COMMUNITY ASSISTANCE
PLANNING REPORT NO. 155

A LAKE MICHIGAN SHORELINE EROSION MANAGEMENT PLAN FOR NORTHERN MILWAUKEE COUNTY WISCONSIN

SOUTHEASTERN WISCONSIN REGIONAL PLANNING COMMISSION

COMMUNITY ASSISTANCE PLANNING REPORT
NUMBER 155

A LAKE MICHIGAN SHORELINE EROSION MANAGEMENT PLAN FOR NORTHERN MILWAUKEE COUNTY WISCONSIN

Prepared by the

Southeastern Wisconsin Regional Planning Commission
P. O. Box 1607
Old Courthouse
916 N. East Avenue
Waukesha, Wisconsin 53187-1607

In Cooperation with the
WISCONSIN COASTAL
MANAGEMENT PROGRAM

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

December 1988

Inside Region $5.00
Outside Region $10.00

(This page intentionally left blank)

SOUTHEASTERN WISCONSIN REGIONAL PLANNING COMMISSION

916 N. EAST AVENUE • P.O. BOX 1607 • WAUKESHA, WISCONSIN 53187-1607 • TELEPHONE (414) 547-6721

Serving the Counties of: KENOSHA, MILWAUKEE, OZAUKEE, RACINE, WALWORTH, WASHINGTON, WAUKESHA

December 30, 1988

Mr. John O. Norquist, Mayor
City of Milwaukee
City Hall
200 E. Wells Street
Milwaukee, Wisconsin 53202

Mr. F. R. Dengel, President
Village of Fox Point
Village Hall
7200 N. Santa Monica Boulevard
Fox Point, Wisconsin 53217

Mr. F. Patrick Matthews, President
Village of Whitefish Bay
Village Hall
5300 N. Marlborough Drive
Whitefish Bay, Wisconsin 53217

Mr. Michael J. Spector, President
Village of Shorewood
Village Hall
3930 N. Murray Avenue
Shorewood, Wisconsin 53211

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

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

Gentlemen:

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

Seeking to improve upon this approach to shore protection, representatives of local communities in northern Milwaukee County, in January 1986, entered into a contract with the Regional Planning Commission for the preparation of a comprehensive plan for shoreline erosion control. The study was funded in part by Milwaukee County, the City of Milwaukee, and the Villages of Fox Point, Shorewood, and Whitefish Bay; and in part by the Wisconsin Coastal Management Program. Assisting the Commission in the study were consultants from the University of Wisconsin; Warzyn Engineering, Inc., Milwaukee; W. F. Baird & Associates, Ltd., Ottawa, Canada; Johnson, Johnson & Roy, Inc., Ann Arbor, Michigan; and PTL-Inspectorate, Inc., New Berlin. The study was carried out under the guidance of an Advisory Committee composed of representatives of the affected local communities, Milwaukee County, the Wisconsin Department of Natural Resources, the University of Wisconsin Sea Grant Institute, the University of Wisconsin, and concerned and knowledgeable citizens.

For the 7.3-mile shoreline study area extending from the City of Milwaukee Linnwood Avenue water treatment plant northward to Milwaukee County Doctors Park in the Village of Fox Point, the study provides information useful to local governmental agencies and private property owners on existing shoreline conditions, and guidance on which measures can best protect against wave and ice action and stabilize the bluff slopes on a long-term basis. The study identifies those measures that are needed and are economically feasible, those measures which would not have a significant adverse impact either on adjacent shoreline areas or on the offshore coastal environment, and those measures which would, where practical, provide a shoreline usable for recreational activities. Alternative shoreline erosion control measures are evaluated, and a recommended plan is presented. An implementation program is also recommended to carry out the plan.

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

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

Sincerely,

Kurt W. Bauer
Executive Director

(This page intentionally left blank)

TABLE OF CONTENTS

	Page
Chapter I—INTRODUCTION	1
Background	1
Definition of Shoreline Erosion Management	1
Need for a Shoreline Erosion Management Study	1
Review of Previous Studies	2
Shoreline Erosion Management Study Area	4
Purpose and Scope	6
Summary	6
Chapter II—INVENTORY FINDINGS	9
Introduction	9
Natural Resource Base	9
Bedrock Geology and Glacial Deposits	9
Bluff Characteristics	12
Beach Characteristics	17
Near-shore Bathymetry	18
Groundwater Resources	19
Climate	27
Man-Made Features	29
Civil Divisions	29
Existing Land Use	29
Existing Zoning	29
Coastal Erosion Processes	31
Bluff Erosion	31
Types of Slope Failure	34
Wave Action	35
Lake Michigan Water Level	37
Anticipated Future Lake Levels	42
Ice Formation	43
Groundwater Seepage	43
Vegetative Cover	43
Beach Erosion	43
Existing Regulations Pertaining to Shoreland Development	44
Existing Structural Erosion Control Measures	48
Existing Coastal Erosion Problems	50
Bluff Analysis Sections	50
Bluff Recession Rates	74
Summary	86
Chapter III—EVALUATION OF COASTAL EROSION PROBLEMS AND DAMAGES	89
Introduction	89
Evaluation of Coastal Erosion Problems	89
Definition of Safety Factor	90
Methods of Analysis	90
Bluff Toe Erosion	90
Bluff Slope Instability by Rotational Sliding	92
Distinction Between Deterministic and Probabilistic Slope Stability Analyses	92
Deterministic Slope Stability Analysis	93
Probabilistic Slope Stability Analysis	98
Interpretation of Rotational Slope Stability Analysis Results	100
Bluff Slope Instability by Translational Sliding	101
Results	105
Bluff Analysis Section 1	105
Bluff Analysis Section 2	106
Bluff Analysis Section 3	107
Bluff Analysis Section 4	108
Bluff Analysis Section 5	109
Bluff Analysis Section 6	109
Bluff Analysis Section 7	110
Bluff Analysis Section 8	111
Bluff Analysis Section 9	112
Bluff Analysis Section 10	112
Bluff Analysis Section 11	113
Bluff Analysis Section 12	115
Bluff Analysis Section 13	116
Bluff Analysis Section 14	117
Bluff Analysis Section 15	117
Bluff Analysis Section 16	118
Bluff Analysis Section 17	119
Bluff Analysis Section 18	119
Bluff Analysis Section 19	120
Bluff Analysis Section 20	121
Bluff Analysis Section 21	122
Bluff Analysis Section 22	123
Bluff Analysis Section 23	124
Bluff Analysis Section 24	124
Bluff Analysis Section 25	125
Bluff Analysis Section 26	126
Bluff Analysis Section 27	127
Bluff Analysis Section 28	128
Bluff Analysis Section 29	129

	Page
Bluff Analysis Section 30	130
Bluff Analysis Section 31	130
Bluff Analysis Section 32	131
Bluff Analysis Section 33	132
Bluff Analysis Section 34	133
Bluff Analysis Section 35	134
Analysis Subsection 35A	134
Analysis Subsection 35B	134
Analysis Subsection 35C	134
Analysis Subsection 35D	136
Analysis Subsection 35E	136
Recommendations	136
Bluff Analysis Section 36	136
Summary of the Evaluation of Bluff Analysis Sections	138
Evaluation of Potential Coastal Erosion Damages	142
Potential Property Loss	142
Potential Economic Loss	147
Summary	151
Chapter IV—ALTERNATIVE SHORELINE EROSION CONTROL MEASURES AND A RECOMMENDED SHORELINE EROSION MANAGEMENT PLAN	153
Introduction	153
Plan Design and Analysis	153
Planning Process	153
Analytical Procedures and Design Criteria	154
Conceptual Shore Protection Measures	159
Bluff Toe Protection	160
Revetment	160
Rip-rap	164
Grout-Filled Bags	165
Concrete Structures	165
Bulkhead	167
Concrete Cantilevered Bulkhead	169
Steel Sheet Piling Bulkhead	169
Concrete-Stepped Bulkhead	169
Onshore or Near-shore Beach Systems	169
Groins	171
Armored Headland-Pocket Beach System	173
Near-shore Reefs	173
Perched Cobble Beach System	174
Near-shore Pervious Concrete Sill	178
Manufactured Concrete Beach Containment Systems	180
Offshore Breakwater	181
Rubblemound Breakwater	181
Caisson Breakwater	181
Sheet Pile Breakwater	182
Timber Crib Breakwater	183
Floating Breakwater	184
Offshore Islands and Peninsulas	184
Bluff Slope Stabilization	184
Bluff Slope Regrading	184
Cutback Method	186
Fill Method	186
Cut and Fill Method	187
Terracing Method	187
Groundwater Drainage	187
Horizontal Drains	188
Vertical Drains	188
Trench Drains	189
Surface Water Drainage	189
Revegetation	190
Seeding	190
Transplanting	191
Setback Requirements for New Urban Development	191
Nonstructural Setback Distance	191
Structural Setback Distance	191
Regulation of Lake Michigan Water Levels	193
Alternative Shoreline Erosion Management Plans	195
Bluff Slope Stabilization Plan Element	195
Bluff Toe Protection Plan Element	197
Revetment Alternative Plan	200
Beach Alternative Plan	202
Offshore Alternative Plan	207
Preliminary Shoreline Erosion Management Plan	209
Recommended Shoreline Erosion Management Plan	219
Plan Implementation	221
Maintain Existing Institutional Structure—County-Based Coordination	227
Modified Existing Institutional Structure—Local-Based Coordination	228

	Page
Create New Institutional Structure	229
Review of Implementation Approaches	230
Summary	232
Chapter V—SUMMARY	235
Introduction	235
Purpose and Scope of Study	235
Inventory Findings	235
Evaluation of Coastal Erosion Problems and Damages	237
Alternative Shoreline Erosion Management Measures	237
Preliminary Shoreline Erosion Management Plan	238
Recommended Shoreline Erosion Management Plan	239
Plan Implementation	239
Public Reaction to the Recommended Plan and Subsequent Action of the Advisory Committee	240
Installation of Additional Shoreline Protection Measures Within Those Areas That Do Not Need Protection, or Are Already Protected	242
Opposition to Nourished Gravel Beaches Within the Village of Fox Point	242
Implementation Program	243
Concluding Remarks	245

LIST OF APPENDICES

Appendix			Page
A	Bathymetric Surveys of Lake Michigan by Warzyn Engineering, Inc.: August 1986		254
	Map A-1	Locations of Bathymetric Surveys of Lake Michigan Along the Northern Milwaukee County Shoreline	254
	Figure A-1	Bathymetric Survey, Location 1	255
	Figure A-2	Bathymetric Survey, Location 2	256
	Figure A-3	Bathymetric Survey, Location 3	257
	Figure A-4	Bathymetric Survey, Location 4	258
	Figure A-5	Bathymetric Survey, Location 5	259
	Figure A-6	Bathymetric Survey, Location 6	260
	Figure A-7	Bathymetric Survey, Location 7	261
	Figure A-8	Bathymetric Survey, Location 8	262
	Figure A-9	Bathymetric Survey, Location 9	263
	Figure A-10	Bathymetric Survey, Location 10	264
	Figure A-11	Bathymetric Survey, Location 11	265
	Figure A-12	Bathymetric Survey, Location 12	266
	Figure A-13	Bathymetric Survey, Location 13	267
	Figure A-14	Bathymetric Survey, Location 14	268
	Figure A-15	Bathymetric Survey, Location 15	269

Appendix			Page
B	Inventory of Shore Protection Structures in Northern Milwaukee County: 1986		270
C	Aerial Photographs of the Lake Michigan Shoreline of Northern Milwaukee County: May 1986		272
	Figure C-1	Linnwood Avenue Water Treatment Plant to 3224 E. Hampshire Street City of Milwaukee	272
	Figure C-2	3224 E. Hampshire Street-3228 N. Lake Drive City of Milwaukee	272
	Figure C-3	3252 N. Lake Drive-3052 E. Newport Court City of Milwaukee	273
	Figure C-4	3061 E. Newport Court-3444 N. Lake Drive City of Milwaukee	273
	Figure C-5	3432-3474 N. Lake Drive City of Milwaukee	274
	Figure C-6	3474-3562 N. Lake Drive Village of Shorewood	274
	Figure C-7	3534-3600 N. Lake Drive Village of Shorewood	275
	Figure C-8	3580-3816 N. Lake Drive Village of Shorewood	275
	Figure C-9	3704-3914 N. Lake Drive Village of Shorewood	276
	Figure C-10	3816-3960 N. Lake Drive Village of Shorewood	276
	Figure C-11	3932 N. Lake Drive to Atwater Park Village of Shorewood	277
	Figure C-12	Atwater Park Village of Shorewood	277
	Figure C-13	4060-4136 N. Lake Drive Village of Shorewood	278
	Figure C-14	4108-4216 N. Lake Drive Village of Shorewood	278
	Figure C-15	4162-4400 N. Lake Drive Village of Shorewood	279
	Figure C-16	4240-4442 N. Lake Drive Village of Shorewood	279

Appendix Page

Figure C-17 4408-4480 N. Lake Drive
Village of Shorewood 280

Figure C-18 4460-4614 N. Lake Drive
Village of Shorewood 280

Figure C-19 4470-4614 N. Lake Drive
Village of Whitefish Bay 281

Figure C-20 4500-4620 N. Lake Drive
Village of Whitefish Bay 281

Figure C-21 4514-4626 N. Lake Drive
Village of Whitefish Bay 282

Figure C-22 4600-4652 N. Lake Drive
Village of Whitefish Bay 282

Figure C-23 4632-4720 N. Lake Drive
Village of Whitefish Bay 283

Figure C-24 4686-4794 N. Lake Drive
Village of Whitefish Bay 283

Figure C-25 4780-4850 N. Lake Drive
Village of Whitefish Bay 284

Figure C-26 4820-4930 N. Lake Drive
Village of Whitefish Bay 284

Figure C-27 4910 N. Lake Drive to Buckley Park
Village of Whitefish Bay 285

Figure C-28 Big Bay Park
Village of Whitefish Bay 285

Figure C-29 Big Bay Park
Village of Whitefish Bay 286

Figure C-30 Big Bay Park to 5220 N. Lake Drive
Village of Whitefish Bay 286

Figure C-31 1400 E. Henry Clay Street-5312 N. Lake Drive
Village of Whitefish Bay 287

Figure C-32 5270-5400 N. Lake Drive
Village of Whitefish Bay 287

Figure C-33 5320-5486 N. Lake Drive
Village of Whitefish Bay 288

Figure C-34 5436-5570 N. Lake Drive
Village of Whitefish Bay 288

Figure C-35 5530 N. Lake Drive-5684 N. Shore Drive
Village of Whitefish Bay 289

Appendix Page

Figure C-36 910 E. Silver Spring Drive-5738 N. Shore Drive
Village of Whitefish Bay 289

Figure C-37 5626-5770 N. Shore Drive
Village of Whitefish Bay 290

Figure C-38 5684-5842 N. Shore Drive
Village of Whitefish Bay 290

Figure C-39 758 E. Day Avenue to Klode Park
Village of Whitefish Bay 291

Figure C-40 5866-6030 N. Shore Drive
Village of Whitefish Bay 291

Figure C-41 6010 N. Shore Drive-601 E. Lake Terrace
Village of Whitefish Bay 292

Figure C-42 6110 N. Lake Drive Court-6216 N. Lake Drive
Village of Whitefish Bay 292

Figure C-43 611 E. Lake Hill Court-6310 N. Lake Drive
Village of Whitefish Bay 293

Figure C-44 6250-6400 N. Lake Drive
Village of Whitefish Bay 293

Figure C-45 6350-6448 N. Lake Drive
Village of Fox Point 294

Figure C-46 6420-6516 N. Lake Drive
Village of Fox Point 294

Figure C-47 6464-6702 N. Lake Drive
Village of Fox Point 295

Figure C-48 6600-6730 N. Lake Drive
Village of Fox Point 295

Figure C-49 6702 N. Lake Drive-6820 N. Barnett Lane
Village of Fox Point 296

Figure C-50 6810-6880 N. Barnett Lane
Village of Fox Point 296

Figure C-51 6836-6942 N. Barnett Lane
Village of Fox Point 297

Figure C-52 6928-6990 N. Barnett Lane
Village of Fox Point 297

Figure C-53 6960 N. Barnett Lane-7000 N. Beach Drive
Village of Fox Point 298

Figure C-54 7000-7124 N. Beach Drive
Village of Fox Point 298

Appendix | Page

Figure C-55 7000-7152 N. Beach Drive
Village of Fox Point 299

Figure C-56 7106-7200 N. Beach Drive
Village of Fox Point 299

Figure C-57 7134-7234 N. Beach Drive
Village of Fox Point 300

Figure C-58 7210-7328 N. Beach Drive
Village of Fox Point 300

Figure C-59 7250 N. Beach Drive-Beach Court
Village of Fox Point 301

Figure C-60 7415-7515 N. Beach Drive
Village of Fox Point 301

Figure C-61 7481-7540 N. Beach Drive
Village of Fox Point 302

Figure C-62 7540-7704 N. Beach Drive
Village of Fox Point 302

Figure C-63 7644-7736 N. Beach Drive
Village of Fox Point 303

Figure C-64 7736-7828 N. Beach Drive
Village of Fox Point 303

Figure C-65 7828-7930 N. Beach Drive
Village of Fox Point 304

Figure C-66 7930-7954 N. Beach Drive
Village of Fox Point 304

Figure C-67 7944-8005 N. Beach Drive
Village of Fox Point 305

Figure C-68 8005-8035 N. Beach Drive
Village of Fox Point 305

Figure C-69 8025-8064 N. Beach Drive
Village of Fox Point 306

Figure C-70 8064-8120 N. Beach Drive
Village of Fox Point 306

Figure C-71 8110-8130 N. Beach Drive
Village of Fox Point 307

Figure C-72 Doctors Park
Village of Fox Point 307

Figure C-73 Doctors Park
Village of Fox Point 308

Appendix		Page
D	Plan Summary Distributed Prior to Public Hearings on the Northern Milwaukee County Shoreline Erosion Management Plan	309
E	Minutes of Public Hearings on the Northern Milwaukee County Shoreline Erosion Management Plan	316
	Village of Shorewood	316
	Village of Fox Point	319
	Village of Whitefish Bay	324

LIST OF TABLES

Table		Page
	Chapter II	
1	Summary of Bluff Heights Along the Lake Michigan Shoreline of Northern Milwaukee County: 1985	12
2	Bluff Composition Along the Lake Michigan Shoreline of Northern Milwaukee County: 1986	15
3	Selected Properties of Bluff Materials Within the Lake Michigan Shoreline of Northern Milwaukee County	16
4	Beach Widths of the Lake Michigan Shoreline of Northern Milwaukee County: 1986	18
5	Beach Characteristics of the Lake Michigan Shoreline of Northern Milwaukee County: 1986	19
6	Beach Slopes Within the Lake Michigan Shoreline of Northern Milwaukee County: 1986	21
7	Estimated Electrical Resistivity Values of Selected Bluff Materials Within Northern Milwaukee County	24
8	Average Monthly Air Temperature at Milwaukee: 1951 Through 1985	27
9	Average Monthly Precipitation and Snow and Sleet at Milwaukee: 1951 Through 1985	28
10	Area and Shoreline Length of Civil Divisions Within the Northern Milwaukee County Lake Michigan Shoreline Study Area: 1986	31
11	Existing Land Use Within the Northern Milwaukee County Lake Michigan Shoreline Study Area: 1985	31
12	Summary of Selected Existing Zoning Regulations in the Northern Milwaukee County Lake Michigan Shoreline Study Area	34
13	Summary of Hourly Wind Direction at Milwaukee: 1975-1984	36
14	Lake Michigan Fetches Used for Deep Water Wave Hindcasting	37
15	Permits Required for Shore Protection Activities in Northern Milwaukee County	47
16	Summary of Northern Milwaukee County Structural Shore Protection Survey: 1986	50
17	Physical and Erosion-Related Characteristics of Bluff Analysis Sections: 1986	52
18	Shoreline Recession Rates Along the Northern Milwaukee County Lake Michigan Shoreline	80
19	Summary of Bluff Recession Rates and Shore Material Loss Along the Northern Milwaukee County Lake Michigan Shoreline	84
20	Comparison of Bluff Recession Rates Measured for the Lake Michigan Shoreline of Southeastern Wisconsin by the Regional Planning Commission	86

Table		Page
	Chapter III	
21	Location of Profile Sites	94
22	Sources of Stratigraphic Data Used for the Slope Stability Analyses at Profile Sites	97
23	Soil Properties Used in the Deterministic Slope Stability Analysis for Rotational Sliding	98
24	Sources of Groundwater Data Used for the Slope Stability Analyses at Profile Sites	99
25	Variation in Soil Properties Assigned in the Probabilistic Slope Stability Analysis	100
26	Guidelines for Classification of Bluff Slopes for Rotational Sliding	102
27	Potential for Translational Sliding Under Bluff Conditions Found in Northern Milwaukee County	104
28	Summary of Deterministic and Probabilistic Slope Stability Analysis Results for Rotational Sliding	139
29	Summary of Evaluations of Bluff Conditions: Summer 1986	140
30	Indicated Shore Protection Measures to Control Shoreline Erosion and Stabilize the Bluff Slopes: Summer 1986	143
31	Extent of Indicated Shore Protection Measures in Northern Milwaukee County: 1986	145
32	Areal Extent of Study Area, Area Directly Adjacent to the Lake Michigan Shoreline, and Area Potentially Subject to Shoreline Erosion Within Each Civil Division: 1986	145
33	Extent of Land and Facilities Lying Within 10 Feet, and Within 25 Feet, of the Edge of an Existing Marginal or Unstable Bluff or Terrace: 1986	147
34	Average Economic Value of Properties Directly Adjacent to the Lake Michigan Shoreline in Northern Milwaukee County: 1986	148
35	Economic Value of Land and Residential Buildings Lying Within 10 Feet, and Within 25 Feet, of the Edge of Marginal or Unstable Bluffs or Terraces Within the Lake Michigan Shoreline of Milwaukee County: 1986	149
	Chapter IV	
36	Recommended Site-Specific Inventories, Analyses, and Design Criteria for Shore Protection Measures	155
37	Comparison of Bluff Toe Protection Measures	161
38	Estimated Beach Slopes that Would Form on Various Beach Fill Materials	173
39	Estimated Effect of Existing Diversion Rates on Great Lakes Water Levels	195
40	Selection Criteria and Estimated Capital and Maintenance Unit Costs of the Bluff Slope Stabilization Plan Components	198
41	Estimated Cost of the Bluff Slope Stabilization Plan for Northern Milwaukee County	199
42	Selection Criteria and Estimated Capital and Maintenance Unit Costs of the Revetment Alternative Plan Components	203
43	Estimated Cost of the Revetment Alternative Plan	204
44	Selection Criteria and Estimated Capital and Maintenance Unit Costs of the Beach Alternative Plan Components	206
45	Estimated Cost of the Beach Alternative Plan	208
46	Selection Criteria and Estimated Capital and Maintenance Unit Costs of the Offshore Alternative Plan Components	211
47	Estimated Cost of the Offshore Alternative	212
48	Estimated Cost of the Preliminary Shoreline Erosion Management Plan for Northern Milwaukee County	214

Table | Page

Table		Page
49	Distribution of the Estimated Cost of the Preliminary Shoreline Erosion Management Plan	218
50	Estimated Cost of the Recommended Shoreline Erosion Management Plan for Northern Milwaukee County	221
51	Distribution of the Estimated Cost of the Recommended Shoreline Erosion Management Plan	224
52	Recommended Implementation Segments for Northern Milwaukee County	226
53	Comparison of Alternative Implementation Approaches	231
	Chapter V	
54	Plan Implementation Authorities Under the Final Recommended Implementation Program	244
55	Estimated Cost of the Final Recommended Shoreline Erosion Management Plan for Northern Milwaukee County	247
56	Distribution of the Estimated Cost of the Final Recommended Shoreline Erosion Management Plan	250
57	Final Recommended Implementation Segments for Northern Milwaukee County	251

LIST OF FIGURES

Figure		Page
	Chapter II	
1	Longitudinal Section Through the Lake Michigan Shoreline of Northern Milwaukee County Showing Bluff Height and Composition: 1986	13
2	Soil Boring Data Collected Under the Northern Milwaukee County Lake Michigan Shoreline Erosion Management Study	15
3	Plan View and Longitudinal Profile of the Groundwater Drainage Tunnel Installed Along 5350-5415 N. Lake Drive, Whitefish Bay: 1932	25
4	Drainage Well at 4430 N. Lake Drive, Shorewood: 1966	26
5	Typical Pattern of Waves Approaching a Beach	36
6	Milwaukee Deep Water Wave Height and Period Probability of Exceedence: Wave Direction from the North, Northeast, Southeast, East and All	37
7	Predicted Lake Michigan Deep Water Wave Conditions During Storms at Milwaukee	38
8	Inshore Wave Height Probability of Exceedence	40
9	Inshore Wave Period Probability of Exceedence	41
10	Lake Michigan Water Levels at Milwaukee: 1860-1986	42
11	Variations in Monthly Mean Lake Michigan Levels: 1900-1986	42
12	Beach Erosion in Response to Wave Action	44
13	Bluff Analysis Section 1	56
14	Bluff Analysis Section 2	57
15	Bluff Analysis Section 3	57
16	Bluff Analysis Section 4	57
17	Bluff Analysis Section 5	58
18	Bluff Analysis Section 6	59
19	Bluff Analysis Section 7	59
20	Bluff Analysis Section 8	60
21	Bluff Analysis Section 9	61

22 Bluff Analysis Section 10 61
23 Bluff Analysis Section 11 62
24 Bluff Analysis Section 12 62
25 Bluff Analysis Section 13 63
26 Bluff Analysis Section 14 63
27 Bluff Analysis Section 15 64
28 Bluff Analysis Section 16 64
29 Bluff Analysis Section 17 65
30 Bluff Analysis Section 18 65
31 Bluff Analysis Section 19: 1976 and 1986 66
32 Bluff Analysis Section 20 67
33 Bluff Analysis Section 21: 1976, 1983, 1986 68
34 Bluff Analysis Section 22: 1973 and 1986 69
35 Bluff Analysis Section 23 70
36 Bluff Analysis Section 24 70
37 Bluff Analysis Section 25 71
38 Bluff Analysis Section 26 71
39 Bluff Analysis Section 27 72
40 Bluff Analysis Section 28 72
41 Bluff Analysis Section 29 73
42 Bluff Analysis Section 30 74
43 Bluff Analysis Section 31 75
44 Bluff Analysis Section 32 75
45 Bluff Analysis Section 33 76
46 Bluff Analysis Section 34 76
47 Bluff Analysis Section 35 77
48 Bluff Analysis Section 36 77
49 Measured Bluff Recession Rates Along the Lake Michigan Shoreline of Northern Milwaukee County 85

Chapter III

50 Common Types of Slope Failures in Lake Michigan Coastal Bluffs 90
51 Effect of Rotational and Translational Sliding on Slope Stability 91
52 Forces Acting on a Typical Section in the Modified Bishop Method of Rotational Slope Stability Analysis 93
53 Variation of Bluff Conditions and the Resultant Safety Factors Calculated by the Probabilistic Slope Stability Analysis 101
54 Concept of the Infinite Slope Analysis for Translational Sliding 103
55 Deterministic Bluff Slope Stability Analysis for Profile 1: Bluff Analysis Section 1 105
56 Deterministic Bluff Slope Stability Analysis for Profile 2: Bluff Analysis Section 2 106
57 Deterministic Bluff Slope Stability Analysis for Profile 3: Bluff Analysis Section 3 107
58 Deterministic Bluff Slope Stability Analysis for Profile 4: Bluff Analysis Section 3 107
59 Deterministic Bluff Slope Stability Analysis for Profile 5: Bluff Analysis Section 4 108
60 Deterministic Bluff Slope Stability Analysis for Profile 6: Bluff Analysis Section 5 109
61 Deterministic Bluff Slope Stability Analysis for Profile 7: Bluff Analysis Section 6 110
62 Deterministic Bluff Slope Stability Analysis for Profile 8: Bluff Analysis Section 7 110

Figure		Page

63 Deterministic Bluff Slope Stability Analysis for Profile 9: Bluff Analysis Section 8 111
64 Deterministic Bluff Slope Stability Analysis for Profile 10: Bluff Analysis Section 9 112
65 Deterministic Bluff Slope Stability Analysis for Profile 11: Bluff Analysis Section 10 113
66 Deterministic Bluff Slope Stability Analysis for Profile 12: Bluff Analysis Section 11—Prefill Conditions 114
67 Deterministic Bluff Slope Stability Analysis for Profile 13: Bluff Analysis Section 11—Fill Under Construction A 114
68 Deterministic Bluff Slope Stability Analysis for Profile 14: Bluff Analysis Section 11—Fill Under Construction B 114
69 Deterministic Bluff Slope Stability Analysis for Profile 15: Bluff Analysis Section 12 115
70 Deterministic Bluff Slope Stability Analysis for Profile 16: Bluff Analysis Section 12 115
71 Deterministic Bluff Slope Stability Analysis for Profile 17: Bluff Analysis Section 13 116
72 Deterministic Bluff Slope Stability Analysis for Profile 18: Bluff Analysis Section 14 117
73 Deterministic Bluff Slope Stability Analysis for Profile 19: Bluff Analysis Section 15 118
74 Deterministic Bluff Slope Stability Analysis for Profile 20: Bluff Analysis Section 16 118
75 Deterministic Bluff Slope Stability Analysis for Profile 21: Bluff Analysis Section 17 119
76 Deterministic Bluff Slope Stability Analysis for Profile 22: Bluff Analysis Section 17 119
77 Deterministic Bluff Slope Stability Analysis for Profile 23: Bluff Analysis Section 18 120
78 Deterministic Bluff Slope Stability Analysis for Profile 24: Bluff Analysis Section 19 121
79 Deterministic Bluff Slope Stability Analysis for Profile 25: Bluff Analysis Section 20 121
80 Deterministic Bluff Slope Stability Analysis for Profile 26: Bluff Analysis Section 21 122
81 Deterministic Bluff Slope Stability Analysis for Profile 27: Bluff Analysis Section 21 123
82 Deterministic Bluff Slope Stability Analysis for Profile 28: Bluff Analysis Section 22 123
83 Deterministic Bluff Slope Stability Analysis for Profile 29: Bluff Analysis Section 23 124
84 Deterministic Bluff Slope Stability Analysis for Profile 30: Bluff Analysis Section 24 125
85 Deterministic Bluff Slope Stability Analysis for Profile 31: Bluff Analysis Section 25 126
86 Failure of the Klode Park Bluff Slope in Bluff Analysis Section 25: 1986 and 1987 126
87 Deterministic Bluff Slope Stability Analysis for Profile 32: Bluff Analysis Section 26 127
88 Deterministic Bluff Slope Stability Analysis for Profile 33: Bluff Analysis Section 27 127

Figure		Page
89	Deterministic Bluff Slope Stability Analysis for Profile 34: Bluff Analysis Section 28	128
90	Deterministic Bluff Slope Stability Analysis for Profile 35: Bluff Analysis Section 28	129
91	Deterministic Bluff Slope Stability Analysis for Profile 36: Bluff Analysis Section 29	129
92	Deterministic Bluff Slope Stability Analysis for Profile 37: Bluff Analysis Section 30	130
93	Deterministic Bluff Slope Stability Analysis for Profile 38: Bluff Analysis Section 31	131
94	Deterministic Bluff Slope Stability Analysis for Profile 39: Bluff Analysis Section 32	132
95	Deterministic Bluff Slope Stability Analysis for Profile 40: Bluff Analysis Section 32	132
96	Deterministic Bluff Slope Stability Analysis for Profile 41: Bluff Analysis Section 33	133
97	Deterministic Bluff Slope Stability Analysis for Profile 42: Bluff Analysis Section 34	133
98	Deterministic Bluff Slope Stability Analysis for Profile 43: Bluff Analysis Section 36	138
99	Deterministic Bluff Slope Stability Analysis for Profile 44: Bluff Analysis Section 36	138

Chapter IV

100	Typical Rip-rap Revetment	164
101	Typical Grout-Filled Bag Revetment	166
102	Typical Interlocking Concrete Block Revetment	167
103	Typical Cast-in-Place Concrete Units	168
104	Effect of a Bulkhead on the Bluff Top Cutback Distance Required to Achieve a Stable Bluff Slope	169
105	Typical Concrete Cantilevered Bulkhead	170
106	Typical Steel Sheet Piling Bulkhead	171
107	Typical Concrete-Stepped Bulkhead	172
108	Typical Rock Groin System with Artificially Nourished Beach	174
109	Typical Steel Sheet Pile Groin with Artificially Nourished Beach	175
110	Typical Armored Headland and Pocket Beach System	176
111	Typical Near-shore Stone Reef with Nourished Coarse Sand and Gravel Beach	177
112	Typical Perched Cobble Beach System	178
113	Typical Pervious Concrete Sill	179
114	Typical Concrete Beach Containment System with Nourished Coarse Sand and Gravel Beach	180
115	Typical Segmented Rubblemound Breakwater System	182
116	Miscellaneous Alternative Types of Breakwaters	183
117	Typical Offshore Island or Peninsula	185
118	Alternative Methods of Bluff Slope Stabilization	186
119	Horizontal Drainage System	188
120	Vertical Drainage System	189
121	Trench Drains	189
122	Stormwater Drainage System to Prevent Excessive Storm Runoff Over the Top of the Bluff	190
123	Procedure Utilized to Estimate Nonstructural Erosion Risk Distance and Nonstructural Setback Distance	192

Figure		Page
124	Procedure Utilized to Estimate Structural Erosion Risk Distance and Structural Setback Distance	193

LIST OF MAPS

Map		Page
	Chapter I	
1	Northern Milwaukee County Lake Michigan Shoreline Erosion Management Study Area	5
	Chapter II	
2	Soils Within the Lake Michigan Shoreline of Northern Milwaukee County	11
3	Location of Soil Boring Sites Within the Lake Michigan Shoreline of Northern Milwaukee County	14
4	Existing Beach Conditions Along the Northern Milwaukee County Lake Michigan Shoreline 1986-1987	20
5	Near-shore Bathymetry of Lake Michigan Along the Northern Milwaukee County Shoreline: 1986	22
6	Locations Where the Elevation of the Groundwater was Identified Within the Northern Milwaukee County Lake Michigan Shoreline: 1986	23
7	Civil Divisions Within the Northern Milwaukee County Lake Michigan Shoreline Study Area: 1986	30
8	Existing Land Uses Within the Northern Milwaukee County Lake Michigan Shoreline Study Area: 1985	32
9	Existing Zoning Districts Along the Immediate Lake Michigan Shoreline of Northern Milwaukee County: 1986	33
10	Shore Orientations Assumed for the Inshore Wave Analysis	39
11	Lake Michigan Shoreline Area Where a Lake Bed Grant Has Been Issued to Milwaukee County by the State of Wisconsin	45
12	Existing Shore Protection Structures in the Northern Milwaukee County Lake Michigan Shoreline Study Area: 1986	49
13	Bluff Analysis Sections in the Northern Milwaukee County Lake Michigan Shoreline Study Area	51
14	Bluff Recession Reaches in the Northern Milwaukee County Lake Michigan Shoreline Study Area	79
	Chapter III	
15	Location of Profile Sites	95
16	Beach Drive Sanitary Sewer and Bluff Analysis Section 35 Subsections: 1986	135
17	Bluff Analysis Section 35 Shoreline Protection: 1986	137
18	Shoreline Conditions in Northern Milwaukee County: 1986	141
19	Indicated Shore Protection Measures for the Northern Milwaukee County Lake Michigan Shoreline: 1986	144
20	Summary of Potential Erosion Damages	150

Map Page

Chapter IV

21 Bluff Slope Stabilization Plan Element for Northern Milwaukee County 196
22 Revetment Alternative Plan for Northern Milwaukee County 201
23 Beach Alternative Plan for Northern Milwaukee County 205
24 Offshore Alternative Plan for Northern Milwaukee County 210
25 Preliminary Shoreline Erosion Management
Plan for Northern Milwaukee County 213
26 Recommended Shoreline Erosion Management
Plan for Northern Milwaukee County 220
27 Implementation Segments for the Recommended
Shoreline Erosion Management Plan 225

Chapter V

28 Final Recommended Shoreline Erosion
Management Plan for Northern Milwaukee County 246

(This page intentionally left blank)

Chapter I

INTRODUCTION

BACKGROUND

On November 1984, representatives of the Villages of Shorewood, Whitefish Bay, and Fox Point, the City of Milwaukee, Milwaukee County, and concerned property owners requested that the Regional Planning Commission provide assistance in defining and seeking solutions to the severe and costly erosion problems that exist along a 7.3-mile reach of the Lake Michigan shoreline of northern Milwaukee County. That reach includes all of the shoreline in the Villages of Shorewood, Whitefish Bay, and Fox Point, and the northernmost 0.7 mile of shoreline in the City of Milwaukee. Subsequently, the Milwaukee County Department of Parks, Recreation and Culture, with assistance from the Commission, in July 1985 applied for and obtained a grant under the Wisconsin Coastal Management Program in partial support of the conduct of a shoreline erosion management study for this shoreline reach of Milwaukee County.

The northern Milwaukee County Lake Michigan shoreline erosion management study was conducted between December 1985 and August 1987. The study was carried out under the guidance of an Intergovernmental and Citizens Advisory Committee created by the Regional Planning Commission. The Committee consisted of representatives of the Villages of Fox Point, Shorewood, and Whitefish Bay; the City of Milwaukee; Milwaukee County; the Wisconsin Department of Natural Resources; the University of Wisconsin Sea Grant Institute; the University of Wisconsin-Milwaukee; and concerned citizens. The functions of the Committee were to articulate the purpose and define the scope and content of the study, as well as to guide the development of a recommended shoreline erosion management plan for the northern Milwaukee County Lake Michigan shoreline. The study includes an inventory and analysis of the existing shoreline erosion and bluff recession conditions, evaluates alternative shoreline erosion control measures, and recommends a comprehensive shoreline erosion management plan.

The shoreline management plan set forth in this report is the culmination of two separate, but coordinated, studies which were conducted simultaneously. A study of bluff conditions and onshore structural and nonstructural protection measures—including bluff slope stabilization measures—was conducted by the staff of the Regional Planning Commission, with the assistance of consultants. A study of coastal processes and offshore structural protection measures was conducted by Warzyn Engineering, Inc.; Johnson, Johnson & Roy, Inc.; and W. F. Baird & Associates, Coastal Engineers, Ltd. Both the onshore and offshore protection studies considered the potential use of tunnel debris from the Milwaukee Metropolitan Sewerage District deep tunnel construction project as a fill material to help reduce the cost of certain protection structures.

This study report sets forth the findings and recommendations of the northern Milwaukee County Lake Michigan shoreline erosion management study conducted by the Commission, together with the related findings of the offshore study.

DEFINITION OF SHORELINE EROSION MANAGEMENT

For the purposes of this study, shoreline erosion management is defined as a coordinated set of measures designed to abate shoreline erosion and reduce the attendant property losses, undesirable aesthetic impacts, and risks to human safety which result from such erosion. Erosion management measures include both onshore and offshore structural measures—such as the construction of revetments, bulkheads, groins, breakwaters, 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 management is the preservation or enhancement of the overall quality of life of the residents of the study area through the selective protection of high-value,, physical resources and those environmental values—recreational, aesthetic, ecological, and cultural—normally associated with, and concentrated in, coastal areas.

NEED FOR A SHORELINE EROSION MANAGEMENT STUDY

The erosion, and subsequent recession, of coastal terraces and bluffs constitutes one of the most adverse impacts of coastal erosion processes.

Bluff and terrace recession rates in the northern Milwaukee County study area range up to 1.5 feet per year, resulting in the annual loss of nearly 12,000 square feet of land and over 585,000 cubic feet of shore material from the study area. To protect both public property and private property from erosion damage, various types of shore protection measures and fill projects have been constructed in the past. Some of these structures, because of improper design, construction, or maintenance, were ineffective and soon were damaged or destroyed. In addition, significant concern has been expressed by the local units of government and citizens concerned that some of these erosion control measures are unsightly, and could cause accelerated shoreline erosion rates in adjacent shoreline areas, particularly "down drift" areas. There is therefore a need to reconsider the current approach to shoreline protection, and the need to modify that approach.

The Advisory Committee for the study identified several primary shoreline erosion issues that needed to be addressed in the study. These issues were:

1. The adequacy of county and local shoreline protection regulations and mitigative requirements.

2. The adequacy of the knowledge of specific conditions and processes which contribute to the erosion of the shoreline, and of the inventories of these conditions.

3. The adverse as well as beneficial effects of the various shore protection measures being used in the study area.

4. The proper role of the county and local units of government in the design of shore protection measures; the development and enforcement of shore protection design and construction standards and regulations; the coordination of the installation of large structures by groups of property owners; the development of financing arrangements for property owners; public education efforts; and the control of erosion on public property.

Although all these issues were considered to be important and were addressed in this study, there was one issue of primary concern. That issue was the need to develop economically feasible shore protection measures to protect private shoreline property—measures which could be implemented by groups of property owners on a section-by-section basis; which would have no significant adverse effects on adjacent shoreline areas; and which would offer a shoreline desired and usable by the property owners and other area citizens. There was also a need to develop the financial arrangements and administrative procedures needed to successfully implement these shore protection measures, and thereby minimize the existing "piecemeal" approach to the problem.

The significant data base and analyses set forth in this study report provide an opportunity for the affected property owners and local governmental units to reach an understanding on the severity and causes of the erosion problems. Accordingly, this report is intended to serve as a data resource which can help the County and local units of government concerned in the assessment of specific erosion problems and solutions for general sections of the shoreline. These data represent typical conditions within the sections and are not necessarily applicable to specific properties within any section. Thus, the data presented in this report should be used with judgment with respect to any specific property.

REVIEW OF PREVIOUS STUDIES

An important element of the study was the collation and analysis of previously collected data on shoreline erosion and recession in northern Milwaukee County. Unlike the previous studies, this study presents inventory data, analyses, and recommendations for specific sections of the shoreline. Issues such as implementation mechanisms and financing arrangements are also addressed in greater detail than in previous studies. The following section briefly describes the major shore erosion studies heretofore conducted within northern Milwaukee County. The findings and recommendations of these studies are incorporated, as appropriate, into Chapter II of this report.

1. <u>Proposed Extension of Lincoln Memorial Drive from Lake Park to Green Tree Road</u>, Thorsten Lindberg, Tentative Plan of the Milwaukee County Regional Planning Department, 1934.

This early proposal recommended that a series of offshore islands be constructed from Lake Park in the City of Milwaukee to Green Tree Road in the Village of Fox Point.

The proposed islands were designed to provide protection against wave erosion, create additional public lake frontage, allow extension of Lincoln Memorial Drive to Green Tree Road, and provide protection for small boating activities. This proposal showed that construction of offshore islands within the study area may be technically feasible. The proposal was not implemented, apparently, due primarily to a lack of funding.

2. "Stabilizing a Lake Michigan Bluff," C. S. Whitney, Civil Engineering, Vol. 6, No. 5, 1936.

   An investigation of the causes of a major bluff failure which occurred in the late 1920's and early 1930's on the Lake Michigan shoreline between Henry Clay Street and Silver Spring Drive in the Village of Whitefish Bay was completed in 1936. The study included information on the characteristics of the beach and bluff, and on the topography and subsoil conditions within the Lake Michigan near-shore area. In addition, the study described an erosion control method used to minimize further bluff failure which included the use of drainage tunnels to reduce the groundwater level and to relieve the hydrostatic pressure within about 530 feet of shoreline. The drainage system was implemented in 1932 and continued to operate until about 1960.

3. Beach Erosion Study, Lake Michigan Shore Line of Milwaukee County, Wis., U. S. Army Corps of Engineers, Beach Erosion Board, House Document No. 526, 79th Congress, 2nd Session, 1945.

   In 1945, the U. S. Army Corps of Engineers completed a study to determine the best method 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, weather conditions, near-shore bathymetry, sources and movement of the beach material, the effects of lake levels on the shore, and the effect of ice action on shore protection structures.

   The study recommended that the shoreline from the City of Milwaukee Linnwood Avenue water treatment plant northward to the Fox Point terrace be protected by an extension of Lincoln Memorial Drive along a lakefront fill having large sand beaches at intervals, including at Atwater Park and Big Bay Park; and that the remainder of the Fox Point shoreline be protected by a series of groins artificially nourished with sand. The study also concluded that the federal government should not provide funds for the implementation of shore protection measures in northern Milwaukee County. The recommendation concerning the extension of Lincoln Memorial Drive was again found to be technically feasible, but was not acted on. Most groins constructed within the study area have been only moderately successful because of limited littoral drift and because beach nourishment has generally not been allowed by State regulation.

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, presented information on Lake Michigan water levels, geologic conditions, the extent of shore erosion, shoreline recession rates, existing shore protection structures, and beach conditions. Alternative types of shore protection measures were reviewed, and, for private property, it was recommended that rip-rap revetments or concrete bulkheads be installed, along with groins for those areas where a beach was desired. The study committee noted that an effective solution to erosion problems in the study area would be to extend Lincoln Memorial Drive on fill placed at the base of the bluff northward to Fox Point. The report stated that such an alternative would not only provide shore protection, but would provide improved public access to the lakeshore. 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, are still used to provide privately funded shore protection. No funds have become available for the proposed extension of Lincoln Memorial Drive.

5. "Problems of Great Lakes Shore Erosion," W. T. Painter, Paper Presented at First World Congress on Water Resources, Chicago, Illinois, September 1973.

   This study presented general information on Great Lakes geology, soil mechanics, groundwater flow, and slope stability. In addition, this study documented an investigation of the causes of a major bluff failure that occurred in April 1973 on the Lake Michigan shoreline at 5270 N. Lake Drive, Village of Whitefish Bay. The report also contained a description of the erosion control methods used to stabilize the bluff slope at the site, which included a vertical groundwater drainage system, bluff slope regrading by placing soil, rock, and concrete rubble on the bluff slope, and bluff toe protection, consisting of a concrete riprap revetment. The project was completed in September 1973.

6. Lake Michigan Shoreline, Milwaukee County, Wisconsin, Preliminary Feasibility Report, U. S. Army Corps of Engineers, 1975.

   This study was initially intended to investigate the degree of shore erosion, and to develop and analyze alternative solutions to the erosion problems, along the publicly owned shorelands in Milwaukee County. However, at the request of Wisconsin Congressman Henry S. Reuss, the scope of the study was subsequently expanded to include further 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 the Fox Point terrace. This study was more comprehensive than the earlier studies, and presented data on climate, population, income, transportation facilities, recreational resources and demands, shore erosion damages to land and structures, and environmental impacts of alternative erosion control measures. The study concluded that although there were several alternative methods of erosion control which were technically feasible for northern Milwaukee County, none of the alternatives could be economically justified. This study therefore concurred with the 1945 recommendation by the Corps of Engineers that no federal funds be used for the protection of the shoreline in northern Milwaukee County.

7. 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.

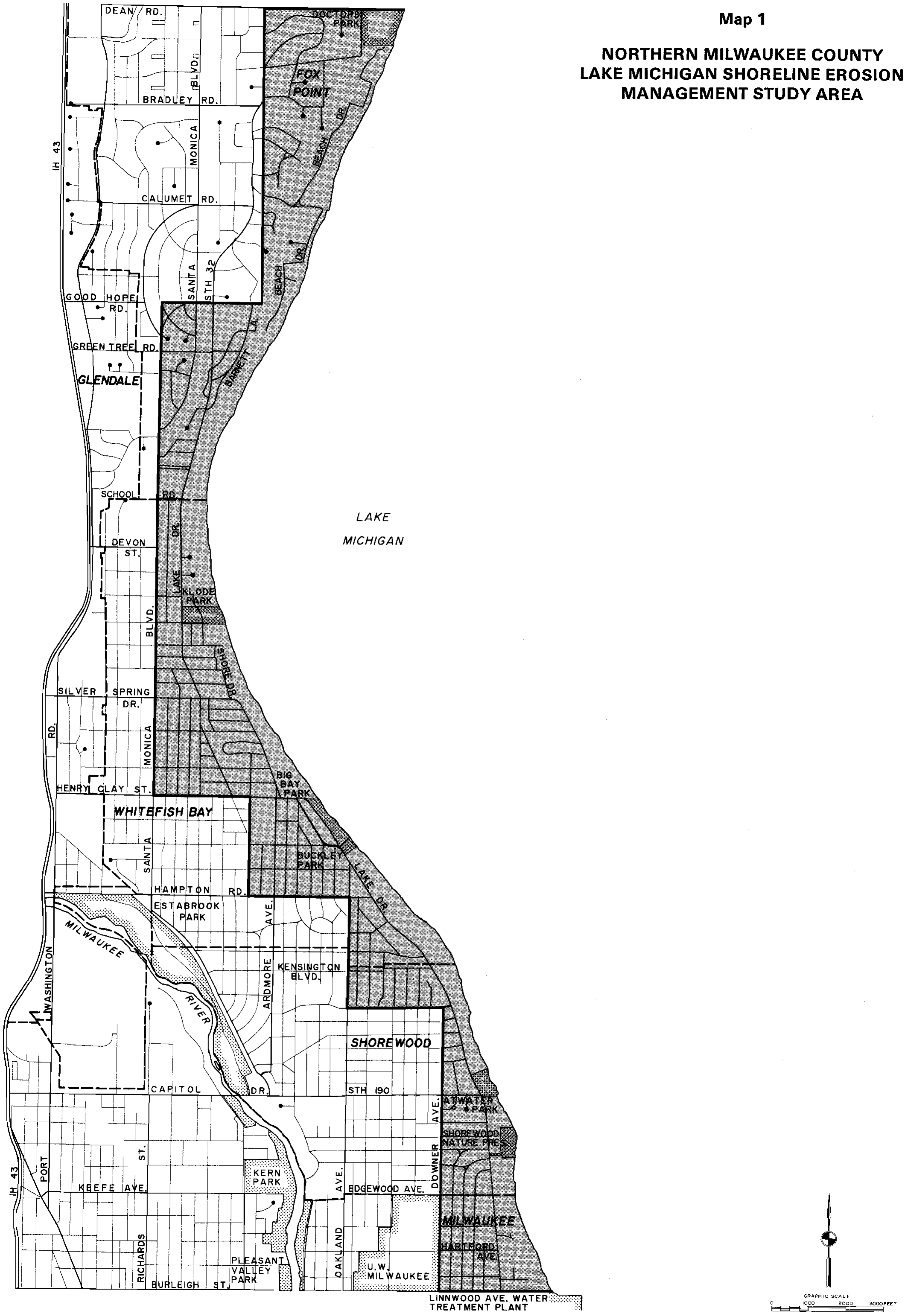
   An inventory of shoreline conditions within Milwaukee County was completed in 1977. The county shoreline was divided into six reaches, each reach having similar physical and erosion-related characteristics. The reach identified as Reach 10 comprised the northern Milwaukee County study area and was rated as the sixth most critical erosion area of a total of 32 reaches along the entire Lake Michigan shoreline of Wisconsin. The study estimated long-term—110-year—bluff recession rates ranging from one to six feet per year for this reach. The study reported data on beach, bluff, and geologic characteristics; observed shore damages; and shore protection structures. Twenty bluff slope stability analyses and one soil boring were conducted under the shore erosion study within Reach 10. The study did not recommend specific types of shore protection measures to be implemented in Reach 10.

SHORELINE EROSION MANAGEMENT STUDY AREA

The northern Milwaukee County shoreline erosion management study area consists of the 1,726 acres of land adjoining Lake Michigan along 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, as shown on Map 1. The total

Map 1

NORTHERN MILWAUKEE COUNTY LAKE MICHIGAN SHORELINE EROSION MANAGEMENT STUDY AREA



Source: SEWRPC.

study area contains about 7.3 miles of Lake Michigan shoreline. The study area thus consists of that portion of the City of Milwaukee and North Shore communities that affects, or is affected by, Lake Michigan resources and 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 residential development and unique recreational opportunities which attract users from well inland. Due consideration is given in this study to these and other important linkages between the study area and the balance of the local communities and Milwaukee County. Consideration is also given to the reaches of the Lake Michigan shoreline adjacent to the study area, since some shore protection measures may have effects on nearby shoreline areas.

PURPOSE AND SCOPE

The primary purpose of the northern Milwaukee County shoreline erosion management study is to define the risk of erosion damage along the Lake Michigan shoreline; to explore alternative and to recommend effective, economically feasible, and environmentally acceptable measures for erosion control; and to identify implementation mechanisms, financing arrangements, and local regulations needed to successfully carry out the recommended plan. An important objective of the study was 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 eliminates or 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. This study is not intended to provide "facility planning" level preliminary engineering designs for individual shore protection measures. Rather, the study presents "systems level" recommendations on the type of control measures which should be implemented, and provides guidelines and general information which should be applied and followed in the subsequent detailed design of erosion control measures.

To accomplish these objectives, 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 and bluff recession in the study area and to the characteristics of the natural resource base which affect shoreline management.

2. The preparation of large-scale, one inch equals 100 feet scale topographic maps of the shoreline area of northern Milwaukee County, together with attendant horizontal and vertical survey control, which were used to compile inventory data and to help determine the need for, and to design parameters for, both structural and nonstructural shore protection measures.

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

4. The development and evaluation of alternative onshore and offshore protection measures based upon the inventory and erosion hazard data and which include both nonstructural and structural measures to reduce damages by shoreline erosion and bluff recession.

5. The recommendation of specific types of nonstructural and structural erosion control measures, as well as implementation mechanisms and financing arrangements.

SUMMARY

In November 1984, the City of Milwaukee, the Villages of Shorewood, Whitefish Bay, and Fox Point, and Milwaukee County requested that the Regional Planning Commission provide assistance in seeking solutions to the severe shoreline erosion problem occurring along the Lake Michigan shoreline of northern Milwaukee County. Subsequently, with financial assistance from the Wisconsin Coastal Management Program, a shoreline erosion management study was undertaken under the guidance of an intergovernmental and citizens advisory committee. The shoreline management plan set forth in this report is the result of two separate but coordinated studies. The first study of bluff conditions and onshore protection measures was conducted by the Regional Planning Commission staff with the assistance of consultants under contract to the Commission. The second study of coastal processes and offshore protection measures was conducted by Warzyn Engineering, Inc.; Johnson, Johnson & Roy, Inc.; and W. F. Baird & Associates, Coastal Engineers, Ltd.

Shoreline erosion management is defined as a coordinated set of measures designed to abate shoreline erosion and reduce attendant property losses, aesthetic impacts, and risks to human safety. Erosion rates within the study area range up to 1.5 feet per year, with the loss of over 585,000 cubic feet of shore material per year. A number of shore erosion-related issues which need to be addressed were defined by the Intergovernmental and Citizens Advisory Committee. These issues involve the basic understanding of the causes of shoreline erosion and bluff slope failure; the characteristics and effects of various types of shore protection measures; the role of local communities in shoreline management; appropriate shore protection measures for both public and private property; and suitable financial arrangements and administrative procedures to help implement shore protection measures.

The study area consists of the Lake Michigan shoreline area 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. The study area consists of about 1,726 acres and contains about 7.3 miles of shoreline.

The primary purpose of the study is to define the risk of erosion damage, recommend effective erosion control measures, and identify appropriate implementation actions. To achieve this purpose, the study consists of an inventory of erosion-related data; the preparation of large-scale topographic maps; the identification of erosion risk areas and shoreline recession rates; the development and evaluation of alternative shore protection measures; and the preparation of a recommended comprehensive shoreline erosion management plan.

(This page intentionally left blank)

Chapter II

INVENTORY FINDINGS

INTRODUCTION

In order to identify and evaluate alternative structural and nonstructural shoreland protection measures, the risk of erosion damages must be determined, 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 within those areas. Accordingly, this chapter describes the Lake Michigan shoreland study area, and provides pertinent information on the elements of the natural resource base relevant to coastal erosion, on the existing land use and zoning patterns, and on the types, causes, and rates of shoreline erosion and bluff recession occurring within the coastal area of northern Milwaukee County.

The study area, as defined in Chapter I and shown on Map 1, includes that portion of northern Milwaukee County which most directly affects, and is most affected by, Lake Michigan shoreline erosion. Some of the data presented herein, including data on bluff characteristics, groundwater resources, and types and causes of bluff erosion, were collected through special surveys conducted by the University of Wisconsin-Madison, the University of Wisconsin-Milwaukee, and PTL-Inspectorate, Inc., working under contract to the Regional Planning Commission. Deep water wave characteristics presented in this chapter were provided by W. F. Baird & Associates, Coastal Engineers, Ltd., and near-shore bathymetry was provided by Warzyn Engineering, Inc. Other inventory data—such as data on the geology and climate of the area—were collated from Commission files. Detailed information on the topographic and cultural features of the area was provided by 1 inch equals 100 feet scale, 2-foot contour interval, topographic and cadastral maps prepared by Aerometric Engineering, Inc., photogrammetric engineers, to Commission specifications and 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 segments of the immediate shoreland area.

This chapter consists of seven sections following the introduction. The first section describes the natural resource base pertinent to coastal erosion management. The second section describes the existing land use pattern of the study area, and provides information on the comprehensive zoning district boundaries and related regulations within the shoreland area. The third section describes coastal erosion processes. The fourth section concerns existing regulations—other than zoning—relating to shoreland development. Existing structural shore protection measures 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 bluff recession rates.

NATURAL RESOURCE BASE

This section describes those aspects of the natural resource base that 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, and climate 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. Silurian dolomite, primarily Niagara dolomite, lies above the Ordovician rock formations, and is approximately 300 feet thick. 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 Milwaukee County study area. The

Devonian Formations are covered by glacial deposits ranging up to 150 feet in thickness within the study area. The Milwaukee Formation crops out at the base of the bluff near 6818 N. Barnett Lane in the Village of Fox Point.

Materials directly deposited by glacial ice are called till. Several layers of till can be identified within the study area. The lowest layer of till exposed by bluffs within the study area is known as the New Berlin Formation. This formation ranges in thickness up to 70 feet and consists of a lower sand and gravel member and an upper member, sandy in texture with pebbles, cobbles, and some boulders. Directly above the New Berlin Formation lies a layer known as the Oak Creek Formation. The Oak Creek Formation, whose maximum thickness ranges up to 115 feet, is composed of pebbly, silty clay loam; lacustrine clay, silt, and sand; and glaciofluvial sand and gravel. The layer nearest the surface and overlying the Oak Creek Formation is known as the Ozaukee Member of the Kewaunee Formation. The till of the Ozaukee Member is fine-grained, typically silty clay or silty clay loam which is reddish in color.

All three glacial formations are exposed by the bluffs within the study area. Within the exposed bluffs, the Kewaunee Formation ranges from 20 to 85 feet in thickness, the Oak Creek Formation ranges up to 30 feet in thickness, and the New Berlin Formation ranges up to 20 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. Additional glacial deposits are located beneath the lake bed.

Soils

Soil properties influence the rate and amount of stormwater runoff, thereby affecting the severity of surface erosion at the top of the lake 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 intensively developed areas. The findings of the soil surveys have been published in SEWRPC Planning Report No. 8, Soils of Southeastern Wisconsin (1966). The surveys have provided data on the physical, chemical, and biological properties of the mapped soils and, more importantly, have provided interpretations of the soil properties for planning, engineering, agricultural, and resource conservation purposes.

Within the study area, the detailed soils mapping was conducted only within the Village of Fox Point. Detailed soils mapping was not conducted within the City of Milwaukee and Villages of Shorewood and Whitefish Bay because, owing to the density of the urban development, 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.

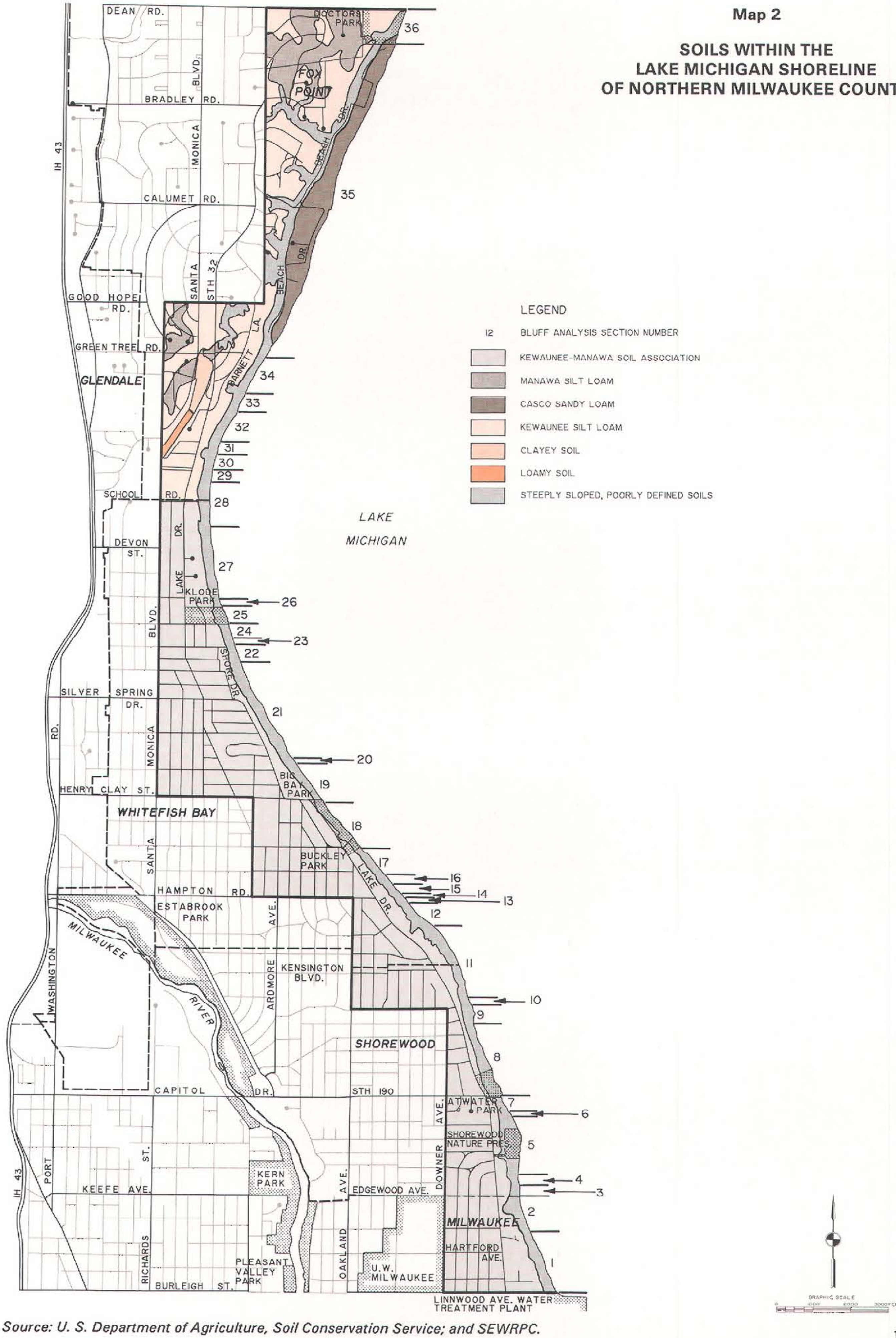
As shown on Map 2, Kewaunee and Manawa silt loams cover about 536 acres, or 80 percent, of the Village of Fox Point. Casco sandy loam covers about 64 acres, or 10 percent of the Village—generally, the terraced portion of the shoreline. About six acres, or less than 1 percent of the village shoreline, is covered by loamy or clayey land. The remaining 66 acres, or 10 percent of the village area, contains steeply sloped land covered by shallow, poorly defined soils.

Map 2 also shows that the Villages of Whitefish Bay and Shorewood and the City of Milwaukee are covered by soils collectively referred to by the U. S. Soil Conservation Service as the Kewaunee-Manawa Association. Thus, the soils in these areas are similar to the soils surveyed in the upland portions of the Village of Fox Point.

The Kewaunee and Manawa soils form in this loess and silty clay glacial till, on moraines, and in depositional areas. A large amount of stormwater runoff can be generated from these soils, as well as from the loamy and clayey lands, because of the low infiltration capacity, low permeability, and poor drainage characteristics of the soils. Areas covered by these soils may therefore contribute substantial surface runoff over the top of the bluffs, causing surface erosion of the bluff face. Casco soils, which form over

Map 2

SOILS WITHIN THE LAKE MICHIGAN SHORELINE OF NORTHERN MILWAUKEE COUNTY



Source: U. S. Department of Agriculture, Soil Conservation Service; and SEWRPC.

calcareous sand and gravel outwash, would generate low to moderate amounts of stormwater runoff because of the moderate infiltration capacity, moderate permeability, and good drainage characteristics of the soils.

Bluff Characteristics

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

Table 1 summarizes the lengths of shoreline with various bluff heights. Bluff heights are also shown in Figure 1. In the southernmost portion of the study area within the City of Milwaukee, bluffs generally range in height from 75 to 100 feet. Northward through the Village of Shorewood, the bluff heights increase somewhat, ranging from 90 to 115 feet. The height of the bluffs within the portion of Whitefish Bay south of E. Lake Terrace generally ranges from 65 to 95 feet. North of E. Lake Terrace to Green Tree Road the bluff heights increase, ranging from 100 to 130 feet. North of Green Tree Road, a relatively wide terrace exists in front of the bluffs, which extends to a maximum width of approximately 900 feet and ranges from 4 to 10 feet in height. About 24 percent of the shoreline within the study area is located within the terraced area. Near the northernmost portion of the study area, within Doctors Park, the terrace disappears and the bluff heights are about 90 feet. About 17 percent of the shoreline has bluffs ranging from 60 to 80 feet in height; about 30 percent has bluffs ranging from 81 to 100 feet in height; and about 25 percent has bluffs ranging from 101 to 120 feet in height. Less than 4 percent of the shoreline has bluffs higher than 120 feet.

The natural bluffs of northern Milwaukee County are composed of a variety of glacial meltwater and lacustrine deposits. Field surveys were conducted in May 1986 to identify these materials as exposed in the bluff faces. 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. Map 3 shows locations where soil boring data were available prior to this study. Nine additional soil borings were taken in October and November 1986 by PTL-Inspectorate, 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. The locations where the additional soil borings were taken are also shown on Map 3, and the results illustrated in Figure 2.

The composition of the bluffs, based on all of the above data, is described on the longitudinal section in Figure 1. Table 2 indicates the relative predominance of the various materials on the face of the bluff. Ozaukee till was found to be the predominant bluff material, covering about 26 percent of the total bluff face surface area in a vertical plane within the study area. Oak Creek till was found to be the second most common bluff material, covering about 10 percent of the total bluff face. A combination of silt and sand,

Table 1

SUMMARY OF BLUFF HEIGHTS ALONG THE LAKE MICHIGAN SHORELINE OF NORTHERN MILWAUKEE COUNTY: 1985

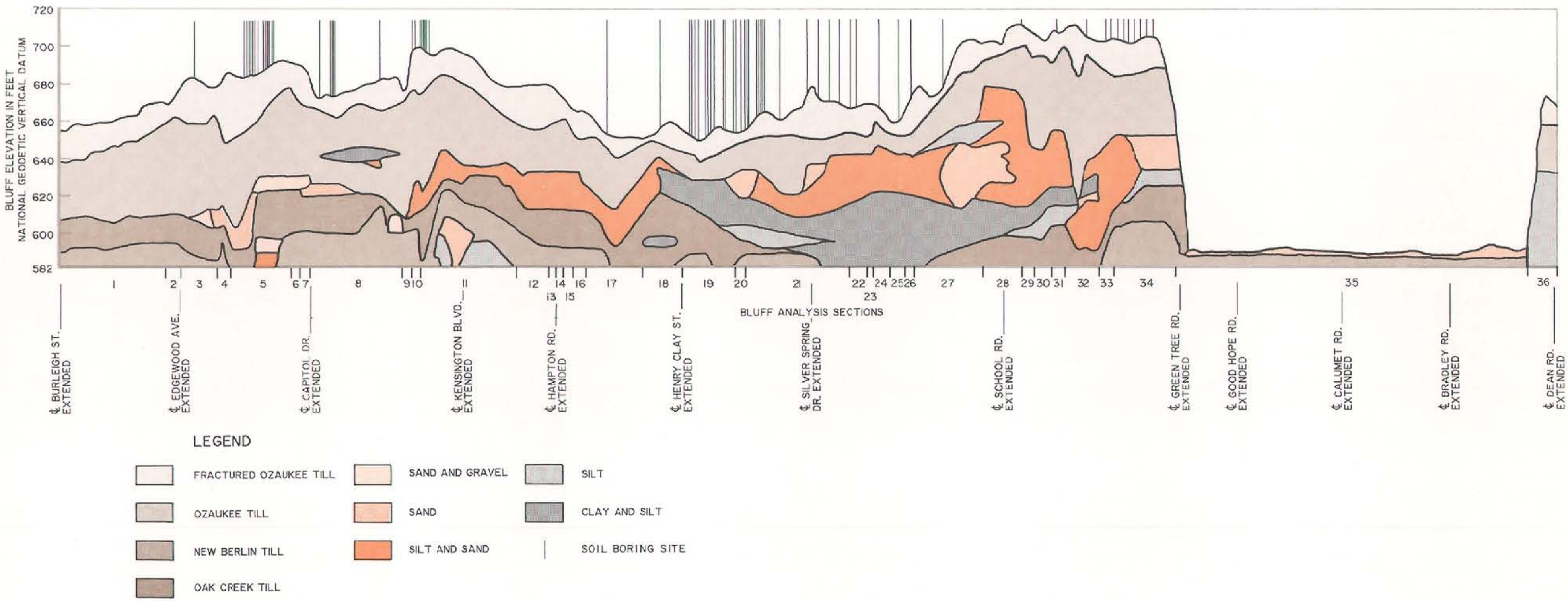
Bluff Height (feet)	Length of Shoreline (feet)	Percent of Total Study Area Shoreline Length
61 - 70	3,660	9.4
71 - 80	4,800	12.4
81 - 90	5,340	13.8
91 - 100	6,040	15.6
101 - 110	2,640	6.8
111 - 120	5,600	14.9
121 - 130	1,210	3.1
Total	29,460[a]	76.0[a]

[a]Excludes the Fox Point terrace, which covers 9,070 feet, or about 24 percent of the total shoreline, and which has a height of less than 10 feet.

Source: SEWRPC.

Figure 1

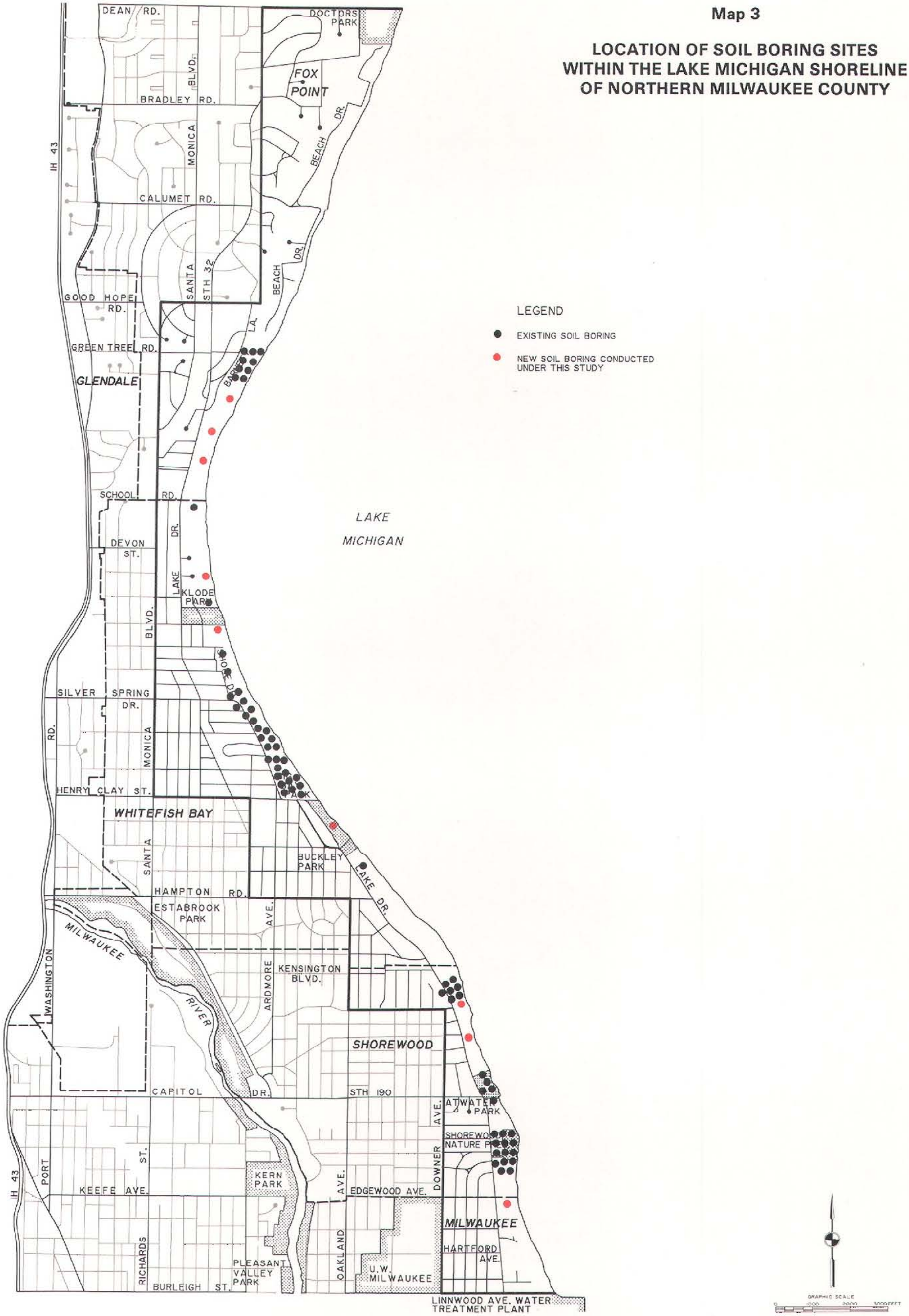
LONGITUDINAL SECTION THROUGH THE LAKE MICHIGAN SHORELINE OF NORTHERN MILWAUKEE COUNTY SHOWING BLUFF HEIGHT AND COMPOSITION: 1986



Source: D. M. Mickelson and SEWRPC.

Map 3

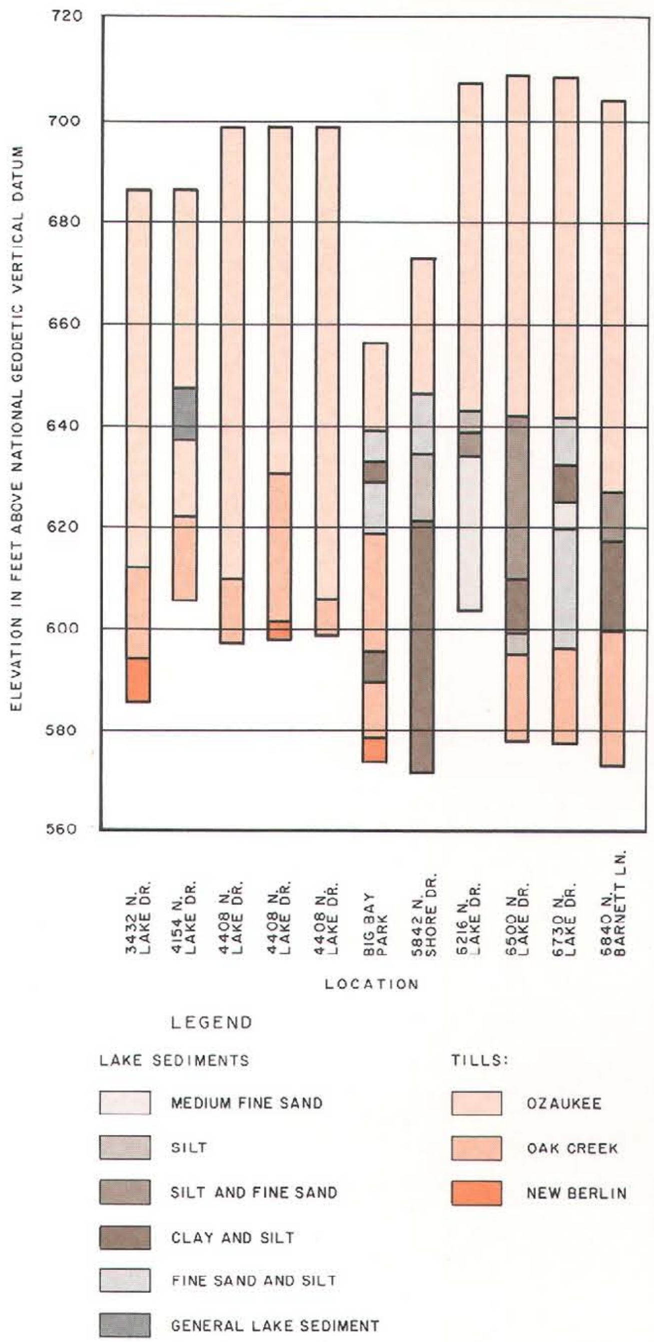
LOCATION OF SOIL BORING SITES WITHIN THE LAKE MICHIGAN SHORELINE OF NORTHERN MILWAUKEE COUNTY



Source: SEWRPC.

Figure 2

SOIL BORING DATA COLLECTED UNDER THE NORTHERN MILWAUKEE COUNTY LAKE MICHIGAN SHORELINE EROSION MANAGEMENT STUDY



Source: SEWRPC.

Table 2

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

Bluff Composition	Percent of Bluff Face Surface Area in the Vertical Plane
Ozaukee Till	26
Oak Creek Till	10
New Berlin Till	6
Silt and Sand	9
Sand	4
Clay and Silt	2
Sand and Gravel	1
Silt	1
Undetermined	41
Total	100

Source: D. M. Mickelson and SEWRPC.

and New Berlin till were found to cover about 9 and 6 percent of the total bluff face, respectively. Sand and gravel, silt, clay and silt, and sand were also identified in the bluff face, totaling about 8 percent of the total bluff face. The material constituting the remaining 41 percent of the bluff face was undetermined because no stratigraphic data were available and the slopes were considered to be stable and well vegetated. Laboratory analyses of the bluff materials collected in the field by grab samples in May 1986, and through the soil borings conducted in October and November 1986, were performed by the Department of Civil Engineering, University of Wisconsin-Madison. The results of the laboratory analyses, as set forth in Table 3, provide a quantitative determination of the soil properties that determine the resistance of the soil to slope failure. The moisture content, liquid and plastic limits, and silt and clay fraction of soil samples provide information useful in calculating the ability of the soil materials to resist 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 dry 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 13.7 to 48.1 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

Table 3

SELECTED PROPERTIES OF BLUFF MATERIALS WITHIN THE LAKE MICHIGAN SHORELINE OF NORTHERN MILWAUKEE COUNTY

Soil Type	Location	Depth (feet)	Liquid Limit (percent)	Plastic Limit (percent)	Plasticity Index (percent)	Percent Gravel	Percent Sand	Percent Silt	Percent Clay	Unified Soil Classification
Grab Sample										
Glacial Tills										
New Berlin	3850 N. Lake Drive	- -	15.4	12.0	3.4	5.8	50.2	28.0	16.0	SM
	4480 N. Lake Drive	- -	19.6	14.2	5.4	3.7	37.7	38.6	20.0	CL-ML
	4626 N. Lake Drive	- -	20.1	14.4	5.7	7.8	36.3	36.9	19.0	CL-ML
	6818 N. Barnett Lane	- -	13.7	12.9	0.8	23.2	32.3	33.5	11.0	SM
	7004 N. Barnett Lane	- -	16.2	13.7	2.5	16.2	17.9	58.0	8.0	ML
Oak Creek	3562 N. Lake Drive	- -	29.7	15.8	13.8	0.8	12.5	47.7	39.0	CL
	6818 N. Barnett Lane	- -	34.4	17.3	17.1	3.1	7.1	43.8	46.0	CL
Ozaukee	3510 N. Lake Drive	- -	32.7	17.1	15.6	2.2	7.6	47.2	43.0	CL
	4100 N. Lake Drive	- -	32.1	16.6	15.5	2.2	6.9	46.9	44.0	CL
	4460 N. Lake Drive	- -	31.7	15.9	15.8	0.3	6.6	49.1	44.0	CL
	6330 N. Lake Drive	- -	37.5	17.1	20.4	2.9	8.1	45.0	44.0	CL
Lake Sediments										
Medium Fine Sand	6424 N. Lake Drive	- -	- -	- -	- -	0.0	90.2	9.8	- -	SP
Silt	3590 N. Lake Drive	- -	- -	18.2	- -	0.0	12.2	81.8	6.0	ML
	6124 N. Lake Drive	- -	- -	20.9	- -	0.0	7.0	84.0	9.0	SW
Clay and Silt	3432 N. Lake Drive	- -	48.1	22.3	25.9	0.0	2.8	61.2	36.0	CL
	4700 N. Lake Drive	- -	35.3	17.3	17.9	0.0	0.0	56.0	44.0	CL
	6430 N. Lake Drive	- -	34.5	11.6	22.9	0.0	0.0	56.0	44.0	CL
Soil Borings										
Glacial Tills										
New Berlin	4408 N. Lake Drive	100	19.0	13.6	5.4	4.0	32.0	46.0	18.0	CL
Oak Creek	3432 N. Lake Drive	80	21.3	15.9	5.4	8.0	20.0	54.0	18.0	CL
	6730 N. Lake Drive	130	31.6	16.1	15.4	0.5	8.0	50.0	41.5	CL
	6840 N. Barnett Lane	110	31.8	16.5	15.3	2.0	7.0	53.0	38.0	CL
Lake Sediments										
Medium Fine Sand	6730 N. Lake Drive	85	- -	- -	- -	0.0	81.0	19.0	- -	SP
Silt	5842 N. Shore Drive	35	18.1	17.8	0.3	0.0	8.0	79.9	13.0	ML
	5842 N. Shore Drive	40	47.9	76.6	- -	0.0	0.0	93.0	7.0	ML
	6216 N. Lake Drive	65	19.2	18.9	0.6	0.0	1.0	90.0	9.0	ML
	6500 N. Lake Drive	112	18.4	17.2	1.3	0.0	2.0	88.0	10.0	ML
Silt and Fine Sand	4154 N. Lake Drive	45	- -	- -	- -	0.0	68.0	3.0	29.0	SW
	6216 N. Lake Drive	70	- -	- -	- -	0.0	55.0	43.7	1.3	SP
	6216 N. Lake Drive	85	- -	- -	- -	0.0	11.0	83.0	6.0	ML
Clay and Silt	Big Bay Park	25	27.3	32.1	- -	0.0	1.0	71.2	27.8	ML
	5842 N. Lake Drive	35	28.6	17.4	11.2	0.0	1.5	63.5	35.0	ML
	5842 N. Lake Drive	55	30.0	18.5	11.6	0.0	0.0	67.0	33.0	CL
	5842 N. Lake Drive	75	34.1	18.2	15.9	0.0	0.0	56.0	44.0	CL
	6500 N. Lake Drive	110	22.2	13.3	9.0	1.0	23.0	52.0	24.0	CL
	6730 N. Lake Drive	80	26.2	15.4	10.8	2.0	11.0	58.0	29.0	CL
Fine Sand and Silt	4154 N. Lake Drive	40	23.2	13.8	9.4	0.0	25.5	69.5	5.0	CL
	Big Bay Park	20	- -	18.4	- -	0.0	21.0	73.5	5.5	ML
	5842 N. Shore Drive	30	- -	- -	- -	0.0	23.0	74.0	3.0	ML
	6730 N. Lake Drive	70	- -	- -	- -	0.0	36.0	64.0	- -	ML
	6730 N. Lake Drive	90	- -	- -	- -	0.0	36.0	64.0	- -	ML

Source: T. B. Edil and D. M. Mickelson, 1986.

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 0.3 to 25.9 percent. With a known liquid limit and plasticity index, the measured moisture content of a soil sample can be used to estimate the behavior of that soil as a liquid or as a plastic.

The fraction of the soil that is composed of silt- and clay-size particles is an indicator of 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 9.8 percent to 100 percent.

All bluff soil samples were classified on the basis of the Unified Soil Classification system. This system classifies soils primarily for engineering purposes. CL soils are relatively fine-grained, impervious soils with a high clay content, low plasticity, and a liquid limit of less than 50 percent. CL soils generally have very low shear strengths. ML soils are fine-grained, fair to poorly drained soils with a high silt and silty clay content, low plasticity, and a liquid limit of less than 50 percent. ML soils tend to have low shear strengths. SM soils are relatively coarse-grained, well-drained soils that are poorly graded, with an appreciable amount of fine-grained particles. SP soils are coarse-grained, well-drained soils that are poorly graded, with little or no fine-grained particles. SW soils are coarse-grained, well-drained soils that are well graded, with little or no fine-grained particles. SP, SM, and SW soils tend to have higher shear strengths than do ML and CL soils. The soil properties associated with these Unified Soil Classification groups were used—in conjunction with the measured values set forth in Table 3—in the slope stability analyses.

Values for the effective friction angle and the effective cohesion intercept of selected bluff materials within the study area were obtained through triaxial compression tests completed for four of the soil samples collected through soil borings. The effective friction angle and the effective cohesion intercept are coefficients related to the frictional resistance and cohesiveness of the soil to shearing when placed under stress. 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. For sand, the effective friction angle is that angle at which the soil would achieve a stable slope if no groundwater were present within the soil. Effective friction angles within the study area were found to be relatively uniform, ranging from 26 to 32 degrees. Effective cohesion intercept values are generally higher for soils that contain appreciable amounts of fine-grained particles. Within the study area, effective cohesion intercept values were found to range from 0 to 4,820 pounds per square foot.

Beach Characteristics

A beach may be defined as an area of unconsolidated material which extends landward from the ordinary low-water line to the line marking a distinct change in physiographic form, or the beginning of permanent terrestrial vegetation. The width of a beach and the size and character of the sediments found on beaches vary widely in response to the lake water level, the degree of wave action affecting the beach, the slope of the beach face and the near-shore lake bottom, the kinds of material available near the shore for the formation of beaches, and man-made structures. Beach materials are supplied by littoral drift transporting particles contributed to the lake by watershed drainage and up-current shoreline erosion and bluff recession. As already noted, the bluffs within the study area are composed largely of glacial till. Only a small portion of this till is sand size or larger, which would help form beaches. Tables 4 and 5 set forth beach characteristics for the northern Milwaukee County shoreline of Lake Michigan as surveyed in August 1986.

Table 5 indicates 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 sand, gravel, and cobbles, since clay and silt are more readily kept in suspension and carried out into the lake. These finer materials tend ultimately to settle out in calmer, deeper, offshore waters. In 1986, about 69 percent of the

Table 4

BEACH WIDTHS OF THE LAKE MICHIGAN SHORELINE OF NORTHERN MILWAUKEE COUNTY: 1986

Beach Width (feet)	Shoreline Length (feet)	Percent of Total Shoreline Length
0 - 10	26,690	68.8
11 - 30	3,400	8.8
31 - 50	4,620	11.9
51 - 70	2,380	6.1
71 - 90	680	1.8
> 90	1,000	2.6
Total	38,770	100.0

Source: SEWRPC.

northern Milwaukee County shoreline exhibited either 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. The primary beach surface and subsurface material within the study area was sand and gravel, covering about 10 percent of the total shoreline.

Map 4 shows the distribution of various beach materials along the shoreline. Sand, and sand and gravel were predominant along the far southern shoreline reaches, the shoreline near Atwater Park, the central portion of the Fox Point terrace, and the southern portion of Doctors Park. Beach areas containing larger portions of gravel were found near Klode Park and in the southern portion of Big Bay Park. Much of the remainder of the shoreline area exhibited little or no beach owing to the protective structures present and the high water levels extant at the time of the field survey.

Table 4 and Map 4 also indicate the beach widths along the coast. About 20 percent of the shoreline had a beach ranging in width from 11 to 50 feet; and about 8 percent had a beach ranging in width from 51 to 90 feet. Only about 3 percent of the shoreline, located at Atwater Park, had a beach over 90 feet in width as of early summer 1986.

The beach slopes of the northern Milwaukee County shoreline are shown on Map 4. Generally, beach slopes ranged up to 10 degrees. However, steeper beach slopes ranging from 10 to 20 degrees were measured near the southern portion of Big Bay Park and the northern portion of Atwater Park. Table 6 summarizes beach slopes with respect to the total shoreline length. No beach slope determination was made for the portion of the shoreline that had a beach width of 10 feet or less—approximately 69 percent of the total shoreline. Of the remaining shoreline, 4,080 feet, or 10 percent of the total shoreline, had a beach slope ranging from 0 to 6 degrees; 7,170 feet, or 19 percent, had a beach slope ranging from 7 to 12 degrees; and 830 feet, or 2 percent, had a beach slope greater than 12 degrees. Generally, the wider beaches had slightly flatter slopes and were composed of finer-grained materials, whereas the narrower beaches had steeper slopes and were composed of coarser-grained materials.

Near-shore Bathymetry

The near-shore bathymetry, or lake bottom elevations, within 1,200 to 2,200 feet of the shoreline was surveyed by Warzyn Engineering, Inc., in October 1986. The bottom elevations, shown at three-foot contour intervals, are presented on Map 5, and the individual near-shore profiles are presented in Appendix A. The near-shore bathymetry 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. As shown on the map, the near-shore slopes were most gentle—about 0.5 degrees, or about 1 on 100 or 1 percent—off the Village of Fox Point terrace, near the boundary between the Villages of Shorewood and Whitefish Bay, and near the southern end of the study area in the City of Milwaukee. The near-shore slopes were steepest—about 1.0 degree, or about 1 on 50 or 2 percent—along the northeast-facing shoreline of the Village of Whitefish Bay between Klode Park and Big Bay Park, and in the Village of Shorewood just north of Atwater Park.

The near-shore bathymetry within the study area was previously surveyed in 1871, 1912, and 1944.[1] A review of these early data indicated

---

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

Table 5

BEACH CHARACTERISTICS OF THE LAKE MICHIGAN SHORELINE OF NORTHERN MILWAUKEE COUNTY: 1986

Category	Beach Composition		Shoreline Length (feet)	Percent of Total Shoreline Length
	Surface 0 to 6 Inches	Subsurface 7 to 12 Inches		
I	Sand	Sand	1,700	4.4
II	Sand	Sand and gravel	780	2.0
III	Sand	Gravel	700	1.8
IV	Sand and gravel	Sand	2,740	7.2
V	Sand and gravel	Sand and gravel	3,980	10.2
VI	Gravel	Sand	360	0.9
VII	Gravel	Sand and gravel	1,620	4.2
VIII	Sand, gravel, and cobbles	Sand and gravel	200	0.5
Shoreline Area with a Beach Width 10 Feet or Less			26,690	68.8
Total			38,770	100.0

Source: SEWRPC.

that in 1871 and 1912 the near-shore slopes were more gentle than those surveyed in 1986. The bathymetric survey results in 1944 were similar to the 1986 conditions, although the 1986 gentle-sloped areas were even more gentle in 1944, while some 1986 steeper-sloped areas were even more steep. Because of the high water levels in 1985 and 1986, and the declining availability of littoral drift as more shore protection structures are installed, it is expected that, in general, the near-shore zone will become somewhat steeper in the future unless measures such as beach nourishment are implemented.

Groundwater Resources

The occurrence, distribution, direction, and quantity of groundwater flow have important impacts on the stability of the bluff slopes. Along the northern 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.

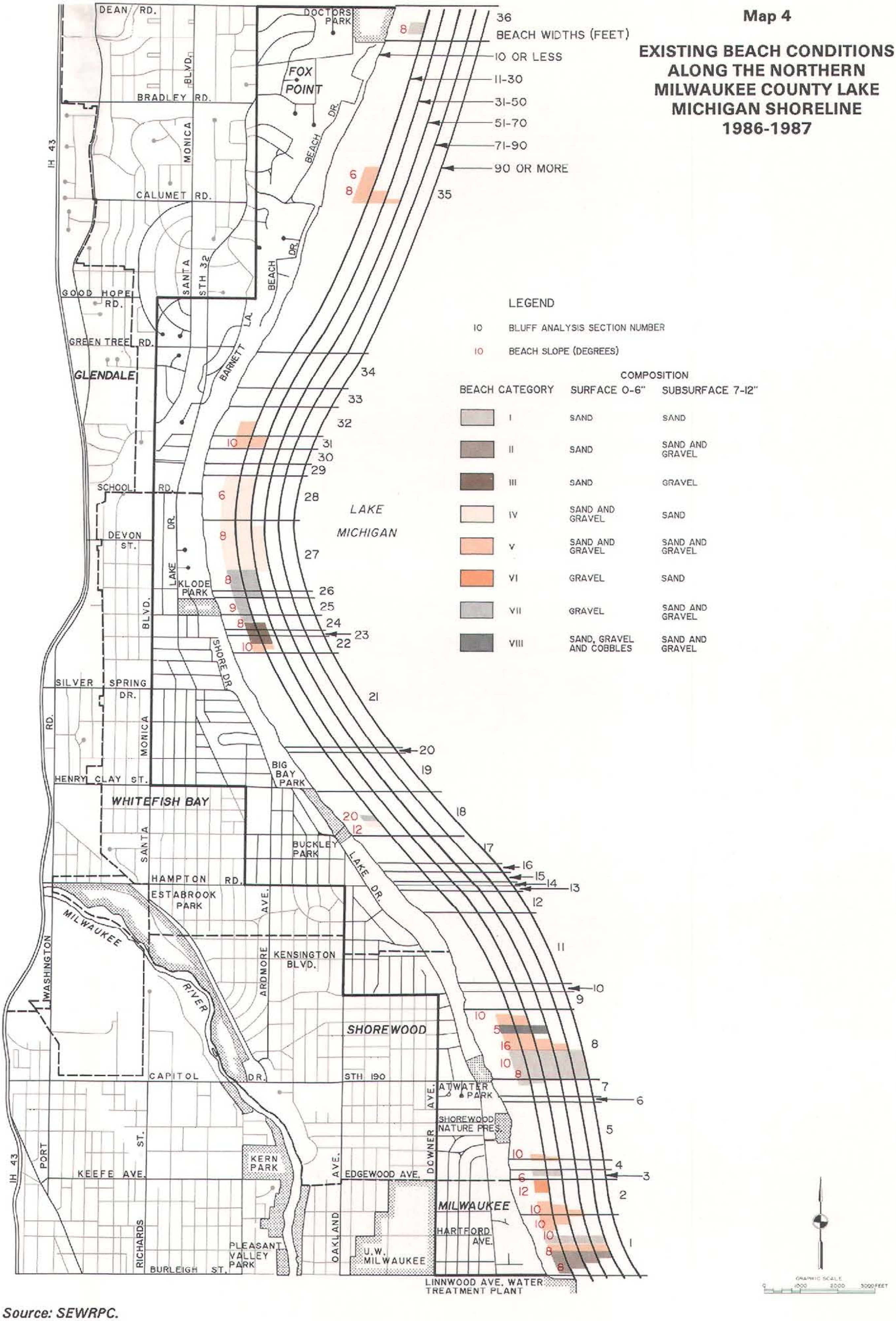
There are two major aquifers beneath the northern Milwaukee County study area. These aquifers are commonly called the "deep sandstone" aquifer and the "shallow limestone" aquifer. The aquifers differ widely in water yield capabilities and extend to great depths.

The deep sandstone aquifer, which is known to be 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, also 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 level of Lake Michigan. Recharge of this aquifer is by the downward seepage of precipitation which falls within, and

Map 4

EXISTING BEACH CONDITIONS ALONG THE NORTHERN MILWAUKEE COUNTY LAKE MICHIGAN SHORELINE 1986-1987



Source: SEWRPC.

Table 6

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

Average Beach Slope (degrees)	Length of Shoreline (feet)	Percent of Study Area Shoreline Length
No Significant Beach[a]	26,690	68.8
0 - 3	0	- -
4 - 6	4,080	10.5
7 - 9	2,730	7.0
10 - 12	4,440	11.5
13 - 15	620	1.6
> 15	210	0.6
Total	38,770	100.0

[a]*Beach width of 10 feet or less.*

Source: SEWRPC.

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 up to 150 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 northern Milwaukee County bluffs. Estimated groundwater levels for the study area were based on either field observations or soil boring or well data, or were determined using electrical resistivity methods.

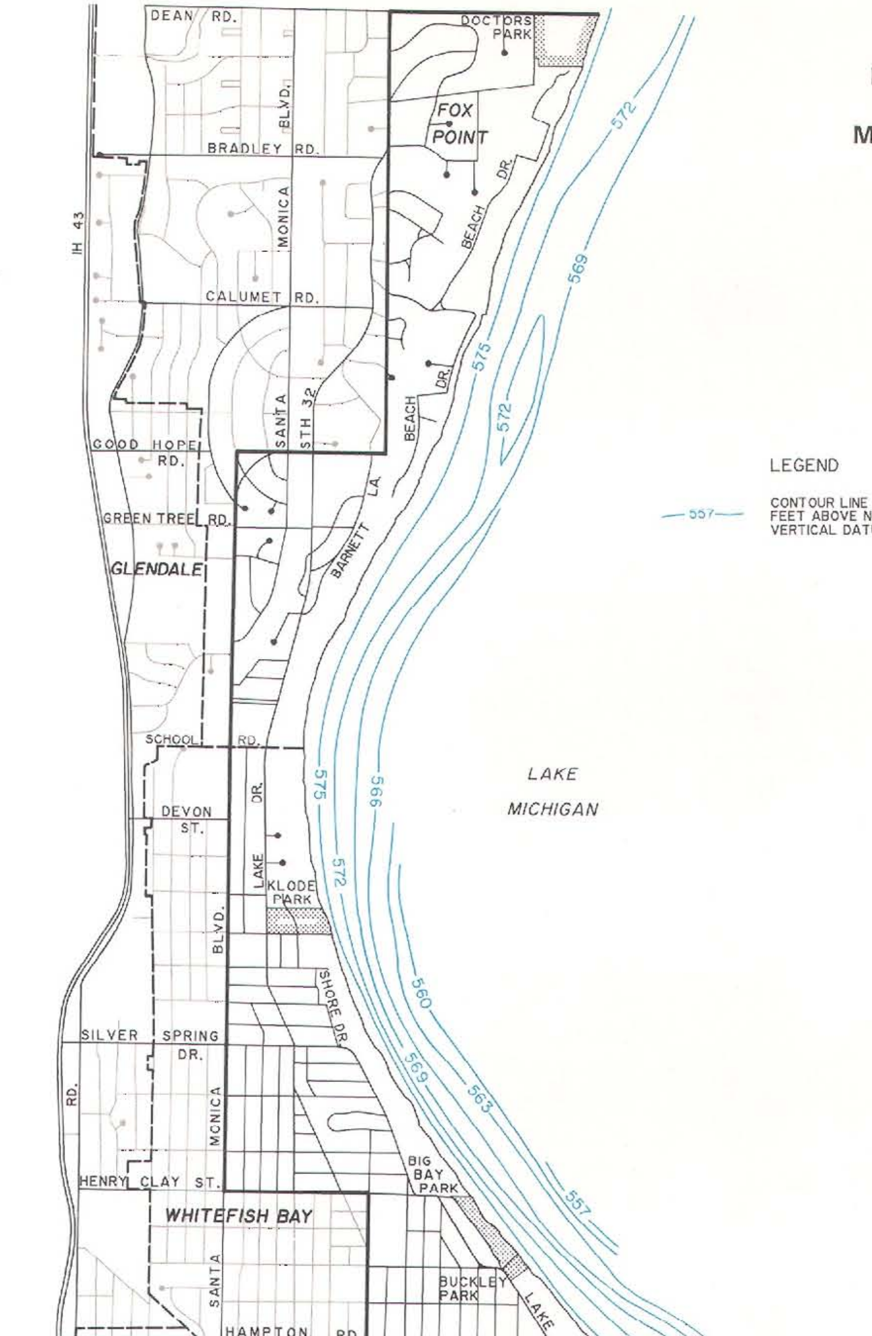
As shown on Map 6, there were eight locations where the level of the water table was identified by observation of groundwater seepage in May 1986. Most of these seepage zones were observed in the northern portion of the study area from E. Lakeview Avenue north to the southern end of N. Barnett Avenue.

As already noted, nine soil borings were taken 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 October and November 1986—the depth to the water table, including any perched water tables, was identified. At two of the soil boring sites, groundwater observation wells were installed by the property owners. The depth to the water table in these wells was measured on a regular basis. The locations of the new soil boring sites and the observation wells are shown on Map 6.

The depths to the water tables were also determined using electrical resistivity methods in October and November 1986. Measurements were made at 10 locations along the study area shoreline, as shown on Map 6, where existing groundwater data were not available and where it was thought the level of the water table could influence the overall stability of the bluff slope. The resistivity analyses were conducted by Dr. William F. Kean, Professor of Geological Sciences at the University of Wisconsin-Milwaukee. Electrical resistivity methods are often used in groundwater studies as an aid in locating the water table. 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 material, its porosity, the pore fluid conductivity, and the degree of saturation. Resistivity changes with depth are monitored by varying the spacing between the electrodes. For this study, the electrodes were placed on lines parallel to the bluff edge and at a distance of 30 to 100 feet from the bluff edge. Table 7 presents estimated resistivity values for selected bluff materials. In general, dry bluff materials have high resistivity values.

Map 5

NEAR-SHORE BATHYMETRY OF LAKE MICHIGAN ALONG THE NORTHERN MILWAUKEE COUNTY SHORELINE: 1986



LEGEND

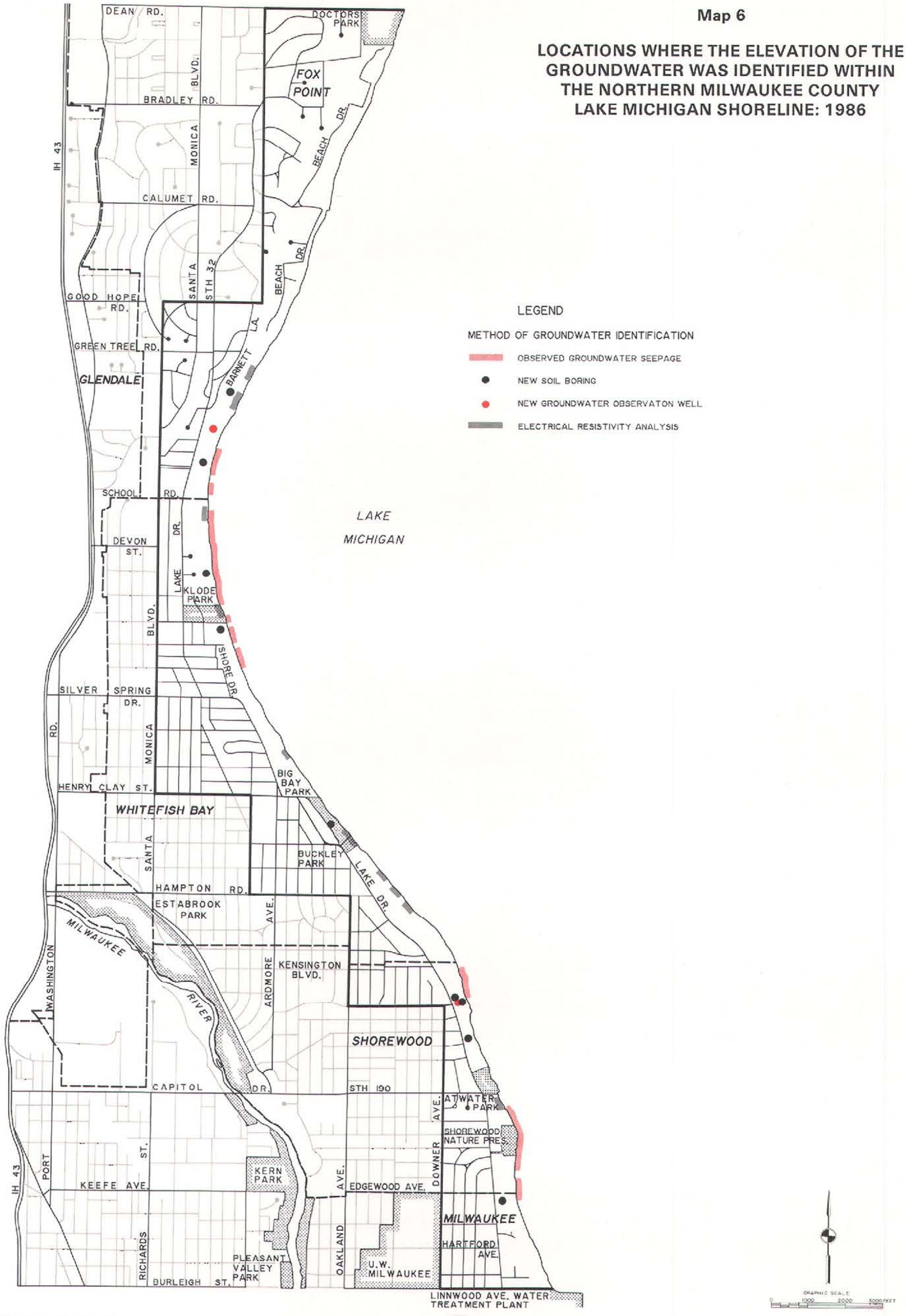
557 CONTOUR LINE AND ELEVATION IN FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

GRAPHIC SCALE

Source: Warzyn Engineering, Inc.

Map 6

LOCATIONS WHERE THE ELEVATION OF THE GROUNDWATER WAS IDENTIFIED WITHIN THE NORTHERN MILWAUKEE COUNTY LAKE MICHIGAN SHORELINE: 1986



Source: SEWRPC.

Table 7

ESTIMATED ELECTRICAL RESISTIVITY VALUES OF SELECTED BLUFF MATERIALS WITHIN NORTHERN MILWAUKEE COUNTY

Resistivity (ohm-feet)	Bluff Material
1 - 39	Saturated sand
40 - 65	Partially saturated sand and silt and fractured clayey tills
66 - 164	Surface soils
165 - 320	Moist, clayey tills and dry sand and silt
321 - 6,500	Dry, clayey tills and bedrock

Source: W. F. Kean.

Based on the results of the groundwater seepage observations, soil borings, groundwater observation wells, and electrical resistivity analyses, three general groundwater systems were identified within northern Milwaukee County bluffs. A perched water table was usually found within the fractured Ozaukee till. A second water table was generally located within a lake sediment layer lying between Ozaukee till and Oak Creek till. A third water table was generally found at about the level of Lake Michigan—usually in Oak Creek or New Berlin till. In some sections, the entire bluff beneath the second water table was saturated; in other sections, a layer of unsaturated till separated the second water table from the third water table.

Control of groundwater and seepage conditions within bluff slopes by drainage may improve stability. Groundwater drainage methods that have been used within the study area include drainage tunnels, well systems, and horizontal drains. The following is a description of the known groundwater drainage systems installed within northern Milwaukee County, along with an assessment of their effectiveness in improving the stability of the bluff slopes.

1. 5350-5415 N. Lake Drive, Whitefish Bay. A drainage tunnel 4 feet wide and 6 feet high was constructed along 530 linear feet of shoreline in 1932.[2] The tunnel was designed to help control a massive, slow-moving slide which was believed to have been due to the accumulation of hydrostatic pressure by groundwater, and which covered about 3,000 feet of shoreline and extended 20 feet below lake level.

   The tunnel was driven into the face of the bluff to a point beneath the east side of Lake Drive. The tunnel then branched north and south to protect a total of five properties. As shown in Figure 3, the tunnel was driven into the middle of a water-bearing sand stratum approximately 10 feet below the water level previously recorded in test holes. In order to relieve the hydrostatic pressure at the bottom of the sand, holes were drilled in the tunnel floor, and sections of 12-inch-diameter sewer pipe were placed into the sand. Similar drainage holes 24 inches in diameter were drilled in the drainage tunnel beneath manholes located at the northern and southern ends of the tunnel. A 24-inch-diameter reinforced concrete pipe was laid with open joints on the bottom of the tunnel, and was covered with pea gravel. A cast-iron pipe outlet discharged into the lake.

   Immediately following the completion of the tunnel, the water levels in test borings reportedly dropped several feet. The drainage system reportedly reduced the slope failure and continued to operate until approximately 1960 when the outfall was damaged, preventing any further discharge. In the 1970's, fill was placed on the toe and face of the bluff by Foundation Engineering, Inc., providing substantial protection against further shore erosion and slope failure.

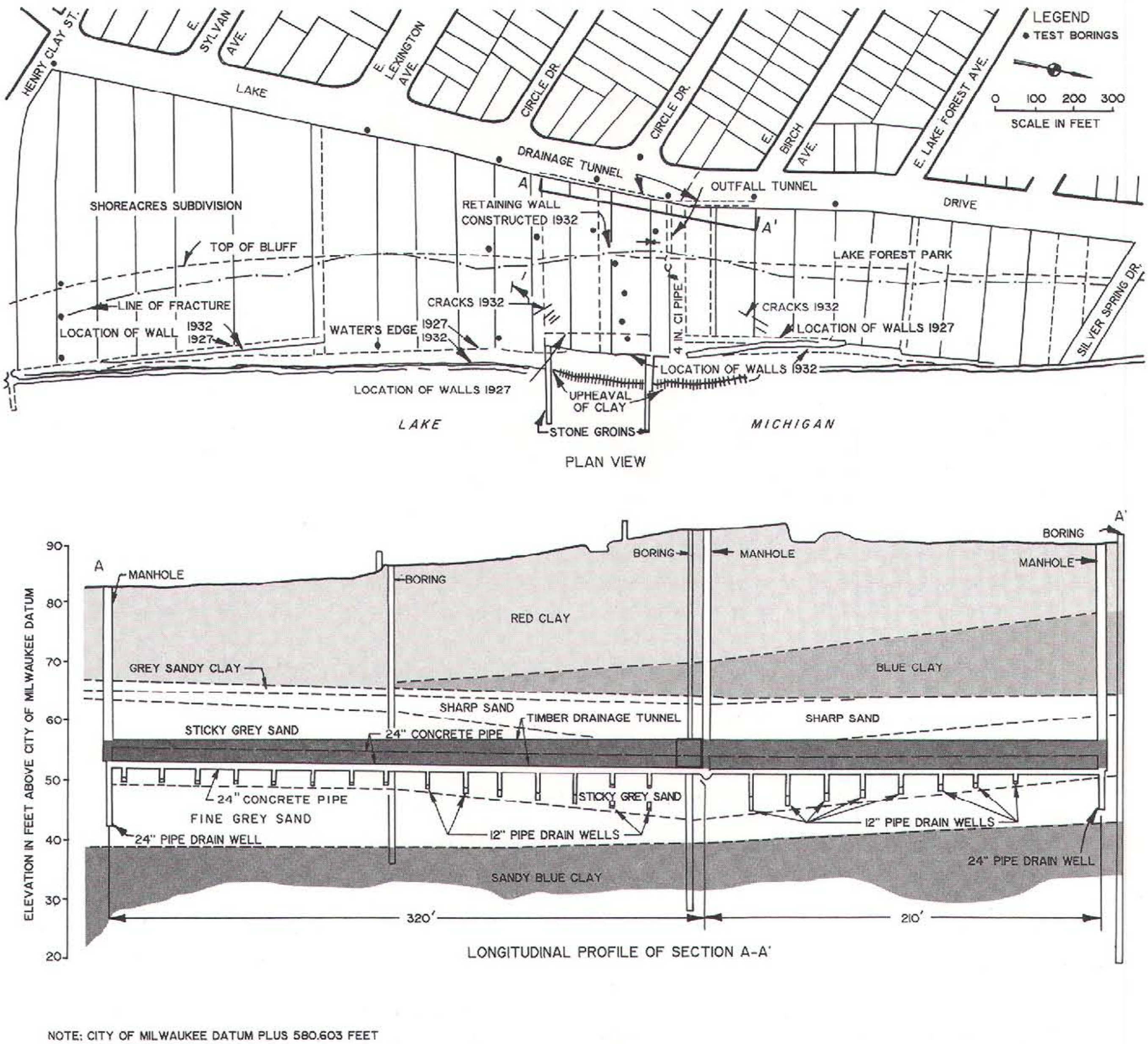
2. 4430 N. Lake Drive, Shorewood. In 1966, a drainage well was installed on this property to reduce the groundwater level within

---

[2] *C. S. Whitney, "Stabilizing a Lake Michigan Bluff, Construction of Drainage Tunnel Relieves Hydrostatic Pressure and Stops Sliding," Civil Engineering, Vol. 6, No. 5, May 1936, pp. 303-319.*

Figure 3

PLAN VIEW AND LONGITUDINAL PROFILE OF THE GROUNDWATER DRAINAGE TUNNEL INSTALLED ALONG 5350-5415 N. LAKE DRIVE, WHITEFISH BAY: 1932



Source: C. S. Whitney.

a relatively thin water-bearing layer, and thereby improve the stability of the bluff slope. The well was installed on the west side of the house about 100 feet from the edge of the bluff. As shown in Figure 4, installation of the well required a soil boring ranging in diameter from 10 to 16 inches and extending to a depth of approximately 78 feet. A six-inch-diameter casing was placed in the well and backfilled with gravel up to a depth of three-and-one-half feet below the surface. The gravel was then covered with a six-inch layer of bentonite, and capped with three feet of concrete. A one-half horse-power submersible pump was installed, and controlled by two sensors placed at depths of 64 and 70 feet. Thus, the well was designed to lower the level of the groundwater a maximum of six feet. From one to 10 gallons per day were pumped from the well. The drainage well was still operating in 1986.

This small pumpage rate apparently reduced the seepage rate within the thin water-bearing layer. While this groundwater drainage system may have significantly reduced the rate of bluff slope failure, the bluff nevertheless continued to recede, and in 1986, Coast-Tec Construction Company, Ltd., was in the process of constructing bluff toe protection, as well as placing fill on the lower portion of the face of the bluff.

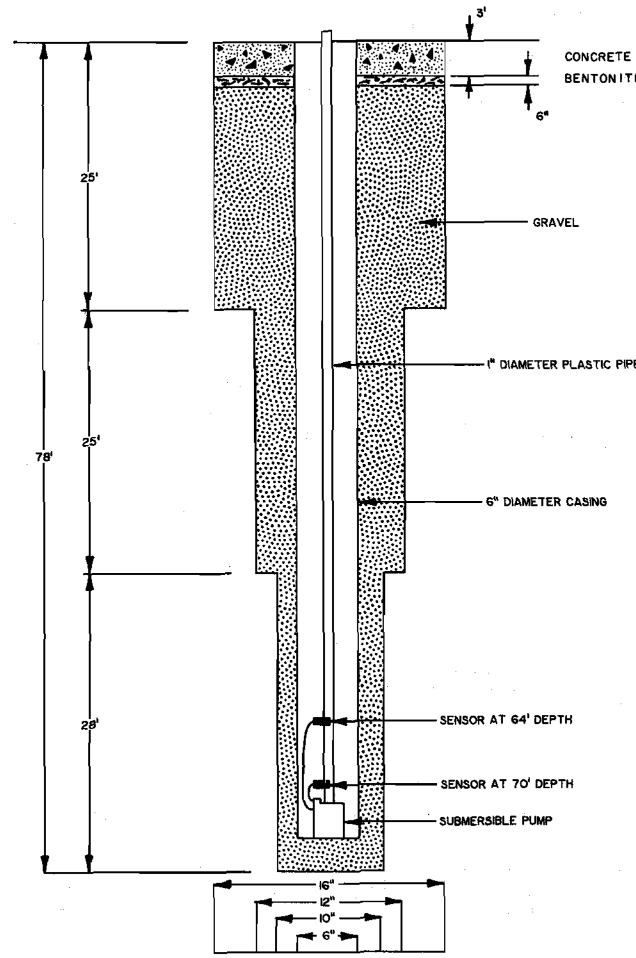
3. 4920 N. Lake Drive, Whitefish Bay. In 1968, three drainage wells were installed 25 to 35 feet from the edge of the bluff to reduce excessive groundwater seepage from the face of the bluff. One well was drilled to a depth of 64 feet and the other two wells extended between 40 and 50 feet deep. Installation of the wells required three-foot-diameter soil borings. Twenty-four-inch-diameter slotted casings were placed in the wells, which were then backfilled with gravel. Water was pumped from the wells by submersible electric pumps controlled by sensors. Drain tiles were used to discharge the well water to a storm sewer located in Lake Drive. The deeper well pumped 24 hours per day, and the shallower wells pumped intermittently. After the construction of the wells, water continued to seep out of the face of the bluff, even though the wells intercepted a large amount of water.

   Fill was placed on the face of the bluff in 1975 by Henry L. Munch Co., Inc. The fill was composed of concrete rubble, overlain by clay. A second fill was placed at this site in 1976 by Foundation Engineering, Inc. Since the completion of the fill projects, minor seepage has continued, but it has not had an apparent effect on the stability of the slope.

4. 4620 N. Lake Drive, Whitefish Bay. A drainage system was installed in the early 1980's with the intention of draining surface water to a ravine north of the property rather than allowing it to seep into the ground or flow over the top of the bluff. Surface water runoff from several properties to the south also drained toward the site. The drainage system consisted of several shallow holes ranging in depth up to 16 feet, with openings at the surface. The collected water was conveyed to the ravine. The drainage system did not apparently reduce the groundwater level, and in 1986 fill was being placed on the face of the bluff by Shoreline of Wisconsin, Inc.

Figure 4

DRAINAGE WELL AT 4430 N. LAKE DRIVE, SHOREWOOD: 1966



Source: University of Wisconsin-Milwaukee

5. 4700 N. Lake Drive, Whitefish Bay. A drainage system was installed in the early 1980's to intercept surface runoff which flows over the top of the bluff and infiltrates into the ground surface, subsequently seeping from the face of the bluff. The drainage system consisted of roof

gutter drains, an eight-foot-deep footing tile which drains the near-surface groundwater at the house, three surface collection grills, a 10-foot-deep catch basin with outfall pipes extending down to the lake, and a five-foot-deep French drain filled with concrete which extends from the north side of the house diagonally to the catch basin.

While effectively draining surface water, the drainage system has had a limited effect on groundwater seepage. Such seepage has been observed by the property owner after large storm events, although little seepage has been noted during dry-weather conditions.

6. Klode Park, Whitefish Bay. A groundwater drainage system was installed at the footing of the North Shore Water Commission pumping station at Klode Park in about 1962. The interceptor, constructed of perforated Armco pipe, discharged the water at the base of the bluff. This drainage system may have helped lower the groundwater level. Both the field surveys and the electrical resistivity analyses conducted in 1986 under this study indicated that the water table was located near the foot of the pumping station. During a December 1986 storm, a portion of the bulkhead in Klode Park collapsed and the lower portion of the bluff slumped into the lake. A second, more massive slump occurred north of the raw water intake pumping station in April 1987. Excessive groundwater seepage, combined with previous slumping at the toe, may have contributed to the April slide.

Climate

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

Air temperature impacts primarily include the formation of ice on the lake, the initiation of freeze-thaw actions on soils, and high stormwater runoff rates from frozen soils. Table 8 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, average from 64.9° to 70.3° F.

Table 8

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	37.7	46.1

Source: National Weather Service and SEWRPC.

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

Precipitation within the study area takes the form of rain, sleet, hail, and snow, and ranges from gentle showers of trace quantities to brief but intense and potentially destructive thunderstorms or major rainfall-snowmelt events causing severe bluff and beach erosion. Average monthly and annual total precipitation and snowfall for the Milwaukee National Weather Service station are presented in Table 9. 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 9

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

Month	Average Total Precipitation (inches)[a]	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

[a]*Expressed as water equivalent.*

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.[3] A one-hour, 10-year recurrence interval storm may be expected to have a total rainfall of about 1.8 inches; and a 24-hour, 10-year recurrence interval storm may be expected to have a total rainfall of about 3.7 inches. Extended wet periods may result in unusually high coastal losses. Over the period 1841 through 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.[4] The maximum monthly precipitation amount was 10.83 inches, which occurred in June 1917. In late 1986, unusually high levels of precipitation occurred in Milwaukee and throughout the Lake Michigan drainage area, resulting 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.

---

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

[4]*National Weather Service, Wisconsin Statistical Reporting Service, and SEWRPC.*

The presence of Lake Michigan tends to moderate the climate of the northern Milwaukee County study area. 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 midday summer temperatures to be about 10° F lower in shoreline areas than in inland areas because of the cooling lake breezes. Lake Michigan does not have as pronounced an effect on precipitation as it does on temperature. A minor Lake Michigan effect is apparent in the late spring and summer, when there is about 0.5 inch less rainfall per month in coastal areas than in areas farther inland. This difference may be attributed to the cool lake waters maintaining a cooler lower atmosphere which inhibits convective precipitation. However, during the winter, Lake Michigan can serve as a source of moisture, resulting in slightly higher snowfalls near the lake.

MAN-MADE FEATURES

An understanding of the existing civil divisions, land use patterns, and zoning regulations is essential to the formation of practical shoreline management guidelines for the coastal area experiencing shoreline erosion. Accordingly, this section describes the existing civil divisions, land use, and zoning within the study area.

Civil Divisions

Local civil division boundaries within the study area are shown on Map 7. The study area, which lies entirely within Milwaukee County, contains portions of the City of Milwaukee and the Villages of Shorewood, Whitefish Bay, and Fox Point. The area and proportion of each municipality within the study area in 1986, 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 10. As indicated in Table 10, the Village of Fox Point encompassed 39 percent of the study area, or 672 acres, and accounted for 38 percent of the Lake Michigan shoreline within the study area, or 14,580 feet. The Village of Whitefish Bay occupied 41 percent of the study area, or 701 acres, and accounted for 38 percent of the shoreline, or 14,680 feet. The Village of Shorewood occupied 12 percent of the study area, or 211 acres, and accounted for 17 percent of the shoreline, or 6,590 feet. The City of Milwaukee accounted for the remaining 8 percent of the study area, or 141 acres, and contained the remaining 7 percent of the northern Milwaukee County Lake Michigan shoreline, or 2,920 feet.

Existing Land Use

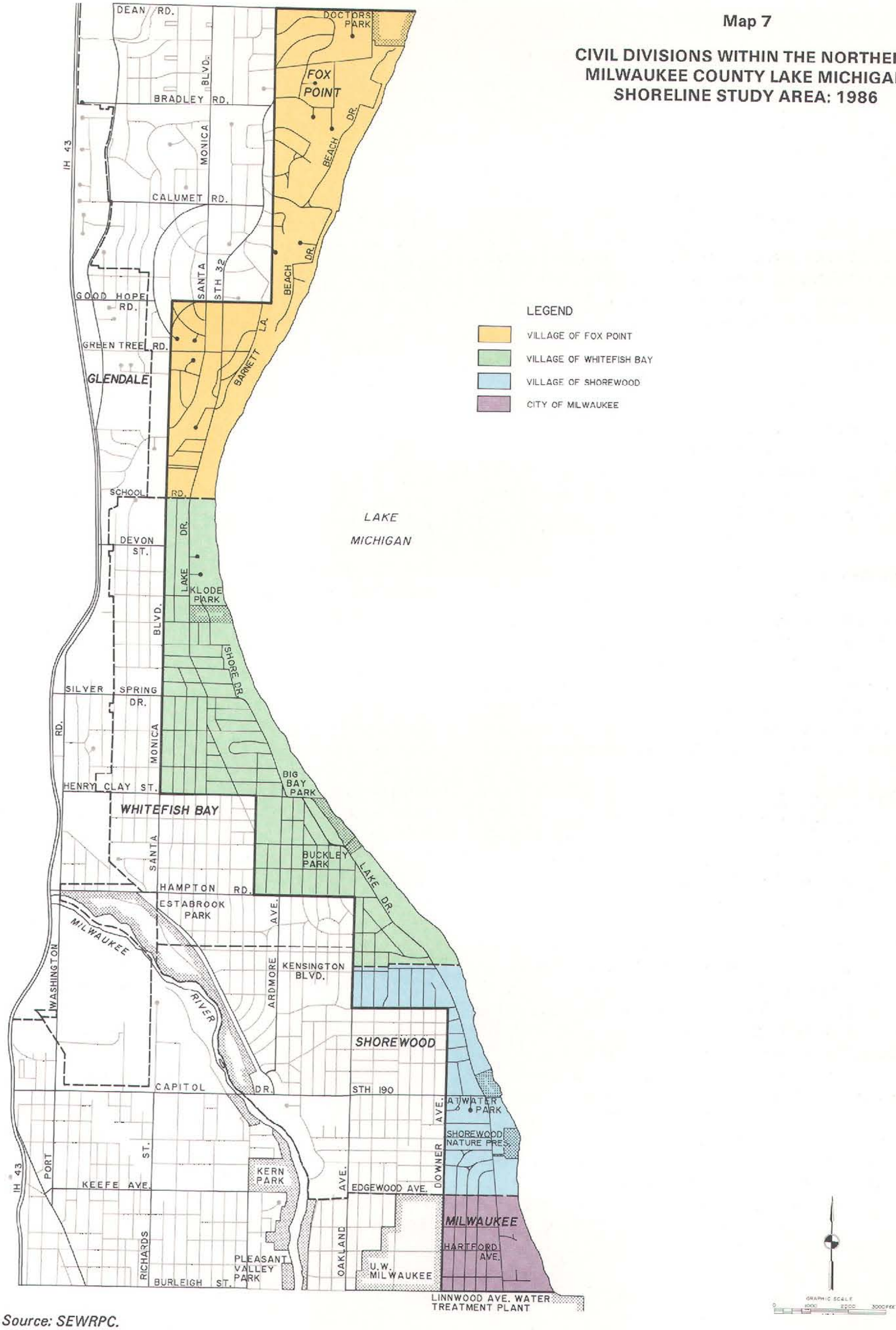
The type and spatial distribution of the major categories of land use existing within the coastal erosion study area of northern Milwaukee County in 1985 are shown on Map 8. The areal extent of the various major categories of land use within the shoreland study area, which encompasses a total of 1,726 acres, is presented in Table 11. As shown on Map 8 and indicated in Table 11, a significant portion of the study area, 1,448 acres, or 84 percent, was devoted to intensive urban uses in 1985, including residential, transportation and utility, governmental and institutional, and commercial uses. Of these urban land uses, residential uses constituted the largest proportion—1,066 acres, or 74 percent of the developed urban area. Recreational uses constituted an additional 33 acres, or 2 percent of the total area. Remaining undeveloped lands, including unused urban land and woodlands, encompassed 245 acres, or 14 percent of the total study area.

Existing Zoning

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 four civil divisions which have jurisdiction in the Lake Michigan coastal erosion study area in northern Milwaukee County. Areas likely to be affected by amendments to existing zoning ordinances which would regulate, in the public interest, land uses in relation to the risk of shoreline erosion and bluff recession are shown on Map 9. In the City of Milwaukee, this area includes the land east of N. Lake Drive. In the Villages of Shorewood and Whitefish Bay, this area includes that land designated in the Lake Drive Residential District and the Lake Shore Residential District, respectively. In the Village of Fox Point, this area generally includes the land classified in the A-1 and B Residential Districts east of N. Lake Drive within the study area. Table 12 summarizes some of the pertinent regulations set forth in the local zoning codes governing the affected zoning districts.

Map 7

CIVIL DIVISIONS WITHIN THE NORTHERN MILWAUKEE COUNTY LAKE MICHIGAN SHORELINE STUDY AREA: 1986



Source: SEWRPC.

Table 10

AREA AND SHORELINE LENGTH OF CIVIL DIVISIONS WITHIN THE NORTHERN MILWAUKEE COUNTY LAKE MICHIGAN SHORELINE STUDY AREA: 1986

Civil Division	Area (acres)	Percent of Study Area	Lake Michigan Shoreline Length (feet)	Percent of Shoreline
City of Milwaukee	141.1	8.2	2,920	7.5
Village of Shorewood	211.1	12.2	6,590	17.0
Village of Whitefish Bay	701.3	40.6	14,680	37.9
Village of Fox Point	672.4	39.0	14,580	37.6
Study Area Total	1,725.9	100.0	38,770	100.0

Source: SEWRPC.

Table 11

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

	Village of Fox Point		Village of Shorewood		Village of Whitefish Bay		City of Milwaukee		Total Study Area	
Land Use Category	Acres	Percent of Total	Acres	Percent of Total	Acres	Percent of Total	Acres	Percent of Total	Acres	Percent of Total
Residential	431.1	64.1	136.2	64.5	418.5	59.7	80.0	56.7	1,065.8	61.8
Land Under Urban Development	- -	- -	0.2	0.1	- -	- -	0.9	0.6	1.1	0.1
Commercial	1.5	0.2	0.7	0.3	8.0	1.2	0.6	0.4	10.8	0.6
Transportation and Utilities	101.8	15.2	43.1	20.4	151.6	21.6	28.8	20.4	325.3	18.8
Governmental and Institutional	4.7	0.7	- -	- -	32.3	4.6	8.3	5.9	45.3	2.6
Recreational	13.5	2.0	6.5	3.1	12.6	1.8	- -	- -	32.6	1.9
Unused Urban Land	17.0	2.5	2.1	1.0	56.3	8.0	- -	- -	75.4	4.4
Woodlands	102.8	15.3	22.3	10.6	22.0	3.1	22.5	16.0	169.6	9.8
Total	672.4	100.0	211.1	100.0	701.3	100.0	141.1	100.0	1,725.9	100.0

Source: SEWRPC.

COASTAL EROSION PROCESSES

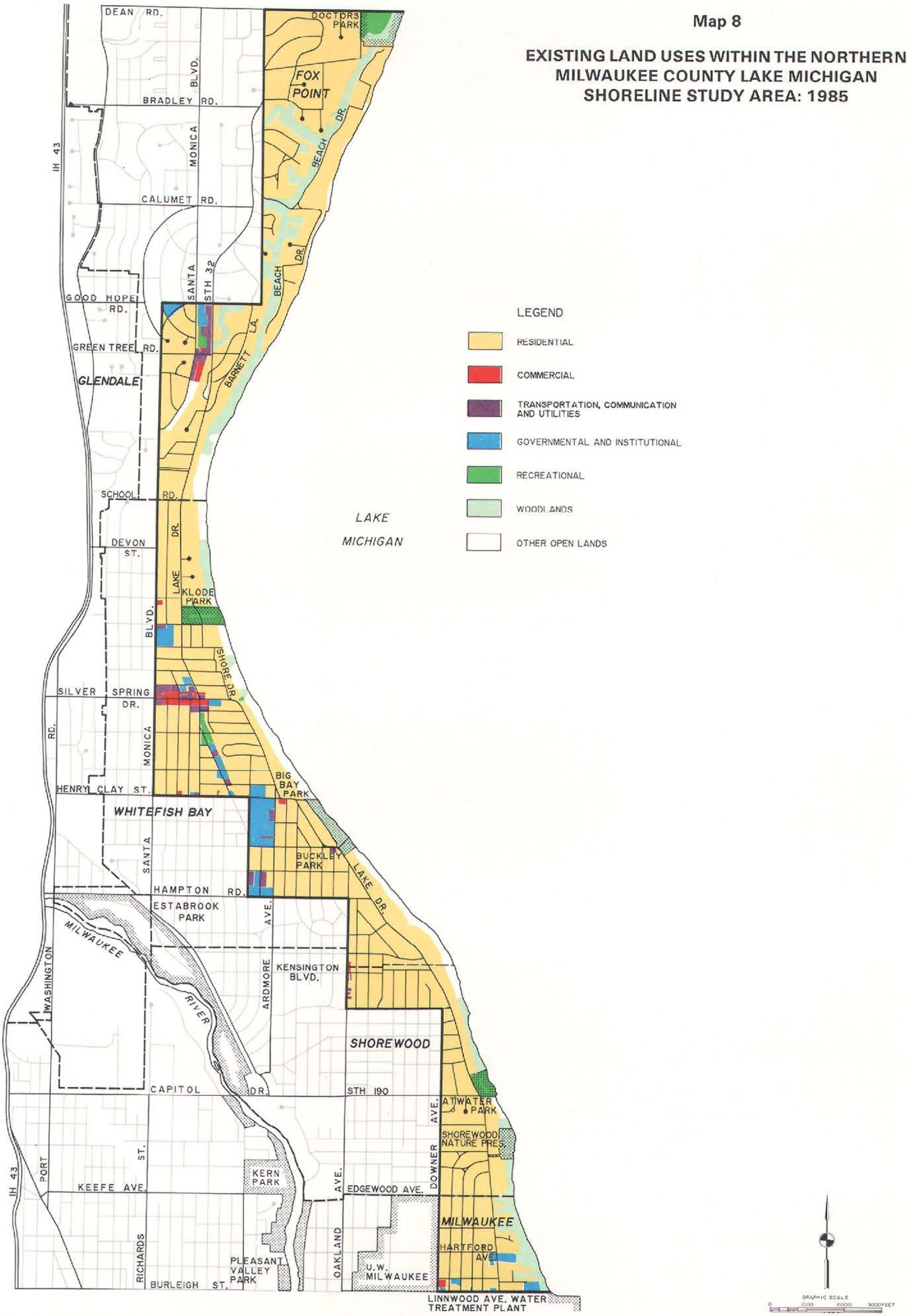
Erosion of the Lake Michigan shoreline is a natural process which can be accelerated—such as by increasing the rate and volume of stormwater runoff—or decelerated—such as by the construction of shore protection measures—by human activities. Shoreline erosion includes two processes: bluff erosion and beach erosion. Various factors contribute to bluff erosion and beach erosion, including 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

Map 8

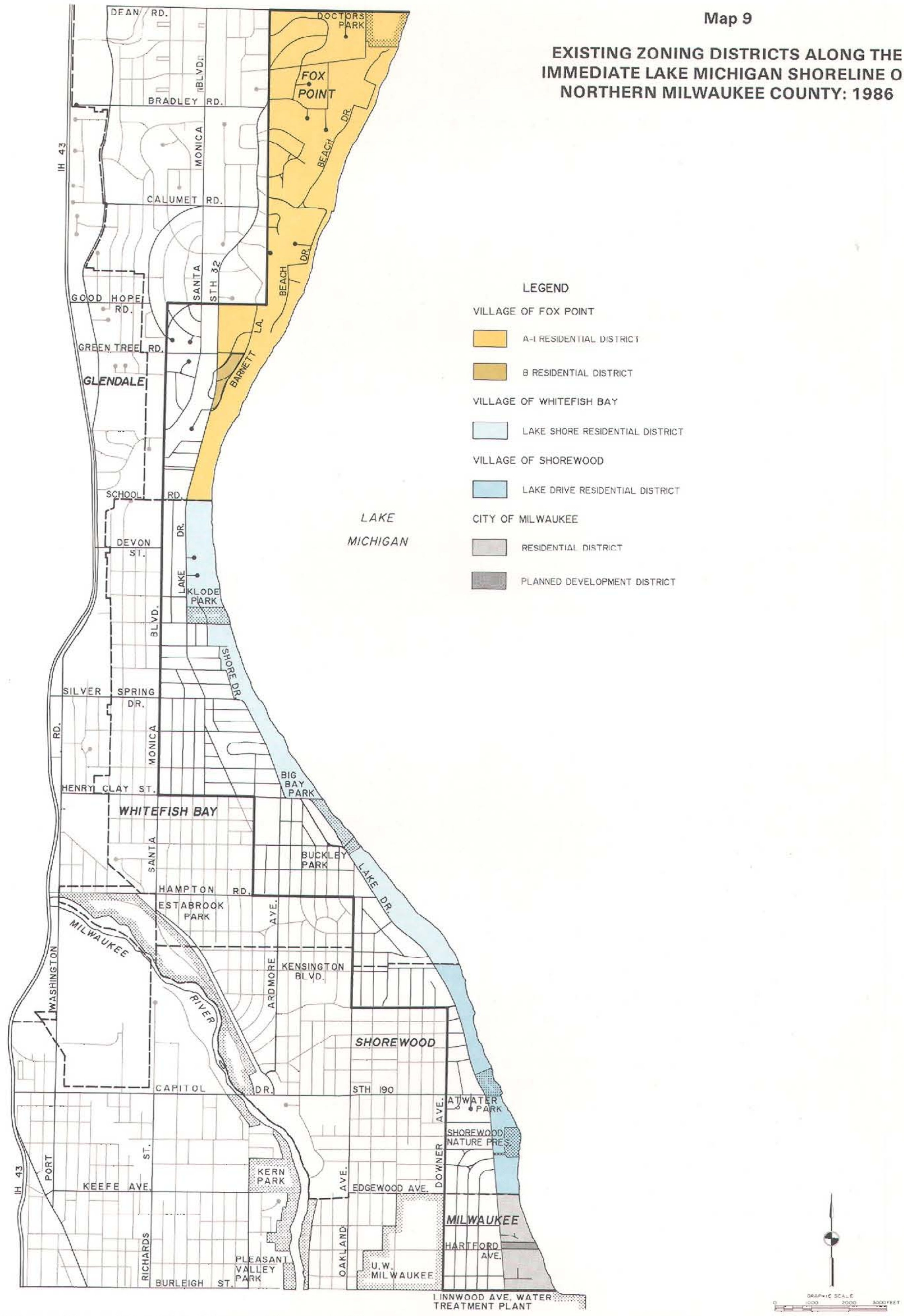
EXISTING LAND USES WITHIN THE NORTHERN MILWAUKEE COUNTY LAKE MICHIGAN SHORELINE STUDY AREA: 1985



Source: SEWRPC.

