COMMUNITY ASSISTANCE
PLANNING REPORT NO. 152

A STORMWATER DRAINAGE AND FLOOD CONTROL SYSTEM PLAN FOR THE MILWAUKEE METROPOLITAN SEWERAGE DISTRICT

Part 1 of 2

CHAPTERS 1-10

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Special acknowledgement is due Mr. Gary A. Gagnon, Group Manager, Planning and Engineering Support, Milwaukee Metropolitan Sewerage District, for his contribution to the preparation of this report.

*The system planning was initiated in 1986 during Mr. Marchese's tenure as Executive Director of the Milwaukee Metropolitan Sewerage Commission. Mr. White succeeded Mr. Marchese in 1989 as the work of the Advisory Committee was in the final stages of completion.

COMMUNITY ASSISTANCE PLANNING REPORT
NUMBER 152

A STORMWATER DRAINAGE AND FLOOD CONTROL SYSTEM PLAN FOR THE MILWAUKEE METROPOLITAN SEWERAGE DISTRICT

Prepared by the

Southeastern Wisconsin Regional Planning Commission
P. O. Box 1607
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December 1990

Inside Region $25.00
Outside Region $50.00

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SOUTHEASTERN WISCONSIN REGIONAL PLANNING COMMISSION

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December 7, 1990

Mr. Wallace White, Executive Director
Milwaukee Metropolitan Sewerage District
260 W. Seeboth Street
Milwaukee, Wisconsin 53204

Dear Mr. White:

In accordance with the provisions of the contractual agreement entered into between the Milwaukee Metropolitan Sewerage District and the Regional Planning Commission, on April 25, 1985, the Commission has completed and is providing to you herewith a stormwater drainage and flood control system plan for the District. The comprehensive stormwater drainage and flood control plan provided for in the aforereferenced agreement consists of two parts, a policy plan and a system plan. The policy plan, set forth in SEWRPC Community Assistance Planning Report No. 130, was completed in 1986 and adopted by the Milwaukee Metropolitan Sewerage District. It identifies those streams and watercourses for which the District, as an areawide agency, should assume jurisdiction; identifies the types of improvements for which the District should assume responsibility; and sets forth the manner in which improvement costs are to be shared between the District and benefited local municipalities.

This report presents the companion system plan for stormwater drainage and flood control. The system plan was prepared under the guidance of an Advisory Committee. The membership of the Committee included knowledgeable County and local officials, representatives of the Wisconsin Department of Natural Resources and the District, and concerned citizens. Alternative and recommended flood control and related drainage system plans are presented for the streams proposed to be under the jurisdiction of the District for flood control purposes in the Kinnickinnic River, Lake Michigan Direct Drainage Area, Oak Creek, Root River, Milwaukee River, and Menomonee River watersheds.

The Regional Planning Commission is appreciative of the assistance offered by the technical staffs of the District and of the cities and villages concerned in the preparation of this report. The Advisory Committee efforts are particularly acknowledged and appreciated. The Commission and Commission staff stand ready to assist the District in considering adoption of, and in administering, over time, this system plan.

Sincerely,

Kurt W. Bauer
Executive Director

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Chapter I

INTRODUCTION

The Milwaukee Metropolitan Sewerage District is charged by Section 66.89 of the Wisconsin Statutes with the function and duty of planning, designing, constructing, maintaining, and operating a sewerage system for the collection, transmission, and disposal of all sewage and drainage generated within its service area. Specifically, that function and duty includes the provision and management of a system of facilities for the collection, transmission, and disposal of stormwater and groundwater, as well as of sanitary sewage. The District is accordingly authorized to plan, design, construct, maintain, and operate storm sewers and other facilities and structures for the collection and transmission of stormwater and is authorized to improve watercourses within the District by deepening or widening or other changes needed to carry off surface or drainage waters.[1] The District is also authorized to make such improvements outside the geographic limits of the District on any watercourses that flow out of the District, and may divert stormwater from surface watercourses into drains, conduits, and storm sewers. Sound public administration, as well as good planning and engineering practice, dictates that these broad responsibilities for stormwater management be carried out within explicit policy guidelines set forth by the governing body of the District, as well as within the context of a comprehensive stormwater drainage and flood control system plan consistent with those policies.

Recognizing the need for both a policy plan and a system plan that could be used to guide the development over time of drainage and flood control facilities within the greater Milwaukee area, the Milwaukee Metropolitan Sewerage District on January 25, 1985, requested the Southeastern Wisconsin Regional Planning Commission to prepare, in cooperation with the District, a comprehensive stormwater drainage and flood control plan. That plan was to consist of two elements—a policy plan and a system plan. In response to that request, the Commission prepared a prospectus documenting the need for the requested two-part plan, specifying the scope and content of that plan, and identifying the work required to produce the plan, together with means for funding and accomplishing that work.[2] Based upon that prospectus, a contract governing the preparation of the desired policy and system plans was entered into between the District and the Commission on April 22, 1985.

The policy plan was completed by the Regional Planning Commission in conjunction with the Milwaukee Metropolitan Sewerage District on March 21, 1986,[3] and unanimously adopted by the governing body of the Milwaukee Metropolitan Sewerage District on June 19, 1986. On July 30, 1986, the policy plan was transmitted by the Regional Planning Commission on behalf of the District to the governing bodies of Milwaukee County and the municipalities within the District and the District contract service area with a request that these governing bodies adopt the policy plan.

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[1] *Implementation of certain drainage and flood control improvements within the existing geographical jurisdiction of the Milwaukee Metropolitan Sewerage District may require the prior approval of certain regulatory agencies, including the Wisconsin Department of Natural Resources and the U. S. Army Corps of Engineers. The regulatory process involved is complex and has been the subject of extended discussion between the District and the regulatory agencies concerned. Accordingly, the District should seek legal counsel prior to proceeding with any drainage or flood control project that involves the construction or hydraulic improvement of artificial waterways connecting to navigable waters; the alteration or enclosure of navigable waterways; the placement of deposits or structures in the bed of navigable waterways; the removal of material from the beds of navigable waterways; or the filling of wetlands.*

[2] *Stormwater Drainage and Flood Control Planning Program Prospectus for the Milwaukee Metropolitan Sewerage District, SEWRPC, March 1985.*

[3] *The policy plan is documented in SEWRPC Community Assistance Planning Report No. 130, A Stormwater Drainage and Flood Control Policy Plan for the Milwaukee Metropolitan Sewerage District, March 1986.*

As of June 1988, the Milwaukee County Board of Supervisors and nine of the 29 cities and villages concerned had adopted the policy plan.[4] Adoption of the policy plan permitted work to proceed on the preparation of the required system plan.

NEED FOR A STORMWATER MANAGEMENT PLAN

Stormwater drainage and flood control facilities are among the most important of public works influencing the development of an urbanizing region. The location and adequacy of these facilities affect the public health, safety, and welfare; the overall quality of the environment; recreational activities; industrial productivity; and the value and use to which land may be put, and therefore property values. If not properly attended to, stormwater drainage and flood control system development will inevitably emerge as a major obstacle to the sound growth and development of an area and become a major issue facing public officials, citizen leaders, and technicians.

Stormwater drainage and flood control planning efforts of various types are not new to the geographic area served by the Milwaukee Metropolitan Sewerage District. Such studies have been carried out at various times in the past by many of the 18 incorporated municipalities which comprise the District, as well as by Milwaukee County and the District and its predecessor agencies. Importantly, as of 1988 the Southeastern Wisconsin Regional Planning Commission had completed comprehensive watershed plans for the five major watersheds which lie wholly or partly within the District, as shown on Map 1. These watershed plans identify the flooding and water pollution problems existing within each watershed and make recommendations for the resolution of these problems. These watershed plans, which are documented in a series of planning reports, were prepared over an almost 20-year period, with the first of such plans, that for the Root River watershed, having been completed in 1966 and the last in 1986.[5] These watershed plans address flooding as opposed to drainage problems.[6] Nevertheless, if supplemented as necessary to address drainage as well as flood problems, and if integrated over the geographic area of the District, these watershed plans provide a sound basis for the development of a comprehensive stormwater drainage and flood control plan for the District.

The completed watershed studies document potential monetary flood damages along perennial streams within the Milwaukee Metropolitan Sewerage District and environs totaling about $33.3

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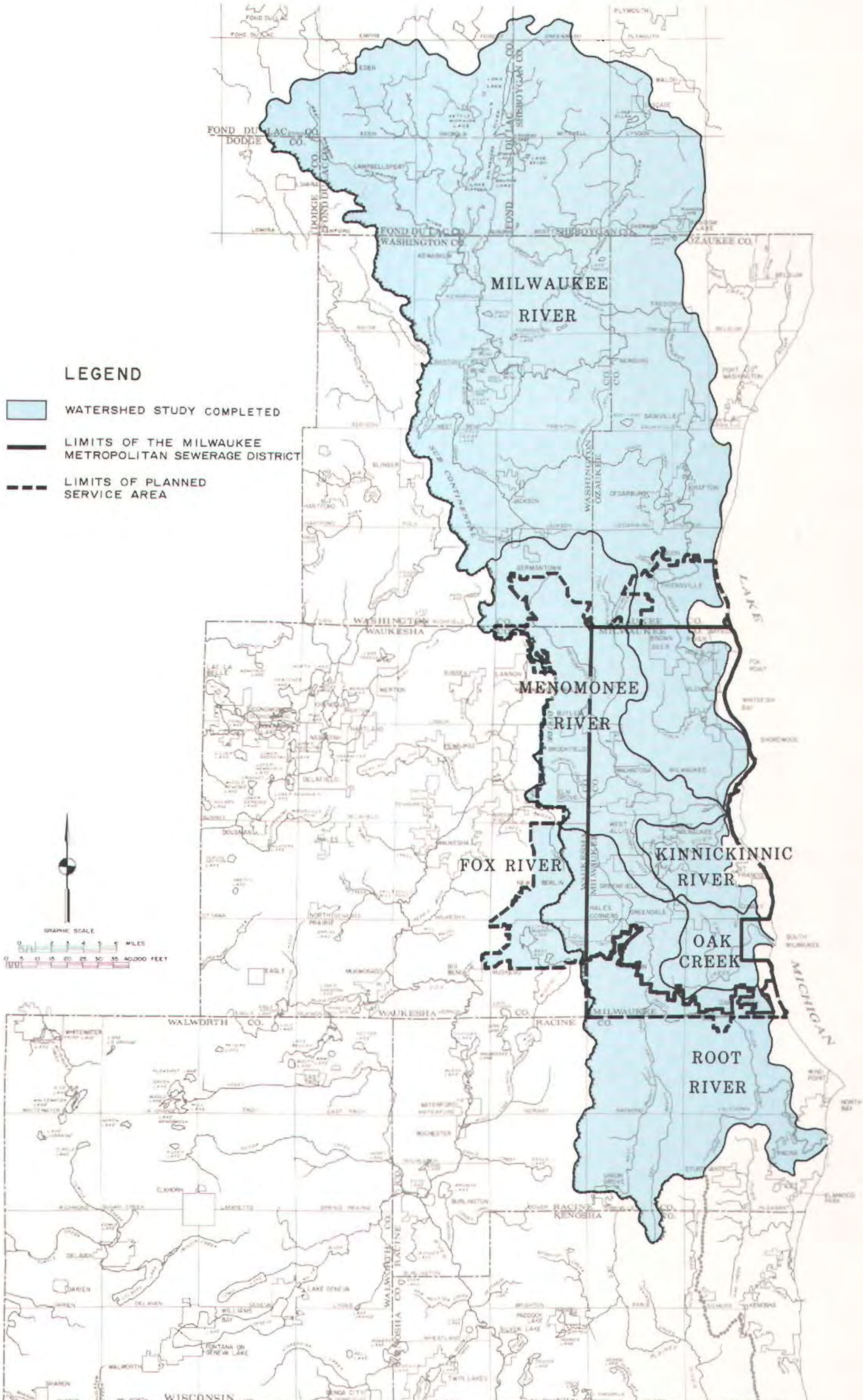
[4] *The municipal plan adoption actions as of the end of 1986 were as follows: City of Greenfield, December 3, 1986; City of Franklin, September 2, 1986; City of Wauwatosa, September 17, 1986; City of Milwaukee, September 23, 1986; City of West Allis, November 4, 1986; Village of Brown Deer, October 6, 1986; Village of Shorewood, October 6, 1986; Village of River Hills, October 16, 1986; Milwaukee County, September 11, 1986; City of Oak Creek, July 7, 1987.*

[5] *SEWRPC Planning Report No. 9, A Comprehensive Plan for the Root River Watershed, July 1966; SEWRPC Planning Report No. 13, A Comprehensive Plan for the Milwaukee River Watershed, Volume One, Inventory Findings and Forecasts, December 1970, and Volume Two, Alternative Plans and Recommended Plan, October 1971; SEWRPC Planning Report No. 26, A Comprehensive Plan for the Menomonee River Watershed, Volume One, Inventory Findings and Forecasts, October 1976, and Volume Two, Alternative Plans and Recommended Plan, October 1976; SEWRPC Planning Report No. 32, A Comprehensive Plan for the Kinnickinnic River Watershed, December 1978; SEWRPC Planning Report No. 36, A Comprehensive Plan for the Oak Creek Watershed, August 1986; and SEWRPC Community Assistance Planning Report No. 13 (2nd Edition), Flood Control Plan for Lincoln Creek, Milwaukee County, Wisconsin, September 1982.*

[6] *The Regional Planning Commission has, for planning and engineering purposes, differentiated between flooding and stormwater drainage problems. Flooding problems have been defined as caused by the inundation of the natural floodlands of a watershed that occurs along the major river and stream channels as a direct result of water moving out of, and away from, those channels. Stormwater drainage problems have been defined as resulting from inundation that occurs when stormwater runoff moving toward rivers and streams and other low-lying areas of a watershed encounters inadequate conveyance or storage facilities and results in localized ponding and surcharging of natural watersheds and artificial storm sewers. Different techniques are thus required to define and address these two problems.*

Map 1

STATUS OF WATERSHED PLANNING WITHIN THE MILWAUKEE METROPOLITAN SEWERAGE DISTRICT AND ENVIRONS



Source: SEWRPC.

million for a 100-year recurrence interval flood event and almost $2.5 million on an average annual basis expressed in 1985 dollars. The major flood-prone reaches, as shown on Map 2, include, among others, reaches of the Root, Milwaukee, and Menomonee Rivers and Oak Creek, Underwood Creek, Wilson Park Creek, and Lincoln Creek. These damages affect literally thousands of residences, businesses, and industries, as well as public buildings and facilities, and are accompanied by severe public safety and health hazards. These damages are, moreover, attributable solely to flooding, as defined by the Regional Planning Commission, and exclude damages caused by inadequate drainage or by the surcharging of sanitary sewers. Clearly, drainage and flood control problems within the District are real, costly, and well documented, and deserve resolution by the District and the local municipalities concerned.

In addition to the serious and costly flood problems that exist within the District, at least five other factors contribute to the need for the preparation of a stormwater drainage and flood control plan for the District at this time. These are:

1. The need to review, update, and integrate into a single policy and system plan and plan implementation program the flood control recommendations contained in the comprehensive watershed plans completed for the five watersheds lying wholly or partly within the Milwaukee Metropolitan Sewerage District.

   Such review, reevaluation, and integration is required in order to determine whether the flood control recommendations contained within the watershed plans are still valid, given changes which may have occurred since the adoption of some of the plans; to bring the costs and benefits to a common base year; and, importantly, to establish priorities for the needed projects between watersheds.

2. The need to expand the scope of the completed comprehensive watershed plans to include consideration of drainage as well as flooding problems, thereby more fully responding to the statutory functions and duties of the District.

3. The need to provide the Milwaukee Metropolitan Sewerage District, as an agency, with the documented stormwater drainage and flood control plan which good public administration and planning and engineering practice would dictate be available as a guide to District actions over time directed at the abatement of drainage and flood control problems.

   It is axiomatic that drainage and flood control facilities must function as integrated systems over entire watersheds, and that system plans are therefore required for the resolution of drainage and flooding problems. Since the Milwaukee Metropolitan Sewerage District encompasses a number of watersheds, however, it is evident that the proper execution of the District drainage and flood control responsibilities also requires the integration of the flood control recommendations contained in plans for the individual watersheds across the entire District.

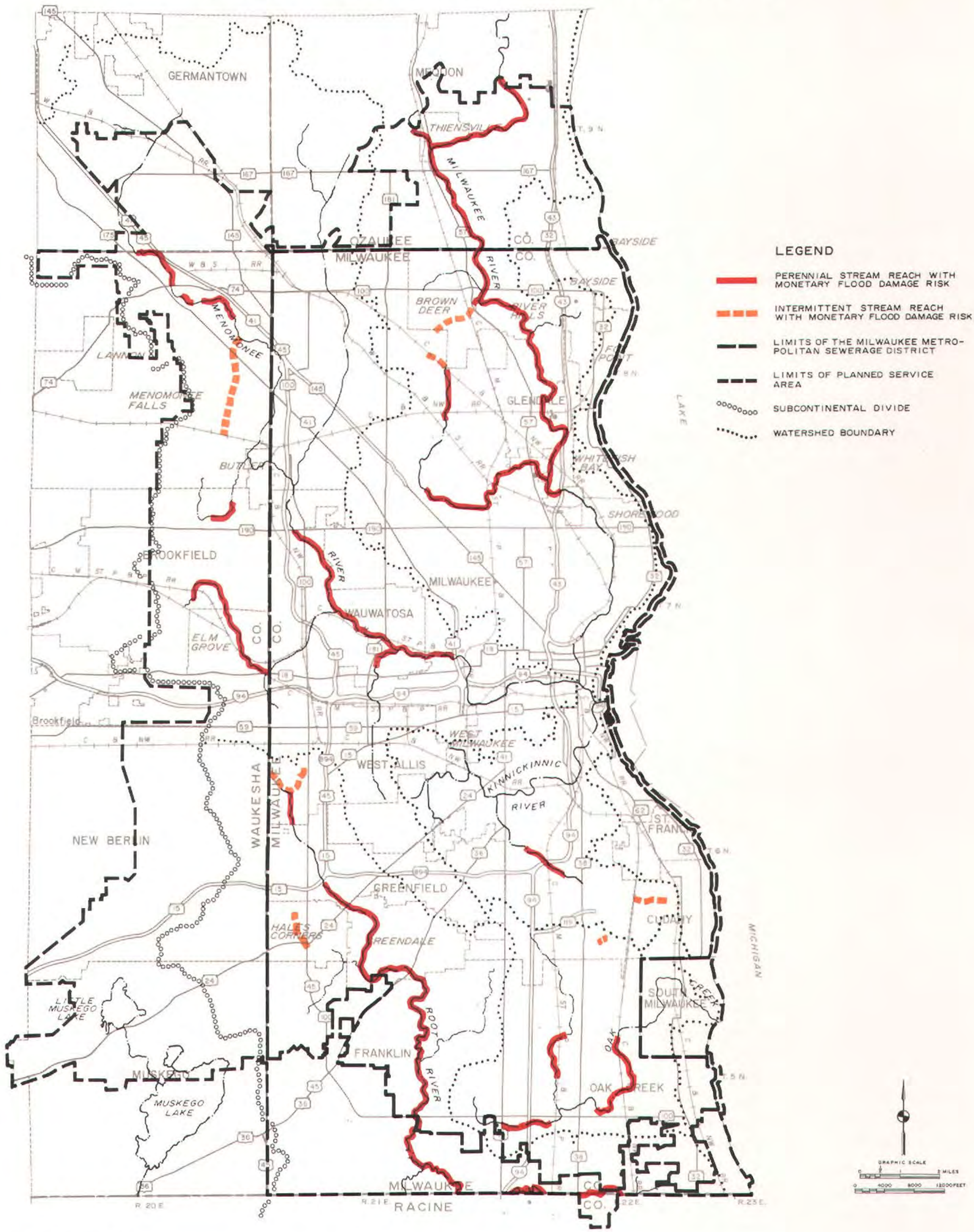
4. The need to provide an opportunity for the local municipalities comprising the Milwaukee Metropolitan Sewerage District and Milwaukee County, and for concerned citizens, to participate in the necessary policy and system plan formulation.

   As already noted, drainage and flooding problems are among the most serious and costly problems of concern to local units of government and affected citizens. Such problems not only result in property damage and disruption of socioeconomic activities, but may constitute serious threats to public health and safety. Such problems, moreover, affect the development potential of real property, and therefore property values. Accordingly, the local municipalities and individual citizens affected by and concerned about these problems should be afforded an opportunity to guide the formulation of a District drainage and flood control policy and system plan. Only if a true consensus is achieved on the location, extent, and severity of the problems, and on the most effective solutions thereto, can a plan be said to exist within the District.

5. The need to integrate surface water objectives and supporting water quality standards with drainage and flood control recommendations.

Map 2

MAJOR FLOOD DAMAGE PRONE STREAM REACHES WITHIN THE MILWAUKEE METROPOLITAN SEWERAGE DISTRICT AND ENVIRONS



Source: SEWRPC.

Recent studies conducted by the Regional Planning Commission and the Wisconsin Department of Natural Resources have established water use objectives and supporting water quality standards for all of the streams considered under the system planning effort. It is important to consider these established water use objectives when examining alternative stormwater drainage and flood control measures. The evaluation of such alternative measures should consider how those measures would contribute to meeting the established water use objectives and drainage and flood control objectives.

PURPOSE OF A STORMWATER DRAINAGE AND FLOOD CONTROL PLAN

The primary purpose of the District drainage and flood control planning program is the development of two separate but interrelated plans to guide the staged development of needed drainage and flood control facilities within the District, while promoting implementation of adopted local and areawide land use plans and assuring the protection and wise use of the natural resource base. The resulting plans are intended to provide the responsible public officials with technically sound guides that can be used in the making of decisions concerning the need for, most effective means of, and desirable scheduling of the construction of needed drainage and flood control works. More specifically, the plans would:

1. Identify those streams and watercourses for which the Milwaukee Metropolitan Sewerage District should assume jurisdiction for the resolution of drainage and flood control problems.[7]

2. Provide the technical staffs concerned with a complete and definitive inventory of the location and capacity of all of the streams and watercourses for which the District should assume jurisdiction. This inventory would provide the data on the physical characteristics of the drainage structures and intervening stream reaches necessary to permit calculation of flood flows and stages and channel capacities, identification of reaches of inadequate capacity, and identification of the causes of those inadequacies.

3. Provide elected and appointed public officials and concerned citizens with accurate information on the existing and probable future drainage and flood control problems within the District; on their locations, extent, and severity; and on the most effective means for their resolution.

As already noted, the prospectus specifies that the drainage and flood control plan is to consist of two elements—a policy plan and a system plan. The system plan is to identify the type, general location, and horizontal and vertical alignments of needed drainage and flood control facilities. To this end, the system plan as set forth herein recommends the approximate elevation, size, grade, and capacity of channels and appurtenant bridge waterway openings, major storm sewers, detention and retention basins, pumping stations, and other appurtenances of areawide significance, and provides data on flood stages under existing and planned conditions and on any hazards to public health and safety as may be required for sound decisions concerning land use development and redevelopment relating to floodplains and floodways. The system plan is in sufficient depth to provide a sound basis for local flood control planning and design, as well as for proceeding with final engineering for the watercourse and major drainage projects recommended to be constructed by the District. Particularly careful attention has been given in the system planning to the provision of needed outlets for existing and committed local drainage facilities. The system plan identifies the costs and benefits of the recommended improvements and identifies an order of priority and a schedule for their construction over time, constituting, in effect, a capital improvements program for areawide drainage and flood control works within the District and District contract service areas. In addition, the system plan is intended to provide planning and engineering data useful in local drainage system and facility planning and design, and in the resolution of local drainage problems. This report is intended to document the system plan element of the overall drainage and flood control plan for the greater Milwaukee area.

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[7]*It is recognized that, given the State Statutes governing the operation of the District, the term "jurisdiction" may have certain legal implications. Within the context of this policy and system plan, however, the term is defined to mean those streams and watercourses for which the District is recommended to act as the primary management agency with respect to the construction and maintenance of needed drainage and flood control works.*

For purposes of the system planning, committed local drainage facilities were defined to include: 1) proposed facilities for which outlets have been constructed at elevations which require the related proposed upstream facilities for a properly functioning system; 2) proposed facilities for which tributary upstream facilities have been constructed which require the related proposed downstream facilities for a properly functioning system; and 3) proposed facilities documented in a locally prepared stormwater management plan which has been formally adopted by the local government or governments concerned.

The policy plan provides an important basis for the preparation of the system plan. The policy plan 1) recommends those streams and watercourses for which the District, as an areawide agency, should assume jurisdiction; 2) recommends the types of improvements for which the District should assume responsibility; and 3) recommends the manner in which improvement and maintenance costs are to be shared between the District and the benefited local municipalities. The policy plan also provides a basis for prioritizing and scheduling, as a part of the system plan, the needed drainage and flood control improvements to be constructed by the District.

STAFF AND COMMITTEE STRUCTURE

These policy and system plans were prepared by the staffs of the Regional Planning Commission and Milwaukee Metropolitan Sewerage District working under the guidance of an Advisory Committee created for this purpose. This Committee, appointed jointly by the Commission and the District, includes representatives of the Cities of Milwaukee, Wauwatosa, and West Allis; the "North Shore" suburban units of government in Milwaukee County; the "South Shore" suburban units of government in Milwaukee County; the County; the Wisconsin Department of Natural Resources; and the District and the Regional Planning Commission; as well as three citizen members knowledgeable and concerned about drainage and flood control problems and related environmental problems. The basic purpose of the Advisory Committee was to actively involve the various units and agencies of government concerned, as well as citizen interests, in the drainage and flood control planning process, placing the knowledge and experience of the Committee members at the disposal of the study and, to the extent practicable, ensuring intergovernmental agreement on policy and system plan recommendations. The full membership of the Advisory Committee is set forth on the inside front cover of this report.

SCHEME OF PRESENTATION

The findings and recommendations of the system planning effort of the overall drainage and flood control planning effort are documented and presented in this report. In addition to this introductory chapter, this report consists of 10 chapters. Chapter II describes the study area to be considered and documents the basic inventories, water use objectives, and forecasts relating to those natural and man-made features most directly related to stormwater drainage and flood control. Chapter III sets forth a set of objectives, principles, and standards, as well as design criteria, to be used in the development and evaluation of alternative flood control system plans. Following Chapter III, there are six chapters relating the plan to each of the major watersheds within the study area—one chapter for each watershed. Each chapter identifies the existing and probable future flooding and related drainage problems; describes alternative and recommended flood control and related drainage improvement measures and associated costs and benefits; and recommends the means for implementing the recommended measures. Chapter X presents an integrated description of the recommended system plans described in the six chapters addressing the individual watersheds, and sets forth a priority schedule for implementing the system plan. The eleventh and final chapter summarizes the findings and recommendations of the system planning effort. This report is intended to allow careful, critical review of the alternative plan elements by public officials, agency staff personnel, and citizen leaders within the Milwaukee Metropolitan Sewerage District and planned District service area, and to provide the basis for plan adoption and implementation by the federal, state, and local agencies of government concerned.

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Chapter II

DESCRIPTION OF THE STUDY AREA: MAN-MADE FEATURES AND THE NATURAL RESOURCE BASE

INTRODUCTION

The planning and design of urban stormwater drainage and flood control systems requires knowledge about, and consideration of, certain man-made and natural features of the area to be served. The type, density, and spatial distribution of land use are important determinants of the rate and volume of stormwater runoff. The demography and the economy of the service area not only are determinants of the land use pattern, but are important in any consideration of the means of funding needed facilities. Stormwater drainage and flood control facilities also have direct and indirect impacts on the land use pattern of an area and on the resident population and economy. The climate and weather, topography, soils, vegetation, wetlands, and surface water features of the service area are also important determinants of the rate and volume of stormwater runoff. The land use pattern is supported and influenced by utility and transportation facilities. These natural features, together with others, such as the quality of the surface waters, must also be carefully considered to ensure that the proposed drainage and flood control facilities will not unnecessarily and adversely affect these invaluable resources and thereby affect the quality of life. Accordingly, this chapter describes those man-made and natural features of the planning area that affect and are affected by stormwater drainage and flood control facilities.

STUDY AREA

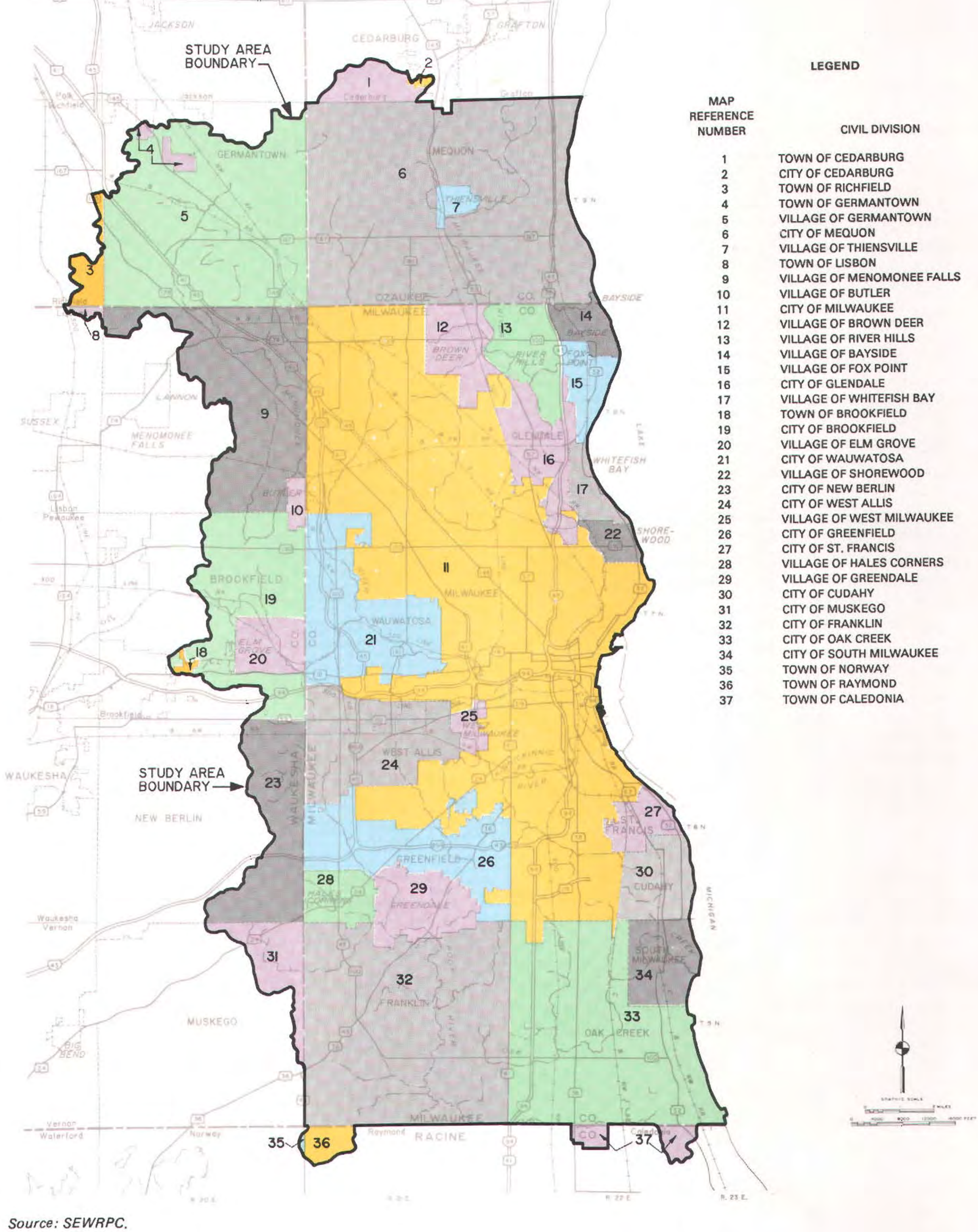
Stormwater drainage and flood control facilities must function as an integrated system over entire watersheds. Land use patterns which determine the amount and spatial distribution of the hydraulic loadings to be accommodated by such facilities, however, develop over an entire urban region in response to basic social and economic forces and to the operation of the urban land market without regard to either natural watershed boundaries or artificial county and municipal corporate limit lines. The stormwater drainage and flood control facilities, in turn, determine to an important degree the use to which land may be put, particularly in riverine areas. These facilities often cross corporate limit lines, but generally do not cross watershed boundaries. Thus, stormwater drainage and flood control planning cannot be accomplished successfully within the context of a single municipality or county if that municipality or county is part of a larger urban complex, nor can such planning be accomplished successfully solely within natural watershed areas. Rather, such planning must be accomplished on the basis of a geographic area which recognizes the configuration of the natural watersheds, including at a minimum, with respect to this study, the tributary drainage areas of those streams for which the Milwaukee Metropolitan Sewerage District has assumed jurisdiction; the major factors which influence the pattern of urban development in the greater Milwaukee area; and the legal and institutional factors that affect the development of drainage and flood control works of areawide significance.

The geographic area delineated for drainage and flood control system planning in the greater Milwaukee area under the study is shown on Map 3. This area includes the Milwaukee Metropolitan Sewerage District; the balance of land in Milwaukee County not currently in the District, namely the City of South Milwaukee and the southern portions of the Cities of Franklin and Oak Creek; all of the City of Mequon and the Village of Thiensville and those portions of the Town and City of Cedarburg that are tributary to Pigeon Creek in Ozaukee County; those portions of the Village of Germantown and the Towns of Germantown and Richfield that are tributary to the Menomonee River in Washington County; all land east of the subcontinental divide in Waukesha County, including all of the Villages of Butler and Elm Grove, and portions of the Cities of Brookfield, Muskego, and New Berlin, the Village of Menomonee Falls, and the Towns of Brookfield and Lisbon; and portions of the Towns of Caledonia, Norway, and Raymond in Racine County, which either are part of the District's contract service area or have lands tributary to streams recommended for District jurisdiction.

The combined study area is about 379 square miles in extent; occupies portions of five counties in the Southeastern Wisconsin Region—Milwaukee, Ozaukee, Racine, Washington, and Waukesha—and encompasses all or portions of 15 cities, 14 villages,

Map 3

CIVIL DIVISIONS IN THE STUDY AREA: 1985



LEGEND

MAP REFERENCE NUMBER	CIVIL DIVISION
1	TOWN OF CEDARBURG
2	CITY OF CEDARBURG
3	TOWN OF RICHFIELD
4	TOWN OF GERMANTOWN
5	VILLAGE OF GERMANTOWN
6	CITY OF MEQUON
7	VILLAGE OF THIENSVILLE
8	TOWN OF LISBON
9	VILLAGE OF MENOMONEE FALLS
10	VILLAGE OF BUTLER
11	CITY OF MILWAUKEE
12	VILLAGE OF BROWN DEER
13	VILLAGE OF RIVER HILLS
14	VILLAGE OF BAYSIDE
15	VILLAGE OF FOX POINT
16	CITY OF GLENDALE
17	VILLAGE OF WHITEFISH BAY
18	TOWN OF BROOKFIELD
19	CITY OF BROOKFIELD
20	VILLAGE OF ELM GROVE
21	CITY OF WAUWATOSA
22	VILLAGE OF SHOREWOOD
23	CITY OF NEW BERLIN
24	CITY OF WEST ALLIS
25	VILLAGE OF WEST MILWAUKEE
26	CITY OF GREENFIELD
27	CITY OF ST. FRANCIS
28	VILLAGE OF HALES CORNERS
29	VILLAGE OF GREENDALE
30	CITY OF CUDAHY
31	CITY OF MUSKEGO
32	CITY OF FRANKLIN
33	CITY OF OAK CREEK
34	CITY OF SOUTH MILWAUKEE
35	TOWN OF NORWAY
36	TOWN OF RAYMOND
37	TOWN OF CALEDONIA

Source: SEWRPC.

and 8 towns. The area of each of these civil divisions, and the portion of the total study area lying within these civil divisions, are set forth in Table 1. Geographic boundaries of the civil divisions are an important consideration, since the civil divisions form the basic foundation of the public decision-making framework within which stormwater and flood control problems must be addressed.

MAN-MADE FEATURES

Man-made features that are important to any stormwater drainage and flood control planning effort include land use patterns, public utility networks, and transportation systems. Together with the population and economic activities of the study area, these features may be thought of as the socioeconomic base of the study area. A description of the socioeconomic base of the study area is herein presented in four sections. The first section describes the demographic and economic base of the study area in terms of the population size, distribution, and density, selected characteristics of the population, and employment levels and distribution. The second section describes the land use patterns in the study area in terms of historical development and current conditions. The third and fourth sections describe the public utility and transportation facility systems within the study area.

Demographic and Economic Base

There is a direct relationship between the resident population of a study area and the demand for land, water, and other important elements of the natural resource base, as well as the demand for transportation, utility, recreation, and other community services and facilities, including drainage and flood control facilities. Thus, an understanding of the size, spatial distribution, and characteristics of this population is essential for a sound, comprehensive planning effort. The size and characteristics of the population of an area are greatly influenced by growth and other changes in economic activity. Population levels and economic activities must therefore be considered together.

Population Size: The resident population of the study area in 1985 was estimated at 1,042,600 persons, or about 60 percent of the estimated 1,742,700 persons then residing within the seven-county Southeastern Wisconsin Region. As shown in Figure 1 and Table 2, the study area exhibited significant increases in population from 1950 to 1970 and a marked decline in resident population from 1970 through 1985. The resident population of the study area increased by about 208,600 persons, or over 23 percent, between 1950 and 1960, and by almost 50,000 persons, or about 5 percent, between 1960 and 1970. Over the 10-year from 1970 to 1980, the study area population decreased by about 85,000 persons, or about 7.4 percent—from 1,151,600 in 1970 to about 1,066,400 persons in 1980. The population continued to decline from 1980 to 1985, decreasing by about 24,000 persons, or 2.2 percent—from 1,066,400 in 1980 to 1,042,600 in 1985. The relative gain in population in the study area from 1950 to 1960—23.4 percent—was significantly higher than in the State of Wisconsin—15.1 percent—or the United States—18.5 percent—but somewhat lower than in the Region—26.8 percent—during the same time period. The rate of population increase in the study area from 1960 to 1970—4.5 percent—was lower than in the Region, the State, and the United States, which experienced gains of 11.6 percent, 11.8 percent, and 13.4 percent, respectively. Finally, while the study area and Region experienced population declines from 1970 to 1985, the State of Wisconsin and the United States continued to experience population increases, although at a lower rate than during previous decades.

Population Distribution: The distribution of the resident population of the study area by civil division for the years 1950, 1960, 1970, 1980, and 1985 is also presented in Table 2. As indicated in this table, only three civil divisions experienced population declines from 1950 to 1960, and four civil divisions from 1960 to 1970. However, a majority of the civil divisions in the study area—23 of 37, or 62 percent—experienced population decreases from 1970 to 1980, and 25 of 37 civil divisions, or 67 percent, experienced population declines from 1980 to 1985. The largest absolute loss in population from 1970 to 1985 occurred in the City of Milwaukee, the population of which declined by about 105,000 persons—from 717,400 in 1970 to about 612,100 in 1985. The largest absolute gain during this time period occurred in the City of Greenfield, the population of which increased by over 7,600 persons—from about 24,400 in 1970 to about 32,100 in 1985.

The average population density by civil division within the study area for the year 1985 is shown in Table 3. As indicated in this table, the average population density for the entire study area was 2,753 persons per square mile. The civil divisions in Milwaukee County exhibited the highest population densities of all the civil divisions in the study

Table 1

AREAL EXTENT OF CIVIL DIVISIONS IN THE STUDY AREA: 1985

County or Civil Division	Total County or Civil Division Area (square miles)	County or Civil Division Area Included Within Study Area (square miles)	Percent of County or Civil Division Area Within Study Area	Percent of Study Area Within County or Civil Division
Milwaukee County	242.44	242.44	100.0	64.1
Village of Bayside	2.38[a]	2.38[a]	100.0	0.6
Village of Brown Deer	4.37	4.37	100.0	1.2
City of Cudahy	4.80	4.80	100.0	1.3
Village of Fox Point	2.85	2.85	100.0	0.7
City of Franklin	34.69	34.69	100.0	9.2
City of Glendale	5.98	5.98	100.0	1.6
Village of Greendale	5.59	5.59	100.0	1.5
City of Greenfield	11.52	11.52	100.0	3.0
Village of Hales Corners	3.20	3.20	100.0	0.8
City of Milwaukee	96.67[b]	96.67[b]	100.0	25.5
City of Oak Creek	28.41	28.41	100.0	7.5
Village of River Hills	5.32	5.32	100.0	1.4
City of St. Francis	2.55	2.55	100.0	0.7
Village of Shorewood	1.50	1.50	100.0	0.4
City of South Milwaukee	4.82	4.82	100.0	1.3
City of Wauwatosa	13.24	13.24	100.0	3.5
City of West Allis	11.43	11.43	100.0	3.0
Village of West Milwaukee	1.12	1.12	100.0	0.3
Village of Whitefish Bay	2.11	2.11	100.0	0.6
Ozaukee County	235.08[a]	50.88[a]	21.6	13.4
City of Cedarburg	3.26	0.17	5.2	- -[c]
Town of Cedarburg	26.69	2.58	9.7	0.7
City of Mequon	47.00	47.00	100.0	12.4
Village of Thiensville	1.04	1.04	100.0	0.3
Racine County	340.47	3.06	0.9	0.8
Town of Caledonia	46.24	1.65	3.6	0.4
Town of Norway	35.68	0.07	0.2	- -[c]
Town of Raymond	35.75	1.34	3.7	0.4
Washington County	435.68[b]	31.55[b]	7.2	8.3
Village of Germantown	34.50	29.28	84.9	7.7
Town of Germantown	1.72	0.76	44.2	0.2
Town of Richfield	36.37	1.49	4.1	0.4
Waukesha County	580.60	50.72	8.7	13.4
City of Brookfield	26.30	13.59	51.7	3.6
Town of Brookfield	6.43	0.19	3.0	- -[c]
Village of Butler	0.80	0.80	100.0	0.2
Village of Elm Grove	3.25	3.25	100.0	0.9
Town of Lisbon	32.75	0.34	1.0	0.1
Village of Menomonee Falls	33.39	18.93	56.7	5.0
City of Muskego	36.01	3.90	10.8	1.0
City of New Berlin	36.85	9.72	26.4	2.6
Total	- -	378.65	- -	100.0

[a] *Includes that portion of the Village of Bayside in Ozaukee County.*

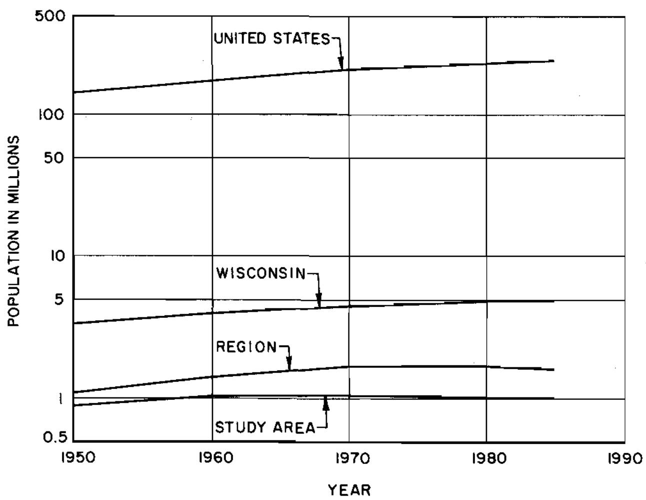
[b] *Includes that portion of the City of Milwaukee in Washington County.*

[c] *Less than 0.1 percent.*

Source: SEWRPC.

Figure 1

POPULATION OF THE STUDY AREA, THE REGION, WISCONSIN, AND THE UNITED STATES: 1950-1985



Source: U. S. Bureau of the Census, Wisconsin Department of Administration, and SEWRPC.

area, including the Village of Shorewood with a population density of almost 9,500 persons per square mile, the Village of Whitefish Bay with a population density of 6,760 persons per square mile, and the City of Milwaukee with a population density of about 6,330 persons per square mile. Population density in the study area is graphically represented on Map 4.

Population Characteristics: Table 4 shows the median household income by civil division in the study area for the years 1959 to 1979 as expressed in constant 1979 dollars. The median 1979 household income in the study area ranged from a low of $16,400 in the Village of West Milwaukee in Milwaukee County to a high of almost $49,000 in the Village of River Hills in Milwaukee County. While all of the civil divisions for which data are available experienced a gain in median household income from 1959 to 1969, 18 of the 37 civil divisions in the study area, or almost 50 percent, experienced declines in median household income from 1969 to 1979.

Table 5 indicates average household size by civil division in the study area for the years 1960, 1970, and 1980. It is interesting to note that with the exception of the Town of Germantown whose household size remained constant, every civil division in the study area experienced a decline in household size from 1960 to 1980. As of 1970, 10 civil divisions in the study area exhibited household sizes of greater than four persons per household, and only two civil divisions—the Villages of Shorewood and West Milwaukee in Milwaukee County—had average household sizes of fewer than three persons per household. As of 1980, one civil division in the study area—the Town of Germantown in Washington County—had a household size of four persons per household, while 15 civil divisions exhibited household sizes of fewer than three persons per household.

The median age of the resident population by civil division in the study area for the years 1960, 1970, and 1980 is presented in Table 6. A review of this table indicates that the population of the study area is aging. As of 1960, there were 27 civil divisions in the study area whose resident population had a median age of less than 30 years. In 1980, there were only six civil divisions in the study area whose population exhibited a median age of less than 30 years. Indeed, in 1980 three communities for the first time had a population whose median age was greater than 40 years—the Village of Fox Point in Milwaukee County, 41.0 years; the Village of Thiensville in Ozaukee County, 40.3 years; and the Village of Elm Grove in Waukesha County, 42.1 years.

Economic Base: Changes in the population of the study area are related to, among other factors, changes in economic activity within the study area. This is true not only because population migration patterns and trends within and between areas are dependent in part upon available job opportunities, but also because jobs must ultimately be available to sustain population increases due to natural increase and to prevent a forced out-migration of young residents entering the labor force.

The total employment in the study area by civil division for the years 1972, 1980, and 1985 is indicated in Table 7. As shown in this table, total jobs in the study area increased by more than 53,100, or about 10 percent—from about 538,700 in 1972 to about 591,800 in 1980. The City of Wauwatosa experienced the largest absolute increase in jobs over this time period, over 14,600, or 40 percent—from about 36,000 in 1972 to about 50,600 in 1980. The Village of West Milwaukee experienced the largest absolute decrease, almost 9,300 jobs, or 46 percent—from about 19,980 jobs in 1972 to about 10,700 jobs in 1980. The greatest percentage increase in jobs over this time period was exhibited by the City of Oak

Table 2

POPULATION OF THE STUDY AREA, THE REGION, WISCONSIN, AND THE UNITED STATES: 1950, 1960, 1970, 1980, AND 1985

Civil Division	1950	1960	Change 1950-60		1970	Change 1960-70		1980	Change 1970-80		1985	Change 1980-85	
			Absolute	Percent		Absolute	Percent		Absolute	Percent		Absolute	Percent
Milwaukee County													
Village of Bayside[a]	--	3,078[p]	--	--	4,461[p]	1,383	44.9	4,724[p]	263	5.9	4,689[p]	-35	-0.7
Village of Brown Deer[b]	--	11,280	--	--	12,582	1,302	11.5	12,921	339	2.7	12,599	-332	-2.5
City of Cudahy	12,182	17,975	5,793	47.6	22,078	4,103	22.8	19,547	-2,531	-11.5	19,042	-505	-2.6
Village of Fox Point	2,585	7,315	4,730	183.0	7,939	624	8.5	7,649	-290	-3.7	7,174	-475	-6.2
City of Franklin[c]	--	10,006	--	--	12,247	2,241	22.4	16,871	4,624	37.8	18,530	1,659	9.8
Town of Franklin[d]	3,886	--	--	--	--	--	--	--	--	--	--	--	--
City of Glendale[e]	--	9,537	--	--	13,426	3,889	40.8	13,882	456	3.4	13,625	-257	-1.9
Town of Granville[d]	11,784	--	--	--	--	--	--	--	--	--	--	--	--
Village of Greendale	2,752	6,843	4,091	148.7	15,089	8,246	120.5	16,928	1,839	12.2	16,770	-158	-0.9
City of Greenfield[f]	--	17,636	--	--	24,424	6,788	38.5	31,353	6,929	28.4	32,050	697	2.2
Town of Greenfield[d]	20,907	--	--	--	--	--	--	--	--	--	--	--	--
Village of Hales Corners[g]	--	5,549	--	--	7,771	2,222	40.0	7,110	-661	-8.5	6,842	-268	-3.8
Town of Lake[d]	18,956	--	--	--	--	--	--	--	--	--	--	--	--
City of Milwaukee	637,392	741,324	103,932	16.3	717,372[q]	-23,952	-3.2	636,297[q]	-81,075	-11.3	612,085[q]	-24,212	-3.8
Town of Milwaukee[d]	5,857	--	--	--	--	--	--	--	--	--	--	--	--
City of Oak Creek[h]	--	9,372	--	--	13,928	4,556	48.6	16,932	3,004	21.6	18,002	1,070	6.3
Town of Oak Creek[d]	4,807	--	--	--	--	--	--	--	--	--	--	--	--
Village of River Hills	567	1,257	690	121.7	1,561	304	24.2	1,642	81	5.2	1,639	-3	-0.2
City of St. Francis[i]	--	10,065	--	--	10,489	424	4.2	10,095	-394	-3.8	9,724	-371	-3.7
Village of Shorewood	16,199	15,990	-209	-1.3	15,576	-414	-2.6	14,327	-1,249	-8.0	14,247	-80	-0.6
City of South Milwaukee	12,855	20,307	7,452	58.0	23,297	2,990	14.7	21,069	-2,228	-9.6	20,512	-557	-2.6
City of Wauwatosa	33,324	56,923	23,599	70.8	58,676	1,753	3.1	51,308	-7,368	-12.6	50,234	-1,074	-2.1
Town of Wauwatosa[d]	23,941	--	--	--	--	--	--	--	--	--	--	--	--
City of West Allis	42,959	68,157	25,198	58.7	71,649	3,492	5.1	63,982	-7,667	-10.7	64,066	84	0.1
Village of West Milwaukee	5,429	5,043	-386	-7.1	4,405	-638	-12.7	3,535	-870	-19.8	3,595	60	1.7
Village of Whitefish Bay	14,665	18,390	3,725	25.4	17,402	-988	-5.4	14,930	-2,472	-14.2	14,264	-666	-4.5
Subtotal	871,047	1,036,047	165,000	18.9	1,054,372	18,325	1.8	965,102	-89,270	-8.5	939,689	-25,413	-2.6
Ozaukee County													
City of Cedarburg	55	222	167	303.6	540	318	143.2	429	-111	-20.6	358	-71	-16.6
Town of Cedarburg	180	376	196	108.9	928	552	146.8	817	-111	-12.0	787	-30	-3.7
City of Mequon[j]	--	8,543	--	--	12,150	3,607	42.2	16,193	4,043	33.3	16,003	-190	-1.2
Town of Mequon[j]	4,065	--	--	--	--	--	--	--	--	--	--	--	--
Village of Thiensville	897	2,507	1,610	179.5	3,182	675	26.9	3,341	159	5.0	3,104	-237	-7.1
Subtotal	5,197	11,648	6,451	124.1	16,800	5,152	44.2	20,780	3,980	23.7	20,252	-528	-2.5
Racine County													
Town of Caledonia	88	1,280	1,192	1,354.5	1,408	128	10.0	1,136	-272	-19.3	1,056	-80	-7.0
Town of Norway	37	44	7	18.9	69	25	56.8	56	-13	-18.8	52	-4	-7.1
Town of Raymond	98	136	38	38.8	350	214	157.4	306	-44	-12.6	272	-34	-11.1
Subtotal	223	1,460	1,237	554.7	1,827	367	25.1	1,498	-329	-18.0	1,380	-118	-7.9
Washington County													
Village of Germantown	2,173	4,275	2,102	96.7	5,713	1,438	33.6	10,260	4,547	79.6	11,434	1,174	11.4
Town of Germantown	225	311	86	38.2	357	46	14.8	106	-251	-70.3	119	13	12.3
Town of Richfield	61	94	33	54.1	248	154	163.8	376	128	51.6	408	32	8.5
Subtotal	2,459	4,680	2,221	90.3	6,318	1,638	35.0	10,742	4,424	70.0	11,961	1,219	11.3
Waukesha County													
City of Brookfield[k]	--	14,779	--	--	17,788	3,009	20.4	17,535	-253	-1.4	18,860	1,325	7.6
Town of Brookfield	6,399	74	-6,325	-98.8	318	244	329.7	139	-179	-56.3	139	0	0.0
Village of Butler	1,047	2,274	1,227	117.2	2,261	-13	-0.6	2,059	-202	-8.9	2,002	-57	-2.8
Village of Elm Grove[l]	--	4,994	--	--	7,201	2,207	44.2	6,735	-466	-6.5	6,239	-496	-7.4
Town of Lisbon	12	15	3	25.0	25	10	66.7	25	0	0.0	25	0	0.0
Town of Menomonee Falls[m]	2,860	--	--	--	--	--	--	--	--	--	--	--	--
Village of Menomonee Falls[m]	--	15,553	--	--	26,975	11,422	73.4	23,481	-3,494	-13.0	23,311	-170	-0.7
City of Muskego[n]	--	--	--	--	4,082	--	--	4,817	735	18.0	4,805	-12	-0.2
Town of Muskego[n]	251	3,104	2,853	113.7	--	--	--	--	--	--	--	--	--
City of New Berlin[o]	--	7,104	--	--	13,657	6,553	92.2	13,516	-141	-1.0	13,930	414	3.1
Town of New Berlin[o]	3,627	--	--	--	--	--	--	--	--	--	--	--	--
Subtotal	14,196	47,897	33,701	237.4	72,307	24,410	51.0	68,307	-4,000	-5.5	69,311	1,004	1.5
Study Area Total	893,122	1,101,732	208,610	23.4	1,151,624	49,892	4.5	1,066,429	-85,195	-7.4	1,042,593	-23,836	-2.2
Region Total	1,240,618	1,573,614	332,996	26.8	1,756,083	182,469	11.6	1,764,919	8,836	0.5	1,742,700	-22,219	-1.3
Wisconsin Total	3,434,575	3,951,777	517,202	15.1	4,417,821	466,044	11.8	4,705,767	287,946	6.5	4,779,021	73,254	1.6
United States Total	151,325,798	179,323,175	27,997,377	18.5	203,302,031	23,978,856	13.4	226,504,825	23,202,794	11.4	237,677,000	11,173,175	4.9

Footnotes to Table 2

[a]The Village of Bayside was incorporated in 1953.

[b]The Village of Brown Deer was incorporated in 1955.

[c]The City of Franklin was incorporated in 1956.

[d]Between 1950 and 1960, all remaining unincorporated territory in Milwaukee County became incorporated either through annexation to existing cities and villages or through direct incorporation, and the Towns of Franklin, Granville, Greenfield, Lake, Milwaukee, Oak Creek, and Wauwatosa ceased to exist.

[e]The City of Glendale was incorporated in 1950 after the conduct of the 1950 census.

[f]The City of Greenfield was incorporated in 1957.

[g]The Village of Hales Corners was incorporated in 1952.

[h]The City of Oak Creek was incorporated in 1955.

[i]The City of St. Francis was incorporated in 1951.

[j]In 1957, the remaining territory of the Town of Mequon was incorporated as the City of Mequon, and the Town of Mequon ceased to exist.

[k]The City of Brookfield was incorporated in 1954.

[l]The Village of Elm Grove was incorporated in 1955.

[m]Between 1950 and 1960, the remaining territory of the Town of Menomonee was annexed by the Village of Menomonee Falls, and the Town of Menomonee ceased to exist.

[n]In 1964, the Town of Muskego was incorporated as the City of Muskego, and the Town of Muskego ceased to exist.

[o]In 1959, the Town of New Berlin was incorporated as the City of New Berlin, and the Town of New Berlin ceased to exist.

[p]Between 1953 and 1960, the Village of Bayside annexed territory in Ozaukee County. The populations presented for the Village in 1960, 1970, 1980, and 1985 include the population of that portion of the Village in Ozaukee County.

[q]In 1963, the City of Milwaukee annexed territory in Washington County. The populations presented for the City in 1970, 1980, and 1985 include the population of that portion of the City in Washington County.

Source: U. S. Bureau of the Census, Wisconsin Department of Administration, and SEWRPC.

Creek, which experienced a 220 percent increase in jobs over this time period—from about 3,670 jobs in 1972 to over 11,900 jobs in 1980.

Total jobs in the study area increased by about 400, or less than 1 percent—from about 591,800 in 1980 to about 592,200 in 1985. The City of Brookfield experienced the largest absolute increase in jobs over this time period, about 4,000, or about 23 percent—from about 17,400 in 1980, to about 21,400 in 1985. The City of Milwaukee experienced the largest absolute decrease, almost 7,800 jobs, representing about a 2 percent decrease—from about 348,900 jobs in 1980 to about 341,100 jobs in 1985. The greatest percentage increase in jobs over this time period was exhibited by the Village of Bayside, which experienced an increase of about 340 jobs—from about 500 jobs in 1980 to about 840 jobs in 1985, an increase of about 67 percent.

Table 8 sets forth total employment by major industry group in the study area for the years 1972, 1980, and 1985. As indicated in this table, the "industrial" group provided the largest number of jobs in the study area in 1972—over 208,900 jobs, or almost 39 percent. Service jobs exhibited the largest absolute and percentage increase from 1972 to 1980, increasing by about 31,500 jobs, or over 21 percent—from 148,780 jobs in 1972 to about 180,300 jobs in 1980.

In 1985, the service group provided the largest number of jobs, over 199,100, or about 34 percent of the jobs in the study area. The service group also exhibited the largest absolute and percentage increases during the 1980 to 1985 time period, increasing by about 18,800 jobs, or about 10 percent, over the 180,300 such jobs in 1980. The industrial group exhibited the largest absolute decrease during the 1980 to 1985 time period, decreasing by about 27,700 jobs, or about 13 percent—from about 210,400 jobs in 1980 to about 182,700 jobs in 1985.

Employment density as indicated by the number of jobs per square mile in the study area in 1985 is shown on Map 5.

Table 3

TOTAL POPULATION AND DENSITY IN THE STUDY AREA: 1985

Civil Division	Population Within Study Area	Percent of Study Area Population	Area Included in Study Area (square miles)	Percent of Study Area Within Civil Division	Average Gross Population Density per Square Mile
Milwaukee County					
Village of Bayside	4,689[a]	0.45	2.38[a]	0.6	1,970
Village of Brown Deer	12,599	1.21	4.37	1.2	2,883
City of Cudahy	19,042	1.83	4.80	1.3	3,967
Village of Fox Point	7,174	0.69	2.85	0.7	2,517
City of Franklin	18,530	1.78	34.69	9.2	534
City of Glendale	13,625	1.31	5.98	1.6	2,278
Village of Greendale	16,770	1.61	5.59	1.5	3,000
City of Greenfield	32,050	3.07	11.52	3.0	2,782
Village of Hales Corners	6,842	0.66	3.20	0.8	2,138
City of Milwaukee	612,085[b]	58.71	96.67[b]	25.5	6,332
City of Oak Creek	18,002	1.73	28.41	7.5	634
Village of River Hills	1,639	0.16	5.32	1.4	308
City of St. Francis	9,724	0.93	2.55	0.7	3,813
Village of Shorewood	14,247	1.37	1.50	0.4	9,498
City of South Milwaukee	20,512	1.97	4.82	1.3	4,256
City of Wauwatosa	50,234	4.82	13.24	3.5	3,794
City of West Allis	64,066	6.14	11.43	3.0	5,605
Village of West Milwaukee	3,595	0.34	1.12	0.3	3,210
Village of Whitefish Bay	14,264	1.37	2.11	0.6	6,760
Subtotal	939,689	90.13	242.55	64.1	3,874
Ozaukee County					
City of Cedarburg	358	0.03	0.17	- -[c]	2,106
Town of Cedarburg	787	0.08	2.58	0.7	305
City of Mequon	16,003	1.53	47.00	12.4	340
Village of Thiensville	3,104	0.30	1.04	0.3	2,985
Subtotal	20,252	1.94	50.79	13.4	399
Racine County					
Town of Caledonia	1,056	0.10	1.65	0.4	640
Town of Norway	52	- -[c]	0.07	- -[c]	743
Town of Raymond	272	0.03	1.34	0.4	203
Subtotal	1,380	0.13	3.06	0.8	451
Washington County					
Village of Germantown	11,434	1.10	29.28	7.7	391
Town of Germantown	119	0.01	0.76	0.2	157
Town of Richfield	408	0.04	1.49	0.4	274
Subtotal	11,961	1.15	31.53	8.3	379
Waukesha County					
City of Brookfield	18,860	1.81	13.59	3.6	1,388
Town of Brookfield	139	0.01	0.19	- -[c]	732
Village of Butler	2,002	0.19	0.80	0.2	2,503
Village of Elm Grove	6,239	0.60	3.25	0.9	1,920
Town of Lisbon	25	- -[c]	0.34	0.1	74
Village of Menomonee Falls	23,311	2.24	18.93	5.0	1,231
City of Muskego	4,805	0.46	3.90	1.0	1,232
City of New Berlin	13,930	1.34	9.72	2.6	1,433
Subtotal	69,311	6.65	50.72	13.4	1,367
Total	1,042,593	100.00	378.65	100.0	2,753

[a] *Includes that portion of the Village of Bayside in Ozaukee County.*

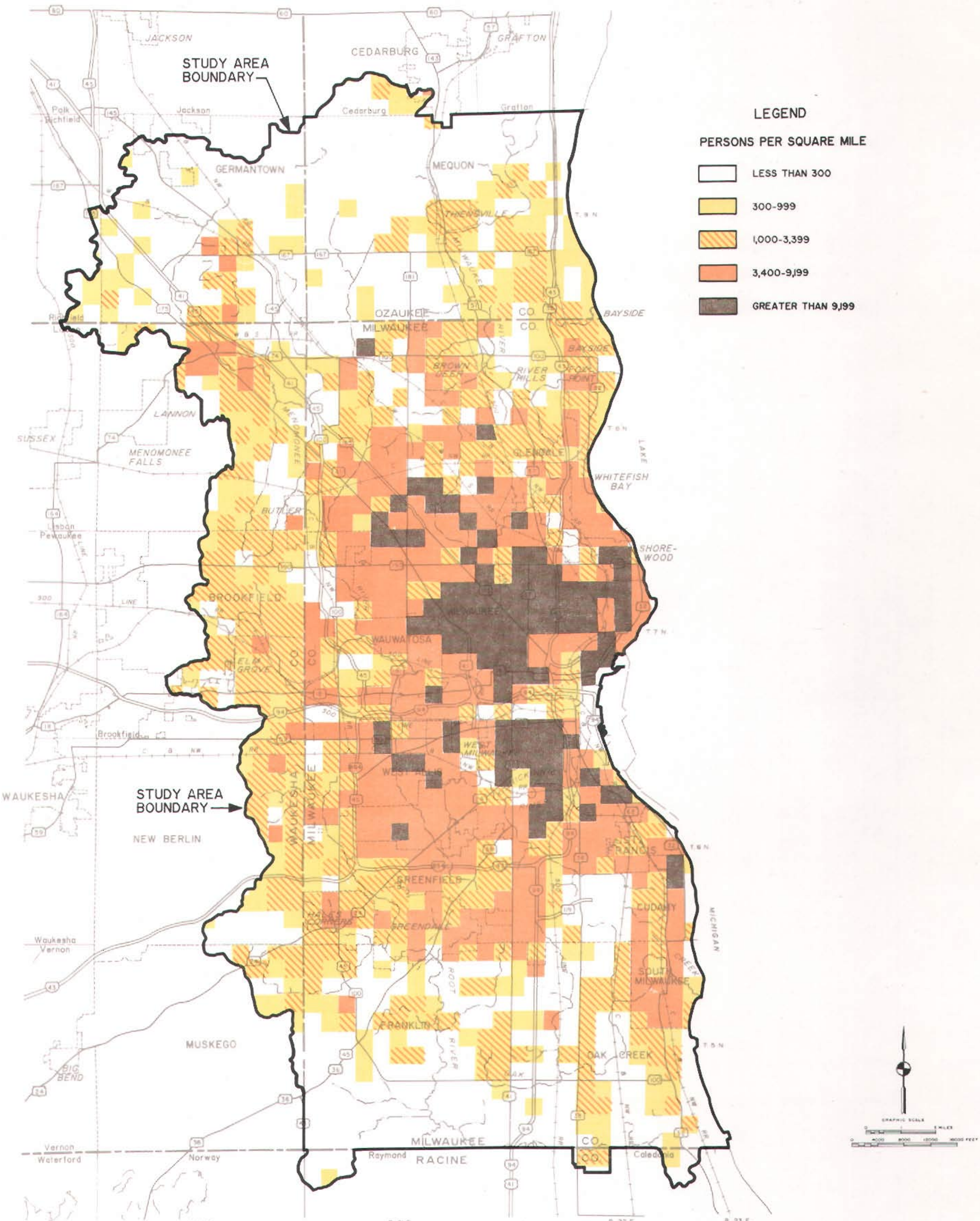
[b] *Includes that portion of the City of Milwaukee in Washington County.*

[c] *Less than 0.05 percent.*

Source: Wisconsin Department of Administration and SEWRPC.

Map 4

POPULATION DENSITY IN THE STUDY AREA: 1985



Source: SEWRPC.

Table 4

MEDIAN HOUSEHOLD INCOME IN THE STUDY AREA: 1959, 1969, AND 1979

Civil Division	Median Household Income[a]			Change 1959-1979	
	1959[b]	1969[b]	1979	Number	Percent
Milwaukee County					
Village of Bayside	$ - -[c]	$43,744	$42,848	- -	- -
Village of Brown Deer	19,174	27,151	25,888	6,714	35.0
City of Cudahy	16,328	20,514	20,305	3,977	24.4
Village of Fox Point	- -[c]	36,115	34,614	- -	- -
City of Franklin	17,143	23,158	25,464	8,321	48.5
City of Glendale	19,516	27,465	27,195	7,679	39.3
Village of Greendale	18,491	26,726	27,718	9,227	50.0
City of Greenfield	18,647	23,353	22,137	3,490	18.7
Village of Hales Corners	21,240	27,380	24,892	3,652	17.2
City of Milwaukee	14,104	16,437	16,028	1,924	13.6
City of Oak Creek	16,851	22,615	23,413	6,562	38.9
Village of River Hills	N/A	43,280	48,766	- -	- -
City of St. Francis	15,310	18,478	20,231	4,921	32.1
Village of Shorewood	18,847	21,030	19,570	723	3.8
City of South Milwaukee	16,848	21,555	20,850	4,002	23.7
City of Wauwatosa	19,769	23,492	23,288	3,519	17.8
City of West Allis	16,197	19,917	18,686	2,489	15.4
Village of West Milwaukee	14,681	16,972	16,430	1,749	11.9
Village of Whitefish Bay	25,535	29,707	29,130	3,595	14.1
Ozaukee County					
City of Cedarburg	15,449	23,187	22,716	7,267	47.0
Town of Cedarburg	N/A	25,105	30,462	- -	- -
City of Mequon	18,426	28,238	33,510	15,084	81.9
Village of Thiensville	19,722	26,788	23,385	3,663	18.6
Racine County					
Town of Caledonia	N/A	23,215	25,815	- -	- -
Town of Norway	N/A	19,777	23,685	- -	- -
Town of Raymond	N/A	21,413	23,329	- -	- -
Washington County					
Village of Germantown	N/A	25,415	25,314	- -	- -
Town of Germantown	N/A	19,586	25,313	- -	- -
Town of Richfield	N/A	22,972	27,099	- -	- -
Waukesha County					
City of Brookfield	21,746	31,037	32,159	10,413	47.9
Town of Brookfield	N/A	27,278	30,979	- -	- -
Village of Butler	N/A	23,459	10,444	- -	- -
Village of Elm Grove	- -[c]	35,754	38,922	- -	- -
Town of Lisbon	N/A	24,533	27,487	- -	- -
Village of Menomonee Falls	17,852	25,671	26,804	8,952	50.1
City of Muskego	N/A	23,994	25,648	- -	- -
Town of Muskego[d]	N/A	- -	- -	- -	- -
City of New Berlin	18,290	25,902	28,547	10,257	56.1

NOTE: N/A indicates data not available.

[a]*Median income values shown are for entire municipality.*

[b]*Data have been converted to constant 1979 dollars.*

[c]*1960 Census did not compute an actual medium value for this community, but rather listed the median value as greater than $10,000.*

[d]*The Town of Muskego was incorporated as part of the City of Muskego between 1960 and 1970.*

Source: U. S. Bureau of the Census and SEWRPC.

Table 5

PERSONS PER HOUSEHOLD IN THE STUDY AREA: 1960, 1970, AND 1980

Civil Division	Persons per Household[a]			Change 1960-1980	
	1960	1970	1980	Number	Percent
Milwaukee County					
Village of Bayside	3.7	3.6	3.0	-0.7	-18.9
Village of Brown Deer	4.0	3.6	2.8	-1.2	-30.0
City of Cudahy	3.4	3.2	2.8	-0.6	-17.6
Village of Fox Point	3.6	3.4	3.3	-0.3	-8.3
City of Franklin	4.0	4.0	3.0	-1.0	-25.0
City of Glendale	3.4	3.4	2.7	-0.7	-20.6
Village of Greendale	3.7	3.8	3.1	-0.6	-16.2
City of Greenfield	3.8	3.5	2.6	-1.2	-31.6
Village of Hales Corners	3.8	3.6	2.8	-1.0	-26.3
City of Milwaukee	3.1	3.0	2.6	-0.5	-16.1
City of Oak Creek	4.0	3.9	3.0	-1.0	-25.0
Village of River Hills	3.5	3.4	3.1	-0.4	-11.4
City of St. Francis	3.8	3.3	2.5	-1.3	-34.2
Village of Shorewood	2.8	2.6	2.2	-0.6	-21.4
City of South Milwaukee	3.6	3.5	2.8	-0.8	-22.2
City of Wauwatosa	3.2	3.1	2.6	-0.6	-18.8
City of West Allis	3.3	3.0	2.5	-0.8	-24.2
Village of West Milwaukee	2.7	2.4	2.0	-0.7	-25.9
Village of Whitefish Bay	3.5	3.2	3.4	-0.1	-2.9
Ozaukee County					
City of Cedarburg	3.5	3.4	2.7	-0.8	-22.9
Town of Cedarburg	3.8	3.9	3.5	-0.3	-7.9
City of Mequon	3.6	3.8	3.1	-0.5	-13.9
Village of Thiensville	3.7	3.6	2.5	-1.2	-32.4
Racine County					
Town of Caledonia	3.9	3.9	3.3	-0.6	-15.4
Town of Norway	3.7	3.7	3.4	-0.3	-8.1
Town of Raymond	3.9	4.0	3.4	-0.5	-12.8
Washington County					
Village of Germantown	3.8	4.0	3.1	-0.7	-18.4
Town of Germantown	4.0	3.6	4.0	0.0	0.0
Town of Richfield	4.0	3.9	3.5	-0.5	-12.5
Waukesha County					
City of Brookfield	3.9	3.9	3.3	-0.6	-15.4
Town of Brookfield	3.9	3.9	3.5	-0.4	-10.3
Village of Butler	3.9	3.7	2.6	-1.3	-33.3
Village of Elm Grove	3.9	3.8	3.0	-0.9	-25.6
Town of Lisbon	4.1	3.9	3.5	-0.6	-24.4
Village of Menomonee Falls	3.9	4.0	3.1	-0.8	-20.5
City of Muskego[b]	--	3.9	3.3	--	--
Town of Muskego[b]	3.9	--	--	--	--
City of New Berlin	4.0	4.0	3.3	-0.7	-17.5

[a] *Persons per houshold rates are for entire municipality.*

[b] *The Town of Muskego was incorporated as part of the City of Muskego between 1960 and 1970.*

Source: U. S. Bureau of the Census and SEWRPC.

Table 6

MEDIAN AGE IN THE STUDY AREA: 1960, 1970, AND 1980

Civil Division	Median Age[a]			Change 1960-1980	
	1960	1970	1980	Number	Percent
Milwaukee County					
Village of Bayside	28.2	32.2	39.6	11.4	40.4
Village of Brown Deer	25.0	26.6	32.7	7.7	30.8
City of Cudahy	27.9	26.6	30.4	2.5	8.9
Village of Fox Point.	32.0	34.3	41.0	9.0	28.1
City of Franklin	25.5	23.6	28.8	3.3	12.9
City of Glendale	30.7	35.2	39.5	8.8	28.7
Village of Greendale.	25.2	23.6	30.6	5.4	21.4
City of Greenfield	26.9	27.2	31.8	4.9	18.2
Village of Hales Corners	27.7	27.7	33.8	6.1	22.0
City of Milwaukee	30.4	28.2	28.8	-1.6	-5.3
City of Oak Creek	23.7	22.9	28.2	4.5	19.0
Village of River Hills	34.3	34.2	36.7	2.4	7.0
City of St. Francis	25.7	26.9	30.1	4.4	17.1
Village of Shorewood	39.2	36.4	34.4	-4.8	-12.2
City of South Milwaukee	26.4	25.9	30.3	3.9	14.8
City of Wauwatosa.	37.6	35.9	36.9	-0.7	-1.9
City of West Allis	30.7	30.5	33.5	2.8	9.1
Village of West Milwaukee.	38.1	38.9	38.2	0.1	0.3
Village of Whitefish Bay	32.2	32.7	35.7	3.5	10.9
Ozaukee County					
City of Cedarburg	27.4	26.8	31.9	4.5	16.4
Town of Cedarburg	26.5	24.1	30.6	4.1	15.5
City of Mequon.	29.2	28.4	33.0	3.8	13.0
Village of Thiensville	27.8	30.1	40.3	12.5	45.0
Racine County					
Town of Caledonia	24.6	23.0	28.3	3.7	15.0
Town of Norway.	25.7	24.7	28.7	3.0	11.7
Town of Raymond	25.2	22.9	28.8	3.6	14.3
Washington County					
Village of Germantown.	25.0	22.4	27.9	2.9	11.6
Town of Germantown	24.3	25.2	27.6	3.3	13.6
Town of Richfield	23.6	23.1	28.1	4.5	19.1
Waukesha County					
City of Brookfield	27.7	26.5	34.7	7.0	25.3
Town of Brookfield	26.1	25.2	31.3	5.2	19.9
Village of Butler	23.4	24.3	31.9	8.5	25.3
Village of Elm Grove	31.6	32.0	42.1	10.5	33.2
Town of Lisbon	23.3	24.5	29.3	6.0	25.8
Village of Menomonee Falls. . . .	23.8	22.3	31.1	7.3	30.7
City of Muskego[b]	- -	23.8	29.6	- -	- -
Town of Muskego[b]	25.4	- -	- -	- -	- -
City of New Berlin.	25.2	23.2	30.3	5.1	20.2

[a] *Median ages shown are for entire municipality.*

[b] *The Town of Muskego was incorporated as part of the City of Muskego between 1960 and 1970.*

Source: U. S. Bureau of the Census and SEWRPC.

Table 7

TOTAL EMPLOYMENT IN THE STUDY AREA: 1972, 1980, AND 1985

Civil Division	Employment						
	1972	1980	Change 1972-1980		1985	Change 1980-1985	
			Number	Percent		Number	Percent
Milwaukee County							
Village of Bayside	512	506	-6	-1.2	846	340	67.2
Village of Brown Deer	3,577	5,521	1,944	54.3	6,969	1,448	26.2
City of Cudahy	10,772	14,162	3,390	31.5	12,419	-1,743	-12.3
Village of Fox Point	1,326	1,959	633	47.7	2,066	107	5.5
City of Franklin	1,937	3,567	1,630	84.2	3,948	381	10.7
City of Glendale	18,114	20,500	2,386	13.2	20,668	168	0.8
Village of Greendale	4,749	6,281	1,532	32.3	6,885	604	9.6
City of Greenfield	5,291	7,735	2,444	46.2	9,717	1,982	25.6
Village of Hales Corners	1,974	2,481	507	25.7	2,990	509	20.5
City of Milwaukee	351,227	348,850	-2,377	-0.7	341,085	-7,765	-2.2
City of Oak Creek	3,673	11,924	8,251	224.6	15,131	3,207	26.9
Village of River Hills	623	836	213	34.2	1,090	254	30.4
City of St. Francis	1,472	2,597	1,125	76.4	2,711	114	4.4
Village of Shorewood	3,105	3,304	199	6.4	3,873	569	17.2
City of South Milwaukee	4,779	7,802	3,023	63.3	6,187	-1,615	-20.7
City of Wauwatosa	36,031	50,630	14,599	40.5	52,503	1,873	3.7
City of West Allis	36,202	39,822	3,620	10.0	36,506	-3,316	-8.3
Village of West Milwaukee	19,978	10,720	-9,258	-46.3	6,883	-3,837	-35.8
Village of Whitefish Bay	3,043	2,935	-108	-3.5	2,504	-431	-14.7
Subtotal	508,385	542,132	33,747	6.6	534,981	-7,151	-1.3
Ozaukee County							
City of Cedarburg	--	--	--	--	--	--	--
Town of Cedarburg	53	40	-13	-24.5	26	-14	-35.0
City of Mequon	3,855	5,718	1,863	48.3	7,615	1,897	33.2
Village of Thiensville	1,038	1,191	153	14.7	1,120	-71	-6.0
Subtotal	4,946	6,949	2,003	40.5	8,761	1,812	26.1
Racine County							
Town of Caledonia	52	90	38	73.1	88	-2	-2.2
Town of Norway	--	--	--	--	--	--	--
Town of Raymond	4	7	3	75.0	7	0	0.0
Subtotal	56	97	41	73.2	95	-2	-2.1
Washington County							
Village of Germantown	1,193	2,869	1,676	140.5	2,970	101	3.5
Town of Germantown	30	121	91	303.3	120	-1	-0.8
Town of Richfield	51	69	18	35.3	69	0	0.0
Subtotal	1,274	3,059	1,785	140.1	3,159	100	3.3
Waukesha County							
City of Brookfield	9,907	17,396	7,489	75.6	21,414	4,018	23.1
Town of Brookfield	--	--	--	--	--	--	--
Village of Butler	2,314	4,148	1,834	79.3	3,759	-389	-9.4
Village of Elm Grove	1,181	1,842	661	56.0	1,825	-17	-0.9
Town of Lisbon	--	--	--	--	--	--	--
Village of Menomonee Falls	8,714	13,543	4,829	55.4	15,453	1,910	14.1
City of Muskego	319	854	535	167.7	718	-136	-15.9
City of New Berlin	1,608	1,806	198	12.3	2,004	198	11.0
Subtotal	24,043	39,589	15,546	64.7	45,173	5,584	14.1
Total	538,704	591,826	53,122	9.9	592,169	343	0.1

Source: Wisconsin Department of Industry, Labor and Human Relations; and SEWRPC.

Table 8

TOTAL EMPLOYMENT BY MAJOR INDUSTRY GROUP IN THE STUDY AREA: 1972, 1980, AND 1985

Industry Group	1972		1980		Change 1972-1980		1985		Change 1980-1985	
	Number	Percent	Number	Percent	Number	Percent	Number	Percent	Number	Percent
Agricultural	5,558	1.0	3,491	0.6	-2,067	-37.2	3,073	0.5	-418	-12.0
Retail.	83,821	15.6	89,393	15.1	5,572	6.6	93,253	15.7	3,860	4.3
Service	148,780	27.6	180,303	30.5	31,523	21.2	199,121	33.6	18,818	10.4
Industrial.	208,972	38.8	210,394	35.5	1,422	0.7	182,746	30.9	-27,648	-13.1
Governmental and Institutional. . . .	64,043	11.9	78,120	13.2	14,077	22.0	83,398	14.1	5,278	6.8
Transportation, Communication, and Utilities	27,530	5.1	30,125	5.1	2,595	9.4	30,578	5.2	453	1.5
Total	538,704	100.0	591,826	100.0	53,122	9.9	592,169	100.0	343	0.1

Source: Wisconsin Department of Industry, Labor and Human Relations; and SEWRPC.

Land Use

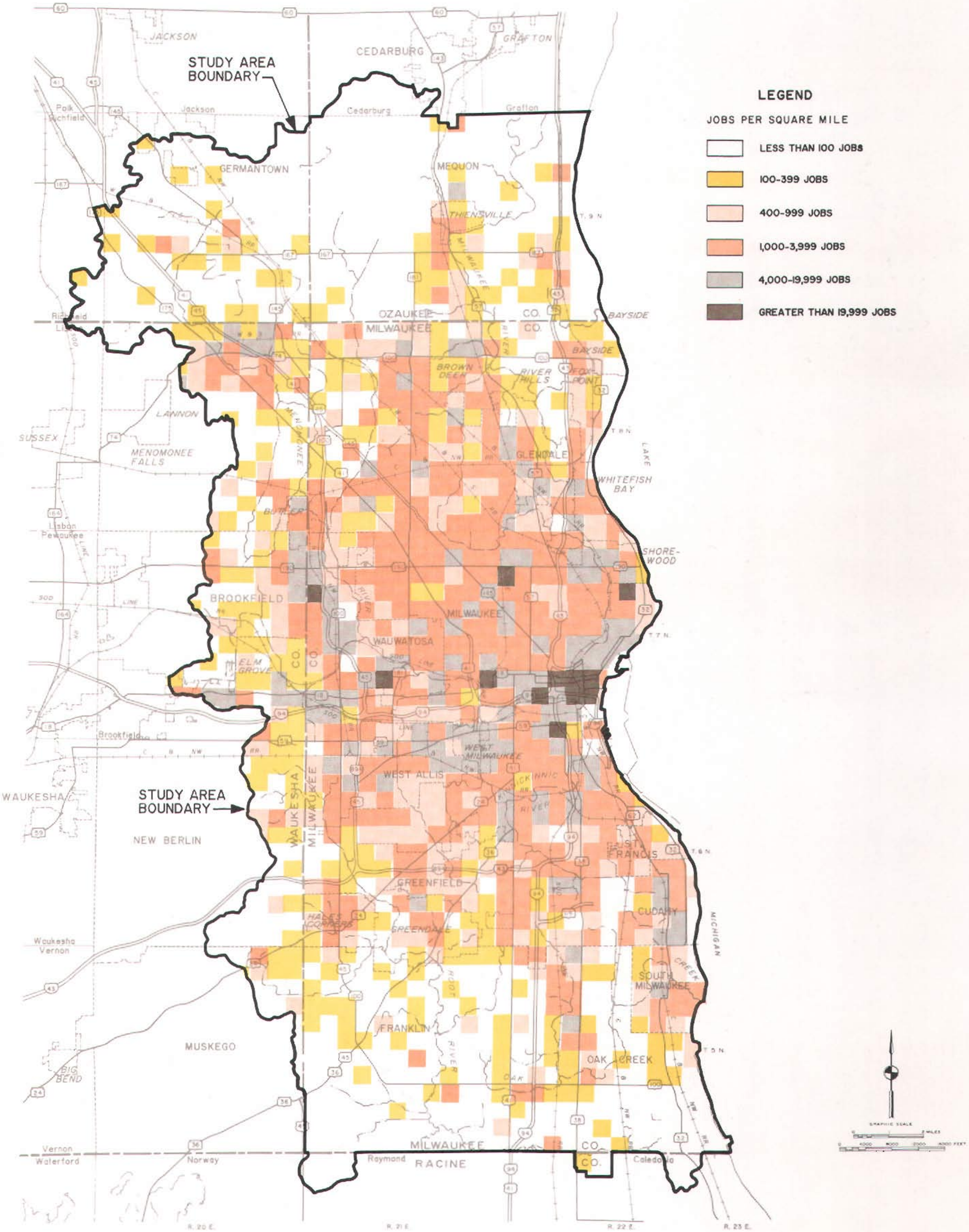
As already noted, the type, density, and spatial distribution of land uses are important determinants of the rate, volume, and quality of stormwater runoff, as well as the intensity of flooding and dollar amount of flood damages. The amount and spatial distribution of impervious areas, and the type of stormwater drainage facilities, vary with land use. The existing land use pattern in the study area can best be understood within the context of its historical development. Accordingly, attention is focused herein on historical as well as existing land use development patterns within the study area.

Historical Growth Patterns: The first permanent European settlement within the study area was a trading post established in 1795 on the east side of the Milwaukee River just north of what is now Wisconsin Avenue. The movement of European settlers into the Region was well underway by 1830, and most of the cities and villages within the study area can trace their origins to trading posts and mills established in the early nineteenth century. Completion of the U. S. Public Land Survey in the Region by 1836 and subsequent sale of public lands brought many settlers from New England, Germany, Austria, and Scandanavia. By the late 1800's, there were many small scattered areas of urban development within the study area. In addition to the larger urban center associated with the City of Milwaukee, traces of early urban development are evident in many of the smaller communities within the study area. These include the Cities of Cudahy, South Milwaukee, and Wauwatosa in Milwaukee County, the Village of Thiensville and the unincorporated community of Freidstadt, which is now in the City of Mequon, in Ozaukee County, the Village of Germantown in Washington County, and the Villages of Elm Grove and Menomonee Falls in Waukesha County. The pattern of historical urban growth in the study area from the years 1850 to 1985 is shown on Map 6.

Existing Land Use: The general pattern of existing land use within the study area for the years 1963, 1970, 1980, and 1985 is indicated in Table 9. Urban land uses encompassed over 176 square miles, or about 47 percent, of the study area in 1963. Residential land uses and transportation, communication, and utility uses together encompassed over 142 square miles, or almost 38 percent, of the study area in 1963. Urban lands in the study area increased to more than 198 square miles, or over 52 percent, of the study area in 1970, and to 216 square miles, or over 57 percent, of the study area in 1980. By 1985, urban land uses encompassed over 221 square miles, or more than 58 percent of the study area. The largest increases in urban land uses during this time period occurred in residential land use, which increased by over 19 square miles, and transportation and communication utility land uses, which increased over 15 square miles. Rural land uses, consisting primarily of agricultural and other open lands, decreased by 45 square miles—from about 202 square miles in 1963 to about 157 square miles in 1985.

Map 5

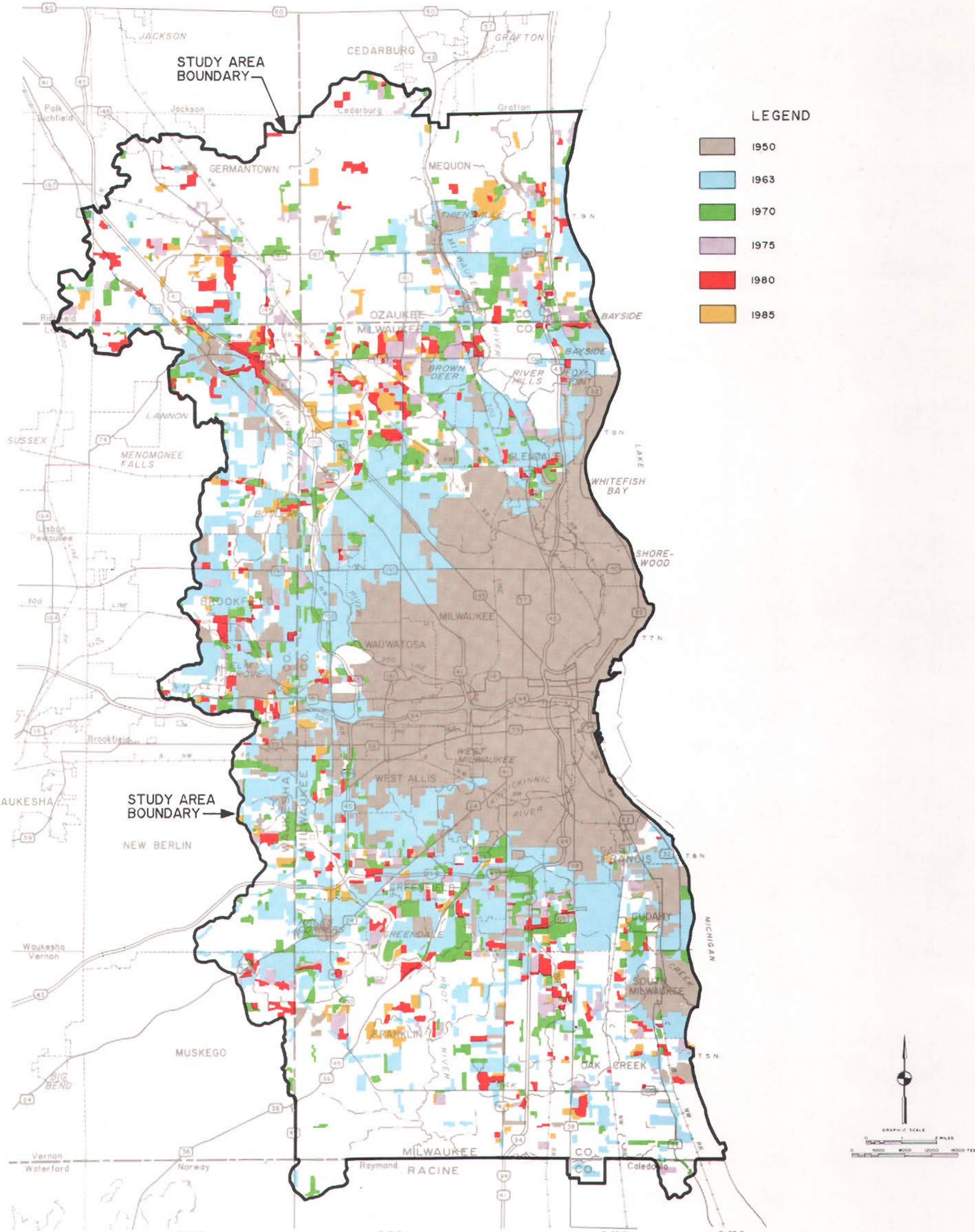
EMPLOYMENT DENSITY IN THE STUDY AREA: 1985



Source: SEWRPC.

Map 6

HISTORICAL URBAN DEVELOPMENT IN THE STUDY AREA: 1850-1985



Source: SEWRPC.

Table 9

LAND USE IN THE STUDY AREA: 1963, 1970, 1980, AND 1985

Land Use Category	1963		1970		1980		1985		Change 1963-1985	
	Square Miles	Percent	Square Miles	Percent	Square Miles	Percent	Square Miles	Percent	Square Miles	Percent
Urban										
Residential.	88.4	23.4	96.6	25.6	105.7	28.0	107.8	28.5	19.4	21.9
Commercial	4.8	1.3	5.5	1.4	6.4	1.7	6.8	1.8	2.0	41.7
Industrial.	7.1	1.9	8.0	2.1	9.2	2.4	9.8	2.6	2.7	38.0
Transportation, Communication, and Utilities	53.8	14.2	62.7	16.6	68.1	18.0	69.2	18.3	15.4	28.6
Governmental and Institutional. . . .	11.4	3.0	12.9	3.4	13.3	3.5	13.4	3.5	2.0	17.5
Recreational	10.9	2.9	12.5	3.3	13.7	3.6	14.4	3.8	3.5	32.1
Subtotal	176.4	46.7	198.2	52.4	216.4	57.2	221.4	58.5	45.0	25.5
Rural										
Agricultural	123.8	32.7	106.5	28.2	93.1	24.7	88.0	23.4	-35.8	-28.9
Open Land.	77.8	20.6	73.3	19.4	68.5	18.1	68.6	18.1	-9.2	-11.8
Subtotal	201.6	53.3	179.8	47.6	161.6	42.8	156.6	41.5	-45.0	-22.3
Total	378.0	100.0	378.0	100.0	378.0	100.0	378.0	100.0	--	--

Source: SEWRPC.

The 1985 land uses in the study area are shown on Map 7.

Public Utility Base

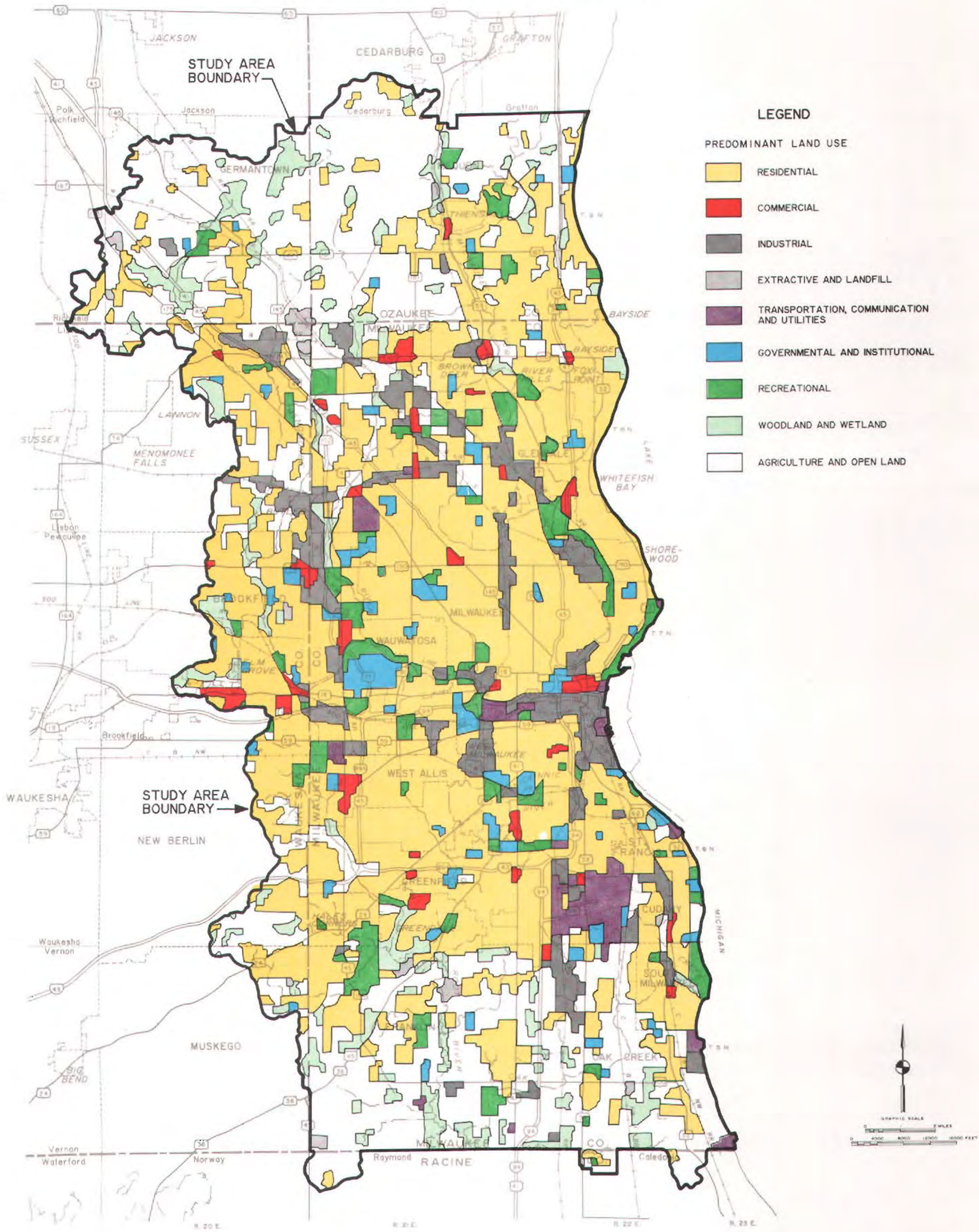
Sanitary Sewer Service: In 1985, sanitary sewage generated within the study area was conveyed to and treated at six public sewage treatment plants. Two of these plants, the Jones Island sewage treatment plant and the South Shore sewage treatment plant, are operated by the Milwaukee Metropolitan Sewerage District, which serves the entire study area except that portion in the City of Cedarburg which is tributary to the City of Cedarburg sewage treatment plant; that portion in the Village of Germantown which is tributary to the Village of Germantown sewage treatment plant; that portion in the Village of Thiensville which is tributary to the Village of Thiensville sewage treatment plant; and that portion in the City of South Milwaukee which is tributary to the City of South Milwaukee sewage treatment plant. In 1987, the Village of Germantown and Village of Thiensville sewage treatment plants were in the process of abandonment, with the tributary areas to be connected to the Milwaukee metropolitan sewerage system.

The existing public sanitary sewer service area and the location of the existing sewage treatment facilities within the study area are shown on Map 8. About 221 square miles, or 58 percent of the total study area, and approximately 1,019,900 persons, or about 98 percent of the total study area population, were served by public sanitary sewerage facilities in 1985.

Water Supply Service: Most of the water supply service within the study area is provided by public water utilities. In 1985, there were a total of 21 publicly owned water utilities within the study area. The existing service areas of these utilities is shown on Map 9. In addition to the publicly owned water utilities, there were 105 nonmunicipal cooperatively owned residential water supply systems within the study area. Many of these systems served isolated enclaves of residential development and are governed by Chapters NR 108, 109, 111, and 112 of the Wisconsin Administrative Code. The locations of these 105 private water supply systems are also shown on Map 9. About 188 square miles, or about 50 percent of the total study area, and approximately 962,500 persons, or 92 percent of the total study area population, were served with public water supply facilities in 1985.

Map 7

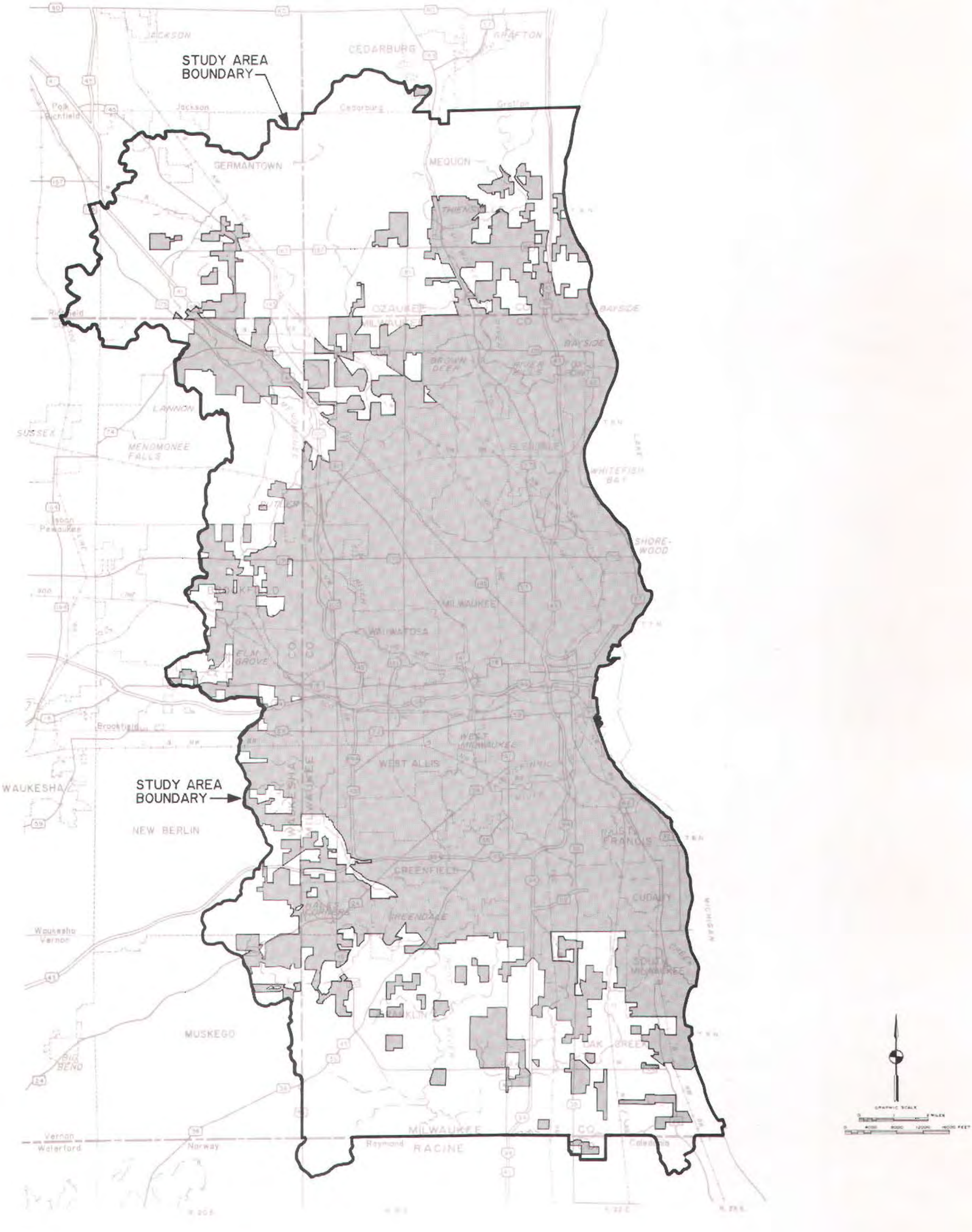
LAND USE IN THE STUDY AREA: 1985



Source: SEWRPC.

Map 8

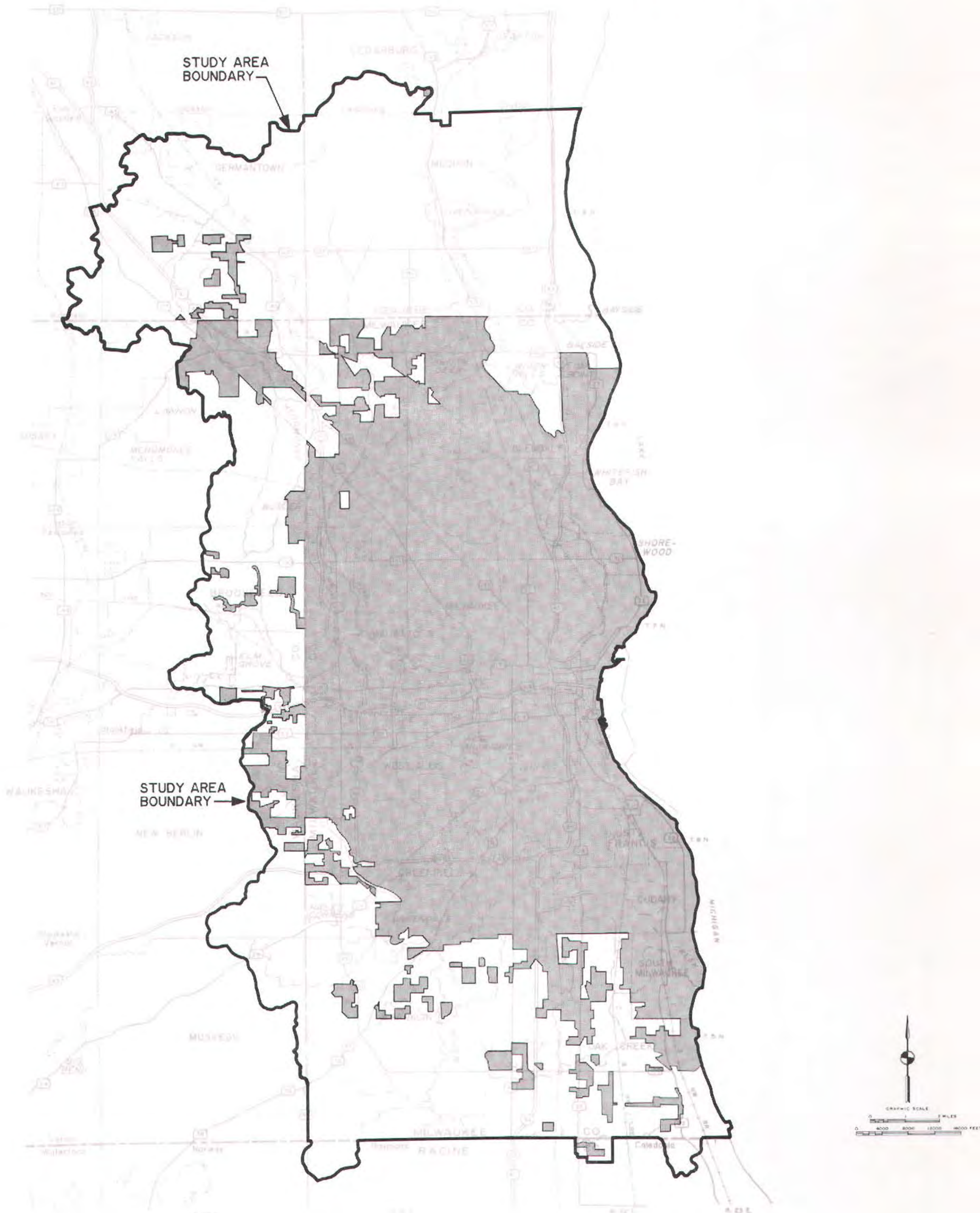
EXISTING PUBLIC SANITARY SEWER SERVICE AREAS IN THE STUDY AREA: 1985



Source: SEWRPC.

Map 9

EXISTING PUBLIC WATER UTILITY SERVICE AREAS IN THE STUDY AREA: 1985



Source: SEWRPC.

Electric Power and Gas Service: Electric power is provided to all portions of the study area by the Wisconsin Electric Power Company, while natural gas service is provided in part by the Wisconsin Natural Gas Company and in part by the Wisconsin Gas Company. Both electric power and natural gas service may be considered to be ubiquitous within the study area and thus do not constitute a constraint on the location or intensity of urban development within the study area.

Transportation Facilities

Highways: As shown on Map 10, in 1985 the study area was served by an extensive street and highway system, including about 85 linear miles of freeway and about 905 linear miles of surface arterials. In addition, there were about 2,633 linear miles of collector and land access streets within the study area. The extensive street and highway system within the study area serves to provide ease of access to residential, commercial, and industrial land uses in the area.

Bus Service: Two types of bus service were provided in the study area in 1985: urban mass transit service and intercity bus service. Urban mass transit service was provided by the Milwaukee County Transit System and Waukesha County. About 152 square miles, or 40 percent of the study area, and 871,600 persons, or 84 percent of the resident population, were within the mass transit service area in 1985.

Intercity bus service is provided in the study area by various private carriers connecting the Milwaukee central business district with outlying areas.

Railway Service: Railway service within the study area in 1985 was limited to freight hauling except for scheduled Amtrak passenger service over the lines of the Soo Line Railroad Company between the Amtrak passenger station in the City of Milwaukee, which is the only stop in the study area, and Chicago to the south and Minneapolis-St. Paul to the northwest. The Amtrak passenger station is the only rail passenger terminal within the study area.

As shown on Map 11, extensive railway freight service was provided throughout the study area by the Soo Line Railroad Company; the Wisconsin & Southern Railroad Company; and the Chicago & North Western Transportation Company. The heavily industrialized portion of the Menomonee River Valley in the City of Milwaukee, in particular, contains a large concentration of the Soo Line Railroad Company classification yard railway trackage. Also within the study area, the "Butler" classification yard of the Chicago & North Western Transportation Company is located immediately east of the Village of Butler in the Cities of Milwaukee and Wauwatosa. In addition, one shortline railway operates within the study area. This railway—the Wisconsin & Southern Railroad Company—provides trackage rights to the Soo Line Railroad Company.

Airports: There are three public-use airports located within the study area. Residents and businesses within the study area are provided with commercial airline service at Milwaukee County's General Mitchell International Airport (General Mitchell Field). In 1985, this airport was served by 15 air carriers providing passenger service on a regularly scheduled basis, as well as air cargo service. General Mitchell International Airport and Timmerman Field, located on the northwest side of the City of Milwaukee, also provide a variety of facilities and fixed-base operation services for all types of general aviation activity, including business and corporate jets. General aviation activity was also served by Rainbow Airport located in the City of Franklin.

NATURAL FEATURES

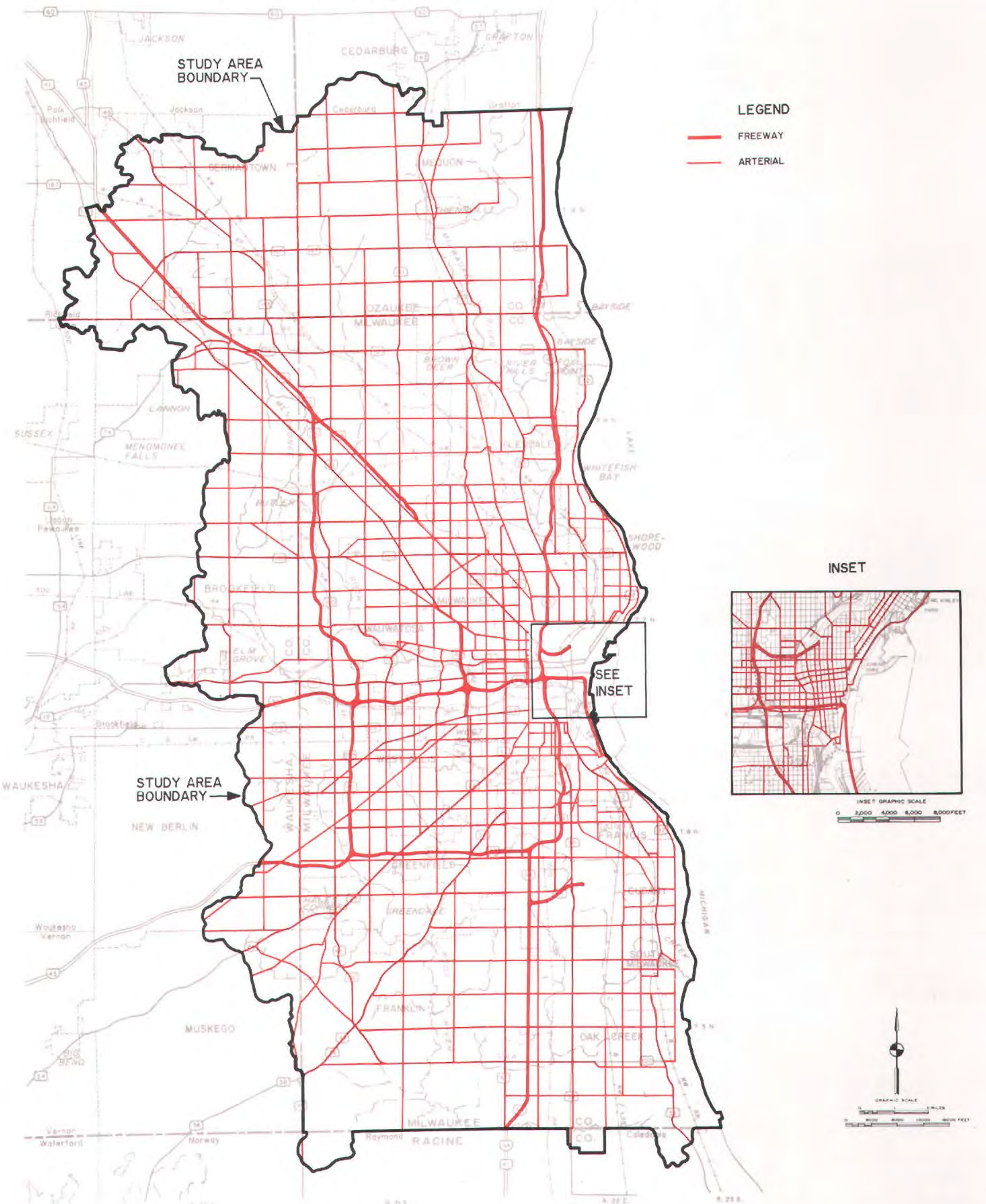
The natural resource base is an important determinant of the development potential of an area, as well as of the ability of the area to provide a pleasant and habitable environment for all forms of life. The principal elements of the natural resource base are: climate, physiographic and topographic features, geology, soils, vegetation, and surface water features. Without a proper understanding of these elements and of their interrelationships, human use and alteration of the natural environment proceed at a risk of excessive costs in terms of both monetary expenditures and destruction of the nonrenewable or slowly renewable resources. In this age of high resource demand, urban expansion, and rapidly changing technology, it is particularly important that the natural resource base be an important consideration in any planning effort.

Climate

Air temperatures and the type, intensity, and duration of precipitation events are major determinants of the rate and volume of stormwater runoff. The study area has a typical continental-type climate characterized primarily by a continuous progression of markedly different seasons and a wide

Map 10

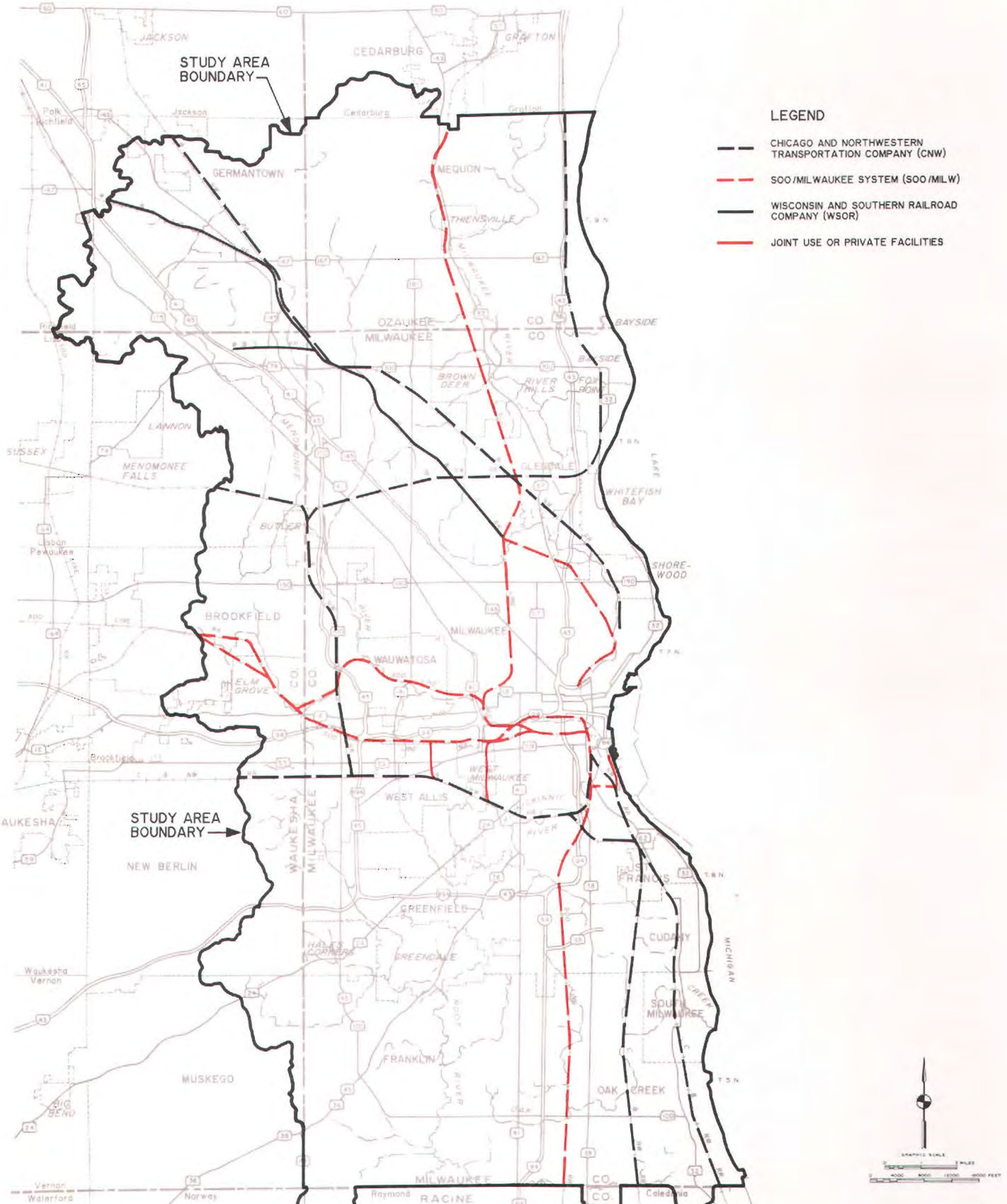
ARTERIAL STREETS AND HIGHWAYS IN THE STUDY AREA: 1985



Source: SEWRPC.

Map 11

COMMON CARRIER RAILWAY FREIGHT LINES IN THE STUDY AREA: 1985



Source: SEWRPC.

Table 10

NORMAL AIR TEMPERATURES AT SELECTED METEOROLOGICAL OBSERVATION STATIONS IN THE STUDY AREA: 1951-1980[a]

Month	Meteorological Station Location						Two Station Average		
	Milwaukee (Mitchell Field)			Germantown					
	Average Daily Maximum	Average Daily Minimum	Mean	Average Daily Maximum	Average Daily Minimum	Mean	Average Daily Maximum	Average Daily Minimum	Mean
January	26.0	11.3	18.7	26.0	8.2	17.1	26.0	9.7	17.9
February	30.1	15.8	23.0	30.6	12.5	21.6	30.4	14.1	22.3
March	39.2	24.9	32.1	40.2	22.3	31.3	39.7	23.6	31.7
April.	53.5	35.6	44.6	55.4	34.1	44.8	54.4	34.8	44.7
May	64.8	44.7	54.8	67.8	43.4	55.6	66.3	44.1	55.2
June	75.0	54.7	64.9	77.3	52.9	65.1	76.1	53.8	65.0
July	79.8	61.1	70.5	82.0	58.0	70.0	80.9	59.5	70.2
August	78.4	60.2	69.3	80.3	56.9	68.6	79.3	58.5	68.9
September . . .	71.2	52.5	61.9	72.6	49.2	60.9	71.9	50.8	61.4
October.	59.9	41.9	50.9	61.3	39.5	50.4	60.6	40.7	50.6
November . . .	44.7	29.9	37.3	45.4	27.7	36.5	45.1	28.8	36.9
December . . .	32.0	18.2	25.1	32.0	15.2	23.6	32.0	16.7	24.3
Annual	54.6	37.6	46.1	55.9	35.0	45.5	55.2	36.3	45.8

[a]The 30-year period 1951-1980 is the "standard normal" period which conforms to the World Metropolitan Organization standard for climatological normals.

Source: National Climatic Center and SEWRPC.

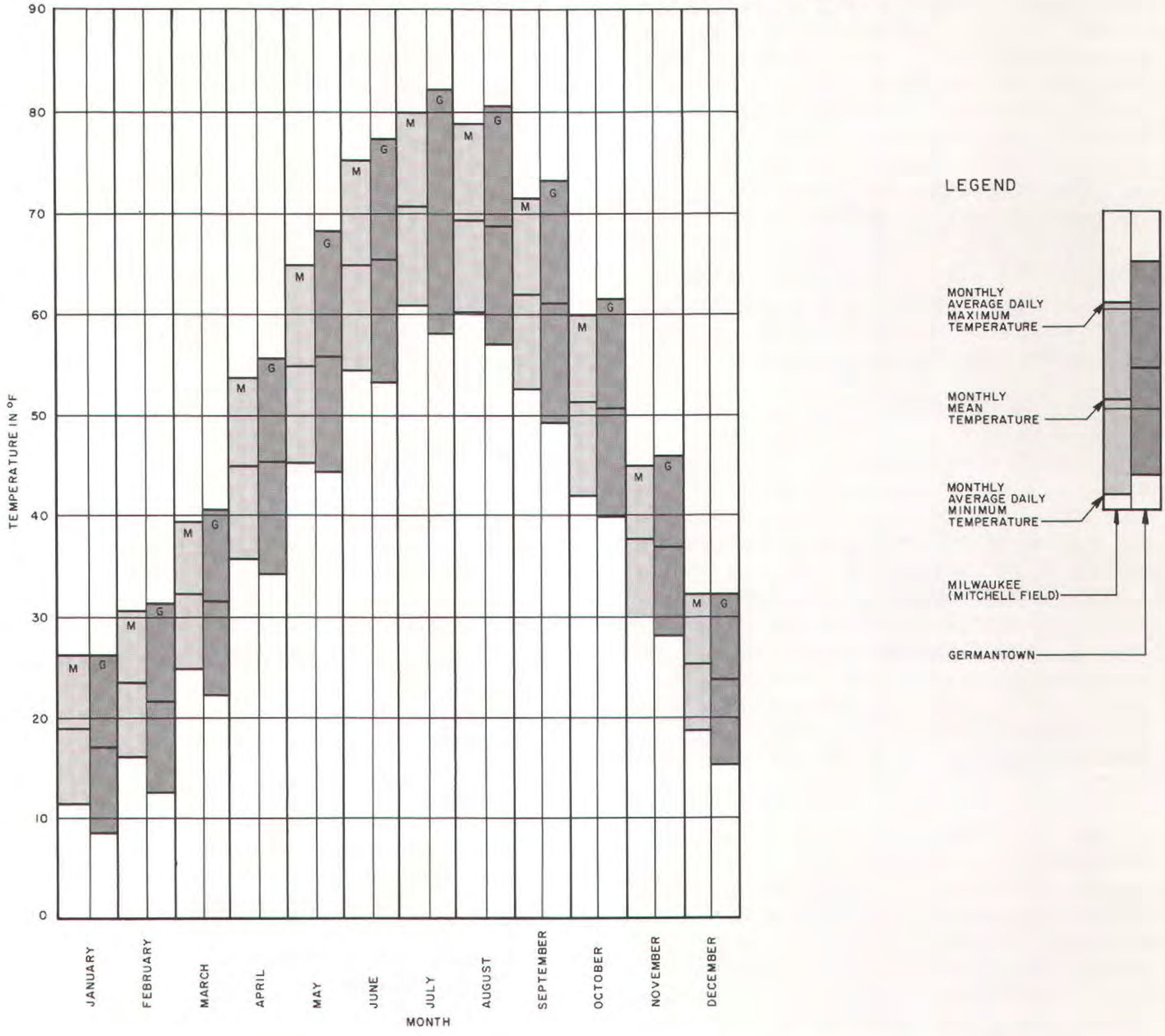
range in monthly temperatures. The study area lies in the path of both low-pressure storm centers moving from the west and southwest, and high-pressure fair weather centers moving in a generally southeasterly direction. The confluence of these air masses results in frequent weather changes, particularly during spring and winter. These temporal weather changes consist of marked variations in temperature, precipitation, relative humidity, wind speed and direction, and cloud cover. The meteorological events influence the rate and amount of stormwater runoff, the severity of storm drainage and flooding problems, and the required capacity of stormwater conveyance and storage facilities. Definitive long-term meteorological data are available for the study area from the Milwaukee National Weather Service station located at General Mitchell Field, as well as from a meteorological weather station located in the Village of Germantown.

Temperature: Temperature data for the two selected meteorological observation stations within the study area, Milwaukee at Mitchell Field and Germantown, are presented in Table 10 and Figure 2. The air temperature data used to develop the table and figure represent the monthly climatic normal averages for the period 1951 through 1980. The use of the 30-year climatic normal period provides for a consistent period of record between stations with varying years of operation, and thus enables more accurate comparisons to be made of prevailing temperature conditions. From a statistical standpoint, a 30-year period of record may be expected to encompass about 95 percent of the total variation experienced in a particular meteorological event at a given location.

The temperature data for the study area, as reflected by the monthly mean temperatures at the Milwaukee and Germantown observation stations,

Figure 2

TEMPERATURE DATA FOR SELECTED METEOROLOGICAL OBSERVATION STATIONS IN THE STUDY AREA: 1951-1980



Source: National Climatic Center and SEWRPC.

indicate both spatial and temporal variations. The temperature data also illustrate how air temperatures in the study area lag approximately one month behind the winter and summer solstice during the annual cycle; as a result, July is the warmest month in the study area and January is the coldest.

Mean summer temperatures in July and August are in the 70°F range within the study area. Average daily maximum temperatures for these two summer months range from 78°F to 82°F, whereas the average daily minimum temperatures vary from 57°F to 61°F. With respect to the daily minimum temperatures, the meteorological station network is not sufficiently dense to reflect the effects of topography. During nighttime hours, cold air, because of its greater density, flows into low-lying areas. Because of this phenomenon, the average daily minimum temperatures in these topographically low areas, particularly during the summer months, may be expected to be lower than those recorded at the meteorological stations.

Temperatures within the study area, as measured by the monthly means for January and February, range from 17° F to 23° F. Average daily maximum temperatures within the study area for these two months vary from about 26° F to 31° F, whereas the average daily minimum temperatures range from about 8° F to 16° F. The temperature data presented in Table 10 and Figure 2 provide evidence of the moderating effect of Lake Michigan on near-shore temperatures. For example, the Germantown meteorological station exhibits average daily maximum temperatures, particularly during the summer months, of 1° F to 2° F higher than those exhibited at the Mitchell Field station in the City of Milwaukee. Thus, the presence of Lake Michigan and its associated lake breeze phenomenon act to reduce the incidence of higher temperatures in the near-shore environment.

The temperature data for these two stations also provide evidence of an "urban heat island effect." Large urban complexes have been observed to exhibit higher air temperatures than surrounding rural areas. This temperature differential is greatest during the evening hours on clear days and is partly attributable to the numerous heat sources distributed throughout the urban environment. Another factor is the gradual loss of this heat to the atmosphere because of the dense pattern of urban structures emitting the radiating heat toward each other rather than into the open atmosphere as in rural areas, and because of the presence of atmospheric contaminants which form a barrier to nighttime radiation from the earth back to the atmosphere.

As shown in Table 10 and Figure 2, average daily minimum temperatures at Mitchell Field are consistently 2° F to 3° F higher than average daily minimum temperatures at the Germantown station. Moreover, although the annual average daily maximum temperature at Mitchell Field is on the order of 1° F lower than at Germantown, the annual average temperature at Mitchell Field is about 1° F higher than at the Germantown station. These differences may be the result of the heat island effect which causes the average minimum temperatures at Mitchell Field to be about 1° F to 2° F higher on an annual basis than the average minimum temperatures at the Germantown station.

Precipitation: Precipitation within the study area takes the form of rain, sleet, hail, and snow. Precipitation events may range from gentle showers of trace quantities to destructive thunderstorms, as well as major rainfall and snowmelt events causing property damage, inundation of poorly drained areas, and stream flooding.

Table 11 and Figures 3 and 4 indicate the average precipitation by month for the climatic period 1951 to 1980 for the two meteorological stations in the study area. Table 11 also presents average snowfall data for these two stations for varying periods of record. The average annual total precipitation in the study area, based upon the numerical average of data from Mitchell Field and Germantown, is 29.8 inches expressed as water equivalent, while the average annual snowfall is 49.7 inches.

Average total monthly precipitation within the study area ranges from a low of 1.08 inches in February to a high of 3.55 inches in July. The principal snowfall months are December, January, February, and March, when average snowfalls are 11.7, 11.8, 10.0, and 10.9 inches, respectively, and during which time about 92 percent of the average annual snowfall may be expected to occur. Snowfall is the predominant form of precipitation during these months, totaling about 60 percent of the total precipitation expressed as water equivalent.

More than 19 inches, or about 64 percent, of the average annual precipitation normally occurs during the April through October growing season, primarily as rainfall. Assuming that 10 inches of measured snowfall is equivalent to one inch of water, the average annual snowfall of 49.7 inches is equal to about 4.97 inches of water; therefore, only about 16 percent of the average annual precipitation occurs as snowfall.

Snow Cover: The likelihood of snow cover and the depth of that cover on the ground are important factors influencing the planning, design, construction, and maintenance of stormwater drainage and flood control facilities. Because snow acts as a thermal insulator, snow cover, particularly early in the winter season, significantly influences the depth and the duration of frozen ground, which in turn affects runoff as well as certain types of engineered works.

Snow depth as measured at Milwaukee for the 70-year period 1900 through 1969, and published in Snow and Frost in Wisconsin, a Wisconsin Statistical Reporting Service publication, is summarized in Table 12. It should be noted that the tabulated data pertaining to snow depth on the ground, as measured at the time and place of observation, are not a direct measure of average snowfall. Recognizing that snowfall and temperatures, and therefore snow accumulation on the ground, vary spatially within the study area, the Milwaukee data presented in Table 12 should be considered only an

Table 11

PRECIPITATION CHARACTERISTICS AT SELECTED LOCATIONS WITHIN THE STUDY AREA

Month	Observation Station				Two Station Average	
	Milwaukee (Mitchell Field)		Germantown			
	Average Normal Precipitation (1951-1980)	Average Snow and Sleet (1951-1980)	Average Normal Precipitation (1951-1980)	Average Snow and Sleet (1961-1976)	Average Normal Precipitation (1951-1980)	Average Snow and Sleet (1961-1970)
January	1.64	13.5	1.04	10.2	1.34	11.8
February	1.33	10.5	0.83	9.4	1.08	10.0
March	2.58	10.1	1.84	11.7	2.21	10.9
April.	3.37	2.1	2.81	2.6	3.09	2.3
May	2.66	Trace	2.78	0.0	2.72	0.0
June	3.59	0.0	3.42	0.0	3.50	0.0
July	3.54	0.0	3.56	0.0	3.55	0.0
August	3.09	0.0	3.55	0.0	3.32	0.0
September . . .	2.88	Trace	3.14	Trace	3.01	0.0
October.	2.25	0.2	2.36	0.1	2.31	0.2
November . . .	1.98	3.4	1.97	2.2	1.97	2.8
December . . .	2.03	11.4	1.46	12.1	1.74	11.7
Annual	30.94	51.2	28.76	48.3	29.84	49.7

Source: National Climatic Center and SEWRPC.

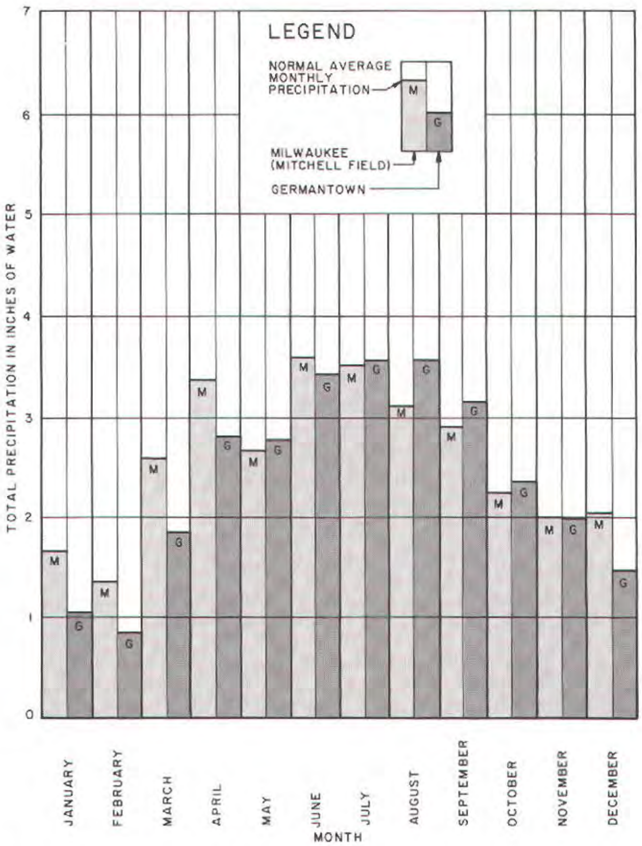
approximation of conditions throughout the entire study area. As indicated by the data, snow cover is most likely during the months of December, January, and February, during which there is at least a 40 percent probability of having one inch or more of snow cover in Milwaukee. Furthermore, during January and the first half of February, there is at least a 25 percent probability of having five or more inches of snow on the ground. During March, the month in which severe spring snowmelt-rainfall flood events are most likely to occur, there is at least a 30 percent probability of having one inch or more of snow on the ground during the first half of the month, with the probability of that amount of snow cover diminishing to 7 percent by the end of the month.

The data presented in Table 12 can be used to estimate the probability that a given snow cover will exist or be exceeded at any given time. It should, therefore, be useful in planning winter outdoor work construction activities, as well as in estimating runoff for hydrologic purposes. There is, for example, a 7 percent probability of having one inch or more of snow cover on November 15 of any year, whereas there is a much higher probability, 61 percent, of having that much snow cover on January 15.

Frost Depth: Ground frost, or frozen ground, refers to that condition in which the ground contains variable amounts of water in the form of ice. Frost influences hydrologic processes, particularly the percent of rainfall and snowmelt that will run off the land directly into storm sewers and surface watercourses, in contrast to that which will enter and be temporarily detained in the soil. Snow cover is a primary determinant of the depth of frost penetration and of the duration of frozen ground. Thermal conductivity of snow cover is less than one-fifth that of moist soil, and thus heat loss from the soil to the cold atmosphere is greatly inhibited by an insulating snow cover. Frost conditions in the Region were published by the Wisconsin Agricultural Reporting Service for the months of November through April, based upon an eight-year period of record for 1961 through 1977, and are summarized on a semi-monthly basis in Table 13. Table 13 indicates that frozen ground is likely to exist in the study area for approximately

Figure 3

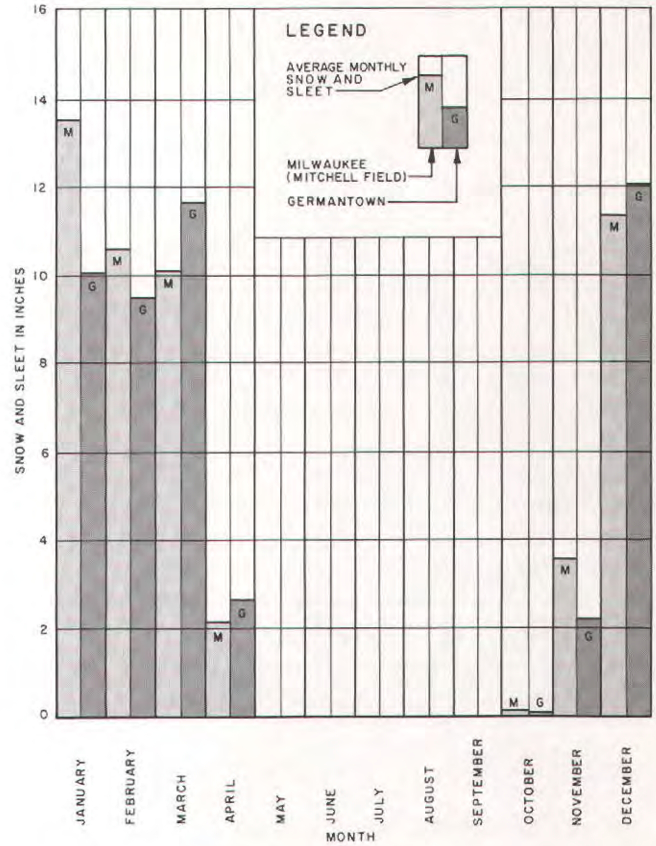
PRECIPITATION CHARACTERISTICS AT SELECTED LOCATIONS WITHIN THE STUDY AREA



Source: National Climatic Center and SEWRPC.

Figure 4

AVERAGE MONTHLY SNOW AND SLEET DATA AT SELECTED LOCATIONS WITHIN THE STUDY AREA



Source: National Climatic Center and SEWRPC.

four months each winter season, extending from late November through March, with more than six inches of frost normally occurring in January, February, and the first half of March. Historical data indicate that the most severe frost conditions normally occur in February, when 15 or more inches of frost may be expected in unpaved areas. However, since stormwater drainage facilities are often located in areas covered by roadway pavements, or in adjacent areas which are frequently cleared of snow cover, stormwater management planning must also consider substantially deeper frost penetration depths. The City of West Allis Engineering Department reports frost penetration depths of up to 48 inches being observed under roadway pavements in the vicinity of water mains that were damaged by frost action. This observation is consistent with engineering practice in the Milwaukee area which generally provides for a minimum cover of 36 to 48 inches over utility facilities to avoid freezing problems.

Physiographic and Topographic Features

An understanding of the physiography and topography of the study area can aid in understanding the hydrologic and hydraulic characteristics of the service water drainage system.

Watersheds Comprising the Study Area: As indicated in Table 14 and shown on Map 12, the study area is comprised of all or portions of seven watersheds, including 24.9 square miles—or all—of the Kinnickinnic River watershed, which encompasses about 6.6 percent of the study area; 135.9 square miles—or all—of the Menomonee River watershed, which comprises about 35.9 percent of the study area; 92.6 square miles, or about 13.4 percent, of the Milwaukee River watershed, which comprises about 24.5 percent of the study area; 27.2 square miles—or all—of the Oak Creek watershed, which comprises about 7.2 percent of the study area; 74.8 square miles, or about 38 percent, of the Root River watershed, which comprises 19.8 per-

Table 12

SNOW COVER PROBABILITIES AT MILWAUKEE BASED ON DATA FOR THE PERIOD 1900-1969

Date		Snow Cover[a]									
		1 Inch or More		5 Inches or More		10 Inches or More		15 Inches or More		Average (inches)	
Month	Day	Number of Occurrences[b]	Probability of Occurrence[c]	Number of Occurrences[b]	Probability of Occurrence[c]	Number of Occurrences[b]	Probability of Occurrence[c]	Number of Occurrences[b]	Probability of Occurrence[c]	Per Occurrence[d]	Overall[e]
November	15	5	0.07	0	0.00	0	0.00	0	0.00	1.2	0.09
	30	12	0.17	1	0.01	1	0.01	0	0.00	2.8	0.49
December	15	33	0.47	10	0.14	0	0.00	0	0.00	3.3	1.54
	31	32	0.46	9	0.13	1	0.01	0	0.00	3.6	1.66
January	15	43	0.61	17	0.24	4	0.06	2	0.03	4.9	2.94
	31	48	0.69	22	0.31	9	0.13	4	0.06	6.2	4.26
February	15	44	0.63	23	0.33	7	0.10	3	0.04	6.0	3.69
	28	27	0.39	8	0.11	3	0.04	1	0.01	4.5	1.69
March	15	23	0.33	6	0.09	4	0.06	0	0.00	3.9	1.21
	31	5	0.07	1	0.01	1	0.01	0	0.00	3.4	0.24

[a] *Data pertain to snow depth on the ground as it was measured at the time and place of observation, and are not a direct measure of average snowfall.*

[b] *Number of occurrences is the number of times during the 70-year period of record when measurements revealed that the indicated snow depth was equaled or exceeded on the indicated date.*

[c] *Probability of occurrence for a given snow depth and date is computed by dividing the number of occurrences by 70, and is defined as the probability that the indicated snow cover will be reached or exceeded on the indicated date.*

[d] *Average snow cover per occurrence is defined as the sum of all snow cover measurements in inches for the indicated date divided by the number of occurrences for that date—that is, the number of times in which 1.0 inch or more snow cover was recorded.*

[e] *Overall average snow cover is defined as the sum of all snow cover measurements in inches for the indicated date divided by 70—that is, the number of observation times.*

Source: National Weather Service, Wisconsin Statistical Reporting Service, and SEWRPC.

cent of the study area; and 22.8 square miles, or 23.9 percent, of the land directly tributary to Lake Michigan, which comprises about 6 percent of the study area. In addition, about one-half square mile of the Fox River watershed located in the southwest portion of the City of Franklin is within the study area. This represents less than one-half of 1 percent of the total study area acreage.

Physiographic Features: Physiographic features or surficial land forms have been determined largely by the underlying bedrock and the overlying glacial deposits of the watersheds. The major surficial land forms of the study area resulting from this glaciation are shown on Map 13.

The Niagara cuesta, on which the study area lies, is a gently eastward sloping bedrock surface, with the eastern border of the study area being about 300 feet lower in elevation than the western border. Glacial deposits overlying the bedrock formations form the irregular surface topography of the study area characterized by rounded hills or groups of hills, ridges, broad underlying, undulating plains, and poorly drained wetlands.

Table 13

AVERAGE FROST DEPTH IN SOUTHEASTERN WISCONSIN: NOVEMBER TO APRIL

Month and Day	Nominal Frost Depth (inches)[a]
November 30	1
December 15	3
December 31	4
January 15.	9
January 31.	12
February 15	14
February 28	15
March 15	13
March 31	7
April 7	3

[a] *Based on 1961-1977 frost depth data for cemeteries as reported by funeral directors and cemetery officials. Since cemeteries have soils that are overlain by an insulating layer of turf, the mapped frost depths should be considered minimum values.*

Source: Wisconsin Agricultural Reporting Service, Snow and Frost in Wisconsin, October 1978.

Table 14

AREAL EXTENT OF WATERSHEDS IN THE STUDY AREA

Watershed	Total Watershed Area (square miles)	Watershed Area Included in Study Area (square miles)	Percent of Watershed Within Study Area	Percent of Study Area Within Watershed
Fox	934.3	0.5	--[a]	--[a]
Kinnickinnic	24.9	24.9	100.0	6.6
Menomonee	135.9	135.9	100.0	35.9
Milwaukee	692.1	92.6	13.4	24.5
Oak Creek	27.2	27.2	100.0	7.2
Root	196.9	74.8	38.0	19.8
Lake Michigan Direct Drainage	95.5	22.8	23.9	6.0
Total	--	378.7	--	100.0

[a] *Less than one-tenth of 1 percent.*

Source: SEWRPC.

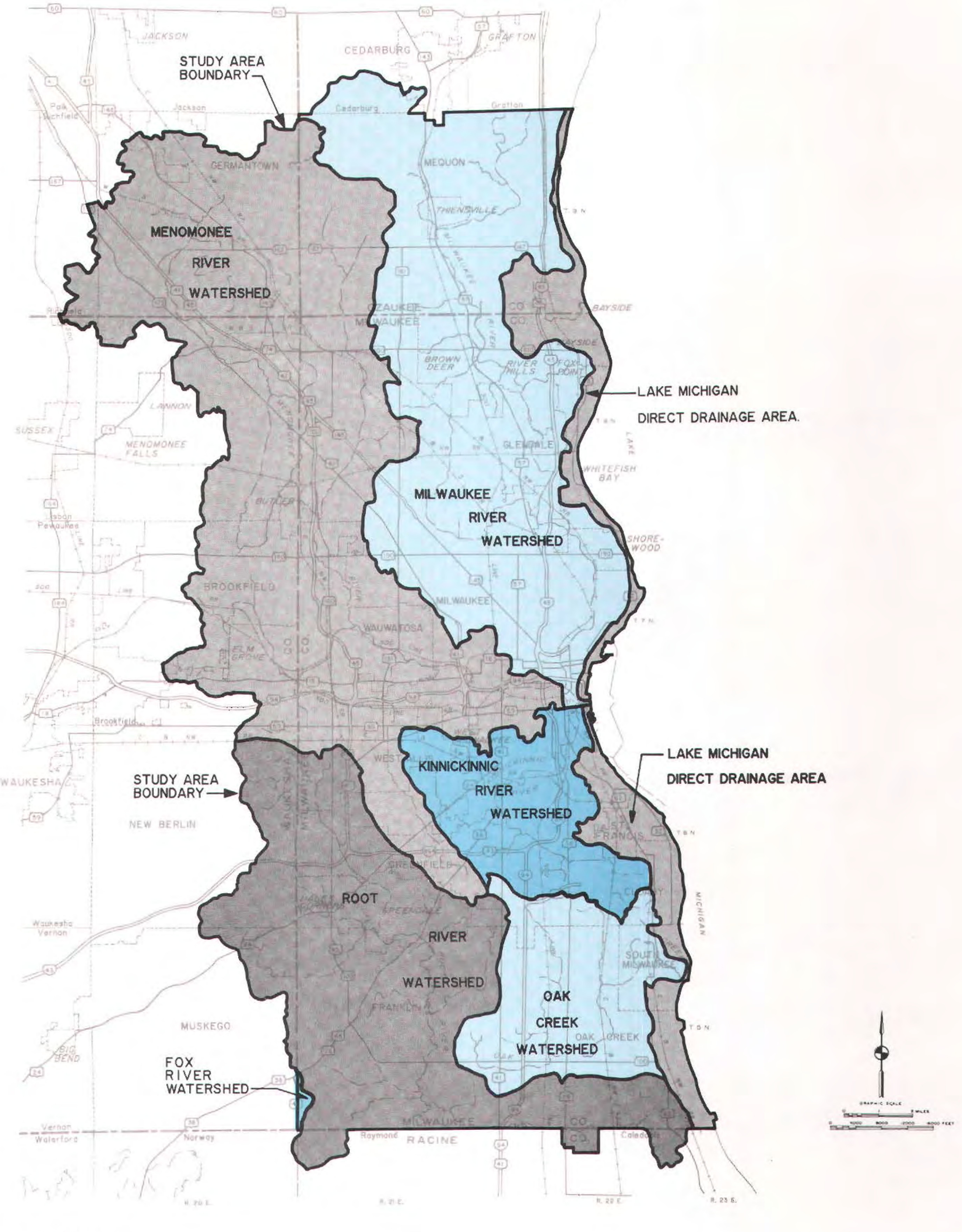
Topography: The topography or variation in elevation of the study area is an important factor determining the hydrologic response within the study area to rainfall and rainfall-snowmelt events.

The Commission prepares, and encourages local units of government to prepare, one inch equals 100 feet scale, and one inch equals 200 feet scale, two-foot contour interval topographic maps based on a Commission-recommended monumented control survey network relating the U. S. Public Land Survey System to the State Plane Coordinate System. As shown on Map 14, large-scale topographic maps prepared to Commission standards, which include monumented control, are available for 222 square miles, or 59 percent, of the study area in 1987. A total of 1,412 U. S. Public Land Survey Corners in the study area have been, or are being, relocated, monumented, and tied in to the State Plane Coordinate System, representing 80.6 percent of such corners in the study area. These large-scale topographic maps facilitate the hydrologic and hydraulic studies required for drainage and flood control planning and engineering, including the delineation of drainage basins, stream network configuration, and stream profiles and cross-sections. The maps are essential for the accurate and precise delineation of flood hazard areas along streams and watercourses and in the determination of monetary flood damages. Finally, such maps are essential for the sound preliminary and final engineering of required drainage and flood control improvements, for the acquisition of flood hazard areas for park and open space purposes, and for the proper exercise of public land use controls to protect floodwater conveyance and storage capacities. These maps are also useful for many other types of municipal planning and public works functions.

As shown on Map 15, surface elevations within the study area range from a high of over 1,100 feet above National Geodetic Vertical Datum (NGVD) —Mean Sea Level Datum—in the Town of Richfield, a portion of the Menomonee River watershed located in the northwest area of the study area, to approximately 580 feet above NGVD at the mouth of the Milwaukee River as it enters Lake Michigan. Most of the study area is covered by gently sloping ground moraine—heterogeneous materials deposited beneath the ice, end moraines consisting of material deposited at the forward margins of the ice sheet, and outwash plains formed by the action of flowing glacial meltwater.

Map 12

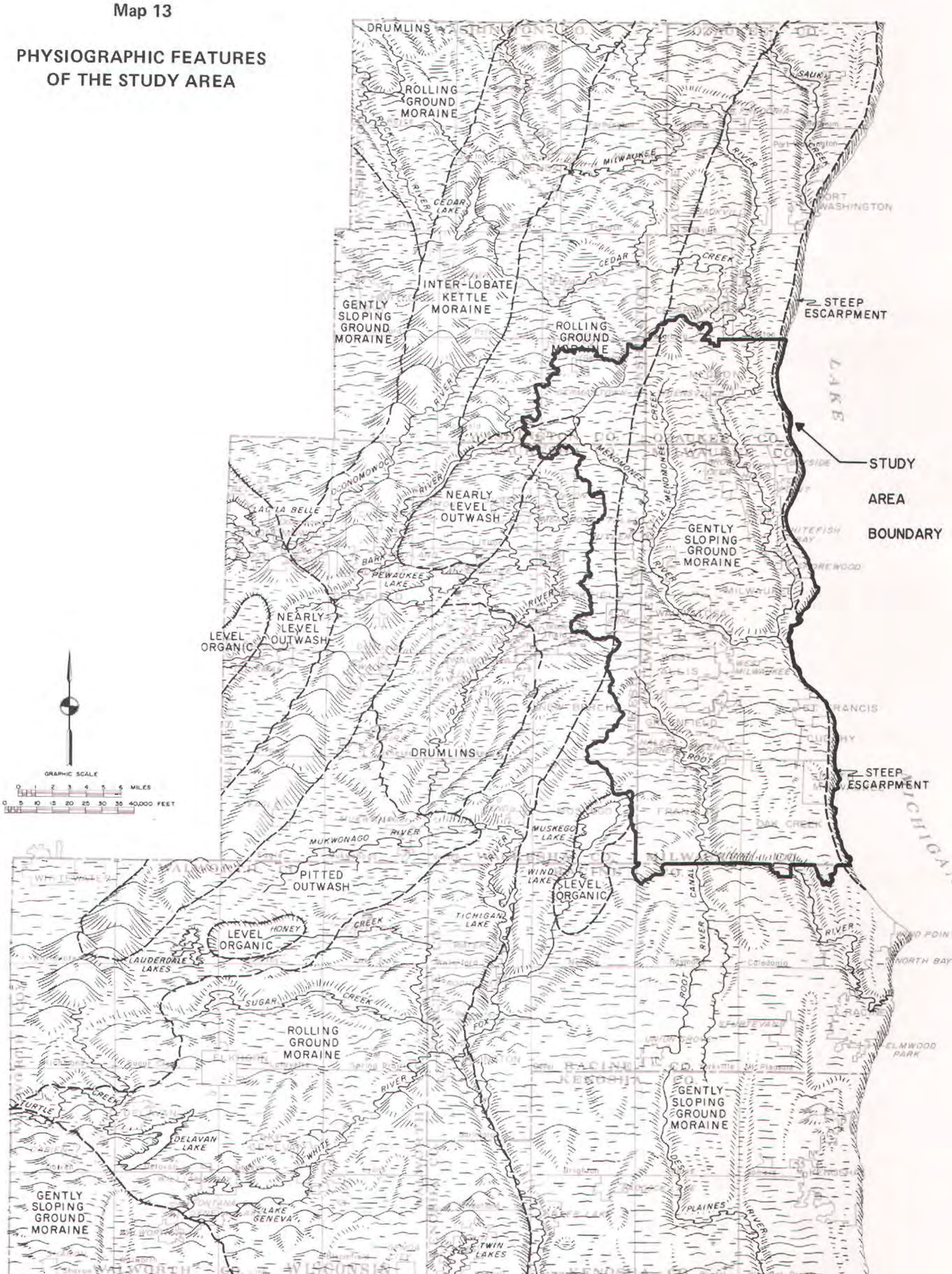
WATERSHEDS IN THE STUDY AREA



Source: SEWRPC.

Map 13

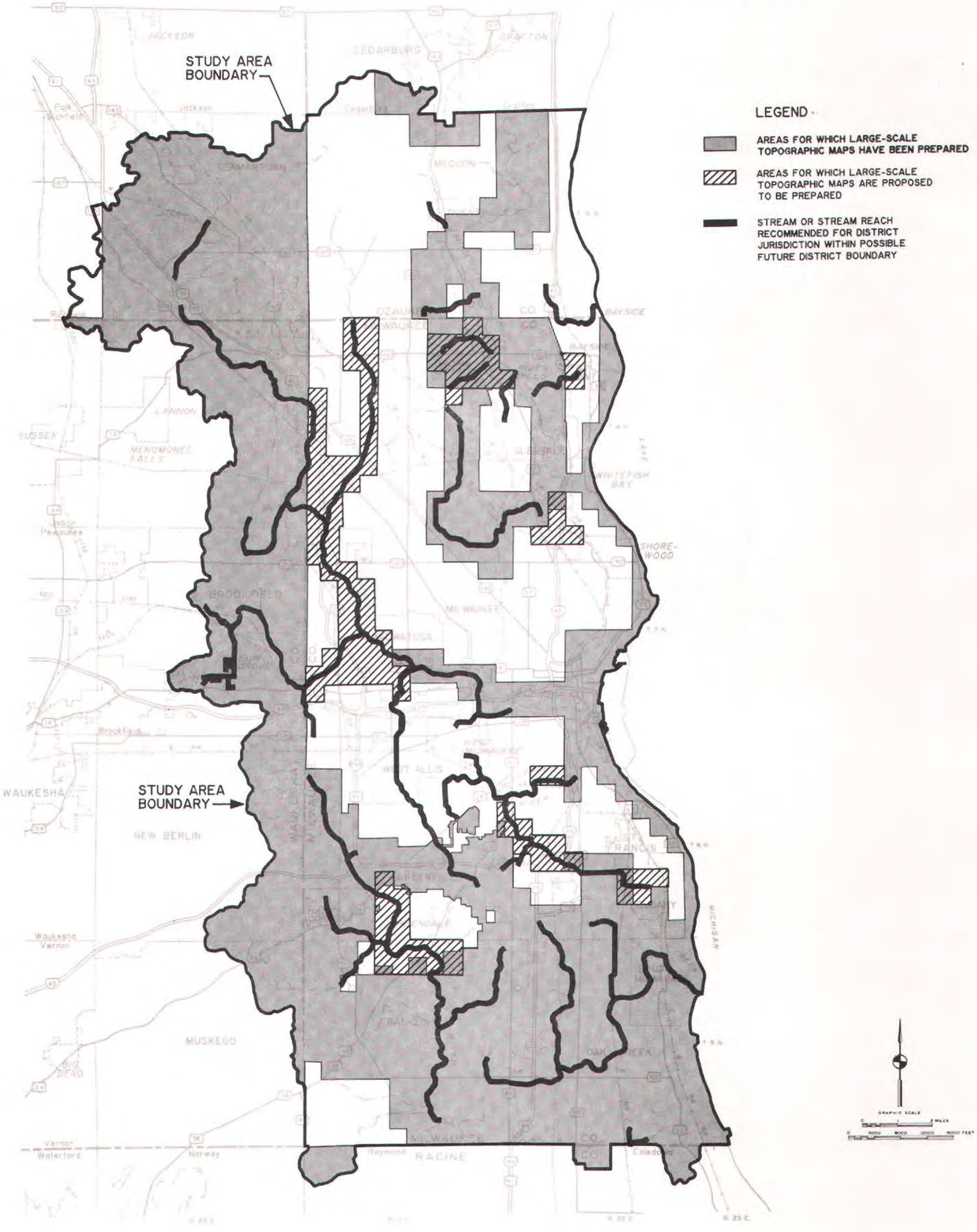
PHYSIOGRAPHIC FEATURES OF THE STUDY AREA



Source: SEWRPC.

Map 14

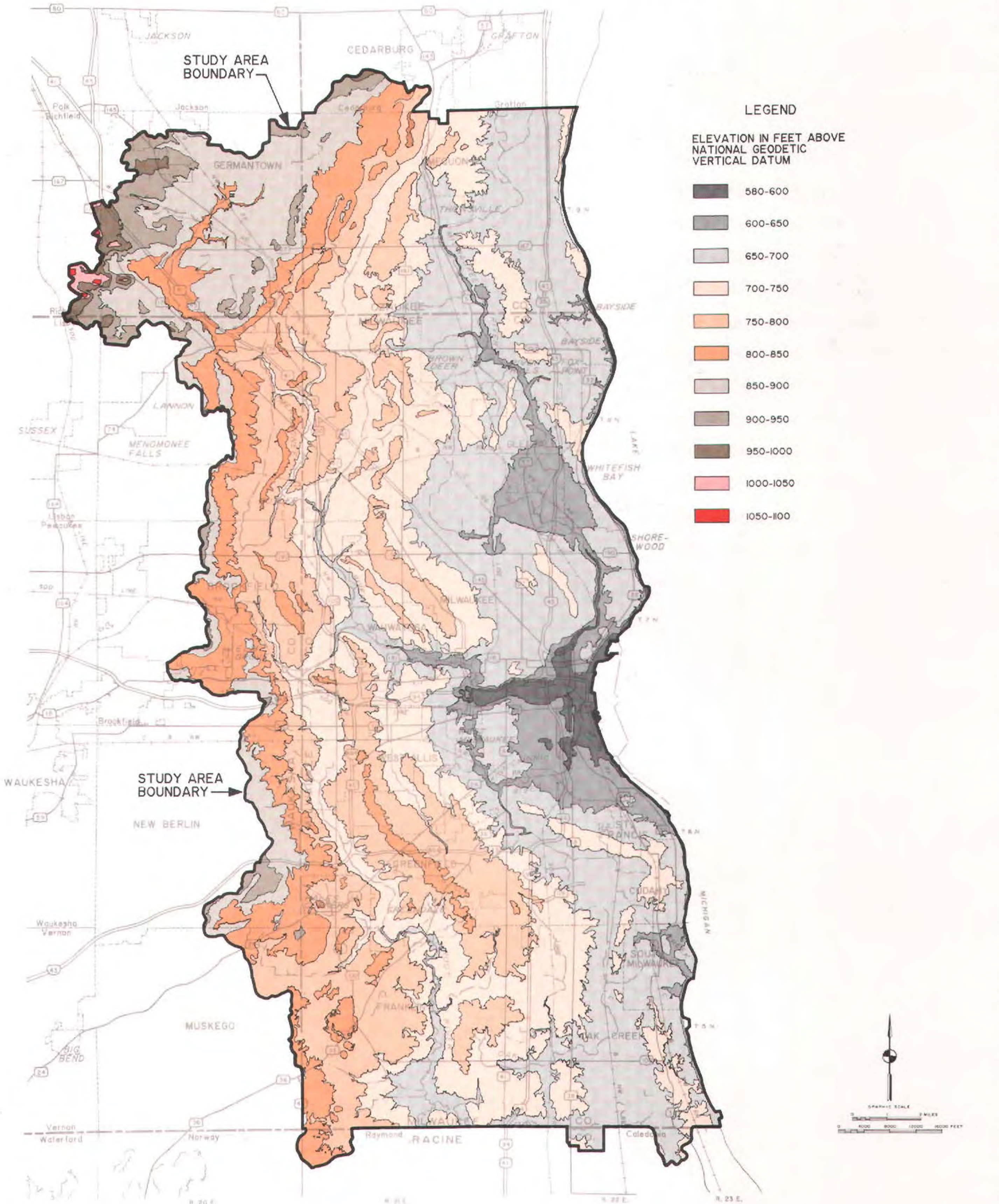
AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPS IN THE STUDY AREA: 1987



Source: SEWRPC.

Map 15

TOPOGRAPHIC CHARACTERISTICS OF THE STUDY AREA



Source: SEWRPC.

Geology

The bedrock formations underlying the unconsolidated surficial deposits of the study area consist of Precambrian crystalline rocks; Cambrian sandstone; Ordovician dolomite, sandstone, and shale; and Silurian and Devonian dolomite. All of these rock units slope toward the east. The bedrock geology of the study area is shown on Map 16, a map of the surface of the bedrock, and is supplemented by Figure 5, which presents two vertical sections through the study area. The uppermost bedrock unit throughout most of the study area is Silurian dolomite, primarily Niagara dolomite underlain by a relatively impervious layer of Maquoketa shale.

Bedrock topography was shaped by preglacial and glacial erosion of the exposed bedrock. The consolidated bedrock underlying the study area generally dips eastward at a rate of 25 to 30 feet per mile. The bedrock surface ranges from about 800 feet NGVD in the western part of the study area to less than 400 feet NGVD at the mouth of the Milwaukee River. The glacial deposits above the bedrock include end moraine, ground moraine, outwash, and lake-basin deposits.

The combined thickness of unconsolidated glacial deposits, alluvium, and marsh deposits varies from less than 20 feet, with scattered bedrock outcrops, in the northern portion of the study area, to about 400 feet in south-central Milwaukee County. Map 17 indicates the spatial variation of the thickness of unconsolidated deposits overlying the bedrock in the study area.

Because of the glacial deposits, southeastern Wisconsin has few bedrock exposures, either natural or artificial, available for scientific or recreation purposes. Most of the exposures that were formerly available have been destroyed. In addition, the natural exposures are not very extensive, and many of these have also been covered. There has been little or no effort in the past to preserve any of these bedrock exposures, whether natural or artificial, even though many were located in public parks. This failure to preserve these outcroppings can be attributed to a lack of appreciation of the value of such rock exposures.

The value of these exposures is demonstrated by the many different usages possible. Of particular importance is their historic value from both a scientific and industrial standpoint. These exposures have provided significant evidence of value to the understanding of local and regional geology, and to establishing some new geologic concepts such as the presence of fossil reefs. On a local scale it is impossible to study the bedrock geology when there are few or no exposures available. This presents problems when new geologic concepts are proposed, since it is important for concerned scientists to be able to reexamine exposures at such times, no matter how thoroughly such exposures may have been studied in the past.

The industrial history of the Region is also related to these rock exposures. Some of the earliest business ventures in Milwaukee County were quarries. These quarries provided commodities, such as dimensional building stone, crushed stone, lime, and natural cement, which were necessary for the development of the Region.

Rock exposures may also play an important role in education. Field trips are an important teaching aid in elementary school, high school, and university science and geology classes. The location of rock exposures near an urban area makes such learning devices available to a large number of students. These rock exposures also provide research material for university-level thesis projects.

The locations of important geologic sites in Milwaukee County are shown on Map 18 and more fully described in SEWRPC Technical Record Vol. 4, No. 3, February 1982, "Preservation of Scientifically and Historically Important Geologic Sites in Milwaukee County, Wisconsin." Most of these sites are threatened by various construction projects—including stormwater drainage and flood control improvements—and thus efforts to preserve them must be taken now before the sites are covered or access to them is otherwise lost.

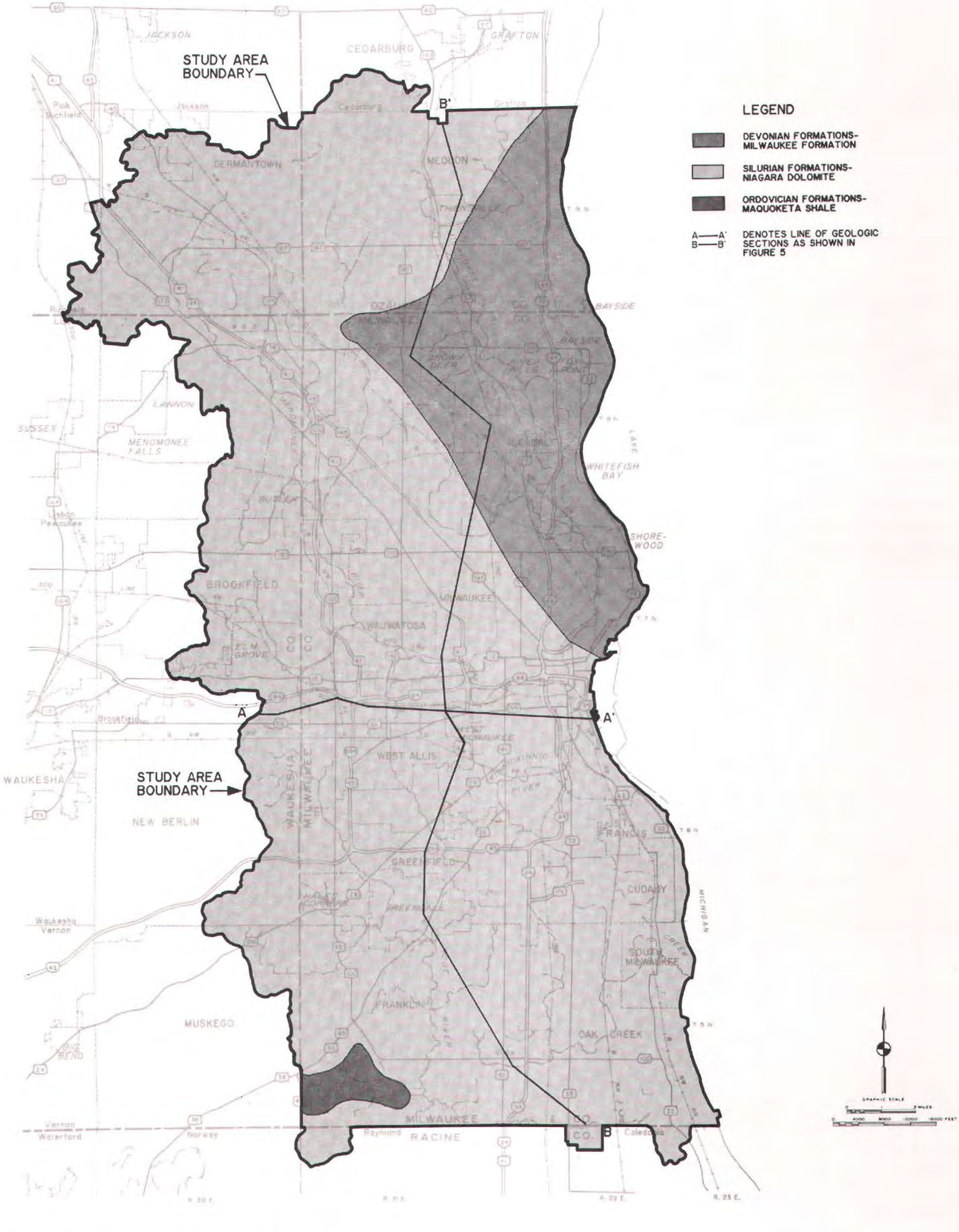
Soils

The nature of soils within the study area has been determined primarily by the interaction of the parent glacial deposits covering the study area and by the topography, climate, plants, animals, and time.

To assess the significance of the diverse soils found in southeastern Wisconsin, the Southeastern Wisconsin Regional Planning Commission in 1963 negotiated a cooperative agreement with the U. S. Soil Conservation Service under which detailed operational soil surveys were completed for the entire Planning Region. The results of the soil surveys are published in SEWRPC Planning Report No. 8, Soils of Southeastern Wisconsin. The regional soil surveys, a sample of which is presented in Figure 6, have resulted in the mapping of

Map 16

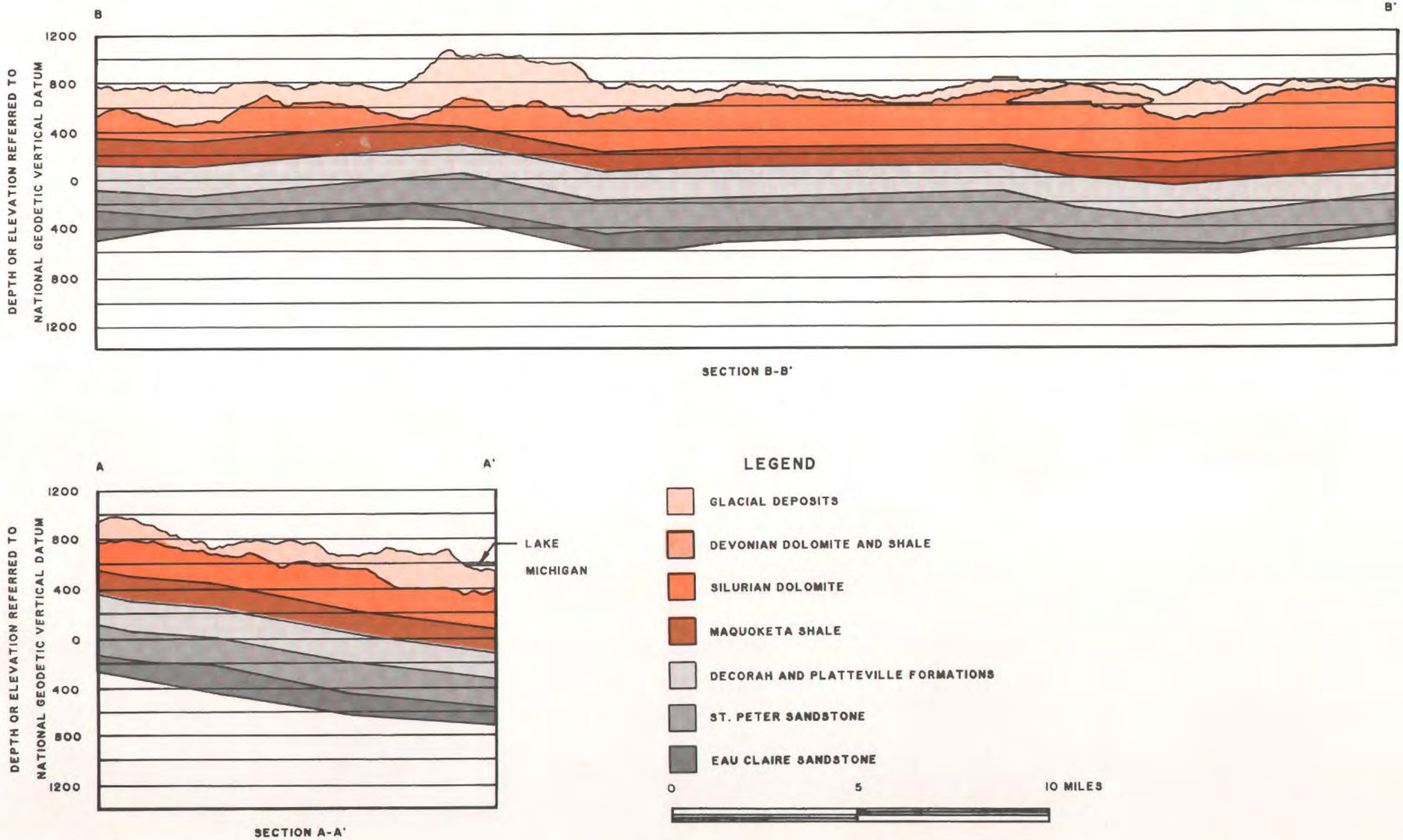
BEDROCK GEOLOGY OF THE STUDY AREA



Source: U. S. Geological Survey and SEWRPC.

Figure 5

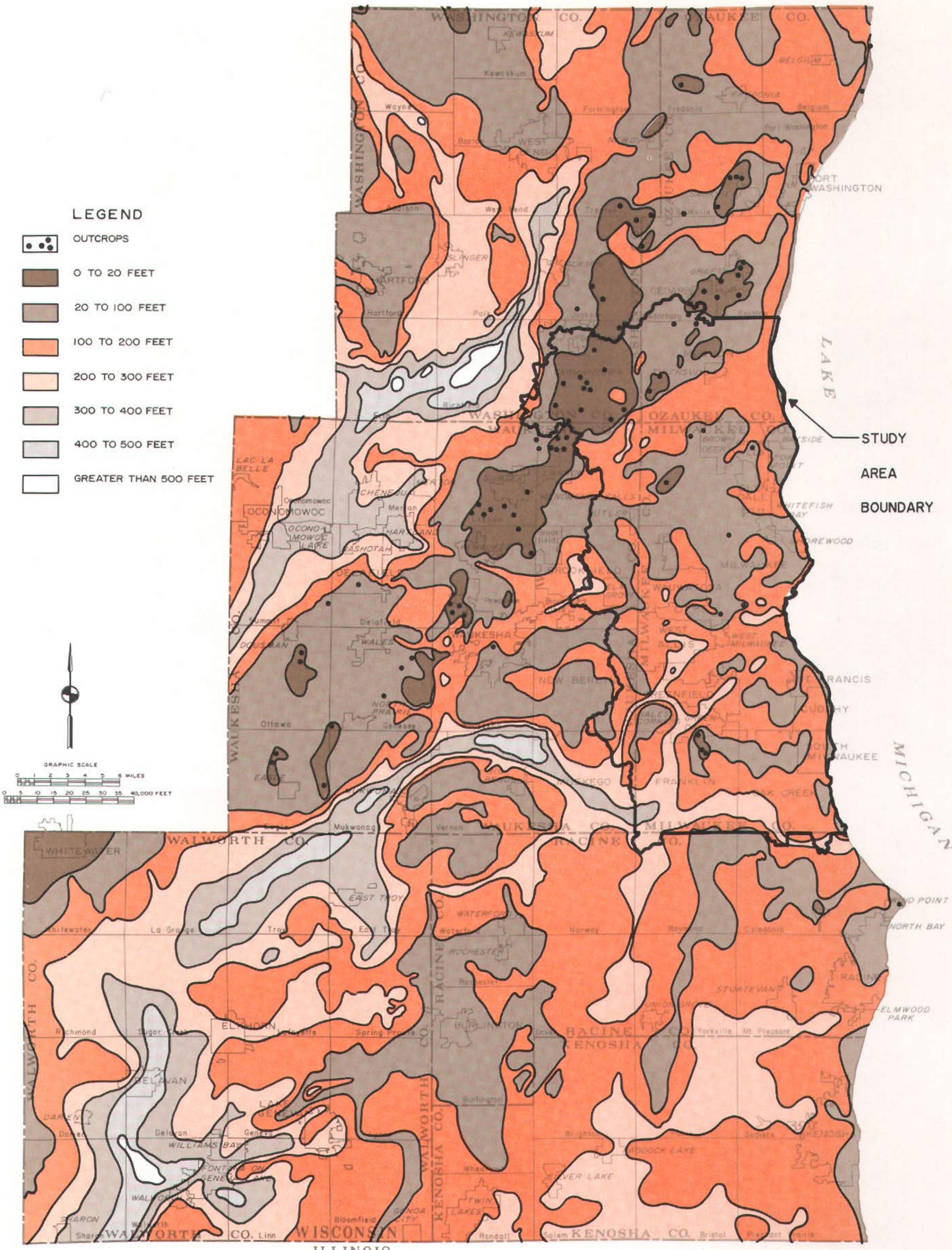
PROFILE OF GEOLOGIC STRUCTURE IN THE STUDY AREA



Source: U. S. Geological Survey and SEWRPC.

Map 17

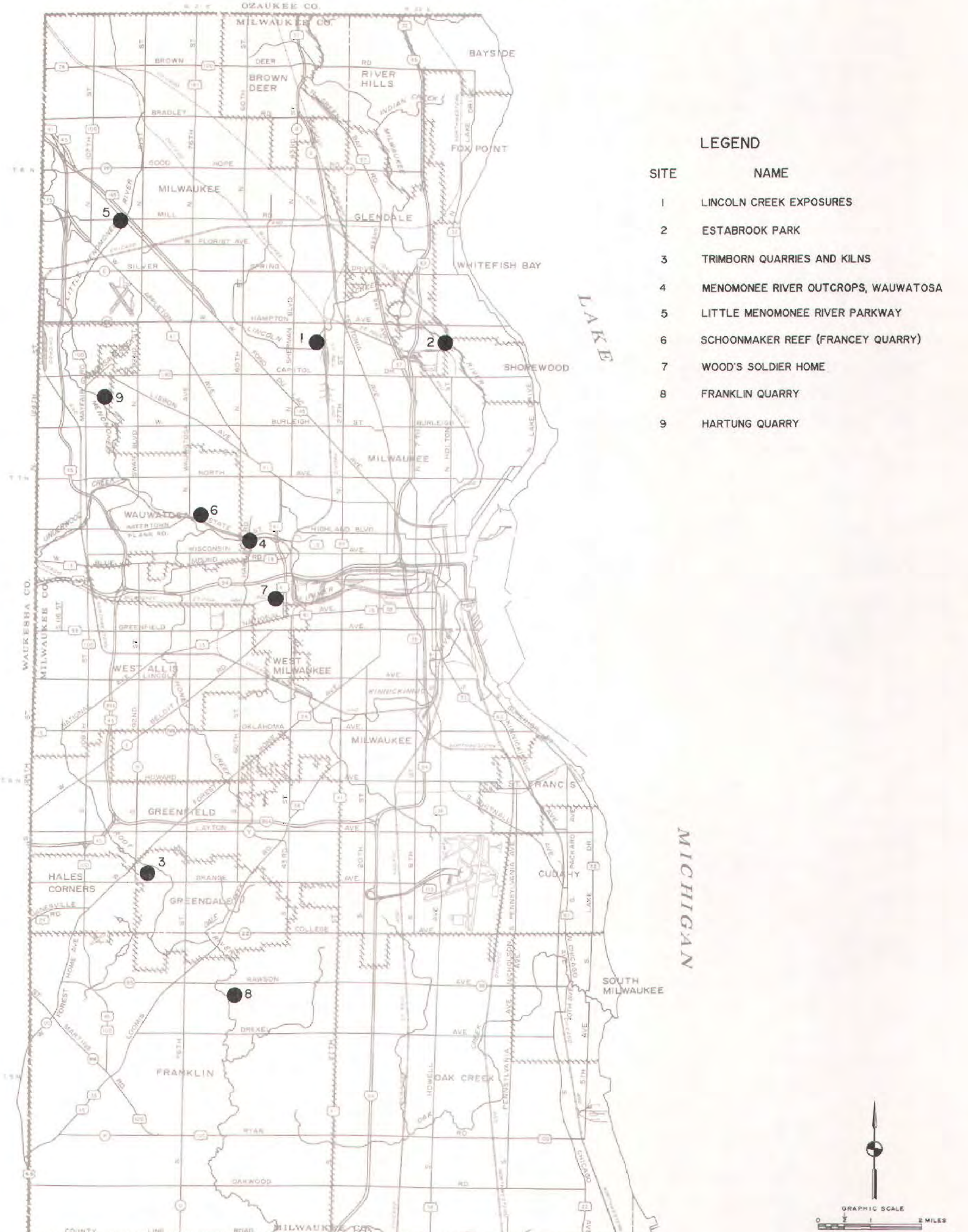
THICKNESS OF GLACIAL DEPOSITS AND THE LOCATION OF BEDROCK OUTCROPS IN THE STUDY AREA



Source: SEWRPC.

Map 18

LOCATION OF IMPORTANT GEOLOGIC SITES IN MILWAUKEE COUNTY



LEGEND

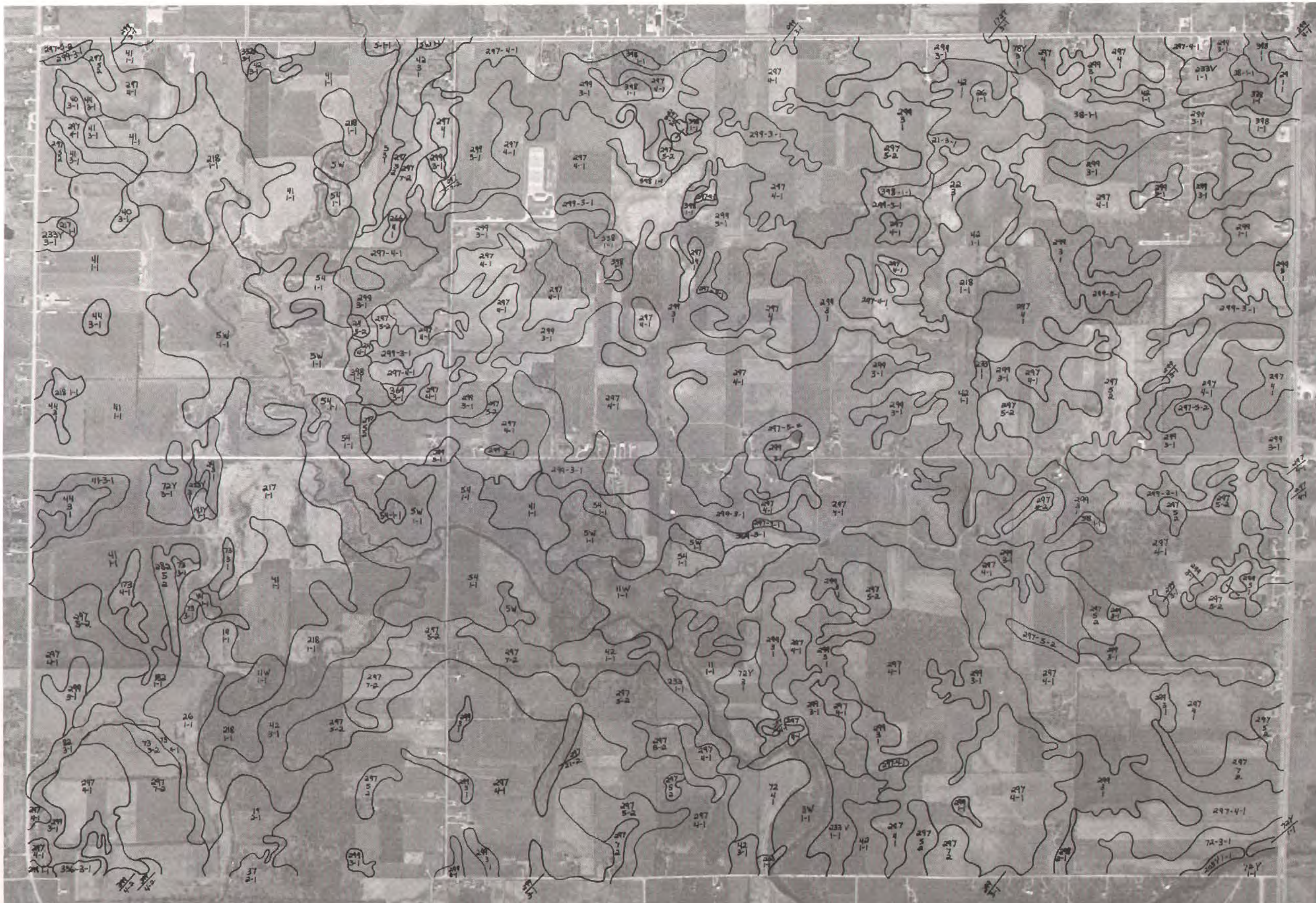
SITE	NAME
1	LINCOLN CREEK EXPOSURES
2	ESTABROOK PARK
3	TRIMBORN QUARRIES AND KILNS
4	MENOMONEE RIVER OUTCROPS, WAUWATOSA
5	LITTLE MENOMONEE RIVER PARKWAY
6	SCHOONMAKER REEF (FRANCEY QUARRY)
7	WOOD'S SOLDIER HOME
8	FRANKLIN QUARRY
9	HARTUNG QUARRY

Source: Donald Mikulic and Joanne Klussendorf.

Figure 6

SOIL SURVEY SHEET, MILWAUKEE COUNTY, WISCONSIN

U. S. Public Land Survey Sections 25, 26, 27, 34, 35, and 36, Township 5 North, Range 21 East



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

the soils within the Region in great detail. At the same time, the surveys have provided data on the physical, chemical, and biological properties of the soils and, more importantly, have provided interpretations of the soil properties for planning, engineering, agricultural, and resource conservation purposes.

Soil properties are an important factor influencing the rate and volume of stormwater runoff from land surfaces. The type of soil is also an important consideration in the evaluation of shallow groundwater aquifer recharge and stormwater storage. The soil characteristics and the slope and vegetative cover of the land surface also affect the degree of soil erosion which occurs during runoff events.

With respect to watershed hydrology, the most significant soil interpretation for stormwater management is the categorization of soils into hydrologic soil groups A, B, C, and D. In terms of runoff characteristics, these four hydrologic soil groups are defined as follows:

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

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

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

- Hydrologic Soil Group D: Very large amounts of runoff because of very low infiltration, low permeability, and extremely poor drainage.

The spatial distribution of the four hydrologic soil groups within the study area is shown on Map 19. Hydrologic soil groups A, B, C, and D cover 1 percent, 13 percent, 49 percent, and 14 percent, respectively, of the study area. The remaining 23 percent is covered by disturbed soils. It is important to note that nearly 63 percent of the study area is covered by soils having poor or very poor drainage characteristics, which therefore may be expected to generate relatively large amounts of stormwater runoff.

Vegetation

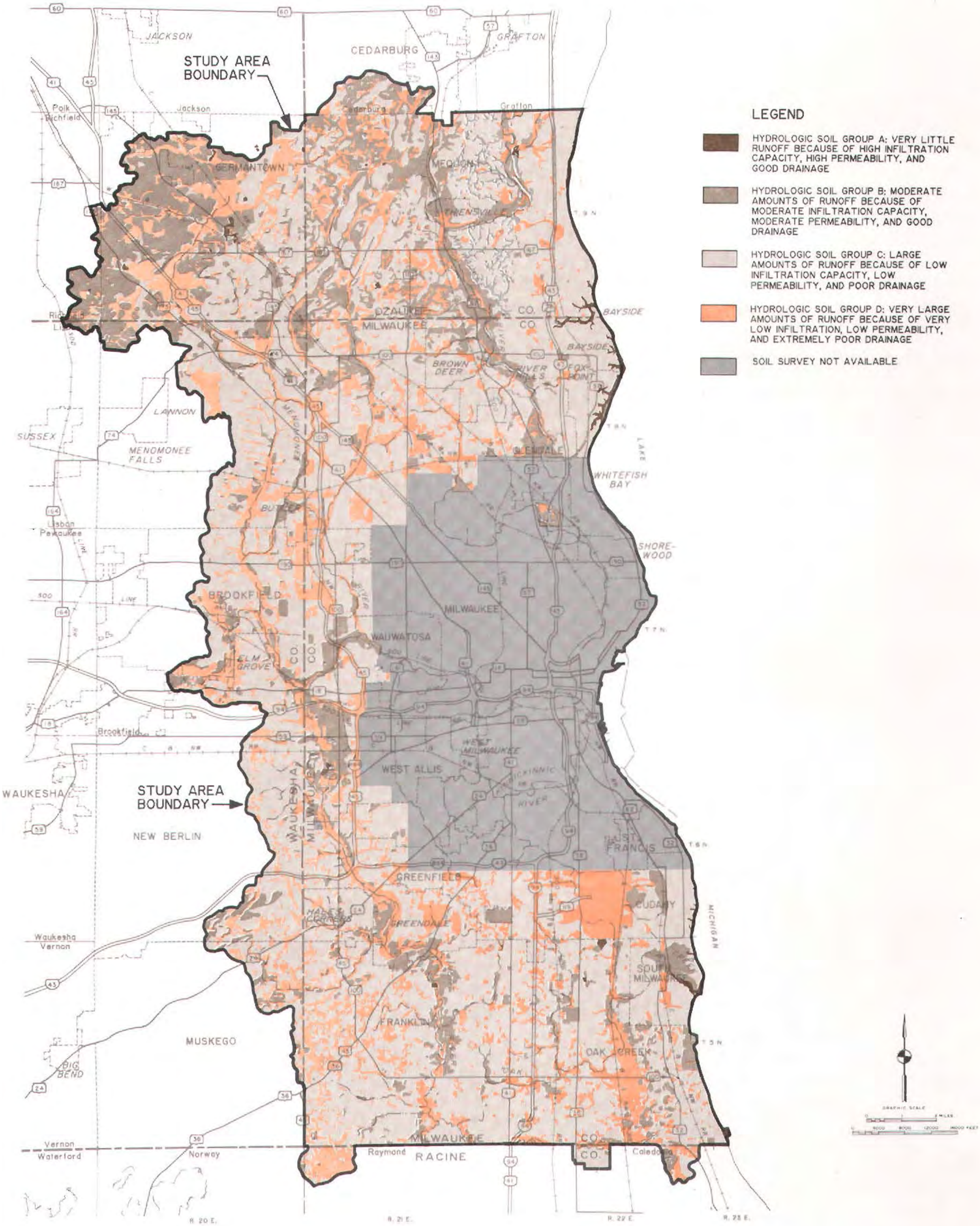
Vegetation in any location at any given time is determined by, or the result of, a variety of factors, including climate, topography, occurrence of fire, soil characteristics, proximity of bedrock, drainage features, and, of course, the activities of man. Because of the temporal and spatial variability of these factors and the sensitivity of vegetation to most of them, vegetation throughout the study area has been a changing mosaic of different types. The terrestrial vegetation in the study area occupies sites which may be divided into two broad land classifications: wetland and woodland. Wetlands are defined as those lands which are wholly or partially covered with hydrophytic plants and wet and spongy organic soils and which are generally covered with shallow standing water intermittently inundated or having a high water table.

Chapters NR 115 and NR 117 of the Wisconsin Administrative Code require counties and incorporated communities in Wisconsin to place wetlands within shoreland areas in a shoreland/wetland zoning district. Implementation of the shoreland/wetland zoning provision of these chapters will ensure the preservation of many wetland areas within the Region and throughout the State. By law, shorelands are defined as all areas located within 1,000 feet of the ordinary high-water mark of a navigable lake, pond, or flowage; or within 300 feet of the ordinary high-water mark of a navigable river or stream, or to the landward edge of the floodplain, whichever distance is greater. Shoreland/wetland zoning regulations and any subsequent proposed amendments are subject to review and approval by the Wisconsin Department of Natural Resources (DNR), thus making wetland zoning in effect joint state-local zoning.

Wetland inventory maps at a scale of one inch equals 2,000 feet were prepared for the seven southeastern Wisconsin counties by the Regional Planning Commission under contract to the DNR, and provided a basis for the regulation of shoreland/wetlands within the Region under Chapters NR 115 and NR 117. Upon receipt of the preliminary wetland inventory maps from the DNR, each county or local municipality concerned was given 90 days to review the maps and subsequently hold a public hearing to receive comments on the accuracy and completeness of the maps. After the required public hearing, each county or local municipality concerned was required to return the inventory maps to the DNR, annotated to identify any areas believed to be incorrectly designated.

Map 19

HYDROLOGIC SOIL GROUPS IN THE STUDY AREA



Source: SEWRPC.

The DNR then scheduled a meeting to discuss the potential inaccuracies of the preliminary maps, make the necessary corrections, and transmit final wetland maps to the county or local municipality concerned. The county or local municipality then had six months to amend its shoreland/wetland zoning to protect the final mapped wetlands. As of June 1988, final wetland maps have been prepared for 19 communities within the MMSD study area, including the Cities of Glendale and Oak Creek and the Villages of Greendale and River Hills in Milwaukee County; the City of Mequon and Town of Cedarburg in Ozaukee County; the Towns of Caledonia, Norway, and Raymond in Racine County; the Village of Germantown and the Towns of Germantown and Richfield in Washington County; and the Cities of Brookfield, Muskego, and New Berlin, the Villages of Elm Grove and Menomonee Falls, and the Towns of Brookfield and Lisbon in Waukesha County.

Woodlands are defined as those upland areas, one acre or more in size, having 17 or more deciduous trees per acre, each measuring at least four inches in diameter at breast height, and having at least a 50 percent canopy cover. In addition, coniferous tree plantations and reforestation projects are identified as woodlands.

The location, extent, type, and quality of wetland and woodland areas are key determinants of the environmental quality of the study area. Wetland and woodland areas can, for example, support a variety of outdoor recreation activities. They contribute to the beauty and visual diversity of the environment, and function as visual and acoustical shields or barriers. Such areas and the vegetation contained within them serve important ecological functions, since they are typically, on a unit-area basis, the biologically most productive portions of the study area; provide continuous wildlife range and sanctuary for native biota; and help to maintain surface water quality by functioning as sediment and nutrient traps. Finally, certain wetland and woodland areas can be excellent outdoor laboratories for educational and research activities.

Because of the heavily urbanized nature of the study area, in 1985 wetlands covered only 17.7 square miles, or about 5 percent, of the study area; while woodlands covered about 12.5 square miles, or 3 percent, of the study area. The spatial distribution of these wetlands and woodlands is shown on Map 20.

Water Resources

Surface water resources consisting of streams and associated floodlands form the singularly most important element of the natural resource base in the study area. Their contribution to the study area in terms of economic development, recreational activity, and aesthetic quality is immeasurable. The groundwater resources of the study area are hydraulically connected to the surface water resources, inasmuch as they provide the base flows of the streams. The groundwater resources, along with Lake Michigan, constitute the major sources of supply for domestic, municipal, and industrial water uses.

Surface Water Resources: There are no major lakes, that is lakes having 50 acres or more of surface area, located within the study area. There are, however, a number of minor lakes within the study area which, together with the surface areas of the streams within the study area, encompass approximately three square miles, or about eight-tenths of 1 percent of the total study area. The value of most of these minor lakes and ponds is largely aesthetic. Because of the lack of lakes capable of supporting reasonable recreational use with little degradation of the resource in the heavily populated Milwaukee urban area, recreation pressures will be more heavily exerted on Lake Michigan and the streams and lakes in the adjacent tributary watershed lands outside the study area.

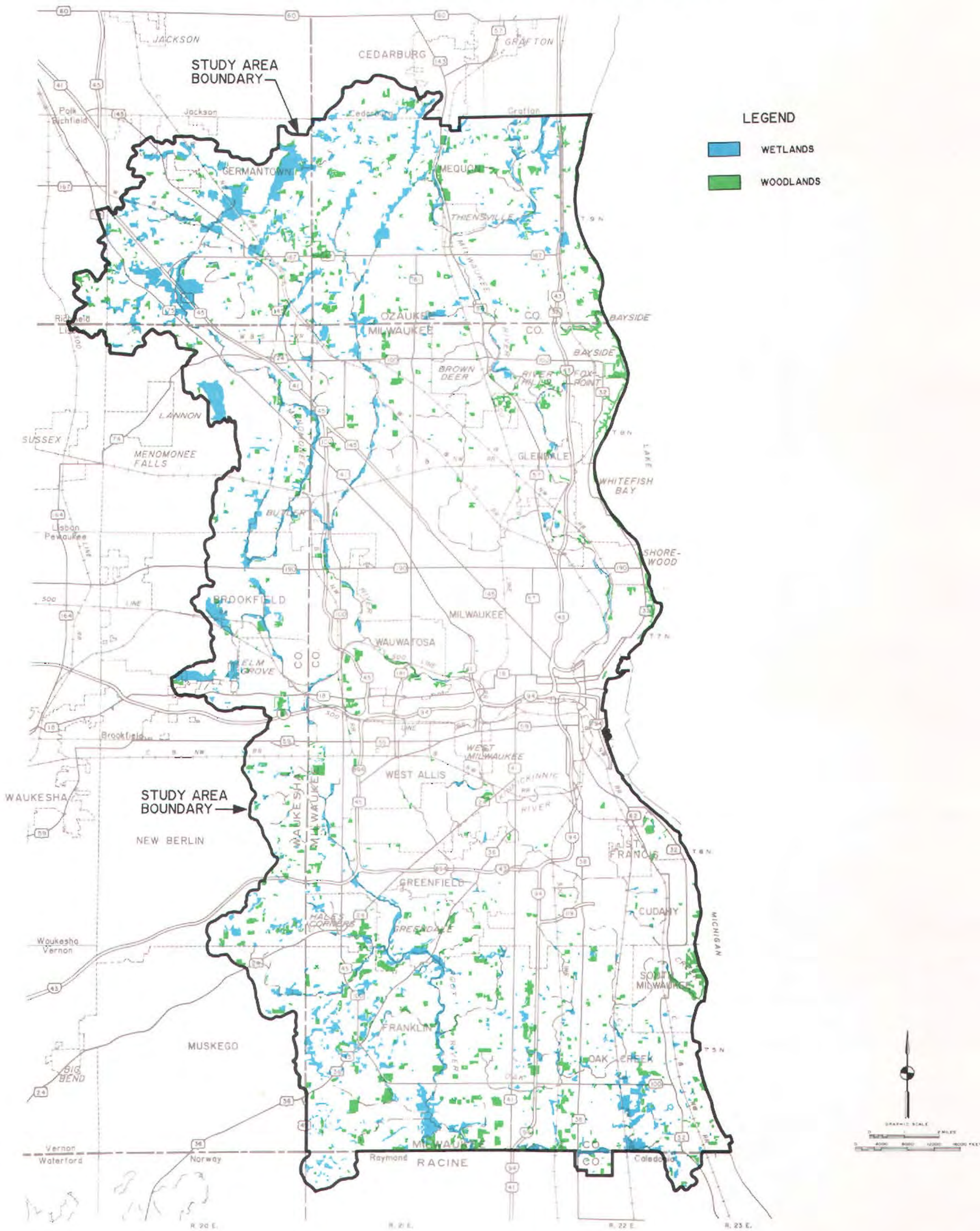
Streams: One of the most interesting, variable, and occasionally unpredictable features of the resource base is its river and stream system with its ever-changing and sometimes widely fluctuating discharges and stages. Within the study area, the stream system receives a relatively uniform flow of water from the underlying shallow groundwater reservoir. This groundwater discharge constitutes the base flow of the streams. The streams also periodically receive surface water runoff from rainfall and snowmelt which, when superimposed on the base flow, sometimes causes the streams to leave their channels and occupy the adjacent floodplains.

All perennial and certain selected intermittent streams within the study area are listed in Table 15 and shown on Map 21. These streams total about 203 linear miles.

Community Assistance Planning Report No. 130, A Stormwater Drainage and Flood Control Policy Plan for the Milwaukee Metropolitan Sewerage District, identifies district jurisdiction of streams

Map 20

WETLANDS AND WOODLANDS IN THE STUDY AREA



Source: SEWRPC.

Table 15

PERENNIAL AND SELECTED INTERMITTENT STREAMS IN THE STUDY AREA

Stream	Stream Length (miles)	Stream or Stream Reach Recommended for Milwaukee Metropolitan Sewerage District Jurisdiction		Stream or Stream Reach Not Recommended for Milwaukee Metropolitan Sewerage District Jurisdiction
		Existing District Boundary	Possible Future District Boundary	
Kinnickinnic River Watershed				
Edgerton Channel[a]	2.6[b]	2.6	2.6	- -
Holmes Avenue Creek[a]	1.6	- -	- -	1.6
Kinnickinnic River	8.1	5.7	5.7	2.4
Lyons Creek[a]	1.3	1.3	1.3	- -
Villa Mann Creek	1.7	1.7	1.7	- -
South 43rd Street[a]	1.1	1.1	1.1	- -
Wilson Park Creek[a]	3.5	3.5	3.5	- -
Lake Michigan Direct Drainage Area				
Fish Creek[a]	3.1	2.1	3.1	- -
Menomonee River Watershed				
Butler Ditch	3.7	- -	3.7	- -
Dousman Ditch	5.5	- -	5.5	- -
Honey Creek	8.8	8.8	8.8	- -
Little Menomonee Creek	2.5	- -	- -	2.5
Little Menomonee River	9.6	6.9	6.9	2.7
Menomonee River[a]	27.9	15.9	25.1	2.8
South Branch Underwood Creek	1.6[c]	1.6	1.6	- -
Underwood Creek	8.2	2.6	8.2	- -
West Branch Menomonee River	1.7	- -	- -	1.7
Woods Creek	1.1	1.1	1.1	- -
Unnamed Tributary Section 12, T9N, R20E	1.4	- -	- -	1.4
Unnamed Tributary Section 14, T7N, R20E	1.0	- -	- -	1.0
Milwaukee River Watershed				
Beaver Creek	1.9	1.9	1.9	- -
Brown Deer Creek[a]	1.9	1.9	1.9	- -
Indian Creek	1.9	1.9	1.9	- -
Lincoln Creek	8.5[d]	8.5	8.5	- -
Milwaukee River[a]	26.2	- -	- -	26.2
Pigeon Creek	2.4	- -	0.8	1.6
South Branch Creek	1.5[b]	1.5	1.5	- -
Unnamed Tributary Section 2, T7N, R21E	0.5	- -	- -	0.5
Unnamed Tributary Section 7, T9N, R22E	1.7	- -	- -	1.7
Unnamed Tribulary Section 18, T9N, R22E	1.4	- -	- -	1.4
Unnamed Tributary Section 35, T9N, R21E	2.0	- -	2.0	- -
Unnamed Tributary Section 36, T9N, R21E	0.2	- -	- -	0.2
Oak Creek Watershed				
Mitchell Field Drainage Ditch[a]	3.3[e]	3.3	3.3	- -
North Branch Oak Creek[a]	5.8	5.8	5.8	- -
Oak Creek	13.1	8.4	13.1	- -
Root River Watershed				
Crayfish Creek	1.0	- -[f]	1.0[f]	- -
East Branch Root River[a]	4.7	4.7	4.7	- -
Root River	21.6[e]	14.2	16.5	5.1
Root River Canal	1.3	- -	- -	1.3
Tess Corners Creek[a]	2.6	2.6	2.6	- -
Whitnall Park Creek[a]	3.0[g]	3.0	3.0	- -
Unnamed Tributary Section 20, T6N, R21E	0.4	0.4	0.4	- -
Total	202.9	113.0	148.8	54.1

[a]*It should be noted that for these 14 streams, the stream reach lengths have been revised somewhat when compared with the stream lengths set forth in SEWRPC Community Assistance Planning Report No. 130, A Stormwater Drainage and Flood Control Policy Plan for the Milwaukee Metropolitan Sewerage District, March 1986. The revisions, which range from 0.1 mile on five streams to 2.6 miles on the Milwaukee River, are the result of: 1) refinements in the study area boundaries--as in the case of the Milwaukee River stream length revision where the study area now being used extends to the northerly limits of the City of Mequon rather than to about two miles south of the city limits as was the case in the 1986 policy plan; 2) the development of more precise locations of the stream reach limits as documented in SEWRPC Memorandum Report No. 28, Streams and Watercourses for Which the Milwaukee Metropolitan Sewerage District Has Assumed Jurisdiction for Drainage and Flood Control Purposes, August 1987; 3) more precise delineation and measurements of the stream lengths using large-scale topographic mapping and channel profiles; and 4) physical alterations to the stream channels.*

[b]*Intermittent stream.*

[c]*Includes 0.5 mile of intermittent reach.*

[d]*Includes 0.4 mile of intermittent reach.*

[e]*Includes 0.9 mile of intermittent reach.*

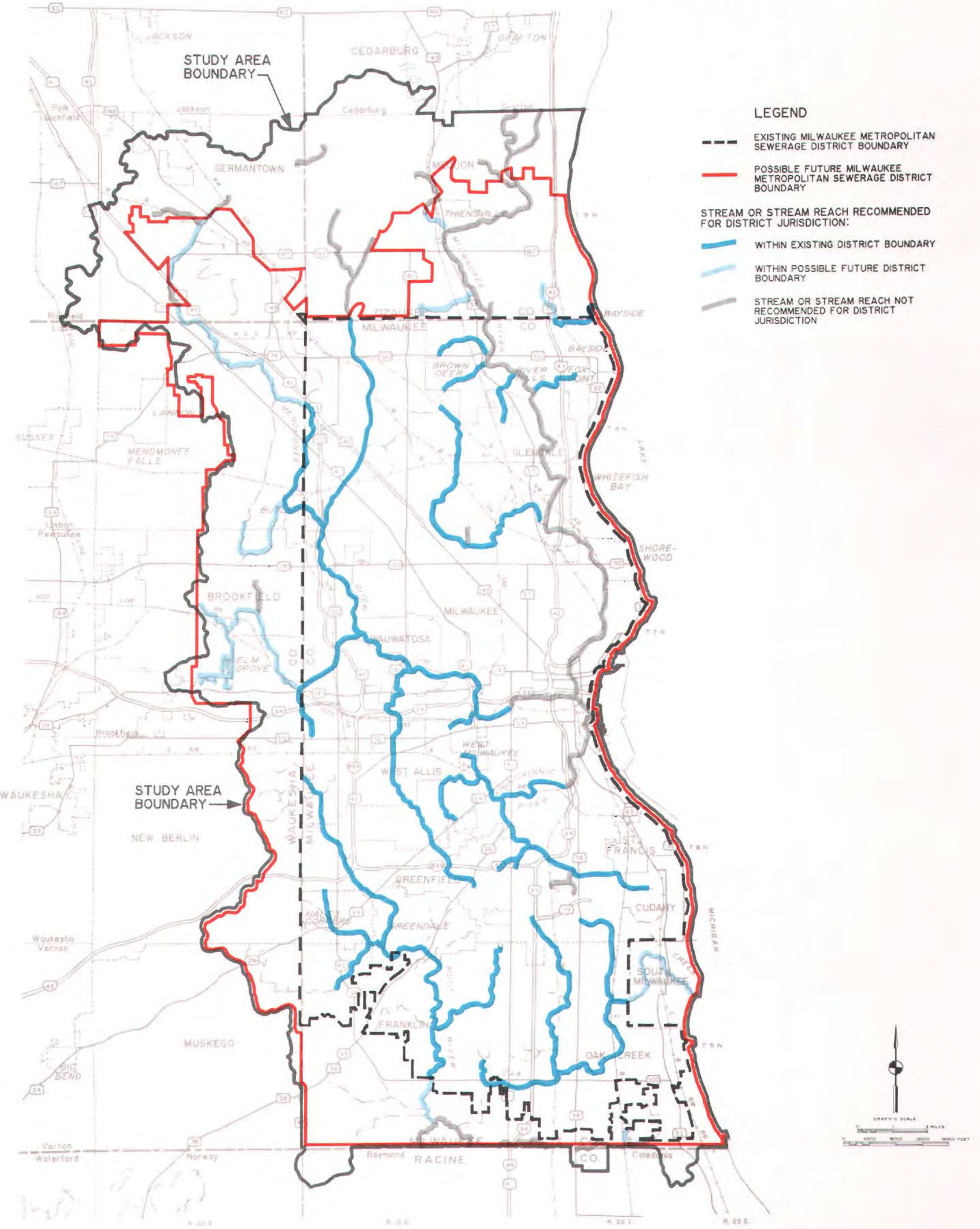
[f]*During 1988, the Milwaukee Metropolitan Sewerage District added the Crayfish Creek area to the District.*

[g]*Includes 1.8 miles of intermittent reach.*

Source: SEWRPC.

