--or between slices--are ignored. The resulting equation for calculating the safety factor is:

$$SF = \frac{\sum [c' b + (W - ub) \tan \phi'] \quad 1/m}{\sum W \sin \alpha}$$

where: SF = Safety factor

$m = \cos \alpha \left[1 + \left(\tan \alpha \tan \frac{\phi'}{SF} \right) \right]$

W = weight of individual slice

b = width of slice

α = slope angle

u = pore water pressure

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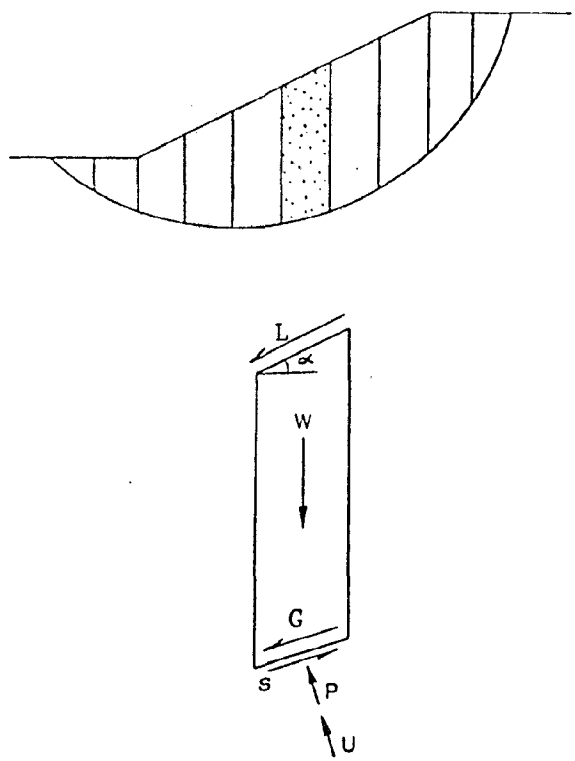
[1] R. A. Siegel, STABL User Manual, Joint Highway Research Project, Purdue University and the Indiana State Highway Commission, JHRP-75-9, June 1975.

Figure III - 3

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



where: L = Length of Failure Surface
α = Angle of Inclination
W = Weight of Slice
G = Gravity
s = Shear Strength Forces (Cohesion and Friction)
P = Normal Force
U = Seepage Force

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

c = cohesion intercept
ø = internal friction angle

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

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. 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 location. The second version utilized a probabilistic approach which allowed the input data to vary randomly within specified dispersions.[2] 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. The 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 northern Milwaukee County were used in the deterministic slope stability analysis. The location of the profile sites, which are presented in Table III-1, were selected to be representative of bluff areas with different physical characteristics and different causes and types of slope failure. From one to three profiles were prepared for 82 of the bluff analysis sections. The 18 remaining analysis sections included shoreline areas where the bluff has either been previously regraded to a stable slope angle, or is protected by a major lake front facility; and where a natural bluff is not located directly

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

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Table III-1

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

Table III-1 (cont'd)

Civil Division	Bluff Analysis Section	Profile Number	Location
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' 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		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

Table III-1 (cont'd)

Civil Division	Bluff Analysis Section	Profile Number	Location
Village of Whitefish Bay (cont'd)	86	88	5960 N. Shore Drive
	87	89	614 E. Lake Hill Court
	88	90	6330 N. Lake Drive
Village of Fox Point		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

at the water's edge. Within these sections, no slope stability analysis was conducted.

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 most important in bluffs greater than 80 feet high.[3] 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 period for the stability of Lake Michigan coastal bluffs for both deep-seated and shallow

slides.[4] 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.

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

[4]L. E. Vallejo and T. B. Edil, 1979, "Design Charts for Devlopment and Stability of Evolving Slopes," J. Civil Eng. Design, 1(3):231-252.

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 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.[5]

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. In the evaluation of the stability of this type of slope, the failure surface having the lowest safety factor is most important, even if that failure surface with the lowest safety factor would affect a small portion of the bluff slope.

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[5]Ibid.

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 northern Milwaukee County, historical geologic records of soil boring data, and 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 III-2. 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 historic 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 calculate the strength 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 III-3.

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

Table III-2

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

Civil Division	Bluff Analysis Section	Profile Number	Location	Field Observation of Exposed Bluff Face Within Section 1986-1987[a]	Soil Boring Within Section 1986	Soil Boring Within Section 1987	Soil Boring Within Section Pre-1986	Field Observation of Exposed Bluff Face Within Adjacent Sections[a]	Soil Boring Within Adjacent Sections
City of Oak Creek	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	--	--	--	--	--
	10	13	9006 S. 5th Avenue	--	--	--	--	--	X
	11	14	9006 S. 5th Avenue	--	--	--	X	--	--
	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	--	--	--
	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	--	--	--
	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	--	--
	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	--

-continued-

Table III-2 (cont'd)

Civil Division	Bluff Analysis Section	Profile Number	Location	Field Observation of Exposed Bluff Face Within Section 1986-1987[a]	Soil Boring Within Section 1986	Soil Boring Within Section 1987	Soil Boring Within Section Pre-1986	Field Observation of Exposed Bluff Face Within Adjacent Sections[a]	Soil Boring Within Adjacent Sections
City of Milwaukee	48	55	South Shore Park	--	--	X	--	--	--
	50	56	South Shore Park	--	--	--	--	--	X
	61	57	3252 N. Lake Drive	--	--	--	--	--	X
	62	58	100' 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		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	--	--	--	--
Village of Fox Point	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	--	--	--	--
	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	--	--
	96	99	Doctors Park	X[b]	--	--	--	--	--
		100	Doctors Park	X[b]	--	--	--	--	--
Village of Bayside	97	101	9360 N. Lake Drive	--	--	--	--	--	X
	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 summer of 1986, allowing field determination of the stratigraphy.

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

Source: SEWRPC

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Table III-3

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 (degree)
Tills				
New Berlin	138	138	10	34-36
Oak Creek	135	135	10-100	27-31
Ozaukee	134	134	150	30
Fractured Ozaukee ..	134	134	10	30
Tiskilwa	10	130	350	27
Lake Sediments				
Medium Fine Sand ...	120	120	0	33-43
Sand and Gravel	120	120	0-1340	22-33
Silt	130	130	200-4000	27-32
Silt and Fine Sand .	110	110	0-10	31-43
Clay and Silt	130	130	150-1340	22-27
Fine Sand and Silt .	110-125	110-125	100	33
General Lake Sediment	125	125	0-100	26-31
Coarse Sand	120	120	0	33
Fill				
Concrete Rubble and Soil	130	130	0	33-35

Source: SEWRPC.

elevation of the groundwater within each of the bluff analysis sections was determined based on the sources of data previously set forth in Table II-8.

For each of the profile sites, the deterministic version of STABL was used to generate 100 potential failure surfaces and to calculate the corresponding safety factors. The ten failure surfaces with the lowest safety factor were identified. The three lowest safety factors are shown in this report for each profile site.

Probabilistic Slope Stability Analysis--The probabilistic version of STABL was developed for use in this study by Professors Peter J. Bosscher and Tuncer B. Edil of the University of Wisconsin-Madison under contract to the Commission. The probabilistic model[6] 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. The 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 either obviously 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.

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[6]T. B. Edil and M. N. Schultz, Landslide Hazard Potential Determination Along A Shoreline Segment, Wisconsin Sea Grant Institute, 1983.

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 which were allowed to vary randomly.

The data dispersions used for the probabilistic analyses were selected by Professor Tuncer B. Edil. 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 three to ten 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 judgement 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. However, numerous repetitions of the analysis, each corresponding to a combination of the variable parameters randomly fixed within their dispersions, helps assess the likelihood of slope failure. The probabilistic analysis thus helps quantify the risk 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 interface lines as they proceed into the bluff were 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

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Table III-4

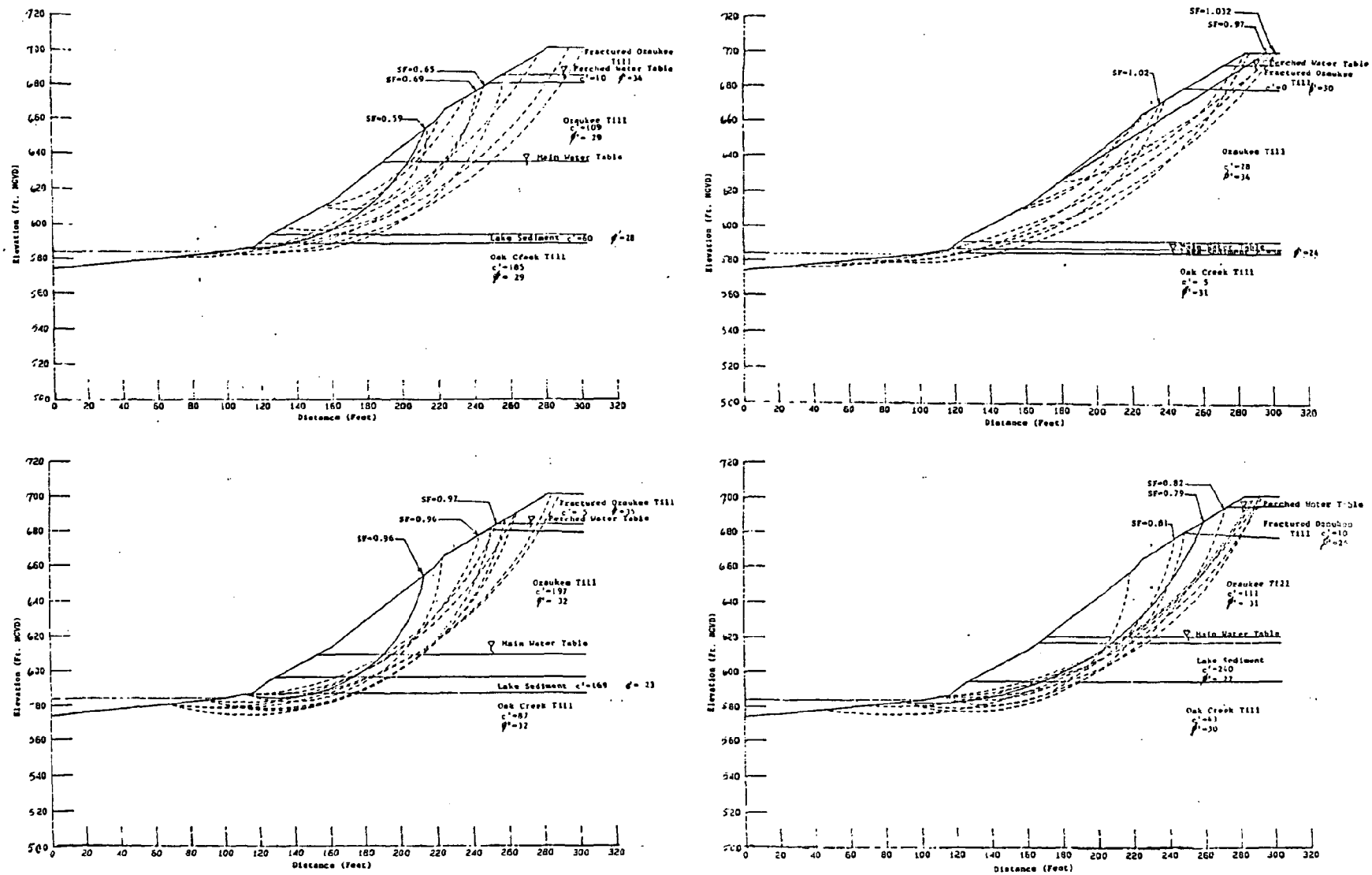
VARIATION IN SOIL MPROPERTIES USED IN THE PROBABILISTIC SLOPE STABILITY ANALYSIS

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

Figure III - 4

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.

NOTE: c'=Effective Cohesion Intercept in Pounds per Square Foot.
ϕ'=Internal Friction Angle in Degrees.
SF = Safety Factor

Table III- 5

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 both 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 are considered. In those cases, a final classification was determined based on the subjective judgement of the Commission staff and its consultants.

Source: SEWRPC.

Table III- 5

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
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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 both 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 are considered. In those cases, a final classification was determined based on the subjective judgement of the Commission staff and its consultants.

Source: SEWRPC.

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 ten lowest safety factors were considered.

Bluff Slope Instability by Translational Sliding: 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 analyses for translational sliding is illustrated in Figure III-5. 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.

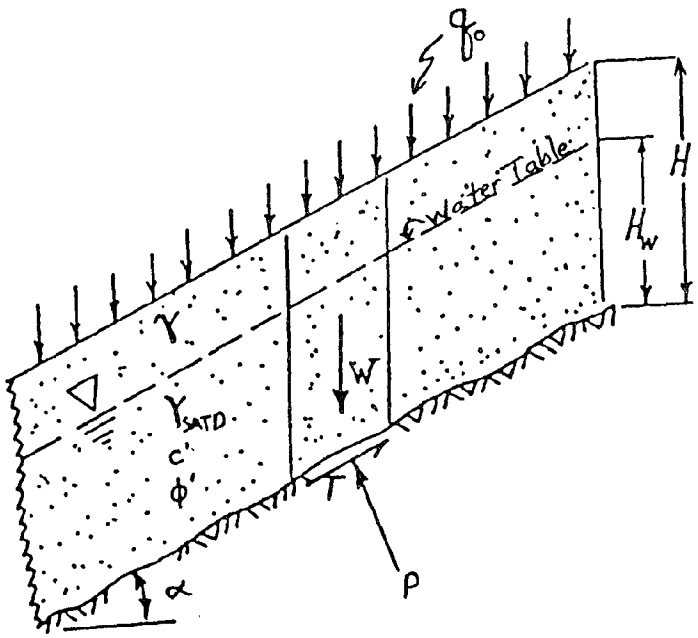
The safety factor for translational sliding based on the infinite slope analysis is calculated with the following equation:

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

where:
- SF = Safety factor
- o' = internal friction angle
- c' = cohesion intercept
- α = slope angle
- γ = moist density of soil
- γ_{SATD} = saturated density of soil
- γ_{BUOY} = buoyant density of soil ($\gamma_{BUOY} = \gamma_{SATD} - \gamma_w$)
- γ_w = density of water
- H = vertical thickness of sliding mass
- H_w = piezometric height above sliding surface
- q = uniform vertical surcharge stress on slope

Figure III-5

CONCEPT OF THE INFINITE SLOPE ANALYSIS
FOR TRANSLATIONAL SLIDING



WHERE: q_o = vertical surcharge

W = weight of soil mass

P = Normal Force

γ = Unsaturated Density of Soil

γ_{SATD} = Saturated Density of Soil

c' = Cohesion Intercept

ϕ' = Internal Friction Angle

H = Vertical Thickness of Sliding Mass

Hw = Piezometric Height Above Sliding Surface

T = Tensite Strength of Vegetation Roots

α = Slope Angle

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

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 previously 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 estimated to be three feet. This thickness is typical of shallow sliding masses along the Lake Michigan shoreline. A depth of three feet also approximates the average 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 III-3.

The effect of groundwater was evaluated under three conditions. The first condition assumed the soil to be unsaturated. 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.

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. 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 severe likelihood of failure. Such bluffs were classified 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 III-6 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 III-6.

RESULTS

An evaluation of each bluff analysis section, including a determination of the likelihood of bluff slope instability by rotational sliding and translational sliding, and an assessment of the severity of bluff toe erosion, is presented below. A summary of the results of the evaluation of shoreline erosion and bluff instability within the entire study area is also presented.

The results of the analyses were used to determine the shoreline protection needs for the study area. For each bluff analysis section, the types of shoreland protection measures needed to fully stabilize the bluff slope and protect the toe against erosion are presented. Effective shore protection 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 by either filling, or cutting back the slope. In order to maintain the natural aesthetic properties and natural drainage characteristic of the bluff, modification of the bluff slope by filling, or cutting back the slope was recommended only where the other control measures--which would maintain or reestablish these natural characteristics--would not effectively stabilize the slope. It is recognized that filling could effectively be used to stabilize many slopes in lieu of other types of control measures. Chapter IV describes and evaluates the specific alternative shore protection measures available.

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Table III-6

POTENTIAL FOR TRANSLATIONAL SLIDING UNDER BLUFF CONDITIONS
FOUND IN MILWAUKEE COUNTY

Soil Type	Vegetated Bluff Face: Groundwater Condition in Bluff	Slope Angle 10°	20°	30°	40°	Unvegetated Bluff Face: Groundwater Condition in Bluff	Slope Angle 10°	20°	30°	40°
Tills										
Ozaukee	Unsaturated	S[a]	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	M	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
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	S1	S1	S1	S1	Unsaturated	S	S	M	U
	Seepage parallel to face	S1	S1	S1	S1	Seepage parallel to face	S	U	U	U
	Seepage emerging from face	S1	S1	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]Potential for Translational Sliding: S Stable Bluff Slope
M Marginal
U Unstable Bluff Slope

Source: SEWRPC

The results set forth in this report are based on systems level, somewhat generalized analyses which determined the condition of each bluff analysis section, and identified the actions needed to protect the shoreline and bluff slope. The evaluation of individual lakeshore properties and the detailed design of shore protection measures requires a site specific analysis by a professional geotechnical or coastal engineer.

Bluff Analysis Section 1: The entire shoreline of Section 1, located at the Wisconsin Electric Power Company Oak Creek plant, was protected by an interlocking sheet pile bulkhead and rip-rap revetment. The natural bluff has been graded to a stable slope. No erosion of the bluff was observed during the field survey conducted during the fall of 1987.

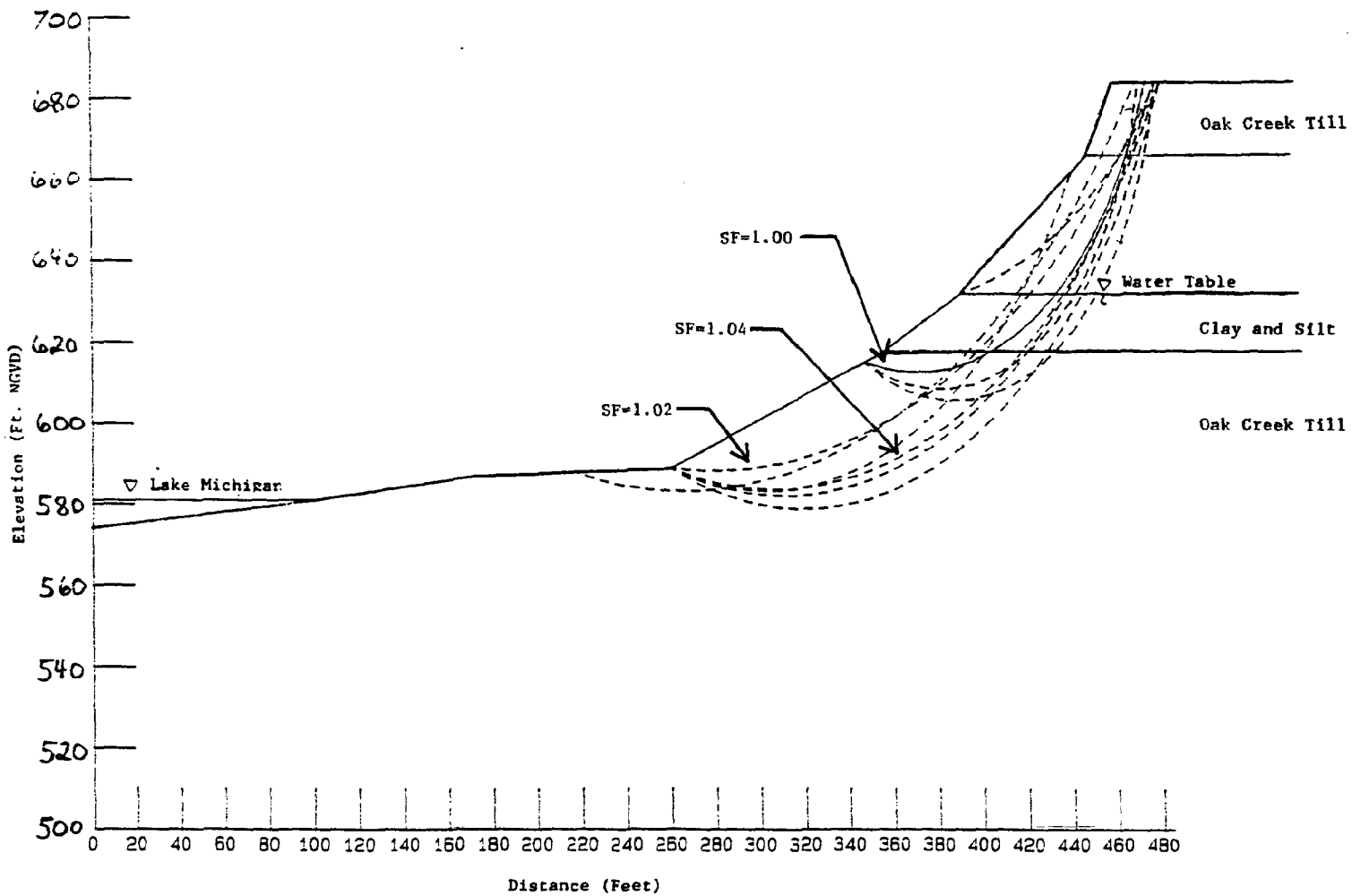
Scour at the base of the revetment which fronts 80 to 90 percent of the bulkhead was observed during the field survey. Toe erosion control measures, other than maintenance of the existing structure, are not needed to insure bluff stability in this section.

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 plan which extends from Elm Road to Oakwood Road, was characterized by the use of Profile No. 1, Profile No. 2, and Profile No. 3.

The results of the deterministic slope stability analyses shown in Figure III-6, Figure III-7, and Figure III-8 for Profile Nos. 1, 2, and 3, respectively, indicate portions of the bluff slope within Section 2 were 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 remaining nine 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 remaining nine 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 remaining nine safety factors ranged from 1.25 to 1.34.

Figure III -6

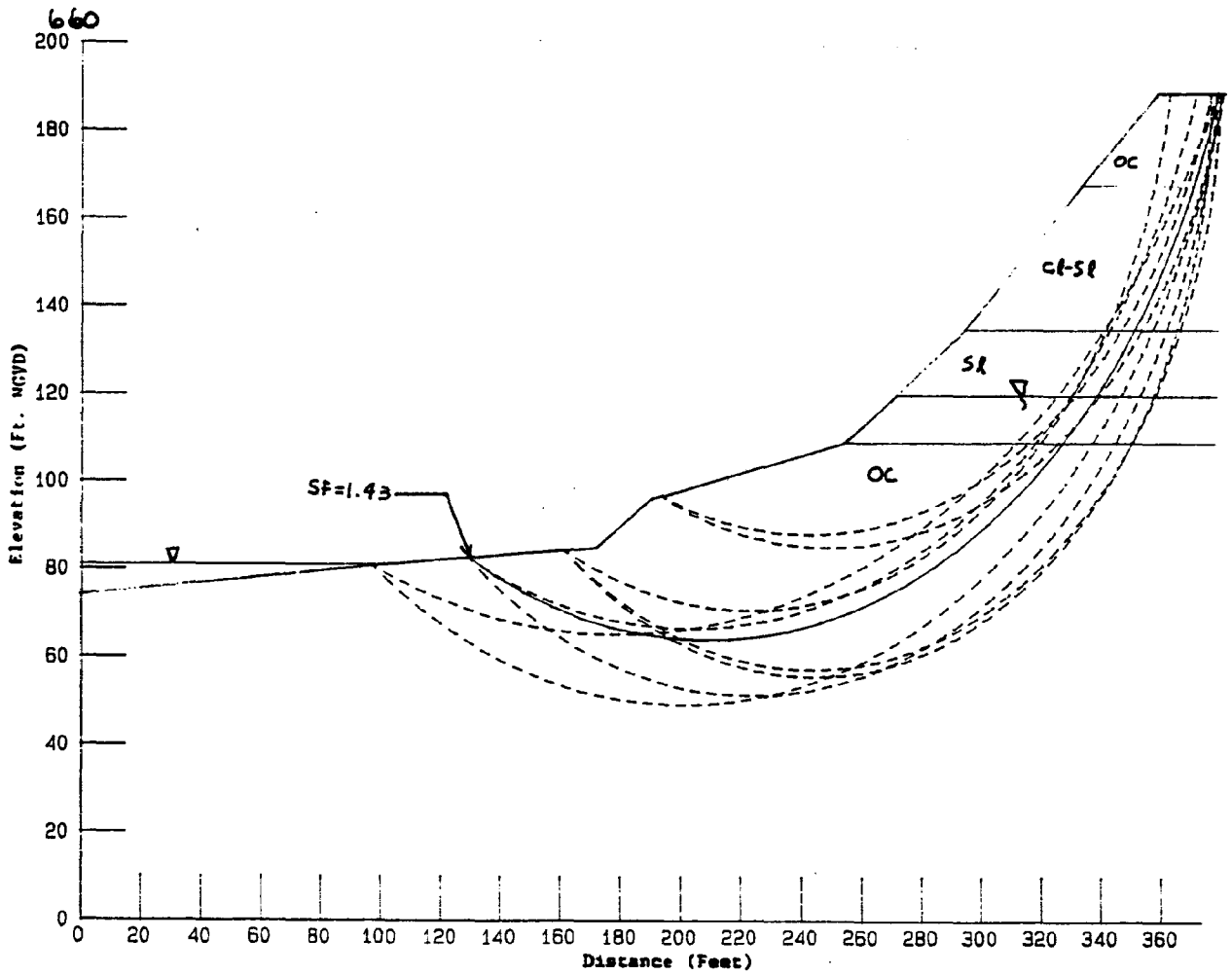
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 1



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

Figure III-7

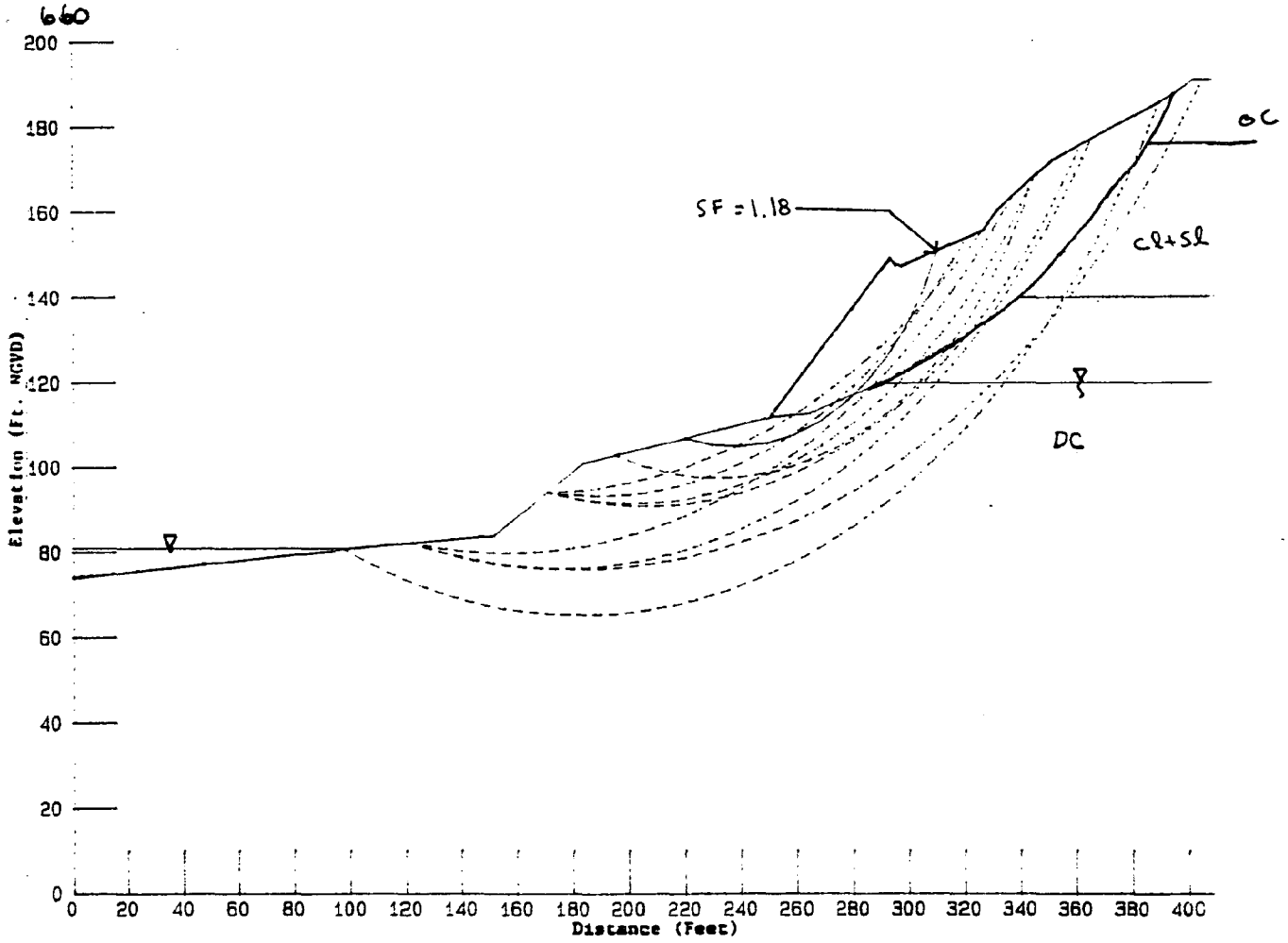
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 2



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

Figure III -8

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 3



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

Of the 20 probabilistic stability analyses conducted for Profile No. 1, the lowest safety factors ranged from 0.73 to 1.37, with 11, or 55 percent, having a safety factor of less than 1.0. Of the total 200 failure surfaces evaluated at Profile No. 1, 87, or 43 percent of the surfaces, had safety factors of less than 1.0. Of the 20 probabilistic stability analyses conducted for Profile No. 2, the lowest safety factors ranged from 0.79 to 1.36, with 9, or 45 percent, having a safety factor of less than 1.0. Of the total 200 failure surfaces evaluated at Profile No. 2, 64, or 32 percent of the surfaces, had safety factors of less than 1.0. Of the 20 probabilistic analyses conducted for Profile No. 3, the lowest safety factors ranged from 0.99 to 1.62, with two, or 10 percent of the surfaces, having safety factors less than 1.0. Of the total 200 failure surfaces evaluated at Profile No. 3, two, or 1 percent of the surfaces, had safety factors of less than 1.0. Based on the results of the probabilistic slope stability analyses, Section 2 was considered to have marginal potential for slope failure, depending on specific conditions within the bluff.

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 northern two-thirds of the section, the lower segment of the bluff slope was unstable, despite relatively good vegetative cover. Profuse groundwater seeps observed during the 1987 field survey were considered to be the major influence in slope failure here. In the southernmost end, steep slope angles were considered to be the major factor causing failure by translational motion.

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 Oak Creek power plant bulkhead. In the northern third, toe erosion was

observed, but did not appear to be threatening the stability of the overall bluff slope. No shore protection structures were located within this section as of 1987.

Surface water runoff and high groundwater levels enhance failure by both translational and rotational sliding. A groundwater drainage system should be installed to minimize this affect. Also, a system controlling surface water runoff should be emplaced. It does not appear necessary at this time to provide additional protection against wave and ice action at the toe of the bluff.

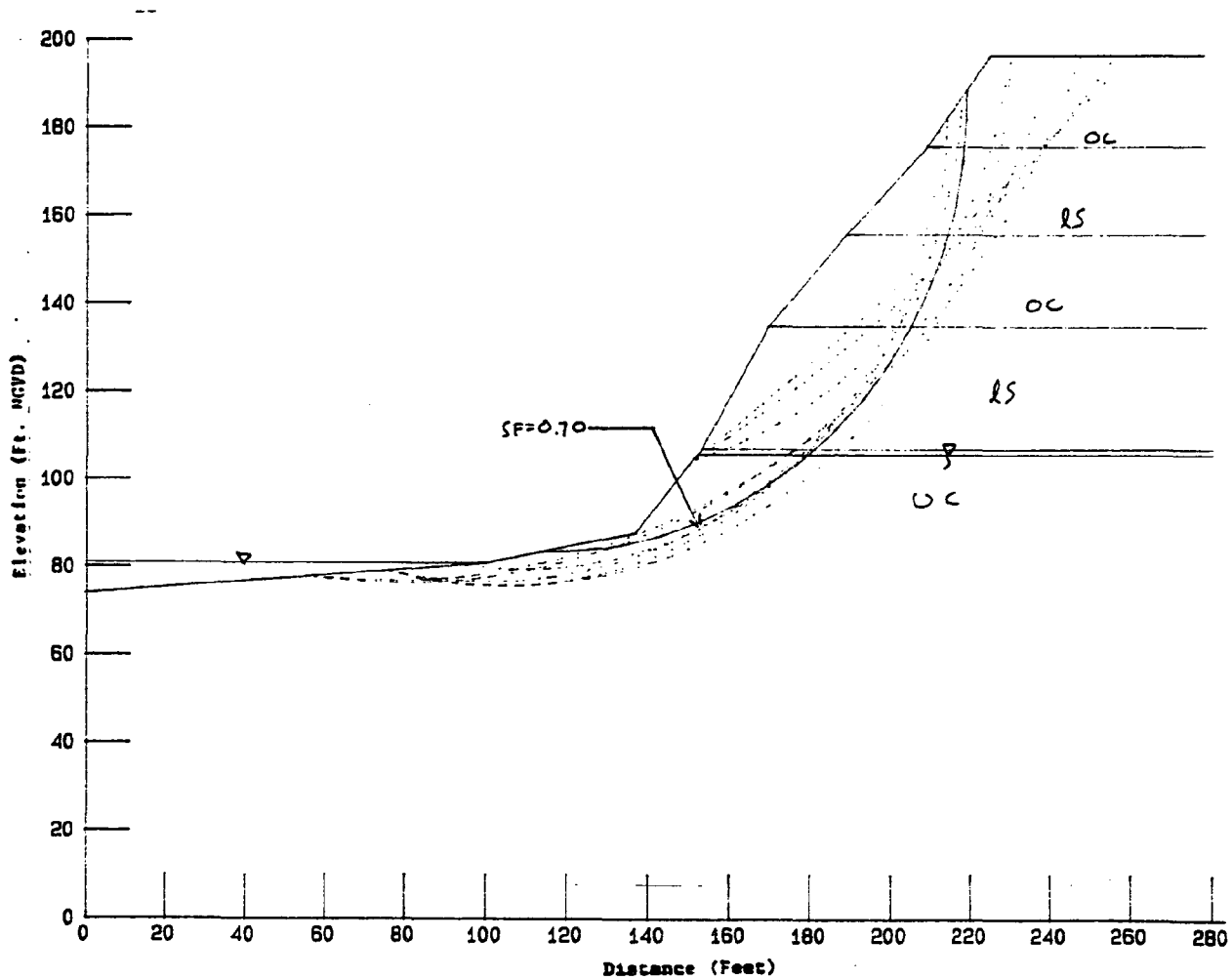
Bluff Analysis Section 3: The stability of the bluff slope within Section 3, located in Bender Park in the City of Oak Creek, was characterized by the use of Profile No. 4, Profile No. 5, and Profile No. 6.

The results of the deterministic slope stability analyses, shown in Figures III-9 and III-10 for Profile Nos. 4 and 6, respectively, indicate 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 nine remaining 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 nine remaining lowest safety factors ranged from 0.76 to 0.83. Profile No. 5, shown in Figure III-11, a site of recent slumping, was stable with respect to rotational sliding. The lowest failure surface calculated at Profile 5 had a safety factor of 1.13 and included the entire bluff face. The nine remaining safety factors ranged from 1.13 to 1.19. Based on the deterministic slope stability analyses and on observed bluff conditions, Section 3 was considered to have an unstable bluff slope with respect to rotational sliding.

Overall, Section 3 was also considered to have an unstable bluff slope with respect to translational sliding. This was due to the lack of vegetative cover on the bluff face, the steepness of the bluff slope and the accumulation of stormwater runoff. Groundwater seeps in the lower segment of the slope also contribute to the overall instability. A recently slumped portion of the

Figure III -9

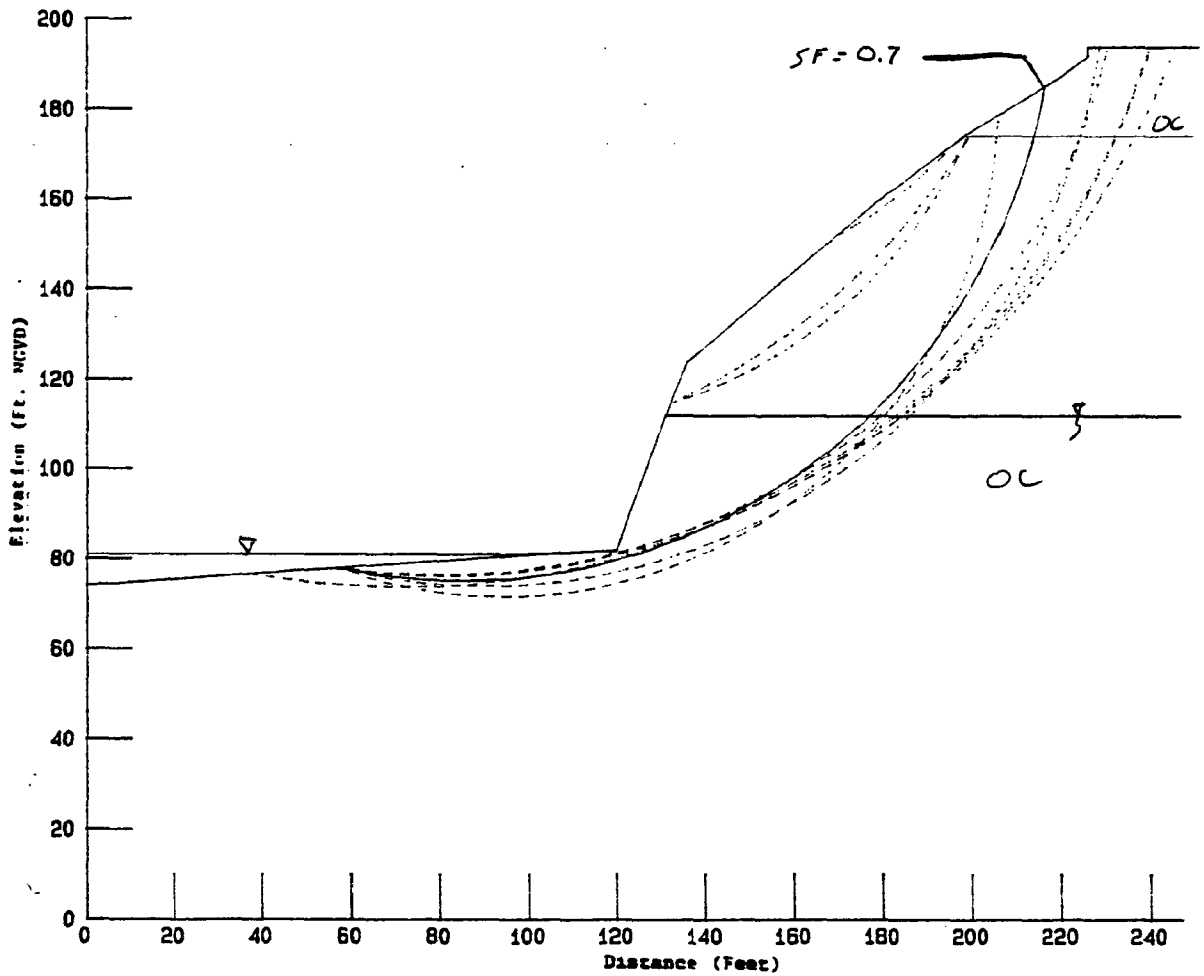
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 4



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

Figure III - 10

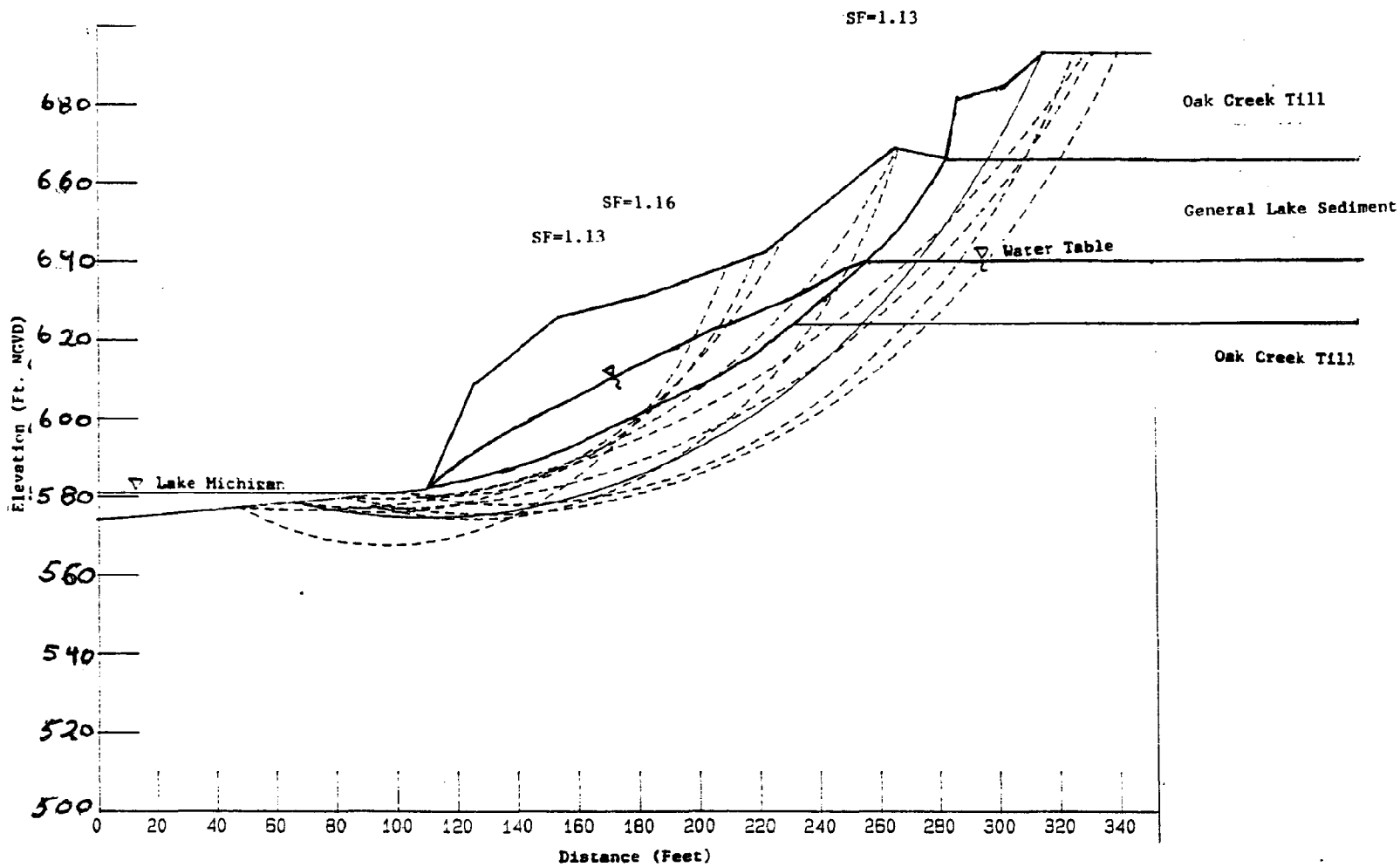
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 6



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

Figure III- 11

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 5



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

section had a more gentle slope angle than the unfailed bluff slope. The potential for translational sliding in the slump block is marginal.

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 as of 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. 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, was characterized by the use of Profile No. 7 and Profile No. 8.

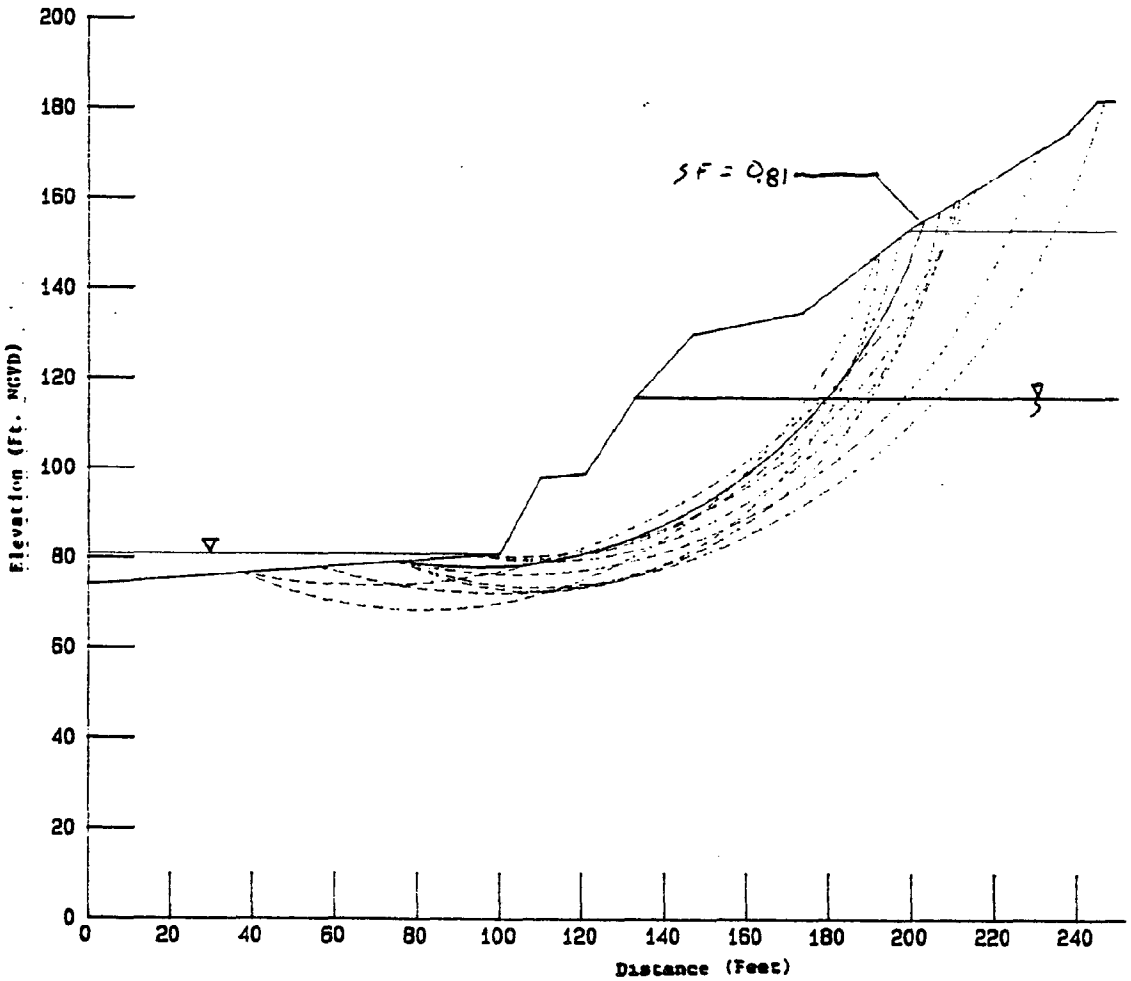
The results of the deterministic slope stability analyses shown in Figures III-12 and III-13 for Profile Nos. 7 and 8, respectively, indicate the bluff slope in Section 4 is unstable with respect to rotational sliding. The lowest failure surface calculated at Profile No. 7 had a safety factor of 0.81 and included the lower two-thirds of the bluff. The nine remaining 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 included the lower portion of the bluff. The nine remaining lowest safety factors ranged from 0.77 to 0.84. The probabilistic slope stability analysis was not conducted for this section because of the degree of instability observed in the field and verified under 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 in the upper and lower portions of the slope, and groundwater seeps 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 toe erosion was contributing significantly to the instability of the bluff slope. No shore protection structures were located within the section of of 1987.

Figure III-12

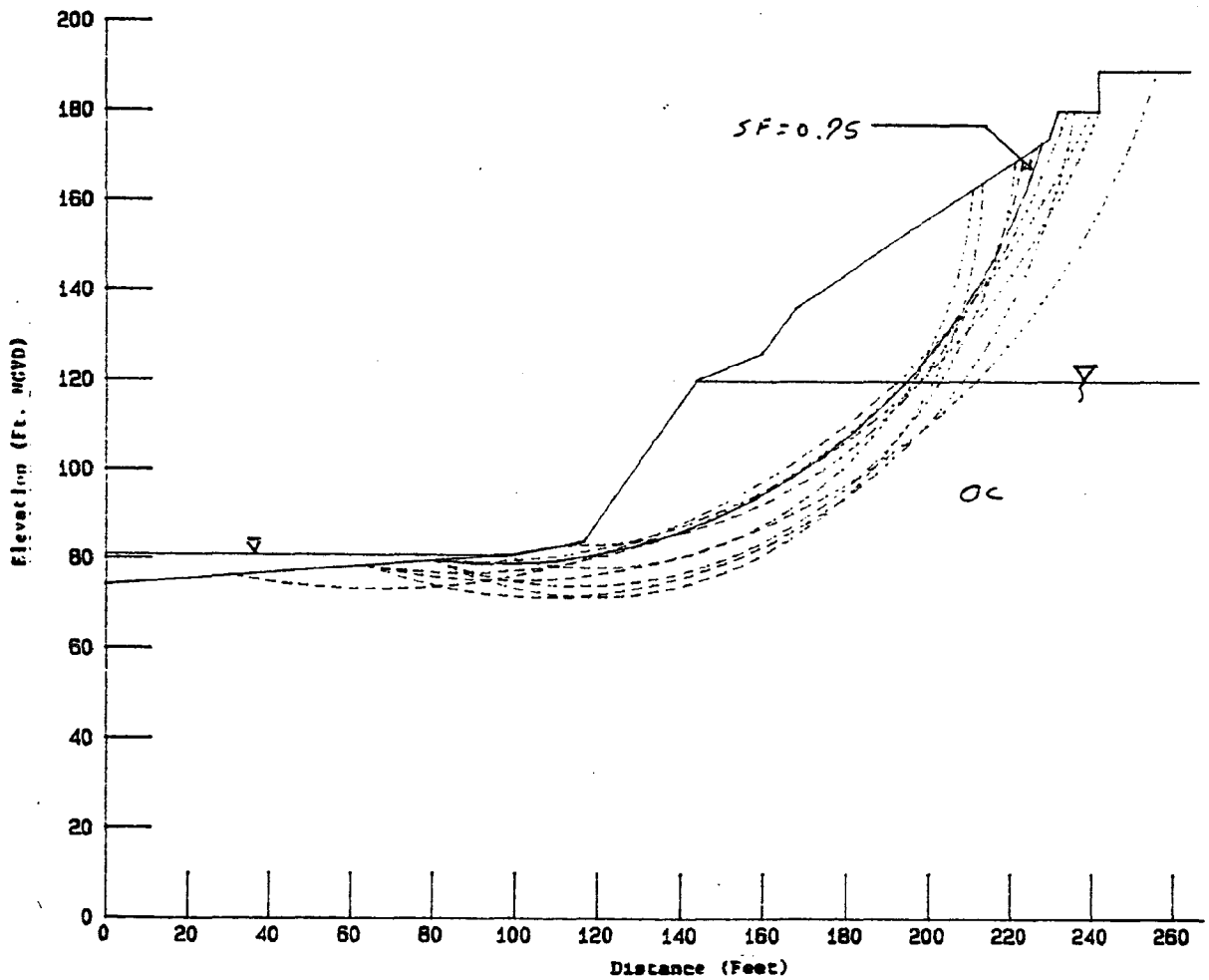
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 7



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

Figure III -13

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 8



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

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. 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 was characterized by the use of Profile No. 9.

The results of the deterministic slope stability analysis, shown in Figure III-14, 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 remaining nine lowest safety factors ranged from 0.89 to 1.03.

Of the 20 probabilistic stability analyses conducted, the lowest safety factors ranged from 0.36 to 0.83. Of the total of 200 failure surfaces evaluated, 177 surfaces, or 88 percent, had safety factors of less than 1.0. Based on both 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.

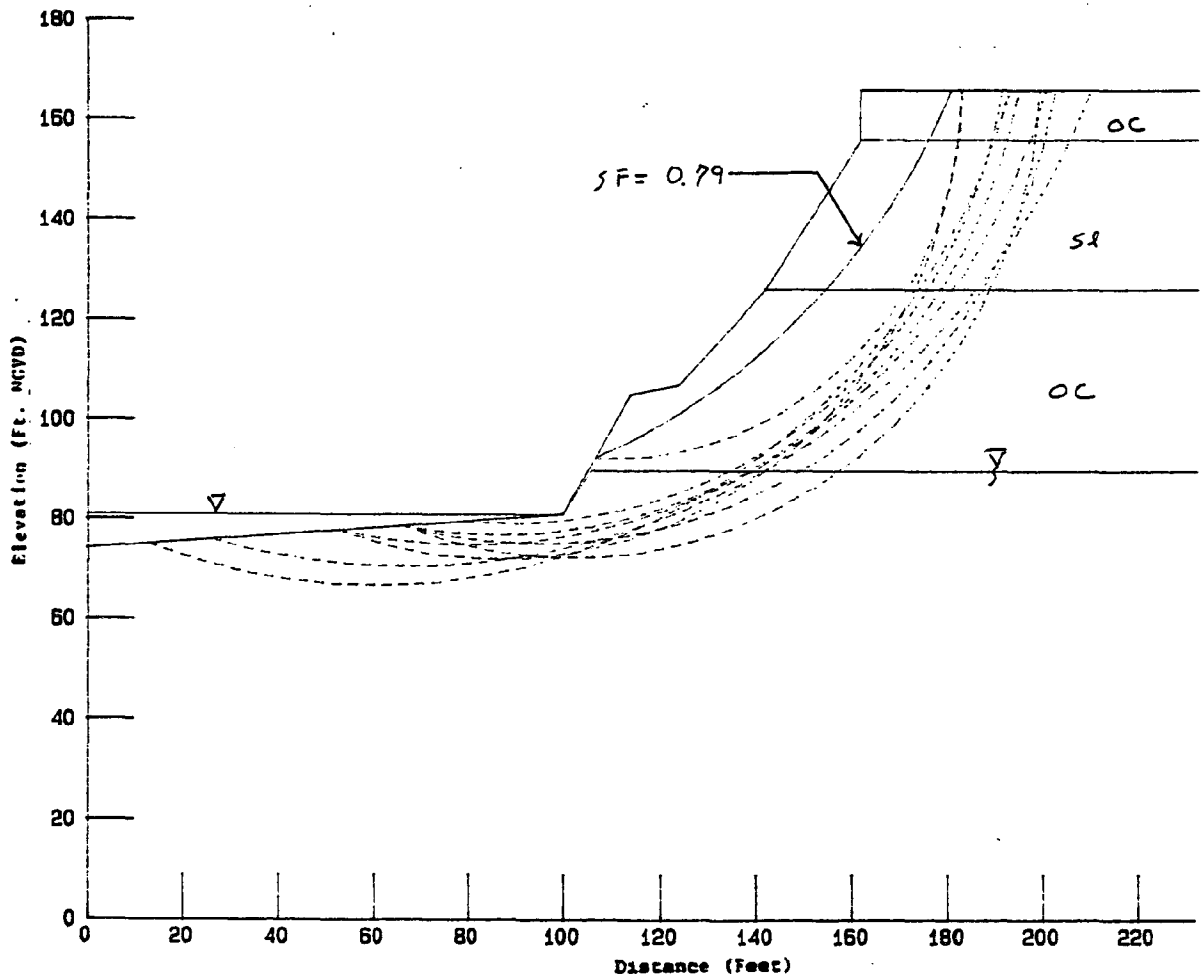
Section 5 was also considered to have an unstable slope with respect to translational sliding. This was due in part to the lack of vegetative cover on most of the bluff slope, and in part to the relatively steep angle of the bluff slope.

Bluff toe erosion was observed within the entire shoreline of Section 5 during the 1987 field survey. This toe erosion was affecting the stability of the bluff 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. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Figure III -14

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 9



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

Bluff Analysis Section 6: The stability of the bluff slope within Section 6, located at 9300 S. 5th Avenue, was characterized by the use of Profile No. 10.

The results of the deterministic slope stability analysis, shown in Figure III-15, indicate that Profile No. 10 had an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.86, and was located within the lower two-thirds of the bluff slope. The remaining nine lowest safety factors ranged from 0.87 to 0.99.

Of the probabilistic stability analyses conducted, the lowest safety factors ranged from 0.65 to 1.03, with 19 failure surfaces, or 95 percent, having a safety factor less than 1.0. Of the total of 200 failure surfaces evaluated, 159, or 99 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.

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 was mostly unvegetated. These conditions contributed significantly to the instability of the bluff in this section.

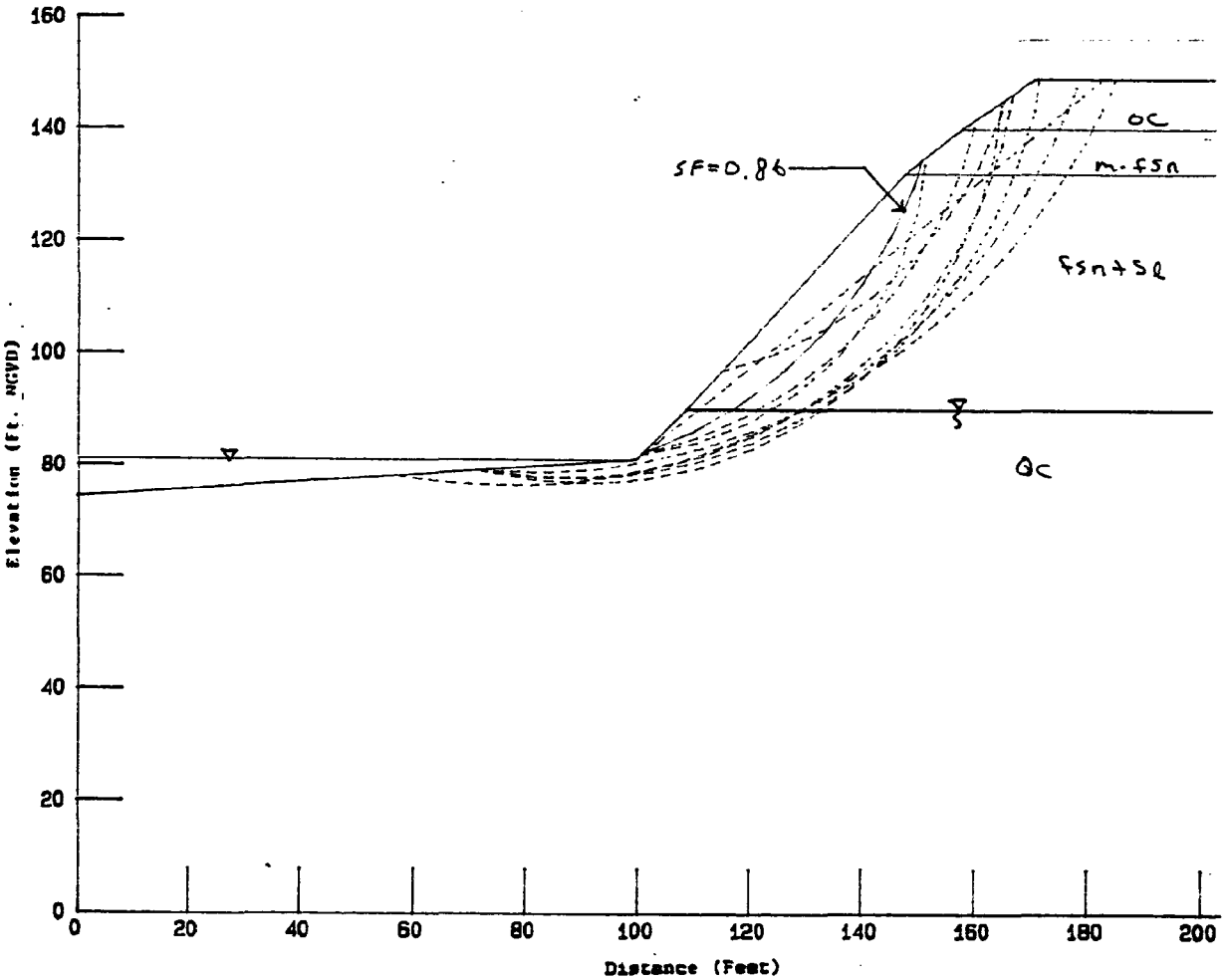
Bluff toe erosion was observed in the entire shoreline of Section 6 and was identified as a primary cause of bluff slope failure. As of 1986 no shore protection structures were located within this section.

In order to fully stabilize the bluff in Section 6, it is recommended that the bluff slope be regraded to a stable slope angle. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 7: The stability of the bluff slope underlying the concrete slab fill within Section 11, located at 9180 S. 5th Avenue, was characterized by Profile No. 11.

Figure III -15

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 10



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

The deterministic slope stability model was used within Section 7 to assess the potential for rotational sliding for conditions observed in the field. The results shown in Figure III-16 indicate that Profile No. 11 had 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 remaining nine lowest safety factors range from 0.88 to 1.03.

Of the 20 probabilistic stability analyses conducted, the lowest safety factors ranged from 0.17 to 0.83 with all of the failure surfaces, or 100 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated, 200, or 100 percent of the surfaces had safety factors less than 1.0. Based on both 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 in the top of the fill area was observed 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.

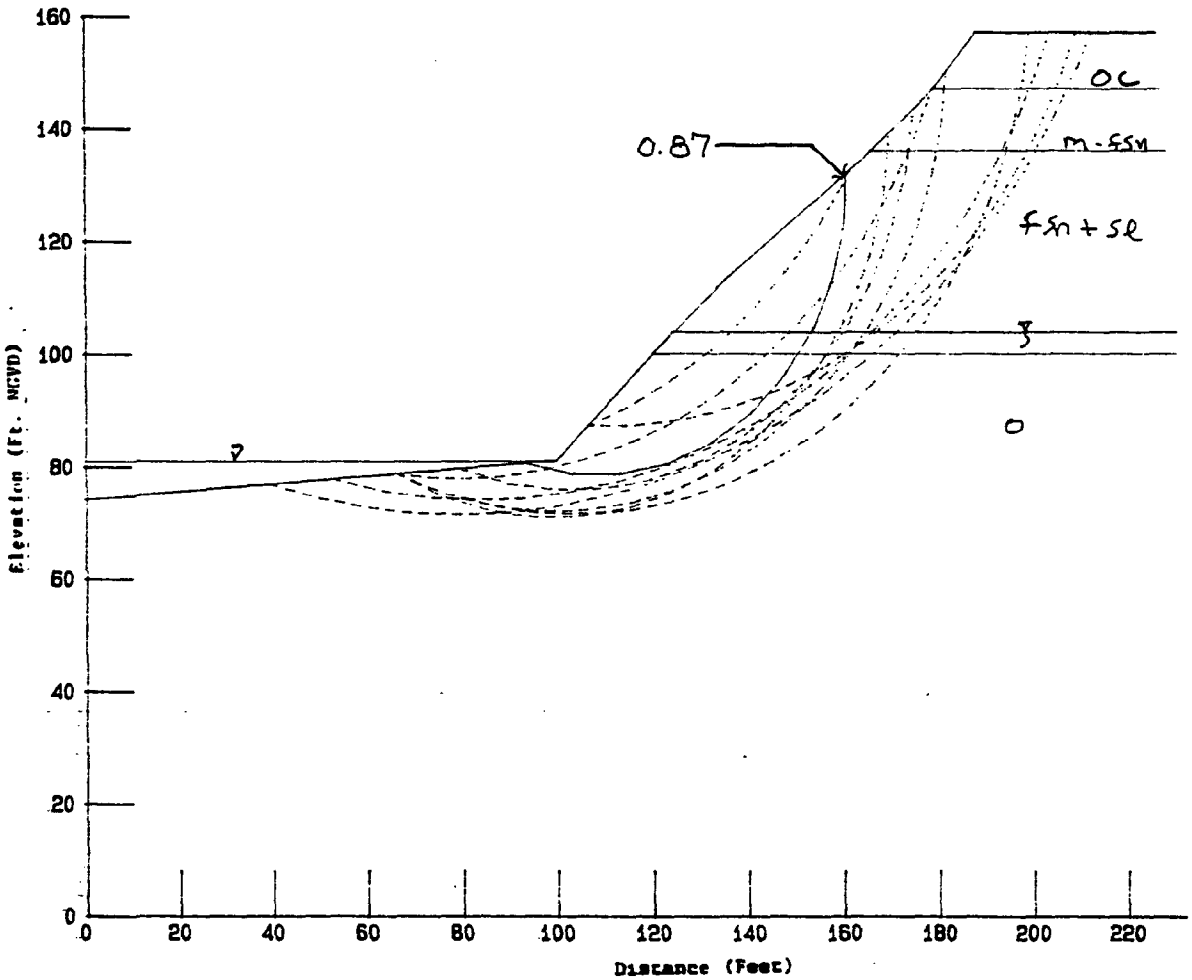
No significant toe erosion was observed in Section 7 during the field survey. Toe protection along the entire shoreline of this section was provided by a concrete slab revetment.

Several small ponds in the southern end of Section 7 were noted during the fall 1987 field survey. The contribution of the ponds to the site hydrogeology could not be determined in the field. 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.

Shore protection measures needed to fully stabilize this section cannot be determined at this time. Based on the observed and modeled bluff conditions, both bluff toe protection and slope regrading are appropriate preliminary

Figure III -16

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 11



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

protection measures. These measures will not be completely effective, however, if surface water is shown to be a factor contributing to slope instability.

Bluff Analysis Section 8: The entire shoreline of Section 8, located at 9170 S. 5th Avenue, is protected by the Oak Creek water intake bulkhead, a structure of sheet piling reinforced with a concrete wall and quarrystone. At the time of the field survey conducted during the fall of 1987, the structure appeared to be well maintained; no erosion was observed. No further measures, other than proper maintenance of the bulkhead, are required to insure the stability of the bluff in Section 8.

Bluff Analysis Section 9: The stability of the bluff slope within Section 9, located at 4301 E. Depot Road, was characterized by the use of Profile No. 12.

The results of the deterministic slope stability analysis, shown in Figure III-17, indicate that Profile No. 12 had an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.87, and was located within the lower two-thirds of the bluff slope. The remaining nine lowest safety factors ranged from 0.94 to 1.07.

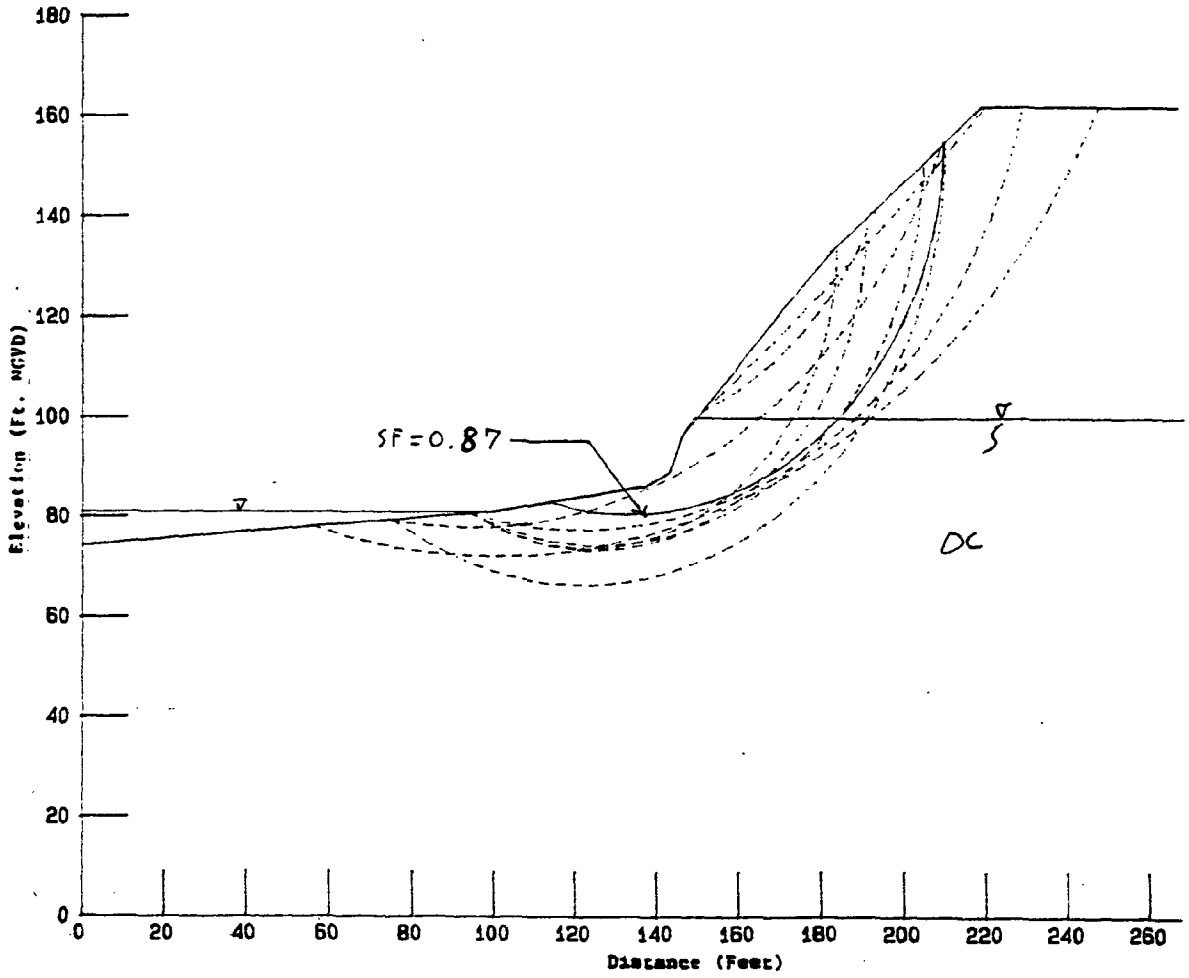
Of the 20 probabilistic stability analyses conducted, the lowest safety factors ranged from 0.21 to 0.92, with all of the failure surfaces, or 100 percent, having a safety factor < 1.0. Of the total of 200 failure surfaces evaluated, 170, or 85 percent, had safety factors of less than 1.0.

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 moderately stable bluff slope with respect to translational sliding. In the northern part of the section, solifluction and numerous planar slides were observed during the field survey conducted during the fall of 1987. Lack of vegetative cover and moderately steep slope angles were considered to be the primary influences on slope failure by translational sliding in this section.

Figure III -17

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 12



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

Significant toe erosion was observed within the entire shoreline of Section 9 during the field survey conducted in the fall of 1987. The Peter Cooper breakwater, a low, quarrystone structure located approximately 200 feet offshore, was submerged throughout this section. It was virtually ineffective in providing toe protection. Equally ineffective was the rubble revetment located in the southern third of this section.

In order to fully stabilize the bluff in this section, adequate, well maintained toe protection is needed in conjunction with bluff slope regrading.

Bluff Analysis Section 10: The stability of the bluff slope within Section 10, located at 9006 S. 5th Avenue, was characterized by the use of Profile No. 13.

The results of the deterministic slope stability analysis shown in Figure III-18 for Profile No. 13 indicate a potential 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 remaining nine lowest safety factors ranged from 0.98 to 1.06.

Of the 20 probabilistic stability analyses conducted, the lowest safety factors ranged from 0.70 to 1.14, with 14 of the failure surfaces, or 70 percent, having a safety factor less than 1.0. Of the 200 failure surfaces evaluated, 91, or 45 percent of the surfaces had safety factors less than 1.0.

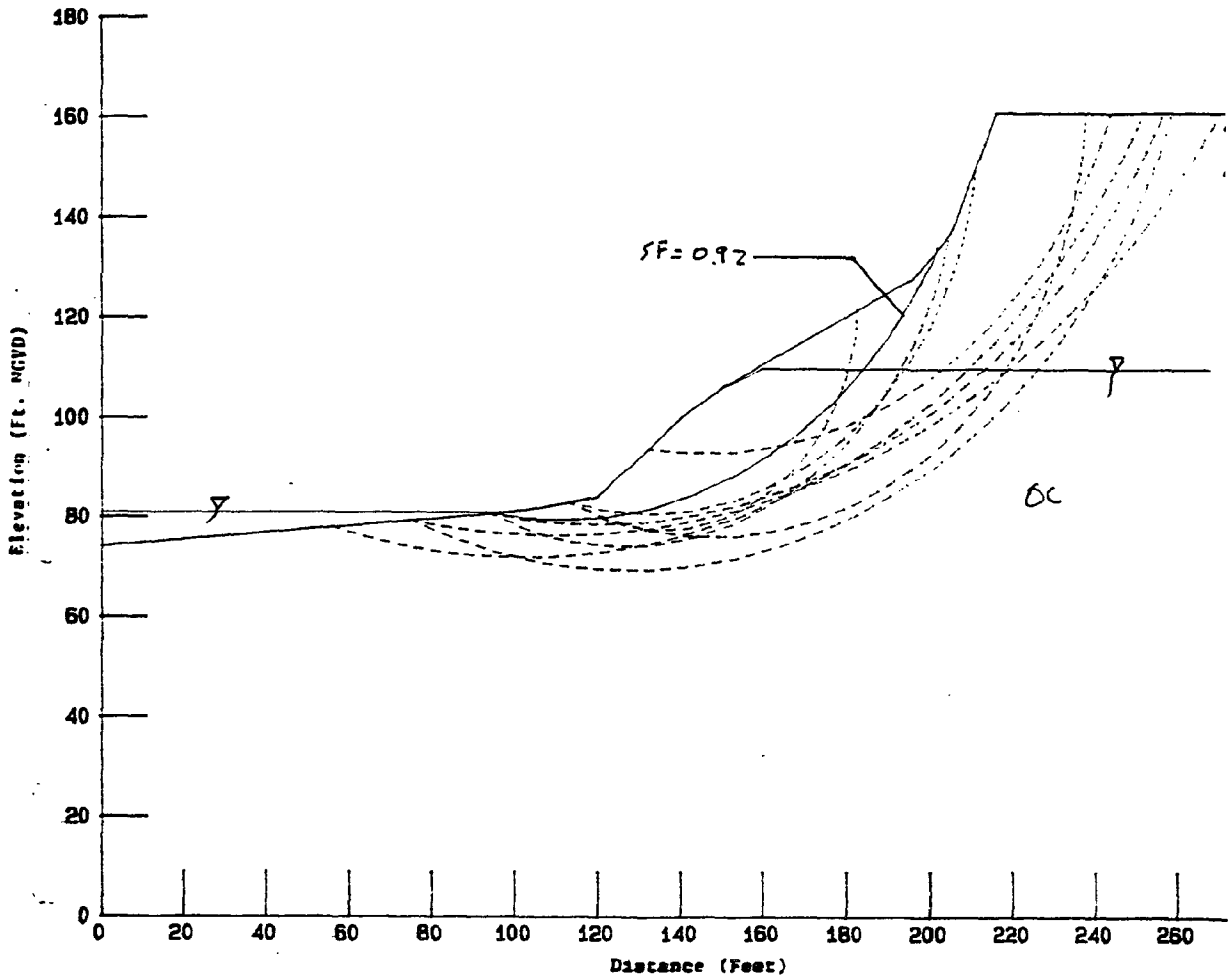
Based on both the deterministic and probabilistic slope stability analyses and on the observed bluff conditions, Section 10 was considered to have a marginal bluff slope with respect to rotational sliding.

During the field surveys conducted in the fall of 1987, 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 reddish slag.

Section 10 was 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

Figure III-18

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 13



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

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 surveys was considered a significant threat to bluff stability. The presence of the Peter Cooper breakwater, a low, partially-submerged, quarrystone structure located approximately 200 feet offshore was ineffective in preventing shoreline erosion in the southern end of this section. Where the structure is emergent in the northern end, it was marginally effective.

Action necessary to fully stabilize the bluff in this section is two-fold. First, partial regrading concentrating on the base of the bluff slope should be provided. Second, bluff toe protection should be provided and properly maintained.

Bluff Analysis Section 11: The stability of the bluff slope within Section 11, which extends from 9006 S. 5th Avenue to 8740 S. 5th Avenue, was characterized by the use of Profile No. 14.

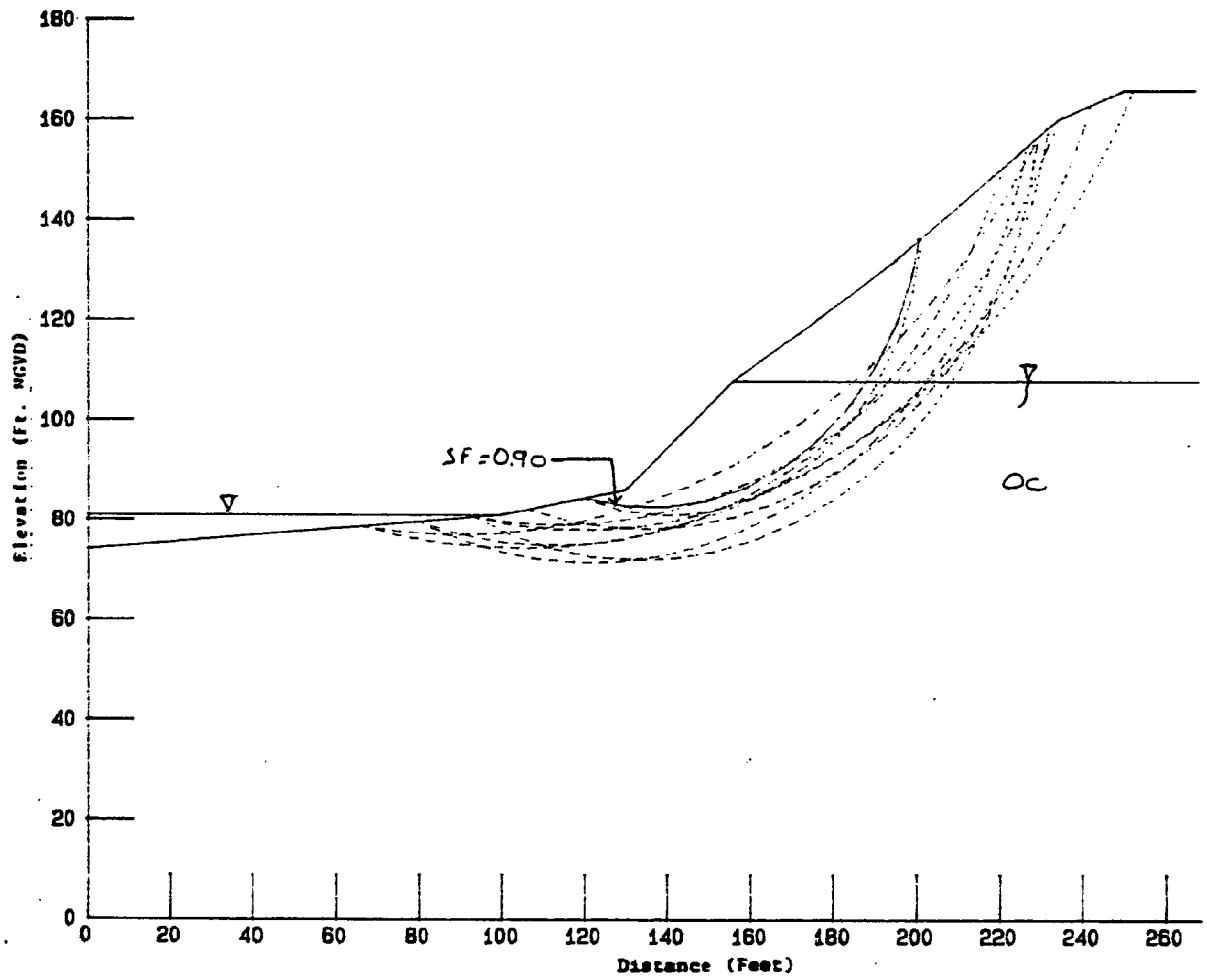
The results of the deterministic slope stability analysis, shown in Figure III-19, indicate that Profile No. 14 had an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.90, and was located within the lower two-thirds of the bluff slope. The remaining nine lowest safety factors ranged from 0.92 to 1.02.

Of the 20 probabilistic stability analyses conducted, the lowest safety factors ranged from 0.38 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, 157 surfaces, or 78 percent, had safety factors of less than 1.0.

During the field survey conducted during the fall of 1987, recent slope failures were observed in the southern end of this section. Shallow, planar slides were observed through the section.

Figure III -19

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 14



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

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 is partially vegetative with a slope angle of 40 degrees. The balance of the bluff is moderately well-vegetated and has a slope angle of 25 degrees. The potential for the translational sliding is, therefore, greater in the lower portion of the slope, as field observations have verified.

Bluff toe erosion was observed in the entire shoreline of Section 10 and was identified as a contributing factor in bluff slope instability. No structures were located in this section as of 1986.

In order to attain maximum bluff stability in this section, bluff slope regrading and toe protection are needed.

Bluff Analysis Section 12: The entire shoreline of Section 12, located at the South Shore Treatment Plant, is protected by the treatment plant bulkhead, a structure of double row steel sheet piling, with armor stone filling the splash apron. The natural bluff has been graded to a stable slope. No erosion of the bluff was observed during the field survey conducted in the fall of 1987.

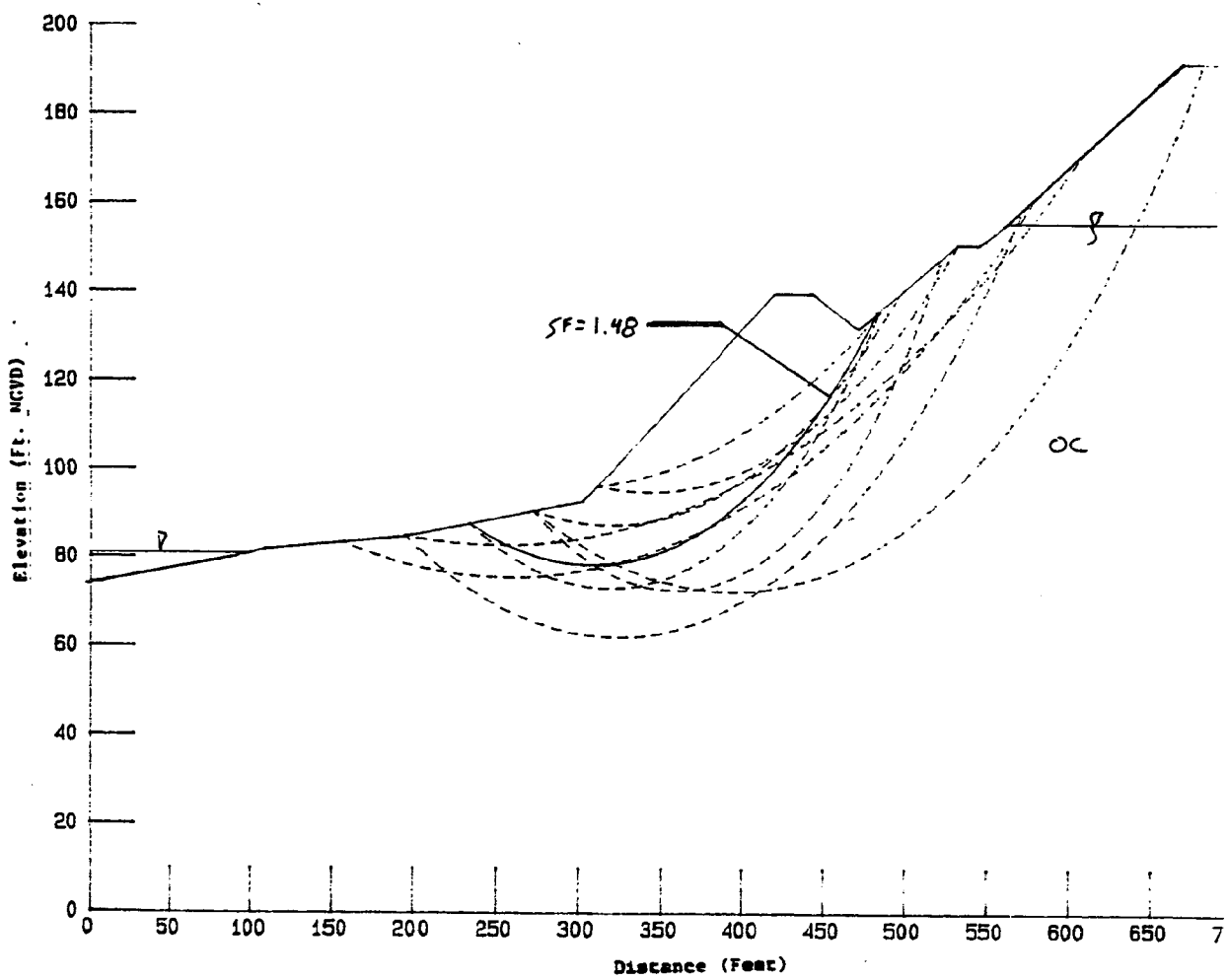
Although the bulkhead was reported to be in relatively good condition during a 1988 field inspection, evidence of overtopping was noted. Inside the facility, relocation of gravel and erosion at the base of the wall were observed. It is recommended that the bulkhead be examined by site engineers and that corrective action be taken as needed to properly maintain the structure.

Bluff Analysis Section 13: The stability of the bluff slope within Section 13, located at 8400 S. 5th Avenue, was characterized by the use of Profile No. 15.

The results of the deterministic slope stability analysis, shown in Figure III-20, indicate that Profile No. 15 had a stable slope with respect to

Figure III -20

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 15



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

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 remaining nine lowest safety factors ranged from 1.50 to 1.75.

Based on the deterministic slope stability analyses and on the observed bluff conditions, Section 13 was considered to have a stable bluff slope with respect to rotational sliding.

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 South Shore wastewater treatment facility bulkhead. No shore protection structures were located in Section 13 as of 1986.