Map 9

EXISTING ZONING DISTRICTS ALONG THE IMMEDIATE LAKE MICHIGAN SHORELINE OF NORTHERN MILWAUKEE COUNTY: 1986



Source: Village of Fox Point, Village of Shorewood, Village of Whitefish Bay, City of Milwaukee, and SEWRPC.

Table 12

SUMMARY OF SELECTED EXISTING ZONING REGULATIONS IN THE NORTHERN MILWAUKEE COUNTY LAKE MICHIGAN SHORELINE STUDY AREA

Civil Division	Zoning District	Permitted Uses	Conditional Uses	Minimum Lot Area (square feet)	Minimum Lot Width (feet)	Minimum Front Setback (feet)	Minimum Side Setback (feet)	Minimum Rear Setback (feet)
City of Milwaukee	R/F 1 Single-Family Residential District	Single-family dwellings, family day care homes, convents, churches, elementary and secondary schools, colleges, governmental structures, public parks and playgrounds, telephone central offices, transmitter towers, farming truck gardening, nurseries or greenhouses	More than one principal residential building per lot, community living arrangements, commercial uses in public parks and playgrounds	6,000	50	25	6	25
	PD Planned Development District	- -	- -	14,000	- -	25	25	25
Village of Shorewood	Lake Drive Residential District	Single-family dwellings; noncommercial greenhouses, nurseries and gardens; private garages	- -	7,000	60	24	5	- -
Village of Whitefish Bay	Lake Shore Residential District	Single-family dwellings; noncommercial greenhouses, nurseries and gardens; private garages	- -	9,600	80	- -	3	16
Village of Fox Point	A-1 Residential District	Residential dwellings; accessory uses	- -	40,000	120	30	20	20
	B Residential District	Residential dwellings; accessory uses	- -	15,000	80	30	10	20

Source: City of Milwaukee, Village of Shorewood, Village of Whitefish Bay, Village of Fox Point, and SEWRPC.

slopes gravity acts to move material on the slope to a lower elevation. On most slopes that 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 northern 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 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 and very wet conditions occur. 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. Sapping can result in the collapse of overlying materials. The sapping results from the seepage of groundwater from permeable seams exposed in the bluff face or from extrusions of water-saturated plastic sediment in exposed seams.

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, that 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 caused by 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 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.

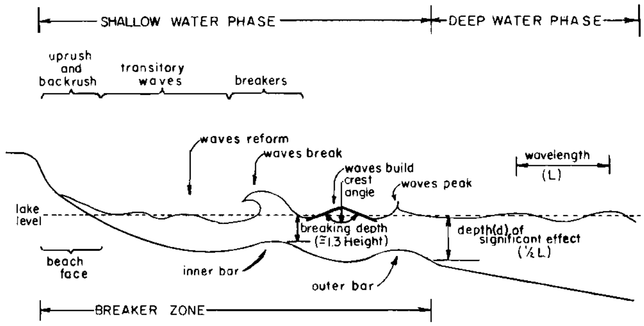
A fifth type of slope failure is rock or soil fall. This type of failure takes place when undercutting is extreme and near-vertical cliffs are produced. Even though some such segments of bluff are present along the northern Milwaukee County shoreline, these are generally small, and rock or soil fall 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.

<u>Wave Action</u>: Several factors affect the type of slope failure that occurs and the severity of that failure. The physical characteristics of the beach and bluff have a major influence on the resistance of the slope to failure. Numerous other factors affect the external stresses 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 5 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.

Figure 5

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.

Table 13

SUMMARY OF HOURLY WIND DIRECTION AT MILWAUKEE: 1975-1984

Wind Direction	Total Number of Hours	Percent of Total Hours
North	5,200	14.0
Northeast	11,300	30.4
East	5,100	13.7
Southeast	11,600	31.2
South	4,000	10.7
Total	37,200	100.0

Source: W. F. Baird & Associates Coastal Engineers, Ltd.

Waves may be characterized by their height, period or frequency, and length. Knowledge of these wave characteristics is necessary in order to predict wave energy impact against the beach and bluff, and to design shore protection structures properly. In deep water, the major determinants of wave height are wind speed, wind duration, and fetch length. In shallow water, wave height is primarily determined by the height of the incoming deep-water waves and by the water depth. Wave period is defined as the time which elapses between two successive wave crests passing a fixed point. Wave length is defined as the distance between the crests of two successive waves and is determined by wind speed, wind duration, and water depth.

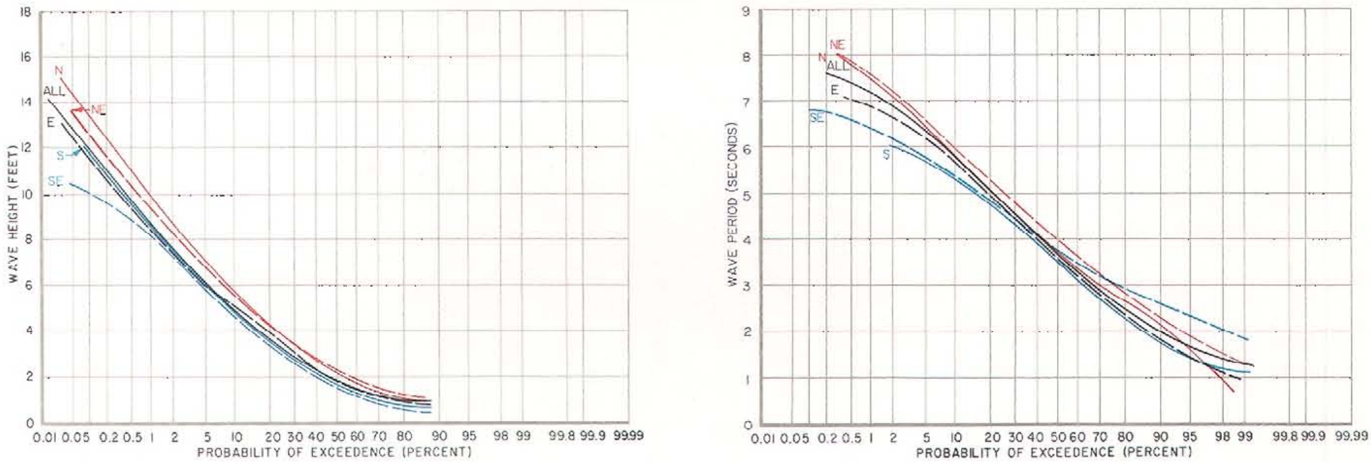
The degree of wave energy affecting toe erosion is related to the slope of the beach and offshore areas, the orientation of the beach in relation to storm wind and waves, the lake distance over which waves can develop, and the elevation of the water surface relative to the elevation of the base of the bluff.

A summary of deep-water wave conditions for Milwaukee was compiled by W. F. Baird & Associates, Coastal Engineers, Ltd., in 1986 for the northern Milwaukee County offshore erosion control study. Hourly values of wind speed and direction recorded at Milwaukee over the period 1975 to 1984 were used to predict the occurrence of various wave heights and periods for the northern Milwaukee County study area. Table 13 summarizes hourly wind direction over the 10-year period, and Table 14 illustrates the fetch—that is, the length of water over which the wind can blow unhindered—for the various wind directions. As indicated in Table 13, the most frequently occurring waves offshore of Milwaukee are from the southeast and the northeast, occurring 31 and 30 percent of the time, respectively. Waves from the northeast have the largest fetch—approximately 250 miles, and may be associated with the most damaging storms. Figure 6 sets forth predicted Lake Michigan deep-water wave conditions for each wind direction. The figure shows the percent of time a given wave height or wave period is likely to be exceeded based on wind data from 1974 to 1984.

Figure 7 presents deep-water wave height and wave period estimates at Milwaukee for various recurrence interval storm events. The reciprocal of the recurrence interval is the likelihood of that storm event occurring in any given year. For example, a 20-year recurrence interval storm event has a 5 percent chance of occurring during any given year. That same storm event has a 40 percent chance of occurring in any 10-year period, a 72 percent chance of occurring in any 25-year period, and a 92 percent chance of occurring in any 50-year period. Deep-water wave heights can range up to 25 feet, and wave periods may range up to 12 seconds during major storm events. In general, the largest

Figure 6

MILWAUKEE DEEP WATER WAVE HEIGHT AND PERIOD PROBABILITY OF EXCEEDENCE: WAVE DIRECTION FROM THE NORTH, NORTHEAST, SOUTH, SOUTHEAST, EAST, AND ALL



Source: W. F. Baird & Associates Coastal Engineers, Ltd., and SEWRPC.

Table 14

LAKE MICHIGAN FETCHES USED FOR DEEP WATER WAVE HINDCASTING

Direction	Angle from Due North (degrees)	Fetch (miles)
North	10	0
North of Northeast	11	100
North of Northeast	23	250
Northeast	45	190
East of Northeast	68	90
East	90	90
East of Southeast	113	110
Southeast	135	110
South of Southeast	158	110
South of Southeast	169	100
South	170	0

Source: W. F. Baird & Associates, Coastal Engineers, Ltd.

storm-generated waves are most likely to occur during early winter, and least likely to occur during spring.[5]

A similar analysis was also completed by W. F. Baird & Associates, Coastal Engineers, Ltd., for inshore wave conditions—those to a water depth of 10 feet. The inshore wave data differ from offshore data in that refraction-shoaling coefficients were applied along with changes in wave angle between deep water and the inshore water depth to obtain predicted wave heights and periods. As shown on Map 10, the study area was broken down into four segments, each reflecting a separate shoreline orientation. Figures 8 and 9 graphically illustrate the predicted inshore wave heights and periods within each shoreline segment.

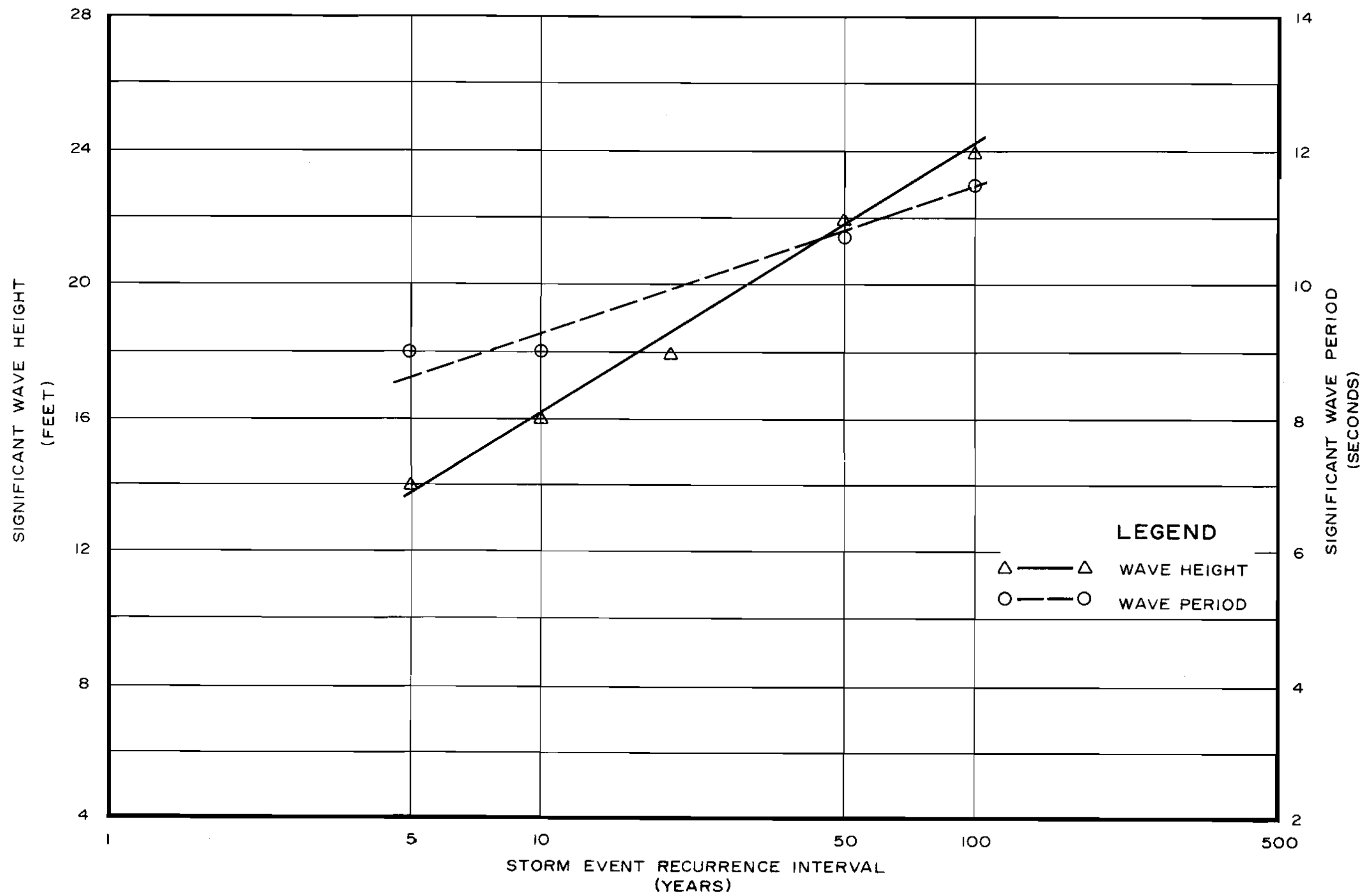
Lake Michigan Water Level: Lake water level fluctuations affect the rates of wave-induced shoreline erosion. High and rising water levels result in more rapid recession of the shoreline. When the water level is low, wave energy is expended as waves break along the beach. When water levels rise, waves can break directly on the

---

[5] *J. P. Keillor, University of Wisconsin-Sea Grant Institute, Letter to Earl K. Anderson, Port of Milwaukee Harbor Engineer, September 14, 1983.*

Figure 7

PREDICTED LAKE MICHIGAN DEEP WATER WAVE CONDITIONS DURING STORMS AT MILWAUKEE



NOTE: THE SIGNIFICANT WAVE PERIOD AND WAVE HEIGHT ARE THOSE AVERAGE CHARACTERISTICS EXHIBITED BY THE LARGEST ONE-THIRD OF THE WAVES OCCURRING DURING THE RECURRENCE INTERVAL STORM EVENT.

Source: J. P. Keillor, University of Wisconsin-Sea Grant Institute, Letter to Earl K. Anderson, Port of Milwaukee Harbor Engineer, September 14, 1983; and U. S. Army Corps of Engineers, Design Wave Information for the Great Lakes, Report No. 3, Lake Michigan, Technical Report H-76-1, November 1986.

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 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 along the Lake Michigan shoreline.[6]

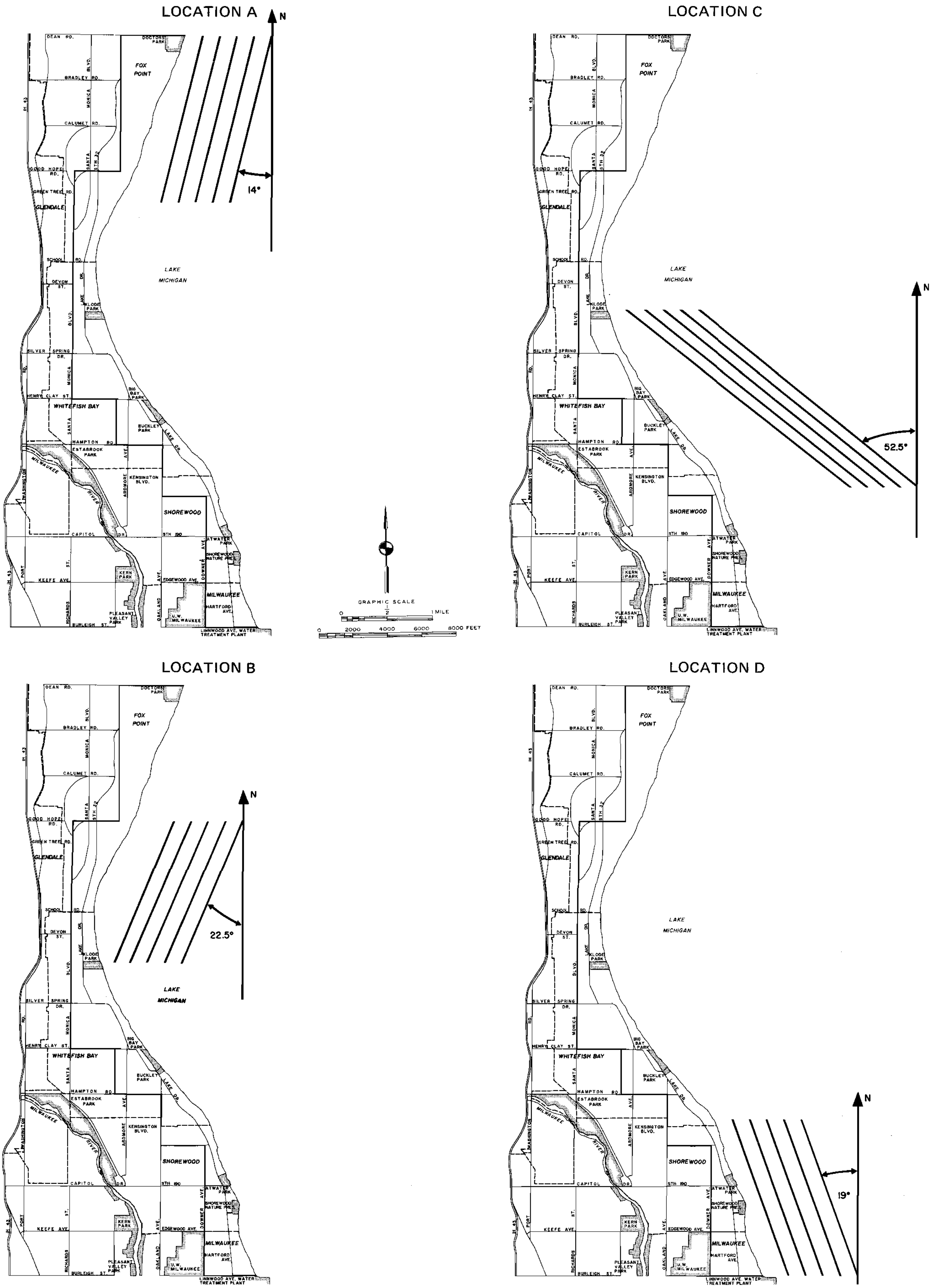
Since 1860, average annual surface elevations of Lake Michigan at Milwaukee have ranged from a low in 1964 of 577.06 feet above National Geodetic Vertical Datum (NGVD)—formerly referred to as Mean Sea Level Datum—to a high

---

[6]*R. C. Berg and C. Collinson, Bluff Erosion Recession Rates and Volumetric Losses on the Lake Michigan Shore in Illinois, Illinois Geologic Survey, Environmental Geology Note No. 76, 1976.*

Map 10

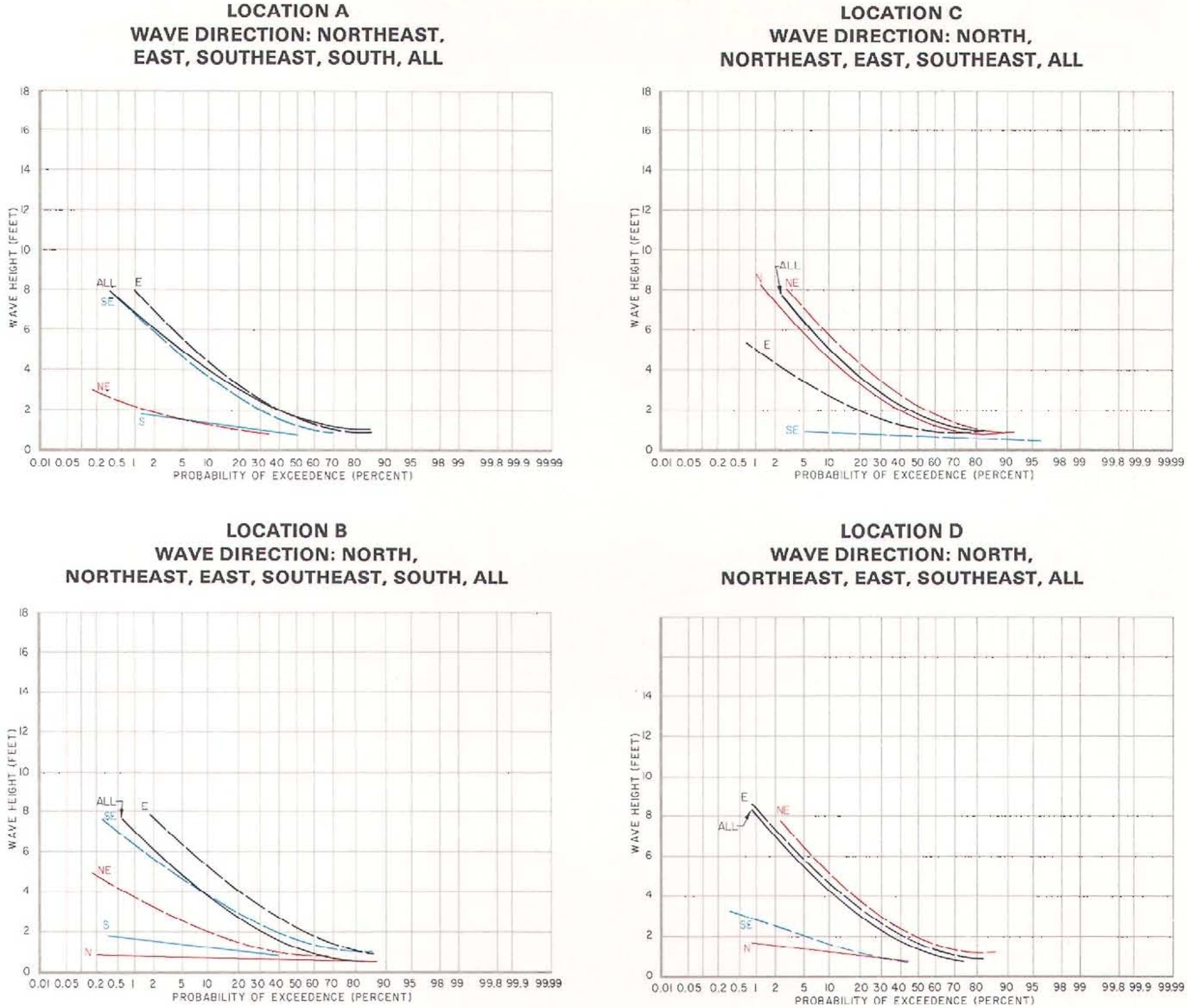
SHORE ORIENTATIONS ASSUMED FOR THE INSHORE WAVE ANALYSIS



Source: W. F. Baird & Associates Coastal Engineers, Ltd.

Figure 8

INSHORE WAVE HEIGHT PROBABILITY OF EXCEEDENCE



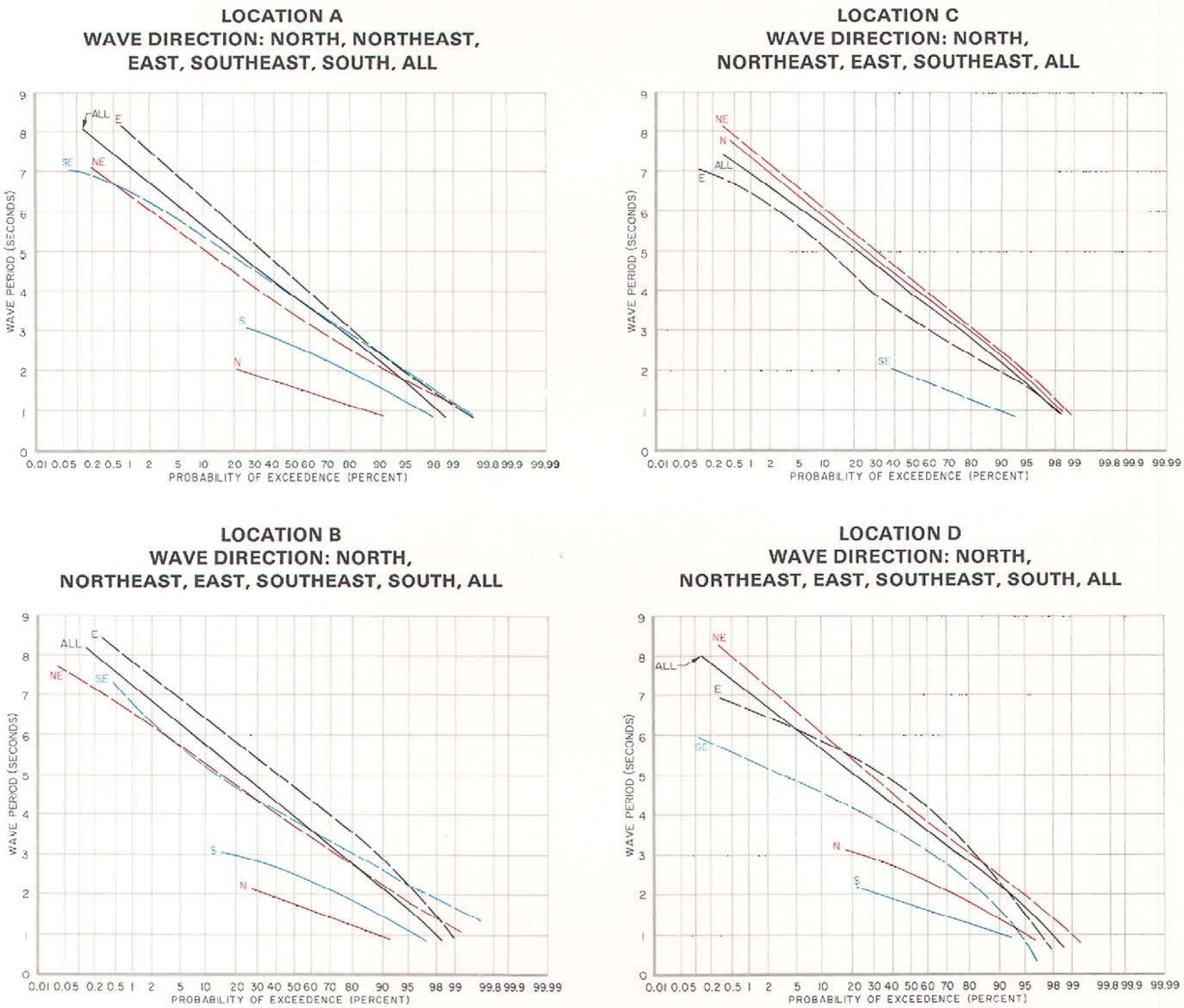
Source: W. F. Baird & Associates Coastal Engineers, Ltd., and SEWRPC.

of 582.24 feet above NGVD in 1986 (see Figure 10). The National Ocean Survey reports water level data using the International Great Lakes Datum (IGLD), 1955 adjustment. At Milwaukee, elevation in feet NGVD can be converted to feet IGLD by subtracting 1.34 feet, as determined by first order leveling conducted by the Regional Planning Commission. Major dredging of the St. Clair River at the outlet of Lake Huron in the late 19th century and early 20th century caused the levels of Lakes Michigan and Huron to decrease about one foot. Therefore, direct comparisons of the present-day lake levels with the relatively high levels recorded in the 19th century are inappropriate. Indeed, 20th century record levels would have exceeded the 19th century levels if the St. Clair River had not been dredged.

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 Straits of Mackinac, evaporation from the lake surface, and resulting changes in

Figure 9

INSHORE WAVE PERIOD PROBABILITY OF EXCEEDENCE



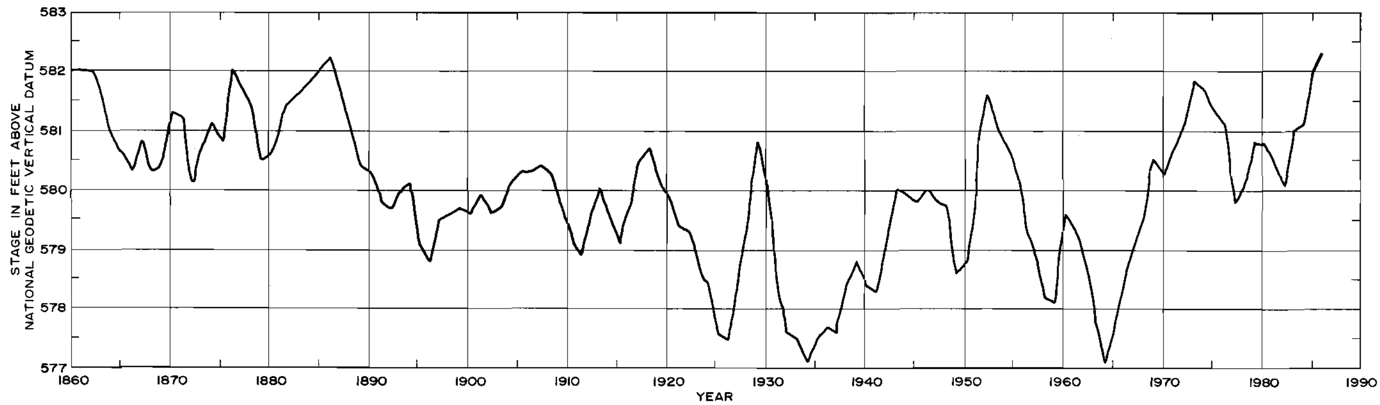
Source: W. F. Baird & Associates Coastal Engineers, Ltd., and SEWRPC.

the storage—volume of water—in the lake. The annual cycle in Lake Michigan water level elevations is shown in Figure 11. The highest water level elevations generally occur in June, July, and August, and the lowest occur in January, February, and March. Two components of the hydrologic budget—precipitation, which runs off the land or falls directly on the lakeshore, and evaporation—dominate the seasonal hydrologic cycle, with the result being that inflow to the lake generally exceeds outflow during the six-month February-through-July period, and outflow generally exceeds inflow during the remaining six months. Accordingly, lake storage and the lake levels rise from February through July and fall during 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.

Twentieth century-record high monthly mean lake levels at Milwaukee were experienced throughout 1986, as shown in Figure 11. These

Figure 10

LAKE MICHIGAN WATER LEVELS AT MILWAUKEE: 1860-1986



Source: U. S. Army Corps of Engineers; U. S. Department of Commerce, National Oceanic and Atmospheric Administration; and SEWRPC.

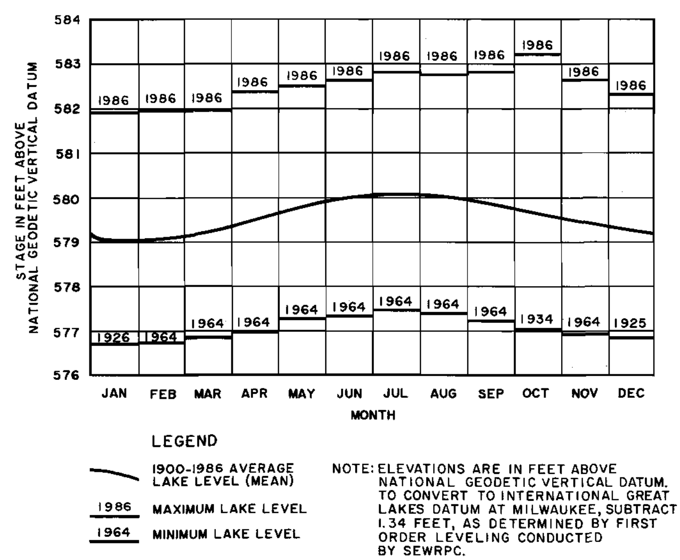
high lake levels were caused by unusually large amounts of precipitation. Short-term fluctuations in lake levels also occur.

Geological evidence indicates that within the the last 1,000 years Lake Michigan levels have at least five times exceeded by more than two feet the annual mean level recorded in 1985 (582.0 feet NGVD).[7] These episodes of high water may have lasted for many decades, perhaps centuries. The hydrologic conditions which produced these high levels could reoccur. It should be noted, however, that if the prehistoric hydrologic conditions had occurred with present-day outlet channel hydraulics, then the prehistoric levels of Lakes Michigan-Huron would have been at least one foot lower than indicated by the geologic record. Fluctuations in lake levels over several years are episodic, not cyclic. The only known cycle in lake levels is the annual cycle described above.

Anticipated Future Lake Levels: Assuming that the level of Lake Michigan continues to fluctuate as it has in the past, an estimate was made of the annual mean lake levels that may be expected to occur within the next 50 years.

Figure 11

VARIATIONS IN MONTHLY MEAN LAKE MICHIGAN LEVELS: 1900-1986



Source: U. S. Army Corps of Engineers; U. S. Department of Commerce, National Oceanic and Atmospheric Administration; and SEWRPC.

Analyses of water level records for the period 1915 through 1985 for Lake Michigan at Milwaukee adjusted to existing hydraulic outlet conditions during periods of rising levels were made to characterize relatively long-term rises. In order to estimate the maximum lake level that could be reached, a review was conducted of water level frequency curves developed from the

---

[7]*Curtis E. Larsen, A Stratigraphic Study of Beach Features on the Southwestern Shore of Lake Michigan: New Evidence of Holocene Lake Level Fluctuations, Illinois State Geological Survey, Environmental Geology Notes 112, 1985.*

1915 to 1985 level data and of data developed by the National Oceanic and Atmospheric Administration-Great Lake Environmental Research Laboratory relating to the outlet capacities of the lakes. This review indicated that there is a potential for the Lake Michigan water level to rise by about two feet over the mean 1986 levels. If precipitation amounts over a several-year period return to mean 20th century levels, the lake levels may be expected to be two to three feet lower than those measured in 1986. Record low lake levels experienced in 1963 and 1964 were about five feet lower than the 1986 levels.

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

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

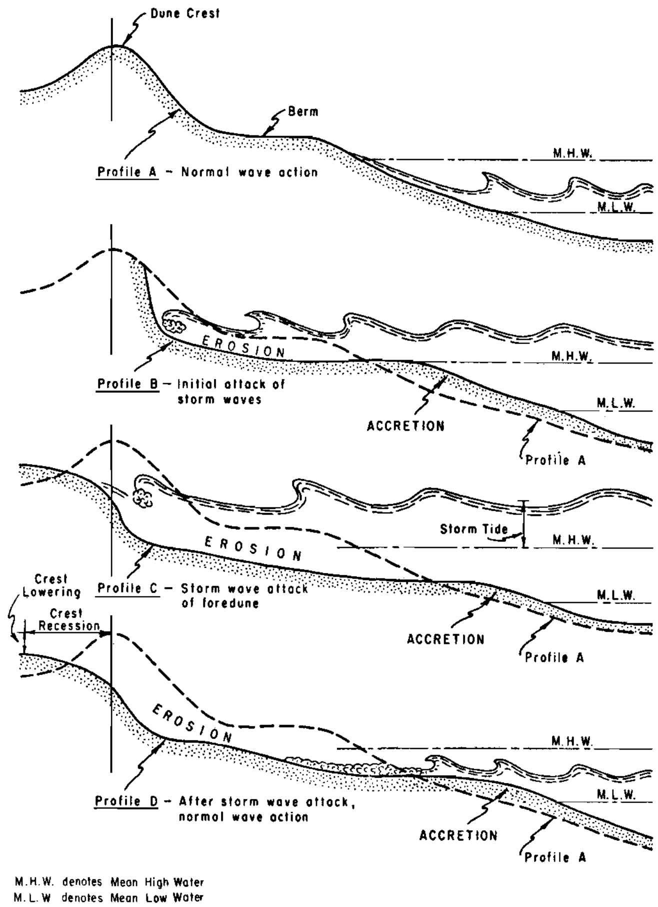
Vegetative Cover: Vegetation can also have an effect on bluff stability and erosion. The aboveground portion of the vegetation physically intercepts raindrops, thereby reducing their potential to loosen particles on the bluff face, reducing the impact of wind, and serving to trap windblown sediment. 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, and thereby contribute to its stability. Vegetation on the top of the bluff may serve to intercept and divert some surface runoff, thus preventing it from moving down the bluff face. The roots of vegetation, however, may induce infiltration by slowing runoff and providing infiltration passages into the bluff face, thereby possibly contributing to a decrease in bluff stability as a result of increased groundwater content and level. Probably one of the most significant aspects of the lack of vegetation on a bluff face is that it serves as an effective indicator of recent erosion.

Beach Erosion

The features of a beach and the materials composing the beach are continuously in a state of flux as a result of the near-shore transport of sand and gravel, primarily in response to wave action. There is a constantly changing interaction between the forces that bring sand ashore and those that move it lakeward, with the position and configuration of the main mass of sand at any time serving as an index of the dominant forces. Large waves which often occur during storm events tend to erode beaches by removing material from them and transporting it in a lakeward direction. In contrast, the small waves—characteristic of periods between storm events—tend to build beaches up through a net landward transport of sediment. Thus, the beaches exhibit a continuous cyclic pattern of erosion and accretion in response to the nature of the waves impinging on the beach and the elevation of the lake. Figure 12 shows the

Figure 12

BEACH EROSION IN RESPONSE TO WAVE ACTION



Source: U. S. Army Corps of Engineers.

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. In 1986, primarily because of the high lake levels, all beaches in the study area were in a state of erosion.

Sediment is transported parallel to the shoreline along the beach by long-shore currents. Long-shore 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 northern 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.[8] Only a portion of this amount is sand size or larger, suitable for forming beaches.

EXISTING REGULATIONS PERTAINING TO SHORELAND DEVELOPMENT

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

The U. S. Army Corps of Engineers is the primary federal agency responsible for the regulation of structures and work related to surface waters. Initial Corps authority to regulate structures or work in, or affecting, navigable waters stems from the River and Harbor Act of 1899. Corps regulatory authority was expanded 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.

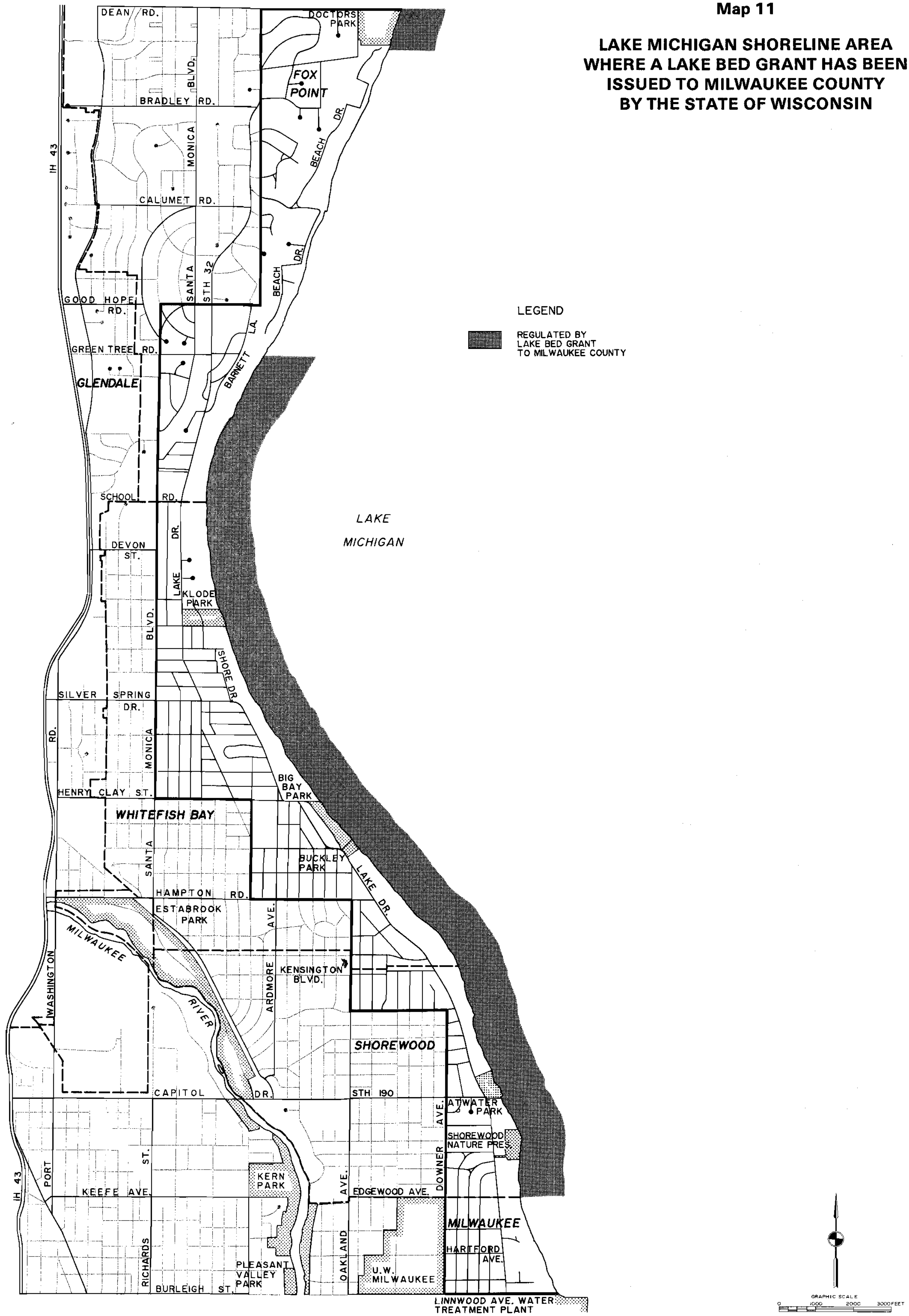
The State of Wisconsin, through the Department of Natural Resources (DNR), regulates shore protection activities under the provisions of Chapter 30 of the Wisconsin Statutes. State regulatory authority for shore protection and

---

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

Map 11

LAKE MICHIGAN SHORELINE AREA WHERE A LAKE BED GRANT HAS BEEN ISSUED TO MILWAUKEE COUNTY BY THE STATE OF WISCONSIN



Source: SEWRPC.

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 rip-rap 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 activities throughout most of the Lake Michigan shoreline of the State, 68 percent of the shoreline within the study area is regulated under a Lake Bed Grant issued to Milwaukee County, as shown in Map 11. The Lake Bed Grant, issued by the Wisconsin Legislature in 1933, ceded to Milwaukee County a strip of submerged land extending into Lake Michigan for a distance of 2,400 feet to be held and used by the County for public park, parkway, and highway purposes. To protect the public interest, the County administers a permit program for shore protection measures and dredge and fill activities which requires the submittal of a plan and which may require that certain conditions established by the County be met. The Wisconsin Department of Natural Resources does, however, have 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's interest.

As shown in Table 15, the construction of shore protection structures may require permits from the U. S. Army Corps of Engineers, Wisconsin Department of Natural Resources, Milwaukee County, and the local communities. 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. Prior to granting a permit, the Corps of Engineers requests review comments from the U. S. Fish and Wildlife Service, the Wisconsin Department of Natural Resources, and the State Historical Society. However, many smaller structures—those involving the placement of less than one cubic yard of material per linear foot of shoreline for a shoreline length of less than 500 feet—are covered under what is referred to as a Nationwide permit, and the Corps must simply be notified of the proposed construction. Outside the Lake Bed Grant shoreline area, a permit is also required from the Wisconsin Department of Natural Resources for all structures extending below the ordinary high-water mark. Within the Lake Bed Grant shoreline area, water quality certification is required from the Department of Natural Resources, and a permit is required from Milwaukee County. For all structures that extend above the ordinary high-water mark, a building permit is required from each of the villages within their jurisdictional boundaries. For all structures that extend below the ordinary high-water mark, a special shore protection permit is required from the villages. Within the City of Milwaukee, a building permit is required, regardless of whether the structure lies above or below the ordinary high-water mark. In addition, the local communities 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.

In addition, local units of government have been granted a variety of regulatory powers which can be used to guide development within the Lake Michigan shoreland area in the public interest. Among the most important of these are the comprehensive zoning and land subdivision regulations. The existing zoning regulations that apply within the shoreline portion of the study area have been previously described. There being relatively little undeveloped land within the shoreland area of the study area, land subdivision regulations have, as a practical matter, little application to the control of erosion hazards. A review of the existing subdivision control ordinances indicates that there are no specific provisions for the minimization of Lake Michigan shoreline erosion hazards. In the

Table 15

PERMITS REQUIRED FOR SHORE PROTECTION ACTIVITIES IN NORTHERN MILWAUKEE COUNTY

Regulatory Agency	Installation of Shore Protection Structures Below Ordinary High-Water Mark	Placement of Fill Material Above Ordinary High-Water Mark	Placement of Fill Material Below Ordinary High-Water Mark	Grading or Removal of Topsoil	Hauling of Fill Material
U. S. Army Corps of Engineers	A Nationwide permit requirement may apply if the structure covers less than 500 feet of shoreline. The Nationwide permit requirement could be applicable both within and outside the Lake Bed Grant area A General permit requirement may apply if a permit for the structure has been issued by the Wisconsin DNR, and if the project does not cover more than 2,000 feet of shoreline. The General permit requirement is applicable only outside the Lake Bed Grant area, where the DNR has permit authority If the above do not apply, an individual permit is required	- -	A Nationwide permit requirement may apply if the structure covers less than 500 feet of shoreline and less than one cubic yard of material is placed per lineal foot of shoreline. The Nationwide permit requirement could be applicable both within and outside the Lake Bed Grant area A General permit requirement may apply if a permit for the fill has been issued by the Wisconsin DNR, and if the project does not cover more than 2,000 feet of shoreline. There is no limit on the amount of fill placed. The Nationwide permit requirement may only be used outside the Lake Bed Grant area, where the DNR has permit authority If the above do not apply, an individual permit is required	- -	- -
Wisconsin Department of Natural Resources	Within Lake Bed Grant area, water quality certification required Outside Lake Bed Grant area, permit required	- -	Within Lake Bed Grant area, water quality certification required Outside Lake Bed Grant area, permit required	Permit required for grading or removing topsoil from the bank where area exposed exceeds 10,000 square feet	- -
Milwaukee County	Permit required within Lake Bed Grant area	- -	Permit required within Lake Bed Grant area	- -	- -
City of Milwaukee	Permit required	Permit required	Permit required	- -	Permit required
Village of Shorewood	- -	Permit required	Permit required	- -	Permit required
Village of Whitefish Bay	- -	Permit required	Permit required	- -	Permit required
Village of Fox Point	- -	Permit required	Permit required	- -	Permit required

NOTE: Permits are generally not required for the maintenance of shore protection measures.

Source: SEWRPC.

Village of Fox Point, however, water-related setbacks are included in the village construction regulations which restrict cutting on banks of ravines and lake bluffs.

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 by stabilizing bluff slopes. Structural protective measures installed both by public agencies and by private shoreline property owners are costly and have had varying degrees of success. In addition, many structures were not properly designed or constructed, and many are not adequately maintained, resulting in severe deterioration within a period of time much shorter than the life of the facilities they were designed to protect.

Onshore protective structures include bulkheads and revetments constructed at or near the base of a bluff. Bulkheads, or seawalls, have two functions: 1) to serve 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 type of onshore and near-shore protective structure is the groin, which is connected to and built perpendicular to the beach and is intended to partially obstruct the longshore current which results in the accumulation of transported sand on the beach up-current of a structure. A similar but temporary result may be achieved with artificial beach nourishment, although this approach is still under study—and not generally permitted—by the Wisconsin Department of Natural Resources. The resulting beach absorbs wave energy and reduces toe erosion along the adjacent bluffs. It should be noted that 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. Groins, as well as all other shore protection structures, require periodic maintenance, extension, and sometimes replacement.

Breakwaters 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, breakwaters may accelerate beach and bluff erosion downdrift of the protected areas, as sediments settle in the sheltered water behind the breakwater.

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.

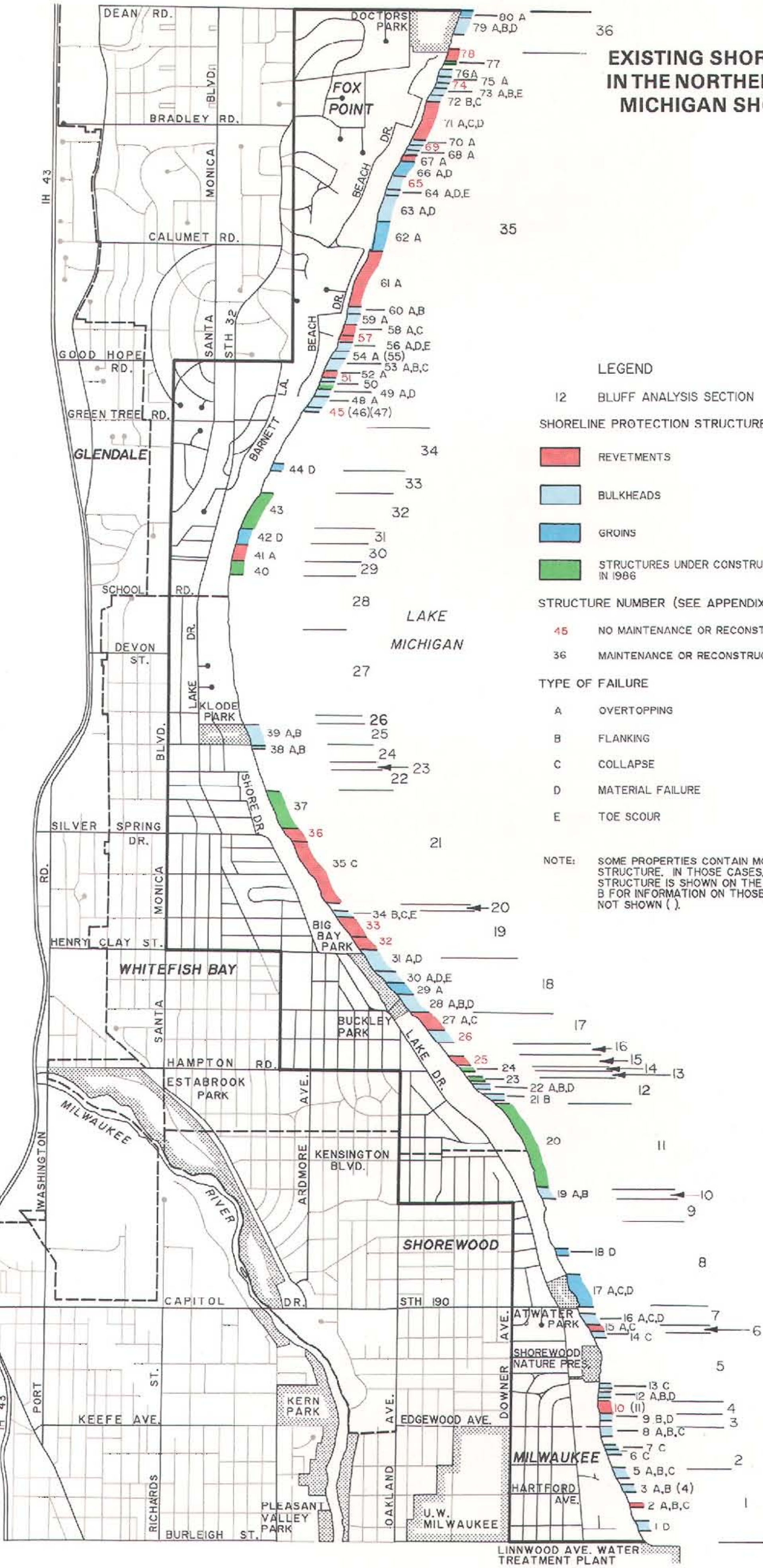
Another slope stabilization procedure involves the installation of internal drains to lower the water table within the bluff and thus reduce the likelihood of bluff slope failure. Slope stabilization can also include maintenance of a protective cover of vegetation. Slope stabilization measures usually include a combination of these methods.

A variety of shoreline protection structures have been installed by public and private property owners, thereby reducing shoreline erosion along certain portions of the northern Milwaukee County shoreline. In 1986, 80 shoreline protection structures were located within the study area. Of these 80 structures, 25, or 31 percent, were revetments; 43, or 54 percent, were bulkheads; 11, or 14 percent, were groins; and one, or 1 percent, was a breakwater. Of the total, eight, or about 10 percent of the structures, were located in the City of Milwaukee; 12, or about 15 percent, were located in the Village of Shorewood; 19, or 24 percent, were located in the Village of Whitefish Bay; and 41, or 51 percent, were located in the Village of Fox Point. As shown on Map 12, approximately 61 percent of the northern Milwaukee County shoreline, or 23,700 feet, was protected by onshore structures, although some of these structures were not providing adequate protection against shoreline erosion.

The quality and effectiveness of shoreline protection structures varies considerably. An inventory of the condition of shoreline protection structures within the northern Milwaukee County study area was conducted in August 1986. The results of this survey are presented in Appendix B and summarized in Table 16. The table indicates that only 24 percent of the structures, including 52 percent of revetments

Map 12

EXISTING SHORE PROTECTION STRUCTURES IN THE NORTHERN MILWAUKEE COUNTY LAKE MICHIGAN SHORELINE STUDY AREA: 1986



LEGEND

12 BLUFF ANALYSIS SECTION

SHORELINE PROTECTION STRUCTURES

- REVETMENTS
- BULKHEADS
- GROINS
- STRUCTURES UNDER CONSTRUCTION IN 1986

STRUCTURE NUMBER (SEE APPENDIX B)

- 45 NO MAINTENANCE OR RECONSTRUCTION REQUIRED
- 36 MAINTENANCE OR RECONSTRUCTION REQUIRED

TYPE OF FAILURE

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

NOTE: SOME PROPERTIES CONTAIN MORE THAN ONE STRUCTURE. IN THOSE CASES, ONLY THE PRIMARY STRUCTURE IS SHOWN ON THE MAP. SEE APPENDIX B FOR INFORMATION ON THOSE STRUCTURES NOT SHOWN ( ).

GRAPHIC SCALE

0 ½ 1 MILE

0 1000 2000 3000 4000 FEET

Source: SEWRPC.

Table 16

SUMMARY OF NORTHERN MILWAUKEE COUNTY STRUCTURAL SHORE PROTECTION SURVEY: 1986

Maintenance Required	Type of Structure									
	Revetment		Bulkhead		Groin		Breakwater		Total	
	Number	Percent	Number	Percent	Number	Percent	Number	Percent	Number	Percent
Yes	12	48	37	86	11	100	1	100	61	76
No	13	52	6	14	0	0	0	0	19	24
Total	25	100	43	100	11	100	1	100	80	100
	Type of Failure[a]									
Toe Scour	0	0	4	9	0	0	0	0	4	5
Overtopping	12	48	28	65	8	73	1	100	49	61
Flanking	1	4	18	42	1	9	0	0	20	25
Collapse	6	24	11	26	2	18	0	0	19	24
Material Failure	0	0	14	33	5	45	0	0	19	24
None	13	52	6	14	0	0	0	0	19	24

[a]*Some structures had more than one type of failure.*

Source: SEWRPC.

and 14 percent of bulkheads, had no observable failures and were not in need of any significant maintenance work. The remaining structures were observed to have some type of failure. Table 16 summarizes the type of failures affecting these structures.

The predominant type of structural failure was overtopping, where the water level, or the wave heights, exceeded the top of the structure. Overtopping, which erodes material from behind revetments and bulkheads, and which reduces the effectiveness of groins and breakwaters, affected about 61 percent of the structures inventoried, including about 48 percent of the revetments, 73 percent of the groins, 65 percent of the bulkheads, and the single breakwater. This indicates either that most structures were 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 types of failure included flanking—where the sides of the structure are eroded; collapsing; material failure; and toe scour. Flanking affected 25 percent of the structures inventoried, including about 4 percent of the revetments, 42 percent of the bulkheads, and 9 percent of the groins. About 24 percent of the structures surveyed experienced collapsing, 24 percent had material failure, and 5 percent were undercut at the structure toe.

EXISTING COASTAL EROSION PROBLEMS

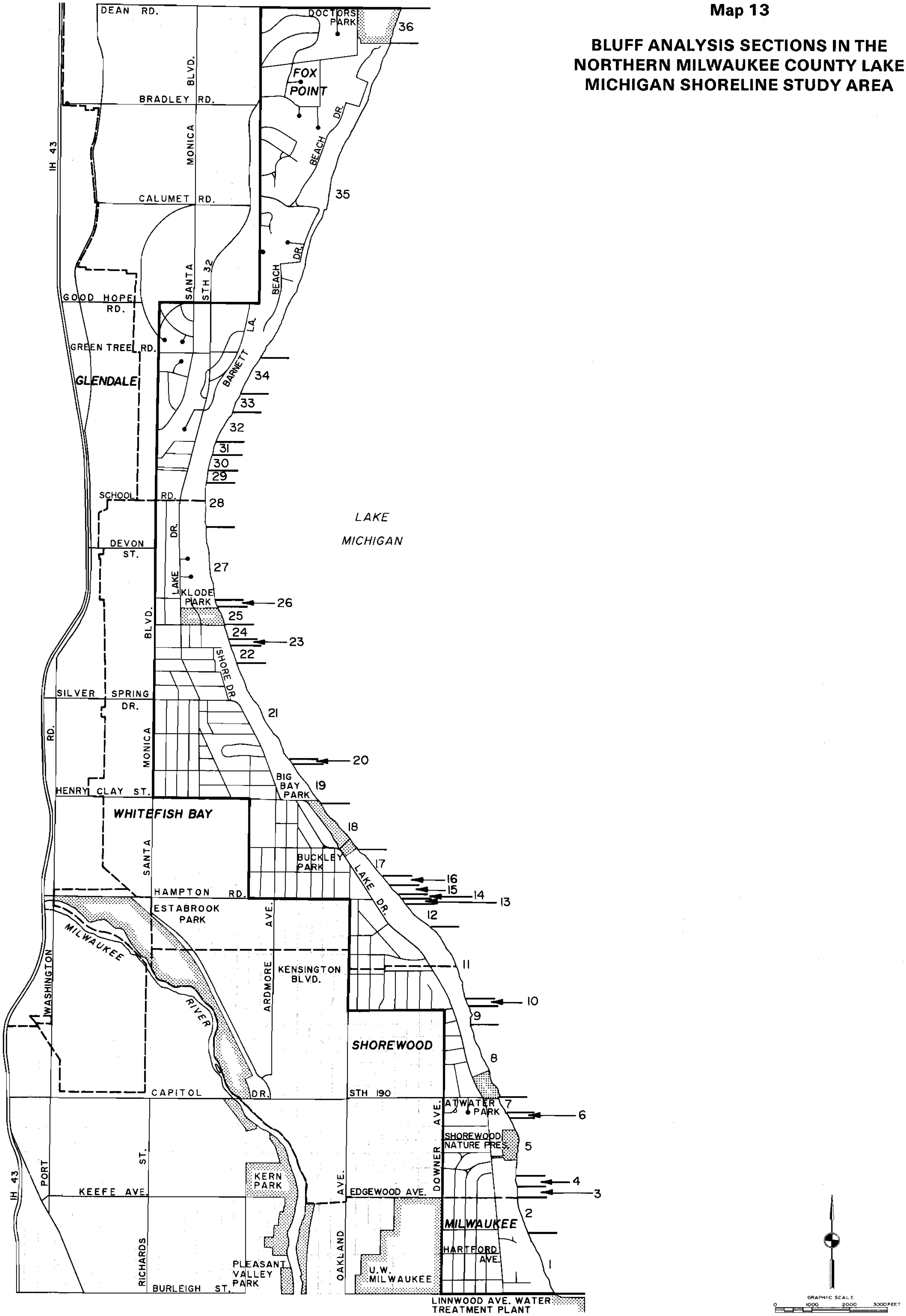
Bluff recession results in the loss of extensive land areas; and the sometimes major, unexpected, and rapid slope failures caused by slumping and sliding may pose a threat to human safety. The erosion or accretion of the beaches is a related process in that the extent of the beach affects the degree 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 erosion of the Fox Point Terrace also results in the loss of land area and poses a threat of extensive damage to those properties on the terrace with lake frontage, as well as to N. Beach Drive.

Bluff Analysis Sections

The study area shoreline was divided into 36 sections, each with similar physical and erosion-related characteristics. These 36 bluff analysis sections are shown on Map 13. The boundaries of the sections are located on property boundary lines. Field surveys were conducted in May 1986 to delineate the section boundaries and to inventory the physical characteristics of, and identify the causes and types of, shoreline erosion and slope failure occurring within each section. Table 17 summarizes the physical and

Map 13

BLUFF ANALYSIS SECTIONS IN THE NORTHERN MILWAUKEE COUNTY LAKE MICHIGAN SHORELINE STUDY AREA



Source: SEWRPC.

Table 17

PHYSICAL AND EROSION-RELATED CHARACTERISTICS OF BLUFF ANALYSIS SECTIONS: 1986

Civil Division	Bluff Analysis Section	Address	Shoreline Length (feet)	Beach Width (feet)	Beach Composition	Bluff Height (feet)	General Bluff Composition	Groundwater Conditions	Fill Project	Cause and Type of Bluff Slope Failure
City of Milwaukee	1	Linnwood Avenue Water Treatment Plant—3052 Newport Court	1,970	0-80	Sand and gravel	75-90	Undetermined; vegetated	No major groundwater seeps were noted	No	No apparent bluff failures
	2	3378-3474 N. Lake Drive	950	60	Sand and gravel	90-100	Undetermined; vegetated	No major groundwater seeps were noted	No	No apparent bluff failures
Village of Shorewood	3	3510 N. Lake Drive	300	0-40	Sand and gravel	95-100	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	No	Wave erosion and groundwater seepage contribute to slope failure. Failure occurs as shallow slides and deep-seated slumps
	4	3534 N. Lake Drive	290	⩽ 10	- -	95-100	Ozaukee till at top of bluff, underlain by sand, Oak Creek till, and New Berlin till	No major groundwater seeps were noted	Yes	- -
	5	3650-3914 N. Lake Drive	1,710	⩽ 10	Sand and gravel	100-110	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 the sand and gravel layer	No	No apparent bluff failures
	6	3926 N. Lake Drive	170	⩽ 10	- -	110	Ozaukee till at top of bluff, underlain by sand and gravel, Oak Creek till, and New Berlin till	No major groundwater seeps were noted	Yes	- -
	7	3932-3966 N. Lake Drive	380	⩽ 10	- -	110	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	No	Wave erosion contributes to bluff failure. Also, top of the bluff has several broken drainage tiles which are leaking water on slope. Failure occurs as sloughing and shallow slides
	8	Atwater Park—4216 N. Lake Drive	2,170	25-130	Sand, gravel, and cobbles	90-105	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	No	No apparent bluff failures
	9	4226-4320 N. Lake Drive	520	⩽ 10	- -	115	Undetermined; vegetated	No major groundwater seeps were noted	No	No apparent bluff failures
	10	4400-4408 N. Lake Drive	240	⩽ 10	- -	110-115	Ozaukee till at top of bluff, underlain by silt and sand, Oak Creek till, and New Berlin till	No major groundwater seeps were noted	No	Wave erosion contributes to bluff failure. Although a concrete bulkhead protects the toe of the bluff, erosion by wave action is still occurring over the top of the wall

Table 17 (continued)

Civil Division	Bluff Analysis Section	Address	Shoreline Length (feet)	Beach Width (feet)	Beach Composition	Bluff Height (feet)	General Bluff Composition	Groundwater Conditions	Fill Project	Cause and Type of Bluff Slope Failure
Village of Shorewood (continued)	11	4424-4652 N. Lake Drive	2,370	$\leqslant$ 10	- -	95-115	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	Yes	Wave erosion contributed to the bluff failure that was occurring prior to the construction of the fill project. A groundwater drainage system installed in 1966 at 4430 N. Lake Drive may have helped reduce groundwater seepage and associated adverse effects on the bluff
Village of Whitefish Bay	12	4668-4730 N. Lake Drive	850	$\leqslant$ 10	- -	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	No	Wave erosion contributes to bluff failure. Failure occurs as surface sloughing, slumping, and shallow slides
	13	4744-4762 N. Lake Drive	190	$\leqslant$ 10	- -	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	No	- -
	14	4780 N. Lake Drive	160	$\leqslant$ 10	- -	90-95	Ozaukee till at top of bluff, underlain by silt and sand, Oak Creek till, and New Berlin till	No major groundwater seeps were noted	No	Wave erosion contributes to bluff failure. Failure occurs as surface sloughing, slumping, and shallow slides
	15	4790-4800 N. Lake Drive	310	$\leqslant$ 10	- -	80-90	Ozaukee till at top of bluff, underlain by silt and sand, Oak Creek till, and New Berlin till	No major groundwater seeps were noted	No	- -
	16	4810-4840 N. Lake Drive	360	$\leqslant$ 10	- -	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	No	Wave erosion contributes to bluff failure. Slope failure occurs as shallow slumps and slides
	17	4850-4940 N. Lake Drive	810	$\leqslant$ 10	Sand and gravel	65-80	At southern end of section, Ozaukee till at top of bluff, underlain by silt and sand, Oak Creek till, and New Berlin till. The remainder of the bluff is undetermined vegetated	No major groundwater seeps were noted	Yes	- -
	18	Buckley Park—southern portion of Big Bay Park	1,660	0-25	Sand and gravel	65-80	Undetermined; vegetated	No major groundwater seeps were noted	No	Large slope failure occurred in November 1982 at southern end of this section. Bulkhead at base of bluff was overtopped and groundwater seepage may also have contributed to the failure

Table 17 (continued)

Civil Division	Bluff Analysis Section	Address	Shoreline Length (feet)	Beach Width (feet)	Beach Composition	Bluff Height (feet)	General Bluff Composition	Groundwater Conditions	Fill Project	Cause and Type of Bluff Slope Failure
Village of Whitefish Bay (continued)	19	Northern portion of Big Bay Park to 5270 N. Lake Drive	1,480	≤ 10	- -	65-80	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	Yes	- -
	20	5290 N. Lake Drive	130	≤ 10	- -	70	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	No	Minor slope failure was occurring as shallow slides within top portion of bluff
	21	5300 N. Lake Drive–808 Lakeview Avenue	2,970	≤ 10	- -	80-85	Undetermined	No major groundwater seeps were noted	Yes	- -
	22	5722-5770 N. Shore Drive	490	0-50	Sand and gravel	80-85	Ozaukee till at top of bluff, underlain by silt and sand, and Oak Creek till	Some groundwater seepage in the Ozaukee till layer	No	Groundwater seepage and wave erosion contribute to bluff failure. Wave erosion is minimized by the relatively wide beach. Slope failure occurs as slumping and shallow slides
	23	758 E. Day Avenue	140	35	Sand and gravel	85	Ozaukee till at top of bluff, underlain by silt and sand, and Oak Creek till	No major groundwater seeps were noted	Yes	- -
	24	740 E. Day Avenue–5866 N. Shore Drive	430	25-35	Sand and gravel	80-85	Ozaukee till at top of bluff, underlain by silt and sand, and Oak Creek till	Many groundwater seeps were noted	No	Groundwater seepage and wave erosion contributed to bluff failure. Slope failure occurs as slumping along silt and sand layers
	25	Klode Park	480	20-35	Sand and gravel	75-80	Ozaukee till at top of bluff, underlain by silt and sand and Oak Creek till	No major groundwater seeps were noted	No	No apparent bluff failures
	26	5960 N. Shore Drive	170	35-40	Sand and gravel	80-90	Ozaukee till at top of bluff, underlain by silt and sand, and Oak Creek till	Some groundwater seepage in the silt and sand layer	No	Groundwater seepage contributes to bluff failure. Wave erosion is minimal due to the wide beach. Slope failures occur as slumping
	27	6000 N. Shore Drive–6260 N. Lake Drive	1,950	35-50	Sand and gravel	90-120	Ozaukee till at top of bluff, underlain by silt and sand, a layer of sand, a layer of silt and sand, and Oak Creek till	Groundwater seeps are common from mid-height on bluff to base	No	Groundwater seepage contributes to bluff failure. Although a relatively wide beach was present, toe erosion was occurring. Slope failures occur as small slips

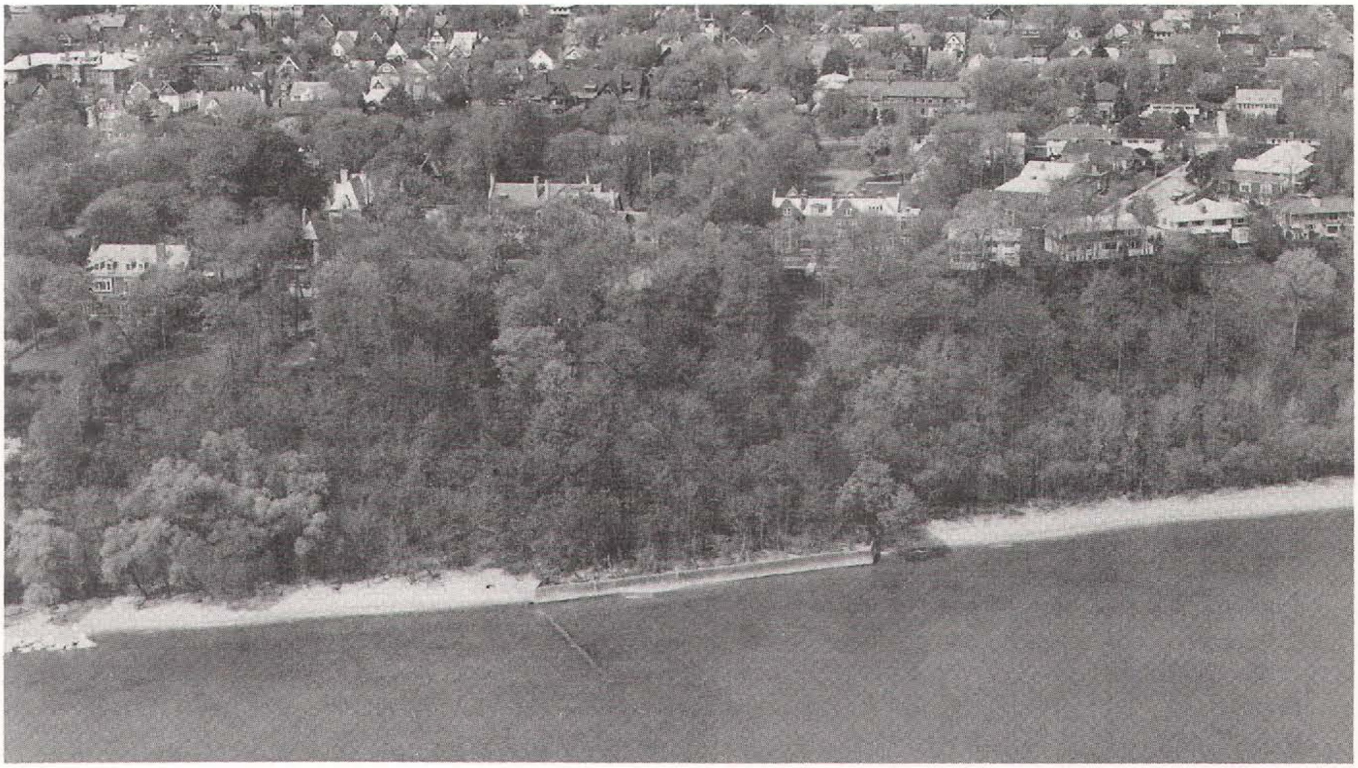
Table 17 (continued)

Civil Division	Bluff Analysis Section	Address	Shoreline Length (feet)	Beach Width (feet)	Beach Composition	Bluff Height (feet)	General Bluff Composition	Groundwater Conditions	Fill Project	Cause and Type of Bluff Slope Failure
Village of Whitefish Bay (continued)	28	6310-6424 N. Lake Drive	1,150	25-40	Sand and gravel	115-125	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	No	Wave erosion and groundwater seepage contribute to bluff failure and are occurring mainly along the lower portion of the bluff slope. Slope failures occur as surface sliding and slumping
Village of Fox Point	29	6430-6448 N. Lake Drive	320	⩽ 10	- -	120-125	Ozaukee till at top of bluff, underlain by silt and sand	No major groundwater seeps were noted	Yes	- -
	30	6464-6530 N. Lake Drive	470	⩽ 10	- -	115-125	Ozaukee till at top of bluff, underlain with silt and sand	Groundwater seepage noted at top of the silt layer	No	Wave erosion and groundwater seepage contribute to bluff failure. Slope failures occur as shallow slides and slumps
	31	6600-6702 N. Lake Drive	510	0-50	Sand and gravel	120-125	Ozaukee till at top of bluff, underlain by silt and sand	No major groundwater seeps were noted	No	Wave erosion contributes to the minimal bluff failure. Slope failures occur as shallow slides and small slumps
	32	6720 N. Lake Drive—6818 N. Barnett Lane	770	0-15	Sand and gravel	115-125	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 lie below Oak Creek till, and below New Berlin till lies bedrock	No major groundwater seeps were noted	No	Wave erosion contributes to failure. Slope failures occur as rapid surface sliding and small slumps
	33	6820-6840 N. Barnett Lane	530	⩽ 10	- -	115-120	Ozaukee till at top of bluff, underlain by silt and sand, Oak Creek till and New Berlin till	No major groundwater seeps were noted	No	Wave erosion contributes to bluff failure. Slope failures occur as slumping and shallow slides
	34	6868-7004 N. Barnett Lane	1,460	⩽ 10	- -	115-120	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	No	Wave erosion contributes to bluff failure. Slope failures occur as shallow slides and slumps
	35	7000-8130 N. Beach Drive	9,070	0-65	Sand and gravel	4-10	Sand at the top of the terrace underlain by New Berlin till	No major groundwater seeps were noted	No	Terrace is being eroded by wave action at some sites
	36	Doctors Park	840	⩽ 10	- -	90	Undetermined; vegetated	No major groundwater seeps were noted	No	Apparent bluff failure, although wave erosion was occurring at southern end of section

Source: SEWRPC.

Figure 13

BLUFF ANALYSIS SECTION 1



Source: SEWRPC.

erosion-related characteristics of the 36 sections, and presents the addresses included within each section.

Section 1, which extends from the City of Milwaukee Linnwood Avenue water treatment plant to 3052 E. Newport Court, had a beach width ranging from 0 to 80 feet and a bluff height ranging from 75 to 90 feet. The composition of the bluff was undetermined because of the vegetative cover on the bluff face (see Figure 13). The slope of the bluff was relatively gentle, with an angle of approximately 18 degrees. No bluff failures were observed during the May 1986 field survey. There was no observed wave erosion at the bluff toe, probably owing to the relatively wide beach, which was formed by the groin-like action of the Linnwood Avenue water treatment plant.

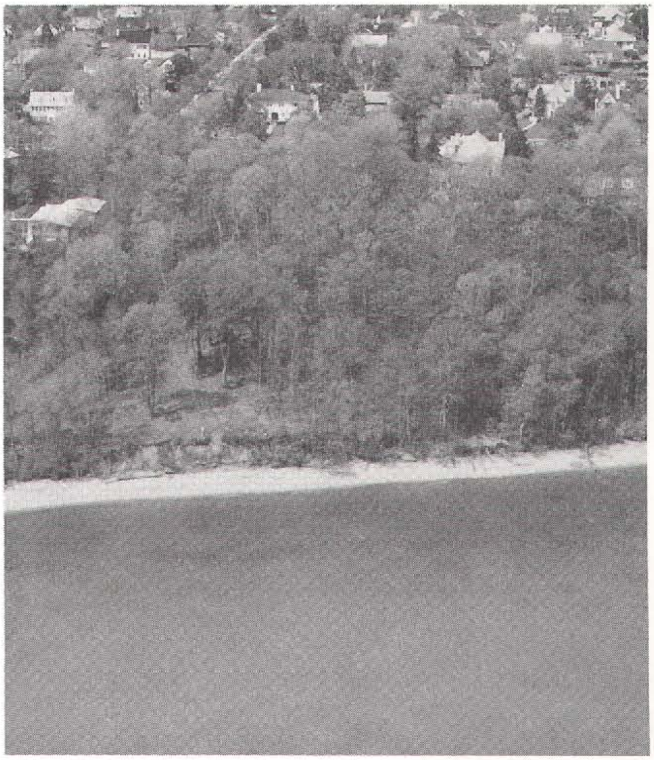
Section 2, which extends from 3378 to 3474 N. Lake Drive, had a beach width of about 60 feet and a bluff height ranging from 90 to 100 feet. The composition of the bluff was again undetermined because of the heavy vegetation on the bluff face, and no bluff failures were observed at the time of the survey. Within this section, the bluff forms two ravines which have side slopes of approximately 25 degrees. At the base of the southern ravine there was an alluvial fan of approximately 100 feet in width composed primarily of Ozaukee till, probably from an old slump block, covered with three feet of sand and gravel deposited by runoff from the ravine. As shown in Figure 14, the face of the fan has experienced slope failure in the form of shallow slides caused by wave erosion. The erosion of this alluvial fan does not affect the overall stability of the bluff.

Section 3, at 3510 N. Lake Drive, had a beach width ranging from 0 to 40 feet and a bluff height ranging from 95 to 100 feet. Ozaukee till constituted the top of the bluff, and was underlain by sand and gravel, Oak Creek till, and New Berlin till. As shown in Figure 15, the face of the bluff was vegetated, with a slope of 28 degrees. The upper portion of the bluff slope was less stable than in Sections 1 or 2, experiencing shallow slides and slumping. All of the material at the toe of the bluff is from old slumps. Erosion by wave action was observed within this section. Groundwater seepage was observed along the lower two-thirds of the slope.

Section 4, at 3534 N. Lake Drive, was a fill, constructed in 1980, covering 90 linear feet of shoreline. There was no beach in this section and the bluff height was about 95 to 100 feet. As shown in Figure 16, the bottom two-thirds of the

Figure 14

BLUFF ANALYSIS SECTION 2



Source: SEWRPC.

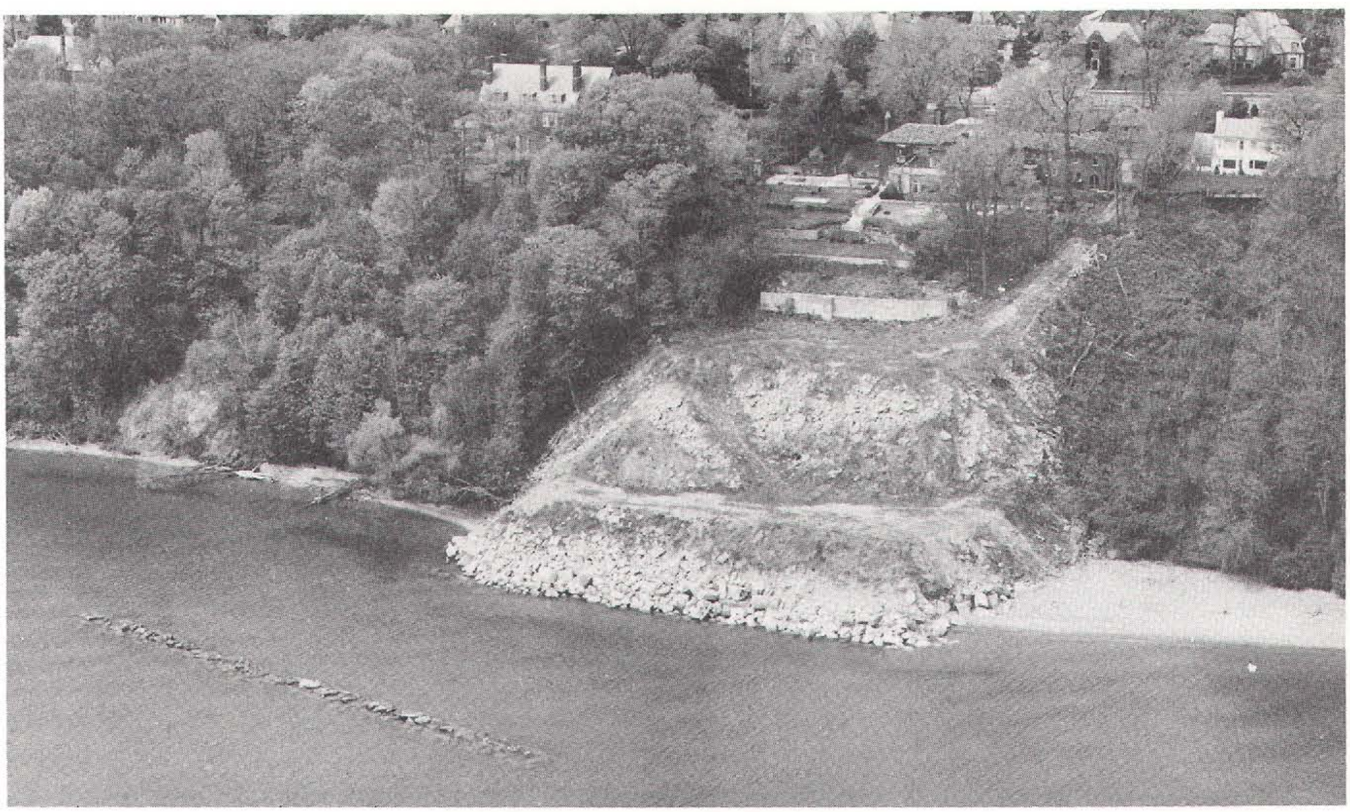
Figure 15

BLUFF ANALYSIS SECTION 3

Source: SEWRPC.

Figure 16

BLUFF ANALYSIS SECTION 4



Source: SEWRPC.

Figure 17

BLUFF ANALYSIS SECTION 5

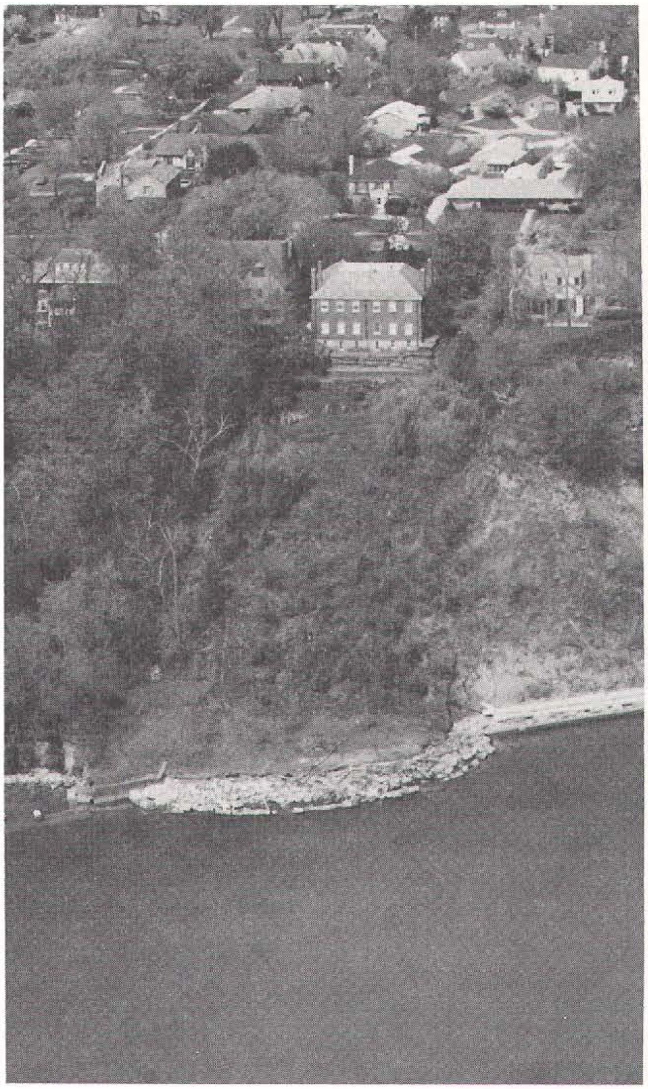
Source: SEWRPC.

slope was composed of concrete rubble with sparse vegetative cover. Slope stabilization within the top third of the bluff was provided by the placement of a series of retaining walls within the regraded bluff slope. The retaining walls were constructed of concrete and ranged in height from one to 10 feet. A revetment protected the toe of the fill. Additional protection was provided by a breakwater which was located approximately 120 feet offshore.

As shown in Figure 17, Section 5, which extends from 3550 to 3914 N. Lake Drive, contained a beach less than 10 feet wide and bluff heights ranging from 100 to 110 feet. The bluff was composed of Ozaukee till underlain by sand and gravel, Oak Creek till, and New Berlin till. The bluff face was heavily vegetated, with a slope of approximately 20 degrees. During the field survey, there was some evidence of older mass movement, but no sign of recent movement. At the base of the bluff, a lake sand terrace—known as the Nipissing terrace—was present, reaching a maximum width of 300 feet. The terrace, which was approximately 20 feet high, was composed of erodible sand and gravel, and was experiencing erosion from wave action. Minor groundwater seeps were noted at the base of the sand and gravel layer on the till surface.

Figure 18

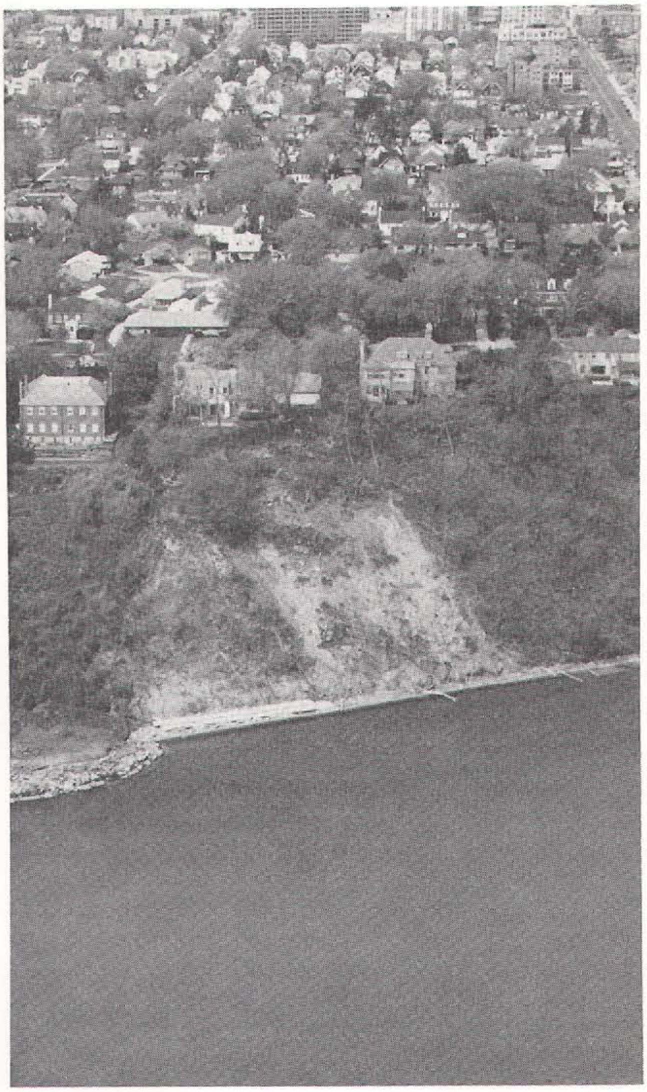
BLUFF ANALYSIS SECTION 6



Source: SEWRPC.

Figure 19

BLUFF ANALYSIS SECTION 7



Source: SEWRPC.

Section 6, at 3926 N. Lake Drive, was a fill, constructed in 1977, covering about 170 linear feet of shoreline. There was no beach in this section and the bluff height was about 110 feet. In this section, bluff slope stabilization was accomplished by cutting back the bluff to a 20-degree angle. A concrete block revetment was installed to protect the toe of the fill. As shown in Figure 18, a good vegetative cover was established on the fill.

In Section 7, which extends from 3932 to 3966 N. Lake Drive, the beach was less than 10 feet wide and the bluff height was about 110 feet. Ozaukee till constituted the top of the bluff and was underlain sand and gravel, sand, Oak Creek till, and New Berlin till. The bluff face was almost entirely exposed, with an average slope of 29 degrees. The primary cause of bluff failure was wave erosion at the bluff toe, although slope failure may have been exacerbated by broken drainage tiles which are leaking onto the bluff face. As shown in Figure 19, slope failure occurred as surface sloughing and shallow slides.

Section 8, which extends from Atwater Park to 4216 N. Lake Drive, had the widest beach within the study area—up to 130 feet in May 1986—built up primarily by the Atwater Park groin system. Bluff heights ranged from 90 to 105 feet. The

Figure 20

BLUFF ANALYSIS SECTION 8

Source: SEWRPC.

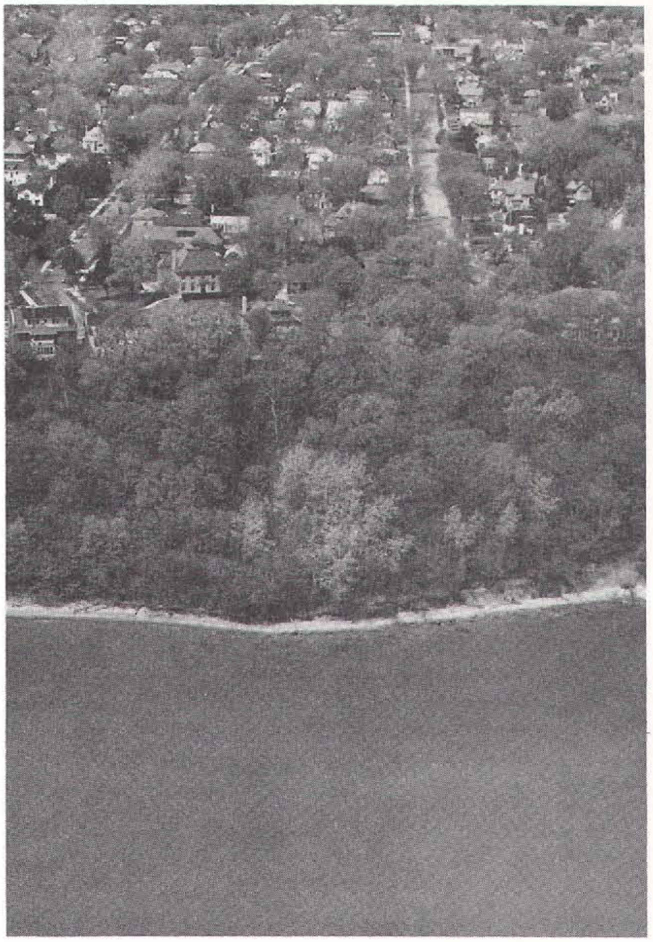
bluff was composed primarily of Ozaukee till underlain by sand, Oak Creek till, and New Berlin till. As shown in Figure 20, the bluff was heavily vegetated. The slope of the bluff was stable and terraced. North of Atwater Park, there was evidence of past slope surface movement. At the far northern end of the section, the Nipissing terrace was again present. The terrace, which is composed of sand and gravel underlain by New Berlin till, appeared to be stable and vegetated, although some wave erosion was noted at the base of the terrace.

In Section 9, which extends from 4226 to 4320 N. Lake Drive, the beach width was less than 10 feet and the bluff height was about 115 feet. The composition of the bluff was undetermined because of the heavy vegetation on the bluff face. The slope of the bluff was approximately 28 degrees. No bluff failures were observed during the field surveys. The toe of the bluff was protected by the Nipissing terrace, which reached a width of 100 feet within this section. There was wave erosion observed at the base of the terrace, as shown in Figure 21.

As shown in Figure 22, Section 10, which extends from 4400 to 4408 N. Lake Drive, did not have a beach, and the bluff height ranged from 100 to 115 feet. The composition of the bluff was Ozaukee till, underlain by silt and sand, Oak Creek till, and New Berlin till. The face of the bluff was heavily vegetated, with a slope of 24 degrees. No groundwater seepage was noted. The toe of the bluff was protected by a concrete bulkhead which was being overtopped. While the bulkhead offered some protection, there was continued erosion from waves washing over the top of the structure.

Figure 21

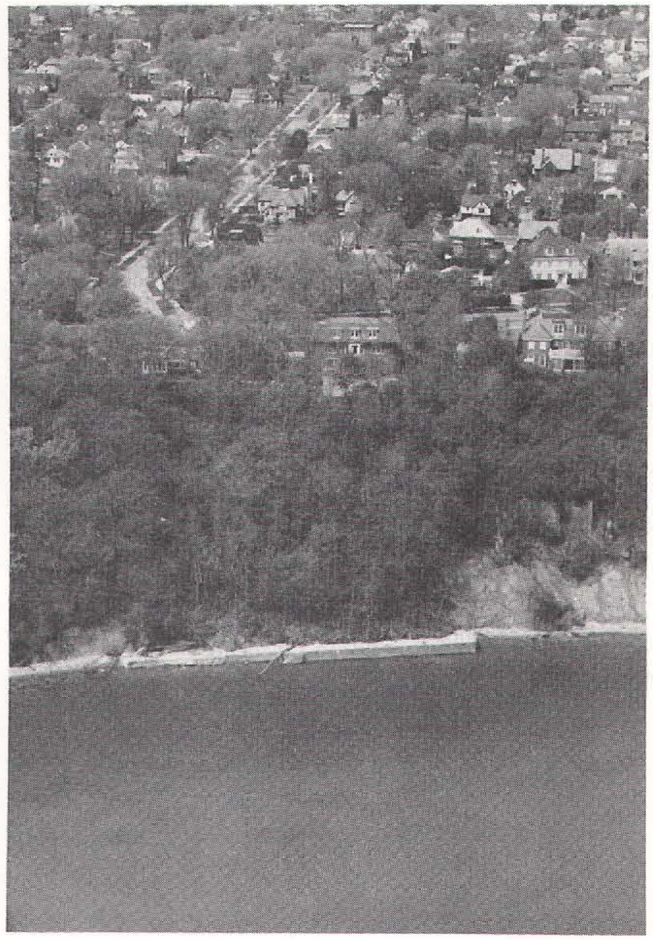
BLUFF ANALYSIS SECTION 9



Source: SEWRPC.

Figure 22

BLUFF ANALYSIS SECTION 10



Source: SEWRPC.

Section 11, which extends from 4424 to 4652 N. Lake Drive, was a fill project under construction in 1986. The beach was generally less than 10 feet wide and the bluff height ranged from 95 to 115 feet. The composition of the bluff was Ozaukee till, underlain by silt and sand, Oak Creek till and New Berlin till, and silt and sand layers. As shown in Figure 23, recent major slumping had occurred within a large portion of this section. A clay pressure ridge had been formed approximately 30 feet offshore by this deep-seated failure, which indicated that a very large slip surface, covering the entire bluff slope, had experienced movement. Some groundwater seeps were observed in the silt and sand layer. Wave erosion contributed to the bluff failure that was occurring prior to the construction of the fill project. A groundwater drainage system installed at 4430 N. Lake Drive may have helped reduce groundwater seepage and associated adverse effects within that localized area of the bluff.

In Section 12, which extends from 4668 to 4730 N. Lake Drive, the beach was less than 10 feet wide, and the bluff height was about 95 feet. The composition of the bluff was Ozaukee till underlain by silt and sand, Oak Creek till, and New Berlin till. The slope of the bluff was very steep, with an angle of approximately 38 degrees, and was free of vegetation except within ravines. Severe wave erosion was rapidly removing sediment at the base of the bluff and was the primary cause of bluff failure. As shown in Figure 24, slope failure occurred as slumping and shallow slides.

Section 13, which extends from 4744 to 4762 N. Lake Drive, was a fill project under construction in 1986, and covered 190 linear feet of shoreline (see Figure 25). The beach was less than 10 feet wide and the bluff height was about 95 feet. The fill was generally composed of rubble and concrete slabs. The slope of the fill was still quite

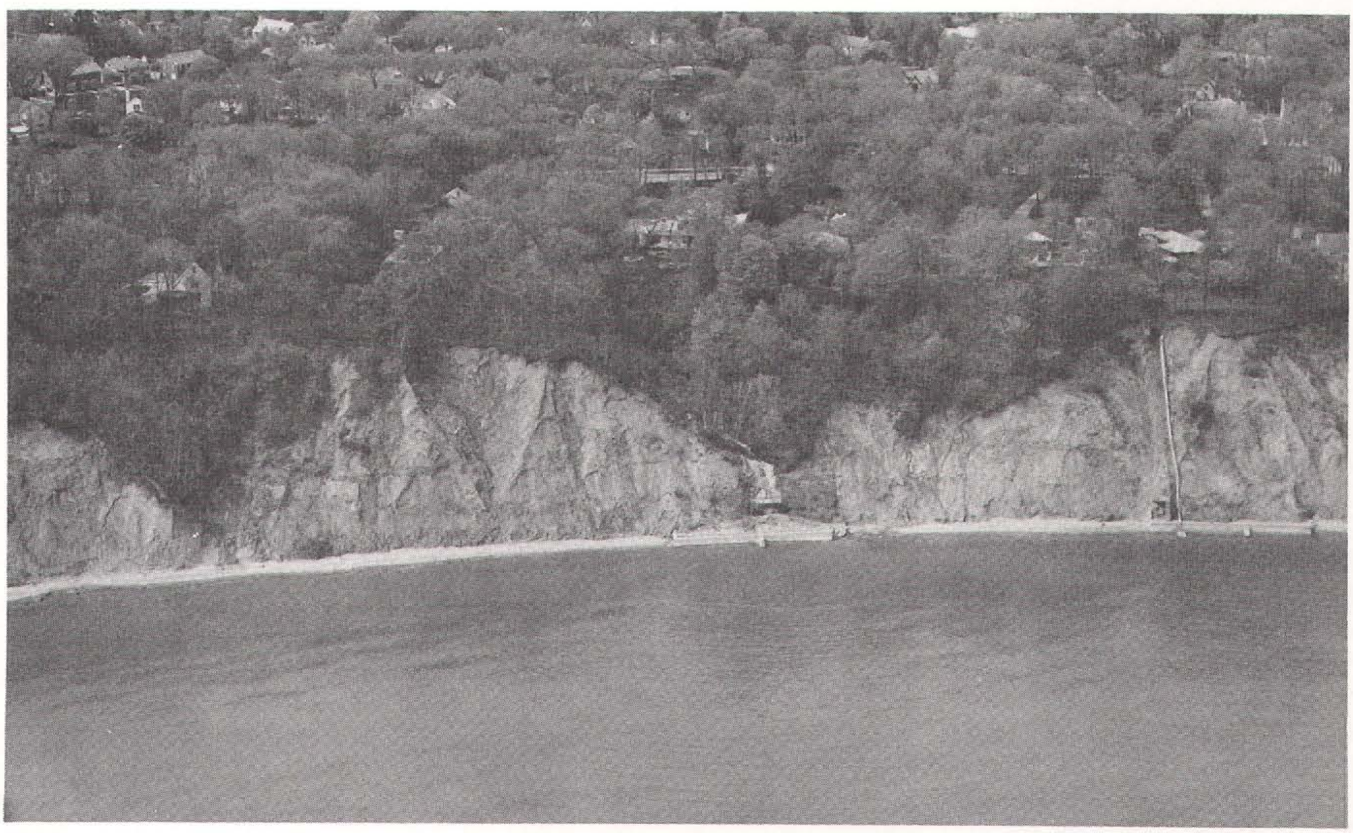
Figure 23

BLUFF ANALYSIS SECTION 11

Source: SEWRPC.

Figure 24

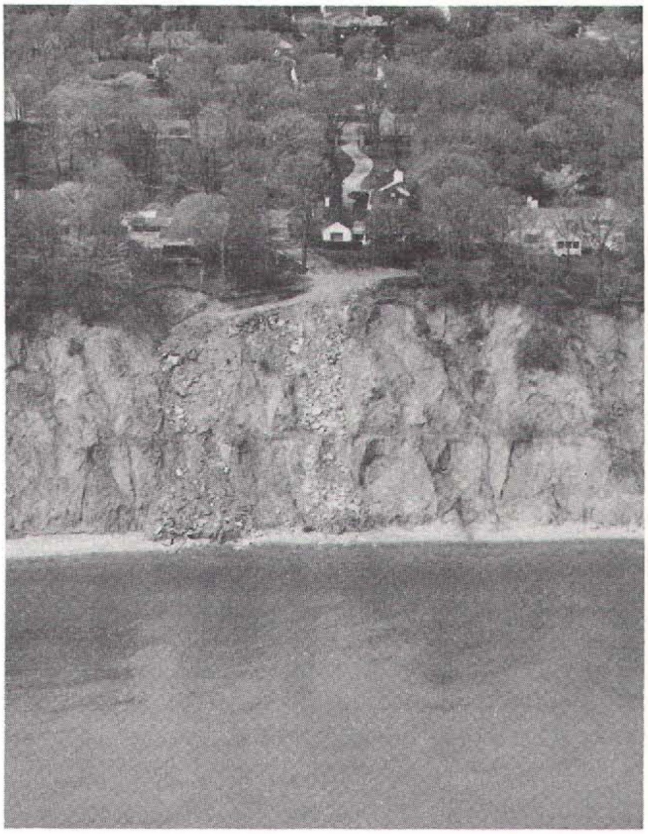
BLUFF ANALYSIS SECTION 12



Source: SEWRPC.

Figure 25

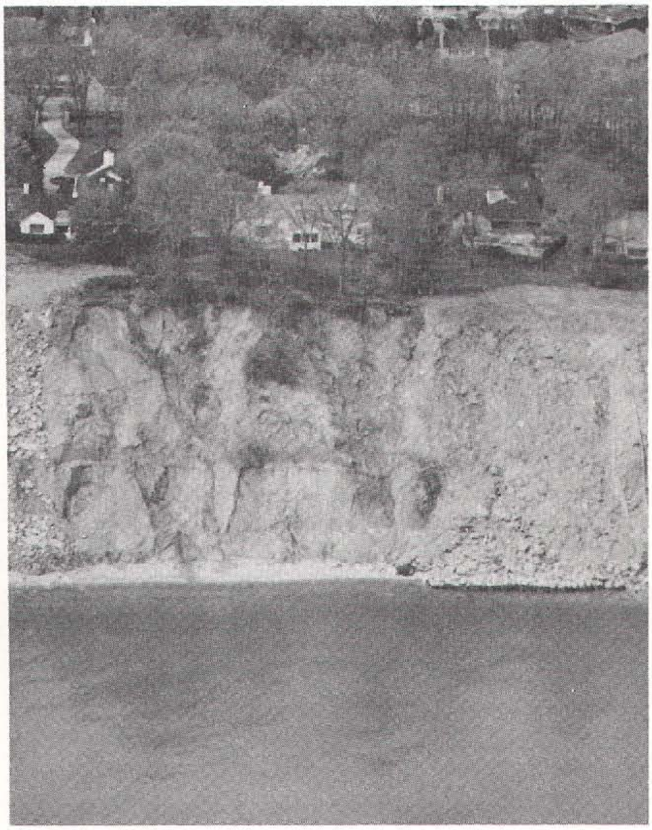
BLUFF ANALYSIS SECTION 13



Source: SEWRPC.

Figure 26

BLUFF ANALYSIS SECTION 14



Source: SEWRPC.

steep, approximating 33 degrees. At the time of the field survey in May 1986, there was no bluff toe protection provided in the section.

Section 14, at 4780 N. Lake Drive, had a beach less than 10 feet wide and a bluff height ranging from 90 to 95 feet. The composition of the bluff was Ozaukee till underlain by layers of silt and sand, Oak Creek till, and New Berlin till. As shown in Figure 26, the slope of the bluff was fairly steep, with an angle of 36 degrees. Like Section 12, wave erosion at the bluff toe was the primary cause of bluff recession, as it was rapidly removing the material at the base of the bluff. Slope failure occurred as shallow slides and slumping. No groundwater seepage was noted during the field surveys.

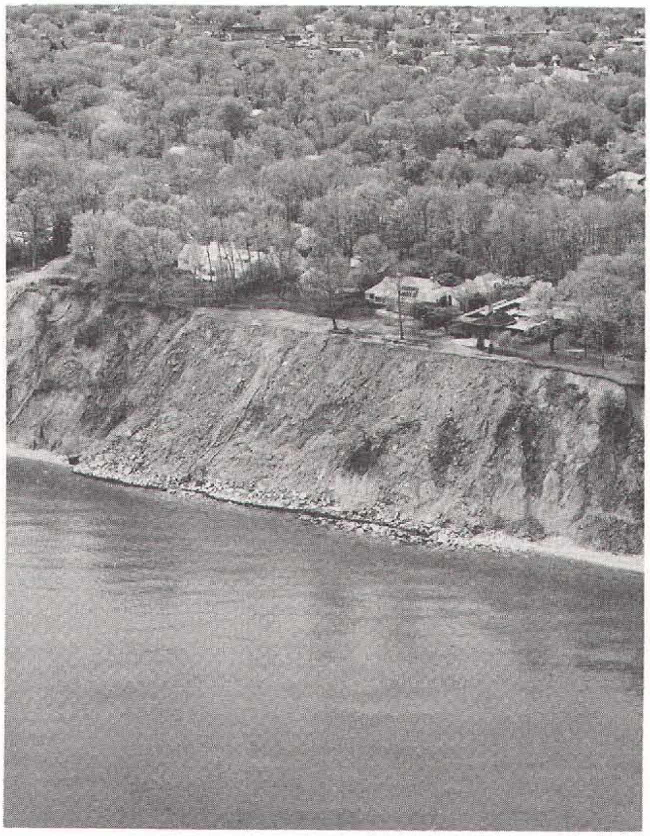
Section 15, which extends from 4790 to 4800 N. Lake Drive, was a fill which covered 310 linear feet of shoreline. Construction of the fill began in 1985. The width of the beach was less than 10 feet and the bluff height ranged from 80 to 90 feet. The slope of the bluff remained relatively steep, approximating 36 degrees. Figure 27 shows a tension crack that was observed at the top of the bluff. The fill project included a steel crib installed along the base of the bluff and a revetment composed of concrete, rubble, stone blocks, and grout-filled bags to protect the toe of the bluff, as shown in Figure 27. Two metal drainage pipes extended down the face of the bluff, discharging at the toe.

Section 16, which extends from 4810 to 4840 N. Lake Drive, had a beach width of less than 10 feet and bluff height was about 80 feet. The composition of the bluff was Ozaukee till underlain by silt and sand, Oak Creek till, and New Berlin till. As shown in Figure 28, the slope of the bluff was extremely steep, approximating 40 degrees, and the face of the bluff was free of vegetation. The houses within this section are set back over 100 feet from the edge of the bluff, and are separated from the bluff edge by a ravine. Wave erosion at the bluff toe was the primary cause of bluff recession. Slope failure occurred as shallow slumps and slides. No groundwater seepage was noted during the field surveys.

Section 17, which extends from 4850 to 4940 N. Lake Drive, was a fill project constructed during the period 1975 to 1986. The southern 200 feet of

Figure 27

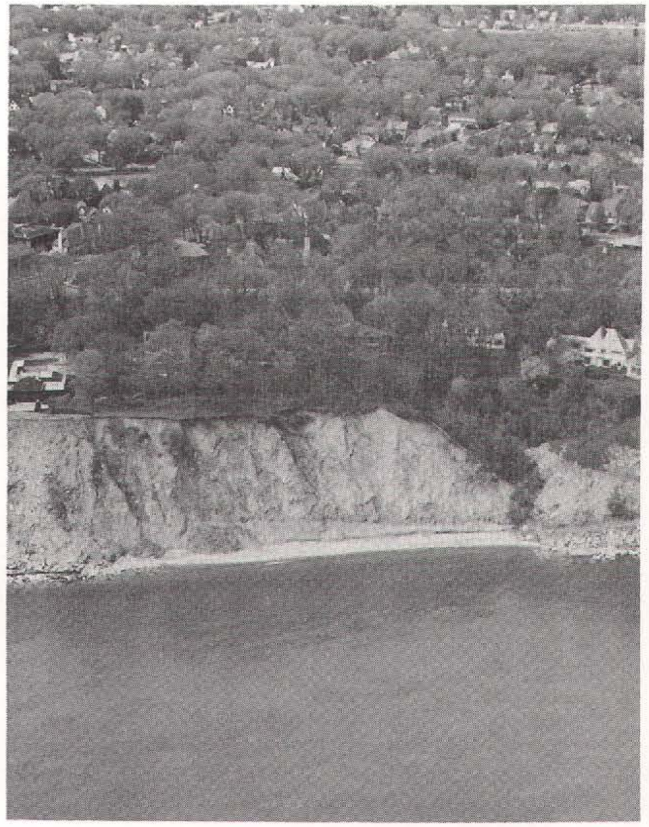
BLUFF ANALYSIS SECTION 15



Source: SEWRPC.

Figure 28

BLUFF ANALYSIS SECTION 16



Source: SEWRPC.

this section was a coarse rubble fill that, at the time of the field survey in May 1986, was still under construction. There was no beach, and the bluff height ranged from 65 to 80 feet. A revetment composed of rubble and concrete slabs was being placed at the toe of the fill for protection against wave action. The fill adjacent to the north side of this project extended for a distance of 1,200 feet and included the southern portion of Buckley Park. As shown in Figure 29, the bluff had been terraced and was well vegetated. The toe of the fill was protected by a concrete slab revetment which was being overtopped. The fill was therefore being eroded at the toe of the bluff.

Section 18, which included Buckley Park and the southern portion of Big Bay Park, had a beach width up to 25 feet and bluff heights ranging from 65 to 80 feet. The composition of the bluff was Ozaukee till, underlain by silt and sand, Oak Creek till, and New Berlin till. The bluff face was heavily vegetated and had a slope of 24 degrees, as shown in Figure 30. No bluff failures were observed during the May 1986 field surveys; however, some dislocation of trees was noted. In November 1986, a very large slump occurred at the southern end of this section, in Buckley Park. The toe of the bluff was protected by a concrete-stepped bulkhead, which was being overtopped.

Section 19, which extends from the northern portion of Big Bay Park to 5270 N. Lake Drive, was a series of fill projects constructed between 1973 and 1982 and covering a shoreline length of 1,480 linear feet. There was no beach within this section, and the bluff heights ranged from 65 to 80 feet. Figure 31 illustrates the severe erosion problem that was occurring within the southern portion of this section in 1976, and the improved slope stability provided by the fill projects, as shown in the 1986 photograph. The fill, which has been terraced, was beginning to hold a vegetative cover. Generally, a revetment protected the toe of the fill; however, in the northern portion of the fill area, a concrete bulkhead supported by wooden pilings protected the toe.

Figure 29

BLUFF ANALYSIS SECTION 17

Source: SEWRPC.

Figure 30

BLUFF ANALYSIS SECTION 18

Source: SEWRPC.

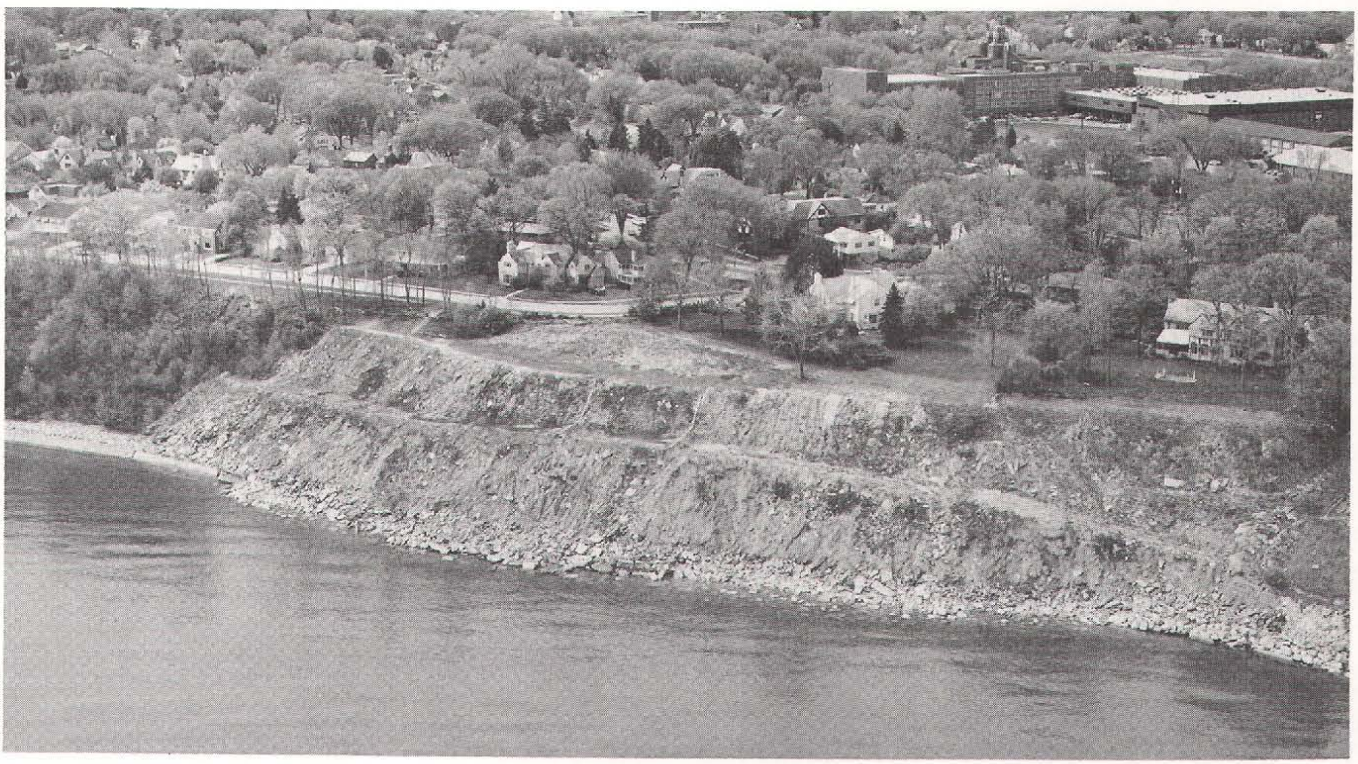
Figure 31

BLUFF ANALYSIS SECTION 19: 1976 AND 1986

1976

Source: James G. Rosenbaum.

1986



Source: SEWRPC.

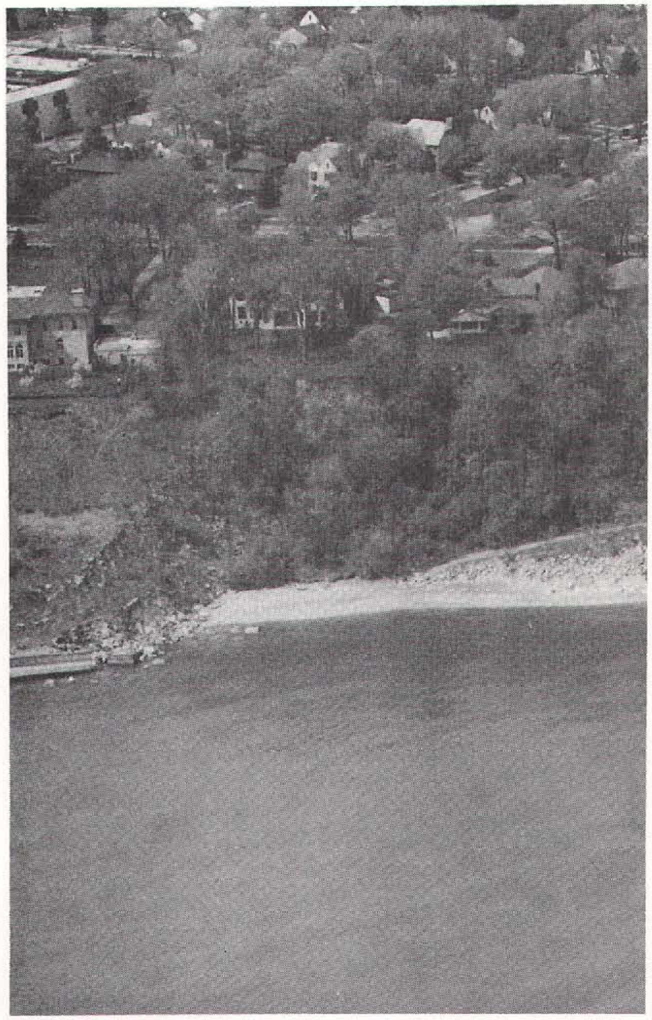
Section 20, at 5290 N. Lake Drive, was a 130-foot-long section adjoined on the north and south by fill projects. This section had a beach less than 10 feet in width and a bluff height of about 70 feet. The composition of the bluff was Ozaukee till underlain by layers of silt and clay, Oak Creek till, and New Berlin till. The bluff face was almost entirely vegetated, with a slope of 29 degrees. Bluff failure in the form of shallow slides was observed on the top portion of the bluff. The toe of the bluff was protected by a beach that had been built up from the fill projects to the north and south (see Figure 32). No groundwater seepage was noted during the May 1986 field surveys.

Section 21, which extends from 5300 N. Lake Drive to 808 Lakeview Avenue, was a series of fill projects constructed from 1974 to 1986. There was no beach within this section, and bluff heights ranged from 80 to 85 feet. As shown in Figure 33, a photograph of the southern portion of the section taken in September 1976 showed a large amount of fill placed along the lower bluff slope to protect the toe of the bluff. A concrete slab revetment protects the base of that fill. A photograph taken in May 1983 north of E. Silver Spring Drive, as also shown in Figure 33, shows that severe slumping occurred prior to the construction of a fill project north of E. Silver Spring Drive. That fill, as shown in a 1986 photograph in Figure 33, was terraced, with a concrete rubble and rock revetment protecting the base of the fill.

In Section 22, which extends from 5722 to 5770 N. Shore Drive, the beach width ranged up to 50 feet and bluff heights ranged from 80 to 85 feet. The composition of the bluff was Ozaukee till underlain by silt and sand and Oak Creek till. A comparison of the bluff in 1973 and 1986 (see Figure 34) appears to indicate that the bluff had reached a more stable configuration in 1986. It is common after each incremental slope failure for the soil masses to assume a temporary stable configuration until the net effect of many factors again decreases slope stability. In May 1986, the face of the bluff was vegetated, with a slope angle of approximately 35 degrees. Groundwater seepage was observed in this section. Because of the fairly wide beach, the base of the bluff was experiencing only minor wave erosion. Slope failure was occurring as slumping and shallow slides.

Figure 32

BLUFF ANALYSIS SECTION 20



Source: SEWRPC.

Section 23, at 758 E. Day Avenue, covered 140 linear feet of shoreline. This section was a coarse rubble fill constructed prior to 1976. The beach width in this section was 35 feet and the bluff height was 85 feet. As shown in Figure 35, except at the toe of the bluff, the fill is vegetated with a slope of 30 degrees. A 20-foot-high scarp at the toe of the fill caused by wave erosion was observed during the survey of May 1986.

Section 24, which extends from 740 E. Day Avenue to 5866 N. Shore Drive, had a beach width of 25 to 35 feet and a bluff height of 80 to 85 feet. The bluff was composed of Ozaukee till underlain by silt and sand and Oak Creek till. The face of the bluff was vegetated, with a

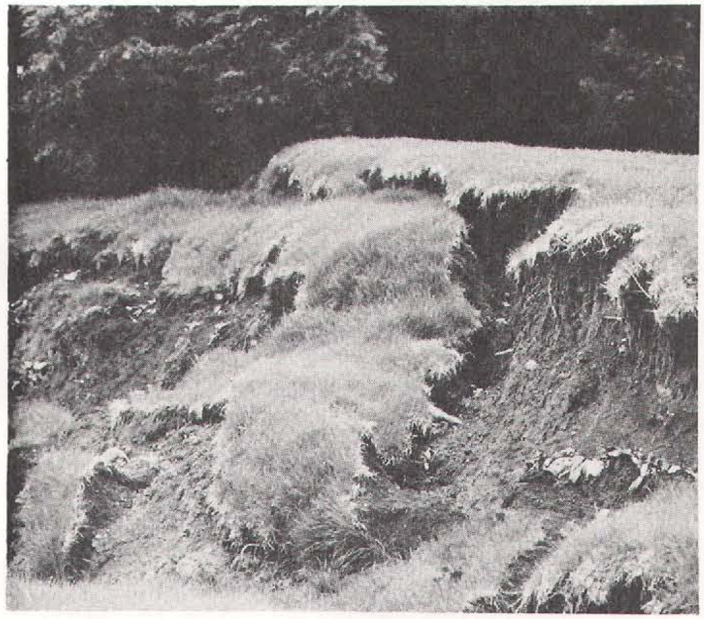
Figure 33

BLUFF ANALYSIS SECTION 21: 1976, 1983, 1986

1976

Source: James G. Rosenbaum.

1983



Source: SEWRPC.

1986

Source: SEWRPC.

Figure 34

BLUFF ANALYSIS SECTION 22: 1973 AND 1986

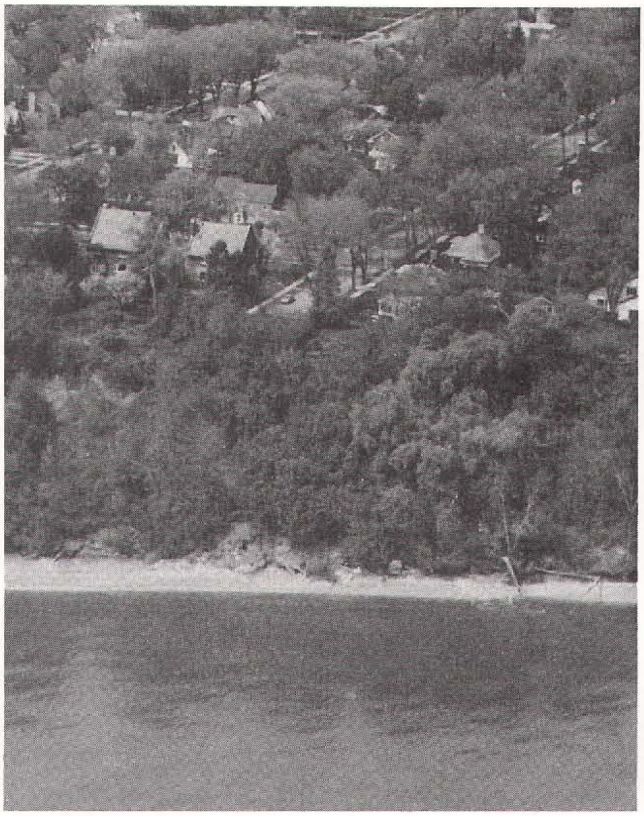
1973

1986

Source: SEWRPC.

Figure 35

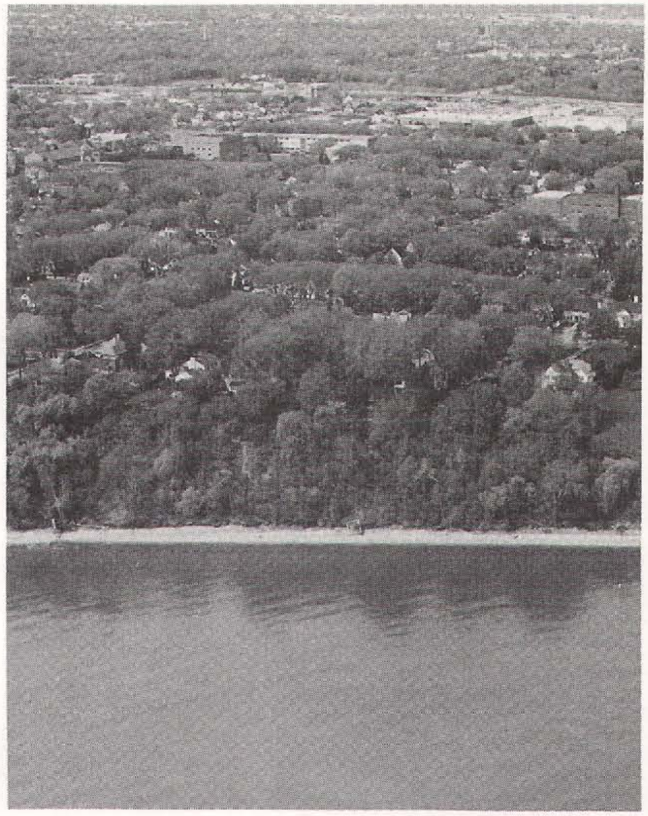
BLUFF ANALYSIS SECTION 23



Source: SEWRPC.

Figure 36

BLUFF ANALYSIS SECTION 24



Source: SEWRPC.

slope of 35 degrees. Groundwater seepage was observed from the sand and silt layer. Wave erosion also appeared to be a major cause of bluff failure. As shown in Figure 36, slope failure occurred as slumping along the silt and sand layer. The base of the bluff was composed of slumped lake sediment and till material.

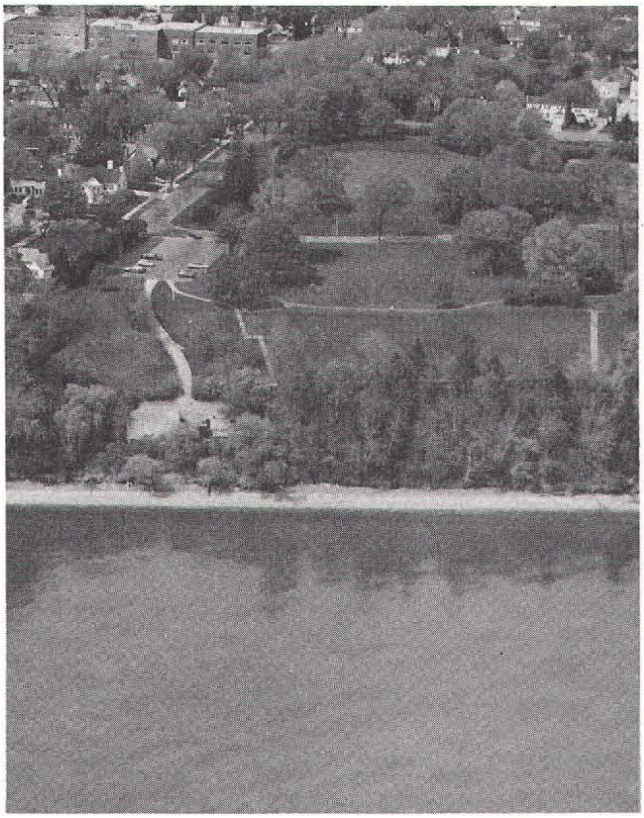
Section 25, which comprises the shoreline of Klode Park, had a beach width ranging from 20 to 35 feet and bluff heights ranging from 75 to 80 feet. As in Section 24, the bluff was composed of Ozaukee till underlain by silt and sand and Oak Creek till. The top portion of the slope was regraded to a stable slope, and the face of the lower portion of the bluff was vegetated, with a slope of approximately 27 degrees, as shown in Figure 37. Although groundwater seepage was observed in the lower portion of the bluff, there were no major bluff failures evident during the May 1986 field survey. The toe of the bluff was protected by a bulkhead which showed evidence of overtopping. As noted above, a portion of this bulkhead collapsed during a December 1986 storm, and in April 1987, a massive slope failure occurred just north of the North Shore Water Commission pumping station in Klode Park.

Section 26, at 5960 N. Shore Drive, directly north of Klode Park, had a beach width ranging from 35 to 40 feet and a bluff height of 80 to 90 feet. The composition of the bluff was Ozaukee till underlain by silt and sand and Oak Creek till. The face of the bluff was mostly vegetated, and the slope was approximately 32 degrees. Groundwater seepage in the silt and sand layer had caused major slumping to occur, as shown in Figure 38. The wide beach was providing some protection against wave erosion at the base of the bluff in the summer of 1986.

Section 27, which extends from 6000 N. Shore Drive to 6260 N. Lake Drive, had a beach width of 35 to 50 feet and bluff heights ranging from 90 to 120 feet. The composition of the bluff was Ozaukee till underlain by silt and sand, sand, and Oak Creek till. In the southern portion of the section, the bottom of the bluff was covered by

Figure 37

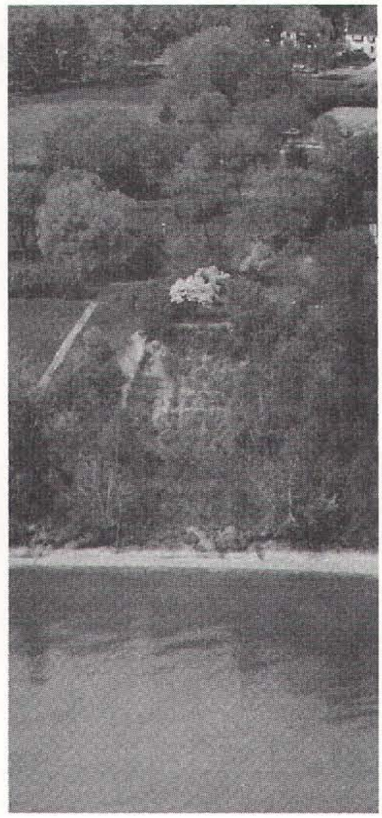
BLUFF ANALYSIS SECTION 25



Source: SEWRPC.

Figure 38

BLUFF ANALYSIS SECTION 26



Source: SEWRPC.

debris from an old slump block. The slope was completely vegetated, with an angle of about 26 degrees. Generally, the top of the bluff appeared stable, while the bottom portion exhibited some slope failure in the form of small slips and slump blocks being caused by groundwater seepage and wave erosion, as shown in Figure 39.

Section 28, which extends from 6310 to 6424 N. Lake Drive, had a beach width ranging from 25 to 40 feet and bluff heights ranging from 115 to 125 feet. The composition of the bluff was Ozaukee till underlain by alternating layers of fine sand and silt and sand, and by Oak Creek till. As shown in Figure 40, there was more evidence of slope failure than in Section 27, and the bluff slope was slightly steeper, approximately 30 degrees. Some groundwater seepage was observed in the silt and sand layer, contributing to the slumping which was occurring on the bottom half of the slope. Slumped till and silt covered the toe of the bluff. Wave erosion had formed a rapidly eroding toe with a steep lower slope and more gentle upper slope. Slope failure was in the form of surface sliding and slumping.

As shown in Figure 41, Section 29, which extends from 6430 to 6448 N. Lake Drive, was a concrete rubble fill project under construction in 1986, covering about 320 linear feet of shoreline. There was no beach in this section, and the bluff height ranged from 120 to 125 feet. The slope of the fill was terraced, with an overall slope angle of approximately 25 degrees. Protection of the base of the fill was being provided by a revetment composed of large concrete blocks and slabs.

Section 30, which extends from 6464 to 6530 N. Lake Drive, had a beach less than 10 feet wide and bluff heights ranging from 115 to 125 feet. The composition of the bluff was Ozaukee till underlain by silt and sand. As shown in Figure 42, the lower half of the slope was unvegetated, while the upper half was well vegetated with a slope of 30 degrees. The primary cause of bluff failure in this section was wave action. Ground-

Figure 39

BLUFF ANALYSIS SECTION 27

Source: SEWRPC.

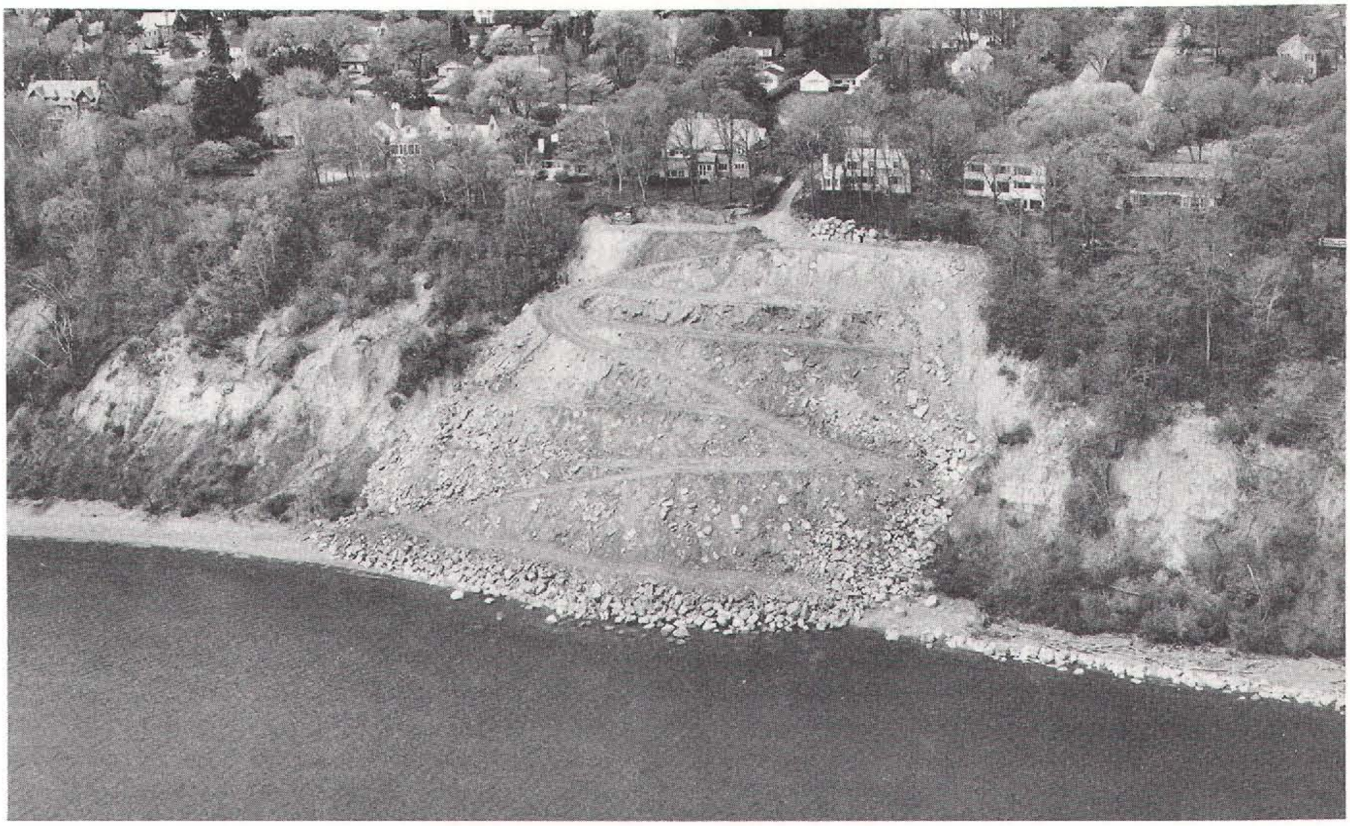
Figure 40

BLUFF ANALYSIS SECTION 28

Source: SEWRPC.

Figure 41

BLUFF ANALYSIS SECTION 29



Source: SEWRPC.

water seepage was also present on the top of the silt layer. Slope failure was occurring as shallow slides and slumps. As in Section 28, the material at the toe of the bluff was slumped till and silt debris.

In Section 31, which extends from 6600 to 6702 N. Lake Drive, a beach had been built up to 50 feet in width by a small groin system. A bulkhead present behind the groin system offered additional protection against wave action. Bluff heights ranged from 120 to 125 feet. The bluff was composed of Ozaukee till underlain by silt and sand. As shown in Figure 43, the entire face of the bluff was vegetated, with a slope of 35 degrees. Although the bluff slope was more stable than in Section 30, shallow slides and small slumps were observed during the May 1986 field survey. Minimal wave erosion was observed owing to the relatively wide beach. No groundwater seepage was observed on the bluff face.

Section 32, which extends from 6720 N. Lake Drive to 6818 N. Barnett Lane, had a beach up to 15 feet wide and a bluff height ranging from 115 to 125 feet. The composition of the bluff in the southern portion of the section was Ozaukee till underlain by silt and sand, which extended down to the lake level. In the northern portion of the section, the silt and sand layer was underlain by Oak Creek till and New Berlin till, and a small exposure of bedrock was present at the lake level. The lower portion of the slope was unvegetated, as shown in Figure 44. The bluff had an overall slope of about 32 degrees. Much of the debris on the bottom portion of the bluff was slump material, composed of sand and silt. Wave erosion in the summer of 1986 was the primary cause of bluff failure, and no significant groundwater seepage was observed. Within this section, surface slides and small slumps were present along the lower portion of the slope.

Section 33, which extends from 6820 to 6840 N. Barnett Lane, did not have a beach, and bluff heights ranged from 115 to 120 feet. The composition of the bluff was Ozaukee till underlain by silt and sand, Oak Creek till, and New Berlin till. As shown in Figure 45, nearly the entire face of the bluff was vegetated, with a slope of approximately 26 degrees. The upper portion of the bluff appeared to be stable. Wave erosion at the bluff toe had, however, caused some slumping and shallow slides on the lower portion of the slope. No groundwater seepage was noted during the May 1986 survey.