Map 21

PERENNIAL AND SELECTED INTERMITTENT STREAMS IN THE STUDY AREA: 1987



Source: SEWRPC.

and watercourses within the study area for drainage and flood control purposes. More specifically, the Advisory Committee overseeing the work on that policy plan recommended that—after the application of certain overriding considerations[1] —the Milwaukee Metropolitan Sewerage District jurisdiction for perennial streams for the resolution of drainage and flood control problems include all perennial streams which meet at least one of the following three criteria:

1. Streams within the District for which the District has completed channel improvements.

2. Streams within the District with significant monetary flood damage risk.

3. Streams within the District having a tributary drainage area in more than one community.

In addition, the Advisory Committee recommended that the Milwaukee Metropolitan Sewerage District jurisdiction for the resolution of drainage and flood control problems include intermittent streams which meet any two of the above three criteria.

The application of the overriding considerations and the criteria to streams within the study area resulted in a recommendation that 113.0 linear miles of streams within the existing district boundaries, or 56 percent of the streams in the study area, be recommended for District jurisdiction. In addition, it was recommended that 148.8 linear miles of streams within the possible future District boundary, or 73.0 percent of the linear miles of streams in the study area, be under District jurisdiction in the future.

[1] *The overriding considerations set forth by the Committee were: 1) the estuary reaches of the Kinnickinnic, Menomonee, and Milwaukee Rivers should be excluded from District jurisdiction since these reaches are more properly the responsibility of state and federal levels of government; and 2) major stream reaches having 50 percent or more of their tributary drainage area outside the study area should be excluded from District jurisdiction. Through the application of these overriding considerations, 2.4 miles of the Kinnickinnic River estuary, 2.2 miles of the Menomonee River estuary, and 3.2 miles of the Milwaukee River estuary were recommended to be excluded from District jurisdiction. Similarly, 23.0 miles of the main stem of the Milwaukee River, the remainder of the Milwaukee River in the policy plan study area outside the estuary, and 4.8 miles of the Root River were recommended to be excluded from District jurisdiction.*

As indicated in Table 16 and shown on Map 22, the tributary drainage area of the streams recommended for District jurisdiction within existing District boundaries encompasses 261 square miles, or about 69 percent of the total study area. The tributary drainage areas of the streams recommended for District jurisdiction within possible future district boundaries total over 298 square miles, or almost 79 percent of the total study area.

Floodlands: The natural floodplain of a river is a wide, flat, gently sloping area contiguous to and usually lying on both sides of a channel. The floodplain, which is normally bounded on its outer edges by higher topography, is gradually formed over a long period of time by the river during floodstage and as that river meanders in the floodplain, continuously eroding material from concave banks and depositing it on convex banks. A river or stream may be expected to occupy and flow on its floodplain on the average of once every two years; therefore, the floodplain should be considered to be an integral part of a natural stream system. The extent to which a natural floodplain would be occupied by any given flood will depend on the severity of the flood, and, more particularly, upon its elevation or stage. Thus, an infinite number of outer limits of the natural floodplain may be delineated, each related to a specified flood recurrence interval. The Regional Planning Commission recommends that the natural floodplains of a river or stream be more specifically defined as those corresponding to a flood having a recurrence interval of 100 years, with the natural floodlands being defined as consisting of the river channel plus the 100-year floodplain.

Floodlands within the study area are shown on Map 23. These floodlands occupy a total of 28.2 square miles, or about 7.5 percent, of the study area. The delineation of natural floodlands is extremely important for sound land use planning and development. Because of flood hazards, high water tables, and inadequate soils, floodland areas are generally not well suited to urban development. Furthermore, floodlands are not needed for incremental urban development in that there are sufficient suitable land areas outside the floodlands for development purposes. Floodland areas, however, are generally prime locations for much-needed park and open space areas, and contain many of the best remaining wetlands and wildlife habitat areas of the Region. Floodlands also have important floodwater conveyance and storage functions.

Table 16

TRIBUTATY DRAINAGE AREA OF STREAMS IN THE STUDY AREA WHICH ARE RECOMMENDED FOR MILWAUKEE METROPOLITAN SEWERAGE DISTRICT JURISDICTION

Watershed	Watershed Area Within Study Area (square miles)	Tributary Drainage Area of Streams Recommended for District Jurisdiction (square miles)		Percent of Watershed in Study Area Having Tributary Drainage Area Attendant to Streams Recommended for District Jurisdiction		Percent of Study Area Having Tributary Drainage Area Attendant to Streams Recommended for District Jursidiction	
		Existing District Boundary	Possible Future District Boundary	Existing District Boundary	Possible Future District Boundary	Existing District Boundary	Possible Future District Boundary
Fox	0.5	0.0	0.0	- -	- -	- -	- -
Kinnickinnic.	24.9	20.5	20.5	82.3	82.3	5.4	5.4
Menomonee	135.9	132.1	132.1	97.2	97.2	34.9	34.9
Milwaukee	92.6	31.3	47.0	33.8	48.9	8.3	12.4
Oak Creek	27.2	22.7	27.2	83.5	100.0	6.0	7.2
Root	74.8	49.3	66.4	65.9	88.8	13.0	17.5
Lake Michigan Direct Drainage . . .	22.8	5.3	5.3	23.2	23.2	1.4	1.4
Total	378.7	261.2	298.5	- -	- -	69.0	78.8

Source: SEWRPC.

Environmental Corridors

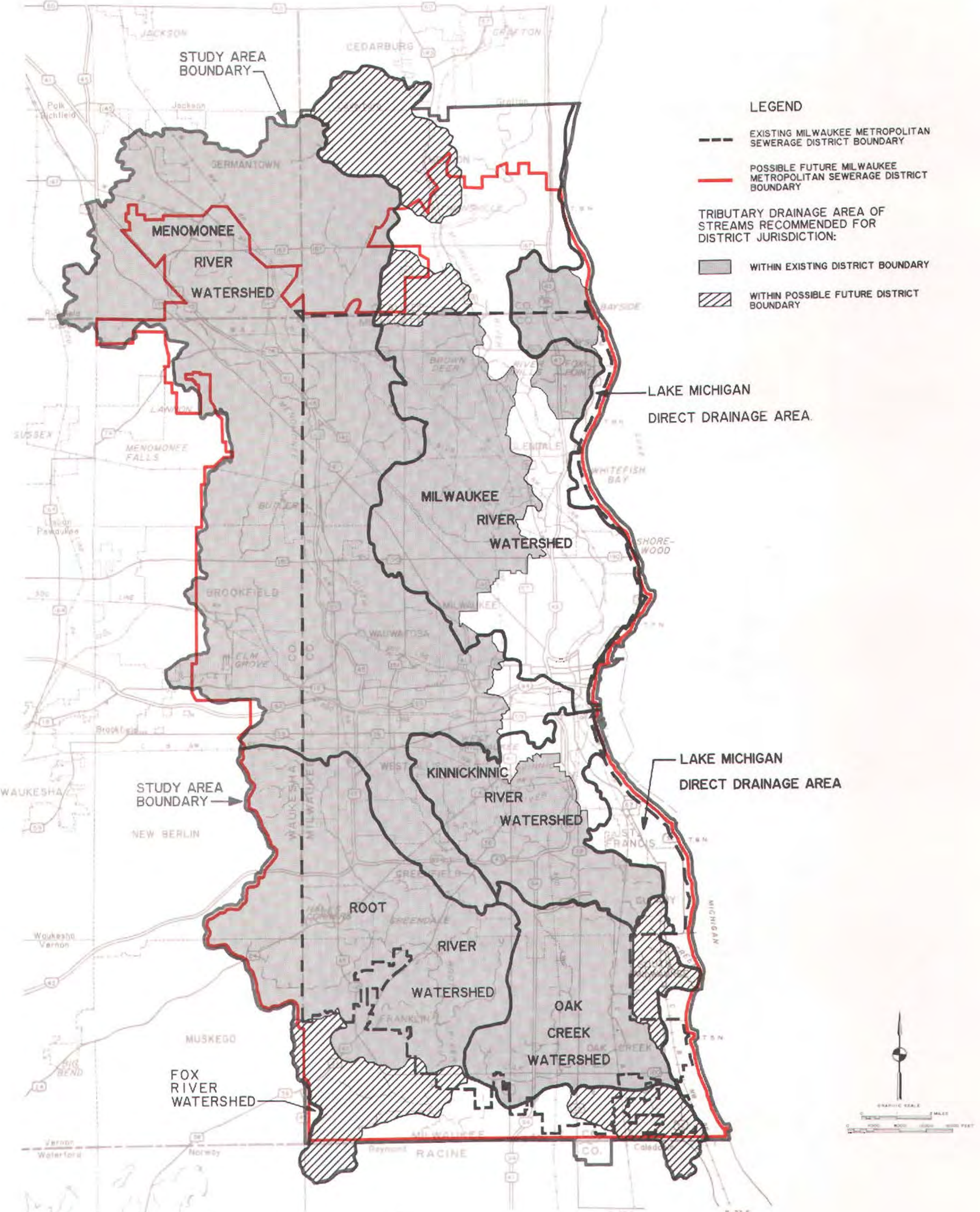
Environmental Corridor Concept: One of the most important tasks undertaken by the Regional Planning Commission as part of its planning efforts was the identification and delineation of those areas of the Region having high concentrations of natural, recreational, historic, aesthetic, and scenic resources, and which therefore should be preserved and protected in order to maintain the overall quality of the environment. Such areas normally include one or more of the following seven elements of the natural resource base which are essential to the maintenance of both the ecological balance and the natural beauty of the Region: 1) lakes, rivers, and streams and their associated undeveloped shorelands and floodplains; 2) wetlands; 3) woodlands; 4) prairies; 5) wildlife habitat areas; 6) wet, poorly drained, and organic soils; and 7) rugged terrain and high-relief topography. While the foregoing seven elements constitute integral parts of the natural resource base, there are five additional elements which, although not a part of the natural resource base per se, are closely related to or centered on that base, and therefore are important considerations in identifying and delineating areas with scenic, recreational, and educational value. These additional elements are: 1) existing outdoor recreation sites; 2) potential outdoor recreation and related open space sites; 3) historic, archaeological, and other cultural sites; 4) significant scenic areas and vistas; and 5) natural scientific areas.

The delineation of these 12 natural resource and resource-related elements on a map results in an essentially linear pattern of relatively elongated areas which have been termed "environmental corridors" by the Commission. Primary environmental corridors include a wide variety of the important resource and resource-related elements and are at least 400 acres in size, two miles in length, and 200 feet in width. Secondary environmental corridors typically connect with primary environmental corridors and are at least 100 acres in size and one mile in length.

It is important to point out that, because of the many interlocking and interacting relationships between living organisms and their environment, the destruction or deterioration of one element of the total environment may lead to a chain reaction of destruction and deterioration. The drainage of wetlands, for example, may have far-reaching effects since such drainage may destroy fish

Map 22

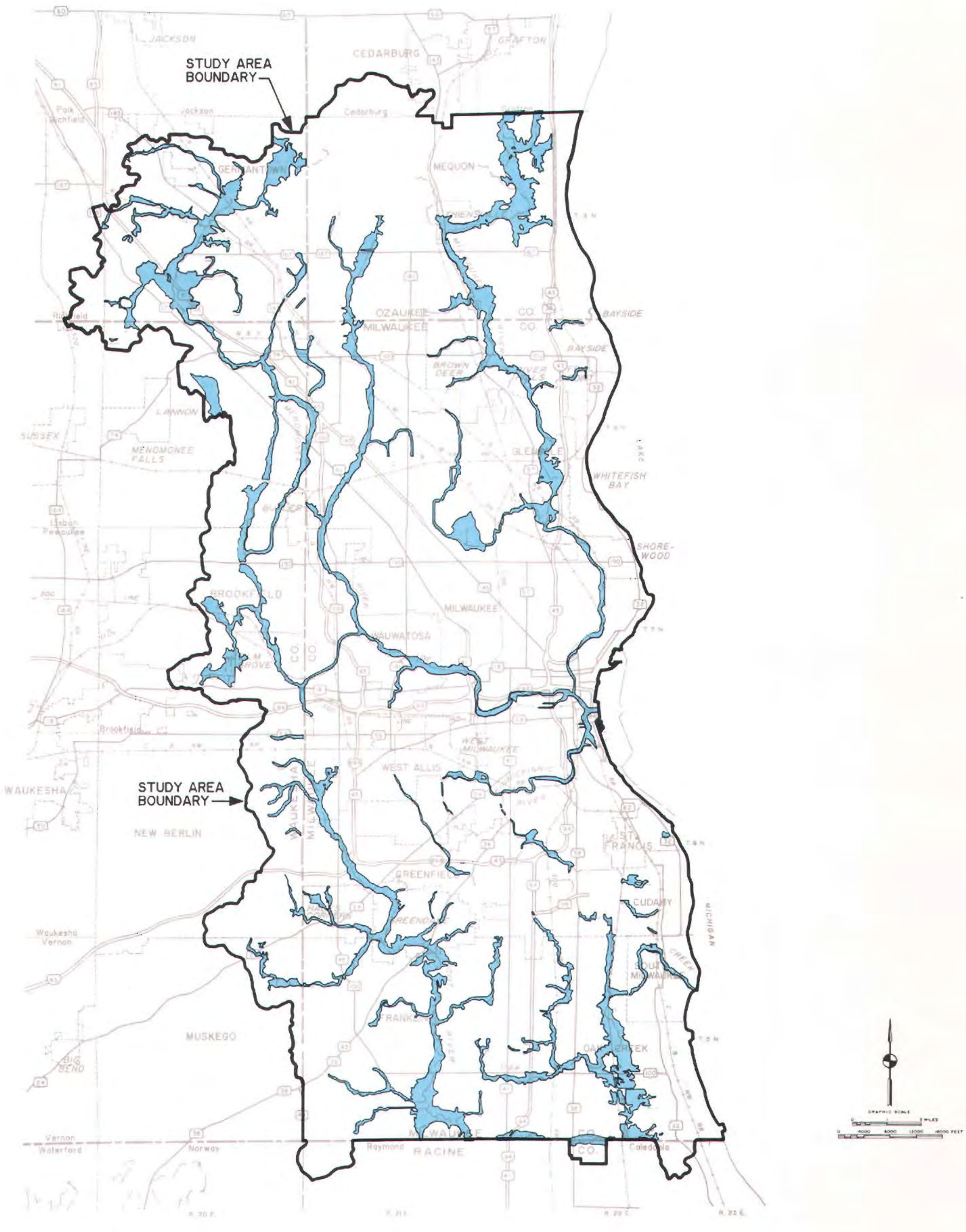
TRIBUTARY DRAINAGE AREA OF STREAMS IN THE STUDY AREA WHICH ARE RECOMMENDED FOR MILWAUKEE METROPOLITAN SEWERAGE DISTRICT JURISDICTION



Source: SEWRPC.

Map 23

FLOODLANDS IN THE STUDY AREA



Source: SEWRPC.

Table 17

ENVIRONMENTAL CORRIDORS AND ISOLATED NATURAL AREAS IN THE STUDY AREA: 1985

Natural Features	Primary Environmental Corridors		Secondary Environmental Corridors		Isolated Natural Areas		Total	
	Square Miles	Percent	Square Miles	Percent	Square Miles	Percent	Square Miles	Percent
Surface Water	2.3	8.5	0.3	2.8	0.3	4.1	2.9	6.4
Wetlands	11.5	42.4	4.2	38.9	2.0	27.4	17.7	39.2
Woodlands	5.3	19.6	3.1	28.7	4.2	57.5	12.6	27.9
Other	8.0	29.5	3.2	29.6	0.8	11.0	12.0	26.5
Total	27.1	100.0	10.8	100.0	7.3	100.0	45.2	100.0

Source: SEWRPC.

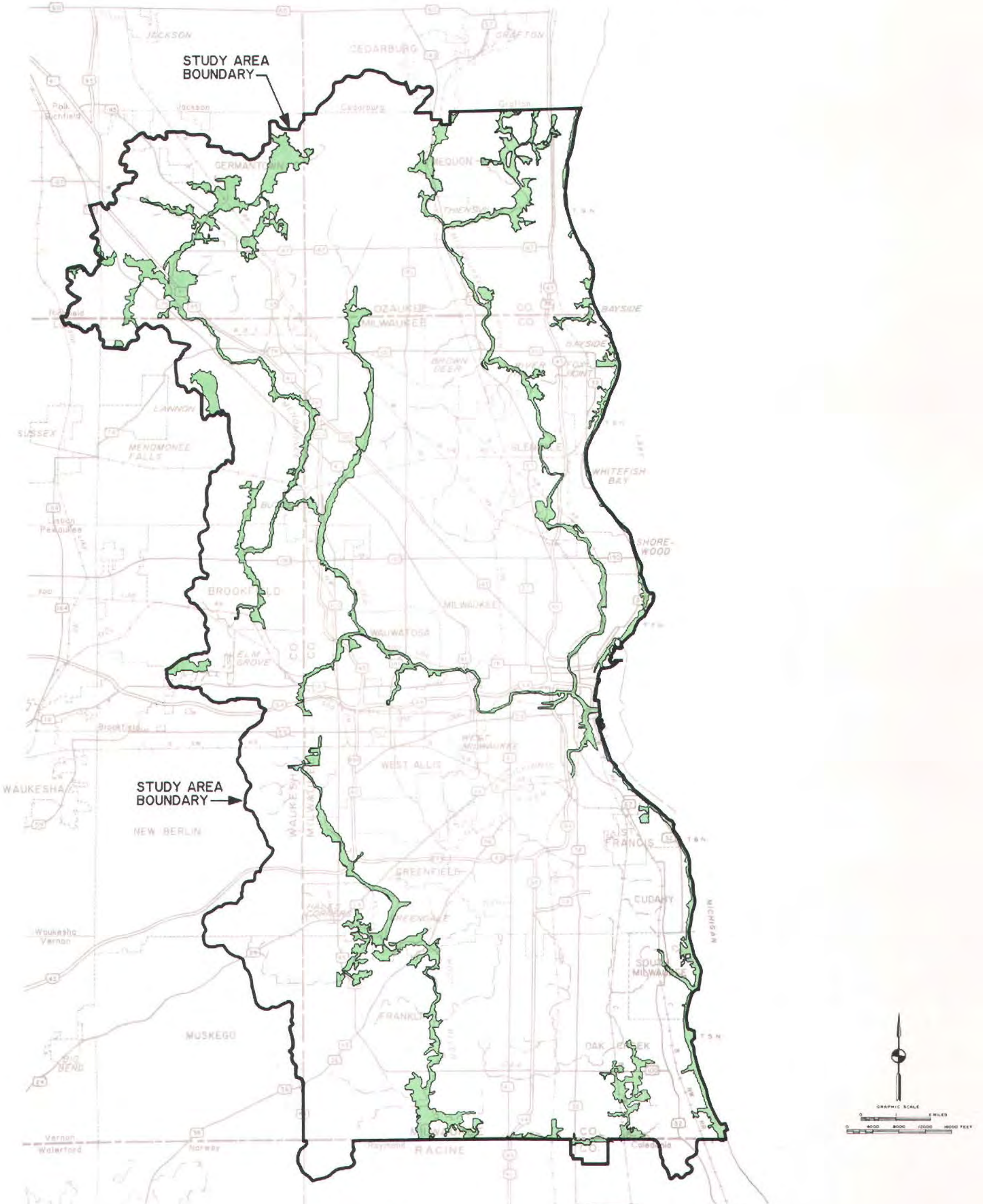
spawning grounds, wildlife habitat, groundwater recharge areas, and the natural filtration of floodwater storage areas of interacting lake and stream systems. The resulting deterioration of surface water quality may, in turn, lead to a deterioration of the quality of groundwater. Groundwater serves as a source of domestic, municipal, and industrial water supplies and provides a basis for low flows in rivers and streams. Similarly, destruction of woodland cover, which may have taken a century or more to develop, may result in soil erosion and stream siltation and in more rapid runoff and increased flooding, as well as the destruction of wildlife habitat. Although the effects of any one of these environmental changes may not in itself be overwhelming, the combined effects may lead eventually to the deterioration of the underlying and supporting natural resource base and to the overall quality of the environment for life. The need to protect and preserve the remaining environmental corridors within the study area should thus be apparent.

Primary environmental corridors in the study area generally lie along the major stream valleys and contain almost all of the remaining high-value woodlands, wetlands, and wildlife habitat and surface waters and undeveloped floodlands in the study area. These corridors also contain some of the best remaining potential park sites, and are, in fact, a composite of the best individual elements of the natural resource base. As indicated in Table 17 and shown on Map 24, primary environmental corridors encompassed about 27.1 square miles, or about 7 percent, of the total study area in 1985—including 2.3 square miles of surface water, 11.5 square miles of wetlands, 5.3 square miles of woodlands, and 8.0 square miles of other lands, including wildlife habitat, prairies, and scenic areas and vistas.

Secondary environmental corridors within the study area are generally located along intermittent streams or serve as links between segments of primary environmental corridor. Secondary corridors contain a variety of resource elements, often remnant resources from primary corridors which have been developed for intensive agricultural purposes or urban land uses. Secondary environmental corridors facilitate surface water drainage, maintain "pockets" of natural resource features, and provide for the movement of wildlife, as well as for the movement and dispersal of seeds for a variety of plant species. Such corridors, while not as important as the primary environmental corridors, should be preserved in essentially open, natural uses as urban development proceeds within the study area, particularly when opportunities are presented to incorporate the corridors into urban stormwater detention areas, associated drainageways, and neighborhood parks. As indicated in Table 17, secondary environmental corridors encompassed 10.8 square miles, or about 2.9 percent, of the total study area in 1985. Such corridors included about 0.3 square mile of surface water, 4.2 square miles of wetlands, 3.1 square miles of woodlands, and 3.2 square miles of other lands.

Map 24

PRIMARY ENVIRONMENTAL CORRIDORS IN THE STUDY AREA: 1985



Source: SEWRPC.

In addition to primary and secondary environmental corridors, other, small concentrations of natural resource base elements exist within the study area. These resource base elements are isolated from environmental corridors by urban development or agricultural uses but, although separated from the environmental corridor network, also have important natural values. Isolated natural areas may provide the only available wildlife habitat in an area, provide good locations for local parks and nature study areas, and lend aesthetic character or natural diversity to an area. Important isolated natural features include a geographically well-distributed variety of isolated wetlands, woodlands, and wildlife habitat. These isolated natural features should also be protected and preserved in a natural state whenever possible. As indicated in Table 17, isolated natural areas encompassed approximately 7.3 square miles, or 1.9 percent, of the total study area in 1985. Such areas include approximately 0.3 square mile of surface water, 2.0 square miles of wetlands, 4.2 square miles of woodlands, and 0.8 square mile of other lands.

In total, the primary and secondary environmental corridors and isolated natural areas encompassed almost 45 square miles, or 12 percent, of the study area in 1985, and included about 2.9 square miles of surface water, 17.7 square miles of wetlands, 12.6 square miles of woodlands, and 12.0 square miles of other lands.

ANTICIPATED GROWTH AND CHANGE

In any planning effort, forecasts are required of all future conditions which are considered beyond the scope of the plans to be prepared, but which may affect either the design of the plans or the implementation of the plans over time. Future demands on the resources within the study area are determined primarily by the size, spatial distribution, and characteristics of the future population and economic activities within the study area, and by the resultant land use pattern associated with changes in population and economic activity. Land use patterns, in turn, markedly influence stormwater runoff. Conversion of land from rural to urban use and the associated increase in impervious area will tend to increase both the rate and volume of stormwater for a given rainfall event and decrease the time of runoff. Unless special stormwater management measures are taken, the typical effect of urbanization is to produce an increase both in the peak rates of stormwater runoff and in the total volume of runoff. Stormwater runoff from urban lands also carries pollutants that are different from those carried by runoff from rural lands, as well as greater amounts of pollutants. Finally, changes in land use over time affect the stormwater runoff process, and therefore the loadings on the stormwater management system. Therefore, consideration of both the probable future and existing land use pattern in the study area is necessary for effective development of stormwater drainage and flood control plans.

Although the spatial distribution of future population and economic activity can be influenced by public land use regulation, and although upper limits can be set on population and economic activity levels through such regulation, the control of population and economic activity levels lies largely beyond the scope of government activity, at least at the regional and local levels. Neither the levels of population and employment within the study area, nor the rates of change in these levels, can be prescribed in this plan; rather, such levels and changes will be a function of the attractiveness of the study area and of the Southeastern Wisconsin Region relative to other areas of the Region and other regions of the United States. In the preparation of the stormwater drainage and flood control plan, therefore, future population and economic activity levels had to be forecast. These forecasts could then be converted to demands for land within the study area, and stormwater drainage and flood control plans could be prepared to meet such demands.

Basis of Population and Economic Activity Forecasts

It is important to note that the population and employment forecasts presented in this section were not independently prepared for the study area, but were based upon forecasts prepared for and used in the preparation of other regional plan elements, including areawide land use, transportation, sanitary sewerage system, and watershed plans. The use of these forecasts helped to assure consistency between the study area plan and other long-range areawide plan elements.

The population, employment, and land use demand forecasts selected as the basis for the preparation of the stormwater drainage, flood control planning effort were based upon a regional forecast developed using an "alternative futures" approach. Under this approach, alternative futures were postulated for the Region considering potential changes in the key external factors affecting the development of the Region, including the cost and availability of energy, individual and family lifestyles, and the ability of the Region to compete with other regions of the United States for development. The range of population and economic activity levels attendant to these alternative futures was believed to represent reasonable extremes of future development conditions within the Region. Alternative land use patterns were then developed for each of these extremes in order to provide a range of spatial distributions, population, and economic activity levels within the Region.

Two of the resulting four alternative futures, the "optimistic growth centralized development" future and the "optimistic growth decentralized development" future, envision moderate growth in resident population and economic activity levels within the Region. One of these futures envisions that this growth will be accommodated in a centralized manner, with new urban development occurring largely at medium densities and contiguous to, and outward from, existing urban centers in the Region. The other envisions that much of this growth will be accommodated in a decentralized manner, with new urban development occurring at low densities in a defused pattern well beyond the limits of existing centers of the Region. The other two futures envision only slight growth in economic activity and an actual decline in resident population levels. One of these two futures, the "pessimistic growth centralized development" future, envisions that any redistribution of population and employment will be accommodated in a centralized manner. The other of these futures, the "pessimistic growth decentralized development" future, envisions that any redistribution of population and employment will be accommodated in a decentralized manner.

It was determined by the Technical Advisory Committee overseeing the study that the population and employment levels envisioned under the optimistic growth centralized development alternative would be used in the planning process. The use of this alternative future represents a conservative approach to the stormwater drainage and flood control planning process. This alternative, as already noted, envisions a moderate increase in population and economic activity levels within the study area, and therefore represents a reasonable extreme of land use development which could occur within the study area within the next two decades. In addition, this future would have the greatest effect on stormwater drainage and flooding conditions within the study area. Moreover, the spatial distribution, population, and economic activity under this future is based upon adopted regional and local land use development objectives, and is consistent with federal and state policies which seek to promote more centralized urban development patterns and protect environmentally significant areas within the study area.

Population Growth

As indicated in Table 18 and Figure 7, the regional population forecast selected as a basis for the study area planning effort anticipates that the resident population of the Region will reach 2.2 million persons by the year 2000. This would represent an increase of about 476,000 persons, or 27.3 percent, over the 1985 level of 1.74 million persons. This anticipated population increase—equivalent to about 31,700 persons per year from 1985 to 2000 —is more than twice the actual rate of increase of 13,400 persons per year experienced from 1930 to 1985, but still less than the actual rate of increase of 33,300 persons per year experienced from 1950 to 1960. Under the pessimistic growth scenario, the population of the Region in the year 2000 could be as low as 1,690,00. This level represents a decrease of 3,500 persons per year from 1985 to 2000. The study area forecast, based upon normative areawide land use development objectives, envisions a reversal of recent trends. Under the forecast, the population of the study area may be expected to increase from the 1985 level of 1,042,600 to 1,249,000 in the plan design year 2000. This represents an increase of about 206,500 persons, or about 19.8 percent, over the 1985 population level. This anticipated population increase—equivalent to about 13,700 persons per year from 1985 to 2000—is more than twice the actual rate of increase of 5,600 persons per year experienced from 1930 to 1985, but significantly less than the actual rate of increase of 20,800 persons per year experienced from 1950 to 1960. Under the pessimistic growth decentralized development scenario, the population of the study area in the plan design year 2000 could be as low as 851,900 persons.

Employment Growth

Employment activity as measured in terms of employment opportunities does not link functionally to the study area pattern within southeastern Wisconsin. Rather, the forces determining economic activity originate and are sustained over the entire urbanized region. Under the alternative

Table 18

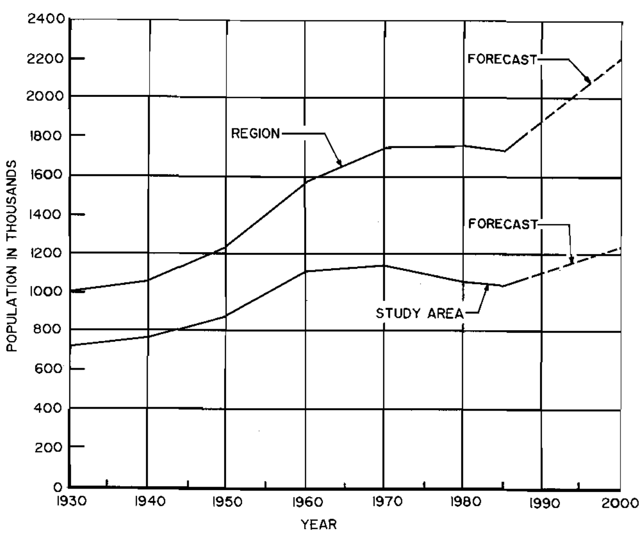
POPULATION WITHIN THE REGION AND STUDY AREA: SELECTED YEARS 1930-2000

Area	Population											
	1930	1940	1950	1960	1970	1980	1985	Change 1930-1985		Year 2000	Change 1985-2000	
								Absolute	Percent		Absolute	Percent
Region	1,006,100	1,067,700	1,240,600	1,573,600	1,756,100	1,764,900	1,742,700	736,600	73.2	2,219,300	476,000	27.3
Study Area	733,600	777,500	893,100	1,101,700	1,151,600	1,066,400	1,042,600	309,000	42.1	1,249,100	206,500	19.8

Source: U. S. Bureau of the Census and SEWRPC.

Figure 7

POPULATION TRENDS AND FORECASTS FOR THE REGIONAL STUDY AREA: 1930-2000



Source: U. S. Bureau of the Census and SEWRPC.

future selected as the basis for this planning program, employment levels within the Region are envisioned to increase substantially between 1985 and the plan design year. As indicated in Table 19, the regional employment level, which stood at 871,900 in 1985, is envisioned to increase by 144,100 employees, or 16.5 percent, to a level of 1,016,000 by the year 2000. Under the pessimistic growth scenario, however, employment in the Region in the year 2000 could be as low as 886,900. Within the study area, based upon the alternative future selected as a basis for study area planning, the employment level of about 592,200 in 1985 was envisioned to increase by 60,100, or 10.1 percent, to 652,300 in the year 2000. Under the pessimistic growth decentralized development scenario, however, employment in the study area in the plan design year 2000 could be as low as 566,000.

Land Use Demand

Because of the population and employment increases envisioned within the study area by the year 2000, the continued conversion of rural land to urban use may be expected to be required within the study area. Between 1963 and 1985, 45.0 square miles of land were converted from rural to urban use within the study area, increasing the proportion of the total area of the study area in urban use from about 47 percent in 1963, or about 176 square miles, to 58.5 percent in 1985, or about 221.4 square miles. The alternative future selected as a basis for study area planning envisions the conversion of an additional 20 square miles of land from rural to urban use between 1985 and the year 2000, an increase of about 9 percent. By the plan design year, then, approximately 241 square miles, or about 64 percent, of the 378-square-mile study area would be in urban use.

SUMMARY

The planning and design of urban stormwater drainage and flood control systems requires knowledge about and consideration of certain man-made and natural features of the area to be served. This chapter describes those man-made and natural features of the planning area that affect and are affected by stormwater drainage and flood control facilities.

Table 19

EMPLOYMENT WITHIN THE REGION AND STUDY AREA: EXISTING 1972-1985 AND PROJECTED YEAR 2000

Area	Employment						
	1972	1985	Change 1972-1985		Year 2000	Change 1985-2000	
			Absolute	Percent		Absolute	Percent
Region	748,800	871,900	123,100	16.4	1,016,000	144,100	16.5
Study Area	538,700	592,200	53,500	9.9	652,300	60,100	10.1

Source: Wisconsin Department of Industry, Labor and Human Relations; and SEWRPC.

The geographic area delineated for drainage and flood control planning in the greater Milwaukee area encompasses about 379 square miles and occupies all of Milwaukee County, as well as portions of Ozaukee, Racine, Washington, and Waukesha Counties. This area includes all or portions of 15 cities, 14 villages, and 8 towns.

The resident population of the study area in 1985 was estimated at 1,042,600 persons, or about 60 percent of the estimated 1,742,700 persons then residing within the seven counties in the Southeastern Wisconsin Region. The study area exhibited significant increases in population from 1950 to 1970, and then marked decreases from 1970 to 1985. The relative gain in population in the study area over the 1950 to 1960 time period—about 23 percent—was significantly higher than that in the State of Wisconsin or the United States, but somewhat lower than that in the Region, during the same time period. While the study area and the Region experienced population declines from 1970 to 1985, the State of Wisconsin and the United States continued to experience population increases, although at a lower rate than during previous decades.

The median household income in the study area showed a significant gain between 1959 and 1969, but almost half of the civil divisions in the study area experienced losses in household income as expressed in constant 1979 dollars between 1969 and 1979. Average household size in the study area continued to decline in all civil divisions between 1960 and 1980. As of 1980, only one civil division in the study area had an average household size of greater than four persons, while 15 civil divisions exhibited average household sizes of fewer than three persons. The median age of the resident population in the study area continued to increase. In 1960, 27 civil divisions within the study area had resident populations with a median age of less than 30 years. In 1980, only six civil divisions within the study area had a population exhibiting a median age of less than 30 years, and there were three communities which, for the first time, had median ages of greater than 40 years.

Total jobs in the study area increased by over 53,100, or 10 percent—from 538,700 in 1972 to about 591,800 in 1980. The industrial group included the largest number of jobs in the study area in 1972—over 208,900 jobs, or almost 39 percent. Service jobs, however, exhibited the largest absolute percentage increase from 1972 to 1980, about 31,500 jobs, or 21 percent—from 148,800 jobs in 1972 to about 180,300 jobs in 1980. In 1985, the service group provided the largest number of jobs, over 199,100, or about 34 percent of the jobs in the study area. The service group also exhibited the largest absolute and percentage increases during the 1980 to 1985 time period, increasing by about 18,800 jobs, or about 10 percent, over 1980 levels. The industrial group exhibited the largest absolute decrease from 1980 to 1985, decreasing by about 27,000 jobs, or about 13 percent—from about 210,400 in 1980 to about 182,700 jobs in 1985.

Urban land uses in the study area encompassed over 176 square miles, or about 47 percent of the study area, in 1963. Urban lands in the study area increased to over 198 square miles, or 52 percent of the study area, by 1970, and to 216 square miles, or 57 percent of the study area, by 1980. By 1985, urban land uses encompassed over 221 square miles, or more than 58 percent of the study area. The largest increases in urban land uses during this time period occurred in residential land use, which increased over 19 square miles; and in transportation, communication, and utility uses, which increased more than 15 square miles.

In 1985, sanitary sewage generated in the study area was conveyed to and treated at six public sewage treatment plants. About 221 square miles, or 58 percent of the total study area, and approximately 1,019,900 persons, or 98 percent of the total study area population, were served by public sanitary sewerage facilities in 1985.

In 1985, there were a total of 21 publicly owned water utilities, as well as 105 nonmunicipal cooperatively owned residential water supply systems in the study area. About 188 square miles, or about 50 percent of the total study area, and approximately 962,500 persons, or 92 percent of the study area population, were served with public water supply facilities in 1985.

The study area is well served with highway, bus, railway, and air transportation facilities. In 1985, the study area included an extensive street and highway system, including 85 linear miles of freeway and about 905 linear miles of surface arterials. In addition, there were about 2,633 linear miles of collector and land access streets in the study area. Two types of bus service were provided in 1985—urban mass transit and inner city bus service. Urban mass transit service within the study area was provided by the Milwaukee County Transit System and Waukesha County, which together provided service to 152 square miles, or 40 percent of the study area, and 871,600 persons, or 84 percent of the resident population of the study area. Railway service in 1985 was limited to freight hauling except for scheduled Amtrak passenger service over the lines of the Soo Line Railroad between the Amtrak passenger station in the City of Milwaukee, the only stop in the study area, and Chicago to the south and Minneapolis-St. Paul to the northwest. There are three public-use airports located in the study area, including Milwaukee County's General Mitchell International Airport (General Mitchell Field), which is served by 15 air carriers providing passenger service on a regularly scheduled basis, as well as air cargo service. In addition to Mitchell Field, the study area is served by Timmerman Field located on the northwest side of the City of Milwaukee, and Rainbow Airport, which is limited to general aviation activity, located in the City of Franklin.

Air temperatures and the type, intensity, and duration of precipitation events are major determinants of the rate and volume of stormwater runoff. The temporal weather changes consist of marked variations in temperature, precipitation, relative humidity, wind speed and direction, and cloud cover. These meteorological events influence the rate and amount of stormwater runoff, the severity of stormwater drainage and flooding problems, and the required capacity of stormwater conveyance and storage facilities.

Mean summer temperatures in July and August in the study area are in the 70°F range. Average daily maximum temperatures for these two months range from 78°F to 82°F, whereas average daily minimum temperatures vary from 57°F to 61°F. Temperatures in the study area measured by monthly means for January and February range from 17°F to 23°F. Average daily maximum temperatures for these two months vary from about 26°F to 31°F, whereas average daily minimum temperatures range from about 8°F to 15°F.

Average annual total precipitation in the study area is about 29.8 inches expressed as water equivalent, while the average annual snowfall is 49.7 inches. More than 19 inches, or 64 percent, of the average annual precipitation normally occurs during the April to October growing season, primarily as rainfall. Assuming that 10 inches of measured snowfall is equivalent to one inch of water, the average annual snowfall of about 49.7 inches is equal to about 4.97 inches of rain, and therefore only about 16 percent of the average annual precipitation occurs as snowfall. Ground frost or frozen ground is likely to exist in the study area for approximately four months each winter season, extending from late November to March, with more than six inches of frost normally occurring in January, February, and the first half of March.

The study area is comprised of all or portions of seven watersheds, including 24.9 square miles, or all, of the Kinnickinnic River watershed, which encompasses about 6.6 percent of the study area; 135.9 square miles, or all, of the Menomonee River watershed, which comprises 35.9 percent of the study area; 92.6 square miles, or about 13.4 percent, of the Milwaukee River watershed, which comprises about 24.5 percent of the study area; 27.2 square miles, or all, of the Oak Creek watershed, which comprises about 7.2 percent of the study area; 78 square miles, or about 38 percent, of the Root River watershed, which comprises 19.8 percent of the study area; and 22.8 square miles, or 23.9 percent, of the land directly tributary to Lake Michigan, which comprises about 6 percent of the study area. In addition, about one-half square mile of the Fox River watershed located within the southwest portion of the City of Franklin is located within the study area.

Large-scale topographic maps prepared to Commission standards, which include monumented control, are available for 222 square miles, or 59 percent, of the study area. A total of 1,412 U. S. Public Land Survey corners in the study area have been, or are being, relocated, monumented, and tied to the State Plane Coordinate System, representing 80.6 percent of such corners in the study area.

These large-scale maps facilitate the hydrologic and hydraulic studies required for drainage and flood control planning and engineering, including the delineation of drainage basins, stream network configuration, and stream profiles and cross-sections. The maps are essential for the accurate and precise delineation of flood hazard areas along streams and watercourses and in the determination of monetary flood damages. Finally, such maps are essential for the sound preliminary and final engineering of required drainage and flood control improvements, for the acquisition of flood hazard areas for park and open space purposes, and for the proper exercise of public land use controls to protect floodwater conveyance and storage capacities. These maps are also useful for many other types of municipal planning and public works functions.

Surface elevations in the study area range from a high of over 1,100 feet above National Geodetic Vertical Datum (NGVD)—Mean Sea Level Datum—in the Town of Richfield in the northwest portion of the study area, to approximately 580 feet above NGVD at the mouth of the Milwaukee River as it enters Lake Michigan.

Bedrock topography was shaped by preglacial and glacial erosion of the exposed bedrock. The consolidated bedrock underlying the study area generally dips eastward at a rate of 25 to 30 feet per mile. The bedrock surface ranges from about 800 NGVD in the western part of the study area, to less than 400 feet NGVD at the mouth of the Milwaukee River.

Because of the glacial deposits, few bedrock exposures, either natural or artificial, are available for scientific or recreational purposes. The industrial history of the Region is related to these bedrock exposures. In addition, such exposures may also play an important role in education. These exposures have provided significant evidence of value to the understanding of local and regional geology and to establishing some new geologic concepts such as the presence of fossil reefs. Such bedrock exposures are threatened by various construction projects, including potential stormwater drainage and flood control improvement projects.

Soil properties are an important factor influencing the rate and volume of stormwater runoff from land surfaces. The soil characteristics, slope, and vegetative cover of the land affect the degree of soil erosion which occurs during runoff events. With respect to watershed hydrology, the most significant soil interpretation for stormwater management is the categorization of soils into hydrologic soil groups A, B, C, and D, with soils in the hydrologic soil group A category experiencing very little runoff because of high infiltration capacity, high permeability, and good drainage, and soils in hydrologic soil groups B, C, and D experiencing progressively larger amounts of runoff because of lower infiltration, low permeability, and poor drainage. Hydrologic soil groups A, B, C, and D cover 1 percent, 13 percent, 49 percent, and 14 percent, respectively, of the study area. The remaining 23 percent of the study area is covered by disturbed soils. Sixty-three percent of the study area is covered by soils having poor or very poor drainage characteristics; such soils may be expected to generate relatively large amounts of stormwater runoff.

The terrestrial vegetation in the study area may be divided into two broad land classifications—wetlands and woodlands. Wetlands are those lands that are fully or partially covered with hydrophytic plants and wet or spongy organic soils and that are generally covered with shallow standing water intermittently inundated or having a high water table. Woodlands are defined as those upland areas one acre or more in size having 17 or more deciduous trees per acre, each measuring at least four inches in diameter at breast height, and having at least a 50 percent canopy cover. In 1985, wetlands covered 17.7 square miles, or about 5 percent, of the study area, while woodlands covered about 12.5 square miles, or about 3 percent, of the study area.

There are no major lakes—that is, lakes having 50 acres or more of surface area—within the study area. There are a number of minor lakes, which, together with the surface area of streams within the study area, encompass about three square miles, or less than 1 percent of the total study area. The value of these minor lakes and ponds is largely aesthetic.

One of the most interesting, variable, and occasionally unpredictable features of the resource base is its river and stream system with its ever-changing and sometimes widely fluctuating discharges and stages. Perennial and certain intermittent streams within the study area total about 203 linear miles. Through the application of certain overriding considerations and three specific jurisdictional criteria to streams within the study area, the Milwaukee Metropolitan Sewerage District has assumed jurisdiction for drainage and flood control purposes 113.0 linear miles of streams within existing District boundaries, or about 56 percent of the streams within the study area. In addition, 148.8 linear miles of streams within the possible future District boundaries, or about 73 percent of the linear miles of streams in the study area, are eligible for District jurisdiction. The tributary drainage areas of streams recommended for District jurisdiction within existing District boundaries encompass 261 square miles, or 69 percent, of the total study area, and 298 square miles, or 79 percent, of the study area within possible future District boundaries.

The natural floodplain of a river is a wide, flat, gently sloping area, contiguous to and usually lying on both sides of the channel. A river or stream may be expected to occupy and flow on its floodplain an average of once every two years. Therefore, the floodplain should be considered to be an integral part of the natural stream system. The extent to which a natural floodplain would be occupied by any given flood will depend upon the severity of the flood and, more particularly, upon its elevation or stage. An infinite number of outer limits of the natural floodplain may be delineated, each related to a specified flood recurrence interval. The natural floodlands, for purposes of this study, are defined as consisting of the river channel plus the 100-year floodplain. Floodlands within the study area occupy a total of 28.2 square miles, or approximately 7.5 percent of the study area.

The delineation of selected natural resource and resource-related elements on a map results in an essentially linear pattern of relatively elongated areas which have been termed environmental corridors by the Commission. Primary environmental corridors include a wide variety of important resource and resource-related elements and are at least 400 acres in size, two miles in length, and 200 feet in width. Secondary environmental corridors typically connect with primary environmental corridors and are at least 100 acres in size and one mile in length. Primary environmental corridors in the study area generally lie along the major stream valleys, and contain almost all of the remaining high-value woodlands, wetlands, wildlife habitat, surface waters, and undeveloped floodlands in the study area. They also contain some of the best remaining potential park sites and are a composite of the best individual elements of the natural resource base. Primary environmental corridors encompassed about 27.1 square miles, or 7 percent, of the total study area in 1985. The secondary environmental corridors in the study area are generally located along intermittent streams or serve as links between segments of primary environmental corridor. Secondary corridors also contain a variety of resource elements, but are often remnant resources from the primary corridors which have been developed for other uses. Secondary environmental corridors in the study area, while not as important as the primary environmental corridors, should be preserved in essentially open, natural uses as urban development proceeds, particularly when opportunities are presented to incorporate such corridors into urban stormwater detention areas, associated drainageways, and neighborhood parks. Secondary environmental corridors encompassed 10.8 square miles, or about 2.9 percent, of the total study area in 1985.

Population, employment, and land use demand forecasts selected as a basis for the preparation of the stormwater drainage and flood control planning effort were based upon a regional forecast developed using an "alternative futures" approach. Using this approach, alternative futures were postulated for the Region considering potential changes in the key external factors affecting the development of the Region, including the cost and availability of energy, individual and family lifestyles, and the ability of the Region to compete with other regions of the United States for development. The range of population and economic activity levels attendant to these alternative futures was believed to represent reasonable extremes of future development conditions in the Region. Alternative land use patterns were then developed for each of these extremes in order to provide a range of spatial distributions, population, and economic activity levels within the Region.

The regional population forecast selected as basis for the study area planning efforts—the optimistic growth, centralized development future—anticipates that the resident population of the Region will reach 2.2 million persons by the year 2000. This would represent an increase of about 476,000 persons, or 27 percent, over the 1985 level of 1.74 million persons. Under the pessimistic growth scenario, however, the population of the Region in the year 2000 could be as low as 1,690,000. The study area forecast is based upon normative areawide land use development objectives and envisions a reversal of recent trends. Under the forecast, the population of the study area may be expected to increase from the 1985 level of 1,042,600 to 1,249,000 in the plan design year 2000. This represents an increase of about 206,500 persons, or about 19.8 percent, over the 1985 population level. Under the pessimistic growth scenario, however, the population of the study area in the year 2000 could be as low as 851,900 persons.

Employment levels within the Region under the optimistic growth scenario are envisioned to increase from 871,900 in 1985 to 1,016,000 by the year 2000—an increase of 144,100 employees, or 16.5 percent, over the time period. Under the pessimistic growth scenario, however, employment in the Region in the year 2000 could be as low as 886,900. Within the study area, based upon the optimistic growth scenario, employment is envisioned to increase by 60,100, or about 10.1 percent—from 592,200 in 1985 to 652,300 in the year 2000. Under the pessimistic growth scenario, however, employment in the study area in the design year could be as low as 566,000.

Because of the population and employment level increases envisioned within the study area by the year 2000, the continued conversion of rural land to urban use may be required. Between 1963 and 1985, 45 square miles of land were converted from rural to urban use within the study area, increasing the proportion of the total area of the study area in urban use from about 47 percent in 1963 to over 58 percent in 1985. Under the alternative future selected as the basis for study area planning, the conversion of an additional 20 square miles of land from rural to urban use may be expected between 1985 and the plan design year 2000, an increase of about 9 percent. By the plan design year, then, approximately 241 square miles, or about 64 percent, of the 378-square-mile study area would be in urban use.

Chapter III

DRAINAGE AND FLOOD CONTROL OBJECTIVES, PRINCIPLES, AND STANDARDS AND DESIGN CRITERIA

INTRODUCTION

It is axiomatic that stormwater drainage and flood control facilities must function as integrated systems over entire watersheds and that system plans are, therefore, required for the resolution of drainage and flooding problems. Major portions of the Kinnickinnic, Menomonee, Milwaukee, Oak Creek, and Root River watersheds, along with part of the Lake Michigan direct drainage area, are included within the existing and possible future Milwaukee Metropolitan Sewerage District boundaries. These watersheds comprise the logical units for flood control and stormwater drainage planning within the District. Certain aspects of the Commission watershed planning process are therefore applicable to the District drainage and flood control system planning program.

The formulation of development objectives and supporting standards is one of the most important steps in the Commission watershed planning process applicable to the District drainage and flood control system planning program. Soundly conceived drainage and flood control system development objectives should incorporate the knowledge of people who are best informed not only about the drainage and flood control problems of the greater Milwaukee area, but also about drainage and flood control planning and engineering. To the maximum extent possible, such objectives should be established by public officials legally assigned this task, assisted as necessary not only by planners and engineers but by interested and concerned citizens as well. This participation by public officials and concerned citizens as well as by knowledgeable technicians is important because of the value judgments inherent in any set of development objectives.

The required broad level of participation in the District drainage and flood control system planning process is provided by the governing body of the District itself, which is composed of elected and appointed public officials and citizens; and by the Advisory Committee on Stormwater Drainage and Flood Control Planning for the Milwaukee Metropolitan Sewerage District and District Service Areas created by the governing body of the District to guide the drainage and flood control policy and system planning effort. That Committee is comprised of knowledgeable state, district, county, and municipal officials, and concerned citizens. The full composition of this Committee is listed on the inside of the front cover of this report. One of the important functions of this Committee was to assist in the formulation of a set of drainage and flood control system development objectives which could provide a sound basis for drainage and flood control system plan design, test, and evaluation.

This chapter sets forth the set of drainage and flood control system development objectives and supporting principles and standards approved by the Committee. Some of these objectives, principles, and standards were originally formulated by the Regional Planning Commission under regional water resources planning programs but were deemed relevant to formulation of a drainage and flood control system plan for the Milwaukee Metropolitan Sewerage District. Others were formulated specifically for the District system planning effort.

In addition to presenting drainage and flood control system development objectives, principles, and standards, this chapter discusses certain engineering design criteria and analytic procedures used in the system planning effort to design alternative plan subelements, test the physical feasibility of those subelements, and make necessary economic comparisions between such subelements. The description of these criteria and procedures in this chapter is intended to help provide an understanding by all concerned of the level of detail entailed in the system plan preparation, as well as of the need for refinement of some aspects of that system plan prior to implementation.

BASIC CONCEPTS AND DEFINITIONS

The term "objective" is subject to a wide range of interpretation and application, and is closely linked to other terms often used in planning which are similarly subject to a wide range of interpretation and application. The following definitions have therefore been adopted by the Regional Planning Commission in order to provide a common frame of reference:

1. Objective: a goal or end toward the attainment of which plans and policies are directed.
2. Principle: a fundamental, primary, or generally accepted tenet used to support objectives and prepare standards and plans.
3. Standard: a criterion used as a basis of comparison to determine the adequacy of plan proposals to attain objectives.
4. Plan: a design which seeks to achieve the agreed-upon objectives.
5. Policy: a rule or course of action used to ensure plan implementation.
6. Program: a coordinated series of policies and actions to carry out a plan.

Although this chapter deals primarily with the first three of these terms, an understanding of the interrelationship of the foregoing definitions and the basic concepts which they represent is essential to an understanding of the following discussion of drainage and flood control system development objectives, principles, and standards.

FLOOD CONTROL OBJECTIVES, PRINCIPLES, AND STANDARDS

In order to be useful in the drainage and flood control system planning process, objectives not only must be logically sound and related in a demonstrable and measurable way to alternative physical development proposals, but must be consistent with, and grow out of, broader, areawide development objectives. This is essential if the drainage and flood control plan is to comprise an integral element of a comprehensive plan for the physical development of the greater Milwaukee area, and if sound coordination of areawide land use and drainage and flood control facility development is to be achieved.

In its planning efforts to date, the Southeastern Wisconsin Regional Planning Commission has adopted a number of regional development objectives relating to drainage and flood control, after careful review and recommendation by various advisory and coordinating committees. These objectives, together with their supporting principles and standards, are set forth in previous Commission planning reports. One of the specific water control facility development objectives adopted by the Commission under other planning programs is applicable to the District drainage and flood control system planning effort. The second objective for the District planning effort was developed from one of the standards set forth in previous Commission planning reports. Within the context of the District planning effort, this standard is more appropriately expressed as an objective. These objectives are:

1. An integrated system of drainage and flood control facilities and floodland management programs which will effectively reduce flood damage under the existing land use pattern within the District boundaries and promote the implementation of the adopted land use plans for the watersheds in the District, meeting the anticipated runoff loadings generated by the existing and proposed land uses.
2. An integrated system of flood control and stormwater management facilities designed to minimize the negative impacts on fish and other aquatic life and to support the water use objectives set forth in the regional water quality management plan.

Principles and Standards

Complementing each of the foregoing water control facility development objectives are a planning principle which supports the objective and asserts its inherent validity, and a set of quantifiable planning standards which can be used to evaluate the relative or absolute ability of alternative plan designs to meet the stated objective. These principles and standards, as they apply to drainage and flood control system planning, are set forth in Table 20, and serve to facilitate quantitative application of the objectives during plan design, test, and evaluation. In addition, Map 25 illustrates the application of the recommended water use objectives for streams within the Milwaukee Metropolitan Sewerage District and flood control study area.

It should be noted that the planning standards herein recommended for adoption fall into two groups: comparative and absolute. The comparative standards, by their very nature, can be applied only through a comparison of alternative plan proposals. Absolute standards can be applied

Table 20

WATER CONTROL FACILITY DEVELOPMENT OBJECTIVES, PRINCIPLES, AND STANDARDS FOR THE MILWAUKEE METROPOLITAN SEWERAGE DISTRICT

OBJECTIVE NO. 1

An integrated system of drainage and flood control facilities and floodland management programs which will effectively reduce flood damage under the existing land use pattern within the District boundaries and promote the implementation of the adopted land use plans for the watersheds in the District, meeting the anticipated runoff loadings generated by the existing and proposed land uses.

PRINCIPLE

Reliable local municipal stormwater drainage facilities cannot be properly planned, designed, or constructed except as integral parts of an areawide system of floodwater conveyance and storage facilities centered on major waterways and designed so that the hydraulic capacity of each waterway opening and channel reach abets the common aim of providing for the storage, as well as the movement, of floodwaters. Not only does the land use pattern of the tributary drainage area affect the required hydraulic capacity of the drainage and flood control facilities, but the effectiveness of the floodwater conveyance and storage facilities affects the uses to which land within the tributary watershed, and particularly within the riverine areas of the watershed, may properly be put.

STANDARDS

1. All new and replacement bridges and culverts over waterways shall be designed so as to accommodate, according to the categories listed below, the designated flood events without overtopping of the related roadway or railway track and resultant disruption of traffic by floodwaters.

   a. Minor and collector streets used or intended to be used primarily for access to abutting properties: a 10-year recurrence interval flood discharge.

   b. Arterial streets and highways, other than freeways and expressways, used or intended to be used primarily to carry heavy volumes of fast, through traffic: a 50-year recurrence interval flood discharge.

   c. Freeways and expressways: a 100-year recurrence interval flood discharge.

   d. Railways: a 100-year recurrence interval flood discharge.

2. All new and replacement bridges and culverts over waterways, including pedestrian and other minor bridges, in addition to meeting the applicable requirements of paragraph number 1 above, shall be designed so as to accommodate the 100-year recurrence interval flood event without raising the peak stage, either upstream or downstream, 0.01 foot or more above the peak stage for the 100-year recurrence interval flood, as established in the adopted drainage and flood control plan.[a] Larger permissible flood stage increases may be acceptable for reaches having topographic or land use conditions which could accommodate the increased stage without creating additional flood damage potential upstream or downstream of the proposed structure.

3. The waterway opening of all new and replacement bridges shall be designed so as to readily facilitate the passage of ice floes and other floating debris, and thereby avoid blockages often associated with bridge failure and with unpredictable backwater effects and flood damages. In this respect, it should be recognized that clear spans and rectangular openings are more efficient than interrupted spans and curvilinear openings in allowing the passage of ice floes and other floating debris.

4. Certain new or replacement bridges and culverts over waterways, including pedestrian and other minor bridges, so located with respect to the stream system that the accumulation of floating ice or other debris may cause significant backwater effects with attendant danger to life, public health, or safety, or attendant serious damage to homes, industrial and commer-

cial buildings, and important public utilities, shall be designed so as to pass the 100-year recurrence interval flood with at least 2.0 feet of freeboard between the peak stage and the low concrete or steel in the bridge span.

5. Standards 1, 3, and 4 shall also be used as the criteria for assessing the adequacy of the hydraulic capacity and structural safety of existing bridges or culverts over waterways and thereby serve, within the context of the adopted drainage and flood control system plan, as the basis for crossing modification or replacement recommendations designed to alleviate flooding and other problems.

6. All new and replacement bridges and culverts over waterways shall be designed so as not to inhibit fish passage in areas that are supporting, or which are capable of supporting, valuable recreational sport and forage fish species.

7. Channel modifications, dikes, and floodwalls should be restricted to the minimum number and extent absolutely necessary for the protection of existing and proposed land use development, consistent with the land use elements of the adopted comprehensive plans for the watersheds within the District. The upstream and downstream effect of such structural works on flood discharges and stages shall be determined, and any such structural works which may significantly increase upstream or downstream peak flood discharges should be used only in conjunction with complementary facilities for the storage and movement of the incremental floodwaters through the watershed stream system. Channel modifications, dikes, or floodwalls shall not increase the height of the 100-year recurrence interval flood 0.01 foot or more in any unprotected upstream or downstream stream reaches.[a] Increases in flood stages that are equal to or greater than 0.01 foot resulting from any channel, dike, or floodwall construction shall be contained within the upstream or downstream extent of the channel, dike, or floodwall, except where topographic or land use conditions could accommodate the increased stage without creating additional flood damage potential.

8. In cases where a dike or floodwall is intended to protect human life, the minimum dike or floodwall top elevation shall be determined using whichever of the following produces the highest profile.

a. The 100-year recurrence interval flood profile plus three feet of freeboard, or

b. The 500-year recurrence interval flood profile.

The height of low dikes or floodwalls that are not intended to protect human life shall be based on the high-water surface profiles for the 100-year recurrence interval flood prepared under the drainage and flood control plan, and shall be capable of passing the 100-year recurrence interval flood with a freeboard of at least 2.0 feet.

9. The construction of channel modifications, dikes, or floodwalls shall be deemed to change the limits and extent of the associated floodways and floodplains. However, no such change in the extent of the associated floodways and floodplains shall become effective for the purposes of land use regulation until such time as the channel modifications, dikes, or floodwalls are actually constructed and operative. Any development in a former floodway or floodplain located to the landward side of any dike or floodwall shall be provided with adequate drainage so as to avoid ponding and associated damages.

10. Reduced regulatory flood protection elevations and accompanying reduced floodway or floodplain areas resulting from any proposed dams or diversion channels shall not become effective for the purposes of land use regulation until the reservoirs or channels are actually constructed and operative.

11. All water control facilities other than bridges and culverts, such as dams and diversion structures, shall be designed to meet the spillway discharge capacity requirements of Chapter NR 333 of the Wisconsin Administrative Code.[b] According to Chapter NR 333, dams whose failure would present a low hazard to downstream human life and property shall have a minimum spillway capacity ranging from the 50-year recurrence interval flood to the 200-year recurrence interval flood, depending on the size of the dam and on downstream land use and land use control classifications; dams whose failure could present a significant hazard to downstream human life and/or property shall have a minimum spillway capacity ranging from the 200-year recurrence interval flood to the 500-year recurrence interval flood, depending on the size of the dam; and dams whose failure could present a high hazard to downstream human life and/or property shall have a minimum spillway capacity ranging from the 500-year recurrence interval flood to the 1,000-year recurrence interval flood, depending on the size of the dam. As applied by the Commission, the definition of hazard to property includes damage to homes, industrial and commercial buildings, and important public utilities and closure of principal transportation routes.

12. All water control facilities should be compatible with existing local stormwater management plans and as flexible as practical to accommodate future local stormwater management planning which shall include consideration of onsite control.

PRINCIPLE

Floodlands that are unoccupied by, and not committed to, urban development should be retained in an essentially natural open space condition supplemented with the development of selected areas for public recreational uses. Maintaining floodlands in open uses will serve to protect downstream riverine communities from the adverse effects of the actions of upstream riverine communities by discouraging floodland development that would significantly aggravate existing flood problems or create new flood problems; will preserve natural floodwater conveyance and storage capacities; will avoid increased peak flood discharges and stages; will contribute to the preservation of wetland, woodland, and wildlife habitat as part of a continuous linear system of open space; and will enhance the quality of life for both the urban and rural population by preserving and protecting the recreational, aesthetic, ecological, and cultural values of riverine areas.

STANDARDS

1. All public land acquisitions, easements, floodland use regulations, and other measures intended to eliminate the need for water control facilities shall, in all areas not already in intensive urban use or committed to such use, encompass at least all of the riverine areas lying within the 100-year recurrence interval flood inundation line.

2. Where hydraulic floodways are to be delineated, they shall to the maximum extent feasible accommodate existing, committed, and planned floodplain land uses.

3. In the determination of a hydraulic floodway, the hydraulic effect of the potential floodplain encroachment shall be limited so that the peak stage of the 100-year recurrence interval flood is not raised by 0.01 foot or more.[a] Larger stage increases may be acceptable if appropriate legal arrangements are made with affected local units of government and property owners.

OBJECTIVE NO. 2

An integrated system of flood control and stormwater management facilities designed to minimize the negative impacts on fish and other aquatic life and to support the water use objectives set forth in the regional water quality management plan.

PRINCIPLE

Surface water is one of the most valuable resources of southeastern Wisconsin; and, even under the effects of increasing population and economic activity levels, the potential of natural streamwaters to serve a reasonable variety of beneficial uses, in addition to the functions of flood-flow conveyance and waste transport and assimilation, should be protected and preserved.

STANDARDS

1. Stormwater drainage and flood control facilities should not degrade the existing water quality in streams and watercourses and should support those water use objectives and supporting water quality standards set forth in the adopted areawide water quality management plan as documented in SEWRPC Planning Report No. 30, A Regional Water Quality Management Plan for Southeastern Wisconsin: 2000, Volume Three, Recommended Plan, June 1979, and subsequent duly adopted amendments thereto. The applicable water use objectives for the streams and watercourses concerned are indicated on Map 25. The supporting water quality standards were originally presented in SEWRPC Planning Report No. 30, and were revised in Volume Two of SEWRPC Planning Report No. 37, A Comprehensive Plan for the Milwaukee Harbor Estuary, December 1987.

2. Water control facilities should be designed to minimize adverse impacts on wetlands.

[a]Regional Planning Commission watershed studies conducted prior to the Kinnickinnic River watershed study used a standard of 0.5 foot. That standard was reduced in the Kinnickinnic River, Pike River, and Oak Creek watershed plans in order to be consistent with revisions to the Wisconsin Administrative Code. Chapter NR 116 of the Code was revised by the Wisconsin Department of Natural Resources in July 1977 so as to specify a maximum computed stage increase of only 0.1 foot. The July 1977 edition of Chapter NR 116 was repealed and a new Chapter NR 116 was created effective March 1, 1986. The new NR 116 provides that the maximum computed increase in flood stage must be less than 0.01 foot. In effect, the new code permits no increase in flood stage. Deviations from this Department standard may be approved by the Department if "the appropriate legal arrangements have been made with all property owners affected by the increased flood elevations" and if "any affected municipality (meets) all legal requirements for amending its water surface profiles, floodplain zoning maps, and zoning ordinances."

Although the Regional Planning Commission has adopted the numerically more stringent standard in order to be consistent with the Wisconsin Administrative Code, the Commission staff has repeatedly expressed concern with the use of 0.01 foot and, more particularly, with the accuracy of hydraulic computations that is implied by that standard. The Commission staff, in a June 15, 1984, letter to Mr. Larry A. Larsen, Chief of the Floodplain-Shoreland Management Section of the Wisconsin Department of Natural Resources, stated that "with the use of computerized methods of analysis, the numerical output of an evaluation could indicate an increase in stages of less than 0.1 foot but greater than 0.01 foot which, when reviewed, would be considered to be the result of the modeling technique and would, in reality, indicate that no increase in stage may be expected."

[b]Regional Planning Commission studies conducted prior to the Milwaukee Metropolitan Sewerage District stormwater drainage and flood control planning effort used a design spillway discharge capacity equal to the 100-year recurrence interval flood if failure of a water control facility would damage only agricultural lands and isolated farm buildings, and a design discharge capacity equal to the standard project flood or the more severe probable maximum flood where failure of a water control facility could jeopardize public health and safety, cause loss of life, or seriously damage homes, industrial and commercial buildings, and important public utilities, or result in closure of principal transportation routes. The spillway discharge capacity standards applied by the Commission prior to the District drainage and flood control planning effort were as stringent as or, in the case of use of the standard project flood or probable maximum flood, more stringent than the Wisconsin Department of Natural Resources guidelines applied prior to adoption of Chapter NR 333. The Commission has revised the previous standards to be consistent with Chapter NR 333 of the Wisconsin Administrative Code as it was created effective June 1, 1985.

Source: SEWRPC.

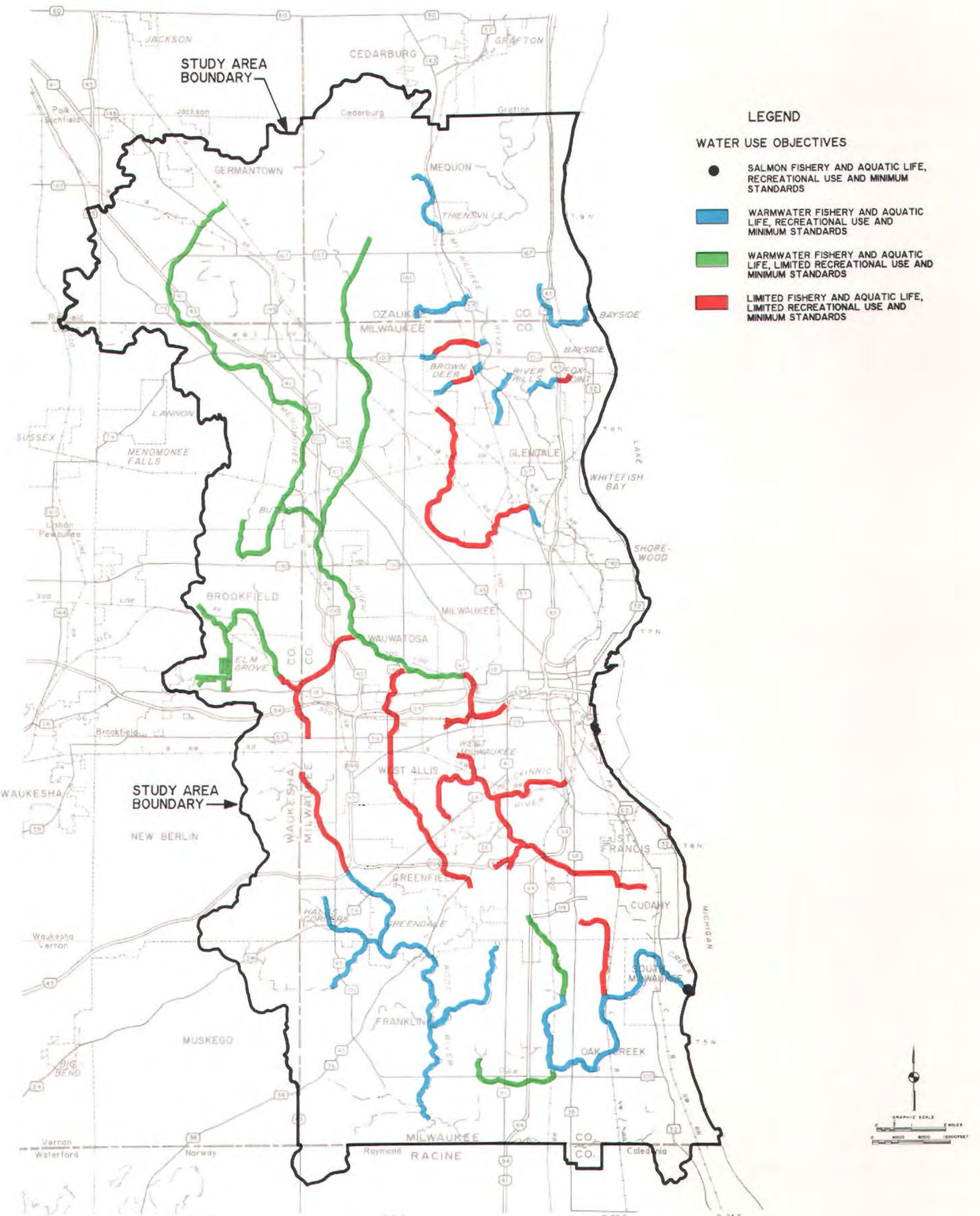
individually to each alternative plan proposal since they are expressed in terms of maximum, minimum, or desirable values. The standards set forth herein should serve as aids not only in the development, test, and evaluation of water control facility plans, but also in the development of plan implementation policies and programs.

Overriding Considerations

When applying the drainage and flood control system development objectives, principles, and standards to the design, test, and evaluation of alternative plan elements, several overriding considerations must be recognized. First, it must be recognized that any proposed water control management facilities must constitute integral parts of a total system. It is not possible through application of the objectives and standards alone, however, to assure such system integration, since the objectives and standards cannot be used to determine the effect of individual facilities and controls on each other or on the system as a whole. This requires the application of planning and engineering techniques developed for this purpose—such as hydrologic and hydraulic simulation—to quantitatively test the performance of the proposed facilities as part of a total system, thereby permitting adjustment of the spatial distribution and capacities of the facilities to the existing and future runoff as derived from the adopted land use plans. Second, it must be recognized that it is unlikely that any one plan proposal will meet all the standards completely. Thus, the extent to which each standard is met, exceeded, or violated must serve as a measure of the ability of each alternative plan proposal to achieve the specific objective which the given standard complements. Third, it must be recognized that the objectives may in some cases be in conflict, and that such conflict will require resolution through compromise; such compromise is an essential part of any design effort.

Map 25

RECOMMENDED WATER USE OBJECTIVES FOR STREAMS WITHIN THE MILWAUKEE METROPOLITAN SEWERAGE DISTRICT DRAINAGE AND FLOOD CONTROL STUDY AREA



Source: SEWRPC.

ALTERNATIVE FLOOD CONTROL MEASURES

Drainage and flood control measures may be broadly subdivided into two categories: structural measures and nonstructural measures. Structural measures include floodwater storage facilities, such as wet and dry detention basins; percolation basins and infiltration devices; diversions; containment facilities, such as earthen dikes and concrete floodwalls; conveyance facilities, such as major channel modifications; and bridge, culvert, and dam modifications or replacements. Nonstructural measures include the preservation of floodlands for recreational and other open space uses; land use regulation, both within and outside floodland areas; utility extension policies; the extension of information; and structure floodproofing and removal. Table 21 lists structural and nonstructural measures for flood control that may be applied individually or in various combinations to portions of the streams and watercourses within the planning area. Structural measures tend to be more effective in achieving the objectives of flood control in riverine areas that have already been urbanized, while nonstructural measures are generally more effective in riverine areas that have not been converted to flood-prone development but have the potential for such development.

Structural Measures

Each of the six structural floodland management measures set forth in Table 21 is described briefly below. Emphasis is placed on the function of each measure; on the key factors, or basic requirements, used to determine if the measure is applicable to a particular stream reach and related riverine area; and on some of the more significant general advantages and disadvantages of each measure.

Storage: From the perspective of floodland management, the function of floodwater storage facilities is either to detain floodwaters upstream of flood-prone areas for subsequent gradual release—as is the case with a detention pond—or to retain floodwaters for gradual release and evaporation or for groundwater recharge—as is the case with a retention pond—thereby decreasing downstream discharges and flood stages and associated flood damages. A key consideration in applying this alternative is the existence of sites of sufficient storage volume that are properly positioned upstream of flood-prone riverine areas and that are located so as to control the runoff from a significant portion of the total drainage area tributary to the flood-prone reaches. In addition, the site must be available; that is, it must not contain significant urban development.

Centralized floodwater storage facilities, consisting of a relatively few but large facilities, may be directly located on the stream system, such as the case of a conventional reservoir, or may be located off the channel system as in an abandoned quarry or in excavated chambers in the underlying bedrock. Decentralized storage facilities, consisting of relatively many but small facilities, may also be provided in the headwater areas of a stream system.

Storage reservoirs may be of the detention type or of the retention type. The former type is designed to hold back or delay stormwater runoff temporarily. The latter is designed for the long-term storage of stormwater without release to the surface water drainage system.

Storage facilities have the advantage of being able to potentially mitigate flooding along downstream channel reaches, in contrast with other structural measures which generally provide only more local flood relief. Storage facilities may also be multiuse, providing recreational, low-flow augmentation, water supply, and flood control benefits. The wet-type storage facilities may also provide nonpoint source water pollution control benefits downstream from the facility. The disadvantages of storage include high capital costs; large land area requirements; in some cases potentially adverse water quality conditions within the impoundments; potentially unfavorable impacts on fisheries; relatively high ongoing maintenance costs; and the false sense of security regarding flood dangers that may be engendered in downstream reaches, leading to the possible influx of urban development into the remaining flood-prone areas.

Infiltration Devices: Infiltration devices are designed to serve the dual purpose of reducing the volume and rate of stormwater runoff and reducing the pollutant contribution to receiving waters. Such devices include roof drain leaders directed to lawns, soak-away pits, infiltration trenches, percolation basins, grass swales and waterways, porous pavements, and perforated drainage systems. Roof drains connected to lawns, soak-away pits, grass swales, and infiltration trenches may be located in upland areas to control runoff and pollutants at the source. Percolation basins are generally located

Table 21

ALTERNATIVE FLOODLAND MANAGEMENT MEASURES

Alternative: Major Category	Alternative: Name	Function	Comment
Structural	Storage	To detain floodwaters upstream of flood-prone reaches for subsequent gradual release	May be accomplished by on-channel reservoirs or by off-channel or underground storage
	Infiltration devices	To reduce stormwater runoff volumes, flow rates, and contaminant contributions to receiving waters	Include soak-away pits, infiltration trenches, percolation basins, grass swales and waterways, porous pavements, and perforated drainage systems
	Diversion	To divert waters from a point upstream of the flood-prone reaches and discharge to an acceptable receiving watercourse outside the watershed	Man entail legal problems
	Dikes and floodwalls	To prevent the occurrence of overland flow from the channel to floodland structures and facilities	- -
	Channel modification and enclosure	To convey flood flows through a river reach at significantly lower stages	May be accomplished by straightening, lowering, widening, lining, and otherwise modifying a channel or by enclosing a major stream, including construction of a new length of channel for the purpose of bypassing a reach of natural stream
	Bridge and culvert alteration or replacement	To reduce the backwater effect of bridges and culverts	May be accomplished by increasing the waterway opening or otherwise substantially altering the crossing or by replacing it
Nonstructural	Reservation of floodlands for recreational and related open space use	To minimize flood damage by using floodlands for compatible recreational and related open space uses and also to retain floodwater storage and conveyance	May be accomplished through private development, such as a golf course, or by public acquisition of the land or of an easement
	Floodland regulations	To control the manner in which new urban development is carried out in the floodlands so as to assure that it does not aggravate upstream and downstream flood problems	May be accomplished through zoning, land subdivision control, sanitary and building ordinances
	Control of land use outside the floodlands	To control the manner in which urban development occurs outside the floodlands so as to minimize the hydrologic impact on downstream floodlands	- -
	Flood insurance	To minimize monetary loss or reduce monetary impact on structure owner	Premiums may be subsidized or actuarially determined
	Lending institution policies	To discourage acquisition or construction of flood-prone structures by means of mortgage granting procedures	- -
	Realtor policies	To discourage acquisition or construction of flood-prone structures by providing flood hazard information to prospective buyers	- -
	Community utility policies	To discourage construction in flood-prone areas by controlling the extension of utilities and services	- -
	Emergency programs	To minimize the danger, damage, and disruption from impending flood events	Such a program may include installation of remote stage sensors and alarms, road closures, and evacuation of residents
	Structure floodproofing	To minimize damage to structures by applying a combination of protective measures and procedures on a structure-by-structure basis	- -
	Structure removal	To eliminate damage to existing structures by removing them from flood-prone areas	- -

Source: SEWRPC.

at stormwater outfalls and can be used in conjunction with grass waterways, porous pavements, and perforated drainage systems to reduce runoff and pollutant loadings from all sources contributing to the drainage network. The detailed design of infiltration devices is highly site-specific and may require field testing of soils and infiltration rates.

Infiltration devices have the advantage of controlling both stormwater runoff and pollutant loadings. Such devices may also reduce downstream storm drainage system costs by reducing flow rates and volumes, may increase groundwater recharge, and may aid in the maintenance of base streamflows during dry-weather periods. Infiltration devices have the greatest practical application, and the greatest flexibility in installation, in rural and suburban areas. The disadvantages of such devices are the difficulties entailed in introducing such devices into existing urbanized areas; frequent and costly maintenance requirements; adverse secondary effects such as wet basements, excessive operation of sump pumps, excessive infiltration of clear water into sanitary sewerage systems, and groundwater contamination; and the creation of unstable conditions for and damage to pavements and structures, particularly in wet, cold climates.

Diversion: The function of a diversion is to intercept potentially damaging flood flows at a location upstream of the flood-prone reach and to convey those flows to an acceptable receiving watercourse beyond the flood-prone reach or outside the watershed in which the flood mitigation is required. Diversion alternatives require a control structure located on the stream channel that establishes the stage at which the diversion process will begin and the rate at which it will occur; and an open channel or closed conduit conveyance facility to carry the diverted floodwaters from the stream to the point of discharge. A key consideration in assessing the applicability of diversion is the availability of a suitable diversion route, or alignment, and an adequate receiving watercourse to which the floodwaters may be diverted without harmful physical effects or legal challenge.

Diversion, like storage, has the potential to abate flooding along downstream channel reaches. Disadvantages include high capital costs, potential legal liabilities entailed in the transfer of water between watersheds, and the false sense of security regarding downstream flood dangers that may develop as a result of the construction of a diversion facility. This alternative does not lend itself to the ready incorporation of nonpoint source water pollution abatement actions.

Dikes and Floodwalls: Earthen dikes and concrete or sheet steel floodwalls are means of providing flood control in certain damage-prone stream reaches. The function of dikes and floodwalls is to contain the floodwaters; that is, to prevent the occurrence of lateral overland flow from the channel to adjacent floodland areas containing flood damage-prone structures and facilities. A key consideration in the application of this measure is the availability of sufficient space between the stream channel and the land uses that are to be protected to permit the construction of the dikes or floodwalls, the latter having the advantage of requiring a narrower strip of land.

During major flood events, high river levels may reverse the flow in the local stormwater drainage system, resulting in the movement of floodwaters from the stream into developed riverine areas, causing inundation and damage. To prevent such backflow into protected areas, dikes and floodwalls normally must be supplemented by backwater gates on storm sewer and drainage outlets that have inlets at elevations approximating the design flood stage. Backwater gates function as valves that normally pass the stormwater to the river but close when the hydraulic head on the river side of the hinged gate exceeds the head on the opposite side of the gate.

Backwater gates may, however, create local drainage problems attributable to the accumulation of stormwater runoff which does not have access to the stream because of the closed gate. Areas susceptible to the resulting inundation can be afforded protection through the provision of pumping stations to convey the impounded storm drainage over the dikes and floodwalls to the stream during major flood events.

An important factor which must be considered in the design of dikes and floodwalls is the flood stage against which protection is to be provided. This stage may be higher than the "natural" stage as a result of the lateral constriction imposed on the stream by the dikes and floodwalls. This higher stage, together with an appropriate freeboard, must be used to establish the crest elevation of the dikes and floodwalls.

An advantage of dikes and floodwalls is that they can generally provide local protection quickly. Disadvantages of such facilities include high capital costs, the potential for increasing upstream flood stages, and the potential for reducing the floodwater storage capacity of the stream and attendant floodlands, thereby increasing downstream discharges and associated stages. These facilities can also have a perceived negative aesthetic impact, and may engender a false sense of security about flood dangers.

Channel Modification and Enclosure: Channel modification may include one or more of the following changes to the natural stream channel, all designed to increase the capacity of that channel: straightening, and deepening and widening; placement of a concrete invert and partial sidewalls; and reconstruction of selected bridges and culverts as needed. In some instances, a completely new length of channel may be constructed. The stream channel may also be placed in a large covered conduit along or close to the alignment of the stream reach to convey floodwaters through an area in a manner that may substantially reduce overland flooding.

The function of channel modifications or enclosures is to provide a lower, hydraulically more efficient waterway through which a given flood discharge can be conveyed at a substantially lower stage relative to that which would exist under natural or prechannelized conditions. Key considerations in applying this measure include the availability of required right-of-way of sufficient width to accommodate the modified or relocated channel, and the length of upstream and downstream natural channel reaches that must be modified to provide an acceptable transition from the natural channel and floodplain to the channelized or enclosed reach.

A key advantage of channelization or enclosure is that it can be quickly applied to local stream reaches. Such channels also have low maintenance costs. Disadvantages include a possible perceived negative aesthetic impact and the potential, because of the loss of channel storage, to aggravate downstream problems by increasing downstream discharges and stages. Channelization incorporating concrete inverts and sidewalls may have a harmful effect on fish and other biota and may result in the loss of recreational uses. These structures may have a high capital cost and may contribute to increased flood stages and channel degradation in natural downstream reaches.

Bridge and Culvert Alteration or Replacement: Highway and railway bridges and culverts may significantly affect upstream flood stages and downstream flood stages and discharges and thereby aggravate existing flood problems or create such problems. Bridge and culvert alteration or replacement is intended to avoid or minimize the adverse hydrologic and hydraulic effects of existing bridges and culverts on flood flows and stages. This structural measure is normally most applicable in areas where the waterway crossings are relatively old and undersized. The usefulness of this structural alternative in a watershed is contingent upon identifying those bridges and culverts that produce major backwater effects as a result of inadequate hydraulic capacity, and identifying those structures that are impassable during major flood events.

Although bridge and culvert modification usually entails increasing the waterway opening of the structures to increase their capacity, there are situations in which it may be desirable to maintain the waterway opening of the existing structure or to actually decrease that opening in order to decrease downstream flood flows and stages.

Nonstructural Measures

Each of the nonstructural floodland management measures presented in Table 21 is discussed briefly below. The function of each measure is described and the key factors or basic requirements needed to determine if the given alternative applies to a riverine area or portion of a watershed are discussed. In addition, some of the more significant advantages and disadvantages of the various measures are identified.

Reservation of Floodlands for Recreational and Related Open Space Uses: There is a need in metropolitan areas for active and passive recreational and open space lands readily accessible to residents. Floodplains provide an ideal location for such lands both because recreational use frequently is compatible with the flood hazard and because other forms of intensive flood damage-prone urban development are incompatible with the flood hazard. Recreational and related open space use of floodlands may be accomplished by several mechanisms, including public purchase or other acquisition in fee simple or purchase or other acquisition of easements. The principal advantage of this alternative is its definitive nature and legal incontestability. The key disadvantage is the cost. Land developers may be receptive to dedicating floodlands to public open space use since floodlands are usually not well suited to urban development, not only because of the flood hazard but also because of soil and groundwater conditions and utility availability, since land subdivision regulations often require developers to provide a minimum amount of recreational land as a part of a proposed urban development, and since existing floodland regulations may limit the extent of floodland development. It should also be noted that the preservation of floodlands for recreation and open space uses may also have a favorable impact on the value of property in proximity to the riverine area.

Floodland Regulations: Floodland regulations take the form of, or are incorporated into, zoning, land subdivision, sanitary, and building ordinances adopted by counties, cities, villages, and towns under the police powers granted them by the State Legislature. Such regulations are intended to mitigate flood damage by controlling the manner in which new urban development is carried out in the floodlands so as to assure that it is not flood-prone and, equally important, that it does not aggravate upstream and downstream flood problems.

Floodlands in Wisconsin are governed primarily by rules and regulations adopted by the Wisconsin Department of Natural Resources pursuant to Wisconsin Statutes. All counties, cities, and villages are expected to adopt reasonable and effective floodland regulations under the enabling statutes. The principal advantages of floodland regulations are that they control the manner in which new development occurs in riverine areas, and also control selected practices by which existing urban or rural lands are managed. The principal disadvantage is that they offer no relief from existing flood damage.

Floodland use regulations as promulgated by the Wisconsin Department of Natural Resources promote the use of a two-district floodway-floodplain fringe approach. This approach, in practice, promotes the development of all floodplain fringe areas located beyond the limits of the floodway. To avoid this problem, a three-district approach is often used in order to preserve as much of the floodplain fringe area in open uses as possible, thereby preserving the natural floodwater storage capacity of the riverine area. The Wisconsin Administrative Code requires that floodways be delineated so as to essentially not cause any increase in the regulatory or 100-year recurrence interval flood stage.

Although stipulation of an essentially "no stage increase" floodway eliminates or reduces some of the problems associated with the two-district floodway-floodplain fringe approach to floodland regulations, several significant disadvantages remain. Under the Department's two-district approach, filling and development of the floodland fringe area is indiscriminately permitted under specified conditions. Such filling and development may lead to a marked increase in downstream flood discharges and stages. The delineation of a floodway, by constricting the cross-sectional flow area, may also increase flood stages, thereby laterally extending the floodplain boundary and subjecting additional lands and structures to floodland regulation. Also, floodland fill with development outside the floodway limits, but within environmentally critical areas, may lead to the destruction of environmentally sensitive riverine areas.

Floodland and other land use recommendations can be made more effective for environmental corridor protection as well as for flood damage mitigation. For example, more comprehensive floodland regulations in as yet undeveloped areas may simply designate a single floodland district from which all flood-prone development is excluded, or, as already noted, may incorporate a floodway, a developable floodplain fringe, and an undevelopable conservancy district.

Chapter NR 116 of the Wisconsin Administrative Code provides for, but does not require, use of alternative floodland districts. For instance, Chapter NR 116 contains the designation of a flood storage district. The flood storage district is comparable to a floodplain conservancy district. If development would remove storage volume from a flood storage district, that development is not permitted unless either compensatory storage volume is provided or the entire flood storage district is rezoned to a floodland fringe district. In the shallow depth flooding district, development that would cause an obstruction to flood flows and would increase the 100-year recurrence interval flood elevation is not permitted unless the entire shallow flooding district is rezoned to a floodland fringe district.

Control of Land Use Outside Floodlands: It is important to regulate the manner in which urban development occurs outside, as well as within, floodlands so as to minimize the hydrologic and hydraulic impacts on floodland areas receiving runoff from tributary watershed areas. The hydrologic and hydraulic interdependence between the land surface and the streamflow regimen of a watershed suggests that areawide land use planning is an essential part of effective flood control.[1] It is important, therefore, that structural and nonstructural flood control measures be based upon an areawide land use plan which considers the hydrologic-hydraulic consequences of the location of future urban development, the amount of impervious surface in that development, and the manner in which stormwater runoff from new development is controlled.

Federal Flood Insurance: The federal government encourages the purchase of flood insurance by individual landowners to reduce the need for periodic federal disaster assistance. From the perspective of the owner of flood-prone residential, commercial, or industrial structures, federal flood insurance provides a means of distributing monetary flood losses in the form of an annual flood insurance premium. One of the requirements that must be met by a community before landowners can participate in the federal flood insurance program is that the community must enact land use controls which meet federal standards for floodland protection and development. A very close tie, therefore, exists between two of the nonstructural floodland measures—the federal flood insurance program and floodland regulations.

Lending Institution and Realtor Policies: Lending institutions and realtors have gradually become more aware of the flood hazards associated with properties located in floodland areas. The interest of lending institutions and realtors in the flood-prone status of property has been intensified by the federal flood insurance program, which requires the purchase of flood insurance for any structure within a flood hazard area when the purchaser seeks a mortgage through a federally supervised lending institution. Under state regulation, it is incumbent on real estate brokers, salesmen, or their agents to inform potential purchasers of property of any flood hazards which may exist. The purpose of this regulation is to reduce the unwitting acquisition or construction of flood-prone structures by providing information to prospective buyers.

Utility Extension Policies: Under state regulation, sanitary sewer service may not be extended into flood hazard areas to the extent that such areas are a part of an environmental corridor.[2] Local communities may supplement this regulation by policies which prevent the extension of sewers and other public utility services, such as water supply, into any flood-prone areas. Such policies discourage the development of flood-prone areas and help to avoid the need to construct flood control works.

Emergency Programs: The function of an emergency program is to minimize the damage and disruption associated with flooding through a coordinated, preplanned action which is to be taken when a flood is impending or occurring. Such a program may include the installation of remote upstream sensors and alarms, preplanned road closures, evacuation of residents, and mobilization of

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[1] *For a graphic demonstration of the potential impact of land use changes outside floodland areas on flood discharges, stage, and damage, refer to SEWRPC Planning Report No. 26, A Comprehensive Plan for the Menomonee River Watershed, Volume Two, Alternative Plans and Recommended Plan, October 1976, pp. 72-97.*

[2] *An environmental corridor is defined by the Regional Planning Commission as an elongated area in the landscape encompassing the best remaining natural resource features of an area, including its lakes and streams and associated floodlands and shorelands; its woodlands, wetlands, and wildlife habitat; areas of groundwater discharge and recharge; organic soils; and significant geological formations and physiographic features. By maintaining such corridors in essentially natural, open uses—through appropriate floodland and conservancy zoning and through acquisition for public park and parkway purposes—groundwater and surface water quality will be protected and enhanced, soil erosion and sedimentation abated, air cleansed, wildlife population maintained, and important scientific and educational areas protected. Such corridors are generally well suited to outdoor recreational use, but poorly suited to intensive urban uses. The exclusion of such urban uses from the corridors will minimize costly flood damages and attendant hazards to public health and safety, avoid excessive infiltration of clear water into sanitary sewer systems, and avoid wet basements and failures of foundations for buildings and pavements. The maintenance of such environmental corridors in natural, open uses will lend form and structure to urban development and provide a natural boundary to urban neighborhoods. In addition, such corridors provide excellent buffers between incompatible urban land uses, thus contributing to the aesthetic character and economic value of urban development and the stability of urban residential neighborhoods.*

portable pumping equipment to relieve the surcharge of sanitary sewers. In small watersheds, the "flashy" nature of the hydrologic-hydraulic system may preclude, as a practical matter, the effective implementation of any warning system as a part of the emergency program.

Structure Floodproofing: Residential, commercial, and industrial structures located within or adjacent to floodlands are vulnerable to flood damage because of the variety of ways in which floodwaters can enter such structures. It is possible and generally practicable for individual owners to make adjustments to their structures and to employ certain measures or procedures that will significantly reduce flood damages. This approach is referred to as floodproofing.

Floodproofing techniques may be designed to prevent the entry of floodwaters into the structure or to ensure continuation of utility and other services during flood events, thereby protecting the structure contents in the event that floodwaters do, by design or otherwise, enter the building. Floodproofing measures should be applied only under the guidance of a registered professional engineer who has carefully inspected the building and contents, and has analyzed its structural integrity and evaluated the flood threat. A program of floodproofing could be initiated and supervised by the local community concerned.

Floodproofing measures may include the installation of backwater valves in sanitary sewer building connections, the operation of sump pumps to remove any floodwaters that enter the basement of a structure through foundation drains or other openings, the installation of waterproof seals at structural joints, the construction of earthen berms or masonry walls around the structure or cluster of structures, and the installation of glass block in basement window openings and flood shields over doorways or windows or other structure openings. Such measures may also include the elevation of electrical machinery and equipment above flood stage, and the elevation of existing structures to raise their first floors above flood stage.

The principal advantage of floodproofing is that it provides a means by which individual property owners can unilaterally take action to protect flood-prone structures against flood damage. A significant negative aspect of floodproofing is the possibility that it may be applied without adequate professional engineering guidance, thereby leading to possible major damage to the structure, as well as posing a threat to the health and safety of the owners, tenants, and users of the structure. Another negative attribute of floodproofing is the possibility that the technique will not be applied in a coordinated way throughout the entire flood-prone reach of the streams, thereby leaving a significant residual demand for flood relief. It should be noted that under current regulations, floodproofing will not remove the federal requirement for flood insurance.

The preceding paragraphs discuss floodproofing to alleviate structure flooding caused by overbank flows from main watercourses. Another source of structure flooding is inadequate local drainage of stormwater runoff from private land. Such inadequate drainage can be caused by improper grading of individual lots, poor landscaping practices, and poorly maintained roof drainage and drain tile systems. These problems can only be resolved by private landowners on the basis of sound, site-specific, engineering guidance. Because of the site-specific nature of structure flooding caused by such localized drainage inadequacies, the alleviation of such flooding is not addressed in this system plan.

Structure Removal: The removal of structures, in particular those structures having first-floor elevations at or below the design flood stage, may constitute a cost-effective approach to flood damage control. The cost of removing a residential structure from a flood-prone area is computed as the sum of the structure and site acquisition cost, structure demolition or moving cost, site restoration costs, and occupant relocation cost, the last of which is provided to the displaced homeowner or tenant in compensation for expenses incurred as a result of moving.

This approach has the advantage of enhancing the opportunity to develop the aesthetic appearance and recreational potential of riverine areas by restoring floodlands to an essentially natural, open use. A disadvantage of this alternative is the opposition likely to be encountered from some property owners even if offered an equitable price for the flood damage-prone property. Although some of the value placed on a home may be intangible, and therefore cannot be expressed in monetary terms, it is nevertheless real and must be considered when structure removal alternatives are proposed. The removal of such structures may also result in a loss in tax base to the local civil division. The net cost to the community, however, may be considerably less than the amount of the taxes lost because of the compensating effect of other factors, including reduced costs of municipal services and of flood-related emergency services, and the likelihood that some of the evacuated residents will construct new residences within the civil division on previously undeveloped land, thereby restoring some of the lost tax base.

ENGINEERING DESIGN CRITERIA AND ANALYTIC PROCEDURES

Certain engineering design criteria and analytic procedures were utilized in the preparation of the District drainage and flood control system plan. More specifically, these criteria and procedures were used in the design of alternative plan subelements, in the test of the technical feasibility of those subelements, and in the making of the necessary economic comparisons. Although these engineering criteria and procedures are widely accepted and firmly based in current engineering practice, it is, nevertheless, believed useful to document them here.

Rainfall Intensity-Duration-Frequency Relationships

If local stormwater control and areawide watercourse flood control measures are to be compatible and function in a coordinated manner, plans for both must be based on consistent engineering design criteria. A fundamental criterion for both local and areawide drainage and flood control planning and design is the rainfall intensity-duration-frequency relationship representative of the area.

The Commission has developed rainfall intensity-duration-frequency relationships based on precipitation records at the Milwaukee National Weather Service station. These relationships are shown graphically in Figure 8 and in mathematical equation form in Table 22. The curves in Figure 8 and the equations in Table 22 are directly applicable to urban stormwater control system design using the rational formula,[3] with the equations being intended primarily for incorporation into digital computer programs used in stormwater control system analysis and design.

The curves in Figure 9, which relate total rainfall to duration and frequency, are convenient for use in basinwide hydrologic analysis. The variation of rainfall depth with tributary area and the seasonal variation of rainfall probability are shown in Figures 10 and 11, respectively. The relationships presented in Figure 11 indicate that severe rainfall events, as defined by their duration and recurrence interval, are most likely to occur during the months of July, August, and September. All these rainfall relationships are directly applicable within the existing and possible future District boundaries, as well as within the Southeastern Wisconsin Planning Region.

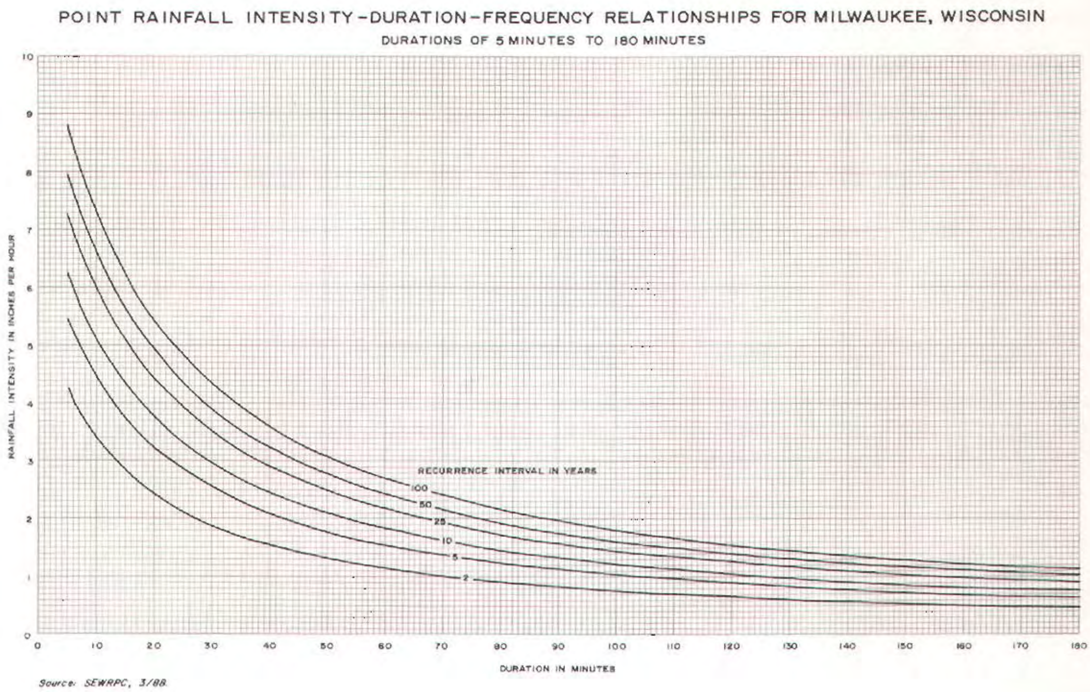
As noted above, the rainfall intensity-duration-frequency relationships are based on analyses of precipitation records compiled at the Milwaukee National Weather Service station. These relationships were initially developed by the Commission in 1965 using rainfall data collected at the Milwaukee station during the 49-year period from 1903 through 1951. The statistical analysis of that rainfall data was revised under the Commission's Fox and Milwaukee River watershed studies in 1969 to incorporate 15 years of additional rainfall data to determine if the additional historical record would alter the rainfall intensity-duration-frequency relationships. Only slight alterations were found, but as a result of the supplemental analysis, rainfall intensity-duration-frequency relationships were available for the 64-year period from 1903 through 1966.

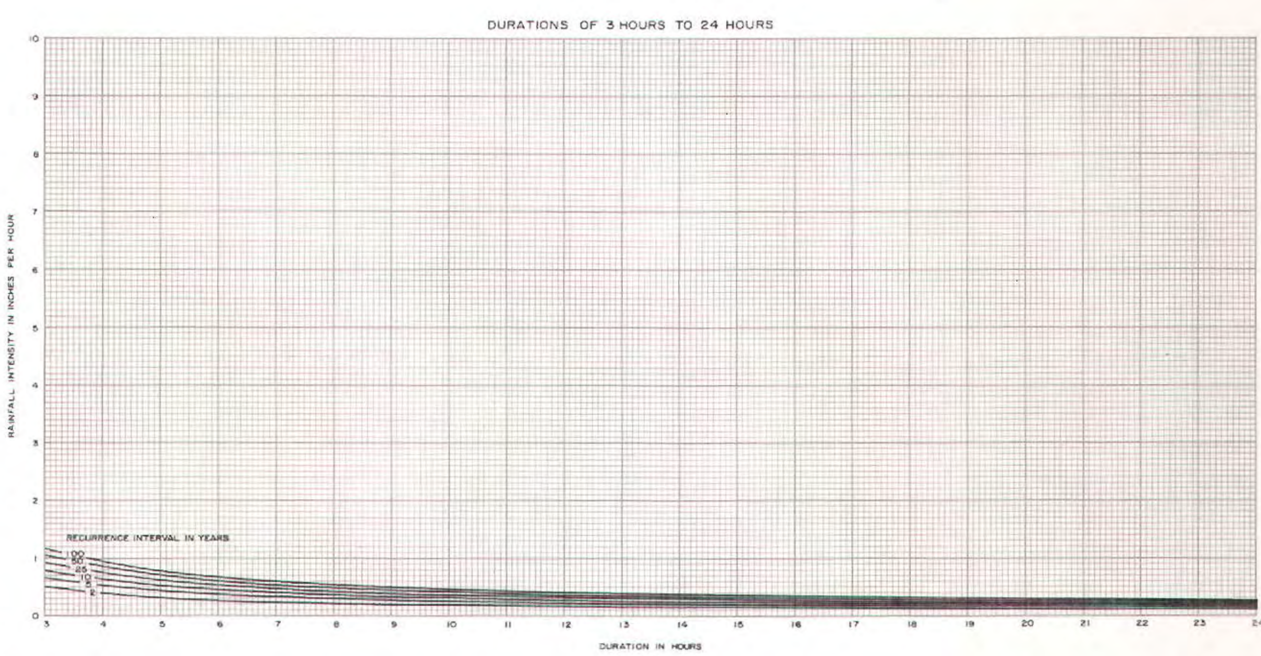
The Commission once again revised the statistical analysis of historical rainfall data under the District study, incorporating 20 more years of record. The results of this analysis are incorporated in the figures and mathematical equations herein presented. Special consideration was given to the impact of the August 6, 1986, rainfall event on the revised rainfall intensity-duration-frequency relationships, since this event resulted in a total of 6.84 inches of rain over a 24-hour period. Separate statistical analyses were completed for the period 1903 through 1985, and the period 1903 through 1986, which included the August 6, 1986, rainfall event. These statistical analyses, which determine rainfall depths for specific combinations of duration and recurrence interval, produced somewhat different results. Therefore, further analyses were conducted to determine if the rainfall data result-

[3]*For a detailed description of the rational method with emphasis on the use of soils, land use, and hydrologic data available for the seven-county Southeastern Wisconsin Planning Region, refer to "Determination of Runoff for Urban Storm Water Drainage System Design" by K.W. Bauer, SEWRPC Technical Record, Vol. 2, No. 4, April-May 1965. The procedures used to obtain equations for intensity-duration-frequency relationships are described in "Development of Equations for Intensity-Duration-Frequency Relationships" by Stuart G. Walesh, SEWRPC Technical Record, Vol. 3, No. 5, March 1973.*

Figure 8

POINT RAINFALL INTENSITY-DURATION-FREQUENCY CURVES FOR MILWAUKEE, WISCONSIN[a]





[a] The curves are based on Milwaukee rainfall data for the 84-year period of 1903 to 1986. These curves are applicable within an accuracy of ± 10 percent to the entire Southeastern Wisconsin Planning Region.

Revised 3/88

Source: SEWRPC.

Table 22

POINT RAINFALL INTENSITY-DURATION-FREQUENCY EQUATIONS FOR THE MILWAUKEE METROPOLITAN SEWERAGE DISTRICT AND THE REGION[a]

Recurrence Interval (years)	Duration of Five Minutes or More But Less than 60 Minutes[b]	Duration of 60 Minutes or More Through 24 Hours[b]
2	$i = \frac{85.1}{14.8 + t}$	$i = 26.9\ t^{-0.771}$
5	$i = \frac{118.9}{16.7 + t}$	$i = 36.4\ t^{-0.771}$
10	$i = \frac{143.0}{17.8 + t}$	$i = 43.3\ t^{-0.773}$
25	$i = \frac{172.0}{18.7 + t}$	$i = 51.0\ t^{-0.772}$
50	$i = \frac{193.4}{19.2 + t}$	$i = 56.8\ t^{-0.771}$
100	$i = \frac{214.4}{19.4 + t}$	$i = 63.0\ t^{0.773}$

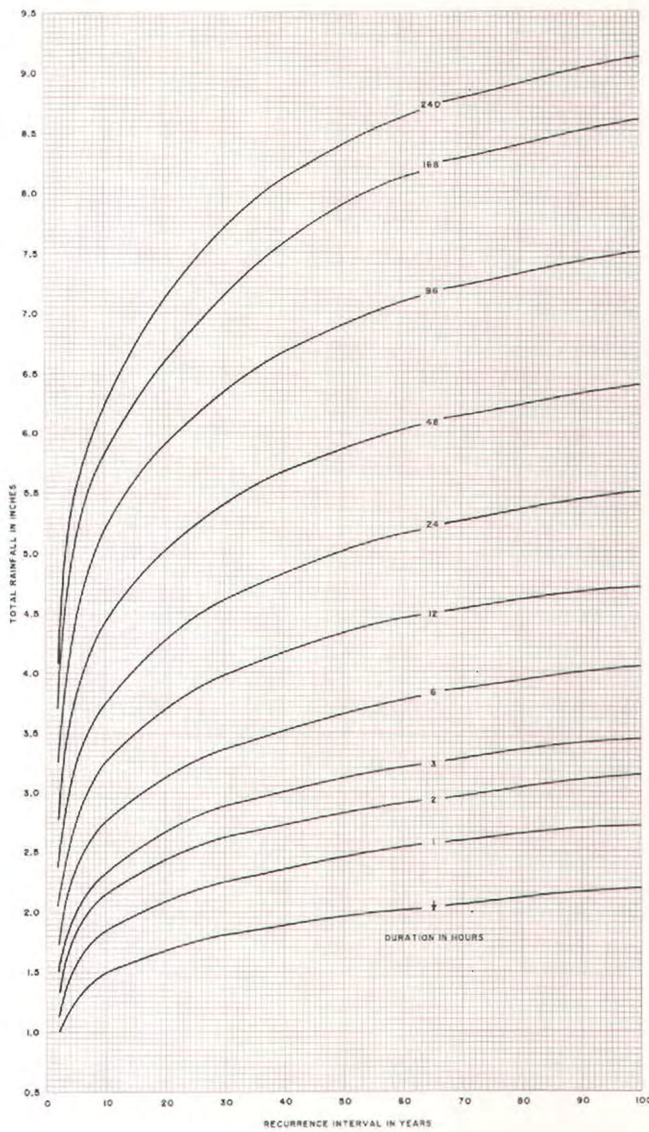
[a]The equations are based on Milwaukee rainfall data for the 84-year period from 1903 to 1986. These equations are applicable, within accuracy of ±10 percent, to the entire Southeastern Wisconsin Planning Region.

[b]i = Rainfall intensity in inches per hour
t = Duration in minutes

Source: SEWRPC.

Figure 9

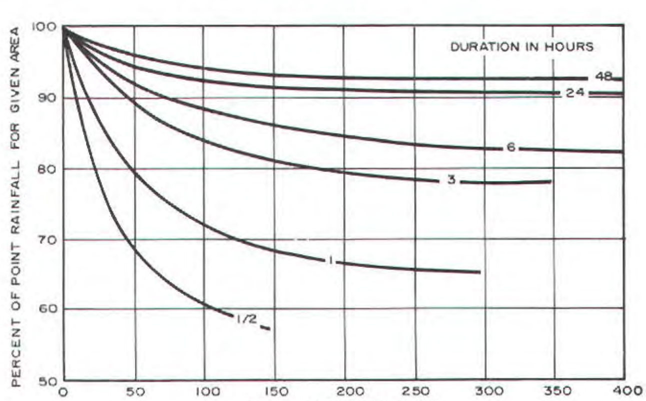
POINT RAINFALL DEPTH-DURATION-FREQUENCY RELATIONSHIPS IN THE MILWAUKEE METROPOLITAN SEWERAGE DISTRICT AND THE REGION



Source: SEWRPC.

Figure 10

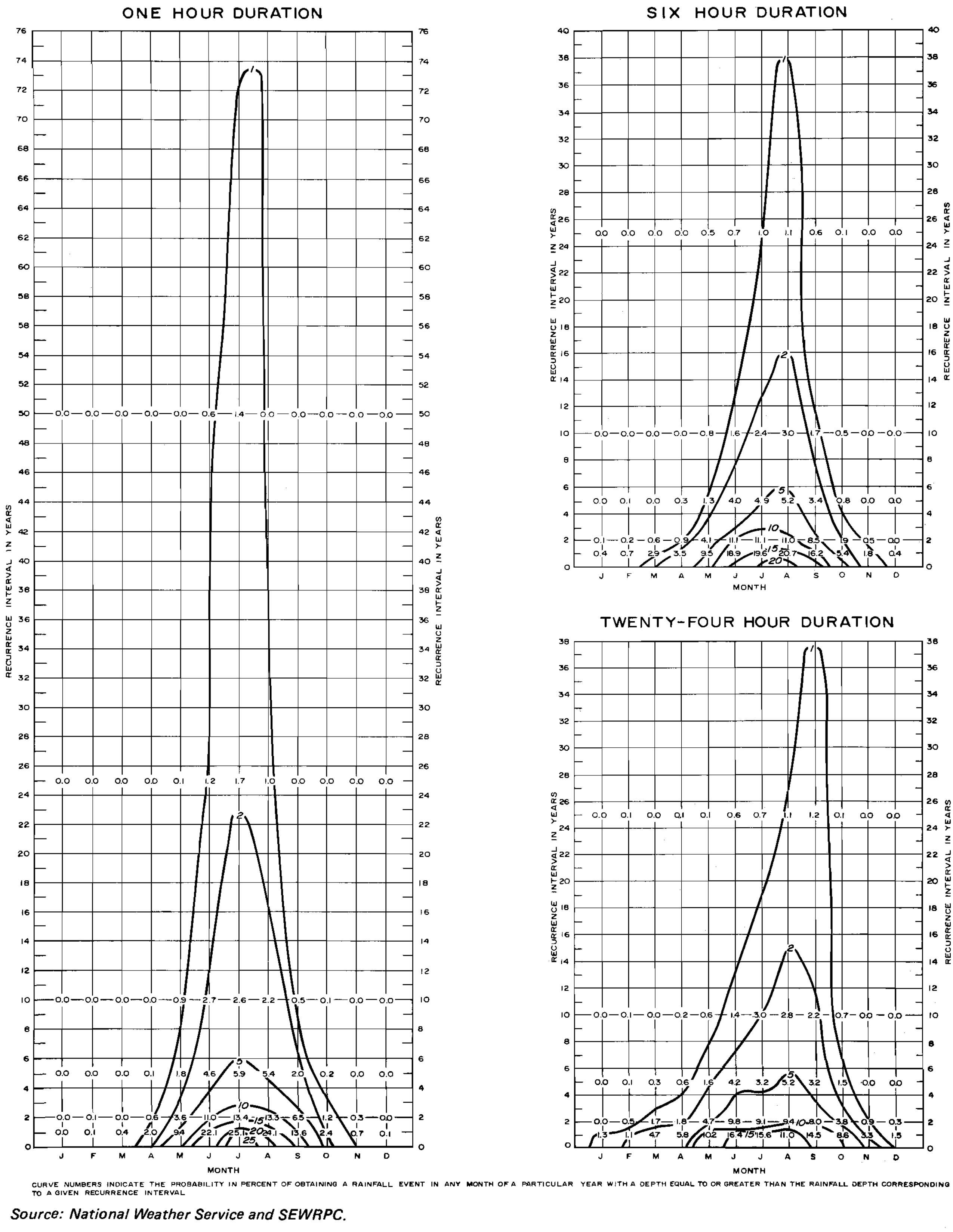
RAINFALL DEPTH-DURATION-AREA RELATIONSHIPS IN THE MILWAUKEE METROPOLITAN SEWERAGE DISTRICT AND THE REGION



Source: National Weather Service and SEWRPC.

Figure 11

SEASONAL VARIATION OF RAINFALL EVENT DEPTH IN THE MILWAUKEE METROPOLITAN SEWERAGE DISTRICT AND THE REGION

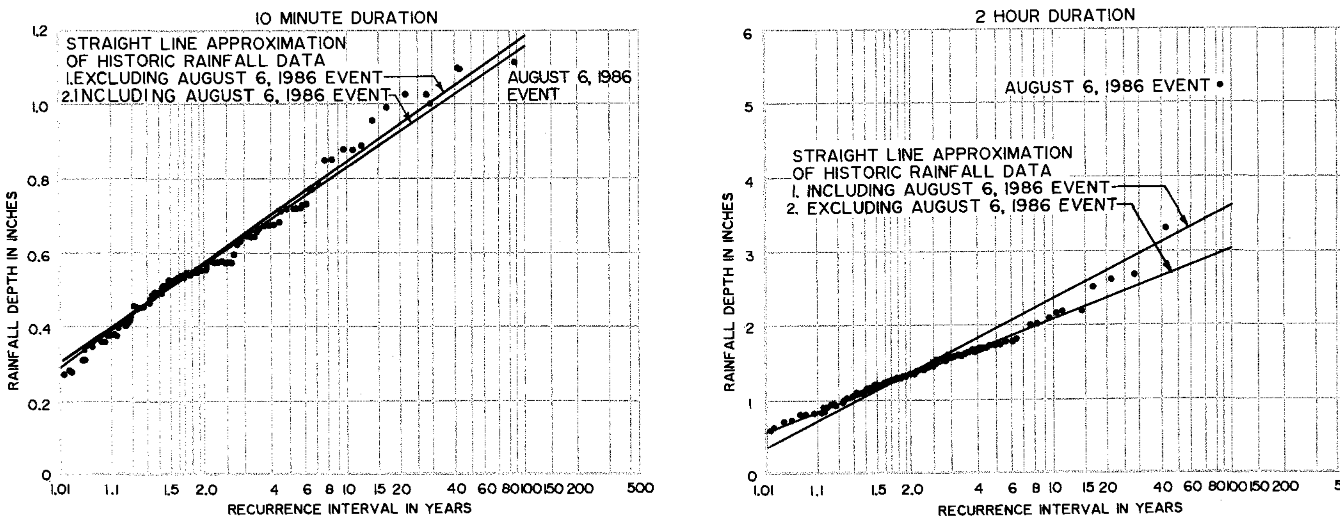


CURVE NUMBERS INDICATE THE PROBABILITY IN PERCENT OF OBTAINING A RAINFALL EVENT IN ANY MONTH OF A PARTICULAR YEAR WITH A DEPTH EQUAL TO OR GREATER THAN THE RAINFALL DEPTH CORRESPONDING TO A GIVEN RECURRENCE INTERVAL

Source: National Weather Service and SEWRPC.

Figure 12

PROBABILITY PLOTS OF HISTORIC ANNUAL PEAK RAINFALL DEPTHS FOR 10-MONTH AND TWO-HOUR DURATIONS



Source: SEWRPC.

ing from the August 6, 1986, rainfall event should be used to develop the revised rainfall intensity-duration-frequency relationships.

The historical rainfall data used to develop the rainfall intensity-duration-frequency relationships consist of annual peak rainfall depths for specified durations. A probability plot of the 84 historical peak rainfall depths for each specified duration was completed.[4] Two of these plots are set forth in Figure 12 as examples. The statistical analysis conducted produces a straight line approximation of the historical data, as shown in Figure 12, which are extrapolated to determine rainfall depths having recurrence intervals greater than the period of recorded data. As shown in Figure 12, the 10-minute rainfall depth associated with the August 6, 1986, rainfall event plots near the straight line approximation of the depths for the 83-year period from 1903 through 1985. Therefore, a straight line approximation of the 84-year period from 1903 through 1986, which includes the August 6, 1986, event, varies only slightly from that for the 83-year period. This was also true for all of the other specified durations except the two-hour duration. As shown in Figure 12, the two-hour rainfall depth associated with the August 6, 1986, rainfall event does not plot near a straight line approximation of the previous 83 annual peak rainfall depths. Therefore, a straight line approximation of the 84-year period from 1903 through 1986, which includes the August 6, 1986, event, varies significantly from that for the 83-year period for a two-hour duration, as shown in Figure 12. When performing statistical analyses such as this, the addition of a single data value should not be allowed to significantly affect the resulting frequency relationships. Therefore, the two-hour rainfall depth associated with the August 6, 1986, rainfall event was regarded as an anomaly, and not included in the final statistical analysis which produced the revised rainfall intensity-duration-frequency relationships.

[4] *The most efficient formula for computing plotting positions for unspecified distributions, and the one most commonly used for most sample data, is*

$$T = \frac{n+1}{m}$$

where: T is an estimate of the recurrence interval, n is the number of years of record, and m is the rank of the particular sample value, with the largest value equal to one.

Hydrologic and Hydraulic Analyses

Recently developed water resources planning and engineering techniques make it possible to calculate existing and probable future hydrologic and hydraulic conditions in a watershed. These techniques involve the formulation and application of mathematical models that simulate the behavior of the surface water system based upon information such as meteorological, land use, and soil data, and stream characteristics that determine the manner in which runoff from the land moves through the stream system. These models, which are usually programmed for digital computer application, permit the quantitative analyses of hydrology and hydraulics under existing and alternative future conditions as required in a sound planning effort.

Model Selection Criteria: For areawide planning purposes, the mathematical simulation model should be able to:

1. Simulate hydrologic, hydraulic, and water quality conditions in both rural and urban areas;

2. Compute flood discharges and stages for a wide range of recurrence intervals, including the 100-year recurrence interval, with sufficient accuracy for use in delineating floodland regulatory districts and areas and for designing and evaluating alternative flood control measures and works;

3. Incorporate the effects of hydraulic structures such as bridges, culverts, and dams and of localized floodland encroachments on upstream and downstream flood discharges and stages;

4. Incorporate the hydrologic and hydraulic effects of land use changes—particularly the effects of the conversion of land from rural to urban uses—not only within the floodlands, but within the entire tributary watershed; and

5. Incorporate the hydrologic and hydraulic effects of alternative structural flood control works, such as channelization, dikes and floodwalls, and storage impoundments.

Model Selection: As noted in Chapter I of this report, hydrologic and hydraulic analyses have been completed under previous Regional Planning Commission work programs for a majority of the streams for which recommended flood control systems were developed under the District study. Each of the hydrologic and hydraulic models developed under these previous work programs was reviewed under the District study. Where the base data for the models adequately reflected the existing and future land use conditions applied under the District study, the analyses made with those models were used directly. Where existing or future land use conditions differed from those on which the previous models were based, the models were revised as necessary for use in the District study.

Four different mathematical models were used in the development of flood discharges for the study area. Each of these models, which are described below, is considered to provide a proper simulation of the hydrologic performance of the particular watershed concerned. The selection of the model to be used depended upon the specific characteristics of the watersheds and upon the availability of necessary input data.

The Hydrocomp hydrologic model was used for the development of flood discharges for the Menomonee River watershed, Kinnickinnic River watershed, and most of the stream reaches studied in the Root River watershed.[5] The Hydrological Simulation Program-Fortran (HSPF), which represents a refined version of the Hydrocomp model, was used for the development of flood discharges for the Oak Creek watershed.[6] The Hydrocomp and HSPF models simulate streamflow on a continuous basis using recorded climatological data as input. Flood discharges are developed by conducting discharge-frequency analyses of the simulated annual peak discharges generated by the hydrologic model according to the log Pearson Type III method of analyses, as recommended by the U. S. Water Resources Council[7] and as specified by the Wisconsin Department of Natural Resources.[8] Flood discharges for three streams in the Root River watershed—Whitnall Park Creek, WEMP Branch, and 113th Street Branch—were developed

[5] *See Hydrocomp, Inc., Hydrocomp Simulation Programming Operations Manual, 4th Edition, January 1976.*

[6] *U. S. Environmental Protection Agency, Environmental Research Laboratory, Hydrological Simulation Program-Fortran, User's Manual for Release 8.0, Athens, Georgia, April 1984.*

[7] *United States Water Resources Council, "Guidelines for Determining Flood Flow Frequency," Bulletin No. 17 of the Hydrology Committee, Washington, D. C., March 1976.*

[8] *"Wisconsin's Floodplain Management Program," Wisconsin Administrative Code, Chapter NR 116, February 1986.*

using the model known as the Illinois Urban Drainage Area Simulator (ILLUDAS).[9] This model uses discrete rainfall patterns for the selected recurrence interval design storms. Peak flow rates are determined by applying the rainfall patterns to contributing drainage areas to produce runoff hydrographs which are combined to form instream discharges.

The "Computer Program for Project Formulation-Hydrology" (TR20) was used for the development of flood flows for Lincoln Creek in the Milwaukee River watershed.[10] This hydrologic model converts design rainfall events into various recurrence interval flood discharges by computing land surface runoff and employing channel and reservoir routing techniques to develop instream discharges.

Flood discharges for the remaining streams in the Milwaukee River watershed and for Fish Creek in the Lake Michigan direct drainage area were developed using the U. S. Army Corps of Engineers program called, "Flood Hydrograph Package."(HEC-1)[11] This hydrologic model, like the ILLUDAS and TR20 models, simulates the surface runoff response of a drainage basin to a particular precipitation event. The model represents a drainage basin as an interconnected system of hydrologic and hydraulic components, such as a surface runoff entity, a stream channel, or a reservoir. Representation of a component requires a set of parameters which specify the particular characteristics of the component and mathematical relations which describe the physical processes. The result of this modeling process is the computation of streamflow hydrographs at desired locations in the drainage basin.

[9]*Michael L. Terstriep and John B. Stall, "The Illinois Urban Drainage Area Simulator, ILLUDAS," Bulletin 58, Illinois State Water Survey, 1974.*

[10]*U. S. Department of Agriculture, Soil Conservation Service—Engineering Division, "Computer Program for Project Formulation-Hydrology," Technical Release No. 20, May 1965.*

[11]*U. S. Army Corps of Engineers, Hydrologic Engineering Center, Computer Program 723-X6-L2010, HEC-1, Flood Hydrograph Package Users Manual, Davis, California, September 1981 (revised January 1985).*

For each of the models that used a design storm, the rainfall was distributed using the median distribution for a first-quartile storm as developed by F. A. Huff.[12] That distribution is representative of heavy storms within the Region.

Flood profiles for all the streams for which recommended flood control systems were developed under the District study were developed using the U. S. Army Corps of Engineers program called "Water Surface Profiles."[13] This hydraulic model uses a computational procedure known as the "standard step method" in floodland reaches between hydraulic structures such as bridges, culverts, and dams. Given a discharge and stage at a starting floodland cross-section, a trial stage is selected for the next upstream cross-section. The Manning equation for open channel flow is used to calculate the mechanical energy loss between the two cross-sections, then a check is made to determine if the conservation of energy principle is satisfied. If not, another upstream stage is selected and tested, and the process repeated until the unique upstream stage is found at which the conservation of energy principle is satisfied. This computational process is then repeated for successive upstream floodland reaches. The result is a calculated flood stage at each of the cross-section locations. The model also determines the hydraulic effect of a bridge or culvert and the associated approach roadways by computing the upstream stage as a function of the downstream stage, flood discharge, and the physical characteristics of the hydraulic structure.

Design Flood

The design flood adopted for the District drainage and flood control plan is that event having a 100-year recurrence interval peak discharge under year 2000 planned watershed land use and floodland development conditions. This discharge was determined for key locations selected throughout the watershed stream system and was used to delineate the 100-year recurrence interval floodlands, which in turn served as the basis for develop-

[12]*F. A. Huff, "Time Distribution of Rainfall in Heavy Storms," Water Resources Research, Vol. 3, No. 4, 1967, pp. 1007-1019.*

[13]*U. S. Army Corps of Engineers, Hydrologic Engineering Center, Computer Program 723-X6-L202A, HEC-2, Water Surface Profiles Users Manual, Davis, California, September 1982.*

ment and testing of alternative plans and selection of the recommended plan. For example, the 100-year recurrence interval flood hazard line was used to define those structures included in the calculation of annual flood damages.

The selection of the design flood should be dictated by careful consideration of factors such as available hydrologic data, watershed flood characteristics, and costs attributable to flooding relative to benefits accruing from various floodplain management alternatives. In the final analysis, however, the selection of the design flood is as much a matter of public policy as it is of engineering practice and economic analysis. Sound engineering practice dictates that the flood used to delineate floodlands for land use regulation purposes have a specific recurrence interval so that the costs and benefits of alternative flood control plans can be analyzed, along with the advantages and disadvantages of various levels and combinations of police power regulations, public acquisition, and public construction for flood damage abatement and prevention. The Regional Planning Commission has selected the 100-year recurrence interval flood as the design flood for all of its watershed planning efforts for the following reasons:

1. A 100-year recurrence interval flood generally—with certain unusual exceptions—approximates, with respect to the amount of land inundated, the largest known floods that have actually occurred in the Region since its settlement by Europeans. Not all streams within the Region have experienced floods as large as the 100-year recurrence interval flood, and only one stream has experienced a flood flow which substantially exceeded a 100-year recurrence interval event. For example, the largest flood of record for the Menomonee River watershed, as recorded near the watershed outlet at Wauwatosa, is estimated to have had a recurrence interval of about 100 years. The two largest floods of record for the Milwaukee River watershed, as measured near the watershed outlet at Milwaukee, are also estimated to have had a recurrence interval of about 100 years. The largest flood of record for the Fox River watershed, as observed near the watershed outlet at Wilmot near the Wisconsin-Illinois border, is estimated to have had a recurrence interval of about 37 years. The largest flood of record for the Root River watershed, as determined in Racine at the watershed outlet, is estimated to have had a recurrence interval of about 80 years. The largest flood of record for the Pike River watershed, as determined in Kenosha at the mouth of the river, is estimated to have had a recurrence interval of about 60 years. Within the Region, only the Kinnickinnic River has actually experienced a flood event significantly larger than the 100-year recurrence interval event. The peak flow on the Kinnickinnic River at 15th Street in the City of Milwaukee during the flood of August 6, 1986, is estimated to have had a recurrence interval of greater than 500 years. On other streams in the Region, however, the highest recurrence interval flood caused by the storm of August 6, 1986, was a 40-year recurrence interval flood on the lower Menomonee River. For regulatory purposes, the use of a flood event that is similar in terms of peak flood stages and area of inundation to the most severe flood that has actually occurred on several streams within the Region provides a means by which engineers, planners, and community leaders can meaningfully relate the seriousness of the flood problem to the public, and thereby obtain an understanding of the need for floodland management.

2. The 100-year recurrence interval flood is judged to be a reasonably conservative choice when viewed in the context of the full range of possible regulatory flood events which could be used. A primary function of the regulatory flood is to define, by means of a floodplain and associated floodway, those riverine areas in which urbanization should be prohibited or strictly controlled. The regulatory flood should be at least as severe as the 10-year recurrence interval flood, since it would not be in the best interest of either the public in general or potential riverine property owners in particular to allow or encourage urban development in areas that are subject to inundation as frequently as, or more frequently than, an average of once every 10 years. This is particularly true where the flooding may endanger the health or safety of floodplain inhabitants and require that costly rescue, cleanup, and repair work be undertaken by local units of government.

   The inadequacy of the 10-year flood event as the regulatory flood thus requires selection of a more severe event, such as the recurrence interval floods of 25, 50, and 100 years. Hydrologic and hydraulic analyses completed as part of comprehensive Commission watershed studies indicate that the streams and rivers of southeastern Wisconsin generally exhibit relatively small incremental differences in stage and areas of inundation as floods increase in severity from the 10- to the 100-year event. Flood discharges in this range exceed channel capacity so that the river occupies and flows on its floodplain. Because of the large cross-sectional area of flow made available on the relatively broad floodplains characteristic of the streams of the Planning Region, large increments of additional discharge are accommodated with relatively small stage increases. Therefore, the stage of a 100-year recurrence interval flood will normally be only a few feet above the 10-year stages, although discharges of the former are usually almost twice those of the latter. The differences between the stages of a 25- or 50-year recurrence interval flood event and of a 100-year recurrence interval flood event are generally even smaller. The floodplains, moreover, are normally bounded on the outer fringes by relatively steep slopes leading to higher topography and, as a consequence of this lateral confinement, the area subject to inundation increases relatively little as floods increase in severity from the 10-year to the 100-year event.

Use of the 100-year recurrence interval flood event thus provides a greatly reduced probability of occurrence, yet entails only a relatively small incremental increase in stage, and therefore in the area subject to regulation. Thus, the 100-year event, as opposed to the 25- or 50-year event, is recommended as the basis for floodland regulation.

3. The 100-year recurrence interval flood was recommended for use by federal agencies for floodplain management purposes in 1969[14] by the U. S. Water Resources Council, an organization composed of representatives of federal offices and agencies concerned with water resource problems. This recommendation, in effect, formalizes a generally accepted practice followed by federal agencies, such as the U. S. Army Corps of Engineers and the U. S. Soil Conservation Service, of using the 100-year recurrence interval flood as the design flood for water resources planning purposes. The Regional Planning Commission use of the 100-year recurrence interval flood as the design flood results in watershed plans that have floodland management recommendations which are in accord with federal water resources planning procedures. This is particularly important with respect to any plan recommendations that require federal participation for implementation.

4. Subsequent to the Commission recommendation that the 100-year recurrence interval flood serve as the basis for floodland regulations in southeastern Wisconsin, the Wisconsin Legislature, in August 1966, enacted the State Water Resources Act. The Act authorizes and directs the Wisconsin Department of Natural Resources to carry out a statewide program leading to the adoption of reasonable and effective floodland regulations by all counties, cities, and villages. One of the requirements of the resulting state floodplain management program is that floodland regulations be based on the regional flood, which is defined by the Department as being the 100-year recurrence interval flood. Therefore, the use of the 100-year flood for land use regulatory purposes, as originally recommended by the Commission, is now mandatory within Wisconsin.

Flood Control Facility Design Criteria

Design criteria for the structural flood control measures listed in Table 21 are given below.

Storage: When properly planned and designed, floodwater storage facilities can alleviate existing flooding problems and can mitigate the impacts of flooding under future land use conditions. Typical storage facilities are shown in Figure 13. The following design criteria are to be used in the system planning for storage facilities.

1. Storage facilities should be designed to mitigate downstream flood damages for floods up to and including the 100-year recurrence interval event under planned land use and channel system conditions. The performance of each facility should be evaluated for selected recurrence interval floods ranging from the 10-year recurrence interval flood up to the 100-year recurrence interval flood. Such evaluation will ensure

[14] *U. S. Water Resources Council, Proposed Flood Hazard Evaluation Guidelines for Federal Executive Agencies, Washington, D. C., September 1969.*

that the storage facility provides adequate reduction in flood peaks throughout the range of potential floods.

2. Where practical, storage facilities should be designed to limit the design outflow to no more than the capacity of the existing downstream conveyance and storage systems.

3. Where modification to, or replacement of, the existing downstream conveyance and storage system is necessary, any proposed upland storage facilities that are required should be sized to maximize the difference between benefits and costs of the combined storage and conveyance system.

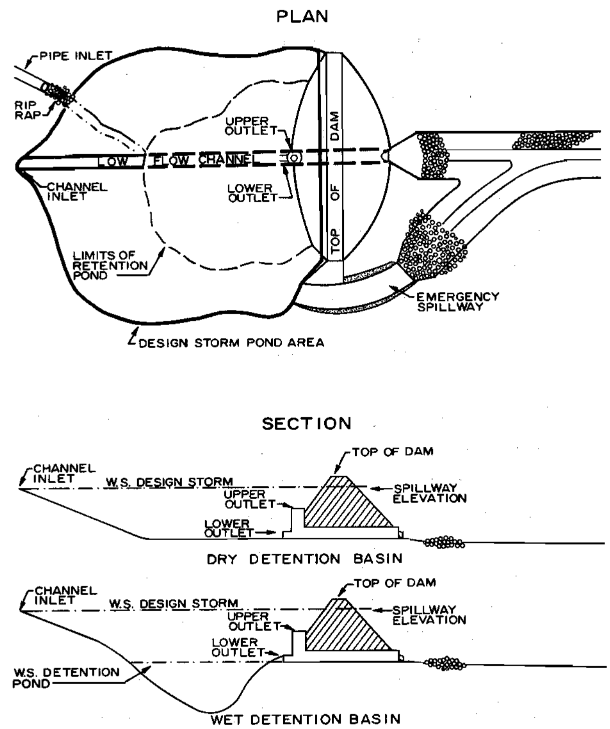
4. The effects of storage facilities on the duration and magnitude of downstream flooding under future conditions as compared to existing conditions should be carefully examined. Because of routing through a storage facility, the outflow hydrograph should be significantly flattened in comparison to the inflow hydrograph, flows should be reduced, and the duration of peak, or near-peak, flows increased. When prolongation of near-peak flows causes those flows to coincide with near-peak flows of downstream tributaries, storage facilities should be sized so as not to increase combined future downstream flood peaks above the existing peaks. In cases where the increased duration of downstream flooding creates an unacceptable level of damages, even though the flood peak was limited to the existing condition peak, the storage facility should be sized to reduce the peak outflow and the duration of downstream flooding to more acceptable levels. In some instances, the peak storage facility outflow may need to be reduced to an amount less than the existing subbasin outflow in order to reduce the effects of a prolonged peak.

5. To the extent practical, storage facilities should be designed to prevent downstream bank erosion during frequent storm events.

6. Storage provided through the use of dry detention basins minimizes maintenance. Accordingly, wet detention basins should be used only on a site-specific basis when warranted for recreational, aesthetic, water quality, or water supply purposes.

Figure 13

TYPICAL STORMWATER DETENTION STORAGE STRUCTURES



Source: SEWRPC.

7. Where the use of wet detention basins is warranted for water quality purposes according to criterion 6, consideration should be given to selecting a pond area-outflow rate relationship that will increase particle settling efficiencies without compromising the flood control objectives for the basin.

8. To avoid short circuiting of flow and to maximize the detention efficiency of wet detention basins, the basin length-to-width ratio should approximate five, or baffles should be provided to increase the flow length.

9. The average depths of wet detention basins should range between three and eight feet. A three-foot minimum depth is needed to minimize scour and resuspension of deposited sediments, and an eight-foot maximum depth will aid in reducing aquatic plant growth and will increase winter survival of fish.

10. Storage depths on parking lots, truck stopping areas, and similar open spaces should not exceed six inches during the design flood event.

11. Storage facilities that include dams or earth embankments to detain floodwaters should include an emergency spillway to pass flows in excess of the 100-year design flood, and should satisfy the applicable criteria of Chapter NR 333 of the Wisconsin Administrative Code.

Diversion: Peak flows, flood levels, and damages in a flood-prone reach can be reduced by diverting flow from the reach to a downstream point on the same watercourse, to another watercourse in the same watershed as the flood-prone reach, to a watercourse in another watershed, or to Lake Michigan outside the watershed. A typical diversion facility is shown in Figure 14. The following design criteria are to be used in the system planning for a diversion and the component parts of a diversion, which include the control structure, the diversion channel and/or conduit, and the outlet.

1. Diversion facilities should be designed to mitigate downstream flood damages for floods up to and including the 100-year recurrence interval event under planned land use and channel conditions.

2. The diverted flow should not cause flooding of unprotected lands adjacent to the diversion facilities except where topographic or land use conditions could accommodate the increased stage without creating additional flood damage potential.

3. Open channels that are part of a diversion system should be designed according to the applicable criteria listed in the "Channel Modification and Enclosure" subsection of this chapter.

4. Culverts or conduits that are part of a diversion system should be designed according to the applicable criteria listed in the "Bridge and Culvert Alteration or Replacement" subsection of this chapter.

5. If practical, the outlet of the channel or conduit should be aligned to permit a smooth transition for flow into the receiving watercourse.

6. Appropriate energy dissipation and/or erosion protection should be provided at the control structure, at any grade control structures, at the diversion channel outlet, along the diversion channel sides and bottom, and at conduits. The type of protection will be dictated by site-specific hydraulic considerations.

7. Interior drainage facilities should be provided where diversion channel dikes are constructed.

Dikes and Floodwalls: Where the floodplain topography is flat and there is considerable damage-prone development in the floodplain, dikes and floodwalls can be used to provide flood control. Typical dike and floodwall cross-sections are shown in Figure 15. The following design criteria are to be used in the system planning for dikes and floodwalls.

1. Dikes and floodwalls should be designed to mitigate flood damages for floods up to and including the 100-year recurrence interval event under planned land use and channel conditions.

2. The upstream and downstream effect of dikes and floodwalls on flood discharges and stages shall be determined, and any such structural works which may significantly increase upstream or downstream peak flood discharges should be used only in conjunction with complementary facilities for the storage and movement of the incremental floodwaters through the watershed stream system.

3. Dikes or floodwalls shall not increase the height of the 100-year recurrence interval flood by 0.01 foot or more in any unprotected upstream or downstream reaches. Increases in flood stages that are equal to or greater than 0.01 foot resulting from any dike or floodwall construction shall be contained within the upstream or downstream extent of the dike or floodwall, except where topographic or land use conditions could accommodate the increased stage without creating additional flood damage potential.

4. In cases where a dike or floodwall is intended to protect human life, the minimum dike or

Figure 14

TYPICAL DIVERSION CHANNEL AND APPURTENANT STRUCTURES

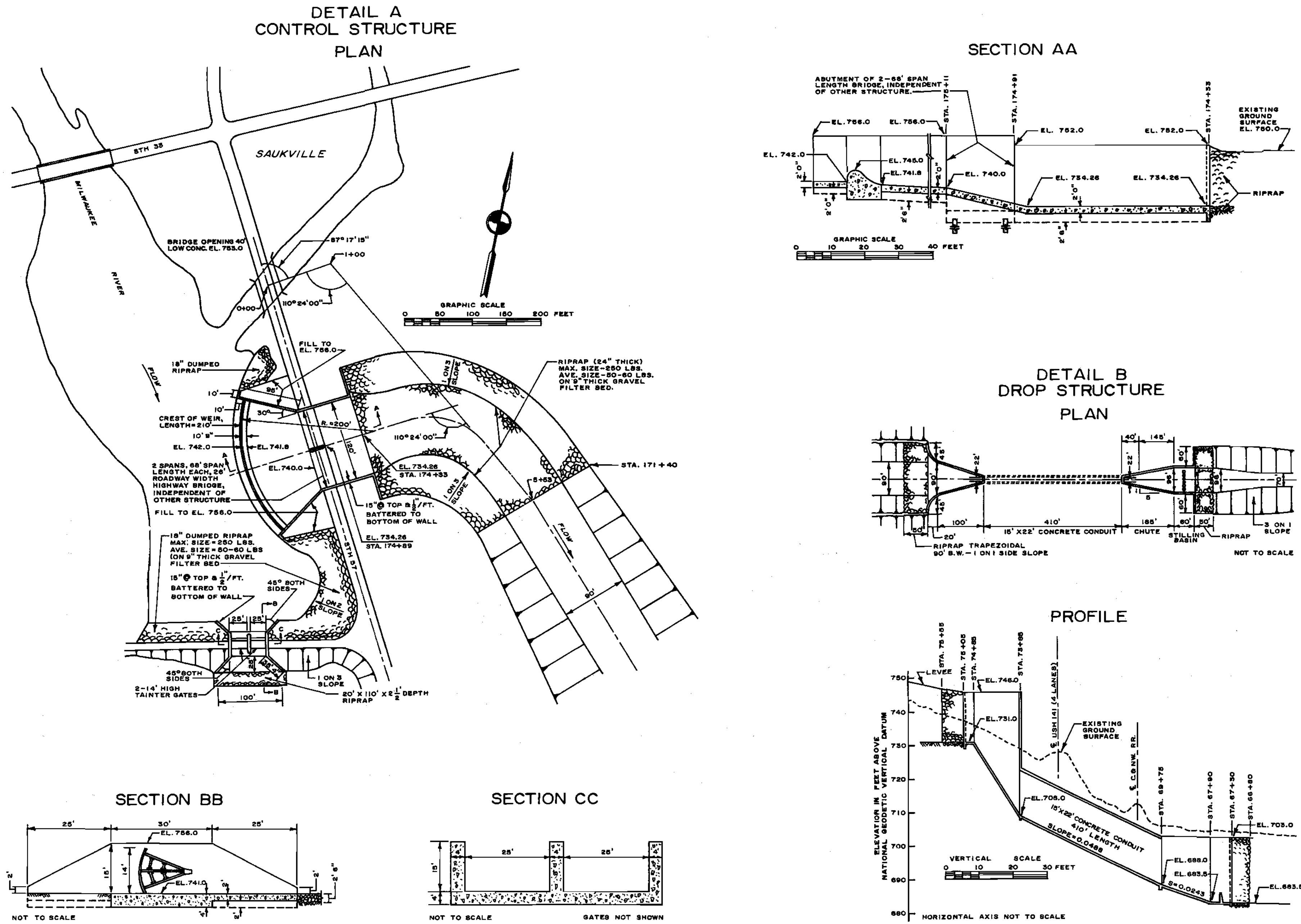


Figure 14 (continued)

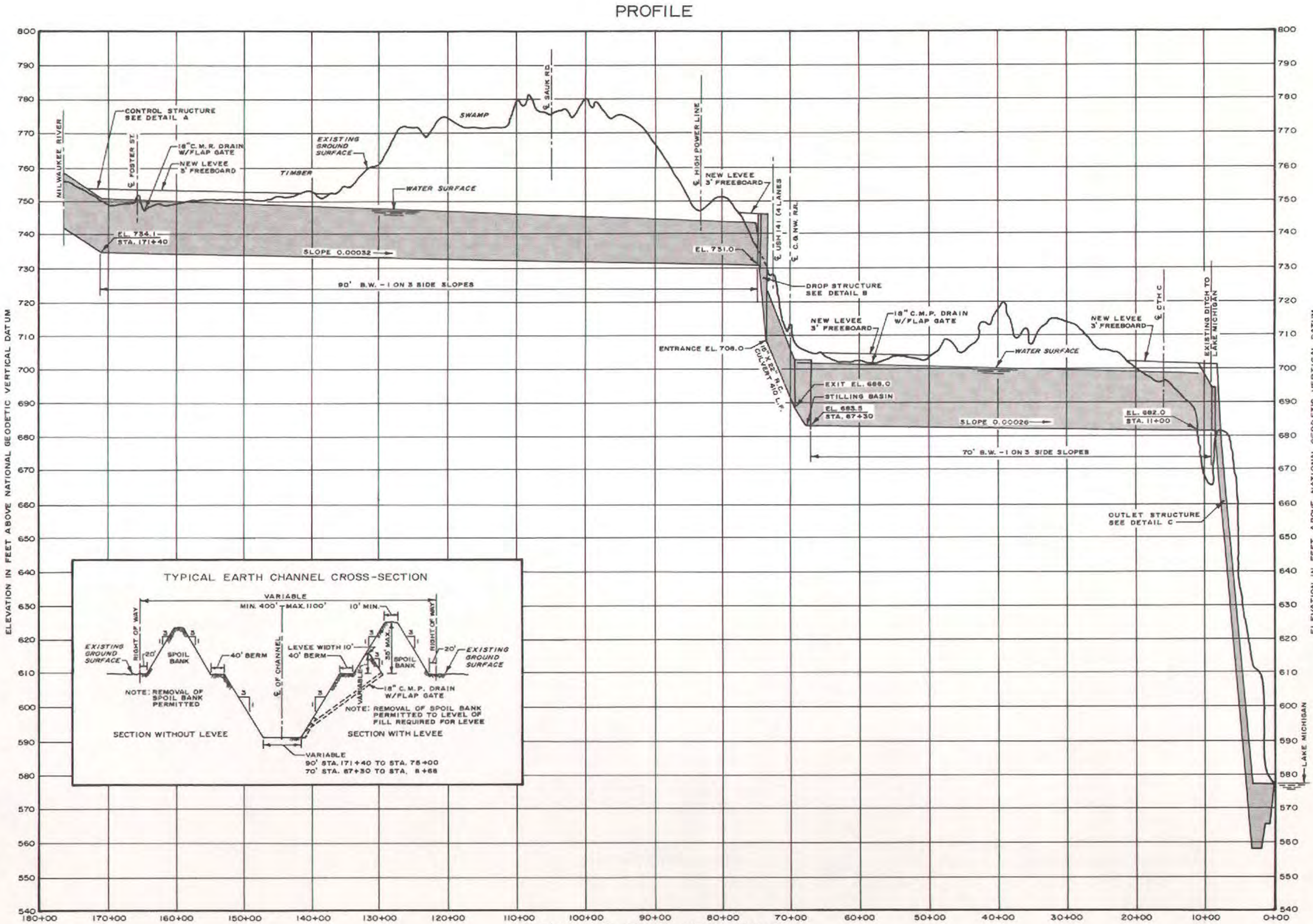
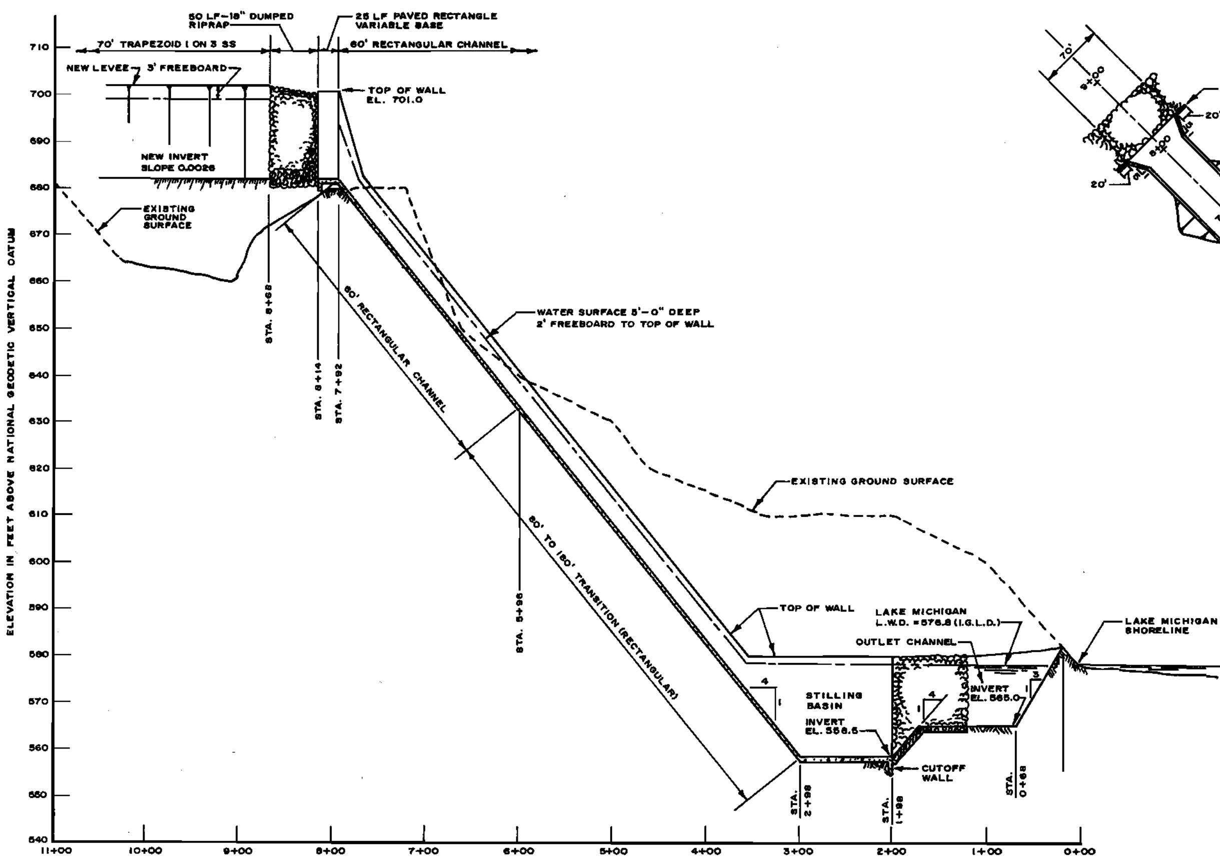


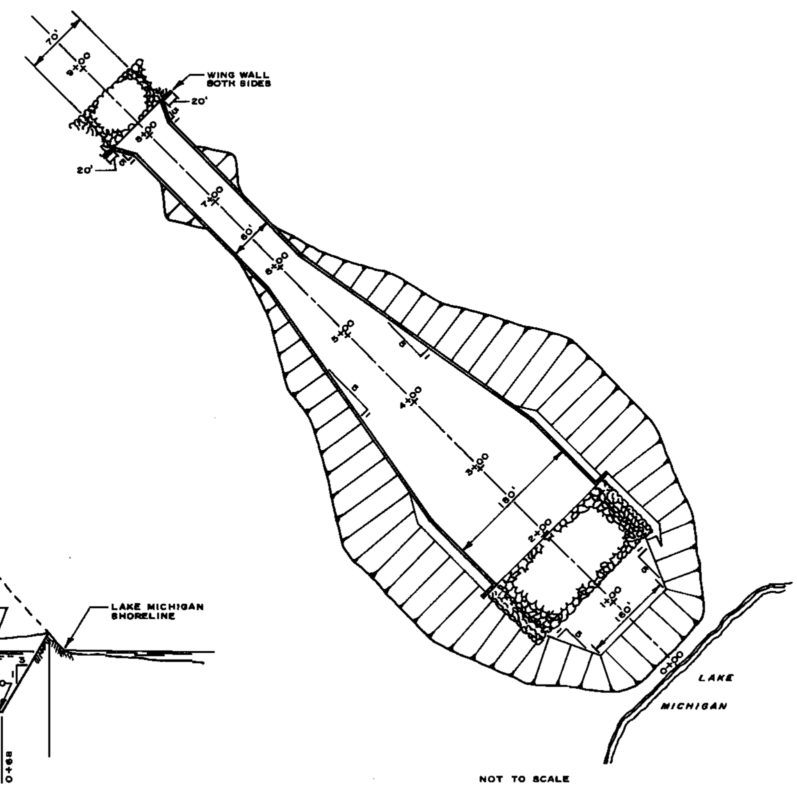
Figure 14 (continued)

DETAIL C
OUTLET STRUCTURE

PROFILE



PLAN VIEW



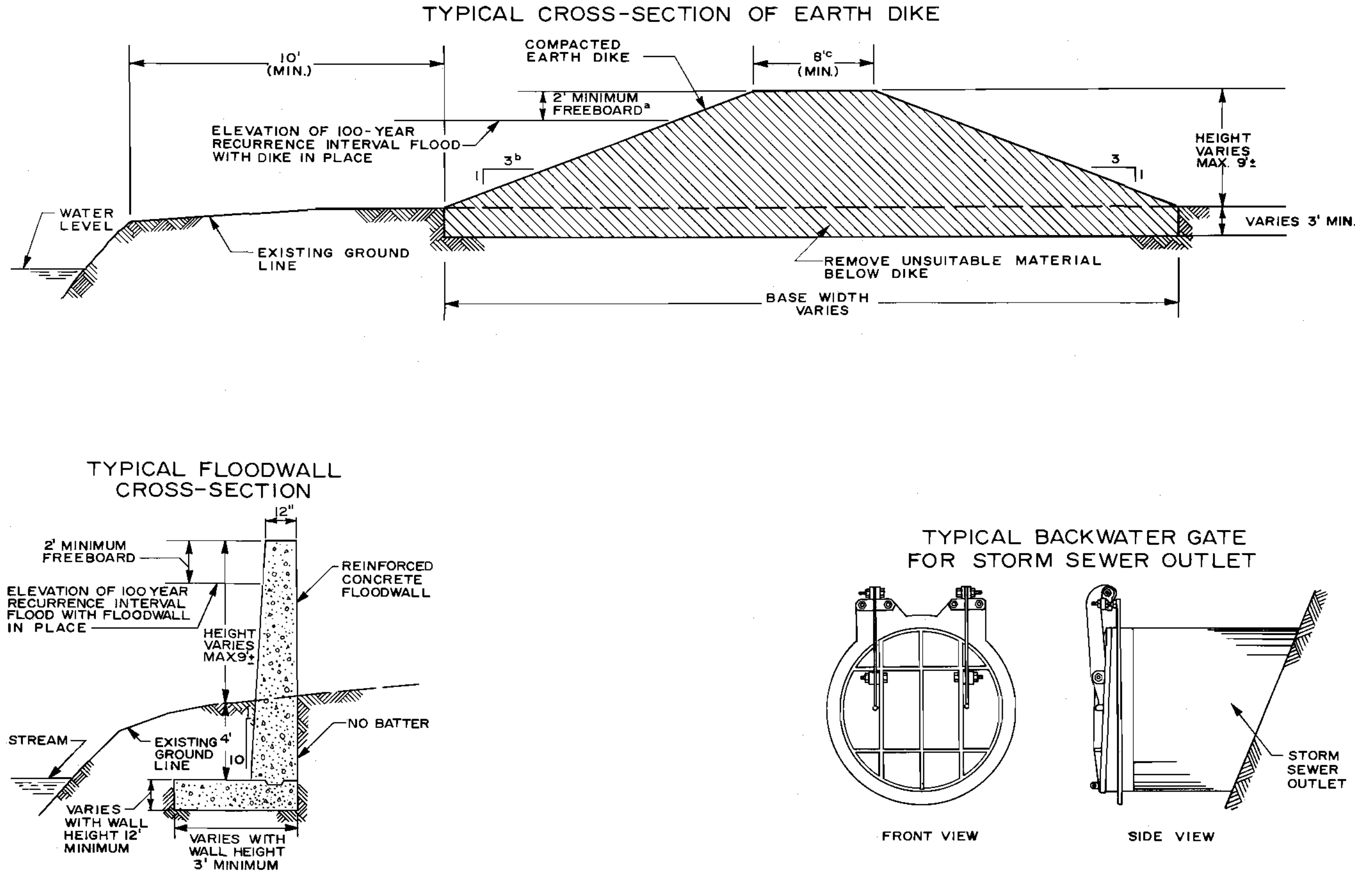
NOTE: (1) THIS DIVERSION SYSTEM PLAN IS REPRODUCED, WITH THE DELETION OF SOME MINOR DETAILS, FROM PLATES 4, 5, AND 6 AS PUBLISHED IN SURVEY REPORT FOR FLOOD CONTROL–MILWAUKEE RIVER AND ITS TRIBUTARIES, WISCONSIN, U.S. ARMY ENGINEER DISTRICT–CHICAGO, NOVEMBER, 1964.

(2) ALL ELEVATIONS ARE REFERED TO NATIONAL GEODETIC VERTICAL DATUM, 1929 ADJUSTMENT.

Source: U.S. Army Corps of Engineers and SEWRPC.

Figure 15

CONTAINMENT FACILITIES: TYPICAL EARTH DIKE, CONCRETE FLOODWALL, AND BACKWATER GATE



[a]*A two-foot minimum freeboard is permissible where dikes or floodwalls are not intended to protect human life. Otherwise a three-foot minimum freeboard is required.*

[b]*Dike side slopes of one vertical on three horizontal should be used wherever feasible. The minimum slope should be no steeper than one vertical on two and one-half horizontal.*

[c]*A six-foot top width should be used for heights of six feet or less. An eight-foot width should be used for higher dikes.*

Source: Water Resources Research Institute and SEWRPC.

floodwall top elevation shall be determined using whichever of the following produces the highest profile:

a. The 100-year flood profile plus three feet of freeboard, or

b. The 500-year flood profile.

The height of low dikes or floodwalls that are not intended to protect human life shall be based on the high water surface profiles for the 100-year recurrence interval flood prepared under the drainage and flood control plan, and shall be capable of passing the 100-year recurrence interval flood with a freeboard of at least two feet.

5. Where practical, dike slopes should be one vertical on three horizontal. Slopes should be no steeper than one vertical on two and one-half horizontal.

6. For dikes with heights of six feet or less, the minimum top width should be six feet. For dikes with heights greater than six feet, the minimum top width should be eight feet.

Channel Modification and Enclosure: The following design criteria for channel modification and enclosure are to be applied to this plan:

1. Channel modifications and enclosures should be designed to accommodate flood flows up to and including the 100-year recurrence interval event under planned land use and channel conditions.

2. Features to mitigate adverse impacts on fish and wildlife habitat should be considered in the design of channel modifications and enclosures.[15]

3. The upstream and downstream effect of channel modifications on flood discharges and stages shall be determined, and any such structural works that may significantly increase upstream or downstream peak flood discharges should be used only in conjunction with complementary facilities for the storage and movement of the incremental floodwaters through the watershed stream system.

4. Channel modifications shall not increase the height of the 100-year recurrence interval flood by 0.01 foot or more in any unprotected upstream or downstream stream reaches. Increases in flood stages that are equal to or greater than 0.01 foot resulting from any channel construction shall be contained within the upstream or downstream extent of the channel, except where topographic or land use conditions could accommodate the increased stage without creating additional flood damage potential.

5. Alternative cross-sections for modified channels using turf or rip-rap are shown in Figure 16. Selected design criteria for the various channel types are summarized in Figure 16 and Table 23.

   a. Turf-lined, or Type A, channels should be used wherever practicable. Where there is adequate right-of-way, such channels should have maximum side slopes of one vertical on four horizontal. In no instance should the side slopes be steeper than one vertical on two horizontal. A Manning's "n" value of 0.030 should be used and the maximum velocity during the 100-year recurrence interval flood should not exceed six feet per second. A maintenance access road should be located along the top of the bank, or along a 12-foot-wide maintenance bench as shown in Figure 16. Where wetland vegetation bottom channels are deemed important for environmental reasons, as discussed below, base flow should be conveyed in either a trickle channel or a low-flow channel.

   b. Rip-rap-lined, or Type B, channels should be provided if erosive velocities are to be expected in turf-lined channels. A typical channel section for this situation is shown as Type B in Figure 16. Where feasible, riprap-lined channel side slopes should be one vertical on three horizontal, but they should not be steeper than one vertical on two horizontal. A Manning's "n" value of 0.035 should be used and the maximum velocity should be no more than 10 feet per second.

6. Where right-of-way restrictions or hydraulic considerations prevent the use of turf-lined channels, fully or partially lined concrete channels may be used, as shown in Figure 16, Types C through F. A Manning's "n" value of 0.015 should be used for concrete channels. Composite turf- and concrete-lined channels should be designed using the appropriate "n" for each segment of the channel cross-section.

   a. Partially turf-lined, or Type C, channels with a concrete invert may be used in residential areas. Where practical, the turf-lined side slopes should be one vertical on four horizontal, but in no instance should they be steeper than one vertical on two horizontal. During the 100-year recurrence interval flood, the maximum velocity should be six feet per second.

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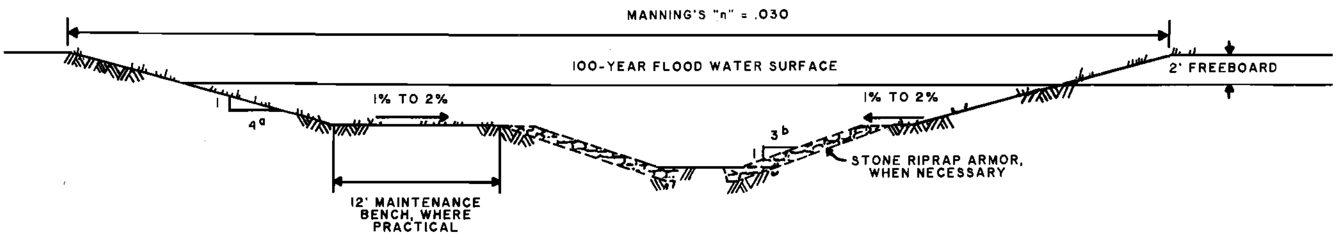
[15] *A work group to develop mitigative procedures for channelized streams in Wisconsin has been formed by the Wisconsin Department of Natural Resources and includes representatives of the Milwaukee Metropolitan Sewerage District, the Southeastern Wisconsin Regional Planning Commission, local public works officials, the U. S. Army Corps of Engineers, the U. S. Fish and Wildlife Service, the University of Wisconsin, the Wisconsin Society of Civil Engineers, and the Wisconsin Society of Professional Engineers.*

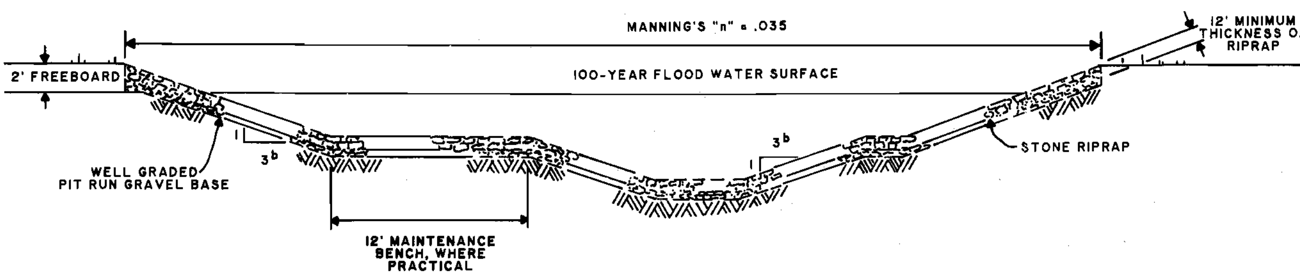
Figure 16

TYPICAL MODIFIED CHANNEL CROSS-SECTIONS

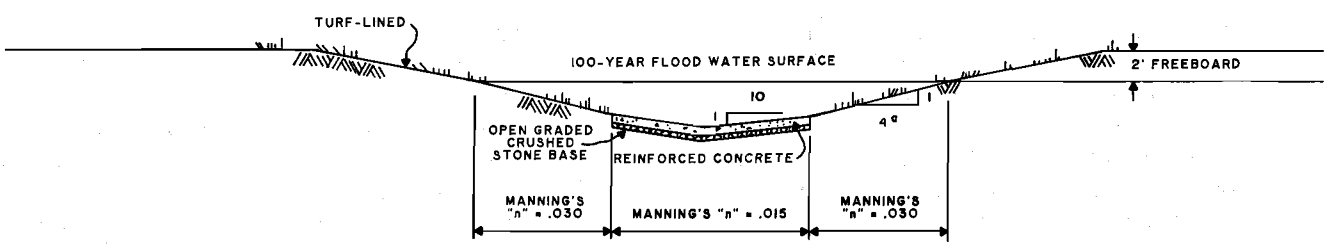
TYPE A
TURF-LINED CHANNEL



TYPE B
RIPRAP-LINED CHANNEL



TYPE C
PARTIALLY TURF-LINED WITH CONCRETE INVERT CHANNEL



TYPE D
PARTIALLY CONCRETE-LINED CHANNEL

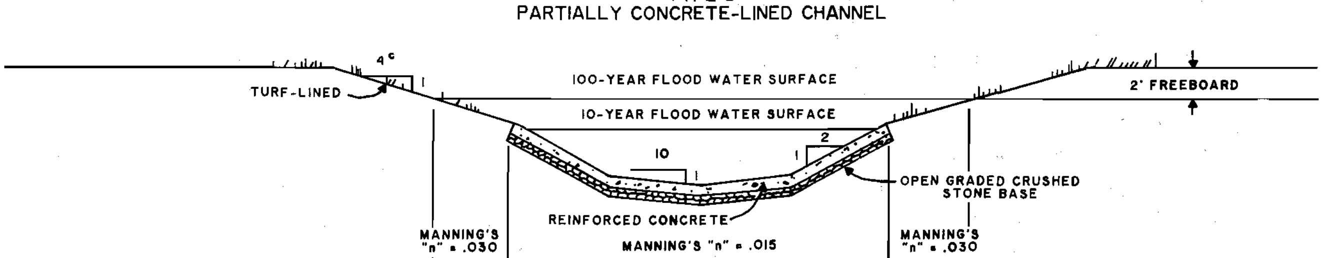
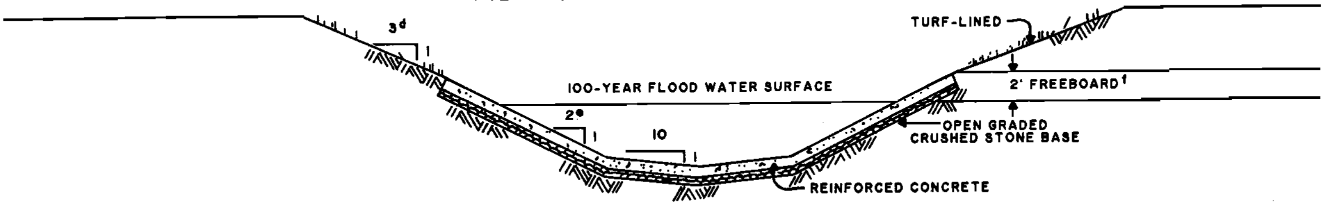
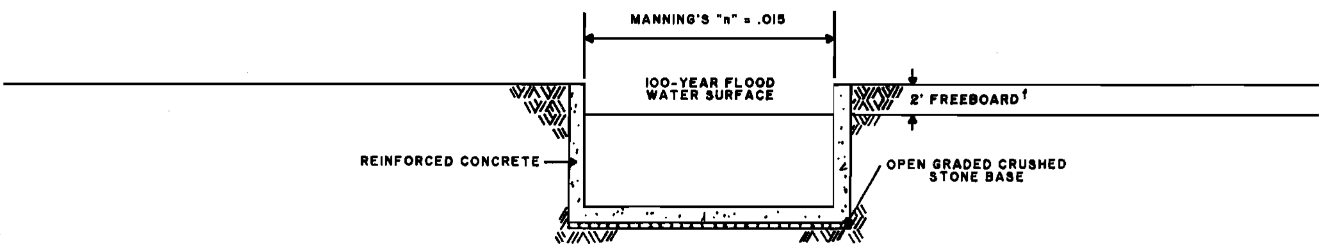


Figure 16 (continued)

TYPE E
FULLY CONCRETE-LINED TRAPEZOIDAL CHANNEL



TYPE F
CONCRETE-LINED RECTANGULAR CHANNEL



[a]*Desirable side slope is one vertical on four horizontal. Steepest allowable side slope is one vertical on two horizontal.*

[b]*Desirable side slope is one vertical on three horizontal. Steepest allowable side slope is one vertical on two horizontal.*

[c]*Desirable side slope is one vertical on four horizontal. Steepest allowable side slope is one vertical on two and one-half horizontal.*

[d]*Desirable side slope is one vertical on three horizontal. Steepest allowable side slope is one vertical on two horizontal.*

[e]*Desirable side slope is one vertical on two horizontal. Steepest allowable side slope is one vertical on one horizontal.*

[f]*A freeboard of two feet is desirable. The minimum permissible freeboard is one foot.*

Source: SEWRPC.

b. Partially concrete-lined, or Type D, channels may be used in residential areas and in some industrial and commercial areas where there are right-of-way limitations. The slope of the concrete-lined portions should be no steeper than one vertical on two horizontal. Turf-lined slopes should be one vertical on four horizontal if practicable, but should be no steeper than one vertical on two and one-half horizontal. The 10-year recurrence interval flood should be conveyed within the concrete channel. The maximum velocity should be nine feet per second for the 10-year recurrence interval flood and 11 feet per second for the 100-year recurrence interval flood.

c. Fully concrete-lined, or Type E, trapezoidal channels may be used in industrial and commercial areas with restricted right-of-way. This type of channel is designed to carry the 100-year recurrence interval flood flow within the concrete channel. It is desirable to have two feet of freeboard to the top of the concrete, but a minimum of one foot is permissible. The slope of the concrete-lined portions can range from one vertical on two horizontal to one vertical on one horizontal. It is desirable for turf-lined side slopes to be one vertical on three horizontal, but slopes of one vertical on two horizontal are permissible where right-of-way is restricted. The maximum allowable average velocity during the 100-year recurrence interval flood is 12 feet per second.

d. Concrete-lined rectangular, or Type F, channels may be used in commercial and industrial areas with highly restricted right-of-way. The freeboard requirements are the same as for Type E channels. The maximum velocity during the 100-year recurrence interval flood should not exceed 12 feet per second.

Table 23

CHANNEL MODIFICATION DESIGN CRITERIA

Modification Type	Turf- or Riprap-Lined Side Slopes	Concrete-Lined Side Slopes	Maximum Allowable Velocity (feet/sec)
A	1V:2H to 1V:4H	--	6
B	1V:2H to 1V:3H	--	10
C	1V:2H to 1V:4H	--[a]	6
D	1V:2.5H to 1V:4H	1V:2H	9[b], 11[c]
E	1V:2H to 1V:3H	1V:1H to 1V:2H	12
F	--	Vertical	12

[a]*Only the channel bottom is concrete.*

[b]*For the 10-year recurrence interval flood.*

[c]*For the 100-year recurrence interval flood.*

Source: SEWRPC.

7. The Manning's "n" value criteria for modified channels may be adjusted somewhat in cases where site-specific conditions, such as anticipated vegetative growth and frequency of maintenance, dictate such adjustment.

8. The maximum allowable velocities for modified channels may be increased in localized reaches where site-specific conditions create higher velocities. Adequate erosion protection should be provided in those reaches.

9. Where practicable, grade control structures should be provided as necessary to reduce the channel gradient and obtain flow velocities within the accepted limits. Channel bottom drop structures should not be used in streams with existing or potential valuable fisheries.

10. Where feasible, modified channels should have a two-foot freeboard above the design flood elevation.

11. At channel bends, the freeboard should be referenced to the super-elevated water surface elevation.

12. Channel bends should have a minimum radius equal to twice the design flow top width, or of 100 feet, whichever is greater.

13. Culverts or conduits that are part of a channel enclosure project should be designed according to the applicable criteria listed in the "Bridge and Culvert Alteration or Replacement" subsection of this chapter.

14. Appropriate energy dissipation and erosion protection should be provided at any grade control structures and at conduit outlets. The type of protection will be dictated by site-specific hydraulic considerations.

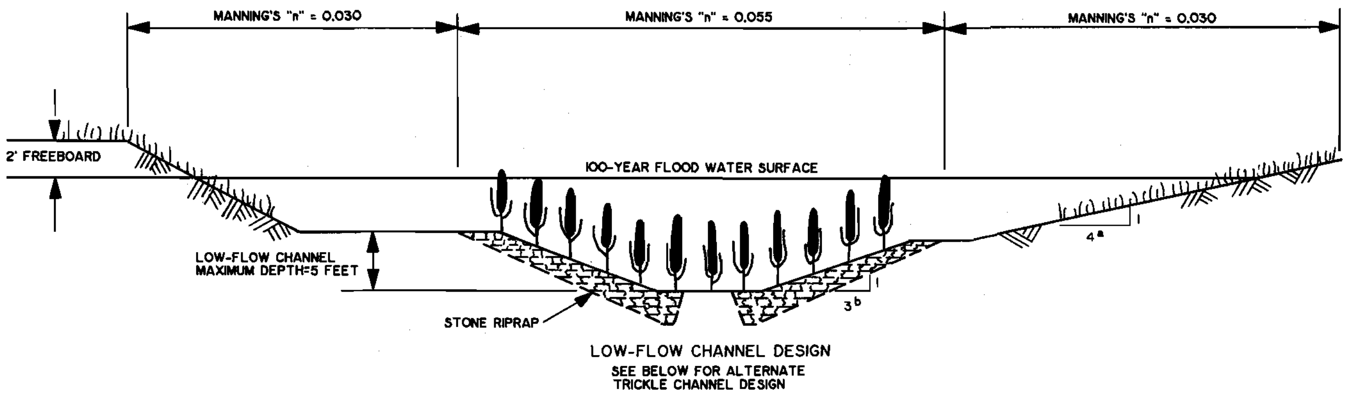
15. Where modified channels are to be located in an existing wetland area, turf-lined channels should be constructed with wetland vegetation in the bottoms, as shown in Figure 17. The choice between using a trickle channel or using a low flow channel should be based on the overall size of the modified channel relative to the size of the trickle or low-flow channel required to pass the design flow. The trickle channel or the low-flow channel should have a design capacity of about 3 percent of the 100-year recurrence interval flood. Trickle channel depths should be about 1.5 feet. The maximum low-flow channel depth should be about five feet. These design flow and depth criteria are intended to be used as guidelines that may be adjusted to meet site-specific conditions. The design criteria for turf-lined channels apply to wetland bottom channels with the following modifications and additions.

    a. The longitudinal channel slope and the initial channel shape, which are selected to meet the design velocity criteria, should be determined using Manning's "n" values characteristic of newly constructed channels. The Manning's "n" values given above for turf- and riprap-lined channels should be used.

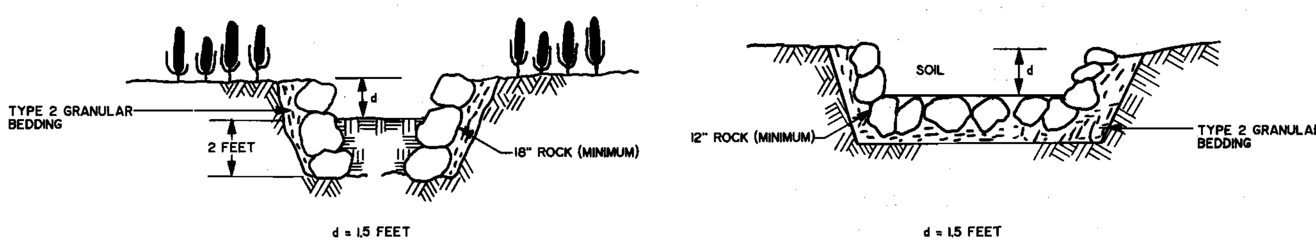
    b. The design water surface profile should be determined using Manning's "n" values characteristic of mature wetland channels. Wetland channels with trickle channels should be designed using Manning's "n" values determined from Figure 18. The turf-lined portions of wetland channels with low-flow channels should be designed using a Manning's "n" value of 0.030, and the low-flow channels should be designed using a Manning's "n" value of 0.055.

Figure 17

WETLAND BOTTOM CHANNEL CROSS-SECTIONS



TRICKLE CHANNELS FOR WETLAND VEGETATION BOTTOMS



[a]*Desirable side slope is one vertical on four horizontal. Steepest allowable side slope is one vertical on two horizontal.*

[b]*Desirable side slope is one vertical on three horizontal. Steepest allowable side slope is one vertical on two horizontal.*

Source: Denver Urban Drainage and Flood Control District, and SEWRPC.

Bridge and Culvert Alteration or Replacement: Design criteria for bridge and culvert alteration or replacement are listed below:

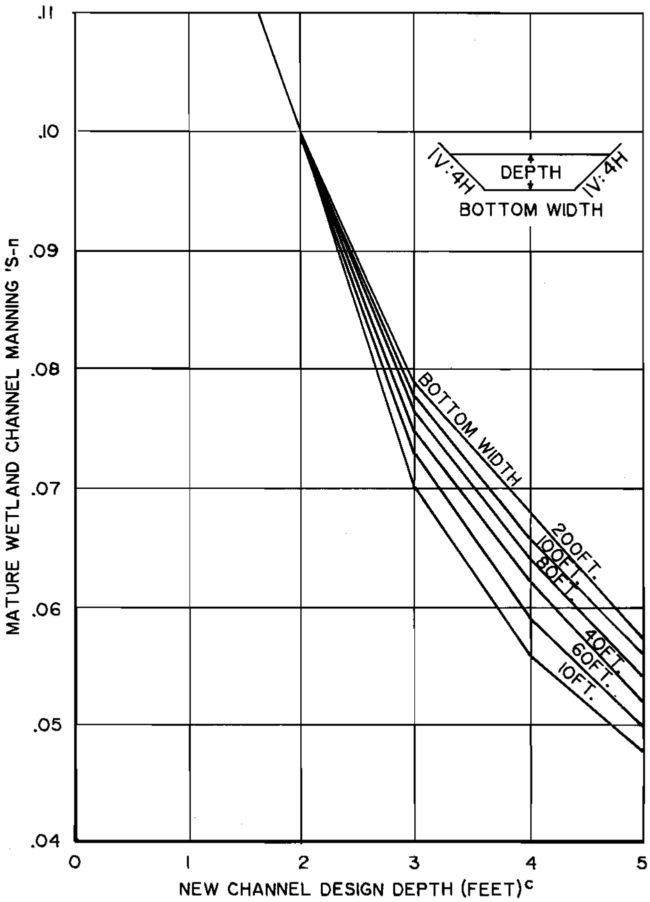
1. For reaches having topographic or land use conditions that could accommodate stage increases greater than 0.01 foot without creating additional flood damage potential upstream of the proposed structure, and having substantial floodplain storage volume for reducing flood peaks, consideration should be given to maintaining undersized bridge or culvert waterway openings or to actually decreasing the waterway opening in order to decrease downstream flood flows and stages.

2. Except at structures where blockage of the waterway opening is identified as an historical problem, backwater computations should be made assuming proper waterway opening design and maintenance so that the full waterway opening of each proposed or existing bridge or culvert is available for the conveyance of flood flow.

3. At existing structures where significant blockage of the waterway opening has consistently occurred during past floods, the backwater computations for determination of the design flood profile under existing conditions should be made assuming a degree of blockage of the opening commensurate with available historical observations.

Figure 18

MANNING'S "n" FOR WETLAND BOTTOM CHANNELS WITH TRICKLE CHANNELS[a,b]



[a]FOR DESIGN, USE MANNING'S n= 0.030 FOR A NEW (IMMATURE) CHANNEL TO SET THE CHANNEL'S LONGITUDINAL SLOPE. USING THIS LONGITUDINAL SLOPE, ADJUST THE DEPTH OR WIDTH FOR THE WETLAND MANNING'S n IN THIS CHART.

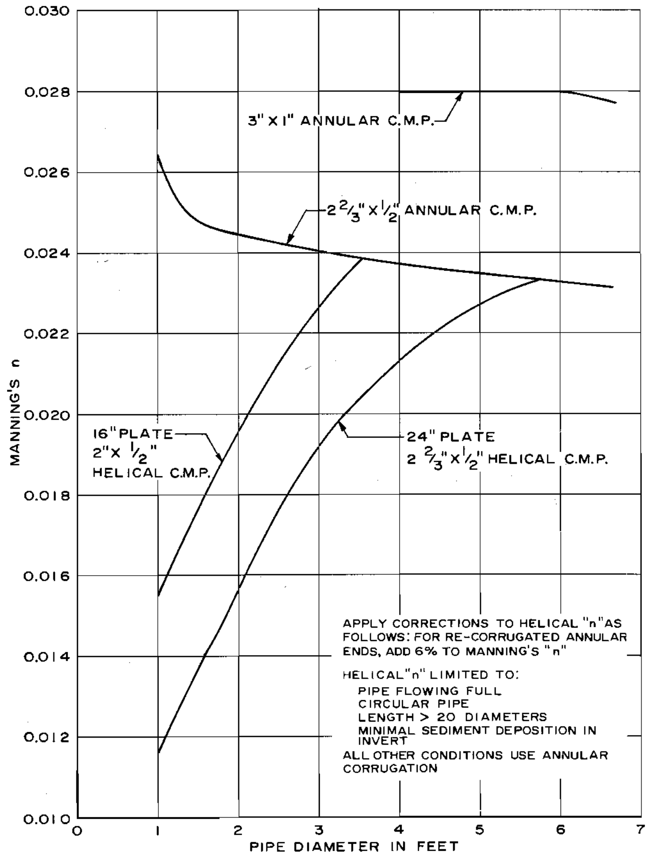
[b]FOR CHANNEL DESIGN DEPTH GREATER THAN FIVE FEET, USE THE DEPTH OF FIVE FEET IN THE ABOVE CHART.

[c]DEPTH OF CHANNEL BEFORE WETLAND VEGETATION MATURES.

Source: SEWRPC.

Figure 19

MANNING'S "n" VERSUS DIAMETER FOR CORRUGATED METAL PIPE CULVERTS FLOWING FULL



Source: U.S. Department of Transportation, Hydraulic Flow Resistance Factors for Corrugated Metal Conduit, and SEWRPC.

4. Manning's "n" values as shown in Figure 19 should be used for properly installed and maintained corrugated metal pipe and pipe arch culverts.

5. A Manning's "n" value of 0.012 should be used for well-constructed concrete pipe flowing full.

6. Where analyses indicate that pipes would flow less than full at design loading, the hydraulic element charts set forth in Figures 20 and 21 should be used to determine critical characteristics required for solution of Manning's equation.

7. For culverts, the minimum desirable velocity during the design flood is 2.5 feet per second.

8. The minimum culvert size should be 12 inches in diameter.

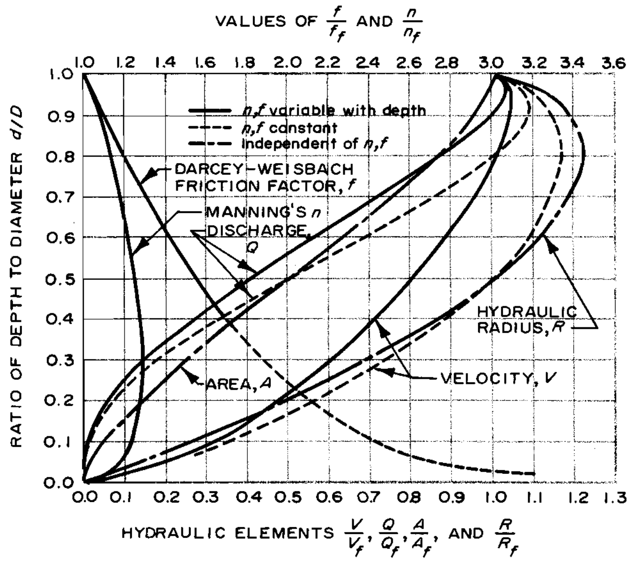
9. Culverts should be laid on an uninterrupted uniform gradient.

10. Where practical, the culvert location should provide a direct exit, avoiding an abrupt change in direction at the outlet end, and, desirably, at the inlet end.

11. Appropriate energy dissipation and/or erosion protection should be provided at culverts and bridges. The type of protection will be dictated by site-specific hydraulic considerations.

Figure 20

HYDRAULIC ELEMENTS
GRAPH FOR CIRCULAR SEWERS

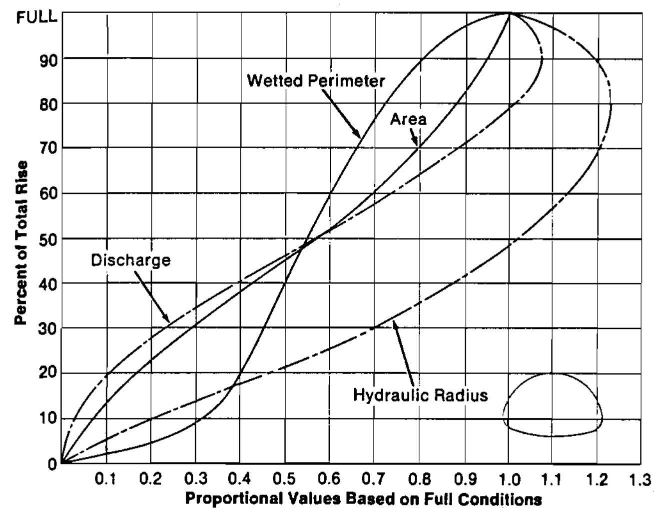


NOTE: n, V, Q, AND R REPRESENT PARTIAL FLOW CONDITIONS. n_f, V_f, Q_f, AND R_f REPRESENT FULL FLOW CONDITIONS.

Source: American Society of Civil Engineers.

Figure 21

HYDRAULIC PROPERTIES OF CORRUGATED STEEL AND STRUCTURAL PLATE PIPE-ARCHES



Source: American Iron and Steel Institute.

12. In streams with an existing or potential valuable fishery, the bottoms of bridges and culverts should be designed to allow for the free passage of aquatic organisms for a variety of flow extremes.

Stormwater Drainage Facility Design Criteria

There are two distinct drainage systems to be considered in the development of the stormwater drainage elements of this system plan: the minor system and the major system. The minor stormwater drainage system is intended to minimize the inconveniences attendant to inundation from more frequent storms. The minor drainage system consists of sideyard and backyard drainage swales, street curbs and gutters, roadside swales, storm sewers and appurtenances, and some storage facilities. It is composed of the engineered paths provided for the stormwater runoff to reach the receiving streams and watercourses during these more frequent storm events.

The major stormwater drainage system is designed for conveyance of stormwater runoff during major storm events when the capacity of the minor system is exceeded. The major stormwater drainage system consists of the entire street cross-section and interconnected drainage swales, watercourses, and stormwater storage facilities. Portions of the streets, therefore, serve as components of both the minor and major stormwater drainage systems. When providing transport of overland runoff to the piped storm sewer system, the streets function as a part of the minor drainage system; when utilized to transport overflow from surcharged pipe storm sewers and culverts and overflowing roadside swales, the streets function as a part of the major drainage system. Major drainage system components must be carefully studied to identify areas subject to inundation during major storm events.

To ensure that the stormwater drainage system is able to effectively control the stormwater runoff in a cost-effective manner, storm events of specified recurrence intervals must be selected as a basis for the design and evaluation of both the minor and major drainage systems. The selection of these design storm events should be dictated by careful consideration of the frequency of inundation which can be accepted versus the cost of protection. This involves value judgments which should be made by the responsible local officials involved and applied consistently in both the public and private sectors.

The average frequency of rainfall used for design purposes determines the degree of protection afforded by the stormwater management system. This protection should be consistent with the damage to be prevented. In practice, however, the calculation of benefit-cost ratios is not deemed warranted for ordinary urban drainage facilities, and a design storm recurrence interval is selected on the basis of engineering judgment and experience with the performance of stormwater management facilities in similar areas.

In this respect, it should be noted that the cost of storm sewers and other drainage facilities is not directly proportional to either the design storm frequency or the flow rates. A 10-year recurrence interval storm produces approximately 16.5 percent greater rainfall intensities and 26 percent greater runoff intensities than a 5-year recurrence interval storm. This higher runoff rate requires sewer pipe diameters to be on the order of 10 percent larger. However, drainage systems are limited to commercially available pipe sizes which, in the most frequently used range of 15- to 66-inch diameter, have incremental diameter increases of 10 to 20 percent, corresponding incremental capacity increases of 27 to 58 percent, and corresponding average in-place cost increases of 15 to 23 percent. However, the incremental cost increases on a systemwide basis may be expected to be on the order of 15 percent, because only portions of any given system will require modified sizes.

Another consideration in evaluating alternative design recurrence intervals for drainage facilities is the risk of exceeding capacity. A 5-year recurrence interval event, which is expected to occur on the average of 20 times in 100 years, has a 50 percent chance of being exceeded in about 3.5 years, a period which may be unacceptable from a public relations point of view. In contrast, a 10-year recurrence interval event, which is expected to occur on the average of 10 times in 100 years, has a 50 percent chance of being exceeded in about seven years.

The municipalities in the District stormwater drainage and flood control system plan study area generally use either a 5- or 10-year recurrence interval storm event for the design of minor stormwater drainage elements, including storm sewers. For this system plan, the design storm frequency used by the local municipality was used in the systems analyses.

The stormwater drainage system components to be considered in this drainage and flood control plan are listed in the policy plan that was prepared by the Regional Planning Commission.[16] Those components include storm sewers, detention and retention basins, pumping stations, and other appurtenances of areawide significance. Interior drainage facilities should also be considered as possible elements of a stormwater drainage system. Interior drainage facilities are necessitated when the construction of diversions, flood protection dikes, or floodwalls may be expected to obstruct natural, or man-made, drainage patterns to receiving streams and watercourses, or when dikes or floodwalls raise the stage in the receiving stream to a level that will cause backwater in storm sewers and flooding of protected areas. Interior drainage systems combine conveyance measures, or conveyance and storage measures, to transmit stormwater runoff from the landward side of dikes or floodwalls to streams or watercourses. Such systems may include all, or some, of the following elements: 1) open drainage channels, including grass swales; 2) cross culverts to convey stormwater under streets, highways, railways, or other embankments; 3) stormwater storage facilities; 4) storm sewers; 5) pumping stations; and 6) backwater gates for storm sewer outlets. While several of these elements are generally considered to be part of the minor drainage system, their interrelationship with the major drainage and flood control systems requires that their functions be evaluated during storms in excess of the minor system design storm.

Design criteria for open drainage channels, cross culverts, and stormwater storage facilities have all been addressed in the preceding section on flood control facility design criteria. Design criteria for major storm sewers, grass swales, and pumping stations are given below.

Storm Sewers:

1. Sewer modifications or additions should be designed to accommodate the peak runoff from the design storm used by the municipality in which the modifications or additions are to be made. If a storm sewer crosses corporate limits and one municipality has a more stringent design storm requirement than the other, the more stringent require-

[16] *SEWRPC Community Assistance Planning Report No. 130, A Stormwater Drainage and Flood Control Policy Plan for the Milwaukee Metropolitan Sewerage District, March 1986.*

ment should be used. In no instance should storm sewers be sized for less than a 5-year recurrence interval storm.

2. A Manning's "n" of 0.012 should be used for well-constructed, precast, concrete pipe sewer lines flowing full.

3. Where the analyses indicate that sewers would flow less than full at design loadings, the hydraulic element chart set forth in Figure 20 should be used to determine the critical hydraulic characteristics.

4. The minimum desirable velocity during the design storm event is 2.5 feet per second.

5. The minimum depth of cover over the top of the sewer should be 3.0 feet.

6. Where the elevations of street inlets, or other storm sewer entry points, in protected areas are at or below the design flood stage on the receiving stream, backwater gates should be provided on storm sewer outlets.

7. Storm sewer outlet invert elevations should be above the channel bottom elevations of the receiving watercourses. This criterion assumes that there is periodic cleaning and maintenance of stream channels.

Grass Swales: In some areas of low-density urban development, grass swales are used to convey stormwater runoff to collector storm sewers or to receiving streams. Such systems are generally used in conjunction with culverts at driveway and street crossings. In addition to providing a relatively low-cost means of conveyance, as compared to conventional storm sewers, grass swales may be designed to reduce peak flows and to remove some pollutants from the runoff through deposition and infiltration. The effectiveness of a grass swale in removing pollutants can be increased through the addition of an infiltration trench in the bottom of the swale.

1. Grass swale components of the minor stormwater drainage system should be designed to accommodate the peak runoff from the frequency design storm used for a minor system of the municipality in which the swales are located.

2. Swales may be designed to flow full with no freeboard.

3. The depth of the swale bottom below the street shoulder should be from one and one-half to three feet.

4. Manning's "n" factors ranging from 0.035 to 0.10 should be used for swale design, depending on the swale location, anticipated frequency of mowing, longitudinal swale bottom slope, and design flow depth.

5. All swales should be designed to provide a maximum flow velocity of five feet per second when accommodating the design storm.

6. Culverts that are part of a grass swale system should be designed according to the applicable criteria listed in the "Bridge and Culvert Alteration or Replacement" subsection of this chapter.

7. Cross culverts at roadways and railways should be designed to meet the applicable flow capacity standards listed in Standard No. 1 of Objective No. 1 in Table 20 of this chapter.

Pumping Stations: The purpose of stormwater pumping is to remove stormwater from low-lying areas that cannot be effectively drained by gravity.

When closed backwater gates prevent stormwater drainage from areas protected by dikes or floodwalls, temporary or permanent pumping stations can be used to convey the impounded storm drainage over the dikes or floodwalls to the stream during major flood events. Stormwater pumping stations are also associated with stormwater storage facilities that have limited land surface available and are restricted to deep storage. Pumping should not be included as a component of the drainage and flood control plan when an alternative providing gravity drainage is practicable.

At the systems planning level, only recommendations concerning the location, type, and capacity of the pumping facility are provided. More detailed engineering at the facilities planning and final design stages will be needed to determine the type of pumps, type of drives and motor requirements, type of electrical controls, and size and configuration of intake facilities.

The following criteria and assumptions relating to stormwater pumping facilities should be used in the development of the drainage and flood control system plan:

1. An evaluation should be made of the ability of the pumping station to provide protected areas with relief from flooding during floods with recurrence intervals ranging from 5 to 100 years.

2. The pumping station should be designed with a gravity overflow to the major drainage system.

3. For systems planning purposes, it should be assumed that the pumps will be high-capacity, low-head centrifugal pumps powered by constant-speed electric motors designed for intermittent service.

4. Each pumping station should be designed with a backup power source.

Flood Control Facility Safety Design Criteria: Because of the detailed nature of the design of most safety measures for flood control facilities, such design is most appropriately accomplished at the final design stage rather than at the system planning stage. Therefore, this system plan does not include criteria relating to specific safety measures. Potential safety hazards were considered to be intangible elements in the comparison of alternative plans.

Economic Evaluation

The concepts of economic analysis and economic selection are vital to the public planning process. In dealing with a single system of public works, such as drainage and flood control facilities, the evaluation of alternative projects and the establishment of a priority order among potential projects can probably best be achieved through economic analysis and selection. All decisions concerning monetary expenditures, public or private, are based on an evaluation, objective or subjective, of benefits and costs. This is not to imply that a formal economic analysis is made before every such decision. The process of decision-making itself, however, consists of an evaluation as to whether or not the benefits to be received will be worth the costs to be incurred. Benefits are not necessarily accountable in monetary terms, but the very act of spending money—or resources—for an intangible benefit implies that the benefit is perceived to be worth at least the cost incurred.

In addition, consideration should be given to possible alternative benefits that could be received for alternative expenditures within the limits of the available resources. Alternative benefits are compared, and the project which is considered to give the greatest value for the costs entailed selected. One alternative that should always be considered is the benefit that would be received from investment in the money market. This benefit is expressed in the prevailing interest rates.

Personal and private decisions, while implying at least subjective consideration of benefits and costs broadly defined, are not, as already noted, necessarily based on either an objective or explicit evaluation of monetary benefits and costs. Public officials, however, have a responsibility to objectively and explicitly evaluate the monetary benefits and costs of alternative investments to assure that the public will receive the greatest possible benefits from the always limited monetary resources available.

It is then a goal of good public administration that every public expenditure return to the public a value at least equal to the amount proposed to be expended for a project plus the interest income foregone from the ever-present alternative of private investment. Accordingly, benefit-cost analysis has been used for many years by the Regional Planning Commission to select from among alternatives the most economically efficient means of resolving a problem—or of reaching an objective. Other agencies utilizing this method in planning and engineering include the U. S. Army Corps of Engineers and the Wisconsin Department of Transportation. The method is recommended by such national agencies as the American Association of Highway and Transportation Officials, and is usually one of the major methods set forth in textbooks on engineering economics. Benefit-cost analysis can also be adapted to provide a basis for prioritizing projects, all intended to attain the same objectives. Thus, benefit-cost analysis, together with certain overriding considerations which include technical feasibility, environmental soundness, public health and safety factors, and regulatory constraints, was used to compare the alternative projects considered for this system plan and to establish a preliminary construction priority order among otherwise eligible drainage and flood control improvements.

Benefit-Cost Analysis: The benefit-cost analysis method of evaluating government investment in public works came into general use after the adoption of the federal Flood Control Act of 1936. The Act stated that waterways should be improved "if the benefits to whomsoever they may accrue are in excess of the estimated costs." The monetary value of benefits is defined as the amount of money which an individual would pay for that benefit if given the market choice of purchase. Monetary costs are defined as the total value of the resources used in the construction of the project.

In order to assure that public funds are invested most profitably, alternative plans or projects should be investigated and analyzed. Such investigations and analyses are properly conducted at the systems planning level and have been included in this drainage and flood control plan.

The recommended plan should be selected from those alternatives which meet watershed development objectives only after consideration of the following hierarchy of economic considerations:

1. Benefits, including intangible values, must exceed costs in order for a project to be economically justified.

2. An excess of benefits over costs, however, is not a sufficient criterion on which to base a recommendation; and, therefore, among those alternative plan elements exhibiting benefit-cost ratios greater than one, the alternative with the greatest difference between benefits and costs, not the greatest benefit-cost ratio, will produce the largest absolute return on the investment.

3. Maximization of benefits minus costs is not, however, in and of itself a sufficient criterion for selection of an alternative, since the amount of public funds available or potentially available, and public attitudes toward a particular plan element, must be considered in selecting from among various plan elements. It may be politically and financially impossible to obtain support and funding for a plan element even though it, among all the available alternatives, would produce the greatest return on the investment.

The benefits and often the costs of drainage and flood control projects accrue over long periods of time. Moreover, each project is likely to have a different time flow of benefits and costs. Benefits of one project may be realized earlier than those of another, while the time flow of costs may vary from one large initial investment for one project to small, but recurrent, expenditures over a long period of time for another. In order to place these projects with varying time flows of benefits and costs on a comparable basis, the concept of the time value of money must be applied. A dollar benefit or a dollar cost at some time in the future has a value less than a dollar at present. The variation of the value of benefits and costs with respect to time is expressed through the mathematics of compound interest. Use of an interest rate also incorporates consideration of the ever-present possibility of private investment as an alternative. To be economical, a project should return to the public a benefit approximating that which might be obtained through private investment. Money invested privately is currently expected to return from 4 to 8 percent interest after taxes. Since implementation of the drainage and flood control plan should return benefits to the public similar to those which could be attained through private investment by small investors, an interest rate of 6 percent is recommended for use in the economic evaluation of plans.

The benefit-cost analysis must also be based on a specified number of years, usually equal to the physical or economic life of the project. Drainage and flood control improvement will often continue to furnish benefits for an indefinite period of time, particularly land use control and public park elements. Accordingly, the Regional Planning Commission has selected 50 years as the period of economic analysis for drainage and flood control works. Benefits accrued after 50 years discounted to the present at 6 percent are very small.

Project Benefits: The benefits from a drainage and flood control project can be classified as tangible—that is, measurable in monetary terms—and as intangible. Tangible benefits include flood damage reduction, enhancement of property values, and that part of outdoor recreation to which a monetary value can be assigned. Intangible benefits include aesthetic factors and such benefits as improved efficiencies in public utilities that have monetary values, but values that cannot be practically calculated. The specific benefits of water quality improvements were considered to be intangible in the sense that these benefits are difficult to measure, although very real since a high level of recreational use of surface waters is possible only if applicable water quality standards are met.[17]

Project Costs: The direct costs of water resource development include the construction, operation, and maintenance costs of physical elements of the plan; the cost of acquiring land; income foregone as a result of land use regulation; and expenditures for engineering, legal work, and project administration. The costs of structural facilities were calculated using 1986 unit prices, which reflect the magnitude of work, the location in the urban region, and regional labor costs. The cost of land acquisition was based on 1986 market prices for land in the District.

Method of Calculating Benefit-Cost Ratio: The accepted rule for economic efficiency is to select the set of improvement projects having the greatest excess of benefits over costs—that is, the maximum net present value. In situations where there is a budgetary constraint, the net present value of the potential projects can be maximized by selecting the combination of projects that have maximum present value but in total do not violate the budgetary constraint. For a number of projects, benefit-cost ratios can be defined with maintenance and operating costs and residual, or salvage, value in the numerator as follows:

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[17] *More specifically, flood damage is defined as the physical deterioration or destruction caused by floodwaters. Flood loss refers to the net effect of flood damages on the economy and is usually expressed in monetary terms. All losses resulting from a flood can be broadly classified as direct, indirect, depreciation, or intangible. Reduction of flood loss by flood protection measures creates benefits equal to the damages protected against.*

Direct losses are defined as the monetary costs entailed in restoring flood-damaged property to preflood condition. This includes the cost of restoring flood-damaged residential, commercial, and industrial properties and the value of farm crops destroyed by flooding.

Indirect losses are defined as the monetary costs of flood-fighting and floodproofing, and of flood-caused loss of wages, sales, and production. Increased costs of carrying on normal operations during periods of flood disruption, and increased costs of transportation because of flood-caused detours, are also defined as indirect losses.

Depreciation losses are defined as the reduction in the value of real property when the risk of flooding becomes known. Property values after a flood are reduced by the amount of money which will have to be expended for flood repairs. Accordingly, depreciation losses should be equal to the probable direct losses from future floods. In the Regional Planning Commission approach to flood control planning, the direct flood losses, rather than the depreciation losses, are used in the economic analyses.

Intangible losses are defined as losses that cannot be measured in monetary terms. Intangible losses include loss of life, health hazard, interruption of schooling, loss of police and fire protection, and mental aggravation. Although these losses cannot be measured in monetary terms, they often constitute the most severe flood damage experienced by the public, monetary costs notwithstanding.

Flood damages may also be classified into public sector and private sector losses. Direct public sector losses include road and bridge repairs, basement pumping, and flood clean-up operations. Indirect public sector losses include highway traffic rerouting and control, and relief and health services. In the Commission flood control planning work, road-user detour costs are calculated on the basis of traffic volume, detour length, time of closures, and average per-mile vehicle costs over the normal routes and over the detour routes.

Direct private sector losses include damage to residential, commercial, and industrial properties and to agricultural crops, with such damages being related to the type of building or structure involved, the value of the structures and contents, and the depths and durations of inundation. Damages to structures include, among others, damages to electrical, heating, and ventilating equipment; to ceilings, walls, floors, and fittings; to carpeting; to furniture and appliances; and to other contents.

$$B/C = \frac{PV(\Delta U) + PV(\Delta R)}{PV(\Delta I) + PV(\Delta M)}$$

where: B/C = the benefit-cost ratio

$PV(\Delta U)$ = the present value of benefits relative to the "do nothing" alternative; these benefits are measured in terms of the monetary value of the direct and indirect flood damages avoided by the project;

$PV(\Delta M)$ = the present value of maintenance and operating costs relative to the do nothing alternative;

$PV(\Delta R)$ = the present value of the project residual, or salvage, value relative to the do nothing alternative; and

$PV(\Delta I)$ = the present value of the project capital cost relative to the do nothing alternative.

Relationship of Economic and Financial Analysis: The distinction between economic feasibility and financial feasibility is of particular importance in the consideration of the costs of land already in public ownership. A financial analysis involves an examination of the liquidating characteristics of the project from the point of view of the particular government agency undertaking the project. The relevant matters are the monetary disbursements and monetary receipt of the project. The financial analysis determines whether or not the prospective available funds are adequate to cover all the costs.

On the other hand, and as described above, an economic analysis determines if the project benefits to whomsoever they accrue exceed the costs to whomsoever they accrue. Since one of the legitimate objectives of government is to promote the general welfare, it is necessary to consider the effect of a proposed project on all of the people who may be affected, not just on the income and expenditures of a particular agency. The economic valuation of the benefits and costs may differ considerably from the actual income and expenditures of a government agency.

Basic Cost Data: In order to provide a consistent basis for the determination of the costs of the structural and nonstructural flood control and drainage components of the alternative plans considered, basic cost data were developed for construction and annual operation and maintenance. Those data are presented in Appendix A.

Where feasible, construction cost curves for entire components are presented. Such curves are given for surface storage facilities, storm sewers, dikes, floodwalls, circular culverts, tunnels, and pumping stations. For other structural flood control measures, unit construction costs for each element of the particular measure are tabulated. Unit-cost tabulations are provided for bridge alteration or replacement, channel modifications, and channel enclosures.

Figures A-1 through A-10 and Tables A-1 through A-6 in Appendix A represent 1986 construction or operation and maintenance costs based on an Engineering News-Record Construction Cost Index (CCI) of 4520 as has been adopted for the Milwaukee area by the District. When estimating total project costs, the costs obtained from those figures and tables should be increased by 35 percent to account for engineering, administration, and contingencies. Where applicable, the cost of land acquisition or easements should be added.

Cost data were obtained from several of the more recent District flood control projects, from bid tabulations for other recent flood control and drainage projects within the Region, from past Regional Planning Commission studies, from studies conducted by the U. S. Army Corps of Engineers, and from the 1982 Dodge Guide to Public Works and Heavy Construction Costs.[18] Where pre-1986 data were used in the development of cost curves or unit costs, the Milwaukee CCI was used to adjust the costs to 1986.

Cost data for the structural measures considered were adopted after comparison and evaluation of data from the sources listed previously. The validity of the adopted unit-cost data for the typical elements of a channel modification project was verified by using the data to estimate the costs of several constructed District flood control projects for which total costs were available.