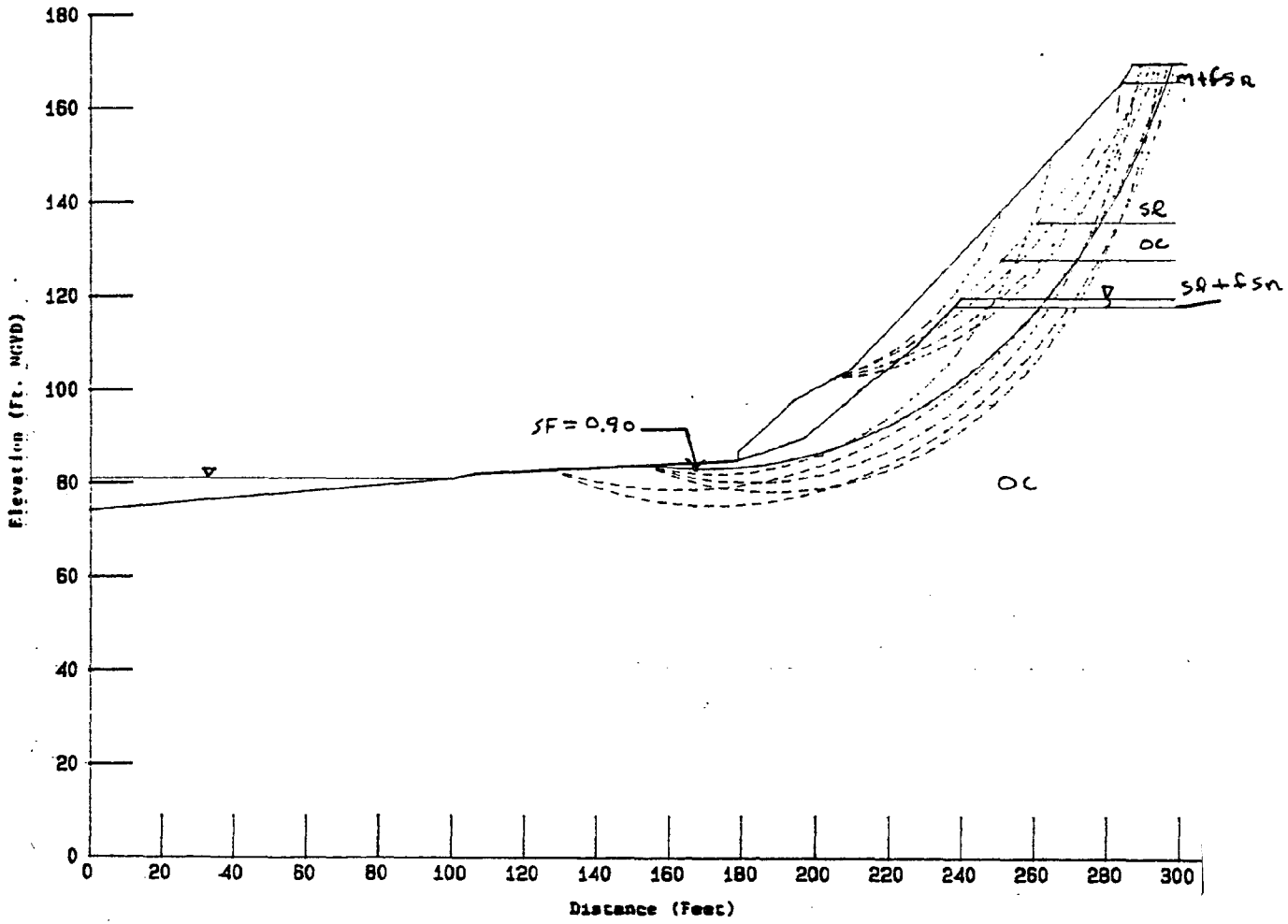
To ensure continued bluff stability in this section, the small number of exposed areas on the bluff slope should be revegetated.

Bluff Analysis Section 14: The stability of the bluff slope in Section 14, which extends from 3817 to 3509 3rd Avenue, was characterized by the use of Profile No. 16 and Profile No. 17.

The results of the deterministic slope stability analyses, are shown in Figures III-21 and III-22, 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 remaining 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 remaining nine lowest safety factors ranged from 0.75 to 0.88. These results indicate an unstable bluff slope with respect to rotational sliding.

Figure III -21

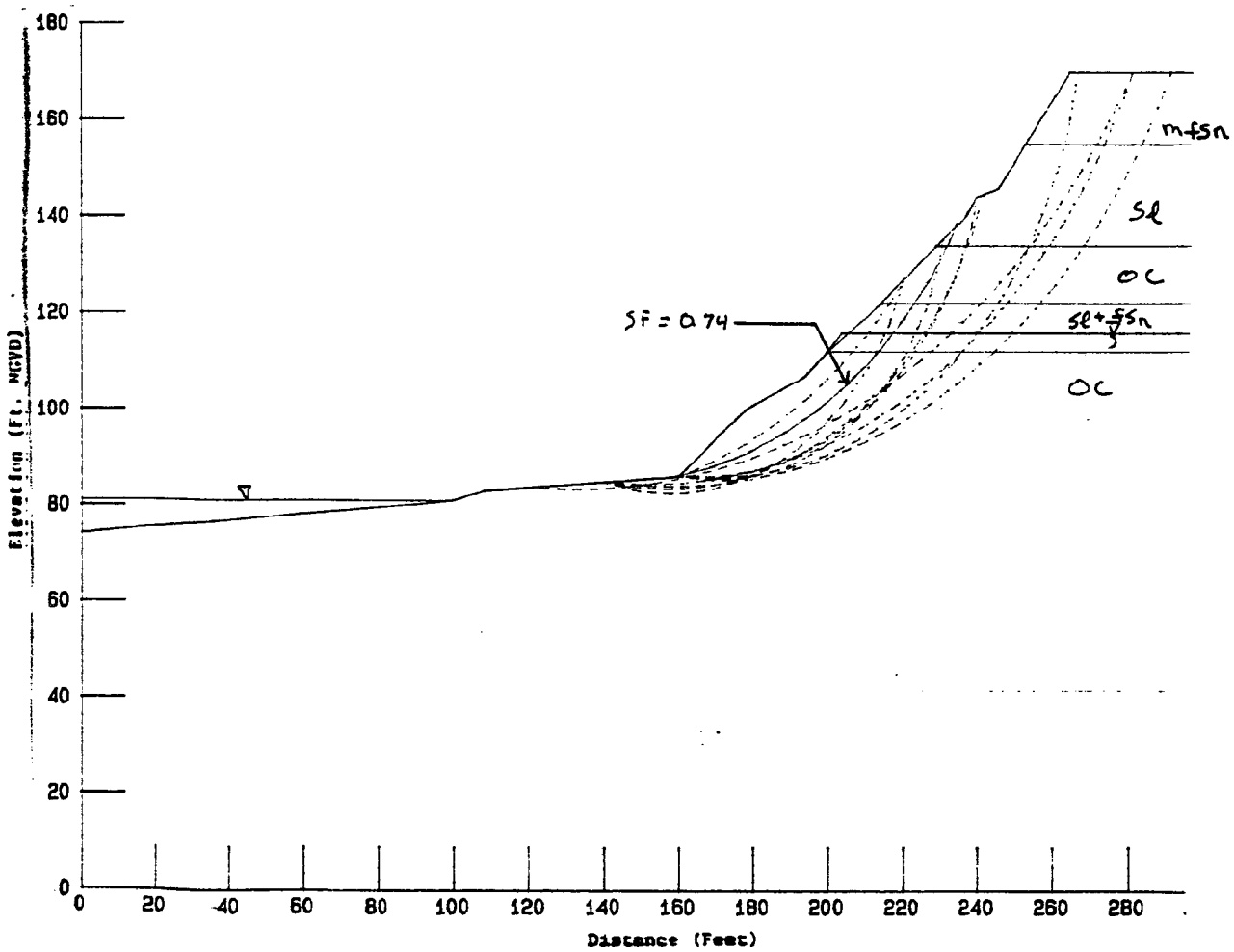
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 16



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

Figure III -22

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 17



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

Of the 20 probabilistic stability analyses conducted for Profile No. 16, the lowest safety factors ranged from 0.36 to 0.84, or 100 percent of the critical surfaces had safety factors less than 1.0. Of the total of 200 failure surfaces evaluated at Profile No. 16, 100 percent had safety factors less than 1.0. The probabilistic stability analyses was not conducted for Profile No. 17.

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 small planar slides in the upper portion of the natural bluff slope was noted. Here, the slope angle was greater than 40 degrees and the vegetation was patchy. 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 contributing to the instability of the bluff slope was observed along the entire shoreline of Section 14 during the fall of 1987 field surveys. No shore protection structures were located within this section as of 1987.

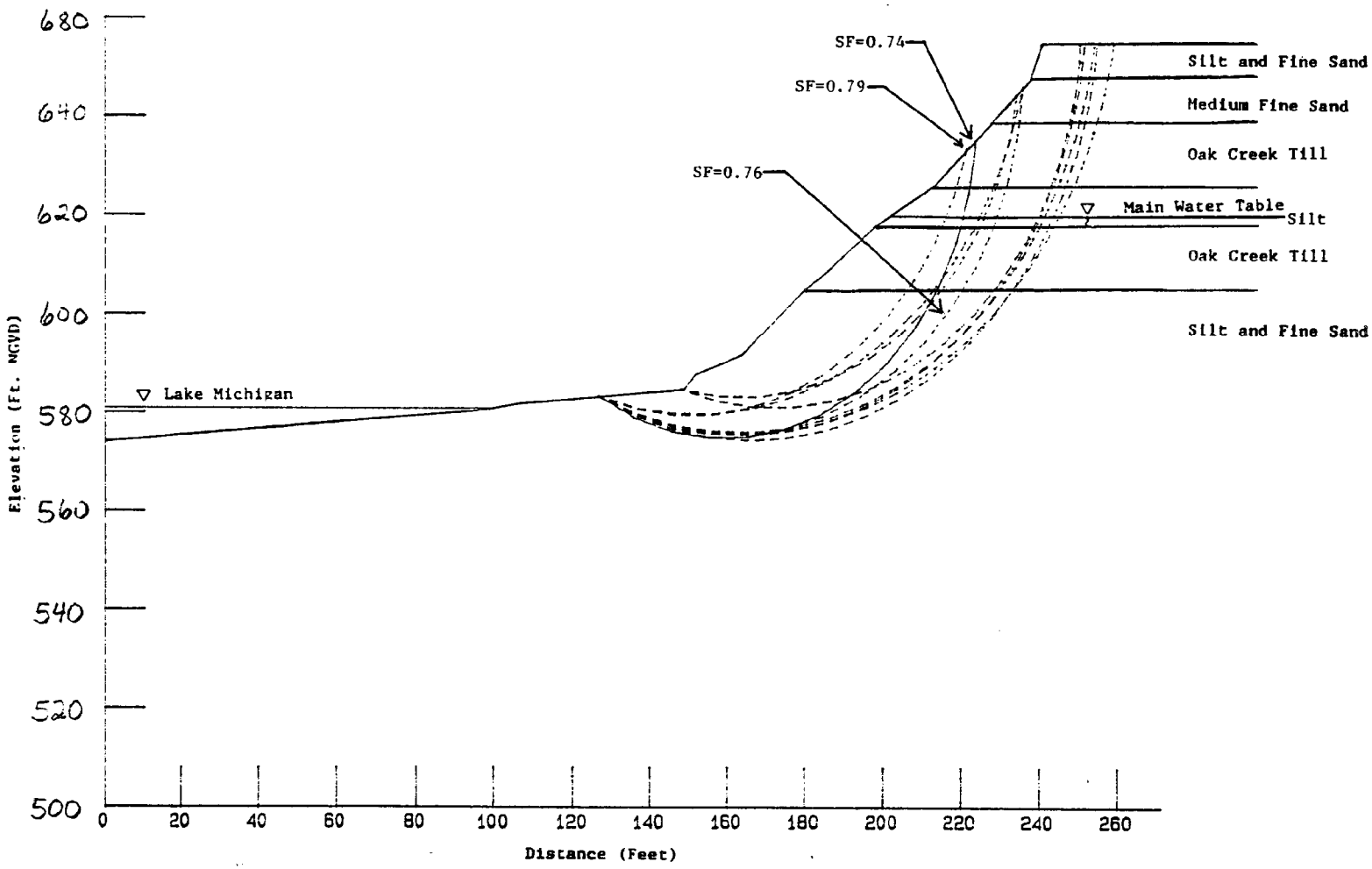
The unplanned dumping of concrete rubble fill at the site of Profile No. 16 has had a modest, local effect on improving bluff stability. However, the fill at the base of the bluff is susceptible to toe erosion. In order to fully stabilize the bluff in this section, bluff toe protection and bluff slope regrading are needed.

Bluff Analysis Section 15: The stability of the bluff slope within Section 15, which extends from 235 Lakeview Avenue to 3303 Marina Road, was characterized by the use of Profile 18 in the northern half, and by field observations in the southern half.

The results of the deterministic slope stability analysis, shown in Figure III-23, indicate that Profile No. 18 had an unstable bluff slope with respect

Figure III -23

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 18



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

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 remaining nine lowest safety factors ranged from 0.76 to 0.83.

Of the 20 probabilistic stability analyses conducted for this profile site, the lowest safety factors ranged from 0.70 to 1.10, with 19 of the critical surfaces, or 95 percent, having a safety factor less than 1.0. Of the total of 200 failure surfaces evaluated, 188, or 94 percent of the surfaces, had safety factors 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 half of this section. The southern half, however, was covered with concrete waste which had been dumped over the top of the bluff. This action had been effective in stabilizing that portion of the bluff.

Based on both deterministic and probabilistic slope stability analyses, and on observed bluff conditions, the northern half of Section 15 was considered to have an unstable slope with respect to rotational sliding. The southern half of Section 15 was stable.

Overall, the northern half of Section 15 was considered to have an unstable slope with respect to translational sliding. Just north of the fill site, slope angles were greater than 40 degrees, and the slope was barren of vegetation. Further north, behind the yacht club launch, the slope was partially vegetated and had been graded to a more gentle angle, decreasing the potential for failure. The southern half of Section 15 was considered to have a stable slope with respect to translational sliding.

No significant bluff toe erosion was observed during the 1987 field surveys. A small sand beach had accumulated north of the concrete fill. In the northern end of the section, the yacht club boat launch protected the bluff from toe erosion. The structure itself, however, was suffering from erosion on the front face and undermining.

Shore protection measures needed to fully stabilize Section 15 focus on the northern half of the section. Bluff toe protection and bluff slope regrading are necessary in the currently unprotected zone. Maintenance of the structure built by the yacht club is needed to ensure continued 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, was characterized by the use of Profile No. 19.

The results of the deterministic slope stability analysis, shown in Figure III-24, indicate that Profile No. 19 had 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 is located within the middle third of the bluff slope. The remaining nine lowest safety factors ranged from 1.14 to 1.16.

Based on the deterministic slope stability analysis and on the observed bluff conditions, Section 16 was considered to have a stable bluff slope with respect to rotational sliding.

Section 16 as also considered to have a stable bluff slope with respect to translational sliding. The bluff slope was generally well-vegetated with only small patches of exposed soil.

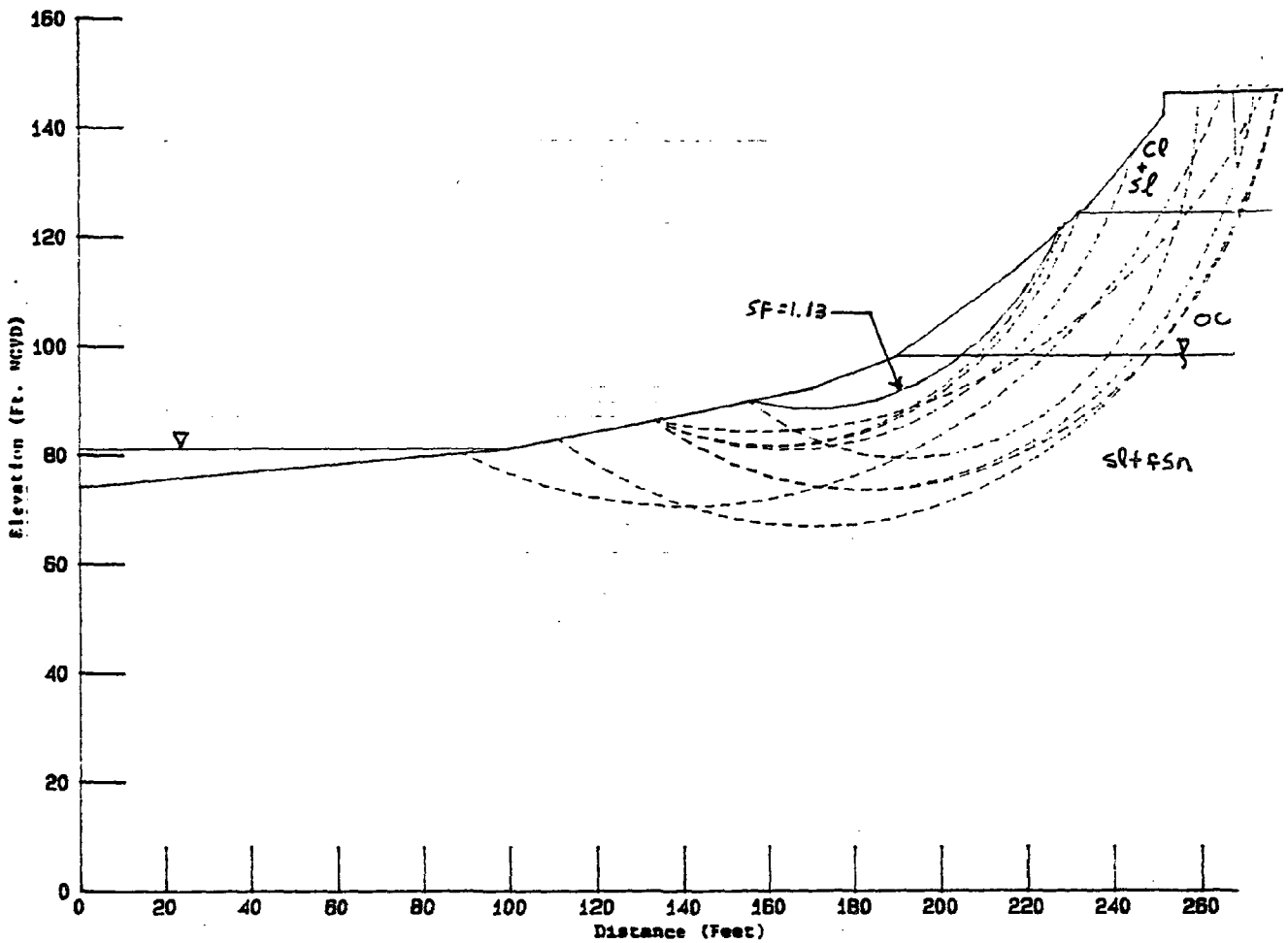
The toe of the bluff was protected by a wide sand beach which had accumulated to the north of the yacht club boat launch. The beach thins in the northern end of the section exposing the bluff to minor toe erosion. No shore protection structures were located in Section 16 as of 1986.

To ensure continued bluff stability in this section, the disturbed soil areas on the bluff slope should be revegetated.

Bluff Analysis Section 17: The stability of the bluff slope within Section 17, located at 3333 5th Avenue, was characterized by the use of Profile No. 20.

Figure III -24

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 19



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

The results of the deterministic slope stability analysis, shown in Figure III-25, indicated Profile 20 had 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 remaining nine lowest safety factors ranged from 0.80 to 0.92.

Based on the deterministic slope stability analyses and on observed bluff conditions, Section 17 was considered to have an unstable bluff slope with respect to rotational sliding.

Section 17 was also considered to have an unstable bluff slope with respect to translational sliding. Near vertical slope angles in the upper portion of the bluff and absence of vegetative cover on the entire bluff slope throughout the section increase the potential for translational sliding. Evidence of recent shallow falls and slides was noted during the field surveys 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 surveys. No shore protection structures were located in this section as of 1986.

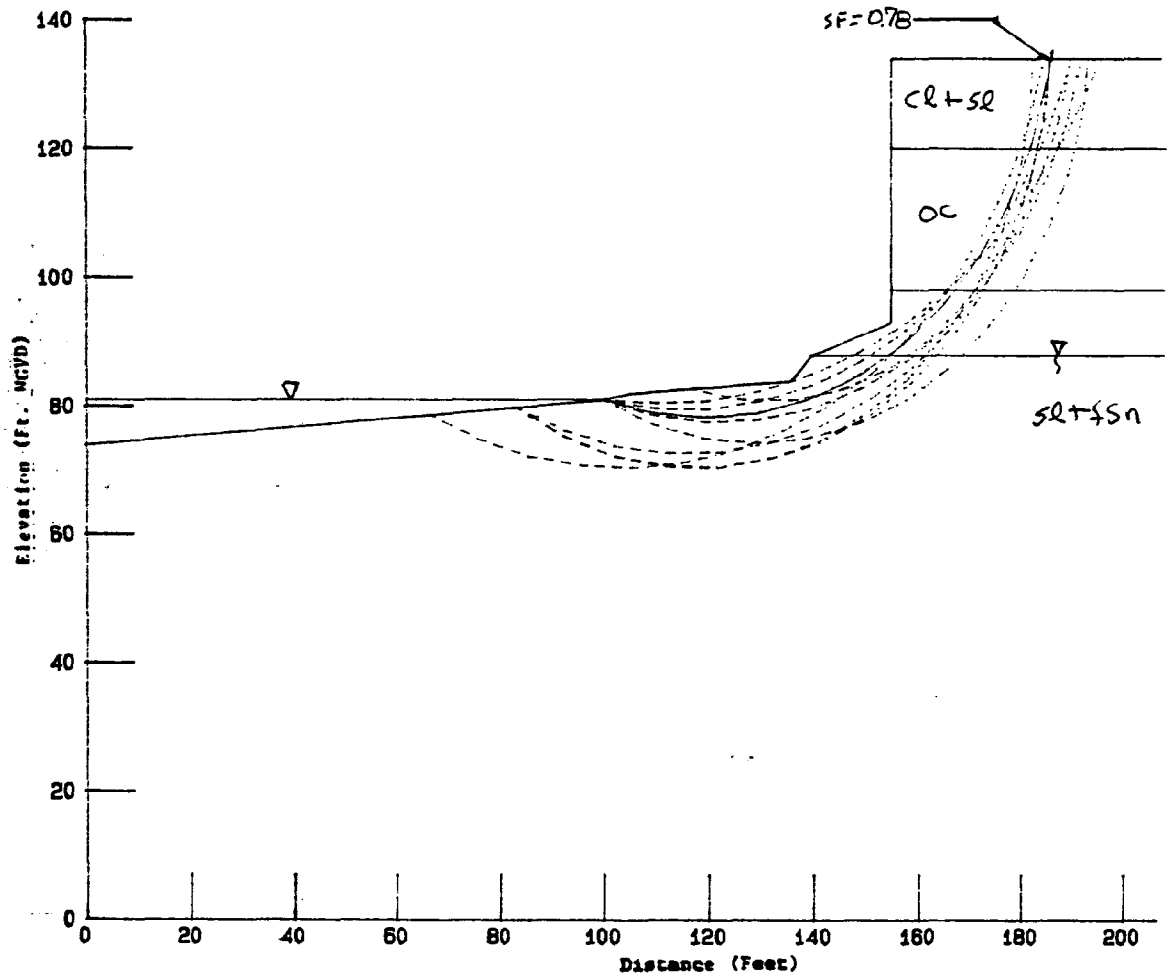
In order to fully stabilize the bluff in this section, both bluff slope regrading and bluff toe protection are necessary.

Bluff Analysis Section 18: The stability of the bluff slope within Section 18, located at the Marshall Avenue South Milwaukee Water Utility, was characterized by the use of Profile No. 21 and Profile No. 22. The entire slope in this section had been covered with a layer of rubble fill.

The results of the deterministic slope stability analyses are shown in Figure III-26 and III-27, and represent Profile Nos. 21 and 22, respectively. 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 nine remaining lowest safety factors ranged from 1.43 to 1.63. 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

Figure III -25

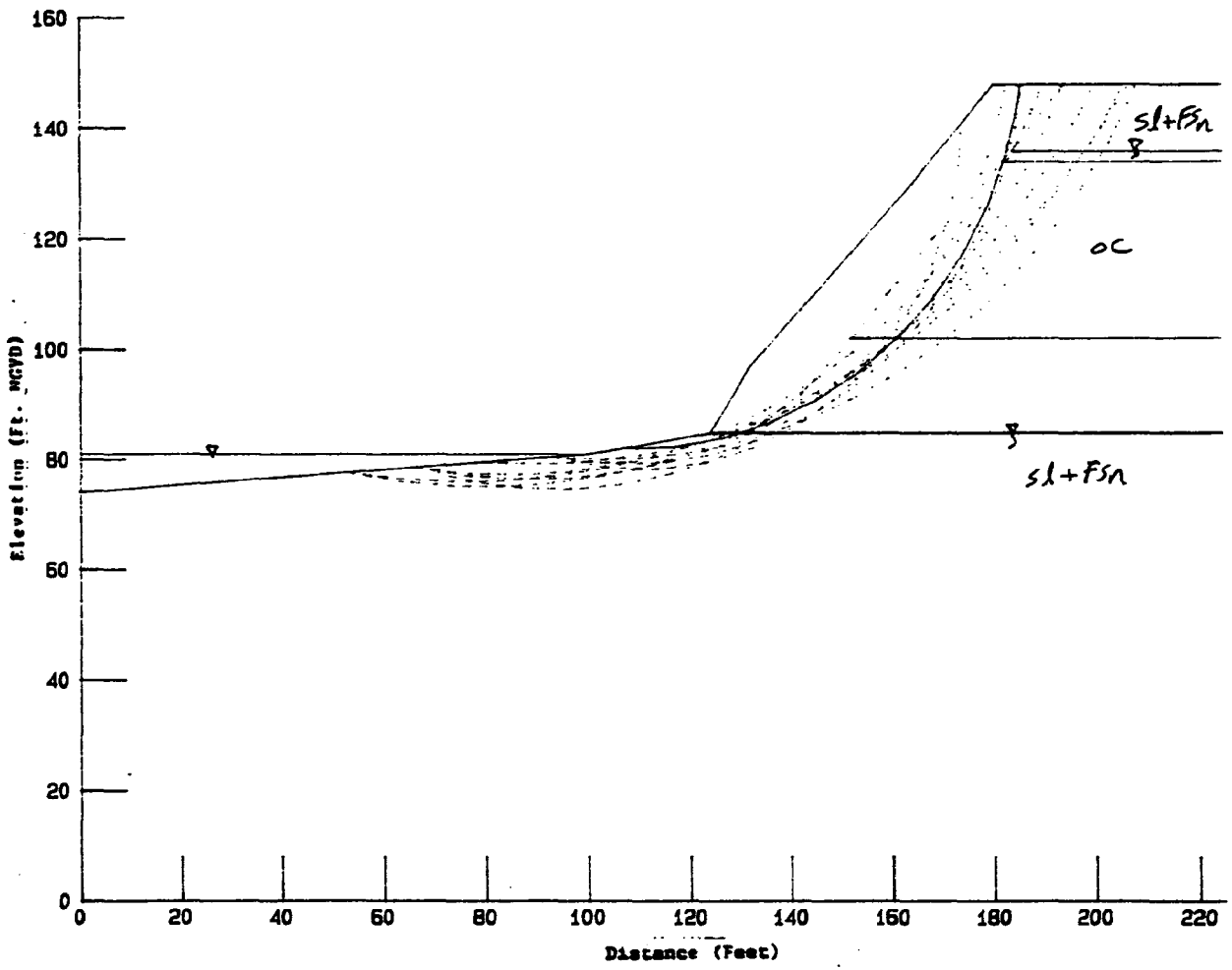
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 20



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

Figure III -27

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 22



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

of 0.87, and included the entire bluff slope. The nine remaining lowest safety factors ranged from 0.91 to 1.04. The results indicate the bluff slope of the recently slumped area was stable with respect to rotational sliding. The bluff slope in the unfailed area, which represents the majority of this section, showed a potential threat of failure with respect to rotational sliding.

A probabilistic slope stability analysis, under which bluff conditions at the profile site were varied, was conducted for Profile No. 22. Of the 20 probabilistic slope stability analyses conducted, the lowest safety factors ranged from 0.51 to 0.84, with 100 percent of the failure surfaces having safety factors less than 1.0. Of the total of 200 failure surfaces evaluated, 195, or 97 percent of the surfaces, had safety factors 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 is greatest in the unvegetated previously failed areas. The balance of the section is moderately vegetated, with several areas of exposed soil.

Bluff toe erosion is a major factor contributing to failure in the majority end of Section 18. In the northernmost end, a sand beach protects the toe of the bluff from wave and ice action. No shore protection structures were located in this section as of 1986.

Bluff toe protection coupled with bluff slope regrading are appropriate shore protection measures to fully stabilize the bluff in Section 18.

Bluff Analysis Section 19: The stability of the bluff slope within Section 19, located at the South Milwaukee Yacht Club and the southern end of Grant Park, was characterized by observed bluff conditions.

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 outlet of Oak Creek was located just north of the breakwater. A rubble mound groin at the southern end of Grant Park maintained the position of the outlet on the downdrift side of the structure, and trapped a wide sand beach on the updrift side. The bluff slope was well protected from toe erosion by the wide beach. Overall, the bluff slope was considered stable with respect to rotational and translational sliding. The bluff face was well vegetated and had experienced only minor soil creep.

Proper maintenance of the existing shore protection structures is necessary to ensure continued bluff stability in Section 19.

Bluff Analysis Section 20: The stability of the bluff slope within Section 20, located at Grant Park, was characterized by the use of Profile No. 23.

The results of the deterministic slope stability analysis shown in Figure III-28, indicate that Profile No. 24 had an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.71, and included the entire bluff slope. The remaining nine lowest safety factors ranged from 0.71 to 0.78.

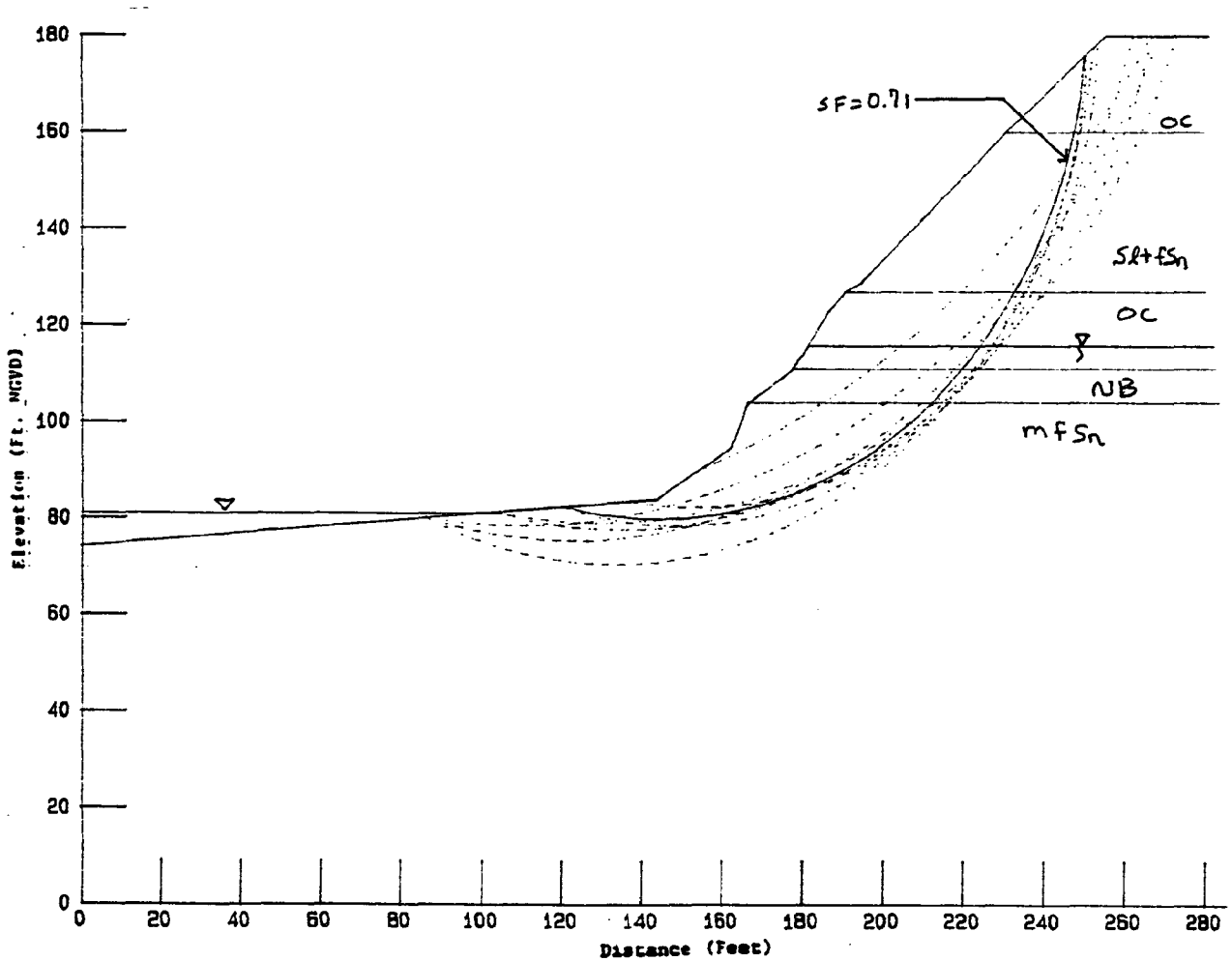
Based on the deterministic slope stability analyses and on observed bluff conditions, Section 20 was considered to have an unstable bluff with respect to rotational sliding.

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, however, 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 timber groin located in the middle of Section 20 had trapped a narrow sand beach, but this did not decrease the effect of wave action on toe erosion.

Figure III -28

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 23



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

In order to fully stabilize the bluff in this section, bluff toe protection is needed. In addition, the lower portion of the bluff slope should be regraded.

Bluff Analysis Section 21: The stability of the bluff slope within Section 21, located at Grant Park, was characterized by the use of Profile No. 24.

The results of the deterministic slope stability analyses shown in Figure III-29, for Profile No. 24, indicate a potential threat for bluff slope failure with respect to rotational sliding. The lowest failure surface calculated had a safety factor of 0.92, and included the entire bluff slope. The remaining nine lowest safety factors ranged from 0.92 to 1.03.

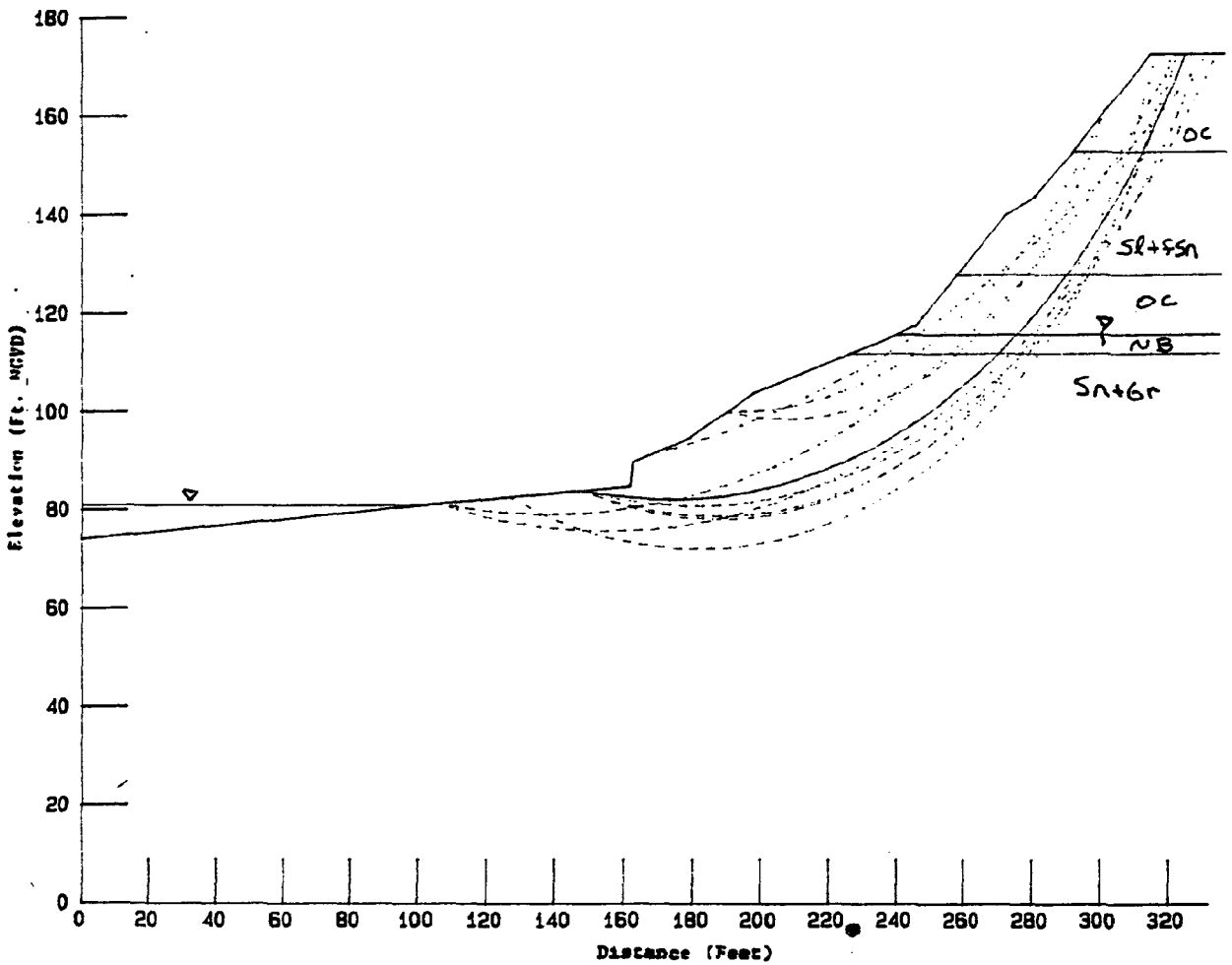
A probabilistic slope stability analysis, under which the bluff conditions at the profile site were varied, was conducted to help characterize the stability of the bluff within the entire section. Of the 20 probabilistic stability analyses conducted, the lowest safety factors ranged from 0.82 to 1.07, with 15, or 75 percent, having a safety factor of less than 1.0. Of the total of 200 failure surfaces evaluated, 98 surfaces, or 49 percent, had safety factors less than 1.0. Groundwater seeps observed during the field survey conducted during the fall of 1987 were considered to be a factor in initiating failure by rational movement. 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 growth covering 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 timber groin in the southern portion of the section had trapped a sand beach approximately 100 feet wide on the updrift side of the structure. This beach protected the toe of the bluff from erosion by wave action. Toward the northern end, the beach thinned to approximately 25 feet--the toe of the bluff was no longer adequately protected and the stability of the bluff was threatened.

Figure III -29

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 24



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

In order to fully stabilize the bluff in this section, additional bluff toe protection is needed. The current structure should also be properly maintained. Finally, a groundwater drainage system should be emplaced to reduce the potential for failure by rotational sliding.

Bluff Analysis Section 22: The stability of the bluff slope within Section 22, located in Grant Park, was characterized by the use of Profile No. 25.

The results of the deterministic slope stability analysis, shown in Figure III-30, indicate that Profile No. 25 had 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 remaining nine lowest safety factors ranged from 0.85 to 0.98.

Of the 20 probabilistic slope stability analyses conducted, the lowest safety factors ranged from 0.65 to 1.06 with 16, or 80 percent, having a safety factor of less than 1.0. Of the total of 200 failure surfaces evaluated at Profile No. 25, 129, or 64 percent of the surfaces, had safety factors less than 1.0.

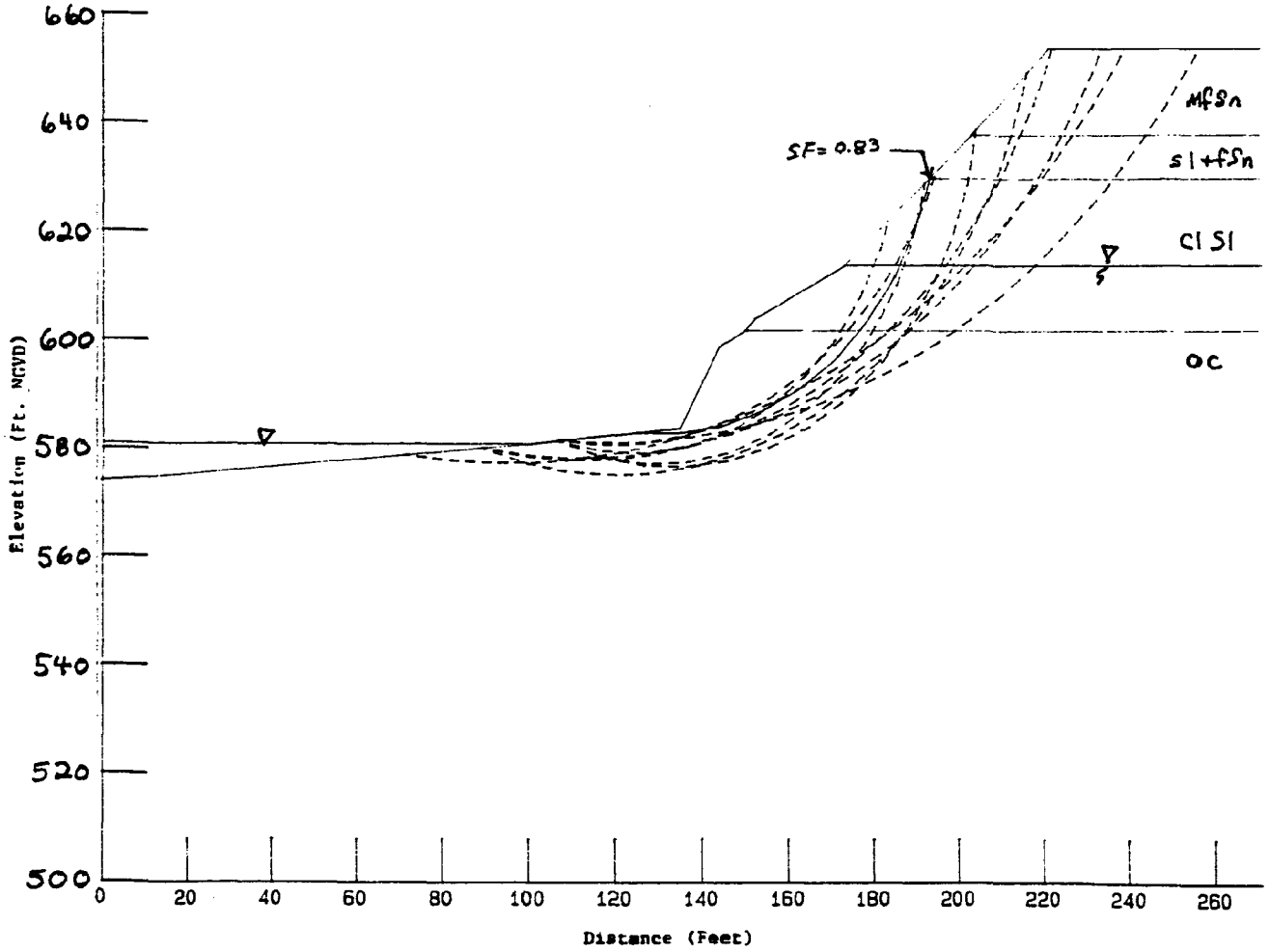
Based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 22 was considered to have an unstable slope with respect to rotational sliding.

Overall, Section 22 was considered to have a marginal bluff slope with respect to translational sliding. In the southern end of the section, the base of the bluff was barren and the upper portion was well-vegetated. The entire bluff face was without vegetation in the northern end. Numerous groundwater seeps were observed at the base of the bluff during the field survey conducted during the fall of 1987. The potential for failure by translational motion was considered to be the greatest in areas where vegetation was sparse and where groundwater was emergent.

Bluff toe erosion was observed throughout Section 22 during the 1987 field survey. The effect on bluff stability varied. In the southern end, the stability of the bluff did not appear to be threatened. However, in the northern

Figure III -30

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 25



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

end, toe erosion was considered to have a significant impact on bluff stability. No shore protection structures were located in this section as of 1986.

To fully stabilize the bluff in Section 22, both bluff slope regrading of the lower portion of the bluff and bluff toe protection are needed.

Bluff Analysis Section 23: The stability of the bluff slope in Section 23, located in Grant Park, was characterized by the use of Profile No. 26.

The results of the deterministic slope stability analysis, shown in Figure III-31, indicate that Profile No. 26 had 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 remaining 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. Of the 20 probabilistic stability analyses conducted, the lowest safety factors ranged from 0.29 to 0.94. Of the total of 200 failure surfaces evaluated, 180,or 90 percent, had safety factors less than 1.0.

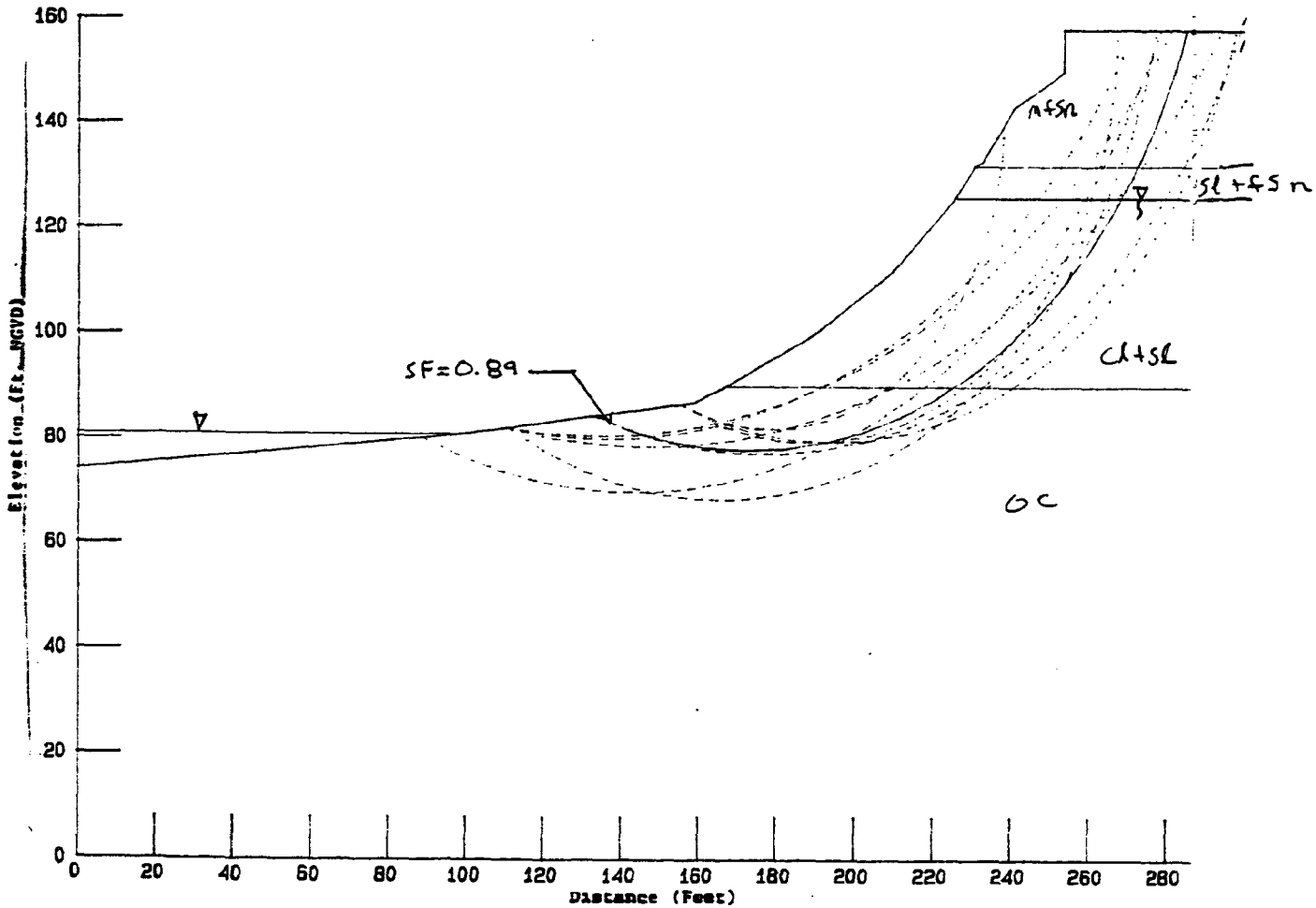
Based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 23 was considered to have an unstable slope with respect to rotational sliding.

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, many active seeps were observed in the middle portion of the bluff face. Evidence of sapping, minor slumping, and planar sliding were also noted. The entire bluff face in this section was unvegetated. Slope failure in Section 23 was affected by both the lack of vegetation and the position of the water table.

Bluff toe erosion, observed throughout the section, also contributed to bluff instability. As of 1987, no shore protection structures were located in Section 23.

Figure III-31

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 26



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

Shore protection measures are required to fully stabilize the bluff in this section. Both bluff toe protection and bluff slope regrading are necessary to accomplish this task.

Bluff Analysis Section 24: The stability of the bluff slope within Section 24, located in Grant Park, was characterized by the use of Profile No. 27.

The results of the deterministic slope stability analysis, shown in Figure III-32, 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 remaining 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. Of the 20 probabilistic stability analyses conducted, the lowest safety factors ranged from 0.71 to 1.19 with 14, or 70 percent, having a safety factor of less than 1.0. Of the total of 200 failure surfaces evaluated, 65, or 32 percent, had safety factors 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.

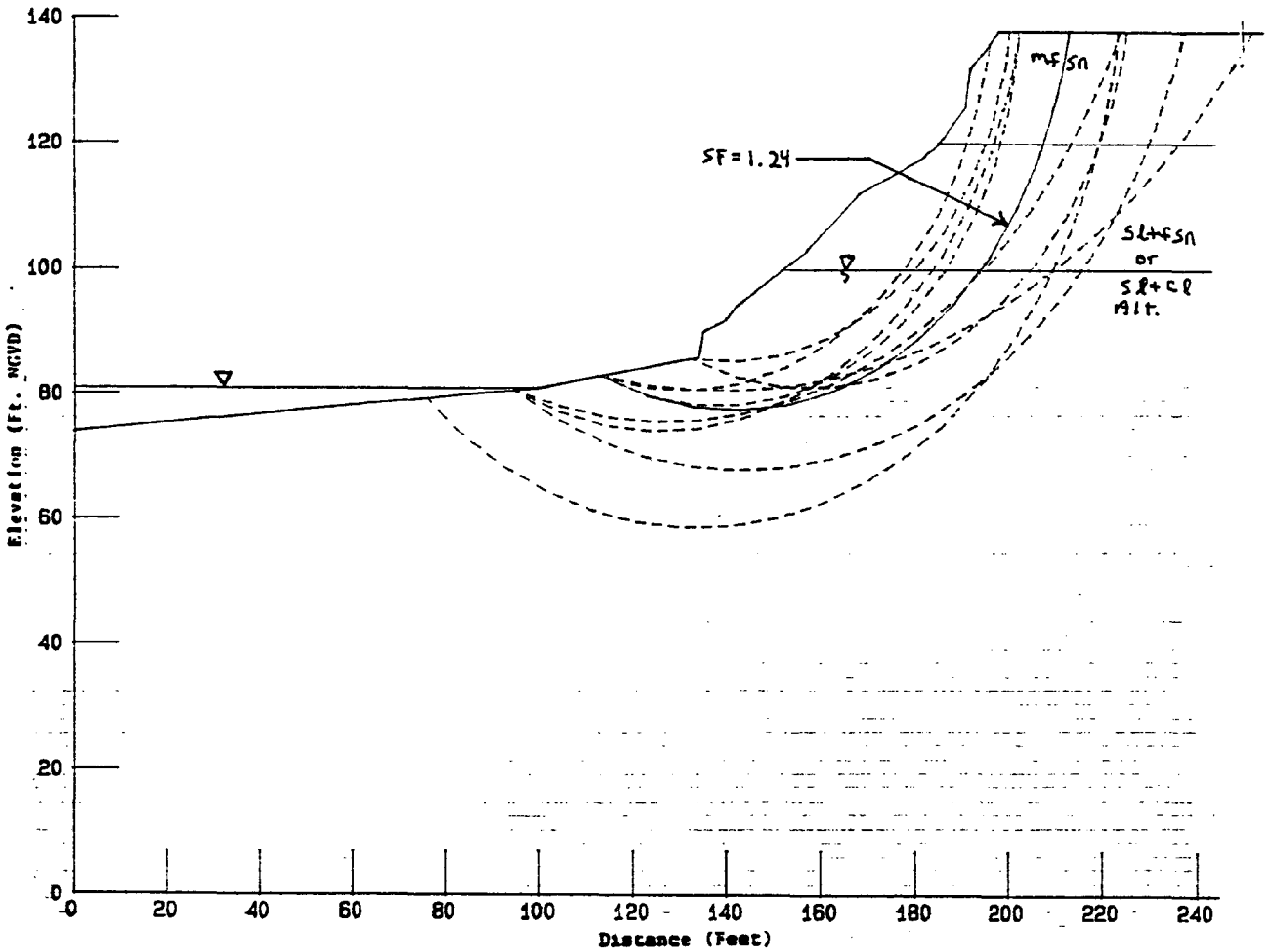
Section 24 was considered to have an unstable slope with respect to translational sliding. This is due, in part, to the steep angle of the bluff slope and the lack of vegetative cover on the bluff face.

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. A groin in the southern end of the section had trapped a small beach on its updrift side. In a short distance, however, the beach narrowed to a width of approximately 20 feet and provided very little protection against ice and wave action.

Regrading of the lower bluff and bluff toe protection are necessary to fully stabilize the bluff in Section 24.

Figure III -32

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 27



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

Bluff Analysis Section 25: The stability of the bluff slope in Section 25, located in Grant Park, was characterized by the use of Profile No. 28.

The results of the deterministic slope stability analysis, shown in Figure III-33, indicate that Profile No. 28 had 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.

Based on the deterministic slope stability analysis and on the observed bluff conditions, Section 25 was considered to have an unstable slope with respect to rotational sliding.

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. The bluff face was sparsely vegetated. During the field survey conducted in the fall of 1987, evidence of solifluction and shallow slides were observed.

Although bluff toe erosion was observed in Section 25 during the 1987 field surveys, it was not considered to be a threat to the overall stability of the bluff slope. No shore protection structures were present in this section as of 1987.

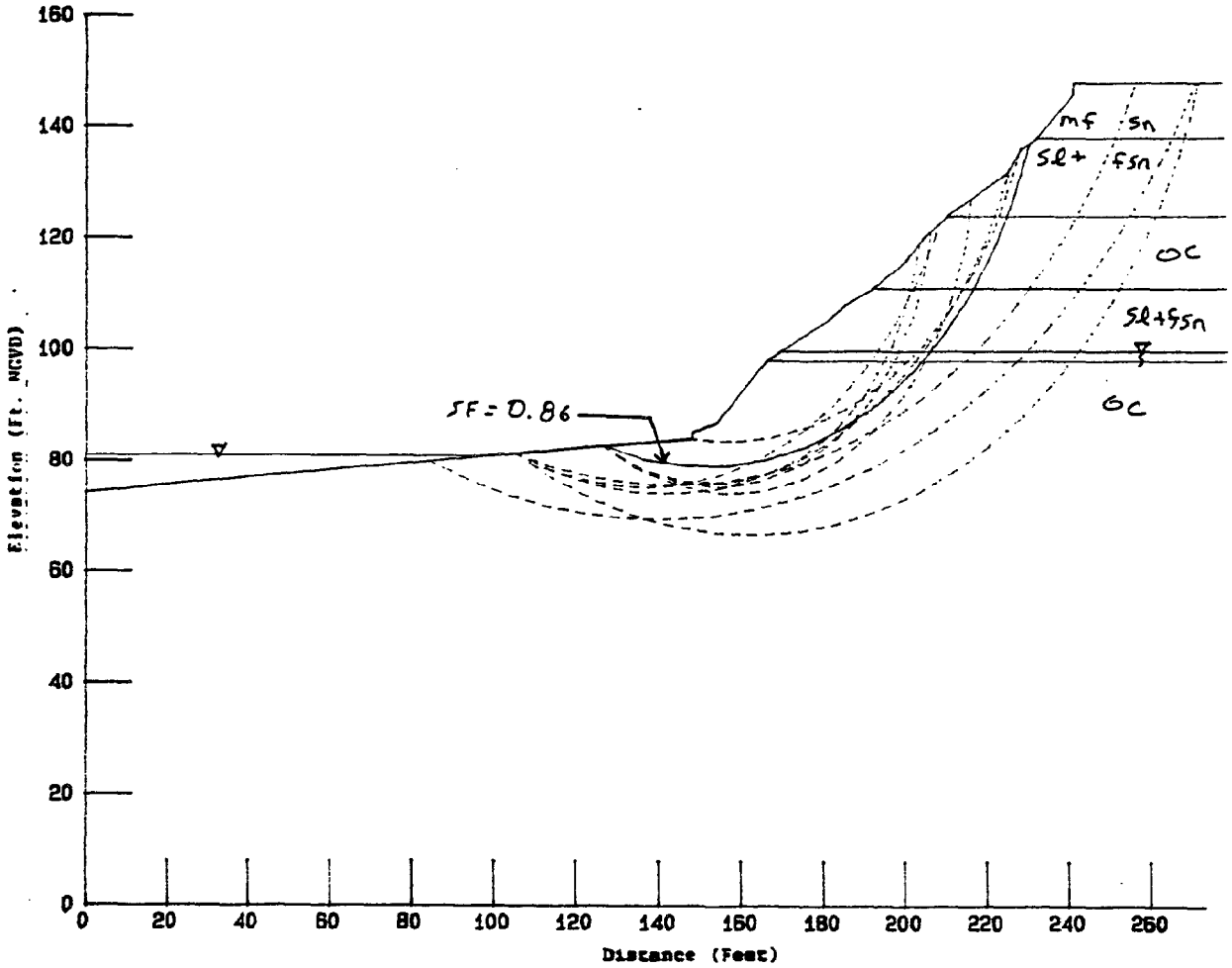
Shore protection measures are needed to fully stabilize the bluff in Section 25. Bluff slope regrading and bluff toe protection are suggested.

Bluff Analysis Section 26: The stability of the bluff slope in Section 26, which extends from College Avenue to Warnimont Park, was characterized by the use of Profile No. 29.

The results of the deterministic slope stability analysis, shown in Figure III-34, indicate that Profile No. 29 had an unstable slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.73 and included the entire bluff slope. The remaining nine lowest safety factors ranged from 0.75 to 0.86.

Figure III -33

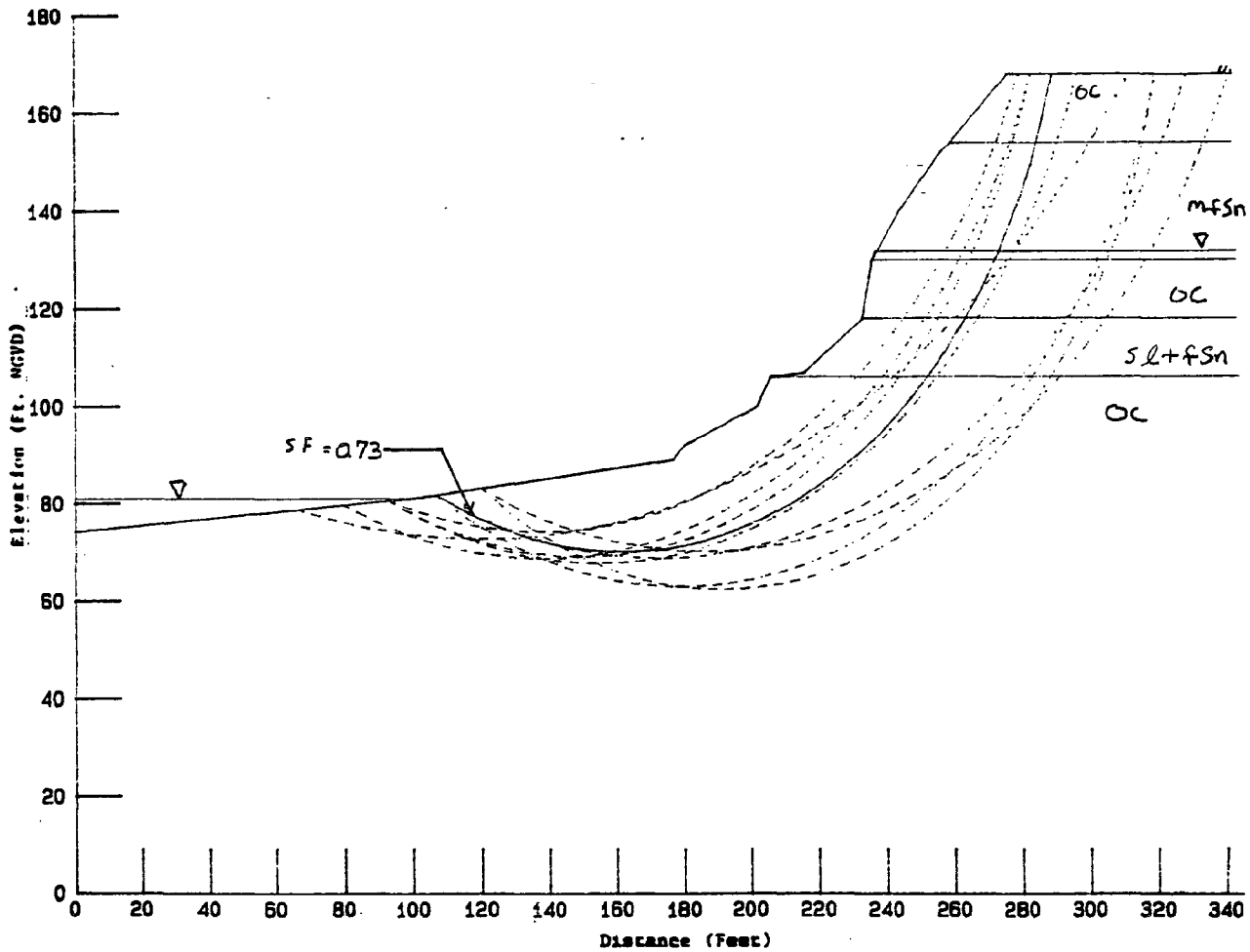
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 28



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

Figure III -34

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 29



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

Based on the deterministic slope stability analysis and on the observed bluff conditions, Section 26 was considered to have an unstable slope with respect to rotational sliding.

Section 26 was also considered to have an unstable slope with respect to translational sliding. The upper slope was moderately 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. Failure due to translational motion was, therefore, precipitated by the relative position of the water table and the angle of the bluff slope.

Bluff toe erosion was observed throughout Section 26 and was considered a major influence on bluff instability through over-steepening of the bluff slope. No shore protection structures were present in Section 26 as of 1987.

Full stabilization of the bluff slope in Section 26 would require implementing shore protection measures. Bluff slope regrading and bluff toe protection should be considered.

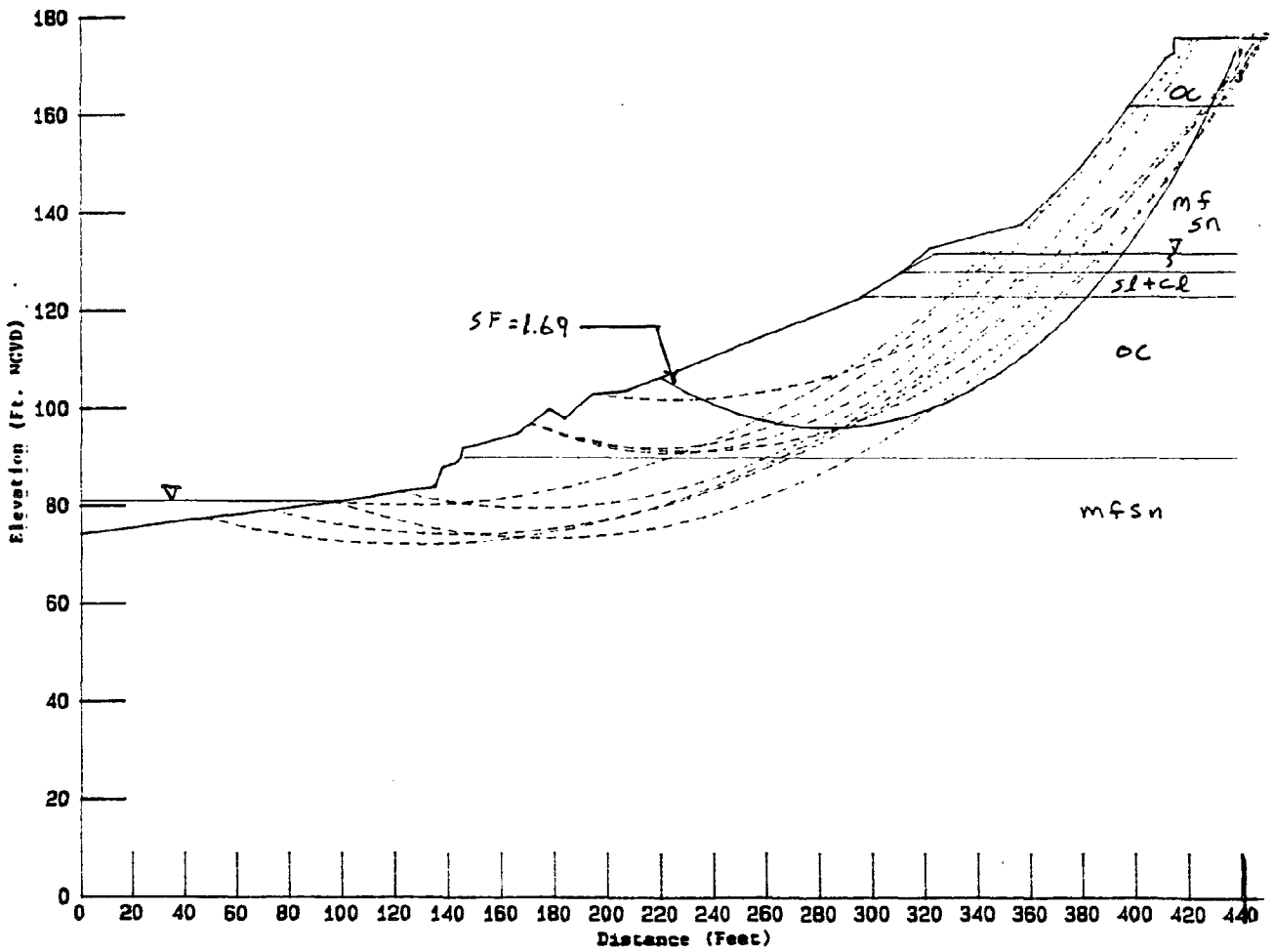
Bluff Analysis Section 27: The stability of the bluff slope in Section 27, located in Warnimont Park, was characterized by the use of Profile Nos. 30 and 31.

The results of the deterministic slope stability analyses are shown in Figures III-35 and III-36 for Profiles 30 and 31, respectively. Profile 30 was taken on a recently slumped area in the southern end of the section. The results of the deterministic slope analysis on this site indicate the slope was 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 remaining nine lowest safety factors ranged from 1.73 to 1.95. 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 0.70, and was located in the upper two-thirds of the bluff slope. The remaining nine lowest safety factors ranged from 0.73 to 1.0.

Probabilistic stability analyses were conducted for both profile sites. Of the 20 analyses conducted for Profile No. 30, the lowest safety factors ranged

Figure III -35

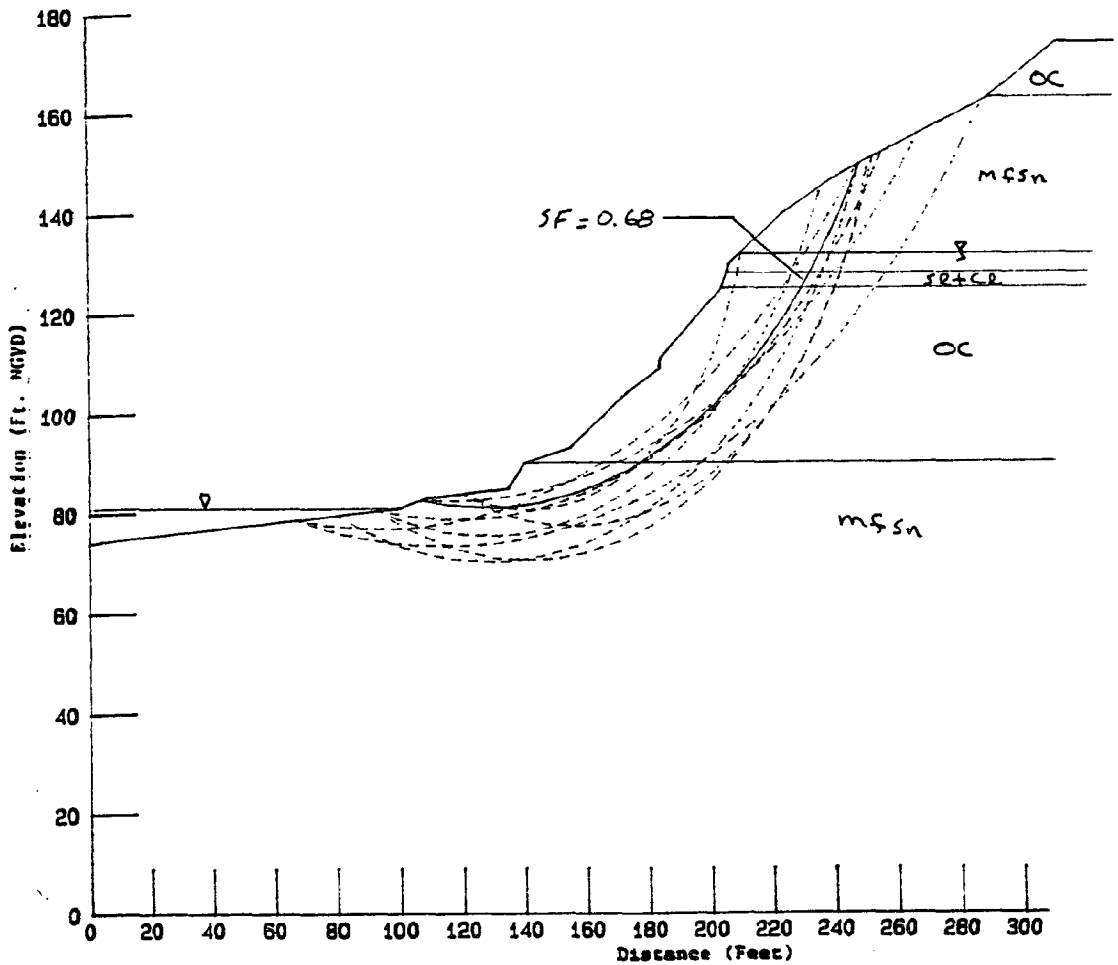
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 30



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

Figure III -36

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 31



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

from 1.09 to 1.79. None of the calculated safety factors were less than 1.0. Of the total of 200 failure surfaces evaluated at Profile No. 30, none had safety factors less than 1.0. 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 less than 1.0. Of the total of 200 failure surfaces evaluated at Profile No. 31, 165, or 82 percent, have 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 motion generally occurred in the lower portion of the bluff slope due to the presence of major groundwater seeps. It also occurred in the upper slope between bowl-shaped failure scars, although this portion of the slope is generally well-vegetated.

Bluff toe erosion was observed throughout the section during the field survey conducted during the fall of 1987. It is severe enough to affect the stability of the bluff in places, but is generally not the major cause for slope failure in Section 27.

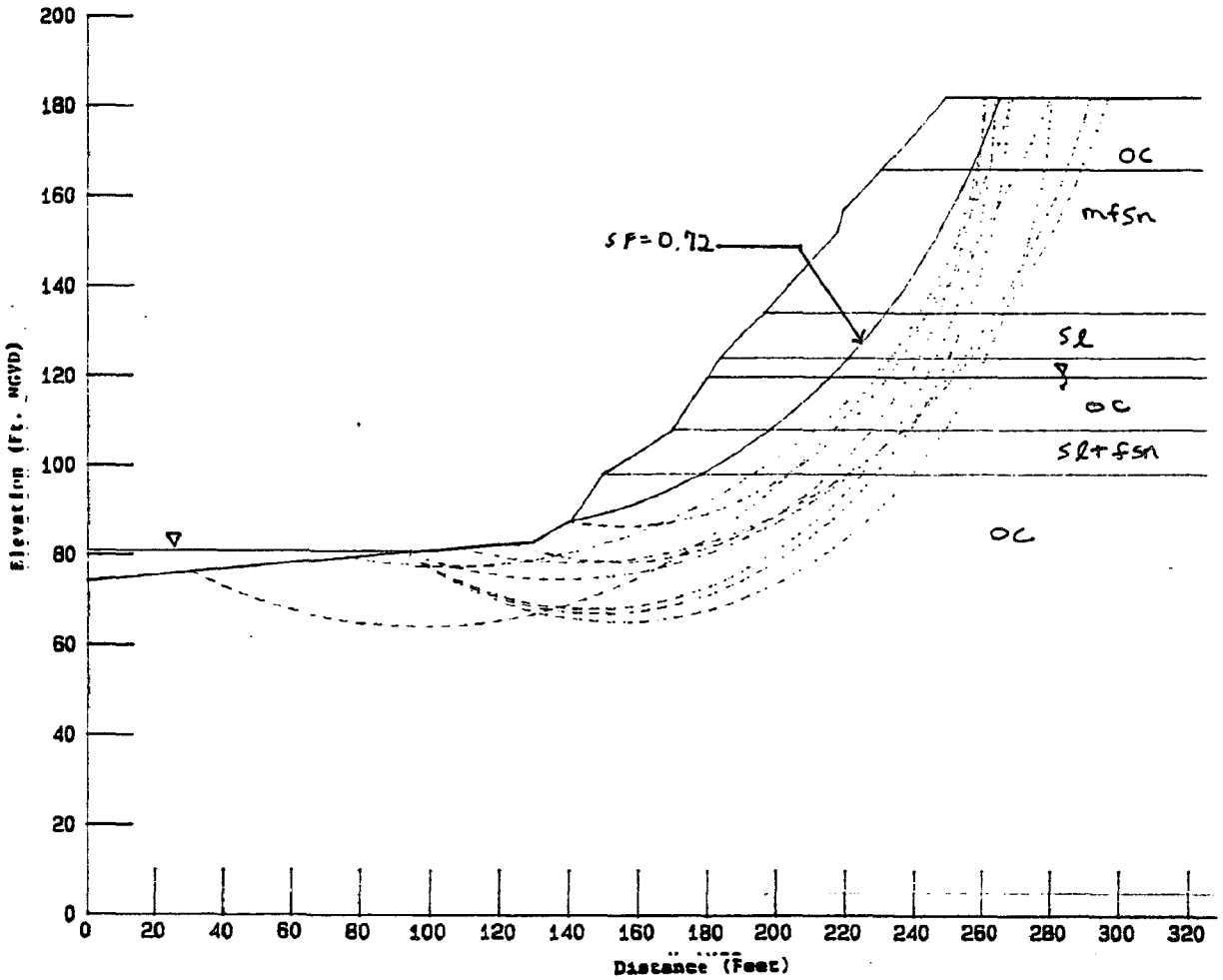
In order to fully stabilize the bluff slope in Section 27, shore protection measures are needed. Bluff toe protection and bluff slope regrading should be considered.

Bluff Analyses Section 28: The stability of the bluff slope within Section 28, which is located in Warnimont Park, was characterized by the use of Profile No. 32.

The results of the deterministic slops stability analysis shown in Figure III-37 indicate that Profile No. 32 had 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 remaining nine lowest safety factors ranged from 0.72 to 0.83. Based on the deterministic slope stability analysis and on observed bluff conditions,

Figure III -37

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 32



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

Section 28 was considered to have unstable bluff slope with respect to rotational sliding.

Section 28 was considered to have a marginal bluff slope with respect to translational sliding. During the field survey conducted during the fall of 1987, Section 28 was reported to be a steep, unvegetated bluff face deeply cut by several well-vegetated ravines. Failure by translational motion was concentrated on the bare bluff face. Groundwater seeps at the base of the bluff were also observed during the field survey. This discharge contributed to the occurrence of shallow, planar failures.

The 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 as of 1987.

Both bluff toe protection and bluff slope regrading are necessary to fully stabilize the bluff slope in this section.

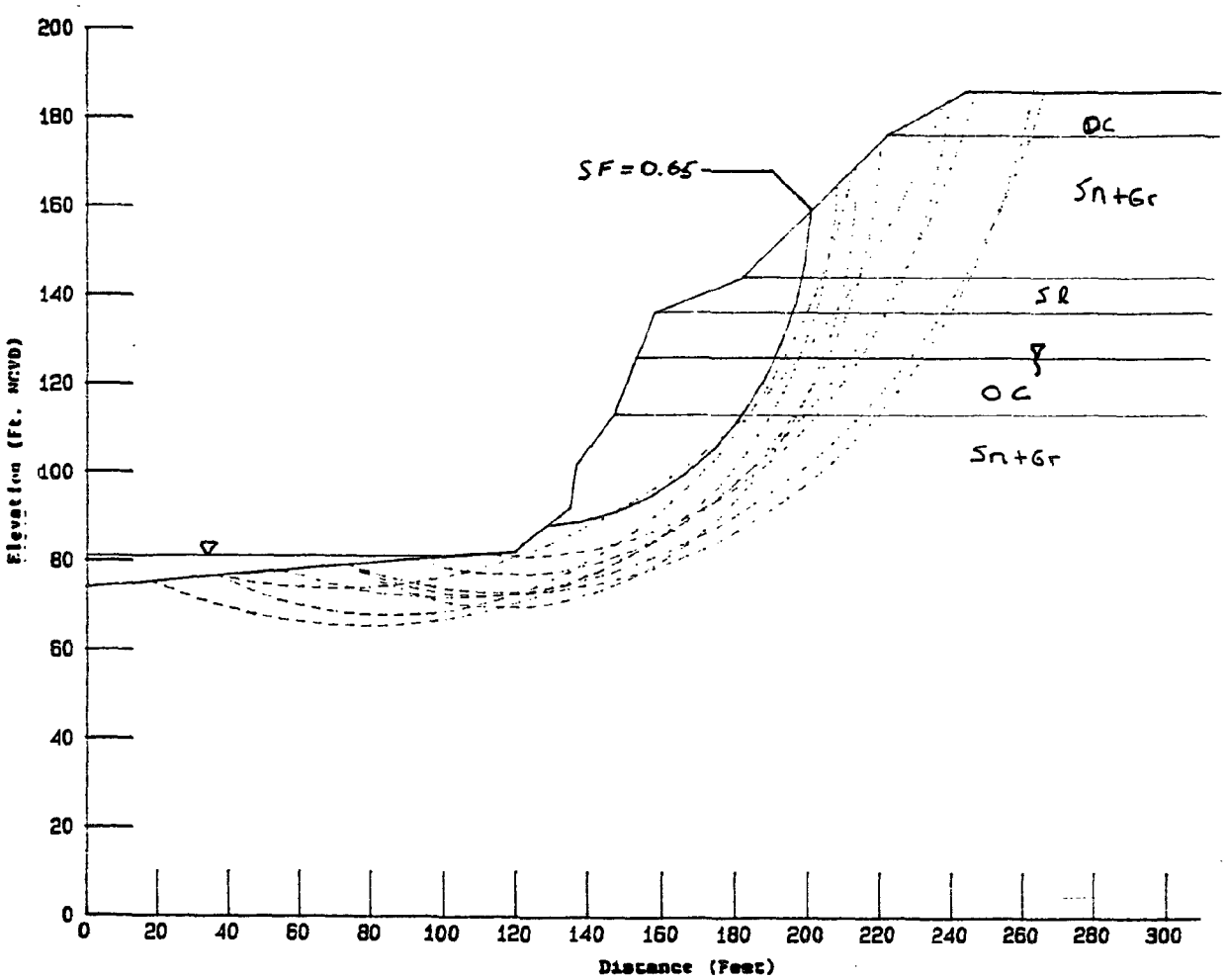
Bluff Analysis Section 29: The stability of the bluff slope within Section 29, located in Warnimont Park, was characterized by the use of Profile No. 33.

The results of the deterministic slope stability analysis, shown in Figure III-38, indicate that Profile No. 33 had an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.65, and was located within the lower two-thirds of the bluff slope. The remaining nine lowest safety factors ranged from 0.66 to 0.84. Based on the deterministic slope stability analysis and on observed bluff conditions, Section 29 was considered to have an unstable slope with respect to rotational sliding.

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 steepness of the slope were cited as the major causes for failure by translational motion.

Figure III -38

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 33



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

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. The effect was most acute in the he northern end of the section. No shore protection structures were located within this section as of 1987.

In order to fully stabilize the bluff in this section, both bluff toe protection and bluff slope regrading are necessary.

Bluff Analysis Section 30: The stability of the bluff slope within Section 30 located in Warnimont Park was characterized by the use of Profile No. 34.

The results of the deterministic slope stability analysis, shown in Figure III-39, indicate that Profile No. 34 had an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.81, and included the entire bluff slope. The remaining nine lowest safety factors ranged from 0.82 to 0.90. Based on both the deterministic slope stability analysis and on the observed bluff conditions, Section 30 was considered to have an unstable slope with respect to rotational sliding.

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 and falls over the lower stratigraphic units were observed during the field survey conducted during the fall of 1987.

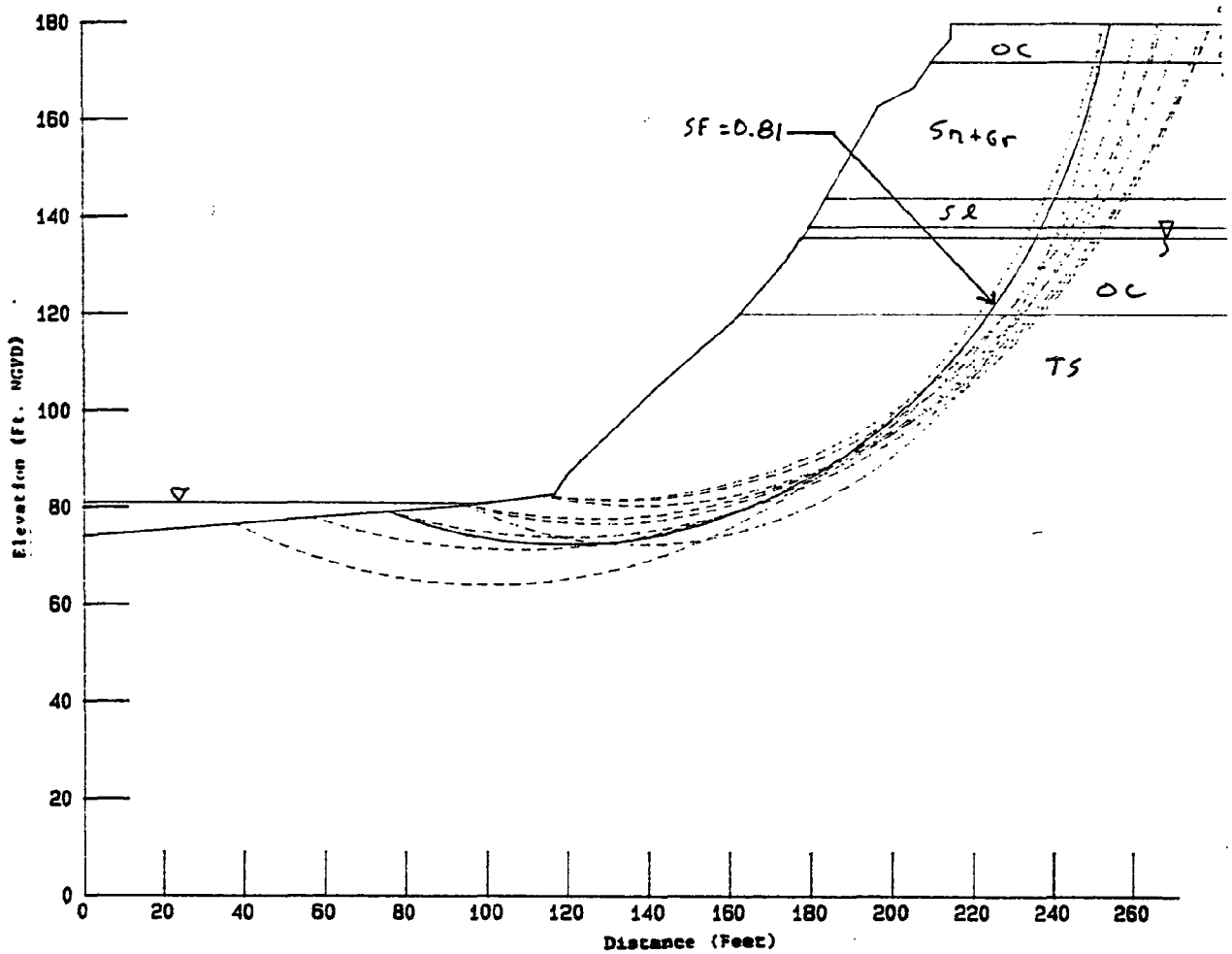
Toe erosion contributing to the instability of the bluff slope along the entire shoreline of Section 30 was observed during the fall of 1987 field survey.

In order to fully stabilize the bluff in this section, bluff toe protection and bluff slope regrading are necessary.

Bluff Analysis Section 31: The stability of the bluff slope within Section 31, which is located in Warnimont Park, was characterized by the use of Profile No. 35.

Figure III -39

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 34



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

The results of the deterministic slope stability analysis, shown in Figure III-40, indicate that Profile No. 35 had an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.81, and was included in the entire bluff slope. The remaining nine lowest safety factors ranged from 0.82 to 0.87.

Of the 20 probabilistic stability analyses conducted, the lowest safety factors ranged from 0.50 to 0.89. Of the total of 200 failure surfaces evaluated, all 200 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 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.

Despite the presence of two small permeable groins in the southern end of the section, bluff toe erosion affecting the stability of the bluff slope was observed during the fall of 1987 field survey.