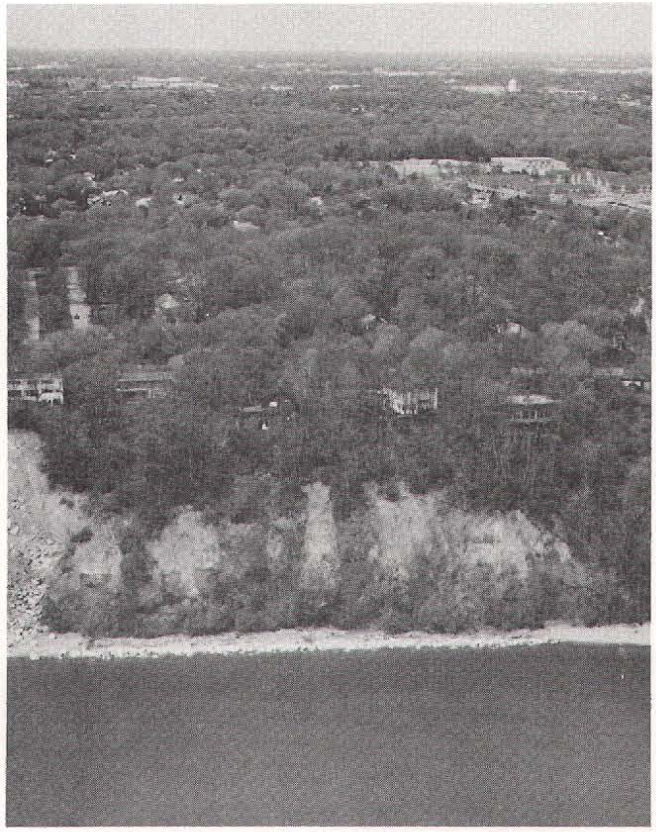
Section 34, which extends from 6868 to 7004 N. Barnett Lane, had a beach width of less than 10 feet and bluff heights ranging from 115 to 120 feet. The composition of the bluff was Ozaukee till underlain by sand, silt, Oak Creek till, and New Berlin till. As shown in Figure 46, the lower half of the slope was unvegetated and the upper half was well vegetated. The slope of the lower portion of the bluff was extremely steep, approximately 45 degrees, with an average overall bluff slope of about 35 degrees. The primary cause of bluff failure in this section was wave erosion, which caused shallow sliding and slumping of the undercut upper units. No groundwater seepage was observed during the field surveys.

Section 35, which extends from 7000 to 8130 N. Beach Drive, was comprised of a 9,070-foot-long terrace. As shown in Figure 47, most of this section has no beach, although a 65-foot-wide beach contained by a groin was present directly north of the east-west segment of Beach Drive. The height of the terrace ranged from 4 to 10 feet, and the terrace was composed of sand underlain by New Berlin till. Although a variety of shoreline protection structures have been constructed along the terrace, wave and ice erosion damage was observed at several sites. About 9,000 feet of eight-inch-diameter vitrified clay sanitary sewer is located along the shoreline in Section 35, along with about 34 manholes. Although originally built on the shore, much of the sewer and several of the manholes, because of erosion of the terrace, now lie within Lake Michigan and are subject to direct damage from wave and ice action. The sewer was constructed in about 1935, and shows signs of advanced deterioration.

Section 36, which comprises the portion of the study area within Doctors Park, had a beach width of less than 10 feet and a bluff height of about 90 feet. The composition of the bluff was undetermined because of the heavy vegetation on the bluff face (see Figure 48). The slope of the bluff was gentle, with an angle of approximately 25 degrees. No bluff slope failures were observed during the May 1986 field surveys. The toe of most of the bluff was protected by a concrete bulkhead, although the extreme southern portion of the section was not protected. This southern portion was experiencing wave erosion, although this erosion did not appear to be affecting the stability of the bluff slope itself.

Figure 42

BLUFF ANALYSIS SECTION 30



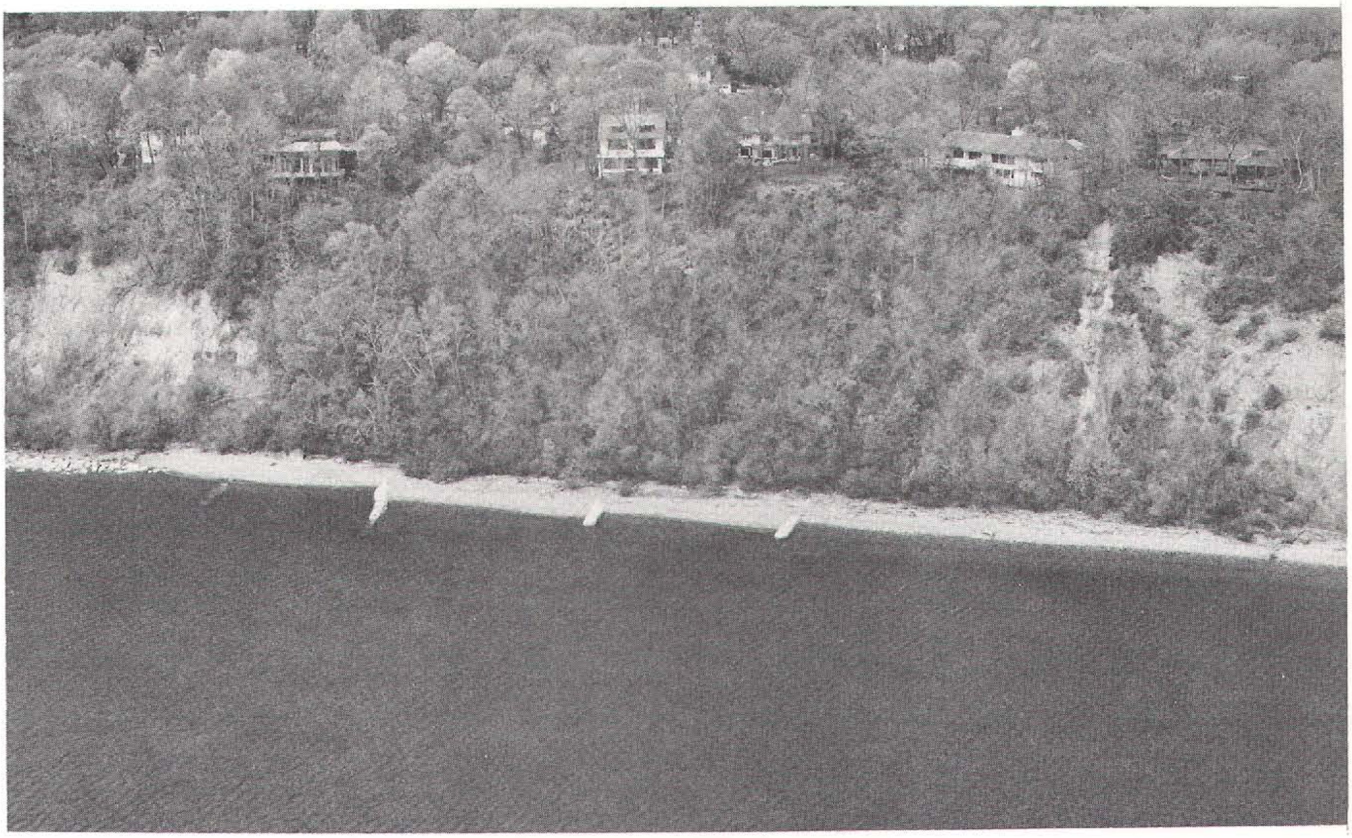
Source: SEWRPC.

BLUFF RECESSION RATES

The rate of bluff recession may be estimated by measuring the change in location of a bluff edge over a specified time period. For the purpose of this report, the term bluff recession includes the erosion and recession of the entire shoreline—both the bluffs and the Fox Point terrace. Bluff recession rates for northern Milwaukee County were measured for two different time spans using Regional Planning Commission ratioed and rectified, one inch equals 400 feet scale aerial photographs; the Commission one inch equals 100 feet scale, two-foot contour interval topographic maps; and U. S. Army Corps of Engineers one inch equals 200 feet scale aerial photographs. 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 bluff recession perpendicular to the shoreline. The

Figure 43

BLUFF ANALYSIS SECTION 31



Source: SEWRPC.

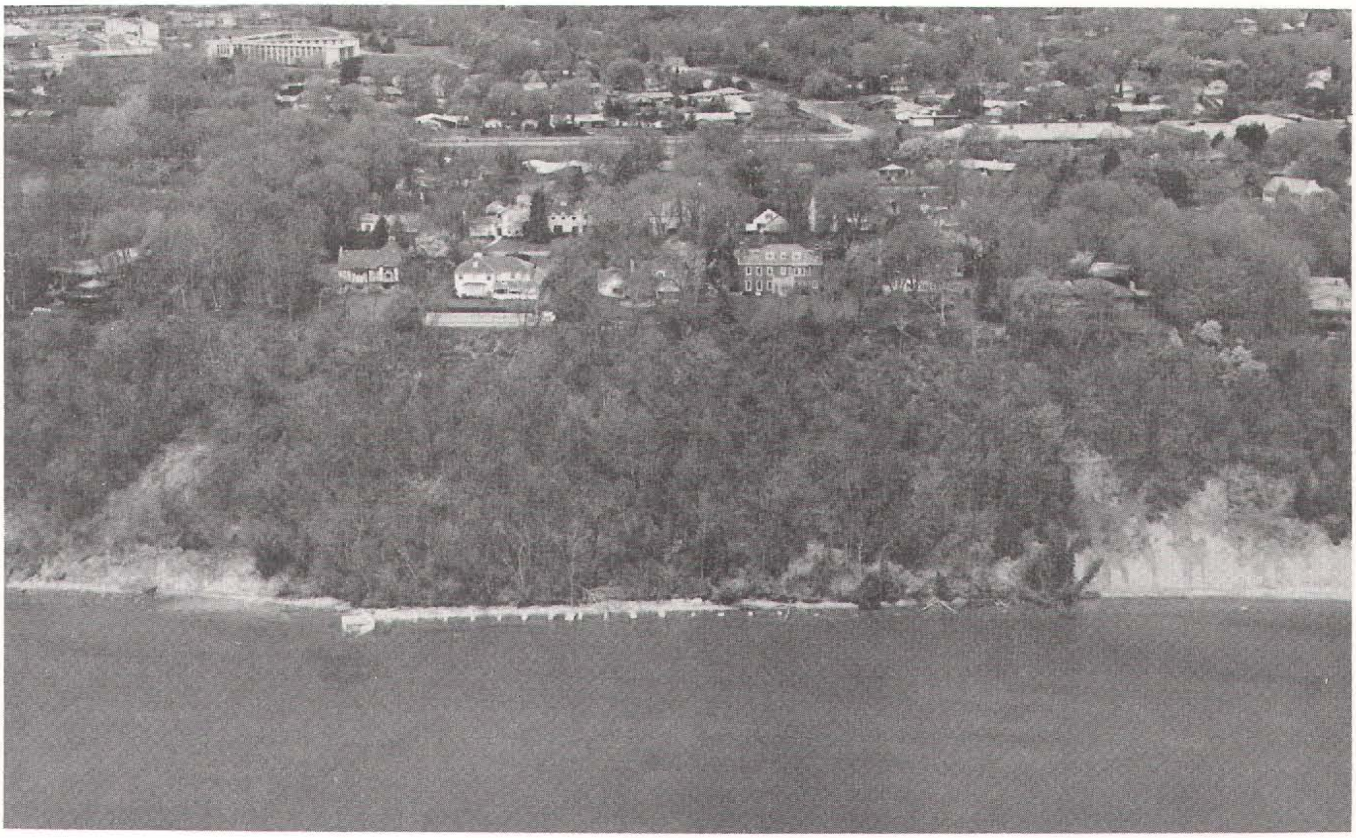
Figure 44

BLUFF ANALYSIS SECTION 32

Source: SEWRPC.

Figure 45

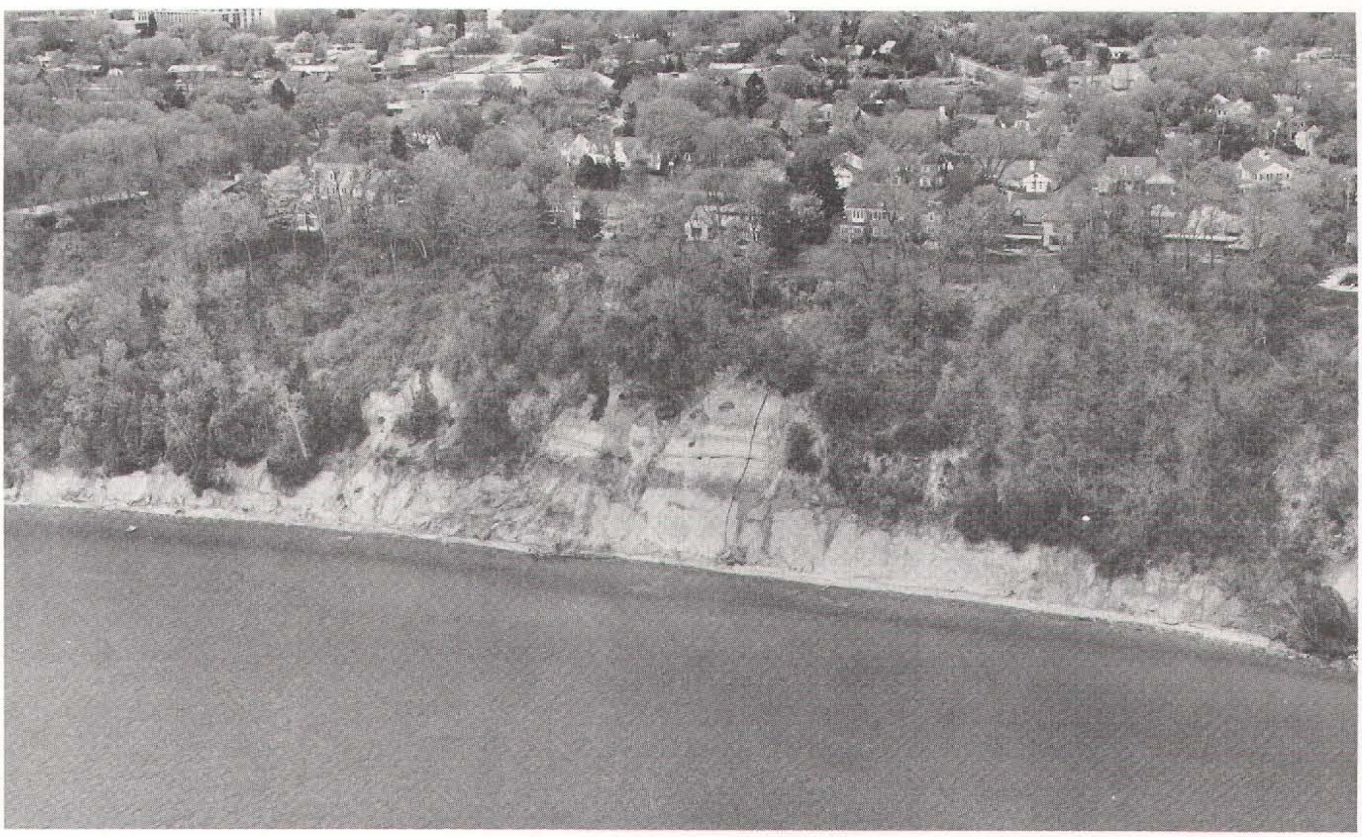
BLUFF ANALYSIS SECTION 33



Source: SEWRPC.

Figure 46

BLUFF ANALYSIS SECTION 34



Source: SEWRPC.

Figure 47

BLUFF ANALYSIS SECTION 35

Source: SEWRPC.

Figure 48

BLUFF ANALYSIS SECTION 36

Source: SEWRPC.

recession rates were calculated by measuring the distance from a fixed landmark to the top edge of the bluff.

Bluff 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 173 bluff recession reaches, which are shown on Map 14. The shoreline length of these reaches ranges from 200 feet to 350 feet, with many reaches having a shoreline length exceeding 200 feet, the shoreline often being not parallel to the north-south section lines. The combined length of the bluff recession reaches is 38,770 feet. The recession rate estimates made for the period March 1963 to April 1985 covered a period of rising lake levels from record low levels in 1964 to near record high levels in 1985. The period did not include the record high levels and severe storm conditions in the fall of 1985 and in 1986. Thus, the rates estimated may be somewhat conservatively lower than would be expected if the data were based upon this most recent 24 years of record. However, the rates should be representative of a long-term trend with a range of lake levels. Recession rate estimates made for the period April 1978 to April 1985 can be used to help assess the benefits of those shore protection structures that were installed in the 1970's and 1980's.

Table 18 presents the measured bluff recession rates for the time periods 1963 through 1985, and 1978 through 1985 for each bluff recession reach. For comparison purposes, long-term recession rates over the period 1836 through 1969, as reported by the U. S. Army Corps of Engineers, are also shown in the table for a few locations. Shoreline length, bluff height, and the volume of bluff material lost for each reach are also presented in the table. The recession rates for the period 1963 through 1985 ranged from less than 0.5 foot per year to 1.5 feet per year, with the length-weighted mean recession rate for those areas with rates greater than or equal to 0.5 foot per year being 0.59 foot per year. The highest recession rates were measured at the southern portion of the Fox Point terrace, and just south of E. Silver Spring Drive. Fill was placed on the bluff south of E. Silver Spring Drive in the mid 1970's. For the period 1978 through 1985 the recession rates ranged from less than 0.5 foot per year to 1.0 foot per year, with a length-weighted mean rate of 0.52 foot per year.

The volume of bluff material lost by erosion annually is also set forth in Table 18. Bluff recession, as measured from 1963 through 1985, resulted in the annual loss of approximately 585,600 cubic feet of bluff material.

A summary of measured bluff top recession rates and associated shoreline lengths and the volume of material loss to erosion is provided in Table 19. About 95 and 98 percent of the shoreline measured in 1963 through 1985 and 1978 through 1985, respectively, had bluff recession rates of 0.5 foot per year or less. About 1 percent of the shoreline measured in 1963 through 1985, and none of the shoreline measured in 1978 through 1985, had bluff recession measurements exceeding 1.0 foot per year. The 5 percent of the total study area shoreline exhibiting a recession rate exceeding 0.5 foot per year from 1963 through 1985 accounted for about 16 percent of the total bluff material loss in the study area. Figure 49 graphically illustrates the recession rates measured for 1963 through 1985, and for 1978 through 1985.

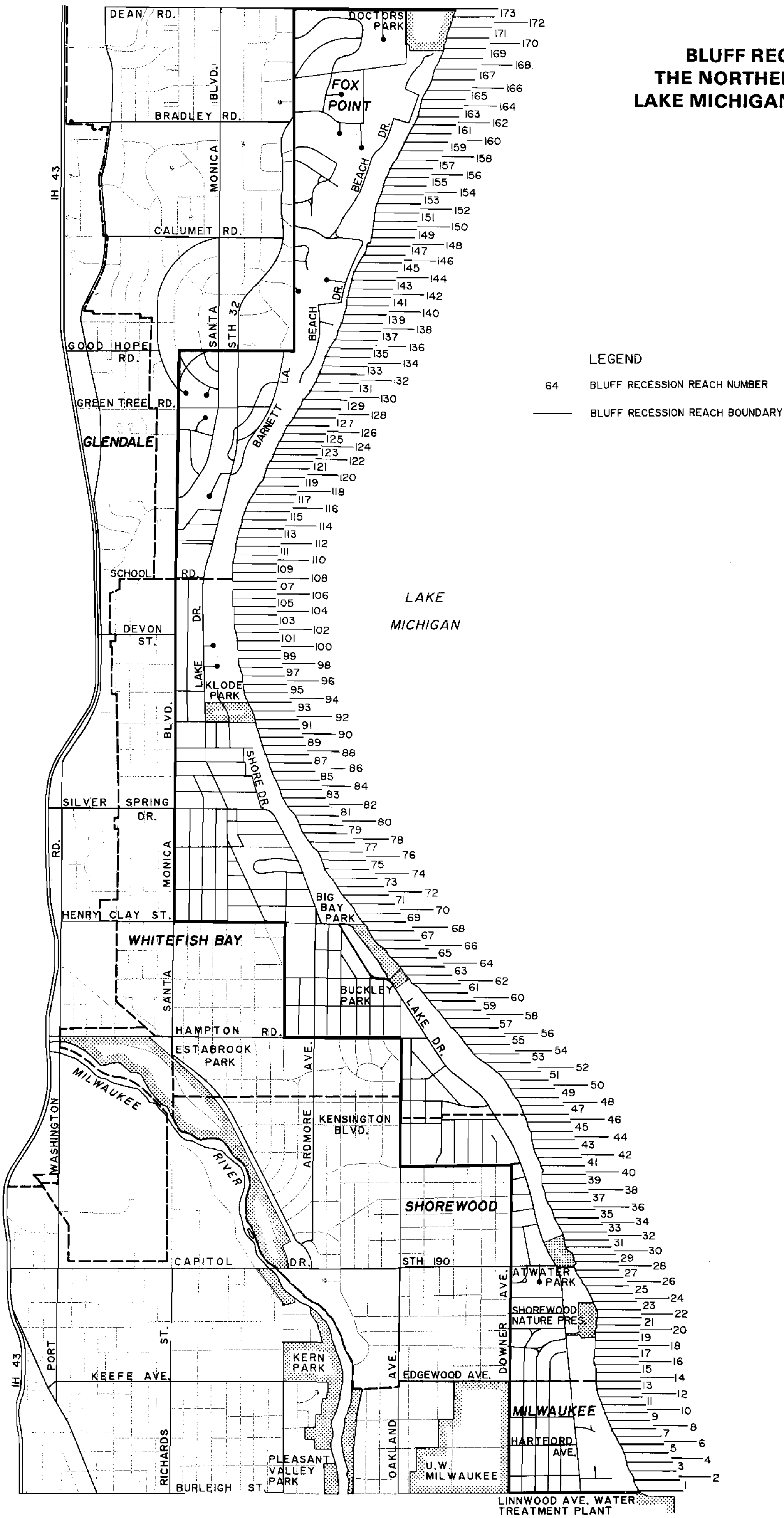
The actively eroding shoreline areas within northern Milwaukee County are apparently receding at a much slower rate than other shoreline areas in southeastern Wisconsin studied by the Regional Planning Commission. Table 20 compares the average recession rates measured in northern Milwaukee County to recession rates measured in Racine County, the City of St. Francis in southern Milwaukee County, and the Chiwaukee Prairie-Carol Beach shoreline area in southern Kenosha County.[9] The table indicates that the recession rates in these other southeastern Wisconsin shoreline areas averaged 1.5 to 5.6 feet per year, or over

---

[9]*See SEWRPC Community Assistance Planning Report No. 86, A Lake Michigan Coastal Erosion Management Study for Racine County, Wisconsin, 1982; SEWRPC Community Assistance Planning Report No. 110, A Lake Michigan Coastal Erosion and Related Land Use Management Study for the City of St. Francis, Wisconsin, 1984; and SEWRPC Community Assistance Planning Report No. 88, A Land Use Management Plan for the Chiwaukee Prairie-Carol Beach Area of the Town of Pleasant Prairie, Kenosha County, Wisconsin, 1985.*

Map 14

BLUFF RECESSION REACHES IN THE NORTHERN MILWAUKEE COUNTY LAKE MICHIGAN SHORELINE STUDY AREA



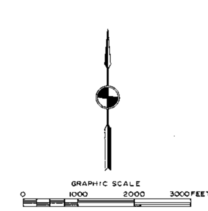


Table 18

SHORELINE RECESSION RATES ALONG THE NORTHERN MILWAUKEE COUNTY LAKE MICHIGAN SHORELINE

Bluff Analysis Section	Shoreline Recession Reach	Shoreline Length (feet)	Bluff Height (feet)	Annual Recession Rates (feet/year): Long Term[a] 1836-1969	Annual Recession Rates (feet/year): Short Term 1963-1985	Annual Recession Rates (feet/year): Short Term 1978-1985	Annual Volume of Shore Material Loss[b] (cubic feet per year)
1	1	210	75	1.5	< 0.5	< 0.5	- -
	2	210	80	- -	0.5	< 0.5	8,400
	3	230	75	- -	< 0.5	< 0.5	- -
	4	200	80	- -	< 0.5	< 0.5	- -
	5	200	80	- -	0.5	< 0.5	8,000
	6	200	90	- -	< 0.5	< 0.5	- -
	7	230	90	- -	< 0.5	< 0.5	- -
	8	240	90	- -	< 0.5	0.5	- -
	9	320	90	- -	< 0.5	0.5	- -
2	10	200	100	- -	< 0.5	< 0.5	- -
	11	200	100	- -	0.5	0.5	10,000
	12	210	100	- -	< 0.5	< 0.5	- -
	13	210	95	- -	< 0.5	0.5	- -
	14	200	95	- -	0.5	0.5	9,500
3	15	210	95	- -	< 0.5	0.5	- -
4	16	200	100	- -	0.5	< 0.5	10,000
	17	210	100	- -	0.5	1.0	10,500
	18	200	105	- -	0.5	< 0.5	10,500
	19	200	100	- -	< 0.5	< 0.5	- -
	20	200	100	- -	0.5	< 0.5	10,000
	21	200	100	- -	< 0.5	< 0.5	- -
5	22	200	100	- -	< 0.5	< 0.5	- -
	23	250	105	- -	0.5	< 0.5	13,100
	24	290	110	- -	0.5	0.5	16,000
6	25	270	110	- -	< 0.5	< 0.5	- -
7	26	210	110	- -	0.5	0.5	11,600
	27	240	105	- -	< 0.5	0.5	- -
	28	200	90	- -	< 0.5	< 0.5	- -
8	29	250	90	- -	< 0.5	< 0.5	- -
	30	200	99	- -	< 0.5	< 0.5	- -
	31	210	90	- -	< 0.5	< 0.5	- -
	32	210	95	- -	0.5	0.5	10,000
	33	210	95	- -	< 0.5	0.5	- -
	34	220	95	- -	< 0.5	< 0.5	- -
	35	200	100	- -	0.5	0.5	10,000
	36	200	100	- -	< 0.5	< 0.5	- -
	37	200	100	- -	0.5	< 0.5	10,000
9	38	200	90	- -	< 0.5	< 0.5	- -
	39	220	115	- -	< 0.5	< 0.5	- -
10	40	200	115	- -	0.5	1.0	11,500
	41	200	110	- -	< 0.5	0.5	- -

Table 18 (continued)

Bluff Analysis Section	Shoreline Recession Reach	Shoreline Length (feet)	Bluff Height (feet)	Annual Recession Rates (feet/year) Long Term[a] 1836-1969	Short Term 1963-1985	Short Term 1978-1985	Annual Volume of Shore Material Loss[b] (cubic feet per year)
11	42	200	110	--	< 0.5	0.5	--
	43	200	115	--	< 0.5	0.5	--
	44	200	115	--	< 0.5	0.5	--
	45	230	110	--	< 0.5	< 0.5	--
	46	350	110	--	< 0.5	0.5	--
	47	250	100	--	< 0.5	< 0.5	--
	48	240	100	--	< 0.5	< 0.5	--
	49	260	95	--	1.0	0.5	24,700
	50	350	95	--	< 0.5	0.5	--
12	51	290	95	--	< 0.5	< 0.5	--
	52	260	95	--	0.5	0.5	12,400
	53	260	95	--	< 0.5	1.0	--
13	54	250	95	--	< 0.5	0.5	--
14	55	270	90	--	0.5	0.5	12,200
15	56	280	80	--	< 0.5	0.5	--
16	57	260	80	--	< 0.5	0.5	--
	58	250	80	--	< 0.5	< 0.5	--
17	59	250	70	--	< 0.5	< 0.5	--
	60	300	70	--	< 0.5	< 0.5	--
	61	250	70	--	< 0.5	< 0.5	--
	62	270	70	--	< 0.5	< 0.5	--
	63	250	65	--	< 0.5	< 0.5	--
	64	270	70	--	< 0.5	< 0.5	--
18	65	250	70	--	< 0.5	< 0.5	--
	66	260	75	--	< 0.5	< 0.5	--
	67	250	70	--	< 0.5	< 0.5	--
	68	240	70	--	< 0.5	< 0.5	--
19	69	300	65	--	< 0.5	< 0.5	--
	70	250	70	--	< 0.5	< 0.5	--
	71	250	75	--	< 0.5	< 0.5	--
	72	300	70	--	< 0.5	< 0.5	--
20	73	240	70	--	0.5	< 0.5	8,400
	74	240	70	--	0.5	< 0.5	8,400
21	75	210	80	--	1.0	< 0.5	16,800
	76	260	80	--	1.0	< 0.5	20,800
	77	240	80	--	0.5	< 0.5	9,600
	78	230	75	--	1.0	< 0.5	17,200
	79	250	80	--	< 0.5	< 0.5	10,000
	80	250	80	1.5	< 0.5	< 0.5	--
	81	250	85	--	< 0.5	< 0.5	--
	82	250	85	--	< 0.5	< 0.5	--
	83	210	85	--	< 0.5	< 0.5	--
	84	210	85	--	< 0.5	< 0.5	--
	85	230	80	--	0.5	0.5	--

Table 18 (continued)

Bluff Analysis Section	Shoreline Recession Reach	Shoreline Length (feet)	Bluff Height (feet)	Annual Recession Rates (feet/year)			Annual Volume of Shore Material Loss[b] (cubic feet per year)
				Long Term[a]	Short Term		
				1836-1969	1963-1985	1978-1985	
22	86	210	80	- -	< 0.5	0.5	- -
23	87	200	85	- -	0.5	0.5	8,500
24	88	210	85	- -	< 0.5	0.5	- -
	89	210	85	- -	< 0.5	0.5	- -
	90	200	80	- -	< 0.5	0.5	- -
25	91	200	75	- -	0.5	< 0.5	- -
	92	200	75	- -	0.5	< 0.5	7,500
	93	200	80	- -	0.5	< 0.5	8,000
26	94	200	90	- -	< 0.5	< 0.5	- -
	95	200	95	- -	< 0.5	< 0.5	- -
27	96	200	90	- -	< 0.5	0.5	- -
	97	200	90	- -	< 0.5	< 0.5	- -
	98	200	90	- -	0.5	0.5	9,000
	99	200	105	- -	0.5	0.5	10,500
	100	200	115	- -	0.5	< 0.5	11,500
	101	200	120	- -	< 0.5	0.5	- -
	102	200	120	- -	0.5	0.5	12,000
	103	200	115	- -	< 0.5	< 0.5	- -
	104	200	115	- -	0.5	0.5	11,500
28	105	200	120	- -	< 0.5	0.5	- -
	106	200	115	- -	< 0.5	0.5	- -
	107	200	125	1.5	0.5	0.5	12,500
	108	200	125	- -	0.5	< 0.5	12,500
	109	200	125	- -	0.5	< 0.5	12,500
29	110	200	120	- -	0.5	< 0.5	12,000
	111	220	120	- -	0.5	0.5	13,200
30	112	210	115	- -	0.5	0.5	12,100
	113	210	125	- -	0.5	< 0.5	13,100
31	114	200	120	- -	< 0.5	0.5	- -
	115	200	125	- -	< 0.5	< 0.5	- -
	116	200	125	- -	< 0.5	0.5	- -
32	117	250	120	- -	0.5	0.5	15,000
	118	250	120	- -	< 0.5	< 0.5	- -
	119	240	115	- -	0.5	< 0.5	13,800
33	120	200	120	- -	0.5	0.5	12,000
	121	220	120	- -	0.5	0.5	13,200
	122	220	115	- -	0.5	0.5	12,600
34	123	210	115	- -	< 0.5	0.5	- -
	124	240	120	- -	< 0.5	< 0.5	- -
	125	240	120	- -	< 0.5	0.5	- -
	126	240	120	- -	< 0.5	0.5	- -
	127	240	120	- -	0.5	< 0.5	14,400
35	128	220	25	- -	1.0	< 0.5	5,500

Table 18 (continued)

Bluff Analysis Section	Shoreline Recession Reach	Shoreline Length (feet)	Bluff Height (feet)	Annual Recession Rates (feet/year) Long Term[a] 1836-1969	Short Term 1963-1985	Short Term 1978-1985	Annual Volume of Shore Material Loss[b] (cubic feet per year)
35 (continued)	129	230	10	--	1.0	< 0.5	2,300
	130	250	5	--	< 0.5	< 0.5	--
	131	200	5	--	1.5	0.5	1,500
	132	210	5	--	< 0.5	< 0.5	--
	133	220	5	--	0.5	< 0.5	600
	134	220	5	--	0.5	< 0.5	600
	135	200	5	--	< 0.5	< 0.5	--
	136	200	5	0.5	0.5	< 0.5	500
	137	200	5	--	1.0	0.5	1,000
	138	200	5	--	< 0.5	< 0.5	--
	139	200	5	--	1.5	0.5	1,500
	140	200	5	--	< 0.5	< 0.5	--
	141	200	5	--	< 0.5	< 0.5	--
	142	200	5	--	< 0.5	< 0.5	--
	143	200	5	--	< 0.5	< 0.5	--
	144	210	5	--	< 0.5	< 0.5	--
	145	210	5	--	< 0.5	< 0.5	--
	146	290	5	--	< 0.5	< 0.5	--
	147	230	5	--	0.5	< 0.5	600
	148	200	5	--	0.5	< 0.5	500
	149	200	5	--	0.5	0.5	500
	150	200	5	--	< 0.5	0.5	--
	151	200	5	--	< 0.5	0.5	--
	152	210	5	--	< 0.5	0.5	--
	153	200	5	--	< 0.5	< 0.5	--
	154	210	5	--	< 0.5	< 0.5	--
	155	230	5	--	< 0.5	< 0.5	--
	156	200	5	--	< 0.5	< 0.5	--
	157	240	5	--	0.5	0.5	600
	158	230	5	--	< 0.5	< 0.5	--
	159	240	5	--	< 0.5	< 0.5	--
	160	270	5	--	0.5	0.5	700
	161	200	5	0.5	0.5	< 0.5	500
	162	200	5	--	< 0.5	< 0.5	--
	163	210	5	--	< 0.5	< 0.5	--
	164	220	5	--	< 0.5	0.5	--
	165	210	10	--	< 0.5	< 0.5	--
	166	220	10	--	< 0.5	< 0.5	--
	167	220	10	--	< 0.5	< 0.5	--
	168	250	10	--	< 0.5	< 0.5	--
	169	200	5	--	< 0.5	< 0.5	--
	170	200	10	--	< 0.5	< 0.5	--
	171	200	90	--	< 0.5	< 0.5	--

Table 18 (continued)

Bluff Analysis Section	Shoreline Recession Reach	Shoreline Length (feet)	Bluff Height (feet)	Annual Recession Rates (feet/year) Long Term[a] 1836-1969	Short Term 1963-1985	Short Term 1978-1985	Annual Volume of Shore Material Loss[b] (cubic feet per year)
36	172 173	210 270	90 90	- - - -	0.5 0.5	< 0.5 < 0.5	9,500 12,200
Total		38,770	- -	- -	- -	- -	585,600
Percent of Total Shoreline Length with Recession ≥0.5 Foot per Year				- -	35.0	37.0	- -
Mean Recession Rate of Shoreline ≥0.5 Foot per Year				1.1	0.59	0.52	- -

[a]*Long-term bluff recession rates were reported in U. S. Army Corps of Engineers, Lake Michigan Shoreline, Milwaukee County, Wisconsin, Preliminary Feasibility Report, 1975.*

[b]*The annual volume of material loss was calculated using the 1963 to 1985 recession rates.*

Source: SEWRPC.

Table 19

SUMMARY OF BLUFF RECESSION RATES AND SHORE MATERIAL LOSS ALONG THE NORTHERN MILWAUKEE COUNTY LAKE MICHIGAN SHORELINE

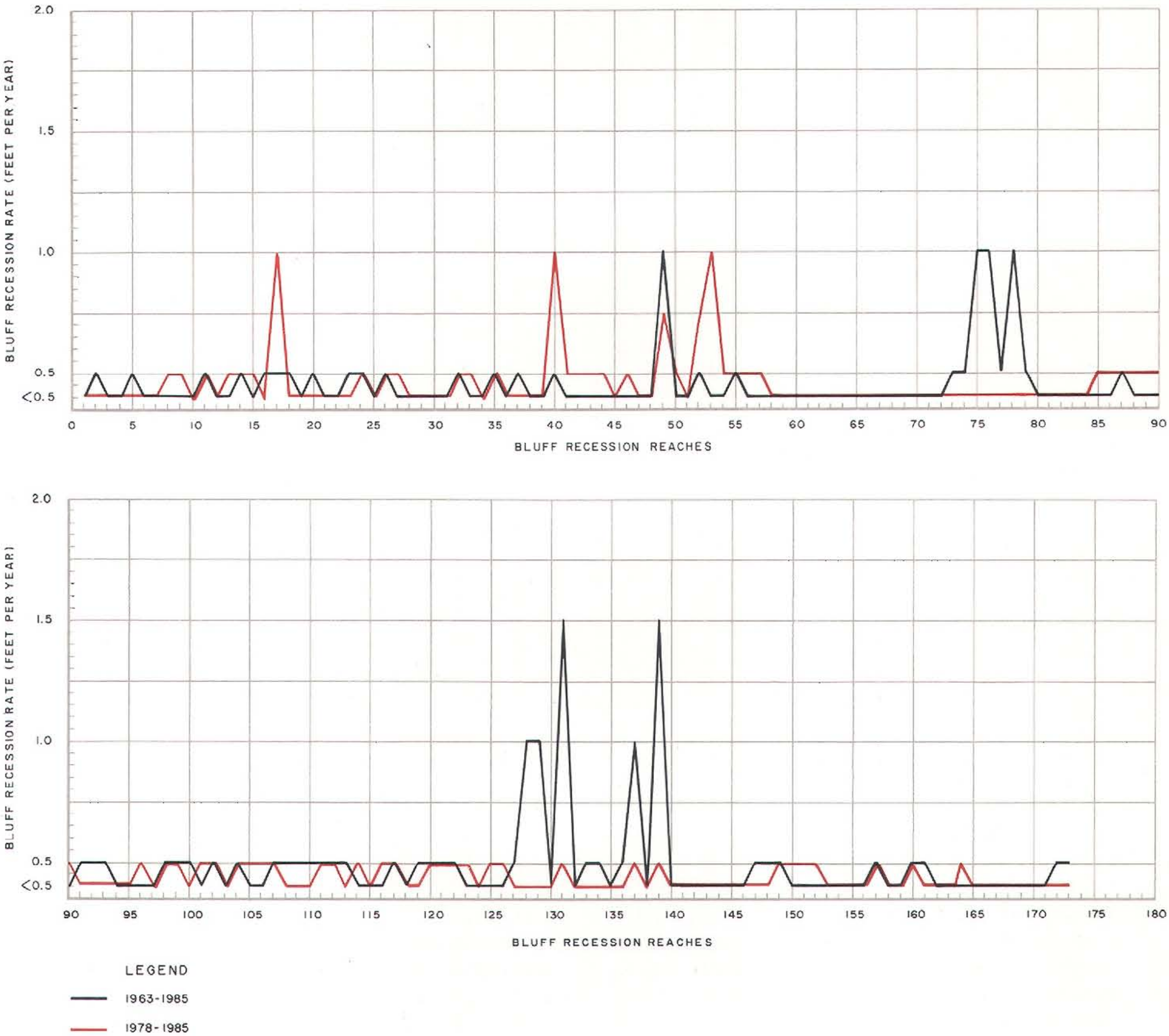
Bluff Recession Rate (feet per year)	SEWRPC 1963-1985 Shoreline Length (feet)	SEWRPC 1963-1985 Percent of Total	SEWRPC 1978-1985 Shoreline Length (feet)	SEWRPC 1978-1985 Percent of Total	Annual Volume of Shore Material Loss[a] Cubic Feet per Year	Annual Volume of Shore Material Loss[a] Percent of Total
< 0.5	25,210	65	24,450	63	- -	- -
0.5	11,550	30	13,650	35	494,300	84
1.0	1,610	4	670	2	88,300	15
1.5	400	1	0	0	3,000	1
Total	38,770	100	38,770	100	585,600	100

[a]*The annual volume of material loss was calculated using the 1963 to 1985 recession rates.*

Source: SEWRPC.

Figure 49

MEASURED BLUFF RECESSION RATES ALONG THE LAKE MICHIGAN SHORELINE OF NORTHERN MILWAUKEE COUNTY



Source: SEWRPC.

three times higher than the mean recession rates of 0.5 to 0.6 foot per year measured in this study area.

The shoreline erosion rates are apparently lower in northern Milwaukee County for several reasons. First, in 1986 about 61 percent of the shoreline was protected by shore protection measures or by fill placed on the bluff slope. Even where these structures were found to be failing and required reconstruction or maintenance, they have apparently effectively reduced the rate of shoreline erosion. This portion of the northern Milwaukee County shoreline protected by structures is greater than in the other shoreline areas studied in southeastern Wiscon-

Table 20

COMPARISON OF BLUFF RECESSION RATES MEASURED FOR THE LAKE MICHIGAN SHORELINE OF SOUTHEASTERN WISCONSIN BY THE REGIONAL PLANNING COMMISSION

Lake Michigan Shoreline Area	Period of Measurement	Bluff Recession Rates (feet per year)		
		Minimum	Mean	Maximum
Racine County	1963-1980	0.0	1.5	10.2
	1975-1980	0.0	2.1	10.2
Chiwaukee Prairie-Carol Beach Kenosha County	1970-1980	0.0	5.6	12.6
City of St. Francis Milwaukee County	1963-1980	0.2	2.7	5.6
	1970-1980	0.0	3.2	6.3
Northern Milwaukee County Study Area	1963-1985	< 0.5	0.6[a]	1.5
	1978-1985	< 0.5	0.5[a]	1.0

[a] *Length-weighted mean recession rate of those areas showing a recession rate of 0.5 foot per year or more.*

Source: SEWRPC.

sin. Second, the near-shore bathymetry, or slope of the lake bottom, is fairly gentle along the Fox Point terrace, which accounts for nearly one-fourth of the study area shoreline. This gentle offshore slope absorbs wave energy and helps prevent the erosion of the terrace from being more severe. Third, many of the bluffs subject to slope failure are experiencing deep-seated slumps or rotational slides, rather than shallow translational slides. For a period of time, the slumps result in a more stable slope as the overall angle of the slope is reduced and a large amount of slope debris is deposited at the base of the bluff. Thus, slope failure in those bluff areas subject to deep-seated slumping tends to be very sporadic, where an episode of severe bluff recession may be followed by a long period—perhaps decades—of relative slope stability. On an average annual basis over a limited time period, these bluff areas may exhibit a very low recession rate, although a significant risk of severe slope failure may exist.

SUMMARY

This chapter presents an inventory of certain elements of the natural resource base relevant to shoreline erosion and bluff recession; summarizes existing land use and zoning patterns; and sets forth the findings of an inventory and analysis of the types, causes, and rates of shoreline erosion and bluff recession occurring within northern 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 northern 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 150 feet in thickness. Several layers of glacial debris, including the Kewaunee Formation, the Oak Creek Formation, and the New Berlin Formation, can be identified on the eroding bluff faces along the northern part of the County's Lake Michigan shoreline.

Soil properties influence the rate of stormwater runoff and the severity of surface erosion. The soils within the upland portions of the study area 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. The sandy soils which cover the terrace within the Village of Fox Point are more likely to generate low to moderate amounts of stormwater runoff because of their moderate infiltration capacity, moderate permeability, and good drainage.

Bluff heights along the shoreline range up to nearly 130 feet above beach levels. About one-half of the shoreline has bluffs ranging from 80 to 20 feet in height. The terraced area within the Village of Fox Point, which lies 4 to 10 feet above the beach, covers approximately 24 percent of the shoreline within the study area. The most dominant bluff material identified was the Ozaukee till, covering about 26 percent of the total bluff face surface within the study area. Other common bluff materials found were silt and sand, Oak Creek till, and New Berlin till. The composition of the bluff slopes along about 41 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 in May 1986 were sand, gravel, and cobbles. The most extensive beach, exceeding 90 feet in width, was found at Atwater Park in the Village of Shorewood, and was composed of sand. 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. About 69 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. Beach slopes generally were less than 10 degrees.

Along the northern 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 16 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 study area, which lies entirely within Milwaukee County, contains portions of the City of Milwaukee and the Villages of Shorewood, Whitefish Bay, and Fox Point, and encompasses a total of 1,726 acres. About 1,448 acres, or 84 percent of the study area, was devoted to intensive urban uses in 1985. About 74 percent of the urban land area was in residential use.

Zoning ordinances are currently in effect in each of the four civil divisions within the study area. In general, those areas likely to be affected by amendments to existing zoning ordinances which would regulate land uses in relation to the risk of shoreline erosion and bluff recession have been placed in residential zoning districts.

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, 68 percent of the shoreline within the study area is regulated under a Lake Bed Grant issued to Milwaukee County. Local zoning ordinances regulate land uses within the shoreland area, but are generally devoid of provisions pertaining to Lake Michigan shoreline erosion hazards.

An inventory of shore protection structures conducted in June and July of 1986 indicated that a variety of structures, including bulkheads, revetments, groins, and a breakwater, have been installed along the northern 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 both by private shoreline property owners and by public agencies, have had varying degrees of success. An inventory of all 80 shore protection structures in the study area indicated that only about 24 percent of the structures had no observable failure and were not in need of 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 in northern Milwaukee County was conducted in May 1986. The results of the inventory indicated that the primary cause of bluff recession in the study area was bluff toe erosion caused by wave action. Groundwater seepage also was a major cause of slope failure in some portions of the study area. Most slope failure was occurring as shallow slides, although many areas were experiencing deep-seated slumps.

Bluff recession rates for the northern Milwaukee County study area were measured using the Regional Planning Commission aerial photographs taken in 1963 and 1985, and U. S. Army Corps of Engineers aerial photographs taken in 1978. For the period 1963 through 1985, about 65 percent of the study area shoreline exhibited bluff recession rates of less than 0.5 foot per year. Only about 1 percent of the shoreline exhibited a bluff recession rate exceeding 1.0 foot per year. The highest recession rate measured from 1963 through 1985 was approximately 1.5 feet per year, which occurred in the terraced portion of the Village of Fox Point. The mean recession rate over the period 1963 through 1985 of those areas showing a recession rate of 0.5 foot per year or more was 0.6 foot per year. In general, the average annual bluff recession rates measured over the period 1978 through 1985 were slightly lower than the annual recession rates measured over the period 1963 through 1985. Bluff recession in the study area resulted in the loss of nearly 600,000 cubic feet of bluff material annually.

Chapter III

EVALUATION OF COASTAL EROSION PROBLEMS AND DAMAGES

INTRODUCTION

The identification of those shoreland areas that are affected by shoreline erosion and bluff recession is essential to the evaluation of alternative structural and nonstructural shoreline erosion control measures. The purposes of this chapter are to describe the reaches of the Lake Michigan shoreline through northern Milwaukee County experiencing bluff toe erosion and having the potential for bluff slope failure; to identify the 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 generally to 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 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, 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 require a site-specific analysis by a professional engineer specializing in coastal management. It is intended that this report provide guidance and direction to property owners on the types of shore protection measures that may be needed and should be investigated further. The information presented in this report should also be used to help coordinate the 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 geotechnical engineering techniques used to evaluate the existing shore erosion problems and to identify needed control measures, and presents the results of this evaluation for each of 36 bluff analysis sections. The second section assesses the damages that 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 northern Milwaukee County.

EVALUATION OF COASTAL EROSION PROBLEMS

The Lake Michigan shoreline erosion problems of primary concern are 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 extent and severity of bluff toe erosion was determined by aerial photo interpretation and by observations made during field surveys conducted in 1986. 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, the degree to which toe erosion was contributing to the slope failure was assessed. 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.

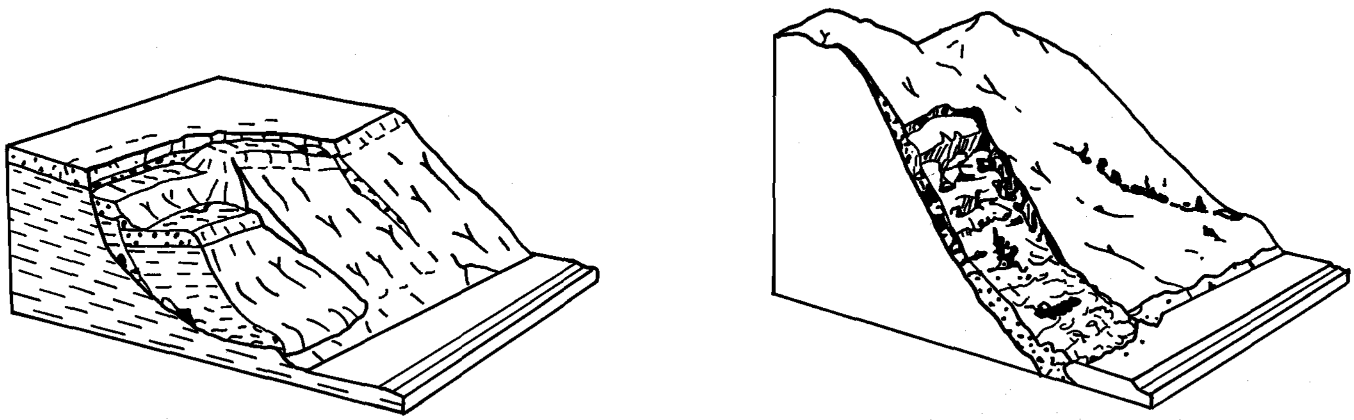
The bluff slope stability analyses were conducted to determine the likelihood of bluff slope failure within the various bluff analysis sections; to determine whether the most likely failures would be deep-seated slumps or shallow slides; to relate slope failures to bluff strata and groundwater conditions;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

Figure 50

COMMON TYPES OF SLOPE FAILURES IN LAKE MICHIGAN COASTAL BLUFFS

ROTATIONAL SLIDING TRANSLATIONAL SLIDING



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

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 50 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 51, 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.

Definition of Safety Factor

Using shear strengths and stresses, a factor of safety was determined related to the potential failure surfaces within the bluff. This 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.

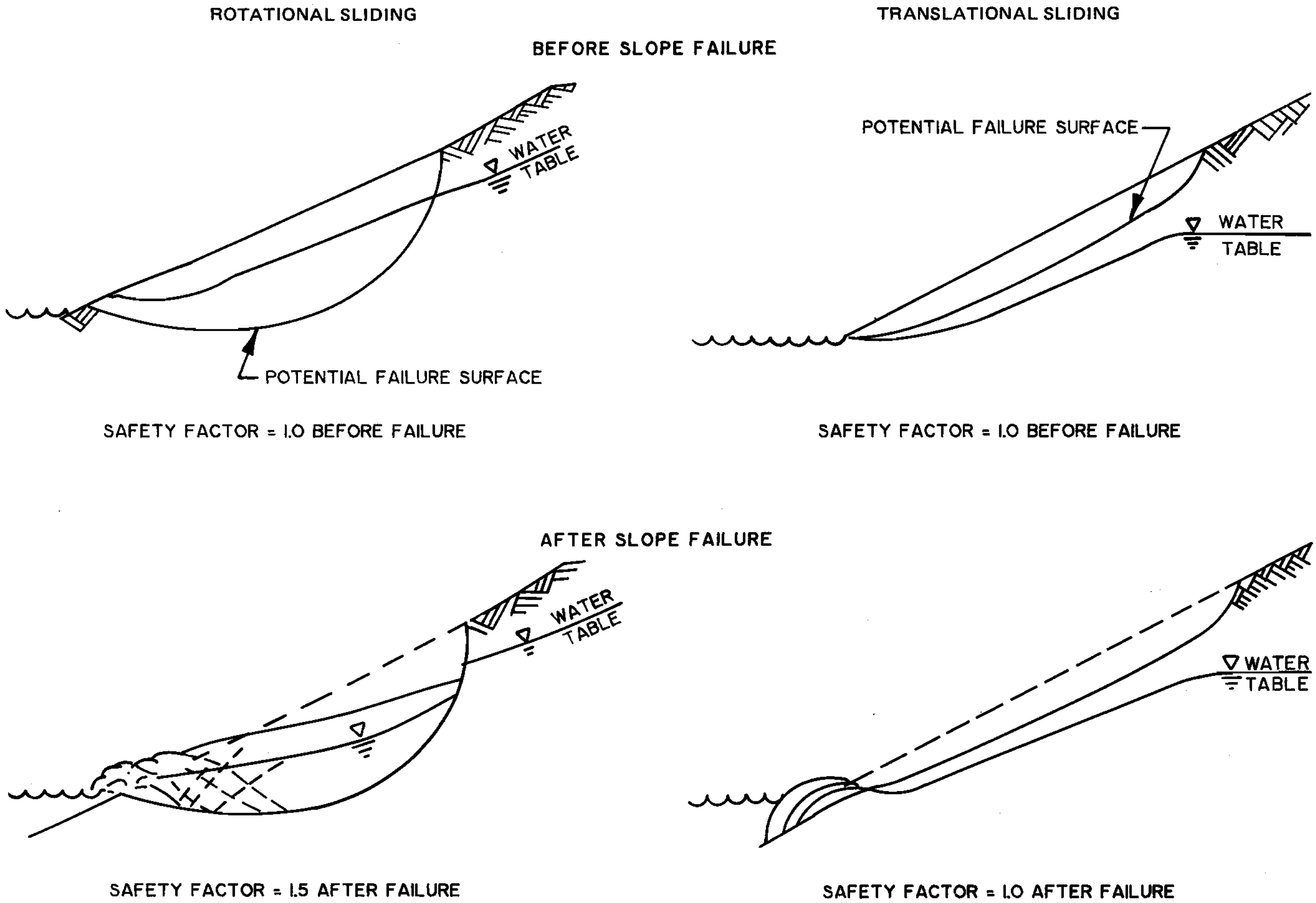
Methods of Analysis

The 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 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 bluff site vary from the general conditions described herein. This 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 may trigger slope failures on the upper bluff slope. Therefore, such erosion

Figure 51

EFFECT OF ROTATIONAL AND TRANSLATIONAL SLIDING ON SLOPE STABILITY



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

must be considered in any bluff stability analysis. Toe erosion also affects the erosion and recession of the low terrace in the Village of Fox Point.

During the 1986 field surveys, those portions of the study area shoreline that were experiencing wave erosion at the toe of the bluff were identified and mapped. Those affected shoreline reaches included areas where waves were observed to be attacking an unprotected bluff; where there was noticeable evidence of recent toe erosion; or where existing shore protection structures were failing, 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 1986. These photographs are reproduced, but not in color, in Appendix C.

Detailed measurements of the geometry of the bluff slope, which were conducted at 44 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 the bluff. Slope stability analyses for rotational slides provide not only an indication of the likelihood of circular slips, but also an overall indication of the resistance of the slopes to all types of massive slope failures. In reality, massive slope failure surfaces are rarely truly circular; most are more planar with a steeper upper portion at the rupture surface, and with a progressively decreasing slope angle. Slope stability analyses were performed for the bluffs within each bluff analysis section 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 Associate 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 52. 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:

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

where: SF = safety factor

$m_\alpha = \cos\alpha\ [1 + (\tan\alpha\ \tan\phi')/SF]$

W = weight of individual slice

b = width of slice

α = slope angle

u = pore water pressure

c' = effective cohesion intercept

ϕ' = 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.

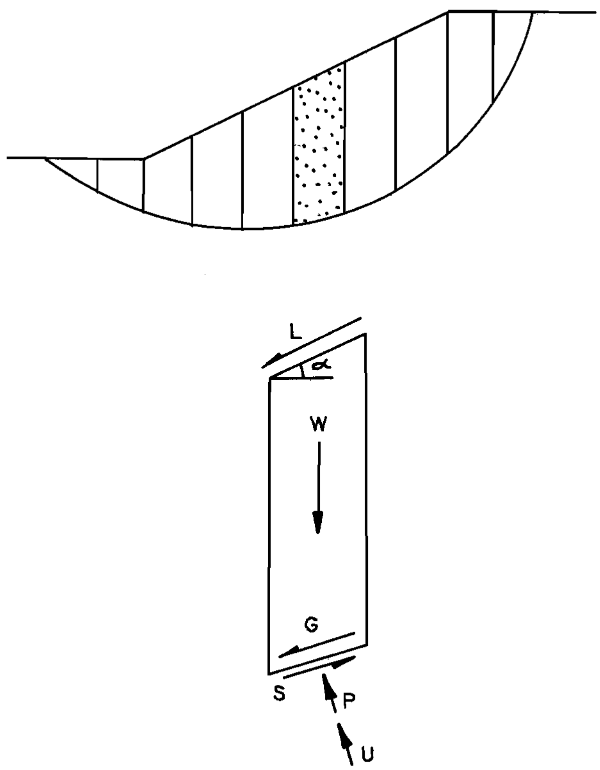
Distinction Between Deterministic and Probabilistic Slope Stability Analyses: Two separate versions of the STABL program were used in the slope stability analysis for the northern Milwaukee County shoreline. The first version utilized a deterministic approach in which site-specific data collected at 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,

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

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

Figure 52

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



WHERE: L = LENGTH OF FAILURE SURFACE, IN FEET
α = ANGLE OF INCLINATION, IN DEGREES
W = WEIGHT OF SLICE, IN POUNDS
G = GRAVITY, IN POUNDS PER FOOT
s = SHEAR STRENGTH FORCES (COHESION AND FRICTION), IN POUNDS PER SQUARE FOOT
P = NORMAL FORCE, IN POUNDS
U = SEEPAGE FORCE, IN POUNDS PER SQUARE FOOT

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

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 analysis are presented below.

Deterministic Slope Stability Analysis: A total of 44 bluff profiles, prepared during the field surveys conducted in the summer of 1986, were used in the deterministic slope stability analysis. The locations of the profile sites, which are presented in Table 21 and shown on Map 15, 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 each bluff analysis section except Section 35, which includes the low terrace in the Village of Fox Point, where 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 the most important in bluffs higher than 80 feet.[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 has been found to be the most critical period for the stability of Lake Michigan coastal bluffs for

[3] *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.*

Table 21

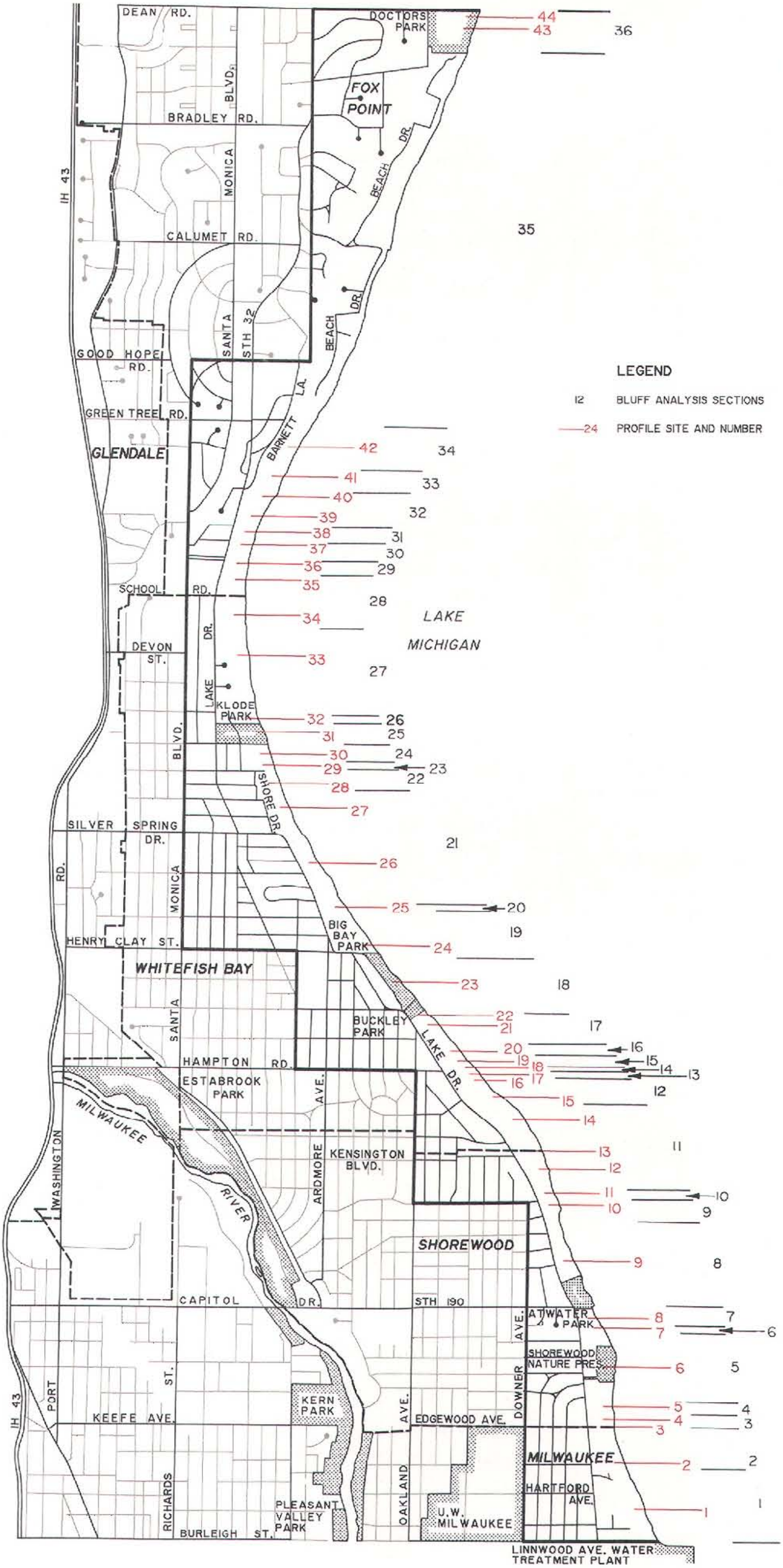
LOCATION OF PROFILE SITES

Civil Division	Bluff Analysis Section	Profile Number	Location
City of Milwaukee	1	1	3252 N. Lake Drive
	2	2	100′ north of E. Newport Avenue
Village of Shorewood	3	3	3510 N. Lake Drive
		4	3510 N. Lake Drive
	4	5	3534 N. Lake Drive
	5	6	3704 N. Lake Drive
	6	7	3926 N. Lake Drive
	7	8	3932 N. Lake Drive
	8	9	4098 N. Lake Drive
	9	10	4308 N. Lake Drive
	10	11	4408 N. Lake Drive
	11	12	4460 N. Lake Drive
Village of Whitefish Bay		13	4500 N. Lake Drive
		14	4620 N. Lake Drive
	12	15	4652 N. Lake Drive
		16	4730 N. Lake Drive
	13	17	4762 N. Lake Drive
	14	18	4780 N. Lake Drive
	15	19	4794 N. Lake Drive
	16	20	4810 N. Lake Drive
	17	21	4890 N. Lake Drive
		22	4930 N. Lake Drive
	18	23	Big Bay Park
	19	24	Henry Clay Street
	20	25	5290 N. Lake Drive
	21	26	5486 N. Lake Drive
		27	5674 N. Shore Drive
	22	28	5738 N. Shore Drive
	23	29	758 Day Street
	24	30	5822 N. Shore Drive
	25	31	Klode Park
	26	32	5960 N. Shore Drive
	27	33	614 E. Lake Hill Court
	28	34	6330 N. Lake Drive
Village of Fox Point		35	6424 N. Lake Drive
	29	36	6448 N. Lake Drive
	30	37	6530 N. Lake Drive
	31	38	6610 N. Lake Drive
	32	39	6720 N. Lake Drive
		40	6818 N. Barnett Lane
	33	41	6840 N. Barnett Lane
	34	42	6960 N. Barnett Lane
	36	43	Doctors Park
		44	Doctors Park

Source: SEWRPC.

Map 15

LOCATION OF PROFILE SITES



Source: SEWRPC.

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. The elevation of the water table is affected by many of the same factors that result in fluctuations in the level of Lake Michigan. In some bluffs, the groundwater may be hydraulically connected to the lake; in such bluffs, the elevation of the water table is 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 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 the most important, even if that surface would affect only a small portion of the bluff slope.

The second common type of slope evolution involves the retreat of the bluff generally parallel to the existing face. Large, deep, rotational slips may also occur. This type of slope evolution typically occurs in bluffs with a steep slope—greater than 30 degrees—and in bluffs composed of noncohesive glacial tills and sand. The evaluation of the stability of this second type of slope involves 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 summer of 1986, 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 22. The reliability of the slope stability evaluations was greater at some profile sites than at others because the quantity and precision of available inventory data varied substantially between sites.

The results of laboratory analyses of the properties of soils identified in the study area were summarized in Chapter II. The soil property summaries were based on historical data and on the geotechnical laboratory analyses of grab samples collected in May 1986, and of soil boring samples collected in October and November 1986. These soil properties were used to calculate the ability of the soil materials to resist slope failure. The soil properties of the bluff materials used in the deterministic slope stability analyses are presented in Table 23.

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

[4] *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.*

[5] *Ibid.*

Table 22

SOURCES OF STRATIGRAPHIC DATA USED FOR THE SLOPE STABILITY ANALYSES AT PROFILE SITES

Civil Division	Bluff Analysis Section	Profile Number	Location	Field Observation of Exposed Bluff Face Within Section: 1986[a]	Soil Boring Within Section: 1986	Soil Boring Within Section: Pre-1986	Field Observation of Exposed Bluff Face Within Adjacent Sections[a]	Soil Boring Within Adjacent Sections
City of Milwaukee	1	1	3252 N. Lake Drive	--	--	--	--	3432 N. Lake Drive (1986)
	2	2	100' north of E. Newport Avenue	--	3432 N. Lake Drive	--	--	--
Village of Shorewood	3	3	3510 N. Lake Drive	--	--	--	--	14 borings in Section 5 (1970's)
		4	3510 N. Lake Drive	--	--	--	--	14 borings in Section 5 (1970's)
	4	5	3534 N. Lake Drive	--	--	--	--	14 borings in Section 5 (1970's)
	5	6	3704 N. Lake Drive	--	--	14 borings (1970's)	--	--
	6	7	3926 N. Lake Drive	--	--	--	X	--
	7	8	3932 N. Lake Drive	X	--	--	--	--
	8	9	4098 N. Lake Drive	--	4154 N. Lake Drive	--	--	--
	9	10	4308 N. Lake Drive	--	--	--	--	3 borings at 4408 N. Lake Drive (1986)
	10	11	4408 N. Lake Drive	--	4408 N. Lake Drive	--	--	--
	11	12	4460 N. Lake Drive	X	--	--	--	--
Village of Whitefish Bay		13	4500 N. Lake Drive	X	--	--	--	--
		14	4620 N. Lake Drive	X	--	--	--	--
	12	15	4652 N. Lake Drive	X	--	--	--	--
		16	4730 N. Lake Drive	X	--	--	--	--
	13	17	4762 N. Lake Drive	X	--	--	--	--
	14	18	4780 N. Lake Drive	X	--	--	--	--
	15	19	4794 N. Lake Drive	X	--	--	--	--
	16	20	4810 N. Lake Drive	X	--	--	--	--
	17	21	4890 N. Lake Drive	--	--	--	X	--
		22	4930 N. Lake Drive	--	--	4930 N. Lake Drive (1970's)	--	--
	18	23	Big Bay Park	--	Big Bay Park	--	--	--
	19	24	Henry Clay Street	--	--	8 borings within 1,000 feet (1935-1970's)	--	--
	20	25	5290 N. Lake Drive	--	--	4 borings (1935)	--	--
	21	26	5486 N. Lake Drive	--	--	12 borings within 1,000 feet (1935-1970's)	--	--
		27	5674 N. Shore Drive	--	--	4 borings within 1,000 feet (1953)	--	--
	22	28	5738 N. Shore Drive	--	--	5722 N. Shore Drive (1953)	--	--
	23	29	758 Day Street	--	--	--	--	5842 N. Shore Drive (1986)
	24	30	5822 N. Shore Drive	--	5842 N. Shore Drive	--	--	--
	25	31	Klode Park	--	--	--	X	--
	26	32	5960 N. Shore Drive	X	--	5960 N. Shore Drive (1953)	--	--
	27	33	614 E. Lake Hill Court	--	6216 N. Lake Drive	--	--	--
	28	34	6330 N. Lake Drive	--	--	6330 N. Lake Drive (1970's)	--	--
Village of Fox Point		35	6424 N. Lake Drive	--	--	6330 N. Lake Drive (1970's)	--	--
	29	36	6448 N. Lake Drive	--	--	--	--	6500 N. Lake Drive (1986)
	30	37	6530 N. Lake Drive	--	6500 N. Lake Drive	--	--	--
	31	38	6610 N. Lake Drive	--	--	--	--	6500 N. Lake Drive (1986)
	32	39	6720 N. Lake Drive	--	6730 N. Lake Drive	--	--	--
		40	6818 N. Barnett Lane	--	--	--	--	6730 N. Lake Drive (1986)
	33	41	6840 N. Barnett Lane	--	6840 N. Barnett Lane	--	--	--
	34	42	6960 N. Barnett Lane	X	--	6 borings (1935)	--	--
	36	43	Doctors Park	X[b]	--	--	--	--
		44	Doctors Park	X[b]	--	--	--	--

[a] *X denotes that at least a portion of the bluff was unvegetated and exposed during the summer of 1986, allowing field determination of the stratigraphy.*

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

Source: SEWRPC.

Table 23

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 Friction Angle (degrees)
Tills				
New Berlin	138	138	10	34
Oak Creek	135	135	100	30.5
Ozaukee	134	134	150	30
Fractured Ozaukee	134	134	10	30
Lake Sediments				
Medium Fine Sand	120	120	0	33
Sand and Gravel	120	120	0	33
Silt	130	130	4,000	31
Silt and Fine Sand	110	110	10	31
Clay and Silt	130	130	450	27
Fine Sand and Silt	125	125	100	33
General Lake Sediment	125	125	100	27
Fill				
Concrete Rubble and Soil Fill	130	130	0	35

Source: T. B. Edil and SEWRPC.

elevation of the groundwater was estimated based on the depth of permeable soil layers. The elevation of the groundwater at each of the profile sites was determined based on the sources of data set forth in Table 24.

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 10 failure surfaces with the lowest safety factors 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 Associate Professor Peter J. Bosscher and Professor Turner 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 only 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 30 of the 44 profile sites that were analyzed using the deterministic slope stability analysis method. The remaining 14 profile sites—5, 7, 12, 13, 14, 17, 19, 21, 22, 24, 26, 27, 29, and 36—were sites where fill had been placed on the face of the bluff. The probabilistic method was not suitable for evaluating the stability of fill sites.

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

Table 24

SOURCES OF GROUNDWATER DATA USED FOR THE SLOPE STABILITY ANALYSES AT PROFILE SITES

Civil Division	Bluff Analysis Section	Profile Number	Location	Field Observation of Seepage: 1986	1986 Soil Boring	Observation Well Measurements	Electrical Resistivity Analysis	Estimated Based on Location of Permeable Soil Strata
City of Milwaukee	1	1	3252 N. Lake Drive	--	--	--	--	X
	2	2	100 feet north of E. Newport Avenue	--	X	--	--	--
Village of Shorewood	3	3	3510 N. Lake Drive	X	--	--	--	--
		4	3510 N. Lake Drive	X	--	--	--	--
	4	5	3534 N. Lake Drive	--	--	--	--	X
	5	6	3704 N. Lake Drive	X	--	--	--	--
	6	7	3926 N. Lake Drive	--	--	--	--	X
	7	8	3932 N. Lake Drive	--	--	--	X	X
	8	9	4098 N. Lake Drive	--	X	--	--	X
	9	10	4308 N. Lake Drive	--	--	--	--	X
	10	11	4408 N. Lake Drive	--	X	X	--	--
	11	12	4460 N. Lake Drive	X	--	--	--	--
Village of Whitefish Bay		13	4500 N. Lake Drive	X	--	--	--	--
		14	4620 N. Lake Drive	X	--	--	--	--
	12	15	4652 N. Lake Drive	--	--	--	X	--
		16	4730 N. Lake Drive	--	--	--	X	--
	13	17	4762 N. Lake Drive	--	--	--	--	X
	14	18	4780 N. Lake Drive	--	--	--	X	--
	15	19	4794 N. Lake Drive	--	--	--	--	X
	16	20	4810 N. Lake Drive	--	--	--	X	--
	17	21	4890 N. Lake Drive	--	--	--	--	X
		22	4930 N. Lake Drive	--	--	--	X	--
	18	23	Big Bay Park	--	X	--	--	X
	19	24	Henry Clay Street	--	--	--	--	X
	20	25	5290 N. Lake Drive	--	--	--	X	X
	21	26	5486 N. Lake Drive	--	--	--	--	X
		27	5674 N. Shore Drive	--	--	--	--	X
	22	28	5738 N. Shore Drive	X	--	--	--	--
	23	29	758 Day Street	--	--	--	--	X
	24	30	5822 N. Shore Drive	X	X	--	--	--
	25	31	Klode Park	X	--	--	X	--
	26	32	5960 N. Shore Drive	X	--	--	--	--
	27	33	614 E. Lake Hill Court	X	X	--	--	--
	28	34	6330 N. Lake Drive	--	--	--	X	X
Village of Fox Point		35	6424 N. Lake Drive	X	--	--	--	--
	29	36	6448 N. Lake Drive	--	--	--	--	X
	30	37	6530 N. Lake Drive	X	X	--	--	X
	31	38	6610 N. Lake Drive	--	--	--	--	X
	32	39	6720 N. Lake Drive	--	X	X	--	X
		40	6818 N. Barnett Lane	--	--	--	--	X
	33	41	6840 N. Barnett Lane	--	X	--	X	--
	34	42	6960 N. Barnett Lane	--	--	--	X	--
	36	43	Doctors Park	--	--	--	--	X
		44	Doctors Park	--	--	--	--	X

Source: SEWRPC.

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 was then randomly varied within a distribution determined from a review of observed conditions within each bluff analysis section, and other available data. The allowed dispersion of data was specified for each profile site by assigning a standard deviation of those bluff parameters that were allowed to vary randomly.

The data dispersions used for the probabilistic analysis were selected by the geotechnical engineering consultants to the Commission. 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 10 test results were available for each soil type. The dispersions were assigned by examining the dispersion of the available test data and the nature of the soil. These soil property data are presented in Chapter II. The dispersions used for the elevation of the groundwater and the elevation and inclination of the soil interface lines were not specifically calculated, rather being estimated based upon a review of the range of variability of these characteristics within each bluff analysis section. Thus, experienced judgment was used in establishing the range of variation of bluff characteristics for the probabilistic analysis. It must be recognized that because of the nature of the probabilistic analysis, there is substantial uncertainty as to whether 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, help assess the likelihood of slope failure. The probabilistic analysis thus helps quantify the risk of slope failure associated with variable bluff conditions.

The locations of the soil interface lines on the bluff face as well as the angle of inclination of these lines as they proceed into the bluff were varied. The degree of variability differed at each profile site, but in general, the elevation of the soil interface lines ranged from 0 to 20 feet about the mean, and the angle of inclination ranged from 0 to 3 degrees about the mean. The lowest variability of soil interface lines was selected for those sites where the strata were well defined and the represented bluff analysis section was small—sometimes including only one property.

Table 25

VARIATION IN SOIL PROPERTIES ASSIGNED IN THE PROBABILISTIC SLOPE STABILITY ANALYSIS

Soil Type	Standard Deviation of Effective Cohesion Intercept (pounds per square foot)	Standard Deviation of Internal Friction Angle (degrees)
Glacial Tills		
New Berlin	5	3
Oak Creek	75	2
Ozaukee	100	3
Fractured Ozaukee	5	3
Lake Sediments		
Medium Fine Sand	5	2
Sand and Gravel	5	2
Silt	1,000	2
Silt and Fine Sand	10	2
Clay and Silt	350	2
Fine Sand and Silt	75	2
General Lake Sediment	75	3

Source: SEWRPC.

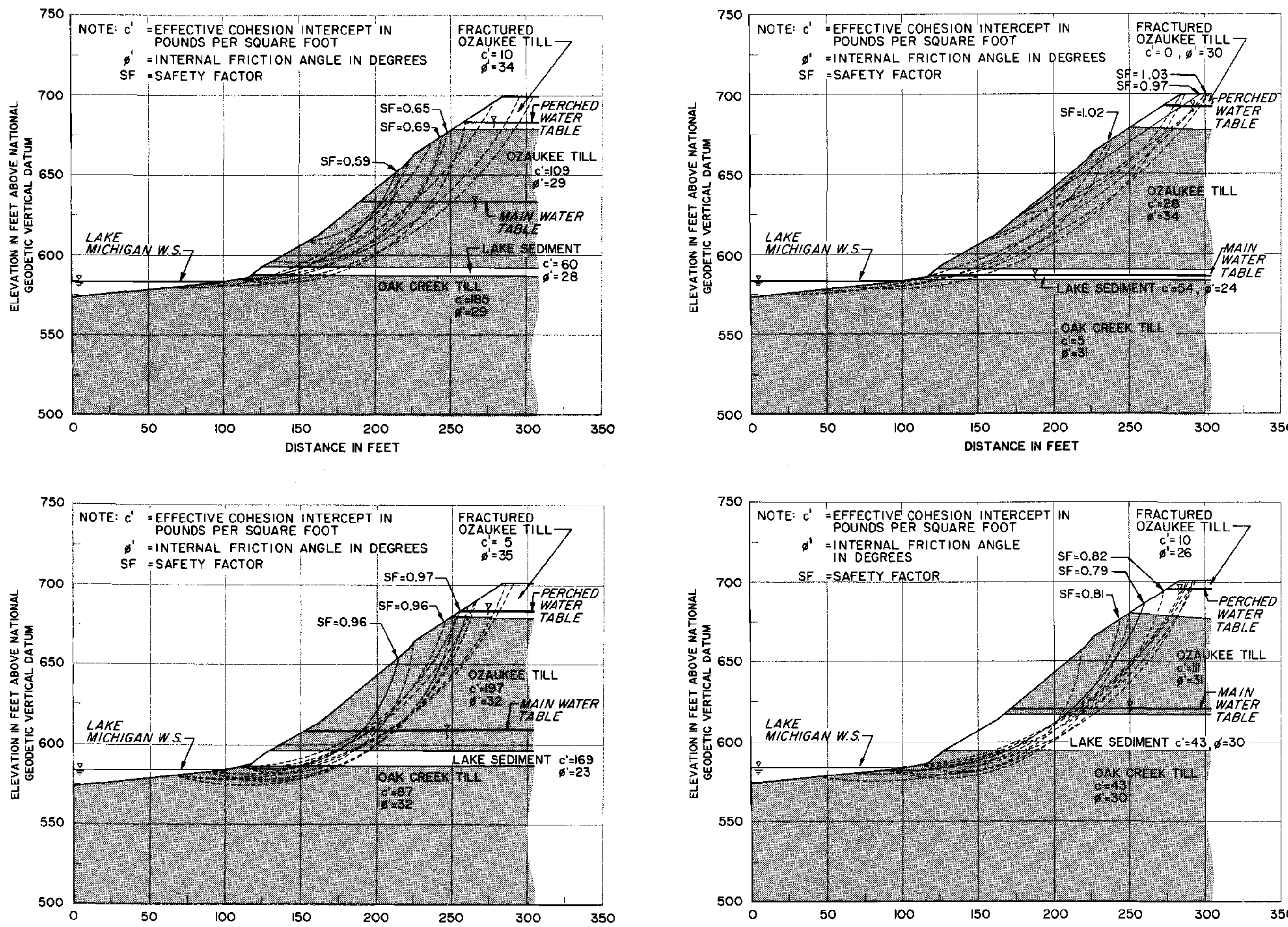
The dispersion of soil properties from the means used in the probabilistic analysis are shown in Table 25. The elevation of the groundwater was varied based on available water data and on the type and thickness of the lake sediment layers within each profile site. In general, the elevation of the main water table ranged from 2 to 20 feet from the mean, and the elevation of the perched water table, located in the fractured Ozaukee layer, ranged from 2 to 7.5 feet from the mean. An illustration of the variability of bluff parameters and the resultant effects on the safety factors calculated with the probabilistic analysis is shown in Figure 53. Compared to the deterministic analysis, the probabilistic analysis yields both higher and lower safety factors.

For each of the 30 profile sites evaluated with the probabilistic analysis, a minimum of 20 stability analyses were performed using a random combination of variable bluff parameters. Each stability analysis involved the generation of 100 failure surfaces and the calculation of the corresponding safety factors. For each analysis, the 10 failure surfaces with the lowest safety factors were identified.

Interpretation of Rotational Slope Stability Analysis Results: The stability of the bluff slopes with respect to rotational sliding was determined for each bluff analysis section. The bluff slopes within

Figure 53

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.

each section were classified as stable, marginal, or unstable. The stability classifications were based on a combined interpretation of the deterministic slope stability analysis, the probabilistic slope stability analysis, observed slope conditions in the summer of 1986, and historical records of previous slope failures.

A set of general guidelines developed to classify the bluff slopes on the basis of the deterministic and probabilistic slope stability analysis results is presented in Table 26. These guidelines were used to provide a general indication of slope stability—the final classifications set forth in this chapter being determined by a review of all available data. In interpreting the results of the deterministic and probabilistic stability analyses, both the lowest safety factor and the 10 lowest safety factors were considered in order to evaluate slope stability on a long-term basis.

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 northern

Table 26

GUIDELINES FOR CLASSIFICATION OF BLUFF SLOPES FOR ROTATIONAL SLIDING

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

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

Source: SEWRPC.

Milwaukee County shoreline was analyzed with the computer program INSLOPE (Infinite Slope Analysis). INSLOPE was developed by Professor Donald H. Gray at the University of Michigan. The program calculates the safety factors of slopes where the thickness of failed material is small in comparison to the height of the slope, and where the failure surface is parallel to the slope surface. The concept of the infinite slope stability analysis for translational sliding is illustrated in Figure 54. 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:

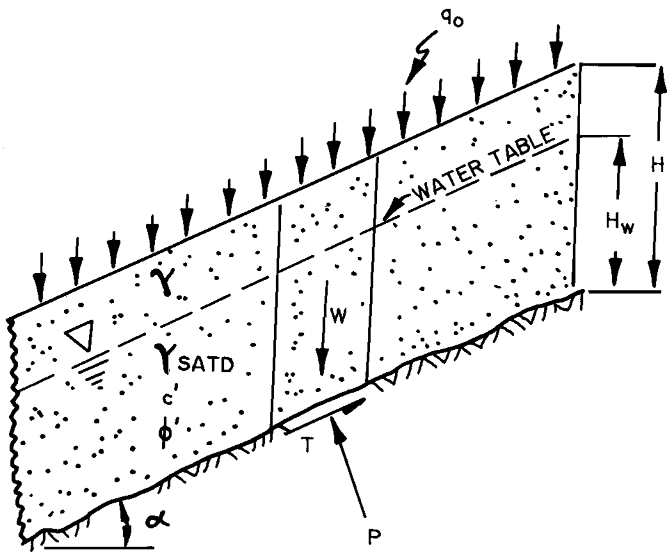
$$SF = \frac{\left[\dfrac{c'}{\cos^2\alpha \tan\phi'} + (q_o + \gamma H) + (\gamma_{BUOY} - \gamma)H_W\right]\dfrac{\tan\phi'}{\tan\alpha}}{[(q_o + \gamma H) + (\gamma_{SATD} - \gamma)H_W]}$$

where:

SF = safety factor
ϕ' = internal friction angle
c' = effective cohesion intercept
α = slope angle
γ = moist density of soil
γSATD = saturated density of soil
γBUOY = buoyant density of soil ($_{BUOY} = \gamma_{SATD} - \gamma_w$)
γw = density of water
H = vertical thickness of sliding mass
Hw = piezometric height above sliding surface
q_o = uniform vertical surcharge stress on slope

Figure 54

CONCEPT OF THE INFINITE SLOPE ANALYSIS FOR TRANSLATIONAL SLIDING



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

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

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 23.

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 pounds 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 as unstable. Bluff slopes with safety factors ranging from 1.0 to 1.5 were classified as marginal. Bluff slopes with safety factors greater than 1.5 were classified as stable. Table 27 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 27.

Table 27

POTENTIAL FOR TRANSLATIONAL SLIDING UNDER BLUFF CONDITIONS FOUND IN NORTHERN MILWAUKEE COUNTY

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

[a]*Potential for Translational Sliding: S - Stable Bluff Slope*
M - Marginal Bluff Slope
U - Unstable Bluff Slope

Source: SEWRPC.

RESULTS

An evaluation of each bluff analysis section, including a determination of the likelihood of bluff slope instability by rotational sliding and translational sliding, is presented below, along with an assessment of the severity of bluff toe erosion. 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 evaluation were used to determine the shoreline protection needs of 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 either by filling or by cutting back the slope. In order to maintain the natural aesthetic properties and drainage characteristics of the bluff, modification of the bluff slope by filling, or by cutting back the slope, was recommended only where other control measures—which would maintain or reestablish these natural characteristics—were judged to be unable to 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.

The results set forth in this report are based on systems level, generalized analyses which determine the condition and needs of entire bluff analysis reaches—herein termed sections—and identify 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.

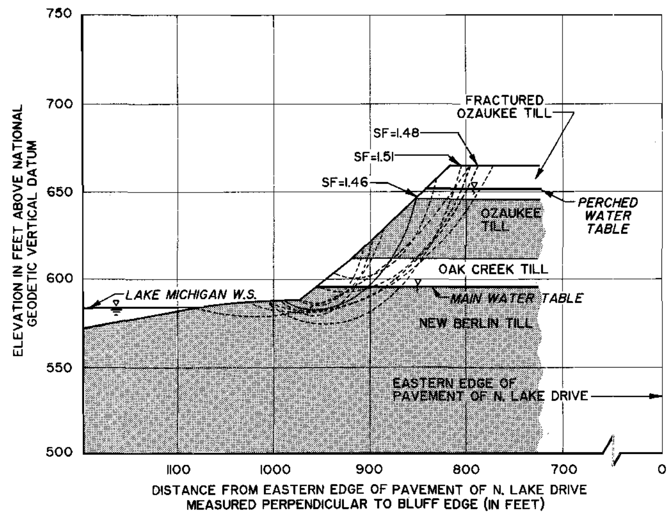
Bluff Analysis Section 1

The stability of the bluff slope within Section 1, which extends from the City of Milwaukee Linnwood Avenue water treatment plant to 3052 E. Newport Court, was characterized by the use of Profile No. 1.

The results of the deterministic slope stability analysis, shown in Figure 55, indicate that Profile No. 1 has a stable bluff slope with respect to rotational sliding. The lowest failure surface

Figure 55

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 1: BLUFF ANALYSIS SECTION 1



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

calculated at this profile site had a safety factor of 1.46, and was located within the lower two-thirds of the bluff slope. The next nine lowest safety factors ranged from 1.48 to 1.61.

A probabilistic slope stability analysis, under which the bluff conditions at the profile site were varied, was conducted to help characterize the stability of the bluff slope within the entire section, and to help determine whether, under certain conditions, the slope would be unstable. Twenty probabilistic stability analyses were conducted; the lowest safety factors in the analyses 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 1 was considered to have a stable bluff slope with respect to rotational sliding.

Overall, Section 1 was considered to have a marginal bluff slope with respect to translational sliding. Although the bluff slope was at a gentle angle, and a good vegetative growth covered the bluff face, there were small disturbed soil areas observed on the upper portion of the bluff slope where translational sliding may have occurred. These small isolated slides, however, did not appear to be threatening the stability of the overall bluff slope.

Due primarily to the relatively wide beach built up in Section 1, 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 primarily in the northern portion of the section. If the lake levels would return to the mean 20th century levels, the resulting beach within most of Section 1 would approximate 60 to 100 feet in width.

No measures are needed to prevent rotational sliding within Bluff Analysis Section 1. Revegetation of the scattered disturbed soil areas within the upper portion of the bluff slope is recommended to prevent the occurrence of translational sliding. Additional toe protection measures are recommended within the northern portion of the section to prevent erosion from wave and ice action.

Bluff Analysis Section 2

The stability of the bluff slope within Section 2, which extends from 3378 to 3474 N. Lake Drive, was characterized by the use of Profile No. 2.

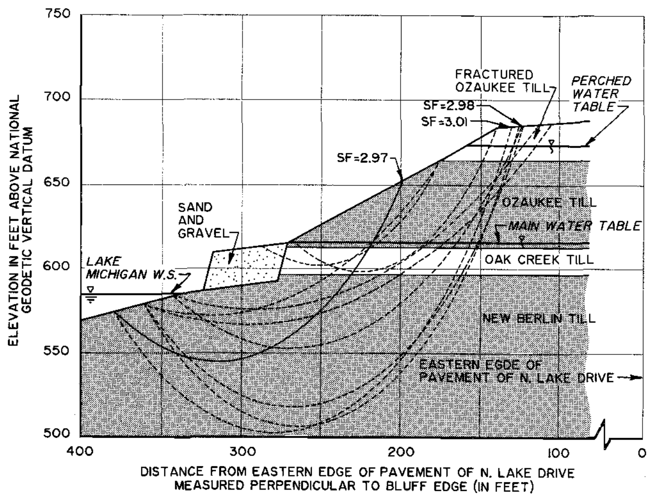
The results of the deterministic slope stability analysis, shown in Figure 56, indicate that Profile No. 2 has a stable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 2.97, and was located within the lower two-thirds of the bluff slope. The next nine lowest safety factors ranged from 2.98 to 3.13.

The lowest safety factors indicated by the 20 probabilistic stability analyses 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 2 was considered to have a stable bluff slope with respect to rotational sliding.

Overall, Section 2 was considered to have a marginal bluff slope with respect to translational sliding. Although the bluff slope was at a gentle angle and a good vegetative growth covered the bluff face, 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.

Figure 56

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 2: BLUFF ANALYSIS SECTION 2



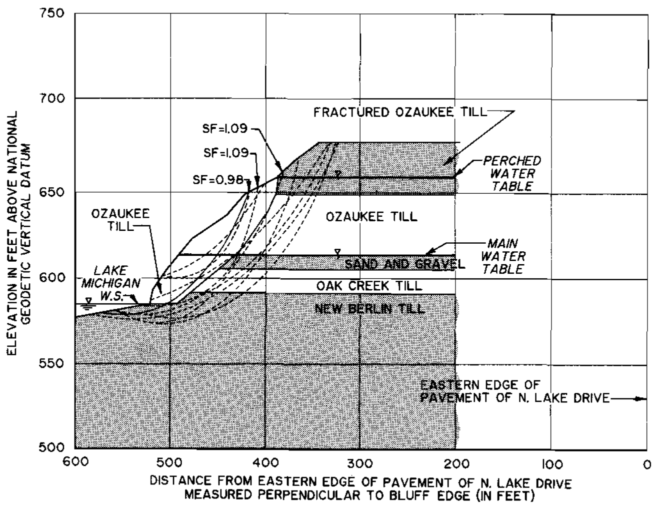
Source: T. B. Edil, D. M. Mickelson, and SEWRPC.

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, the resulting beach within Section 2 would approximate 30 feet in width.

No measures are needed to prevent rotational sliding within Bluff Analysis Section 2. Surface runoff control and the establishment of a good vegetative cover on the land that was cleared is recommended within this section, especially on the steep ravine slopes, to prevent the occurrence of translational sliding. Bluff toe protection is recommended to prevent erosion by wave and ice action.

Figure 57

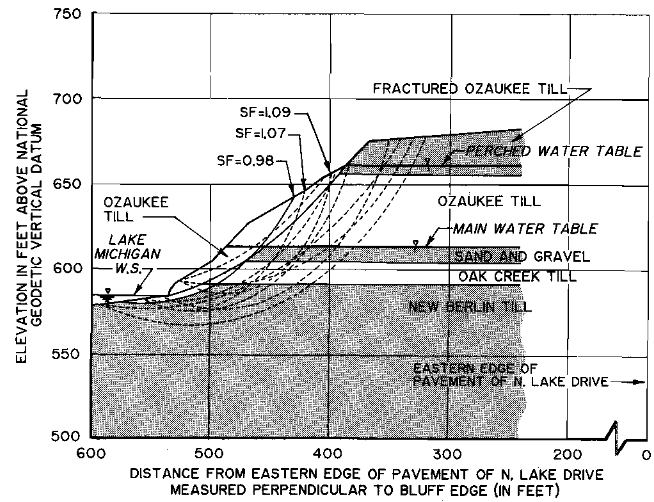
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 3: BLUFF ANALYSIS SECTION 3



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

Figure 58

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 4: BLUFF ANALYSIS SECTION 3



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

Bluff Analysis Section 3

The stability of the bluff slopes within Section 3, which is located at 3510 N. Lake Drive, was characterized by the use of Profile No. 3 and Profile No. 4. The results of the deterministic slope stability analyses, shown in Figure 57 for Profile No. 3 and Figure 58 for Profile No. 4, indicate a threat of bluff slope failure with respect to rotational sliding. The lowest failure surface calculated at Profile No. 3 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 next nine lowest safety factors ranged from 1.09 to 1.38. The lowest failure surface calculated at Profile No. 4 had a safety factor of 0.98, and was also located within the lower two-thirds of the bluff slope. The next nine lowest safety factors ranged from 1.07 to 1.38.

The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 3 ranged from 0.62 to 1.08, with 13, or 65 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated at Profile No. 3, 63, or 32 percent, had safety factors of less than 1.0. The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 4 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. 4, 29, or 15 percent, had safety factors of 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 3 was considered to have a marginal bluff slope with respect to rotational sliding.

Overall, Section 3 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 observed 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 3 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 that covers the lower portion of the slope. Shore protection structures present in the section in 1986 included one concrete bulkhead covering about 150 feet, or 50 percent of shoreline within the section. This structure was in need of major maintenance or reconstruction at the time of the survey. If the lake levels would return to the mean 20th century levels, the resulting beach within Section 3 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 that lies at the base of the slope in the northern part of Section 3. 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 will not be damaged by slight movement of the slump block. Toe protection should be provided along the entire shoreline of the section. Maintenance of a good vegetative cover on the entire bluff slope is recommended to prevent translational sliding.

Bluff Analysis Section 4

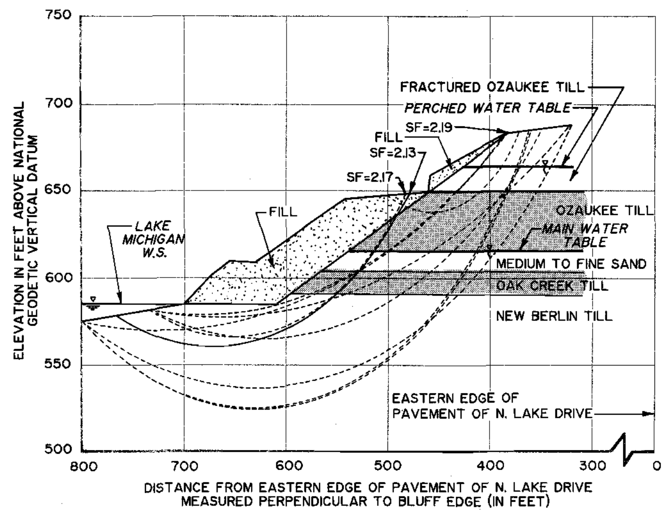
The stability of the fill and the underlying bluff slope within Section 4, which is located at 3534 N. Lake Drive, was characterized by the use of Profile No. 5.

The results of the deterministic slope stability analysis for Profile No. 5 are shown in Figure 59. The lowest failure surface calculated at this profile site had a safety factor of 2.13, and was located on the lower portion of the bluff slope beneath the fill layer. The next nine lowest safety factors ranged from 2.17 to 2.31. A probabilistic slope stability analysis was not conducted for this section because it is a fill site. Based on the deterministic slope stability analysis and on the observed bluff conditions, Section 4 was considered to have a stable bluff slope with respect to rotational sliding.

Section 4 was also considered to have a stable bluff slope with respect to translational sliding. In general, translational sliding within sites covered with concrete rubble and soil fill was considered unlikely because of the ability of the fill material to maintain a relatively steep slope, and because of the benefits realized by loading the base of the slope. A large amount of fill material had been placed at the base of the natural bluff slope within Section 4.

Figure 59

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 5: BLUFF ANALYSIS SECTION 4



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

Primarily because of the effectiveness of the rock and rubble revetment placed at the toe of the fill in Section 4, 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, a significant beach would not be expected to develop within Section 4.

Although the risk of rotational sliding was slight, it is recommended that the top of the terraced fill be regraded to allow surface water to flow toward the lake, rather than to accumulate 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 5

The stability of the bluff slope within Section 5, which extends from 3550 to 3914 N. Lake Drive, was characterized by the use of Profile No. 6.

The results of the deterministic slope stability analysis, shown in Figure 60, indicate that Profile No. 6 has a stable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 1.12. The next nine lowest safety factors ranged from 1.17 to 1.25.

The lowest safety factors indicated by the 20 probabilistic stability analyses ranged from 0.86 to 1.23, with four failure surfaces, or 20 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated, 17 surfaces, or 8 percent, had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 5 was considered to have a stable bluff slope with respect to rotational sliding. However, the probabilistic analysis did indicate that there is a slight potential for slope failure depending upon the specific conditions within the bluff.

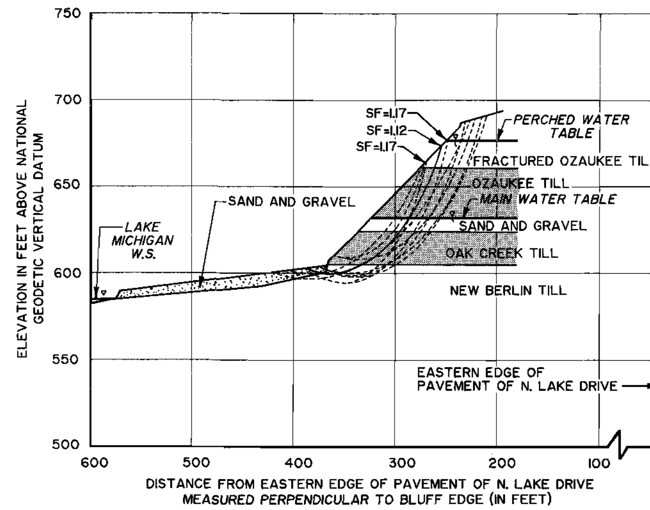
Overall, Section 5 was also considered to have a stable bluff slope with respect to translational sliding. This was due primarily to the good vegetative growth which covered the entire bluff face, and also to the relatively low bluff slope angle of about 25 degrees. Since there were no disturbed soil areas observed during the 1986 field survey within this section, the potential for translational sliding appeared to be minimal.

The Nipissing terrace present at the base of the bluff had experienced significant erosion because of inadequate protection against wave and ice action, and because the material the terrace is composed of is easily eroded. However, because the terrace is 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, the resulting beach within Section 5 would approximate 10 to 20 feet in width.

No measures are needed to prevent rotational or translational sliding within Bluff Analysis Section 5. Bluff toe protection is recommended to protect the terraced portion of the section, which includes the Shorewood Nature Preserve.

Figure 60

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 6: BLUFF ANALYSIS SECTION 5



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

Bluff Analysis Section 6

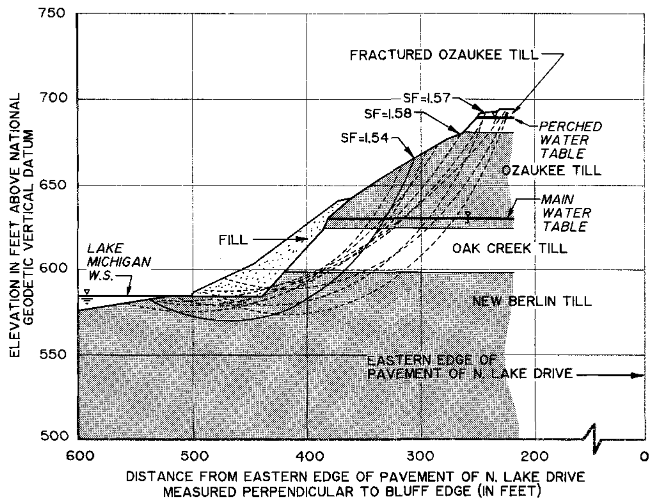
The stability of the fill and the underlying bluff slope within Section 6, which is located at 3926 N. Lake Drive, was characterized by the use of Profile No. 7.

The results of the deterministic slope stability analysis for Profile No. 7 are shown in Figure 61. The lowest failure surface calculated at this profile site had a safety factor of 1.54. The next nine lowest safety factors ranged from 1.57 to 1.61. A probabilistic slope stability analysis was not conducted for this section because it is a fill site. Based on the deterministic slope stability analysis and on the observed bluff conditions, Section 6 was considered to have a stable bluff slope with respect to rotational sliding.

Section 6 was also considered to have a stable bluff slope with respect to translational sliding. In general, translational sliding within sites covered with concrete rubble and soil fill was considered unlikely because of the ability of the fill material to maintain a relatively steep slope, and because of the benefits realized by loading the base of the slope. A large amount of fill material had been placed at the base of the natural bluff slope which reduced the overall slope angle. Furthermore, a good vegetative cover had been established on the fill.

Figure 61

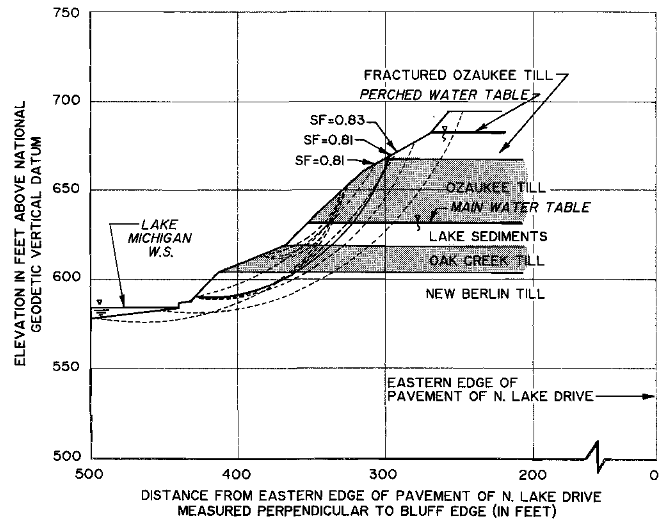
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 7: BLUFF ANALYSIS SECTION 6



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

Figure 62

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 8: BLUFF ANALYSIS SECTION 7



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

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. Even if the lake levels would return to the mean 20th century levels, a significant beach would not be expected to develop within Section 6.

No measures are needed to prevent rotational or translational sliding within Bluff Analysis Section 6. Additional bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 7

The stability of the bluff slope within Section 7, which extends from 3932 to 3966 N. Lake Drive, was characterized by the use of Profile No. 8.

The results of the deterministic slope stability analysis, shown in Figure 62, indicate that Profile No. 8 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.81, and was located within the middle portion of the bluff slope. The next nine lowest safety factors ranged from 0.81 to 0.88.

The lowest safety factors indicated by the 20 probabilistic stability analyses ranged from 0.51 to 0.90. Of the 200 failure surfaces evaluated, 193 surfaces, or 96 percent, had safety factors of less than 1.0. Four houses were located within 50 feet of the edge of the bluff. Based on both the deterministic and probabilistic slope stability analyses and on the observed bluff conditions, Section 7 was considered to have an unstable bluff slope with respect to rotational sliding.

Section 7 was also considered to have an unstable bluff slope with respect to translational sliding. This was due in part to the lack of vegetative cover on most of the bluff slope, and in part to the relatively steep angle of the slope. The potential for translational sliding was further increased by 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 7 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, the resulting beach within Section 7 would probably 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 located as close as 20 feet from the bluff edge. Additional bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 8

The stability of the bluff slope within Section 8, which extends from Atwater Park to 4216 N. Lake Drive, was characterized by the use of Profile No. 9.

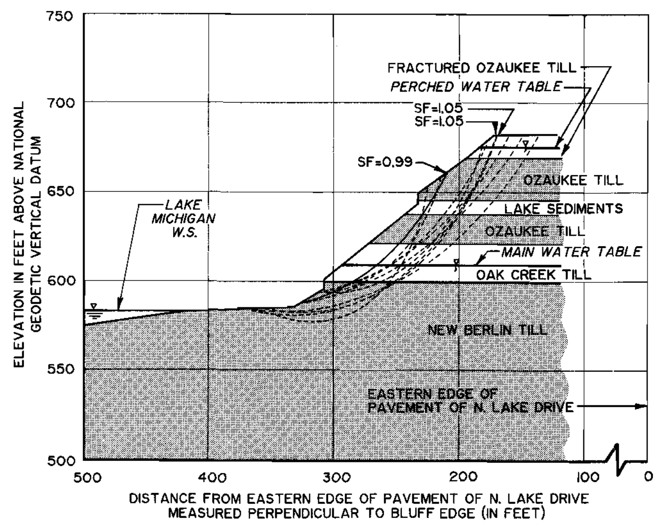
The results of the deterministic slope stability analysis, shown in Figure 63 for Profile No. 9, indicate a threat of bluff slope failure with respect to rotational sliding. The lowest failure surface calculated at Profile No. 9 had a safety factor of 0.99, and was located on the lower two-thirds of the bluff. The next nine lowest safety factors ranged from 1.05 to 1.15.

The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 9 ranged from 0.66 to 1.17, with 12 failure surfaces, or 60 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated at Profile No. 9, 90, or 45 percent, 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.

Figure 63

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 9: BLUFF ANALYSIS SECTION 8



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

Section 8 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 8, 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 marginal portion of the section. If the lake levels would return to the mean 20th century levels, 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 8, 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 1,380 feet of Section 8 to prevent erosion from wave and ice action.

Bluff Analysis Section 9
The stability of the bluff slope within Section 9, which extends from 4226 to 4320 N. Lake Drive, was characterized by the use of Profile No. 10.

The results of the deterministic slope stability analysis, shown in Figure 64, indicate that Profile No. 10 has a stable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 1.79. The next nine lowest safety factors ranged from 1.79 to 1.90. The Nipissing terrace present at the base of the bluff helped improve the stability of the bluff slope.

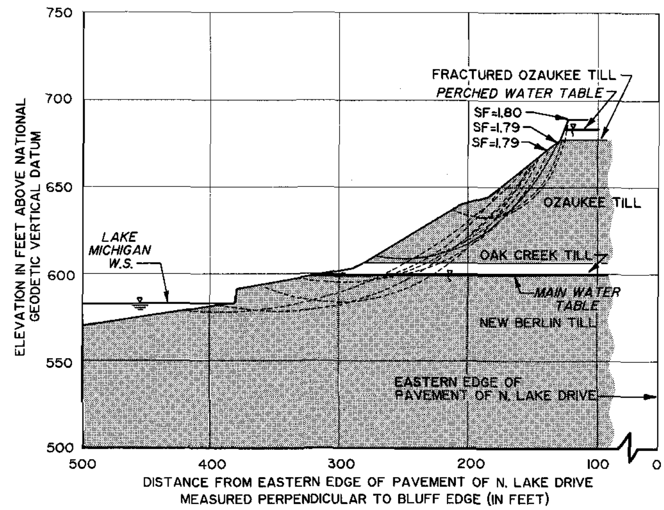
The lowest safety factors indicated by the 20 probabilistic stability analyses ranged from 0.74 to 1.99, with only one failure surface, or 5 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated, only four surfaces, or 2 percent, had a safety factor of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 9 was considered to have a stable bluff slope with respect to rotational sliding.

Overall, Section 9 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 ranges from about 30 to 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 narrower at the northern end of the section. Further toe erosion in this shoreline area may in time begin to affect the stability of the

Figure 64

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 10: BLUFF ANALYSIS SECTION 9



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

bluff slope. If the lake levels would return to the mean 20th century levels, the resulting beach within Section 9 would approximate 10 to 20 feet in width.

No measures are needed to prevent rotational or translational sliding within Bluff Analysis Section 9. Bluff toe protection is recommended to protect the terraced portion of the section, especially within the northern 240 feet of the section where the terrace narrows.

Bluff Analysis Section 10
The stability of the bluff slope within Section 10, which extends from 4400 to 4408 N. Lake Drive, was characterized by the use of Profile No. 11.

The results of the deterministic slope stability analysis, shown in Figure 65 for Profile No. 11, indicate a threat of bluff slope failure with respect to rotational sliding. The lowest failure surface calculated at Profile No. 11 had a safety factor of 0.68, and was located on the lower half of the bluff. The next nine lowest safety factors ranged from 0.69 to 0.88.

The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 11 ranged from 0.61 to 0.97. Of the 200 failure surfaces evaluated at Profile No. 11, 160, or 80 percent, had safety factors of less than 1.0.

When the field surveys were conducted in the summer of 1986, the overall bluff slope within Section 10 was well vegetated, although some slope movement had occurred, and some soil areas were exposed. The elevation of the groundwater shown in Figure 65 was measured in an observation well installed in 1986 at 4408 N. Lake Drive. The slope stability analyses indicated that some slope failures may be expected to occur on the slump block lying on the lower portion of the slope. Both houses within this section were located within 50 feet of the top edge of the bluff. A bulkhead present at the base of the slope had 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 is structurally intact. The probability that the bulkhead will be able to effectively hold the slope and prevent a major failure cannot be evaluated at the systems planning level. It is therefore recommended that a site-specific analysis be conducted to properly evaluate the effect of the bulkhead on the stability of the bluff slope. Based on this information, Bluff Analysis Section 10 was considered to have a marginal bluff slope with respect to rotational sliding; however, the bluff slope should be classified as unstable if it is shown in the site-specific analysis that the bulkhead is not providing suitable protection.

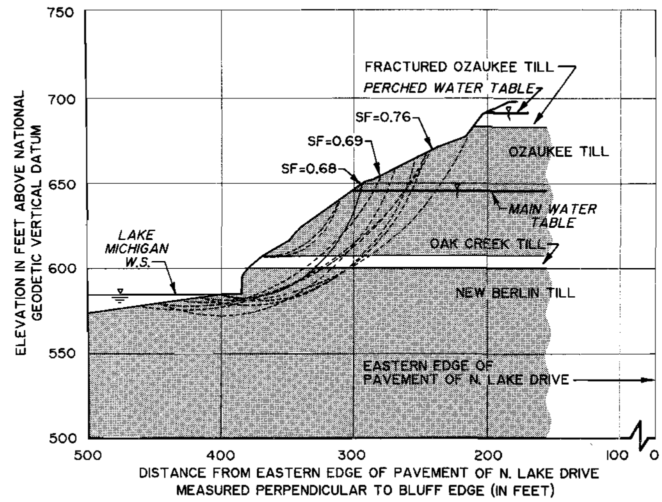
Overall, Section 10 was considered to have a marginal bluff slope with respect to translational sliding. Generally, a good vegetative growth covered most of the bluff slope; however, in areas of little vegetation, there would be a moderate potential for translational sliding because of the relatively steep angle of the bluff slope and the relatively high elevation of the groundwater.

Bluff toe erosion was observed in Section 10 during the field surveys conducted in the summer of 1986. The 200-foot-long concrete bulkhead 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 threatening the stability of the bluff slope. Even if the lake levels would return to the mean 20th century levels, a significant beach would not be expected to develop within Section 10.

To prevent rotational sliding, as well as to provide protection against wave and ice action

Figure 65

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 11: BLUFF ANALYSIS SECTION 10



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

at the toe of the bluff, it is recommended that adequate bluff toe protection be provided within Section 10. 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.

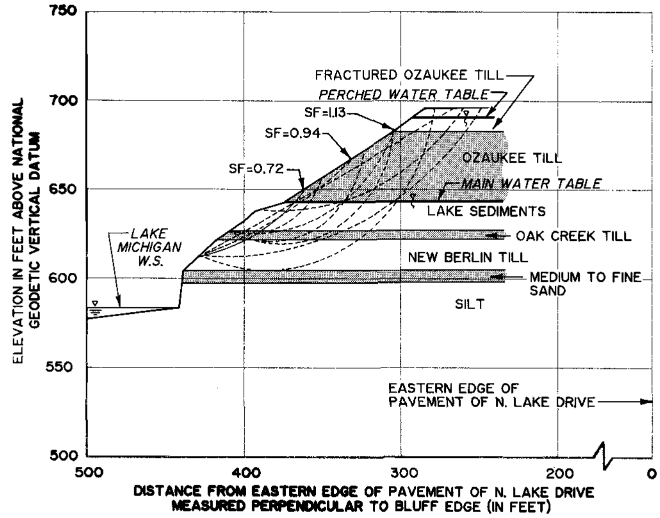
Bluff Analysis Section 11

Bluff Analysis Section 11 was a fill project under construction during the summer of 1986. The stability of the fill and the underlying bluff slope within Section 11, 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. 12 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. 13 and Profile No. 14 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 66, indicate that Profile No. 12 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.72. The next nine lowest safety factors ranged from 0.94

Figure 66

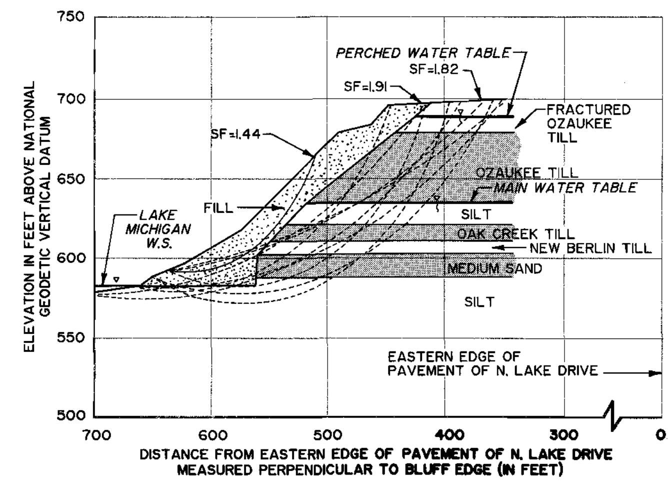
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 12: BLUFF ANALYSIS SECTION 11—PREFILL CONDITIONS



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

Figure 67

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 13: BLUFF ANALYSIS SECTION 11—FILL UNDER CONSTRUCTION A



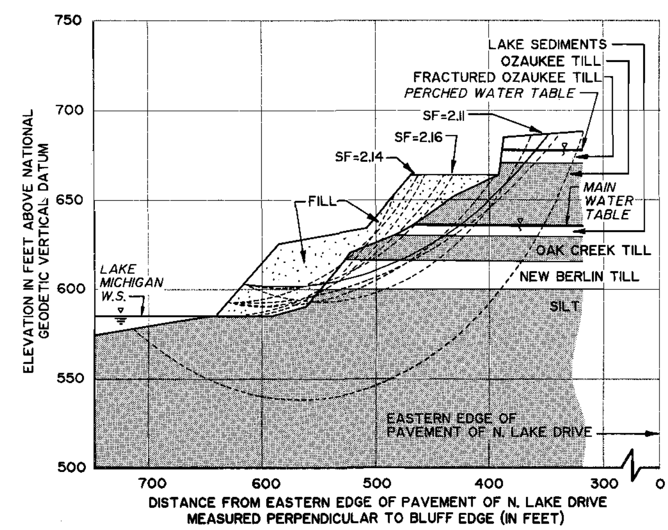
Source: T. B. Edil, D. M. Mickelson, and SEWRPC.

to 1.38. The results of the deterministic slope stability analyses, shown in Figure 67 for Profile No. 13 and Figure 68 for Profile No. 14, indicate that the bluff slope was stable during the construction of the fill. The lowest failure surface calculated at Profile No. 13 had a safety factor of 1.44, and was located within the fill material. The next nine lowest safety factors ranged from 1.82 to 2.48. The lowest failure surface calculated at Profile No. 14 had a safety factor of 2.11. The next nine lowest safety factors ranged from 2.14 to 3.37. A probabilistic slope stability analysis was not conducted for this section because it is a fill site. Based on the deterministic slope stability analyses at Profile Nos. 13 and 14, on the observed bluff conditions, and on the anticipated geometry of the fill project when completed, Section 11 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. 13 and 14.

Section 11 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

Figure 68

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 14: BLUFF ANALYSIS SECTION 11—FILL UNDER CONSTRUCTION B

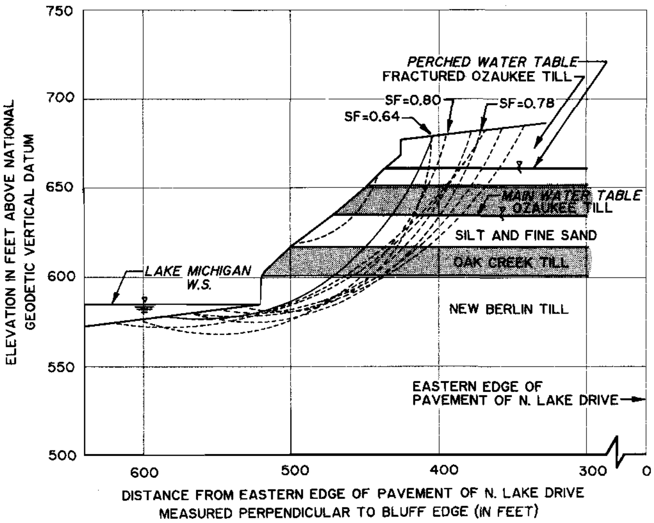


Source: T. B. Edil, D. M. Mickelson, and SEWRPC.

considered unlikely because of the ability of the fill material to maintain a relatively steep slope, and because of the benefits realized by loading the base of the slope. It was anticipated that a large amount of fill material would be placed at the base of the natural bluff slope within Section 11.

Figure 69

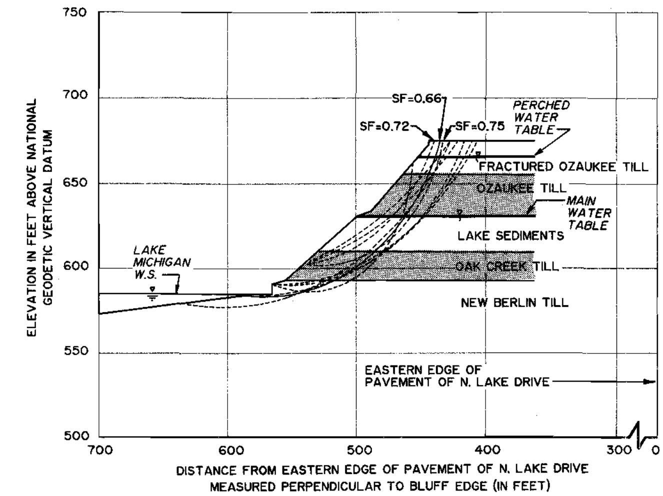
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 15: BLUFF ANALYSIS SECTION 12



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

Figure 70

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 16: BLUFF ANALYSIS SECTION 12



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

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, a significant beach would not be expected to develop within Section 11.

No additional measures are recommended to prevent rotational or translational sliding within Bluff Analysis Section 11 other than the completion of the fill project. 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 12

The stability of the bluff slope within Section 12, which extends from 4668 to 4730 N. Lake Drive, was characterized by the use of Profile No. 15 and Profile No. 16.

The results of the deterministic slope stability analyses, shown in Figure 69 for Profile No. 15 and Figure 70 for Profile No. 16, indicate that the bluff slope is unstable with respect to rotational sliding. The lowest failure surface calculated at Profile No. 15 had a safety factor of 0.64 and included the entire bluff slope. The next nine lowest safety factors ranged from 0.78 to 0.94. The lowest failure surface calculated at Profile No. 16 had a safety factor of 0.66, and also included the entire bluff slope. The next nine lowest safety factors ranged from 0.72 to 0.83.

The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 15 ranged from 0.50 to 0.97. Of the 200 failure surfaces evaluated at Profile No. 15, 185, or 92 percent, had safety factors of less than 1.0. The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 16 ranged from 0.52 to 0.81. Of the 200 failure surfaces evaluated at Profile No. 16, 186, or 93 percent, had safety factors of less than 1.0. Two houses were located within 50 feet of the top edge of the bluff. Based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 12 was considered to have an unstable bluff slope with respect to rotational sliding.

Section 12 was also considered to have an unstable bluff slope with respect to translational sliding. This was due to the lack of vegetative cover on most of the bluff slope, and to the relatively steep angle of the bluff slope.

Bluff toe erosion was observed along the entire shoreline of Section 12 during the field surveys conducted in the summer of 1986, and was identified as a primary cause of bluff slope failure. Shore protection structures present in the section in the summer of 1986 included two concrete bulkheads, each 100 feet in length, that were being overtopped and flanked and which were in need of maintenance at the time of the survey, and a 100-foot-long revetment still under construction composed of limestone rock and grout-filled bags. The remaining shoreline within the section was not protected by shore protection structures at the time of the survey. Even if the lake levels would return to the mean 20th century levels, a significant beach would not be expected to develop within Section 12.

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 13

Bluff Analysis Section 13 was a fill under construction during the summer of 1986. The stability of the fill and the underlying bluff slope within Section 13, which extends from 4744 to 4762 N. Lake Drive, was characterized by the use of Profile No. 17.

The results of the deterministic slope stability analysis, shown in Figure 71, indicate that Profile No. 17 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.61 and was located beneath the layer of fill material, within the natural bluff. The next nine lowest safety factors ranged from 0.67 to 0.86. A probabilistic slope stability analysis was not conducted for this profile because it is a fill site. One house was located within 50 feet of the top edge of the bluff. Based on the conditions of the bluff during the summer of 1986, Section 13 was considered to have an unstable bluff slope with respect to rotational sliding.

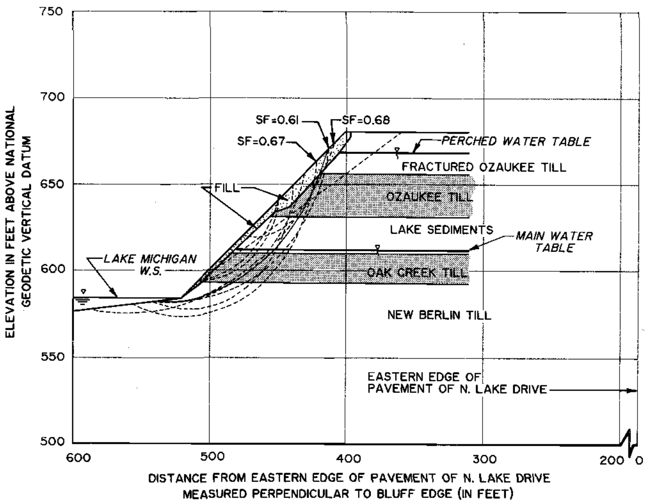
Although translational sliding within fill areas was generally considered unlikely, the potential for sliding was evaluated within this section because of the thin layer of fill placed on the natural bluff slope. Based on the conditions of the bluff slopes during the summer of 1986, Section 13 was considered to have an unstable bluff slope with respect to translational sliding. This was due to the lack of vegetative cover on the bluff slope, and to the steep angle of the bluff slope.

Bluff toe erosion was observed within the entire shoreline of Section 13 during the field surveys conducted in the summer of 1986. This 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, a significant beach would not be expected to develop within Section 13.

To abate the severe potential for both rotational and translational sliding, it is recommended that the bluff slope be regraded to a stable slope angle. This action may require filling, since cutting back the top of the slope may not be feasible because houses at the top of the bluff are as close as 40 feet from the bluff edge. Bluff toe protection is recommended to prevent erosion from wave and ice action. As previously noted, the evaluation for Section 13 was based on the conditions of the bluff slope as of the summer of 1986, at which time a fill project was in progress, and therefore does not reflect the condition of the completed fill.

Figure 71

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 17: BLUFF ANALYSIS SECTION 13



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

Bluff Analysis Section 14

The stability of the bluff slope within Section 14, located at 4780 N. Lake Drive, was characterized by the use of Profile No. 18.

The results of the deterministic slope stability analysis, shown in Figure 72, indicate that Profile No. 18 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.80, and included the entire bluff slope. The next nine lowest safety factors ranged from 0.81 to 0.97.

The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 18 ranged from 0.55 to 0.82. All of the 200 failure surfaces evaluated at Profile No. 18 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 14 was considered to have an unstable bluff slope with respect to rotational sliding.

Section 14 was also considered to have an unstable bluff slope with respect to translational sliding. This was due to the lack of vegetative cover on most of the bluff slope, and to the relatively steep angle of the bluff slope.

Bluff toe erosion was observed within Section 14 during the field surveys conducted in the summer of 1986. This toe erosion was affecting the stability of the bluff slope. As of 1986, no shore protection structures were located within this section. Even if the lake levels would return to the mean 20th century levels, a significant beach would not be expected to develop within Section 14.

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 15

Bluff Analysis Section 15 was a fill under construction during the summer of 1986. The stability of the fill and the underlying bluff slope within Section 15, which extends from 4790 to 4800 N. Lake Drive, was characterized by the use of Profile No. 19.

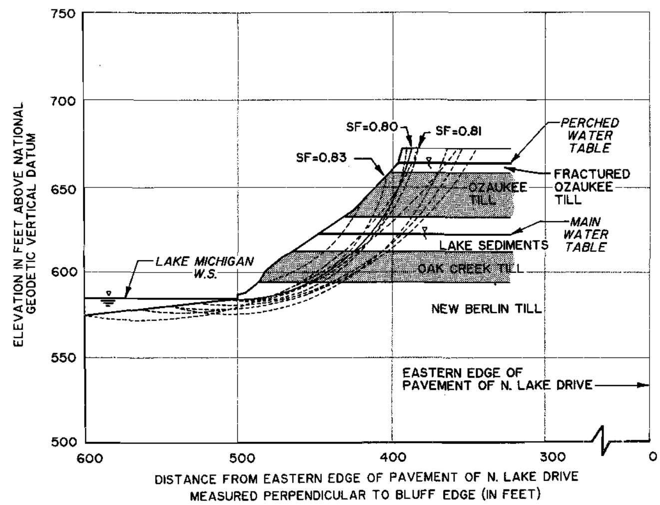
The results of the deterministic slope stability analysis, shown in Figure 73, indicate that Profile No. 19 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.90, and was located within the upper portion of the fill material. The next nine lowest safety factors ranged from 0.91 to 0.96. 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. Based on the conditions of the bluff slope during the summer of 1986, Section 15 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 15 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 15 during the field surveys conducted in May 1986. This erosion was contributing to the instability of the bluff slope. During the summer of 1986, a 300-foot-long revetment composed of stone blocks and grout-filled bags was under construction. The effectiveness of this structure was not evaluated.

Figure 72

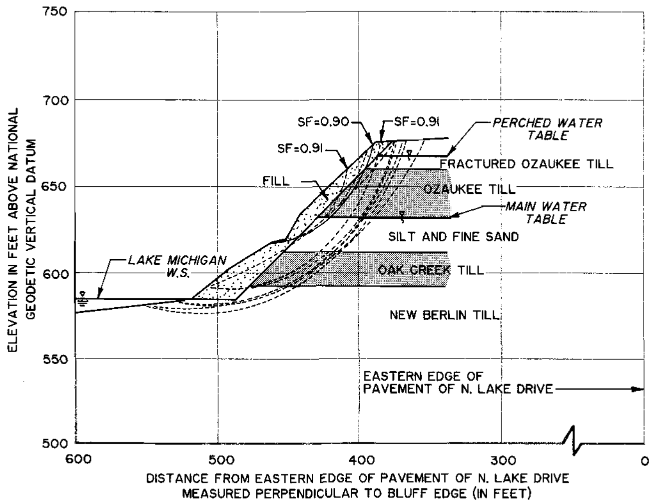
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 18: BLUFF ANALYSIS SECTION 14



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

Figure 73

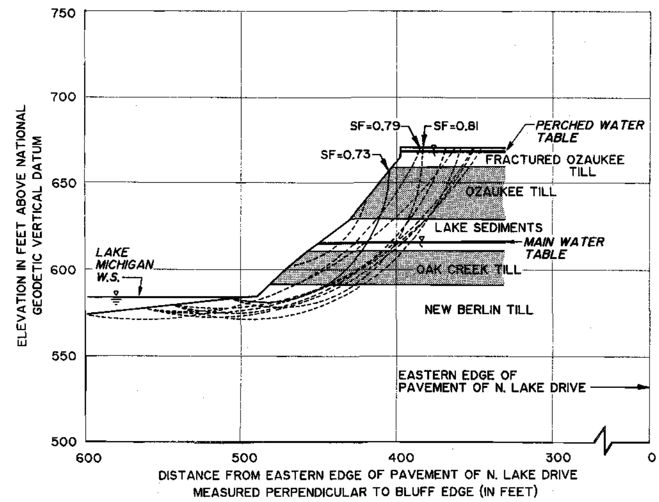
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 19: BLUFF ANALYSIS SECTION 15



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

Figure 74

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 20: BLUFF ANALYSIS SECTION 16



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

Even if the lake levels would return to the mean 20th century levels, a significant beach would not be expected to develop within Section 15.

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 25 feet from the bluff edge. Bluff toe protection is recommended to prevent erosion from wave and ice action. As previously noted, the evaluation for Section 15 was based on the conditions of the bluff slope as of the summer of 1986, at which time a fill project was in progress, and therefore does not reflect the condition of the completed fill.

Bluff Analysis Section 16

The stability of the bluff slope within Section 16, which extends from 4810 to 4840 N. Lake Drive, was characterized by the use of Profile No. 20.

The results of the deterministic slope stability analysis, shown in Figure 74, indicate that Profile No. 20 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.73, and was located within the lower portion of the bluff slope. The next nine lowest safety factors ranged from 0.79 to 0.94.

The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 20 ranged from 0.53 to 0.83. Of the 200 failure surfaces evaluated at Profile No. 20, 190, or 95 percent, had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 16 was considered to have an unstable bluff slope with respect to rotational sliding.

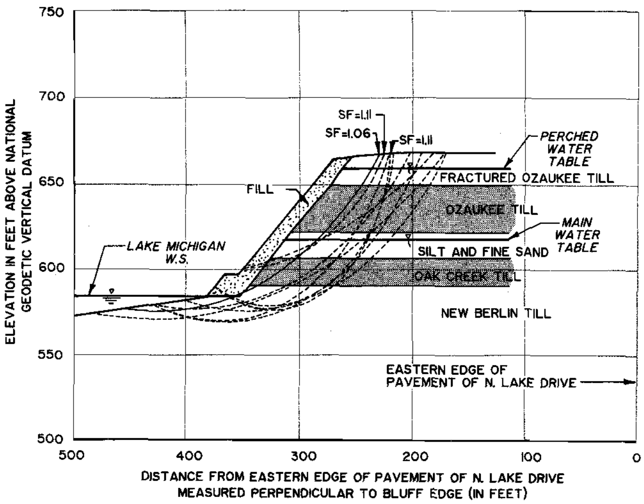
Section 16 was also considered to have an unstable bluff slope with respect to translational sliding. This was due to the lack of vegetative cover on most of the bluff slope, and to the relatively steep angle of the bluff slope.

Bluff toe erosion was observed within Section 16 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 this section. Even if the lake levels would return to the mean 20th century levels, a significant beach would not be expected to develop within Section 16.

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 16.

Figure 75

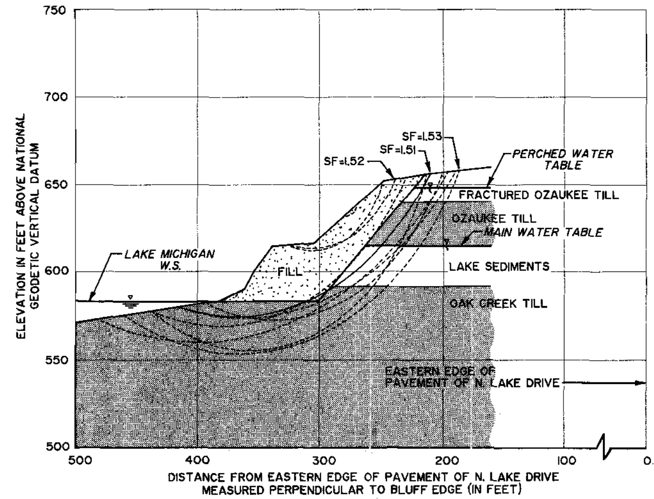
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 21: BLUFF ANALYSIS SECTION 17



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

Figure 76

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 22: BLUFF ANALYSIS SECTION 17



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

Bluff Analysis Section 17

The stability of the fill and the underlying bluff slope within Section 17, which extends from 4850 to 4940 N. Lake Drive, was characterized by the use of Profile No. 21 and Profile No. 22.

The results of the deterministic slope stability analyses, shown in Figure 75 for Profile No. 21 and Figure 76 for Profile No. 22, indicate stable bluff slopes with respect to rotational sliding. The lowest failure surface calculated at Profile No. 21 had a safety factor of 1.06, and was located beneath the fill layer. The next nine lowest safety factors ranged from 1.11 to 1.44. The lowest failure surface calculated at Profile No. 22 had a safety factor of 1.51, and was also located beneath the fill layer. The next nine lowest safety factors ranged from 1.52 to 1.59. A probabilistic slope stability analysis was not conducted for this section because it is a fill site. Therefore, based on the deterministic slope stability analysis and on the observed bluff conditions, Section 17 was considered to have a stable bluff slope with respect to rotational sliding.

Section 17 was also considered to have a stable bluff slope with respect to translational sliding. In general, translational sliding within fill areas was considered unlikely because of the ability of the fill material to maintain a relatively steep slope, and because of the benefits realized by loading the base of the slope. A large amount of fill material had been placed at the base of the natural bluff slope within Section 17.

Bluff toe erosion was observed within the northern portion of Section 17 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 erosion was not affecting the stability of the bluff slope. Within the southern portion of the section, where the fill project was still under construction in 1986, a revetment composed of rubble and concrete slabs was being placed at the toe of the fill for protection against wave action. Because the structure had not been completed, the degree of bluff toe protection provided could not be determined. Even if the lake levels would return to the mean 20th century levels, a significant beach would not be expected to develop within Section 17.

No measures are needed to prevent rotational or translational sliding within Bluff Analysis Section 17. Additional bluff toe protection is recommended at the base of the fill to prevent erosion from wave and ice action.

Bluff Analysis Section 18

The stability of the bluff slope within Section 18, which includes Buckley Park and the southern portion of Big Bay Park, was characterized by the use of Profile No. 23.

The results of the deterministic slope stability analysis, shown in Figure 77 for Profile No. 23, indicate a threat of bluff slope failure as a result of rotational sliding. The lowest failure surface calculated at Profile No. 23 had a safety factor of 0.91, and included the entire bluff slope. The next nine lowest safety factors ranged from 0.98 to 1.07.

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

In the 1986 summer field 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 Buckley Park. Therefore, based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 18 was considered to have a marginal bluff slope with respect to rotational sliding.

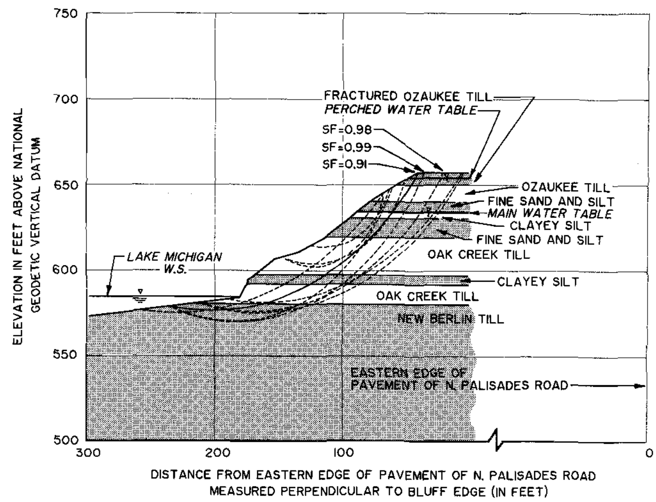
Overall, Section 18 was considered to have a stable bluff slope with respect to translational sliding. This was due to the good vegetative growth that covered most of the bluff face. There were, however, small disturbed soil areas observed on portions of the bluff slope, especially within the recent slope failure, where there was a moderate potential for translational sliding.

In the summer of 1986, the toe of the bluff was protected by a concrete bulkhead. While the bulkhead offered some protection, there was erosion of the bluff toe by waves washing over the top of the structure. This erosion was contributing to the instability of the bluff slope. A portion of this bulkhead collapsed when the bluff slope failed in November 1986. Even if the lake levels would return to the mean 20th century levels, it is anticipated that a significant beach would develop within Section 18.

To prevent rotational sliding within the northern 1,060 feet of Section 18, it is recommended that a groundwater drainage system be installed to lower the groundwater elevation. Within the southern 600 feet of the section, which includes Buckley Park, it is recommended that the bluff slope be regraded to a stable slope angle. Also, additional toe erosion control measures are recommended along the entire section to prevent erosion from wave and ice action.

Figure 77

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 23: BLUFF ANALYSIS SECTION 18



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

Bluff Analysis Section 19

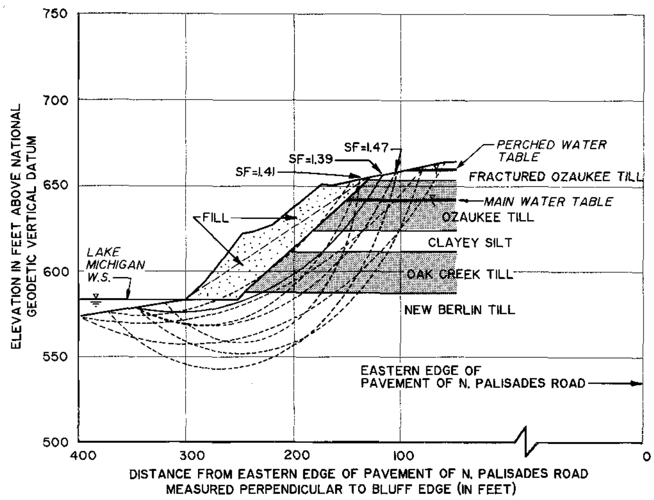
The stability of the fill and the underlying bluff slope within Section 19, which extends from the northern portion of Big Bay Park to 5270 N. Lake Drive, was characterized by the use of Profile No. 24.