[18] *Leonard A. McMahon, 1982 Dodge Guide to Public Works and Heavy Construction Costs, ed. Percival E. Pereira, Annual Edition No. 14, McGraw-Hill, Princeton, New Jersey, 1981.*

Cost estimating data and procedures for nonstructural flood control methods are given in Tables A-7 through A-10. The data were developed from past studies by the Regional Planning Commission and from studies conducted within the Region by the U. S. Army Corps of Engineers. These data represent total 1986 costs and they should not be increased for engineering, administration, and contingencies.

For both structural and nonstructural flood control measures, the adopted base cost data are those which are considered most applicable to the types of projects considered for the District drainage and flood control plan. The cost data presented in Appendix A were used in the economic evaluation of alternative systems plans, and are not intended to be used for project estimating purposes. Actual costs will vary from these estimates, reflecting site-specific conditions, local availability and supply of materials, and labor costs. Any necessary land acquisition costs were estimated utilizing the latest available state equalized assessed valuations.

Staged Development and Priority Determination: Because rarely, if ever, are sufficient public monies available to construct simultaneously all of the public works facilities which may be needed in a functional area, it becomes necessary to establish a program of construction projects arranged in order of priority. Desirably, that order would provide the greatest return on the public funds invested in the projects over time.

An attractive feature of many water resource developments is their divisibility into several individual projects which may be assigned priorities and then financed and built at different times. Staged construction based on prioritization of individual projects permits lower initial capital investments, reduces interest costs, and allows for flexibility in continued planning. Staging developments may also allow an element to be deferred until increased demands raise its benefit-cost ratio. In planning for staged development, however, consideration must be given to the possibilities of higher costs in the future and unavailability of land. In any development, staging also serves to lower risks incurred because of unavailability of data during preparation and partial implementation of initial plans.

Variations on the means by which benefit-cost analyses are applied in planning and engineering work are possible, including application of the benefit-cost method in a manner which considers the difference between benefits and costs. For the purpose of the prioritization of the District drainage and flood control projects, however, it is recommended that the method be applied in its simplest and most direct form. This not only maintains simplicity and thereby promotes public understanding, but recognizes that maximization of benefits minus costs will in some cases be insufficient in and of itself for the final prioritization of proposed projects. Other factors, including the amount of public funds available, or potentially available, and public attitudes toward, and understanding of, a particular improvement proposal must also be considered in prioritizing projects if the political and financial feasibility of the program is to be assured.

To rank independent improvements, the project with the highest benefit-cost ratio is selected first, and other projects—always with the next highest benefit-cost ratio—are added to the list until all projects are accounted for. Based upon consideration of the total cost of the program and estimates of the available funding for the program, a five-year capital improvements program can then be developed. This program should be reviewed annually, at which time the first year of the program would be proposed for inclusion in the District annual budget, and an additional year would be added to the end of the program—thus always maintaining a five-year program—until all of the needed drainage and flood control improvements are in place.

Another intangible factor which should be considered is the history of citizen concern in a problem area. In many instances, elected and appointed officials and interested citizens have spent considerable effort over long periods of time in attempting to resolve problems, in some cases with no tangible progress. While it is not possible to directly consider such histories in the prioritization owing to their intangible nature, it is possible for the Advisory Committee to indirectly consider such histories in arriving at a final improvement schedule.

Overriding Considerations: The following overriding considerations must be met before applying the benefit-cost analysis to the consideration of alternatives and to the prioritization of the drainage and flood control projects:

1. Each project to be considered must have been shown at the systems level of planning to be technically feasible and economically

and environmentally sound. The determination of technical feasibility should be based upon analyses, preferably hydrologic and hydraulic simulation model studies such as those conducted for this plan. Those analyses should clearly indicate that the proposed project will achieve the reductions in peak flood flows or peak flood stages, or both, that are necessary to abate the flood damages concerned without exacerbating such problems either upstream or downstream of the proposed project.

2. The project should be shown to be economically sound by benefit-cost analysis. While such analysis applied in the classic manner would require that the benefit-cost ratio of a project be greater than one, it must be recognized that other objectives which cannot be directly quantified monetarily, such as providing adequate outlets for municipal stormwater sewers or abating public health and safety hazards resulting from the backup of sanitary sewers surcharged by floodwaters into basements of buildings, may make it politically desirable to construct a project having a benefit-cost ratio of less than one. However, in such cases it should always be demonstrated that the project, while having a benefit-cost ratio of less than one, has the highest benefit-cost ratio of the feasible alternatives.

3. The project must have been shown at the systems level of planning to be environmentally sound by explicitly considering potential impacts on surface- and groundwater quality and existing and potential fish and wildlife habitats and populations. The project must qualify for all legally required regulatory agency approvals.

Only if a project meets the foregoing overriding considerations should it be considered for selection as a recommended alternative and for prioritization utilizing the benefit-cost analysis results presented in this plan.

Once the projects have been prioritized on the basis of the benefit-cost analysis, two additional overriding criteria may increase the order of priority of a given project. First would be evidence of a foreseeable danger to human life. Second would be evidence that the timing of the project must be changed in order to coordinate its construction with the construction of other major public works, such as highways, sanitary sewerage systems, or water supply facilities.

Chapter IV

EVALUATION OF ALTERNATIVE PLANS AND SELECTION OF RECOMMENDED FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN— KINNICKINNIC RIVER WATERSHED

INTRODUCTION

The adopted Milwaukee Metropolitan Sewerage District drainage and flood control policy plan recommends that the District assume jurisdiction over six perennial streams in the Kinnickinnic River watershed. These six streams, totaling 15.9 lineal miles in length, include the Kinnickinnic River, Lyons Park Creek, Wilson Park Creek, S. 43rd Street Ditch, Villa Mann Creek, and Villa Mann Creek Tributary. Three of these streams—the Kinnickinnic River, Lyons Park Creek, and Wilson Park Creek—have been studied under previous Commission planning programs.[1] Each of these six streams is considered in the following sections of the chapter. Data are presented on existing and probable future flood problems, alternative and recommended flood control and related drainage improvement measures, and recommended implementation actions.

KINNICKINNIC RIVER WATERSHED FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN

Flood control improvements for the Kinnickinnic River were considered in a comprehensive watershed plan prepared by the Commission in December 1978.[2] That plan recommended the following flood control measures for the Kinnickinnic River: 1) removal of 14 bridges between the abandoned Chicago, North Shore & Milwaukee right-of-way and S. 16th Street; 2) construction of new bridges at S. 6th Street, S. 9th Place, S. 13th Street, and S. 16th Street; and 3) channel reconstruction and widening between S. 5th Street extended and S. 6th Street, and between S. 8th Street and S. 12th Street. These measures were designed to prevent overbank flooding in this reach for floods up to and including the 100-year recurrence interval event, and were completed by the District and the City of Milwaukee in 1985. Hydrologic and hydraulic analyses conducted under the Commission's watershed study indicated no further reaches along the Kinnickinnic River where flood damages are to be expected for floods up to and including the 100-year recurrence interval event under either existing or planned land use conditions. Accordingly, no further flood control measures for the Kinnickinnic River were considered under that study.

As a result of the heavy rainfall that occurred on August 6, 1986, severe flood damages were again experienced along that reach of the Kinnickinnic River between S. 6th Street and S. 16th Street. That flood, however, had an estimated recurrence interval in excess of 500 years. As a result of that flood event, the City of Milwaukee Common Council adopted a resolution requesting that the District consider diverting all, or part, of the flow from Wilson Park Creek to Lake Michigan in order to reduce flood flows exceeding the 100-year recurrence interval flows along the Kinnickinnic River. Accordingly, alternative diversion measures were investigated as part of this system plan. Those measures are presented later in this chapter under the discussion of the Wilson Park Creek subwatershed.

Overview of the Study Area

The Kinnickinnic River watershed is located largely within the City of Milwaukee. Portions of the watershed are also located in the Cities of Cudahy, Greenfield, West Allis, and St. Francis and in the Village of West Milwaukee. From its headwater area at S. 60th Street, the Kinnickinnic River flows in a generally easterly direction for approximately 8.0 miles, and drains an area of 24.78 square miles (see Map 26). Of this total drainage area, 18.70 square miles, or about 75 percent, lie within the City of Milwaukee; 1.47 square miles, or about 6 percent, lie within the City of Cudahy; 2.32 square miles, or 9 percent, lie within the City of Greenfield; 1.66 square miles, or about 7 percent, lie within the City of West Allis; 0.51 square mile, or 2 percent, lies within the Village of West Milwaukee; and 0.12 square mile, or about 0.5 percent, lies in the City of St. Francis.

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[1] *See SEWRPC Planning Report No. 32, A Comprehensive Plan for the Kinnickinnic River Watershed, December 1978.*

[2] *Ibid.*

Map 26

THE KINNICKINNIC RIVER WATERSHED

Source: SEWRPC.

More specifically, from its origin at a storm sewer outfall at S. 60th Street immediately south of W. Kinnickinnic River Parkway Drive, the Kinnickinnic River flows in a generally easterly direction to about S. 4th Street, at which point it turns to flow in a north-northeasterly direction to join the Milwaukee River near Lake Michigan. The entire 8.0-mile reach described is classified as perennial. This entire stream length is recommended for District jurisdiction in the policy plan companion to this system plan.

The Kinnickinnic River watershed is almost completely developed for urban use, including residential, commercial, industrial, institutional, and urban open space uses. The open space uses are comprised mainly of public parks and cemeteries. The developed areas of the Kinnickinnic River watershed are generally provided with a full range of municipal street improvements, including paved streets with curbs and gutters and attendant storm sewers. Accordingly, surface runoff is generally conveyed rapidly from each individual site to the Kinnickinnic River through storm sewers.

Specific data on pertinent characteristics of the watershed, including hydrologic soil types, land slopes, and land use, appear in Chapter II of this report. The planned land use conditions utilized in the system planning assume that the watershed will be fully urbanized by the design year of the plan. However, existing open space uses, such as parks and cemeteries, will remain.

Flooding, in various degrees, has been a common occurrence adjacent to the Kinnickinnic River. Flooding along the river has increased proportionally to the degree of conversion of land from open, rural uses to urban uses. Channel improvements have been made along 6.2 miles, or 77 percent, of the stream length to accommodate the increased streamflows. The channel has been physically altered by deepening, straightening, lining with concrete or stone, construction of sills or drop spillways, and enclosure in culverts.

Flooding and Related Drainage Problems

As already noted, flooding, including first-floor flooding, yard flooding, and basement flooding, has occurred frequently along the Kinnickinnic River. Historically, the area that has sustained the worst flooding is that along the 0.77-mile reach between S. 6th Street and S. 16th Street. Flood control works that were completed in 1985 for this reach have greatly reduced this problem. Those improvements, which were described earlier, were designed to prevent overbank flooding for floods up to and including the 100-year recurrence interval event.

During 1986, there were seven storm events for which flooding and water-related problems were reported, based upon records maintained by the City Engineer of the City of Milwaukee. Map 27 shows those areas for which problems were reported. More than 340 separate flooding problems were documented during 1986, with the most—230—being reported during the August 6 rainfall. The majority of the reported problems were due to either sanitary sewer backups or localized stormwater drainage problems. Structure damages due to overbank flooding from the Kinnickinnic River were, however, documented for the August 6, 1986, rainfall. These damages were located mainly along the reach between S. 6th Street and S. 16th Street. As previously noted, that event had an estimated recurrence interval in excess of 500 years, far beyond the design capacity of the completed flood control works. Flood damages from the August 6 event would have been much worse had the flood control improvements recommended in the adopted Kinnickinnic River watershed plan not been in place. Map 28 shows the extent of flooding from the August 6 event, as well as the estimated floodplain had these improvements not been implemented. As shown on this map, the August 6 event resulted in the flooding of about 20 acres of land along the river and resulting damages to about 80 structures. Had the channel improvements not been in place, the area impacted by this flood event would have encompassed about 90 acres, and the flooding would have resulted in damages to about 460 structures. It should be noted that the flood control measures considered under this system plan are aimed primarily at alleviating flood damages from direct overland flooding along the stream studied, as well as at providing an adequate outlet for local storm sewers. These measures will help to reduce, but will not necessarily eliminate, flooding due to localized stormwater drainage problems or sanitary sewer backup.

The drainage and flood control objectives and supporting principles and standards set forth in Chapter III specify the flood events which bridges shall accommodate without overtopping the related roadway. Based on those criteria, two bridges, S. Chase Avenue and S. 43rd Street, are considered hydraulically inadequate as shown in Appendix B.

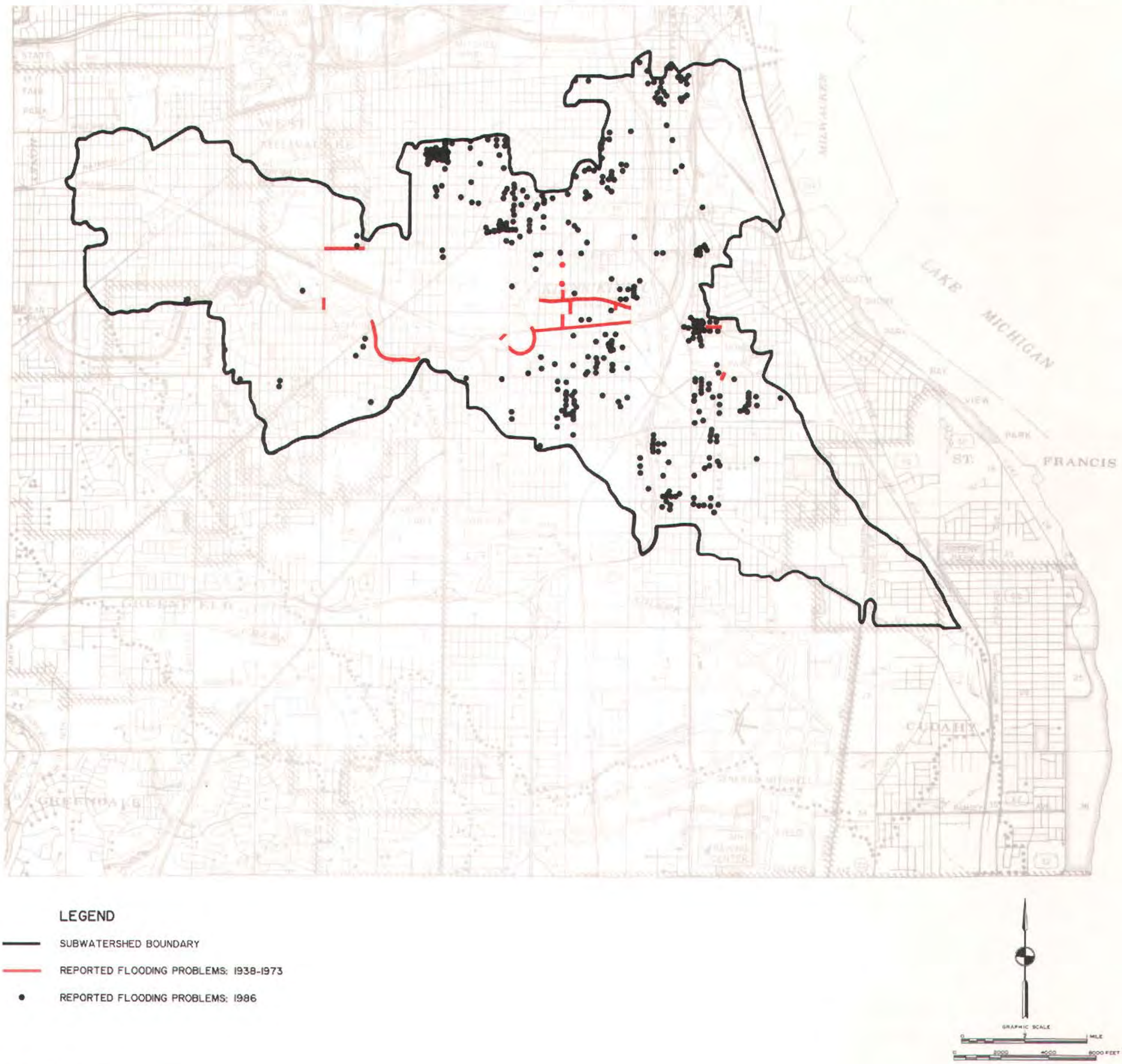
Flood Discharges and Stages

As noted in Chapter III of this report, the hydrologic model used in developing design discharges for the Kinnickinnic River simulates streamflow on a continuous basis using recorded climatological data as input. Flood discharges are developed by conducting discharge-frequency analyses of simulated annual peak discharges generated by the hydrologic model according to the log Pearson Type III method of analysis, as recommended by the U. S. Water Resources Council[3] and as speci-

[3] *United States Water Resources Council, "Guidelines for Determining Flood Flow Frequency," Bulletin No. 17 of the Hydrology Committee, Washington, D. C., March 1976.*

Map 27

AREAS WITH REPORTED FLOODING AND DRAINAGE PROBLEMS IN THE KINNICKINNIC RIVER WATERSHED



Source: SEWRPC.

fied by the Wisconsin Department of Natural Resources.[4] These analyses were conducted for planned land use and existing channel conditions at seven locations along the Kinnickinnic River. The flood discharges that were developed were then checked by incorporating the discharges into a hydraulic model to develop stages and comparing those stages to available historical high water mark data. Such data were available at one location on the channel from the U. S. Geological Survey, at 11 locations on the channel from the City Engineer

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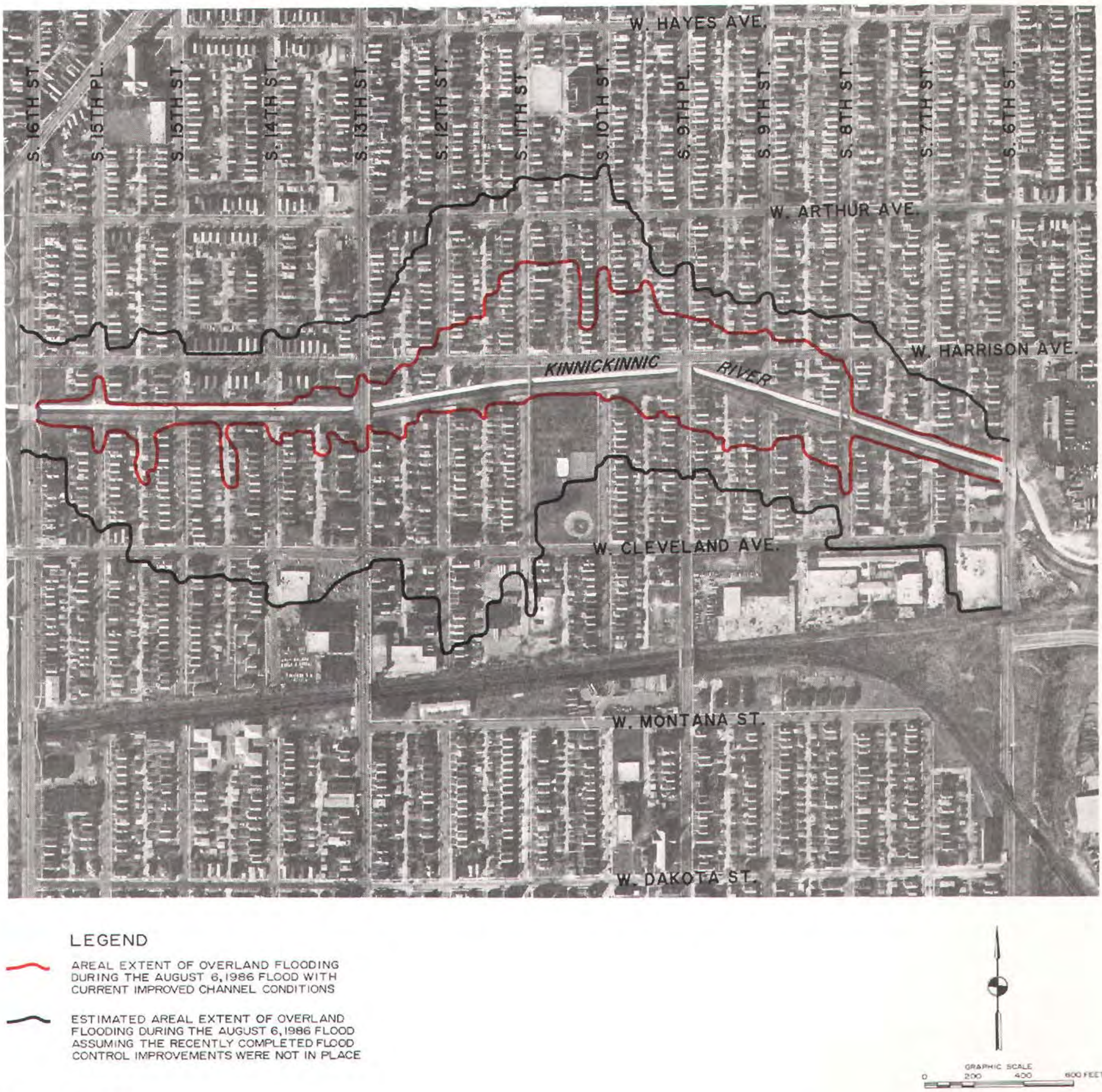
[4] *"Wisconsin's Floodplain Management Program," Wisconsin Administrative Code, Chapter NR 116, February 1986.*

Map 28

EXTENT OF OVERBANK FLOODING ON THE KINNICKINNIC RIVER BETWEEN 6TH AND 16TH STREETS—UNDER EXISTING CHANNEL CONDITIONS AND ASSUMING CHANNEL CONDITIONS PRIOR TO RECENT IMPROVEMENTS: AUGUST 6, 1986 STORM EVENT



Source: SEWRPC.

of the City of Milwaukee, and at 10 locations on the channel from the Milwaukee Metropolitan Sewerage District.

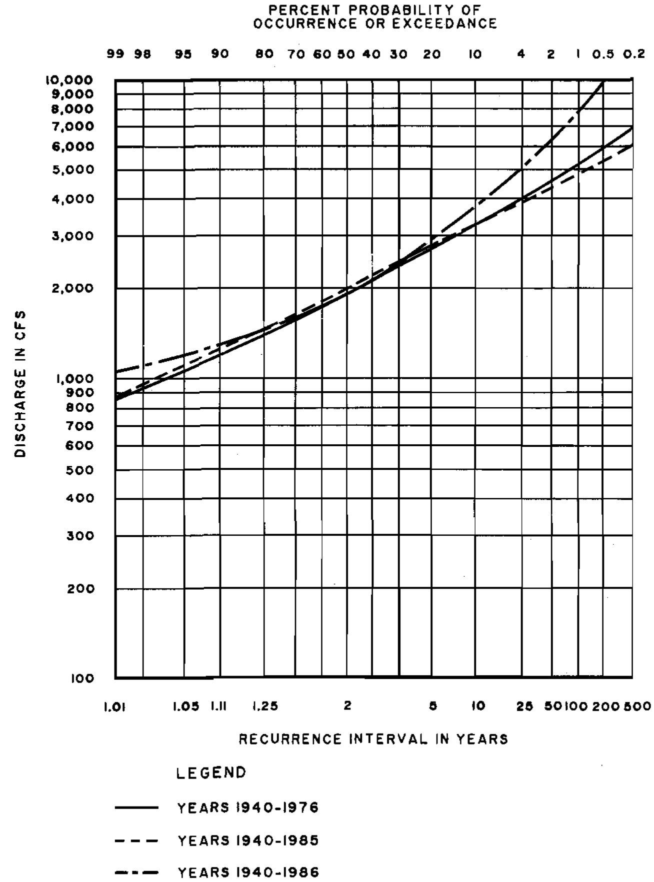
In preparing the Kinnickinnic River watershed comprehensive plan, streamflows were developed on a continuous basis for the 37-year period from 1940 through 1976. Annual peak discharges for these 37 years were used in the log Pearson Type III analysis to determine flood discharges of the required frequencies. As part of this system plan, the previously published flood discharges were reviewed in light of the availability of an additional 10 years of climatological data. Special consideration was given to the impact of the August 6, 1986, storm event on the resulting discharge-frequency relationship, since the event produced a discharge that was significantly higher than any previous flood. Log Pearson Type III analyses were conducted for the Kinnickinnic River at River Mile 2.72, the site of a continuous stage recorder. Recorded annual peak discharges were added to the 37 years of simulated peak discharges in order to extend the period of record through both water year 1985 and water year 1986.

The resulting discharge-frequency curves for these two periods of record, as well as for the 37-year period used in the Kinnickinnic River watershed study, are shown in Figure 22. As shown in Figure 22, the data for the periods of 1940 through 1976 and 1940 through 1985 produce similar discharge-frequency relationships. The addition of the August 6, 1986, flood discharge, however, results in a significant increase in the flood discharge, particularly for the less frequent events. When performing statistical analyses such as this, the addition of a single data value should not be allowed to significantly alter the resulting frequency relationship, particularly when a relatively long period of record is used, as in this case. Therefore, the discharge for the August 6 flood event was regarded as an anomaly, and was not included in the extended record discharge-frequency analysis. Furthermore, since the revised discharges developed by extending the period of record through 1985 do not differ significantly from the discharges developed under the Kinnickinnic River watershed study, particularly for the 100-year recurrence interval design flood, the added time and cost of developing revised discharges based on an extended period of record for the streams in the Kinnickinnic River watershed were not considered warranted.

The estimated peak flood discharges under year 2000 planned land use and existing (1986) channel conditions are set forth in Table 24. Flood stage profiles were determined for the 10-, 50-, and 100-year recurrence interval runoff events under planned land use and existing channel conditions. These profiles, which encompass the full 8.0-mile-long reach of the Kinnickinnic River studied, constitute a graphic representation of the flood stages along the Kinnickinnic River under the specified recurrence interval flood discharges. In addition to providing an overall representation of flood stages relative to familiar points of reference, such as the channel bottom and bridge deck surfaces, the profiles, because of their continuity, permit the determination of flood stages at any point along

Figure 22

DISCHARGE-FREQUENCY RELATIONSHIPS FOR THE KINNICKINNIC RIVER AT RIVER MILE 2.72



Source: SEWRPC.

the stream channel. The flood profiles are shown in Figure 23. The extent of the 100-year recurrence interval floodplain for the Kinnickinnic River under planned land use and existing channel conditions is shown on Map 29.

Recommended Flood Control System for the Kinnickinnic River

As previously noted, channel modifications along with bridge removal and replacement were completed by the District in 1985 for that reach of the Kinnickinnic River between S. 5th Street extended and S. 16th Street. Those improvements are designed to eliminate overbank flooding in this reach for floods up to and including the 100-year recurrence interval event. Overbank flooding during a 100-year recurrence interval event along the Kinnickinnic River upstream of S. 16th Street would be confined to Milwaukee County-owned parkway lands, and therefore no structure damages

Table 24

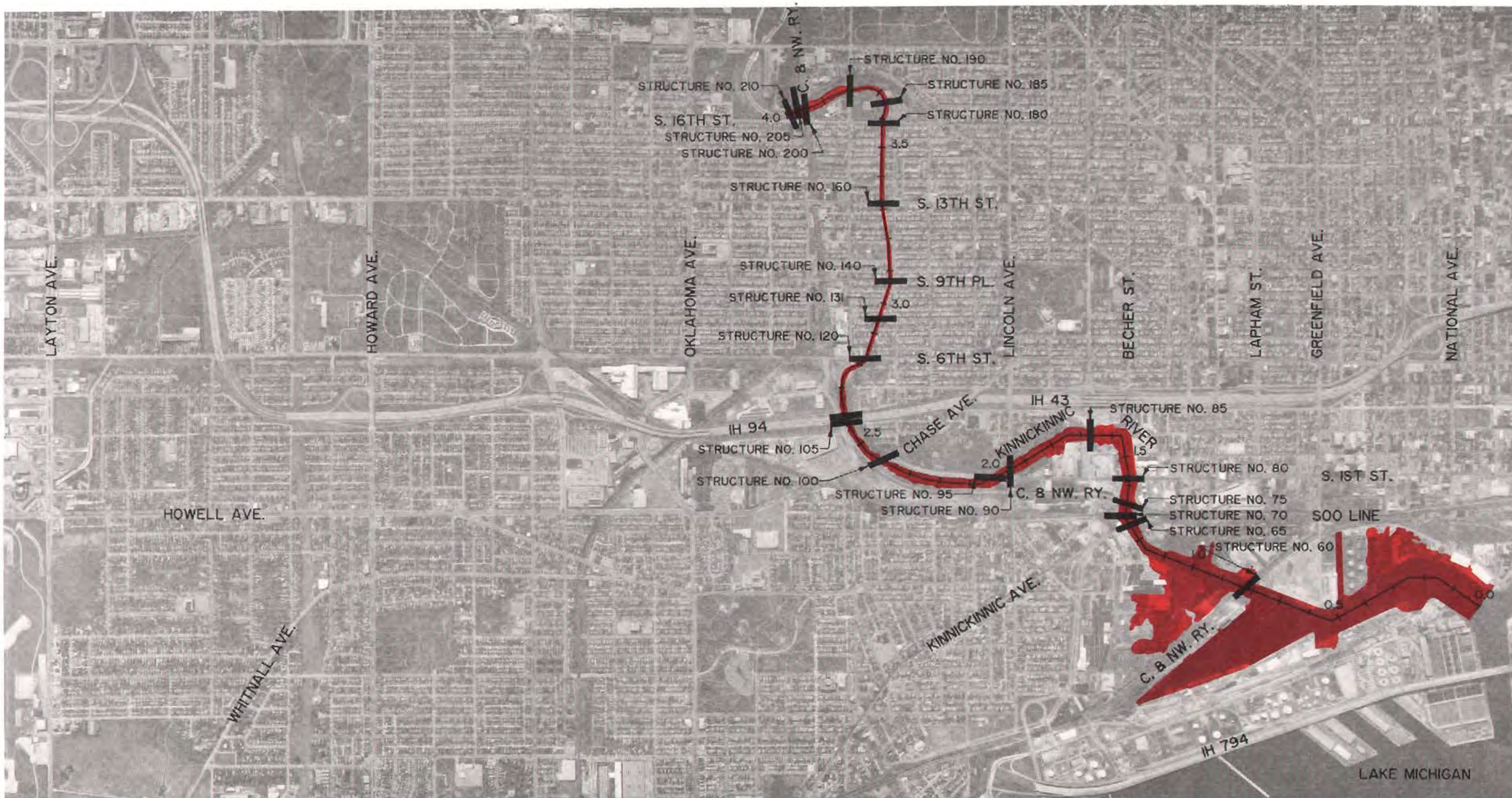
FLOOD DISCHARGES FOR THE KINNICKINNIC RIVER FOR YEAR 2000 LAND USE AND EXISTING CHANNEL CONDITIONS

Location	River Mile	Peak Flood Discharge (cubic feet per second)		
		10-Year	50-Year	100-Year
Chicago & North Western Railway	0.84	4,550	6,500	7,400
Kinnickinnic Avenue (STH 32)	1.28	4,550	6,500	7,400
Soo Line Railroad	1.31	4,540	6,500	7,400
Chicago & North Western Railway	1.35	4,550	6,500	7,400
S. 1st Street	1.43	4,350	6,200	7,000
W. Becher Street	1.67	4,350	6,200	7,000
W. Lincoln Avenue	1.96	4,350	6,200	7,000
S. 1st Street	2.01	4,350	6,200	7,000
S. Chase Avenue	2.40	4,350	6,200	7,000
IH 94	2.57	4,350	6,200	7,000
Abandoned Chicago, North Shore & Milwaukee Railroad	2.72	3,750	5,300	6,000
S. 6th Street	2.81	3,750	5,300	6,000
S. 9th Place	3.08	3,750	5,300	6,000
S. 13th Street	3.32	3,750	5,300	6,000
S. 16th Street	3.58	3,550	5,000	5,700
Pedestrian Bridge	3.65	3,550	5,000	5,700
W. Cleveland Avenue	3.79	3,550	5,000	5,700
Chicago & North Western Railway	3.94	3,550	5,000	5,700
Chicago & North Western Railway Spur	3.96	3,550	5,000	5,700
Drop Structure	3.995	3,550	5,000	5,700
S. 20th Street	4.32	3,550	5,000	5,700
Chicago & North Western Railway Spur	4.44	3,550	5,000	5,700
S. 27th Street (USH 41)	4.91	3,550	5,000	5,700
S. 29th Street	5.03	3,550	5,000	5,700
Drop Structure	5.12	3,550	5,000	5,700
Kinnickinnic River Parkway	5.14	3,550	5,000	5,700
Pedestrian Bridge	5.21	1,600	2,250	2,550
S. 35th Street	5.45	1,600	2,250	2,550
W. Forest Home Avenue	5.71	1,600	2,250	2,550
Jackson Park Drive	5.87	1,600	2,250	2,550
Jackson Park Tunnel Outlet Structure	6.01	1,600	2,250	2,550
Jackson Park Tunnel Inlet Structure	6.14	1,600	2,250	2,550
Drop Structure	6.271	1,600	2,250	2,550
Pedestrian Bridge	6.44	1,600	2,250	2,550
S. 43rd Street	6.51	790	1,100	1,200
Pedestrian Bridge	7.16	790	1,100	1,200
S. 60th Street Outfall	8.05	790	1,100	1,200

Source: SEWRPC.

Map 29

100-YEAR RECURRENCE INTERVAL FLOODPLAIN FOR THE KINNICKINNIC RIVER UNDER YEAR 2000 PLANNED LAND USE WITH EXISTING CHANNEL CONDITIONS



LEGEND

100 YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

1.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR THE KINNICKINNIC RIVER IS SHOWN IN APPENDIX H

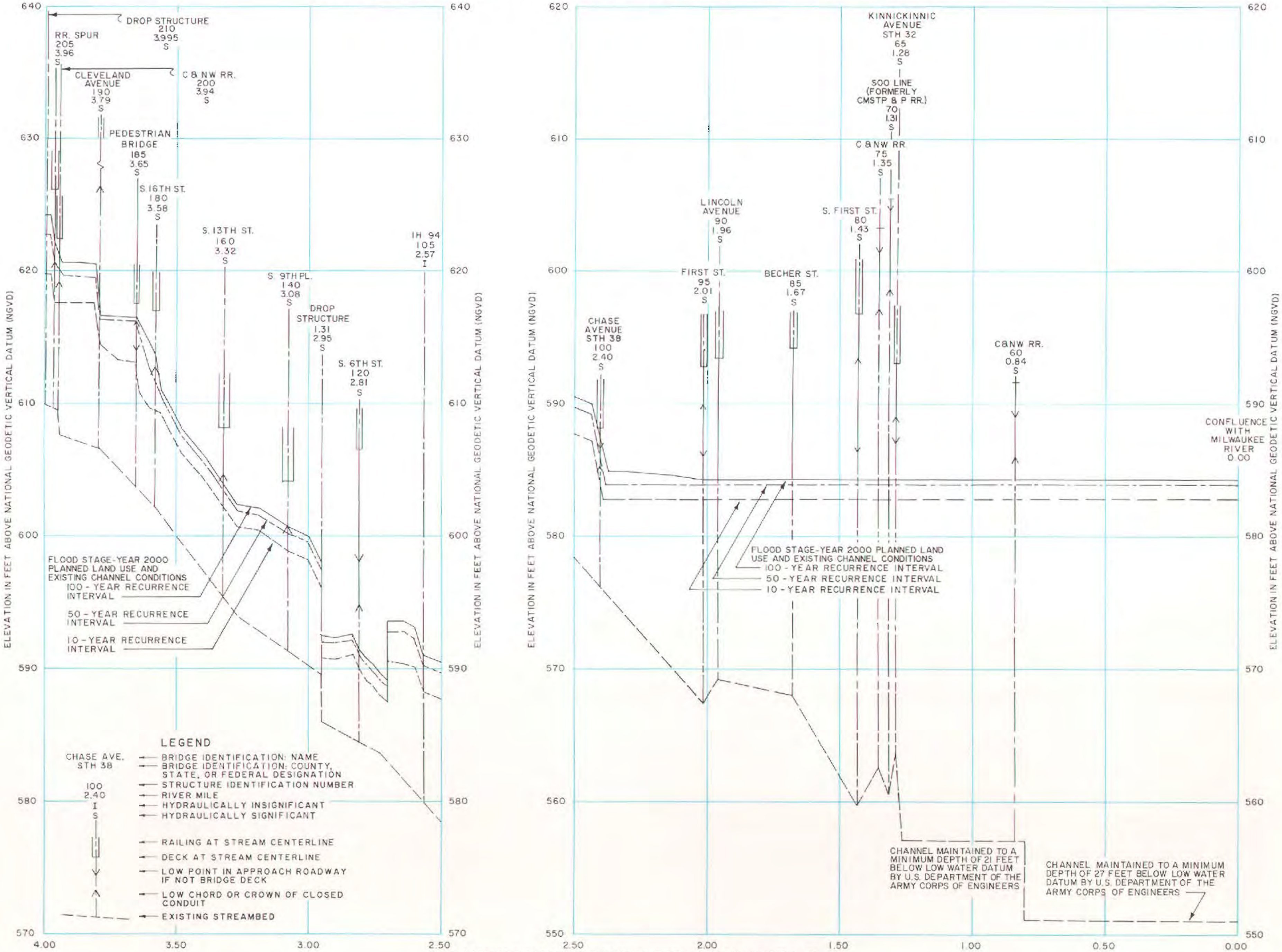
GRAPHIC SCALE

0 1/2 1 MILE

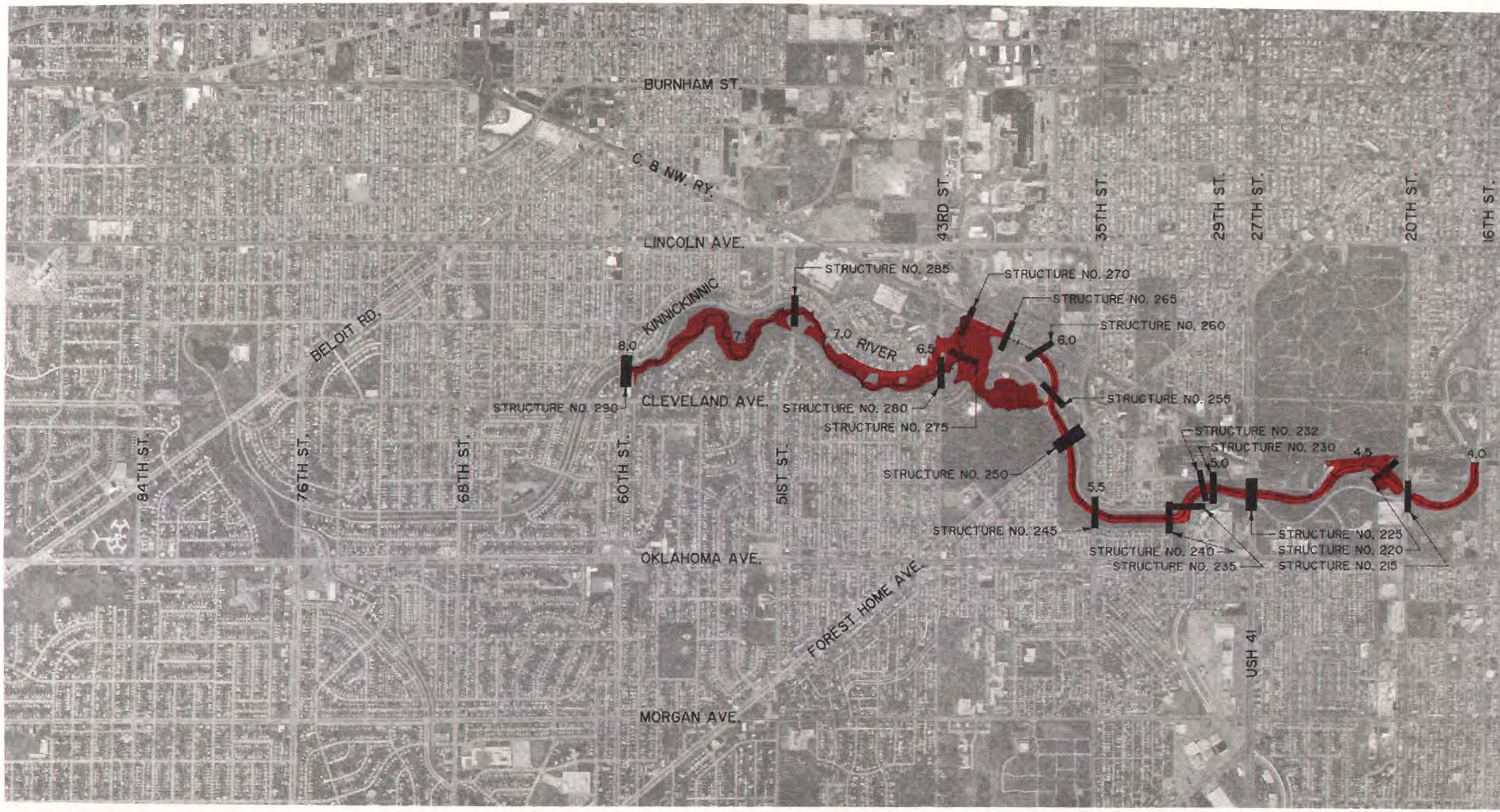
DATE OF PHOTOGRAPHY: APRIL 1986

Figure 23

FLOOD STAGE AND STREAMBED PROFILE FOR THE KINNICKINNIC RIVER



Map 29 (continued)



LEGEND

100 YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

1.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR THE KINNICKINNIC RIVER IS SHOWN IN APPENDIX H

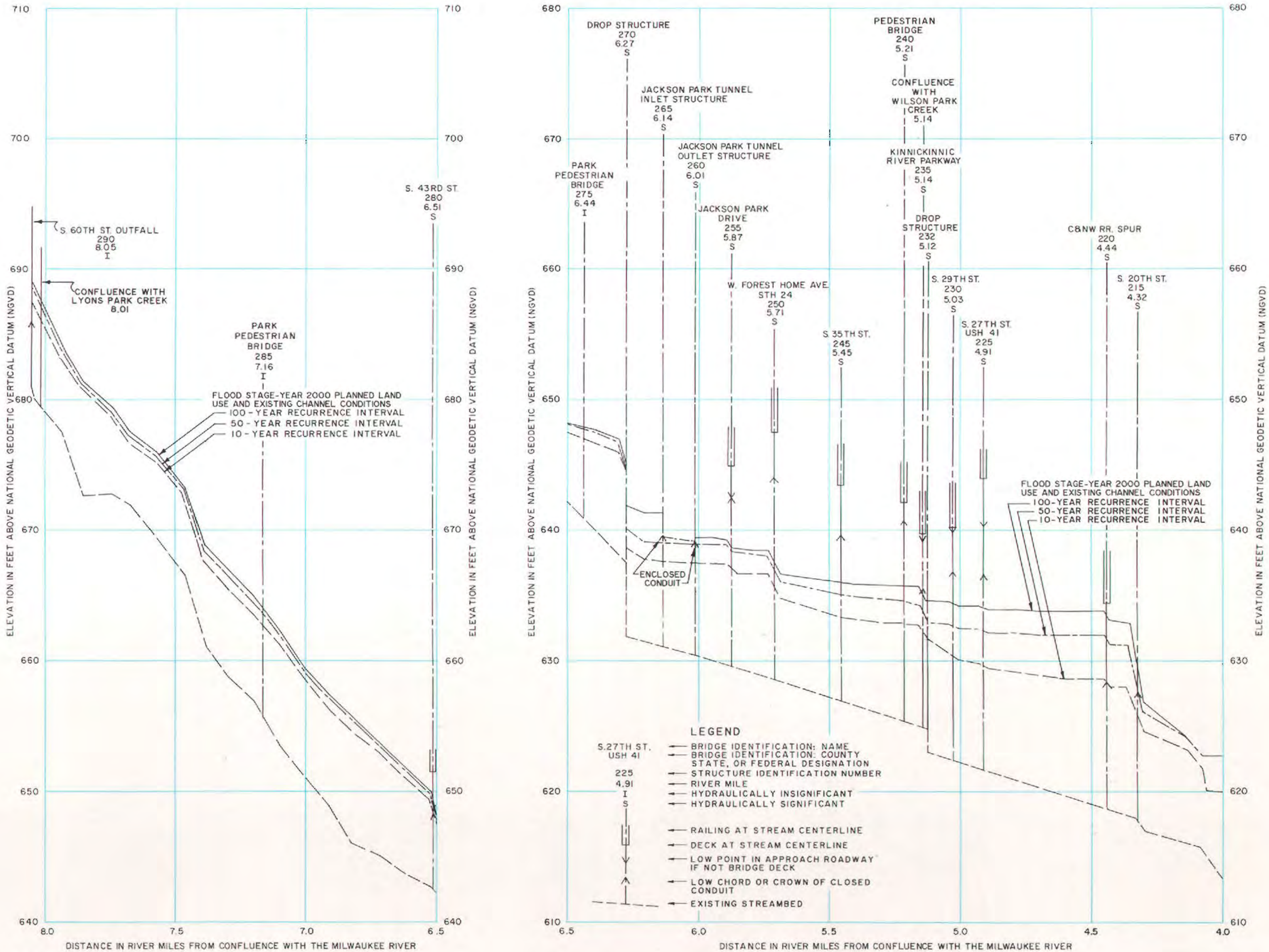
GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Source: SEWRPC.

Figure 23 (continued)



Source: SEWRPC.

are expected. Accordingly, no additional flood control or drainage system measures are recommended for the Kinnickinnic River.

As previously noted, numerous problems resulting from localized stormwater drainage were reported in the Kinnickinnic River watershed in 1986. It is recommended that a local drainage system analysis be conducted by the City of Milwaukee Engineering Department for the entire Kinnickinnic River watershed in order to more adequately define problem areas and to develop solutions to the problems. It is also recommended that replacement bridges for those structures shown in Appendix B as having inadequate hydraulic capacities be designed so as to pass the recommended design flood flow without overtopping of the attendant roadway. Such replacement is not required for flood control purposes, but rather should be carried out for transportation or other purposes.

Finally, it is recommended that large-scale topographic maps be prepared by the Milwaukee Metropolitan Sewerage District for those areas adjacent to the Kinnickinnic River. Such maps are essential in assessing the extent of flooding along the river, as well as in identifying stormwater drainage patterns and possible low-lying areas near the channel where ponding of stormwater or storm sewer surcharging could cause flood problems. Although orthophotographs prepared by the U. S. Geological Survey are available for most of the Kinnickinnic River, as well as a portion of Wilson Park Creek, they often do not show ground elevations beyond the channel banks or the immediate vicinity of the channel. Also, these orthophotographs do not reflect the channel modifications and new bridge constructed along the reach between S. 5th Street and S. 16th Street. No cost has been assigned to the flood control plan as these maps could also be used for other purposes.

LYONS PARK CREEK SUBWATERSHED FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN

Hydrologic and hydraulic analyses of Lyons Park Creek were conducted under the comprehensive watershed plan for the Kinnickinnic River prepared by the Commission in 1978. That plan also assessed existing and possible future flood problems along the stream. Flood flows and stages developed under that study have been incorporated into this system plan.

Map 30

THE LYONS PARK CREEK SUBWATERSHED

Source: SEWRPC.

Overview of the Study Area

Lyons Park Creek is a tributary of the Kinnickinnic River. The Lyons Park Creek subwatershed is located almost entirely within the City of Milwaukee, with a small portion of the subwatershed being located in the City of Greenfield. From its headwater area near S. 50th Street, Lyons Park Creek flows in a generally northerly direction for approximately 1.5 miles, and drains an area of about 1.32 square miles (see Map 30). Of this total drainage area, 1.18 square miles, or about 89 percent, lie within the City of Milwaukee, and 0.14 square mile, or about 11 percent, lies within the City of Greenfield.

More specifically, from its origin near S. 50th Street and W. Colonial Court, Lyons Park Creek flows in a generally northwesterly direction to the vicinity of S. 57th Street and W. Euclid Avenue.

From this point, the creek flows in a generally northerly direction to its confluence with the Kinnickinnic River near N. 59th Street and W. Cleveland Avenue. Of the 1.5-mile reach described, 1.3 miles, or 87 percent, is classified as perennial, while the remaining 0.2 mile, or 13 percent, is classified as intermittent. The entire perennial stream length is recommended for District jurisdiction in the policy plan companion to this system plan.

The Lyons Park Creek subwatershed is almost completely developed for urban use, including residential, commercial, institutional, and urban open space uses. The open space uses are comprised mainly of public parks. The developed areas of the Lyons Park Creek subwatershed are generally provided with a full range of municipal street improvements, including paved streets with curbs and gutters and attendant storm sewers. Accordingly, surface runoff is generally conveyed rapidly from each individual site to Lyons Park Creek through storm sewers.

Specific data on pertinent characteristics of the watershed, including hydrologic soil types, land slopes, and land use, appear in Chapter II of this report. The planned land use conditions utilized in the system planning assume that the watershed will be fully urbanized by the design year of the plan. However, existing open space uses, such as parks, will remain.

Channel improvements have been made along the entire 1.3-mile perennial stream length to accommodate the increased streamflows. The channel has been physically altered by deepening, straightening, lining with concrete or stone, construction of sills or drop spillways, and enclosure in culverts.

Flooding and Related Drainage Problems

The investigations of historical flood problems along Lyons Park Creek that were conducted under the comprehensive plan for the Kinnickinnic River watershed, as well as under this system plan, indicated few problems. Reported problems were limited to minor street flooding and isolated basement sewer backups. This lack of any serious flooding problems can be attributed, in part, to the extensive channel modifications that have been implemented along Lyons Park Creek.

The results of the hydrologic and hydraulic analyses indicate that no structure flood damages are expected to occur along Lyons Park Creek for floods up to and including the 100-year recurrence interval event under year 2000 planned land use and existing channel conditions.

Flood Discharges and Stages

As noted in Chapter III of this report, the hydrologic model used in developing design discharges for Lyons Park Creek simulates streamflow on a continuous basis using recorded climatological data as input. Flood discharges are developed by conducting discharge-frequency analyses of simulated annual peak discharges generated by the hydrologic model according to the log Pearson Type III method of analysis, as recommended by the U. S. Water Resources Council and as specified by the Wisconsin Department of Natural Resources. These analyses were conducted for planned land use and channel conditions at two locations along Lyons Park Creek. The flood discharges that were developed were then checked by incorporating the discharges into a hydraulic model to develop stages and comparing those stages to available historical high water mark data. Such data were available for three locations on the channel from the City Engineer of the City of Milwaukee.

The estimated peak flood discharges under year 2000 planned land use and existing (1986) channel conditions are set forth in Table 25. Flood stage profiles were determined for the 10-, 50-, and 100-year recurrence interval runoff events under planned land use and existing channel conditions. These profiles, which encompass the full 1.3-mile-long reach of Lyons Park Creek studied, constitute a graphic representation of the flood stages along Lyons Park Creek under the specified recurrence interval flood discharges. In addition to providing an overall representation of flood stages relative to familiar points of reference, such as the channel bottom and bridge deck surfaces, the profiles, because of their continuity, permit the determination of flood stages at any point along the stream channel. The flood profiles are shown in Figure 24. The extent of the 100-year recurrence interval floodplain under planned land use conditions is shown on Map 31.

Recommended Flood Control System for Lyons Park Creek

As previously noted, no structure flood damages are expected along Lyons Park Creek for floods up to and including the 100-year recurrence interval event under year 2000 planned land use and

Table 25

FLOOD DISCHARGES FOR LYONS PARK CREEK FOR YEAR 2000 LAND USE AND EXISTING CHANNEL CONDITIONS

Location	River Mile	Peak Flood Discharge (cubic feet per second)		
		10-Year	50-Year	100-Year
Confluence with Kinnickinnic River	0.00	670	980	1,150
Parking Lot Tunnel Outlet	0.06	670	980	1,150
W. Cleveland Avenue Tunnel Inlet	0.12	670	980	1,150
Pedestrian Bridge	0.20	670	980	1,150
W. Stack Drive	0.36	670	980	1,150
W. Bennett Avenue Tunnel Outlet	0.54	670	980	1,150
W. Lakefield Drive Extension Tunnel Inlet	0.70	670	980	1,150
S. 57th Street	0.84	670	980	1,150
Pedestrian Bridge	0.87	670	980	1,150
Pedestrian Bridge	0.89	670	980	1,150
Pedestrian Bridge	1.07	670	980	1,150
S. 55th Street	1.17	670	980	1,150
W. Forest Home Avenue	1.31	475	640	710

Source: SEWRPC.

existing channel conditions. Minor overland flooding of a segment of one collector street—W. Stack Drive—may be expected to result from storm sewer surcharging, however. Any overbank flooding of Lyons Park Creek would be limited to that reach through Lyons Park.

Because of the lack of structure flood damages and only minor road flooding, no flood control or drainage alternatives were considered for Lyons Park Creek. Accordingly, no flood control or drainage system plans are recommended for Lyons Park Creek.

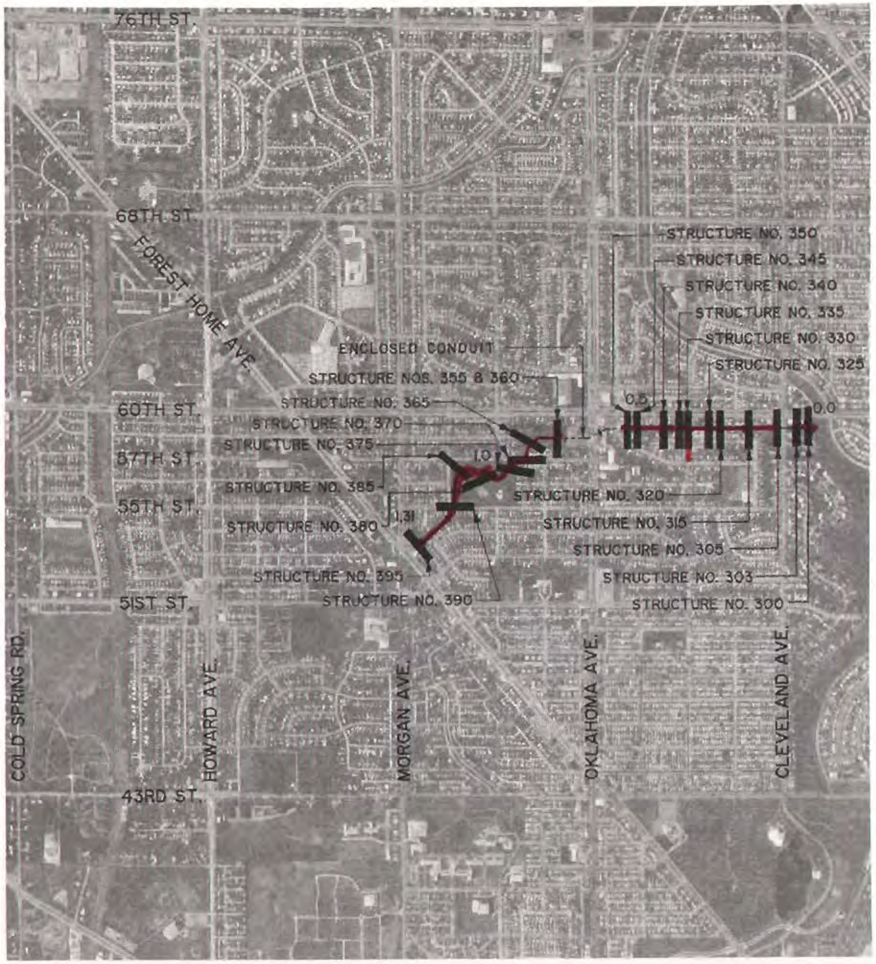
Because of a lack of large-scale topographic maps for Lyons Park Creek, the floodplain shown on Map 31 can be considered only an approximation. This delineation was based on field-surveyed cross-sections supplemented by construction drawings for the channel modifications. In order to provide a more precise delineation of the Lyons Park Creek floodplain, it is recommended that the Milwaukee Metropolitan Sewerage District prepare large-scale topographic maps for the floodplain areas along this stream. Because these maps would have multiple uses, no cost has been assigned to the flood control plan.

WILSON PARK CREEK SUBWATERSHED FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN

Flood control improvements for Wilson Park Creek were considered in a comprehensive watershed plan prepared by the Commission in December 1978. Alternative flood control measures developed under that study were limited to that portion of Wilson Park Creek—also known as the Edgerton Channel—that is located upstream of Milwaukee General Mitchell International Airport in the City of Cudahy. Plans developed by the Milwaukee Metropolitan Sewerage District that called for major channel modifications to Wilson Park Creek between W. Euclid Avenue and S. 6th Street were considered as being committed, and therefore no alternative flood control measures were considered for that reach. As of December 1986, the District had completed the channel modifications between W. Euclid Avenue and S. 20th Street. The remaining 1.3 miles of channel modifications between S. 20th Street and S. 6th Street had not been implemented. This includes a 0.4-mile reach through Wilson Park. Local residents have expressed concerns to the District that the proposed channel modifications would have an adverse impact on a

Map 31

100-YEAR RECURRENCE INTERVAL FLOODPLAIN FOR LYONS PARK CREEK UNDER YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS



LEGEND

100 YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

1.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR LYONS PARK CREEK IS SHOWN IN APPENDIX H

GRAPHIC SCALE

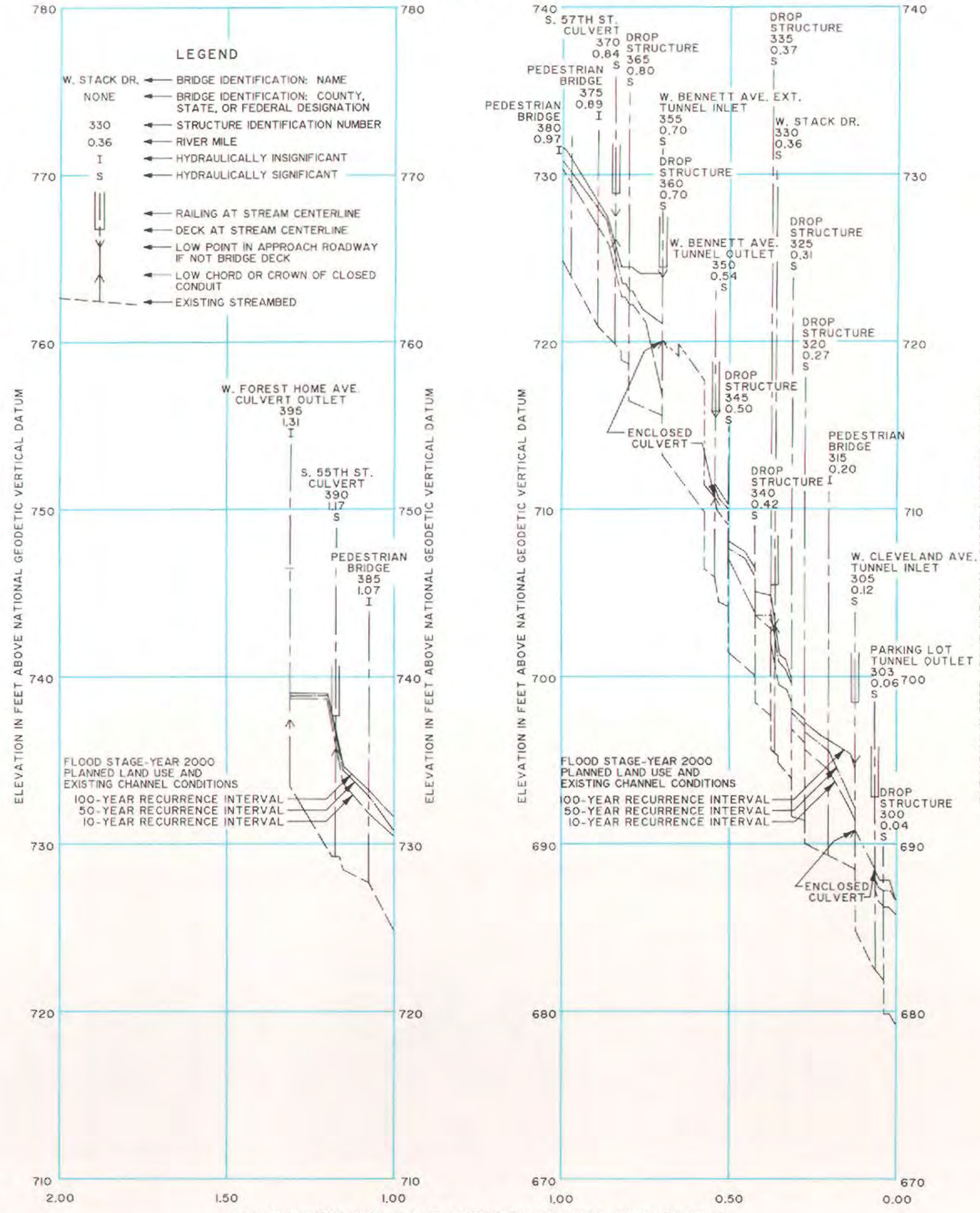
0 1/2 1 MILE

DATE OF PHOTOGRAPHY APRIL 1986

Source: SEWRPC.

Figure 24

FLOOD STAGE AND STREAMBED PROFILE FOR LYONS PARK CREEK



Source: SEWRPC.

resident population of ducks that nest in the park. Accordingly, the District requested that the Commission consider alternative flood control measures for Wilson Park Creek between S. 20th Street and S. 6th Street. Also, as a result of the severe flood damages that occurred along the Kinnickinnic River during the August 6, 1986, flood event, the City of Milwaukee Common Council adopted a resolution requesting the District to consider diverting all, or part, of the flow in Wilson Park Creek to Lake Michigan in order to reduce flood flows along the Kinnickinnic River. Accordingly, alternative diversion measures were investigated as part of this system plan and are presented herein.

Overview of the Study Area

Wilson Park Creek is a tributary of the Kinnickinnic River. The Wilson Park Creek subwatershed is located almost entirely within the City of Milwaukee. Small portions of the subwatershed are located in the Cities of Cudahy, Greenfield, and St. Francis. From its headwater area near S. Whitnall Avenue, Wilson Park Creek flows in a generally northwesterly direction for approximately 6.1 miles, and drains an area of 11.19 square miles (see Map 32). Of this total drainage area, 7.58 square miles, or about 68 percent, lie within the City of Milwaukee; 1.47 square miles, or about 13 percent, lie within the City of Cudahy; 2.02 square miles, or 18 percent, lie within the City of Greenfield; and 0.12 square mile, or about 1 percent, lies within the City of St. Francis.

More specifically, from its origin near S. Whitnall Avenue, Wilson Park Creek flows in a generally westerly direction to the vicinity of S. Howell Avenue and W. Layton Avenue. From this point, the creek flows in a generally northwesterly direction to W. Morgan Avenue, and thence in a generally northerly direction to its confluence with the Kinnickinnic River near S. 30th Street and W. Oklahoma Avenue. Of the 6.1-mile reach described, 5.2 miles, or 85 percent, is classified as perennial, while the remaining 0.9 mile, or 15 percent, is classified as intermittent. As already noted, the entire 6.1-mile stream length is recommended for District jurisdiction in the policy plan companion to this system plan.

The Wilson Park Creek subwatershed is almost completely developed for urban use, including residential, commercial, industrial, institutional, and urban open space uses. The open space uses are comprised mainly of public parks and cemeteries. The developed areas of the Wilson Park Creek subwatershed are generally provided with a full range of municipal street improvements, including paved streets with curbs and gutters and attendant storm sewers. Accordingly, surface runoff is generally conveyed rapidly from each individual site to Wilson Park Creek through storm sewers.

Specific data on pertinent characteristics of the watershed, including hydrologic soil types, land slopes, and land use, appear in Chapter II of this report. The planned land use conditions utilized in the system planning assume that the watershed will be fully urbanized by the design year of the system plan. However, existing open space uses, such as parks and cemeteries, will remain.

Flooding, in various degrees, is a common occurrence adjacent to the Wilson Park Creek and the Edgerton Channel. Flooding along the creek has increased proportionally to the degree of conversion of land from open, rural uses to urban uses. Channel improvements have been made along the entire stream length to accommodate the increased streamflows. The channel has been physically altered by deepening, straightening, lining with concrete or stone, construction of sills or drop spillways, and enclosure in culverts.

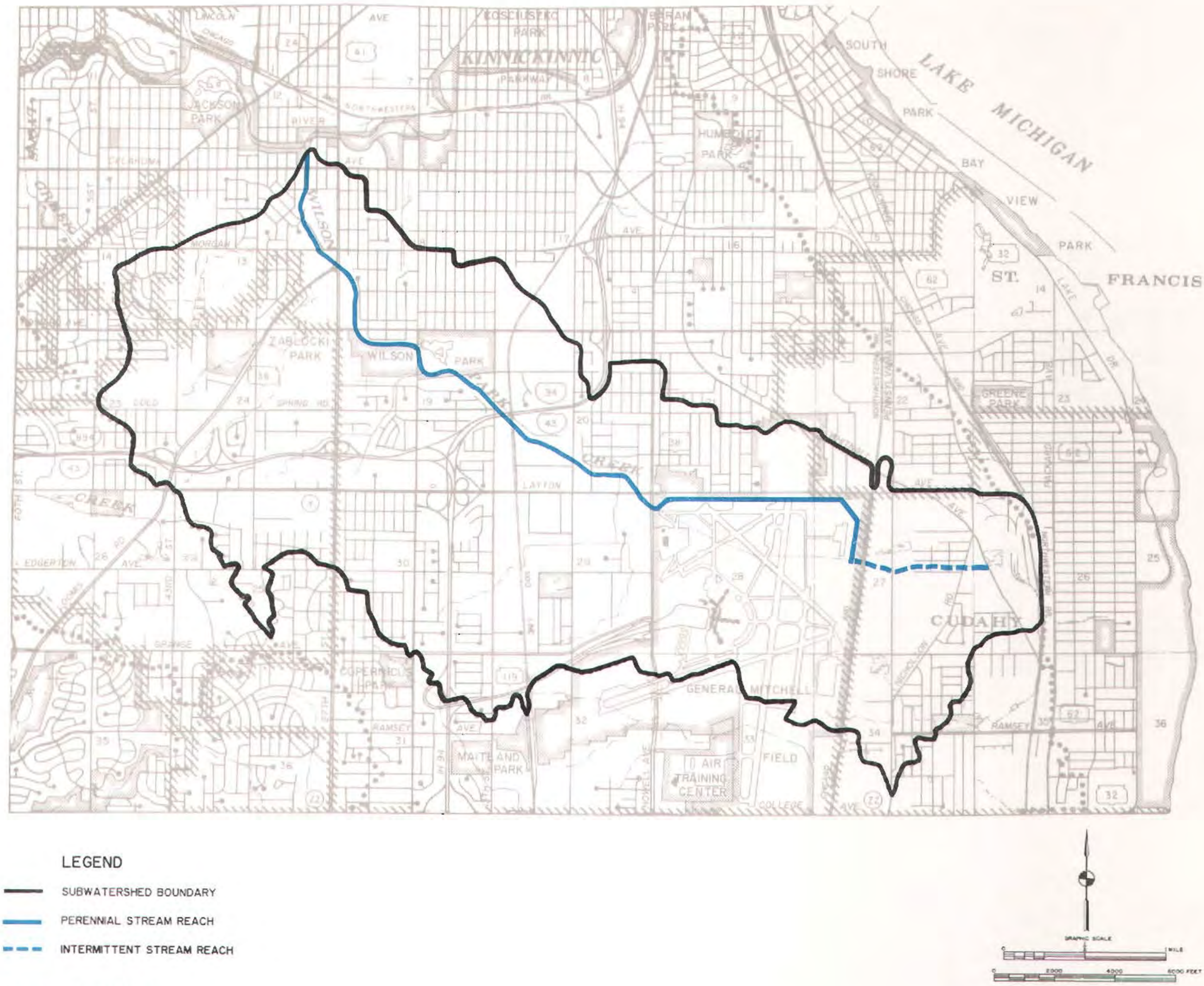
Flooding and Related Drainage Problems

As already noted, flooding, including first-floor flooding, yard flooding, and basement flooding, occurs frequently within the Wilson Park Creek subwatershed.

During 1986 alone, there were three storm events for which flooding and water-related problems were reported, based upon records maintained by the City Engineer of the City of Milwaukee. Map 33 shows those areas for which problems were reported. More than 130 flooding problems were documented during 1986, with the most—115—being reported during the August 6 rainfall. Additional areas in which flooding problems have occurred—including areas along the Edgerton Channel—were documented in the Kinnickinnic River watershed study and are also shown on Map 33. Flooding of roadways and underpasses has also occurred frequently in the watershed. It should be noted that the flood control measures considered under this system plan are aimed primarily at alleviating flood damages from direct overland flooding along the stream studied, as well as at providing an adequate outlet for local storm sewers. These measures will help to reduce, but will not necessarily eliminate, flooding due to localized stormwater drainage problems or sanitary sewer backups.

Map 32

THE WILSON PARK CREEK SUBWATERSHED



Source: SEWRPC.

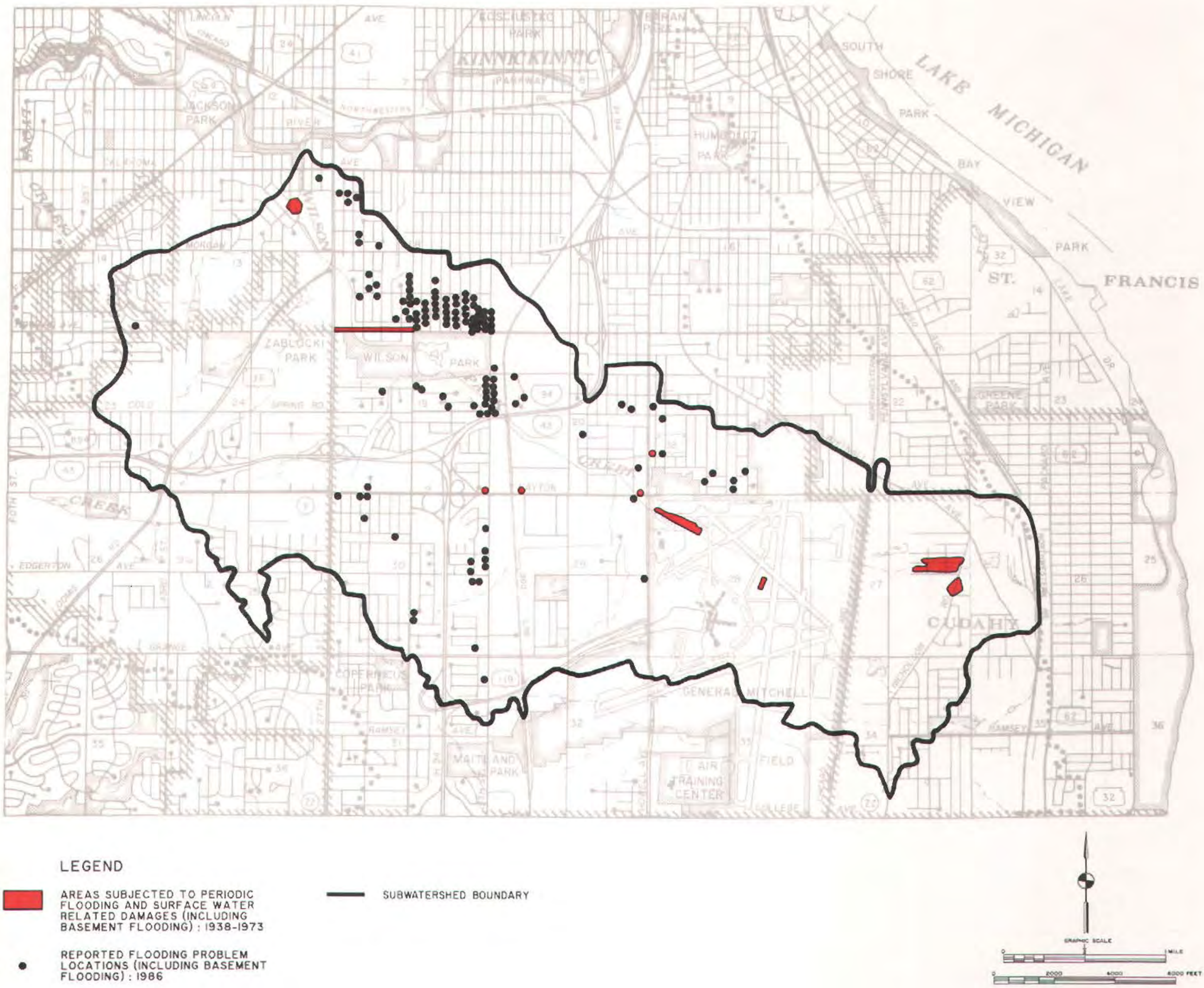
The costs of flooding were estimated using damage cost curves prepared by the Commission and described in Chapter III. The dollar amount of the flood damages is based upon the depth of inundation and the assessed valuation of the building. Damages to building contents are included in the total costs.

Flooding, as defined herein, includes basement flooding, yard inundation, and flooding above the first-floor level. The total number of existing residences that may be expected to experience direct flooding along Wilson Park Creek in the City of Milwaukee is as follows:

Flood Event Recurrence Interval	Number of Existing Homes Flooded Year 2000 Land Use Conditions
100	36
50	26
10	0

Map 33

AREAS WITH REPORTED FLOODING AND DRAINAGE PROBLEMS IN THE WILSON PARK CREEK SUBWATERSHED



Source: SEWRPC.

The number of existing industrial and commercial properties that may be expected to experience direct flooding along Wilson Park Creek in the City of Milwaukee is as follows:

Flood Event Recurrence Interval	Number of Existing Industrial and Commercial Properties Flooded Year 2000 Land Use Conditions
100	8
50	5
10	0

Additional homes and industrial and commercial properties may be expected to experience indirect flood damages through sewer backup. The eight commercial and industrial buildings that are expected to experience flood damage include De Paul Rehabilitation Hospital. This facility incurred extensive flood damage as a result of the August 6, 1986, storm event, including the flooding of a cafeteria area, numerous offices, and a gymnasium, and the loss of the hospital telephone system and portions of the electrical system. All of these facilities were located in the basement or subbasement levels of the hospital.

The total average annual flood losses—damages—for Wilson Park Creek in the City of Milwaukee are estimated at $39,000 under year 2000 planned land use and existing channel conditions. Flood losses of about $900,000 may be expected from a 100-year recurrence interval flood under year 2000 planned land use and existing channel conditions.

The total number of existing residences that may be expected to experience direct flooding along Wilson Park Creek-Edgerton Channel in the City of Cudahy is as follows:

Flood Event Recurrence Interval	Number of Existing Homes Flooded Year 2000 Land Use Conditions
100	177
50	175
10	140

The total number of existing industrial and commercial properties that may be expected to experience direct flooding along Wilson Park Creek-Edgerton Channel in the City of Cudahy is as follows:

Flood Event Recurrence Interval	Number of Existing Industrial and Commercial Properties Flooded Year 2000 Land Use Conditions
100	5
50	5
10	3

Additional homes and industrial and commercial properties be expected to experience indirect flood damages through sewer backup.

The total average annual flood losses—damages—for Edgerton Channel in the City of Cudahy are estimated at $142,100 under year 2000 planned land use and existing channel conditions. Flood losses of about $486,000 may be expected from a 100-year recurrence interval flood under year 2000 planned land use and existing channel conditions.

The drainage and flood control objectives and supporting principles and standards set forth in Chapter III specify the flood events which bridges shall accommodate without overtopping the related roadway. Based on those criteria, three bridges are considered hydraulically inadequate, as shown in Appendix B. These three structures are at S. Pennsylvania, S. Nicholson, and S. Whitnall Avenues.

Flood Discharges and Stages

As noted in Chapter III of this report, the hydrologic model used in developing design discharges for Wilson Park Creek simulates streamflow on a continuous basis using recorded climatological data as input. Flood discharges are developed by conducting discharge-frequency analyses of simulated annual peak discharges generated by the hydrologic model according to the log Pearson Type III method of analyses, as recommended by the U. S. Water Resources Council[5] and as specified by the Wisconsin Department of Natural Resources.[6] These analyses were conducted for both existing and planned land use and channel conditions at six locations along Wilson Park Creek. The flood discharges that were developed were then checked by incorporating the discharges into a hydraulic model to develop stages and comparing those stages to available historical high-water-mark data. Such data were available for three locations on the channel from the City Engineer of the City of Milwaukee, and for six locations on the channel from the Milwaukee Metropolitan Sewerage District.

The flood discharges presented in the Kinnickinnic River watershed study for planned land use and existing channel conditions were developed with the assumption that the District's plans for major channelization along Wilson Park Creek between W. Euclid Avenue and S. 6th Street would be fully implemented. As previously noted, as of December 1986, this channelization had been completed except for a 1.3-mile reach between S. 6th Street and S. 20th Street. Therefore, for purposes of this system plan, a new set of flood flows and stages was developed for Wilson Park Creek assuming year 2000 planned land use and existing (1986) channel conditions. This change in channel conditions along Wilson Park Creek did not produce any significant change in the discharges for the Kinnickinnic River presented in the Kinnickinnic River watershed study. Therefore, planned land use and existing channel flood flows and stages were revised only for Wilson Park Creek.

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[5] *United States Water Resources Council, loc. cit.*

[6] *"Wisconsin's Floodplain Management Program," Wisconsin Administrative Code, Chapter NR 116, February 1986.*

Table 26

FLOOD DISCHARGE FOR WILSON PARK CREEK—EDGERTON CHANNEL FOR YEAR 2000 LAND USE AND EXISTING CHANNEL CONDITIONS

Location	River Mile	Peak Flood Discharge (cubic feet per second)		
		10-Year	50-Year	100-Year
Mouth at Kinnickinnic River	0.00	1,970	2,730	3,070
W. Oklahoma Avenue Tunnel Outlet	0.05	1,970	2,730	3,070
W. Euclid Avenue Tunnel Inlet	0.32	1,970	2,730	3,070
W. Lakefield Drive	0.49	1,970	2,730	3,070
W. Morgan Avenue Tunnel Outlet	0.68	1,670	2,310	2,600
S. 27th Street Tunnel Inlet	0.87	1,670	2,310	2,600
W. Howard Avenue	1.30	1,670	2,310	2,600
S. 20th Street	1.70	1,670	2,310	2,600
Pedestrian Bridge	1.83	1,670	2,310	2,600
S. 13th Street	2.42	1,250	1,690	1,880
IH 94	2.50	1,250	1,690	1,880
Soo Line Railroad	2.57	1,250	1,690	1,880
S. 6th Street	3.03	1,250	1,690	1,880
S. 5th Street	3.18	1,250	1,690	1,880
W. Layton Avenue Tunnel Outlet	3.51	520	660	710
S. Howell Avenue Tunnel Inlet	3.65	520	660	710
Airport Tunnel Outlet	3.86	520	660	710
Airport Tunnel Inlet	4.76	400	550	620
Airport Service Road	4.96	400	550	620
Drop Structure	5.28	400	550	620
Chicago & North Western Railway	5.34	400	550	620
Utility Lane	5.36	400	550	620
S. Pennsylvania Avenue	5.54	350	450	510
Frontage Road	5.98	350	450	510
S. Nicholson Avenue	5.99	350	450	510
S. Whitnall Avenue	6.12	350	450	510

Source: SEWRPC.

The estimated peak flood discharges under year 2000 planned land use and existing (1986) channel conditions are set forth in Table 26. Flood stage profiles were determined for the 10-, 50-, and 100-year recurrence interval runoff events under planned land use and existing channel conditions. These profiles, which encompass the full 6.1-mile-long reach of Wilson Park Creek studied, constitute a graphic representation of the flood stages along Wilson Park Creek under the specified recurrence interval flood discharges. In addition to providing an overall representation of flood stages relative to familiar points of reference such as the channel bottom and bridge deck surfaces, the profiles, because of their continuity, permit the determination of flood stages at any point along the stream channel. The flood profiles are shown in Figure 25. The extent of the 100-year recurrence interval floodplain for Wilson Park Creek under planned land use and existing channel conditions is shown on Map 34.

Alternative Flood Control and Related Drainage System Plans for Wilson Park Creek in the City of Milwaukee

In the preparation of this system plan, six alternative flood control plans were considered for alleviating the flood damage problems along Wilson Park Creek in the City of Milwaukee: 1) Alternative System 1—no action; 2) Alternative System 2—structure floodproofing and elevation; 3) Alternative System 3—major channelization; 4) Alternative System 4—major channelization with concrete lining through Wilson Park; 5) Alternative System 5—bridge replacement with limited channelization; and 6) Alternative System 6—diversion.

Table 27

COST ESTIMATES FOR FLOOD CONTROL ALTERNATIVES FOR WILSON PARK CREEK IN THE CITY OF MILWAUKEE

			Annual Costs				Benefit-Cost Analysis			
Alternative	Description	Capital Cost	Amortized Capital[a]	Operation and Maintenance	Other	Total	Annual Benefits	Annual Benefits Minus Annual Costs	Benefit-Cost Ratio	Economic Ratio Greater than One
1. No Action	--	$ 0	$ 0	$ 0	$39,000	$ 39,000	$ 0	$ -39,000	--	No
2. Structure Floodproofing and Elevation	a. Floodproof up to 36 residential and 8 industrial and commercial structures	$ 1,185,000	$ 79,000	$ --	$ --	$ 79,000	$39,000	$ -40,000	0.49	No
	b. Elevate 2 residential structures	62,000								
	Subtotal	$ 1,247,000								
3. Major Channelization	a. 1.3 miles of major channelization	$ 554,000	$ 37,000	$ 2,600	$ --	$ 39,600	$39,000	$ -600	0.98	No
	b. Modification of pedestrian bridge	30,000								
	Subtotal	$ 584,000								
4. Major Channelization with Concrete Lining through Wilson Park	a. 0.9 mile of channelization with turf lining	$ 333,000	$ 79,400	$ 2,600	$ --	$ 82,000	$39,000	$ -43,000	0.48	No
	b. 0.4 mile of channelization with concrete lining	888,000								
	c. Modification of pedestrian bridge	30,000								
	Subtotal	$ 1,251,000								
5. Bridge Replacement with Limited Channelization	a. Replacement of one bridge	$ 778,000	$ 60,400	$ 2,600	$ --	$ 63,000	$39,000	$ -24,000	0.62	No
	b. 1.3 miles of limited channelization	166,000								
	Subtotal	$ 944,000								
6. Diversion	18,350 feet of 13-foot-diameter diversion tunnel	$48,307,000	$3,063,000	$18,000	$ --	$3,081,000	$39,000	$-3,042,000	0.01	No
7. Combination of Major Channelization and Channel Enclosure	a. 0.93 mile of channelization with truf lining	$ 342,000	$ 299,800	$ 4,200	$ --	$ 304,000	$39,000	$ -265,000	0.13	No
	b. 1,450 feet of channel enclosure	4,382,000								
	Subtotal	$ 4,724,000								
8. Detention Storage	a. 65 acre-foot detention basin	$ 945,000	$ 91,000	$26,000	$ --	$ 117,000	$39,000	$ -78,000	0.33	No
	b. Culverts under railroad spur	48,000								
	c. Land acquisition	443,000								
	Subtotal	$ 1,436,000								

[a] *Amortized capital cost is based on an interest rate of 6 percent and a project life of 50 years.*

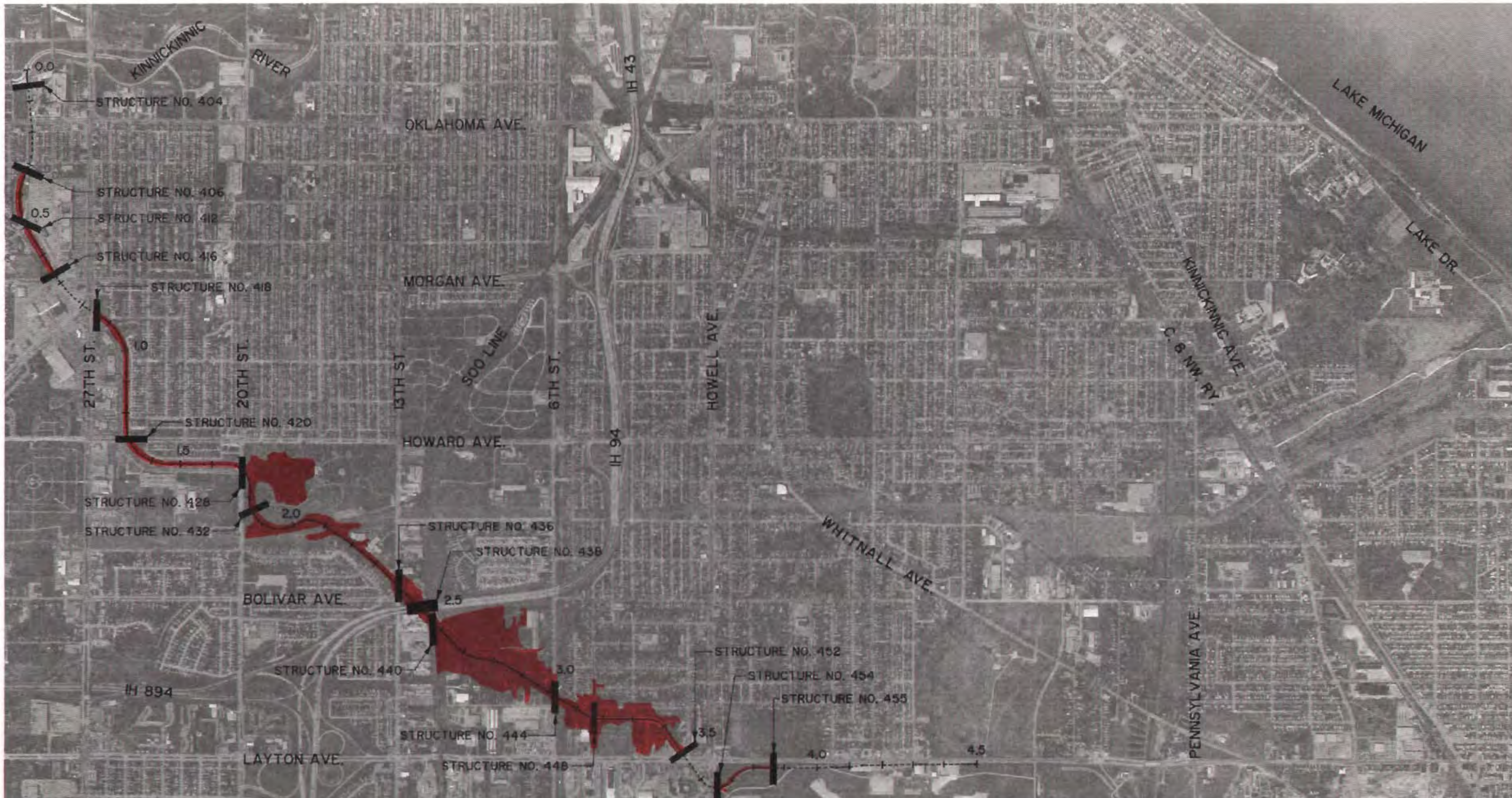
Source: SEWRPC.

Each alternative is described below. The economic benefits and costs attendant to the alternatives are provided in Table 27.

Alternative System Plan 1—No Action: One alternative course of action to alleviate the flood problem along Wilson Park Creek in the City of Milwaukee is to do nothing—that is, to recognize the inevitability of extensive flooding but to decide not to mount a collective, coordinated program to abate the flood damages. Under year 2000 planned land use and existing channel conditions, the average annual flood damages along this reach would approximate $39,000. There are no monetary benefits associated with this alternative, and the average annual cost would be equivalent to the average annual flood damage cost of $39,000. The estimated damages associated with the 100-year recurrence interval flood under year 2000 planned land use and existing channel conditions total $900,000.

Map 34

100-YEAR RECURRENCE INTERVAL FLOODPLAIN FOR WILSON PARK CREEK—EDGERTON CHANNEL UNDER YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS



LEGEND

100 YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

1.0 — APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR WILSON PARK CREEK IS SHOWN IN APPENDIX H

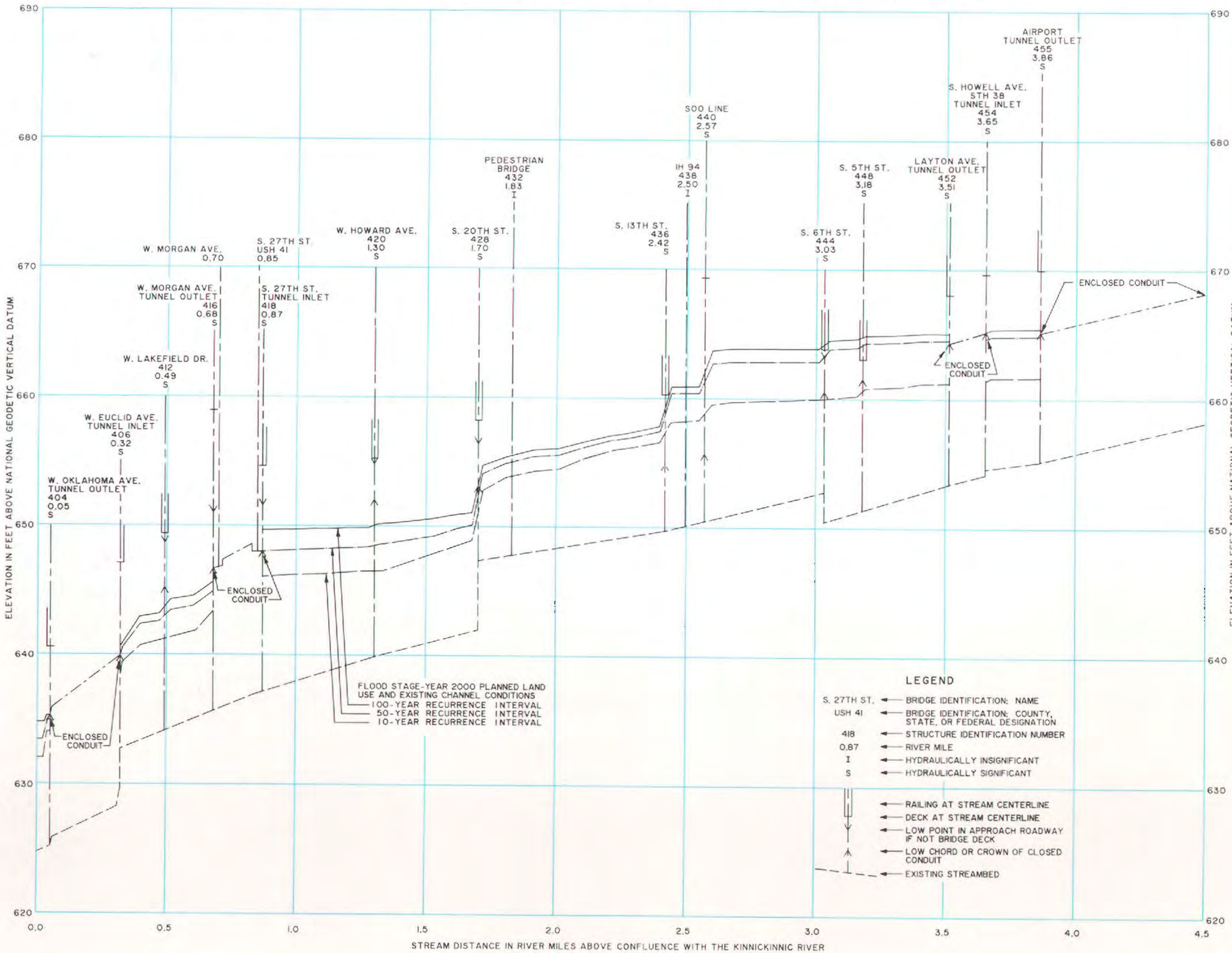
GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Figure 25

FLOOD STAGE STREAMBED PROFILE FOR WILSON PARK CREEK—EDGERTON CHANNEL



Map 34 (continued)

LEGEND

100 YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

1.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR WILSON PARK CREEK IS SHOWN IN APPENDIX H

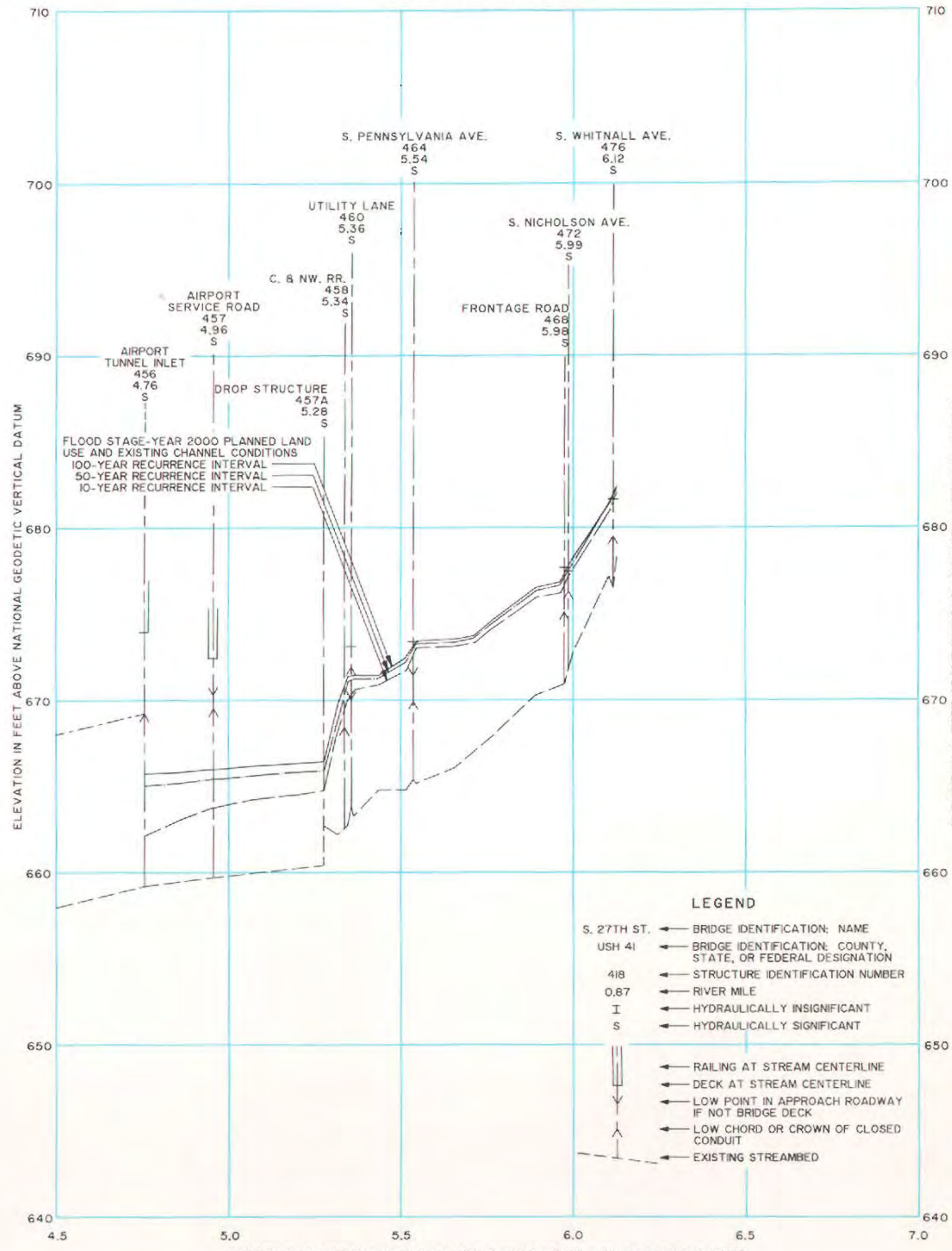
GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Source: SEWRPC.

Figure 25 (continued)



Source: SEWRPC.

Alternative System Plan 2—Structure Floodproofing and Elevation, and Removal: A structure floodproofing and elevation alternative flood control system was analyzed to determine if such a structure-by-structure approach would be a technically feasible and economically viable solution to the flood problem along Wilson Park Creek in the City of Milwaukee. For analytical purposes, the 100-year recurrence interval flood stage under year 2000 planned land use and existing channel conditions was used to estimate the number of existing flood-prone structures to be floodproofed, elevated, or removed, and the costs involved.

In the case of residential structures, floodproofing was assumed to be feasible if the design flood stage was below the first-floor elevation. Structure elevation was considered feasible for residential structures with basements if the estimated cost of elevating the structure was less than the estimated removal cost. Structures to be elevated were assumed to have the first-floor raised to an elevation of at least two feet above the 100-year recurrence interval flood stage to provide adequate freeboard. For aesthetic reasons, structure elevation was limited to a maximum of four feet. Structures that would have to be elevated more than four feet were considered for removal.

Floodproofing was considered to be feasible for all nonresidential structures provided the flood stage was not more than seven feet above the first-floor elevation. The floodproofing costs were assumed to be a function of the depth of water over the first floor.

As shown on Map 35, of the 44 structures that may be expected to incur flood damage, 42 would have to be floodproofed and two would have to be elevated. No structures would need to be removed. Damage from floods up to and including the 100-year recurrence interval event would be virtually eliminated.

Assuming that these structure floodproofing measures would be fully implemented, and utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of this alternative is estimated at $79,000. This cost consists of the amortization of the $1,247,000 capital cost—$1,185,000 for floodproofing and $62,000 for structure elevation. The average annual flood damage abatement benefit was estimated at $39,000, yielding a benefit-cost ratio of 0.49.

Map 35

ALTERNATIVE PLAN 2: STRUCTURE FLOODPROOFING AND ELEVATION ALONG WILSON PARK CREEK IN THE CITY OF MILWAUKEE

Source: SEWRPC.

Alternative System Plan 3—Major Channelization: This alternative consists of completing the major channel modifications along the 1.3-mile reach between S. 6th Street and S. 20th Street, as shown on Map 36. These channel modifications are the same as those that have been proposed by the Milwaukee Metropolitan Sewerage District and were included in the Kinnickinnic River watershed study with the exception that the resulting channel would have a turf lining as opposed to a concrete lining.

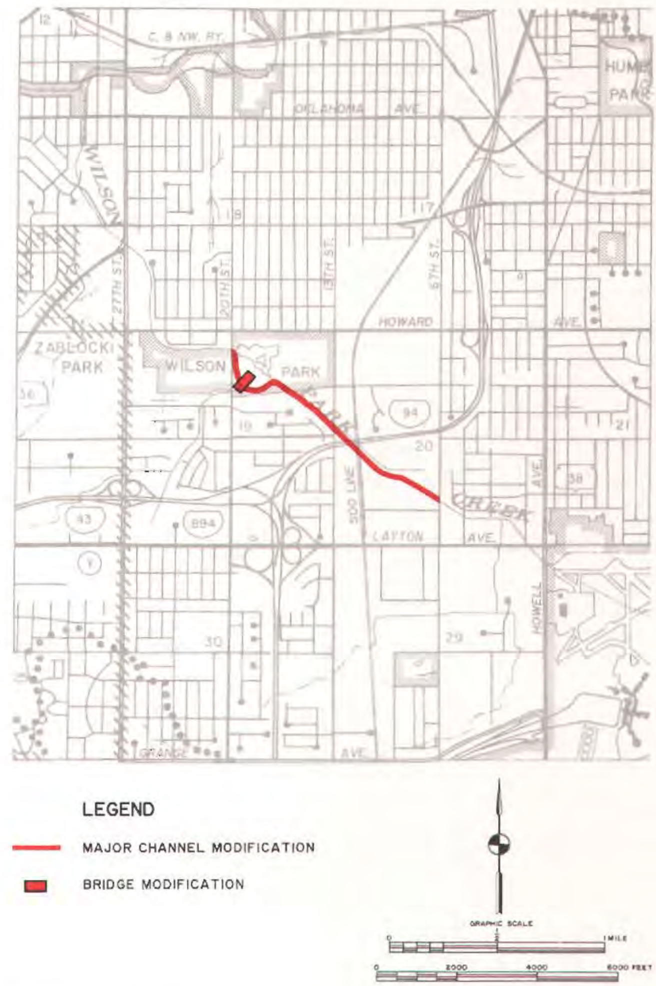
Under this alternative, the streambed would be lowered from 2.3 to 5.4 feet in order to match the existing streambed along those reaches for which channel modifications have already been completed. The proposed channel would have a bottom width of 20 feet and side slopes of one on four. The resulting top width would average 110 feet. Modification of the bridge piers for the one pedestrian bridge in Wilson Park would be required. No other bridge reconstruction or modification would be required since the bridges within this stream reach have already been constructed to match the proposed channel invert.

Implementation of this alternative would eliminate all structure flood damages due to overland flooding and storm sewer surcharging for floods up to and including the 100-year recurrence interval event.

Utilizing an annual interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this major channelization alternative is estimated at $39,600. This cost consists of the amortization of the $554,000 capital cost of the channel modification, the $30,000 cost for modification of one pedestrian bridge, and $2,600 in annual operation and maintenance costs. The proposed channel is located entirely within either a drainage right-of-way that is owned by the District or a county-owned park, and therefore no land acquisition costs have been included. The average annual flood abatement benefit is estimated at $39,000, resulting in a benefit-cost ratio of 0.98.

Map 36

ALTERNATIVE PLAN 3: MAJOR CHANNELIZATION ALONG WILSON PARK CREEK IN THE CITY OF MILWAUKEE



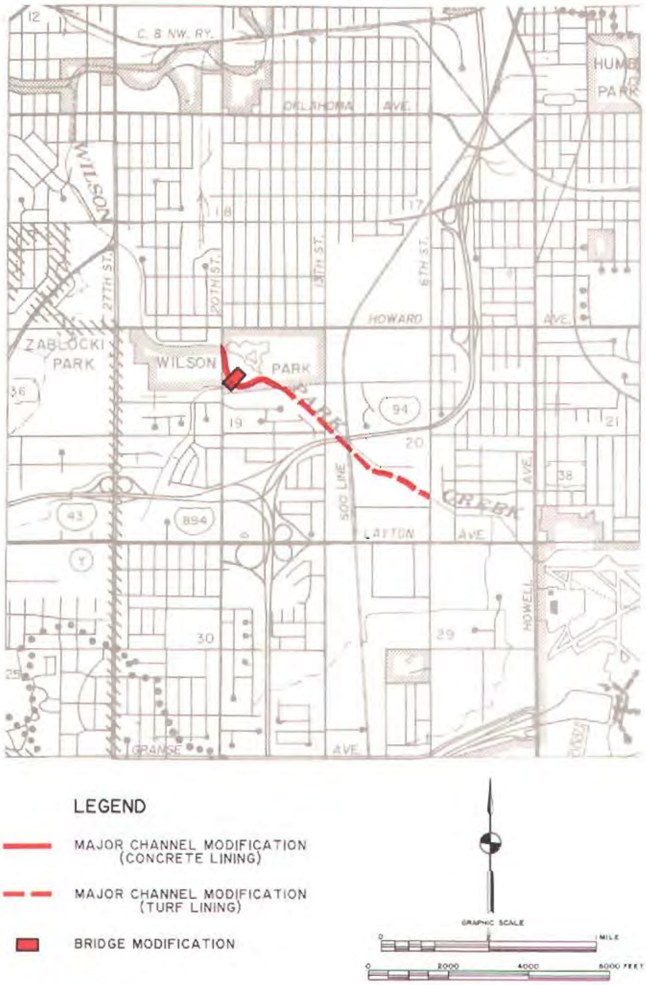
Source: SEWRPC.

<u>Alternative System Plan 4—Major Channelization with Concrete Lining Through Wilson Park:</u> This alternative flood control system is similar to Alternative System 3 in that it includes major channel modifications along the 1.3-mile reach between S. 6th Street and S. 20th Street, as shown on Map 37. These channel modifications are different, however, for the 0.4-mile reach through Wilson Park in that the proposed channel would be fully concrete lined, which would permit the use of a steeper side slope and, accordingly, would result in a narrower channel cross-section. This narrower channel would reduce the number of trees that would need to be removed from the park as a result of the channelization. Current practice of the Milwaukee County Parks Department requires compensation for county-owned trees that are removed by other agencies for public works construction.

Under this alternative, the streambed would be lowered from 2.3 to 5.4 feet in order to match the existing streambed at S. 6th Street and at S. 20th Street. For the 0.9-mile reach between Wilson Park and S. 6th Street, the proposed channel would be turf lined and would have a bottom width of 20 feet and side slopes of one on four. The resulting top width would average 110 feet. For the 0.4-mile reach through Wilson Park, the channel would be concrete lined and would have a bottom width of 20 feet and side slopes of one on two. The resulting top width would average 50 feet. A suitable transition between the two channel cross-sections would be provided immediately upstream of Wilson Park. It would be necessary to modify the bridge piers for one pedestrian bridge in Wilson

Map 37

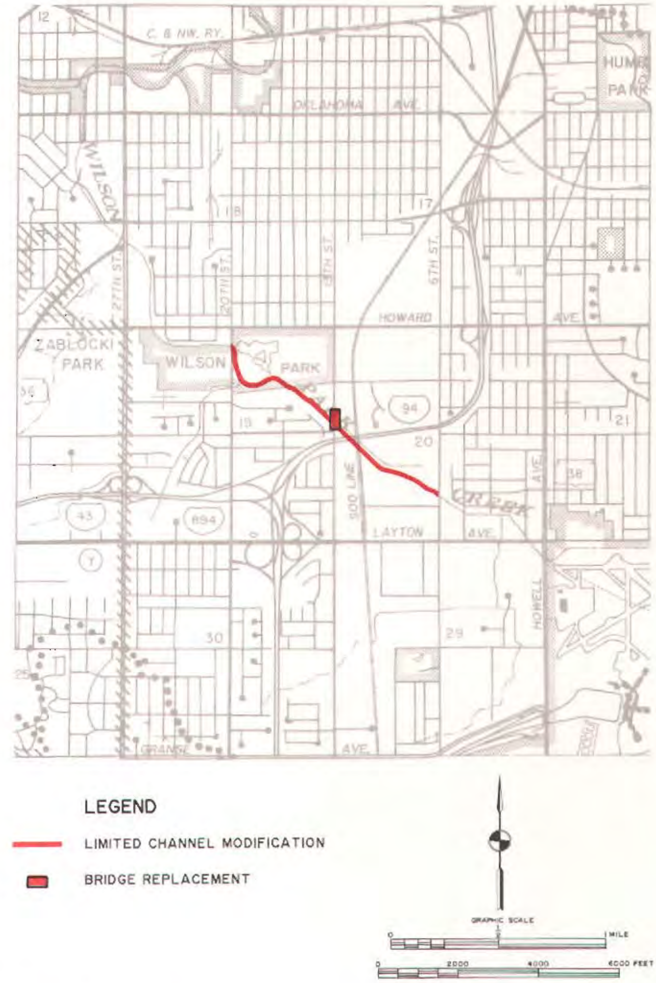
ALTERNATIVE PLAN 4: MAJOR CHANNELIZATION ALONG WILSON PARK CREEK IN THE CITY OF MILWAUKEE WITH CONCRETE LINING THROUGH WILSON PARK



Source: SEWRPC.

Map 38

ALTERNATIVE PLAN 5: BRIDGE REPLACEMENT AND LIMITED CHANNELIZATION ALONG WILSON PARK CREEK IN THE CITY OF MILWAUKEE



Source: SEWRPC.

Park. No other bridge replacement or modification would be required. Implementation of this alternative would eliminate all structure flood damages due to overland flooding and storm sewer surcharging for floods up to and including the 100-year recurrence interval event.

Utilizing an annual interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this major channelization alternative is estimated at $82,000. This cost consists of the amortization of the $1,221,000 capital cost of the channel modification, $30,000 for modification of one pedestrian bridge, and $2,600 in annual operation and maintenance costs. The average annual flood abatement benefit is estimated at $39,000, resulting in a benefit-cost ratio of 0.48.

Alternative System Plan 5—Bridge Replacement with Limited Channelization: This flood control system alternative is shown on Map 38 and consists of replacing one existing bridge over Wilson Park Creek—at S. 13th Street at River Mile 2.42. Under existing channel conditions, this bridge produces a backwater of 3.0 feet for a 100-year recurrence interval flood with year 2000 planned land use. This bridge would be replaced with a clear span structure designed to produce no backwater during a 100-year recurrence interval flood event.

In addition to the bridge replacement, minor channel modifications would be carried out along the 1.3-mile reach of Wilson Park Creek between S. 6th Street and S. 20th Street. These modifications are intended to eliminate stagnant water conditions caused by a 2.0-foot jump in the streambed at S. 6th Street, and also to help alleviate flood damages at De Paul Rehabilitation Hospital. The existing channel invert would be lowered from 1.3 to 2.3 feet within this reach. The proposed streambed would match the existing streambed upstream of S. 6th Street, while a drop of about 4.0 feet would remain at S. 20th Street. Between S. 6th Street and S. 13th Street, the proposed channel would have a bottom width of 10 feet and side slopes of one on three. Between S. 13th Street and S. 20th Street, the proposed channel would have a bottom width of 20 feet and side slopes of one on three.

Implementation of this alternative would essentially eliminate all damages attendant to floods up to and including the 100-year recurrence interval event.

Utilizing an annual interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this alternative is estimated at $63,000. This cost consists of the amortization of the $778,000 capital cost of the bridge replacement, the $166,000 capital cost of the channel modification, and $2,600 in annual operation and maintenance costs. The average annual flood abatement benefit is estimated at $39,000, resulting in a benefit-cost ratio of 0.62.

Alternative System Plan 6—Diversion: In analyzing a floodwater diversion system for Wilson Park Creek in the City of Milwaukee, three subalternative systems were considered. These subalternatives considered diverting stormwater runoff from three different drainage areas through a gravity flow conduit located along E. and W. Layton Avenue, discharging to Lake Michigan at Sheridan Park. These drainage basins are shown on Map 39, and include the following areas: 1) that portion of the Wilson Park Creek subwatershed located in the City of Cudahy; 2) that area which is tributary to Holmes Avenue Creek; and 3) that area which is tributary to the confluence of Wilson Park Creek and Holmes Avenue Creek. Under each subalternative, the diversion tunnel would be designed to convey the peak discharge for a 100-year recurrence interval flood event from the respective drainage area.

Hydrologic and hydraulic analyses conducted under this alternative indicate that the diversion of runoff from that area located in the City of Cudahy would not reduce flood discharges enough to eliminate flood damages along Wilson Park Creek in the City of Milwaukee during a 100-year recurrence interval flood. Therefore, that subalternative was eliminated from further consideration.

Implementation of either of the two remaining subalternatives would serve to eliminate flood damages from floods up to and including the 100-year recurrence interval event. Both subalternatives consist of the construction of about 18,350 feet of diversion tunnel along Layton Avenue between the Holmes Avenue Creek crossing and Lake Michigan at Sheridan Park. This tunnel would have a total drop of about 62 feet along its 18,350-foot length. A tunnel 11 feet in diameter would be required to divert the runoff from the Holmes Avenue Creek drainage basin. A tunnel diameter of 13 feet would be required to divert runoff from that area tributary to the confluence of Wilson Park Creek and Holmes Avenue Creek. Since these two subalternatives are very similar in terms of the capital costs entailed, that subalternative which provides for the diversion of runoff from the area tributary to the confluence of Wilson Park Creek and Holmes Avenue Creek was chosen for further consideration. This subalternative would provide the greatest reduction in flood discharge along Wilson Park Creek as well as along the Kinnickinnic River. This subalternative is shown on Map 40.

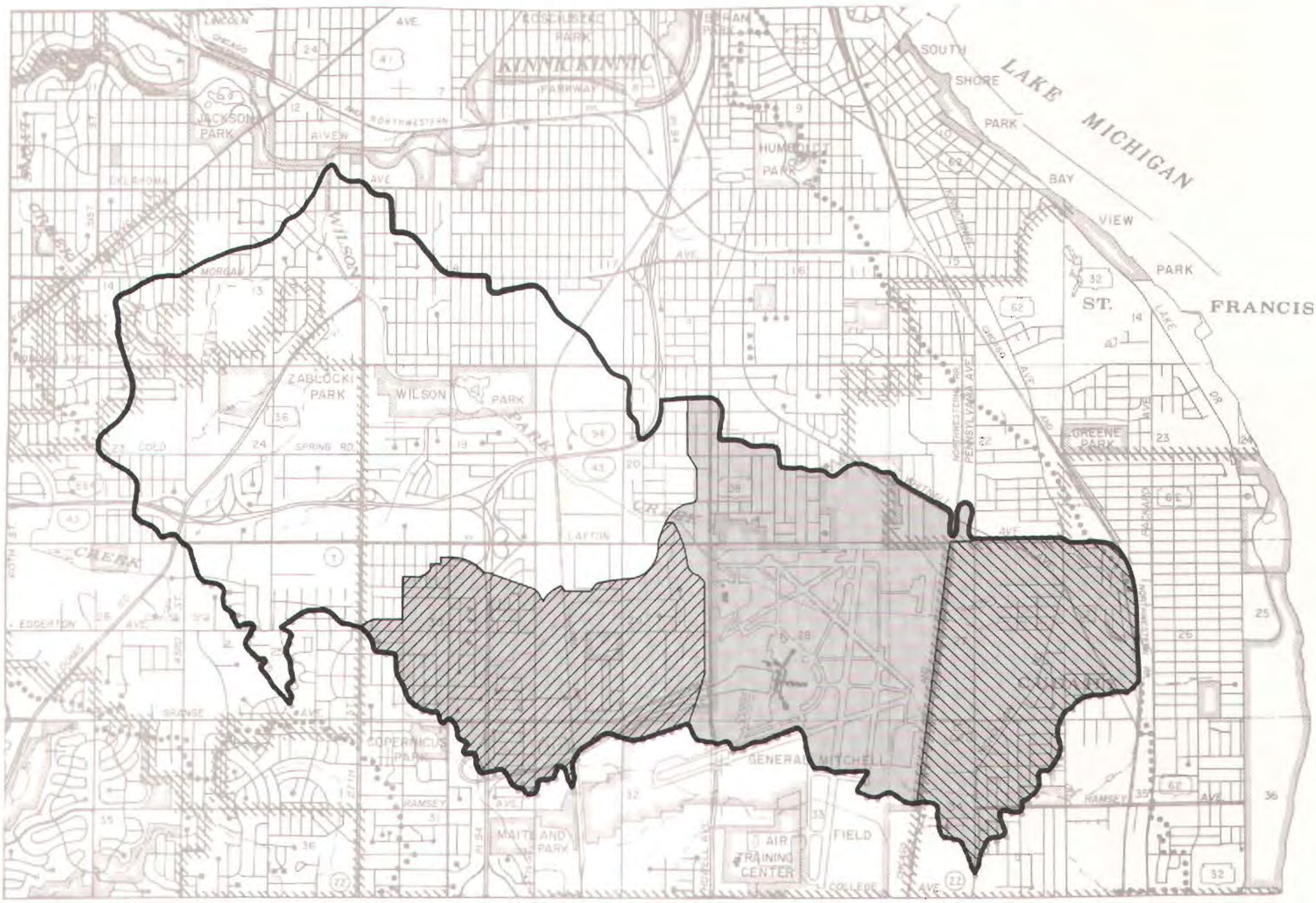
An alternative diversion tunnel route which could be considered in any more detailed feasibility studies would route the diversion north at about Howell Avenue to the upper reaches of the inner harbor at about Chase Avenue.

Utilizing an annual interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this floodwater diversion alternative is estimated at $3,081,000. This cost consists of the amortization of the $48,307,000 capital cost of the diversion tunnel, and $18,000 in annual operation and maintenance costs. The average annual flood abatement benefit is estimated at $39,000, resulting in a benefit-cost ratio of 0.01.

Alternative System Plan 7—Combination of Major Channelization and Channel Enclosure: This alternative flood control system is shown on Map 41 and includes major channel modifications along the 0.9-mile reach between Wilson Park and S. 6th

Map 39

DRAINAGE BASINS WITHIN WILSON PARK CREEK SUBWATERSHED CONSIDERED FOR DIVERSION TO LAKE MICHIGAN UNDER ALTERNATIVE PLAN 6



LEGEND

AREA DIVERTED SUBALTERNATIVE 1

AREA DIVERTED SUBALTERNATIVE 2

AREA DIVERTED SUBALTERNATIVE 3

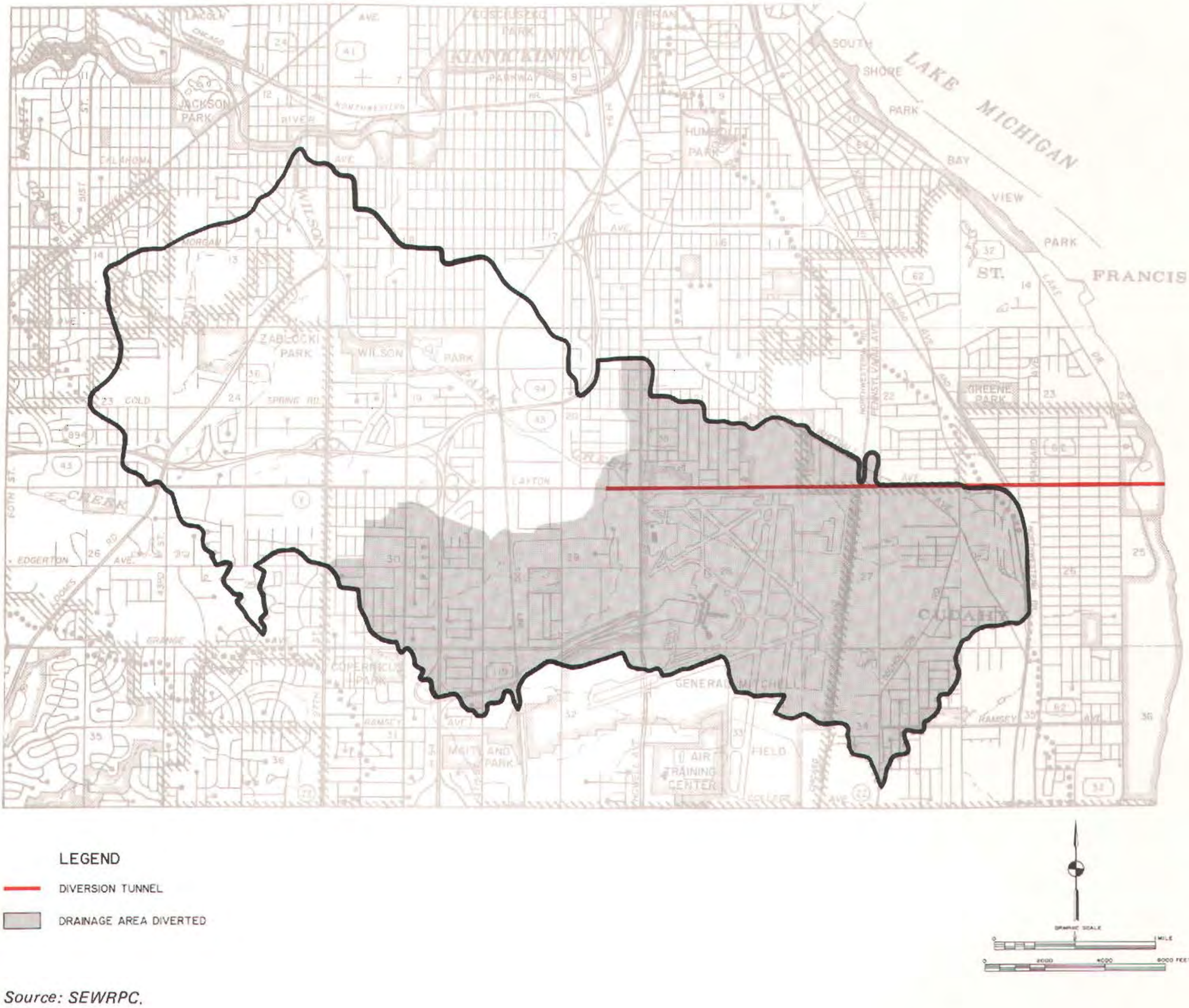
Source: SEWRPC.

Street, as well as the construction of a bypass tunnel through Wilson Park. This alternative was considered since it would avoid the need for major changes to the existing stream channel through Wilson Park.

Under this alternative, the existing Wilson Park Creek channel would be lowered from 2.3 to 5.2 feet between S. 6th Street and the Wisconsin Electric Power Company right-of-way located at the upstream end of Wilson Park. The proposed channel through this reach would be turf lined and would have a bottom width of 20 feet and side slopes of one on four. The resulting top width would average 110 feet. Within Wilson Park the existing channel would remain unchanged except for the first 150 feet upstream of S. 20th Street. The channel in this reach would be lowered 5.4 feet and widened, with a 20-foot bottom width and side slopes of one on three. The resulting

Map 40

ALTERNATIVE PLAN 6: DIVERSION OF FLOODWATERS FROM WILSON PARK CREEK IN THE CITY OF MILWAUKEE



Source: SEWRPC.

channel would be turf lined. In addition, a bypass tunnel would be constructed through Wilson Park following an alignment similar to that shown on Map 41. This tunnel would consist of two 12-foot by 10-foot concrete box culverts and one 10-foot by 8-foot concrete box culvert, each culvert being about 1,450 feet in length. These culverts would be placed on a slope that would match the proposed channel inverts at either end. This tunnel would carry some normal flow and most wet-weather flow from Wilson Park Creek. During major storm events, excess flow would be conveyed through the park by the existing channel. The existing channel would continue to convey the normal flow from Villa Mann Creek, as well as localized drainage.

Implementation of this alternative would eliminate all structure flood damages due to overland flooding and storm sewer surcharging for floods up to and including the 100-year recurrence interval event.

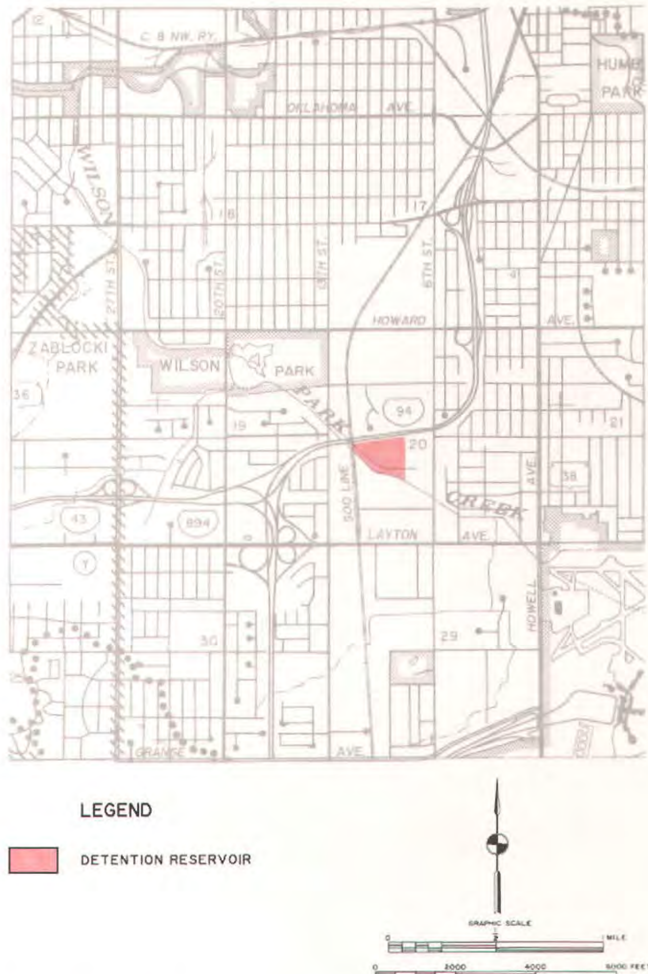
Map 41

ALTERNATIVE PLAN 7: COMBINATION OF MAJOR CHANNELIZATION AND CHANNEL ENCLOSURE ALONG WILSON PARK CREEK IN THE CITY OF MILWAUKEE

Source: SEWRPC.

Map 42

ALTERNATIVE PLAN 8: DETENTION STORAGE ALONG WILSON PARK CREEK IN THE CITY OF MILWAUKEE



Source: SEWRPC.

Utilizing an annual interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this alternative is estimated at $304,000. This cost consists of the amortization of the $342,000 capital cost of the channel modification, the $4,382,000 capital cost of the bypass tunnel, and $4,200 in annual operation and maintenance costs. The average annual flood abatement benefit is estimated at $39,000, resulting in a benefit-cost ratio of 0.13.

Alternative System Plan 8—Detention Storage: This flood control system alternative consists of the construction of a detention reservoir along Wilson Park Creek immediately upstream of the Soo Line Railroad crossing at River Mile 2.57, as shown on Map 42. The resulting reservoir would encompass about 18 acres and would provide about 65 acre-feet of storage under a 100-year recurrence interval flood event. An existing railroad spur that services the Central Steel & Wire Company currently traverses the site of the proposed reservoir. This railroad spur would remain, with excavation of the reservoir occurring on either side of the embankment. In order to ensure access of floodwaters to that portion of the reservoir that lies north of the railroad embankment, culverts would be placed along the embankment at a spacing of 100 feet.

Implementation of this alternative would eliminate all structure flood damages due to overland flooding and storm sewer surcharging for floods up to and including the 100-year recurrence interval event.

Utilizing an annual interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this alternative is estimated at $117,000. This cost includes the amortization of the $945,000 capital cost of the reservoir, the $48,000 capital cost of the railroad spur culverts, $443,000 for land acquisition, and $26,000 in annual operation and maintenance costs. The average annual flood abatement benefit is estimated at $39,000, resulting in a benefit-cost ratio of 0.33.

Evaluation of Flood Control Alternatives for Wilson Park Creek in the City of Milwaukee

In selecting recommended flood control measures for Wilson Park Creek, not only must the costs of each alternative plan be considered, but also the environmental and aesthetic impacts, and the implementability of the alternative. Also, the noneconomic, or intangible, benefits of each alternative plan must be considered.

Excluding the "no action" alternative, all of the flood control alternatives developed for Wilson Park Creek in the City of Milwaukee are considered technically feasible. The results of an economic analysis of these alternatives revealed that only one—major channelization—produced a benefit-cost ratio close to one. Alternative Plan 4—major channelization with concrete lining through Wilson Park—and Alternative Plan 5—bridge replacement with limited channelization—would alleviate flood damages, but at higher costs. The remaining five alternatives were eliminated from further consideration for various reasons. The "no action" alternative, while offering the lowest cost, does nothing to alleviate the existing flood problem and does not represent a sound approach to flood control. In addition to having a benefit-cost ratio of less than one, Alternative Plan 2—structure floodproofing and elevation, and removal—presents several problems in implementation. First, complete implementation of a voluntary structure floodproofing and elevation program is unlikely and, with partial implementation, the City of Milwaukee would be left with a significant residual problem whenever a major flood event occured. Also, yard damages and cleanup costs would remain under the structure floodproofing and removal alternative. Finally, some floodproofing is very likely to be applied without adequate professional advice. As a result, structure damage is likely to occur, and, once again, the City of Milwaukee is likely to be asked to assist in resolution of the problem. It should be noted that in some instances a structure floodproofing and elevation alternative may be a viable solution to a flooding problem. Such would be the case where structure damages are relatively low and are widely scattered along a stream. Structural measures, such as channel modifications or detention storage, may not be an economical solution in such instances, since long reaches of the channel would have to be modified by structural improvements to resolve widely scattered problems.

Implementation of Alternative Plan 6—diversion—would provide for additional intangible benefits in that the plan would result in a low-level outlet for local stormwater drainage systems all along the route of the diversion tunnel to Lake Michigan. However, the present damages and thus the benefits associated with the local drainage system improvement have not been specifically quantified. The extremely high cost of Alternative Plan 6 relative to the direct flood abatement benefits along Wilson Park Creek and the attendant very low benefit-cost ratio make this alternative impractical from an economic standpoint. However, should funding from other than local sources be found for implementing this project, it could be considered further because of the potential for providing a means of improving local drainage systems.

Alternative Plan 7—channel enclosure and major channelization—and Alternative Plan 8—detention storage—were not considered further because of their very low benefit-cost ratios.

In choosing between the remaining three alternatives—major channelization, major channelization with concrete lining through Wilson Park, and bridge replacement with limited channelization—several factors must be considered. In terms of impacts on the natural environment, it appears that Alternative 5—bridge replacement with limited channelization—would be preferable, since required changes to the existing channel configuration would be more limited. In this respect, however, it should be noted that extensive channel modifications have already been carried out along Wilson Park Creek, including along the 0.4-mile reach through Wilson Park. These modifications included widening and straightening of the channel. Therefore, the creek is no longer in its "natural" state. By utilizing a turf lining under Alternative Plan 3—major channelization—the appearance of the proposed channel could be made more consistent with the urban park environment along Wilson Park Creek. In addition, the impact on wildlife in Wilson Park—such as the resident duck population—would be lessened by use of the turf lining as opposed to a concrete lining. Alternative Plan 4—major channelization with concrete lining through Wilson Park—would require fewer trees to be removed in Wilson Park. The remaining trees would provide a visual barrier along the channel, lessening the aesthetic impact of the concrete lining.

In terms of environmental impacts on developed areas, all three alternatives would serve to reduce flood stages along Wilson Park Creek between S. 20th Street and W. Layton Avenue. Because of the more extensive channelization and lower channel invert, Alternatives 3 and 4, providing for major channelization, result in lower flood profiles, particularly through that reach along De Paul Rehabilitation Hospital.

In terms of implementability, all three alternatives would present few problems. Favorable aspects of Alternative Plans 3 and 4 include the fact that they are consistent with channelization plans prepared by the Milwaukee Metropolitan Sewerage District except for the use of a turf lining as opposed to concrete, and the fact that they utilize the bridges that have already been built to accommodate the proposed channel invert. A favorable aspect of Alternative Plan 2 is that it has a lesser impact on the existing channel system, and thus should make the regulatory agency and the general public more favorably inclined to implementation.

Upon consideration of the eight alternative floodland management systems, a ninth alternative was developed. This alternative also consists of major channelization along the 1.3-mile reach between S. 6th Street and S. 20th Street. The streambed would be lowered from 2.3 to 5.4 feet along this reach in order to match the existing inverts at S. 6th Street and S. 20th Street. For the 0.4-mile reach through Wilson Park, the channel would have a bottom width of 20 feet and side slopes of one on three. The resulting channel top width would average 70 feet. For the 0.9-mile reach between Wilson Park and S. 6th Street, the proposed channel would have a bottom width of 20 feet and side slopes of one on four. The resulting channel top width would average 110 feet. The channel along the entire 1.3-mile reach would be turf lined. It would be necessary to modify the piers on one pedestrian bridge located in Wilson Park. Implementation of this alternative would eliminate all structure flood damages due to overland flooding and storm sewer surcharging for floods up to and including the 100-year recurrence interval event.

Utilizing an annual interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this major channelization alternative is estimated at $35,600. This cost consists of the amortization of the $490,000 capital cost of the channel modification, the $30,000 cost for modification of one pedestrian bridge, and $2,600 in annual operation and maintenance costs. The average annual flood abatement benefit is estimated at $39,000, resulting in a benefit-cost ratio of 1.10.

It is recommended that the revised Alternative Plan 3 with side slopes of one on three through Wilson Park be adopted to resolve flooding problems along Wilson Park Creek in the City of Milwaukee. At a June 19, 1987, public hearing on the recommended plan, a number of citizens requested that the modified channel include concrete lining rather than turf lining. As a result of that hearing, it was decided that the Milwaukee Metropolitan Sewerage District should conduct an opinion poll of residents along the channel in order to determine public support for the concrete lining of portions of the channel. The results of that poll would be used by the District in determining whether or not to provide a concrete lining on a part of the channel cross-section in the final design of the channel modifications.

In recommending the channel modification flood control plan for Wilson Park Creek, the Advisory Committee recognized that there was interest among local officials in providing a greater level of flood protection and a better means of solving local drainage problems through construction of the floodwater diversion tunnel. That interest led the Milwaukee County Board of Supervisors and the Common Council of the City of Milwaukee to urge the District to proceed with the recommended channel modifications along Wilson Park Creek as an interim measure, and to urge further that the

District seek funding for construction of the floodwater diversion alternative. The standards described in Chapter III of this system plan call for a 100-year recurrence interval level of protection in the design of flood control works by the District; the Advisory Committee thus concluded that the design and construction of flood control measures intended to protect against events with recurrence intervals in excess of 100 years and to provide a deep local drainage outlet would be the responsibility of the local communities, with assistance from state or federal agencies, such as the U. S. Army Corps of Engineers. This conclusion would not preclude the investigation of the potential for federal funding of this alternative, but would place the burden of seeking such assistance on the local community.

Alternative Flood Control and Related Drainage System Plans for Wilson Park Creek-Edgerton Channel in the City of Cudahy

In preparing the comprehensive plan for the Kinnickinnic River watershed, 11 alternative flood control systems were considered for alleviating the flood damage problem along Wilson Park Creek in the City of Cudahy—also referred to as the Edgerton Channel: 1) Alternative System 1—no action; 2) Alternative System 2—diversion; 3) Alternative System 3—structure floodproofing and elevation; 4) Alternative System 4—major channelization; 5) Alternative System 5—major channelization with channel enclosure; 6) Alternative System 6—dikes and floodwalls; 7) Alternative System 7—detention storage; 8) Alternative System 8—detention storage, channel enclosure, and channel realignment; 9) Alternative System 9—bridge and culvert modification, replacement, and removal; 10) Alternative System 10—channel enclosure; and 11) Alternative System 11—major channelization with turf lining and channel enclosure.

Each alternative system is described briefly below. The economic benefits and costs attendant to each alternative are provided in Table 28. More detailed descriptions of these 11 alternatives are set forth in SEWRPC Planning Report No. 32.

Alternative System Plan 1—No Action: Under this "do nothing" alternative, the average annual cost would be equivalent to the average annual flood damages of $142,100 for this reach. Estimated damages of about $486,000 for the 100-year recurrence interval flood would remain along this stream reach. There would be no monetary benefits associated with this alternative.

Alternative System Plan 2—Diversion: This flood control system alternative is shown on Map 43 and consists of constructing a gravity flow conduit from the Edgerton Channel immediately downstream of S. Nicholson Avenue to Lake Michigan through the City of Cudahy along the alignment of Edgerton Avenue. The diversion conduit would be approximately 6.0 feet in diameter and would have a total length of 7,500 feet. This conduit would be able to carry the 100-year recurrence interval flood discharge under year 2000 planned land use conditions at the point of diversion. This alternative would eliminate flood damages along this reach for floods up to and including the 100-year recurrence interval event.

Utilizing an annual interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this alternative is estimated at $729,200. This cost consists of the amortization of the $11,390,000 capital cost for the diversion conduit and $7,100 in annual operation and maintenance costs. The average annual flood abatement benefit is estimated at $142,100, resulting in a benefit-cost ratio of 0.19.

Alternative System Plan 3—Structure Floodproofing and Elevation: A structure floodproofing and elevation alternative flood control system was analyzed to determine if such a structure-by-structure approach would be a technically feasible and economically viable solution to the flood problem along Edgerton Channel in the City of Cudahy. The analysis indicated that 169 structures would have to be floodproofed and 11 structures would have to be elevated. These structures are shown on Map 44. No structures would be removed under this alternative. Implementation of this alternative would essentially eliminate all damages from floods up to and including the 100-year recurrence interval event.

Utilizing an annual interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this alternative is estimated at $55,800. This cost consists of the amortization of the $878,000 capital cost—$531,900 for floodproofing and $346,000 for structure elevation. The average annual flood abatement benefit is estimated at $142,100, yielding a benefit-cost ratio of 2.55.

Alternative System Plan 4—Major Channelization: Under this alternative, about 0.8 mile of major channel modifications would be carried out along Edgerton Channel between the upstream limit of the existing airport channelization at River Mile 5.28 and S. Whitnall Avenue, as shown on Map 45.

Table 28

COST ESTIMATES FOR FLOOD CONTROL ALTERNATIVES FOR EDGERTON CHANNEL IN THE CITY OF CUDAHY

Alternative	Capital Cost	Annual Costs: Amortized Capital[a]	Annual Costs: Operation and Maintenance	Annual Costs: Other	Annual Costs: Total	Benefit-Cost Analysis: Annual Benefits	Benefit-Cost Analysis: Annual Benefits Minus Annual Costs	Benefit-Cost Analysis: Benefit-Cost Ratio	Benefit-Cost Analysis: Economic Ratio Greater than One
1. No Action	$ 0	$ 0	$ 0	$142,100	$142,100	$ 0	$-142,100	- -	No
2. Diversion	11,390,000	722,100	7,100	0	729,200	142,100	-587,100	0.19	No
3. Structure Floodproofing and Elevation	878,000	55,800	0	0	55,800	142,100	86,300	2.55	Yes
4. Major Channelization	2,614,000[d]	166,000	1,700	0	167,700	142,100	-25,600	0.85	No
5. Major Channelization with Channel Enclosure	2,663,000[d]	168,800	1,900	0	170,700	142,100	-28,600	0.83	No
6. Dikes and Floodwalls	3,409,000	226,700[c]	53,200	0	279,900	142,100	-137,800	0.51	No
7. Detention Storage	980,000	62,200	10,000	0	72,200	117,500	45,300	1.63	Yes
8. Detention Storage, Channel Enclosure, and Channel Realignment	2,109,000[d]	133,900	10,500	0	144,400	142,100	-2,300	0.98	No
9. Bridge and Culvert Modification, Replacement, and Removal[b]	- -	- -	- -	- -	- -	- -	- -	- -	- -
10. Channel Enclosure	4,466,000	283,500	2,500	0	286,000	142,100	-143,900	0.50	No
11. Major Channelization with Turf Lining and Channel Enclosure	2,154,000[d]	136,600	1,900	0	138,500	142,100	3,600	1.02	Yes

[a]*Amortized capital cost is based on an interest rate of 6 percent and a project life of 50 years.*

[b]*No costs were determined as this alternative was not found to be technically feasible.*

[c]*Assumes replacement of pumping equipment after 25 years.*

[d]*No cost was assigned for S. Pennsylvania Avenue bridge replacement since this structure is scheduled for replacement under the Commission's adopted regional transportation plan.*

Source: SEWRPC.

The proposed channel would be concrete lined and would have a bottom width of 10 feet and side slopes of one on two to one on three. The top width of the proposed channel would vary from 40 to 50 feet. The existing streambed would be lowered from 1.5 to 2.5 feet. In addition to the major channel modifications, the following five bridges would require modification, replacement, or removal: 1) the Chicago & North Western Railway bridge, 2) the railroad utility road bridge at River Mile 5.36, 3) the S. Pennsylvania Avenue bridge, 4) the frontage road bridge at River Mile 5.98, and 5) the S. Nicholson Avenue bridge. These bridges would be designed to span the entire modified channel and to cause no backwater during a 100-year recurrence interval flood event. Implementation of this alternative would essentially eliminate all damages attendant to floods up to and including the 100-year recurrence interval event. Utilizing an annual interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this alternative is estimated at $167,700. This cost consists of the amortization of the $2,614,000 capital cost—$1,362,000 for channelization and $1,252,000 for bridge replacement[7]—and $1,700 in annual operation and maintenance costs. The average annual flood abatement benefit is estimated at $142,100, resulting in a benefit-cost ratio of 0.85.

Alternative System Plan 5—Major Channelization with Channel Enclosure: A major channel modifi-

[7]*No cost was assigned for S. Pennsylvania Avenue bridge replacement since this structure is scheduled for replacement under the Commission's adopted regional transportation plan.*

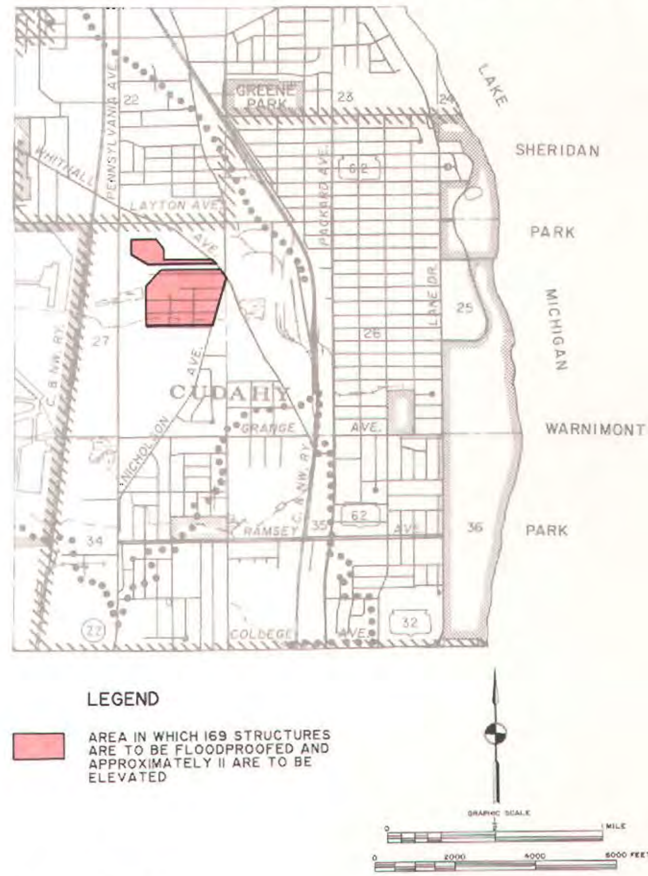
Map 43

ALTERNATIVE PLAN 2: DIVERSION OF FLOODWATERS FROM EDGERTON CHANNEL IN THE CITY OF CUDAHY

Source: SEWRPC.

Map 44

ALTERNATIVE PLAN 3: STRUCTURE FLOODPROOFING AND ELEVATION ALONG EDGERTON CHANNEL IN THE CITY OF CUDAHY



Source: SEWRPC.

cation and channel enclosure alternative was analyzed for Edgerton Channel in the City of Cudahy and is shown on Map 46. This alternative consists of major channel modifications along 0.5 mile of this stream reach—0.38 mile between the upstream limit of the existing airport channelization and River Mile 5.66, and 0.13 mile between S. Nicholson Avenue and S. Whitnall Avenue. The proposed channel would be concrete lined and would have a bottom width of 10 feet and side slopes ranging from one on two to one on three. The existing channel invert would be lowered from 2.1 to 4.5 feet. This alternative also includes the enclosure of 0.3 mile of channel in a reinforced concrete box culvert between River Mile 5.66 and S. Nicholson Avenue. This box culvert would be 10 feet wide by 6 feet high. In addition, this alternative calls for the modification or replacement of three bridges: 1) the Chicago & North Western Railway bridge, 2) the railroad utility road bridge at River Mile 5.36, and 3) the S. Pennsylvania Avenue bridge. Implementation of this alternative would essentially eliminate all damages attendant to floods up to and including the 100-year recurrence interval event.

Utilizing an annual interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this alternative is estimated at $170,700. This cost consists of the amortization of the $2,663,000 capital cost—$680,000 for channelization, $1,058,000 for channel enclosure, and $925,000 for bridge replacement[8]—and $1,900 in annual operation and main-

[8] *Ibid.*

Map 45

ALTERNATIVE PLAN 4: MAJOR CHANNELIZATION ALONG EDGERTON CHANNEL IN THE CITY OF CUDAHY

Source: SEWRPC.

Map 46

ALTERNATIVE PLAN 5: MAJOR CHANNELIZATION AND CHANNEL ENCLOSURE ALONG EDGERTON CHANNEL IN THE CITY OF CUDAHY

Source: SEWRPC.

tenance costs. The average annual flood abatement benefit is estimated at $142,100, resulting in a benefit-cost ratio of 0.83.

Alternative System Plan 6—Dikes and Floodwalls: The dike and floodwall flood control system alternative for Wilson Park Creek in the City of Cudahy consists of the construction of 0.9 mile of earthen dikes and 0.8 mile of concrete or sheet steel floodwalls, as shown on Map 47. These dikes and floodwalls would be designed to pass the 100-year recurrence interval flood with 3.0 feet of freeboard. The maximum height of the dikes and floodwalls would be 8.0 feet. Also under this alternative, eight stormwater pumping stations would be required in order to drain low-lying areas behind the dikes and floodwalls. Finally, this alternative includes the replacement of the bridge at S. Nicholson Avenue and the removal of the frontage road bridge at River Mile 5.98. Implementation of this alternative would essentially eliminate all damages attendant to floods up to and including the 100-year recurrence interval event.

Utilizing an annual interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this alternative is estimated at $279,900. This cost consists of the amortization of the $3,409,000 capital cost—$383,000 for dikes, $1,562,400 for floodwalls, $78,000 for bridge replacement, and $1,386,000 for pumping stations—and $53,200 in annual operation and maintenance costs. The average annual flood abatement benefit is estimated at $142,100, resulting in a benefit-cost ratio of 0.51.

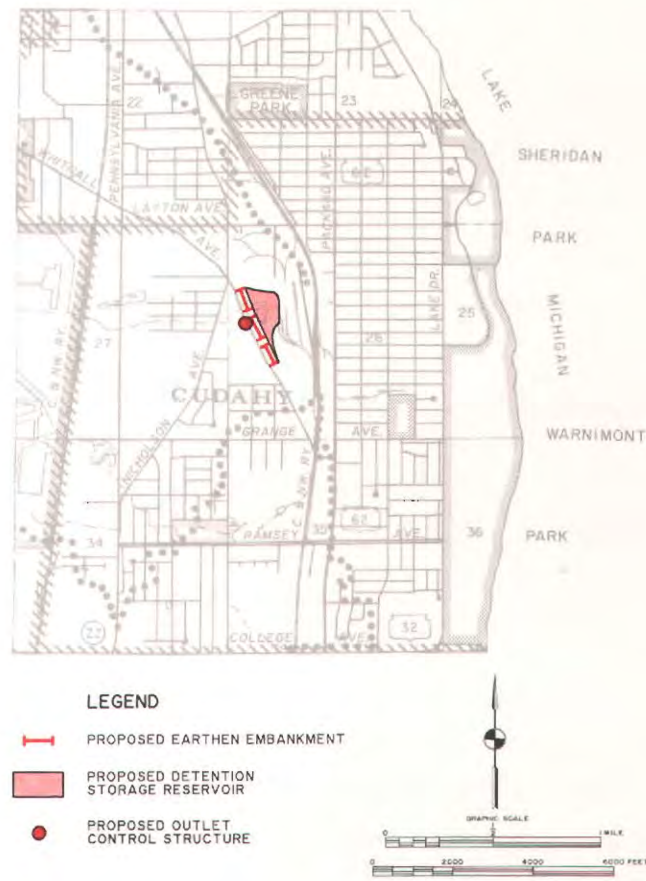
Map 47

ALTERNATIVE PLAN 6: DIKES AND FLOODWALLS ALONG EDGERTON CHANNEL IN THE CITY OF CUDAHY

Source: SEWRPC.

Map 48

ALTERNATIVE PLAN 7: DETENTION STORAGE ALONG EDGERTON CHANNEL IN THE CITY OF CUDAHY



Source: SEWRPC.

Alternative System Plan 7—Detention Storage: The detention storage alternative for Edgerton Channel in the City of Cudahy would provide for the construction of a reservoir along Edgerton Channel immediately upstream of S. Whitnall Avenue, as shown on Map 48. An earthen embankment approximately 1,600 feet long and varying from 2 to 13 feet in height would be constructed immediately east and parallel to S. Whitnall Avenue. The reservoir would provide about 65 acre-feet of storage under a 100-year recurrence interval flood event. This detention reservoir would serve to reduce, but not eliminate, flood damages along this reach of Edgerton Channel. The residual flood damages would amount to about $24,600 on an average annual basis.

Utilizing an annual interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this alternative is estimated at $72,200. This cost consists of the $980,000 capital cost of the detention reservoir and $10,000 in annual operation and maintenance costs. The average annual flood abatement benefit is estimated at $117,500, yielding a benefit-cost ratio of 1.63.

Alternative System Plan 8—Detention Storage, Channel Enclosure, and Channel Realignment: Under this flood control system alternative, shown on Map 49, a detention reservoir would be constructed immediately upstream of S. Whitnall

Map 49

ALTERNATIVE PLAN 8: DETENTION STORAGE, CHANNEL ENCLOSURE, AND BRIDGE ALTERATION ALONG EDGERTON CHANNEL IN THE CITY OF CUDAHY

Source: SEWRPC.

Avenue. An earthen embankment approximately 1,600 feet in length and 2.0 to 13 feet in height would be constructed immediately east of S. Whitnall Avenue. In addition to the reservoir, a 0.31-mile reach of the channel between River Miles 5.66 and 5.97 would be enclosed in a 10-foot-wide by 6-foot-high reinforced concrete box culvert. Finally, the existing channel would be realigned between the downstream end of the box culvert and the western city limits in order to be consistent with local planning and with easements previously acquired for this purpose by the City of Cudahy. In addition, a new bridge at S. Pennsylvania Avenue would need to be constructed to match the new channel alignment. Implementation of this alternative would essentially eliminate all damages attendant to floods up to and including the 100-year recurrence interval event.

Utilizing an annual interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this alternative is estimated at $144,400. This cost consists of the $2,109,000 capital cost—$980,000 for the detention reservoir, $1,085,000 for channel enclosure, and $44,200 for channel realignment[9]—and $10,500 in annual operation and maintenance costs. The average annual flood abatement benefit is estimated at $142,100, resulting in a benefit-cost ratio of 0.98.

Alternative System Plan 9—Bridge and Culvert Modification, Replacement, and Removal: A bridge and culvert modification, replacement, and removal alternative was considered for alleviating flood damages along Edgerton Channel in the City of Cudahy. This alternative is shown on Map 50. Under this alternative, selected bridges would be either removed, replaced, or modified so as to produce no backwater under a 100-year recurrence interval flood event. The bridges considered under this alternative were: 1) the Chicago & North Western Railway bridge at River Mile 5.34, 2) the utility road bridge at River Mile 5.36, 3) the S. Pennsylvania bridge at River Mile 5.54, 4) the frontage road bridge at River Mile 5.98, and 5) the S. Nicholson Avenue bridge at River Mile 5.99. The results of the hydraulic analysis conducted under this alternative indicate that the replacement or removal of these bridges would not have a significant impact on flood damages along this reach. Thus, this alternative is not considered to be technically or economically feasible. No costs were determined for this alternative.

Alternative System Plan 10—Channel Enclosure: The channel enclosure alternative would provide for the enclosure of a 0.8-mile reach of the Edgerton Channel between the upstream end of the existing airport channel modifications and S. Whitnall Avenue. The channel enclosure would consist of a single 10-foot-wide by 6-foot-high reinforced concrete box culvert between S. Whitnall Avenue and a point approximately 1,600 feet

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[9]*No cost was assigned for S. Pennsylvania Avenue bridge replacement since this structure is scheduled for replacement under the Commission's adopted regional transportation plan.*

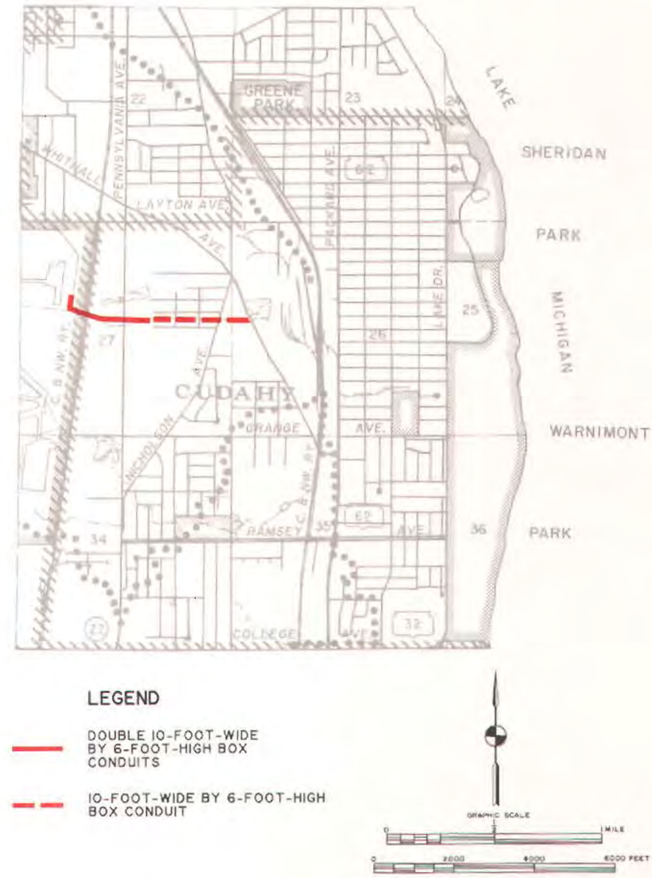
Map 50

ALTERNATIVE PLAN 9: BRIDGE AND CULVERT MODIFICATION, REPLACEMENT, AND REMOVAL ALONG EDGERTON CHANNEL IN THE CITY OF CUDAHY

Source: SEWRPC.

Map 51

ALTERNATIVE PLAN 10: CHANNEL ENCLOSURE ALONG EDGERTON CHANNEL IN THE CITY OF CUDAHY



Source: SEWRPC.

downstream of S. Nicholson Avenue. At that point, a transition would be made to a double concrete box culvert, each box being 10 feet wide by 6 feet high, which would extend downstream to join the existing improved channel through Milwaukee General Mitchell International Airport. This alternative is shown on Map 51. Implementation of this alternative would essentially eliminate all damages from floods up to and including the 100-year recurrence interval event.

Utilizing an annual interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this channel enclosure alternative is estimated at $286,000. This cost consists of the amortization of the $4,466,000 capital cost of the channel enclosure, and $2,500 in annual operation and maintenance costs. The average annual flood abatement benefit is estimated at $142,100, yielding a benefit-cost ratio of 0.50.

Alternative System Plan 11—Major Channelization with Turf Lining and Channel Enclosure: A second flood control alternative combining major channelization and channel enclosure was analyzed for Edgerton Channel. This alternative is shown on Map 52 and consists of major channel modifications along 0.5 mile of the stream—0.4 mile between the upstream limit of the existing airport channelization and River Mile 5.66, and 0.1 mile between S. Nicholson Avenue and S. Whitnall Avenue. The proposed channel would be turf lined and would have a bottom width of 10 feet and side slopes of one on three. The existing channel invert would be lowered from 2.1 to 4.5 feet. Between

Map 52

ALTERNATIVE PLAN 11: MAJOR CHANNELIZATION WITH TURF LINING AND CHANNEL ENCLOSURE ALONG EDGERTON CHANNEL IN THE CITY OF CUDAHY

Source: SEWRPC.

the Cudahy city limits at River Mile 5.31 and River Mile 5.66, the channel would be realigned to accommodate local land use proposals and existing drainage easements. Also under this alternative, 0.3 mile of channel between River Mile 5.66 and S. Nicholson Avenue would be enclosed in a reinforced concrete box culvert. This box culvert would be 10 feet wide by 6 feet high. In addition, this alternative calls for the replacement of the Chicago & North Western Railway bridge at River Mile 5.34 and the railroad utility road bridge at River Mile 5.36, as well as the construction of a new bridge at the Pennsylvania Avenue crossing at River Mile 5.54.

Utilizing an annual interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this alternative is estimated at $138,500. This cost consists of the amortization of the $2,154,000 capital cost—$148,000 for channelization, $1,058,000 for channel enclosure, and $934,000 for bridge replacement[10]—and $1,900 for annual operation and maintenance costs. The average annual flood abatement benefit is estimated at $142,100, resulting in a benefit-cost ratio of 1.02.

Evaluation of Flood Control Alternatives for Edgerton Channel in the City of Cudahy

The principal features of, and the costs and benefits associated with, each of the floodland management alternatives considered for the Edgerton Channel are summarized in Table 28. Excluding the "no action" alternative, all of the alternatives were found to be technically feasible with the exception of Alternative Plan 9—bridge and culvert modification, replacement, and removal. Of the remaining nine alternatives, four were found to have benefit-cost ratios of one or more, and one additional alternative—Alternative Plan 10, providing for channel enclosure—was found to have sufficient intangible benefits to be maintained as a viable alternative.

Alternative Plan 3—structure floodproofing and elevation—presents several problems in implementation. First, complete implementation of a voluntary structure floodproofing and elevation program is unlikely and, with partial implementation, the City of Cudahy would be left with a significant residual problem whenever a major flood event occurred. Second, even if a voluntary structure floodproofing program were completely carried out, the City of Cudahy would still be subjected to extensive overland flooding that would hamper routine access to and from some riverine area structures, continue to close local streets to automobile traffic, and interfere with the rapid movement of emergency vehicles. Furthermore, yard and street damages and cleanup costs remain with the structure floodproofing and elevation alternative. Finally, some floodproofing is very likely to be applied without adequate professional advice. As a result, structure damage is likely to occur, and once again city officials are likely to be asked to assist in resolution of the problem. Therefore, this alternative was eliminated from further consideration. It should be noted that in some instances a structure floodproofing and elevation alternative may be a viable solution to a flooding problem.

[10] *Ibid.*

Such would be the case where structure damages are relatively low and are widely scattered along a stream. Structural measures, such as channel modifications or detention storage, may not present an economical solution in those instances, since long reaches of the channel would have to be modified by structural improvements to resolve widely scattered problems.

Although Alternative Plan 7—detention storage—exhibits a benefit-cost ratio of greater than one, this alternative would abate only about 80 percent of the flood problem in the City of Cudahy as measured by reduction in average annual flood damages, resulting in only a partial resolution to the flood problem. Therefore, this alternative was dropped from further consideration.

The remaining three alternatives are similar in that they all provide for enclosure of the channel through the existing residential development between River Mile 5.66 and S. Nicholson Avenue. In addition to the flooding of homes, this reach has been the subject of several nonflood-related problems, including the deposition of trash in the channel, as well as health and safety problems relating to children playing in the channel. The open channel safety problems in this reach are the result of the narrow right-of-way between the buildings, necessitating the construction of near-vertical side slopes along the channel. These problems were brought up by City of Cudahy officials at interagency staff meetings and at the October 12, 1978, public hearing conducted as part of the Kinnickinnic River watershed study.

In terms of implementability, it appears that Alternative Plan 8, providing for detention storage, channel enclosure, and channel realignment, would be objected to by city officials as indicated at the October 12, 1978, public hearing mentioned above. In addition to the concerns expressed above, officials objected to the use for floodwater storage of lands proposed for industrial development in local plans and zoning. Of the two remaining alternatives, Alternative Plan 10, calling for complete channel enclosure, would be consistent with City of Cudahy plans to enclose Edgerton Channel within the City. This alternative has a very low benefit-cost ratio, however.

After consideration of the various technical and economic features of the alternative floodland management measures, it was recommended that Alternative Plan 11, providing for a combination of major channelization with turf lining and channel enclosure, be adopted to resolve flooding problems along Edgerton Channel in the City of Cudahy.

Alternative Floodwater Diversion Alternatives for Wilson Park Creek and the Kinnickinnic River

As previously noted, the City of Milwaukee Common Council had requested that, as part of this system plan, the possibility of diverting floodwaters from all or part of Wilson Park Creek in order to reduce flood discharges on the Kinnickinnic River be investigated. As a result of such an investigation, two such diversion alternatives were developed: 1) diversion of runoff from that portion of the Wilson Park Creek subwatershed which drains to the confluence of Wilson Park Creek and Holmes Avenue Creek; and 2) diversion of all flow from Wilson Park Creek. These two alternatives are described below.

Alternative System Plan 1—Diversion of Flow from Wilson Park Creek at Confluence with Holmes Avenue Creek: This alternative is the same as Alternative System 5 for Wilson Park Creek in the City of Milwaukee and is shown on Map 40. Under this alternative, the flow in Wilson Park Creek at the confluence with Holmes Avenue Creek for floods up to and including the 100-year recurrence interval event would be diverted to Lake Michigan at Sheridan Park by a tunnel to be constructed along Layton Avenue. Flow in excess of the 100-year recurrence interval event would still be carried along Wilson Park Creek. This tunnel would be about 18,350 feet in length and would have a diameter of about 13 feet, assuming a total available drop of 62 feet between the point of diversion and Lake Michigan. The impact of this diversion tunnel on flood discharges along the Kinnickinnic River for both the 100-year recurrence interval flood and the August 6, 1986, flood event—the flood of record with an estimated recurrence interval in excess of 500 years—is shown in Table 29. As shown in this table, the diversion alternative would serve to reduce the 100-year recurrence interval flood discharge on the Kinnickinnic River by 14 to 18 percent. The August 6, 1986, flood event discharge would be reduced by 11 to 16 percent.

Utilizing an annual interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this diversion alternative is estimated at $3,081,000. This cost consists of the amortization of the $48,307,000 capital cost of the diversion tunnel, and $18,000 in annual

Table 29

IMPACT OF FLOODWATER DIVERSION ALTERNATIVES ON FLOOD DISCHARGES ALONG THE KINNICKINNIC RIVER

Location	River Mile	100-Year Recurrence Interval[a]					August 6, 1986 Flood[b]				
		Existing Condition Discharge (cubic feet per second)	Diversion Alternative 1		Diversion Alternative 2		Existing Condition Discharge (cubic feet per second)	Diversion Alternative 1		Diversion Alternative 2	
			Discharge (cubic feet per second)	Percent of Reduction	Discharge (cubic feet per second)	Percent of Reduction		Discharge (cubic feet per second)	Percent of Reduction	Discharge (cubic feet per second)	Percent of Reduction
Mouth	0.00	7,400	6,320	14	4,790	35	12,160	10,680	12	7,740	36
S. 1st Street	1.43	7,000	5,900	16	4,330	38	12,860	11,390	11	8,360	35
Abandoned North Shore & Milwaukee Road.	2.72	6,000	5,060	16	3,490	42	11,030	9,390	15	5,790	48
S. 16th Street	3.58	5,700	4,700	18	3,110	45	9,990	8,360	16	4,760	52

[a] *Discharges are based upon year 2000 land use and existing channel conditions.*

[b] *Discharges listed were simulated using hydrologic model for Kinnickinnic River watershed.*

Source: SEWRPC.

operation and maintenance costs. The average annual flood damage abatement benefit was estimated at $89,000, yielding a benefit-cost ratio of 0.03. It should be noted that unlike the average annual benefit of the previously described flood control alternative, the average annual benefit of this alternative, and therefore the benefit-cost ratio, includes benefits derived from reducing damages for floods having recurrence intervals of greater than 100 years.

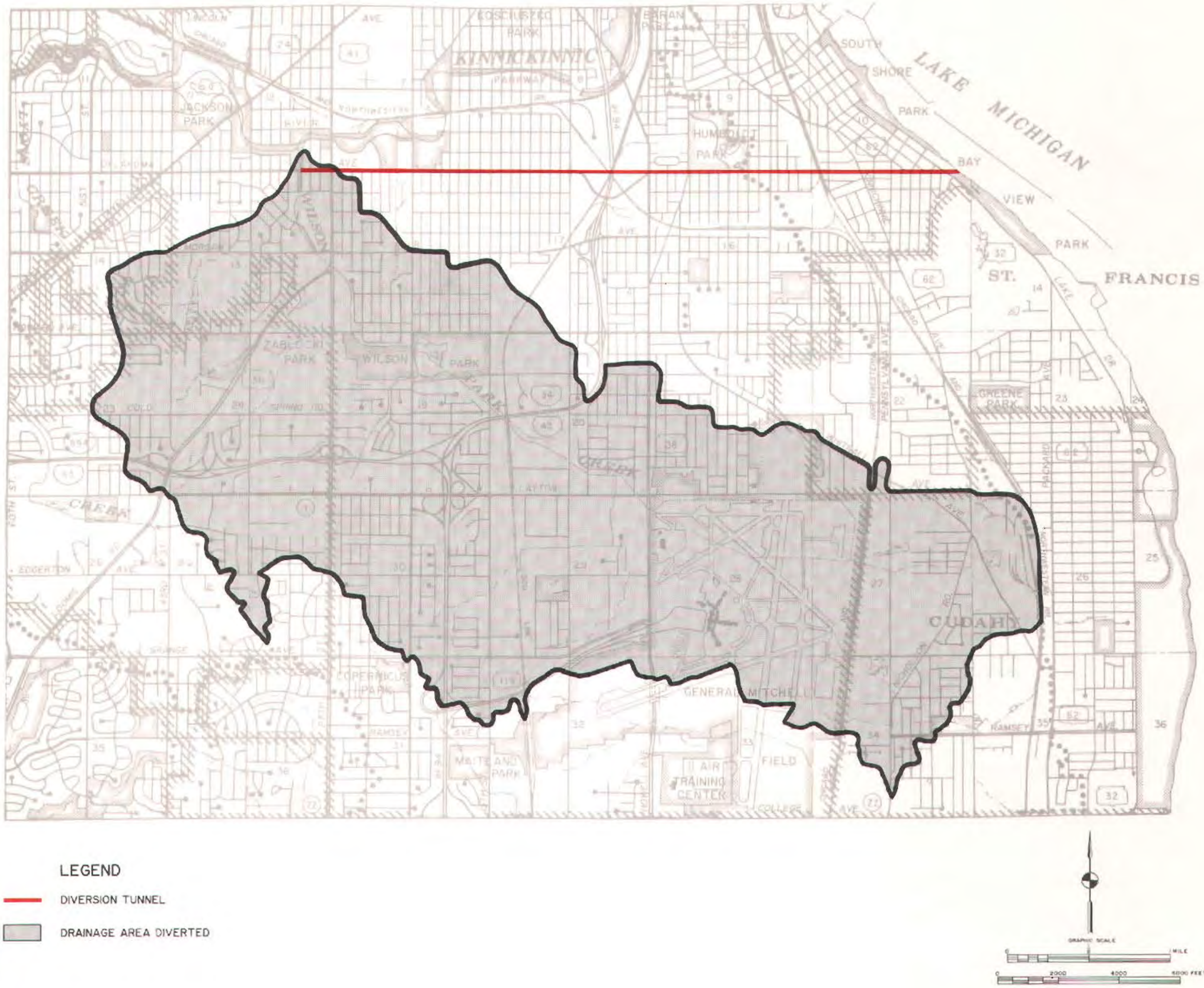
Alternative System Plan 2—Diversion of Flow from Wilson Park Creek: Under this flood control alternative, all flow from Wilson Park Creek for floods up to and including the 100-year recurrence interval event would be diverted to Lake Michigan at South Shore Park by a tunnel to be constructed along Oklahoma Avenue. Flows in excess of the 100-year recurrence interval event would continue to discharge to the Kinnickinnic River. The proposed tunnel would be about 21,700 feet in length and would have a diameter of about 18 feet, assuming a total available drop of 39 feet between the point of diversion and Lake Michigan. This alternative is illustrated on Map 53. The impact of this diversion tunnel on flood discharges along the Kinnickinnic River for both the 100-year recurrence interval flood and the August 6, 1986, flood event is shown in Table 29. As shown in this table, this alternative would reduce the 100-year recurrence interval flood discharge on the Kinnickinnic River by 35 to 45 percent. The August 6, 1986, flood event discharge would be reduced by 35 to 52 percent. It should be noted, however, that implementation of this diversion alternative would not serve to reduce or eliminate flood problems along Wilson Park Creek. Additional flood control measures would be required to alleviate those problems.

An alternative diversion tunnel that could be considered in any more detailed feasibility studies would route the diversion north at about Howell Avenue to the upper reaches of the inner harbor at about Chase Avenue.

Utilizing an annual interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this major diversion is estimated at $4,669,000. This includes the amortization of the $73,305,000 capital cost of the diversion tunnel, and $21,000 in annual operation and maintenance costs. The average annual flood damage abatement benefit was estimated at $50,000, yielding a benefit-cost ratio of 0.01. As with the first diversion alternative, the average annual benefit is based on a reduction of damages from floods having a recurrence interval of greater than 100 years.

Map 53

DIVERSION ALTERNATIVE SYSTEM 2: DIVERSION OF FLOODWATER FROM WILSON PARK CREEK



Source: SEWRPC.

Evaluation of Floodwater Diversion Alternatives for Wilson Park Creek and the Kinnickinnic River

Both of the floodwater diversion alternatives considered would serve to reduce flood discharges along the Kinnickinnic River. Alternative Plan 2—diversion of flow from Wilson Park Creek—would have the greater impact on flood discharges along the Kinnickinnic River. This alternative would not, however, reduce flood discharges or flood-related damages along Wilson Park Creek. Additional flood control measures for Wilson Park Creek would therefore be required. Alternative Plan 1—diversion of flow from Wilson Park Creek at the confluence with Holmes Avenue Creek—would have a lesser impact on flood discharges along the Kinnickinnic River. This alternative would, however, reduce flood discharges along Wilson Park Creek and would alleviate flood damage along Wilson Park Creek in the City of Milwaukee for floods up to, and including, the 100-year recurrence interval event. Also, the capital cost of this alternative would be significantly lower than the capital cost of Alternative Plan 2.

Because of the high costs associated with the floodwater diversion alternatives, it is unlikely that either alternative could be implemented utilizing funds from either the Milwaukee Metropolitan Sewerage District or the City of Milwaukee. If there is sufficient local interest in a floodwater diversion system, state or federal funds for the project should be sought. If such funds do become available, it is recommended that Alternative Plan 1, providing for diversion of flow from Wilson Park Creek at the confluence with Holmes Avenue Creek, be implemented, since this alternative offers flood discharge reductions along more of the stream system in the watershed. Since Alternative Plan 2 would serve to divert floodwaters from the Kinnickinnic River directly to Lake Michigan within the South Shore breakwater, any further consideration of that alternative should include an evaluation of the impacts on water quality conditions within both the inner and outer harbors.

Recommended Flood Control System for Wilson Park Creek-Edgerton Channel

Based upon consideration of the technical feasibility, economic viability, environmental impacts, potential public acceptance, and practicality of each of the alternatives considered, it was recommended that a revised Alternative Plan 3 calling for major channelization with turf lining be adopted for Wilson Park Creek in the City of Milwaukee; and that Alternative Plan 11—major channelization with turf lining and channel enclosure—be adopted for Edgerton Channel in the City of Cudahy. Refinements have been made to the previously recommended plan for Wilson Park Creek in the City of Milwaukee based upon a final detailed design which followed the system planning. These refinements are included in the recommended plan which is described below.

The total capital cost of the recommended combined flood control plan for Wilson Park Creek-Edgerton Channel is estimated at $2,674,000 in 1986 dollars. Annual operation and maintenance costs are estimated at $4,500. The recommended plan is shown on Map 54. The peak flood profile that would be attendant to planned future land use and channel conditions in the subwatershed is shown in Figure 26. Flood discharges that may be expected along Wilson Park Creek as a result of the recommended channel modification and enclosure are shown in Table 30. The recommended plan is not expected to have a significant impact on flood flows and stages along the Kinnickinnic River. The stages would not be altered by more than 0.5 foot and the stages would be contained within the improved channel of the Kinnickinnic River, since that section of the river was designed assuming the implementation of improvements similar to those being recommended for Wilson Park Creek.

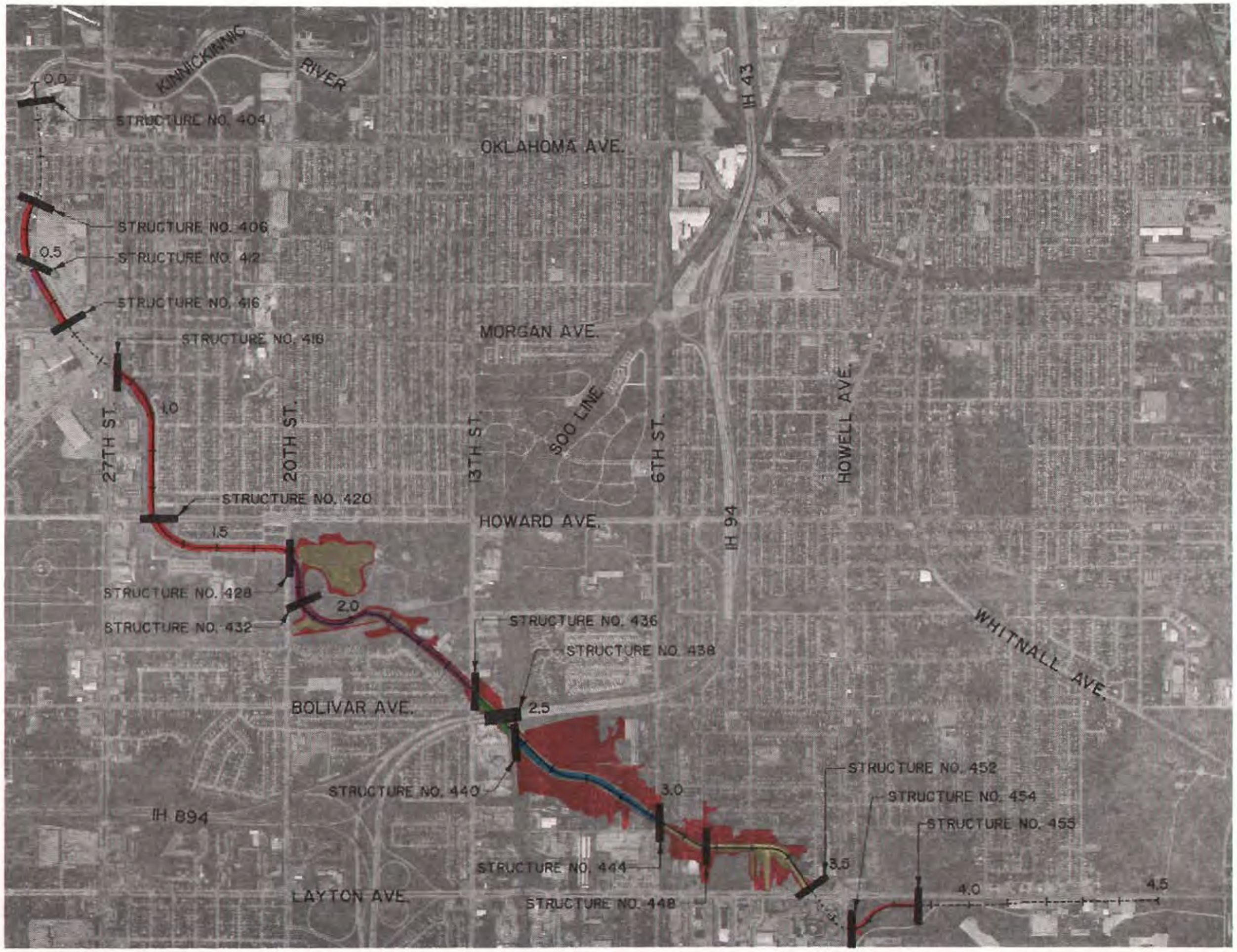
Implementation of the recommended plan would essentially eliminate all flood-related damages to existing structures along the entire Wilson Park Creek channel for floods up to, and including, the 100-year recurrence interval event under planned land use conditions. The plan is more fully described below.

The recommended flood control plan for Wilson Park Creek in the City of Milwaukee is shown on Map 54, and consists of completing major channel modifications along the 1.3-mile reach between S. 6th Street and S. 20th Street. Between S. 13th Street and S. 20th Street, the channel would be lowered up to 5.4 feet, with the resulting channel having a bottom width of 20 feet and side slopes of one foot vertical on 2.5 feet horizontal. Within this reach, the channel would be lined with rip-rap up to an elevation that is two feet above the channel invert, with the remainder to be lined with turf. Between S. 13th Street and the Soo Line Railroad crossing at River Mile 2.57, the channel would be lowered up to five feet, with the resulting channel having a bottom width of 14 feet and side slopes of one foot vertical on two feet horizontal. Within this reach, the channel would be lined with concrete up to an elevation that is seven feet above the channel invert, with the remainder to be turf lined. Between River Mile 2.57 and S. 6th Street, the channel would be lowered up to 4.8 feet, with the resulting channel having a bottom width of 20 feet and side slopes varying from one foot vertical on three feet horizontal to one foot vertical on four feet horizontal. Within this reach, the channel would again be lined with rip-rap up to an elevation that is two feet above the channel invert, with the remainder being turf lined. In addition to the concrete lining noted above, a concrete channel lining would also be utilized at all bridge transitions, at the confluence with Villa Mann Creek, and at two storm sewer outfalls. No bridge reconstruction would be required, as all bridges within this 1.3-mile reach have been constructed to accommodate the proposed channel invert.

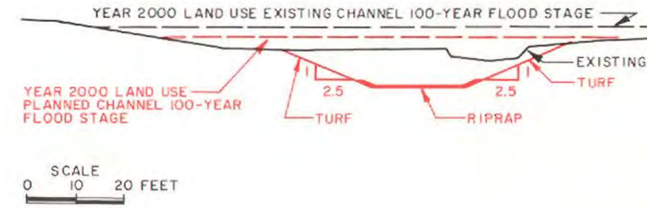
Map 54

RECOMMENDED FLOOD CONTROL PLAN FOR WILSON PARK CREEK—EDGERTON CHANNEL

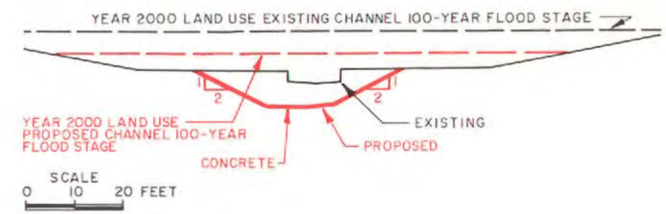
CITY OF MILWAUKEE



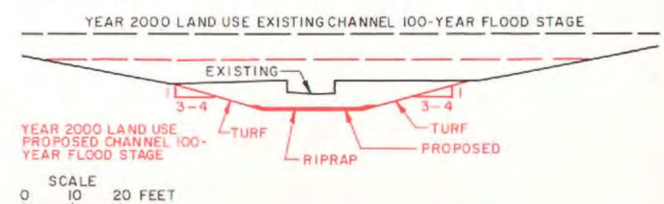
TYPICAL CROSS SECTION OF THE EXISTING AND PROPOSED CHANNEL ALONG WILSON PARK CREEK BETWEEN S. 20TH STREET AND S. 13 TH STREET.



TYPICAL CROSS SECTION OF THE EXISTING AND PROPOSED CHANNEL ALONG WILSON PARK CREEK BETWEEN S. 13TH STREET AND THE SOO LINE RAILROAD



TYPICAL CROSS SECTION OF THE EXISTING AND PROPOSED CHANNEL ALONG WILSON PARK CREEK BETWEEN THE SOO LINE RAILROAD AND S. 6TH STREET



LEGEND

- 1.0 — APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING
- 100-YEAR RECURRENCE INTERVAL FLOODPLAIN - YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS
- 100-YEAR RECURRENCE INTERVAL FLOODPLAIN - YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS
- MAJOR CHANNEL MODIFICATION (TURF LINING WITH RIPRAP INVERT AND 2.5 : 1 SIDE SLOPES)
- MAJOR CHANNEL MODIFICATION (CONCRETE LINING WITH 2 : 1 SIDE SLOPES)
- MAJOR CHANNEL MODIFICATION (TURF LINING WITH RIPRAP INVERT AND 4 : 1 SIDE SLOPES)

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR WILSON PARK CREEK IS SHOWN IN APPENDIX H

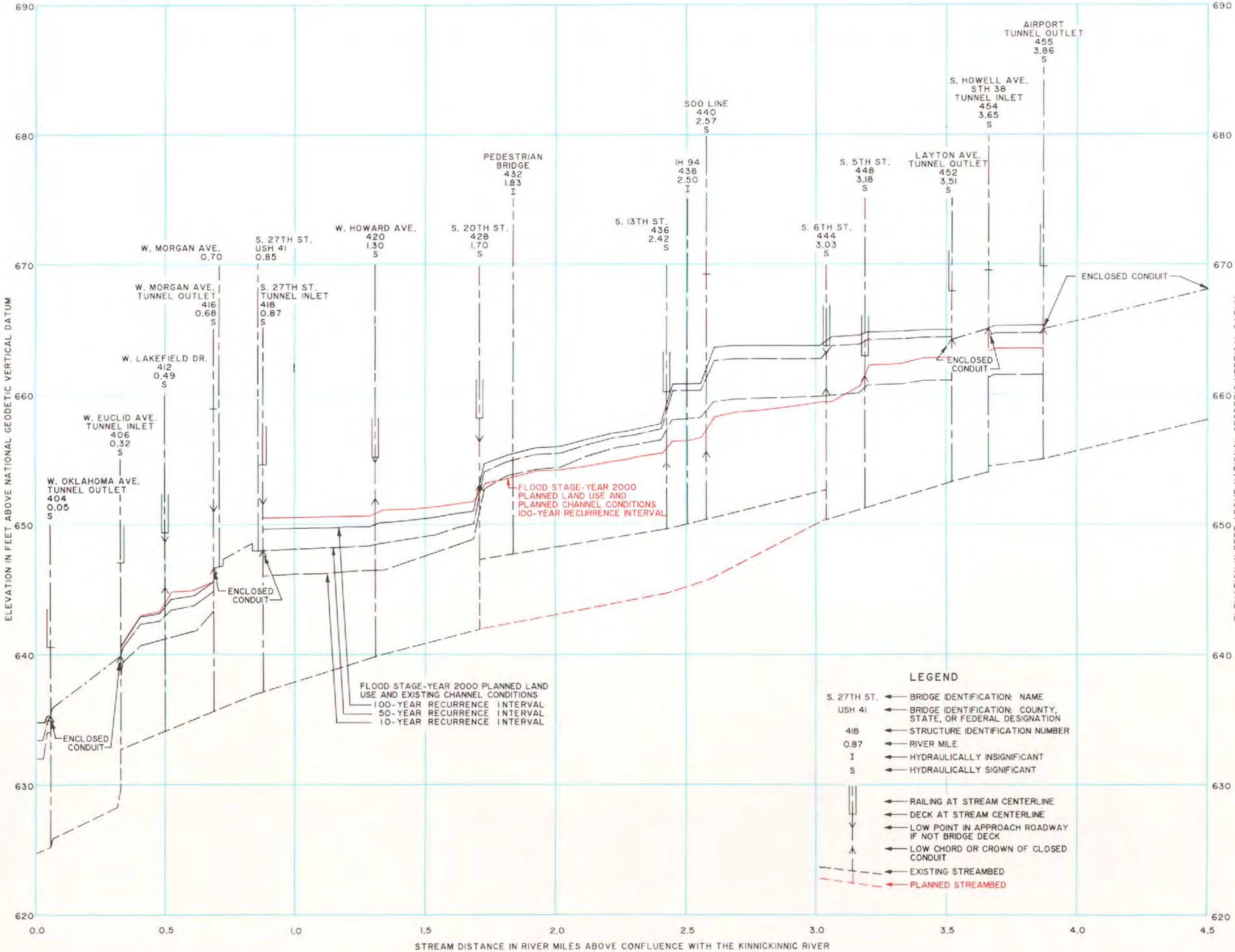
DUE TO MAP SCALE LIMITATIONS, THE DIFFERENCE BETWEEN THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS, AND THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS, MAY NOT APPEAR ON THIS MAP. WHERE NO DIFFERENCE APPEARS REFERENCE SHOULD BE MADE TO THE FLOOD STAGE PROFILE SHOWN BELOW

DATE OF PHOTOGRAPHY: APRIL 1986

Figure 26

RECOMMENDED PLAN FLOOD STAGE PROFILE FOR WILSON PARK CREEK

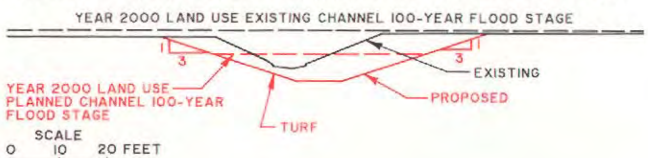
CITY OF MILWAUKEE



Map 54 (continued)

CITY OF MILWAUKEE

TYPICAL CROSS SECTION OF THE EXISTING AND PROPOSED CHANNEL ALONG EDGERTON CHANNEL BETWEEN RIVER MILE 5.28 AND RIVER MILE 5.66 AND BETWEEN S. NICHOLSON AVENUE AND S. WHITNALL AVENUE



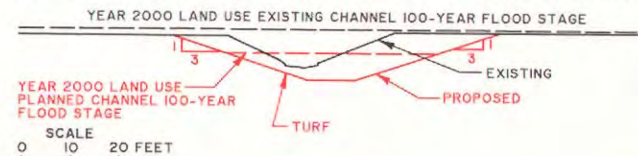
LEGEND

- 100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS
- PROPOSED CHANNEL MODIFICATIONS
- PROPOSED BRIDGE REPLACEMENT
- PROPOSED STRUCTURE REMOVAL
- 1.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

DATE OF PHOTOGRAPHY: APRIL 1986

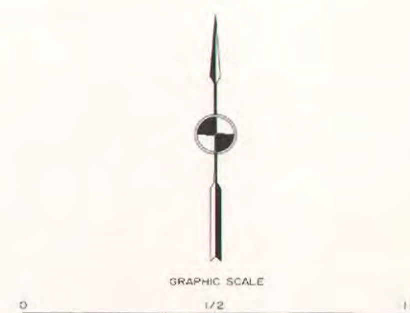
CITY OF CUDAHY

TYPICAL CROSS SECTION OF THE EXISTING AND PROPOSED CHANNEL ALONG EDGERTON CHANNEL BETWEEN RIVER MILE 5.28 AND RIVER MILE 5.66 AND BETWEEN S. NICHOLSON AVENUE AND S. WHITNALL AVENUE



LEGEND

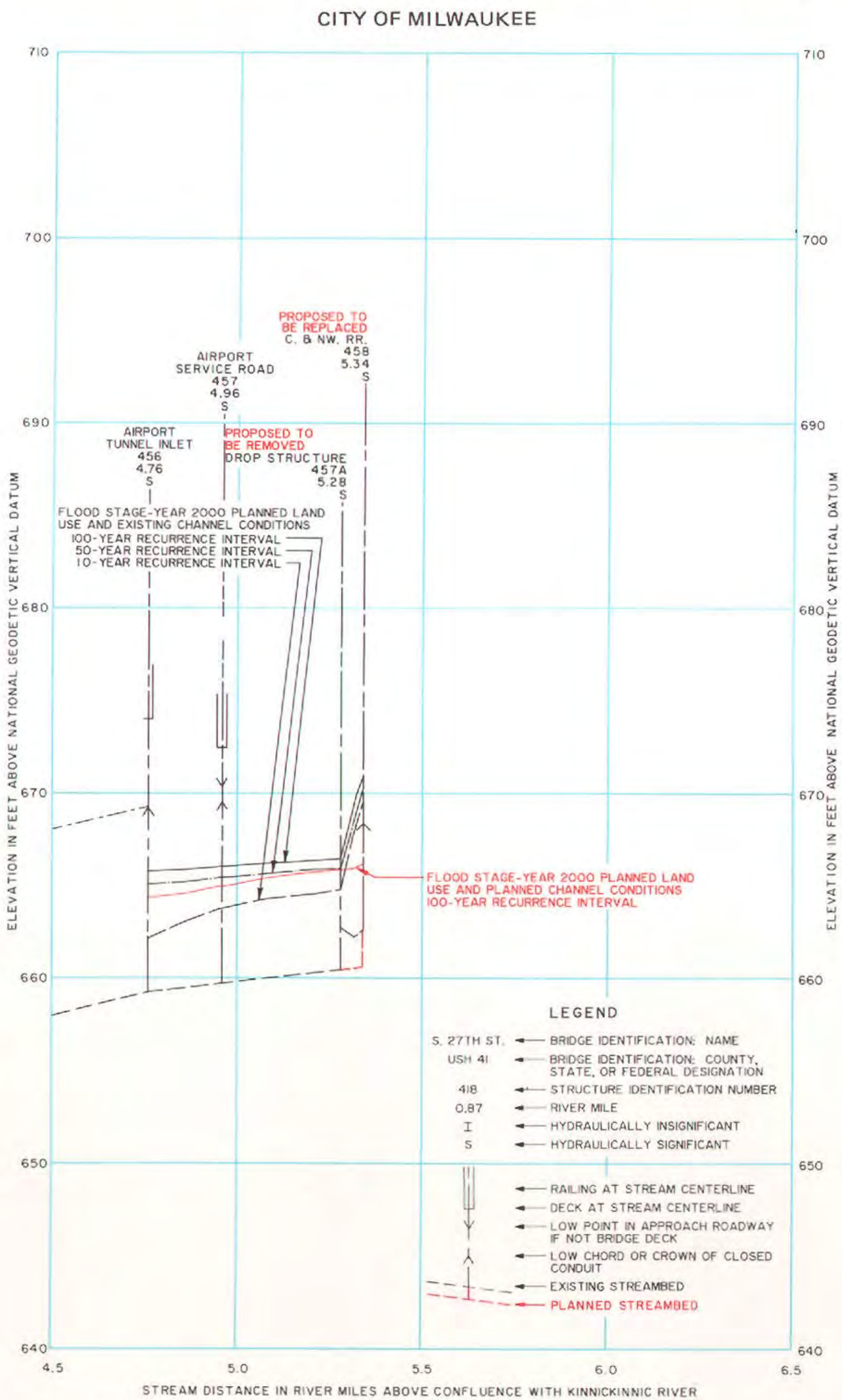
- 100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS
- PROPOSED ENCLOSED CONDUIT
- PROPOSED CHANNEL MODIFICATIONS
- PROPOSED BRIDGE REPLACEMENT
- 1.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

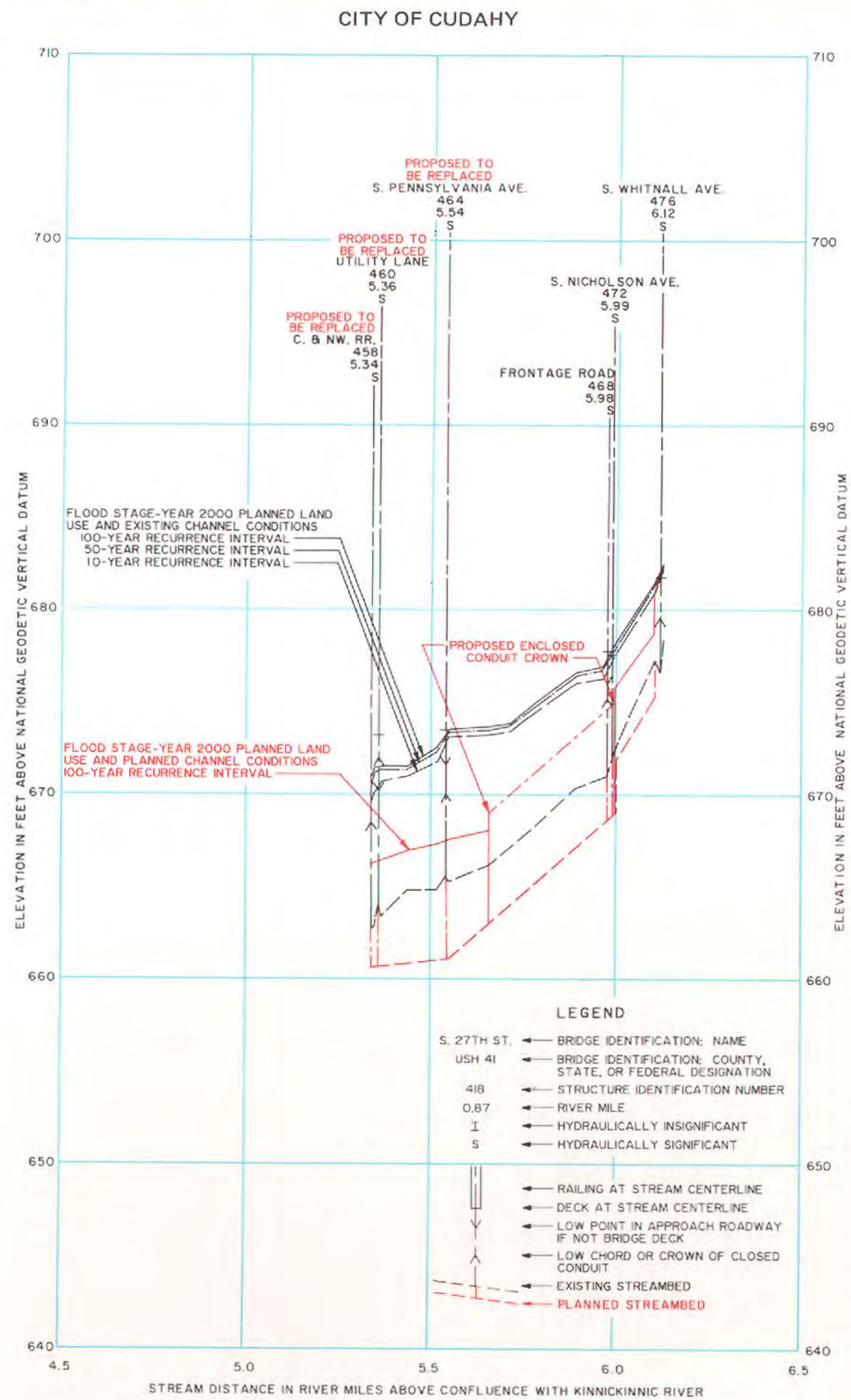


DATE OF PHOTOGRAPHY: APRIL 1986

Source: SEWRPC.

Figure 26 (continued)





Source: SEWRPC.

Table 30

IMPACT OF RECOMMENDED FLOOD CONTROL PLAN FOR WILSON PARK CREEK-EDGERTON CHANNEL ON 100-YEAR RECURRENCE INTERVAL FLOOD DISCHARGE

Location	River Mile	100-Year Recurrence Interval Flood Discharges—Year 2000 Planned Land Use		Percent Increase
		Existing Channel Condition	Recommended Plan Condition	
Mouth of Wilson Park Creek	0.00	3,070	3,090	1
W. Morgan Avenue Tunnel	0.68	2,600	2,840	9
Upstream of Confluence with Villa Mann Creek	1.88	1,880	2,340	24
Layton Avenue Tunnel Outlet . . .	3.51	710	830	17
Airport Tunnel Outlet	3.85	620	620	0

Source: SEWRPC.

The recommended flood control plan for Edgerton Channel in the City of Cudahy is shown on Map 54, and consists of widening and deepening about 0.5 mile of the channel—0.4 mile between the upstream limit of the existing airport channelization and River Mile 5.66, and 0.1 mile between S. Nicholson Avenue and S. Whitnall Avenue. The existing streambed would be lowered 2.1 to 4.5 feet, with the resulting channel having a bottom width of 10 feet and side slopes of one on three. In addition, 0.3 mile of the channel between River Mile 5.66 and S. Nicholson Avenue is to be enclosed in a concrete box culvert approximately 10 feet wide and 6 feet high.

Since the recommended flood control plan for Edgerton Channel is different from that which was recommended under the Kinnickinnic River watershed plan, this portion of the system plan would constitute an amendment to the comprehensive watershed plan.

It is recommended that when the bridge at S. Whitnall Avenue is replaced for transportation purposes, it be designed so as to accommodate the 50-year recurrence interval flood flow without overtopping the attendant roadway.

Finally, it is recommended that large-scale topographic maps be prepared for Wilson Park Creek, including the Edgerton Channel. As noted previously in this chapter, orthophotographs are available for Wilson Park Creek downstream of W. Layton Avenue. Those maps do not, however, reflect recent channel modifications completed by the Milwaukee Metropolitan Sewerage District downstream of S. 20th Street. Also, those maps often do not provide topographic information for those areas beyond the immediate vicinity of the channel. Such information is useful in determining low-lying areas where storm sewer surcharging or local stormwater ponding could cause structure flooding. Large-scale topographic mapping for the Edgerton Channel was prepared in 1958 and does not represent more recent development near the channel. Since the new maps would serve multiple purposes, no cost has been assigned to the flood control plan.

Flood Control and Related Drainage System Plan Implementation

It is recommended that the structural measures developed for the abatement of flood problems along Wilson Park Creek be implemented expeditiously through the cooperative efforts of the City of Cudahy, Milwaukee County, and the Milwaukee Metropolitan Sewerage District. More specifically, it is recommended that the District design, construct, and maintain the major channel modifications recommended along Wilson Park Creek in the City of Milwaukee between S. 6th Street and S. 20th Street, a distance of 1.3 miles, and along Edgerton Channel in the City of Cudahy, a distance of 0.5 mile. It is further recommended that the District design, construct, and maintain the box culvert intended to enclose Edgerton Channel in the City of Cudahy between River Mile 5.66 and S. Nicholson Avenue, a distance of 0.3 mile. Finally, it is recommended that the District prepare large-scale topographic maps for those areas along Wilson Park Creek and the Edgerton Channel.

It is further recommended that the City of Cudahy reconstruct the roadway over Edgerton Channel at S. Nicholson Avenue and construct a replacement bridge at S. Pennsylvania Avenue upon completion of the recommended channel modification and enclosure by the District. It is also recommended that the City assist the District in acquiring the necessary construction easements and rights-of-way.

It is further recommended that, upon the completion of the recommended channel modification by the District, the District work with the Chicago & North Western Transportation Company in the reconstruction of the railway crossing at River Mile 5.34 and of the utility road at River Mile 5.36.

Finally, it is recommended that Milwaukee County cooperate fully in the major channelization through the provision of attendant construction easements and rights-of-way. It is also recommended that the County modify, as necessary, the pedestrian bridge located in Wilson Park in order to accommodate the proposed channel.

The capital costs associated with the various components of the recommended plan are summarized by agency in Table 31.

S. 43RD STREET DITCH SUBWATERSHED FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN

South 43rd Street Ditch was not studied under previous Commission planning programs. Analyses of the hydrologic and hydraulic characteristics of the Ditch and the subwatersheds tributary to the Ditch were accordingly conducted under this system planning effort.

Table 31

SUMMARY OF RECOMMENDED PLAN CAPITAL COSTS

Implementing Agency	Improvements	Estimated Capital Cost
Milwaukee Metropolitan Sewerage District	Channel Improvements	$ 652,000
	Channel Enclosure	1,058,000
	Bridge Removal and Replacement[a,b]	930,000
	Subtotal	$ 2,640,000
Milwaukee County	Bridge Modification	$ 30,000
	Subtotal	$ 30,000
City of Cudahy	Bridge Replacement[b]	$ 4,000
	Subtotal	$ 4,000
	Total	$ 2,674,000

[a] *Cost for bridge replacement may be allocated to Chicago & North Western Transportation Company.*

[b] *No cost has been assigned for the removal and replacement of the S. Pennsylvania Avenue bridge since this structure is scheduled for replacement under the Commission's adopted regional transportation plan.*

Source: SEWRPC.

Overview of the Study Area

South 43rd Street Ditch is a tributary of the Kinnickinnic River. The S. 43rd Street Ditch subwatershed is primarily located within the City of West Allis; however, the ditch is located in the Village of West Milwaukee and the City of Milwaukee. Of the total 1.10-mile perennial stream length, 0.76 mile is located in the Village of West Milwaukee and the remaining 0.34 mile is located in the City of Milwaukee. South 43rd Street Ditch drains an area of about 1.69 square miles (see Map 55). Of this total drainage area, 1.16 square miles, or about 69 percent, lie within the City of West Allis; 0.43 square mile, or about 25 percent, lies within the Village of West Milwaukee; and 0.10 square mile, or about 6 percent, lies within the City of Milwaukee.

From its origin near the intersection of S. 50th Street extended and W. Rogers Street extended, S. 43rd Street Ditch flows 0.15 mile in an easterly direction to the W. Electric Avenue bridge. From that bridge, the ditch flows 0.29 mile in a generally easterly direction to S. 43rd Street. At that point, the ditch will enter a double concrete box culvert with two 6-foot by 16-foot cells. The double box culvert is currently under construction as part of the S. 43rd Street Arterial Project. The culvert will run 0.32 mile in a southerly direction to a point just north of W. Lincoln Avenue, where it will transition to an existing double 7-foot by 10.5-foot concrete box. That structure runs about 200 feet in a southerly direction underneath W. Lincoln Avenue before turning 90 degrees and running in an easterly direction for about 155 feet. At that point, the box culvert transitions to a double 8.4-foot by 11-foot structural plate pipe arch culvert which runs a total of 360 feet easterly and then southerly. From the pipe arch outlet, the ditch flows 0.1 mile in a southerly direction to a double 7.6-foot by 11.5-foot structural plate pipe arch culvert, which runs 0.10 mile in a southerly direction under tracks of the Chicago & North Western Railway to the Kinnickinnic River. As already noted, the entire 1.10-mile stream length is recommended for District jurisdiction in the policy plan companion to this system plan.

At present, the S. 43rd Street Ditch subwatershed is almost completed developed for urban use, including residential, commercial, industrial, institutional, and urban open space uses. The open space uses are comprised mainly of public parks and open land associated with industrial areas. The developed areas of the S. 43rd Street Ditch subwatershed are generally provided with a full range of municipal street improvements, including paved streets with curbs and gutters and attendant storm sewers. Accordingly, surface runoff is generally conveyed rapidly from each individual site to the ditch through storm sewers.

Specific data on pertinent characteristics of the watershed, such as hydrologic soil types, land slopes, and land use, appear in Chapter II of this report. This system plan assumes that the subwatershed has reached ultimate development conditions and that the land use will undergo little change by the design year of the system plan.

Map 55

THE S. 43RD STREET DITCH SUBWATERSHED

Source: SEWRPC.

Flooding and Related Drainage Problems

Prior to construction of the double 6-foot by 16-foot box culvert along S. 43rd Street, localized flooding occurred almost annually along the north-south reach of the ditch north of W. Lincoln Avenue. There is no history of flooding problems along the east-west reach of the ditch. During the flood of March 30, 1960, stormwater inundation occurred on the Village of West Milwaukee-City of Milwaukee boundary along W. Lincoln Avenue between S. 37th Street and S. 43rd Street. Street flooding was reported and several buildings incurred damage as the result of basement flooding. The flood of August 3, 1960, caused inundation in the same area and electrical equipment was damaged at the Froedtert Malt Corporation in the Village of West Milwaukee. No flooding problems were reported along W. Lincoln Avenue between S. 37th Street and S. 43rd Street during major floods after 1960, including the unusually large event of August 6, 1986. Modifications to the Kinnickinnic River channel and to S. 43rd Street Ditch south of W. Lincoln Avenue following the floods of 1960 apparently reduced flood levels sufficiently to prevent flooding or stormwater drainage problems along W. Lincoln Avenue during subsequent floods.

As noted above, significant reaches of the ditch have been, or are being, enclosed in culverts. Over a period of many years, the entire channel was modified. The east-west reach of the ditch has relatively steep, irregular banks which are heavily overgrown with trees and brush. The open channel reach south of W. Lincoln Avenue has uniformly sloping, well-maintained grass sides with a concrete cunette along the channel bottom.

Flood Discharges and Stages

As noted in Chapter III of this report, the hydrologic model used in developing design discharges for S. 43rd Street Ditch simulates streamflow on a

Table 32

FLOOD DISCHARGE FOR S. 43RD STREET DITCH FOR YEAR 2000
LAND USE AND EXISTING CHANNEL AND STORM SEWER CONDITIONS

Location	River Mile	Peak Flood Discharge (cubic feet per second)		
		10-Year	50-Year	100-Year
Mouth at Kinnickinnic River (Chicago & North Western Railway Tunnel Outlet) . . .	0.00	520	630	670
Chicago & North Western Railway Tunnel Inlet	0.10	520	630	670
Immediately Upstream of Chicago & North Western Railway Tunnel Inlet	0.11	490	600	640
W. Lincoln Avenue Tunnel Outlet.	0.20	490	600	640
S. 43rd Street Tunnel Inlet	0.66	440	510	540
Upstream of S. 43rd Street Tunnel Inlet . . .	0.71	400	470	490
Downstream of W. Electric Avenue Bridge . .	0.93	330	380	400
W. Electric Avenue Bridge.	0.95	330	380	400
Upstream End of Ditch at S. 50th Street Extended	1.10	320	370	380

Source: SEWRPC.

continuous basis using recorded climatological data as input. Flood discharges are developed by conducting discharge-frequency analyses of simulated annual peak discharges generated by the hydrologic model according to the log Pearson Type III method of analysis, as recommended by the U. S. Water Resources Council and as specified by the Wisconsin Department of Natural Resources. These analyses were conducted at five locations along S. 43rd Street Ditch. The flood discharges that were developed were then checked by incorporating the discharges into a hydraulic model to develop stages and comparing those stages to available historical high-water observations. Such observations were available at the W. Electric Avenue bridge from employees of adjacent businesses.