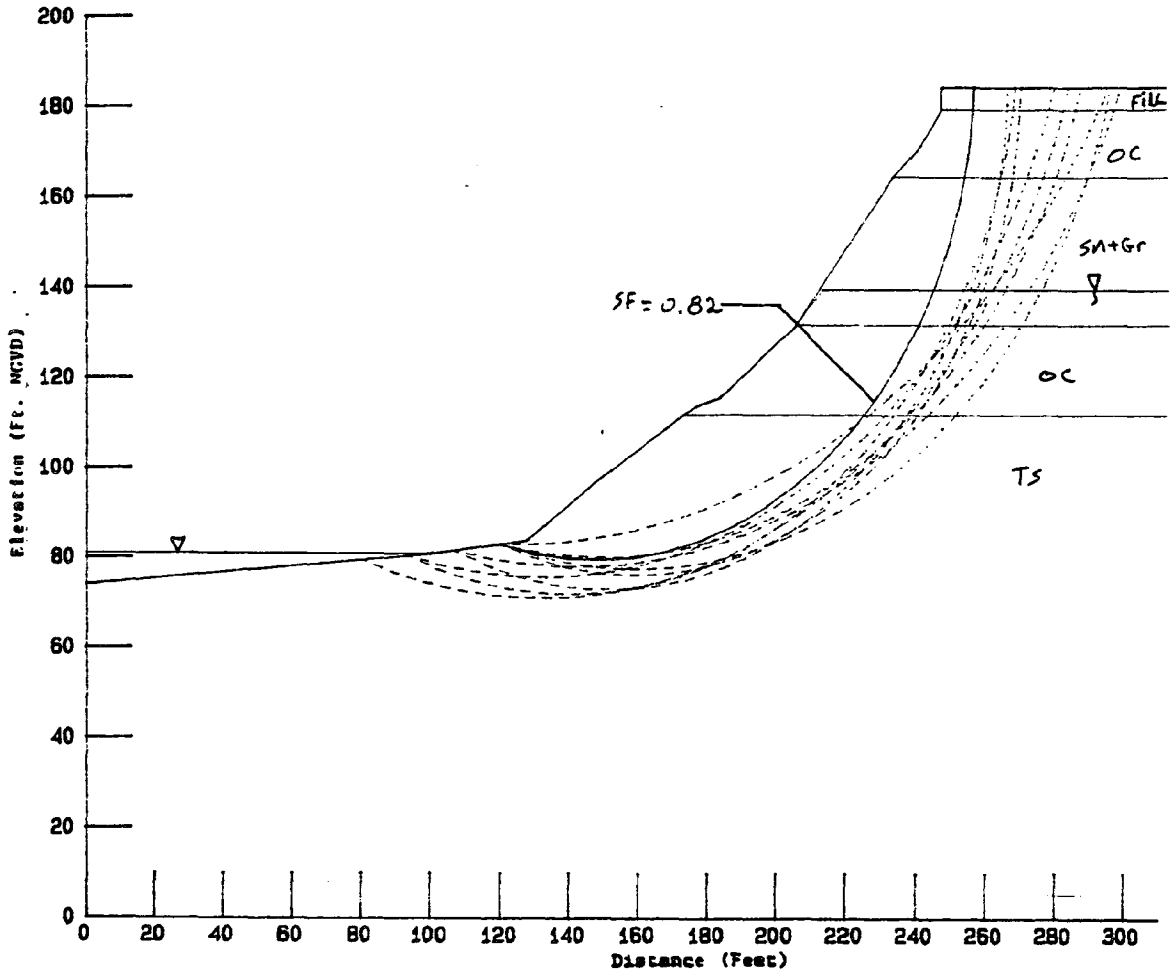
In order to fully stabilize the bluff in this section, two types of shoreline protection must be implemented. First, the protection must be provided and properly maintained. This action could include repairing the two existing structures. Second, bluff slope regrading is needed.

Bluff Analysis Section 32: The entire shoreline of Section 32, located at the Cudahy Water Intake, was protected by a concrete bulkhead fortified with a rip-rap revetment. The natural bluff has been graded to a stable slope. No erosion of the bluff was observed during the field survey conducted during the fall of 1987.

Evidence of overtopping was observed during a field inspection conducted during the spring of 1988; also, several locations where the rip-rap revetment had settled away from the bulkhead, exposing steel reinforcement rods. Maintenance of the existing structure is necessary to insure continued bluff stability in this section.

Figure III-40

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 35



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

Bluff Analysis Section 33: The stability of the bluff slope within Section 33, which is located in Warnimont Park, was characterized by the use of Profile No. 36.

The results of the deterministic slope stability analysis, shown in Figure III-41, indicate that Profile No. 36 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 included the entire bluff slope. The remaining nine lowest safety factors ranged from 0.82 to 0.92. Based on the results of the deterministic slope stability analysis and on the observed bluff conditions, Section 33 was considered to have an unstable bluff slope with respect to rotational sliding.

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 observed during the field survey conducted during the fall of 1987 support this classification.

Although toe erosion was observed in Section 33 during the 1987 field survey, it was not considered to be a threat to the overall stability of the bluff slope. No shore protection structures were present in this section as of 1987.

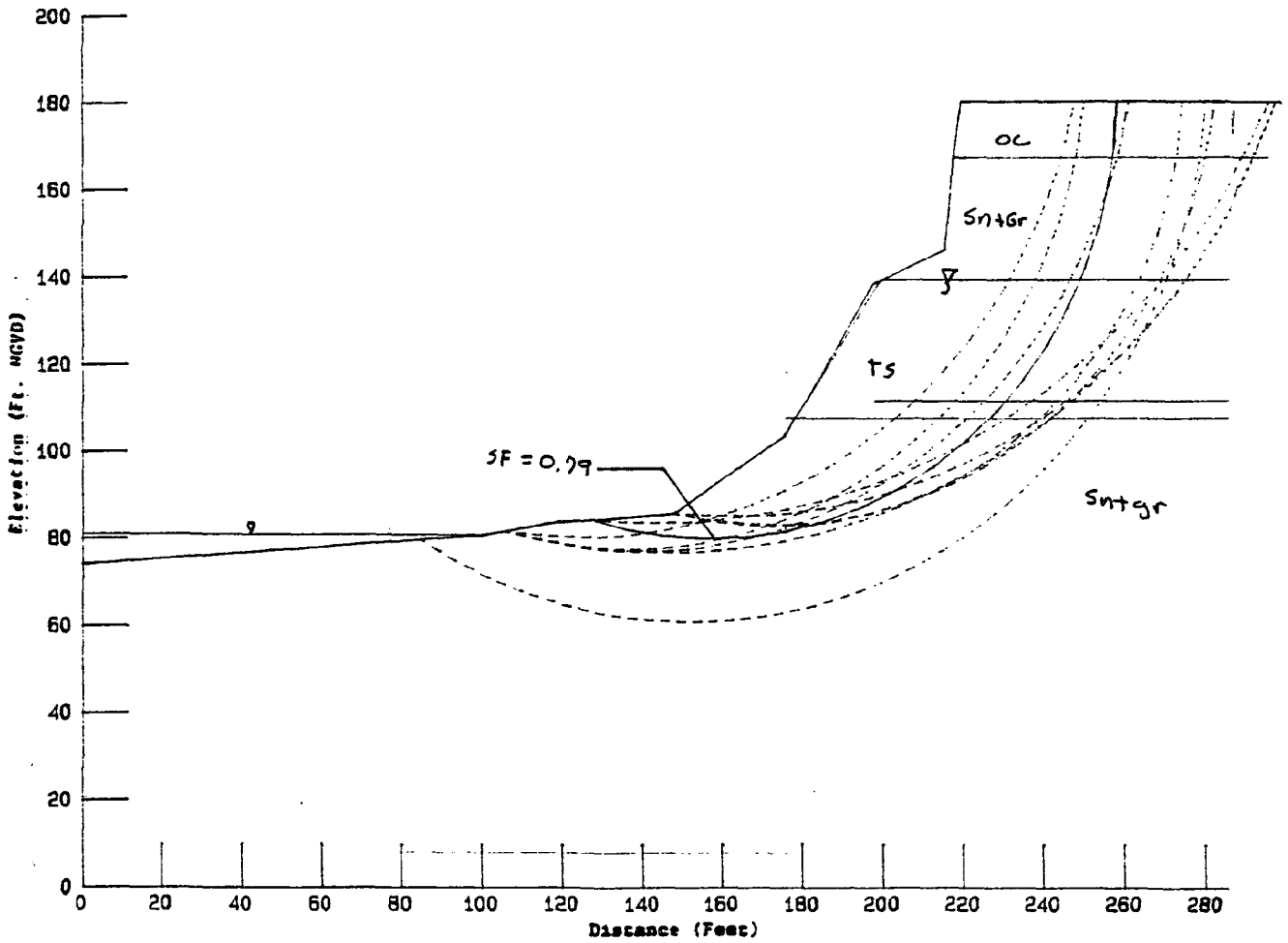
Shore protection measures are needed to fully stabilize the bluff in Section 33. Bluff slope regrading and bluff toe protection are suggested.

Bluff Analysis Section 34: The stability of the bluff within Section 34, located at Sheridan Park, was characteristic of the use of Profile No. 37 and Profile No. 38.

The results of the deterministic slope stability analyses, shown in Figure III-42 and Figure III-43 for Profile No. 37 and Profile No. 38, respectively, indicate that Section 34 has a stable bluff slope 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 remaining nine lowest safety factors range from 1.25 to 1.34. The lowest failure surface calculated

Figure III -41

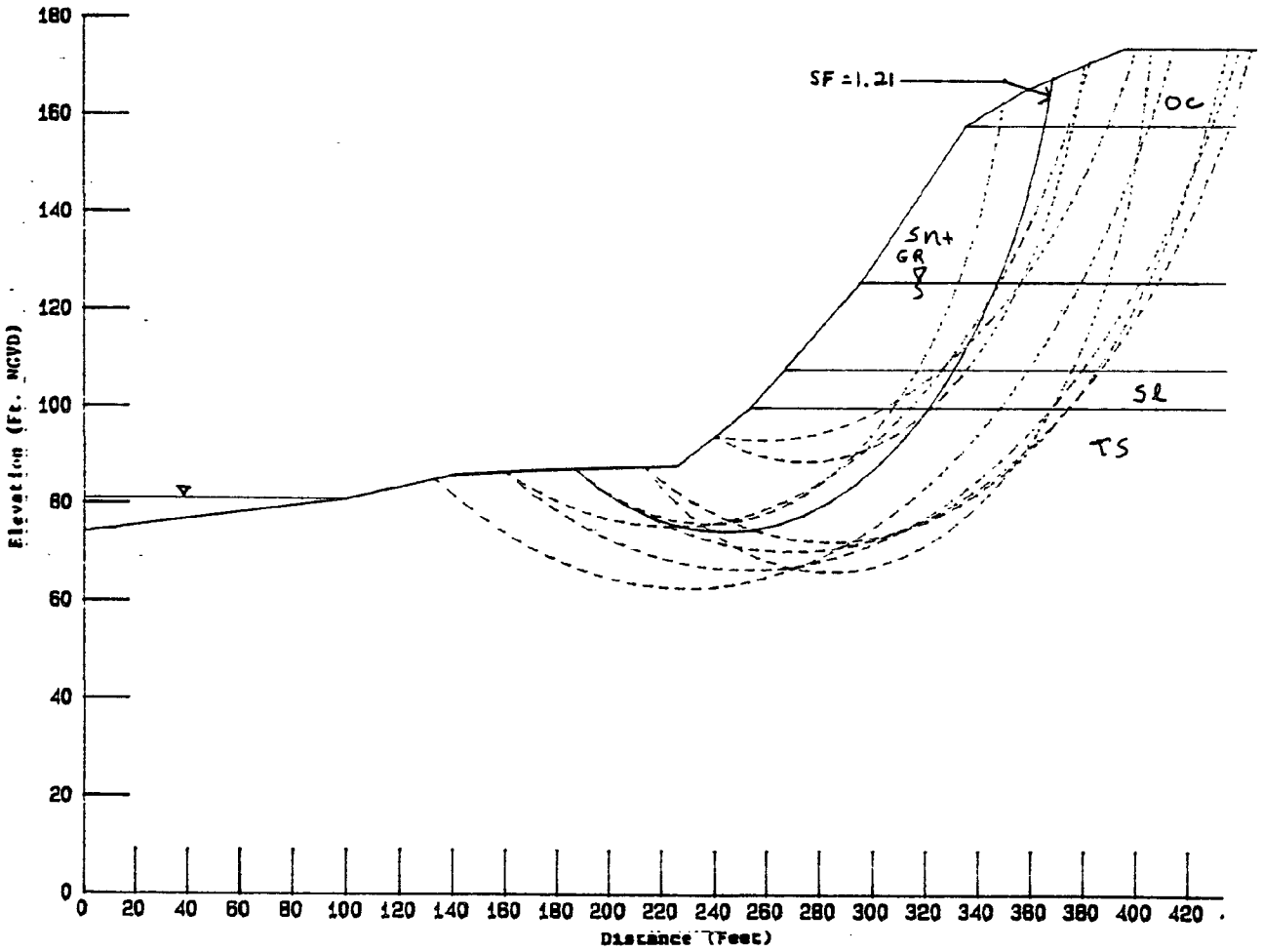
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 36



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

Figure III -42

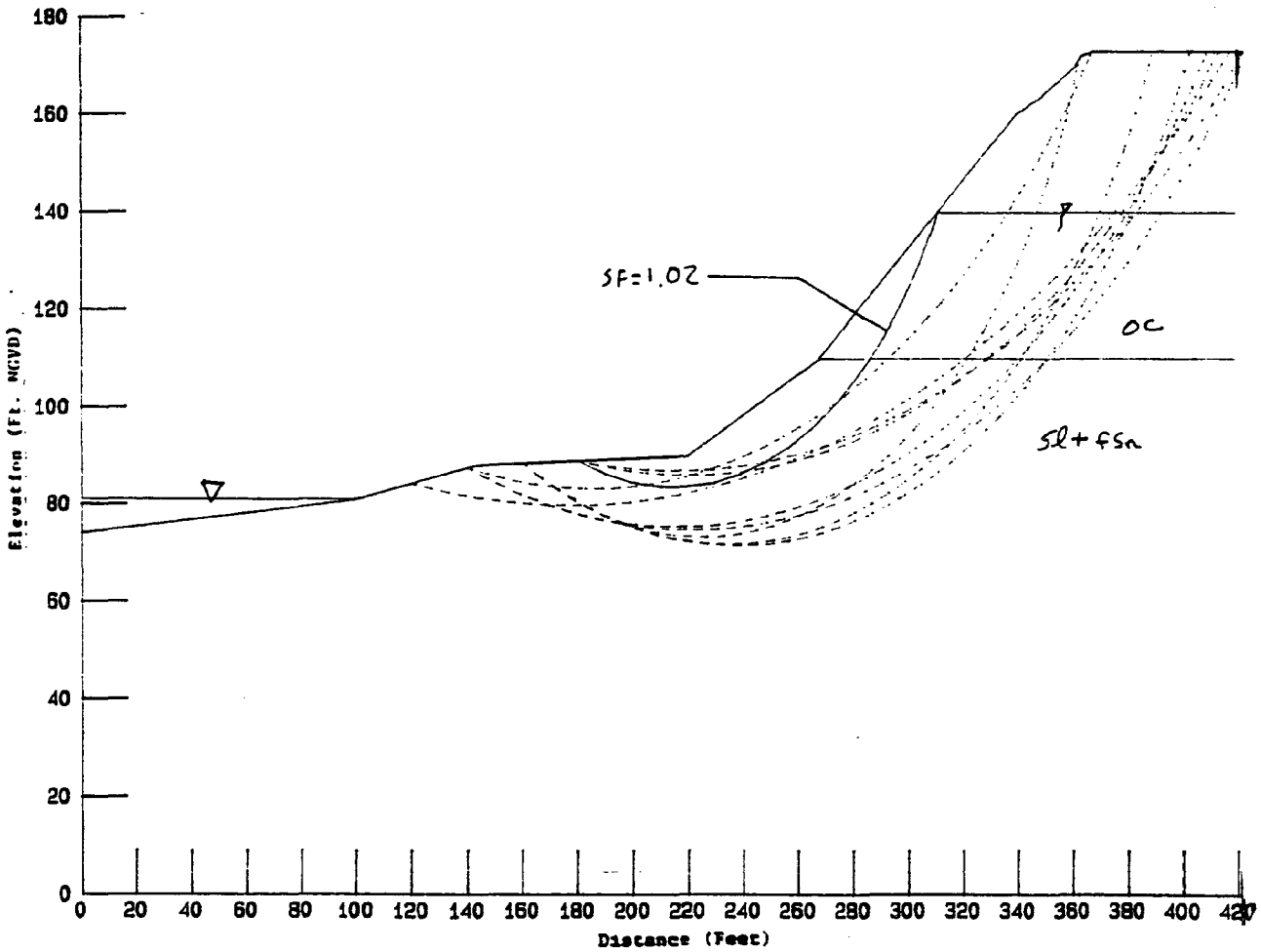
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 37



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

Figure III -43

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 38



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

at Profile No. 38 had a safety factor of 1.02 and included the lower two-thirds of the bluff slope. The remaining nine lowest safety factors range 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. Of the 20 probabilistic stability analyses conducted at Profile No. 37, the lowest safety factors ranged from 0.73 to 1.25, with 13 failure surfaces, or 65 percent, having safety factors less than 1.0. Of the total of 200 failure surfaces evaluated at this site, 40, or 20 percent of the surfaces,had safety factors less than 1.0. Of the 20 probabilistic stability analyses conducted at Profile No. 38, the lowest safety factors ranged from 0.71 to 1.25, with 11 failure surfaces, or 55 percent, having safety factors less than 1.0. Of the total of 200 failure surfaces evaluated at this site, 72, or 36 percent of the surfaces, had safety factors less than 1.0. 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 was well-vegetated, terraced, and had an average slope angle of less than 30 degrees.

Bluff toe erosion in Section 34 was considered to be slight throughout the section. This was due to the protection provided by a groin field. A fairly wide beach had been trapped on the updrift side of each structure. To insure continued bluff protection, these structures must be properly maintained.

In addition to maintenance of existing shore protection structures, minor bluff slope re-vegetation is needed to fully stabilize the bluff slope in Section 34.

Bluff Analysis Section 35: The stability of the bluff slope within Section 35, located at Sheridan Park, was characterized by the use of Profile No. 39.

The results of the deterministic slope stability analysis, shown in Figure III-44, indicate that Profile No. 39 had an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.74, and was located within the lower two-thirds of the bluff slope. The remaining nine lowest safety factors ranged from 0.90 to 1.02

Of the 20 probabilistic stability analyses conducted, the lowest safety factors ranged from 0.58 to 0.94. Of the total of 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.

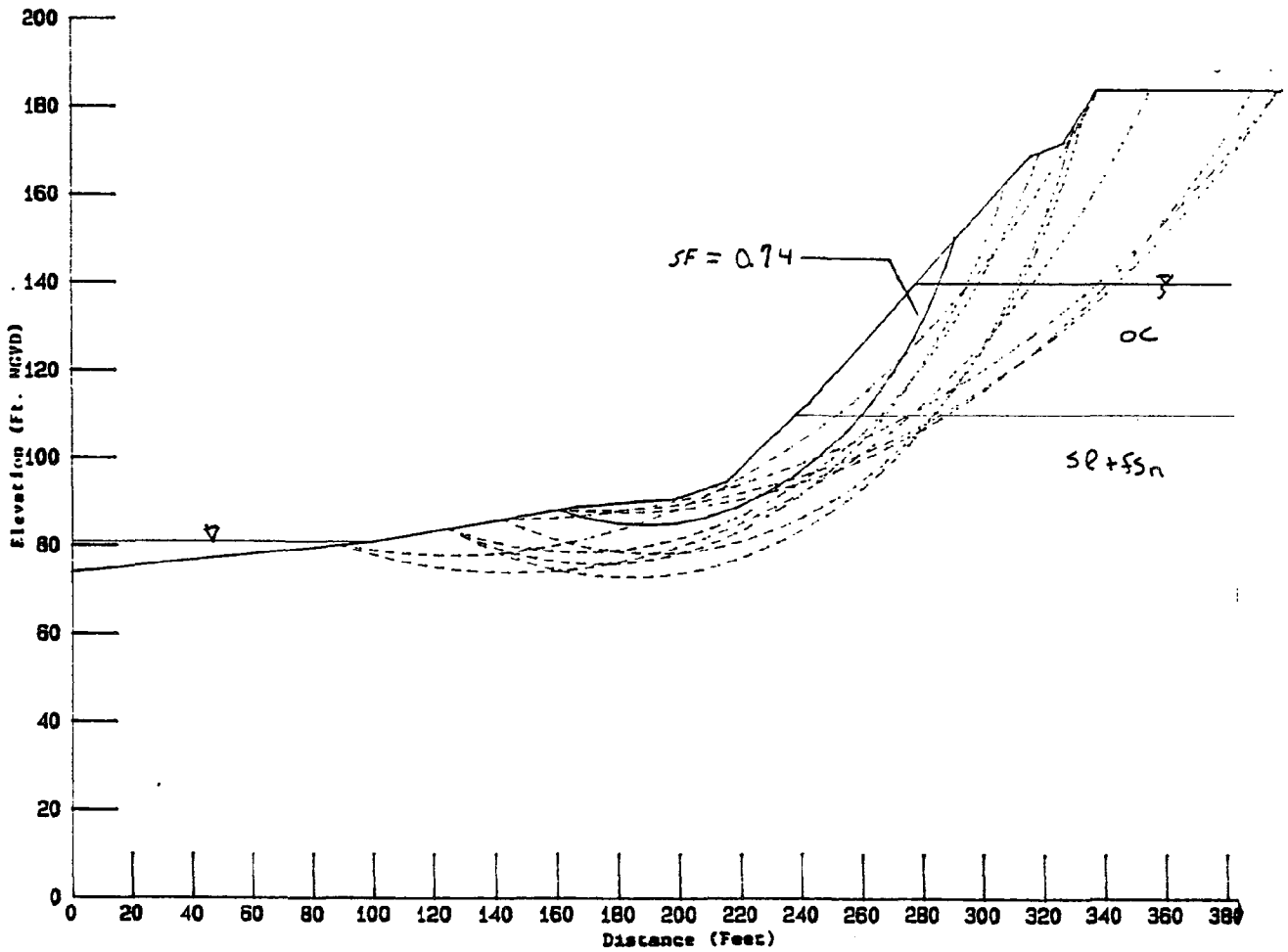
Section 35 was considered to have a marginal bluff slope with respect to translational sliding. The relatively steep angle of the slope, coupled with the sparse vegetation, contributed to the observed tendency to fail by translational motion reported during the field survey conducted in the fall of 1987. Groundwater seeps observed during the field survey were also considered to be a major influence on bluff slope failure in this section.

Slight bluff toe erosion observed throughout Section 35 was considered to have no effect on the stability of the bluff slope. The groin field, which begins at the southern end of Section 34, continues through Section 35, providing toe protection by an accumulated beach. Although Sections 34 and 35 had similar stratigraphy and bluff toe protection, the former had a more stable bluff slope than the latter. The position of the groundwater table in each section seemed to be the factor causing this disparity in results. The impact of a small surface water impoundment in Section 35 on the site hydrogeology could not be evaluated at the time of the fall 1987 field survey. A thorough hydrogeologic investigation of this section must be conducted before final remedial protection measures are outlined.

Preliminarily, the maintenance of existing shore protection structures, bluff slope revegetation, and groundwater drainage are considered necessary to fully stabilize the bluff slope in Section 35.

Figure III -44

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 39



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

Bluff Analysis Section 36: The stability of the bluff slope in Section 36, located at Sheridan Park, was characterized by the use of Profile No. 40.

The results of the deterministic slope stability analysis, shown in Figure III-45, indicates that Profile No. 40 had 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 remaining nine lowest safety factors ranged from 0.95 to 1.09.

Of the 20 probabilistic stability analyses conducted, the lowest safety factors ranged from 1.05 to 1.45. Of the 200 failure surfaces evaluated, none had safety factors less than 1.0. Although the probabilistic analyses indicate that the bluff was stable, observations reported during the field survey conducted during the fall of 19087, and the deterministic analysis, indicate a potential for bluff slope failure by rotational sliding in this section. The final bluff slope classification, therefore, was considered to be marginal.

Section 36 was considered to be stable with respect to translational sliding. This was due to the good vegetative cover and relatively gentle average slope angle. Some disturbed soil areas in the upper section of the slope were noted during the fall 1987 field survey, but these areas were not indicative of overall slope instability.

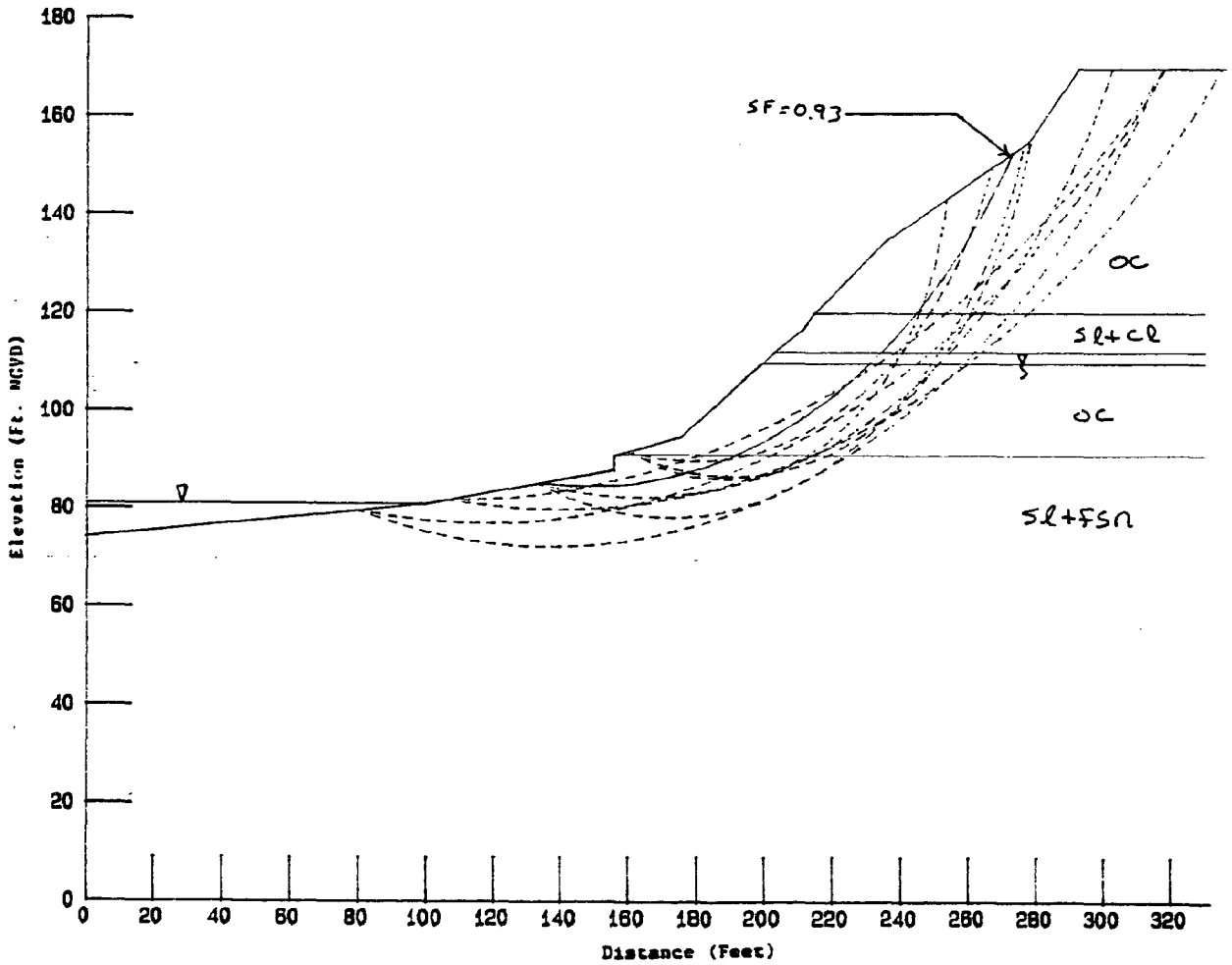
Toe erosion was slight in Section 36 due to the presence of two timber groins. The beach accumulated on the updrift side of these structures was generally wide enough to protect the toe of the bluff.

In order to fully stabilize the bluff in Section 36, the aforementioned structures must be maintained. In addition, revegetation of the bluff slope is necessary.

Bluff Analysis Section 37: The stability of the bluff slope within Section 37, located at Sheridan Park, was characterized by the use of Profile No. 41 and Profile No. 42.

Figure III -45

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 40



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

The results of the deterministic slope stability analyses shown in Figure III-46 and Figure III-47, for Profile No. 41 and Profile No. 42, respectively, indicate that Section 37 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at Profile No. 41 has a safety factor of 0.81, and was located in the lower two-thirds of the bluff slope. The remaining nine lowest safety factors range 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 remaining nine lowest safety factors range from 0.86 to 0.99. Based on the deterministic slope stability analyses and on observed bluff conditions, Section 37 was determined to be unstable with respect to rotational sliding.

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 as of 1987.

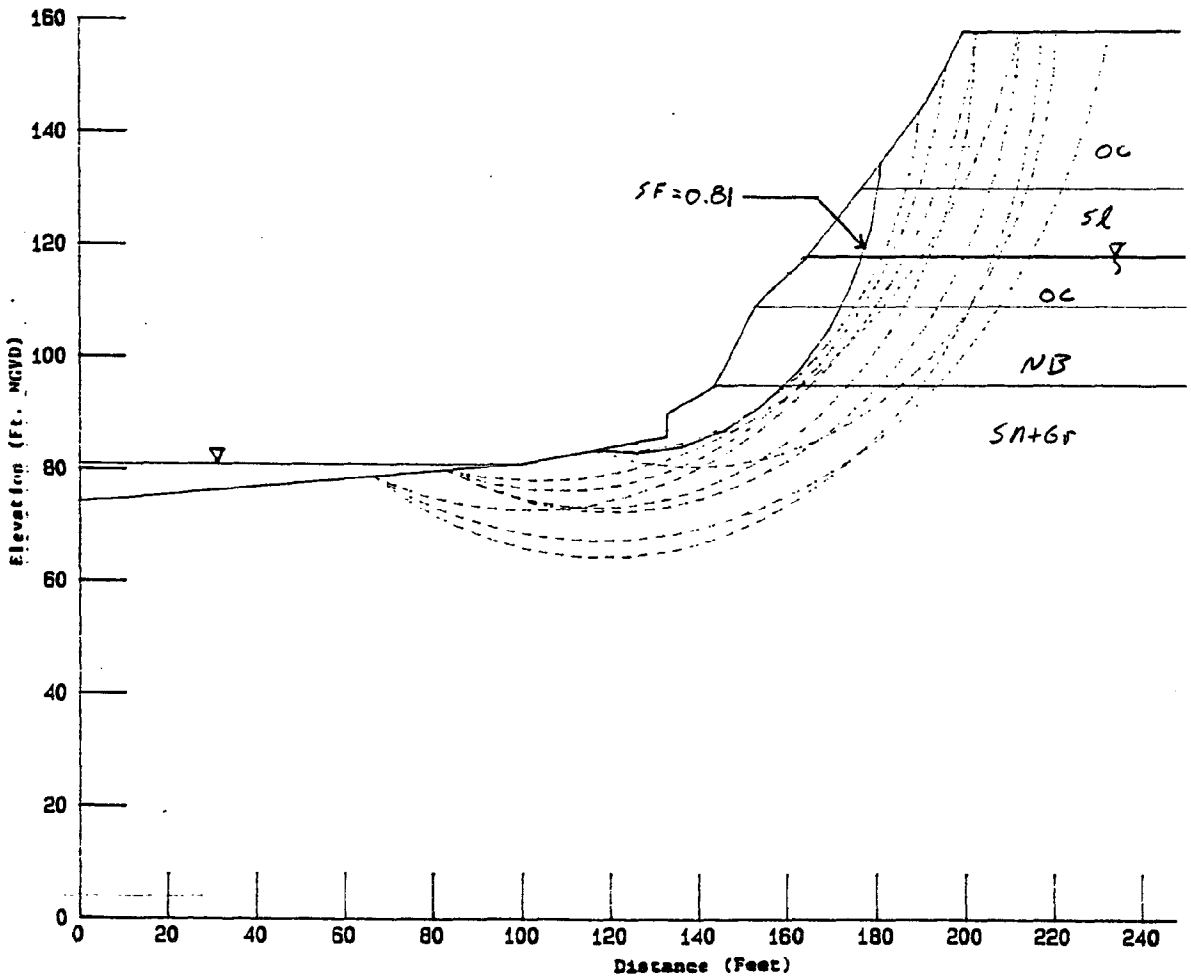
To fully stabilize the bluff slope in Section 37, shoreline protection measures are needed. Bluff toe protection, coupled with bluff slope regrading, are considered to be the best alternatives.

Bluff Analysis Section 38: The stability of the fill and underlying bluff slope within Section 38, which extends from Lunham Avenue to Denton Avenue, was characterized by the use of Profile No. 43.

The results of the deterministic slope stability analysis, shown in Figure III-48, indicate a potential threat of bluff slope failure with respect to rotational sliding. The critical 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 remaining nine failure surfaces had safety factors ranging from 0.85 to 1.56. The probabilistic slope stability analysis was not conducted for this profile because it is a fill site.

Figure III -46

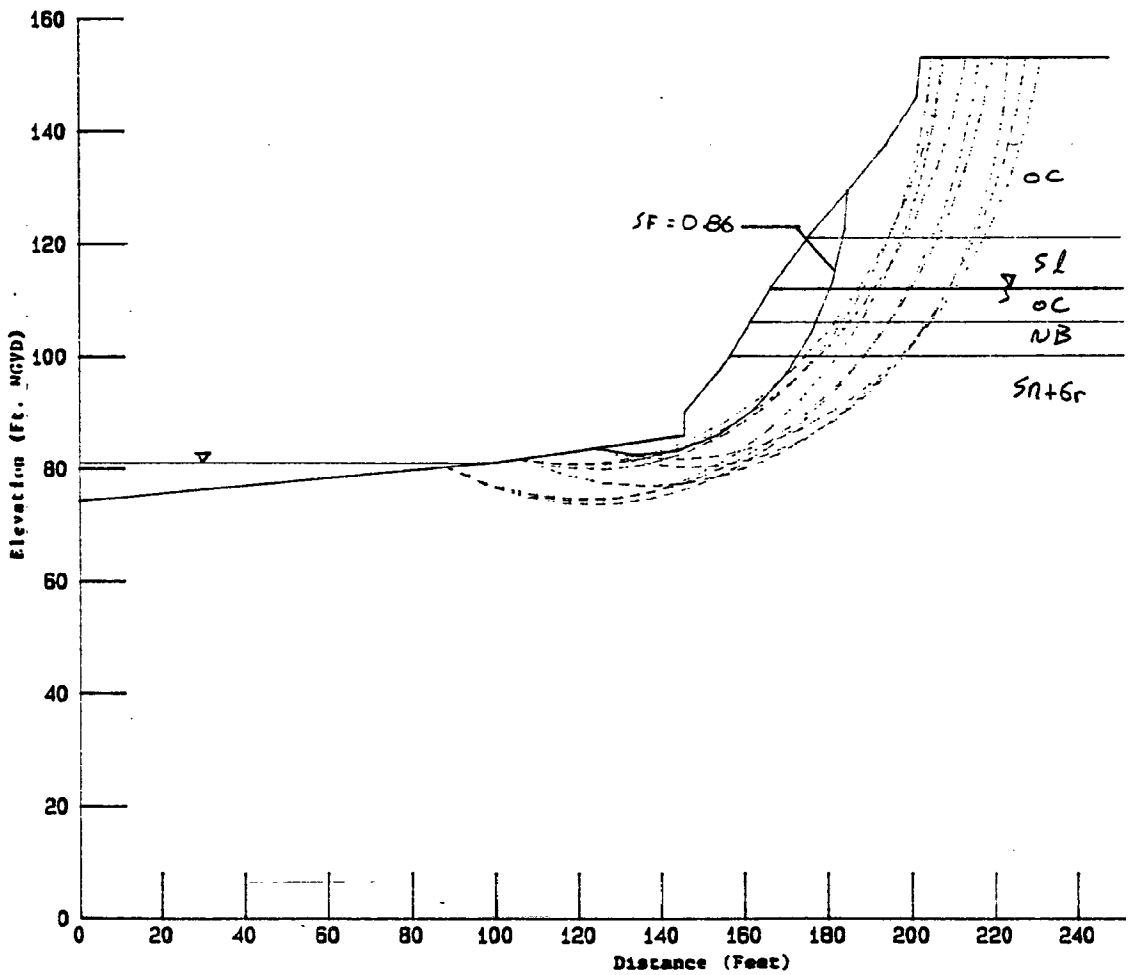
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 41



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

Figure III -47

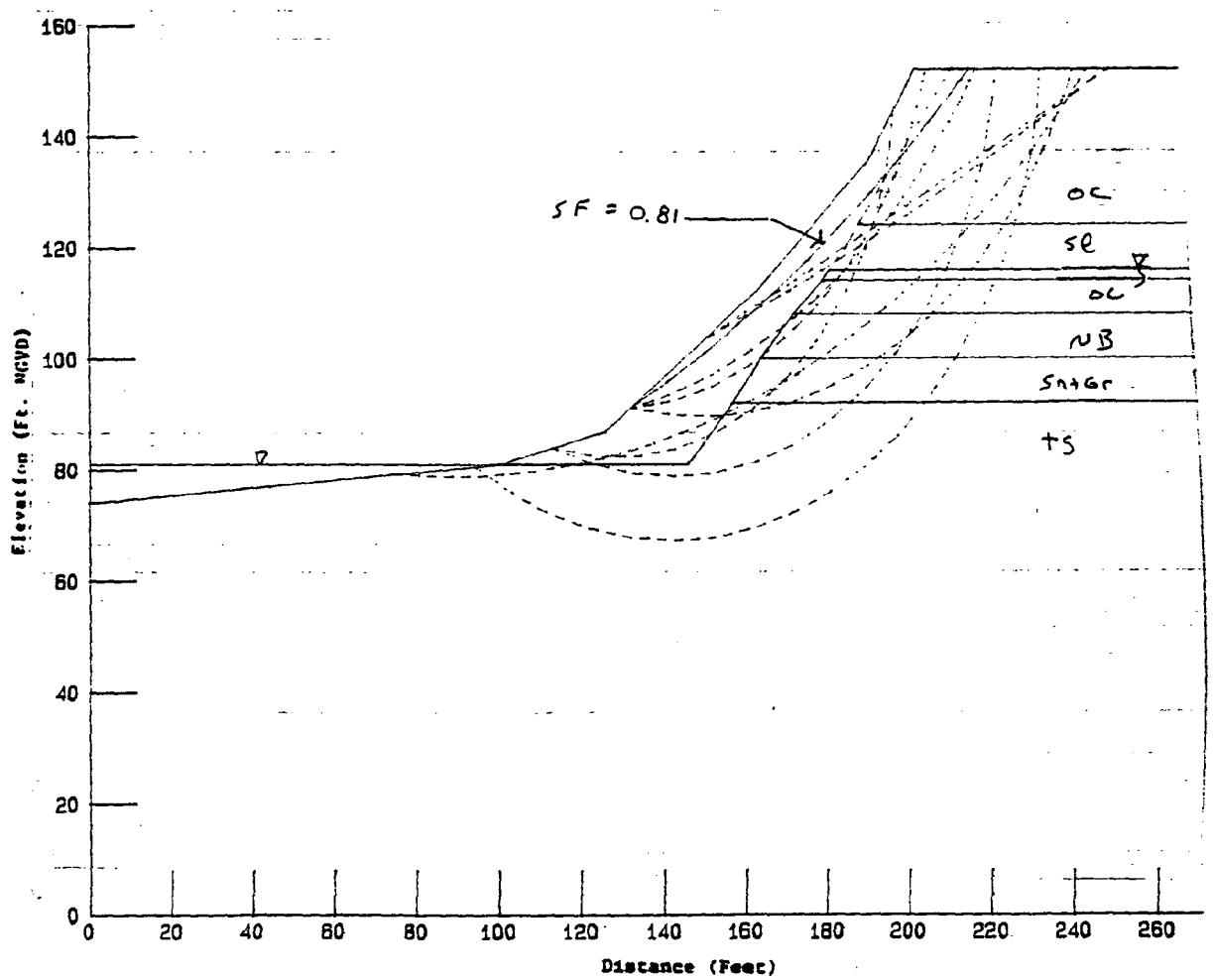
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 42



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

Figure III -48

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 43



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

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.

Moderate bluff toe erosion was observed within the entire shoreline of Section 38 during the field surveys 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 moderate effect on the stability of the bluff slope at the present time. No shore protection structures were located within this section as of 1987.

Although the bluff slope in this section has been filled, it is not stable. To fully stabilize the bluff, toe protection and bluff slope regrading are needed.

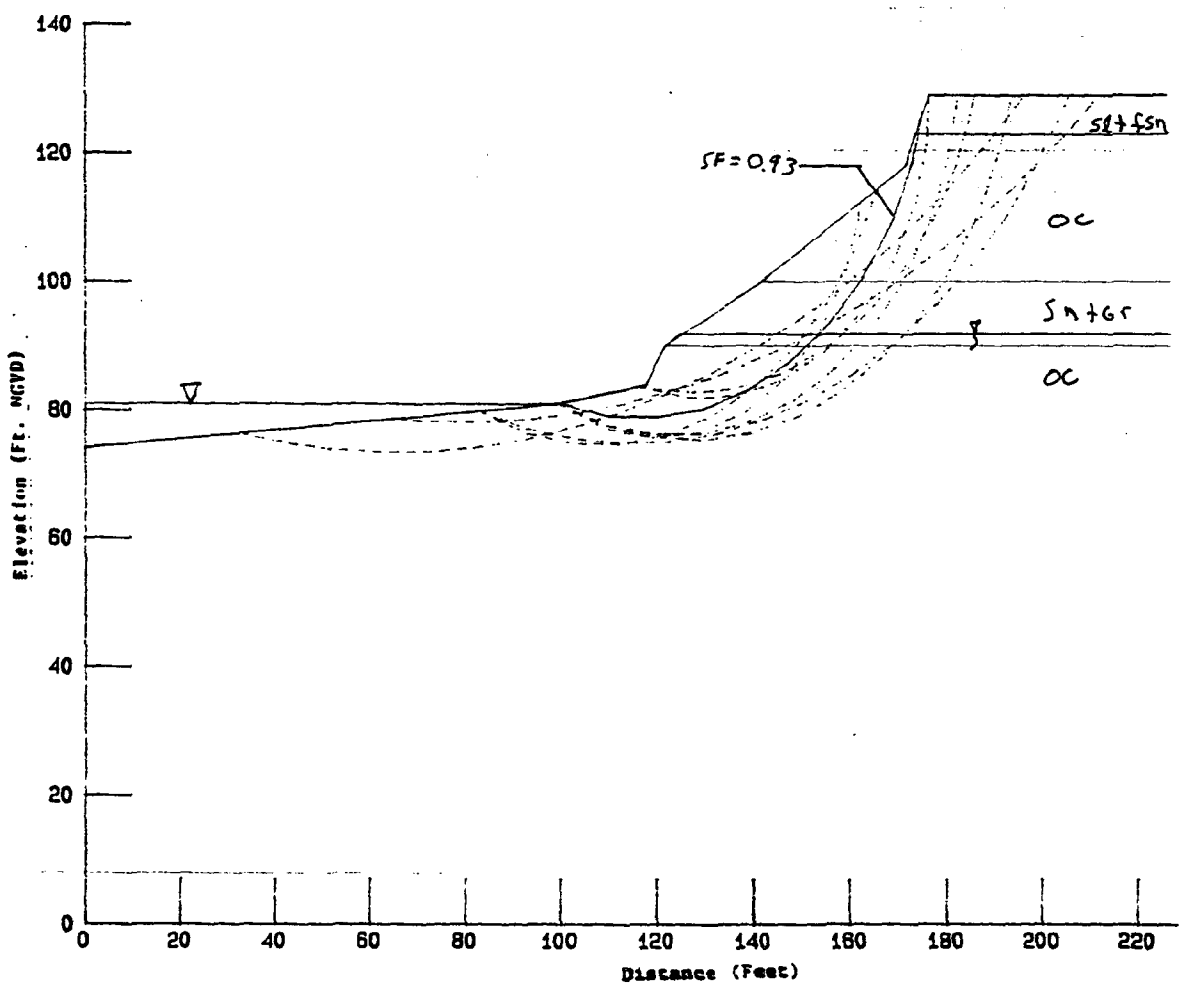
Bluff Analysis Section 39: The stability of the bluff slope within Section 39, which extends from Denton Avenue to the 100 block of Howard Avenue, was characterized by the use of Profile No. 44 and Profile No. 45.

The results of the deterministic slope stability analysis, shown in Figure III-49 and Figure III-50, for Profile No. 44 and Profile No. 45, respectively, indicate that the bluff slope in Section 39 is unstable. The critical failure surface calculated at Profile No. 44 had a safety factor of 0.93, and included the entire bluff slope. The remaining nine failure surfaces evaluated had safety factors ranging from 0.97 to 1.45. The critical 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 remaining nine failure surfaces evaluated had safety factors ranging from 0.79 to 0.97.

Of the 20 probabilistic stability analyses conducted at Profile No. 44, the lowest safety factors ranged from 0.73 to 1.28, with six surfaces, or 30 percent, having safety factors less than 1.0. Of the total of 200 failure surfaces evaluated at this profile site, 15, or 7 percent of the surfaces, had safety factors less than 1.0. Of the 20 probabilistic stability analyses

Figure III -49

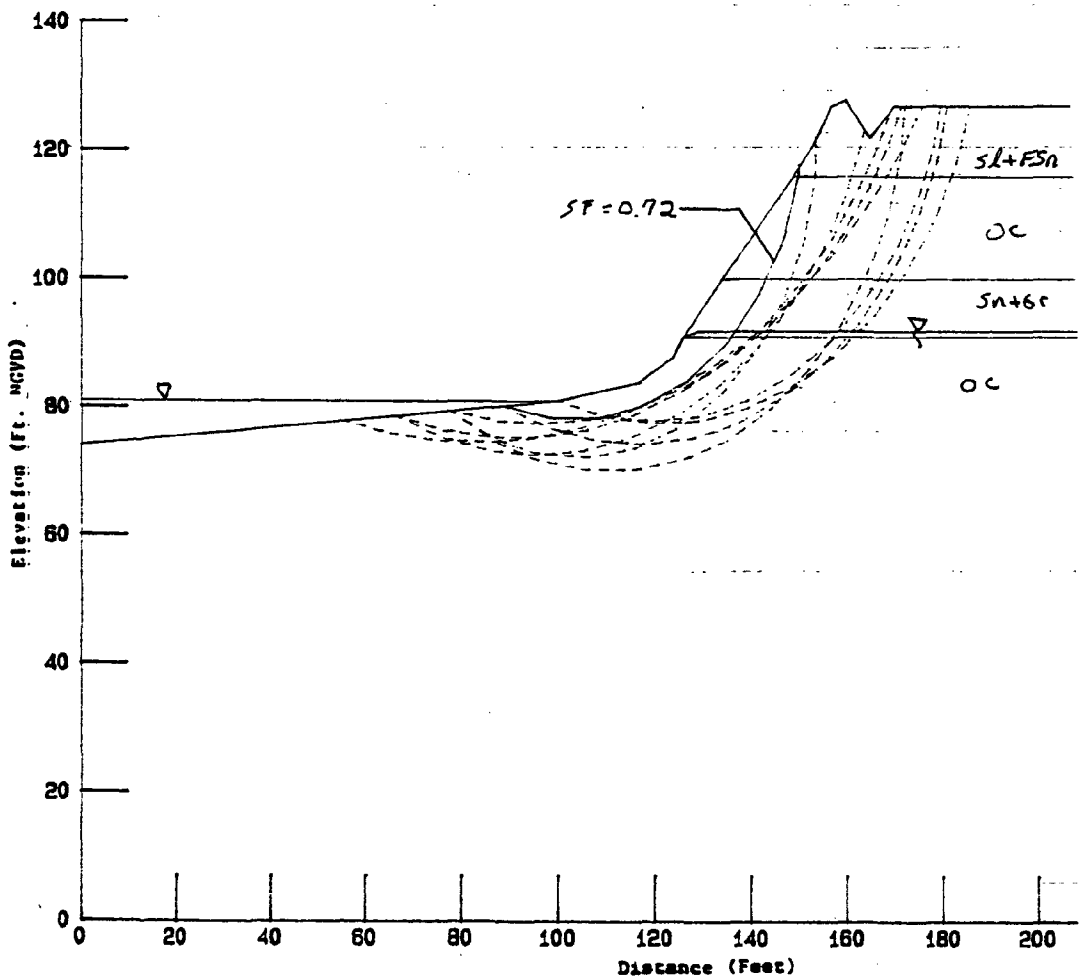
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 44



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

Figure III -50

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 45



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

conducted at Profile No. 45, the lowest safety factors ranged from 0.72 to 1.14, with 13 surfaces, or 65 percent, having safety factors less than 1.0. Of the total of 200 failure surfaces evaluated at this profile site, 62, or 31 percent of the surfaces, had safety factors 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.

Toe erosion throughout the entire section contributed to bluff failure by undercutting. Although considered moderate in comparison to toe erosion observed in other sections, this process had a significant negative effect on bluff stability in Section 39.

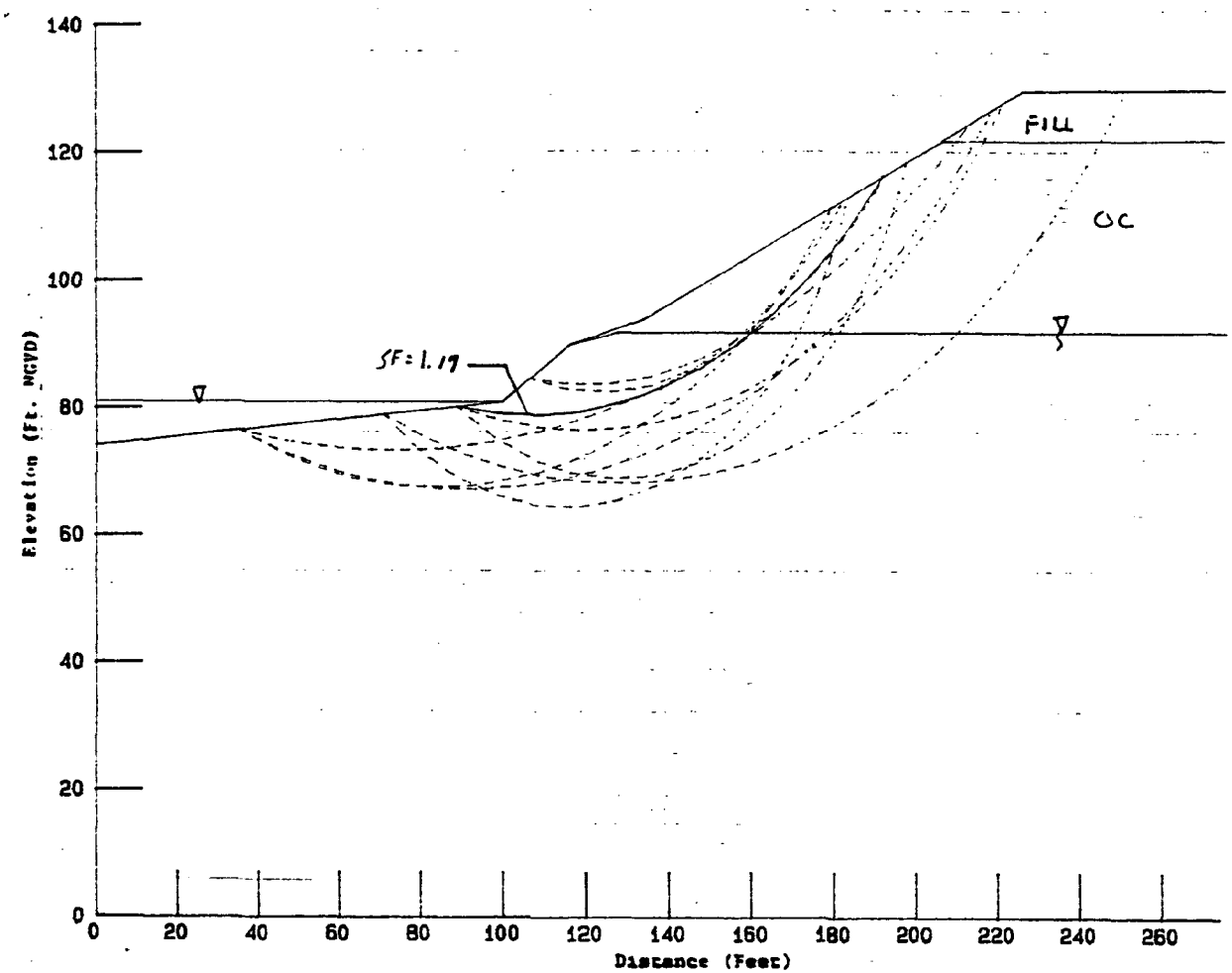
Shoreline protection measures are needed to fully stabilize the bluff in this section. Both bluff slope regrading and bluff toe protection are considered necessary.

Bluff Analysis Section 40: The stability of the bluff slope within Section 40, which extends from the 100 block of Howard Avenue to the Wisconsin Electric Lakeside Power Plant Breakwater, was characterized by the use of Profile No. - 46.

The results of the deterministic slope stability analysis, shown in Figure III-51, indicate that Profile No. 46 had 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 remaining nine lowest safety factors ranged from 1.24 to 1.45. The 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.

Figure III -51

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 46



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

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.

Although the toe of the bluff in the entire section was protected by a rip-rap revetment, bluff toe erosion was observed where the revetment was overtopped. The present structure must be maintained to insure maximum protection, hence fully stability of the bluff slope.

Bluff Analysis Section 41: The entire shoreline of Section 41, located at the Wisconsin Electric Power Company Lakeside Plant, was protected by an outer rubble mound breakwater, and a rubble revetment. A field inspection of these structures was made in the spring of 1988. The outer breakwater was reported to have deteriorated during the recent period of high lake levels. As a result, the revetment was subjected to greater wave heights than those for which it was designed. In places where rubble had been mounded higher, the revetment was effective; otherwise, overtopping had occurred and toe erosion was observed. The existing structures must be repaired and properly maintained to insure continued bluff stability in this section.

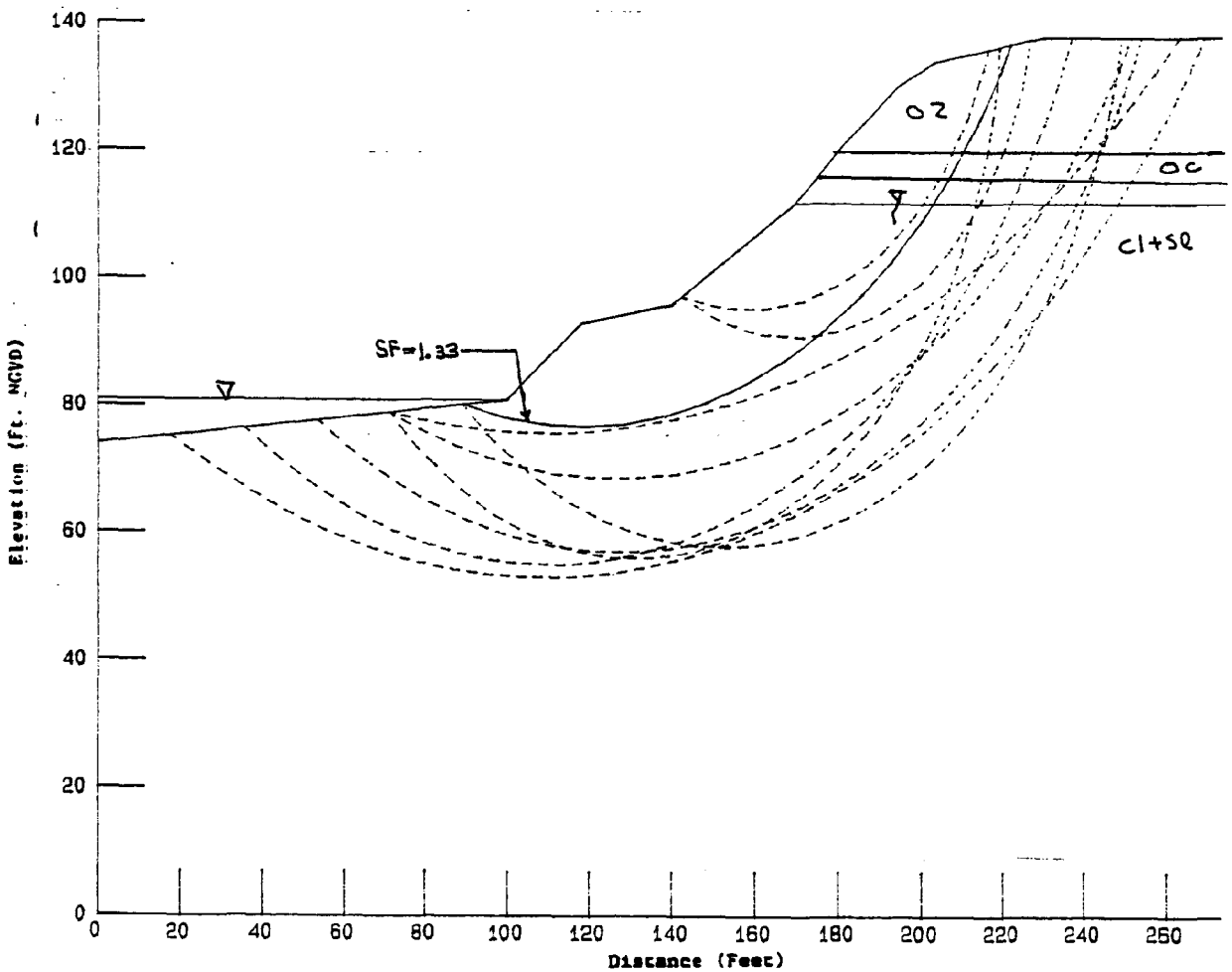
Bluff Analysis Section 42: The stability of the bluff within Section 42, which extends from the Wisconsin Electric Power Company lakeside breakwater to Packard Avenue, was characterized by the use of Profile No. 47.

The results of the deterministic slope stability analysis, shown in Figure III-52, indicate that Profile No. 47 had 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 remaining nine lowest safety factors ranged from 1.40 to 1.59. Based on the deterministic slope stability analysis and on the observed bluff conditions, Section 42 was considered to have a stable bluff slope with respect to rotational sliding.

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

Figure III-52

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 47



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

the steeper sections of bluff slope, especially in the southern end of this section. Translational sliding may have occurred at these locations.

Bluff toe erosion was slight in Section 42. A concrete rubble revetment extended from the power company breakwater in the south through the northern end of the section. The revetment offered sufficient toe protection.

In order to maintain full stability of the bluff in this section, the existing bluff toe protection structure must be maintained. To retard the tendency to fail by translational sliding in the southern end of this section, revegetation of the disturbed soil areas should be considered.

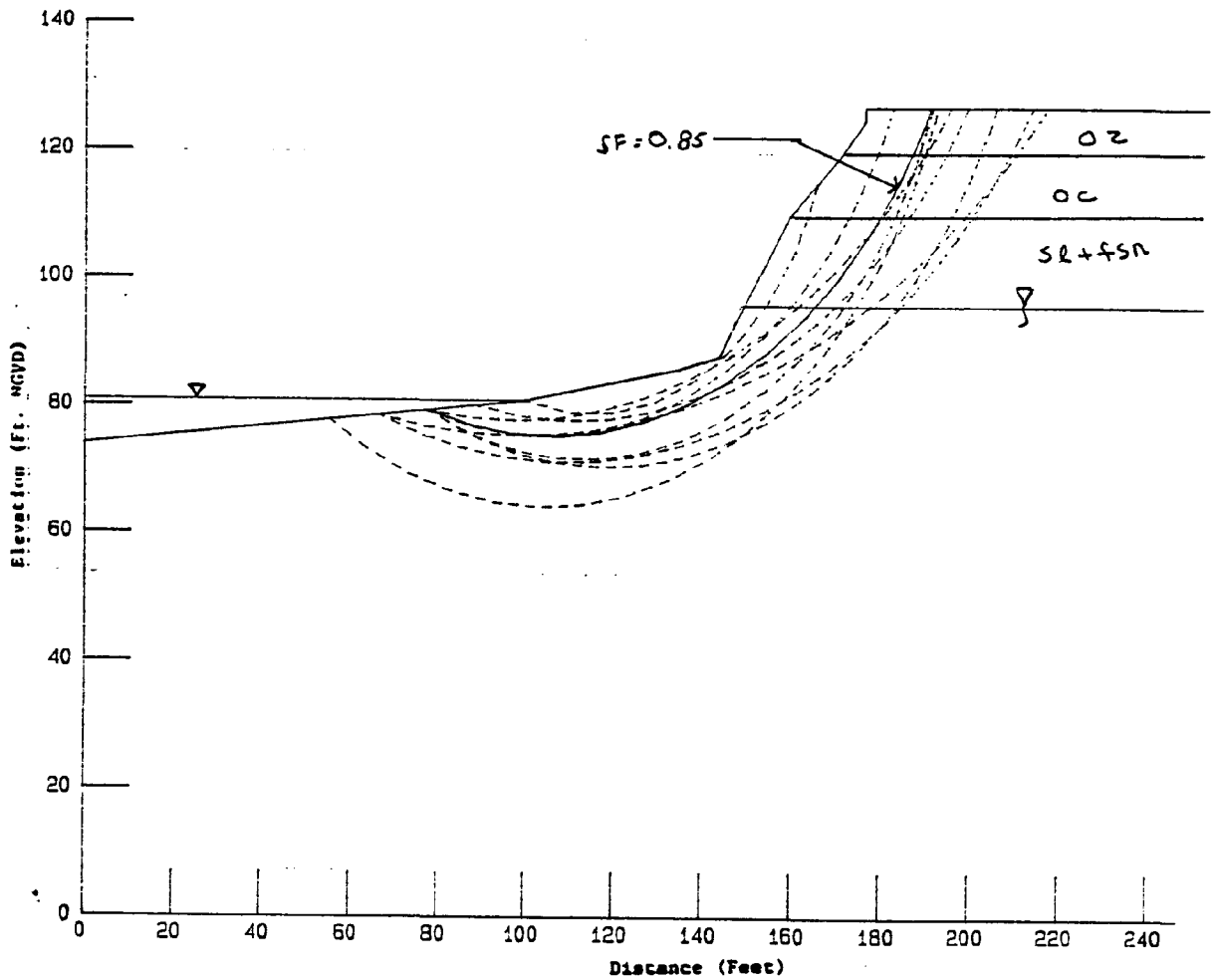
Bluff Analysis Section 43: The stability of the bluff within Section 43, located in Bay View Park, was characterized by the use of Profiles No. 48, 49, and 50.

The results of the deterministic stability analyses, shown in Figures III-53, III-54, and III-55 for Profiles No. 48, 49, and 50, respectively, indicate that Section 43 had 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 remaining 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 included the entire bluff slope. The remaining 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 range of the nine remaining lowest safety factors was 0.83 to 1.07.

Of the 20 probabilistic stability analyses conducted for Profile No. 48, the lowest safety factors ranged from 0.74 to 1.15, with 13, or 65 percent, having a safety factor less than 1.0. Of the total of 200 failure surfaces evaluated at this profile site, 47, or 23 percent of the surfaces, 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 0.50 to 1.15, with 19, or 95 percent, having a safety factor less than 1.0. Of the 200 failure surfaces evaluated at Profile No. 49, 90, or 45 percent of the surfaces, had safety factors less than 1.0. Of the 20 probabilistic stability analyses conducted at Profile No. 50, the lowest safety factors ranged from 0.52 to 1.04, with

Figure III -53

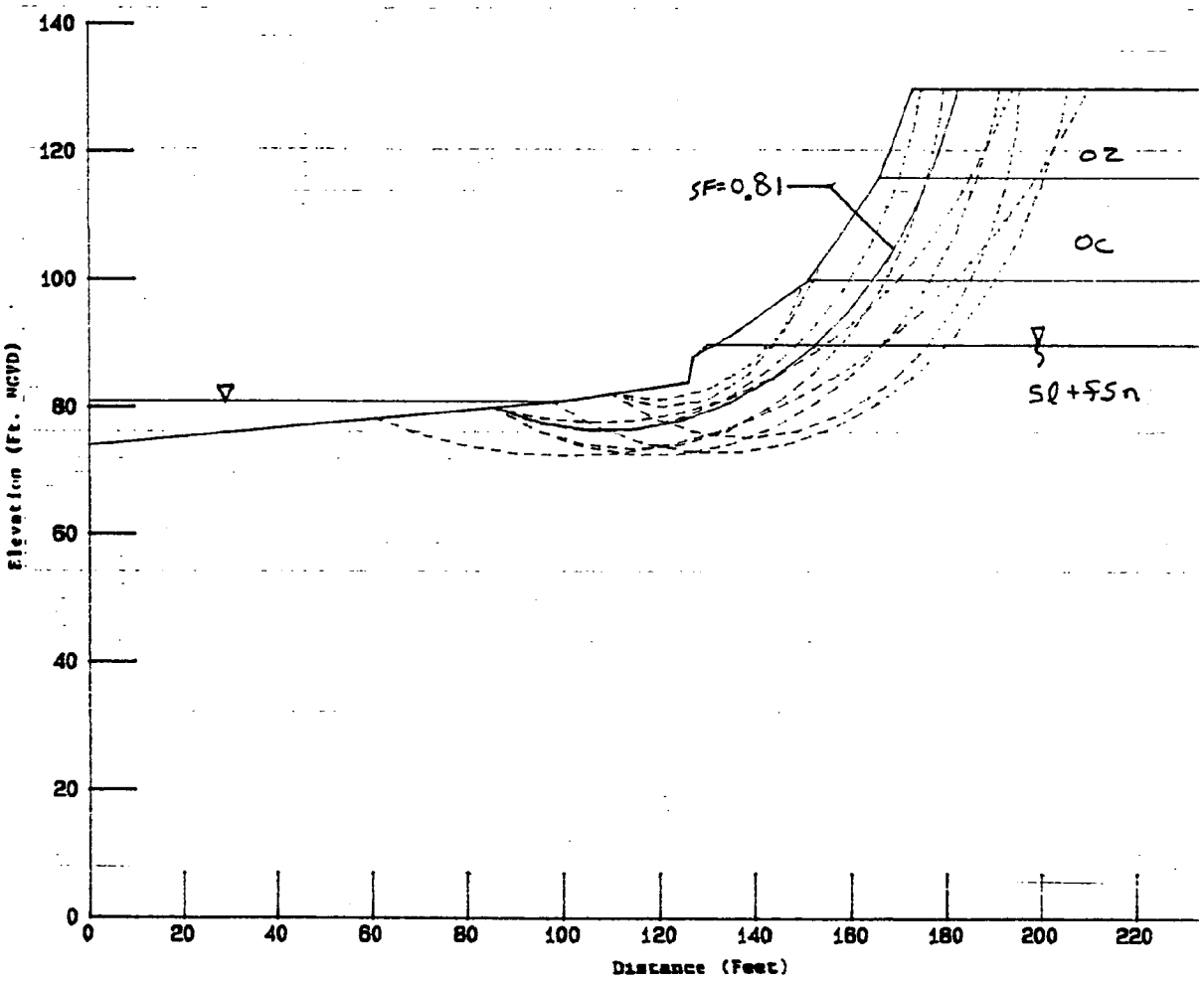
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 48



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

Figure III -54

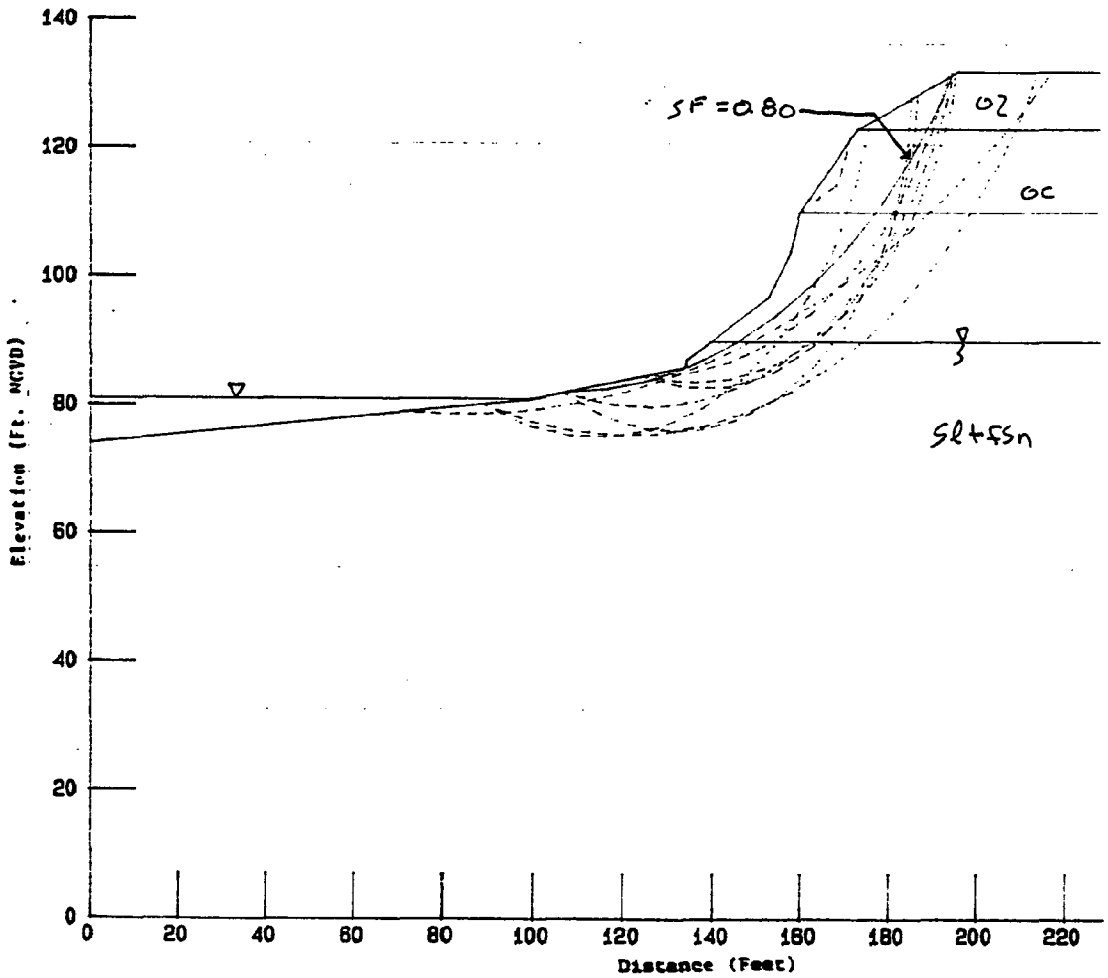
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 49



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

Figure III-55

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 50



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

16, or 80 percent, having a safety factor less than 1.0. Of the total of 200 failure surfaces evaluated at this profile site, 88, or 44 percent of the surfaces, had safety factors 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 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 field survey. This appeared to be a significant factor, contributing to the overall instability of the bluff. As of 1986, no shoreline protection structures wee present in this section.

In order to fully stabilize the bluff in Section 43, shoreline protection measures are needed. Both bluff toe protection and bluff slope regrading are suggested.

Bluff Analysis Section 44: The stability of the bluff in Section 44, located in Bay View Park, was characterized by the use of Profile No. 51.

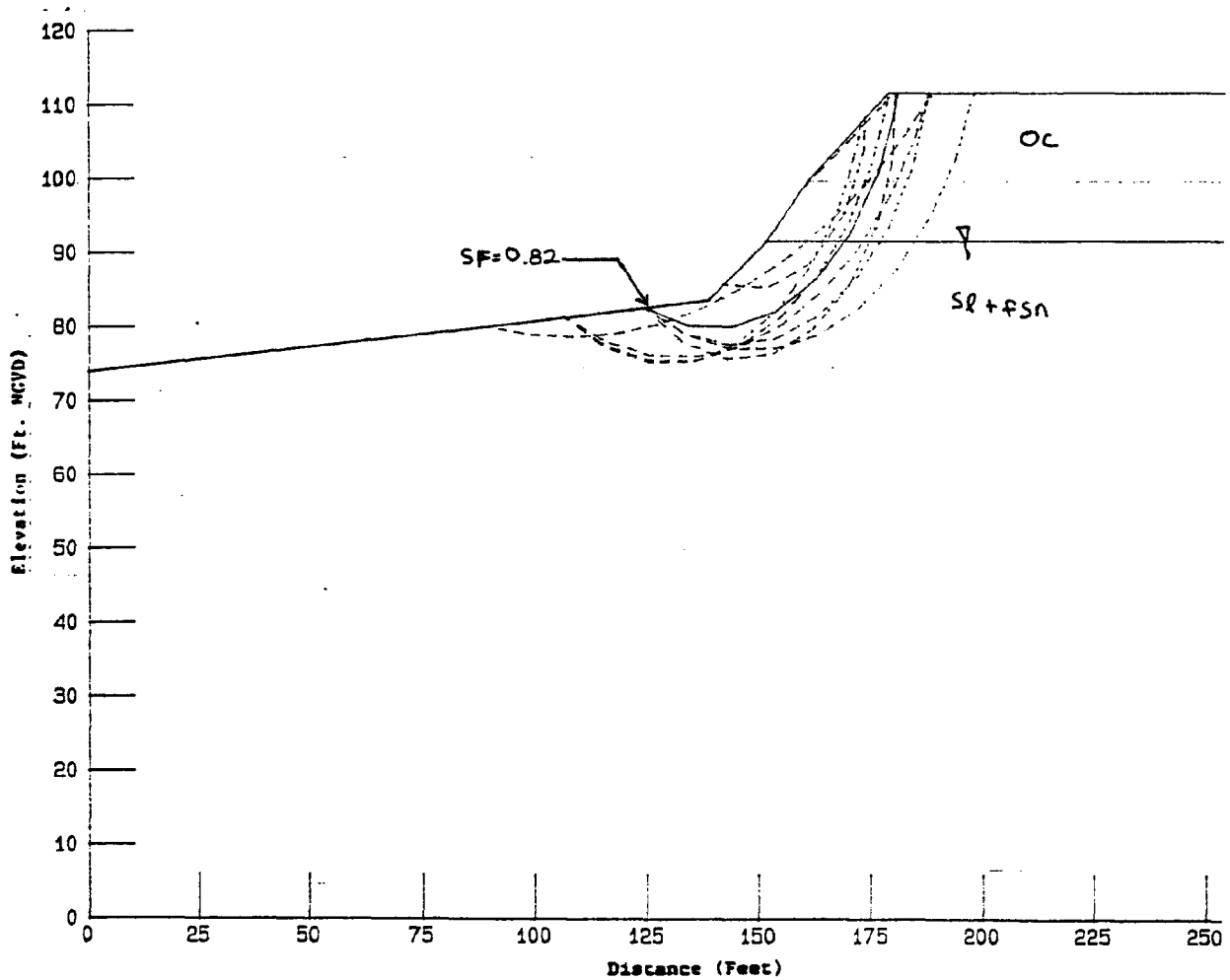
The results of the deterministic slope stability analysis, shown in Figure III-56, indicate that Profile No. 51 had an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.82 and included the entire bluff slope. The remaining nine lowest safety factors ranged from 0.89 to 1.11.

Based on the deterministic slope stability analysis and on observed field conditions, Section 44 was considered to have an unstable slope with respect to rotational sliding.

Section 44 was considered to have an unstable slope with respect to translational sliding. This is particularly true in the lower portion of the bluff slope. Minor seeps observed in this portion of the bluff slope, coupled with

Figure III -56

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 51



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

lack of vegetation and a relatively steep slope angle, all contribute to the tendency to fail by translational sliding in this section.

Moderate bluff toe erosion was observed throughout Section 44 during the field survey conducted during the fall of 1987. It was not reported that this contributed significantly to slope instability in the section. No shoreline protection structures were present in this section at the time of the 1987 field survey.

To fully stabilize the bluff slope in Section 44, both bluff slope regrading and bluff toe protection are needed.

Bluff Analysis Section 45: The stability of the bluff in Section 45, located in Bay View Park, was characterized by the use of Profile No. 52.

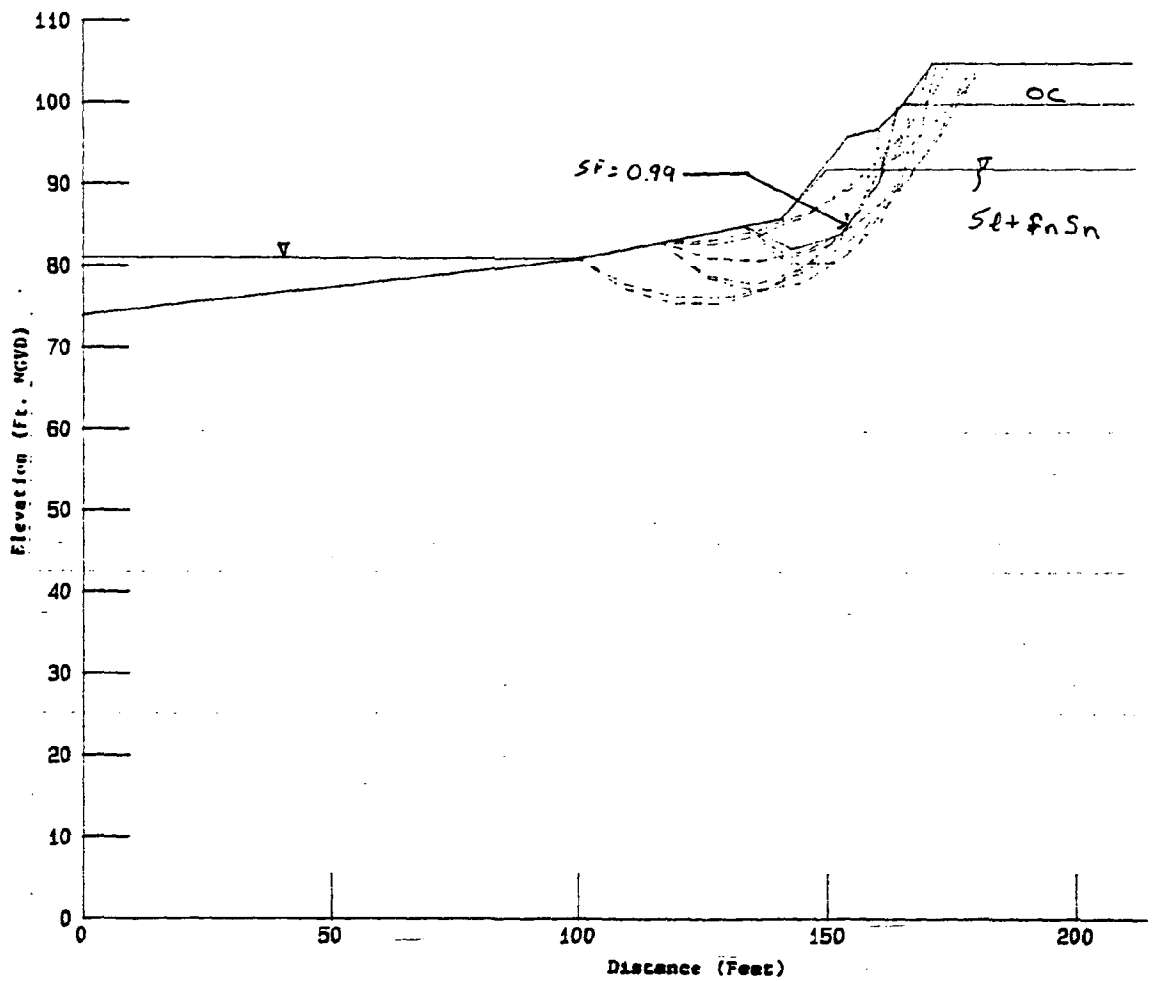
The results of the deterministic analysis, shown in Figure III-57, indicate that Profile No. 52 had a marginally unstable slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.99, and was located in the middle third of the bluff slope. The remaining nine lowest safety factors ranged from 1.01 to 1.23.

Of the 20 probabilistic slope stability analyses conducted for Profile No. 52, the lowest safety factors ranged from 0.53 to 1.64, with 10, or 50 percent, having a safety factor less than 1.0. Of the 200 failure surfaces evaluated, 27, or 13 percent of the surfaces, had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analysis, 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 be marginally unstable with respect to translational sliding. During the field survey conducted during the fall of 1987, evidence of numerous shallow slides in the thin layer of fill which covered the bluff slope were observed. Steepness of the bluff slope, as well as lack of vegetative cover, were considered the major causes of bluff failure by translational sliding in this section.

Figure III -57

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 52



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

Bluff toe erosion in Section 45 was moderate, and did not contribute significantly to bluff slope instability. No shoreline protection structures were present in Section 45 at the time of the fall 1987 field survey.

In order to fully stabilize the bluff in Section 45, bluff toe protection and bluff slope regrading are necessary.

Bluff Analysis Section 46: The stability of the bluff in Section 46, located in Bay View Park, was characterized by the use of Profile No. 53.

The results of the deterministic slope stability analysis, shown in Figure III-58, indicate that Profile No. 53 had a marginally unstable 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 remaining nine lowest safety factors ranged from 0.98 to 1.26.

Of the 20 probabilistic stability analyses conducted, the lowest safety factors ranged from 0.75 to 1.48, with six, or 30 percent, having a safety factor less than 1.0. Of the 200 failure surfaces evaluated, nine, or 4 percent of the surfaces, 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.

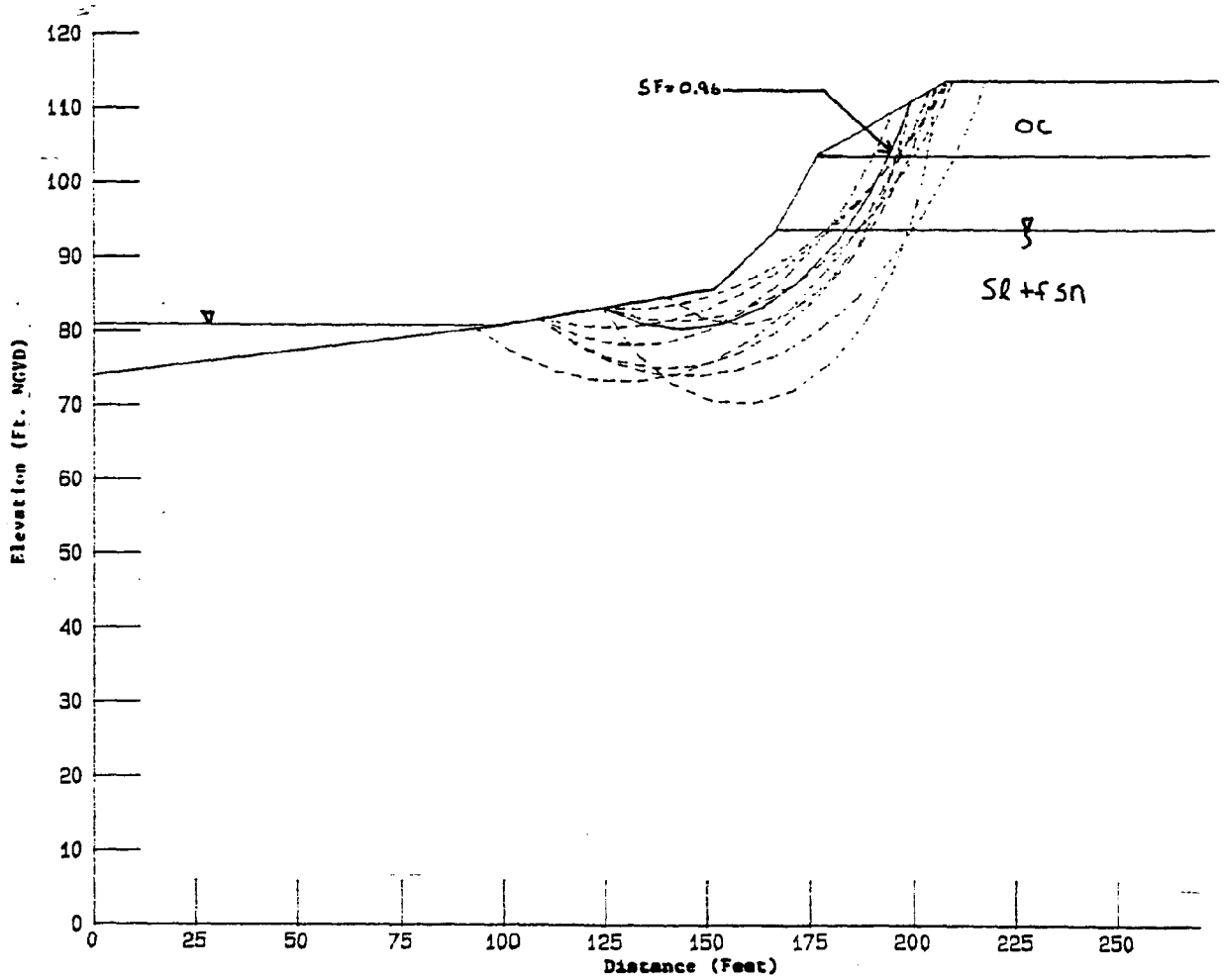
Section 46 was also considered to have a marginal bluff slope with respect to translational sliding. This was due to the relatively steep angle of the bluff slope and sparse vegetation.

Moderate toe erosion was observed in Section 46 during the fall of 1987 survey. Toe erosion was not considered a significant factor in bluff instability evaluation.

Bluff toe protection and bluff slope regrading are necessary to fully stabilize the bluff in Section 46.

Figure III-58

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 53



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

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 III-59, indicate that Profile No. 54 had 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 remaining nine lowest safety factors ranged from 1.19 to 1.43.