The results of the deterministic slope stability analysis, shown in Figure 78, indicate that Profile No. 24 has a stable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 1.39, and was located beneath the fill. The next nine lowest safety factors ranged from 1.41 to 1.71. A probabilistic slope stability analysis was not conducted for this section because it is a fill area.

Section 19 was also considered to have a stable bluff slope with respect to translational sliding. In general, translational sliding within fill areas was considered unlikely because of the ability of the fill material to maintain a relatively steep slope, and because of the benefits realized by

Figure 78

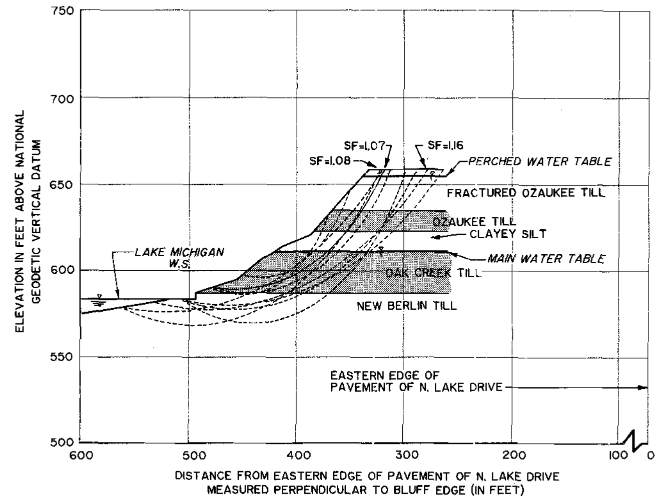
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 24: BLUFF ANALYSIS SECTION 19



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

Figure 79

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 25: BLUFF ANALYSIS SECTION 20



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

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 19.

A small amount of bluff toe erosion was observed in Section 19 during the field surveys conducted in the summer of 1986. A revetment composed of rock and concrete rubble and a concrete bulkhead located within the section were not providing adequate toe protection against wave and ice action. Because of the large amount of fill material placed at the base of the bluff, the observed toe erosion was not affecting the stability of the bluff. Even if the lake levels would return to the mean 20th century levels, a significant beach would not be expected to develop within Section 19.

No measures are needed to prevent rotational or translational sliding within Bluff Analysis Section 19. Additional bluff toe protection is recommended at the base of the fill to prevent erosion from wave and ice action.

Bluff Analysis Section 20

The stability of the bluff slope within Section 20, located at 5290 N. Lake Drive, was characterized by the use of Profile No. 25.

The results of the deterministic slope stability analysis, shown in Figure 79 for Profile No. 25, indicate that the bluff slope is just barely stable with respect to rotational sliding. The lowest failure surface calculated had a safety factor of 1.07, and was located mainly within the upper portion of the bluff slope. The remaining nine lowest safety factors ranged from 1.08 to 1.33.

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

In the field 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 20 was considered to have a marginal bluff slope with respect to rotational sliding.

Section 20 was also considered to have a marginal bluff slope with respect to translational sliding. The base of the bluff had good vegetative cover, with a relatively gentle slope angle of approximately 20 degrees. The upper portion of the bluff slope contained disturbed soil areas, with a much steeper slope of approximately 35 degrees.

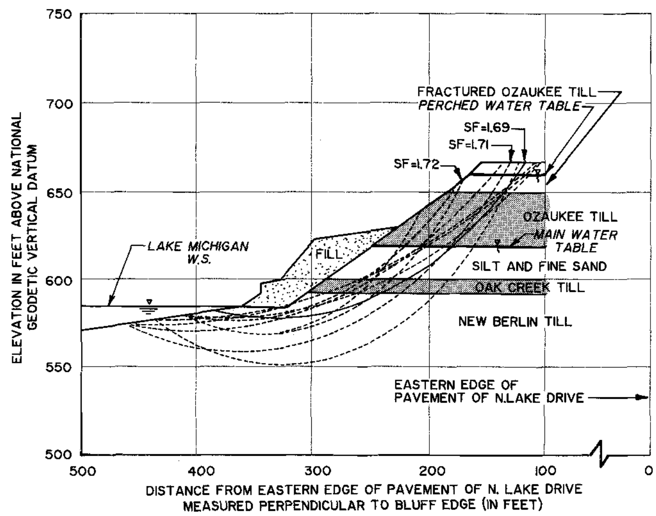
Therefore, the potential for translational sliding was far greater on the upper portion of the bluff slope than on the lower bluff slope.

Due primarily to the relatively wide beach built up in Section 20, 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, the resulting beach within Section 20 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.

Figure 80

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 26: BLUFF ANALYSIS SECTION 21



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

Bluff Analysis Section 21

The stability of the fill and underlying bluff slope within Section 21, which extends from 5300 N. Lake Drive to 808 Lakeview Avenue, was characterized by the use of Profile No. 26 and Profile No. 27.

The results of the deterministic slope stability analyses, shown in Figure 80 for Profile No. 26 and Figure 81 for Profile No. 27, indicate stable bluff slopes with respect to rotational sliding. The lowest failure surface calculated at Profile No. 26 had a safety factor of 1.69, and was located beneath the fill layer. The next nine lowest safety factors ranged from 1.71 to 1.81. The lowest failure surface calculated at Profile No. 27 had a safety factor of 1.75, and was located beneath the top portion of the fill layer. The next nine lowest safety factors ranged from 1.80 to 2.02. A probabilistic slope stability analysis was not conducted for this section because it is a fill area. Therefore, based on the deterministic slope stability analysis and on observed bluff conditions, Section 21 was considered to have a stable bluff slope with respect to rotational sliding.

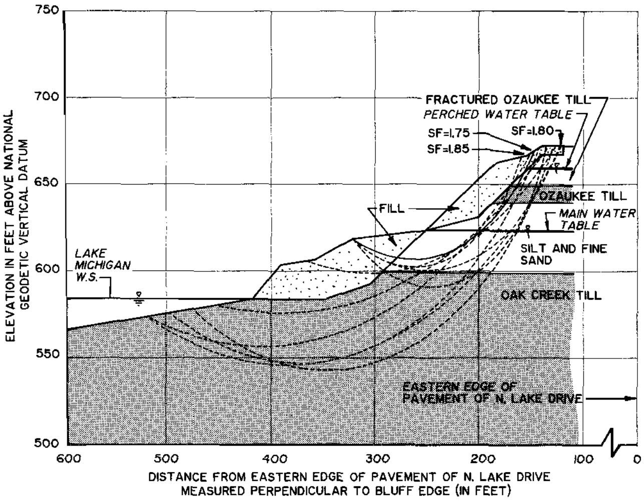
Section 21 was also considered to have a stable bluff slope with respect to translational sliding. In general, translational sliding within fill areas was considered unlikely because of the ability of the fill material to maintain a relatively steep slope, and because of the benefits realized by loading the base of the slope. Within the northern portion of Section 21, translational sliding was considered unlikely to occur because of the large amount of fill material placed on nearly the entire natural bluff slope. 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 21, 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. North of Silver Spring Drive, where the fill project was still under construction in 1986, a rock revetment was being placed at the toe of the fill for protection. Because the structure had not been completed as of the time of the field surveys, the degree of bluff toe protection was not evaluated. Even if the lake levels would return to the mean 20th century levels, a significant beach would not be expected to develop within Section 21.

No measures are needed to prevent rotational sliding within Bluff Analysis Section 21. Only minimal translational sliding may be expected

Figure 81

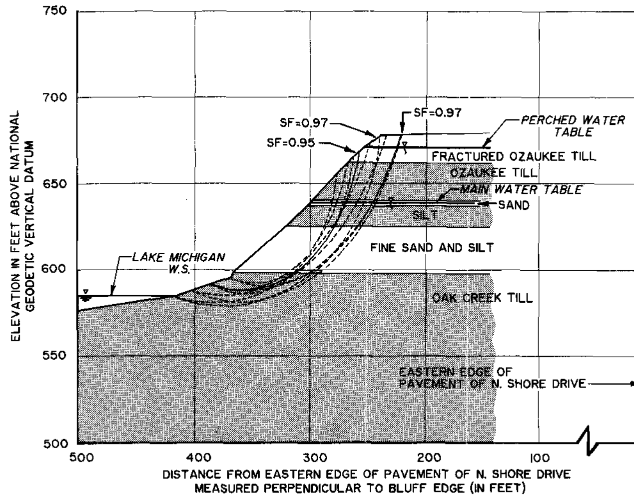
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 27: BLUFF ANALYSIS SECTION 21



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

Figure 82

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 28: BLUFF ANALYSIS SECTION 22



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

to occur—primarily on the upper bluff slope in the southern portion of the section. Additional toe erosion control should be provided along the southern 1,700 feet of the section south of Silver Spring Drive, and adequate toe protection is recommended north of Silver Spring Drive, when the fill project is completed in that area, to prevent erosion by wave and ice action.

Bluff Analysis Section 22

The stability of the bluff slope within Section 22, which extends from 5722 to 5770 N. Shore Drive, was characterized by the use of Profile No. 28.

The results of the deterministic slope stability analysis, shown in Figure 82, indicate that Profile No. 28 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated had a safety factor of 0.95, and was located within the middle portion of the bluff slope. The next nine lowest safety factors ranged from 0.97 to 0.99.

The lowest safety factors indicated by the 20 probabilistic stability analyses ranged from 0.47 to 1.12, with 17 of the failure surfaces, or 85 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated, 159, or 80 percent, had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 22 was considered to have an unstable bluff slope with respect to rotational sliding.

Section 22 was considered to have a marginal bluff slope with respect to translational sliding. This was due to the relatively steep slope of the bluff, and the abundance of disturbed soil areas located throughout the section. The potential for translational sliding was greater on the lower portion of the bluff, where groundwater seepage was noted during the 1986 field surveys.

Minor erosion of the toe of the bluff due to wave action was observed. Should it continue, this erosion may affect the stability of the bluff slope. In the summer of 1986, the toe of the bluff was protected by a relatively wide beach. However, during the study period, the beaches were eroding rapidly. If the lake levels would return to the mean 20th century levels, the resulting beach within Section 22 would approximate 60 feet in width.

To prevent rotational sliding within Section 22, it is recommended that a groundwater drainage system be installed to lower the groundwater elevation, and 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 23
The stability of the fill and the underlying bluff slope within Section 23, which is located at 758 E. Day Avenue, was characterized by the use of Profile No. 29.

The results of the deterministic slope stability analysis, shown in Figure 83, indicate that Profile No. 29 has a stable bluff with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 1.14, and was located beneath the fill. The next nine lowest safety factors ranged from 1.14 to 1.18. A probabilistic slope stability analysis was not conducted for this section because it is a fill area. Therefore, based on the deterministic slope stability analysis and on observed bluff conditions, Section 23 was considered to have a stable bluff slope with respect to rotational sliding.

Section 23 was also considered to have a stable bluff slope with respect to translational sliding. In general, translational sliding within fill areas was considered unlikely because of the ability of the fill material to maintain a relatively steep slope, and because of the benefits realized by loading the base of the slope. The fill material placed on the natural bluff slope, especially within the lower portion of the slope, should minimize the potential for translational sliding.

Bluff toe erosion was observed within Section 23 during the field surveys conducted in the summer of 1986. This 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, the resulting beach within Section 23 would approximate 40 to 50 feet in width.

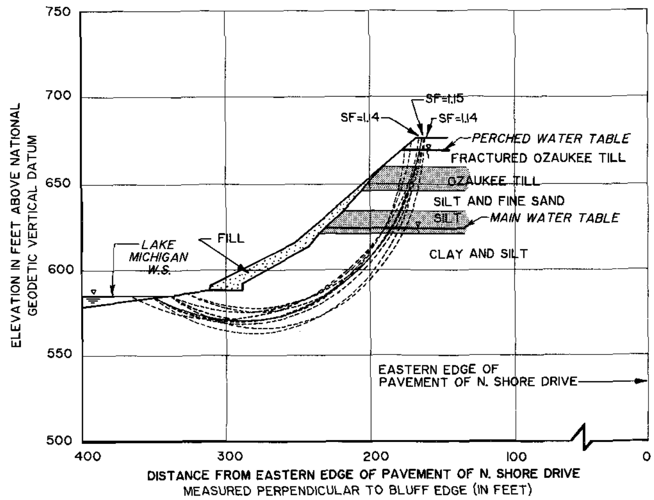
No measures are needed to prevent rotational or translational sliding within Bluff Analysis Section 23. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 24
The stability of the bluff slope within Section 24, which extends from 740 E. Day Avenue to 5866 N. Shore Drive, was characterized by the use of Profile No. 30.

The results of the deterministic slope stability analysis, shown in Figure 84 for Profile No. 30, indicate a threat of bluff slope failure with

Figure 83

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 29: BLUFF ANALYSIS SECTION 23



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

respect to rotational sliding. The lowest failure surface calculated at Profile No. 30 had a safety factor of 0.96, and included the entire bluff slope. The next nine lowest safety factors ranged from 0.96 to 1.03.

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

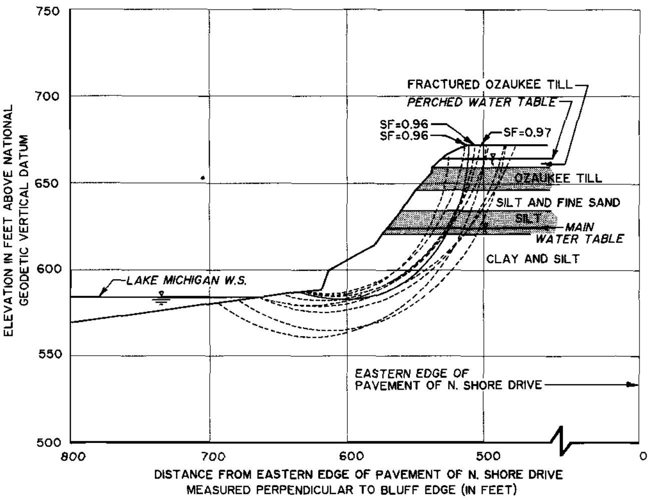
In the 1986 field surveys, small slips and slumps were noted throughout the section. Based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 24 was considered to have an unstable bluff slope with respect to rotational sliding.

Section 24 was considered to have a marginal bluff slope with respect to translational sliding. This was due to the relatively steep slope of the bluff, and the abundance of disturbed soil areas located throughout the section. The potential for translational sliding was greater on the lower portion of the bluff, where groundwater seepage was noted during the 1986 field 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

Figure 84

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 30: BLUFF ANALYSIS SECTION 24



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

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, the resulting beach within Section 24 would approximate 50 feet in width.

To prevent rotational and translational sliding within Section 24, 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 25

The stability of the bluff slope within Section 25, which is located at Klode Park, was characterized by the use of Profile No. 31.

The results of the deterministic slope stability analysis, shown in Figure 85 for Profile No. 31, indicate a threat of bluff slope failure with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.65, and was located within the lower portion of the bluff slope. The next nine lowest safety factors were much higher, ranging from 0.99 to 1.19.

The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 31 ranged from 0.53 to 1.13, with 14 of the failure surfaces, or 70 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated at Profile No. 31, 70, or 35 percent, had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions in the summer of 1986, Section 25 was considered to have an unstable bluff slope with respect to rotational sliding.[7]

---

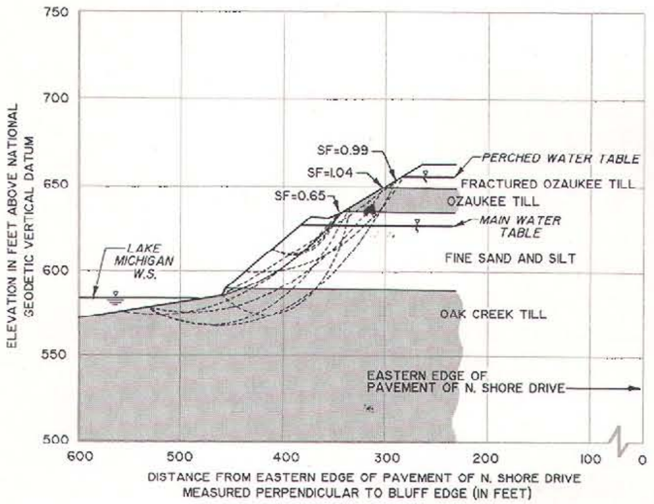
[7] *In December 1986, 240 lineal feet of a concrete bulkhead protecting the northern portion of Section 25 north of the North Shore Water Commission pumping station collapsed. During the same storm event, a slope failure involving the lower 10 to 15 feet of the bluff along the entire section occurred, as shown in Figure 86. To protect the pumping station, approximately 4.5 tons per lineal foot of 500- to 2,000-pound rock rip-rap and fill were placed behind the remaining bulkhead in the southern portion of the section in January 1987. Following the placement of the rip-rap and fill, a slope stability analysis conducted by Warzyn Engineering, Inc., indicated that the safety factor for the southern reach of the pumping station was approximately 1.25.*

During April 1987, a large rotational slide occurred at Klode Park north of the pumping station and reinforced bulkhead, changing the configuration of the slope, as shown in Figure 86. The failure occurred generally along the plane indicated in Figure 85, which had a safety factor of about 0.65, indicating a high potential for failure. A water-bearing sand layer was exposed on the bluff failure surface, near the top of the slide. Two soil borings conducted by Warzyn Engineering, Inc., at beach level in Klode Park indicated two and one-half to six feet of sand and gravel, underlain by Oak Creek till.

To protect Klode Park and the pumping station, the Village of Whitefish Bay authorized in the summer of 1987 the design and construction of a sand and gravel beach to be contained by three offshore breakwaters and steel sheet pile groins. To assure the proper performance of the design, the proposed shore protection measures were physically modeled at a scale of 1 to 20 in the Canadian National Research Laboratory's Hydraulic Offshore Wave Basin in Ottawa, Canada, in June and July of 1987. Construction of the project began in the fall of 1987, and the new beach system was opened to the public in July 1988.

Figure 85

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 31: BLUFF ANALYSIS SECTION 25



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

Overall, Section 25 was considered to have a marginal bluff slope with respect to translational sliding. Generally, good vegetative growth 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 (see Figure 86).

Because of the importance of the North Shore Water Commission pumping station, it was essential that a site-specific analysis of the stability of the bluff slope be conducted, and that additional protection measures be installed to protect the toe of the bluff.[8]

Bluff Analysis Section 26

The stability of the bluff slope within Section 26, which is located at 5960 N. Shore Drive, was characterized by the use of Profile No. 32.

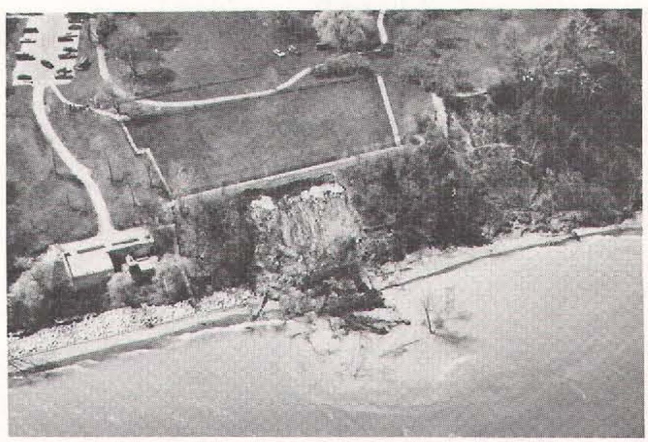
The results of the deterministic slope stability analysis, shown in Figure 87, indicate that Profile No. 32 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a

Figure 86

FAILURE OF THE KLODE PARK BLUFF SLOPE IN BLUFF ANALYSIS SECTION 25: 1986 AND 1987

1986

1987



Source: SEWRPC.

safety factor of 0.70, and was located within the middle portion of the bluff slope. The next nine lowest safety factors ranged from 0.71 to 0.88.

The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 32 ranged from 0.52 to 1.10, with 18 of the failure surfaces, or 90 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated at Profile No. 32, 163, or 82 percent, had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 26 was considered to have an unstable bluff slope with respect to rotational sliding.

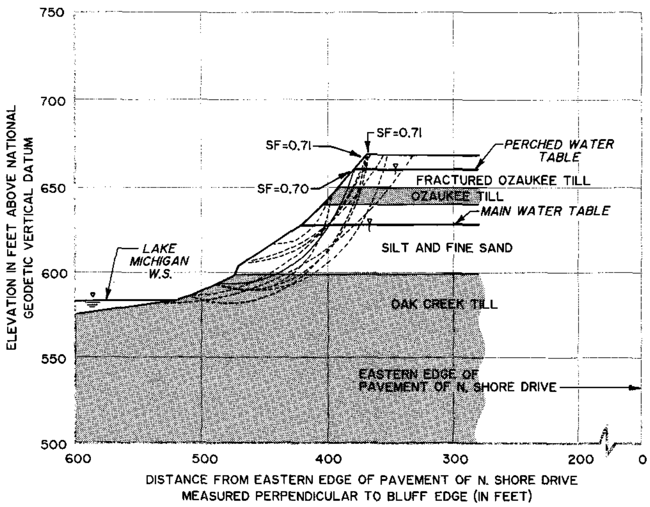
Section 26 was also considered to have an unstable bluff slope with respect to translational sliding.

---

[8]*As noted in footnote 7, measures to protect the bluff in Section 25 were undertaken by the Village of Whitefish Bay during 1986 and 1987.*

Figure 87

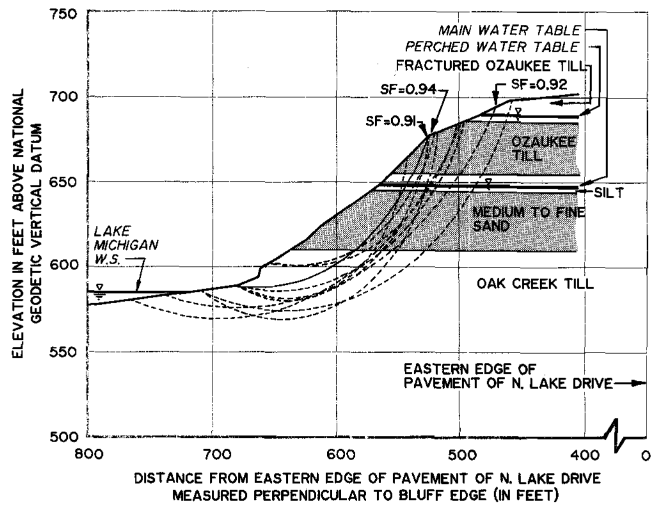
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 32: BLUFF ANALYSIS SECTION 26



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

Figure 88

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 33: BLUFF ANALYSIS SECTION 27



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

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 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, the resulting beach within Section 26 would approximate 40 feet in width.

To prevent rotational and translational sliding within Section 26, 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 27

The stability of the bluff slope within Section 27, which extends from 6000 N. Shore Drive to 6260 N. Lake Drive, was characterized by the use of Profile No. 33.

The results of the deterministic slope stability analysis, shown in Figure 88 for Profile No. 33, indicate a threat of bluff slope failure with respect to rotational sliding. The lowest failure surface calculated at Profile No. 33 had a safety factor of 0.91, and was located on the lower portion of the bluff slope. The next nine lowest safety factors ranged from 0.92 to 1.03.

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

In the 1986 field surveys, the overall bluff slope within Section 27 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 27, there was a potential for deep-seated failures. Therefore, based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 27 was considered to have a marginal bluff slope with respect to rotational sliding.

Overall, Section 27 was also considered to have a marginal bluff slope with respect to translational sliding. Generally, a good vegetative growth covered most of the bluff face. There were, however, small disturbed soil areas observed on portions of the bluff slope, especially on the lower 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, the resulting beach within Section 27 would approximate 30 feet in width.

To prevent rotational sliding in Section 27, 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 28

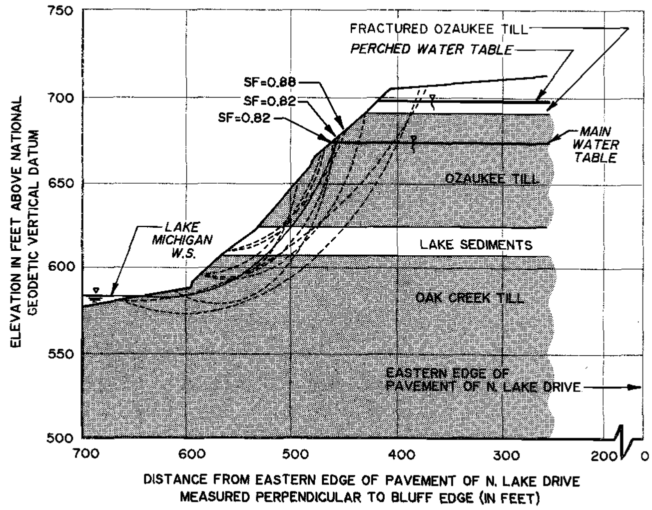
The stability of the bluff slope within Section 28, which extends from 6310 to 6424 N. Lake Drive, was characterized by the use of Profile No. 34 and Profile No. 35.

The results of the deterministic slope stability analyses, shown in Figure 89 for Profile No. 34 and Figure 90 for Profile No. 35, indicate the bluff slope is unstable with respect to rotational sliding. The lowest failure surface calculated at Profile No. 34 had a safety factor of 0.82, and was located within the lower two-thirds of the bluff slope. The next nine lowest safety factors calculated at Profile No. 34 ranged from 0.82 to 0.95. The lowest failure surface calculated at Profile No. 35 had a safety factor of 0.86, and was also located within the lower portion of the bluff slope. The next nine lowest safety factors calculated at Profile No. 35 ranged from 0.88 to 1.02.

The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 34 ranged from 0.63 to 1.00, with 19 of the failure surfaces, or 95 percent, having a safety factor of less than 1.0. Of the 200 failure

Figure 89

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 34: BLUFF ANALYSIS SECTION 28



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

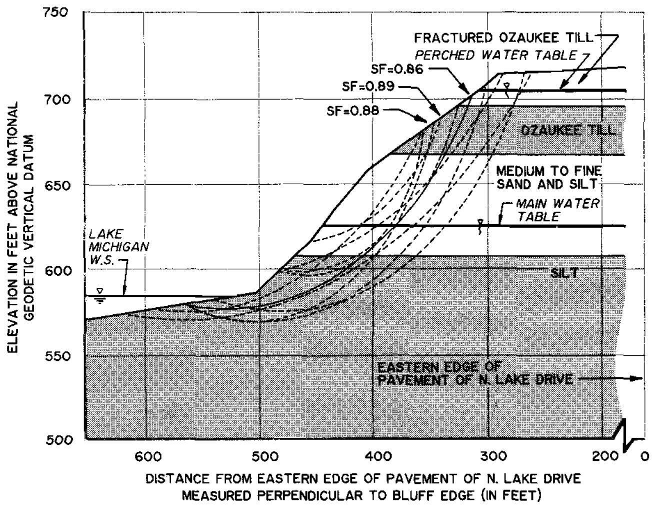
surfaces evaluated at Profile No. 34, 137, or 68 percent, had safety factors of less than 1.0. Of the 20 probabilistic stability analyses conducted for Profile No. 35, the lowest safety factors ranged from 0.51 to 1.03, with 18 of the failure surfaces, or 90 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated at Profile No. 35, 167, or 84 percent, had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 28 was considered to have an unstable bluff slope with respect to rotational sliding.

Section 28 was also considered to have an unstable bluff slope with respect to translational sliding. This was due to the lack of vegetative cover on most of the bluff face, and to the relatively steep angle of the bluff slope. The potential for translational sliding was even greater within the lower portion of the bluff slope because of the groundwater seepage occurring in the silt and sand layers.

Bluff toe erosion was observed within the entire shoreline of Section 28 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

Figure 90

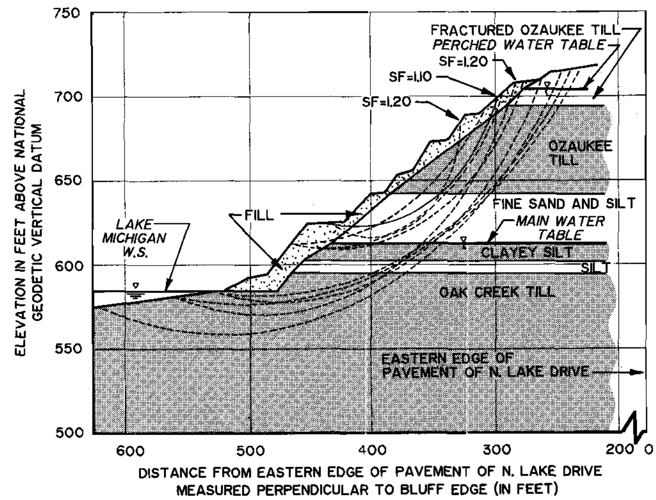
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 35: BLUFF ANALYSIS SECTION 28



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

Figure 91

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 36: BLUFF ANALYSIS SECTION 29



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

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, the resulting beach within Section 28 would approximate 40 feet in width.

To abate the potential for both rotational and translational sliding within Section 28, 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 29

Bluff Analysis Section 29 was a fill under construction during the summer of 1986. The stability of the fill and the underlying bluff slope within Section 29, which extends from 6430 to 6448 N. Lake Drive, was characterized by the use of Profile No. 36.

The results of the deterministic slope stability analysis, shown in Figure 91, indicate that Profile No. 36 has a stable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 1.10, and was located beneath the middle portion of the fill. The next nine lowest safety factors ranged from 1.20 to 1.24. A probabilistic slope stability analysis was not conducted for this section because it is a fill area.

Section 29 was also considered to have a stable bluff slope with respect to translational sliding. In general, translational sliding within fill areas was considered unlikely because of the ability of the fill material to maintain a relatively steep slope, and because of the benefits realized by loading the base of the slope. A large amount of fill material had been placed at the base of the natural bluff slope within Section 29.

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, a significant beach would not be expected to develop within Section 29.

No additional measures are needed to prevent rotational or translational sliding within Bluff

Analysis Section 29, other than the completion of the fill project. It is recommended that adequate toe protection be provided at the base of the fill, when the project is completed, to prevent erosion by wave and ice action.

Bluff Analysis Section 30

The stability of the bluff slope within Section 30, which extends from 6464 to 6530 N. Lake Drive, was characterized by the use of Profile No. 37.

The results of the deterministic slope stability analysis, shown in Figure 92, indicate that Profile No. 37 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.91, and was located within the middle portion of the bluff slope. The next nine lowest safety factors ranged from 0.92 to 0.98.

The lowest safety factors indicated by the 20 probabilistic stability analyses ranged from 0.41 to 0.90. All of the 200 failure surfaces evaluated 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 30 was considered to have an unstable bluff slope with respect to rotational sliding.

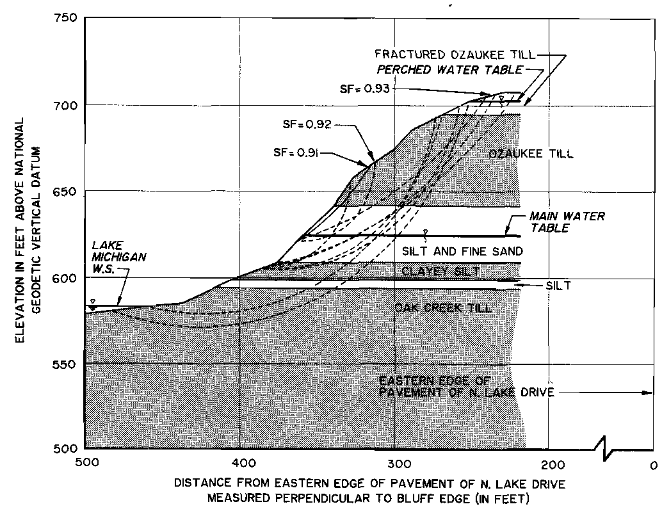
Section 30 was also considered to have an unstable bluff slope with respect to translational sliding. This was due to the lack of vegetative cover on the lower portion of the bluff slope, and to the relatively steep angle of the bluff slope. The potential for translational sliding was even greater within the lower portion of the bluff slope because of the groundwater seepage at the top of the silt and sand layer.

Bluff toe erosion was observed within the entire shoreline of Section 30 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, a significant beach would not be expected to develop within Section 30.

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 some houses at the top of the bluff are within 10 feet of the bluff edge. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Figure 92

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 37: BLUFF ANALYSIS SECTION 30



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

Bluff Analysis Section 31

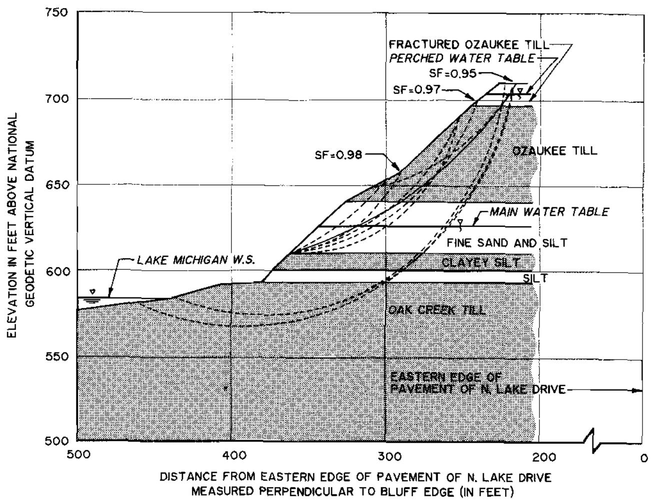
The stability of the bluff slope within Section 31, which extends from 6600 to 6702 N. Lake Drive, was characterized by the use of Profile No. 38.

The results of the deterministic slope stability analysis, shown in Figure 93, indicate that Profile No. 38 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.95, and was located within the upper two-thirds of the bluff slope. The next nine lowest safety factors ranged from 0.97 to 1.07.

The lowest safety factors indicated by the 20 probabilistic stability analyses ranged from 0.45 to 1.03, with 18 of the failure surfaces, or 90 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated, 180, or 90 percent, had safety factors of less than 1.0. Three houses were located within 50 feet of the top edge of the bluff. Based on both the deterministic and probabilistic slope stability analyses and on the observed bluff conditions, Section 31 was considered to have a marginal bluff slope with respect to rotational sliding.

Figure 93

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 38: BLUFF ANALYSIS SECTION 31



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

Overall, Section 31 was also considered to have a marginal bluff slope with respect to translational sliding. A good vegetative growth covered most of the 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 31, no significant bluff toe erosion was observed during the field surveys conducted in the summer of 1986. The toe of the bluff was further protected by a concrete bulkhead which lies above the beach. However, during the study period, beaches were eroding rapidly. Should beach erosion continue or the lake levels remain relatively high, erosion of the bluff could occur, which would increase the potential for slope failure. If the lake levels would return to the mean 20th century levels, the resulting beach within Section 31 would approximate 60 feet in width.

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

Bluff Analysis Section 32

The stability of the bluff slope within Section 32, which extends from 6720 N. Lake Drive to 6818 N. Barnett Lane, was characterized by the use of Profile No. 39 and Profile No. 40.

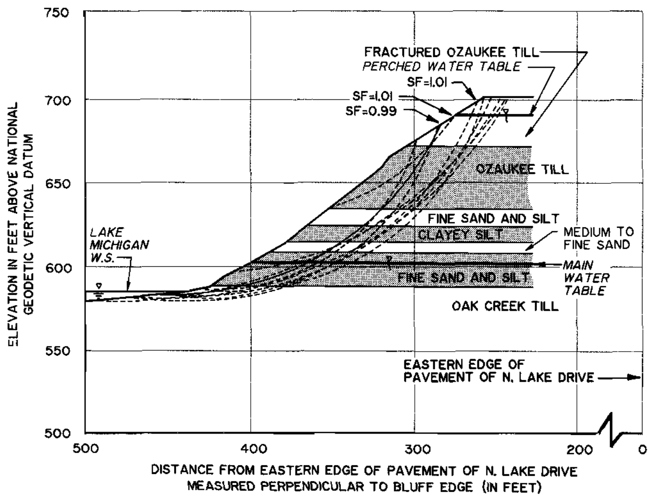
The results of the deterministic slope stability analyses, shown in Figure 94 for Profile No. 39 and Figure 95 for Profile No. 40, indicate a threat of bluff slope failure with respect to rotational sliding. The lowest failure surface calculated at Profile No. 39 had a safety factor of 0.99, and included the entire bluff slope. The next nine lowest safety factors calculated at Profile No. 39 ranged from 1.01 to 1.06. The lowest failure surface calculated at Profile No. 40 had a safety factor of 0.99, and was located on the lower portion of the bluff slope. The next nine lowest safety factors calculated at Profile No. 40 ranged from 0.99 to 1.14.

The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 39 ranged from 0.74 to 1.10, with 14 of the failure surfaces, or 70 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated, 123, or 62 percent, had safety factors of less than 1.0. The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 40 ranged from 0.55 to 1.54, with three failure surfaces, or 15 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated, 22, or 11 percent, had safety factors of less than 1.0. Profile No. 40 was significantly more stable than Profile No. 39 because bedrock was present at the base of the bluff in Profile No. 40. 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 32 was considered to have a marginal bluff slope with respect to rotational sliding.

Section 32 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 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 slope.

Figure 94

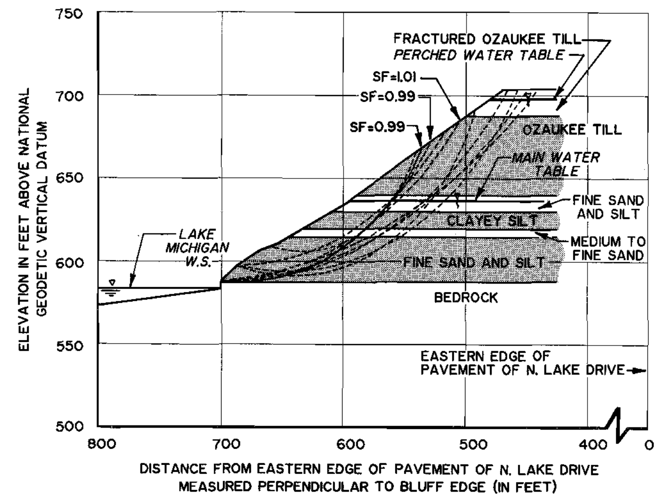
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 39: BLUFF ANALYSIS SECTION 32



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

Figure 95

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 40: BLUFF ANALYSIS SECTION 32



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

Bluff toe erosion was observed along the entire shoreline of Section 32 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. However, in the fall of 1986, grout-filled bags were placed at the base of the bluff along a portion of the shoreline. The bags, which were placed to a height of about 10 feet, are intended to minimize the further erosion of the toe. If the lake levels would return to the mean 20th century levels, the resulting beach within portions of Section 32 would be as wide as 40 feet.

To prevent rotational and translational sliding within Bluff Analysis Section 32, it is recommended that the groundwater level be monitored and that a good vegetative cover be maintained on the bluff slope. The bluff toe protection measures installed in 1986 should be maintained to prevent erosion from wave and ice action.

Bluff Analysis Section 33

The stability of the bluff slope within Section 33, which extends from 6820 to 6840 N. Barnett Lane, was characterized by the use of Profile No. 41.

The results of the deterministic slope stability analysis, shown in Figure 96 for Profile No. 41, indicate a threat of bluff slope failure with respect to rotational sliding. The lowest failure surface evaluated had a safety factor of 0.96, and was located within the lower portion of the bluff slope. The next nine lowest safety factors ranged from 1.01 to 1.21.

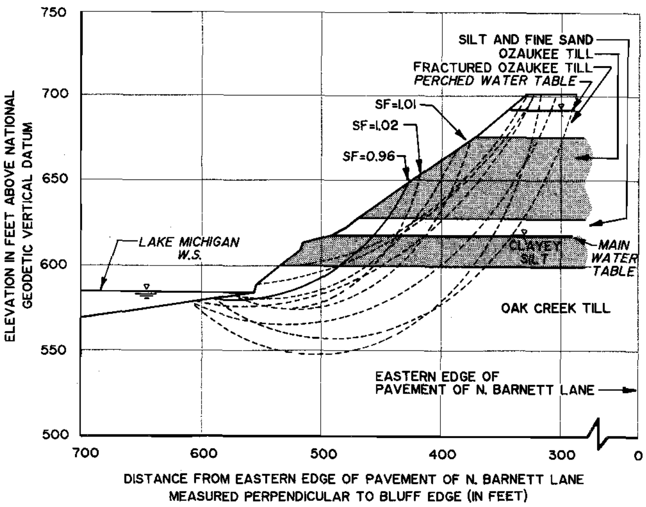
The lowest safety factors indicated by the 20 probabilistic stability analyses ranged from 0.60 to 1.35, with eight of the failure surfaces, or 40 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated, 51, or 26 percent, had safety factors of less than 1.0.

During the field surveys, while the overall bluff slope appeared to be stable, some slumps and shallow slides were observed, especially on the lower portion of the bluff slope. Therefore, based on both the deterministic and probabilistic slope stability analyses, and on the observed bluff conditions, Section 33 was considered to have a marginal bluff slope with respect to rotational sliding.

Overall, Section 33 was also considered to have a marginal bluff slope with respect to translational sliding. A good vegetative growth generally covered most of the bluff face. There were, however, small disturbed soil areas observed on the lower portion of the bluff slope where the potential for translational sliding would be greater.

Figure 96

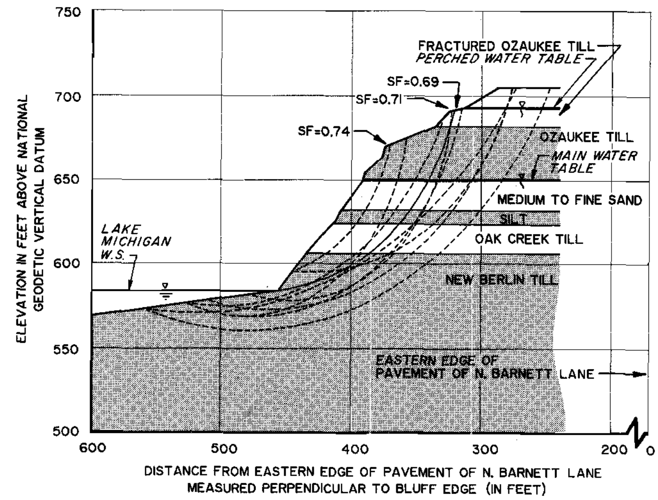
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 41: BLUFF ANALYSIS SECTION 33



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

Figure 97

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 42: BLUFF ANALYSIS SECTION 34



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

Erosion of the toe of the bluff due to wave action was observed. Continued bluff toe erosion within this section would affect the stability of the bluff slope. In the summer of 1986, the toe of the bluff was receiving partial protection from the pilings of an old mining railroad system. If the lake levels would return to the mean 20th century levels, the resulting beach within Section 33 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 33. It is also recommended that a good vegetative cover be maintained on the bluff slope to prevent translational sliding.

Bluff Analysis Section 34

The stability of the bluff slope within Section 34, which extends from 6868 to 7004 N. Barnett Lane, was characterized by the use of Profile No. 42.

The results of the deterministic slope stability analysis, shown in Figure 97, indicate that Profile No. 42 has an unstable bluff slope with respect to rotational sliding. The lowest failure surface calculated at this profile site had a safety factor of 0.69, and was located within the lower portion of the bluff slope. The next nine lowest safety factors ranged from 0.71 to 1.00.

The lowest safety factors indicated by the 20 probabilistic stability analyses ranged from 0.51 to 0.73. Of the 200 failure surfaces evaluated, 196, or 98 percent, had safety factors of less than 1.0. Three houses were located within 50 feet of the top edge of the bluff. Based on both the deterministic and probabilistic slope stability analyses and on the observed bluff conditions, Section 34 was considered to have an unstable bluff slope with respect to rotational sliding.

Section 34 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 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, the resulting beach within Section 34 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 some houses are within 10 feet of the bluff edge. Bluff toe protection is recommended to prevent erosion from wave and ice action.

Bluff Analysis Section 35

The evaluation of Section 35 differs from that for the other analysis sections because it is comprised of a 9,070-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 on Map 16. For the purposes of this analysis, the section was divided into five subsections based on ownership. As shown on Map 16, three of the subsections, which include about 6,750 feet, or 74 percent, of the total shoreline within the section, are in private ownership. The two remaining subsections, containing about 2,320 feet, or 26 percent of the total shoreline, are comprised of public land, the immediate shoreline being owned by the Village of Fox Point.

Analysis Subsection 35A: Subsection 35A extends from 7038 to 7328 N. Beach Drive and includes 2,390 feet, or 26 percent, of the total shoreline within Section 35. All of the shoreline within this subsection is privately owned.

A variety of shoreline protection structures have been installed by private property owners along the shoreline to reduce the erosion of the terrace by wave action. In 1986, approximately 2,150 feet, or 90 percent, of the shoreline was protected by onshore structures, such as bulkheads, revetments, and groins. However, only 840 feet, or 35 percent of this subsection, was protected by structures that had no observable failures or that were not in need of significant maintenance work. About 240 feet of the shoreline, or 10 percent, was not protected by any onshore structures and was eroding. If the lake levels would return to the mean 20th century levels, the resulting beach with Subsection 35A 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 35B: Subsection 35B 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 18 percent, of the total shoreline within the Section 35. The shoreline is owned by the Village of Fox Point.

The terrace within this subsection contained a revetment composed of concrete blocks and rubble. In the summer of 1986, the revetment was being overtopped, allowing erosion to occur behind the structure. This erosion posed a threat to N. Beach Drive, which was located as close as 25 feet from the edge of the terrace. At the southern end of this subsection, at the turnaround point of N. Beach Drive, lies a bulkhead composed of concrete and cut stone slabs. Located at the northern end of this subsection is a concrete groin, extending approximately 140 feet in length, 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, the resulting beach within Subsection 35B would approximate 50 feet in width.

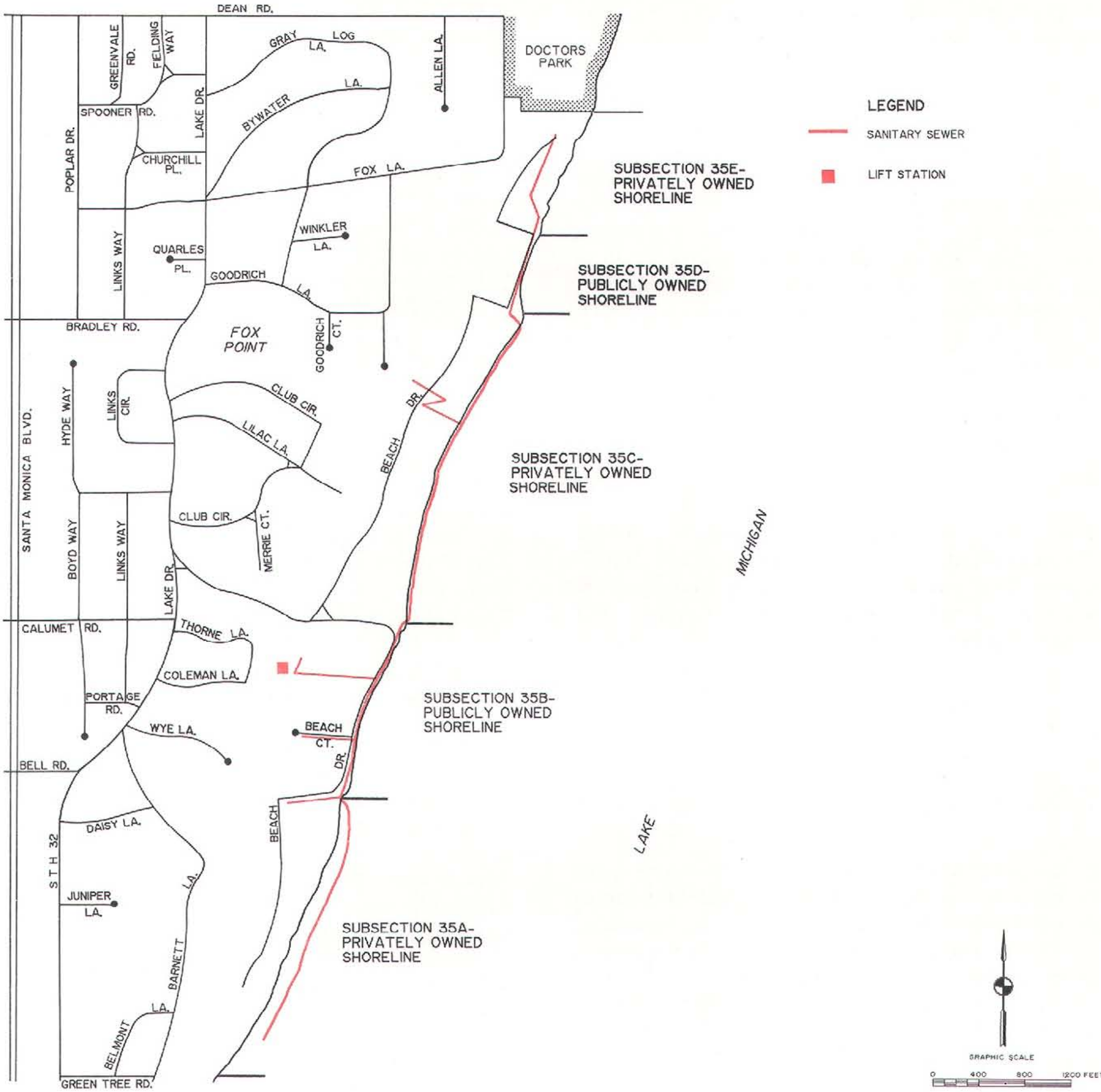
The portion of sanitary sewer within this subsection was located partially within the lake and partially on land immediately adjacent to the lake. The southernmost manhole within this subsection was located one- and one-half feet below lake level, making it vulnerable to damage from wave and ice action.

Analysis Subsection 35C: Subsection 35C extends from 7540 to 7966 N. Beach Drive, and includes 3,000 feet, or 33 percent, of the total shoreline within Section 35. All of the shoreline within this subsection is privately owned.

A variety of shoreline protection structures have been installed by private property owners 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 that had no observable failures or that were not in need of significant maintenance work. About 360 feet of the shoreline, or 12 percent, was not protected

Map 16

BEACH DRIVE SANITARY SEWER AND BLUFF ANALYSIS SECTION 35 SUBSECTIONS: 1986



Source: Donohue & Associates, Inc., and SEWRPC.

by onshore structures and was eroding. If the lake levels would return to the mean 20th century levels, the resulting beach within Subsection 35C would approximate 40 feet in width.

Within Subsection 35C, a beach was present along most of the shoreline in the summer of 1986. The portion of the sanitary sewer 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 35D: Subsection 35D 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 35. The shoreline is owned by the Village of Fox Point.

The terrace within this subsection contained a revetment composed of blocks and concrete rubble. In the field 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, the resulting beach within Subsection 35D would approximate 20 feet in width.

The portion of sanitary sewer within Subsection 35D 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 35E: Subsection 35E extends from 8035 to 8130 N. Beach Drive, and includes 1,360 feet, or 15 percent, of the total shoreline within Section 35. All of the shoreline within this subsection is privately owned.

A variety of shoreline protection structures have been installed by private property owners to reduce the erosion of the terrace by wave action. In 1986 approximately 1,140 feet of the shoreline, or 84 percent, was protected by onshore structures. However, only 240 feet, or 18 percent of this subsection, was protected by structures that had no observable failures or that were not in need of significant maintenance work. About 220 feet, or 16 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, the resulting beach within Subsection 35E would approximate 40 feet in width.

The portion of the sanitary sewer within Subsection 35E 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: It is recommended that adequate shoreline protection be provided along the entire shoreline of Bluff Analysis Section 35. Such protection may require the maintenance of existing shore protection structures, the reconstruction of existing structures, and the construction of new structures. As shown on Map 17, in 1986, 18 percent of the shoreline within the section was protected by structures that did not require maintenance, about 71 percent was protected by structures that were in need of maintenance, and 11 percent 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 36

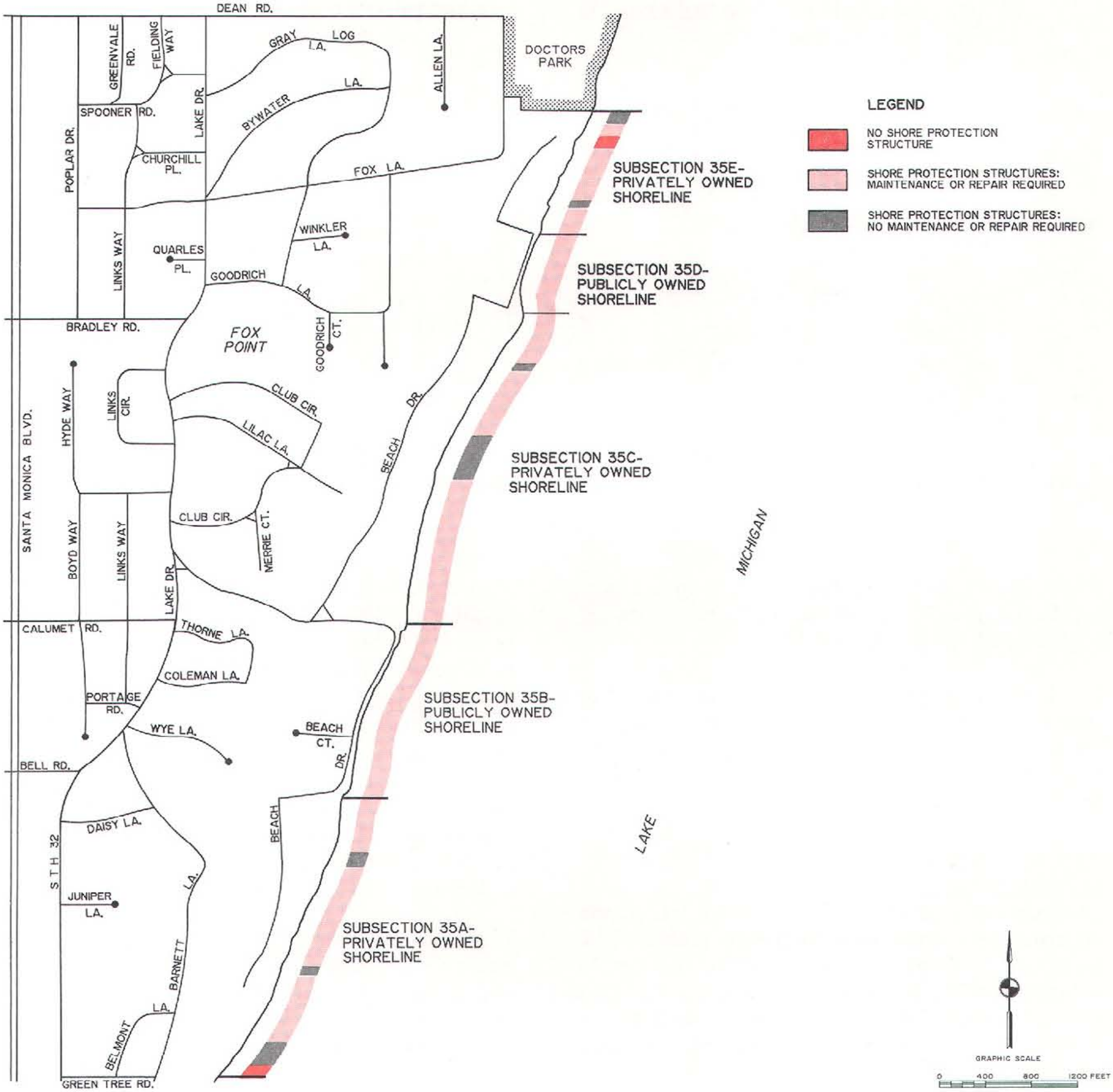
The stability of the bluff slope within Section 36, which is located within Doctors Park, was characterized by the use of Profile No. 43 and Profile No. 44.

The results of the deterministic slope stability analyses, shown in Figure 98 for Profile No. 43 and Figure 99 for Profile No. 44, indicate stable bluff slopes with respect to rotational sliding. The lowest failure surface evaluated at Profile No. 43 had a safety factor of 1.16, and was located within the upper two-thirds of the bluff slope. The next nine lowest safety factors evaluated ranged from 1.18 to 1.24. The lowest failure surface calculated at Profile No. 44 had a safety factor of 1.22, and included the entire bluff slope. The remaining failure surfaces had safety factors ranging from 1.23 to 1.37.

The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 43 ranged from 0.95 to 1.38, with three failure surfaces, or 15 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated, 12, or 6 percent, had safety factors of less than 1.0. The lowest safety factors indicated by the 20 probabilistic stability analyses conducted for Profile No. 44 ranged from 0.79 to 1.42, with three failure surfaces, or 15 percent, having a safety factor of less than 1.0. Of the 200 failure surfaces evaluated, 26, or 13 percent, had safety factors of less than 1.0. Based on both the deterministic and probabilistic slope stability analyses and on the observed bluff conditions, Section 36 was considered to have a stable bluff slope with respect to rotational sliding. However, the probabilistic analysis indicated that under certain conditions, there may be expected to be a slight risk of slope failure.

Map 17

BLUFF ANALYSIS SECTION 35 SHORELINE PROTECTION: 1986



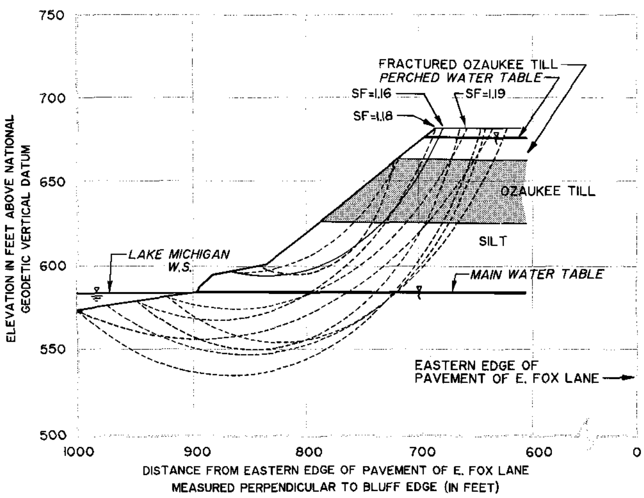
Source: SEWRPC.

Section 36 was also considered to have a stable bluff slope with respect to translational sliding. This was due to the gentle angle of the bluff slope, and to the good vegetative growth covering the entire bluff face.

Within Section 36, the bluff slope was protected by a concrete bulkhead. However, there was erosion of the bluff toe from waves washing over the top of the bulkhead. This erosion was not affecting the overall stability of the bluff slope. If the lake

Figure 98

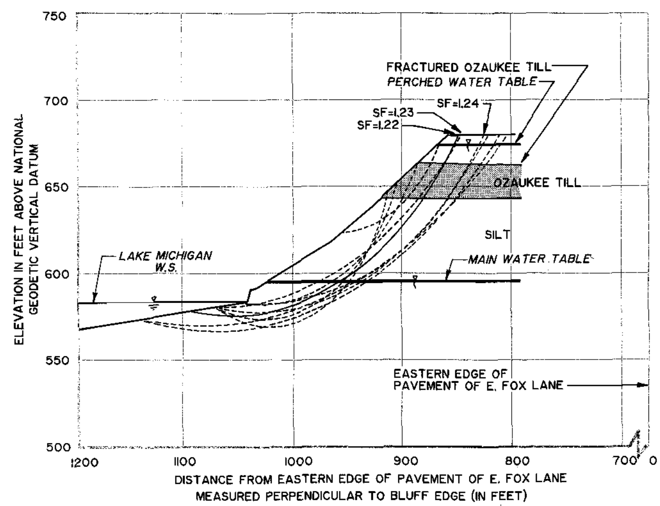
DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 43: BLUFF ANALYSIS SECTION 36



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

Figure 99

DETERMINISTIC BLUFF SLOPE STABILITY ANALYSIS FOR PROFILE 44: BLUFF ANALYSIS SECTION 36



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

levels would return to the mean 20th century levels, the resulting beach within Section 36 would approximate 60 feet in width.

No measures are needed to prevent rotational or translational sliding within Bluff Analysis Section 36. Toe protection in addition to the existing concrete bulkhead is recommended to prevent wave overtopping.

Summary of the Evaluation of Bluff Analysis Sections

The analyses of each of the 36 bluff analysis sections were conducted to better quantify the risks of bluff slope failure with respect to rotational sliding and translational sliding, and of bluff toe erosion. A summary of the deterministic and probabilistic slope stability analysis results for rotational sliding for each profile site is set forth in Table 28. The evaluations of the bluff conditions as of the summer of 1986 are presented in Table 29, and shown on Map 18. 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, 13 bluff analysis sections, which cover 14,540 feet of shoreline, or 38 percent of the total study area shoreline, were found to have stable bluff slopes. A total of nine bluff analysis sections, which cover 8,260 feet of shoreline, or 21 percent of the total study area shoreline, were found to have marginal bluff slopes. A total of 13 bluff analysis sections, which cover 6,900 feet of shoreline, or 18 percent of the total study area shoreline, were found to have unstable slopes. Bluff slope stability was not evaluated for Section 35, consisting of the Fox Point terrace, which includes the remaining 9,070 feet, or 23 percent, of the total study area shoreline.

With respect to translational sliding, 13 bluff analysis sections, which cover 15,450 feet of shoreline, or 40 percent of the total study area shoreline, were considered to have stable bluff slopes. Twelve bluff analysis sections, covering 8,750 feet of shoreline, or 23 percent of the total study area shoreline, were considered to have marginal bluff slopes. Ten bluff analysis sections, covering 5,500 feet of shoreline, or 14 percent of the total study area shoreline, were considered to have unstable bluff slopes.

Seven bluff analysis sections, covering about 7,760 feet of shoreline, or 20 percent of the total study area shoreline, were exhibiting insignificant or slight bluff toe erosion in 1986. The remaining 30 bluff analysis sections, covering

Table 28

SUMMARY OF DETERMINISTIC AND PROBABILISTIC SLOPE STABILITY ANALYSIS RESULTS FOR ROTATIONAL SLIDING

Civil Division	Bluff Analysis Section	Profile Number	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	Model-Indicated Stability Classification of Section
City of Milwaukee	1	1	1.46	0	0.98-1.60	5	1	Stable
	2	2	2.97	0	2.01-2.89	0	0	Stable
Village of Shorewood	3	3	0.98	10	0.62-1.08	65	32	Marginal
		4	0.98	10	0.81-1.15	55	15	- -
	4	5	2.13	0	- -	- -	- -	Stable (fill)
	5	6	1.12	0	0.86-1.23	20	8	Stable
	6	7	1.54	0	- -	- -	- -	Stable (fill)
	7	8	0.81	100	0.51-0.90	100	96	Unstable
	8	9	0.99	10	0.66-1.17	60	45	Marginal
	9	10	1.79	0	0.74-1.99	5	2	Stable
	10	11	0.68	100	0.61-0.97	100	80	Marginal
	11	12	0.72	20	- -	- -	- -	Stable (fill)
Village of Whitefish Bay		13	1.44	0	- -	- -	- -	- -
		14	2.11	0	- -	- -	- -	- -
	12	15	0.64	100	0.50-0.97	100	92	Unstable
		16	0.66	100	0.52-0.81	100	93	- -
	13	17	0.61	100	- -	- -	- -	Unstable (fill)
	14	18	0.80	100	0.55-0.82	100	100	Unstable
	15	19	0.90	100	- -	- -	- -	Unstable (fill)
	16	20	0.73	100	0.53-0.83	100	95	Unstable
	17	21	1.06	0	- -	- -	- -	Stable (fill)
		22	1.51	0	- -	- -	- -	- -
	18	23	0.91	30	0.54-1.06	70	78	Marginal
	19	24	1.39	0	- -	- -	- -	Stable (fill)
	20	25	1.07	0	0.76-1.44	25	13	Marginal
	21	26	1.69	0	- -	- -	- -	Stable (fill)
		27	1.75	0	- -	- -	- -	- -
	22	28	0.95	100	0.47-1.12	85	80	Unstable
	23	29	1.14	0	- -	- -	- -	Stable (fill)
	24	30	0.96	70	0.54-1.06	90	64	Unstable
	25	31	0.65	20	0.53-1.13	70	35	Unstable
	26	32	0.70	100	0.52-1.10	90	82	Unstable
	27	33	0.91	50	0.62-1.60	65	48	Marginal
	28	34	0.82	100	0.63-1.00	95	68	Unstable
Village of Fox Point		35	0.86	90	0.51-1.03	90	84	- -
	29	36	1.10	0	- -	- -	- -	Stable (fill)
	30	37	0.91	100	0.41-0.90	100	100	Unstable
	31	38	0.95	40	0.45-1.03	90	90	Marginal
	32	39	0.99	10	0.74-1.10	70	62	Marginal
		40	0.99	20	0.55-1.54	15	11	- -
	33	41	0.96	10	0.60-1.35	40	26	Marginal
	34	42	0.69	90	0.51-0.73	100	98	Unstable
	36	43	1.16	0	0.95-1.38	15	6	Stable
		44	1.22	0	0.79-1.42	15	13	- -

NOTE: The evaluation 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.

Table 29

SUMMARY OF EVALUATIONS OF BLUFF CONDITIONS: SUMMER 1986

Civil Division	Bluff Analysis Section	Profile Number	Shoreline Length (feet)	Potential for Rotational Sliding	Potential for Translational Sliding	Existing Bluff Toe Erosion[a]
City of Milwaukee	1	1	1,970	Stable	Marginal	I Slight
	2	2	950	Stable	Marginal	II Moderate
Village of Shorewood	3	3, 4	300	Marginal	Marginal	III Severe
	4	5	290	Stable (fill)	Stable	I Slight
	5	6	1,710	Stable	Stable	II Moderate
	6	7	170	Stable (fill)	Stable	II Moderate
	7	8	380	Unstable	Unstable	III Severe
	8	9	2,170	Marginal	Stable	I Slight
	9	10	520	Stable	Stable	III Severe
	10	11	240	Marginal	Marginal	III Severe
	11	12-14	2,370	Stable (fill)	Stable	I Slight
Village of Whitefish Bay	12	15, 16	850	Unstable	Unstable	III Severe
	13	17	190	Unstable (fill)	Unstable	III Severe
	14	18	160	Unstable	Unstable	III Severe
	15	19	310	Unstable (fill)	Unstable	III Severe
	16	20	360	Unstable	Unstable	III Severe
	17	21, 22	810	Stable (fill)	Stable	II Moderate
	18	23	1,660	Marginal	Stable	III Severe
	19	24	1,480	Stable (fill)	Stable	II Moderate
	20	25	130	Marginal	Marginal	I Slight
	21	26, 27	2,970	Stable (fill)	Stable	II Moderate
	22	28	490	Unstable	Marginal	II Moderate
	23	29	140	Stable (fill)	Stable	II Moderate
	24	30	430	Unstable	Marginal	III Severe
	25	31	480	Unstable	Marginal	III Severe
	26	32	170	Unstable	Unstable	II Moderate
	27	33	1,950	Marginal	Marginal	II Moderate
	28	34, 35	1,150	Unstable	Unstable	III Severe
Village of Fox Point	29	36	320	Stable (fill)	Stable	I Slight
	30	37	470	Unstable	Unstable	III Severe
	31	38	510	Marginal	Marginal	I Slight
	32	39, 40	770	Marginal	Marginal	III Severe
	33	41	530	Marginal	Marginal	III Severe
	34	42	1,460	Unstable	Unstable	III Severe
	35	- -[b]	9,070	- -[b]	- -[b]	II Moderate
	36	43, 44	840	Stable	Stable	II Moderate

NOTE: The elevation of the stability of a bluff slope at individual lakeshore properties requires a site-specific analysis by a professional geologist or geotechnical engineer.

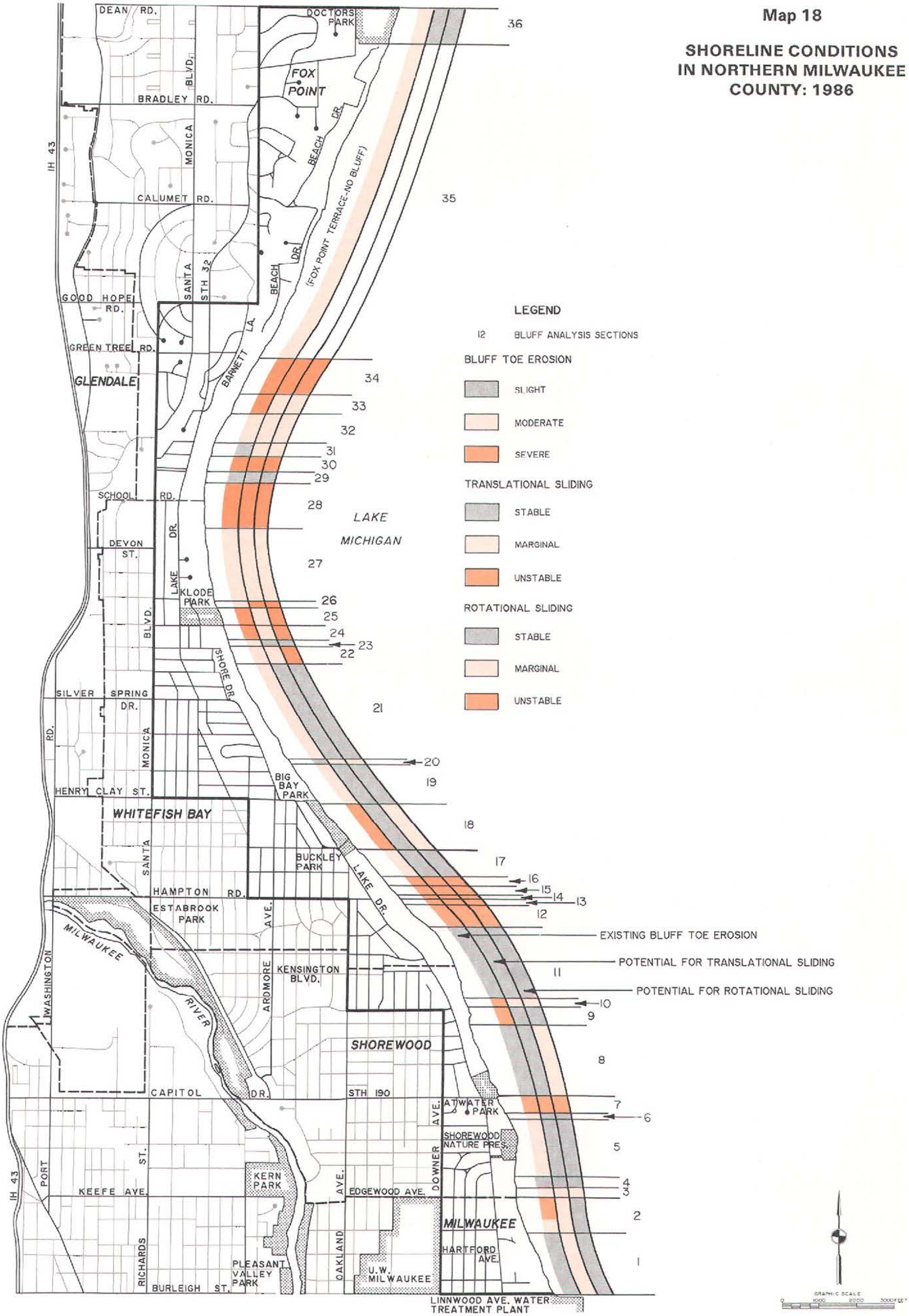
[a]*Category I includes minor, or slight, toe erosion. Category II, defined as moderate toe erosion, includes substantial erosion of the toe of the bluff which may not be expected to affect the stability of the bluff slope. Category III, defined as severe toe erosion, includes toe erosion which may be expected to affect the stability of the slope.*

[b]*A bluff slope stability analysis was not conducted for Bluff Analysis Section 35, which is a terrace.*

Source: SEWRPC.

Map 18

SHORELINE CONDITIONS IN NORTHERN MILWAUKEE COUNTY: 1986



Source: SEWRPC.

31,010 feet, or 80 percent of the total study area shoreline, were exhibiting substantial erosion of the bluff toe. The erosion occurring within 17 bluff analysis sections, covering 10,260 feet, or 33 percent of the eroding shoreline, is considered to be affecting the overall stability of the bluff slopes.

Based on the conditions of the bluff slopes in the summer of 1986, measures to protect the shoreline and stabilize the slopes were identified for each of the 36 bluff analysis sections. The types of shore protection measures indicated are listed in Table 30 and shown on Map 19. The identified measures include regrading the bluff slope to a stable angle; installing a groundwater drainage system to lower the elevation of the groundwater; constructing surface water runoff control measures; revegetating the bluff slopes; and protecting 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 31.

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 all or portions of 18 bluff analysis sections, which include about 10,420 feet, or 27 percent, of the study area shoreline. Groundwater drainage systems were indicated for all or portions of six bluff analysis sections covering about 6,160 feet, or 16 percent, of the shoreline. Detailed studies of the groundwater systems should be conducted within these six sections to determine the feasibility of lowering the elevation of the groundwater. If these studies indicate that drainage of the groundwater would not be feasible, regrading of at least a portion of the bluff slope would probably be necessary. Control of surface water runoff was indicated for three bluff analysis sections, which cover about 1,540 feet, or 4 percent, of the shoreline. Revegetation of at least a portion of the bluff face was indicated for 10 bluff analysis sections covering about 8,000 feet, or 21 percent, of the shoreline. Protection of the toe of the bluff against wave and ice action was indicated for all or portions of 34 bluff analysis sections, which have a combined shoreline of about 37,470 feet, or 97 percent of the total study area shoreline.

EVALUATION OF POTENTIAL 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 potential loss of land and buildings at the top of the bluffs and the Fox Point terrace. The historical bluff recession rates presented in Chapter II were not used to estimate future bluff recession because the recession rates along most shoreline areas were too low to be precisely measured over a 22-year period of record. 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 marginal or unstable—as well as the Fox Point terrace—would recede. In order to determine the extent and economic value of the land and buildings subject to a risk of erosion damage, those areas lying within 10 feet, and within 25 feet, of the edge of existing marginal or unstable bluff slopes or the Fox Point terrace were delineated. 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 northern 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 northern Milwaukee County communities. Table 32 sets forth for each local unit of government the area within the entire study area; the area directly adjacent to the Lake Michigan shoreline, which is comprised of those properties abutting the lake; and the area potentially subject to shoreline erosion—that is, lying within marginal or unstable bluff analysis sections and directly adjacent to the shoreline. As shown in the table, although 19 to 51 percent of the Villages of Shorewood, Whitefish Bay, and Fox Point lie within the study area, only 6 to 11 percent of the land within those Villages lies directly adjacent to Lake Michigan, and only 2 to 7 percent of the land within those Villages may be subject to shoreline erosion. About 0.2 percent of the City of Milwaukee lies within the study area, less

Table 30

INDICATED SHORE PROTECTION MEASURES TO CONTROL SHORELINE EROSION AND STABILIZE THE BLUFF SLOPES: SUMMER 1986

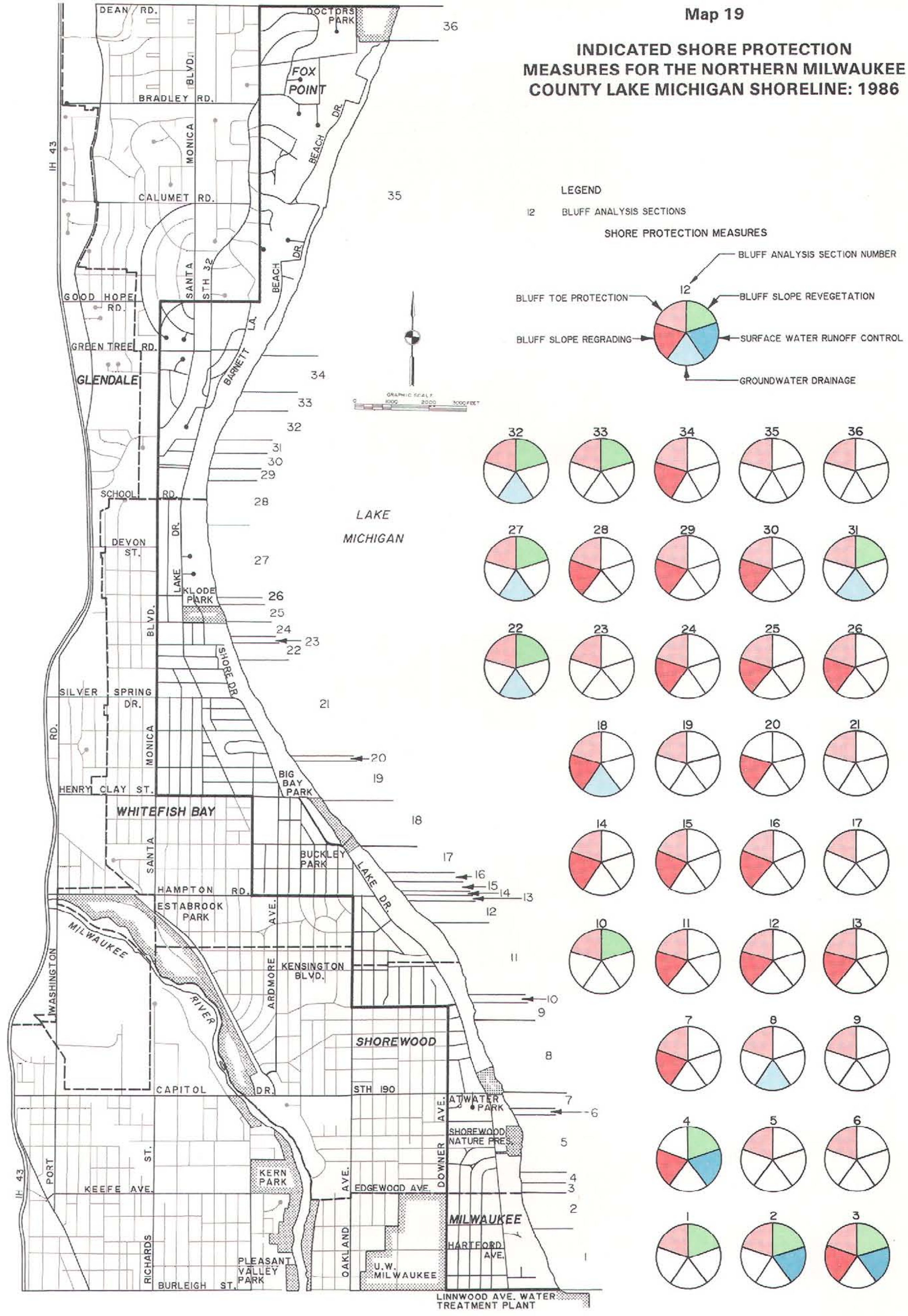
Bluff Analysis Section	Bluff Toe Protection	Bluff Slope Revegetation	Surface Water Runoff Control	Groundwater Drainage	Bluff Slope Regrading
1	X	X	--	--	--
2	X	X	X	--	--
3	X	X	X	--	X
4	--	X	X	--	X
5	X	--	--	--	--
6	X	--	--	--	--
7	X	--	--	--	X
8	X	--	--	X	--
9	X	--	--	--	--
10	X	X	--	--	--
11	X	--	--	--	X
12	X	--	--	--	X
13	X	--	--	--	X
14	X	--	--	--	X
15	X	--	--	--	X
16	X	--	--	--	X
17	X	--	--	--	--
18	X	--	--	X	X
19	X	--	--	--	--
20	--	--	--	--	X
21	X	--	--	--	--
22	X	X	--	X	--
23	X	--	--	--	--
24	X	--	--	--	X
25	X	--	--	--	X
26	X	--	--	--	X
27	X	X	--	X	--
28	X	--	--	--	X
29	X	--	--	--	X
30	X	--	--	--	X
31	X	X	--	X	--
32	X	X	--	X	--
33	X	X	--	--	--
34	X	--	--	--	X
35	X	--	--	--	--
36	X	--	--	--	--

NOTE: The selection of measures needed to control shoreline erosion and stabilize the slope, and the detailed design of the selected measures, requires a site-specific analysis by a professional geotechnical or coastal engineer.

Source: SEWRPC.

Map 19

INDICATED SHORE PROTECTION MEASURES FOR THE NORTHERN MILWAUKEE COUNTY LAKE MICHIGAN SHORELINE: 1986



Source: SEWRPC.

Table 31

EXTENT OF INDICATED SHORE PROTECTION MEASURES IN NORTHERN MILWAUKEE COUNTY: 1986

Shore Protection Measure	City of Milwaukee		Village of Shorewood		Village of Whitefish Bay		Village of Fox Point		Total Study Area	
	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
Bluff Toe Protection	2,040	70	6,300	96	14,550	99	14,580	100	37,470	97
Bluff Slope Revegetation.	2,920	100	830	13	2,440	17	1,810	12	8,000	21
Surface Water Runoff Control.	950	33	590	9	0	0	0	0	1,540	4
Groundwater Drainage	0	0	1,380	21	3,500	24	1,280	9	6,160	16
Bluff Slope Regrading	0	0	1,780	26	5,780	39	2,860	20	10,420	27

Source: SEWRPC.

Table 32

AREAL EXTENT OF STUDY AREA, AREA DIRECTLY ADJACENT TO THE LAKE MICHIGAN SHORELINE, AND AREA POTENTIALLY SUBJECT TO SHORELINE EROSION WITHIN EACH CIVIL DIVISION: 1986

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
Village of Fox Point.	1,843	672	36.5	121	6.6	103	5.6
Village of Shorewood.	1,088	211	19.4	69	6.3	19	1.7
Village of Whitefish Bay	1,363	701	51.4	146	10.7	97	7.1
City of Milwaukee	61,840	141	0.2	42	<0.1	12	<0.1
Total	66,134	1,725	- -	378	- -	231	- -

[a]*The area potentially subject to shoreline erosion is defined as that land lying within a marginal or unstable bluff analysis section—including the Fox Point terrace—and directly adjacent to the shoreline.*

Source: SEWRPC.

than 0.1 percent lies 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 219 acres, or 95 percent, are privately owned, while the remaining 12 acres, or 5 percent, are 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 areas 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 that were determined to have marginal or unstable bluff slopes in the slope stability analyses. Approximately 15,160 feet, or 39 percent, of the study area shoreline was found to be within the marginal or unstable bluff analysis sections. Potential erosion hazard areas were also delineated for the 9,070 feet, or 23 percent of the shoreline, located within Bluff Analysis Section 35, which includes the Fox Point terrace. The land and facilities lying within 25 feet of the edge of the existing bluff or terrace were considered to be at some risk of erosion damage. The land and facilities lying within 10 feet of the edge would have the greatest risk of erosion damage.

Loss of land and facilities may result from continued shoreline erosion and the parallel retreat of the bluff, or from additional slope failure. It cannot be assumed that the bluff face will remain at its existing angle, and the potential exists for the bluff slope to rapidly, and perhaps catastrophically, recede to a more gentle, and stable, slope angle. The existing bluffs within northern Milwaukee County could recede to a slope angle as gentle as one on two and one-half, or about 22 degrees, although some 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.[9] Another report by Vallejo and Edil[10] 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 limited time periods; and

3. Groundwater conditions can change significantly as the bluff recedes and strata of permeable bluff materials are eroded, covered, or disturbed.

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 the 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 33, approximately 5.4 acres (234,400 feet^2) of land, or about 2 percent of the land directly adjacent to Lake Michigan and potentially subject to shoreline erosion, lies within 10 feet of the edge of a marginal or unstable bluff or terrace. Of this total area, about 3.1 acres, or 57 percent, is located within the Village of Fox Point; about 1.7 acres, or 32 percent, is located within the Village of Whitefish Bay; and about 0.6 acre, or 11 percent, is located within the Village of Shorewood. There were no marginal or unstable bluff analysis sections located within the City of Milwaukee. Privately owned land comprises about 4.2 acres, or 78 percent, of this land. Publicly owned land comprises the remaining 1.2 acre, or 22 percent. A total of 23 buildings lie, in whole or in part, within 10 feet of the edge.

Approximately 13.4 acres (586,000 feet^2) of land, or about 6 percent of the land directly adjacent to Lake Michigan and potentially subject to shoreline erosion, lies within 25 feet of the edge of a marginal or unstable bluff or terrace. Of this total area, about 7.8 acres, or 58 percent, is located within the Village of Fox Point; about 4.1

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

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

Table 33

EXTENT OF LAND AND FACILITIES LYING WITHIN 10 FEET, AND WITHIN 25 FEET, OF THE EDGE OF AN EXISTING MARGINAL OR UNSTABLE BLUFF OR TERRACE: 1986

Civil Division	Marginal or Unstable Bluff Analysis Section	Shoreline Length (feet)	Extent of Land and Number of Residential Buildings Near Edge of Existing Marginal or Unstable Bluffs			
			Within 10 Feet		Within 25 Feet	
			Area (feet^2)	Residential Buildings (number)	Area (feet^2)	Residential Buildings (number)
Village of Shorewood	3	300	3,000	0	7,500	0
	7	380	3,800	2	9,500	3
	8[a]	1,380	13,800	6	34,500	12
	10	240	2,400	2	6,000	2
Subtotal	- -	2,300	23,000	10	57,500	17
Village of Whitefish Bay	12	850	8,500	0	21,250	1
	13	190	1,900	0	4,750	0
	14	160	1,600	0	4,000	0
	15	310	3,100	1	7,750	1
	16	360	3,600	0	9,000	0
	18	1,660	16,600	0	41,500	0
	20	130	1,300	0	3,250	0
	22	490	4,900	0	12,250	0
	24	430	4,300	1	10,750	2
	25	480	4,800	0	12,000	0
	26	170	1,700	0	4,250	0
	27	1,950	19,500	2	48,750	5
	28	540	5,400	0	13,500	0
Subtotal	- -	7,720	77,200	4	193,000	9
Village of Fox Point	28	610	6,100	0	15,250	0
	30	470	4,700	3	11,750	3
	31	510	5,100	2	12,750	2
	32	770	7,700	1	19,250	2
	33	530	5,300	0	13,250	0
	34	1,460	14,600	2	36,500	3
	35	9,070	90,700	1	226,750	4
Subtotal	- -	13,420	134,200	9	335,500	14
Total	- -	23,440	234,400	23	586,000	40

[a] *Includes only the portion of Section 8 north of Atwater Park, which was determined to have a marginal bluff slope.*

Source: SEWRPC.

acres, or 31 percent, is located within the Village of Whitefish Bay; and about 1.5 acres, or 11 percent, is located within the Village of Shorewood. Privately owned land comprises a total of 10.5 acres, or 78 percent, of this land, while publicly owned land comprises the remaining 2.9 acres, or 22 percent. A total of 40 buildings lie, in whole or in part, within 25 feet of the edge.

Potential Economic Loss

As noted above, the northern Milwaukee County shoreline provides a unique setting for high-value residential development. Approximately 274 residential properties lie directly adjacent to the Lake Michigan shoreline within the northern Milwaukee County study area. Table 34 sets forth the 1986 average economic value of the properties directly adjacent to the Lake Michigan shoreline which lie within the boundaries of each local unit of government. As shown in Table 34, the average total lakefront property values within the North Shore communities ranged from $222,400 in the Village of Fox Point to $296,200 in the City of Milwaukee, with an

Table 34

AVERAGE ECONOMIC VALUE OF PROPERTIES DIRECTLY ADJACENT TO THE LAKE MICHIGAN SHORELINE IN NORTHERN MILWAUKEE COUNTY: 1986

Civil Division	Average Land Value	Percent of Total Value	Average Improvement Value	Percent of Total Value	Average Total Economic Value
City of Milwaukee	$70,800	24	$225,400	76	$296,200
Village of Fox Point	93,300	42	129,100	58	222,400
Village of Shorewood	57,200	22	202,200	78	259,400
Village of Whitefish Bay	92,300	37	157,800	63	250,100
Total	$78,400	30	$178,600	70	$257,000

Source: Real Estate Data, Inc., and SEWRPC.

overall average total property value of $257,000. The improvement value accounted for approximately 58 to 78 percent of the total property values, while the land value comprised the remaining 22 to 42 percent of the total. These lakeshore properties account for approximately 2 percent of the total tax base within the Villages of Fox Point, Shorewood, and Whitefish Bay.

The potential economic losses resulting from continued bluff recession were estimated by determining the economic value of the land and facilities located within 10 and 25 feet of the edge of any marginal or unstable bluff or terrace. The value of land and facilities in those areas was estimated based upon the 1986 values presented in the Milwaukee County statistical report of property valuations prepared by Real Estate Data, Inc. Table 35 and Map 20 set forth the approximate economic value of the land and facilities contained within 10 feet, and within 25 feet, of the edge of a marginal or unstable bluff or terrace for each civil division in the study area. The economic values presented in the table do not include the value of public utilities and improvements such as streets and sewers.

The total economic value of land and residential buildings lying within 10 feet of the edge of marginal or unstable bluffs or terraces is approximately $3.8 million, of which about $0.4 million, or 11 percent, represents the value of the land, and about $3.4 million, or 89 percent, represents the value of the buildings. Of the total value, $1.3 million, or 34 percent, represents the potential loss to the Village of Fox Point; about $0.8 million, or 21 percent, represents the potential loss to the Village of Whitefish Bay; and $1.7 million, or 45 percent, represents the potential loss to the Village of Shorewood.

The total economic value of land and residential buildings lying within 25 feet of the edge of marginal or unstable bluffs or terraces is approximately $6.9 million, of which about $1.1 million, or 15 percent, represents the value of the land, and about $5.8 million, or 85 percent, represents the value of the buildings. Of the total value, $2.3 million, or 33 percent, represents the potential loss to the Village of Fox Point; $1.8 million, or 26 percent, represents the potential loss to the Village of Whitefish Bay; and $2.8 million, or 41 percent, represents the potential loss to the Village of Shorewood.

A complete analysis of the economic impact of shoreline erosion must consider the effects on the market value of lakefront property. Since shoreline erosion and bluff recession within the North Shore communities are not recent phenomena, the damages that could result from erosion are generally understood by the sellers and the real estate brokers who represent them. Since Wisconsin law prohibits the omission of

Table 35

ECONOMIC VALUE OF LAND AND RESIDENTIAL BUILDINGS LYING WITHIN 10 FEET, AND WITHIN 25 FEET, OF THE EDGE OF MARGINAL OR UNSTABLE BLUFFS OR TERRACES WITHIN THE LAKE MICHIGAN SHORELINE OF NORTHERN MILWAUKEE COUNTY: 1986

Civil Division	Land			Residential Buildings			Total
	Extent (acres)	Economic Value[a]	Percent of Total Economic Value	Number	Economic Value[a]	Percent of Total Economic Value	Economic Value[a]
Within 10 Feet							
Village of Fox Point	3.1	$ 242,400	19	9	$1,049,100	81	$1,291,500
Village of Whitefish Bay	1.8	147,600	19	4	633,600	81	780,900
Village of Shorewood	0.5	35,400	2	10	1,668,100	98	1,703,500
Total	5.4	$ 425,400	11	23	$3,350,800	89	$3,775,900
Within 25 Feet							
Village of Fox Point	7.7	$ 602,100	26	14	$1,677,000	74	$2,279,100
Village of Whitefish Bay	4.4	360,800	20	9	1,434,200	80	1,795,000
Village of Shorewood	1.3	92,000	3	17	2,710,700	97	2,802,700
Total	13.4	$1,054,900	15	40	$5,821,900	85	$6,876,800

[a]*Economic values are in 1986 dollars.*

Source: SEWRPC.

pertinent facts by sellers and real estate brokers in the sale of real estate, buyers are also being made aware of the potential erosion problems associated with lakefront property. The effect of shoreline erosion on the number of lakefront properties sold, and on the actual sales prices, was evaluated for the Village of Shorewood.[11]

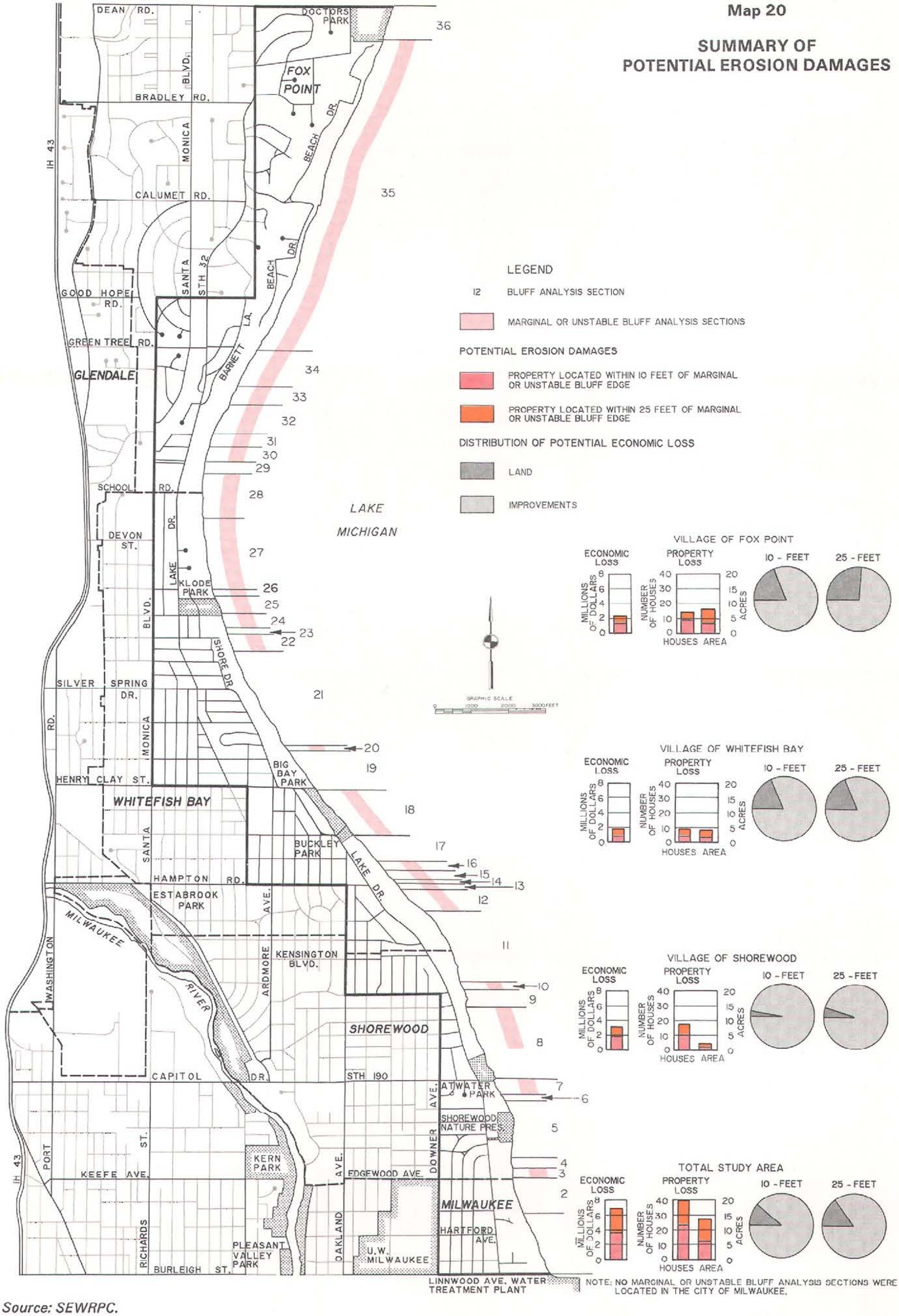
[11]*J. R. Wronski, Assistant Assessor, Shorewood Memorandum on information relative to the impact of lakeshore erosion on property values. Prepared for Lucia Petrie, Village Trustee. January 5, 1987.*

The evaluation indicated that shoreline erosion has not significantly restricted the number of lakefront properties sold. In the Village of Shorewood, which contains 63 lakefront properties, 25 sales occurred over the period 1981 through 1986, with an increased trend in sales over the latter years.

The evaluation also suggested that recent shoreline erosion has not had an adverse impact on the sales prices of most lakefront property. The analysis of the sales of lakefront property occurring between 1981 and 1986 in the Village of Shorewood indicated an increasing trend in

Map 20

SUMMARY OF POTENTIAL EROSION DAMAGES



Source: SEWRPC.

sales prices. There was one case, however, in which a major bluff failure which posed a threat to the security of the building had an adverse impact on the market value of the property. Although the properties along the lakefront are threatened by shoreline erosion in varying degrees, the adverse impacts on property values appear to be significant only when such erosion poses a serious threat to the foundation of the building. However, with the rise in the level of Lake Michigan that occurred in 1986, and the associated publicity and public concern over shoreline erosion, the market values of lakefront property could be impacted to a greater degree in the future.

SUMMARY

This chapter evaluates the shoreline erosion and bluff recession occurring within the study area, identifies those factors causing the erosion and bluff recession, identifies the types of control measures needed to abate shoreline erosion and bluff recession, and summarizes the property and economic losses that may be experienced if shoreline protection is not provided. The identification of the shoreland areas that 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 coastal erosion problems within each of 36 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, 38 percent of the total study area shoreline was determined to have stable bluff slopes, 21 percent of the shoreline was determined to have marginal bluff slopes, and 18 percent was determined to have unstable bluff slopes. Bluff slope stability was not evaluated for the remaining 23 percent of the shoreline, consisting of the Fox Point terrace.

With respect to translational sliding, 40 percent of the total study area shoreline was determined to have stable slopes, 23 percent of the shoreline was determined to have marginal bluff slopes, and 14 percent was determined to have unstable bluff slopes.

With respect to bluff toe erosion, 20 percent of the total study area shoreline was observed to have little or no evidence of toe erosion in 1986. About 47 percent of the shoreline was experiencing erosion at the toe of the bluff that did not appear to affect the overall stability of the bluff slope. The remaining 33 percent of the shoreline was experiencing toe erosion which was threatening the overall stability of the bluff slope.

The shore protection needs of each of the bluff analysis sections within the study area were identified based on the bluff slope conditions observed in the summer of 1986. It was indicated that the bluff slopes within about 27 percent of the study area shoreline should be regraded to a stable slope angle; that groundwater drainage systems should be installed to lower the elevation of the groundwater along about 16 percent of the shoreline; that surface water runoff control measures should be implemented along about 4 percent of the shoreline; that additional toe protection should be provided to about 97 percent of the shoreline; and that the bluff slope should be revegetated along about 21 percent of the shoreline. It is important to note that no bluff analysis sections were found to be fully protected in 1986, requiring no maintenance or corrective actions.

The land area lying within 10 feet, and within 25 feet, of the edge of a marginal or unstable bluff or terrace was delineated on large-scale topographic maps. The area lying within 10 feet of the edge of the marginal or unstable bluffs and terraces includes about 5.4 acres of land, or about 2 percent of the study area directly adjacent to Lake Michigan and potentially subject to shoreline erosion, and 23 residential buildings. About 4.2 acres, or 78 percent of the land within 10 feet of the edge, were privately owned, while the remaining 1.2 acre, or 22 percent, was in public ownership. About 13.4 acres of land, or about 6 percent of the study area directly adjacent to Lake Michigan and potentially subject to shoreline erosion, and 40 residential buildings lie within 25 feet of the edge of the marginal or unstable bluffs and terraces. About 10.5 acres, or 78 percent of the land within 25 feet of the edge, were privately owned, while the remaining 2.9 acres, or 22 percent, were publicly owned.

The economic value of the land and buildings located within 10 feet of the edge was approximately \$3.8 million. The economic value of the land and buildings located within 25 feet of the edge was approximately \$6.9 million. These economic values do not include the value of public utilities and improvements such as streets and sewers. The areas identified as subject to potential erosion damages would be protected if adequate bluff toe protection and slope stabilization measures were implemented.

(This page intentionally left blank)

Chapter IV

ALTERNATIVE SHORELINE EROSION CONTROL MEASURES AND A RECOMMENDED SHORELINE EROSION MANAGEMENT PLAN

INTRODUCTION

Alternative measures to protect the shoreline and stabilize the bluff slopes within northern Milwaukee County were identified under the study to resolve the erosion 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 comprehensive shoreline erosion management plan for northern 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 northern Milwaukee County 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 analytical procedures used in the development and evaluation of the alternative control measures. The second section describes the conceptual measures that could be utilized within the study area. The third section describes 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 comprehensive shoreline management plan for northern 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 management 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 comprehensive shoreline management 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, and environmental 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. The preliminary engineering phase of project implementation should concentrate on the solution identified in the recommended system plan, and should involve the collection and analysis of more detailed geotechnical and coastal engineering data as described in Table 36. The preliminary engineering phase, using more detailed site-specific data, should either reaffirm or revise the solution set forth in the recommended plan, and determine the best way to carry out the recommended solution.

The third phase, or final design, should be carried out by the implementing units of government and private property owners. The final design phase consists of the development of construction plans and specifications needed to completely implement the needed shore protection measures. The final design should include layout drawings, construction details, materials specifications, a schedule for construction, and access arrangements. The final design plan should also include the existing and proposed profile of the bluff slope, the quantity of materials to be used, material placement instructions, and an inspection and quality assurance program to ensure compliance with plans.

For many reasons, the planning process for shore protection often does not proceed in the simple three-step process described above. In some cases, an iterative process occurs whereby a reexamination of an earlier phase is required. This frequently occurs where additional data are collected and analyzed. Changes in federal and state regulations can also disrupt the planning process. In planning for shore protection, there is a tendency to circumvent critical steps in the planning process—sometimes in an attempt to minimize costs, and sometimes in response to intense concern and controversy over a particularly severe problem. This approach may achieve short-term benefits in that it leads to a prompt resolution of the pressing problem. Unfortunately, however, circumvention of key planning steps may lead 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 limited the effectiveness of shore protection projects. Chapter II demonstrated that the existing shore protection measures in northern Milwaukee County are providing varying levels of protection, with nearly 80 percent of the structures in need of maintenance and exhibiting some type of damage. Proposals for new shore protection measures have generally not included an analysis of potential adverse impacts on adjacent shoreline areas. In many cases, shore protection measures are designed and constructed without a thorough understanding of the coastal processes and hydrogeologic features affecting the site concerned, or of the interaction of that site with adjacent shoreline reaches.

The site-specific analytical procedures and design criteria for shore protection presented herein represent a consistent set of guidelines which can and should be applied not only in the systems level phase, but also in the preliminary engineering and final design phases of the shore protection development process. These procedures and criteria are intended to promote a better understanding of the data collection and analysis efforts needed prior to plan implementation. The design criteria were used to design the alternative plans set forth in this systems level planning report, and to help test and evaluate their technical feasibility, and to ensure their comparability.

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 36. These procedures and criteria provide the means for quantitatively sizing and

Table 36

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 sandbars, if present 2. Calculate the anticipated wave height and runup at the structure under the design water level and storm wave conditions, and under an appropriate range of water level and wave conditions 3. Evaluate the impacts on adjacent shoreline areas of wave reflection or interruption of the littoral drift. Estimate the amount of potential beach material 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. Major 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 584.5 feet NGVD (583.2 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. Major 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 3. Structures should be designed to prevent severe damage from undercutting, flanking, or overtopping during the design storm. Positive drainage for water that overtops the structure and for groundwater that 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 artificially nourished with beach material as needed. Groin construction should begin at the downdrift end of the shoreline segment, and the beach fill should be placed promptly following completion of the groins 6. Inflexible gravity structures should not be installed on sand and gravel or soft clay deposits. Glacial till containing boulders is generally acceptable for gravity structures, but is often difficult for pile driving 7. Suitable measurements should be incorporated into the site plan to allow ready access by heavy construction equipment, as needed to maintain the structure on a long-term basis
Bluff Slope Instability	Regrading of Bluff Slope Utilizing Cutback, Filling, and/ or Terracing	1. Survey the bluff geometry and groundwater conditions and take 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, and monitor seasonal fluctuations in the water table elevation. 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

Table 36 (continued)

Shoreline Problem	Potentially Applicable Shore Protection Measures	Site Specific Inventories and Analyses	Design Criteria
Bluff Slope Instability (continued)	**Regrading of Bluff Slope Utilizing Cutback, Filling, and/ or Terracing (continued)**	2. Conduct a detailed slope stability analysis of the existing bluff slope. Conduct additional stability analyses where the bluff profile, stratigraphy, or groundwater conditions vary substantially 3. Conduct a slope stability analysis of the bluff slope anticipated to exist at the completion of regrading, or following the construction of terraces	22°, or one on two and one-half, unless a detailed slope stability analysis indicates that a steeper slope angle would be stable. Filling only the lower portion of the slope, cutting back the top of 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 important existing buildings or facilities are located less than 50 feet from the bluff edge. Fill should be placed only on the lower portion of the bluff slope, unless additional fill is required to stabilize the slope or to provide access to the lower portion. Fill may be used to construct a safe roadway, suitable for haul trucks, down the face of the bluff. Where an access roadway must be constructed from the top down, the fill material should be distributed along the face of the bluff to avoid large accumulations of fill material on top of the bluff. The amount of fill used, and the extension of the fill, if any, into Lake Michigan, should be the minimum needed to stabilize adequately the bluff slope or to provide a configuration aligned with the adjacent shoreline 4. Where fine-grained material is used as fill, a coarse gravel drainage layer with a suitable outlet should be provided beneath the fill. This drainage system must be maintained on a long-term basis to freely drain the fill layer at all times 5. The slope stability analyses and observed angles of similar fill slopes should be used to specify the stable slope angle for fills composed of mixtures of soil, concrete rubble, rock, and similar materials 6. Fill material may include granular soil, broken concrete, rock, and other clean material. Lumber, metal, asphalt, tires, clay soils, easily corroded material, and litter should not be used for fill 7. The fill material should be deposited at the base of the bluff first, and then filled upward 8. Granular fill material should be covered with a two-foot layer of finer grained silt or loam soil to allow rapid revegetation of the bluff slope. Impermeable clay soils should not be used to cover fill material. No rocks or broken concrete should be visible on the completed surface 9. Bluff toe protection and surface water and groundwater drainage control should be incorporated into a fill project in accordance with the guidelines provided in this table. Provision should be made for drainage of groundwater where water-bearing strata or groundwater seepage is observed

Table 36 (continued)

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 Dissipaters, Outlet Structures, Drop Structures, Slope Drains, Erosion Control Measures	1. Review condition of gullies and channels. Identify eroded or scoured waterways, areas of sheet and rill erosion, and poorly drained areas 2. Identify sources of surface water runoff and evaluate condition and capacity of outlets. Identify discharge sites for rooftop and driveway runoff 3. Estimate peak flow discharges and flow velocities in critical channels and gullies	1. Stormwater drainage systems should be designed to utilize to the fullest extent practicable the natural drainage system, and to provide the most economical installation of gravity flow systems. A primary objective of stormwater management is the maintenance of a good vegetative cover on drainageways and on the bluff slope, and the prevention of soil erosion 2. Stormwater drainage outlets should be located and designed to avoid discharging surface runoff over the top of the bluff, unless suitable conveyance facilities are provided to accommodate the flow without causing soil erosion or reducing the stability of the bluff slope 3. To prevent excessive scouring of open drainage channels, flow velocities during a 10-year recurrence interval design storm should be limited to a maximum of six feet per second for turf-lined channels which, if necessary, may contain a concrete cunette; and to a maximum of 10 feet per second for rip-rap-lined channels. Where practicable, grade control structures should be provided as necessary to reduce the channel gradient and obtain flow velocities within accepted limits. Turf-lined side slopes should be limited to a maximum of one on two 4. The use of measures to enhance infiltration of stormwater which would increase groundwater levels or seepage rates should be avoided 5. Water should not be allowed to accumulate or pond at the top of the bluff, on terraced bluff slopes, or on top of slump blocks 6. Stormwater discharge outlets at the base of the bluff should be designed to prevent scouring or erosion

Table 36 (continued)

Shoreline Problem	Potentially Applicable Shore Protection Measures	Site Specific Inventories and Analyses	Design Criteria
Poorly Vegetated Bluff Slope Which Allows Surface Erosion or Shallow Sliding	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. For successful vegetation, conduct a thorough site analysis of climate, soils, slope, and water availability. Identify specific needs of carefully selected plant species with respect to control of surface water and groundwater, slope shaping, and soil management 3. Survey the existing vegetation to identify what vegetation exists and effectively controls erosion on the slope 4. Identify aesthetic and functional preferences	1. Where bluff revegetation is indicated, the bluff may be allowed to reestablish 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 that 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.

thereby ensuring the performance of shore protection measures, thus providing a uniform and consistent base of reference for use in project development and design. Because of the variability of coastal and hydrogeologic conditions along the shoreline, step-by-step instructions to properly analyze or design a shore protection project cannot be provided. Table 36 lists those issues which should be addressed in site-specific analyses, recognizing that the actual analyses may have to be varied depending on the site characteristics.

Total shore protection at a site will often involve the implementation of more than one specific management measure. The application of these recommended procedures and criteria alone will not assure that the total shore protection project is properly integrated, or that the project is fully consistent with adjacent shore protection projects. Thus, 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 discussion. The design of bluff toe protection structures requires an estimate to be made of the highest water levels that may reasonably be expected to occur during the life of the structure. 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 Straits of Mackinac, evaporation from the lake surface, and resulting changes in the storage—the volume of water—in the lake. Record high Lake Michigan water levels experienced in 1986 were primarily due to the unusually large amounts of precipitation that occurred during 1985 and 1986. Under a

Commission study of the Milwaukee Harbor estuary,[1] it was determined that the 100-year recurrence interval maximum annual mean water level of Lake Michigan at the Milwaukee Harbor is 582.9 feet above National Geodetic Vertical Datum (NGVD). The 100-year recurrence interval maximum instantaneous lake level—which would include seiche effects and wind setup during storms—is 584.5 feet NGVD. The period of record considered under the Milwaukee Harbor estuary study was 1915 through 1985. It should be noted that the record high mean lake level in 1986 was 582.5 feet NGVD, or about 0.4 foot lower than the 100-year recurrence interval high maximum annual mean water level. The mean annual level of Lake Michigan at Milwaukee over the period of 1915 through 1985 was 579.5 feet NGVD. The low water datum, which is a relatively low level of 578.1 feet NGVD, measured in 1955, serves as a plane of reference to which navigation chart depths and navigation improvement depths are referenced.

It is recommended that shore protection structures be designed to prevent severe damage at the 100-year recurrence interval maximum instantaneous lake level of 584.5 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.

The design of shore protection structures to protect essential public works facilities or high-value facilities often dictates that more conservative design considerations than those noted above for lake levels be utilized. There are no standard design practices in this regard, and each project design engineer must make those decisions based upon the specifics of the particular project. Geological evidence indicates that within the last 1,000 years there have been at least two episodes in which Lake Michigan levels have exceeded the 1986 record high annual mean level by about four feet. Such evidence, along with some studies of long-term climatological data, indicates that the lake levels may be in a long-term rising trend. In light of these data, the studies conducted by the Regional Planning Commission for the Milwaukee Harbor estuary study included an analysis of possible future water levels assuming that Lake Michigan is in a long-term rising trend. Based upon analyses conducted in that study, it was concluded that, on a long-term basis, there is a reasonable probability that the lake could rise to about two feet higher than the 1985 levels. That would result in a design lake level of 585.9 feet NGVD, or about 1.4 feet higher than the 100-year recurrence interval maximum instantaneous lake level noted above. This higher design level, or other high levels, could be appropriately considered in the design of important public works or other major facilities. More extensive research on global climatic trends is needed to properly assess the potential for significant long-term changes in lake levels.

The maximum wave height generated by a storm which can reach the shore is limited by the available water depth. At the design lake levels described above, it is recommended that onshore protection structures be designed to prevent severe damage by the maximum wave capable of being supported in the near-shore environment. Within the study area, the maximum design wave for onshore structures will generally have a recurrence interval of less than 10 years owing to the shallow bathymetry offshore. It is recommended that offshore protection structures be designed to prevent severe damage by the 50-year recurrence interval wave height.

CONCEPTUAL SHORE PROTECTION MEASURES

The analysis of the need for, and the selection of, 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 36 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 36 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

---

[1] *SEWRPC Planning Report No. 37, A Water Resources Management Plan for the Milwaukee Harbor Estuary, Volume One, Inventory Findings, March 1987.*

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 was based on systems level analyses. This section describes the alternative shore protection measures that 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, is presented for each protection measure for use in the systems level planning effort. The alternative structural designs and associated costs presented in this chapter represent typical structural designs for Lake Michigan shoreline areas. All costs are presented in 1987 dollars.

Bluff Toe Protection

Shoreline areas exhibiting significant bluff toe erosion in 1986 were identified in Chapter III of this report and include approximately 31,010 feet, or 80 percent, of the study area shoreline. Alternative bluff toe protection measures evaluated for the northern Milwaukee County study area included both onshore and offshore structures. Onshore structures include revetments, bulkheads, and groins; offshore structures include breakwaters, barrier reefs, and islands. A general comparison of selected characteristics of bluff toe protection measures is provided in Table 37. The table presents certain requirements for successful application of the structures, lists the advantages and disadvantages of each type of structure, and notes the compatibility of the structure with alternative shoreline uses. These data serve as the basis for determining which structures should be evaluated for individual bluff analysis sections. There is no single type of structure that should be used in all cases; consideration of the specific characteristics of each section to be protected is essential in the planning and design of bluff toe protection measures.

The following sections describe common structural toe protection measures presently used in the Great Lakes and provide guidelines for the application of these measures. 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 the U. S. Army Corps of Engineers' Shore Protection Manual (1984). Where appropriate, some manufactured shore protection systems are herein described. These are intended to be representative of the types of products commercially available, and the description should not be considered an endorsement of those products. The project design engineer should consider the particular advantages, disadvantages, costs, and performance record of the products available within the Great Lakes area.

Frequently, there are opportunities to trade off capital and maintenance costs. Larger, or multiple, structures, requiring higher capital costs but resulting in lower maintenance costs, may be utilized to provide the same level of shore protection recommended herein. The plan costs set forth in this report were based upon a judicious combination of facilities providing capital and maintenance costs which were considered to be reasonable for lakeshore property owners. This should not preclude the use of a somewhat different trade off of capital and maintenance costs based upon more detailed analyses developed in project-specific preliminary engineering.

Revetment: Various types of revetments are commonly used to provide toe protection within northern Milwaukee County. Revetments contain a flattened slope at the bluff toe armored with material resistant to wave erosion and ice damage, and 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

Table 37

COMPARISON OF BLUFF TOE PROTECTION MEASURES

Bluff Toe Protection Measure	Type	Advantages	Disadvantages	Compatibility with Alternative Shoreline Uses					Capital Cost ($/lineal foot of shoreline)[a]	Annual Maintenance Cost ($/lineal foot of shoreline)[a]
				Walking	Swimming	Fishing	Boating	Aesthetics		
Revetment	Rip-rap	Easy to construct and maintain Flexible, durable	Limits access to shoreline Heavy equipment required for installation May reflect wave energy	Fair	Poor	Fair	Poor	Good	200-700	5-20
	Grout-filled bags	Constructed where access limited Adaptable to add-on construction	Limits access to shoreline Relatively inflexible Not as durable as quarry stone	Fair	Poor	Poor	Poor	Fair	200-250	10-20
	Manufactured concrete systems	Provides uniform appearance Adaptable to add-on construction Concrete units interlock for stability	Relatively inflexible Generally not as durable as quarry stone Heavy equipment required for installation	Fair	Fair	Fair	Fair	Fair	150-450	15-20
Bulkhead	Concrete cantilevered	Uniform appearance Infrequent maintenance requirements Durable	Loss of beach may be intensified Relatively inflexible Maintenance, when required, is difficult and costly Reflects wave energy	Good	Fair	Good	Fair	Fair	400	10-15
	Steel sheet piling	Uniform appearance Infrequent maintenance requirements Durable	Loss of beach may be intensified Relatively inflexible Maintenance, when required, is difficult and expensive Special pile-driving equipment required to install Reflects wave energy	Good	Fair	Good	Fair	Fair	650	5-10
	Concrete-stepped	Provides uniform appearance and usable shoreline Infrequent maintenance requirements Durable	Relatively inflexible Loss of beach may be intensified Maintenance, when required, is difficult and costly Reflects wave energy	Good	Fair	Good	Good	Fair	1,300	5-10

Table 37 (continued)

Bluff Toe Protection Measure	Type	Advantages	Disadvantages	Compatibility with Alternative Shoreline Uses					Capital Cost ($/lineal foot of shoreline)[a]	Annual Maintenance Cost ($/lineal foot of shoreline)[a]
				Walking	Swimming	Fishing	Boating	Aesthetics		
Onshore or Near-shore Beach Systems	Rock groins with nourished gravel beach	Provides usable shoreline Flexible Absorbs wave energy Feeds littoral transport system Adjusts to variable water levels	The beach would need to be periodically re-nourished Trapping sand supply in littoral drift may reduce the available sand for down-current beach areas	Good	Good	Good	Good	Good	400-900	10-40
	Armored headland-pocket beach system	Flexible, durable Provides usable shoreline Pocket beaches absorb wave energy Adjusts to variable water levels	May require large amount of fill to construct Beach would need to be periodically re-nourished to maintain sand or fine gravel Trapping sand supply in littoral drift may reduce the available sand for down-current beach areas Armored headlands may reflect wave energy	Good	Good	Good	Good	Good	600-1,200	10-40
	Near-shore reefs with nourished gravel beaches	Flexible Provides uniform appearance and continuous usable shoreline Feeds littoral transport system Adjusts to variable water levels	The beach would need to be periodically re-nourished Trapping sand supply in littoral drift may reduce the available sand for down-current beach areas Reefs are subject to large wave attack and thus more susceptible to damage than are onshore structures Limits view of horizon	Good	Good	Good	Fair	Fair	450-1,200	15-50
	Perched cobble beach without covering of sand or gravel	Cobbles absorb considerable wave energy without causing scouring from wave reflection Adjusts to variable water levels	Limits use of shoreline	Poor	Poor	Poor	Poor	Fair	350-400	20

Table 37 (continued)

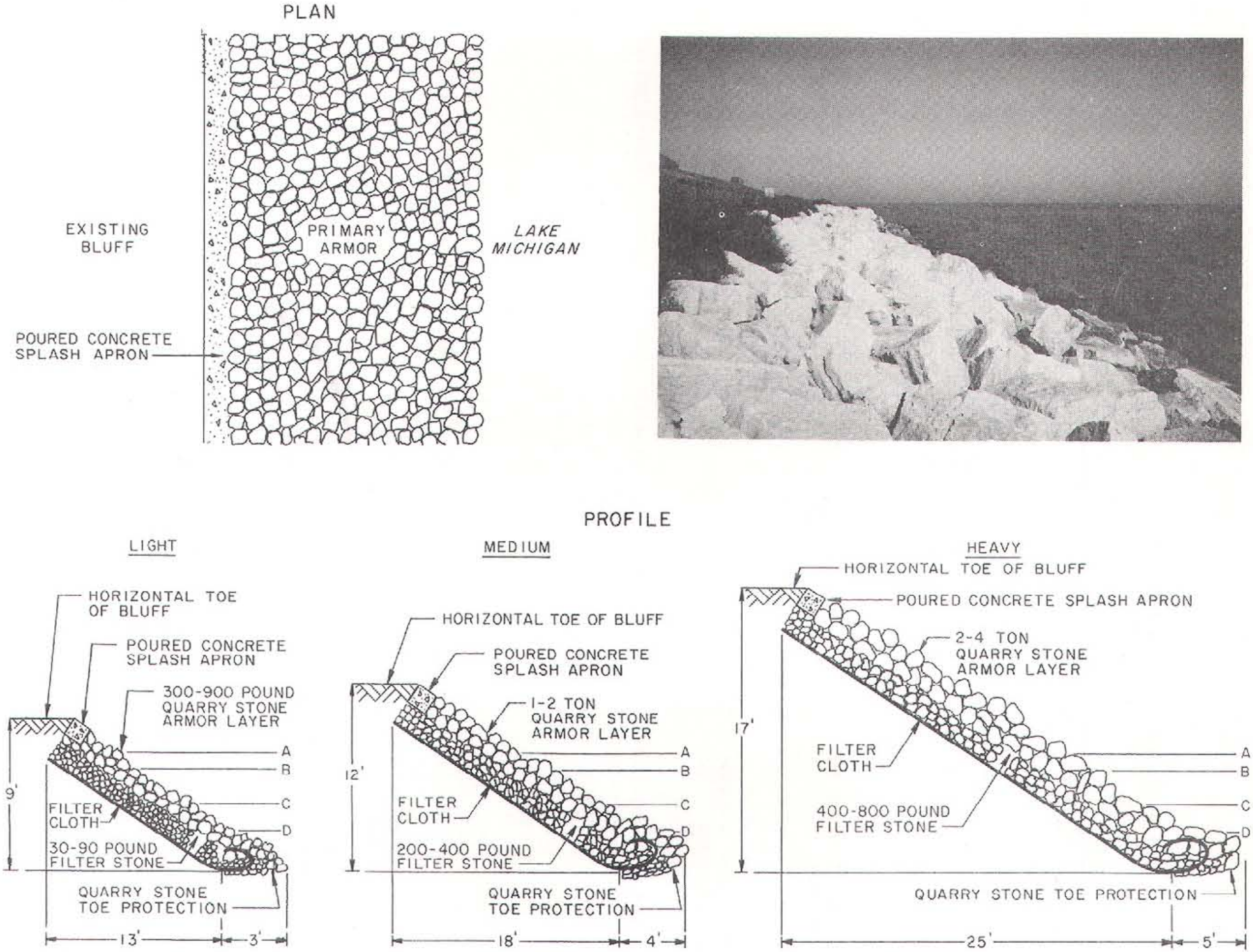
Bluff Toe Protection Measure	Type	Advantages	Disadvantages	Compatibility with Alternative Shoreline Uses					Capital Cost ($/lineal foot of shoreline)[a]	Annual Maintenance Cost ($/lineal foot of shoreline)[a]
				Walking	Swimming	Fishing	Boating	Aesthetics		
Beach Systems (continued)	Near-shore pervious concrete sill	Reduces wave attack by tripping and slowing waves Enhances sediment accretion near shore	Requires some beach, gentle offshore slope, and low-wave-energy environment Interferes with small boat navigation near shore	Good	Good	Fair	Poor	Good	200-300	10
	Manufactured concrete systems nourished with sand or gravel	Provides partially usable shoreline	Limits access to water Blocks may settle or move out of alignment	Good	Fair	Fair	Fair	Fair	200-300	15-50
Offshore Breakwater with Nourished Sand Beach	Rubble mound	Provides substantial protection Use of shoreline not restricted Provides large sand beach	Heavy equipment mounted on barges may be required for installation and maintenance Trapping sand supply in littoral drift may reduce the available sand for down-current beaches	Good	Good	Good	Good	Good	1,000-2,000	20-50
Offshore Island or Peninsula	- -	Additional land created for recreational use Provides substantial protection Use of shoreline not restricted	Large amount of fill material required to construct Degree of protection needed on lakeward side of island Heavy equipment mounted on barges may be required for installation and maintenance	Good	Good	Good	Good	Good	800-1,200	20-40

[a]The costs shown are estimates of the likely costs entailed where these measures could effectively be used. Because, at any one site, the different structures would not all offer the same level of protection, and because the structure unit costs are site specific, a direct comparison of the costs for the different structures may not be appropriate.

Source: SEWRPC.

Figure 100

TYPICAL RIP-RAP REVETMENT



LEGEND

DESIGN WATER LEVELS

A DESIGN HIGH STILL WATER LEVEL PLUS WIND SETUP 584.5 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

B DESIGN HIGH STILL WATER LEVEL 582.9 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

C 1915 TO 1985 ANNUAL MEAN WATER LEVEL 579.5 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

D LOW WATER DATUM 578.1 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

NOTE: THE DESIGN SPECIFICATIONS SHOWN HEREIN ARE FOR A TYPICAL STRUCTURE. THE DETAILED DESIGN OF SHORE PROTECTION MEASURES MUST BE BASED ON A DETAILED ANALYSIS OF WAVE CLIMATE, COST AND AVAILABILITY OF CONSTRUCTION MATERIAL, SPECIFIC GRAVITY AND QUALITY OF THE STONE, TYPE OF LAKEBED MATERIAL, AND EXISTING SHORELINE GEOMETRY.

Source: SEWRPC.

revetment, and an interlocking concrete block revetment.

<u>Rip-rap</u>: As shown in Figure 100, a rip-rap revetment utilizes rock or quarry stone as its armor layer. To be durable, the armor stone should be free of laminations and cracks. The stone should be angular, with the greatest dimension no greater than three times the least dimension. Slab-shaped stones are not desirable for an armor layer. Rip-rap revetments providing three levels of protection are illustrated in Figure 100. A light revetment may require two to three tons of stone per lineal foot of shoreline; a medium revetment, three to five tons of stone per foot; and a heavy revetment, five to 10 tons of stone per foot. The size of the armor stones needed to provide adequate protection is depen-

dent on the wave height, the specific gravity and quality of the stone, the slope of the structure, and the degree of interlocking of the individual stones. An alternative rip-rap revetment design, known as a berm revetment, utilizes a thick layer of variable-size armor stone with an average weight typically less than one-half the weight of the stone required by conventional design methods. Wave action shapes the thick armor layer into a berm which dissipates the wave energy.

The advantages of a rip-rap revetment are that it is relatively easy to construct and maintain; it is flexible, and can therefore withstand some movement or displacement without total failure; and it provides a relatively natural appearance to the shoreline.

The primary disadvantages of a rip-rap revetment are that the structure generally makes use of the immediate shoreline area for recreational activities difficult, and access to the water may be precluded. A rip-rap 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. Rip-rap revetments, particularly steep structures, reflect wave energy, although less than would most bulkheads. This reflected energy may scour offshore lakebed material, especially immediately in front of the structure. A steeper offshore slope would allow larger waves to reach the shoreline.

The life of a rip-rap revetment depends on the durability of the rock used for construction and on the degree of maintenance performed. Rip-rap revetments may be affected by settling and displacement. If armor stones are moved by wave action, the entire structure may be weakened if not maintained. Rip-rap revetments placed on sand without proper filter material and those utilizing undersized armor stone are particularly prone to failure.

The cost of rip-rap 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 rip-rap revetment range from $5.00 to $20 per lineal foot.

Grout-Filled Bags: Large grout-filled bags have been placed at the toe of bluffs to form revetments within the study area. These bags are typically six feet deep by two and one-half feet high, and up to 20 feet long. The 20-foot-long bags weigh about 14 tons each. As shown in Figure 101, the bags should be placed parallel to the shore with reinforcing bars installed both vertically and horizontally to hold the bags together. A filter cloth and a gravel bed should be placed beneath the bags to provide drainage and prevent the underlying soil from being washed away by waves or groundwater seepage. The bags are the most appropriate for low- to moderate-wave-energy environments.

The primary advantage of a grout-filled bag revetment is that it can be constructed where access is limited. A grout pump which can be operated from the top of a bluff is used to fill the bags. In addition, the structure is readily adaptable to add-on construction if additional structure height is necessary. The bags are rounded, providing limited access to the shoreline.

The primary disadvantage of a grout-filled bag revetment is that it is relatively inflexible, and is therefore more vulnerable to wave forces than is an equivalent rip-rap 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, and may be susceptible to bottom scouring. Since concrete is not as dense as natural rock, a larger volume of concrete is required to provide the same weight and therefore protection as natural rock.

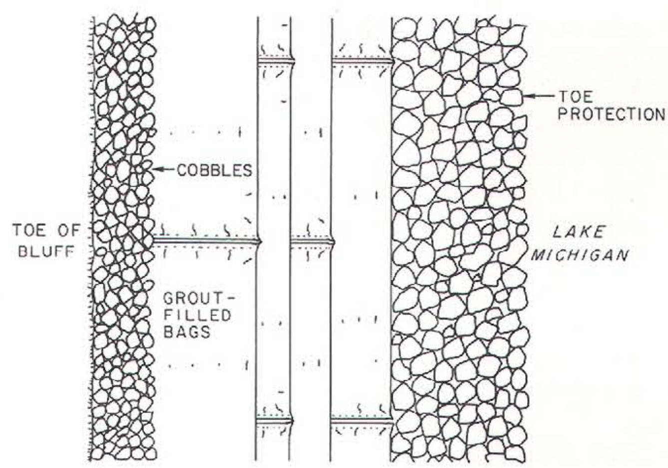
The capital cost of a grout-filled bag revetment is influenced by design water level and depth, wave environment, material cost, and other site-specific factors, but in general ranges from $200 to $250 per lineal foot of shoreline. Average annual maintenance costs may range from $10 to $20 per lineal foot.

Concrete Structures: Several different types of manufactured concrete structures are commercially available. The interlocking concrete blocks or slabs fit together to form a revetment. These blocks or slabs typically weigh up to 1,000 pounds each. As shown in Figure 102, the blocks or slabs are usually perforated with holes or

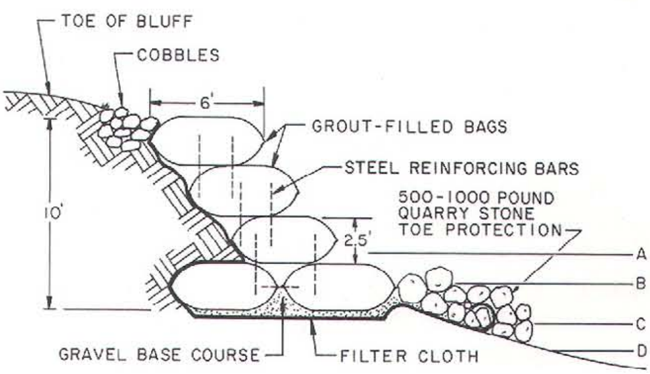
Figure 101

TYPICAL GROUT-FILLED BAG REVETMENT

PLAN



PROFILE



LEGEND

DESIGN WATER LEVELS

A DESIGN HIGH STILL WATER LEVEL PLUS WIND SETUP 584.5 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

B DESIGN HIGH STILL WATER LEVEL 582.9 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

C 1915 TO 1985 ANNUAL MEAN WATER LEVEL 579.5 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

D LOW WATER DATUM 578.1 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

NOTE: THE DESIGN SPECIFICATIONS SHOWN HEREIN ARE FOR A TYPICAL STRUCTURE. THE DETAILED DESIGN OF SHORE PROTECTION MEASURES MUST BE BASED ON A DETAILED ANALYSIS OF WAVE CLIMATE, COST AND AVAILABILITY OF CONSTRUCTION MATERIAL, SPECIFIC GRAVITY AND QUALITY OF THE STONE, TYPE OF LAKEBED MATERIAL, AND EXISTING SHORELINE GEOMETRY.

Source: U. S. Army Corps of Engineers and SEWRPC.

slots to neutralize pressure from changing water levels and to absorb energy from wave action. A filter cloth should be placed beneath the concrete units to prevent the underlying soil from being washed away by waves or groundwater seepage, and stone should be placed at the toe of the revetment to prevent scouring.

The advantage of an interlocking concrete system is that it provides a uniform appearance and a usable shoreline which may be suitable for some recreational activities. In addition, the system is readily adaptable to add-on construction.

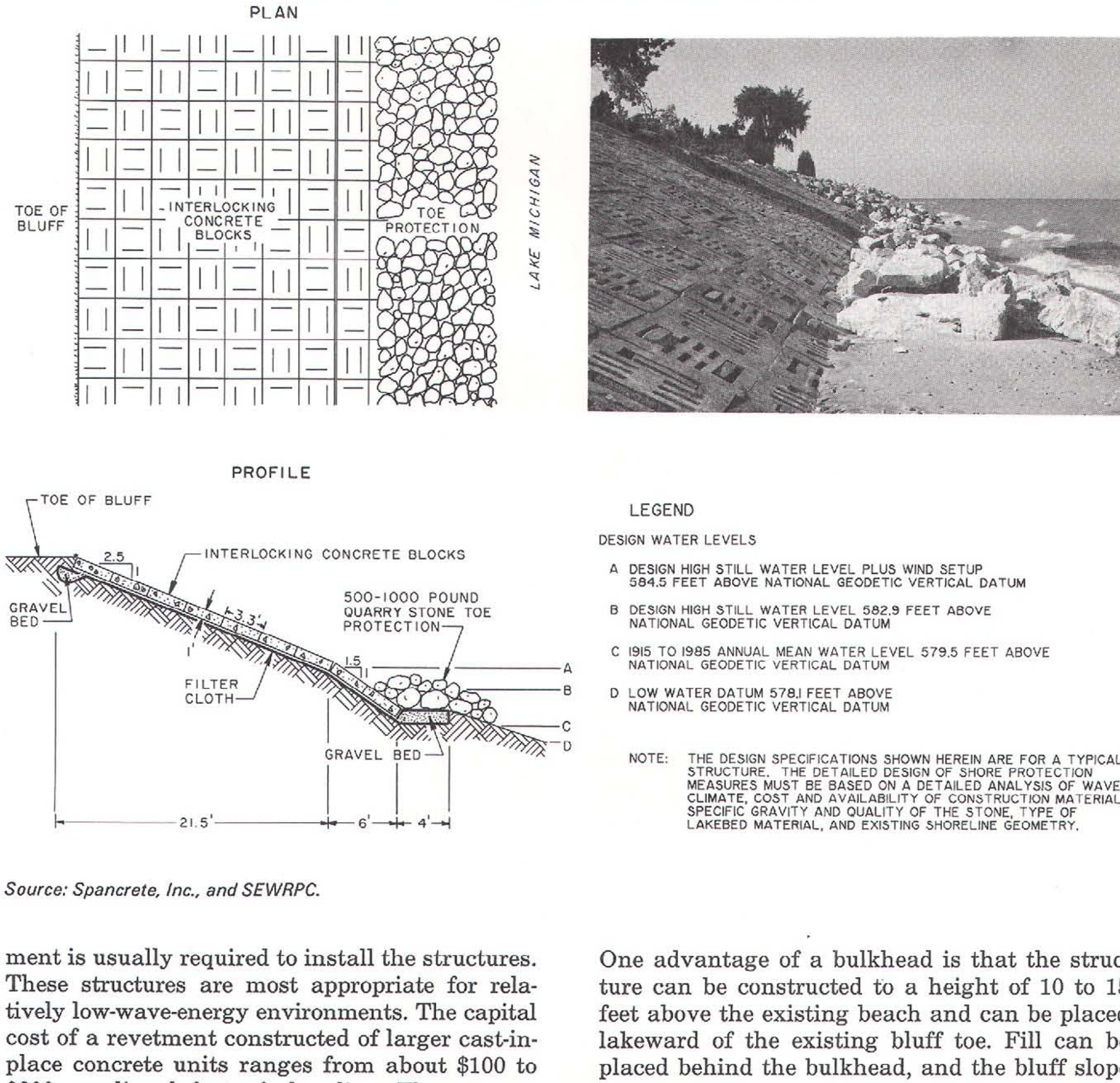
A major disadvantage of interlocking concrete blocks in general is that the failure of one block can lead to rapid failure of adjacent blocks. In some applications, the blocks may not be as durable as a rip-rap revetment. Failure of the subgrade will quickly result in excessive movement of the blocks. The interlocking concrete block systems are most appropriate for relatively low-wave-energy environments.

The capital cost of an interlocking concrete block system, depending on site characteristics, approximates \$150 to \$450 per lineal foot of shoreline. The average annual operation and maintenance cost would approximate \$15 to \$20 per lineal foot.

Larger concrete units, usually cast in place, which do not specifically interlock are also commercially available. These units can be placed along the shoreline to create a revetment. The units, an example of which is shown in Figure 103, vary in size, often ranging from two to three tons each. Heavy construction equip-

Figure 102

TYPICAL INTERLOCKING CONCRETE BLOCK REVETMENT



Source: Spancrete, Inc., and SEWRPC.

ment is usually required to install the structures. These structures are most appropriate for relatively low-wave-energy environments. The capital cost of a revetment constructed of larger cast-in-place concrete units ranges from 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.