The estimated peak flood discharges under year 2000 planned land use and existing (1987) channel conditions are set forth in Table 32. Flood stage profiles were determined for the 10-, 50-, and 100-year recurrence interval runoff events under planned land use and existing channel conditions. These profiles, which encompass the full 1.10-mile-long reach of the S. 43rd Street Ditch studied, constitute a graphic representation of the flood stages along the ditch under the specified recurrence interval flood discharges. In addition to providing an overall representation of flood stages relative to familiar points of reference such as the channel bottom and bridge deck surfaces, the profiles, because of their continuity, permit the determination of flood stages at any point along the stream channel. The flood profiles are shown in Figure 27.

Flood profiles were determined using surveyed cross-sections, surveyed bridge data, and Wisconsin Department of Transportation (WisDOT) construction drawings for the S. 43rd Street arterial box culvert. There are no current large-scale topographic maps available for the S. 43rd Street Ditch area north of W. Lincoln Avenue; therefore, the extent of flooding during the 100-year recurrence interval flood was delineated on Map 56 using field-surveyed data on ground and building elevations, supplemented by Wisconsin Department of Transportation construction drawings for the S. 43rd Street arterial box culvert. Because of the lack of large-scale topographic mapping, the delineation of inundated areas on Map 56 is only an approximation.

The data presented above indicate that under year 2000 land use and existing and committed channel conditions, the 100-year recurrence interval flood in S. 43rd Street Ditch would not cause any flooding of structures, collector streets, arterial streets, or main railways. During the 100-year recurrence interval flood, there could be shallow flooding of the open area located south of the ditch, west of S. 43rd Street, north of the railway spur for the General Electric Company plant buildings, and east of the General Electric Company warehouse located immediately adjacent to the ditch, as shown on Map 56. The warehouse entrance elevation—652.8 feet National Geodetic Vertical Datum (NGVD)—should, however, be well above the 100-year recurrence interval flood level of 649.8 feet NGVD. There could also be shallow flooding of the open area to the west of the General Electric Company warehouse and between the warehouse and the General Electric Company factory, but no buildings should be flooded. The 100-year recurrence interval flood would be completely contained within the double box culvert that is being constructed as part of the S. 43rd Street arterial project. The flooding that occurred along S. 43rd Street in the past should be abated by the double box culvert.

Recommended Flood Control
System for S. 43rd Street Ditch

Because no flooding of structures, collector streets, arterial streets, or main railways would be expected from a 100-year recurrence interval flood under planned land use conditions, it was not necessary to consider further flood control and drainage alternatives for S. 43rd Street Ditch. No flood control or drainage system measures are recommended other than completion of the 43rd Street arterial double box culvert.

The City of West Allis has sized a larger trunk storm sewer to replace one of the existing trunk sewers in the S. 43rd Street subwatershed. The trunk sewer drains the subbasin located south of W. Burnham Street and west of the ditch. The stormwater drainage problems in that subbasin are not severe enough to merit construction of the new trunk sewer at the present time, but, as portions of the existing sewer system require replacement, it is likely that all, or part, of the trunk sewer will be constructed. The new trunk sewer was designed to handle the peak runoff from a five-year recurrence interval rainfall event, accounting for development that has occurred since construction of the existing storm sewer. The larger trunk sewer pipes would have a greater hydraulic capacity than the existing pipes; therefore, construction of the larger sewer would increase the 10-, 50-, and 100-year recurrence interval flows in S. 43rd Street Ditch downstream of the storm sewer outfall. The increase in the various recurrence interval flows can be determined by comparing Tables 32 and 33.

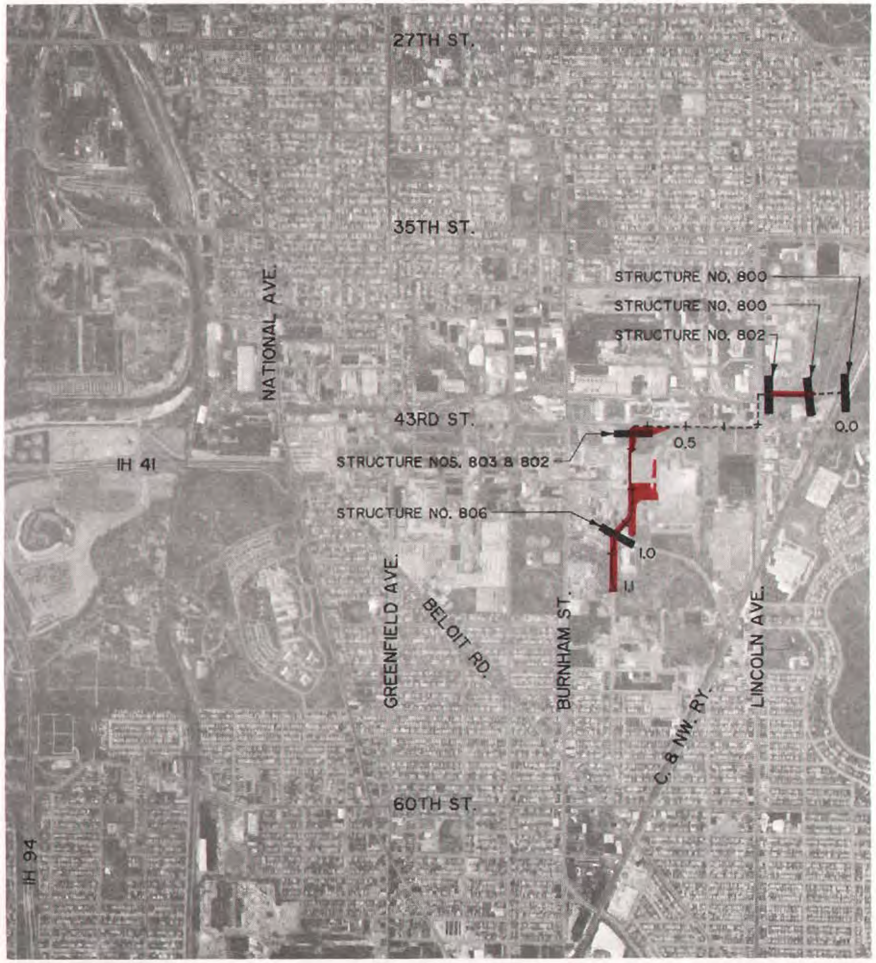
The greater 100-year recurrence interval flood flows following completion of the new trunk sewer as presently sized by the City would increase flood levels in S. 43rd Street Ditch by about one to three feet. Although the lack of large-scale topographic mapping precludes the precise assessment of damages along the ditch under these flood conditions, the survey data collected during the preparation of this system plan along with the Wisconsin Department of Transportation plans for the S. 43rd Street arterial indicate that structure flooding and flooding of collector and arterial streets could occur. There could be some flooding of S. 43rd Street and W. Lincoln Avenue as a result of flow leaving the channel upstream of the S. 43rd Street arterial box culvert. There could also be some flooding of buildings along S. 43rd Street and along the ditch west of the box culvert. The box culvert headwall would not be overtopped.

Three alternative structural flood control measures for abating flood damages along S. 43rd Street Ditch resulting from the increased flows from the trunk sewer were considered. These include construction of dikes and floodwalls, construction of dikes in conjunction with debrushing of the channel banks, and channel widening and deepening.

Each of the three alternatives is technically feasible and should eliminate flooding of structures, collector streets, and arterial streets during the 100-year recurrence interval flood. The dike construction and channel debrushing alternative is considered to be the most economically feasible based on an estimated capital cost of $135,000 and an annual maintenance cost of $8,500. That alter-

Map 56

100-YEAR RECURRENCE INTERVAL FLOODPLAIN FOR S. 43RD STREET DITCH UNDER YEAR 2000 PLANNED LAND USE WITH EXISTING CHANNEL CONDITIONS



LEGEND

100 YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

1.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR THE SOUTH 43RD STREET DITCH IS SHOWN IN APPENDIX H

GRAPHIC SCALE

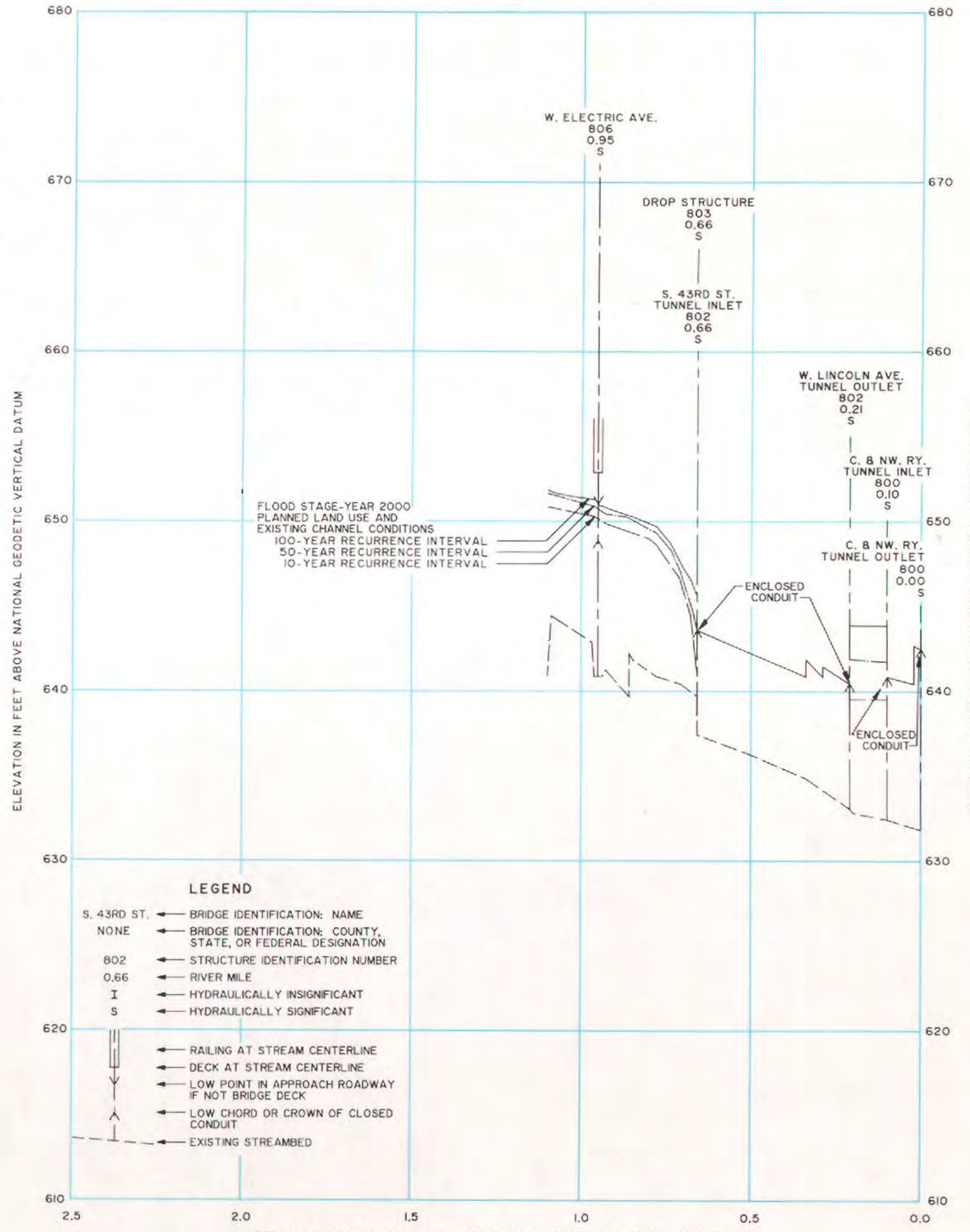
0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Source: SEWRPC.

Figure 27

FLOOD STAGE AND STREAMBED PROFILE FOR S. 43RD STREET DITCH



Source: SEWRPC.

Table 33

FLOOD DISCHARGES FOR S. 43RD STREET DITCH FOR YEAR 2000 LAND USE, EXISTING CHANNEL CONDITIONS, AND NEW CITY OF WEST ALLIS TRUNK SEWER

Location	River Mile	Peak Flood Discharge (cubic feet per second)		
		10-Year	50-Year	100-Year
Mouth at Kinnickinnic River (Chicago & North Western Railway Tunnel Outlet) . . .	0.00	580	760	840
Chicago & North Western Railway Tunnel Inlet	0.10	580	760	840
Immediately Upstream of Chicago & North Western Railway Tunnel Inlet	0.11	550	750	820
W. Lincoln Avenue Tunnel Outlet.	0.20	550	750	820
S. 43rd Street Tunnel Inlet	0.66	530	690	750
Upstream of S. 43rd Street Tunnel Inlet . . .	0.71	510	660	720
Downstream of W. Electric Avenue Bridge . .	0.93	460	600	660
W. Electric Avenue Bridge.	0.95	460	600	660
Upstream End of Ditch at S. 50th Street Extended	1.10	450	590	640

Source: SEWRPC.

native includes construction of a 250-foot-long dike with an average height of about 2.5 feet across a swale between the Chicago & North Western Railway culvert inlet at River Mile 0.10 and the railroad tracks; construction of a 280-foot-long dike with an average height of 5.5 feet along the south bank just upstream of the S. 43rd Street arterial box culvert; construction of a 450-foot-long dike with an average height of 6.0 feet along the right bank between River Miles 0.79 and 0.87; construction of a 360-foot-long dike with an average height of 4.5 feet along the south bank from River Mile 0.87 to the W. Electric Avenue bridge at River Mile 0.95; and annual clearing and debrushing of the main channel of the ditch from the S. 43rd Street arterial box culvert inlet at River Mile 0.66 to the upstream end of the ditch at River Mile 1.10. Because of the lack of large-scale topographic mapping, all dike lengths and heights are approximate.

The implementation of this plan element is not required until such time as the upstream sewer system is enlarged. It is recommended that the dike construction and channel debrushing alternative be refined when the City of West Allis decides to construct the new trunk sewer. It is suggested that the final trunk sewer design be coordinated with the refinement of the flood control measures for S. 43rd Street Ditch to enable selection of the most acceptable combination of storm sewer modification and flood control measures. It is also recommended that the Milwaukee Metropolitan Sewerage District prepare large-scale topographic mapping of the areas along S. 43rd Street Ditch following construction of the S. 43rd Street arterial and enclosure of the ditch along S. 43rd Street. Since these topographic maps would serve multiple purposes, no cost has been assigned to the flood control plan.

VILLA MANN CREEK AND VILLA MANN CREEK TRIBUTARY SUBWATERSHEDS FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLANS

Villa Mann Creek and Villa Mann Creek Tributary were not studied under previous Commission planning programs. Analyses of the hydrologic and hydraulic characteristics of the creek and tributary and their subwatersheds were conducted for this system plan.

Overview of the Study Area

Villa Mann Creek Tributary is a tributary of Villa Mann Creek. Villa Mann Creek is a tributary of Wilson Park Creek which is a tributary of the Kinnickinnic River. All of the 0.76-mile length of Villa Mann Creek is classified as perennial stream and is located in the City of Milwaukee. Of the 0.88-mile length of Villa Mann Creek Tributary, 0.65 mile is classified as perennial stream and the remaining 0.23 mile is classified as intermittent stream. Of the 0.65 mile of perennial stream, 0.27 mile is located in the City of Milwaukee and 0.38 mile is located in the City of Greenfield. All of the 0.23 mile of intermittent stream is located in the City of Greenfield. The perennial stream lengths of Villa Mann Creek and Villa Mann Creek Tributary are recommended for District jurisdiction in the policy plan companion to this system plan.

Villa Mann Creek Tributary drains an area of about 0.66 square mile (see Map 57). Of this total drainage area, 0.62 square mile, or about 94 percent, lies within the City of Greenfield, and 0.04 square mile, or about 6 percent, lies within the City of Milwaukee.

Excluding the area drained by Villa Mann Creek Tributary, Villa Mann Creek drains an area of 0.65 square mile. Of this total drainage area, 0.50 square mile, or about 77 percent, lies within the City of Milwaukee, and 0.15 square mile, or about 23 percent, lies within the City of Greenfield.

From its origin, Villa Mann Creek Tributary flows 0.23 mile east and then north to W. Colony Drive, where it passes through two four-foot-diameter

Map 57

THE VILLA MANN CREEK SUBWATERSHED

Source: SEWRPC.

concrete culverts. From those culverts the stream flows 0.15 mile in a generally easterly direction to IH 894, where it flows to the north through a 5.5-foot by 8.0-foot box culvert. The stream then flows 0.23 mile in a generally northeasterly direction to the S. 27th Street tunnel, which runs 0.27 mile in a generally easterly direction to the confluence with Villa Mann Creek. The tunnel consists of a 318-foot-long, 5-foot by 9.7-foot concrete box culvert followed by two 1,033-foot-long concrete culverts, one 66 inches in diameter and one 60 inches in diameter.

From its origin just to the north of IH 894, Villa Mann Creek flows 0.13 mile in a northeasterly direction to the confluence with Villa Mann Creek Tributary. Beginning at the confluence, and extending 0.63 mile downstream to the mouth of the creek, the stream channel has been widened, deepened, and partially lined with concrete. From the confluence, the creek runs 0.23 mile in a generally northeasterly direction to W. Bolivar Avenue, where it passes through a double 6-foot by

12-foot box culvert. Downstream from W. Bolivar Avenue, the creek flows 0.16 mile in a northerly direction to W. Plainfield Avenue, where it passes through another double 6-foot by 12-foot box culvert. From W. Plainfield Avenue, the creek flows 0.17 mile in a generally easterly direction to the S. 20th Street bridge. There are two drop structures located in that reach. Downstream from S. 20th Street, the creek flows 0.07 mile in a generally northeasterly direction to the confluence with Wilson Park Creek.

Present land use in the Villa Mann Creek and Villa Mann Creek Tributary subwatersheds is predominantly medium-density residential, with some areas in high-density residential, transportation, local commercial, governmental and institutional, and park uses. The developed areas of the two subwatersheds are generally provided with a full range of municipal street improvements, including paved streets with curbs and gutters and attendant storm sewers. Accordingly, surface runoff is generally conveyed rapidly from each individual site to the streams through storm sewers.

Specific data on pertinent characteristics of the watershed, such as hydrologic soil types, land slopes, and land use, appear in Chapter II of this report. This system plan assumes that the subwatershed land use conditions will undergo little change by the design year of the system plan.

Flooding and Related Drainage Problems

The channel widening and deepening, partial concrete lining, and bridge and culvert replacement project undertaken along the lower 0.63-mile of Villa Mann Creek in the 1960's was presumably in response to some then-existing or anticipated flood problems. However, the Cities of Milwaukee and Greenfield have no record of flooding problems along Villa Mann Creek or Villa Mann Creek Tributary aside from an isolated complaint during the unusually large flood of August 6, 1986. A resident along the reach of Villa Mann Creek just upstream of the modified channel who was interviewed during preparation of this system plan reported no flooding problems or exceptionally high water levels at her home in her 29 years of residence. Based on the lack of reported or recorded flooding problems along either Villa Mann Creek or Villa Mann Creek Tributary, it appears that both of the existing stream channels have been adequate to pass the floods that have occurred at least since the 1960's.

Flood Discharges and Stages

As noted in Chapter III of this report, the hydrologic model used in developing design discharges for Villa Mann Creek and Villa Mann Creek Tributary simulates streamflow on a continuous basis using recorded climatological data as input. Flood discharges are developed by conducting discharge-frequency analyses of simulated annual peak discharges generated by the hydrologic model according to the log Pearson Type III method of analysis, as recommended by the U. S. Water Resources Council and as specified by the Wisconsin Department of Natural Resources. These analyses were conducted at three locations along Villa Mann Creek and three locations along Villa Mann Creek Tributary. The estimated peak flood discharges under year 2000 planned land use and existing (1987) channel conditions for Villa Mann Creek and Villa Mann Creek Tributary are set forth in Tables 34 and 35, respectively. Flood stage profiles were determined for the 10-, 50-, and 100-year recurrence interval runoff events under planned land use and existing channel conditions. These profiles, which encompass the full 0.76-mile-long reach of Villa Mann Creek and the 0.65-mile-long reach of Villa Mann Creek Tributary recommended for District jurisdiction, constitute a graphic representation of the flood stages along the creek and tributary under the specified recurrence interval flood discharges. In addition to providing an overall representation of flood stages relative to familiar points of reference such as the channel bottom and bridge deck surfaces, the profiles, because of their continuity, permit the determination of flood stages at any point along the stream channel. The flood profiles are shown in Figures 28 and 29.

Flood profiles for Villa Mann Creek were determined using surveyed cross-sections, surveyed bridge data, and City of Milwaukee as-built construction drawings for the 0.63 mile of improved channel. Flood profiles for Villa Mann Creek Tributary were determined using cross-sections from large-scale topographic maps and surveyed bridge data.

The data described above indicated that under year 2000 land use and existing channel conditions, the 100-year recurrence interval floods in Villa Mann Creek and Villa Mann Creek Tributary would not cause any flooding of structures, collector streets, arterial streets, or freeways and expressways, as shown on Maps 58 and 59. For both Villa Mann Creek and Villa Mann Creek Tributary, the 100-

Table 34

FLOOD DISCHARGES FOR VILLA MANN CREEK FOR YEAR 2000 LAND USE AND EXISTING CHANNEL CONDITIONS

Location	River Mile	Peak Flood Discharge (cubic feet per second)		
		10-Year	50-Year	100-Year
Mouth at Wilson Park Creek	0.00	360	530	600
S. 20th Street Bridge	0.07	360	530	600
W. Plainfield Avenue	0.24	360	530	600
W. Bolivar Avenue	0.40	360	530	600
Immediately Upstream of W. Bolivar Avenue	0.41	290	440	510
Confluence with Villa Mann Creek Tributary	0.63	290	440	510
Immediately Upstream of Confluence with Villa Mann Creek Tributary	0.64	120	170	190

Source: SEWRPC.

Table 35

FLOOD DISCHARGES FOR VILLA MANN CREEK TRIBUTARY FOR YEAR 2000 LAND USE AND EXISTING CHANNEL CONDITIONS

Location	River Mile	Peak Flood Discharge (cubic feet per second)		
		10-Year	50-Year	100-Year
Mouth at Villa Mann Creek	0.00	180	270	320
S. 27th Street Tunnel Inlet	0.27	160	240	290
IH 894	0.50	160	240	290
Immediately Upstream of IH 894	0.51	130	200	230
W. Colony Drive	0.65	130	200	230

Source: SEWRPC.

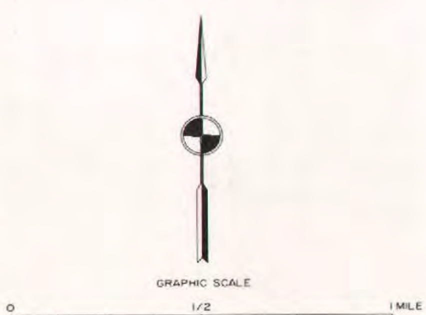
Map 58
100-YEAR RECURRENCE INTERVAL FLOODPLAIN FOR VILLA MANN CREEK UNDER YEAR 2000 LAND USE WITH EXISTING CHANNEL CONDITIONS

LEGEND

100 YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

1.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

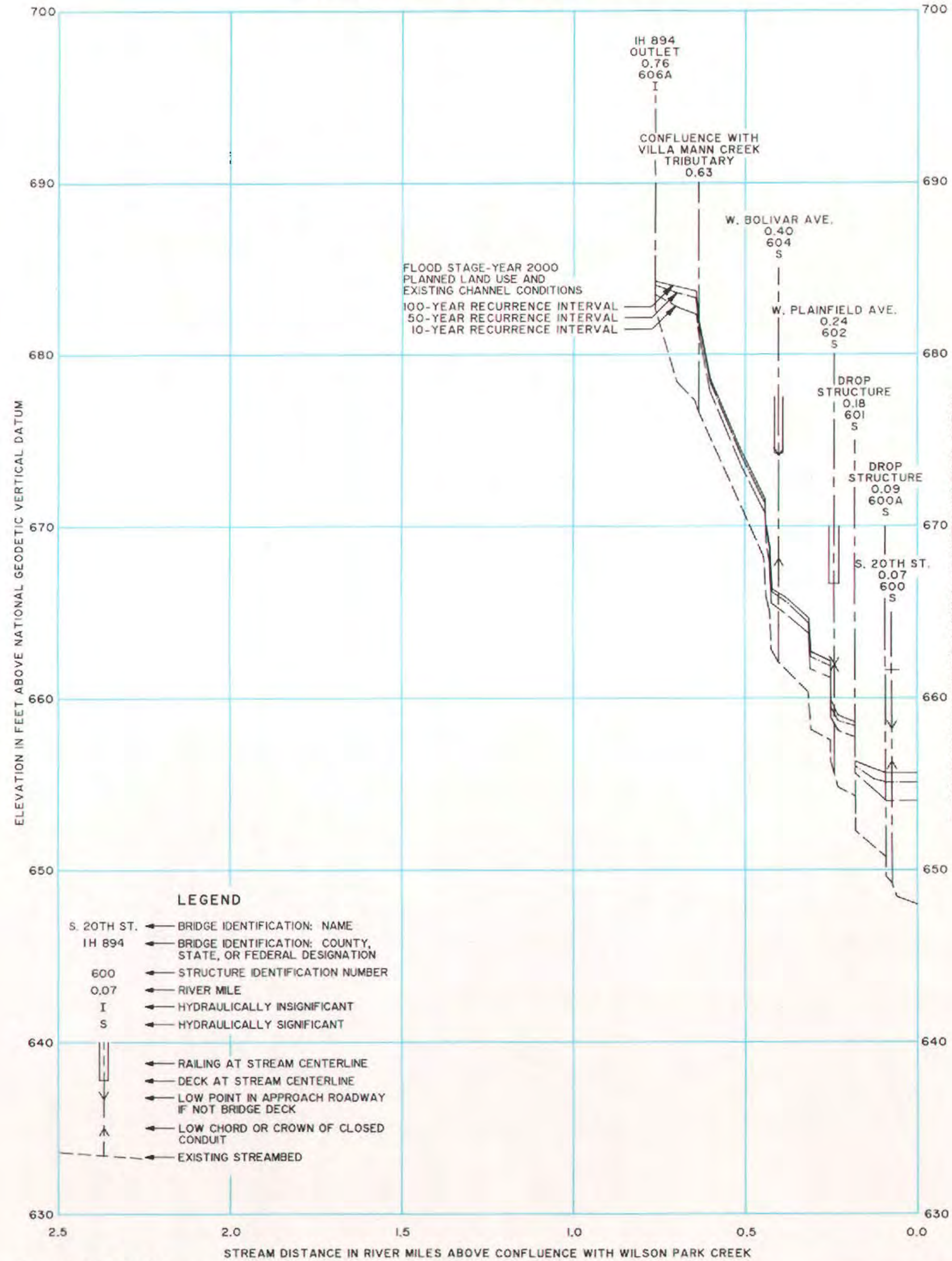
NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR VILLA MANN CREEK IS SHOWN IN APPENDIX H



DATE OF PHOTOGRAPHY: APRIL 1986

Source: SEWRPC.

Figure 28
FLOOD STAGE AND STREAMBED PROFILE FOR VILLA MANN CREEK



Source: SEWRPC.

Map 59

100-YEAR RECURRENCE INTERVAL FLOODPLAIN FOR VILLA MANN CREEK TRIBUTARY UNDER YEAR 2000 LAND USE WITH EXISTING CHANNEL CONDITIONS

LEGEND

100 YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

1.0 — APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR VILLA MANN TRIBUTARY IS SHOWN IN APPENDIX H

GRAPHIC SCALE

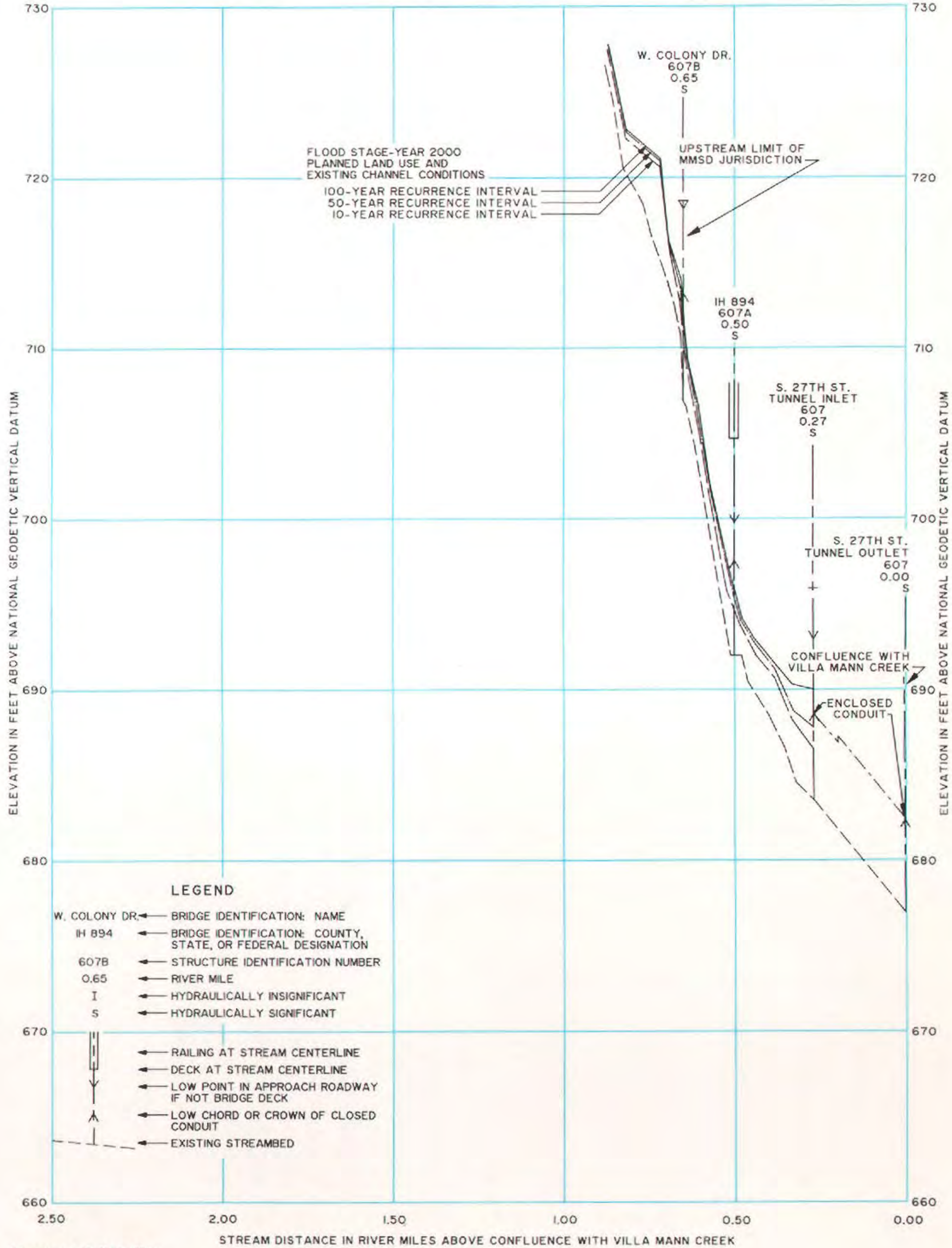
0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Source: SEWRPC.

Figure 29

FLOOD STAGE AND STREAMBED PROFILE FOR VILLA MANN CREEK TRIBUTARY



Source: SEWRPC.

year recurrence interval flood would be contained within the main stream channel with the exception of several reaches along Villa Mann Creek where undeveloped open areas could be flooded. The open areas along Villa Mann Creek that could be flooded during a 100-year recurrence interval flood include the area downstream of S. 20th Street within Wilson Park and the area along the 0.1-mile reach upstream of the confluence with Villa Mann Creek Tributary.

Recommended Flood Control System for Villa Mann Creek and Villa Mann Creek Tributary

Because no flooding of structures, collector streets, arterial streets, or freeways and expressways would be expected to result from a 100-year recurrence interval flood under year 2000 land use conditions, it was not necessary to consider further flood control and drainage alternatives for Villa Mann Creek and Villa Mann Creek Tributary. Therefore, no flood control or drainage system measures are recommended.

Because of a lack of large-scale topographic mapping for Villa Mann Creek, the floodplain boundaries shown on Maps 58 and 59 were delineated using surveyed cross-section data and construction drawings for the channel modifications completed by the City of Milwaukee. Therefore, this delineation can be considered only an approximation. It is recommended that the Milwaukee Metropolitan Sewerage District prepare large-scale topographic maps for Villa Mann Creek, at which time a more precise delineation of the floodplain should be made. Large-scale topographic mapping for Villa Mann Creek Tributary upstream of S. 27th Street is currently available from the City of Greenfield. Such mapping is useful not only in delineating floodplains, but also in determining stormwater drainage patterns to the stream, as well as in identifying areas away from the channel where storm sewer backup may result in structure flooding. Since these topographic maps would serve multiple purposes, no cost has been assigned to the flood control plan.

Chapter V

EVALUATION OF ALTERNATIVES AND SELECTION OF RECOMMENDED FLOOD CONTROL PLAN—OAK CREEK WATERSHED

INTRODUCTION

Three streams in the Oak Creek watershed are recommended to be under the jurisdiction of the Milwaukee Metropolitan Sewerage District. These three streams, totaling 22.2 lineal miles in length, are Oak Creek, North Branch Oak Creek, and the Mitchell Field Drainage Ditch. All three of these streams have been studied under the Commission's Oak Creek watershed planning program.[1] The data on existing and probable future flood problems, alternative and recommended flood control measures and related costs, and recommended means for implementation presented herein were developed under this previous planning program. The costs of the recommended measures, however, have been updated to a 1986 base.

OVERVIEW OF THE STUDY AREA

About 64 percent of the area of the Oak Creek watershed lies within the City of Oak Creek. The remaining 36 percent of the watershed area lies within the Cities of Cudahy, Franklin, Greenfield, Milwaukee, and South Milwaukee. The Oak Creek watershed, as shown on Map 60, is a 27.24-square-mile surface water drainage basin that discharges to Lake Michigan in the City of South Milwaukee. In 1980, a 0.76-square-mile area was diverted from the direct Lake Michigan drainage basin to the Oak Creek drainage basin, the area diverted being shown on Map 60. This interbasin diversion increased the area of the Oak Creek drainage basin by 2.9 percent. Oak Creek acts as an estuary of Lake Michigan from its mouth to the first Oak Creek Parkway bridge, a distance of about 0.3 mile. As shown on Map 60, the source of Oak Creek which, as a perennial stream, has a length of about 13.1 miles, is in Section 24, Town 5 North, Range 21 East in the City of Franklin. From its source near the intersection of Sherwood Drive and Southland Drive, the creek flows southerly for approximately 0.5 mile to just south of Ryan Road and thence easterly for approximately 5.2 miles to 15th Avenue and the Oak Creek Parkway, and then finally flows southeasterly for about 3.0 miles to its confluence with Lake Michigan.

The North Branch of Oak Creek which, as a perennial stream, has a length of about 5.8 miles, has its origin in Section 31, Town 6 North, Range 22 East in the City of Milwaukee just south of the IH 94/General Mitchell International Airport (Mitchell Field) spur freeway interchange. From this origin, the stream flows southerly to its confluence with Oak Creek north of Ryan Road between 13th Street and Howell Avenue.

The third perennial stream in the Oak Creek watershed recommended to be under the jurisdiction of the Milwaukee Metropolitan Sewerage District is the Mitchell Field Drainage Ditch, which has a perennial length of about 2.4 miles and an intermittent length of about 0.9 mile. From its source east of the Mitchell Field north-south runway in Section 33, Town 6 North, Range 22 East in the City of Milwaukee, the stream flows southerly to its confluence with Oak Creek north of Drexel Avenue.

In 1980, about 15 square miles of the watershed area, or 53 percent of the total area of the watershed, was still in rural use, with agriculture and related open uses occupying about 12 square miles, or about 45 percent of the total watershed area. In 1980, urban land uses occupied about 13 square miles, or about 47 percent of the total area of the watershed, of which residential land use accounted for over five square miles, or over 19 percent of the total watershed area. Also of significance is the transportation, communication, and utilities land use category, which accounted for over five square miles, or about 19 percent of the total watershed area. From 1963 to 1980, approximately 3.6 square miles, or 13 percent of the watershed, was converted from rural to urban

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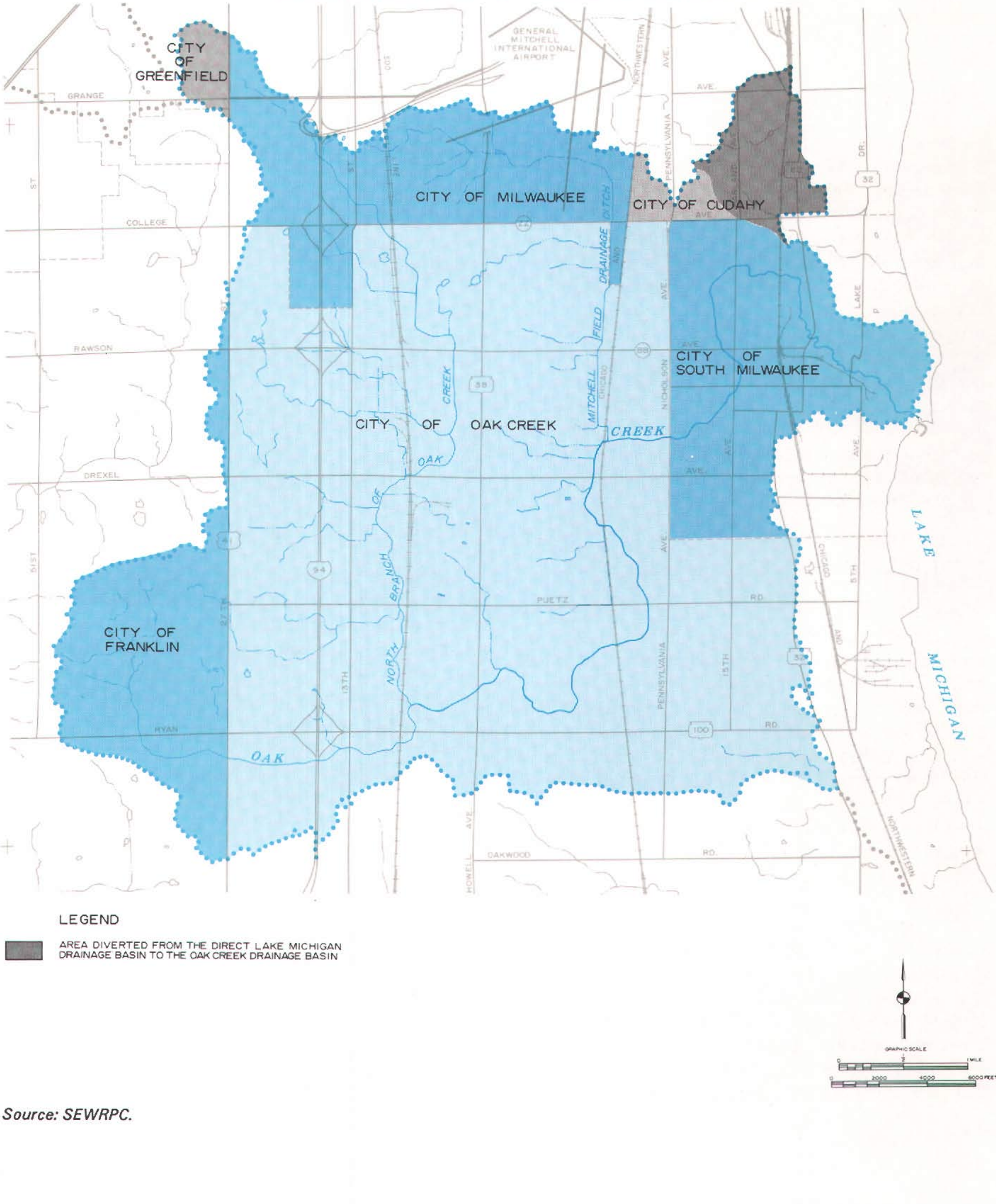
[1] *See SEWRPC Planning Report No. 36, A Comprehensive Plan for the Oak Creek Watershed, August 1986.*

Map 60

CIVIL DIVISIONS IN THE OAK CREEK WATERSHED



Source: SEWRPC.

use, resulting in a rate of urbanization of about 0.2 square mile per year.

In order to meet the needs of the expected resident population and employment, the amount of land devoted to urban use within the watershed is projected to increase from the 1980 total of about 13 square miles, or about 47 percent of the total area of the watershed, to about 24 square miles by the year 2000, or about 87 percent of the total area of the watershed. The demand for urban land will have to be satisfied primarily through the conversion of a large portion of the remaining agricultural and other open lands of the watershed to urban uses. Such rural land uses may thus be expected to decline collectively from about 14.6 square miles in 1980 to about 3.5 square miles in the year 2000, a decrease of about 76 percent between 1980 and 2000. Therefore, in this report it is assumed that the watershed will be fully urbanized in the year 2000.

Flooding of the stream system of the Oak Creek watershed has been, and in the absence of corrective action may be expected to continue to be, a common and natural occurrence. In portions of the watershed, the streams leave their channels and occupy portions of the adjacent natural floodplains almost annually as a result of late winter-early spring snowmelt or snowmelt-rainfall events, or in response to spring, summer, and fall thunderstorms. Unnecessary occupancy of the natural floodlands by flood-vulnerable land uses, together with development-induced changes in the flow characteristics of the streams, has produced serious flood problems in the watershed. Some of these problems, but not all, have been at least partially resolved through the construction of channel improvements. In some cases channel improvements have aggravated flooding problems downstream.

FLOODING AND RELATED DRAINAGE PROBLEMS

Research of the available historical records indicated the occurrence of eight major flooding periods in the Oak Creek watershed in the recent past. These major floods, each of which caused significant damage to property as well as disruption of normal social and economic activities in the watershed, occurred on June 22, 1917; June 23, 1940; March 30, 1960; June 11, 1967; June 26, 1968; September 18, 1972; April 21, 1973; and September 13, 1978. Findings of the research into historical flood problems leads to the conclusion that flood problems in the urbanized portions of the Oak Creek watershed are relatively minor compared to flood damages in agricultural areas. The majority of the urban flood damages have been concentrated in the southwest area of the City of South Milwaukee. Major channel improvements made to Oak Creek between Rawson Avenue and Pennsylvania Avenue have done much to alleviate flooding in this area.

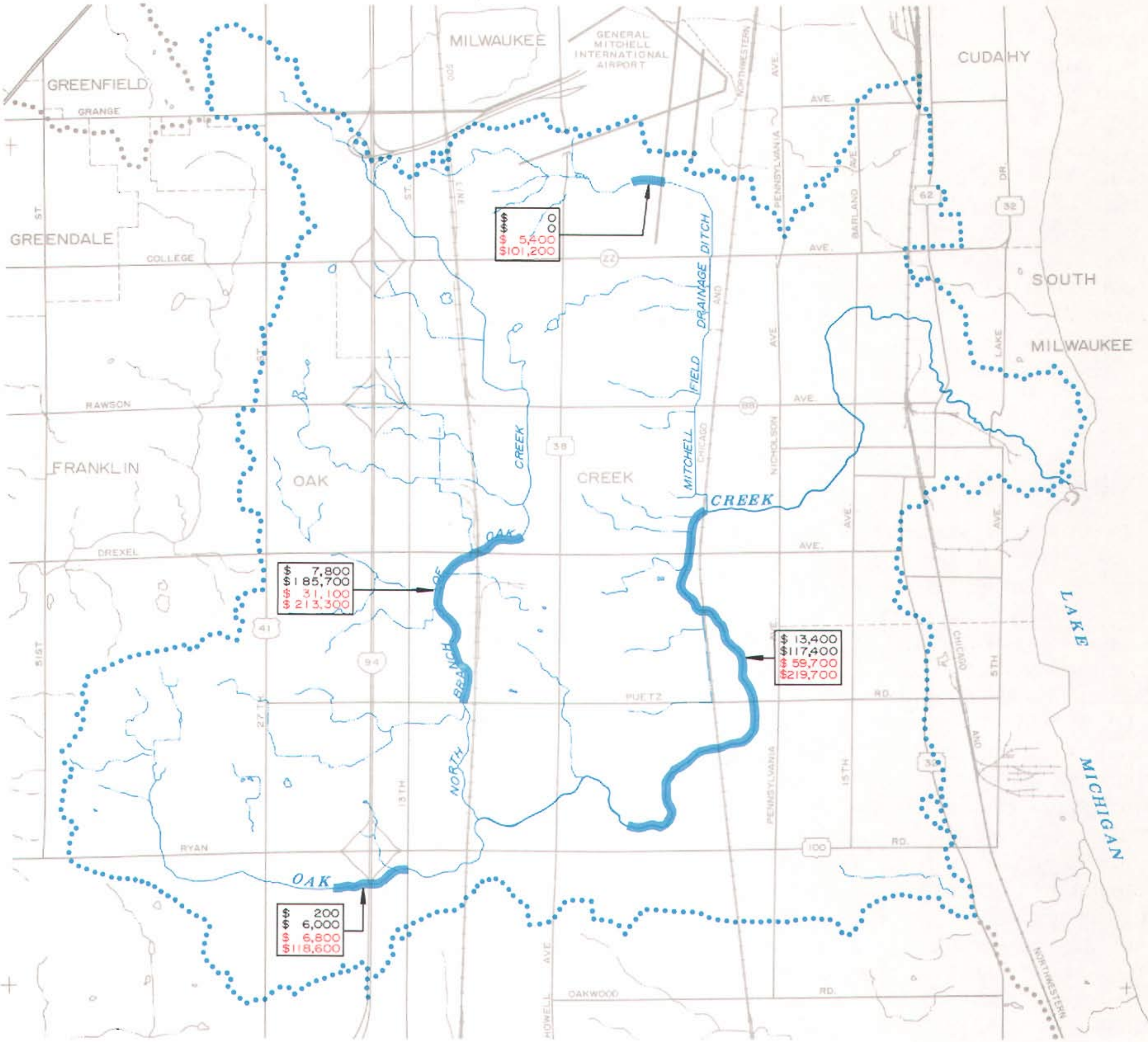
Because until recently the Oak Creek watershed has been primarily rural in nature, relatively few residences have been flooded in the past. However, uncontrolled urbanization and lack of adequate floodplain management could result in significant increases in flooding damage, not only to existing but also to future residential development in the watershed.

The areas that most frequently experience flooding problems in the watershed are outlined on Map 61. The total average annual monetary flood damages within the Oak Creek watershed resulting from direct overland flooding are estimated at $21,400 under 1980 land use conditions, and $103,000 under planned, year 2000 land use conditions. Additional homes and commercial properties may, however, experience indirect flood damages through sanitary sewer backup. In this respect it should again be noted that the flood control measures considered under this system plan are primarily intended to alleviate flood damages from direct overland flooding along the stream studied, as well as to provide an adequate outlet for local storm sewers and drainageways. These measures, however, may be expected to help to reduce flooding due to localized stormwater drainage problems or sanitary sewer backups.

The drainage and flood control objectives and supporting principles and standards set forth in Chapter III specify the flood events which bridges should accommodate without overtopping of the related roadway. Based on those criteria, eight bridges on Oak Creek, four bridges on the North Branch of Oak Creek, and one bridge on the Mitchell Field Drainage Ditch are considered hydraulically inadequate, as shown in Appendix C. These structures are located at E. Forest Hill Avenue, E. Puetz Road, S. Nicholson Road, S. Shepard Avenue, W. Ryan Road, S. 13th Street, S. 20th Street, and W. Puetz Road

Map 61

EXISTING AND POTENTIAL AVERAGE ANNUAL AND 100-YEAR RECURRENCE INTERVAL FLOOD DAMAGES ALONG SELECTED STREAM REACHES IN THE OAK CREEK WATERSHED



LEGEND

SELECTED FLOOD DAMAGE REACH

ESTIMATED FLOOD DAMAGES

EXISTING

$ 13,400 AVERAGE ANNUAL
$117,400 100-YEAR RECURRENCE INTERVAL

FUTURE

$ 59,700 AVERAGE ANNUAL
$219,700 100-YEAR RECURRENCE INTERVAL

NOTE: ESTIMATED FUTURE FLOOD DAMAGES ASSUME YEAR 2000 PLANNED LAND USE CONDITIONS AND EXISTING CHANNEL CONDITIONS

Source: SEWRPC.

on Oak Creek; W. Puetz Road, Wildwood Drive, S. 6th Street, and W. Marquette Avenue on the North Branch of Oak Creek; and E. College Avenue on the Mitchell Field Drainage Ditch.

FLOOD DISCHARGES AND STAGES

A noted in Chapter III of this report, the hydrologic model used for development of design flood discharges for the Oak Creek watershed uses recorded climatological data as input. Streamflow was simulated on a continuous basis by the hydrologic model and compared to recorded streamflow data, with necessary adjustments made to calibrate the model. Flood discharges were then developed by conducting discharge-frequency analyses of simulated annual peak discharges generated by the hydrologic model. The flood discharges simulated by the hydrologic model were again checked by incorporating the discharges into a hydraulic model to develop stages, and comparing those stages to historical high-water data.

The estimated peak flood discharges under existing and planned, year 2000, land use conditions and existing channel conditions are set forth in Table 36. Flood stage profiles were determined for the 10-, 50-, and 100-year recurrence interval runoff events under planned and existing channel conditions for each of the three streams studied, and constitute graphic representations of flood stages under specified recurrence interval flood discharges. In addition to providing an overall representation of flood stages relative to familiar points of reference such as the channel bottom and bridge deck surfaces, the profiles, because they are continuous, permit the determination of flood stages at any point along the stream channel. The flood profiles for planned land use and existing channel conditions are shown in Figure 30. The extent of the 100-year recurrence interval floodplain under planned land use and existing channel conditions is shown on Map 62 for the Oak Creek watershed.

ALTERNATIVE FLOOD CONTROL PLANS FOR THE OAK CREEK WATERSHED

In preparing the recommended flood control plan under the Commission Oak Creek watershed planning program, the following 12 alternative plans were considered for alleviating the flood damage problems in the watershed: 1) No Action Alternative 1; 2) No Action Alternative 2; 3) Structure Floodproofing, Elevation, and Removal Alternative; 4) Major Channelization Alternative 1; 5) Major Channelization Alternative 2; 6) Major Channelization Alternative 3; 7) Decentralized Storage Alternative; 8) Centralized Storage Alternative; 9) Combination Major Channelization, Channel Deepening and Shaping, and Structure Floodproofing, Elevation, and Removal Alternative; 10) Combination Major Channelization, Channel Deepening and Shaping, Centralized Storage, and Structure Floodproofing, Elevation, and Removal Alternative; 11) Combination Minimum Channel Modification and Structure Floodproofing, Elevation, and Removal Alternative; and 12) Combination Channel Deepening and Shaping, and Structure Floodproofing, Elevation, and Removal Alternative.

Each alternative plan is described briefly below. The economic benefits and costs attendant to each alternative are provided in Table 37. More detailed descriptions of these 12 alternatives are set forth in SEWRPC Planning Report No. 36.

No Action Alternative

One alternative course of action for addressing the flood problems of the Oak Creek watershed is to do nothing—that is, to recognize the inevitability of flooding in the watershed, but to decide not to mount a collective, coordinated program to abate the flood damages. Under this alternative, one of two flood damage situations would remain, depending on which land use development policy for the year 2000 is adopted within the watershed. If it is assumed that new urban development will be allowed to occur within the floodplain fringe areas—that is, within the flood hazard area but outside the floodway—with all new structures being placed on fill or otherwise protected from flood damage, then under year 2000 land use and existing channel conditions, the average annual flood damages in the watershed may be expected to approximate $103,000. If it is assumed that no further development of flood hazard-prone areas within the watershed will be permitted, the average annual flood damages in the watershed may be expected to approximate $98,000. Under either condition, damage to crops would be minor and concentrated in the southeast portion of the watershed. There are no monetary benefits associated with this "do nothing" alternative,

(Continued on Page 189)

Table 36

FLOOD DISCHARGES FOR THE OAK CREEK WATERSHED UNDER EXISTING AND PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

Location	River Mile	Peak Flood Discharge (cfs)					
		Existing Land Use and Existing Channel Conditions			Planned Land Use and Existing Channel Conditions		
		10-Year	50-Year	100-Year	10-Year	50-Year	100-Year
Oak Creek							
Pedestrian Bridge	0.14	1,140	1,590	1,780	1,910	2,540	2,810
1st Oak Creek Parkway Bridge	0.35	1,140	1,590	1,780	1,910	2,540	2,810
2nd Oak Creek Parkway Bridge	0.88	1,140	1,590	1,780	1,910	2,540	2,810
Mill Road	0.94	1,140	1,590	1,780	1,910	2,540	2,810
Oak Creek Parkway Dam	0.95	1,140	1,590	1,780	1,910	2,540	2,810
3rd Oak Creek Parkway Bridge	1.18	1,140	1,590	1,780	1,910	2,540	2,810
4th Oak Creek Parkway Bridge	1.32	1,140	1,590	1,780	1,910	2,540	2,810
Chicago Avenue/STH 32	1.61	1,140	1,590	1,780	1,890	2,510	2,770
5th Oak Creek Parkway Bridge	2.14	1,140	1,590	1,780	1,890	2,510	2,770
Pedestrian Bridge	2.24	1,140	1,590	1,780	1,890	2,510	2,770
Chicago & North Western Railway	2.35	1,140	1,590	1,780	1,890	2,510	2,770
15th Avenue	2.84	1,090	1,560	1,780	1,840	2,440	2,700
Pedestrian Bridge	3.18	1,090	1,560	1,780	1,840	2,440	2,700
Pine Street	3.37	1,090	1,560	1,780	1,840	2,440	2,700
E. Rawson and 16th Avenues	3.65	1,090	1,560	1,780	1,840	2,440	2,700
15th Avenue	3.76	1,090	1,560	1,780	1,840	2,440	2,700
Pedestrian Bridge	3.89	1,090	1,560	1,780	1,840	2,440	2,700
Milwaukee Avenue	4.01	1,090	1,560	1,780	1,840	2,440	2,700
15th Avenue	4.06	1,090	1,560	1,780	1,840	2,440	2,700
Pedestrian Bridge	4.18	1,090	1,560	1,780	1,840	2,440	2,700
S. Pennsylvania Avenue	4.71	1,090	1,560	1,780	1,840	2,440	2,700
Chicago & North Western Railway	5.25	850	1,290	1,500	1,500	2,030	2,270
E. Drexel Avenue	5.56	850	1,290	1,500	1,500	2,030	2,270
Chicago & North Western Railway	6.06	850	1,290	1,500	1,500	2,030	2,270
E. Forest Hill Avenue	6.25	850	1,290	1,500	1,500	2,030	2,270
E. Puetz Road	6.83	850	1,290	1,500	1,500	2,030	2,270
Chicago & North Western Railway	7.34	1,130	1,780	2,080	2,080	2,870	3,220
S. Nicholson Road	7.44	1,130	1,780	2,080	2,080	2,870	3,220
S. Shepard Avenue	8.11	1,130	1,780	2,080	2,080	2,870	3,220
S. Howell Avenue/Northbound STH 38	9.22	1,130	1,780	2,080	2,080	2,870	3,220
S. Howell Avenue/Southbound STH 38	9.24	1,130	1,780	2,080	2,080	2,870	3,220
W. Ryan Road/STH 100	10.06	400	790	1,030	1,030	1,620	1,830
Spillway	10.12	400	790	1,030	1,030	1,620	1,830
Soo Line Railroad	10.24	400	790	1,030	1,030	1,620	1,830
Private Bridge	10.25	400	790	1,030	1,030	1,620	1,830
Private Bridge	10.46	400	790	1,030	1,030	1,620	1,830
Private Bridge	10.60	400	790	1,030	1,030	1,620	1,830
S. 13th Street/CTH V	10.69	400	790	1,030	1,030	1,620	1,830
Pedestrian Bridge	10.72	400	790	1,030	1,030	1,620	1,830
IH 94 Northbound	10.97	310	600	790	690	1,140	1,330
IH 94 Southbound	10.99	310	600	790	690	1,140	1,330
S. 20th Street	11.24	310	600	790	690	1,140	1,330
S. 27th Street/STH 41	11.70	220	430	570	400	700	840
S. 31st Street	11.97	160	310	410	200	390	490
Private Bridge	12.23	160	310	410	200	390	490
W. Ryan Road/STH 100	12.52	160	310	410	200	390	490
Concrete Drop Sill	12.69	160	310	410	200	390	490
Concrete Drop Sill	12.90	160	310	410	200	390	490

Table 36 (continued)

Location	River Mile	Peak Flood Discharge (cfs)					
		Existing Land Use and Existing Channel Conditions			Planned Land Use and Existing Channel Conditions		
		10-Year	50-Year	100-Year	10-Year	50-Year	100-Year
Oak Creek (continued)							
Concrete Drop Sill	13.07	100	170	210	100	170	210
W. Southland Drive	13.18	100	170	210	100	170	210
W. Woodward Drive	13.31	50	90	110	50	90	110
W. Glenwood Drive	13.58	10	30	40	10	30	40
Private Drive	13.60	10	30	40	10	30	40
Private Drive	13.62	10	30	40	10	30	40
W. Maple Crest Drive	13.64	10	30	40	10	30	40
Reservoir Outlet	13.65	10	30	40	10	30	40
Private Bridge	13.76	20	50	60	20	50	60
W. Puetz Road	13.79	20	50	60	20	50	60
Mitchell Field Drainage Ditch							
Chicago & North Western Railway	0.14	350	590	730	580	900	1,050
E. Rawson Avenue/CTH EB	0.80	320	560	680	560	830	950
E. College Avenue/CTH ZZ	1.83	310	450	520	450	560	620
Private Bridge	2.15	310	450	520	450	560	620
Airport Runway Culvert	2.60	240	310	310	305	310	315
Private Bridge	2.74	330	600	740	640	1,010	1,180
Pedestrian Bridge	2.80	330	600	740	640	1,010	1,180
Private Bridge	3.10	330	600	740	640	1,010	1,180
S. Howell Avenue/STH 38	3.31	330	600	740	640	1,010	1,180
North Branch Oak Creek							
Soo Line Railroad	0.10	710	1,400	1,670	1,210	2,000	2,320
Private Bridge	0.21	710	1,400	1,670	1,210	2,000	2,320
Private Bridge	0.34	710	1,400	1,670	1,210	2,000	2,320
W. Puetz Road	0.92	650	1,195	1,450	1,130	1,750	1,940
Private Bridge	1.71	650	1,195	1,450	1,130	1,750	1,940
W. Wildwood Drive	2.00	500	800	930	940	1,190	1,260
W. Drexel Avenue	2.21	500	800	930	940	1,190	1,260
Soo Line Railroad	2.25	430	750	880	890	1,130	1,190
S. 6th Street	2.41	430	750	880	890	1,130	1,190
W. Marquette Avenue	3.04	260	430	520	560	820	900
W. Rawson Avenue/CTH BB	3.51	260	430	520	560	820	900
S. 6th Street	3.86	260	430	520	560	820	900
Spillway	3.90	260	430	520	560	820	900
Spillway	4.20	260	430	520	560	820	900
Private Bridge	4.35	100	140	160	150	220	240
Private Bridge	4.59	100	140	160	150	220	240
Private Bridge	4.62	100	140	160	150	220	240
Private Bridge	4.67	100	140	160	150	220	240
Private Bridge	4.74	100	140	160	150	220	240
Soo Line Railroad	4.75	80	140	150	120	200	220
W. College Avenue/CTH ZZ	4.91	125	235	260	145	250	280
Private Bridge	4.94	170	330	370	190	350	390
S. 13th Street	5.21	170	330	370	190	350	390
W. Ramsey Avenue and IH 94 Box Culvert	5.65	240	360	400	250	370	410
IH 94 Exit Ramp	5.85	240	360	400	250	370	410

Source: SEWRPC.

Map 62

100-YEAR RECURRENCE INTERVAL FLOODPLAIN FOR THE OAK CREEK WATERSHED UNDER YEAR 2000 PLANNED LAND USE WITH EXISTING CHANNEL CONDITIONS

OAK CREEK

LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

1.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR OAK CREEK IS SHOWN IN APPENDIX H

GRAPHIC SCALE

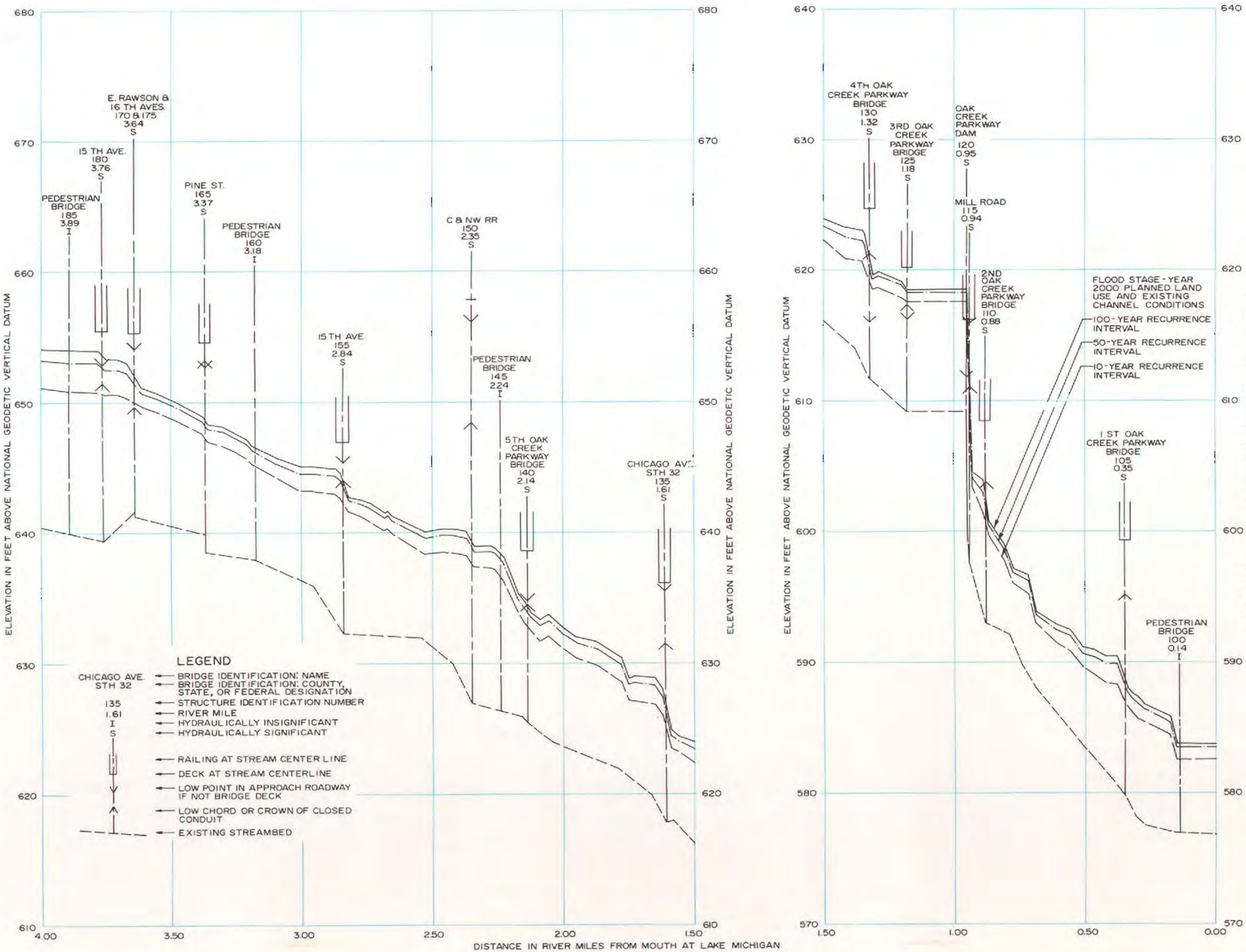
0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Figure 30

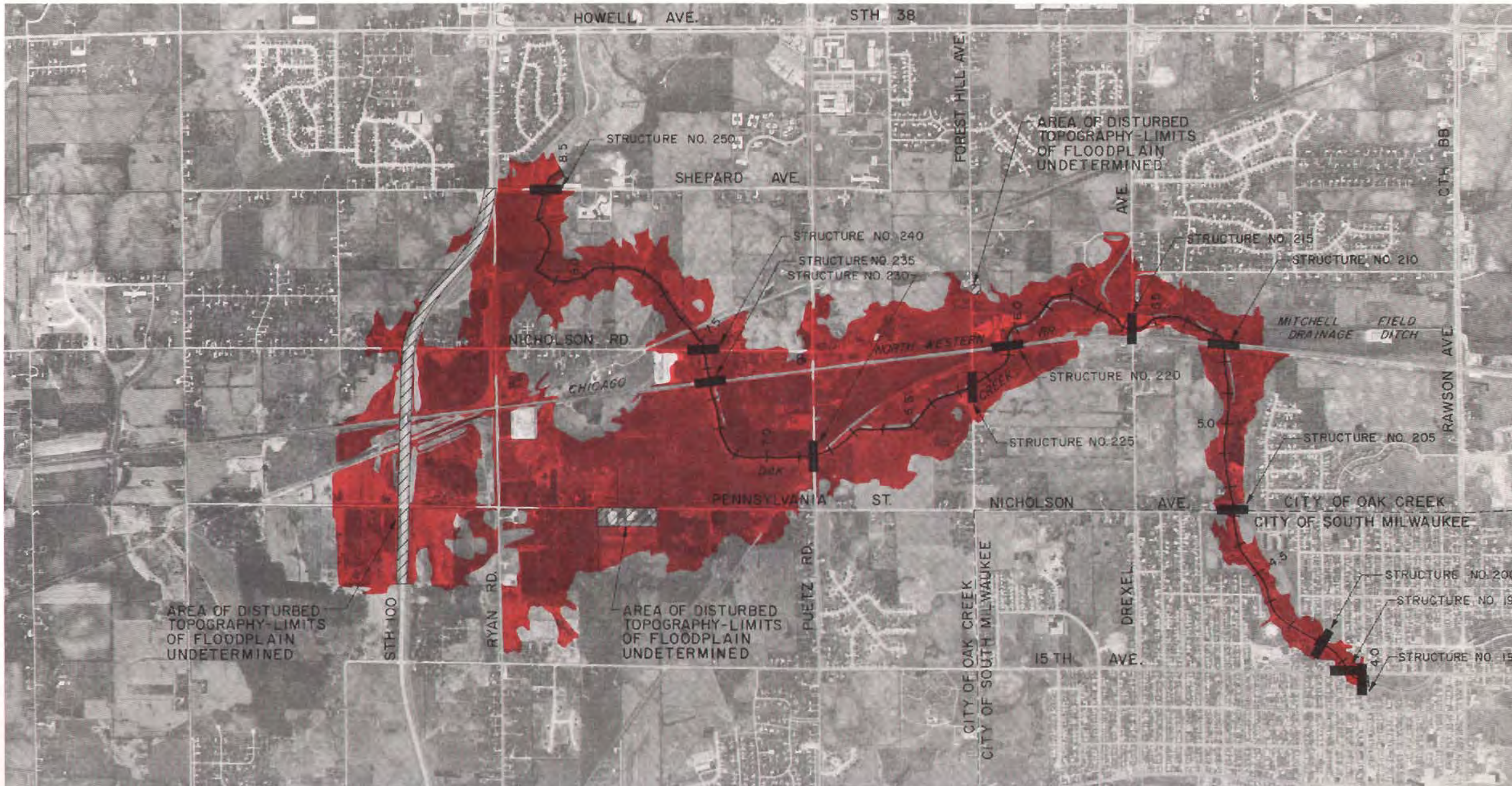
FLOOD STAGE AND STREAMBED PROFILE FOR THE OAK CREEK WATERSHED

OAK CREEK



Map 62 (continued)

OAK CREEK



LEGEND

APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

100-YEAR RECURRENCE INTERVAL FLOODLANDS-- PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR OAK CREEK IS SHOWN IN APPENDIX H

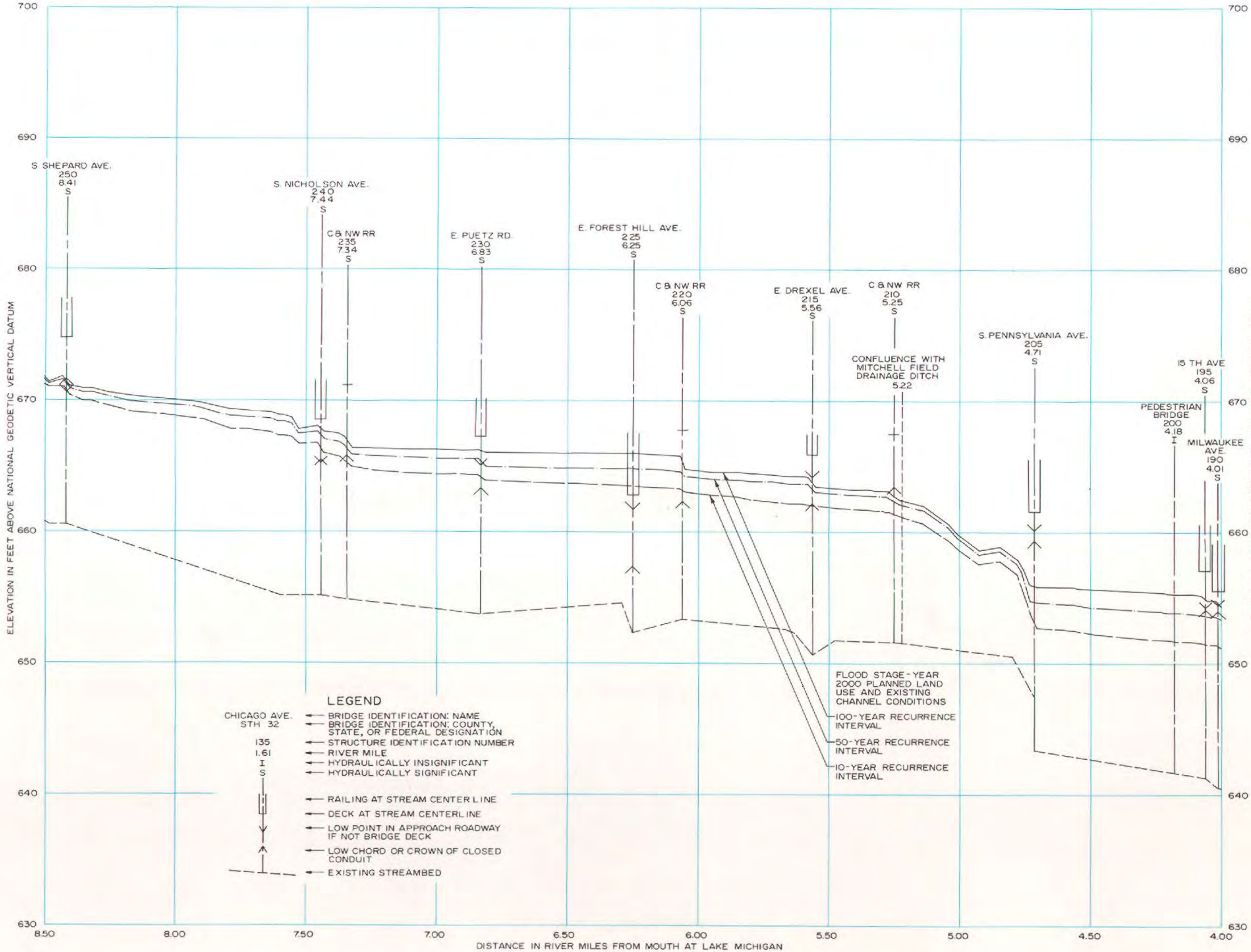
GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY APRIL 1980

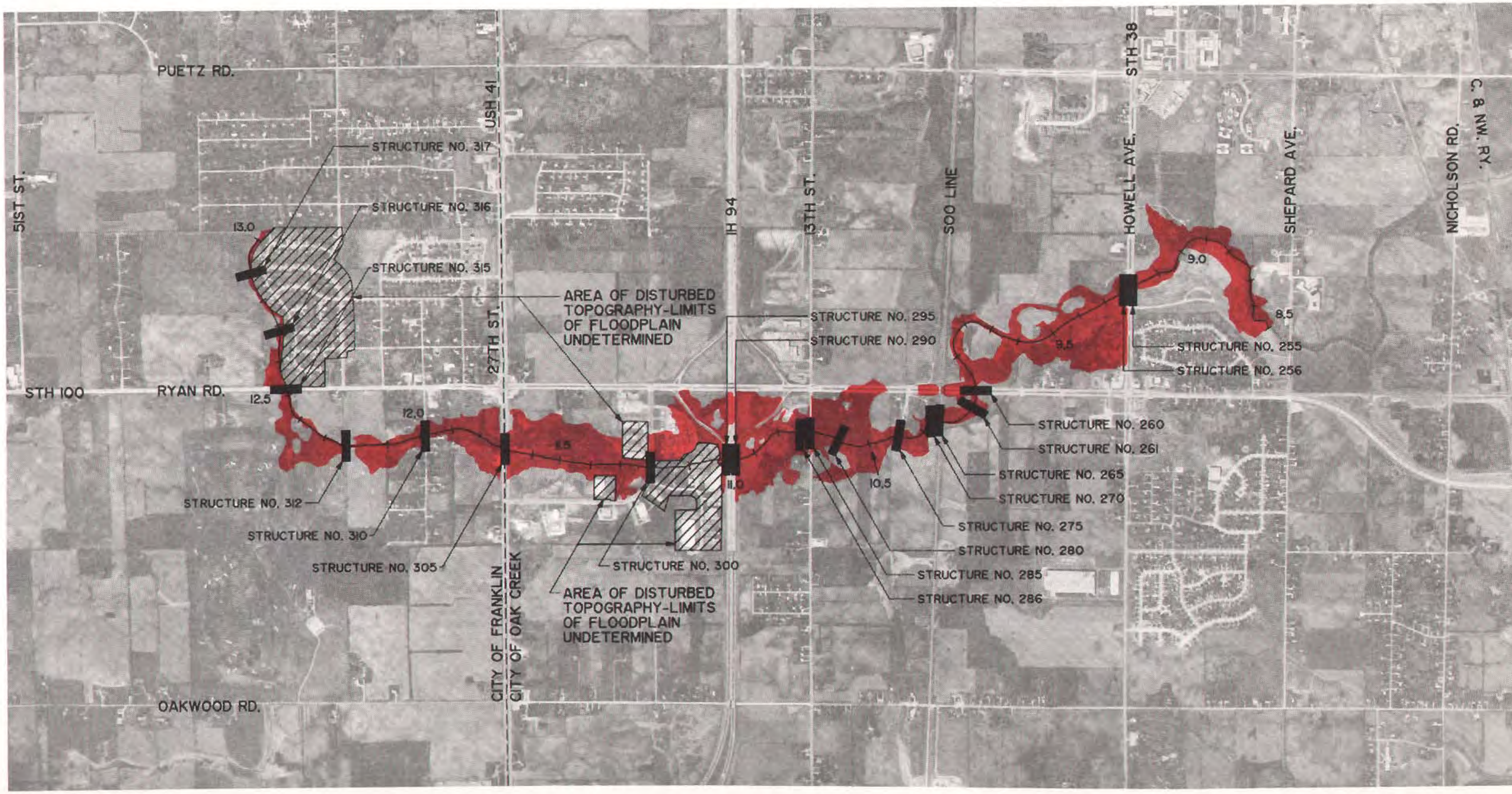
Figure 30 (continued)

OAK CREEK



Map 62 (continued)

OAK CREEK



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

1.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR OAK CREEK IS SHOWN IN APPENDIX H

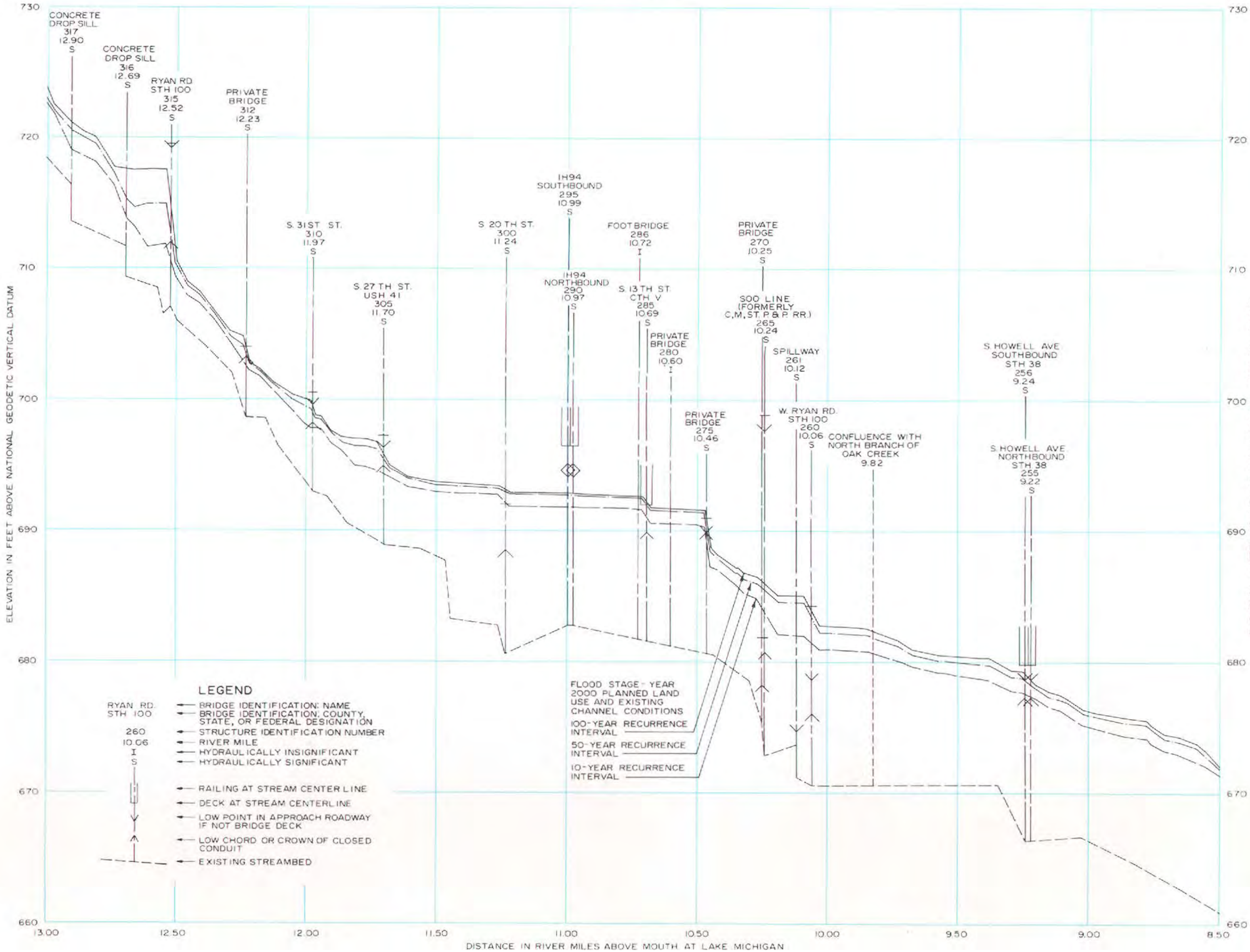
GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

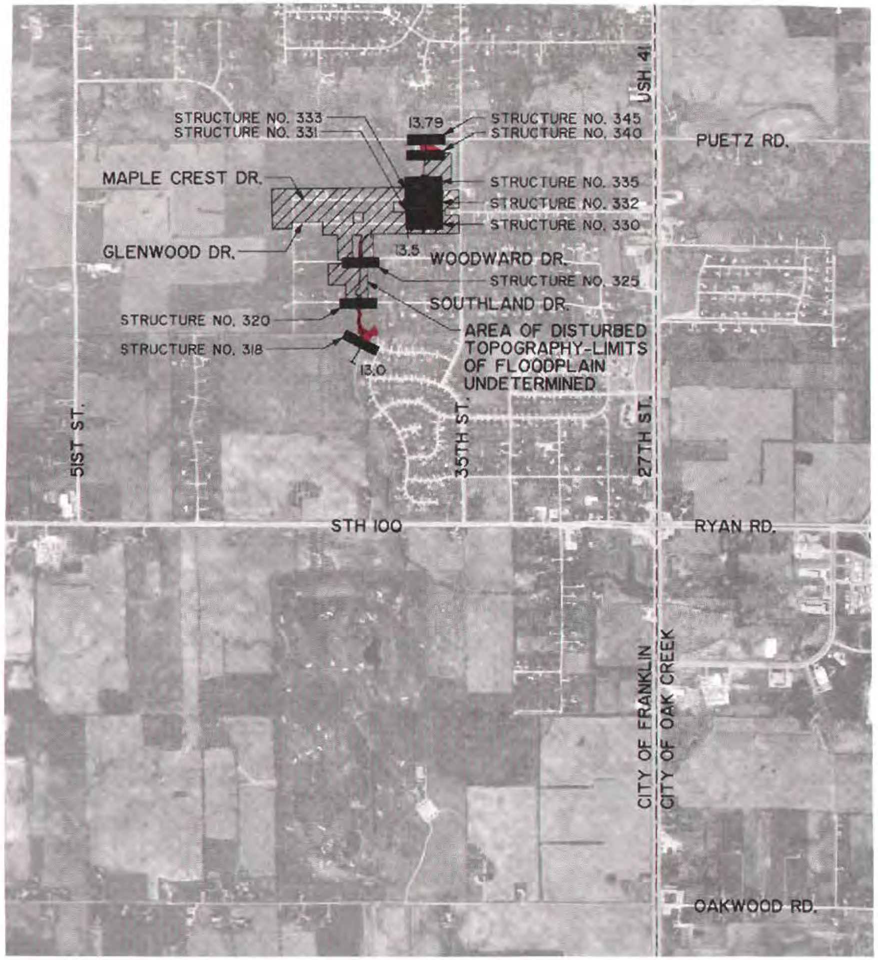
Figure 30 (continued)

OAK CREEK



Map 62 (continued)

OAK CREEK



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

1.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR OAK CREEK IS SHOWN IN APPENDIX H

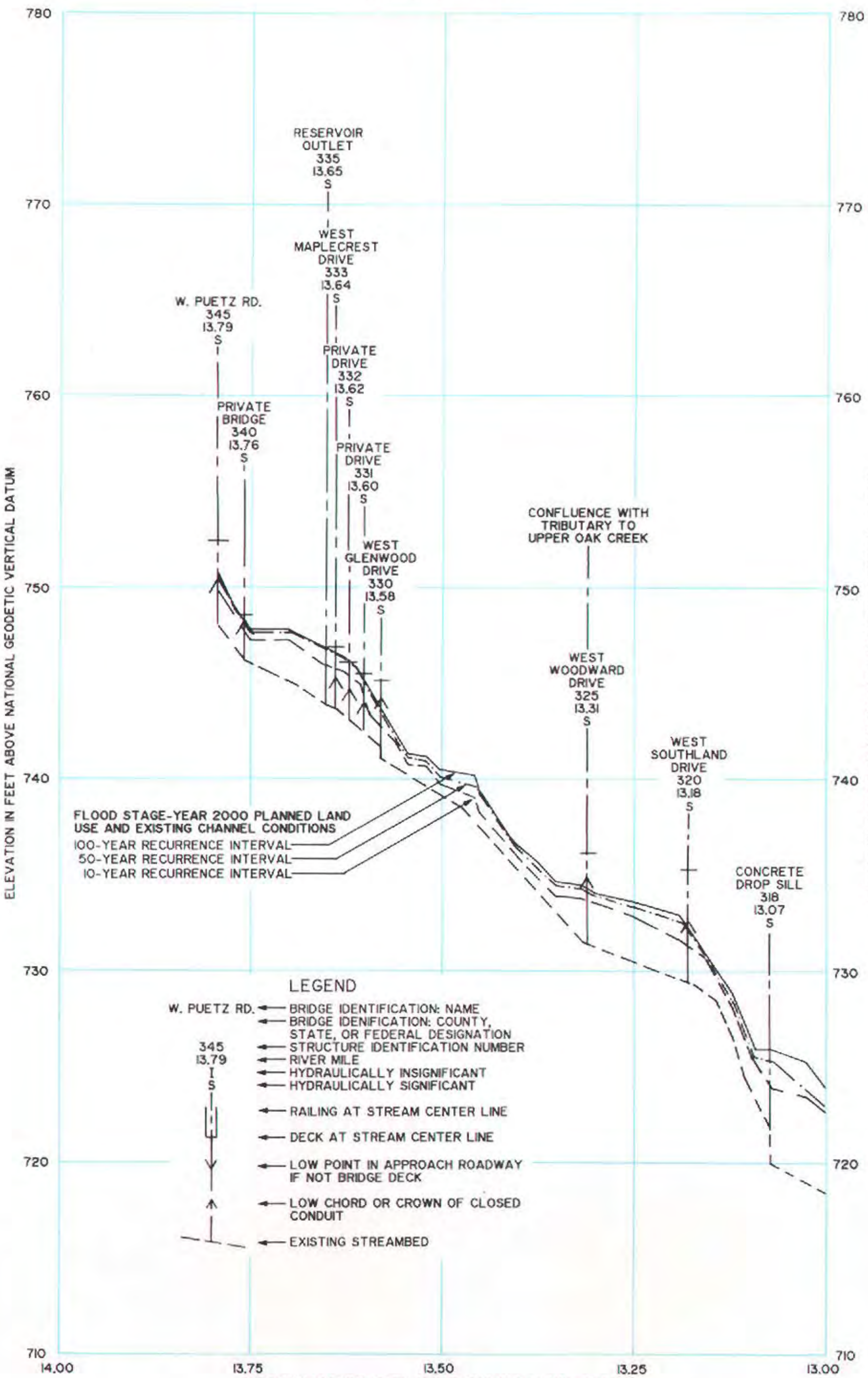
GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Figure 30 (continued)

OAK CREEK



and the average annual cost would be equivalent to the average annual flood damages.

Structure Floodproofing, Elevation, and Removal Alternative

A structure floodproofing, elevation, and removal alternative flood control plan was prepared and evaluated to determine if such a structure-by-structure approach would be a technically feasible and economically sound solution to the urban flood damage problems within the Oak Creek watershed. As shown on Map 63, of the 30 structures that are expected to incur flood damage, 22 would have to be floodproofed, 6 would have to be elevated, and 2 would have to be removed under this alternative. Future flood damage to the existing structures in the watershed would be virtually eliminated by these measures. The potential for damage to crops in the watershed would remain, however.

Assuming that these structure floodproofing measures would be fully implemented, and utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of this alternative is estimated at $50,000. This cost consists of the amortization of the $788,000 capital cost—$463,000 for floodproofing, $193,000 for structure elevation, and $132,000 for structure removal. The average annual flood damage abatement benefit was estimated at $78,000, yielding a benefit-cost ratio of 1.56.

Major Channelization Alternatives

Three alternatives utilizing major channel modifications were developed and analyzed for the Oak Creek watershed to determine if such measures would provide technically and economically sound solutions to existing and future flood problems, as well as to accommodate existing local development and stormwater drainage plans. The purpose of the first two alternatives was two-fold: 1) to help abate the existing and future flood damages; and 2) to provide an adequate outlet for existing urban stormwater drainage systems which, as designed and built, may experience a loss of required hydraulic capacity as a result of the inverts of the outlets being at an elevation below the existing streambed, or as a result of surcharging from the receiving stream to which the outlets discharge. Those outfalls with invert elevations below the channel bottom and within two feet above that bottom may be expected to experience loss of capacity, which may cause drainage problems due to storm sewer surcharging. The locations of outfalls that enter the stream with invert elevations less than two feet above the channel bottom are shown on Map 64.

Major Channelization Alternative 1: The first flood control alternative utilizing major channel improvements is shown on Map 65. Under this alternative, major channel modifications would be required along Oak Creek starting at the upstream end of the parkway impoundment in the City of South Milwaukee and continuing upstream to S. 27th Street—a distance of 9.8 miles. The proposed channel would have a bottom width ranging from 16 to 42 feet, with side slopes of one on two for the lower half of the channel cross-section, and of one on three for the upper half of the channel cross-section. The lower half of the channel would be concrete-lined and the top half turf-lined. Major channel modifications would also be required along the North Branch of Oak Creek from its confluence with Oak Creek upstream to W. Ramsey Avenue, a distance of 5.6 miles, and along the Mitchell Field Drainage Ditch from its confluence with Oak Creek upstream to W. Rawson Avenue, a distance of 0.8 mile. The proposed channel geometry would be the same as that for Oak Creek, with bottom widths of eight feet for the North Branch of Oak Creek and 24 feet for the Mitchell Field Drainage Ditch. In addition, this alternative includes widening and shaping of the channel of the Mitchell Field Drainage Ditch along the Wisconsin Air National Guard property. These improvements would begin at the upstream end of the south runway culvert and continue upstream for approximately 800 feet. The proposed channel would be turf-lined, with a bottom width of 10 feet and side slopes of one on three. It should be noted that the 100-year flood would not be contained within the channel along certain stream reaches under this alternative.

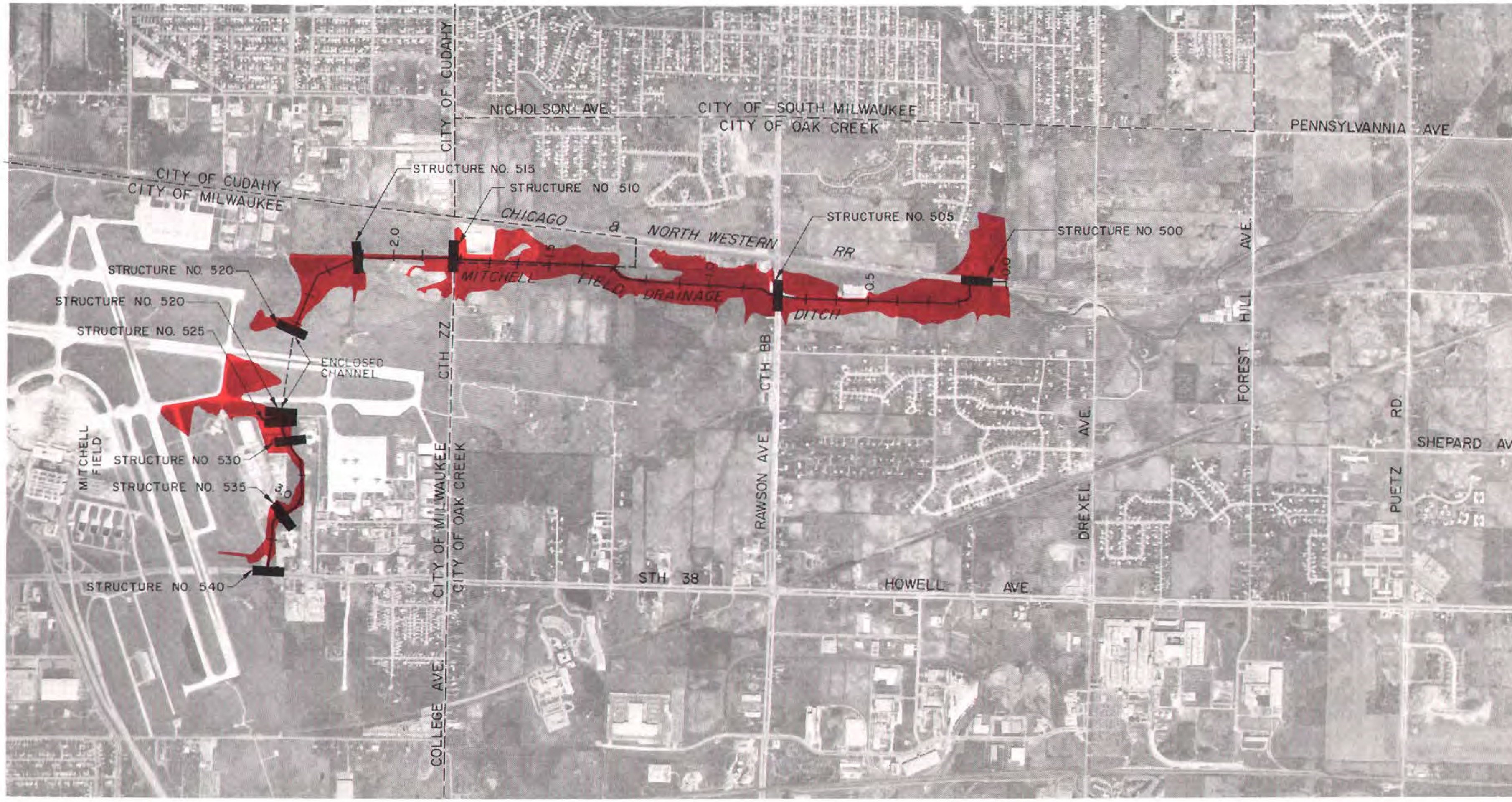
This alternative plan element also includes the replacement of 25 bridges on Oak Creek and six on the North Branch of Oak Creek. No bridge replacement would be required on the Mitchell Field Drainage Ditch.

Utilizing an annual interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this major channelization alternative is estimated at $1,409,000. This cost consists of the amortization of the $22,130,000 capital cost of the major

(Continued on Page 198)

Map 62 (continued)

MITCHELL FIELD DRAINAGE DITCH



LEGEND

APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

100-YEAR RECURRENCE INTERVAL FLOODLANDS--PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR MITCHELL FIELD DRAINAGE DITCH IS SHOWN IN APPENDIX H

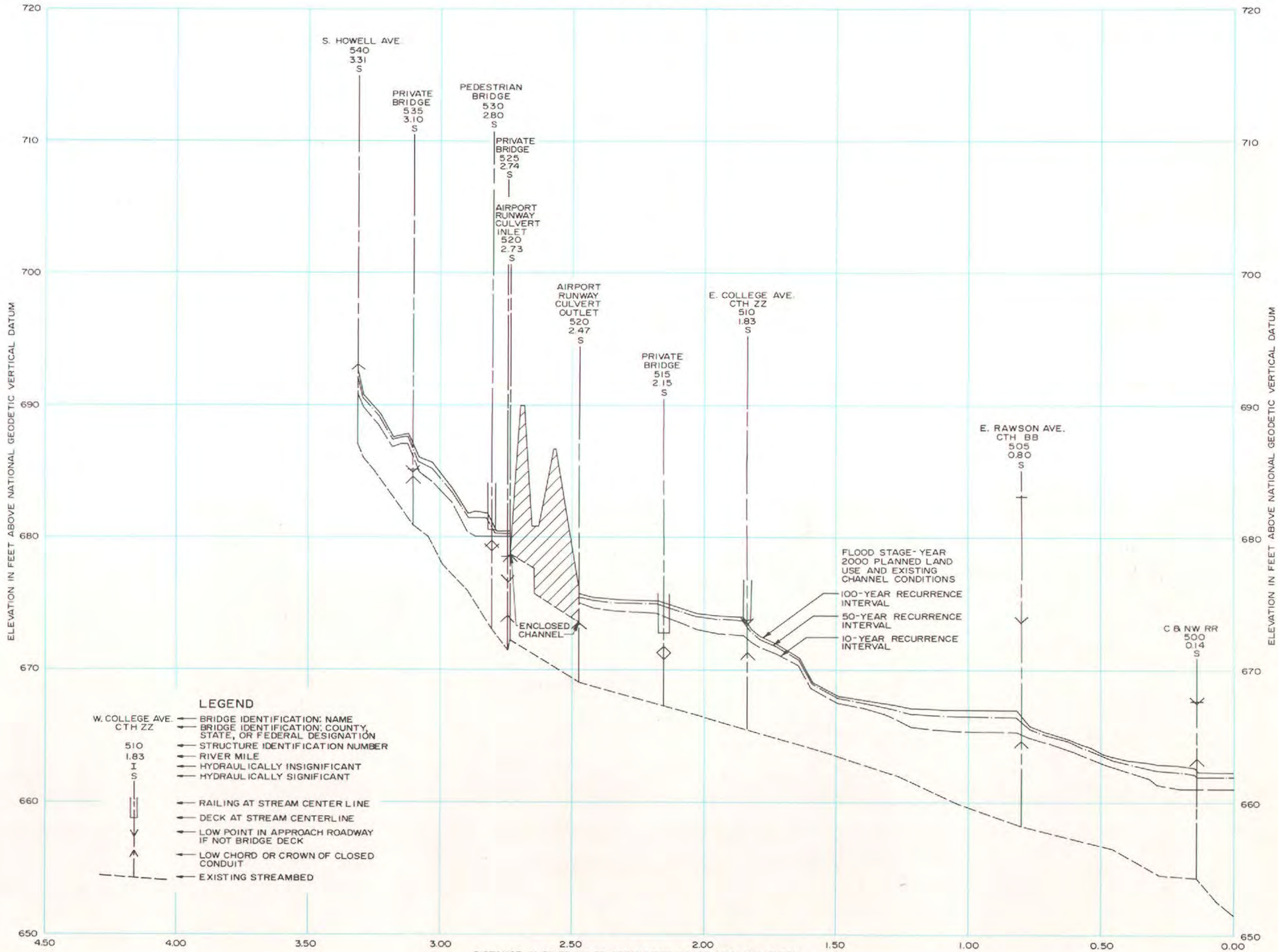
GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

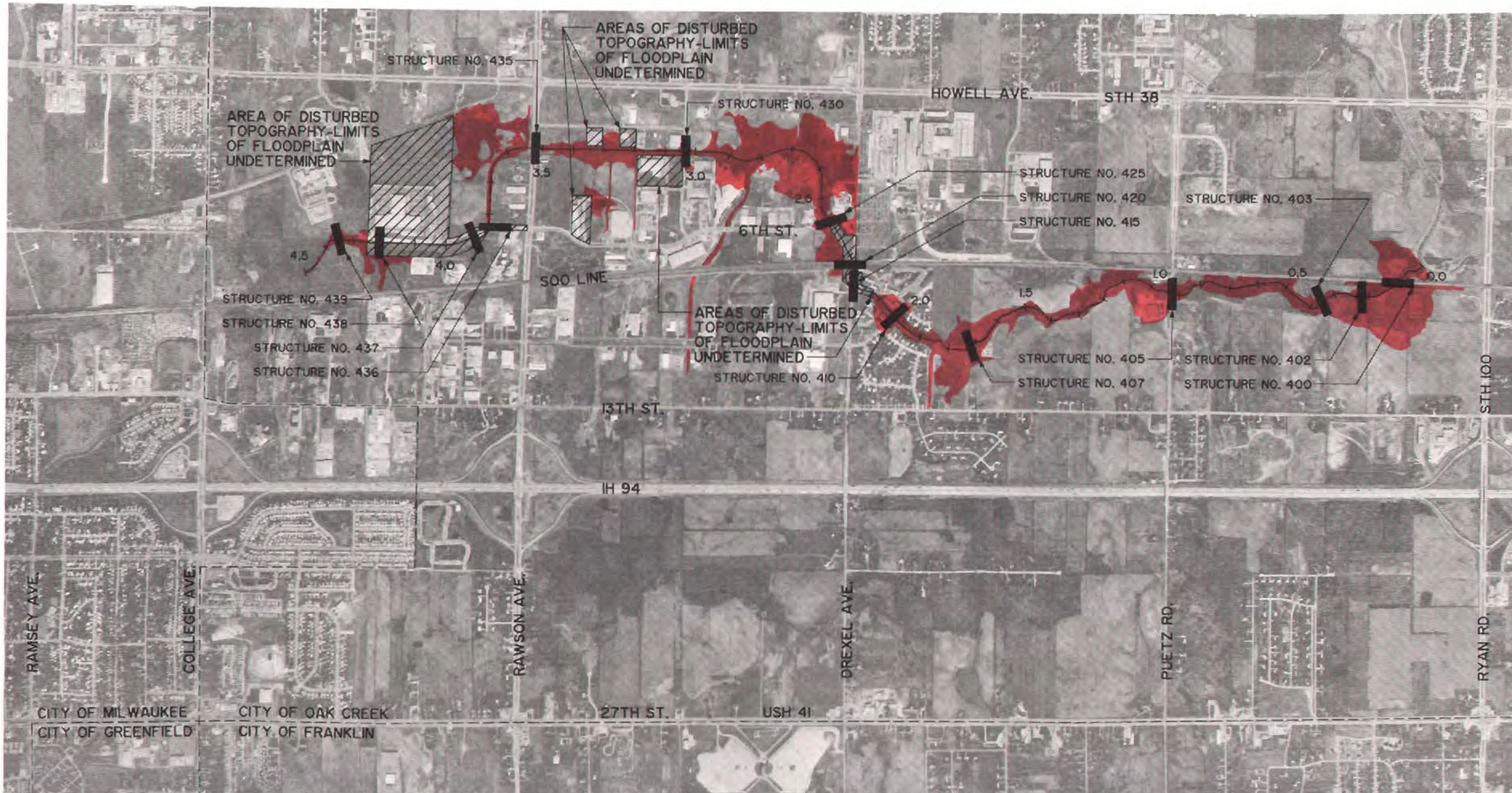
Figure 30 (continued)

MITCHELL FIELD DRAINAGE DITCH



Map 62 (continued)

NORTH BRANCH OF OAK CREEK



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

1.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR NORTH BRANCH OF OAK CREEK IS SHOWN IN APPENDIX H

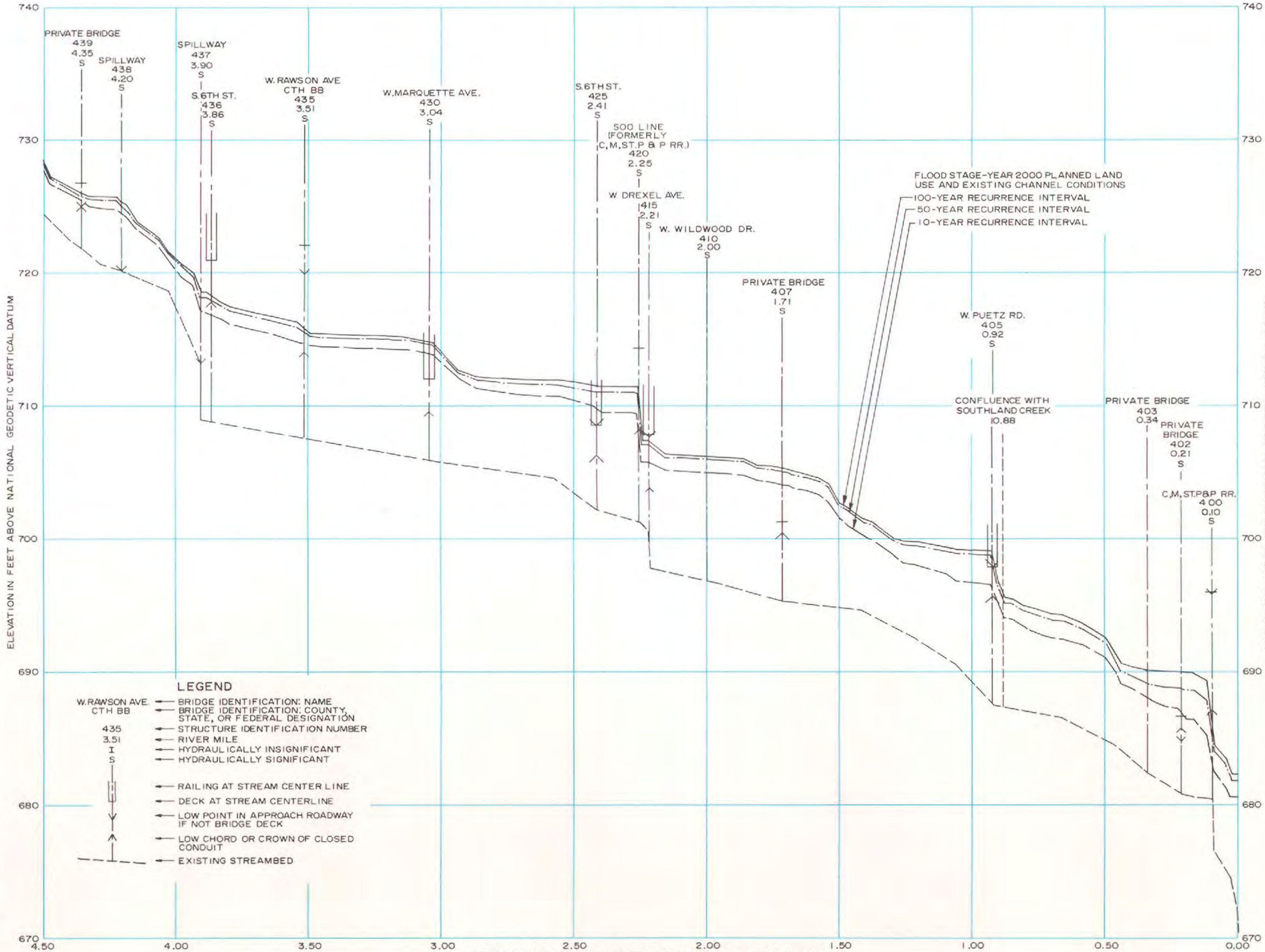
GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

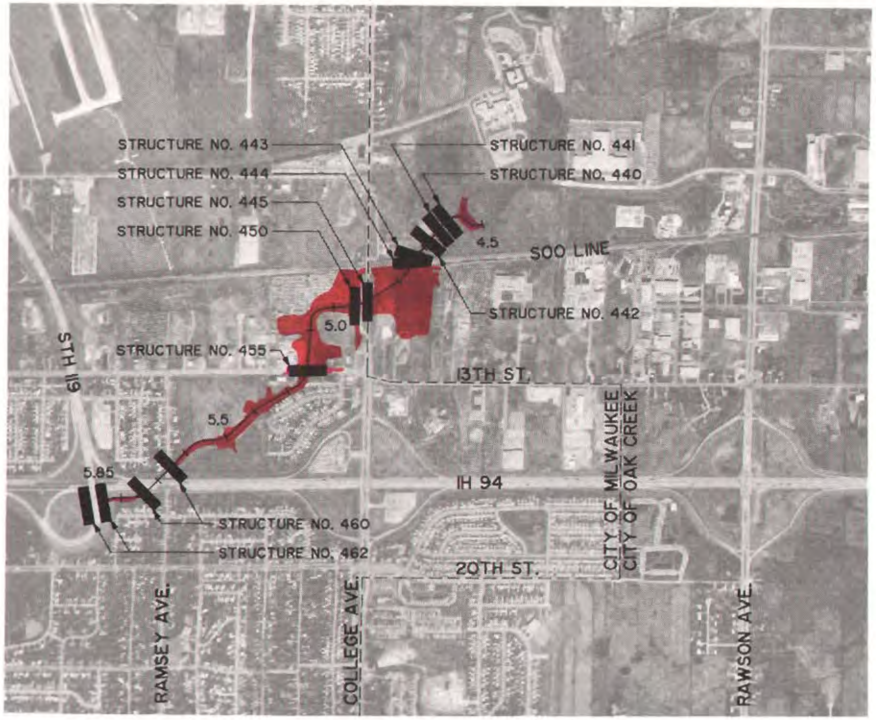
Figure 30 (continued)

NORTH BRANCH OF OAK CREEK



Map 62 (continued)

NORTH BRANCH OF OAK CREEK

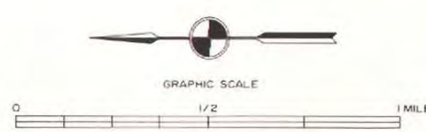


LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

1.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR NORTH BRANCH OF OAK CREEK IS SHOWN IN APPENDIX H

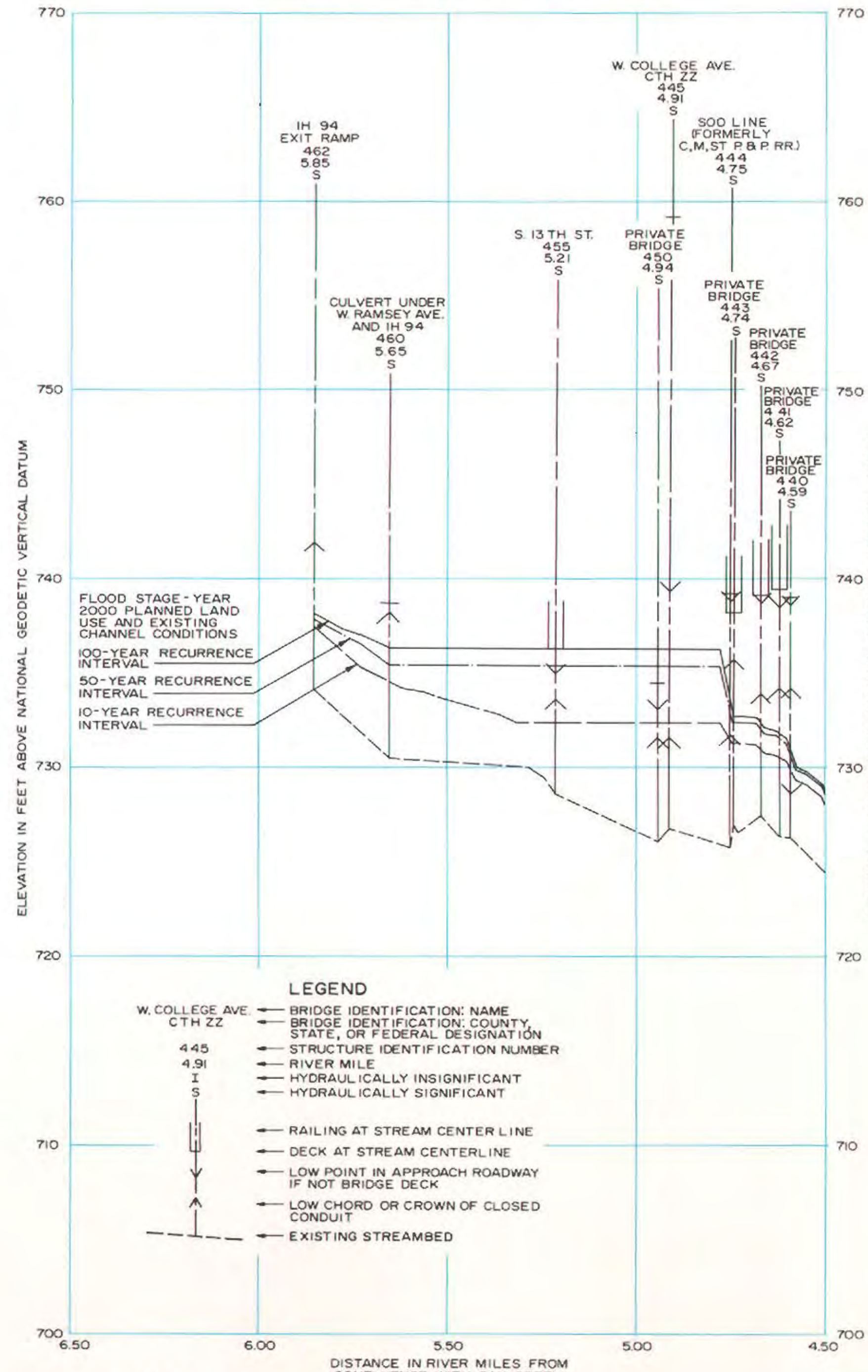


DATE OF PHOTOGRAPHY: APRIL 1986

Source: SEWRPC.

Figure 30 (continued)

NORTH BRANCH OF OAK CREEK



Source: SEWRPC.

Table 37

PRINCIPAL FEATURES, COSTS, AND BENEFITS OF THE ALTERNATIVE FLOOD CONTROL PLANS FOR THE OAK CREEK WATERSHED

Alternative			Technically Feasible	Capital Cost		Economic Analysis[a]								
Number	Name	Description		Item	(thousands)	Annual Amortized Capital Cost (thousands)	Annual Operation and Maintenance Cost (thousands)	Total[b] Annual Cost (thousands)	Annual[b] Benefits (thousands)	Excess of Annual Benefits Over Costs (thousands)	Benefit-Cost Ratio	Benefit-Cost Ratio Greater than 1.0	Nontechnical and Noneconomic Considerations: Positive	Nontechnical and Noneconomic Considerations: Negative
1	No Action-No development in floodplain area	- -	Yes	- -	$ - -	$ - -	$ - -	$ 98[c]	$ - -	$ - -	- -	No	- -	Continue to incur average annual flood damages of $98,000
2	No Action-Development in floodplain fringe area only	- -	Yes	- -	- -	- -	- -	103[c]	- -	- -	- -	No	- -	Continue to incur average annual flood damages of $103,000
3	Structure floodproofing, elevation, and removal	a. Floodproof up to 22 residential and commercial structures b. Elevate up to six residential structures c. Remove up to two residential structures	Yes	Floodproofing Elevating Removal Subtotal	463 193 132 788	50	- -	50	78	28	1.56	Yes	Immediate partial flood relief at discretion of property owners Most of the costs would be borne by beneficiaries	Complete, voluntary implementation unlikely and therefore left with a significant residual flood problem. Overland flooding and some attendant problems remain. Some floodproofing is likely to be applied without adequate professional advice and, as a result, structure damage may occur. Partial resolution of flood problem
4	Major Channelization 1	a. 16.2 miles of major channelization b. Modification or replacement of 26 bridges	Yes	Major channelization Bridge modification and replacement Subtotal	16,047 6,083 22,130	1,394	15	1,409	93	-1,316	0.07	No	Consistent with commitment of communities within the watershed as reflected by the location, size, and grade of existing storm sewers and bridges	Aesthetic impact of concrete lining, partial resolution of flood problem
5	Major Channelization 2	a. 16.2 miles of major channelization b. Replacement of 41 bridges	Yes	Major channelization Bridge replacement Subtotal	18,084 10,822 28,906	1,821	15	1,836	103	-1,733	0.06	No	Consistent with commitment of communities within the watershed as reflected by the location, size, and grade of existing storm sewers and bridges	Aesthetic impact of concrete lining
6	Major Channelization 3	a. 4.3 miles of major channelization b. Replacement of 12 bridges	Yes	Major channelization Bridge replacement Subtotal	1,142 4,799 5,941	375	4	379	34	-345	0.09	No	Consistent with locally committed plans for development of industrial parks and residential neighborhood in City of Oak Creek	Partial resolution of flood problem
7	Decentralized Storage	Provide onsite detention storage facilities	Yes	Onsite detention facilities Land cost Subtotal	4,580 450 5,030	317	234	551	73	-478	0.13	No	Potential to retain public open space. Impact on instream water quality	No assurance of long-term commitment by local units of government to require onsite detention facilities. Difficult to apply to small-scale development proposals. Partial resolution of flood problem.
8	Centralized Storage	Construction of five detention storage reservoirs	Yes	Reservoirs and outlet culverts Earthen embankment Land acquisition Subtotal	612 12 99 723	46	18	64	58	-6	0.91	No	Potential to retain public open space	Partial resolution of flood problem

Table 37 (continued)

Alternative			Technically Feasible	Economic Analysis[a]										
				Capital Cost		Annual Amortized Capital Cost (thousands)	Annual Operation and Maintenance Cost (thousands)	Total[b] Annual Cost (thousands)	Annual[b] Benefits (thousands)	Excess of Annual Benefits Over Costs (thousands)	Benefit-Cost Ratio	Benefit-Cost Ratio Greater than 1.0	Nontechnical and Noneconomic Considerations	
Number	Name	Description		Item	(thousands)								Positive	Negative
9	Combination of Major Channelization and Floodproofing	a. 6.4 miles of major channelization b. 3.7 miles of channel deepening and shaping c. Replacement of 19 bridges d. Floodproof, elevate, or remove up to 24 structures	Yes	Major channelization Channel deepening and shaping Bridge replacement Structure floodproofing, elevation, and removal Subtotal	$ 1,676 401 6,541 347 8,965	$ 583	$ 10	$ 593	$ 92	$ -501	0.16	No	Consistent with locally committed plans for development of industrial parks and residential neighborhood in City of Oak Creek. Provides sufficient outlet for existing storm sewers	Partial resolution of flood problem
10	Combination of Major Channelization, Channel Deepening and Shaping, Centralized Storage, Structure Floodproofing and Elevation	a. 6.4 miles of major channelization b. 3.7 miles of channel deepening and shaping c. Replacement of eight bridges d. Construction of five detention basins e. Floodproof or elevate up to 18 structures	Yes	Major channelization Channel deepening and shaping Bridge replacement Detention basins Structure floodproofing and elevation Subtotal	1,574 401 3,317 723 107 6,122	386	28	414	98	-316	0.24	No	Consistent with locally committed plans for development of industrial parks and residential neighborhood in City of Oak Creek. Provides sufficient outlet for existing storm sewers. Potential to retain public open space	Partial resolution of flood problem
11	Combination of Minimum Channelization and Structure Floodproofing, Elevation, and Removal	a. 5.7 miles of major channelization b. 3.7 miles of channel deepening and shaping c. Replacement of 11 bridges d. Floodproof, elevate, or remove up to 26 structures	Yes	Major channelization Channel deepening and shaping Bridge replacement Structure floodproofing, elevation, and removal Subtotal	1,076 401 3,049 521 5,047	318	9	327	93	-234	0.28	No	Consistent with locally committed plans for development of industrial parks and residential neighborhood in City of Oak Creek. Provides sufficient outlet for existing storm sewers	Partial resolution of flood problem
12	Combination of Channel Deepening and Shaping, and Structure Floodproofing, Elevation, and Removal	a. 2.4 miles of channel deepening and shaping b. Replacement of two bridges c. Floodproof, elevate, or remove up to 29 structures	Yes	Channel deepening and shaping Bridge replacement Structure floodproofing, elevation, and removal Subtotal	207 110 692 1,009	64	1	65	78	13	1.20	Yes	Immediate partial flood relief at discretion of property owners Provides sufficient outlet for a storm sewer outfall which is currently below channel grade and eliminates a negative channel slope between IH 94 and S. 27th Street	Complete, voluntary implementation unlikely and therefore left with a significant residual flood problem. Overland flooding and some attendant problems remain. Some floodproofing is likely to be applied without adequate professional advice and, as a result, structure damage may occur. Partial resolution of flood problem

[a] *Economic analyses are based on an annual interest rate of 6 percent and a 50-year amortization period and project life.*

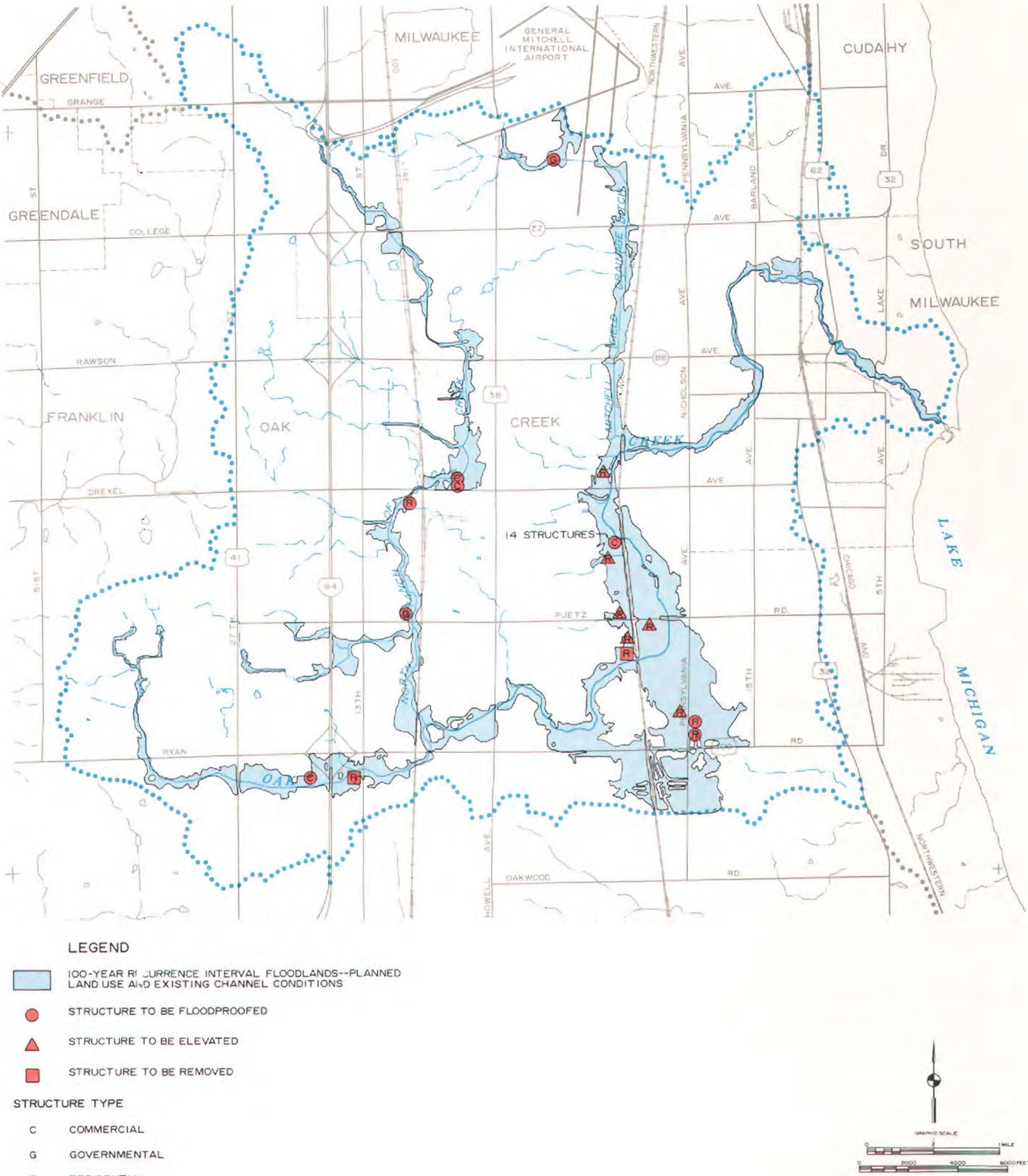
[b] *Annual benefits and costs used in the benefit-cost analysis include only the direct benefits derived from the abatement of monetary flood damages, and the direct costs attendant to implementation of the flood control measures, including capital and operation and maintenance costs. Environmental and recreational benefits and costs were not addressed in the benefit-cost analysis since these represent intangible benefits and costs and, therefore, cannot be readily quantified.*

[c] *The total cost of this alternative consists of the average annual monetary flood damages.*

Source: SEWRPC.

Map 63

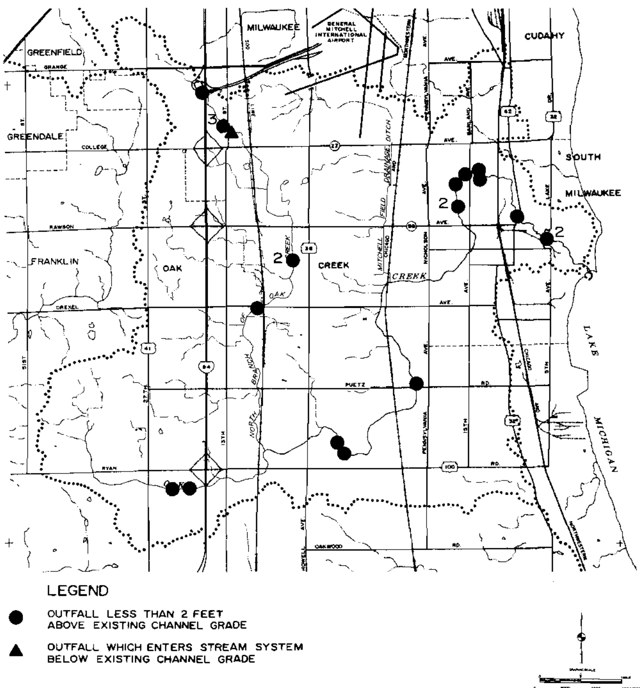
STRUCTURE FLOODPROOFING, ELEVATION, AND REMOVAL ALTERNATIVE FOR THE OAK CREEK WATERSHED



Source: SEWRPC.

Map 64

EXISTING STORM SEWER OUTFALLS IN THE OAK CREEK WATERSHED WHICH ENTER THE STREAM SYSTEM WITHIN TWO FEET OF THE EXISTING CHANNEL GRADE



Source: SEWRPC.

channelization and bridge replacement entailed, and $15,000 in annual operation and maintenance costs. The average annual flood abatement benefit is estimated at $93,000, resulting in a benefit-cost ratio of 0.07.

Major Channelization Alternative 2: The second major channelization flood control alternative, shown on Map 66, is essentially the same as the first alternative, except that the channel is designed to contain the 100-year recurrence interval flood discharge while providing a minimum of two feet of freeboard. Major channel modifications would be required along Oak Creek between the parkway impoundment in the City of South Milwaukee and S. 27th Street, a distance of 9.8 miles; along the North Branch of Oak Creek from its confluence with Oak Creek to W. Ramsey Avenue, a distance of 5.6 miles; and along the Mitchell Field Drainage Ditch from its confluence with Oak Creek to E. Rawson Avenue, a distance of 0.8 mile. Under this alternative, the proposed channel would need to be widened and/or deepened beyond that proposed in a 1967 report prepared by Klug & Smith Company for the Milwaukee Metropolitan Sewerage District, entitled Report on Oak Creek Flood Survey on Entire Basin for the Metropolitan Sewerage Commission of the County of Milwaukee.[2] For some reaches the proposed channel would need to be widened an additional 10 to 36 feet, while the channel bottom would need to be lowered an additional two feet. For the reach of the Mitchell Field Drainage Ditch along the Wisconsin Air National Guard property, no further channel widening was considered under this alternative. In addition to the required widening and lowering of the channel bottom, 16 additional bridges would need to be replaced under this alternative.

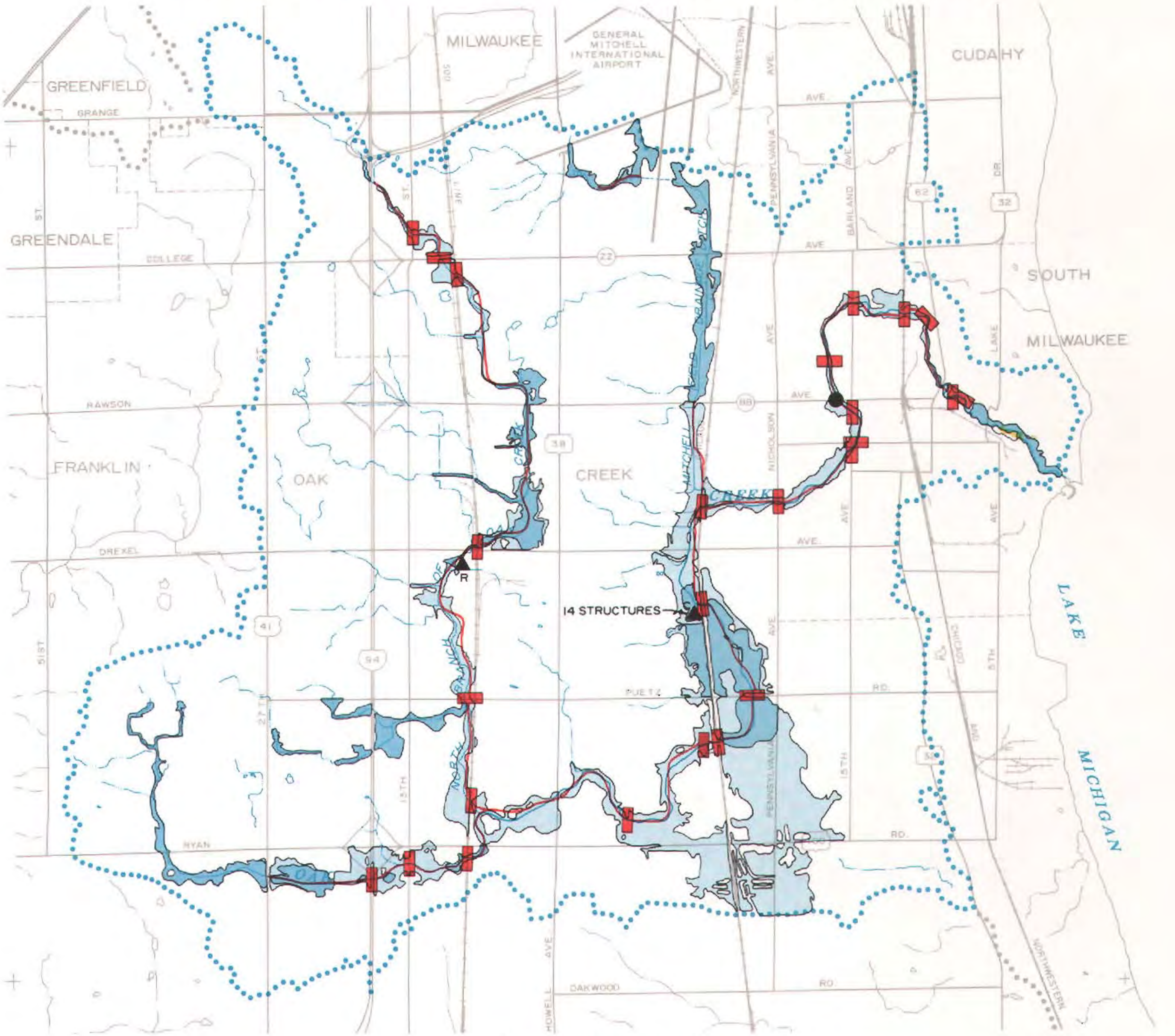
Utilizing an annual interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this channel modification alternative is estimated at $1,836,000, consisting of the amortization of the $28,906,000 capital cost of major channelization and bridge replacement, and $15,000 in annual operation and maintenance costs. The average flood abatement benefit is estimated at $103,000, resulting in a benefit-cost ratio of 0.06.

Major Channelization Alternative 3: Under the third flood control alternative utilizing major channelization, shown on Map 67, major channel modifications would be carried out only along that portion of Oak Creek beginning at the Soo Line Railroad and extending upstream to S. 27th Street—a distance of about 1.5 miles. Peak flood discharges and channel slopes would be less under this alternative than under the first two channelization alternatives, thereby resulting in lower velocities along the modified channel reaches and allowing the use of turf lining in place of concrete lining. The proposed channel would have a bottom width of 20 feet with side slopes of one on three. Major channel

[2] *The Klug & Smith report recommended major channel modifications for much of the Oak Creek watershed stream system, and its recommendations have been incorporated by the District and other state and local units and agencies of government into the design of bridges, channel improvements, and urban stormwater drainage systems.*

Map 65

MAJOR CHANNELIZATION ALTERNATIVE 1 FOR THE OAK CREEK WATERSHED



LEGEND

- 100-YEAR RECURRENCE INTERVAL FLOODLANDS--PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS
- 100-YEAR RECURRENCE INTERVAL FLOODLANDS--PLANNED LAND USE WITH MAJOR CHANNELIZATION
- PROPOSED MAJOR CHANNELIZATION
- PROPOSED BRIDGE REPLACEMENT
- ● PROPOSED BRIDGE MODIFICATION
- ▲ STRUCTURE EXPERIENCING DAMAGES UNDER FLOOD CONTROL ALTERNATIVE

STRUCTURE TYPE

C COMMERCIAL

R RESIDENTIAL

GRAPHIC SCALE

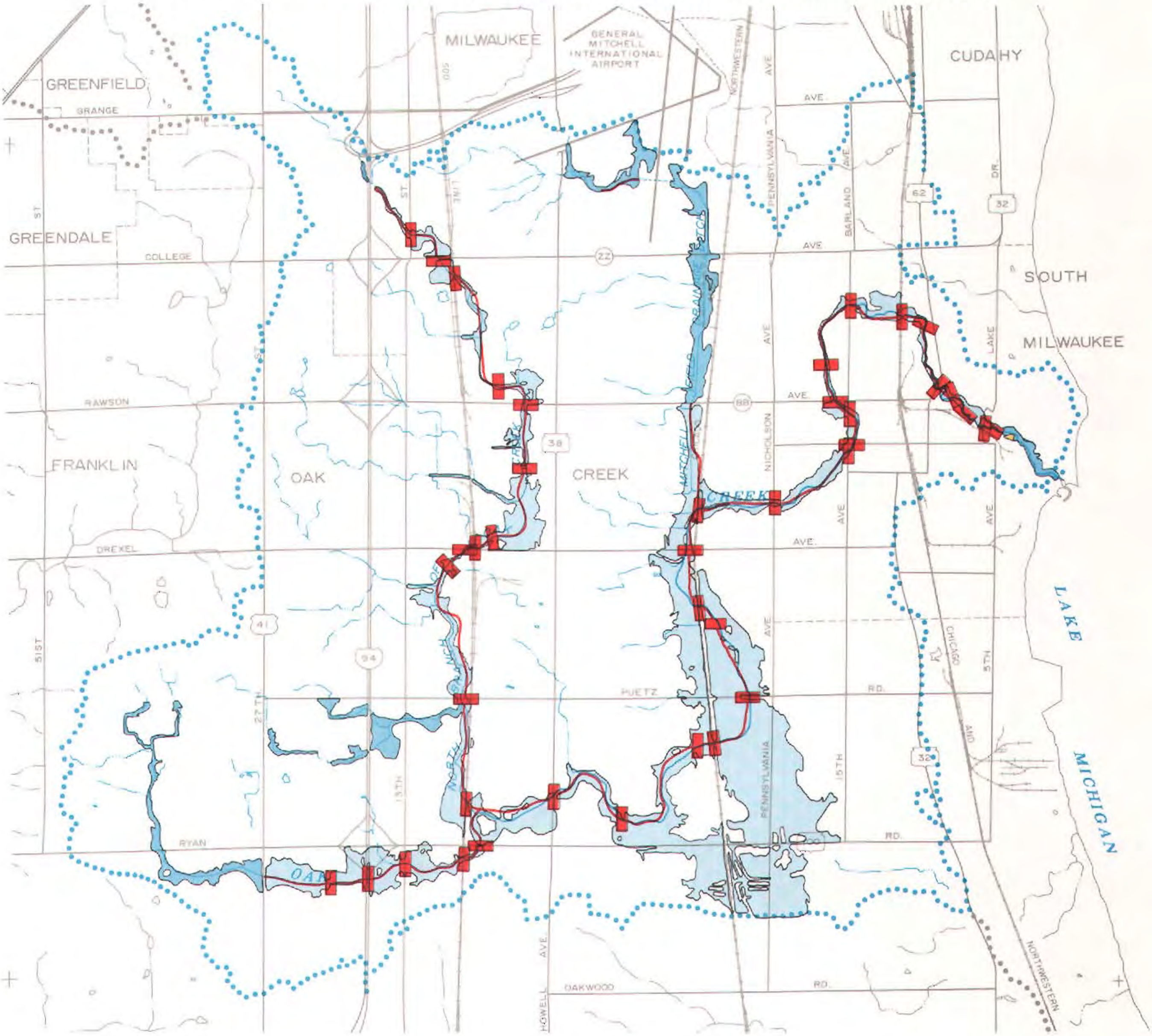
0 1 MILE

0 2000 4000 6000 FEET

Source: SEWRPC.

Map 66

MAJOR CHANNELIZATION ALTERNATIVE 2 FOR THE OAK CREEK WATERSHED



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODLANDS--PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

100-YEAR RECURRENCE INTERVAL FLOODLANDS--PLANNED LAND USE WITH MAJOR CHANNELIZATION

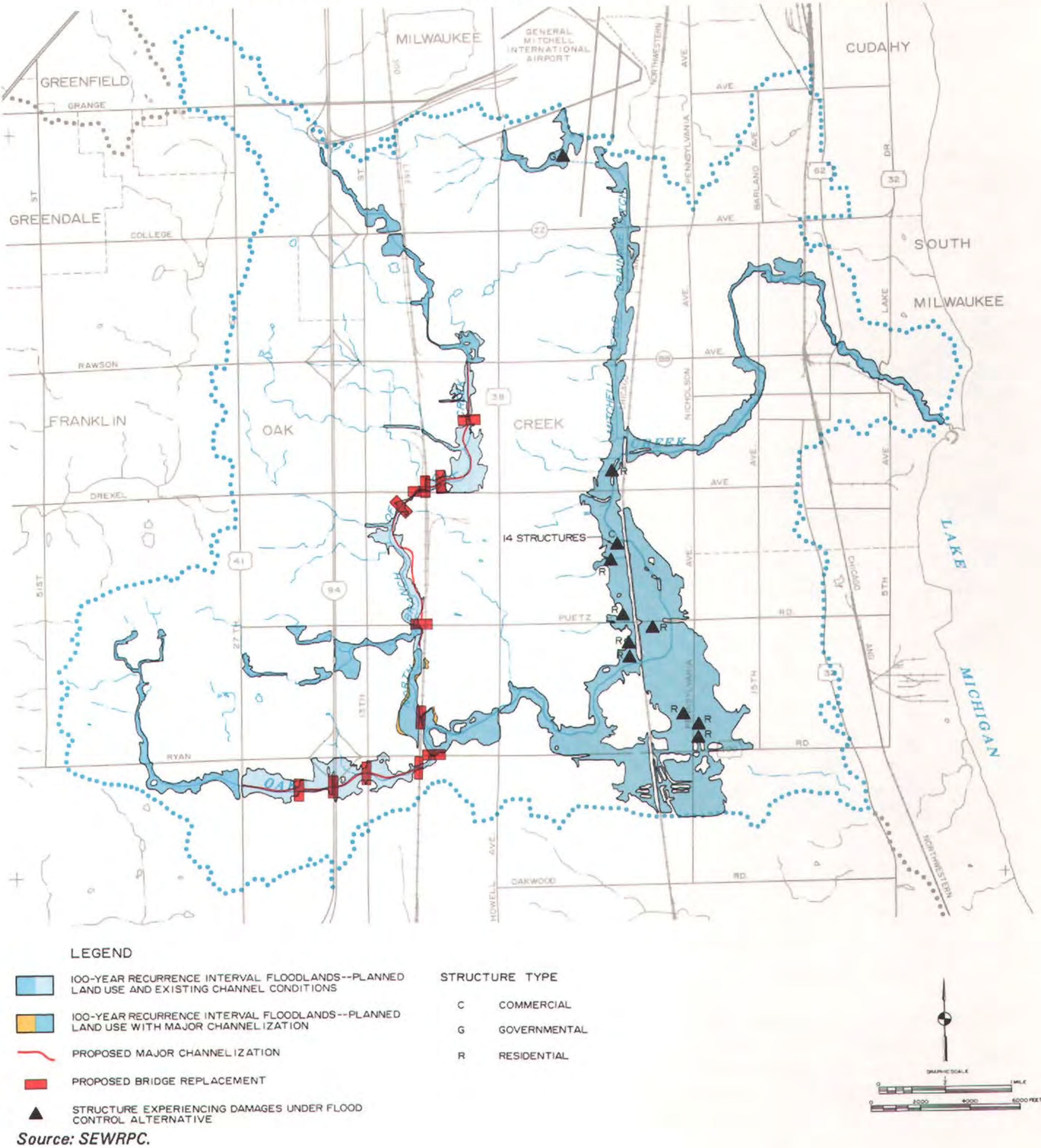
PROPOSED MAJOR CHANNELIZATION

PROPOSED BRIDGE REPLACEMENT

Source: SEWRPC.

Map 67

MAJOR CHANNELIZATION ALTERNATIVE 3 FOR THE OAK CREEK WATERSHED



Source: SEWRPC.

modifications would also be required along the North Branch of Oak Creek beginning about 960 feet downstream of the confluence with Southland Creek and extending upstream to W. Rawson Avenue—a distance of about 2.8 miles. This proposed channel would also be turf-lined and have a bottom width of 20 feet with side slopes of one on three. This alternative plan would also require the replacement of five bridges on Oak Creek and seven bridges on the North Branch of Oak Creek.

The hydrologic-hydraulic analyses conducted under this alternative indicates that the reduction in floodwater storage created by the channel modifications would serve to increase stages downstream of the Oak Creek modification a distance of about 10.2 miles, with the increases over this length varying from 0.1 to 0.5 foot. Stages would also be increased downstream of the North Branch modification for a distance of about 0.7 mile, with the increases over this length varying from 0.1 foot to 1.1 feet. Chapter NR 116 of the Wisconsin Administrative Code requires that flooding easements be obtained from all property owners affected by any increase of more than 0.1 foot in the 100-year recurrence interval flood profile.[3] Accordingly, such flooding easements would have to be obtained under this alternative for that reach from the mouth of Oak Creek to the beginning of the major channel improvements.

Utilizing an annual interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this alternative is estimated at $379,000, consisting of the amortization of the $5,941,000 capital cost of the channel improvements and bridge replacements, and $4,000 in annual operation and maintenance costs. The average annual flood abatement benefit is estimated at $34,000, resulting in a benefit-cost ratio of 0.09.

Decentralized Storage Alternative: An alternative flood control plan consisting of decentralized—or off-stream, onsite—storage was considered. This alternative, shown on Map 68, assumes that all of the communities in the watershed will adopt policies requiring that onsite stormwater detention facilities be provided as land is converted from rural to urban use in order to ensure that stormwater runoff from developing areas will not exceed such runoff under pre-development conditions. Such a policy would serve to limit peak flood discharges to those under the existing land use conditions in the watershed. In addition, the construction of onsite storage facilities could reduce the costs of local urban stormwater facilities, as well as provide some water quality benefits by limiting the amount of urban nonpoint source pollution entering the stream system.

Under this alternative plan, 90 detention basins were assumed to be installed throughout the watershed to serve new development, each one to two acres in size and serving about 80 acres of tributary drainage area. It was recognized that other types of facilities such as infiltration trenches could also be used to minimize stormwater flows, and that the best type of facility would have to be determined on the basis of site-specific analyses.

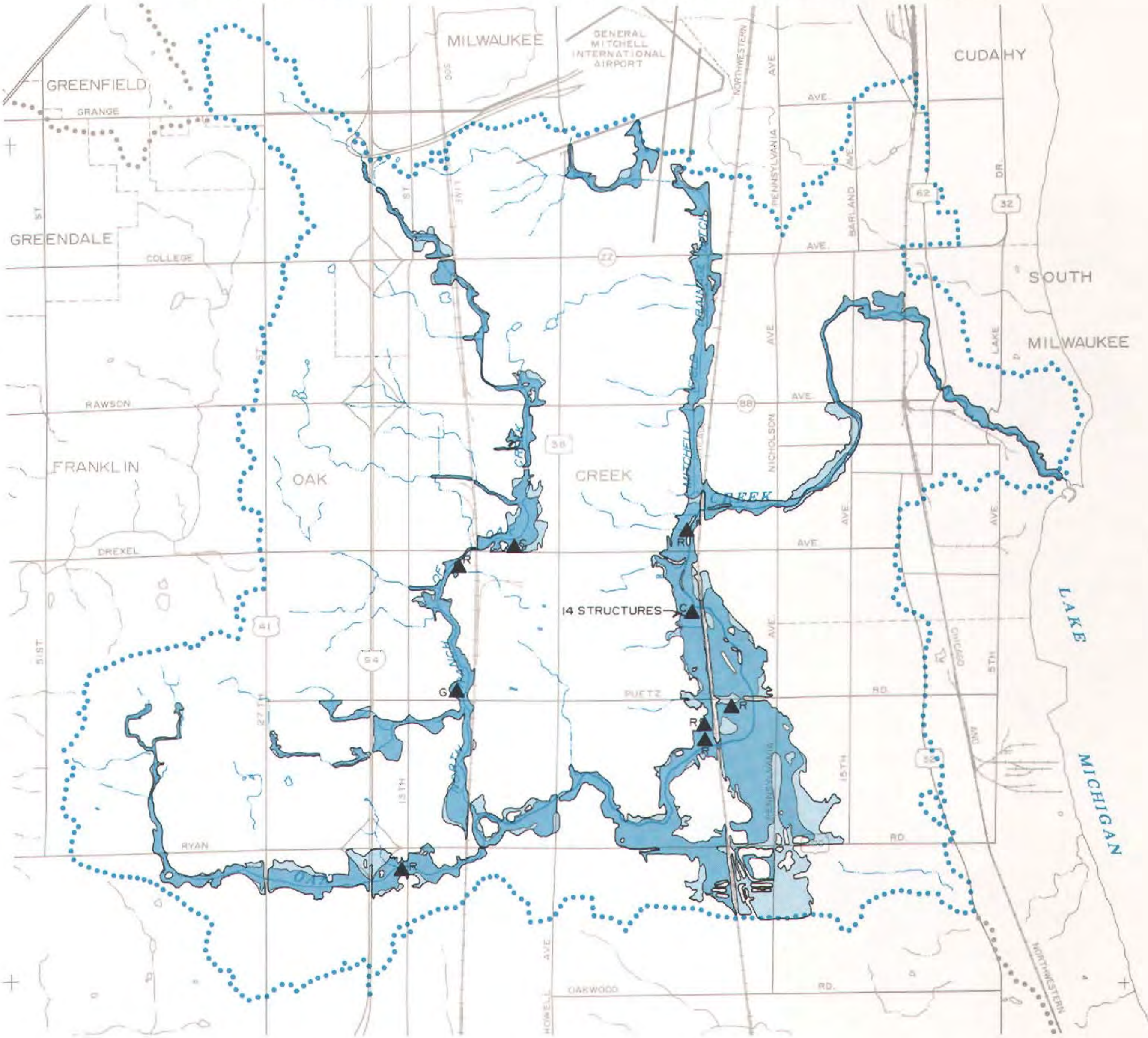
Utilizing an interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this alternative was estimated at $551,000, consisting of the amortization of the $5,030,000 land cost and capital cost of the onsite detention facilities, and $234,000 in annual operation and maintenance costs. The average annual flood abatement benefit was estimated at $73,000, the difference between the potential average annual flood damage under existing land use and channel and floodplain conditions and year 2000 planned land use and existing channel conditions with floodplain fringe development. The benefit-cost ratio was thus estimated at 0.13.

Centralized Storage Alternative: A centralized—or on-stream—detention storage alternative flood control plan was also prepared and evaluated. As shown on Map 69, this alternative consists of the construction of on-stream detention basins at the following five locations: 1) upstream of S. Howell Avenue on the Oak Creek main stem; 2) upstream of S. 27th Street on the Oak Creek main stem; 3) upstream of S. 31st Street on the Oak Creek main stem; 4) upstream of the first S. 6th Street crossing of

[3]*A revised version of Chapter NR 116 became effective in March 1986 limiting this increase to 0.01 foot.*

Map 68

DECENTRALIZED STORAGE ALTERNATIVE FOR THE OAK CREEK WATERSHED



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODLANDS--PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS WITH ONSITE DETENTION STORAGE

100-YEAR RECURRENCE INTERVAL FLOODLAND--PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

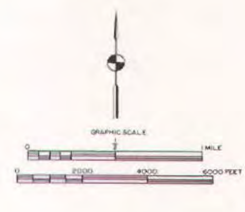
▲ STRUCTURE EXPERIENCING DAMAGES UNDER FLOOD CONTROL ALTERNATIVE

STRUCTURE TYPE

C COMMERCIAL

G GOVERNMENTAL

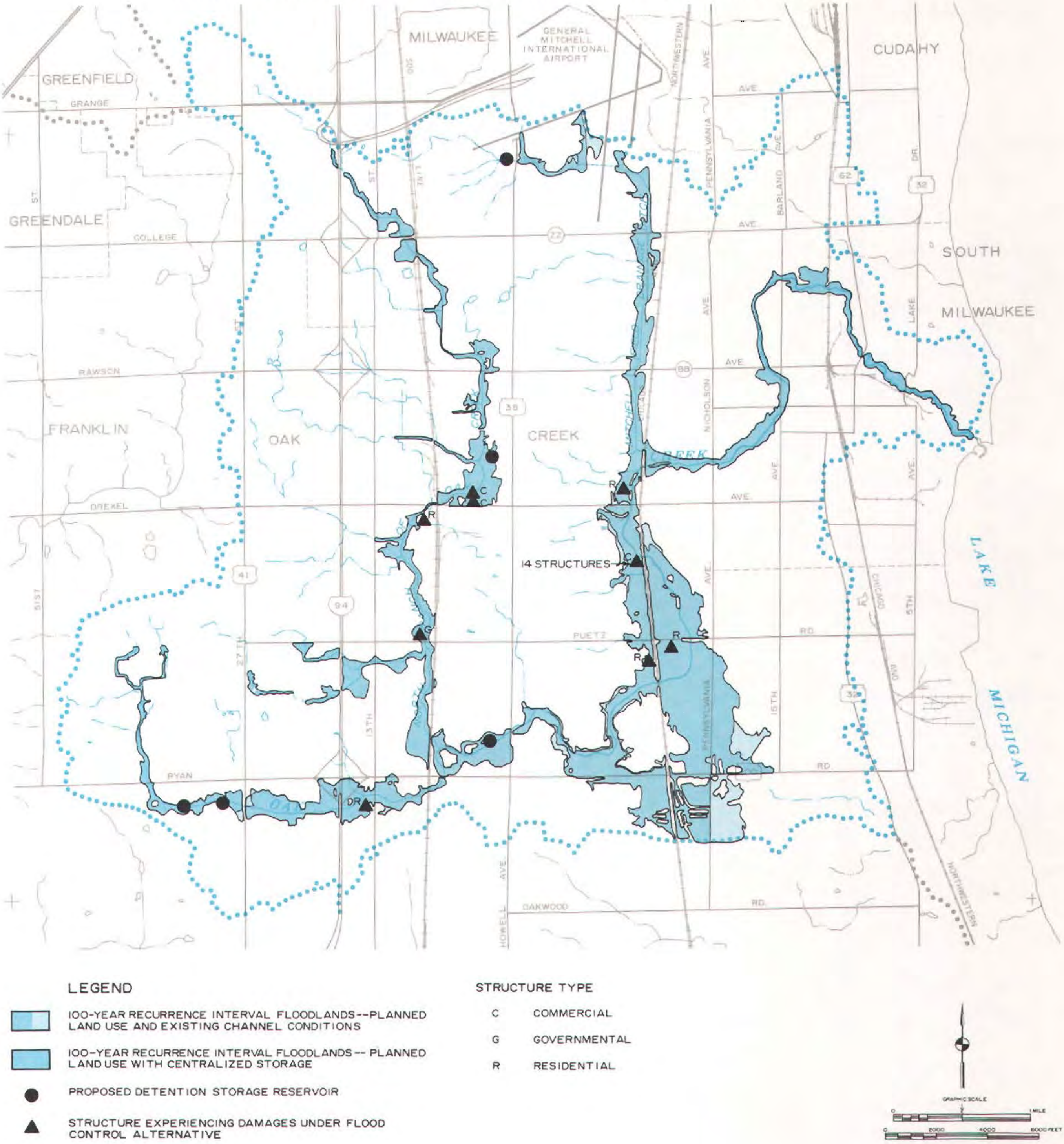
R RESIDENTIAL



Source: SEWRPC.

Map 69

CENTRALIZED STORAGE ALTERNATIVE FOR THE OAK CREEK WATERSHED



Source: SEWRPC.

the North Branch of Oak Creek; and 5) upstream of S. Howell Avenue on the Mitchell Field Drainage Ditch. These sites were selected because of their proximity to reaches with potential flood damages, and because basins at these sites would have the greatest potential impact on downstream peak flood discharges.

The results of the hydrologic-hydraulic simulation modeling conducted under this alternative indicate that some flood damage potential would remain under this alternative. The locations of the residual flood damages attendant to the 100-year recurrence interval flood event under year 2000 planned land use and existing channel conditions with floodplain fringe development are shown on Map 69.

Utilizing an interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this alternative was estimated at $64,000, consisting of the amortization of the $723,000 land cost and capital cost of constructing the five detention basins, and $18,000 in annual operation and maintenance costs. The average annual flood abatement benefit is estimated at $58,000, resulting in a benefit-cost ratio of 0.91.

<u>Combination Major Channelization, Channel Deepening and Shaping, and Structure Floodproofing, Elevation, and Removal Alternative</u>: Based upon the results of the analyses of the flood control alternatives, a ninth flood control alternative was developed. This alternative, as shown on Map 70, incorporates the major channel modifications described under the third major channelization alternative; that is, modifications would be carried out along Oak Creek from the Soo Line Railroad crossing upstream to S. 27th Street, a distance of 1.5 miles; and along the North Branch of Oak Creek from about 960 feet downstream of the confluence with Southland Creek and extending upstream to W. Rawson Avenue, a distance of 2.8 miles. These channels would have bottom widths of 20 feet and side slopes of one on three, and would be turf-lined. Major channel modifications would also be made along an additional 2.1 miles of the North Branch of Oak Creek from W. Rawson Avenue to W. Ramsey Avenue. In addition to these major channel modifications, deepening and shaping of the channel would be required along three stream reaches: 1) Oak Creek between S. Pennsylvania Avenue and E. Puetz Road, a distance of 2.1 miles; 2) Oak Creek extending from a point about 0.5 mile downstream of S. Shepard Avenue to a point about 0.3 mile upstream of S. Shepard Avenue, a distance of 0.8 mile; and 3) the Mitchell Field Drainage Ditch from its confluence with Oak Creek upstream to E. Rawson Avenue, a distance of 0.8 mile. In these reaches the streambed would be lowered an average of three feet in order to provide an adequate outlet for existing storm sewer outfalls.

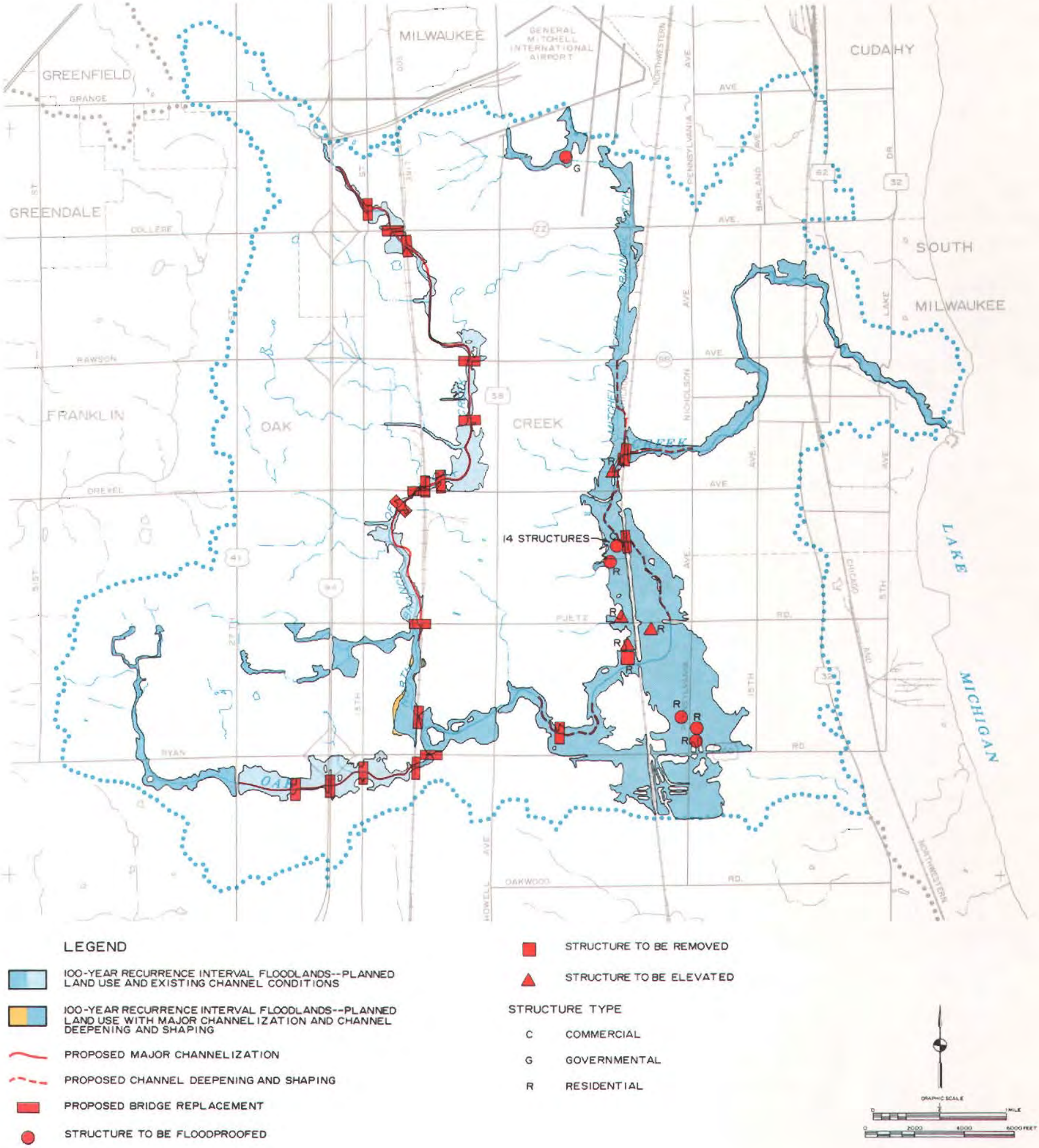
The hydrologic-hydraulic analysis conducted under this alternative indicates that the loss of floodwater storage would result in an increase of 0.1 to 0.8 foot in the 100-year recurrence interval flood profile for Oak Creek downstream of the proposed channel modifications. The increase in the flood profile on the North Branch of Oak Creek is expected to range from 0.8 foot to 1.4 feet. Therefore, flooding easements would have to be obtained under this alternative for the Oak Creek main stem and the North Branch of Oak Creek downstream of the proposed channel modifications. In order to alleviate the residual structure damages which would be expected to remain, this alternative includes the floodproofing of 19 buildings, the elevation of four buildings, and the removal of one building. Remaining flood damages under this alternative would be limited to crop damages.

Utilizing an interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this alternative was estimated at $593,000, consisting of the amortization of the $8,965,000 capital cost of channel improvements and bridge replacements, and of structure floodproofing, elevation and removal, and $10,000 in annual operation and maintenance costs. The average annual flood abatement benefit is estimated at $92,000, resulting in a benefit-cost ratio of 0.16.

<u>Combination Major Channelization, Channel Deepening and Shaping, Centralized Storage, and Structure Floodproofing and Elevation Alternative</u>: A second flood control alternative was developed which combines several of the flood control elements described in previous alternatives. Under this alternative, as shown on Map 71, major channel modifications would be made along Oak Creek from the Soo Line Railroad crossing upstream to S. 27th Street, a distance of 1.5 miles; and along the North

Map 70

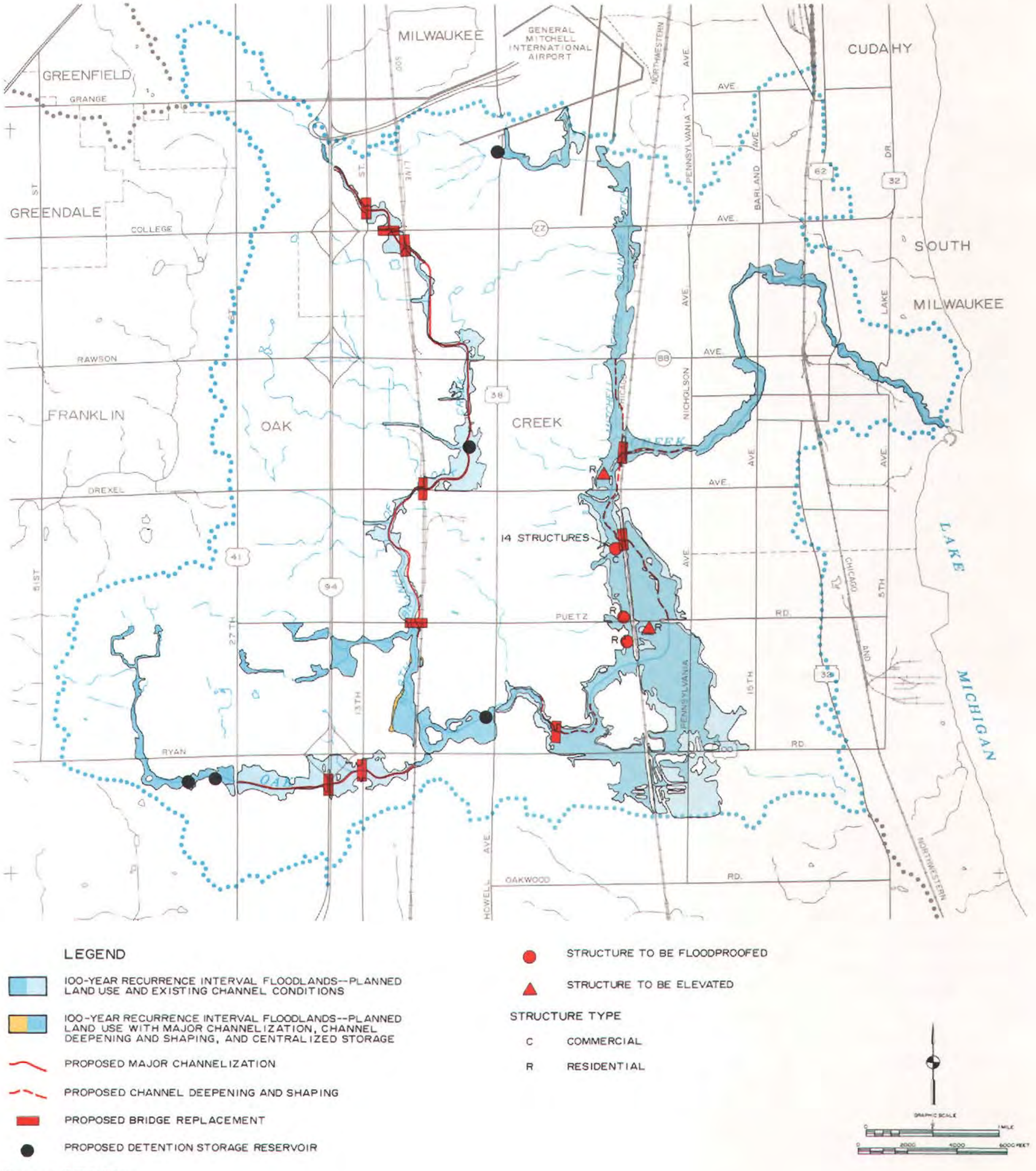
COMBINATION MAJOR CHANNELIZATION, CHANNEL DEEPENING AND SHAPING, AND STRUCTURE FLOODPROOFING, ELEVATION, AND REMOVAL ALTERNATIVE FOR THE OAK CREEK WATERSHED



Source: SEWRPC.

Map 71

COMBINATION MAJOR CHANNELIZATION, CHANNEL DEEPENING AND SHAPING, CENTRALIZED STORAGE, AND STRUCTURE FLOODPROOFING AND ELEVATION ALTERNATIVE FOR THE OAK CREEK WATERSHED



Source: SEWRPC.

Branch of Oak Creek from about 960 feet downstream of the confluence of Southland Creek and extending upstream to W. Ramsey Avenue, a distance of 4.9 miles. These channels would be turf-lined and would have a bottom width of 20 feet and side slopes of one on three. Channel deepening and shaping would also be required along: 1) Oak Creek between S. Pennsylvania Avenue and E. Puetz Road, a distance of 2.1 miles; 2) Oak Creek from a point 0.5 mile downstream of S. Shepard Avenue to a point 0.3 mile upstream of S. Shepard Avenue, a distance of 0.8 mile; and 3) the Mitchell Field Drainage Ditch from its confluence with Oak Creek upstream to E. Rawson Avenue, a distance of 0.8 mile. In these reaches the streambed would be lowered an average of three feet.

In addition to the above channel improvements, five centralized, or on-stream, detention basins would be constructed at the following locations: 1) upstream of S. Howell Avenue on Oak Creek; 2) upstream of S. 27th Street on Oak Creek; 3) upstream of S. 31st Street on Oak Creek; 4) upstream of the first S. 6th Street crossing of the North Branch of Oak Creek; and 5) upstream of S. Howell Avenue on the Mitchell Field Drainage Ditch.

This alternative plan also includes the replacement of five bridges on Oak Creek and five bridges on the North Branch of Oak Creek.

Residual structure damages in the watershed would be alleviated under this alternative by the floodproofing of 16 buildings and the elevation of two buildings. Remaining flood damages under this alternative would be limited to crop damages.

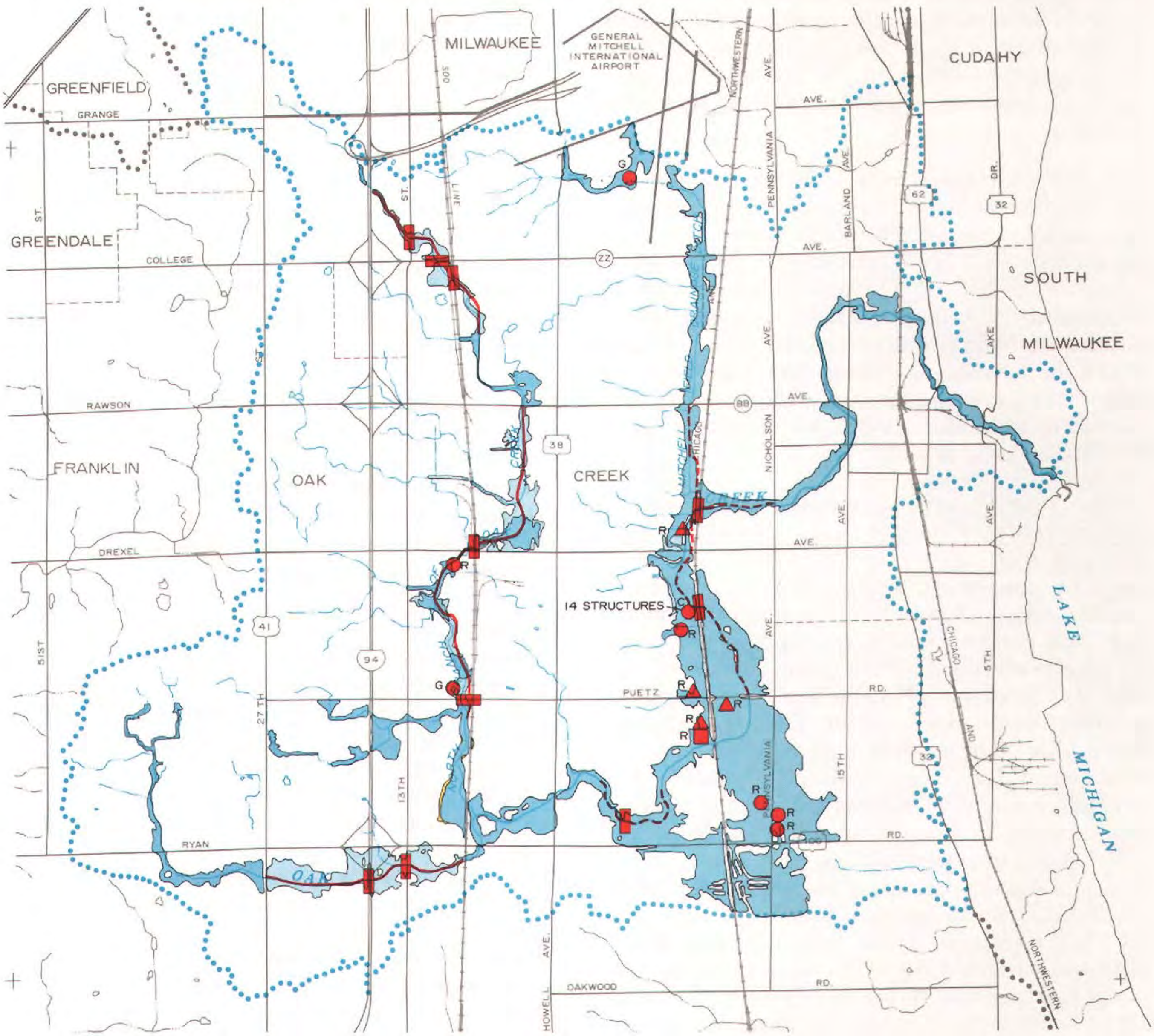
Utilizing an interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this alternative was estimated at $414,000, consisting of the amortization of the $6,122,000 land cost of the detention basins and capital cost of channel improvements, bridge replacements, and detention basins, and of structure floodproofing, elevation, and removal, and $28,000 in annual operation and maintenance costs. The average annual flood abatement benefit is estimated at $98,000, resulting in a benefit-cost ratio of 0.24.

Combination Minimum Channel Modification and Structure Floodproofing, Elevation, and Removal Alternative: A flood control alternative was developed for the watershed which incorporates the minimum amount of channel modifications required to provide an adequate outlet for the existing storm sewer outfalls. This alternative would allow for the implementation of local neighborhood and industrial park development plans. Under this alternative, as shown on Map 72, channel deepening and shaping would occur along Oak Creek between S. Pennsylvania Avenue and E. Puetz Road, a distance of 2.1 miles; Oak Creek from a point 0.5 mile downstream of S. Shepard Avenue to a point about 0.3 mile upstream of S. Shepard Avenue, a distance of 0.8 mile; and the Mitchell Field Drainage Ditch from its confluence with Oak Creek to E. Rawson Avenue, a distance of 0.8 mile. Further channel modifications made along Oak Creek, and the North Branch of Oak Creek would be designed to contain flood discharges up to and including a 10-year recurrence interval event, as opposed to a 100-year recurrence interval event which was used under the other alternatives. Major channelization would be made along Oak Creek from the Soo Line Railroad crossing south of W. Ryan Road upstream to S. 27th Street, a distance of 1.5 miles. The proposed channel would be turf-lined and have a bottom width of 10 feet and one on three side slopes. Major channel modifications would also be made along two reaches on the North Branch of Oak Creek: 1) from a point 960 feet downstream of W. Puetz Road upstream to W. Rawson Avenue, a distance of 2.8 miles; and 2) from the sheet pile spillway located west of the United Parcel Service distribution center upstream to W. Ramsey Avenue, a distance of 1.4 miles. The proposed channel would also be turf-lined, with a bottom width of 10 feet and one on three side slopes.

Because of the loss of floodwater storage under this alternative, stage increases in the 100-year recurrence interval flood of 0.1 to 0.7 foot along the main stem of Oak Creek and of 0.1 to 1.0 foot along the North Branch of Oak Creek may be expected. Therefore, under State law flood easements would be required along the main stem of Oak Creek and along the North Branch of Oak Creek downstream of the proposed channel modifications.

Map 72

COMBINATION MINIMUM CHANNEL MODIFICATION AND STRUCTURE FLOODPROOFING, ELEVATION, AND REMOVAL ALTERNATIVE FOR THE OAK CREEK WATERSHED



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODLANDS--PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

100-YEAR RECURRENCE INTERVAL FLOODLANDS--PLANNED LAND USE WITH MAJOR CHANNELIZATION AND CHANNEL DEEPENING AND SHAPING

PROPOSED MAJOR CHANNELIZATION

PROPOSED CHANNEL DEEPENING AND SHAPING

PROPOSED BRIDGE REPLACEMENT

STRUCTURE TO BE FLOODPROOFED

STRUCTURE TO BE ELEVATED

STRUCTURE TO BE REPLACED

STRUCTURE TYPE

C COMMERCIAL

G GOVERNMENTAL

R RESIDENTIAL

GRAPHIC SCALE

0 1 MILE

0 2000 4000 8000 FEET

Source: SEWRPC.

This alternative plan also includes the replacement of five bridges on Oak Creek and six bridges on the North Branch of Oak Creek. The residual structure damages which would be expected to remain would be alleviated by the floodproofing of 21 buildings, elevation of four buildings, and removal of one building. Remaining flood damages would thus be limited to crop damages.

Utilizing an interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this alternative was estimated at $327,000, consisting of the amortization of the $5,047,000 capital cost of channel improvements, bridge replacements, and structure floodproofing, elevation, and removal, and $9,000 in annual operation and maintenance costs. The average annual flood abatement benefit is estimated at $93,000, resulting in a benefit-cost ratio of 0.28.

Combination Channel Deepening and Shaping, and Structure Floodproofing, Elevation, and Removal Alternative: An alternative flood control plan consisting of limited channel deepening and shaping and structure floodproofing, elevation, and removal was prepared and evaluated for the watershed. This plan is shown on Map 73. Under this plan, channel deepening and shaping would occur along Oak Creek from River Mile 10.30 upstream to S. 27th Street, a distance of 1.4 miles. Within this reach, the streambed would be lowered an average of three feet in order to provide an adequate outlet for existing storm sewer outfalls, and also to eliminate the negative channel slope between IH 94 and S. 20th Street. Between River Mile 10.30 and IH 94, the channel would have a bottom width of 10 feet with side slopes of one on three, similar to the existing side slopes in this reach. Between IH 94 and S. 27th Street, the channel would have a bottom width of 10 feet with side slopes of one on three in order to facilitate maintenance of the channel through the industrial park. Overland flooding would still be expected to occur during major runoff events along this reach of channel deepening. Flows associated with minor runoff events having recurrence intervals of two years or less would, however, be confined to the reach of the channel through the planned industrial park between IH 94 and S. 27th Street.

Channel deepening and shaping would also be required along the North Branch of Oak Creek starting at the steel sheet pile spillway located west of the United Parcel Service distribution center and extending upstream to S. 13th Street, a distance of 1.0 mile. Within this reach, the streambed would be lowered an average of three feet in order to provide an outlet for a storm sewer outfall that is currently below the streambed. The proposed channel would have a bottom width of 10 feet with side slopes of one on two to one on five, similar to the existing side slopes in this reach. Overland flooding would still be expected to occur through this reach during major runoff events. More frequent events having recurrence intervals of two years or less would, however, be confined to that reach of the channel beginning at the north end of the MATC-South Campus and extending upstream to S. 13th Street.

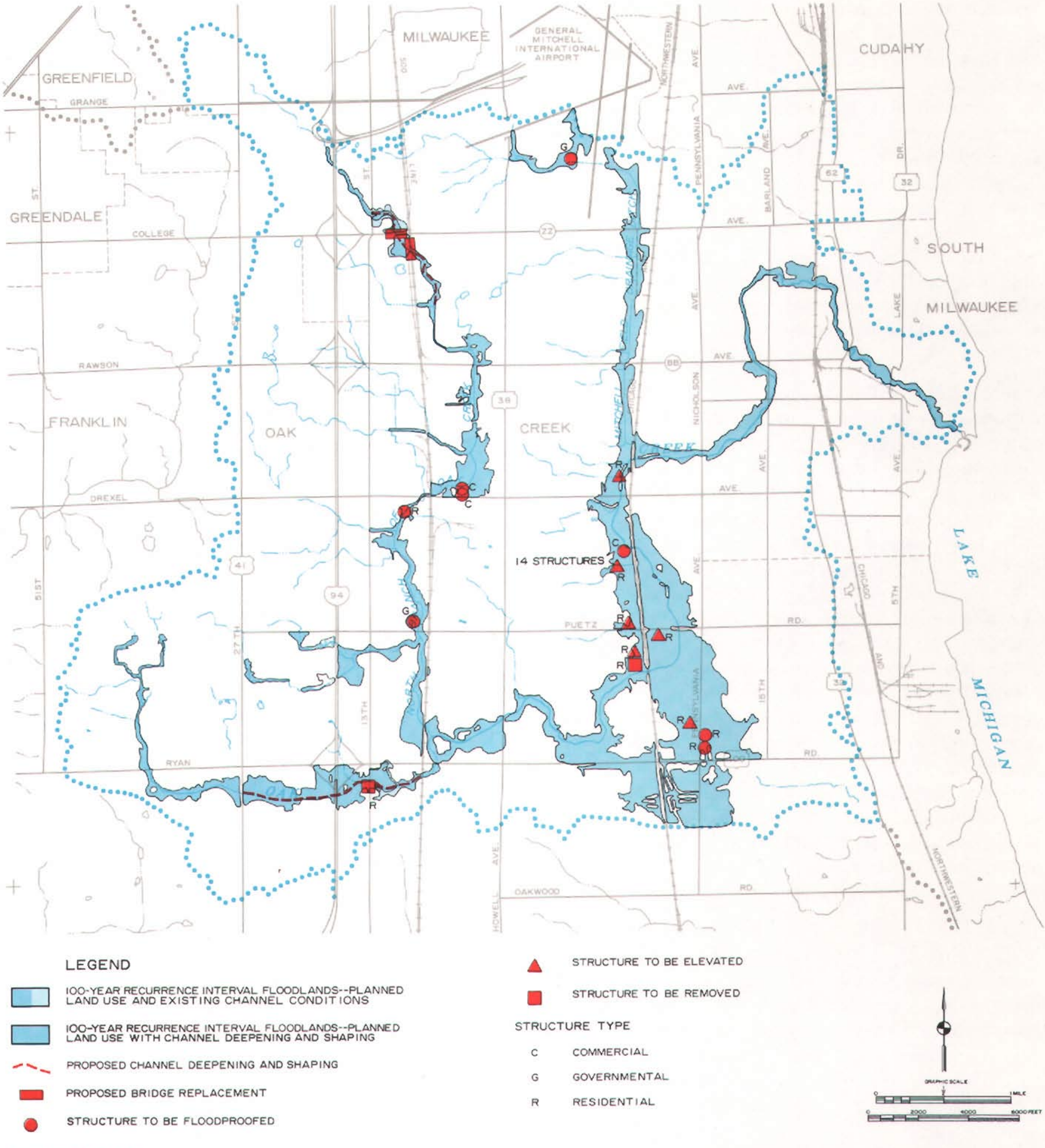
Also under this alternative plan, two bridges on the North Branch of Oak Creek would be replaced, and structure damages would be alleviated by the floodproofing of 21 buildings, elevation of six buildings, and removal of two buildings. Remaining flood damages would be limited to crop damages.

Utilizing an interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this alternative was estimated at $65,000, consisting of the amortization of the $1,009,000 capital cost of channel modification, bridge replacement, and structure floodproofing, elevation, and removal, and $1,000 in annual operation and maintenance costs for the Oak Creek channel through the proposed industrial park between IH 94 and S. 27th Street and for the North Branch channel between College Avenue and S. 13th Street—this reach currently being maintained by the City of Milwaukee. The average annual flood abatement benefits are estimated at $78,000, resulting in a benefit-cost ratio of 1.20.

As part of this flood control alternative, three subalternatives were considered for providing an adequate outlet for the storm sewer with the outfall located below the existing channel invert: 1) raising the storm sewer so that the outfall from this sewer matches the existing channel invert; 2) constructing a new storm sewer parallel to the North Branch of Oak Creek channel which would convey the flow from the restricted sewer to a point downstream where an adequate outlet can be achieved; and 3) providing a pumping station at the outlet of the sewer.

Map 73

COMBINATION LIMITED CHANNEL DEEPENING AND SHAPING AND STRUCTURE FLOODPROOFING, ELEVATION, AND REMOVAL ALTERNATIVE FOR THE OAK CREEK WATERSHED



Source: SEWRPC.

The sewer elevation subalternative would require raising the storm sewer which enters the North Branch of Oak Creek at S. 13th Street a minimum of 3.6 feet. The storm sewer is constructed at the minimum required depth of cover, and elevation of this storm sewer would therefore not be practical. If the elevation of this sewer could be achieved, the cost entailed would be about $120,000. Besides being impractical, this subalternative would not alleviate the poor drainage conditions which exist in the North Branch of Oak Creek between W. College Avenue and the private bridge located at River Mile 4.67. These drainage conditions occur under periods of low flow and are caused by the negative channel slopes in this stream reach. Because of these problems, this subalternative was not considered further.

Under the parallel storm sewer subalternative which was analyzed under this alternative, approximately 0.8 mile of 54-inch-diameter pipe would be laid along the east side of the North Branch of the Oak Creek channel between S. 13th Street and the private bridge at River Mile 4.35. This intercepting sewer is shown in Figure 31. This sewer would have sufficient capacity to accommodate runoff from rainfall events having recurrence intervals of up to and including five years. The cost of providing the intercepting sewer is estimated at $410,000. This subalternative would not alleviate the problem of poor drainage in the stream channel because of the negative slope in the channel. Because of the relatively high cost of this subalternative, and the fact that the channel would continue to have standing water under low-flow periods, the laying of the intercepting storm sewer was not considered further.

The final subalternative considered was the installation of a lift station at the outlet of the restricted storm sewer. The cost of installing this lift station is estimated at $350,000, with annual operation and maintenance costs of $2,500. This subalternative would not alleviate poor drainage conditions in the channel reach described above. Because of the relatively high cost of this subalternative, as well as the fact that poor drainage conditions would remain in the channel reach described above, the installation of a lift station was not considered further.

Recommended Flood Control
Plan for the Oak Creek Watershed

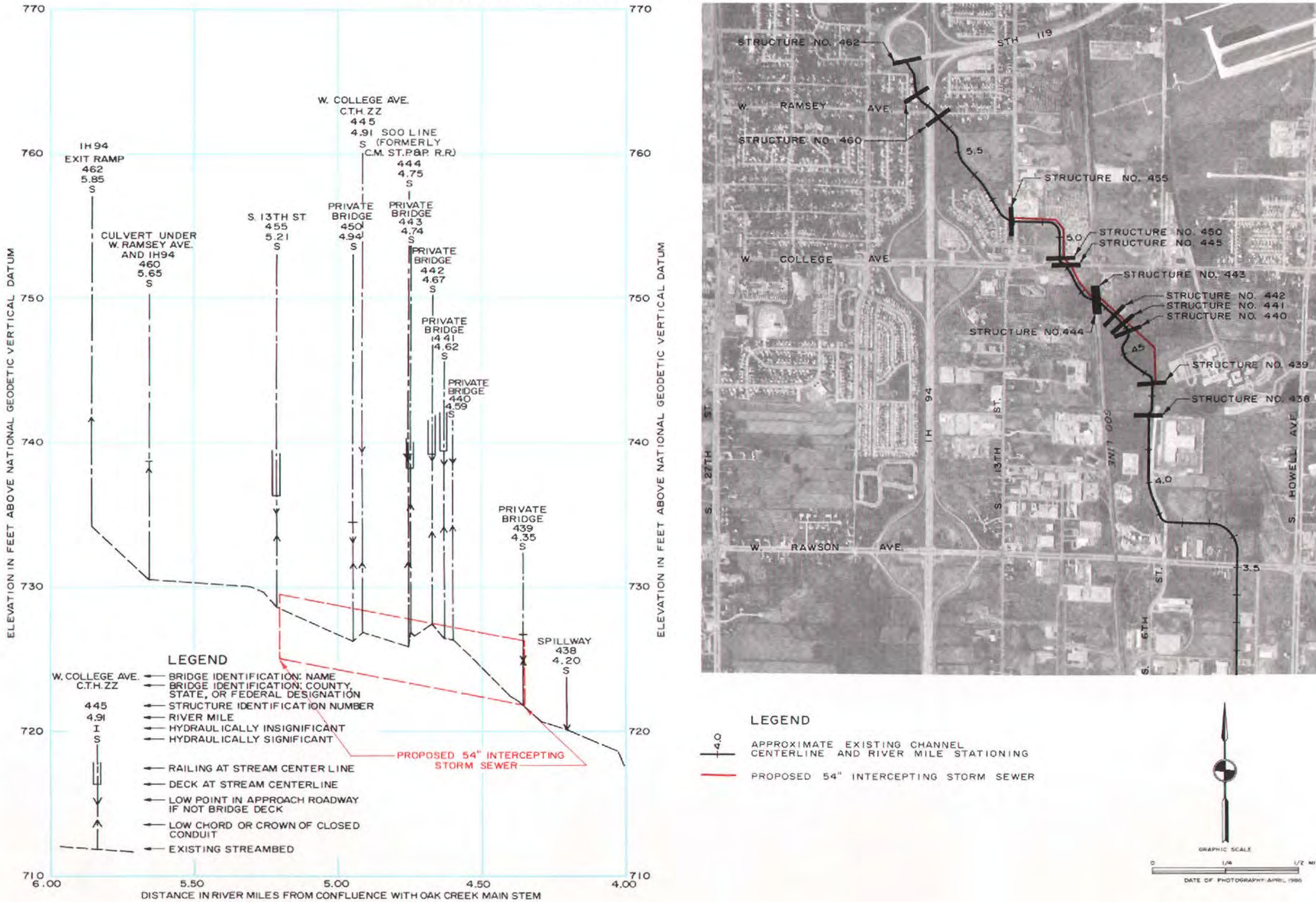
Based upon consideration of the technical feasibility, economic viability, environmental impacts, potential public acceptance, and practicality of each of the alternatives considered, it was recommended that the combination limited channel deepening and shaping, and structure floodproofing, elevation, and removal alternative be adopted and implemented for the Oak Creek watershed. This recommended alternative consists of the following components: 1) channel deepening and shaping of 1.4 miles of the Oak Creek channel between River Mile 10.30 and the S. 27th Street crossing, and of 1.0 mile of the North Branch of Oak Creek between the steel sheet pile spillway located west of the United Parcel Service distribution center and the S. 13th Street crossing; 2) the floodproofing of 21 buildings, the elevation of six buildings, and the removal of two buildings; and 3) the replacement of two bridges on the North Branch of Oak Creek.

Of the 21 buildings recommended for floodproofing, 16 are located along the main stem of Oak Creek, consisting of two houses and 14 commercial buildings; four are located along the North Branch of Oak Creek, consisting of two commercial buildings, one apartment building, and one municipal garage; and one office and warehouse building is located along the Mitchell Field Drainage Ditch. All six of the buildings recommended to be elevated, as well as the two buildings recommended for removal, are houses located along the main stem of Oak Creek.

With respect to bridge replacement, the two bridges recommended to be replaced are the W. College Avenue and Soo Line Railroad crossings. It should be noted that the W. College Avenue crossing was replaced in 1987 by Milwaukee County in accordance with the recommended plan. Thus, the waterway opening hydraulic capacity and channel bottom elevation of the new bridge should accommodate the proposed channel improvements. It should also be noted that the Soo Line Railroad crossing must be replaced with a structure having the same hydraulic characteristics as the existing crossing, which has a considerable backwater effect and results in significant floodwater storage under major flood conditions. Since there

Figure 31

PROFILE ILLUSTRATION OF INTERCEPTING STORM SEWER FOR A PORTION OF NORTH BRANCH OF OAK CREEK



Source: SEWRPC.

Table 38

ECONOMIC ANALYSIS OF THE RECOMMENDED FLOOD CONTROL PLAN FOR THE OAK CREEK WATERSHED

Recommended Flood Control Measures	Costs				Benefit-Cost Analysis			
		Annual						
	Capital	Amortized Capital[a]	Operation and Maintenance	Total	Annual Benefits (dollars)	Annual Benefits Minus Annual Costs (dollars)	Benefit Cost Ratio	Economic Ratio Greater than One
Channel Deepening and Shaping	$ 347,000	$22,000	$2,000	$24,000				
Bridge Replacement	201,000	13,000	- -	13,000				
Structure Floodproofing, Elevation, and Removal	645,000	41,000	- -	41,000				
Subtotal	$1,193,000	$76,000	$2,000	$78,000	$88,000	$10,000	1.13	Yes

[a]*Amortized capital cost is based on an interest rate of 6 percent and a project life of 50 years.*

Source: SEWRPC.

are no flood damages resulting from this restriction to flow, a replacement crossing must exhibit the same floodwater storage effect as the existing crossing to prevent any increase in downstream flood flows and stages.

The average annual cost of this alternative, assuming an interest rate of 6 percent and a project life and amortization period of 50 years, is estimated at $78,000, consisting of the following: amortization of the $347,000 capital cost of channel deepening and shaping; amortization of the $201,000 capital cost of bridge replacement; amortization of the $645,000 capital cost of the floodproofing, elevation, and removal of 29 buildings; and $2,000 in annual operation and maintenance costs. The recommended flood control plan for the Oak Creek watershed is graphically summarized on Map 74, and attendant costs set forth in Table 38. The 100-year recurrence interval flood profile and streambed profile under the recommended flood control plan are shown in Figure 32.

Implementation of this flood control plan would result in the abatement of all flood damages in the watershed caused by flood events up to and including the 100-year recurrence interval event under year 2000 planned land use conditions. Implementation of the flood control plan would not, however, serve to eliminate local stormwater drainage problems in the watershed. The abatement of those problems should be addressed through the preparation of stormwater management system plans.

It is recommended that when those bridges identified in Appendix C as having inadequate hydraulic capacity are replaced for transportation purposes, they be designed to accommodate the respective recommended design frequency flood event without overtopping of the attendant roadway. Those structures to be designed to accommodate a 10-year recurrence interval flood flow include S. Shepard Avenue and S. 20th Street on Oak Creek and Wildwood Drive, S. 6th Street, and W. Marquette Avenue on the North Branch of Oak Creek. Those structures to be designed to accommodate a 50-year recurrence interval flood flow include E. Forest Hill Avenue, E. Puetz Road, S. Nicholson Road, W. Ryan Road, S. 13th Street, and W. Puetz Road on Oak Creek; W. Puetz Road on the North Branch of Oak Creek; and W. College Avenue on the Mitchell Field Drainage Ditch.

Finally, it is recommended that large-scale topographic maps be prepared for two reaches of Oak Creek and one reach of the North Branch of Oak Creek. Existing topographic maps do not reflect the significant amount of development—including channel replacement—which has occurred in these areas. It is also recommended that new maps be prepared for the south one-half of U. S. Public Land Survey Section 5 and the north one-half of U. S. Public Land Survey Section 30 in the City of Oak Creek (Township 5 North, Range 22 East), and the southwest one-quarter of U. S. Public Land Survey Section 24 in the City of Franklin (Township 5 North, Range 21 East). Since these new maps would serve multiple purposes, none of the attendant costs have been assigned to the flood control plan.

Impacts of Recommended Land Use and Floodland Management Plans on Flood Flows and Stages: Implementation of the recommended land use and floodland management plans may be expected to have a significant impact on flood flows and stages in the Oak Creek watershed. The impacts of plan implementation on the regulatory 100-year recurrence interval flood are given for selected locations along the stream system of the Oak Creek watershed in Table 39. Future urban land use development proposed for the watershed accounts for the increase in peak flood flows and stages. Along those stream reaches where channel deepening and shaping is recommended, peak flood stages may be expected to be lower than under planned land use development and existing channel conditions.

Flood Control Plan Implementation

The major floodland management recommendation of the District's drainage and flood control system plan is the institution of sound floodland zoning regulations throughout the watershed and the acquisition for public park and open space use of primary environmental corridor lands along the lower reaches of Oak Creek and in the southeast area of the watershed. It is important to note, however, that the floodland zoning measures to be applied need to be coordinated with the implementation of the recommended structural flood control measures developed under the Commission Oak Creek watershed planning program and set forth in this report. That is, the local zoning agencies need to apply appropriate floodland zoning to the existing floodlands in the watershed, particularly along Oak Creek and the North Branch of Oak Creek, based upon future land use and existing channel conditions until such time as the recommended channel improvements are undertaken. At that time, the floodland zoning regulations may be adjusted to reflect the improvements that have actually been put in place.

It is recommended that the Milwaukee Metropolitan Sewerage District make the needed channel improvements within the Oak Creek watershed. In particular, it is recommended that the District carry out the recommended channel deepening and shaping along 1.4 miles of Oak Creek between River Mile 10.30 and the S. 27th Street crossing, and along 1.0 mile of the North Branch of Oak Creek between the steel sheet pile spillway located west of the United Parcel Service distribution center and the S. 13th Street crossing.

The recommended plan also calls for structure floodproofing, elevation, and removal measures to be undertaken along Oak Creek and the North Branch of Oak Creek in the City of Oak Creek, and along the Mitchell Field Drainage Ditch in the City of Milwaukee. Structure floodproofing and elevation would be undertaken by the property owners directly affected, as, for example, by the Oak Creek Floral Company with respect to its greenhouses located along Oak Creek. It is recommended, however, that the professional services required to prepare plans for floodproofing and the elevation of individual buildings be made available, at no cost, to property owners by the two cities involved through the city engineers. In addition, it is recommended that the Cities of Milwaukee and Oak Creek review their local building ordinances to ensure that appropriate floodproofing regulations are included. Also, it is recommended that these two cities explore on behalf of the property owners directly affected any state and federal aids available for such floodproofing measures. With regard to the two buildings recommended for removal, it is recommended that these properties be acquired by the Milwaukee Metropolitan Sewerage District and subsequently dedicated to the Milwaukee County Department of Parks, Recreation and Culture for parkway purposes.

It is recommended that the District prepare the large-scale topographic maps recommended for certain stream reaches in the watershed.

Concluding Remarks

Implementation of this drainage and flood control system plan would result in the abate-

(Continued on Page 229)

Map 74

RECOMMENDED FLOOD CONTROL SYSTEM PLAN FOR THE OAK CREEK WATERSHED

OAK CREEK

LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS

1.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR OAK CREEK IS SHOWN IN APPENDIX H

NOTE: DUE TO MAP SCALE LIMITATIONS, THE DIFFERENCE BETWEEN THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS, AND THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS, MAY NOT APPEAR ON THIS MAP. WHERE NO DIFFERENCE APPEARS REFERENCE SHOULD BE MADE TO THE FLOOD STAGE PROFILE SHOWN BELOW.