Based on the deterministic slope stability analysis, and on the observed bluff conditions, Section 47 was considered to have a stable bluff slope with respect to rotational sliding.

Section 47 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 the very good vegetative cover.

Roughly 95 percent of Section 47 was located inside the South Shore breakwater. As a result of this protection, a wide, sand beach and small sand terrace had built up along the entire length of shoreline in Section 47.

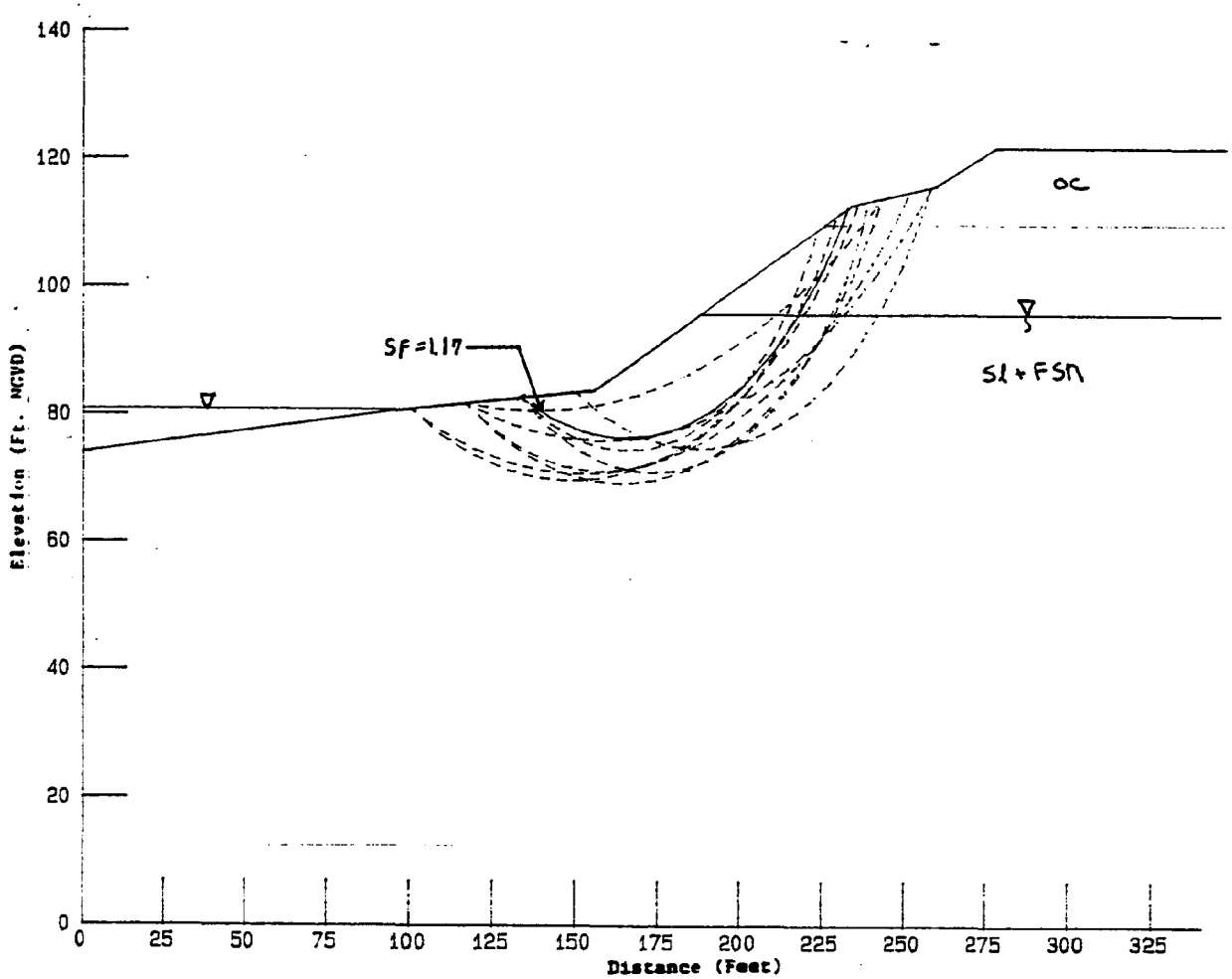
No bluff toe erosion was observed during the field survey conducted in the fall of 1987.

Maintenance of the existing structure is imperative to ensure continued bluff stability in Section 47.

Bluff Analysis Section 48: The stability of the bluff in Section 48, located in South Shore Park, was characterized by the use of Profile No. 55.
The results of the deterministic slope stability analyses, shown in Figure III-60, indicate that Profile No. 55 had 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 in the upper two-thirds of the bluff slope. The remaining nine lowest safety factors ranged from 1.28 to 1.48.

Figure III -59

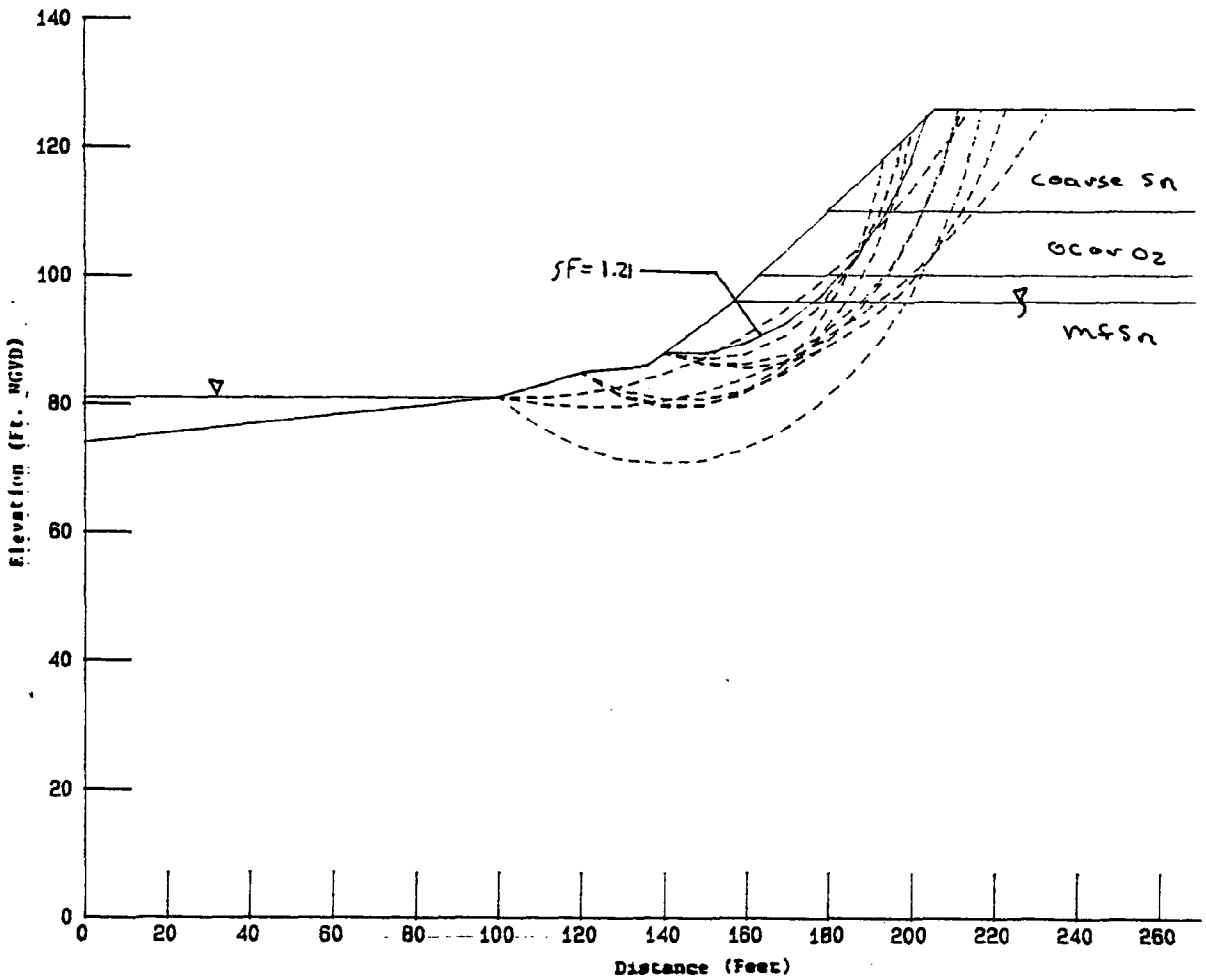
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 54



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

Figure III -60

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 55



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

Of the 20 probabilistic stability analyses conducted, the lowest safety factors ranged from 1.04 to 1.56. In fact, all 200 of the surfaces evaluated had safety factors greater than 1.0. 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 also had a stable slope with respect to translational sliding. The bluff slope was very well vegetated and has a gentle slope angle. The entire section was inside the South Shore breakwater, which appeared effective in decreasing the effects of erosion due to wave action.

The entire length of shoreline in this section was also protected by a rip-rap revetment. Bluff toe erosion in Section 48 was reported only where the revetment had been overtopped. This was not extensive, therefore, bluff toe erosion was not considered to negatively impact bluff stability in this section.

Maintenance of both the South Shore breakwater and the rip-rap revetment is necessary to ensure continued stability of the bluff in Section 48.

Bluff Analysis Section 49: The entire shoreline of Section 49, located at the Texas Street water intake, was protected by a rip-rap revetment. The natural bluff has been extended 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.

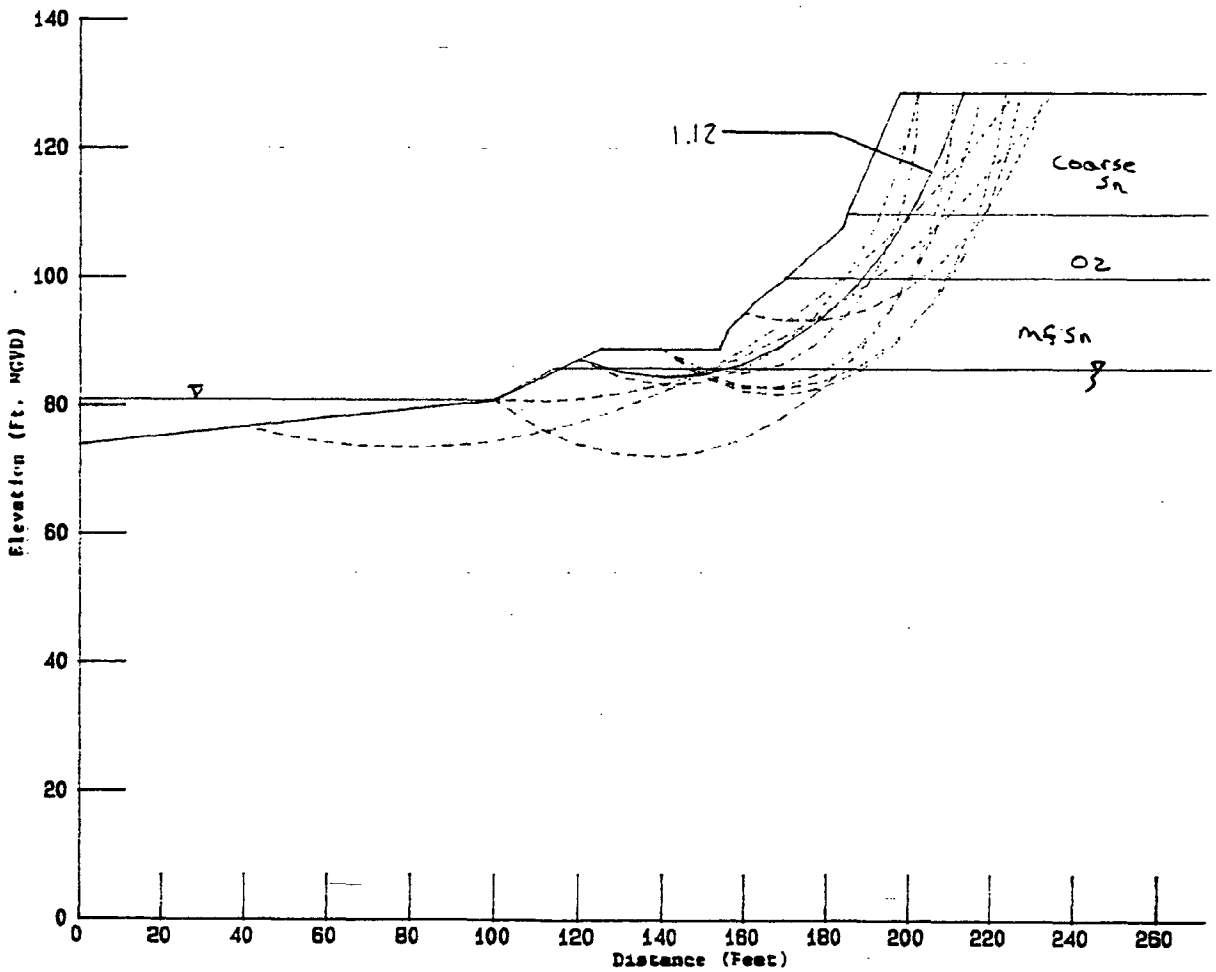
Some damage to the rip-rap on the north side of the structure was observed in the spring of 1988. Maintenance of damage such as this is necessary to ensure bluff stability in this section.

Bluff Analysis Section 50: The stability of the bluff slope within Section 50, located in South Shore Park, was characterized by the use of Profile No. 56.

The results of the deterministic slope stability analysis, shown in Figure III-61, indicate that Profile No. 50 had 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, and was located within the upper two-thirds

Figure III -61

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 56



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

of the bluff slope. The remaining nine lowest safety factors ranged from 1.28 to 1.48.

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 0.78 to 1.49, with five failure surfaces, or 25 percent, having a safety factor of less than 1.0. Of the total 200 failure surfaces evaluated, six, or 3 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 50 was considered to have a stable bluff slope with respect to rotational sliding.

Overall, Section 50 was considered to have an unstable bluff slope with respect to translational sliding. This was due to the steepness of the bluff slope.

The entire shoreline of Section 50 was protected by a concrete rubble rip-rap revetment. No obvious signs of damage to the structure during a field survey conducted in the spring of 1988. No toe erosion of the bluff in this section was observed either in the spring of 1988 survey or in a previous field survey conducted during the fall of 1987.

Bluff toe protection is integral in maintaining bluff stability in Section 50. To this end, the shoreline protection structure in place should be inspected periodically and repaired as necessary.

Bluff Analysis Section 51: The entire shoreline of Section 51, located in South Shore Park, was protected by the South Shore breakwater and the South Shore Park revetments. The natural bluff has been graded to a stable slope.

During a field survey conducted during the spring of 1988, moderate erosion landward of the revetment was observed. This was believed to have occurred as a result of overtopping during the recent episode of elevated lake levels.

The present structure must be properly repaired to ensure maximum shoreline protection.

Bluff Analysis Section 52: The entire shoreline of Section 52, located at South Shore Beach, was protected by the South Shore breakwater and a wide sand beach. The natural bluff has been graded to a stable slope. No erosion of the bluff was observed during the field survey conducted during the fall of 1987. Maintenance of the existing structure is necessary to ensure the continuance of the shoreline protection currently provided.

Bluff Analysis Section 53: The entire shoreline of Section 53, which is located at the South Shore Yacht Club, is inside the South Shore breakwater. The shoreline in the section is protected by an interlocking steel bulkhead or concrete rubble revetment. The natural bluff had been graded to a very gentle angle. No additional shoreline protection measures are needed in Section 53; existing structures need only be maintained.

Bluff Analysis Section 54: The entire shoreline of Section 54, which is located at South Shore Park, is inside the South Shore breakwater. The U. S. Coast Guard Station bulkhead and revetment further protect the shoreline from wave action. The natural bluff slope has been graded to a very gentle angle. At this time, maintenance of the existing shoreline protection structures will ensure continued effectiveness.

Bluff Analysis Section 55: Section 55 extends from E. Russell Avenue to the Jones Island sewage treatment plant and is located within the Milwaukee Harbor breakwater. A variety of structures line the shoreline within Section 55. The dredged material storage site revetment covers 30 percent of the shoreline. Approximately 25 percent of the shoreline is protected by the S. Lincoln Memorial Drive bulkhead. Outdoor harbor slips protect 30 percent. Finally, the Jones Island sewage treatment plant protects 15 percent. During a field survey conducted during the spring of 1988, these structures were examined and found to be in good repair. Continued maintenance of these structures will ensure their continued effectiveness and shoreline protection measures.

Bluff Analysis Section 56: Section 56 extends from the Marcus Amphitheater to McKinley Marina, and is located within the Milwaukee Harbor breakwater. A

number of structures are located along the shoreline within this section. The Summerfest bulkhead and revetment protect 15 and 20 percent of the shoreline, respectively. Another 10 percent is protected by the Municipal Pier bulkhead. The War Memorial bulkhead covers 10 percent and the McKinley Marina landfill bulkhead 20 percent. The McKinley Marina protects 25 percent of the shoreline. A field survey of these structures, conducted during the spring of 1988, reported no problems with appearance or effectiveness of each structure. Current maintenance practices should be continued to prevent degradation of the structures and, ultimately, loss of the shoreline protection afforded by them.

Bluff Analysis Section 57: The entire shoreline of Section 57 is protected by revetments. The McKinley Beach Project, a revetment and pocket beach system effectively protects 80 percent of this section; the North Point revetment protects the remaining 20 percent. These structures must be maintained to ensure continued shoreline protection.

Bluff Analysis Section 58: The entire shoreline of Section 58 is protected by Bradford Beach, a sand beach nearly 200 feet in width.

Bluff Analysis Section 59: The entire shoreline of Section 59, located in Lake Park, is protected by a revetment. Erosion behind the structure, evidence of overtopping by waves, was noted during the field survey conducted during the spring of 1988. Evaluation of the damage to the structure, as well as repair of the structure, are needed to restore maximum protection. A plan for future maintenance is also advised to prevent another future degradation of the structures.

Bluff Analysis Section 60: The entire shoreline of Section 60 is protected by the Linnwood Avenue purification plant bulkhead. A field inspection of the structure was conducted during the spring of 1988. Deterioration of the concrete slab which tops the bulkhead, a double-walled, armor stone-lined design structure, was reported. Also, damage due to waves overtopping the structure was noted in the site perimeter fence and in the asphalt access road. Finally, the vertical sheet pile wall, the primary protection structure in this section, was reported to be of suspect longevity. It was not clear if the wall was tied back to a competent existing structure. The concrete sea wall

directly in front of the building itself was reported to be in relatively good condition. Clearly, this section needs to be thoroughly evaluated, and a plan for repair and maintenance developed if this section is to continue to be protected from the effects of wave and ice erosion.

Bluff Analysis Section 61: The stability of the bluff slope within Section 61, which extends from the Linwood Purification Plant to 3052 E. Newport Court, was characterized by the use of Profile No. 57.

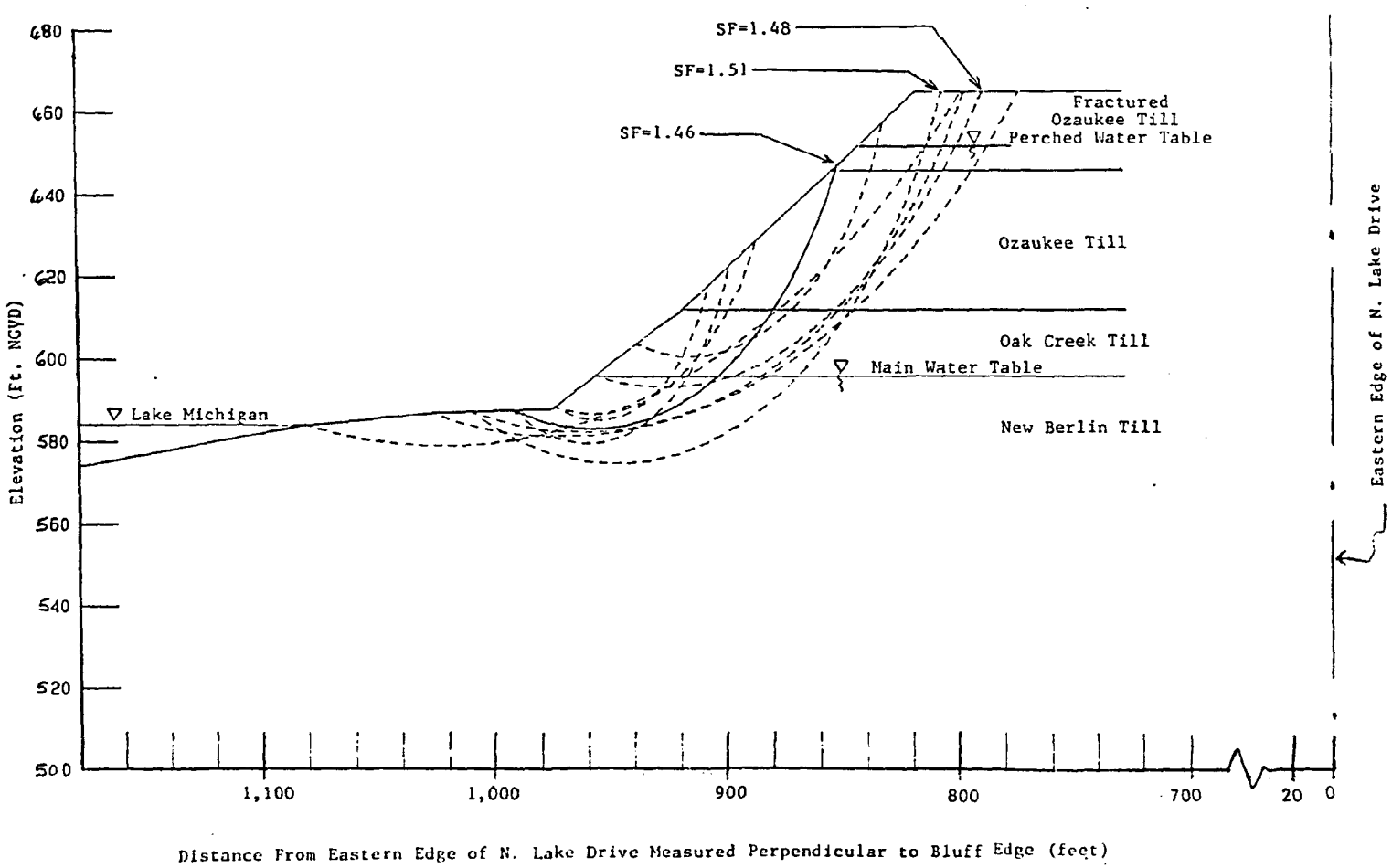
The results of the deterministic slope stability analysis, shown in Figure III-62, indicate that Profile No. 57 had a stable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 1.46, and was located within the lower two-thirds of the bluff slope. The remaining 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 total of 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.

Figure III-62

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 57:3252 N. LAKE DRIVE



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

Due primarily to the relatively wide beach built up in Section 61, no significant bluff toe erosion was observed during the field surveys 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. If the lake levels would return to the mean 20th Century levels, it is anticipated that the resulting beach within most of Section 61 would approximate 60 to 100 feet in width.

No measures are needed to prevent rotational sliding within Bluff Analysis Section 61. Revegetation of the scattered disturbed soil areas within the upper portion of the bluff slope is recommended to prevent the occurrence of translational sliding. 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 62: The stability of the bluff slope within Section 62, which extends from 3378 to 3474 N. Lake Drive, was characterized by the use of Profile No. 58.

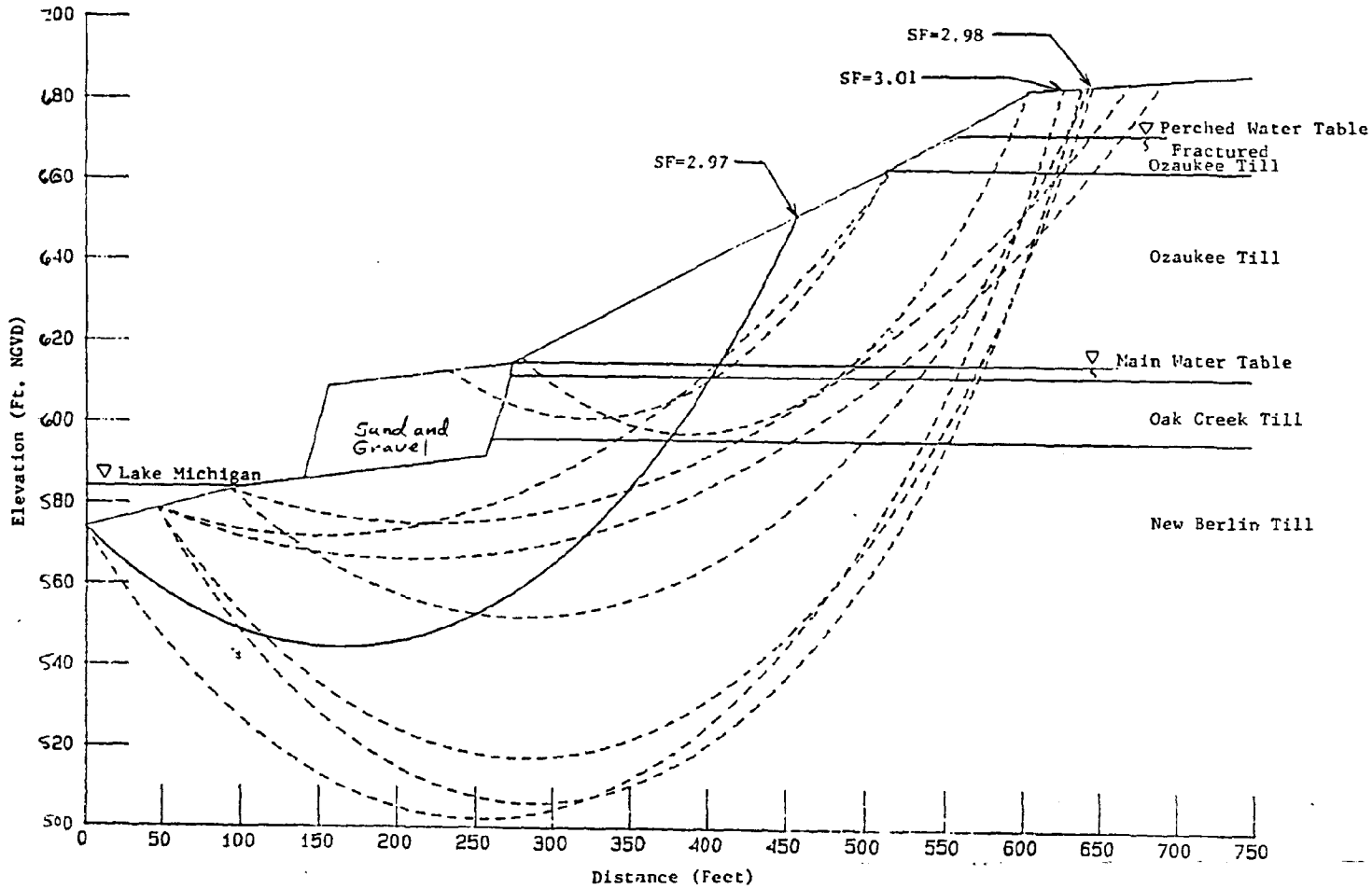
The results of the deterministic slope stability analysis, shown in Figure III-63, indicate that Profile No. 58 had a stable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 2.97, and was located within the lower two-thirds of the bluff slope. The remaining nine lowest safety factors ranged from 2.98 to 3.13.

Of the 20 probabilistic stability analyses conducted, the lowest safety factors 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

Figure III-63

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 58:100 FEET NORTH OF NEWPORT AVE.



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

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 surveys 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 at the present time. Should erosion of the fan continue, the attendant risk of the toe erosion affecting the overall stability of the bluff would increase. If the lake levels would return to the mean 20th Century levels, it is anticipated that the resulting beach within Section 62 would approximate 30 feet in width.

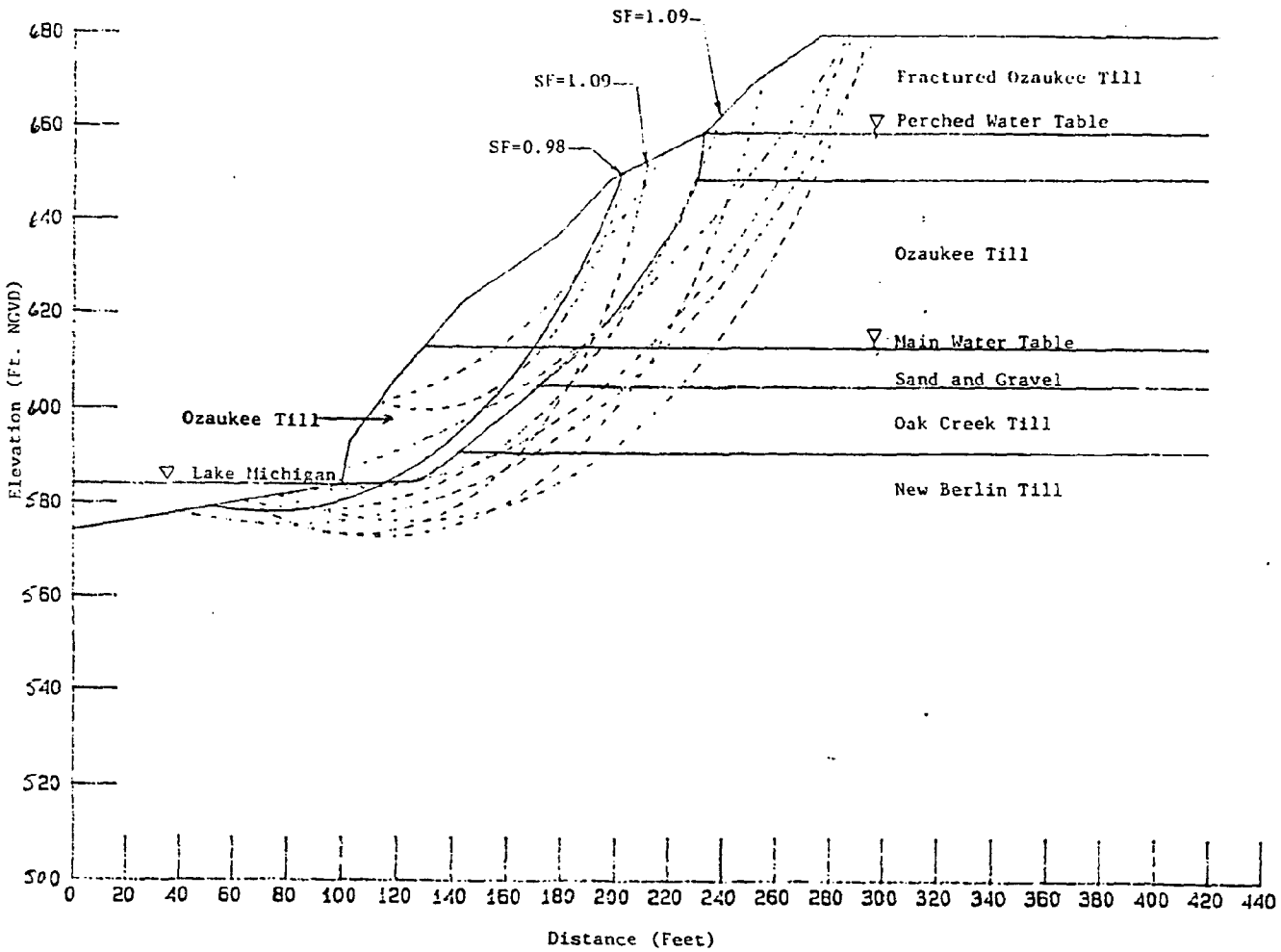
No measures are needed to prevent rotational sliding within Bluff Analysis Section 62. Surface runoff control and the establishment of a good vegetative cover on the land which was cleared is recommended within this section, especially on the steep ravine slopes, to prevent the occurrence of translational sliding. Bluff toe protection is recommended to prevent the erosion by wave and ice action.

Bluff Analysis Section 63: The stability of the bluff slopes within Section 63, which is located at 3510 N. Lake Drive, was characterized by the use of Profile No. 59 and Profile No. 60.

The results of the deterministic slope stability analyses, shown in Figure III-64 and Figure III-65 for Profile No. 59 and Profile No. 60, respectively, indicate a potential 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 slope within old slump block material. The remaining 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-

Figure III-64

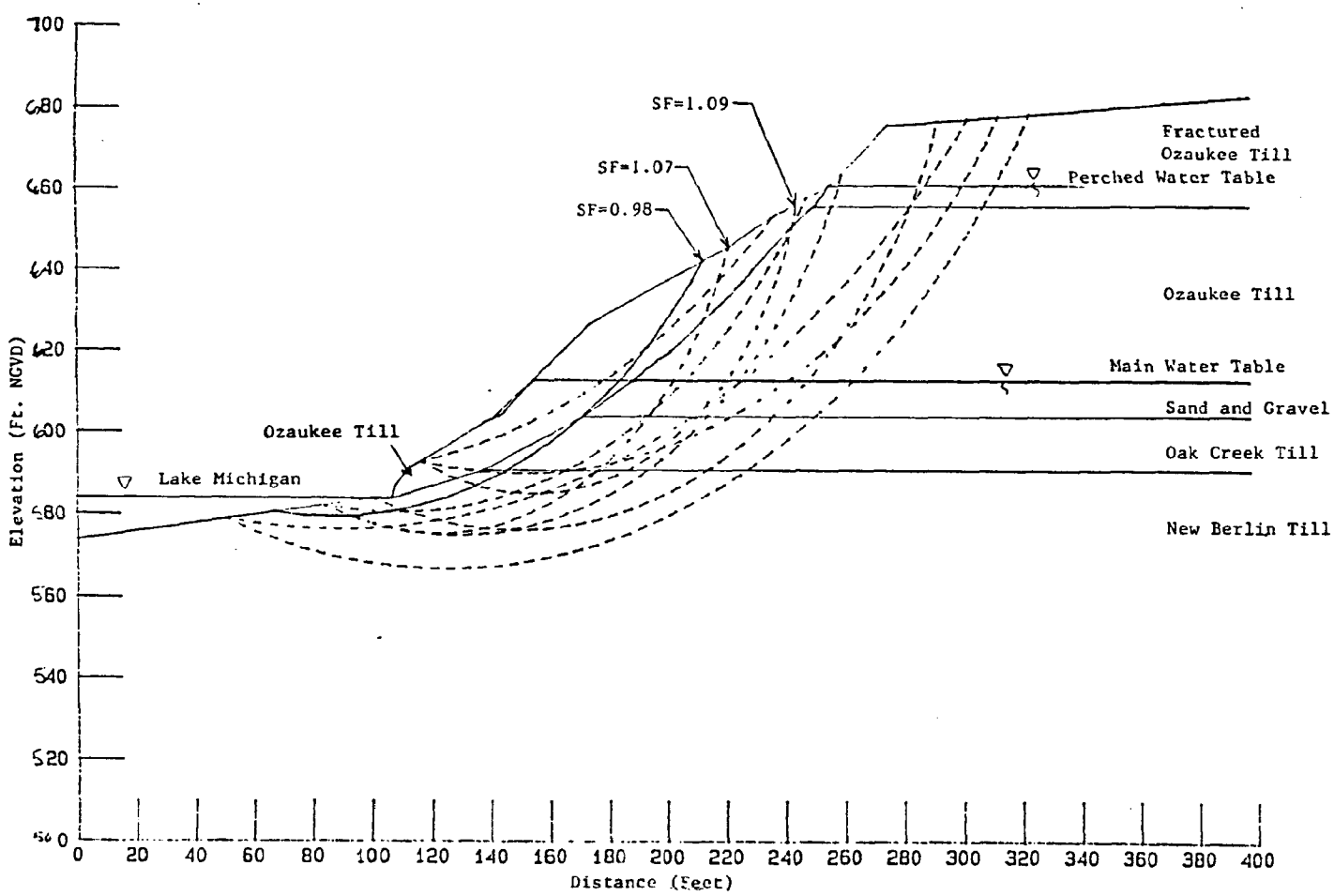
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 59: 3510 N. LAKE DRIVE



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

Figure III-65

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 60: 3510 N. LAKE DRIVE



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

thirds of the bluff slope. The remaining nine lowest safety factors ranged from 1.07 to 1.38.

Of the 20 probabilistic stability analyses conducted for Profile No. 59, the lowest safety factors ranged from 0.62 to 1.08, with 13, or 65 percent, having a safety factor less than 1.0. Of the total of 200 failure surfaces evaluated at Profile No. 59, 63, or 32 percent of the surfaces, had safety factors of less than 1.0. Of the 20 probabilistic stability analyses conducted for Profile No. 60, the lowest safety factors ranged from 0.81 to 1.15, with 11, or 55 percent, having a safety factor of less than 1.0. Of the total of 200 failure surfaces evaluated at Profile No. 60, 29, or 15 percent of the surfaces, had safety factors less than 1.0.

During the field surveys 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 surveys 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 covered the lower portion of the slope. Shore protection structures present in the section in 1986 included one concrete bulkhead covering a total of 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. If the lake levels would return to the mean 20th Century levels, it is anticipated that the resulting beach within Section 63 would approximate 10 to 20 feet in width.

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, 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 would not be damaged by slight movement of the slump block. Toe protection should be provided along the entire shoreline of the section. Maintenance of would 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 vegetation cover on the entire bluff slope is recommended to prevent the occurrence of translational sliding.

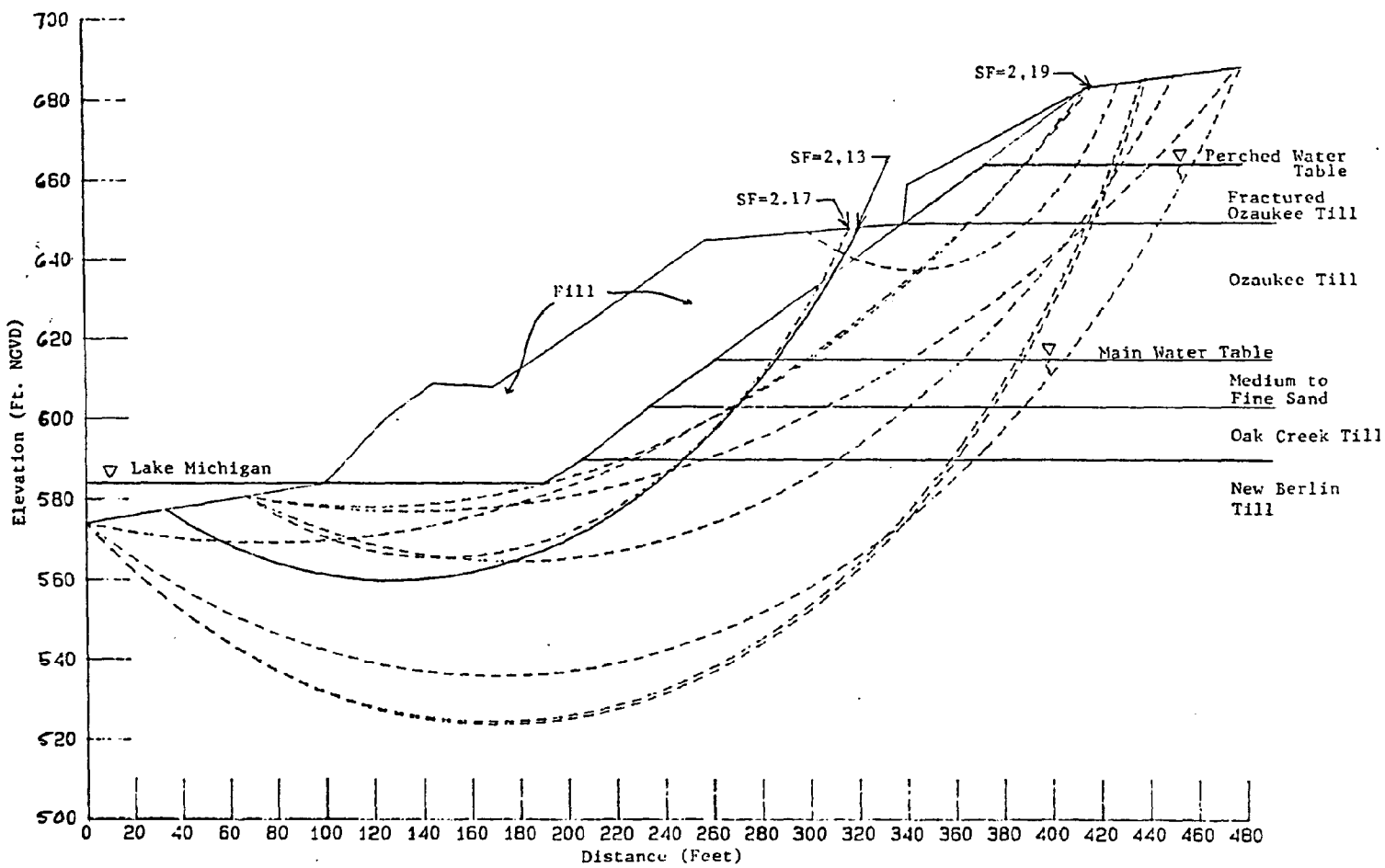
Bluff Analysis Section 64: The stability of the fill and the underlying bluff slope within Section 64, which is located at 3534 N. Lake Drive, was characterized by the use of Profile No. 61.

The results of the deterministic slope stability analysis for Profile No. 61, are shown in Figure III-66. 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 remaining nine lowest safety factors ranged from 2.17 to 2.31. The 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

Figure III-66

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 61: 3534 N. LAKE DRIVE



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

of the benefits of 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.

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 surveys 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. Even if the lake levels would return to the mean 20th Century levels, it is not anticipated that a significant beach would develop within Section 64.

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 towards 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.

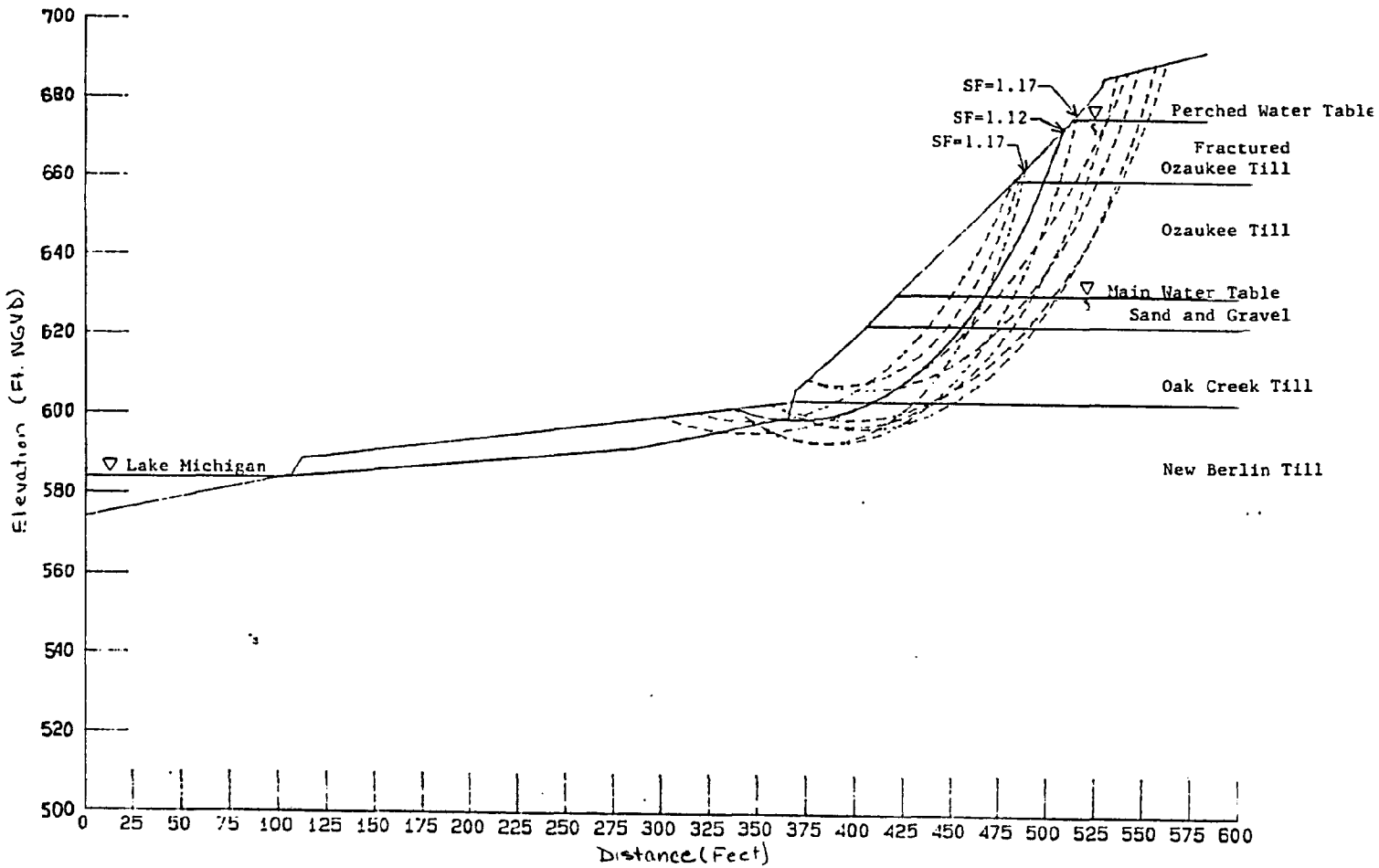
Bluff Analysis Section 65: The stability of the bluff slope within Section 65, which extends from 3550 to 3914 N. Lake Drive, was characterized by the use of Profile No. 62.

The results of the deterministic slope stability analysis, shown in Figure III-67, indicate that Profile No. 62 had 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 remaining nine lowest safety factors ranged from 1.17 to 1.25.

Of the 20 probabilistic stability analyses conducted, the lowest safety factors 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 total of 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

Figure III- 67

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 62:3704 N. LAKE DRIVE



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

probabilistic analysis did indicate that there is 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 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. If the lake levels would return to the mean 20th Century levels, it is anticipated that the resulting beach within Section 65 would approximate 10 to 20 feet in width.

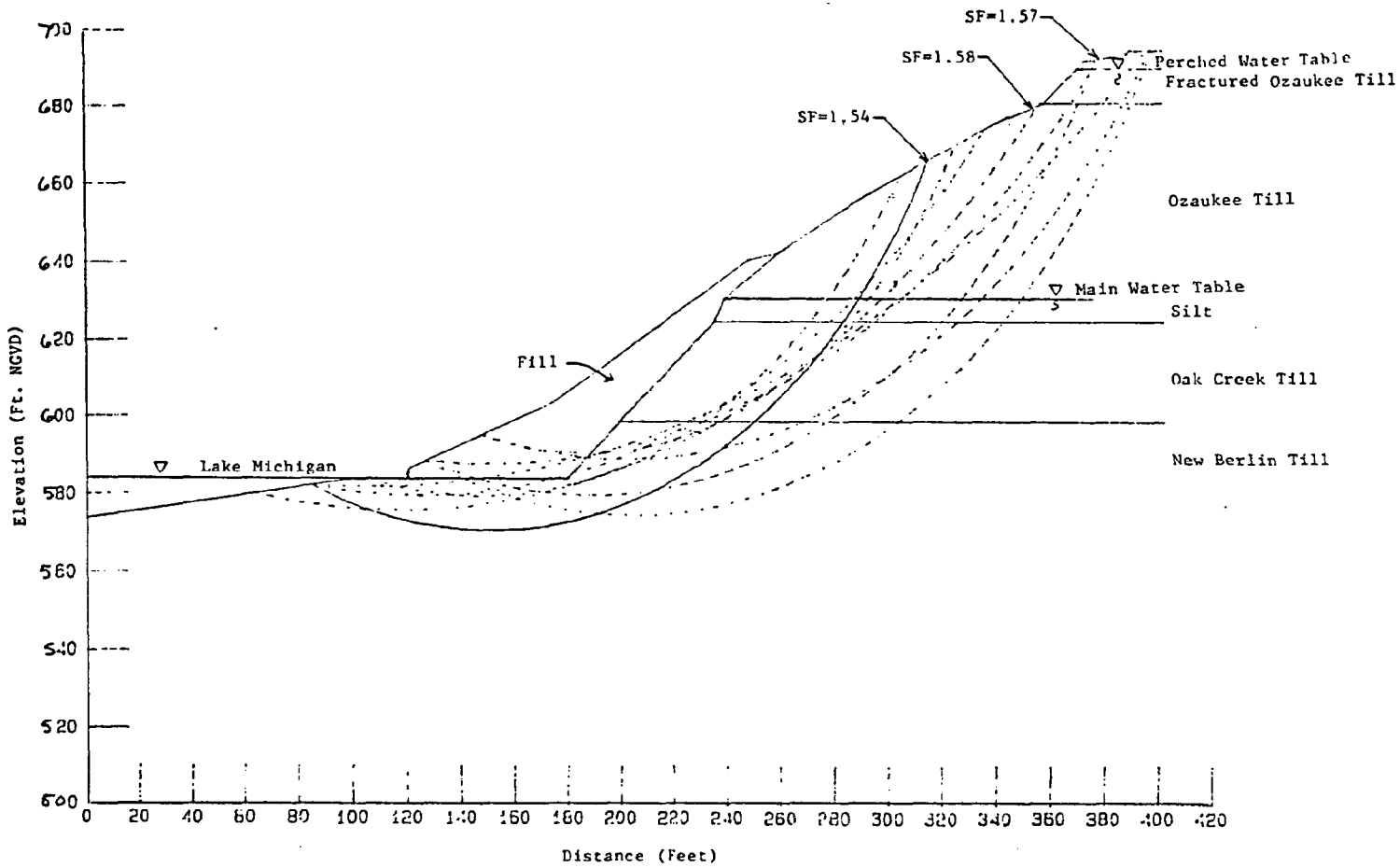
No measures are needed to prevent rotational or translational sliding within Bluff Analysis Section 65. Bluff toe protection is recommended to protect the terraced portion of the section, which includes the Shorewood Nature Preserve.

Bluff Analysis Section 66: The stability of the fill and the underlying bluff slope within Section 66, which is located at 3926 N. Lake Drive, 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 III-68. The lowest failure surface calculated at this profile site had a safety factor of 1.54. The remaining nine lowest safety factors ranged from 1.57 to 1.61. The 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.

Figure III- 68

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 63:3926 N. LAKE DRIVE



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

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 of 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 at the present time. Even if the lake levels would return to the mean 20th Century levels, it is not anticipated that a significant beach would develop within Section 6.

No measures are needed to prevent rotational or translational sliding within Bluff Analysis Section 66. Toe erosion control should include maintenance of the existing concrete block revetment.

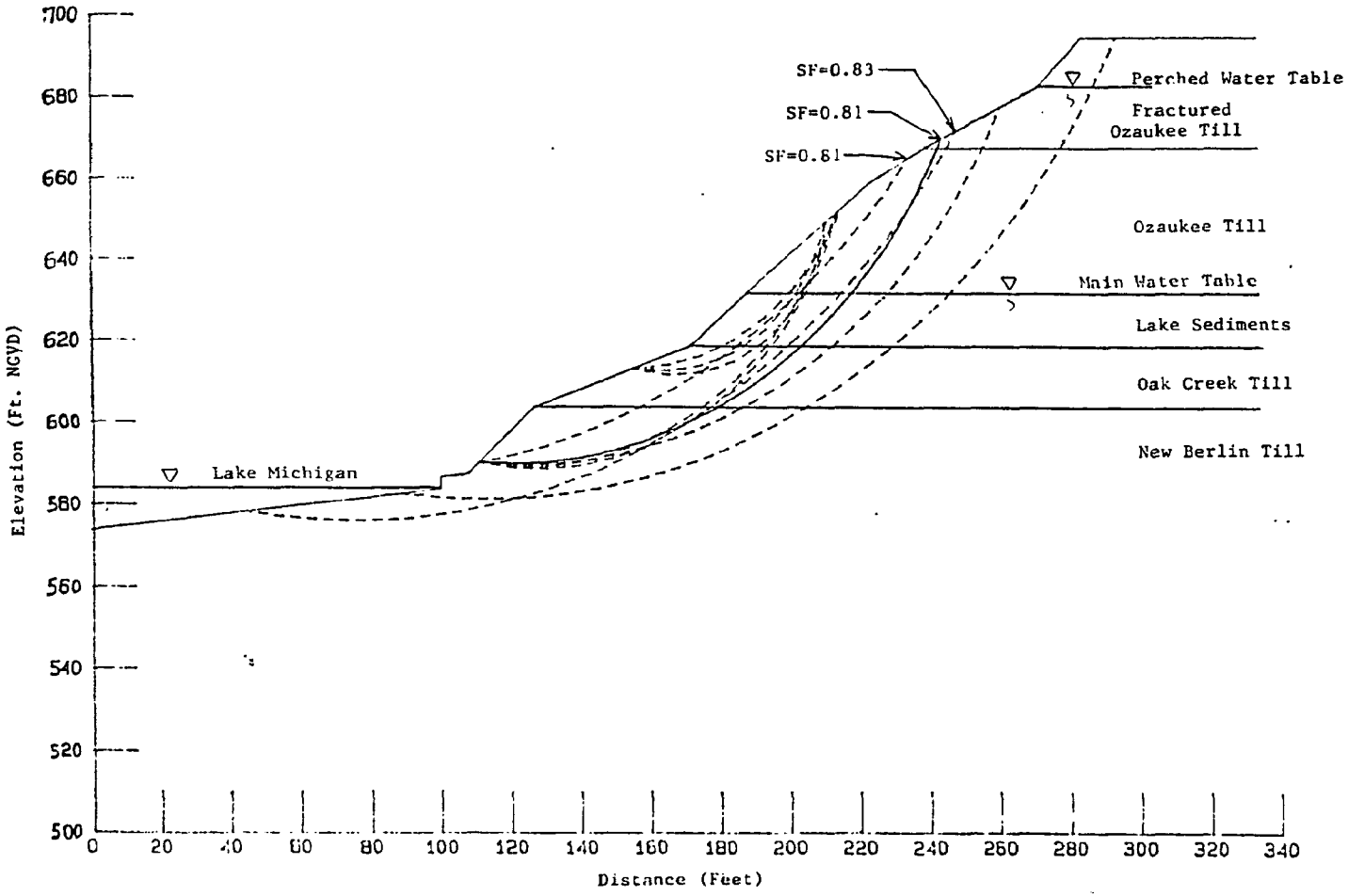
Bluff Analysis Section 67: The stability of the bluff slope within Section 67, which extends from 3932 to 3966 N. Lake Drive, was characterized by the use of Profile No. 64.

The results of the deterministic slope stability analysis, shown in Figure III-69, indicate that Profile No. 64 had an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.81, and was located within the middle portion of the bluff slope. The remaining 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 total of 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

Figure III- 69

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 64:3932 N. LAKE DRIVE



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

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. If the lake levels would return to the mean 20th Century levels, it is anticipated that the resulting beach within Section 67 would be less than 20 feet in width.

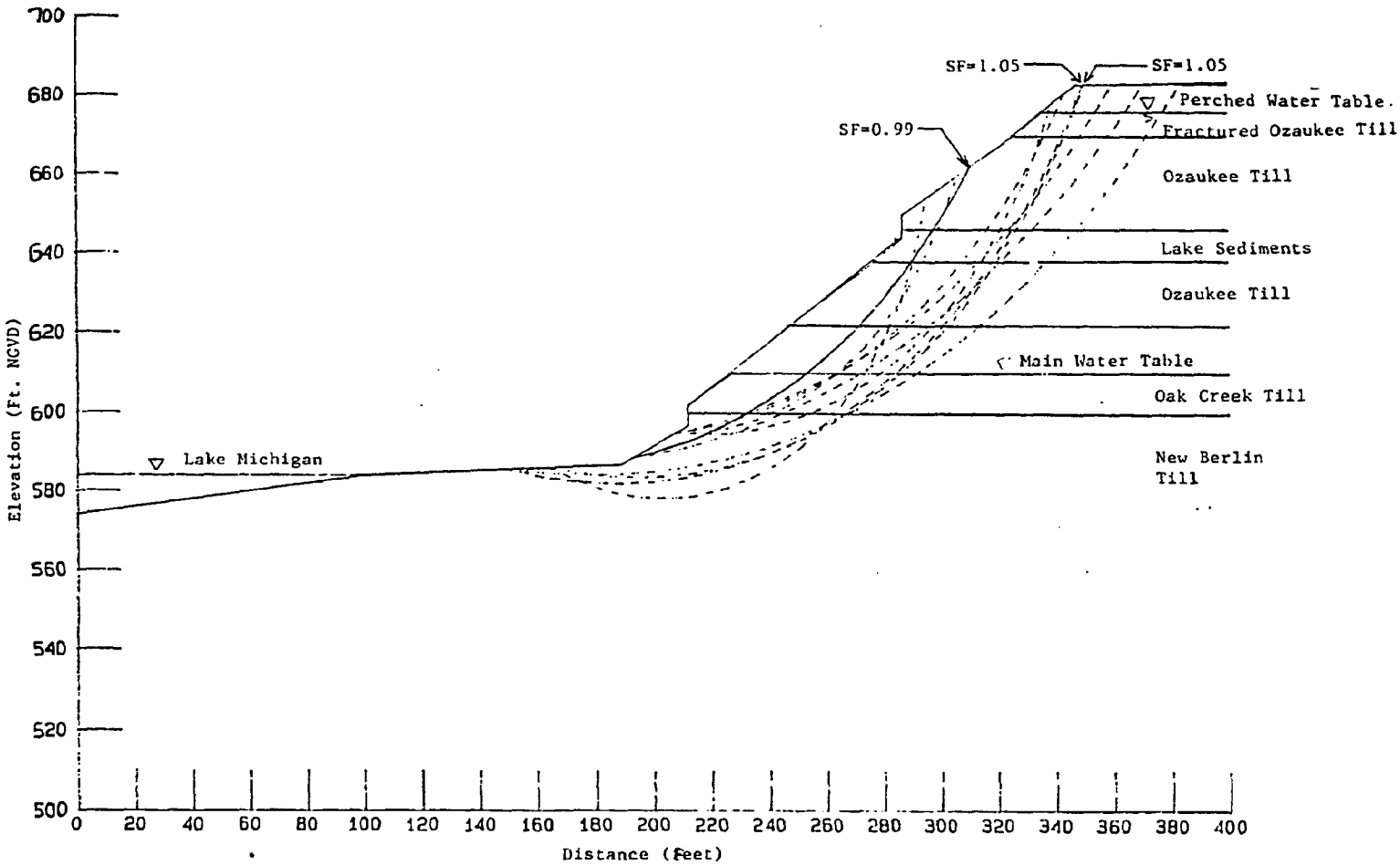
To abate the severe potential for both rotational and translational sliding, it is recommended that the bluff slope be regraded to a stable slope angle. This action may require filling, since cutting back the top of the slope may not be feasible because some houses at the top of the bluff are as close 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, was characterized by the use of Profile No. 65.

The results of the deterministic slope stability analysis shown in Figure III-70, for Profile No. 65 indicate a potential 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 remaining nine lowest safety factors ranged from 1.05 to 1.15.

Figure III-70

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 65:4098 N. LAKE DRIVE



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

Of the 20 probabilistic stability analyses conducted for Profile No. 65, the lowest safety factors ranged from 0.66 to 1.17, with 12 failure surfaces, or 60 percent, having a safety factor less than 1.0. Of the total of 200 failure surfaces evaluated at Profile No. 65, 90, or 45 percent of the surfaces, had safety factors of less than 1.0.

During the 1986 field surveys, 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 surveys 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. If the lake levels would return to the mean 20th Century levels, it is anticipated that the resulting beach within Section 8 north of Atwater Park would approximate 80 feet in width.

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 groundwater elevation. Bluff toe protection is recommended within the northern 1,650 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, was characterized by the use of Profile No. 66.

The results of the deterministic slope stability analysis, shown in Figure III-71, indicate that Profile No. 66 had 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 remaining 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.

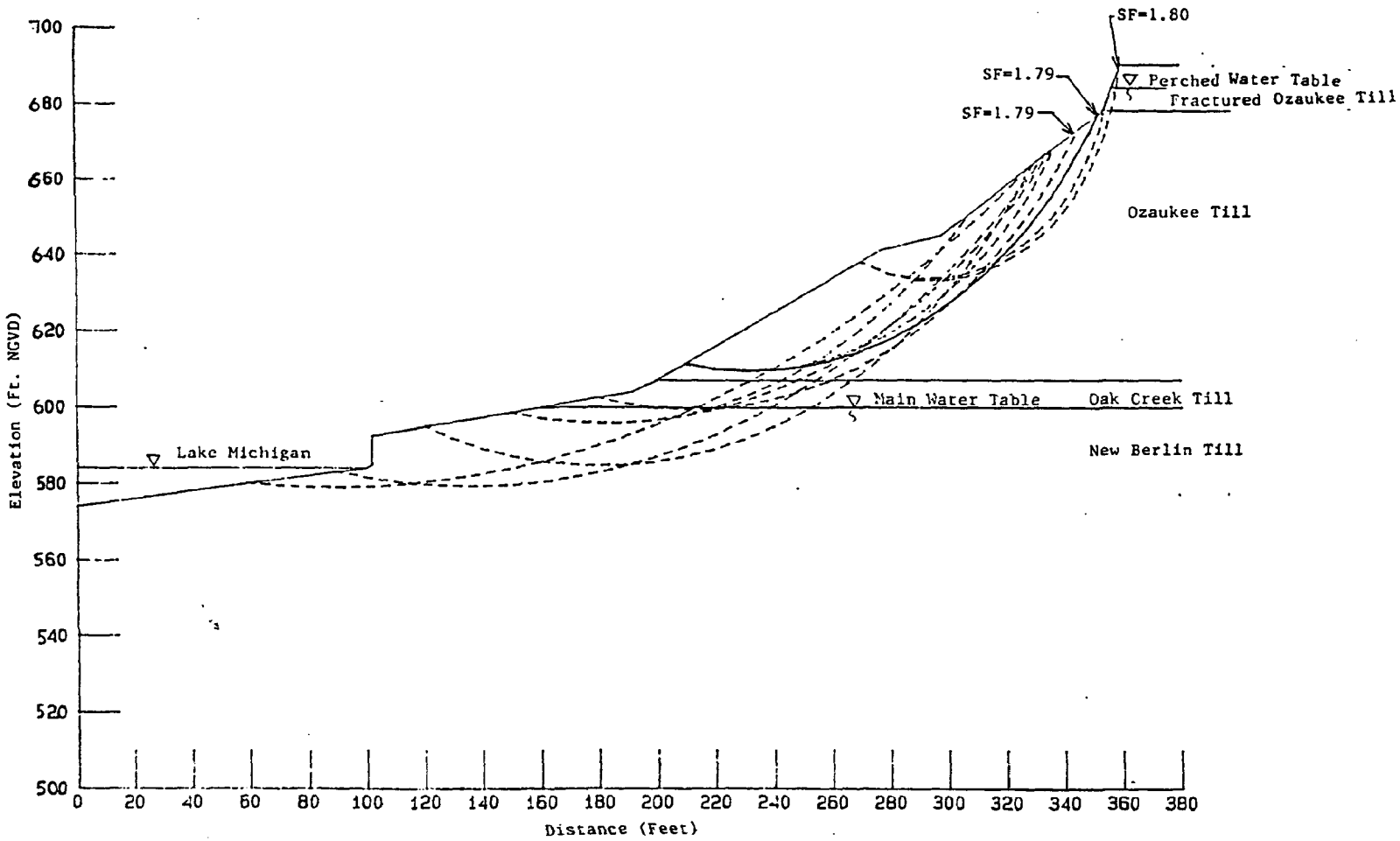
Of the 20 probabilistic stability analyses conducted, the lowest safety factors ranged from 0.74 to 1.99, with only one failure surface, or 5 percent, having a safety factor of less than 1.0. Of the total of 200 failure surfaces evaluated, only four surfaces, or 2 percent, had a safety factor of less 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 about 100 feet in width, the erosion would not be expected to affect the stability of the bluff slope at the present time. 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. If the lake levels would return to the mean

Figure III-71

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 66 : 4308 N. LAKE DRIVE



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

20th Century levels, it is anticipated that the resulting beach within Section 69 would approximate 10 to 20 feet in width.

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.

Bluff Analysis Section 70: The stability of the bluff slope within Section 70, which extends from 4400 to 4408 N. Lake Drive, was characterized by the use of Profile No. 67.

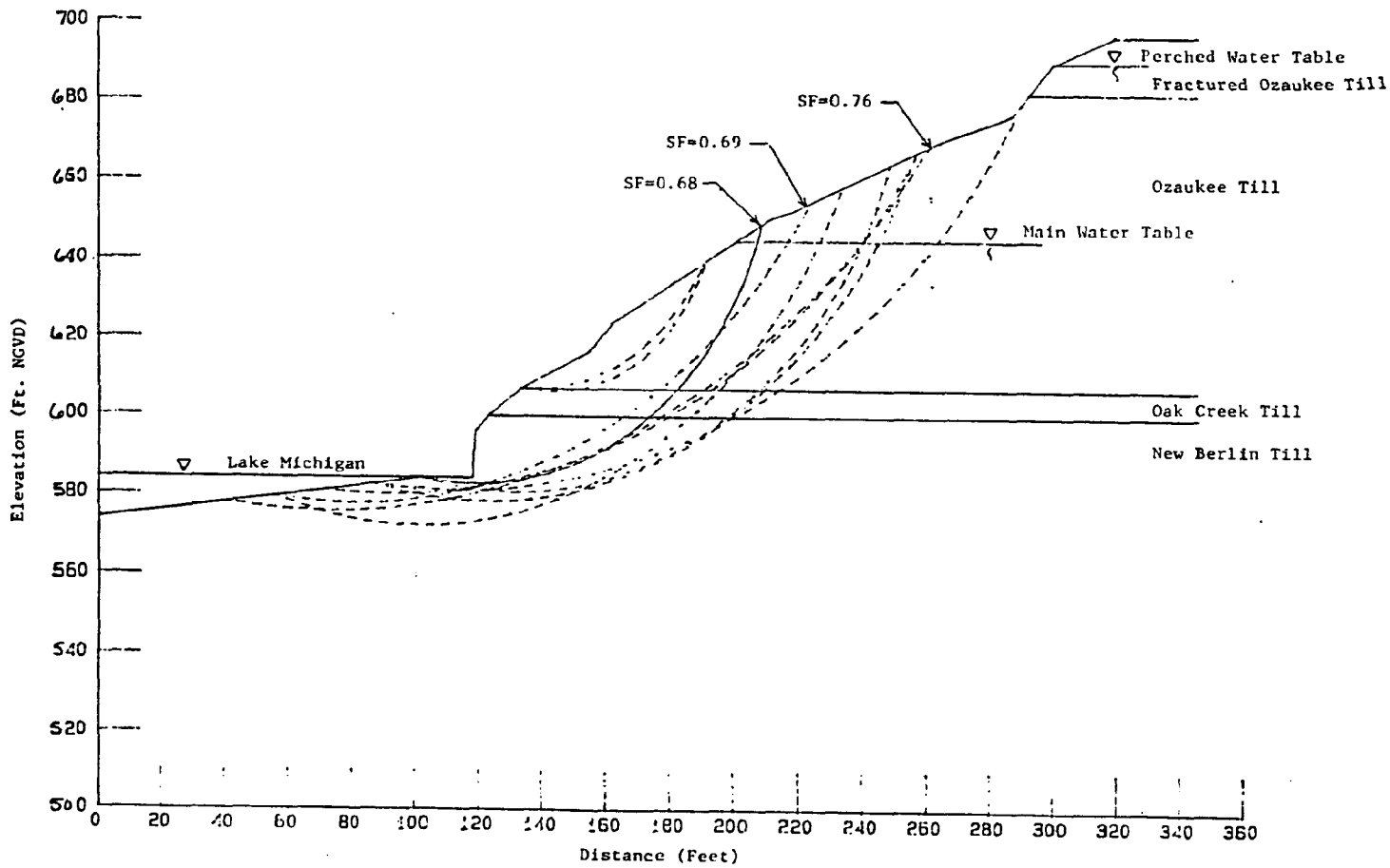
The results of the deterministic slope stability analysis, shown in Figure III-72, for Profile No. 67, indicate that the bluff slope was potentially unstable with respect to rotational sliding. The lowest failure surface calculated at Profile No. 67 had a safety factor of 0.68, and was located on the lower half of the bluff. The remaining nine lowest safety factors ranged from 0.69 to 0.88.

Of the 20 probabilistic stability analyses conducted for Profile No. 67, the lowest safety factors ranged from 0.61 to 0.97. Of the total of 200 failure surfaces evaluated at Profile No. 67, 160, or 80 percent of the surfaces, had safety factors of less than 1.0.

In the field surveys conducted in the Summer of 1986, the overall bluff slope within Section 70 was well vegetated, although some slope movement had previously occurred, and some soil areas were exposed. The elevation of the groundwater shown in Figure III-72 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 has been 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

Figure III- 72

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 67 : 4408 N. LAKE DRIVE



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

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 surveys 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. Even if the lake levels would return to the mean 20th Century levels, it is not anticipated that a significant beach would develop within Section 70.

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, especially in the southern portion of the section. 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: 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.

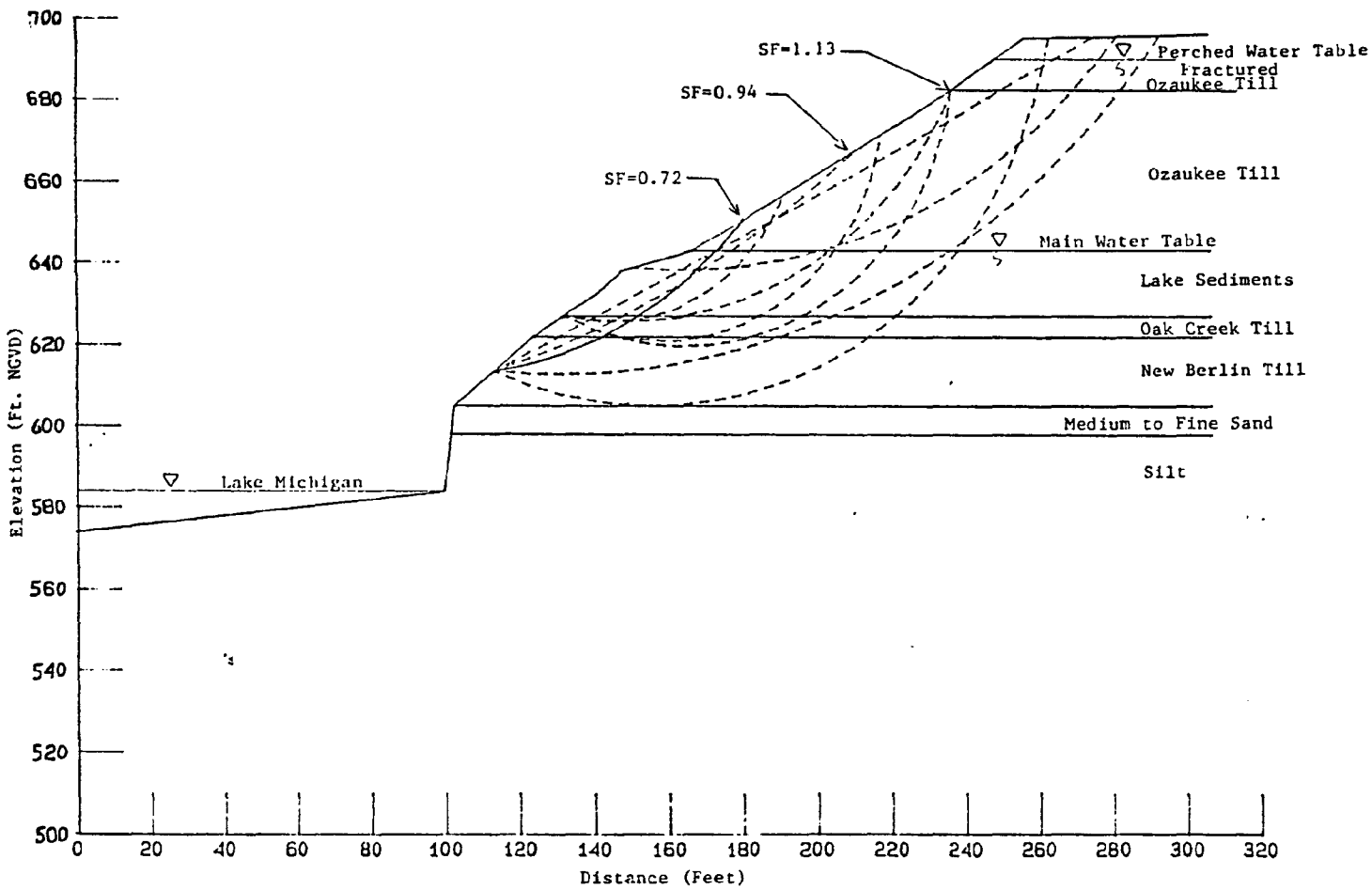
The results of the deterministic slope stability analysis for the prefill conditions shown in Figure III-73, indicate that Profile No. 68 had an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.72. The remaining nine lowest safety factors ranged from 0.94 to 1.38. The results of the deterministic slope stability analyses, shown in Figure III-74, and Figure III-75 for Profile No. 69 and Profile No. 70, respectively, 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 remaining 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 remaining nine lowest safety factors ranging from 2.14 to 3.37. The 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 of 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. Even if the lake levels would return to the mean 20th Century levels, it is not anticipated that a significant beach would develop within Section 71.

Figure III-73

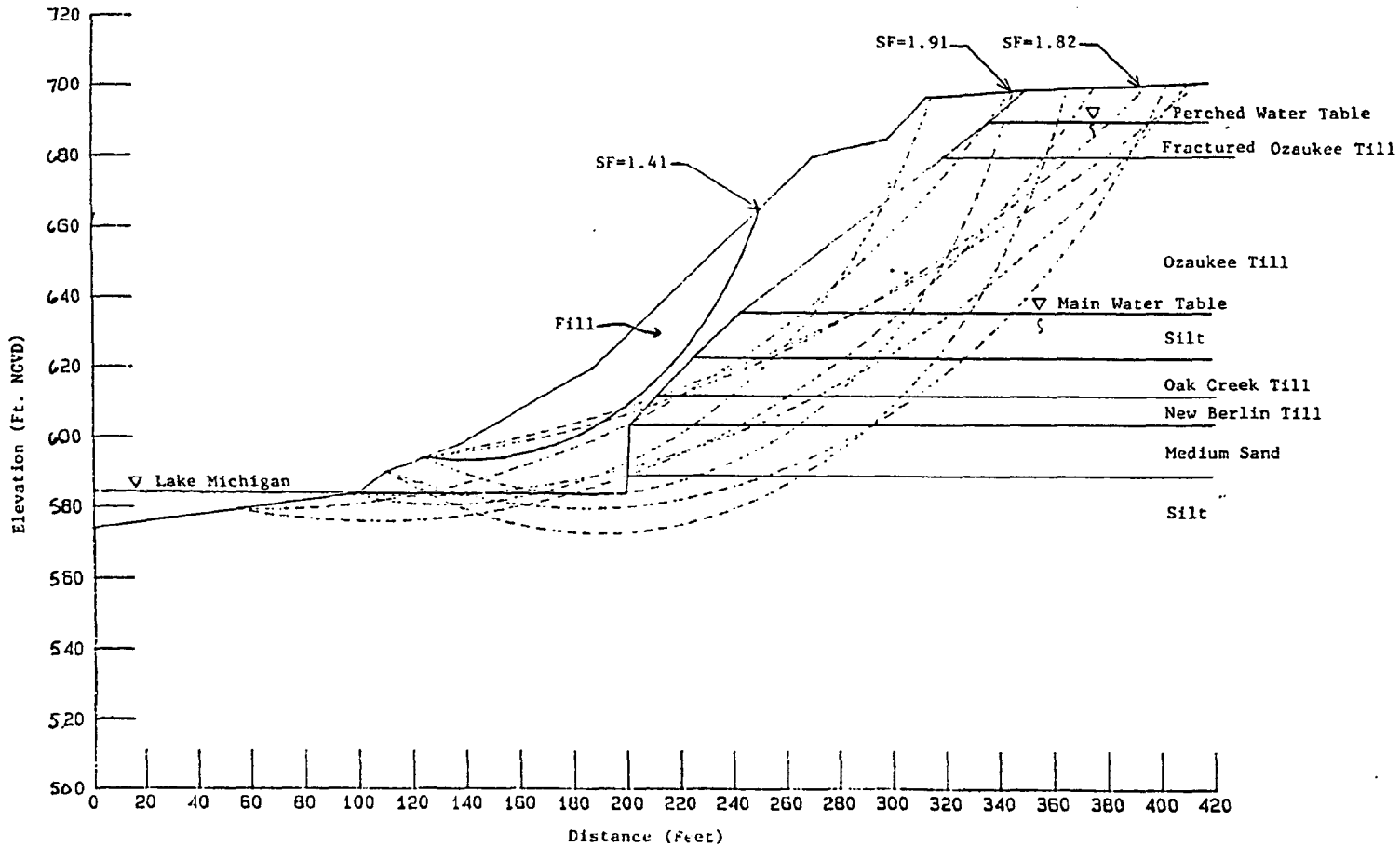
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 68: 4460 N. LAKE DRIVE



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

Figure III-74

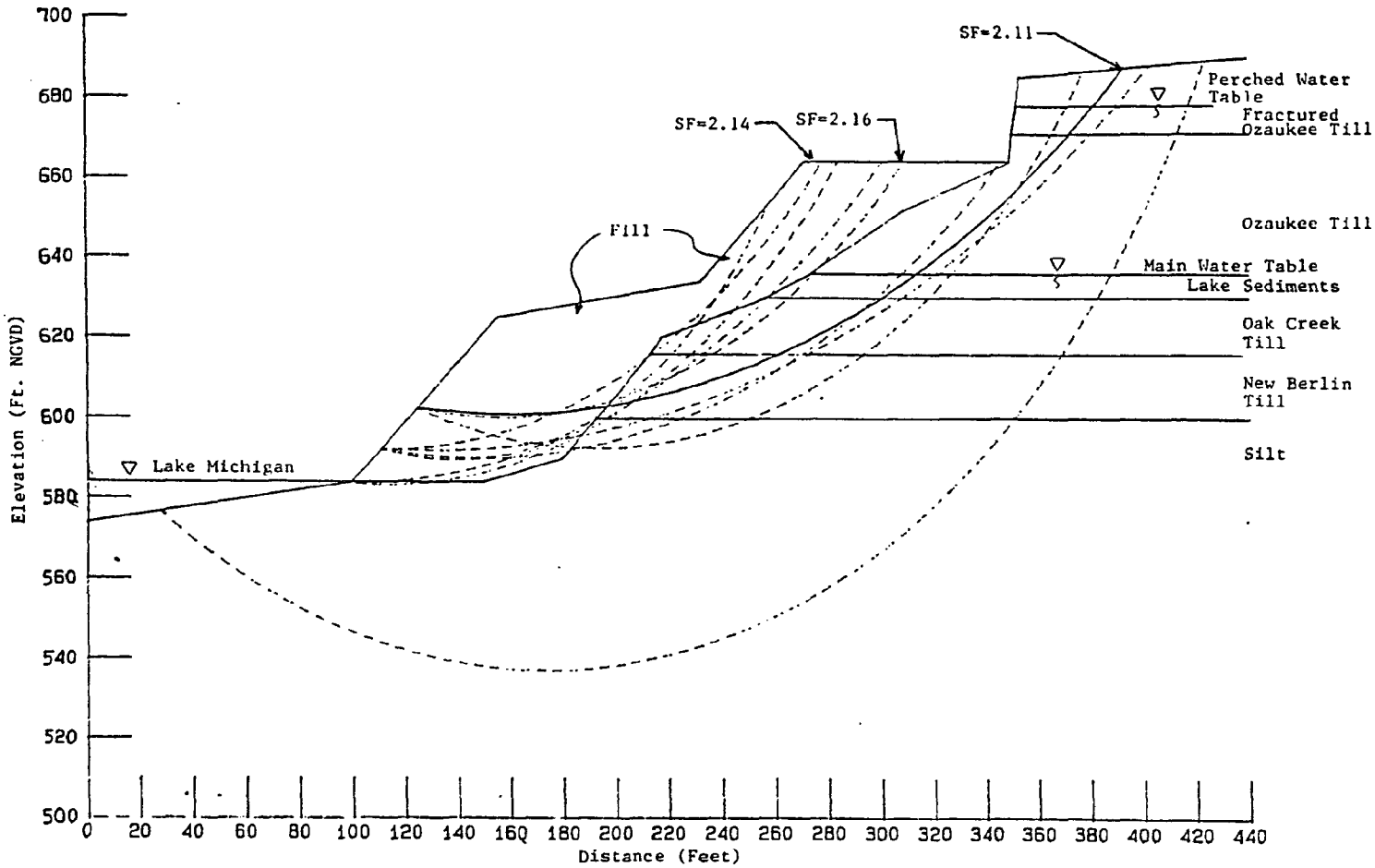
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 69: 4500 N. LAKE DRIVE



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

Figure III-75

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 70: 4620 N. LAKE DRIVE



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

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, was characterized by the use of Profile No. 71 and Profile No. 72.

The results of the deterministic slope stability analyses, shown in Figure III-76, and Figure III-77 for Profile No. 71 and Profile No. 72, respectively, indicate that the bluff slope was 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 remaining 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 remaining nine lowest safety factors ranged from 0.72 to 0.83.

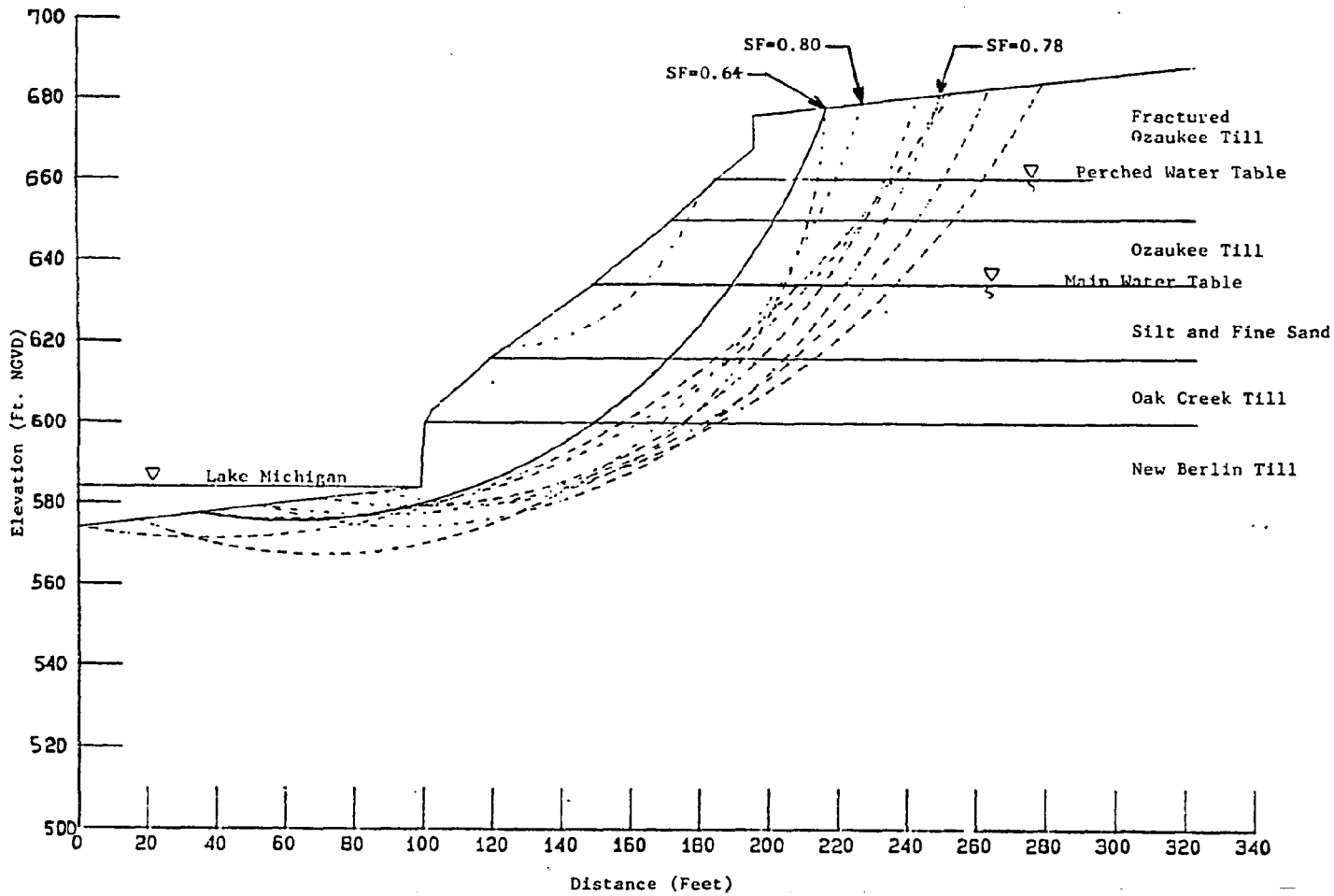
Of the 20 probabilistic stability analyses conducted for Profile No. 71, the lowest safety factors ranged from 0.50 to 0.97. Of the total of 200 failure surfaces evaluated at Profile No. 15, 185, or 92 percent, of the surfaces had safety factors of less than 1.0. Of the 20 probabilistic stability analyses conducted for Profile No. 72, the lowest safety factors ranged from 0.52 to 0.81. Of the total of 200 failure surfaces evaluated at Profile No. 72, 186, or 93 percent, of the surfaces 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 surveys conducted in the summer of 1986, and was identified as a

Figure III-76

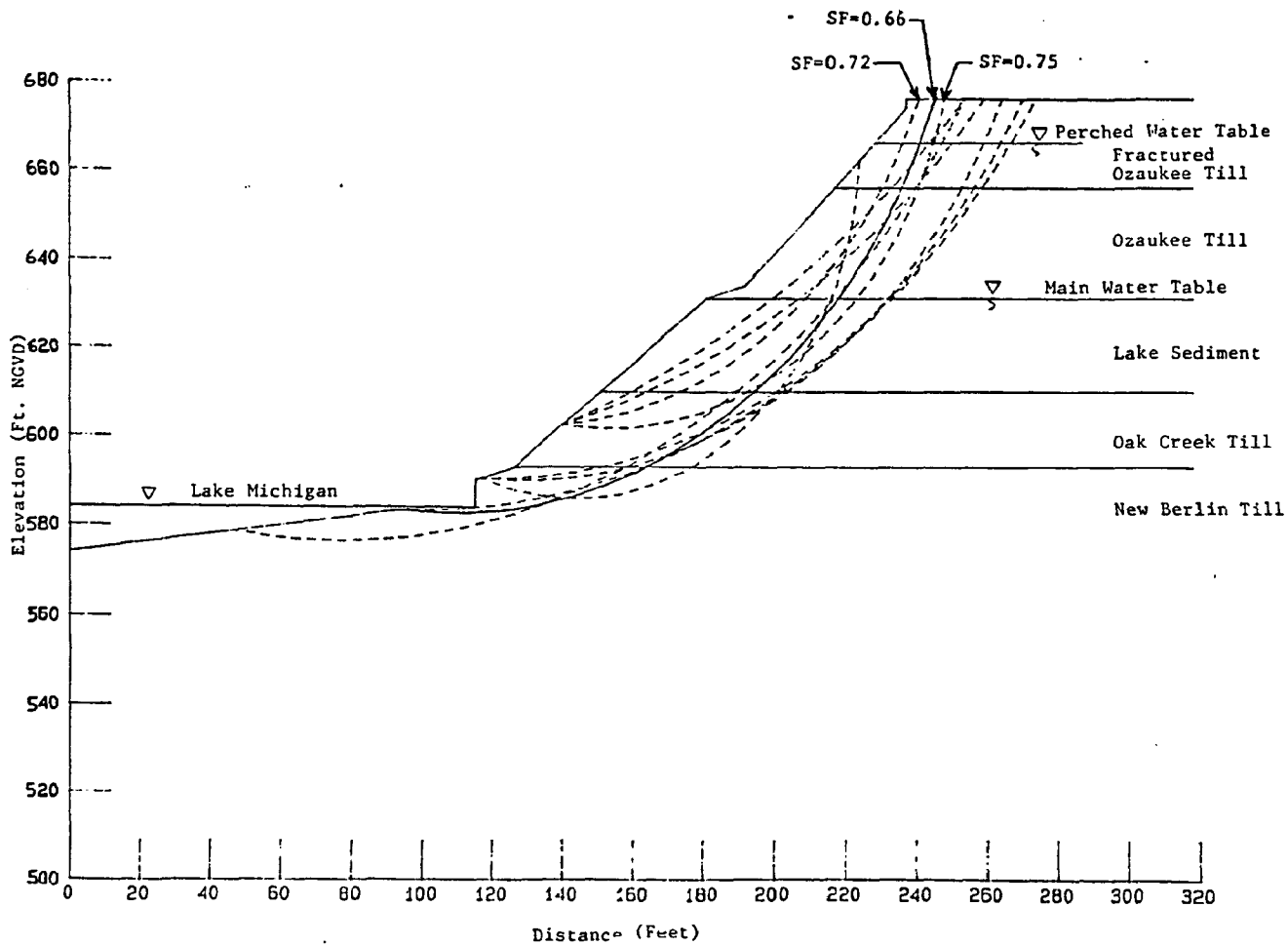
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 71: 4652 N. LAKE DRIVE



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

Figure III-77

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 72: 4730 N. LAKE DRIVE



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

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 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. 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 were as close as 40 feet from the bluff edge. Bluff toe protection is recommended to prevent erosion from wave and ice action.