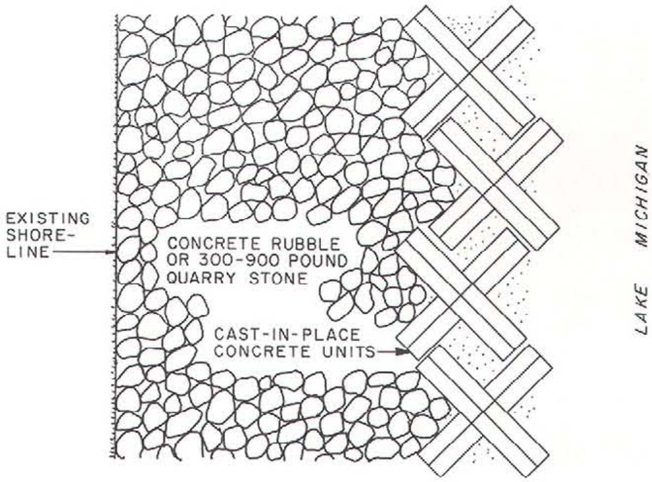
Bulkhead: Bulkheads are vertical retaining walls constructed of concrete, steel sheet piling, or timber which supports the base of the bluff and provides protection against wave and ice action. Historically, bulkheads have been the most commonly used shore protection structure in northern Milwaukee County, with most being constructed of concrete.

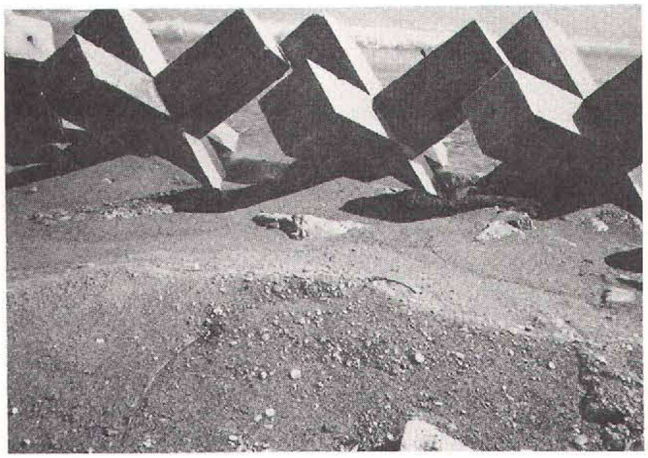
One advantage of a bulkhead is that the structure can be constructed to a height of 10 to 15 feet above the existing beach and can be placed lakeward of the existing bluff toe. Fill can be placed behind the bulkhead, and the bluff slope can be regraded from the top of the bulkhead rather than from the existing bluff toe. This effectively reduces the required bluff top regrading distance to achieve a stable bluff slope, as shown in Figure 104. Thus, the necessary cutting back of the top of the bluff to form a stable slope could be significantly reduced if a bulkhead is constructed. Another advantage of a bulkhead is that it provides a uniform appearance and may be suited for recreational facilities such as walkways, piers, and boat slips which may enhance the use of the shoreline.

Figure 103

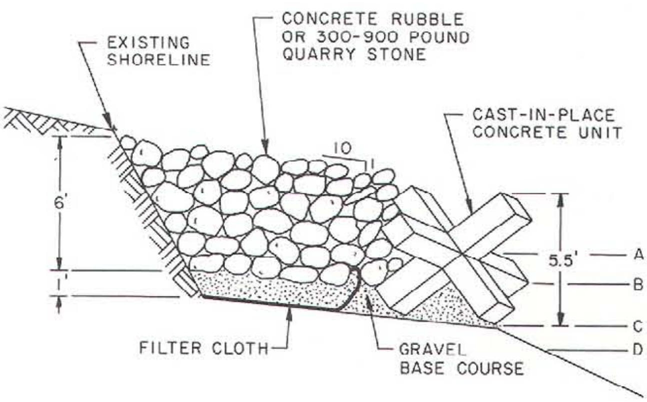
TYPICAL CAST-IN-PLACE CONCRETE UNITS

PLAN

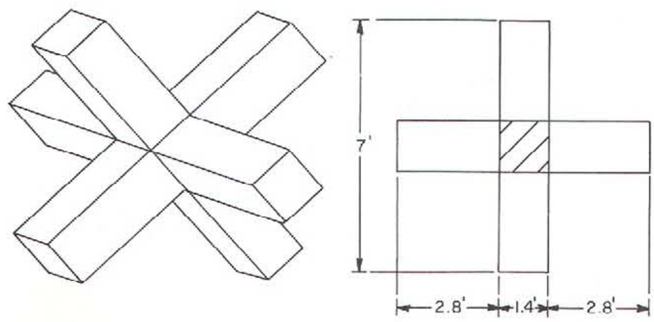




PROFILE



TYPICAL CONCRETE UNIT



LEGEND

DESIGN WATER LEVELS

A DESIGN HIGH STILL WATER LEVEL PLUS WIND SETUP 584.5 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

B DESIGN HIGH STILL WATER LEVEL 582.9 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

C 1915 TO 1985 ANNUAL MEAN WATER LEVEL 579.5 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

D LOW WATER DATUM 578.1 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

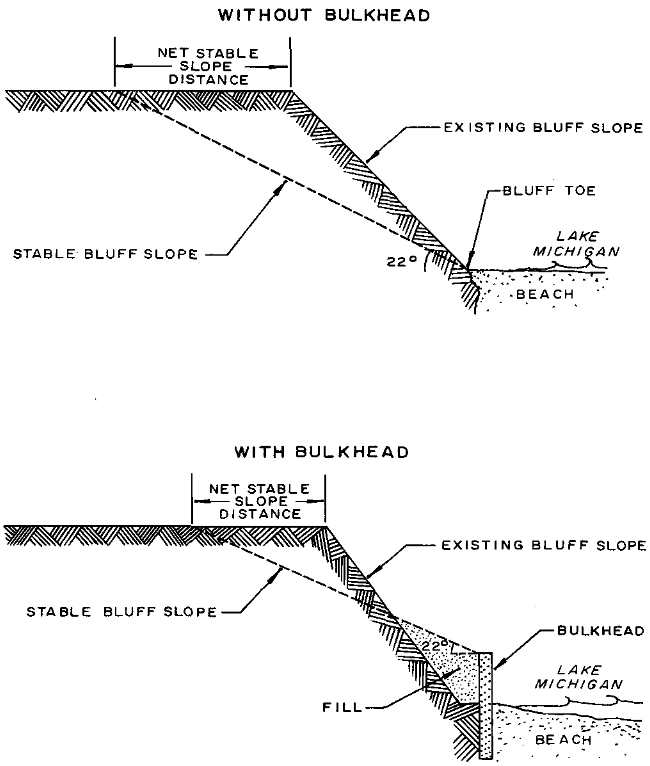
NOTE: THE DESIGN SPECIFICATIONS SHOWN HEREIN ARE FOR A TYPICAL STRUCTURE. THE DETAILED DESIGN OF SHORE PROTECTION MEASURES MUST BE BASED ON A DETAILED ANALYSIS OF WAVE CLIMATE, COST AND AVAILABILITY OF CONSTRUCTION MATERIAL, SPECIFIC GRAVITY AND QUALITY OF THE STONE, TYPE OF LAKEBED MATERIAL, AND EXISTING SHORELINE GEOMETRY.

Source: Dan Libecki Grading and SEWRPC.

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

Figure 104

EFFECT OF A BULKHEAD ON THE BLUFF TOP CUTBACK DISTANCE REQUIRED TO ACHIEVE A STABLE BLUFF SLOPE



Source: SEWRPC.

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 105, 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. Rip-rap 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 northern Milwaukee County would entail a capital cost of approximately $400 per lineal foot of shoreline. Average annual maintenance costs would range from $10 to $15 per lineal foot.

Steel Sheet Piling Bulkhead: A steel sheet piling bulkhead, as shown in Figure 106, 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 106. Rip-rap toe protection and weep holes for drainage should be provided. The structure should be backfilled with coarse granular material. Special pile-driving equipment is required to install the structure.

Construction of a steel sheet piling bulkhead along the Lake Michigan shoreline of northern Milwaukee County would require a capital cost of approximately $650 per lineal foot of shoreline. Average annual maintenance costs would range from $5.00 to $10 per lineal foot.

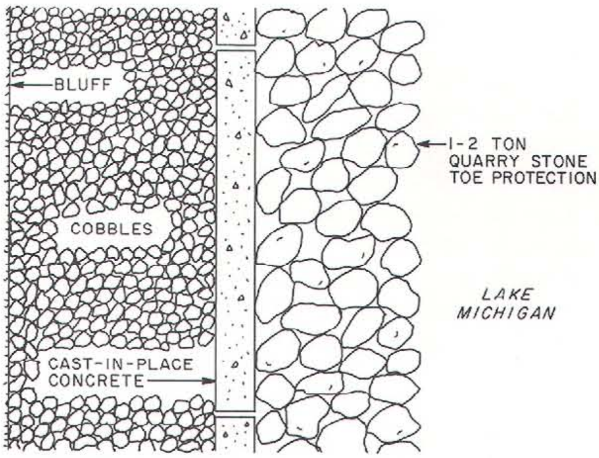
Concrete-Stepped Bulkhead: A third type of bulkhead is a cast-in-place, concrete-stepped bulkhead, as shown in Figure 107. The bulkhead, cast as a massive, gravity-held structure to resist overturning by wave action or soil pressures, should include a splash apron along the crest of the bulkhead to prevent 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.

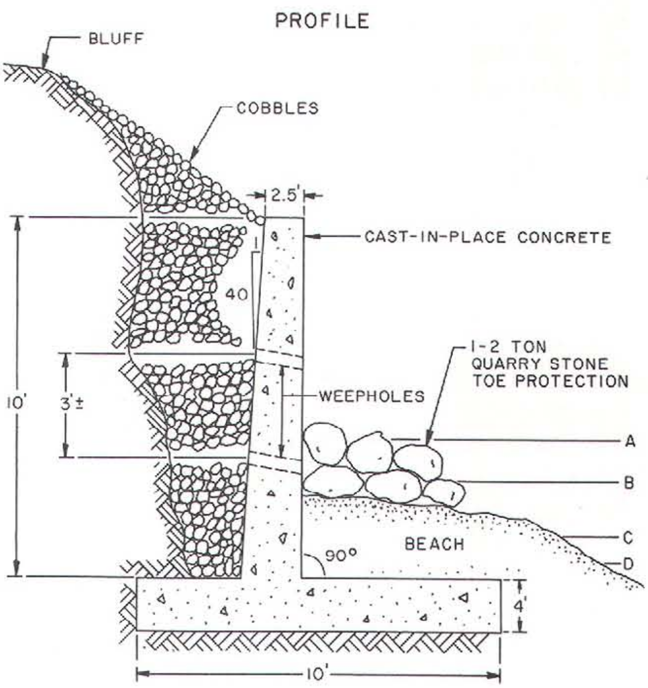
Construction of a concrete-stepped bulkhead along the Lake Michigan shoreline of northern Milwaukee County would entail a capital cost of approximately $1,300 per lineal foot of shoreline. Average annual maintenance costs would range from $5.00 to $10 per lineal foot.

Onshore or Near-Shore Beach Systems: There are several onshore or near-shore protection structures which may support a beach and in turn protect the bluff toe against wave action, while providing opportunities for the pursuit of recreational activities such as walking, swimming, and boating. Beach systems require

Figure 105

TYPICAL CONCRETE CANTILEVERED BULKHEAD





LEGEND

DESIGN WATER LEVELS

- A DESIGN HIGH STILL WATER LEVEL PLUS WIND SETUP 584.5 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM
- B DESIGN HIGH STILL WATER LEVEL 582.9 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM
- C 1915 TO 1985 ANNUAL MEAN WATER LEVEL 579.5 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM
- D LOW WATER DATUM 578.1 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

NOTE: THE DESIGN SPECIFICATIONS SHOWN HEREIN ARE FOR A TYPICAL STRUCTURE. THE DETAILED DESIGN OF SHORE PROTECTION MEASURES MUST BE BASED ON A DETAILED ANALYSIS OF WAVE CLIMATE, COST AND AVAILABILITY OF CONSTRUCTION MATERIAL, SPECIFIC GRAVITY AND QUALITY OF THE STONE, TYPE OF LAKEBED MATERIAL, AND EXISTING SHORELINE GEOMETRY.

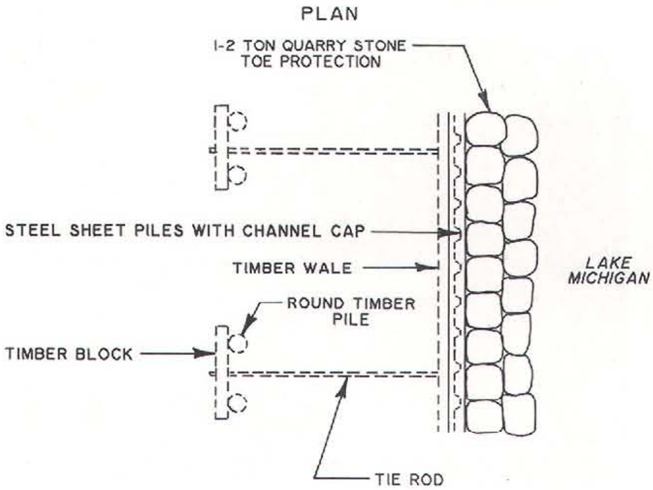
Source: SEWRPC.

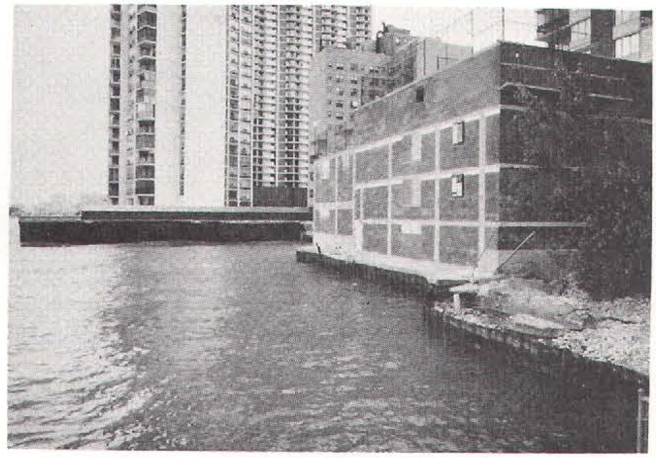
structures which are built out from the shoreline, or placed in the lake in shallow water. The structures are intended to prevent wave action from eroding a natural or artificially nourished beach. Because the supply of sand in the littoral drift is limited, it is often necessary to artificially nourish 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 38 lists the beach slopes expected to form on different sized beach material. As shown in the table, while sand beaches would generally have a slope of less than 5 degrees, gravel beaches may frequently have slopes approximating 10 degrees.

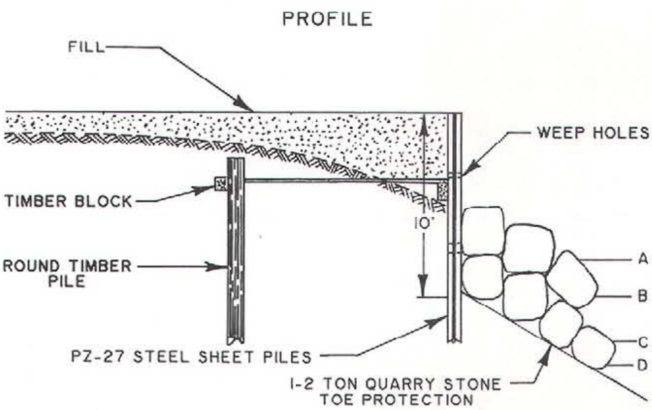
The major advantage of an onshore beach system is that an extended beach is provided to protect the bluff toe against wave action and to allow access to the lake for walking, swimming, and fishing.

Figure 106

TYPICAL STEEL SHEET PILING BULKHEAD







LEGEND

DESIGN WATER LEVELS

A DESIGN HIGH STILL WATER LEVEL PLUS WIND SETUP 584.5 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

B DESIGN HIGH STILL WATER LEVEL 582.9 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

C 1915 TO 1985 ANNUAL MEAN WATER LEVEL 579.5 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

D LOW WATER DATUM 578.1 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

NOTE: THE DESIGN SPECIFICATIONS SHOWN HEREIN ARE FOR A TYPICAL STRUCTURE. THE DETAILED DESIGN OF SHORE PROTECTION MEASURES MUST BE BASED ON A DETAILED ANALYSIS OF WAVE CLIMATE, COST AND AVAILABILITY OF CONSTRUCTION MATERIAL, SPECIFIC GRAVITY AND QUALITY OF THE STONE, TYPE OF LAKEBED MATERIAL, AND EXISTING SHORELINE GEOMETRY.

Source: U. S. Army Corps of Engineers and SEWRPC.

The disadvantages of beach systems include the potential for increasing downdrift erosion if the littoral drift is obstructed to form the beach, and the considerable maintenance that may be required to keep the extended beach intact. Also, insufficient bluff toe protection may be provided by the beach during large storm events, especially during high lake levels.

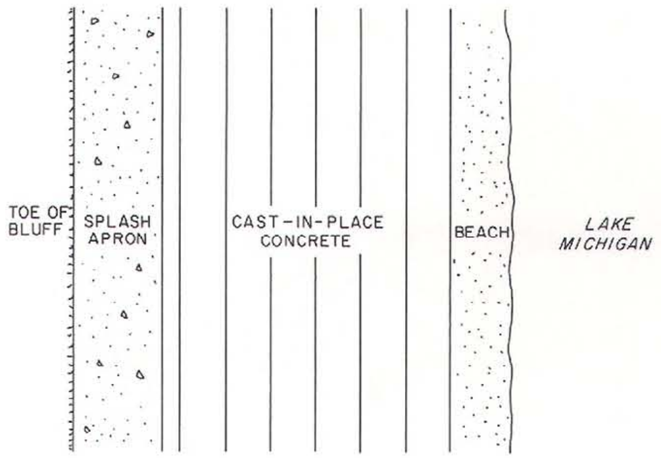
Described below are several types of onshore and near-shore beach system designs: groins, an armored headland-pocket beach system, near-shore reefs, and manufactured concrete systems.

Groins: Groins are the most common type of structure used to create beaches. Groins can be constructed of 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 in northern Milwaukee County 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

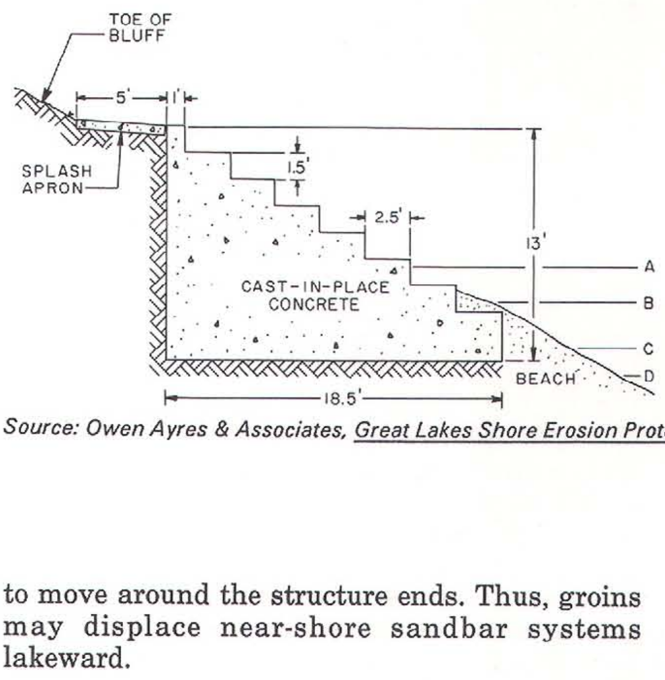
Figure 107

TYPICAL CONCRETE-STEPPED BULKHEAD

PLAN



PROFILE



LEGEND

DESIGN WATER LEVELS

A DESIGN HIGH STILL WATER LEVEL PLUS WIND SETUP 584.5 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

B DESIGN HIGH STILL WATER LEVEL 582.9 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

C 1915 TO 1985 ANNUAL MEAN WATER LEVEL 579.5 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

D LOW WATER DATUM 578.1 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

NOTE: THE DESIGN SPECIFICATIONS SHOWN HEREIN ARE FOR A TYPICAL STRUCTURE. THE DETAILED DESIGN OF SHORE PROTECTION MEASURES MUST BE BASED ON A DETAILED ANALYSIS OF WAVE CLIMATE, COST AND AVAILABILITY OF CONSTRUCTION MATERIAL, SPECIFIC GRAVITY AND QUALITY OF THE STONE, TYPE OF LAKEBED MATERIAL, AND EXISTING SHORELINE GEOMETRY.

Source: Owen Ayres & Associates, Great Lakes Shore Erosion Protection, Structural Design Examples, 1978, and SEWRPC.

to move around the structure ends. Thus, groins may displace near-shore sandbar systems lakeward.

Figures 108 and 109 show examples of rock and sheet pile groin systems designed to maintain a beach composed of gravel. The onshore portion of the groins would be constructed with a top elevation about seven feet above the existing beach level to retain the beach fill. The orientation and spacing of a groin system is highly dependent on the site-specific details of the project location, but spacing should generally be equal to about one and one-half to twice the groin length. The groins should be of sufficient height to prevent excessive overtopping. Periodic replenishment of the beach material will be required.

The height, orientation, and shape of groins may be modified, depending on the site characteristics, to either maximize beach containment or minimize trapping of the littoral drift. For example, the lakeward end of groins may be sloped downward to reduce downdrift impacts. The capital cost of a groin system ranges from $200 to $500 per lineal foot of shoreline, with an additional cost ranging up to $115 per lineal foot

Table 38

ESTIMATED BEACH SLOPES THAT WOULD FORM ON VARIOUS BEACH FILL MATERIALS

Breaking Wave Height	Very Coarse Sand 0.06 inch (1.5mm)	Very Fine Gravel 0.12 inch (3mm)	Fine Gravel 0.24 inch (6mm)	Medium Gravel 0.48 inch (12mm)	Coarse Gravel 0.96 inch (24mm)
3 feet (0.9m)	4°	6°	8°	12°	16°
6 feet (1.8m)	3°	4°	6°	8°	12°
9 feet (2.7m)	2°	3°	5°	7°	10°
12 feet (3.7m)	2°	3°	4°	6°	8°

NOTE: Calculated using the following formula from J. W. Kamphuis, M. H. Davies, R. B. Nairn, and O. J. Sayao: "Calculation of Littoral Sand Transport Rates," Coastal Engineering, Vol. 10, pp. 1-21, 1986:

$$m = \tan^{-1}\left[1.8\left(\sqrt{\frac{H}{D}}\right)^{-1}\right]$$

where: m = beach slope (degrees)
H = breaking wave height (m)
D = beach particle diameter (m)

Source: SEWRPC.

of shoreline to artificially nourish the beach by shore, or an additional cost of up to $500 per lineal foot of shoreline to artificially nourish the beach by barge. Annual maintenance costs depend upon the need for additional fill material, and range from $10 to $40 per lineal foot.

Armored Headland-Pocket Beach System: An armored headland and pocket beach system acts 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 110. The headlands are usually protected with an armor stone revetment. A headland beach system may create a relatively large amount of land for recreational use. Design considerations for the headlands are similar to those for a revetment.

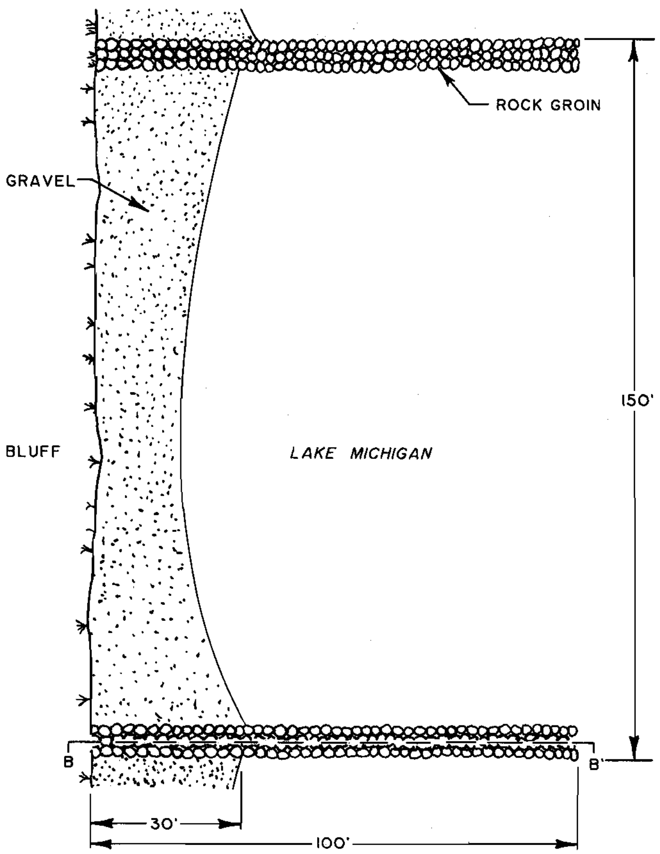
The capital cost of a headland and pocket beach system would range from $600 to $1,200 per lineal foot of shoreline. Average annual maintenance costs would range from $10 to $40 per lineal foot.

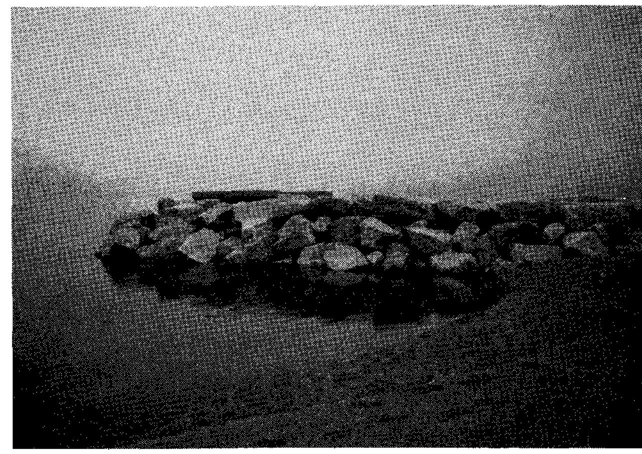
Near-shore Reefs: Near-shore reefs are constructed of stone and placed generally parallel to the shoreline in a water depth of four to five feet. Such reefs are generally located less than 100 feet from the shoreline, as shown in Figure 111. In some applications, the reefs may curve into the shoreline, or the system may be supplemented by groins. In a typical installation, a filter cloth would be placed on the lake bottom, covered with 5- to 90-pound stone, and then by 300- to 900-pound stone. An armor layer, consisting of 3- to 5-ton stone, would then be placed. The reefs would extend to a height about two feet above the design maximum instantaneous water level. A beach nourished with coarse sand or gravel would be maintained behind the reefs. As with the other beach systems, periodic addition of beach fill would likely be required.

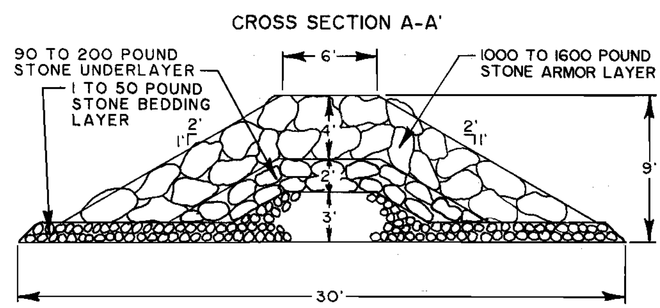
The capital cost of a near-shore reef ranges from $350 to $600 per lineal foot of shoreline, with an additional cost ranging up to $115 per lineal foot of shoreline to artificially nourish the beach by shore, or an additional cost of up to approximately $500 per lineal foot of shoreline to artificially nourish the beach by barge. Annual operation and maintenance costs would depend upon the need for periodic re-nourishment of the beach material, and are estimated to range from $15 to $50 per lineal foot.

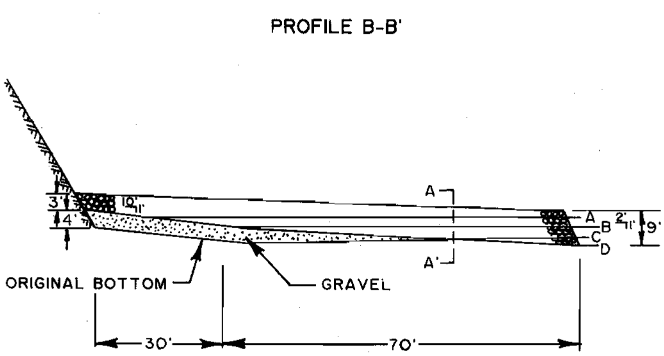
Figure 108

TYPICAL ROCK GROIN SYSTEM WITH ARTIFICIALLY NOURISHED BEACH









LEGEND

DESIGN WATER LEVELS

A DESIGN HIGH STILL WATER LEVEL PLUS WIND SETUP 584.5 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

B DESIGN HIGH STILL WATER LEVEL 582.9 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

C 1915 TO 1985 ANNUAL MEAN WATER LEVEL 579.5 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

D LOW WATER DATUM 578.1 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

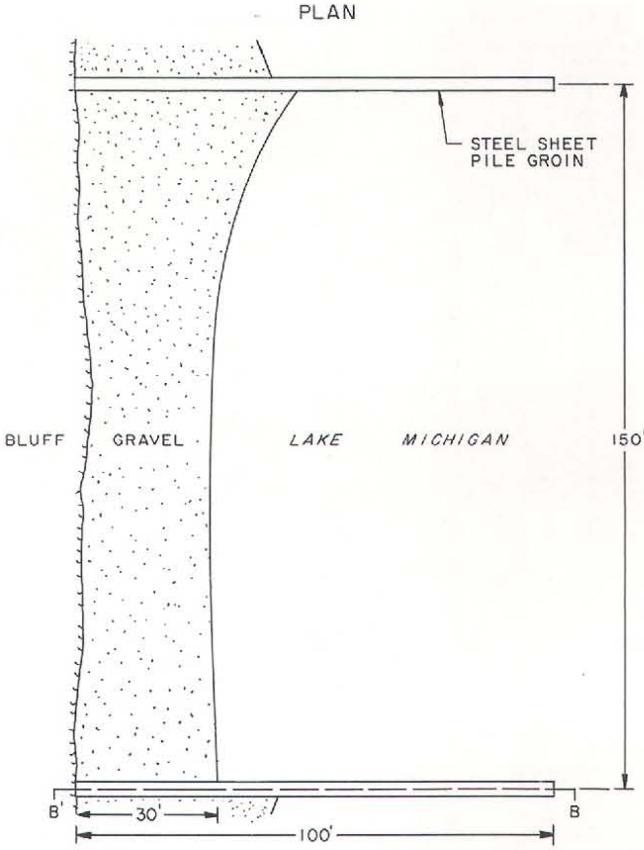
NOTE: THE DESIGN SPECIFICATIONS SHOWN HEREIN ARE FOR A TYPICAL STRUCTURE. THE DETAILED DESIGN OF SHORE PROTECTION MEASURES MUST BE BASED ON A DETAILED ANALYSIS OF WAVE CLIMATE, COST AND AVAILABILITY OF CONSTRUCTION MATERIAL, SPECIFIC GRAVITY AND QUALITY OF THE STONE, TYPE OF LAKEBED MATERIAL, AND EXISTING SHORELINE GEOMETRY.

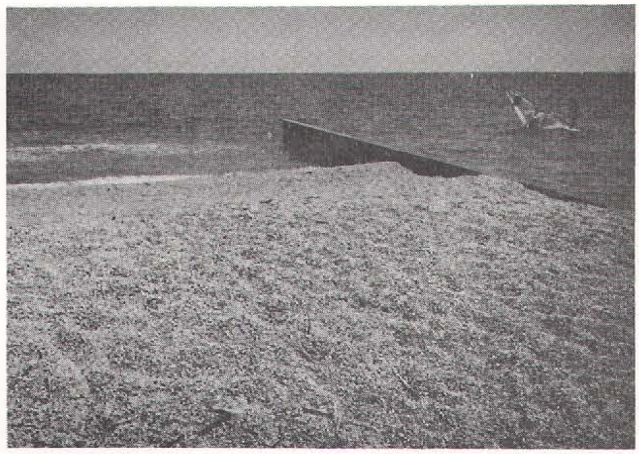
Source: U. S. Army Corps of Engineers and SEWRPC.

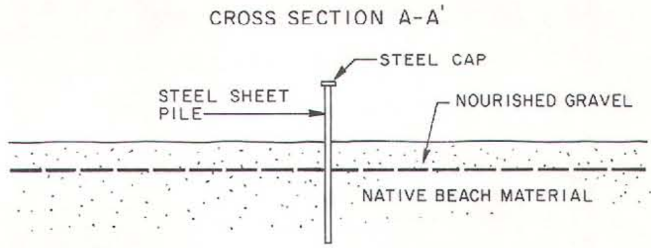
Perched Cobble Beach System: Perched beaches constructed of cobbles would serve as wave-absorbing structures, particularly suitable where the shoreline water is deep. A beach constructed of cobble stones ranging from 3 to 12 inches in diameter, as shown in Figure 112, would be able to absorb considerable wave energy while staying intact better than do beaches composed of sand and gravel. The reduced wave reflection would help prevent scouring of the lakebed by wave energy normally reflected by bulkheads or rip-rap revetments. The cobbles are typically swept by the storm surge to form raised ridges on the backshore, adding protection to bluffs. Lateral migration of the cobbles can be controlled by constructing barriers similar to groins on the downdrift sides. The disadvantage of a cobble beach system is that the use of the shoreline and access to the water may be severely limited, depending on the size and

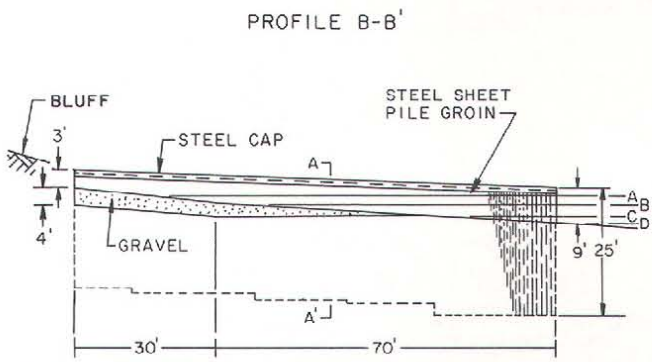
Figure 109

TYPICAL STEEL SHEET PILE GROIN SYSTEM WITH ARTIFICIALLY NOURISHED BEACH









LEGEND

DESIGN WATER LEVELS

A DESIGN HIGH STILL WATER LEVEL PLUS WIND SETUP 584.5 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

B DESIGN HIGH STILL WATER LEVEL 582.9 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

C 1915 TO 1985 ANNUAL MEAN WATER LEVEL 579.5 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

D LOW WATER DATUM 578.1 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

NOTE: THE DESIGN SPECIFICATIONS SHOWN HEREIN ARE FOR A TYPICAL STRUCTURE. THE DETAILED DESIGN OF SHORE PROTECTION MEASURES MUST BE BASED ON A DETAILED ANALYSIS OF WAVE CLIMATE, COST AND AVAILABILITY OF CONSTRUCTION MATERIAL, SPECIFIC GRAVITY AND QUALITY OF THE STONE, TYPE OF LAKEBED MATERIAL, AND EXISTING SHORELINE GEOMETRY.

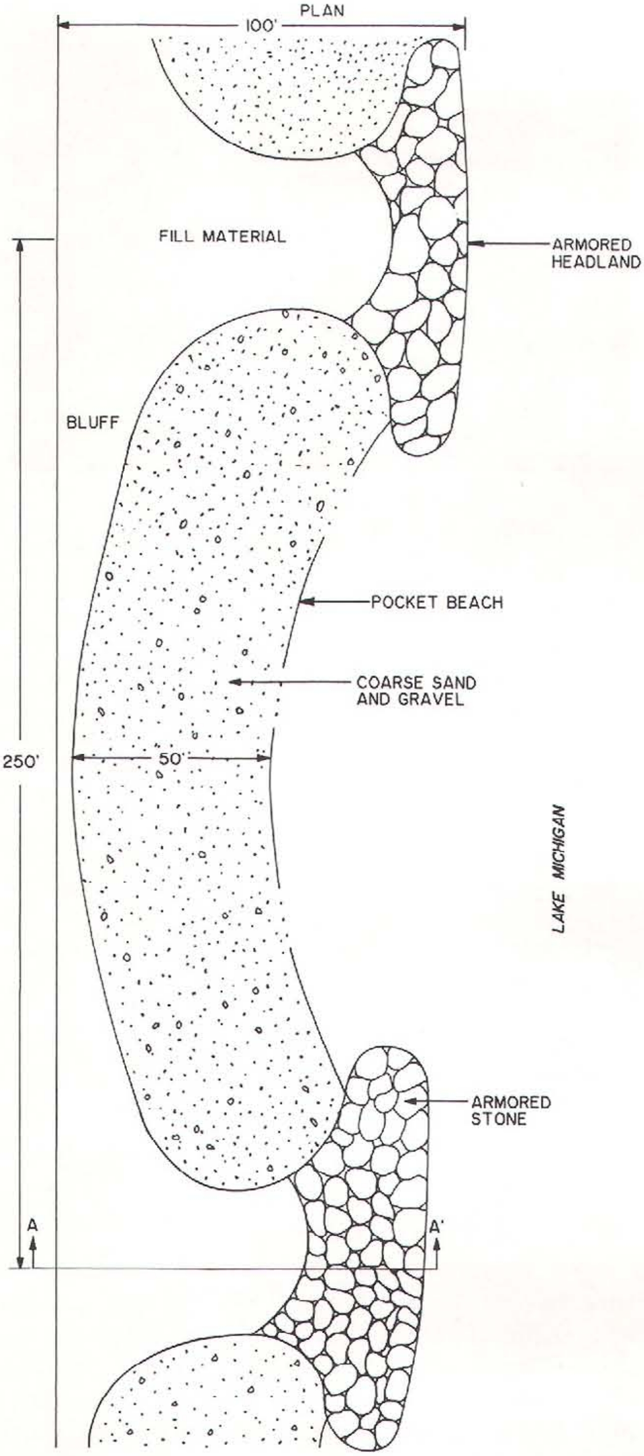
Source: Warzyn Engineering, Inc., and SEWRPC.

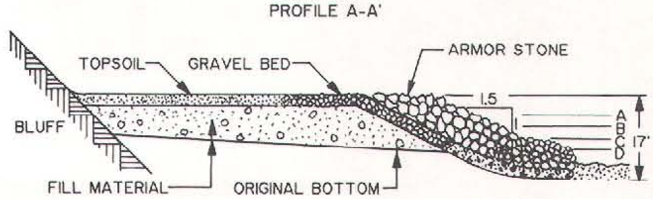
shape of the cobbles. The usability of cobble beaches installed primarily for erosion control can be enhanced by the placement of a one- to two-foot layer of gravel on top of the cobbles. Although the gravel layer would need re-nourishment, the stability of the cobble base and the perched beach design would reduce the need for replacement material.

To increase the effectiveness of the cobble beach and prevent the migration of the cobbles, a sill would be placed lakeward of the original shoreline to create a perched beach and a new shoreline. The sills could be constructed of precast concrete units, quarry stone, or steel sheet pile. As shown in Figure 112, the permeable, precast, steel fiber-reinforced concrete units typically weigh two tons each, and measure approximately four feet high, four feet wide, and six feet deep. They are usually set adjacent to each other in water typically from three to six feet deep. The structures are secured to each other with steel cables. The manufacturers of concrete units report that the sloped front and

Figure 110

TYPICAL ARMORED HEADLAND AND POCKET BEACH SYSTEM





LEGEND

DESIGN WATER LEVELS

- A DESIGN HIGH STILL WATER LEVEL PLUS WIND SETUP 584.5 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM
- B DESIGN HIGH STILL WATER LEVEL 582.9 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM
- C 1915 TO 1985 ANNUAL MEAN WATER LEVEL 579.5 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM
- D LOW WATER DATUM 578.1 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

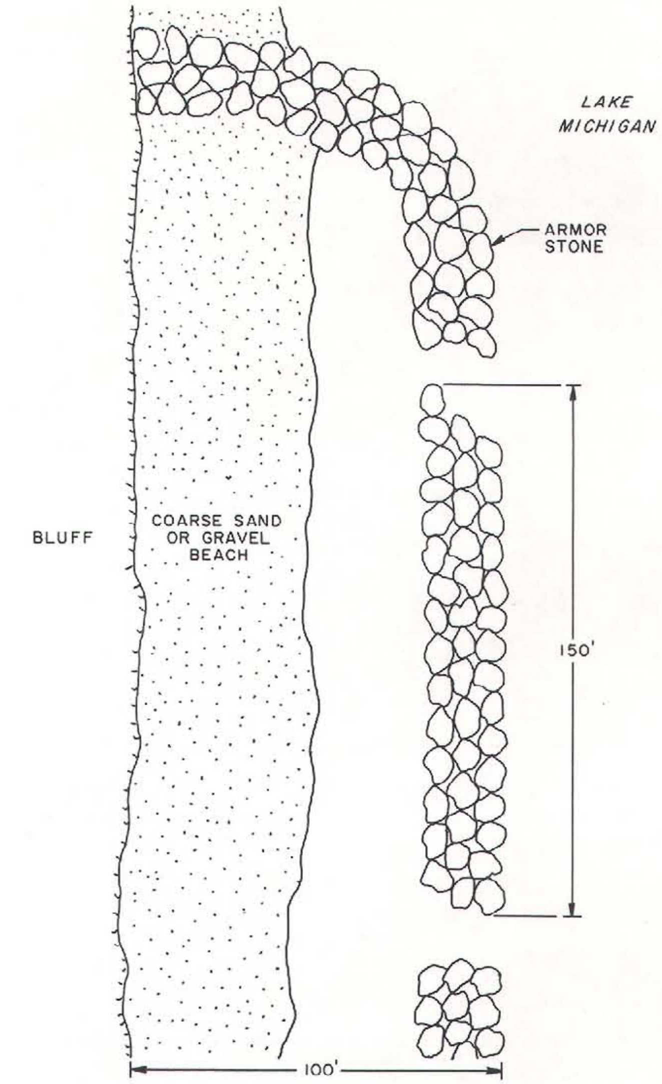
NOTE: THE DESIGN SPECIFICATIONS SHOWN HEREIN ARE FOR A TYPICAL STRUCTURE. THE DETAILED DESIGN OF SHORE PROTECTION MEASURES MUST BE BASED ON A DETAILED ANALYSIS OF WAVE CLIMATE, COST AND AVAILABILITY OF CONSTRUCTION MATERIAL, SPECIFIC GRAVITY AND QUALITY OF THE STONE, TYPE OF LAKEBED MATERIAL, AND EXISTING SHORELINE GEOMETRY.

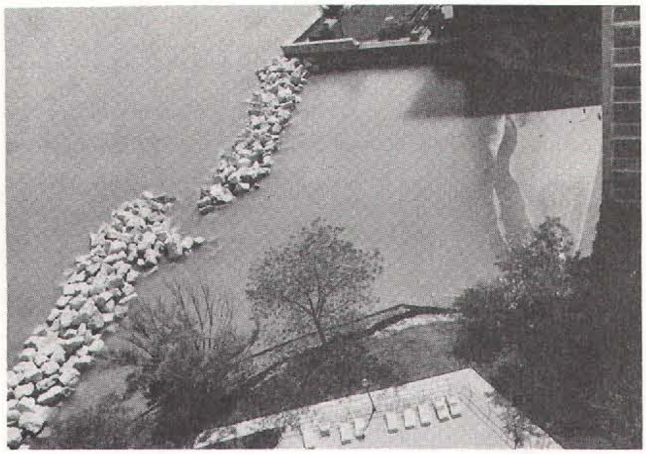
Source: Warzyn Engineering, Inc., and SEWRPC.

Figure 111

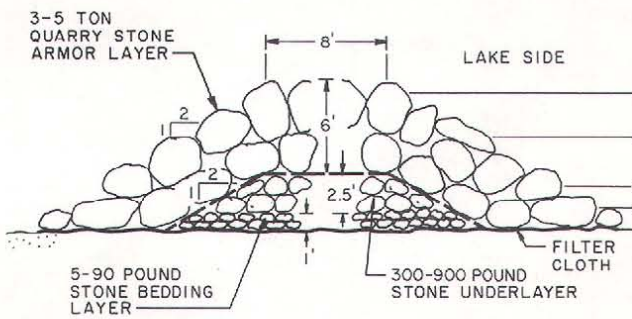
TYPICAL NEAR-SHORE STONE REEF WITH NOURISHED COARSE SAND AND GRAVEL BEACH

PLAN





PROFILE DETAIL



LEGEND

DESIGN WATER LEVELS

A DESIGN HIGH STILL WATER LEVEL PLUS WIND SETUP 584.5 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

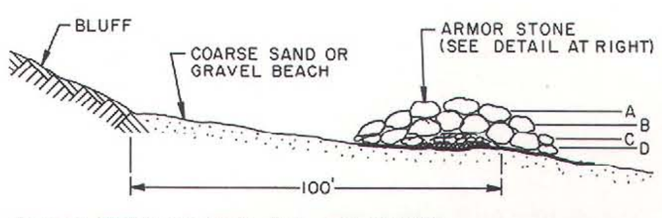
B DESIGN HIGH STILL WATER LEVEL 582.9 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

C 1915 TO 1985 ANNUAL MEAN WATER LEVEL 579.5 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

D LOW WATER DATUM 578.1 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

NOTE: THE DESIGN SPECIFICATIONS SHOWN HEREIN ARE FOR A TYPICAL STRUCTURE. THE DETAILED DESIGN OF SHORE PROTECTION MEASURES MUST BE BASED ON A DETAILED ANALYSIS OF WAVE CLIMATE, COST AND AVAILABILITY OF CONSTRUCTION MATERIAL, SPECIFIC GRAVITY AND QUALITY OF THE STONE, TYPE OF LAKEBED MATERIAL, AND EXISTING SHORELINE GEOMETRY.

PROFILE



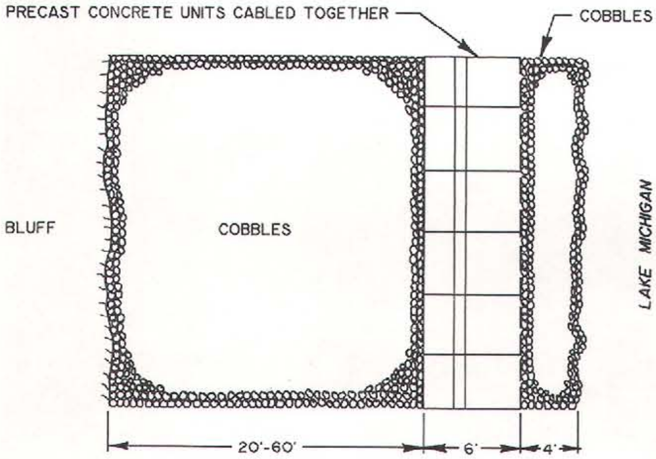
Source: STS Consultants, Ltd., and SEWRPC.

back profile, and tapered openings, temper wave energy but allow enough energy transmission to avoid scouring of the lakebed from wave reflection. Accretion of beach material has been reported both lakeward and landward of some pervious sills. Heavy construction equipment is required to install the structures. The capital cost of precast concrete units is approximately \$250 per lineal foot of shoreline. A sill constructed of quarry stone would have a capital cost of about \$250 per lineal foot of shoreline, and a sill constructed of sheet pile would have a capital cost of about \$600 per lineal foot of shoreline.

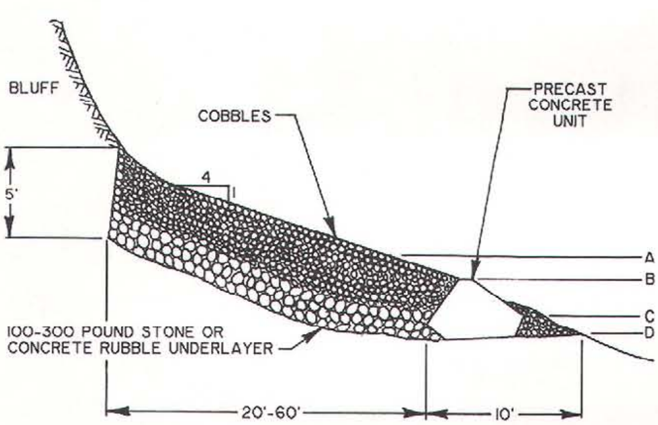
Figure 112

TYPICAL PERCHED COBBLE BEACH SYSTEM

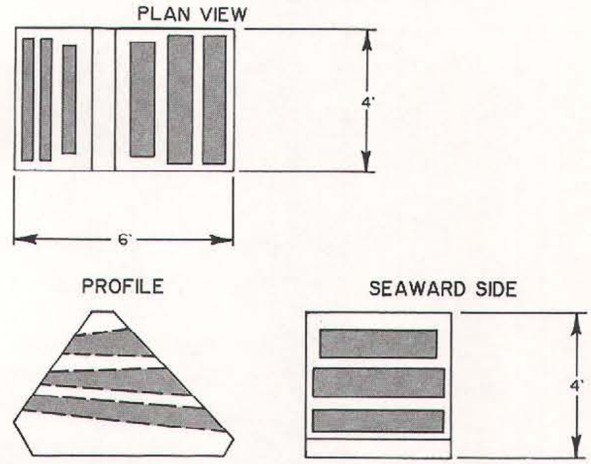
PLAN



PROFILE



TYPICAL PRECAST CONCRETE UNIT



LEGEND

DESIGN WATER LEVELS

A DESIGN HIGH STILL WATER LEVEL PLUS WIND SETUP 584.5 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

B DESIGN HIGH STILL WATER LEVEL 582.9 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

C 1915 TO 1985 ANNUAL MEAN WATER LEVEL 579.5 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

D LOW WATER DATUM 578.1 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

NOTE: THE DESIGN SPECIFICATIONS SHOWN HEREIN ARE FOR A TYPICAL STRUCTURE. THE DETAILED DESIGN OF SHORE PROTECTION MEASURES MUST BE BASED ON A DETAILED ANALYSIS OF WAVE CLIMATE, COST AND AVAILABILITY OF CONSTRUCTION MATERIAL, SPECIFIC GRAVITY AND QUALITY OF THE STONE, TYPE OF LAKEBED MATERIAL, AND EXISTING SHORELINE GEOMETRY.

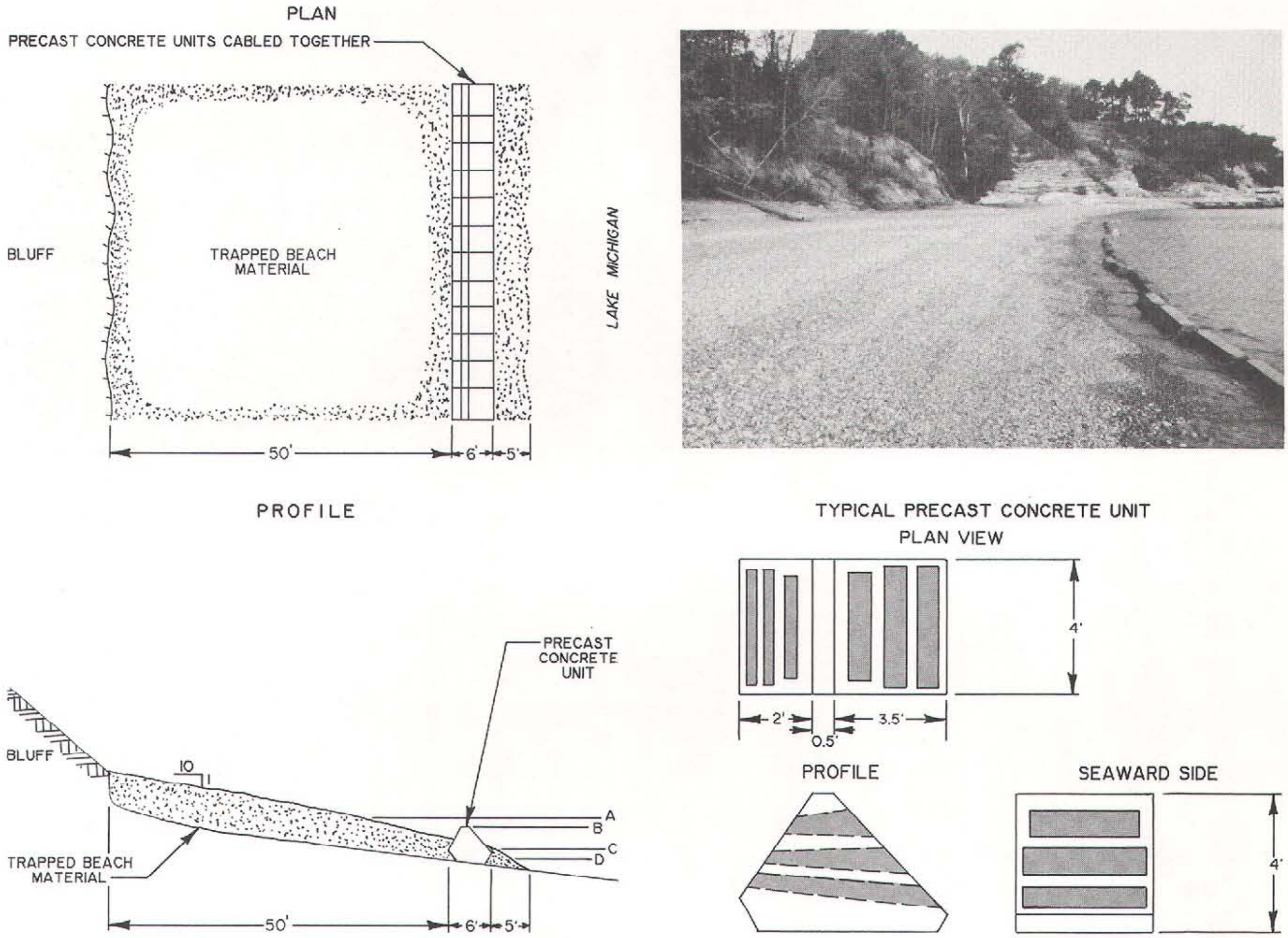
Source: Great Lakes Environmental Marine, Ltd., and SEWRPC.

The cobble beach system, including a concrete unit or quarry stone sill, would entail a total capital cost of $350 to $400 per lineal foot of shoreline. Annual maintenance costs, which depend upon the need to re-nourish the supply of cobbles, would approximate $20 per foot.

Near-Shore Pervious Concrete Sill: Where an existing natural beach of sand or coarser material is present, the erosion of the beach can be at least partially controlled by the installation of a pervious concrete sill in the surf zone, as shown in Figure 113. Precast concrete units similar to those described above for perched cobble beaches are currently placed parallel to the shore and connected with steel cables, typically 40 to 60 feet offshore in water two to six feet deep.

Figure 113

TYPICAL PERVIOUS CONCRETE SILL



LEGEND

DESIGN WATER LEVELS

A DESIGN HIGH STILL WATER LEVEL PLUS WIND SETUP 584.5 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

B DESIGN HIGH STILL WATER LEVEL 582.9 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

C 1915 TO 1985 ANNUAL MEAN WATER LEVEL 579.5 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

D LOW WATER DATUM 578.1 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

NOTE: THE DESIGN SPECIFICATIONS SHOWN HEREIN ARE FOR A TYPICAL STRUCTURE. THE DETAILED DESIGN OF SHORE PROTECTION MEASURES MUST BE BASED ON A DETAILED ANALYSIS OF WAVE CLIMATE, COST AND AVAILABILITY OF CONSTRUCTION MATERIAL, SPECIFIC GRAVITY AND QUALITY OF THE STONE, TYPE OF LAKEBED MATERIAL, AND EXISTING SHORELINE GEOMETRY.

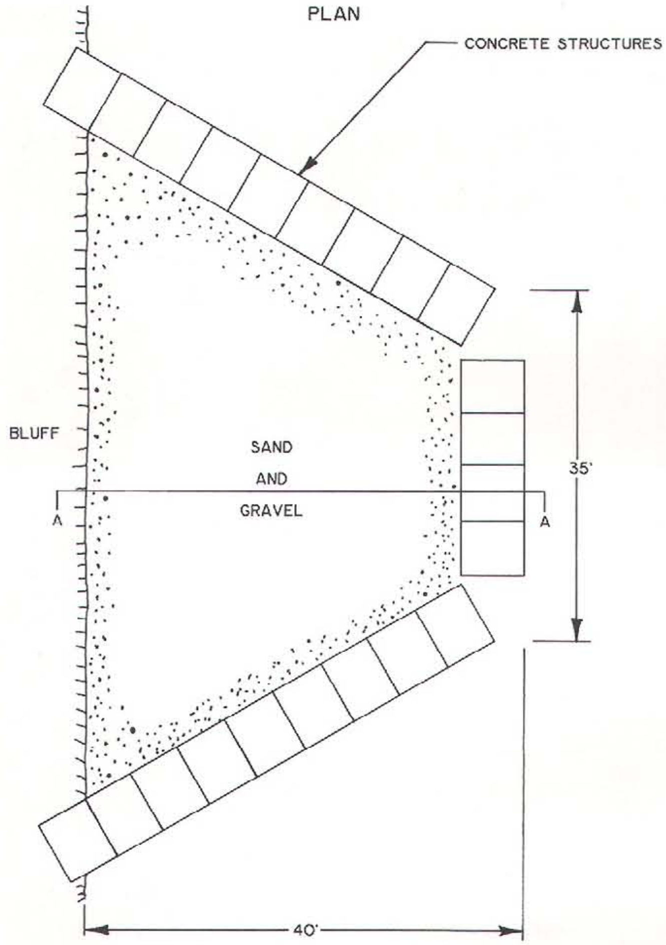
Source: Great Lakes Environmental Marine, Ltd., and SEWRPC.

Wave attack on the shore and bluff is reduced by the sill's attenuating effects on the waves when they are still offshore. The sill trips and slows the waves. In addition to tempering storm surge and backwash, the sill system enhances the deposition of sediment from the littoral drift along shore. Accretion of sediment may occur both landward and lakeward of the sill. The sill would be the most effective in a shallow, low-wave-energy environment which contains a substantial amount of littoral drift material. Heavy construction equipment is required to install the structures. A disadvantage of a sill is that it interferes with small boat navigation near the shore.

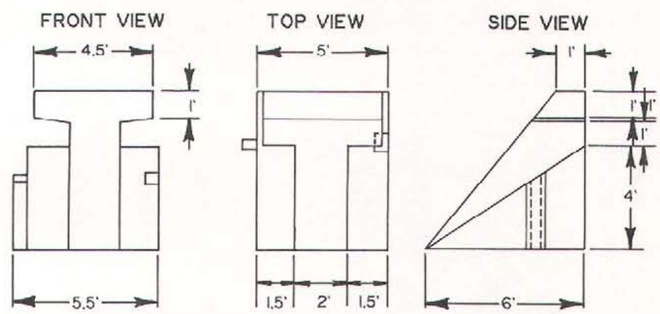
The near-shore pervious concrete sill would entail a total capital cost of $200 to $300 per lineal foot of shoreline. Annual maintenance costs would approximate $10 per foot.

Figure 114

TYPICAL CONCRETE BEACH CONTAINMENT SYSTEM WITH NOURISHED COARSE SAND AND GRAVEL BEACH



TYPICAL CONCRETE BEACH CONTAINMENT UNIT



LEGEND

DESIGN WATER LEVELS

A DESIGN HIGH STILL WATER LEVEL PLUS WIND SETUP 584.5 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

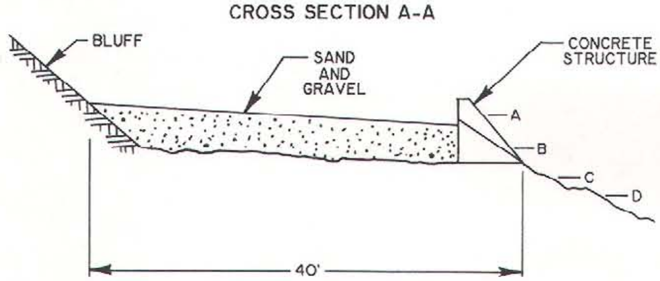
B DESIGN HIGH STILL WATER LEVEL 582.9 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

C 1915 TO 1985 ANNUAL MEAN WATER LEVEL 579.5 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

D LOW WATER DATUM 578.1 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

NOTE: THE DESIGN SPECIFICATIONS SHOWN HEREIN ARE FOR A TYPICAL STRUCTURE. THE DETAILED DESIGN OF SHORE PROTECTION MEASURES MUST BE BASED ON A DETAILED ANALYSIS OF WAVE CLIMATE, COST AND AVAILABILITY OF CONSTRUCTION MATERIAL, SPECIFIC GRAVITY AND QUALITY OF THE STONE, TYPE OF LAKEBED MATERIAL, AND EXISTING SHORELINE GEOMETRY.

CROSS SECTION A-A



Source: NSP Associates and SEWRPC.

Manufactured Concrete Beach Containment Systems: Different types of manufactured concrete structures can be used to contain a beach area. Large, steel-reinforced concrete blocks, being about six feet high and weighing about six or seven tons each, can be placed offshore side by side in water three to four feet deep, as shown in Figure 114. The blocks can also be placed along the side of the contained beach. The structure is intended to allow waves to run up along the face and over the top, trapping the coarser, waterborne particles behind the blocks. The beach could also be artificially nourished. Toe protection and a filter layer could help to prevent scouring and the uneven settling of the blocks.

The advantages of the concrete beach containment system are that the structures directly protect the beach material from wave action, and they may—because of the height of the structures—allow the development of a beach in deeper water environments. A disadvantage is the poor suitability of the beaches for swimming or wading because of the presence of the concrete structures. The concrete structures may also be subject to displacement, which would result in the loss of beach material.

The capital cost of a concrete beach containment system would be approximately $250 per lineal foot of shoreline, with an additional cost ranging up to $115 per lineal foot of shoreline to artificially nourish the beach by shore, or up to $500 per lineal foot of shoreline to artificially nourish the beach by barge. The annual operation and maintenance cost would be $15 to $50 per lineal foot, depending primarily upon the need for periodic re-nourishment 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 be used to contain large, nourished sand beaches. The structures are generally constructed of stone, although some designs use rock-filled concrete caissons, cellular sheet piles, timber cribs, and floating devices. One advantage of any near-shore, or offshore, protection system is that the structures are positioned off the existing shoreline, thereby providing recreational benefits while protecting the shore from erosion. Breakwaters, if properly designed, provide effective protection during periods of widely fluctuating water levels. Breakwaters can be designed to provide substantial protection without becoming complete barriers to littoral transport. A major disadvantage of a breakwater is that a large quantity of material must be deposited in relatively deep water. Heavy equipment mounted on barges is normally required for installation and continued maintenance. Because breakwaters may extend well above the water, they may interfere with the scenic view of the horizon for beach users.

Construction of an offshore breakwater along the Lake Michigan shoreline of northern Milwaukee County would entail a capital cost of $1,000 to $2,000 per lineal foot of shoreline. Average annual maintenance costs would approximate $20 to $50 per lineal foot.

Described below are five alternative breakwater designs: a rubblemound breakwater, a caisson breakwater, a sheet pile breakwater, a timber crib breakwater, and a floating breakwater.

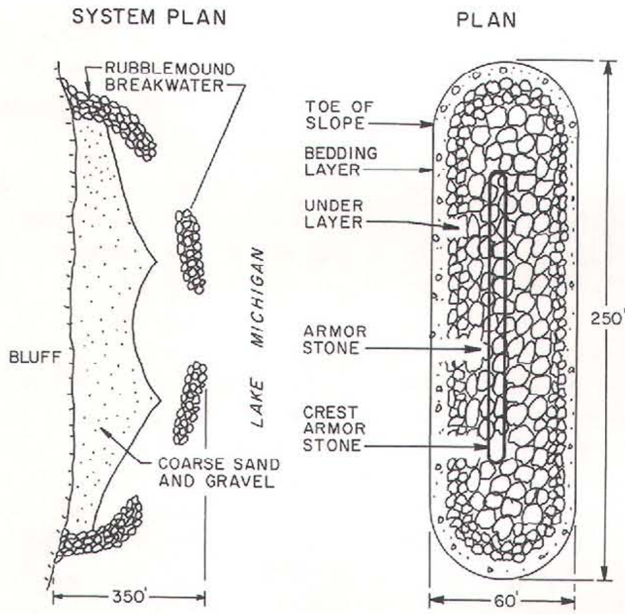
Rubblemound Breakwater: A rubblemound breakwater is the most common type of breakwater in the Great Lakes. The structure, as shown in Figure 115, is usually constructed of several layers of quarry stone, rubble, or concrete units. In a typical rubblemound breakwater, the core of the breakwater is constructed of small size stone, each weighing 1 to 50 pounds. Armor stone forms the outer layer of the breakwater. An intermediate layer acts as a filter layer to prevent the inner core materials from being washed out through the larger armor stone. Depending on the water depth and on the subsurface conditions in the area of the breakwater structure, a filter cloth is sometimes used to prevent bottom scouring and settlement of the structure.

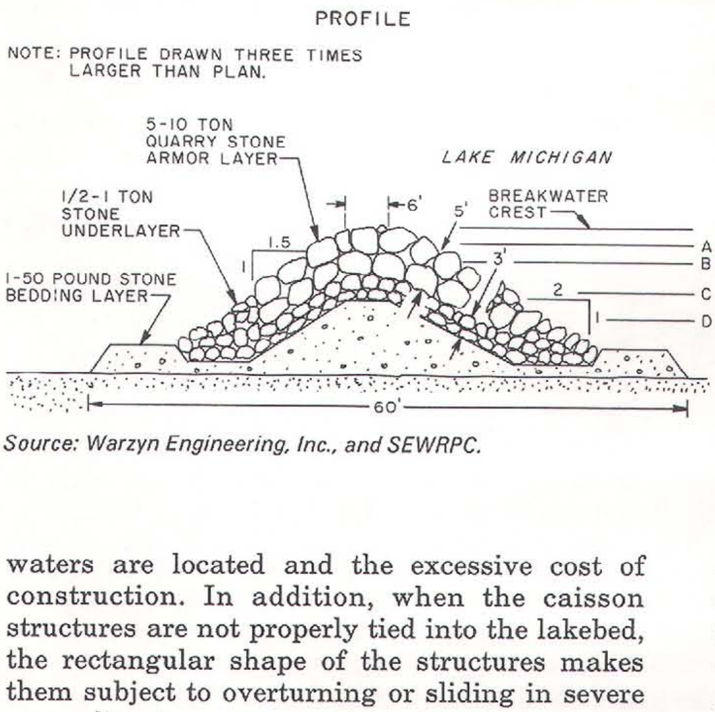
The rubblemound breakwater is intended to prevent or reduce the transmission of wave energy behind it by absorbing much of the energy and reflecting some of the remaining energy back to the main water body. If rubble breakwaters are too porous, they allow a high percentage of longer period wave energy to pass through, causing excessive wave action behind the structure.

Caisson Breakwater: A caisson breakwater, as shown in Figure 116, 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. Rip-rap protection is then placed along the toe of the structure to prevent tilting or overturning due to scour. Caisson breakwaters were used extensively in the Great Lakes, including at the Port of Milwaukee, during the early 1900's for construction of commercial harbors. At that time, the caissons provided distinct construction advantages in 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 because of the relatively shallow water in which the break-

Figure 115

TYPICAL SEGMENTED RUBBLEMOUND BREAKWATER SYSTEM





LEGEND

DESIGN WATER LEVELS

A DESIGN HIGH STILL WATER LEVEL PLUS WIND SETUP 584.5 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

B DESIGN HIGH STILL WATER LEVEL 582.9 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

C 1915 TO 1985 ANNUAL MEAN WATER LEVEL 579.5 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

D LOW WATER DATUM 578.1 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

NOTE: THE DESIGN SPECIFICATIONS SHOWN HEREIN ARE FOR A TYPICAL STRUCTURE. THE DETAILED DESIGN OF SHORE PROTECTION MEASURES MUST BE BASED ON A DETAILED ANALYSIS OF WAVE CLIMATE, COST AND AVAILABILITY OF CONSTRUCTION MATERIAL, SPECIFIC GRAVITY AND QUALITY OF THE STONE, TYPE OF LAKEBED MATERIAL, AND EXISTING SHORELINE GEOMETRY.

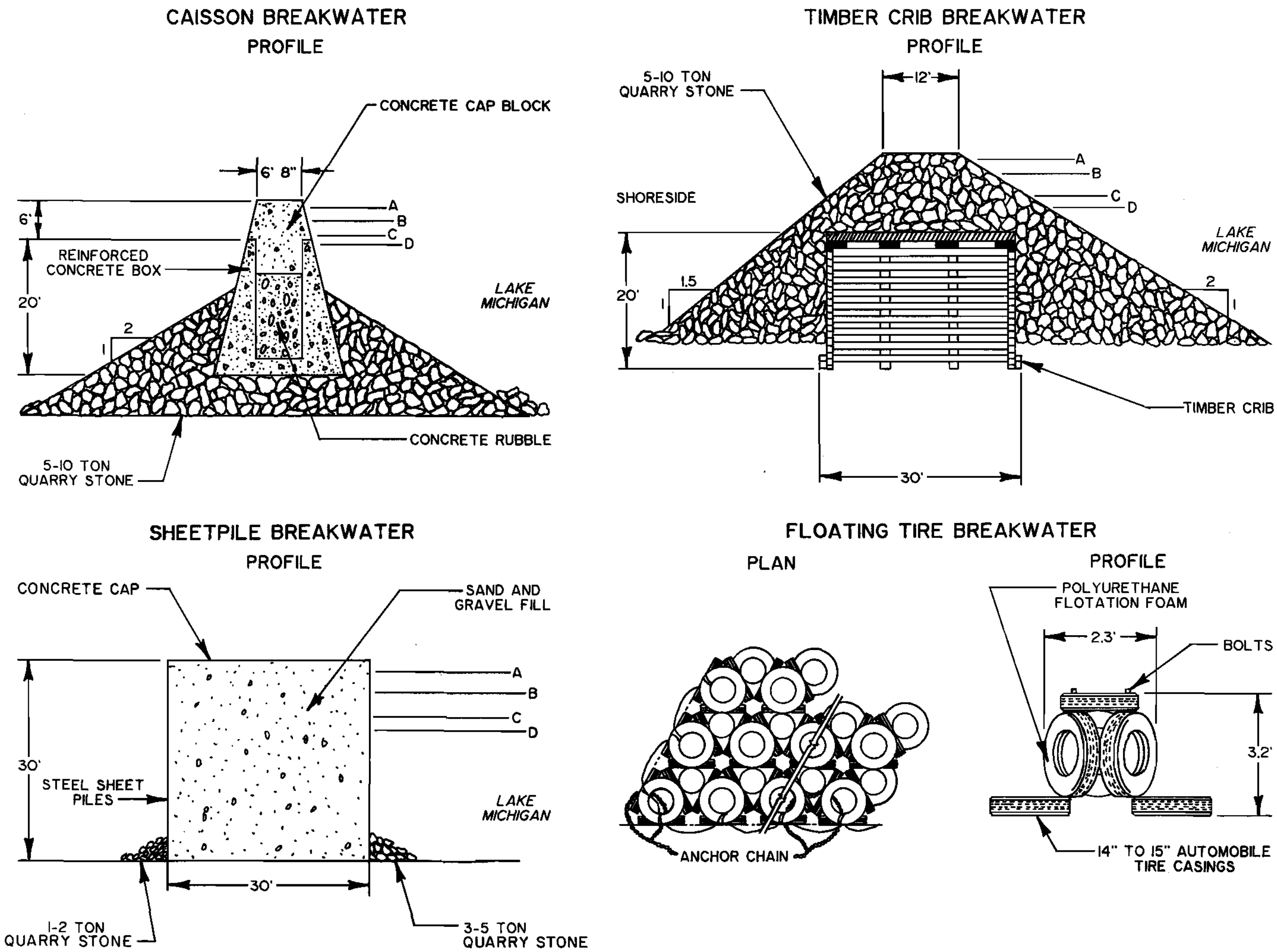
Source: Warzyn Engineering, Inc., and SEWRPC.

waters are located and the excessive cost of construction. In addition, when the caisson structures are not properly tied into the lakebed, the rectangular shape of the structures makes them subject to overturning or sliding in severe wave climates.

Sheet Pile Breakwater: Breakwaters can also be constructed of steel sheet piles. Many variations are found in the design of sheet pile breakwaters. One design provides a series of circular cells constructed of steel sheet piling and filled with either stone or rubble and capped with concrete, as shown in Figure 116. 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. Rip-rap toe protection is required along the base of all sheet pile breakwaters to prevent scouring. Sheet pile breakwater structures provide navigable water up to their edge. In addition to a high initial cost, a disadvantage of the steel sheet pile breakwater is that the face of the structure does not absorb wave energy and, if improperly located, may cause severe reflected wave conditions.

Figure 116

MISCELLANEOUS ALTERNATIVE TYPES OF BREAKWATERS



LEGEND

DESIGN WATER LEVELS

A DESIGN HIGH STILL WATER LEVEL PLUS WIND SETUP 584.5 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

B DESIGN HIGH STILL WATER LEVEL 582.9 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

C 1915 TO 1985 ANNUAL MEAN WATER LEVEL 579.5 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

D LOW WATER DATUM 578.1 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

NOTE: THE DESIGN SPECIFICATIONS SHOWN HEREIN ARE FOR A TYPICAL STRUCTURE. THE DETAILED DESIGN OF SHORE PROTECTION MEASURES MUST BE BASED ON A DETAILED ANALYSIS OF WAVE CLIMATE, COST AND AVAILABILITY OF CONSTRUCTION MATERIAL, SPECIFIC GRAVITY AND QUALITY OF THE STONE, TYPE OF LAKEBED MATERIAL, AND EXISTING SHORELINE GEOMETRY.

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

Timber Crib Breakwater: A fourth type of breakwater is known as a timber crib breakwater and is illustrated in Figure 116. 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 1900's, timber cribs were frequently used for the construction of harbors, including for the Port of Milwaukee. The advantages of using timber cribs for construction of a breakwater is that they provide navigable water space immediately adjacent to the structure and can also accommodate a walkway. In addition, timber crib structures are substantially more effective at absorbing wave energy than 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. Timber cribs should be designed to remain submerged, and covered by armor stone, as shown in Figure 116.

Floating Breakwater: Floating breakwaters, as shown in Figure 116, are constructed of buoyant materials such as logs, hollow concrete boxes, and rubber tires. Floating breakwaters have not been able to 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 water depths. However, since floating breakwaters are effective only against small to moderate, short-period waves, they could be used only as supplementary protection in northern Milwaukee County. Most floating breakwaters would need to be removed during the winter to prevent ice damage to the structure.

Offshore Islands and Peninsulas: Islands and peninsulas lying 250 to 1,500 feet offshore could be constructed to provide substantial protection from wave action while creating additional recreational land. The islands or peninsulas, as shown in Figure 117, would be constructed of fill material consisting of rubble, soil, or tunnel construction debris. The fill material should be protected from wave action by the use of a revetment or an armored headland and pocket beach system. The offshore islands or peninsulas, like offshore breakwaters, dissipate deep-water 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 deep-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, assuming that fill material is available at a minimal cost. Average annual maintenance costs approximate $20 to $40 per lineal foot.

Bluff Slope Stabilization

In Chapter III of this report, 22 bluff analysis sections covering 15,160 feet, or 39 percent of the total study area 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.

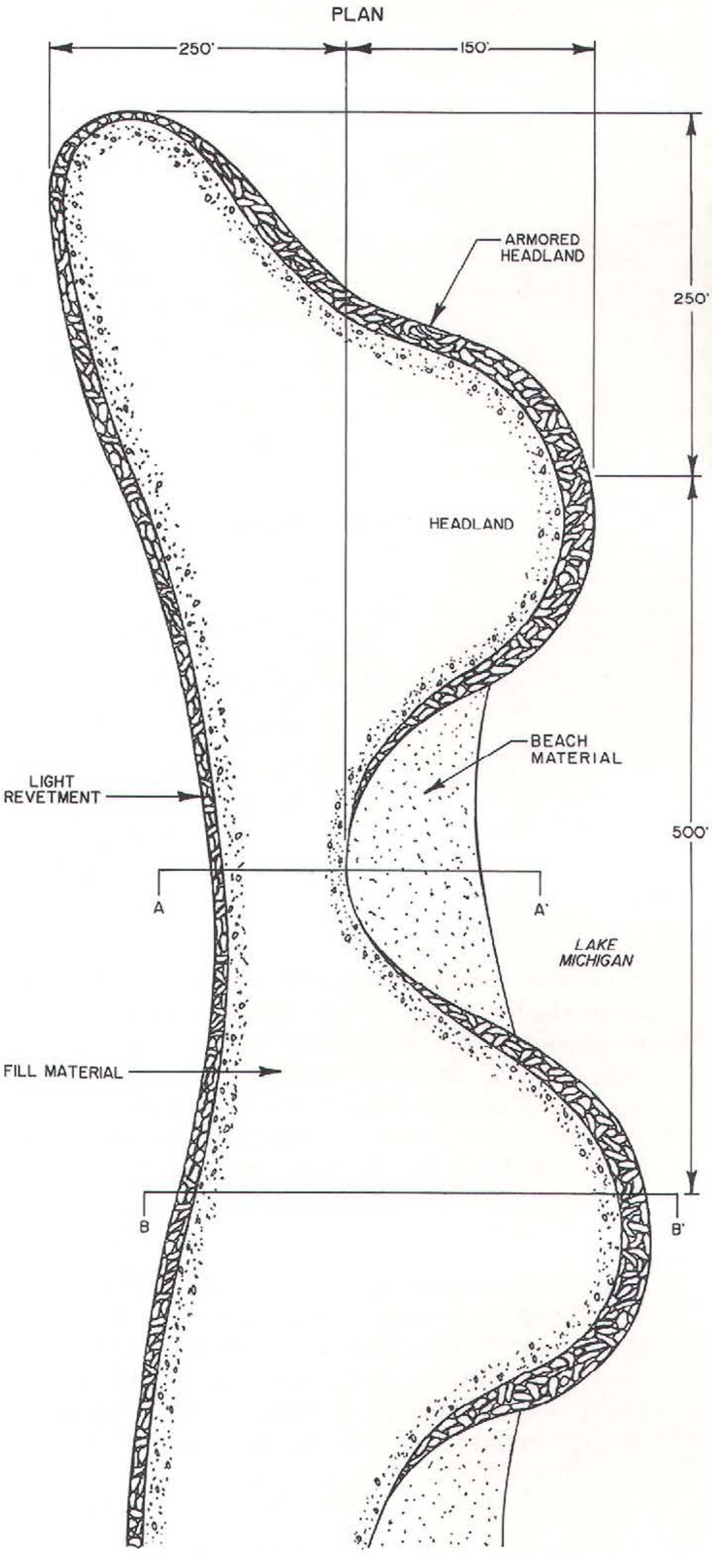
Bluff Slope Regrading: Regrading the bluff slope to a stable angle was indicated for 18 bluff analysis sections covering 10,420 feet, or 27 percent of the study area shoreline. Bluff analysis sections identified as needing bluff slope regrading were those in which other economically feasible measures would not effectively stabilize the slopes. A primary advantage of bluff slope regrading is that further bluff recession is prevented—if bluff toe protection and surface- and groundwater drainage are also provided where needed. Slope regrading will also provide structural stability to the bluff toe protection measures, preventing them from being buried by bluff material.

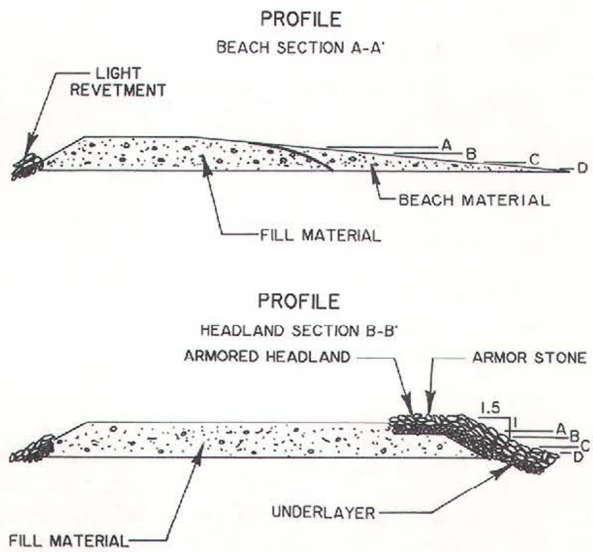
The disadvantage of bluff slope stabilization is that the natural aesthetic properties and drainage characteristics of the bluff are disrupted. In addition, there are problems, albeit temporary, related to the truck and heavy equipment traffic moving to and from the site, as well as to the impacts of the dust and noise at the construction site itself.

Four alternative methods for bluff slope regrading, as shown in Figure 118 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.

Figure 117

TYPICAL OFFSHORE ISLAND OR PENINSULA





LEGEND

DESIGN WATER LEVELS

A DESIGN HIGH STILL WATER LEVEL PLUS WIND SETUP 584.5 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

B DESIGN HIGH STILL WATER LEVEL 582.9 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

C 1915 TO 1985 ANNUAL MEAN WATER LEVEL 579.5 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

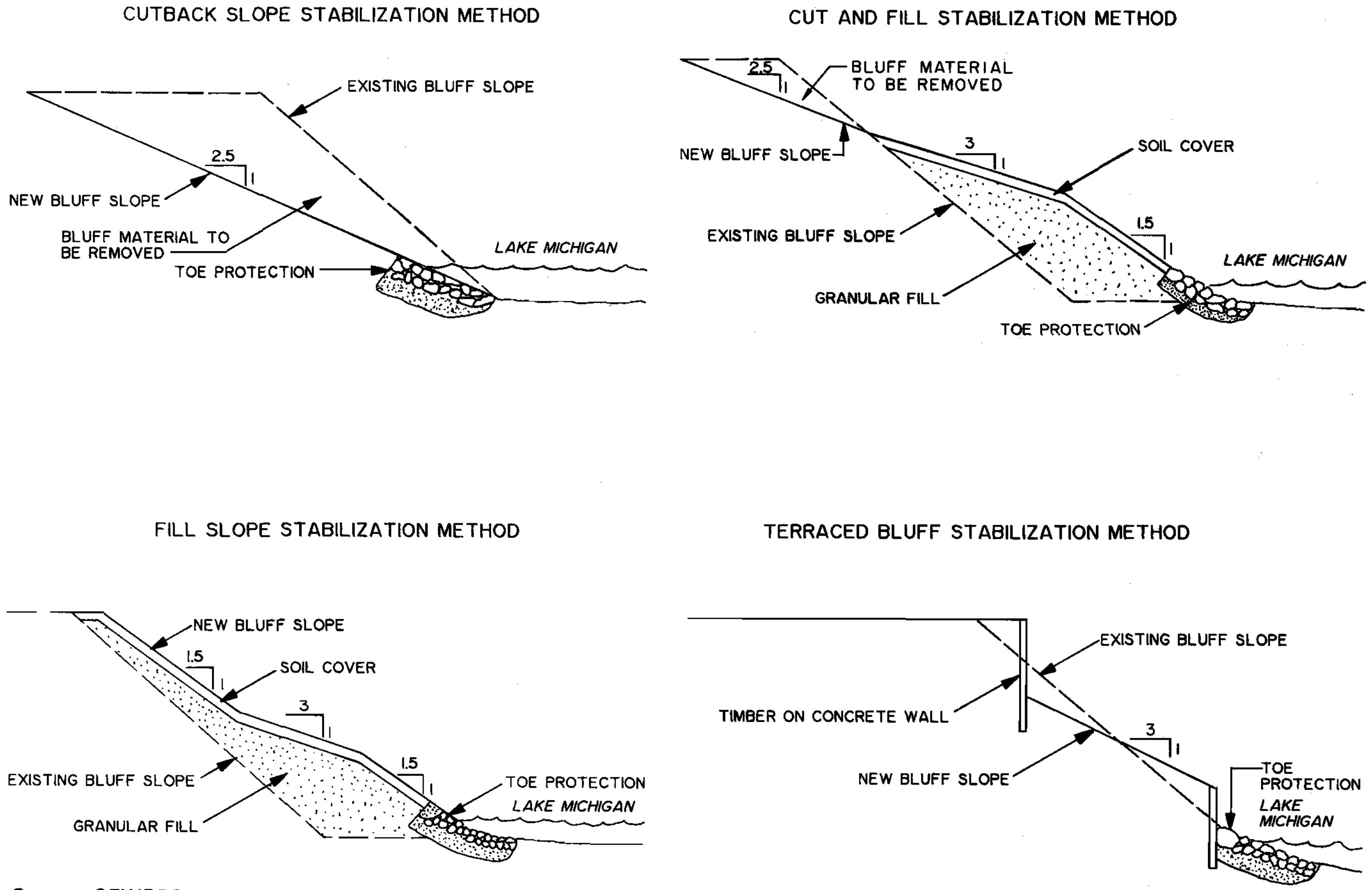
D LOW WATER DATUM 578.1 FEET ABOVE NATIONAL GEODETIC VERTICAL DATUM

NOTE: THE DESIGN SPECIFICATIONS SHOWN HEREIN ARE FOR A TYPICAL STRUCTURE. THE DETAILED DESIGN OF SHORE PROTECTION MEASURES MUST BE BASED ON A DETAILED ANALYSIS OF WAVE CLIMATE, COST AND AVAILABILITY OF CONSTRUCTION MATERIAL, SPECIFIC GRAVITY AND QUALITY OF THE STONE, TYPE OF LAKEBED MATERIAL, AND EXISTING SHORELINE GEOMETRY.

Source: Warzyn Engineering, Inc., and SEWRPC.

Figure 118

ALTERNATIVE METHODS OF BLUFF SLOPE STABILIZATION



Source: SEWRPC.

Cutback Method: Bluff slope regrading can be accomplished by using earth-moving equipment to regrade the face of the slope to a flatter, more stable profile, as shown in Figure 118. As already noted, a bluff slope of one on two and one-half will usually provide a stable bluff slope in the study area. The cutback method can be used only in areas where the houses concerned are located a sufficient distance from the edge of the bluff. Within the portion of the study area recommended for bluff slope regrading, three bluff analysis sections covering 900 feet, or 2 percent of the study area shoreline, should be suitable for application of the cutback method. Regrading the bluff slopes to a stable angle within these sections would require cutting and removing about 100,000 cubic yards of bluff material. 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. Up to two acres of land at the top of the bluff could be lost by cutting back the slopes within the three bluff analysis sections.

Bluff slope regrading using the cutback method would entail a capital cost of \$100 to \$150 per lineal foot of shoreline. Maintenance costs are assumed to be about \$15 per lineal foot during the first three years following bluff slope regrading, primarily for the maintenance of a new vegetative cover.

Fill Method: Bluff slope regrading can also be accomplished by transporting soil, concrete rubble, and other clean fill from an outside source and placing it on the face of the bluff to provide a more stable profile. Filling will likely be required for those bluff analysis sections where the houses are located close to the edge of the bluff. The fill materials, as shown in Figure 118, should be granular. Fine-grained, clay-type materials are not suitable for fill material in those areas susceptible to groundwater drainage problems. Depending on the type of material used for filling, a slightly steeper angle—often approximating 35 degrees—may be utilized for portions of the regraded bluff slopes. Slopes constructed of fill material are normally terraced, or contain compound slopes. Filling should begin at the slope bottom, and some bluffs may need to be filled only along the lower portions of the slope. The fill method could be used for all the 16 bluff analysis sections recommended for bluff slope regrading within the study area. Depending upon the portions of the slopes that needed to be filled, filling all 16 bluff analysis sections could require from 0.5 to 2.0 million cubic yards of fill material. Soil placement, seeding, and mulching would be required to develop a vegetative cover. Adequate toe protection would also be provided to maintain and protect the fill material.

The primary benefit of using the fill method is that land at the top of the bluff is not removed, which is particularly advantageous in areas where houses are located within 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 $150 to $250 per lineal foot of shoreline. Maintenance costs are assumed to be $10 to $15 per lineal foot during the first three years following bluff slope regrading, primarily for the maintenance of a new vegetative cover.

Cut and Fill Method: A combination of cutting the upper unstable portion of the bluff slope, and placing that material—along with additional fill material, if necessary—at the base of the bluff slope can also provide a stable bluff slope. The cut and fill method is also shown in Figure 118. The cut and fill method is limited in use to those areas in which houses are located at least 50 feet from the edge of the bluff slope. Soil placement, seeding, and mulching are required to develop a protective vegetative cover; and adequate toe protection should be provided to maintain the regraded bluff slope.

The advantage of using the cut and fill method over the cutback method is that less land is lost at the top of the bluff slope. The majority of the material needed for filling is already at the site, and, compared to the total fill method, less fill material would extend out into the lake.

Bluff slope regrading using the cut and fill method would entail a capital cost of $100 to $200 per lineal foot of shoreline. Maintenance costs would range from $10 to $15 per lineal foot during the first three years following bluff slope regrading, primarily for the maintenance of a vegetative cover.

Terracing Method: Slope stabilization can also be provided by the placement of a series of vertical retaining walls within the regraded bluff slope, as shown in Figure 118. The retaining walls may be constructed of stone, timber, interlocking concrete blocks, steel sheet pile, or gabions. The bluff slope between the retaining walls is regraded to a slope of one on three or flatter, and vegetated. The terracing method can provide improved access to the shoreline if a suitable walkway is provided. Depending upon the design of the terrace system, less bluff material may need to be removed at the top of the bluff than under the cutback method, or under the cut and fill method.

The primary disadvantages of the terracing method are its relatively high cost, and construction difficulty. Construction of a bluff slope that is entirely terraced may entail a capital cost of $1,000 to $3,500 per lineal foot of shoreline. Annual maintenance costs would be $10 to $15 per lineal foot during the first three years following bluff slope regrading, primarily for the maintenance of a new vegetative cover. Because of this relatively high cost, it is most feasible to construct terraces on only a portion, i.e., the top one-third, of the bluff slope.

Groundwater Drainage: Groundwater drainage was indicated to enhance slope stability in six bluff analysis sections, covering 6,160 feet, or 16 percent, of the study area shoreline. The groundwater conditions and stratigraphy assumed within these marginal or unstable sections was such that lowering the level of the water table may be expected to significantly help stabilize the bluff slopes. Detailed, site-specific analyses of the groundwater conditions must be conducted at the preliminary engineering phase to affirm the feasibility of groundwater drainage systems. Groundwater drainage is also recommended to be considered during, and following, the construction of fill projects to prevent excess hydrostatic pressures caused by the compression of saturated soils by the weight of the fill material and the blocking of seepage paths. Drainage systems require relatively minor maintenance and should not limit the use of the shoreline. A groundwater drainage system would also not disturb the vegetative cover on the bluff slope, nor require changing the slope geometry. A limitation of groundwater drainage as a slope stabilization control measure is that drainage is usually economically feasible only in granular layers. The removal of water within clay glacial till layers is usually too costly and difficult. Three alternative groundwater drainage systems are described below: horizontal drains, vertical drains, and trench drains.

Horizontal Drains: A horizontal drain is a small diameter boring drilled into the face of the bluff slope on a 5 to 10 percent grade and fitted with a perforated pipe. As shown in Figure 119, a system of collector pipes or ditches is provided to carry the collected water to the base of the bluff or to a suitable outlet. A horizontal drainage system is most effective in layers of granular material containing sand and gravel. Drains are usually spaced across the face of the bluff slope at suitable intervals based on the anticipated flow rates and soil permeability.

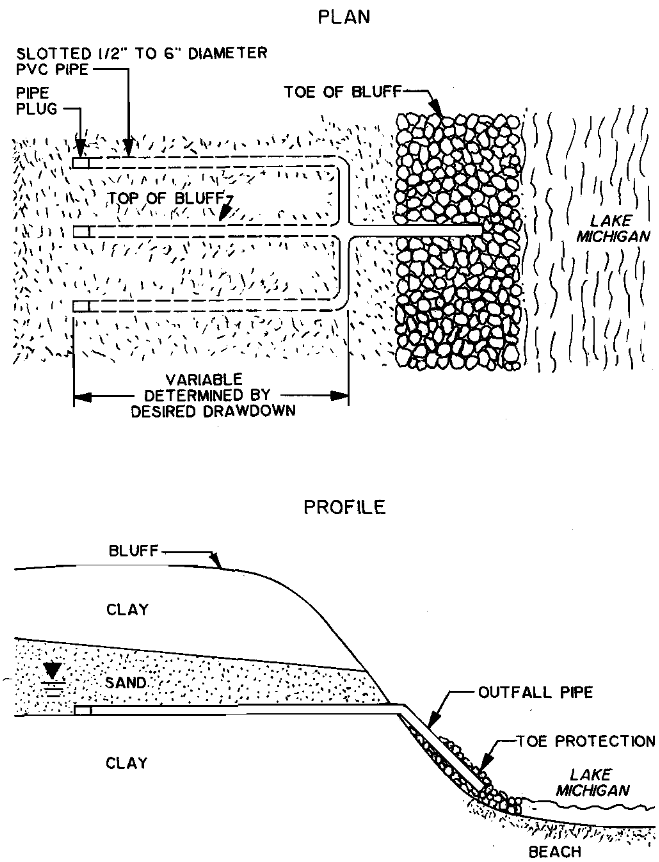
Advantages of a horizontal drain system are that the system drains by gravity, and requires relatively little maintenance. The primary disadvantage of the system is that access to the base of the bluff to install the drains is often difficult.

Construction of a horizontal drain system to lower the level of groundwater would entail a capital cost of $30 to $75 per lineal foot of shoreline. The annual operation and maintenance cost would range from $5.00 to $10 per lineal foot.

Vertical Drains: A vertical drain, or well, usually consists of an 18- to 36-inch-diameter boring drilled vertically from the top of the bluff into the water-bearing strata. Water can be either

Figure 119

HORIZONTAL DRAINAGE SYSTEM

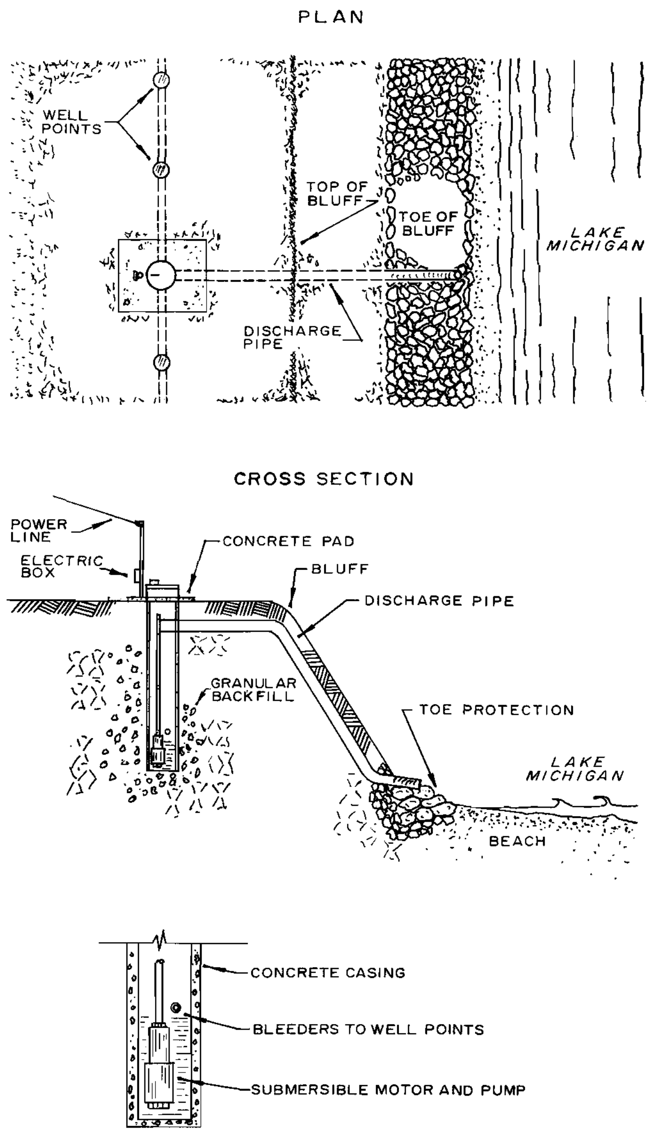


Source: Owen Ayres & Associates, Great Lakes Shore Erosion Protection, Structural Design Examples, 1978, and SEWRPC.

pumped from the well, or tapped with a gravity outlet, as shown in Figure 120. Gravity-drained vertical wells can be connected to horizontal drains which carry the collected water out of the bluff to a safe point of disposal. Water pumped from a vertical well can be discharged to the base of the bluff or to a suitable surface water outlet. Unlike most horizontal drains, vertical drains can be designed to drain several water-bearing strata separated by impermeable layers. Detailed geotechnical analyses should be conducted in the preliminary engineering phase to determine the necessary location, spacing, depth, and pumping rate of the well points. Under favorable conditions, relatively large amounts of water can be pumped from the wells to lower the groundwater table. In addition, access to install the drains is generally not a problem because vertical drains are installed from the top of the bluff. Disadvantages of this system are that the wells must be pumped

Figure 120

VERTICAL DRAINAGE SYSTEM



Source: SEWRPC.

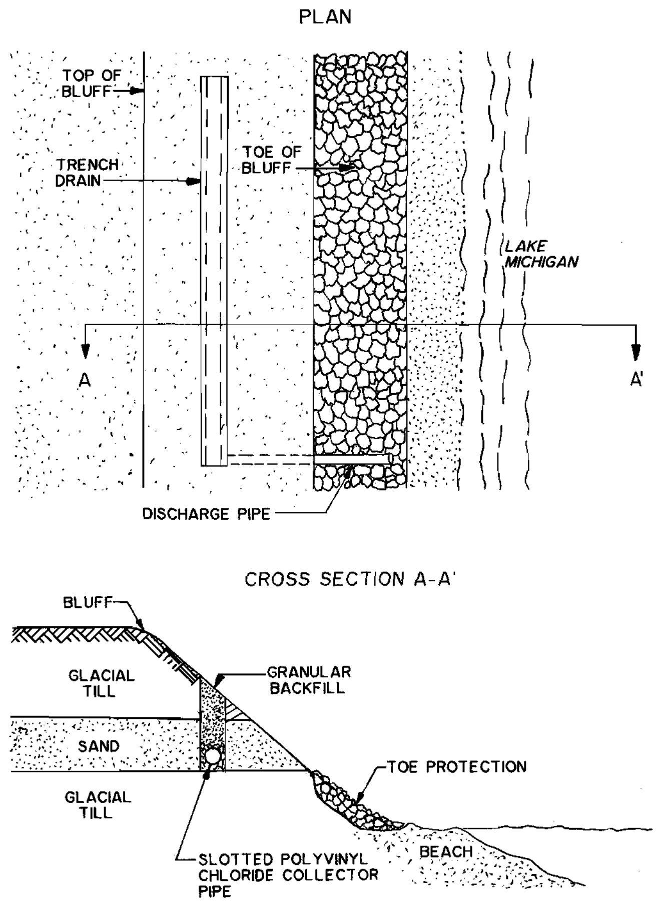
continuously to maintain the lower water table, and substantial maintenance of the wells and pumps may be required.

Construction of a vertical drain system would entail a capital cost of $50 to $150 per lineal foot of shoreline. The annual maintenance cost would range up to $20 per lineal foot.

Trench Drains: The purpose of a trench drain is to intercept and divert shallow seepage. A typical design consists of a narrow trench, dug parallel to the edge of the bluff, in which a perforated collector pipe is installed. The pipe is connected to a discharge outlet and the trench

Figure 121

TRENCH DRAINS



Source: SEWRPC.

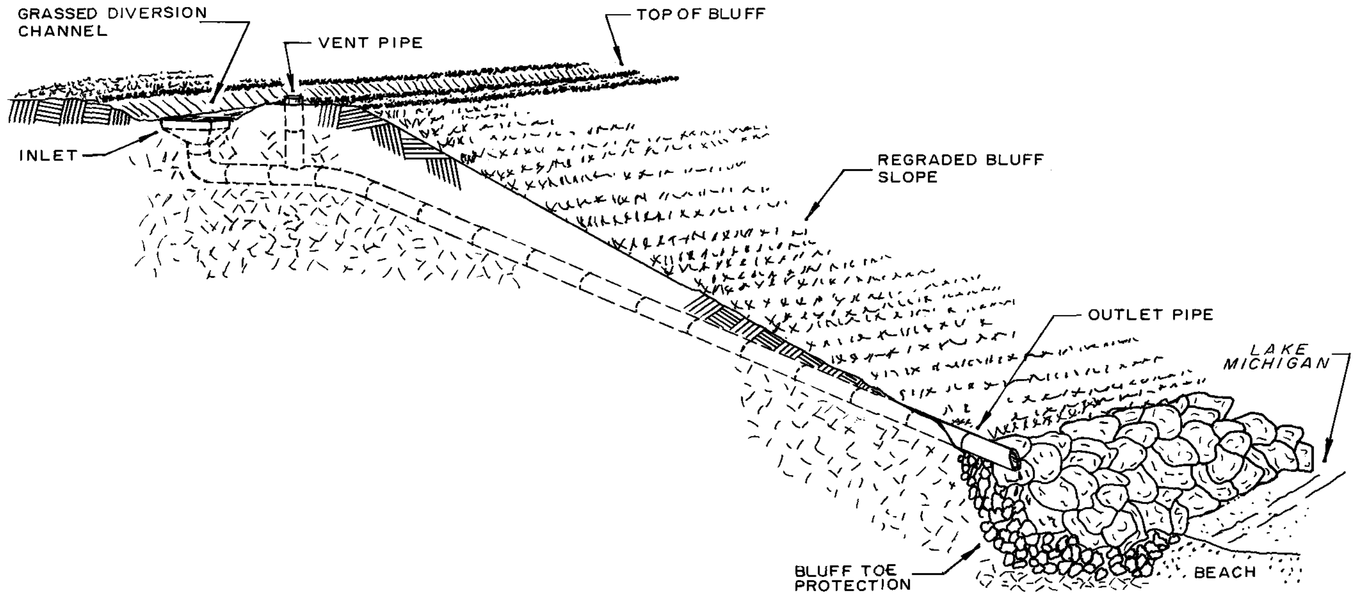
backfilled with granular material, as shown in Figure 121. Drainage trenches are typically two to six feet deep, and 18 to 24 inches wide. A trench drain is relatively inexpensive and easy to install, and drains by gravity. The disadvantage of this system is that it is limited to areas of shallow seepage, although deeper water-bearing strata can sometimes be drained by constructing the trench on the face of the bluff.

Construction of a trench drain may entail a capital cost of $20 to $80 per lineal foot of shoreline, with an annual maintenance cost of up to $5.00 per lineal foot.

Surface Water Drainage: Uncontrolled storm runoff can pond water at the top of the bluff, on top of slump blocks, and behind shore protection structures, as well as form gullies on bluff slopes. Surface water drainage control is particularly indicated for three bluff analysis sections, covering 1,540 feet, or 4 percent, of the study

Figure 122

STORMWATER DRAINAGE SYSTEM TO PREVENT EXCESSIVE STORM RUNOFF OVER THE TOP OF THE BLUFF



Source: SEWRPC.

area shoreline. Specific drainage problems which reduced the stability of the bluff slopes were identified within each of these sections. Surface water drainage measures include various types of structures intended to prevent the ponding of water, to reduce surface flows over the top of the bluff, to prevent scouring and erosion of drainage channels and gullies, and to prevent excessive infiltration into the bluff. An example of a stormwater drainage system to prevent excessive runoff over the top of the bluff is shown in Figure 122. 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 \$15 to \$150 per lineal foot of shoreline, with an annual maintenance cost of up to \$5.00 per lineal foot.

Revegetation: Revegetation of the bluff slope as a means to enhance slope stability was indicated for portions of 10 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 of revegetating bluff slopes include seeding and transplanting.

Seeding: Grass and other herbaceous plant mixtures can be seeded by scattering the seed on the bluff face by hand; by hydroseeding, which distributes the seed in a mixture of water, fertilizer, and mulch; or by drilling, in which a seed and fertilizer are inserted into the soil and covered. Hydroseeding and drilling, which are best suited for large-scale planting and for planting on steep slopes, are labor and equipment intensive and therefore more expensive methods of seeding. With hand broadcast seeding, fertilizer 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 1,000 square feet if scattered by hand, to $40 per 1,000 square feet if hydroseeding or drilling were used. Annual maintenance costs for the first three years following seeding would range from $5.00 per 1,000 square feet for hand scattering, to $10 per 1,000 square feet for hydroseeding or drilling.

Transplanting: Transplanting may be necessary to revegetate difficult sites, and can be used for establishing grasses, shrubs, and trees. Typically conducted by hand, transplanting would require careful attention for excavation of the holes, placement of the plants, fertilization, and watering. Transplanting provides the benefits of an immediate vegetative cover and allows the individual plants to be arranged as desired. It is, however, highly labor intensive.

The capital cost of revegetating a bluff slope by transplanting would range from $200 to $500 per 1,000 square feet. Annual maintenance costs would range from $40 to $100 per 1,000 square feet for the first three years following planting.

Setback Requirements for New Urban Development

Setback requirements for new urban development directly related to erosion hazards can be incorporated into existing city and village zoning ordinances. These requirements are intended to prevent the placement of new urban development in areas with a substantial risk of erosion damage over the economic life of the facilities. Setback distances would be comprised of two components: an erosion risk distance, and a minimum facility setback distance. Erosion risk distances would be the distance from the existing bluff edge that 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 both to prevent facilities from being placed too close to the bluff edge, and to provide an open space area which could 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 county setback distances generally consist of a stable slope component based on a stable bluff slope of one on two and one-half, plus the recession of the bluff that may be expected to occur over a period of approximately 50 years.

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 123. 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.

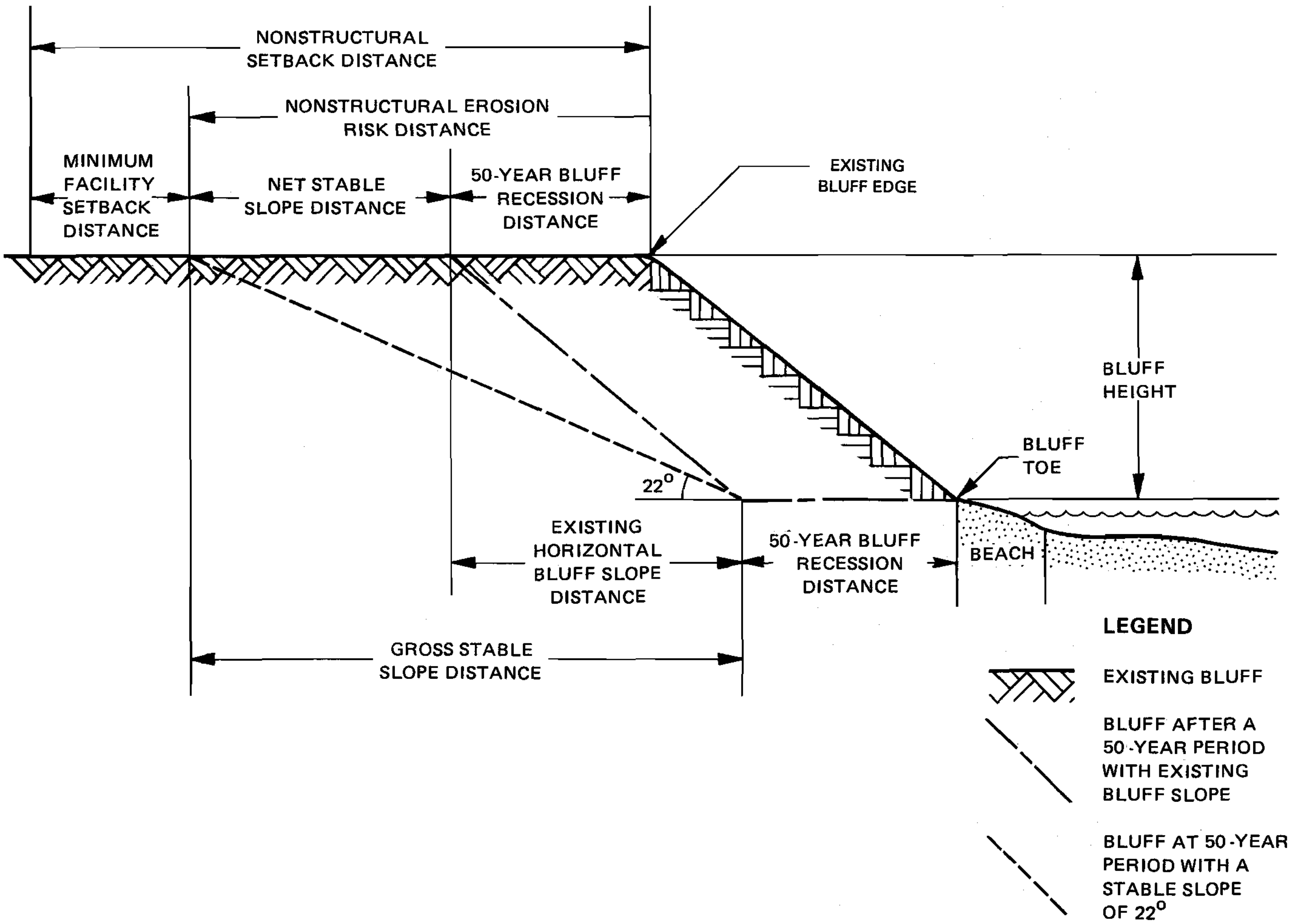
Nonstructural erosion risk distances are comprised of a bluff recession distance over a given time period, plus the distance required to grade the bluff face to a stable slope. Erosion risk distances are delineated for a 50-year period of continued bluff recession. The faces of the bluffs are assumed 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.

Minimum facility setback distances are recommended because future bluff recession rates could differ substantially from the historic 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 124. Structural setback distances consist of the sum of the structural erosion risk distance and a minimum facility setback distance. Structural setback distances would also apply to those

Figure 123

PROCEDURE UTILIZED TO ESTIMATE NONSTRUCTURAL EROSION RISK DISTANCE AND NONSTRUCTURAL SETBACK DISTANCE



NONSTRUCTURAL EROSION RISK DISTANCE = NET STABLE SLOPE DISTANCE + 50-YEAR BLUFF RECESSION DISTANCE

NONSTRUCTURAL SETBACK DISTANCE=NONSTRUCTURAL EROSION RISK DISTANCE+MINIMUM FACILITY SETBACK DISTANCE

WHERE: NET STABLE SLOPE DISTANCE = GROSS STABLE SLOPE DISTANCE - EXISTING HORIZONTAL BLUFF SLOPE DISTANCE

$$\text{GROSS STABLE SLOPE DISTANCE} = \frac{\text{BLUFF HEIGHT}}{\text{TAN } 22^{\circ}} = \frac{\text{BLUFF HEIGHT}}{0.4}$$

MINIMUM FACILITY SETBACK DISTANCE: INTENDED TO PROVIDE A SAFETY ZONE, PROVIDE AESTHETIC BENEFITS, AND ALLOW PROVISION OF FUTURE SURFACE WATER AND GROUNDWATER DRAINAGE SYSTEMS

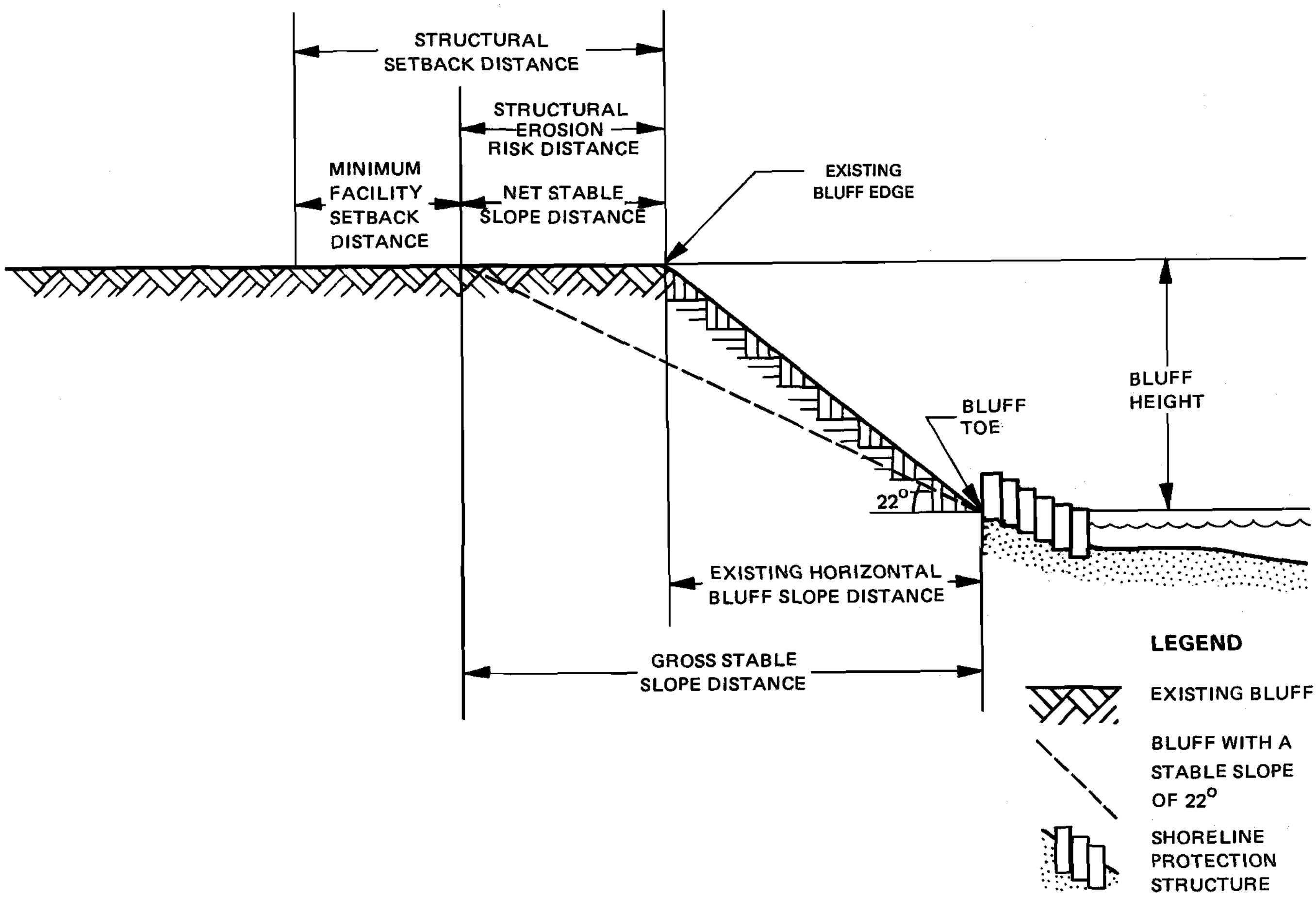
Source: SEWRPC.

portions of the Lake Michigan shoreline that are currently stabilized, even if no shore protection structure is in place.

The rate of bluff recession would be assumed to be zero once the structural measures were in place, the bluff toe protected, and the bluff slope stabilized. A structural erosion risk distance would therefore consist of that distance required to form a stable bluff slope of one on two and one-half, or about 22 degrees. A minimum facility setback distance of 50 feet is recommended for all permanent buildings and facilities.

Figure 124

PROCEDURE UTILIZED TO ESTIMATE STRUCTURAL EROSION RISK DISTANCE AND STRUCTURAL SETBACK DISTANCE



STRUCTURAL EROSION RISK DISTANCE=NET STABLE SLOPE DISTANCE

STRUCTURAL SETBACK DISTANCE=STRUCTURAL EROSION RISK DISTANCE+MINIMUM FACILITY SETBACK DISTANCE

WHERE: NET STABLE SLOPE DISTANCE = GROSS STABLE SLOPE DISTANCE - EXISTING HORIZONTAL BLUFF SLOPE DISTANCE

$$\text{GROSS STABLE SLOPE DISTANCE} = \frac{\text{BLUFF HEIGHT}}{\text{TAN } 22^{\circ}} = \frac{\text{BLUFF HEIGHT}}{0.4}$$

MINIMUM FACILITY SETBACK DISTANCE: INTENDED TO PROVIDE A SAFETY ZONE, PROVIDE AESTHETIC BENEFITS, AND ALLOW PROVISION OF FUTURE SURFACE WATER AND GROUNDWATER DRAINAGE SYSTEMS

Source: SEWRPC.

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 modification of existing diversions into and out of the lakes, and by construction of new diversions.

There are five major artificial diversions on the Great Lakes, which change the natural supply of water to the lake or which permit water to bypass a natural lake outlet. These are the Long Lac, Ogoki, and Chicago diversions, the Welland Canal, and the New York State Barge Canal.

Although they are separate diversions, the Ogoki and Long Lac diversions are frequently considered together because they both divert into Lake Superior water from the Albany River Basin that would otherwise drain to Hudson Bay. Completed in 1941, the Long Lac diversion connects the headwaters of the Kenogami River with the Aguasabon River, which flows into Lake Superior. Completed in 1943, the Ogoki diversion diverts water from the Ogoki River to Nipigon Lake, which is located in the Lake Superior Basin. These diversions were developed for the purpose of generating hydroelectric power. The Long Lac diversion was also developed to help transport pulpwood logs southward.

The combined average flow for the Long Lac and Ogoki diversions is about 5,600 cubic feet per second (cfs). This diversion can be compared with the annual average outflow from Lake Superior of 76,000 cfs for the period 1900 to 1986.

It should be noted that the diversion of water from the Ogoki River was temporarily reduced or stopped during the high-water periods of 1951 through 1953 and 1972 through 1974, and, most recently, in 1985. The 1985 reduction is estimated to have caused a 0.03-foot reduction in the level of Lake Superior.[2]

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 to divert the effluent from Lake Michigan. The diversion also facilitates navigation on the Chicago Sanitary and Ship Canal and hydroelectric power generation in Illinois. The rate of flow is subject to the jurisdiction of the U.S. Supreme Court, the current average authorized flow being 3,200 cfs.

The Welland Canal diverts water from Lake Erie across the Niagara Peninsula to Lake Ontario, thereby bypassing the Niagara River and Niagara Falls, primarily for navigation and hydroelectric power generation. The canal was originally built in 1829 and has been modified and realigned several times. The rate of flow through the canal is about 9,200 cfs.

The New York State Barge Canal diverts water primarily for navigation purposes from the Niagara River at Tonawanda, New York, ultimately discharging it to Lake Ontario. The rate of flow varies seasonally; the average rate is estimated to be 700 cfs and the maximum rate during the navigation season is estimated to be 1,100 cfs.

The effects of these diversions, other than the New York State Barge Canal, on Great Lakes water levels—as estimated by the International Great Lakes Diversions and Consumptive Uses Study Board of the International Joint Commission—are indicated in Table 39. The New York State Barge Canal, it should be noted, has little effect on the water levels of the Great Lakes.

Water levels in the Great Lakes can be partially regulated by means of artificial outlet control structures. 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 governments of the United States and Canada, in August 1981, requested that the International Joint Commission undertake a comprehensive study of methods of alleviating the adverse impacts of changing water levels, ranging from very high to very low levels, on the Great Lakes/St. Lawrence River Basin. The study is to be conducted in two phases. The first phase of the study consists of examination of

[2]*Great Lakes Commission, Water Level Changes—Factors Influencing the Great Lakes, 1986.*

Table 39

ESTIMATED EFFECT OF EXISTING DIVERSION RATES ON GREAT LAKES WATER LEVELS

Diversion	Rate (cfs)	Effect on Mean Water Level (feet)			
		Lake Superior	Lakes Michigan-Huron	Lake Erie	Lake Ontario
Long Lac/Ogoki	5,600	0.21	0.37	0.25	0.22
Lake Michigan at Chicago	3,200	-0.07	-0.21	-0.14	-0.10
Welland Canal	9,400[a]	-0.06	-0.18	-0.44	0

[a] *The effects on lake levels were evaluated for a rate of 9,400 cfs, slightly higher than the current rate of 9,200 cfs. An evaluation based upon the current rate would yield similar results.*

Source: International Great Lakes Diversions and Consumptive Uses Study Board of the International Joint Commission.

short-term alternatives—not involving major structural improvements—to minimize the adverse impacts of fluctuating water levels. The second phase, which is scheduled to be completed in 1989, will include a comprehensive evaluation of potential solutions, including structural improvements, land use planning, and other management activities.

ALTERNATIVE SHORELINE EROSION MANAGEMENT PLANS

Alternative shoreline erosion management plans were developed for two plan elements: a bluff slope stabilization element and a bluff toe protection element. Each of the alternative plans is described in detail below. The descriptions include an estimate of the total capital cost and annual maintenance cost of each plan. To facilitate the comparison of the alternative plans on an economic basis, the equivalent annual cost—or the equivalent present value of a series of future expenditures—is also provided. An economic analysis period of 50 years and an interest rate of 6 percent were used in the economic analysis.

Bluff Slope Stabilization Plan Element

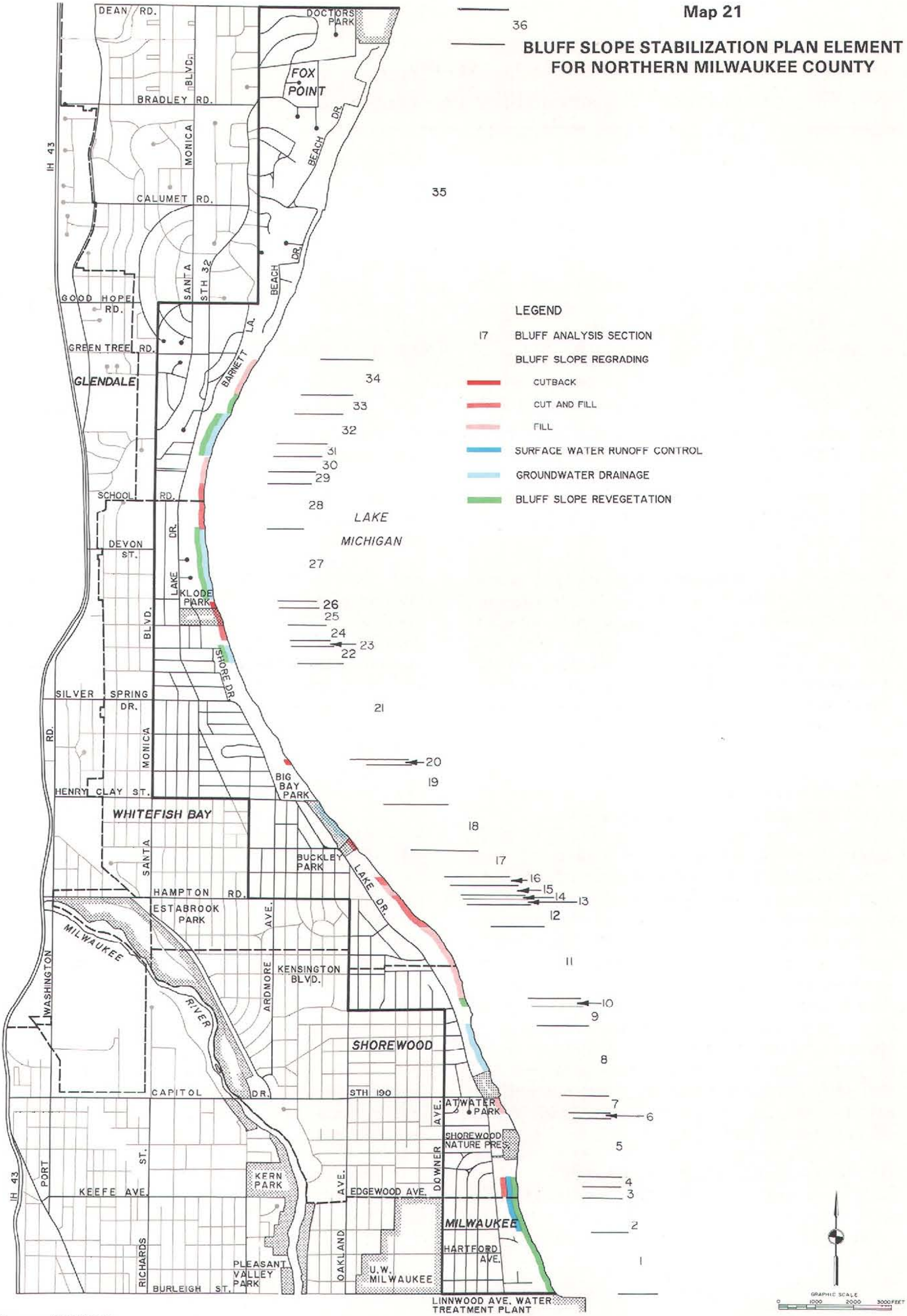
The bluff slope stabilization plan generally identifies for each major section of shoreline the measures expected to be needed to regrade and revegetate the bluff face and to control surface- and groundwater flow. Site-specific stabilization methods which can effectively abate problems of bluff instability can be evaluated only in the preliminary engineering and final design phases of plan implementation. The bluff slope stabilization measures thus represent the first element of the recommended plan and of alternatives thereto.

The bluff slope stabilization plan element consists of measures that would stabilize the bluff slopes within each of the bluff analysis sections, as indicated in Chapter III of this report. The bluff slope stabilization plan element is illustrated on Map 21. This plan element addresses those factors that need to be controlled to fully stabilize the bluff slopes, and identifies an appropriate method for slope regrading. The specific type of control measure to be used, however, would be selected in the preliminary engineering phase of the planning process. Thus, no alternatives to the bluff stabilization plan element were considered. The design and evaluation of specific methods for slope stabilization requires more site-specific data and analyses.

Components of the plan would include bluff slope regrading and revegetation, and control of groundwater and surface water flow. To help minimize the amount of fill required to stabilize

Map 21

BLUFF SLOPE STABILIZATION PLAN ELEMENT FOR NORTHERN MILWAUKEE COUNTY



Source: SEWRPC.

the slope, as well as the disruption of the natural aesthetic properties and drainage characteristics of the bluff slope within areas where slope regrading was indicated as a control measure, three methods were considered: the cutback method, the cut and fill method, and the fill method. The method recommended for bluff slope regrading within a particular bluff analysis section was selected based upon the distance from the existing houses to the edge of the bluff, and the presence or absense of an adjacent fill project. In order to maintain the regraded bluff slope, surface- and groundwater drainage would be provided, where needed, as well as topsoil placement, seeding, and mulching to develop a protective vegetative cover.

The criteria used in the selection of the bluff slope regrading component, along with the estimated cost of each slope stabilization component, are set forth in Table 40. Bluff slopes would be regraded to a stable angle along about 10,420 feet of shoreline, or 27 percent of the total study area shoreline. Detailed studies to determine the feasibility of installing groundwater drainage systems along 6,160 feet of shoreline, or 16 percent of the total study area, would be conducted. Surface water runoff control would be provided along about 1,540 feet of shoreline, or 4 percent of the study area. Revegetation of at least a portion of the bluff face would be provided along about 8,000 feet of shoreline, or 21 percent of the study area.

The selected plan components and estimated cost of bluff slope stabilization are listed for each bluff analysis section in Table 41. The bluff slope stabilization plan would have a capital cost of about $1.9 million, and an average annual maintenance cost of about $228,300. About 73 percent of the maintenance cost, however, would be required only during the first three years following construction. The equivalent annual cost over a 50-year period would be about $222,000.

Bluff Toe Protection Plan Element

The bluff toe protection plan represents the second element of the recommended plan and of alternatives thereto. Three conceptual alternative plans were developed to protect the toe of the bluff from wave and ice erosion. The first alternative would utilize revetments wherever practicable to protect the shoreline. For systems level planning purposes, it was assumed that the revetments would be constructed of quarry stone, although other types of revetments could also be used. The revetment alternative would have a relatively low cost.

The second alternative plan for bluff toe protection would provide, wherever practicable, artificially nourished beach systems with either onshore or near-shore structures being used to help maintain the beaches. The beach alternative plan would provide a usable beach composed of coarse sand or gravel for a large portion of the study area shoreline. For the purposes of systems level planning, 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 near-shore stone reefs—could also be used. The beach alternative would have a relatively moderate cost.

The third alternative plan for bluff toe protection would utilize offshore islands and breakwaters—along with some onshore structures—to protect the shoreline and provide limited sand beaches. This alternative plan would create about 30 acres of new public land for passive recreational uses. The offshore alternative would have a relatively high cost.

In the development of the alternative bluff toe protection plans, a number of important assumptions were made concerning local preferences and priorities. It was assumed that large sandy beaches would be desired at Atwater Park, Klode Park, and Doctors Park. It was further assumed that lakeshore residents of the Fox Point terrace—Bluff Analysis Section 35—would oppose any structures that would obstruct the view of the lake from the residences. Finally, it was assumed that most lakeshore residents would desire a usable shoreline—though not necessarily requiring a sand beach.

The potential shore protection measures previously described in this chapter were then screened to determine which types of measures should be included in the alternative plans. Based upon that screening, it was concluded that the construction of new bulkheads should not be considered further within the study area. Bulkheads are generally difficult and costly to maintain; often reflect wave energy to cause scouring of the lakebed; and generally do not provide an attractive, natural appearance to the shoreline. Some of the alternative plans, however, recommend the continued maintenance of

Table 40

SELECTION CRITERIA AND ESTIMATED CAPITAL AND MAINTENANCE UNIT COSTS OF THE BLUFF SLOPE STABILIZATION PLAN COMPONENTS

Plan Component	Criteria for Selection	Estimated Unit Cost ($/lineal foot of shoreline unless otherwise indicated)	
		Total Capital	Annual Maintenance
Regrading Bluff Slope—Cutback Method	Slope regrading needed to stabilize slope, and the distance between the bluff edge and existing houses is greater 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	150	15[a]
Regrading Bluff Slope—Fill Method	Slope regrading needed to stabilize slope, and the distance between the bluff edge and existing houses is less than 50 feet	150	15[a]
Regrading Bluff Slope—Cut and Fill Method	Slope regrading needed to stabilize slope, and the 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	150	15[a]
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	Variable depending upon type of drainage problem	Variable depending upon type of drainage problem
Revegetation	Areas where lack of vegetation could cause translational sliding	350/1,000 ft^2	10/1,000 ft^2 [a]

[a]*Annual maintenance costs apply only to the first three years following construction of the bluff slope stabilization method.*

Source: SEWRPC.

Table 41

ESTIMATED COST OF THE BLUFF SLOPE STABILIZATION PLAN FOR NORTHERN MILWAUKEE COUNTY

Civil Division	Bluff Analysis Section	Shoreline Length (feet)	Plan Component	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual Cost
City of Milwaukee	1	1,970	Revegetation	$ 20,000	$ 3,900[a]	$ 30,000	$ 2,000
	2	950	Revegetation, surface water control	14,000	2,800[a]	22,000	1,000
Village of Shorewood	3	300	Bluff slope regrading—cut and fill, surface water control, revegetation	30,000	3,000[a]	38,000	2,000
	4	290	Bluff slope regrading—cut and fill, surface water control, revegetation	23,000	2,900[a]	31,000	2,000
	5	1,710	- -	- -	- -	- -	- -
	6	170	- -	- -	- -	- -	- -
	7	380	Bluff slope regrading—fill	57,000	5,700[a]	72,000	5,000
	8	790	- -	- -	- -	- -	- -
	8	1,380	Groundwater drainage	69,000	13,800	287,000	18,000
	9	520	- -	- -	- -	- -	- -
	10	240	Revegetation	4,000	700[a]	6,000	< 1,000
	11	2,370	Bluff slope regrading—fill	356,000	35,600[a]	419,000	27,000
Village of Whitefish Bay	12	850	Bluff slope regrading—cut and fill	128,000	12,800[a]	162,000	10,000
	13	190	Bluff slope regrading—cut and fill	28,000	2,800[a]	283,000	18,000
	14	160	Bluff slope regrading—cut and fill	24,000	2,400[a]	30,000	2,000
	15	310	Bluff slope regrading—fill	46,000	4,600[a]	59,000	4,000
	16	360	Bluff slope regrading—cut and fill	54,000	5,400[a]	68,000	4,000
	17	810	- -	- -	- -	- -	- -
	18	600	Bluff slope regrading—cut and fill	90,000	9,000[a]	114,000	7,000
	18	1,060	Groundwater drainage	53,000	10,600	220,000	14,000
	19	1,480	- -	- -	- -	- -	- -
	20	130	Bluff slope regrading—cutback	13,000	1,300[a]	16,000	1,000
	21	2,970	- -	- -	- -	- -	- -
	22	490	Groundwater drainage, revegetation	32,000	5,900[b]	112,000	7,000
	23	140	- -	- -	- -	- -	- -
	24	430	Bluff slope regrading—fill	64,000	6,400[a]	82,000	5,000
	25	480	Bluff slope regrading—cut and fill	18,000	1,800[a]	23,000	1,000
	26	170	Bluff slope regrading—cutback	26,000	2,600[a]	32,000	2,000
	27	1,950	Groundwater drainage, revegetation	117,000	23,400[b]	435,000	28,000
	28	1,150	Bluff slope regrading—cut and fill	172,000	17,200[a]	219,000	14,000
Village of Fox Point	29	320	Bluff slope regrading—fill	48,000	4,800[a]	61,000	4,000
	30	470	Bluff slope regrading—fill	70,000	7,000[a]	89,000	6,000
	31	510	Groundwater drainage, revegetation	33,000	6,100[b]	116,000	7,000
	32	770	Groundwater drainage, revegetation	62,000	12,300[b]	195,000	12,000
	33	530	Revegetation	8,000	1,600[a]	12,000	1,000
	34	1,460	Bluff slope regrading—fill	219,000	21,900[a]	278,000	18,000
	35	9,070	- -	- -	- -	- -	- -
	36	840	- -	- -	- -	- -	- -
	Total	38,770	- -	$1,878,000	$228,300[c]	$3,511,000	$222,000

[a] *Annual maintenance costs would apply only for first three years following bluff slope regrading or revegetation.*

[b] *Of the total annual maintenance cost of $5,900 for Bluff Analysis Section 22, $1,000, or 17 percent, would be required only for the first three years following revegetation. Of the total annual maintenance cost of $23,400 for Bluff Analysis Section 27, $3,900, or 17 percent, would be required only for the first three years following revegetation. Of the total annual maintenance cost of $6,100 for Bluff Analysis Section 31, $1,000, or 16 percent, would be required only for the first three years following revegetation. Of the total annual maintenance cost of $12,300 for Bluff Analysis Section 32, $4,600, or 37 percent, would be required only for the first three years following revegetation.*

[c] *About $166,700, or 73 percent, of the total annual maintenance cost would be required only for the first three years following bluff slope regrading or revegetation.*

Source: SEWRPC.

existing bulkheads. The alternative bluff toe protection plans thus considered the use of quarry stone revetments; nourished coarse sand or gravel beach systems with short groins; nourished sand beaches with long groins or offshore breakwaters; and offshore islands and peninsulas. For the purposes of 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.

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 suitable protection and should not be used along the Lake Michigan shoreline. Steel sheet piling is reliable, but reflects wave energy which tends to increase bottom scouring. Large interlocking concrete units, concrete blocks, and grout-filled bags are generally not as durable as high-quality quarry stone, but can be used to provide effective shore protection at certain locations.