GRAPHIC SCALE

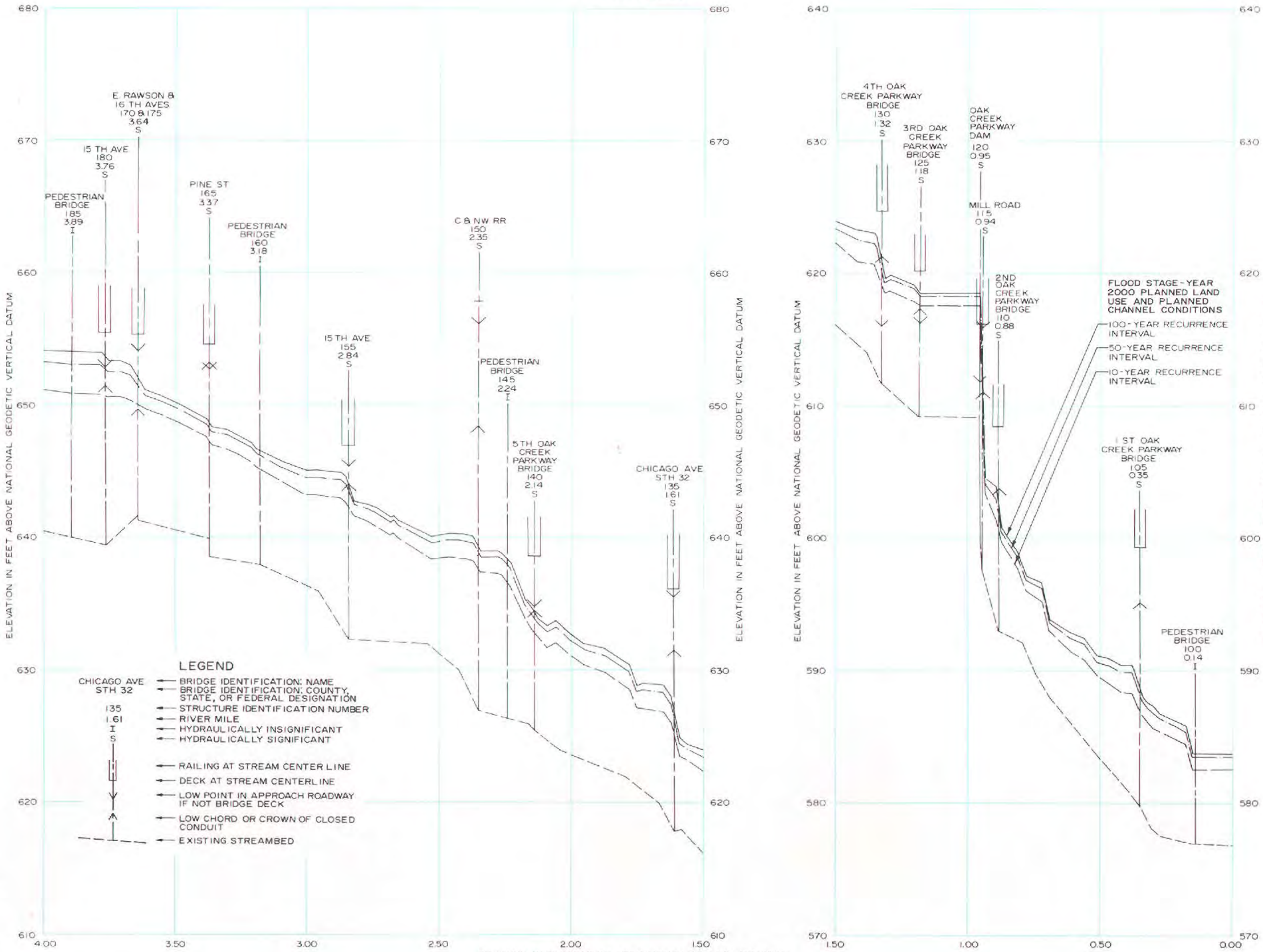
0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Figure 32

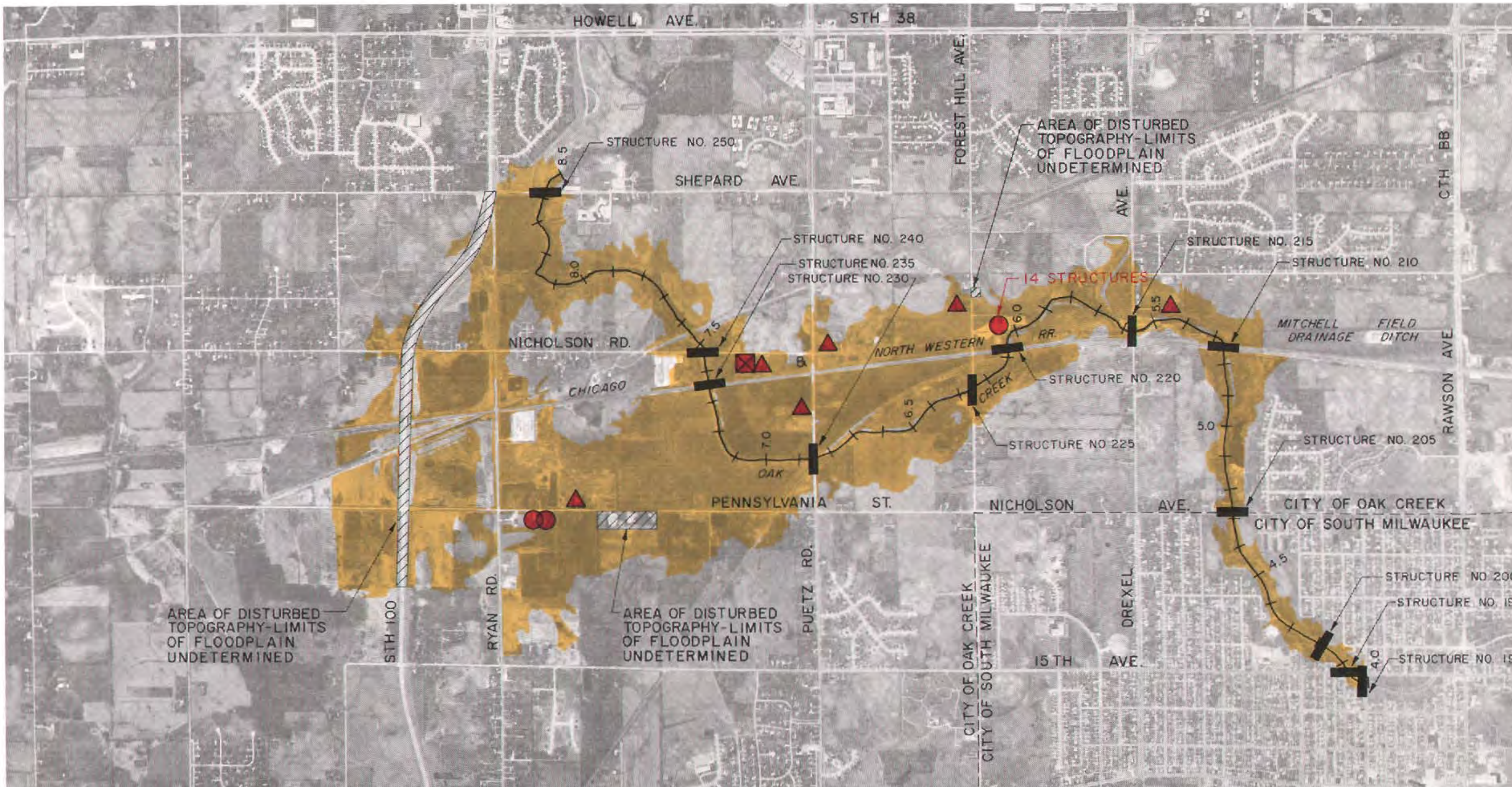
RECOMMENDED PLAN FLOOD STAGE PROFILE FOR THE OAK CREEK WATERSHED

OAK CREEK



Map 74 (continued)

OAK CREEK



LEGEND

1.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

100-YEAR RECURRENCE INTERVAL FLOODLANDS--PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS

STRUCTURE FLOODPROOFING

STRUCTURE ELEVATION

STRUCTURE REMOVAL

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR OAK CREEK IS SHOWN IN APPENDIX H

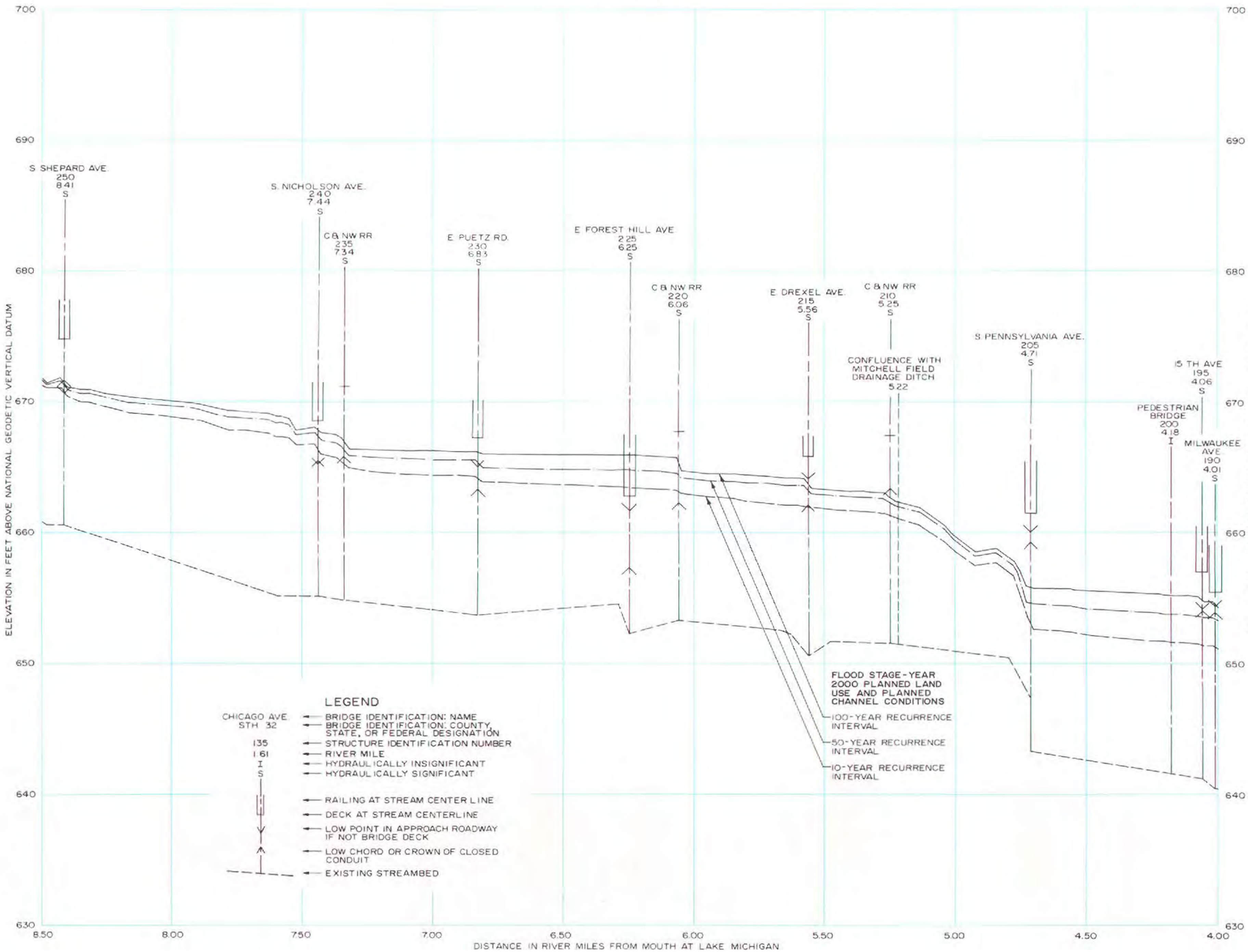
GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1980

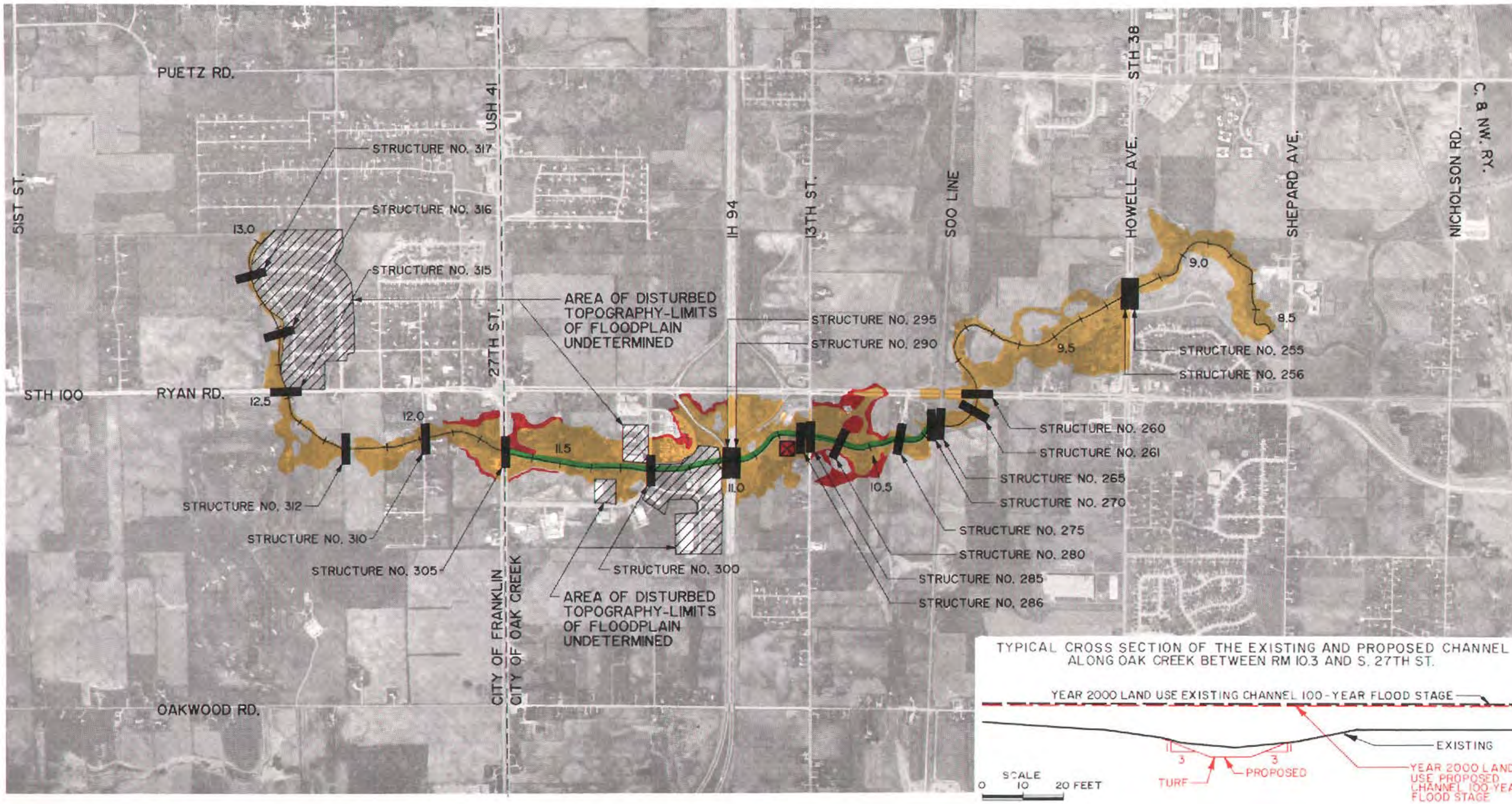
Figure 32 (continued)

OAK CREEK



Map 74 (continued)

OAK CREEK



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS

1.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

CHANNEL DEEPENING AND SHAPING

STRUCTURE TO BE REMOVED

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR OAK CREEK IS SHOWN IN APPENDIX H

NOTE: DUE TO MAP SCALE LIMITATIONS, THE DIFFERENCE BETWEEN THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS, AND THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS, MAY NOT APPEAR ON THIS MAP. WHERE NO DIFFERENCE APPEARS REFERENCE SHOULD BE MADE TO THE FLOOD STAGE PROFILE SHOWN BELOW

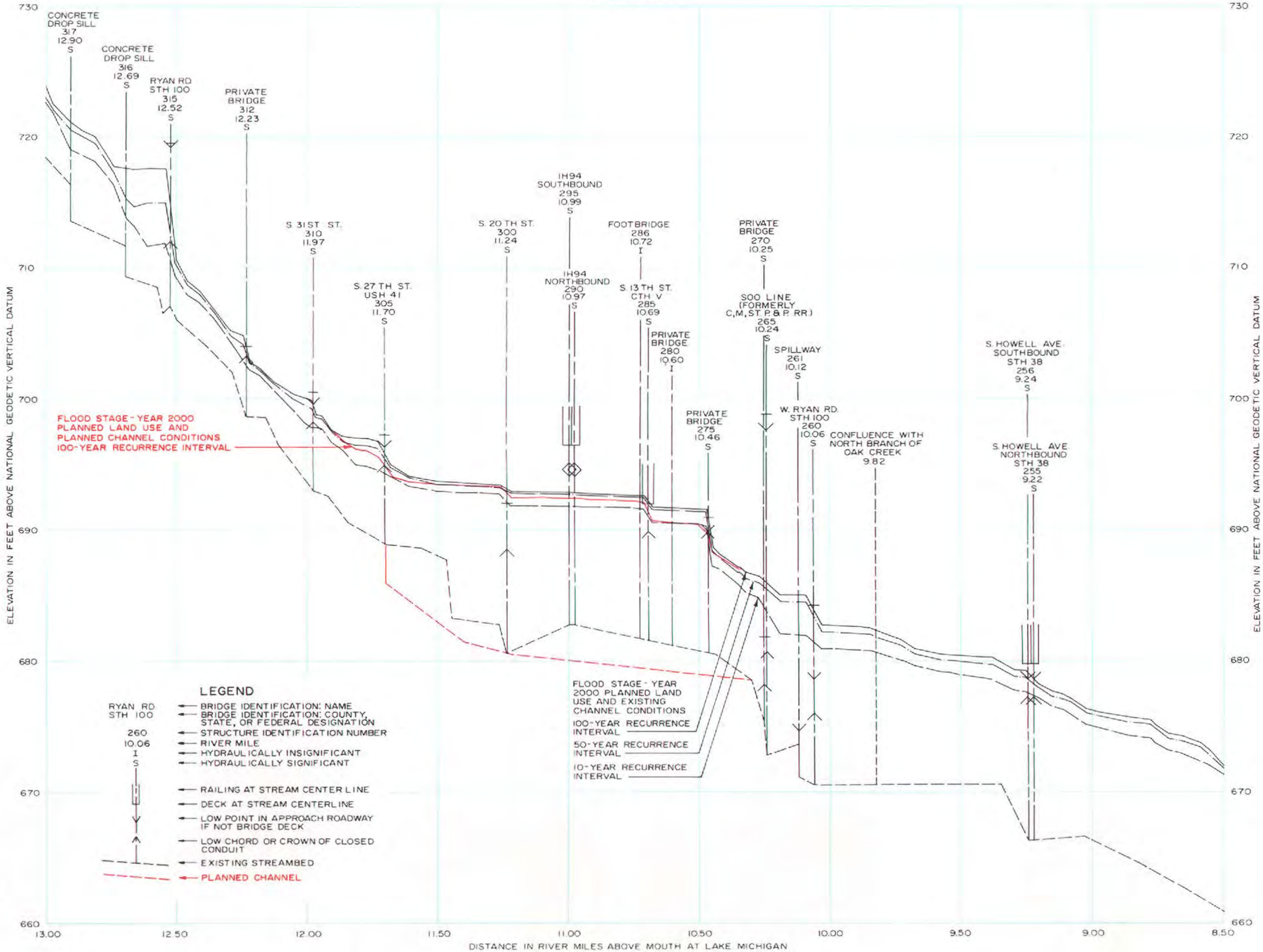
GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Figure 32 (continued)

OAK CREEK



Map 74 (continued)

OAK CREEK

LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS

1.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR OAK CREEK IS SHOWN IN APPENDIX H

GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Figure 32 (continued)

OAK CREEK

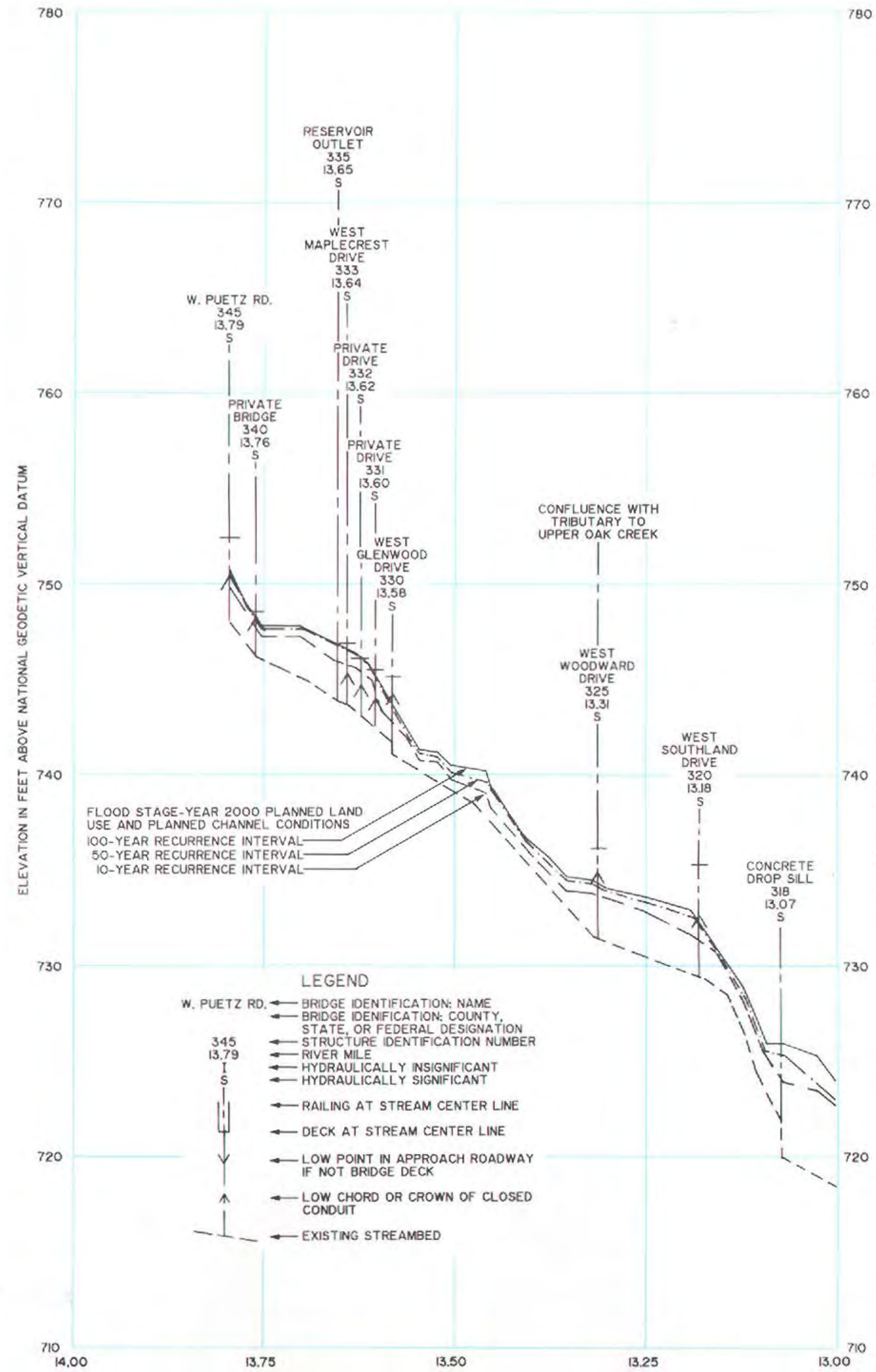


Table 39

100-YEAR RECURRENCE INTERVAL FLOOD DISCHARGES AND STAGES AT SELECTED LOCATIONS IN THE OAK CREEK WATERSHED: EXISTING LAND USE AND CHANNEL CONDITIONS AND PLANNED LAND USE AND CHANNEL CONDITIONS

Location	Existing Land Use and Channel Conditions		Planned Land Use and Channel Conditions	
	Peak Flood Discharge (cfs)	Peak Flood Stage (feet NGVD[a])	Peak Flood Discharge (cfs)	Peak Flood Stage (feet NGVD[a])
Oak Creek				
Confluence with Lake Michigan	1,780	582.1	2,810	583.4
Parkway Dam	1,780	617.5	2,810	618.5
Chicago Avenue	1,780	625.7	2,770	628.0
15th Avenue	1,780	642.5	2,700	644.6
Upstream of Marquette Boulevard Extended	1,780	651.7	2,700	655.5
Upstream of Confluence with Mitchell Field Drainage Ditch	1,500	661.2	2,270	662.4
East Forest Hill Avenue	1,500	663.7	2,270	666.2
Abandoned Chicago, North Shore & Milwaukee Railroad	2,080	666.7	3,220	668.1
South Shepard Avenue	2,080	673.7	3,220	675.1
Upstream of Confluence with North Branch of Oak Creek	1,030	680.7	1,830	682.4
IH 94	790	691.9	1,330	692.7
South 31st Street	410	699.2	490	699.9
Downstream of W. Southland Drive	210	725.6	--[b]	726.0
Upstream of W. Woodward Drive	50	734.6	--[b]	734.6
North Branch of Oak Creek				
Confluence with Oak Creek	1,670	680.5	2,320	682.3
Downstream of W. Puetz Road	1,450	694.6	1,940	695.5
Downstream of Wildwood Drive	930	705.1	1,260	705.8
Soo Line Railroad	880	709.3	1,190	711.4
West Marquette Avenue	520	714.0	900	714.9
MATC-South Campus	160	724.6	240	725.6
Soo Line Railroad	150	733.2	220	736.3
CTH V/S. 13th Street	370	733.4	390	733.4
Mitchell Field Drainage Ditch				
Confluence with Oak Creek	730	661.2	1,050	662.2
CTH BB/W. Rawson Avenue	680	665.9	950	667.0
CTH ZZ/W. College Avenue	520	673.2	620	674.0
Private Drive	740	680.4	1,180	680.7

[a]*NGVD-National Geodetic Vertical Datum.*

[b]*No change in land use.*

Source: SEWRPC.

Map 74 (continued)

MITCHELL FIELD DRAINAGE DITCH

LEGEND

APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

100-YEAR RECURRENCE INTERVAL FLOODLANDS--PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS

STRUCTURE FLOODPROOFING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR OAK CREEK IS SHOWN IN APPENDIX H

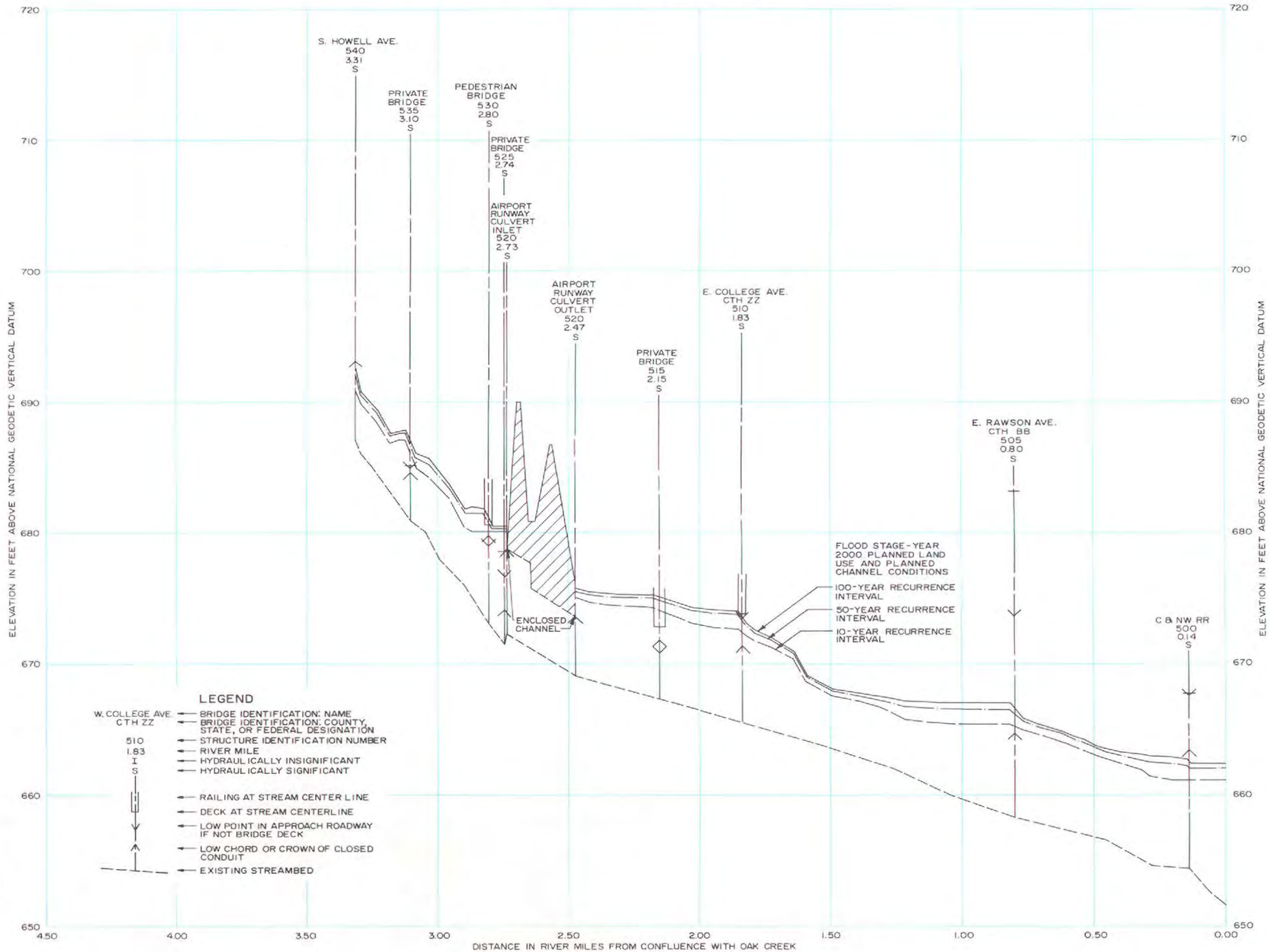
GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

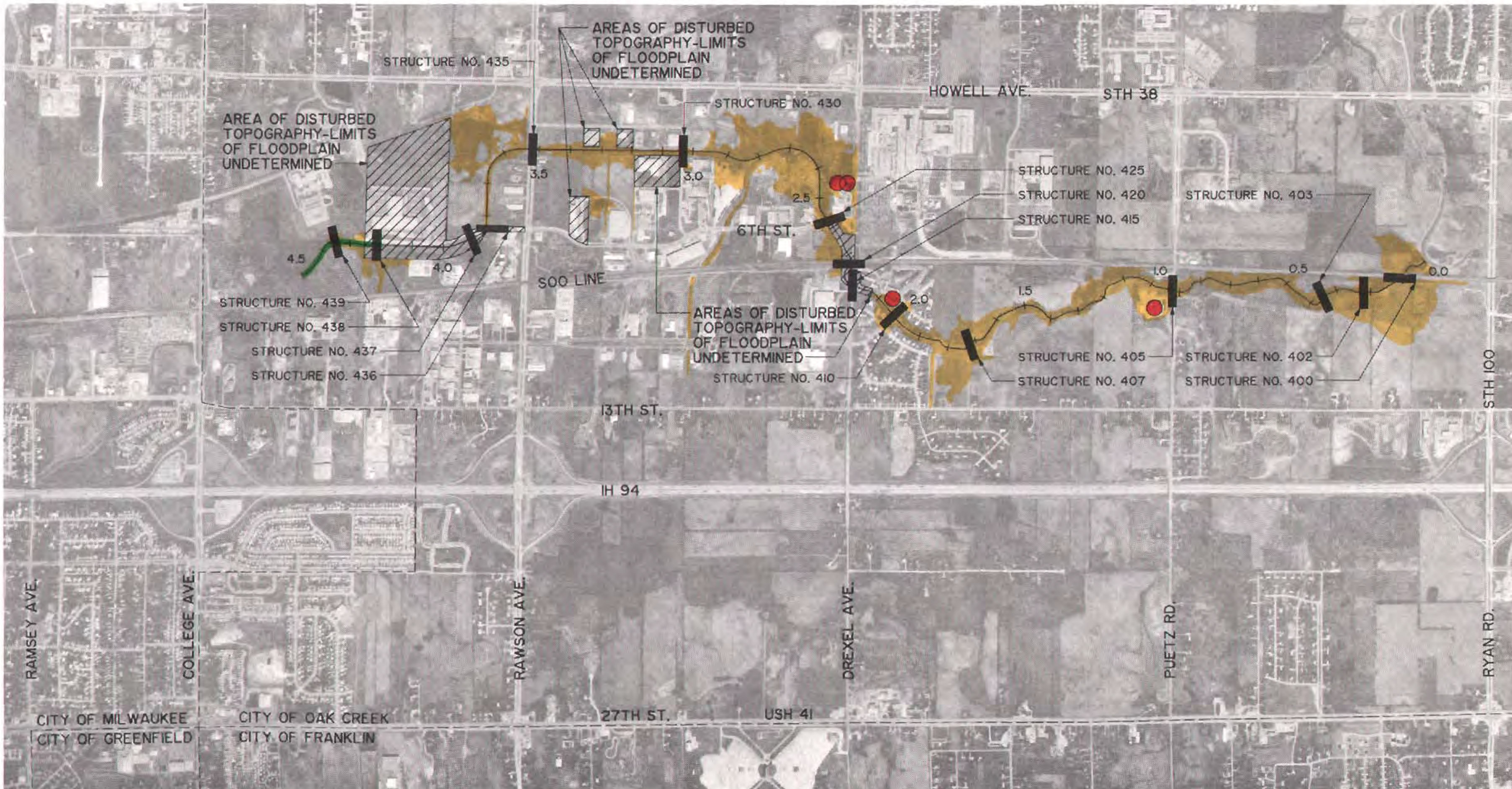
Figure 32 (continued)

MITCHELL FIELD DRAINAGE DITCH



Map 74 (continued)

NORTH BRANCH OF OAK CREEK



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS

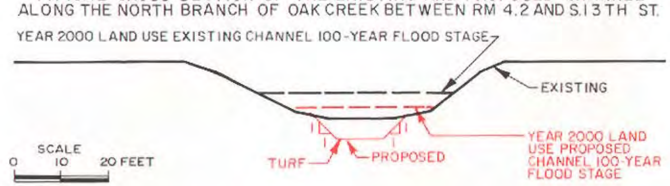
1.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

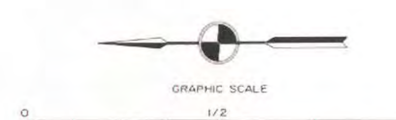
CHANNEL DEEPENING AND SHAPING

STRUCTURE FLOODPROOFING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR NORTH BRANCH OF OAK CREEK IS SHOWN IN APPENDIX H

TYPICAL CROSS SECTION OF THE EXISTING AND PROPOSED CHANNEL ALONG THE NORTH BRANCH OF OAK CREEK BETWEEN RM 4.2 AND S.13TH ST.

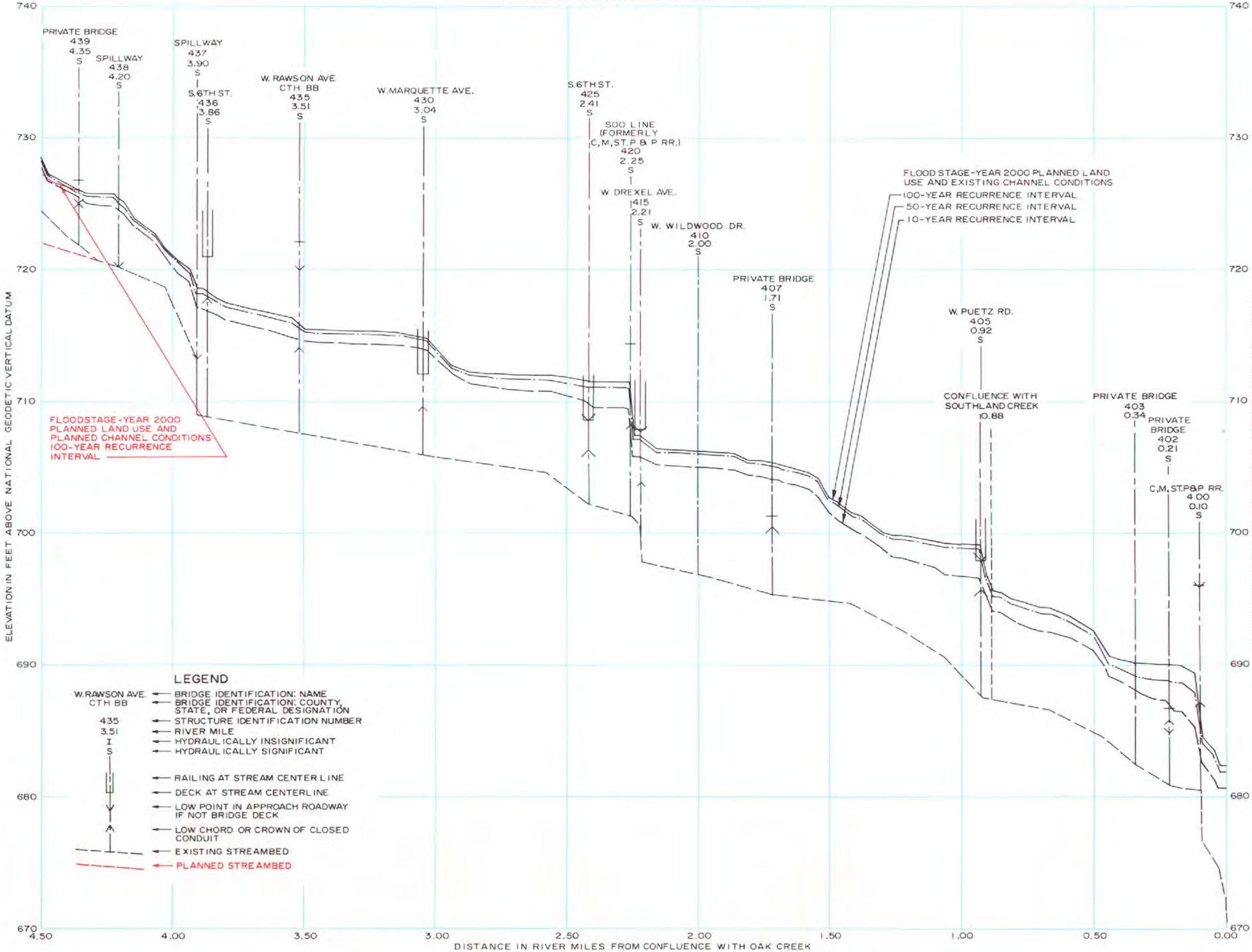




DATE OF PHOTOGRAPHY: APRIL 1986

Figure 32 (continued)

NORTH BRANCH OF OAK CREEK



Map 74 (continued)

NORTH BRANCH OF OAK CREEK

LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS

1.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

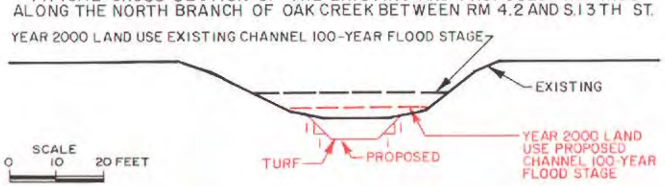
CHANNEL DEEPENING AND SHAPING

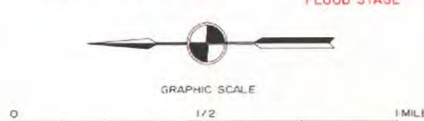
STRUCTURE TO BE REPLACED

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR NORTH BRANCH OF OAK CREEK IS SHOWN IN APPENDIX H

NOTE: DUE TO MAP SCALE LIMITATIONS, THE DIFFERENCE BETWEEN THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS, AND THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS, MAY NOT APPEAR ON THIS MAP. WHERE NO DIFFERENCE APPEARS REFERENCE SHOULD BE MADE TO THE FLOOD STAGE PROFILE SHOWN BELOW

TYPICAL CROSS SECTION OF THE EXISTING AND PROPOSED CHANNEL ALONG THE NORTH BRANCH OF OAK CREEK BETWEEN RM 4.2 AND S.13TH ST.



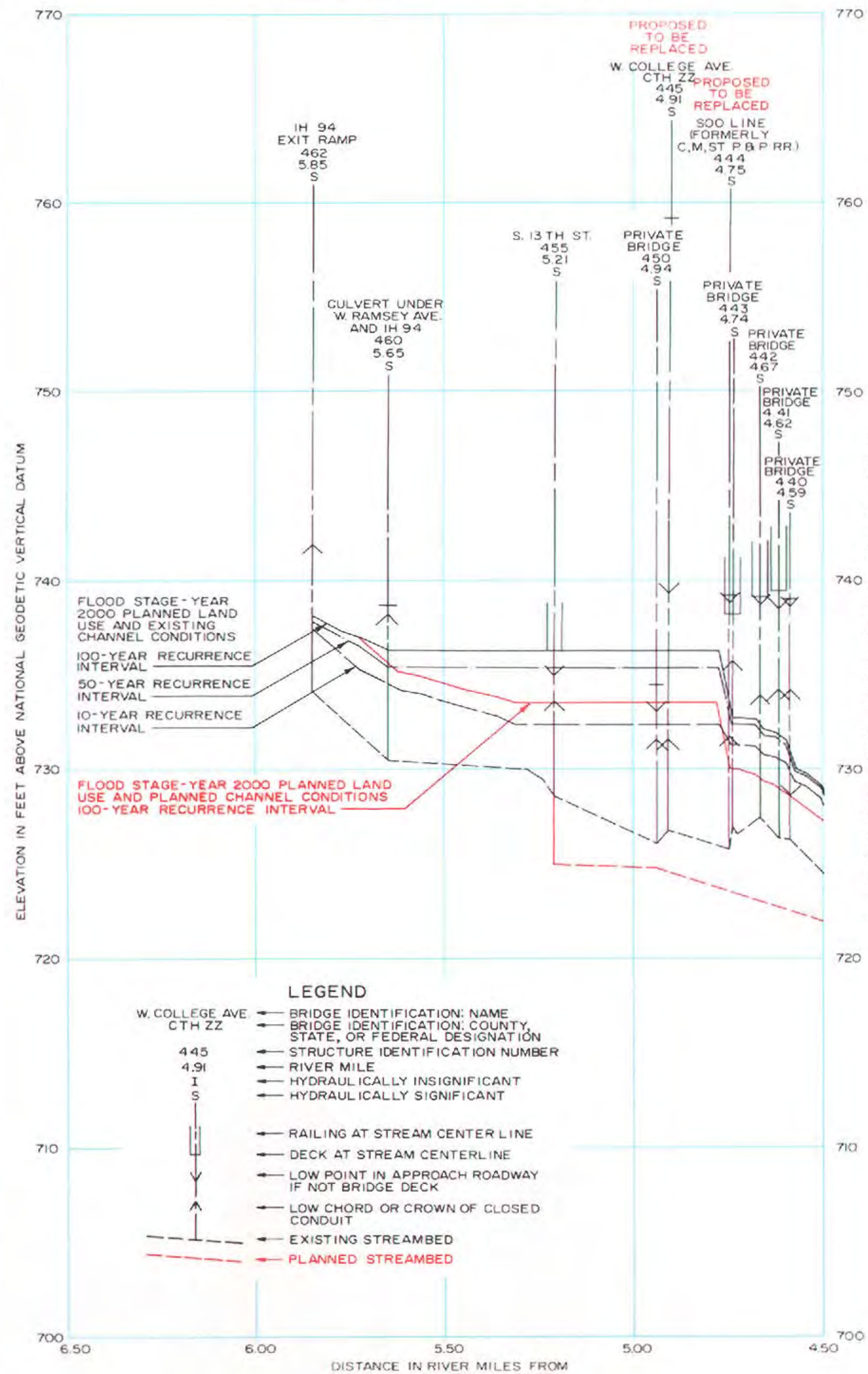


DATE OF PHOTOGRAPHY: APRIL 1986

Source: SEWRPC.

Figure 32 (continued)

NORTH BRANCH OF OAK CREEK



ment of all flood damages in the watershed caused by flood events up to and including the 100-year recurrence interval event under year 2000 planned land use conditions. Implementation of the recommended flood control plan would not, however, serve to eliminate local stormwater drainage problems in the watershed associated with storm sewers having outlets near the proposed channel bottom. Damage estimates relative to overland flooding of the stream system were not high enough to warrant the high cost of the major channel modifications which could alleviate these local stormwater drainage problems. Therefore, the abatement of local stormwater drainage problems should be addressed through the preparation of stormwater system plans. Preliminary indications are that there are no known areas of the watershed which could not be provided with adequate stormwater management facilities under the recommended channel and land use conditions. However, should more detailed stormwater management plans reveal certain stream segments where no technically or economically feasible alternatives exist to conveyance as a means of local stormwater runoff management, additional channel modifications may be indicated. Such channel modifications may be incorporated into the drainage and flood control system plan at a future date provided that it is demonstrated that: 1) there are indeed no feasible alternatives to the additional channelization; 2) the additional channelization would have no significant adverse impacts on downstream flood flows and stages; and 3) proper instream mitigation measures are provided, such as the use of turf-lined as opposed to concrete-lined channels.

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Chapter VI

EVALUATION OF ALTERNATIVE AND SELECTION OF RECOMMENDED FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN—ROOT RIVER WATERSHED

INTRODUCTION

The drainage and flood control policy plan companion to this system plan recommends that the Milwaukee Metropolitan Sewerage District assume jurisdiction for seven perennial and two intermittent streams in the Root River watershed. These nine streams, totaling 24.4 miles in length, consist of the North Branch of the Root River, East Branch of the Root River, Tess Corners Creek, Whitnall Park Creek, North Branch of Whitnall Park Creek, Northwest Branch of Whitnall Park Creek, Crayfish Creek, Caledonia Branch of Crayfish Creek, and an unnamed tributary to the Root River, the latter herein unofficially referred to as the 104th Street Branch. One stream not recommended for District jurisdiction—Hale Creek—was also included in the system planning effort because flood problems existing along that stream necessitate the implementation of flood control measures which may impact on flood flows and stages on, as well as flood control recommendations for, the Root River. Of the 10 streams thus studied under the system planning effort, all but three—the 104th Street Branch, Tess Corners Creek, and the East Branch of the Root River—were studied under previous Commission planning programs.[1] Each of these 10 streams is considered in the following sections of this chapter. Data are presented on existing and probable future drainage and flood control problems, alternative and recommended flood control and related drainage improvement measures, and recommended implementation actions.

[1] *See SEWRPC Planning Report No. 9, A Comprehensive Plan for the Root River Watershed, July 1966; SEWRPC Community Assistance Planning Report No. 121, A Stormwater Management Plan for the Village of Hales Corners, Milwaukee County, Wisconsin, March 1986; SEWRPC Memorandum Report No. 35, A Stormwater Management Plan for the Crayfish Creek Subwatershed, City of Oak Creek, Milwaukee County, Wisconsin, June 1988; and SEWRPC Letter Report to Milwaukee County Board of Supervisors concerning a reevaluation of the adopted Root River watershed plan for the Root River and Hale Creek in the City of West Allis, January 2, 1974.*

NORTH BRANCH OF THE ROOT RIVER SUBWATERSHED FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN

Hydrologic and hydraulic analyses were made and alternative flood control measures evaluated for the North Branch of the Root River in a comprehensive watershed plan prepared by the Commission in 1966.[2] The present system planning effort represents a refinement of that earlier study.

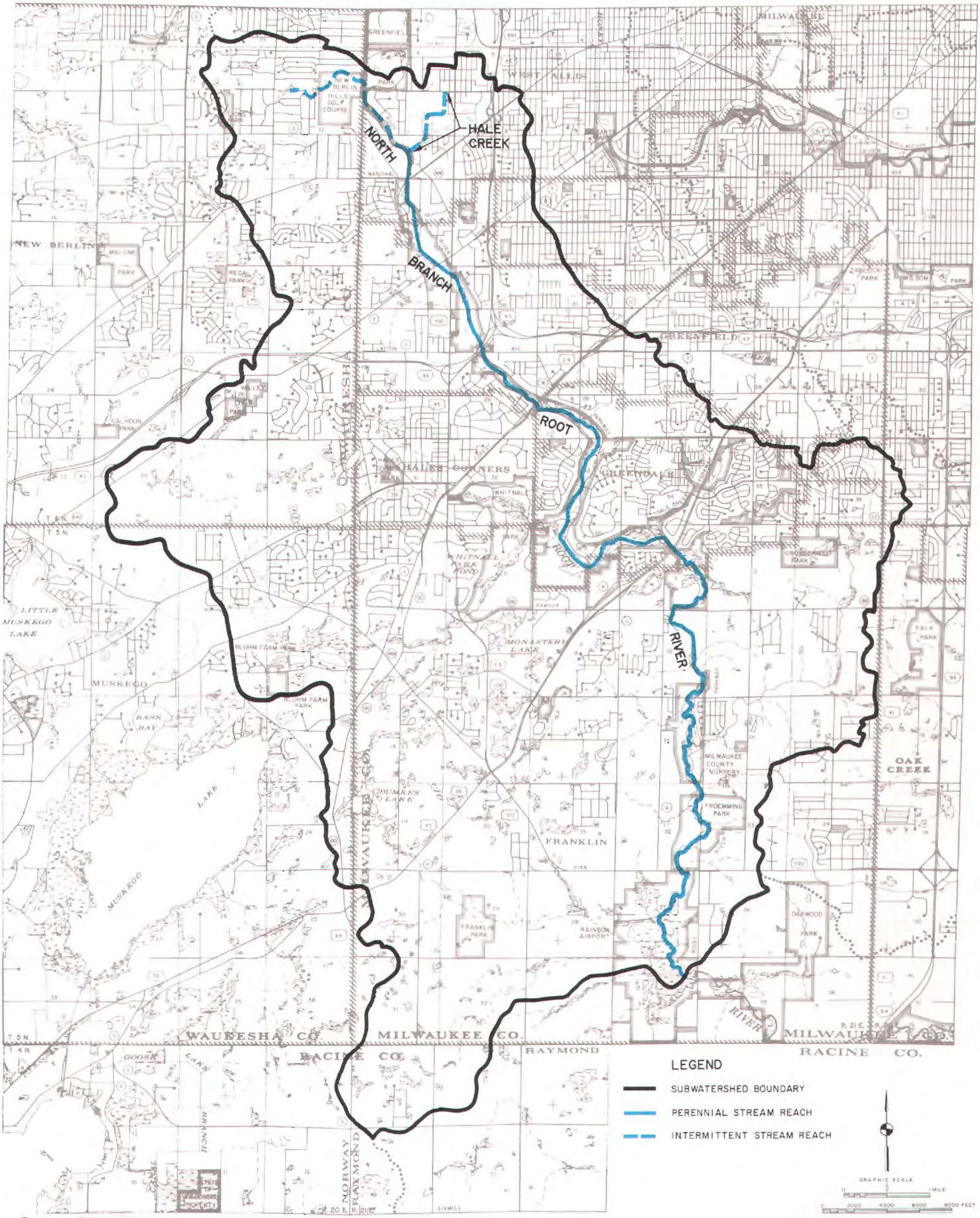
Overview of the Study Area

The North Branch of the Root River subwatershed is located largely within southwestern Milwaukee County, with small portions extending into southeastern Waukesha County and northern Racine County. The subwatershed includes all or portions of the Cities of Franklin, Greenfield, Milwaukee, Muskego, New Berlin, Oak Creek, and West Allis; the Villages of Greendale and Hales Corners; and the Towns of Norway and Raymond. From its origin near the intersection of Sunny Slope Road and Ferguson Road in the City of New Berlin, the North Branch of the Root River flows in a generally southerly direction for a distance of about 17.6 miles to the confluence with the Root River Canal in the City of Franklin to form the Root River. The North Branch of the Root River drains an area of about 60.92 square miles, as shown on Map 75. The extent of the subwatershed area within each minor civil division involved is given in Table 40.

[2] *See SEWRPC Planning Report No. 9, A Comprehensive Plan for the Root River Watershed, July 1966.*

Map 75

THE NORTH BRANCH ROOT RIVER SUBWATERSHED



Source: SEWRPC.

Table 40

AREAL EXTENT OF CIVIL DIVISIONS IN THE NORTH BRANCH ROOT RIVER SUBWATERSHED

County or Civil Division	County or Civil Division Area Included Within Subwatershed (square miles)	Percent of Subwatershed Area Within County or Civil Division
Milwaukee County		
City		
Franklin	26.92	44.3
Greenfield	6.22	10.2
Milwaukee	1.48	2.4
Oak Creek	0.05	0.1
West Allis	2.98	4.9
Village		
Greendale	5.45	8.9
Hales Corners	3.24	5.3
Racine County		
Town		
Norway	0.07	0.1
Raymond	1.29	2.1
Waukesha County		
City		
Muskego	3.84	6.3
New Berlin	9.38	15.4
Total	60.92	100.0

Source: SEWRPC.

More specifically, from its origin near the intersection of Sunny Slope and Ferguson Roads in the City of New Berlin, the North Branch of the Root River flows in an easterly direction to W. Lincoln Avenue in the City of West Allis, a distance of about 1.1 miles; thence southerly for about 2.4 miles to W. Beloit Road in the City of Greenfield; thence southeasterly for about 3.1 miles to W. Grange Avenue in the Village of Greendale; thence southerly for about 1.0 mile to W. College Avenue; thence southeasterly for about 3.3 miles to W. Rawson Avenue in the City of Franklin; and thence southerly for about 6.7 miles to its confluence with the Root River Canal to form the Root River. Of the 17.6-mile reach described, 15.6 miles, or 89 percent, are classified as perennial; the remaining 2.0 miles, or 11 percent, are classified as intermittent. The entire perennial stream length, as well as 0.9 mile of intermittent stream extending to W. Lincoln Avenue, is recommended for District jurisdiction in the policy plan companion to this system plan.

As already noted, hydrologic and hydraulic analyses were also made and flood control alternatives evaluated under this system planning effort for Hale Creek, a tributary to the North Branch of the Root River. A history of flood damage problems, as well as plans by the City of West Allis to widen and deepen this tributary, required that this stream be included in the system planning effort, since any flood control measures carried out along this stream could impact on flood flows and stages and recommended flood control measures along the North Branch of the Root River. Hale Creek is classified as an intermittent stream, with its origin at a storm sewer outfall near the intersection of S. 111th Street and W. Lincoln Avenue in the City of West Allis. From this outfall, Hale Creek flows in a generally southwesterly direction for about 1.0 mile to its confluence with the North Branch of the Root River.

In 1985, about 42 percent of the North Branch of the Root River subwatershed was developed for urban use, including residential, commercial, institutional, and urban open space uses. Most of the developed land, about 59 percent, is concentrated in the northern portion of the subwatershed in the Cities of Greenfield, Milwaukee, and West Allis, and the Villages of Greendale and Hales Corners. The developed areas of the subwatershed are generally provided with a full range of municipal street improvements, including paved streets with curbs and gutters and attendant storm sewers. Accordingly, surface runoff is generally conveyed quickly from most individual sites through storm sewers to the North Branch of the Root River.

Information on certain pertinent characteristics of the watershed, such as hydrologic soil types, land slopes, and land use, appears in Chapter II of this report. The planned land use conditions utilized in the system planning effort assume that the watershed will be about 80 percent urbanized by the design year of the system plan. The remaining rural lands are proposed to be located in the southern portion of the subwatershed in the City of Franklin and in Racine County.

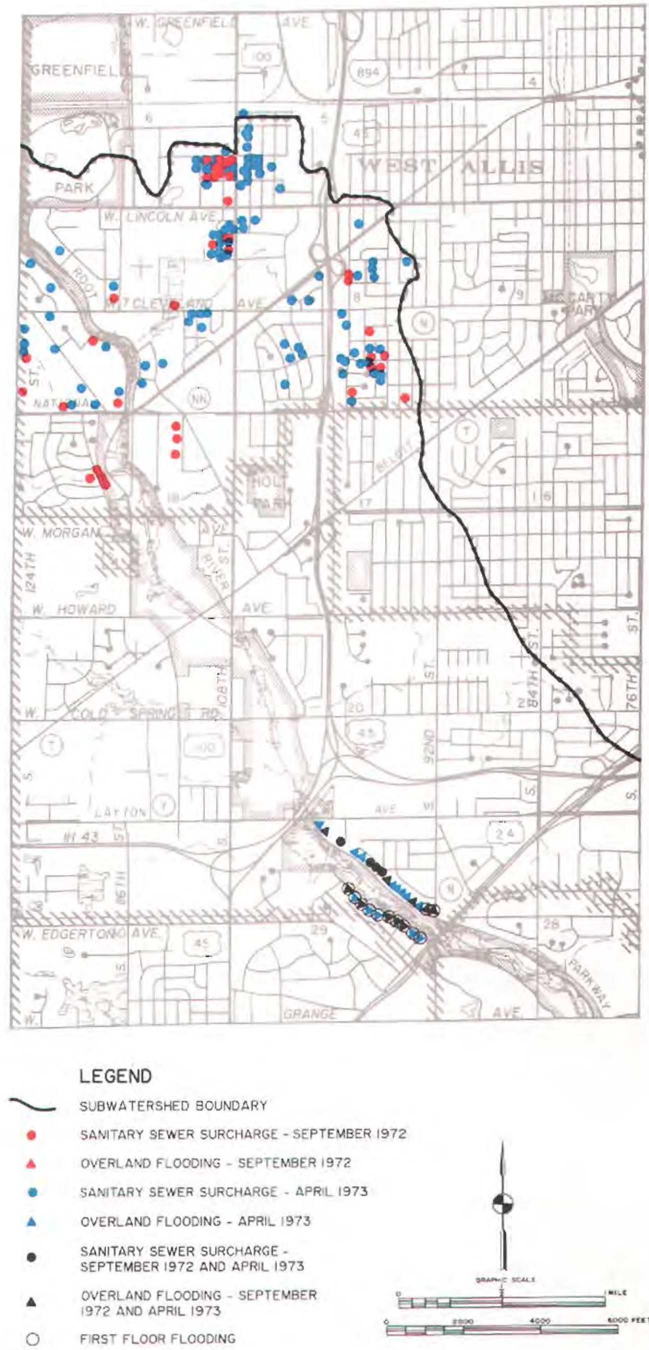
The North Branch of the Root River remains largely unimproved and is incorporated into a parkway along its entire 8.92-mile length between W. Lincoln Avenue in the City of West Allis and W. Rawson Avenue in the City of Franklin. Of the remaining 6.7 miles of stream, all but 1.2 miles flow through open park and institutional lands owned by Milwaukee County. In addition to providing a natural environment in an urban setting, these open parkway, park, and institutional lands provide temporary storage for floodwaters, thus reducing peak flood discharges.

Flooding and Related Drainage Problems

Flooding, in various degrees, is a common occurrence along the North Branch of the Root River. As indicated on Map 76, most of the structural flood damages are concentrated along two reaches: between W. Forest Home Avenue and W. Layton Avenue in the City of Greenfield; and between W. National Avenue and W. Lincoln Avenue in the City of West Allis. Both of these reaches are located in relatively narrow parkway lands and serve to illustrate the consequences of allowing urban development to take place too close to a major stream channel. Structure damages due to overland flooding have been more severe along the City of Greenfield reach, with several homes having experienced first-floor flooding. Flooding along the North Branch of the Root River in the City of West Allis is also common. Mr. Arthur D. Kastner, a citizen member of the Advisory Committee who lives along the Root River Parkway, stated that inundation of the Root River Parkway Drive north of W. National Avenue occurs an average of four times a year. Although such flooding is a nuisance to those people who rely on the Parkway Drive for access their homes, the majority of the monetary damages along this reach have been due to basement flooding caused by sanitary sewer backups. A survey conducted by the Regional Planning Commission after the April 21, 1973, flood indicated that, of 190 structures surveyed, 122 experienced basement flooding, while only one experienced first floor flooding. For 79, or 65 percent, of the 122 structures that experienced basement flooding, the flooding was attributed to sanitary sewer backup, with the flooding of the remainder being attributed to a combination of sanitary sewer backup, seepage through cracks in basement walls, and overland flow. Recent rehabilitation of the sanitary sewer system in the City of West Allis, including elimination of four lift station overflows and construction of the Root River interceptor sewer, has relieved the problem of sanitary sewer backups in the Root River watershed portion of West Allis.

Map 76

AREAS WITH REPORTED FLOODING AND DRAINAGE PROBLEMS IN THE NORTH BRANCH ROOT RIVER SUBWATERSHED



Source: SEWRPC.

Another stormwater drainage problem in the City of West Allis concerns the construction of storm sewers which have been designed to discharge to the North Branch of the Root River and Hale Creek with invert elevations below the existing streambeds. These sewers were constructed under the assumption that major channel modifications, including lowering of the streambed, would be carried out along these two stream reaches. These storm sewers operate with either partially blocked or negatively sloped outfalls, thus reducing their effective conveyance capacity and resulting in poor drainage and street and structure flooding in areas away from the stream channels. Frequent surcharging of the storm sewer system discharging to Hale Creek at W. Lincoln Avenue and S. 111th Street has been documented by the City of West Allis engineering department. The storm sewer outfall at this location consists of a reinforced concrete box culvert 6.5 feet wide by 4.0 feet high carrying runoff from about 170 acres of high-density residential development located north of W. Lincoln Avenue. Since this outfall was constructed with an invert elevation about two feet below the existing streambed, only one-half of its intended conveyance area is available. This has resulted in surcharging of the tributary storm sewers and flooding of residential streets several times a year. Investigations conducted under this system planning effort revealed six storm sewer outfalls—three on the North Branch of the Root River and three on Hale Creek—with inverts located below the existing streambed. These outfalls are shown on Map 77 and in Figure 33.

The costs of flooding were estimated using damage cost curves prepared by the Regional Planning Commission and described in Chapter III. The dollar amount of the flood damages is based upon the depth of inundation and the assessed valuation—required by law to approximate full market value—of the building. Damages to building contents are included in the total costs.

Flooding, as defined herein, includes basement flooding, yard inundation, and flooding above the first-floor level. The number of existing residences that may be expected to experience direct flooding along the North Branch of the Root River and Hale Creek is given in Table 41.

Additional homes and commercial properties may, however, experience indirect flood damages through sanitary sewer backup. It should be noted that the flood control measures considered under this system plan are primarily intended to alleviate flood damages from direct overland flooding along the stream studied, as well as to provide an adequate outlet for local storm sewers and drainageways. These measures, although not specifically designed to do so, may be expected to reduce damages due to localized stormwater drainage problems or sanitary sewer backups.

The total average annual flood losses, together with such losses anticipated under a 100-year recurrence interval flood event, are listed in Table 42 for the North Branch of the Root River and Hale Creek.

The drainage and flood control objectives and supporting principles and standards set forth in Chapter III specify the flood events which bridges shall accommodate without overtopping of the related roadway. Based on these criteria, 14 bridges on the North Branch of the Root River and one bridge on Hale Creek are considered hydraulically inadequate, as shown in Appendix E. These bridges are at W. Drexel Avenue; W. Rawson Avenue; S. 76th Street; W. Layton Avenue; W. Cold Spring Road; S. 108th Street; W. Beloit Road; W. Morgan Avenue; S. 116th Street; W. Oklahoma Avenue; W. Cleveland Avenue; the Root River Parkway drives at River Miles 33.96, 37.39, and 41.95 on the North Branch of the Root River; and W. Cleveland Avenue on Hale Creek.

Flood Discharges and Stages

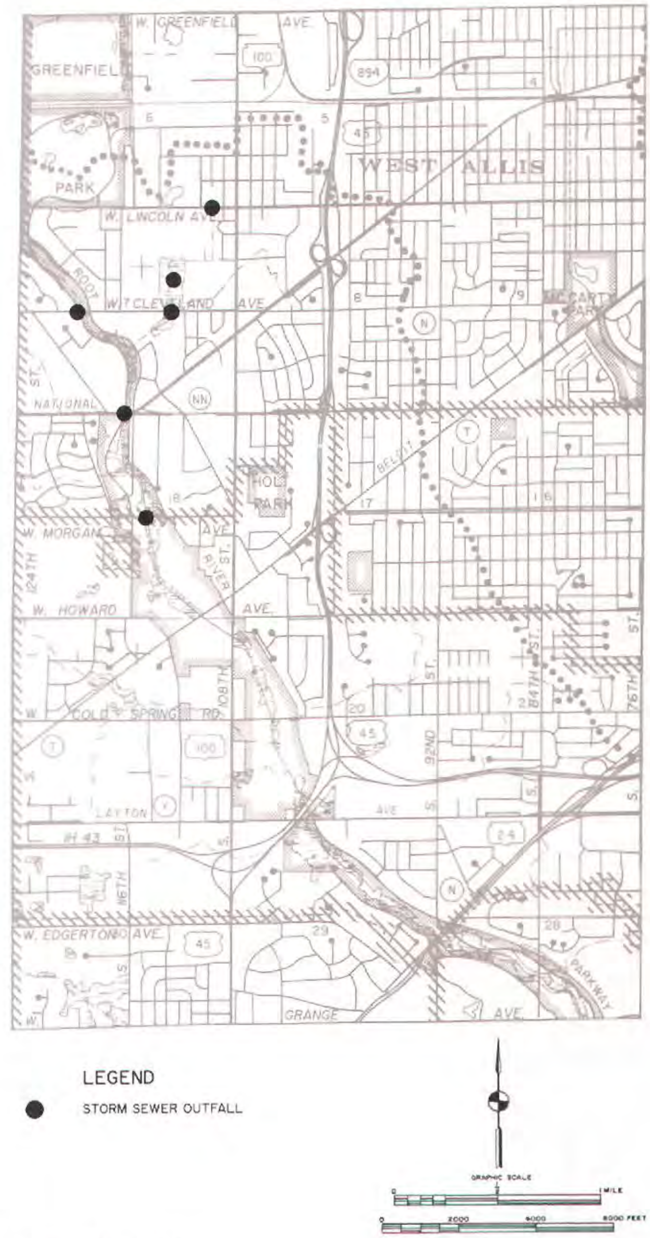
As noted in Chapter III of this report, the hydrologic model used for development of design discharges for the North Branch of the Root River simulates streamflow on a continuous basis, using recorded climatological data as input. Using this model, stream discharges were computed at an hourly time interval. Peak flood discharges were developed by performing discharge-frequency analyses using the log Pearson Type III method of analysis of simulated annual peak discharges generated by the hydrologic model. Because of the urbanized nature of the northern portion of the subwatershed, and its attendant rapid conveyance of runoff through storm sewers, it was suspected that the time of peak discharge on the stream was very short and may have been missed in the analysis utilizing an hourly time interval. Additional simulations were performed, therefore, using a 15-minute time interval and design rainfall events as input. The use of design rainfall events was necessary because the time and cost of simulating continuous streamflow at 15-minute intervals for the 39 years of available climatological data would be prohibitive.

The design rainfall events were developed using 10-, 50-, and 100-year rainfall volumes obtained from the point rainfall depth-duration-frequency relationships developed by the Commission as described in Chapter III. The rainfall distribution utilized for each design storm was the median distribution of a first-quartile storm, as shown in Chapter III. The design storm duration was determined for a given recurrence interval by simulating the peak discharge at a given location for a range of storm durations. The storm duration and associated rainfall volume which produced the largest peak discharge at a given location for a given recurrence interval was selected as the design storm for that location. This analysis was conducted for both existing and planned land use and existing channel conditions at 11 locations along the North Branch of the Root River and at two locations along Hale Creek. The estimated peak flood discharges under existing and planned, year 2000 land use conditions and existing channel conditions are set forth in Table 43. The reduction in flows in a downstream direction is due to storage effects in the Root River Parkway.

A comparison of peak flood discharges developed under this system planning effort and those developed under the Commission's Root River watershed study is provided in Table 44. As shown in this table, the revised flows are generally higher. The differences are due, in part, to the use of different simulation models for the development of flood discharges. As previously noted, the HydroComp hydrologic model was used to develop flows under this system planning effort, while the U. S. Soil Conservation Service TR 20 hydrologic model was used to develop flows under the Root River watershed study. Although the use of different, properly calibrated models should not produce significant—greater than 20 percent—variations in calculated streamflows, some variation is to be expected. Very little recorded streamflow data were available for hydrologic model calibration on the North Branch of the Root River at the time of the Root River watershed study. A continuous recorder stage gage operated by the U. S. Geological Survey in cooperation with the Commission and the Milwaukee Metropolitan Sewerage District was installed on the North Branch of the Root River at W. Ryan Road in 1963. Thus, only two years of recorded streamflow data were available for model calibration purposes at the time of the Root River watershed study. Therefore, that study had to rely heavily on recorded stage data available at Spring Street in the City of Racine, about 26 miles downstream of the North Branch of the Root River,

Map 77

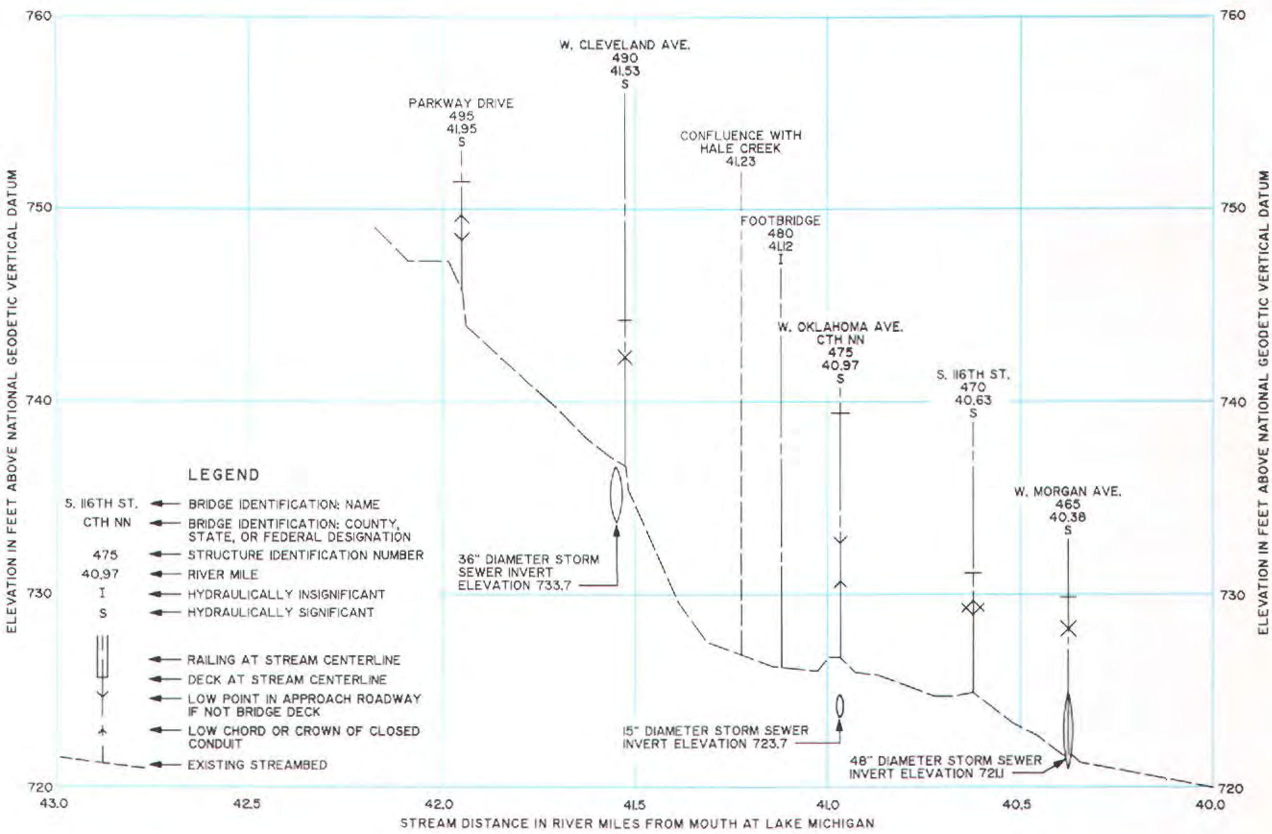
LOCATIONS OF STORM SEWER OUTFALLS WITH INVERT ELEVATIONS BELOW EXISTING STREAMBED

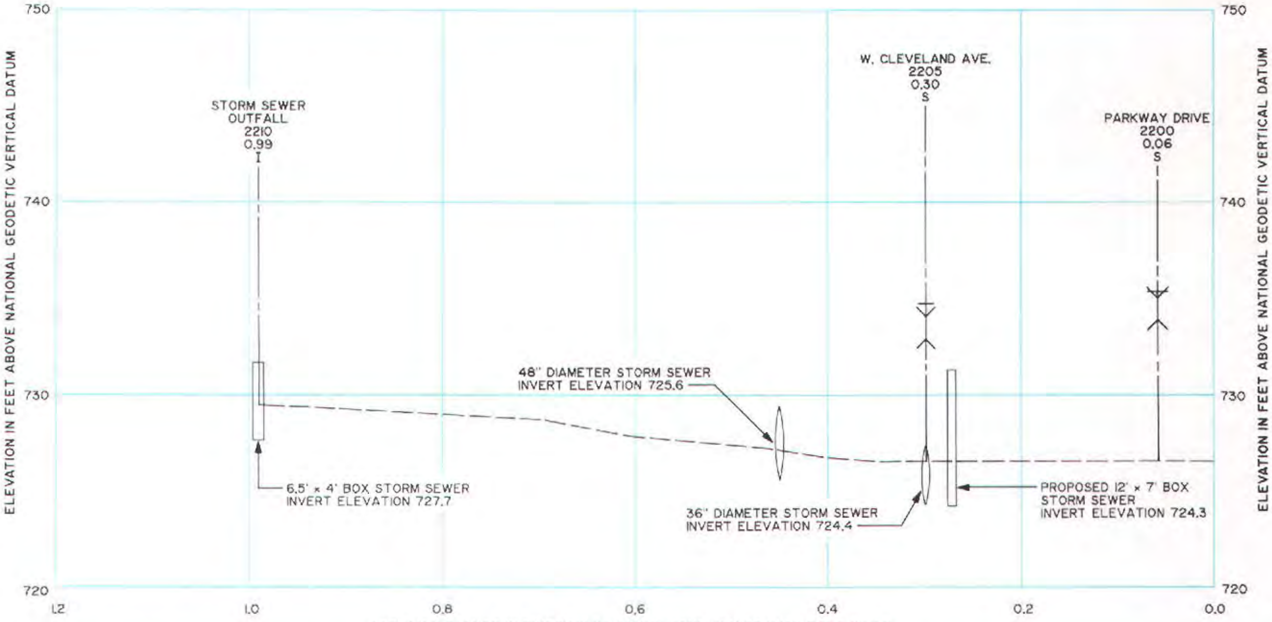


Source: SEWRPC.

Figure 33

STORM SEWER OUTFALLS LOCATED BELOW THE EXISTING STREAMBED ALONG THE NORTH BRANCH ROOT RIVER AND HALE CREEK IN THE CITY OF WEST ALLIS





Source: SEWRPC.

Table 41

STRUCTURE FLOODING ALONG THE NORTH BRANCH ROOT RIVER AND HALE CREEK

Stream	Community	Recurrence Interval (years)	Approximate Number of Existing Homes Flooded		Approximate Number of Existing Industrial and Commercial Properties Flooded	
			Existing Land Use Existing Channel	Planned Land Use Existing Channel	Existing Land Use Existing Channel	Planned Land Use Existing Channel
North Branch of the Root River	Franklin	10 50 100	1 2 2	1 2 2	2 2 2	2 2 2
	Greenfield	10 50 100	10 35 40	17 39 43	0 0 0	0 0 0
	West Allis	10 50 100	0 6 10	3 8 13	0 0 0	0 0 0
Hale Creek	West Allis	10 50 100	3 7 9	5 9 9	0 8 9	0 9 9

Source: SEWRPC.

Table 42

ESTIMATED FLOOD DAMAGES ALONG THE NORTH BRANCH ROOT RIVER AND HALE CREEK

Stream	Community	Average Annual Flood Damage		100-Year Recurrence Interval Flood Damage	
		Existing Land Use Existing Channel	Planned Land Use Existing Channel	Existing Land Use Existing Channel	Planned Land Use Existing Channel
North Branch of the Root River	Franklin	$ 7,600	$ 9,600	$ 63,000	$ 65,000
	Greenfield	25,000	45,000	395,000	465,000
	West Allis	3,000	5,500	76,000	95,000
Hale Creek	West Allis	$22,000	$26,000	$ 595,000	$ 601,000
Total	- -	$57,600	$86,100	$1,129,000	$1,226,000

Source: SEWRPC.

Table 43

FLOOD DISCHARGES FOR EXISTING AND YEAR 2000 LAND USE AND EXISTING CHANNEL CONDITIONS

Location	River Mile	Peak Flood Discharge (cubic feet per second) Existing Land Use Existing Channel Conditions 10-Year	50-Year	100-Year	Year 2000 Planned Land Use, Existing Channel Conditions 10-Year	50-Year	100-Year
North Branch of the Root River							
Confluence with Root River Canal	25.66	2,650	4,200	4,800	2,850	4,450	4,900
Upstream of W. Oakwood Road	26.69	2,400	3,800	4,500	2,600	4,000	4,650
Upstream of W. Ryan Road	28.07	2,400	3,900	4,600	2,600	4,100	4,800
Upstream of Confluence with East Branch of the Root River	30.15	2,500	4,100	4,950	2,650	4,250	5,100
Upstream of W. Drexel Avenue	30.94	2,600	4,200	5,000	2,800	4,300	5,150
W. Rawson Avenue	32.37	2,700	4,400	5,300	2,900	4,600	5,450
Upstream of Confluence with Tess Corners Creek	35.58	1,740	2,700	3,160	1,950	2,910	3,350
W. Forest Home Avenue	37.67	1,720	3,240	3,830	2,160	3,720	4,280
W. Cold Spring Road	39.17	1,570	2,790	3,230	1,800	3,000	3,500
W. National Avenue	40.97	970	1,570	1,850	1,050	1,780	2,000
Upstream of Confluence with Hale Creek	41.25	580	1,090	1,230	710	1,260	1,410
Hale Creek							
Mouth at North Branch of Root River	0.00	810	1,250	1,430	940	1,420	1,520
Upstream of W. Cleveland Avenue	0.31	300	530	580	300	530	580

Source: SEWRPC.

for model calibration. Twenty-three years of streamflow data recorded at the W. Ryan Road gage were available for model calibration purposes under this system planning effort. Finally, the most significant reason for these differences was the simulation under this study of discharges utilizing a 15-minute time interval. Use of this shorter time interval—compared to use of a one-hour interval—resulted in a more accurate representation of the runoff hydrograph and generally higher peak discharges. Discharges used in the Root River watershed plan were developed using a one-hour time interval, as was done under the Root River watershed study.

Table 44

COMPARISON OF PLANNED LAND USE AND EXISTING CHANNEL CONDITION FLOOD FLOWS: ROOT RIVER WATERSHED STUDY AND MMSD SYSTEM PLAN

Location	River Mile	100-Year Recurrence Interval Flood Discharge (cubic feet per second)		Percent Change
		Root River Watershed Study	MMSD System Plan	
Upstream of W. Ryan Road	28.07	6,500	4,800	-26
W. Drexel Avenue	30.89	5,370	5,150	-4
W. Rawson Avenue	32.37	5,135	5,450	6
W. College Avenue	35.66	2,930	3,350	14
W. Forest Home Avenue	37.67	3,660	4,280	17
W. Cold Spring Road	39.17	3,075	3,500	14
W. Morgan Avenue	40.38	3,010	3,500	16
W. National Avenue	40.97	2,050[a]	2,000	-2

[a]A value of 2,975 cfs for this location is listed in the Commission's Root River watershed study but includes runoff from the West Branch of the Root River. At the time of that study this tributary discharged to the North Branch of the Root River upstream of W. National Avenue. As part of the reconstruction of W. National Avenue, that tributary was relocated so that it now discharges downstream of W. National Avenue. Accordingly, the flow was adjusted to provide a valid comparison.

Source: SEWRPC.

Nevertheless, the flows developed under the two studies compare well, falling with but one exception within less than 17 percent of each other.

Flood stage profiles were developed for the 10-, 50-, and 100-year recurrence interval runoff events under planned land use and existing channel conditions. These profiles, which encompass the full 16.5-mile-long reach of the North Branch of the Root River recommended for District jurisdiction and the 0.99-mile-long reach of Hale Creek, constitute a graphic representation of the flood stages under the specified recurrence interval flood discharges, and under planned land use and existing channel conditions. In addition to providing an overall representation of flood stages relative to familiar points of reference, such as the channel bottom and bridge deck surfaces, the profiles, because of their continuity, permit the determination of flood stages at any point along the stream channel. The flood profile for the North Branch of the Root River is shown in Figure 34, and the flood profile for Hale Creek is shown in Figure 35. The extent of the 100-year recurrence interval floodplain under planned land use conditions is shown on Map 78 for both the North Branch of the Root River and Hale Creek.

Overview of Previous Commission-Developed Alternative Flood Control and Related Drainage System Plans for the North Branch of the Root River and Hale Creek in the City of West Allis

The Regional Planning Commission had previously evaluated flood control alternatives for the North Branch of the Root River and Hale Creek in the City of West Allis, with the findings of that study presented in a January 2, 1974, letter report from the Commission to the Milwaukee County Board of Supervisors.[3] Eight flood control alternatives were developed under that previous study: 1) no action; 2) major channelization; 3) structure floodproofing; 4) earthen dikes; 5) selected bridge replacement; 6) minor channel clearing, deepening, widening, and shaping; 7) floodwater storage; and 8) major channelization-structure floodproofing composite. Of these eight alternatives, three were recommended for further consideration: 1) the major channelization alternative; 2) the structure floodproofing alternative; and 3) the major channelization-structure floodproofing composite alternative. The other five alternatives were eliminated from further consideration for various reasons. The no action alternative would do nothing to eliminate structure flood damages. The selected bridge replacement alternative and the minor channel clearing, deepening, widening, and shaping alternative would also do little to reduce structure flood damages. The floodwater storage alternative was eliminated from further consideration since it, too, would entail residual structure flood damages. Finally, the earthen dikes alternative was eliminated because the dikes would have a severe adverse impact on the aesthetic appearance of the Root River Parkway. The three flood control alternatives recommended for further consideration were found to have similar benefit-cost ratios. All three of these alternatives would eliminate anticipated structure damages from floods up to and including the 100-year recurrence interval event.

The flood control alternatives evaluated under this system planning effort represent a refinement of the three alternatives recommended for consideration under the previous study.

<u>Alternative Flood Control and Related Drainage System Plans for the Root River and Hale Creek in the City of West Allis</u>

Four alternative flood control plans were considered and evaluated for alleviating the flood damage problems along the Root River and Hale Creek in the City of West Allis: Alternative Plan 1—No Action; Alternative Plan 2—Structure Floodproofing, Elevation, and Removal with Stormwater Pumping; Alternative Plan 3—Structure Floodproofing, Elevation, and Removal with Minor Channel Deepening; and Alternative Plan 4—Major Channel Modification.

Both a detention storage and a diking alternative were considered but were not evaluated in detail under this system planning effort. Preliminary investigation of possible detention reservoir sites indicated only one available site remaining in the City of West Allis, that being along Hale Creek between W. Cleveland and W. Lincoln Avenues, east of Nathan Hale High School, as shown on Map 79. This site had also been evaluated under the aforenoted Commission flood control study for the City of West Allis. Urban development which has occurred in this area since 1974 has further reduced the floodwater storage capacity to about 110 acre-feet. An estimated 390 acre-feet of storage would be required to significantly reduce downstream flood damages in the City of West Allis. This alternative was therefore eliminated from further consideration.

Preliminary investigations of a diking alternative revealed that about 6,900 feet of earthen dikes ranging in height from four to eight feet, and about 800 feet of concrete floodwall ranging in height from seven to eight feet, would be required along the Root River main stem, at an estimated cost of $1,073,000. An additional 7,000 feet of earthen dikes ranging in height from five feet to seven feet and about 900 feet of concrete floodwalls ranging in height from six to seven feet would be required along Hale Creek, at an estimated cost of $1,078,000. Stormwater drainage facilities, including stormwater pumping stations, which would be required to adequately convey runoff from behind these dikes and floodwalls, as well as to relieve those storm sewers with outfalls below the existing channel bottom, would cost in excess of $5,000,000. Due to the severe adverse aesthetic impact of dikes and floodwalls up to eight feet in height along these streams, as well as to the relatively high cost, this alternative was also eliminated from further consideration.

Each of the four alternatives evaluated further is described below. The estimated economic benefits and costs attendant to each alternative are provided in Table 45.

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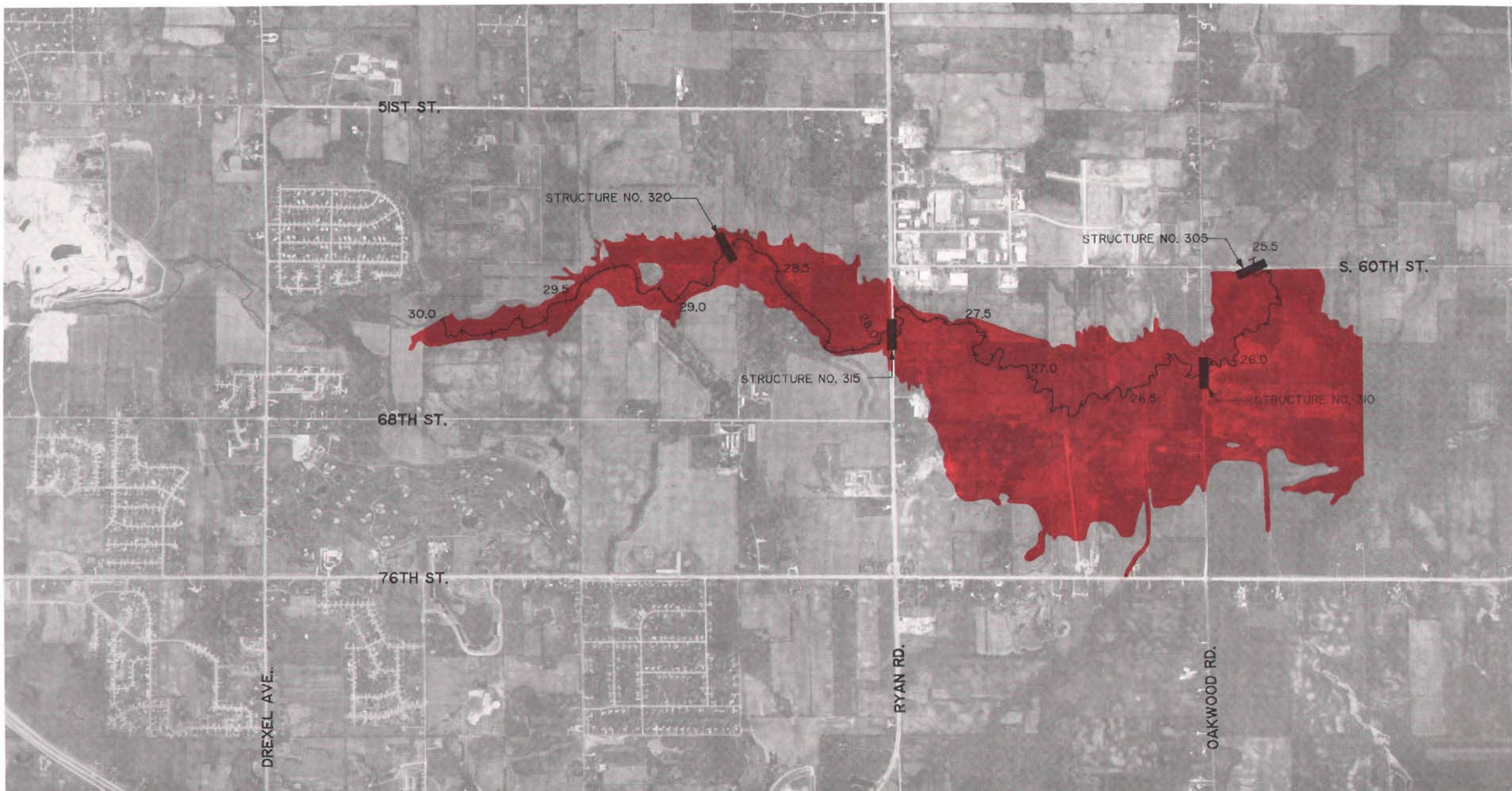
[3]*See SEWRPC Letter Report to Milwaukee County Board of Supervisors concerning a reevaluation of the adopted Root River watershed plan for the Root River and Hale Creek in the City of West Allis, January 2, 1974.*

Map 78

100-YEAR RECURRENCE INTERVAL FLOODPLAIN FOR THE NORTH BRANCH ROOT RIVER AND HALE CREEK UNDER YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

30.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR NORTH BRANCH ROOT RIVER IS SHOWN IN APPENDIX H

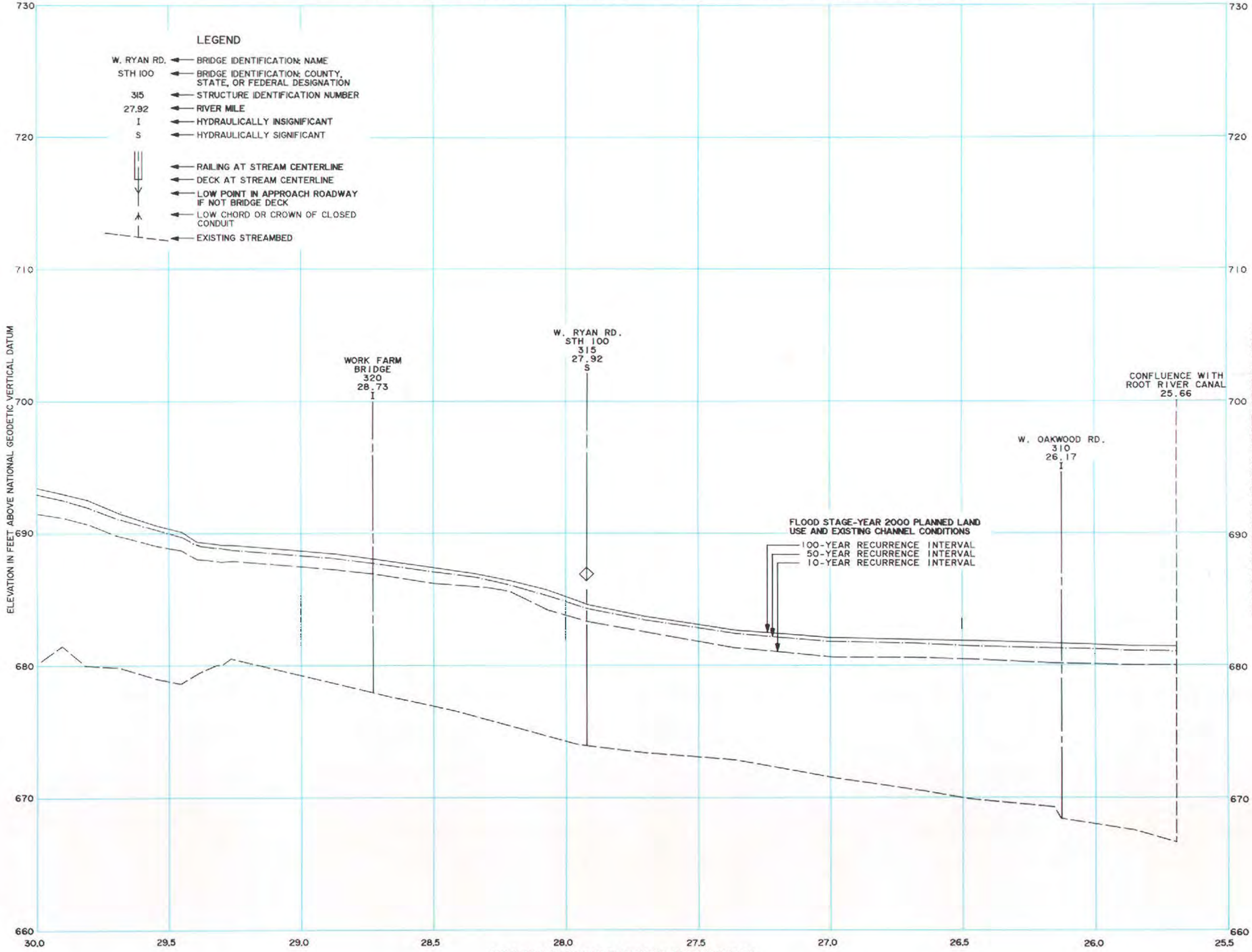
GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Figure 34

FLOOD STAGE AND STREAMBED PROFILE FOR THE NORTH BRANCH ROOT RIVER



Map 78 (continued)

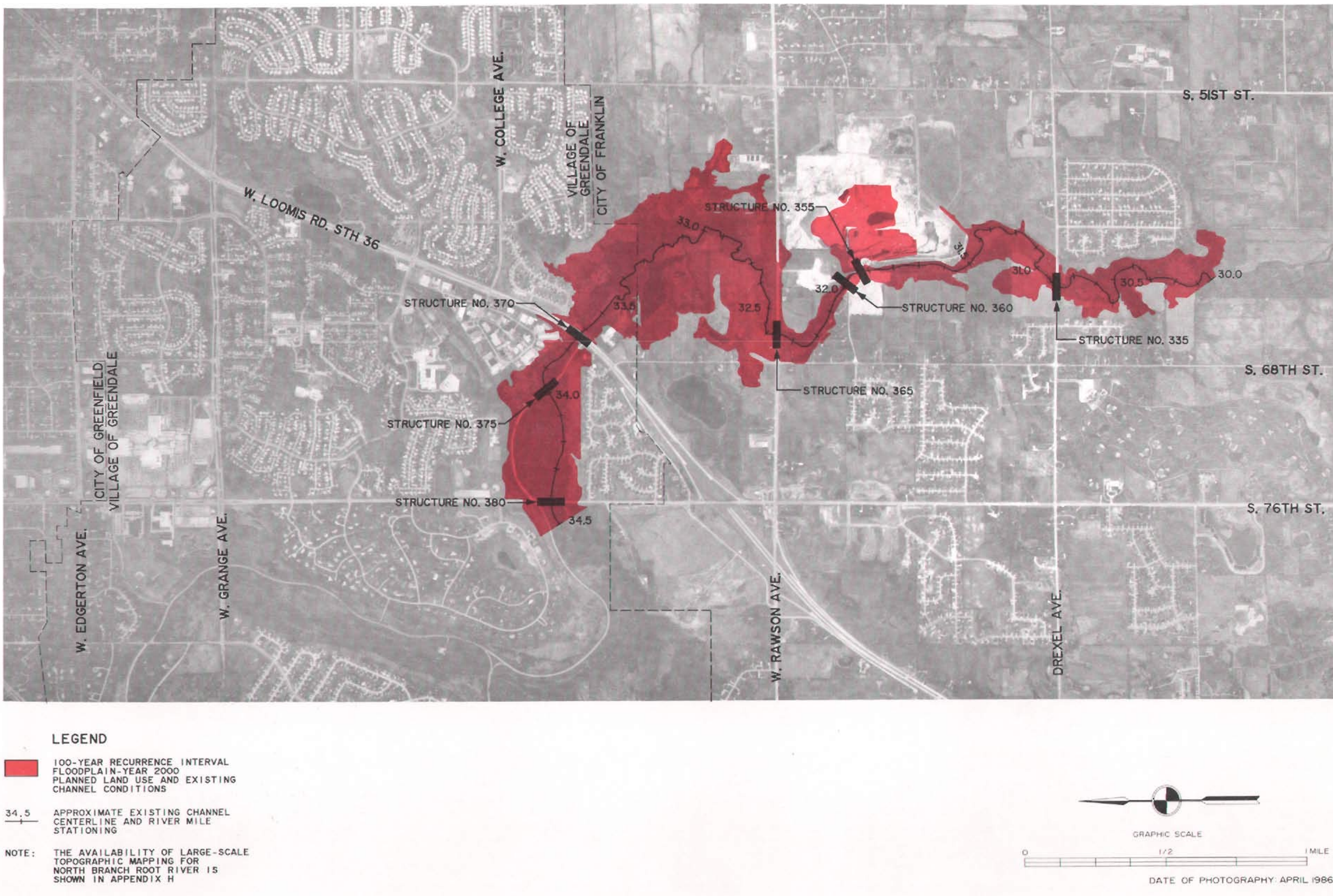
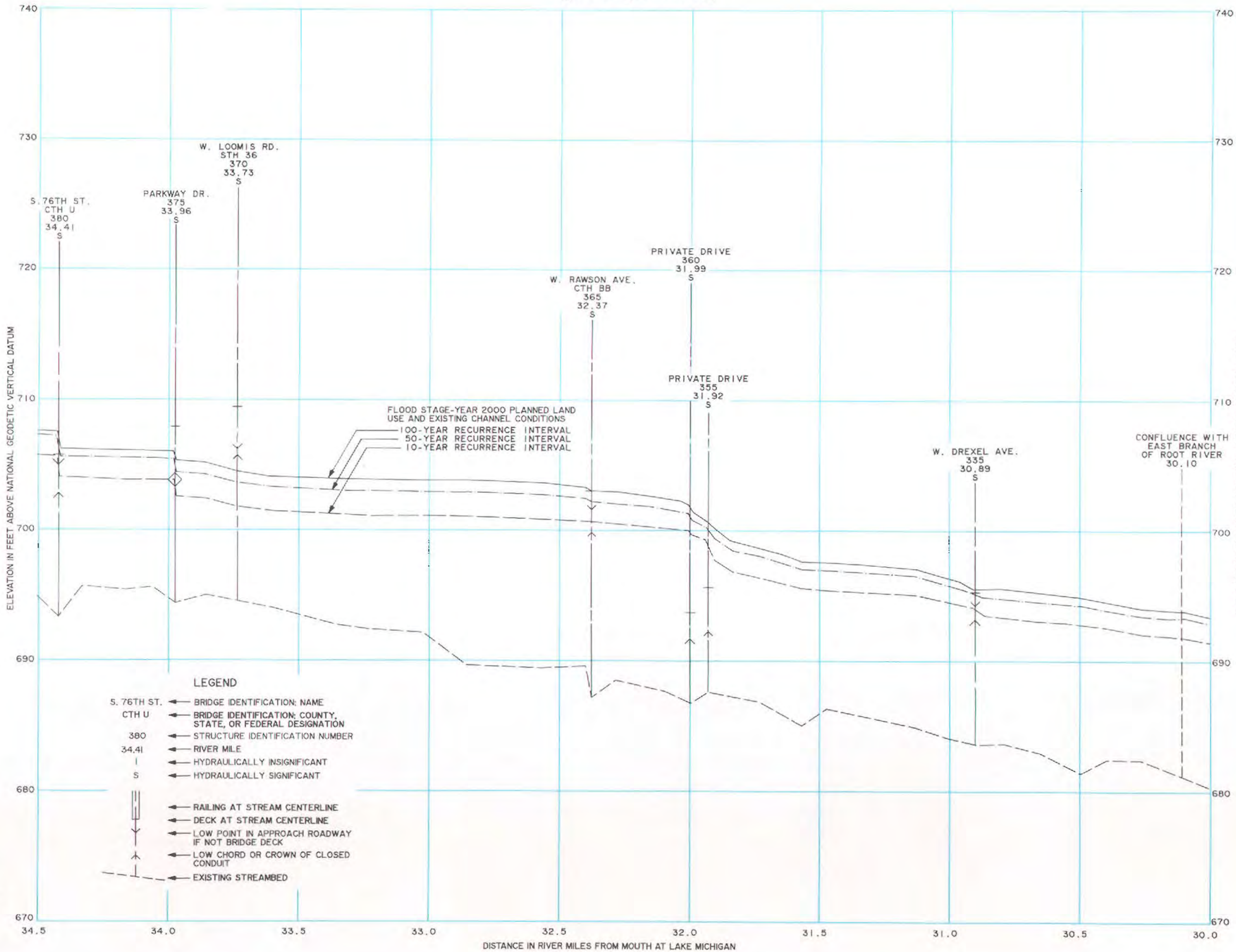
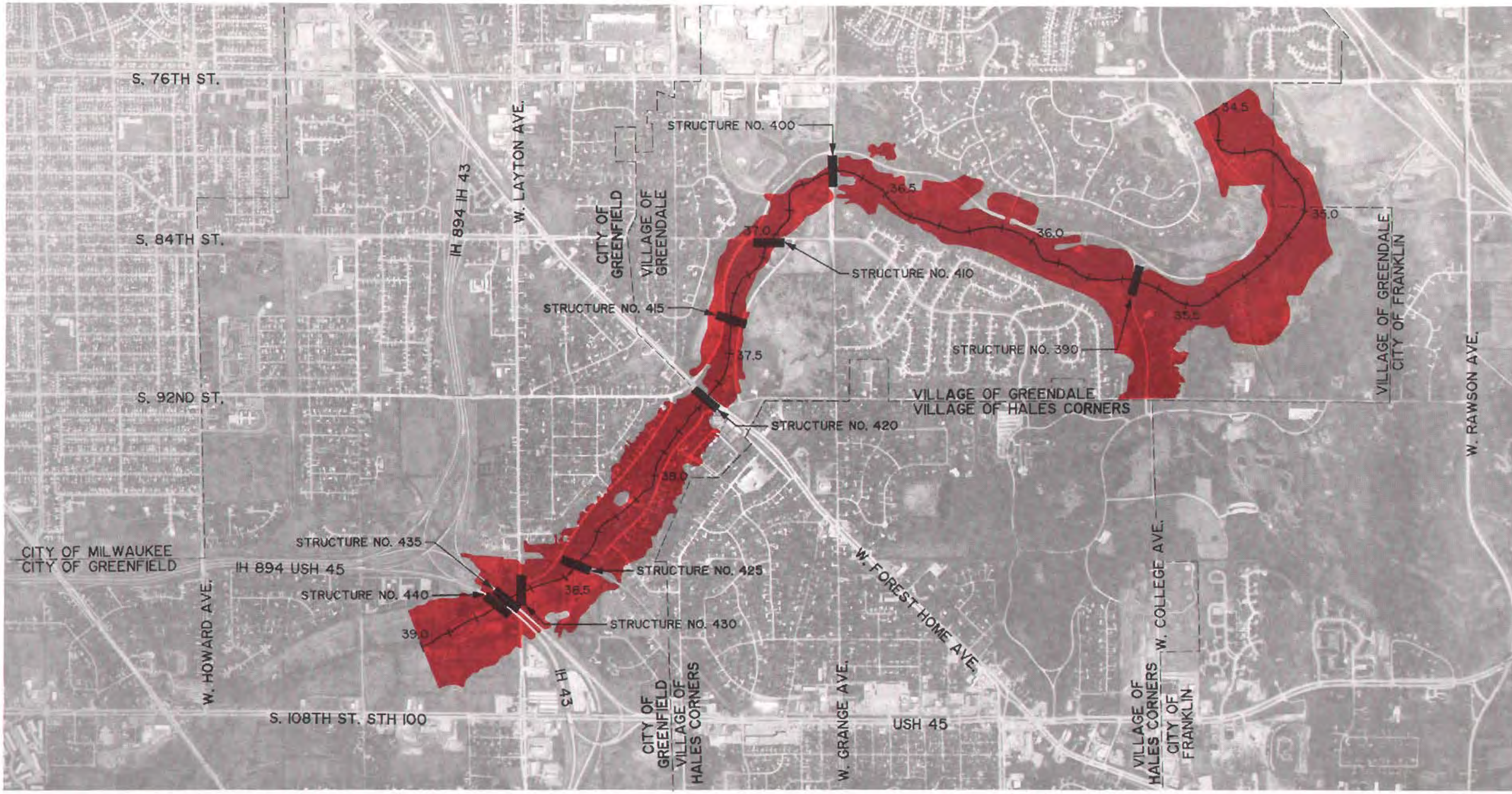


Figure 34 (continued)



Map 78 (continued)



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

39.0 — APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

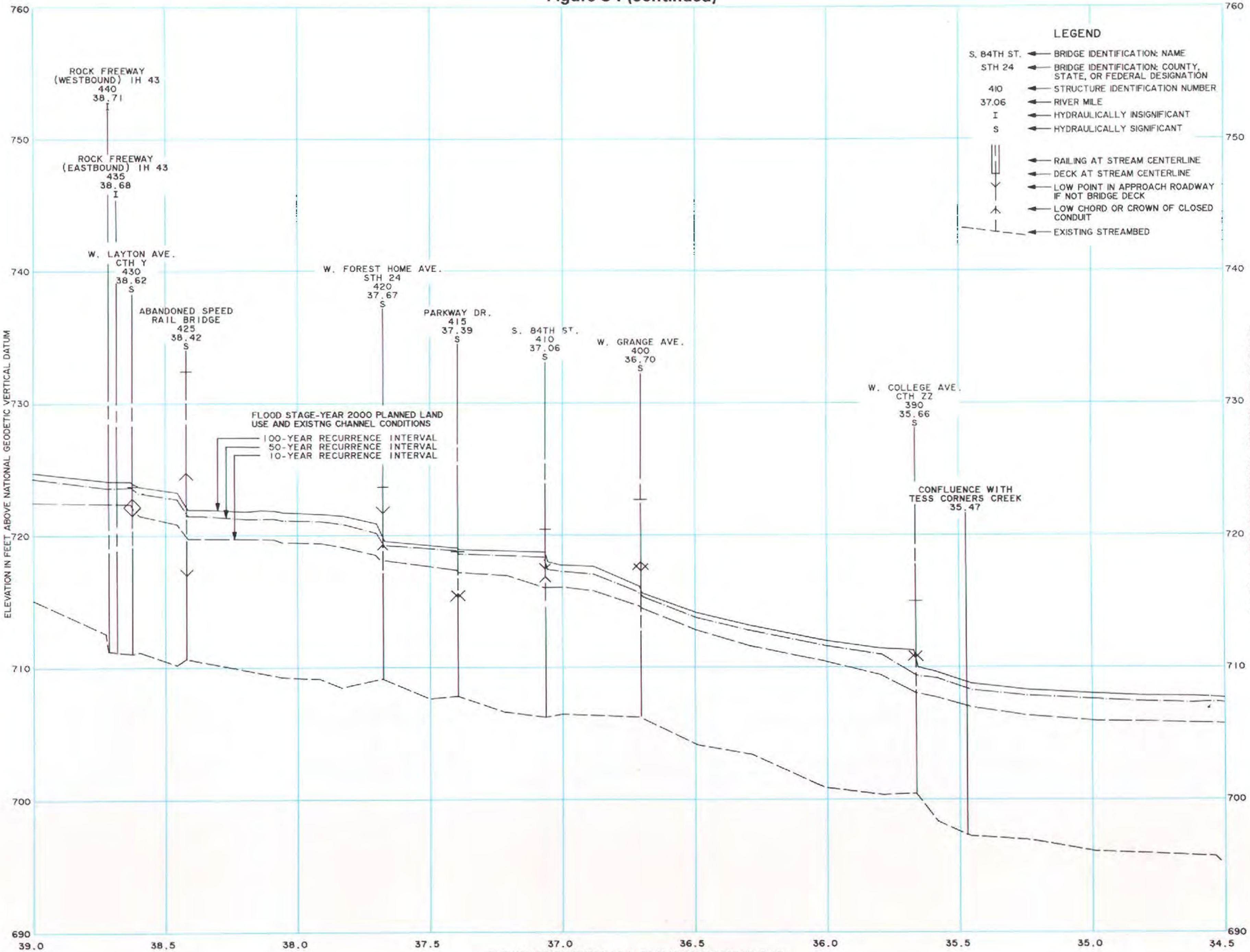
NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR NORTH BRANCH ROOT RIVER IS SHOWN IN APPENDIX H

GRAPHIC SCALE

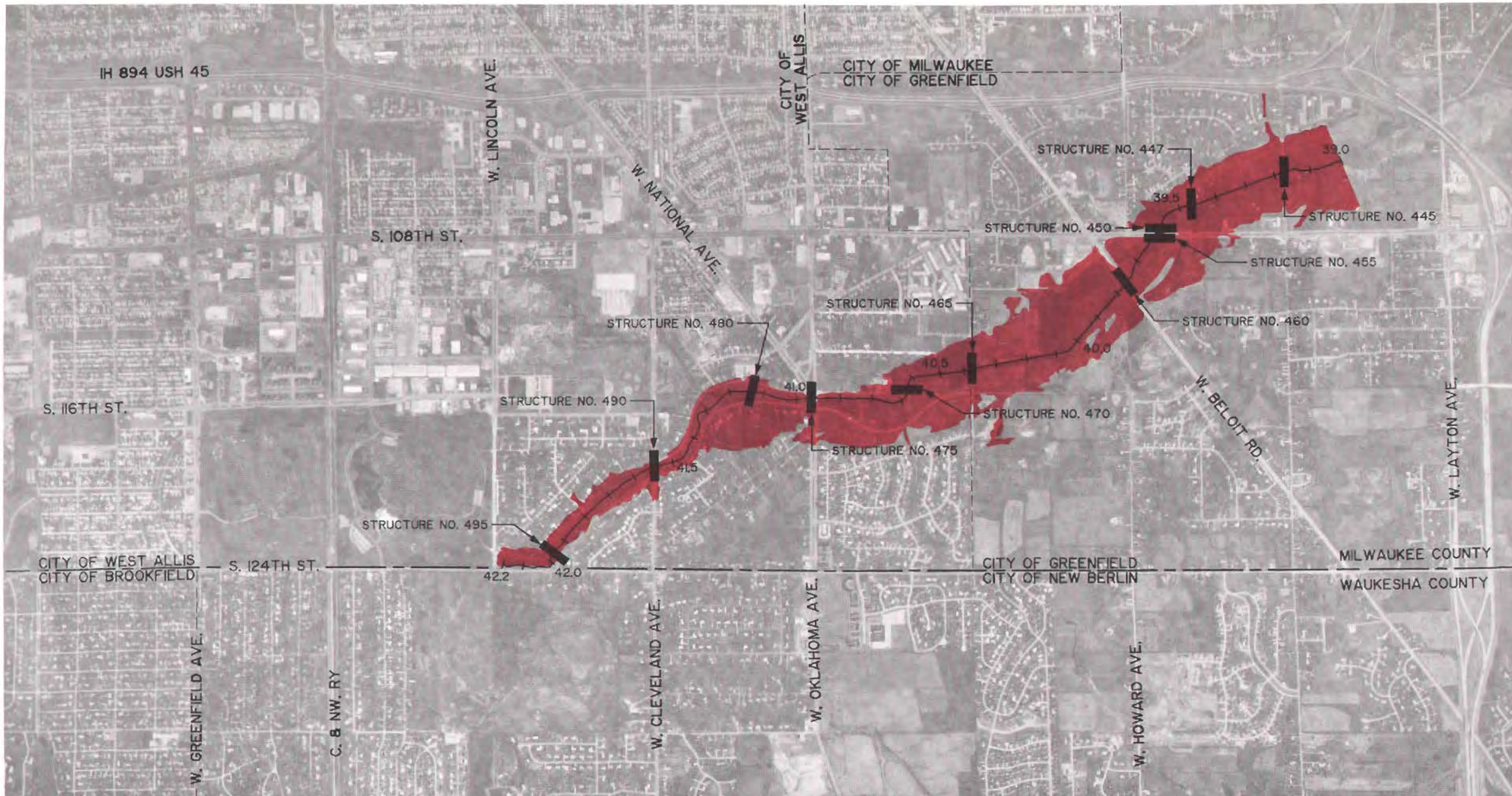
0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Figure 34 (continued)



Map 78 (continued)



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

42.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

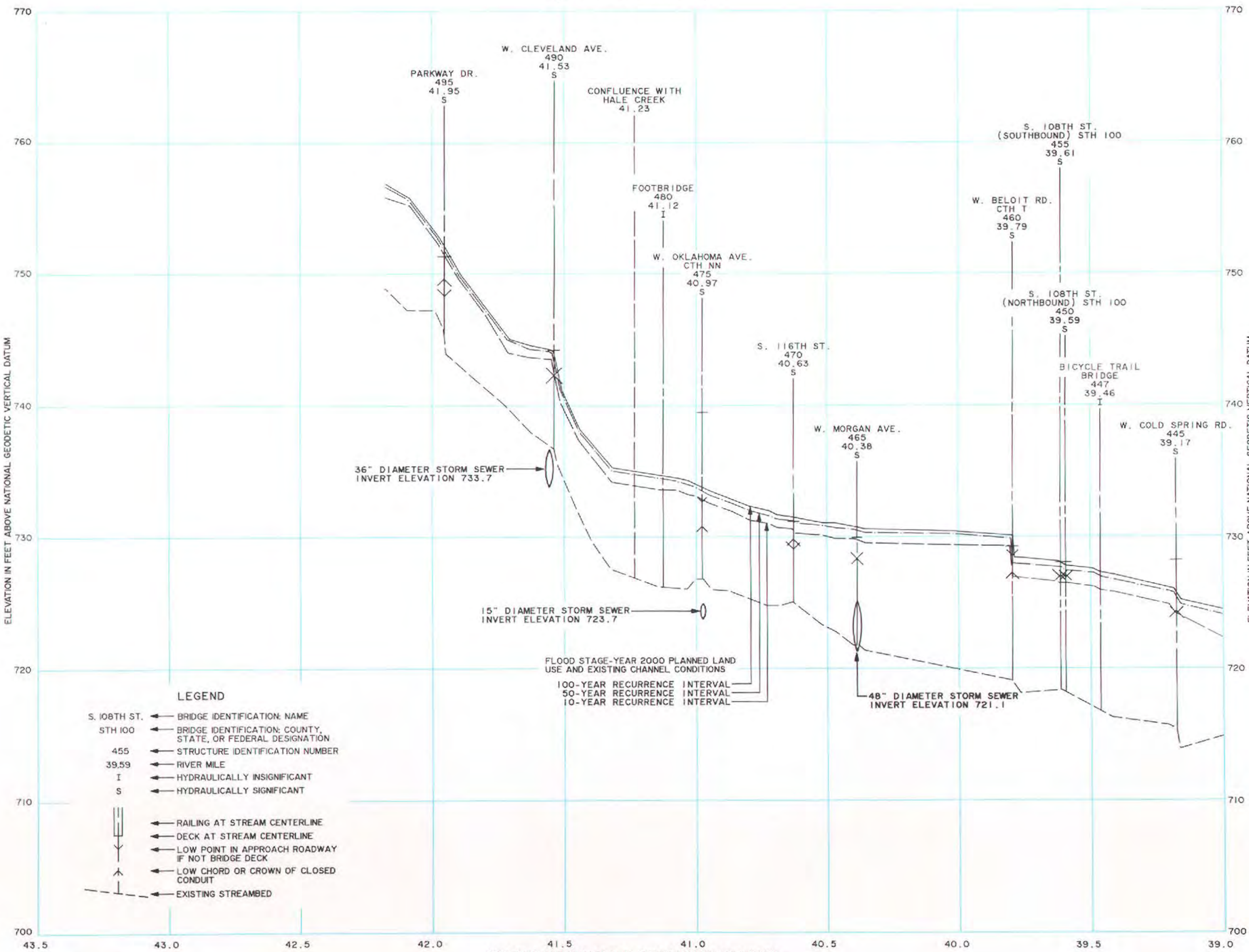
NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR NORTH BRANCH ROOT RIVER IS SHOWN IN APPENDIX H

GRAPHIC SCALE

0 1/2 1 MILE

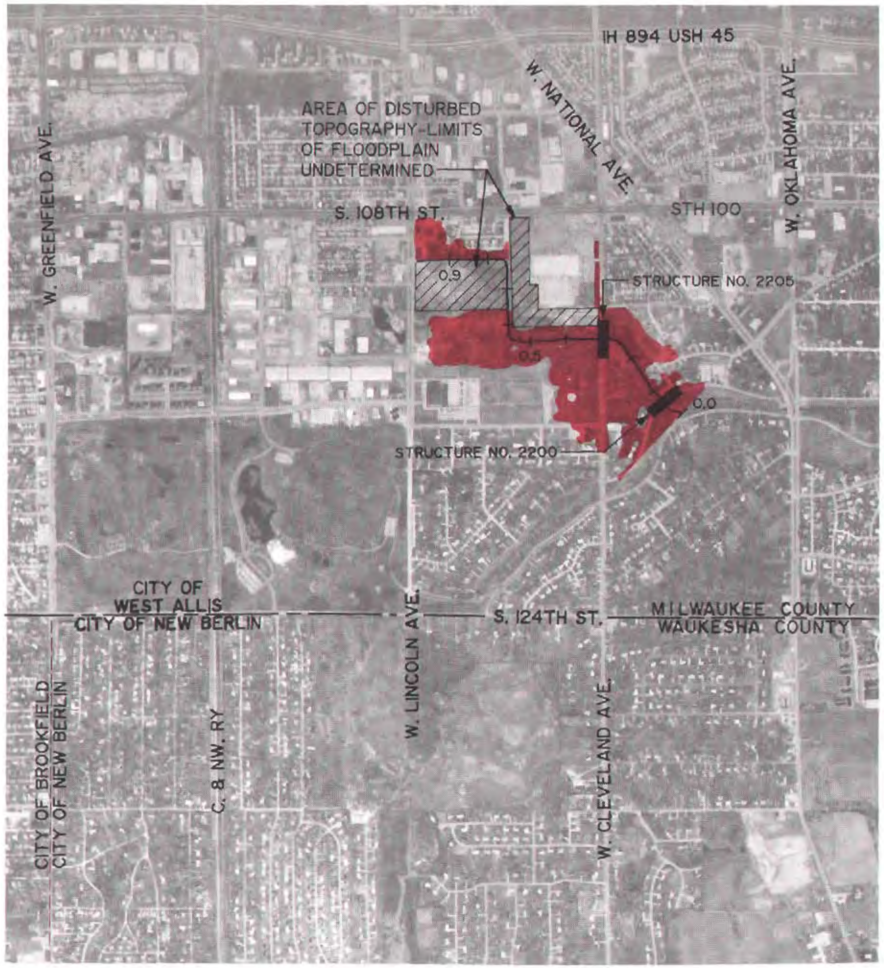
DATE OF PHOTOGRAPHY: APRIL 1986

Figure 34 (continued)



Source: SEWRPC.

Map 78 (continued)



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

0.5 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR HALE CREEK RIVER IS SHOWN IN APPENDIX H

GRAPHIC SCALE

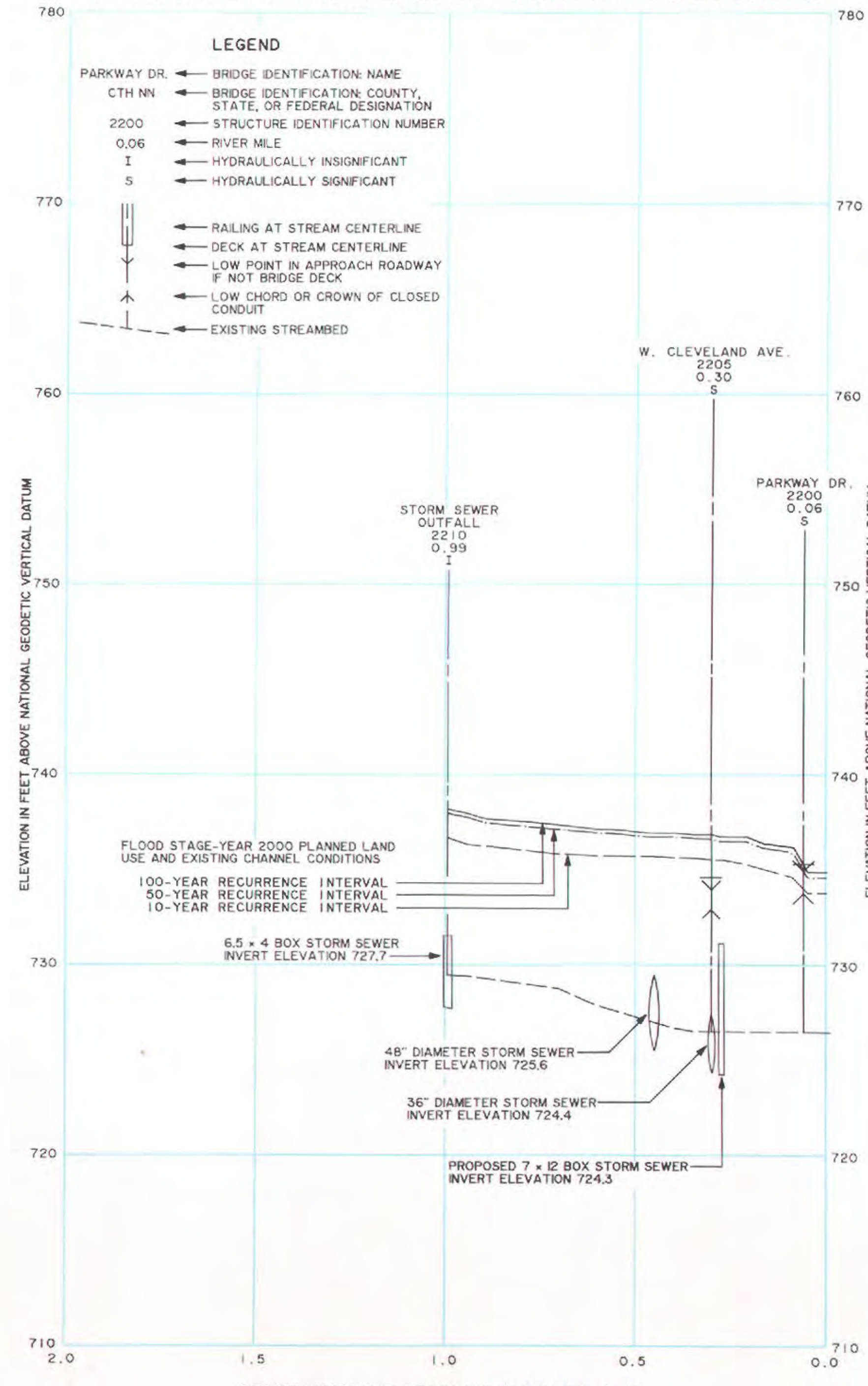
0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Source: SEWRPC.

Figure 35

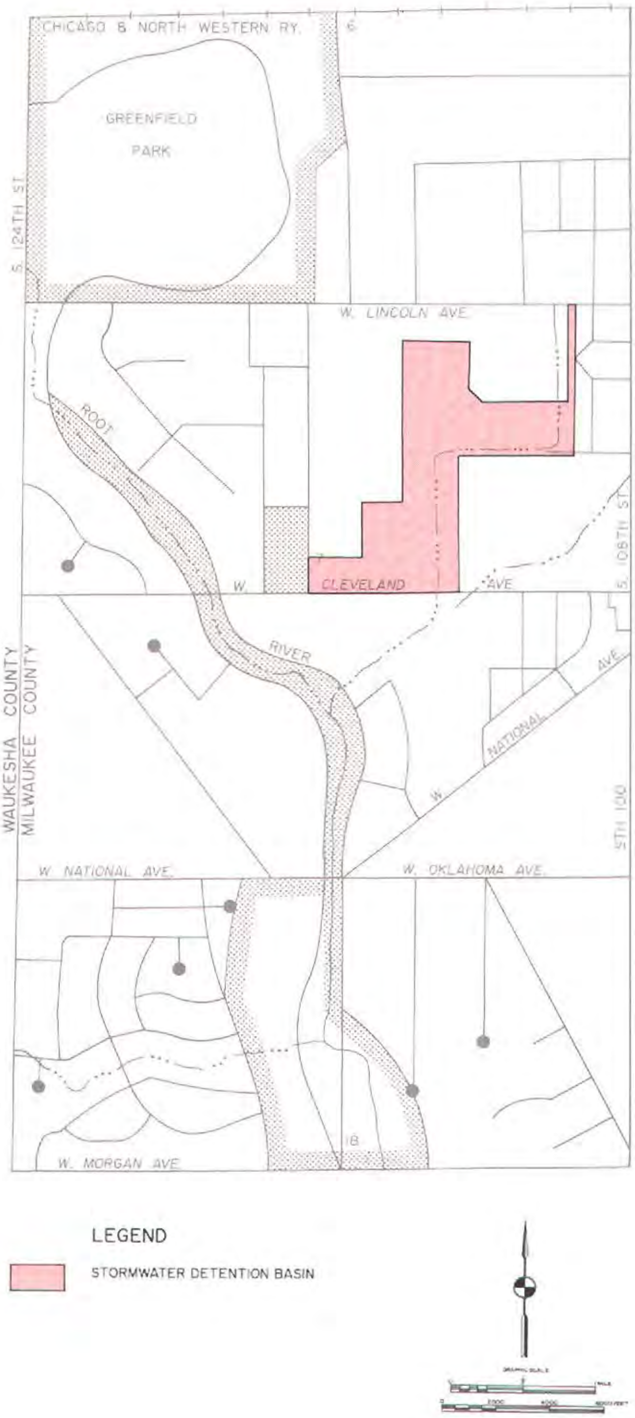
FLOOD STAGE AND STREAMBED PROFILE FOR HALE CREEK



Source: SEWRPC.

Map 79

POTENTIAL STORMWATER DETENTION STORAGE SITE ALONG HALE CREEK IN THE CITY OF WEST ALLIS



Source: SEWRPC.

Alternative 1—No Action: One alternative course of action for addressing the flood problem along the North Branch of the Root River and Hale Creek in the City of West Allis is to do nothing—that is, to recognize the inevitability of extensive flooding but to deliberately decide not to mount a collective, coordinated program to abate the flood damages. Under 1985 land use and existing channel conditions, the average annual flood damages along these two streams would approximate $25,000. The damages from a 100-year recurrence interval flood may be expected to approximate $671,000. Under planned, year 2000 land use and existing channel conditions, the average annual flood damages along these two streams would approximate $31,500. The damages from a 100-year recurrence interval flood may be expected to approximate $696,000. There are no monetary benefits associated with this alternative. The average annual cost would be equivalent to the average of the annual flood damage costs under existing and planned land use conditions, or $28,250.

Alternative 2—Structure Floodproofing, Elevation, and Removal with Stormwater Pumping: A structure floodproofing, elevation, and removal alternative was evaluated to determine if such a structure-by-structure approach would be a technically feasible and economically viable solution to the flood problem along the North Branch of the Root River and Hale Creek in the City of West Allis. The 100-year recurrence interval flood stage under planned, year 2000 land use and existing channel conditions was used to estimate the number of existing structures to be floodproofed, elevated, or removed and the approximate costs involved.

In the case of residential structures, floodproofing was assumed to be feasible if the design flood stage was below the first-floor elevation. Structure elevation was considered feasible for residential structures with basements if the estimated cost of elevating the structure and floodproofing the basement was less than the estimated removal cost. Structures to be elevated were assumed to have the first floor raised to an elevation at least two feet above the 100-year recurrence interval flood stage to provide adequate freeboard. For aesthetic reasons, structure elevation was limited to a maximum of four feet. Structures that would have to be elevated more than four feet were considered for removal.

Table 45

COST ESTIMATES FOR FLOOD CONTROL ALTERNATIVES FOR THE NORTH BRANCH ROOT RIVER AND HALE CREEK IN THE CITY OF WEST ALLIS

Alternative		Costs					Benefit-Cost Analysis			
			Annual							
Name	Description	Capital	Amortized Capital[a]	Operation and Maintenance	Other	Total	Annual Benefits	Annual Benefits Minus Annual Costs	Benefit-Cost Ratio	Economic Ratio Greater than One
1—No Action	- -	$ 0	$ 0	$ 0	$28,250	$ 28,250	$ 0	$ -28,750	- -	No
2—Structure Floodproofing, Elevation, and Removal with Stormwater Pumping	Floodproof 29 structures	$ 377,000	$301,600	$24,000	$ - -	$325,600	$28,250	$-297,350	0.1	No
	Elevate one structure	30,000								
	Remove one structure	90,000								
	Four stormwater pumping stations	4,260,000								
	Subtotal	$4,757,000								
3—Structure Floodproofing, Elevation, and Removal with Minor Channel Deepening	2.6 miles of channel modification	$1,134,000	$ 89,200	$ 5,100	$ - -	$ 94,300	$28,250	$ -66,050	0.3	No
	Replace four bridges	118,000[b]								
	Floodproof 14 structures	65,000								
	Remove one structure	90,000								
	Subtotal	$1,407,000								
4—Major Channel Modification	4.6 miles of channel modification	$5,806,000	$430,800	$18,700	$ - -	$449,500	$28,250	$-421,250	0.1	No
	Replace seven bridges	915,000[b]								
	1,700 feet of earthen dikes	74,000								
	Subtotal	$6,795,000								

[a] *Amortized capital cost is based on an interest rate of 6 percent and a project life of 50 years.*

[b] *Costs for bridges at W. Beloit Road and W. Cleveland Avenue on the North Branch of the Root River and W. Cleveland Avenue on Hale Creek were previously assigned under the Commission's adopted regional transportation system plan.*

Source: SEWRPC.

Floodproofing was considered to be feasible for all nonresidential structures provided the flood stage was not more than seven feet above the first floor elevation. The floodproofing costs were assumed to be a function of the depth of the water over the first floor.

As indicated on Map 80, of the 31 structures which may be expected to incur flood damage, 29 would have to be floodproofed, one would have to be elevated, and one would have to be removed. Future damage from floods up to and including the 100-year recurrence interval event would be virtually eliminated.

In addition to these floodproofing measures, this alternative includes the installation of four permanent stormwater pumping stations. These would be provided in order to relieve storm sewers that were constructed with outlet inverts at elevations below the existing channel bottom. These sewers include the following: 1) a 15-inch storm sewer on the west side of the North Branch of the Root River at W. National Avenue; 2) a 36-inch storm sewer on the west side of the North Branch of the Root River at W. Cleveland Avenue; 3) a 48-inch storm sewer on the west side of Hale Creek about 0.15 mile north of W. Cleveland Avenue; and 4) a 6.5-foot-wide by

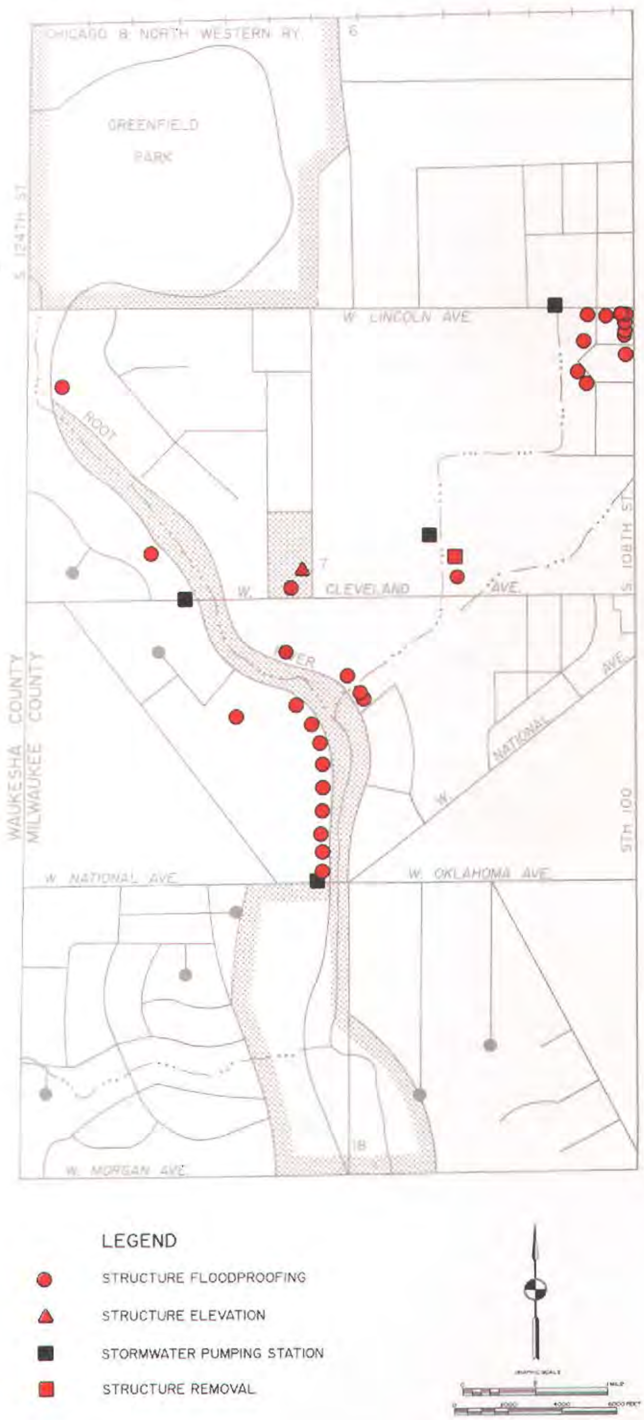
4-foot-high storm sewer box culvert discharging to Hale Creek at W. Lincoln Avenue.

Assuming that the structure floodproofing measures would be fully implemented, and utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of this alternative is estimated at $325,600. This cost consists of the amortization of the $4,757,000 capital cost—$377,000 for floodproofing, $30,000 for structure elevation, $90,000 for structure removal, and $4,260,000 for four pumping stations—and $24,000 in annual operation and maintenance costs. The average annual flood damage abatement benefit is estimated at $28,250, yielding a benefit-cost ratio of 0.1.

Alternative Plan 3—Structure Floodproofing, Elevation, and Removal with Minor Channel Deepening: This alternative system plan is shown on Map 81 and consists of lowering the streambed along the North Branch of the Root River and Hale Creek to accommodate the inverts of the existing storm sewer outlets with outlet invert elevations below the existing channel bottom. This deepening would be required along the 1.57-mile-long reach of the North Branch of the Root River between W. Morgan Avenue and the Parkway Drive bridge at River Mile 41.95, with the channel bottom being lowered up to 4.2 feet; and along the entire length of Hale Creek, with the channel bottom being lowered up to 2.6 feet. The proposed channel along the North Branch of the Root River would have a bottom width of 10 feet between W. Morgan Avenue and the confluence with Hale Creek, and a bottom width of eight feet between the confluence with Hale Creek and the Parkway Drive, while the proposed channel along Hale Creek would have a bottom width of eight feet. The proposed channel would have side slopes of one on three and would have a riprap lining to an elevation of two feet above the channel bottom, with the remainder being turf-lined. In order to accommodate the channel deepening, it would be necessary to replace the bridges at S. 116th Street and W. Cleveland Avenue and a pedestrian bridge at River Mile 41.12 on the North Branch of the Root River, and the W. Cleveland Avenue bridge on Hale Creek. Replacement of the Parkway Drive bridge on Hale Creek was not included in this plan as this bridge is scheduled for replacement by Milwaukee County in 1988 for transportation

Map 80

ALTERNATIVE PLAN 2: STRUCTURE FLOODPROOFING, ELEVATION, AND REMOVAL WITH STORMWATER PUMPING ALONG THE NORTH BRANCH ROOT RIVER AND HALE CREEK IN THE CITY OF WEST ALLIS



Source: SEWRPC.

improvement purposes and would be designed to accommodate a lowered channel invert. No costs were assigned to the two W. Cleveland Avenue bridge replacements, as those costs were assigned under the Commission's adopted regional transportation plan.

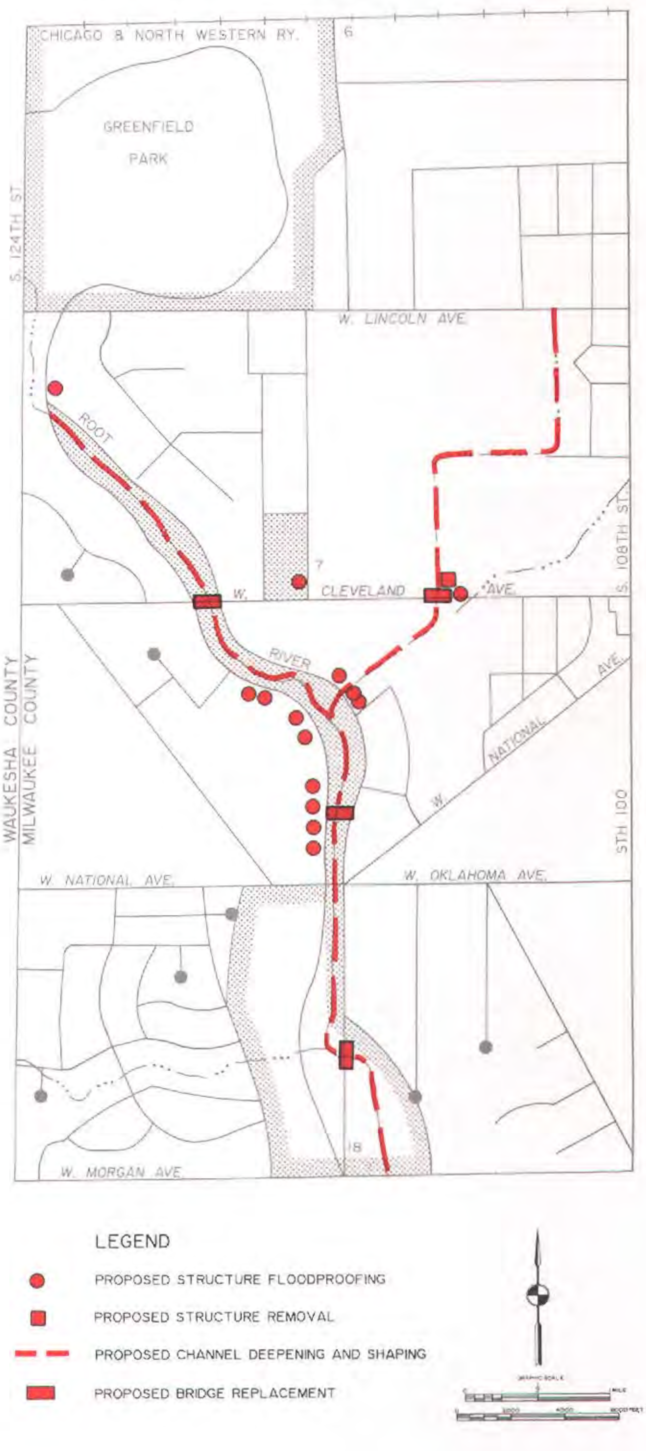
Inundation of lands along the North Branch of the Root River and Hale Creek would continue, although the frequency and severity of such inundation would be reduced. As a result of the proposed channel modifications and bridge replacement, no structures along the North Branch of the Root River, and only two structures along Hale Creek in the City of West Allis, would be expected to incur flood damages from a 10-year recurrence interval event under planned land use conditions; only 15 structures would be expected to incur flood damages from a 100-year recurrence interval event under planned land use conditions. Of these 15 structures, it is recommended that 14 be floodproofed and that one be removed.

Assuming that the structure floodproofing measures would be fully implemented, and utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of this alternative is estimated at $94,300. This cost consists of the amortization of the $1,407,000 capital cost—$1,134,000 for channel modification; $118,000 for bridge replacement; $65,000 for structure floodproofing; and $90,000 for structure removal—and $5,100 in annual operation and maintenance costs. The average annual flood damage abatement benefit is estimated at $28,250, yielding a benefit-cost ratio of 0.30.

Alternative 4—Major Channel Modification: This alternative flood control plan is shown on Map 82 and consists of widening and deepening, with some realignment, of the channel along a 3.56-mile-long reach of the North Branch of the Root River between W. Layton Avenue and W. Lincoln Avenue, and along the entire 0.99-mile-long reach of Hale Creek. Along the North Branch of the Root River the channel would be lowered up to 8.3 feet, with the resulting channel having a bottom width of 10 feet and side slopes of one on four between W. Layton Avenue and the confluence with Hale Creek, and a bottom width of eight feet and side slopes of one on four between Hale Creek and W. Lincoln Avenue. The entire channel would be lined with riprap up to an elevation of two feet above the channel

Map 81

ALTERNATIVE PLAN 3: STRUCTURE FLOODPROOFING, ELEVATION, AND REMOVAL WITH MINOR CHANNEL DEEPENING ALONG NORTH BRANCH ROOT RIVER AND HALE CREEK IN THE CITY OF WEST ALLIS



Source: SEWRPC.

bottom, with the remainder being turf-lined. A 1,000-foot-long reach of channel downstream of S. 108th Street would be realigned so as to match an existing bridge constructed by Milwaukee County in 1968 and intended to carry a planned parkway drive. This bridge was constructed about 100 feet east of the existing channel in anticipation of major channel modifications along the North Branch of the Root River. Some minor realignment of the channel of the North Branch of the Root River may also be necessary for the reach upstream of W. National Avenue in order to accommodate the proposed channel within the existing parkway.

Along Hale Creek the channel would be realigned so as to match a proposed drainage right-of-way as shown on City of West Allis sewer system maps. The channel would be lowered up to 6.8 feet, with the resulting channel having a bottom width of eight feet and side slopes of one on four between the confluence with the North Branch of the Root River and River Mile 0.70, and a bottom width of six feet and side slopes of one on four between River Mile 0.70 and W. Lincoln Avenue. The entire channel would be lined with riprap up to an elevation of two feet above the channel bottom, with the remainder being turf-lined.

In addition to the proposed channel modification, this alternative includes the replacement of six bridges on the North Branch of the Root River and one bridge on Hale Creek. These bridges are those at W. Cold Spring Road, W. Beloit Road, W. Morgan Avenue, S. 116th Street, and W. Cleveland Avenue, and a pedestrian bridge over the North Branch of the Root River at River Mile 41.12; and the bridge at W. Cleveland Avenue over Hale Creek. It was assumed that these bridges would be replaced with clear-span structures. Replacement of the Parkway Drive bridge over Hale Creek was not included in this alternative as that bridge is scheduled for replacement by Milwaukee County in 1988 for transportation improvement purposes and would be designed to accommodate the proposed channel invert and realignment. Also, no costs were assigned for the bridge replacements at W. Beloit Road and W. Cleveland Avenue on the North Branch of the Root River and at W. Cleveland Avenue on Hale Creek as those costs were assigned under the Commission's adopted regional transportation system plan.

Map 82

ALTERNATIVE PLAN 4: MAJOR CHANNEL MODIFICATION ALONG THE NORTH BRANCH ROOT RIVER AND HALE CREEK IN THE CITY OF WEST ALLIS

Source: SEWRPC.

Finally, this alternative includes the construction of about 1,700 feet of earthen dike along the west side of the North Branch of the Root River upstream of W. National Avenue. This dike is intended to prevent street and yard inundation in the residential area located west of the parkway. The proposed dike would average four feet in height and would be designed to contain the 100-year recurrence interval flood discharge under planned land use conditions with two feet of freeboard. As this dike is intended only to prevent minor inundation of residential yards and Parkway Drive and is not required to prevent structure damages, it was deemed unnecessary to use three feet of freeboard as required by the Federal Emergency Management Agency.

Implementation of this alternative would essentially eliminate all damages attendant to floods up to and including the 100-year recurrence interval event under planned land use conditions.

Utilizing an annual interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this alternative is estimated at $449,500. This cost consists of the amortization of the $6,795,000 capital cost—$5,806,000 for channel modification, $915,000 for bridge replacement, $74,000 for dikes—and $18,700 in annual operation and maintenance costs. The average annual flood abatement benefit is estimated at $28,250, yielding a benefit-cost ratio of about 0.1.

Evaluation of Flood Control Alternatives for the North Branch of the Root River and Hale Creek in the City of West Allis

The principal features of, and the costs and benefits associated with, each of the floodland management alternatives considered for the North Branch of the Root River and Hale Creek in the City of West Allis have been summarized in Table 45. All of the alternatives considered were found to be technically feasible. None of the alternatives, however, were found to have a benefit-cost ratio of one or more. The "no action" alternative, while offering the lowest cost, does nothing to alleviate the flood problem, and therefore does not represent a publicly acceptable approach.

Alternative 2—Structure Floodproofing, Elevation, and Removal with Stormwater Pumping—presents several problems in implementation. First, complete implementation of a voluntary structure floodproofing and elevation program is unlikely; and with partial implementation, the City of West Allis would be left with a residual problem whenever a major flood event occurred. Also, yard damages and cleanup costs would remain under this alternative. The high cost of the pumping facilities also makes this alternative unattractive. A desirable feature of this alternative is that it would provide relief for those storm sewers that were constructed with outlet invert elevations below the existing channel bottom. These sewers operate with partially blocked or negative slope outlets, thereby reducing their effective conveyance capacity. This situation results in the potential for street and building flooding in areas away from the channel. It should be noted that if the structure floodproofing measures designed to alleviate damage due to overland flooding from the channel system were considered separately, this alternative would have the highest benefit-cost ratio, 0.90, of all the alternatives considered.

Alternative 3—Structure Floodproofing, Elevation, and Removal with Minor Channel Deepening—also presents problems relating to implementation of the structure floodproofing measures. In addition, the proposed channel deepening and reshaping would serve to reduce the "natural" appearance of the present channel on the North Branch of the Root River, as well as to increase downstream flood discharges and stages. The 100-year flood discharge under planned land use conditions may be expected to increase by up to 11 percent downstream of W. Morgan Avenue as a result of the channel modification, with no increases anticipated, however, downstream of the confluence with the East Branch of the Root River. This increase in flood discharge would result in relatively small stage increases, with the largest increase being 0.3 foot immediately downstream of W. Beloit Road. These higher flood stages would occur mostly on Milwaukee County parklands, although some private properties in the City of Greenfield would also be affected, requiring that proper legal arrangements be made with the affected property owners. This alternative would, however, provide an adequate outlet for those storm sewers with partially blocked or negatively sloped outfalls, and at a much lower cost than stormwater pumping facilities. The larger, more hydraulically efficient channel should also reduce the frequency with which floodwaters inundate the Root River Parkway Drive upstream of W. National Avenue, although inundation during less frequent events would still occur. This alternative also presents the highest benefit-cost ratio of the alternatives considered—0.3.

Alternative 4—Major Channel Modification—would also serve to eliminate structure damages from floods up to and including a 100-year recurrence interval event, as well as provide an adequate outlet for those storm sewers with partially blocked or negatively sloped outfalls. This alternative would also provide the greatest reduction in the frequency of yard and street flooding of all the alternatives considered, with the floodwaters being confined almost entirely to parkway lands under a 100-year recurrence interval event. This alternative would also serve to implement the Commission's adopted Root River watershed plan, which had considered these channel modifications committed. It would also be in conformance with the planned drainage right-of-way for Hale Creek, which is shown on the City of West Allis sewer system maps. However, like Alternative 3, this alternative would have an impact on the "natural" appearance of the existing channel. This alternative would also produce the largest increase in downstream flood flows and stages of the four alternatives considered, with increases of up to 13 percent in the 100-year recurrence interval flood discharge under planned land use conditions downstream of W. Layton Avenue. Again, no significant increase in flood flows would occur downstream of the confluence with the East Branch of the Root River. The largest attendant downstream stage increase would be about 0.5 foot and would occur approximately 800 feet downstream of W. Layton Avenue. Finally, the high cost of this alternative relative to anticipated flood damages makes it economically unattractive. By comparison, the cost of purchasing all of the flood-damage-prone structures along the North Branch of the Root River and Hale Creek in the City of West Allis is about $3,702,000, or a little over one-half the cost of the major channel modifications.

Alternative Flood Control and Related Drainage System Plans for the North Branch of the Root River in the City of Greenfield

In order to reduce the number of combinations of flood control alternatives to be considered for the reach of the Root River through the City of Greenfield, it was assumed that Alternative 3, which calls for a combination of structure floodproofing, elevation, and removal plus minor channel deepening along the Root River and Hale Creek in the City of West Allis, would be implemented. Four alternative flood control plans were considered and evaluated for alleviating flood damage problems along the North Branch of the Root River through the City of Greenfield: 1) No Action; 2) Structure Floodproofing, Elevation, and Removal; 3) Major Channel Modification; and 4) Combination of Detention Storage and Structure Floodproofing, Elevation, and Removal.

A fifth alternative—diking—was also considered but eliminated from further study. Preliminary investigations indicated that about 7,400 feet of earthen dike averaging eight feet in height would be required along the Root River between W. Forest Home Avenue and W. Layton Avenue. The cost of these dikes is estimated at $1,179,000. Stormwater drainage facilities, including stormwater pumping stations, which would be required to adequately convey runoff from behind these dikes would cost in excess of $2,000,000. Because of the severe adverse aesthetic impacts eight-foot-high dikes would have along the Root River Parkway Drive, as well as the high costs that would be incurred in providing stormwater drainage facilities adequate to accommodate localized runoff behind these dikes, this alternative was not considered further. A diking alternative had previously been analyzed as part of a 1983 study of flood control alternatives for the City of Greenfield by the U. S. Army Corps of Engineers.[4] That study also concluded that diking would not be a cost-effective solution to the flood problems concerned.

Each of the four alternatives evaluated is described below. The estimated economic benefits and costs attendant to each alternative are provided in Table 46.

Alternative 1—No Action: One alternative course of action for addressing the flood problem along the North Branch of the Root River in the City of Greenfield is to do nothing—that is, to recognize the inevitability of extensive flooding but to deliberately decide not to mount a collective, coordinated program to abate the flood damages. Under existing land use and existing channel conditions, the average annual flood

[4] *U. S. Army Corps of Engineers, "Section 205 Report on Flood Control on the Root River, at the City of Greenfield, Milwaukee County, Wisconsin," June 17, 1983.*

Table 46

COST ESTIMATES FOR FLOOD CONTROL ALTERNATIVES FOR THE NORTH BRANCH ROOT RIVER IN THE CITY OF GREENFIELD

Alternative		Costs					Benefit-Cost Analysis			
			Annual							
Name	Description	Capital	Amortized Capital[a]	Operation and Maintenance	Other	Total	Annual Benefits	Annual Benefits Minus Annual Costs	Benefit-Cost Ratio	Economic Ratio Greater than One
1—No Action	- -	$ 0	$ 0	$ 0	$35,000	$ 35,000	$ 0	$ -35,000	- -	No
2—Structure Floodproofing, Elevation, and Removal	Floodproof 15 structures	$ 69,000	$94,500	$ 0	$ - -	$ 94,500	$35,000	$ -59,500	0.37	No
	Elevate 17 structures	563,000								
	Remove nine structures	790,000								
	Relocate two structures on lots	68,000								
	Subtotal	$1,490,000								
3—Major Channel Modification	3.0 miles of channel modification	$4,566,000	$391,000	$ 6,200	$ - -	$397,200	$35,000	$-362,200	0.10	No
	Replace five bridges	1,600,000[b]								
	Subtotal	$6,166,000								
4—Combination of Detention Storage and Structure Floodproofing, Elevation, and Removal	Stormwater detention basin	$ 800,000	$221,500	$30,000	$ - -	$251,500	$35,000	$-216,500	0.14	No
	Elevation of W. Cold Spring Road	144,000								
	Stormwater pumping station	1,688,000								
	Floodproof 20 structures	92,000								
	Elevate 13 structures	420,000								
	Remove four structures	350,000								
	Subtotal	$3,494,000								

[a] *Amortized capital cost is based on an interest rate of 6 percent and a project life of 50 years.*

[b] *Cost for W. Layton Avenue bridge was previously assigned under the Commission's adopted regional transportation system plan.*

Source: SEWRPC.

damages along the stream would approximate $25,000. The damages from a 100-year recurrence interval flood may be expected to approximate $395,000. Under planned, year 2000 land use and existing channel conditions, the average annual flood damages along the stream would approximate $45,000. The damages from a 100-year recurrence interval flood may be expected to approximate $465,000. There are no monetary benefits associated with this alternative, and the average annual cost would be equivalent to the average of the existing and planned land use average annual flood damage costs, or $35,000.

Alternative 2—Structure Floodproofing, Elevation, and Removal: A structure floodproofing, elevation, and removal alternative was evaluated to determine if such a structure-by-structure approach would be a technically feasible and economically viable solution to the flood problem along the North Branch of the Root River in the City of Greenfield. The 100-year recurrence interval flood stage under planned year 2000 land use and planned channel conditions was used to estimate the number of existing flood-prone structures to be floodproofed, elevated, or removed and the approximate costs involved.

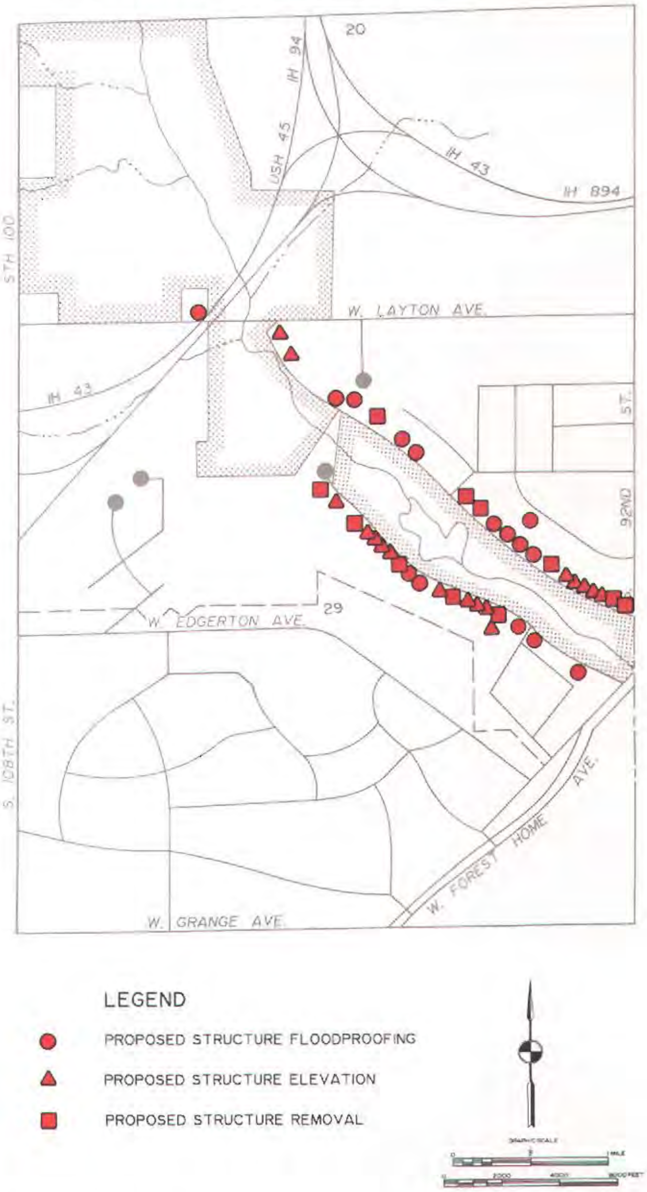
In the case of residential structures, floodproofing was assumed to be feasible if the design flood stage was below the first-floor elevation. Structure elevation was considered feasible for residential structures with basements if the estimated cost of elevating the structure and floodproofing the basement was less than the estimated removal cost. Structures to be elevated were assumed to have the first floor raised to an elevation at least two feet above the 100-year recurrence interval flood stage to provide adequate freeboard. For aesthetic reasons, structure elevation was limited to a maximum of four feet. Structures that would have to be elevated more than four feet were considered for removal.

As shown on Map 83, of the 43 structures which may be expected to incur flood damage from a 100-year recurrence interval flood, 15 would have to be floodproofed, 17 would have to be elevated, and 11 would have to be removed. Future damage from floods up to and including the 100-year recurrence interval event would be virtually eliminated. Of the 11 structures requiring removal, two could be moved to new locations on their lots, beyond the limit of the 100-year recurrence interval floodplain, and subsequently resold. These homes are located along W. Root River Parkway north of W. Brookside Drive. It is assumed that the purchase price of these homes would be returned through resale, with the net cost being the cost of moving the homes on the lots. The remaining seven structures could not be relocated on their present lots as these lots are either too small or are located completely within the 100-year recurrence interval floodplain. It is possible, however, that those structures could be relocated to lots within the general vicinity but beyond the 100-year recurrence interval floodplain. Such relocation would have to be considered on a site-specific basis, with the cost entailed depending on the purchase price of the new lot and the distance the house needs to be moved.

A similar structure floodproofing and removal alternative was presented in the Commission's Root River watershed plan. Under that study, it was determined that 29 structures would incur damages from a 100-year recurrence interval flood. Of those 29 structures, it was recommended that six be floodproofed and that 23 be removed. The discrepancy in the number of houses concerned is due, in part, to the fact that seven houses have been constructed within the 100-year recurrence interval floodplain since the preparation of the Root River watershed plan, and in part to the availability of more detailed topographic information for this system plan, particularly the field-surveyed first-floor elevations for homes along the parkway.

Map 83

ALTERNATIVE PLAN 2: STRUCTURE FLOODPROOFING, ELEVATION, AND REMOVAL ALONG THE NORTH BRANCH ROOT RIVER IN THE CITY OF GREENFIELD



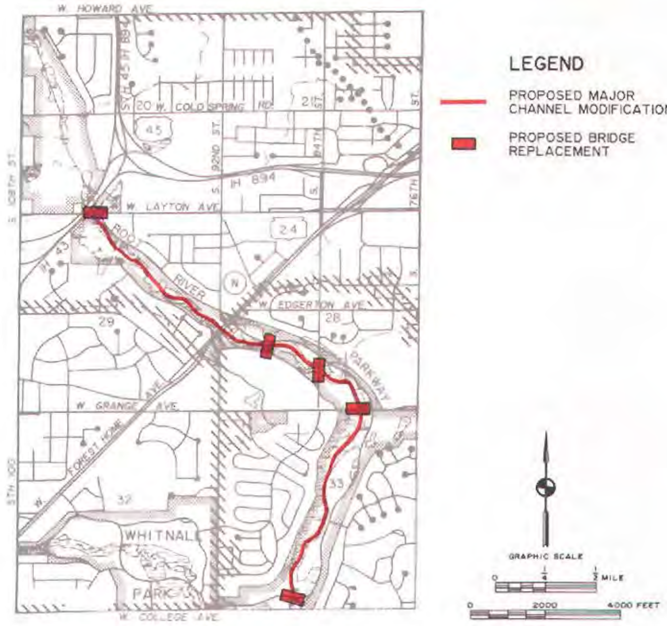
Source: SEWRPC.

Assuming that these structure floodproofing measures would be fully implemented, and utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of this alternative is estimated at $94,500. This cost consists of the amortization of the $1,490,000 capital cost—$69,000 for floodproofing, $563,000 for structure elevation, $790,000 for structure removal, and $68,000 for structure relocation. The average annual flood damage abatement benefit is estimated at $35,000, yielding a benefit-cost ratio of 0.37.

Alternative 3—Major Channel Modification: This alternative system for the resolution of the flood problems along the North Branch of the Root River in the City of Greenfield is shown on Map 84. This plan consists of lowering the existing streambed along a 3.02-mile-long reach between W. College Avenue and the Rock Freeway (IH 43). Along this reach the streambed would be lowered up to 6.4 feet, with the resulting channel having a bottom width of 20 feet and side slopes of one on four. The entire channel would be lined with riprap to an elevation two feet above the invert, with the remainder being turf-lined.

Map 84

ALTERNATIVE PLAN 3: MAJOR CHANNEL MODIFICATION ALONG THE NORTH BRANCH ROOT RIVER IN THE CITY OF GREENFIELD



Source: SEWRPC.

In addition to the channel modification, this alternative would require the replacement of five bridges. These bridges are located at W. College Avenue, W. Grange Avenue, S. 84th Street, the Parkway Drive at River Mile 37.39, and W. Layton Avenue. It was assumed that these bridges would be replaced with clear-span structures. It should be noted that no cost was assigned under this system plan for the replacement of the W. Layton Avenue bridge as that cost was assigned under the Commission's adopted regional transportation system plan.

Implementation of this alternative would serve to eliminate structure flood damages from floods up to and including a 100-year recurrence interval event under planned land use conditions.

Utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of this alternative is estimated at $397,200. This cost consists of the amortization of the $6,166,000 capital cost—$4,566,000 for channel modification, and $1,600,000 for bridge replacement—and $6,200 in annual operation and maintenance costs. The average annual flood abatement benefit is estimated at $35,000, resulting in a benefit-cost ratio of 0.1.

Alternative 4—Combination of Detention Storage and Structure Floodproofing, Elevation, and Removal: This alternative flood control plan is shown on Map 85 and consists of constructing a stormwater detention basin along Milwaukee County Parkway lands between W. Layton Avenue and W. Cold Spring Road. This basin would be created by constructing about 1,050 feet of earthen dike along a northeast to southwest alignment, immediately upstream of the Rock Freeway (IH 43). This dike would have an average height of 12 feet with side slopes of one on three. Additional dikes would be required along the proposed Milwaukee Area Aquatic Park located along the southwest end of the detention basin. It would also be necessary to elevate about 1,650 feet of W. Cold Spring Road to at least two feet above the design pool elevation of 727.6 feet above National Geodetic Vertical Datum (NGVD). Outflow from the detention basin would be accommodated by two reinforced concrete box culverts, each being 10 feet wide by 10 feet high. The entire detention basin would be located on lands currently owned by Milwaukee County with the exception of one residential property located along W. Layton Avenue. A stormwater pumping station would also be required to accommodate stormwater runoff from the Aquatic Park, as well as lands located west of S. 108th Street.

Construction of this detention basin would serve to eliminate flood damages to six existing structures during a 100-year recurrence interval event under planned land use conditions. Of the 37 structures that would still be expected to sustain flood damages, 20 would have to be floodproofed, 13 would have to be elevated, and four would have to be removed.

Implementation of this alternative would serve to eliminate structure flood damages along the North Branch of the Root River in the City of Greenfield for floods up to and including the 100-year recurrence interval event under planned land use conditions.

Utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of this alternative is estimated at $251,500. This cost consists of the amortization of the $3,494,000 capital cost—$800,000 for the detention basin including land acquisition, $144,000 for elevation of W. Cold Spring Road, $1,688,000 for stormwater pumping, $92,000 for structure floodproofing, $420,000 for structure elevation, and $350,000 for structure removal—and $30,000 in annual operation and maintenance costs. The average annual flood abatement benefit is estimated at $35,000, resulting in a benefit-cost ratio of 0.14.

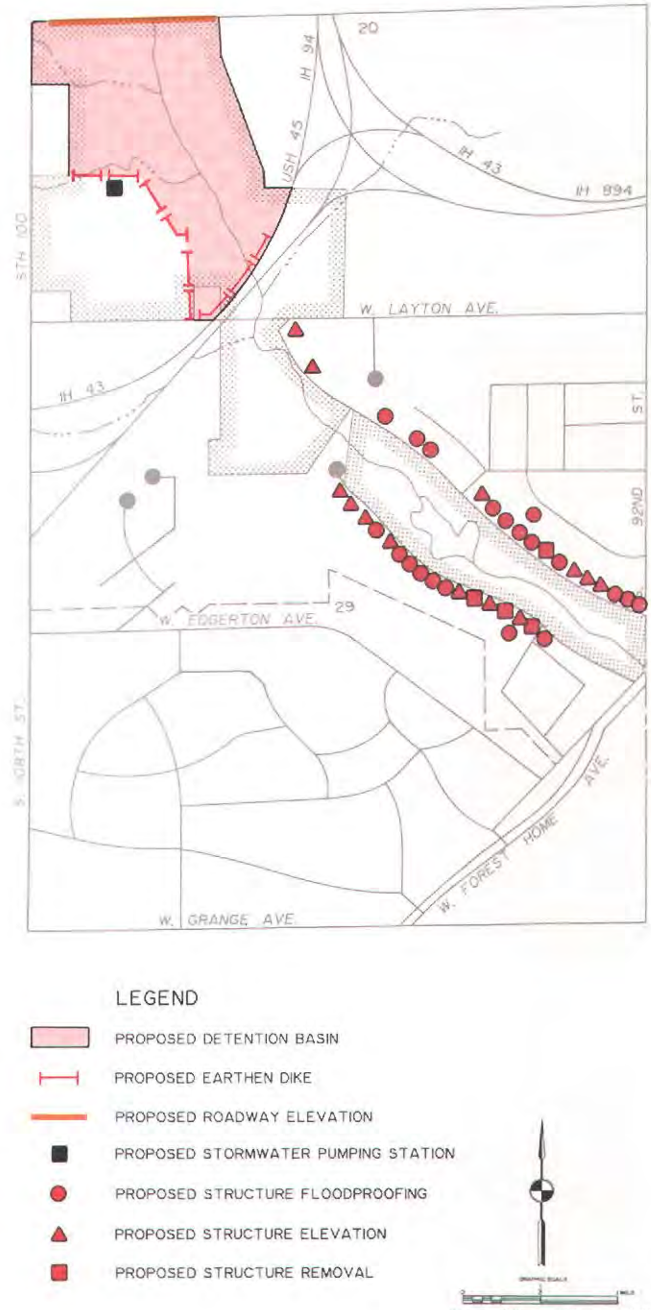
Evaluation of Flood Control Alternatives for the North Branch of the Root River in the City of Greenfield

The principal features of, and the costs and benefits associated with, each of the floodland management alternatives considered for the North Branch of the Root River in the City of Greenfield have been summarized in Table 46. All of the alternatives considered were found to be technically feasible.

None of the alternatives were found to provide a benefit-cost ratio of one or more. The "no action" alternative, while offering the lowest cost, does nothing to alleviate the flood problem, and therefore does not represent a sound approach to flood control.

Map 85

ALTERNATIVE PLAN 4: COMBINATION OF DETENTION STORAGE AND STRUCTURE FLOODPROOFING, ELEVATION, AND REMOVAL ALONG THE NORTH BRANCH ROOT RIVER IN THE CITY OF GREENFIELD



Source: SEWRPC.

Alternative 2—Structure Floodproofing, Elevation, and Removal—presents several problems in implementation. First, complete implementation of a voluntary structure and elevation program is unlikely, and with partial implementation, the City of Greenfield would be left with a residual problem whenever a major flood event occurred. Also, yard damages and cleanup costs would remain under this alternative.

Alternative 3—Major Channel Modification—would serve to eliminate structure flood damages while preserving all existing structures. Some homeowners along this stream reach have opposed removal of homes in the past. This alternative would, however, have a significant aesthetic impact on the channel, including the removal of a small lagoon located upstream of W. Forest Home Avenue. Also, problems may arise in implementation of this alternative, as Village of Greendale officials have, in the past, expressed opposition to channelization along the North Branch of the Root River in the Village of Greendale, as well as in the City of Greenfield. Finally, the high cost of this alternative makes it unattractive.

Alternative 4—Combination of Detention Storage and Structure Floodproofing, Elevation, and Removal—would require the removal of only four structures. Also, construction of the detention basin would serve to reduce downstream flood flows and stages. Flood stages would increase, however, between W. Cold Spring Road and W. Morgan Avenue, although not enough to cause additional structure damages.

Alternative Flood Control and Related Drainage System Plans for the North Branch of the Root River in the City of Franklin

In order to reduce the number of combinations of flood control alternatives to be considered for this portion of the North Branch of the Root River, it was assumed that Alternative 3—Structure Floodproofing, Elevation, and Removal with Minor Channel Deepening—and Alternative 2—Structure Floodproofing, Elevation, and Removal—would be implemented to solve flood damage problems in the Cities of West Allis and Greenfield, respectively. Three alternative flood control plans were considered and evaluated for alleviating flood damage problems in the City of Franklin: 1) No Action; 2) Structure Floodproofing, Elevation, and Removal; and 3) Major Channel Modification. Each of the three alternatives is described below. The estimated economic benefits and costs attendant to each alternative are provided in Table 47.

Alternative 1—No Action: One alternative course of action for addressing the flood problem along the North Branch of the Root River in the City of Franklin is to do nothing—that is, to recognize the inevitability of extensive flooding but to deliberately decide not to mount a collective, coordinated program to abate the flood damages. Under existing land use and existing channel conditions, the average annual flood damages along the stream would approximate \$7,600. The damages from a 100-year recurrence interval flood may be expected to approximate \$63,000. Under planned year 2000 land use and existing channel conditions, the average annual flood damages along the stream would approximate \$9,600. The damages from a 100-year recurrence interval flood may be expected to approximate \$65,000. There are no monetary benefits associated with this alternative, and the average annual cost would be equivalent to the average of the existing and planned land use average annual flood damage costs, or \$8,600.

Alternative 2—Structure Floodproofing, Elevation, and Removal: A structure floodproofing, elevation, and removal alternative was evaluated to determine if such a structure-by-structure approach would be a technically feasible and economically viable solution to the flood problem along the North Branch of the Root River in the City of Franklin. The 100-year recurrence interval flood stage under planned, year 2000 land use and planned channel conditions was used to estimate the number of existing structures to be floodproofed, elevated, or removed and the approximate costs involved.

In the case of residential structures, floodproofing was assumed to be feasible if the design flood stage was below the first-floor elevation. Structure elevation was considered feasible for residential structures with basements if the estimated cost of elevating the structure and floodproofing the basement was less than the estimated removal cost. Structures to be elevated were assumed to have the first floor raised to an elevation at least two feet above the 100-year recurrence interval flood stage to provide adequate freeboard. For aesthetic reasons, structure

Table 47

COST ESTIMATES FOR FLOOD CONTROL ALTERNATIVES FOR THE NORTH BRANCH ROOT RIVER IN THE CITY OF FRANKLIN

Alternative		Costs					Benefit-Cost Analysis			
			Annual							
Name	Description	Capital	Amortized Capital[a]	Operation and Maintenance	Other	Total	Annual Benefits	Annual Benefits Minus Annual Costs	Benefit-Cost Ratio	Economic Ratio Greater than One
1—No Action	- -	$ 0	$ 0	$ 0	$8,600	$ 8,600	$ 0	$ -8,600	- -	No
2—Structure Floodproofing, Elevation, and Removal	Floodproof three structures	$ 30,000	$ 3,800	$ 0	$ - -	$ 3,800	$8,600	$ 4,800	2.26	Yes
	Elevate one structure	30,000								
	Subtotal	$ 60,000								
3—Major Channel Modification	0.8 mile of channel modification	$800,000	$60,200	$1,700	$ - -	$61,900	$8,600	$-53,300	0.14	No
	Replace two bridges	116,000								
	Floodproof four structures	33,000								
	Subtotal	$949,000								

[a]*Amortized capital cost is based on an interest rate of 6 percent and a project life of 50 years.*

Source: SEWRPC.

elevation was limited to a maximum of four feet. Structures that would have to be elevated more than four feet were considered for removal.

Floodproofing was considered to be feasible for all nonresidential structures provided the flood stage was not more than seven feet above the first-floor elevation. The floodproofing costs were assumed to be a function of the depth of the water over the first floor. As shown on Map 86, of the four structures that may be expected to incur flood damages, three would have to be floodproofed and one would have to be elevated.

Assuming that these structure floodproofing measures would be fully implemented, and utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of this alternative is estimated at $3,800. This cost consists of the amortization of the $60,000 capital cost—$30,000 for floodproofing and $30,000 for elevation. The average annual flood damage abatement benefit is estimated at $8,600, yielding a benefit-cost ratio of 2.26.

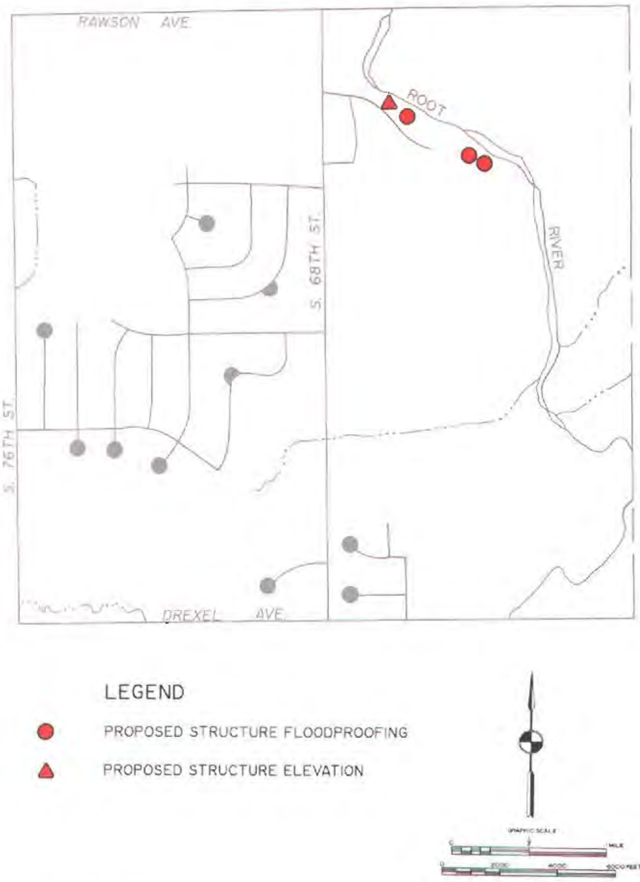
Alternative Plan 3—Major Channel Modification: This alternative flood control plan is shown on Map 87 and consists of deepening and widening the North Branch of the Root River along a 0.81-mile-long reach between River Mile 31.56 and W. Rawson Avenue. Within this reach, the streambed would be lowered up to 1.6 feet, with the resulting channel having a bottom width of 30 feet and side slopes of one on three. The proposed channel would be lined with riprap to an elevation two feet above the streambed, with the remainder being turf-lined.

This alternative also includes the replacement of culverts under two private drives located within the Franklin Aggregates Company stone quarry in the northeast one-quarter of U. S. Public Land Survey Section 10, Township 5 North, Range 21 East. Each structure consists of four 48-inch-diameter corrugated metal pipes. These culverts would be replaced with two reinforced concrete box culverts, each being 10 feet wide by 6 feet high.

The results of the evaluation of this alternative indicated that all four structures that are expected to incur flood damages would still need

Map 86

ALTERNATIVE PLAN 2: STRUCTURE FLOODPROOFING, ELEVATION, AND REMOVAL ALONG THE NORTH BRANCH ROOT RIVER IN THE CITY OF FRANKLIN



Source: SEWRPC.

Map 87

ALTERNATIVE PLAN 3: MAJOR CHANNEL MODIFICATION ALONG THE NORTH BRANCH ROOT RIVER IN THE CITY OF FRANKLIN

Source: SEWRPC.

to be floodproofed. Because of the high cost of the channel modification relative to anticipated flood damages, more extensive channel modifications were considered economically infeasible, and therefore were not evaluated.

Utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of this alternative is estimated at $61,900. This cost consists of the amortization of the $949,000 capital cost—$800,000 for channel modification, $116,000 for bridge replacement, and $33,000 for structure floodproofing—and $1,700 in annual operation and maintenance costs. The average annual flood damage abatement benefit is estimated at $8,600, yielding a benefit-cost ratio of 0.14.

Evaluation of Flood Control Alternatives for the North Branch of the Root River in the City of Franklin

The principal features of, and the costs and benefits associated with, each of the floodland management alternatives considered for the North Branch of the Root River in the City of Franklin have been summarized in Table 47. All of the alternatives considered were found to be technically feasible.

Only one of the alternatives—structure floodproofing, elevation, and removal—was found to have a benefit-cost ratio of one or more. The "no action" alternative, while offering the lowest cost, does nothing to alleviate the flood problem, and therefore does not represent a sound approach to flood control.

Alternative 2—Structure Floodproofing, Elevation, and Removal—presents several problems in implementation. First, complete implementation of a voluntary structure floodproofing and elevation program is unlikely, and with partial implementation the City of Franklin would be left with a residual problem whenever a major flood event occurred. Also, yard damages and cleanup costs would remain under the structure floodproofing, elevation, and removal alternative.

Alternative 3—Major Channel Modification—would abate structure flood damages but would also require that all four buildings expected to experience flood damages be floodproofed. The higher cost of extensive channel modifications required to eliminate the need for floodproofing would be economically unacceptable. Also, channel modifications would have an adverse aesthetic impact on the stream channel.

Recommended Flood Control System for the North Branch of the Root River and Hale Creek

Based upon considerations of the technical feasibility, economic viability, environmental impacts, potential public acceptance, and practicality of each of the alternatives considered, it is recommended that Alternative 3—Structure Floodproofing, Elevation, and Removal with Minor Channel Deepening—which would provide existing storm sewers with adequate outfalls, be adopted for the North Branch of the Root River and Hale Creek in the City of West Allis; that Alternative 2—Structure Floodproofing, Elevation, and Removal—be adopted for the North Branch of the Root River in the City of Greenfield; and that a revision of Alternative 2—Structure Floodproofing, Elevation, and Removal—be adopted for the North Branch of the Root River in the City of Franklin.

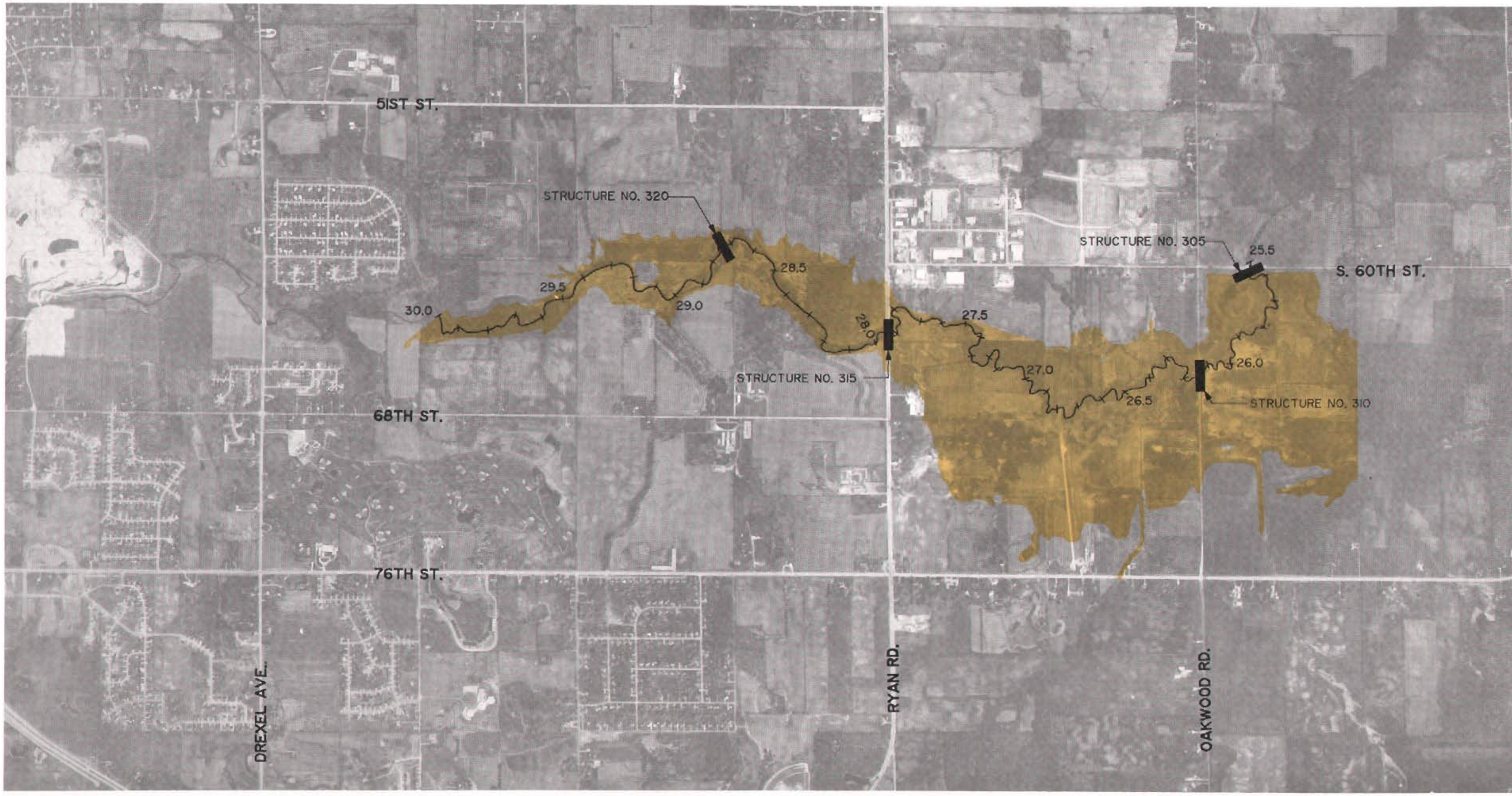
In July and August of 1989, the Milwaukee Metropolitan Sewerage District conducted a field survey of buildings affected by the recommended plan. Data obtained through that survey were used to refine the recommended plan. Those refinements are included in the plan costs and description given below.

The total capital cost of the combined recommended flood control plan for the North Branch of the Root River and Hale Creek is estimated at $3,488,000 in 1986 dollars, $2,740,000 for the North Branch of the Root River and $748,000 for Hale Creek. Annual operation and maintenance costs are estimated at $5,100—$3,100 for the North Branch of the Root River and $2,000 for Hale Creek. The recommended plan is shown graphically on Map 88. The peak flood profile attendant to planned land use and channel conditions in the subwatershed is shown in Figure 36 for the North Branch of the Root River, and Figure 37 for Hale Creek. Flood discharges which may be expected along the North Branch of the Root River and Hale Creek as a result of the recommended channel deepening are provided in Table 48. These increased flood discharges may be expected to result in increases of 0.1 foot to 0.3 foot in the 100-year recurrence interval flood stage under planned land use conditions in the reach between W. Forest Home Avenue and S. 116th Street. Stage increases downstream of W. Forest Home Avenue would be less than 0.1 foot. Implementation of the recommended plan would essentially eliminate all flood-related damages to structures along the North Branch of the Root River for floods up to and including the 100-year recurrence interval event under planned land use conditions. The recommended plan is more fully described below.

The recommended flood control plan for the North Branch of the Root River and Hale Creek in the City of West Allis is shown on Map 88, and consists of lowering the streambed by up to 4.2 feet along a 1.6-mile-long reach of the North Branch of the Root River between W. Morgan Avenue and the Parkway Drive bridge at River Mile 41.95, and by up to 2.6 feet along the entire 1.0-mile length of Hale Creek. This deepening is intended to provide an outlet for existing storm sewers that were constructed with outlet inverts at elevations below the existing channel bottom. The proposed channel would have bottom widths ranging from 6 to 10 feet and side slopes of one on three. The channel would be riprap-lined to an elevation two feet above the proposed streambed, with the remainder being turf-lined. In order to accommodate the lower streambed profile, bridge replacement would be required at S. 116th Street, at W. Cleveland Avenue on the North Branch of the Root River, and at W. Cleveland Avenue on Hale Creek. Also, a pedestrian bridge at River Mile 41.12 would need to be replaced. In addition, it is recommended that three houses along the North Branch of the Root River and five houses along Hale Creek be floodproofed and that one house along the North Branch of the Root River and one house along Hale Creek be removed.

Map 88

RECOMMENDED FLOOD CONTROL SYSTEM PLAN FOR THE NORTH BRANCH ROOT RIVER AND HALE CREEK



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS

30.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR NORTH BRANCH ROOT RIVER IS SHOWN IN APPENDIX H

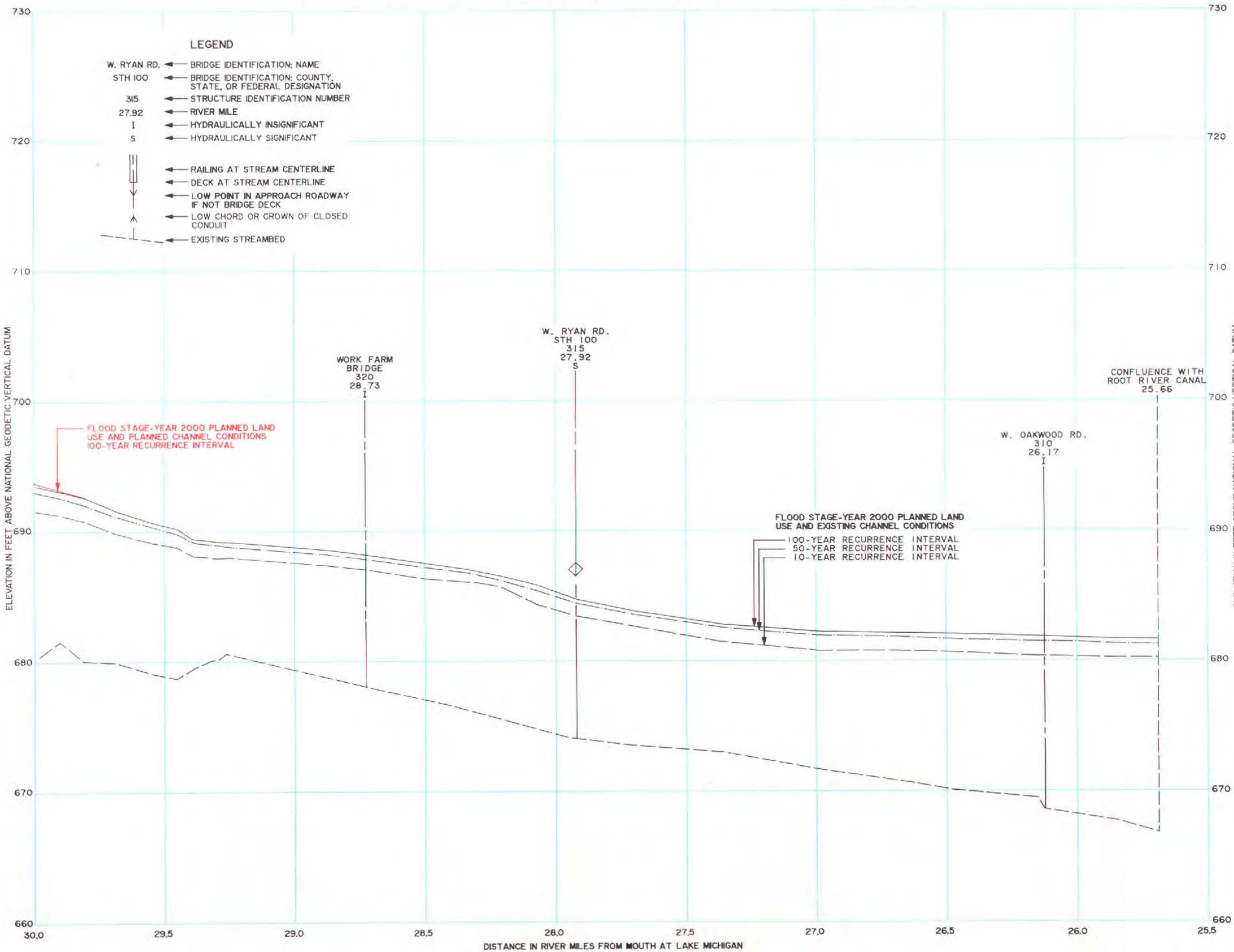
GRAPHIC SCALE

0 1/2 1 MILE

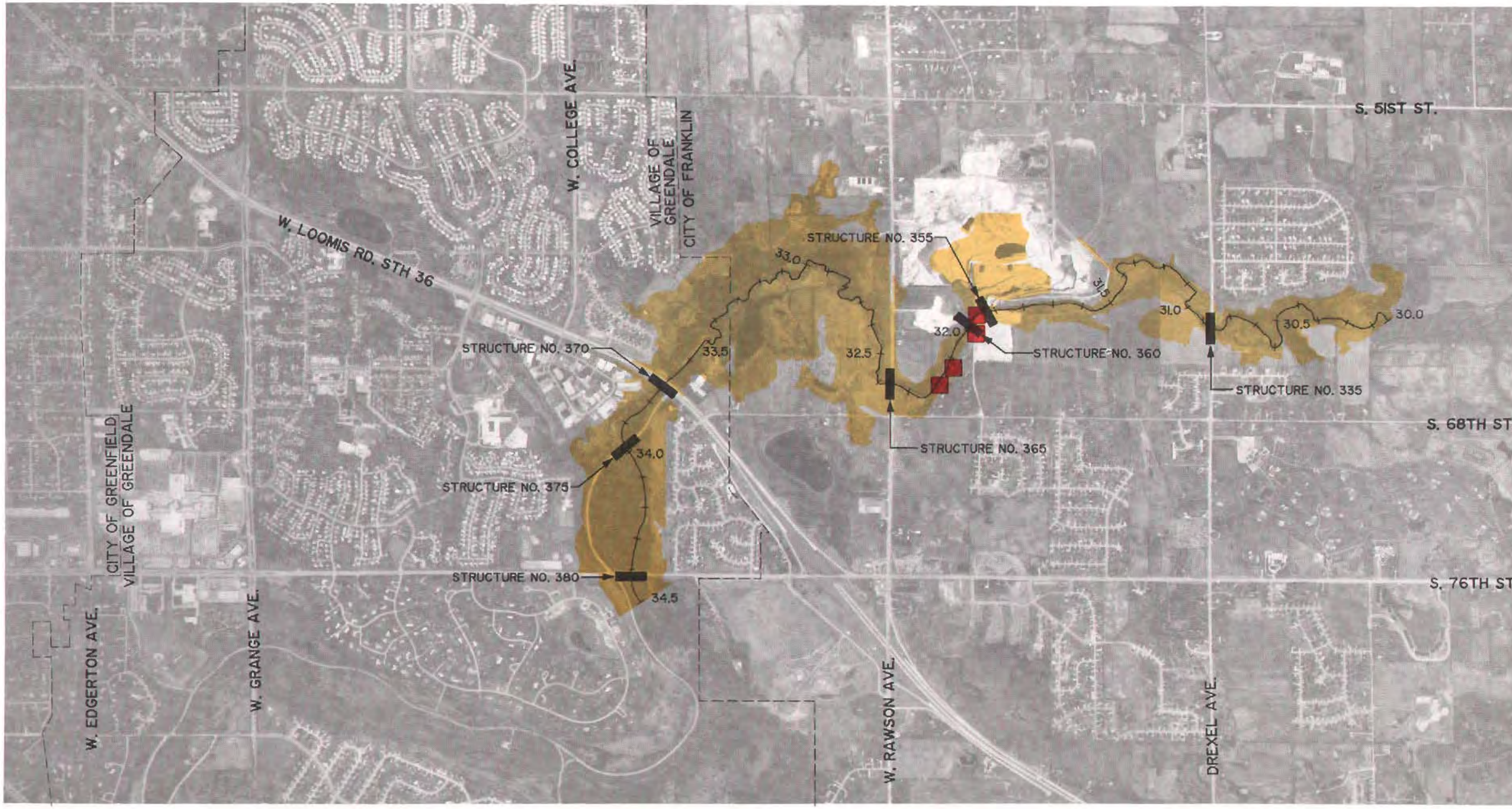
DATE OF PHOTOGRAPHY: APRIL 1986

Figure 36

RECOMMENDED PLAN FLOOD STAGE PROFILE FOR THE NORTH BRANCH ROOT RIVER



Map 88 (continued)



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS

34.5 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

STRUCTURE TO BE REMOVED

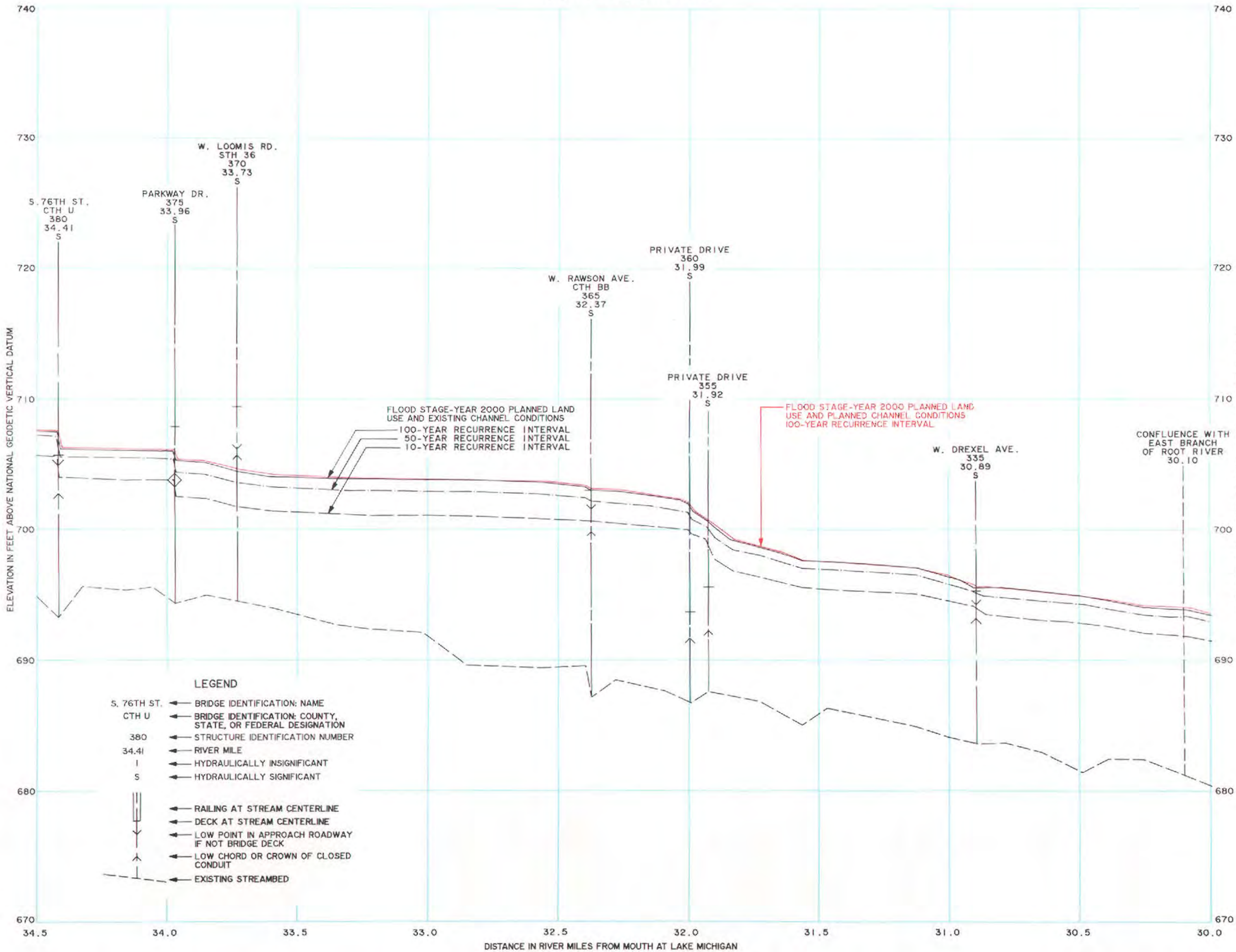
NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR NORTH BRANCH ROOT RIVER IS SHOWN IN APPENDIX H

GRAPHIC SCALE

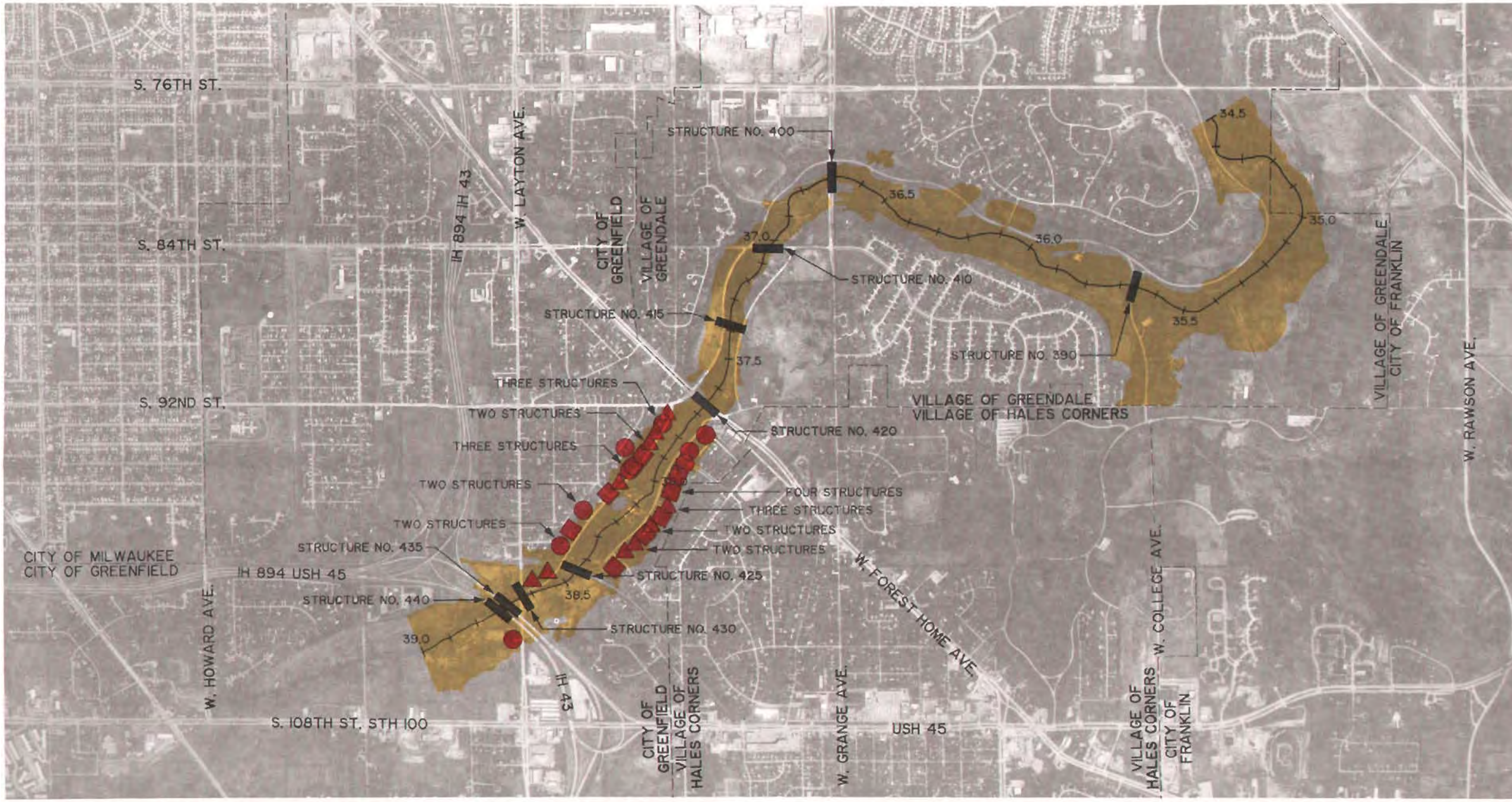
0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Figure 36 (continued)



Map 88 (continued)



LEGEND

- 100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS
- 39.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING
- STRUCTURE FLOODPROOFING
- STRUCTURE ELEVATION
- STRUCTURE TO BE REMOVED

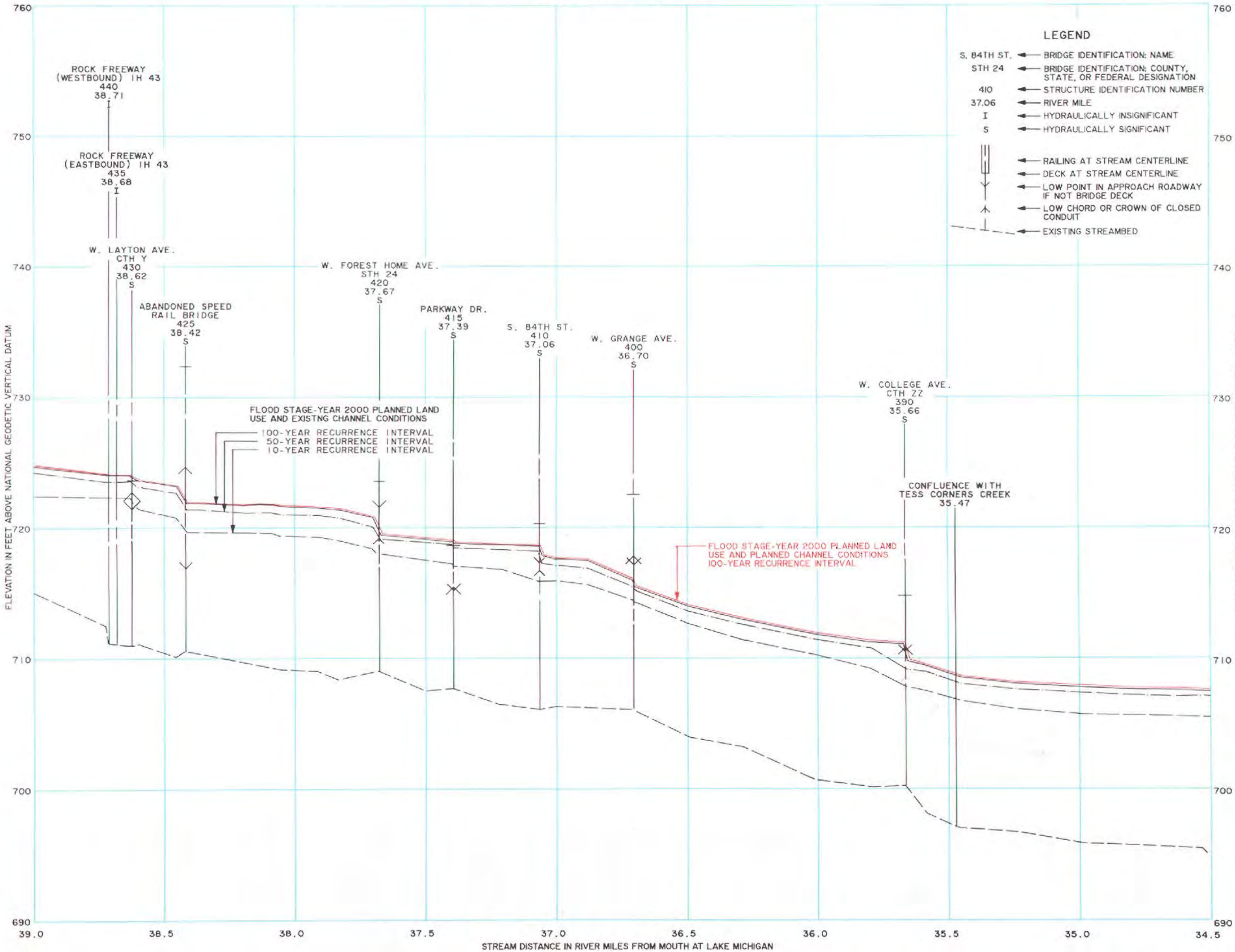
NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR NORTH BRANCH ROOT RIVER IS SHOWN IN APPENDIX H

GRAPHIC SCALE

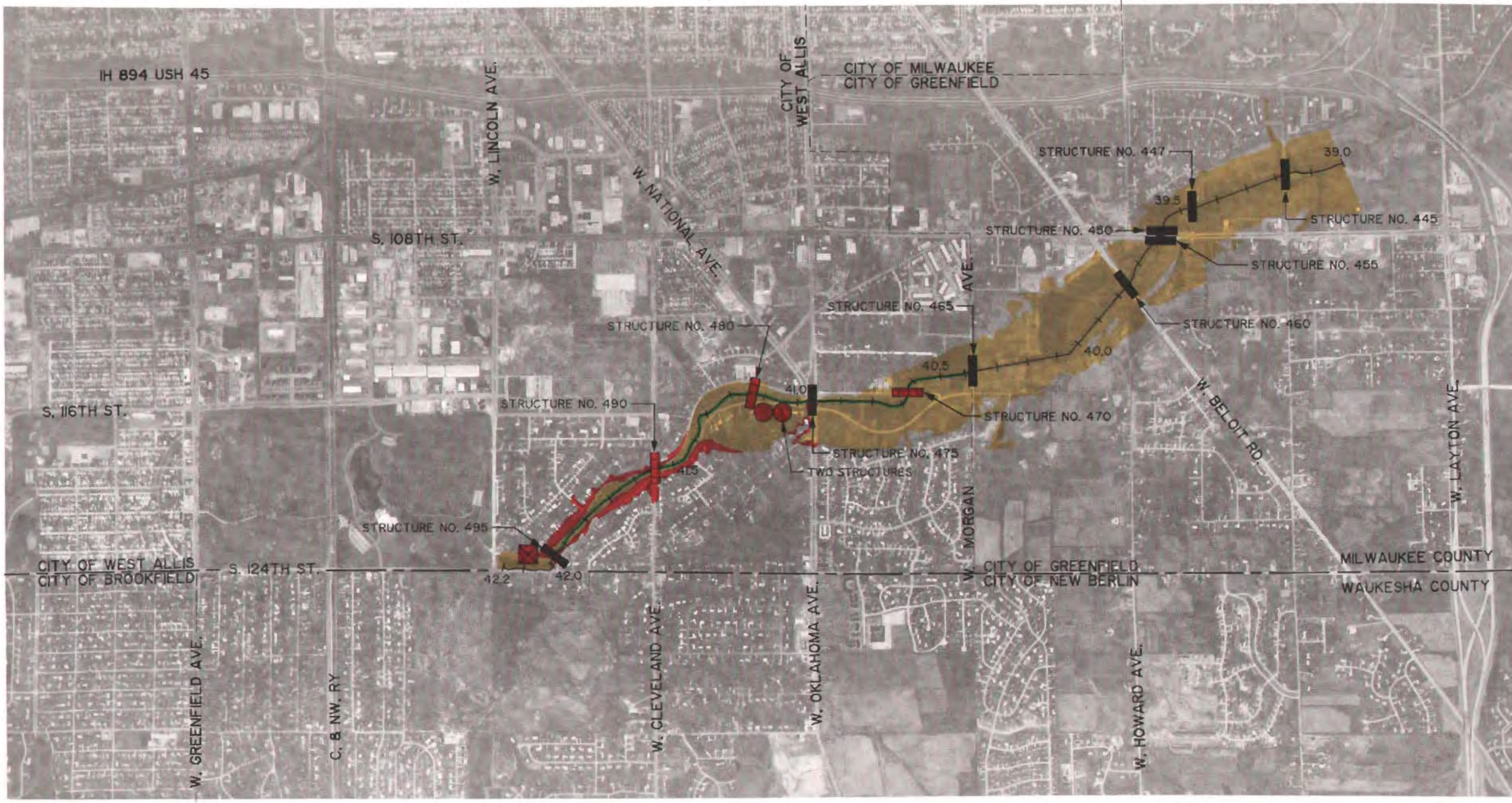
0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

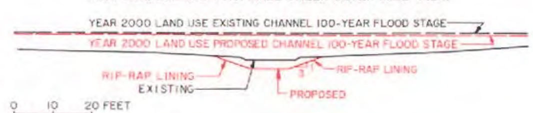
Figure 36 (continued)



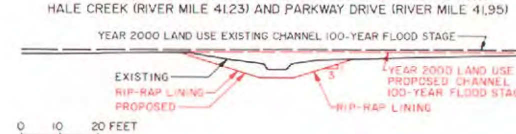
Map 88 (continued)



TYPICAL CROSS SECTION OF EXISTING AND PROPOSED CHANNEL ALONG NORTH BRANCH ROOT RIVER BETWEEN W. MORGAN AVENUE AND CONFLUENCE WITH HALE CREEK (RIVER MILE 41.23)



TYPICAL CROSS SECTION OF EXISTING AND PROPOSED CHANNEL ALONG NORTH BRANCH ROOT RIVER BETWEEN CONFLUENCE WITH HALE CREEK (RIVER MILE 41.23) AND PARKWAY DRIVE (RIVER MILE 41.95)



LEGEND

- 100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS
- 100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS
- 42.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING
- CHANNEL DEEPENING AND RESHAPING
- BRIDGE REPLACEMENT
- STRUCTURE FLOODPROOFING
- STRUCTURE TO BE REMOVED

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR NORTH BRANCH ROOT RIVER IS SHOWN IN APPENDIX H

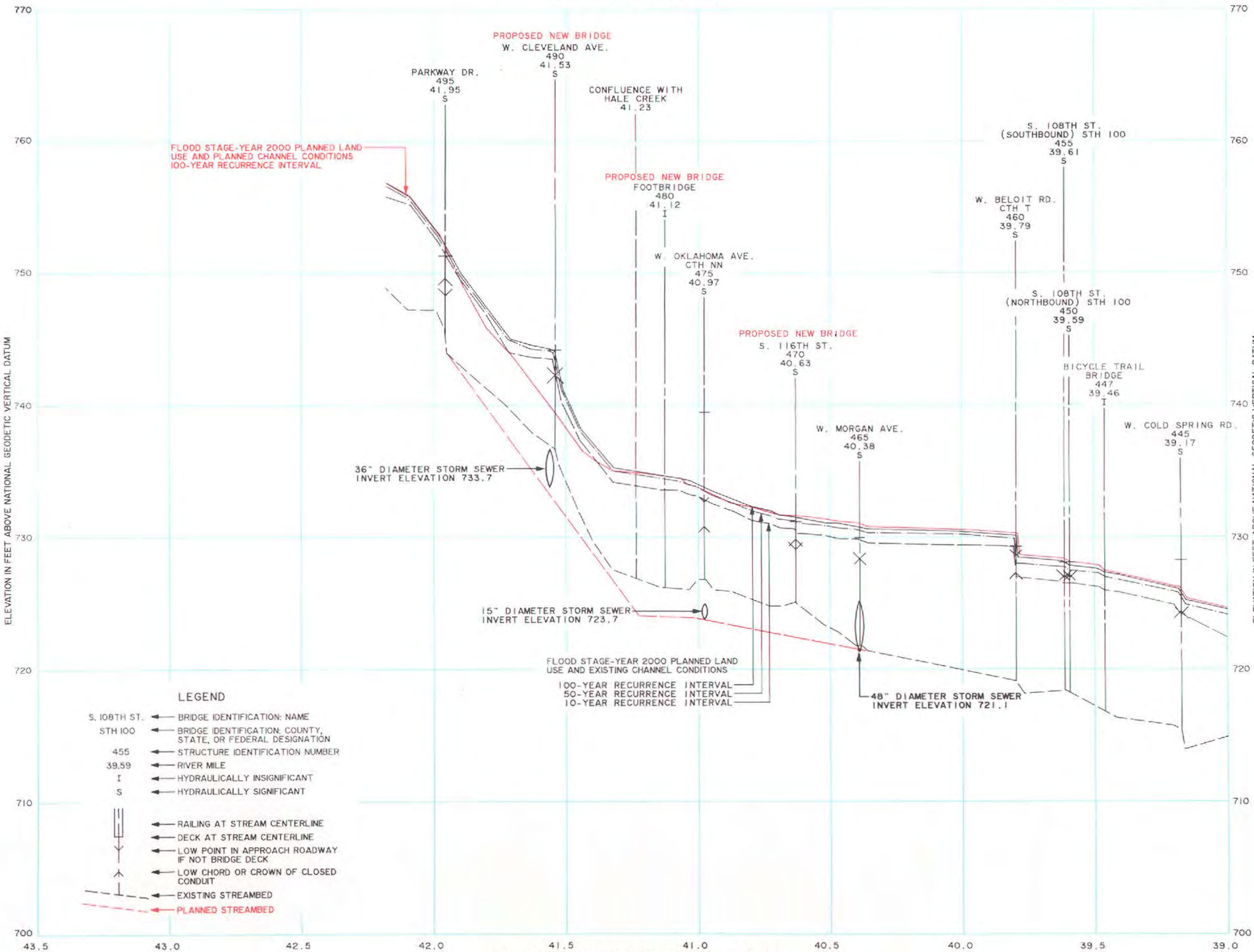
DUE TO MAP SCALE LIMITATIONS, THE DIFFERENCE BETWEEN THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS, AND THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS, MAY NOT APPEAR ON THIS MAP. WHERE NO DIFFERENCE APPEARS REFERENCE SHOULD BE MADE TO THE FLOOD STAGE PROFILE SHOWN BELOW

GRAPHIC SCALE

0 1/2 1 MILE

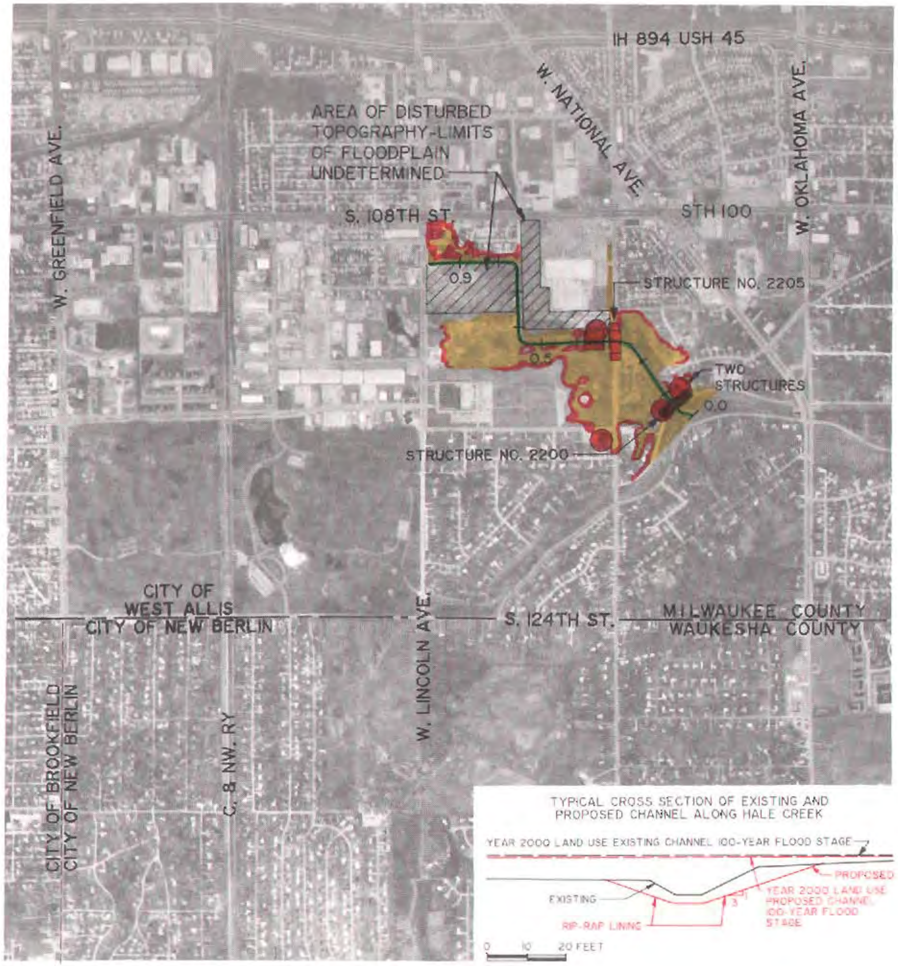
DATE OF PHOTOGRAPHY: APRIL 1986

Figure 36 (continued)



Source: SEWRPC.