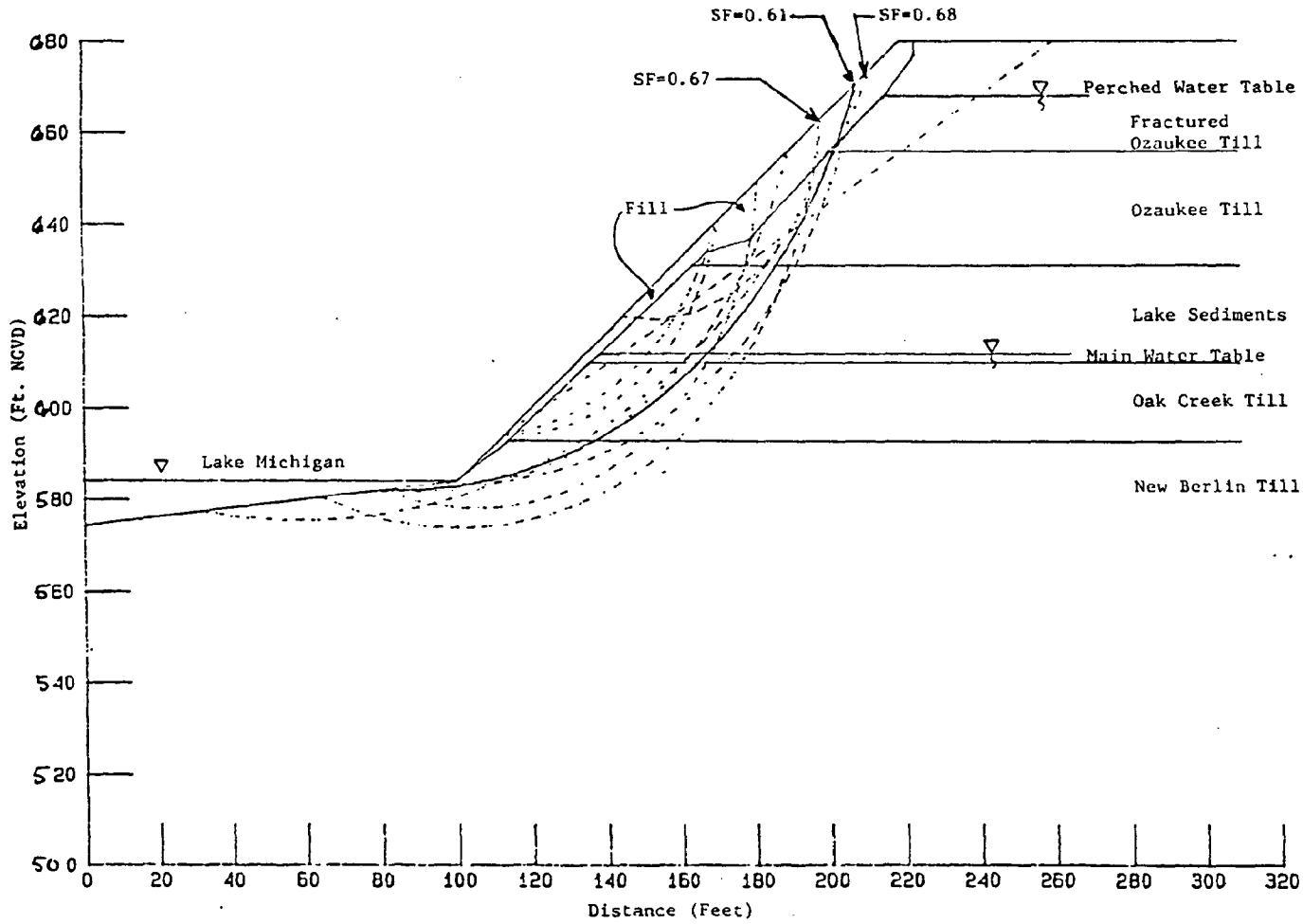
Bluff Analysis Section 73: Bluff Analysis Section 73 was a fill project under construction during the summer of 1986. The stability of the fill and the underlying bluff slope within Section 73, which extends from 4744 to 4762 N. Lake Drive, was characterized by the use of Profile No. 73.

The results of the deterministic slope stability analysis, shown in Figure III-78, indicate that Profile No. 73 had an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.61 and was located beneath the layer of fill material, within the natural bluff. The remaining nine lowest safety factors ranged from 0.67 to 0.86. The probabilistic slope stability analysis was not conducted for this profile 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 bluff slope.

Figure III-78

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 73: 4762 N. LAKE DRIVE



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

Bluff toe erosion was observed within the entire shoreline of Section 73 during the field surveys conducted in the summer of 1986. This bluff toe erosion was contributing to the instability of the bluff slope. No shore protection structures were located within this section as of 1986. Even if the lake levels would return to the mean 20th Century levels, it is not anticipated that a significant beach would develop within Section 73.

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.

Bluff Analysis Section 74: The stability of the bluff slope within Section 74, located at 4780 N. Lake Drive, was characterized by the use of Profile No. 74.

The results of the deterministic slope stability analysis, shown in Figure III-79, indicate that Profile No. 74 had an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.80, and included the entire bluff slope. The remaining nine lowest safety factors ranging from 0.81 to 0.97.

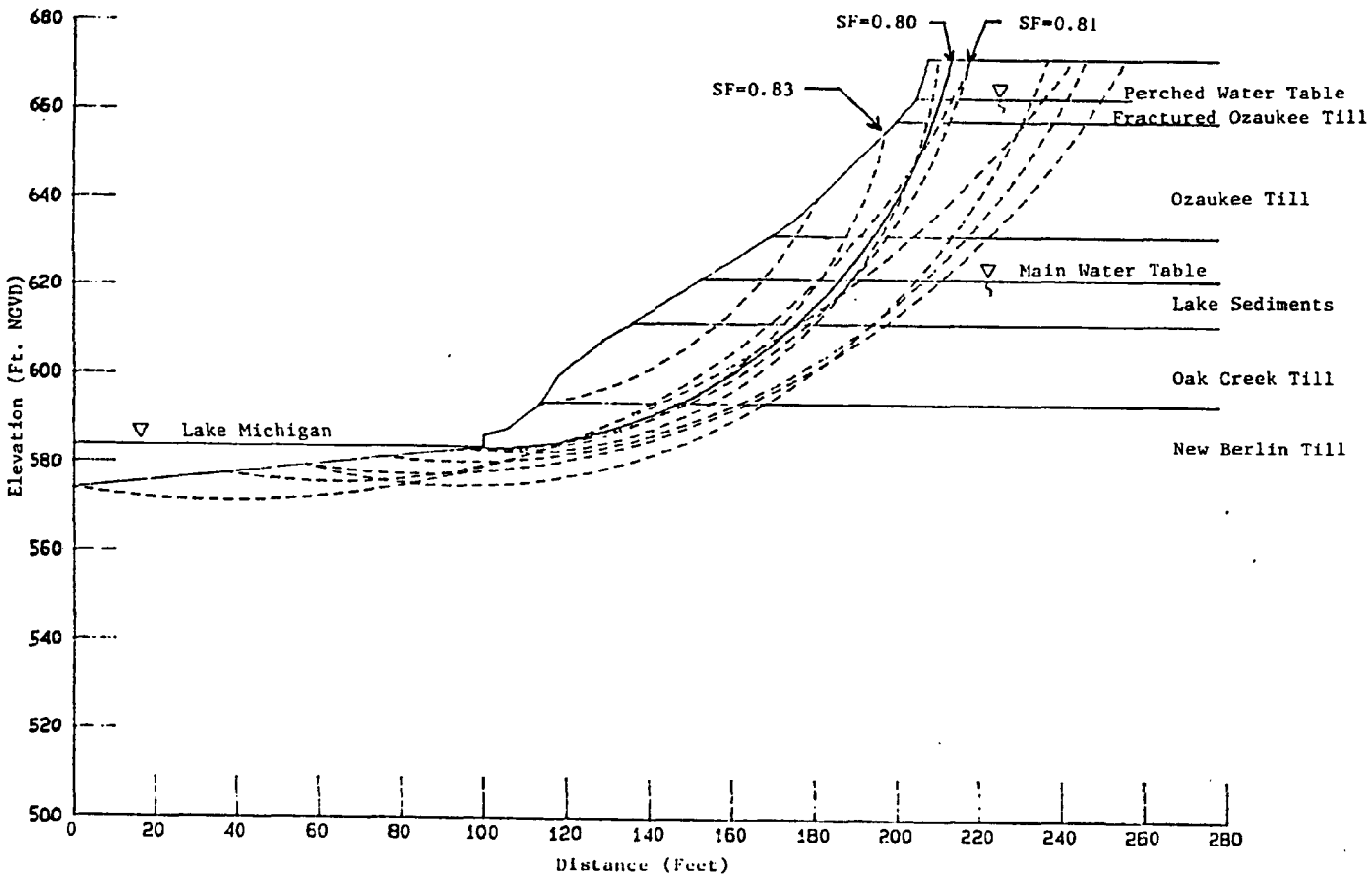
Of the 20 probabilistic stability analyses conducted for Profile No. 74, the lowest safety factors ranged from 0.55 to 0.82. Of the total of 200 failure surfaces evaluated at Profile No. 74, all the surfaces had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 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 surveys conducted in the summer of 1986. This toe erosion was affecting the stability of

Figure III-79

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 74: 4780 N. LAKE DRIVE



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

the bluff slope. As of 1986, no shore protection structures were located within this section. Even if the lake levels would return to the mean 20th Century levels, it is not anticipated that a significant beach would develop within Section 74.

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. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 75: Bluff Analysis Section 75 was a fill project which began 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, was characterized by the use of Profile No. 75.

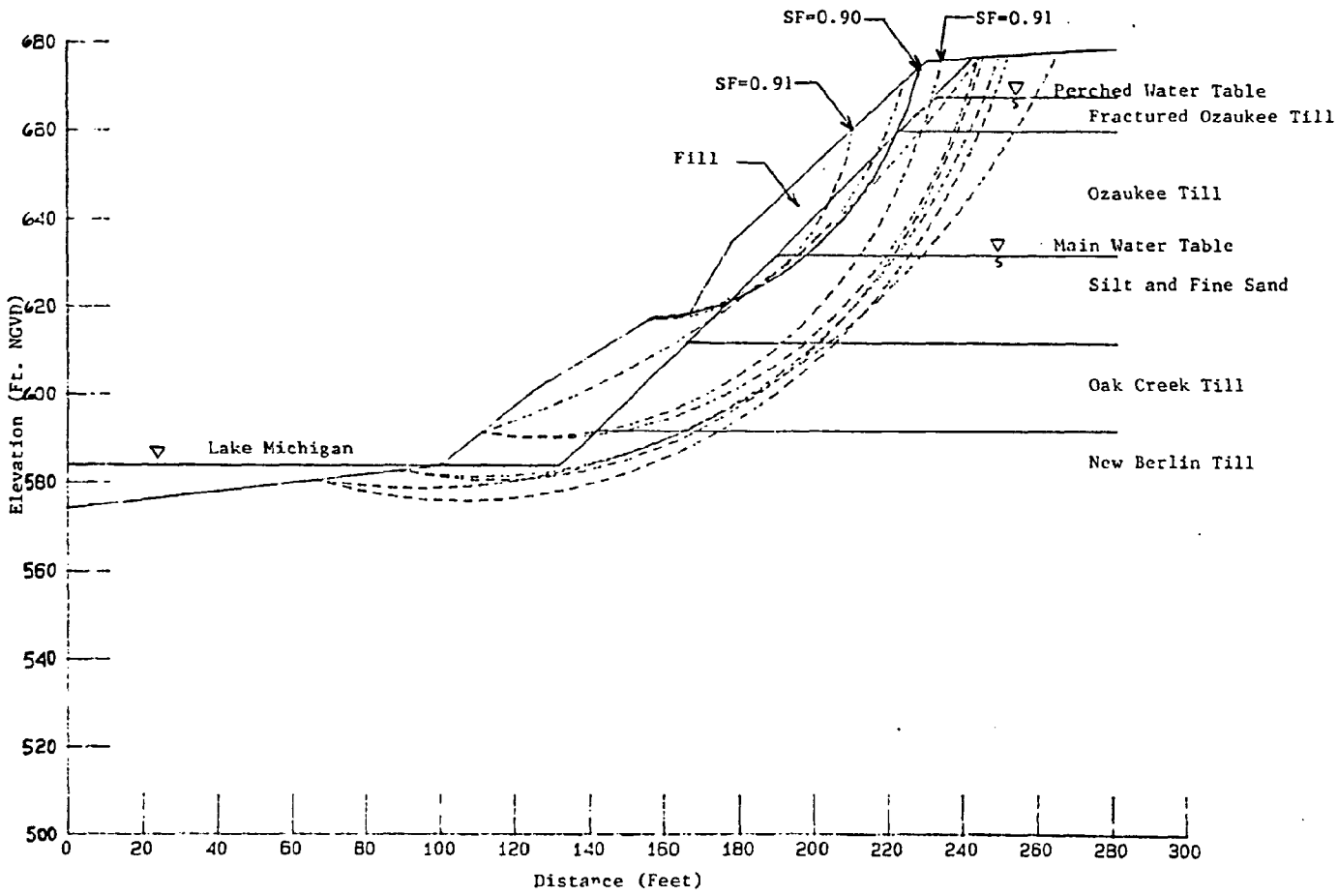
The results of the deterministic slope stability analysis, shown in Figure III-80, indicate that Profile No. 75 had an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.90, and was located within the upper portion of the fill material. The remaining nine lowest safety factors ranged from 0.91 to 0.96. The probabilistic slope stability analysis was not conducted for this profile 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 surveys conducted in May 1986. This toe 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. Even if the lake levels

Figure III-80

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 75: 4794 N. LAKE DRIVE



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

would return to the mean 20th Century levels, it is not anticipated that a significant beach would develop within Section 75.

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 required 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.

Bluff Analysis Section 76: The stability of the bluff slope within Section 76, which extends from 4810 to 4840 N. Lake Drive, was characterized by the use of Profile No. 76.

The results of the deterministic slope stability analysis, shown in Figure III-81, 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 remaining nine lowest safety factors ranged from 0.79 to 0.94.

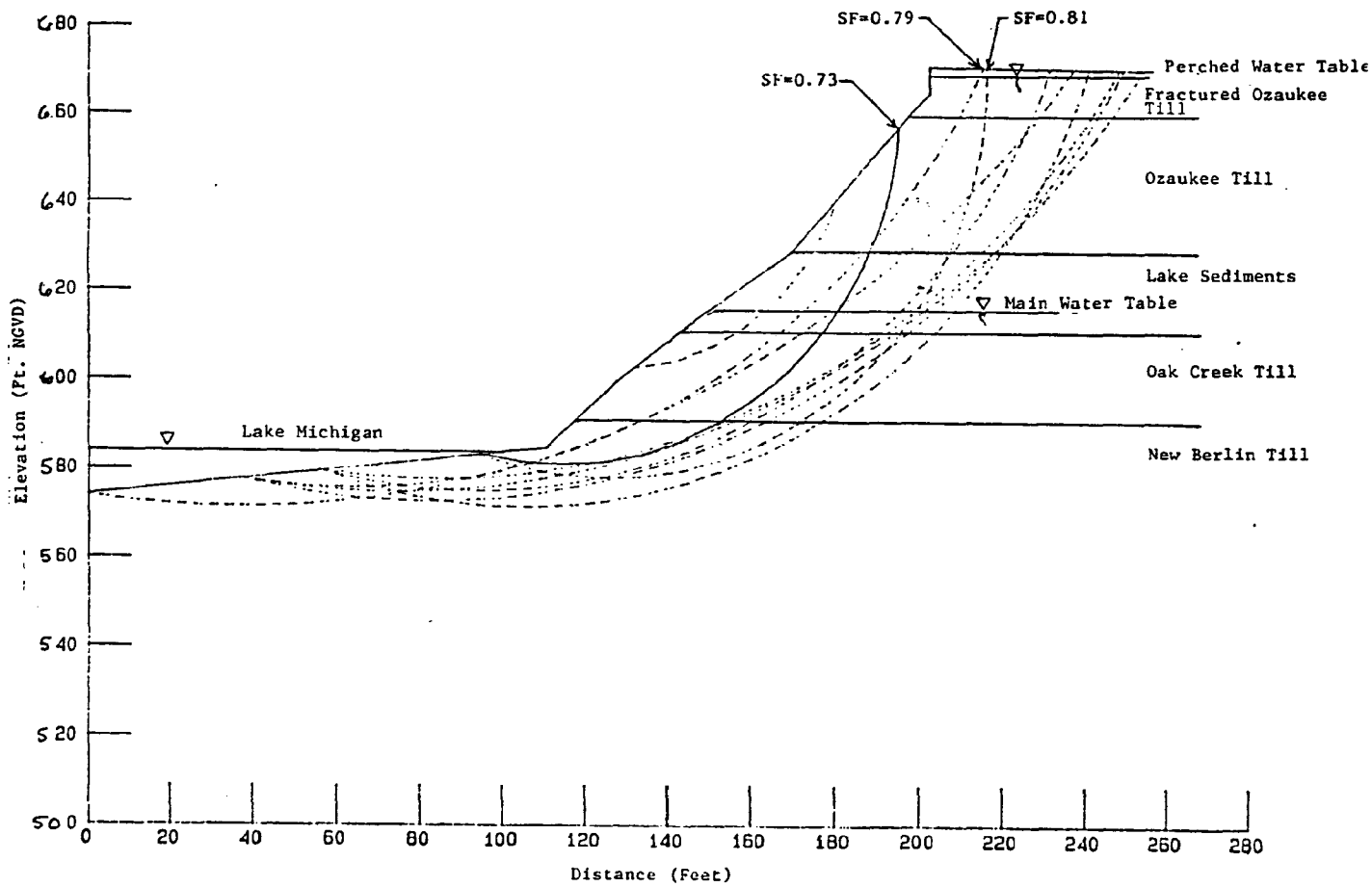
Of the 20 probabilistic stability analyses conducted for Profile No. 76, the lowest safety factors ranged from 0.53 to 0.83. Of the total of 200 failure surfaces evaluated at Profile No. 76, 190, or 95 percent of the surfaces, had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 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 surveys 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

Figure III-81

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 76: 4810 N. LAKE DRIVE



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

this section. Even if the lake levels would return to the mean 20th Century levels, it is not anticipated that a significant beach would develop within Section 76.

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 that bluff toe protection be provided within Section 76.

Bluff Analysis Section 77: 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 Buckly Park, was characterized by the use of Profile No. 77 and Profile No. 78.

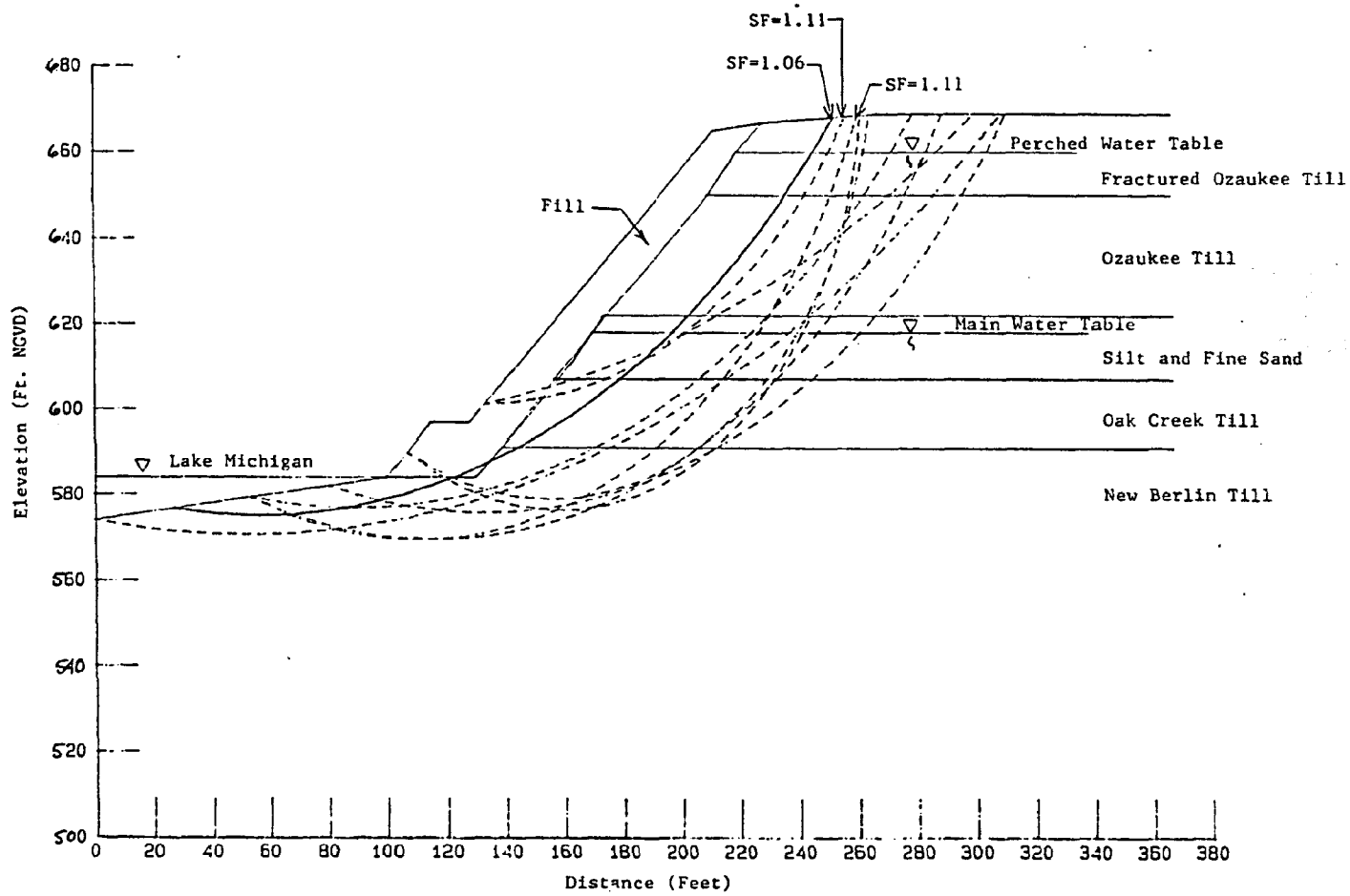
The results of the deterministic slope stability analyses, shown in Figure III-82 and Figure III-83, for Profile No. 77 and Profile No. 78, respectively, indicate stable bluff slopes with respect to rotational sliding. The lowest failure surface calculated at Profile No. 77 had a safety factor of 1.06, and was located beneath the fill layer. The remaining 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 remaining nine lowest safety factors ranged from 1.52 to 1.59. The 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 of 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.

Bluff toe erosion was observed within the northern portion of Section 77 during the field surveys conducted in the summer of 1986. However, because of the large amount of fill material at the base of the bluff, this toe erosion

Figure III-82

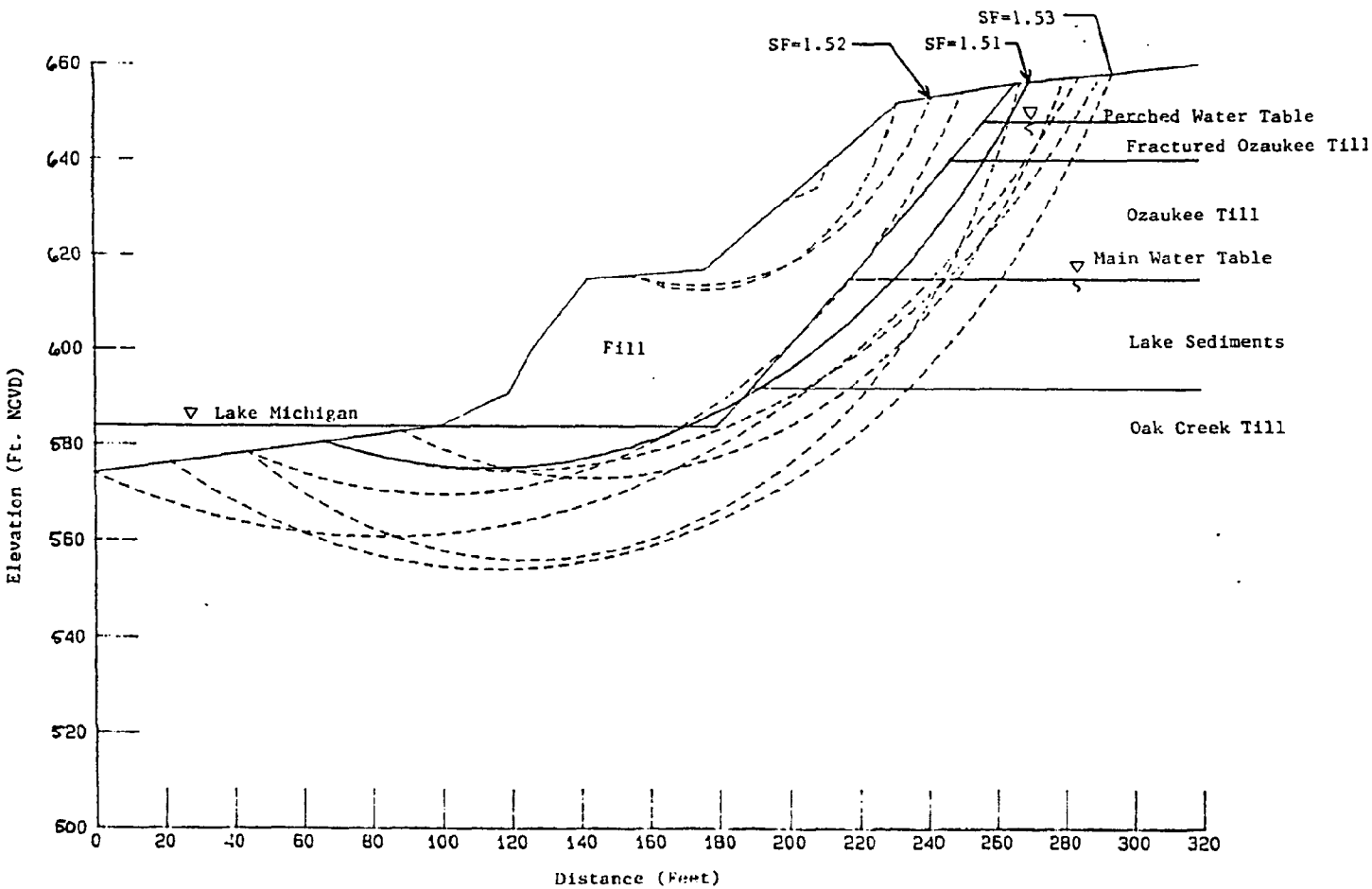
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 77: 4890 N. LAKE DRIVE



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

Figure III-83

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 78: 4930 N. LAKE DRIVE



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

was not affecting the stability of the bluff slope at the present time. 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 was not completed during the time of the field surveys, the degree of bluff toe protection provided could not be determined. Even if the lake levels would return to the mean 20th Century levels, it is not anticipated that significant beach would develop within Section 77.

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 northern portion of Buckly Park and the southern portion of Big Bay Park, was characterized by the use of Profile No. 79.

The results of the deterministic slope stability analysis, shown in Figure III-84, for Profile No. 79, indicate a potential 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 remaining nine lowest safety factors ranged from 0.98 to 1.07.

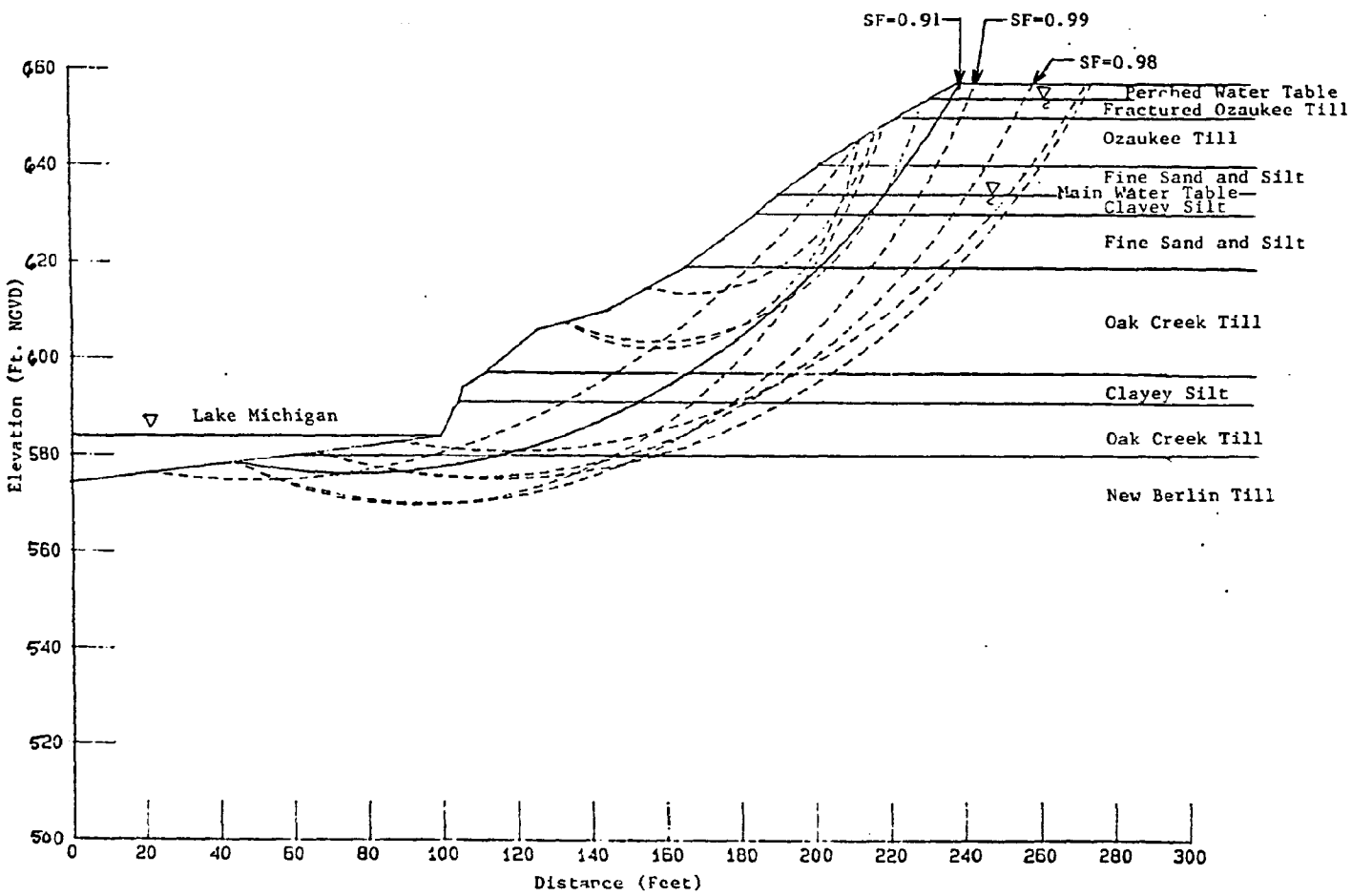
Of the 20 probabilistic stability analyses conducted, the lowest safety factors ranged from 0.54 to 1.06, with 17 failure surfaces, or 70 percent, having a safety factor less than 1.0. Of the total of 200 failure surfaces evaluated, 155, or 78 percent of the surfaces, had safety factors of less than 1.0.

In the 1986 summer field surveys, no bluff failures were observed, although some dislocation of trees was noted. However, in November 1986, a very large slump occurred at the southern end of this section in Buckly Park. Therefore, based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 78 was considered to have an unstable bluff slope with respect to rotational sliding.

Overall, Section 78 was considered to have a stable bluff slope with respect to translational sliding. This was due to the good vegetative growth that

Figure III-84

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 79: BIG BAY PARK



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

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. Even if the lake levels would return to the mean 20th Century levels, it is anticipated that a significant beach would develop within Section 78.

To prevent rotational sliding within Section 78, it is recommended that a groundwater drainage system be installed to lower the groundwater elevation. Toe erosion control measures are also recommended to prevent erosion from wave and ice action. At a minimum, adequate toe protection would require the maintenance and/or reconstruction of the existing concrete bulkhead.

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, was characterized by the use of Profile No. 80.

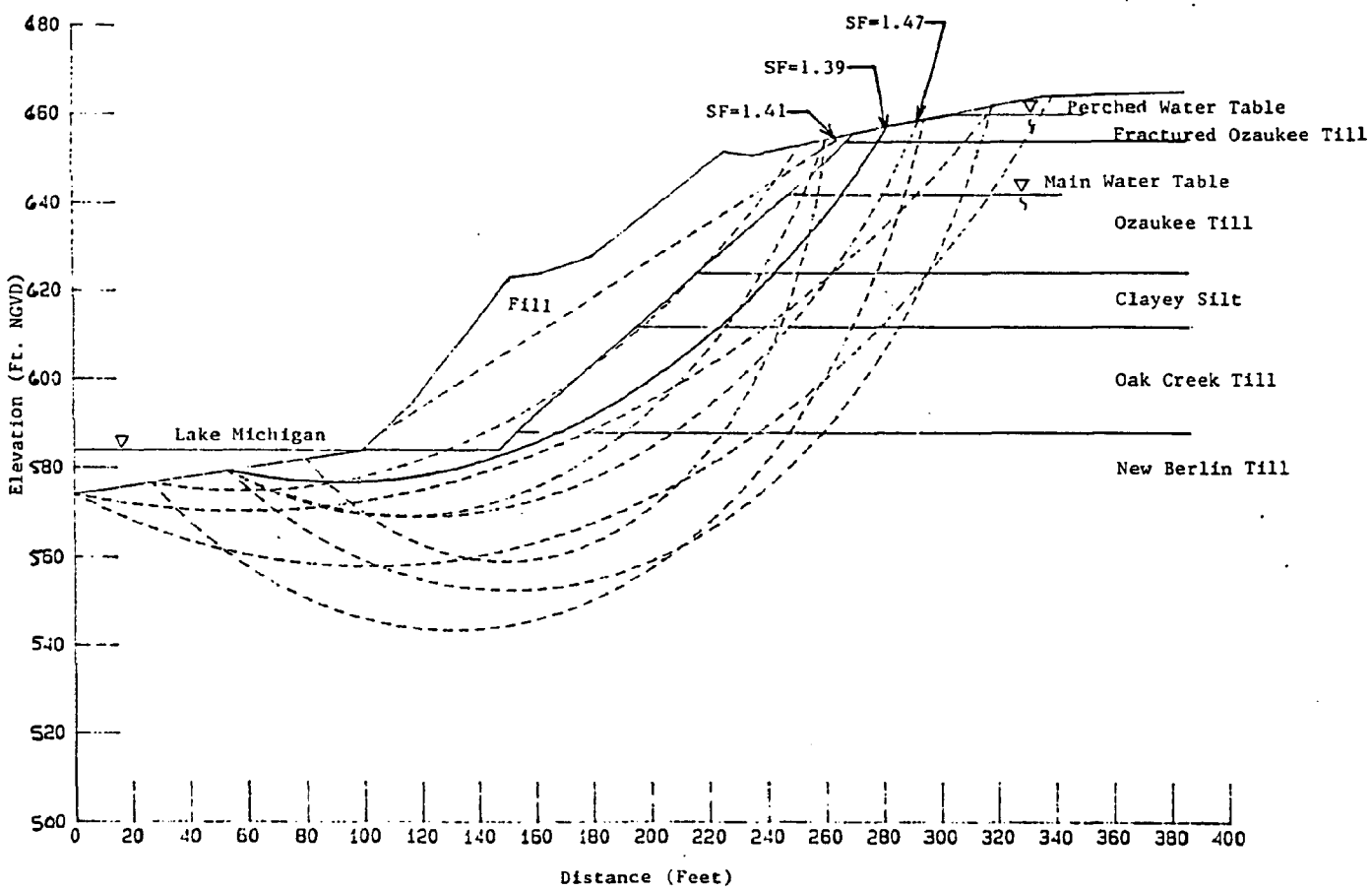
The results of the deterministic slope stability analysis, shown in Figure III-85, indicate that Profile No. 80 had a stable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile had a safety factor of 1.39, and was located beneath the fill. The remaining nine lowest safety factors ranged from 1.41 to 1.71. The 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 of 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 surveys conducted in the Summer of 1986. A revetment composed of rock

Figure III-85

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 80 : HENRY CLAY STREET



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

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 at the present time. Even if the lake levels would return to the mean 20th Century levels, it is not anticipated that a significant beach would develop within Section 79.

No measures are needed to prevent rotational or translational sliding within Bluff Analysis Section 79. Maintenance of the existing rock and concrete rubble revetment and concrete bulkheads is recommended to protect the toe of the bluff against wave and ice action.

Bluff Analysis Section 80: The stability of the bluff slope within Section 80, located at 5290 N. Lake Drive, was characterized by the use of Profile No. 81.

The results of the deterministic slope stability analysis, shown in Figure III-86, for Profile No. 81 indicate that the bluff slope was 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 remaining nine lowest safety factors ranged from 1.08 to 1.33.

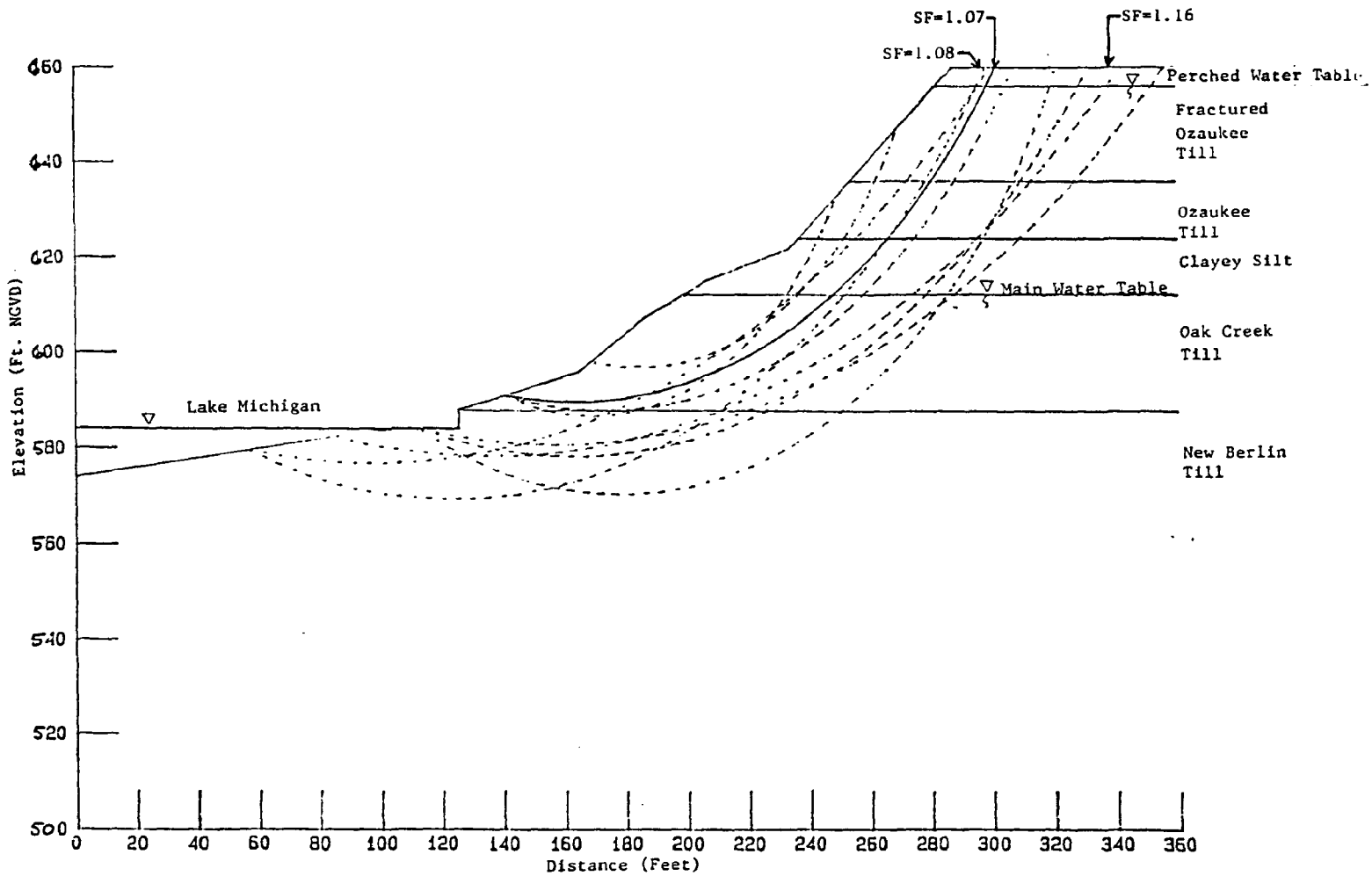
Of the 20 probabilistic stability analyses conducted, the lowest safety factors ranged from 0.76 to 1.44, with 15 of the failure surfaces, or 25 percent, having a safety factor less than 1.0. Of the total of 200 failure surfaces evaluated, 26, or 13 percent of the surfaces, had safety factors of less than 1.0.

In the field surveys 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 slope 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.

Figure III- 86

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 81: 5290 N. LAKE DRIVE



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

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 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 surveys 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. If the lake levels would return to the mean 20th Century levels, it is anticipated that the resulting beach within Section 80 would approximate 60 feet in width.

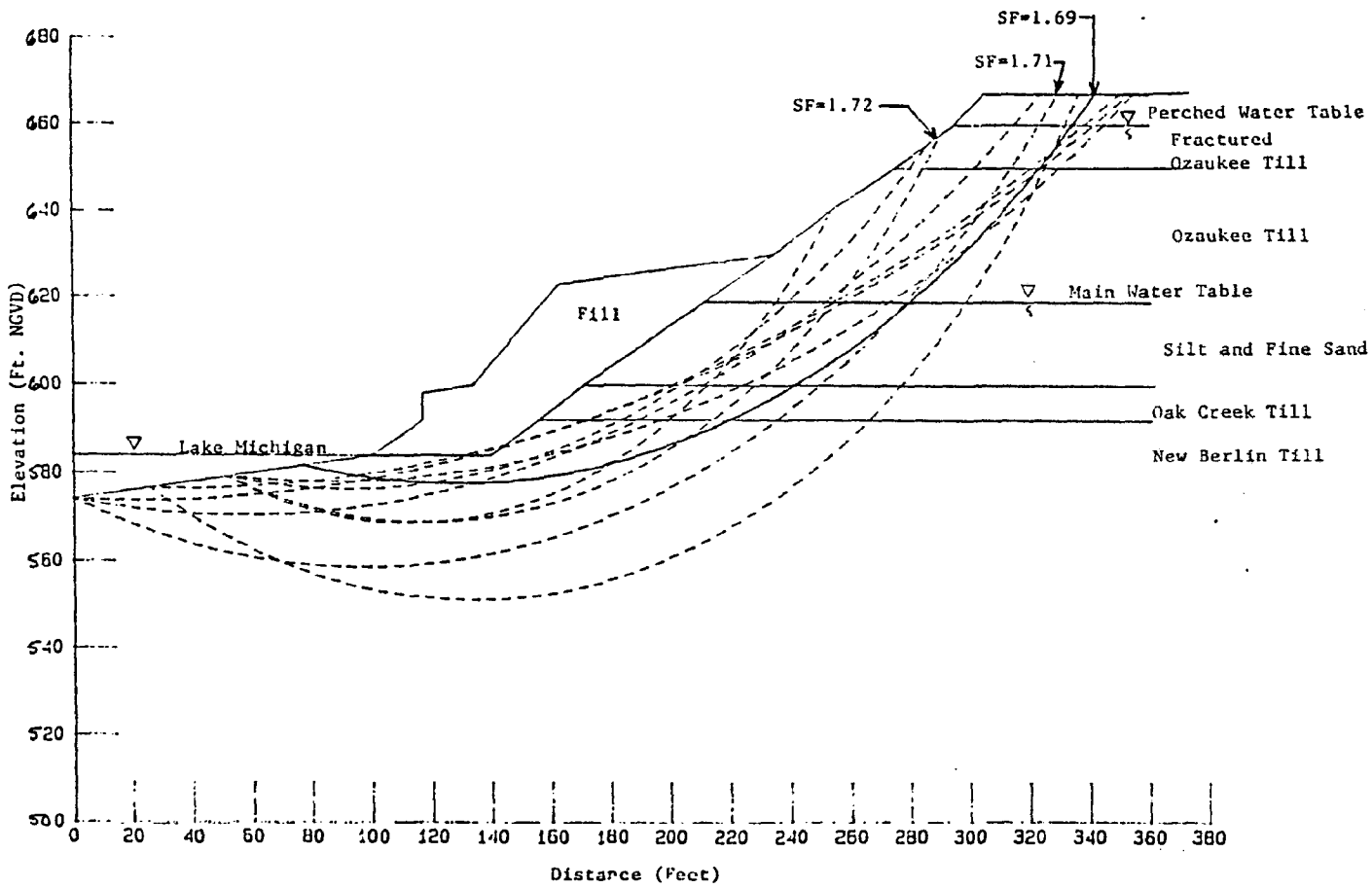
It is recommended that the upper portion of the bluff slope be regraded to a stable slope angle. It does not appear necessary at this time to provide additional protection against wave and ice action at the toe of the bluff.

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, was characterized by the use of Profile No. 82 and Profile No. 83.

The results of the deterministic slope stability analyses, shown in Figure III-87 and Figure III-88, for Profile No. 82 and Profile No. 83, respectively, 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 remaining 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 remaining nine lowest safety factors ranged from 1.80 to 2.02. The probabilistic slope stability analysis was not conducted for this section because it is a fill area.

Figure III-87

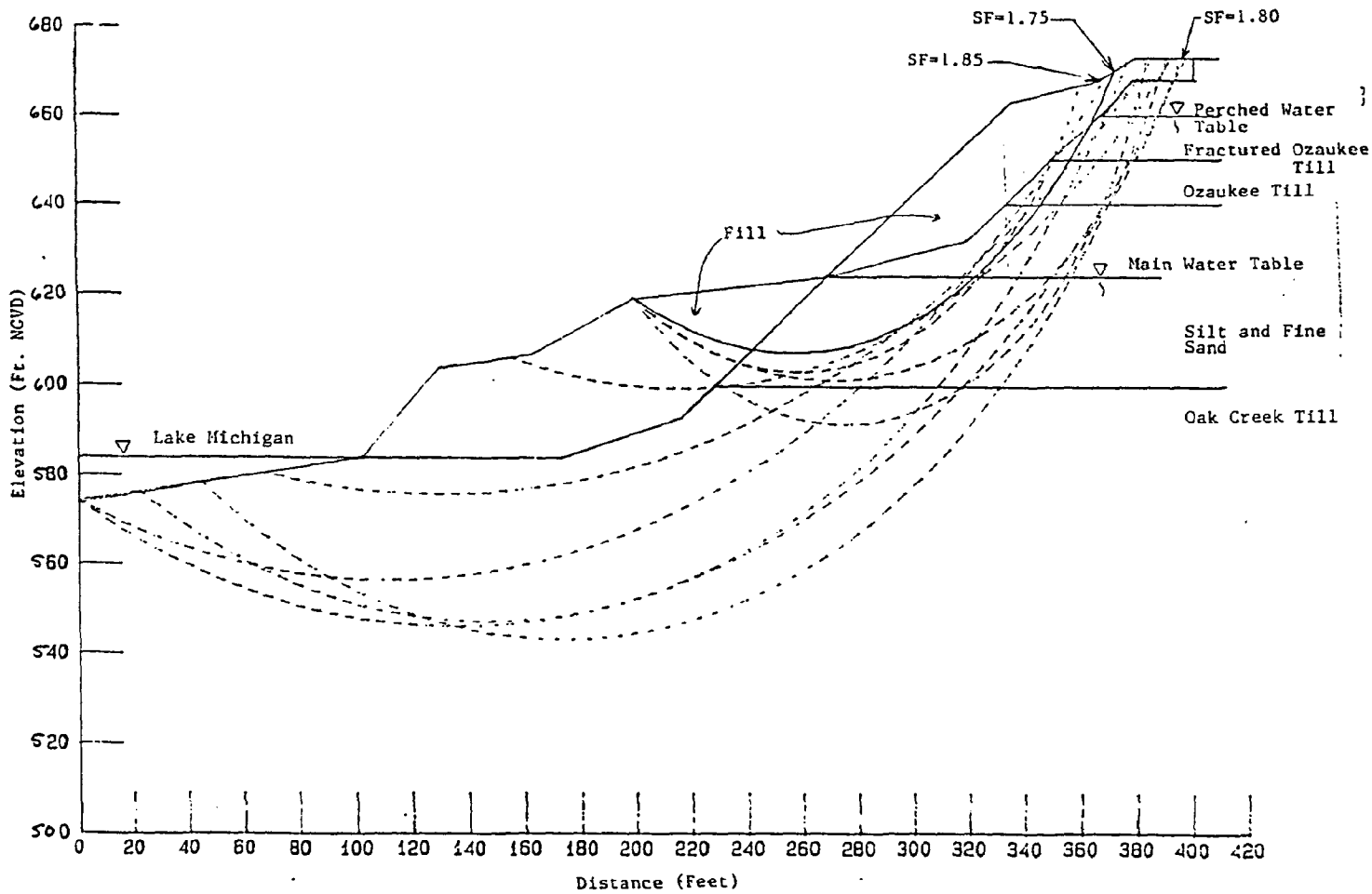
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 82: 5486 N. LAKE DRIVE



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

Figure III-88

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 83 : 5674 N. SHORE DRIVE



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

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 of loading the base of the bluff. Within the northern portion of Section 81, because of the large amount of fill material placed on nearly the entire natural bluff slope, translational sliding was not considered likely to occur. In the southern portion of the section, however, fill material was 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 surveys 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 at the present time. 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 was not completed during the time of the field surveys, an evaluation of the degree of bluff toe protection provided was not made. Even if the lake levels would return to the mean 20th Century levels, it is not anticipated that a significant beach would develop within Section 81.

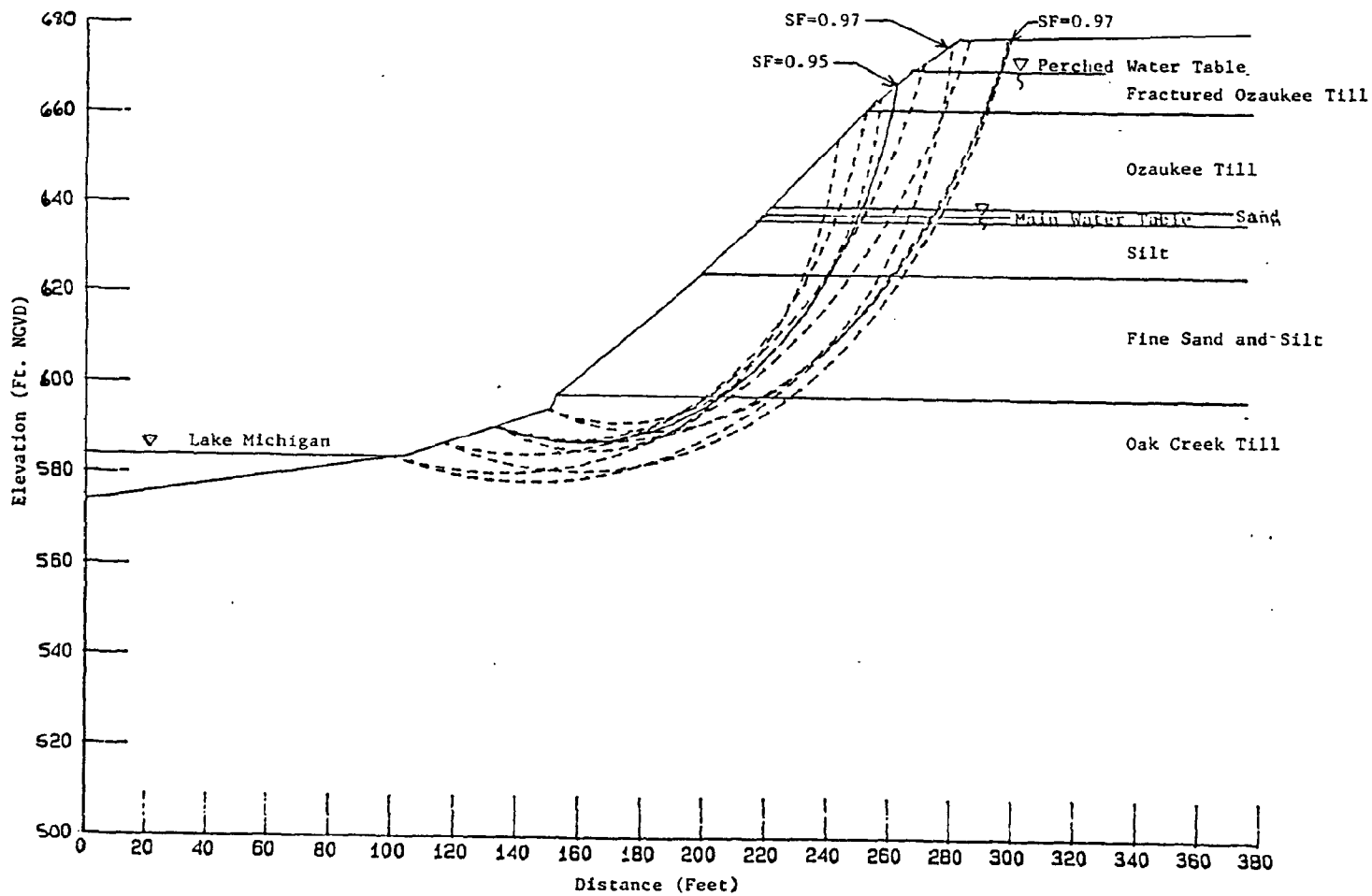
No measures are needed to prevent rotational sliding within Bluff Analysis Section 81. Only minimal translational sliding may be expected to occur--primarily on the upper bluff slope in the southern portion of the section. Toe erosion control should be provided by maintenance of the existing rock and concrete rubble revetment located within the southern 1,700 feet of the section south of Silver Spring Drive.

Bluff Analysis Section 82: The stability of the bluff slope within Section 82, which extends from 5722 to 5770 N. Shore Drive, was characterized by the use of Profile No. 84.

The results of the deterministic slope stability analysis shown in Figure III-89, indicate that Profile No. 84 had an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated had a safety

Figure III-89

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 84: 5738 N. SHORE DRIVE



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

factor of 0.95, and was located within the middle portion of the bluff slope. The remaining nine lowest safety factors ranged from 0.97 to 0.99.

Of the 20 probabilistic stability analyses conducted, the lowest safety factors ranged from 0.47 to 1.12, with 17 of the failure surfaces, or 70 percent, having a safety factor less than 1.0. Of the total of 200 failure surfaces evaluated, 159, or 80 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 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 increased on the lower portion of the bluff where groundwater seepage was noted during the 1986 field surveys.

Minor erosion of the toe of the bluff due to wave action was observed. This toe erosion, should it continue, 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. If the lake levels would return to the mean 20th Century levels, it is anticipated that the resulting beach within Section 82 would approximate 60 feet in width.

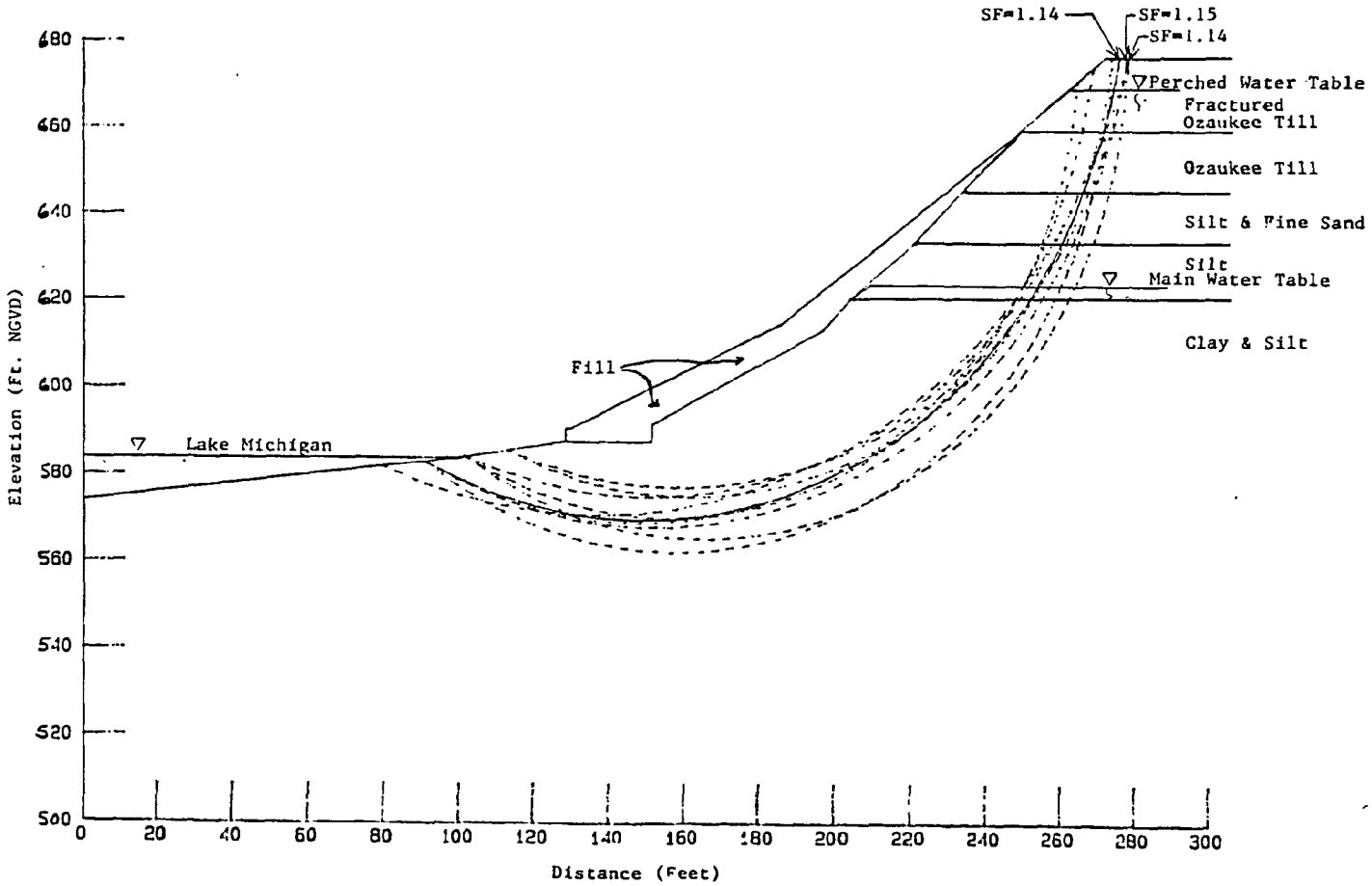
To prevent rotational sliding within Section 82, it is recommended that a groundwater drainage system be installed to lower the groundwater elevation. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 83: The stability of the fill and the underlying bluff slope within Section 83 which is located at 758 E. Day Street was characterized by the use of Profile No. 85.

The results of the deterministic slope stability analysis, shown in Figure III-90, indicate that Profile No. 85 had a stable bluff with respect to

Figure III- 90

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 85: 758 DAY STREET



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

rotational sliding. The lowest failure surface calculated at this profile had a safety factor of 1.14, and was located beneath the fill. The remaining nine lowest safety factors ranged from 1.14 to 1.18. The 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 of 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 surveys 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. If the lake levels would return to the mean 20th Century levels, it is anticipated that the resulting beach within Section 83 would approximate 40 to 50 feet in width.

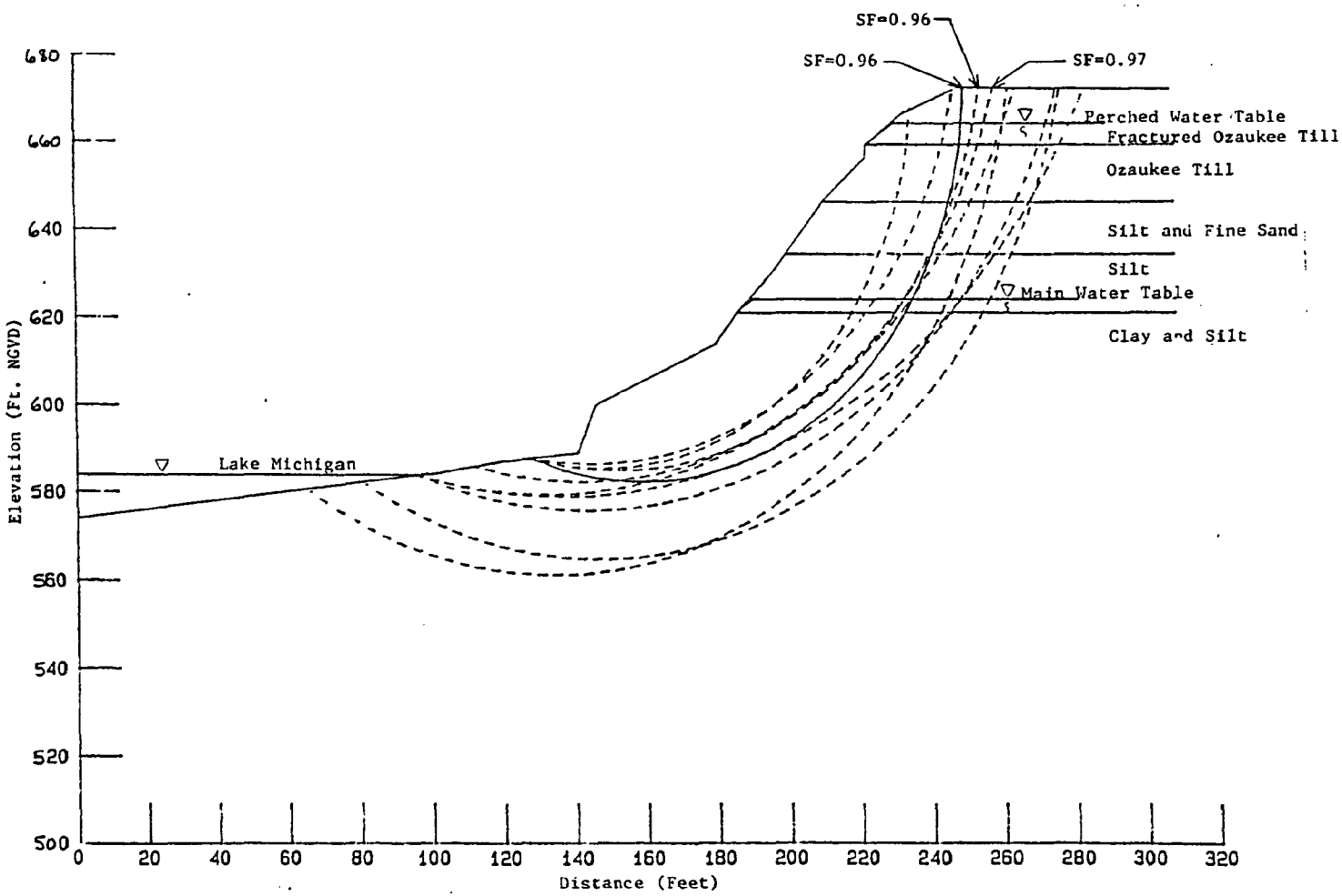
No measures are needed to prevent rotational or translational sliding within Bluff Analysis Section 83. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 84: The stability of the bluff slope within Section 84, which extends from 740 E. Day Street to 5866 N. Shore Drive, was characterized by the use of Profile No. 86.

The results of the deterministic slope stability analysis, shown in Figure III-91 for Profile No. 86, indicate a potential 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 remaining nine lowest safety factors ranged from 0.96 to 1.03.

Figure III-91

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 86: 5822 N. SHORE DRIVE



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

Of the 20 probabilistic stability analyses conducted, the lowest safety factors ranged from 0.54 to 1.06, with 18 of the failure surfaces, or 90 percent, having a safety factor less than 1.0. Of the total of 200 failure surfaces evaluated, 184, or 64 percent of the surfaces, had safety factors of less than 1.0.

In the 1986 field surveys, 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.

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 increased on the lower portion of the bluff where groundwater seepage was noted during the 1986 field surveys.

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. If the lake levels would return to the mean 20th Century levels, it is anticipated that the resulting beach within Section 84 would approximate 50 feet in width.

To prevent rotational and translational sliding within Section 84, it is recommended that the lower portion of the bluff slope be regraded to a stable slope angle. 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 III-92 for Profile No. 87, indicate a potential 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 remaining nine lowest safety factors were much higher, ranging from 0.99 to 1.19.

Of the 20 probabilistic stability analyses conducted for Profile No. 87, the lowest safety factors ranged from 0.53 to 1.13, with 14 of the failure surfaces, or 70 percent, having a safety factor less than 1.0. Of the total of 200 failure surfaces evaluated at Profile No. 87, 70, or 35 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 in the summer of 1986, Section 85 was considered to have an unstable bluff slope with respect to rotational sliding.

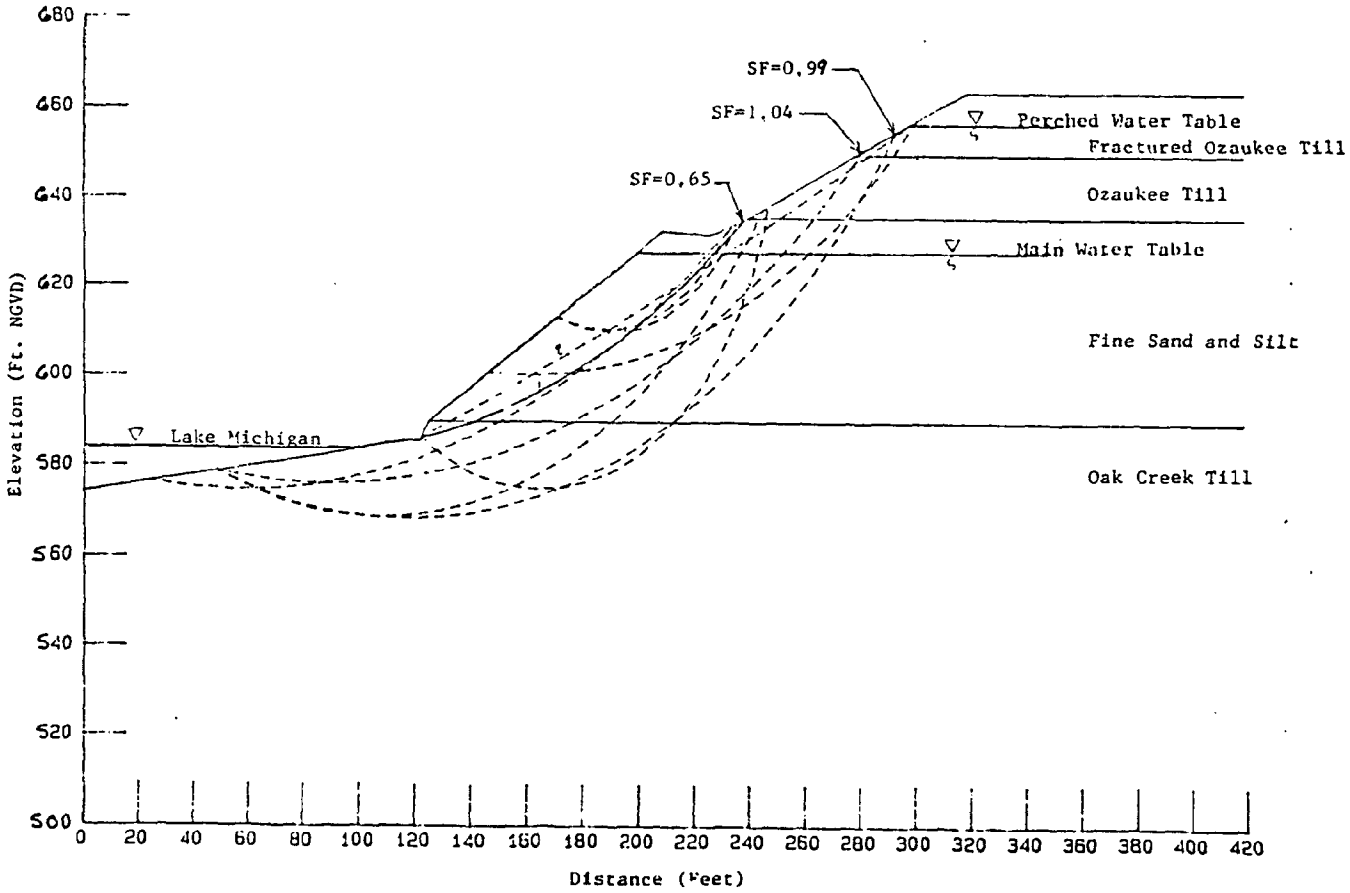
During December of 1986, the lower portion of the bluff slope within Section 85 failed. The resulting slope failure was located within the predicted lowest failure surface identified in the deterministic analysis. Following the slope failure within the southern portion of the section, the concrete bulkhead protecting the toe of the bluff was backfilled with quarry stone in January 1987. This corrective action minimized the threat of additional deep-seated rotational slips in the southern portion of Section 85. There remained a substantial risk of slope failures within the northern portion of the section, however. In April of 1987, a major slump occurred within the northern portion of the section.

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.

As previously mentioned, the southern portion of the section received additional bluff toe protection in January 1987. The bluff toe within this area was protected by a concrete bulkhead, backfilled with quarry stone. In the

Figure III- 92

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 87: KLODE PARK



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

northern portion of Section 85, however, the concrete bulkhead protecting the bluff slope collapsed in December 1986, allowing direct wave action on the bluff toe, which increased the potential for slope failure. If the lake levels would return to the mean 20th Century levels, it is anticipated that the resulting beach within Section 85 would approximate 40 feet in width.

No measures are recommended to prevent rotational or translational sliding within the southern 250 feet of Section 85. Within this portion of the section, toe erosion control measures are not needed, other than the continued maintenance of the concrete bulkhead, backfilled with quarry stone. Within the northern 230 feet of the section, it is recommended that the lower portion of the bluff slope be regraded to a stable slope angle. A good vegetative cover should be maintained on the bluff slope. Additional toe erosion control measures are needed within this portion of the section to prevent erosion from wave and ice action.

Bluff Analysis Section 86: The stability of the bluff slope within Section 86, which is located at 5960 N. Shore Drive, was characterized by Profile No. 88.

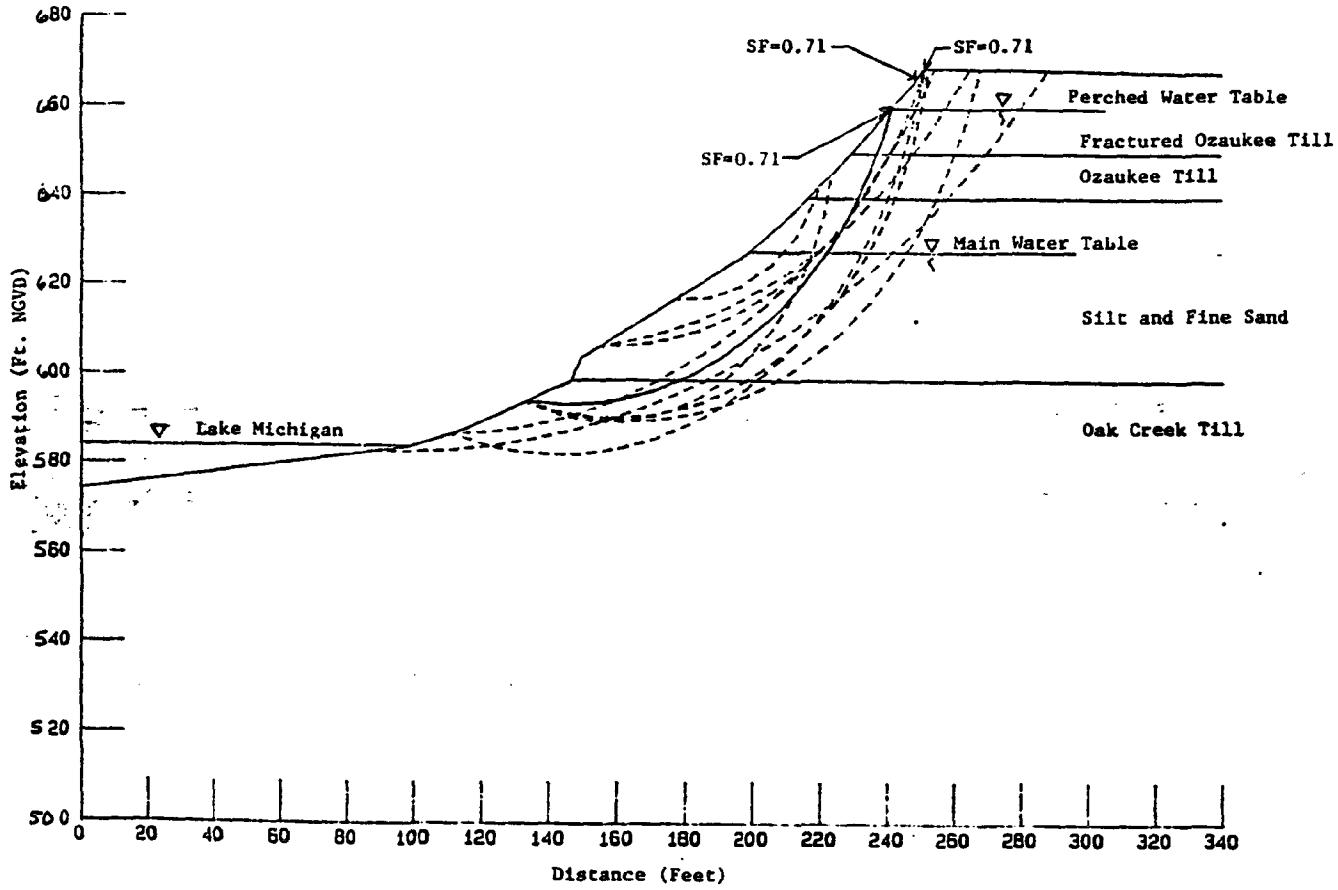
The results of the deterministic slope stability analysis, shown in Figure III-93, indicate that Profile No. 88 had an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.70, and was located within the middle portion of the bluff slope. The remaining nine lowest safety factors ranged from 0.71 to 0.88.

Of the 20 probabilistic stability analyses conducted for Profile No. 88, the lowest safety factors ranged from 0.52 to 1.10, with 18 of the failure surfaces, or 90 percent, having a safety factor less than 1.0. Of the total of 200 failure surfaces evaluated at Profile No. 32, 163, or 82 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 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

Figure III-93

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 88: 5960 N. SHORE DRIVE



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

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 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. If the lake levels would return to the mean 20th Century levels, it is anticipated that the resulting beach within Section 86 would approximate 40 feet in width.

To prevent rotational and translational sliding within Section 86, it is recommended that the bluff slope be regraded to a stable slope angle. Bluff toe protection is recommended to prevent erosion from wave and ice action.

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, was characterized by the use of Profile No. 89.

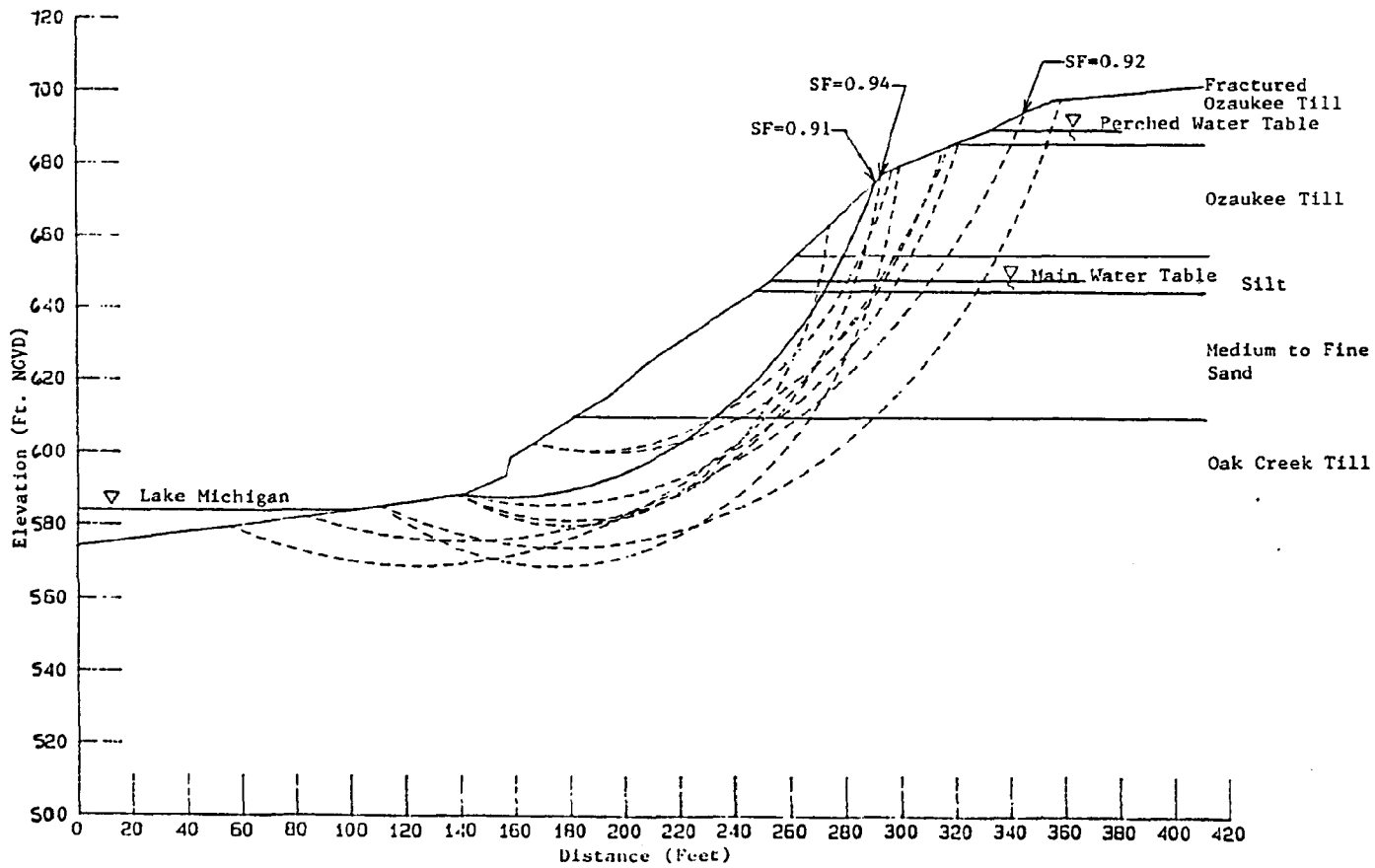
The results of the deterministic slope stability analysis, shown in Figure III-94 for Profile No. 89, indicate a potential threat of bluff slope failure with respect to rotational sliding. The lowest failure surface calculated at Profile No. 89 had a safety factor of 0.91, and was located on the lower portion of the bluff slope. The remaining nine lowest safety factors ranged from 0.92 to 1.03.

Of the 20 probabilistic stability analyses conducted, the lowest safety factors ranged from 0.62 to 1.60, with 13 of the failure surfaces, or 65 percent, having a safety factor less than 1.0. Of the total of 200 failure surfaces evaluated, 96, or 48 percent of the surfaces, had safety factors of less than 1.0.

In the 1986 field surveys, 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

Figure III-94

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 89: 614 EAST LAKE HILL COURT



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

deterministic and probabilistic slope stability analyses, and on these 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. If the lake levels would return to the mean 20th Century levels, it is anticipated that the resulting beach within Section 27 would approximate 30 feet in width.

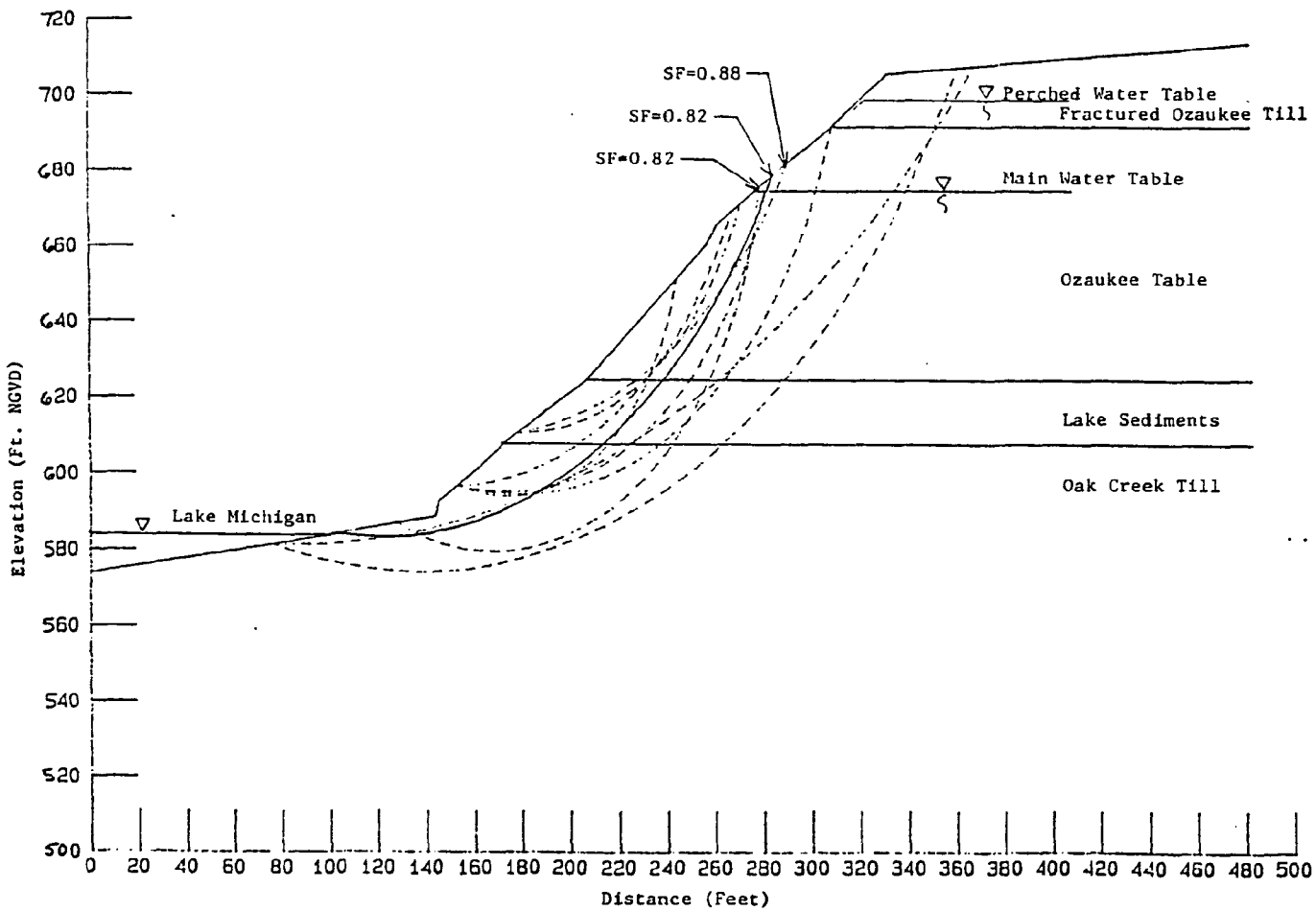
To prevent rotational sliding in Section 87, it is recommended that a groundwater drainage system be installed to lower the groundwater elevation. To prevent translational sliding, it is recommended that a good vegetative cover be maintained on the bluff slope. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 88: The stability of the bluff slope within Section 88, which extends from 6310 to 6424 N. Lake Drive, was characterized by the use of Profile No. 90 and Profile No. 91.