Geotextile filter cloths are required at the base of most quarry stone shore protection structures to protect against undermining, except where structures are constructed offshore in a water depth greater than three times the maximum wave height, where the anticipated current velocities are too weak to move the average size bed material, or where a structure is constructed directly on bedrock.[3] 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.[4]

[3]*U. S. Army Corps of Engineers, Shore Protection Manual, Volume II, Coastal Engineering Research Center, 1984.*

[4]*U. S. Army Corps of Engineers, Low Cost Protection, Final Report on the Shoreline Erosion Control Demonstration (Section 54) Program, 830 pp., 1981.*

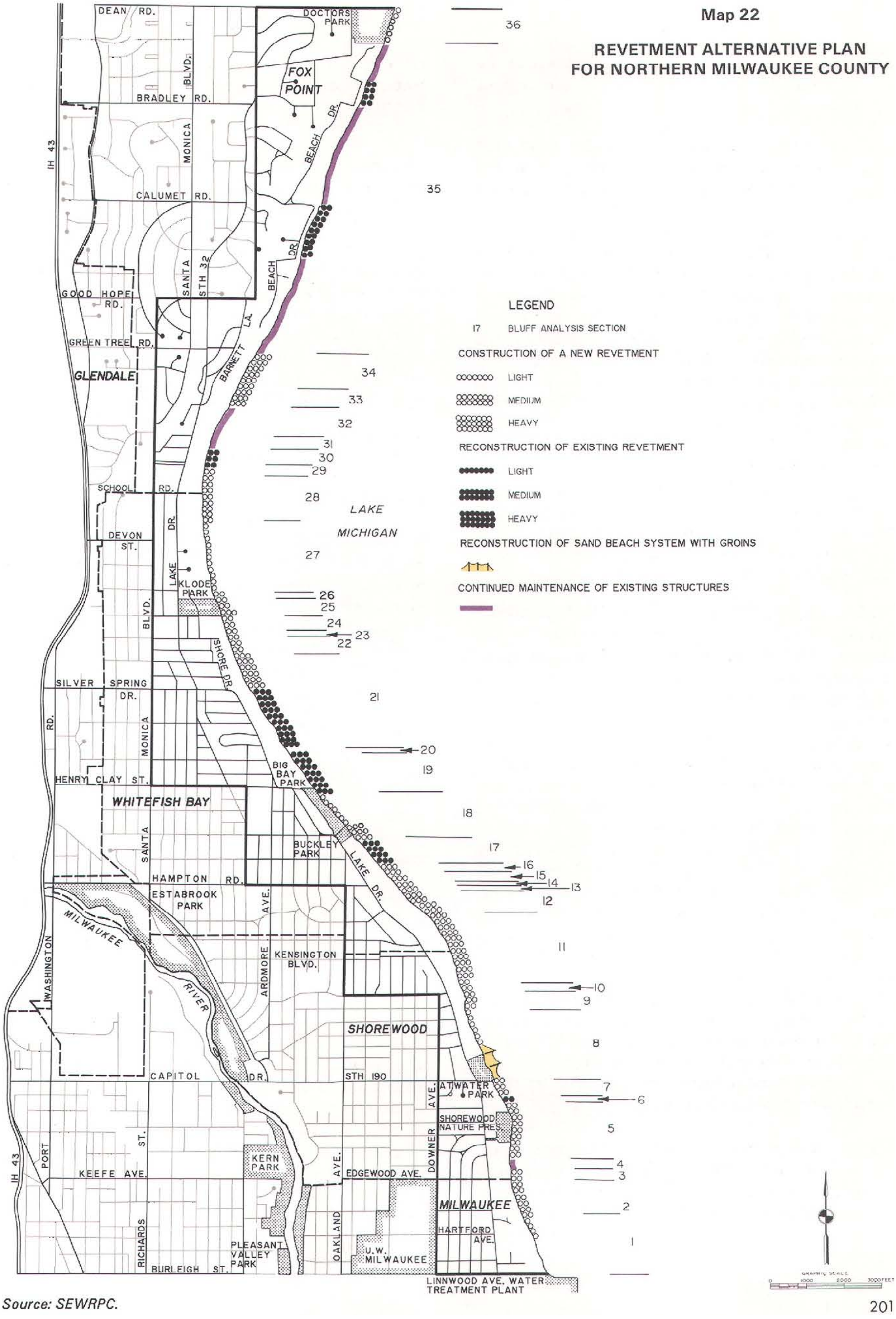
Woven filter cloths are usually composed of polypropylene or polyvinylidene chloride. The cloth made of polyvinylidene chloride—usually dark green—is heavier than water and should be used when constructing below the water surface. Polypropylene cloth—usually dark brown—is lighter than water and stiffer, stronger, and less costly than polyvinylidene chloride cloth. Polypropylene cloth should be used for construction above the water surface. Filter cloth with very small pore sizes should not be used. This grade of fabric is almost impermeable to hydraulic transients; hence, it causes considerable uplift pressures as the wave energy flows along it.[5] 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 bluff toe 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. In addition, different types of construction, including construction of berms using mixed stone sizes, could be utilized for revetments. The plan is illustrated on Map 22, and would include the construction, or reconstruction, of quarry stone revetments; the construction of a sand beach containment system at Atwater Park; and the continued maintenance of existing structures in Bluff Analysis Sections 4, 31, 32, and 35. The existing Doctors Park sand beach, located just north of the study area, would also be maintained. To provide an appropriate level of protection, three types of revetments were considered: light, medium, and heavy. The type of revetment required for a particular bluff analysis section was selected based upon the degree of toe erosion observed in 1986, the existing beach width and near-shore slope, and the anticipated wave heights. For cost purposes, it was assumed that new construction would require about two to three tons of stone per lineal foot of shoreline for a light revetment; about three to five tons of stone per foot for a medium revetment; and about five to 10 tons of stone per foot for a heavy revetment. Lesser amounts of stone would be

[5]*Charles Johnson, Coastal Engineer, U. S. Army Corps of Engineers, Chicago, Illinois, Personal Communication, July 27, 1987.*

Map 22

REVETMENT ALTERNATIVE PLAN FOR NORTHERN MILWAUKEE COUNTY



Source: SEWRPC.

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 42. A new revetment would be constructed along about 21,700 feet of shoreline, or 56 percent of the total study area shoreline of 38,770 feet. Existing revetments would be reconstructed along about 6,950 feet of shoreline, or 18 percent of the study area shoreline. Nourished sand beaches would be maintained along the 790-foot shoreline of Atwater Park, covering 2 percent of the study area, using long groins. The existing sand beach at Doctors Park, which lies just north of the study area, would also be maintained. Existing shore protection measures would be maintained and repaired as needed along about 8,320 feet of shoreline, or 21 percent of the study area shoreline. The remaining 1,010 feet of shoreline, or 3 percent, was not eroding and would not require toe protection under this alternative.

Within the Fox Point terrace—Bluff Analysis Section 35—the Village of Fox Point was, in August 1987, considering relocating portions of a sanitary sewer which lies along the shoreline and, in some areas, within the lake. In particular, the Village was considering relocating the sewer to N. Beach Drive in Subsections 35A and 35C. Relocation of the sewer would not interfere with the continued maintenance of shore protection structures within these subsections, as recommended in this alternative plan. Within Subsection 35B, the sewer lies beneath the existing revetment, which would be reconstructed under this alternative plan. Reconstruction of the revetment would further protect the sewer, although access for inspection and maintenance would continue to be difficult. Some manholes may need to be raised and a storm sewer outfall that discharges at the southern end of Subsection 35B may need to be extended farther out toward the lake. The sewers within Subsection 35D and 35E lie west of the immediate shoreline and are not seriously threatened by lakeshore erosion at this time.

The selected plan component and estimated cost of bluff toe protection are listed for each bluff analysis section in Table 43. The revetment alternative plan would have a total capital cost of about $8.3 million, and an annual maintenance cost of about $597,500. The equivalent annual cost over a 50-year period would be about $1.1 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 essentially represent a continuation of the existing approach to shore protection, although the proposed structures would be better designed, maintained, and coordinated than most existing structures. The plan could be readily implemented by individual property owners, or, preferably, by groups of property owners, and by municipalities.

A major disadvantage of the revetment alternative plan is the lack of a usable shoreline. Significant beaches that would protect the toe of the bluff would be present along only about 3,300 feet, or 8 percent, of the study area shoreline, and would include portions of Sections 1, 8, 20, 31, and 35. In some sections, revetments could have an adverse effect on the littoral environment, which could, in the long term, increase wave action against the shoreline. Revetments tend to reflect wave energy—although less so than bulkheads—and do not feed the littoral transport system. Over time, the near-shore slopes of areas with erodible offshore sand deposits would become somewhat steeper, which would increase the wave height capable of reaching the shore. The Fox Point terrace—Bluff Analysis Section 35—would be particularly susceptible to this effect because of its extensive offshore sand deposits. However, in most of the study area, the offshore sand deposits are believed to be quite shallow, with the relatively erosion-resistant clay hardpan lying close to the surface of the lake bottom. Wave reflection from revetments probably would not significantly steepen the offshore slopes constructed of erosion-resistant clay.

<u>Beach Alternative Plan</u>: The beach alternative plan would provide a usable beach composed of sand or gravel along 61 percent of the study area shoreline. The plan would also include the construction of perched cobble beaches instead of quarry stone revetments along an additional 39 percent of the shoreline. The perched cobble beaches would protect the toe of existing or proposed fill projects which regrade the slope. The beach alternative plan is shown on Map 23.

Table 42

SELECTION CRITERIA AND ESTIMATED CAPITAL AND MAINTENANCE UNIT COSTS OF THE REVETMENT ALTERNATIVE PLAN COMPONENTS

Plan Component	Criteria for Selection	Estimated Unit Cost ($/lineal foot of shoreline)	
		Total Capital	Annual Maintenance
Construction of a New Light Revetment	1. Slight shoreline or bluff toe erosion observed in 1986 2. Existing beach width greater than 20 feet 3. Near-shore slope less than 4 degrees 4. Less than 3 percent probability of onshore waves exceeding 6 feet in height	150	10
Construction of a New Medium Revetment	1. Moderate or severe shoreline or bluff toe erosion observed in 1986 2. Existing beach width less than 20 feet 3. Near-shore slope less than 4 degrees 4. Three to 4 percent probability of onshore waves exceeding 6 feet in height	250	15
Construction of a New Heavy Revetment	1. Moderate or severe shoreline or bluff toe erosion observed in 1986 2. Existing beach width less than 10 feet 3. Near-shore slope greater than 4 degrees 4. Greater than 4 percent probability of onshore waves exceeding 6 feet in height	350	20
Construction of New Heavy Revetment for Public Facility	1. Existing public works facility susceptible to catastrophic wave damage	820	15
Reconstruction of an Existing Revetment	1. Existing revetments which, as of 1986, required a substantial amount of repair	Light-100 Medium-200 Heavy-300	10 15 20
Construction of a New Structure Other than a Revetment	1. 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	1. Structure which was protecting against shoreline erosion in 1986 and which, if maintained, could provide continued effective protection	Variable depending upon type of structure	Variable depending upon type of structure
No Shoreline Protection	1. No significant shoreline or bluff toe erosion observed in 1986 and none expected to occur	0	0

Source: SEWRPC.

Table 43

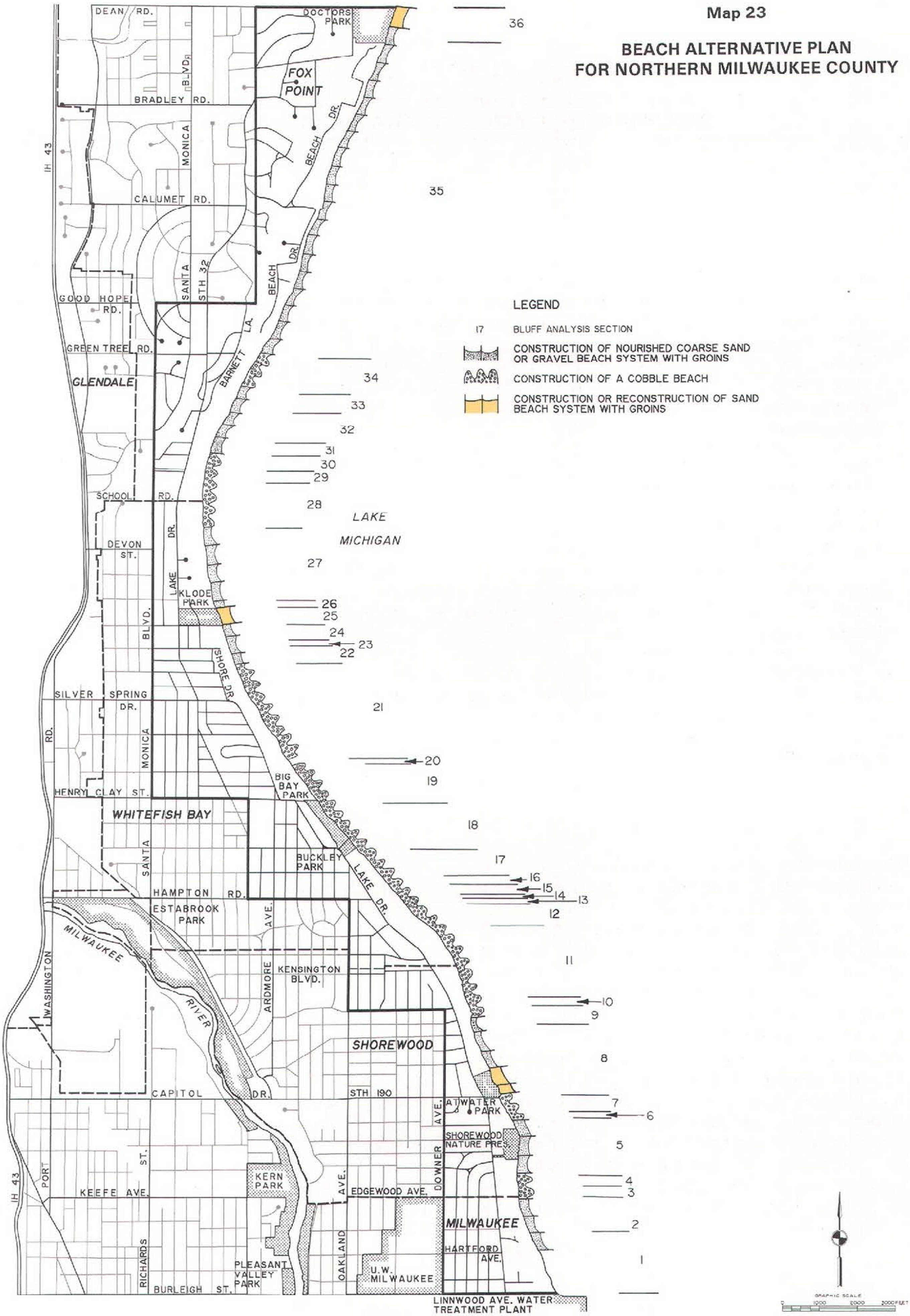
ESTIMATED COST OF THE REVETMENT ALTERNATIVE PLAN

Civil Division	Bluff Analysis Section	Shoreline Length (feet)	Plan Component	Cost per Lineal Foot		Total Cost			
				Capital	Annual Maintenance	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual Cost
City of Milwaukee	1	880	No toe protection	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0
		1,090	Construction of new light revetment	150	10	164,000	10,900	336,000	21,000
	2	950	Construction of new medium revetment	250	15	238,000	14,200	462,000	29,000
Village of Shorewood	3	300	Construction of new medium revetment	250	15	75,000	4,500	146,000	9,000
	4	290	Continued maintenance of existing structures	- -	15	- -	4,300	68,000	4,000
	5	1,710	Construction of new medium revetment	250	15	428,000	25,600	832,000	53,000
	6	170	Reconstruction of existing revetment-medium	200	15	34,000	2,500	73,000	5,000
	7	380	Construction of new medium revetment	250	15	95,000	5,700	185,000	12,000
	8	790	Construction of groin system with nourished sand beach	1,000	50	790,000	39,500	1,413,000	90,000
		1,380	Construction of new light revetment	150	10	207,000	13,800	425,000	27,000
	9	520	Construction of new medium revetment	250	15	130,000	7,800	253,000	16,000
	10	240	Construction of new medium revetment	250	15	60,000	3,600	117,000	7,000
	11	2,370	Construction of new heavy revetment	350	20	830,000	47,400	1,577,000	100,000
Village of Whitefish Bay	12	850	Construction of new heavy revetment	350	20	298,000	17,000	566,000	36,000
	13	190	Construction of new heavy revetment	350	20	66,000	3,800	126,000	8,000
	14	160	Construction of new heavy revetment	350	20	56,000	3,200	106,000	7,000
	15	310	Construction of new heavy revetment	350	20	109,000	6,200	207,000	13,000
	16	360	Construction of new heavy revetment	350	20	126,000	7,200	239,000	15,000
	17	810	Reconstruction of revetment-heavy	300	20	243,000	16,200	498,000	32,000
	18	600	Construction of new heavy revetment	350	20	210,000	12,000	399,000	25,000
	18	1,060	Construction of new light revetment	150	10	159,000	10,600	326,000	21,000
	19	1,480	Reconstruction of existing revetment-heavy	300	20	444,000	29,600	911,000	58,000
	20	130	No toe protection	0	0	0	0	0	0
	21	1,700	Reconstruction of existing revetment-heavy	300	20	510,000	34,000	1,046,000	66,000
		1,270	Construction of new heavy revetment	350	20	445,000	25,400	845,000	54,000
	22	490	Construction of new medium revetment	250	15	123,000	7,300	238,000	15,000
	23	140	Construction of new medium revetment	250	15	35,000	2,100	68,000	4,000
	24	430	Construction of new medium revetment	250	15	108,000	6,400	209,000	13,000
	25	300	Construction of new heavy revetment	820	15	246,000	4,500	317,000	20,000
		180	Construction of new medium revetment	250	15	45,000	2,700	88,000	6,000
	26	170	Construction of new medium revetment	250	15	43,000	2,500	82,000	5,000
	27	1,950	Construction of new light revetment	150	10	293,000	19,500	600,000	38,000
	28	1,150	Construction of new medium revetment	250	15	288,000	17,200	559,000	35,000
Village of Fox Point	29	320	Construction of new medium revetment	250	15	80,000	4,800	156,000	10,000
	30	470	Reconstruction of existing revetment-medium	200	15	94,000	7,000	204,000	13,000
	31	510	Continued maintenance of existing structures	- -	30	- -	15,300	241,000	15,000
	32	770	Continued maintenance of existing structures	- -	20	- -	15,400	243,000	15,000
	33	530	Construction of new medium revetment	250	15	133,000	7,900	258,000	16,000
	34	1,460	Construction of new heavy revetment	350	20	511,000	29,200	971,000	62,000
	35A	2,390	Continued maintenance of existing structures	- -	10	- -	23,900	377,000	24,000
	35B	1,600	Reconstruction of existing revetment-medium	200	15	320,000	24,000	698,000	44,000
	35C	3,000	Continued maintenance of existing structures	- -	10	- -	30,000	473,000	30,000
	35D	720	Reconstruction of existing revetment-medium	200	15	144,000	10,800	206,000	13,000
	35E	1,360	Continued maintenance of existing structures	- -	10	- -	13,600	214,000	14,000
	36	840	Construction of new light revetment	150	10	126,000	8,400	258,000	16,000
			Total	- -	- -	$8,306,000	$597,500	$17,546,000	$1,116,000

Source: SEWRPC.

Map 23

BEACH ALTERNATIVE PLAN FOR NORTHERN MILWAUKEE COUNTY



Source: SEWRPC.

Table 44

SELECTION CRITERIA AND ESTIMATED CAPITAL AND MAINTENANCE UNIT COSTS OF THE BEACH ALTERNATIVE PLAN COMPONENTS

Plan Component	Criteria for Selection	Estimated Unit Cost ($/lineal foot of shoreline)	
		Total Capital	Annual Maintenance
Construction of a Nourished Coarse Sand or Gravel Beach System with Short Groins	1. Shoreline or bluff toe erosion observed in 1986 2. Bluff slope regrading has not previously been conducted, and is not required	400	20
Construction of Long Groins with a Nourished Sand Beach	1. Strong community support for a large sand beach	1,000	50
Construction of a Cobble Beach	1. Bluff slope regrading has been conducted, or is required to stabilize the bluff slope	400	20
No Shoreline Protection	1. No significant shoreline or bluff toe erosion observed in 1986 and none expected to occur	0	0

Source: SEWRPC.

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 44. The beach alternative plan includes the maintenance of large sand beaches at Atwater, Klode, and Doctors Parks. Nourished coarse sand or gravel beaches contained by short groins would be created along about 21,460 feet, or 55 percent, of the study area shoreline. These beaches could also be contained by armored headlands or near-shore reefs. These containment structures would be constructed of quarry stone. A perched cobble beach would be constructed along 15,030 feet, or 39 percent, of the shoreline. No toe protection would be required along 1,010 feet or 3 percent, of the study area shoreline, which presently contains a stable beach.

This alternative plan would thus envision sand or gravel beaches along the entire shoreline, exclusive of the existing and proposed fill projects, and along selected high-wave-energy shoreline areas. Although in most cases a sand or gravel beach system technically could be constructed to protect the toe of a fill project, this type of 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 study area, and most of the bluff slopes that are filled or proposed to be filled face northeasterly. Hence, most fill areas will be subjected to the largest storm waves attacking the study area shoreline. It would be difficult—and costly—to maintain a coarse sand or gravel beach on a long-term basis in such a high-wave-energy environment. Second, since the fill projects generally require the placement of fill toward, or into, the lake, the nourished beach, and the attendant containment groins, constructed 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 groins 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. Sand or gravel beaches were not proposed for Bluff Analysis Sections 9 and 10, which lie immediately south of an existing large fill project, or for Bluff Analysis Section 18, which includes Big Bay Park. Because of shoreline orientation and configuration, and/or because of steep offshore slopes, it would be difficult to maintain a beach at these sites.

With respect to the sanitary sewer problem in Bluff Analysis Section 35, relocation of the sewer in Subsections 35A and 35C would not interfere with the construction of a nourished beach system, as recommended under this alternative plan. Within Subsection 35B, where the sewer lies beneath the existing revetment, construction of a nourished beach would make it easier to inspect and maintain the sewer. Some manholes may need to be raised in Subsection 35B. The sewers within Subsections 35D and 35E, which lie west of the shoreline, would not be affected by the construction of the beach.

The selected plan component and estimated cost of bluff toe protection are listed for each bluff analysis section in Table 45. The beach alternative plan would have an estimated total capital cost of $15.9 million, and an annual maintenance cost of about $0.8 million. The equivalent annual cost over a 50-year period would be about $1.8 million.

The major advantage of the beach alternative plan is the provision of a more usable shoreline. The coarse sand or gravel beaches not only would offer access and recreational opportunities while protecting the shoreline from erosion, but would also reduce wave reflection and, to a limited extent, feed the littoral transport system, thereby reducing adverse effects on the littoral environment. Since access over the perched cobble beaches is usually difficult, these beaches would not provide a usable shoreline unless the cobbles were covered with at least a one- to two-foot layer of gravel. The cobble beaches reflect less wave energy than does a rip-rap revetment. 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 the maintenance of the beach systems—the systems could not be implemented in a piecemeal manner. The coarse sand and gravel beaches would likely increase the use of public shoreline areas by the general public. Such increased use may be opposed by some private property owners who desire limited access to public shoreline property which lies adjacent to certain residential areas.

<u>Offshore Alternative Plan</u>: The offshore alternative plan, developed by Warzyn Engineering, Inc.; Johnson, Johnson & Roy, Inc.; and W. F. Baird and Associates, Ltd., would provide a series of offshore islands and breakwaters for the entire shoreline except the Fox Point terrace—Bluff Analysis Section 35. The islands, composed of construction debris, or perhaps rock spoil from the Milwaukee Metropolitan Sewerage District deep tunnel project, could be protected on the lakeward side by an armored headland-pocket beach system, and on the landward side by a light revetment. The islands, which would be constructed with land-based equipment, would be located 300 to 1,000 feet offshore at an approximate water depth of 10 to 12 feet. The publicly owned islands would not be connected and would be utilized for passive recreational uses, primarily by boaters. Offshore breakwaters with nourished sand beaches would be constructed at Atwater Park, Klode Park, the southern portion of North Beach Drive in the Village of Fox Point which lies directly adjacent to the lake, and Doctors Park. The Doctors Park beach would involve a southward expansion of the existing beach, extending to the Fox Point terrace. In addition to these offshore structures, light 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

Table 45

ESTIMATED COST OF THE BEACH ALTERNATIVE PLAN

Civil Division	Bluff Analysis Section	Shoreline Length (feet)	Plan Component	Cost per Lineal Foot		Total Cost			
				Capital	Annual Maintenance	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual Cost
City of Milwaukee	1	880	No toe protection	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0
		1,090	Nourished beach system	400	20	436,000	21,800	780,000	49,000
	2	950	Nourished beach system	400	20	380,000	19,000	679,000	43,000
Village of Shorewood	3	300	Cobble beach	400	20	120,000	6,000	215,000	14,000
	4	290	Cobble beach	400	20	116,000	5,800	207,000	13,000
	5	1,710	Nourished beach system	400	20	684,000	34,200	1,223,000	78,000
	6	170	Cobble beach	400	20	68,000	3,400	122,000	8,000
	7	380	Cobble beach	400	20	152,000	7,600	272,000	17,000
	8	790	Construction of long groins with a nourished sand beach (Atwater)	1,000	50	790,000	39,500	1,413,000	90,000
		1,380	Nourished beach system	400	20	552,000	27,600	987,000	63,000
	9	280	Nourished beach system	400	20	112,000	5,600	200,000	13,000
		240	Cobble beach	400	20	96,000	4,800	172,000	11,000
	10	240	Cobble beach	400	20	96,000	4,800	172,000	11,000
	11	2,370	Cobble beach	400	20	948,000	47,400	1,695,000	108,000
Village of Whitefish Bay	12	850	Cobble beach	400	20	340,000	17,000	608,000	39,000
	13	190	Cobble beach	400	20	76,000	3,800	136,000	9,000
	14	160	Cobble beach	400	20	64,000	3,200	114,000	7,000
	15	310	Cobble beach	400	20	124,000	6,200	222,000	14,000
	16	360	Cobble beach	400	20	144,000	7,200	257,000	16,000
	17	810	Cobble beach	400	20	324,000	16,200	579,000	37,000
	18	1,660	Cobble beach	400	20	664,000	33,200	1,187,000	75,000
	19	1,480	Cobble beach	400	20	592,000	29,600	1,058,000	67,000
	20	130	No toe protection	0	0	0	0	0	0
	21	2,970	Cobble beach	400	20	1,188,000	59,400	2,124,000	135,000
	22	490	Nourished beach system	400	20	196,000	9,800	350,000	22,000
	23	140	Nourished beach system	400	20	56,000	2,800	100,000	6,000
	24	430	Nourished beach system	400	20	172,000	8,600	308,000	20,000
	25	300	Construction of long groins with a nourished sand beach and new medium revetment	1,250	65	375,000	19,500	682,000	43,000
		180	Construction of long groins with a nourished sand beach	1,000	50	180,000	9,000	322,000	20,000
	26	170	Nourished beach system	400	20	68,000	3,400	122,000	8,000
	27	1,950	Nourished beach system	400	20	780,000	39,000	1,395,000	89,000
	28	1,150	Nourished beach system	400	20	460,000	23,000	822,000	52,000
Village of Fox Point	29	320	Cobble beach	400	20	128,000	6,400	229,000	15,000
	30	470	Cobble beach	400	20	188,000	9,400	336,000	21,000
	31	510	Nourished beach system	400	20	204,000	10,200	365,000	23,000
	32	770	Nourished beach system	400	20	308,000	15,400	551,000	35,000
	33	530	Nourished beach system	400	20	212,000	10,600	379,000	24,000
	34	1,460	Cobble beach	400	20	584,000	29,200	1,044,000	66,000
	35	9,070	Nourished beach system	400	20	3,628,000	181,400	6,487,000	412,000
	36	840	Nourished beach system	400	20	336,000	16,800	601,000	38,000
			Total	- -	- -	$15,941,000	$797,800	$28,515,000	$1,811,000

Source: SEWRPC.

maintained along the shoreline not protected by offshore structures, and in a few areas where construction of a light revetment would not be feasible. The offshore alternative plan is illustrated on Map 24.

The criteria used in the selection of an offshore alternative plan component, along with the estimated cost of each component, are set forth in Table 46. Offshore islands would be created along about 27,590 feet, or 71 percent, of the study area shoreline. Offshore breakwaters would be constructed along about 3,710 feet, or 10 percent, of the study area shoreline. Existing toe protection structures would be maintained along 8,030 feet, or 21 percent, of the study area shoreline. To supplement the offshore structures, new light revetments would be constructed or reconstructed along 18,890 feet, or 49 percent, of the study area shoreline.

The effect of the offshore alternative plan on the sanitary sewer problem in Bluff Analysis Section 35 would be the same as under the revetment alternative plan except in Subsection 35B, where a beach contained by offshore breakwaters would be constructed. The construction of the beach would improve the ease of inspection and maintenance of the sewer which lies beneath the existing revetment. Some manholes within Subsection 35B would probably need to be raised.

The selected plan component and estimated cost of bluff toe protection are listed for each bluff analysis section in Table 47. The offshore alternative plan would have a total capital cost of about $35.3 million, and an annual maintenance cost of about $1.0 million. The equivalent annual cost over a 50-year period would be about $3.3 million.

The major advantages of the offshore alternative plan would be the creation of approximately 30 acres of new public lakeshore parkland; the provision of about 300 acres of protected surface water; the creation of about 10 miles of new shoreline; the expansion of large public sand beaches at four sites; and the provision of new wildlife and fishery habitat. The plan would minimize the disruption and expenditures 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 an opportunity to utilize public funds to create new public parkland while helping to protect private property.

A disadvantage of the offshore alternative plan, in addition to its high cost, is the need for over five million cubic yards of fill material for construction of the islands. The plan probably could not be implemented by groups of private property owners; thus, 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.

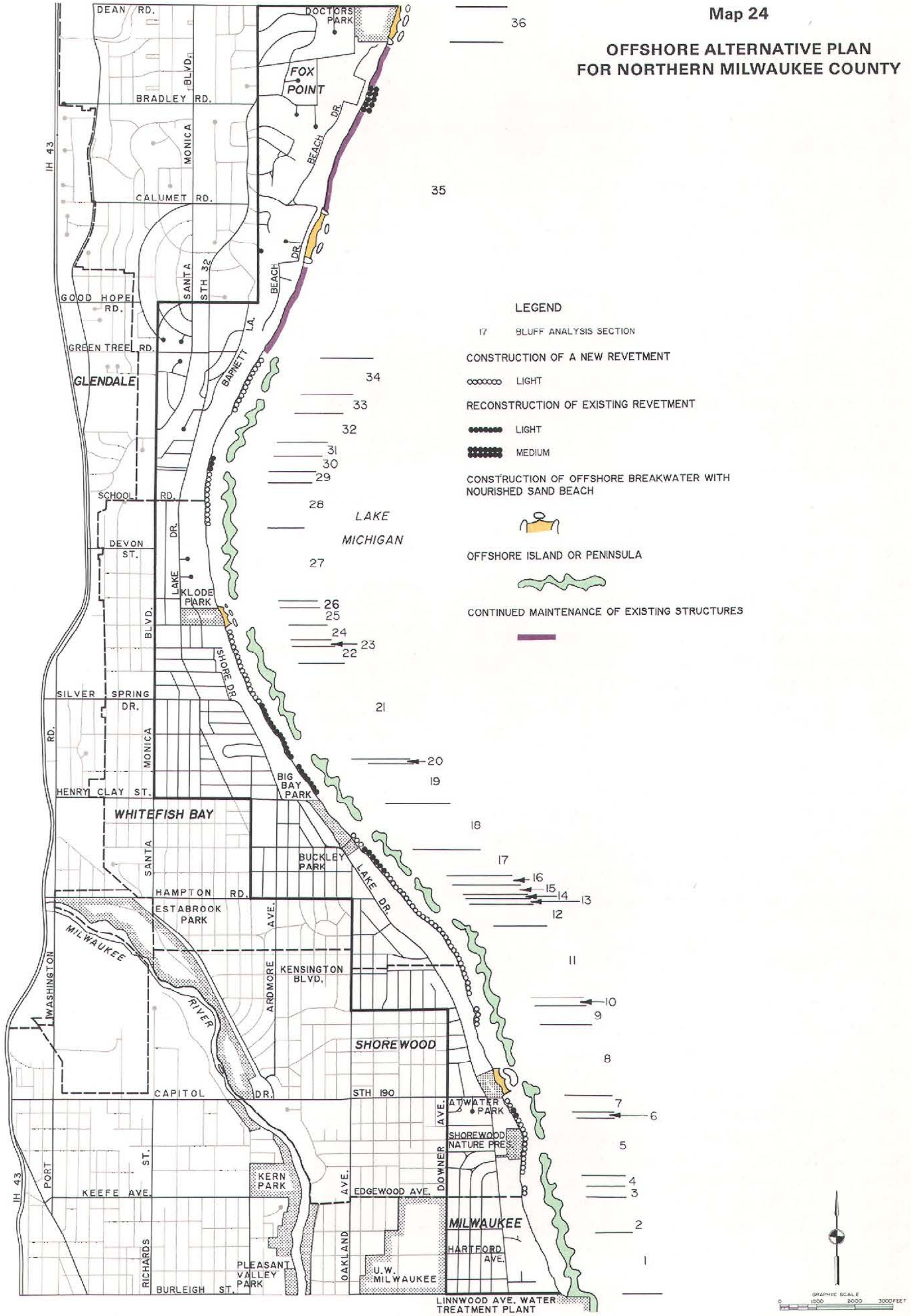
PRELIMINARY SHORELINE EROSION MANAGEMENT PLAN

The preliminary shoreline erosion management plan for northern Milwaukee County consists of a bluff slope stabilization element and a shoreline and bluff toe 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 preliminary 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 be economically feasible and implementable; and would provide—where practicable—a usable shoreline to be enjoyed by the general public as well as by lakefront property owners.

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 protection must at times be compromised—particularly when shore protection is not undertaken until a severe erosion problem has developed and real property is threatened. The plan, however, attempts to minimize adverse environmental impacts, as well as potential adverse impacts on adjacent shoreline areas, by recommending carefully selected sets of needed protection measures most appropriate for the different coastal environments within the study area. The plan seeks to ensure that the recommended

Map 24

OFFSHORE ALTERNATIVE PLAN FOR NORTHERN MILWAUKEE COUNTY



Source: SEWRPC.

Table 46

SELECTION CRITERIA AND ESTIMATED CAPITAL AND MAINTENANCE UNIT COSTS OF THE OFFSHORE ALTERNATIVE PLAN COMPONENTS

Plan Component	Criteria for Selection	Estimated Unit Cost ($/lineal foot of 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	1,000	20
Breakwater System with Nourished Sand Beach	1. Existing public sand or fine gravel beaches 2. Strong community support for a public sand beach 3. Desire to provide additional public access to restricted shoreline areas	1,500	50
Construction of a New Light Revetment	1. Shoreline areas that require substantial bluff slope regrading, and fill projects under construction in 1986 2. Areas exhibiting moderate or severe shoreline or bluff toe erosion in 1986 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 1986, required a substantial amount of repair in order to provide additional protection beyond that provided by the offshore structure	Light-100 Medium-150	10 10
Continued Maintenance of Existing Structure	1. Structure which was protecting against shoreline erosion in 1986 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.

measures will have minimal long-term harmful effects on the overall coastal environment—including the offshore bathymetry, sediments, and ecosystem.

After carefully weighing the advantages, disadvantages, and costs of the alternative plans considered, the Commission staff selected a preliminary plan comprised of the best components of each of the alternative plans considered. The preliminary plan is illustrated on Map 25. Individual plan components and associated costs are listed in Table 48.

The preliminary plan envisioned that the bluff slopes would be stabilized by regrading the

Table 47

ESTIMATED COST OF THE OFFSHORE ALTERNATIVE

Bluff Analysis Section	Shoreline Length (feet)	Plan Components and Costs						Section Costs			
			Cost per Lineal Foot			Cost per Lineal Foot					
		Onshore Components	Capital	Annual Maintenance	Offshore Components	Capital	Annual Maintenance	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual Cost
1	1,970	- -	$ - -	$- -	Island	$1,000	$20	$ 1,970,000	$ 39,400	$ 2,591,000	$ 164,000
2	950	- -	- -	- -	Island	1,000	20	950,000	19,000	1,249,000	79,000
3	300	New light revetment	150	10	Island	1,000	20	345,000	9,000	487,000	31,000
4	290	- -	- -	- -	Island	1,000	20	290,000	5,800	381,000	24,000
5	1,710	New light revetment	150	10	Island	1,000	20	1,966,000	51,300	2,775,000	176,000
6	170	Reconstruction of existing revetment-light	100	10	Island	1,000	20	187,000	5,100	267,000	17,000
7	380	New light revetment	150	10	Island	1,000	20	437,000	11,400	617,000	39,000
8	790	- -	- -	- -	Breakwater with nourished sand beach	1,500	50	1,185,000	39,500	1,808,000	115,000
	1,380	- -	- -	- -	Island	1,000	20	1,380,000	27,600	1,815,000	115,000
9	520	New light revetment	150	10	Island	1,000	20	598,000	15,600	844,000	54,000
10	240	- -	- -	- -	Island	1,000	20	240,000	4,800	316,000	20,000
11	2,370	New light revetment	150	10	Island	1,000	20	2,725,000	71,100	3,847,000	244,000
12	,850	New light revetment	150	10	Island	1,000	20	978,000	25,500	1,380,000	88,000
13	190	New light revetment	150	10	Island	1,000	20	218,000	5,700	308,000	20,000
14	160	New light revetment	150	10	Island	1,000	20	184,000	4,800	260,000	16,000
15	310	New light revetment	150	10	Island	1,000	20	356,000	9,300	503,000	32,000
16	360	New light revetment	150	10	Island	1,000	20	414,000	10,800	584,000	37,000
17	810	Reconstruction of existing revetment-light	100	10	Island	1,000	20	891,000	24,300	1,274,000	81,000
18	600	New light revetment	150	10	Island	1,000	20	690,000	18,000	974,000	62,000
	1,060	- -	- -	- -	Island	1,000	20	1,060,000	21,200	1,394,000	88,000
19	1,480	Reconstruction of existing revetment-light	100	10	Island	1,000	20	1,628,000	44,400	2,328,000	148,000
20	130	- -	- -	- -	Island	1,000	20	130,000	2,600	171,000	11,000
21	1,700	Reconstruction of existing revetment-light	100	10	Island	1,000	20	1,870,000	51,000	2,674,000	170,000
	1,270	New light revetment	150	10	Island	1,000	20	1,460,000	38,100	2,061,000	131,000
22	490	New light revetment	150	10	Island	1,000	20	564,000	14,700	796,000	50,000
23	140	New light revetment	150	10	Island	1,000	20	161,000	4,200	227,000	14,000
24	430	New light revetment	150	10	Island	1,000	20	494,000	12,900	697,000	44,000
25	480	- -	- -	- -	Breakwater with nourished sand beach	1,500	50	720,000	24,000	1,098,000	70,000
26	170	- -	- -	- -	Island	1,000	20	170,000	3,400	224,000	14,000
27	1,950	- -	- -	- -	Island	1,000	20	1,950,000	39,000	2,565,000	163,000
28	1,150	New light revetment	150	10	Island	1,000	20	1,322,000	34,500	1,866,000	118,000
29	320	New light revetment	150	10	Island	1,000	20	368,000	9,600	519,000	33,000
30	470	Reconstruction of existing revetment-light	100	10	Island	1,000	20	517,000	14,100	739,000	47,000
31	510	Continued maintenance of existing structures	- -	35	Island	1,000	20	510,000	28,000	951,000	60,000
32	770	Continued maintenance of existing structures	- -	20	Island	1,000	20	770,000	30,800	1,255,000	80,000
33	530	New light revetment	150	10	Island	1,000	20	609,000	15,900	860,000	55,000
34	1,460	New light revetment	150	10	Island	1,000	20	1,679,000	43,800	2,369,000	150,000
35A	2,390	Continued maintenance of existing structures	- -	10	- -	- -	- -	- -	23,900	377,000	24,000
35B	1,600	- -	- -	- -	Breakwater with nourished sand beach	1,500	50	2,400,000	80,000	3,661,000	232,000
35C	3,000	Continued maintenance of existing structures	- -	10	- -	- -	- -	- -	30,000	473,000	30,000
35D	720	Reconstruction of existing revetment-medium	150	10	- -	- -	- -	108,000	7,200	221,000	14,000
35E	1,360	Continued maintenance of existing structures	- -	10	- -	- -	- -	- -	13,600	214,000	14,000
36	840	- -	- -	- -	Breakwater with nourished sand beach	1,000	50	840,000	42,000	1,502,000	95,000
		Total	- -	- -	- -	- -	- -	$35,334,000	$1,026,900	$51,522,000	$3,269,000

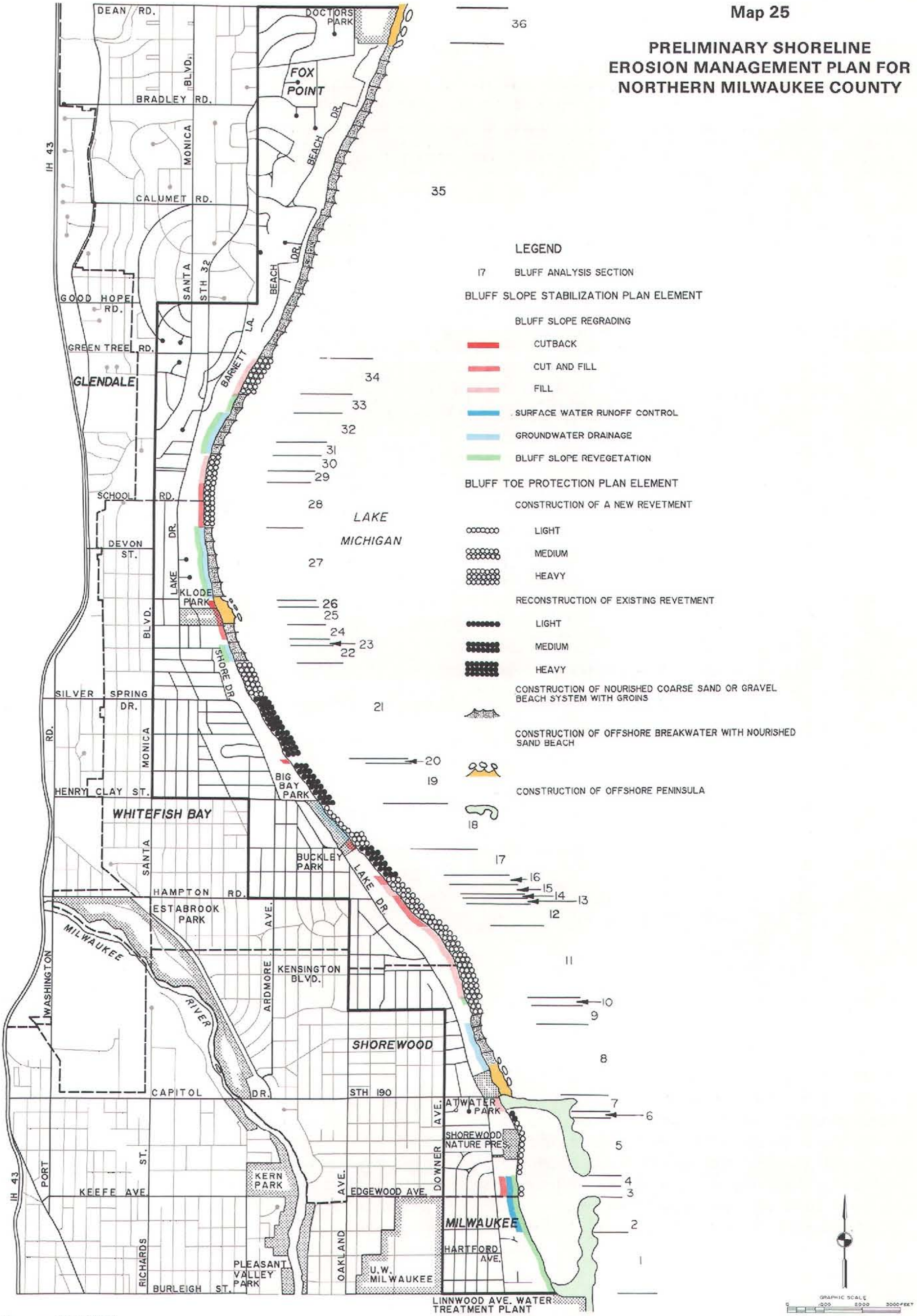
Source: SEWRPC.

slopes, revegetating the slopes, and constructing groundwater and surface water drainage systems, as described in the section on the bluff slope stabilization plan element. The slope stabilization measures would entail a capital cost of about $1.9 million and an annual maintenance cost of about $228,000, with an equivalent annual cost over a 50-year period of $222,000.

The preliminary plan recommended that two offshore peninsulas be constructed—one extend-

Map 25

PRELIMINARY SHORELINE EROSION MANAGEMENT PLAN FOR NORTHERN MILWAUKEE COUNTY



Source: SEWRPC.

Table 48

ESTIMATED COST OF THE PRELIMINARY SHORELINE EROSION MANAGEMENT PLAN FOR NORTHERN MILWAUKEE COUNTY

BLUFF SLOPE STABILIZATION							
Civil Division	Bluff Analysis Section	Shoreline Length (feet)	Plan Component	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual Cost
City of Milwaukee	1	1,970	Revegetation	$ 20,000	$ 3,900[a]	$ 30,000	$ 2,000
	2	950	Revegetation, surface water control	14,000	2,800[a]	22,000	1,000
Village of Shorewood	3	300	Bluff slope regrading—cut and fill, surface water control, revegetation	30,000	3,000[a]	38,000	2,000
	4	290	Bluff slope regrading—cut and fill, surface water control, revegetation	23,000	2,900[a]	31,000	2,000
	5	1,710	- -	- -	- -	- -	- -
	6	170	- -	- -	- -	- -	- -
	7	380	Bluff slope regrading—fill	57,000	5,700[a]	72,000	5,000
	8	790	- -	- -	- -	- -	- -
	8	1,380	Groundwater drainage	69,000	13,800	287,000	18,000
	9	520	- -	- -	- -	- -	- -
	10	240	Revegetation	4,000	700[a]	6,000	< 1,000
	11	2,370	Bluff slope regrading—fill	356,000	35,600[a]	419,000	27,000
Village of Whitefish Bay	12	850	Bluff slope regrading—cut and fill	128,000	12,800[a]	162,000	10,000
	13	190	Bluff slope regrading—cut and fill	28,000	2,800[a]	283,000	18,000
	14	160	Bluff slope regrading—cut and fill	24,000	2,400[a]	30,000	2,000
	15	310	Bluff slope regrading—fill	46,000	4,600[a]	59,000	4,000
	16	360	Bluff slope regrading—cut and fill	54,000	5,400[a]	68,000	4,000
	17	810	- -	- -	- -	- -	- -
	18	600	Bluff slope regrading—cut and fill	90,000	9,000[a]	114,000	7,000
	18	1,060	Groundwater drainage	53,000	10,600	220,000	14,000
	19	1,480	- -	- -	- -	- -	- -
	20	130	Bluff slope regrading—cutback	13,000	1,300[a]	16,000	1,000
	21	2,970	- -	- -	- -	- -	- -
	22	490	Groundwater drainage, revegetation	32,000	5,900[b]	112,000	7,000
	23	140	- -	- -	- -	- -	- -
	24	430	Bluff slope regrading—cut and fill	64,000	6,400[a]	82,000	5,000
	25	480	Bluff slope regrading—cut and fill	18,000	1,800[a]	23,000	1,000
	26	170	Bluff slope regrading—cutback	26,000	2,600[a]	32,000	2,000
	27	1,950	Groundwater drainage, revegetation	117,000	23,400[b]	435,000	28,000
	28	1,150	Bluff slope regrading—cut and fill	172,000	17,200[a]	219,000	14,000
Village of Fox Point	29	320	Bluff slope regrading—fill	48,000	4,800[a]	61,000	4,000
	30	470	Bluff slope regrading—fill	70,000	7,000[a]	89,000	6,000
	31	510	Groundwater drainage, revegetation	33,000	6,100[b]	116,000	7,000
	32	770	Groundwater drainage, revegetation	62,000	12,300[b]	195,000	12,000
	33	530	Revegetation	8,000	1,600[a]	12,000	1,000
	34	1,460	Bluff slope regrading—fill	219,000	21,900[a]	278,000	18,000
	35	9,070	- -	- -	- -	- -	- -
	36	840	- -	- -	- -	- -	- -
	Total	38,770	- -	$1,878,000	$228,300[c]	$3,511,000	$222,000

ing northward from the City of Milwaukee Linnwood Avenue water treatment plant and one extending southward from the southern end of Atwater Park. The peninsula extending northward would form an extension of Milwaukee County's Lake Park, and would help protect the Linnwood Avenue water treatment plant. The peninsula extending from the Village of Shorewood Atwater Park would provide an extension of that park and help protect the village nature preserve located in Bluff Analysis Section 5. These offshore peninsulas would protect a total of 5,770 feet of shoreline, or 15 percent of the shoreline of the study area. The recommended peninsulas—along with required supplementary onshore revetments—would

Table 48 (continued)

BLUFF TOE PROTECTION							
Civil Division	Bluff Analysis Section	Shoreline Length (feet)	Plan Component	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual Cost
City of Milwaukee	1	1,970	Island	$ 1,970,000	$ 39,400	$ 2,591,000	$ 164,000
	2	950	Island	950,000	19,000	1,249,000	79,000
Village of Shorewood	3	300	Island	300,000	6,000	395,000	25,000
		300	Construction of light revetment	45,000	3,000	92,000	6,000
	4	290	Island	290,000	5,800	381,000	24,000
	5	1,710	Island	1,710,000	34,200	2,249,000	143,000
		1,710	Construction of light revetment	256,000	17,100	526,000	33,000
	6	170	Island	170,000	3,400	224,000	14,000
		170	Construction of light revetment	26,000	1,700	53,000	3,000
	7	380	Island	380,000	7,600	500,000	32,000
		380	Construction of light revetment	57,000	3,800	117,000	7,000
	8	790	Offshore breakwater with nourished sand beach	1,185,000	39,500	1,808,000	115,000
		1,380	Nourished beach system	552,000	27,600	987,000	63,000
	9	280	Nourished beach system	112,000	5,600	200,000	13,000
		240	Construction of medium revetment	60,000	3,600	117,000	7,000
	10	240	Construction of medium revetment	60,000	3,600	117,000	7,000
Village of Whitefish Bay	11	2,370	Construction of heavy revetment	830,000	47,400	1,577,000	100,000
	12	850	Construction of heavy revetment	298,000	17,000	566,000	36,000
	13	190	Construction of heavy revetment	66,000	3,800	126,000	8,000
	14	160	Construction of heavy revetment	56,000	3,200	106,000	7,000
	15	310	Construction of heavy revetment	108,000	6,200	206,000	13,000
	16	360	Construction of heavy revetment	126,000	7,200	239,000	15,000
	17	810	Reconstruction of revetment-heavy	243,000	16,200	498,000	32,000
	18	600	Construction of heavy revetment	210,000	12,000	399,000	25,000
		1,060	Construction of light revetment	159,000	10,600	326,000	21,000
	19	1,480	Reconstruction of revetment-heavy	444,000	29,600	911,000	58,000
	20	130	No toe protection	0	0	0	0
	21	1,700	Reconstruction of revetment-heavy	510,000	34,000	1,046,000	66,000
		1,270	Construction of heavy revetment	444,000	25,400	844,000	54,000
	22	490	Nourished beach system	196,000	9,800	350,000	22,000
	23	140	Nourished beach system	56,000	2,800	100,000	6,000
	24	430	Nourished beach system	172,000	8,600	308,000	20,000
	25	480	Offshore breakwater with nourished sand beach	720,000	24,000	1,098,000	70,000
	26	170	Nourished beach system	68,000	3,400	122,000	8,000
	27	1,950	Nourished beach system	780,000	39,000	1,395,000	89,000
Village of Fox Point	28	1,150	Construction of medium revetment	288,000	17,200	559,000	35,000
	29	320	Construction of medium revetment	80,000	4,800	156,000	10,000
	30	470	Reconstruction of revetment-medium	94,000	7,000	204,000	13,000
	31	510	Nourished beach system	204,000	10,200	365,000	23,000
	32	770	Nourished beach system	308,000	15,400	551,000	35,000
	33	530	Nourished beach system	212,000	10,600	379,000	24,000
	34	1,460	Construction of heavy revetment	511,000	29,200	971,000	62,000
	35	9,070	Nourished beach system	3,628,000	181,400	6,487,000	412,000
	36	840	Offshore breakwater with nourished sand beach	1,260,000	42,000	1,922,000	122,000
	Total	38,770	- -	$20,194,000	$838,900	$33,417,000	$2,121,000

Table 48 (continued)

PRELIMINARY TOTAL PLAN						
Civil Division	Bluff Analysis Section	Shoreline Length (feet)	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual Cost
City of Milwaukee	1	1,970	$ 1,990,000	$ 43,300	$ 2,621,000	$ 166,000
	2	950	964,000	21,800	1,271,000	80,000
Village of Shorewood	3	300	375,000	12,000	525,000	33,000
	4	290	313,000	8,700	412,000	26,000
	5	1,710	1,966,000	51,300	2,775,000	176,000
	6	170	196,000	5,100	277,000	17,000
	7	380	494,000	17,100	689,000	44,000
	8	2,170	1,806,000	80,900	3,082,000	196,000
	9	520	172,000	9,200	317,000	20,000
	10	240	64,000	4,300	123,000	7,000
	11	2,370	1,186,000	83,000	1,996,000	127,000
Village of Whitefish Bay	12	850	426,000	29,800	728,000	46,000
	13	190	94,000	6,600	409,000	26,000
	14	160	80,000	5,600	136,000	9,000
	15	310	154,000	10,800	265,000	17,000
	16	360	180,000	12,600	307,000	19,000
	17	810	243,000	16,200	498,000	32,000
	18	1,660	512,000	42,200	1,059,000	67,000
	19	1,480	444,000	29,600	911,000	58,000
	20	130	13,000	1,300	16,000	1,000
	21	2,970	954,000	59,400	1,890,000	120,000
	22	490	228,000	15,700	462,000	29,000
	23	140	56,000	2,800	100,000	6,000
	24	430	236,000	15,000	390,000	25,000
	25	480	738,000	25,800	1,121,000	71,000
	26	170	94,000	6,000	154,000	10,000
	27	1,950	897,000	62,400	1,830,000	117,000
	28	1,150	460,000	34,400	778,000	49,000
Village of Fox Point	29	320	128,000	9,600	217,000	14,000
	30	470	164,000	14,000	293,000	19,000
	31	510	237,000	16,300	481,000	30,000
	32	770	370,000	27,700	746,000	47,000
	33	530	220,000	12,200	391,000	25,000
	34	1,460	730,000	51,100	1,249,000	80,000
	35	9,070	3,628,000	181,400	6,487,000	412,000
	36	840	1,260,000	42,000	1,922,000	122,000
	Total	38,770	$22,072,000	$1,067,200	$36,928,000	$2,343,000

[a]*Annual maintenance costs would apply only for first three years following bluff slope regrading or revegetation.*

[b]*Of the total annual maintenance cost of $5,900 for bluff slope stabilization within Bluff Analysis Section 22, $1,000, or 17 percent, would be required only for the first three years following revegetation. Of the total annual maintenance cost of $23,400 for bluff slope stabilization within Bluff Analysis Section 27, $3,900, or 17 percent, would be required only for the first three years following revegetation. Of the total annual maintenance cost of $12,300 for bluff slope stabilization within Bluff Analysis Section 32, $4,600, or 37 percent, would be required only for the first three years following revegetation. Of the total annual maintenance cost of $6,100 for bluff slope stabilization within Bluff Analysis Section 31, $1,000, or 16 percent, would be required only for the first three years following revegetation.*

[c]*About $166,700, or 73 percent, of the total annual maintenance cost of the bluff slope stabilization plan element would be required only for the first three years following bluff slope regrading or revegetation.*

Source: SEWRPC.

entail a capital cost of about $6.1 million and an annual maintenance cost of about $141,000, with an equivalent annual cost over a 50-year period of $530,000.

Under the preliminary plan, nourished sand beaches contained by offshore breakwaters would be constructed at Atwater Park, Klode Park, and Doctors Park. These breakwaters would protect about 2,110 feet of shoreline, or 5 percent of the study area shoreline. These breakwaters would contain about nine acres of public sand beach, substantially increasing recreational opportunities for swimming and sunbathing within the study area. The recommended sand beach systems would entail a capital cost of about $3.2 million and an annual maintenance cost of about $105,500, with an equivalent annual cost over a 50-year period of $307,000.

The preliminary plan envisioned that nourished gravel beach systems contained by short groins would be located just north of Atwater Park, both north and south of Klode Park, along a small portion of the Fox Point bluff, and along the entire Fox Point terrace. These nourished gravel beach systems would protect about 15,720 feet of shoreline, or 41 percent of the study area shoreline. They would entail a capital cost of about $6.3 million and an annual maintenance cost of about $314,000, with an equivalent annual cost over a 50-year period of $715,000.

The preliminary plan proposed that quarry stone revetments be constructed or reconstructed to protect nearly all existing or proposed bluff slope fill projects. Beaches were 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 about 15,040 feet of shoreline, or approximately 39 percent of the study area shoreline. The revetments would entail a capital cost of about $4.6 million, an annual maintenance cost of about $278,000, and an equivalent annual cost over a 50-year period of about $569,000.

The total capital cost of the preliminary shoreline management plan was estimated to be $22.1 million, and the annual maintenance cost, $1.1 million. The equivalent annual cost of the preliminary plan over a 50-year period was estimated to be $2.3 million. The public and private sector costs of the plan within each civil division are summarized in Table 49. Of the total equivalent annual cost of the preliminary plan, about $1.0 million, or 43 percent, would be financed by the public sector, and $1.3 million, or 57 percent, would be financed by the private sector.

The plan costs are best estimates at the systems planning level. Depending on site-specific characteristics, individual projects may cost substantially more or less than herein estimated. Where new 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 would eventually decrease as engineers and contractors became more familiar with the structure designs that 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 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 many areas.

2. The plan envisions new public facilities, inclusive of new lakefront parkland and large sand beaches, which will increase the opportunity for enjoyment of the lakeshore by the general public.

3. The plan recommends the creation of extensive reaches of gravel beaches which would greatly increase the usability of the immediate shoreline and access to the water, primarily for the lakeshore private property owners.

4. The plan proposes that revetments be constructed to provide effective toe protection at the base of all existing or new bluff fill projects.

It is also recommended that low-cost general shoreline management practices be followed by lakefront property owners, and that such owners consider the impact of land use or disturbance activities on the stability of the bluff slopes and

Table 49

DISTRIBUTION OF THE ESTIMATED COST OF THE PRELIMINARY SHORELINE EROSION MANAGEMENT PLAN

Civil Division	Public or Private Sector	Capital		Annual Maintenance		50-Year Present Worth		Equivalent Annual Cost	
		Cost	Percent of Total	Cost	Percent of Total	Cost	Percent of Total	Cost	Percent of Total
City of Milwaukee	Public[a]	$ 2,920,000	13.2	$ 58,400	5.5	$ 3,840,000	10.4	$ 243,000	10.4
	Private	34,000	0.2	6,700	0.6	52,000	0.1	3,000	0.1
	Subtotal	$ 2,954,000	13.4	$ 65,100	6.1	$ 3,892,000	10.5	$ 246,000	10.5
Village of Shorewood	Public	$ 4,291,000	19.5	$ 113,600	10.6	$ 6,083,000	16.5	$ 386,000	16.5
	Private	1,498,000	6.8	103,300	9.7	2,795,000	7.6	176,000	7.5
	Subtotal	$ 5,789,000	26.3	$ 216,900	20.3	$ 8,878,000	24.1	$ 562,000	24.0
Village of Whitefish Bay	Public[b]	$ 1,286,000	5.8	$ 70,000	6.6	$ 2,253,000	6.1	$ 142,000	6.1
	Private	5,062,000	22.9	342,700	32.1	9,707,000	26.3	617,000	26.3
	Subtotal	$ 6,348,000	28.7	$ 412,700	38.7	$11,960,000	32.4	$ 759,000	32.4
Village of Fox Point	Public[c]	$ 2,254,000	10.2	$ 93,200	8.7	$ 3,702,000	10.0	$ 235,000	10.0
	Private	4,727,000	21.4	279,500	26.2	8,496,000	23.0	541,000	23.1
	Subtotal	$ 6,981,000	31.6	$ 372,500	34.9	$12,198,000	33.0	$ 776,000	33.1
Study Area Total	Public	$10,751,000	48.8	$ 335,000	31.4	$15,878,000	43.0	$1,006,000	43.0
	Private	$11,321,000	51.2	$ 732,200	68.6	$21,050,000	57.0	$1,337,000	57.0
	Total	$22,072,000	100.0	$1,067,200[d]	100.0	$36,928,000	100.0	$2,343,000	100.0

[a] *The offshore peninsula extending northward from the Linnwood Avenue water treatment plant, which would be the only public shore protection measure within the City of Milwaukee in the study area, could be jointly implemented by the City of Milwaukee and Milwaukee County.*

[b] *A groundwater drainage system and a light revetment at Big Bay Park would be implemented by Milwaukee County at a capital cost of about $212,000, and an annual maintenance cost of about $21,200.*

[c] *The offshore breakwater with a sand beach at Doctors Park would be implemented by Milwaukee County at a capital cost of about $1,260,000 and an annual maintenance cost of about $42,000.*

[d] *Of the total maintenance cost of $1,067,200, $166,700, or 16 percent, would be required only for the first three years following bluff slope regrading or revegetation.*

Source: SEWRPC.

the protection of the shoreline. More specifically, 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. 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, 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 123 and 124 as advisory in the administration of their zoning and subdivision control ordinances. Provision should be made for the modification of the setback distances upon submittal to the local units of government of an acceptable engineering study report which clearly indicates not only that the property would be adequately protected at a different setback distance, but that the stability of the bluffs would not be adversely affected.

RECOMMENDED SHORELINE EROSION MANAGEMENT PLAN

Following consideration of the preliminary plan recommendations, several modifications to the plan were made by the Advisory Committee. The recommended plan is shown in graphic summary form on Map 26. The recommended plan reflects local concerns and desires expressed by and through the Committee members.

The key revisions to the preliminary plan made by the Advisory Committee are as follows:

1. The two offshore peninsulas—one which would have extended northward from Lake Park in the City of Milwaukee, and another which would have extended southward from Atwater Park in the Village of Shorewood—were eliminated from the plan. Instead, the shoreline south of Atwater Park in the Village of Shorewood and all but the southernmost 900 feet of shoreline in the study area within the City of Milwaukee—which requires no protection—would be protected by a nourished gravel beach.

   The Committee decided that the peninsulas should not be recommended because the cost was relatively high; there was no great desire expressed by the Village of Shorewood for additional lakefront parkland; the provision of additional lakefront parkland would also likely be a low priority in Milwaukee County at this time; and there were concerns about access and law enforcement problems. The recommended beaches would provide a usable shoreline and suitable lakefront access for construction and maintenance of the recommended shore protection measures.

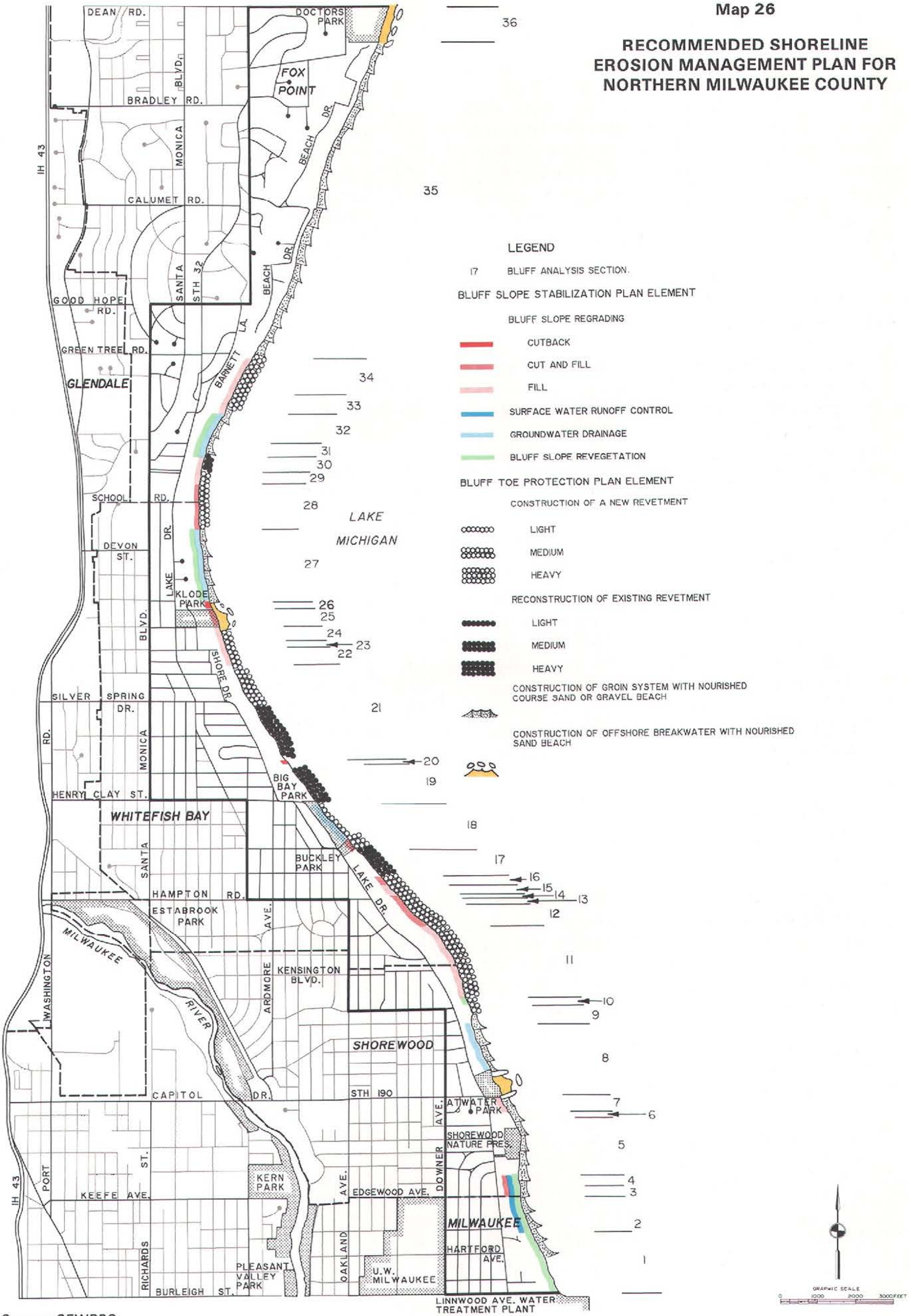
2. The recommended type of bluff toe protection for two shoreline areas was changed from a nourished gravel beach to a quarry stone revetment. The first shoreline area—which consists of Bluff Analysis Sections 22, 23, and 24—extends from 5722 to 5866 N. Shore Drive. The second shoreline area—Bluff Analysis Section 33—extends from 6820 to 6840 N. Barnett Lane. In both shoreline areas, projects were underway in 1987 to construct revetments and place fill on the bluff slopes. The bluff slope stabilization plan element was also revised to include bluff slope regrading with fill within those shoreline areas. These revisions to the preliminary plan reflect the need to incorporate into the plan committed shore protection actions which were made in the absence of an adopted shoreline erosion management plan.

The components and associated costs of the recommended plan are listed in Table 50. The slope stabilization measures would entail a capital cost of about $2.0 million and an annual maintenance cost of about $238,000, with an equivalent annual cost over a 50-year period of $228,000. The bluff toe protection measures would entail a capital cost of about $15.8 million, an annual maintenance cost of about $790,000, and an equivalent annual cost of about $1,795,000. The total capital cost of the recommended shoreline erosion management plan is approximately $17.8 million, and the annual maintenance cost, about $1,029,000. The equivalent annual cost of the recommended plan over a 50-year period is approximately $2.0 million. The public and private sector costs of the recommended plan within each civil division are set forth in Table 51. Of the total equivalent annual cost of the plan, about $0.6 million, or 28 percent, would be financed by the public sector, and $1.4 million, or 72 percent, would be financed by the private sector.

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 plan provides guidance for plan implementation, and serves as a point of departure for the necessary site-specific analyses. Adoption and implementation of the recommended plan should ensure the provision of a high-quality, well-managed coastal environment for northern Milwaukee County.

Map 26

RECOMMENDED SHORELINE EROSION MANAGEMENT PLAN FOR NORTHERN MILWAUKEE COUNTY



Source: SEWRPC.

Table 50

ESTIMATED COST OF THE RECOMMENDED SHORELINE EROSION MANAGEMENT PLAN FOR NORTHERN MILWAUKEE COUNTY

BLUFF SLOPE STABILIZATION							
Civil Division	Bluff Analysis Section	Shoreline Length (feet)	Plan Component	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual Cost
City of Milwaukee	1	1,970	Revegetation	$ 20,000	$ 3,900[a]	$ 30,000	$ 2,000
	2	950	Revegetation, surface water control	14,000	2,800[a]	22,000	1,000
Village of Shorewood	3	300	Bluff slope regrading—cut and fill, surface water control, revegetation	30,000	3,000[a]	38,000	2,000
	4	290	Bluff slope regrading—cut and fill, surface water control, revegetation	23,000	2,900[a]	31,000	2,000
	5	1,710	- -	- -	- -	- -	- -
	6	170	- -	- -	- -	- -	- -
	7	380	Bluff slope regrading—fill	57,000	5,700[a]	72,000	5,000
	8	790	- -	- -	- -	- -	- -
	8	1,380	Groundwater drainage	69,000	13,800	287,000	18,000
	9	520	- -	- -	- -	- -	- -
	10	240	Revegetation	4,000	700[a]	6,000	< 1,000
	11	2,370	Bluff slope regrading—fill	356,000	35,600[a]	419,000	27,000
Village of Whitefish Bay	12	850	Bluff slope regrading—cut and fill	128,000	12,800[a]	162,000	10,000
	13	190	Bluff slope regrading—cut and fill	28,000	2,800[a]	283,000	18,000
	14	160	Bluff slope regrading—cut and fill	24,000	2,400[a]	30,000	2,000
	15	310	Bluff slope regrading—fill	46,000	4,600[a]	59,000	4,000
	16	360	Bluff slope regrading—cut and fill	54,000	5,400[a]	68,000	4,000
	17	810	- -	- -	- -	- -	- -
	18	600	Bluff slope regrading—cut and fill	90,000	9,000[a]	114,000	7,000
	18	1,060	Groundwater drainage	53,000	10,600	220,000	14,000
	19	1,480	- -	- -	- -	- -	- -
	20	130	Bluff slope regrading—cutback	13,000	1,300[a]	16,000	1,000
	21	2,970	- -	- -	- -	- -	- -
	22	490	Bluff slope regrading—fill	74,000	7,400[a]	93,000	6,000
	23	140	Bluff slope regrading—fill	21,000	2,100[a]	27,000	2,000
	24	430	Bluff slope regrading—fill	64,000	6,400[a]	82,000	5,000
	25	480	Bluff slope regrading—cut and fill	18,000	1,800[a]	23,000	1,000
	26	170	Bluff slope regrading—cutback	26,000	2,600[a]	32,000	2,000
	27	1,950	Groundwater drainage, revegetation	117,000	23,400[b]	435,000	28,000
	28	1,150	Bluff slope regrading—cut and fill	172,000	17,200[a]	219,000	14,000
Village of Fox Point	29	320	Bluff slope regrading—fill	48,000	4,800[a]	61,000	4,000
	30	470	Bluff slope regrading—fill	70,000	7,000[a]	89,000	6,000
	31	510	Groundwater drainage, revegetation	33,000	6,100[b]	116,000	7,000
	32	770	Groundwater drainage, revegetation	62,000	12,300[b]	195,000	12,000
	33	530	Bluff slope regrading—fill	80,000	8,000[a]	101,000	6,000
	34	1,460	Bluff slope regrading—fill	219,000	21,900[a]	278,000	18,000
	35	9,070	- -	- -	- -	- -	- -
	36	840	- -	- -	- -	- -	- -
	Total	38,770	- -	$ 2,013,000	$238,300[c]	$ 3,608,000	$ 228,000

PLAN IMPLEMENTATION

The recommended bluff stabilization and shore protection plan for the Lake Michigan shoreline of northern Milwaukee County as described in the foregoing section of this chapter requires proper implementation throughout entire reaches of shoreline having similar physiographic characteristics. The recommended control measures—bluff slope regrading with quarry stone revetment toe protection; artificially nourished gravel beach systems, sometimes with additional slope stabilization measures; and sand beaches contained by offshore breakwaters—cannot be properly implemented on a piecemeal basis. To ensure proper design and

Table 50 (continued)

BLUFF TOE PROTECTION								
Civil Division	Bluff Analysis Section	Shoreline Length (feet)	Plan Component	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual Cost
City of Milwaukee	1	880	No toe protection	$ 0	$ 0	$ 0	$ 0
		1,090	Nourished beach system	436,000	21,800	780,000	49,000
	2	950	Nourished beach system	380,000	19,000	679,000	43,000
Village of Shorewood	3	300	Nourished beach system	120,000	6,000	215,000	14,000
	4	290	Nourished beach system	116,000	5,800	207,000	13,000
	5	1,710	Nourished beach system	684,000	34,200	1,223,000	78,000
	6	170	Nourished beach system	68,000	3,400	122,000	8,000
	7	380	Nourished beach system	152,000	7,600	272,000	17,000
	8	790	Offshore breakwater with nourished sand beach	1,185,000	39,500	1,808,000	115,000
		1,380	Nourished beach system	552,000	27,600	987,000	63,000
	9	280	Nourished beach system	112,000	5,600	200,000	13,000
		240	Construction of medium revetment	60,000	3,600	117,000	7,000
	10	240	Construction of medium revetment	60,000	3,600	117,000	7,000
	11	2,370	Construction of heavy revetment	830,000	47,400	1,577,000	100,000
Village of Whitefish Bay	12	850	Construction of heavy revetment	298,000	17,000	566,000	36,000
	13	190	Construction of heavy revetment	66,000	3,800	126,000	8,000
	14	160	Construction of heavy revetment	56,000	3,200	106,000	7,000
	15	310	Construction of heavy revetment	108,000	6,200	206,000	13,000
	16	360	Construction of heavy revetment	126,000	7,200	239,000	15,000
	17	810	Reconstruction of revetment-heavy	243,000	16,200	498,000	32,000
	18	600	Construction of heavy revetment	210,000	12,000	399,000	25,000
		1,060	Construction of light revetment	159,000	10,600	326,000	21,000
	19	1,480	Reconstruction of revetment-heavy	444,000	29,600	911,000	58,000
	20	130	No toe protection	0	0	0	0
	21	1,700	Reconstruction of revetment-heavy	510,000	34,000	1,046,000	66,000
		1,270	Construction of heavy revetment	444,000	25,400	844,000	54,000
	22	490	Construction of medium revetment	122,000	7,400	238,000	15,000
	23	140	Construction of medium revetment	35,000	2,100	68,000	4,000
	24	430	Construction of medium revetment	108,000	6,400	209,000	13,000
	25	480	Offshore breakwater with nourished sand beach	720,000	24,000	1,098,000	70,000
	26	170	Nourished beach system	68,000	3,400	122,000	8,000
	27	1,950	Nourished beach system	780,000	39,500	1,395,000	89,000
	28	1,150	Construction of medium revetment	288,000	17,200	559,000	35,000
Village of Fox Point	29	320	Construction of medium revetment	80,000	4,800	156,000	10,000
	30	470	Reconstruction of revetment-medium	94,000	7,000	204,000	13,000
	31	510	Nourished beach system	204,000	10,200	365,000	23,000
	32	770	Nourished beach system	308,000	15,400	551,000	35,000
	33	530	Construction of heavy revetment	186,000	10,600	353,000	22,000
	34	1,460	Construction of heavy revetment	511,000	29,200	971,000	62,000
	35	9,070	Nourished beach system	3,628,000	181,400	6,487,000	412,000
	36	840	Offshore breakwater with nourished sand beach	1,260,000	42,000	1,922,000	122,000
	Total	38,770	- -	$15,811,000	$790,400	$28,269,000	$1,795,000

maintenance, and to minimize construction impacts, these measures can be successfully implemented only along entire specified reaches of shoreline. These reaches, referred to as implementation segments, are shown on Map 27. The shoreline length and location of each segment, along with the recommended shore protection measures, are provided in Table 52. There are 18 implementation segments within the study area, with shoreline lengths ranging from 480 to 6,130 feet. The shoreline contained within each segment would, under the recommended plan, have a relatively uniform type of shore protection, and implementation of a

Table 50 (continued)

RECOMMENDED TOTAL PLAN						
Civil Division	Bluff Analysis Section	Shoreline Length (feet)	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual Cost
City of Milwaukee	1	1,970	$ 456,000	$ 25,700	$ 810,000	$ 51,000
	2	950	394,000	21,800	701,000	44,000
Village of Shorewood	3	300	150,000	9,000	253,000	16,000
	4	290	139,000	8,700	238,000	15,000
	5	1,710	684,000	34,200	1,123,000	78,000
	6	170	68,000	3,400	122,000	8,000
	7	380	209,000	13,300	344,000	22,000
	8	2,170	1,806,000	80,900	3,082,000	196,000
	9	520	172,000	9,200	317,000	20,000
	10	240	64,000	4,300	123,000	7,000
	11	2,370	1,186,000	83,000	1,996,000	127,000
Village of Whitefish Bay	12	850	426,000	29,800	728,000	46,000
	13	190	94,000	6,600	409,000	26,000
	14	160	80,000	5,600	136,000	9,000
	15	310	154,000	10,800	265,000	17,000
	16	360	180,000	12,600	307,000	19,000
	17	810	243,000	16,200	498,000	32,000
	18	1,660	512,000	42,200	1,059,000	67,000
	19	1,480	444,000	29,600	911,000	58,000
	20	130	13,000	1,300	16,000	1,000
	21	2,970	954,000	59,400	1,890,000	120,000
	22	490	196,000	14,800	331,000	21,000
	23	140	56,000	4,200	95,000	6,000
	24	430	172,000	12,800	291,000	18,000
	25	480	738,000	25,800	1,121,000	71,000
	26	170	94,000	6,000	154,000	10,000
	27	1,950	897,000	62,400	1,830,000	117,000
	28	1,150	460,000	34,400	778,000	49,000
Village of Fox Point	29	320	128,000	9,600	217,000	14,000
	30	470	164,000	14,000	293,000	19,000
	31	510	237,000	16,300	481,000	30,000
	32	770	370,000	27,700	746,000	47,000
	33	530	266,000	18,600	454,000	28,000
	34	1,460	730,000	51,100	1,249,000	80,000
	35	9,070	3,628,000	181,400	6,487,000	412,000
	36	840	1,260,000	42,000	1,922,000	122,000
	Total	38,770	$17,824,000	$1,028,700	$31,877,000	$2,023,000

[a] *Annual maintenance costs would apply only for first three years following bluff slope regrading or revegetation.*

[b] *Of the total annual maintenance cost of $23,400 for stabilizing the bluff slope within Bluff Analysis Section 27, $3,900, or 17 percent, would be required only for the first three years following revegetation. Of the total annual maintenance cost of $6,100 for stabilizing the bluff slope within Bluff Analysis Section 31, $1,000, or 16 percent, would be required only for the first three years following revegetation. Of the total annual maintenance cost of $12,300 for stabilizing the bluff slope within Bluff Analysis Section 32, $4,600, or 37 percent, would be required only for the first three years following revegetation.*

[c] *About $181,600, or 76 percent, of the total annual maintenance cost of the bluff slope stabilization plan element would be required only for the first three years following bluff slope regrading or revegetation.*

Source: SEWRPC.

project within an entire specified segment would not be expected to have an adverse effect on adjacent segments.

Map 27 also identifies the proposed general location of nine permanent access sites which would be used for the construction and continued maintenance of the recommended shore protection measures. The locations of these access sites are general, with the specific locations to be determined based upon the needs of individual projects. Each of the sites could contain a permanent roadway, suitable for trucks and heavy construction equipment, extending down

Table 51

DISTRIBUTION OF THE ESTIMATED COST OF THE RECOMMENDED SHORELINE EROSION MANAGEMENT PLAN

Civil Division	Public or Private Sector	Capital		Annual Maintenance		50-Year Present Worth		Equivalent Annual Cost	
		Cost	Percent of Total	Cost	Percent of Total	Cost	Percent of Total	Cost	Percent of Total
City of Milwaukee	Public	$ 0	0	$ 0	0	$ 0	0	$ 0	0
	Private	850,000	4.8	47,500	4.6	1,511,000	4.7	95,000	4.7
	Subtotal	$ 850,000	4.8	$ 47,500	4.6	$ 1,511,000	4.7	$ 95,000	4.7
Village of Shorewood	Public	$ 1,869,000	10.5	$ 73,700	7.2	$ 3,031,000	9.5	$ 193,000	9.5
	Private	1,826,000	10.2	117,500	11.4	3,350,000	10.5	212,000	10.5
	Subtotal	$ 3,695,000	20.7	$ 191,200	18.6	$ 6,381,000	20.0	$ 405,000	20.0
Village of Whitefish Bay	Public[a]	$ 1,317,000	7.4	$ 72,200	7.0	$ 2,312,000	7.3	$ 146,000	7.2
	Private	4,935,000	27.7	338,900	33.0	9,412,000	29.5	599,000	29.6
	Subtotal	$ 6,252,000	35.1	$ 411,100	40.0	$11,724,000	36.8	$ 745,000	36.8
Village of Fox Point	Public[b]	$ 2,254,000	12.6	$ 93,000	9.0	$ 3,702,000	11.6	$ 235,000	11.6
	Private	4,773,000	26.8	285,900	27.8	8,559,000	26.9	543,000	26.9
	Subtotal	$ 7,027,000	39.4	$ 378,900	36.8	$12,261,000	38.5	$ 778,000	38.5
Study Area Total	Public	$ 5,440,000	30.5	$ 238,900	23.2	$ 9,045,000	28.4	$ 574,000	28.3
	Private	$12,384,000	69.5	$ 789,800	76.8	$22,832,000	71.6	$1,449,000	71.7
	Total	$17,824,000	100.0	$1,028,700[c]	100.0	$31,877,000	100.0	$2,023,000	100.0

[a] *A groundwater drainage system and a light revetment at Big Bay Park would be implemented by Milwaukee County at a capital cost of about $212,000, and an annual maintenance cost of about $21,200.*

[b] *The offshore breakwater with a sand beach at Doctors Park would be implemented by Milwaukee County at a capital cost of about $1,260,000 and an annual maintenance cost of about $42,000.*

[c] *Of the total maintenance cost of $1,028,700, $181,600, or 18 percent, would be required only for the first three years following bluff slope regrading or revegetation.*

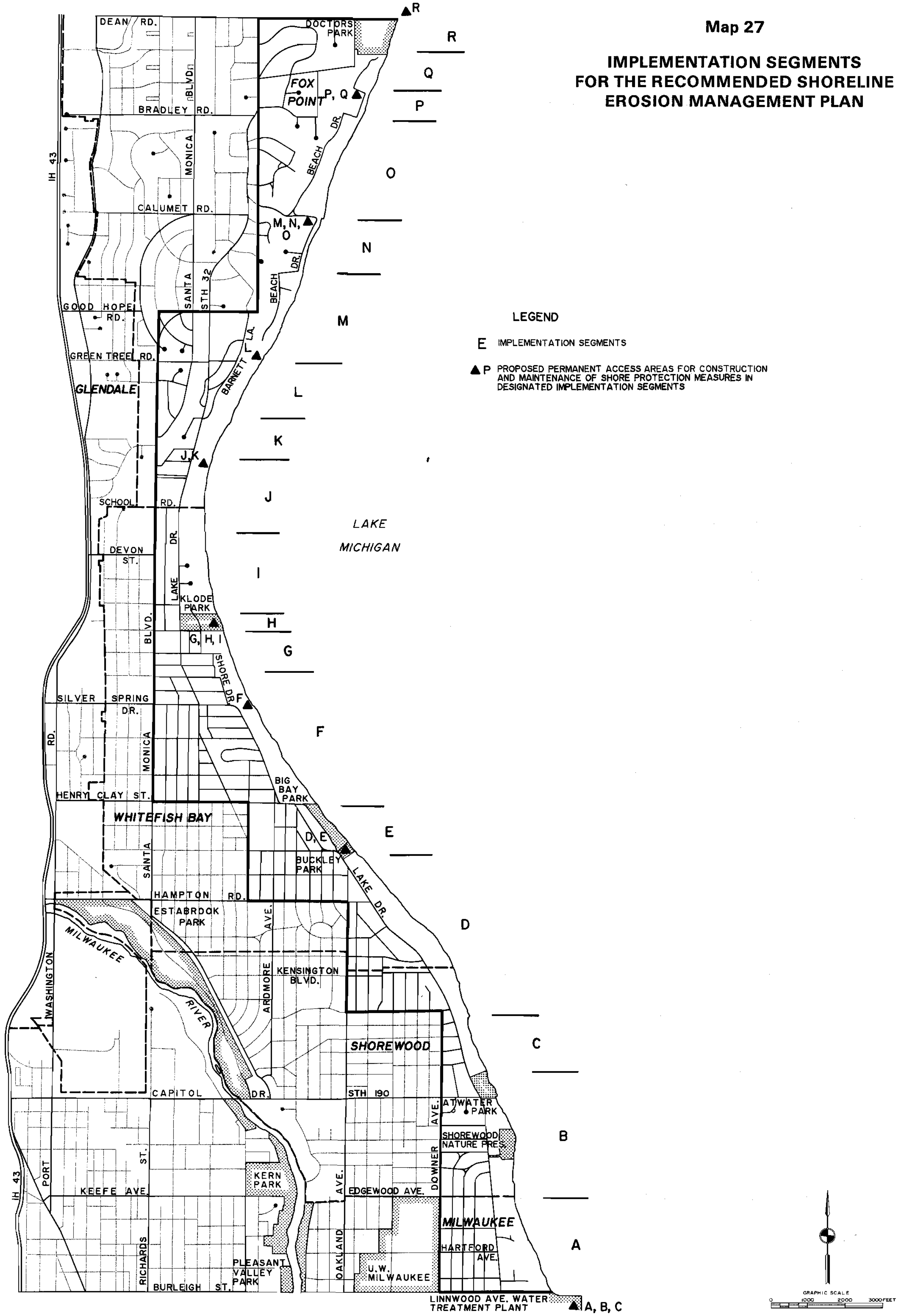
Source: SEWRPC.

to the shoreline. The implementation segments that would be served by each of the proposed access sites are also designated on the map. It is recognized that failure to implement the plan in an orderly fashion—with construction initiating at the proposed access sites—would require that additional temporary access locations be developed. Furthermore, the bluff slope regrading operations may also require additional temporary access locations in order to minimize the movement of large quantities of fill material. The provision of the proposed permanent access sites would help centralize and thereby reduce the areawide impacts caused by the movement of heavy equipment and large volumes of material.

There appear to be three basic approaches that could be taken to implement the plan, two of which would basically rely upon the existing institutional structure. A third alternative approach would involve the creation of an

Map 27

IMPLEMENTATION SEGMENTS FOR THE RECOMMENDED SHORELINE EROSION MANAGEMENT PLAN



Source: SEWRPC.

Table 52

RECOMMENDED IMPLEMENTATION SEGMENTS FOR NORTHERN MILWAUKEE COUNTY

Implementation Segment	Shoreline Length (feet)	Address	Civil Division	Bluff Analysis Sections	Recommended Bluff Slope Stabilization Measures[a]	Recommended Bluff Toe Protection Measures[a]
A	2,920	Linnwood Avenue water treatment plant-3474 N. Lake Drive	Milwaukee	1-2	Surface water runoff control, revegetation	Nourished gravel beach system
B	3,640	3510 N. Lake Drive-Atwater Park	Shorewood	3-8	Cut and fill, fill, surface water runoff control, revegetation	Nourished gravel beach system, offshore breakwater with nourished sand beach
C	1,660	4060-4240 N. Lake Drive	Shorewood	8-9	Groundwater drainage	Nourished gravel beach system with short groins
D	5,530	4300-4940 N. Lake Drive	Shorewood-Whitefish Bay	9-17	Cut and fill, fill, revegetation	Medium and heavy revetments
E	1,660	Buckley Park-Big Bay Park	Whitefish Bay	18	Groundwater drainage, cut and fill	Light revetment
F	4,580	Big Bay Park-808 Lakeview Avenue	Whitefish Bay	19-21	Cutback	Heavy revetment
G	1,060	5722-5866 N. Shore Drive	Whitefish Bay	22-24	Fill	Medium revetment
H	480	Klode Park	Whitefish Bay	25	Cut and fill	Offshore breakwater with nourished sand beach
I	2,120	5960 N. Shore Drive-6260 N. Lake Drive	Whitefish Bay	26-27	Cutback, groundwater drainage, revegetation	Nourished gravel beach system
J	1,940	6310-6530 N. Lake Drive	Whitefish Bay-Fox Point	28-30	Cut and fill, fill	Medium revetment
K	1,280	6600 N. Lake Drive-6818 N. Barnett Lane	Fox Point	31-32	Groundwater drainage, revegetation	Nourished gravel beach system
L	1,990	6820-7010 N. Barnett Lane	Fox Point	33-34	Fill	Heavy revetment
M	2,390	7038-7828 N. Beach Drive	Fox Point	35a	- -	Nourished gravel beach system
N	1,600	7405-7535 N. Beach Drive	Fox Point	35b	- -	Nourished gravel beach system
O	3,000	7540-7966 N. Beach Drive	Fox Point	35c	- -	Nourished gravel beach system
P	720	8005-8035 N. Beach Drive	Fox Point	35d	- -	Nourished gravel beach system
Q	1,360	8040-8135 N. Beach Drive	Fox Point	35e	- -	Nourished gravel beach system
R	840	Doctors Park	Fox Point	36	- -	Offshore breakwater with nourished sand beach

[a] *Not all listed slope stabilization and bluff toe protection measures may be required for the entire implementation segment.*

Source: SEWRPC.

entirely new institutional structure that would provide the basis for plan implementation. Each of these three alternatives is described below.

Maintain Existing Institutional Structure—County-Based Coordination

Under this approach, Milwaukee County, through its Department of Parks, Recreation and Culture, would be designated the lead governmental organization in coordinating plan implementation efforts as projects are proposed by the many private and public property owners concerned. A total of 274 individual private property owners reside directly adjacent to the Lake Michigan shoreline within the study area. In addition, five governmental agencies own land along the shoreline: Milwaukee County, the City of Milwaukee, and the Villages of Shorewood, Whitefish Bay, and Fox Point. The regulatory structure already put in place by Milwaukee County in carrying out its duties and responsibilities under the lakebed grants from the State of Wisconsin would provide the basic mechanism for coordinating the individual projects that may be proposed over time by the lakeshore property owners. In order for this alternative to work effectively over the entire study area, however, it would be necessary for Milwaukee County to seek and receive a lakebed grant for those portions of the Lake Michigan shoreline extending from the Linnwood Avenue water treatment plant northward to Edgewood Avenue, which is the boundary between the City of Milwaukee and the Village of Shorewood; and from Green Tree Road extended northward to the south property line of Doctors Park, within the Village of Fox Point. As noted in Chapter II, primary regulatory authority over these particular portions of the lakebed remains with the Wisconsin Department of Natural Resources.

In order for Milwaukee County to properly coordinate the regulation and installation of structural measures along the lakeshore, it would be necessary to relate the existing regulatory authority directly to the plan recommendations. As a first step toward this end, the plan recommendations would be adopted by the Milwaukee County Board of Supervisors and by the governing bodies of the four local units of government concerned: the City of Milwaukee and the Villages of Shorewood, Whitefish Bay, and Fox Point. In addition, the plan recommendations should be endorsed by the Wisconsin Department of Natural Resources and the U. S. Department of the Army, Corps of Engineers.

Once overall policy level agreement is thus reached on the basic thrust of the plan recommendations, it would be necessary for Milwaukee County to amend its existing ordinance to require that all permits henceforth issued for proposed structural measures along the affected reach of the Lake Michigan shoreline be fully consistent with the plan recommendations. Such an implementation strategy would require a strong long-term commitment to the plan on the part of the County. Prior to issuance of any permit for a proposed structure, then, Milwaukee County would have to make a finding that the project was designed in accordance with the criteria set forth in the plan and that the geographic scope of the project is fully in accordance with the plan. This might mean that in some instances, permits for proposed shore protection structures would not be approved. Such a situation would occur if an individual property owner or group of property owners proposed to install a shore protection structure of a type substantially different from the recommendations in the plan, if a proposed structure design was not consistent with the design criteria set forth in the plan, or if a particular proposal failed to encompass an entire implementation segment. The main deficiency attendant to this approach to plan implementation lies in the fact that a particular proposal for which a permit is sought may fail to encompass an entire implementation segment, thus requiring denial of the permit. There would be no way under this alternative approach to plan implementation to require appropriate groups of property owners to act collectively in implementing the plan.

In addition to the regulatory efforts of Milwaukee County, there would always remain in effect other existing institutional requirements that could affect the implementability of the plan. There would remain, for example, the need in every case to obtain a U. S. Army Corps of Engineers permit for any new shore protection structure. Presumably, if the Corps of Engineers endorsed the plan recommendations, and if the proposed structures were found to be in accordance with the plan recommendations, Corps of Engineers issuance of a permit would be routine. In addition, there would remain in effect zoning, erosion control, hauling and filling, and possibly other regulatory ordinances administered by the local governments concerned, the provisions of which would have to be met. Any local ordinance that requires a permit for hauling of fill, for example, would constitute a possible constraint on a given project. Again, however, if the local governments endorsed the plan and if a particular proposed shoreline structure or bluff stabilization project along the shoreline were found to be in conformance with the plan by Milwaukee County, then issuance of any necessary local permits should be routine.

Modified Existing Institutional Structure—Local-Based Coordination

Under the second alternative approach, each of the four local units of government involved—the City of Milwaukee and the Villages of Shorewood, Whitefish Bay, and Fox Point—would be the lead governmental organization in coordinating plan implementation efforts within their respective jurisdictions. Under this approach, Milwaukee County would relinquish its lakebed grants, and new lakebed grants would be issued to the individual local units of government. Consequently, the local units of government rather than Milwaukee County would regulate individual structural projects that may be proposed over time by the lakeshore property owners. Milwaukee County would continue to be involved, but only insofar as it owned land along the lakeshore within the affected reach.

As in the first alternative approach, it would be necessary for the governmental agencies involved at the local, county, state, and federal levels to formally adopt the recommended plan. A second step would involve securing new lakebed grants to the four local units of government involved. This would require state legislative action.

Each of the four local units of government would then take steps to adopt appropriate ordinances that would have the effect of requiring all permits for structural measures along the Lake Michigan shoreline to be issued only upon a finding of full consistency with the plan recommendations. Unlike the first alternative approach, this approach would not need to rely upon the voluntary cooperation of all of the individual property owners within an implementation segment. It would be within the power of municipalities to generally take action to protect property within their jurisdiction from destruction. The exercise of this power must serve a public purpose, even though private property may be benefited. It would, therefore, be possible for the municipalities concerned to undertake—if necessary, without full voluntary cooperation—the construction and maintenance of the recommended control measures within an entire implementation segment. To finance the improvements, the municipalities could levy taxes or special assessments against benefiting properties for specific projects.

Since some of the plan recommendations involve shoreline segments that extend across municipal boundary lines, it would be necessary under this alternative to establish some type of ongoing intergovernmental cooperative mechanism. Such a mechanism could be an intergovernmental committee established jointly by the four communities concerned. That committee would be advisory to the governing bodies of the four municipalities and would be called upon in particular to provide guidance in those cases where projects affected more than one municipality.

The four municipalities concerned could also provide the necessary intergovernmental coordination by the joint exercise of powers set forth in Section 66.30 of the Wisconsin Statutes. This Statute permits the joint exercise by municipalities of any power or duty required of, or authorized individually to, such municipalities. Under this authority, a cooperative contract commission could be created by the four local units of government. Because the powers of such a commission are limited, the local units of government would have to continue to exercise their municipal authorities to help implement the recommended plan.

Under this approach, the units of government involved would formally adopt the recommended plan. Those municipalities would form a cooperative contract commission. State legislative action would then be sought to secure new lakebed grants to either the municipalities concerned, or the newly created commission. The commission could be given the authority to issue permits for structural shore protection measures only upon a finding of full consistency with the plan recommendations. The commission could also be given the authority to compile and distribute information on shoreline conditions, review and grant permits for new shore protection work, administer shore protection projects, enter into contracts to construct and maintain the recommended measures, and monitor compliance with the recommended plan. The commission could have authority to recommend an annual budget and retain a technical staff qualified to administer the projects. The commission budget would need to be approved by each of the local units of government. An administrative budget could be financed by general fund dollars or by special assessments, at each community's discretion.

The powers of such a commission are limited in two important ways. First, a commission created under Section 66.30 does not have the power to levy taxes or special assessments. A commission is authorized only to issue revenue bonds under Section 66.066 of the Wisconsin Statutes. This financing mechanism would not be appropriate because shore protection projects are not revenue-producing enterprises. Second, the commission could, if necessary, condemn property for the purpose of constructing or maintaining shore protection measures only with the approval of the local unit of government in whose jurisdiction the condemnation is proposed. Because of these limitations, such a commission could not implement the recommended plan without the full cooperation of the municipalities concerned.

In order to overcome the problems arising from the limited powers of a commission created under Section 66.30, the agreement creating the commission should encompass commitments by the local units of government to exercise their municipal condemnation and taxing powers to implement the recommended plan. Within Implementation Segments D and J, both of which lie within the boundaries of two municipalities, the condemnation and taxing powers would be jointly exercised by the two municipalities. The individual local units of government would decide how to finance the projects within their jurisdiction, and issue tax levies or special assessments where appropriate. Although the exercise of condemnation powers could be done by the local units of government directly, it may be preferable that the condemnation powers be granted to the contract commission, subject to the approval of the local unit of government concerned. Thus, the initial determination of condemnation would be made by an entity other than the local governing body. This approach would not rely on the voluntary, cooperative action of all of the property owners within an implementation segment. Projects would be undertaken only upon an appropriate petition from the property owners within an implementation segment to the commission. The recommended plan could be effectively implemented using the foregoing approach.

Create New Institutional Structure

Under a third alternative approach, an entirely new institutional structure would be created for the specific purpose of directing implementation of the shoreline erosion management plan. Under this approach, state legislation would be sought to enable the county and local units of government concerned to create cooperatively a single, special-purpose unit of government to manage the lakeshore erosion and bluff stabilization problem for the entire study area.

In concept, a lakeshore management district would be similar to the inland lake protection and rehabilitation district authorized in Chapter 33 of the Wisconsin Statutes. A lakeshore management district would be created for the specific purpose of undertaking a program of erosion management and bluff stabilization within its boundaries. Such a district would have a governing body that could be either elected or appointed. A district could be empowered to acquire, through condemnation if necessary, real property; construct and maintain shore protection works as specified in the plan; construct and maintain bluff stabilization projects as specified in the plan; and carry out such other acts as may be necessary to fully implement the plan. In order to finance improvements, such a district would have to be empowered to levy a tax upon all real property within the district, to make special assessments to benefiting parties for specific projects, and to contract and discharge debt.

The details of the organization and operations of such a special-purpose district would have to be specified in the enabling legislation. This approach would not rely on the voluntary, cooperative action of all of the property owners within an implementation segment. Upon an appropriate petition, for example, the governing body of such a district could undertake the construction and maintenance of the recommended control measures within an implementation segment on some basis other than full voluntary participation from all property owners.

Under this alternative approach, it would be necessary for all lakebed grants to be made to the special-purpose district. Since the district, rather than individual property owners, would be responsible for all shore protection and bluff stabilization activities—private property owners presumably being precluded from so doing in the enabling legislation—the district would be responsible for obtaining any necessary federal and local regulatory permits. Presuming that the basic recommended plan would be approved and endorsed by all of the agencies concerned, and because the district would be constrained in its project undertaking to those projects identified in the plan, or in any amendment thereto that may be agreed upon by all parties, any federal or local required permits should be routinely forthcoming.

Review of Implementation Approaches

The range of authorities that could be assigned to the various implementing agencies under the alternative approaches considered are listed in Table 53. The implementation functions could be performed by Milwaukee County, the municipalities, a cooperative contract commission, or a lakeshore management district. The table indicates that under the local-based coordination alternative approach, a number of functions could be performed by the municipalities acting either individually or through a cooperative contract commission. The functions necessary to implement the plan could be assigned to provide maximum municipal authority or maximum commission authority, or the authority could be shared between the agencies.

Given the basic need to ensure implementation of the plan for entire implementation segments, and given the unlikelihood that there will always be voluntary, cooperative action in the plan implementation process in a timely way, it is apparent that the plan can be best implemented either by a newly created lakeshore management district or by a cooperative contract commission created under Section 66.30 of the Wisconsin Statutes working jointly with the municipalities. Both the creation of a Section 66.30 commission and the creation of a lakeshore management district provide several distinct advantages over plan implementation by the County or by local units of government alone. First, the purpose of a commission or district—unlike the purpose of general-purpose local units of government—would solely be the provision of shore protection. Thus, the local planning efforts and decision-making process would be focused on providing shore protection, with no detraction from nonrelated issues. Second, it would be more efficient for a commission or for a special-purpose district to acquire the expertise and experience needed to properly coordinate and direct the necessary site-specific studies, project designs, and maintenance programs. The necessary technical staff would be retained and directed by a single agency, rather than by several municipalities. Third, a commission or a district would likely provide for a more consistent approach within the study area with respect to the design and maintenance of shore protection measures. The effective implementation of the recommended shore protection measures within entire implementation segments would be difficult to achieve if permit procedures and forms, schedules for municipal approval of projects, design criteria, inspection procedures, and construction and maintenance techniques varied from municipality to municipality. It is unlikely that the municipalities, acting alone, would implement the plan on a consistent basis. Fourth, the governing body of a commission or a district would be more representative of, and responsive to, the lakeshore property owners if, in fact, the governing body consisted of lakeshore residents or individuals experienced in protecting the shoreline.

Thus, clearly, the plan could be successfully implemented either by a new lakeshore management district or by the individual municipalities working with and through a new cooperative contract commission. The cooperation and support needed to carry out the plan must, in any case, be provided by the municipalities concerned. Therefore, in order to maximize the local government's role in protecting the shoreline, it is recommended that the plan be implemented by the municipalities with the assistance of a cooperative contract commission created by the municipalities for this purpose. A cooperative contract commission would be created by the municipalities under Section 66.30 of the Wisconsin Statutes, and the municipalities would cooperatively agree to exercise their condemnation and taxing powers to help the commission implement the plan. Permits for shore protection measures could be issued by the commission, although permits for filling and

Table 53

COMPARISON OF ALTERNATIVE IMPLEMENTATION APPROACHES

			Local-Based Coordination				
				Joint Municipal-Commission Coordination			
Plan Implementation Authority	Existing Conditions	County-Based Coordination	Municipal Coordination	Maximum Municipal Authority	Municipal-Commission Shared Authority	Maximum Cooperative Contract Commission Authority	New Institutional Structure: Lakeshore Management District
Administration, Coordination, and Information[a]	County, municipalities, and informal intergovernmental advisory committee	County	Municipalities, and informal intergovernmental advisory committee	Commission	Commission	Commission	District
Lakebed Grant Designee	County, State	County	Municipalities	Municipalities	Commission	Commission	District
Review Proposed Projects	Not conducted	County	Municipalities, and informal intergovernmental advisory committee	Municipalities	Commission and municipalities	Commission	District
Issue Construction Permit for Shore Protection Structures[b]	County	County	Municipalities	Municipalities	Commission and municipalities	Commission	District
Contract to Construct and Maintain Shore Protection Structures	Individual property owners	Individual property owners	Municipalities	Municipalities	Commission and municipalities	Commission	District
Levy Taxes or Special Assessments	Not conducted	Not conducted	Municipalities	Municipalities	Municipalities	Municipalities	District
Condemn Property as Necessary	Not conducted	Not conducted	Municipalities	Municipalities	Municipalities or commission with approval of municipalities	Commission with approval and authority of municipalities	District
Monitor Plan Compliance	Not conducted	County	Municipalities, and informal intergovernmental advisory committee	Commission	Commission	Commission	District

[a]*Includes compilation and provision of shoreline erosion-related information, coordination of shore protection projects, and establishment of design criteria.*

[b]*These permits are for specific project construction. In addition to local permits, permits from the U. S. Army Corps of Engineers and permits or water quality certification from the Wisconsin Department of Natural Resources would be required under all alternative approaches. Municipalities currently regulate filling and hauling activities, and administer zoning ordinances. These regulatory functions would remain with the municipalities under all alternative approaches.*

Source: Village of Shorewood, Village of Whitefish Bay, and SEWRPC.

hauling would continue to be issued by the municipalities. Permits would no longer be issued by Milwaukee County, and arrangements would be made to ensure that federal and state permits would be routinely granted for projects in conformance with the plan once the plan is approved by the U. S. Army Corps of Engineers and the Wisconsin Department of Natural Resources.

The specific plan implementation functions to be carried out by the commission and by the municipalities would be identified by negotiations between the municipalities concerned. State legislative action would be sought to secure new lakebed grants to either the newly created commission or the municipalities for the entire study area shoreline except offshore of county parkland. Under the recommended implementation approach, Milwaukee County would retain ownership of the lakebed off county parkland, and would remain responsible for protecting the county-owned shoreline, without the need to obtain approval or permits from the municipalities or from a newly formed commission. However, it is recommended that the County cooperate with other property owners to implement projects within entire implementation segments. This recommended implementation approach would ensure local control and management of the lakeshore; require local governmental approval for decisions related to condemnation of property and the financing of projects; provide a new agency to assist the municipalities whose sole purpose is protecting the shoreline; and provide an efficient and consistent mechanism for regulating shore protection measures and for ensuring the proper design, construction, and maintenance of such measures.

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 comprehensive shoreline erosion management plan for northern Milwaukee County. Various methods of implementing the recommended plan were considered, and an implementation program was proposed as part of the recommended plan. The recommended plan reflects the concerns and preferences of the local units of government and private lakefront property owners concerned.

This study is intended to constitute the first, or systems planning, phase of what may be regarded as a three-phase shore protection development process. Preliminary engineering is the second phase in this sequential process, with final design being the third and final phase. Analytical 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 bluff toe protection, bluff slope stabilization, surface water and groundwater drainage control, and revegetation will be required to adequately prevent bluff recession. Bluff toe protection measures described included four types of revetments, three types of bulkheads, five types of onshore or near-shore beach systems, and six types of offshore structures. The capital costs of these structures were estimated to range from $150 to $2,000 per lineal foot of shoreline, with annual maintenance costs ranging from $5.00 to $50 per lineal foot. Bluff slope stabilization could be accomplished by cutting back, filling, cutting and filling, or terracing the bluff slope with retaining walls, at a capital cost ranging from $100 to $3,500 per lineal foot of shoreline, and an average annual maintenance cost of $5.00 to $15 per lineal foot, for the first three years after construction. Groundwater drainage could be provided at a capital cost of $20 to $150 per lineal foot of shoreline, with an average annual maintenance cost of $5.00 to $20 per lineal foot. Surface water drainage control could be provided at a capital cost of $10 to $150 per lineal foot, with annual maintenance costs of up to $5.00 per lineal foot. Revegetating the bluff slope could be accomplished at a capital cost of $20 to $500 per 1,000 square feet, with an average annual maintenance cost of up to $15 per 1,000 square feet for three years. 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 study area shoreline. The comprehensive shoreline management plan consists of two elements: a bluff slope stabilization element and a bluff toe protection element. A single bluff slope stabilization plan is presented which specifies the measures needed to regrade or revegetate the slope and control groundwater or surface water flow. The capital cost of the bluff slope stabilization plan is estimated at $1.9 million, the average annual maintenance cost at $228,000, and the equivalent annual cost over a 50-year period at $222,000. Three alternative bluff toe 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 a capital cost of about $8.3 million, an annual maintenance cost of about $597,000, and an equivalent annual cost over a 50-year period of $1.1 million. The second alternative plan for bluff toe protection would provide, wherever practicable, artificially nourished beach systems. The beach alternative plan would have a capital cost of about $15.9 million, an average annual maintenance cost of about $798,000, and an equivalent annual cost over a 50-year period of $1.8 million. The third alternative plan for bluff toe protection would utilize offshore islands, peninsulas, and breakwaters to protect the shoreline and provide limited sand beaches, creating over 30 acres of new land for passive recreational uses. The offshore alternative plan would have a capital cost of about $35.3 million, an average annual maintenance cost of about $1.0 million, and an equivalent annual cost over a 50-year period of $3.3 million.

A preliminary shoreline erosion management plan for northern Milwaukee County was developed 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 preliminary plan consisted of the slope stabilization plan; and carefully selected components of all three alternative bluff toe protection plans.

The preliminary shoreline erosion management plan envisioned that the bluff slopes would be stabilized by regrading and revegetating the bluff slopes, and by installing groundwater and surface water drainage systems, where needed. To protect the immediate shoreline from wave and ice action, two offshore peninsulas would be constructed, one extending northward from the Linnwood Avenue water treatment plant, and one extending southward from Atwater Park in the Village of Shorewood. Nourished sand beaches contained by offshore breakwaters would be constructed at Atwater, Klode, and Doctors Parks. These breakwaters would contain a total of about nine acres of public sand beach. Nourished gravel beaches contained by rock groins would be located just north of Atwater Park, both north and south of Klode Park, along a small portion of the Fox Point bluff, and along the entire Fox Point terrace. These nourished beaches would protect about 15,700 feet, or 41 percent, of the study area shoreline. Finally, rip-rap revetments would be constructed or reconstructed to protect nearly all existing or proposed bluff slope fill projects. These revetments would protect about 15,000 feet, or 39 percent, of the study area shoreline.

The total capital cost of the preliminary shoreline erosion management plan is about $22 million, and the annual maintenance cost about $1.1 million. The equivalent annual cost of the preliminary plan over a 50-year period is approximately $2.3 million. Of the total plan cost, about 57 percent would be financed by the private sector and 43 percent by the public sector.

After careful consideration of the preliminary plan recommendations, two major revisions to the preliminary plan were made by the study Advisory Committee. These revisions were the elimination of the two offshore peninsulas, with the shoreline instead to be protected by nourished gravel beaches contained by rock groins; and the replacement of two beaches with rock revetments, because such revetments are already under construction.

The recommended plan would include about 2,100 lineal feet of large public sand beaches contained by offshore breakwaters at Atwater Park, Klode Park, and Doctors Park; 19,000 lineal feet of nourished gravel beaches; 16,600 lineal feet of quarry stone revetments; and bluff slope stabilization measures. Only about 1,000 lineal feet of shoreline would not require any bluff toe protection. The total capital cost of the recommended plan is approximately $17.8 million, with an average annual maintenance cost of about $1.0 million. The equivalent annual cost of the recommended plan over a 50-year period is about $2.0 million. Of the total recommended plan cost, about 72 percent would be financed by the private sector and 28 percent by the public sector.

The recommended plan cannot be implemented on a piecemeal basis. The needed shore protection measures can best be implemented within 18 portions of the shoreline referred to as implementation segments. The provision of nine proposed permanent access areas would help centralize, and thereby reduce, the areawide impacts caused by the movement of trucks and heavy equipment during construction and maintenance operations.

Several alternative methods of implementing the plan were considered: having Milwaukee County coordinate the implementation activities; creating a new lakeshore management district; and placing primary responsibility with the municipalities.

The successful implementation of needed shoreline erosion and bluff recession control projects within entire implementation segments will, in any case, require the cooperation of Milwaukee County and the municipalities concerned: the City of Milwaukee and the Villages of Fox Point, Shorewood, and Whitefish Bay. Thus, it is recommended that the municipalities assume primary responsibility for carrying out the plan. To enhance the efficiency and coordination of these local functions, it is recommended that the municipalities jointly form a cooperative contract commission under the provisions of Section 66.30 of the Wisconsin Statutes. Such a commission could efficiently promote plan implementation, although it could not levy taxes or special assessments. It could not condemn property without the approval of the individual municipalities concerned. Examples of commissions created under Section 66.30 include the North Shore Water Commission and the North Shore Library Cooperative.

The specific duties to be carried out by the proposed commission would have to be agreed upon in negotiations between the municipalities concerned. Individual municipal ordinances would remain in effect with respect to zoning and regulation of filling, hauling, and other construction activities. The process for obtaining permits to construct new shore protection measures would be simplified and designed to maximize local control while carrying out the recommended plan in an integrated fashion within shoreline reaches having similar physiographic characteristics.

Chapter V

SUMMARY

INTRODUCTION

The erosion and recession of shorelines and bluffs constitutes one of the more difficult and costly problems facing private property owners and local governments along the Lake Michigan coastline. Shoreline and bluff recession rates in northern Milwaukee County range up to 1.6 feet per year, averaging about 0.5 foot per year. This recession results in an average annual loss of nearly 8,000 square feet of land surface and nearly 600,000 cubic feet of shore material.

In the past, to protect both private and public property from erosion damage, various types of shore protection measures and bluff stabilization facilities were constructed along the north shore. Some of these facilities were ineffective; some were subsequently damaged by wave action; some were perceived to be unsightly; and some may have accelerated erosion in adjacent shoreline areas. Significant concern about the existing approaches to protecting the shoreline was publicly expressed by local citizens at hearings and meetings held to discuss certain shore protection projects initiated in the early 1980's. Therefore, a need developed to critically reexamine the approaches taken to protect the shoreline, and to develop more cost-effective approaches to shore protection. Responding to the need for information and for proper guidelines and procedures to help lakefront property owners, the local communities in northern Milwaukee County asked the Southeastern Wisconsin Regional Planning Commission to conduct a shoreline erosion and bluff recession study.

PURPOSE AND SCOPE OF STUDY

The northern Milwaukee County shoreline erosion and bluff recession management study was intended to define the risk of erosion and bluff recession damage along the Lake Michigan shoreline; to explore alternative, and to recommend effective, economically feasible, and environmentally acceptable measures for erosion and bluff recession control; and to identify implementation mechanisms needed to carry out the recommended plan. To achieve these purposes, the study consisted of an inventory of erosion- and recession-related characteristics of the shoreline area, including the preparation of large-scale topographic maps of that area based upon a monumented system of survey control; the identification of erosion risk areas and shoreline recession rates; the development and evaluation of alternative shore protection and bluff recession control measures; and the preparation of a recommended shoreline erosion and bluff recession management plan.

The study was carried out under the guidance of an Advisory Committee created by the Regional Planning Commission and composed of representatives of the Villages of Shorewood, Whitefish Bay, and Fox Point; the City of Milwaukee; Milwaukee County; the Wisconsin Department of Natural Resources; the University of Wisconsin Sea Grant Institute; the University of Wisconsin-Milwaukee; and concerned and knowledgeable citizens. The shoreline management plan set forth in this report is the culmination of two separate, but coordinated, studies which were conducted simultaneously. A study of bluff conditions and of onshore structural and nonstructural protection measures was conducted by the staff of the Regional Planning Commission, with the assistance of consultants. These consultants included three professors from the University of Wisconsin-Madison who assisted in the bluff slope stability analyses; a professor from the University of Wisconsin-Milwaukee who conducted the electrical resistivity analyses to help evaluate groundwater conditions; PTL-Inspectorate, Inc., which conducted the soil borings; Robert T. McCoy, who took oblique aerial photographs of the shoreline; and Aerometric Engineering, Inc., which prepared the topographic mapping. A study of coastal processes and offshore structural protection measures was conducted by civil engineers from Warzyn Engineering, Inc.; planners and landscape architects from Johnson, Johnson & Roy, Inc.; and coastal engineers from W. F. Baird & Associates, Ltd.

INVENTORY FINDINGS

The northern Milwaukee County shoreline erosion and bluff recession management study area was defined as the area lying along Lake Michigan from the City of Milwaukee Linnwood Avenue water treatment plant northerly through the Villages of Shorewood, Whitefish Bay, and Fox Point to Doctors Park. The study area is comprised of those lands that are most directly affected by the Lake Michigan erosion processes and encompasses approximately 1,726 acres of land, and approximately 7.3 miles of shoreline.

Those elements of the natural resource base within the study area pertinent to an understanding of coastal erosion and bluff recession processes were inventoried, including the bedrock geology and glacial deposits; soils; height, slope, vegetative cover, stratigraphy, and stability of bluffs; beaches; groundwater conditions; and climate. The study area is underlain by Silurian, Ordovician, Cambrian, and Precambrian bedrock. Up to 150 feet of unconsolidated glacial deposits cover the bedrock, and include layers of the Kewaunee Formation, the Oak Creek Formation, and the New Berlin Formation. The soils covering the upland portions of the study area generally have low infiltration capacity, low permeability, and poor drainage. The sandy soils which cover the terrace within the Village of Fox Point have moderate infiltration capacity, moderate permeability, and good drainage.

The bluffs along the northern Milwaukee County shoreline range up to nearly 130 feet in height above beach levels, with about one-half of the length of shoreline within the study area having bluffs ranging from 80 to 120 feet in height. The terraced area within the Village of Fox Point, which lies four to 10 feet above the beach level, covers approximately 24 percent of the study area. The bluffs are generally comprised of glacial till, silt, clay, sand, and gravel. At the time of the field surveys conducted under the study in the summer of 1986, when record high lake levels were recorded, most of the shoreline had a beach width of less than 10 feet, although in places the beach width exceeded 90 feet.

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

Climate impacts on coastal erosion and bluff recession include freeze-thaw actions within bluff material; high surface runoff from frozen soils; lake ice effects; and high surface runoff and soil erosion during intense storms. Frozen ground and snow cover may be expected for approximately four months each winter season. About 17 percent of the average annual precipitation of 31.81 inches occurs as snowfall and sleet. Lake ice formation begins in late November or early December and ice breakup normally occurs in late March or early April.

The type, degree, and extent of shore erosion and bluff recession damage is determined by the interrelationship of the natural and man-made features of the study area. In 1985 about 1,448 acres of the study area, or 84 percent of the total study area, was devoted to intensive urban uses. About 74 percent of the urban land area was in residential use.

Shoreland development and activities are regulated by federal, state, and local units and agencies of government. The U. S. Army Corps of Engineers is the primary federal agency responsible for regulating certain structures, dredging, and wetland protection. Although the Wisconsin Department of Natural Resources regulates shore protection-related activities throughout most of the Lake Michigan shoreline of the State, 68 percent of the length of shoreline within the study area is regulated under a lakebed grant made to Milwaukee County by the State Legislature in 1933. Local zoning ordinances are presently in effect in each of the four municipalities within the study area, but are generally devoid of provisions pertaining to Lake Michigan shoreline erosion and bluff recession hazards.

Numerous types of shore protection structures exist along the northern Milwaukee County shoreline. The effectiveness of these structures—which include groins, bulkheads, revetments, and breakwaters—has varied. A field inspection of all 80 shore protection structures in the study area conducted in 1986 indicated that 76 percent exhibited some type of damage and required repair; very little maintenance is performed on most structures.

The most serious Lake Michigan coastal problem in northern Milwaukee County is recession of the bluffs. A survey was conducted under the study in May 1986 to evaluate the physical and erosion-related characteristics of the bluffs. The results of the inventory indicated that the primary cause of bluff recession in the study area at that time was bluff toe erosion by wave action. Groundwater seepage was also a major cause of slope failure in some portions of the study area. Shallow sliding was the most common type of slope failure experienced, although many areas were experiencing deep-seated slumps. From 1963 through 1985, the bluff recession rate along the study area shoreline averaged 0.5 foot per year. The highest recession rate measured from 1963 through 1985 was 1.6 feet per year, which occurred in the terraced portion of the Village of Fox Point.

EVALUATION OF COASTAL EROSION PROBLEMS AND DAMAGES

The identification of the shoreland areas that are expected to be affected by shoreline erosion and bluff recession enables public officials and concerned and affected private property owners to better assess potential erosion losses and to evaluate alternative shoreline erosion control measures. Analytical procedures and geotechnical engineering techniques used to evaluate the existing and potential coastal erosion problems under the study included a determination of the stability of the bluff slope with respect to both rotational and translational sliding, and an assessment of the severity of bluff toe erosion.

With respect to rotational sliding, 38 percent of the total length of the study area shoreline was determined to have stable bluff slopes; 21 percent marginal bluff slopes; and 18 percent unstable bluff slopes, as shown on Map 18 in Chapter III. Bluff slope stability is not a problem in the Fox Point terrace area.

With respect to translational sliding, 40 percent of the length of shoreline in the study area was determined to have stable bluff slopes; 23 percent marginal bluff slopes; and 14 percent unstable bluff slopes.

With respect to toe erosion, only 20 percent of the length of shoreline within the study area was observed to exhibit little or no evidence of toe erosion in 1986. About 54 percent of the length of shoreline was found to be exhibiting erosion at the toe of the bluff, but the erosion did not appear to affect the overall stability of the bluff slope. The remaining 26 percent of the shoreline length was observed to exhibit toe erosion which was threatening the overall stability of the bluff slope.

The shore protection needs for each of the 36 bluff analysis sections within the study area were identified. It was indicated that the bluff slopes within about 27 percent of the length of shoreline within the study area should be regraded to a stable slope angle; that groundwater drainage systems should be installed to lower the elevation of the groundwater along about 16 percent of the length of shoreline; that surface water runoff control measures should be implemented along about 4 percent of the length of shoreline; that additional toe protection should be provided to about 97 percent of the length of shoreline; and that the bluff slope along about 18 percent of the length of shoreline should be revegetated. It is important to note that no entire bluff analysis sections were found to be fully protected, requiring no maintenance or corrective actions, in 1986.

The land area lying within 10 feet, and within 25 feet, of the edge of a marginal or unstable bluff or terrace was delineated for the entire study area shoreline. The area lying within 10 feet of the edge of marginal or unstable bluffs and terraces was found to total about five acres in area and in 1986 to encompass 23 residential buildings, having a 1986 economic value of about $3.8 million. The area lying within 25 feet of the edge of marginal or unstable bluffs and terraces was found to total about 13 acres of land, and to contain 40 residential buildings having a 1986 economic value of about $6.9 million. In all, 274 residential properties having an average 1986 economic value of $257,000, and a combined total economic value of private property exceeding $70 million, were located directly adjacent to the shoreline within the study area.

ALTERNATIVE SHORELINE EROSION MANAGEMENT MEASURES

Specific structural shore protection measures required at any particular site can be properly determined only on the basis of a detailed engineering analysis of the physical characteristics of the site; the causes of erosion on the site; the degree of erosion expected; and property values. Bluff toe protection measures evaluated for northern Milwaukee County included four types of revetments; three types of bulkheads; five types of onshore or near-shore beach systems; and six types of offshore structures. The installation of the bluff toe protection structures would entail a capital investment of $150 to $2,000 per lineal foot of shoreline, with average annual maintenance costs ranging from $5.00 to $50 per lineal foot. Bluff slope stabilization could be accomplished by cutting back, filling, cutting and filling, or terracing the bluff slope with retaining walls at a capital cost ranging from $100 to $3,500 per lineal foot of shoreline, and an average annual maintenance cost of less than $5.00 per lineal foot. Improved groundwater drainage could be provided at a capital cost of $20 to $150 per lineal foot of shoreline, with an average annual maintenance cost ranging from $5.00 to $15 per lineal foot. Improved surface water drainage control could be provided at a capital cost of approximately $10 to $150 per lineal foot of shoreline, with average annual maintenance costs of up to $5.00 per lineal foot. Revegetating the bluff slope could be accomplished at a capital cost of $20 to $500 per 1,000 square feet, with an average annual maintenance cost of from $5.00 to $15 per 1,000 square feet for the first three years, after which the vegetation should be established.

Alternative shoreline protection and bluff recession control plans presented for the study area shoreline consisted of two elements: a bluff slope stabilization plan element and a bluff toe protection plan element. A single bluff slope stabilization plan was presented along with three alternative bluff toe protection plans. The bluff slope stabilization plan, which specifies the measures needed to regrade or revegetate the slope and control groundwater or surface water flow, and which should be implemented regardless of the toe protection measures selected, would entail a capital cost of approximately $1.9 million, an average annual maintenance cost of about $228,300, and an equivalent annual cost over a 50-year period of $222,000. The bluff slope stabilization plan is shown on Map 21 in Chapter IV.

The revetment alternative bluff toe protection plan shown on Map 22 in Chapter IV, which proposes the use of quarry stone revetments wherever practicable to protect the shoreline, would entail a capital cost of about $8.3 million, an average annual maintenance cost of about $598,000, and an equivalent annual cost over a 50-year period of about $1.1 million. The nourished beach alternative bluff toe protection plan shown on Map 23 in Chapter IV, which would provide wherever practicable artificially nourished beach systems, would entail a capital cost of about $15.9 million, an average annual maintenance cost of about $0.8 million, and an equivalent annual cost over a 50-year period of $1.8 million. The offshore alternative toe protection plan shown on Map 24 in Chapter IV, which would utilize offshore islands and breakwaters to protect the shoreline, would entail a capital cost of approximately $35.3 million, an average annual maintenance cost of about $1.0 million, and an equivalent annual cost over a 50-year period of $3.3 million.

PRELIMINARY SHORELINE EROSION MANAGEMENT PLAN

A preliminary plan was prepared by the Commission staff which attempted to both fully stabilize the bluff slopes and protect the immediate shoreline from wave and ice erosion on a long-term basis. This plan, which was comprised of the bluff slope stabilization plan and a combination of the best components of each of the three alternative bluff toe protection plans considered, sought those shore protection measures which, when applied on a reach-by-reach basis, would effectively abate the erosion problems; would be economically feasible and implementable; and would provide—where practicable—a usable shoreline.

The preliminary shoreline erosion management plan, which is shown in graphic summary form on Map 25 in Chapter IV, envisioned large public sand beaches contained by offshore breakwaters at Atwater Park, Klode Park, and Doctors Park; two offshore peninsulas, one extending northward from Milwaukee County's Lake Park and one extending southward from the Village of Shorewood's Atwater Park; 15,720 feet of nourished gravel beaches contained by rock groins; 15,040 feet of quarry stone revetments; and bluff slope stabilization measures. The total capital cost of the preliminary shoreline erosion management plan was approximately $22.1 million, and the average annual maintenance cost about $1.1 million. The equivalent annual cost of the plan over a 50-year period approximated $2.3 million.

RECOMMENDED SHORELINE EROSION MANAGEMENT PLAN

Upon careful consideration of the preliminary plan prepared by the Commission staff, the Advisory Committee made several modifications to that plan. The recommended plan, shown in graphic summary form on Map 26 in Chapter IV, includes two major revisions of the preliminary plan. First, the two offshore peninsulas originally proposed were eliminated from the plan, with the shoreline concerned instead to be protected by nourished gravel beaches contained by rock groins. Second, the preliminary plan was revised to reflect two new revetment and fill projects which were recently initiated in the absence of an adopted shoreline erosion management plan. Thus, the construction of a revetment and bluff slope filling were recommended for these two shoreline areas; the preliminary plan proposed that these areas be protected by a nourished gravel beach with less extensive bluff slope stabilization work.

The recommended plan would include about 2,100 lineal feet of large public sand beaches contained by offshore breakwaters at Atwater Park, Klode Park, and Doctors Park; about 19,000 feet of nourished gravel beaches contained by rock groins; about 16,600 feet of riprap revetments; and bluff slope stabilization measures. About 1,000 lineal feet of shoreline would not require any bluff toe protection. The total capital cost of the recommended plan approximates $17.8 million, with an average annual maintenance cost of about $1.0 million. The 50-year equivalent annual cost of the recommended plan approximates $2.0 million.

The scope of the recommended 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 protection must at times be compromised—particularly when shore protection is not undertaken until a severe erosion problem has developed and real property is threatened. The plan, however, attempts to minimize adverse environmental impacts, as well as potential adverse impacts on adjacent shoreline areas, by recommending carefully selected sets of needed protection measures most appropriate for the different coastal environments within the study area. The plan seeks to ensure that the recommended measures will have minimal long-term harmful effects on the overall coastal environment—including the offshore bathymetry, sediments, and ecosystem.

PLAN IMPLEMENTATION

The recommended plan cannot be implemented on a piecemeal basis—the shore protection measures can be properly constructed and maintained only within specified reaches of shoreline referred to as implementation segments. Eighteen implementation segments were delineated and are shown on Map 27 in Chapter IV. Nine permanent access sites to the shoreline to be used for construction and continued maintenance of the recommended measures were identified.

The possibility of using the existing institutional structure and having Milwaukee County coordinate the implementation activities was considered. Such implementation would rely upon the voluntary cooperation of all of the property owners within an implementation segment. Individual property owners would be able to effectively thwart proper plan implementation. Thus, this alternative method of implementation was not recommended.

Another alternative approach considered was the creation of a new lakeshore management district whose specific purpose would be to stabilize the bluff slopes and protect the shoreline. Upon adoption of the plan by Milwaukee County and the local units of government concerned, and endorsement by the Wisconsin Department of Natural Resources and the U. S. Army Corps of Engineers, State legislation would be sought to enable the County and the local units of government concerned to cooperatively create a lakeshore management district. Upon an appropriate petition from the property owners within a designated implementation segment, the district would construct and maintain the recommended bluff stabilization and shore protection measures. In order to finance the improvements, the district governing body would have to be empowered to levy a tax upon all real property within the district, to make special assessments to benefiting parties for specific projects, and to contract and discharge debt. This approach would not have to rely upon the voluntary cooperation of all property owners within an implementation segment. All lakebed grants would be made to the lakeshore management district. This approach was not recommended because of anticipated opposition to the creation of a new taxing body, and because elected officials of the general-purpose units of government desired a more active role in the plan implementation process.

A third approach to implementing the recommended plan would place primary responsibility with the municipalities. The municipalities could continue to issue permits for new shore protection measures; enter into contracts; levy taxes or special assessments; condemn property if necessary; and monitor compliance with the plan. To enhance the efficiency of these functions, the municipalities could, under Section 66.30 of the Wisconsin Statutes, form a cooperative contract commission. This commission could exercise all of these functions with the following exceptions: it could not levy taxes or special assessments and it could not condemn property without the approval of the municipality.

This third approach to plan implementation, the formation of a cooperative contract commission by the municipalities under Section 66.30, is recommended. Because the powers of such a commission are limited, the municipalities would have to exercise their municipal authorities to help implement the plan—especially with respect to the financing of the projects and the condemnation of property. It is recommended that the specific plan implementation functions to be carried out by the commission and by the municipalities be negotiated among the municipalities concerned.

PUBLIC REACTION TO THE RECOMMENDED PLAN AND SUBSEQUENT ACTION OF THE ADVISORY COMMITTEE

Formal public hearings were held on the recommended shoreline erosion and bluff recession control plan within each of the three villages involved. A public hearing was not held within the City of Milwaukee, the decision in this respect having been made by the City. The public hearings, which were held between March 29, 1988, and May 16, 1988, were conducted by the Villages of Fox Point, Shorewood, and Whitefish Bay with the village presidents presiding. The purpose of the hearings was to present the preliminary findings and recommendations of the shoreline erosion management study for review and consideration by lakefront property owners, public officials, and interested citizens. The hearings were announced through news releases sent to the media serving northern Milwaukee County, and through the distribution of a public information summary prepared by the Commission staff and reviewed by a subcommittee appointed by the Advisory Committee Chairman. A summary report distributed prior to the hearings is presented in Appendix D to this report. The Village of Shorewood hearing was held at 7:30 p.m. on March 29, 1988, in the Shorewood Library. The Village of Fox Point hearing was held at 7:00 p.m. on April 26, 1988, in the Fox Point Municipal Building. The Village of Whitefish Bay hearing was held at 7:30 p.m. on May 16, 1988, in the Village Hall. The public hearings were well attended, with 40 citizens present at the Shorewood hearing, 85 at the Fox Point hearing, and 65 at the Whitefish Bay hearing. Minutes of the public hearings were taken and published by each of the municipalities involved and provided to both the Advisory Committee and the Regional Planning Commission for review and consideration prior to final adoption of a recommended plan. The minutes of the public hearings are presented in Appendix E.

In general, the Village of Shorewood residents who made comments at the March 29, 1988, hearing supported the overall plan. The recommendations of the physical elements of the plan were well received, with one exception: the recommendation to provide additional erosion control structures in areas that presently receive substantial protection, such as the northern portion of Bluff Analysis Section 8 and the southern portion of Bluff Analysis Section 5 where significant beaches have formed. Some residents commented that those property owners whose shoreline is adequately protected should not be required to install additional shore protection measures. There were also several concerns expressed by Village of Shorewood residents with respect to the recommended implementation program for the plan. It was suggested that more detailed guidelines be developed in the plan with regard to the implementation program so that property owners would have a better understanding of what to expect. Specific questions raised on the implementation program were as follows: 1) Which governmental agencies would have authority to implement the plan? 2) When and under what circumstances could property owners be forced to comply with the plan? 3) How would financing and assessments be issued?

The residents of the Village of Fox Point who made comments at the April 26, 1988, hearing expressed substantial opposition to key elements of the plan. There was testimony that the residents found the inventories and analyses that were conducted under the study useful, and there was some support expressed for long-range planning for shore protection. However, major concern was expressed over the physical elements of the plan. This concern focused on the recommendation that a nourished gravel beach be developed along the Fox Point terrace, referred to in the plan as Bluff Analysis Section 35. All owners of property along N. Beach Drive who spoke at the hearing were opposed to the development of the gravel beach on the basis that a beach system adjacent to the road would attract too many users, thereby increasing the traffic, parking, and trespassing problems that already exist in that area. The beaches, some residents said, may increase property owner liabilities for injuries suffered by persons using the beach, and may result in increased police protection requirements. Furthermore, some property owners had recently installed other types of structures for erosion control—primarily bulkheads and revetments—and therefore did not wish to install additional measures. There was also opposition expressed to the recommendation that the municipalities, under certain circumstances, could require unwilling property owners to comply with the plan and participate in a particular project. Some property owners believed it was the right and obligation of individual property owners to protect their own property in the manner they preferred.

On January 13, 1988, prior to the Village of Fox Point public hearing, the Regional Planning Commission received a letter from the Fox Point Village Manager on behalf of the Village Board requesting that the revetment alternative for the Village be included in the final plan rather than the nourished gravel beach alternative. The revetment alternative calls for the reconstruction of the existing revetment for the village portion of the shoreline along N. Beach Drive, and continued maintenance of the existing structures protecting the private property along N. Beach Drive. The revetment alternative also calls for continued maintenance of existing structures in Bluff Analysis Sections 31 and 32, which lie within the bluff portion of the Village of Fox Point. The letter from the Village also noted that on September 22, 1987, the Village Board adopted a resolution opposing the creation of a new governmental entity with tax levy powers that would have jurisdiction over the shore areas. Prior to the public hearings, the Advisory Committee revised an initial Commission staff recommendation for the formation of a new lakeshore management district, and recommended instead that the municipalities jointly create a cooperative contract commission to help coordinate plan implementation activities and improve local control over shoreline protection. A cooperative contract commission would not represent a new taxing authority.

In general, the Village of Whitefish Bay residents who made comments at the May 16, 1988, hearing supported the plan. It was noted at the hearing that the plan is designed to address many of the problems that have in the past been caused by shoreline protection projects in the Village. It was also observed that the types of bluff stabilization and shoreline protection measures recommended in the plan are very similar to those that have previously been used, and to those that are currently being installed, in the Village. It was generally agreed by those who commented that the proposed implementation program is needed to successfully carry out the recommended shore protection projects.

In summary, three major issues were raised at the public hearings on the recommended plan: 1) the installation of additional shoreline protection measures within areas that do not need protection, or that are already protected; 2) the recommended development of nourished gravel beaches within portions of the Village of Fox Point; and 3) the recommended plan implementation program.

The Advisory Committee carefully considered these three issues in light of the testimony given at the public hearings. Certain major changes were made by the Committee in the initially proposed shoreline erosion control plan in direct response to the public reaction to the plan.

Installation of Additional Shoreline Protection Measures Within Those Areas That Do Not Need Protection, or Are Already Protected

Concerns were expressed at both the Shorewood and Fox Point hearings with regard to the recommended installation of shoreline protection measures in some areas where such protection does not appear necessary. Within the Village of Shorewood the comments were made in reference to shoreline areas which presently did not contain any type of shore protection structure and on which a significant beach had formed as an indirect benefit of a downdrift structure. Within the Village of Fox Point the comments were made in reference to the Fox Point terrace shoreline along N. Beach Drive, where many property owners had installed erosion control structures other than the type recommended in the plan.

In response to these concerns, it was concluded by the Advisory Committee that while under present conditions it appears that only minimal toe erosion is occurring along portions of the shoreline, the plan recommendations are intended to protect the shoreline from wave and ice erosion on a long-term basis, and therefore the components of the plan are designed to provide adequate protection under high Lake Michigan water levels. Moreover, the recommended implementation program provides that no work would be undertaken until a majority of the property owners within a proposed project area agreed that protection was needed and desirable. Finally, it was recognized that, in some cases, it may be necessary to install protection measures on property which does not require such a high level of protection in order to successfully construct and maintain shore protection measures within entire implementation segments or reaches. While the required installation of shore protection measures in such cases may appear unfair to the few property owners concerned, the requirement is justified and necessary to efficiently and effectively protect the entire shoreline of the study area. The plan implementation program would allow the municipalities flexibility in assigning appropriate costs to property owners on the basis of the benefits received.

Opposition to Nourished Gravel Beaches Within the Village of Fox Point

Concern was expressed by both the Village of Fox Point and the lakefront property owners along N. Beach Drive over the recommended development of nourished gravel beaches along portions of the Village of Fox Point shoreline. These concerns related in part to the problems of increased traffic and parking on public lands, and trespassing and vandalism on private lands which could result from the increased use of the lakefront by the public. It was feared that the construction of a beach system along the terrace would attract too many inland users. The beaches were also objected to because some property owners along the terrace had already installed shore protection measures different from those recommended in the plan.

The Committee noted that the recommendation for the beach system was made following a careful consideration of alternatives. The beach system, which could only be considered in relatively low-wave-energy environments—such as the Fox Point terrace—was recommended primarily for the following reasons:

1. Compared to traditional revetments and bulkheads, less wave energy is reflected by beaches, thereby reducing associated damages to adjacent shoreline reaches, to the littoral drift, and to the offshore coastal environment.

2. Properly designed beaches are flexible, energy absorptive, and durable, adjusting and remolding in response to storm and water level conditions.