Map 88 (continued)



LEGEND

- 100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS
- 100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS
- 0.5 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING
- STRUCTURE FLOODPROOFING
- STRUCTURE TO BE REMOVED
- CHANNEL DEEPENING AND RESHAPING
- BRIDGE REPLACEMENT

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR HALE CREEK RIVER IS SHOWN IN APPENDIX H

DUE TO MAP SCALE LIMITATIONS, THE DIFFERENCE BETWEEN THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS, AND THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS, MAY NOT APPEAR ON THIS MAP. WHERE NO DIFFERENCE APPEARS REFERENCE SHOULD BE MADE TO THE FLOOD STAGE PROFILE SHOWN BELOW

GRAPHIC SCALE

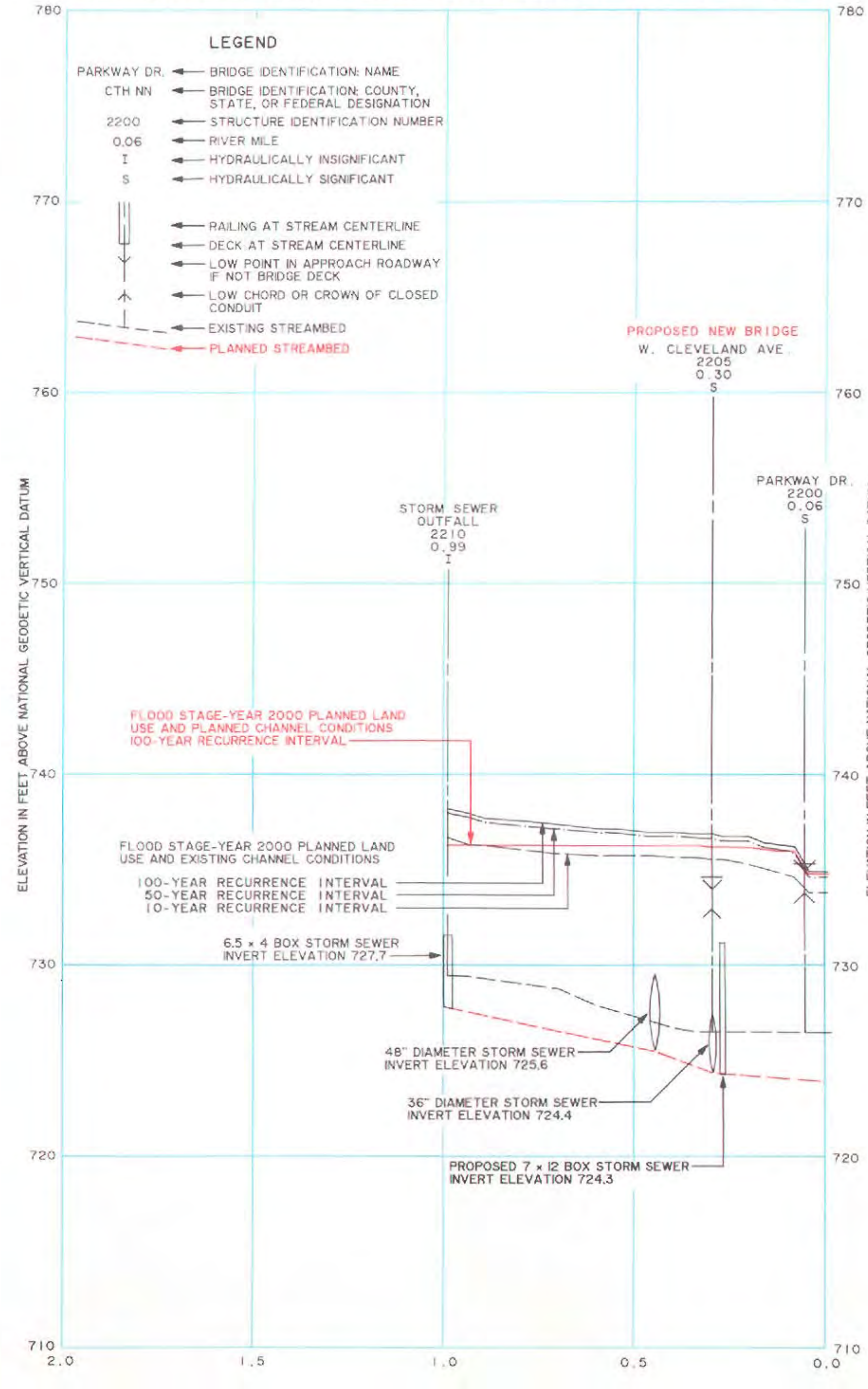
0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Source: SEWRPC.

Figure 37

RECOMMENDED PLAN FLOOD STAGE PROFILE FOR HALE CREEK



Source: SEWRPC.

Table 48

IMPACT OF RECOMMENDED FLOOD CONTROL PLAN FOR THE NORTH BRANCH ROOT RIVER AND HALE CREEK ON 100-YEAR RECURRENCE INTERVAL FLOOD DISCHARGE

Stream	Location	100-Year Recurrence Interval Flood Discharges (cubic feet per second) Year 2000 Planned Land Use			
		River Mile	Existing Channel Condition	Recommended Plan Condition	Percent Increase
North Branch of the Root River	Confluence with Root River and Root River Canal	25.66	4,900	4,900	0
	Upstream of W. Ryan Road	28.07	4,800	4,800	0
	W. Drexel Avenue	30.89	5,100	5,150	1
	W. Rawson Avenue	32.37	5,450	5,550	2
	W. College Avenue	35.66	3,350	3,430	2
	W. Forest Home Avenue	37.67	4,280	4,440	4
	W. Cold Spring Road	39.17	3,500	3,880	11
	W. National Avenue	40.97	2,000	2,840	42
	Upstream of Confluence with Hale Creek	41.25	1,410	1,510	7
Hale Creek	At Mouth	0.00	1,520	1,560	3
	W. Cleveland Avenue	0.30	580	590	2

Source: SEWRPC.

The recommended flood control plan for the North Branch of the Root River in the City of Greenfield is shown on Map 88, and consists of floodproofing 14 houses, elevating 15 houses above the flood elevation, and removing 13 houses from the floodplain. Of the 13 houses to be removed from the floodplain, it is recommended that one house along W. Root River Parkway be moved to new a location on its lot, beyond the 100-year recurrence interval floodplain, and subsequently resold. It is also recommended that moving the remaining 12 houses to new lots located in the general vicinity but beyond the 100-year recurrence interval floodplain be investigated.

The recommended flood control plan for the North Branch of the Root River in the City of Franklin is shown on Map 88, and consists of removing two houses and two commercial storage buildings. The decision to recommend removal of these four structures is based upon a recommendation made in the Commission's comprehensive plan for the Root River watershed for the development of a parkway along the entire length of the Root River between Greenfield Park in the City of West Allis and STH 38 in the City of Racine. These properties are located along the only reach of the Root River in Milwaukee County that is not currently in public ownership. The acquisition of these properties will facilitate the development of the recommended parkway. The cost of removing these four structures is estimated at $329,000.

As noted earlier, the recommended flood control plan would result in stage increases along the North Branch of the Root River of up to 0.3 foot for a 100-year recurrence interval flood under planned land use conditions. Thus, flood easements would have to be obtained from all downstream property owners affected by the stage increases. In recommending this flood control plan, the Advisory Committee recognized that some affected property owners may not be willing to grant such easements, particularly since no structural measures are recommended to control flooding of their homes. The Committee suggested several actions that could be taken by the implementing agency in order to satisfy state requirements concerning increases in the flood stage. These actions included:

1. Offer to purchase any of the affected properties that did not want to grant required easements.

2. Offer to pay the incremental cost of floodproofing and elevation measures required as a result of the stage increase.

3. Change the policy plan to provide for payment of floodproofing and elevation costs which would then be related to securing an easement.

4. Request the Wisconsin Department of Natural Resources Board to grant a waiver from the flood easement requirement.

5. Seek legislation which would change Wisconsin Administrative Code, Chapter NR 116, to remove the flood easement requirement in certain instances.

6. Seek legislation which would change Wisconsin Administrative Code, Chapter NR 116, to allow local communities and other government agencies to follow the same procedures as state agencies in such matters. Current regulations require state agencies only to inform property owners of the potential stage increase and of the owners' right to take legal action against the State. These agencies are not required to obtain flood easements.

It is recommended that the implementing agency pursue one or more of these alternatives if it encounters problems obtaining the flooding easements needed to implement the recommended plan.

It is recommended that replacement bridges for those structures shown in Appendix E as having inadequate hydraulic capacities be designed so as to pass the recommended design flood flow without overtopping of the attendant roadway. Such replacement is not required for flood control purposes, but rather should be carried out for transportation or other purposes.

Adequate large-scale topographic mapping is available for the North Branch of the Root River except for the reach between W. Rawson Avenue in the City of Franklin and W. Forest Home Avenue in the Village of Greendale. The only maps available for this reach are Milwaukee Sewerage Commission maps prepared in 1958, based on photographs taken in 1951 and 1952. These maps no longer reflect the many changes that have taken place within the floodplain and surrounding drainage area since they were prepared. Therefore, it is recommended that new large-scale topographic maps be prepared for the following U. S. Public Land Survey Sections: the west one-half of Section 2 and all of Sections 3 and 4 in the City of Franklin (Township 5 North, Range 21 East); and all except the northeast one-quarter of Section 28 and all of Section 33 in the Village of Greendale (Township 6 North, Range 21 East). Also, significant development has occurred within the floodplain of the North Branch of the Root River north of W. National Avenue and along Hale Creek in the City of West Allis since the preparation of large-scale topographic maps in 1973. Therefore, it is recommended that a new large-scale topographic map be prepared for U. S. Public Land Survey Section 7 in the City of West Allis (Township 6 North, Range 21 East).

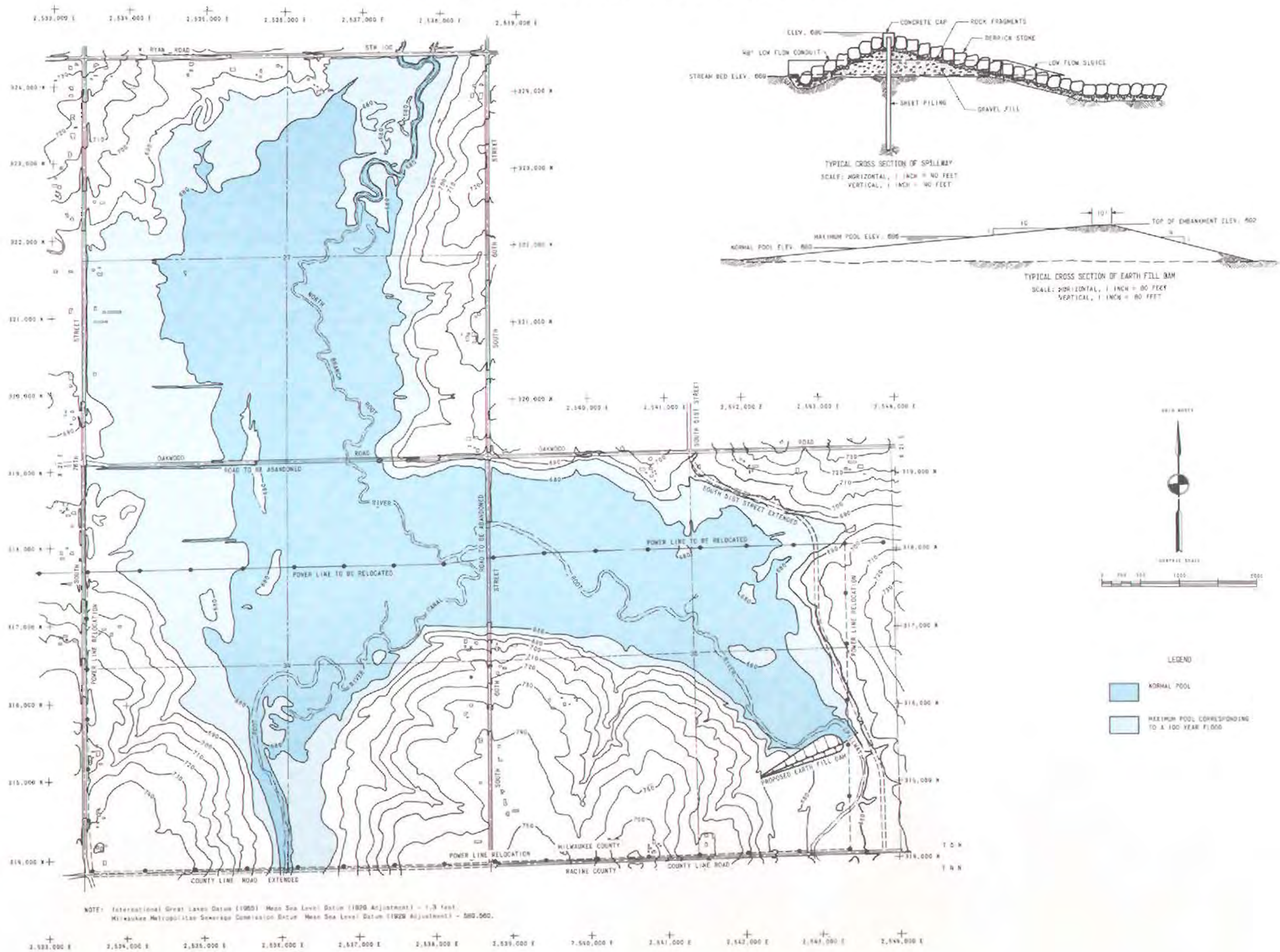
Impact of Oakwood Lake on Flood Flows and Stages

The Commission's adopted comprehensive plan for the Root River watershed recommended the development of a permanent multipurpose reservoir near the confluence of the Root River and the North Branch of the Root River in the City of Franklin. Lowlands lying in this area form a natural reservoir during flood periods, the outflow of which is regulated by a narrow cross-section of the Root River channel and floodplain near W. County Line Road. The recommended reservoir, which has been named Oakwood Lake, is shown on Map 89. This lake would artificially increase the flood regulation effect of the natural reservoir and would provide a water body for recreation, conservation, and low-flow augmentation purposes.

As proposed in the adopted Root River watershed plan, the normal water surface area of the

Map 89

GENERAL PLAN OF OAKWOOD LAKE AS PROPOSED IN THE COMPREHENSIVE PLAN FOR THE ROOT RIVER WATERSHED



Source: SEWRPC.

lake would be about 660 acres. It was proposed that about 400 acres of land underlying the lake be excavated to provide for such recreational pursuits as boating and fishing. The remaining 260 acres of lake area were envisioned to provide shallow water for fish and wildlife habitat. The normal water surface of the lake would be held between elevations of 679 feet and 680 feet above National Geodetic Vertical Datum by means of a low rock dam. Water stored between these elevations would be available for release for streamflow augmentation at a rate varying from three to five cubic feet per second (cfs), depending upon lake level. A flow of three cfs would result in a stream 24 feet wide and 6 inches deep flowing at a velocity of 0.25 foot per second. In the recreation portion of the proposed lake, a mean bottom elevation of 675 feet would be established to provide a mean water depth of four to five feet. As proposed in the plan, the lake would have a normal shoreline of about five miles. The plan envisioned that a portion of the shoreline would be developed for recreational use, with the remainder left in a natural state.

The flood control aspects of the reservoir were previously evaluated by the Commission in conjunction with work done for the U. S. Department of Housing and Urban Development as part of a flood insurance study for Racine County. That work, which involved the estimation of flood discharges and stages associated with the 100-year recurrence interval flood for the Root River, confirmed the determination of the earlier Root River watershed study that the flood control effects of the reservoir would be modest. Under that most recent work, it was estimated that the reservoir would reduce the 100-year recurrence interval peak flood stage in the City of Racine by 0.5 foot, essentially the same as the value of 0.4 foot estimated in the adopted Root River watershed plan. Consequently, as found in the original watershed study, construction of the reservoir would result in no major flood damage-abatement benefits. The reservoir would, however, provide recreational and water quality benefits. Therefore, it is recommended that development of Oakwood Lake continue to be pursued by state and local officials.

Flood Control and Related
Drainage System Plan Implementation

The recommended flood control plan for the North Branch of the Root River is largely nonstructural in that it emphasizes structure floodproofing, elevation, and removal as a means of alleviating flood damages. The structure floodproofing and elevation measures would be undertaken by the property owners directly affected. It is recommended that the City of West Allis assume the cost of removing one house located along Hale Creek. It is also recommended that the Milwaukee Metropolitan Sewerage District assume the cost of purchasing one house along the North Branch of the Root River in the City of Greenfield, relocating this house on its lot outside the 100-year recurrence interval flood hazard area, and then making it available for sale. It is recommended that the District also assume the cost of removing one house along the North Branch of the Root River in the City of West Allis, 12 houses in the City of Greenfield and two houses and two commercial buildings in the City of Franklin. Once these buildings have been removed, it is recommended that the cleared land be used for parkway or local drainage easement purposes. It is recommended that the District investigate the possibility of moving the one house in West Allis, 12 houses in Greenfield, and the two houses in Franklin to nearby lots located outside the 100-year recurrence interval floodplain. It is further recommended that the professional services required to prepare plans for the floodproofing and elevation of individual buildings be made available, at no cost, to the property owners by the engineering departments of the Cities of Greenfield and West Allis. Also, it is recommended that these communities review their building ordinances to ensure that appropriate floodproofing regulations are included.

It is recommended that structural measures recommended for the North Branch of the Root River and Hale Creek be implemented through the cooperative efforts of the Milwaukee Metropolitan Sewerage District, the City of West Allis, and Milwaukee County. More specifically, it is recommended that the District design, construct, and maintain the channel modifications recommended along the 1.6-mile reach of the North Branch of the Root River between W. Morgan Avenue and the Parkway Drive bridge at River Mile 41.95. It is also recommended that the District remove the bridges at S. 116th Street and Cleveland Avenue, and the pedestrian bridge at River Mile 41.12. Finally, it is recommended that the District prepare large-scale topographic maps for the following U. S. Public

Table 49

SUMMARY OF RECOMMENDED PLAN CAPITAL COSTS—NORTH BRANCH ROOT RIVER AND HALE CREEK

Implementing Agency	Improvements	Estimated Capital Cost
Milwaukee Metropolitan Sewerage District	Channel modification—North Branch of the Root River	$ 678,000
	Bridge removal[a]	12,000
	Removal of 17 structures	1,518,000
	Relocation of one house on lot	34,000
	Subtotal	$2,242,000
City of West Allis	Channel modification—Hale Creek	$ 456,000
	Bridge replacement[a]	88,000
	Removal of one house	90,000
	Subtotal	$ 634,000
Milwaukee County	Replacement of one pedestrian bridge	$ 18,000
	Total	$2,894,000[b]

[a]No costs have been assigned for the removal and replacement of the W. Cleveland Avenue bridges over the North Branch of the Root River and Hale Creek since these structures are scheduled for replacement under the Commission adopted regional transportation plan. Those costs are estimated at $264,000 and $179,000, respectively.

[b]Does not include $594,000 cost for structure floodproofing and elevation, which would be borne by property owners directly affected.

Source: SEWRPC.

Land Survey Sections: the west one-half of Section 2 and all of Sections 3 and 4 in Township 5 North, Range 21 East; and all of Sections 7 and 33 and all but the northeast one-quarter of Section 28 in Township 6 North, Range 21 East.

It is further recommended that the City of West Allis design, construct, and maintain the channel modifications recommended along Hale Creek, including removing the existing bridge at W. Cleveland Avenue. It is further recommended that the City design and construct the replacement bridges at S. 116th Street and W. Cleveland Avenue over the North Branch of the Root River and at W. Cleveland Avenue over Hale Creek.

Finally, it is recommended that Milwaukee County design and construct the replacement pedestrian bridge at River Mile 41.12 over the North Branch of the Root River.

The capital costs associated with the various components of the recommended plan are summarized in Table 49.

EAST BRANCH OF THE ROOT RIVER SUBWATERSHED FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN

Hydrologic and hydraulic analyses of the East Branch of the Root River were previously conducted by the Commission for the City of

Franklin. The findings of these analyses were not published, however; and alternative flood control measures were not prepared and evaluated. The East Branch of the Root River was also included in the federal flood insurance study for the City of Franklin. The hydrologic and hydraulic analyses made under that study represent a refinement of the earlier Commission study.

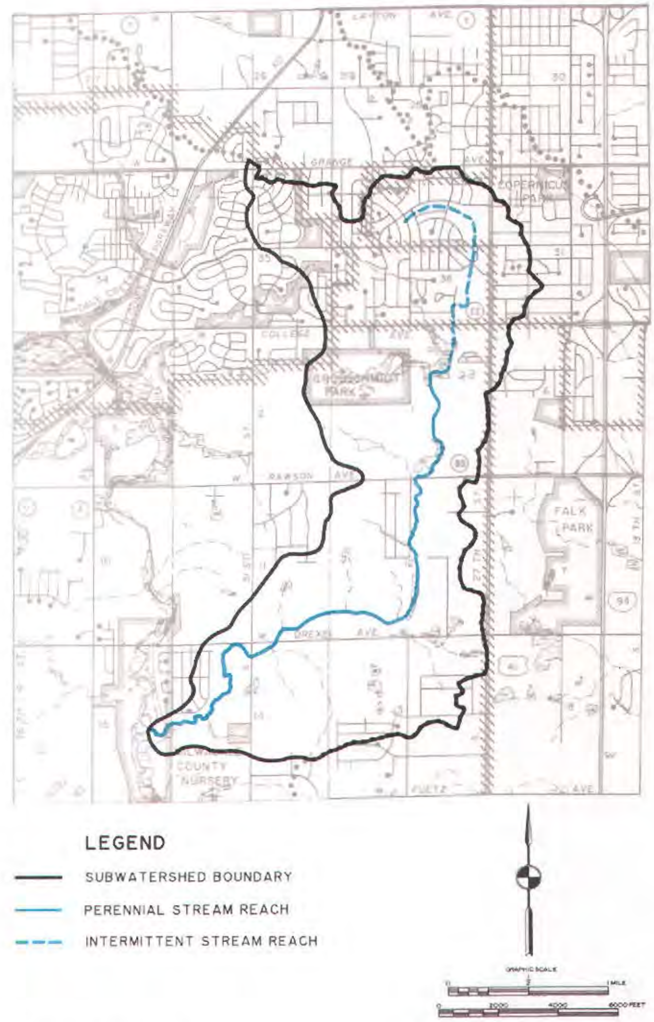
Overview of the Study Area

The East Branch of the Root River subwatershed is located mainly in the City of Franklin, with small portions located in the Cities of Greenfield, Milwaukee, and Oak Creek, and the Village of Greendale. From its origin near the intersection of S. Melinda Street and W. Parnell Avenue in the City of Milwaukee, the East Branch of the Root River flows in a generally southwesterly direction for about 6.0 miles to its confluence with the North Branch of the Root River, and drains an area of about 4.83 square miles (see Map 90). Of this total drainage area, 3.31 square miles, or about 69 percent, lies within the City of Franklin; 0.59 square mile, or about 12 percent, lies within the City of Greenfield; 0.55 square mile, or about 11 percent, lies within the City of Milwaukee; 0.05 square mile, or about 1 percent, lies within the City of Oak Creek; and 0.33 square mile, or about 7 percent, lies within the Village of Greendale.

More specifically, from its origin near the intersection of S. Melinda Street and W. Parnell Avenue, the East Branch of the Root River flows in an easterly direction to W. Green Avenue extended, a distance of about 0.4 mile; thence southerly for about 3.0 miles to N. 35th Street extended; thence westerly for about 0.9 mile to W. Drexel Avenue; and thence southwesterly about 1.7 miles to its confluence with the North Branch of the Root River. Of the 6.0-mile length of reach described, 4.7 miles south of W. Carrington Avenue extended, or 78 percent, are classified as perennial, while the remaining 1.3 miles, or 22 percent, are classified as intermittent. The entire perennial stream length is recommended for District jurisdiction in the policy plan companion to this system plan. Hydraulic analyses were also conducted under this system planning effort of the 0.2-mile segment of intermittent stream extending from W. Carrington Avenue extended to W. College Avenue. Reported flooding problems along this reach made it necessary to consider flood control

Map 90

EAST BRANCH ROOT RIVER WATERSHED



Source: SEWRPC.

alternatives, since such measures may impact on required downstream flows and stages and flood control measures.

In 1985, the subwatershed tributary to the East Branch of the Root River was still largely undeveloped for urban use, with 2.96 square miles, or about 61 percent, devoted to agricultural and other open uses. The remaining 1.87 square miles was developed for urban use, with 1.73 square miles, or about 92 percent of the developed portion, consisting of residential land uses, and the remaining 0.14 square mile consisting of commercial and industrial uses. Much of the developed area of the subwatershed was located in the headwater area, and was generally provided with a full range of municipal street improvements, including paved streets with curbs and gutters and attendant storm sewers which convey surface runoff rapidly to the stream. In the City of Franklin, most of the surface runoff is accommodated in roadside ditches and other small tributary watercourses, and therefore does not reach the main channel as rapidly as runoff from the headwater area.

Information on certain pertinent characteristics of the watershed, such as hydrologic soil types, land slopes, and land use, is provided in Chapter II of this report. The planned land use conditions utilized in the system planning effort assume that the entire subwatershed tributary to the East Branch of the Root River will be fully urbanized by the design year of the system plan. Some existing open space uses, such as parks, should remain.

The East Branch of the Root River north of W. College Avenue has been channelized to accommodate the increased runoff from the tributary portion of the subwatershed. In the City of Franklin, the stream remains "unimproved," although it is apparent that much of the channel has been realigned and straightened over the years since settlement of the area by Europeans for agricultural purposes.

Flooding and Related Drainage Problems

Investigations of historical flood problems along the East Branch of the Root River indicate that few significant problems exist within the subwatershed. This is due primarily to the relatively undeveloped character of the subwatershed. Flood problems are limited primarily to the area along two reaches of the stream: 1) an area of residential development along N. 35th Street south of W. Rawson Avenue; and 2) within the Franklin Mobile Estates mobile home court located south of W. College Avenue at S. 27th Street. This latter area is located upstream of the segment recommended for District jurisdiction. Damage to crops which has occurred owing to overland flooding may be expected to diminish as the subwatershed develops for urban use.

The monetary flood damages within the subwatershed were estimated using the depth-damage cost curves prepared by the Commission and described in Chapter III. The dollar amount of the flood damages is based upon the depth of inundation and the assessed valuation of the buildings concerned. Damages to building contents were included in the total costs.

Flooding, as defined herein, includes basement flooding, yard inundation, and flooding above the first-floor level. The total number of residences existing in 1985 that may be expected to experience direct flooding along the East Branch of the Root River is as follows:

Flood Event Recurrence Interval	Approximate Number of Existing Homes Flooded Existing Land Use and Existing Channel Conditions	Approximate Number of Existing Homes Flooded Planned Land Use and Existing Channel Conditions
10	6	10
50	16	18
100	18	19

The number of industrial and commercial properties existing in 1985 that may be expected to experience direct flooding along the East Branch of the Root River is as follows:

Flood Event Recurrence Interval	Approximate Number of Existing Industrial and Commercial Properties Flooded Existing Land Use and Existing Channel Conditions	Approximate Number of Existing Industrial and Commercial Properties Flooded Planned Land Use and Existing Channel Conditions
10	0	0
50	1	1
100	1	1

Additional homes and commercial properties may, however, experience indirect damages through sanitary sewer backup. It should be noted that the flood control measures considered under this system plan are primarily intended to alleviate flood damages from direct overland flooding along the stream reaches studied, as well as to provide an adequate outlet for local storm sewers and drainageways. These measures may help to reduce problems attendant to inadequate local stormwater drainage and problems of sanitary sewer backups.

The total average annual flood losses—damages—incurred along the East Branch of the Root River are estimated at $5,000 under existing land use and channel conditions, and at $8,100 under planned land use and existing channel conditions. Flood losses from a 100-year recurrence interval event are estimated to total $89,000 under existing land use and channel conditions, and about $100,000 under planned land use and existing channel conditions.

The drainage and flood control objectives and supporting principles and standards set forth in Chapter III specify the flood events which bridges shall accommodate without overtopping of the related roadway. Based on these criteria, four bridges on the East Branch of the Root River are considered hydraulically inadequate, as shown in Appendix E. These bridges are at S. 51st Street; W. Drexel Avenue; W. Rawson Avenue; and W. College Avenue.

Flood Discharges and Stages

As noted in Chapter III of this report, the hydrologic model used for development of design discharges for the East Branch of the Root River simulates streamflow on a continuous basis, using recorded climatological data as input. Discharges were computed at an hourly time interval. Flood discharges were developed by conducting discharge-frequency analyses of simulated annual peak discharges generated by the hydrologic model using the log Pearson Type III method of analysis. Because of the relatively small tributary drainage area of this subwatershed, it was suspected that the time of peak discharge on the stream was very short and may have been missed in the analyses utilizing an hourly time interval. Therefore, additional simulations were performed using a 15-minute time interval with design rainfall events as input. The use of design rainfall events was necessary because the time and cost of simulating continuous streamflows at 15-minute intervals for the 39 years of available climatological data would be prohibitive.

The design rainfall events were developed using 10-, 50-, and 100-year rainfall volumes obtained from the updated point rainfall depth-duration-frequency relationships developed by the Commission as described in Chapter III. The rainfall distribution utilized for each design storm was the median distribution of a first-quartile storm, as shown in Chapter III. The design storm duration was determined for a given recurrence interval by simulating the peak discharge at a given location for a range of storm durations. The storm duration and associated rainfall volume which produced the largest peak discharge at a given location for a given recurrence interval was selected as the design storm for that location. This analysis was conducted for both existing and planned land use and existing channel conditions at five locations along the East Branch of the Root River. The estimated peak flood discharges under existing and planned, year 2000 land use conditions and existing channel conditions are set forth in Table 50.

Flood stage profiles were determined for the 10-, 50-, and 100-year recurrence interval runoff events under planned land use and existing channel conditions. These profiles, which encompass the full 4.7-mile-long reach recommended for District jurisdiction, constitute a graphic representation of the flood stages along the East Branch of the Root River under the specified recurrence interval flood discharges, and under planned land use and existing channel conditions. In addition to providing an overall representation of flood stages relative to familiar points of reference such as the channel bottom and bridge deck surfaces, the profiles, because of their continuity, permit the ready determination of flood stages at any point along the stream channel. The flood profile is shown in Figure 38. The attendant extent of the 100-year recurrence interval floodplain under planned land use conditions is shown on Map 91. This delineation of the flood hazard area was accomplished using large-scale topographic maps prepared by the City of Franklin in 1964.

Alternative Flood Control and Related Drainage System Plans for the East Branch of the Root River

Three alternative flood control plans were considered for alleviating the flood damage problems along the East Branch of the Root River: 1) No Action; 2) Structure Floodproofing, Elevation, and Removal; and 3) Combination of Channel Modification, Bridge Replacement, and Structure Floodproofing.

Each alternative is described below. The estimated economic benefits and costs attendant to each alternative are provided in Table 51.

Alternative 1—No Action: One alternative course of action for addressing the flood problem along the East Branch of the Root River is to do nothing—that is, to recognize the inevitability of flooding, but to deliberately decide not to mount a collective, coordinated program to abate the flood damages. Under existing land use and existing channel conditions, the average annual flood damages along the stream approximate \$5,000. The damages from a 100-year recurrence interval flood may be expected to approximate \$89,000. Under planned, year 2000 land use and existing channel conditions, the average annual flood damages along this reach may be expected to approximate \$8,100. The damages from a 100-

Table 50

FLOOD DISCHARGES FOR THE EAST BRANCH ROOT RIVER FOR EXISTING AND YEAR 2000 LAND USE AND EXISTING CHANNEL CONDITIONS

Location	River Mile	Peak Flood Discharge (cubic feet per second)					
		Existing Land Use Existing Channel Conditions			Year 2000 Planned Land Use, Existing Channel Conditions		
		10-Year	50-Year	100-Year	10-Year	50-Year	100-Year
Mouth at Root River	0.00	430	950	1,160	700	1,280	1,490
N. 51st Street	1.48	430	950	1,160	700	1,280	1,490
- -	2.21	420	910	1,100	610	1,160	1,390
- -	3.14	320	660	810	460	860	1,010
W. Rawson Avenue	3.66	300	630	770	440	820	960
- -	4.70	260	430	510	260	430	510

Source: SEWRPC.

year recurrence interval flood may be expected to approximate $100,000. There are no monetary benefits associated with this alternative, and the average annual cost would be equivalent to the average of the existing and planned land use average annual costs, or $6,600.

Alternative 2—Structure Floodproofing, Elevation, and Removal: A structure floodproofing, elevation, and removal alternative was evaluated to determine if such a structure-by-structure approach would be a technically feasible and economically viable solution to the flood problem along the East Branch of the Root River. The 100-year recurrence interval flood stage under planned, year 2000 land use and existing channel conditions was used to estimate the number of existing structures to be floodproofed, elevated, or removed and to estimate the costs involved.

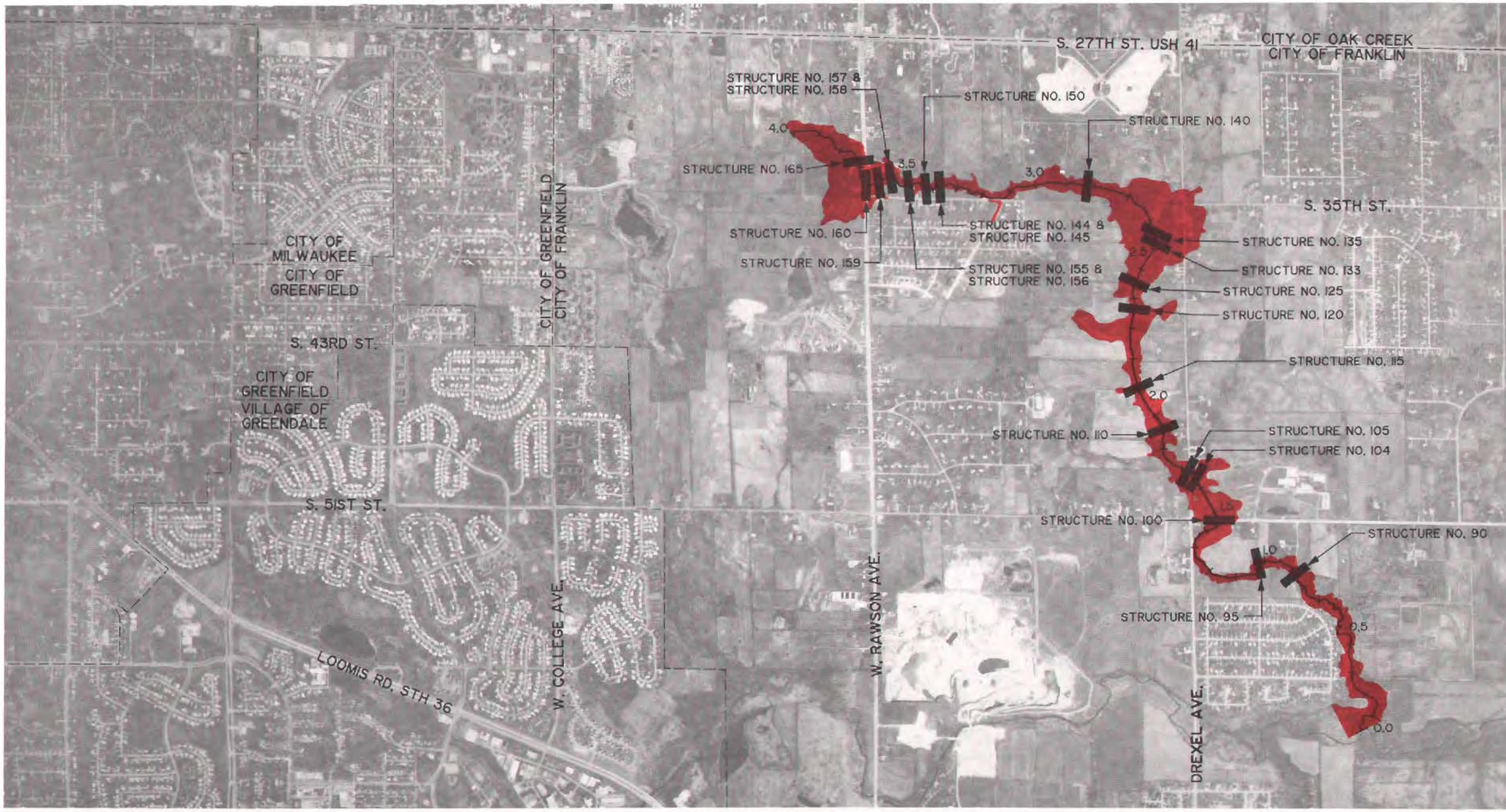
In the case of residential structures, floodproofing was assumed to be feasible if the design flood stage was below the first-floor elevation. Structure elevation was considered feasible for residential structures with basements if the estimated cost of elevating the structure and floodproofing the basement was less than the estimated removal cost. Structures to be elevated were assumed to have the first floor raised to an elevation at least two feet above the 100-year recurrence interval flood stage to provide adequate freeboard. For aesthetic reasons, structure elevation was limited to a maximum of four feet. Structures that would have to be elevated more than four feet were considered for removal. In the case of mobile homes, it was assumed that these structures would be relocated out of the floodplain.

Floodproofing was considered to be feasible for all nonresidential structures provided the flood stage was not more than seven feet above the first-floor elevation. The floodproofing costs were assumed to be a function of the depth of the water over the first floor.

As shown on Map 92, of the 20 structures which may be expected to incur flood damage, five would have to be floodproofed, three would have to be elevated, and 12 would have to be removed. These 12 structures consist of mobile homes which would be readily relocated to lots outside the 100-year recurrence interval floodplain, either within the same mobile home court or to

Map 91

100-YEAR RECURRENCE INTERVAL FLOODPLAIN FOR THE EAST BRANCH ROOT RIVER UNDER YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

4.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR EAST BRANCH ROOT RIVER IS SHOWN IN APPENDIX H

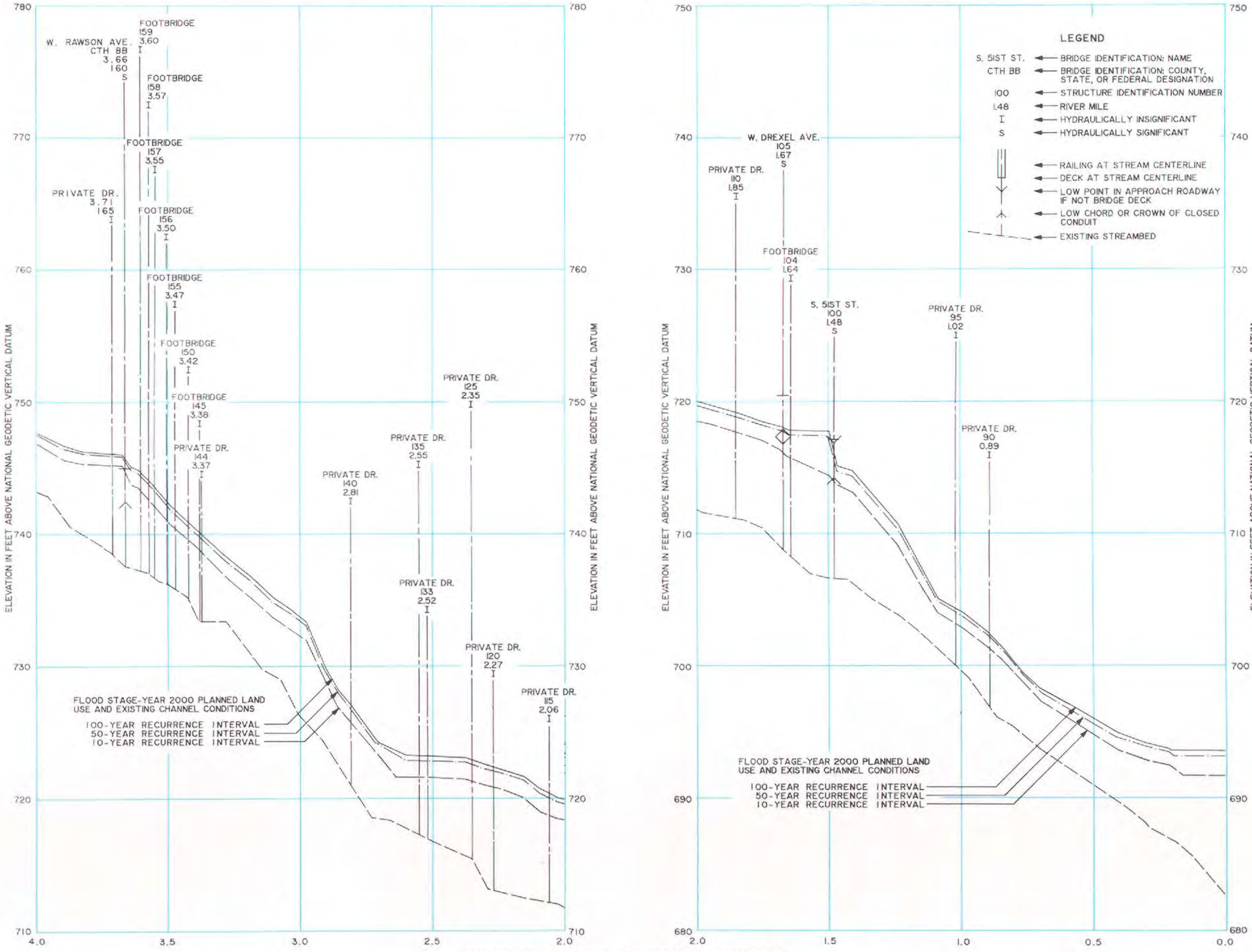
GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Figure 38

FLOOD STAGE AND STREAMBED PROFILE FOR THE EAST BRANCH ROOT RIVER



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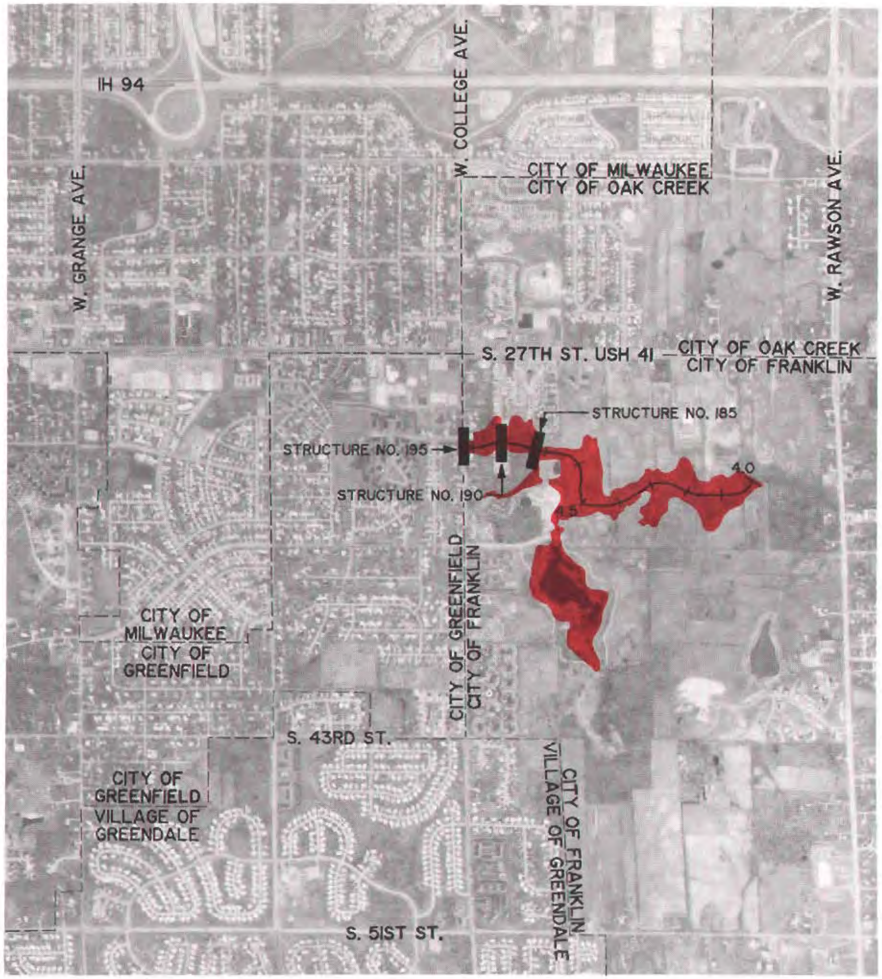
Map 91 (continued)



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

4.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR EAST BRANCH ROOT RIVER IS SHOWN IN APPENDIX H

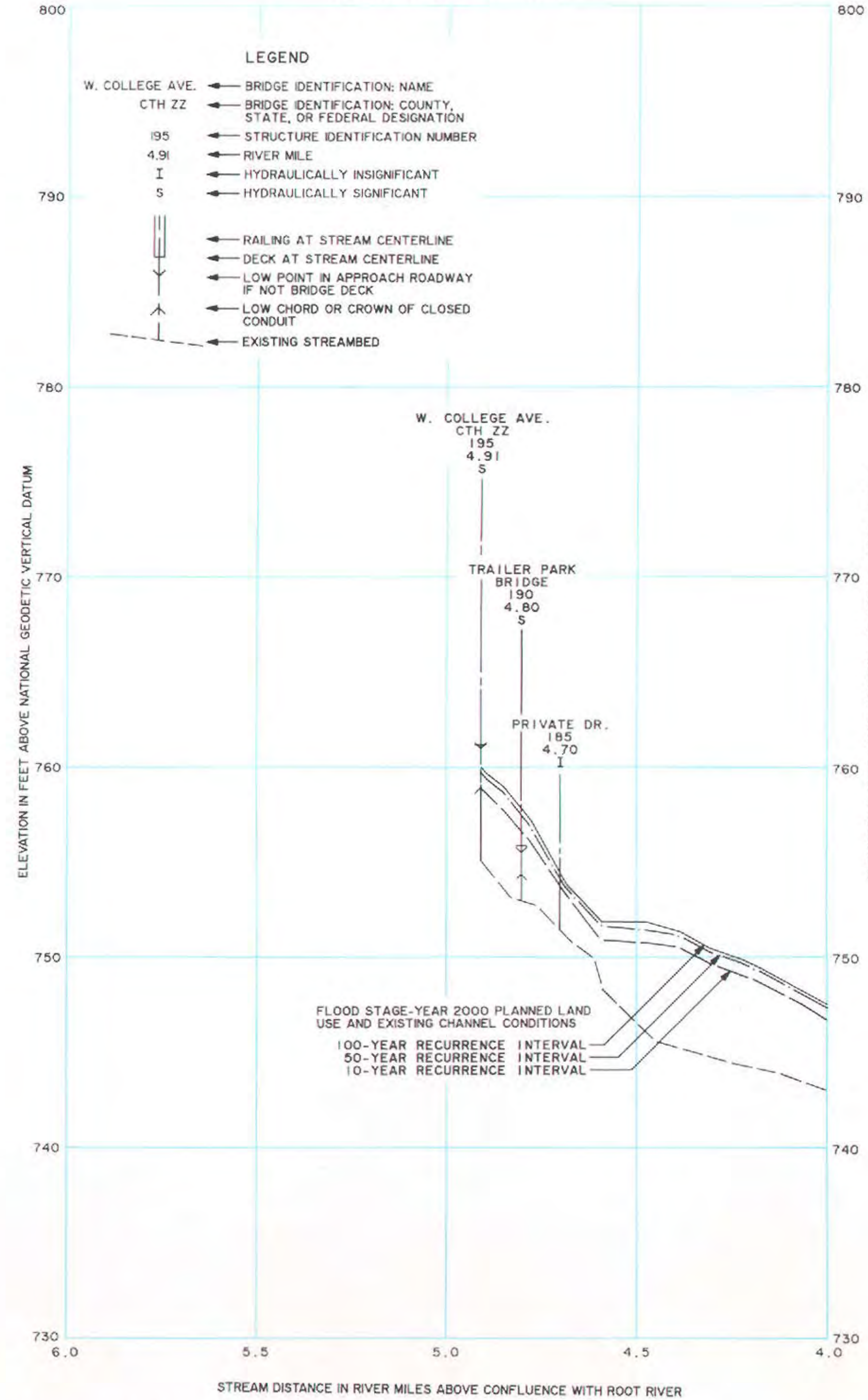
GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Source: SEWRPC.

Figure 38 (continued)



Source: SEWRPC.

Table 51

COST ESTIMATE FOR FLOOD CONTROL ALTERNATIVES FOR THE EAST BRANCH ROOT RIVER

Alternative		Costs					Benefit-Cost Analysis			
			Annual							
Name	Description	Capital	Amortized Capital[a]	Operation and Maintenance	Other	Total	Annual Benefits	Annual Benefits Minus Annual Costs	Benefit-Cost Ratio	Economic Ratio Greater than One
1—No Action	- -	$ 0	$ 0	$ 0	$6,600	$ 6,600	$ 0	$ -6,600	- -	No
2—Structure Floodproofing, Elevation, and Removal	Floodproof five structures	$ 39,000	$13,900	$ 0	$ 0	$13,900	$6,600	$ -7,300	0.47	No
	Elevate three structures	96,000								
	Relocate 12 structures	84,000								
	Subtotal	$219,000								
3—Combination of Channel Modification, Bridge Replacement, and Structure Floodproofing	1.77 miles of channel modification	$392,000	$36,000	$3,600	$ 0	$39,600	$6,600	$-33,000	0.17	No
	Replace three bridges	48,000[b]								
	Modify eight footbridges	54,000								
	Floodproof two structures and elevate one structure	69,000								
	Subtotal	$563,000								

[a]*Amortized capital cost is based on an interest rate of 6 percent and a project life of 50 years.*

[b]*Cost of the W. Rawson Avenue bridge replacement is not included as that cost was previously assigned under the Commission's adopted regional transportation plan.*

Source: SEWRPC.

nearby courts. Future damage from floods up to and including the 100-year recurrence interval event would be virtually eliminated.

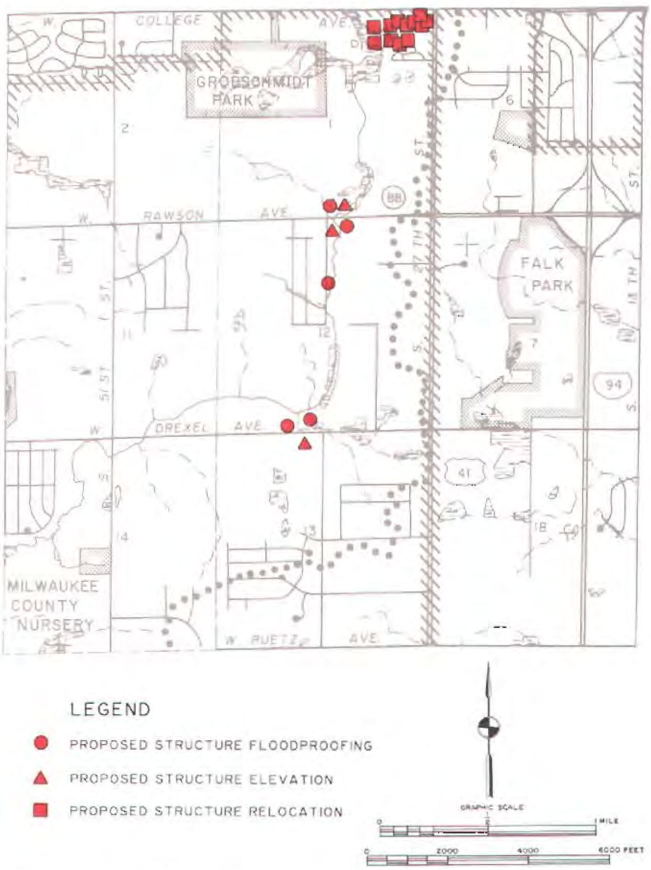
Assuming that these structure floodproofing measures would be fully implemented, and utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of this alternative is estimated at $13,900. This cost consists of the amortization of the $219,000 capital cost—$39,000 for floodproofing, $96,000 for structure elevation, and $84,000 for structure removal. The average annual flood damage abatement benefit was estimated at $6,600, yielding a benefit-cost ratio of 0.47.

Alternative Plan 3—Combination of Channel Modification, Bridge Replacement, and Structure Floodproofing: This alternative for the resolution of the flood problems along the East Branch of the Root River is shown on Map 93. The plan consists of lowering the existing streambed by 0.1 foot to 3.2 feet along the 1.77-mile reach between River Mile 3.14 and W. College Avenue. Between River Mile 3.14 and 3.60, the channel would be turf-lined and would have a bottom width of 10 feet and side slopes of one on three. Between River Mile 3.60 and W. Rawson Avenue at River Mile 3.66, the channel would be fully concrete-lined and would have a bottom width of 15 feet and side slopes of one on two. In order to control channel erosion in the transition from concrete to turf lining, riprap would be used to line about 50 feet of channel downstream of River Mile 3.60. Between W. Rawson Avenue and River Mile 3.74, the channel would be fully concrete-lined and would have a bottom width of 15 feet and side slopes of one on three. From River Mile 3.74 to W. College Avenue, the channel would be turf-lined and would have a bottom width of 10 feet and side slopes of one on three.

In addition to channel modification, this plan calls for replacing the bridges at W. Rawson Avenue, a private drive at River Mile 3.71, and

Map 92

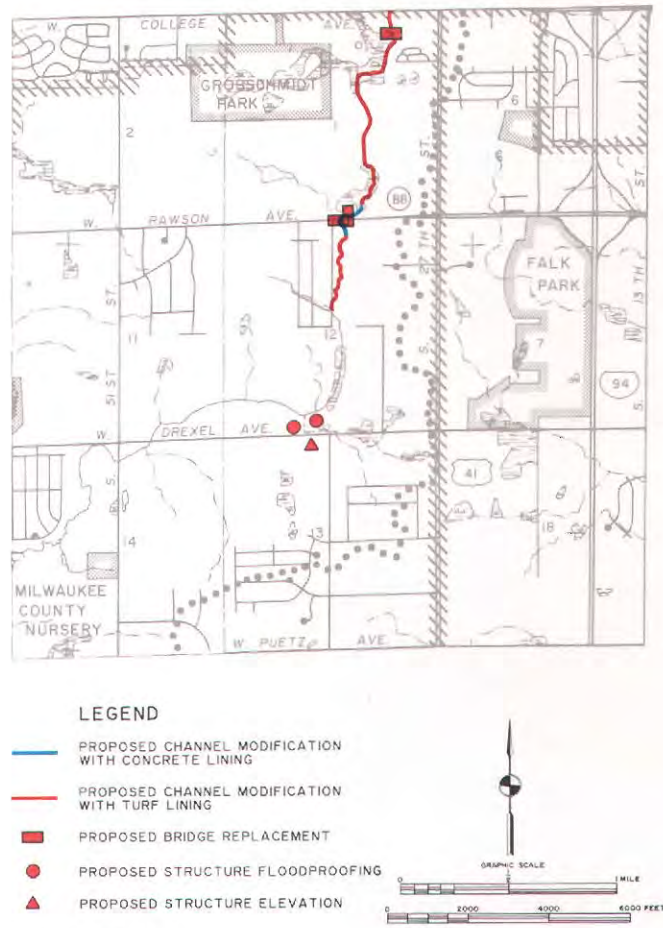
ALTERNATIVE PLAN 2: STRUCTURE FLOODPROOFING, ELEVATION, AND REMOVAL ALONG THE EAST BRANCH ROOT RIVER



Source: SEWRPC.

Map 93

ALTERNATIVE PLAN 3: COMBINATION OF CHANNEL MODIFICATION, BRIDGE REPLACEMENT, AND STRUCTURE FLOODPROOFING ALONG THE EAST BRANCH ROOT RIVER



Source: SEWRPC.

a private drive at River Mile 4.80. At W. Rawson Avenue, the replacement structure would consist of two 12-foot-wide by 8-foot-high reinforced concrete box culverts. The replacement structure for the private drive at River Mile 3.71 would consist of two 12-foot-wide by 6-foot-high reinforced concrete box culverts. The replacement structure for the private drive at River Mile 4.80 would consist of two 8-foot-wide by 5-foot-high reinforced concrete box culverts. In addition to these three culverts, modifications may be required for eight private footbridges located between River Mile 3.37 and W. Rawson Avenue in order to accommodate the channel modifications. It should be noted that W. Rawson Avenue is recommended for improvement under the Commission's adopted regional transportation system plan; therefore, no cost was assigned to the flood control system plan for these replacements.

In order to accommodate the widened channel, it would be necessary to relocate four mobile homes. Finally, this alternative calls for the floodproofing of two houses and the elevation of one house.

Implementation of this alternative would essentially eliminate all damages attendant to floods along the East Branch of Root River up to and including the 100-year recurrence interval event.

Utilizing an annual interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this alternative is estimated at $39,600. This cost consists of the amortization of the $563,000 capital cost—$392,000 for channel modification; $102,000 for bridge replacement and modification; and $69,000 for structure floodproofing, elevation, and relocation—and $3,600 in annual operation and maintenance costs. The average annual flood abatement benefit is estimated at $6,600, resulting in a benefit-cost ratio of 0.17.

Evaluation of Flood Control Alternatives for the East Branch of the Root River

The principal features of, and the costs and benefits associated with, each of the floodland management alternatives considered for the East Branch of the Root River are summarized in Table 51. All of the alternatives considered were found to be technically feasible.

None of the alternatives produces a benefit-cost ratio of one or more. The "no action" alternative, while offering the lowest cost, does nothing to alleviate the existing flood problem, and therefore does not represent a sound approach to flood control.

Alternative 2—Structure Floodproofing, Elevation, and Removal—presents several problems in implementation. First, complete implementation of a voluntary structure and elevation program is unlikely, and with partial implementation, the City of Franklin would be left with a residual problem whenever a major flood event occurred. Also, yard damages and cleanup costs would remain under this alternative. It should be noted, however, that in some instances a structure floodproofing, elevation, and removal alternative may be a viable solution to a flooding problem. Such would be the case where structure damages are relatively low and are more scattered along a stream. Structural measures, such as channel modification or detention storage, may not present an economical solution in those instances.

Implementation of Alternative 3—Combination of Channel Modification, Bridge Replacement, and Structure Floodproofing—would provide an added flood control benefit in that it would eliminate the overtopping of W. Rawson Avenue and two private access drives for floods up to and including the 100-year recurrence interval event. However, the high cost of this alternative relative to damages makes this alternative impractical. In addition, the channel modifications to be provided under this alternative, particularly those calling for a concrete lining, would serve to eliminate the "natural" look of the stream. These modifications would serve to increase downstream flood flows on the East Branch of the Root River, although not enough to cause additional flooding problems.

Recommended Flood Control System for the East Branch of the Root River

Based upon consideration of the technical feasibility, economic viability, environmental impacts, potential public acceptance, and practicality of each of the alternatives considered, it is recommended that Alternative 2—Structure Floodproofing, Elevation, and Removal—be adopted for the East Branch of the Root River.

In July and August of 1989, the Milwaukee Metropolitan Sewerage District conducted a field survey of buildings affected by the recommended plan. Data obtained through that survey were used to refine the recommended plan. Those refinements are included in the plan costs and description given below.

The total capital cost of the recommended flood control plan is estimated at $153,000 in 1986 dollars. The recommended plan is shown on Map 94. The recommended plan should have no impact on flood flows and stages along the East Branch of the Root River.

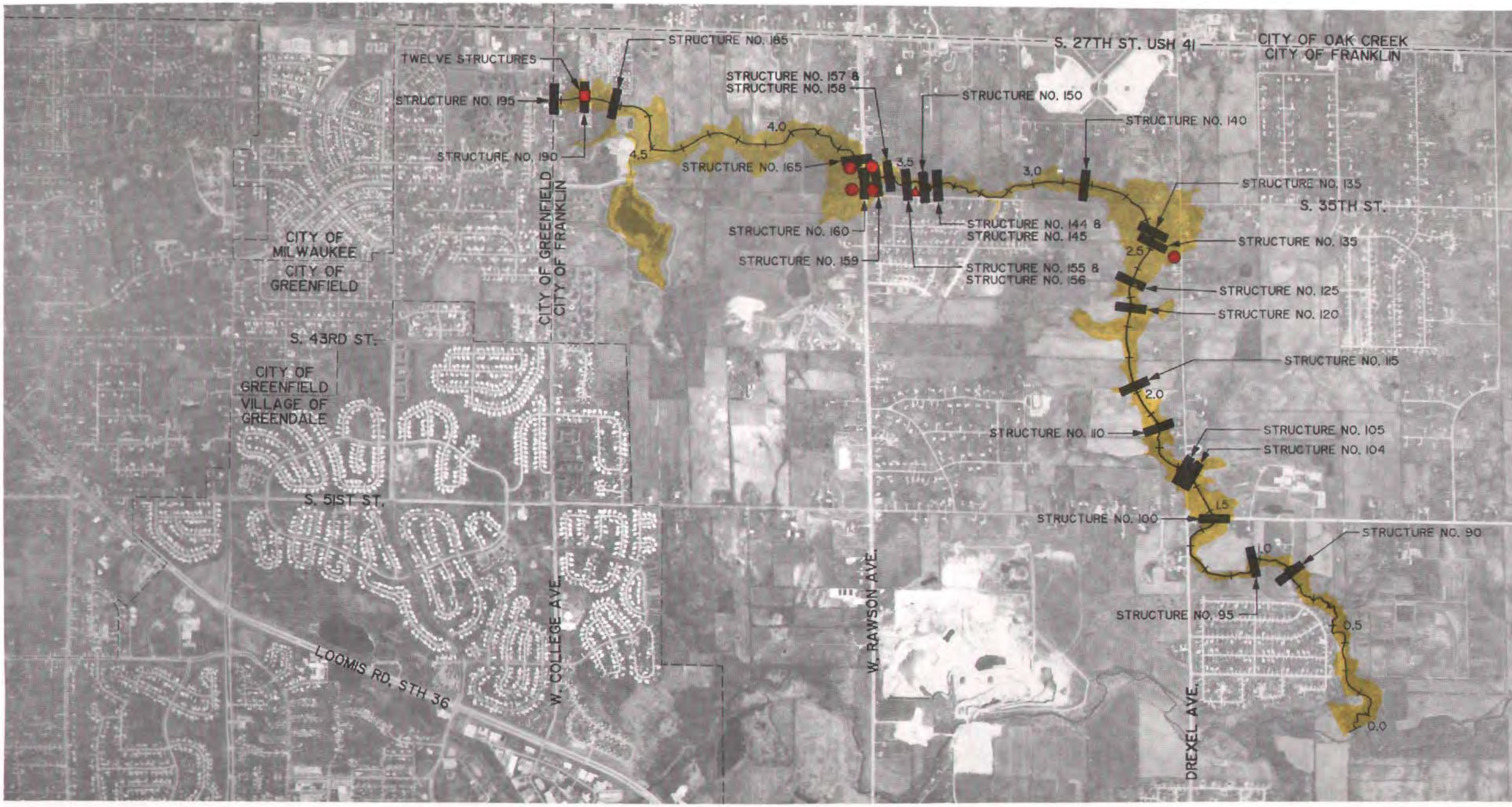
Implementation of the recommended plan would essentially eliminate all flood-related damages to existing structures along the East Branch of the Root River for floods up to and including the 100-year recurrence interval event under planned land use conditions.

The recommended plan consists of the floodproofing of four houses and one office building, the elevation of one house, and the relocation of 12 mobile homes out of the 100-year recurrence interval floodplain, as shown on Map 94.

It is recommended that replacement bridges for those structures shown in Appendix E as having inadequate hydraulic capacities be designed so as to pass the recommended design flood flow without overtopping of the attendant roadway. Such replacement is not required for flood control purposes, but rather should be carried out for transportation or other purposes.

Map 94

RECOMMENDED FLOOD CONTROL SYSTEM PLAN FOR THE EAST BRANCH ROOT RIVER



LEGEND

- 100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS
- 4.5 — APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING
- STRUCTURE FLOODPROOFING
- STRUCTURE TO BE REMOVED
- STRUCTURE ELEVATION

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR EAST BRANCH ROOT RIVER IS SHOWN IN APPENDIX H

THERE IS NO CHANGE BETWEEN THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS, AND THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS. SEE FIGURE 38 ON PAGES 285 AND 286 FOR THE FLOODSTAGE AND STREAMBED PROFILES FOR EAST BRANCH ROOT RIVER.

GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Source: SEWRPC.

Little significant development has occurred along the floodplain of the East Branch of the Root River since the completion of large-scale topographic maps for the City of Franklin in 1964. One exception to this is the development of the Root River Heights and the Hawthorne Glen subdivisions in the northwest one-quarter of U. S. Public Survey Section 14 and the northeast one-quarter of Section 15 in the City of Franklin. In order that a more accurate delineation of the 100-year recurrence interval floodplain can be made, it is recommended that new large-scale topographic maps be prepared for these two quarter sections. Since these maps would serve multiple purposes, none of the attendant costs have been assigned to the flood control plan.

Flood Control and Related Drainage System Plan Implementation

The recommended flood control system plan for the East Branch of the Root River consists of structure floodproofing, elevation, and removal. The structure floodproofing and elevation measures would be undertaken by the property owners directly affected. For the 12 mobile homes to be relocated outside the floodplain, it is recommended that the $84,000 cost for this relocation be borne by the City of Franklin, as these structures are located along a stream reach not recommended for District jurisdiction. It is further recommended that the professional services required to prepare plans for the floodproofing and elevation of individual buildings be made available, at no cost, to property owners by the City of Franklin engineering department. Also, it is recommended that the City of Franklin review its building ordinance to ensure that appropriate floodproofing regulations are included. It is recommended that the City explore, on behalf of the property owners involved, any available state and/or federal aids for such floodproofing measures. It is further recommended that the Milwaukee Metropolitan Sewerage District prepare new large-scale topographic maps for the northwest one-quarter of U. S. Public Land Survey Section 14 and the northeast one-quarter of Section 15, Township 5 North, Range 21 East, City of Franklin.

TESS CORNERS CREEK SUBWATERSHED FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN

As already noted, hydrologic and hydraulic analyses, alternative drainage and flood control system plans, and recommended system plans for three streams under Milwaukee Metropolitan Sewerage District jurisdiction—Whitnall Park Creek, the North Branch of Whitnall Park Creek, and the Northwest Branch of Whitnall Park Creek—were prepared by the Regional Planning Commission under a separate stormwater management planning program for the Village of Hales Corners. The drainage improvement and flood control measures recommended for these three streams under this prior planning program have been incorporated into this system plan. Hydrologic and hydraulic analyses of Tess Corners Creek were conducted under federal flood insurance studies for the Villages of Franklin and Greendale; however, no drainage and flood control alternatives were evaluated under those studies. Accordingly, the system level plan was expanded herein to include Tess Corners Creek in addition to the information for the three streams studied under the Hales Corners stormwater management plan.

Village of Hales Corners Stormwater Management Plan

Drainage and flood control improvements for Whitnall Park Creek, the North Branch of Whitnall Park Creek, and the Northwest Branch of Whitnall Park Creek were considered in the stormwater management planning program conducted by the Regional Planning Commission for the Village of Hales Corners.[5] The village plan seeks to promote the development of an effective stormwater management system for the Village. To the extent practicable, the system is designed to minimize damages attendant to poor drainage while reducing downstream flooding. More specifically, the planning report prepared for the Village:

1. Describes the existing stormwater drainage system and the existing stormwater drainage and related problems in the Village and environs and identifies the causes of these problems;

2. Sets forth proposed future land use conditions and related stormwater management requirements;

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[5] *See SEWRPC Community Assistance Planning Report No. 121, A Stormwater Management Plan for the Village of Hales Corners, Milwaukee County, Wisconsin, March 1986.*

3. Provides a set of stormwater management objectives and supporting standards to guide the development of an effective stormwater management system;

4. Presents alternative stormwater management system plans;

5. Provides a comparative evaluation of the technical, economic, and environmental features of the alternative plans;

6. Recommends a stormwater management plan for the Village and environs consisting of various structural and nonstructural measures; and

7. Identifies the responsibilities of, and actions required by, the various governmental units and agencies that will implement the recommended plan.

The system plans herein presented for Whitnall Park Creek, the North Branch of Whitnall Park Creek, and the Northwest Branch of Whitnall Park Creek provide data on existing and probable future flood problems, alternative and recommended drainage and flood control improvement measures, and recommended implementation actions, and, as already noted, are based upon the findings and recommendations of the stormwater management planning program for the Village of Hales Corners described above. The system plan includes only those major system components of the village plan relating to the streams and watercourses under District jurisdiction.

Overview of the Study Area

The Tess Corners Creek subwatershed, as shown on Map 95, is located within the corporate limits of the Cities of Franklin, Greenfield, Muskego, and New Berlin, and the Villages of Greendale and Hales Corners. The stream reaches recommended for District jurisdiction and included in this subwatershed are as follows: 1) a 0.78-mile reach of the North Branch of Whitnall Park Creek from its confluence with the Northwest Branch of Whitnall Park Creek upstream to W. Edgerton Avenue; 2) a 0.44-mile reach of the Northwest Branch of Whitnall Park Creek from its confluence with Whitnall Park Creek upstream to the confluence of the North Branch of Whitnall Park Creek; 3) a 1.81-mile reach of Whitnall Park Creek from its confluence with Tess Corners Creek upstream to the confluence of the Northwest Branch of Whitnall Park Creek; and 4) a 2.64-mile reach of Tess Corners Creek from its confluence with the Root River to a point about 0.60 mile upstream of the W. Rawson Avenue crossing. Of the stream reaches described above, the 2.64-mile reach of Tess Corners Creek and the downstream 0.64 mile of the 1.81-mile reach of Whitnall Park Creek are classified as perennial; the remaining stream reaches are classified as intermittent.

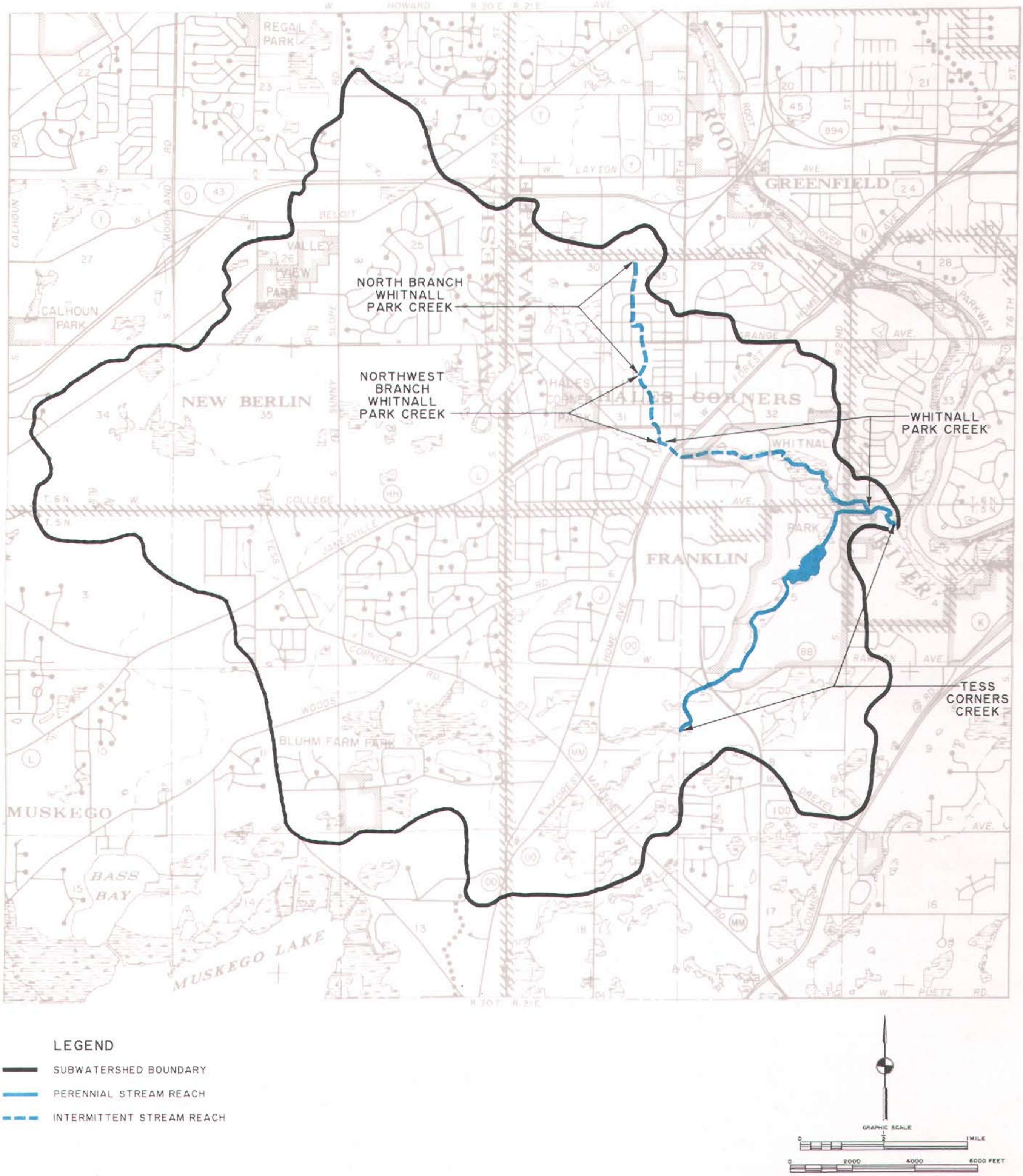
In 1985 the Tess Corners Creek subwatershed was still largely undeveloped, with 8.96 square miles, or about 60 percent, devoted to agricultural and other open space uses. The remaining 6.09 square miles was developed for urban use—with 5.91 square miles, or about 97 percent, consisting of residential uses, and the remaining 0.18 square mile consisting of commercial and industrial uses. Much of the land within the subwatershed developed for urban use is located in the headwater areas. Municipal street improvements consist primarily of paved streets without curbs and gutters and integrated storm sewer facilities. Therefore, most of the surface runoff is carried in roadside ditches which discharge to the natural streams and watercourses of the subwatershed. Whitnall Park Creek upstream of the S. 108th Street crossing, the Northwest Branch of Whitnall Park Creek, and the North Branch of Whitnall Park Creek have been modified in varying degrees by deepening, realignment, or enclosure to accommodate increasing surface runoff resulting from urban development.

Flooding and Related Drainage Problems

The most significant historical drainage problems in the Tess Corners Creek subwatershed exist in the Village of Hales Corners. The most persistent and widespread problems appear to be related to high groundwater levels, which require the excessive operation of sump pumps over extended periods of time and contribute to ponding of stormwater in drainage ditches and low areas during wet weather conditions. The drainage problems are aggravated by drainage ditches with insufficient slopes and conveyance capacities, providing inadequate outlets for the local storm sewer facilities. The insufficient conveyance capacities of the major stream reaches also create the potential for significant overland flooding during major storm events,

Map 95

THE TESS CORNERS CREEK SUBWATERSHED



Source: SEWRPC.

with resulting monetary damage to affected buildings. These problems may be expected to be exacerbated by the further development of agricultural and other open lands within the subwatershed.

The costs of overland flooding from major storm events were estimated using damage cost curves prepared by the Commission and described in Chapter III. The dollar amount of the flood damages was thus based upon the depth of inundation and the assessed valuation of the buildings concerned. Damages to building contents were included in the total damage costs.

Flooding, as defined herein, includes basement flooding, yard inundation, and flooding above the first-floor level of buildings. The total number of residential structures that may be expected to experience direct flooding along the four studied stream reaches in the Tess Corners Creek subwatershed under existing land use and channel conditions is set forth below. These floodprone structures, as shown on Map 96, are located in proximity to the stream channels of the study reaches. No flooding of structures is expected to occur along Tess Corners Creek itself.

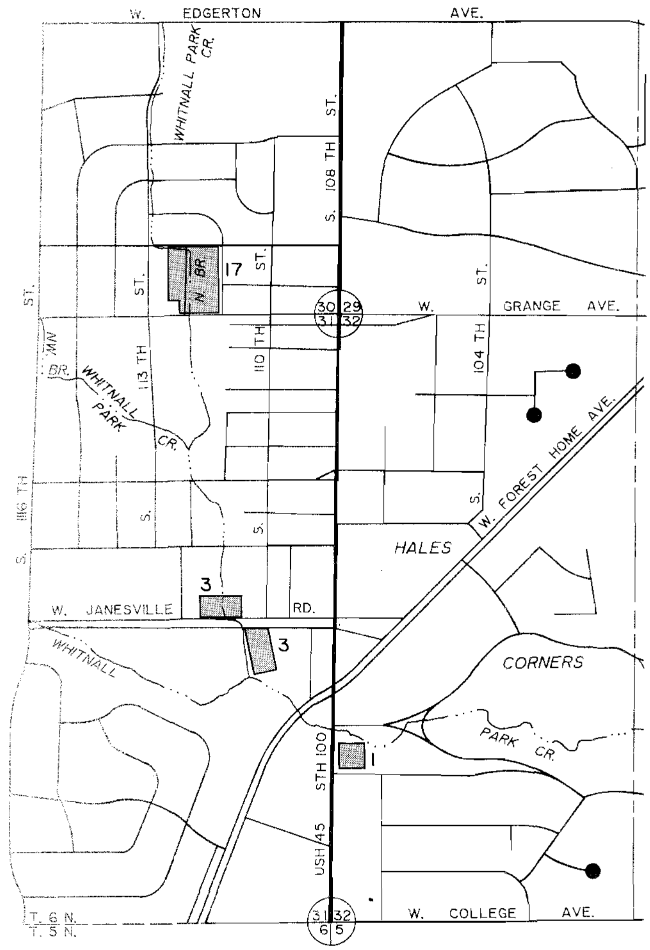
Flood Event Recurrence Interval	Approximate Number of Existing Residential Structures Flooded Existing Land Use and Existing Channel Conditions	Approximate Number of Existing Nonresidential Structures Flooded Existing Land Use and Existing Channel Conditions
10	15	2
50	18	4
100	20	4

Additional structures may, however, experience indirect flood damages through sanitary sewer backup. It should be noted that the flood control measures considered under this system plan are primarily intended to alleviate flood damages from direct overland flooding along the streams studied, as well as to provide an adequate outlet for local storm sewers and drainageways. These measures will also help to reduce flooding due to localized stormwater drainage problems or sanitary sewer backups.

The total average annual flood losses—damages—for the Tess Corners Creek subwatershed are estimated at $51,500 under existing land use and channel conditions. Flood losses from a 100-year recurrence interval event are estimated at $599,000 under existing land use and channel conditions.

Map 96

LOCATION OF FLOOD-PRONE STRUCTURES UNDER EXISTING LAND USE AND DRAINAGE CONDITIONS IN THE TESS CORNERS CREEK AREA



LEGEND

LOCATION OF FLOODPRONE STRUCTURES

3 NUMBER OF FLOODPRONE STRUCTURES

GRAPHIC SCALE

Source: SEWRPC.

Results of the hydrologic and hydraulic analyses conducted of the Tess Corners Creek subwatershed indicated that there should be no significant increase in flood flows and stages under planned land use and existing channel conditions along the four studied stream reaches having the potential for stormwater damages.

This is to be expected because the drainage area tributary to these reaches is already largely developed for urban use. Since no significant increase in potential flood damages is expected under planned land use and existing channel conditions, only the estimated monetary flood damages under existing land use and channel conditions were used in the study.

The drainage and flood control objectives and supporting principles and standards set forth in Chapter III specify the flood events which bridges shall accommodate without overtopping of the related roadway. Based on these criteria, one bridge on Tess Corners Creek, one bridge on Whitnall Park Creek, and six bridges on the North Branch of Whitnall Park Creek are considered hydraulically inadequate, as shown in Appendix E. These bridges are at S. 92nd Street on Tess Corners Creek; S. 92nd Street on Whitnall Park Creek; W. Grange Avenue; S. 112th Street; W. Copeland Avenue; W. Mallory Avenue; W. Abbott Avenue; and W. Woodside Drive on the North Branch of Whitnall Park Creek.

Flood Discharges and Stages

As noted in Chapter III of this report, the hydrologic model used for development of design discharges for the North Branch of Whitnall Park Creek, the Northwest Branch of Whitnall Park Creek, and Whitnall Park Creek upstream of S. 108th Street were developed by the Regional Planning Commission using the model known as the Illinois Urban Drainage Area Simulator (ILLUDAS). This model uses discrete rainfall patterns for the selected recurrence interval design storms. Peak flow rates are determined by applying the rainfall patterns to contributing drainage areas to produce runoff hydrographs which are combined to form instream discharges.

The hydrologic model used for development of design discharges for Tess Corners Creek and Whitnall Park Creek downstream of S. 108th Street, the HydroComp hydrologic model, simulates streamflow on a continuous basis, using recorded climatological data as input. Discharges were computed at hourly time intervals. Flood discharges were developed by conducting discharge-frequency analyses of simulated annual peak discharges generated by the hydrologic model using the log Pearson Type III method of analysis, as recommended by the U. S. Water Resources Council and as specified by the Wisconsin Department of Natural Resources. Because of the relatively small tributary drainage area of these two streams, it was suspected that the time of peak discharge on the stream was relatively short; thus it could be missed utilizing only an hourly time interval in the analyses. Additional simulations were performed, therefore, using a 15-minute time interval and related design rainfall events. The use of design rainfall events was necessary since the time and cost of simulating continuous streamflows at 15-minute intervals for the 39 years of available climatological data would be prohibitive.

The design rainfall events were developed using 10-, 50-, and 100-year recurrence interval rainfall volumes obtained from the updated point rainfall depth-duration-frequency relationships developed by the Regional Planning Commission as described in Chapter III. The rainfall distribution utilized for each design storm was the median distribution of a first-quartile storm, as shown in Chapter III. The design storm duration was determined for a given recurrence interval by simulating the peak discharge at a given location for a range of storm durations. The storm duration and associated rainfall volume which produced the largest peak discharge at a given location for a given recurrence interval was selected as the design storm for that location. The estimated peak flood discharges under existing and planned, year 2000 land use conditions and existing channel conditions are set forth in Table 52.

Flood stage profiles were determined for the 10-, 50-, and 100-year recurrence interval runoff events under planned land use and existing channel conditions. These profiles, which encompass the full 5.67 miles of stream reaches recommended for District jurisdiction in the Tess Corners Creek subwatershed, constitute a graphic representation of the flood stages along the four studied stream reaches under the specified recurrence interval flood discharges, and under planned land use and existing channel conditions. In addition to providing an overall representation of flood stages relative to familiar points of reference such as the channel bottom and bridge deck surfaces, the profiles, because of their continuity, permit the determination of flood stages at any location along the stream channel. The flood profiles are shown on Figure 39. The extent of the 100-year recurrence interval floodplain under planned land use

Table 52

FLOOD DISCHARGES FOR TESS CORNERS CREEK SUBWATERSHED FOR EXISTING AND YEAR 2000 LAND USE AND EXISTING CHANNEL CONDITIONS

Location	River Mile	Peak Flood Discharge (cubic feet per second)					
		Existing Land Use Existing Channel Conditions			Year 2000 Planned Land Use, Existing Channel Conditions		
		10-Year	50-Year	100-Year	10-Year	50-Year	100-Year
Tess Corners Creek							
S. 92nd Street	0.54[a]	690	1,460	1,770	850	1,710	2,030
Whitnall Park Drive	0.58	690	1,460	1,770	850	1,710	2,030
Whitnall Park Dam	0.84	690	1,460	1,770	850	1,710	2,030
W. Rawson Avenue/CTH BB	2.04	690	1,460	1,770	850	1,710	2,030
Private Drive	2.33	910	1,660	1,950	1,080	1,920	2,240
Whitnall Park Creek							
W. College Avenue	0.06[b]	980	1,860	2,190	980	1,860	2,190
S. 92nd Street	0.17	980	1,860	2,190	980	1,860	2,190
Whitnall Park Drive	0.24	980	1,860	2,190	980	1,860	2,190
Whitnall Park Dam	0.26	980	1,860	2,190	980	1,860	2,190
Whitnall Park Drive	0.39	980	1,860	2,190	980	1,860	2,190
Whitnall Park Dam	0.40	980	1,860	2,190	980	1,860	2,190
Whitnall Park Dam	0.64	980	1,860	2,190	980	1,860	2,190
Whitnall Park Drive	0.97	1,000	1,500	1,800	1,000	1,500	1,800
Whitnall Park Drive	1.43	1,000	1,500	1,800	1,000	1,500	1,800
Whitnall Park Drive	1.47	1,000	1,500	1,800	1,000	1,500	1,800
S. 108th Street/STH 100	1.62	697	1,090	1,273	734	1,160	1,373
W. Forest Home Avenue/CTH OO	1.70	546	925	1,116	592	1,000	1,207
Northwest Branch Whitnall Park Creek							
W. Janesville Road	0.09[c]	207	335	394	204	335	398
W. Godsell Road	0.25	201	280	312	200	275	311
W. Parnell Avenue	0.39	201	280	312	200	275	311
North Branch Whitnall Park Creek							
W. Grange Avenue	0.24[d]	93	170	211	100	175	214
S. 112th Street	0.36	122	185	212	128	190	216
W. Copeland Avenue	0.41	122	185	212	128	190	216
W. Mallory Avenue	0.47	122	185	212	128	190	216
W. Upham Avenue	0.53	103	155	178	109	160	182
W. Abbott Avenue	0.58	91	137	158	97	140	163
W. Edgerton Avenue	0.78	62	89	102	59	85	95

[a]*Distance in river miles above confluence with Root River.*

[b]*Distance in river miles above confluence with Tess Corners Creek.*

[c]*Distance in river miles above confluence with Whitnall Park Creek.*

[d]*Distance in river miles above confluence with Northwest Branch of Whitnall Park Creek.*

Source: SEWRPC.

conditions is shown on Map 97. This delineation of the flood hazard area was accomplished using one inch equals 100 feet scale, two-foot contour interval topographic maps prepared to Regional Planning Commission specifications.

Alternative Flood Control and Related Drainage System Plans for the Tess Corners Creek Subwatershed

Two alternative stormwater management system plans were considered for alleviating the drainage and potential flood damage problems in the Tess Corners Creek subwatershed: 1) a conveyance plan; and 2) a detention storage plan. These alternative system plans were developed for Whitnall Park Creek, the Northwest Branch of Whitnall Park Creek, and the North Branch of Whitnall Park Creek under the Village of Hales Corners study. No such alternative plans were developed for Tess Corners Creek or Whitnall Park Creek downstream of the Village of Hales Corners corporate limits since no flooding or related drainage problems exist along these stream reaches.

It should be noted that while the major drainage system components developed under the village plan were designed to eliminate all damages from flood events up to and including the 100-year recurrence interval event, the plan did leave two structures to be floodproofed in order to reduce the cost of the recommended structural flood control works.

Alternative Plan 1—Conveyance: The conveyance alternative plan involves the provision of new storm sewers and engineered open channels and attendant culverts to abate stormwater runoff problems and to effectively serve planned new urban development within the Village. Map 98 shows the location and alignment of proposed channel enclosures and engineered open channels and culverts proposed under the conveyance alternative. The salient characteristics of the proposed channel enclosures, improved channels, and attendant culverts comprising this alternative plan are as follows:

North Branch of Whitnall Park Creek

1. About 1,240 lineal feet of channel enclosure consisting of 48-inch buried conduit in S. 113th Street from 180 feet south of W. Edgerton Avenue to W. Upham Avenue.

2. About 940 lineal feet of channel enclosure consisting of 54-inch buried conduit in S. 113th Street from W. Upham Avenue to W. Copeland Avenue and in W. Copeland Avenue from S. 113th Street to S. 112th Street.

3. About 665 lineal feet of channel enclosure consisting of buried conduits in S. 112th street from W. Copeland Avenue to W. Grange Avenue.

4. About 125 lineal feet of channel enclosure consisting of two 60-inch buried conduits from W. Grange Avenue south.

5. Regrading of 1,100 feet of open channel upstream of the confluence with the Northwest Branch of Whitnall Park Creek.

6. About 1,250 lineal feet of street regrading in W. Copeland Avenue from S. 111th Street to west of S. 112th Street; in S. 112th Street from W. Copeland Avenue to W. Grange Avenue; and in W. Grange Avenue to east and west of S. 112th Street.

Northwest Branch of Whitnall Park Creek

1. Installation of twin 78-inch culvert under driveway immediately north of W. Janesville Road (STH 24).

2. About 400 feet of channel improvement downstream of W. Janesville Road (STH 24) to confluence with Whitnall Park Creek.

3. Installation of twin 78-inch culvert under driveway 200 feet south of W. Janesville Road (STH 24).

4. About 300 feet of channel improvement downstream of the confluence of the North Branch of Whitnall Park Creek.

Whitnall Park Creek

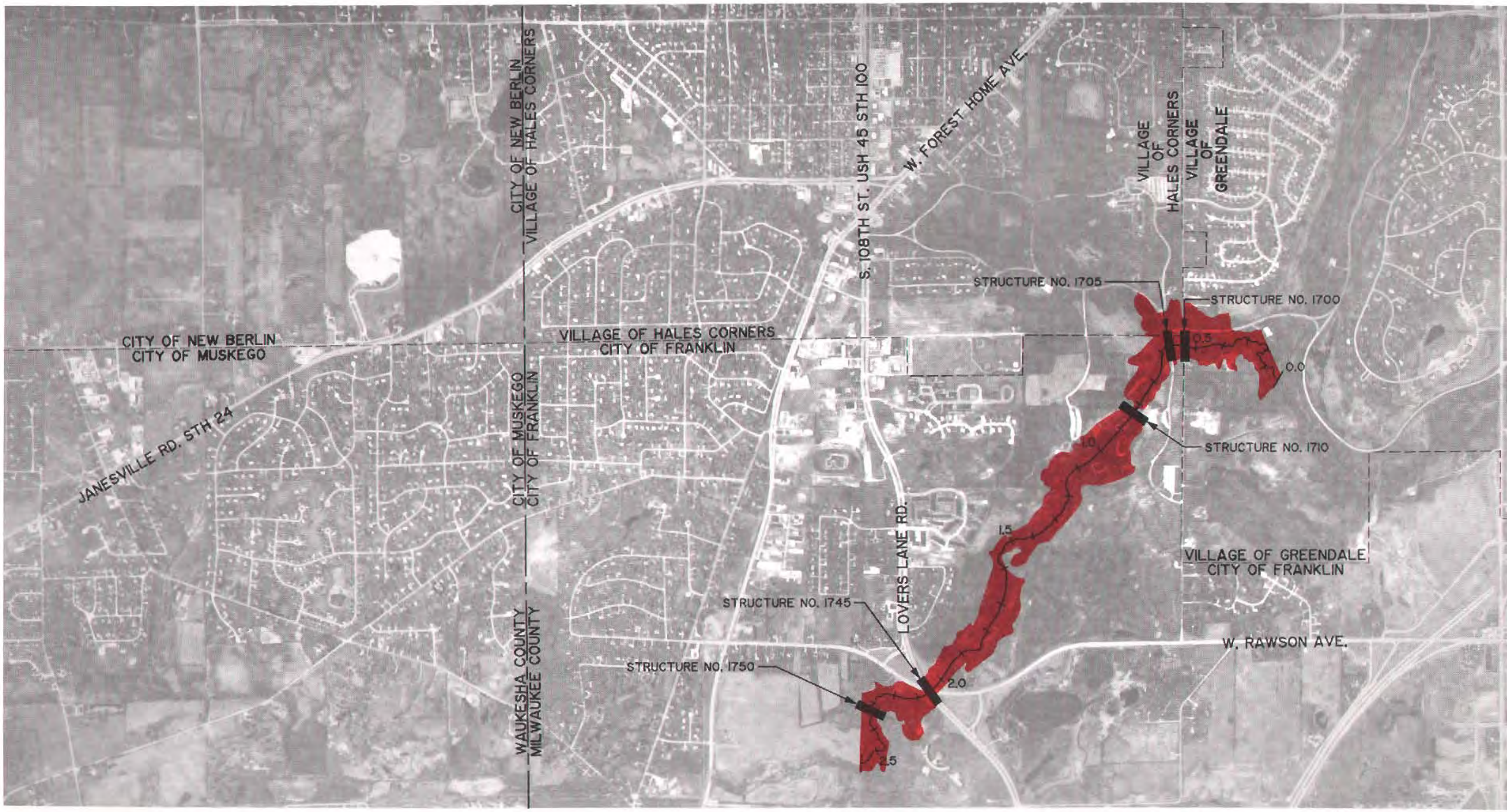
1. About 150 feet of channel improvement downstream of confluence of Northwest Branch of Whitnall Park Creek.

2. Installation of 20-foot by 6-foot box culvert under driveway at confluence with Northwest Branch of Whitnall Park Creek.

3. Removal of the driveway crossing 150 feet downstream of confluence of Northwest Branch of Whitnall Park Creek.

Map 97

100-YEAR RECURRENCE INTERVAL FLOODPLAIN FOR TESS CORNERS CREEK, WHITNALL PARK CREEK, NORTHWEST BRANCH WHITNALL PARK CREEK, AND NORTH BRANCH WHITNALL PARK CREEK—YEAR 2000 LAND USE AND EXISTING CHANNEL CONDITIONS



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

1.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR TESS CORNERS CREEK IS SHOWN IN APPENDIX H

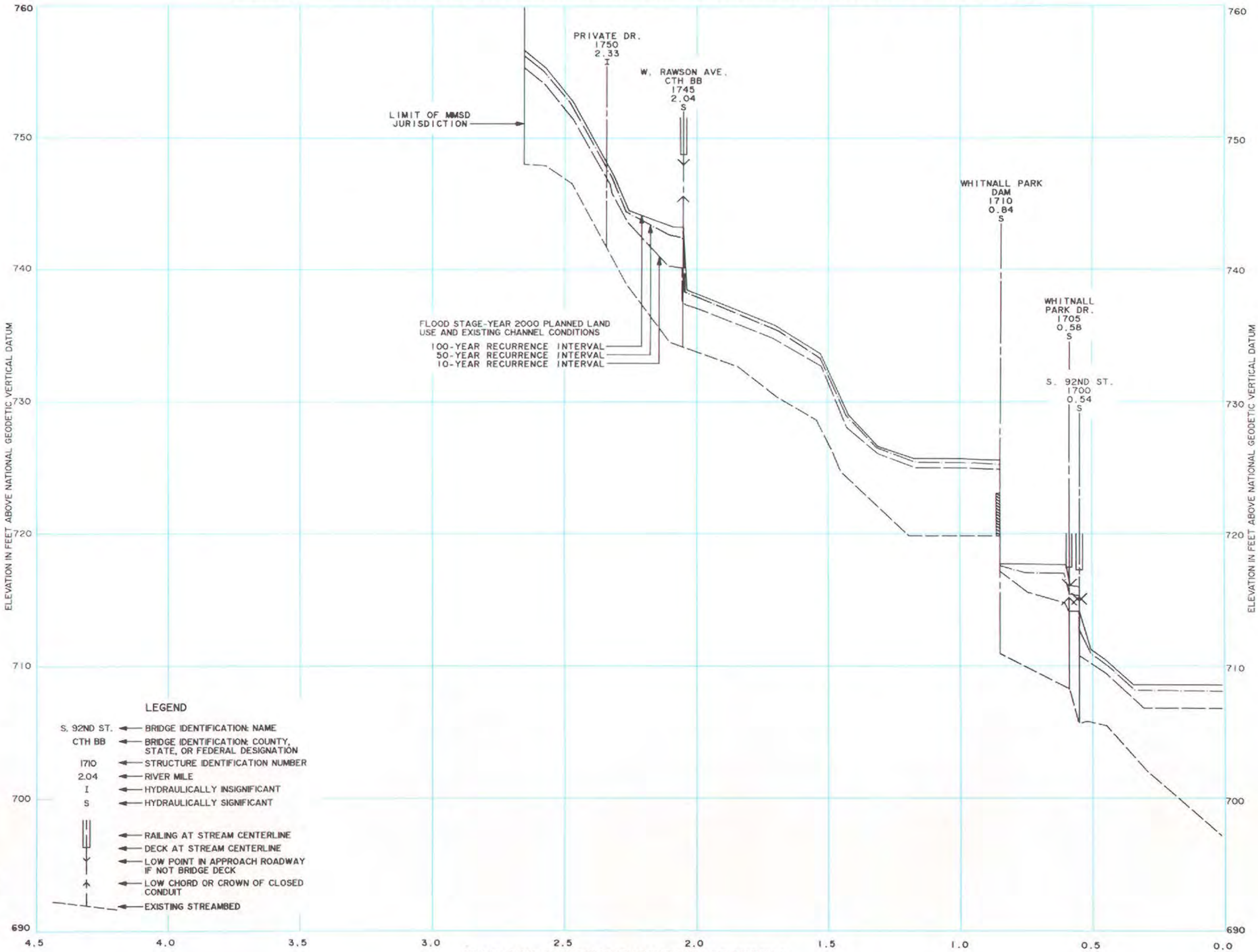
GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Figure 39

FLOOD STAGE AND STREAMBED PROFILES FOR TESS CORNERS CREEK, WHITNALL PARK CREEK, NORTHWEST BRANCH WHITNALL PARK CREEK, AND NORTH BRANCH WHITNALL PARK CREEK



Map 97 (continued)

LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

1.0 — APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

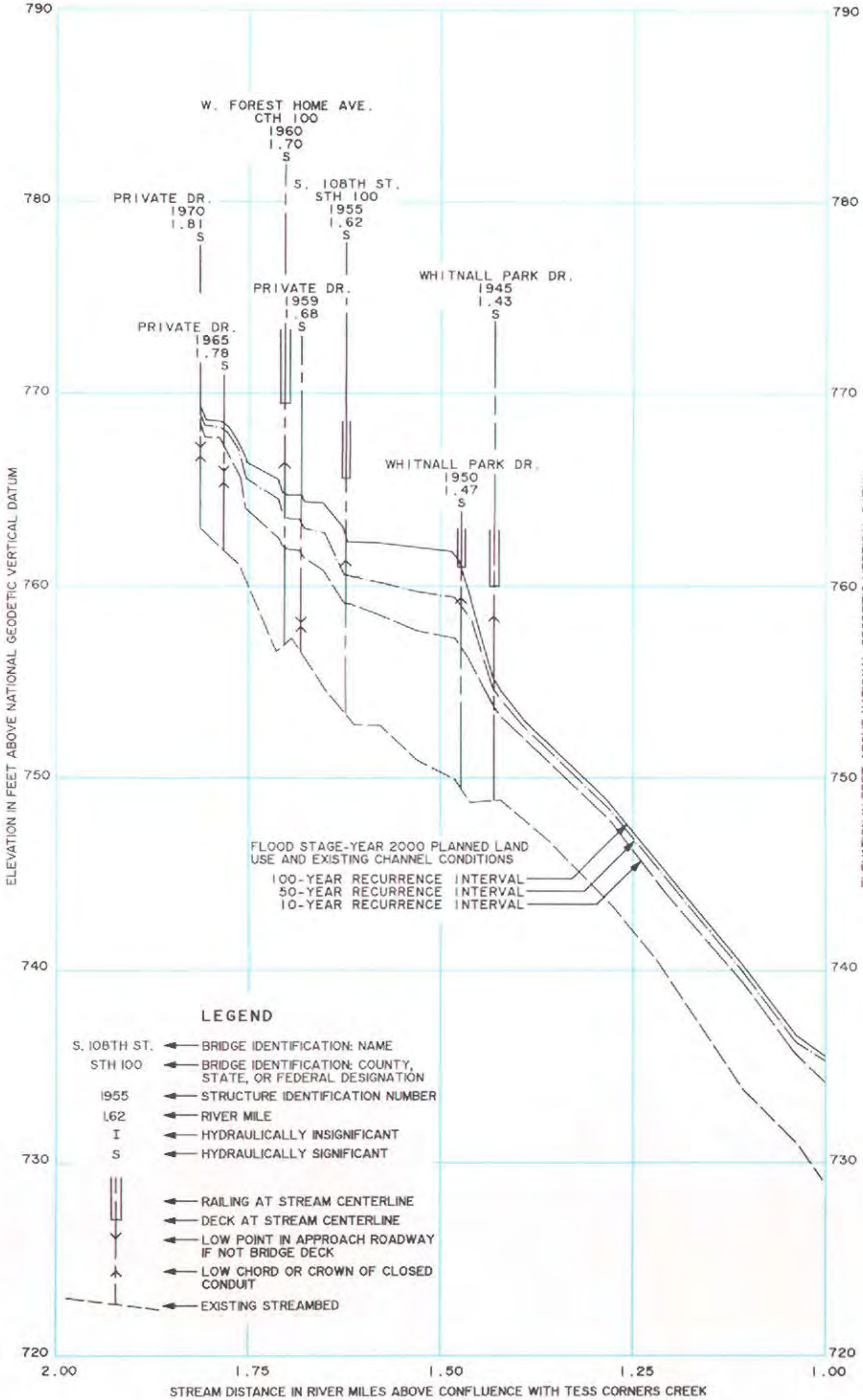
NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR WHITNALL PARK CREEK IS SHOWN IN APPENDIX H

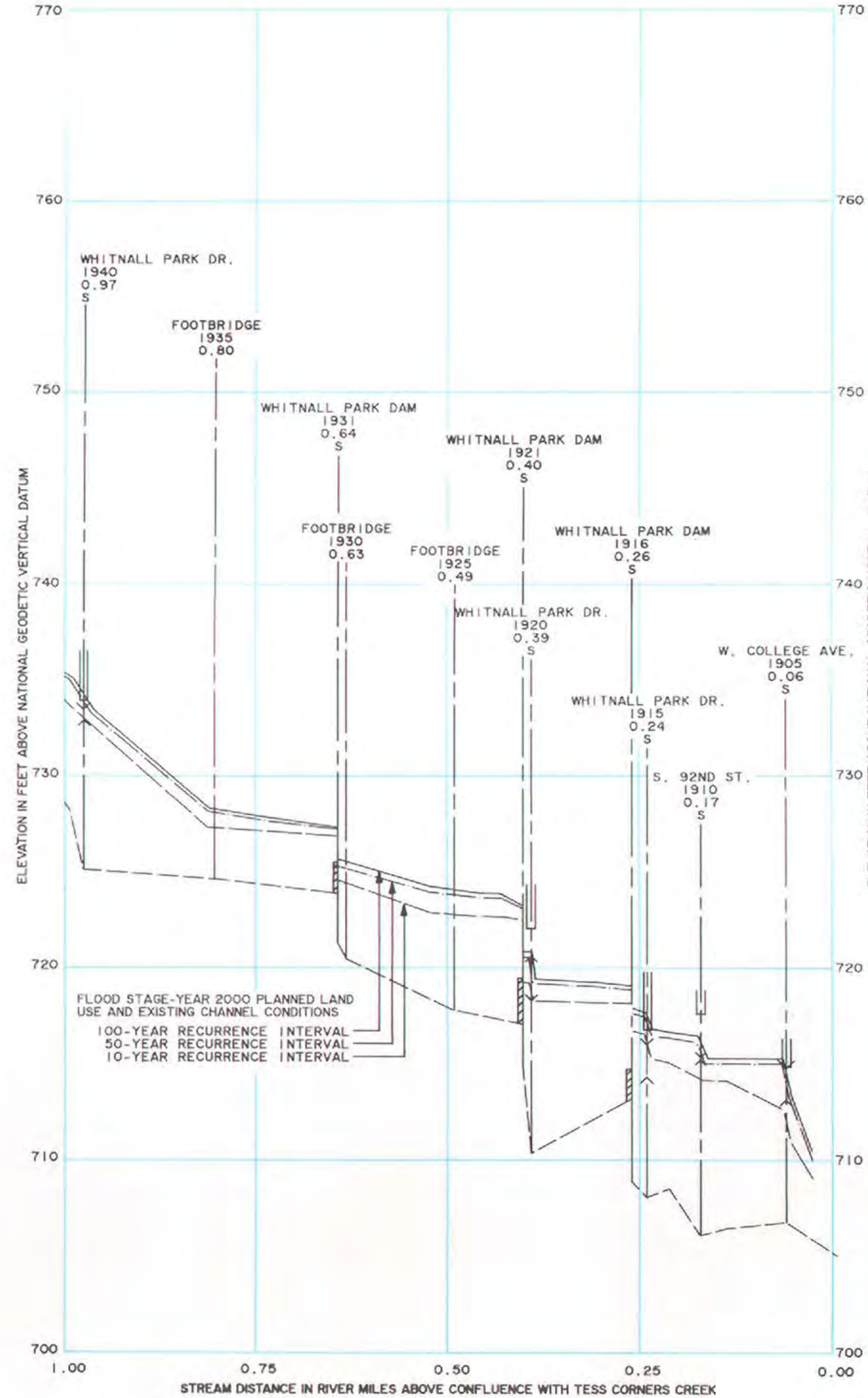
GRAPHIC SCALE

0 400 800 1200 FEET

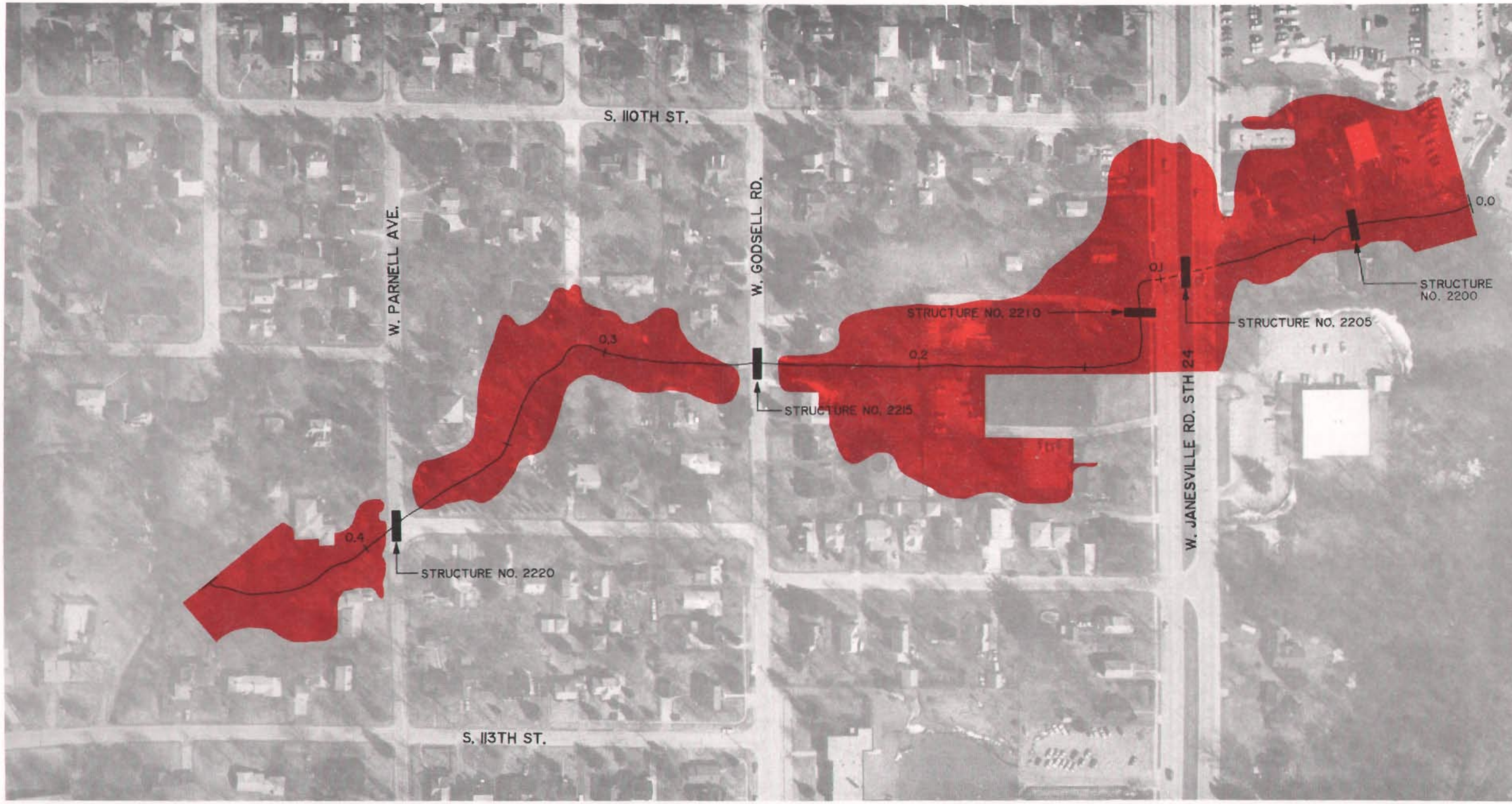
DATE OF PHOTOGRAPHY: MARCH 1985

Figure 39 (continued)





Map 97 (continued)



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

0.2 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

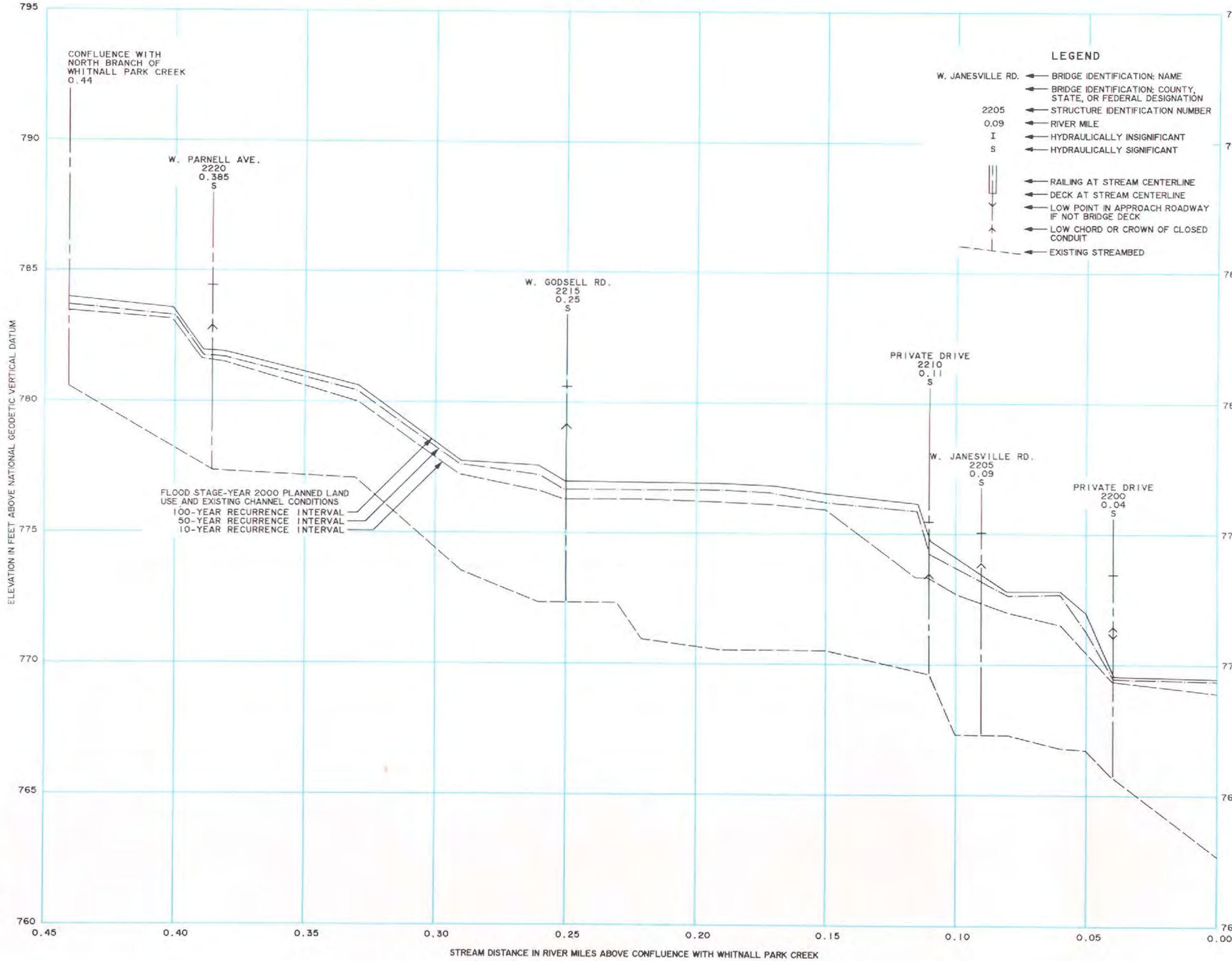
NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR NORTHWEST BRANCH OF WHITNALL PARK CREEK IS SHOWN IN APPENDIX H

GRAPHIC SCALE

0 100 200 300 FEET

DATE OF PHOTOGRAPHY: MARCH 1985

Figure 39 (continued)



Map 97 (continued)

LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

0.5 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

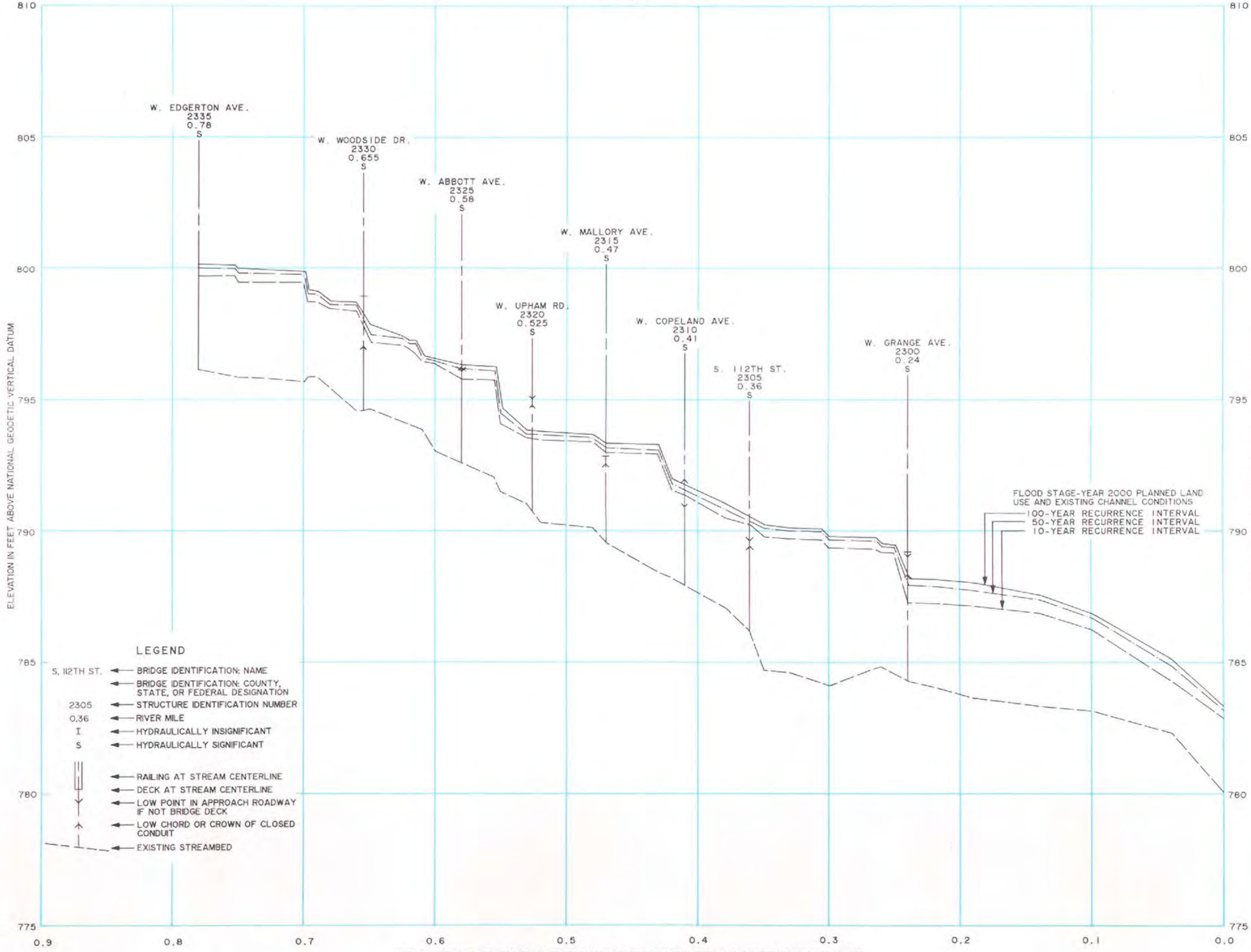
NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR NORTH BRANCH OF WHITNALL PARK CREEK IS SHOWN IN APPENDIX H

GRAPHIC SCALE
0 200 400 600 FEET

DATE OF PHOTOGRAPHY: MARCH 1985

Source: SEWRPC.

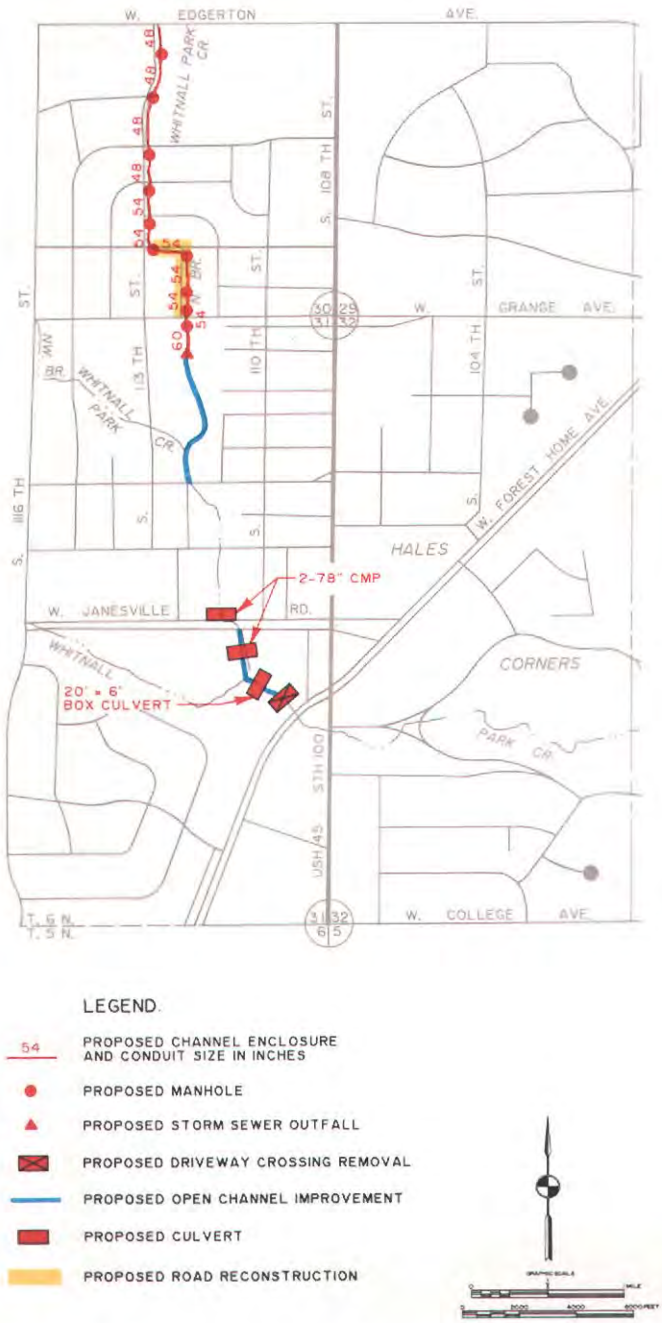
Figure 39 (continued)



Source: SEWRPC.

Map 98

CONVEYANCE ALTERNATIVE FOR TESS CORNERS CREEK SUBWATERSHED



Source: W. G. Nienow Engineering Associates and SEWRPC.

The conveyance alternative thus consists of enclosing 2,970 lineal feet of the North Branch of Whitnall Park Creek in buried conduit ranging in size from 42 to 60 inches in diameter. All buried conduit is assumed to be constructed of reinforced concrete pipe.

About 1,950 lineal feet of new engineered open channels would be provided under this alternative. The new engineered channels would be concrete-lined, or a combination of concrete-lined bottom and turf side slopes. The plan also includes three new culvert installations consisting of two twin 78-inch corrugated metal pipes and a single 20-foot-wide by 6-foot-deep concrete box culvert, and removal of a driveway crossing.

Alternative Plan 2—Detention Storage: The detention storage alternative plan would provide for construction of a parking lot detention facility to reduce downstream discharge, allowing the use of smaller conveyance facilities downstream. The parking lot detention facility, along with supplementary conveyance facilities, would serve to abate stormwater drainage problems and to effectively accommodate increased runoff from new urban development. Map 99 shows the location of the proposed parking lot storage facility and of the major supplementary conveyance facilities. The salient characteristics of the proposed channel enclosure, open channels, and detention facility comprising this plan are as follows:

North Branch of Whitnall Park Creek

1. About 1,420 feet of channel enclosure consisting of 48-inch buried conduit in S. 113th Street from W. Edgerton Avenue to W. Upham Avenue.
2. About 940 feet of channel enclosure consisting of 54-inch buried conduit in S. 113th Street from W. Upham Avenue to W. Copeland Avenue; and in W. Copeland Avenue from S. 113th Street to S. 112th Street.
3. About 1,250 lineal feet of road regrading in W. Copeland Avenue from S. 111th Street to the west of S. 112th Street; in S. 112th Street from W. Copeland Avenue to W. Grange Avenue; and in W. Grange Avenue to east and west of S. 112th Street.
4. A 5.3-acre-foot detention facility at Hales Corners Lutheran School.
5. About 665 feet of channel enclosure consisting of 54-inch buried conduit from detention facility to W. Grange Avenue.
6. About 125 feet of channel enclosure consisting of 60-inch buried conduit from W. Grange Avenue south.

7. About 1,100 feet of channel reconstruction upstream of the confluence with the Northwest Branch of Whitnall Park Creek.

Northwest Branch of Whitnall Park Creek

1. Installation of a 12-foot by 6-foot box culvert under driveway immediately north of W. Janesville Road (STH 24).

2. About 400 feet of channel improvement downstream of W. Janesville Road (STH 24) to confluence with Whitnall Park Creek.

3. Installation of a 12-foot by 6-foot box culvert under driveway 200 feet south of W. Janesville Road (STH 24).

4. About 300 feet of channel improvement downstream of the confluence of the North Branch of Whitnall Park Creek.

Whitnall Park Creek

1. About 150 feet of channel improvement downstream of confluence of Northwest Branch of Whitnall Park Creek.

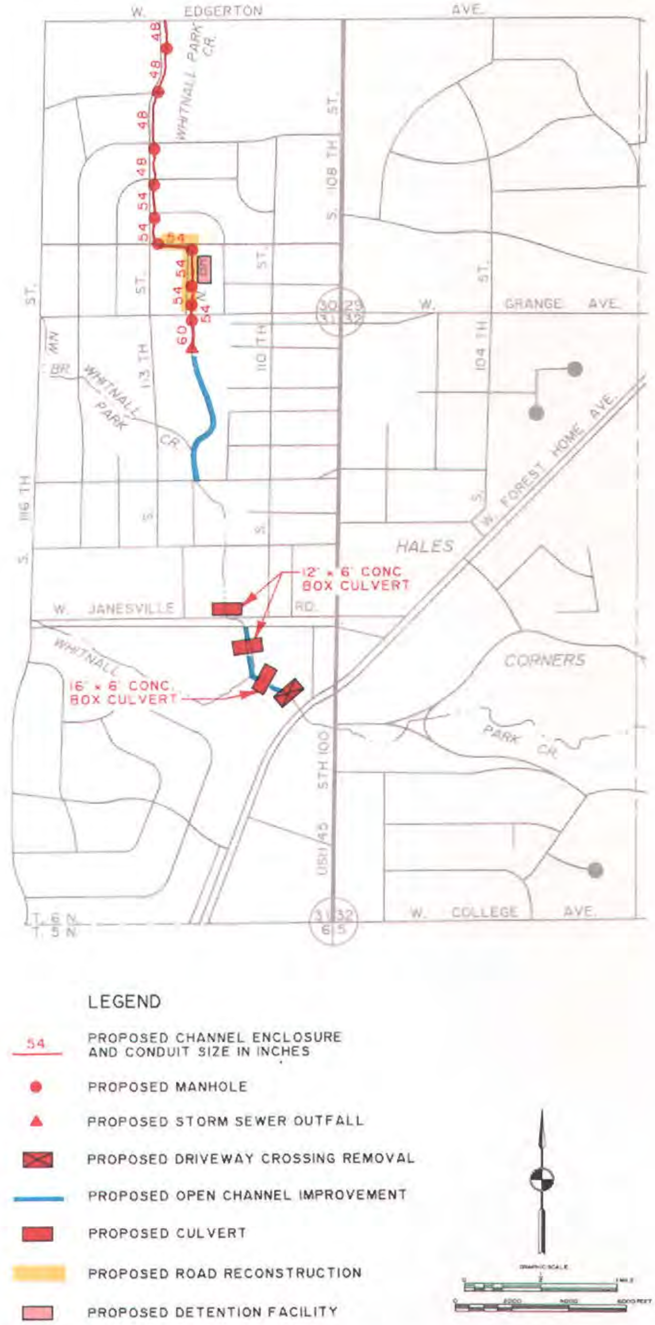
2. Installation of a 16-foot by 6-foot box culvert under driveway at confluence with Northwest Branch of Whitnall Park Creek.

3. Removal of the driveway crossing 150 feet downstream of confluence of Northwest Branch of Whitnall Park Creek.

The detention storage alternative provides for construction of a parking lot detention facility which would have a maximum surface area of 0.2 acre and a storage volume of about 5.3 acre-feet under 10-year recurrence interval runoff conditions. Supplementary conveyance measures include enclosing 3,150 lineal feet of the North Branch of Whitnall Park Creek in buried conduit ranging in size from 42 to 60 inches in diameter. Three new culvert installations are proposed consisting of two 12-foot-wide by 6-foot-deep concrete box culverts and one 16-foot-wide by 6-foot-deep concrete box culvert. Also proposed is removal of a driveway crossing. All buried conduit is assumed to be constructed of reinforced concrete pipe. About 1,950 feet of new engineered open channels would be provided under this alternative, as shown on Map 99. All of the new engineered channels would be turf-lined.

Map 99

DETENTION STORAGE ALTERNATIVE FOR TESS CORNERS CREEK SUBWATERSHED



Source: W. G. Nienow Engineering Associates and SEWRPC.

Alternative Supplementary Flood Control Measures for the Tess Corners Creek Subwatershed

While the alternative flood control and related drainage system plans discussed above would eliminate the majority of drainage and flooding problems of the Tess Corners Creek subwatershed, two structures may be expected to continue to experience damage from a flood having a recurrence interval of 100 years or more—an office building located along the south bank of Whitnall Park Creek immediately downstream of S. 108th Street; and an apartment building located along the east bank of the Northwest Branch of Whitnall Park Creek immediately upstream of W. Janesville Road. Since the plan presented herein is intended to eliminate damages from floods having a recurrence interval up to and including 100 years, two alternative supplementary flood control measures were considered: 1) Structure Floodproofing; and 2) Combination Bridge Replacement and Structure Floodproofing.

Alternative Plan 1 consists of floodproofing the two subject structures at a capital cost of $104,200. Utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of this alternative is estimated at $6,500.

Alternative Plan 2 consists of the replacement of three bridges—the first two parkway drive crossings of Whitnall Park Creek and the W. Janesville Road crossing of the Northwest Branch of Whitnall Park Creek—and floodproofing of the office building along Whitnall Park Creek. Replacement of these three bridges with structures having greater hydraulic capacity would eliminate the potential flood problem for the apartment building concerned and greatly reduce the depth of inundation of the office building concerned, thus minimizing the floodproofing cost incurred. The capital cost of this alternative is $492,000. Utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of this alternative is estimated at $31,200.

Recommended Flood Control and Related Drainage System Plan for the Tess Corners Creek Subwatershed

The recommended stormwater management and flood control system components for the stream reaches recommended for District jurisdiction in the Tess Corners Creek subwatershed, along with the attendant costs, are set forth in Table 53. The recommended plan is summarized in graphic form on Map 100, which identifies the boundary of the 100-year recurrence interval floodplain under planned land use and channel conditions. The 100-year flood profile under planned land use and channel conditions is shown in Figure 40.

The recommended flood control and related drainage system includes a detention storage component and conveyance components. The detention storage component consists of a single surface detention facility with associated inlets and outlets. The conveyance components include: 1) buried conduit and related inlets, manholes, and outfalls; 2) engineered open channels; and 3) street cross-sections. Full street cross-sections are to be utilized to convey flows in excess of those generated by a 10-year recurrence interval runoff event and up to and including the flows generated by a 100-year recurrence interval runoff event. The capacity of the street cross-section under 100-year recurrence interval flood conditions along the North Branch of Whitnall Park Creek in the vicinity of W. Copeland Avenue may be expected to be exceeded under planned land use and existing channel conditions. In this area, however, inundation of land beyond the street cross-section at and below the peak flood stages will not result in property damage or pose a threat to public health and safety. It should be noted that approximate street pavement crown elevations are recommended for all intersections and for all locations of recommended changes in street grade. These are intended to assure the proper functioning of the major stormwater drainage system, and are intended to be used as guides in the establishment of street grades.

As shown in Table 53, the recommended flood control and related drainage system plan components for the stream reaches recommended for Milwaukee Metropolitan Sewerage District jurisdiction in the Tess Corners Creek subwatershed have an estimated capital cost of $1,433,000. Annual operation and maintenance costs are estimated at $6,200, yielding at total average annual cost of $97,200. As presented earlier, the estimated average annual monetary flood damage for these stream reaches is $50,500. Therefore, implementation of the recommended plan for these stream reaches would result in average annual benefits of $50,500, with the recommended plan having a benefit-cost ratio of 0.52. Although this benefit-cost ratio is less than one, the recommended plan presented herein is intended not only to eliminate the monetary damages within the subwatershed resulting from major storm events such as those having a recurrence interval of greater than 10 years, but also to eliminate the stormwater drainage problems which occur frequently as a result of storms of lesser magnitude.

In addition to the plan elements described below, it is recommended that replacement bridges for those structures shown in Appendix E as having inadequate hydraulic capacities be designed so as to pass the recommended design flood flow without overtopping of the attendant roadway. Such replacement is not required for flood control purposes, but rather should be carried out for transportation or other purposes.

Description of the Recommended Stormwater Management System by Stream Reach

A brief summary of the recommended plan components for each of the three stream reaches recommended for District jurisdiction is provided below.

North Branch of Whitnall Park Creek: To improve the stormwater drainage conditions in the problem areas and to accommodate anticipated runoff conditions, approximately 3,150 lineal feet of channel enclosure consisting of buried conduit, ranging in size from 42 inches to 66 inches in diameter, is proposed to be installed. In addition, a 5.3-acre-foot detention basin is proposed to be constructed in the Hales Corners Lutheran School playground and parking lot off S. 112th Street. The following streets are proposed to be lowered and reconstructed: S. 112th Street between W. Copeland Avenue and W. Grange Avenue by 2.1 feet to 1.2 feet; W. Copeland Avenue in the vicinity of S. 112th Street up to 2.1 feet; and W. Grange Avenue in the vicinity of S. 112th Street up to 2.1 feet. The intersection of W. Copeland Avenue and S. 111th Street is proposed to be raised about 0.5 foot. Approximately 1,100 lineal feet of turf-lined channel is recommended to be constructed from the confluence with the Northwest Branch of Whitnall Park Creek.

Northwest Branch of Whitnall Park Creek: To improve the stormwater drainage conditions, two 12-foot-wide by 6-foot-high concrete box culverts are proposed to be installed under the driveway located immediately north of W. Janesville Road and under the driveway located 200 feet south of W. Janesville Road. Approximately 400 lineal feet of open channel downstream of W. Janesville Road to the confluence with Whitnall Park Creek, and 300 lineal feet of open channel downstream of the confluence with the North Branch of Whitnall Park Creek, are proposed to be improved. The street system required to support future urban development should be carefully designed with respect to location, configuration, and horizontal and vertical alignment to provide the necessary major drainage system capacity. In addition, an apartment building located along the east bank of the stream immediately upstream of W. Janesville Road is proposed to be floodproofed.

Whitnall Park Creek: To improve the stormwater drainage conditions, a 12-foot-wide by 6-foot-high concrete box culvert is proposed to be installed under a driveway located near the confluence with the Northwest Branch of Whitnall Park Creek. Approximately 150 lineal feet of open channel downstream of the confluence of the Northwest Branch of Whitnall Park Creek is proposed to be improved; and the driveway crossing located 150 feet downstream of the confluence of the Northwest Branch of Whitnall Park Creek is proposed to be removed. The street system required to support future urban development should be carefully designed with respect to location, configuration, and horizontal and vertical alignment to provide the necessary major drainage system capacity. In addition, an office building located along the south bank of the stream immediately downstream of S. 108th Street is proposed to be floodproofed.

Flood Control and Related Drainage System Plan Implementation

The recommended flood control and related drainage system plan components for the three stream reaches recommended for District jurisdiction in the Tess Corners Creek subwatershed consist of channel enclosure, channel improvement, detention facility construction, bridge replacement, street reconstruction, and structure floodproofing. It is recommended that the costs for channel enclosure, channel improvement, and demolition of bridges proposed to be replaced or removed be borne by the Milwaukee Metropolitan Sewerage District, as set forth in Table 54. It is recommended that the detention facility construction, new bridge construction, and street reconstruction be borne by the Village of Hales Corners. The structure floodproofing measures would be undertaken by the property owners directly affected. It is further recommended that the professional services required to

Table 53

SELECTED CHARACTERISTICS AND COSTS OF THE RECOMMENDED FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN FOR THE TESS CORNERS CREEK SUBWATERSHED

Stream and Component Description	Estimated Cost: Capital	Estimated Cost: Annual Amortized Capital[a]	Estimated Cost: Annual Operation and Maintenance	Total Annual Cost
North Branch Whitnall Park Creek				
1. 1,420 feet of 48-inch buried conduit in 113th Street from Edgerton Avenue to Upham Avenue	$ 364,000	$23,100	$ 0[b]	$23,100
2. 940 feet of 54-inch buried conduit in 113th Street from Upham Avenue to Copeland Avenue and in Copeland Avenue from 113th Street to 112th Street	279,000	17,700	0[b]	17,700
3. 5.3 acre-foot detention facility at Hales Corners Lutheran School	131,000	8,300	5,300	13,600
4. 580 feet of 60-inch storm sewer from detention facility to Grange Avenue	200,000	12,700	0[b]	12,700
5. 210 feet of 66-inch storm sewer from Grange Avenue south 180 feet	82,200	5,200	0[b]	5,200
6. 1,100 feet of channel reconstruction upstream of the confluence with the Northwest Branch Whitnall Park Creek	68,600	4,300	400	4,700
7. 1,250 feet of road reconstruction in Copeland Avenue from 111th Street to west of 112th Street, in 112th Street from Copeland Avenue to Grange Avenue, and in Grange Avenue east and west of 112th Street	80,000	5,100	0	5,100
Subtotal	$1,204,800	$76,400	$5,700	$82,100
Northwest Branch Whitnall Park Creek				
1. Install one 30-foot-long by 12-foot-wide by 6-foot-deep concrete box culvert under driveway immediately north of Janesville Road	$ 32,100	$ 2,000	$ 0[b]	$ 2,000
2. Install one 30-foot-long by 12-foot-wide by 6-foot-deep concrete box culvert under driveway 200 feet south of Janesville Road	32,100	2,000	0	2,000
3. 400 feet of channel improvement downstream of Janesville Road to confluence with Whitnall Park Creek	10,100	700	200	900
4. 300 feet of channel reconstruction downstream of confluence of North Branch Whitnall Park Creek	9,900	600	200	800
5. Floodproof one building	23,900	1,500	0	1,500
Subtotal	$ 108,100	$ 6,800	$ 400	$ 7,200

Table 53 (continued)

Stream and Component Description	Estimated Cost			Total Annual Cost
	Capital	Annual Amortized Capital[a]	Annual Operation and Maintenance	
Whitnall Park Creek				
1. Install one 25-foot-long by 16-foot-wide by 6-foot-deep concrete box culvert under driveway at confluence of Northwest Branch Whitnall Park Creek	$ 33,000	$ 2,100	$ 0	$ 2,100
2. 150 feet of channel improvement downstream of confluence of Northwest Branch Whitnall Park Creek	3,800	300	100	400
3. Remove one driveway crossing 150 feet downstream of confluence of Northwest Branch Whitnall Park Creek	5,000	400	0	400
4. Floodproof one structure	78,500	5,000	0	5,000
Subtotal	$ 120,300	$ 7,800	$ 100	$ 7,900
Total	$1,433,200	$91,000	$6,200	$97,200

[a] *Annual amortized capital cost is based on an annual interest rate of 6 percent and an amortization period and project life of 50 years.*

[b] *Costs were noted to be zero when the project proposed replacement of a component with a component which has similar operation and maintenance costs.*

Source: SEWRPC.

Table 54

SUMMARY OF RECOMMENDED PLAN CAPITAL COSTS FOR THE TESS CORNERS CREEK SUBWATERSHED

Implementing Agency	Improvements	Estimated Capital Cost
Milwaukee Metropolitan Sewerage Commission	Channel enclosure	$ 925,200
	Channel improvement	92,400
	Bridge and culvert removal	20,000
	Subtotal	$1,037,600
Village of Hales Corners	Detention facility	$ 131,000
	Culvert construction	82,200
	Street reconstruction	80,000
	Subtotal	$ 293,200
	Total	$1,330,800

Source: SEWRPC.

Map 100

RECOMMENDED FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN FOR TESS CORNERS CREEK, WHITNALL PARK CREEK, NORTHWEST BRANCH WHITNALL PARK CREEK, AND NORTH BRANCH WHITNALL PARK CREEK UNDER YEAR 2000 LAND USE AND PLANNED CHANNEL CONDITIONS

LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS

1.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR TESS CORNERS CREEK IS SHOWN IN APPENDIX H

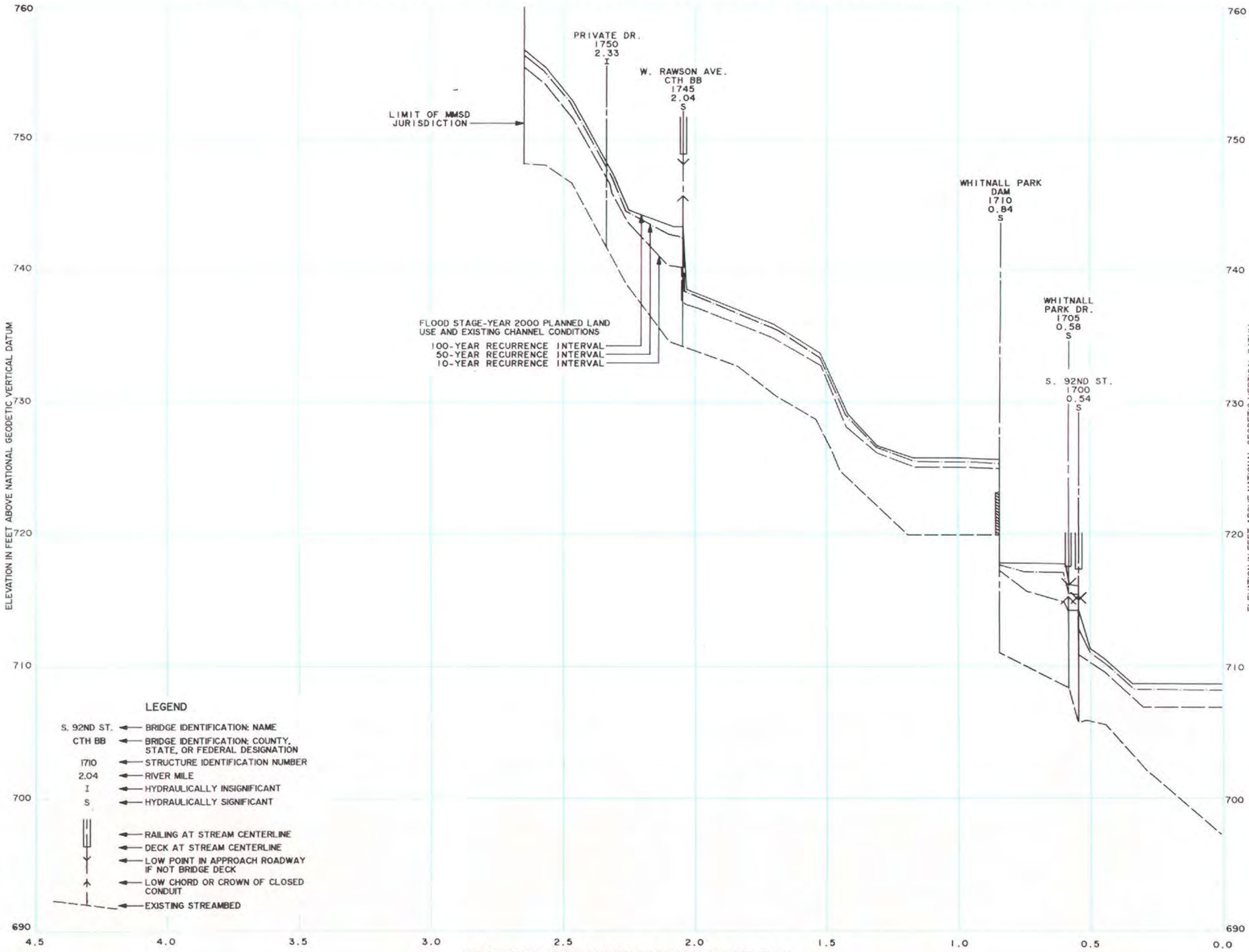
GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Figure 40

RECOMMENDED PLAN FLOOD STAGE PROFILES FOR TESS CORNERS CREEK, WHITNALL PARK CREEK, NORTHWEST BRANCH WHITNALL PARK CREEK, AND NORTH BRANCH WHITNALL PARK CREEK



Map 100 (continued)

LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS

1.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

STRUCTURE FLOODPROOFING

PROPOSED OPEN CHANNEL IMPROVEMENT

PROPOSED DRIVEWAY CROSSING REMOVAL

PROPOSED CULVERT

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR WHITNALL PARK CREEK IS SHOWN IN APPENDIX H

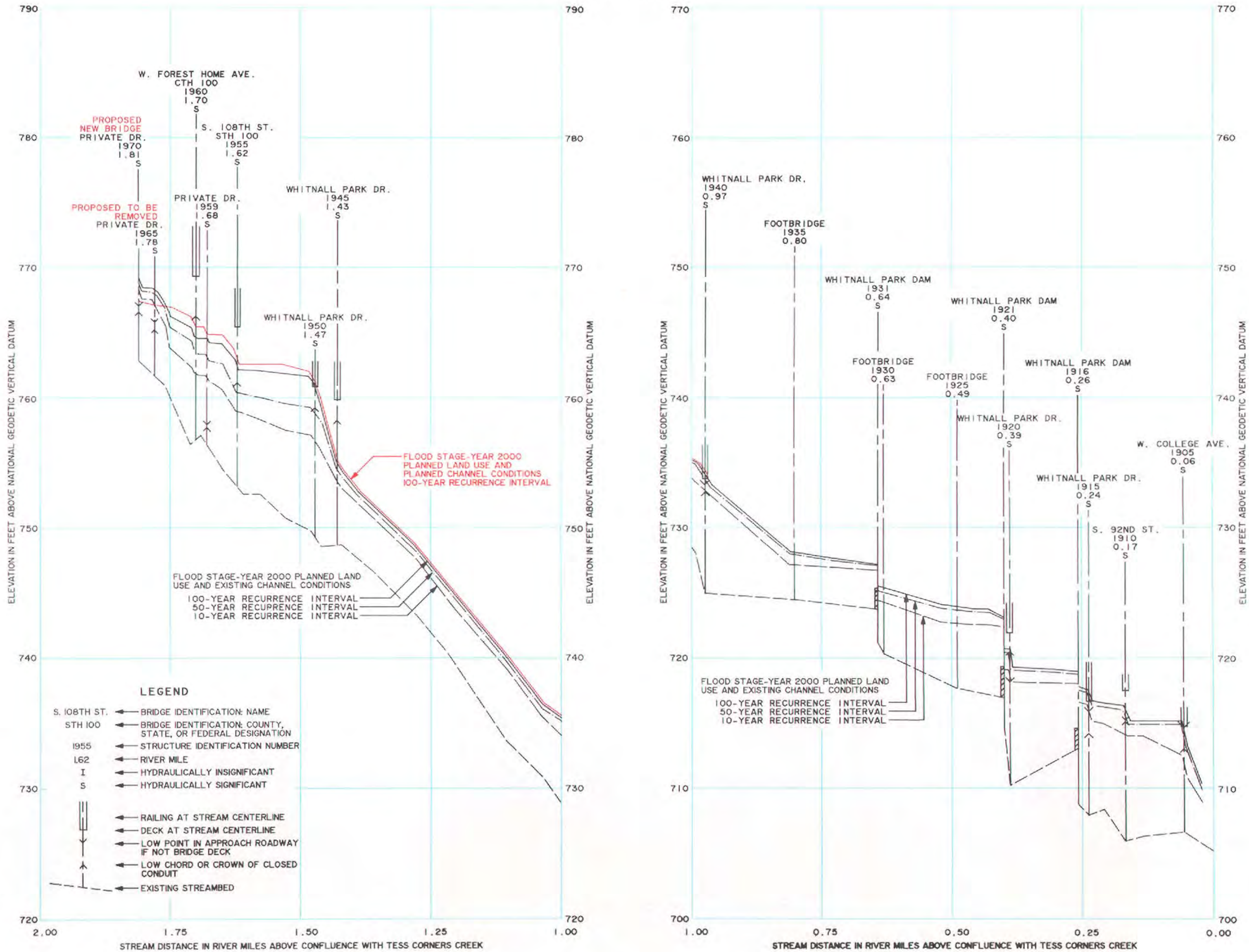
DUE TO MAP SCALE LIMITATIONS, THE DIFFERENCE BETWEEN THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS, AND THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS, MAY NOT APPEAR ON THIS MAP. WHERE NO DIFFERENCE APPEARS REFERENCE SHOULD BE MADE TO THE FLOOD STAGE PROFILE SHOWN BELOW

GRAPHIC SCALE

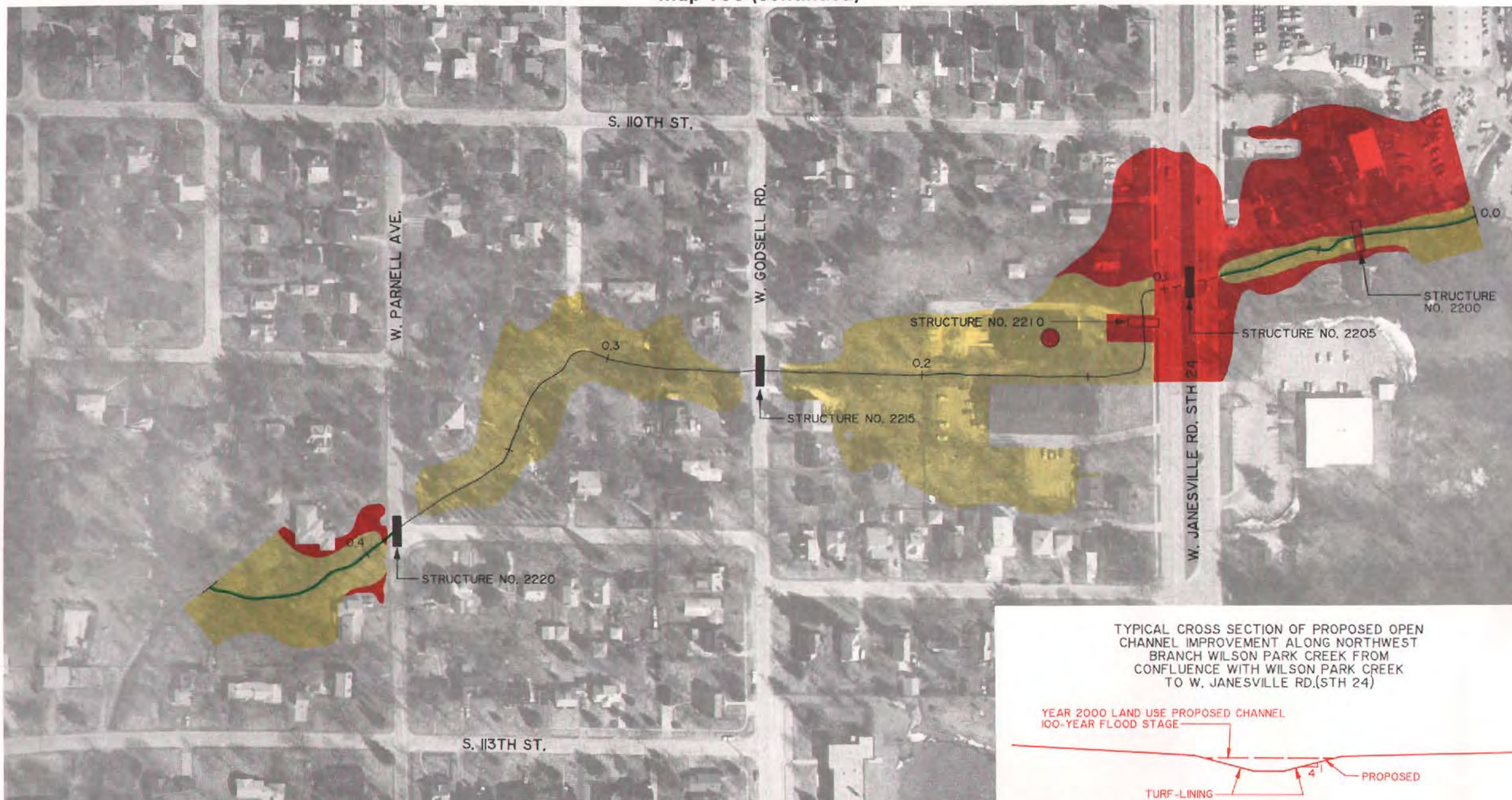
0 400 800 1200 FEET

DATE OF PHOTOGRAPHY: MARCH 1985

Figure 40 (continued)



Map 100 (continued)



TYPICAL CROSS SECTION OF PROPOSED OPEN CHANNEL IMPROVEMENT ALONG NORTHWEST BRANCH WILSON PARK CREEK FROM CONFLUENCE WITH WILSON PARK CREEK TO W. JANESVILLE RD.(STH 24)

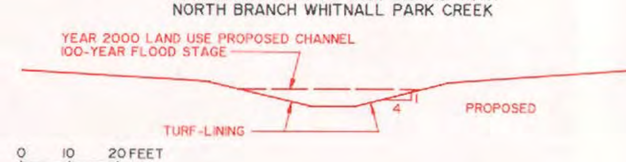
YEAR 2000 LAND USE PROPOSED CHANNEL 100-YEAR FLOOD STAGE

PROPOSED

TURF-LINING

0 10 20 FEET

TYPICAL CROSS SECTION OF PROPOSED OPEN CHANNEL IMPROVEMENT ALONG NORTHWEST BRANCH WILSON PARK CREEK FROM W. PARNELL AVE. TO CONFLUENCE WITH NORTH BRANCH WHITNALL PARK CREEK



LEGEND

- 100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS
- 100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS
- 0.2 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING
- STRUCTURE FLOODPROOFING
- PROPOSED OPEN CHANNEL IMPROVEMENT
- PROPOSED CULVERT

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR NORTHWEST BRANCH OF WHITNALL PARK CREEK IS SHOWN IN APPENDIX H

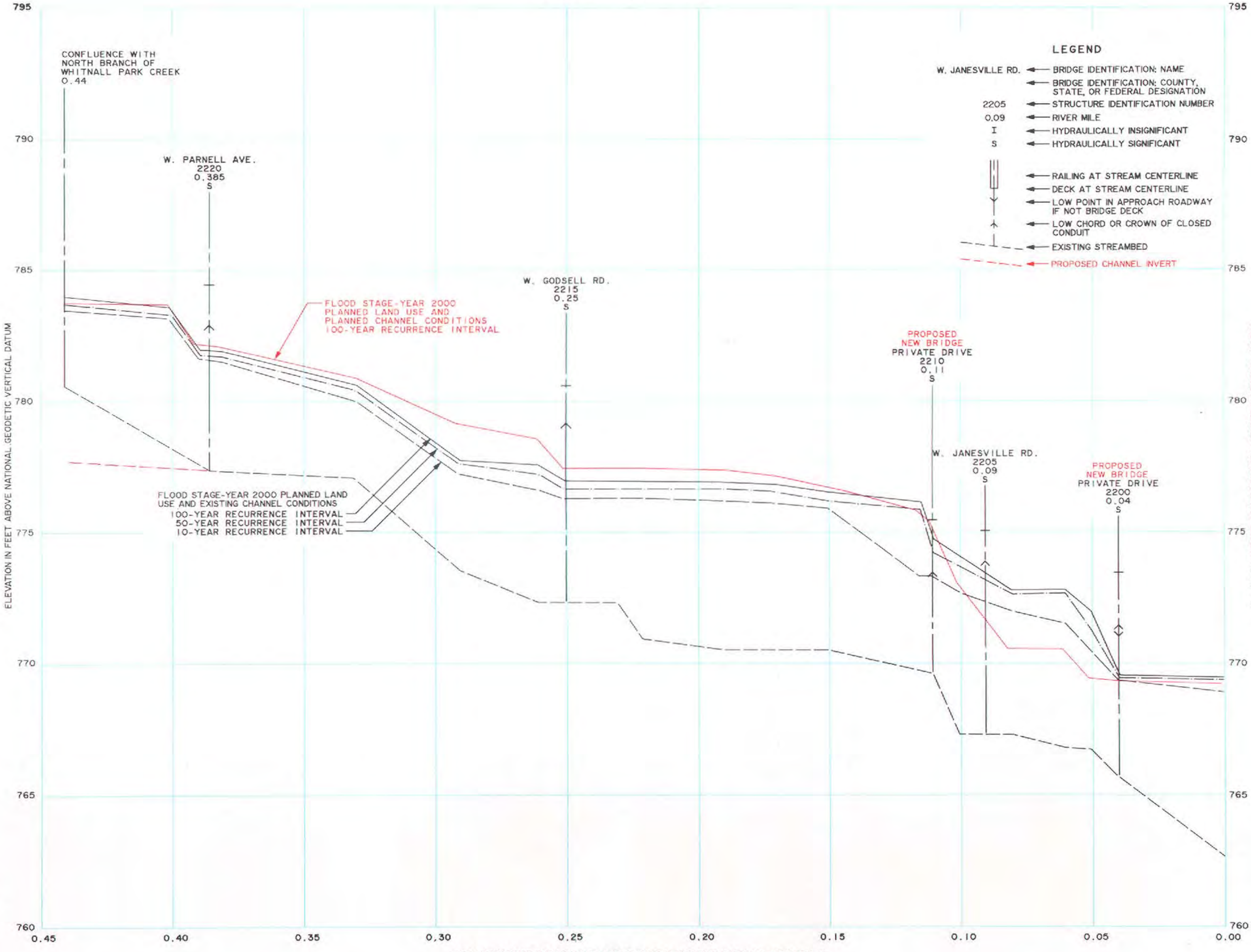
DUE TO MAP SCALE LIMITATIONS, THE DIFFERENCE BETWEEN THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS, AND THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS, MAY NOT APPEAR ON THIS MAP. WHERE NO DIFFERENCE APPEARS REFERENCE SHOULD BE MADE TO THE FLOOD STAGE PROFILE SHOWN BELOW

GRAPHIC SCALE

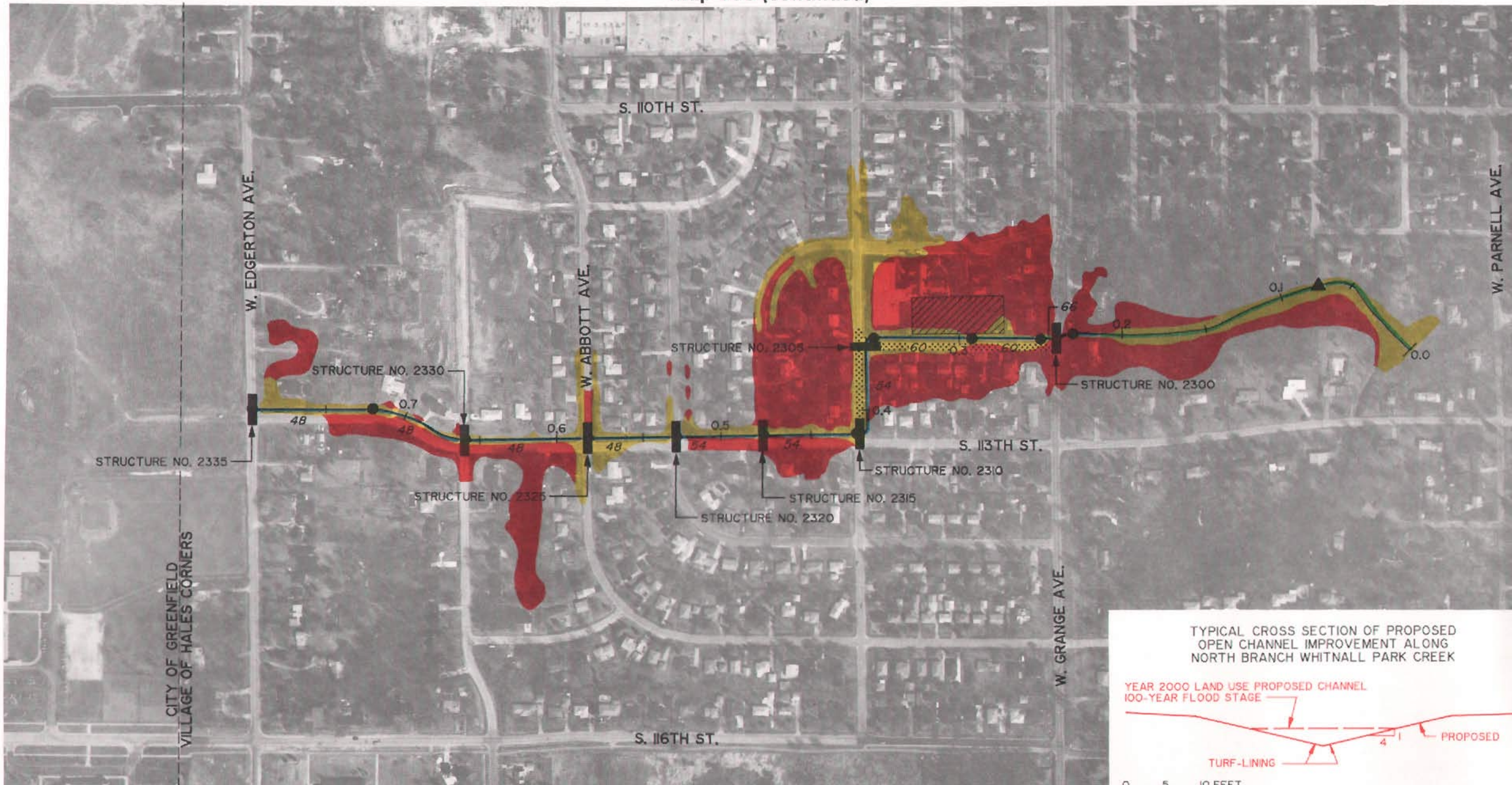
0 100 200 300 FEET

DATE OF PHOTOGRAPHY: MARCH 1985

Figure 40 (continued)



Map 100 (continued)



LEGEND

- 100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS
- 100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS
- 0.5 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING
- 60 PROPOSED CHANNEL ENCLOSURE AND CONDUIT SIZE IN INCHES
- PROPOSED MANHOLE
- PROPOSED STORM SEWER OUTFALL
- PROPOSED OPEN CHANNEL IMPROVEMENT
- PROPOSED DETENTION FACILITY
- PROPOSED ROAD RECONSTRUCTION

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR NORTH BRANCH OF WHITNALL PARK CREEK IS SHOWN IN APPENDIX H

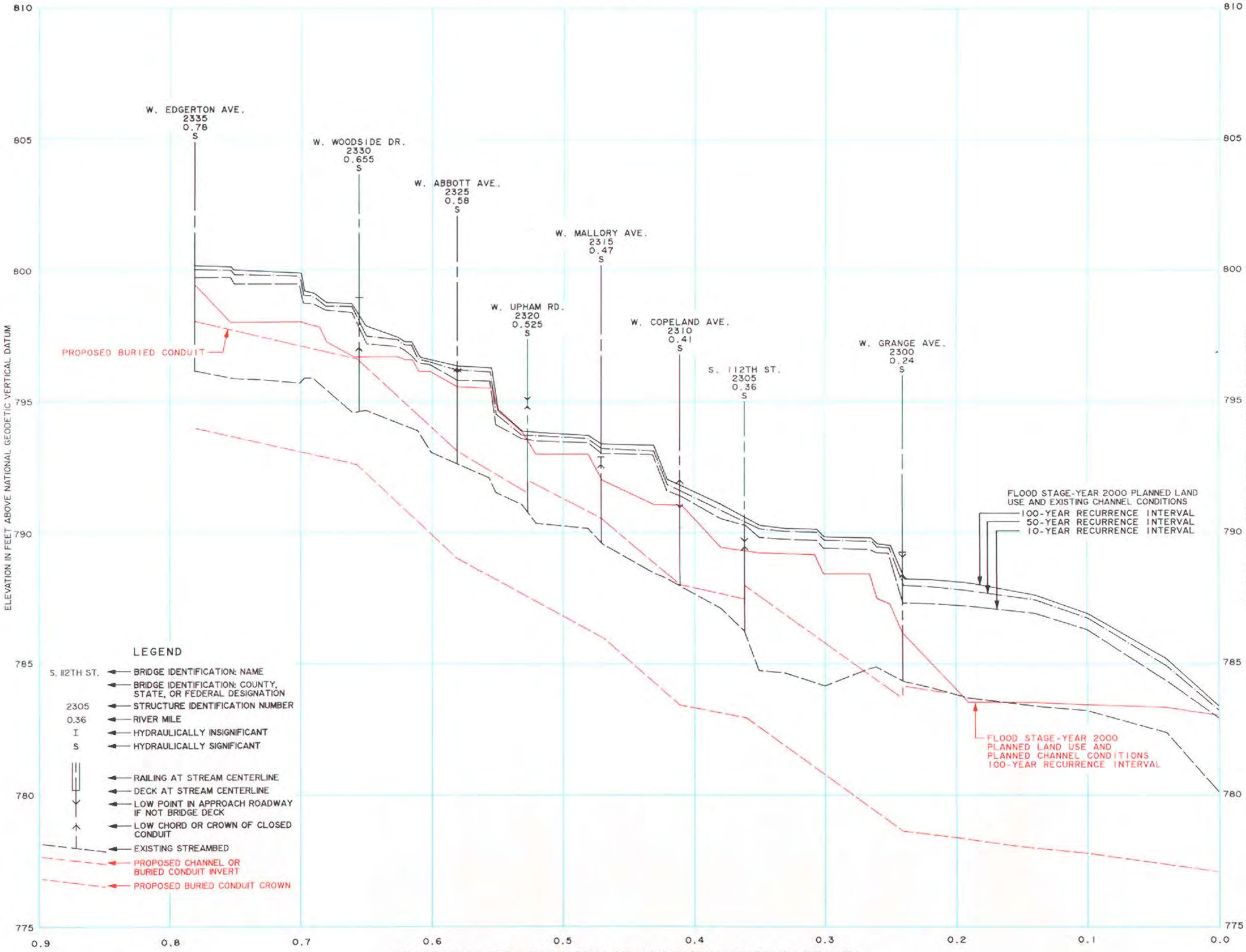
NOTE: DUE TO MAP SCALE LIMITATIONS, THE DIFFERENCE BETWEEN THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS, AND THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS, MAY NOT APPEAR ON THIS MAP. WHERE NO DIFFERENCE APPEARS REFERENCE SHOULD BE MADE TO THE FLOOD STAGE PROFILE SHOWN BELOW

GRAPHIC SCALE
0 200 400 600 FEET

DATE OF PHOTOGRAPHY: MARCH 1985

Source: W. G. Nienow Engineering Associates and SEWRPC.

Figure 40 (continued)



Source: SEWRPC.

prepare plans for the floodproofing and elevation of individual buildings be made available, at no cost, to property owners by the Village of Hales Corners engineering department. Also, it is recommended that the Village of Hales Corners review its building code to ensure that appropriate floodproofing regulations are included. It is recommended that the Village explore, on behalf of the property owners involved, any available state and/or federal aids for such floodproofing measures.

CRAYFISH CREEK SUBWATERSHED FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN

As already noted, drainage and flood control improvements for the Crayfish Creek subwatershed were considered in a stormwater management study conducted by the Regional Planning Commission for the City of Oak Creek in June 1988. The purpose of the study was to describe the stormwater drainage and flood control problems and identify the causes of those problems; evaluate alternative means by which the identified problems could be alleviated and drainage improved; and set forth a recommended stormwater management plan based upon that evaluation. The findings and recommendations of the study are presented in SEWRPC Memorandum Report No. 35, <u>A Stormwater Management Plan for the Crayfish Creek Subwatershed, City of Oak Creek, Milwaukee County, Wisconsin</u>, June 1988.

Overview of the Subwatershed

The Crayfish Creek subwatershed, as shown on Map 101, is located within the corporate limits of the City of Oak Creek and the Town of Caledonia. The stream reaches in the subwatershed recommended for District jurisdiction are as follows: 1) a 0.53-mile reach of Crayfish Creek from the County Line Road crossing upstream to the Elm Road crossing; and 2) a 0.43-mile reach of Caledonia Branch from its confluence with Crayfish Creek upstream to the abandoned railroad crossing. Both of these stream reaches are classified as perennial.

In 1985 the Crayfish Creek subwatershed—with a total area of 5.78 square miles—was still largely undeveloped, with 4.38 square miles, or about 76 percent, devoted to agricultural and other open space uses. The remaining 1.40 square miles of the subwatershed was developed for urban use, with 1.21 square miles, or about 87 percent, consisting of residential uses, and the remaining 0.19 square mile consisting of commercial and industrial uses. Municipal street improvements in the developed portions of the subwatershed generally consist only of paved streets without curbs and gutters and storm sewers. Therefore, most of the surface runoff is carried by roadside ditches which discharge to surface watercourses and streams. Some of the major tributary streams, including the reach of Crayfish Creek recommended for District jurisdiction, have been modified in varying degrees by deepening or realignment. The reach of Caledonia Branch recommended for District jurisdiction, however, has not been modified.

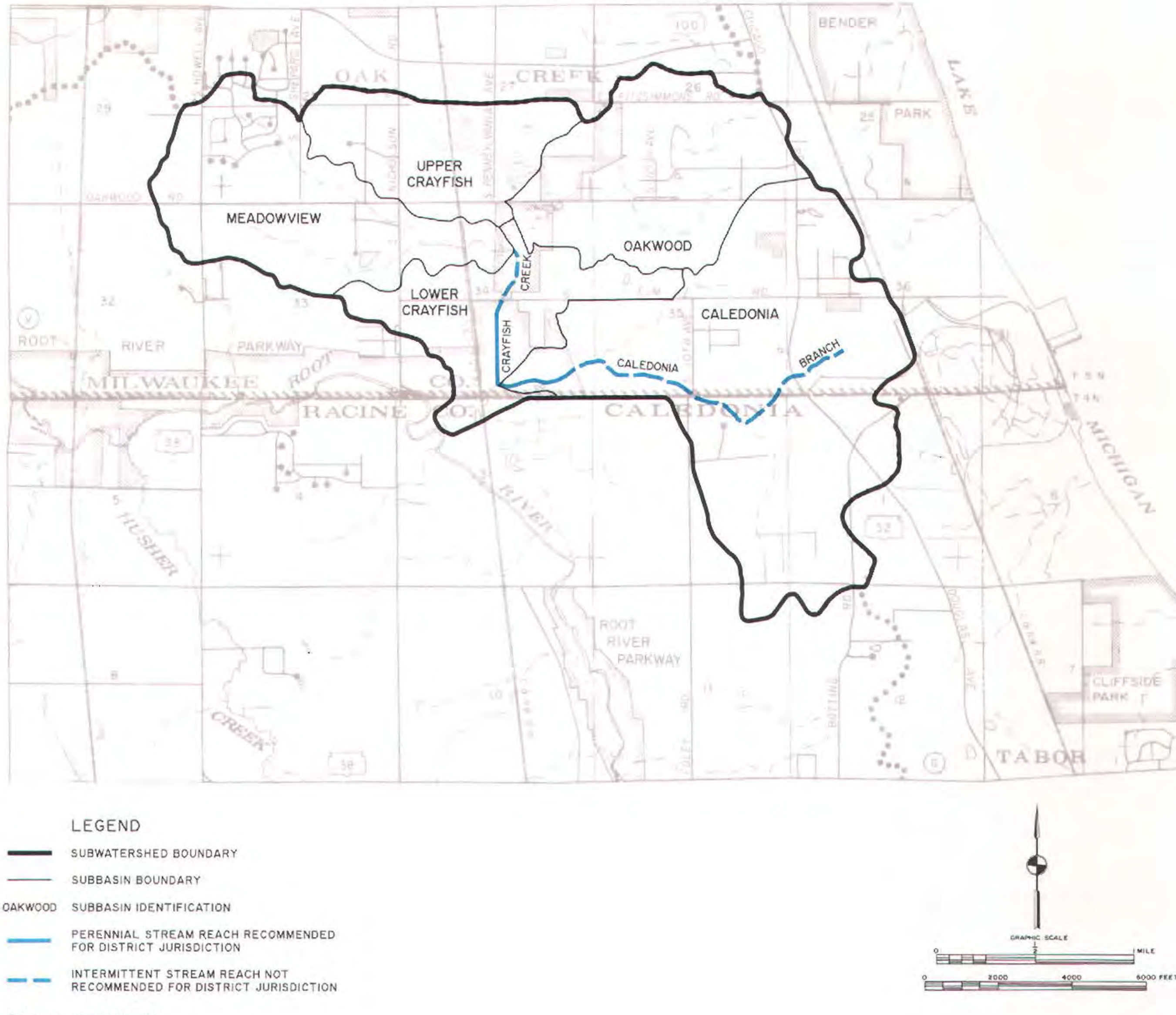
The storm sewer system within the subwatershed consists of approximately 12,350 lineal feet of sewer, ranging in size from 12 inches in diameter to 66 inches in diameter. There are also 900 lineal feet of low-flow ditch drain pipe and 950 lineal feet of ditch enclosure pipe. Most of the sewers and drains are constructed of reinforced concrete pipe.

Flooding and Related Drainage Problems

The Crayfish Creek subwatershed experiences both drainage and flood control problems. The watershed divide generally lies between Fitzsimmons Road extended and Oakwood Road, but because of the level topography is ill-defined. Roadside ditches and culverts near the divide are subject to flooding and backwater from both the main Oak Creek channel and the main Root River channel by flood events as small as a two-year recurrence interval event on either stream. In addition, lands in the vicinity of the drainage divide are subject to flooding from channel backwaters of both streams during flood events as small as a five-year recurrence interval event. Allowing for the flow gradients needed to drain the Crayfish Creek subwatershed into the network of channels leading to the Root River, flooding may be expected to be substantially more frequent than stated. This is borne out by a history of complaints from residents of the subwatershed indicating that flooding may indeed occur several times a year. The drainage and flooding problems are aggravated by the existence of drainage ditches with very flat slopes and conveyance capacities which provide inadequate outlets for local storm drainage facilities.

Map 101

THE CRAYFISH CREEK SUBWATERSHED



Source: SEWRPC.

The presence of wet or poorly drained soils in the area contributes to the drainage and flooding problems. The poorly drained soils can be developed only with the aid of costly special measures such as under-drainage systems and artificial fill. Sanitary sewers located in these areas will be susceptible to high rates of groundwater infiltration. Stormwater which accumulates in these areas during and following storm events may pose health hazards, hamper transportation by inundating streets, flood basements, and serve as breeding sites for mosquitoes.

Although varying degrees of overland flooding occur frequently within the study area, and the potential exists for widespread flooding from a major storm event, the only monetary flood losses that should be expected under existing land use and channel conditions are those associated with crop damages, residential yard flooding, and indirect flooding of basements. No residential structures are located within the limits of the 100-year recurrence interval floodplain. There should be no potential for crop damage under year 2000 land use conditions along the stream reaches recommended for District jurisdiction, since all of the remaining agricultural lands in the watershed are proposed to be converted to residential or urban open space uses.

The drainage and flood control objectives and supporting principles and standards set forth in Chapter III specify the flood events which bridges shall accommodate without overtopping of the related roadway. Based on these criteria, the bridge at E. County Line Road on Crayfish Creek is considered hydraulically inadequate, as shown in Appendix E.

Flood Discharges and Stages

Stormwater runoff rates in the study area were estimated using three techniques. The first technique is one developed specifically for estimating the magnitude and frequency of floods on waterways in urban areas of Wisconsin. The method was developed by the U. S. Geological Survey in a cooperative project involving that agency, the Milwaukee Metropolitan Sewerage District, and the Southeastern Wisconsin Regional Planning Commission. The method uses equations developed by multiple-regression analyses to compute flood magnitudes and frequencies. The significant characteristics considered in the equations are recurrence interval, drainage area, and percent imperviousness based upon land use.

The second technique used was the U. S. Soil Conservation Service TR 55 method. This technique uses general empirical relationships between characteristics that affect runoff and peak rates of discharge in small watersheds to compute flood magnitudes and frequencies. These characteristics include recurrence interval, drainage area, watershed slope, percent ponds and wetland, land use and attendant percent impervious area, vegetative cover, rainfall depth, hydrologic soil group, and maximum watershed hydraulic length.

The third technique used was a mathematical simulation model known as the runoff hydrograph and routing model (HYDROUT). This model uses the continuous rainfall pattern for the selected recurrence interval design storms based on results of the intensity-duration-frequency analyses. Such analyses have been performed by the Regional Planning Commission on Milwaukee area meteorological data. In the application of this method, the study area is divided into catchment areas, and a runoff hydrograph is produced for each area. The hydrograph is a product of the rainfall pattern, the U. S. Soil Conservation Service runoff curve number used in the conversion of rainfall to runoff, and a dimensionless index hydrograph. These hydrographs are combined and hydrologically routed downstream from one critical location in the system to the next to provide system hydraulic loadings in the form of peak flow rates and total flow volumes. The reservoir routing mode allows for the routing of the flow-through hydrograph. These hydrographs are combined and hydrologically routed downstream to provide system hydraulic loadings in the form of peak flow rates and total flow volumes. The reservoir routing mode allows for the routing of the flow through a reservoir based on the storage and outflow characteristics of the reservoir. The output hydrograph produced in this mode can then be combined with additional hydrographs as it is routed downstream via conveyance facilities or through additional reservoirs. This simulation model allows the effects of multiple, sequential reservoir storage facilities on downstream peak flow rates to be evaluated. The estimated peak flood discharges under existing and planned, year 2000 land use conditions and existing channel conditions are set forth in Table 55, and are those derived from the application of the simulation model.

Flood stage profiles were determined for the 10-, 50-, and 100-year recurrence interval runoff events under planned land use and existing channel conditions. These profiles, which encompass the full 0.96 mile of stream reaches recommended for District jurisdiction in the Crayfish Creek subwatershed, constitute a graphic representation of the flood stages along the two studied stream reaches under the specified recurrence interval flood discharges and under planned land use and existing channel conditions. In addition to providing an overall representation of flood stages relative to familiar points of reference, such as the channel bottom and bridge deck surfaces, the profiles, because of their continuity, permit the determination of flood stages at any point along the stream channel. These flood profiles are shown in Figure 41. The extent of the 100-year recurrence interval floodplain under planned land use conditions is shown on Map 102. This delineation of the flood hazard area was accomplished using large-scale topographic maps.

Table 55

PEAK FLOOD DISCHARGES FROM SUBBASINS IN THE CRAYFISH CREEK SUBWATERSHED

Subbasin	Peak Flows (cubic feet per second)					
	Existing 1980 Conditions			Year 2000 Conditions		
	10-Year	50-Year	100-Year	10-Year	50-Year	100-Year
Upper Crayfish	60	85	90	125	175	195
Oakwood	85	115	125	175	240	270
Meadowview	110	113	115	215	220	225
Caledonia	125	170	190	240	350	400
Lower Crayfish	290	360	385	535	720	815

Source: SEWRPC.

Alternative Flood Control and Related Drainage System Plans for the Crayfish Creek Subwatershed

Seventeen alternative plans were developed and evaluated under the Crayfish Creek subwatershed study. The 17 alternative plans included conveyance, detention, and backwater prevention facilities, or a combination of such facilities. In addition, retention and nonstructural measures were incorporated into all of the plan alternatives in that the recommended land use pattern for the subwatershed included maintenance in essentially natural, open uses of the primary environmental corridor lands, including wetland areas, along Crayfish Creek.

All alternatives required the development of an internal system of open channels to provide collection capability. Certain alternatives involved modification of this basic internal open channel collection system. This is true of those alternatives under which the proposed outlet location of the subwatershed was to be in other than at its present location. The collection channels were assumed under each of the alternatives to be turf-lined, with side slopes of one on four.

Because of the complexity of the existing flooding and drainage problems, alternative plans were developed that would mitigate these problems to varying degrees. In the alternative evaluation, the alternatives were placed in one of three categories based on the degree to which they served to mitigate flooding and drainage problems.

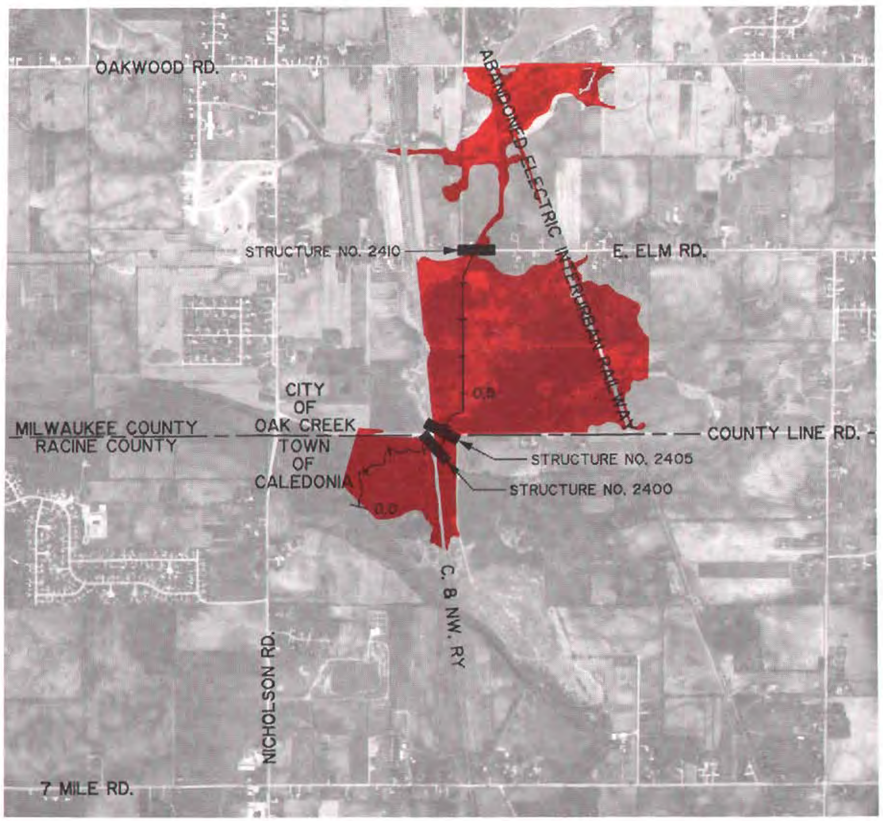
The first category of mitigation applied to alternatives with an improved internal drainage system within the Crayfish Creek subwatershed, but did not provide for the elimination of the backed-up floodwaters from the Root River and Oak Creek into the subwatershed. These alternative were categorized as providing a limited degree of abatement of the flooding and drainage problems.

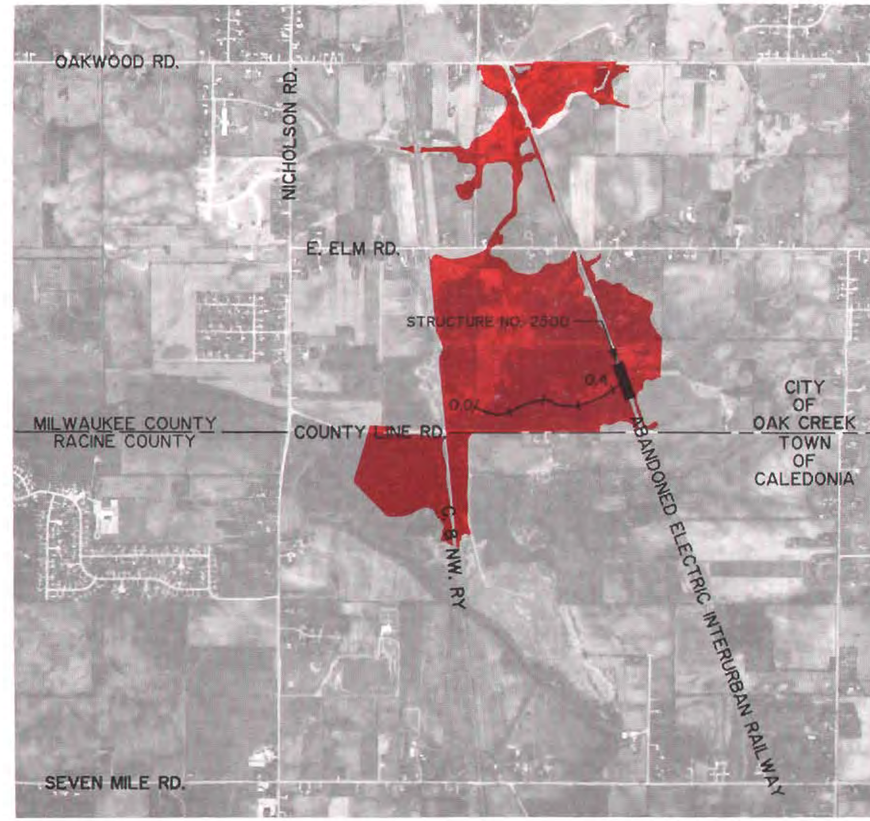
The second category of mitigation applied to alternatives with an improved internal drainage system within the Crayfish Creek subwatershed, and with provisions to seal off floodwaters from the Root River and Oak Creek. Under this category of alternatives, the outlets from the watershed would be closed during flooding periods on the Root River and Oak Creek, and thus some flooding and drainage problems would remain. These alternatives were considered to provide a significant degree of abatement of flooding and drainage problems.

Finally, the third category of mitigation applied to those alternatives with improved internal drainage systems within the Crayfish Creek subwatershed, provisions to seal off the backed-

Map 102

100-YEAR RECURRENCE INTERVAL FLOODPLAIN FOR CRAYFISH CREEK AND CALEDONIA BRANCH UNDER YEAR 2000 LAND USE AND EXISTING CHANNEL CONDITIONS





LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

0.4 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR CRAYFISH CREEK IS SHOWN IN APPENDIX H

GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

0.4 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR CALEDONIA BRANCH IS SHOWN IN APPENDIX H

GRAPHIC SCALE

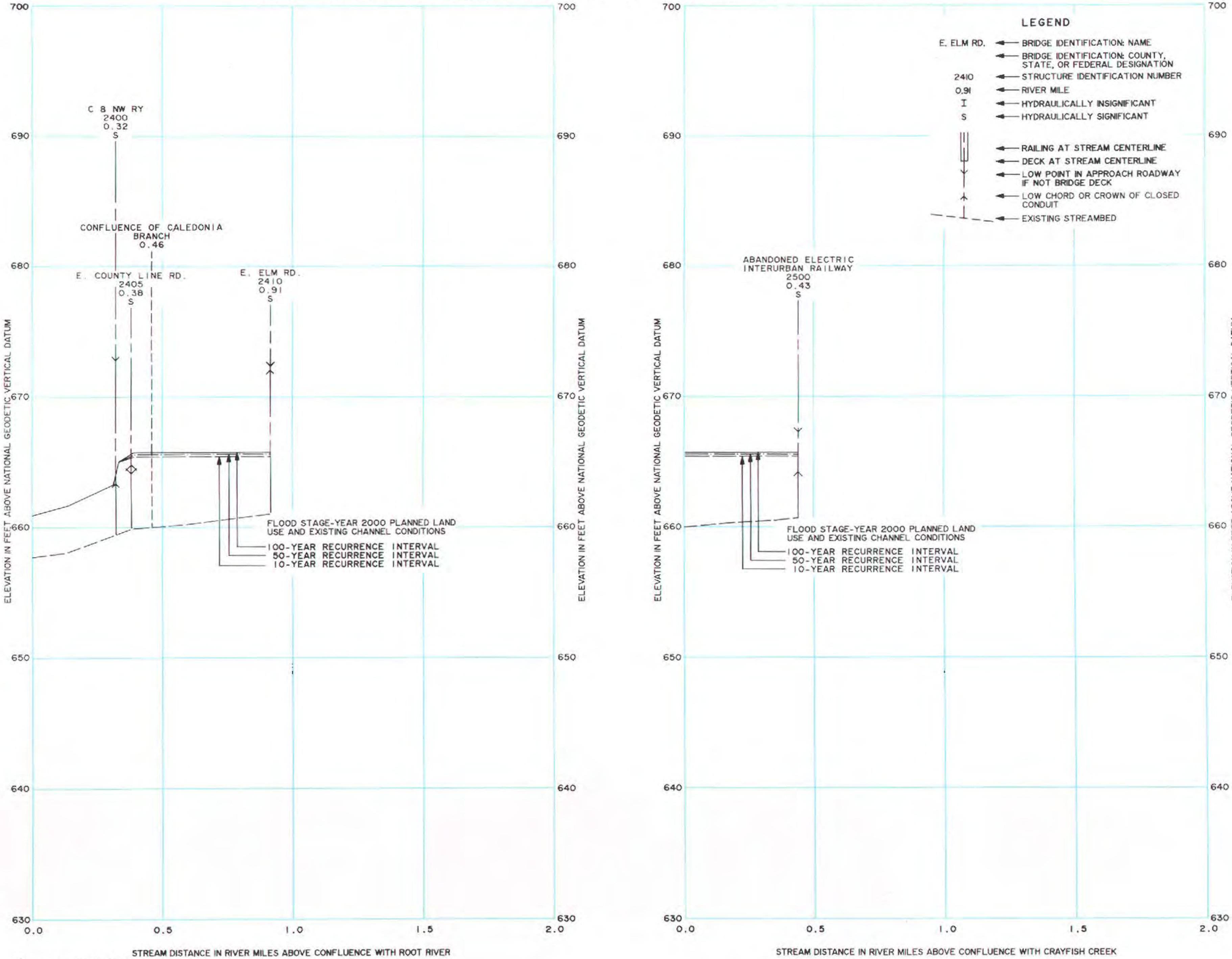
0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Source: SEWRPC.

Figure 41

FLOOD STAGE AND STREAMBED PROFILES FOR CRAYFISH CREEK AND CALEDONIA BRANCH



Source: SEWRPC.

Table 56

PERTINENT CHARACTERISTICS OF THE ALTERNATIVE PLANS CONSIDERED UNDER THE CRAYFISH CREEK SUBWATERSHED STORMWATER MANAGEMENT PLANNING PROGRAM

Alternative Number	Cost Rank	Collection Channel	Conveyance Channel	Backwater Gates	Pump	Storage	Force Main	Capital Cost	
								2-Year Recurrence Interval Design Storm	10-Year Recurrence Interval Design Storm
1	1	X	X					$ 800,000	$ 900,000
2	4	X	X					1,400,000	1,600,000
3	14	X	X					7,900,000	8,900,000
4	15	X	X					7,900,000	8,900,000
5	16	X	X					9,500,000	11,000,000
6	17	X	X					9,500,000	11,000,000
7	2	X	X	X				1,100,000	1,300,000
8	5	X	X	X				1,700,000	1,900,000
9	6	X			X		X	2,300,000	2,800,000
10	9	X	X		X		X	3,200,000	3,800,000
11	13	X	X		X		X	4,800,000	5,700,000
12	10	X	X	X		X		2,600,000	4,200,000
13	12	X	X	X		X		3,600,000	5,400,000
14	7	X			X	X	X	2,300,000	3,100,000
15	8	X			X	X	X	2,800,000	3,500,000
16	11	X	X		X	X	X	4,200,000	4,900,000
17	3	X	X	X				1,200,000	1,400,000

Source: SEWRPC.

up floodwaters from the Root River and Oak Creek, and an adequate outlet from the subwatershed at all times. These alternatives were categorized as providing a high degree of abatement of the flooding and drainage problems.

Pertinent characteristics of each of the alternative approaches are set forth in Table 56. Based upon an evaluation of these alternatives, seven were identified for further consideration. The comparative evaluation of these plans was focused primarily on the cost of the stormwater management system components, and on the ability of the plans to resolve stormwater drainage problems in the subwatershed. Additional criteria considered in the comparative evaluation were water quality protection, development restriction, operation and maintenance requirements, impact on downstream flood flows and stages, and public acceptance. The findings

Table 57

SUMMARY COMPARISON OF ALTERNATIVE STORMWATER MANAGEMENT SYSTEM PLANS

Alternative Plan	Abatement of Drainage and Flooding Problems	Water Quality	50-Year 6 Percent Equivalent Average Annual Cost	Development Restrictions	Operation and Maintenance Requirements	Impact on Downstream Flows	Public Acceptance
4—Diversion Conveyance to Lake Michigan	Flooding and drainage problems would be abated to a high degree	Increased pollutant loadings would be discharged directly to Lake Michigan, while pollutant loadings to the Root River would decrease. Some pollutant removal would be provided by the grass-lined open conveyance channel	$507,000 for 2-year; $571,000 for 10-year	High	Low	Low	Low
7—Conveyance to the Root River via Route A	Flooding and drainage problems would be abated to a significant degree Outlet from the subwatershed would be closed during Root River flooding	Pollutant loadings would continue to be discharged to the Root River. Some pollutant removal would be provided by the grass-lined open conveyance channel	$74,000 for 2-year; $86,000 for 10-year	Low	Low	Low	Moderate
9—Pumping to the Root River	Flooding and drainage problems would be abated to a high degree	Pollutant loadings would continue to be discharged to the Root River. Some pollutant removal would be provided by the grass-lined open conveyance channel	$156,000 for 2-year; $188,000 for 10-year	Low	Moderate	Low	High
11—Diversion Pumping to Lake Michigan	Flooding and drainage problems would be abated to a high degree	Increased pollutant loadings would be discharged directly to Lake Michigan, while pollutant loadings to the Root River would decrease. Some pollutant removal would be provided by the grass-lined open conveyance channel	$317,000 for 2-year; $375,000 for 10-year	Moderate	Moderate	Low	Low
12—Storage and Conveyance to the Root River	Flooding and drainage problems would be abated to a significant degree Outlet from the subwatershed would be closed during Root River flooding	Pollutant loadings would continue to be discharged to the Root River. Some pollutant removal would be provided by the grass-lined open conveyance channel. If a detention pond is utilized, substantial pollutant removal would be achieved	$195,000 for 2-year; $316,000 for 10-year	Low	Moderate	Low	Moderate
14—Storage and Pumping to the Root River	Flooding and drainage problems would be abated to a high degree	Pollutant loadings would continue to be discharged to the Root River. Some pollutant removal would be provided by the grass-lined open conveyance channel. If a detention pond is utilized, substantial pollutant removal would be achieved	$171,000 for 2-year; $242,000 for 10-year	Low	Moderate	Low	Moderate
17—Conveyance to the Root River via Route D	Flooding and drainage problems would be abated to a significant degree Outlet from the subwatershed would be closed during Root River flooding	Pollutant loadings would continue to be discharged to the Root River. Some pollutant removal would be provided by the grass-lined open conveyance channel. Potential exists for use of existing pond located south of County Line Road for pollutant removal	$79,000 for 2-year; $92,000 for 10-year	Moderate	Low	Low	Moderate

Source: SEWRPC.

of this comparative evaluation are summarized in Table 57. The evaluation indicated that Alternative 17, with some refinements and installed in two phases, would best serve the Crayfish Creek subwatershed. That alternative plan would provide for the discharge of stormwater to the Root River at a location about 0.9 mile downstream of the present location; the provision of backwater facilities to prevent flood flows on the Root River and Oak Creek from backing up into Crayfish Creek; and the provision of stormwater detention facilities for both flood flow reduction and water quality management purposes.

Recommended Flood Control and Related Drainage System Plan for the Crayfish Creek Subwatershed

The Crayfish Creek subwatershed plan proposes improved drainage and flood control facilities for the entire subwatershed. The key improvements are listed in Table 58 and Table 59. As already noted, however, only two stream reaches within the subwatershed are recommended for District jurisdiction. The recommended plan components for these two stream reaches are described below. In addition, required improvements on downstream reaches of Crayfish Creek in Racine County are included since they are an integral part of the Crayfish Creek channel improvements and are required to provide an adequate outlet for the Crayfish Creek main channel.

The recommended flood control and related drainage system plan components for the stream reaches in the Crayfish Creek subwatershed specifically recommended for District jurisdiction are set forth in Table 60, along with the associated costs. The recommended plan components for the subject stream reaches consist of minor channel improvement, culvert replacement, and new channel construction and roadway reconstruction. These conveyance components would be designed to accommodate flows up to and including the 100-year recurrence interval event. The recommended plan components for the subject stream reaches are summarized in graphic form on Map 103. The recommended plan 100-year recurrence interval flood profile is shown in Figure 42.

Crayfish Creek: The recommended components of the system plan for the Crayfish Creek subwatershed recommended for District jurisdiction include replacement of the 36-inch corrugated metal pipe (CMP) culvert at the Oakwood Road crossing of Crayfish Creek with a 48-inch CMP culvert. A berm would be constructed parallel to and 50 feet south of Fitzsimmons Road extended from a point 250 feet west of the Chicago & North Western Railway tracks eastward 3,500 feet, to S. 15th Avenue extended. The existing channel in the vicinity of the Oakwood Road crossing would be regraded to eliminate any negative slopes. County Line Road would be raised from just west of the Chicago & North Western tracks to S. 15th Avenue extended. In lieu of raising this 2,500-foot reach of W. County Line Road, an earthen berm could be constructed parallel to and immediately south of the road for this same reach. Four 72-inch-diameter corrugated metal culvert pipes would be installed under the raised roadway and equipped with backwater gates to prevent floodwaters from the Root River from entering the Crayfish Creek channel north of E. County Line Road. One of these culverts should be set at a lower elevation than the other three to accommodate fish migration during low-flow conditions. The details of this culvert system should be reviewed and approved by the Wisconsin Department of Natural Resources fish management personnel prior to installation. It is envisioned that the backwater gates would be manually operated during periods of potential flood flows and stages on the Root River. Automated operation could be provided but is not included in the system costs.

In addition, a new channel would be constructed from E. County Line Road and the Chicago & North Western tracks to the southeast to an existing reservoir, and from the reservoir to the Root River approximately 850 feet north of Seven Mile Road. The new channel would be turf-lined and have a bottom width of 25 feet, with one on four side slopes and an average depth of about five feet. Also, it is recommended that minor debrushing of the Crayfish Creek channel between E. County Line Road and E. Elm Road be carried out, with minor regrading to eliminate negative channel bottom gradients. These channel maintenance measures are recommended to be carried out periodically as necessary.

Recently discovered ground and surface water contamination caused by an abandoned landfill in the vicinity of the planned channel south of County Line Road will require reconsideration of the channel route as part of the detailed design. This evaluation would be done using the results of ongoing remedial plans for the landfill cleanup. The existing contamination problems will delay implementation actions associated with the use of the existing pond south of County Line Road as a sedimentation pond. Thus, in order to provide for improved water quality conditions, it is recommended that four sedimentation basins be constructed in the City in areas upstream of the large wetland complex along Crayfish Creek.

To accommodate stormwater drainage during periods when the Root River flows and stages

Table 58

SELECTED CHARACTERISTICS AND COSTS OF THE RECOMMENDED STORMWATER MANAGEMENT PLAN FOR THE CRAYFISH CREEK SUBWATERSHED—PHASE I

Hydrologic Unit	Project Component Description	Estimated Cost	
		Capital	Annual Operation and Maintenance[a]
Upper Crayfish	1. Replacement of the existing 36-inch corrugated metal pipe (CMP) culvert at Oakwood Road crossing of Crayfish Creek with a 48-inch CMP culvert	$ 4,000	$ 0
	2. Construction of a berm parallel to and 50 feet south of Fitzsimmons Road extended from 250 feet west of the Chicago & North Western Railway tracks 3,500 feet to the east, to S. 15th Avenue extended	60,000	1,000
	3. Regrading of existing channel in the vicinity of Oakwood Road	10,000	0
	Engineering and Contingencies	10,000	0
	Subtotal	$ 84,000	$ 1,000
Lower Crayfish	1. Reconstruction of 2,500 feet of E. County Line Road east of the Chicago & North Western Railway tracks or a berm adjacent to the road	$ 80,000	$ 0
	2. Replacement of the existing 5-foot by 17-foot concrete box culvert at the County Line Road crossing of Crayfish Creek with four 72-inch CMP culverts	40,000	0
	3. Installation of four backwater gates on County Line Road culverts	75,000	8,000
	4. Bulkheading of the four 48-inch CMP culverts at Chicago & North Western Railway crossing of Crayfish Creek	3,000	0
	5. Construction of 700 lineal feet of new open channel from County Line Road to the existing retention pond located 700 feet south of E. County Line Road and adjacent to the Chicago & North Western Railway, and 2,110 feet of new channel from that retention pond to the Root River 850 feet north of Seven Mile Road	90,000	1,000
	6. Inlet and outlet refinements to the existing pond located downstream of County Line Road	3,000	0
	Engineering and Contingencies	25,000	0
	Subtotal	$316,000	$ 9,000
	Total	$397,000	$10,000

[a]*Costs were noted to be zero when the project proposed replacement of a component with a component that has similar operation and maintenance cost.*

Source: SEWRPC.

Table 59

SELECTED CHARACTERISTICS AND COSTS OF THE RECOMMENDED STORMWATER MANAGEMENT PLAN FOR THE CRAYFISH CREEK SUBWATERSHED—PHASE II

Hydrologic Unit	Project Component Description	Estimated Cost	
		Capital	Annual Operation and Maintenance[a]
Oakwood	Subbasin No. 1 Improvements 1. 1,800 feet of 48-inch-diameter reinforced concrete pipe (RCP) along Fitzsimmons Road from 10th Avenue extended to the west	$ 180,000	$ 0
	2. 1,200 feet of 24-inch-diameter RCP along Fitzsimmons Road from S. 10th Avenue extended to the east	50,000	0
	Subbasin No. 2 Improvements 1. 900 feet of 18-inch-diameter RCP along Oakwood Road from Chicago Road to the east	30,000	0
	2. 1,300 feet of 36-inch-diameter RCP along Oakwood Road from Chicago Road to the west to the proposed channel	90,000	0
	3. 4,700 feet of channel modifications from the existing 30-inch-diameter RCP outfall to the west	30,000	0
	4. 1,100 feet of new open channel from the proposed 36-inch-diameter outfall to the subbasin divide	60,000	200
	Engineering and Contingencies	40,000	0
	Subtotal	$ 480,000	$ 200
Caledonia	Subbasin No. 1 Improvements 1. 2,800 feet of 36-inch-diameter RCP along E. Elm Road from the section line between Sections 35 and 36 to just west of S. 10th Avenue	$ 200,000	$ 0
	2. 3,900 feet of channel modifications from the proposed new open channel south of Oakwood Road to the southwest to the primary environmental corridor boundary	40,000	0
	Subbasin No. 2 Improvements 1. 3,200 feet of 60-inch-diameter RCP from E. Elm Road at S. 4th Avenue to the south to E. County Line Road and then west to just west of the section line between Sections 35 and 36	480,000	0
	2. 3,500 feet of channel modifications from E. County Line Road just west of the section line between Sections 35 and 36 to the southwest into Caledonia, and then to the northwest across E. County Line Road up to the primary environmental corridor boundary in the southwest quarter of Section 35	30,000	0
	3. 1,300 feet of channel modifications from S. Elaine Road at East Schmitz Drive to the southwest to the primary environmental corridor boundary	20,000	0
	Engineering and Contingencies	80,000	0
	Subtotal	$ 850,000	$ 0
Meadowview	Subbasin No. 1 Improvements 1. 2,000 feet of channel modifications from the Milwaukee County Parkland boundary in the northwest quarter of Section 34 to the west to the confluence with Subbasin No. 2 and Subbasin No. 3 channels, respectively	$ 0[b]	$ 0

Table 59 (continued)

Hydrologic Unit	Project Component Description	Estimated Cost	
		Capital	Annual Operation and Maintenance[a]
Meadowview (continued)	Subbasin No. 2 Improvements 1. 900 feet of channel modifications from the confluence with the Subbasin No. 3 channel to the northwest to Nicholson Road	$ 0[b]	$ 0
	2. 1,700 feet of 48-inch-diameter RCP along the north side of Oakwood Road from McGraw Drive to Nicholson Road and then south along Nicholson Road to the existing channel	170,000	0
	3. 800 feet of 24-inch-diameter RCP from the proposed detention basin to the southeast to E. Oakwood Road and then along Oakwood Road to the existing channel just west of McGraw Drive	40,000	0
	4. 16.5 acre-foot detention facility north of Oakwood Road just east of Shepard Avenue	210,000	10,000
	5. 300 feet of 63-inch by 98-inch horizontal elliptical RCP from Shepard Avenue to the proposed detention facility	60,000	0
	6. 1,200 feet of channel modifications from Shepard Avenue just west of the proposed detention facility to the northwest to Oak Lane	10,000	0
	Subbasin No. 3 Improvements 1. 4,500 feet of channel modifications from the confluence with the Subbasin No. 2 channel to the west to just west of Shepard Avenue extended	50,000	0
	2. 3,700 feet of 48-inch-diameter RCP along Shepard Avenue extended from the Subbasin No. 3 channel to the north up to Oakwood Road and then west along Oakwood Road to Howell Avenue	370,000	0
	3. 1,200 feet of 36-inch-diameter RCP along Oakwood Road from Howell Avenue to the west	80,000	0
	Engineering and Contingencies	100,000	0
	Subtotal	$1,090,000	$10,000
Upper Crayfish	Subbasin No. 1 Improvements 1. 1,100 feet of new open channel along Fitzsimmons Road extended from Nicholson Road to the east	$ 20,000	$ 400
	2. 900 feet of 42-inch-diameter RCP along Fitzsimmons Road from McGraw Drive to Nicholson Road	80,000	0
	3. 1,500 feet of 36-inch-diameter RCP along Fitzsimmons Road from McGraw Drive to the west	110,000	0
	Subbasin No. 2 Improvements 1. 2,100 feet of 36-inch-diameter RCP along Oakwood Road from Pennsylvania Avenue to Nicholson Road	150,000	0
	2. 900 feet of channel modifications from Nicholson Road at Oak Lane to the east	0[b]	0
	3. 900 feet of 36-inch-diameter RCP along Oak Lane from Nicholson Road to McGraw Drive	60,000	0
	Engineering and Contingencies	40,000	0
	Subtotal	$ 460,000	$ 400

Table 59 (continued)

Hydrologic Unit	Project Component Description	Estimated Cost	
		Capital	Annual Operation and Maintenance[a]
Lower Crayfish	Subbasin No. 1 Improvements 1. None	$ --	$ --
	Subbasin No. 2 Improvements 1. 1,200 feet of 48-inch-diameter RCP along Elm Road in the northwest quarter of Section 35	120,000	0
	2. 1,700 feet of channel modifications along Elm Road from the proposed 48-inch-diameter RCP to the west	10,000	0
	Subbasin No. 3 Improvements 1. 1,000 feet of 72-inch-diameter RCP along E. Elm Road from the Crayfish Creek channel to the west to just west of the Chicago & North Western Railway tracks	200,000	0
	2. 1,000 feet of 54-inch-diameter RCP along E. Elm Road from the Chicago & North Western Railway tracks to the west to just east of Nicholson Road	130,000	0
	Subbasin No. 4 Improvements 1. 2,400 feet of 36-inch-diameter RCP along Nicholson Road from Elm Road to the Root River	170,000	0
	Engineering and Contingencies	130,000	0
	Subtotal	$ 760,000	$ 0
	Total	$3,640,000	$10,600

[a]*Costs were noted to be zero when the project proposed replacement of a component with a component that has a similar operation and maintenance cost.*

[b]*This section of channel has been modified and therefore was not included in the recommended plan cost estimate.*

Source: SEWRPC.

exceed those levels that would allow a free outlet from the Crayfish Creek subwatershed, consideration was given to the need to provide stormwater pumping facilities to facilitate adequate outlet conditions at the lower end of the watershed. To accomplish this, a pumping station could be installed just north of County Line Road with a capacity of about 500 cubic feet per second. This facility would be designed to accommodate a two-year recurrence interval storm event rather than a 10-year or 100-year recurrence interval event, since the two-year event over the Crayfish Creek subwatershed would be an extremely rare event in combination with a major flood event on the Root River itself.