The results of the deterministic slope stability analyses, shown in Figure III-95 and Figure III-96 for Profile No. 90 and Profile No. 91, respectively, indicate the bluff slope was unstable with respect to rotational sliding. The lowest 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 remaining nine lowest safety factors evaluated at Profile No. 90 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

Figure III-95

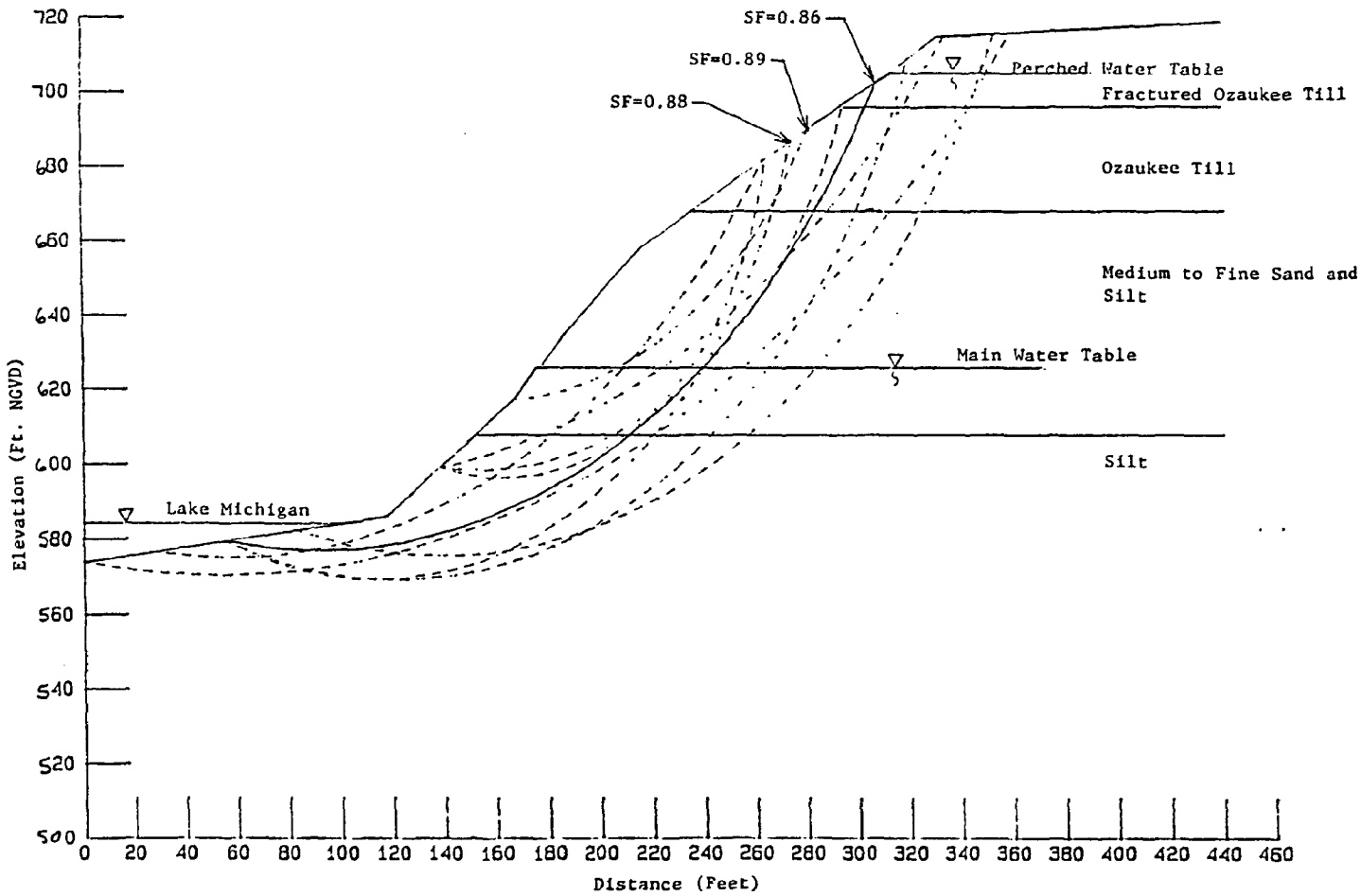
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 9Q: 6330 N. LAKE DRIVE



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

Figure III- 96

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 91 : 6424 N. LAKE DRIVE



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

bluff slope. The remaining nine lowest safety factors evaluated at Profile No. 91 ranged from 0.88 to 1.02.

Of the 20 probabilistic stability analyses conducted for Profile No. 90, the lowest safety factors 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 total of 200 failure surfaces evaluated at Profile No. 90, 137, or 68 percent, of the surfaces had safety factors of less than 1.0. Of the 20 probabilistic stability analyses conducted for Profile No. 91, the lowest safety factors 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 total of 200 failure surfaces evaluated at Profile No. 91, 167, or 84 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 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 surveys 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 surveys; 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. If the lake levels would return to the mean 20th Century levels, it is anticipated that the resulting beach within Section 88 would approximate 40 feet in width.

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. 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, was characterized by Profile No. 92.

The results of the deterministic slope stability analysis, shown in Figure III-97, indicate that Profile No. 92 had 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 remaining nine lowest safety factors ranged from 1.20 to 1.24. The probabilistic slope stability analysis was not conducted for this section because it is a fill area.

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 of 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.

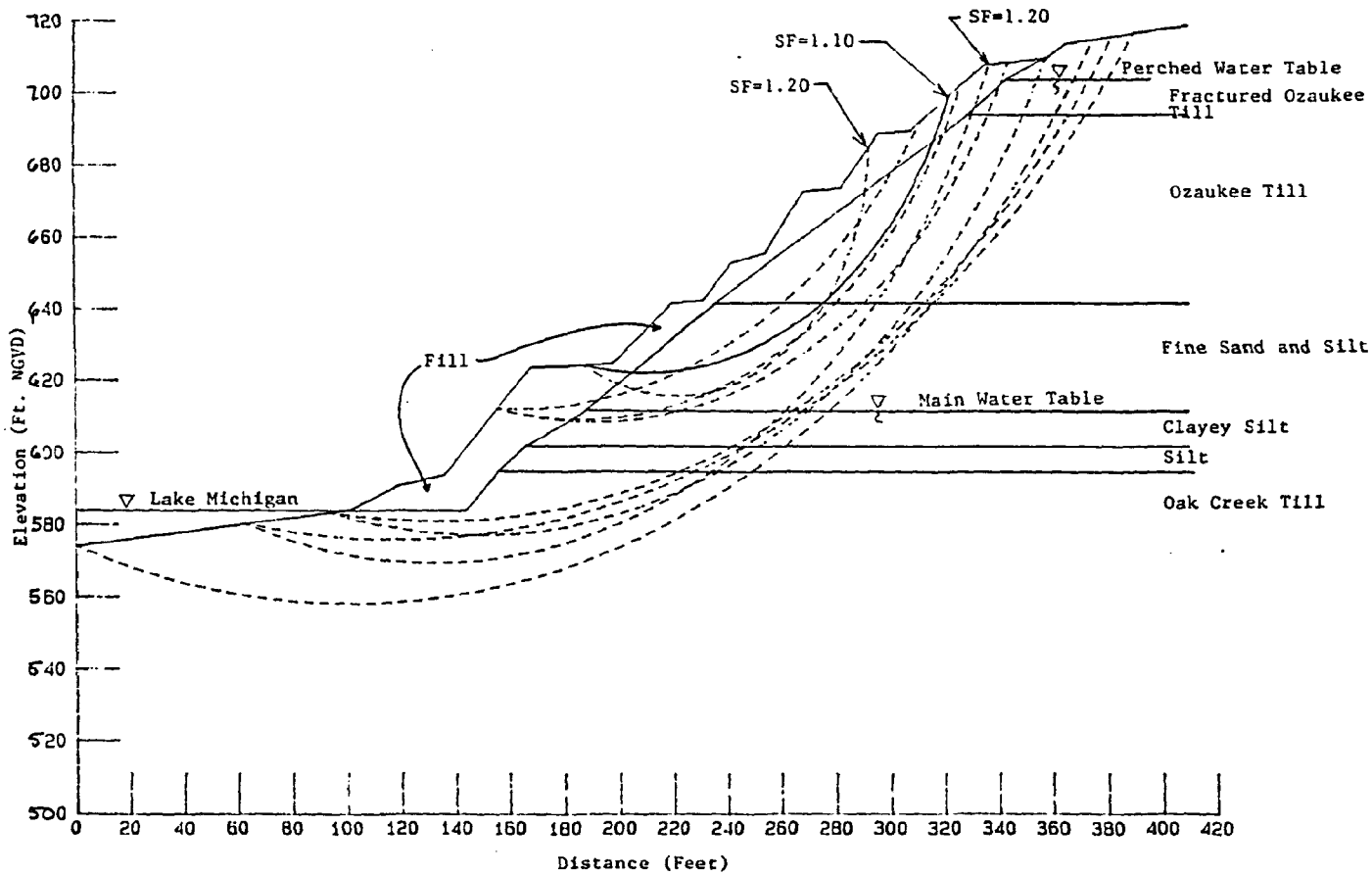
Erosion at the toe of the bluff was not evaluated in this section because the fill was still under construction in 1986. A revetment composed of large concrete blocks and slabs was being placed at the toe of the fill during the 1986 field surveys. Even if the lake levels would return to the mean 20th Century levels, it is not anticipated that a significant beach would develop within Section 89.

No measures are needed to prevent rotational or translational sliding within Bluff Analysis Section 89. It is recommended that adequate toe protection be provided at the base of the fill, when complete, to prevent erosion by 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, was characterized by the use of Profile No. 93.

Figure III-97

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 92: 6448 N. LAKE DRIVE



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

The results of the deterministic slope stability analysis, shown in Figure III-98, indicate that Profile No. 93 had an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.91, and was located within the middle portion of the bluff slope. The remaining nine lowest evaluated safety factors ranged from 0.92 to 0.98.

Of the 20 probabilistic stability analyses conducted, the lowest safety factors ranged from 0.41 to 0.90. Of the total of 200 failure surfaces evaluated, all of the surfaces 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.

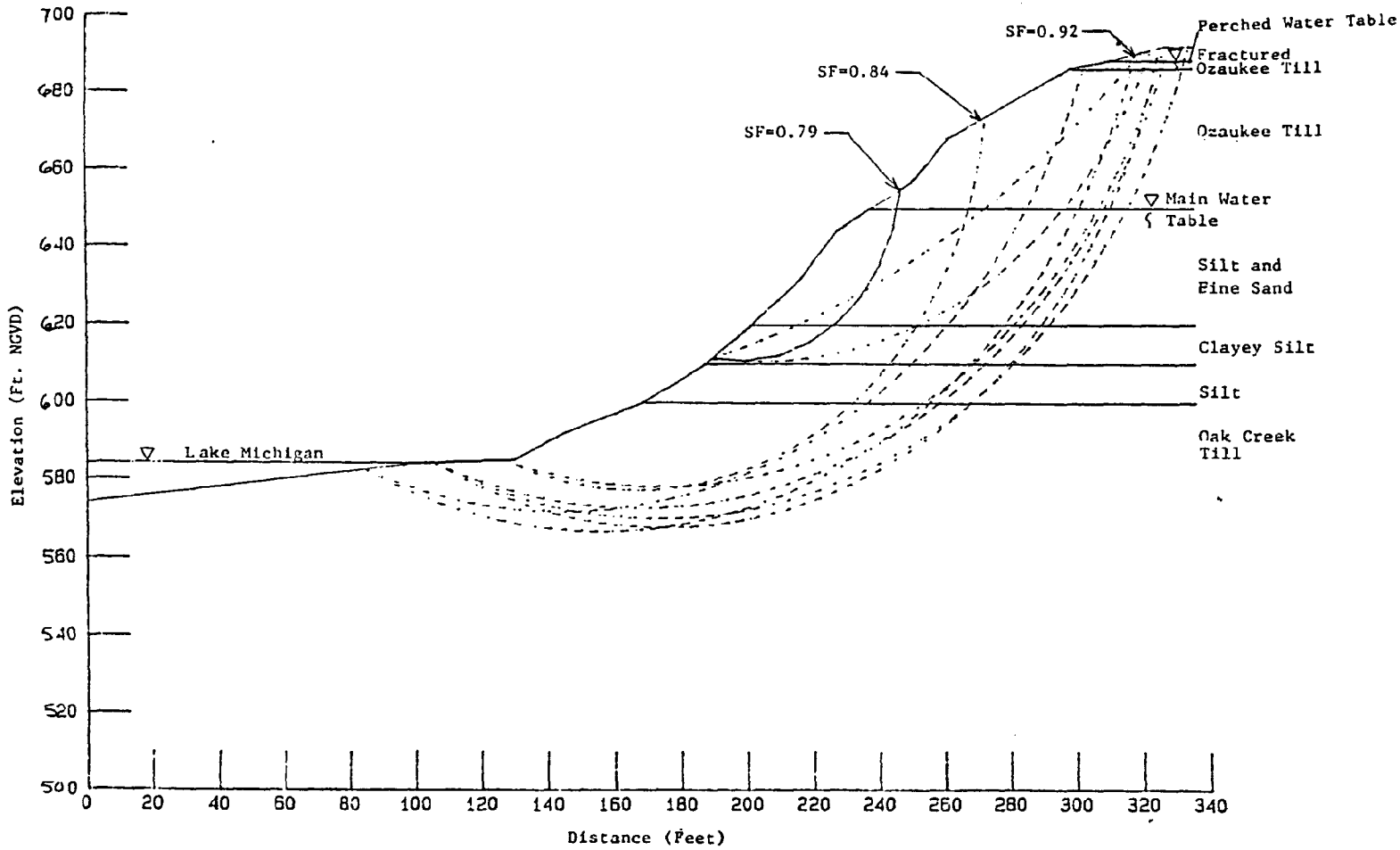
Section 90 was also considered to have an unstable bluff slope with respect to translational sliding. This was due in part to the lack of vegetative cover on the lower portion of the bluff slope, and in part to the relatively steep angle of the bluff slope. Within the lower portion of the bluff slope, 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 surveys 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. Even if the lake levels would return to the mean 20th Century levels, it is not anticipated that a significant beach would develop within Section 90.

To prevent 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 within 10 feet of the bluff edge. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Figure III- 98

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 93 : 6530 N. LAKE DRIVE



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

Bluff Analysis Section 91: The stability of the bluff slope within Section 91, which extends from 6600 to 6702 N. Lake Drive, was characterized by the use of Profile No. 94.

The results of the deterministic slope stability analysis, shown in Figure III-99, indicate that Profile No. 94 had an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.95, and was located within the middle portion of the bluff slope. The remaining nine lowest evaluated safety factors ranged from 0.97 to 1.07.

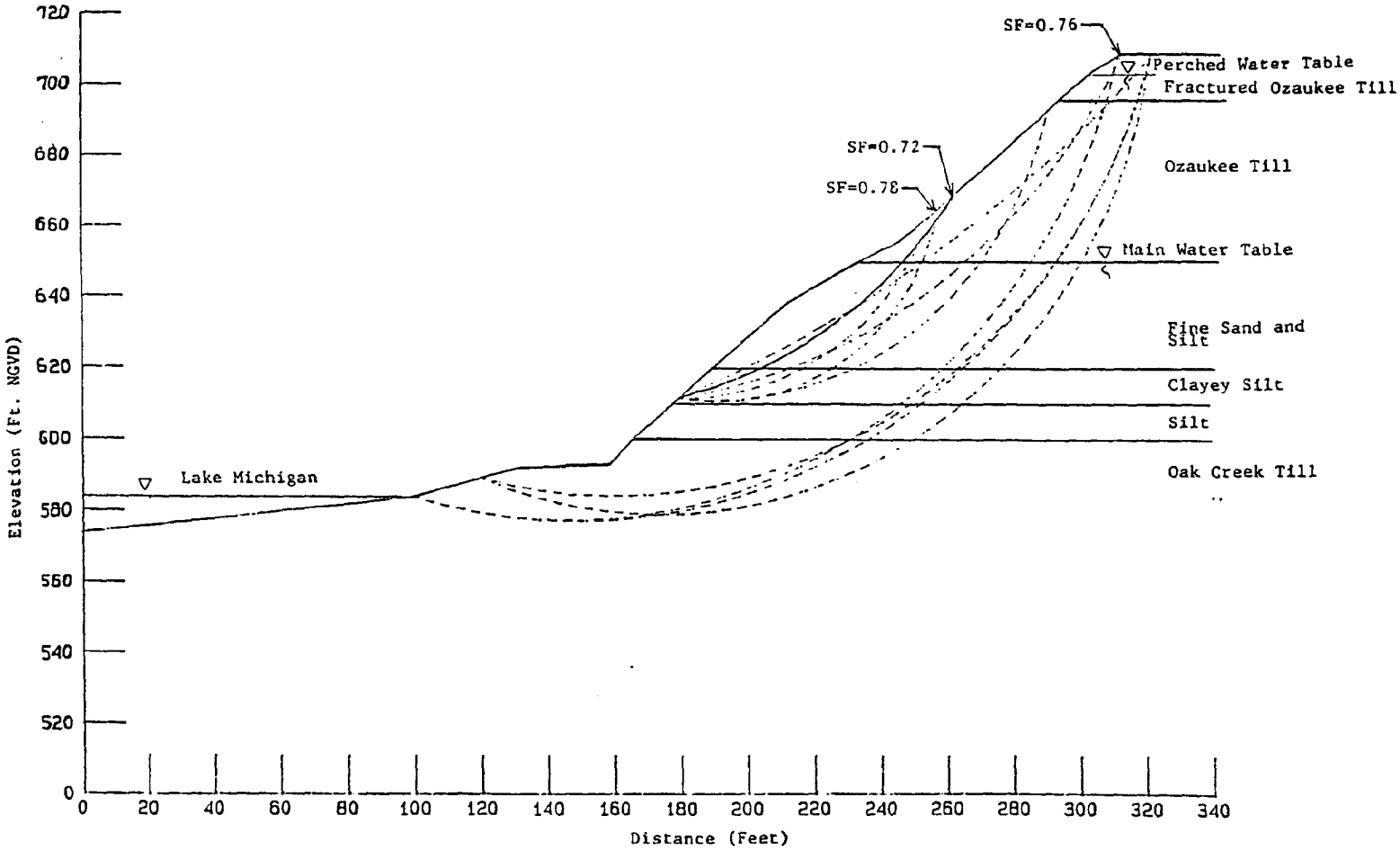
Of the 20 probabilistic stability analyses conducted, the lowest safety factors ranged from 0.45 to 1.03, with 18 of the failure surfaces, or 90 percent, having a safety factor of less than 1.0. Of the total of 200 failure surfaces evaluated, 180, or 90 percent of the surfaces, had safety factors of 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 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 surveys 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. If the lake levels would return to the mean 20th Century levels, it is anticipated that the resulting beach within Section 91 would approximate 60 feet in width.

Figure III- 99

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 94: 6610 N. LAKE DRIVE



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

To prevent 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 within 10 feet of the bluff edge. Bluff toe protection is recommended to prevent erosion from wave and ice action.

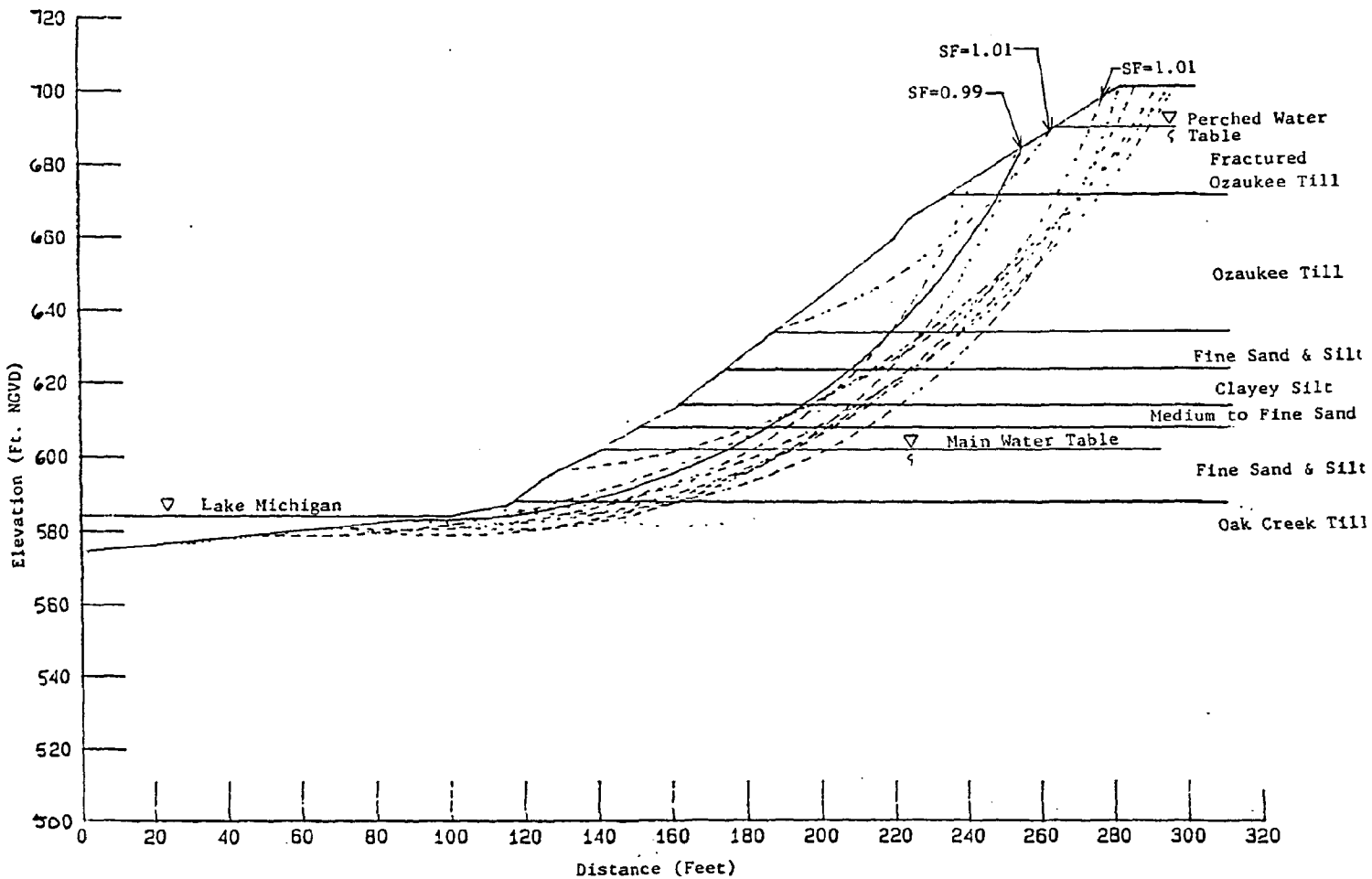
Bluff Analysis Section 92: The stability of the bluff slope within Section 92, which extends from 6720 N. Lake Drive to 6818 N. Barnett Lane, was characterized by the use of Profile No. 95 and Profile No. 96.

The results of the deterministic slope stability analyses shown in Figure III-100 and Figure III-101 for Profile No. 95 and Profile No. 96, respectively, indicate a potential 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 remaining nine lowest safety factors evaluated at Profile No. 95 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 remaining nine lowest safety factors evaluated at Profile No. 96 ranged from 0.99 to 1.14.

Of the 20 probabilistic stability analyses conducted for Profile No. 95, the lowest safety factors 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 total of 200 failure surfaces evaluated, 123, or 62 percent of the surfaces, had safety factors of less than 1.0. Of the 20 probabilistic stability analyses conducted for Profile No. 96, the lowest safety factors 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 total of 200 failure surfaces evaluated, 22, or 11 percent of the surfaces, 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.

Figure III- 100

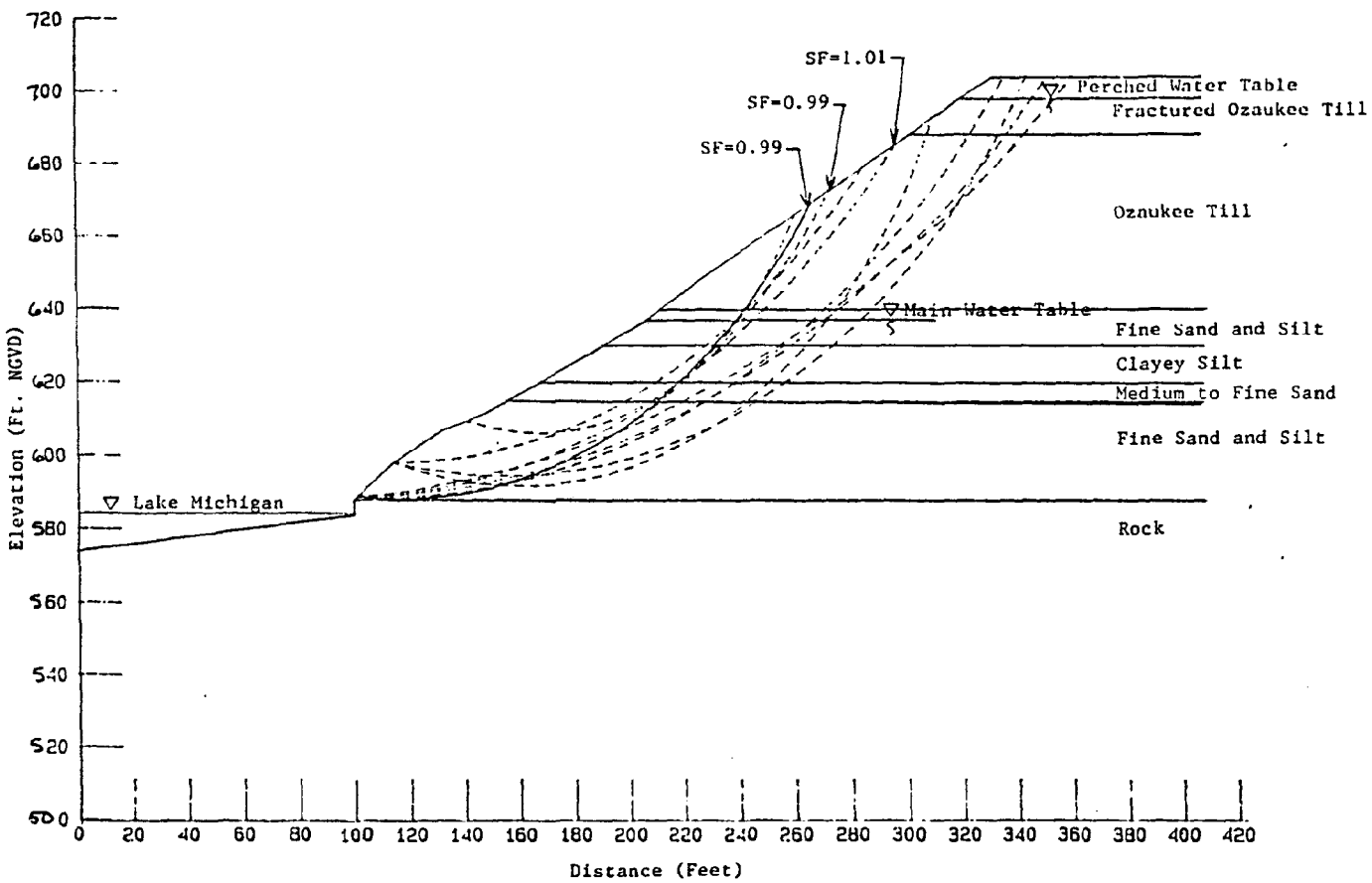
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 95 : 6720 N. LAKE DRIVE



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

Figure III-101

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 96: 6818 N. LAKE DRIVE



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

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 surveys conducted in the Summer of 1986. There were no shore protection structures present within the section during the summer field surveys. If the lake levels would return to the mean 20th Century levels, it is anticipated that the resulting beach within portions of Section 92 would be as much as 40 feet wide.

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 expected 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 installed and that a good vegetative cover be maintained on the bluff slope. The bluff toe protection measures installed in 1987 should be maintained to prevent erosion from wave and ice action.

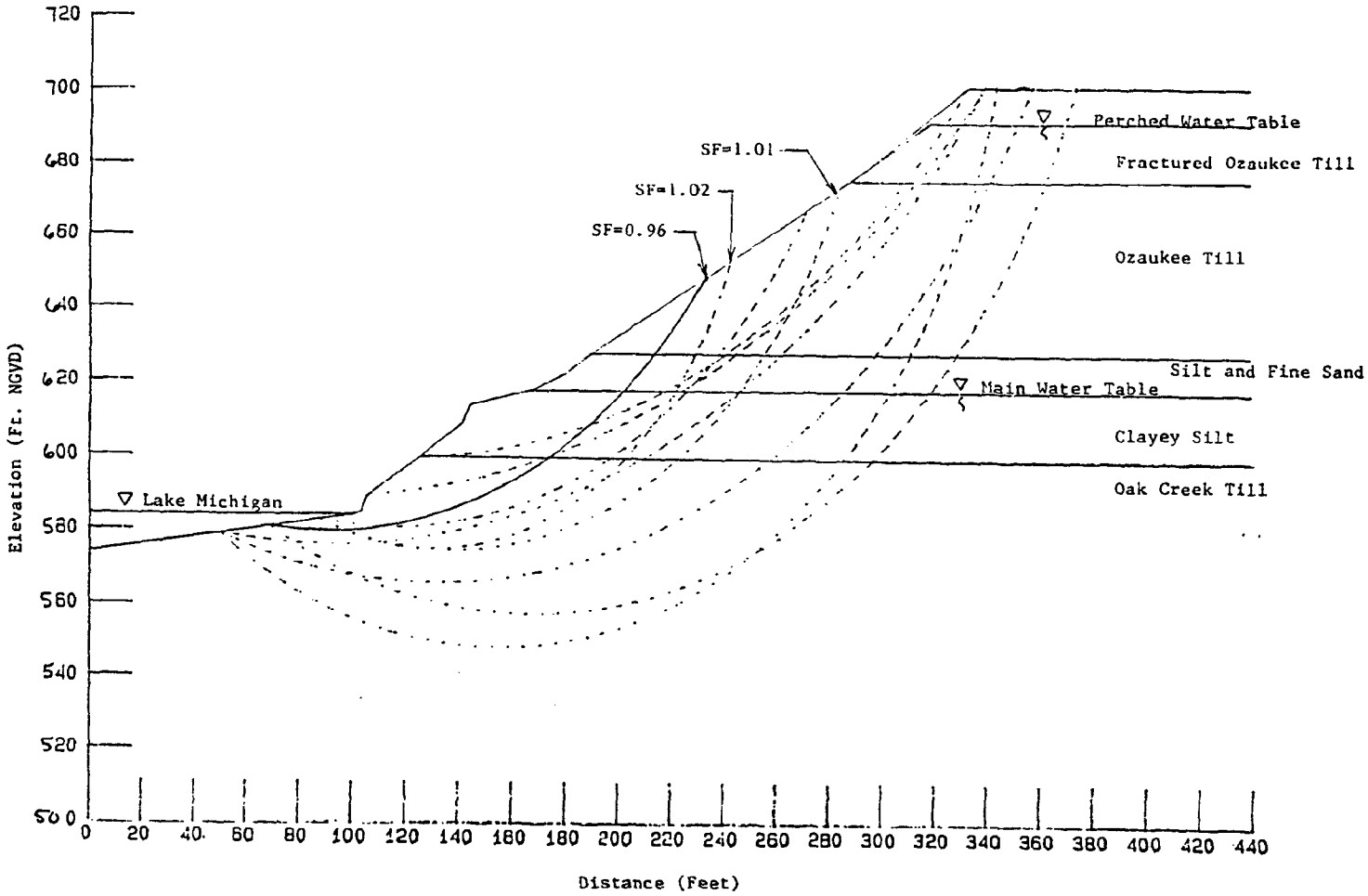
Bluff Analysis Section 93: The stability of the bluff slope within Section 93, which extends from 6820 to 6840 N. Barnett Lane, was characterized by the use of Profile No. 97.

The results of the deterministic slope stability analysis, shown in Figure III-102 for Profile No. 97, indicate a potential 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 remaining nine lowest safety factors ranged from 1.01 to 1.21.

Of the 20 probabilistic stability analyses conducted, the lowest safety factors ranged from 0.60 to 1.35, with 8 of the failure surfaces, or 40 percent,

Figure III-102

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 97: 6840 N. LAKE DRIVE



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

having a safety factor less than 1.0. Of the total of 200 failure surfaces evaluated, 51, or 26 percent of the surfaces, 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. This was due to the good vegetative growth that 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 increase.

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. If the lake levels would return to the mean 20th Century levels, it is anticipated that the resulting beach within Section 93 would approximate 30 feet in width.

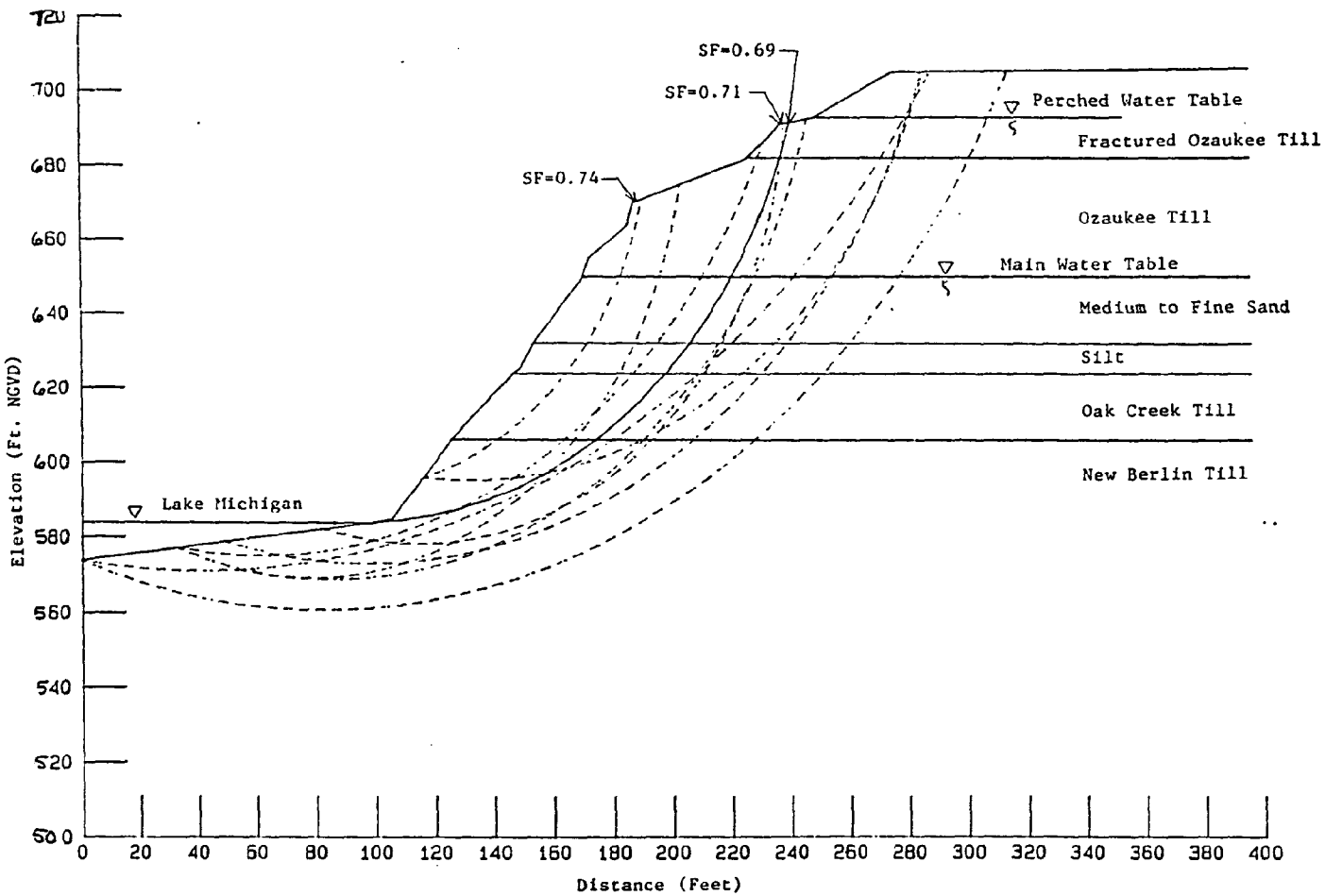
To prevent rotational sliding, as well as to provide protection against wave and ice action at the toe of the bluff, it is recommended that bluff toe protection be provided within Section 93. It is also recommended that a good vegetative cover be maintained on the bluff slope to prevent translational sliding.

Bluff Analysis Section 94: The stability of the bluff slope within Section 94, which extends from 7004 to 7010 No. Barnett Lane, was characterized by the use of Profile No. 98.

The results of the deterministic slope stability analysis, shown in Figure III-103, indicate that Profile No. 98 had an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile

Figure III-103

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 98 : 6960 N. LAKE DRIVE



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

site had a safety factor of 0.69, and was located within the lower portion of the bluff slope. The remaining nine lowest safety factors evaluated ranged from 0.71 to 1.00.

Of the 20 probabilistic stability analyses conducted, the lowest safety factors ranged from 0.51 to 0.73. Of the total of 200 failure surfaces evaluated, 196 of the failure surfaces, 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 34 during the field surveys 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. If the lake levels would return to the mean 20th Century levels, it is anticipated that the resulting beach within Section 94 would approximate 20 feet in width.

To prevent 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 within 10 feet of the bluff edge. Bluff toe protection is recommended to prevent erosion from 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. Special consideration must also be given to this section in the evaluation of the erosion problems because of the vulnerable location of the Beach Drive sanitary sewer, which extends along the Lake Michigan shoreline as shown in Map III-1. For the purposes of this

site had a safety factor of 0.69, and was located within the lower portion of the bluff slope. The remaining nine lowest safety factors evaluated ranged from 0.71 to 1.00.

Of the 20 probabilistic stability analyses conducted, the lowest safety factors ranged from 0.51 to 0.73. Of the total of 200 failure surfaces evaluated, 196 of the failure surfaces, 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.

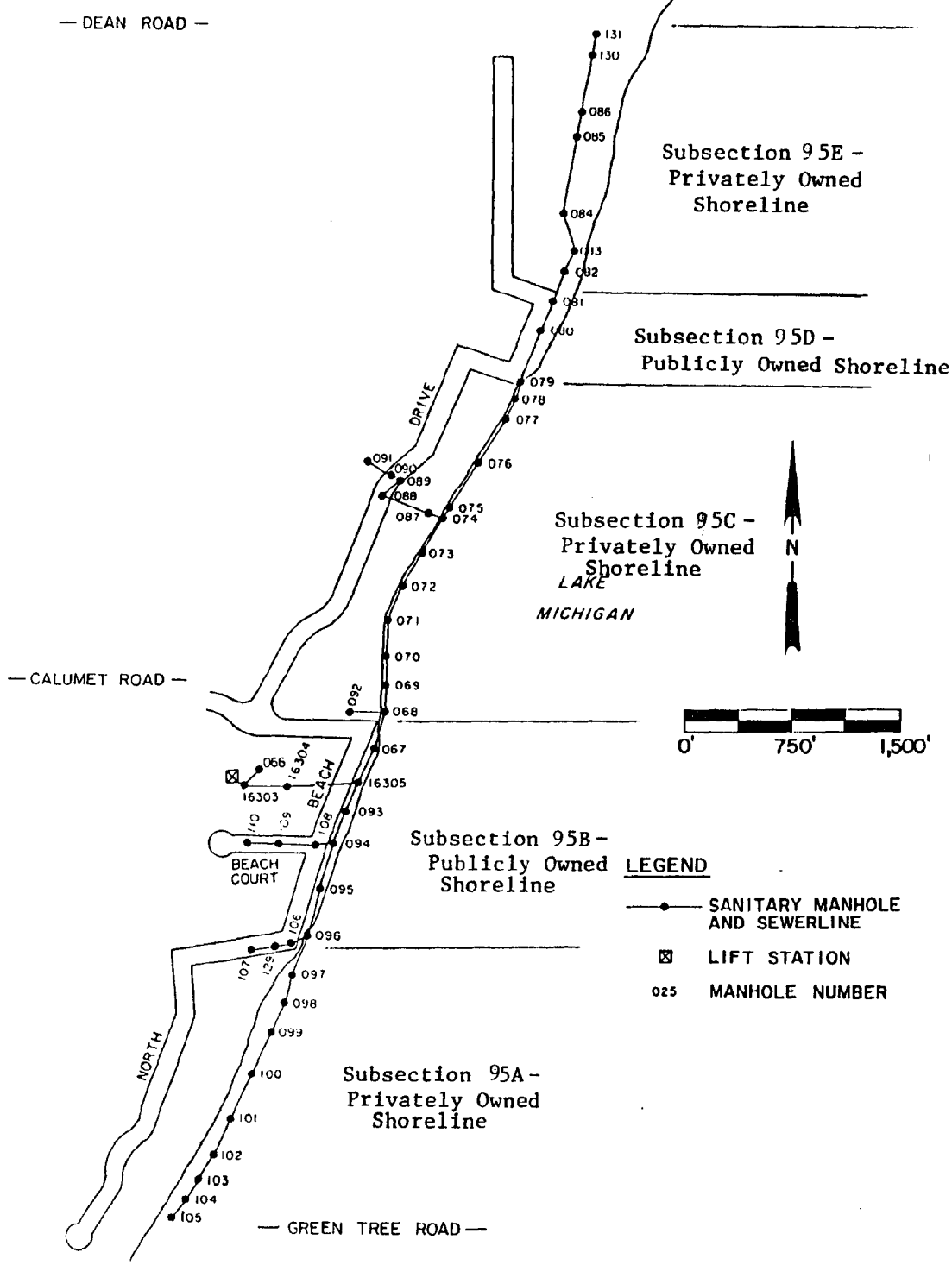
Bluff toe erosion was observed along the entire shoreline of Section 34 during the field surveys conducted in 1986, and was identified as a major cause of bluff slope failure. Aside from a collapsed groin, no shore protection structures were present within this section. If the lake levels would return to the mean 20th Century levels, it is anticipated that the resulting beach within Section 94 would approximate 20 feet in width.

To prevent 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 within 10 feet of the bluff edge. Bluff toe protection is recommended to prevent erosion from 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. Special consideration must also be given to this section in the evaluation of the erosion problems because of the vulnerable location of the Beach Drive sanitary sewer, which extends along the Lake Michigan shoreline as shown in Map III-1. For the purposes of this

Map III - 1

BEACH DRIVE SANITARY SEWER AND BLUFF ANALYSIS SECTION 95 SUBSECTIONS



Source: Donohue Engineers & Architects and SEWRPC.

analysis the section was divided into five subsections based on ownership. As shown on Map III-1, three of the subsections, which include about 6,990 feet, or about 75 percent of the total shoreline within the section, are in private ownership. The two remaining subsections, containing about 2,320 feet, or about 25 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 and includes 2,390 feet, or 26 percent of the total shoreline, within Section 95. The shoreline within this subsection is entirely 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, were not protected by any onshore structures and were eroding. If the lake levels would return to the mean 20th Century levels, it is anticipated that the resulting beach with Subsection 95A would approximate 40 feet in width.

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 are just slightly above the lake level and are extremely vulnerable to wave and ice action. Within the southern portion of this subsection, continued erosion could expose the sewer pipe which lies only one to two feet below the lake bottom.

Analysis Subsection 95B--Subsection 95B includes the shoreline area east of the southern portion of N. Beach Drive which lies adjacent to the lake. It includes about 1,600 feet, or 17 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 turn-around point of N. Beach Drive, lies a bulkhead is 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. If the lake levels would return to the mean 20th Century levels, it is anticipated that the resulting beach within Subsection 95B would approximate 50 feet in width.

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.

Analysis Subsection 95C--Subsection 95C extends from 7540 to 7966 N. Beach Drive, and includes 3,000 feet, or 32 percent of the total shoreline within Section 95. The shoreline within this subsection is entirely 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 2,640 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. If the lake levels would return to the mean 20th Century levels, it is anticipated that the resulting beach within Subsection 95C would approximate 40 feet in width.

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 the study period, beaches were eroding rapidly. Should beach erosion continue or the lake levels remain relatively high, the resulting erosion could expose the manholes and sewer to wave and ice attack.

Analysis Subsection 95D: Subsection 95D includes the shoreline area east of the northern portion of N. Beach Drive 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 entire subsection contained a revetment composed of blocks and concrete rubble. In the field surveys 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. If the lake levels would return to the mean 20th Century levels, it is anticipated that the resulting beach within Subsection 35D would approximate 20 feet in width.

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, and includes 1,600 feet, or 17 percent of the total shoreline within Section 95. The shoreline within this subsection is entirely 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 1,140 feet, or 71 percent of the shoreline, was protected by onshore structures. However, only 240 feet, or 15 percent of this subsection, was protected by structures which had no observable failures, or were not in need of any significant maintenance work. About 460 feet, or 29 percent of the shoreline, was not protected by onshore structures and was eroding. If the lake levels would return to the mean 20th Century levels, it is anticipated that the resulting beach within Subsection 95E would approximate 40 feet in width.

The portion of the sanitary sewer included within Subsection 95E was located approximately 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 which did not require maintenance, about 71 percent of the shoreline was protected by structures which 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.

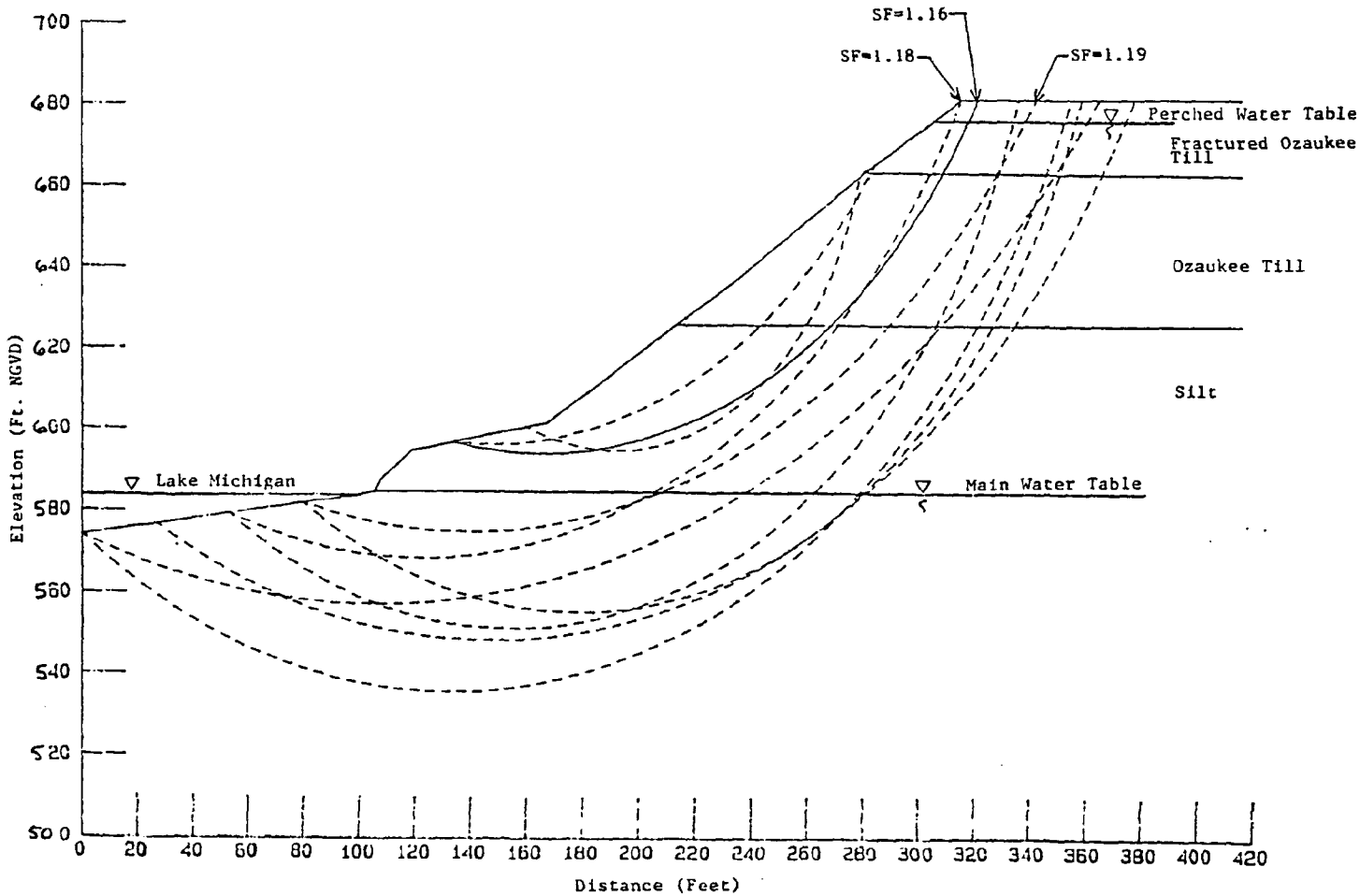
Bluff Analysis Section 96: The stability of the bluff slope within Section 96, which is located within Doctors Park, was characterized by the use of Profile No. 99 and Profile No. 100.

The results of the deterministic slope stability analyses shown in Figure III-104 and Figure III-105 for Profile No. 99 and Profile No. 100, respectively, indicate stable bluff slopes with respect to rotational sliding. The lowest failure surface evaluated at Profile No. 99 had a safety factor of 1.16, and was located within the upper two-thirds of the bluff slope. The remaining nine lowest safety factors evaluated 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 evaluated had safety factors ranging from 1.23 to 1.37.

Of the 20 probabilistic stability analyses conducted for Profile No. 99, the lowest safety factors 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 total of 200 failure surfaces evaluated, 12, or 6 percent of the surfaces, had safety factors of less than 1.0. Of the 20 probabilistic stability analyses conducted for Profile No. 100, the lowest safety factors 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 total of 200 failure surfaces evaluated, 26, or 13 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 96 was considered to have a stable bluff slope with respect to rotational sliding.

Figure III-104

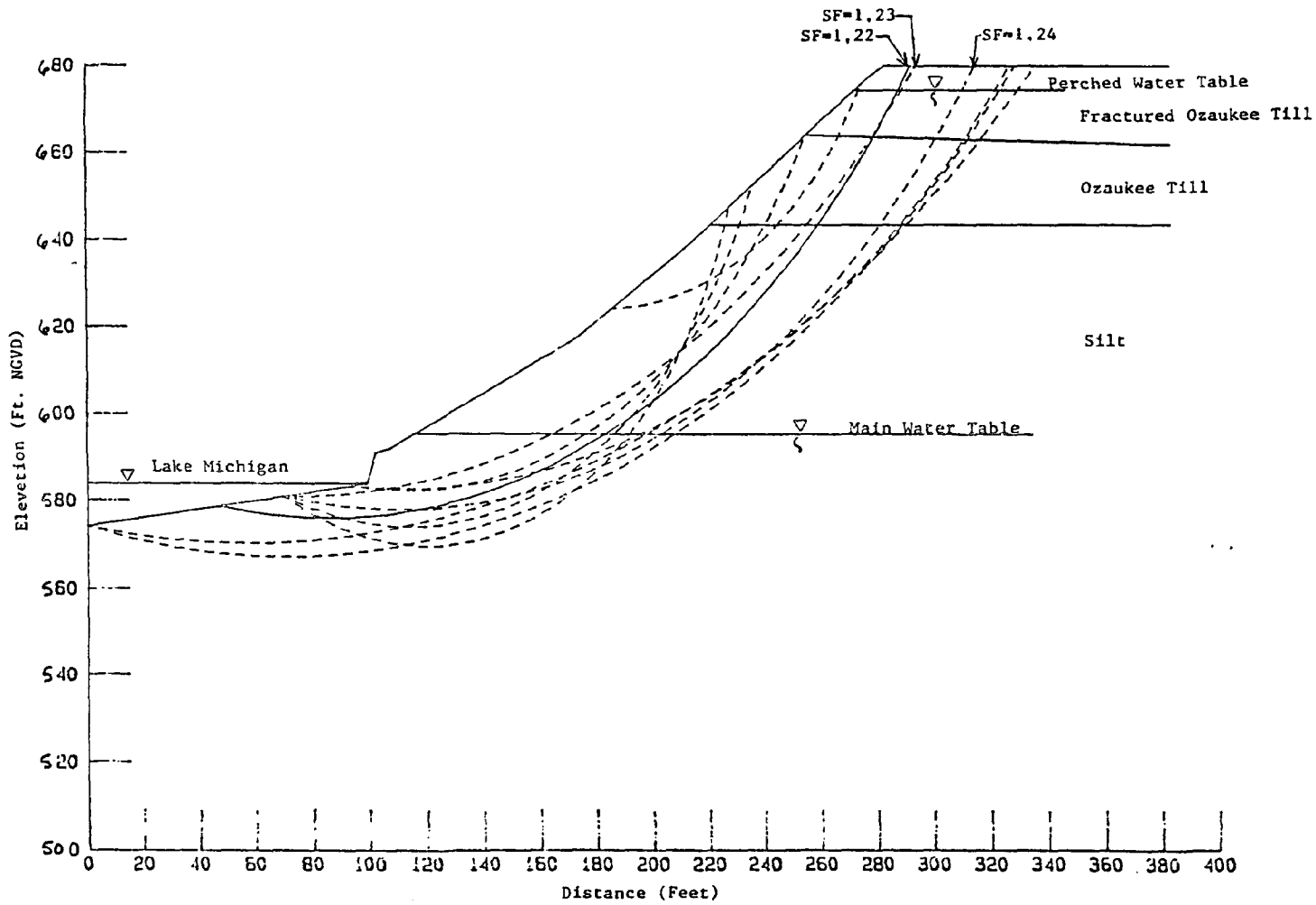
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 99 : DOCTORS PARK



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

Figure III-105

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 100 DOCTORS PARK



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

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. If the lake levels would return to the mean 20th Century levels, it is anticipated that the resulting beach within Section 96 would approximate 100 feet in width.

No measures are needed to prevent rotational or translational sliding within Bluff Analysis Section 96. Maintenance or reconstruction of the existing concrete bulkhead to prevent wave overtopping is recommended.

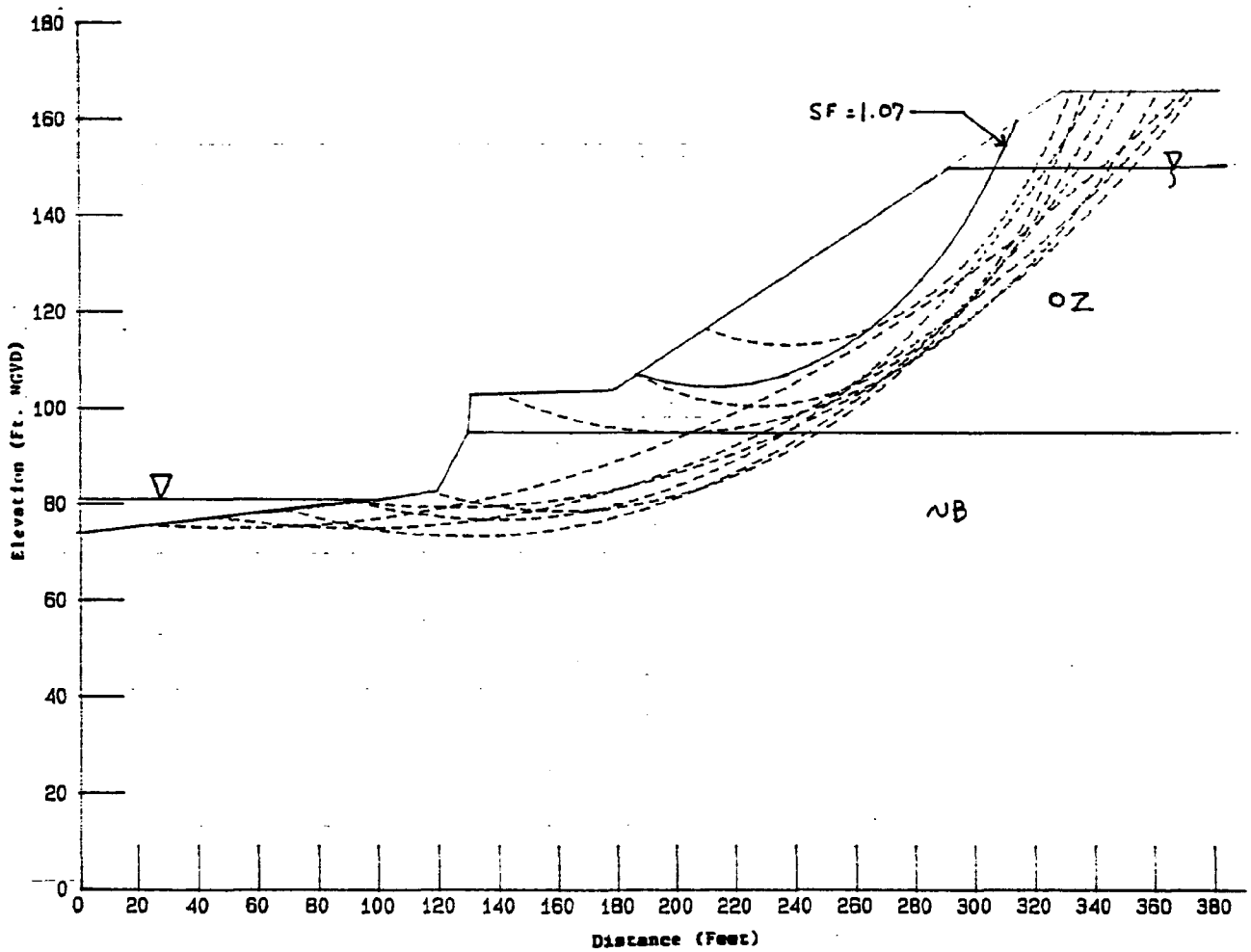
Bluff Analysis Section 97: The stability of the bluff within Section 97, which extends from the Schlitz Audubon Center to 9360 N. Lake Drive, was characterized by the use of Profile No. 101.

The results of the deterministic slope stability analyses, shown in Figure III-106, indicate that Profile No. 101 had 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 remaining 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. Of the 20 probabilistic stability analyses conducted, the lowest safety factors ranged from 0.70 to 1.68, with six, or 30 percent, having a safety factor less than 1.0. Of the total of 200 failure surfaces evaluated, 23 surfaces, or 11 percent, had safety factors of less than 1.0. The results of the probabilistic

Figure III -106

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 101



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

analyses indicate that the bluff slope is marginally stable with respect to rotational sliding.

Based on both the deterministic and probabilistic slope stability analyses, and on observed bluff conditions, Section 97 was considered to have a stable bluff slope with respect to rotational sliding.

Section 97 was also considered to have a stable slope with respect to translational sliding. The bluff slope is very well vegetated, and is protected by a wide terrace.

Erosion of the base of the terrace was observed during the field survey conducted during the fall of 1987. Although this did not affect the stability of the bluff slope, it was recognized as a potential threat to bluff stability, particularly in the northern end of the section, where the terrace was more narrow.

Bluff toe protection is necessary to ensure continued bluff slope stability in this section.

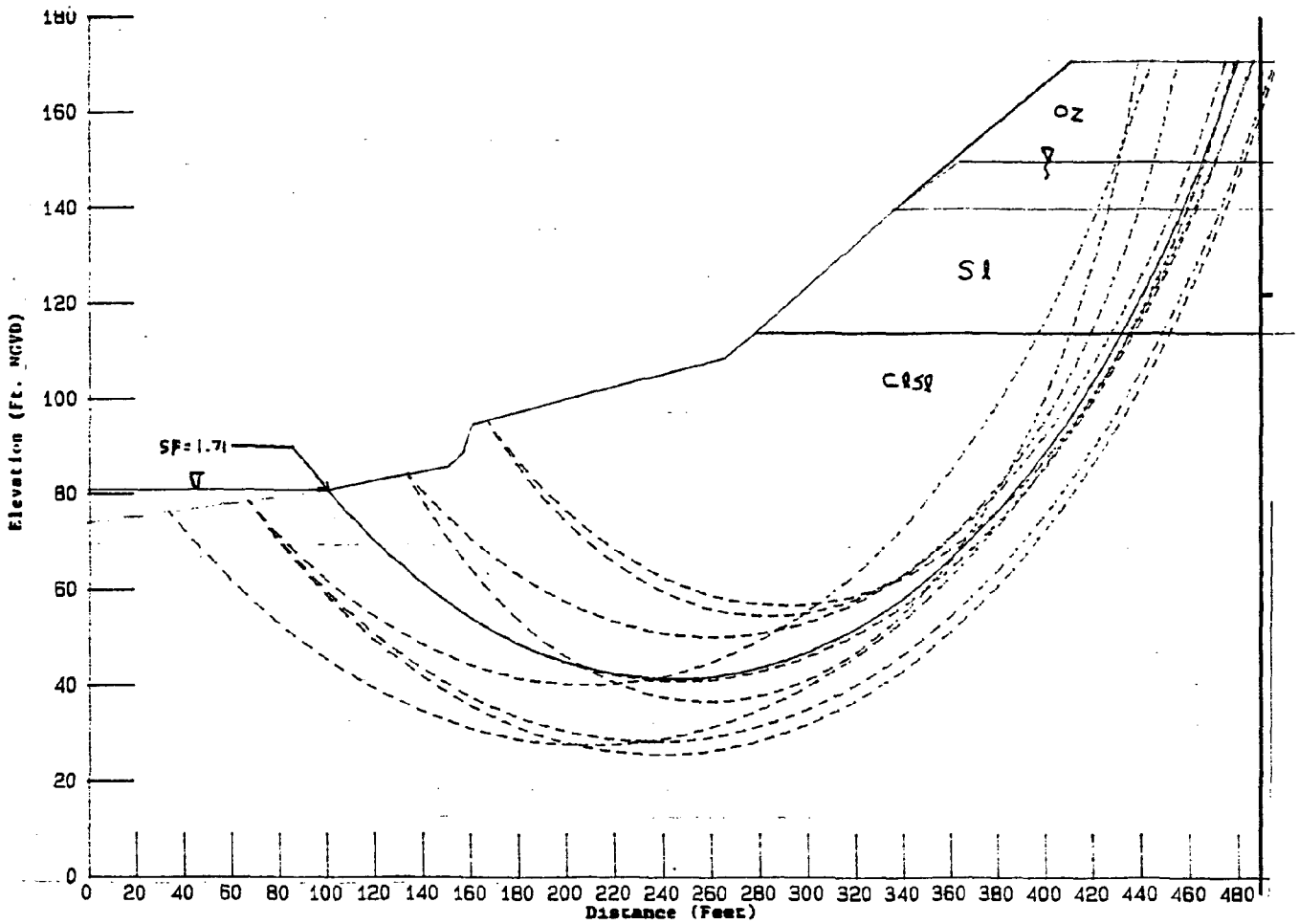
Bluff Analysis Section 98: The shoreline of Section 98, which extends from 1470 to 1434 E. Bay Point Road, was protected by a rip-rap revetment and a bulkhead. These structures appeared to be providing adequate protection of the base of the terrace which fronted the bluff slope. During the field survey conducted in the fall of 1987, no toe erosion was reported. To ensure continued shoreline protection in Section 98, these structures must be maintained.

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, was characterized by the use of Profile No. 102.

The results of the deterministic slope stability analyses, as shown in Figure III-107, indicate that Profile No. 102 had 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 remaining nine lowest safety factors ranged from 1.71 to 1.79.

Figure III -107

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 102



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

Based on the deterministic slope stability analysis and on observed bluff conditions, Section 99 was considered to have a stable bluff slope with respect to rotational sliding.

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 was observed in Section 99 during the field survey conducted during the fall of 1987 and was reported to be affecting only the wide terrace at the base of the bluff. It was recognized that continued erosion of the terrace would negatively impact bluff stability in this section.

Toe protection is, therefore, considered necessary to ensure long-term bluff stability in this section.

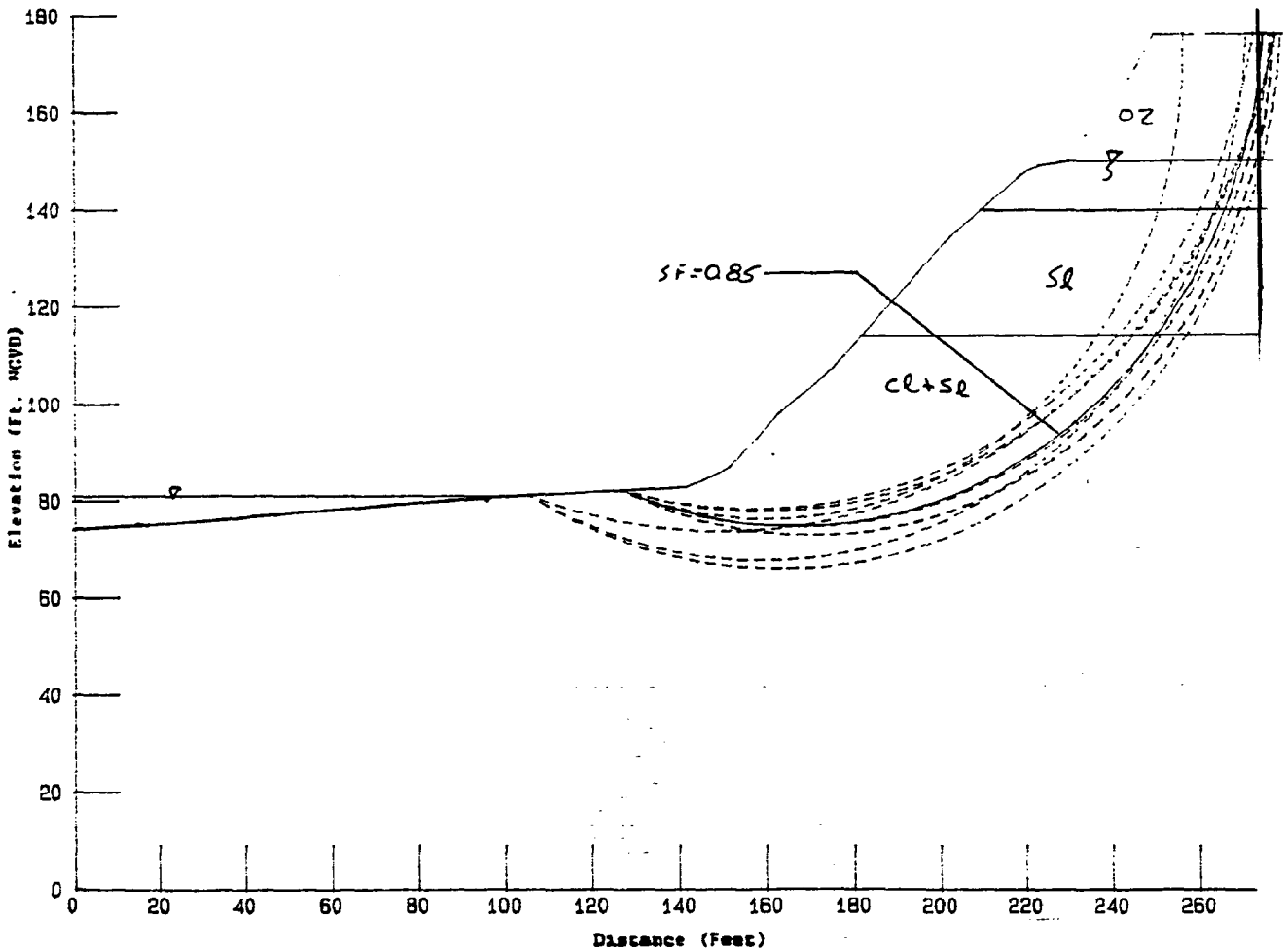
Bluff Analysis Section 100: The stability of the bluff in Section 100, which extends from 9400 to 9578 N. Lake Drive, was characterized by the use of Profile No. 103 and Profile No. 104.

The results of the deterministic slope stability analysis, shown in Figure III-108 and Figure III-109 for Profile No. 103 and Profile No. 104, respectively, indicate that Section 100 had 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 remaining nine lowest safety factors ranged from 0.86 to 0.95. The lowest failure surface calculated at Profile No. 104 had a safety factor of 0.91 and also included the entire bluff slope. The remaining nine lowest safety factors ranged from 0.91 to 1.02

Probabilistic slope stability analyses were also conducted for each profile site. Of the 20 probabilistic stability analyses conducted for Profile 59, the lowest safety factors ranged from 0.88 to 1.12, with 19,or 95 percent, having a safety factor less than 1.0. Of the total of 200 failure surfaces evaluated for Profile No. 103, 142, or 71 percent of the surfaces, had safety factors of less than 1.0. Of the 20 probabilistic stability analyses conducted for Profile No. 104, the lowest safety factors ranged from 0.23 to

Figure III -108

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 103



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

1.10, with 17, or 85 percent, having a safety factor less than 1.0. Of the total of 200 failure surfaces evaluated at Profile No. 104, 127, or 63 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 condition, 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 steep angle of the bluff slope. Groundwater seepage was observed during the field survey conducted during the fall of 1987, and was considered to contribute to the tendency of the bluff slope to fail by translational sliding.

Severe toe erosion contributing to the overall instability of the bluff slope was observed throughout most of Section 100, with the exception of the property at 1240 Donges Bay Court, where a concrete bulkhead and fill project were cited. Here, evidence of overtopping and flanking were reported during the 1987 field survey, allowing some erosion of the toe of the bluff.

An integrated shoreline protection plan, including bluff slope regrading and bluff toe protection, are needed to fully stabilize the bluff in this section.

Summary of the Evaluation of Bluff Analysis Sections

The analyses of each of the 100 bluff analysis sections were conducted to better quantify the risks of bluff slope failure with respect to rotational sliding, translational sliding, and bluff toe erosion. A summary of the deterministic and probabilistic slope stability analysis results for each profile site is set forth in Table III-7. The evaluations of the bluff conditions are presented in Table III-8. While these summaries present the results of the evaluation of bluff analysis sections, it must be recognized that the bluff conditions within any given section can vary substantially.

With respect to rotational sliding, 34 bluff analysis sections, which cover 48,700 feet, or 31 percent of the total study area shoreline, were found to have stable bluff slopes. A total of 16 bluff analysis sections, which cover 16,770 feet, or 10 percent of the total study area shoreline, were found to have marginal bluff slopes. A total of 43 bluff analysis sections, which

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Table III-7

SUMMARY OF DETERMINISTIC AND PROBABILISTIC SLOPE STABILITY ANALYSIS RESULTS FOR ROTATIONAL SLIDING

			Deterministic Analysis		Probabilistic Analysis			
Civil Division	Bluff Analysis Section	Profile Site	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	Stability Classification of Section
City of Oak Creek	2	1	1.0	0	0.73-1.37	55	43	M
		2	1.43	0	0.79-1.36	45	32	--
		3	1.18	0	0.99-1.62	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	0.36-0.83	100	88	U
	6	10	0.86	100	0.65-1.03	95	49	U
	7	11	0.87	80	0.17-0.83	100	100	U
	9	12	0.87	70	0.21-0.92	100	85	U
	10	13	0.92	30	0.70-1.14	70	45	M
	11	14	0.90	70	0.38-1.03	90	78	U
	13	15	1.48	0				S
City of South Milwaukee	14	16	0.90	100	0.36-0.84	100	100	U
		17	0.74	100				--
	15	18	0.74	100	0.70-1.10	95	94	U
	16	19	1.13	100	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
	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	0.29-0.94	100	90	U
	24	27	1.23	0	0.72-1.19	70	32	M
	25	28	0.86	80				U
City of Cudahy	26	29	0.73	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	0.50-0.89	100	100	U
	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	70	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
	42	47	1.33	0				S
	43	48	0.85	70	0.74-1.15	65	23	U
		49	0.81	70	0.50-1.15	95	45	
		50	0.81	80	0.52-1.04	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

-continued-

Table III-7 (cont'd)

Civil Division	Bluff Analysis Section	Profile Site	Deterministic Analysis: Lowest Safety Factor	Deterministic Analysis: Percent of 10 Lowest Safety Factors <1.0	Probabilistic Analysis: Range of Lowest Safety Factors	Probabilistic Analysis: Percent of Lowest Safety Factors < 1.0	Probabilistic Analysis: Percent of 10 Lowest Safety Factors Per Model Run < 1.0	Stability Classification of Section
City of Milwaukee	48	55	1.21	0	1.04-1.56	0	0	S
	50	56	1.12	0	0.78-1.49	25	3	S
	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	U
	71	68	0.72	20	--	--	--	S
Village of Whitefish Bay		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	U
	79	80	1.39	0	--	25	--	S
	80	81	1.07	0	0.76-1.44	25	13	U
	81	82	1.69	0	--	--	--	S
		83	1.75	0	--	--	--	U
	82	84	0.95	100	0.47-1.12	85	80	S
	83	85	1.14	0	--	--	--	--
	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	U
	88	90	0.82	100	0.63-1.00	95	68	U
Village of Fox Point		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
	94	98	0.69	90	0.51-0.73	100	98	U
	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
	99	102	1.71	0				S
	100	103	0.85	100	0.28-1.12	95	71	U
		104	0.91	90	0.23-1.10	85	63	--

NOTE: M - Marginal
U - Unstable
S - Stable

Source: SEWRPC.

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Table III-8

SUMMARY OF THE EVALUATIONS OF BLUFF CONDITIONS

Civil Division	Bluff Analysis Section	Profile Number	Shoreline Length (feet)	Potential for Rotational Sliding	Potential for Translational Sliding	Existing Bluff Toe Erosion[b]
City of Oak Creek	1	--	4,470	Stable	Stable	Slight
	2	1,2,3	2,820	Marginal	Unstable	Slight
	3	4,5,6	2,930	Unstable	Unstable	Severe
	4	7,8	1,980	Unstable	Unstable	Severe
	5	9	1,070	Unstable	Unstable	Severe
	6	10	1,170	Unstable	Unstable	Severe
	7	11	1,000	Unstable	Unstable	Slight
	8	--	540	Unstable	Unstable	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	Stable	Stable	Slight
	17	20	440	Unstable	Unstable	Severe
	18	21,22	1,880	Unstable	Unstable	Severe
	19	--	3,180	Stable	Stable	Slight
	20	23	1,280	Unstable	Unstable	Severe
	21	24	1,060	Marginal	Stable	Severe
	22	25	950	Unstable	Marginal	Severe
	23	26	1,200	Unstable	Unstable	Severe
	24	27	1,910	Marginal	Unstable	Severe
	25	28	880	Unstable	Unstable	Moderate
City of Cudahy	26	29	660	Unstable	Unstable	Severe
	27	30,31	1,850	Unstable	Marginal	Severe
	28	32	2,050	Unstable	Marginal	Severe
	29	33	770	Unstable	Unstable	Severe
	30	34	1,760	Unstable	Unstable	Severe
	31	35	600	Unstable	Unstable	Severe
	32	--	340	Stable	Stable	Slight
	33	36	2,060	Unstable	Unstable	Moderate
	34	37,38	1,780	Marginal	Stable	Slight
	35	39	650	Unstable	Marginal	Slight
	36	40	710	Marginal	Stable	Slight
	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	Moderate
	40	46	820	Stable	Stable	Moderate
	41	--	1,650	Stable	Stable	Slight
	42	47	940	Stable	Marginal	Slight
	43	48,49,50	1,370	Unstable	Unstable	Severe
	44	51	140	Unstable	Unstable	Moderate

Table III-8 (cont'd)

Civil Division	Bluff Analysis Section	Profile Number	Shoreline Length (feet)	Potential for Rotational Sliding	Potential for Translational Sliding	Existing Bluff Toe Erosion[b]
City of St. Francis (cont'd)	45	52	80	Marginal	Marginal	Moderate
	46	53	360	Marginal	Marginal	Moderate
	47	54	2,470	Stable	Stable	Slight
City of Milwaukee	48	55	1,420	Stable	Stable	Slight
	49	--	340	Stable	Stable	Slight
	50	56	1,130	Stable	Unstable	Slight
	51	--	570	Stable	Stable	Moderate
	52	--	450	Stable	Stable	Slight
	53	--	1,320	Stable	Stable	Slight
	54	--	1,360	Stable	Stable	Moderate
	55	--	14,750	--	--	Slight
	56	--	16,060	--	--	Slight
	57	--	3,210	--	--	Slight
	58	--	1,900	--	--	Slight
	59	--	3,540	--	--	Moderate
	60	--	2,210	--	--	Slight
	61	57	1,970	Stable	Stable	Slight
	62	58	950	Stable	Stable	Moderate
Village of Shorewood	63	59,60	300	Marginal	Marginal	Severe
	64	61	290	Stable (fill)	Stable	Slight
	65	62	1,710	Stable	Stable	Moderate
	66	63	170	Stable (fill)	Stable	Moderate
	67	64	380	Unstable	Unstable	Severe
	68	65	2,170	Marginal	Stable	Slight
	69	66	520	Stable	Stable	Severe
	70	67	240	Unstable	Stable	Severe
	71	68,70	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	1,410	Stable (fill)	Stable	Moderate
	78	23	1,060	Unstable	Stabel	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
	85	87	480	Unstable	Stable	Severe
	86	88	170	Unstable	Unstable	Moderate
	87	89	1,950	Marginal	Stable	Moderate
	88	90,91	1,150	Unstable	Unstable	Severe

Table III-8 (cont'd)

Civil Division	Bluff Analysis Section	Profile Number	Shoreline Length (feet)	Potential for Rotational Sliding	Potential for Translational Sliding	Existing Bluff Toe Erosion[b]
Village of Fox Point	89	92	320	Stable (fill)	Stable	Slight
	90	93	470	Unstable	Unstable	Severe
	91	94	510	Marginal	Stable	Slight
	92	95,96	770	Marginal	Marginal	Slight
	93	97	530	Marginal	Stable	Severe
	94	98	1,460	Unstable	Unstable	Severe
	95	—	9,310	--	--	Moderate
	96	89,100	600	Stable	Stable	Moderate
Village of Bayside	97	101	4,660	Stable	Stable	Moderate
	98	--	860	--	--	Slight
	99	102	1,280	Stable	Stable	Moderate
	100	103,104	1,320	Unstable	Unstable	Severe

NOTE: The evalutation of the stability of a bluff slope at individual lakeshore properties requires a site specific analysis by a professional geologist or geotechnical engineer.

Source: SEWRPC

cover 42,660 feet, or 27 percent of the total study area shoreline, were found to have unstable slopes. Bluff slope stability was not evaluated for the remaining seven sections consisting of the shoreline protected by the Milwaukee Harbor breakwater, the terrace directly north of the harbor, to the Linnwood water treatment plant, and the Fox Point terrace, which includes the remaining 50,980 feet, or 32 percent of the total study area shoreline.

With respect to translational sliding, 42 bluff analysis sections, which cover 57,120 feet, or 36 percent of the total study area shoreline, were considered to have stable bluff slopes. Fifteen bluff analysis sections, covering 11,260 feet, or 7 percent of the total study area shoreline, were considered to have marginal bluff slopes. Thirty-six bluff analysis sections, covering 39,750 feet, or 25 percent of the total study area shoreline, were considered to have unstable bluff slopes.

Thirty-four bluff analysis sections, covering about 77,700 feet, or 49 percent of the total study area shoreline, were exhibiting insignificant or slight bluff toe erosion in 1986. The remaining 66 bluff analysis sections, covering 81,410 feet, or 51 percent of the total study area shoreline, were exhibiting substantial erosion of the bluff toe. The erosion occurring within 41 bluff analysis sections, covering 40,490 feet, or 50 percent of the eroding shoreline, is considered to be affecting the overall stability of the bluff slopes.

The measures to protect the shoreline and stabilize the bluff slopes were identified for each of the 100 bluff analysis sections. The types of shore protection measures indicated are listed in Table III-9. 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; revegetation of the bluff slopes; and protection of the toe of the bluff against wave and ice action. The extent of the shoreline within each municipality associated with each of the indicated shore protection measures is summarized in Table III-10.

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, was indicated for 48 bluff analysis sections, which include about 45,490 feet, or 29 percent of the

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Table III-9

INDICATED SHORE PROTECTION MEASURES TO CONTROL
SHORELINE EROSION AND STABILIZE THE BLUFF SLOPES

Civil Division	Bluff Analysis Section	Bluff Toe Protection	Bluff Slope Revegetation	Surface Water Runoff Control	Groundwater Drainage	Bluff Slope Regrading
City of Oak Creek	1	--	--	--	--	--
	2	--	--	X	X	--
	3	X	--	--	--	X
	4	X	--	--	--	X
	5	X	--	--	--	X
	6	X	--	--	--	X
	7	X	--	--	--	X
	8	--	--	--	--	--
	9	X	--	--	--	X
	10	X[b]	--	--	--	X
	11	X	--	--	--	X
	12	--	--	--	--	--
	13	--	--	--	--	--
City of Milwaukee	14	X	--	--	--	X
	15	X[c]	--	--	--	X[c]
	16	--	--	--	--	--
	17	X	--	--	--	X
	18	X	--	--	--	X
	19	--	--	--	--	--
	20	X	--	--	--	X
	21	X	--	--	X	--
	22	X	--	--	--	X
	23	X	--	--	--	X
	24	X	--	--	--	X
	25	X	--	--	--	X
City of Cudahy	26	X	--	--	--	X
	27	X	--	--	--	X
	28	X	--	--	--	X
	29	X	--	--	--	X
	30	X	--	--	--	X
	31	X	--	--	--	X
	32	--	--	--	--	--
	33	X	--	--	--	X
	34	X	X	--	--	--
	35	X	X	--	X	--
	36	X	X	--	--	--
	37	X	--	--	--	X
City of St Francis	38	X	--	--	--	X
	39	X	--	--	--	X
	40	X	--	--	--	--
	41	--	--	--	--	--

Table III-9 (cont'd)

Civil Division	Bluff Analysis Section	Bluff Toe Protection	Bluff Slope Revegetation	Surface Water Runoff Control	Groundwater Drainage	Bluff Slope Regrading
City of St. Francis (cont'd)	42	X	X	--	--	--
	43	X	--	--	--	X
	44	X	--	--	--	X
	45	X	--	--	--	X
	46	X	--	--	--	X
City of Milwaukee	47	--	--	--	--	--
	48	X	--	--	--	--
	49	--	--	--	--	--
	50	X	--	--	--	--
	51	X	--	--	--	--
	52	--	--	--	--	--
	53	--	--	--	--	--
	54	X	--	--	--	--
	55	--	--	--	--	--
	56	--	--	--	--	--
	57	--	--	--	--	--
	58	--	--	--	--	--
	59	X	--	--	--	--
	60	X	--	--	--	--
	61	--	X	--	--	--
	62	X	X	X	--	--
	63	X	X	X	--	X
Village of Shorewood	64	--	X	X	--	X
	65	X	--	--	--	--
	66	X	--	--	--	--
	67	X	--	--	--	X
	68	X	--	--	X	--
	69	X	--	--	--	--
	70	X	X	--	--	--
	71	--	--	--	--	--
Village of Whitefish Bay	72	X	--	--	--	X
	73	X	--	--	--	X
	74	X	--	--	--	X
	75	X	--	--	--	X
	76	X	--	--	--	X
	77	X	--	--	--	X
	78	X	--	--	X	--
	99	X	--	--	--	--
	80	--	--	--	--	X
	81	X	--	--	--	--
	82	X	--	--	--	X
	83	X	--	--	--	X
	84	X	--	--	--	X
	85	--	--	--	--	--
	86	--	--	--	--	X
	87	X	X	--	X	--

Table III-9 (cont'd)

Civil Division	Bluff Analysis Section	Bluff Toe Protection	Bluff Slope Revegetation	Surface Water Runoff Control	Groundwater Drainage	Bluff Slope Regrading
Village of Fox Point	88	X	--	--	--	X
	89	--	--	--	--	--
	90	X	--	--	--	X
	91	X	--	--	--	--
	92	X	X	--	X	--
	93	--	--	--	--	--
	94	--	--	--	--	--
	95	X	--	--	--	--
	96	X	--	--	--	--
Village of Bayside	97	X	--	--	--	--
	98	--	--	--	--	--
	99	X	--	--	--	--
	100	X	--	--	--	X

[a]Upper portion of slope.

[b]Additional protection in southern end of section.

[c]Applies only to segment not protected by Yacht Club.