3. In the long term, wave heights approaching beaches are more apt to remain stable, or possibly to even decrease over time, whereas scouring in front of revetments and bulkheads may be expected to increase the heights of the approaching waves.

4. The beach system would create a usable shoreline.

5. While beach renourishment may be required following a highly erosive storm, massive structural failure is unlikely.

An alternative to the beach system that was considered for the terrace was the reconstruction of the revetment along the village-owned portions of N. Beach Drive, and the continued maintenance of the existing structures—primarily revetments and bulkheads—along the privately owned portions of N. Beach Drive.

In response to the concerns raised by the Village Board and residents of Fox Point, the Advisory Committee recommended that the plan be changed for Bluff Analysis Section 35, which includes the Fox Point terrace, to recommend the revetment alternative. Thus, under the final recommended plan, the continued maintenance of existing structures along the privately owned shoreline of the Fox Point terrace and the reconstruction of the revetments along the village-owned shoreline adjacent to two portions of N. Beach Drive are recommended. The final recommendations are expected to be acceptable to the Fox Point Village Board and to village lakefront residents—and thus more implementable.

The final recommendations for the Fox Point terrace were made reluctantly. The Advisory Committee was concerned about potential long-term adverse effects on adjacent shoreline areas and on the offshore coastal environment which may be caused by wave energy reflected from the existing bulkheads and revetments. These adverse effects are most likely to occur in coastal areas with deep sand deposits and gentle offshore slopes, such as the Fox Point terrace area. The coastal area offshore of the Fox Point terrace is thus more susceptible to these effects than is any other location in northern Milwaukee County, and the low terrace is more susceptible to damage from increased wave attack than are other locations.

To help avoid the occurrence of serious, irreversible adverse effects on the Lake Michigan coast of northern Milwaukee County, it is recommended that a long-term, continuing coastal monitoring program be implemented by the Village of Fox Point along the 9,070-foot-long Fox Point terrace. This monitoring program would be intended to detect the early stages of any significant adverse effects caused by the existing—or any new—shore protection structures along the terrace. If significant effects are detected, corrective action—such as revising the design of some structures, structure modification, and localized beach nourishment—could be undertaken. If serious impacts continue to occur, further revisions to the final recommended plan should be considered. It is recommended that the coastal monitoring program include periodic bathymetric surveys, characterization of the composition of the coastal sediments, and observation of damage to structures. The bathymetric profiles should be prepared at approximately 1,000-foot intervals to a water depth of at least 12 feet below low water datum. Grab samples of the sediment may be used to characterize sediment composition. The structure evaluations would require onsite field inspections. It is recommended that the monitoring program initially be conducted at two- to five-year intervals.

The Advisory Committee also considered the Fox Point Village Manager's request that continued maintenance of existing structures—rather than nourished gravel beaches—be recommended for the entire bluff, as well as terrace, portion of the Village. Nourished gravel beaches are recommended in the plan for Bluff Analysis Sections 31 and 32 along the bluff portion of the Village. The Committee concluded that since the bluff shoreline is not readily accessible to the general public, the beaches would not likely result in increased traffic, parking, trespassing, or vandalism problems. Furthermore, it is likely that some additional method of bluff toe protection will be needed to protect Sections 31 and 32, and Section 31 has historically been protected by a beach contained by groins. No opposition to the recommendation for nourished gravel beaches was expressed at the hearing by residents of Sections 31 and 32. Thus, the Committee reaffirmed its recommendation for a nourished gravel beach in Bluff Analysis Sections 31 and 32.

Implementation Program

The recommended implementation program for the shoreline erosion management plan was met with mixed reaction at the public hearings. Village of Whitefish Bay residents generally supported the implementation program; Village of Shorewood residents also generally supported the program but felt more specific guidelines were needed; and Village of Fox Point residents expressed opposition to the program.

In response to the concerns raised at the public hearings, the Advisory Committee carefully reconsidered the implementation program and concluded that the overall concept of the program was sound. The Committee concluded that some of the opposition to the plan implementation program expressed in the Village of Fox

Table 54

PLAN IMPLEMENTATION AUTHORITIES UNDER THE FINAL RECOMMENDED IMPLEMENTATION PROGRAM

Plan Implementation Authority	Implementation Agency Options
Administration, Coordination, and Information	Cooperative Contract Commission
Lakebed Grant Designee or Delegate	Municipalities
Review Proposed Projects	Cooperative Contract Commission and Municipalities
Issue Construction Permit for Shore Protection Structures	Municipalities or Cooperative Contract Commission
Contract to Construct and Maintain Shore Protection Structures	Cooperative Contract Commission, or Municipalities, or Private Property Owners
Levy Taxes or Special Assessments	Municipalities
Condemn Property if Necessary	Municipalities
Monitor Plan Compliance and Maintenance	Cooperative Contract Commission and Municipalities

Source: SEWRPC.

Point related to a misunderstanding that the proposal involved the establishment of a new taxing authority which would have the power to levy taxes for a project even if property owners were not in favor of the project. The plan implementation program had been revised earlier by the Advisory Committee in a manner that fully addressed this concern. The Committee determined, however, that more specific guidelines should be provided relating to the duties and functions of the cooperative contract commission recommended to be jointly created by the municipalities concerned to help coordinate plan implementation activities. Table 53 in Chapter IV presents the various duties and functions that could be assigned to the various implementation agencies. It was initially recommended that the specific plan implementation functions to be carried out by these agencies be identified by negotiations between the municipalities concerned following adoption of the plan.

Following consideration of the comments expressed at the public hearings, the Advisory Committee recommended that a preferred implementation approach be identified. The duties and functions recommended to be assigned to the municipalities and to the newly formed contract commission are set forth in Table 54. These duties and functions may be modified through negotiations between the municipalities as plan implementation proceeds.

Property owners under certain circumstances may be required to comply with the plan by the municipality. It is recommended that a municipality consider requiring a property owner to comply with the plan only where a petition for a project in conformance with the final recommended plan is submitted to the municipality by a majority of the property owners concerned; where plan compliance is necessary for the successful construction or maintenance of the project; and where failure to comply with the plan could result in an increased risk of damage or a significantly increased cost to other properties within the segment. The fact that a particular property may not require the same degree of added protection as do other properties in the segment should not, in itself, be considered a valid reason for not complying with the plan.

However, the improvements to be built should be carefully designed to reflect the needs of each property owner, while forming an integral part of the shoreline protection measures for the segment of shoreline concerned. Thus, the costs to individual property owners could vary.

It is recommended that the decision to finance projects by levying taxes or special assessments be made by the municipalities on a case-by-case basis. The method of financing should also be determined by the municipalities. Regardless of the method of financing selected, the cost of shore protection projects should be distributed among the property owners concerned based upon the benefits received.

Concluding Remarks

Based upon the results of the public hearings held on the recommended plan, the Advisory Committee made the following changes to the northern Milwaukee County shoreline erosion management plan as that plan was presented at the public hearings:

1. The nourished gravel beach system recommended for Bluff Analysis Section 35, which consists of the Fox Point terrace property along N. Beach Drive, was changed to a revetment alternative. Under the final plan, private property owners within the Fox Point terrace would continue to maintain the existing shore protection structures. The Village of Fox Point would reconstruct the existing revetments on village property that adjoins two portions of N. Beach Drive.

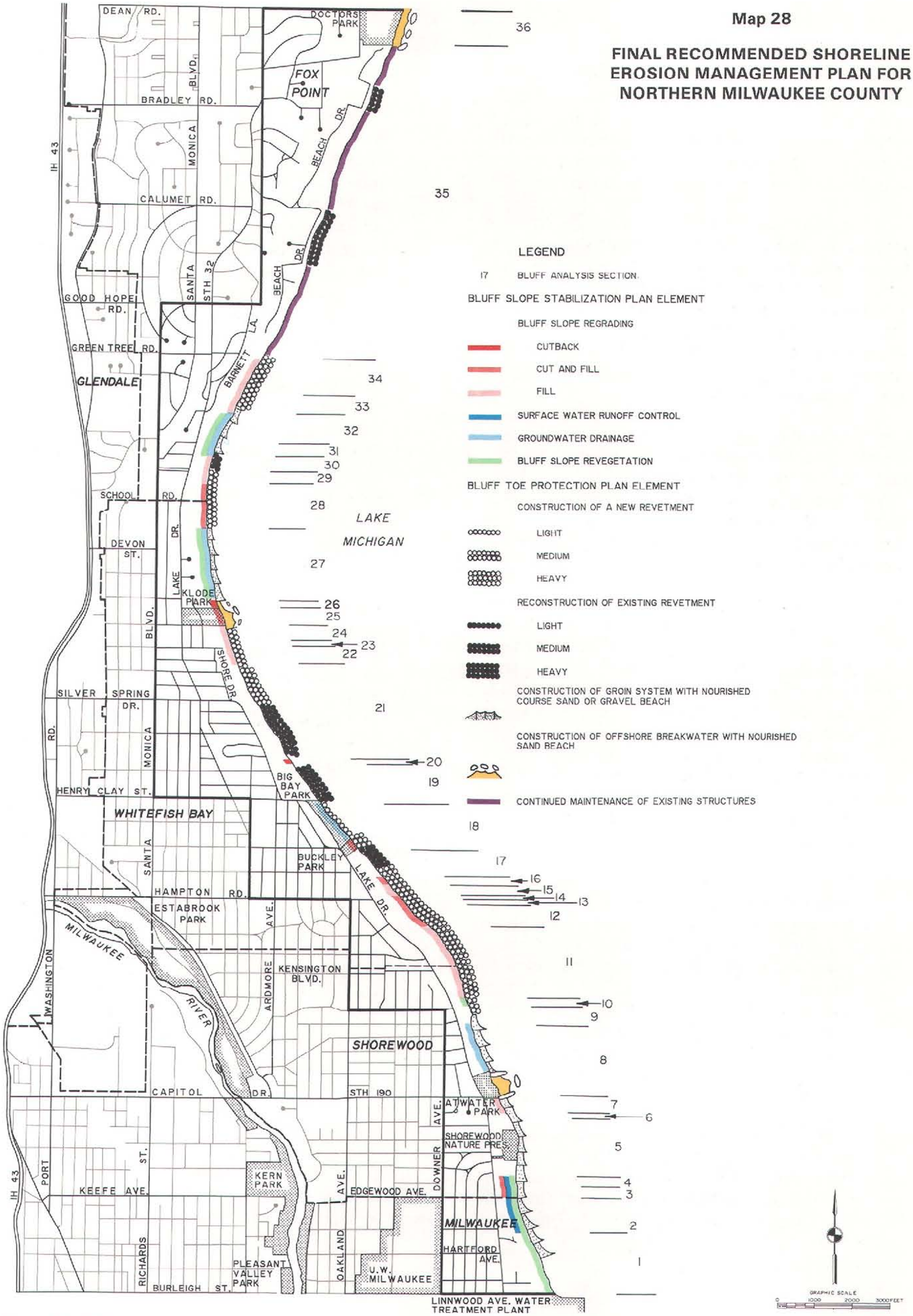
2. A long-term continuing coastal monitoring program was recommended to be implemented by the Village of Fox Point along the Fox Point terrace. This monitoring program, intended to detect any adverse effects caused by existing or new structures, would include a bathymetric survey, characterization of the composition of the coastal sediments, and observations of damage to structures. These surveys would be conducted at two- to five-year intervals.

3. Plan implementation recommendations were expanded to suggest the duties and functions of each implementation agency concerned. The recommended implementation program calls for the creation of a municipal-cooperative contract commission and for shared implementation responsibilities between that commission and the local municipalities. The contract commission, which would be created jointly by the municipalities, would have authority to provide information and administer and coordinate shore protection projects. The contract commission would also share responsibility with the municipalities for reviewing, issuing permits, and providing contracts for construction and maintenance of shore protection structures, and for monitoring plan compliance and the maintenance of shore protection measures. The commission would have no tax levy powers. Only the municipalities would have authority to levy taxes or special assessments to help finance projects. Under the implementation program, projects could be initiated only after a petition by a majority of the property owners within an implementation segment had been submitted to the respective municipality. The municipalities could, under certain circumstances if an appropriate petition was submitted by a majority of the property owners concerned, require property owners to comply with the plan and maintain shore protection measures. Project costs should be distributed based on the benefits received.

The final recommended plan is shown in graphic summary form on Map 28. Individual components and associated costs of the recommended plan are set forth in Table 55. The total capital cost of the final recommended shoreline erosion management plan is approximately \$14.8 million, or \$3.0 million less than the plan as taken to the public hearings. The annual maintenance cost is about \$0.9 million, or about \$80,000 less than the plan as taken to the hearings. The equivalent annual cost of the final recommended plan over a 50-year period is approximately \$1.7 million. The public and private sector costs of the final recommended plan within each civil division are set forth in Table 56. Of the total equivalent annual cost of the plan, about \$1.3 million, or 77 percent, would be financed by the private sector, and about \$0.4 million, or 23 percent, would be financed by the public sector. The

Map 28

FINAL RECOMMENDED SHORELINE EROSION MANAGEMENT PLAN FOR NORTHERN MILWAUKEE COUNTY



Source: SEWRPC.

Table 55

ESTIMATED COST OF THE FINAL RECOMMENDED SHORELINE EROSION MANAGEMENT PLAN FOR NORTHERN MILWAUKEE COUNTY

BLUFF SLOPE STABILIZATION							
Civil Division	Bluff Analysis Section	Shoreline Length (feet)	Plan Component	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual Cost
City of Milwaukee	1	1,970	Revegetation	$ 20,000	$ 3,900[a]	$ 30,000	$ 2,000
	2	950	Revegetation, surface water control	14,000	2,800[a]	22,000	1,000
Village of Shorewood	3	300	Bluff slope regrading—cut and fill surface water control, revegetation	30,000	3,000[a]	38,000	2,000
	4	290	Bluff slope regrading—cut and fill surface water control, revegetation	23,000	2,900[a]	31,000	2,000
	5	1,710	- -	- -	- -	- -	- -
	6	170	- -	- -	- -	- -	- -
	7	380	Bluff slope regrading—fill	57,000	5,700[a]	72,000	5,000
	8	790	- -	- -	- -	- -	- -
	8	1,380	Groundwater drainage	69,000	13,800	287,000	18,000
	9	520	- -	- -	- -	- -	- -
	10	240	Revegetation	4,000	700[a]	6,000	1,000
	11	2,370	Bluff slope regrading—fill	356,000	35,600[a]	419,000	27,000
Village of Whitefish Bay	12	850	Bluff slope regrading—cut and fill	128,000	12,800[a]	162,000	10,000
	13	190	Bluff slope regrading—cut and fill	28,000	2,800[a]	283,000	18,000
	14	160	Bluff slope regrading—cut and fill	24,000	2,400[a]	30,000	2,000
	15	310	Bluff slope regrading—fill	46,000	4,600[a]	59,000	4,000
	16	360	Bluff slope regrading—cut and fill	54,000	5,400[a]	68,000	4,000
	17	810	- -	- -	- -	- -	- -
	18	600	Bluff slope regrading—cut and fill	90,000	9,000[a]	114,000	7,000
	18	1,060	Groundwater drainage	53,000	10,600	220,000	14,000
	19	1,480	- -	- -	- -	- -	- -
	20	130	Bluff slope regrading—cutback	13,000	1,300[a]	16,000	1,000
	21	2,970	- -	- -	- -	- -	- -
	22	490	Bluff slope regrading—fill	74,000	7,400[a]	93,000	6,000
	23	140	Bluff slope regrading—fill	21,000	2,100[a]	27,000	2,000
	24	430	Bluff slope regrading—fill	64,000	6,400[a]	82,000	5,000
	25	480	Bluff slope regrading—cut and fill	18,000	1,800[a]	23,000	1,000
	26	170	Bluff slope regrading—cutback	26,000	2,600[a]	32,000	2,000
	27	1,950	Groundwater drainage, revegetation	117,000	23,400[b]	435,000	28,000
	28	1,150	Bluff slope regrading—cut and fill	172,000	17,200[a]	219,000	14,000
Village of Fox Point	29	320	Bluff slope regrading—fill	48,000	4,800[a]	61,000	4,000
	30	470	Bluff slope regrading—fill	70,000	7,000[a]	89,000	6,000
	31	510	Groundwater drainage, revegetation	33,000	6,100[b]	116,000	7,000
	32	770	Groundwater drainage, revegetation	62,000	12,300[b]	195,000	12,000
	33	530	Bluff slope regrading—fill	80,000	8,000[a]	101,000	6,000
	34	1,460	Bluff slope regrading—fill	219,000	21,900[a]	278,000	18,000
	35	9,070	- -	- -	- -	- -	- -
	36	840	- -	- -	- -	- -	- -
	Total	38,770	- -	$ 2,013,000	$238,300[c]	$ 3,608,000	$ 228,000

Table 55 (continued)

Civil Division	Bluff Analysis Section	Shoreline Length (feet)	Plan Component	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual Cost
BLUFF TOE PROTECTION							
City of Milwaukee	1	880	No toe protection	$ 0	$ 0	$ 0	$ 0
		1,090	Nourished beach system	436,000	21,800	780,000	49,000
	2	950	Nourished beach system	380,000	19,000	679,000	43,000
Village of Shorewood	3	300	Nourished beach system	120,000	6,000	215,000	14,000
	4	290	Nourished beach system	116,000	5,800	207,000	13,000
	5	1,710	Nourished beach system	684,000	34,200	1,223,000	78,000
	6	170	Nourished beach system	68,000	3,400	122,000	8,000
	7	380	Nourished beach system	152,000	7,600	272,000	17,000
	8	790	Offshore breakwater with nourished sand beach	1,185,000	39,500	1,808,000	115,000
		1,380	Nourished beach system	552,000	27,600	987,000	63,000
	9	280	Nourished beach system	112,000	5,600	200,000	13,000
		240	Construction of medium revetment	60,000	3,600	117,000	7,000
	10	240	Construction of medium revetment	60,000	3,600	117,000	7,000
Village of Whitefish Bay	11	2,370	Construction of heavy revetment	830,000	47,400	1,577,000	100,000
	12	850	Construction of heavy revetment	298,000	17,000	566,000	36,000
	13	190	Construction of heavy revetment	66,000	3,800	126,000	8,000
	14	160	Construction of heavy revetment	56,000	3,200	106,000	7,000
	15	310	Construction of heavy revetment	108,000	6,200	206,000	13,000
	16	360	Construction of heavy revetment	126,000	7,200	239,000	15,000
	17	810	Reconstruction of revetment-heavy	243,000	16,200	498,000	32,000
	18	600	Construction of heavy revetment	210,000	12,000	399,000	25,000
		1,060	Construction of light revetment	159,000	10,600	326,000	21,000
	19	1,480	Reconstruction of revetment-heavy	444,000	29,600	911,000	58,000
	20	130	No toe protection	0	0	0	0
	21	1,700	Reconstruction of revetment-heavy	510,000	34,000	1,046,000	66,000
		1,270	Construction of heavy revetment	444,000	25,400	844,000	54,000
	22	490	Construction of medium revetment	122,000	7, 400	238,000	15,000
	23	140	Construction of medium revetment	35,000	2,100	68,000	4,000
	24	430	Construction of medium revetment	108,000	6,400	209,000	13,000
	25	480	Offshore breakwater with nourished sand beach	720,000	24,000	1,098,000	70,000
	26	170	Nourished beach system	68,000	3,400	122,000	8,000
	27	1,950	Nourished beach system	780,000	39,000	1,395,000	89,000
	28	1,150	Construction of medium revetment	288,000	17,200	559,000	35,000
Village of Fox Point	29	320	Construction of medium revetment	80,000	4,800	156,000	10,000
	30	470	Reconstruction of revetment-medium	94,000	7,000	204,000	13,000
	31	510	Nourished beach system	204,000	10,200	365,000	23,000
	32	770	Nourished beach system	308,000	15,400	551,000	35,000
	33	530	Construction of heavy revetment	186,000	10,600	353,000	22,000
	34	1,460	Construction of heavy revetment	511,000	29,200	971,000	62,000
	35a	2,390	Continued maintenance of existing structures	- -	23,900	377,000	24,000
	35b	1,600	Reconstruction of revetment-medium	400,000	24,000	778,000	49,000
	35c	3,000	Continued maintenance of existing structures	- -	30,000	473,000	30,000
	35d	720	Reconstruction of revetment-medium	180,000	10,800	350,000	22,000
	35e	1,360	Continued maintenance of existing structures	- -	13,600	214,000	14,000
	36	840	Offshore breakwater with nourished sand beach	1,260,000	42,000	1,922,000	122,000
	Total	38,770	- -	$12,763,000	$711,300	$23,974,000	$1,522,000

Table 55 (continued)

TOTAL PLAN						
Civil Division	Bluff Analysis Section	Shoreline Length (feet)	Capital	Annual Maintenance	50-Year Present Worth	Equivalent Annual Cost
City of Milwaukee	1	1,970	$ 456,000	$ 25,700	$ 810,000	$ 51,000
	2	950	394,000	21,800	701,000	44,000
Village of Shorewood	3	300	150,000	9,000	253,000	16,000
	4	290	139,000	8,700	238,000	15,000
	5	1,710	684,000	34,200	1,223,000	78,000
	6	170	68,000	3,400	122,000	8,000
	7	380	209,000	13,300	344,000	22,000
	8	2,170	1,806,000	80,900	3,082,000	196,000
	9	520	172,000	9,200	317,000	20,000
	10	240	64,000	4,300	123,000	7,000
	11	2,370	1,186,000	83,000	1,996,000	127,000
Village of Whitefish Bay	12	850	426,000	29,800	728,000	46,000
	13	190	94,000	6,600	409,000	26,000
	14	160	80,000	5,600	136,000	9,000
	15	310	154,000	10,800	265,000	17,000
	16	360	180,000	12,600	307,000	19,000
	17	810	243,000	16,200	498,000	32,000
	18	1,660	512,000	42,200	1,059,000	67,000
	19	1,480	444,000	29,600	911,000	58,000
	20	130	13,000	1,300	16,000	1,000
	21	2,970	954,000	59,400	1,890,000	120,000
	22	490	196,000	14,800	331,000	21,000
	23	140	56,000	4,200	95,000	6,000
	24	430	172,000	12,800	291,000	18,000
	25	480	738,000	25,800	1,121,000	71,000
	26	170	94,000	6,000	154,000	10,000
	27	1,950	897,000	62,400	1,830,000	117,000
	28	1,150	460,000	34,400	778,000	49,000
Village of Fox Point[d]	29	320	128,000	9,600	217,000	14,000
	30	470	164,000	14,000	293,000	19,000
	31	510	237,000	16,300	481,000	30,000
	32	770	370,000	27,700	746,000	47,000
	33	530	266,000	18,600	454,000	28,000
	34	1,460	730,000	51,100	1,249,000	80,000
	35	9,070	580,000	102,300	2,192,000	139,000
	36	840	1,260,000	42,000	1,922,000	122,000
	Total	38,770	$14,776,000	$949,600	$27,582,000	$1,750,000

[a]*Annual maintenance costs would apply only for first three years following bluff slope regrading or revegetation.*

[b]*Of the total annual maintenance cost of $23,400 for stabilizing the bluff slope within Bluff Analysis Section 27, $3,900, or 17 percent, would be required only for the first three years following revegetation. Of the total annual maintenance cost of $6,100 for stabilizing the bluff slope within Bluff Analysis Section 31, $1,000, or 16 percent, would be required only for the first three years following revegetation. Of the total annual maintenance cost of $12,300 for stabilizing the bluff slope within Bluff Analysis Section 32, $4,600, or 37 percent, would be required only for the first three years following revegetation.*

[c]*About $181,600, or 76 percent, of the total annual maintenance cost would be required only for the first three years following bluff slope regrading or revegetation.*

[d]*In addition, it is recommended that the Village of Fox Point implement a coastal monitoring program for the Fox Point terrace, which would require surveys at two- to five-year intervals.*

Source: SEWRPC.

Table 56

DISTRIBUTION OF THE ESTIMATED COST OF THE FINAL RECOMMENDED SHORELINE EROSION MANAGEMENT PLAN

Civil Division	Public or Private Sector	Capital		Annual Maintenance		50-Year Present Worth		Equivalent Annual Cost	
		Cost	Percent of Total	Cost	Percent of Total	Cost	Percent of Total	Cost	Percent of Total
City of Milwaukee	Public	$ 0	0	$ 0	0	$ 0	0	$ 0	0
	Private	850,000	5.7	47,500	5.0	1,511,000	5.5	95,000	5.4
	Subtotal	$ 850,000	5.7	$ 47,500	5.0	$ 1,511,000	5.5	$ 95,000	5.4
Village of Shorewood	Public	$ 1,869,000	12.7	$ 73,700	7.7	$ 3,031,000	11.0	$ 193,000	11.0
	Private	1,826,000	12.4	117,500	12.4	3,350,000	12.1	212,000	12.1
	Subtotal	$ 3,695,000	25.1	$191,200	20.1	$ 6,381,000	23.1	$ 405,000	23.1
Village of Whitefish Bay	Public[a]	$ 1,317,000	8.9	$ 72,200	7.6	$ 2,312,000	8.4	$ 146,000	8.3
	Private	4,935,000	33.4	338,900	35.7	9,412,000	34.1	599,000	34.3
	Subtotal	$ 6,252,000	42.3	$411,100	43.3	$11,724,000	42.5	$ 745,000	42.6
Village of Fox Point[d]	Public[b]	$ 580,000	3.9	$ 34,800	3.7	$ 1,128,000	4.1	$ 71,000	4.1
	Private	3,399,000	23.0	265,000	27.9	6,838,000	24.8	434,000	24.8
	Subtotal	$ 3,979,000	26.9	$299,800	31.6	$ 7,966,000	28.9	$ 505,000	28.9
Study Area Total	Public	$ 3,766,000	25.5	$180,700	19.0	$ 6,471,000	23.5	$ 410,000	23.4
	Private	$11,010,000	74.5	$768,900	81.0	$21,111,000	76.5	$1,340,000	76.6
	Total	$14,776,000	100.0	$949,600[c]	100.0	$27,582,000	100.0	$1,750,000	100.0

[a] *A groundwater drainage system and a light revetment at Big Bay Park would be implemented by Milwaukee County at a capital cost of about $212,000, and an annual maintenance cost of about $21,200.*

[b] *The offshore breakwater with a sand beach at Doctors Park would be implemented by Milwaukee County at a capital cost of about $1,260,000 and an annual maintenance cost of about $42,000.*

[c] *Of the total maintenance cost of $949,600, $181,600, or 19 percent, would be required only for the first three years following bluff slope regrading or revegetation.*

[d] *In addition, it is recommended that the Village of Fox Point implement a coastal monitoring program for the Fox Point terrace which would require surveys at two- to five-year intervals.*

Source: SEWRPC.

implementation segments for the final recommended plan would be the same as those shown on Map 27 in Chapter IV. The final recommended shore protection measures for each implementation segment are presented in Table 57.

The successful implementation of the final recommended plan will require not only a substantial capital investment, but a stable, long-term commitment to carrying out the recommended capital improvement projects and related maintenance programs. Adoption and implementation of the plan should, however, ensure the provision of a high-quality, well-managed coastal environment for northern Milwaukee County.

Table 57

FINAL RECOMMENDED IMPLEMENTATION SEGMENTS FOR NORTHERN MILWAUKEE COUNTY

Implementation Segment	Shoreline Length (feet)	Address	Civil Division	Bluff Analysis Sections	Final Recommended Bluff Slope Stabilization Measures[a]	Final Recommended Bluff Toe Protection Measures[a]
A	2,920	Linnwood Avenue water treatment plant-3474 N. Lake Drive	Milwaukee	1-2	Surface water runoff control, revegetation	Nourished gravel beach system
B	3,640	3510 N. Lake Drive-Atwater Park	Shorewood	3-8	Cut and fill, fill, surface water runoff control, revegetation	Nourished gravel beach system, offshore breakwater with nourished sand beach
C	1,660	4060-4240 N. Lake Drive	Shorewood	8-9	Groundwater drainage	Nourished gravel beach system with short groins
D	5,530	4300-4940 N. Lake Drive	Shorewood-Whitefish Bay	9-17	Cut and fill, fill, revegetation	Medium and heavy revetments
E	1,660	Buckley Park-Big Bay Park	Whitefish Bay	18	Groundwater drainage, cut and fill	Light revetment
F	4,580	Big Bay Park-808 Lakeview Avenue	Whitefish Bay	19-21	Cutback	Heavy revetment
G	1,060	5722-5866 N. Shore Drive	Whitefish Bay	22-24	Fill	Medium revetment
H	480	Klode Park	Whitefish Bay	25	Cut and fill	Offshore breakwater with nourished sand beach
I	2,120	5960 N. Shore Drive-6260 N. Lake Drive	Whitefish Bay	26-27	Cutback, groundwater drainage, revegetation	Nourished gravel beach system
J	1,940	6310-6530 N. Lake Drive	Whitefish Bay-Fox Point	28-30	Cut and fill, fill	Medium revetment
K	1,280	6600 N. Lake Drive-6818 N. Barnett Lane	Fox Point	31-32	Groundwater drainage, revegetation	Nourished gravel beach system
L	1,990	6820-7010 N. Barnett Lane	Fox Point	33-34	Fill	Heavy revetment
M	2,390	7038-7828 N. Beach Drive	Fox Point	35a	- -	Continued maintenance of existing structures
N	1,600	7405-7535 N. Beach Drive	Fox Point	35b	- -	Reconstruction of revetment-medium
O	3,000	7540-7966 N. Beach Drive	Fox Point	35c	- -	Continued maintenance of existing structures
P	720	8005-8035 N. Beach Drive	Fox Point	35d	- -	Reconstruction of revetment-medium
Q	1,360	8040-8135 N. Beach Drive	Fox Point	35e	- -	Continued maintenance of existing structures
R	840	Doctors Park	Fox Point	36	- -	Offshore breakwater with nourished sand beach

[a]*Not all listed slope stabilization and bluff toe protection measures may be required for the entire implementation segment.*

Source: SEWRPC.

(This page intentionally left blank)

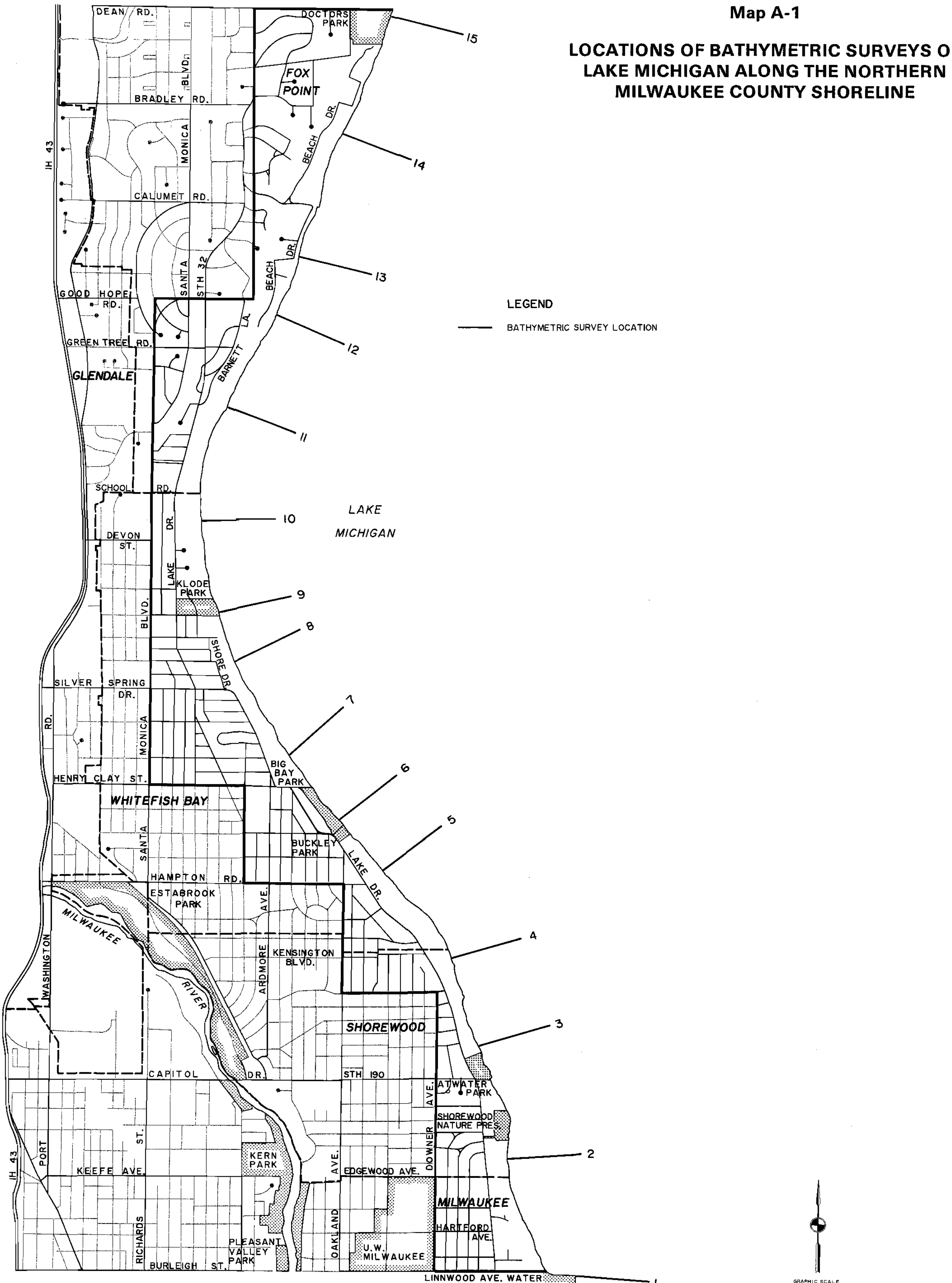
APPENDICES

Appendix A

BATHYMETRIC SURVEYS OF LAKE MICHIGAN BY WARZYN ENGINEERING, INC.: AUGUST 1986

Map A-1

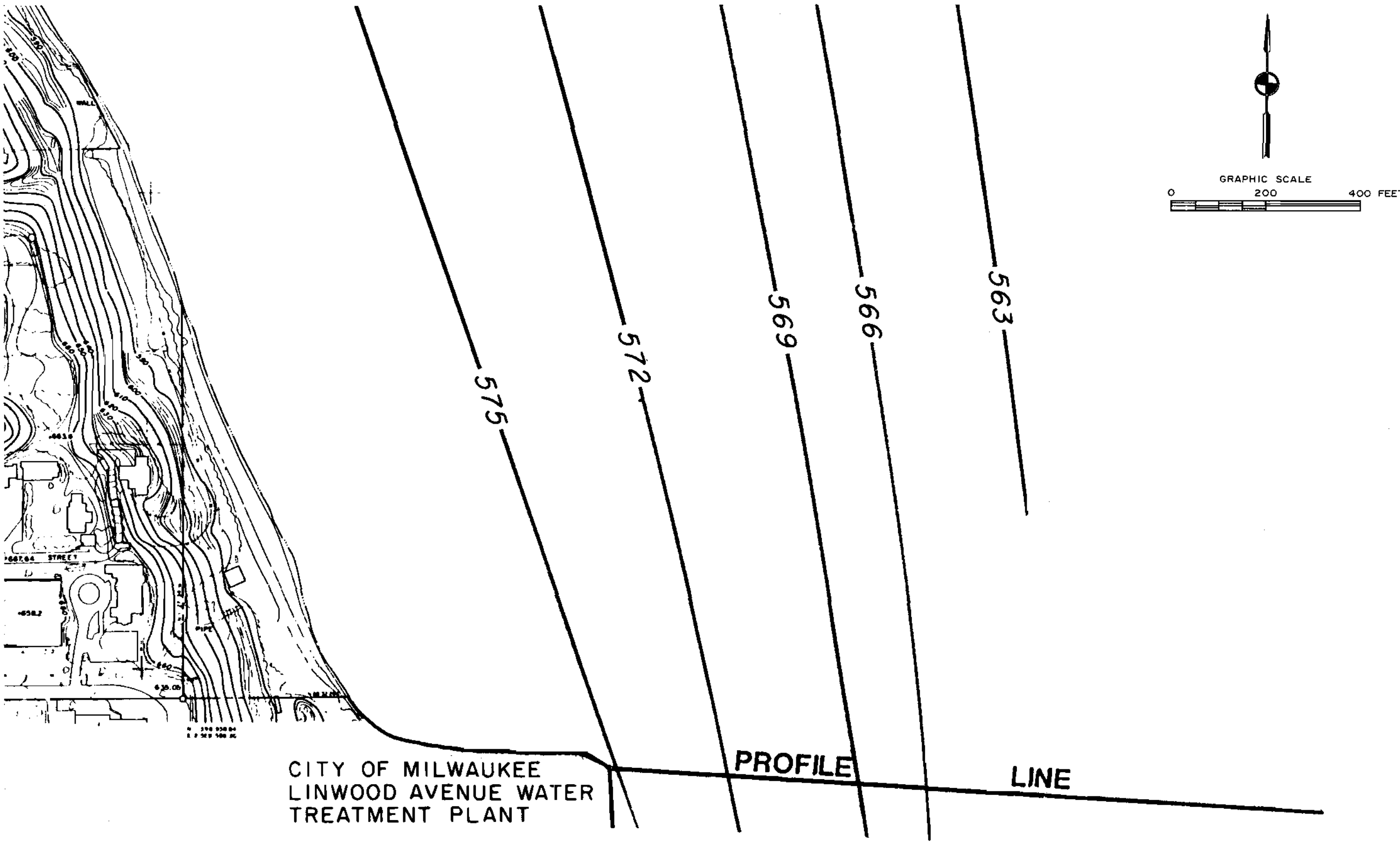
LOCATIONS OF BATHYMETRIC SURVEYS OF LAKE MICHIGAN ALONG THE NORTHERN MILWAUKEE COUNTY SHORELINE



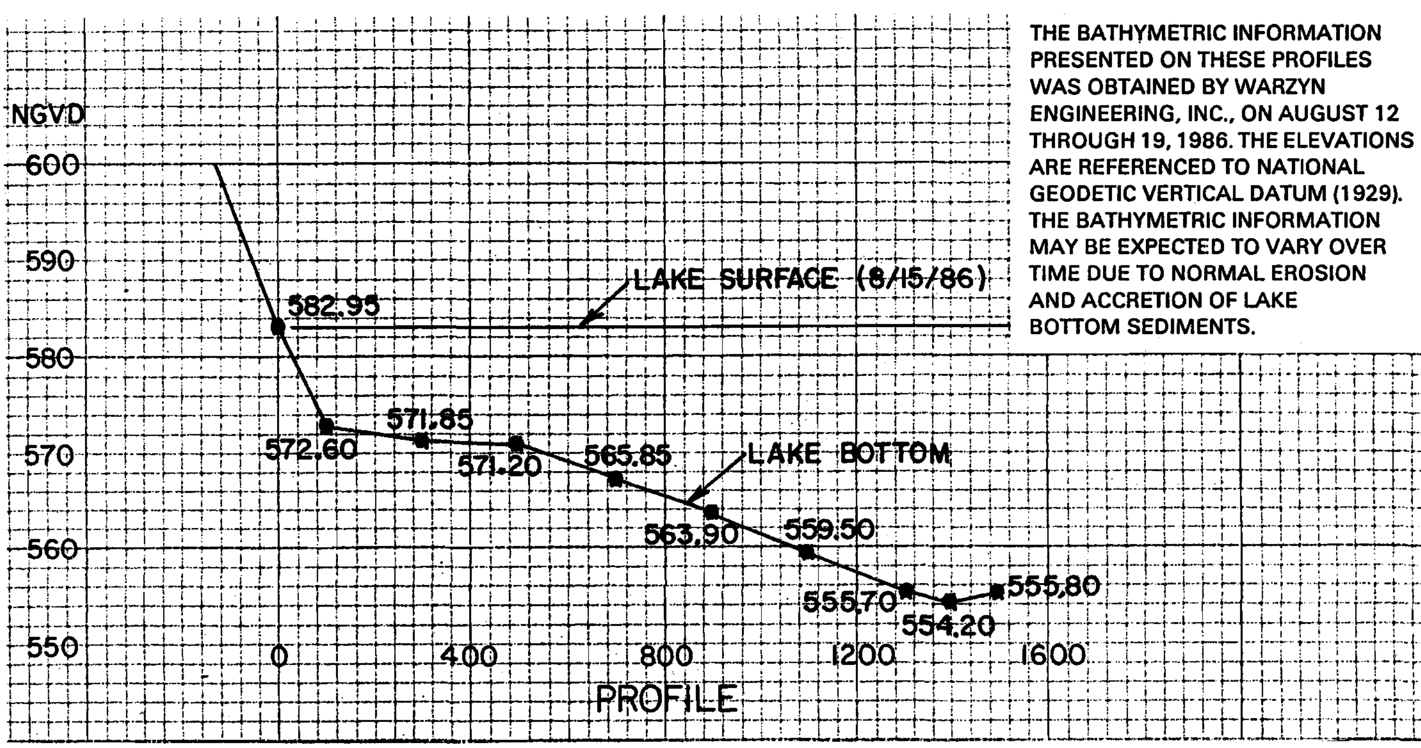
Source: SEWRPC.

Figure A-1

BATHYMETRIC SURVEY, LOCATION 1



PLAN

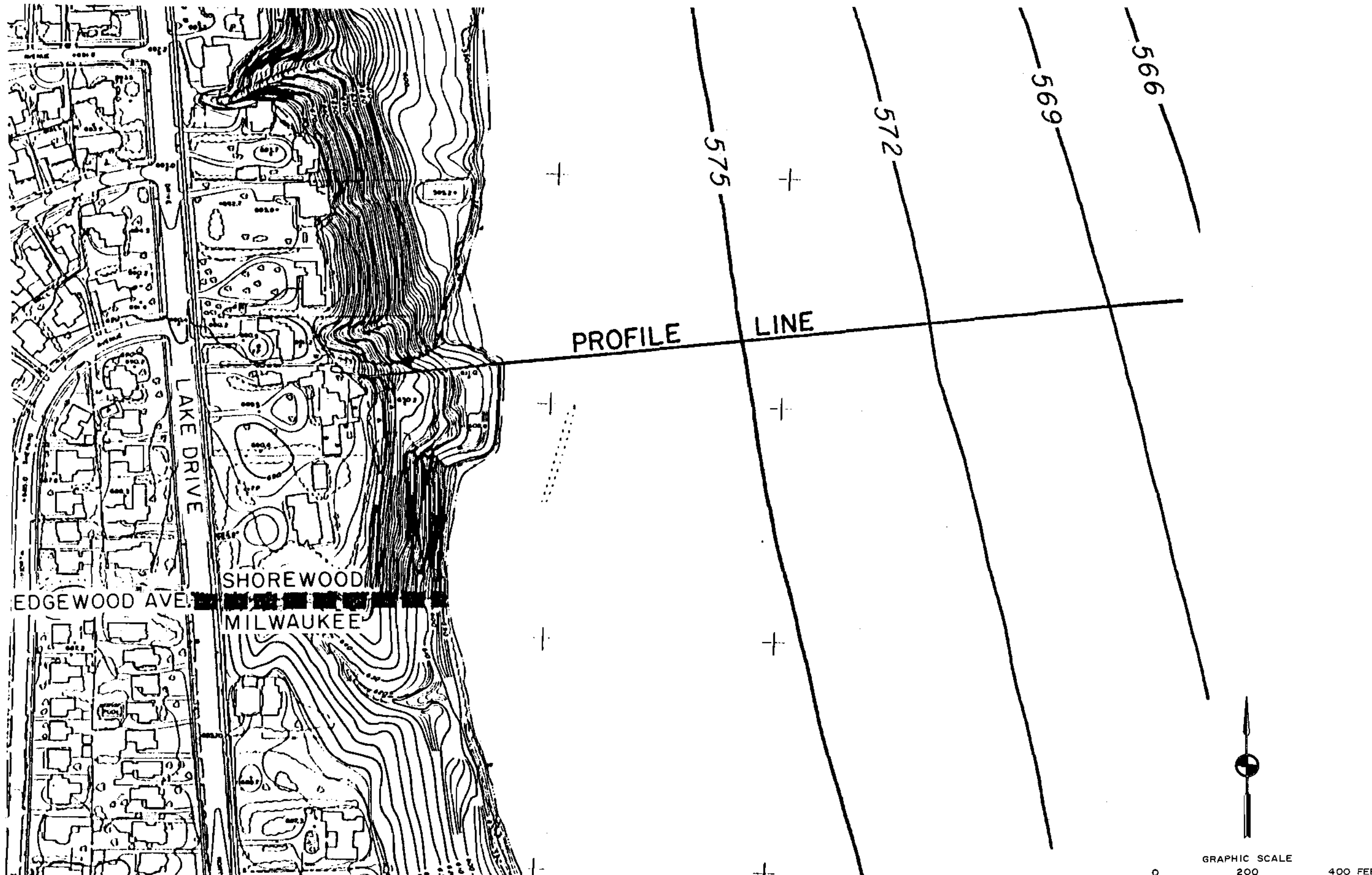


THE BATHYMETRIC INFORMATION PRESENTED ON THESE PROFILES WAS OBTAINED BY WARZYN ENGINEERING, INC., ON AUGUST 12 THROUGH 19, 1986. THE ELEVATIONS ARE REFERENCED TO NATIONAL GEODETIC VERTICAL DATUM (1929). THE BATHYMETRIC INFORMATION MAY BE EXPECTED TO VARY OVER TIME DUE TO NORMAL EROSION AND ACCRETION OF LAKE BOTTOM SEDIMENTS.

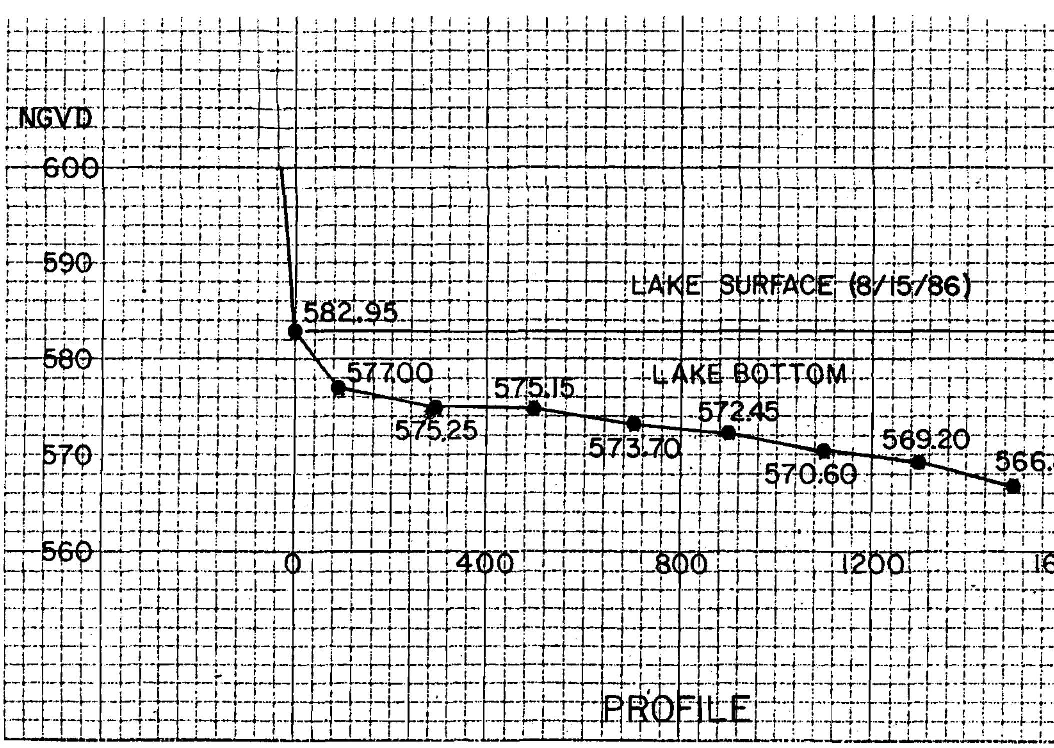
Source: Warzyn Engineering, Inc.

Figure A-2

BATHYMETRIC SURVEY, LOCATION 2



PLAN



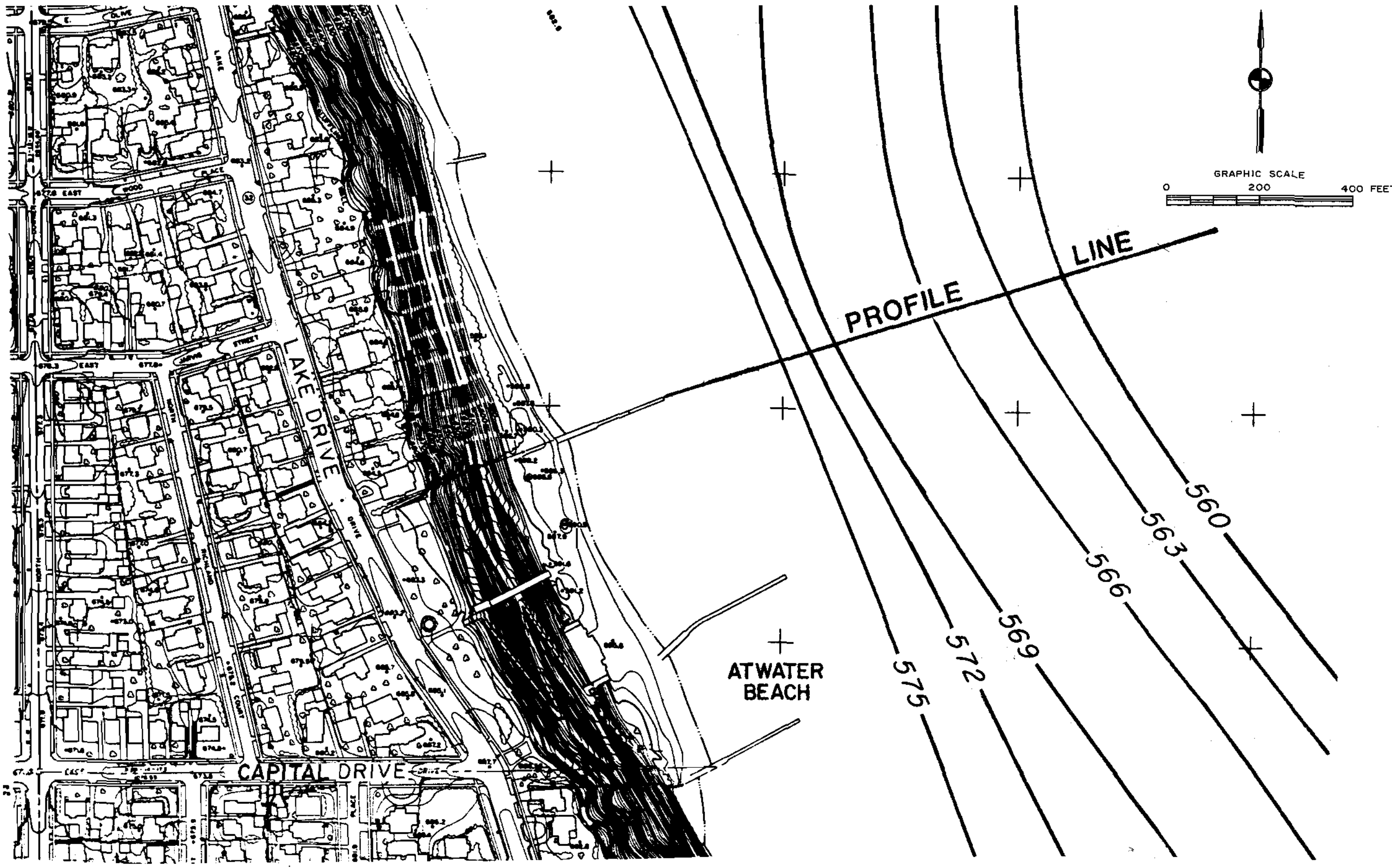
PROFILE

THE BATHYMETRIC INFORMATION PRESENTED ON THESE PROFILES WAS OBTAINED BY WARZYN ENGINEERING, INC., ON AUGUST 12 THROUGH 19, 1986. THE ELEVATIONS ARE REFERENCED TO NATIONAL GEODETIC VERTICAL DATUM (1929). THE BATHYMETRIC INFORMATION MAY BE EXPECTED TO VARY OVER TIME DUE TO NORMAL EROSION AND ACCRETION OF LAKE BOTTOM SEDIMENTS.

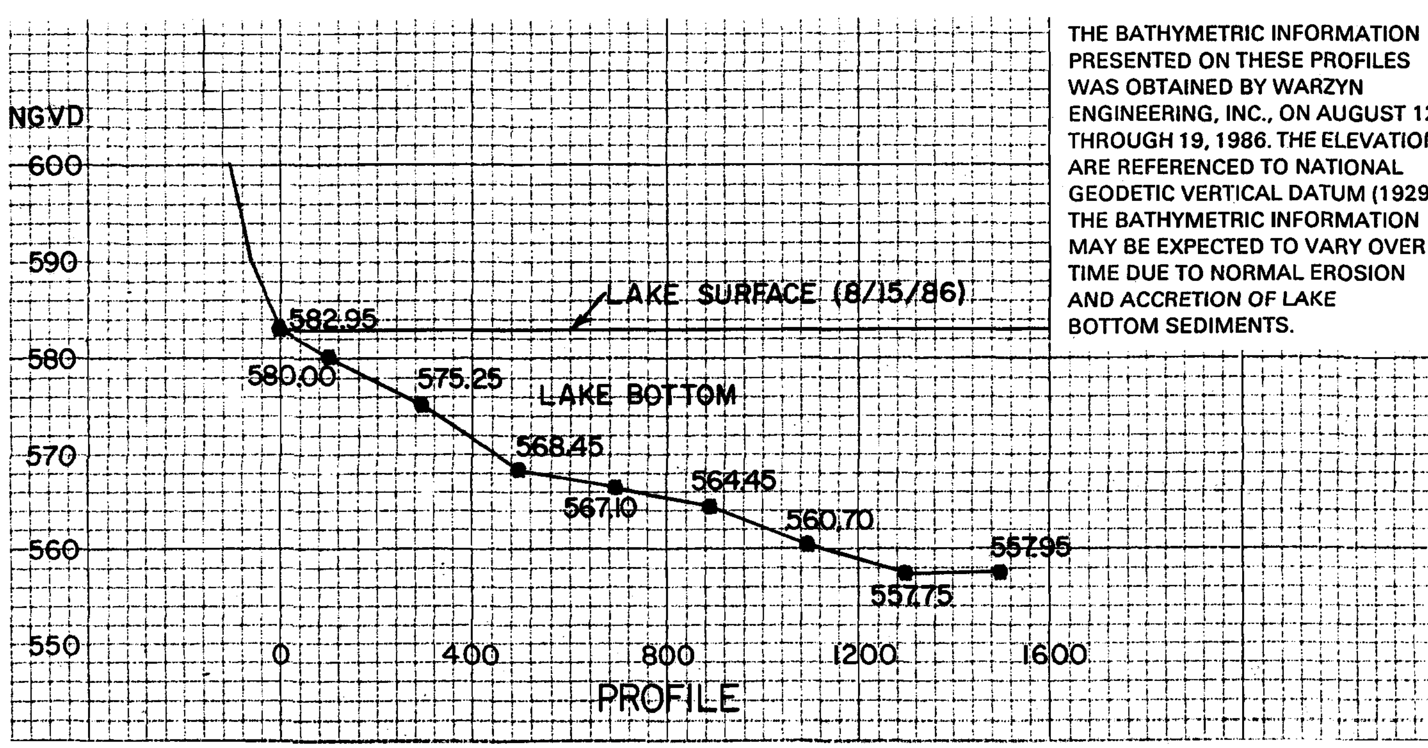
Source: Warzyn Engineering, Inc.

Figure A-3

BATHYMETRIC SURVEY, LOCATION 3



PLAN

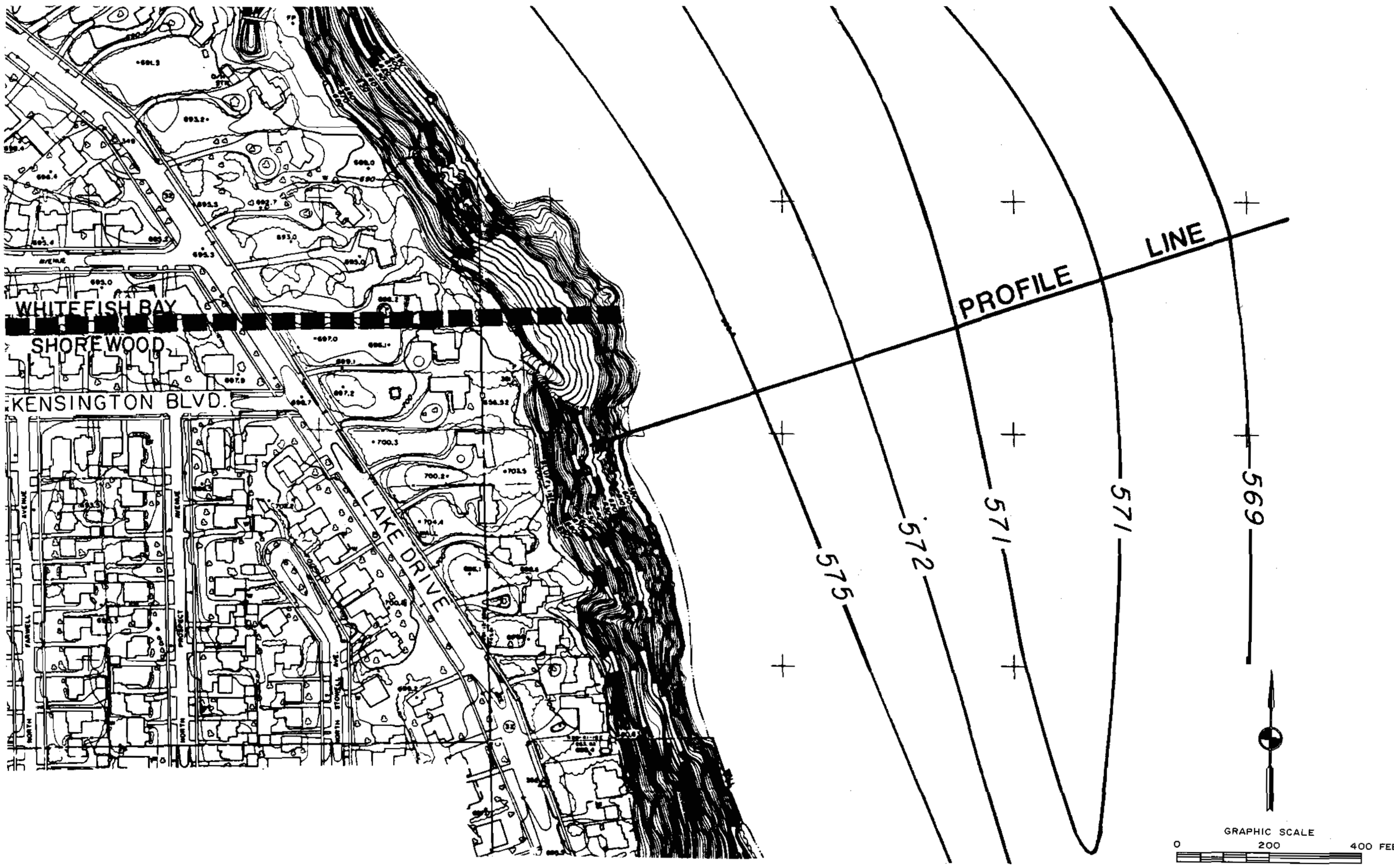


THE BATHYMETRIC INFORMATION PRESENTED ON THESE PROFILES WAS OBTAINED BY WARZYN ENGINEERING, INC., ON AUGUST 12 THROUGH 19, 1986. THE ELEVATIONS ARE REFERENCED TO NATIONAL GEODETIC VERTICAL DATUM (1929). THE BATHYMETRIC INFORMATION MAY BE EXPECTED TO VARY OVER TIME DUE TO NORMAL EROSION AND ACCRETION OF LAKE BOTTOM SEDIMENTS.

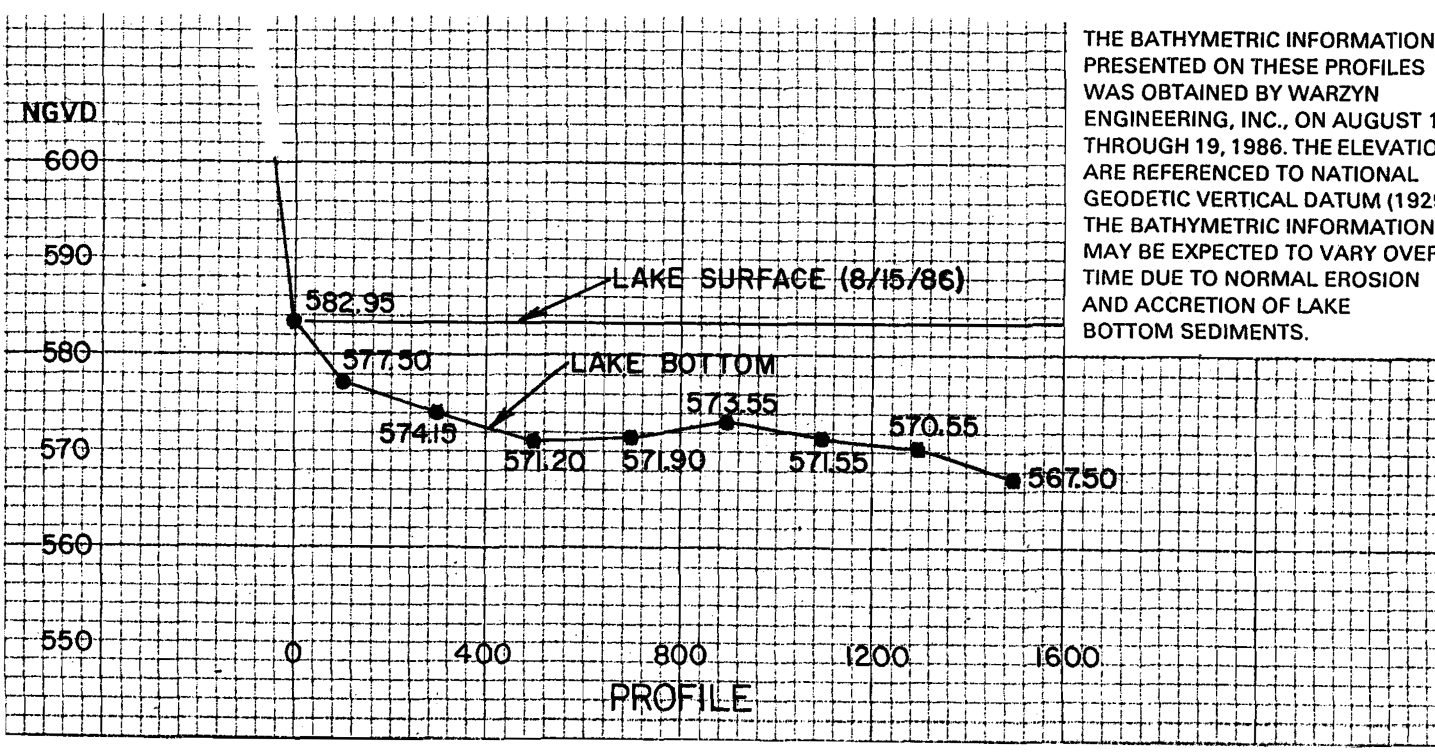
Source: Warzyn Engineering, Inc.

Figure A-4

BATHYMETRIC SURVEY, LOCATION 4



PLAN



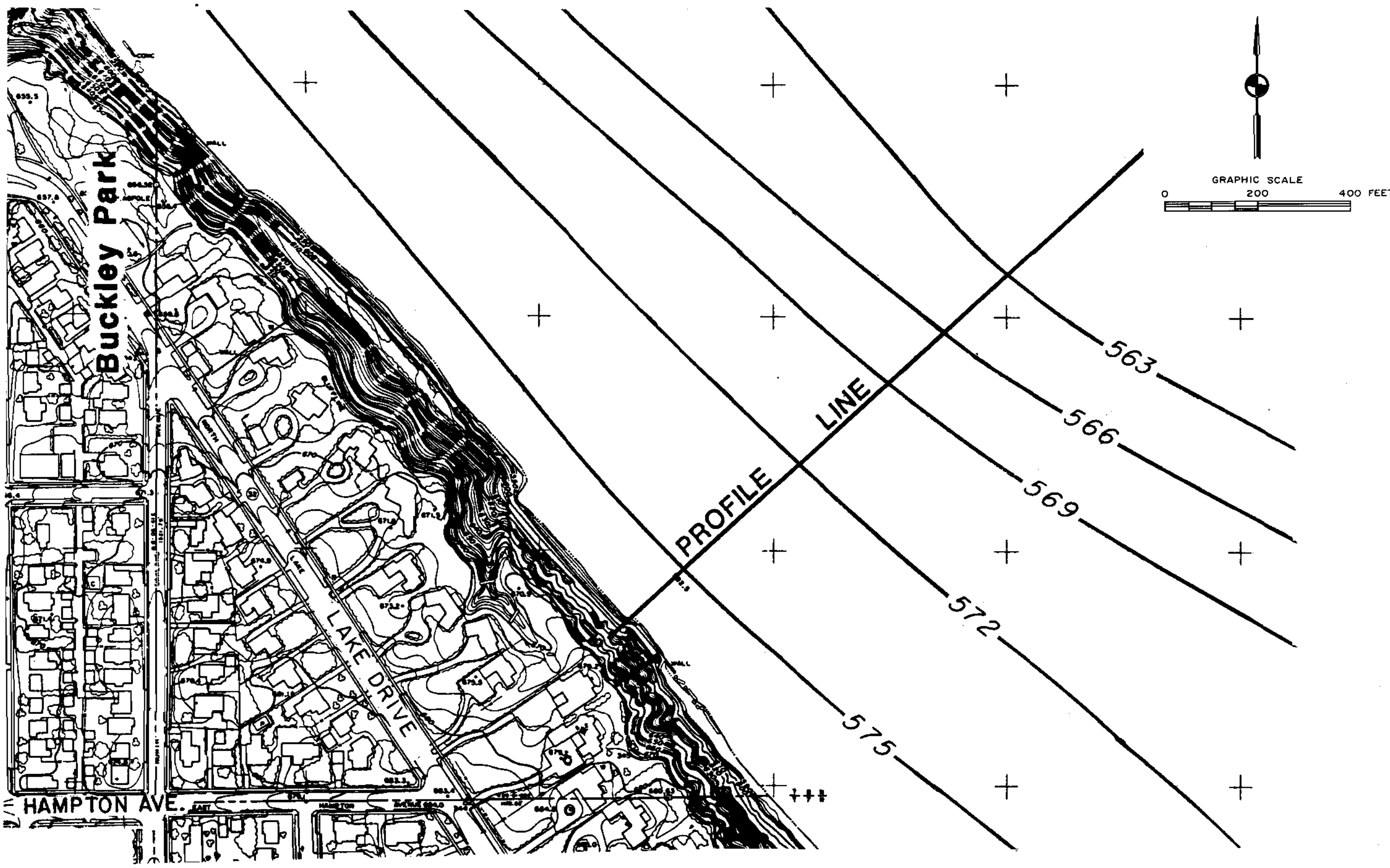
PROFILE

THE BATHYMETRIC INFORMATION PRESENTED ON THESE PROFILES WAS OBTAINED BY WARZYN ENGINEERING, INC., ON AUGUST 12 THROUGH 19, 1986. THE ELEVATIONS ARE REFERENCED TO NATIONAL GEODETIC VERTICAL DATUM (1929). THE BATHYMETRIC INFORMATION MAY BE EXPECTED TO VARY OVER TIME DUE TO NORMAL EROSION AND ACCRETION OF LAKE BOTTOM SEDIMENTS.

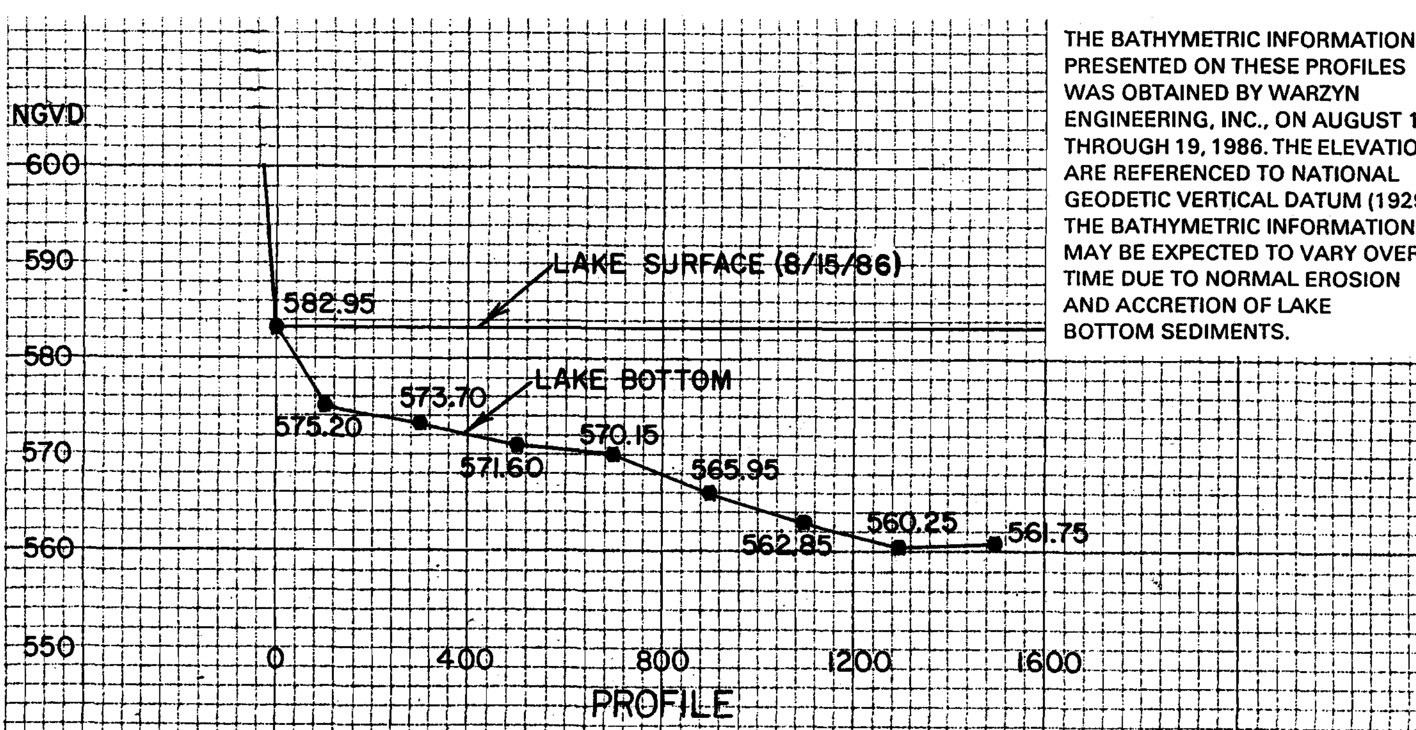
Source: Warzyn Engineering, Inc.

Figure A-5

BATHYMETRIC SURVEY, LOCATION 5



PLAN

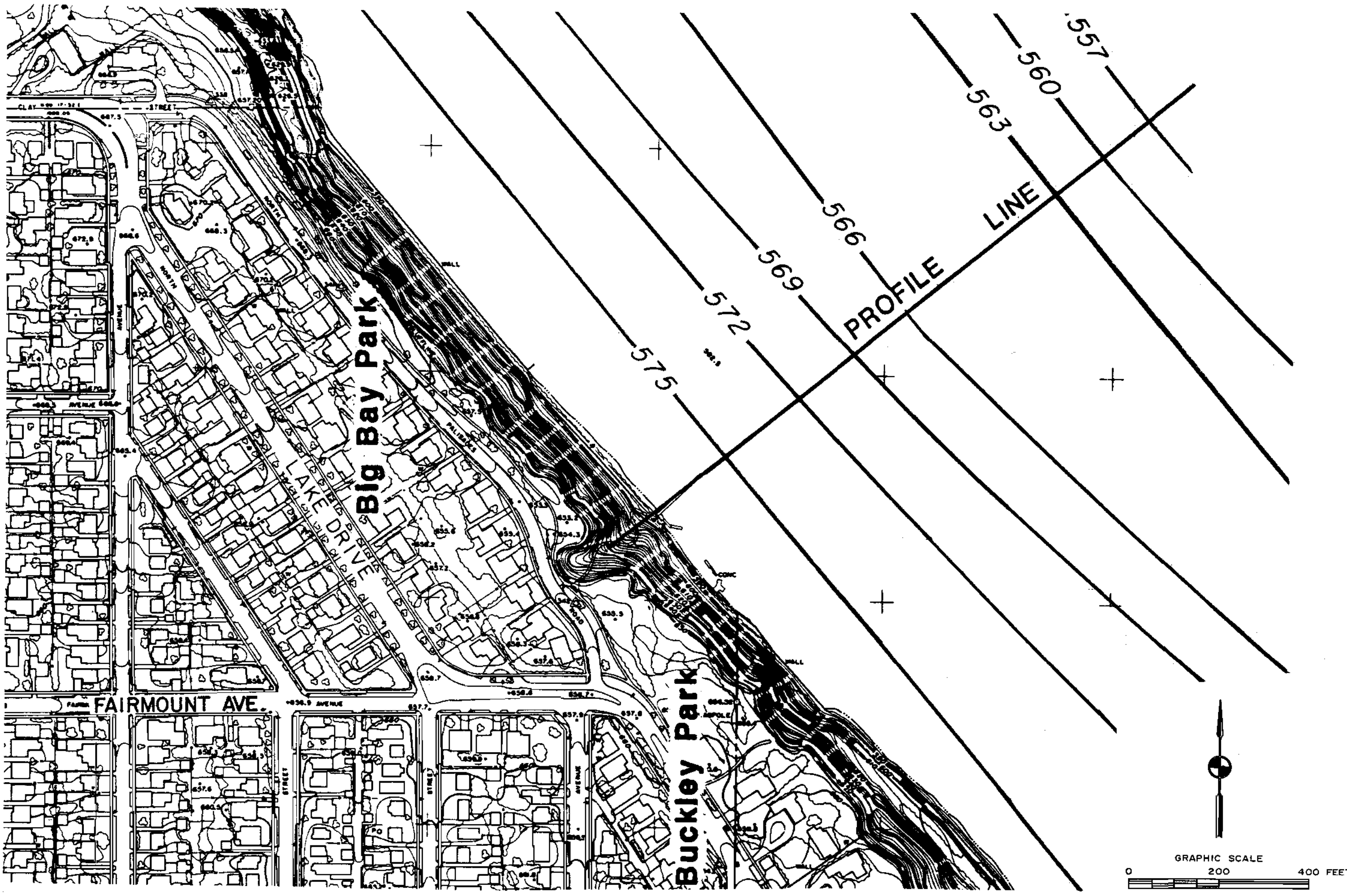


THE BATHYMETRIC INFORMATION PRESENTED ON THESE PROFILES WAS OBTAINED BY WARZYN ENGINEERING, INC., ON AUGUST 12 THROUGH 19, 1986. THE ELEVATIONS ARE REFERENCED TO NATIONAL GEODETIC VERTICAL DATUM (1929). THE BATHYMETRIC INFORMATION MAY BE EXPECTED TO VARY OVER TIME DUE TO NORMAL EROSION AND ACCRETION OF LAKE BOTTOM SEDIMENTS.

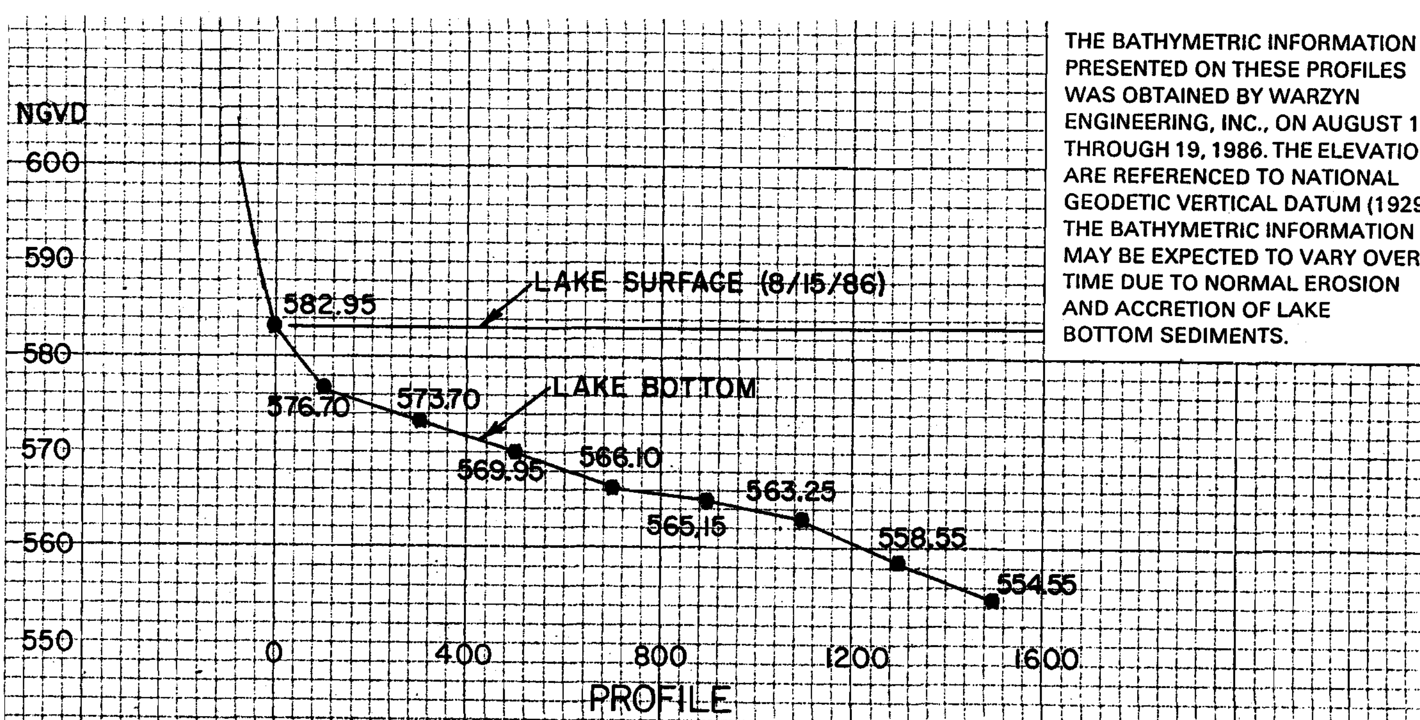
Source: Warzyn Engineering, Inc.

Figure A-6

BATHYMETRIC SURVEY, LOCATION 6



PLAN

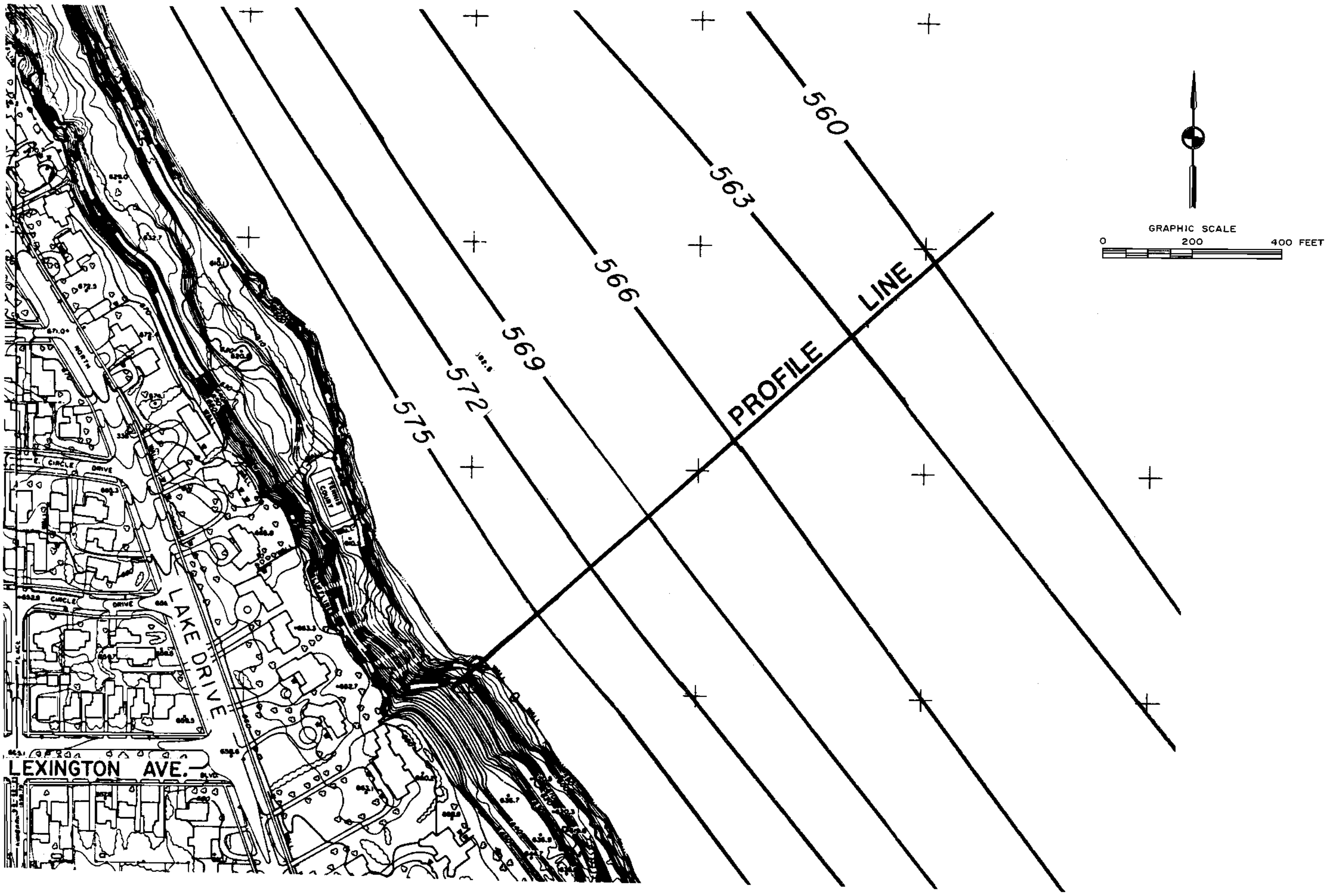


THE BATHYMETRIC INFORMATION PRESENTED ON THESE PROFILES WAS OBTAINED BY WARZYN ENGINEERING, INC., ON AUGUST 12 THROUGH 19, 1986. THE ELEVATIONS ARE REFERENCED TO NATIONAL GEODETIC VERTICAL DATUM (1929). THE BATHYMETRIC INFORMATION MAY BE EXPECTED TO VARY OVER TIME DUE TO NORMAL EROSION AND ACCRETION OF LAKE BOTTOM SEDIMENTS.

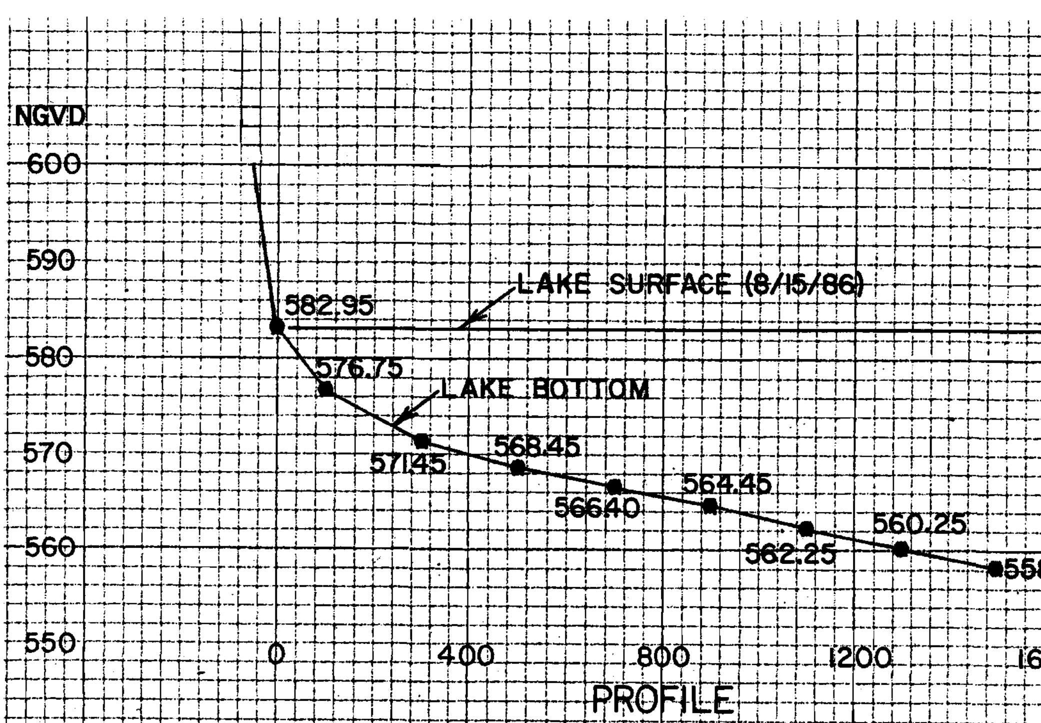
Source: Warzyn Engineering, Inc.

Figure A-7

BATHYMETRIC SURVEY, LOCATION 7



PLAN



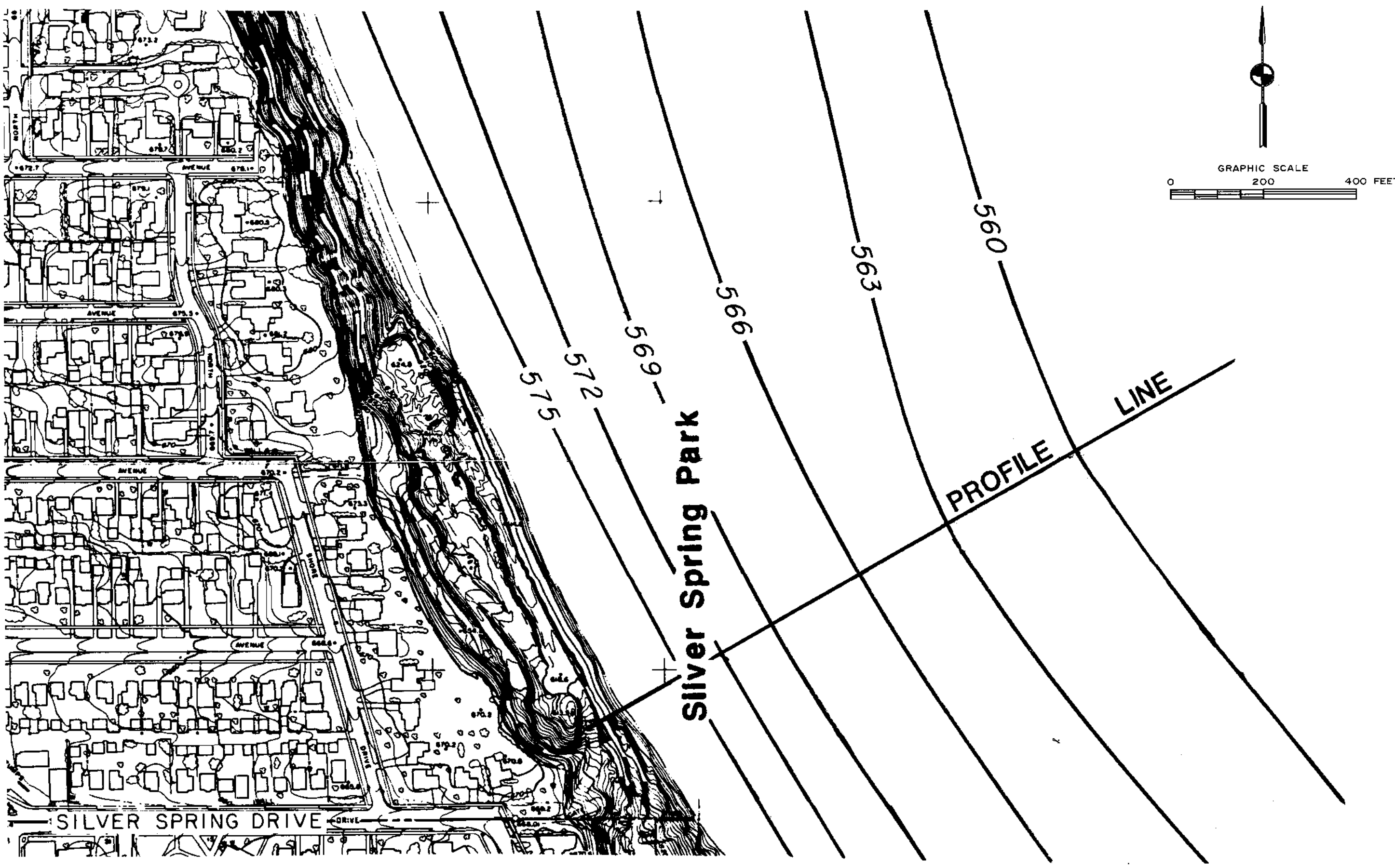
PROFILE

THE BATHYMETRIC INFORMATION PRESENTED ON THESE PROFILES WAS OBTAINED BY WARZYN ENGINEERING, INC., ON AUGUST 12 THROUGH 19, 1986. THE ELEVATIONS ARE REFERENCED TO NATIONAL GEODETIC VERTICAL DATUM (1929). THE BATHYMETRIC INFORMATION MAY BE EXPECTED TO VARY OVER TIME DUE TO NORMAL EROSION AND ACCRETION OF LAKE BOTTOM SEDIMENTS.

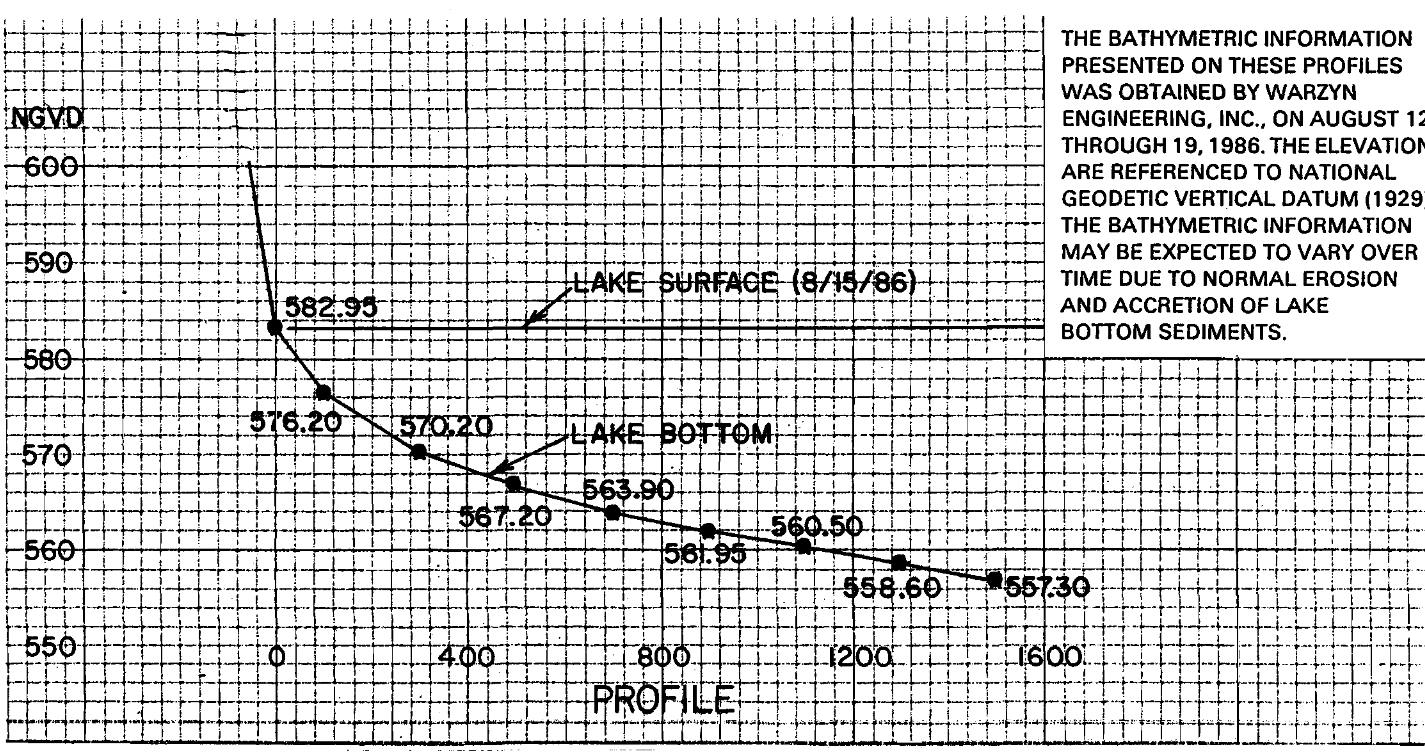
Source: Warzyn Engineering, Inc.

Figure A-8

BATHYMETRIC SURVEY, LOCATION 8



PLAN

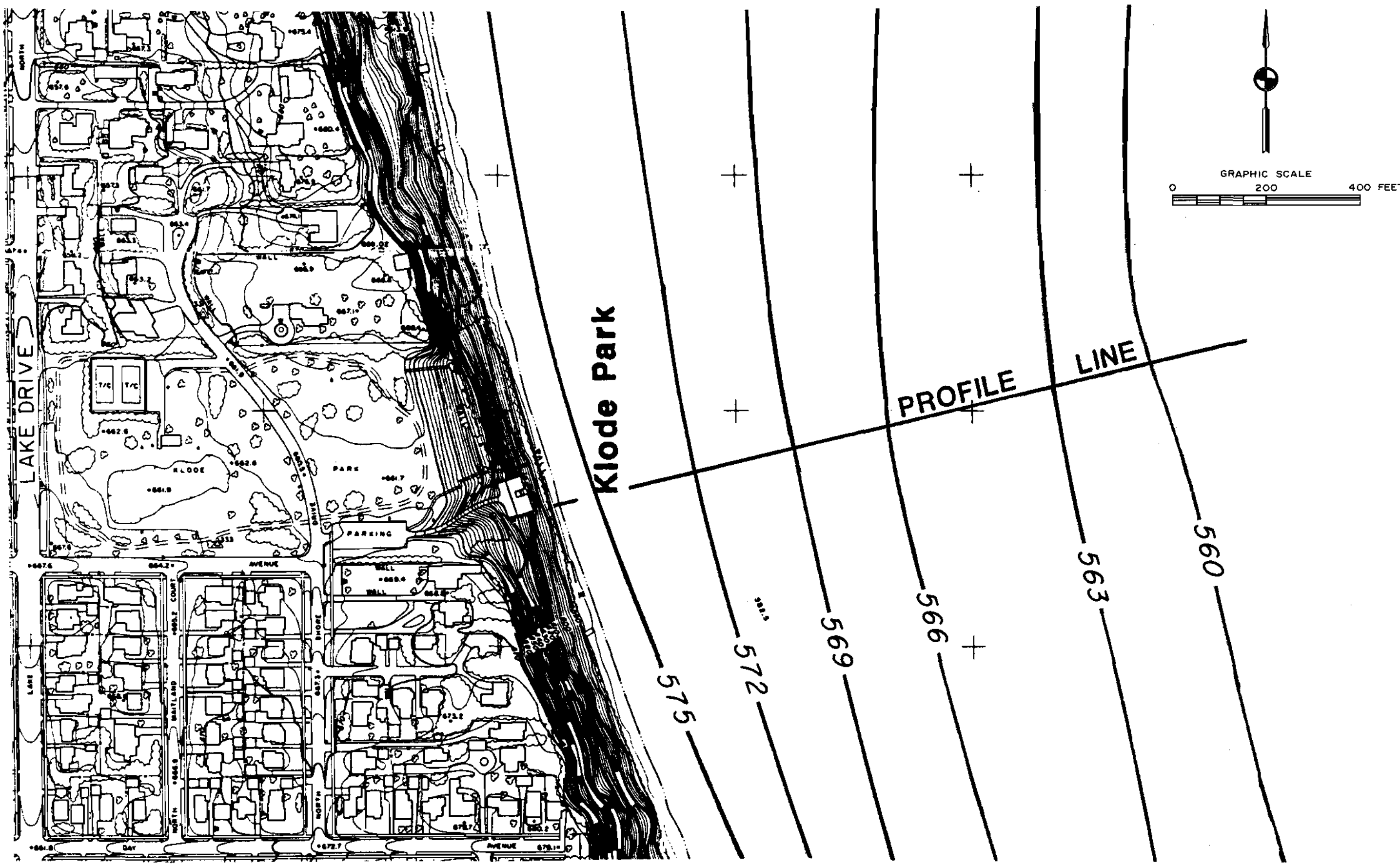


THE BATHYMETRIC INFORMATION PRESENTED ON THESE PROFILES WAS OBTAINED BY WARZYN ENGINEERING, INC., ON AUGUST 12 THROUGH 19, 1986. THE ELEVATIONS ARE REFERENCED TO NATIONAL GEODETIC VERTICAL DATUM (1929). THE BATHYMETRIC INFORMATION MAY BE EXPECTED TO VARY OVER TIME DUE TO NORMAL EROSION AND ACCRETION OF LAKE BOTTOM SEDIMENTS.

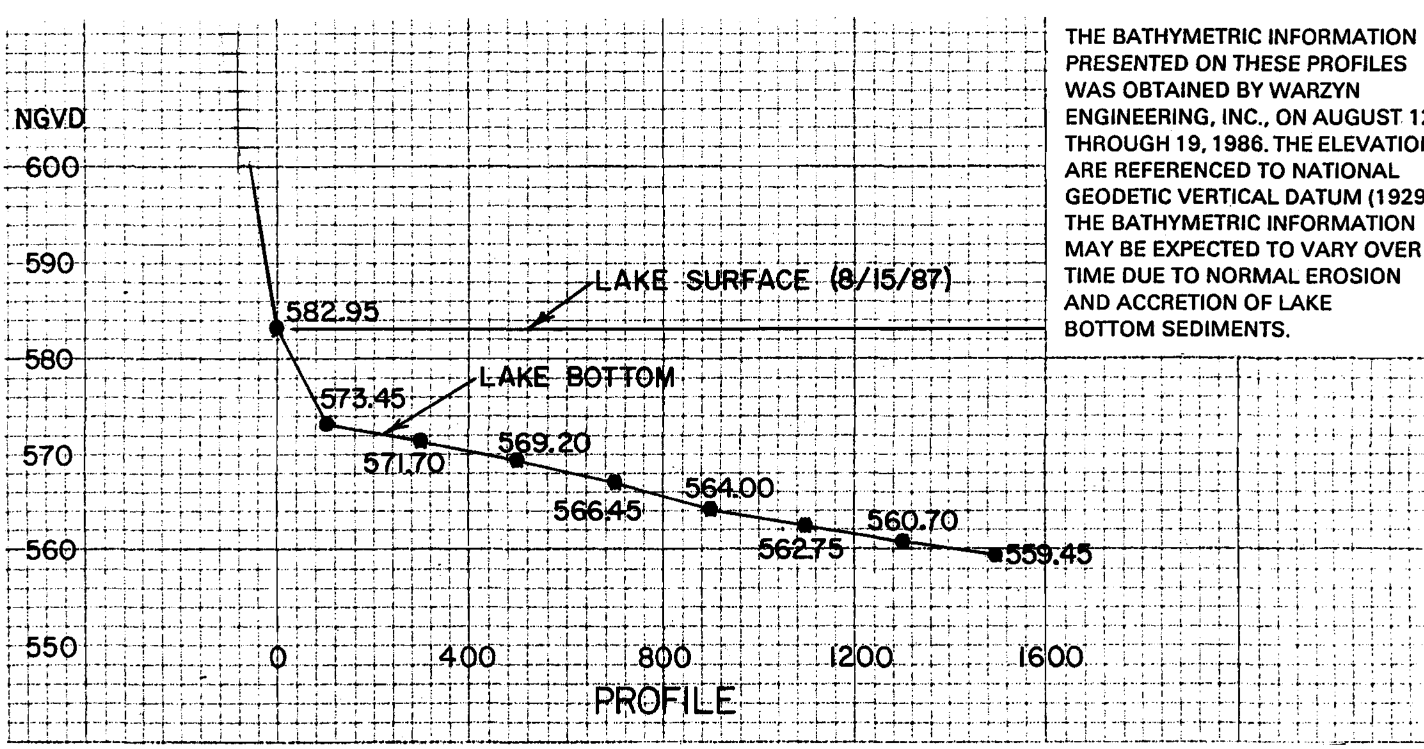
Source: Warzyn Engineering, Inc.

Figure A-9

BATHYMETRIC SURVEY, LOCATION 9



PLAN

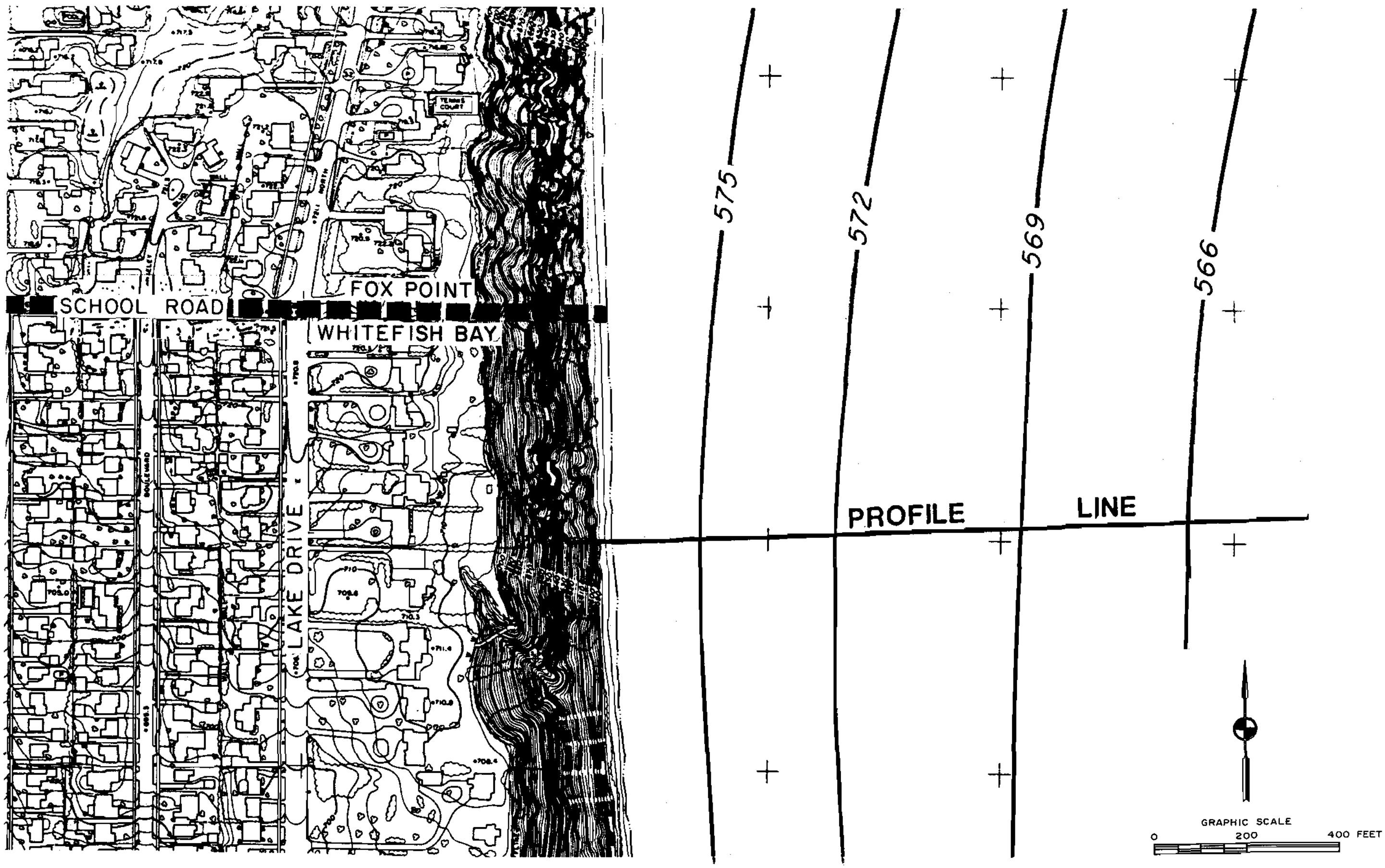


THE BATHYMETRIC INFORMATION PRESENTED ON THESE PROFILES WAS OBTAINED BY WARZYN ENGINEERING, INC., ON AUGUST 12 THROUGH 19, 1986. THE ELEVATIONS ARE REFERENCED TO NATIONAL GEODETIC VERTICAL DATUM (1929). THE BATHYMETRIC INFORMATION MAY BE EXPECTED TO VARY OVER TIME DUE TO NORMAL EROSION AND ACCRETION OF LAKE BOTTOM SEDIMENTS.

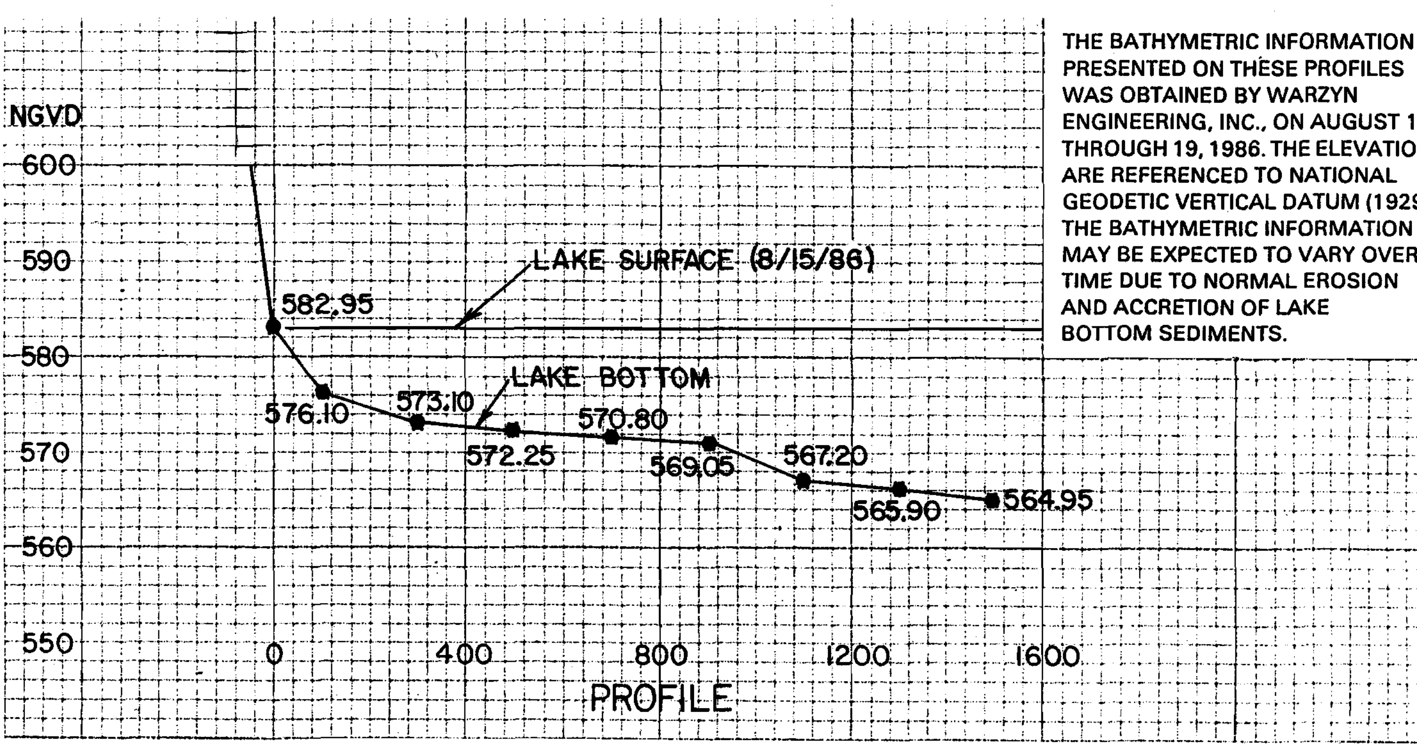
Source: Warzyn Engineering, Inc.

Figure A-10

BATHYMETRIC SURVEY, LOCATION 10



PLAN



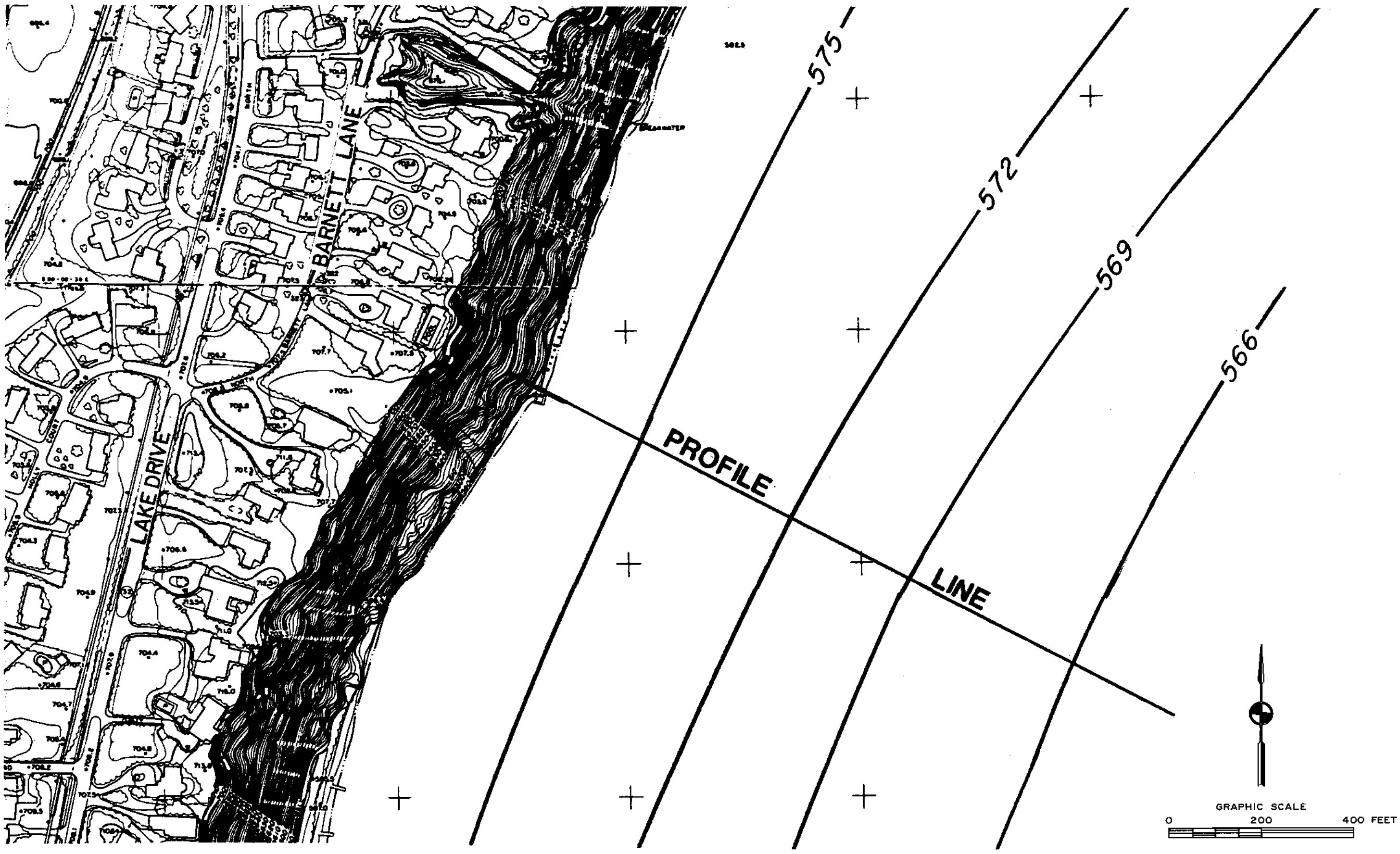
PROFILE

THE BATHYMETRIC INFORMATION PRESENTED ON THESE PROFILES WAS OBTAINED BY WARZYN ENGINEERING, INC., ON AUGUST 12 THROUGH 19, 1986. THE ELEVATIONS ARE REFERENCED TO NATIONAL GEODETIC VERTICAL DATUM (1929). THE BATHYMETRIC INFORMATION MAY BE EXPECTED TO VARY OVER TIME DUE TO NORMAL EROSION AND ACCRETION OF LAKE BOTTOM SEDIMENTS.

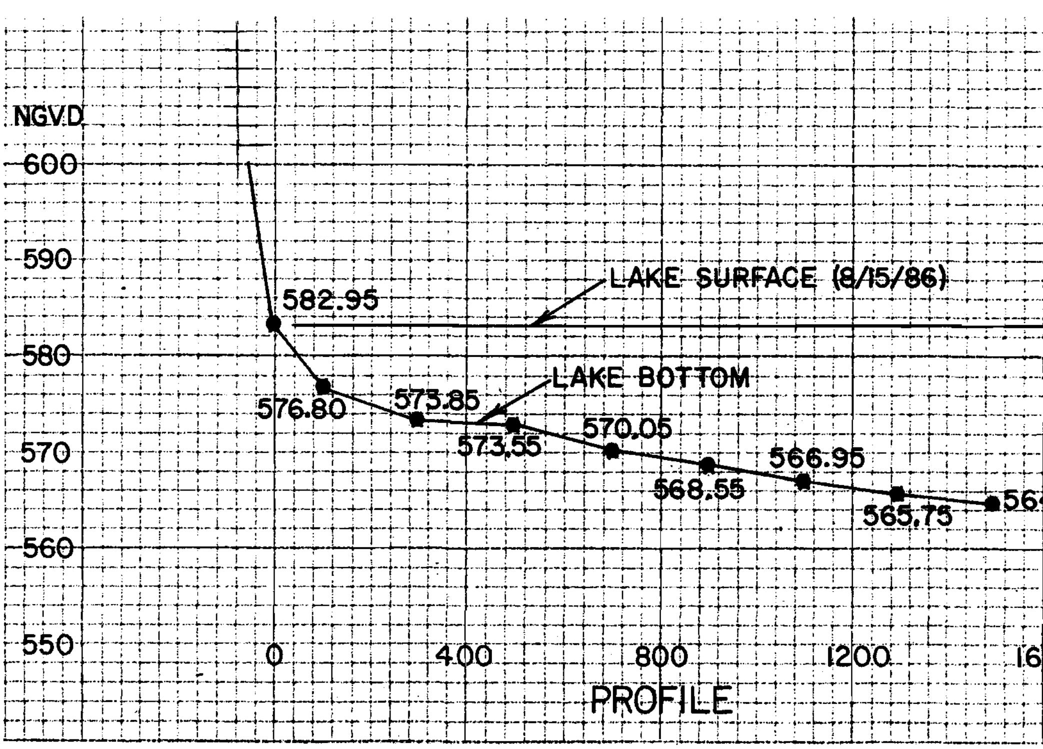
Source: Warzyn Engineering, Inc.

Figure A-11

BATHYMETRIC SURVEY, LOCATION 11



PLAN

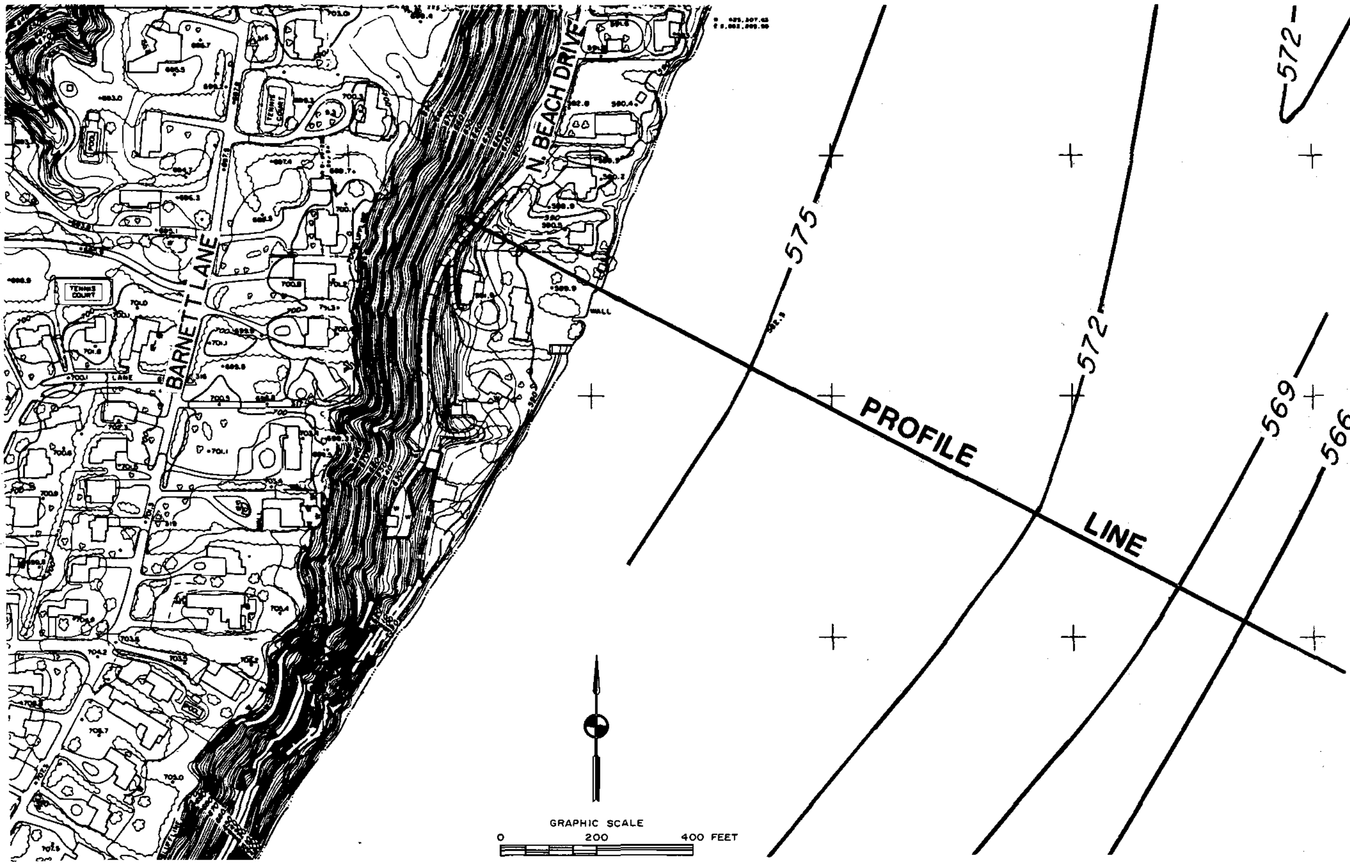


THE BATHYMETRIC INFORMATION PRESENTED ON THESE PROFILES WAS OBTAINED BY WARZYN ENGINEERING, INC., ON AUGUST 12 THROUGH 19, 1986. THE ELEVATIONS ARE REFERENCED TO NATIONAL GEODETIC VERTICAL DATUM (1929). THE BATHYMETRIC INFORMATION MAY BE EXPECTED TO VARY OVER TIME DUE TO NORMAL EROSION AND ACCRETION OF LAKE BOTTOM SEDIMENTS.

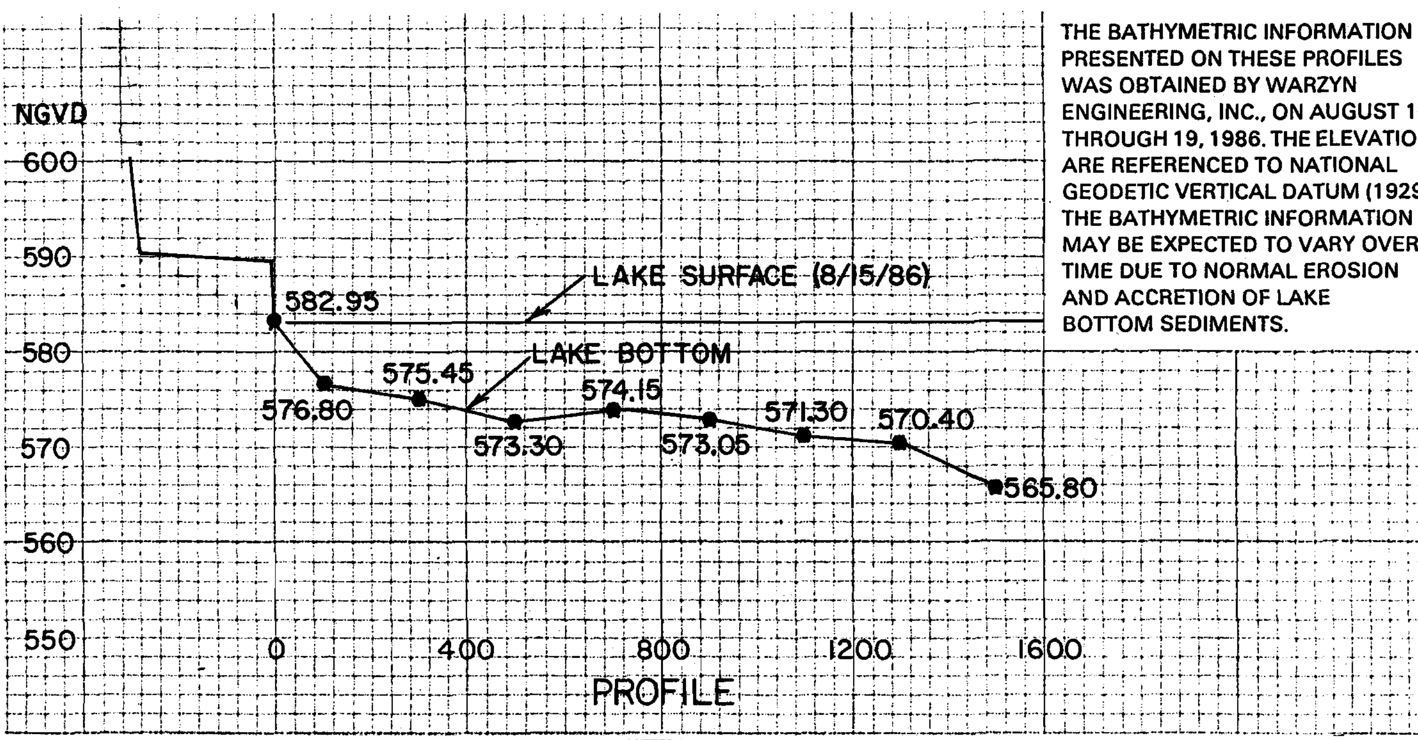
Source: Warzyn Engineering, Inc.

Figure A-12

BATHYMETRIC SURVEY, LOCATION 12



PLAN



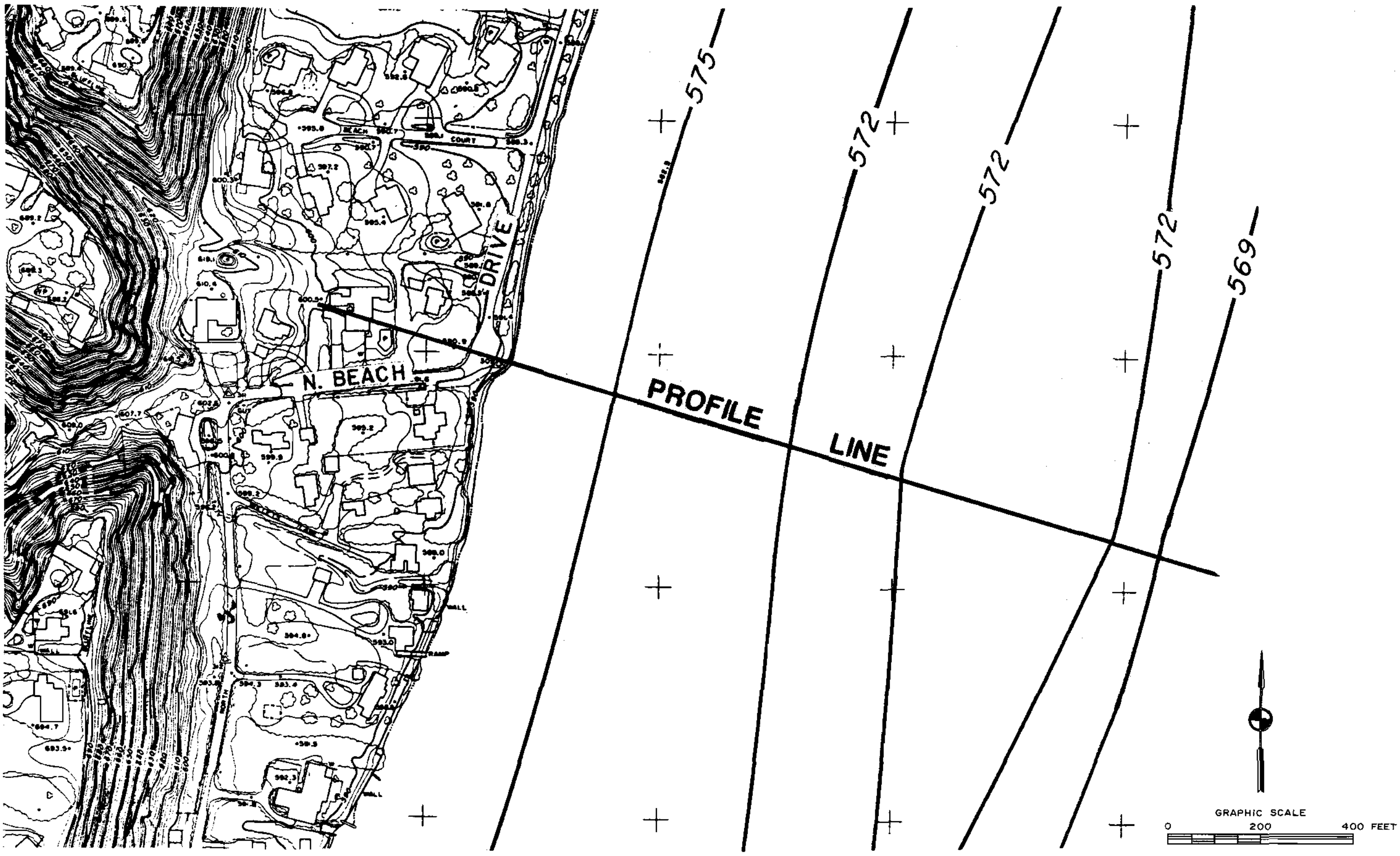
PROFILE

THE BATHYMETRIC INFORMATION PRESENTED ON THESE PROFILES WAS OBTAINED BY WARZYN ENGINEERING, INC., ON AUGUST 12 THROUGH 19, 1986. THE ELEVATIONS ARE REFERENCED TO NATIONAL GEODETIC VERTICAL DATUM (1929). THE BATHYMETRIC INFORMATION MAY BE EXPECTED TO VARY OVER TIME DUE TO NORMAL EROSION AND ACCRETION OF LAKE BOTTOM SEDIMENTS.

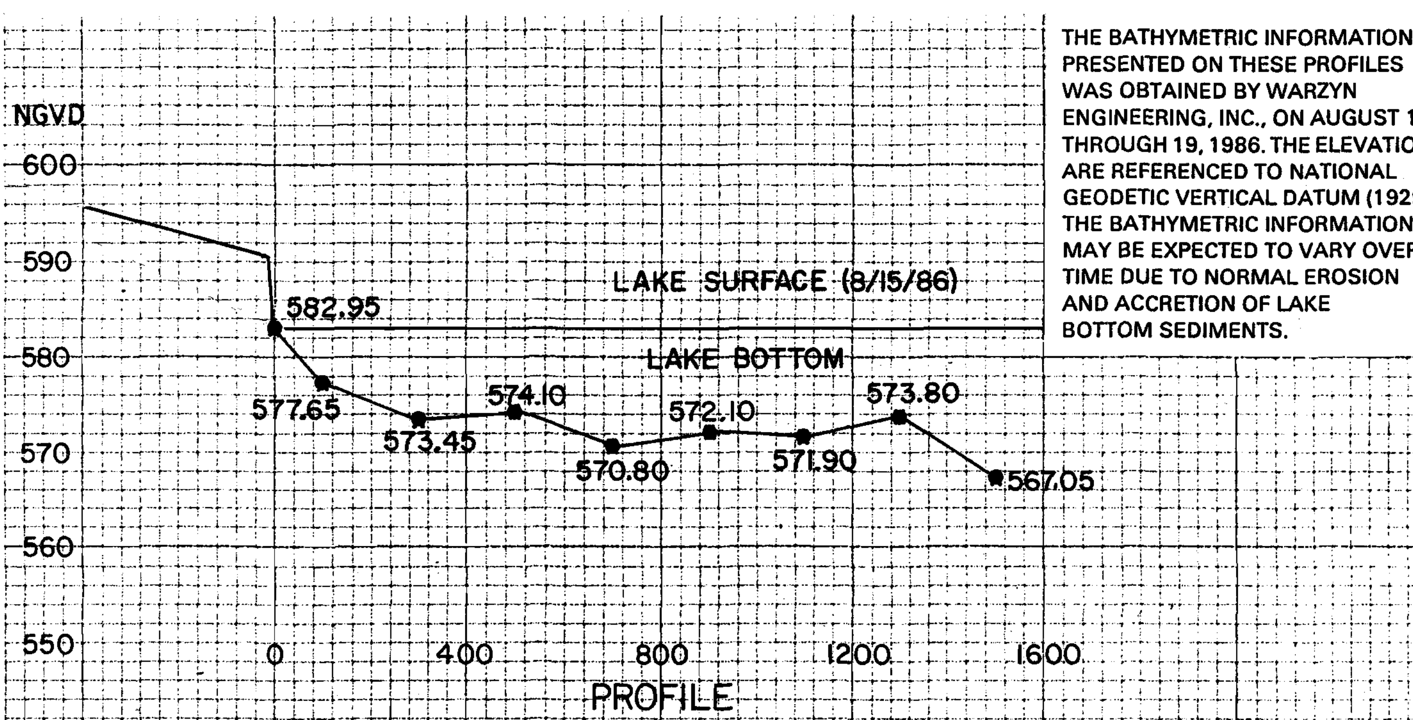
Source: Warzyn Engineering, Inc.

Figure A-13

BATHYMETRIC SURVEY, LOCATION 13



PLAN



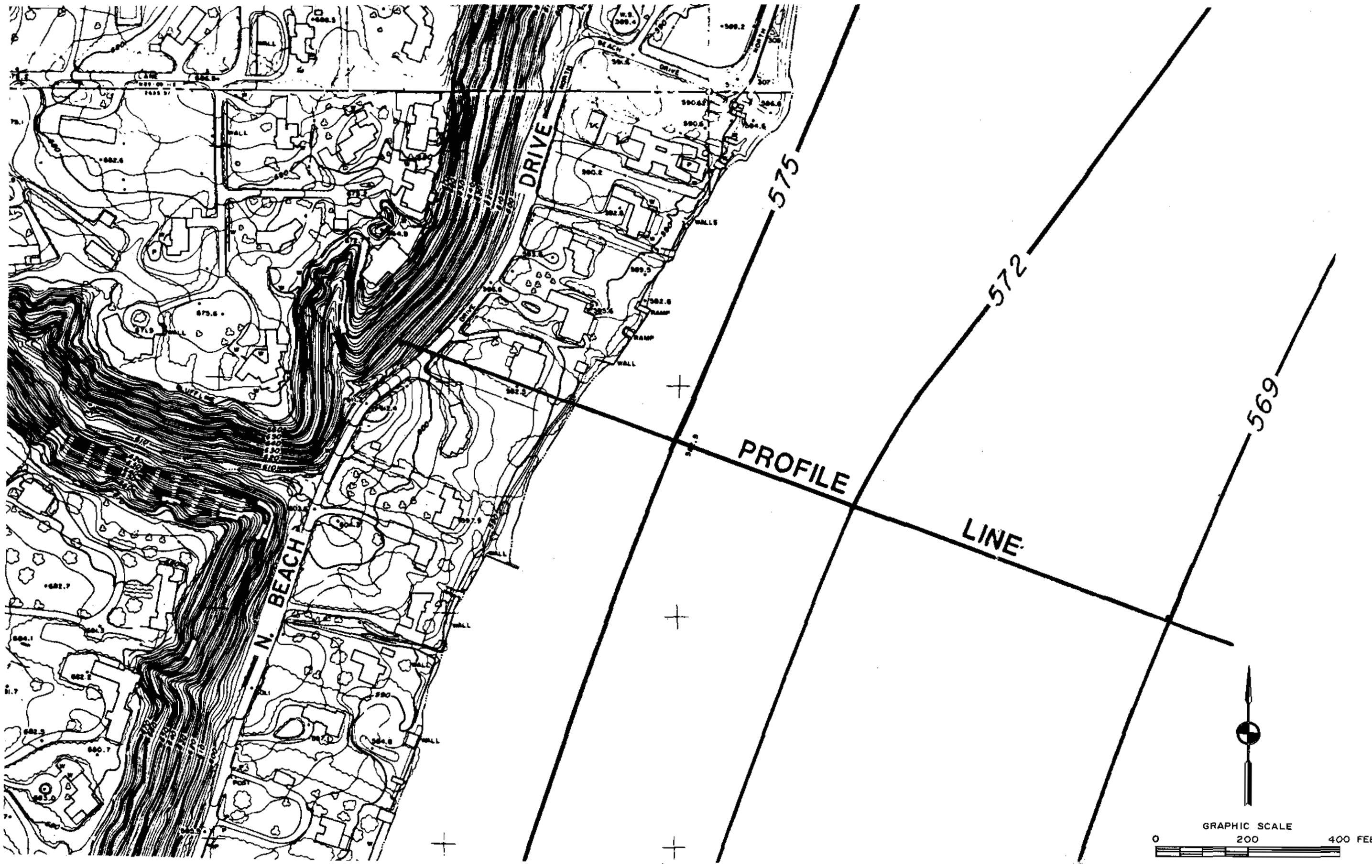
PROFILE

THE BATHYMETRIC INFORMATION PRESENTED ON THESE PROFILES WAS OBTAINED BY WARZYN ENGINEERING, INC., ON AUGUST 12 THROUGH 19, 1986. THE ELEVATIONS ARE REFERENCED TO NATIONAL GEODETIC VERTICAL DATUM (1929). THE BATHYMETRIC INFORMATION MAY BE EXPECTED TO VARY OVER TIME DUE TO NORMAL EROSION AND ACCRETION OF LAKE BOTTOM SEDIMENTS.

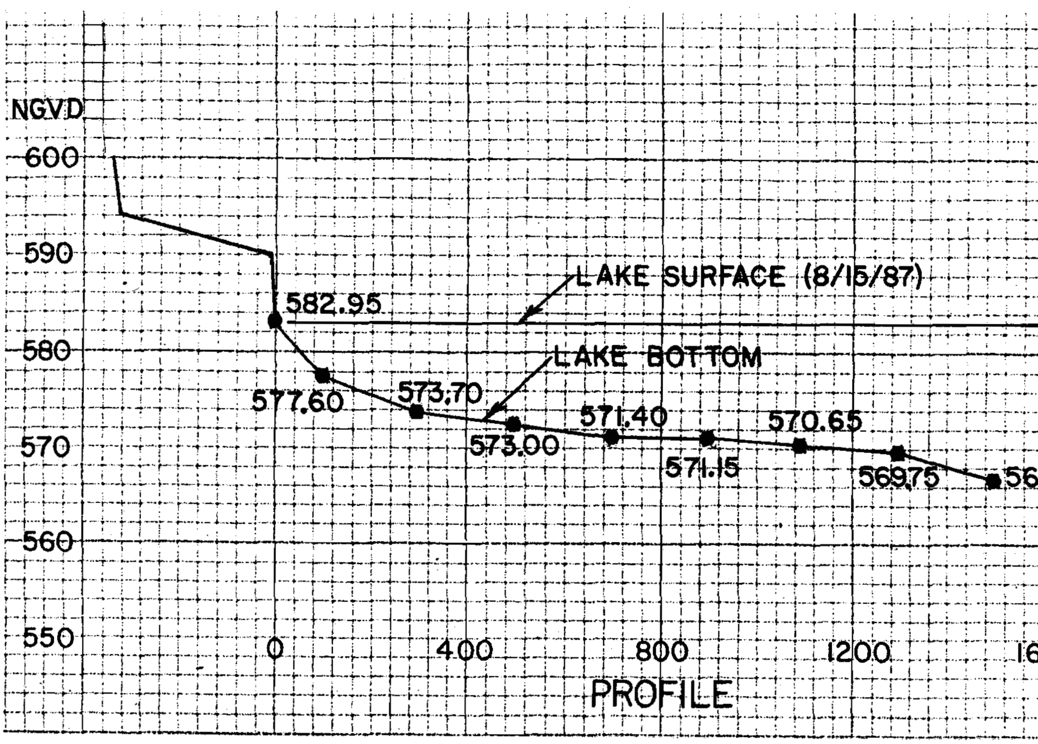
Source: Warzyn Engineering, Inc.