It is recommended, however, that initially no additional stormwater pumping facility be provided in the Crayfish Creek subwatershed at County Line Road. Should experience indicate that the backup of floodwaters from the Root River creates problems more frequently than acceptable, the recommended plan could be modified to readily accommodate a small pumping station at the lower end of the subwatershed just north of County Line Road.

Caledonia Branch: Minor debrushing of the Caledonia Branch channel between its confluence with Crayfish Creek and the abandoned electric interurban railway crossing is recom-

Table 60

SELECTED CHARACTERISTICS AND COSTS OF THE RECOMMENDED FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN FOR THE CRAYFISH CREEK SUBWATERSHED

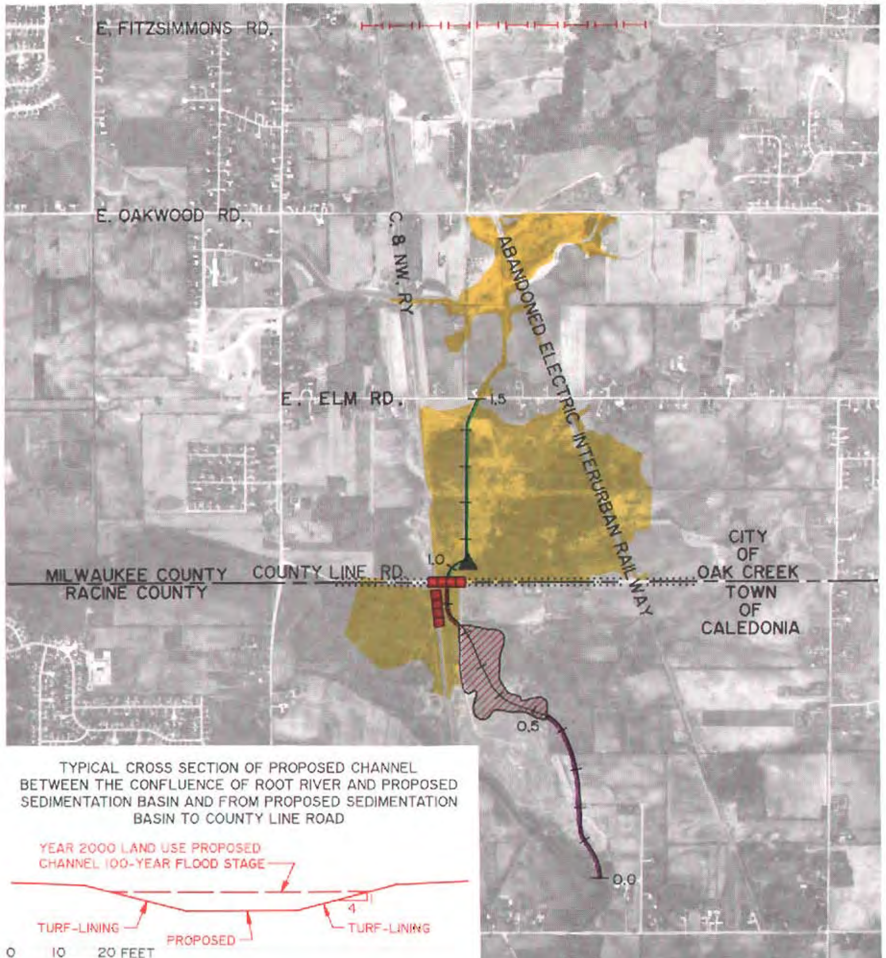
Stream	Project Component Description	Estimated Cost	
		Capital	Annual Operation and Maintenance[a]
Crayfish Creek	1. Replacement of the existing 36-inch corrugated metal pipe (CMP) culvert at Oakwood Road crossing of Crayfish Creek with a 480-inch CMP culvert	$ 5,000	$ 0
	2. Construction of a berm parallel to and 50 feet south of Fitzsimmons Road extended from 250 feet west of the Chicago & North Western Railway tracks 3,500 feet to the east, to S.15th Avenue extended	68,000	1,000
	3. Regrading of existing channel in the vicinity of Oakwood Road	11,000	0
	4. Reconstruction of 2,500 feet of E. County Line Road east of the Chicago & North Western Railway tracks	80,000	0
	5. Replacement of the existing 5-foot by 17-foot concrete box culvert at the County Line Road crossing of Crayfish Creek with four 72-inch CMP culverts	40,000	0
	6. Installation of four backwater gates on County Line Road culverts	75,000	8,000
	7. Bulkheading of the four 48-inch CMP culverts at Chicago & North Western Railway crossing of Crayfish Creek	3,000	0
	8. Construction of 700 lineal feet of new open channel from County Line Road to the existing retention pond located 700 feet south of E. County Line Road and adjacent to the Chicago & North Western Railway, and 2,110 feet of new channel from that retention pond to the Root River 850 feet north of Seven Mile Road	98,000	1,000
	9. Inlet and outlet refinements to the existing pond located downstream of County Line Road	3,000	0
	10. Minor debrushing and regrading of 2,800 feet of Crayfish Creek north of E. County Line Road	9,000	500
	Subtotal	$392,000	$10,500
Caledonia	1. Minor debrushing and regrading of 2,300 feet of Caledonia Branch upstream of its confluence with Crayfish Creek	$ 7,000	$ 400
	Total	$399,000	$10,900

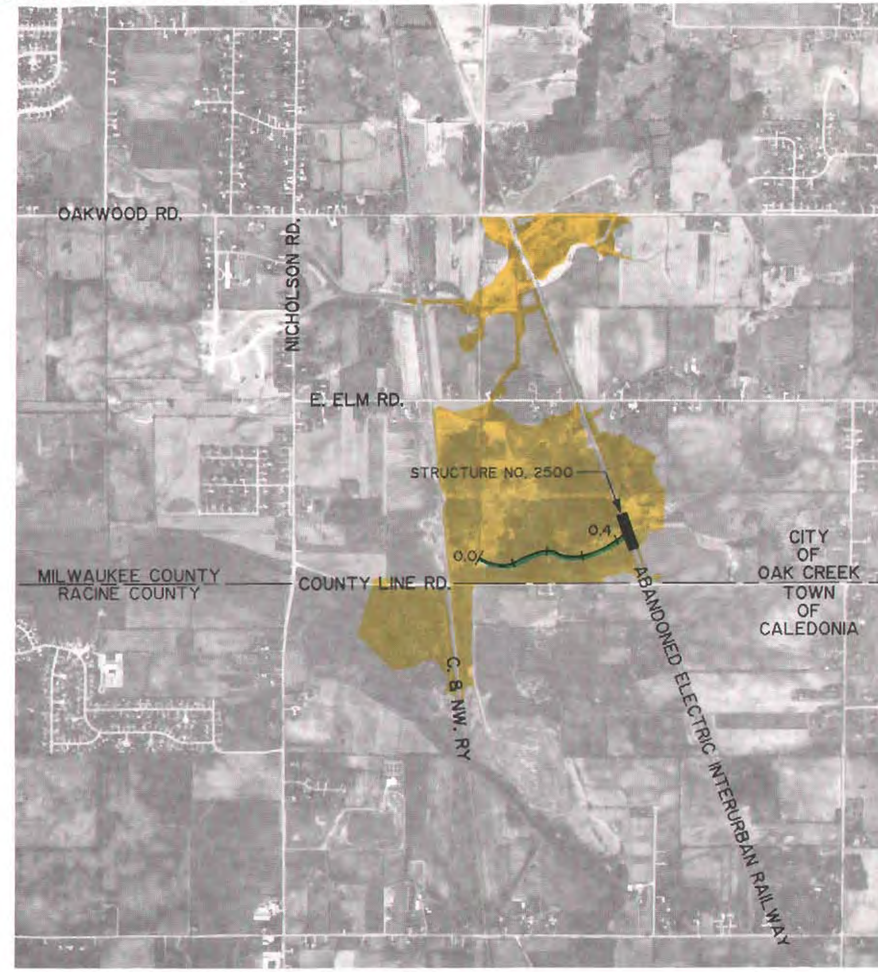
[a]*Costs were noted to be zero when the project proposed replacement of a component with a component that has similar operation and maintenance cost.*

Source: SEWRPC.

Map 103

RECOMMENDED FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN FOR CRAYFISH CREEK AND CALEDONIA BRANCH UNDER YEAR 2000 LAND USE AND PLANNED CHANNEL CONDITIONS





LEGEND

- 100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS
- 0.5 — APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING
- PROPOSED MINOR CHANNEL DEBRUSHING AND REGRADING
- PROPOSED NEW CHANNEL
- PROPOSED CULVERT MODIFICATION
- PROPOSED BERM
- PROPOSED SEDIMENTATION POND
- PROPOSED ELEVATED ROAD OR BERM
- POTENTIAL FUTURE STORMWATER PUMPING STATION TO BE ADDED SHOULD EXPERIENCE INDICATE THAT A GREATER PROTECTION LEVEL IS NEEDED

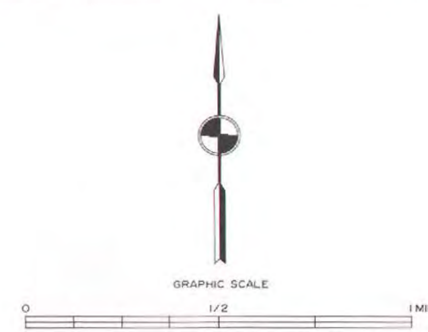
NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR CRAYFISH CREEK IS SHOWN IN APPENDIX H

DATE OF PHOTOGRAPHY: APRIL 1986

LEGEND

- 100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS
- 0.0 — APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING
- PROPOSED MINOR CHANNEL DEBRUSHING AND REGRADING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR CALEDONIA BRANCH IS SHOWN IN APPENDIX H

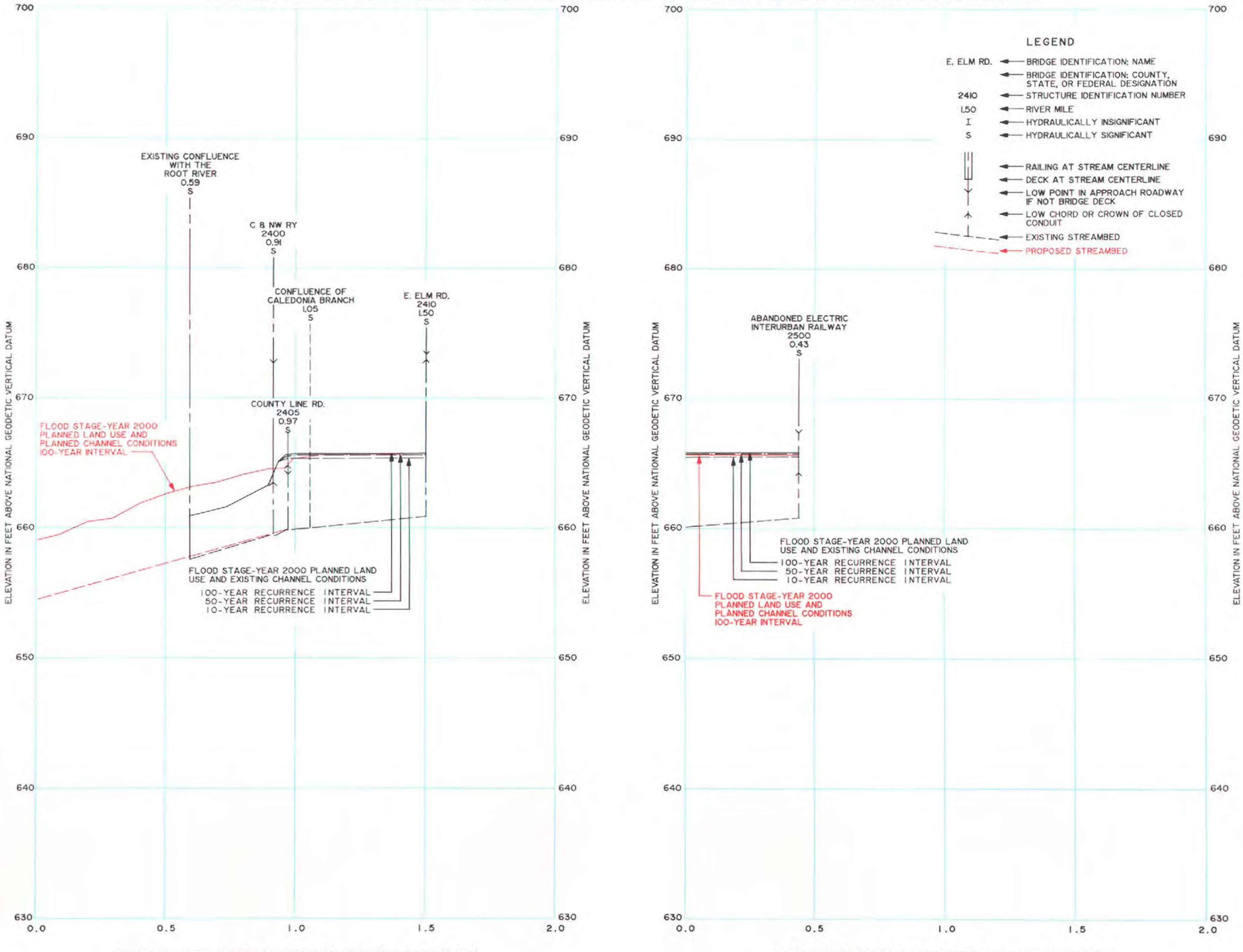


DATE OF PHOTOGRAPHY: APRIL 1986

Source: SEWRPC.

Figure 42

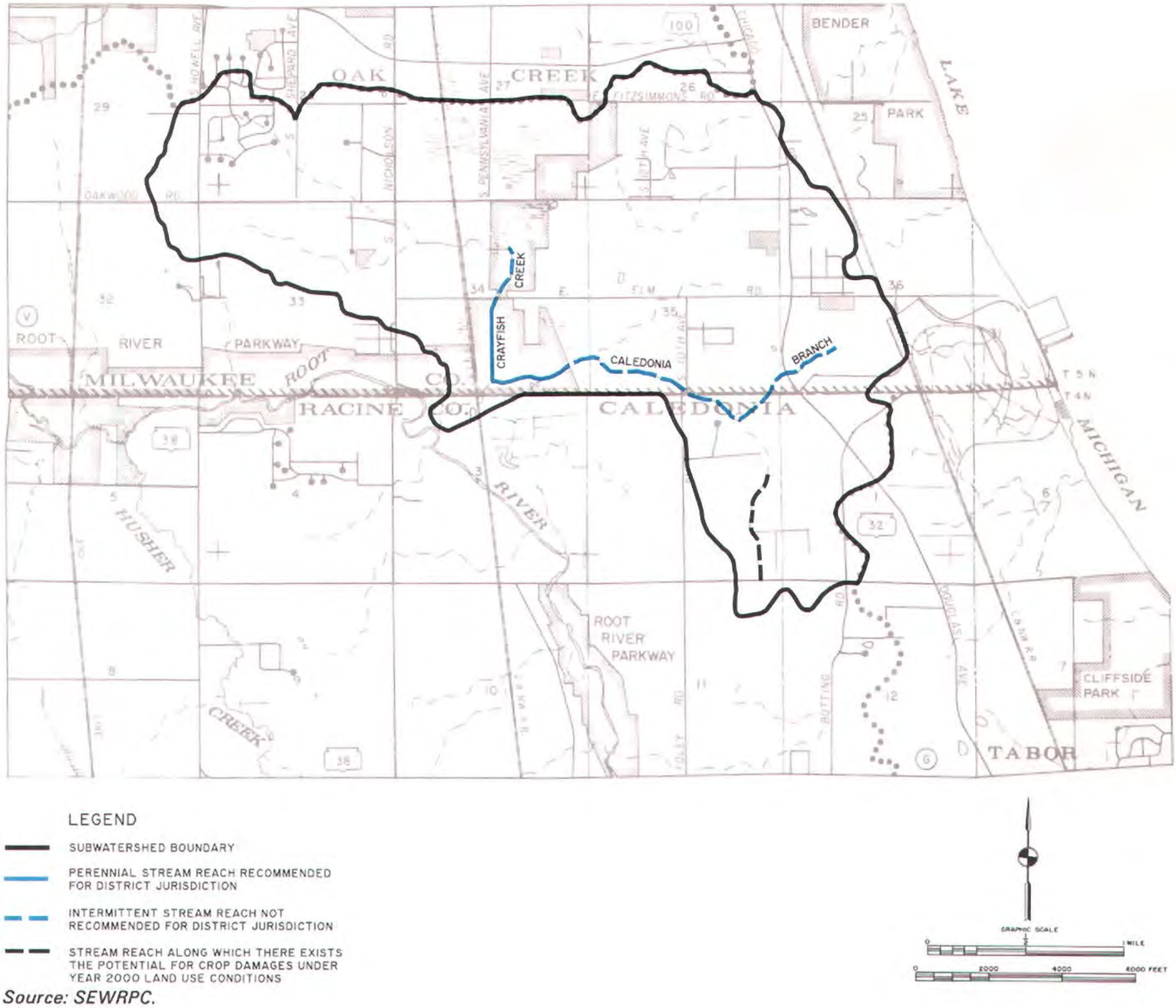
RECOMMENDED PLAN FLOOD STAGE PROFILES FOR CRAYFISH CREEK AND CALEDONIA BRANCH



Source: SEWRPC.

Map 104

POTENTIAL FUTURE CROP DAMAGES WITHIN THE CRAYFISH CREEK SUBWATERSHED IN THE TOWN OF CALEDONIA



Source: SEWRPC.

mended, along with minor regrading to eliminate negative channel bottom gradients. These channel maintenance measures are recommended to be carried out periodically as necessary.

Consideration of Potential Crop Damages in the Town of Caledonia: No potential should remain for monetary losses resulting from floods having a recurrence interval of up to and including 100 years in the Crayfish Creek subwatershed within the City of Oak Creek under year 2000 land use and channel conditions. The potential does exist, however, for minor crop damages along a tributary to Caledonia Branch in the Town of Caledonia, as shown on Map 104. No flood control measures are herein recommended to abate these crop damages. The recommended improvements for the Caledonia Branch and for Crayfish Creek presented herein would, however, provide an adequate outlet for any agricultural flood control measures that may be carried out along the tributary to Caledonia Branch.

Flood Control and Related
Drainage System Plan Implementation
The recommended flood control and related drainage system plan components for the two stream reaches recommended for District jurisdiction in the Crayfish Creek subwatershed consist of culvert modification, roadway reconstruction, new channel construction, and minor channel debrushing and regrading. It is recommended that the costs for all of the recommended plan components be borne by the Milwaukee Metropolitan Sewerage District.

104TH STREET BRANCH SUBWATERSHED FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN

As already noted, the 104th Street Branch was not studied under previous Commission planning programs. Analyses of the hydrologic and hydraulic characteristics of the tributary and its subwatershed were accordingly conducted under this system planning effort.

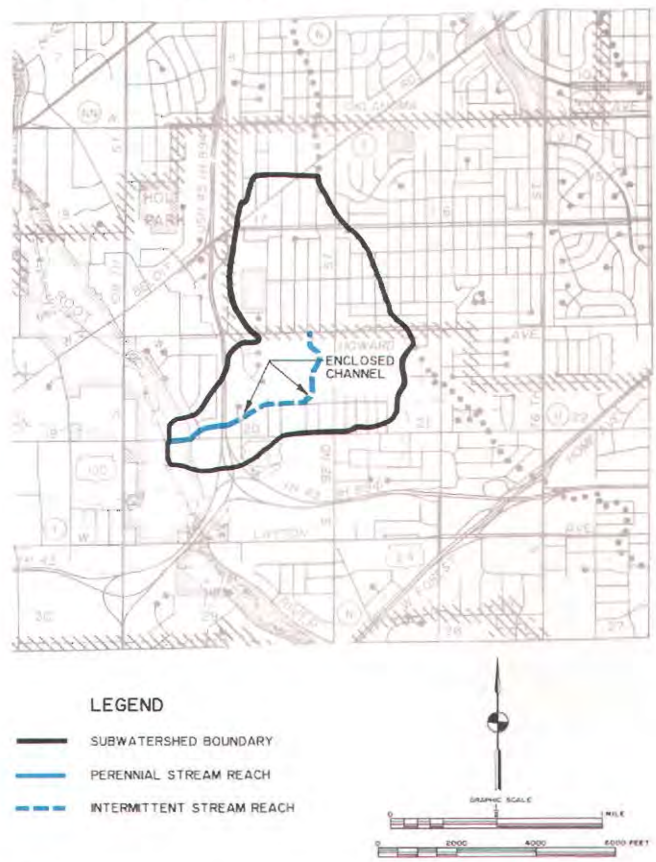
Overview of the Study Area
Of the 1.15-mile total length of the 104th Street Branch, 0.34 mile is classified as perennial stream and the remaining 0.83 mile is classified as intermittent stream. All of the 1.15-mile stream length is located in the City of Greenfield. The entire 0.34-mile perennial stream length and 0.02 mile of intermittent stream length is recommended for District jurisdiction in the policy plan companion to this system plan.

The 104th Street Branch drains an area of about 0.92 square mile, as shown on Map 105. Of this total drainage area, 0.48 square mile, or about 52 percent, lies within the City of Greenfield, and 0.44 square mile, or about 48 percent, lies within the City of Milwaukee.

From its origin at a storm sewer outfall located at the intersection of S. 94th Street and W. Howard Avenue, the 104th Street Branch flows 0.28 mile in a southerly direction to its entrance into a series of storm sewers. The stream then flows about 0.51 mile westerly through storm sewers to an outfall located about 200 feet west of the intersection of S. 99th Street and W. Plainfield Avenue. The 104th Street Branch then flows southwesterly 0.03 mile, where it passes through a 0.06-mile length of concrete box culvert beneath IH 894. From that culvert, the stream flows 0.09 mile westerly to S. 104th Street

Map 105

THE 104TH STREET BRANCH SUBWATERSHED



Source: SEWRPC.

and then 0.18 mile southwesterly to its confluence with the North Branch of the Root River.

Present land use in the 104th Street Branch subwatershed is predominantly residential, with the remaining areas in transportation and open space uses. The developed areas of the subwatershed are generally provided with a full range of municipal street improvements, including paved streets with curbs and gutters and attendant storm sewers. Accordingly, surface runoff is generally conveyed rapidly from each individual site to the stream over impervious surfaces and through storm sewers.

Data relating to certain pertinent characteristics of the subwatershed, such as hydrologic soil types, land slopes, and land use, appear in Chapter II of this report. The subwatershed land use conditions may be expected to undergo little change by the design year of the system plan.

As development of this subwatershed has occurred, the stream channel has been considerably altered through realignment, widening and deepening, and enclosure. Channel modifications have been made along the entire 0.36-mile reach recommended for District jurisdiction.

Flooding and Related Drainage Problems

Investigations conducted under this system planning effort revealed no serious flooding problems along the 104th Street Branch. The City Engineer for the City of Greenfield reported that there is some concern over stream bank erosion along the reach between S. 104th Street and IH 894. Also, about half of the stream is located within Milwaukee County Parkway land, where there is no flood-damage-prone development along the stream channel.

The results of the hydrologic and hydraulic analyses indicate that no structure flood damages may be expected to occur along the 104th Street Branch for floods up to and including the 100-year recurrence interval event under planned, year 2000 land use and existing channel conditions.

Flood Discharges and Stages

As noted in Chapter III of this report, the hydrologic model used for development of design discharges for the Root River watershed simulates streamflow on a continuous basis, using recorded climatological data as input. Flood discharges were computed at an hourly time interval. Because of the relatively small tributary drainage area of the 104th Street Branch subwatershed, it was suspected that the time of peak discharge on the stream was very short and may have been missed in the analyses utilizing an hourly time interval. Simulations were performed, therefore, using a 15-minute time interval with design rainfall events as input. The use of design rainfall events was necessary because the time and cost of simulating continuous streamflows at 15-minute intervals for the 39 years of available climatological data would be prohibitive.

The design rainfall events were developed using 10-, 50-, and 100-year rainfall volumes obtained from the updated point rainfall depth-duration-frequency relationships developed by the Commission as described in Chapter III. The rainfall distribution utilized for each design storm was the median distribution of a first-quartile storm, as shown in Chapter III. The design storm duration was determined for a given recurrence interval by simulating the peak discharge at a given location for a range of storm durations. The storm duration and associated rainfall volume which produced the largest peak discharge at a given location for a given recurrence interval was selected as the design storm for that location. The estimated peak flood discharges under existing and planned, year 2000 land use and existing (1987) channel conditions for the 104th Street Branch are as follows:

			Peak Flood Discharge (cfs)	
River Mile	Location	Recurrence Interval (years)	Existing Land Use, Existing Channel Conditions	Year 2000 Planned Land Use, Existing Channel Conditions
0.00	Mouth at Root River	10	310	310
		50	560	560
		100	620	620

Flood stage profiles were determined for the 10-, 50-, and 100-year recurrence interval runoff events under planned land use and existing channel conditions. These profiles, which encompass the full 0.36-mile-long reach recommended for District jurisdiction, constitute a graphic representation of the flood stages along the 104th Street Branch under the specified recurrence interval flood discharges, and under planned land use and existing channel conditions. In addition to providing an overall representation of flood stages relative to familiar points of reference such as the channel bottom and bridge deck surfaces, the profiles, because of their continuity, permit the determination of flood stages at any point along the stream channel. The flood profile is shown in Figure 43. The extent of the 100-year recurrence interval floodplain under planned land use conditions is shown on Map 106. This delineation of the flood hazard area was accomplished using large-scale topographic maps prepared by the City of Greenfield to Commission standards in 1974 and 1976. It should be noted that for the reach upstream of IH 894, significant changes have been made to the channel and floodplain since the preparation of the large-scale topographic map.

Recommended Flood Control
System for the 104th Street Branch

Because no flooding of structures would be expected to result from a 100-year recurrence interval flood under year 2000 land use conditions, it was not necessary to consider further

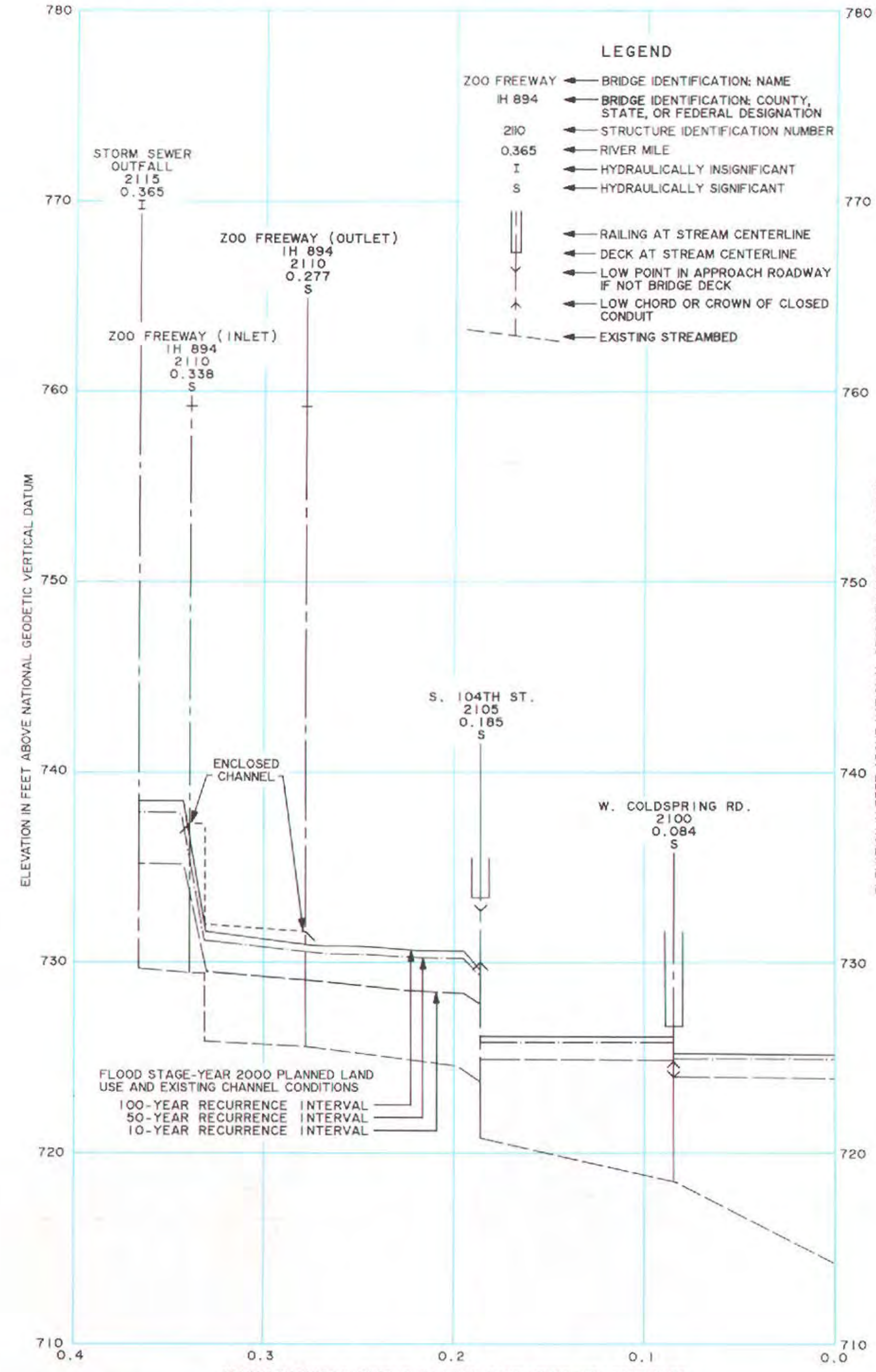
Map 106

100-YEAR RECURRENCE INTERVAL FLOODPLAIN FOR THE 104TH STREET BRANCH UNDER YEAR 2000 LAND USE AND EXISTING CHANNEL CONDITIONS

Source: SEWRPC.

Figure 43

FLOOD STAGE AND STREAMBED PROFILE FOR THE 104TH STREET BRANCH



Source: SEWRPC.

flood control and drainage alternatives for the 104th Street Branch. Therefore, no flood control or drainage system measures are recommended. Since there has been little change in the riverine areas of the 104th Street Branch since the preparation by the City of large-scale topographic maps in 1974 and 1976, and since those maps were prepared to Commission standards, no new topographic mapping is required as a basis for the delineation of the flood hazard area along this watercourse.

IMPACT OF RECOMMENDED FLOOD CONTROL SYSTEM PLAN ON FLOOD FLOWS AND STAGES

Structural flood control measures recommended for streams in the Root River watershed under this system plan will improve the hydraulic efficiency of the channel system, but may be expected to increase, to some degree, downstream flood flows and stages. These measures include channel modification, channel enclosure, bridge and culvert replacement, and channel cleaning and debrushing. Hydrologic and hydraulic analyses were conducted as part of this system plan to determine the impact of the recommended flood control measures on downstream flood flows and stages. These analyses considered the impact along the entire Root River channel to its mouth at Lake Michigan in the City of Racine. A comparison of the 100-year recurrence interval flood flows and stages under planned land use and existing and planned channel conditions is provided in Table 61. Flood stage profiles for the Root River main stem under planned land use and existing and planned channel conditions in the watershed are shown in Figure 44. The extent of the 100-year recurrence interval floodplain along the Root River main stem under planned year 2000 land use and planned channel conditions is shown on Map 107.

As noted earlier in this chapter, implementation of the flood control measures for the North Branch of the Root River and Hale Creek may be expected to increase flood flows and stages along the North Branch of the Root River upstream of the confluence with the East Branch of the Root River. No increases are anticipated downstream of this location. Increases in the 100-year recurrence interval flood stage of 0.1 to 0.3 foot may be expected to occur along the North Branch of the Root River between W. Forest Home Avenue and S. 116th Street. Most of these increases would occur along stream reaches located within Milwaukee County park and parkway lands, although some private properties would also be affected. Proper legal arrangements would have to be made with all affected property owners. No additional existing structures are expected to incur flood damages as a result of the increased flood stages. Stage increases downstream of W. Forest Home Avenue would be less than 0.1 foot.

Flood control and stormwater drainage measures along Whitnall Park Creek and the north and northwest tributaries to Whitnall Park Creek are not expected to increase flood flows and stages downstream of the confluence of Whitnall Park Creek and Tess Corners Creek. Increases in the 100-year recurrence interval flood stage of 0.1 to 0.8 foot are anticipated along Whitnall Park Creek downstream of the private drive located at River Mile 1.78. Again, most of these increases would occur along stream reaches located within county park and parkway lands. Some private properties located between S. 108th Street and River Mile 1.78 would also be affected, although no additional existing structures are expected to incur flood damages. Proper legal arrangements would have to be made with all affected property owners.

The flood control measures recommended along the East Branch of the Root River would not affect flood flows and stages. No flood control measures are recommended along Tess Corners Creek and 104th Street Branch.

During the development of the Commission's stormwater management plan for the Crayfish Creek subwatershed, concerns were raised by officials of the Town of Caledonia that, by eliminating the ability of floodwaters from the Root River to back up into the Crayfish Creek drainage basin, significant floodwater storage would be lost, thus increasing downstream flows. Hydrologic model simulations were performed under this system planning effort to test the impact of the Crayfish Creek plan on Root River flood flows. These simulations took into account not only the loss of storage for Root River floodwaters, but also the elimination of inflow to the Root River from Crayfish Creek during the time that backwater gates along W. County Line Road would be closed. During the time that these

Table 61

IMPACT OF RECOMMENDED FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN ON ROOT RIVER WATERSHED FLOOD DISCHARGES

Stream	Location	River Mile	100-Year Recurrence Interval Flood Discharge Planned Land Use (cfs)		Percent Change	100-Year Recurrence Interval Flood Stage Planned Land Use[a] (feet NGVD)		Change in Flood Stage (feet)
			Existing Channel Conditions	Recommended Plan Conditions		Existing Channel Conditions	Recommended Plan Conditions	
Root River	At mouth	0.00	4,900	4,850	-1	584.6[b]	584.6[b]	0.0
	Upstream of Spring Street	3.92	4,800	4,700	-2	593.2	593.2	0.0
	STH 38	5.97	4,800	4,700	-2	624.8	624.7	-0.1
	Upstream of confluence with Hoods Creek	11.46	4,400	4,310	-2	646.8	646.7	-0.1
	Seven Mile Road	15.91	4,200	4,120	-2	663.2	663.1	-0.1
	Downstream of Chicago, Milwaukee, St. Paul & Pacific Railroad	19.47	4,700	4,700	0	670.6	670.6	0.0
	Downstream of S. 43rd Street	23.14	4,900	4,900	0	678.5	678.5	0.0
North Branch of the Root River	Confluence with Root River	25.66	4,900	4,900	0	681.6	681.6	0.0
	W. Ryan Road	27.92	4,650	4,650	0	684.7	684.7	0.0
	Upstream of confluence with the East Branch of the Root River	30.15	5,100	5,150	1	694.1	694.1	0.0
	W. Rawson Avenue	32.37	5,450	5,550	2	703.2	703.3	0.1
	Upstream of confluence with Tess Corners Creek	35.58	3,350	3,430	2	709.5	709.6	0.1
	W. Forest Home Avenue	37.67	4,280	4,440	4	720.8	720.9	0.1
	Downstream of W. Cold Spring Road	39.16	3,500	3,880	11	725.2	725.4	0.2
	Downstream of W. Beloit Road	39.76	3,500	3,880	11	728.3	728.6	0.3
	W. National Avenue	40.97	2,000	2,840	42	734.0	733.8	-0.2
	Upstream of confluence with Hale Creek	41.25	1,410	1,510	7	734.9	734.9	0.0
Hale Creek	At mouth	0.00	1,520	1,560	3	734.9	734.9	0.0
	W. Cleveland Avenue	0.30	580	590	2	737.0	736.4	-0.6

Table 61 (continued)

Stream	Location	River Mile	100-Year Recurrence Interval Flood Discharge Planned Land Use (cfs)		Percent Change	100-Year Recurrence Interval Flood Stage Planned Land Use[a] (feet NGVD)		Change in Flood Stage (feet)
			Existing Channel Conditions	Recommended Plan Conditions		Existing Channel Conditions	Recommended Plan Conditions	
East Branch of the Root River	At mouth	0.00	1,490	1,490	0	693.8[c]	693.8[c]	0.0
	N. 51st Street	1.48	1,490	1,490	0	717.8	717.8	0.0
	- -	2.21	1,390	1,390	0	722.1	722.1	0.0
	- -	3.14	1,010	1,010	0	736.0	736.0	0.0
	W. Rawson Avenue	3.66	960	960	0	745.9	745.9	0.0
	- -	4.70	510	510	0	754.4	754.4	0.0
Tess Corners Creek	At mouth	0.00	3,730	3,730	0	708.8[c]	708.9[c]	0.1
	Upstream of confluence with Whitnall Park Creek	0.42	2,030	2,030	0	710.1	710.1	0.0
	Private drive	2.33	2,240	2,240	0	748,0	748.0	0.0
Whitnall Park	At mouth	0.00	2,190	2,190	0	710.1	710.1	0.0
	Whitnall Park Drive	0.97	1,800	1,870	4	735.0	735.1	0.1
	S. 108th Street	1.62	1,373	1,554	13	764.0	764.6	0.6
	W. Forest Home Avenue	1.70	1,207	1,398	16	765.3	766.1	0.8
Northwest Branch of Whitnall Park Creek	At mouth	0.00	398	545	37	768.9	768.9	-0.3
	W. Godsell Road	0.25	311	454	46	776.9	776.9	0.0
	W. Parnell Avenue	0.39	311	340	9	782.9	782.9	0.0
North Branch of Whitnall Park Creek	At mouth	0.00	222	247	11	783.3	783.1	-0.2
	S. 112th Street	0.36	216	230	6	790.0	788.9	-0.1
	W. Upham Avenue	0.53	182	191	5	793.8	793.9	0.1
	W. Abbott Avenue	0.58	163	163	0	796.4	796.1	-0.3
Crayfish Creek	W. County Line Road	0.38	815	815	0	665.6	665.6	0.0
Caledonia Branch	At mouth	0.00	400	400	0	665.6	665.6	0.0
104th Street Branch	S. 104th Street	0.18	620	620	0	730.6	730.6	0.0

[a]*Flood stages at road crossings are for the upsteam side of the bridge or culvert.*

[b]*Lake Michigan 100-year recurrence interval water level.*

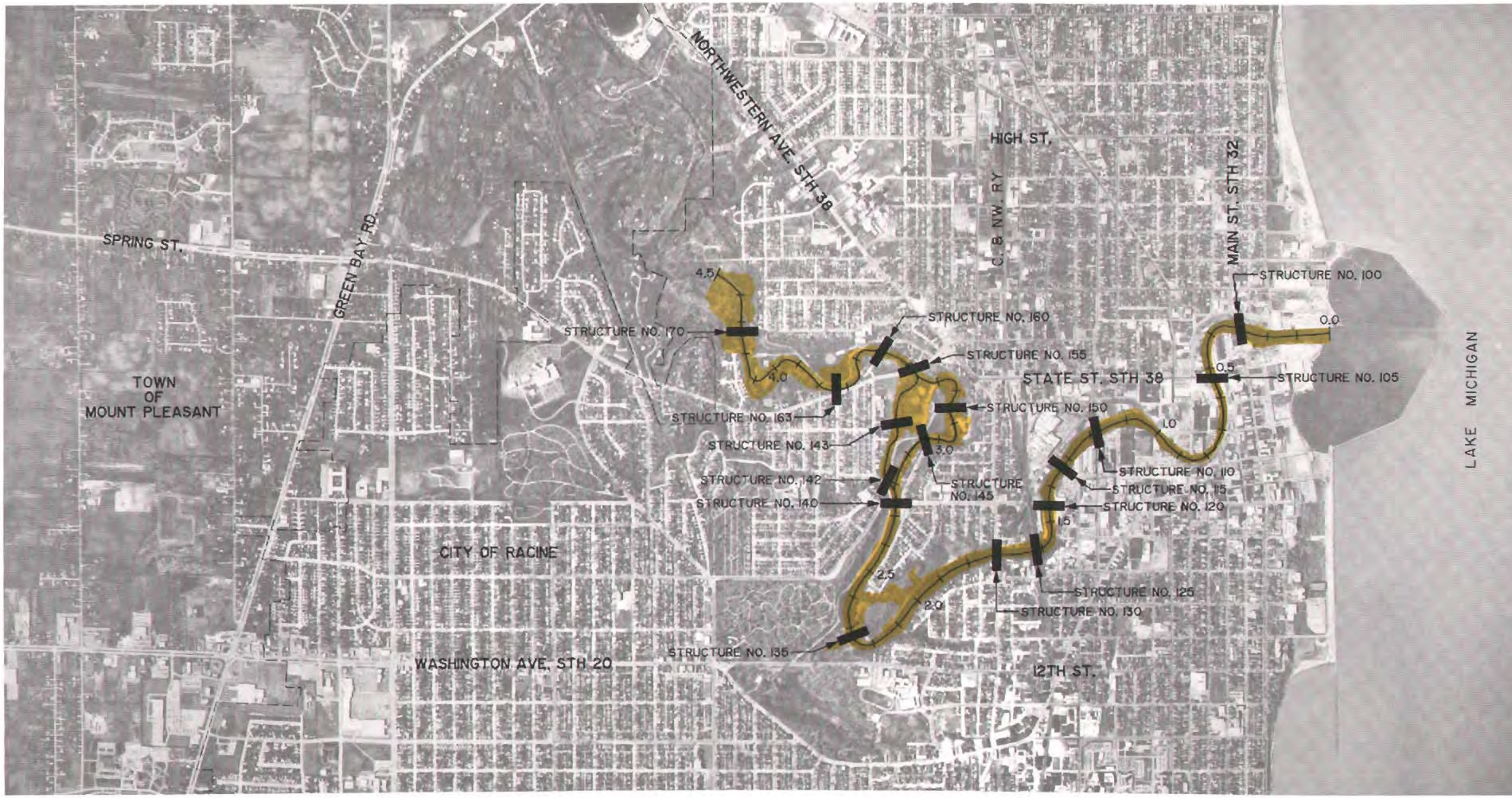
[c]*North Branch of Root River flood stage.*

Source: SEWRPC.

gates are closed, runoff from the Crayfish Creek basin would be stored north of W. County Line Road and would be released only when flood stages on the Root River receded. The results of the hydrologic simulations are shown in Table 61, and indicate that the loss of contributing runoff from the Crayfish Creek basin more than offsets the loss of Root River floodwater storage, thus actually resulting in a small decrease in peak downstream flood flows. This decrease in flood flows would result in downstream stage decreases of only 0.1 foot or less for a 100-year recurrence interval flood event under planned land use conditions. Thus, implementation of these flood control measures would have a negligible impact on downstream flood damages.

Map 107

100-YEAR RECURRENCE INTERVAL FLOODPLAIN FOR THE ROOT RIVER MAIN STEM UNDER YEAR 2000 LAND USE AND PLANNED CHANNEL CONDITIONS



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS

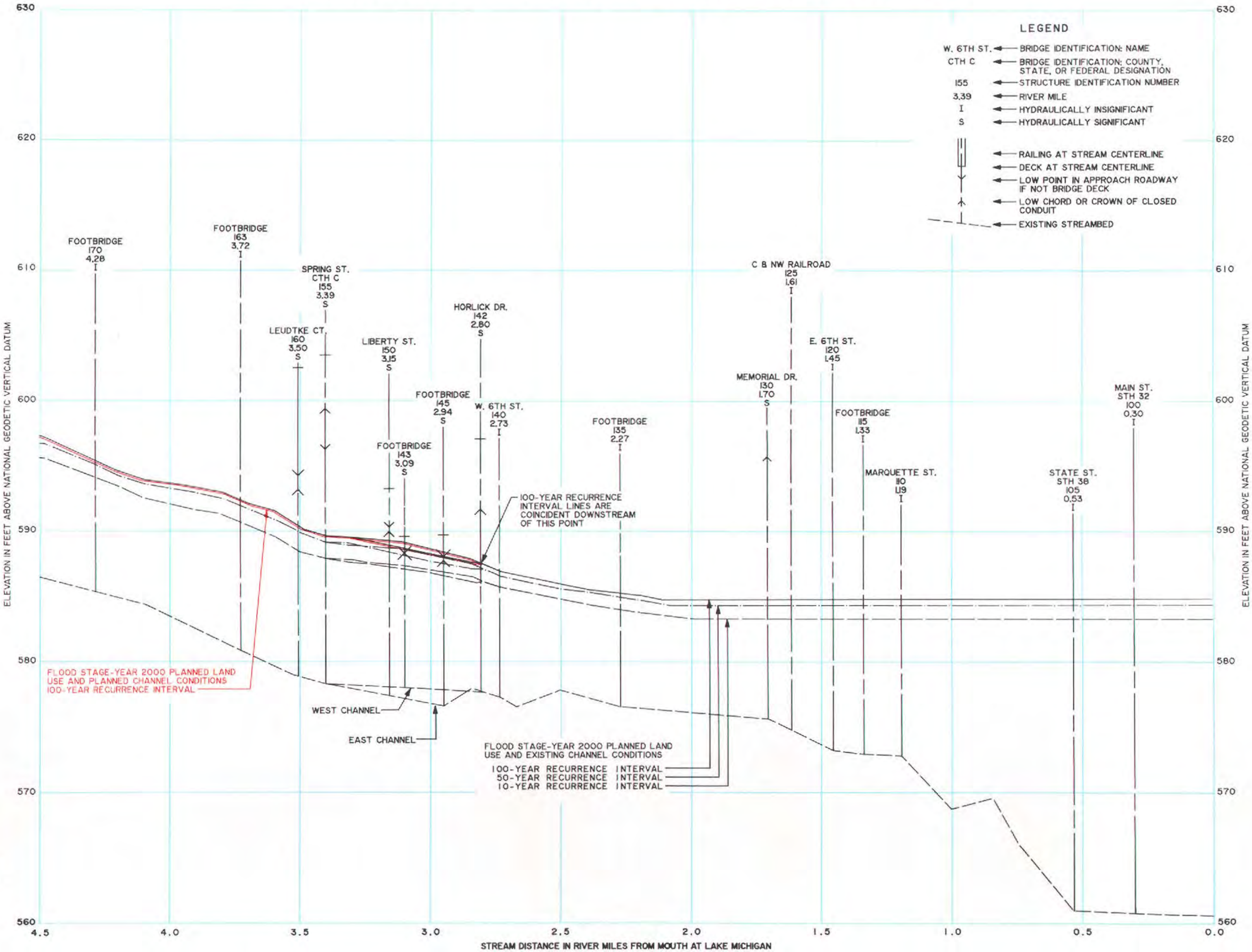
2.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR ROOT RIVER IS SHOWN IN APPENDIX H

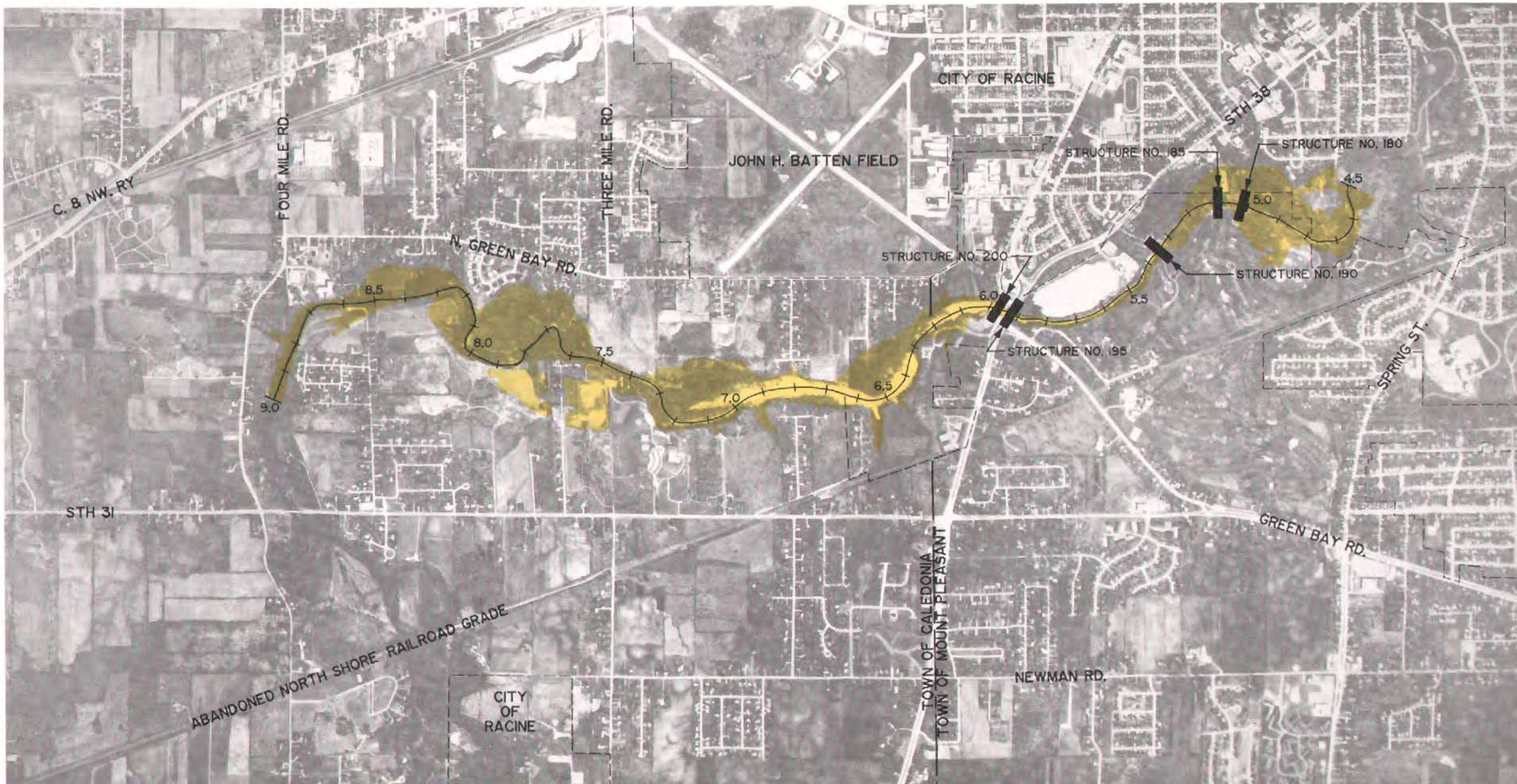
DATE OF PHOTOGRAPHY: APRIL 1986

Figure 44

FLOOD STAGE PROFILES FOR THE ROOT RIVER UNDER YEAR 2000 PLANNED LAND USE AND EXISTING AND PLANNED CHANNEL CONDITIONS



Map 107 (continued)

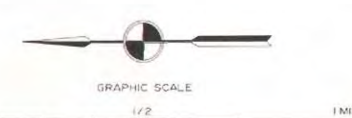


LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS

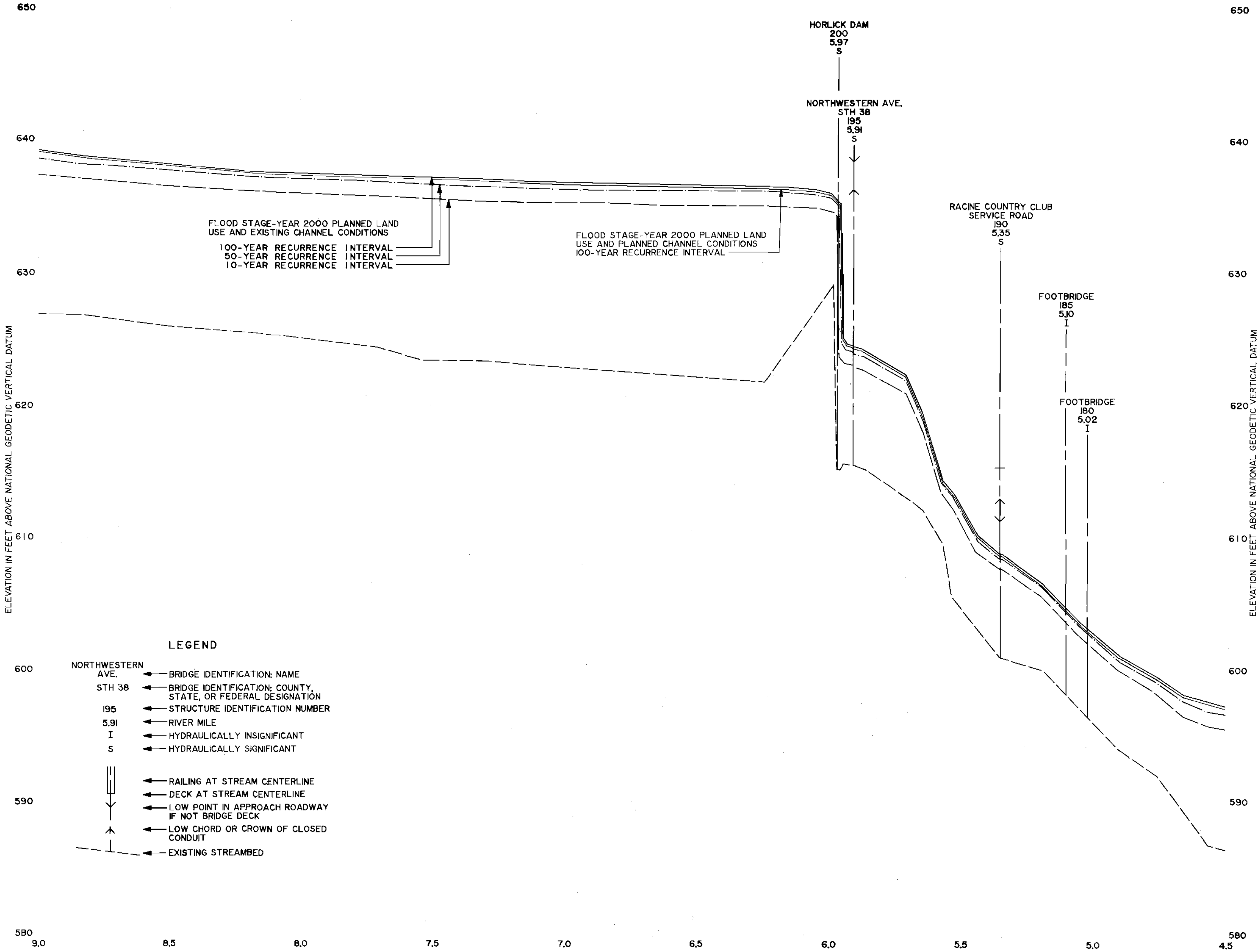
6.5 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR ROOT RIVER IS SHOWN IN APPENDIX H

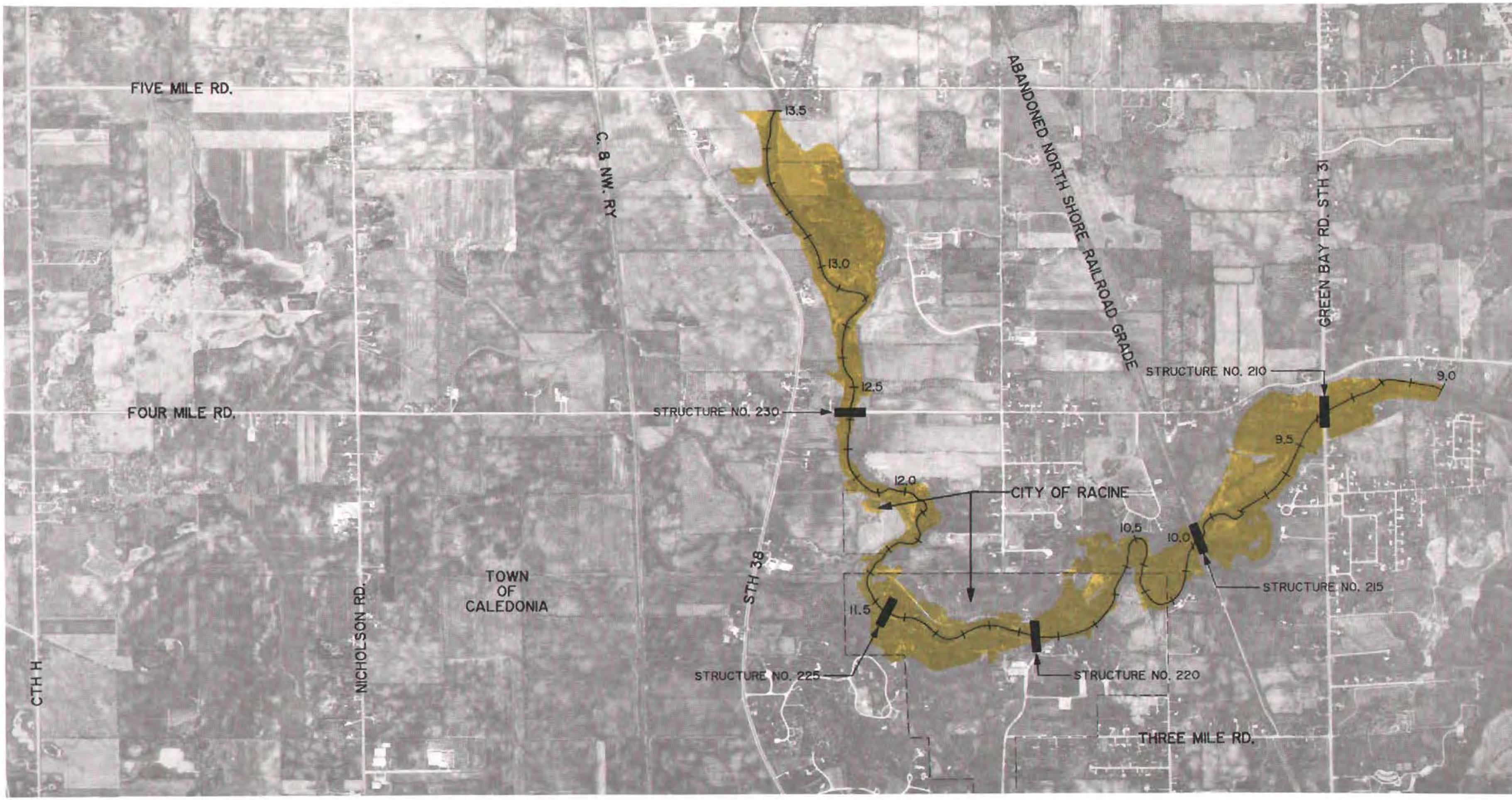


DATE OF PHOTOGRAPHY APRIL 1986

Figure 44 (continued)



Map 107 (continued)



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS

11.5 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

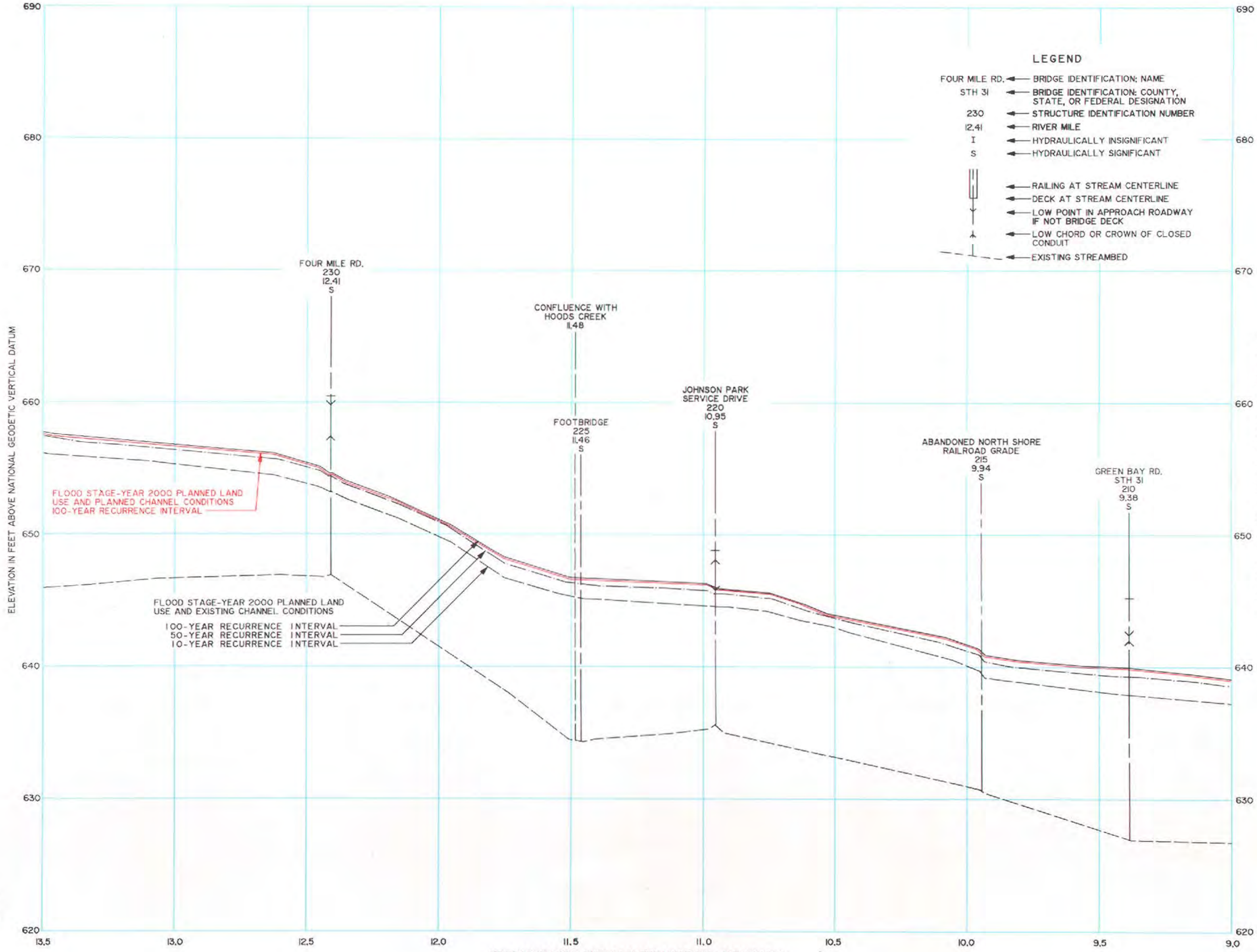
NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR ROOT RIVER IS SHOWN IN APPENDIX H

GRAPHIC SCALE

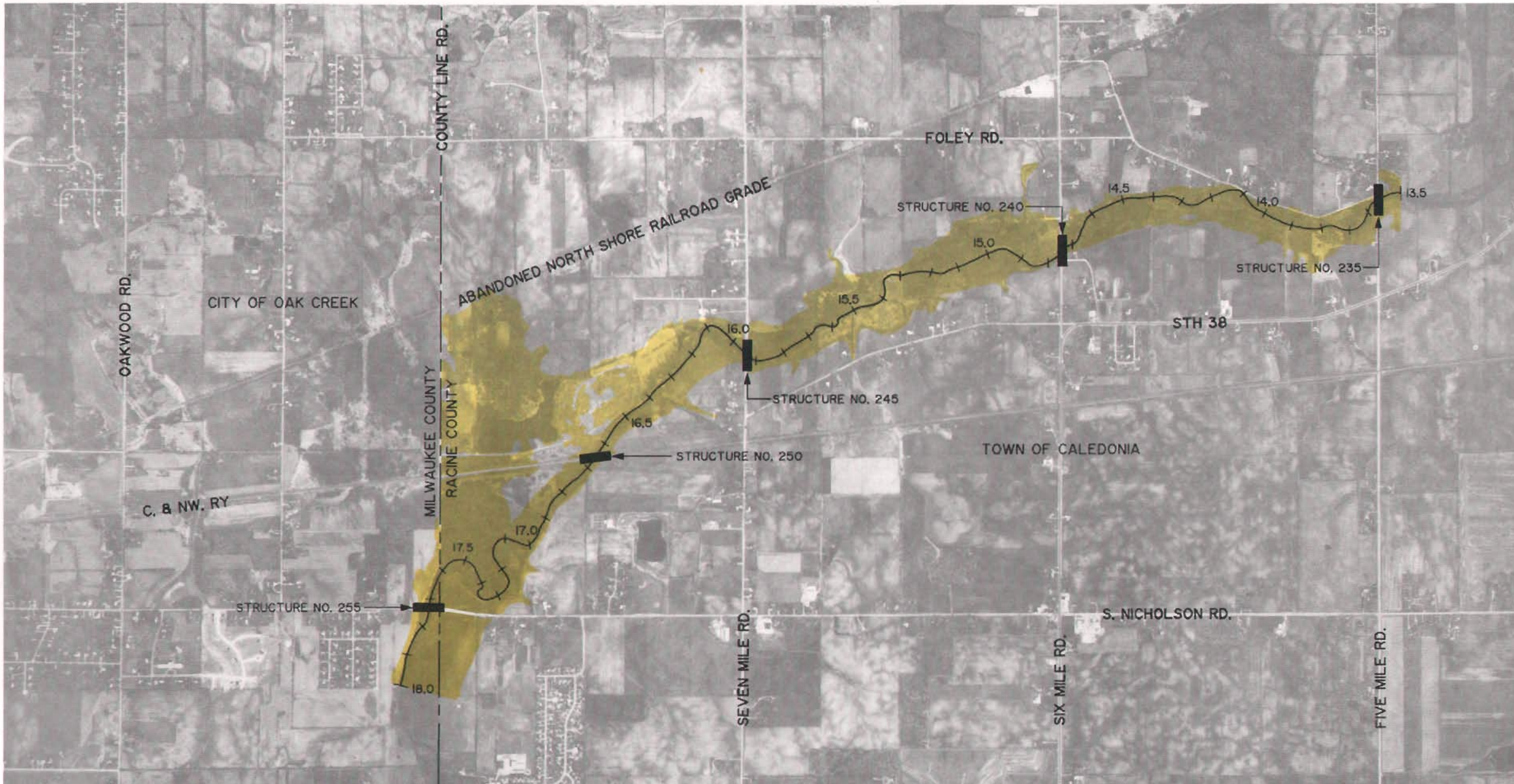
0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Figure 44 (continued)



Map 107 (continued)



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS

15.5 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

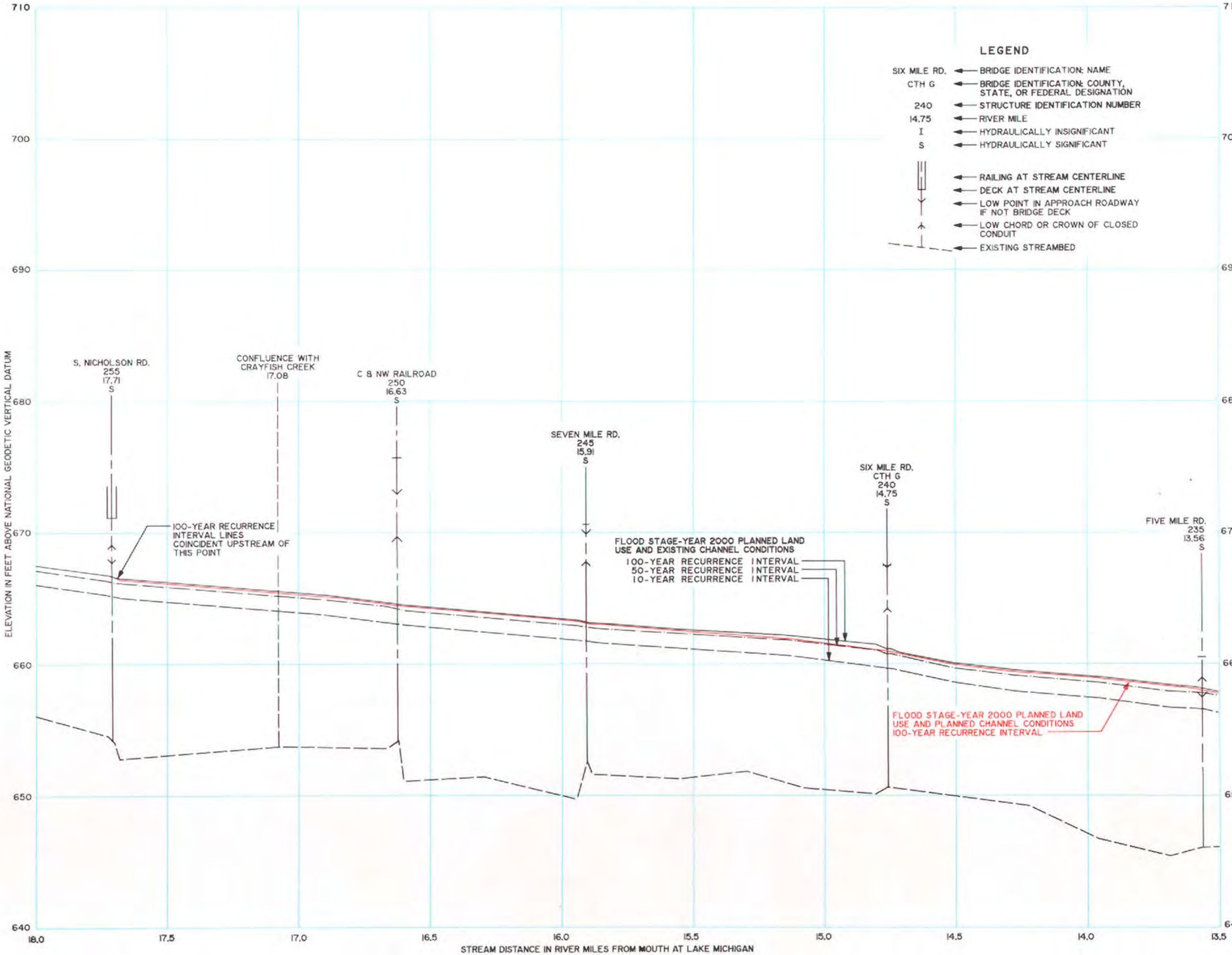
NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR ROOT RIVER IS SHOWN IN APPENDIX H

GRAPHIC SCALE

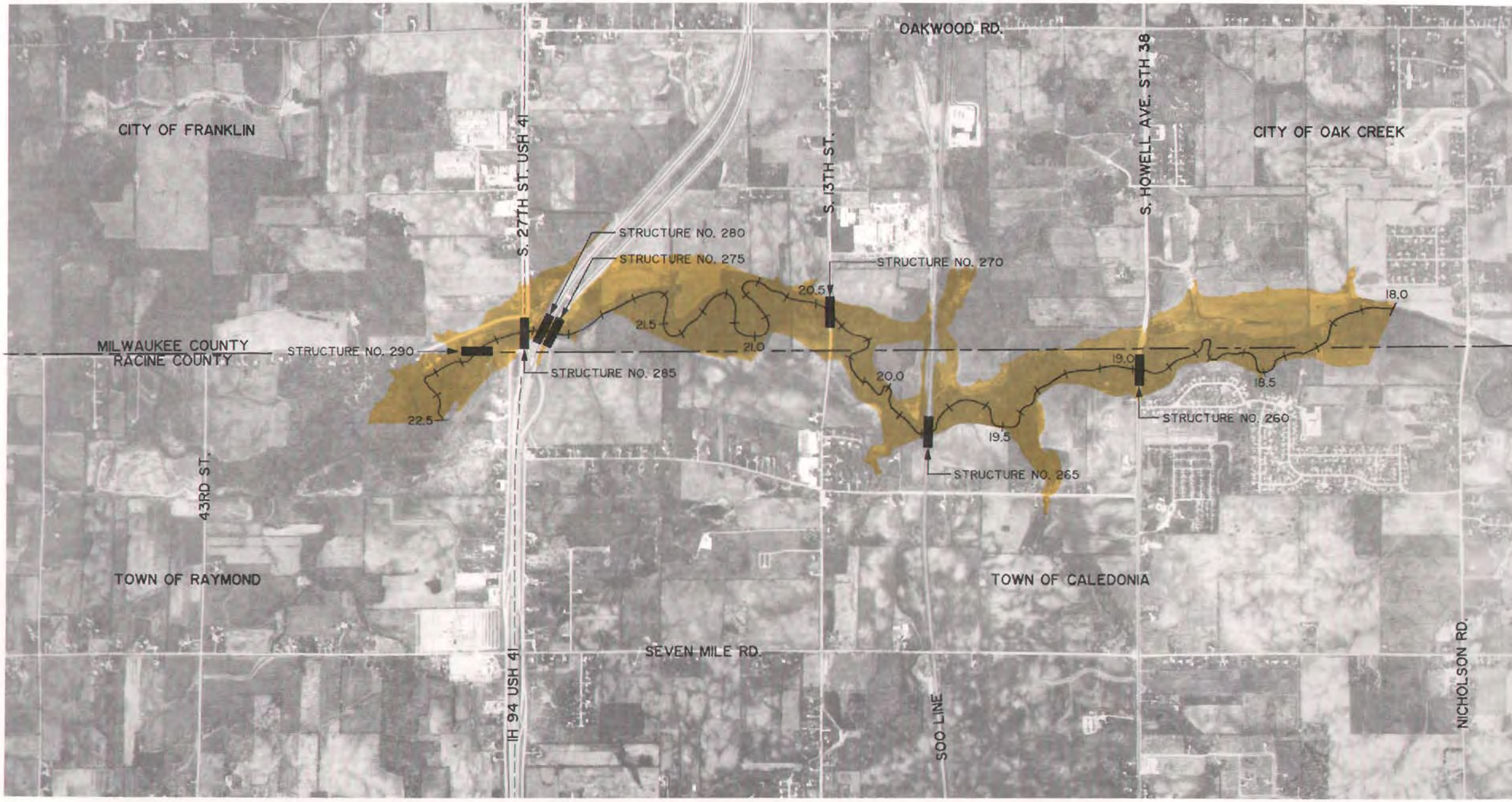
0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Figure 44 (continued)



Map 107 (continued)



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS

20.5 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

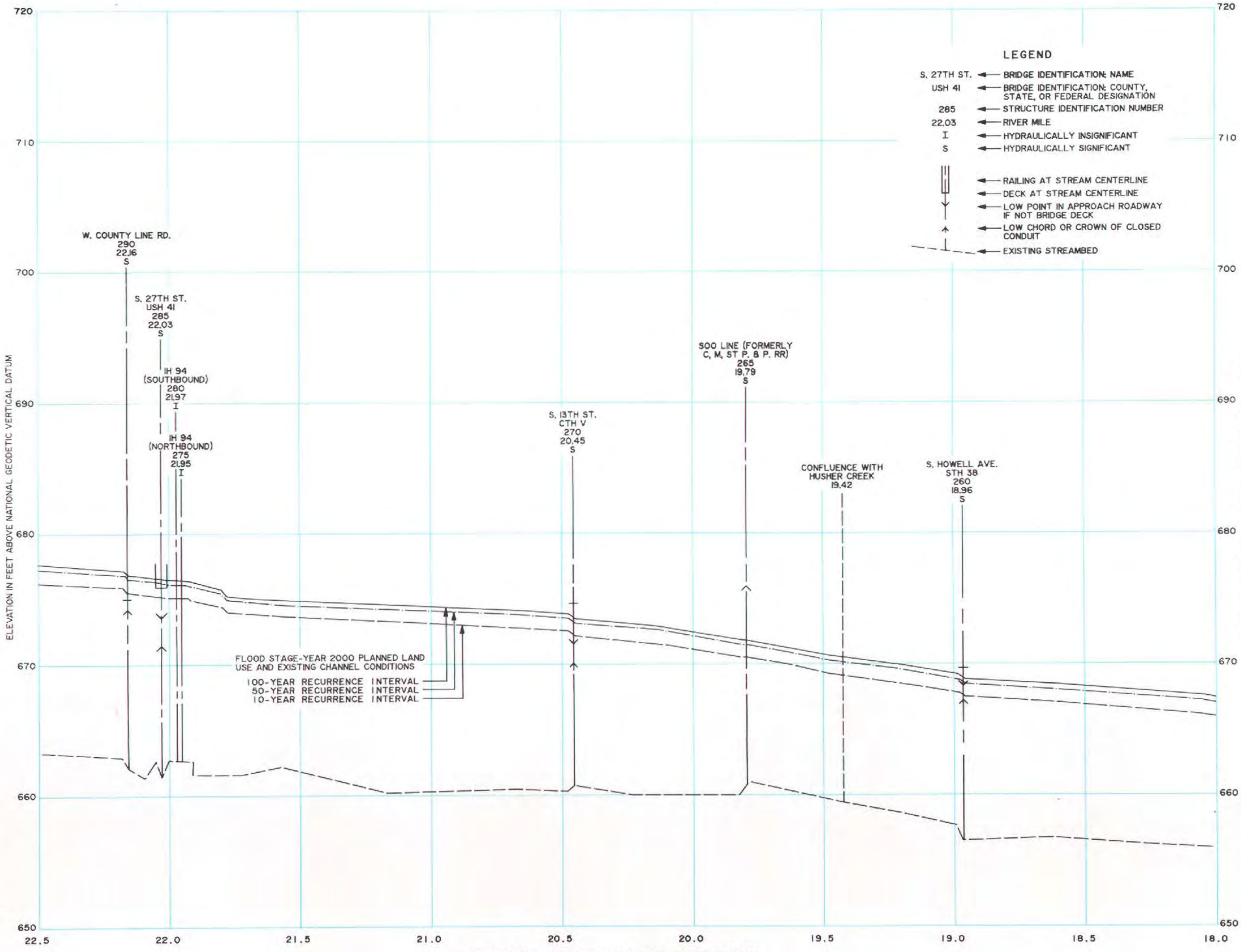
NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR ROOT RIVER IS SHOWN IN APPENDIX H

GRAPHIC SCALE

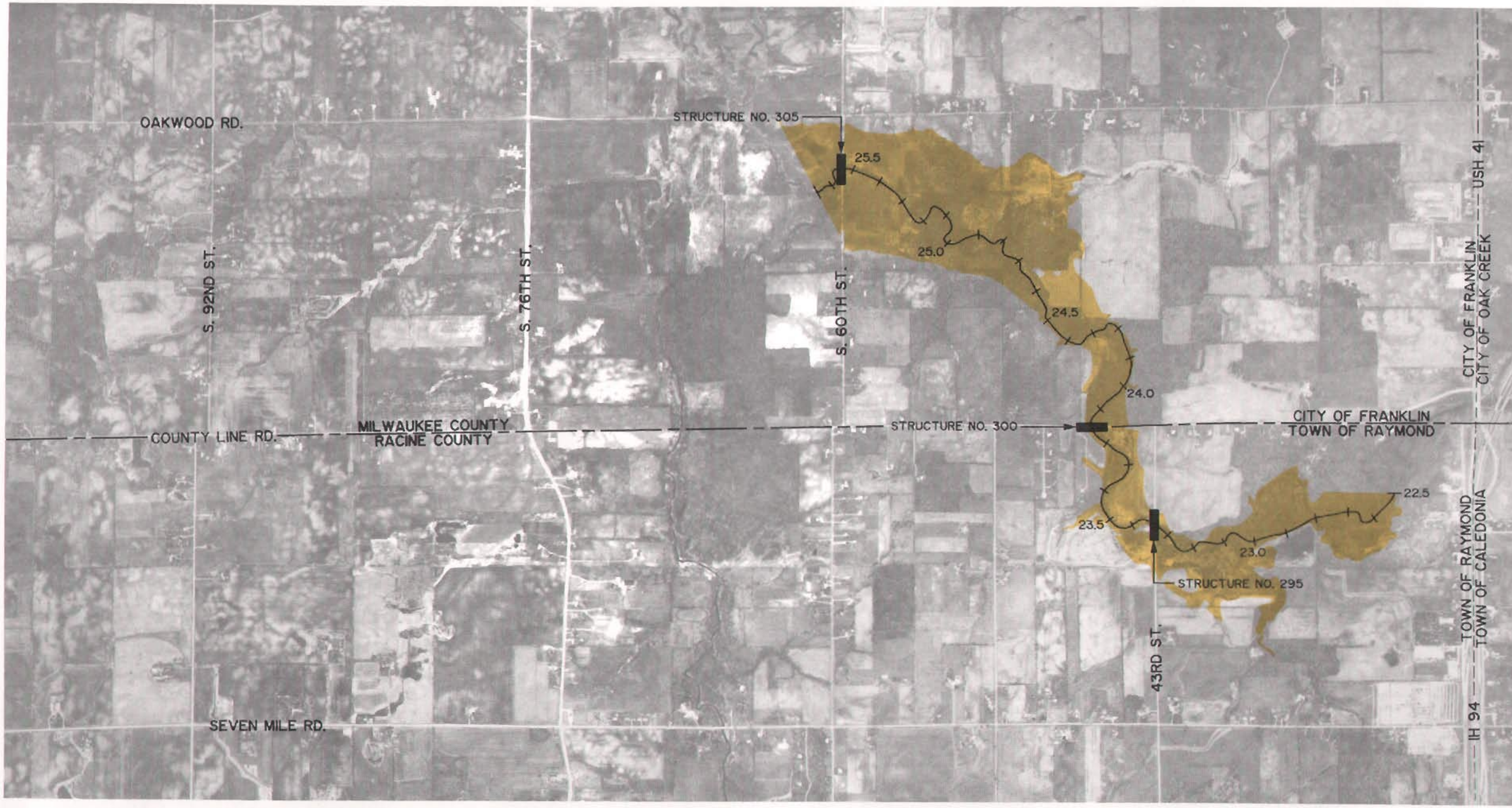
0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Figure 44 (continued)



Map 107 (continued)

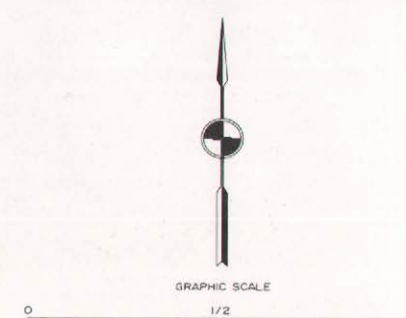


LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS

24.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

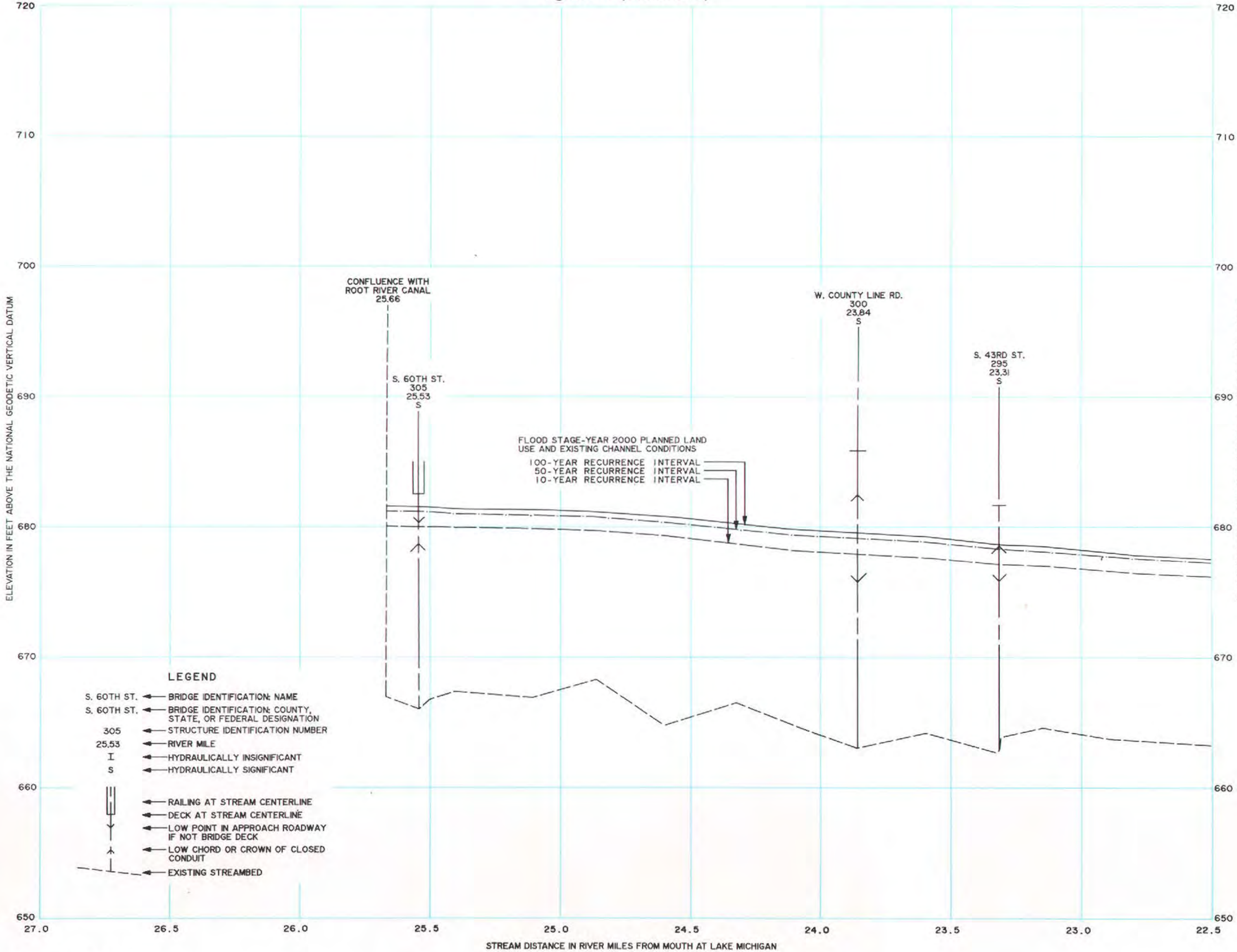
NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR ROOT RIVER IS SHOWN IN APPENDIX H



DATE OF PHOTOGRAPHY: APRIL 1986

Source: SEWRPC.

Figure 44 (continued)



Source: SEWRPC.

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Chapter VII

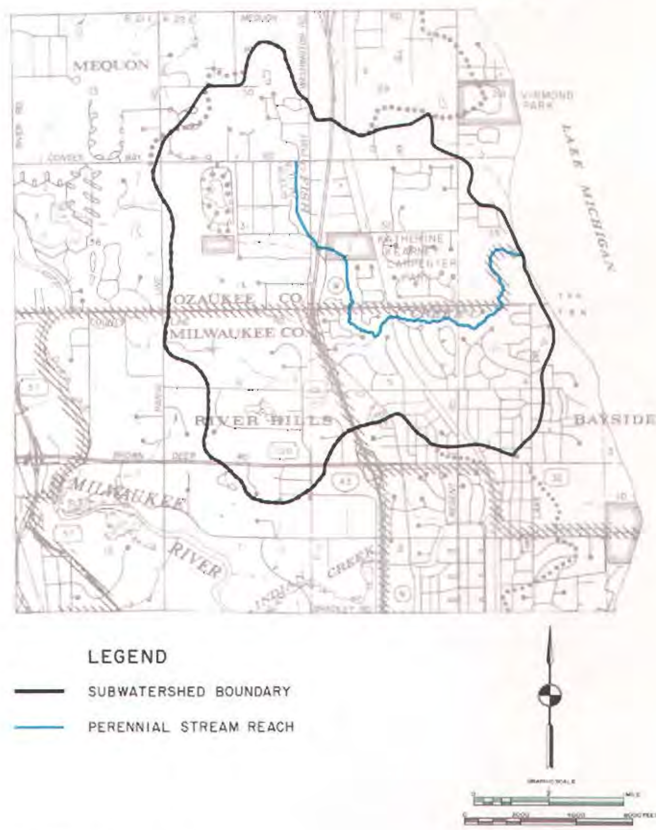
EVALUATION OF ALTERNATIVE AND SELECTION OF RECOMMENDED FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN—LAKE MICHIGAN DIRECT DRAINAGE AREA

INTRODUCTION

The drainage and flood control policy plan companion to this system plan recommends that that the Milwaukee Metropolitan Sewerage District assume jurisdiction for one perennial stream—Fish Creek—and no intermittent streams in the Lake Michigan direct drainage area. Data are presented on existing and probable future flood problems, alternative and recommended flood control and related drainage improvement measures, and recommended implementation actions.

Map 108

THE FISH CREEK SUBWATERSHED



Source: SEWRPC.

FISH CREEK SUBWATERSHED FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN

Fish Creek was not studied under any previous Commission planning programs. Flood flows and stages were developed by the Federal Emergency Management Agency (FEMA) for a portion of the creek through the Village of Bayside as part of the federal flood insurance study for that village. The hydrologic and hydraulic analyses conducted under this system planning effort represent a refinement of that earlier study.

Overview of the Study Area

Fish Creek is a direct tributary to Lake Michigan. The Fish Creek subwatershed is located within the City of Mequon and the Villages of Bayside and River Hills. From its origin at a storm sewer outfall located in Donges Bay Road near its intersection with Port Washington Road in the City of Mequon, Fish Creek drains in a generally easterly direction for a distance of about 3.43 miles, and drains an area of about 5.32 square miles (see Map 108). Of that total drainage area, 2.94 square miles, or about 55 percent, lies within the City of Mequon; 1.33 square miles, or about 25 percent, lies within the Village of Bayside; and 1.05 square miles, or about 20 percent, lies within the Village of River Hills.

More specifically, from its origin near the intersection of Donges Bay and Port Washington Roads, Fish Creek flows in a generally southerly direction to the intersection of Port Washington Road and Zedler Lane. From there it flows southeasterly to County Line Road, thence easterly to a concrete dam located about 0.35 mile downstream of N. Broadmoor Road, and thence northeasterly to its confluence with Lake Michigan. All of the 3.43-mile reach described is classified as perennial and is recommended for District jurisdiction in the policy plan companion to this system plan.

In 1985, about 66 percent of the Fish Creek subwatershed was developed for urban use, including residential, commercial, institutional, and urban open space uses. About 74 percent of the land converted to urban use was devoted to low-density residential development. Such development results in much less impervious land coverage than do more intense uses such as commercial or industrial. In the Village of River Hills, residential development is of a country estates nature, characterized by large lots, generally five acres or more in area. Also, runoff from the developed areas of the Fish Creek subwatershed is conveyed mainly by a series of grassed swales and roadside ditches which serve to attenuate the peak runoff rates. Therefore, peak flood discharges on Fish Creek can be expected to be much less than from areas devoted to more intensive urban use and served by a curb and gutter drainage system with attendant storm sewers.

Specific information on certain pertinent characteristics of the watershed, such as hydrologic soil types, land slopes, and land use, appears in Chapter II of this report. The planned land use conditions utilized in the system planning assume that, with the exception of the Village of River Hills, the watershed will be fully urbanized by the design year of the system plan. Some existing open spaces, such as parks, will remain; and the Village of River Hills is assumed to remain primarily in low-density, country estate-type, residential use.

Channel improvements have been made along about 0.9 mile of Fish Creek, all upstream of Katherine Kearney Carpenter Park in the City of Mequon. These modifications include deepening, straightening, and, in some reaches, lining with concrete. Downstream of the park, the creek is characterized by a deep and wide channel. Downstream of N. Broadmoor Road, the creek flows through a ravine up to 60 feet deep before entering Lake Michigan.

A state-designated scientific area known as Fairy Chasm is located along the downstream reach of Fish Creek. This area extends from the mouth of the creek at Lake Michigan upstream for a distance of about 1.25 miles, covering an area of about 60 acres and including several tributaries. The area is characterized by the deep gorges cut by the creek and its tributaries through glacial and lacustrine deposits. The north-facing slopes of the ravine support northern tree species such as white pine, yellow birch, and white cedar, while the warmer and more exposed south-facing slopes support a xeric hardwood forest. The area has special significance in that many of the plant species found there, such as leatherwood (Dirca palustris), occur only in cold air drainages in southern Wisconsin.

Stream bank erosion occurs along scattered reaches of Fish Creek downstream of Katherine Drive. This erosion may have been exacerbated with the development of the tributary drainage area. Currently, most of the erosion is limited to the "low-flow" channel and does not threaten the downstream ravine slopes such as those along Fairy Chasm. This situation could change as further development occurs within the watershed.

Flooding and Related Drainage Problems

An investigation of historical flood problems along Fish Creek which was conducted under this system plan indicated few problems. This lack of any serious flooding problems can be attributed, in part, to the fact that the natural channel through the Village of Bayside is contained in a deep ravine, and to completed channel modifications within the City of Mequon. Temporary closure of IH 43 did occur during the storm of September 10 and 11, 1986, owing to the accumulation of debris on a trash rack at the upstream end of the freeway culvert, which resulted in a reduction in the culvert's hydraulic capacity.

The results of the hydrologic and hydraulic analyses indicate that no structure flood damages are expected to occur along Fish Creek for floods up to and including the 100-year recurrence interval event under planned, year 2000 land use and existing channel conditions. Some homes and commercial properties may, however, experience indirect flood damages through sanitary sewer backup. It should be noted that the flood control measures considered under this system plan are primarily intended to alleviate flood damages from direct overland flooding along the stream studied, as well as to provide an adequate outlet for local storm sewers and drainageways. These measures, although not specifically designed to do so, may help to reduce flooding due to localized stormwater drainage problems or sanitary sewer backups.

The drainage and flood control objectives and supporting principles and standards set forth in Chapter III specify the flood events which bridges and culverts should be able to accommodate without overtopping the related roadway. Based on those criteria, one culvert—at Port Washington Road and Zedler Lane—is considered hydraulically inadequate, as shown in Appendix F.

Flood Discharges and Stages

As noted in Chapter III of this report, the hydrologic model used for development of design flood discharges for Fish Creek uses design rainfall events as input. The design rainfall events were developed using 10-, 50-, and 100-year rainfall volumes obtained from the updated point rainfall depth-duration-frequency relationships developed by the Commission as discussed in Chapter III. The rainfall distribution utilized for each design storm was the median distribution of a first-quartile storm, as shown in Chapter III for storm durations of one through six hours. Additional simulations were made for a storm duration of 24 hours using the U. S. Soil Conservation Service Type II storm distribution. This distribution provides for a majority of the rainfall to occur during the middle of the storm, after the soil has become partially saturated. Such a distribution would be expected to produce larger peak discharges in less intensively developed watersheds which are not extensively drained by storm sewers, such as the Fish Creek subwatershed. The design storm duration was determined for a given recurrence interval by simulating the peak discharge at a given location for a range of storm durations. The storm duration and associated rainfall volume which produced the largest peak discharge at a given location for a given recurrence interval was selected as the design storm for that location. This analysis was conducted for both existing and planned land use and channel conditions at 19 locations on the main stem of Fish Creek. The estimated peak flood discharges under existing and planned, year 2000 land use and existing channel conditions are set forth in Table 62. The reduction in flow in a downstream direction is due to floodwater storage behind restrictive roadway culverts.

A comparison of peak flood discharges developed under this system planning effort and those developed under the federal flood insurance study for the Village of Bayside is provided in Table 63. As shown in the table, the discharges are in relatively good agreement. The discharges used in the flood insurance study were estimated using regression equations developed for Wisconsin by the U. S. Geological Survey. Discharges resulting from use of those equations were adjusted to account for storage behind roadway culverts.

Flood stage profiles were determined for the 10-, 50-, and 100-year recurrence interval runoff events under planned land use and existing channel conditions. These profiles, which encompass the full 3.4-mile-long reach of Fish Creek studied, constitute a graphic representation of the flood stages along Fish Creek under the specified recurrence interval flood discharges, and under planned land use and existing channel conditions. In addition to providing an overall representation of flood stages relative to familiar points of reference such as the channel bottom and bridge deck surfaces, the profiles, because they are continuous, permit the determination of flood stages at any point along the stream channel. The flood profiles are shown in Figure 45. The extent of the 100-year recurrence interval floodplain under planned land use conditions is shown on Map 109. This delineation of the flood hazard area in the City of Mequon was accomplished using large-scale topographic maps at a scale of 1 inch equals 200 feet and a contour interval of five feet, supplemented by construction plans for the various modifications carried out along the channel. The delineation of the flood hazard area in the Village of Bayside was accomplished using U. S. Geological Survey 7.5-minute quadrangle maps at a scale of 1 inch equals 2,000 feet and a contour interval of 10 feet.

Recommended Flood Control
System for Fish Creek

As previously noted, no structure flood damages are expected along Fish Creek for floods up to and including the 100-year recurrence interval event under planned, year 2000 land use and existing channel conditions. Because of the lack of structure flood damages, no flood control or drainage alternatives were considered for Fish Creek. Accordingly, no flood control or drainage system plans are recommended.

It is recommended, however, that the Milwaukee Metropolitan Sewerage District prepare large-scale topographic maps for those areas along the entire length of Fish Creek. As of 1988, no large-scale topographic mapping was available for that portion of the creek through the Village of

Table 62

FLOOD DISCHARGES FOR EXISTING AND YEAR 2000 LAND USE AND EXISTING CHANNEL CONDITIONS

Location	River Mile	Peak Flood Discharge (cubic feet per second)					
		Existing Land Use Existing Channel Conditions			Year 2000 Planned Land Use, Existing Channel Conditions		
		10-Year	50-Year	100-Year	10-Year	50-Year	100-Year
Mouth at Lake Michigan	0.00	640	950	1,070	650	980	1,100
Upstream of Confluence with Unnamed Tributary	0.30	580	840	940	590	870	970
Dam Located Downstream of N. Broadmoor Road	1.22	510	670	740	520	700	770
At Chicago & North Western Railway	1.57	470	590	640	480	620	670
Upstream of Chicago & North Western Railway	1.59	680	1,090	1,250	710	1,130	1,290
Upstream of Confluence with Unnamed Tributary	1.80	460	720	820	490	750	860
W. County Line Road	2.11	450	690	780	480	720	820
Upstream of Confluence with Unnamed Tributary	2.15	360	560	630	370	570	640
Upstream of Confluence with Unnamed Tributary	2.57	240	370	400	260	370	430
At IH 43	2.75	230	340	380	250	350	420
Upstream of IH 43	2.80	190	270	290	200	270	350
At Zedler Lane	2.89	170	240	260	180	240	330
Upstream of Zedler Lane	3.00	180	300	350	190	310	360
Upstream of Confluence with Cedar Ridge Condominium Lake Outlet	3.20	13	21	24	13	21	24

Source: SEWRPC.

Table 63

COMPARISON OF FLOOD FLOWS: VILLAGE OF BAYSIDE FLOOD INSURANCE STUDY AND MMSD SYSTEM PLAN

Location	River Mile	100-Year Recurrence Interval Flood Discharge (cubic feet per second)		Percent Difference
		Bayside FIS	MMSD System Plan	
Dam Downstream of N. Broadmoor Road	1.22	705	740	5
Chicago & North Western Railway	1.57	625	640	2
Upstream of Chicago & North Western Railway	1.59	1,290	1,250	-3
Upstream of Confluence with Unnamed Tributary	1.80	850	820	-4

Source: SEWRPC.

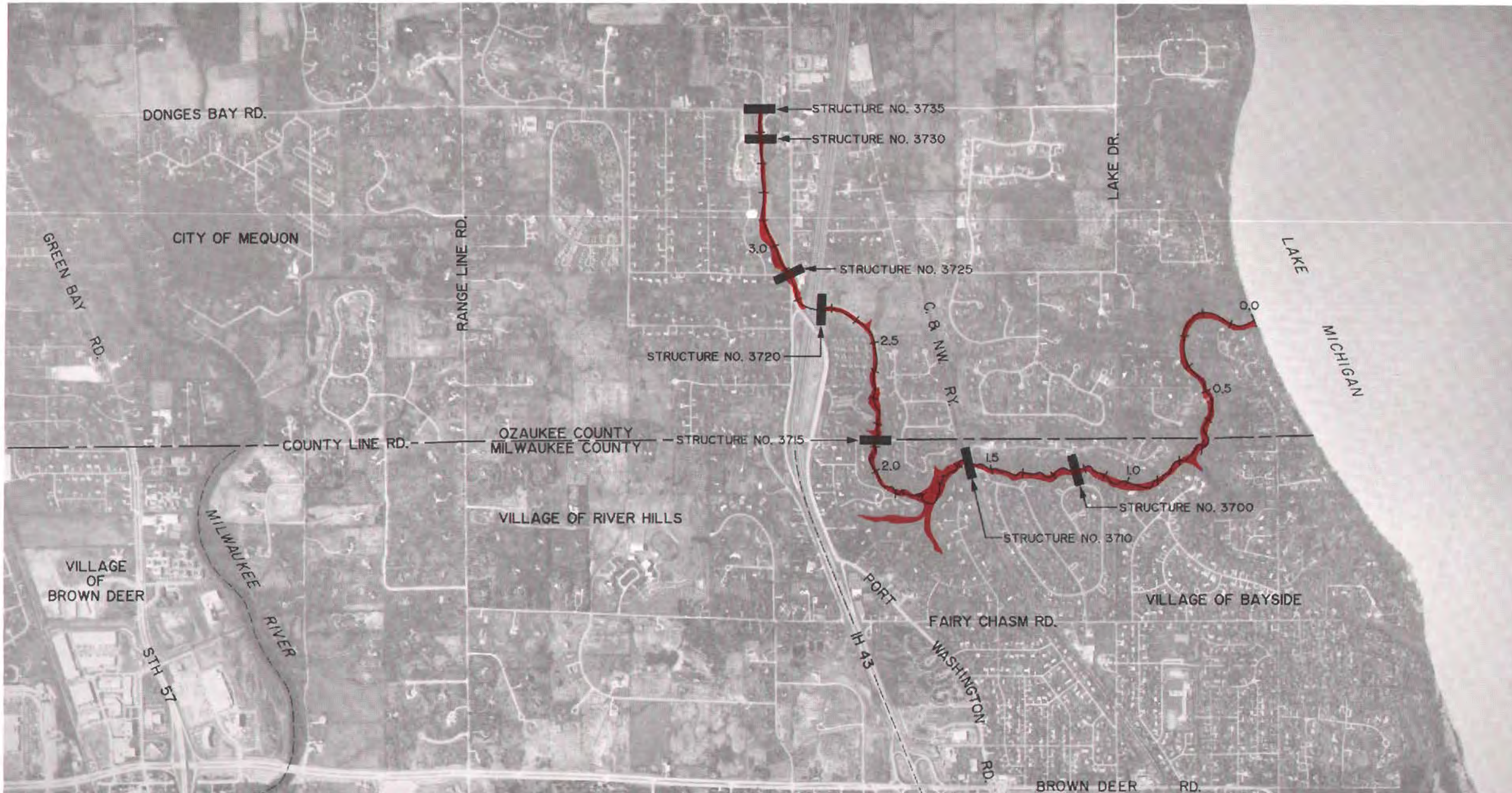
Bayside. In addition, a significant amount of development has occurred along Fish Creek in the City of Mequon since the preparation of large-scale topographic maps by that community. For this reason, the delineation of the flood hazard area along Fish Creek that is shown on Map 109 can only be considered approximate. Since the new large-scale topographic maps would serve multiple uses, no cost has been assigned to the flood control plan.

It is also recommended that when the culvert under Port Washington Road and Zedler Lane is replaced for transportation purposes, it be designed so as to accommodate the 50-year recurrence interval flood flow without overtopping Port Washington Road, and the 10-year recurrence interval flood flow without overtopping Zedler Lane.

Finally, it is recommended that the City of Mequon and the Village of Bayside request their respective county land conservation committees to cooperatively identify the extent of the erosion problems along Fish Creek, identifying the causes of that problem, and developing and implementing a sound solution to the problem.

Map 109

100-YEAR RECURRENCE INTERVAL FLOODPLAIN FOR FISH CREEK UNDER YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

2.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR FISH CREEK IS SHOWN IN APPENDIX H

GRAPHIC SCALE

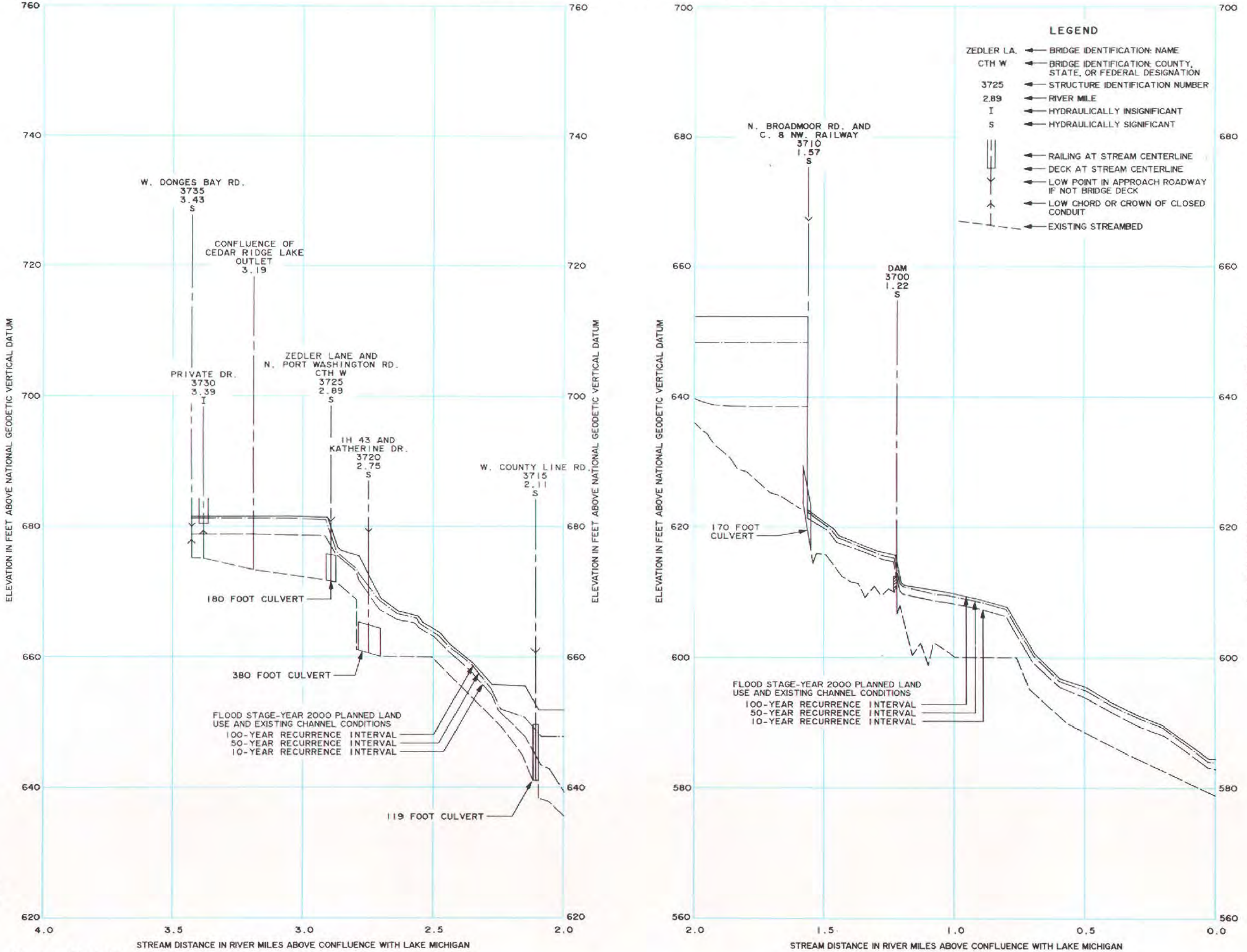
0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Source: SEWRPC.

Figure 45

FLOOD STAGE AND STREAMBED PROFILE FOR FISH CREEK



Source: SEWRPC.

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Chapter VIII

EVALUATION OF ALTERNATIVE AND SELECTION OF RECOMMENDED FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN—MILWAUKEE RIVER WATERSHED

INTRODUCTION

The drainage and flood control policy plan for the greater Milwaukee area, companion to this system plan, recommends that the Milwaukee Metropolitan Sewerage District assume jurisdiction for six perennial streams and one intermittent stream in the Milwaukee River watershed. These seven streams, totaling 16.5 lineal miles in length, include Indian Creek, Lincoln Creek, Pigeon Creek, unnamed Tributary 1, Beaver Creek, Southbranch Creek, and unnamed Tributary 2. Only one of these streams—Lincoln Creek—has been studied under previous Commission planning programs.[1] All but two of these streams are currently located within the District boundaries. These two streams—Pigeon Creek and unnamed Tributary 1—have not been included for detailed study under this system plan. Detailed system plans will be prepared for these two streams at such time that the District boundary is extended to include these watercourses. Each of the remaining five streams is considered in the following sections of this chapter. Data are presented on existing and probable future flood problems, alternative and recommended flood control and related drainage improvement measures, and recommended implementation actions.

INDIAN CREEK SUBWATERSHED FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN

Indian Creek was not studied under any previous Commission planning program. Flood flows and stages were developed by the Federal Emergency Management Agency (FEMA) for that portion of the creek through the Village of Fox Point as part of the federal flood insurance study for that village. An approximate delineation of the flood hazard area along Indian Creek through the Village of River Hills was also made under the federal flood insurance study for that village. The hydrologic and hydraulic analyses conducted under this system planning effort represent a refinement of those earlier studies.

Overview of the Study Area

Indian Creek is a tributary of the Milwaukee River. The Indian Creek subwatershed is located largely within the Villages of Fox Point and River Hills, with small portions located within the City of Glendale and the Village of Bayside. From its origin near the intersection of E. Brown Deer Road and N. Rexleigh Drive in the Village of Bayside, Indian Creek flows in a generally southwesterly direction for approximately 2.64 miles, and drains an area of about 3.12 square miles (see Map 110). Of this total drainage area, 0.19 square mile, or about 6 percent, lies within the City of Glendale; 0.38 square mile, or about 12 percent, lies within the Village of Bayside; 1.45 square miles, or about 47 percent, lie within the Village of Fox Point; and 1.10 square miles, or about 35 percent, lie within the Village of River Hills.

More specifically, from its origin near the intersection of E. Brown Deer Road and N. Rexleigh Drive, Indian Creek flows in a generally southerly direction to E. Spooner Avenue extended. From there it flows westerly to a point about 700 feet west of IH 43, thence southerly for a distance of about 0.3 mile, and thence westerly again to its confluence with the Milwaukee River about 500 feet south of W. Bradley Road. Of the 2.64-mile reach described, 1.94 miles, or 73 percent, is classified as perennial, while the remaining 0.70 mile, or 27 percent, is classified as intermittent. The entire perennial stream length is recommended for District jurisdiction in the policy plan companion to this system plan.

In 1985, about 82 percent of the Indian Creek subwatershed was developed for urban use, including residential, commercial, institutional, and urban open space uses. Most of the remaining open land, about 84 percent, is located in the

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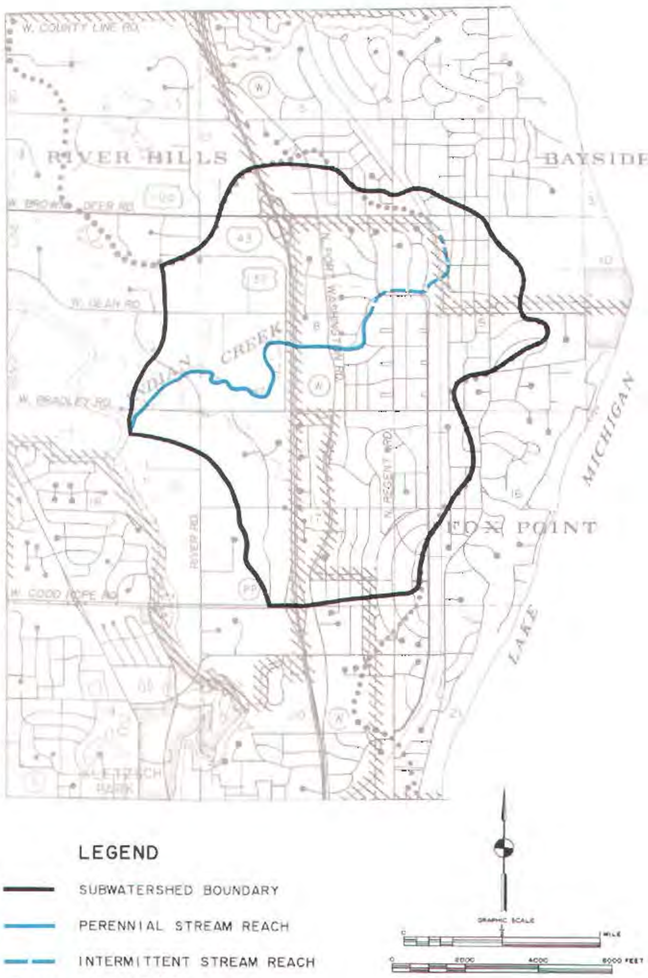
[1] *See SEWRPC Community Assistance Planning Report No. 13, Flood Control Plan for Lincoln Creek, Milwaukee County, Wisconsin (2nd Edition), September 1982.*

Map 110

THE INDIAN CREEK SUBWATERSHED



Source: SEWRPC.

Village of River Hills, where urban development is limited primarily to low-density residential use characterized by large lots, generally five acres or more in area. Runoff from the developed area of the Indian Creek subwatershed is conveyed mainly by a series of roadside drainage ditches. In the headwater area of the creek, many of these ditches are partially lined with concrete, however, so that surface runoff is conveyed relatively rapidly from individual sites to Indian Creek.

Specific information on certain pertinent characteristics of the watershed, such as hydrologic soil types, land slopes, and land use, appears in Chapter II of this report. The planned land use conditions utilized in the system planning assume that, with the exception of the Village of River Hills, the watershed will be fully urbanized by the design year of the system plan. Some existing open space uses, such as parks, will remain; and the Village of River Hills is assumed to remain primarily in low-density, country estate-type, residential use.

Channel improvements have been made along about 0.54 mile of perennial stream length in the Village of Fox Point and along the entire 0.70-mile intermittent stream length to accommodate increased streamflows. The channel has been physically altered by deepening, straightening, and in some reaches lining with concrete.

Flooding and Related Drainage Problems

Investigations of historic flood problems along Indian Creek indicate that few problems exist within the watershed except along Indian Creek Parkway in the Village of Fox Point. Reported problems resulting from the storms of September 10 and 11, 1986, included overland flooding in this area, with inundation of portions of Indian Creek Parkway Drive and some yard flooding. Flooding of homes along the Parkway during this flood event was limited to sanitary sewer backups. The flood discharge resulting from this storm event had a recurrence interval of about 30 years. The locations of reported flooding and drainage problems within the Indian Creek subwatershed during 1986 are shown on Map 111.

The results of hydrologic and hydraulic analyses indicate that the following numbers of existing residences may be expected to experience direct flooding along Indian Creek under existing and planned land use conditions and existing channel conditions:

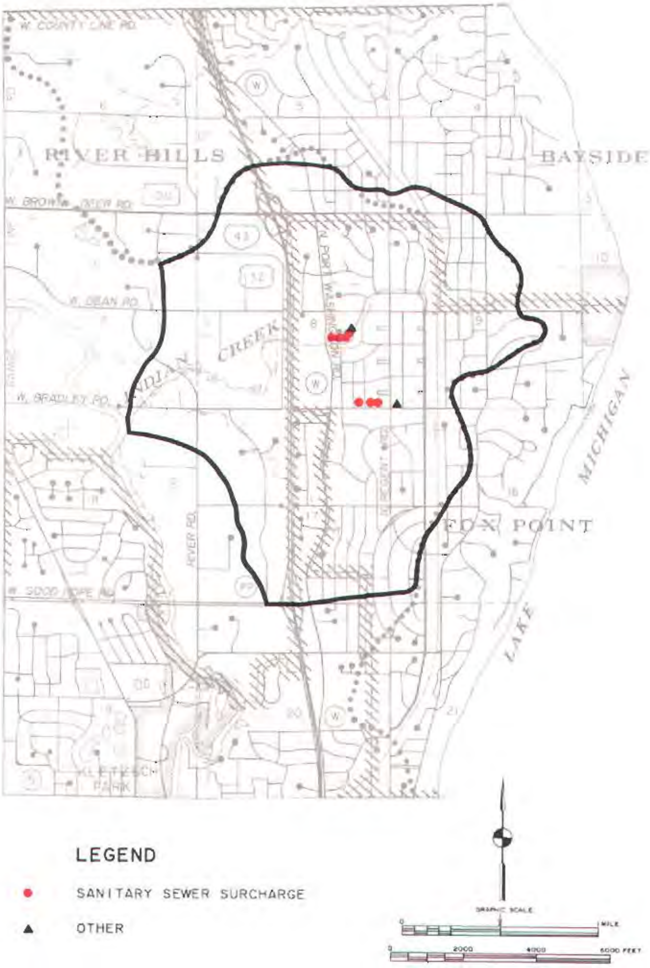
Flood Event Recurrence Interval	Approximate Number of Existing Homes Flooded Existing Land Use and Existing Channel Conditions	Approximate Number of Existing Homes Flooded Planned Land Use and Existing Channel Conditions
100	18	18
50	15	17
10	0	0

All of the homes which may be expected to incur direct flood damages are located within the Village of Fox Point along that reach of the stream between N. Port Washington Road and N. Seneca Avenue.

No commercial properties are expected to experience direct damages for floods up to and including the 100-year recurrence interval event under either existing or planned land use conditions. Additional homes and commercial proper-

Map 111

AREAS WITH REPORTED FLOODING AND DRAINAGE PROBLEMS IN THE INDIAN CREEK SUBWATERSHED: 1986



Source: SEWRPC.

ties may, however, experience indirect flood damages through sanitary sewer backup. It should be noted that the flood control measures considered under this system plan are primarily intended to alleviate flood damages from direct overland flooding along the stream studied, as well as to provide an adequate outlet for local storm sewers and drainageways. These measures may help to reduce flooding due to localized stormwater drainage problems or sanitary sewer backups.

The total average annual flood losses—damages—for Indian Creek are estimated at $17,500 under existing land use and channel conditions; and at $18,500 under planned land use and existing channel conditions. Flood losses from a 100-year recurrence interval event are estimated at $520,000 under existing land use and channel conditions; and at $536,000 under planned land use and existing channel conditions.

The drainage and flood control objectives and supporting principles and standards set forth in Chapter III specify the flood events which bridges shall accommodate without overtopping the related roadway. Based on those criteria, one bridge—E. Dean Road—is considered hydraulically inadequate, as shown in Appendix C.

Flood Discharges and Stages

As noted in Chapter III of this report, the hydrologic model used for development of design flood discharges for Indian Creek uses design rainfall events as input. The design rainfall events were developed using 10-, 50-, and 100-year rainfall volumes obtained from the updated point rainfall depth-duration-frequency relationships developed by the Commission as discussed in Chapter III. The rainfall distribution utilized for each design storm was the median distribution of a first-quartile storm as shown in Chapter III. The design storm duration was determined for a given recurrence interval by simulating the peak discharge at a given location for a range of storm durations. The storm duration and associated rainfall volume which produced the largest peak discharge at a given location for a given recurrence interval was selected as the design storm for that location. This analysis was conducted for both existing and planned land use and channel conditions at five locations along the main stem of Indian Creek. The flood discharges that were developed were then checked by utilizing recorded high-water mark data available for two crest-stage gages maintained by the Milwaukee Metropolitan Sewerage District. These gages are located at N. Pheasant Lane and at E. Dean Road, and as of 1987, each had been in operation for 20 years. Stage-discharge relationships developed under the Village of Fox Point federal flood insurance study were reviewed, found to be adequate, and used to convert the recorded stages to streamflow. A log Pearson Type III analysis was performed on the annual peak discharges for the 20 years of available record. The resulting discharge-frequency relationships were then used to check the discharges developed with the hydrologic model. A comparison of the simulated and recorded discharges at these two gages is provided in Table 64. The

Table 64

COMPARISON OF RECORDED AND SIMULATED DISCHARGES ALONG INDIAN CREEK

MMSD Gage Number	Location	Recurrence Interval	Recorded Discharge (cfs)	Simulated Discharge (cfs)	Percent Difference
SC 2-1	N. Pheasant Lane (downstream side)	10	900	780	-13
		50	1,370	1,320	- 4
		100	1,570	1,580	1
SC 2-2	E. Dean Road (downstream side)	10	640	660	3
		50	1,110	1,110	0
		100	1,380	1,320	- 4

Source: SEWRPC.

Table 65

FLOOD DISCHARGES FOR INDIAN CREEK FOR EXISTING AND YEAR 2000 LAND USE AND EXISTING CHANNEL CONDITIONS

Location	River Mile	Peak Flood Discharge (cfs)					
		Existing Land Use, Existing Channel Conditions			Year 2000 Planned Land Use, Existing Channel Conditions		
		10-Year	50-Year	100-Year	10-Year	50-Year	100-Year
Mouth at Milwaukee River . . .	0.00	890	1,530	1,870	910	1,560	1,890
W. Bradley Road	0.13	890	1,530	1,870	910	1,560	1,890
Private Drive	0.21	890	1,530	1,870	910	1,560	1,890
N. River Road	0.41	760	1,270	1,500	790	1,300	1,520
Private Drive	0.84	760	1,270	1,500	790	1,300	1,520
Private Drive	1.05	760	1,270	1,500	790	1,300	1,520
N. Pheasant Lane	1.36	780	1,320	1,580	810	1,370	1,610
IH 43	1.38	780	1,320	1,580	810	1,370	1,610
N. Port Washington Road	1.57	740	1,260	1,500	780	1,310	1,540
E. Dean Road	1.91	660	1,110	1,320	700	1,160	1,350

Source: SEWRPC.

estimated peak flood discharges under existing and year 2000 planned land use conditions and existing channel conditions are set forth in Table 65.

Flood stage profiles were determined for the 10-, 50-, and 100-year recurrence interval runoff events under planned land use and existing channel conditions. These profiles, which encompass the full 1.9-mile-long reach of Indian Creek studied, constitute a graphic representation of the flood stages along Indian Creek under the specified recurrence interval flood discharges, and under planned land use and existing channel conditions. In addition to providing an overall representation of flood stages relative to familiar points of reference such as the channel bottom and bridge deck surfaces, the profiles, because they are continuous, permit the determination of flood stages at any point along the stream channel. The flood profiles are shown in Figure 46.

Table 66

COMPARISON OF FLOOD FLOWS AND STAGES: VILLAGE OF FOX POINT FLOOD INSURANCE STUDY AND MMSD SYSTEM PLAN

Location	River Mile	100-Year Recurrence Interval Flood Discharge (cfs)			100-Year Recurrence Interval Flood Stage (feet NGVD)		
		Fox Point FIS	MMSD System Plan	Percent Difference	Fox Point FIS	MMSD System Plan	Stage Difference (feet)
IH 43 (upstream side)	1.40	1,200	1,580	30	658.8	660.2	1.4
N. Port Washington Road (downstream side)	1.57	1,200	1,580	30	659.2	660.4	1.2
N. Port Washington Road (upstream side)	1.57	1,000	1,500	50	660.7	665.2	4.5
Upstream of N. Port Washington Road	1.73	1,000	1,500	50	660.8	665.3	4.5
E. Dean Road (downstream side)	1.91	1,000	1,500	50	660.9	665.3	4.4
E. Dean Road (upstream side)	1.91	1,000	1,320	32	661.4	665.4	4.0

Source: SEWRPC.

As shown in Table 66, the flood flows and stages developed under this system planning effort are somewhat higher than those developed under the federal flood insurance study for the Village of Fox Point. It is believed that the discharges developed under this system planning effort constitute a more accurate representation of the watershed performance since they correlate well—generally within 5 percent—with the discharge-frequency relationships for the two District crest-stage gages. Those discharge-frequency relationships are based on 20 years of data, more than double the period of record available for calibration purposes at the time that the federal flood insurance study was prepared. Included in the additional years of record is the September 1986 storm event, which produced the highest recorded stages for the 20-year period of record.

The extent of the 100-year recurrence interval floodplain under planned land use conditions is shown on Map 112. This delineation of the flood hazard area was accomplished using large-scale topographic maps prepared in 1966 and 1970 for that reach within the Village of River Hills. No large-scale topographic mapping is available for the stream reach within the Village of Fox Point. The flood hazard area shown on Map 112 for this reach was delineated using field-surveyed cross-section data, and therefore can only be considered approximate.

Alternative Flood Control and Related Drainage System Plans for Indian Creek

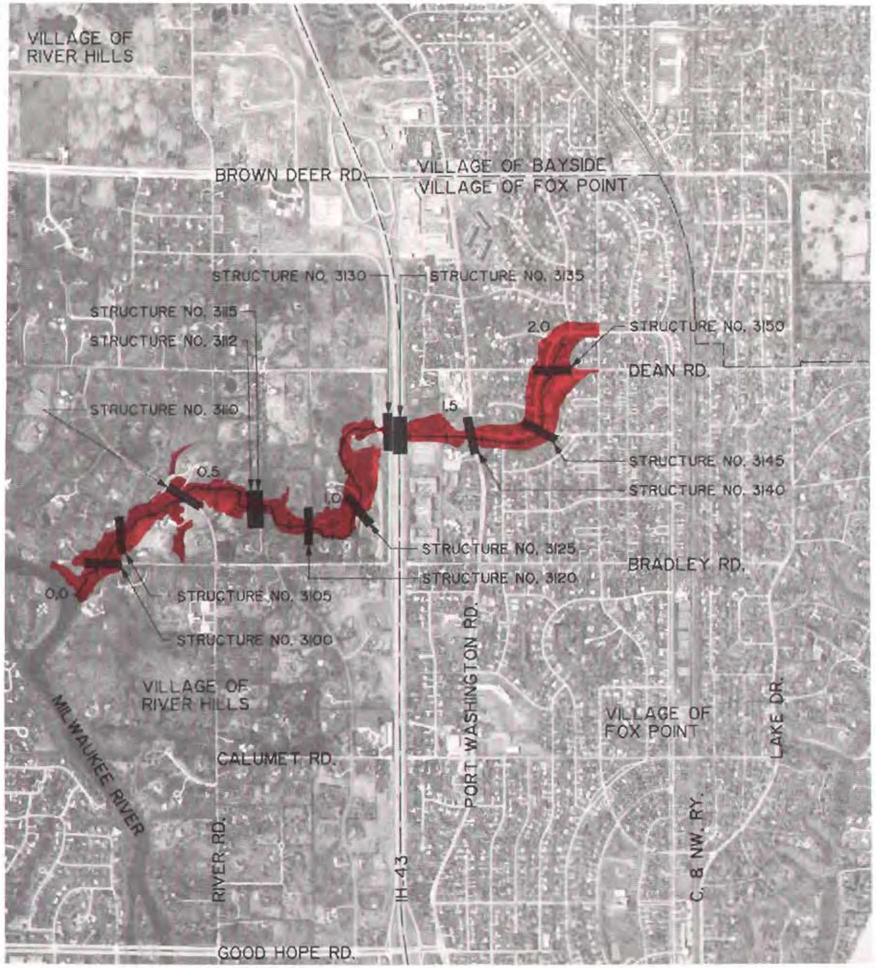
Three alternative flood control plans were considered for alleviating the flood damage problems along Indian Creek: Alternative Plan 1—no action; Alternative Plan 2—structure floodproofing, elevation, and removal; and Alternative Plan 3—culvert replacement.

Each alternative is described below. The estimated economic benefits and costs attendant to each alternative are provided in Table 67.

Alternative Plan 1—No Action: One alternative course of action is to do nothing—that is, to recognize the inevitability of flooding, but to deliberately decide not to mount a collective, coordinated program to abate the flood damages. Under year 2000 planned land use and existing channel conditions, the average annual flood damages along this reach would approximate $18,500. There are no monetary benefits associated with this alternative, and the average annual cost would be equivalent to the average annual flood damage cost of $18,500. The damages associated with the 100-year recurrence

Map 112

100-YEAR RECURRENCE INTERVAL FLOODPLAIN FOR INDIAN CREEK UNDER YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

1.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR INDIAN CREEK IS SHOWN IN APPENDIX H

GRAPHIC SCALE

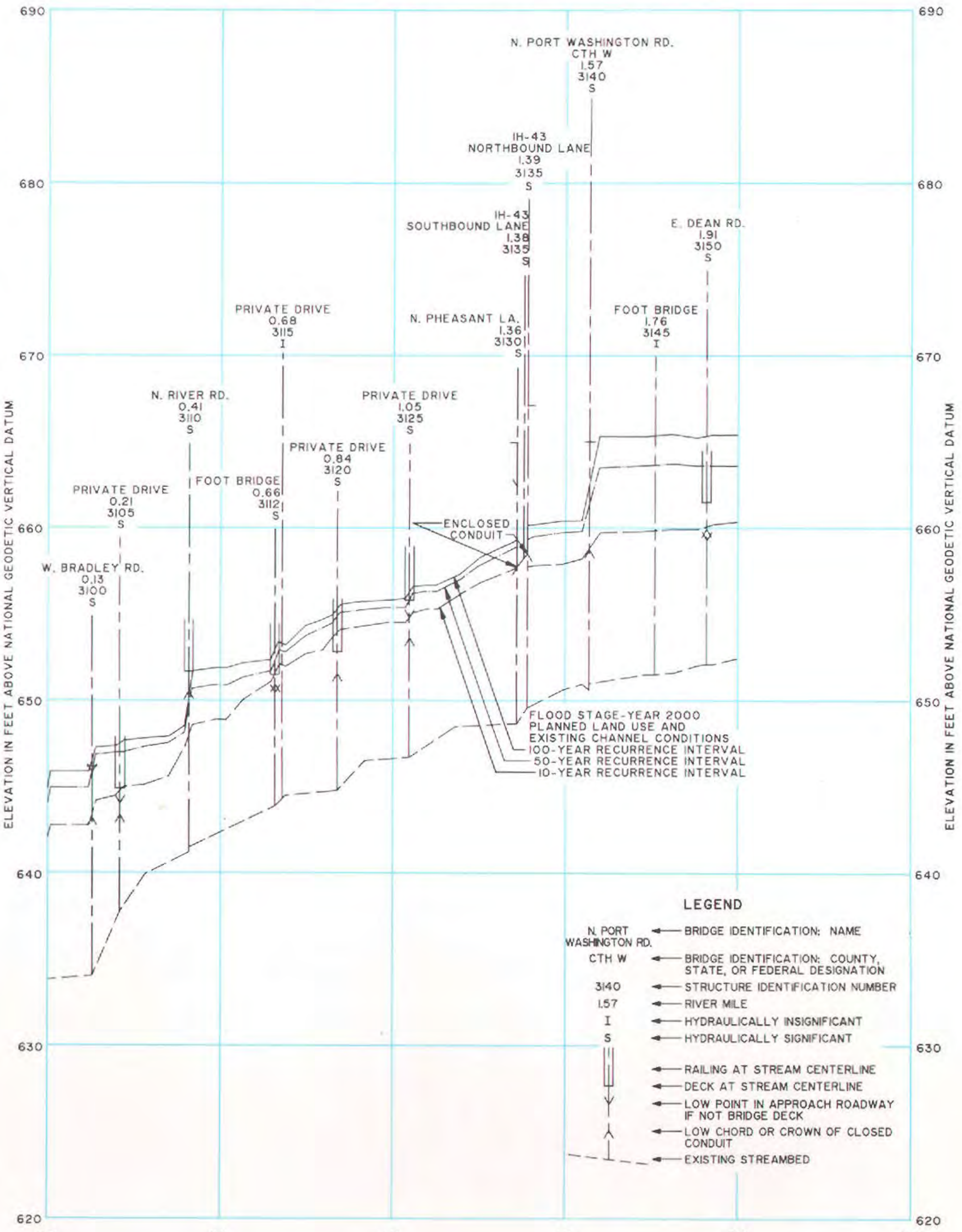
0 1/2 1 MILE

DATE OF PHOTOGRAPHY APRIL 1986

Source: SEWRPC.

Figure 46

FLOOD STAGE AND STREAMBED PROFILE FOR INDIAN CREEK



Source: SEWRPC.

Table 67

COST ESTIMATES FOR FLOOD CONTROL ALTERNATIVES FOR INDIAN CREEK IN THE VILLAGE OF FOX POINT

Alternative	Description	Costs					Benefit-Cost Analysis			
		Capital	Annual				Annual Benefits	Annual Benefits Minus Annual Costs	Benefit-Cost Ratio	Ratio Greater than One
			Amortized Capital[a]	Operation and Maintenance	Other	Total				
1. No Action	--	$ 0	$ 0	$ 0	$18,500	$18,500	$ 0	$-18,500	--	No
2. Structure Floodproofing, Elevation, and Removal	Elevate 18 residential structures	$627,000	$39,800	$ 0	$ 0	$39,800	$18,500	$-21,300	0.46	No
3. Culvert Replacement	Replace culverts at one road crossing	$290,000[b]	$18,400	$ 0	$ 0	$18,400	$18,500	$ 100	1.01	Yes

[a]*Amortized capital cost is based on an interest rate of 6 percent and a project life of 50 years.*

[b]*Cost of this bridge replacement was previously assigned under the Commission's adopted regional transportation plan.*

Source: SEWRPC.

interval flood under year 2000 planned land use and existing channel conditions would approximate $536,000.

Alternative Plan 2—Structure Floodproofing, Elevation, and Removal: A structure floodproofing, elevation, and removal alternative flood control plan was analyzed to determine if such a structure-by-structure approach would be a technically feasible and economically viable solution to the flood problem along Indian Creek. For analytical purposes, the 100-year recurrence interval flood stage under year 2000 planned land use and existing channel conditions was used to estimate the number of existing flood-prone structures to be floodproofed, elevated, or removed and the approximate costs involved.

In the case of residential structures, floodproofing was assumed to be feasible if the design flood stage was below the first floor elevation. Structure elevation was considered feasible for residential structures with basements if the estimated cost of elevating the structure was less than the estimated removal cost. Structures to be elevated were assumed to have the first floor raised to an elevation of at least two feet above the 100-year recurrence interval flood stage to provide adequate freeboard. For aesthetic reasons, structure elevation was limited to a maximum of four feet. Structures that would have to be elevated more than four feet were considered for removal.

As shown on Map 113, all of the 18 houses which may be expected to incur flood damage would have to be elevated. Damage from floods up to and including the 100-year recurrence interval event would be virtually eliminated.

Assuming that these structure floodproofing measures would be fully implemented, and utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of this alternative is estimated at $39,800. This cost consists of the amortization of the $627,000 capital cost for structure elevation. The average annual flood damage abatement benefit is estimated at $18,500, yielding a benefit-cost ratio of 0.46.

Alternative Plan 3—Culvert Replacement: This alternative plan for the resolution of the flood problem along Indian Creek is shown on Map 114, and consists of replacing the existing culverts under N. Port Washington Road at River Mile 1.57. Currently, flow under this road crossing is carried by three 96-inch-diameter corrugated metal pipes, each 132 feet in length. During major flood events, these pipes are submerged, producing significant backwater

Map 113

ALTERNATIVE PLAN 2: STRUCTURE FLOODPROOFING, ELEVATION, AND REMOVAL ALONG INDIAN CREEK

Source: SEWRPC.

Map 114

ALTERNATIVE PLAN 3: CULVERT REPLACEMENT ALONG INDIAN CREEK

Source: SEWRPC.

effects. Under this alternative, these culverts would be replaced by two 12-foot-wide by 12-foot-high reinforced concrete box culverts, each 132 feet in length. It should be noted that this crossing is already designated for replacement under the adopted regional transportation system plan. Therefore, the cost of this bridge replacement has already been assigned under that plan.

Implementation of this alternative would essentially eliminate all damages due to overland flooding for floods up to and including the 100-year recurrence interval event. Because of the reduction in floodwater storage resulting from this culvert replacement, downstream flood flows and stages may be expected to increase as shown in Table 68. The 100-year recurrence interval flood discharge under planned land use conditions may be expected to increase from 8 to 18 percent, causing increases in downstream flood stages ranging from 0.2 to 0.8 foot. No additional structural flood damages, however, are expected as a result of these increased flows. Flood easements or other legal arrangements may, however, have to be obtained from property owners affected by the stage increase.

Utilizing an annual interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this alternative is estimated at $18,400. This cost consists of the amortization of the $290,000 capital cost of the culvert replacement. The average annual flood abatement benefit is estimated at $18,500, yielding a benefit-cost ratio of 1.01.

Table 68

ANTICIPATED INCREASES IN PLANNED LAND USE FLOOD FLOWS AND STAGES ALONG INDIAN CREEK DUE TO IMPLEMENTATION OF ALTERNATIVE SYSTEM 3: CULVERT REPLACEMENT

Location	River Mile	100-Year Recurrence Interval Flood Discharge (cfs)			100-Year Recurrence Interval Flood Stage (feet NGVD)		
		Existing Culverts	Proposed Culverts	Percent Increase	Existing Culverts	Proposed Culverts	Stage Increase (feet)
Mouth	0.00	1,890	2,040	8	646.0	646.3	0.3
W. Bradley Road (upstream side)	0.13	1,890	2,040	8	647.4	647.5	0.1
Private Drive (upstream side)	0.21	1,890	2,040	8	647.8	648.0	0.2
N. River Road (upstream side)	0.41	1,520	1,700	11	651.8	652.6	0.8
Private Drive (upstream side)	0.66	1,520	1,700	11	653.5	653.8	0.3
Private Drive (upstream side)	0.84	1,520	1,700	11	655.6	656.0	0.4
Private Drive (upstream side)	1.05	1,520	1,700	11	656.7	657.0	0.3
N. Pheasant Lane (downstream side)	1.36	1,520	1,700	11	659.3	656.6	0.3
IH 43 (upstream side)	1.39	1,610	1,890	17	660.2	660.9	0.7
N. Port Washington Road (downstream side)	1.57	1,610	1,890	17	660.5	661.0	0.5
N. Port Washington Road (upstream side)	1.57	1,540	1,810	18	665.4	661.8	-3.6
E. Dean Road (upstream side)	1.91	1,350	1,350	0	665.5	662.5	-3.0

Source: SEWRPC.

Evaluation of Flood Control Alternatives for Indian Creek

The costs associated with each of the floodland management alternatives considered for Indian Creek are summarized in Table 67. Both Alternatives 2 and 3 were found to be technically feasible. Alternative 3—culvert replacement—produced a benefit-cost ratio of about one. The "no action" alternative, while offering the lowest cost, does nothing to alleviate the flood problem and does not represent a sound approach to flood control.

Alternative Plan 2—structure floodproofing, elevation, and removal—presents several problems in implementation. First, complete implementation of a voluntary structure floodproofing and elevation program is unlikely, and with partial implementation, the Village of Fox Point would be left with a residual problem whenever a major flood event occurred. Also, yard damages and cleanup costs would remain under this alternative.

Implementation of Alternative Plan 3—culvert replacement—would serve to eliminate structure flooding up to a 100-year recurrence interval event while yielding a benefit-cost ratio of about one. In addition, by reducing the flood profile upstream of N. Port Washington Road, this alternative should help to reduce the severity of sanitary sewer backups in this area. Although this alternative is expected to cause increased flood flows and stages in downstream reaches, these increases are small enough so as to produce no structure flood damages. Implementation of this alternative may be difficult since legal arrangements would have to be made with all affected property owners along the reaches downstream of N. Port Washington Road.

Recommended Flood Control System for Indian Creek

Based upon consideration of the technical feasibility, economic viability, environmental impacts, potential public acceptance, and practicality of each of the alternatives considered, it is recommended that Alternative Plan 3—culvert replacement—be adopted for Indian Creek.

The total capital cost of the recommended flood control plan is estimated at $290,000 in 1986 dollars. The recommended plan is shown on Map 115.

Implementation of the recommended plan would essentially eliminate all flood-related damages to existing structures along the Indian Creek channel for floods up to and including the 100-year recurrence interval event under planned land use conditions.

The recommended plan calls for replacing the existing culverts under N. Port Washington Road with two 12-foot-wide by 12-foot-high reinforced concrete box culverts. As previously noted, the recommended replacement would result in an increase of 0.2 to 0.8 foot in the downstream 100-year recurrence interval flood profile under planned land use conditions, requiring the acquisition of flooding easements from affected property owners. This increase would not, however, result in additional structure flood damage. The recommended plan 100-year recurrence interval flood profile is shown in Figure 47.

In addition to the culvert replacement, it is recommended that large-scale topographic maps be prepared for the area in the Village of Fox Point along that reach of Indian Creek designated for District jurisdiction. No large-scale topographic mapping exists for that area. These maps are needed to accurately delineate the limits of the floodplain. Large-scale topographic maps are available for the area along Indian Creek in the Village of River Hills. These include maps prepared in 1966 for the Village of River Hills, as well as maps prepared in 1970 as part of the Commission's Milwaukee River watershed study. A review of 1985 aerial photographs for this area indicates that few changes have occurred in the riverine area through this reach. Accordingly, no new topographic maps are recommended for the reach of Indian Creek through the Village of River Hills. Since the new maps would serve multiple purposes, none of the attendant costs have been assigned to the flood control plan.

Table 69

SUMMARY OF RECOMMENDED PLAN CAPITAL COSTS—INDIAN CREEK

Implementing Agency	Improvements	Estimated Capital Cost
Milwaukee Metropolitan Sewerage District	Culvert Removal	$ 29,000
Milwaukee County	Culvert Replacement	261,000
Total	- -	$290,000

Source: SEWRPC.

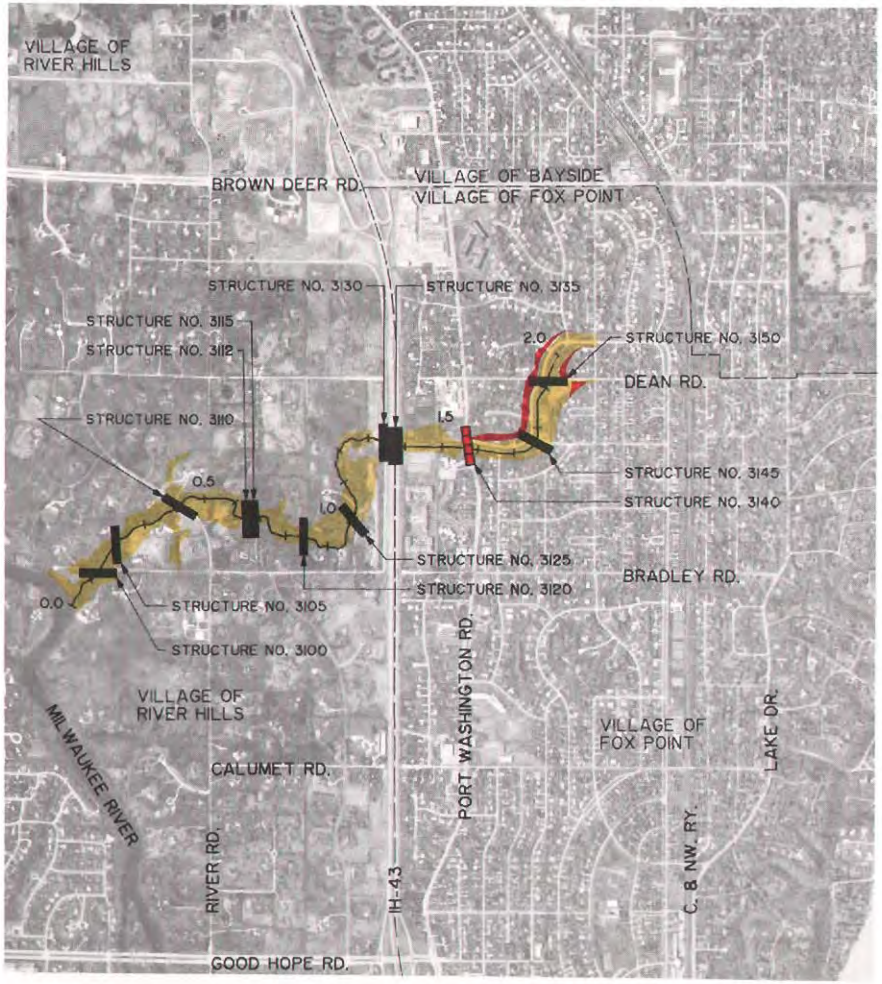
Flood Control and Related Drainage System Plan Implementation

It is recommended that the structural measures developed for the abatement of flood problems along Indian Creek be implemented through the cooperative efforts of Milwaukee County and the Milwaukee Metropolitan Sewerage District. More specifically, it is recommended that the District pay for the removal of the existing culverts, and that the County pay for the design and construction of the replacement culverts at N. Port Washington Road. It is further recommended that the District make appropriate legal arrangements, as necessary, with all downstream property owners who will be affected by the recommended culvert replacement. Finally, it is recommended that the District prepare large-scale topographic maps for the area along that reach of Indian Creek in the Village of Fox Point recommended for District jurisdiction.

The capital costs associated with the recommended plan for Indian Creek are summarized in Table 69.

Map 115

RECOMMENDED FLOOD CONTROL SYSTEM PLAN FOR INDIAN CREEK



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS

PROPOSED CULVERT REPLACEMENT

1.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR INDIAN CREEK IS SHOWN IN APPENDIX H

DUE TO MAP SCALE LIMITATIONS, THE DIFFERENCE BETWEEN THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS, AND THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS, MAY NOT APPEAR ON THIS MAP. WHERE NO DIFFERENCE APPEARS REFERENCE SHOULD BE MADE TO THE FLOOD STAGE PROFILE SHOWN BELOW

GRAPHIC SCALE

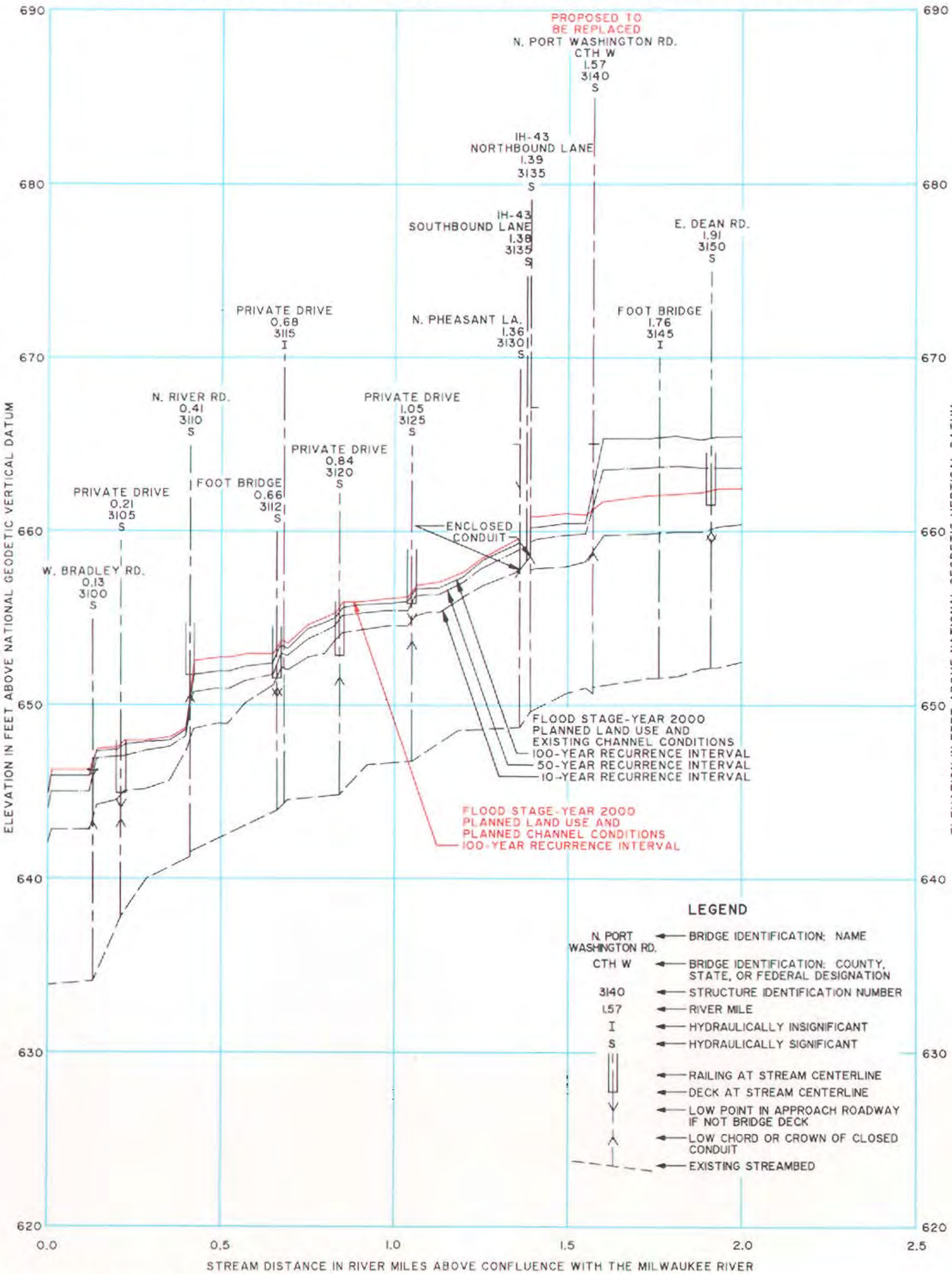
0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Source: SEWRPC.

Figure 47

RECOMMENDED PLAN FLOOD STAGE PROFILE FOR INDIAN CREEK



Source: SEWRPC.

LINCOLN CREEK SUBWATERSHED FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN

Flood control and related drainage improvements for Lincoln Creek were considered in a flood control plan prepared by the Commission in September 1982.[2] That report was completed in the absence of large-scale topographic mapping for all except a small portion of the subwatershed, and thus the work had to rely primarily on surveyed stream valley cross-sections for topographic data. While adequate data were available to characterize the stream channel and to estimate flood flows and stages, it was not possible to precisely delineate the limits of the flood hazard areas. In 1986, new topographic maps at a scale of one inch equals 100 feet, with two-foot contour interval, were prepared for the riverine areas of the subwatershed. The plan herein presented represents a refinement of the previously prepared plan, incorporating a revised flood hazard area delineation using the recently obtained large-scale topographic maps. In addition, rainfall and flooding events that have occurred since completion of the previous plan have been considered in the reevaluation, and the costs of the recommended plan have been updated to a 1986 base.

Overview of the Study Area

Lincoln Creek is a tributary of the Milwaukee River. The Lincoln Creek subwatershed is located almost entirely within the City of Milwaukee. Small portions of the subwatershed are located in the Village of Brown Deer and the City of Glendale. From its headwater area near N. 76th Street and W. Good Hope Road, Lincoln Creek flows in a generally southeasterly direction for a distance of approximately 9.7 miles, and drains an area of about 19.26 square miles (see Map 116).

More specifically, from its origin near N. 76th Street and W. Good Hope Road, Lincoln Creek flows in a generally easterly direction to the vicinity of N. 51st Street and W. Good Hope Road. From this point, the creek flows in a generally southerly direction to the vicinity of N. 60th Street and W. Hampton Avenue, and thence in a generally easterly direction to its confluence with the Milwaukee River in Lincoln Park near N. Green Bay Avenue and W. Villard Avenue. Of the 9.7-mile reach described, 8.1 miles, or 84 percent, is classified as perennial, while the remaining 1.6 miles, or 16 percent, is classified as intermittent. The entire perennial stream length and 0.5 mile of intermittent stream, a total of 8.6 miles, is recommended for District jurisdiction in the policy plan companion to this system plan.

For the purpose of this report, that portion of Lincoln Creek lying north of W. Silver Spring Drive has been designated "Upper Lincoln Creek," and that portion lying south of W. Silver Spring Drive has been designated "Lower Lincoln Creek." Upper Lincoln Creek drains an area of about 4.09 square miles. In 1985, about 50 percent of this area had been developed for urban use. The remaining open space land uses consisted primarily of golf courses and cemeteries, with some agricultural and unused land, and the Havenwoods Environmental Center.

Lower Lincoln Creek drains an area of about 15.17 square miles lying between W. Silver Spring Drive and the Milwaukee River. In 1985, this area was almost completely developed for urban use, including residential, commercial, industrial, institutional, and urban open space uses. The open space uses were comprised of public parks, cemeteries, and a parkway system located along Lincoln Creek from the vicinity of N. 60th Street and W. Hampton Avenue to Lincoln Park on the Milwaukee River. The developed areas of the Lincoln Creek subwatershed are generally provided with a full range of municipal street improvements, including paved streets with curbs and gutters and attendant storm sewers. Accordingly, surface runoff is generally conveyed rapidly from each individual site to Lincoln Creek through storm sewers.

Specific data on certain pertinent characteristics of the subwatershed, such as soil types, land slopes, and land use, appear in Chapter II of this report. It should be noted that the planned land use considered in this report assumes that the subwatershed will be fully urbanized. However, existing open space uses such as parks, cemeteries, golf courses, and the Havenwoods Environmental Center would remain.

Flooding, in various degrees, is a common occurrence adjacent to Lincoln Creek. Flooding along the creek has increased proportionally to

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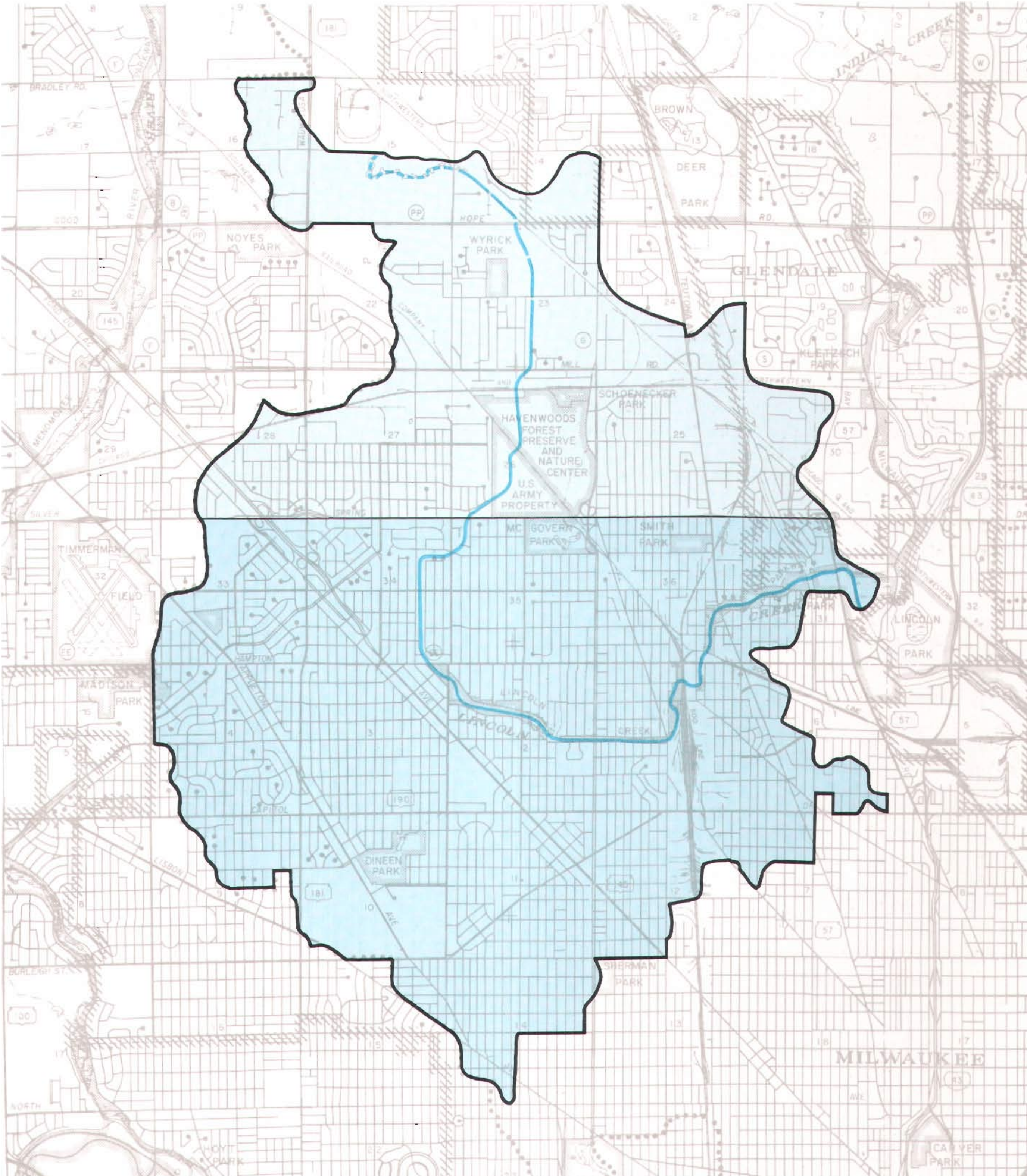
[2] *Ibid.*

Map 116

THE LINCOLN CREEK SUBWATERSHED



LEGEND

SUBWATERSHED BOUNDARY

PERENNIAL STREAM REACH RECOMMENDED FOR MMSD JURISDICTION

INTERMITTENT STREAM REACH RECOMMENDED FOR MMSD JURISDICTION

INTERMITTENT STREAM REACH RECOMMENDED FOR OTHER JURISDICTION

UPPER LINCOLN CREEK

LOWER LINCOLN CREEK

Source: SEWRPC.

the conversion of land from open, rural uses to urban uses. Some channel improvements have been made to accommodate the increased streamflows. Only a short segment of the creek, about 1.3 miles in length, from N. Teutonia Avenue to the confluence with the Milwaukee River is in a relatively natural state. The remainder of the channel has been physically altered by deepening, straightening, or lining with concrete or stone, and by the construction of sills or drop spillways.

Flooding and Related Drainage Problems

Flooding occurs quite frequently along Lincoln Creek. Over the 25-year period 1960 through 1985, more than 1,300 separate flooding and water-related problems have been reported by property owners in the area, including first floor flooding, yard flooding, and basement flooding, based upon records maintained by the City Engineer of the City of Milwaukee. During 1986, there were six storm events for which flooding and water-related problems in the Lincoln Creek subwatershed were documented. More than 640 separate such problems were documented during 1986, with the most—327—being reported during the August 6 rainfall. Problems reported included first floor inundation, basement flooding, and yard flooding, with the most common complaint being basement flooding caused by sewer backup. Flooding of roadways and underpasses has also occurred frequently within the subwatershed.

The areas that most frequently experience flooding problems in the subwatershed are outlined on Map 117. The total average annual monetary flood damages within the Lincoln Creek subwatershed resulting from direct overland flooding are estimated at $618,000 under 1985 land use conditions, and $837,000 under year 2000 planned land use conditions.

In addition to the direct overland flooding problems within the Lincoln Creek subwatershed, drainage problems occur. These problems are related to nine major storm sewer outlets located between the Chicago & North Western Railway crossing at River Mile 6.73 and the upstream crossing of the same railway at River Mile 8.49 which discharge into Lincoln Creek but do not have free outlets. As shown in Figure 48, the inverts of these storm sewer outlets are set at elevations below the existing channel bottom. Some of the storm sewer outlets are provided with smaller diameter outlet pipes which slope upward to allow stormwater to be discharged above the channel bottom, while other sewers are filled with streambed material to the elevation of the existing channel bottom. This condition has been reported by the City Engineer of the City of Milwaukee to cause deposition of solids in the tributary storm sewer systems, resulting in the need for special maintenance and the potential for stormwater ponding in the tributary drainage areas due to the restricted capacity of the storm sewer outlets.

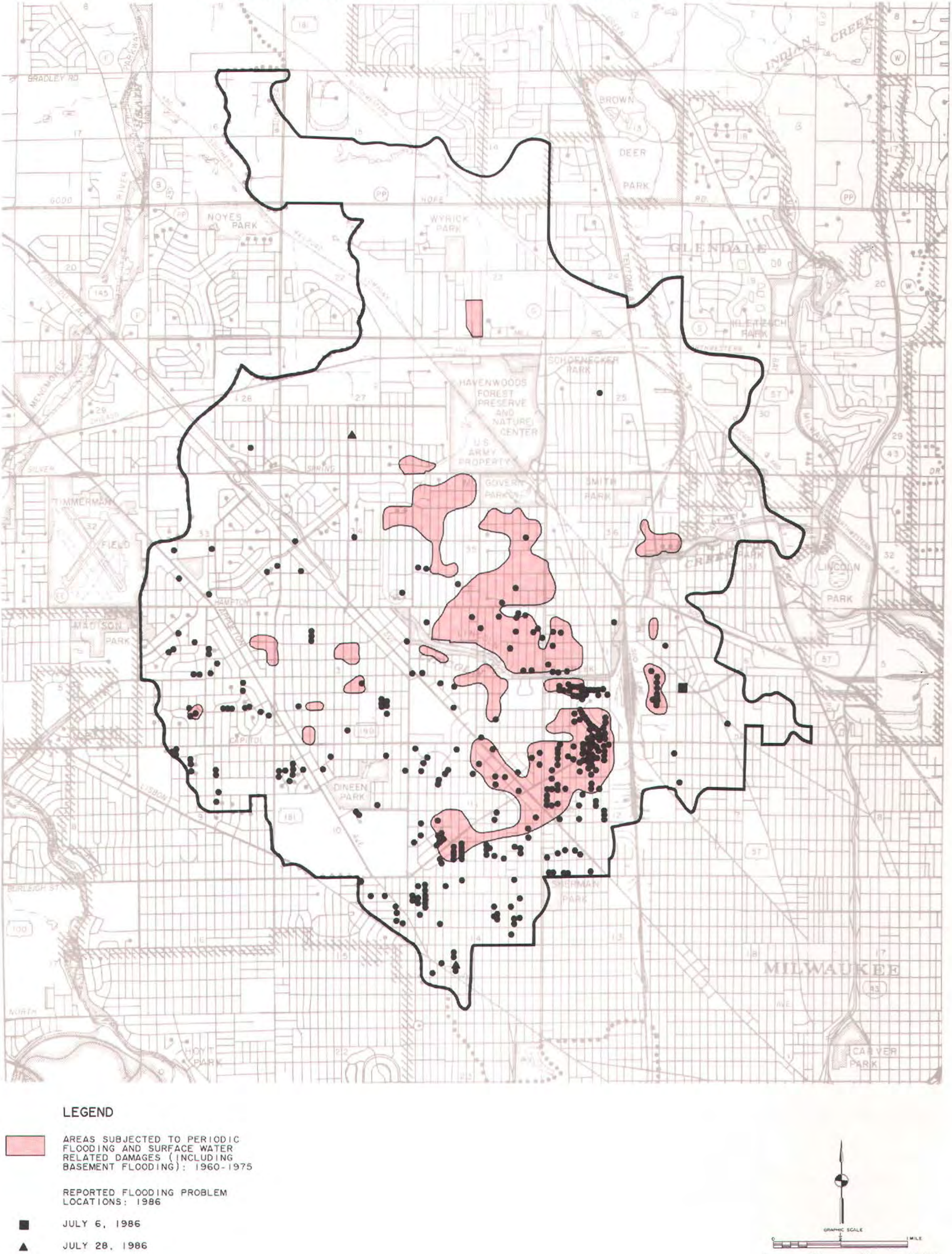
The drainage and flood control objectives and supporting principles and standards set forth in Chapter III specify the flood events which bridges shall accommodate without overtopping the related roadway. Based on those criteria, seven bridges are considered hydraulically inadequate as shown in Appendix C. These bridges are located at N. Teutonia Avenue, N. 35th Street, N. Sherman Boulevard, N. 60th Street and W. Custer Avenue, W. Woolworth Avenue, N. 51st Street, and W. Mill Road.

Flood Discharges and Stages

As noted in Chapter III of this report, the hydrologic model used for developing design flood discharges for Lincoln Creek uses design rainfall events as input. The design rainfall events were developed using 10-, 50-, and 100-year rainfall volumes obtained from the updated point rainfall depth-duration-frequency relationships developed by the Commission as discussed in Chapter III. The rainfall distribution utilized for each design storm was the median distribution of a first-quartile storm, as shown in Chapter III. The design storm duration was determined for a given recurrence interval by simulating the peak discharge at a given location for a range of storm durations. The storm duration and associated rainfall volume that produced the largest peak discharge at a given location for a given recurrence interval was selected as the design storm for that location. This analysis was conducted for both existing and planned land use and channel conditions at 24 locations on the main stem of Lincoln Creek. The flood discharges simulated by the hydrologic model were then checked by incorporating the discharges into a hydraulic model to develop stages and comparing those stages to high-water mark data. Such data were available for a period of up to 22 years at 17 locations on the channel

Map 117

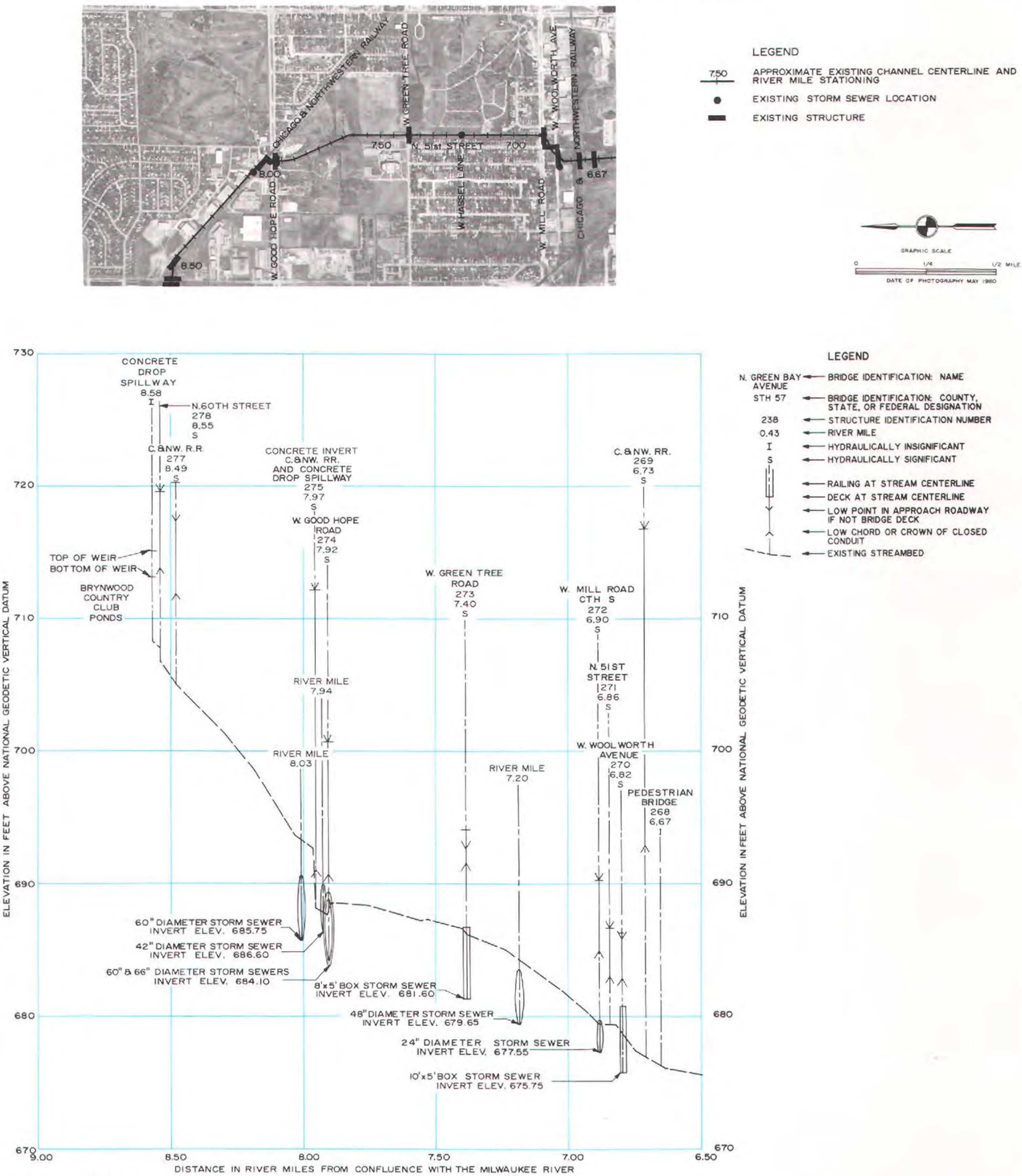
AREAS WITH REPORTED FLOODING AND DRAINAGE PROBLEMS IN THE LINCOLN CREEK SUBWATERSHED: 1960-1986



Source: SEWRPC.

Figure 48

PROFILE ILLUSTRATING THE EXISTING STORM SEWER OUTLETS IN A PORTION OF THE UPPER LINCOLN CREEK SUBWATERSHED



Source: SEWRPC.

from the City Engineer of the City of Milwaukee, and for a period of up to 15 years at eight crest-stage gage locations on the channel, the gages being maintained by the Milwaukee Metropolitan Sewerage District.

In the preparation of the original Lincoln Creek flood control plan, the rainfall relationships were based on the 64-year period of record extending from 1903 through 1966. As noted in Chapter III, the Commission in 1987 revised the point rainfall depth-duration-frequency relationships to incorporate 20 more years of rainfall data. This was done to incorporate certain recent and unusually intensive rainfall events into these relationships. The revised point rainfall depth-duration-frequency relationships, however, were not significantly different from those relationships previously derived from 64 years of record. Moreover, a review of the flood discharges indicated that use of the new relationships would have no significant impacts on the 100-year recurrence interval flood discharges throughout the subwatershed. Accordingly, the development of revised design flood discharges for Lincoln Creek was not considered warranted, and the previously developed flood flows and stages for Lincoln Creek were utilized in the preparation of the refined drainage and flood control system plan for the subwatershed.

The estimated peak flood discharges under existing and year 2000 planned land use conditions and existing channel conditions are set forth in Table 70. Flood stage profiles were determined for the 10-, 50-, and 100-year recurrence interval runoff events under planned land use and existing channel conditions. These profiles, which encompass the full 8.6-mile-long reach of Lincoln Creek studied, constitute a graphic representation of the flood stages along Lincoln Creek under the specified recurrence interval flood discharges, and under planned land use and existing channel conditions. In addition to providing an overall representation of flood stages relative to familiar points of reference such as the channel bottom and bridge deck surfaces, the profiles, because they are continuous, permit the determination of flood stages at any point along the stream channel. The flood profiles are shown in Figure 49. The extent of the 100-year recurrence interval floodplain under planned land use conditions is shown on Map 118.

Alternative Flood Control
Systems for Upper Lincoln Creek

In the preparation of the original flood control plan for the Lincoln Creek subwatershed, five alternative flood control systems were considered for alleviating the flood damage problem along Upper Lincoln Creek: Alternative System 1—no action; Alternative System 2—limited channelization; Alternative System 3—floodwater storage; Alternative System 4—diking; and Alternative System 5—structure floodproofing, elevation, and removal.

Each alternative system is described briefly below. The economic benefits and costs attendant to each alternative are provided in Table 71. More detailed descriptions of these five alternatives are set forth in SEWRPC Community Assistance Planning Report No. 13.

Alternative System 1—No Action: One alternative course of action always available is to do nothing—that is, to recognize the inevitability of extensive flooding but to decide not to mount a collective, coordinated program to abate the flood damages. Given the expressed public concern, it is highly unlikely, however, that a "no action" course should, or indeed could, be followed with respect to Upper Lincoln Creek. Therefore, this alternative, although technically feasible, is probably not practical.

Alternative System 2—Limited Channelization: This alternative system for the resolution of the flood problems along Upper Lincoln Creek consists of about 1.7 miles of channel improvements, including channel deepening, cleaning, and debrushing, as shown on Map 119. This alternative system also includes construction of approximately 80 feet of earthen dikes, removal of a concrete drop spillway, and replacement of a railway crossing culvert, also as shown on Map 119. Implementation of this alternative would essentially eliminate all damages from floods up to and including the 100-year recurrence interval event.

Alternative System 3—Floodwater Storage: The storage alternative for Upper Lincoln Creek would provide for the construction of two detention reservoirs, one having a storage capacity of 84 acre-feet and the other 48 acre-feet. In addition to the proposed storage, cleaning and debrushing or other actions which would result in improved hydraulic efficiency would be

Table 70

FLOOD DISCHARGES FOR LINCOLN CREEK FOR EXISTING AND YEAR 2000 LAND USE AND EXISTING CHANNEL CONDITIONS

Location	River Mile	Peak Flood Discharge (cfs)					
		Existing Land Use, Existing Storage, and Existing Channel Conditions			Planned Land Use, Existing Storage, and Existing Channel Conditions		
		10-Year	50-Year	100-Year	10-Year	50-Year	100-Year
Mouth at Milwaukee River	0.00	5,310	7,350	7,980	5,410	7,370	7,970
N. Green Bay Avenue	0.43	5,310	7,350	7,980	5,410	7,370	7,970
W. Villard Avenue	0.81	4,640	6,120	6,510	4,740	6,120	6,510
Pedestrian Bridge	0.93	4,640	6,120	6,510	4,740	6,120	6,510
N. Teutonia Avenue	1.30	4,640	6,120	6,510	4,740	6,120	6,510
W. Cameron Avenue	1.53	4,480	5,840	6,160	4,580	5,840	6,160
Soo Line Railroad	1.65	4,480	5,840	6,160	4,580	5,840	6,160
W. Hampton Avenue	1.73	4,480	5,840	6,160	4,580	5,840	6,160
N. 32nd Street	1.90	4,480	5,840	6,160	4,580	5,840	6,160
Soo Line Railroad	2.01	4,480	5,840	6,160	4,580	5,840	6,160
Glendale Avenue	2.20	4,480	5,840	6,160	4,580	5,840	6,160
N. 35th Street	2.52	3,640	4,510	4,600	3,730	4,530	4,600
N. 37th Street	2.64	- -	- -	- -	- -	- -	- -
Downstream Side	- -	3,640	4,510	4,600	3,730	4,530	4,600
Upstream Side	- -	3,730	4,900	5,160	3,880	4,960	5,240
N. Sherman Boulevard	3.03	- -	- -	- -	- -	- -	- -
Downstream Side	- -	3,720	4,730	4,990	3,870	4,790	5,060
Upstream Side	- -	4,500	7,070	8,020	4,810	7,440	8,480
N. 51st Street	3.59	3,670	5,860	6,760	4,020	6,290	7,340
Pedestrian Bridge	3.80	3,670	5,860	6,760	4,020	6,290	7,340
N. 58th Street (extended)	4.16	3,670	5,860	6,760	4,020	6,290	7,340
N. 60th Street	4.24	2,840	4,570	5,290	3,190	5,000	5,860
W. Hampton Avenue	4.41	2,840	4,570	5,290	3,190	5,000	5,860
Pedestrian Bridge	4.56	2,150	3,480	4,040	2,490	3,910	4,590
W. Villard Avenue	4.92	830	1,400	1,680	1,130	1,820	2,160
N. 60th Street	5.37	830	1,400	1,680	1,130	1,820	2,160
W. Silver Spring Drive	5.65	- -	- -	- -	- -	- -	- -
Downstream Side	- -	830	1,400	1,680	1,130	1,820	2,160
Upstream Side	- -	470	710	790	530	770	840
Drop Structure	5.79	470	710	790	530	770	840
U. S. Army Bridge	6.06	440	670	740	500	720	780
Wisconsin & Southern Railroad	6.28	400	600	660	440	640	690
Havenwoods Bridge	6.29	400	600	660	440	640	690
Chicago & North Western Railway	6.73	- -	- -	- -	- -	- -	- -
Downstream Side	- -	420	610	660	460	640	700
Upstream Side	- -	610	1,070	1,290	800	1,370	1,640
W. Woolworth Avenue	6.82	490	870	1,040	660	1,110	1,330
N. 51st Street	6.86	490	870	1,040	660	1,110	1,330
W. Mill Road	6.90	490	870	1,040	660	1,110	1,330
W. Greentree Road	7.40	- -	- -	- -	- -	- -	- -
Downstream Side	- -	370	660	790	510	850	1,020
Upstream Side	- -	240	350	390	260	380	400
W. Good Hope Road (structure outlet)	7.92	320	500	560	340	540	630
Chicago & North Western Railway (structure inlet)	7.97	180	250	280	210	290	310
Chicago & North Western Railway	8.49	180	250	280	210	290	310
N. 60th Street	8.55	- -	- -	- -	- -	- -	- -
Downstream Side	- -	180	250	280	210	290	310
Upstream Side	- -	260	470	550	350	590	700

Source: SEWRPC.

Table 71

COST ESTIMATES FOR FLOOD CONTROL ALTERNATIVES FOR THE LINCOLN CREEK SUBWATERSHED

Upper Lincoln Creek, Interest Rate = 6 Percent, 50-Year Period of Economic Analysis										
	Costs					Benefit-Cost Analysis				
		Annual								
Alternative	Capital	Amortized Capital	Operation and Maintenance	Other	Total	Annual Benefits	Annual Benefits Minus Annual Costs	Benefit-Cost Ratio	Economic Ratio Greater than One
1. No Action	$ --	$ --	$ --	$ 32,300	$ 32,300	$ --	$ -32,300	--	No
2. Limited Channelization	329,600	20,800	500	--	21,300	32,300	11,000	1.52	Yes
3. Floodwater Storage	523,000	32,900	1,100	--	34,000	32,300	-1,700	0.95	No
4. Diking	404,000	25,500	700	--	26,200	32,300	6,100	1.23	Yes
5. Structure Floodproofing, Elevation, and Removal	407,000	25,800	--	--	25,800	32,300	6,500	1.25	Yes

Lower Lincoln Creek, Interest Rate = 6 Percent, 50-Year Period of Economic Analysis									
	Costs					Benefit-Cost Analysis			
		Annual							
Alternative	Capital	Amortized Capital	Operation and Maintenance	Other	Total	Annual Benefits	Annual Benefits Minus Annual Costs	Benefit-Cost Ratio	Economic Ratio Greater than One
1. No Action	$ --	$ --	$ --	$617,000	$ 617,000	$ --	$-617,000	--	No
2. Major Channelization	9,591,600	604,000	6,000	--	610,000	617,000	7,000	1.01	Yes
3. Diking and Pumping	12,115,600	763,000	14,000	--	777,000	617,000	-160,000	0.79	No
4. Structure Floodproofing, Elevation, and Removal	20,229,000	1,283,000	--	--	1,283,000	617,000	-666,000	0.48	No

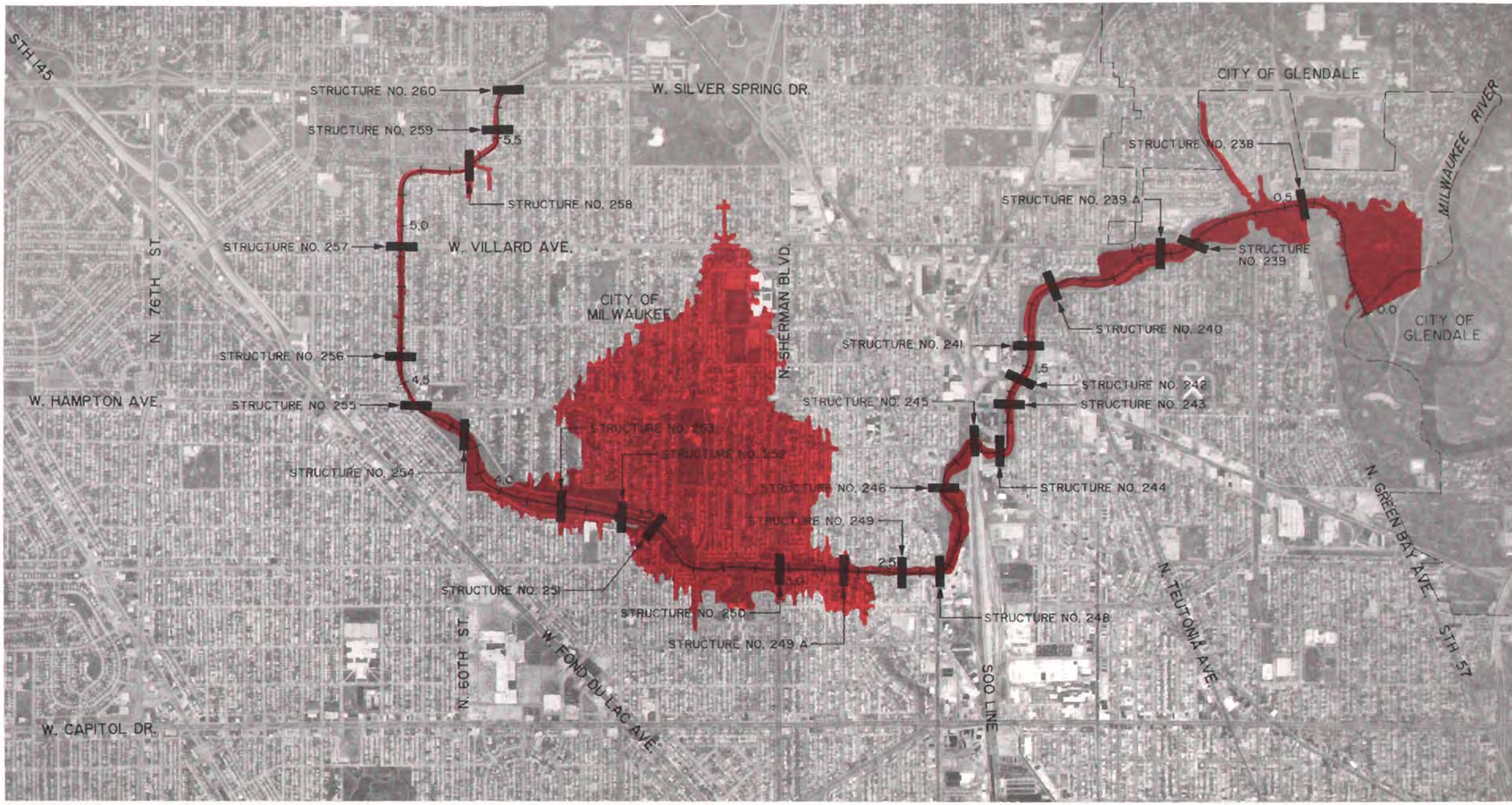
Source: SEWRPC.

required for 1.57 miles of channel. This alternative would also require the replacement of one railroad crossing culvert and the floodproofing of seven structures. These improvements are shown on Map 120. Implementation of this alternative would essentially eliminate all damages from floods up to and including the 100-year recurrence interval event.

One of the storage reservoirs would be located on a 16-acre site located between W. Good Hope Road and W. Green Tree Road. The reservoir dam would be constructed across the existing channel about 50 feet north of W. Green Tree Road, and would have an average height of about seven feet. The structure would be an earthen dam with an outlet spillway, which would consist of one four-foot-diameter concrete pipe. A levee would be constructed along the eastern and northern boundaries of the Daniel Webster Junior High School property west of Lincoln Creek. The levee would extend from the dam upstream approximately 800 feet, and then westward an additional 700 feet to contain floodwaters in the reservoir without flooding the school property. The reservoir would have a maximum storage capacity of 84 acre-feet, and would serve to reduce the 100-year recurrence interval flood below W. Green Tree Road from 1,020 cubic feet per second (cfs) to 900 cfs. The second storage system would involve the existing 11 ponds on the Brynwood Country Club grounds, located immediately west of N. 60th Street. The ponds and adjacent floodlands would remain in their present condition and use. A new earthen dam and control spillway would be constructed at the outlet of the lowest pond to more effectively reduce flood discharges from the series of 11 ponds and increase floodwater storage. The outlet spillway would consist of a four-foot-diameter concrete pipe. This series of reservoirs would have a storage capacity of 48 acre-feet, and would serve to reduce the 100-year recurrence interval flood below N. 60th Street

Map 118

100-YEAR RECURRENCE INTERVAL FLOODPLAIN FOR LINCOLN CREEK UNDER YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

2.5 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR LINCOLN CREEK IS SHOWN IN APPENDIX H

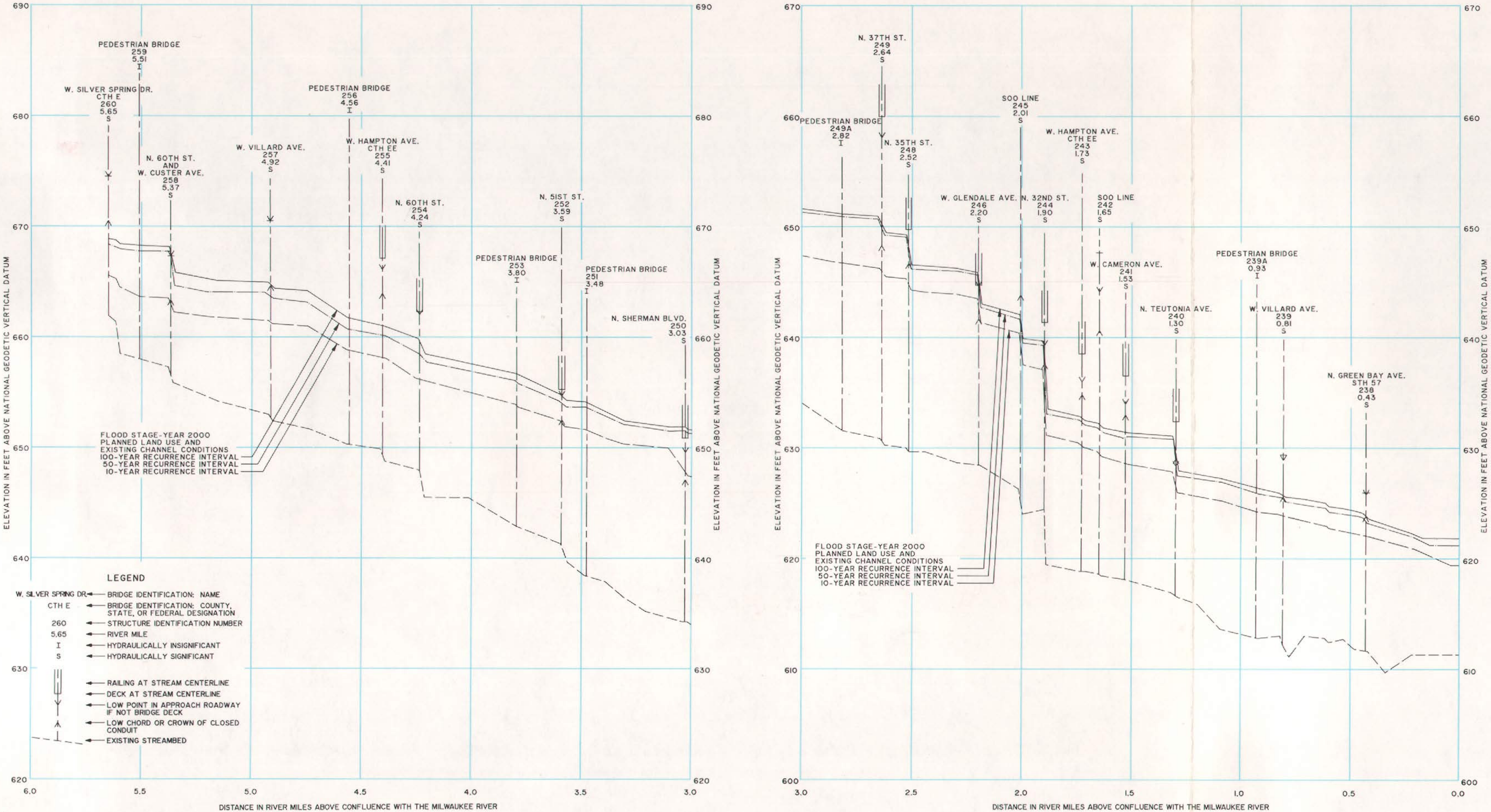
GRAPHIC SCALE

0 1/2 1 MILE

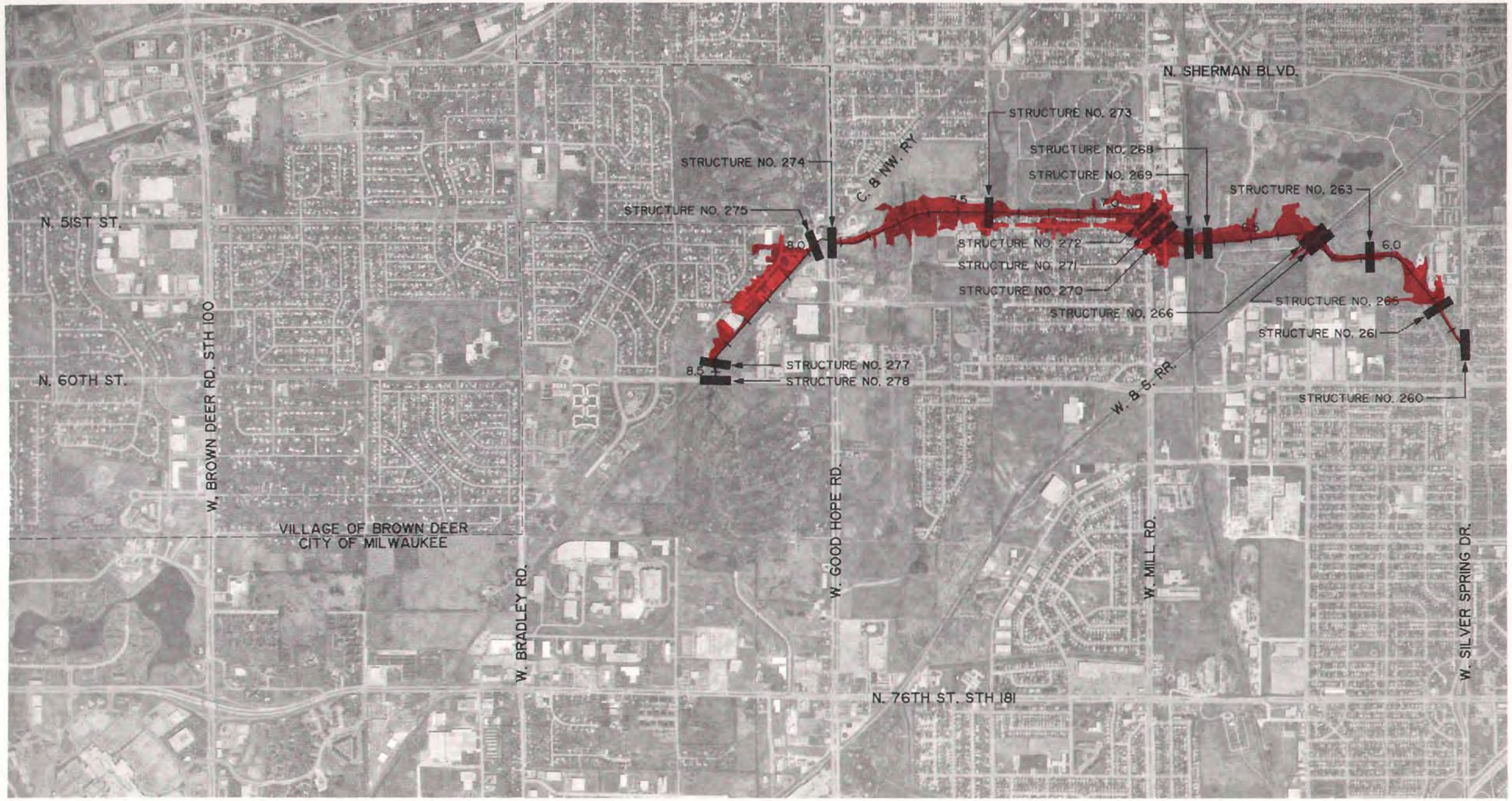
DATE OF PHOTOGRAPHY: APRIL 1986

Figure 49

FLOOD STAGE AND STREAMBED PROFILE FOR LINCOLN CREEK



Map 118 (continued)



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

7.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR LINCOLN CREEK IS SHOWN IN APPENDIX H

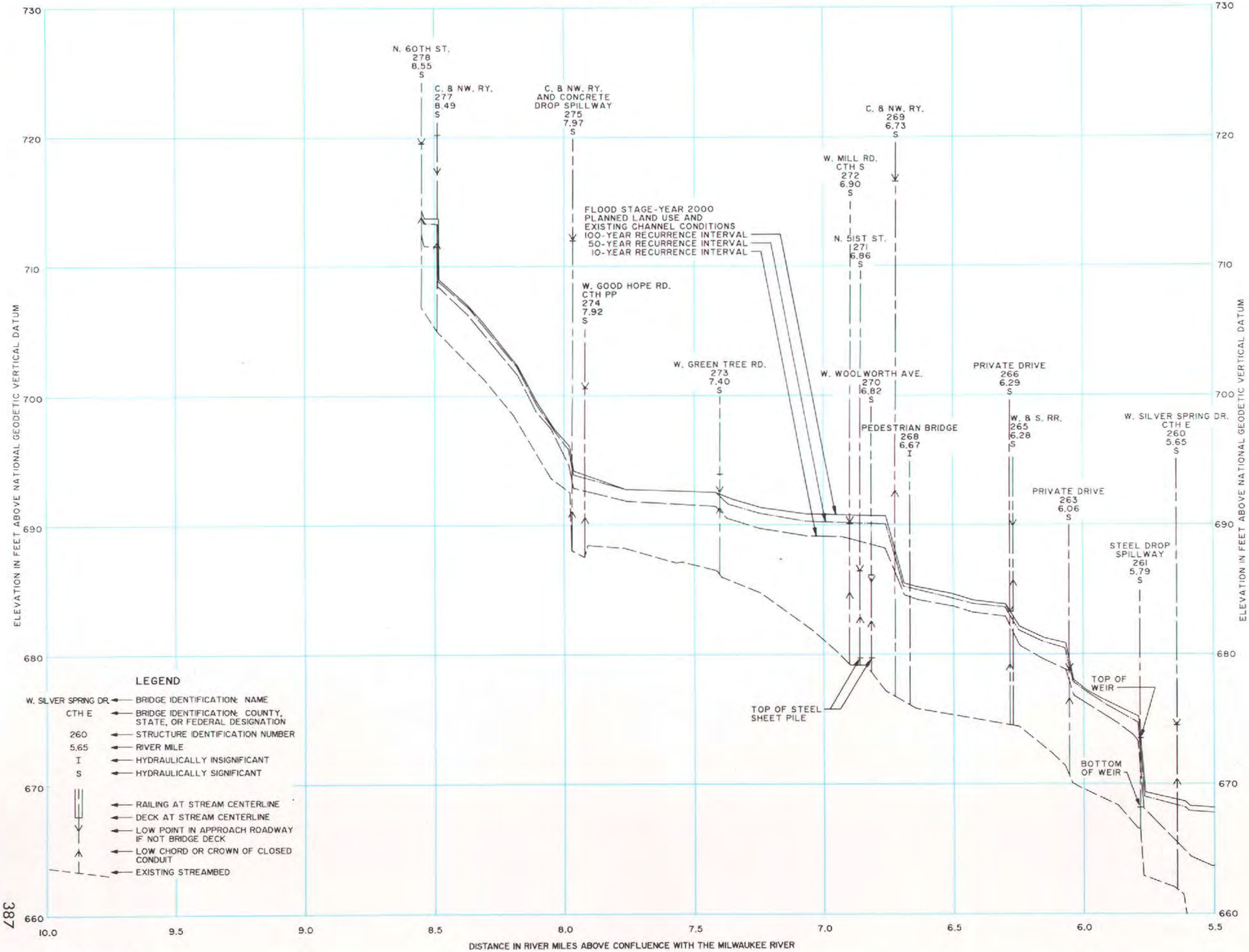
GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Source: SEWRPC.

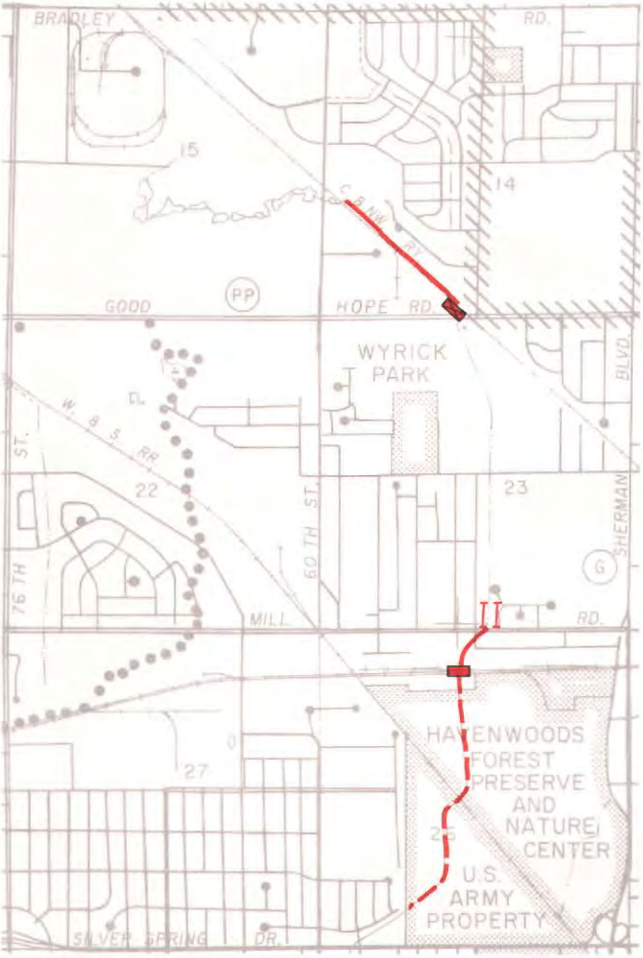
Figure 49 (continued)



Source: SEWRPC.

Map 119

ALTERNATIVE PLAN 2: LIMITED CHANNELIZATION IN THE UPPER LINCOLN CREEK SUBWATERSHED



LEGEND

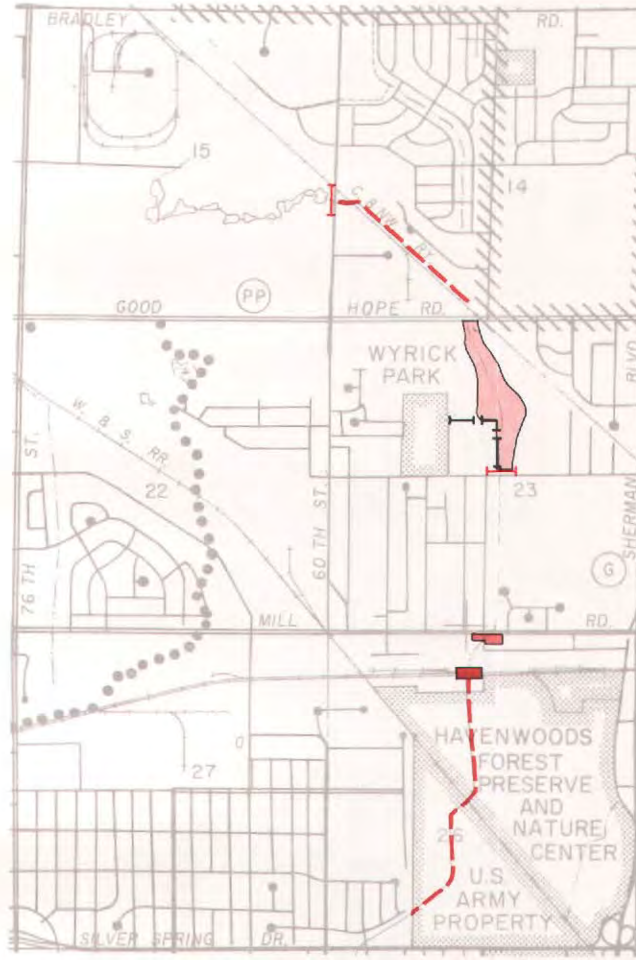
- PROPOSED CHANNEL DEEPENING
- PROPOSED CHANNEL CLEANING AND DEBRUSHING
- PROPOSED EARTHEN DIKE
- PROPOSED REMOVAL OF CONCRETE DROP SPILLWAY
- PROPOSED CULVERT REPLACEMENT

GRAPHIC SCALE 0 1000 2000 3000 FEET

Source: SEWRPC.

Map 120

ALTERNATIVE PLAN 3: FLOOD STORAGE IN THE UPPER LINCOLN CREEK SUBWATERSHED



LEGEND

- PROPOSED CHANNEL CLEANING AND DEBRUSHING
- PROPOSED STRUCTURE FLOODPROOFING
- PROPOSED EARTHEN DAM AND SPILLWAY
- PROPOSED LEVEE
- PROPOSED DETENTION RESERVOIR
- PROPOSED CULVERT REPLACEMENT

GRAPHIC SCALE 0 1000 2000 3000 FEET

Source: SEWRPC.

from 310 cfs to 190 cfs. It should be noted that the existing structures at W. Green Tree Road and N. 60th Street are hydrologically significant and reduce flows significantly.

As part of the analyses conducted under this alternative, an analysis was made of the other potential floodwater storage sites, such as a site in Havenwoods immediately upstream of the Wisconsin & Southern Railroad. The Wisconsin & Southern structure results in a moderate amount of floodwater storage because of its relatively small hydraulic capacity. No significant flood control benefit would be realized by providing additional floodwater storage at this site because flood damages between the site and

W. Silver Spring Drive are minor for both existing and future land use conditions, and flood flows in the heavily urbanized reach downstream of W. Silver Spring Drive would not be significantly reduced by the provision of additional floodwater storage at the Havenwoods site.

Another storage alternative considered was the use of decentralized—or off-stream, onsite—detention and retention facilities. Under this alternative, a large number of relatively small detention or retention basins would be developed throughout the subwatershed. This alternative was not considered further, since the Lincoln Creek subwatershed is almost fully developed, with much of the development being at densities which would make the cost of retrofitting with storage facilities prohibitively high, as well as impractical.

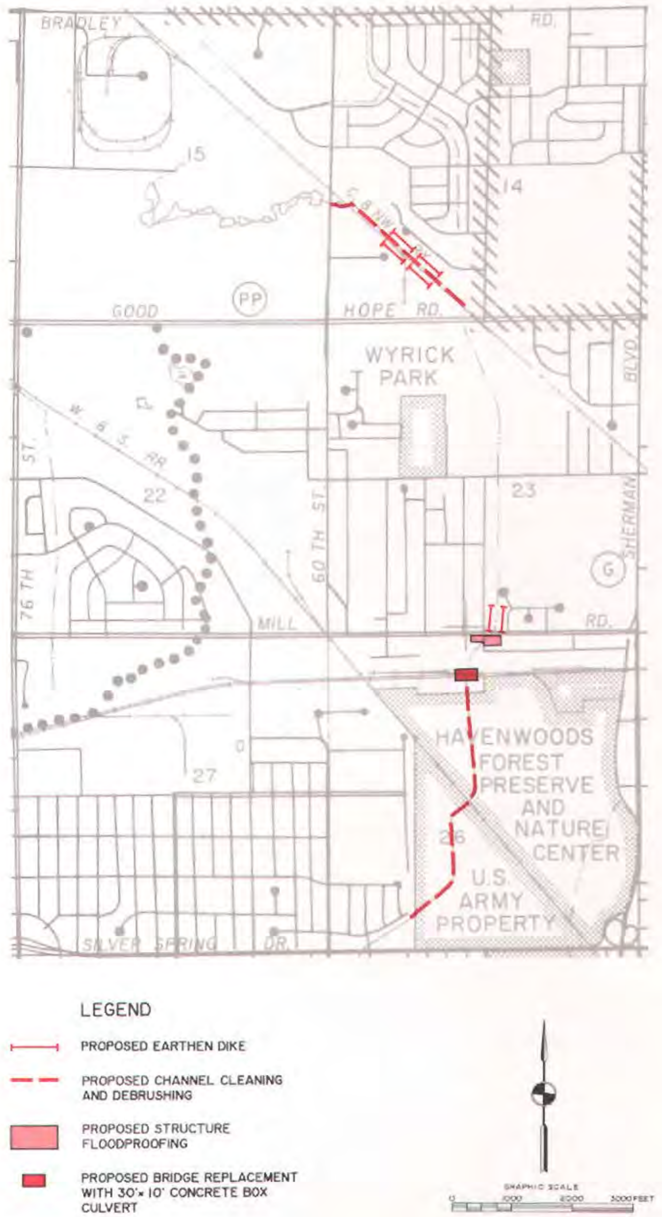
Alternative System 4—Diking: In the diking alternative, 1,200 feet of earthen dike would be constructed along both sides of 0.22 mile of channel, as shown on Map 121, to alleviate flood damages. It would also be necessary to clean out and debrush 1.57 miles of the channel, also as shown on Map 121, to improve the hydraulic efficiency of the existing channel. This alternative would also require the replacement of one railroad crossing culvert and the floodproofing of seven structures. Implementation of this alternative would essentially eliminate all damages attendant to floods up to and including the 100-year recurrence interval event.