Source: SEWRPC

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Table III-10

EXTENT OF INDICATED SHORE PROTECTION MEASURES IN MILWAUKEE COUNTY

Civil Division	Shore Protection Measure									
	Bluff Toe Protection		Bluff Slope Revegation		Surface Water Runoff Control		Groundwater Drainage		Bluff Slope Regrading	
	Shoreline Length (feet)	Percent of Shoreline	Shoreline Length (feet)	Percent of Shoreline	Shoreline Length (feet)	Percent of Shoreline	Shoreline Length (feet)	Percent of Shoreline	Shoreline Length (feet)	Percent of Shoreline
City of Oak Creek..	10,410	46	0	0	2,820	12	2,820	12	10,410	46
City of South Milwaukee	11,700	76	0	0	0	0	1,060	7	10,640	69
City of Cudahy	13,900	98	3,140	22	0	0	650	5	10,760	8
City of St. Francis	6,480	67	940	10	0	0	0	0	4,720	49
City of Milwaukee	7,550	14	0	0	0	0	0	0	0	0
Village of Shorewood	5,490	82	830	13	590	9	2,170	33	970	15
Village of Whitefish Bay	12,340	84	1,950	13	0	0	3,010	21	3,770	26
Village of Fox Point	12,270	84	1,900	13	0	0	770	5	1,590	11
Village of Bayside	8,310	91	0	0	0	0	0	0	1,320	14
Total Study Area	88,450	56	8,760	6	3,410	2	10,480	7	44,180	28

Source: SEWRPC.

study area shoreline. Groundwater drainage systems were indicated for eight bluff analysis sections, covering about 10,970 feet, or 7 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 t the groundwater would not be feasible, it is likely that regarding of at least a portion of the bluff slope would be necessary. Control of surface water run-off was indicated for four bluff analysis sections,which cover about 4,360 feet, or 3 percent of the shoreline. Revegetation of at least a portion of the bluff face was indicated for 13 bluff analysis sections, covering about 10,740 feet, or 7 percent of the shoreline. Protection of the toe of the bluff or shoreline against wave and ice action was indicated for 71 bluff analysis sections, which have a combined shoreline of about 107,380 feet, or about 67 percen of the total study area 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. It was assumed that only those bluff slopes identified as marginally unstable or unstable--as well as the Fox Point terrace--may be expected to recede. 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 1980 by 25 years and 50 years. The 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 III-11 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 unstable bluff analysis sections and directly adjacent to the shoreline. As shown in Table III-11, although from 15 to 51 percent of the Cities of South Milwaukee, Cudahy, and St. Francis, and the Villages of Shorewood, Whitefish Bay, and Fox Point lie within the study area, only from 6 to 11 percent of the land within those municipalities lies directly adjacent to Lake Michigan, and only from 1 to 7 percent of the land within those communities is potentially subject to shoreline erosion. About 4 and 6 percent of the City of Milwaukee and the City of Oak Creek, respectively, lies within the study area, less than 0.1 percent of these municipalities lie directly adjacent to Lake Michigan and less than 0.1 percent is potentially subject to shoreline erosion. Of the total land directly adjacent to Lake Michigan and potentially subject to shoreline erosion, approximately 257 acres, or 75 percent, is privately owned, while the remaining 85 acres, or 25 percent,; is publicly owned. This narrow strip of land, however, is an extremely valuable resource, providing a unique setting for high value residential development and recreational opportunities. These shoreline area also attract users from well inland. It is therefore important to delineate those shoreland areas subject to damages caused by shoreline erosion and bluff recession to help define the need for shore protection measures which would provide a desired and usable shoreline for the property owners as well as for other area citizens.

The property potentially at hazard was delineated for the bluff analysis sections which were determined to have marginally unstable or unstable bluff slopes by the slope stability analyses. Approximately 59,430 feet, or 37 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

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Table III-11

AREAL EXTENT OF STUDY AREA; AREA DIRECTLY ADJACENT TO TO THE 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	--	3,615	--	1,889	--

[a]The area potentially subject to shoreline erosion is defined as that land lying within an unstable bluff analysis section--including the Fox Point Terrace--and directly adjacent to the shoreline.

Source: SEWRPC.

25-year recession distance from the edge would have the greatest risk of erosion damage.

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.[7] Another report by Vallejo and Edil[8] 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. 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 34 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

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[7]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.

[8]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.

3. Groundwater conditions can change significantly as the bluff recedes and strata of permeable bluff materials are eroded, covered, or disturbed.

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.

As set forth in Table III-12, approximately 60.9 acres of land lie within the 25-year bluff recession distance of existing marginal or unstable bluff or terrace. Of this total area, about 27.6 acres, or 45 percent, are located within the City of Oak Creek; about 13.0 acres, or 21 percent, are located with the City of Cudahy; about 8.7 acres, or 14 percent, are located within the City of South Milwaukee; and the remaining 11.6 acres, or 20 percent, are located within the City of St. Francis, and the Villages of Fox Point, Whitefish Bay, Shorewood, and Bayside. There were no unstable bluff analysis sections located within the City of Milwaukee. Privately owned land comprises a total of about 23.1 acres, or 38 percent of this land. Publicly owned land comprises 37.8 acres, or 62 percent.

Approximately 124.4 acres of land lie within the 50-year bluff recession distance of existing marginal or unstable bluff or terrace. Of this total area, about 55 acres, or 44 percent, are located within the City of Oak Creek; about 26 acres, or 21 percent, are located within the City of Cudahy; about 17 acres, or 14 percent, are located within the City of South Milwaukee; and the remaining 26.4 acres, or 21 percent, are located within the City of St. Francis, and the Villages of Fox Point, Whitefish Bay, Shorewood, and Bayside. Privately owned land comprises a total of 48.5 acres, or 39 percent of this land, while publicly owned land comprises 75.9 acres, or 61 percent.

SUMMARY

This chapter evaluates the shoreline erosion and bluff recession occurring within the study area, identifies those specific factors causing the erosion and attendant bluff recession, recommends types of control measures needed to

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Table III-12

SUMMARY OF POTENTIAL EROSION DAMAGES WITHIN THE MILWAUKEE COUNTY SHORELINE

Civil Division	Section	Shoreline Length (feet)	25-Year Recession Distance (feet)	25-Year Total Area (feet^2)	50-Year Recession Distance (feet)	50-Year Total Area (feet^2)	Average Bluff Recession (ft/yr)
City of Oak Creek	2	2,820	60	169,200	120	338,400	2.4
	3	2,930	88	257,840	175	512,750	3.5
	4	1,980	88	174,240	175	346,500	3.5
	5	1,070	160	171,200	320	342,400	6.4
	6	1,170	292	341,640	585	684,450	11.7
	7	1,000	48	48,000	95	95,000	1.9
	9	570	12	6,840	25	14,250	0.5
	10	400	20	8,000	40	16,000	0.8
	11	1,290	18	23,220	35	45,150	0.7
Subtotal	--	13,230	--	1,200,180	--	2,394,900	--
City of South Milwaukee	14	1,310	42	55,020	85	111,350	1.7
	15	790	38	30,020	75	59,250	1.5
	17	440	32	14,080	65	28,600	1.3
	18	1,880	28	52,640	55	103,400	1.1
	20	1,280	12	15,360	25	32,000	0.5
	21	1,060	18	19,080	35	37,100	0.7
	22	950	12	11,400	25	23,750	0.5
	23	1,200	20	24,000	40	48,000	0.8
	24	1,910	42	80,220	85	162,350	1.7
	25	880	88	77,440	175	154,000	3.5
Subtotal	--	11,700	--	379,260	--	759,800	--
City of Cudahy	26	660	28	18,480	55	36,300	1.1
	27	1,850	28	51,800	55	101,750	1.1
	28	2,050	22	45,100	45	92,250	0.9
	29	770	55	42,350	110	84,700	2.2
	30	1,760	100	176,000	200	352,000	4.0
	31	600	52	31,200	105	63,000	2.1
	33	2,060	62	127,720	125	257,500	2.5
	34	1,780	22	39,160	45	80,100	0.9
	35	650	18	11,700	35	22,750	0.7
	36	710	12	8,520	25	17,750	0.5
	37	1,010	12	12,120	25	25,250	0.5
Subtotal	--	13,900	--	564,150	--	1,133,350	--
City of St. Francis	38	1,290	72	92,880	145	187,050	2.9
	39	1,480	70	103,600	140	207,200	2.8
	43	1,370	30	41,100	60	82,200	1.2
	44	140	12	1,680	25	3,500	0.5
	45	80	12	960	25	2,000	0.5
	46	360	12	4,320	25	9,000	0.5
Subtotal	--	4,720	--	244,540	--	490,950	--
Village of Shorewood	63	300	12	3,600	25	7,500	0.5
	67	380	12	4,560	25	9,500	0.5
	68	2,650	12	19,800	25	41,250	0.5
	70	240	12	2,880	25	6,000	0.5
Subtotal	--	2,570	--	30,840	--	64,250	--

(continued)

(Table III-12 continued)

Civil Division	Section	Shoreline Length (feet)	25-Year Recession Distance (feet)	25-Year Total Area (feet^2)	50-Year Recession Distance (feet)	50-Year Total Area (feet^2)	Average Bluff Recession (ft/yr)
Village of Whitefish Bay	72	850	12	10,200	25	21,250	0.5
	73	190	12	2,280	25	4,750	0.5
	74	160	12	1,920	25	4,000	0.5
	75	310	12	3,720	25	7,750	0.5
	76	360	12	4,320	25	9,000	0.5
	78	1,060	12	12,720	25	26,500	0.5
	80	130	12	1,560	25	3,250	0.5
	82	490	12	5,880	25	12,250	0.5
	84	430	12	5,160	25	10,750	0.5
	86	170	12	2,040	25	4,250	0.5
	87	1,950	12	23,400	25	48,750	0.5
	88	540	12	6,480	25	13,500	0.5
Subtotal	--	6,640	--	79,680	--	166,000	0.5
Village of Fox Point	88	610	12	7320	25	15,250	0.5
	90	470	12	5,640	25	11,750	0.5
	91	510	12	6,120	25	12,750	0.5
	92	770	12	9,240	25	19,250	0.5
	95	9,070	12	108,840	25	226,750	0.5
Subtotal	--	11,430	--	137,160	--	285,750	--
Village of Bayside	100	1,320	22	29,040	45	59,400	0.5
Total Study Area	--	65,510	--	2,664,850	--	5,354,400	--

Source: SEWRPC.

abate shoreline erosion and bluff recession, and summarizes the potential property and economic losses which may result if shoreline protection is not implemented. The identification of the shoreland areas which may be expected to continue to be affected by shoreline erosion and bluff recession enables public officials and private property owners to better assess potential erosion losses and evaluate alternative erosion management measures.

Analytic procedures and geotechnical engineering techniques were used to evaluate the existing and potential future 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, 31 percent of the total study area shoreline was determined to have stable bluff slopes, 10 percent of the shoreline was determined to have marginal bluff slopes, and 27 percent of the shoreline was determined 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 Linnwood water treatment plant, and the Fox Point terrace.

With respect to translational sliding, 36 percent of the total study area shoreline was determined to have stable slopes, 7 percent of the shoreline was determined to have marginal bluff slopes, and 25 percent of the shoreline was determined 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 evidence of toe erosion in the field surveys conducted in 1986 and 1987. About 26 percent of the shoreline was experiencing erosion at the toe of the bluff, but the erosion did not appear to affect the overall stability of the bluff slope. The remaining 25 percent of the shoreline was experiencing toe erosion which was threatening the overall stability of the bluff slope.

The shore protection needs for each of the bluff analysis sections within the study area were identified. It was indicated that the bluff slopes within

about 29 percent of the study area shoreline be regraded to a stable slope angle; that groundwater drainage systems be installed to lower the elevation of the groundwater along about 7 percent of the shoreline; that surface water runoff control measures be implemented along about 3 percent of the shoreline; that additional toe protection be provided to about 67 percent of the shoreline; and that revegetation of the bluff slope be provided for about 7 percent of the shoreline.

The land area lying within 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 includes about 60.9 acres of land. About 37.8 acres, or 62 percent of the land within the 25-year bluff recession distance, were publicly owned, while the remaining 23.1 acres, or 38 percent, was in private ownership. About 124.4 acres of land lie within the 50-year bluff recession distance of the marginal or unstable bluffs and terraces. About 75.9 acres, or 61 percent of the land within the 50-year bluff recession distance were publicly owned, while the remaining 48.5 acres, or 39 percent were privately owned. 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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SEWRPC Community Assistance Planning Report No. 163

A LAKE MICHIGAN SHORELINE EROSION, BLUFF RECESSION, AND STORM DAMAGE CONTROL PLAN FOR MILWAUKEE COUNTY

Chapter IV

ALTERNATIVE SHORELINE EROSION CONTROL MEASURES AND A RECOMMENDED SHORELINE EROSION, BLUFF RECESSION, AND STORM DAMAGE CONTROL 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 alternative 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, 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 study Advisory Committee.

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 which potentially could be utilized within the study area. The third section describes the financing and implementation options available to successfully carry out the plan recommendations. The fourth section describes alternative shore protection plans. The fifth section describes the

recommended shoreline erosion, bluff recession, and storm damage control plan for Milwaukee County; 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 areawide, covering the entire reach of shoreline concerned. The preliminary engineering and the final design phases combined are more site-specific, focusing on selected subreaches 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 then 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 often 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 often 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 maintenance and exhibiting some type of failure. Proposals for new shore protection measures have generally not included an analysis of potential adverse impacts on adjacent shoreline areas. In many cases, shore protection measures are designed and constructed without a thorough understanding of the coastal processes and hydro-geologic 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 in not only 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 the 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 IV-1. These procedures and criteria provide the means for quantitatively 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. Due to the variability of coastal and hydro-geologic conditions along the shoreline, step-by-step instructions to properly analyze or design a shore protection project cannot be specified. Table IV-1 lists those issues which should be

Table IV-1

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 at a 10-year recurrence interval, from the north, northeast, east, and southeast. Take shoaling effects into account, as necessary. Deep water wave heights are estimated in this report. Calculate wave heights in the surf zone using Goda's method (Y.Goda, "Irregular Wave Deformation in the Surf Zone," Coastal Engineering in Japan, Vol. 18, pp 13-26, 1975), the TMA method (Steven A. Hughes, The TMA Shallow Water Spectrum-Description and Applications, U.S. Army Corps of Engineers, CERC Technical Report 84.7, December 1984), or a comparable method. 3. Evaluate the impacts on adjacent shoreline areas which may be caused by wave reflection or interruption of the littoral drift. Estimate the amount of potential 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 competence of the lakebed materials to a suitable depth to provide an adequate foundation to support the proposed structure. 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. 5. Identify available access sites for construction and maintenance activities, and the cost and availability of suitable construction materials.	1. Structures should be designed to prevent severe damage and operate well under the 100-year recurrence interval maximum instantaneous 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 minimum instantaneous water level of 574.9 feet NGVD (573.6 feet IGLD) to the maximum instantaneous level. 2. Structures should be designed to prevent severe damage and operate well, at the design lake level, under the 20-year recurrence interval wave height. Consideration should be given to using a 50-year recurrence interval design wave for structures which protect major public facilities where storm damage would have catastrophic impacts. Most storms produce wave spectra which are depth-limited in water depths shallower than about 15 feet. 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 are not recommended and groins and other beach-containing structures should be utilized only when artificially nourished with beach material. 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.
Bluff Slope Instability	Regrading of Bluff Slope Utilizing Cutback, Filling, and/or Terracing	1. Conduct a detailed slope stability analysis of the existing bluff slope. Conduct additional stability analyses where the bluff profile, stratigraphy, or groundwater condition vary substantially. Survey the bluff geometry and groundwater conditions and conduct at least three soil borings to identify the stratigraphy, unless suitable borings were previously conducted. Install at least one groundwater observation well, or piezometer, unless a suitable well was previously installed. Conduct soil tests as necessary.	1. Where cutting back at least a portion of the bluff slope is indicated, the bluff may be allowed to achieve its equilibrium slope naturally, unless such natural uncontrolled slope evolution could damage a building or shore protection structure or pose a safety risk to pedestrians. 2. Where sufficient land exists at the top of the bluff to maintain a 50-foot buffer for existing residential buildings, the bluff edge should be cut back to provide a maximum slope angle of 22°, or one on two and one-half, unless a detailed slope stability analysis indicates that

Shoreline Problem	Potentially Applicable Shore Protection Measures	Site Specific Inventories and Analyses	Design Criteria
Bluff Slope Instability (cont'd)		2. Conduct a slope stability analysis of the bluff slope anticipated to exist at the completion of regrading, or the construction of terraces.	a steeper slope angle would be stable. Filling only the lower portion of the slope, cutting back the top of the 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 residential buildings 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 stabilize adequately the bluff slope or to provide a configuration aligned with the adjacent shoreline. 4. Where fine-grained natural bluff 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 upwards. 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.

Shoreline Problem	Potentially Applicable Shore Protection Measures	Site Specific Inventories and Analyses	Design Criteria
Groundwater Seepage From Face of Bluff Which Threatens the 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 boreholes, 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. 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.	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 resistant 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. 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 reducting 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, may 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 velocitis 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.

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	Revegetation of the Bluff Slope	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 with respect to successful revegetation. Identify specific needs for revegetation relating to control of surface water and groundwater, slope shaping, and soil management. 3. Survey the existing vegetation to identify what effective vegetation currently exists on the slope. 4. Identify aesthetic and functional preferences.	1. Where bluff 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. Some shaping and terracing of the slope may be needed to provide a suitable slope angle and eliminate drainage problems. Groundwater and surface water drainage systems should be installed, as needed, prior to planting. 3. Initial grass or pioneer species should be used to establish a good ground cover first, then trees and shrubs should be planted at 3- to 6-foot spacings. Plantings should be conducted in spring or fall. 4. Maintenance-free deep-rooting plant species which are suitable for the physical site conditions should be selected. 5. Mulch should be applied after seeding. Drilling or hydroseeding may be necessary to successfully establish herbaceous plants on steep slopes. 6. Watering and fertilization after planting should be limited to the minimum needed for successful establishment of the vegetation. 7. All revegetation projects should have provisions for follow-up inspection, care, and maintenance.

Source: SEWRPC

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, some 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 wave--deserve further consideration. It is recommended that shore protection structures be designed to prevent severe damage at the 100-year recurrence interval maximum instantaneous lake level of 584.3 feet NGVD. However, structures should be designed to perform well--and provide a suitable shoreline--under a range of stillwater conditions, as opposed to one design level. Thus, the design of structures should consider performance under lake levels ranging from the low water datum to the maximum instantaneous level.

At the design lake level described above, it is recommended that 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 facilities where shoreline damages could have catastrophic effects, it is recommended that consideration be given to designing structures to prevent severe damage by the 50year recurrence interval wave height, which is about 24.8 feet in deep water.

CONCEPTUAL SHORE PROTECTION MEASURES

The analysis of the need for, and the selection of, potential shore protection measures should first include identification of 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. The selection of measures needed to control shoreline erosion and stabilize bluff slopes set forth in Chapter III are based on systems level analyses. This section sets forth a description of the alternative shore protection measures which should be considered for installation within the study area.

Complete protection of the shoreline will require a combination of bluff toe protection, bluff slope regrading and revegetation, and surface water and groundwater drainage control. The alternative structural shore protection measures presented in this chapter were developed and evaluated based on the inventory data collected and collated, and the analyses performed under this study. A description of alternative structural measures, along with conceptual designs and estimated costs, are presented for each protection measure for use in the system level planning effort. 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.

Bluff Toe Protection

Shoreline areas exhibiting bluff toe erosion were identified in Chapter III of this report and include approximately 81,000 feet, or 51 percent of the County shoreline. Alternative bluff toe protection measures evaluated for the Milwaukee County study area include both onshore and offshore structures. Onshore structures include revetments, bulkheads, and groins; and offshore structures include breakwaters, barrier reefs, and islands. A general comparison of selected characteristics of bluff toe protection measures is provided in Table IV-2. The table presents certain requirements for successful application of the structures, lists the advantages and disadvantages of each type of

Table IV-2

COMPARISON OF BLUFF TOE PROTECTION MEASURES

Bluff Toe Protection Measure	Type	Advantages	Disadvantages	Compatibility with Alternative Shoreline Uses					Capital Cost $/lineal foot of shoreline	Annual Maintenance Cost $/lineal foot of shoreline
				Walking	Swimming	Fishing	Boating	Aesthetics		
Revetment	Riprap	Easy to construct and maintain Flexible, durable	Limits access to shoreline Heavy equipment required for installation Reflects wave energy	Fair	Poor	Poor	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	15-20
	Interlocking Concrete Blocks (Flex-Slabs)	Provides uniform appearance and useable shoreline Adaptable to add-on construction	Relatively inflexible Not as durable as quarry stone Heavy equipment required for installation	Good	Fair	Good	Good	Fair	150-450	15-20
	Erosion Control Units	Massive units interlock and provide good anchorage	Limits access to shoreline Heavy equipment required for installation	Fair	Poor	Poor	Poor	Poor	100-200	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 useable 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 Nearshore Beach Systems	Short Groins with Nourished Sand or Gravel Beach	Provides useable shoreline Flexible Absorbs wave energy Feeds littoral transport system	The beach would need to be periodically renourished Trapping sand supply may reduce the available sand for down-current beach areas	Good	Good	Good	Good	Good	400-900	20-60

Table IV-2 (cont'd)

Bluff Toe Protection Measure	Type	Advantages	Disadvantages	Compatibility with Alternative Shoreline Uses					Capital Cost $/lineal foot of shoreline	Annual Maintenance Cost $/lineal foot of shoreline
				Walking	Swimming	Fishing	Boating	Aesthetics		
Beach Systems (cont'd)	Armored Headland-Pocket Beach System	Flexible, durable Provides useable shoreline Pocket beaches absorb wave energy Feeds littoral transport system if beaches composed of sand or fine gravel	May require large amount of fill to construct Beach would need to be periodically renourished to maintain sand or fine gravel Trapping sand supply may reduce the available sand for down-current beach areas Armored headlands reflect wave energy	Good	Good	Good	Good	Good	600-1,200	20-60
	Nearshore Reefs with Nourished Sand or Gravel Beaches	Flexible Provides uniform appearance and continuous useable shoreline Feeds littoral transport system	The beach would need to be periodically renourished Trapping sand supply may reduce the available sand for down-current beach areas Reefs subject to large wave attack and thus more susceptible to damage than onshore structures	Good	Good	Good	Good	Good	450-1,200	25-70
	Cobble Beach	Cobbles absorb considerable wave energy without causing scouring from wave reflection	Limits use of shoreline Not as durable as conventional riprap revetments	Poor	Poor	Poor	Poor	Fair	450-	30
	Manufactured Concrete Systems (Nellco Beach Building Block) Nourished with Sand or Gravel	Provides partially useable shoreline	Limits access to water Blocks may settle or move out of alignment	Good	Fair	Good	Fair	Fair	250	25-60
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 may reduce the available sand for down-current beaches	Good	Good	Good	Good	Good	1,000-2,000	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	800-1,200	20

Source: SEWRPC

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.

The following sections describe the most common structural toe protection measures presently used in the Great Lakes and provides guidelines for the application of these solutions. The guidelines and general design criteria described relate only to the preliminary design and sizing of bluff toe protection structures; detailed design criteria for structures are set forth in scientific reports such as the U. S. Army Corps of Engineers Shore Protection Manual (1984).

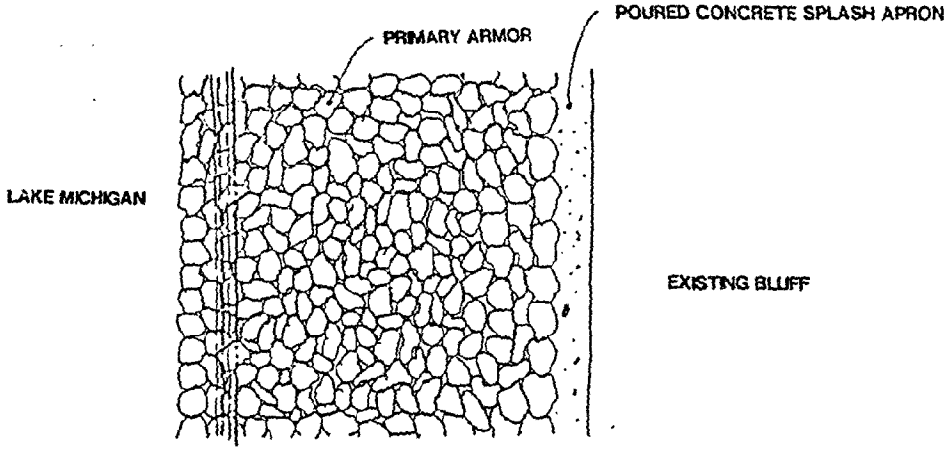
Revetment: Various types of revetments are commonly used to provide toe protection within Milwaukee County. Revetments contain a flattened slope at the bluff toe armored with material resistant to wave erosion and ice damage, and usually underlaid by filter cloth and gravel or cobble bedstone. The armor 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 rip-rap revetment, a grout-filled bag revetment, and an interlocking concrete block revetment.

Riprap--As shown in Figure IV-1, a riprap revetment utilizes rock or quarry stone as its armor layer. The armor stone should be free of laminations, cracks, and undesirable weathering. 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. Armor stone used in Milwaukee County typically ranges from about one to four tons in weight.

Figure IV-1

TYPICAL QUARRY STONE REVETMENT



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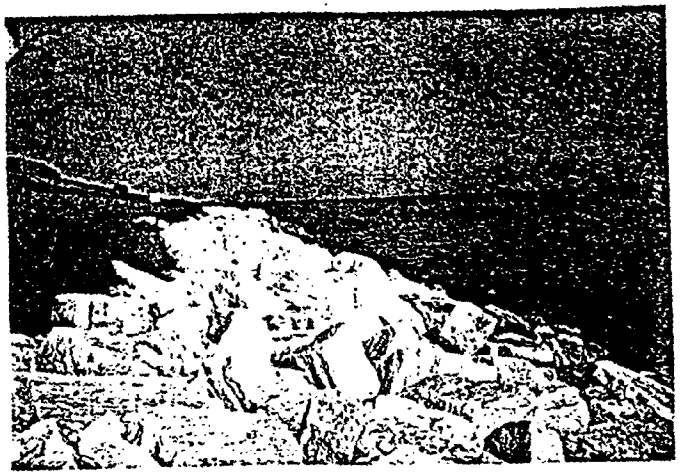
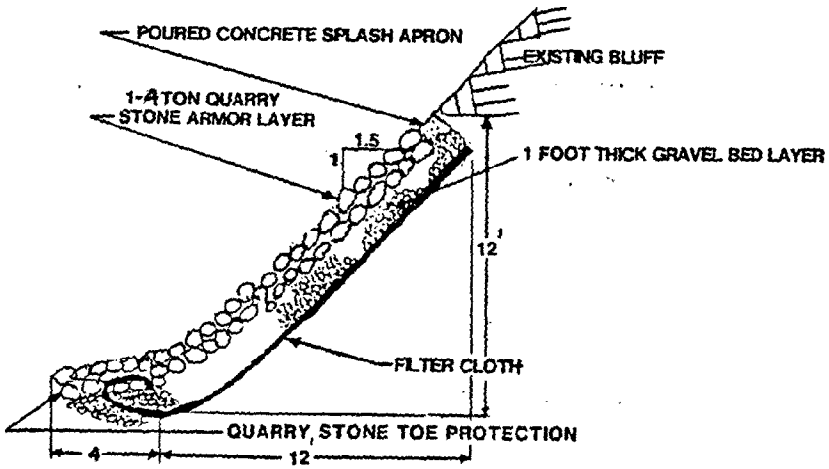




PROFILE

Source: Warzyn Engineering Inc. and SEWRPC

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.

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. The construction of a riprap revetment requires heavy equipment.

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 gives the shoreline a natural appearance. Because of its porosity, the wave runup on a revetment is less than on a bulkhead or beach.

The primary disadvantages of a riprap revetment are that the structure generally makes use of the immediate shoreline area for most 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 wave energy may erode offshore lakebed material and, over time, create a steeper offshore slope which would allow larger waves to reach the shoreline.

The life of a riprap revetment depends on the degree of maintenance performed. Riprap revetments are subject to displacement. The armor stones may be moved by wave action, which may weaken the entire structure if not maintained.

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 to $20 per lineal foot.

Grout-Filled Bags--Large grout-filled nylon bags have been placed at the toe of bluffs to form revetments within the County. The bags which have previously been most commonly used in the County are six feet deep by two and one-

half feet high, and up to 20 feet long. The bags weigh about 14 tons each. As shown in Figure IV-2, 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.

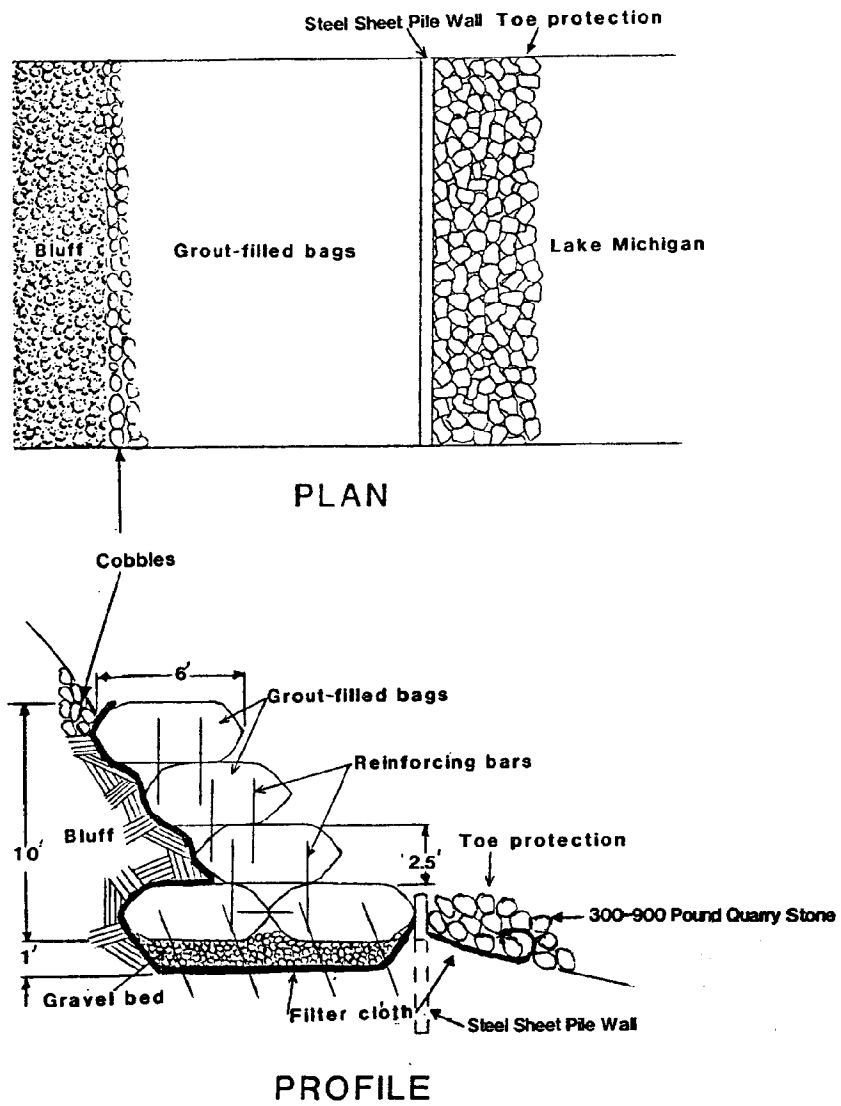
The primary advantage of a grout-filled bag revetment is that it can be constructed where access is limited. A grout pump which can be operated from the top of a bluff is used to fill the bags. In addition, the structure is readily adaptable to add-on construction if additional structure height is necessary. 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 subject to catastrophic failure than would 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 stone in some applications.

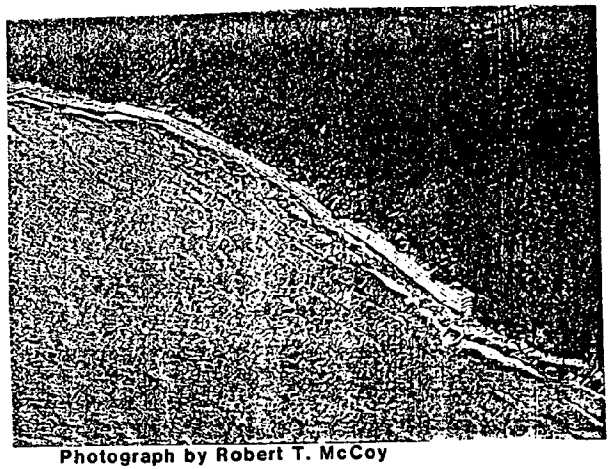
The capital cost of a grout-filled bag revetment is influenced by design water level and depth, wave environment, material cost, and other site-specific factors but, in general, range from $200 to $250 per lineal foot of shoreline. Average annual maintenance costs may range from $15 up to $29 per lineal foot.

Concrete Structures--Several different types of manufactured concrete structures are commercially available. Flex-Slabs, manufactured by Spancrete Industries, Inc., are an example of an interlocking concrete block system in which the slabs fit together to form a revetment. The Flex-Slabs, patented in 1984, each cover 11 square feet, are 12 inches thick, and weigh approximately 1,000 pounds. As shown in Figure IV-3, the slabs interlock with a hook and slot connection. The slabs are perforated with four small and two large slots to neutralize pressure from changing water levels and to absorb energy from wave action. A filter cloth should be placed beneath the slabs 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.

Figure IV-2
TYPICAL GROUT-FILLED BAG REVETMENT



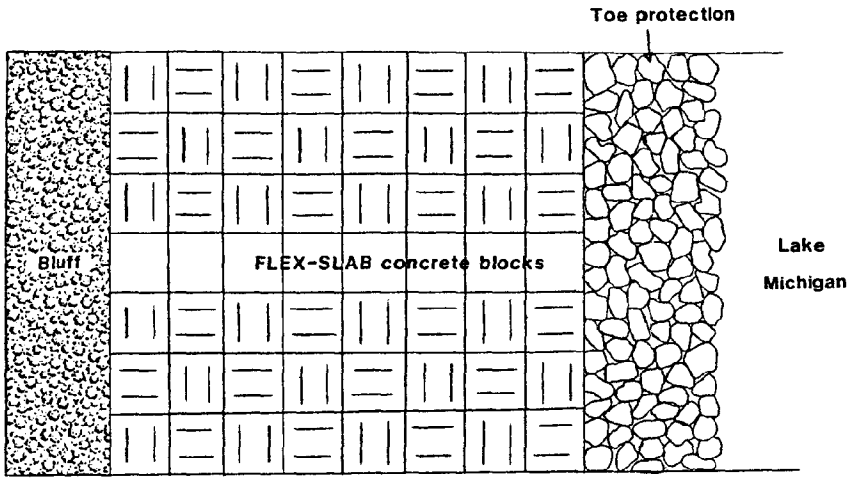
Source: United States Army Corps of Engineers and SEWRPC



Photograph by Robert T. McCoy

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.

Figure IV-3
TYPICAL INTERLOCKING CONCRETE BLOCK
(FLEX-SLAB) REVETMENT



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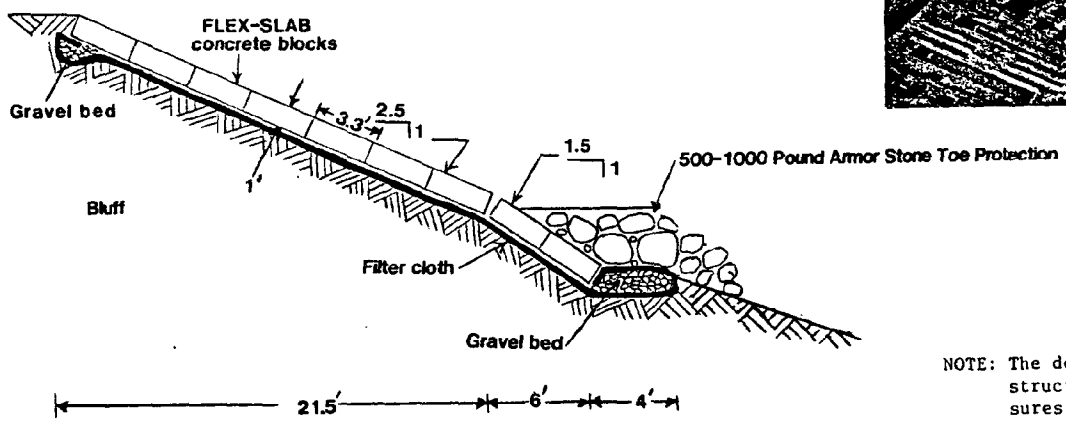
Photograph by Spancete, Inc



PROFILE

Source: Spancrete, Inc. and SEWRPC.

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.

The advantages of the Flex-Slab system are that it provides a uniform appearance and a useable shoreline which may be suitable for some recreational activities. In addition, the Flex-Slab is readily adaptable to add-on construction.

A major disadvantage of interlocking concrete blocks in general is that the failure of one block could 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 capital cost of a Flex-Slab system, depending on site characteristics, approximates $150 to $450 per lineal foot of shoreline. The average annual operation and maintenance cost would approximate $15 to $20 per lineal foot.

Erosion Control Units, manufactured by Libecki Trucking and Grading of Sussex, Wisconsin, are six-sided concrete units which can be placed perpendicular to the shoreline to create a revetment. The units, as shown in Figure IV-4, vary in size, ranging from six to seven feet high and weighing between two to three tons each. Three of the six sides of each unit are sunk into the lake bed and, combined with quarry stone, or concrete rubble, create a revetment. Heavy construction equipment is required to install the structures.

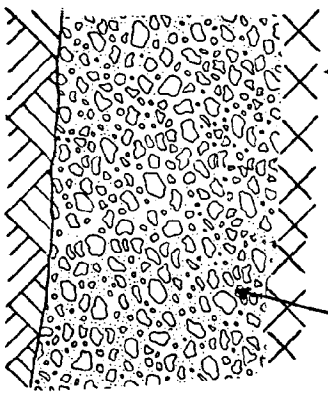
The capital cost of a revetment constructed of Erosion Control Units ranges from about $100 to $200 per lineal foot of shoreline. The average annual operation and maintenance cost would range from $10 to $20 per lineal foot.

Bulkhead: Bulkheads are vertical retaining walls constructed of concrete, steel sheet piling, or timber which supports the base of the bluff and provides protection against wave and ice action. Historically, bulkheads have been one of the most commonly used shore protection structures in Milwaukee County, with most being constructed of concrete.

One advantage of a bulkhead is that the structure can be constructed to a height of 10 to 15 feet above the existing beach and can be placed lakeward of 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

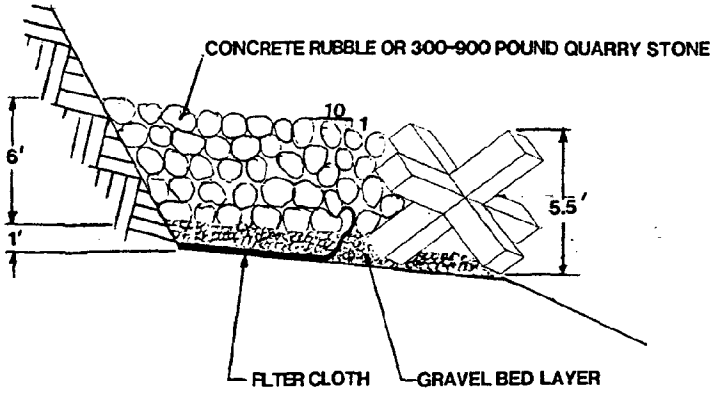
FIGURE IV-4

TYPICAL EROSION CONTROL UNIT REVETMENT

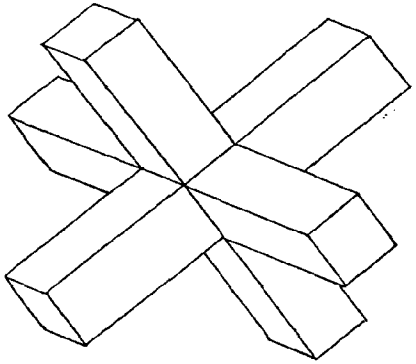


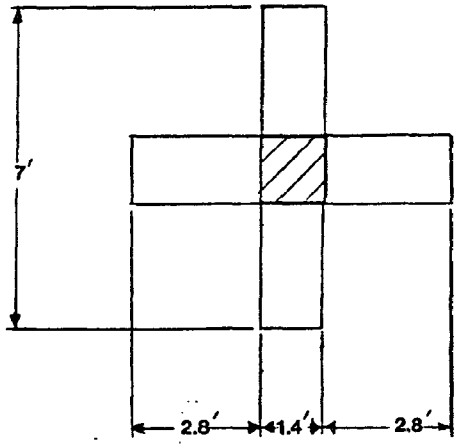
PLAN

Photograph by Dan Libecki Grading



PROFILE





EROSION CONTROL UNIT

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: Dan Libecki Grading and SEWRPC

distance to achieve a stable bluff slope, as shown in Figure IV-5. Thus, the necessary cutting back of the top of the bluff to form a 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.

Disadvantages of a bulkhead are that the structure is inflexible, and maintenance, when required, is difficult and costly. Bulkheads are less suitable during periods of widely fluctuating water levels than are most other 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.

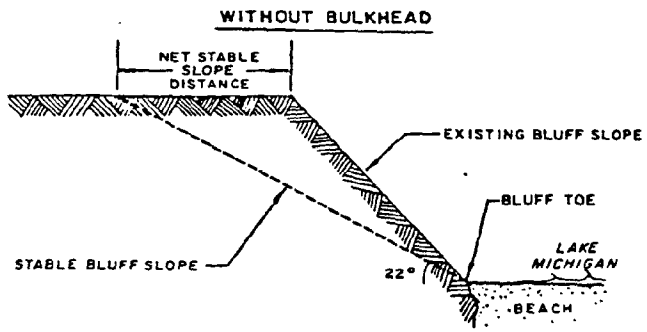
Concrete Cantilevered Bulkhead--A cantilevered, cast-in-place, reinforced concrete bulkhead, as illustrated in Figure IV-6, consists of a concrete base with a cantilevered wall. The wall is constructed with weep holes and backfilled with coarse granular material to prevent hydrostatic pressure buildup and frost heave. Riprap toe protection should be provided. A cantilevered bulkhead derives its support solely from ground penetration, so sufficient embedment is required.

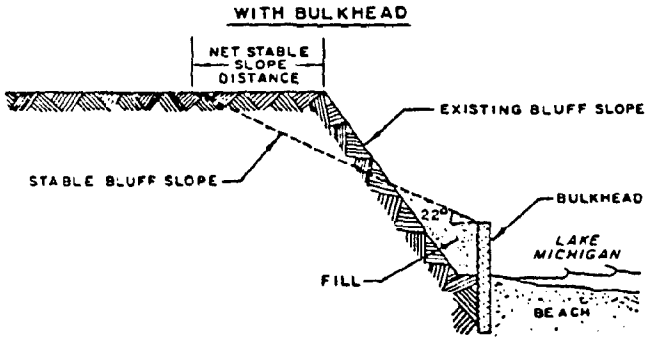
Construction of a concrete cantilevered bulkhead along the Lake Michigan shoreline of Milwaukee County would entail a capital cost of approximately $400 per lineal foot of shoreline. Average annual maintenance costs would range from about $10 to $15 per lineal foot.

Steel Sheet Piling Bulkhead--A steel sheet piling bulkhead, as shown in Figure IV-7, 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 also be anchored with tie backs, as also shown in Figure IV-7. Riprap toe protection and weep holes for drainage should be

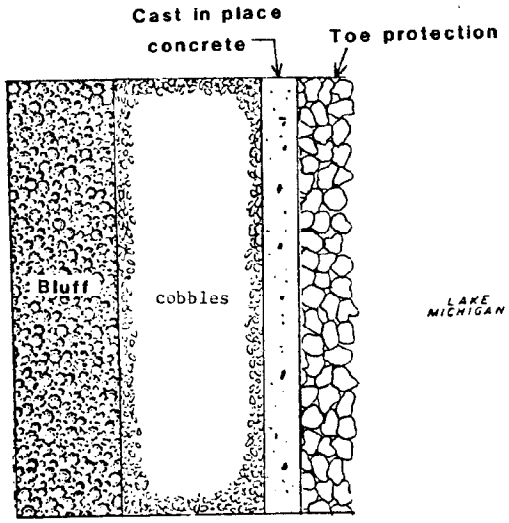
Figure IV-5

EFFECT OF A BULKHEAD ON THE
BLUFF TOP CUTBACK DISTANCE REQUIRED
TO ACHIEVE A STABLE BLUFF SLOPE





Source: SEWRPC



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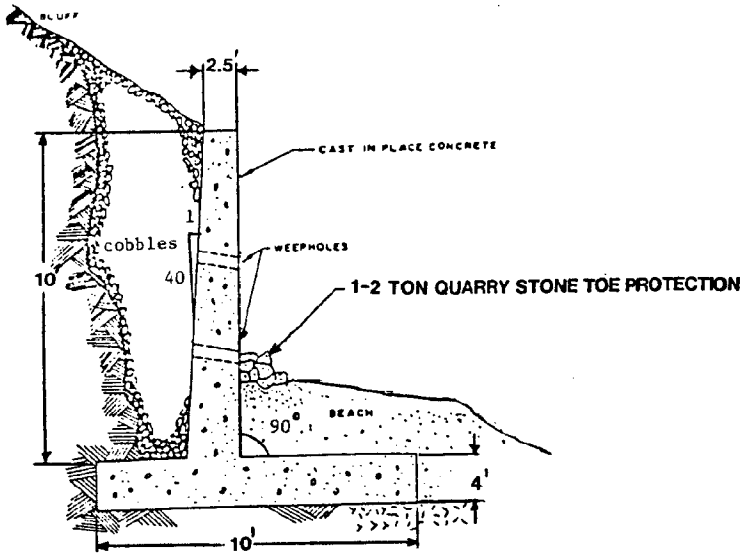
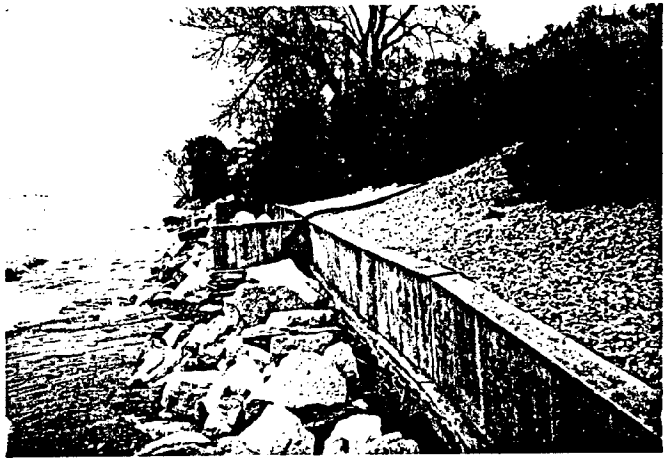


PROFILE

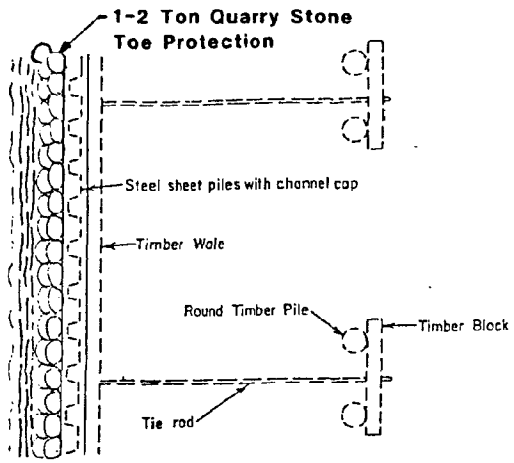
Source: Owen Ayres & Associates, Great Lakes Shore Erosion Protection, Structural Design Examples, 1978. and SEWRPC

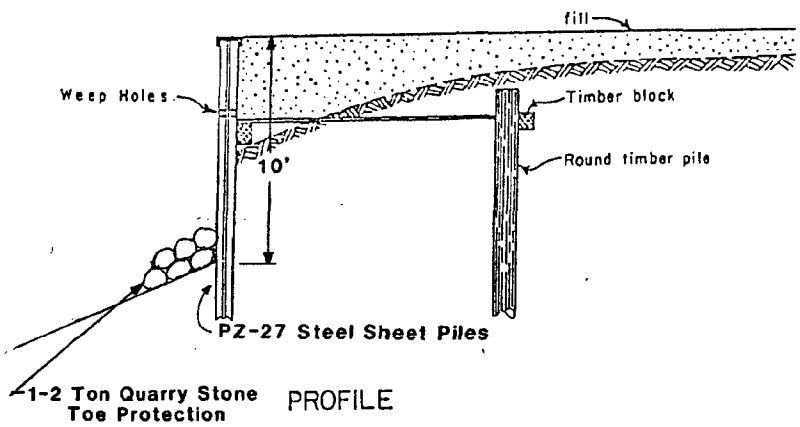
Figure IV-6
Typical Concrete Cantilevered Bulkhead



Photograph by SEWRPC

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.

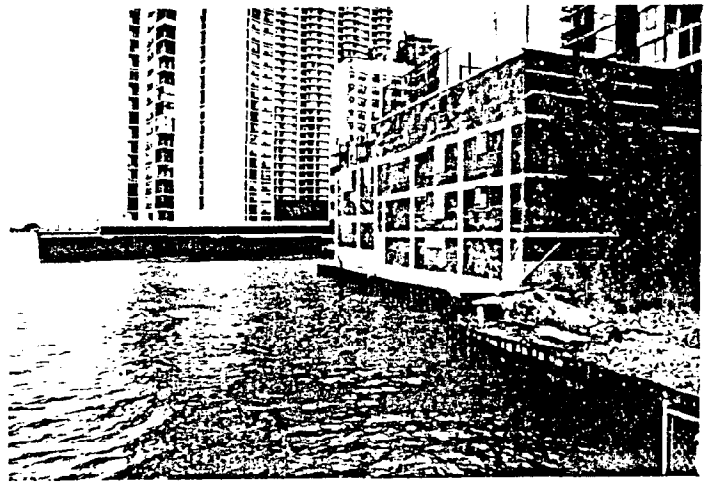




Source: Warzyn Engineering Inc. and SEWRPC

Figure IV-7

TYPICAL STEEL SHEET PILING BULKHEAD



Photograph By SEWRPC

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.

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 about $5 to $10 per lineal foot.

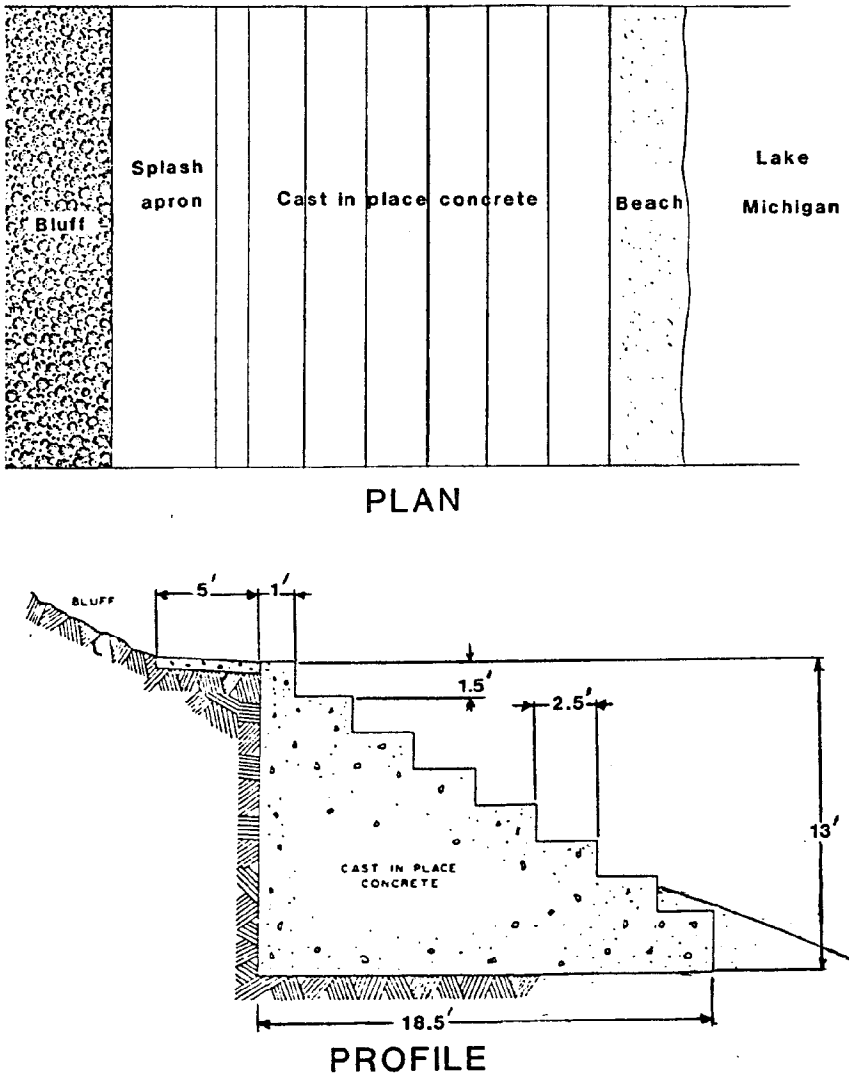
Concrete-Stepped Bulkhead--A third alternative bulkhead design involves construction of a cast-in-place, concrete-stepped bulkhead, as shown in Figure IV-8. The bulkhead, cast as a massive, gravity-held structure to resist overturning by wave action or soil pressures, should include a splash apron along the crest of the bulkhead 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 about $5 to $10 per lineal foot.

Onshore or Nearshore Beach Systems: There are several onshore or nearshore protection structures which may support a beach which may 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 which are built out from the shoreline, or are 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, it is often necessary to artificially nourish the beaches with coarse-grained material, usually coarse sand or gravel. The beaches need to be occasionally re-nourished. Generally, the coarser the beach material, the steeper the beach that would form. Table VI-3 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

Figure IV-8

TYPICAL CONCRETE STEPPED BULKHEAD



Photograph by United States Army Corps of Engineers

Source: Owen Ayres & Associates, Great Lakes Shore Erosion Protection, Structural Design Examples, 1978. and SEWRPC

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.

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Table IV-3

ESTIMATED BEACH SLOPES WHICH 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:

$$m = \tan^{-1}(1.8\ H/D)^{-0.5}$$

where, m = beach slope (degrees)
H = breaking wave height (m)
D = particle size (m)

Source: SEWRPC

of less than 5 degrees, gravel beaches may frequently have slopes approximating 10 degrees.

The major advantage of an onshore beach system is that an extended beach would be provided to protect the bluff toe against wave action and to allow access to the lake 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; the considerable maintenance which may be required to keep the extended beach intact; and insufficient bluff toe protection may be provided by the beach during large storm events, especially during high lake levels.

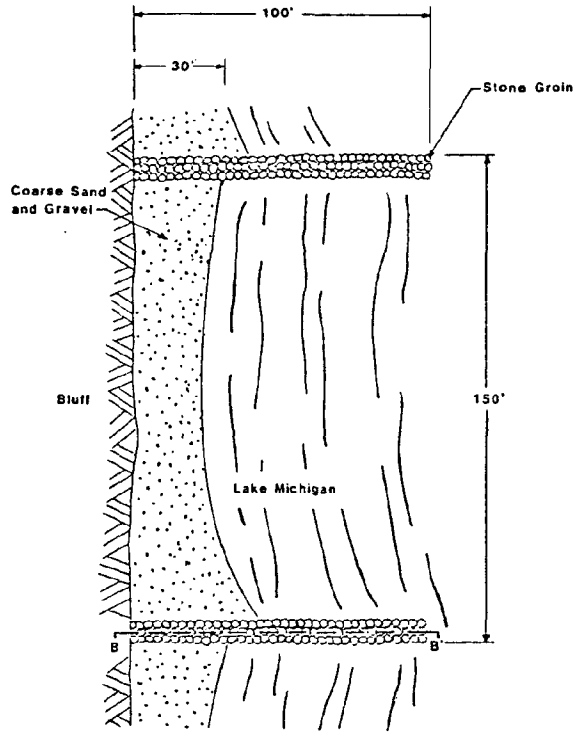
Described below are several types of onshore or nearshore beach system designs: groins, an armored headland-pocket beach system, nearshore reefs, and manufactured concrete systems.

Groins--Groins are the most common type of structure used to create beaches. Groins can be constructed of rock, 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 obstruct the littoral drift, thereby trapping sand 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 appears to be quite limited, it is unlikely that new groin systems would trap enough material to form a substantial beach. Rather, the groins would be designed to hold an artificially nourished beach composed of coarse sand and gravel. Groins do not appreciably reduce the wave energy striking the shore, and sediment moving along shore is forced into deeper water to move around the structure ends, thereby increasing offshore losses. Thus, groins may displace nearshore sandbar systems lakeward.

Figures IV-9 and IV-10 show examples of stone and sheet pile groin systems designed to maintain a beach composed of coarse sand or 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

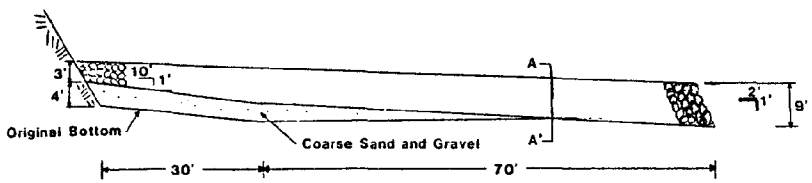
Figure IV-9

TYPICAL STONE GROIN SYSTEM
WITH ARTIFICIALLY NOURISHED BEACH

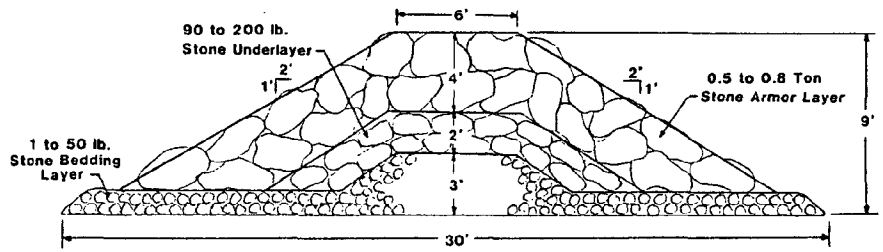


PLAN

Photograph by SEWRPC



PROFILE B-B'



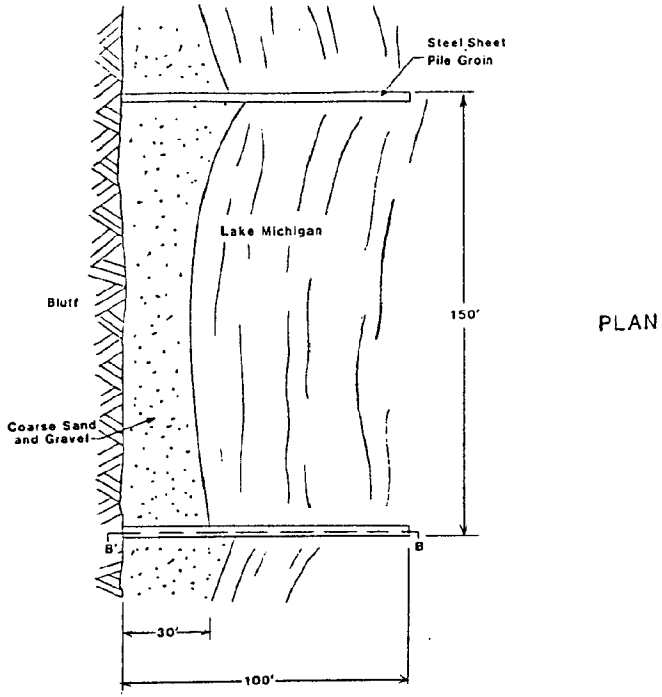
CROSS SECTION A-A'

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: U.S. Army Corps of Engineers and SEWRPC.

Figure IV-10

TYPICAL STEEL SHEET PILE GROIN
SYSTEM WITH ARTIFICIALLY NOURISHED BEACH



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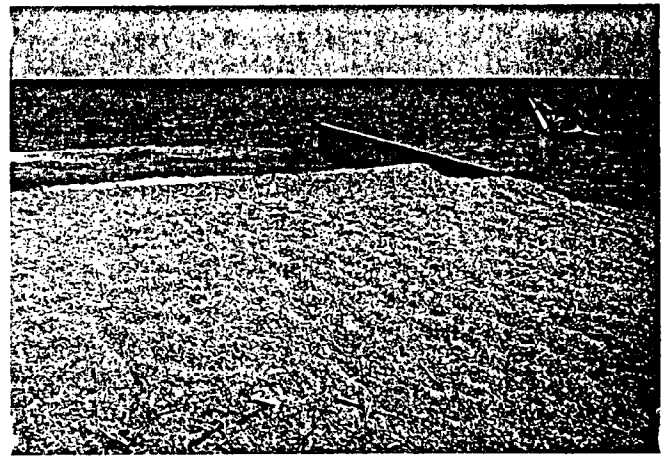
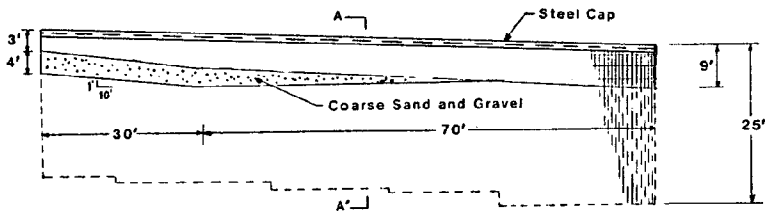
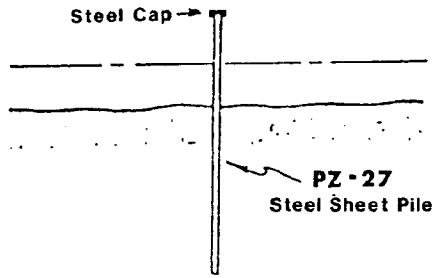


Photograph by U.S. Army Corps of Engineers



PROFILE B-B'



CROSS SECTION A-A'

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.

and spacing of a groin system is highly dependent on the site specific details of the project location, but spacing should generally be equal to about twice the groin length. The groins should be of sufficient height to prevent excessive overtopping. Periodic replenishment of the beach material may be required.

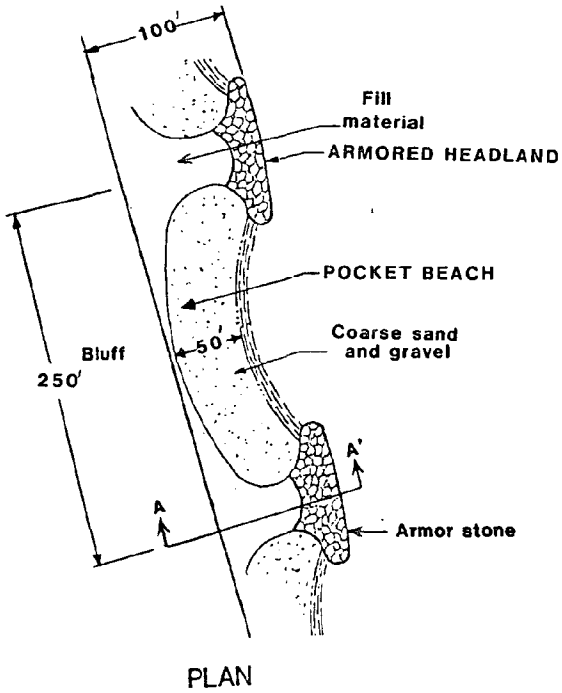
The capital cost of a groin system ranges from $200 to $500 per lineal foot of shoreline, with an additional cost ranging up to approximately $115 per lineal foot of shoreline to artificially nourish the beach by shore, or an additional cost ranging up to approximately $500 per lineal foot of shoreline to artificially nourish the beach by barge. Annual maintenance costs depend upon the need for additional fill material, and are estimated to approximate $20 to $60 per lineal foot.

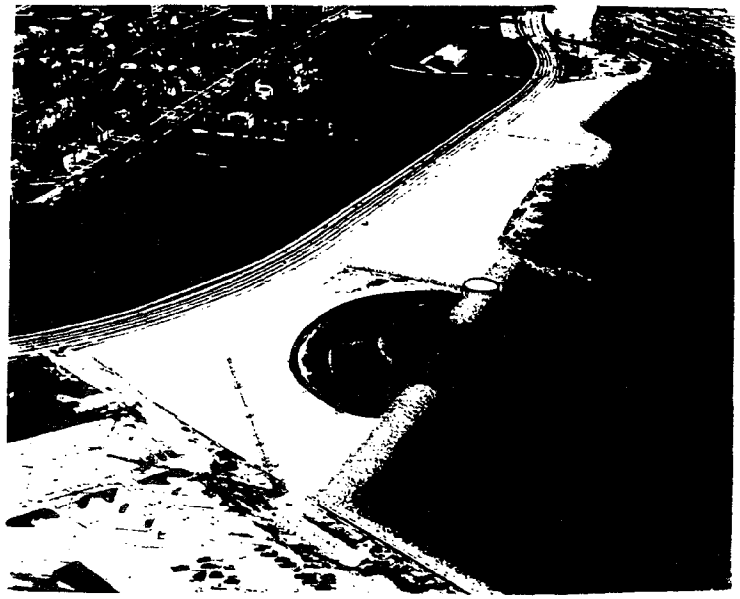
Armored Headland-Pocket Beach System--An armored headland and pocket beach system acts similar to a groin system in that the headland is connected to and extends out from the shoreline. Coarse beach material is trapped or held within the pocket areas of the structure, as shown in Figure IV-11. The headlands 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.

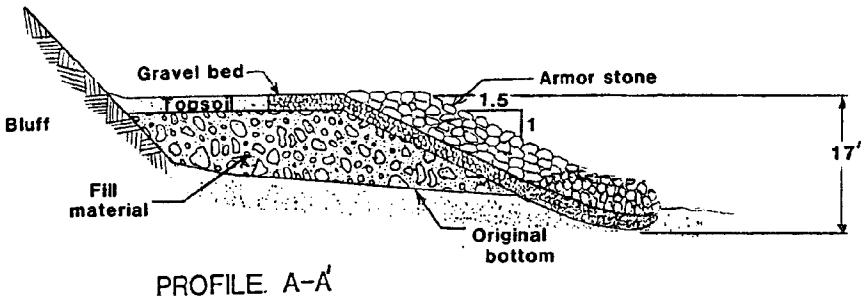
The capital cost of a headland and pocket beach system would range from $600 to $1,200 per lineal foot of shoreline. Average annual maintenance costs would range from $20 to $60 per lineal foot.

Nearshore Reefs--Nearshore reefs are constructed of stone and placed generally parallel to the shoreline in a water depth of about four to five feet. Such reefs are generally located less than 100 feet from the shoreline, as shown in Figure IV-12. 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 may be required.

Figure IV-11
TYPICAL ARMORED HEADLAND AND POCKET BEACH SYSTEM





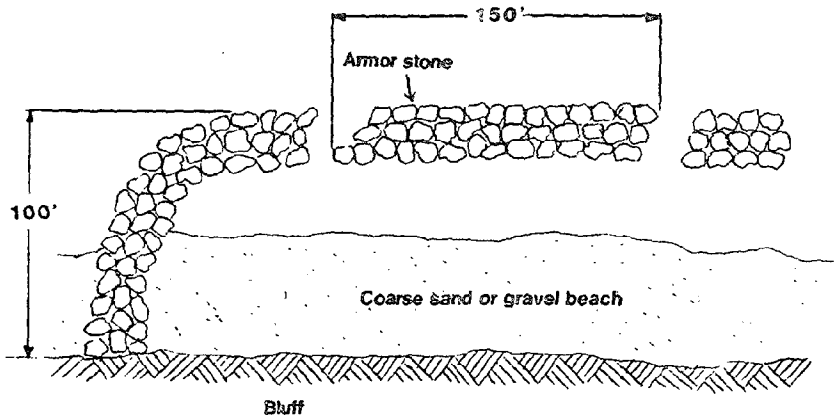


Source: Warzyn Engineering, Inc. and SEWRPC

NOTE: The design specifications shown herein are for a typ structure. The detailed design of shore protection sures must be based on a detailed analysis of wave c ate, cost and availability of construction material, specific gravity and quality of the stone, type of bed material, and existing shoreline geometry.

Figure IV-12

TYPICAL NEARSHORE STONE REEF WITH NOURISHED
COARSE SAND AND GRAVEL BEACH



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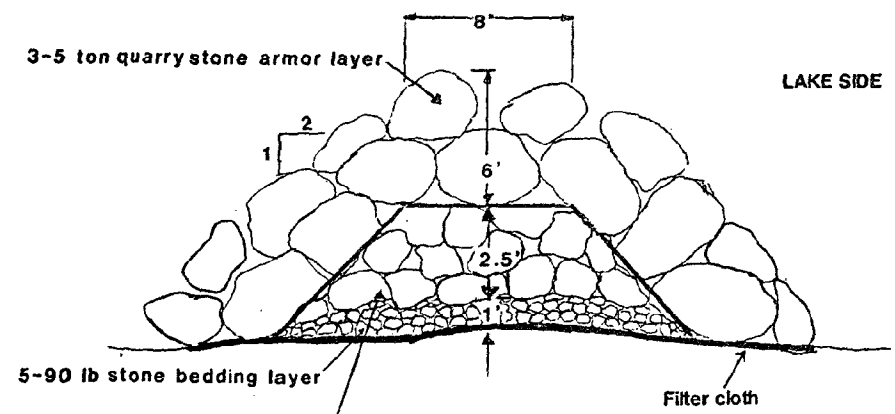


PROFILE

Photograph by STS Consultants,Ltd

Source: STS Consultants, Ltd. and SEWRPC

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.

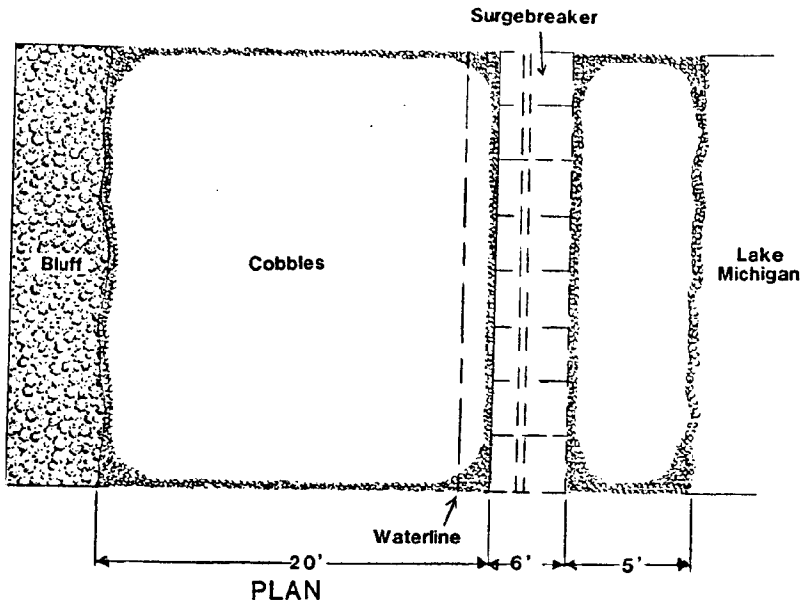
The capital cost of a nearshore reef ranges from $350 to $600 per lineal foot of shoreline, with an additional cost ranging up to $115 per lineal foot of shoreline to artificially nourish the beach by shore, or an additional cost of up to approximately $500 per lineal foot of shoreline to artificially nourish the beach by barge. Annual operation and maintenance costs would depend upon the need for periodic re-nourishment of the beach material, and are estimated to range from $25 to $70 per lineal foot.

Perched Cobble Beach System--Perched beaches constructed of cobbles would serve as wave-absorbing structures. A beach constructed of cobble stones ranging from 3 to 12 inches in diameter, as shown in Figure IV-13, would be able to absorb considerable wave energy while staying intact better than beaches composed of sand and gravel. The reduced wave reflection would help prevent scouring of the lake bed by wave energy normally reflected by bulkheads or riprap revetments. The disadvantage of a cobble beach system is that the use of the shoreline and access to the water is severely limited. The useability of cobble beaches can be enhanced by the placement of a one to two-foot layer of gravel on top of the cobbles. However, it may be difficult to maintain that gravel layer in a high wave energy environment. Cobble beaches may also not be as durable as riprap revetments.

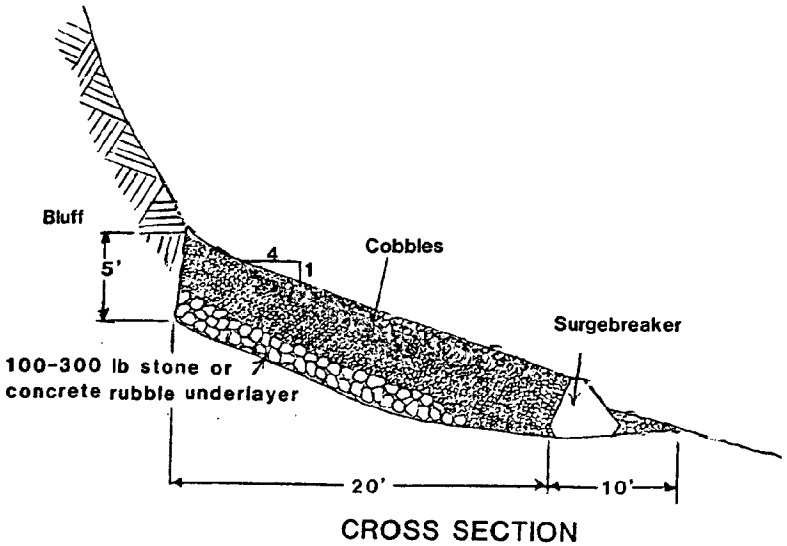
To increase the effectiveness of the cobble beach and prevent the migration of the cobbles, a sill would be placed offshore, which creates a perched beach. The sills could be constructed of precast concrete units called Surgebreakers, quarry stone, or steel sheet pile. Surgebreakers, as shown in Figure IV-13, are permeable, precast, steel-reinforced concrete units weighing approximately 4,000 pounds each, and measuring approximately four feet high, four feet wide, and six feet deep. They are set adjacent to each other in water typically from three to eight feet deep. The structures are secured to each other with steel cables. The Surgebreakers sloped front and back profile, and tapered openings allows transmission wave energy with little wave reflection or resulting scouring of the lake bed. Heavy construction equipment is required to install the structures. The capital cost of Surgebreakers would be approximately $250 per linear 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.

Figure IV-13

TYPICAL PERCHED COBBLE BEACH SYSTEM



Photograph by SEWRPC



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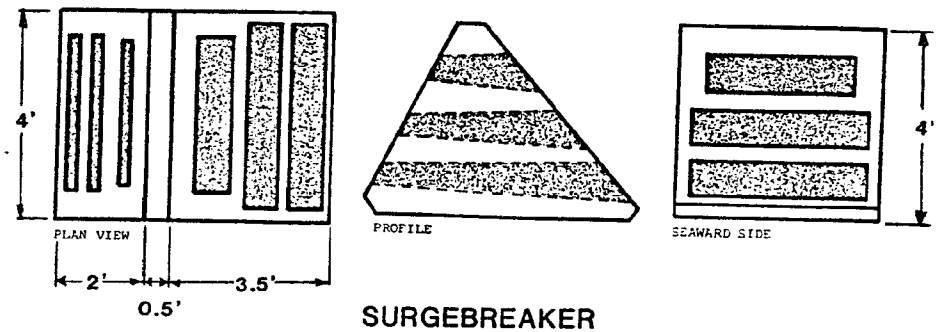


Source: Great Lakes Environmental Marine, Ltd. and SEWRPC

The cobble beach system, including a Surgebreaker sill, would entail a total capital cost of about $400 to $500 per lineal foot of shoreline. Annual maintenance costs, which depend upon the need to renourish the supply of cobbles, would approximate $30 per foot.

Manufactured Concrete Systems--Several types of manufactured concrete structures can be used to construct beach containment facilities. The Nellco Beach Building Blocks are described below as a representative system.

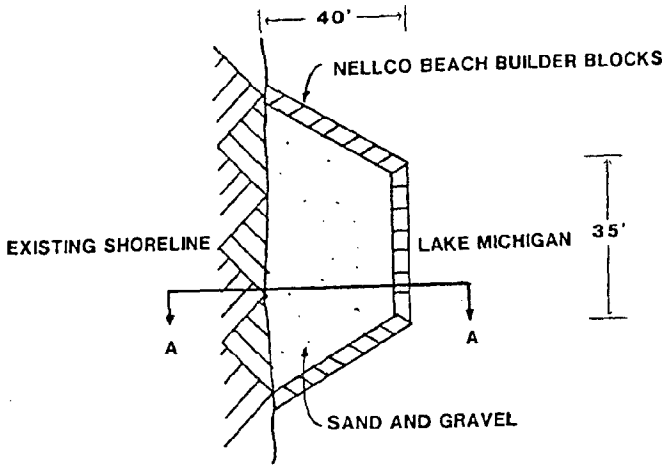
Nellco, Inc., developed a concrete block system designed to contain a beach area. The blocks, which are steel reinforced, have approximate dimensions of six feet by six feet by five feet, and weigh about 13,000 pounds each. As shown in Figure IV-14, the blocks are usually placed offshore side by side in water three to four feet deep. Blocks are also 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, waterborne particles behind the blocks. The beach could also be artificially nourished. Toe protection and a filter layer would be used to prevent scouring and the uneven settling of the blocks.

The capital cost of the Nellco blocks would be approximately $250 per linear foot of shoreline with an additional cost ranging up to $115 per linear 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. The annual operation and maintenance cost would be about $25 to $60 per lineal foot, depending primarily upon the need for periodic re-nourishing of the beach material.

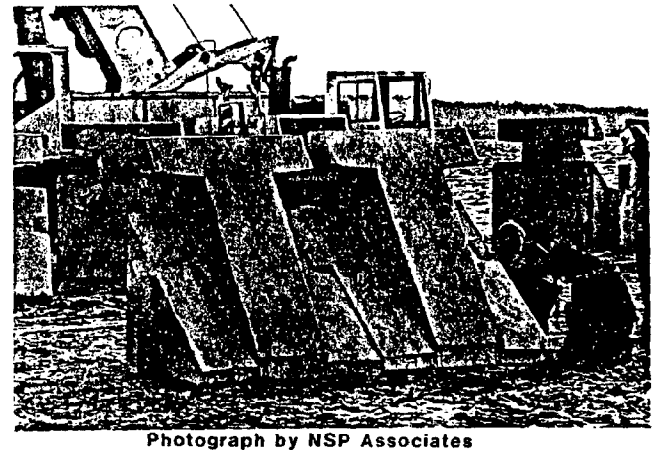
Offshore Breakwater: Breakwaters are protective structures built out from, and generally parallel to, the shore. The breakwaters protect the shore by modifying wave action, reducing deep water wave energy, and usually promoting sediment deposition or maintenance of existing sediment shoreward of the structure. Breakwater systems can help contain large, nourished sand beaches. The structures are generally constructed of stone, although some designs use rock-filled concrete caissions, cellular sheet piles, timber cribs, and floating devices. One advantage of any nearshore, or offshore, protection system is that the structures are positioned off of the existing shoreline, thereby

FIGURE IV-14

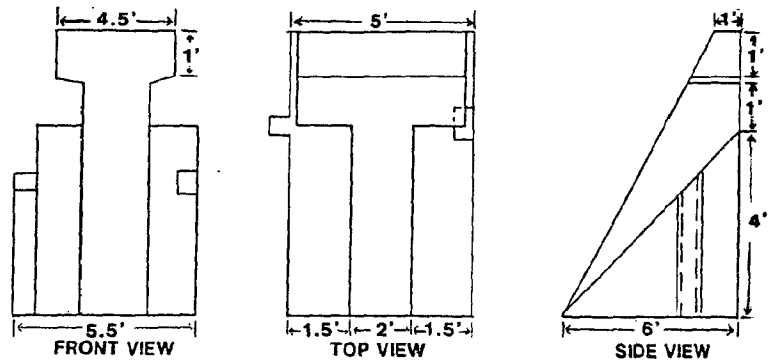
TYPICAL NELCO BEACH BUILDER BLOCKS WITH NOURISHED COARSE SAND AND GRAVEL BEACH

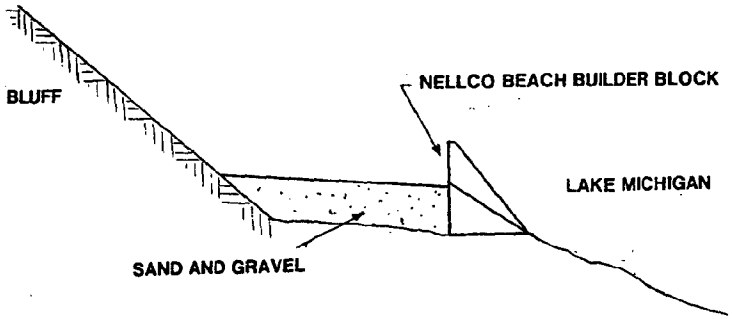


PLAN



Photograph by NSP Associates





CROSS SECTION A-A

NELLCO BEACH BUILDER BLOCK

Source: NSP Associates and SEWRPC

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.

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, nor promoting significant offshore losses. 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 offshore breakwaters along the Lake Michigan shoreline of Milwaukee County would entail a capital cost of approximately $1,000 to $2,000 per lineal foot of shoreline. Average annual maintenance costs would approximate $20 to $50 per lineal foot.

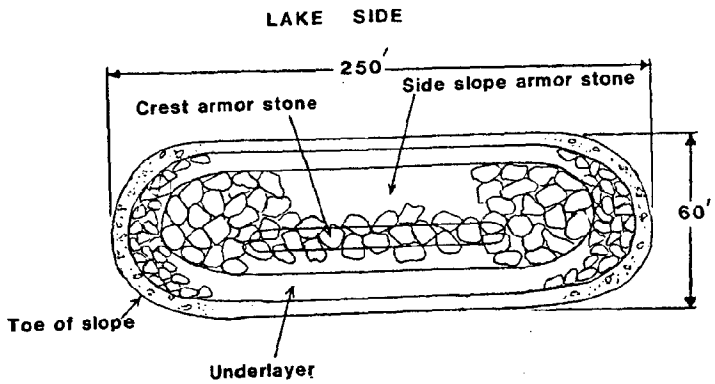
Described below are five alternative breakwater designs: a rubblemound breakwater, a caisson breakwater, a sheet pile breakwater, a timber crib breakwater, and a floating breakwater.

Rubblemound Breakwater--A rubblemound breakwater is the most common type of breakwater in the Great Lakes. The structure, as shown in Figure IV-15, 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 approximately one to 50 pounds. Armor stone forms the outer layer of the breakwater. The 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.

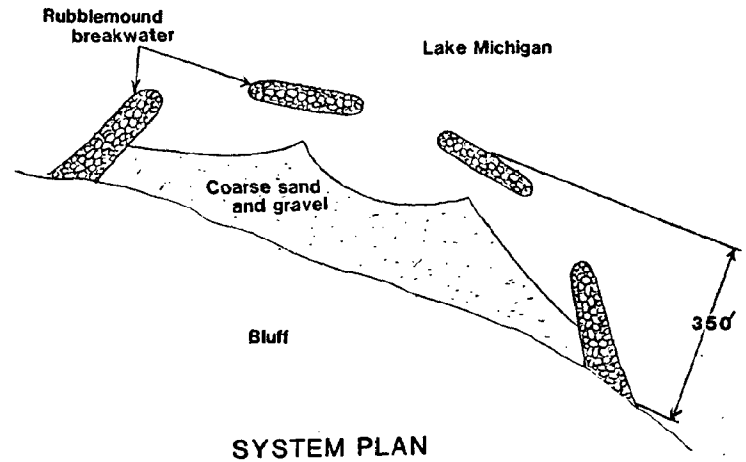
The rubblemound breakwater is intended to prevent or reduce the transmission of wave energy behind it by absorbing much of the energy and reflecting some of the remaining energy back to the main water body. If rubble breakwaters are too porous, they allow a high percentage of longer period wave energy to pass through, causing excessive wave action behind the structure.

Figure IV- 15

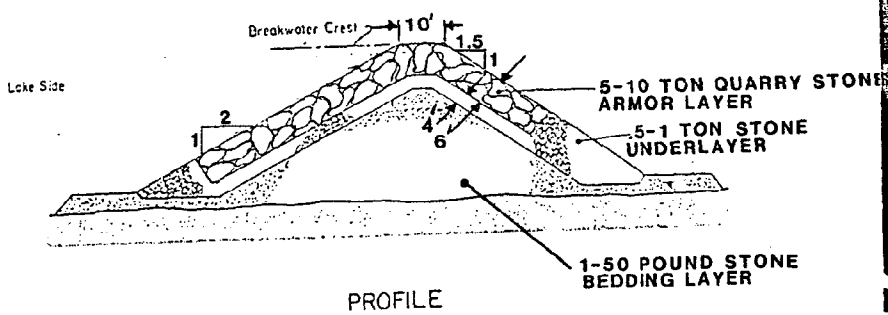
TYPICAL SEGMENTED RUBBLEMOUND BREAKWATER SYSTEM



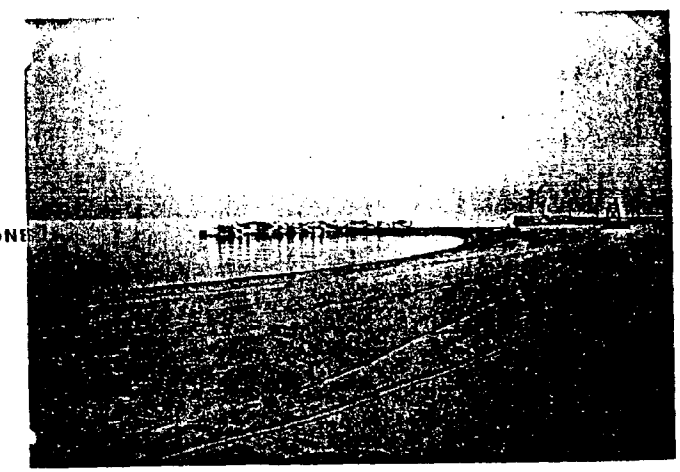
PLAN



SYSTEM PLAN



PROFILE



PHOTOGRAPH BY SEWRPC

Source: Warzyn Engineering, Inc. and SEWRPC

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.