Figure A-14

BATHYMETRIC SURVEY, LOCATION 14



PLAN

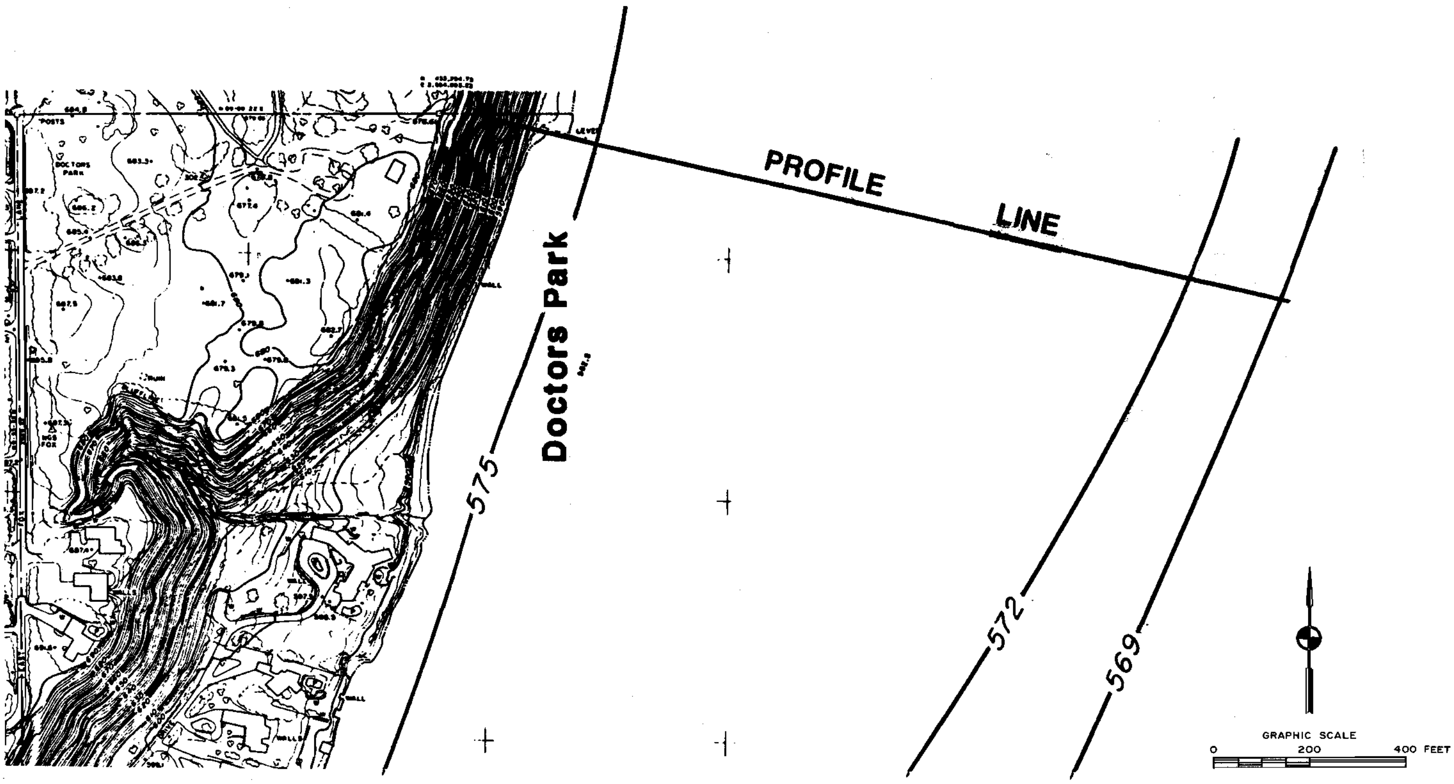


THE BATHYMETRIC INFORMATION PRESENTED ON THESE PROFILES WAS OBTAINED BY WARZYN ENGINEERING, INC., ON AUGUST 12 THROUGH 19, 1986. THE ELEVATIONS ARE REFERENCED TO NATIONAL GEODETIC VERTICAL DATUM (1929). THE BATHYMETRIC INFORMATION MAY BE EXPECTED TO VARY OVER TIME DUE TO NORMAL EROSION AND ACCRETION OF LAKE BOTTOM SEDIMENTS.

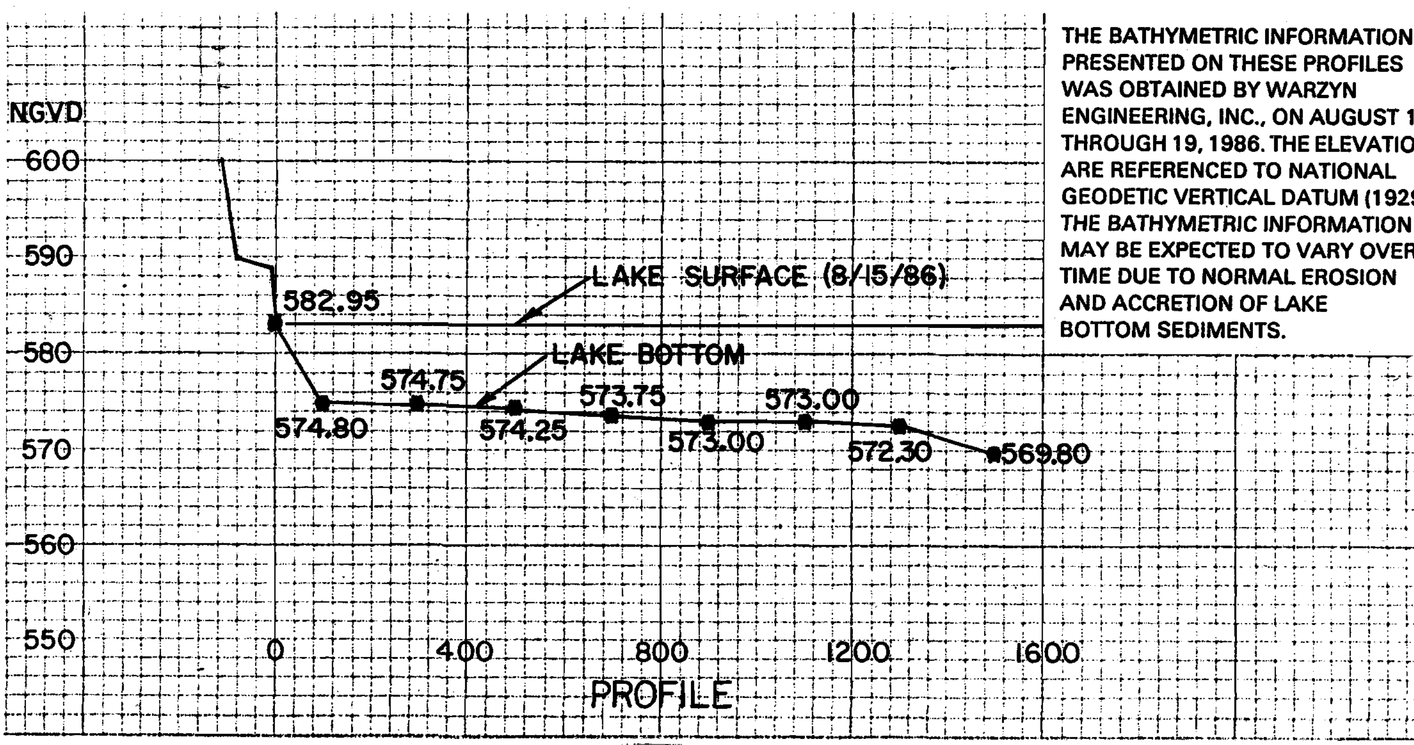
Source: Warzyn Engineering, Inc.

Figure A-15

BATHYMETRIC SURVEY, LOCATION 15



PLAN



PROFILE

THE BATHYMETRIC INFORMATION PRESENTED ON THESE PROFILES WAS OBTAINED BY WARZYN ENGINEERING, INC., ON AUGUST 12 THROUGH 19, 1986. THE ELEVATIONS ARE REFERENCED TO NATIONAL GEODETIC VERTICAL DATUM (1929). THE BATHYMETRIC INFORMATION MAY BE EXPECTED TO VARY OVER TIME DUE TO NORMAL EROSION AND ACCRETION OF LAKE BOTTOM SEDIMENTS.

Source: Warzyn Engineering, Inc.

Appendix B

INVENTORY OF SHORE PROTECTION STRUCTURES IN NORTHERN MILWAUKEE COUNTY: 1986

Structure Number	Address	U. S. Public Land Survey Location			Structure Type	Physical Setting				Length of Structure (feet)	Material Composition of Structure	Maintenance Required	Types of Failure	Date of Construction
		Township	Range	Section		Bluff Height (feet)	Bluff Slope (degrees)	Vegetation[a]	Beach Width (feet)					
1	3224 E. Hampshire Street	7	22	10	Bulkhead	74	17	C	60	370	Poured concrete	Yes	Material failure	pre-1945
2	3252 N. Lake Drive	7	22	10	Revetment	78	20	C	<5	160	Stone	Yes	Overtopping, flanking, collapse	
3	3318-3322 N. Lake Drive	7	22	10	Bulkhead	82	16	C	<5	200	Poured concrete	Yes	Overtopping, flanking	pre-1945
4	3318-3322 N. Lake Drive	7	22	10	Groin	82	16	C	<5	85	Concrete	Yes	Overtopping	1929
5	3063 E. Newport Court	7	22	10	Bulkhead	88	18	C	10	350	Precast concrete	Yes	Overtopping, flanking, collapse	pre-1945
6	3432 N. Lake Drive	7	22	10	Bulkhead	100	15	C	<5	75	Concrete	Yes	Collapse	pre-1945
7	3444 N. Lake Drive	7	22	10	Bulkhead	102	26	C	<5	50	Concrete	Yes	Collapse	pre-1945
8	3474 N. Lake Drive	7	22	10	Bulkhead	103	16	C	<5	275	Concrete	Yes	Overtopping, flanking, collapse	pre-1945
9	3510 N. Lake Drive	7	22	10	Bulkhead	98	29	C	<5	150	Concrete	Yes	Flanking, material failure	pre-1945
10	3534 N. Lake Drive	7	22	10	Revetment	96	19	PC	<5	250	Concrete	No	- -	N/A
11	3534 N. Lake Drive	7	22	10	Breakwater	96	19	PC	<5	225	Stone	Yes	Overtopping	1929
12	3562 N. Lake Drive	7	22	10	Bulkhead	106	20	C	<5	80	Precast concrete	Yes	Overtopping, flanking, material failure	1920
13	3580 N. Lake Drive	7	22	10	Bulkhead	105	16	C	<5	85	Concrete	Yes	Collapse	1920
14	3900 N. Lake Drive	7	22	10	Bulkhead	106	19	C	15	200	Poured concrete	Yes	Collapse	1920
15	3926 N. Lake Drive	7	22	10	Revetment	100	24	C	<5	125	Precast concrete	Yes	Overtopping, collapse	1977
16	3932-3966 N. Lake Drive	7	22	10	Bulkhead	107	29	C	<5	400	Concrete/grout-filled bags	Yes	Overtopping, material failure, collapse	N/A
17	3966-4060 N. Lake Drive	7	22	3	Groin	88	26	C	100	1,150	Concrete	Yes	Collapse, overtopping, material failure	1933
18	4120-4130 N. Lake Drive	7	22	3	Groin	103	27	C	60	95	Sheetpile/concrete	Yes	Material failure	pre-1975
19	4400-4408 N. Lake Drive	7	22	3	Bulkhead	111	29	C	<5	200	Concrete/grout-filled bags	Yes	Overtopping, flanking	pre-1945
20	4442-4668 N. Lake Dive	7	22	3	Revetment	115	33	PC	<5	2,200	Stone	No	- -	1986
21	4676 N. Lake Drive	7	22	3	Bulkhead	102	25	PC	<5	100	Poured concrete/timber	Yes	Flanking	1981
22	4700 N. Lake Drive	7	22	3	Bulkhead	94	38	PC	<5	100	Poured concrete	Yes	Overtopping, flanking, material failure	1981
23	4720 N. Lake Drive	7	22	3	Revetment	97	35	NC	<5	100	Limestone/grout-filled bags	No	- -	1986
24	4762 N. Lake Drive	8	22	34	Revetment	96	33	NC	<5	100	Limestone/grout-filled bags	No	- -	1986
25	4790-4800 N. Lake Drive	8	22	34	Revetment	93	36	NC	<5	300	Stone/grout-filled bags	No	- -	1985
26	4850-4870 N. Lake Drive	8	22	34	Bulkhead	83	31	NC	<5	210	Concrete	No	- -	1986
27	4890-4940 N. Lake Drive	8	22	34	Revetment	71	31	C	<5	600	Concrete/grout-filled bags	Yes	Overtopping, collapse	1976
28	Buckley Park	8	22	34	Bulkhead	66	31	C	<5	350	Concrete	Yes	Overtopping, flanking, material failure	1946
29	Big Bay Park	8	22	33	Groin	73	19	C	40	400	Precast concrete	Yes	Overtopping	1956
30	Big Bay Park	8	22	33	Bulkhead	64	26	C	<5	350	Precast concrete	Yes	Toe scour, overtopping, material failure	1943
31	Big Bay Park	8	22	33	Bulkhead	70	25	C	<5	750	Precast concrete	Yes	Overtopping, material failure	1954
32	1400-1500 E. Henry Clay	8	22	33	Revetment	76	29	PC	<5	655	Concrete	No	- -	1982
33	5220-5240 N. Lake Drive	8	22	33	Revetment	72	19	PC	<5	350	Concrete/stone	No	- -	1981
34	5270 N. Lake Drive	8	22	33	Bulkhead	68	17	C	<5	220	Concrete	Yes	Toe scour, flanking, collapse	pre-1975
35	5312-5570 N. Lake Drive	8	22	33	Revetment	82	19	C	<5	1,700	Concrete/stone	Yes	Collapse	1978
36	5570-5616 N. Lake Drive	8	22	28	Revetment	82	22	NC	<5	370	Concrete/stone	No	- -	1981
37	5626 N. Lake Drive-808 E. Lakeview Drive	8	22	28	Revetment	88	17	NC	10	840	Concrete	No	- -	1986
38	5866 N. Lake Drive	8	22	28	Bulkhead	84	29	C	15	50	Precast concrete	Yes	Overtopping, flanking	1943
39	Klode Park	8	22	28	Bulkhead	74	20	C	20	480	Precast concrete	Yes	Overtopping, flanking	1943
40	6430-6448 N. Lake Drive	8	22	21	Revetment	130	28	NC	<5	240	Concrete	No	- -	1986
41	6530 N. Lake Drive	8	22	21	Revetment	130	31	C	<5	400	Stone	Yes	Overtopping	1972
42	6530-6620 N. Lake Drive	8	22	21	Groin	126	32	C	30	425	Precast concrete	Yes	Material failure	1972
43	6720-6818 N. Barnett Lane	8	22	21	Revetment	120	30	PC	<5	700	Grout-filled bags	No	- -	1986
44	6880 N. Barnett Lane	8	22	21	Groin	108	26	C	10	35	Precast concrete	Yes	Material failure	pre-1975

Appendix B (continued)

Structure Number	Address	U. S. Public Land Survey Location: Township	Range	Section	Structure Type	Physical Setting: Bluff Height (feet)	Bluff Slope (degrees)	Vegetation[a]	Beach Width (feet)	Length of Structure (feet)	Material Composition of Structure	Maintenance Required	Types of Failure	Date of Construction
45	7000 N. Beach Drive	8	22	21	Bulkhead	- -	- -	- -	<5	80	Stone	No	- -	1986
46	7000 N. Beach Drive	8	22	21	Groin	- -	- -	- -	<5	50	Concrete blocks	Yes	Overtopping	N/A
47	7000 N. Beach Drive	8	22	21	Revetment	- -	- -	- -	<5	25	Stone	Yes	Overtopping	1986
48	7038 N. Beach Drive	8	22	21	Bulkhead	- -	- -	- -	<5	160	Concrete	Yes	Overtopping	1982
49	7106 N. Beach Drive	8	22	21	Bulkhead	- -	- -	- -	<5	220	Poured concrete	Yes	Overtopping, material failure	1976
50	7120 N. Beach Drive	8	22	21	Revetment	- -	- -	- -	<5	90	Concrete slabs	No	- -	1986
51	7124 N. Beach Drive	8	22	21	Bulkhead	- -	- -	- -	<5	125	Concrete blocks	No	- -	1986
52	7134 N. Beach Drive	8	22	21	Revetment	- -	- -	- -	<5	100	Concrete slabs and blocks	Yes	Overtopping	pre-1975
53	7152-7200 N. Beach Drive	8	22	21	Bulkhead	- -	- -	- -	<5	270	Concrete blocks, stone	Yes	Overtopping, flanking, collapse	1974
54	7210 N. Beach Drive	8	22	16	Bulkhead	- -	- -	- -	<5	170	Rubble-filled steel crib	Yes	Overtopping	1973
55	7210 N. Beach Drive	8	22	16	Groin	- -	- -	- -	<5	175	Poured concrete/steel	Yes	Overtopping, flanking	N/A
56	7228 N. Beach Drive	8	22	16	Bulkhead	- -	- -	- -	<5	230	Poured concrete	Yes	Material failure, toe scour, overtopping	1975
57	7234-7240 N. Beach Drive	8	22	16	Revetment	- -	- -	- -	<5	165	Stone	No	- -	pre-1975
58	7242-7250 N. Beach Drive	8	22	16	Revetment	- -	- -	- -	<5	200	Concrete	Yes	Overtopping, collapse	pre-1975
59	7254-7328 N. Beach Drive	8	22	16	Bulkhead	- -	- -	- -	<5	200	Concrete block	Yes	Overtopping	1985
60	North of 7328 N. Beach Drive	8	22	16	Bulkhead	- -	- -	- -	<5	65	Concrete slabs, cut stone	Yes	Overtopping, flanking	1974
61	7400-7535 N. Beach Drive	8	22	16	Revetment	- -	- -	- -	<5	1,450	Concrete block and slabs	Yes	Overtopping	1950
62	7540-7710 N. Beach Drive	8	22	16	Groin	- -	- -	- -	<5	560	Concrete	Yes	Overtopping	1945
63	7718-7736 N. Beach Drive	8	22	16	Bulkhead	- -	- -	- -	<5	800	Timber	Yes	Overtopping, material failure	N/A
64	7810 N. Beach Drive	8	22	16	Bulkhead	- -	- -	- -	<5	160	Timber/concrete	Yes	Toe scour, overtopping, material failure	1972
65	7818-7834 N. Beach Drive	8	22	16	Bulkhead	- -	- -	- -	10	400	Stone slabs cemented	No	- -	1982
66	7900-7912 N. Beach Drive	8	22	16	Groin	- -	- -	- -	30	240	Concrete	Yes	Overtopping, material failure	N/A
67	7930 N. Beach Drive	8	22	16	Revetment	- -	- -	- -	<5	175	Concrete slabs/stone	Yes	Overtopping	1971
68	7938 N. Beach Drive	8	22	16	Bulkhead	- -	- -	- -	<5	150	Concrete	Yes	Overtopping	1970
69	7944 N. Beach Drive	8	22	16	Bulkhead	- -	- -	- -	<5	110	Concrete	No	- -	N/A
70	7954 N. Beach Drive	8	22	16	Bulkhead	- -	- -	- -	<5	145	Stone slabs	Yes	Overtopping	1980
71	7966-8035 N. Beach Drive	8	22	16	Revetment	- -	- -	- -	<5	720	Cut stone, rubble	Yes	Overtopping, collapse, material failure	N/A
72	North of 8035 N. Beach Drive	8	22	10	Bulkhead	- -	- -	- -	<5	105	Concrete slabs, stone	Yes	Flanking, collapse	1941
73	8064 N. Beach Drive	8	22	10	Bulkhead	- -	- -	- -	<5	190	Concrete	Yes	Toe scour, overtopping, flanking	N/A
74	8030 N. Beach Drive	8	22	10	Bulkhead	- -	- -	- -	<5	110	Stone slabs	No	- -	1973
75	8090 N. Beach Drive	8	22	10	Bulkhead	- -	- -	- -	<5	130	Concrete	Yes	Overtopping	1972
76	8100-8120 N. Beach Drive	8	22	10	Bulkhead	- -	- -	- -	<5	250	Concrete	Yes	Overtopping	1975-1985
77	8120 N. Beach Drive	8	22	10	Revetment	- -	- -	- -	<5	90	Cut stone	Yes	Overtopping	1986
78	8130 N. Beach Drive	8	22	10	Revetment	- -	- -	- -	<5	350	Cut stone	No	- -	1986
79	Doctors Park	8	22	10	Bulkhead	92	25	C	<5	570	Concrete	Yes	Overtopping, flanking, material failure	pre-1975
80	Doctors Park	8	22	10	Groin	92	25	C	20	40	Concrete	Yes	Overtopping	1964

NOTE: N/A indicates data not available.

[a]*C - Bluff face covered with vegetation.*
PC - Bluff face partly covered with vegetation.
NC - Bluff face not covered with vegetation.

Source: SEWRPC.

Appendix C

AERIAL PHOTOGRAPHS OF THE LAKE MICHIGAN SHORELINE OF NORTHERN MILWAUKEE COUNTY: MAY 1986

Figure C-1

LINNWOOD AVENUE WATER TREATMENT PLANT TO 3224 E. HAMPSHIRE STREET
CITY OF MILWAUKEE

Source: SEWRPC.

Figure C-2

3224 E. HAMPSHIRE STREET-3288 N. LAKE DRIVE
CITY OF MILWAUKEE

Source: SEWRPC.

Figure C-3

3252 N. LAKE DRIVE-3052 E. NEWPORT COURT
CITY OF MILWAUKEE

Source: SEWRPC.

Figure C-4

3061 E. NEWPORT COURT-3444 N. LAKE DRIVE
CITY OF MILWAUKEE

Source: SEWRPC.

Figure C-5

3432-3474 N. LAKE DRIVE
CITY OF MILWAUKEE

Source: SEWRPC.

Figure C-6

3474-3562 N. LAKE DRIVE
VILLAGE OF SHOREWOOD

Source: SEWRPC.

Figure C-7

3534-3600 N. LAKE DRIVE
VILLAGE OF SHOREWOOD

Source: SEWRPC.

Figure C-8

3580-3816 N. LAKE DRIVE
VILLAGE OF SHOREWOOD

Source: SEWRPC.

Figure C-9

3704-3914 N. LAKE DRIVE
VILLAGE OF SHOREWOOD

Source: SEWRPC.

Figure C-10

3816-3960 N. LAKE DRIVE
VILLAGE OF SHOREWOOD

Source: SEWRPC.

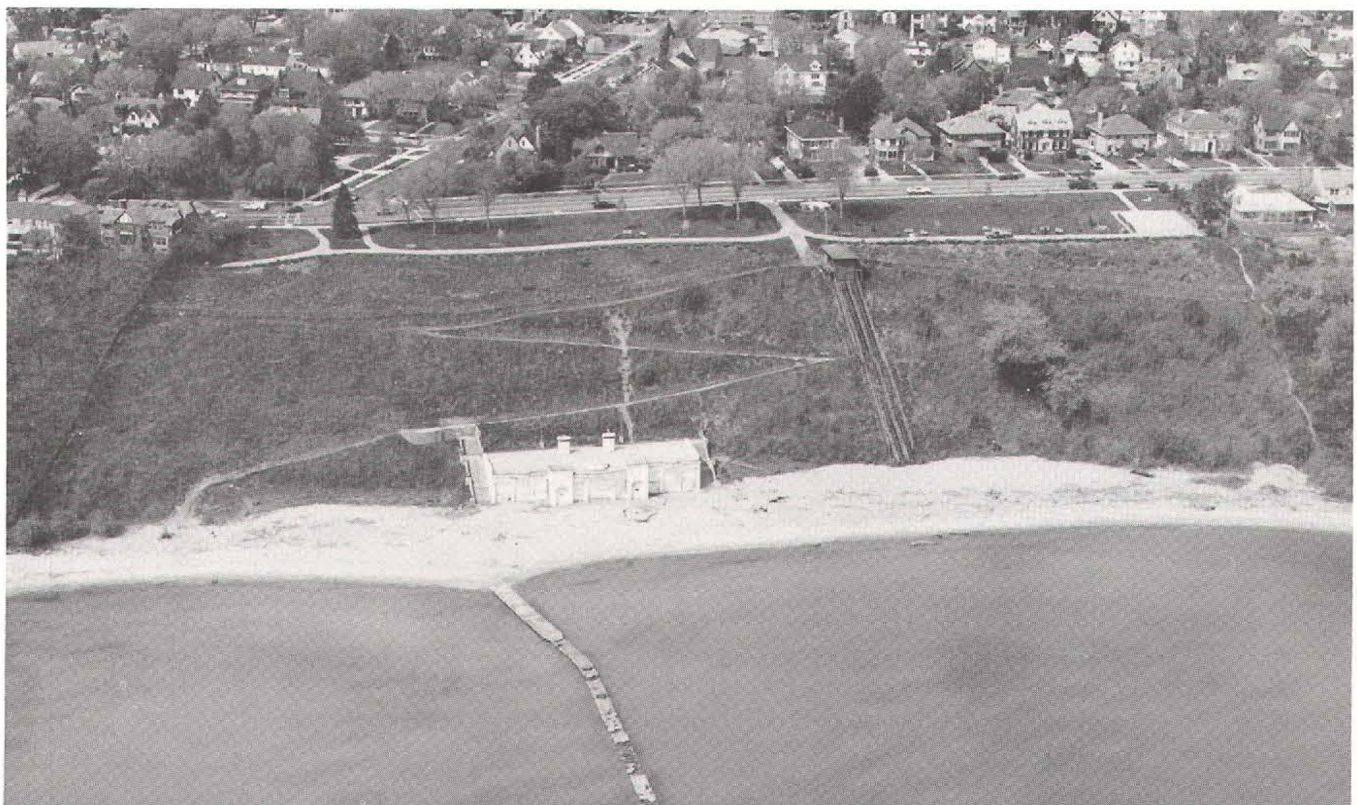
Figure C-11

3932 N. LAKE DRIVE TO ATWATER PARK
VILLAGE OF SHOREWOOD

Source: SEWRPC.

Figure C-12

ATWATER PARK
VILLAGE OF SHOREWOOD



Source: SEWRPC.

Figure C-13

4060-4136 N. LAKE DRIVE
VILLAGE OF SHOREWOOD

Source: SEWRPC.

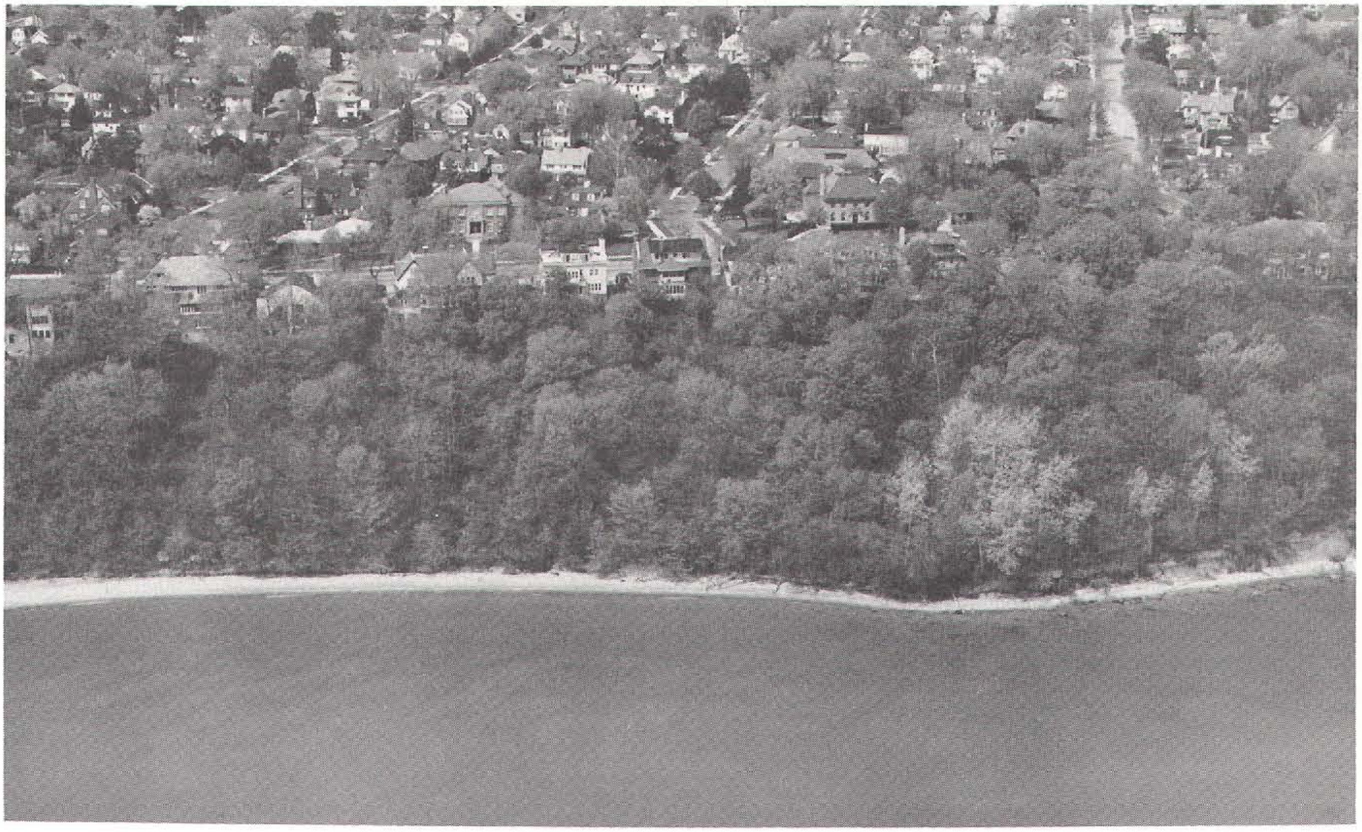
Figure C-14

4108-4216 N. LAKE DRIVE
VILLAGE OF SHOREWOOD

Source: SEWRPC.

Figure C-15

4162-4400 N. LAKE DRIVE
VILLAGE OF SHOREWOOD



Source: SEWRPC.

Figure C-16

4240-4442 N. LAKE DRIVE
VILLAGE OF SHOREWOOD

Source: SEWRPC.

Figure C-17

4408-4480 N. LAKE DRIVE
VILLAGE OF SHOREWOOD

Source: SEWRPC.

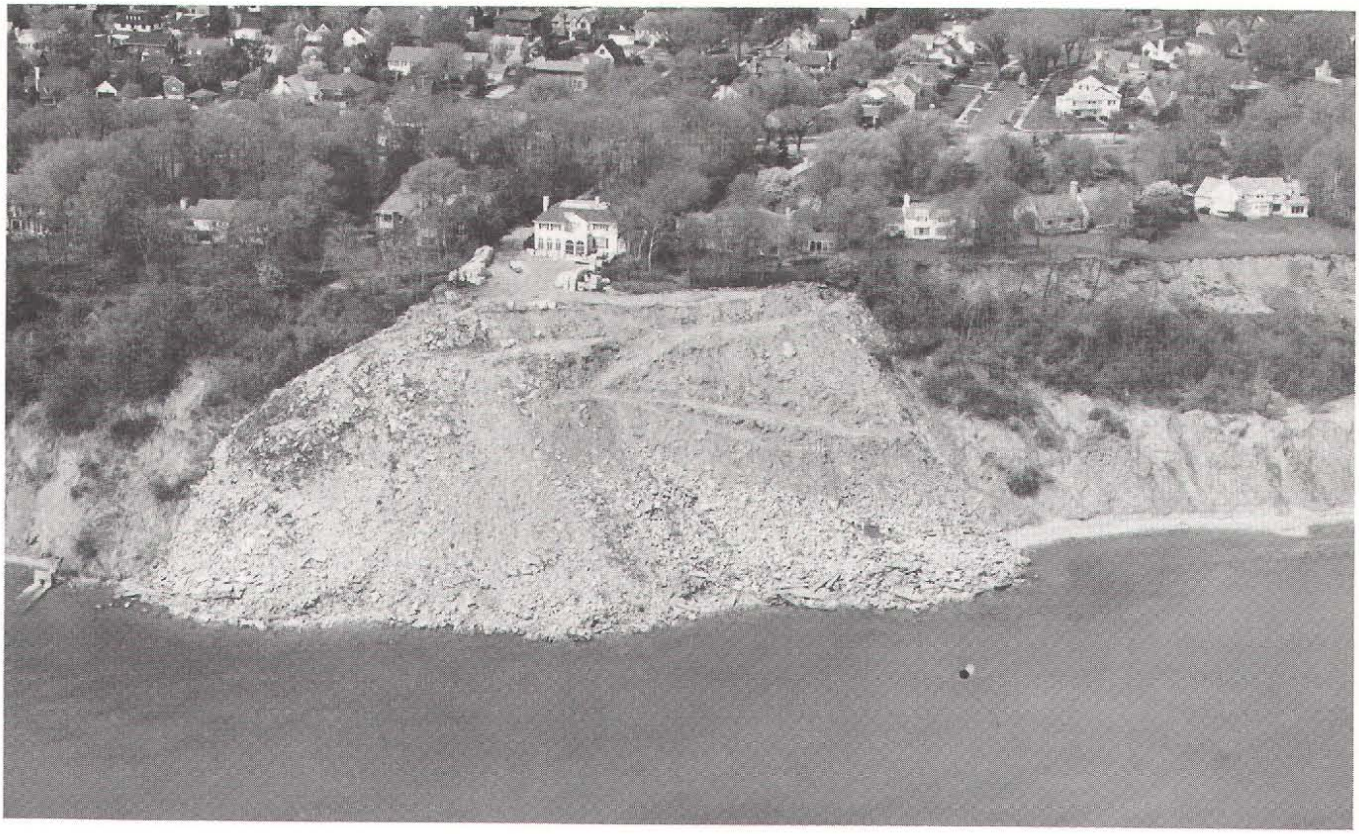
Figure C-18

4460-4614 N. LAKE DRIVE
VILLAGE OF SHOREWOOD

Source: SEWRPC.

Figure C-19

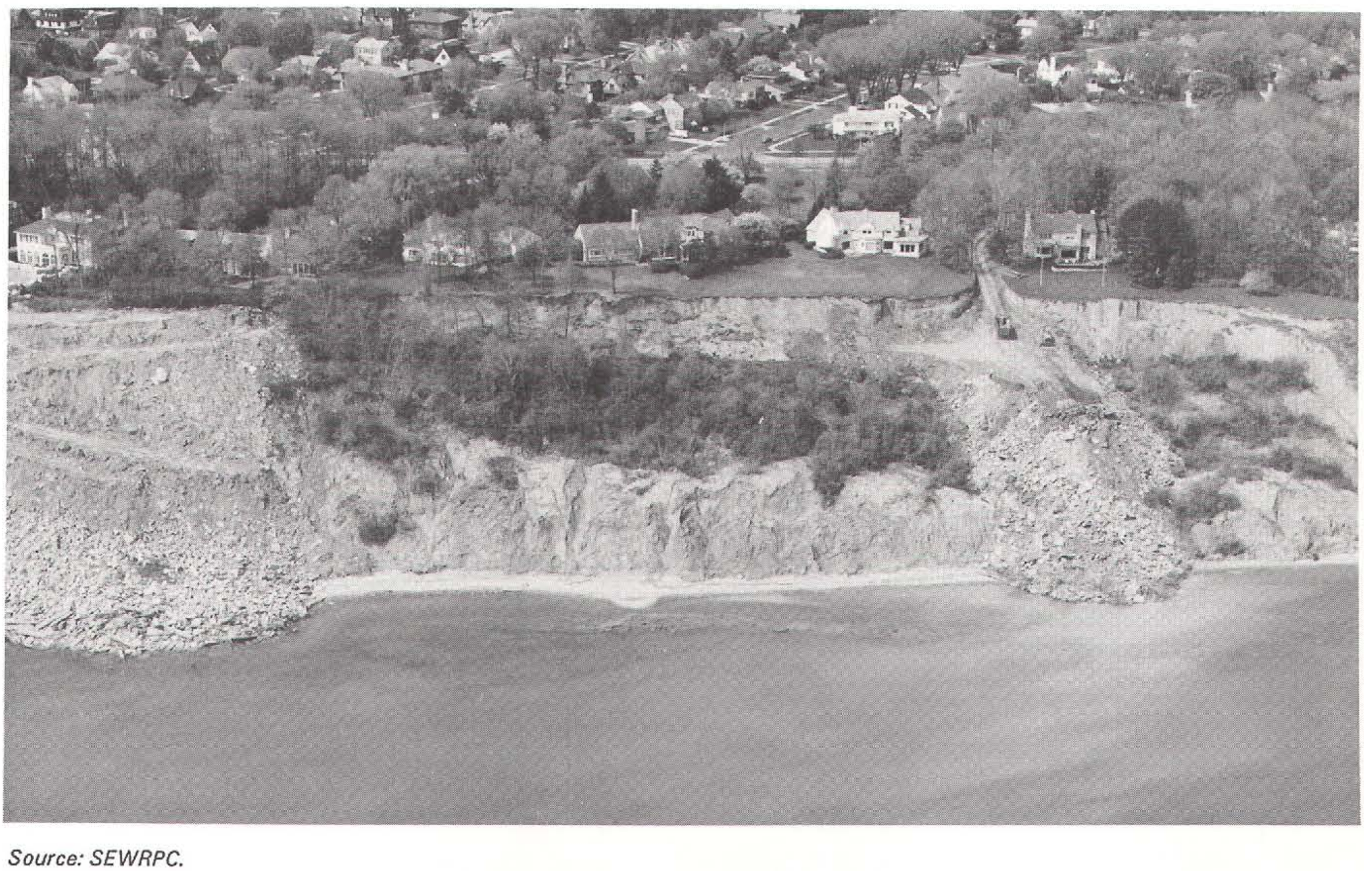
4470-4614 N. LAKE DRIVE
VILLAGE OF WHITEFISH BAY



Source: SEWRPC.

Figure C-20

4500-4620 N. LAKE DRIVE
VILLAGE OF WHITEFISH BAY



Source: SEWRPC.

Figure C-21

4514-4626 N. LAKE DRIVE
VILLAGE OF WHITEFISH BAY

Source: SEWRPC.

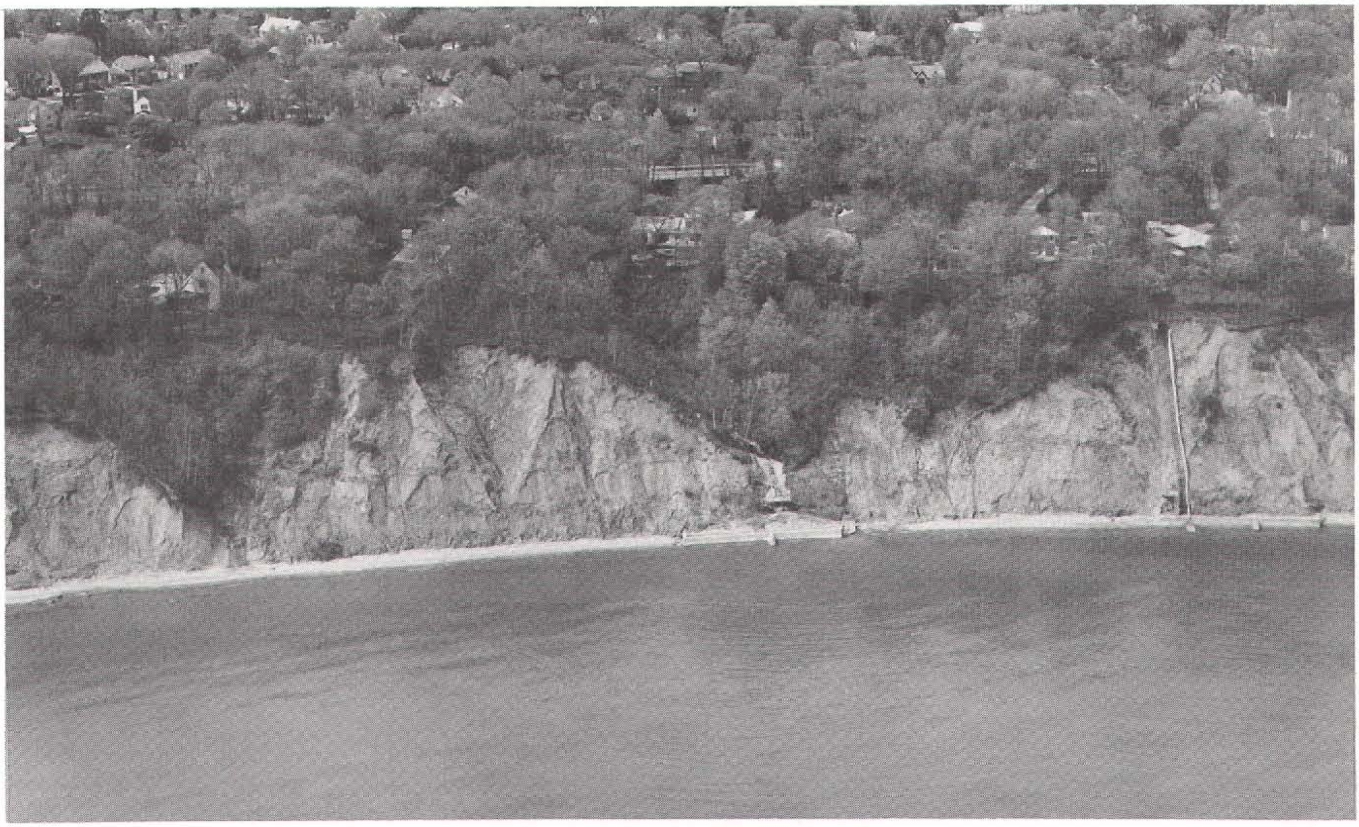
Figure C-22

4600-4652 N. LAKE DRIVE
VILLAGE OF WHITEFISH BAY

Source: SEWRPC.

Figure C-23

4632-4720 N. LAKE DRIVE
VILLAGE OF WHITEFISH BAY



Source: SEWRPC.

Figure C-24

4686-4794 N. LAKE DRIVE
VILLAGE OF WHITEFISH BAY

Source: SEWRPC.

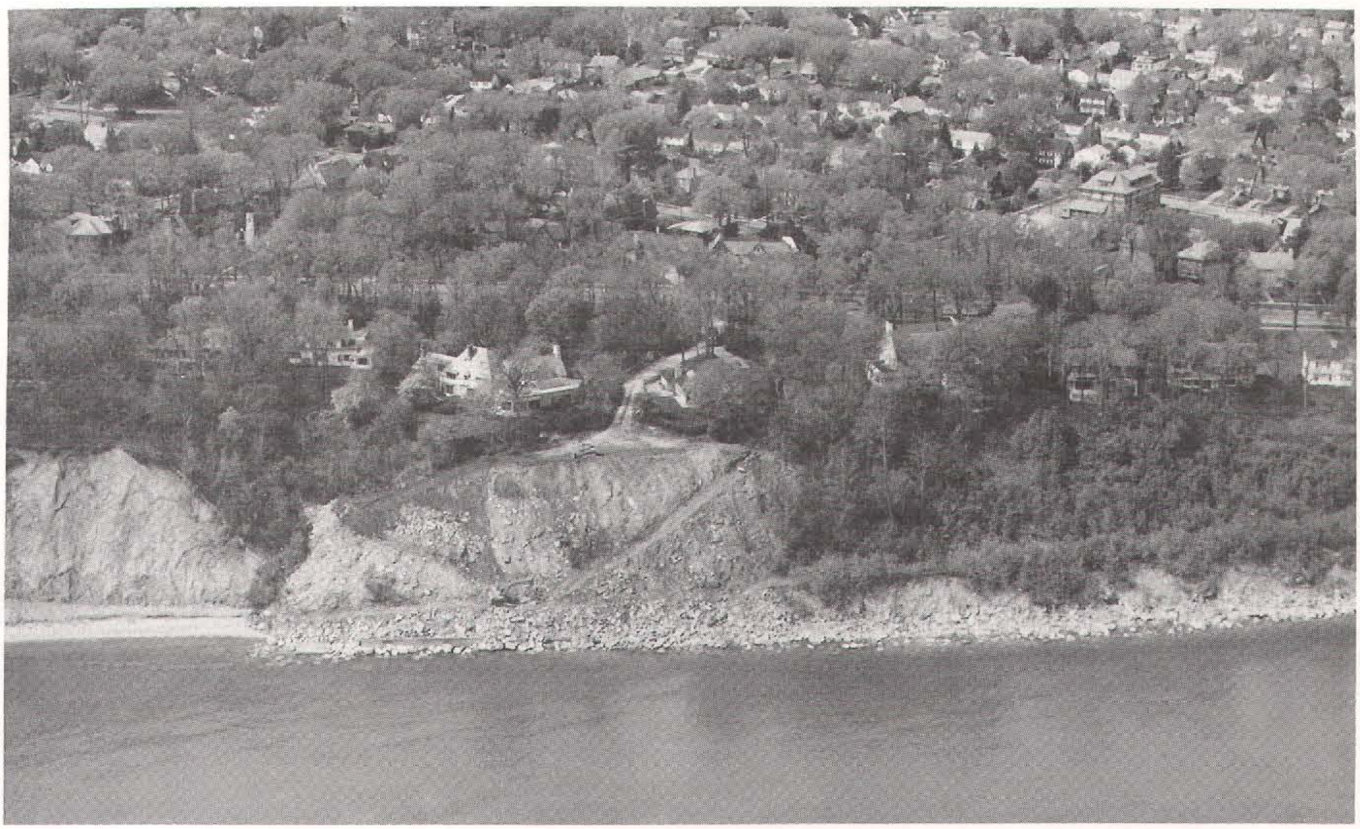
Figure C-25

4780-4850 N. LAKE DRIVE
VILLAGE OF WHITEFISH BAY

Source: SEWRPC.

Figure C-26

4820-4930 N. LAKE DRIVE
VILLAGE OF WHITEFISH BAY



Source: SEWRPC.

Figure C-27

4910 N. LAKE DRIVE TO BUCKLEY PARK
VILLAGE OF WHITEFISH BAY

Source: SEWRPC.

Figure C-28

BIG BAY PARK
VILLAGE OF WHITEFISH BAY

Source: SEWRPC.

Figure C-29

BIG BAY PARK
VILLAGE OF WHITEFISH BAY

Source: SEWRPC.

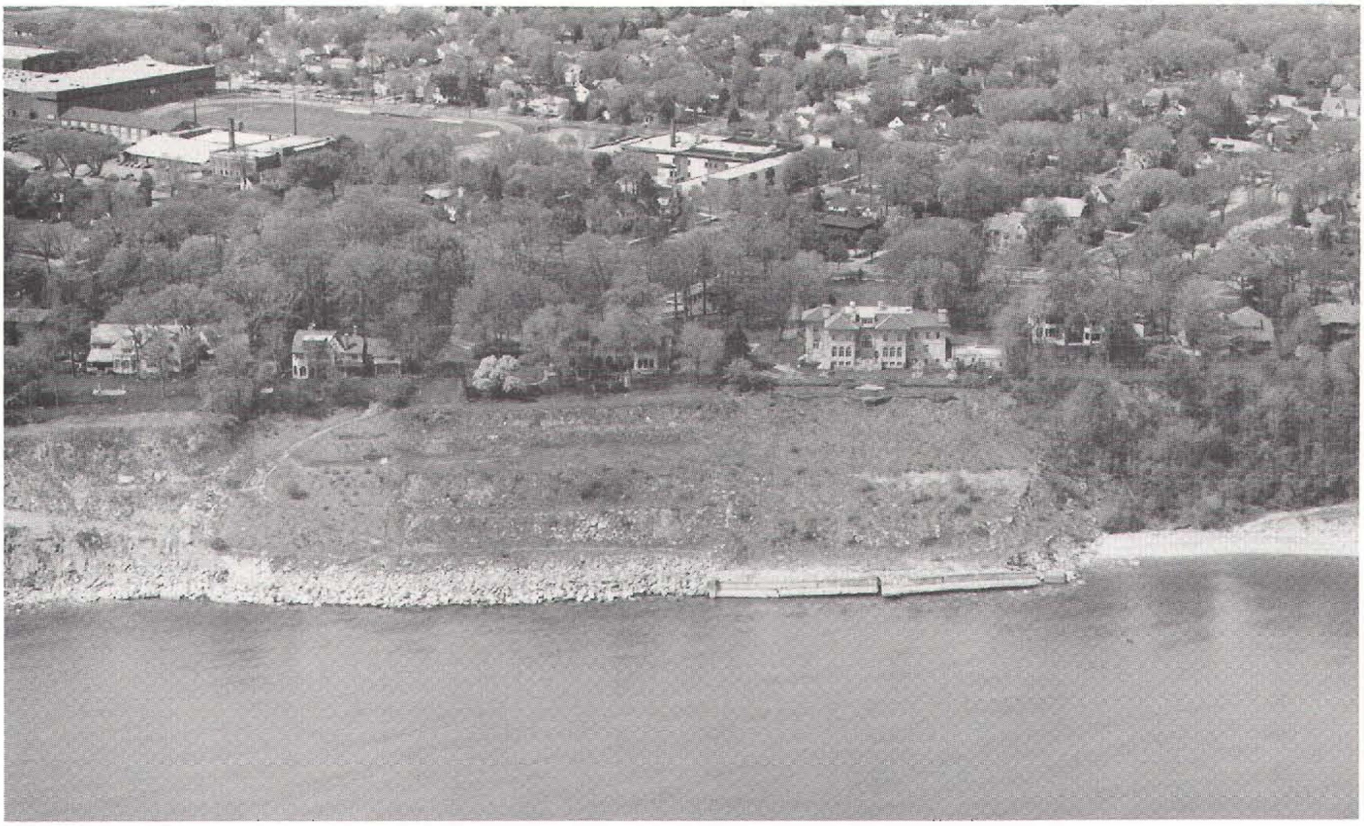
Figure C-30

BIG BAY PARK TO 5220 N. LAKE DRIVE
VILLAGE OF WHITEFISH BAY

Source: SEWRPC.

Figure C-31

1400 E. HENRY CLAY STREET-5312 N. LAKE DRIVE
VILLAGE OF WHITEFISH BAY



Source: SEWRPC.

Figure C-32

5270-5400 N. LAKE DRIVE
VILLAGE OF WHITEFISH BAY

Source: SEWRPC.

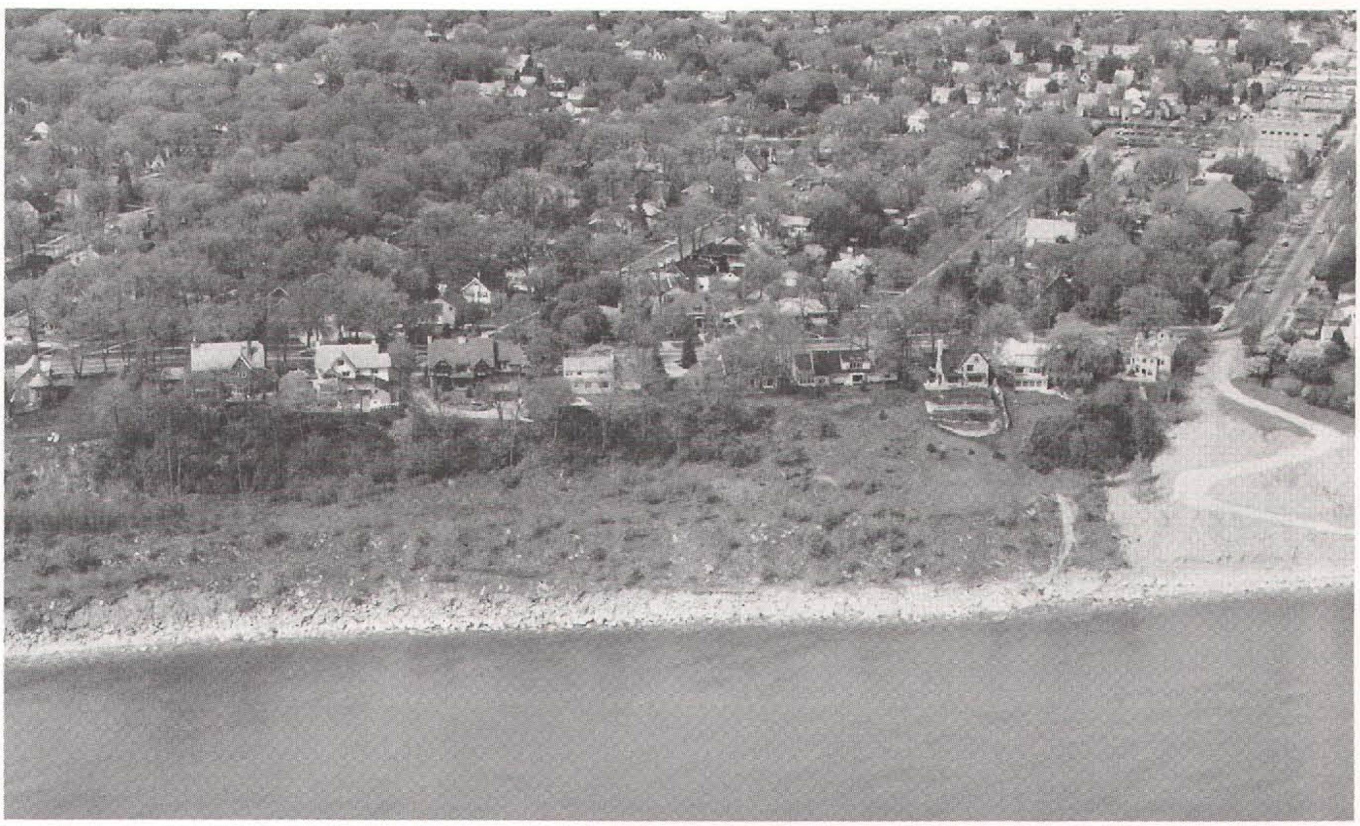
Figure C-33

5320-5486 N. LAKE DRIVE
VILLAGE OF WHITEFISH BAY

Source: SEWRPC.

Figure C-34

5436-5570 N. LAKE DRIVE
VILLAGE OF WHITEFISH BAY



Source: SEWRPC.

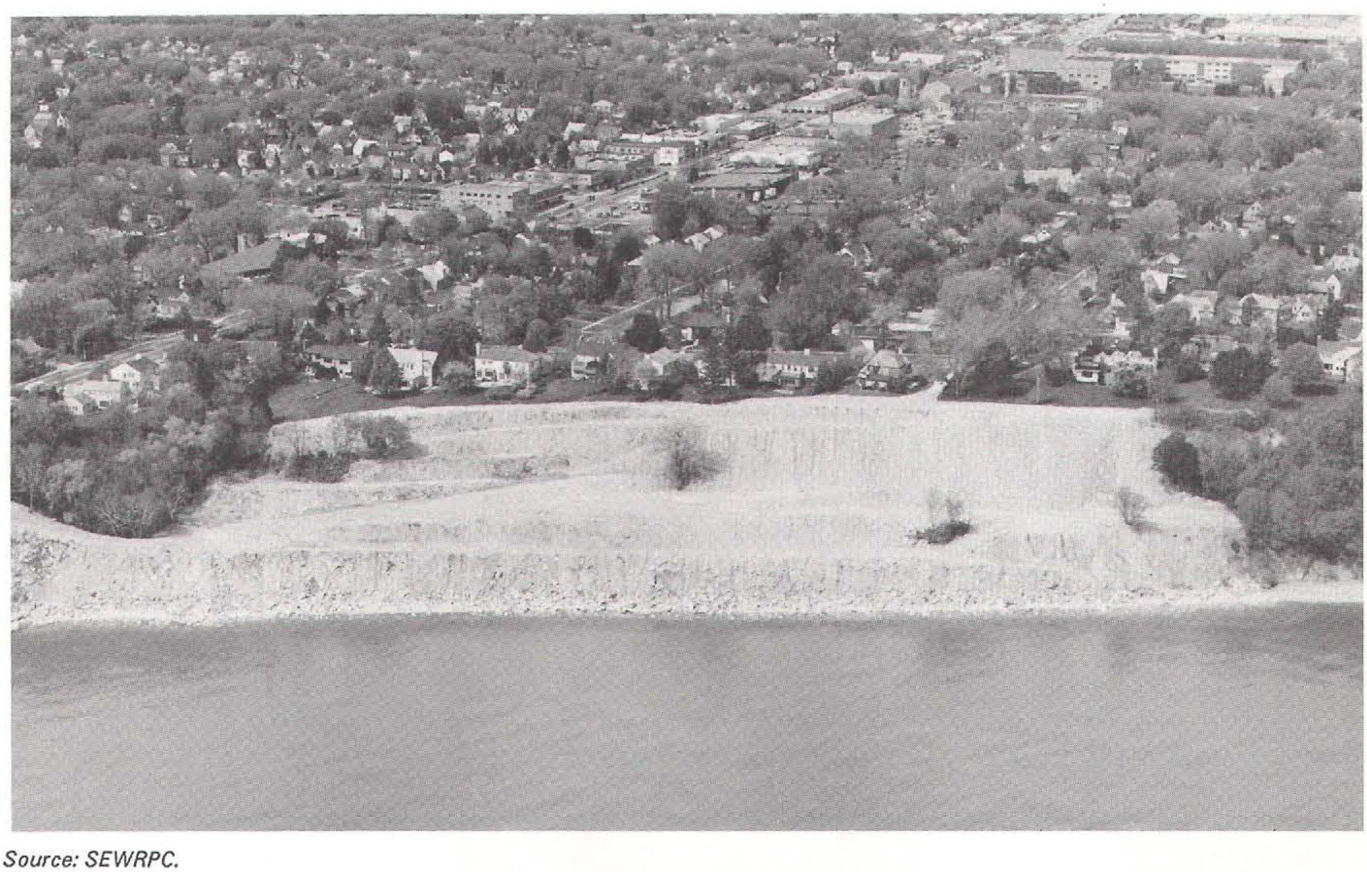
Figure C-35

5530 N. LAKE DRIVE-5684 N. SHORE DRIVE
VILLAGE OF WHITEFISH BAY

Source: SEWRPC.

Figure C-36

910 E. SILVER SPRING DRIVE-5738 N. SHORE DRIVE
VILLAGE OF WHITEFISH BAY



Source: SEWRPC.

Figure C-37

5626-5770 N. SHORE DRIVE
VILLAGE OF WHITEFISH BAY

Source: SEWRPC.

Figure C-38

5684-5842 N. SHORE DRIVE
VILLAGE OF WHITEFISH BAY

Source: SEWRPC.

Figure C-39

758 E. DAY AVENUE TO KLODE PARK
VILLAGE OF WHITEFISH BAY

Source: SEWRPC.

Figure C-40

5866-6030 N. SHORE DRIVE
VILLAGE OF WHITEFISH BAY

Source: SEWRPC.

Figure C-41

6010 N. SHORE DRIVE-601 E. LAKE TERRACE
VILLAGE OF WHITEFISH BAY

Source: SEWRPC.

Figure C-42

6110 N. LAKE DRIVE COURT-6216 N. LAKE DRIVE
VILLAGE OF WHITEFISH BAY

Source: SEWRPC.

Figure C-43

611 E. LAKE HILL COURT-6310 N. LAKE DRIVE
VILLAGE OF WHITEFISH BAY

Source: SEWRPC.

Figure C-44

6250-6400 N. LAKE DRIVE
VILLAGE OF WHITEFISH BAY

Source: SEWRPC.

Figure C-45

6350-6448 N. LAKE DRIVE
VILLAGE OF FOX POINT

Source: SEWRPC.

Figure C-46

6420-6516 N. LAKE DRIVE
VILLAGE OF FOX POINT

Source: SEWRPC.

Figure C-47

6464-6702 N. LAKE DRIVE
VILLAGE OF FOX POINT

Source: SEWRPC.

Figure C-48

6600-6730 N. LAKE DRIVE
VILLAGE OF FOX POINT

Source: SEWRPC.

Figure C-49

6702 N. LAKE DRIVE-6820 N. BARNETT LANE
VILLAGE OF FOX POINT

Source: SEWRPC.

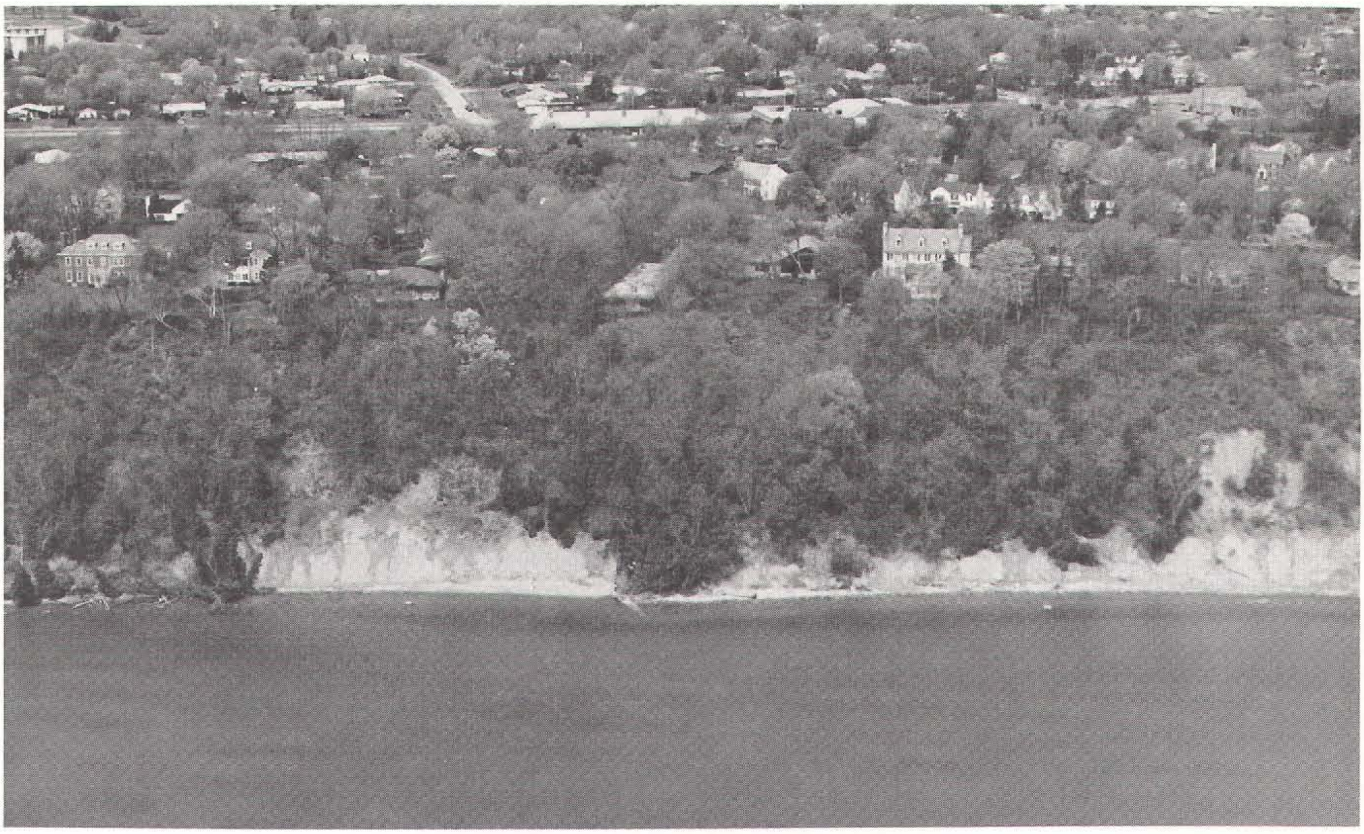
Figure C-50

6810-6880 N. BARNETT LANE
VILLAGE OF FOX POINT

Source: SEWRPC.

Figure C-51

6836-6942 N. BARNETT LANE
VILLAGE OF FOX POINT



Source: SEWRPC.

Figure C-52

6928-6990 N. BARNETT LANE
VILLAGE OF FOX POINT

Source: SEWRPC.

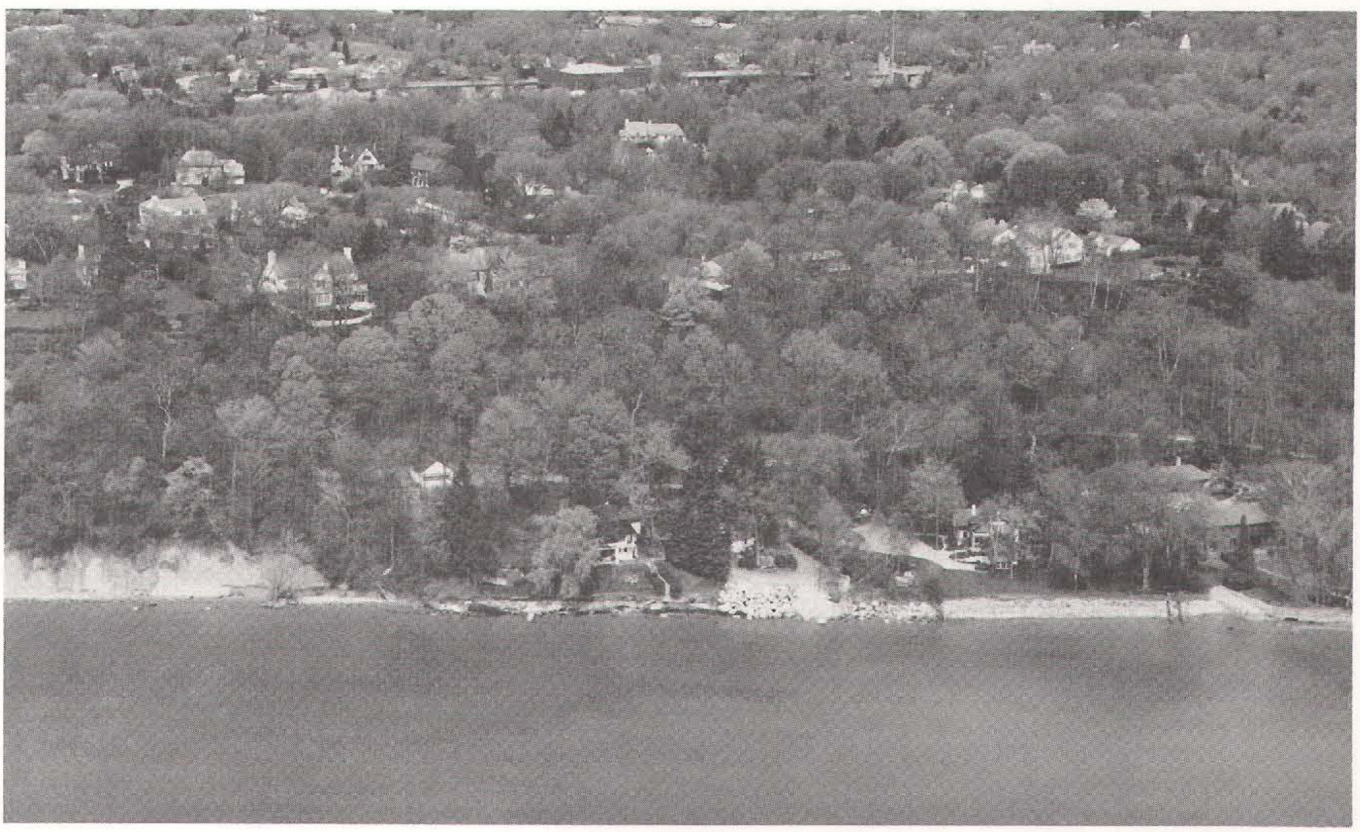
Figure C-53

6960 N. BARNETT LANE-7000 N. BEACH DRIVE
VILLAGE OF FOX POINT

Source: SEWRPC.

Figure C-54

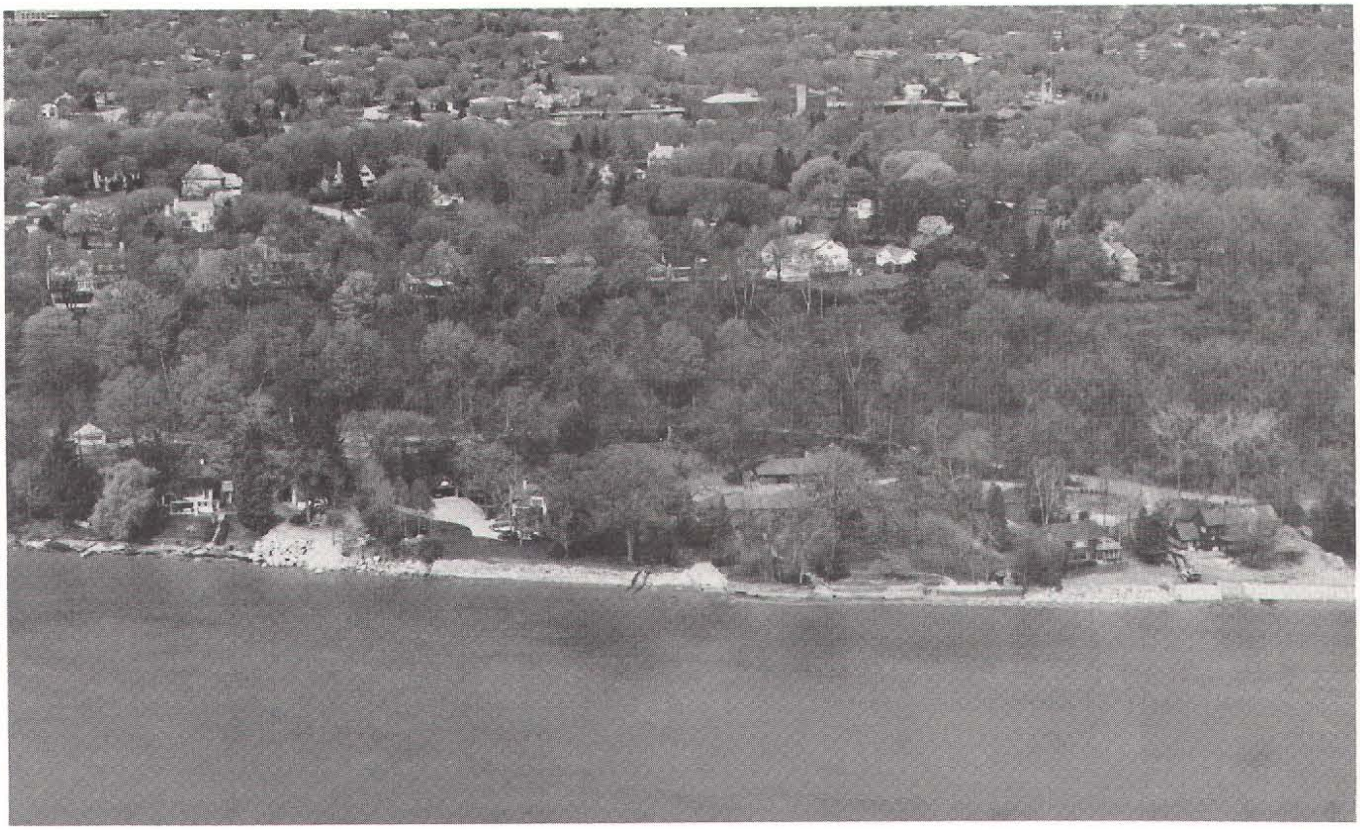
7000-7124 N. BEACH DRIVE
VILLAGE OF FOX POINT



Source: SEWRPC.

Figure C-55

7000-7152 N. BEACH DRIVE
VILLAGE OF FOX POINT



Source: SEWRPC.

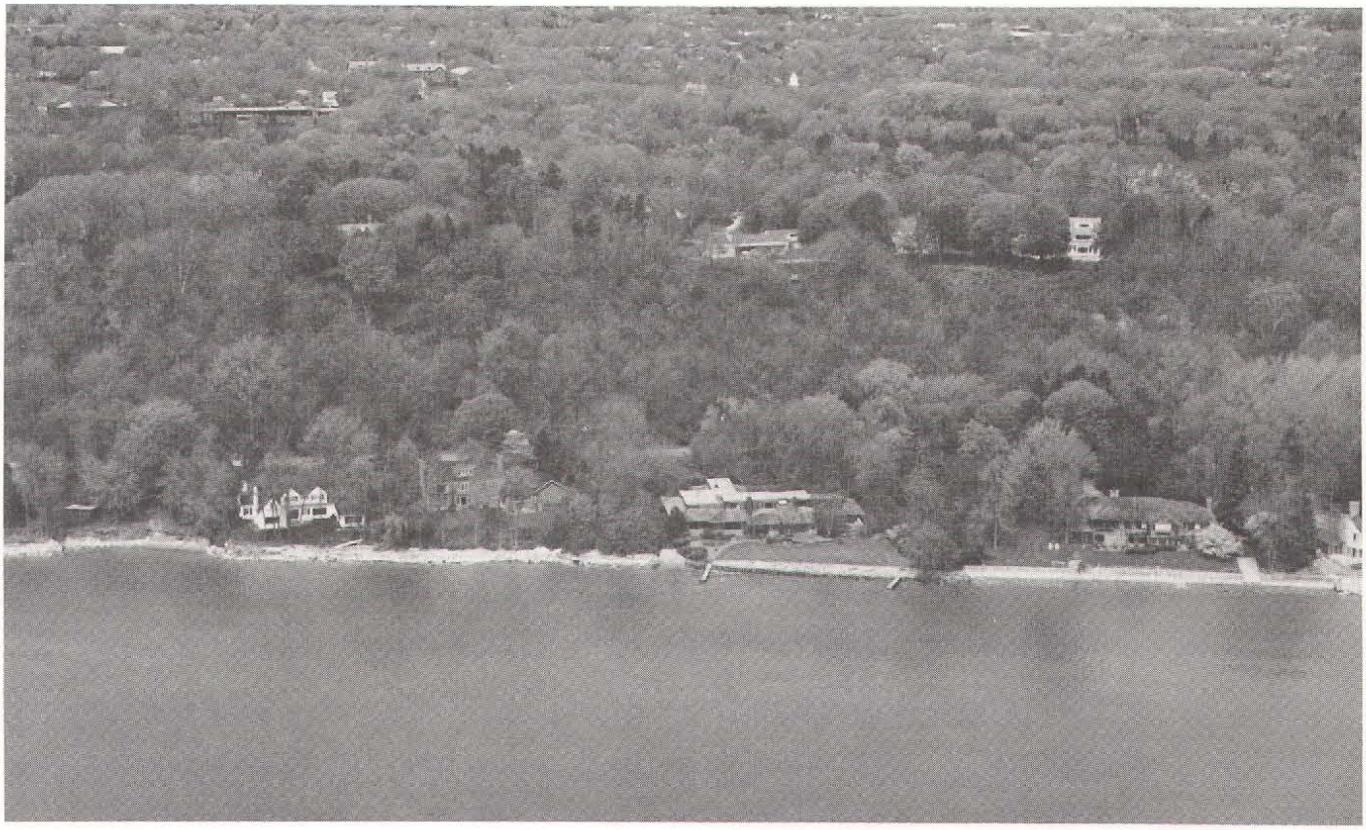
Figure C-56

7106-7200 N. BEACH DRIVE
VILLAGE OF FOX POINT

Source: SEWRPC.

Figure C-57

7134-7234 N. BEACH DRIVE
VILLAGE OF FOX POINT



Source: SEWRPC.

Figure C-58

7210-7328 N. BEACH DRIVE
VILLAGE OF FOX POINT

Source: SEWRPC.

Figure C-59

7250 N. BEACH DRIVE-BEACH COURT
VILLAGE OF FOX POINT

Source: SEWRPC.

Figure C-60

7415-7515 N. BEACH DRIVE
VILLAGE OF FOX POINT

Source: SEWRPC.

Figure C-61

7481-7540 N. BEACH DRIVE
VILLAGE OF FOX POINT

Source: SEWRPC.

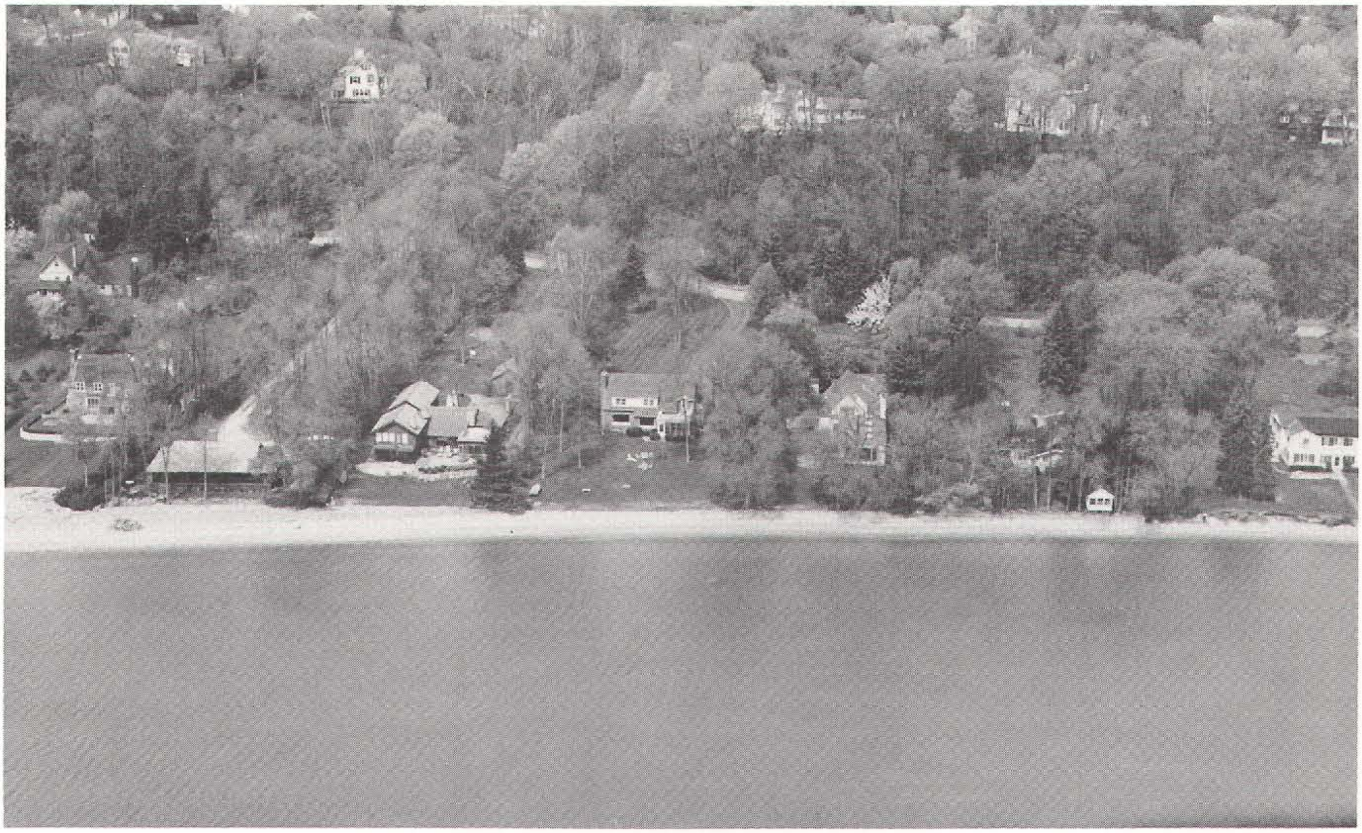
Figure C-62

7540-7704 N. BEACH DRIVE
VILLAGE OF FOX POINT

Source: SEWRPC.

Figure C-63

7644-7736 N. BEACH DRIVE
VILLAGE OF FOX POINT



Source: SEWRPC.

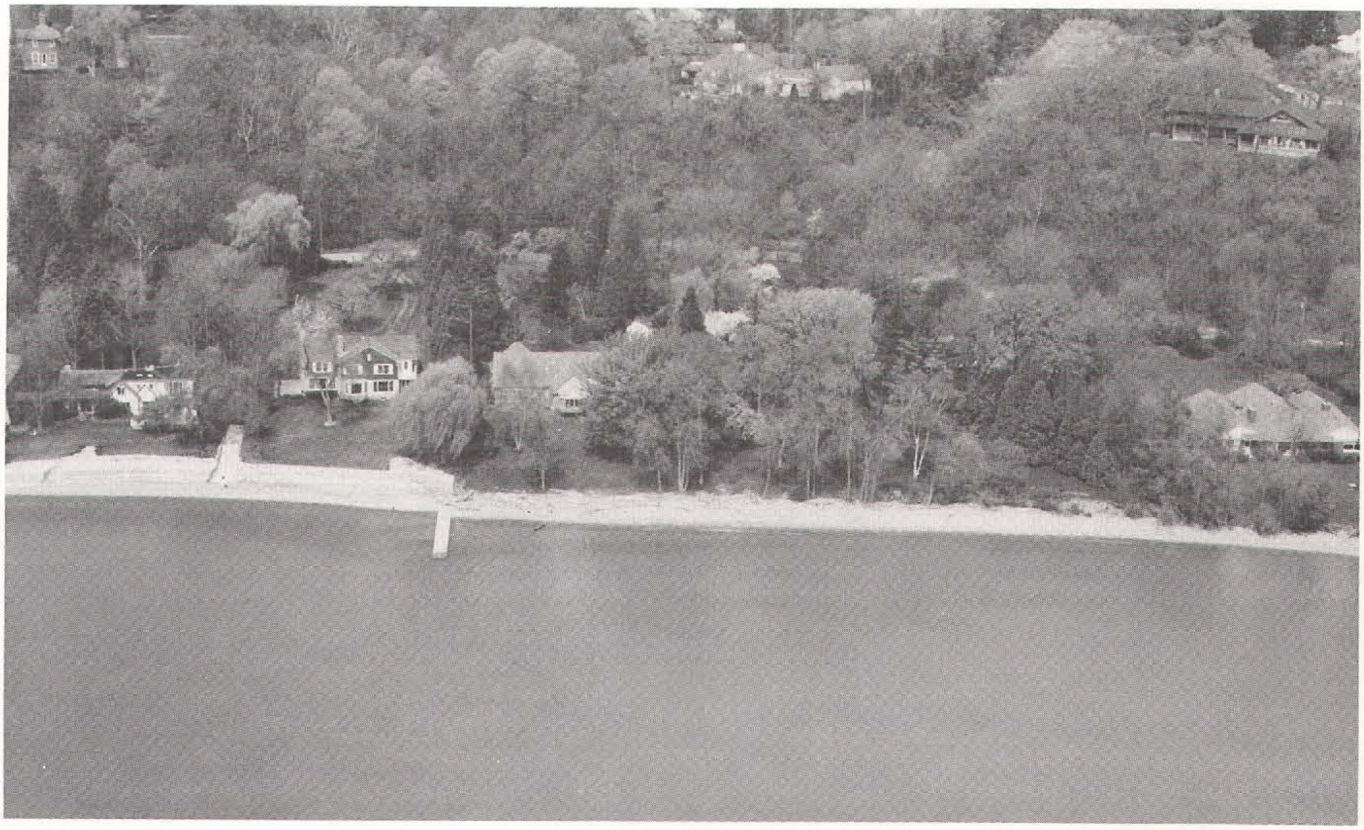
Figure C-64

7736-7828 N. BEACH DRIVE
VILLAGE OF FOX POINT

Source: SEWRPC.

Figure C-65

7828-7930 N. BEACH DRIVE
VILLAGE OF FOX POINT



Source: SEWRPC.

Figure C-66

7930-7954 N. BEACH DRIVE
VILLAGE OF FOX POINT

Source: SEWRPC.

Figure C-67

7944-8005 N. BEACH DRIVE
VILLAGE OF FOX POINT

Source: SEWRPC.

Figure C-68

8005-8035 N. BEACH DRIVE
VILLAGE OF FOX POINT

Source: SEWRPC.

Figure C-69

8025-8064 N. BEACH DRIVE
VILLAGE OF FOX POINT

Source: SEWRPC.

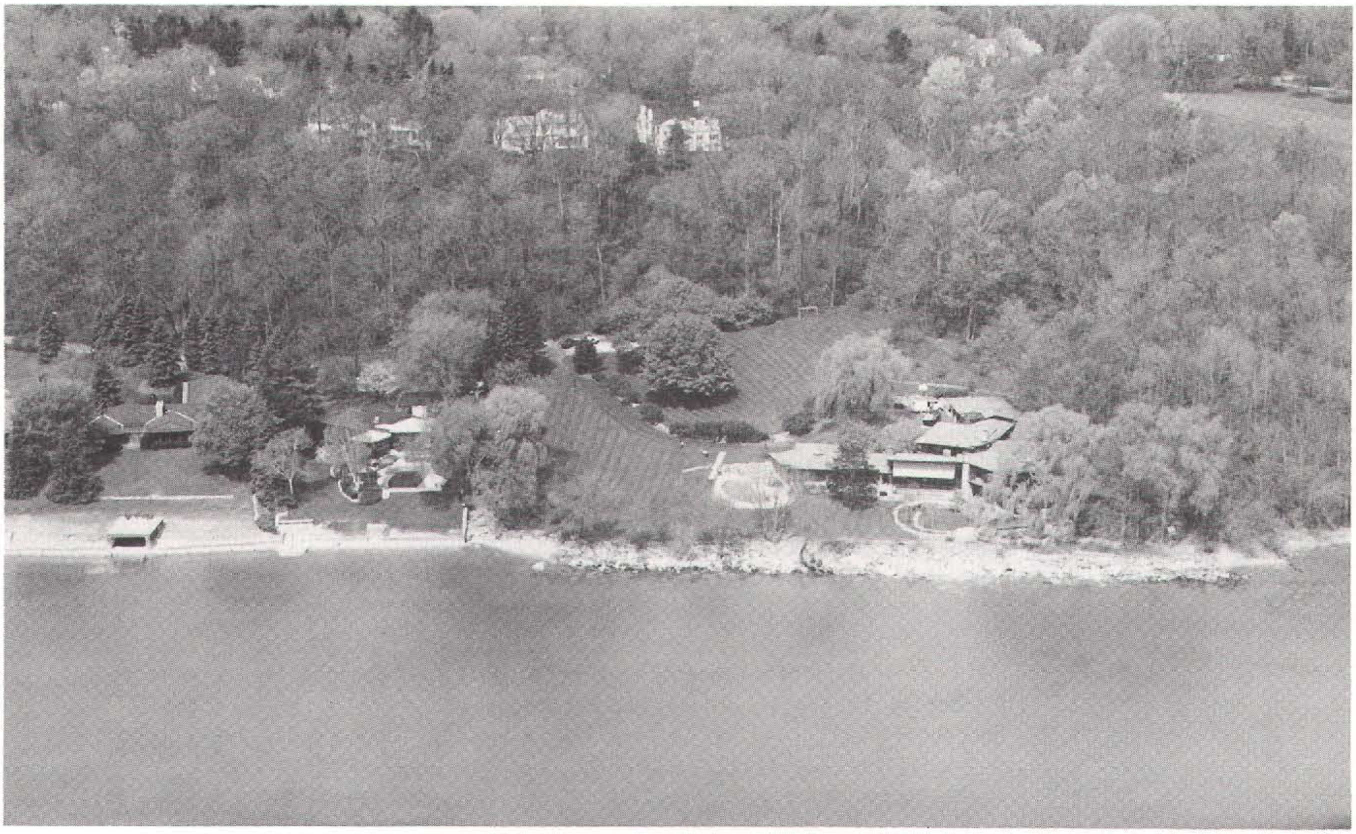
Figure C-70

8064-8120 N. BEACH DRIVE
VILLAGE OF FOX POINT

Source: SEWRPC.

Figure C-71

8110-8130 N. BEACH DRIVE
VILLAGE OF FOX POINT



Source: SEWRPC.

Figure C-72

DOCTORS PARK
VILLAGE OF FOX POINT

Source: SEWRPC.

Figure C-73

DOCTORS PARK
VILLAGE OF FOX POINT

Source: SEWRPC.

Appendix D

PLAN SUMMARY DISTRIBUTED PRIOR TO PUBLIC HEARINGS ON THE NORTHERN MILWAUKEE COUNTY SHORELINE EROSION MANAGEMENT PLAN

NORTHERN MILWAUKEE COUNTY SHORELINE EROSION MANAGEMENT PLAN SUMMARY

OVERVIEW

The shoreline of northern Milwaukee County, always under wave attack, has been severely eroded by relatively high Lake Michigan water levels in the 1970's and 1980's—levels which peaked to record highs in 1986. Two factors above all characterize the condition of the shoreline:

- First, there has been a loss of private and public land owing to the limited effectiveness of those shore protection structures that are not designed for high water levels, and that often are not properly maintained.
- Second, current governmental policies and institutional mechanisms need to be improved to more effectively help lakefront property owners protect their shoreline.

In short, the shoreline is largely protected by structures installed on a piecemeal basis by property owners facing a crisis situation with few alternatives to choose from. While progress continues in protecting many shoreline properties, looming ahead are potential complex, long-term problems, including truck and heavy equipment traffic problems, increased erosion of shoreline areas updrift and downdrift of new structures, increased erosion of the offshore sediments, and the interruption of offshore sediment transport.

Responding to the need for information and for guidelines and procedures to help lakefront property owners, the local shoreline communities in the northern half of Milwaukee County retained the Southeastern Wisconsin Regional Planning Commission to conduct a shore erosion and bluff recession management study. The study was funded in part by the local communities and in part by a grant from the State. The study was carried out under the guidance of an Advisory Committee composed of representatives of the Villages of Fox Point, Shorewood, and Whitefish Bay; the City of Milwaukee; Milwaukee County; the Wisconsin Department of Natural Resources; the University of Wisconsin Sea Grant Institute; the University of Wisconsin-Milwaukee; and concerned and knowledgeable citizens. Assisting the Regional Planning Commission staff in the conduct of the study were consultants from the University of Wisconsin-Madison; University of Wisconsin-Milwaukee; Warzyn Engineering, Inc., Milwaukee, Wisconsin; W. F. Baird & Associates, Ltd., of Ottawa, Canada; and Johnson, Johnson & Roy, Inc., of Ann Arbor, Michigan. The 7.3-mile northern Milwaukee County study area shoreline extends 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.

INVENTORY FINDINGS

About 80 percent of the total shoreline was being eroded by wave action in 1986, when record high lake levels were recorded. Shoreline and bluff recession rates range up to 1.6 feet per year. This recession results in the annual loss of nearly 8,000 square feet of land surface and nearly 600,000 cubic feet of shore material.

Field surveys were conducted to evaluate existing beach characteristics, assess the degree of bluff toe erosion, and determine the adequacy of existing shore protection structures. About 61 percent of the shoreline was found to be protected by revetments, groins, bulkheads, or breakwaters. A 1986 inventory of all 80 shore protection structures in the study area indicated that 76 percent of the structures were in need of substantial repair. The types of structure failure identified included overtopping, where the waves exceeded the top of, and often eroded material behind, the structure; flanking, or erosion at the sides of the structure; material failure; and undercutting. Few structures were found to be properly maintained.

Bluff characteristics and the stability of the bluff slopes were also evaluated. The bluff materials and groundwater conditions were determined by field surveys, soil borings, and electrical resistivity analyses. The bluffs are largely composed of relatively impermeable glacial tills. Sandwiched between these tills, however, are permeable lake sediments—mostly sand and silt. Groundwater seepage within these lake sediments (which discharges from the face of the bluffs) as well as bluff toe erosion by wave action are major causes of slope failure.

Approximately 70 percent of the northern Milwaukee County bluffs exhibited a potential for bluff slope failure in 1986. The stability of the bluff slopes was evaluated by use of mathematical slope stability models. The stability analyses, which were conducted at 44 profile sites, helped quantify the risk of slope failure based on the geometry of the slope, the bluff materials, the strength characteristics of those materials, and the elevation of the groundwater. These analyses also identified those portions of the bluff that were most likely to fail, and helped identify the measures needed to stabilize the slope, such as regrading the slope, or draining groundwater.

THE RECOMMENDED PLAN

A recommended shoreline erosion management plan was prepared to provide guidance to the local communities and lakefront property owners on how to effectively protect the shoreline without adversely affecting other shoreline areas, or the coastal environment. The recommended plan, graphically summarized on Map 1, attempts both to fully stabilize the bluff slopes and to protect the shoreline from wave and ice erosion on a long-term basis. The plan seeks to identify those shore protection measures for individual sections of shoreline which would effectively abate the erosion problems; which recognize the preferences and priorities of the local communities and lakefront property owners; which are economically feasible and implementable; and which would provide—where practicable—a usable shoreline.

To stabilize the bluff slopes, the plan recommends that bluff slopes be regraded—either by filling or by cutting back the top of the bluff—along 29 percent of the shoreline. The plan recommends that groundwater drainage systems be considered for about 15 percent of the shoreline, and surface water control for about 4 percent. Revegetating the bluff slope is recommended for about 18 percent of the shoreline. Bluff slope stabilization may be expected to cost up to $150 per lineal foot of shoreline.

Three alternative means of protecting the bluff toe from wave action were considered: riprap revetments; nourished gravel beaches; and offshore breakwaters, peninsulas, and islands.

Riprap revetments represent the lowest cost alternative, the revetments being relatively easy to construct and maintain. Revetments, however, may result in wave energy which in some areas may erode offshore sand deposits, creating steeper offshore slopes. They do not generally provide a shoreline suitable for most recreational activities. Revetments may be expected to cost from $250 to $350 per lineal foot of shoreline in most areas.

Nourished gravel beaches, which could be contained by rock groins extending out into the lake perpendicular to the shoreline, would provide a more usable shoreline, offering access and recreational opportunities. By resulting in less wave energy than revetments or bulkheads, beaches would cause less scouring and thereby help retain offshore sand deposits. Nourished beaches are generally more costly than revetments and would require periodic renourishment. Nourished gravel beaches may be expected to cost from $300 to $450 per lineal foot of shoreline.

Offshore breakwaters, peninsulas, and islands would create new public lakeshore parkland, provide protected water areas, and minimize the need for shore protection measures along the existing shoreline. When combined with onshore beach systems, offshore structures can reduce the maintenance requirements of the beaches. Offshore structures, however, have a high cost and require a large amount of material for construction. Offshore structures may be expected to cost from $1,000 to $2,000 per lineal foot of shoreline.

The recommended plan integrates the best components of the alternative plans considered. The plan envisions large sand beaches contained by offshore breakwaters at Atwater Park, Klode Park, and Doctors Park; about 19,000 feet of nourished gravel beaches contained by rock groins; nearly 17,000 feet of riprap revetments; and bluff slope stabilization measures. The offshore breakwaters were proposed only for public parks where sand beaches for swimming are desired. Revetments were recommended to protect existing and proposed bluff fill projects, high wave energy environments, and certain locations where revetments were already in place, or under construction. Nourished gravel beaches, which provide a usable shoreline and result in less wave energy, were recommended for essentially all remaining shoreline areas. Beaches were also recommended for some shoreline areas now protected by other structures—such as revetments or bulkheads—where it was concluded that the beaches would have fewer harmful effects on adjacent shoreline areas or on the offshore coastal environment.

The recommended plan would entail a capital cost of about $17.8 million, and an annual maintenance cost of about $1.2 million in 1988 dollars. About 28 percent of the total cost would be financed by the public sector to protect public shoreline property, while the remaining 72 percent of the total cost would be financed by private property owners.

The scope of the recommended plan extends beyond the selection of individual shore protection measures. Coastal processes and the anticipated impacts of the various types of shore protection measures were thoroughly investigated. The plan recognizes that environmental trade-offs must at times be made—particularly when shore protection is not undertaken until a severe erosion problem has developed and real property is threatened. The plan attempts to minimize these environmental trade-offs, as well as potential adverse impacts on adjacent shoreline areas, by trying to foresee problems and by carefully selecting those protection measures which are needed and most appropriate for different coastal environments within the study area. The plan also 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.

PLAN IMPLEMENTATION

The recommended plan must be implemented within entire portions of shoreline, referred to as implementation segments. Eighteen implementation segments, each containing from one to 44 property owners, were identified, as also shown on Map 1. The provision of nine proposed permanent access areas would help centralize and thereby reduce the areawide impacts—including traffic problems—of the movement of trucks and heavy equipment during construction and maintenance operations.

Several alternative methods of implementing the plan were considered: having Milwaukee County coordinate the implementation activities, creating a new lakeshore management district, and placing primary responsibility for implementing the plan with the municipalities.

The cooperation, coordination, and local support needed to successfully implement projects within entire implementation segments can best be provided by the four municipalities concerned: the City of Milwaukee and the Villages of Fox Point, Shorewood, and Whitefish Bay. Thus, it is recommended that the municipalities assume primary responsibility for carrying out the plan.

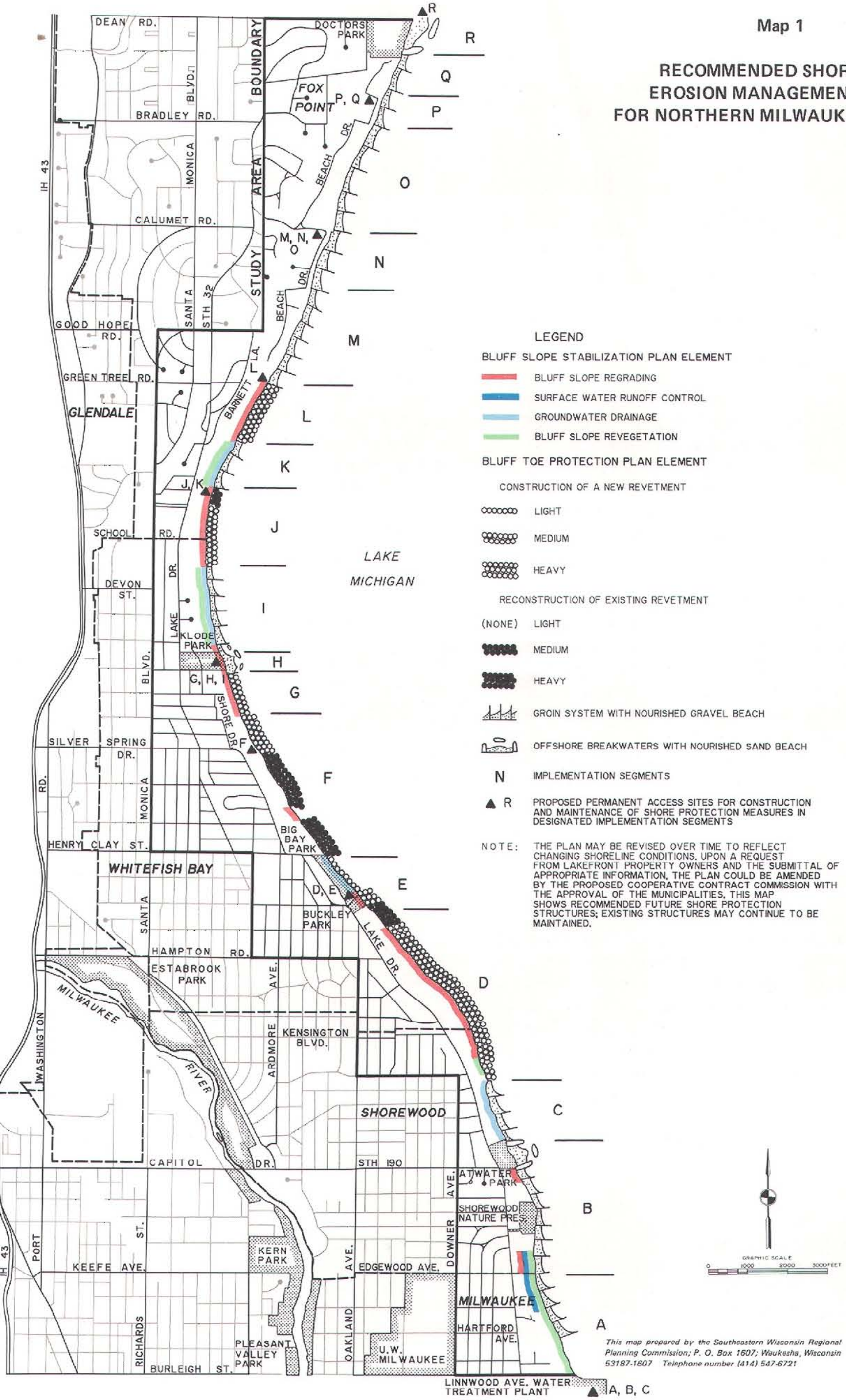
To enhance the efficiency and coordination of the functions needed to carry out the plan, it is recommended that, once the municipalities formally adopt the plan, they jointly form a cooperative contract commission under the provisions of Section 66.30 of the Wisconsin Statutes. Such a commission could efficiently promote plan implementation, although it could not levy taxes or special assessments and could not condemn property without the approval of the individual municipalities concerned. Examples of commissions created under Section 66.30 include the North Shore Water Commission and the North Shore Library Cooperative.

The specific duties to be carried out by the proposed commission would have to be agreed upon by the local elected officials concerned. These duties could include compiling and distributing information on shoreline erosion; reviewing and issuing permits formerly issued by Milwaukee County and the U. S. Army Corps of Engineers; administering shore protection projects; entering into contracts to construct and maintain shore protection structures; and monitoring compliance with the plan. Individual municipal ordinances would remain in effect with respect to zoning, and the regulation of filling, hauling, and other construction activities.

The process for obtaining permits to construct new shore protection measures would be simplified and designed to maximize local control. Under the plan recommendations, permits would no longer be required from Milwaukee County, and permits from the U. S. Army Corps of Engineers and the Wisconsin Department of Natural Resources would be routinely granted for projects in conformance with the plan once these agencies act to approve the plan. Permits for new work may be required only from the newly created commission and the local municipality concerned, thereby assuring both local control and compliance with the recommended plan. The successful implementation of the plan, which requires a stable, long-range commitment to the plan, would provide a high-quality, well-managed coastal environment for northern Milwaukee County.

Map 1

RECOMMENDED SHORELINE EROSION MANAGEMENT PLAN FOR NORTHERN MILWAUKEE COUNTY



ADVISORY COMMITTEE FOR THE NORTHERN MILWAUKEE COUNTY SHORELINE EROSION MANAGEMENT STUDY

Lucia Petrie, Chairman	Trustee, Village of Shorewood
Michael C. Harrigan, Vice-Chairman	Village Manager, Village of Whitefish Bay
Kurt W. Bauer, Secretary	Executive Director, Southeastern Wisconsin Regional Planning Commission
Arthur S. Brooks	Professor, Department of Biological Sciences, University of Wisconsin-Milwaukee
Noreen R. Cook	Village Manager, Village of Fox Point
Leonard E. Engelhardt	Parks Facilities Planning Manager, Milwaukee County Department of Parks, Recreation and Culture
John A. Erickson	City Engineer, City of Milwaukee
Roberta Hazlett	Citizen, Village of Shorewood
Richard A. Keyes	Environmental Engineer, Department of Public Works, Milwaukee County
Ralph P. Knoernschild	Citizen, Village of Whitefish Bay
James F. Lubner	Environmental Specialist, University of Wisconsin-Sea Grant Institute
Edward C. Madere	Village Manager, Village of Shorewood
Kristine Nelson	Trustee, Village of Fox Point
Neal T. O'Reilly	Water Resources Planner, Wisconsin Department of Natural Resources-Southeast District
Louise Petering	Citizen, Village of Fox Point
Howard J. Pincus	Former Professor, Department of Geological Sciences, University of Wisconsin-Milwaukee
Penny E. Podell	Supervisor, Milwaukee County

This publication prepared by the Southeastern Wisconsin Regional Planning Commission; P. O. Box 1607; Waukesha, Wisconsin 53187-1607 Telephone number (414) 547-6721

QUESTIONS AND ANSWERS ABOUT THE NORTHERN MILWAUKEE COUNTY SHORELINE EROSION MANAGEMENT PLAN

1. Q. WHAT PROMPTED THE STUDY?

   A. In the early 1980's, local citizens expressed significant concern about the current piecemeal approach to shore protection. These concerns, which included the type and design of shore protection measures installed, the construction and maintenance of these measures, the appearance of the measures, impacts on adjacent shoreline areas, disturbance by truck and heavy equipment operators, and the control and management of these projects by the units of government involved, were raised publicly at hearings and meetings held to discuss certain shore protection projects initiated in the early 1980's. In response to these citizen concerns, the local units of government formed a committee to discuss the problem. In 1984, the shoreline communities of the northern half of Milwaukee County subsequently retained the Regional Planning Commission to undertake a study of the problems and the best means of their resolution.

2. Q. WHO CONDUCTED THE STUDY?

   A. The study was primarily conducted by the staff of the Regional Planning Commission, with assistance from consultants, under the guidance of an advisory committee. The advisory committee consisted of representatives of the Villages of Fox Point, Shorewood, and Whitefish Bay, the City of Milwaukee, Milwaukee County, the Wisconsin Department of Natural Resources, the University of Wisconsin Sea Grant Institute, the University of Wisconsin-Milwaukee, and concerned citizens.

3. Q. WHAT ARE THE RESULTS OF THE STUDY?

   A. The results of the study are summarized in the accompanying overview and map. Key questions and answers about the study results follow.

4. Q. WOULD RECENTLY COMPLETED WORK BE REDONE UNDER THE PLAN?

   A. No additional work would be undertaken until a majority of the property owners within a proposed project area agreed that protection was desirable. Additional shore protection measures are recommended in some areas where the existing shore protection measures provide an inadequate level of protection against wave action, or where existing measures have an adverse impact on adjacent shoreline areas or on the offshore coastal environment.

5. Q. WHY ARE NOURISHED GRAVEL BEACHES RECOMMENDED FOR SOME SHORELINE AREAS NOW PROTECTED BY OTHER TYPES OF STRUCTURES?

   A. In the long term, the beaches should provide a more desirable shoreline for the property owners. The nourished gravel beaches reduce wave reflection, thereby preventing steepening of the offshore slopes and the associated increased wave damage potential. To a limited extent, the beaches feed the offshore sediment transport system, thereby reducing adverse impacts on the near-shore environment.

6. Q. HOW FLEXIBLE IS THE PLAN?

   A. The plan is flexible in terms of both how and when projects would proceed. Projects would be undertaken only upon an appropriate petition of a majority of the property owners within a proposed project area, or implementation segment. Upon request and the submittal of appropriate information, the implementing agencies—your municipality and the cooperative contract commission jointly formed by the communities of the northern half of Milwaukee County—could amend the plan recommendations as the need arises.

7. Q. DO PRIVATE PROPERTY OWNERS GIVE UP THEIR RIPARIAN RIGHTS BY IMPLEMENTING THE PLAN?

A. No. All existing private property would remain in private ownership. Any new beaches created above the ordinary high water marks become the property of the riparian property owners, for their exclusive use.

8. Q. COULD PRIVATE PROPERTY OWNERS BE REQUIRED TO COMPLY WITH THE PLAN?

A. If an appropriate petition is submitted by the majority of the property owners in a project area and approved, a property owner may be required to comply with the plan. Upon a request from a majority of the property owners within a project area, your municipality may use its special assessment authority to assist in financing shore protection projects at favorable interest rates.

9. Q. HOW WOULD THE PLAN SIMPLIFY THE PERMIT PROCESS?

A. Under existing conditions, permits for shore protection projects are issued by the U. S. Army Corps of Engineers, the Wisconsin Department of Natural Resources, Milwaukee County, and the municipalities. Under the recommended plan, your municipality would continue to issue permits for filling and hauling. The cooperative contract commission created by the municipalities would likely also issue permits for shore protection structures formerly issued by Milwaukee County and the U. S. Army Corps of Engineers. Milwaukee County would no longer issue permits for shore protection projects.

10. Q. WHO WILL PAY FOR THE IMPLEMENTATION OF THE PLAN ON PRIVATE PROPERTY?

A. Lakefront property owners would continue to pay for shore protection measures for their property. The municipalities may assist the property owners in distributing the costs over time by financing the projects and then levying taxes or special assessments to the benefiting parties.

This publication prepared by the Southeastern Wisconsin Regional Planning Commission; P. O. Box 1607; Waukesha, Wisconsin 53187-1607 Telephone number (414) 547-6721

Appendix E

MINUTES OF PUBLIC HEARINGS ON THE NORTHERN MILWAUKEE COUNTY SHORELINE EROSION MANAGEMENT PLAN

VILLAGE OF SHOREWOOD

MARCH 29, 1988

The meeting was called to order by Village President Michael Spector at 7:30 p.m. Mr. Spector summarized the format of the meeting and then called on the representatives from the Southeastern Wisconsin Regional Planning Commission (SEWRPC) to make a summary report on the preliminary plan.

Mr. Robert Biebel (SEWRPC) explained the background that lead up to the lake shore erosion study which includes an area from the water filtration plant in Milwaukee north to Doctors Park in Fox Point. This area, known as "Reach 10," was experiencing varying levels of bluff instability which was being dealt with in an inconsistent manner by property owners. As a result of the efforts to control erosion, other problems such as traffic, dust, noise and concern for the resulting impact on adjacent properties of the erosion control efforts prompted the study in 1985. The study was conducted by SEWRPC under the guidance of a steering committee. The objective was to develop a plan for erosion control measures which, when implemented, would create a stable shoreline in Reach 10.

Mr. David Kendziorski (SEWRPC) summarized the planning process and the elements of the preliminary plan. He stated that the field data collection phase was conducted in 1986. At that time, 11 erosion protection structures existed in the Village of Shorewood. He said that 40% of the bluff was considered stable, 40% was marginal, 5% was unstable and 15% was in the process of being filled or treated in some way.

Mr. Kendziorski showed slides of the various types of treatment suggested in the plan to deal with the erosion problem, including rip-rap, nourished gravel beaches, groins and offshore breakwaters. He said that in general, the plan suggests an offshore breakwater at Atwater Beach, rip-rap revetments at all existing fill areas and nourished gravel beaches for the remainder of the area. The shoreline within the Reach 10 area was broken down into 18 implementation segments. Shorewood's portion of the reach was divided into three segments for implementation. Access to the shore from Shorewood's portion of the work would be at Lake Park in Milwaukee and Buckley Park in Whitefish Bay.

Mr. Kendziorski projected the cost of completing the suggested improvements within the village limits at $3.7 million. He estimated that 50% of the cost would be for protection of village-owned shoreline and the other 50% would be to protect private property. Once completed, he estimated that on the average, the cost to maintain the erosion control improvements would be approximately $150,000 per year for the public area and $90,000 for the private area.

Mr. Biebel outlined several methods by which the plan might be implemented. He said that no specific method had been selected and that others may be considered in the future.

LIST OF IMPLEMENTATION METHODS

A. Place overseeing role with the County.

B. Place overseeing role with each abutting municipality.

C. Create a new unit of government with its own taxing power like a regional government or benefit district.

D. Create an overseeing commission (i.e., a library commission) set up by the abutting municipalities, each having representation.

Following the presentation by SEWRPC, Mr. Spector invited all those in attendance to ask questions and present their views on the issue.

Raymond Zagar, 4074 N. Lake Dr., asked for more specific information regarding the cost to construct and maintain a nourished beach. It was estimated that it would cost between $300-$400 per lineal foot to build and about $30/ft/yr to maintain.

Leslie Muma, 4442 N. Lake Dr., asked if the erosion control projects which are already completed meet the standards of the proposed plan. SEWRPC felt that all the bluff slopes on the current projects were stable, but the revetments at the toe of the slopes were not evaluated as a part of their study.

Nancy Florsheim, 4090 N. Lake Dr., asked how the plan was proposed to be implemented. SEWRPC recommended that large stretches be completed at one time. Shorewood would be divided into three sections. This would make the implementation the most economical. A final determination of the project areas would be made by the municipalities.

In response to a general question on implementation methods, Mr. Spector indicated that no decision had been made as to what the method would be, and that the purpose of the hearing was to get input from the residents as to what the most appropriate method of implementation would be.

Michael Lechter, 4408 N. Lake Dr., asked about the status of existing bulkheads and vehicle access sites to the erosion projects. SEWRPC said that a revetment would be placed in front of any existing bulkhead and also that vehicle access sites were identified on the plan. For Shorewood's project they were located at the Linnwood Ave. Water Treatment Plant in Milwaukee and at Buckley Park in Whitefish Bay.

Mrs. Smith, 4480 N. Lake Dr., asked when the details of the plan would be worked out. Mr. Spector said that a commission of some type would have to be established and work with the various municipalities to iron out the details of the plan.

Lyle G. Henry, 4098 N. Lake Dr., expressed concern about some entity being formed which could ultimately force him to participate in the plan. He urged the board to be very careful in deciding who would have authority to determine when the plan would be implemented and who would pay for it.

Attorney Mark Gehring, representing Gene Wakefield, 3550 N. Lake Dr., said that property owners should be the only ones to decide if the project should be done. He said that the cost to implement and maintain the plan would be economic devastation.

Robert Mazzie, 4100 N. Lake Dr., was concerned that the plan was already decided and that only the decision as to how to tax was left. Mr. Spector reiterated why SEWRPC had developed the plan and why the hearings were held.

In answer to a general question regarding the effect on the plan of low lake levels, SEWRPC indicated the following four points:

A. No one knows whether the lake will rise or fall.

B. Low water levels would allow a project to be deferred.

C. Implementation would probably be easier and cheaper during periods of low water level.

D. No changes to the plan would be warranted since the water would eventually go back up.

Attorney Mark Gehring asked what the tax impact would be of the improvements if done. The answer to this question is not known at this time.

John Beard, 4162 N. Lake Dr., asked why the groins in Segment C were spaced so close together. SEWRPC indicated that the plan was based on a high water level and therefore the design was necessary.

James Sankovitz, 4057 N. Prospect, asked if the property owners on Lake Dr. would be sheltered from the additional burden of their share of the cost to protect the public portions of the lakefront. Mr. Spector said that it might be very difficult to do that under current State law.

A general question was asked regarding what alternatives would be available to protect the publicly owned shoreline if the private property owners did not proceed with their part of the plan. Mr. Spector then summarized the concerns expressed as of that point in the hearing as follows: "Who is going to have the power to order what?"

Since there were no further questions at this point, Mr. Spector thanked all for coming and adjourned the hearing.

Respectfully submitted,

James J. Lynch
Director, Department of City Development

VILLAGE OF FOX POINT

April 26, 1988

President Dengel welcomed the approximately eighty-five persons in attendance to the public hearing on the northern Milwaukee County shoreline management study and proposal. Mr. Dengel said he would serve as moderator, with the format being a presentation by representatives of the Southeastern Wisconsin Regional Planning Commission (SEWRPC), Dave Kendziorski and Bob Biebel, followed by comments by those in favor of the proposal, and by those with concerns and other comments being heard last.

Mr. Dengel said no decision would be made at the meeting; the purpose of the hearing is to receive citizen input regarding the proposal. He said Shorewood has already held a public hearing; one is scheduled in Whitefish Bay for May 16 and Milwaukee will be scheduling a public hearing soon. The Advisory Committee will approve the final report after the public hearings have been completed. They will send the report to the boards of the municipalities for review and/or action.

Lucia Petrie, chairperson of the study committee, Louise Petering, citizen member of the committee, and Trustee Nelson, trustee representative to the committee, were introduced by Mr. Dengel.

Trustee Nelson said action which resulted in this meeting actually began in 1984, at which time Fox Point was asked to join with Whitefish Bay, Shorewood, parts of the City of Milwaukee and Milwaukee County to examine the erosion problems. The purpose was to provide guidance to property owners and to help them in their seeking of solutions to their shoreline problems.

The situation at the time was one of rising lake levels and concern with the effects erosion projects would have on neighboring properties. Nelson said there was controversy over access to the projects and concern over the permitting process. The resurrection of an island project plan from the 1930's, plus the availability of free tunnel rock from the MMSD deep tunnel project as well as the availability of State grant money through SEWRPC, lead to the formation of the Advisory Committee.

Nelson said the purpose of the public hearing was to share what the committee members learned with the residents and to learn residents' concerns.

Bob Biebel thanked the board for holding the public hearing. He said that the study was conducted because beach and bluff erosion accelerated during the seventies and eighties, with lake levels reaching an all-time high late in 1986.

As projects were developed to deal with erosion problems it appeared some long-term problems were being created because of the piecemeal nature of the projects that had been constructed. Mr. Biebel said some problems were created by trucks using residential streets during the fill work. He said some of the projects could cause erosion of the offshore lake bottom which can increase wave height and lead to greater problems.

With these concerns in mind, it was thought desirable to take a broad look at the seven-mile reach of shoreline and develop some alternatives and recommendations to resolve the erosion problems that existed. Mr. Biebel said one objective of the study was to provide information that could be used to assist in evaluating the problems that may exist, as well as the solutions available. He said much of the information developed such as bluff stability analyses and subsurface information has been used by project designers for several projects in the northern Milwaukee County area. A second objective of the study was to examine alternatives, and to recommend a plan, to resolve the erosion problems that exist. Finally, an implementation plan was recommended. Mr. Biebel said he expects the final committee report will be completed in about a month. The report will be entirely advisory; it will not carry any kind of regulatory power and will not require any action.

Dave Kendziorski described some of the physical conditions of the area and some of the inventory findings. He said Fox Point contains about two and three-quarter miles of shoreline and has 87 shoreline property owners. He used maps prepared by SEWRPC to summarize the findings of the inventory conducted during the summer of 1986—a year of high lake levels. He said the lake levels currently are two to three feet lower than at the peak high level.

The maps show approximately 39 shore protection structures which were in place in 1986. Those structures at least partially protected 91% of the Beach Drive terrace shoreland and about 50% of the bluff shoreline. The maps also show shoreline erosion affected about 35% of terraced area and about 70% of bluff area during that time.

Another map summarized the stability of nine bluff analysis sections in Fox Point. Within each of those sections, bluffs were classified as stable, marginal, or unstable. The map has been updated to show fill projects either completed at present or in progress. Mr. Kendziorski showed slides of the existing erosion situation in the Village and some erosion control measures being recommended. He also showed slides taken along the Fox Point shoreline illustrating examples of protection methods such as rip-rap revetments, bulkheads, the groins, and nourished beaches.

Mr. Kendziorski said nourished gravel beaches are being recommended for about 50% of the total study area and for about 75% of Fox Point. Pea-sized gravel is placed at the base of the bluff and beach material is contained by structures such as groins extending perpendicular to the shoreline. He said the groins don't need to be very long, but need to be high because the gravel forms a fairly steep slope.

Mr. Kendziorski said the plan as tentatively approved by the Advisory Committee includes construction of offshore breakwaters at Atwater, Klode and Doctors Parks. The recommended plan includes rip-rap revetments for bluff areas that have already been filled or are proposed to be filled, as well as a few isolated high wave energy areas. For most of the remaining areas, the Committee is recommending nourished gravel beaches. The Committee recommends the plan be implemented as 18 different projects which are designated by letters on the maps.

The total estimated cost to implement the plan is $17.8 million. The total cost to implement the plan in Fox Point is about $6 million. About $1.3 million would be financed by Milwaukee County to protect Doctors Park. The Village portion would be about $700,000, with private property owners financing the remaining $4 million.

Bob Biebel said the Advisory Committee reviewed three methods for implementation of the plan: 1) coordination of projects and permitting handled by Milwaukee County; 2) leaving the responsibility for permitting and overseeing of projects to local government; and 3) creation of a new lakeshore erosion district. He said the consensus of the Committee was that the implementation of the plan was best dealt with by the municipalities with some kind of a cooperative commission being formed with the three communities involved. The commission might have such responsibilities as coordination of projects and permitting.

President Dengel said the board wants to hear what residents have to say. He said the advisory group will listen to all the comments at all of the public hearings and then make a recommendation that will be sent to the various communities, and the communities will determine what, if any, action to take. Mr. Dengel invited those with further information and/or in favor of the plan to speak.

Robert Brill, 6950 N. Barnett Lane, thanked the Committee for their work on behalf of his neighbors and himself. He said if the study hadn't been done, he and his neighbors wouldn't be in a position to have the group project they have today. He said the study information had saved him and his neighbors thousands of dollars in engineering and other costs.

Derrick Andrade, 4039 N. Richland Court, Shorewood, said it is his opinion that rip-rap fill should be allowed to be placed further out in the lake and gradually sloped to lessen wave action. He doesn't feel protrusions into lake cause problems downcurrent. He doesn't believe individual, piecemeal projects are necessarily bad.

Trustee Greene said in his opinion it is crazy to penalize residents who have already protected their properties by imposing an additional tax burden.

Louise Petering, 7229 N. Santa Monica Boulevard, asked the SEWRPC representatives about the tax impact of the plan implementation on residents who already have protected their shoreline.

Mr. Biebel said if the plan were adopted in part or completely, before a project could proceed, a group of people from a particular area would have to apply for permits to go ahead. He said he did not envision any public money being spent to protect private property; therefore, taxes wouldn't be affected. A homeowner wouldn't be special assessed if a project wasn't requested and begun in his reach. Mr. Biebel said this is a long-range plan which could be implemented any time as the need arises. He said there is no intention to force anyone to build an unnecessary project, and that the only way taxes might be impacted is if the Village itself decides to improve some of its shoreline property.

Jerry Frank, 6720 N. Lake Drive, asked what factors affect Lake Michigan levels and whether or not the level is likely to increase or decrease in the future. He also inquired as to conditions on the rest of the Wisconsin shoreline.

Mr. Biebel replied that lake levels are difficult to predict and are basically a result of the amount of precipitation. He said other areas of the Wisconsin shoreline have as severe erosion problems as the studied area, but since they are not as urbanized, erosion doesn't create as much of a problem. He said that a plan was being developed for southern Milwaukee County that and one was previously prepared for Racine County.

Richard Boxer, 7210 N. Beach Drive, said his concerns include the costs, particularly in view of the fact that residents have spent as much as $35,000 to protect just their own property. Mr. Boxer said he is also concerned about who would be liable for injuries that might be sustained by persons using the beach that might be created. He also said the proposed plan would result in a need for more police protection. Mr. Boxer said there would be maintenance costs which would impact on property values of affected residents.

Barbara Hussin, 1015 E. Quarles Place, said because of unpredictable lake levels, the only solution is to look at long-range planning. She said the proposed plan offers a good long-range plan and deserves careful consideration.

John Oscarson, 6430 N. Lake Drive, said the fact that the study was undertaken as a service is to be commended and is appreciated. He said he favors the Village resolving the erosion problems themselves since local control can be lost if a governing body is too large. He said he is pleased to use the results of the study as it applies to individual property owners.

Marvin Wooten, 7743 N. Beach Court, said his concern is the 100-foot beach that might be created in an area with a very narrow road that is already overused. He also fears the creation of a beach will create demand for public access.

Esther Hoffman, 7405 N. Beach Drive, said she attended an early informational meeting at which Lucia Petrie, chairman of the Advisory Committee, indicated that without question, any beach area created would be opened to the public, partly because some public funds would be expended.

Ms. Petrie was in attendance and responded by saying the meeting Ms. Hoffman referred to was held when an offshore project was being considered. This idea has since been rejected.

Andy Gronik, 7124 N. Beach Road, asked if the plan could be forced on owners by either Fox Point or the County. Mr. Biebel replied that in talking with village officials, he didn't get the impression that anyone wanted to force an unwanted project on residents. Mr. Gronik then asked for a show of hands of those in favor of the plan and also of those opposed. The show of hands indicated almost unanimous opposition to the plan.

Ruth Sommer, 7710 N. Beach Drive, said the Committee had done a really good job but she was disappointed property owners weren't represented on the Advisory Committee. Ms. Sommer expressed concern about additional parking and additional usage if additional beach is formed.

Alan Gaulke, 6410 N. Lake Drive, said he has experienced surface water erosion and the study didn't take this type of problem into account.

Dave Kendziorski said surface water erosion had been brought to the Committee's attention since the preliminary plan was drafted. He said the Committee did attempt to identify areas where surface- or groundwater problems were causing bluff instability, and for those areas, a drainage system was recommended.

Jean Lindemann, 8035 N. Beach Drive, said that about two years ago Governor Thompson formed a commission to study the possibility of controlling the level of Lake Michigan. She said water levels are controlled in all but two of the Great Lakes. Ms. Lindemann said lake levels can be controlled by dredging and that the commission findings were given to the Governor seven or eight months ago, but there has been no response. She urged those in attendance to write to the Governor asking where the study is.

Carl Christianson, 7463 N. Beach Court, said he is generally concerned with erosion problems but is not in favor of opening up areas to more traffic because of an attractive beach.

Mr. Biebel said the reason the study committee recommended the beach development as opposed to revetment protection was because the beach development costs are about the same and there isn't the possibility of aggravated scouring erosion that revetments might cause. It was also thought that a usable beach would be a plus. In view of the concerns expressed by the residents, Mr. Biebel said the Committee would take another look at the recommended plan.

President Dengel said the Board had concerns regarding the plan and had voted unanimously against the concept of a separate taxing district. Mr. Dengel said the Board has asked the Advisory Committee to consider recommending alternatives to the beach and groin plan for Beach Drive. He said the Advisory Committee will probably respond to that request after the public hearings are over.

Dick Tollefson, 7730 N. Beach Drive, said when anyone does a fill project, the neighbors are affected. His worry is that as certain sections are completed, neighboring property owners will be forced to do something as well.

Mr. Biebel said that the plan recommended that a group of property owners do a project together. This way, material can be brought to a site more efficiently and traffic problems would be concentrated into a given time frame. The procedure would be for a majority of property owners within a reach to petition for permits. He said a potential problem exists if someone in the middle of a reach doesn't want the project, but this problem would be best dealt with by the municipality.

Hedy Tollefson, 7730 N. Beach Drive, asked who would be liable if groins and jetties constructed caused a problem for neighboring property owners. Mr. Biebel said the plan is designed to prohibit adverse impact on neighboring properties.

William Chase, 7513 N. Seneca Road, said he is a fisherman and he wonders what he and other nonshoreline residents are going to get for the public monies proposed to be spent. He complained about the restriction on fishing before 8:00 a.m. at the public access beach.

Ralph Knoernschild, Whitefish Bay resident and Advisory Committee member, said he also is a director of the Great Lakes Coalition. The Coalition is interested in controlling Lake Michigan level by lowering the water level by dredging. Mr. Knoernschild said dredging can be done without a great expenditure. He said all shoreline residents will be invited to a meeting to be held by the Coalition at which their plan will be explained.

Robert Brill said he believes the residents shouldn't adopt a negative attitude to the proposed plan. He said it is time for the Village Board to take a stand saying: "Here is what we believe in and here is what we're going to do." He expressed disappointment because when he and his neighbors asked the Village for help in the following areas, it was denied: 1) to obtain rock from the sewer tunnel project; 2) for help in financing; and 3) a request that the Village Engineer meet with the engineer hired by N. Barnett Lane neighbors to explore access to the project. Mr. Brill also said he feels the projected costs are too high. He said the N. Barnett Lane landowners are getting a good project for under $230/foot.

President Dengel said it is the opinion of the Board and his personal feeling it is the right and obligation of property owners to protect their own property and the property owner is the only one who can do so.

Arthur Derdarian, 7611 N. Beach Drive, said the only way to stop erosion is to construct jetties out into the lake that would cause a wave to drop back on itself; this will aerate a wave and take the force out of it. He said that in all areas where groins have been used, failure has resulted.

James Greenlee, 6600 N. Lake Drive, said that as the south side of a groin erodes, there is a buildup on the north side.

Louise Petering thanked the Board for the opportunity to be heard. She read a prepared statement in support of the plan saying the plan constitutes an overall blueprint for future action when the existing structures protecting the shore need to be replaced.

Terri Lorenz, 7200 N. Beach Drive, said she feels fortunate to have property on the lake and doesn't want to see it erode away, but those people who use the lake should pay for its protection. She doesn't want to spend money to create a playground for others' use.

David Kingsley, a new resident on Beach Court, said construction of groins would provide more perches for more fishermen.

President Dengel closed the public hearing by thanking everyone for coming and urging the Advisory Committee to bring the study to a rapid conclusion and to get the final report back to the Village so the Board can make a decision. Mr. Dengel said shoreline property owners will be notified when action will be taken on the Advisory Committee report.

Trustee Nelson thanked SEWRPC staff for a clear, concise presentation, noting how skillfully they had concentrated a tremendous amount of material into the brochure distributed prior to the hearing.

Respectfully submitted,

Joann Mock
Deputy Clerk

VILLAGE OF WHITEFISH BAY

MAY 16, 1988

Regular meeting of the Board of Trustees of Whitefish Bay, held in the Village Board Chamber of Village Hall, 5300 N. Marlborough Drive, May 16, 1988.

Pursuant to law, written notice of this meeting was given to the press and by posting on public bulletin board.

Meeting was called to order at 7:30 p.m. by President Matthews.

Present: Trustees Belfus, Ernest, Gormley, Hatfield, Jermain, Riesch and President Matthews.

President Matthews opened the public hearing regarding the proposed northern Milwaukee County shoreline erosion control and shoreline protection plan developed by SEWRPC.

President Matthews said that no action would be taken at this meeting, and that the purpose of this hearing is to receive citizen input regarding the proposal. He said that hearings have already been held in Shorewood and Fox Point, and the County of Milwaukee and City of Milwaukee will also be having hearings on the proposal.

Village Manager Harrigan, who also served as Vice-Chairman of the Committee, said that the process of developing a shoreline protection plan began over two years ago. The situation at the time was one of rising lake levels and concern about the effects that private erosion projects would have on neighboring properties. At that time Whitefish Bay was asked to join with Fox Point, Shorewood and parts of the City of Milwaukee and Milwaukee County to examine the erosion problems. The purpose was to provide guidance to property owners and to help them in their seeking of solutions to their shoreline problems.

Manager Harrigan introduced Bob Biebel and Dave Kendziorski of the Southeastern Wisconsin Regional Planning Commission (SEWRPC) who gave a slide presentation on the effects of erosion to shoreline properties in the Village of Whitefish Bay. Mr. Kendziorski provided examples of the proposed solutions to the problems.

Mr. Biebel said the study was conducted because beach and bluff erosion accelerated during the seventies and eighties, with lake levels reaching an all-time high late in 1986. As projects were developed to deal with erosion problems it appeared some long-term problems were being created because of the piecemeal nature of the projects that had been constructed. Mr. Biebel said that with these concerns in mind, it was thought desirable to take a broad look at the seven-mile reach of shoreline and develop some alternatives and recommendations to resolve the erosion problems that existed.

Manager Harrigan said that what is being recommended is consistent with what is being done by three-quarters of the shoreline owners now. He also said one of the recommendations was that the permit process for erosion projects be simplified and "brought closer to home." Under existing conditions, permits for shore protection projects are issued by the Army Corps of Engineers, Milwaukee County, and Whitefish Bay. Under the recommended plan, permits would be issued only by the Village and a local government commission appointed by the boards of the North Shore communities involved.

Mr. Bill Emory, 5738 N. Shore Drive, asked if the Village had ever denied a fill permit, and who would do the engineering for the proposed projects.

President Matthews said that no fill permit had ever been denied to his knowledge, and that private property owners would be responsible for coordinating engineering between neighbors.

Mr. Emory asked that SEWRPC provide recommendations on standard designs for fill projects.

Mr. Biebel said that standard designs are in the process of development at this time.

Mr. John Strassman, 6120 N. Lake Drive Court, said that the plan seems sound and should be used as a zoning control tool for future projects. He said that poorly designed projects can be harmful to all neighboring properties, and that only high-quality, acceptable projects should be constructed.

Mr. Philip Weinberg, 6240 N. Lake Drive, said that he does not think fill materials should be put in his area. Mr. Kendziorski said that only a sand beach was recommended in that area because it was relatively stable.

Mr. Weinberg asked if the lake level could be artificially controlled.

Mr. Ralph Knoernschild, 5166 N. Berkeley, replied yes. He invited any interested persons to come to a public hearing on June 14th to be held at Whitefish Bay High School at 7:30 p.m. to discuss this issue.

Mr. Jim Hanson, 4696 N. Lake Drive, asked if recommendations could be presented in more specific terms to individual property owners.

Mr. Biebel said that SEWRPC was preparing a specific document and that copies would be provided to the Village and be made available to the public.

Manager Harrigan asked that any interested persons leave their names and addresses and information would be provided when available.

Mr. Marvin Glicklich, 5220 N. Lake Drive, asked if there would be any specific body to oversee and enforce the maintenance of the proposed projects.

Manager Harrigan said that it was not recommended that that level of authority be given to the commission.

Mr. Roy Gust, 4810 N. Lake Drive, asked why revetments were needed and how financing could be arranged.

Mr. Kendziorski said that revetments are used to retard erosion from wave action and that they hold back materials used for fill.

Manager Harrigan said that based on a recent bond counsel opinion, it is legally possible to establish special assessment districts for erosion projects.

Trustee Hatfield said that it would require as many properties as possible to offset the legal costs of issuing bonds for these projects.

Mr. David Hoover, 5560 N. Lake Drive, asked what would be done about the drainage of runoff on top of bluffs, and about permanent access for machinery and materials during project constructions.

Mr. Kendziorski said that where surface runoff is affecting vegetation and instability of bluff slopes, drainage systems are recommended. He also replied that three specific areas have been chosen as access areas for machinery and materials, those being: Buckley Park, Klode Park and the east end of Silver Spring Drive.

Mrs. Arlene Stern, 4800 N. Lake Drive, asked if anyone was available to evaluate work that has already been done, or any proposed work. Was anyone available to coordinate the various plans, and what is the timetable of these discussions?

Manager Harrigan said that there is a growing awareness of the need for the total impact of individual projects to be analyzed. He said that the Village and SEWRPC would be available to advise individuals on how any proposed project would relate to the plan.

Lucia Petrie, Chairman of the Advisory Committee for the Northern Milwaukee County Shoreline Erosion Management Study, said that hearings have to be scheduled for the City and County of Milwaukee sometime this summer. Once hearings are completed a formal recommendation will be given to each community.

Mr. Gary Sanderson, 5005 N. Palisades, said he thinks the proposal is a good idea, and he asked if citizens could have input into the plan.

Manager Harrigan said that specific ideas should be discussed between neighbors and then submitted to the Technical Advisory Committee.

Mr. Frank Banholzer, 4910 N. Lake Drive, commended the work of the Committee and stated that although he is not generally in favor of a strong role for government, he felt consideration should be given to the problems of the single property owner who is in a position to block a number of neighbors from the proper protection of their property.

Mr. Derrick Andrade, 4039 N. Richland Court, Shorewood, said that erosion takes place at any lake level. He gave several suggestions for erosion control.

Mr. Lee Clark, 4700 N. Lake Drive, asked if there has been any study of erosion from groundwater damage.

Mr. Biebel said there had been, and specifics would be included in the detailed report which is being developed.

Mr. David Moss, 5770 N. Shore Drive, asked how to get a recalcitrant homeowner to comply with the majority.

Manager Harrigan said that the local village boards must make a decision on a case-by-case basis. No policy decision has been made.

Penny Podell, Milwaukee County Supervisor, commended Village Manager Michael Harrigan on an outstanding job on the shoreline erosion committee.

There being no further comments, President Matthews thanked all those who attended this meeting, and the public hearing closed at 9:15 p.m.

Respectfully submitted,

Barbara Patin
Clerk-Treasurer

STUDY STAFF

SOUTHEASTERN WISCONSIN
REGIONAL PLANNING COMMISSION

Kurt W. Bauer P.E.	Executive Director
Robert P. Biebel P.E.	Chief Environmental Engineer
David B. Kendziorski	Principal Planner
Judy K. Musich	Planner

UNIVERSITY OF WISCONSIN-MADISON

Peter J. Bosscher, P.E.	Associate Professor of Environmental Engineering
Tuncer B. Edil, P.E.	Professor of Civil and Environmental Engineering
David M. Mickelson	Professor of Geology *and Geophysics*

UNIVERSITY OF WISCONSIN-MILWAUKEE

William F. Kean	Professor of Geological Sciences

WARZYN ENGINEERING, INC.

Patrick J. Harrison, P.E.	General Manager
Cyrus W. Ingraham, P.E.	Senior Engineer

W. F. BAIRD & ASSOCIATES, LTD.

William F. Baird, P.E.	President
Kevin J. MacIntosh, P.E.	Senior Engineer

JOHNSON, JOHNSON & ROY, INC.

Kenneth M. Cobb	Principal
Fredrick A. Klancnik, P.E.	Principal

PTL-INSPECTORATE, INC.

John M. Murphy	Geotechnical Services Division Manager
Scott J. Tromp	Chief Driller

Special acknowledgement is due the private consultants, contractors, and lakefront property owners who supplied valuable information for the study, and a special thanks to those property owners who allowed the Commission to take soil borings on their property. Aerial photography for the study by Robert T. McCoy.