An alternative method of rubblemound breakwater design is to create a berm of porous quarry stone on the lakeward side of the breakwater, as shown in Figure IV-16. This design, known as the "berm breakwater," utilizes a thick layer of armor stones which typically weigh less than one-half the weight of those required by conventional design methods. Wave action consolidates and reshapes the permeable armor layer into a well-nested armored surface. The relatively high porosity of the berm allows waves to propagate into the armor stones, dissipating energy over a large area.

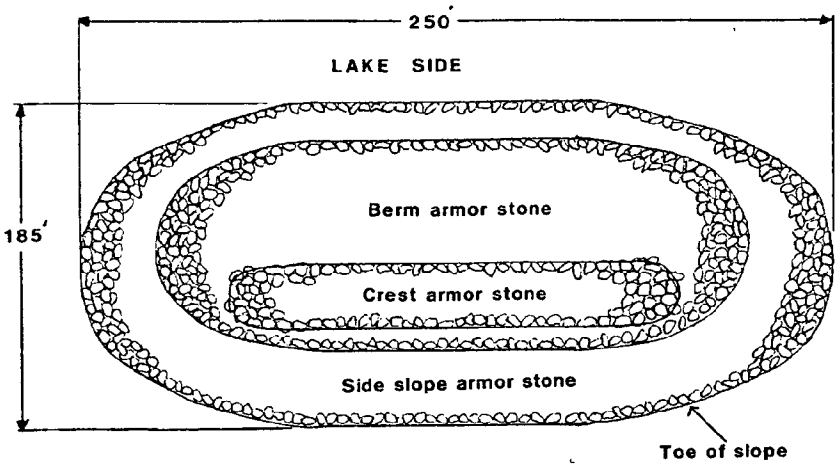
Berm breakwaters may have lower construction costs than conventional breakwaters because of increased local availability of suitably-sized armor stone.

Caisson Breakwater--A caisson breakwater, as shown in Figure IV-17, 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 1900s for construction of commercial harbors. At that time, the caissons provided distinct construction advantages in deep water 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 due to the relatively shallow water in which the breakwaters are located and the excessive cost of construction. In addition, when the caisson structures are not properly tied into the lake bed, 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 method is to provide 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 IV-17. 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. Although sheet pile

Figure IV- 16

TYPICAL BERM BREAKWATER SYSTEM



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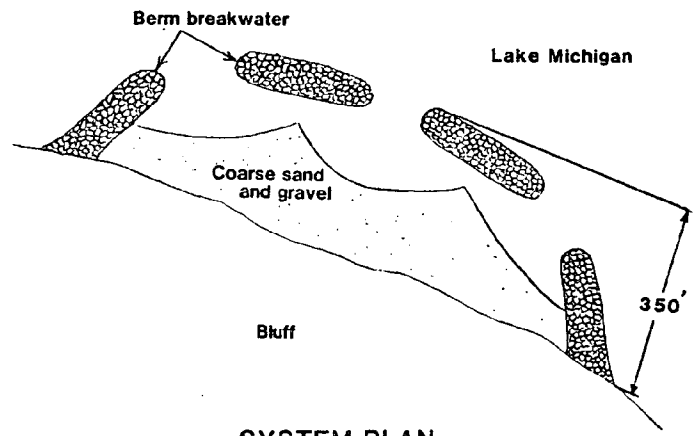
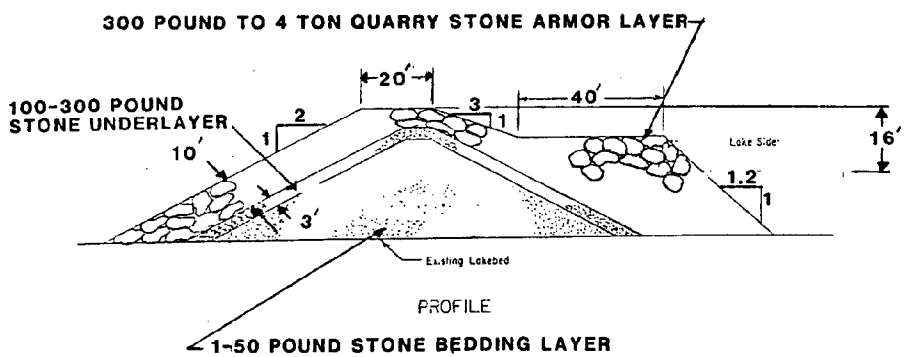
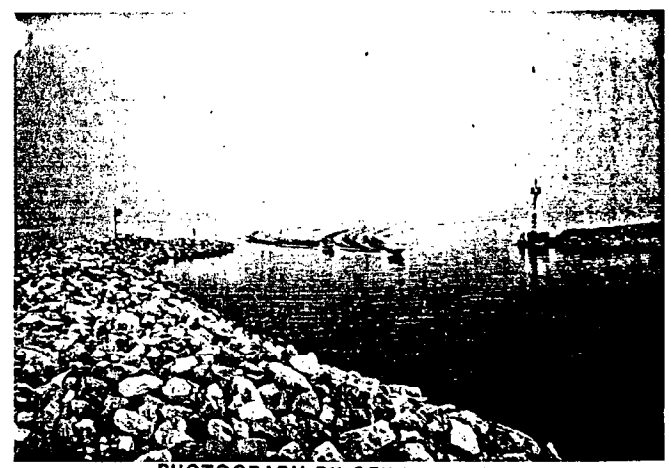


SYSTEM PLAN



Source: Warzyn Engineering, Inc. and SEWRPC



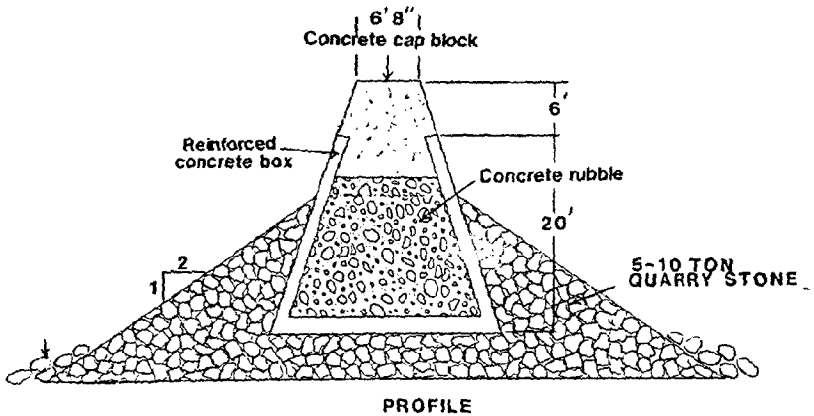
PHOTOGRAPH BY SEWRPC

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.

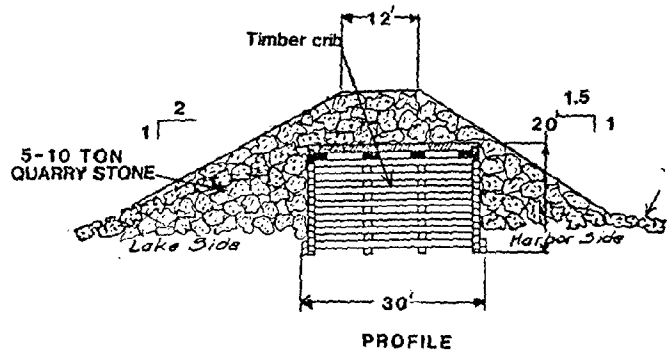
Figure IV- 17

MISCELLANEOUS ALTERNATIVE TYPES OF BREAKWATERS

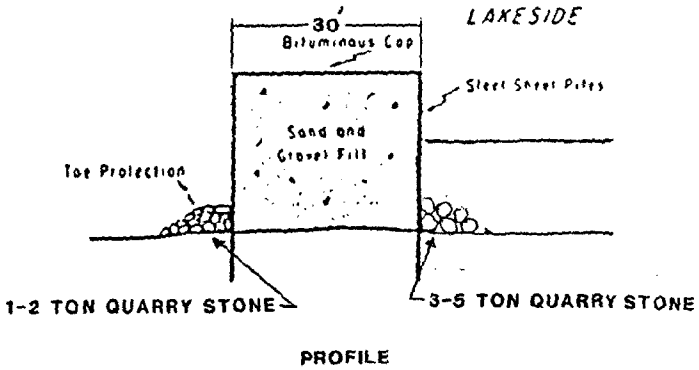
Caisson Breakwater



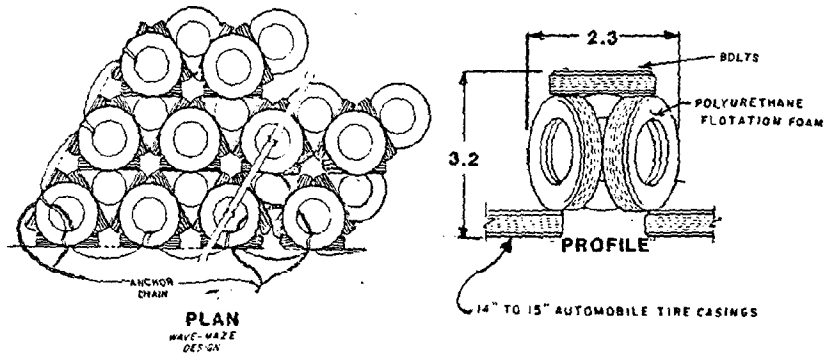
Timber Crib Breakwater



Sheetpile Breakwater



Floating Breakwater



Source: Milwaukee County Park Commission, United States Army Corps of Engineers and SEWRPC

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.

breakwater structures provide navigable water up to their edge, this benefit is usually outweighed by a high initial cost. Another 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.

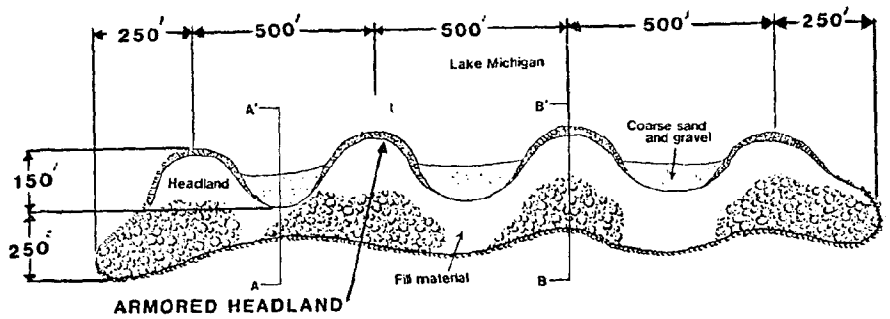
Timber Crib Breakwater--A fourth type of breakwater is known as a timber crib breakwater and is illustrated in Figure IV-17. Similar in construction to the caisson breakwater, the timber cribs are floated into position and settled on a prepared foundation by filling the compartments with stone. The toe of the structure is protected by rip-rap placed at the base of the structure. In the early 1900s, timber cribs were frequently used for the construction of harbors, including for the Port of Milwaukee. Timber crib structures are substantially more effective at dissipating wave energy than the vertical steel sheet pile structures. The major disadvantage of the timber cribs is the limited durability of wood compared to other materials, as exposed timbers are subject to decay.

Floating Breakwater--Floating breakwaters, as shown in Figure IV-17, are constructed of buoyant materials such as logs, hollow concrete boxes, and rubber tires. Floating breakwaters have not been able to both effectively and economically dissipate deep-water 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 excessive water depths. However, since floating breakwaters are effective only against small to moderate, short-period waves, they could only be used as supplementary protection in Milwaukee County. Most floating breakwaters would need to be removed during the winter to prevent ice damage to the structure.

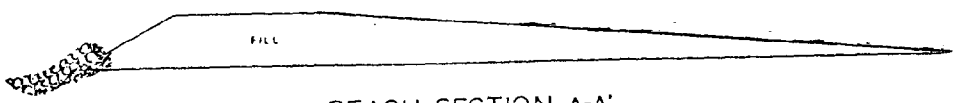
Offshore Islands and Peninsulas: Islands and peninsulas lying approximately 250 to 1,500 feet offshore could be constructed to provide substantial protection from wave action while creating additional recreational land. The islands or peninsulas , as shown in Figure IV-18, would be constructed of fill material consisting of rubble, soil, or tunnel construction debris. The fill material could be protected from wave action by the use of a revetment or an

Figure IV-18

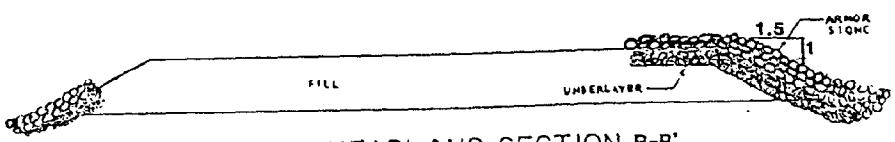
TYPICAL OFFSHORE ISLAND OR PENINSULA



PLAN



BEACH SECTION A-A'



HEADLAND SECTION B-B'

PHOTOGRAPH BY WARZYN ENGINEERING, INC.

Source: Warzyn Engineering Inc. and SEWRPC.

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.

armored headland and pocket beach system. The offshore islands or peninsulas, like offshore breakwaters, dissipate 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 deeper water 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 $1,200 per lineal foot of shoreline. Average annual maintenance costs approximate $20 per lineal foot.

Bluff Slope Stabilization

In Chapter III of this report, 59 bluff analysis sections covering 59,430 feet, or 37 percent of the total Milwaukee County shoreline, were classified as having marginal or unstable bluff slopes. 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 45,490 feet, or 29 percent of the County shoreline. Bluff analysis sections identified as needing bluff slope regrading were those in which other economically feasible measures would not effectively stabilize the bluff slopes. A primary advantage of bluff slope regrading is that further bluff recession would be prevented--if bluff toe protection and surface and groundwater drainage would be provided where needed.

Slope regrading would also provide structural stability to the bluff toe protection measures, preventing them from being buried by bluff material, and from erosion from behind the structure.

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 dust, noise, and aesthetic impacts at the construction site itself.

Four alternative methods for bluff slope regrading, as shown in Figure IV-19 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.

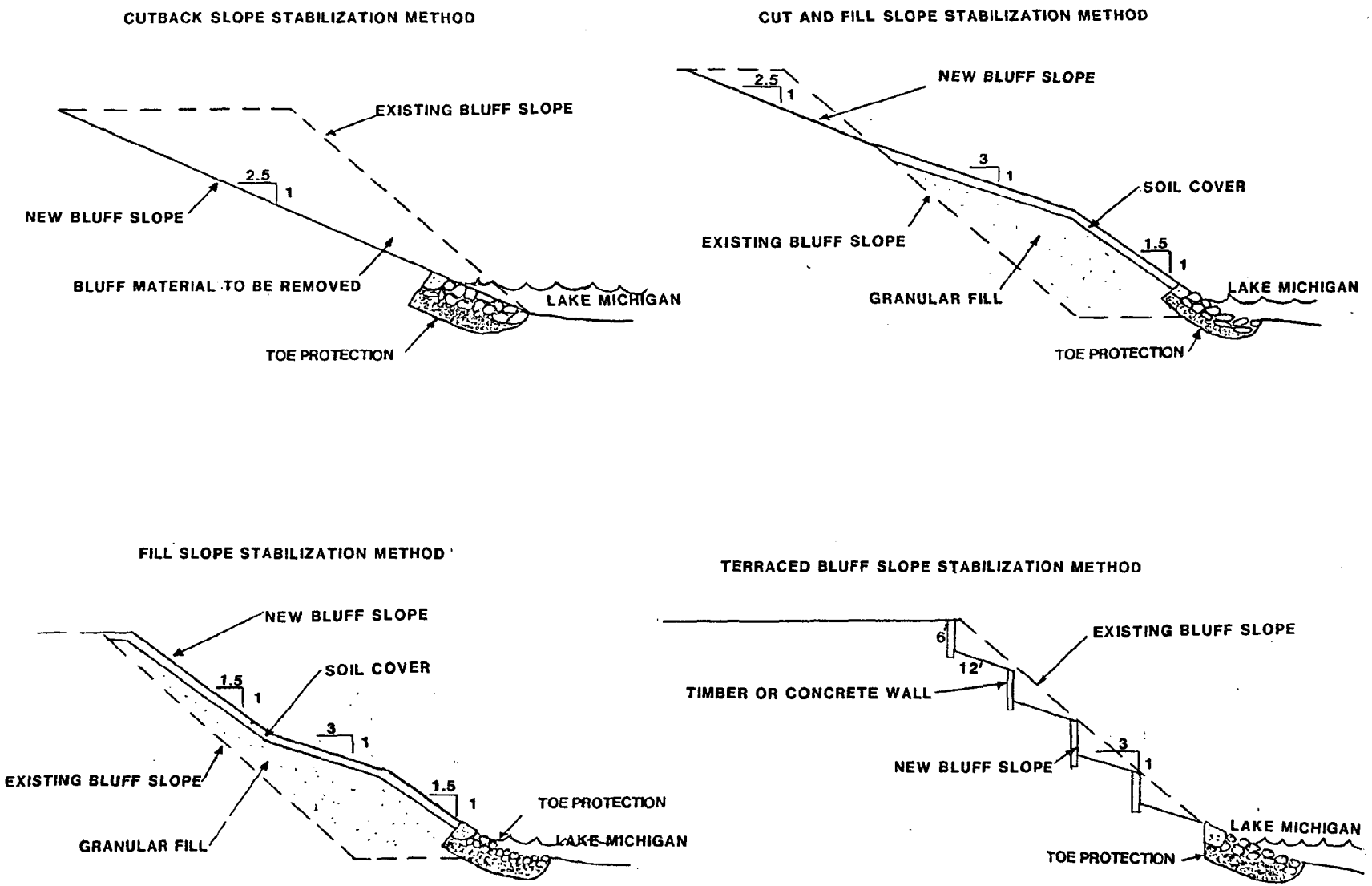
Cutback Method--Bluff slope regrading can be accomplished by using earthmoving equipment to regrade the face of the slope to a flatter, more stable profile, as shown in Figure IV-19. As already noted, a bluff slope of one on two and one-half will usually provide a stable bluff slope in the study area. The cutback method can only be used in areas where houses, or other lakefront structures 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.

Bluff slope regrading using the cutback method would entail a capital cost of approximately $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 the 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 houses or other

Figure IV- 19

ALTERNATIVE METHODS OF BLUFF SLOPE STABILIZATION



Source: SEWRPC.

structures or facilities are located close to the edge of the bluff. The fill materials, as shown in Figure IV-19, should be granular. Fine-grained, clay-type materials are not suitable for fill material because such materials may cause groundwater drainage problems. Depending on the type of material used for filling, a slightly steeper angle--some times 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 protective 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 or other structures are located within about 50 feet of 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 approximately $150 to $250 per linear foot of shoreline. Maintenance costs are assumed to be about $10 to $15 per lineal foot during the first three years following bluff slope regrading, primarily for the maintenance of a 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 IV-19. The cut and fill method is limited in use to only those areas in which houses and other structures 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 approximately $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 IV-19. The retaining walls may be constructed of stone, timber, interlocking concrete blocks, steel sheet pile, or gabions. The bluff slope between the retaining walls are regraded to a slope of one on two and one-half, 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 terraced bluff slope may entail a capital cost of approximately $1,000 to $3,500 per lineal foot of shoreline. Annual maintenance costs would be approximately $10 to $15 per lineal foot during the first three years following bluff slope regrading, primarily for the maintenance of a vegetative cover.

Groundwater Drainage: Groundwater drainage was indicated to enhance slope stability in eight bluff analysis sections, covering 10,970 feet, or 7 percent of the Milwaukee County 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 increased seepage caused by the compression of saturated soils by the weight of the fill material. Drainage systems require 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 economi-

cally feasible only in granular lake sediment 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 IV-20, a system of collector pipes or ditches are provided to carry the collected water to the base of the bluff or to a suitable outlet. A horizontal drainage system is most effective in layers of granular lake sediment containing sand and gravel. Drains are usually spaced across the face of the bluff slope at 20- to 50-foot intervals.

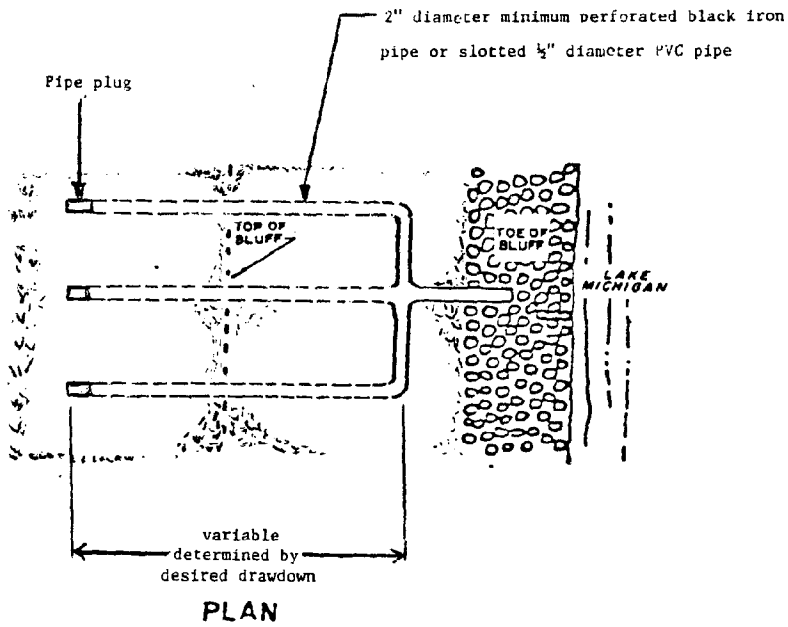
Advantages of a horizontal drain system are that the system drains by gravity, and requires relatively little maintenance. The primary disadvantage of the system is that access to the base of the bluff to install the drains is often difficult.

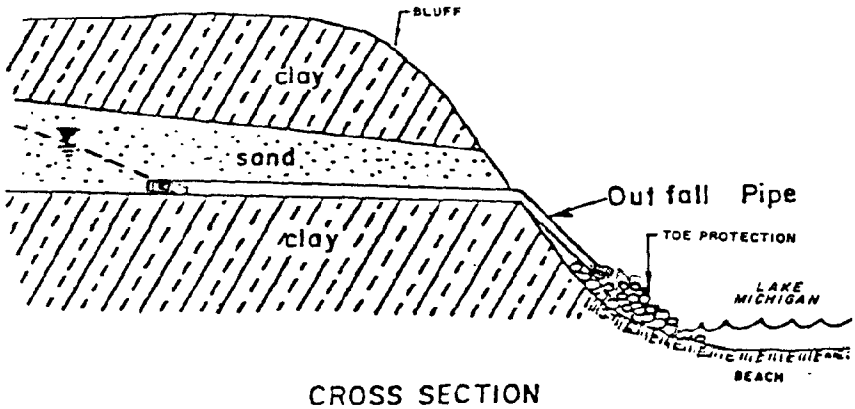
Construction of a horizontal drain system to lower the level of groundwater would entail a capital cost of approximately $30 to $50 per lineal foot of shoreline. The annual operation and maintenance cost would range from $5 to $10 per lineal foot.

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 either be pumped from the well, or tapped with a gravity outlet, as shown in Figure IV-21. 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,

Figure IV -20

HORIZONTAL DRAINAGE SYSTEM



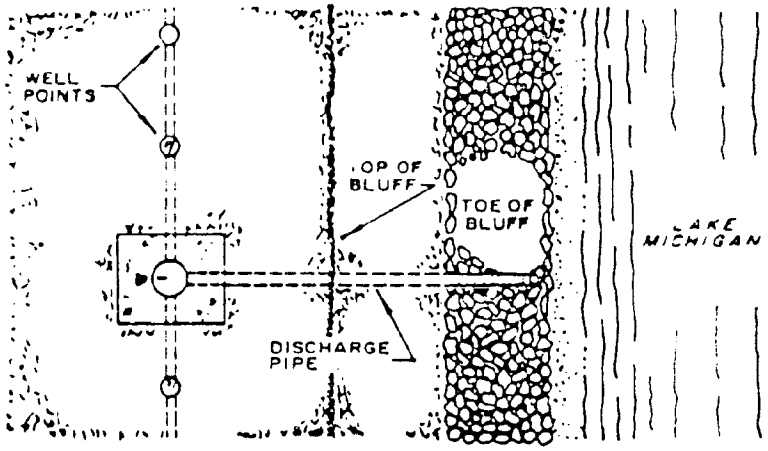


Source: Owen Ayres & Associates, Great Lakes Shore Erosion Protection, Structural Design Examples, 1978. and SEWRPC

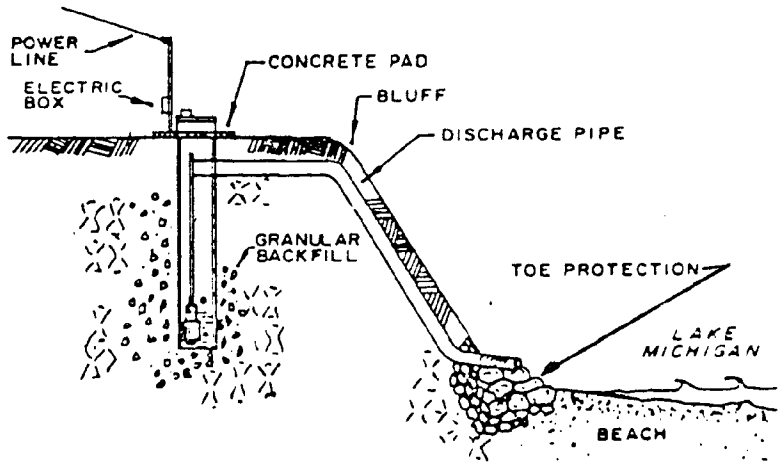
Figure IV - 21

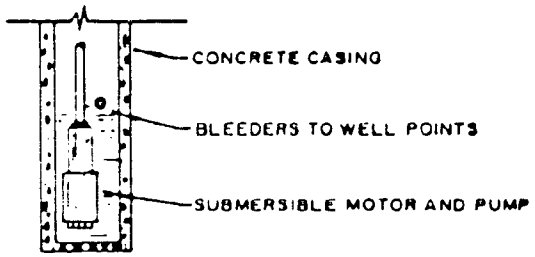
VERTICAL DRAINAGE SYSTEM

PLAN



CROSS SECTION





Source: SEWRPC

access to install the drains is generally not a problem because vertical drains are installed from the top of the bluff. 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 approximately $50 to $75 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 IV-22. 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.

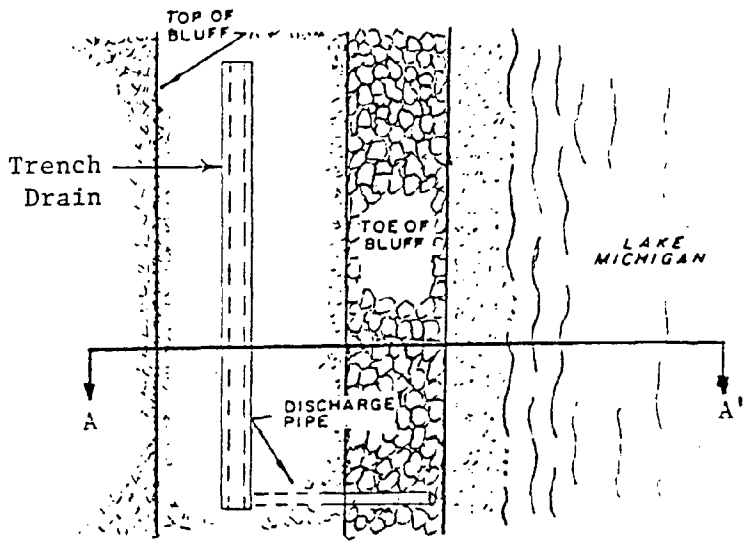
Construction of a trench drain may entail a capital cost of approximately $20 to $40 per lineal foot of shoreline, with an annual maintenance cost of up to $5 per lineal foot.

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 Milwaukee County shoreline. Specific drainage problems were identified within each of these sections, reducing the stability of the bluff slopes. 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 IV-23. Surface water drainage systems have a relatively low cost, require little maintenance, and should not limit the recreational use of the shoreline.

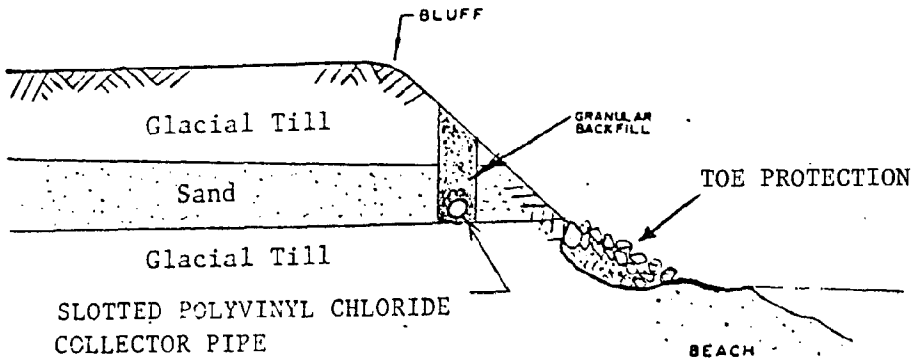
A drainage system would entail a capital cost of about $10 to $70 per lineal foot of shoreline, with an annual maintenance cost of up to $5 per lineal foot.

Figure IV - 22

TRENCH DRAINS



PLAN

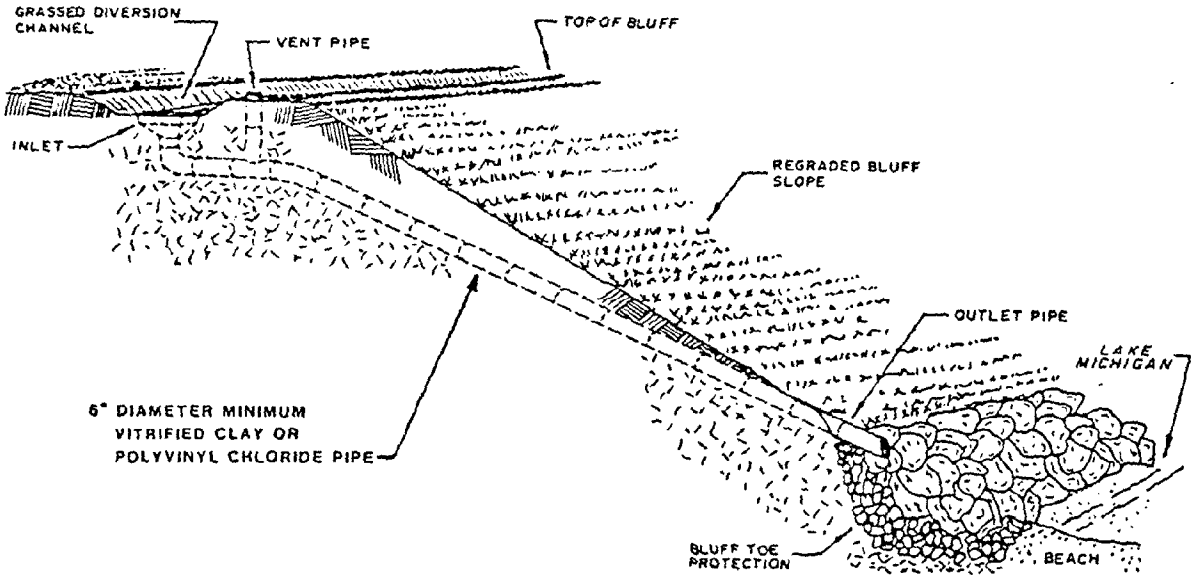


CROSS SECTION A-A'

Source: SEWRPC.

Figure IV-23

STORMWATER DRAINAGE SYSTEM TO PREVENT EXCESSIVE STORM RUNOFF OVER THE TOP OF THE BLUFF



Source: SEWRPC.

Revegetation: Revegetation of the bluff slope as a means to enhance slope stability was indicated for portions of nine bluff analysis sections. 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, 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 to revegetate 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 is 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 would be applied as needed, and mulch would be 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 1000 square feet if scattered by hand, to $40 per 1000 square feet if hydroseeding or drilling were used. Annual maintenance costs for the first three years following seeding would approximate $5 per 1000 square feet for hand scattering, to $10 per 1000 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 1000 square feet. Annual maintenance cost would range from approximately $40 to $100 per 1000 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 setback 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.

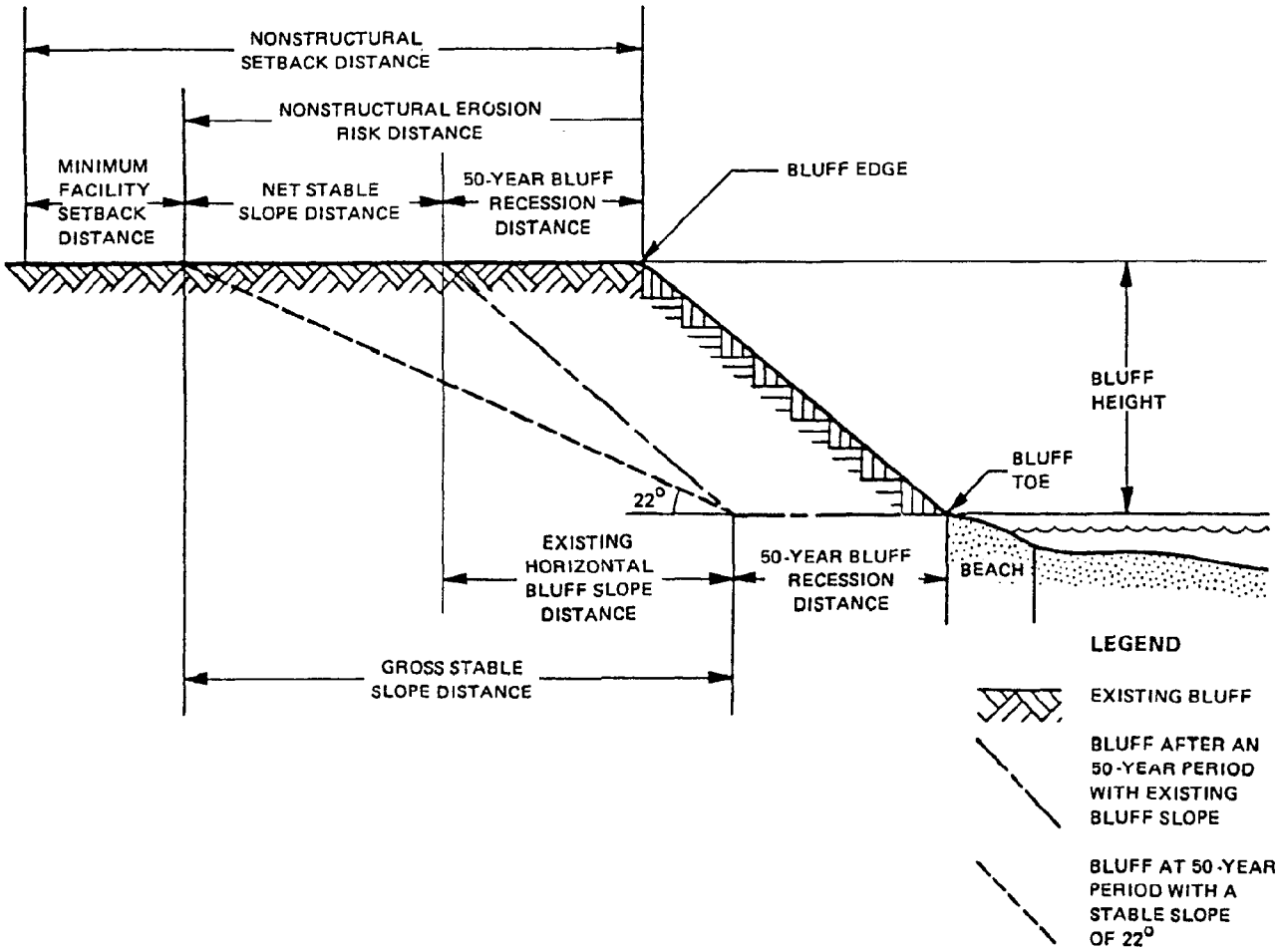
Currently, under the State shoreland zoning legislation, which applies to unincorporated areas, structures must be set back a minimum of 75 feet from the ordinary high water line. In addition, five Wisconsin counties --Douglas, Manitowoc, Sheboygan, Ozaukee, and Racine--have adopted more stringent shoreline setback ordinances which take into account Lake Michigan coastal erosion rates. The setback distances established by the five counties generally consist of a stable slope component based on a stable bluff slope of one on two and one-half, plus the anticipated recession of the bluff which may be expected to occur over an approximate 50-year period.

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 IV-24. 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 delineated for a 50-year period of continued bluff recession. The face of the bluffs are assumed to be graded to a stable slope of approximately one on two and one-half, or about 22 degrees, as discussed in Chapter III of this report.

Figure IV-24

PROCEDURE UTILIZED TO ESTIMATE NONSTRUCTURAL EROSION RISK DISTANCE AND NONSTRUCTURAL SETBACK DISTANCE



NONSTRUCTURAL EROSION RISK DISTANCE=NET STABLE 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: TO PROVIDE A SAFETY FACTOR, FOR AESTHETICS, AND FOR PROVISION OF FUTURE SURFACE WATER AND GROUNDWATER DRAINAGE SYSTEMS

Source: SEWRPC.

Minimum facility setback distances are recommended because future bluff recession rates could differ substantially from the historic 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 IV-25. 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 portions of the Lake Michigan shoreline which 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. A minimum facility setback distance of 50 feet is recommended for all permanent buildings and facilities.

Regulation of Lake Michigan Water Levels

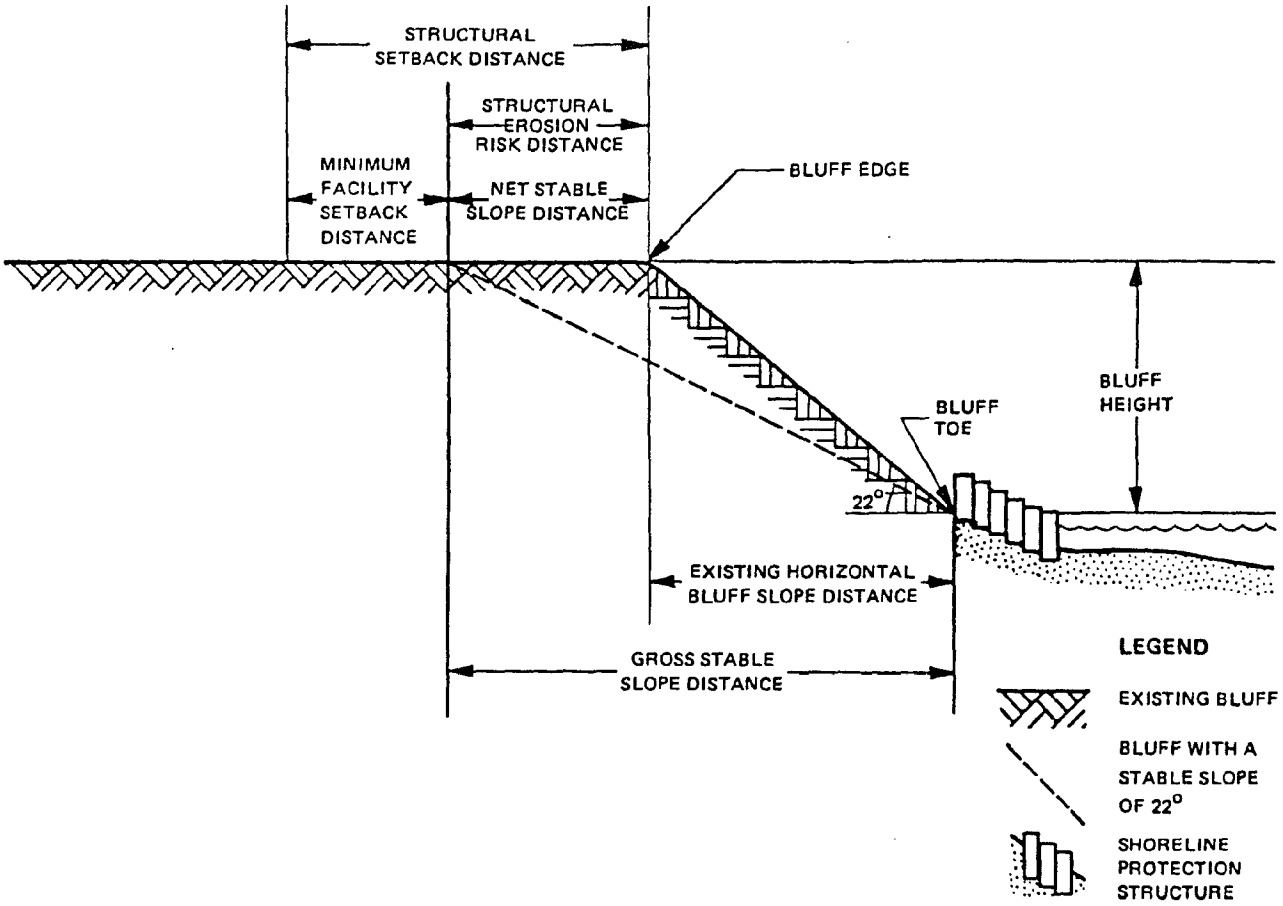
Regulation of Great Lakes water levels has been proposed as one method to help alleviate increased shoreline erosion caused by high water levels. The increased regulation of the water levels could be accomplished by increased dredging of the lakes outlet channels, by modifying existing diversions into and out of the lakes, and by construction of new diversions.

As discussed in Chapter II, 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 include the Long Lac, Ogoki, and Chicago diversions, the Welland Canal, and the New York State Barge Canal. These diversions were discussed in Chapter II.

The combined average flow for the Long Lac and Ogoki diversions is about 5,600 cubic feet per second (cfs). By comparison, the annual average outflow from Lake Superior over the period of 1900 through 1986 is about 76,000 cfs.

Figure IV-25

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: TO PROVIDE A SAFETY FACTOR, FOR AESTHETICS, AND FOR PROVISION OF FUTURE SURFACE WATER AND GROUNDWATER DRAINAGE SYSTEMS

Source: SEWRPC.

It should be noted that the diversion of water from the Ogoki River has been temporarily reduced or stopped during the high water periods of 1951 through 1953, 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.[1]

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 theoretical effects of these diversions, other than the New York State Barge Canal, on Great Lakes water levels--as determined by the International Great Lakes Diversions and Consumptive Uses Study Board of the International Joint Commission--is indicated in Table IV-4. The New York State Barge Canal, it should be noted, has little effect on the water levels of the Great Lakes.

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[1]Great Lakes Commission, "Water Level Changes--Factors Influencing the Great Lakes," 1986.

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Table IV-4

ESTIMATED THEORETICAL EFFECT OF EXISTING
DIVERSION RATES ON GREAT LAKES WATER LEVELS

Diversion	Rate (cfs)	Effect on Mean Water Level (feet)[a]			
		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[b]	-0.06	-0.18	-0.44	0

[a]A positive sign (+) indicates an increase in level; a negative sign (-) indicates a decrease.

[b]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.

Water levels in the Great Lakes can be partially regulated by means of artificial outlet control structures. Presently, 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. The comprehensive study of fluctuating Great Lakes water levels recently undertaken by the International Joint Commission, as described in Chapter II, will examine the potential regulation of Great Lakes water levels.

ALTERNATIVE SHORELINE EROSION, BLUFF RECESSION, AND STORM DAMAGE CONTROL PLANS

Alternative shoreline erosion, bluff recession, and storm damage control plans were developed for two plan elements: a bluff stabilization plan element and a shoreline protection plan element. Each of the alternative plans, described in more detail below, is designed to protect the entire shoreline of Milwaukee County. An estimate of the total capital cost and annual maintenance cost of each plan is also provided.

Bluff Stabilization Plan Element

The bluff 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 groundwater flow. Site specific stabilization methods which can effectively abate problems of bluff instability can be evaluated only in the preliminary engineering and final design phases of plan implementation. The bluff stabilization measures thus will represent the first element of the recommended plan and of alternatives thereto.

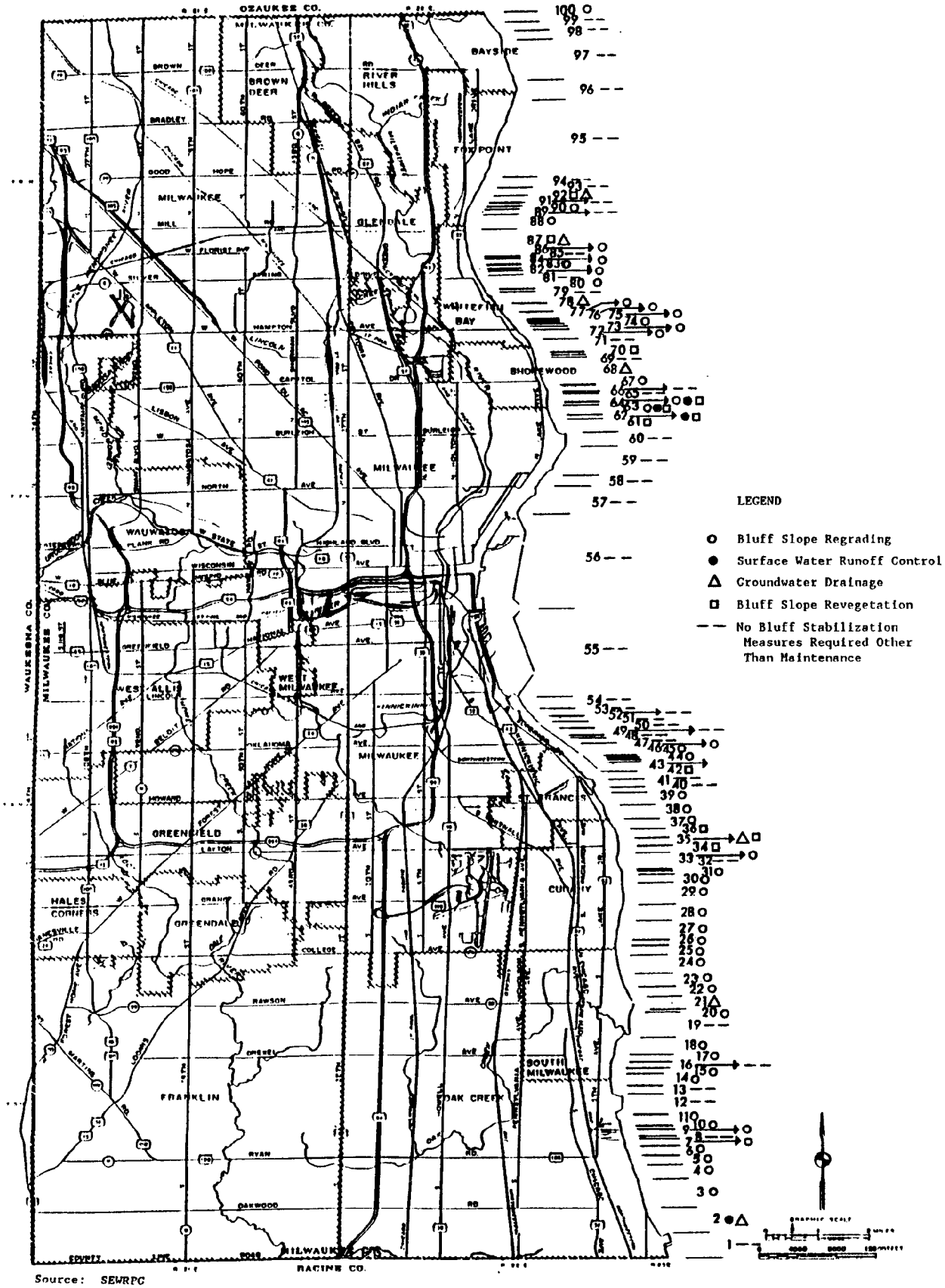
The bluff stabilization plan element consists of measures which would stabilize the bluff slopes within each of the bluff analysis sections, as indicated in Chapter III of this report. The bluff stabilization plan element is illustrated on Map IV-1.

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 slope regrading is indicated as a control measure. The appropriate method for regrading the bluff slope within a particular bluff analysis section--neither the cutback method, the cut and fill method, or the fill method--should be based on more site-specific data and analyses. The appropriate method of bluff regrading should be selected based upon the distance from the existing houses and other structures to the existing edge of the bluff, and the alignment of adjacent shoreline areas. Criteria recommended to be used in the selection of the method of regrading the bluff slope are set forth in Table IV-5.

Selection criteria and the estimated cost of each slope stabilization component, are set forth in Table IV-6. Bluff slopes would be regraded to a stable angle along about 40,400 feet of shoreline, or 25 percent of the total county shoreline. Detailed studies to determine the feasibility of installing groundwater drainage systems along about 11,000 feet of shoreline, or 7 percent of the total county shoreline, would be conducted. Surface water runoff control would be provided along about 4,400 feet of shoreline, or 3 percent of the county shoreline. Revegetation of at least a portion of the bluff face would be provided along about 10,700 feet of shoreline, or 7 percent of the county shoreline.

Map IV-1

BLUFF STABILIZATION PLAN ELEMENT FOR MILWAUKEE COUNTY



Source: SEWRPC

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Table IV-5

RECOMMENDED CRITERIA FOR THE SELECTION OF A BLUFF SLOPE REGRADING METHOD

Preferred Method of Bluff Slope Regrading	Selection Criteria	
	Distance From Existing Bluff Edge House or Structure	Alignment of Adjacent Shoreline Areas
Cutback[a]	Distance between the bluff edge and existing houses or other structures is greater than the distance required to provide a stable bluff slope of one on two and one-half, or 22 degree plus a 50-foot buffer zone.	Adjacent shoreline areas do not extend, nor are proposed to extend, more than 25 feet lakeward of the subject property shoreline.
Fill	Distance between the bluff edge and existing houses is less than 50 feet.	Adjacent shoreline areas extend, or are proposed to extend, at least 50 feet lakeward of the subject property shoreline.
Cut and Fill[a]	Distance between the bluff edge and existing houses is greater than 50 feet, but less than the distance required to provide a stable bluff slope of one on two and one-half, or 22 degrees, plus a 50-foot buffer zone.	Adjacent shoreline areas do not extend, nor are proposed to extend, more than 50 feet lakeward of the subject property shoreline.

[a]For the Cutback or Cut and Fill methods, both criteria should be met.

Source: SEWRPC.

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Table IV-6

SELECTION CRITERIA AND TYPICAL CAPITAL AND MAINTENANCE UNIT COST OF THE 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.	Variable depending upon type of draingae problem.	Variable depending upon type of drainage problem.
Revegation	Areas where lack of vegetation could cause translational sliding.	350/1,000 ft.^{2}	10/1,000 fta
Regrading Bluff Slope	Areas where 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 apply only to the first three years following construction of the bluff slope stabilization method.

Source: SEWRPC.

The bluff stabilization plan element would have an estimated capital cost of about $6.7 million, and an average annual maintenance cost of about $748,000. About 85 percent of the maintenance cost, however, would be required only during the first three years following construction.

Shoreline Protection Plan Element

The shoreline protection plan will represent the second element of the recommended plan and of alternatives thereto. Three conceptual alternative plans were developed to protect the shoreline from wave and ice erosion. The first alternative plan 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 alternative plan for shoreline protection would provide, wherever practicable, artificially nourished beach systems with either onshore or nearshore structures being used to help maintain the beaches. The beach alternative plan would provide a usable beach, in most instance composed of gravel, for a large portion of the study area shoreline. For the purposes of the systems level plan, it was assumed that short groins constructed of quarry stone would be used to help contain the beach material, but other structures--notably steel sheet pile groins, armored headlands, or nearshore stone reefs--could also be used. The beach alternative plan would have a relatively moderate cost.

The third alternative plan for shoreline protection would utilize offshore peninsulas, islands, and breakwaters--along with some onshore structures--to protect the shoreline and provide limited sand beaches. This alternative plan would create over 200 acres of new lakefront land for recreational uses. The offshore alternative would have a relatively high cost.

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 sand beaches would be desired at lakefront parks which historically have contained beaches, and that furthermore, additional beaches would be desired. 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 screened to determine which types of measures should be included in the alternative plans. Based upon that screening, it was concluded that new bulkheads should not be construct, except where shoreline uses such as docking facilities, require such a structure. Bulkheads are generally difficult and costly to maintain; often reflect wave energy to cause scouring of the lake bed; and generally do not provide an attractive, natural appearance to the shoreline. All of the alternative plans, however, recommend the continued maintenance of some existing bulkheads. The alternative shoreline protection plans thus 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 fill material--including possibly debris from the Milwaukee Metropolitan Sewerage District deep tunnel construction project. The maintenance, reconstruction,or demolition of existing shore protection structures are also addressed in the alternative plans.

A variety of shore protection materials and products are commercially available, and some of these systems were described above. 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 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 sized bed material, or where a structure is constructed directly on bedrock.[2] 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.[3] 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 then polyvinylidene chloride cloth. Polypropylene cloth should be used for construction above the water surface. Filter cloth with small pore sizes should not be used. This grade of fabric is almost impermeable to hydraulic transients, and the wave energy can cause considerable uplift pressures.[4] 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.

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 IV-2, would include the construction or reconstruction of quarry stone revetments for about 15.8 miles of shoreline, or about 52 percent of the total county shoreline. The size and associated cost of a revetment required to provide

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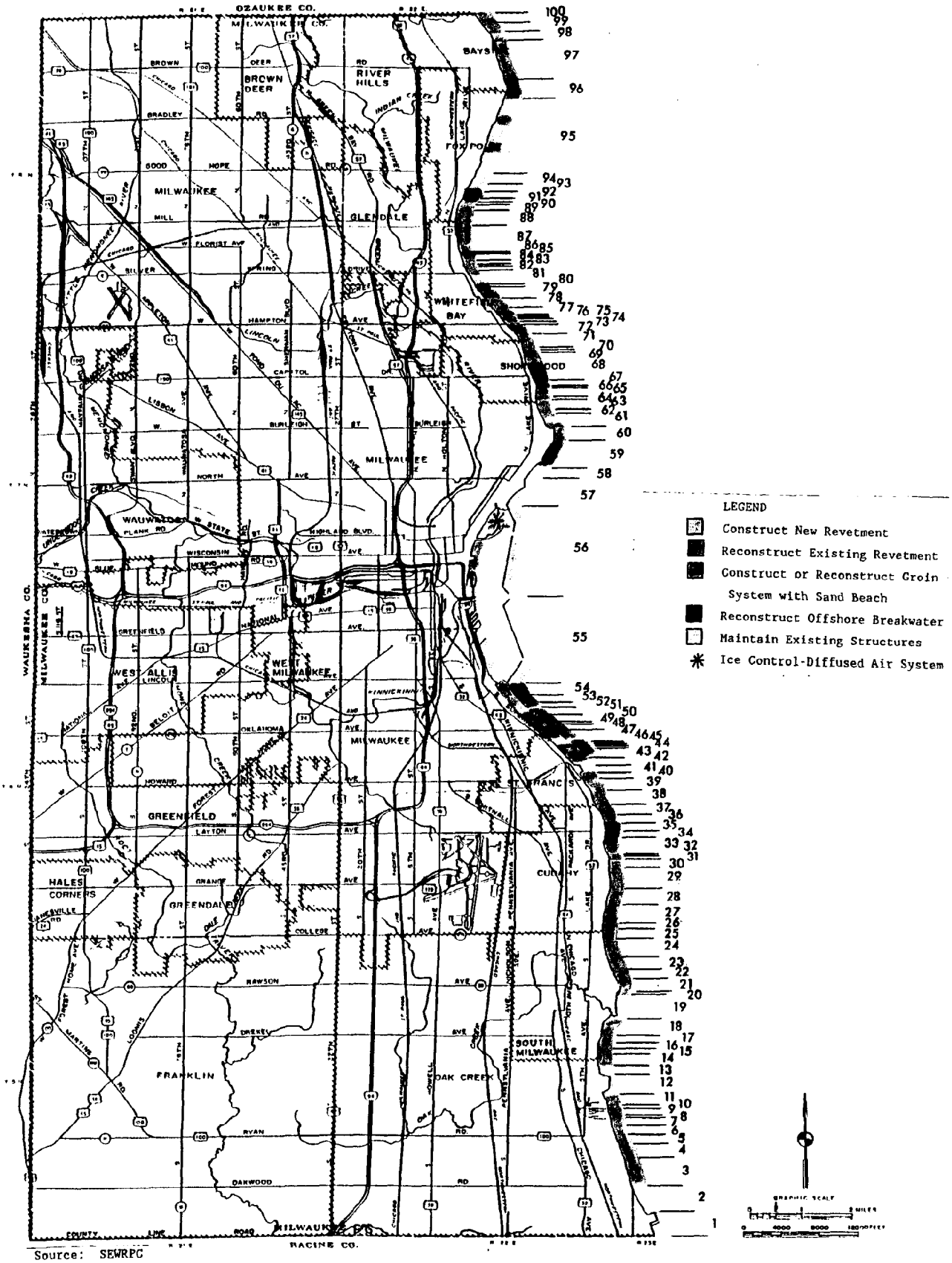
[2]U.S. Army, Corps of Engineers, Shore Protection Manual, Volume II, Coastal Engineering Research Center, 1984.

[3]U.S. Army, Corps of Engineers, Low Cost Shore Protection, Final Report on the Shoreline Erosion Control Demonstration (Section 54) Program, 830 pp., 1981.

[4]Charles Johnson, Coastal Engineer, U.S. Army, Corps of Engineers, Chicago, Illinois, Personal Communication, July 27, 1987.

Map IV-2

REVETMENT ALTERNATIVE PLAN FOR MILWAUKEE COUNTY



Source: SEWRPC

adequate protection for a particular bluff analysis section is dependent upon the degree of toe erosion occurring, the existing beach width and nearshore slope, the anticipated wave heights during storms, and the location and value of the facility or building being protected. For systems level cost purposes, it was assumed that new construction of revetments would typically require about six to nine tons of stone per lineal foot of shoreline, and cost about $250 per foot of shoreline. Lessor 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 IV-7. A new revetment would be constructed along about 68,000 feet of shoreline, or 43 percent of the total county shoreline of 159,100 feet. Existing revetments would be reconstructed along about 15,400 feet of shoreline, or 10 percent of the total county shoreline. Sand beaches contained by groins would be constructed or reconstructed along about 4,300 feet of shoreline, or 3 percent of the total. Existing shore protection measures--including the Milwaukee Harbor breakwater, would be maintained and repaired as needed along about 58,400 feet of shoreline, or 37 percent of the county shoreline. The South Shore breakwater would be reconstructed to its original elevation to protect about 10,200 feet of shoreline, or 7 percent of the total. An ice control system, utilizing a diffused compressed air system, as recommended in SEWRPC Planning Report No. 37,[5] would be installed in the McKinley Marina. About 13,000 feet of shoreline, or 8 percent, was not eroding in 1987 and would not require toe protection under this alternative. The revetment alternative plan would have an estimated total capital cost of about $35.0 million, and an annual maintenance cost of about $3.9 million.

The major advantages of the revetment alternative plan are its relatively low cost, ease of construction and maintenance, and implementability. The proposed shore protection measures would represent an essential continuation of the

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[5]SEWRPC Planning Report No. 37, A Water Resources Management Plan for the Milwaukee Harbor Estuary, Volume Two, Alternative and Recommended Plans, December 1987.

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Table IV-7

SELECTION CRITERIA AND TYPICAL CAPITAL AND MAINTENANCE UNIT COST OF REVETMENT ALTERNATIVE PLAN COMPONENTS

Plan Component	Criteria for Selection	Estimated Unit Cost ($/lineal foot shoreline)	
		Total Capital	Annual Maintenance
Reconstruction of an Existing Revetment	Existing revetments which, as of 1987, required a substantial amount of repair.	Light-100 Medium-200 Heavy-300	10 15 20
Construction of a New Structure Other Than a Revetment	Strong community desire for a particular type of shore protection structure other than a revetment.	Variable Depending Upon Type of Structure	Variable Depending Upon Type of Structure
Continued Maintenance of Existing Structure	Structure which was protecting against erosion in 1987 and which, if maintained, could provide continued effective protection.	Variable Depending Upon Type of Structure	Variable Depending Upon Type of Structure
No Shoreline Protection	No significant shoreline or bluff toe erosion observed in 1987 and none expected to occur.	0	0

Source: SEWRPC.

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 or sheet piling--and do not feed the littoral transport system. Over time, it may be expected that the nearshore slopes of areas with erodible offshore sand deposits would become somewhat steeper, which would increase the wave height capable of reaching 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 not likely significantly steepen the offshore slopes.

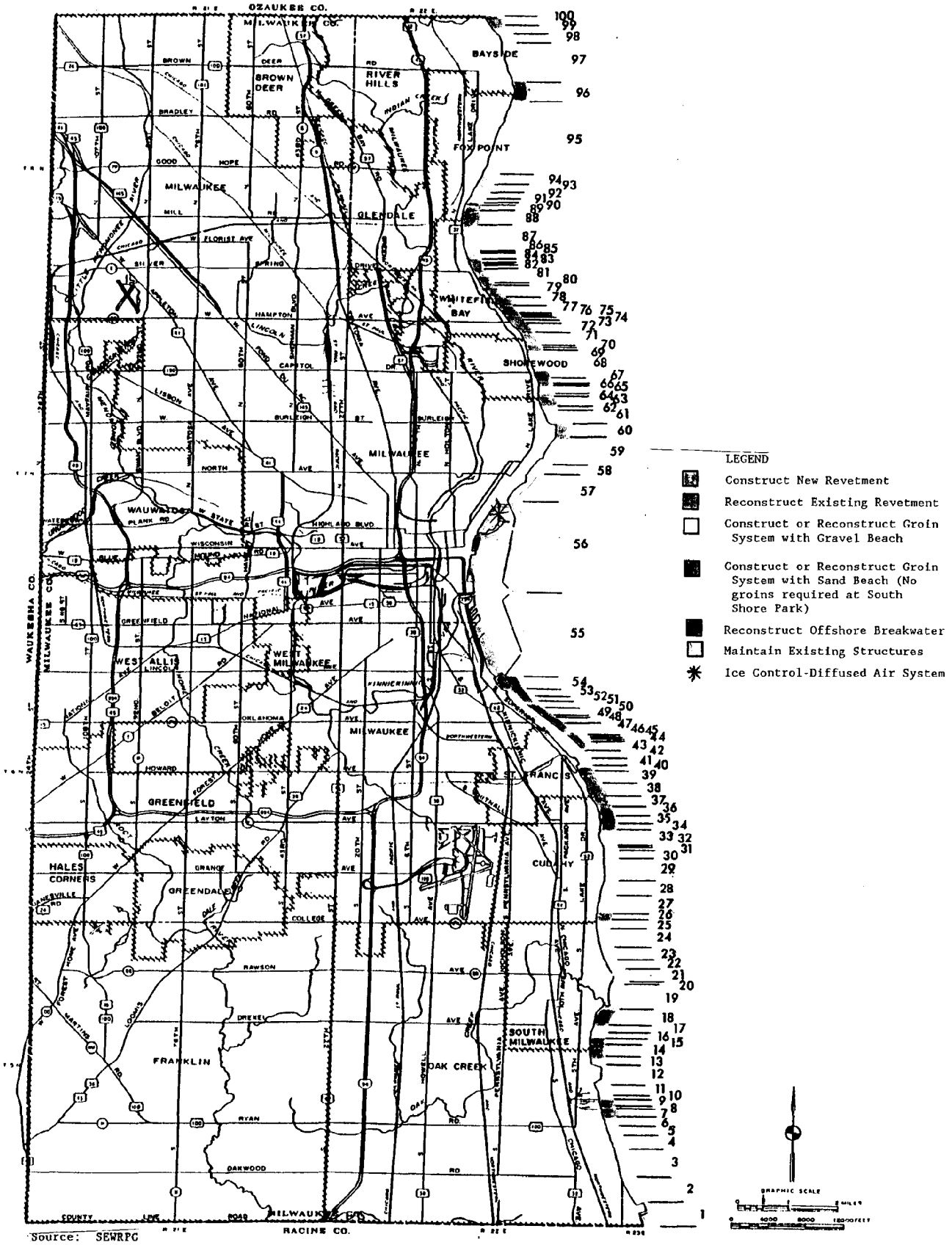
Beach Alternative Plan: The beach alternative plan would include the construction or reconstruction of about 16.3 miles of usable beach composed of sand or gravel. The beach alternative plan is shown on Map IV-3.

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 IV-8. Nourished gravel beaches contained by short groins would be created along about 64,500 feet, or 41 percent of the study area shoreline. These beaches could also be contained by armored headlands or nearshore reefs constructed of quarry stone. New or reconstructed revetments would lie along 27,000 feet of shoreline, or 17 percent of the county total. New or reconstructed sand beaches would cover 3,000 feet of shoreline, or 2 percent. About 51,500 feet of existing structures would be maintained, the South Shore breakwater would be reconstructed, and the ice control system discussed under the revetment alternative plan would be installed in the McKinley Marina. No toe protection would be required along 13,000 feet of shoreline, or 8 percent of the county shoreline.

In general, this alternative plan would not envision gravel beaches along existing or proposed fill projects. Although in most cases a sand or gravel

Map IV-3

BEACH ALTERNATIVE PLAN FOR MILWAUKEE COUNTY



Source: SEWRPC

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Table IV-8

SELECTION CRITERIA AND TYPICAL CAPITAL AND MAINTENANCE UNIT COST OF BEACH ALTERNATIVE PLAN COMPONENTS

Plan Component	Criteria for Selection	Estimated Unit Cost ($/lineal foot shoreline)	
		Total Capital	Annual Maintenance
Construction of a Nourished Gravel Beach System With Short Groins	1. Shoreline or bluff toe erosion observed in 1987. 2. Bluff slope regrading has not previously been conducted, and is not required.	$ 400	$30
Construction or Reconstruction of a Sand Beach	1. Strong community support for a large sand beach.	1,000	50
New or Reconstructed Revetment	1. Bluff slope regrading requiring filling has previously been conducted or is required to stabilize the bluff slope. 2. Strong community support for a revetment.	200-250	15
No Shoreline Protection	1. No significant shoreline or bluff toe erosion boserved in 1986 and none expected to occur.	0	0

Source: SEWRPC.

beach system 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 the fill sites for two major reasons. First, the lakebed bathymetry offshore of the fill projects tends to be steeper than in other portions of the county shoreline, and most of the bluff slopes which are filled or proposed to be filled face due east or northeasterly. Hence, most fill areas will be subjected to the largest storm waves attacking the 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 towards, or into, the lake, the additional construction of a nourished beach, and the attendant containment structures, from a fill site would often extend too far 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. The beach alternative plan would have an estimated total capital cost of about $46.0 million, and an annual maintenance cost of about $4.9 million.

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.

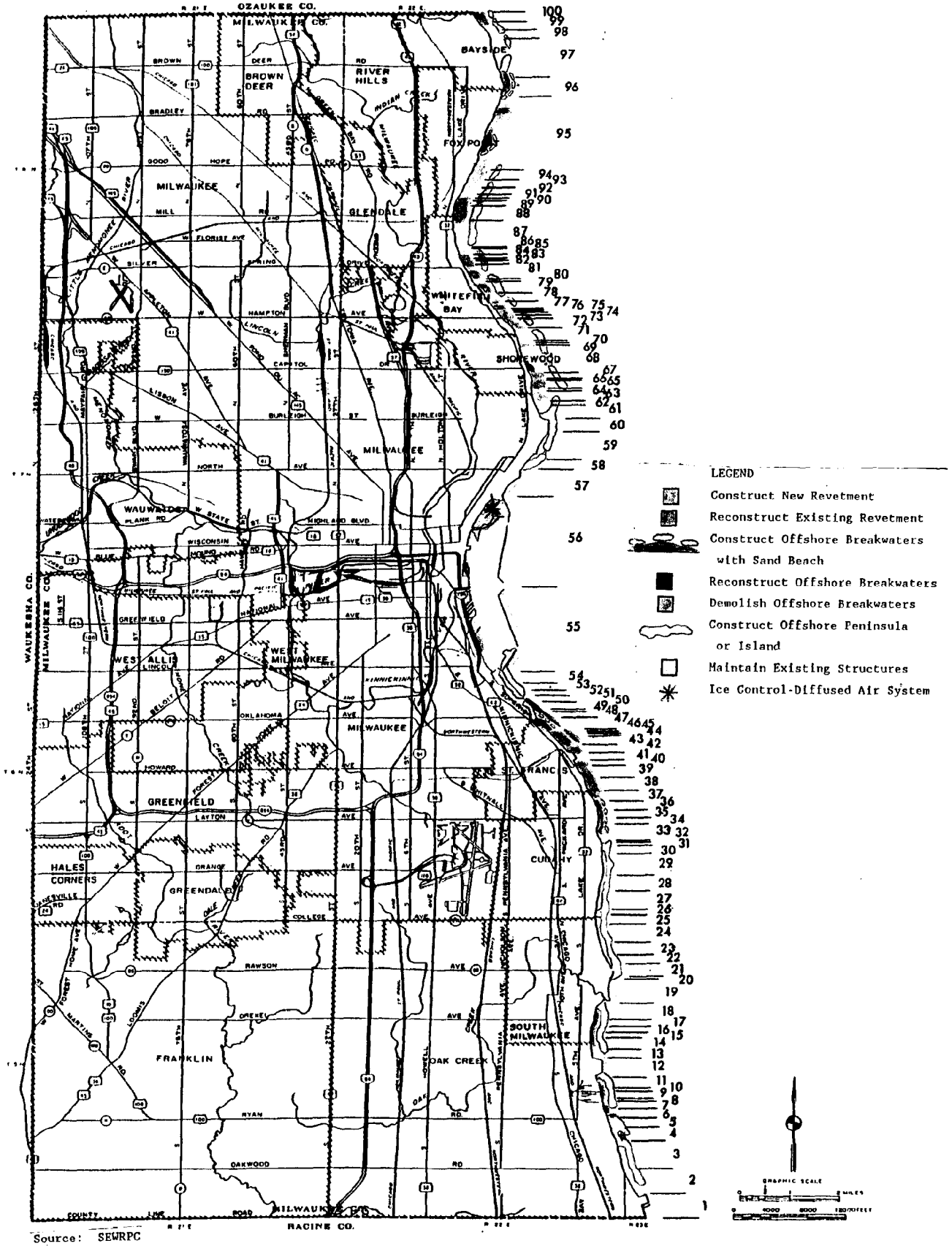
A disadvantage of the beach alternative plan is the increased maintenance and periodic beach nourishment required. To successfully implement the plan, all property owners within the specified beach sections would have to participate in both the construction and maintenance of the beach systems--the systems could not be implemented in a piecemeal manner. The gravel beaches would likely increase the use of the shoreline by the general public, 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 18.6 miles of shoreline, or 62 percent of the total county shoreline. The islands and peninsulas, likely composed of construction debris, or perhaps debris from the Milwaukee Metropolitan Sewerage District deep tunnel project, 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 some would lie in about 30 feet of water. The publicly-owned islands and peninsulas would be utilized for passive recreational uses in most areas, although offshore facilities at Lake Park, South Shore Park, Bay View Park, and Bender Park could perhaps be used for intensive recreational uses. Offshore breakwaters with sand beaches would be constructed at the Village of Shorewood's 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. In addition to these offshore structures, relatively small revetments would be constructed or reconstructed at the existing shoreline in those areas where the offshore structures alone would not be expected to provide sufficient protection against wave action. Existing structures would be maintained along the shoreline not protected by offshore structures, and in a few areas where construction of a revetment would not be feasible. The offshore alternative plan is illustrated on Map IV-4.

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 IV-9. Offshore islands and peninsulas would be created along about 87,590 feet, or 55 percent of the county shoreline. Offshore breakwaters would be constructed along about 10,700 feet, or 7 percent of the county shoreline. Existing shoreline protection structures would be maintained along 58,600 feet of shoreline, or 37 percent of the study area shoreline. To supplement the offshore structures, new revetments would be constructed or reconstructed along 24,800 feet, or 16 percent of the county shoreline. A portion of the South Shore breakwater would be reconstructed and a portion would be demolished under this alternative. An ice control system would be installed in the McKinley Marina, as discussed for the revetment alternative plan. The offshore

Map IV-4

OFFSHORE ALTERNATIVE PLAN FOR MILWAUKEE COUNTY



Source: SEWRPC

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Table IV-9

SELECTION CRITERIA AND TYPICAL CAPITAL AND MAINTENANCE UNIT COST OF OFFSHORE ALTERNATIVE PLAN COMPONENTS

Plan Component	Criteria for Selection	Estimated Unit Cost ($/lineal foot shoreline)	
		Total Capital	Annual Maintenance
Island or Peninsula	1. Entire shoreline, except: a. where breakwaters are proposed to maintain a sand beach; or b. where an unobstructed view of the horizon from a beach or low terrace is desired by property owners. c. Where other existing or proposed structures make use of offshore islands or peninsulas impracticable.	$ 1,000	$20
Breakwater System With Sand Beach	1. Existing public sand or fine gravel beaches. 2. Community spport for a public sand beach. 3. Desire to provide additional public access and usable beach to public shoreline areas.	1,500	50
Construction of a New Revetment	1. Shoreline areas which require substantial bluff slope regrading, and fill projects under construction in 1987. 2. Areas exhibiting moderate or severe shoreline or bluff tow erosion in 1987 and which would require additional protection beyond that provided by the offshore structures.	150	10
Reconstruction of an Existing Revetment	1. 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	10
Continued Maintenance of Existing Structure	1. Structure which was protecting against shoreline erosion in 1987 and which should be maintained to provide continued shore protection in combination with the offshore structures.	Variable Depending Upon Type of Structure	Variable Depending Upon Type of Structure

Source: SEWRPC.

alternative plan would have an estimated total capital cost of about $113.8 million, and an annual maintenance cost of about $4.8 million.

The major advantages of the offshore alternative plan would be the creation of approximately 200 acres of new public lakeshore parkland; the provision of protected surface water; the creation of over 30 miles of new shoreline, the expansion of large public sand beaches; and the provision of new wildlife and fishery habitat. The plan would minimize the disruption associated with protecting the existing 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 construction techniques. The concept of an offshore plan offers a potential opportunity to utilize public funds to create new public parkland while helping to protect both public and private property.

A disadvantage of the offshore alternative plan, in addition to its high cost, is the need for over 6.4 million cubic yards of fill material for construction of the islands. The plan probably could not be implemented by groups of private property owners; so that implementation would have 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 existing shoreline areas would continue to be limited.

RECOMMENDED SHORELINE EROSION, BLUFF RECESSION, AND STORM DAMAGE CONTROL PLAN

Based upon careful consideration of the alternatives, the Advisory Committee for the Milwaukee County Lake Michigan Shoreline Erosion Management Study selected a recommended shoreline erosion, bluff recession, and storm damage control plan for Milwaukee County. As already noted, the recommended plan consists of a bluff stabilization element, and a shoreline protection element. The plan thus represents an attempt both to fully stabilize the bluff slopes, and to protect the immediate shoreline from wave and ice erosion on a long-term basis. Based upon the findings of the inventories and analyses conducted under the study, the plan identifies those shore protection measures which, on a section-by-section basis, 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 practicable--a usable shoreline to be enjoyed by the general public as well as by property owners.

It is important to note that 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 future 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 would not have long-term harmful effects on the overall coastal environment--including the offshore bathymetry, sediments, and ecosystem.

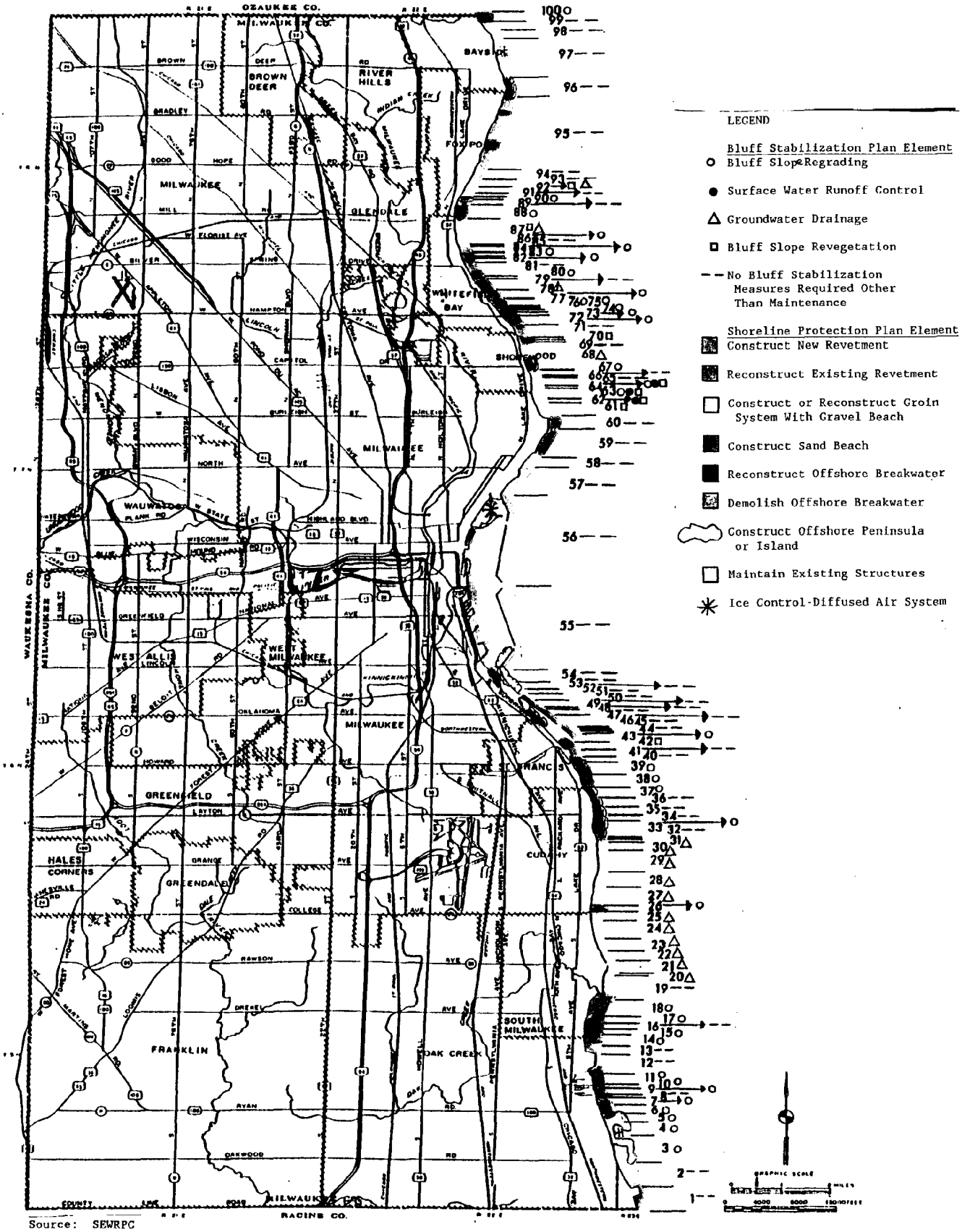
After careful consideration of the advantages, disadvantages, and costs of the alternative plans described above, the Advisory Committee selected a recommended plan comprised of the best components of each of the alternative plans considered. The recommended plan is illustrated on Map IV-5.

The recommended plan envisions that the bluff slopes would be stabilized by regrading the slopes, revegetating the slopes, and constructing groundwater and surface water drainage systems. However, the recommended bluff stabilization measures have a lower cost than as previously described under the bluff stabilization plan element because it is recommended that some bluff slopes be left to regrade back to a stable slope naturally. The bluff stabilization measures would entail a capital cost of about $5.1 million and an annual maintenance cost of about $629,000, although 67 percent of the maintenance cost would be required for only a three-year period after construction.

The plan recommends that about 14,900 feet of offshore islands and peninsulas be constructed, creating about 60 acres of new lakefront public park land. These offshore islands and peninsulas would protect a total of about 9 percent

Map IV-5

RECOMMENDED SHORELINE EROSION, BLUFF RECESSION, AND STORM DAMAGE CONTROL PLAN FOR MILWAUKEE COUNTY



Source: SEWRPC

of the county shoreline. The recommended peninsulas would entail a capital cost of about $14.9 million and an annual maintenance cost of about $298,000.

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, Bay View Park, and Sheridan Park. New sand beaches would also be constructed at South Shore Park and at Bender Park. About two miles of public sand beach, covering nearly 40 acres, would substantially increase recreational opportunities for swimming and sunbathing within the County. The recommended sand beach systems would entail a capital cost of about $15.9 million and an annual maintenance cost of about $534,000.

The recommended plan envisioned that nourished gravel beach systems contained by short groins be located along about 32,600 feet of shoreline, or 20 percent of the county shoreline. The would entail a capital cost of about $13.0 million and an annual maintenance cost of about $977,000.

The recommended plan proposes that about 37,500 feet of quarry stone revetments be constructed or reconstructed, most to protect existing or proposed bluff slope fill projects, or to provide additional protection to supplement offshore islands or peninsulas. As discussed for the beach alternative, beaches are not recommended for the fill projects because the beaches would be subject to high wave energy, which would make the beaches difficult and costly to maintain, and because the beaches would have to extend too far out into the lake, harming downdrift shoreline areas. The revetments would protect approximately 24 percent of the county shoreline. The revetments would entail a capital cost of about $8.8 million, an annual maintenance cost of about $563,000.

Shore protection structures along abut 56,300 feet of shoreline, or 35 percent of the county total, would be maintained under the recommended plan. These structures include the Milwaukee Harbor breakwater and nearly the entire shoreline within the breakwater. The only new structure with the breakwater would be revetment constructed in front of the War Memorial Center's bulkhead. Maintenance of existing structures would entail an annual cost of about $1.9 million. A new ice control system would be installed in the McKinley Marina

at a capital cost of about $300,000 and an annual operation and maintenance cost of about $11,000.

Under the recommended plan, about 7,000 feet of the South Shore breakwater would be demolished, and the rock used to help construct other shore protection structures. The remaining 3,700 feet of the South Shore breakwater would be reconstructed and incorporated into the design of an offshore peninsula and an island. Work on the South Shore breakwater would entail a net capital cost of about $3.7 million, and an annual maintenance cost of about $170,000.

The total capital cost of the recommended shoreline erosion, bluff recession, and storm damage control plan is approximately $61.7 million, and the annual maintenance cost is about $5.1 million. Of the total plan cost, about 35 percent would be financed by the private sector and 65 percent would be financed by the public sector.

The recommended plan costs are best estimates at the system planning level. Depending on site specific characteristics, individual projects may cost substantially more or less than indicated herein. Where new structures are recommended, it was assumed that some of the material--primarily quarry stone--currently protecting the shoreline would be reused. It was also assumed that as the recommended structures are constructed over time, the design costs will eventually decrease as engineers and contractors become 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.

The preliminary recommended plan has four major features:

1. The plan identifies those measures needed to fully stabilize the bluff slopes, which will require bluff slope regrading in some areas;

2. The plan envisions new public facilities, including about 60 acres of lakefront parkland and about 40 acres of sand beaches, which will increase the opportunity for enjoyment of the lakeshore by the general public;

3. The plan recommends the creation of over six miles of gravel beaches which would greatly increase the usability of the immediate shoreline and access to the water, for both private property owners and the general public; and,

4. The plan proposes that revetments be constructed to provide effective toe protection at the base of all existing or new bluff fill projects.

In addition to these specific plan recommendations, it was 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 important, 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. With regard to proposals for new urban development or redevelopment near the shoreline, it is recommended that the local units of government consider the structural and nonstructural setback distances described in Figures IV-24 and IV-25 as advisory in the administration of their zoning and subdivision control ordinances.

The successful implementation of the plan will require substantial expenditures--and a commitment to conducting proper site specific geotechnical and coastal engineering analyses and to carrying out long-term maintenance programs by those responsible for implementing the plan. As a systems level plan, this report provides guidance for plan implementation, and serves as a starting point for the necessary site specific analyses. Adoption and implementation of this recommended plan would 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 effects of those alternative measures as the basis for the selection of a recommended shoreline erosion, bluff recession, and storm damage control plan for Milwaukee County. 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 procedures 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 nearshore beach systems, and six types of offshore structures. The capital costs of these structures were estimated to ranged from $150 to $2,000 per lineal foot of shoreline, with annual maintenance costs ranging from $10 to $100 per lineal foot. Bluff slope stabilization could be accomplished by cutting back, filling, cutting and filling, or by 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 $10 to $15 per lineal foot, for the first three years after construction. Groundwater drainage could be provided at a capital cost of from $20 to $75 per lineal foot of shoreline, with an average annual maintenance cost of about $5 to $20 per lineal foot. Surface water drainage control could be provided at a capital cost of approximately $10 to $70 per lineal foot, with annual maintenance costs of up to $5

per lineal foot. Revegetating the bluff slope could be accomplished at a capital cost of from $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. The procedures developed for delineating both nonstructural and structural setback distances for new buildings and facilities were also presented.

Alternative shore protection plans were presented for the entire Milwaukee County shoreline. The shoreline erosion, bluff recession, and storm damage control 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 bluff stabilization element is estimated at about $6.7 million, and the average annual maintenance cost at about $748,000. 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 about $35.0 million, and an annual maintenance cost of about $3.5 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 about $46.0 million, and an average annual maintenance cost of about $4.9 million. The third alternative plan for shoreline protection 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 about $113.8 million, and an average annual maintenance cost of about $4.9 million.

The recommended shoreline erosion, bluff recession, and storm damage control plan for Milwaukee County seeks to identify those shore protection measures which, on a section-by-section basis, would effectively abate the 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 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; and carefully selected components of three alternative shoreline protection plans.

The recommended shoreline erosion management plan envisions that the bluff slopes be stabilized by regrading and revegetating the bluff slopes, and by installing groundwater and surface water drainage systems, where needed. It is recommended that some bluffs be left to regrade back naturally to a stable slope angle. To protect the immediate shoreline from wave and ice action, quarry stone revetments, gravel beaches contained by short groins, sand beaches contained by offshore breakwaters, and offshore peninsulas and islands are recommended. It is also recommended that existing structures be maintained along 35 percent of the County shoreline.

The total capital cost of the recommended shoreline erosion, bluff recession, and storm damage control plan is about $61.7 million, and the annual maintenance cost is about $5.1 million. Of the total plan cost, about 35 percent would be financed by the private sector and 65 percent would be financed by the public sector.

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