Alternative System 5—Structure Floodproofing, Elevation, and Removal: A structure floodproofing, elevation, and removal alternative flood control system was analyzed to determine if such a structure-by-structure approach would be a technically feasible and economically viable solution to the flood problem along Upper Lincoln Creek. The analysis indicated that 14 structures would have to be elevated and 11 structures would have to be floodproofed. No structures would have to be removed under this alternative. Implementation of this alternative would essentially eliminate all damages from floods up to and including the 100-year recurrence interval event.

The previously described flood control alternatives for Upper Lincoln Creek were designed to resolve overload flooding caused directly by high water levels in the Lincoln Creek channel, and

Map 121

ALTERNATIVE PLAN 4: LOCATION OF PROPOSED DIKES IN THE UPPER LINCOLN CREEK SUBWATERSHED



Source: SEWRPC.

were not intended to address the drainage problems attendant to the restricted capacity of the storm sewer outlets along Upper Lincoln Creek. A separate analysis, therefore, was conducted to evaluate alternative means of ameliorating the restricted storm sewer outlet condition. Two alternative systems were evaluated in addition to the "no action" alternative—a channel deepening alternative and a parallel storm sewer alternative.

Storm Sewer Outlet Relief Alternative 1—No Action: One alternative course of action to consider to deal with the storm sewer outlet problem along Upper Lincoln Creek is to do nothing—that is, to allow the situation to continue in its present state. Because the storm sewer outlets are partially restricted, solids buildup in the sewers may be expected to occur, thereby further restricting the capacity. This situation will result in increased maintenance requirements and the potential for ponding and sanitary sewer backups because of the restricted storm sewer capacity. The storm sewers concerned were designed on the premise that the channel would ultimately be lowered, providing a free outfall. It is accordingly unlikely that a "no action" course can be followed indefinitely, and therefore this alternative is regarded as unacceptable by the City Engineer of the City of Milwaukee.

Storm Sewer Outlet Relief Alternative 2—Channel Deepening: The channel deepening alternative for the portion of Upper Lincoln Creek into which storm sewers discharge would consist of lowering and reconstructing the channel along a reach of about 1.8 miles, as shown in Figure 50. In addition to the channel modification, four bridges would have to be replaced, two bridge openings would have to be cleaned out, and a drop structure would be removed. These improvements would provide free outfalls for all the storm sewer outlets concerned.

Storm Sewer Outlet Relief Alternative 3—Parallel Storm Sewer: Under the parallel storm sewer alternative, approximately 1.23 miles of storm sewer would be laid parallel to the Lincoln Creek channel, as shown in Figure 51, at sufficient depth so that the invert of the new "intercepting" sewer would be below the inverts of the storm sewer outlets. The intercepting sewer would be of sufficient capacity to accommodate runoff from rainfall events having a recurrence interval of about once a year. In addition, a 0.15-mile portion of the Lincoln Creek channel, as shown in Figure 51, would be deepened to accommodate the elevation of another storm sewer outlet.

Alternative Flood Control Systems for Lower Lincoln Creek

In the preparation of the original flood control plan for the Lincoln Creek subwatershed, four alternative flood control systems were considered for alleviating flood damages along Lower Lincoln Creek: 1) Alternative System 1—no action; 2) Alternative System 2—major channelization; 3) Alternative System 3—diking and pumping; and 4) Alternative System 4—structure floodproofing, elevation, and removal.

Each alternative system is described briefly below. The economic benefits and costs attendant to each alternative are provided in Table 71. More detailed descriptions of these four alternatives are set forth in SEWRPC Community Assistance Planning Report No. 13.

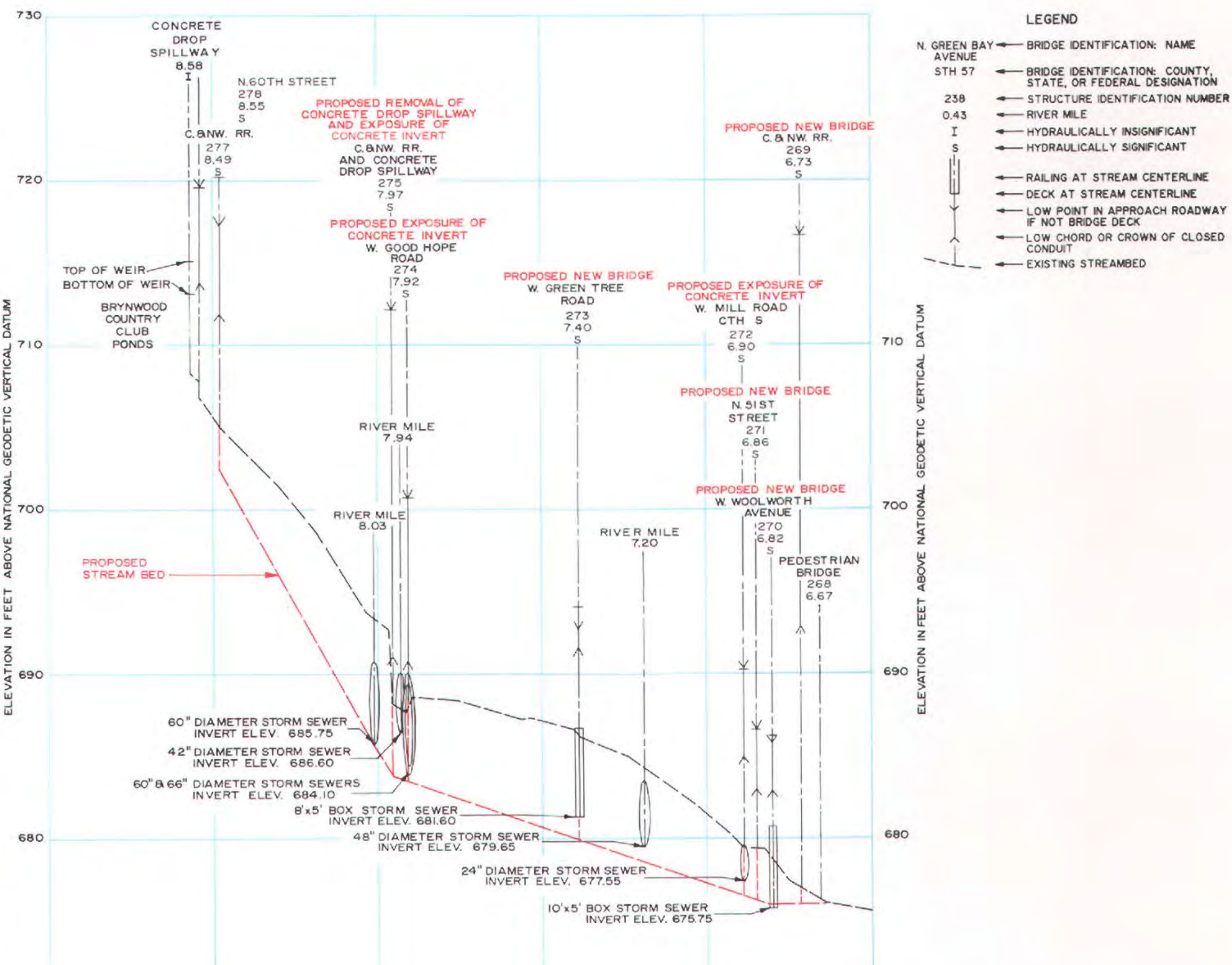
Alternative System 1—No Action: As already noted, one alternative course of action is to do nothing—that is, to recognize the inevitability of extensive flooding but to decide not to mount a collective, coordinated program to abate the flood damages. It is highly unlikely, however, that a no action course should, or indeed could, be followed along Lower Lincoln Creek, since the flood damages are severe and public sentiment demands action. Therefore, this alternative, although technically feasible, is probably not practical.

Alternative System 2—Major Channelization: The major channelization alternative for Lower Lincoln Creek would consist of 2.51 miles of major channel reconstruction and improvement, as shown on Map 122. This alternative would also involve the installation of 8,400 feet of earthen dike and 800 feet of concrete floodwall, and the construction of four permanent stormwater pumping stations and backwater gates. Also, as part of the channel improvements, it would be necessary to modify or replace 14 bridges over Lower Lincoln Creek. Implementation of this alternative would essentially eliminate all damages from floods up to and including the 100-year recurrence interval event.

Alternative System 3—Diking and Pumping: Under this alternative, the following diking and supplemental improvements would be required to alleviate flooding damages: 1) 20,700 feet of earthen dike, 2) 12,200 feet of concrete floodwall, and 3) 16 permanent stormwater pumping stations and backwater gates, all as shown on

Figure 50

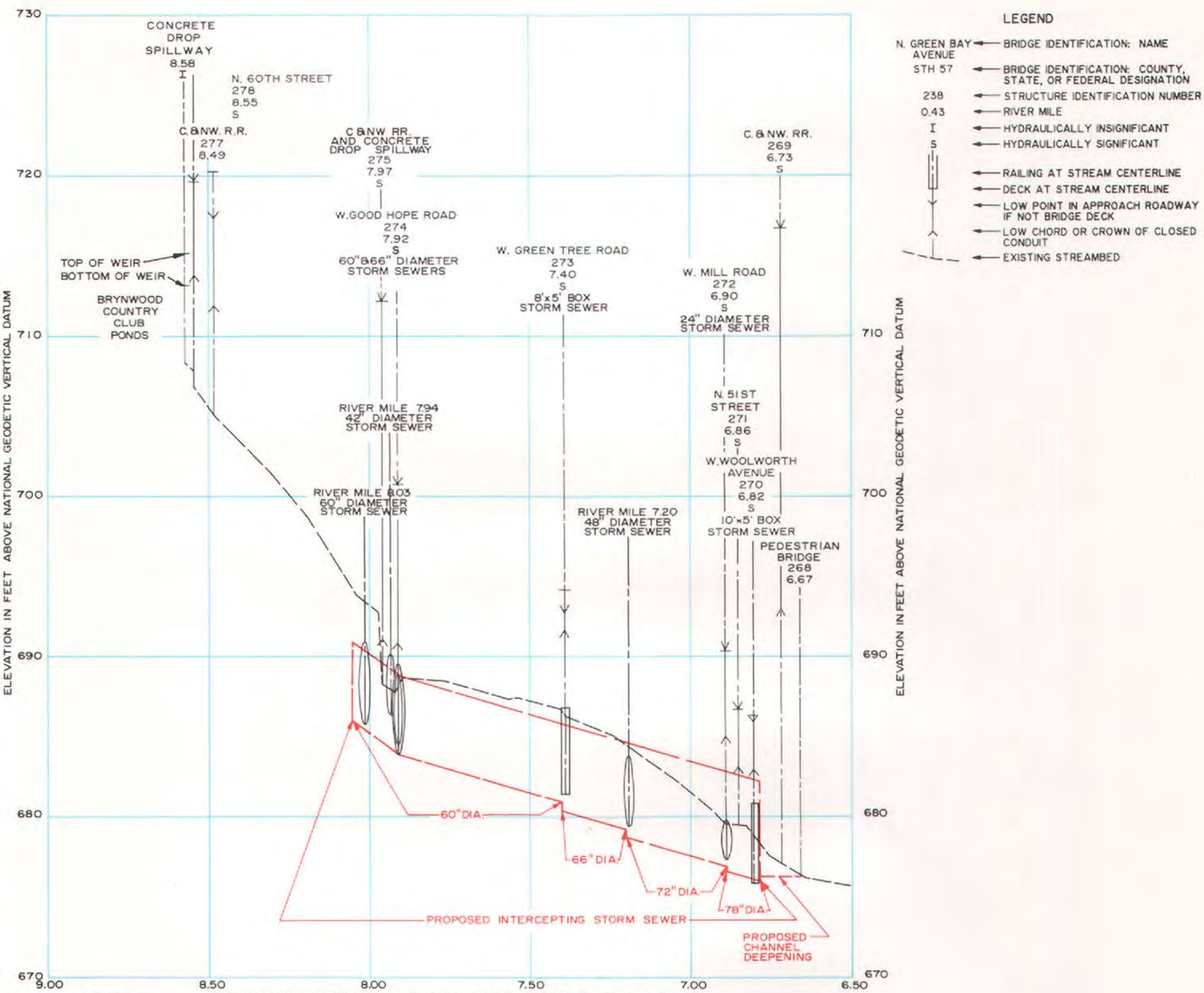
PROFILE ILLUSTRATING CHANNEL DEEPENING ALTERNATIVE FOR A PORTION OF THE UPPER LINCOLN CREEK SUBWATERSHED



Source: SEWRPC.

Figure 51

PROFILE ILLUSTRATING PARALLEL STORM SEWER ALTERNATIVE FOR A PORTION OF THE UPPER LINCOLN CREEK SUBWATERSHED



Source: SEWRPC.

Map 122

ALTERNATIVE PLAN 2: MAJOR CHANNELIZATION IN THE LOWER LINCOLN CREEK SUBWATERSHED

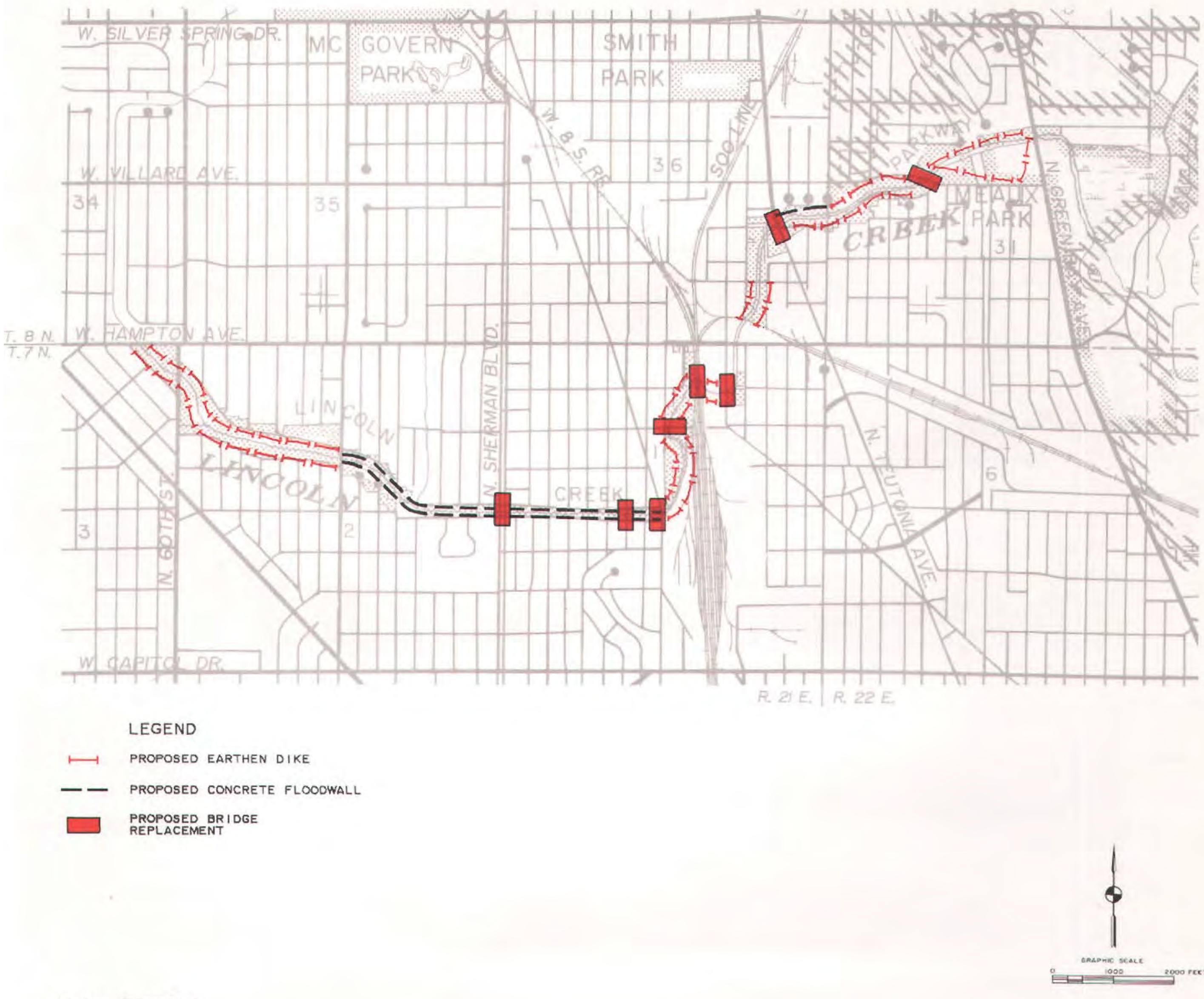
Source: SEWRPC.

Map 123. In addition, it would be necessary to replace eight bridges over Lower Lincoln Creek. Implementation of this alternative would essentially eliminate all damages from floods up to and including the 100-year recurrence interval event.

Alternative System 4—Structure Floodproofing, Elevation, and Removal: A structure floodproofing, elevation, and removal alternative flood control plan was analyzed to determine if such a structure-by-structure approach would be a technically feasible and economically viable solution to the flood problem along Lower Lincoln Creek. The analysis indicated that 825 structures would have to be elevated and 745 structures would have to be floodproofed. No structures would have to be removed under this alternative. Implementation of this alternative would essentially eliminate all damages from floods up to and including the 100-year recurrence interval event.

Map 123

ALTERNATIVE PLAN 3: LOCATION OF PROPOSED DIKES IN THE LOWER LINCOLN CREEK SUBWATERSHED



Source: SEWRPC.

As in Upper Lincoln Creek, there were no available sites in Lower Lincoln Creek for centralized storage facilities at locations where significant reductions in flood flows could be achieved. In addition, decentralized onsite storage facilities were judged to be impractical for use in the tributary drainage area of Lower Lincoln Creek owing to the extent and type of the existing development.

Further Consideration of Flood Control Measures for Lower Lincoln Creek Downstream of N. Teutonia Avenue

Additional analyses were conducted of the dike and floodwall element of the flood control alternatives for Lincoln Creek downstream of N. Teutonia Avenue. These analyses were conducted in order to incorporate the information provided by the new large-scale topographic maps obtained under this study, as well as to provide additional detail relating to the appurtenant stormwater pumping facilities required as requested by the Technical Advisory Committee. The new topographic maps provided more definitive information on the drainage patterns of the lands lying adjacent to Lincoln Creek, as well as on the ground elevations along the creek. This information is essential to determining the extent of potential flood damages along Lincoln Creek; the required height of any dikes or floodwalls; and the number, size, and location of appurtenant stormwater pumping stations.

In addition to the dike and floodwall element, two additional channel modification elements for the stream reach downstream of N. Teutonia Avenue were analyzed. Thus, three refined subalternatives for the lower reaches of Lincoln Creek downstream of Teutonia Avenue were analyzed. The results of these analyses could be readily integrated with the results of the alternative analyses for the remainder of the Lincoln Creek drainage system, since none of the three subalternatives significantly affected the reaches of Lincoln Creek upstream of N. Teutonia Avenue.

<u>Refined Dike and Floodwall Flood Control Plan for Lincoln Creek Downstream of N. Teutonia Avenue</u>: The analysis of the dike and floodwall element of the flood control alternatives for Lincoln Creek downstream of N. Teutonia Avenue resulted in the following refinements to that plan element: 1) the addition of about 1,300 lineal feet of dike, and about 650 lineal feet of floodwall along the east side of Lincoln Creek downstream of N. Green Bay Avenue, and the addition of about 1,900 feet of floodwall along Crestwood Creek north of Lincoln Creek; 2) the proposed replacement of the N. Green Bay Avenue bridge; 3) refinements in the heights of the dikes and floodwalls between N. Teutonia Avenue and N. Green Bay Avenue; and 4) refinements in the number, size, and location of appurtenant stormwater pumping stations. These refinements are discussed below.

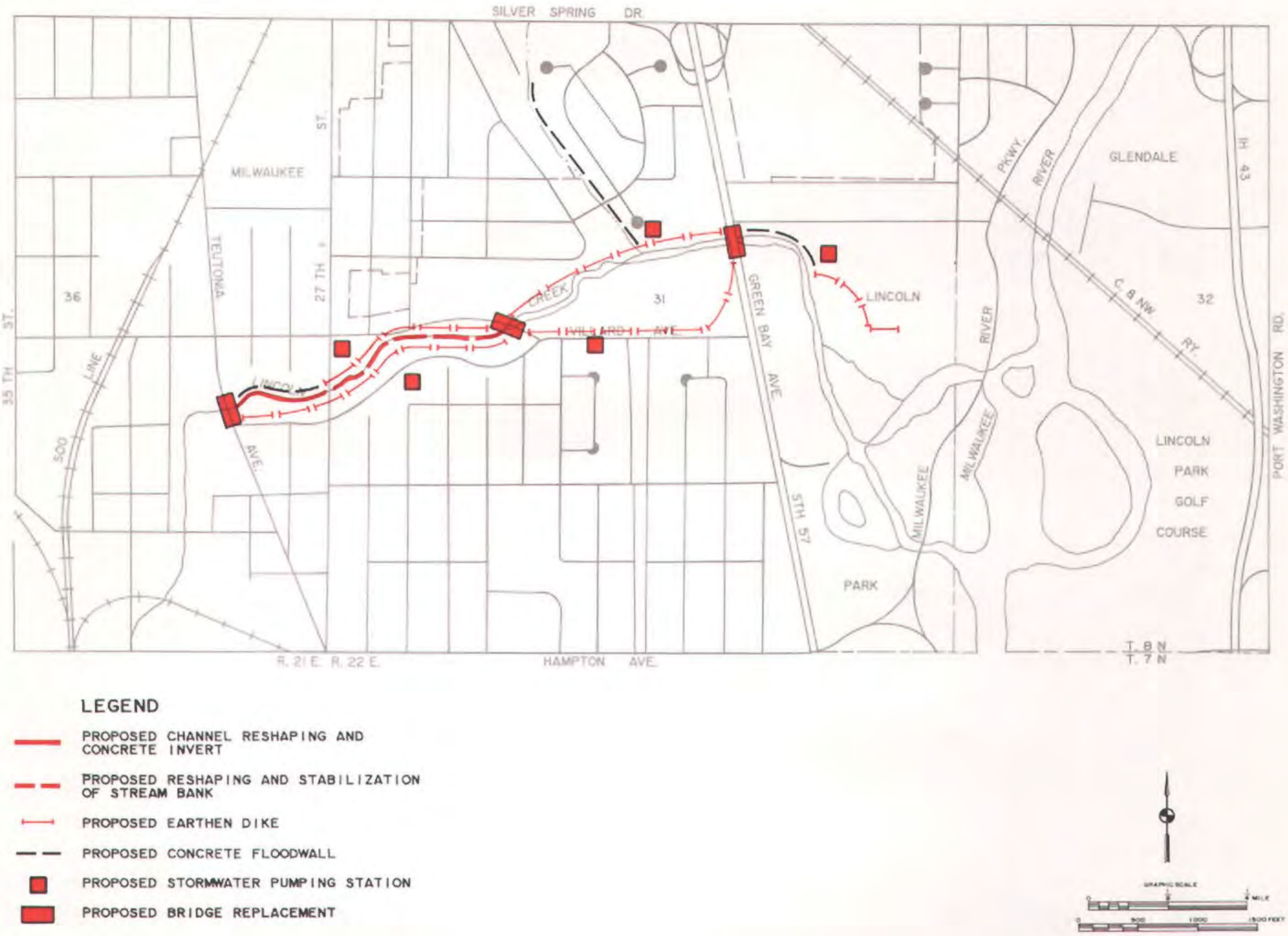
1. The more detailed topographic information made available by the new large-scale topographic maps indicated the potential for flood damages to homes along W. Lawn Avenue east of N. Green Bay Avenue during a 100-year recurrence interval flood under planned land use and channel conditions. In order to protect these homes, an additional 1,300 feet of dike and 650 lineal feet of floodwall are required along the east side of Lincoln Creek, as shown on Map 124. These dikes and floodwalls would be designed to contain the 100-year recurrence interval flood under planned land use and channel conditions with three feet of freeboard. Because of the limited space available between the channel and privately owned lands, a concrete floodwall with an average height of about six feet would be required along the creek for the first 650 feet downstream of N. Green Bay Avenue. An earthen dike with an average total height of about five feet would then extend about 1,300 feet southeasterly through public parklands. A potential for the flooding of homes exists along the east side of Crestwood Creek, a tributary to Lincoln Creek. Protecting these homes would require the construction of about 1,900 lineal feet of concrete floodwall, as shown on Map 124, with an average height of five feet beginning at the confluence with Lincoln Creek and extending to the north along the east bank of Crestwood Creek.

2. The approximately two feet of backwater caused by the bridge at N. Green Bay Avenue under peak design flows may be expected to result in the overtopping of this structure during a 100-year recurrence interval flood under planned land use and channel conditions. The new topographic information available for this area indicates that this may be expected to cause flooding of about 35 structures located along N. Green Bay Avenue. Replacement of this bridge and elevation of the attendant roadway would be required to reduce flood stages and eliminate potential structure flooding in this area.

3. Based on the information provided by the new large-scale topographic maps, the required heights of the dikes and floodwalls between N. Teutonia Avenue and N. Green Bay Avenue were revised as follows: 1) the 800 feet of concrete floodwall along the north side of Lincoln Creek downstream of N. Teutonia Avenue would have an average height of six feet; 2) the 1,800 feet of earthen dike along the north side of Lincoln Creek between N. 27th Street

Map 124

REFINED DIKE AND FLOODWALL FLOOD CONTROL ALTERNATIVE FOR LOWER LINCOLN CREEK DOWNSTREAM OF N. TEUTONIA AVENUE



Source: SEWRPC.

extended and W. Villard Avenue would have an average height of six feet; 3) the 2,000 feet of earthen dike along the north side of Lincoln Creek between W. Villard Avenue and N. Green Bay Avenue would have an average height of six feet; 4) the 2,600 feet of earthen dike along the south side of Lincoln Creek between N. Teutonia Avenue and W. Villard Avenue would have an average height of six feet; and 5) the 2,000 feet of earthen dike along the south side of Lincoln Creek between W. Villard Avenue and N. Green Bay Avenue would have an average height of five feet.

4. The construction of dikes and floodwalls along the Lincoln Creek channel would block the overland flow of stormwater to the channel. It is particularly important that these overland flow routes be able to provide significant drainage relief during those major storm events when the capacity of the storm sewer system is exceeded. Excessive stormwater runoff collecting behind the dikes could result in structural flooding. In addition, the higher flood stages expected along Lower Lincoln Creek and the installation of the backwater gates required for 15 of the storm sewer outfalls

discharging to this stream reach may be expected to cause the tributary storm sewers to surcharge owing to the added head required for these sewers to function, and thus could result in the flooding of low-lying areas located away from the Lincoln Creek channel. The availability of the large-scale topographic maps made it possible to conduct a more detailed analysis of the stormwater drainage patterns along Lincoln Creek in order to identify potential flood problems, and to determine the number, size, and location of stormwater pumping stations needed to relieve the drainage problems that would be created by the construction of the dikes and floodwalls.

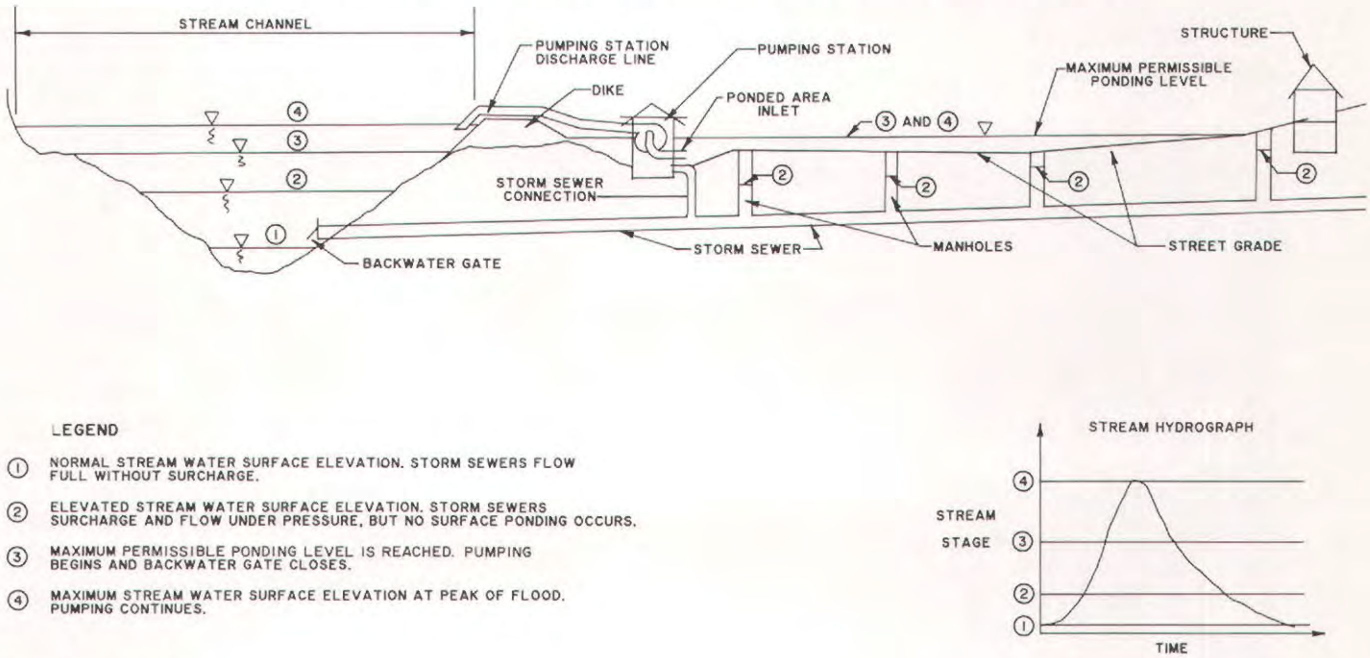
The first step in this analysis was the identification of those low-lying areas along Lincoln Creek where the ponding of excess stormwater and storm sewer surcharging may be expected to occur during major storm events. The drainage area tributary to each of these locations was then delineated on the topographic maps. Runoff hydrographs to each of these locations were then developed using the kinematic wave version of the HEC-1 computer model as described in Chapter III for 100-year recurrence interval storms with durations of 15 minutes, 30 minutes, one hour, and three hours. Generally, the one-hour storm was found to produce the peak rate of discharge for the subbasins analyzed. Consideration was then given to the capacity of the existing storm sewer system, as well as to the timing of the stormwater runoff relative to the streamflow hydrograph for Lincoln Creek, in order to determine the volume of water which may be expected to collect in the low-lying areas concerned. The attendant areas of localized ponding were then delineated on the large-scale topographic maps in order to determine where flooding of structures could be expected to occur. The size and location of stormwater pumping stations that would alleviate this structure flooding were then determined. The capacity of the pumping stations was set so that the stations could handle the flow rates required to limit the volume of excessive runoff to be removed to that necessary to avoid structure flooding. The pumping stations were not sized to alleviate flooding of streets, parking lots, or other open spaces.

A schematic representation of the stormwater pumping station analysis is shown in Figure 52. The analysis determined the discharge capacity required at each storm sewer outlet to Lincoln Creek, assuming the pipe was flowing full, but was not surcharged at either its upstream or downstream end. Pertinent data on storm sewer size, slope, and materials of construction were obtained from the City Engineer and used to calculate the sewer capacities. The full-flow capacity would be available to convey runoff to Lincoln Creek until the water surface of the creek rose to a level at which the storm sewer outlet would be submerged. Once the outlet was submerged, the storm sewer capacity would be reduced and surcharging would occur. However, street flooding would not begin until the hydraulic grade line of the surcharged sewer exceeded the elevations of manhole rims or inlet grates. Surcharging at the first manhole upstream from the outlet would decrease the head and the flow in the upstream storm sewer line, or lines, discharging to that manhole. Thus, the total storm sewer discharge from upstream sewer lines would begin to decrease following outlet submergence. As the upstream storm sewer discharge capacity was reduced, runoff at rates exceeding the reduced capacity would bypass the upstream stormwater inlets and pond in low-lying areas. Such temporary ponding was considered permissible if it did not cause flooding of structures. If the inflow to the ponded area exceeded the discharge capacity of the storm sewer outlet, the excess flow would have to be pumped to prevent structural flooding. Pumping capacity is provided to prevent the ponding elevation in the protected area from reaching the level at which structural flooding would begin. The backwater gates would close when the creek level exceeded the maximum permissible ponding level.

To adequately account for the development of as much available storm sewer outlet discharge capacity as possible prior to reaching the maximum permissible pond-

Figure 52

SCHEMATIC REPRESENTATION OF STORMWATER PUMPING ANALYSIS



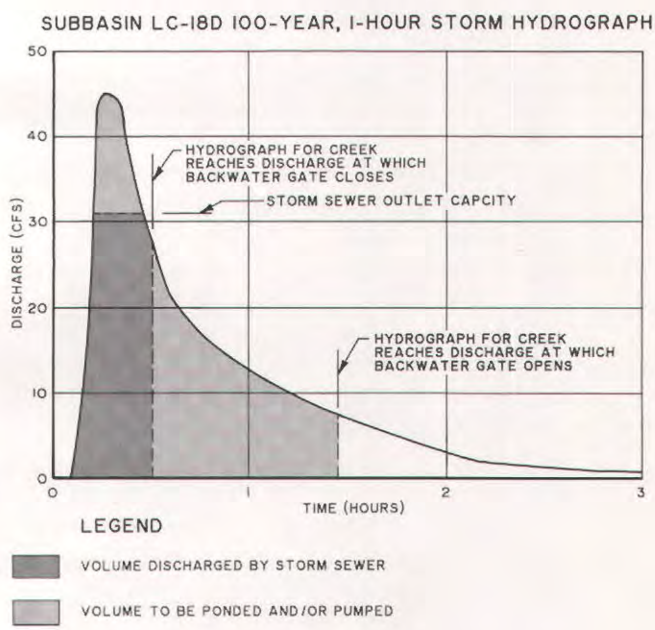
Source: SEWRPC.

ing levels in protected areas, the hydrographs from the protected areas behind the dikes or floodwalls were compared to the Lincoln Creek hydrographs at the storm sewer outlets. An example of one such comparison for subbasin LC18-D is shown in Figure 53. The subbasin hydrograph peaks occurred on the rising limb of the Lincoln Creek hydrograph. A comparison of the relative timing of the flows in each subbasin with those in Lincoln Creek, along with utilization of available storage in streets and parkway areas along the landward side of the proposed dikes, enabled the size and cost of the required pumping stations to be minimized.

Based on the results of the analyses, the refined dike and floodwall alternative for Lincoln Creek downstream of N. Teutonia Avenue, as shown on Map 124, includes the following elements: 1) lining of the channel bottom with concrete from Teutonia Avenue to N. 27th Street extended; 2) the installation of stream bank stabilization measures between N. 27th Street extended and W. Villard Avenue; 3) the construction of about

Figure 53

GRAPHIC REPRESENTATION OF PUMPING STATION ANALYSIS



Source: SEWRPC.

3,350 feet of concrete floodwalls—about 800 feet along the north bank of the creek between Teutonia Avenue and N. 27th Street extended, about 650 feet along the north bank of the creek downstream of N. Green Bay Avenue, and about 1,900 feet along the east bank of Crestwood Creek north of Lincoln Creek; 4) the construction of about 9,700 feet of earthen dikes—about 3,800 feet along the north side of the creek between N. 27th Street extended and N. Green Bay Avenue, about 4,600 feet along the south side of the creek between N. Teutonia Avenue and N. Green Bay Avenue, and about 1,300 feet along the east side of the creek beginning at a point 650 feet downstream of N. Green Bay Avenue; 5) the construction of five stormwater pumping stations with capacities ranging from 35 to 110 cfs; and 6) replacement of the bridges at N. Green Bay Avenue, W. Villard Avenue, and N. Teutonia Avenue.

City of Milwaukee storm sewers are generally designed to accommodate the peak rate of runoff from a five-year recurrence interval storm event. During storms exceeding the design storm, the components of the stormwater drainage system, including the entire street cross-section, convey the runoff. The refined stormwater pumping analysis indicated that there may be certain areas in the Lincoln Creek subwatershed where the major stormwater drainage system is inadequate to prevent structure flooding during a one-hour, 100-year recurrence interval storm event even under existing conditions.

A potential structure flooding problem during a 100-year storm event was identified at St. Michael Hospital, where ponding at a mid-block sag in the grade of N. 25th Street would not be released to Lincoln Creek until shallow flooding of the west end of the hospital, and at about eight nearby residences, occurred. There may be other areas with similar problems. It is accordingly recommended that a local drainage system analysis be conducted by the city staff following the adoption of the recommended stormwater management plan to ascertain that structure flooding problems are not encountered during major storm events.

The total capital cost of the refined dike and floodwall alternative for Lower Lincoln Creek was estimated at $8,991,000 in 1986 dollars. The annual operation and maintenance costs for this portion of the Lincoln Creek flood control plan were estimated at $40,000. A more detailed breakdown of these costs is presented in Table 72.

Major Concrete-Lined Channel Modification Flood Control Alternative for Lincoln Creek Downstream of N. Teutonia Avenue: Additional analyses were conducted of a new concrete-lined channel alternative for Lincoln Creek downstream of N. Teutonia Avenue. These analyses extended downstream of the mouth of Lincoln Creek through the west backwater channel of the Milwaukee River in order to provide an alternative to the refined dike and floodwall alternative. As shown on Map 125, the Milwaukee River at its confluence with Lincoln Creek is divided into three channels—a main channel, and two minor channels on either side of the main channel. It is the west minor channel through which Lincoln Creek drains. The additional analyses indicated that during major storm events, the flow of Lincoln Creek may be expected to divide as it enters this west channel, with a majority of the flow proceeding in a southeasterly direction toward the Milwaukee River main channel, but some of the flow proceeding to the northeast. In addition to Lincoln Creek itself, this refined analysis considered the stream reaches of the Milwaukee River west backwater channel downstream of the mouth of Lincoln Creek to the confluence with the Milwaukee River main channel. The existing channel bottom elevation on the main channel is about 2.5 feet lower than the elevation of the west channel where Lincoln Creek enters it. Consideration of this entire channel system provided an opportunity to develop a refined, improved channel alternative considering all available channel gradient.

Accordingly, a major channel modification alternative which would reduce flood stages by widening, deepening, and lining the Lincoln Creek channel was analyzed for the reach downstream of N. Teutonia Avenue. This alternative is shown on Map 126. As already noted, the analysis conducted for this alternative was carried beyond the confluence of Lincoln Creek with the west channel of the Milwaukee River to the main channel in order to utilize all available channel gradient. By utilizing this additional drop in streambed, the Lincoln Creek channel could be lowered sufficiently to make a channel modification alternative technically feasible for eliminating potential damages from floods up to

Table 72

COST ESTIMATES FOR REFINED FLOOD CONTROL ALTERNATIVES FOR LOWER LINCOLN CREEK DOWNSTREAM OF N. TEUTONIA AVENUE

Alternative	Costs			
		Annual		
	Capital	Amortized Capital[a]	Operation and Maintenance	Total
1. Dikes and Floodwalls				
Bridge Replacement	$2,780,000	$176,000	$ --	$176,000
Dikes and Floodwalls	1,873,000	119,000	10,000	129,000
Concrete Lining	228,000	14,500	--	14,500
Streambank Stabilization	2,851,000	18,100	--	18,100
Stormwater Drainage	621,000	39,400	--	39,400
Pumping Stations	3,204,000	250,000[b]	30,000	280,000
Subtotal	$8,991,000	$617,000	$40,000	$657,000
2. Channelization—Concrete Lining				
Channel Enlargement	$ 618,000	$ 39,200	$ 2,600	$ 41,800
Revegetation	33,000	2,100	--	2,100
Concrete Lining	4,825,000	306,000	--	306,000
Bridge Replacement	2,880,000	183,000	--	183,000
Dike and Floodwall	179,000	11,300	900	12,200
Stormwater Drainage	18,000	1,100	--	1,100
Pumping Stations	150,000	11,700[b]	6,000	17,700
Subtotal	$8,703,000	$554,400	$ 9,500	$563,900
3. Channelization—Turf Lining				
Channel Enlargement	$ 860,000	$ 54,500	$ 2,600	$ 57,100
Revegetation	172,000	10,900	--	10,900
Bridge Replacement	3,060,000	194,000	--	194,000
Concrete Lining	38,000	2,400	--	2,400
Dikes and Floodwalls	372,000	23,600	1,800	25,400
Stormwater Drainage	282,000	17,900	--	17,900
Pumping Stations	714,000	55,800[b]	12,000	67,800
Subtotal	$5,498,000	$359,100	$16,400	$375,500

[a]*Amortized capital cost is based on an interest rate of 6 percent and a project life of 50 years.*

[b]*Amortized capital cost includes the cost of the replacement of pumps after 25 years of operation.*

Source: SEWRPC.

and including the 100-year recurrence interval event under planned land use and channel conditions.

Under this alternative, the streambed would be lowered by up to six feet, with an average depth of excavation of about 2.5 feet between N. Teutonia Avenue and the mouth of Lincoln Creek. The existing channel would be reconstructed with a bottom width of 70 feet and side slopes of one on three. The resulting channel would have an average top width of 160 feet and an average depth of 14 feet. Between the mouth of Lincoln Creek and W. Villard Avenue, the 10-year recurrence interval flood level is at about the same elevation as the top of the bank; therefore, the channel would be fully concrete lined. Between W. Villard Avenue and N. Teutonia Avenue the channel would be concrete lined up to the 10-year recurrence interval flood

Map 125

MILWAUKEE RIVER AT
CONFLUENCE WITH LINCOLN CREEK

Source: SEWRPC.

stage, with the remainder being turf lined. Besides deepening the Lincoln Creek channel, about 1,700 feet of the west channel of the Milwaukee River would need to be dredged in order to maintain an even drop in streambed from the mouth of Lincoln Creek to the confluence with the Milwaukee River main channel. The proposed streambed and high-water profiles for Lincoln Creek under this alternative are shown in Figure 54.

It should be noted that under this alternative, the 100-year recurrence interval flood flows under planned land use and channel conditions would not be confined to the modified channel downstream of W. Villard Avenue. Flood stages between N. Green Bay Avenue and W. Villard Avenue, however, would be sufficiently reduced so as to eliminate potential flood damages along this reach. Downstream of N. Green Bay Avenue there would remain a potential for flood damage to about 10 homes. Eliminating these flood damages would require the construction of about 1,400 feet of dike and floodwall along the east side of Lincoln Creek. About 400 feet of concrete floodwall with an average height of five feet would be required beginning at a point about 250 feet downstream of N. Green Bay Avenue. An additional 1,000 feet of earthen dike with an average height of four feet would be required, ending at a point about 1,300 feet downstream of N. Green Bay Avenue. One stormwater pumping station with a capacity of 10 cfs would be constructed on county parkland downstream of N. Green Bay Avenue. The locations of the dike, floodwall, and pumping station are shown on Map 126.

Also required under this alternative would be the replacement of three street bridges and one pedestrian bridge in order to provide adequate hydraulic capacity to pass flood flows, as well as to accommodate the lower channel invert. The street bridges are located at N. Green Bay Avenue (River Mile 0.43), W. Villard Avenue (River Mile 0.81), and Teutonia Avenue (River Mile 1.30). The pedestrian bridge is located south of St. Michael Hospital at River Mile 0.93.

The total capital cost of this flood control alternative is estimated at $8,703,000, with an annual operation and maintenance cost of about $9,500. A detailed breakdown of these costs is provided in Table 72.

<u>Major Turf-Lined Channel Modification Flood Control Alternative for Lincoln Creek Downstream of N. Teutonia Avenue</u>: A second major channel modification flood control alternative was considered consisting of a turf-lined channel downstream of N. Teutonia Avenue. This alternative is shown on Map 127. Under this alternative, the streambed would be lowered by up to seven feet, with an average depth of excavation of three feet between N. Teutonia Avenue and the mouth of Lincoln Creek. The existing channel would be reconstructed with a bottom width of 80 feet and side slopes of one on three. The resulting channel would have an average top width of 170 feet and an average depth of 15 feet. This channel would be completely turf lined

Map 126

MAJOR CONCRETE-LINED CHANNEL MODIFICATION FLOOD CONTROL ALTERNATIVE FOR LOWER LINCOLN CREEK DOWNSTREAM OF N. TEUTONIA AVENUE

Source: SEWRPC.

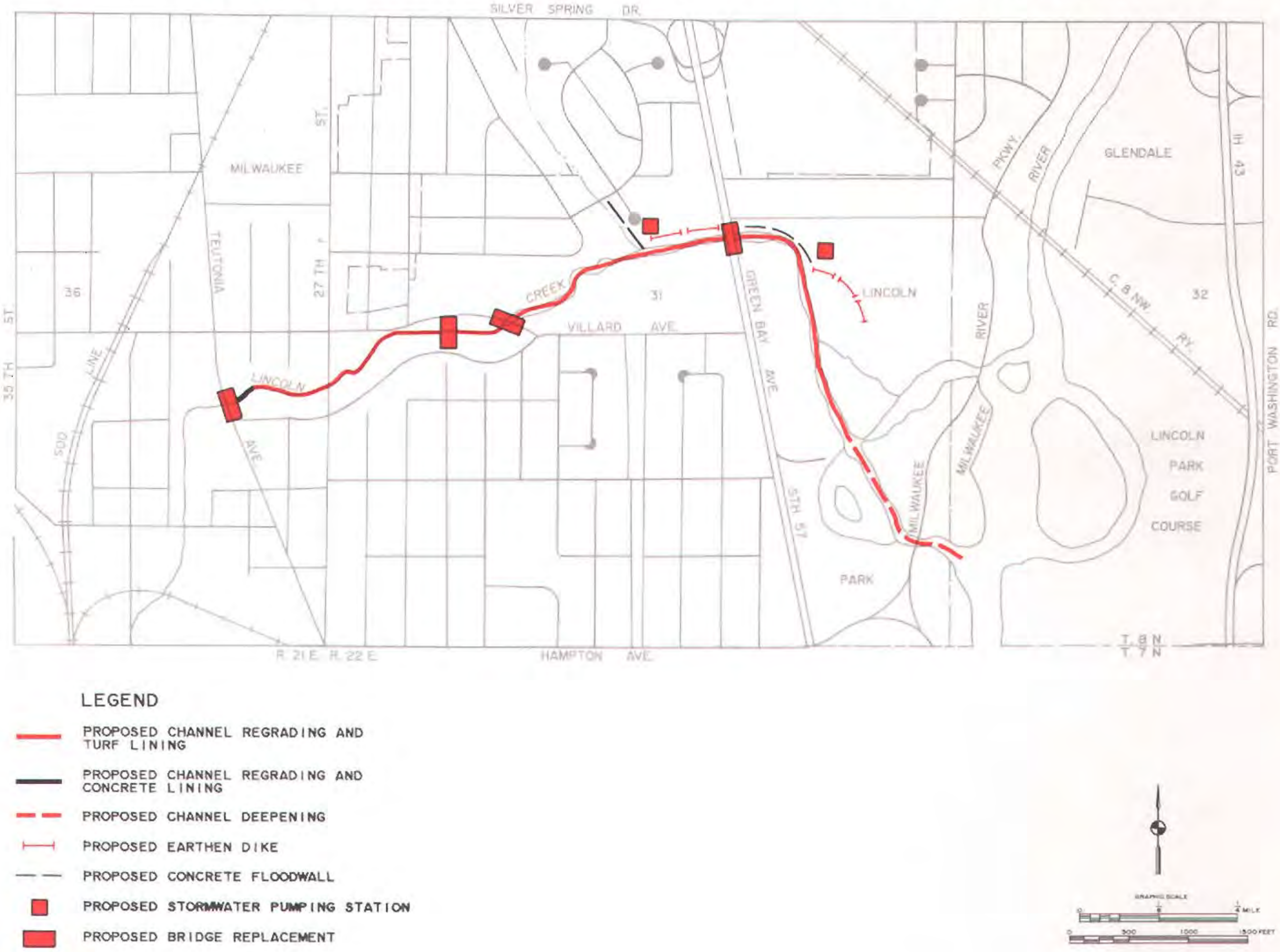
except for the first 150 feet downstream of N. Teutonia Avenue. This short reach would have a concrete invert similar to that which exists upstream of N. Teutonia Avenue in order to control erosion. Besides deepening the Lincoln Creek channel, about 1,700 feet of the west channel of the Milwaukee River would need to be dredged in order to maintain an even drop in streambed from the mouth of Lincoln Creek to the confluence with the Milwaukee River main channel. The proposed streambed and high-water profiles for Lincoln Creek under this alternative are shown in Figure 55.

Hydraulic analyses of this alternative indicated that during a 100-year recurrence interval flood under planned land use and channel conditions, the streamflow velocities in the channel downstream of N. Green Bay Avenue may be expected to exceed somewhat the maximum velocity recommended in Chapter III of this report. The performance of the turf-lined channel within this reach would have to be monitored, and appropriate erosion control measures such as a rip-rap lining added as necessary at critical locations. Streamflow velocities during flood events up to and including the 10-year recurrence interval storm event may be expected to be below the recommended maximum.

Also required under this alternative would be the replacement of three bridges in order to provide adequate hydraulic capacity to pass flood flows, as well as to accommodate the lower channel

Map 127

MAJOR TURF-LINED CHANNEL MODIFICATION FLOOD CONTROL
ALTERNATIVE FOR LOWER LINCOLN CREEK DOWNSTREAM OF N. TEUTONIA AVENUE



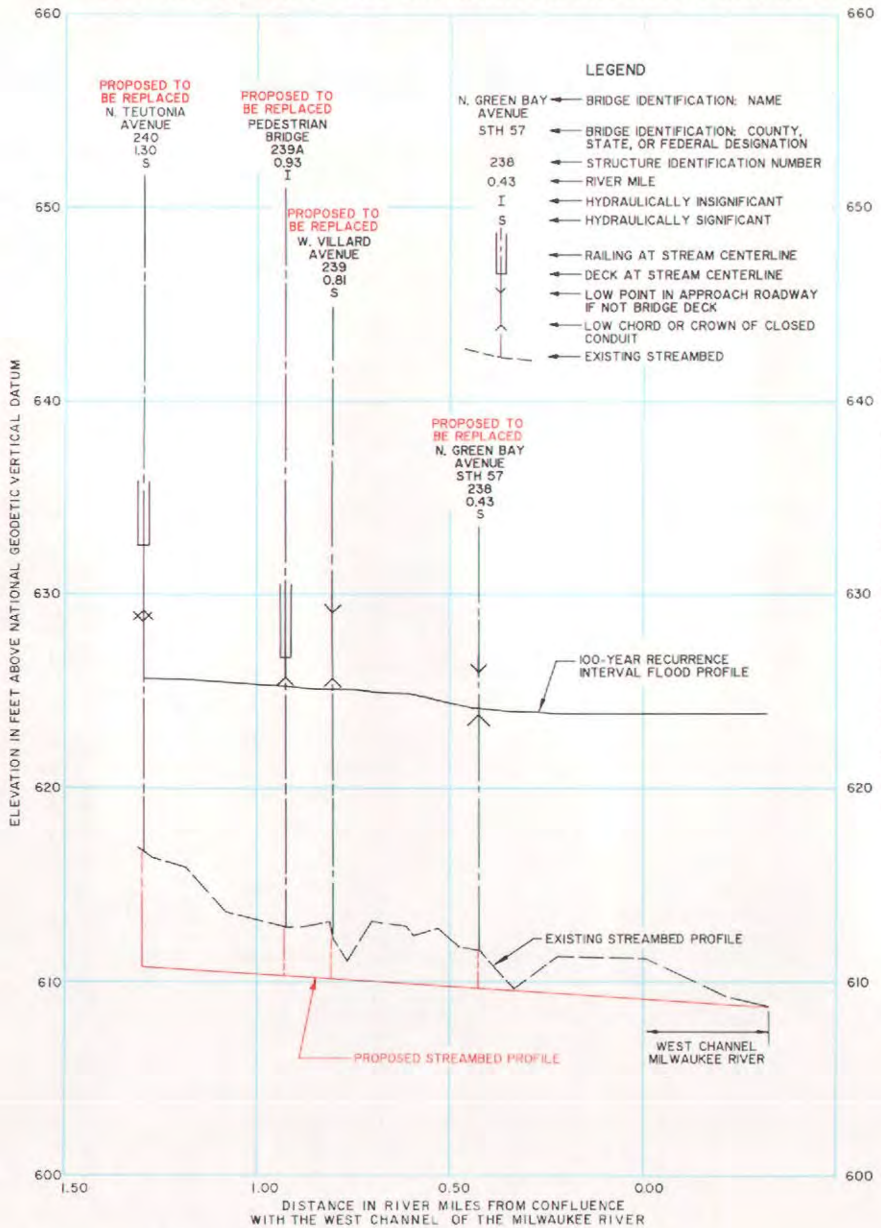
Source: SEWRPC.

invert. These bridges are located at N. Green Bay Avenue (River Mile 0.43), W. Villard Avenue (River Mile 0.81), and N. Teutonia Avenue (River Mile 1.30). The pedestrian bridge at River Mile 0.93 would be removed and would not be replaced.

Under this alternative, the 100-year recurrence interval flood flows under planned land use and channel conditions would not be confined to the modified channel downstream of N. 27th Street extended. Potential flood damages downstream of this location would be significantly reduced, however. Structure flooding may be expected to continue to occur along two reaches—at about eight structures along Crestwood Creek immediately north of Lincoln Creek and at about 25 structures along W. Lawn Avenue east of N. Green Bay Avenue. Eliminating the flood damages along W. Lawn Avenue would require the construction of about 1,400 feet of dike and floodwall along the east side of Lincoln Creek. About 400 feet of concrete floodwall with an average height of five feet would be required beginning at a point about 250 feet downstream of N. Green Bay Avenue. An additional 1,000 feet of earthen dike with an average height of five feet would be required, ending at a point about 1,300 feet downstream of N. Green Bay Avenue. In order to eliminate flood damages along Crestwood Creek, the construction of about 1,300 feet of dike and floodwall would be required. About 750 feet of earthen dike with an average height of five feet would be required along the north side of Lincoln Creek beginning

Figure 54

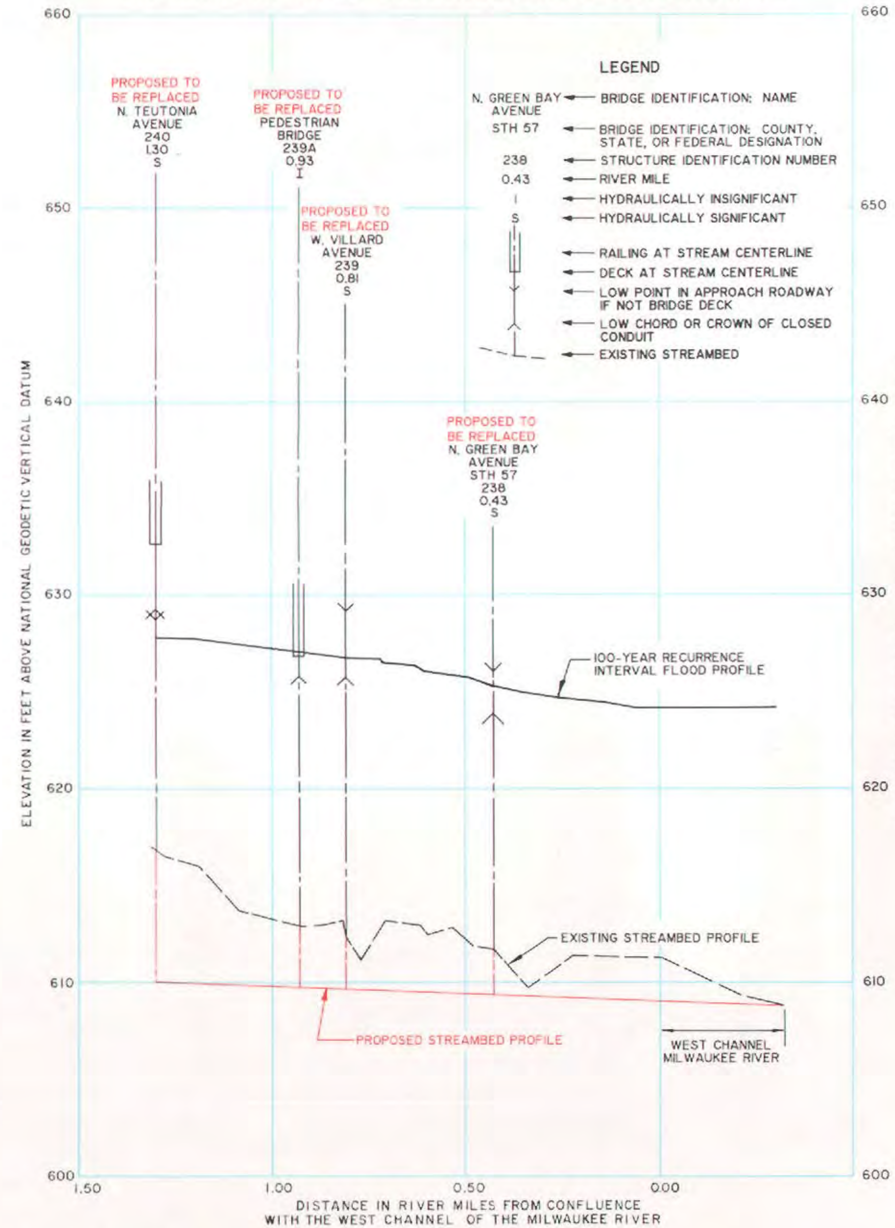
STREAMBED AND FLOOD PROFILES FOR LOWER LINCOLN CREEK DOWNSTREAM OF N. TEUTONIA AVENUE UNDER CONCRETE-LINED CHANNELIZATION ALTERNATIVE



Source: SEWRPC.

Figure 55

STREAMBED AND FLOOD PROFILES FOR LOWER LINCOLN CREEK DOWNSTREAM OF N. TEUTONIA AVENUE UNDER TURF-LINED CHANNELIZATION ALTERNATIVE



Source: SEWRPC.

immediately upstream of N. Green Bay Avenue. An additional 550 feet of concrete floodwall would be required along the east bank of Crestwood Creek beginning at its confluence with Lincoln Creek. One stormwater pumping station with a capacity of 45 cfs would be constructed to the northeast of the confluence of Crestwood Creek and Lincoln Creek. A second stormwater pumping station with a capacity of 10 cfs would be constructed on county parkland downstream from N. Green Bay Avenue. The locations of these dikes, floodwalls, and pumping stations are shown on Map 127.

The total capital cost of this flood control alternative is estimated at $5,498,000, with an annual operation and maintenance cost of about $16,400. A detailed breakdown of these costs is shown in Table 72.

Evaluation of Refined Alternative Flood Control Measures for Lower Lincoln Creek

By incorporating the refined subalternatives for the portion of Lincoln Creek downstream of N. Teutonia Avenue into the alternatives described earlier in this chapter, three refined alternative flood control plans were developed for Lower Lincoln Creek. Each of these plans incorporates the same improvements between W. Silver Spring Road and N. Teutonia Avenue. Those improvements provide for major channel improvements and bridge replacement as described under Alternative 2—major channelization, and shown on Map 122. The alternative components downstream of N. Teutonia Avenue are those described under the three refined subalternatives in the previous section. Each of the three refined alternative flood control plans for Lower Lincoln Creek is described briefly below. The economic benefits and costs attendant to each alternative are provided in Table 73.

Refined Alternative Plan A—Major Concrete-Lined Channelization Upstream of N. Teutonia Avenue with Dikes and Floodwalls Downstream of N. Teutonia Avenue: The major channelization alternative with diking and pumping for Lower Lincoln Creek would consist of about 2.5 miles of major concrete-lined channel reconstruction and improvement, as shown on Maps 122 and 124. This alternative would also involve the installation of about 10,200 feet of earthen dike and about 3,350 feet of concrete floodwall, the construction of six stormwater pumping stations and installation of 17 backwater gates, and some storm sewer modifications in the areas behind the dikes and floodwalls. Also, as part of the channel improvements, it would be necessary to modify, or replace, 14 bridges over Lower Lincoln Creek.

Implementation of this alternative would essentially eliminate all damages from floods up to and including the 100-year recurrence interval event. This alternative has an estimated capital cost of $23,415,000 and an annual average operation and maintenance cost of $56,700.

Refined Alternative Plan B—Major Concrete-Lined Channelization Upstream and Downstream of N. Teutonia Avenue: The major channelization alternative for Lower Lincoln Creek would consist of about 3.8 miles of major concrete-lined channel reconstruction and improvement, as shown on Maps 122 and 126. This alternative would involve the installation of about 1,500 feet of earthen dike and about 400 feet of concrete floodwall, the construction of two stormwater pumping stations and installation of three backwater gates, and some storm sewer modifications in the areas behind the dikes and floodwalls. Also, as part of the channel improvements, it would be necessary to modify or replace 14 bridges over Lower Lincoln Creek.

Implementation of this alternative would essentially eliminate all damages from floods up to and including the 100-year recurrence interval event. This alternative has an estimated capital cost of $23,121,000 and an average annual operation and maintenance cost of $26,200.

Refined Alternative Plan C—Major Concrete-Lined Channelization Upstream of N. Teutonia Avenue with Turf-Lined Channelization Downstream of N. Teutonia Avenue: The major channelization alternative for Lower Lincoln Creek would consist of about 2.5 miles of concrete-lined, and about 1.3 miles of turf-lined, major channel reconstruction and improvement, as shown on Maps 122 and 127. This alternative would involve the installation of about 2,250 feet of earthen dike and about 950 feet of concrete floodwall, the construction of three stormwater pumping stations and installation of four backwater gates, and some storm sewer modifications in the areas behind the dikes and floodwalls. Also, as part of the channel improvements, it would be necessary to modify or replace 14 bridges over Lower Lincoln Creek.

Implementation of this alternative would essentially eliminate all damages from floods up to and including the 100-year recurrence interval

Table 73

ECONOMIC ANALYSIS OF REFINED ALTERNATIVE FLOOD CONTROL PLANS FOR LOWER LINCOLN CREEK[a]

Alternative	Costs				Benefit-Cost Analysis			
	Capital	Annual						
		Amortized Capital[b]	Operation and Maintenance	Total	Annual Benefits	Annual Benefits Minus Annual Costs	Benefit-Cost Ratio	Economic Ratio Greater than One
1. Channelization with Concrete Lining Upstream of N.Teutonia Avenue—Dikes and Floodwalls Downstream of N. Teutonia Avenue	$23,415,000	$1,535,000	$56,700	$1,591,700	$802,000	$-789,700	0.50	No
2. Channelization with Concrete Lining Upstream and Downstream of N. Teutonia Avenue	23,121,000	1,475,000	26,200	1,501,200	802,000	-699,200	0.53	No
3. Channelization with Concrete Lining Upstream of N.Teutonia Avenue—Channelization with Turf-Lining Downstream of N. Teutonia Avenue	19,968,000	1,280,000	27,100	1,307,100	802,000	-505,100	0.61	No

[a]Lower Lincoln Creek is the reach of stream between W. Silver Spring Drive and the confluence of Lincoln Creek with the Milwaukee River.

[b]Amortized capital cost is based on an interest rate of 6 percent and a project life of 50 years, and includes the cost of the replacement of pumps after 25 years of operation.

Source: SEWRPC.

event. This alternative has an estimated capital cost of $19,968,000 and an average annual operation and maintenance cost of $27,100.

Conclusion: Each of the three refined alternatives is technically feasible. However, selection of the recommended flood control measures for Lower Lincoln Creek depends not only upon cost considerations, but also consideration of environmental and aesthetic impacts, and implementability. Also, the noneconomic, or intangible, benefits of each alternative plan must be considered.

The results of an economic analysis of the three refined alternative flood control plans for Lower Lincoln Creek are set forth in Table 73. As shown in Table 73, the average annual cost of Refined Alternative C, providing for major concrete- and turf-lined channels, is about 18 percent and 13 percent lower, respectively, than the equivalent cost of Refined Alternative A providing for major concrete-lined channelization with diking and floodwalls, and Refined Alternative B providing for concrete-lined channelization. As already noted, however, other factors must be considered in selecting a recommended plan.

In terms of impacts on the natural environment, it appears that Alternative A, refined channelization with diking and pumping, would be preferable, since there would be no significant disturbance to the existing Lincoln Creek channel downstream of N. Teutonia Avenue, including the 0.4-mile length through Lincoln Park to the confluence with the west channel of the Milwaukee River.

The second most favorable alternative in this regard would be Refined Alternative C, providing for a combination of concrete- and turf-lined channels. Under this alternative, the channel downstream of N. Teutonia Avenue would be lowered and widened. The channel bottom and sides would be turf-lined, or lined with other types of vegetation. The channel between N. Teutonia Avenue and the Milwaukee River would have a depth of seven to eight feet during summer low-flow conditions under either Refined Alternative B or C, as opposed to a depth of one to six feet under existing conditions and under Refined Alternative A. This added depth under Refined Alternatives B and C would be caused by the backwater from the Milwaukee River extending up the deepened channel. This added depth could be beneficial for certain types of fish and supporting aquatic life, but could present a safety hazard. It is possible that selected instream measures could be developed in the deeper vegetated channel for aquatic life and adjacent habitat, thus minimizing the negative impacts.

In terms of environmental impacts on developed areas, Alternative A, providing for dikes and floodwalls, results in increased stages ranging from 2.0 to 3.5 feet downstream of N. Teutonia Avenue compared to planned land use and existing channel conditions. The alternative that provides for an entirely concrete-lined channel downstream of N. Teutonia Avenue—Refined Alternative B—results in stage changes which range from a reduction of 2.0 feet at N. Teutonia Avenue to an increase of 2.0 feet at the confluence of Lincoln Creek with the Milwaukee River. Refined Alternative C, providing for a turf-lined channel downstream of N. Teutonia Avenue, results in no stage change at N. Teutonia Avenue and about a 2.0-foot increase in stages at the confluence with the Milwaukee River compared to existing conditions. The areas adjacent to the stream reaches experiencing the increased stages would be protected by dikes.

Both the dike and floodwall alternative and the channelization alternative would have negative aesthetic impacts. Construction of dikes and floodwalls under Refined Alternative A may be expected to obstruct the view in significant areas. The construction of floodwalls would eliminate natural stream bank vegetation and wildlife habitat. Under Refined Alternative B, which provides for a concrete-lined channel, there would be a negative aesthetic impact and loss of wildlife habitat adjacent to the stream. The turf-lined channel called for under Refined Alternative C would be perceived as a negative aesthetic impact by those who prefer the visual effect of natural stream channels. However, others may prefer the more urban character of a turf-lined channel.

In terms of implementability, Refined Alternative A, providing for long reaches of dikes and floodwalls and relying on the pumping of stormwater during major events, is the least favorable of the refined alternatives. Under this alternative, flood stages would increase significantly downstream of N. Teutonia Avenue. While the adjacent areas would be protected from flooding by dikes, floodwalls, and pumping stations, the use of such measures could result in local flooding should the pumping systems or appurtenant backwater gates fail. Refined Alternative Plan C appears to be the most implementable and is preferable to Refined Alternative B in this respect as it would entail minimal negative environmental impacts.

After due consideration of the various technical and economic features of the alternative floodland management measures, it is recommended that Refined Alternative C providing for a concrete-lined channel upstream of N. Teutonia Avenue and a turf-lined channel downstream of N. Teutonia Avenue be adopted to abate flooding problems along Lower Lincoln Creek.

Recommended Flood Control System for Lincoln Creek

Based upon consideration of the technical feasibility, economic viability, environmental impacts, potential public acceptance, and practicality of each of the alternatives considered, it was recommended that Alternative Plan 2—limited channelization—in combination with Storm Sewer Relief Alternative 2—channel deepening—be adopted and implemented for Upper Lincoln Creek; and that Refined Alternative Plan C—concrete-lined channelization upstream of N. Teutonia Avenue and turf-lined channelization downstream of N. Teutonia Avenue—be adopted and implemented for Lower Lincoln Creek. Minor refinements have been made to the recommended alternatives as described below. These refinements include changes made as part of the preliminary engineering work carried out subsequent to the system planning.

The total capital cost of the recommended combined flood control plan for Upper and Lower Lincoln Creek is estimated at $21,865,000 in 1986 dollars. This includes the cost of channel deepening—estimated at $813,000—required for storm sewer relief in Upper Lincoln Creek, as shown in Table 74. The recommended plan is

Table 74

ECONOMIC ANALYSIS OF THE RECOMMENDED FLOOD CONTROL PLAN FOR LINCOLN CREEK

Alternative	Costs				Benefit-Cost Analysis			
	Capital	Annual			Annual Benefits	Annual Benefits Minus Annual Costs	Benefit-Cost Ratio	Economic Ratio Greater than One
		Amortized Capital[a]	Operation and Maintenance	Total				
Upper Lincoln Creek[b]								
Channel Modification	$ 33,000	$ 2,100	$ 1,000	$ 3,100				
Channel Cleaning and Debrushing	10,000	600	- -	600				
Dikes	3,000	200	100	300				
Bridge Removal and Replacement	666,000	42,200	- -	42,200				
Subtotal	$ 712,000	$ 45,100	$ 1,100	$ 46,200	$ 42,000	$ -4,200	0.91	No
Lower Lincoln Creek								
Channel Modification and Bridge Removal[c]	$10,083,000	$ 640,000	$ 7,600	$ 647,600				
Bridge Replacement	7,665,000	487,000	- -	487,000				
Dikes and Floodwalls	1,130,000	71,800	7,500	79,300				
Stormwater Drainage	538,000	34,100	- -	34,100				
Pumping Stations[d]	714,000	55,800	12,000	67,800				
Street Regrading	210,000	-13,300	- -	-13,300				
Subtotal	$20,340,000	$1,302,000	$27,100	$1,329,100	$802,000	$-527,100	0.60	No

[a]*Amortized capital cost is based on an interest rate of 6 percent and a project life of 50 years.*

[b]*In addition to the flood control costs for Upper Lincoln Creek set forth in this table, $813,000 would be required for drainage improvements to provide storm sewer relief.*

[c]*Channel modification capital cost includes the cost of $851,000 for demolition of all bridges to be replaced.*

[d]*Amortized capital cost includes the cost of the replacement of pumps after 25 years of operation.*

Source: SEWRPC.

shown graphically on Map 128. The peak flood profile which would be attendant to planned future land use and channel conditions in the subwatershed is shown in Figure 56. Both of the alternative plans which together constitute the recommended plan have the highest benefit-cost ratios of the alternative plans considered—0.91 and 0.60, respectively.

The recommended plan would essentially eliminate all flood-related damages along the Lincoln Creek channel under the 100-year recurrence interval flood for both existing and planned future land use conditions. It would also provide an adequate drainage outlet for the storm sewers and watercourses tributary to Lincoln Creek.

The limits of the floodplain under planned land use and existing and planned channel conditions along Lincoln Creek were mapped using the new, large-scale topographic maps. A sample of the maps produced is shown as Map 129.

The recommended plans make the maximum use of stormwater storage in existing ponding areas and in the channel itself. The channel would be designed to carry the 100-year recurrence interval flood event with two feet of freeboard except that reach located downstream of N. Teutonia Avenue where the channel capacity is exceeded and the adjacent parkway but no structures are flooded. All flooding of existing structures located in the Lincoln Creek subwatershed due to floods up to and including the 100-year recurrence interval flood on Lincoln Creek would be eliminated. The recommended plan is more fully described below.

The recommended flood control plan for Upper Lincoln Creek is best understood by dividing the creek into seven distinct reaches. The recommended plan for these seven reaches is summarized on Map 128, with typical cross-sections shown for the recommended channel for each reach. The plan recommendations for each of these seven reaches are as follows:

1. From the Beginning of Upper Lincoln Creek at N. 76th Street Just North of W. Good Hope Road to the Chicago & North Western Railway Crossing Just Downstream of N. 60th Street. No changes are recommended along this 1.1-mile reach of Upper Lincoln Creek. For the most part, this reach of the creek traverses the Brynwood Country Club.

2. From the Chicago & North Western Railway Crossing Just Downstream of N. 60th Street to W. Good Hope Road. Along this 0.6-mile reach of Upper Lincoln Creek, the existing channel is proposed to be widened and deepened to accommodate flood flows and provide free outlets for two existing storm sewer outfalls which presently have invert elevations lower than the existing channel bottom. The channel deepening would range from 4.0 to 7.0 feet. The new channel would be turf lined, with a rip-rap invert. In order to fit the needed conveyance waterway cross-section within the restrictive right-of-way, reinforced concrete retaining walls ranging from 4.0 to 9.0 feet in height would have to be constructed along the channel for the first 1,350 feet upstream of the existing concrete drop spillway on the upstream side of the W. Good Hope Road culvert. The existing concrete drop spillway at the upstream side of the W. Good Hope Road culvert would be removed, but the culvert itself would not have to be replaced. The culvert would have to be cleaned, however, to make the full depth available for flow.

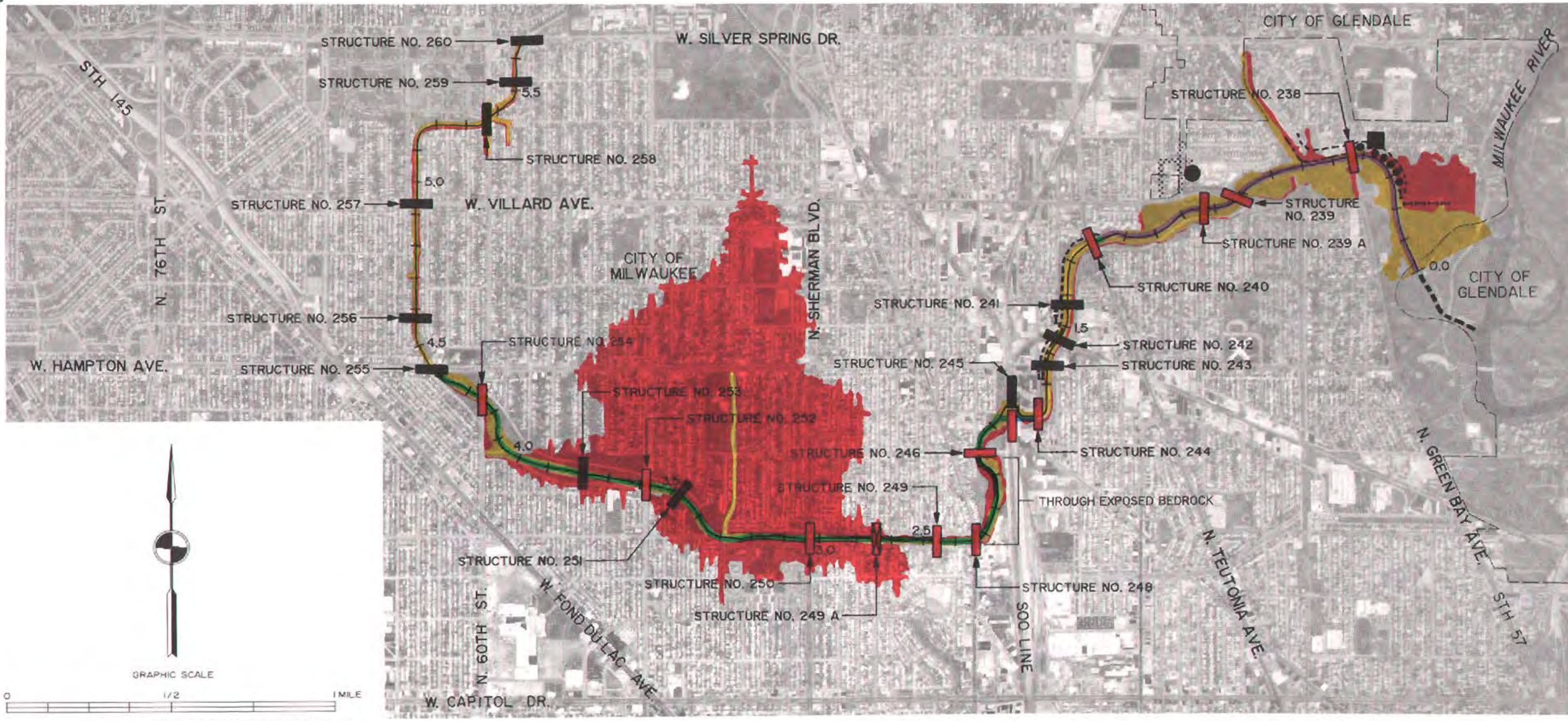
3. From W. Good Hope Road to W. Mill Road. Along this 1.0-mile reach of Upper Lincoln Creek, the channel would be deepened, widened, and turf lined with a rip-rap invert to provide free outlets for four existing storm sewer outfalls which now have inverts below the existing streambed elevation. The deepening would range from 2.0 feet to 7.0 feet. It would be necessary to replace the W. Green Tree Road culvert to accommodate the new channel.

4. From W. Mill Road to W. Woolworth Avenue. Along this 0.1-mile reach of Upper Lincoln Creek, the channel would be lowered about 2.5 feet and would be turf lined, with a rip-rap invert. This lowering is necessary because of the lowered channel proposed upstream, and to help provide a free outlet for the existing storm sewer outfall which discharges within the W. Mill Road structure. The channel lowering would require the replacement of the W. Woolworth Avenue and N. 51st Street culverts. The W. Mill Road culvert was built to accommodate a lower channel; therefore, this culvert need not be replaced. However, it would be necessary to clean out the channel through the culvert down to the design depth of the culvert.

5. From W. Woolworth Avenue to the Existing Pedestrian Bridge Near the Northern Limits of the Havenwoods Environmental Education Center. In this 0.1-mile reach of Upper Lincoln Creek, it is recommended that the channel be widened and deepened in order to both accommodate flood flows and provide a free outlet for a partially buried existing storm sewer outfall at W. Woolworth Avenue. The deepening of the channel bottom would range from about 0.5 foot at the pedestrian bridge to about 2.5 feet at W. Woolworth Avenue. The new channel would be turf lined, with a rip-rap invert. It is also recommended that two houses located along N. 51st Street between W. Woolworth Avenue and W. Mill Road be removed to provide an adequate right-of-way for the proposed channel. These houses would experience flood damage under a 100-year recurrence interval event unless costly channel enclosure measures were taken. In addition, the plan recommends that the existing concrete arch culvert under the Chicago & North Western Railway be removed and replaced with a double concrete box culvert, with each cell being 12 feet high by 10 feet wide. The existing culvert is inadequate and

Map 128

RECOMMENDED FLOOD CONTROL PLAN FOR LINCOLN CREEK



DATE OF PHOTOGRAPHY APRIL 1986

LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS

CHANNEL REGRADING AND CONCRETE LINING

CHANNEL REGRADING AND TURF LINING

CHANNEL ENCLOSURE

EARTHEN DIKE

CONCRETE FLOODWALL

STEEL SHEETPILE FLOODWALL

NEW BRIDGE

STRUCTURE MODIFICATION

CHANNEL DREDGING

STORMWATER PUMPING STATION

STORM SEWER

ROAD GRADE ELEVATION

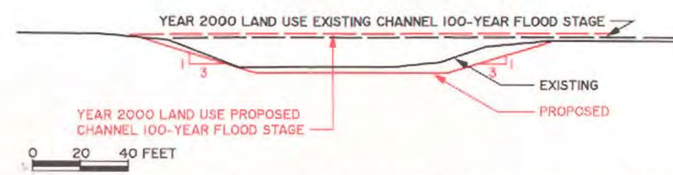
AREA OF POTENTIAL LOCAL DRAINAGE SYSTEM INADEQUACY WHICH IS TO BE ADDRESSED DURING THE FINAL DESIGN

2.5 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR LINCOLN CREEK IS SHOWN IN APPENDIX H

DUE TO MAP SCALE LIMITATIONS, THE DIFFERENCE BETWEEN THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS, AND THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS, MAY NOT APPEAR ON THIS MAP. WHERE NO DIFFERENCE APPEARS REFERENCE SHOULD BE MADE TO THE FLOOD STAGE PROFILE SHOWN BELOW

TYPICAL CROSS SECTION OF THE EXISTING AND PROPOSED CHANNEL ALONG LINCOLN CREEK BETWEEN THE CONFLUENCE WITH THE MILWAUKEE RIVER AND N. TEUTONIA AVENUE



TYPICAL CROSS SECTION OF THE EXISTING AND PROPOSED CHANNEL ALONG LINCOLN CREEK BETWEEN THE SOO LINE RAILWAY CROSSING AT RIVER MILE 2.01 AND THE W. HAMPTON AVENUE CROSSING AT RIVER MILE 4.41

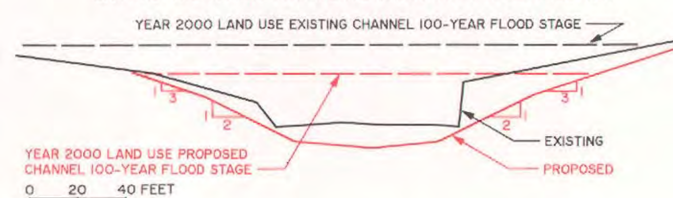
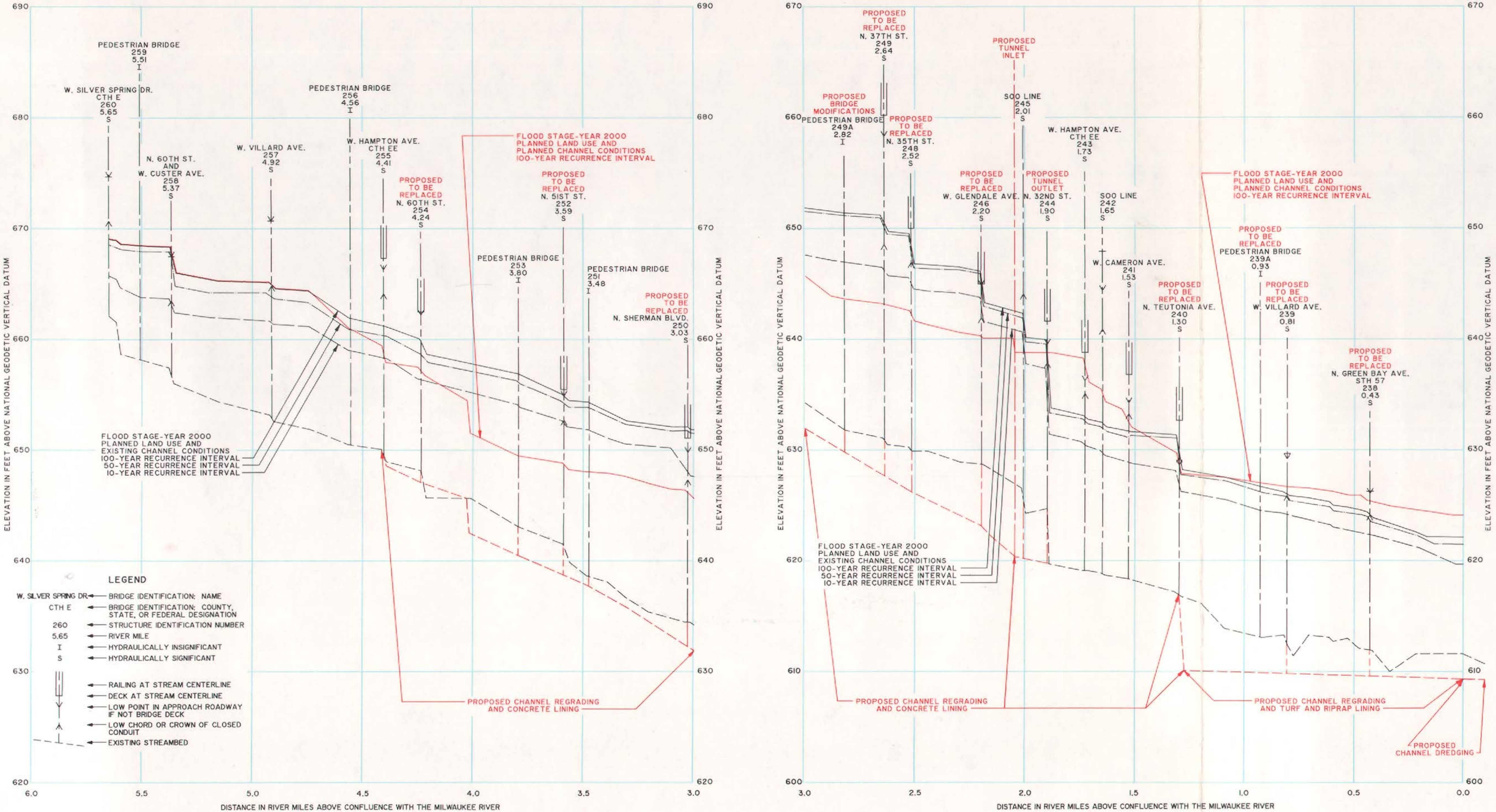
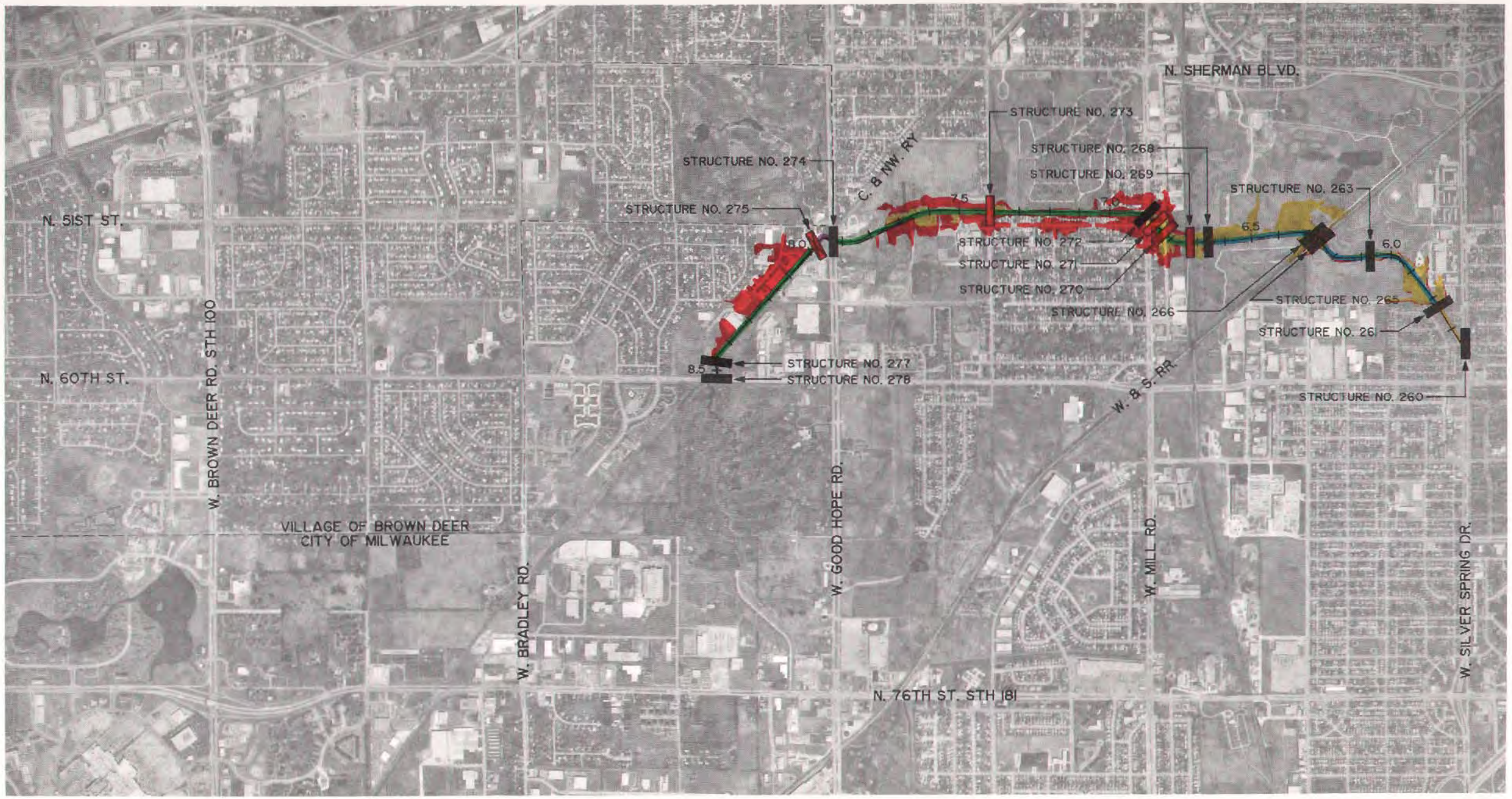


Figure 56

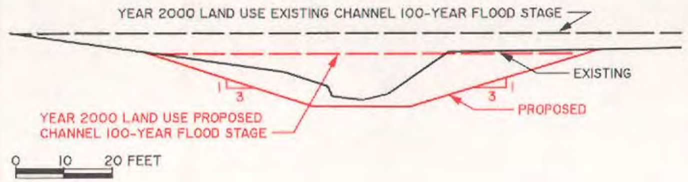
RECOMMENDED PLAN FLOOD STAGE PROFILE FOR LINCOLN CREEK



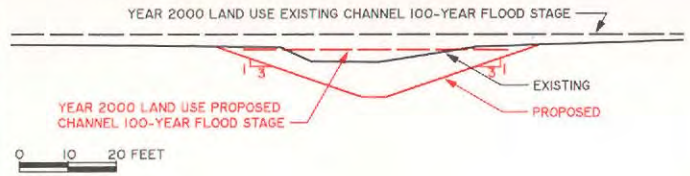
Map 128 (continued)



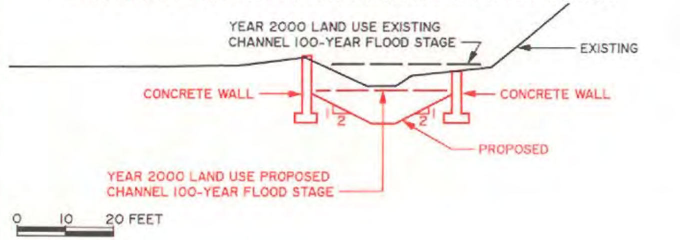
TYPICAL CROSS SECTION OF THE EXISTING AND PROPOSED CHANNEL ALONG LINCOLN CREEK BETWEEN THE PEDESTRIAN BRIDGE NEAR THE NORTHERN LIMITS OF THE HAVENWOODS ENVIRONMENTAL EDUCATION CENTER AND RIVER MILE 6.94



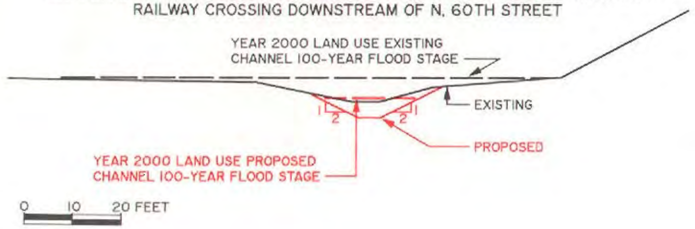
TYPICAL CROSS SECTION OF THE EXISTING AND PROPOSED CHANNEL ALONG LINCOLN CREEK BETWEEN RIVER MILE 6.94 AND W. GOOD HOPE ROAD



TYPICAL CROSS SECTION OF THE EXISTING AND PROPOSED CHANNEL ALONG LINCOLN CREEK BETWEEN W. GOOD HOPE ROAD AND RIVER MILE 8.25



TYPICAL CROSS SECTION OF EXISTING AND PROPOSED CHANNEL ALONG LINCOLN CREEK BETWEEN RIVER MILE 8.25 AND CHICAGO & NORTHWESTERN RAILWAY CROSSING DOWNSTREAM OF N. 60TH STREET

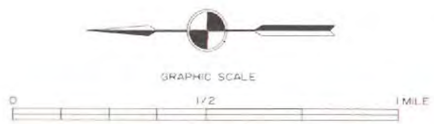


LEGEND

- 100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS
- 100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS
- PROPOSED CHANNEL DEEPENING
- PROPOSED CHANNEL CLEANING AND DEBRUSHING
- PROPOSED EXPOSURE OF EXISTING CONCRETE INVERT
- PROPOSED NEW BRIDGE
- PROPOSED REMOVAL OF CONCRETE DROP SPILLWAY
- 7.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR LINCOLN CREEK IS SHOWN IN APPENDIX H

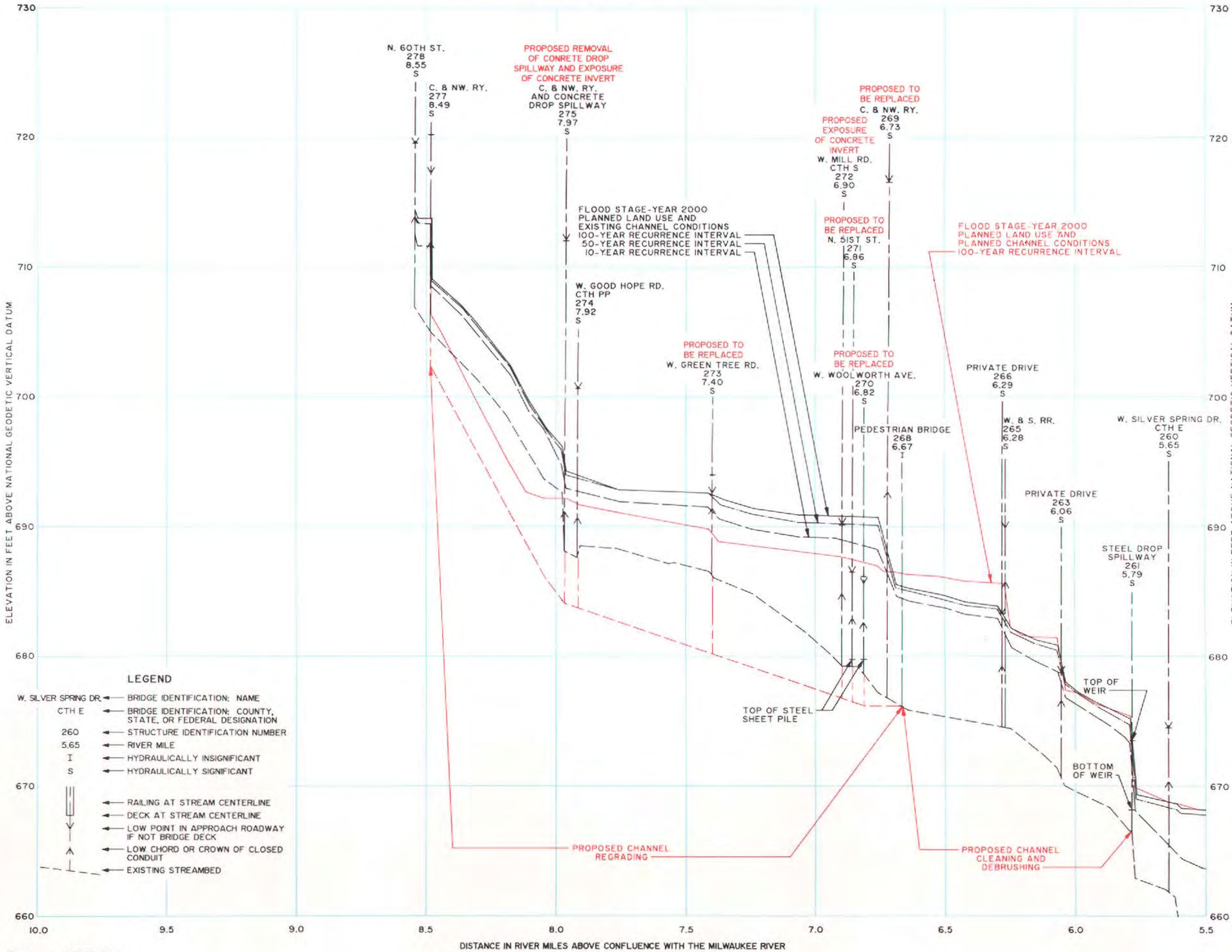
DUE TO MAP SCALE LIMITATIONS, THE DIFFERENCE BETWEEN THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS, AND THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS, MAY NOT APPEAR ON THIS MAP. WHERE NO DIFFERENCE APPEARS REFERENCE SHOULD BE MADE TO THE FLOOD STAGE PROFILE SHOWN BELOW



DATE OF PHOTOGRAPHY APRIL 1986

Source: SEWRPC.

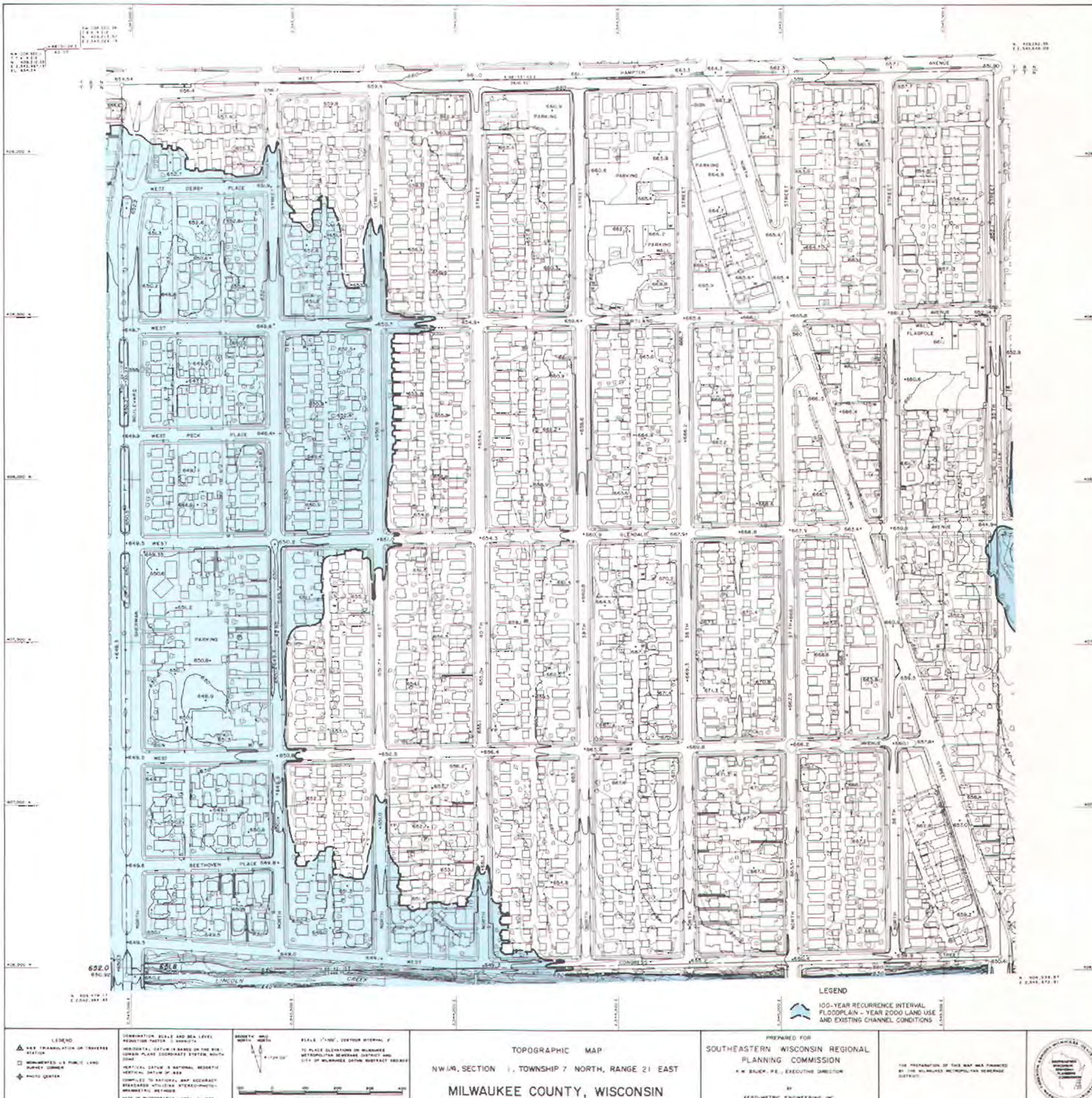
Figure 56 (continued)



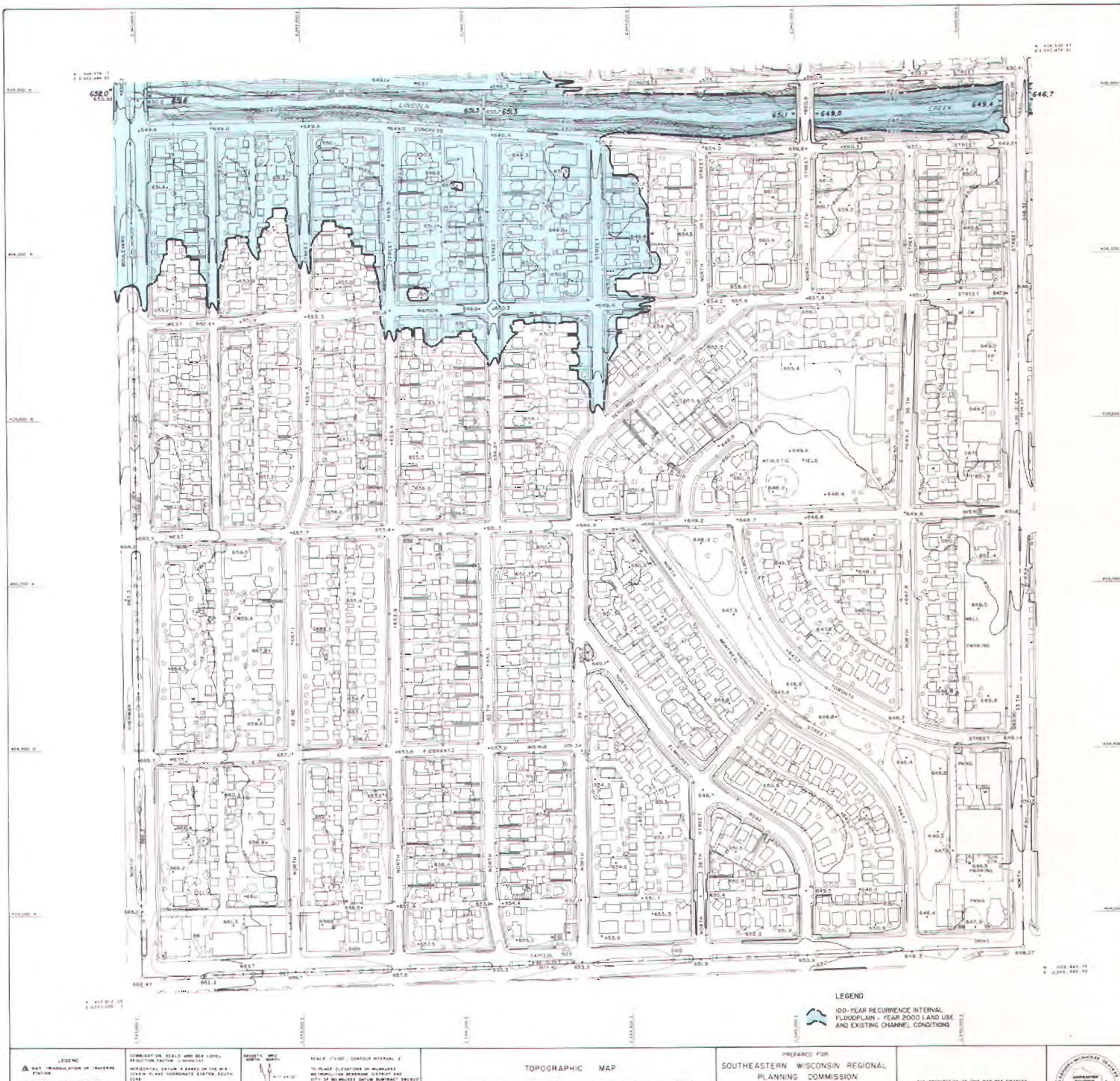
Source: SEWRPC.

Map 129

SAMPLE FLOODPLAIN DELINEATION ON LARGE-SCALE TOPOGRAPHIC MAPS



Map 129 (continued)



Source: SEWRPC.

creates backwater under flood-flow conditions which extends back across W. Woolworth Avenue to W. Mill Road.

6. From the Existing Pedestrian Bridge Near the Northern Limits of the Havenwoods Center to the Existing Steel Drop Spillway Immediately West of the U. S. Army Reserve Training Center. This 0.9-mile reach of Upper Lincoln Creek extends through the U. S. Army property and the Havenwoods Center. It is recommended that the channel in this reach be cleaned and debrushed so as to facilitate and improve flood flows. No channel enlargement or deepening is recommended in this reach. In the event that the Department undertakes any channel modifications for objectives other than flood control—such as for the enhancement of wildlife habitat—it is important that the modifications be designed so as not to increase 100-year recurrence interval flood stages upstream or downstream of Havenwoods. It is important to note that any wetland basin provided in this reach would have insignificant flood control benefits downstream of W. Silver Spring Drive, and the cost of development would have to be justified on other than a flood control basis.

7. From the Existing Steel Drop Spillway Immediately West of the U. S. Army Reserve Training Center to W. Silver Spring Drive. No channel or structure changes are recommended in this 0.1-mile reach of Upper Lincoln Creek. The existing channel has a paved bottom and side slopes and is adequate to accommodate the 100-year recurrence interval flood flow.

The recommended flood control plan for Lower Lincoln Creek is best understood by dividing the creek into six distinct reaches. The recommended plan for these six reaches is summarized on Map 128, with the typical cross-sections shown for the recommended channel for each reach. The plan recommendations for each of these six reaches are as follows:

1. From W. Silver Spring Drive to the W. Hampton Avenue Crossing Just West of N. 60th Street. No changes are required along this 1.2-mile reach of Lower Lincoln Creek. Within this reach the channel has already been deepened, widened, and lined with concrete. The channel has adequate conveyance capacity to accommodate the 100-year recurrence interval flood flow.

2. From the W. Hampton Avenue Crossing Just West of N. 60th Street to N. 32nd Street. Major channel improvements are recommended along this 2.5-mile reach of Lower Lincoln Creek. Throughout this reach the channel bottom would be lowered from 1.0 to 6.5 feet, with an average depth of excavation of about two feet. Except for that portion of this reach between N. 32nd Street and the Soo Line Railroad, the existing channel would be reconstructed with a bottom width of about 30 feet and a top width varying from 100 to 200 feet, depending upon the depth of excavation and the topography adjacent to the existing channel. A concrete lining would be installed in the lower portion of the channel, with revegetation of the upper channel side slopes.

Between N. 32nd Street and a point about 100 feet west of the Soo Line Railroad, the channel would be enclosed in a triple-cell reinforced concrete box culvert. The cells of this culvert would be sized so as to match the culverts under N. 32nd Street, which were constructed by the City of Milwaukee in 1984. The proposed culverts would follow a new alignment which would cross the Soo Line tracks at a point about 200 feet south of the existing channel.

The existing bridges at N. 60th Street, N. 51st Street, N. Sherman Boulevard, N. 37th Street, N. 35th Street, and W. Glendale Avenue would have to be replaced. One pedestrian bridge located at River Mile 2.82 would have to be modified so as to accommodate the proposed lowered channel grade. As noted above, the bridge at N. 32nd Street which was recommended to be replaced in the previously mentioned 1982 flood control report was replaced by the City of Milwaukee in 1984. Since the bridges at N. Sherman Boulevard and N. 37th Street are particularly significant hydrologically and hydraulically—that is, they act as dams during major flood events and reduce downstream flood flows—the plan recommends that these two bridges not be replaced until the recommended channel improvements are carried out.

It should be noted that there are two areas in this reach, as shown in the 1982 flood control report and on Map 128, where bedrock is exposed. The exposed rock is Upper Silurian Waubakee Dolomite. This bedrock exposure is considered to be scientifically important, as it represents the only accessible exposure of this formation in eastern Wisconsin. The design of any channel improvement should seek to preserve these geologic outcrops or to provide comparable exposures after the improvements are completed.

3. From N. 32nd Street to the Soo Line Railroad Crossing Just Downstream of W. Hampton Avenue. The channel along this 0.2-mile reach has already been improved through widening, deepening, and partial lining with concrete. In order to contain future flood flows, however, the plan recommends that a reinforced concrete floodwall be constructed along the west bank between the Soo Line Railroad and W. Hampton Avenue, and that about 100 feet of earthen dike be constructed along the west bank upstream of W. Hampton Avenue. The proposed floodwall would range in height from 2.0 to 5.0 feet, while the dike would range from 3.5 to 4.5 feet in height. Local stormwater drainage from land behind this dike and floodwall would be handled by construction of a 36-inch-diameter storm sewer designed to drain north toward the Soo Line Railroad.

4. From the Soo Line Railroad Crossing Just Downstream of W. Hampton Avenue to W. Cameron Avenue. This 0.1-mile reach of Lower Lincoln Creek has already been improved through widening, deepening, and partial lining with concrete. In order to contain future flood flows, however, the plan recommends that 330 feet of earthen dike and 290 feet of steel sheet pile wall be constructed along the west bank. The dike and wall would range from 3.0 to 4.0 feet in height. The existing 18-inch storm sewer outlet for areas behind this dike would be provided with a backwater gate. Local drainage from areas behind the dike, as well as from the proposed 36-inch-diameter storm sewer upstream of the railroad crossing, would have to be routed downstream of Cameron Avenue into a new, double, 45-inch-wide by 29-inch-high, 2,030-foot-long storm sewer which would discharge to Lincoln Creek downstream of the N. Teutonia Avenue bridge. The use of this new storm sewer represents a modification to Refined Alternative C, which provided for a stormwater pumping station at this location. Subsequent cost-effectiveness analyses indicated that the new storm sewer would be a better subalternative. The existing 12-inch storm sewer in N. 31st Street, which connects with a 48-inch storm sewer outlet in W. Cameron Avenue, would be blocked, and a new 140-foot-long, 12-inch-diameter storm sewer with a backwater gate would be run directly to Lincoln Creek.

5. From W. Cameron Avenue to N. Teutonia Avenue. The plan recommends no changes to the Lincoln Creek channel along this 0.2-mile reach. The channel has already been improved through deepening, widening, and partial concrete lining, and is adequate to convey the 100-year recurrence interval flood flow.

6. From N. Teutonia Avenue to the Mouth of Lincoln Creek. Major channel modifications are recommended along this 1.3-mile reach of Lower Lincoln Creek. Throughout this reach the channel bottom would be lowered from 0.5 foot to 7.0 feet, with an average depth of excavation of about 3.0 feet. The existing channel would be reconstructed with a bottom width of about 80 feet and an average top width of about 170 feet. The modified channel would be turf lined along its entire length except a reach 100 feet downstream of N. Teutonia Avenue. This reach would have a concrete lining similar to that which exists upstream of N. Teutonia Avenue in order to control erosion due to the large drop in streambed elevation. The west channel of the Milwaukee River between the mouth of Lincoln Creek and the main channel of the Milwaukee River would be dredged in order to maintain a continuous slope from Lincoln Creek to the Milwaukee River main channel.

In addition, the plan recommends that about 3,000 feet of dikes and floodwalls be constructed. More specifically, it is recommended that 1,030 feet of steel sheet pile floodwall with an average height of five feet be constructed along the east side of Lincoln Creek downstream of N. Green Bay Avenue. Another 720 feet of earthen dike with an average height of five feet is recommended downstream of this floodwall. Furthermore, 700 feet of concrete floodwall with an average height of five feet is recommended along the north side of Lincoln Creek extending upstream of N. Green Bay Avenue. Finally, 550 feet of concrete floodwall with an average height of five feet is recommended along the east side of Crestwood Creek extending upstream from its confluence with Lincoln Creek. In order to properly handle local drainage in the areas behind these dikes and floodwalls, the storm sewer system would need to be reconstructed and expanded. Four storm sewer outlets would require backwater gates, and one stormwater pumping station, along with associated storm sewer modifications, would need to be constructed to convey stormwater over the dikes during periods of high streamflow in Lincoln Creek.

In order to provide adequate flood-flow capacity, as well as to accommodate the lower streambed elevation, it is recommended that the street bridges at N. Green Bay Avenue, N. Villard Avenue, and N. Teutonia Avenue be replaced. It is also recommended that the pedestrian bridge located at River Mile 0.93 be replaced.

There are three streets where backwater from Lincoln Creek could cause surcharging of storm sewers and shallow flooding at St. Michael Hospital and about eight residences. It is recommended that street grades be raised at these locations to prevent flooding. Raising street grades would be less costly than providing backwater gates on the storm sewer outlets, along with a gravity outlet or pumping capacity to convey stormwater to Lincoln Creek. The locations for street regrading are shown on Map 128.

As previously noted, investigations of the stormwater drainage systems for selected locations along Lincoln Creek downstream of N. Teutonia Avenue revealed potential structure flooding problems as a result of the major stormwater drainage system—storm sewers as well as street cross-sections—being unable to handle the runoff from a 100-year recurrence interval storm. One such location was at St. Michael Hospital, where ponding at a mid-block sag in N. 25th Street may result in shallow flooding of the west end of the hospital, as well as of several nearby residences. It is therefore recommended that the City of Milwaukee engineering department conduct a local drainage system analysis in order to ensure that severe flooding problems during major storm events are not encountered. It is also recommended that when the bridge at N. 60th Street and W. Custer Avenue is replaced for transportation purposes, it be designed so as to accommodate the 50-year recurrence interval flood flow without overtopping the attendant roadway.

Impacts of Recommended
Channel Conditions on Flood Flows

Flood discharges for Lincoln Creek under both existing and year 2000 planned land use conditions and under existing channel conditions were presented earlier in this section. The improvements recommended to alleviate flooding problems in the Lincoln Creek subwatershed will significantly affect flood flows in the subwatershed. The discharges under year 2000 planned land use conditions and recommended channel conditions are set forth in Table 75.

The analyses conducted under the comprehensive watershed planning program for the Milwaukee River, as documented in SEWRPC Planning Report No. 13, A Comprehensive Plan for the Milwaukee River Watershed, were reviewed in order to assess the impact of the increased flows on the Milwaukee River downstream of Lincoln Creek. The 100-year recurrence interval flood discharge for Lincoln Creek under year 2000 planned land use conditions and recommended channel conditions is 14,000 cubic feet per second, substantially higher than the 7,980 cfs expected under existing land use and channel conditions. Because of this increase in peak discharge from Lincoln Creek, it was deemed necessary to consider the impact of this flow downstream of the confluence of Lincoln Creek with the Milwaukee River.

The estimated flow of the Milwaukee River just downstream of Lincoln Creek for a 100-year recurrence interval event under planned

Table 75

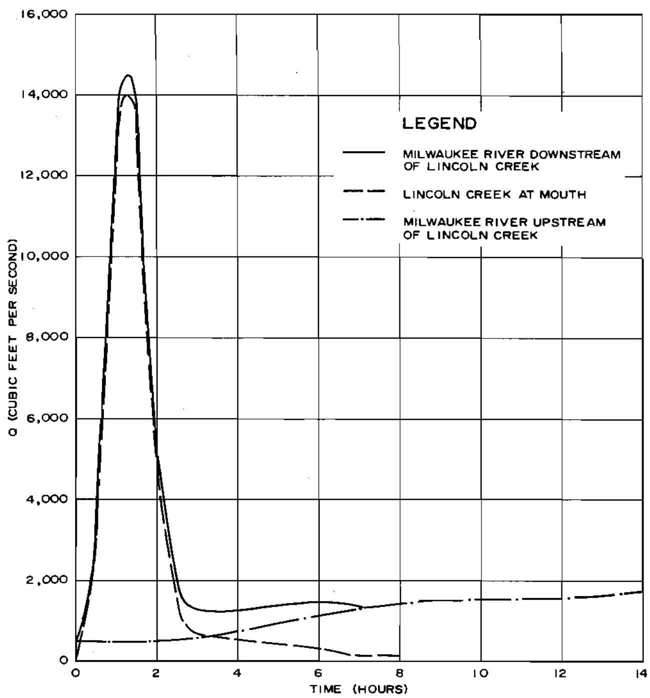
FLOOD DISCHARGES FOR LINCOLN CREEK FOR YEAR 2000
LAND USE CONDITIONS FOR RECOMMENDED CHANNEL CONDITIONS

Location	River Mile	Peak Flood Discharge (cfs) Planned Land Use, Planned Storage, and Recommended Channel Conditions		
		10-Year	50-Year	100-Year
Mouth at Milwaukee River	0.00	7,600	12,460	14,000
N. Green Bay Avenue	0.43	7,600	12,460	14,000
W. Villard Avenue	0.81	6,960	11,050	12,650
Pedestrian Bridge	0.93	6,960	11,050	12,650
N. Teutonia Avenue	1.30	6,960	11,050	12,650
W. Cameron Avenue	1.53	6,700	10,650	12,200
Soo Line Railroad	1.65	6,700	10,650	12,200
W. Hampton Avenue	1.73	6,700	10,650	12,200
N. 32nd Street	1.90	6,700	10,650	12,200
Soo Line Railroad	2.01	6,610	10,540	12,080
W.Glendale Avenue	2.20	6,610	10,540	12,080
N. 35th Street	2.52	5,350	8,540	9,790
N. 37th Street	2.64	5,350	8,540	9,790
N. Sherman Boulevard	3.03	5,140	8,200	9,430
N. 51st Street	3.59	4,030	6,310	7,350
Pedestrian Bridge	3.80	4,030	6,310	7,350
N. 58th Street (extended)	4.16	4,030	6,310	7,350
N. 60th Street	4.24	3,200	5,030	5,860
W. Hampton Avenue	4.41	3,200	5,030	5,860
Pedestrian Bridge	4.56	2,500	3,930	4,600
W. Villard Avenue	4.92	1,140	1,840	2,170
N. 60th Street	5.37	1,140	1,840	2,170
W. Silver Spring Drive	5.65	- -	- -	- -
Downstream Side	- -	1,140	1,840	2,170
Upstream Side	- -	620	980	1,110
Drop Structure	5.79	620	980	1,110
U. S. Army Bridge	6.06	590	930	1,050
Wisconsin & Southern Railroad	6.28	530	850	950
Havenwoods Bridge	6.29	530	850	950
Chicago & North Western Railway	6.73	610	980	1,120
W. Woolworth Avenue	6.82	610	980	1,120
N. 51st Street	6.86	610	980	1,120
W. Mill Road	6.90	610	980	1,120
W. Green Tree Road	7.40	560	850	965
W. Good Hope Road (structure outlet)	7.92	420	640	750
Chicago & North Western Railway (structure inlet)	7.97	210	290	310
Chicago & North Western Railway	8.49	210	290	310
N. 60th Street	8.55	- -	- -	- -
Downstream Side	- -	210	290	310
Upstream Side	- -	350	590	700

Source: SEWRPC.

Figure 57

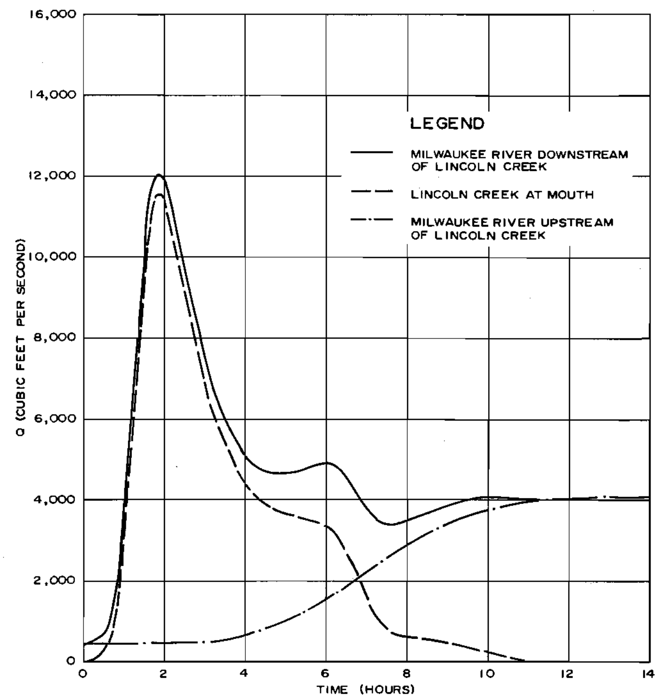
COMPARISON OF FLOOD HYDROGRAPHS FOR MILWAUKEE RIVER AND LINCOLN CREEK FOR A ONE-HOUR DURATION, 100-YEAR RECURRENCE INTERVAL STORM



Source: SEWRPC.

Figure 58

COMPARISON OF FLOOD HYDROGRAPHS FOR MILWAUKEE RIVER AND LINCOLN CREEK FOR A SIX-HOUR DURATION, 100-YEAR RECURRENCE INTERVAL STORM



Source: SEWRPC.

land use and existing channel conditions is 16,400 cfs. Thus, the flow from Lincoln Creek, which can be nearly 14,000 cfs under the recommended channel conditions, could have a significant impact on flood flows in the Milwaukee River.

A review of the timing of the occurrence of peak discharges upstream of Lincoln Creek on the Milwaukee River during major flood-flow events indicated that the earliest peak discharges occur about eight hours after the beginning of a major rainfall event. The peak discharge from the Lincoln Creek subwatershed may be expected generally to occur within three hours of the beginning of a major rainfall event.

The types of storms which may be expected to generate high flows in Lincoln Creek were also reviewed to determine if the flows could be expected to result in a peak flow on the Milwaukee River greater than previously estimated. A comparison of the two synthesized hydrographs shown in Figures 57 and 58 indicates that the peak discharges from Lincoln Creek normally should not be coincident with peak discharges on the main stem of the Milwaukee River. Accordingly, the impacts of the recommended channel improvements on the downstream peak flows should not be significant.

Maximum precipitation storm events with a recurrence interval of 100 years and with varying durations of 1, 2, 4, 5, 6, and 12 hours were evaluated. The analyses indicated that the highest peak rate of flow on the Milwaukee River downstream of Lincoln Creek for these storms—14,500 cfs—may be expected to occur with a one-hour storm, and that the peak rate of flow may be expected to drop as the duration of the rainfall storm event increases. The peak rate of flow on the Milwaukee River downstream of Lincoln Creek was estimated at 8,200 cfs for a 12-hour, 100-year recurrence interval rainfall event. These peak rates of flow compare to the estimated maximum 100-year recurrence interval flood-flow rate of 16,400 cfs that may be expected to be caused by a spring snowmelt condition in the Milwaukee River watershed, the expected critical flood condition on the Milwaukee River. Thus, it may be concluded that the channel improvements recommended for Lincoln Creek should not increase the design flood flows for the Milwaukee River downstream of Lincoln Creek as those flows were used in the preparation of the Milwaukee River watershed plan.

Flood Control and Related Drainage System Plan Implementation

The major floodland management recommendation of the Lincoln Creek flood control plan is the application of structural flood control measures to abate flood problems. It is recommended that the channel modification subelement be implemented expeditiously through the cooperative efforts of the City of Milwaukee, Milwaukee County, and the Milwaukee Metropolitan Sewerage District. More specifically, it is recommended that the District design, construct, and maintain the major channel improvements recommended along Lower Lincoln Creek from the mouth of Lincoln Creek upstream to N. Teutonia Avenue, a distance of 1.3 miles; and from N. 32nd Street upstream to W. Hampton Avenue, a distance of 2.5 miles. It is further recommended that the District design, construct, and maintain the dikes and attendant stormwater pumping stations recommended along Lower Lincoln Creek downstream of N. Green Bay Avenue, a total length of about 1,400 feet; from N. Green Bay Avenue upstream to the confluence with Crestwood Creek and along Crestwood Creek, a total length of about 1,300 feet; and along the west side of the creek between W. Cameron Avenue and the Soo Line Railroad, a total length of about 500 feet.

Along Upper Lincoln Creek, it is recommended that the District design and construct the channel improvements recommended along Upper Lincoln Creek from River Mile 6.67 downstream of the Chicago & North Western Railway line located south of W. Woolworth Avenue to the Chicago & North Western line located just east of N. 60th Street, a distance of about 1.8 miles. It is recommended that the drop structure at the inlet to the W. Good Hope Road culvert be removed when the District makes the channel improvements above this structure. It is recommended that the District remove two houses along N. 51st Street between W. Woolworth Avenue and W. Mill Road. It is further recommended that the District clean out the channel from the drop structure at River Mile 5.79 upstream to the Wisconsin & Southern Railroad, a distance of about 0.5 mile. It is emphasized that in this latter reach, channel enlargement is not called for, but simply the removal of debris and deadfalls, and the removal of live, woody plants smaller than two inches in diameter where they are concentrated in sufficient numbers to significantly impede the flow of floodwaters. Such cleaning should be done carefully to preserve as much as practical of the existing terrestrial and aquatic wildlife habitat and pool and riffle regime in this reach.

It is further recommended that Milwaukee County cooperate fully in the major channel improvements through the provision of attendant construction easements and rights-of-way, modification to the three county-owned pedestrian bridges located at River Miles 2.82, 3.48, and 3.80 to accommodate the proposed lowered channel bottom grade, and replacement of the county-owned pedestrian bridge located at River Mile 0.93.

It is further recommended that the District work with the railroad companies involved in the design and construction of the bridge required to carry the Soo Line Railroad over the recommended channel relocation at River Mile 2.01, and of the replacement bridge or culvert under the Chicago & North Western Railway at River Mile 6.73.

It is recommended that the Wisconsin Department of Natural Resources clean out the channel from the Wisconsin & Southern Railroad upstream through Havenwoods to the Chicago & North Western Railway, also preserving as much as practical of the existing terrestrial and aquatic wildlife habitat and pool and riffle regime.

It is recommended that the District remove the bridges at N. Green Bay Avenue, W. Villard Avenue, N. Teutonia Avenue, W. Glendale Avenue, N. 35th Street, N. 37th Street, N. Sherman Boulevard, W. Woolworth Avenue, N. 51st Street upstream of W. Woolworth Avenue, and W. Green Tree Road, and the pedestrian bridge at River Mile 0.93 to accommodate the recommended channel improvements.

It is recommended that the City of Milwaukee construct replacement bridges at N. Green Bay Avenue, W. Villard Avenue, N. Teutonia Avenue, W. Glendale Avenue, N. 35th Street, N. 37th

Table 76

SUMMARY OF RECOMMENDED PLAN CAPITAL COSTS FOR LINCOLN CREEK SUBWATERSHED

Implementing Agency	Improvements	Estimated Capital Cost
Milwaukee Metropolitan Sewerage District	Channel Improvements	$ 9,502,000
	Dikes and Floodwalls	1,133,000
	Bridge Removal and Replacement[a]	3,672,000
	Pumping Stations	714,000
	Subtotal	$15,021,000
City of Milwaukee	Bridge Replacement or Modification[b]	$ 6,086,000
	Storm Sewer System Improvement	538,000
	Street Regrading	210,000
	Subtotal	$ 6,834,000
Wisconsin Department of Natural Resources	Channel Cleaning and Debrushing	$ 10,000
Total		$21,865,000[c]

[a]*Portions of this cost may be allocated to two railroad companies.*

[b]*Portions of this cost may be allocated to the Wisconsin Department of Transportation.*

[c]*Of the total capital cost of the recommended plan, $21,052,000 may be attributed to flood control, and $813,000 to improved drainage, involving channel deepening to accommodate existing storm sewer outfalls.*

Source: SEWRPC.

Street, N. Sherman Boulevard, W. Woolworth Avenue, N. 51st Street upstream of W. Woolworth Avenue, and W. Green Tree Road; and modify the bridges at N. 51st Street downstream of N. 60th Street and N. 60th Street, all as necessary to provide the required hydraulic capacity and to accommodate channel improvements to be made by the District. The removal and replacement of the N. 37th Street bridge should not be undertaken until downstream channel improvements have been made by the District to accommodate the increased downstream flood flows that will be attendant to the removal of this hydrologically significant structure.

It is similarly recommended that the removal and replacement of the hydrologically significant structure at N. Sherman Boulevard be undertaken only after downstream channel improvements to be made by the District are in place, and only after the N. 37th Street bridge has been replaced. As already noted, the bridge at N. 32nd Street which was recommended to be replaced in the 1982 Lincoln Creek flood control report has already been replaced by the City of Milwaukee.

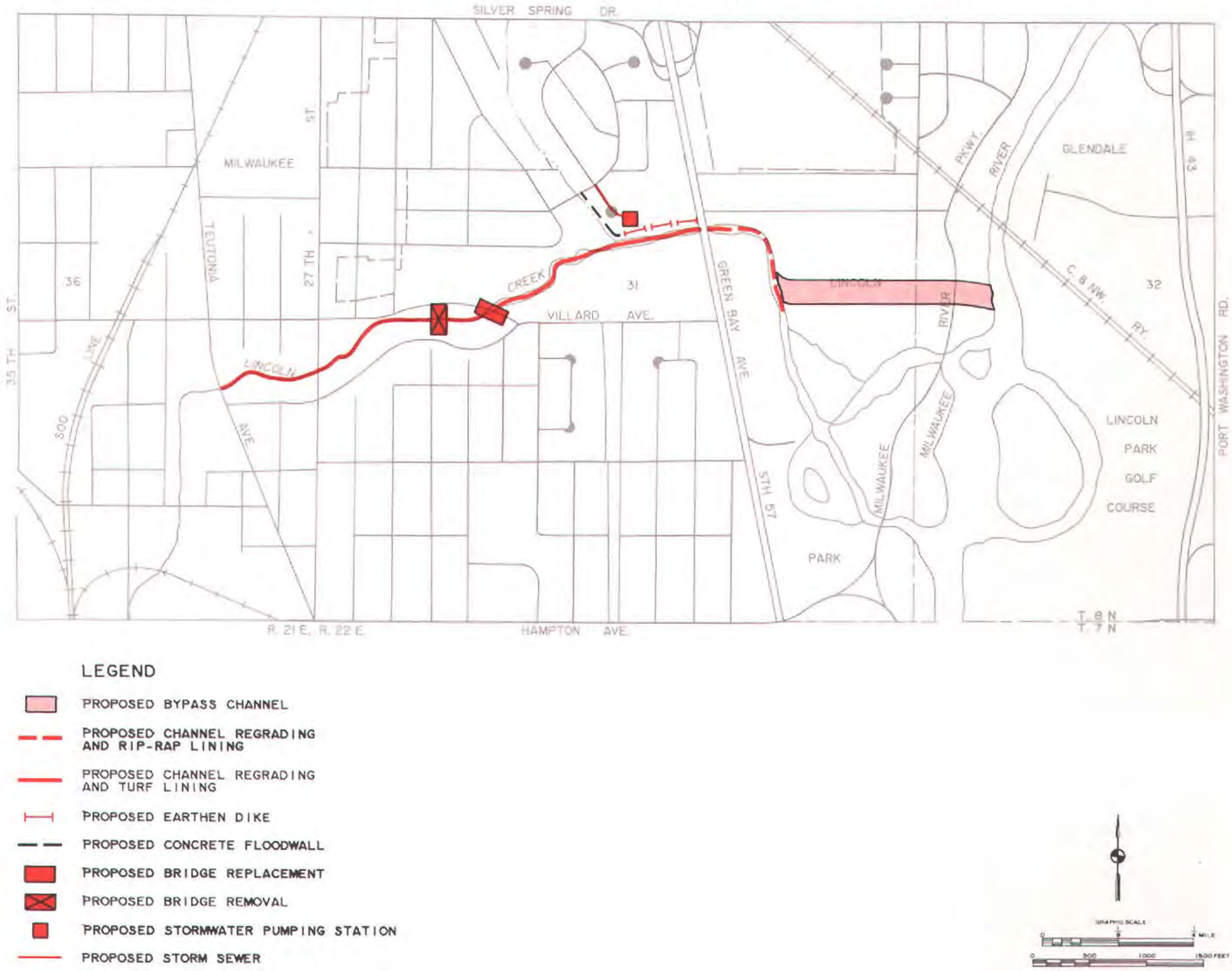
The capital costs associated with the various components of the recommended plan are summarized by agency in Table 76.

Additional Considerations for Recommended Flood Control System Plan for Lincoln Creek in Lincoln Park

Consideration was also given to the construction of a new bypass channel between the existing Lincoln Creek channel and the Milwaukee River parallel to and approximately 800 feet south of W. Lawn Avenue from a point about 800 feet downstream of N. Green Bay Avenue to the main stem of the Milwaukee River, as shown on Map 130. The new channel would be turf lined, with a bottom width of 80 feet, side slopes of one on three, and an average top width of about 170

Map 130

PROPOSED FLOOD CONTROL PLAN FOR LOWER LINCOLN CREEK DOWNSTREAM OF N. TEUTONIA AVENUE WITH BYPASS CHANNEL



Source: SEWRPC.

feet. Construction of the channel would also require the construction of a new bridge to carry N. Milwaukee River Parkway over the channel. The Lincoln Creek channel downstream of the bypass channel would remain in its existing condition. Between the beginning of the bypass channel and N. Green Bay Avenue the Lincoln Creek channel would be widened and deepened, with the enlarged channel having a a bottom width of 80 feet, side slopes of one on three, and an average top width of about 170 feet. The channel would have a rip-rap lining to control erosion. In order to accommodate the new bypass channel, seven residential properties located along N. Milwaukee River Parkway would need to be acquired and removed. An additional seven properties which would be affected by the construction of the channel are owned by Milwaukee County.

The resulting 100-year recurrence interval flood profile downstream of N. Green Bay Avenue under planned land use and channel conditions with the construction of the bypass channel would be from 1.3 to 1.7 feet lower than that which would be expected if a turf-lined channel were constructed along the existing channel alignment. Under these lower stages, flooding of structures along W. Lawn Avenue would not be anticipated, and no appurtenant dikes, floodwalls, or pumping stations would be required. In addition, modification or replacement of the bridge at N. Green Bay Avenue would not be required. The roadway grade for N. Green Bay Avenue would, however, need to be raised 0.5 foot so as to prevent overtopping during a 100-year recurrence interval flood under planned land use and channel conditions. Upstream of N. Green Bay Avenue, the recommended flood control plan would remain unchanged. It should be noted that two guide wire foundations for the WISN radio transmission tower may have to be modified in the final design of this bypass channel.

Incorporation of the bypass channel into the recommended flood control plan would increase the estimated capital cost by $791,000 over the cost of the plan without the channel, excluding modifications to the guide wire foundation for the transmission tower. Annual operation and maintenance costs with the bypass channel would approximate $22,100, a decrease of $5,000 from the recommended plan cost without the channel. On a total annual cost basis, assuming a 6 percent interest rate and a 50-year project life, this modified plan would have a cost of $1,364,000, or about 3 percent more than the cost of the initially recommended plan. However, for the segment of channel involved below N. Green Bay Avenue, this modified plan would result in an increase in total cost of over 50 percent.

The incorporation of this bypass channel is not recommended for the following reasons: 1) the alignment of the bypass channel requiring about a 200-foot-wide area of impact is through an area which is designated by the Wisconsin Department of Natural Resources as a wetland; 2) construction of the channel would result in the removal of 14 privately owned residences, of which seven are presently on the tax roles; and 3) the monetary cost of construction of the bypass channel is more than the cost of the alternative calling for a turf-lined channel and no bypass along the existing channel alignment.

Concluding Remarks

The flooding and related drainage problems in the Lincoln Creek subwatershed are serious and costly. Many hundreds of structures lie in the existing floodplain and are thus subject not only to periodic flood damages, but to federal and state flood insurance and related zoning regulations and requirements. The only feasible and practical way to eliminate such flooding problems and to remove these hundreds of homes and other structures from the floodplain maps under the regulatory requirements of the flood insurance program is to undertake the series of interrelated channel and structure improvements set forth in the recommended Lincoln Creek flood control plan. These improvements, in particular those related to channel deepening, widening, and concrete lining and the construction of related dikes and floodwalls, are recommended by the Commission only as a last resort. There appears to be no other feasible way to resolve the problem.

Urban development in the subwatershed has been permitted to destroy almost all of the wetlands that once provided natural flood-flow regulation on Lincoln Creek. There are no sites available which can provide any significant amount of floodwater detention. A dike and floodwall system alone would not be feasible or cost-effective. Consequently, the only alternative left involves making significant changes to the existing channel by restructuring the creek bed. This is the price that the public, as a whole, must pay for historic lack of foresightedness in protecting the natural floodlands and wetlands of this subwatershed.

BEAVER CREEK SUBWATERSHED FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN

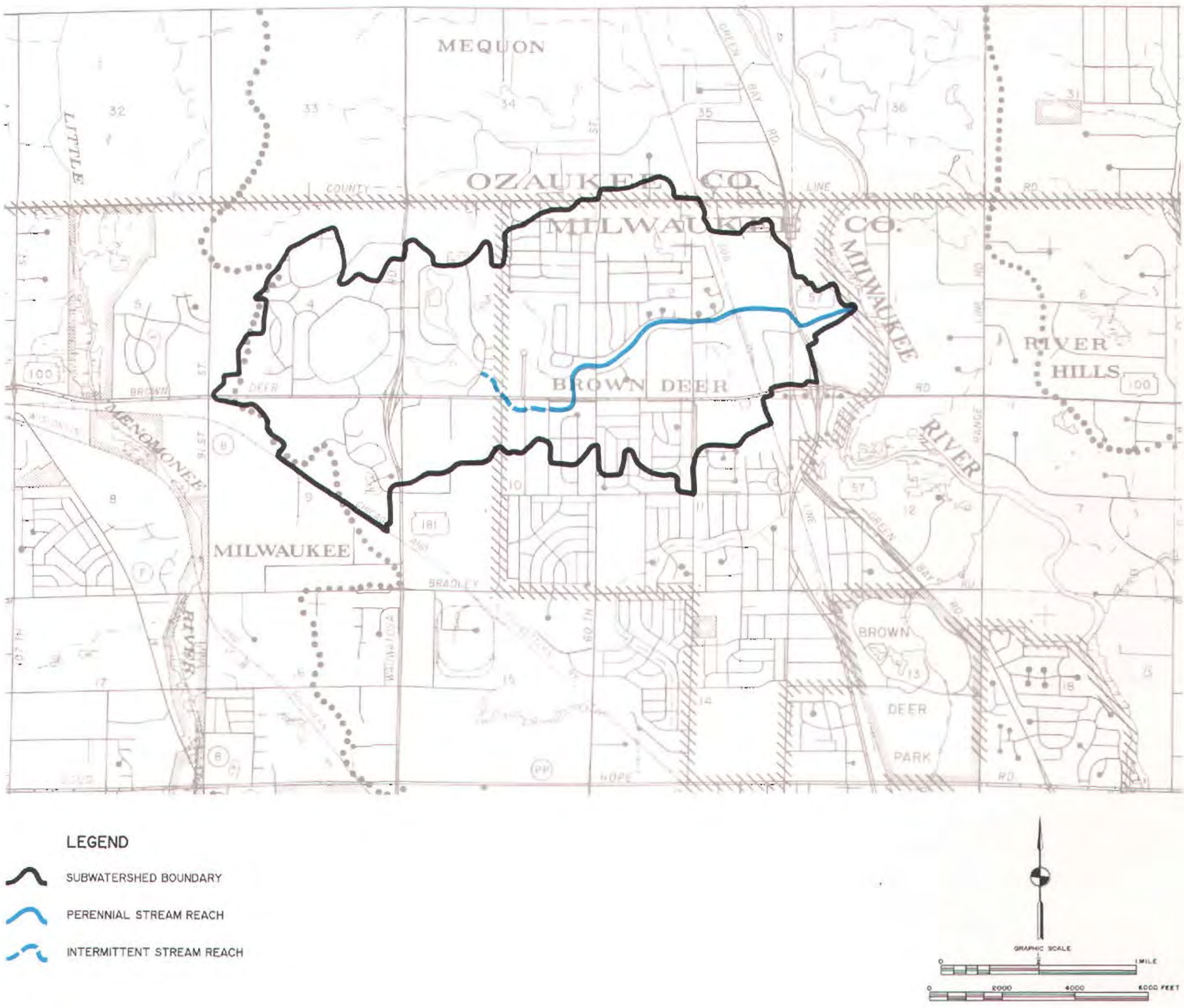
Beaver Creek has not been studied under any previous Commission planning program. Analyses conducted by the Federal Emergency Management Agency for the Village of Brown Deer federal flood insurance study included an approximate delineation of the flood hazard area along Beaver Creek. More detailed analyses of the hydrologic and hydraulic characteristics of the creek and the tributary subwatershed were conducted under this system planning effort.

Overview of the Study Area

Beaver Creek is a tributary of the Milwaukee River. The Beaver Creek subwatershed is located almost equally within the City of Milwaukee and the Village of Brown Deer. From its origin at the outlet of the Northridge Lakes reservoir at N.

Map 131

THE BEAVER CREEK SUBWATERSHED



Source: SEWRPC.

70th Street, Beaver Creek flows in a generally easterly direction for approximately 2.37 miles, and drains an area of about 3.49 square miles (see Map 131). Of this total drainage area, 1.50 square miles, or about 43 percent, lie within the City of Milwaukee, and 1.99 square miles, or about 57 percent, lie within the Village of Brown Deer.

More specifically, from its origin at the outlet from the Northridge Lakes reservoir, Beaver Creek flows in a generally southeasterly direction to the vicinity of N. 66th Street and W. Brown Deer Road. From this point, the creek flows in a generally easterly direction to about N. 62nd Street and W. Brown Deer Road, thence northeasterly to the vicinity of W. Joleno Lane and N. 55th Street extended, and thence easterly to its confluence with the Milwaukee River near the North Suburban YMCA property. Of the 2.37-mile reach described, 1.90 miles, or 80 percent, is classified as perennial, while the remaining 0.47 mile, or 20 percent, is classified as intermittent. The entire perennial stream length is recommended for District jurisdiction in the policy plan companion to this system plan. In addition to the 1.90 miles of stream recommended for District jurisdiction, 0.3 mile of intermittent stream extending to the second W. Brown Deer Road crossing was studied in detail under this system plan. It was decided to include this stream reach in the planning effort since it flows through a residential development and because no flood control measures have been implemented along it. It would therefore be necessary to evaluate the downstream impact of any locally proposed flood control measures along this reach in any case.

By 1985, the Beaver Creek subwatershed was almost completely developed for urban use, including residential, commercial, industrial, institutional, and urban open space uses. Most of the area located in the Village of Brown Deer was developed for residential use, while most of the area in the City of Milwaukee was developed for commercial use. The developed areas of the Beaver Creek subwatershed are generally provided with a full range of municipal street improvements, including paved streets with curbs and gutters and attendant storm sewers, except the residential areas in the Village of Brown Deer which are served by roadside drainage ditches. Accordingly, surface runoff is generally conveyed rapidly from most individual sites to Beaver Creek through storm sewers.

Specific data on pertinent characteristics of the subwatershed, such as hydrologic soil types, land slopes, and land use, appear in Chapter II of this report. The land use conditions utilized in the system planning assume that the subwatershed will be fully urbanized by the design year of the system plan. However, some existing open space uses, such as parks, will remain.

Channel improvements have been made along the entire 1.9-mile perennial stream length as well as along about 0.20 mile of the intermittent stream length to accommodate the increased streamflows. The channel has been physically altered by deepening and straightening, and in some reaches lining with concrete and enclosure in culverts.

Flooding and Related Drainage Problems

An investigation of the historical flood problems along Beaver Creek that was conducted under this system plan indicated few problems. Reported problems were limited to minor flooding of yards and garages along that portion of the creek located between N. 64th Street and N. 68th Street. This lack of any serious flooding problems can be attributed, in part, to the extensive channel modifications that have been implemented along Beaver Creek, as well as to the retention reservoir located within the Northridge Lakes housing development at the headwaters of Beaver Creek. This reservoir greatly reduces the hydrologic impact of the extensive commercial development located along N. Brown Deer Road, west of N. 76th Street.

The results of the hydrologic and hydraulic analyses indicate that no houses may be expected to experience direct flooding along Beaver Creek. Flooding of yards and overtopping of both N. 64th Street and N. 66th Street, however, may be expected along the reach of Beaver Creek between N. 64th Street and the second W. Brown Deer Road crossing at River Mile 2.20. This reach is located upstream of that which is recommended for District jurisdiction under the policy plan companion to this system plan.

No industrial or commercial properties are expected to experience direct damages for floods up to and including the 100-year recurrence interval event under either existing or planned land use. Additional homes and industrial and commercial properties may, however, experience indirect flood damages through sanitary sewer backup. It should be noted that the flood control measures considered under this system plan are primarily intended to alleviate flood damages from direct overland flooding along the stream studied, as well as to provide an adequate outlet for local storm sewers. These measures may help to reduce flooding due to localized stormwater drainage problems or sanitary sewer backups.

Flood Discharges and Stages

As noted in Chapter III of this report, the hydrologic model used for the development of design flood discharges for Beaver Creek uses design rainfall events as input. The design rainfall events were developed using 10-, 50-, and 100-year rainfall volumes obtained from the updated point rainfall depth-duration-frequency relationships developed by the Commission as discussed in Chapter III. The rainfall distribution utilized for each design storm was the median distribution of a first-quartile storm, as shown in

Table 77

COMPARISON OF SIMULATED 100-YEAR RECURRENCE INTERVAL DISCHARGES FOR BEAVER CREEK: WISCONSIN URBAN STREAM REGRESSION EQUATIONS VS. HEC-1 SIMULATION MODEL

River Mile	Location	Discharge Using Regression Equations (cfs)	Discharge Using HEC-1 Model (cfs)	Percent Difference
0.00	At Mouth	1,450	1,110	23
0.92	N. 51st Street	1,310	1,230	6
1.76	W. Brown Deer Road	870	490	44

Source: SEWRPC.

Chapter III. The design storm duration was determined for a given recurrence interval by simulating the peak discharge at a given location for a range of storm durations. The storm duration and associated rainfall volume which produced the largest peak discharge at a given location for a given recurrence interval was selected as the design storm for that location. This analysis was conducted for both existing and planned land use and channel conditions at eight locations on the main stem of Beaver Creek. Because of a lack of recorded high-water data for Beaver Creek, the flood discharges simulated by the hydrologic model were checked by comparison with discharges computed by use of regression equations developed for Milwaukee County by the U. S. Geological Survey under a cooperative program with the Regional Planning Commission and the Metropolitan Sewerage District. As shown in Table 77, the discharges developed with the hydrologic model were lower than those computed using the regression equations, although generally within the standard error of estimate for the equations. Such a result should be expected, since the regression equations do not account for detention storage facilities such as that located at the Northridge Lakes housing development.

The estimated peak flood discharges under existing and planned, year 2000, land use conditions and existing channel conditions selected for use in the system planning are set forth in Table 78. Flood stage profiles were determined for the 10-, 50-, and 100-year recurrence interval runoff events under planned land use and existing channel conditions. These profiles, which encompass the full 2.2-mile-long reach of Beaver Creek studied, constitute a graphic representation of the flood stages along Beaver Creek under the specified recurrence interval flood discharges, and under planned land use and existing channel conditions. In addition to providing an overall representation of flood stages relative to familiar points of reference such as the channel bottom and bridge deck surfaces, the profiles, because they are continuous, permit the determination of flood stages at any point along the stream channel. The flood profiles are shown in Figure 59. The extent of the 100-year recurrence interval floodplain under planned land use conditions is shown on Map 132. This flood hazard area was delineated using large-scale topographic maps prepared in 1964, supplemented with construction plans for the various modifications carried out along the channel.

Alternative Flood Control and Related Drainage System Plans for Beaver Creek

As previously noted, no structure flood damages are anticipated along Beaver Creek. Flooding of yards and overtopping of both N. 64th Street and 66th Street may be anticipated along the reach between N. 64th Street and the second W. Brown Deer Road crossing, however. The Village has considered rezoning the lands located along the south side of W. Brown Deer Road between

Table 78

FLOOD DISCHARGES FOR BEAVER CREEK FOR EXISTING AND YEAR 2000 LAND USE AND EXISTING CHANNEL CONDITIONS

Location	River Mile	Peak Flood Discharge (cfs): Existing Land Use, Existing Channel Conditions 10-Year	50-Year	100-Year	Year 2000 Planned Land Use, Existing Channel Conditions 10-Year	50-Year	100-Year
Mouth at Milwaukee River	0.00	580	970	1,110	640	1,040	1,180
Pedestrian Bridge	0.10	580	970	1,110	640	1,040	1,180
N. Green Bay Road (STH 57) Outlet	0.18	560	930	1,070	620	1,010	1,140
N. Green Bay Road (STH 57) Inlet	0.33	560	930	1,070	620	1,010	1,140
Utility Road	0.67	550	950	1,090	620	1,010	1,130
Wisconsin Central Railroad	0.69	550	950	1,090	620	1,010	1,130
N. 51st Street	0.92	530	1,020	1,230	590	1,090	1,370
N. 60th Street	1.50	360	670	860	410	770	970
W. Brown Deer Road	1.76	220	390	490	260	460	580
N. 64th Street	1.93	190	340	420	230	400	500
N. 66th Street	2.06	190	340	420	230	400	500

Source: SEWRPC.

N. 64th Street and N. 68th Street from residential to commercial use. Therefore, the Village has expressed interest in improving the drainage characteristics of Beaver Creek along this reach. Accordingly, in the preparation of this system plan, three alternative flood control plans were considered for alleviating the flood problems along Beaver Creek, and improving the drainage characteristics of the creek: 1) Alternative Plan 1—no action; 2) Alternative Plan 2—culvert replacement; and 3) Alternative Plan 3—combination of culvert replacement and channel modification.

Each alternative is described below. The estimated capital and operation and maintenance costs attendant to each alternative are provided in Table 79.

Alternative Plan 1—No Action: One alternative course of action for handling the flood problem along Beaver Creek is to do nothing—that is, to recognize the inevitability of flooding but to deliberately decide not to mount a collective, coordinated program to abate the problem.

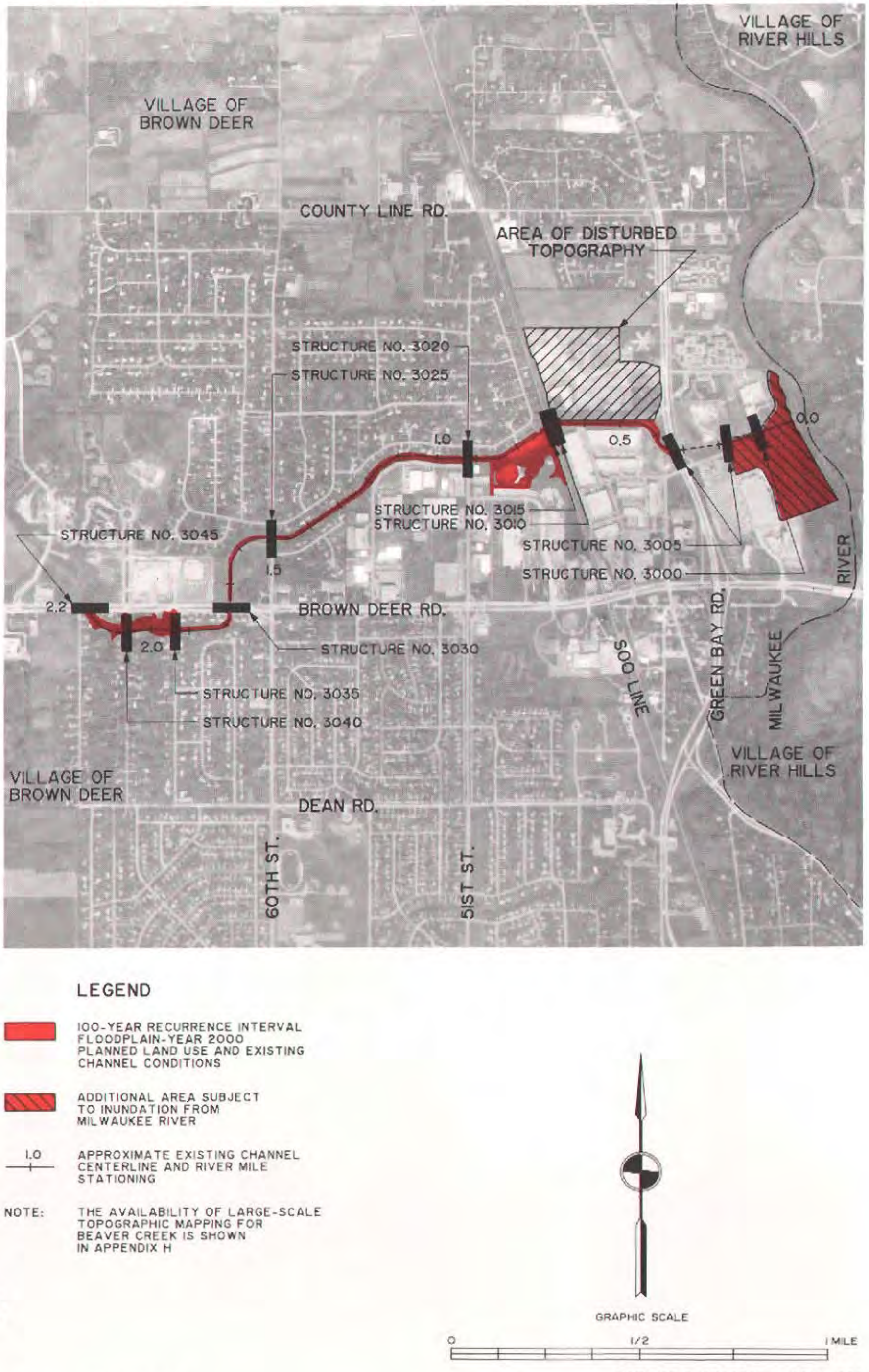
Alternative Plan 2—Culvert Replacement: This alternative plan is shown on Map 133, and consists of replacing the existing culverts at two road crossings—N. 64th Street at River Mile 1.93 and N. 66th Street at River Mile 2.06. In 1987 the road crossing at N. 64th Street consisted of a single corrugated metal pipe arch that was 7 feet 11 inches wide by 5 feet 7 inches high, while the crossing at N. 66th Street consisted of a single 8-foot-wide by 5-foot-high reinforced concrete box culvert. Under this alternative, each of these two structures would be replaced by two 8-foot-wide by 6-foot-high reinforced concrete box culverts.

Utilizing an annual interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this alternative is estimated at $6,300. This cost consists of the amortization of the $99,000 capital cost for culvert replacement.

Alternative Plan 3—Combination of Culvert Replacement and Channel Modification: This alternative is shown on Map 134 and consists of replacing the existing culverts at both N. 64th Street and N. 66th Street, and the modification of the associated channel reach. Each of the two existing structures would be replaced by two 8-foot-wide by 6-foot-high reinforced concrete box

Map 132

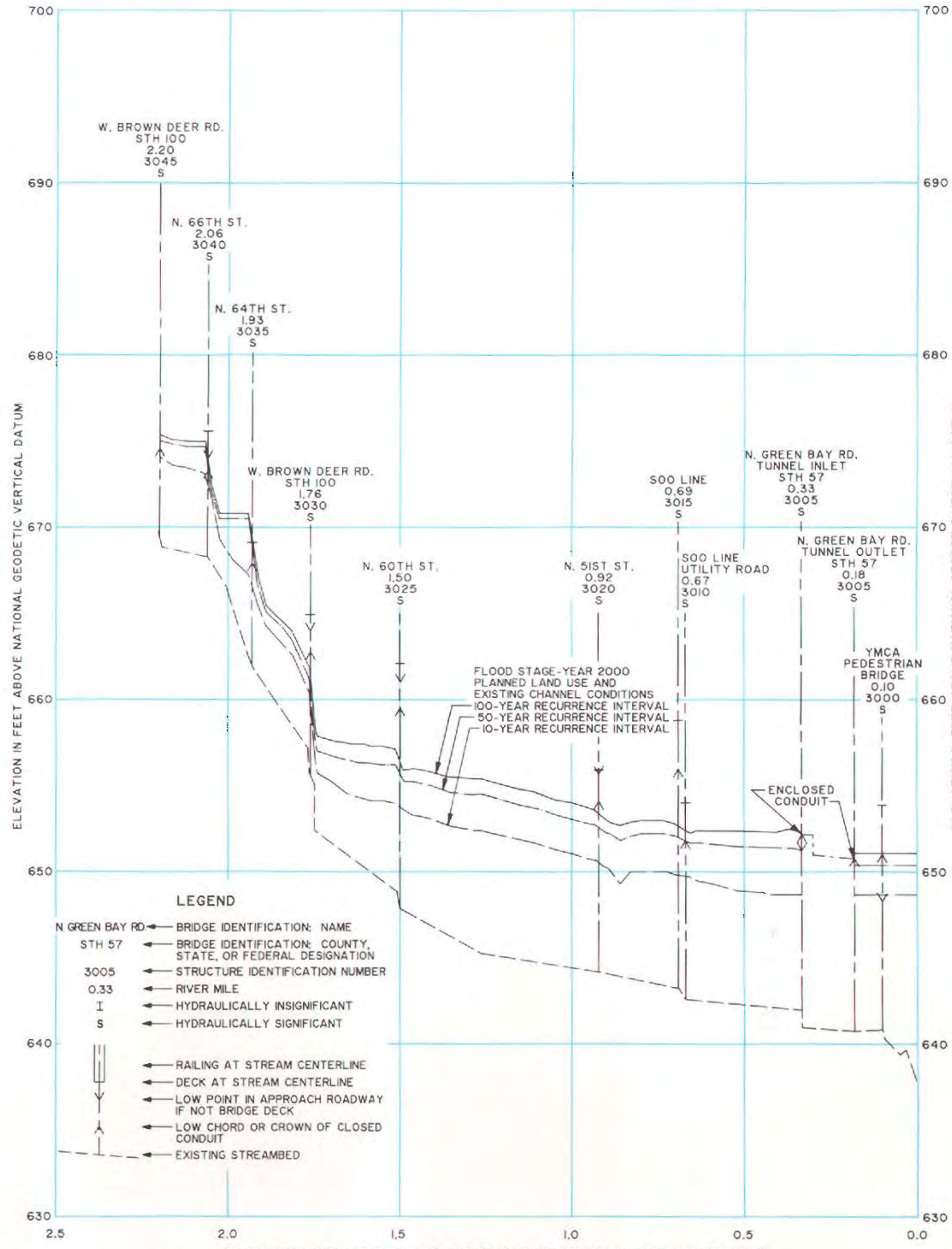
100-YEAR RECURRENCE INTERVAL FLOODPLAIN FOR BEAVER CREEK
UNDER YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS



Source: SEWRPC.

Figure 59

FLOOD STAGE AND STREAMBED PROFILE FOR BEAVER CREEK



Source: SEWRPC.

Table 79

COST ESTIMATES FOR FLOOD CONTROL ALTERNATIVES FOR BEAVER CREEK IN THE VILLAGE OF BROWN DEER

Alternative	Description	Costs				
		Capital	Annual			
			Amortized Capital[a]	Operation and Maintenance	Other	Total
1. No Action	- -	$ 0	$ 0	$ 0	$ 0	$ 0
2. Culvert Replacement	Replace culverts at two road crossings	99,000	6,300	0	0	6,300
3. Combination of Culvert Replacement and Channel Modification	Replace culverts at two road crossings	99,000	13,000	600	0	13,600
	Modify 0.27 mile of channel	103,300				
	Subtotal	$202,300				

[a]*Amortized capital cost is based on an interest rate of 6 percent and a project life of 50 years.*

Source: SEWRPC.

culverts. Channel modifications would be carried out along the 0.27-mile reach of Beaver Creek between N. 64th Street and the W. Brown Deer Road crossing at River Mile 2.20. The existing channel invert would be lowered from 0.1 foot to 3.0 feet within this reach. The proposed channel would have a bottom width of five feet and side slopes of one on two. The resulting top width of the channel would average 25 feet. Due to the high channel velocities within this reach, a riprap channel lining would be required.

Utilizing an annual interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this alternative is estimated at $13,600. This cost consists of the amortization of the $99,000 capital cost of the culvert replacement, the $103,300 capital cost of the channel modification, and $600 in annual operation and maintenance costs.

Evaluation of Flood Control Alternatives for Beaver Creek

The principal features, and costs, of each of the floodland management alternatives considered for Beaver Creek are summarized in Table 79. Excluding the no action alternative, all of the alternatives were found to be technically feasible.

The no action alternative, while offering the lowest cost, does nothing to alleviate the existing flood problem, and therefore does not represent a sound, long-term approach to flood control.

Implementation of either Alternative Plan 2—culvert replacement—or Alternative Plan 3—combination of culvert replacement and channel modification—would eliminate overtopping of both N. 64th Street and N. 66th Street during floods up to and including the 100-year recurrence interval event under planned, year 2000,

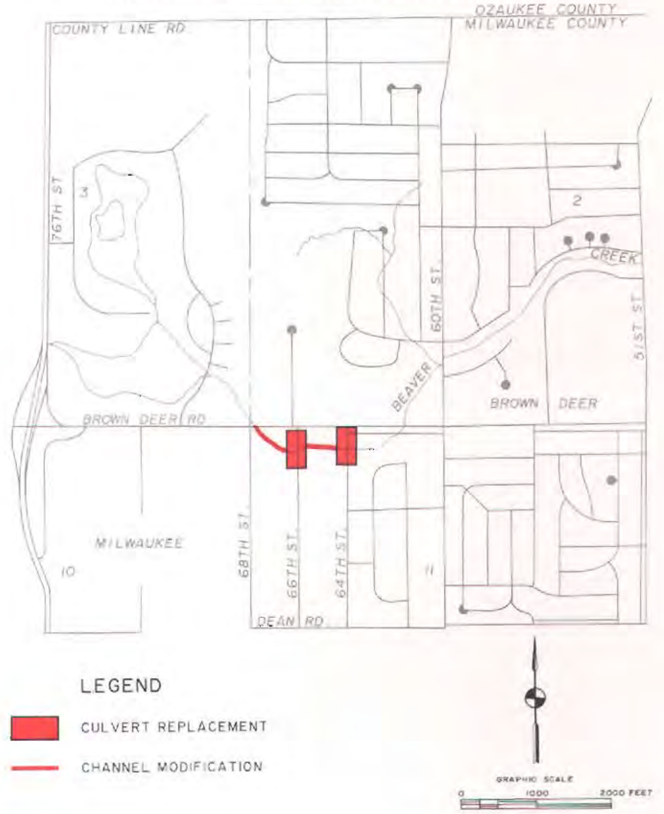
Map 133

ALTERNATIVE PLAN 2: CULVERT REPLACEMENT ALONG BEAVER CREEK

Source: SEWRPC.

Map 134

ALTERNATIVE PLAN 3: COMBINATION OF CULVERT REPLACEMENT AND CHANNEL MODIFICATION ALONG BEAVER CREEK



Source: SEWRPC.

land use conditions. Implementation of Alternative Plan 3 would also eliminate most overland flooding between N. 64th Street and the second W. Brown Deer Road crossing.

Recommended Flood Control System for Beaver Creek

At the Technical Advisory Committee meeting held on January 20, 1988, the representative of the Village of Brown Deer indicated that the Village favored Alternative Plan 3, as it would provide a drainage system adequate to serve proposed land use development in the vicinity of the creek—primarily, the rapidly developing commercial areas along W. Brown Deer Road. Since the reach under consideration is not under Milwaukee Metropolitan Sewerage District jurisdiction and thus is a local responsibility, the Advisory Committee recommended the locally preferred alternative.

The total capital cost of the recommended flood control plan was estimated at $203,000 in 1986 dollars. The recommended plan is shown on Map 135. The peak flood profile that would be attendant to planned land use and planned channel conditions is shown in Figure 60. The recommended plan should have no significant impact on flood flows and stages along Beaver Creek downstream of N. 64th Street.

The recommended plan consists of replacing the existing culverts at N. 64th Street and at N. 66th Street, with each replacement consisting of two 8-foot-wide by 6-foot-high reinforced concrete box culverts. In addition, channel modifications would be carried out along the 0.27-mile reach between N. 64th Street and the W. Brown Deer crossing at River Mile 2.20. The existing channel invert would be lowered from 0.1 foot to 3.0 feet within this reach. The proposed channel would have a bottom width of five feet and side slopes

Map 135

RECOMMENDED FLOOD CONTROL SYSTEM PLAN FOR BEAVER CREEK

LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS

PROPOSED BRIDGE REPLACEMENT

CHANNEL MODIFICATION

1.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR BEAVER CREEK IS SHOWN IN APPENDIX H

NOTE: DUE TO MAP SCALE LIMITATIONS, THE DIFFERENCE BETWEEN THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS, AND THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS, MAY NOT APPEAR ON THIS MAP. WHERE NO DIFFERENCE APPEARS REFERENCE SHOULD BE MADE TO THE FLOOD STAGE PROFILE SHOWN BELOW

GRAPHIC SCALE

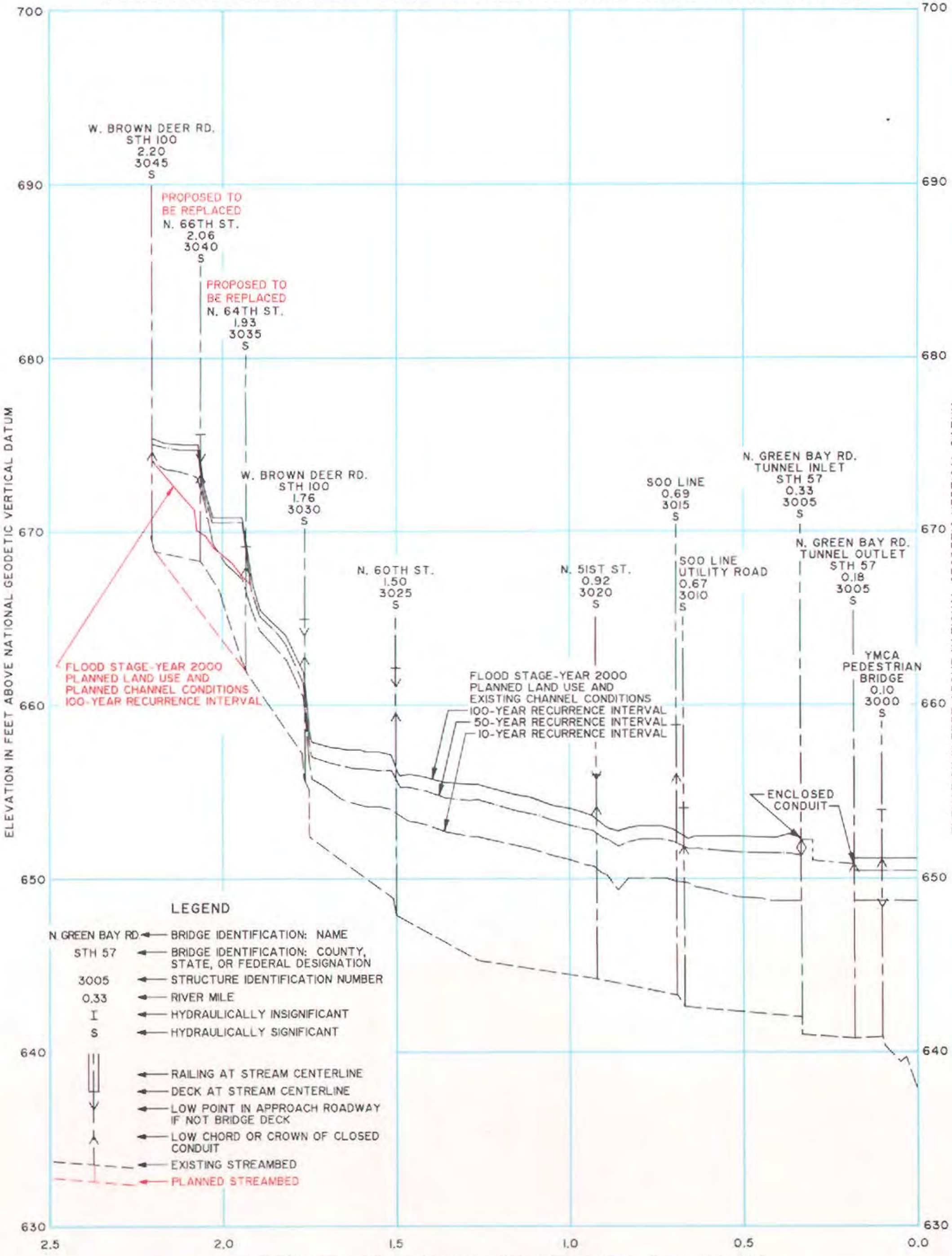
0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Source: SEWRPC.

Figure 60

RECOMMENDED PLAN FLOOD STAGE PROFILE FOR BEAVER CREEK



Source: SEWRPC.

of one on two. Because of the high channel velocities, a rip-rap channel lining would be required along this reach.

In addition to the above measures, it is recommended that new large-scale topographic maps be prepared for the area along Beaver Creek. The large-scale topographic maps currently available from the Village of Brown Deer were prepared in 1964, and do not reflect the significant amount of development that has occurred since then, including major channel modifications and realignment of Beaver Creek. Although the 100-year recurrence interval flood flow is contained within the channel for much of its length, there are two reaches where the flow is expected to exceed the bank-full capacity of the channel. These reaches are between N. Green Bay Road and the west end of the Village Park, and between N. 64th Street and the W. Brown Deer Road crossing at River Mile 2.20. New topographic maps are needed to adequately delineate the limits of the floodplain along these reaches. Since these new maps would serve multiple purposes, none of the attendant costs have been assigned to the flood control plan.

Flood Control and Related Drainage System Plan Implementation

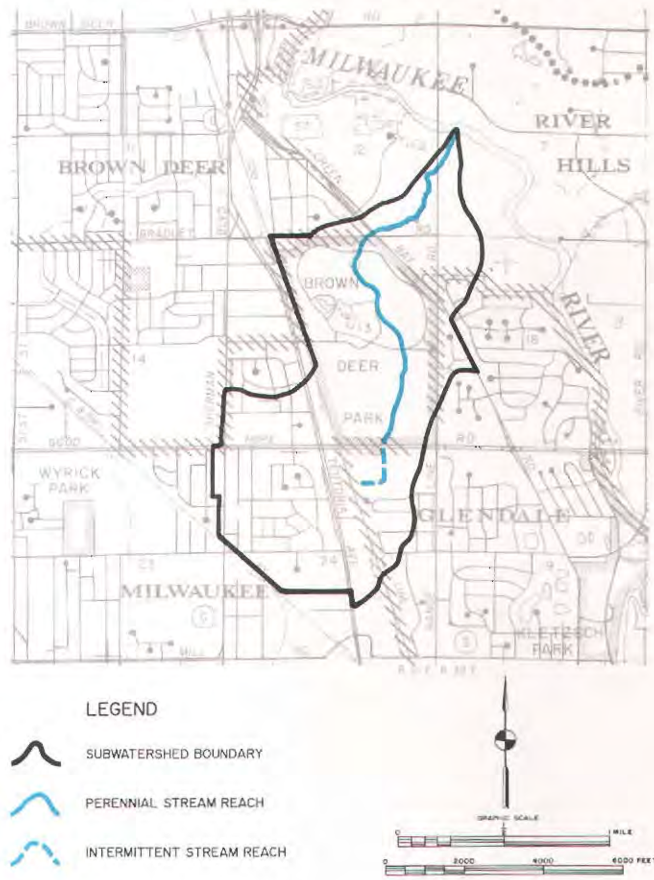
It is recommended that the structural measures developed for the abatement of drainage and flood problems along Beaver Creek be implemented by the Village of Brown Deer. More specifically, it is recommended that the Village design, construct, and maintain the channel modifications recommended along the 0.27-mile reach between N. 64th Street and the W. Brown Deer Road crossing at River Mile 2.20, including the replacement of the existing culverts at N. 64th Street and N. 66th Street. It is further recommended that the Milwaukee Metropolitan Sewerage District prepare new large-scale topographic maps for the area along Beaver Creek.

BROWN DEER PARK CREEK (UNNAMED TRIBUTARY NO. 2) SUBWATERSHED FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN

Unnamed Tributary No. 2, herein referred to by the unofficial name of Brown Deer Park Creek, has not been studied under any previous Commission planning program. Primary analyses of the hydrologic and hydraulic characteristics of the stream and the tributary subwatershed were accordingly conducted under this system planning effort.

Map 136

THE BROWN DEER PARK CREEK SUBWATERSHED



Source: SEWRPC.

Overview of the Study Area

Brown Deer Park Creek is a tributary of the Milwaukee River. The Brown Deer Park Creek subwatershed is located primarily within the City of Milwaukee. Small portions of the subwatershed are located in the City of Glendale and the Villages of River Hills and Brown Deer. From its origin at a storm sewer outlet located east of the Wisconsin Central Railroad and northwest of W. Rochelle Avenue in the City of Glendale, the stream flows in a generally northerly direction for approximately 2.25 miles, and drains an area of about 1.64 square miles (see Map 136). Of this total drainage area, 1.17 square miles, or about 71 percent, lie within the City of Milwaukee; 0.20 square mile, or about 13 percent, lies within the

Village of River Hills; 0.24 square mile, or about 14 percent, lies within the City of Glendale; and 0.03 square mile, or about 2 percent, lies within the Village of Brown Deer.

More specifically, from its origin at the storm sewer outlet, Brown Deer Park Creek flows about 1.51 miles in a generally northerly direction, passing under W. Good Hope Road, Brown Deer Park Drive at two locations, W. Bradley Road, and N. Green Bay Road. From N. Green Bay Road, the stream flows about 0.28 mile in a generally easterly direction to a private dam. From the dam, the stream flows about 0.46 mile in a generally northerly direction, passing under a private drive and N. Range Line Road before reaching its confluence with the Milwaukee River in the area of the Milwaukee Country Club.

Of the 2.25-mile reach described, 1.95 miles, or about 87 percent, is classified as perennial, while the remaining 0.30 mile, or 13 percent, is classified as intermittent. The entire perennial stream length is recommended for District jurisdiction in the policy plan companion to this system plan. The 0.30 mile of intermittent stream consists of two sections of concrete-lined channel separated by a double box culvert. The lined channel and culvert were constructed by the City of Milwaukee. The reach of intermittent stream does not have significant monetary flood damage risk, and the channel improvements in the reach were not constructed by the Milwaukee Metropolitan Sewerage District. Therefore, the 0.30-mile-long reach of intermittent stream does not meet two of the three criteria for establishing District jurisdiction, and the reach was not recommended for District jurisdiction in the policy plan. Only the perennial stream reach was studied under this system plan.

Consideration was also given to the impacts of stormwater drainage improvements proposed by Milwaukee County for the Brown Deer Park golf course. As discussed below, stormwater runoff which flows through the golf course from the west frequently causes disruption to play on the course and damage to trees, with attendant monetary damages. Drainage improvements made to the golf course could have an impact on the flows of Brown Deer Park Creek. Accordingly, the proposed improvements were considered in this study.

At present, that portion of the Brown Deer Park Creek subwatershed within the Cities of Milwaukee and Glendale, with the exception of Brown Deer Park, is almost completely developed for urban use, including residential, commercial, industrial, governmental, institutional, recreational, and urban open space uses. That portion of the subwatershed within the Villages of Brown Deer and River Hills was primarily in low-density residential and recreational uses, with other, less prevalent uses including woodlands, open lands, and governmental and institutional uses. The more intensely developed areas of the subwatershed are located in the upland areas. Those areas are generally provided with a full range of municipal street improvements, including paved streets with curbs and gutters and attendant storm sewers. Accordingly, surface runoff is generally conveyed rapidly from those areas to Brown Deer Park Creek. Runoff from the downstream areas in Brown Deer Park and the Villages of Brown Deer and River Hills generally reaches the stream through natural drainageways or roadside drainage ditches, which convey flow less rapidly than storm sewers. Specific data on pertinent characteristics of the subwatershed, such as hydrologic soil types, land slopes, and land use, appear in Chapter II of this report. The land use conditions utilized in the system planning assume that the subwatershed will be fully developed by the design year of the system plan, consistent with existing uses and with the regional land use plan. Existing open space uses, such as parks, will remain.

As already noted, channel improvements, including lining with concrete and enclosure in culverts, have been made along the entire 0.30-mile-long intermittent stream reach. The perennial stream is essentially in a natural state, although the 0.51-mile-long reach within the Brown Deer Park golf course was deepened by removing accumulated sediment in 1986. Prior to that channel deepening, three low-head wooden dams were maintained across the stream by Milwaukee County. Presently, only the uppermost dam is in place to create a pond to trap sediment entering the golf course. The County has preliminary plans to replace one remaining dam, to replace one of the two dams that have been removed, and to construct two additional dams at new sites. Thus, a total of four dams would exist on the golf course. In addition, a low-head concrete dam spans the stream, creating a small pond in the reach downstream from Green Bay Road.

Flooding and Related Drainage Problems

The investigations of historical flood problems along Brown Deer Park Creek that were conducted under this system plan indicated few problems. This lack of any serious flooding problem can be attributed, in part, to the location of parkland, golf courses, and low-density residential property along all of the perennial stream reach. Overbank storage in Brown Deer Park attenuates peak flows from the upstream developed areas and reduces downstream flooding impacts. It should be noted, however, that there is a stormwater drainage problem on the Brown Deer Park golf course associated with frequent stormwater discharge from a culvert located about 600 feet north of Good Hope Road under the Wisconsin Central Railroad, which borders the golf course along the west side. Stormwater discharge from this culvert flows overland across the golf course to Brown Deer Park Creek, and occupies surface depressions in the golf course terrain. The remaining standing water results in long-term disruption in play and damage to trees located in the flooded areas.

The results of the hydrologic and hydraulic analyses indicate that during the 100-year recurrence interval flood under either existing or planned, year 2000, land use conditions, there should be no first floor flooding of any residential, industrial, or commercial properties along Brown Deer Park Creek. There could be direct basement flooding at one residence along the stream between N. Green Bay Road and W. Bradley Road. There would be no direct basement or first floor flooding of any residential, industrial, or commercial properties along the stream under a 50-year recurrence interval flood and either existing or planned, year 2000, land use conditions.

The total average annual flood losses—damages—for Brown Deer Park Creek are estimated at $90 under both existing and planned, year 2000, land use and existing channel conditions. Flood losses from a 100-year recurrence interval event are estimated at $7,400 under existing land use and channel conditions, and at $7,500 under planned, year 2000, land use and existing channel conditions.

It should be noted that the flood control measures considered under this system plan are primarily intended to alleviate flood damages from direct overland flooding along the stream studied, as well as to provide an adequate outlet for local storm sewers. These measures may help to reduce flooding due to localized stormwater drainage problems or sanitary sewer backups.

The drainage and flood control objectives and supporting principles and standards set forth in Chapter III specify the flood events that bridges shall accommodate without overtopping the related roadway. Based on those criteria, the bridges across Brown Deer Park Creek at N. Range Line Road, N. Green Bay Road, W. Bradley Road, and W. Good Hope Road should all be able to accommodate the 50-year recurrence interval flood event under planned, year 2000, land use conditions. As shown in Appendix C, the only bridges that presently meet this criterion are at N. Green Bay Road and W. Good Hope Road. Although the N. Green Bay Road bridge meets the hydraulic capacity criterion, it would cause about 1.3 feet of backwater during the 50-year flood under planned, year 2000, land use conditions. It may not be practical to achieve the required hydraulic capacity at the N. Bradley Road bridge, which is only 130 feet upstream of N. Green Bay Road, without some corresponding increase in capacity at N. Green Bay Road. It is recommended that when the N. Range Line Road and W. Bradley Road bridges are modified or replaced by local or state highway agencies as part of highway improvement programs, the crossings be designed to provide adequate hydraulic capacity in accordance with recommended standards. It is also recommended that, if possible, replacement of the N. Green Bay Road and W. Bradley Road bridges be coordinated to enable the practical attainment of adequate hydraulic capacity at W. Bradley Road.

The location and design of all new bridges and culverts, as well as the design of replacements of or modifications to existing bridges or culverts, should be based upon the applicable objectives and standards as set forth in Chapter III of this report. Of particular importance is the standard requiring all new or replacement bridges and culverts to be designed to accommodate the 100-year recurrence interval peak flood discharge under planned, year 2000, land use

Table 80

FLOOD DISCHARGES FOR BROWN DEER PARK CREEK FOR EXISTING AND YEAR 2000 LAND USE AND EXISTING CHANNEL CONDITIONS

Location	River Mile	Peak Flood Discharge (cfs)					
		Existing Land Use, Existing Channel Conditions			Year 2000 Planned Land Use, Existing Channel Conditions		
		10-Year	50-Year	100-Year	10-Year	50-Year	100-Year
Mouth at Milwaukee River	0.00	350	520	560	360	520	580
N. Range Line Road	0.19	350	520	560	360	520	580
Private Drive	0.28	350	550	640	360	560	650
Private Dam	0.46	320	510	600	330	520	610
N. Green Bay Road	0.75	330	530	640	340	550	650
W. Bradley Road	0.78	330	530	640	340	550	650
Brown Deer Park Drive (north)	0.88	330	540	640	340	550	650
Brown Deer Park	1.27	330	540	650	350	560	670
Brown Deer Park	1.36	400	640	760	420	670	790
Brown Deer Park Drive (south)	1.44	480	750	880	500	780	910
Brown Deer Park Golf Course Dam (middle)	1.73	480	700	810	510	730	850
Brown Deer Park Golf Course	1.81	510	760	900	540	800	940
Brown Deer Park Golf Course Dam (south)	1.86	530	810	950	560	840	990
W. Good Hope Road	1.95	550	860	1,020	580	900	1,060

Source: SEWRPC.

conditions without raising the corresponding peak flood stage by 0.01 foot or more above the peak stage established in the adopted drainage and flood control system plan. This provision is intended to ensure that new, modified, or replacement river crossings, including their approaches, will not aggravate existing flood problems, create new flood hazards, or unnecessarily complicate the administration of floodland regulations.

Flood Discharges and Stages

As noted in Chapter III of this report, the hydrologic model used for developing design flood discharges for Brown Deer Park Creek uses design rainfall events as input. The design rainfall events were developed using 10-, 50-, and 100-year rainfall volumes obtained from the updated point rainfall depth-duration-frequency relationships developed by the Commission as described in Chapter III. The rainfall distribution utilized for each design storm was the median distribution of a first-quartile storm, as shown in Chapter III. The design storm duration was determined for a given recurrence interval by simulating the peak discharge at a given location for a range of storm durations. The storm duration and associated rainfall volume that produced the largest peak discharge at a given location for a given recurrence interval was selected as the design storm for that location. This analysis was conducted for both existing and planned land use and channel conditions at eight locations on the main stem of Brown Deer Park Creek.

The estimated peak flood discharges under existing and planned, year 2000, land use conditions and existing channel conditions are set forth in Table 80. Upstream of the private drive at River Mile 0.28, there are some reaches where peak flood flows actually decrease in the downstream direction. Those decreases are due to the effects of overbank storage along the stream and to storage in the pond impounded by the private dam at River Mile 0.46. The effects of overbank storage are most pronounced in Brown Deer Park between River Miles 0.88 and

1.95. The decreases in flows for the 50- and 100-year recurrence interval floods at N. Range Line Road are due to some flow being diverted directly to the Milwaukee River by passing over a natural ridge located to the north of Brown Deer Park Creek and upstream of N. Range Line Road.

Flood stage profiles were determined for the 10-, 50-, and 100-year recurrence interval runoff events under planned land use and existing channel conditions. These profiles, which encompass the full 1.95-mile-long reach of Brown Deer Park Creek studied, constitute a graphic representation of the flood stages along the stream under the specified recurrence interval flood discharges, and under planned land use and existing channel conditions. In addition to providing an overall representation of flood stages relative to familiar points of reference such as the channel bottom and bridge deck surfaces, the profiles, because they are continuous, permit the determination of flood stages at any point along the stream channel. The flood profiles are shown in Figure 61. Water surface profiles for the 10-, 50-, and 100-year recurrence interval floods were determined with and without the dams in Brown Deer Park. It was found that the dams would be completely submerged during such floods, and that the water surface profiles would be the same with or without the dams in place.

The accuracy of the hydrologic and hydraulic models was checked using high-water marks along Brown Deer Park Creek from the flood of September 11, 1986. That is the only known large flood for which high-water-mark observations were available. That flood was caused by heavy rains, with a peak intensity of 2.90 inches in four hours falling on saturated ground which had received 2.31 inches of rain in the preceding 33 hours. The recorded rainfall amounts were input to the hydrologic model to simulate the resultant flood hydrographs along the stream. The peak flows from that simulation were then input to the hydraulic model and appropriate adjustments were made to approximate the September 11, 1986, flood profile based on the high-water marks.

The extent of the 100-year recurrence interval floodplain under planned land use conditions is shown on Map 137. The flood hazard area in the reach from the mouth of the stream to W. Bradley Road (River Mile 0.78) was delineated using large-scale topographic maps prepared in 1970. The delineation of the flood hazard area in the reach from W. Bradley Road to Brown Deer Park Drive (south) was based on stream cross-section and hydraulic structure field surveys completed in 1987. The delineation of the flood hazard area in that reach is only approximate owing to the lack of large-scale topographic mapping. Upstream of Brown Deer Park Drive (south), the flood hazard area was delineated using large-scale topographic mapping obtained by Milwaukee County in 1987.

Alternative Flood Control
and Related Drainage System
Plans for Brown Deer Park Creek

As previously noted, anticipated flood damages along Brown Deer Park Creek are relatively minor, being limited to one existing home.

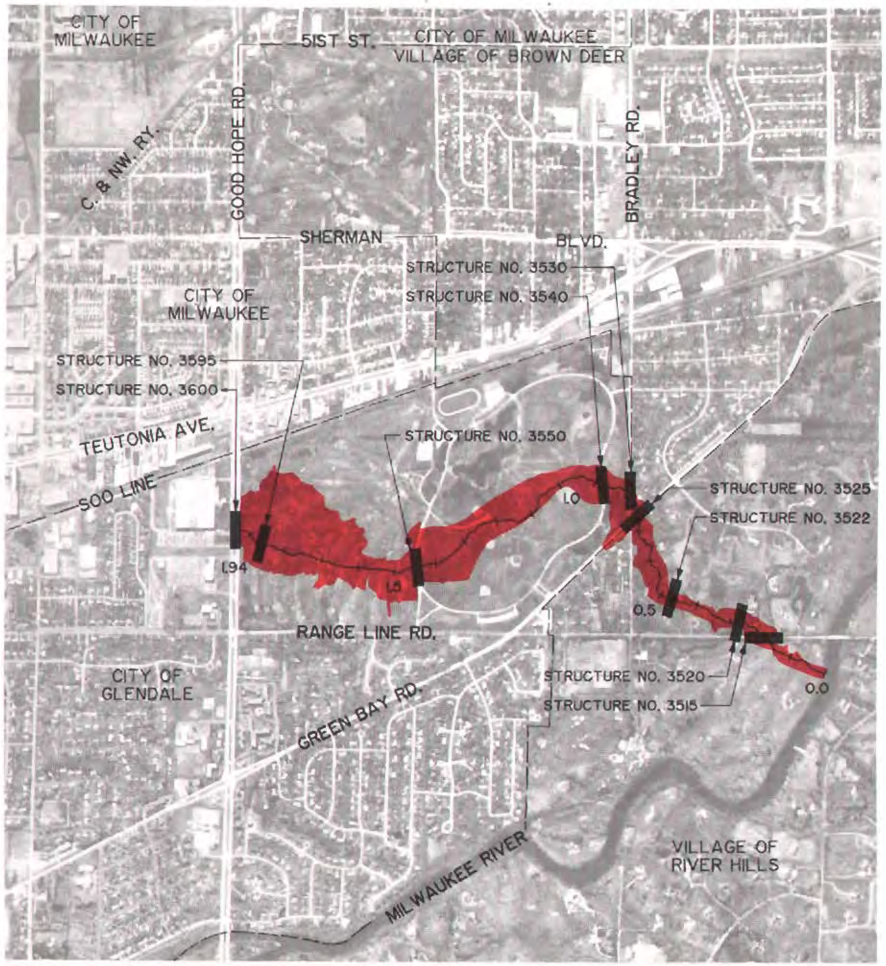
In the preparation of this system plan, three alternative flood control plans were considered for alleviating the flood damage problem along Brown Deer Park Creek: 1) Alternative Plan 1—no action; 2) Alternative Plan 2—structure floodproofing; and 3) Alternative Plan 3—culvert replacement. Each alternative is described below.

Alternative Plan 1—No Action: One alternative course of action that could be taken in response to the flood problem along Brown Deer Park Creek is to do nothing—that is, to recognize the inevitability of flooding but to decide not to mount a collective, coordinated program to abate the flood damages. Under planned, year 2000, land use and existing channel conditions, the average annual flood damages along this reach would approximate \$90. There are no monetary benefits associated with this alternative, and the average annual cost would be equivalent to the average annual flood damage cost of \$90. The estimated damages associated with the 100-year recurrence interval flood under planned, year 2000, land use and existing channel conditions are \$7,500.

Alternative Plan 2—Structure Floodproofing: A structure floodproofing alternative flood control plan was analyzed to determine if such an approach would be a technically feasible and economically viable solution to the flood problem along Brown Deer Park Creek. For analytical purposes, the 100-year recurrence interval flood stage under planned, year 2000, land use and existing channel conditions was used to estimate the number of existing flood-prone structures to

Map 137

100-YEAR RECURRENCE INTERVAL FLOODPLAIN FOR BROWN DEER PARK CREEK UNDER YEAR 2000 PLANNED LAND USE WITH EXISTING CHANNEL CONDITIONS



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

1.0 — APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR BROWN DEER PARK CREEK IS SHOWN IN APPENDIX H

GRAPHIC SCALE

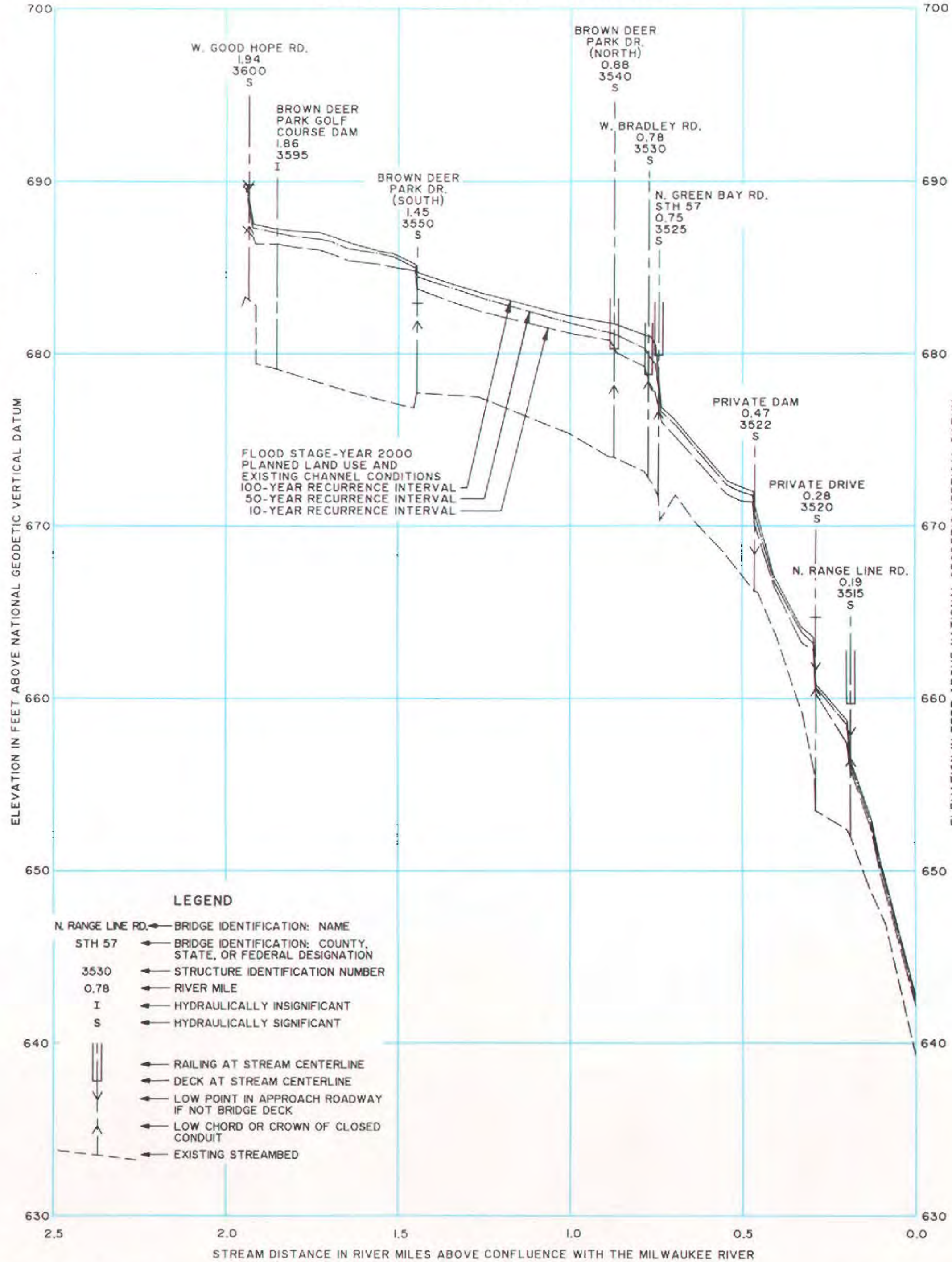
0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Source: SEWRPC.

Figure 61

FLOOD STAGE AND STREAMBED PROFILE FOR BROWN DEER PARK CREEK



Source: SEWRPC.

be floodproofed, and the approximate costs involved. In the case of residential structures, floodproofing was assumed to be feasible if the design flood stage was below the first floor elevation. As shown on Map 137, only one house—located northwest of the intersection of N. Green Bay Road and W. Bradley Road—may be expected to incur flood damage under these conditions. That house could be floodproofed to virtually eliminate future damage from floods up to and including the 100-year recurrence interval event.

Assuming that this structure floodproofing measure would be fully implemented, and utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of this alternative is estimated at $290. This cost consists of the amortization of the $4,600 capital cost for floodproofing. The average annual flood damage abatement benefit was estimated at $90, yielding a benefit-cost ratio of 0.31.

Alternative Plan 3—Culvert Replacement: This alternative plan consists of replacing the existing 10-foot by 5-foot box culvert at N. Green Bay Road with a larger structure. The existing box culvert creates about 3.0 feet of backwater during the 100-year recurrence interval flood under planned, year 2000, land use conditions. Because the potentially flooded home is located only about 70 feet upstream of N. Green Bay Road, installation of a somewhat larger culvert without associated flood control measures would serve to adequately reduce flood levels, eliminating all damages attendant to floods up to and including the 100-year recurrence interval event.

The cost of the culvert replacement would be at least $25,000. Utilizing an annual interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this alternative is estimated to be at least $1,600. This cost consists of the amortization of the capital cost for culvert replacement. The average annual flood abatement benefit is estimated at $90, resulting in a maximum benefit-cost ratio of 0.06.

Evaluation of Flood Control Alternatives for Brown Deer Park Creek

Excluding the no action alternative, all of the alternatives considered for Brown Deer Park Creek were found to be technically feasible. None of the alternatives were found to have a benefit-cost ratio of one or more.

The no action alternative, while offering the lowest cost, does nothing to alleviate the existing flood problem, and therefore does not represent a sound, long-term approach to flood control.

Alternative Plan 2—structure floodproofing—presents several problems in implementation. First, implementation of a voluntary structure floodproofing program may be difficult. Also, yard damages and cleanup costs would remain under the structure floodproofing and elevation alternative. In some instances, a structure floodproofing alternative may be a viable solution to a flooding problem. Such would be the case where structure damages are relatively low and are limited to a few structures along a stream. Structural measures, such as culvert replacement, may not present an economical solution in those instances.

The high cost of Alternative Plan 3—culvert replacement—relative to damages makes the alternative impractical.

Consideration of the Problems Associated with Stormwater Runoff on the Brown Deer Park Golf Course

As noted above, serious drainage problems frequently occur on the Brown Deer Park golf course owing to the discharge of stormwater runoff from adjacent developed areas. Stormwater drainage from an area of approximately 93 acres lying west of N. Teutonia Avenue and developed primarily for industrial use flows overland across the golf course, frequently inundating about an acre of the golf course, causing wet soil conditions, and limiting the use of the course.

The attendance figures for Brown Deer Park golf course that were supplied by the County for the period 1981 through 1987 show an overall declining trend—from a high of 66,211 rounds played in 1981 to a low of 44,175 rounds played in 1987. The number of rounds played decreased every year in that period with the exception of 1985, when 62,840 rounds were played. The County attributes the increased attendance in 1985 to unusually good weather. The County ascribes the declining attendance to interruptions of golf course usage resulting from drainage problems. Figures provided by the County indicate that the loss in revenue in 1987 as compared to 1985, when good weather minimized drainage-related problems, was approximately $113,000. By improving stormwater drainage within the golf course, the County plans to increase attendance and revenue.

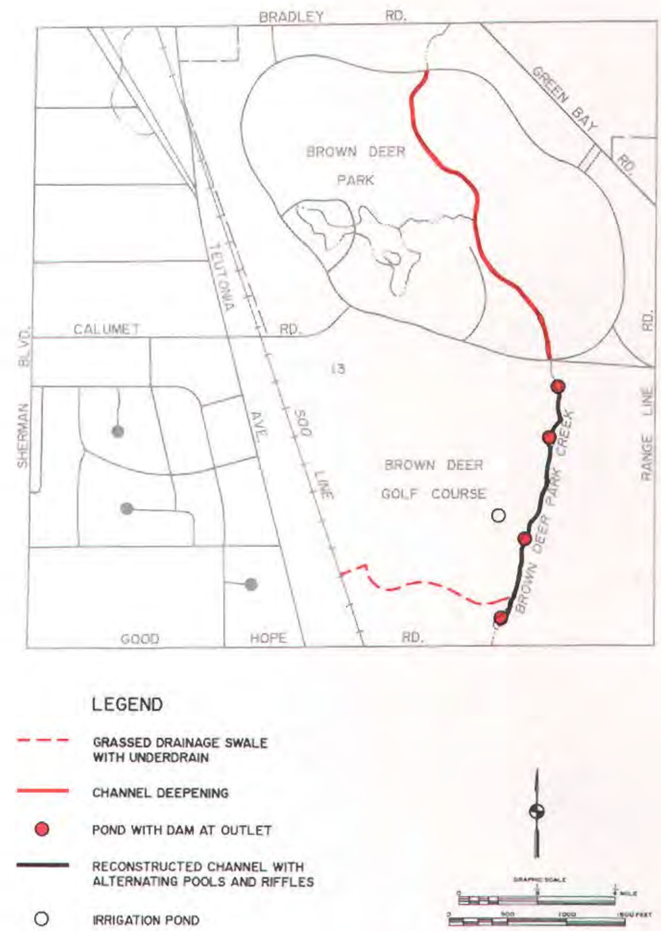
As of February 1988, the County's proposal to alleviate the drainage problem called for implementation of the measures shown on Map 138. Those measures include construction of a 1,650-foot-long trapezoidal grassed swale which would run from the storm sewer outlet at the western boundary of the golf course to a point along Brown Deer Park Creek located about 320 feet downstream of W. Good Hope Road. The proposed swale would have a 10-foot bottom width and side slopes of one vertical on eight horizontal, or flatter. A pipe underdrain system would be installed along approximately the same alignment as the drainage swale. During frequent storm events, the underdrain would pass low flows from the developed area to the west of the golf course and would also collect runoff from areas draining to the swale. During larger storms, most runoff from the west and localized inflow to the swale would be conveyed by the swale.

The swale would follow the alignment of the existing drainage channel for the first 750 feet west of the storm sewer outlet. Under existing conditions, flow beyond that point would travel overland in a generally northeasterly direction before reaching Brown Deer Park Creek. Some of that flow would fill depression areas, causing problems with standing water in the golf course. The County's proposed alternative route for the drainage swale beyond the first 750-foot reach is in a generally southeasterly direction to Brown Deer Park Creek. Stormwater runoff from the area west of the golf course would therefore reach Brown Deer Park Creek at a location about 780 feet upstream of the existing runoff discharge point.

Along the main channel of Brown Deer Park Creek running through the golf course, the County proposes to construct four dams and ponds at the locations shown on Map 138. The two middle ponds are comparable in size to those that existed with the previous system of dams. The farthest upstream and downstream ponds would be enlarged in comparison to the previous

Map 138

MILWAUKEE COUNTY DRAINAGE ALTERNATIVE FOR BROWN DEER GOLF COURSE



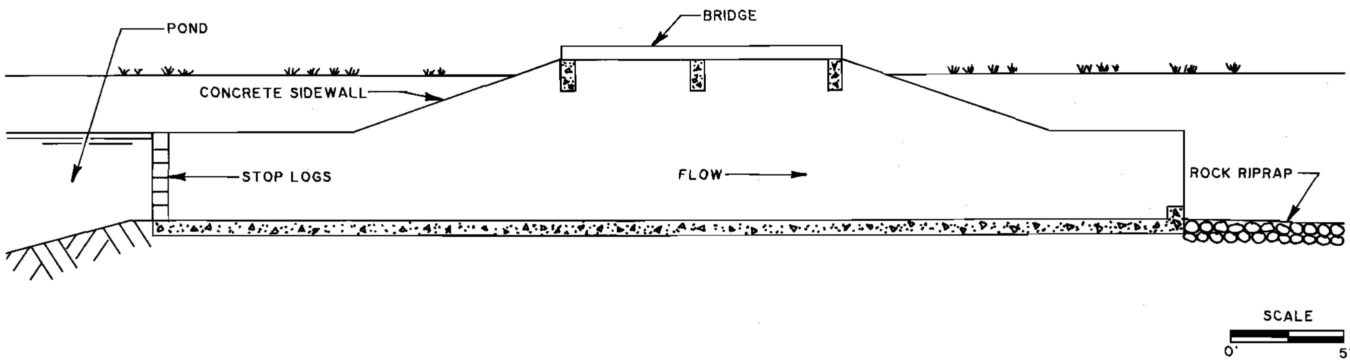
Source: SEWRPC.

ponds. The levels of the ponds would be controlled by box inlet drop spillway structures which would essentially consist of removable timber stop logs set in reinforced concrete sidewalls, as shown in Figure 62. The sidewalls would form a discharge chute for flow over the stop logs and, in the case of the two southernmost dams, would support bridges. The provision of individual, removable stop logs would give the County the ability to lower pond levels, which would increase the hydraulic head on submerged drain outlets and facilitate drainage of the golf course following frequent storms.

The County's plan calls for the reaches of the stream between the ponds to be reconstructed as a series of alternating deep pool and shallow

Figure 62

TYPICAL CROSS-SECTION BOX-INLET DROP SPILLWAY



Source: SEWRPC.

riffle sections. Stream bank side slopes would range from one horizontal on three vertical to one horizontal on eight vertical. The stream banks and bottom in the riffle sections would be lined with rip-rap.

At the present time there are nine bridges crossing the stream within the golf course. The County's plan calls for three of these bridges to be removed and for the remaining six to be reconstructed, with two of the reconstructed bridges being part of the box drop inlet spillways at pond outlets.

The County also proposes to provide subsurface pipe drains for removal of surface runoff which collects in depressions within the floodplain of Brown Deer Park Creek. These drains would be intended to function during frequent storm events, and their outlets would be located above the normal streamwater surface. In addition, the County plans limited filling and/or regrading of depressions within the 100-year recurrence interval floodplain.

An irrigation pond is proposed to be located off the main stream channel, as shown on Map 138. The pond is not intended to be an integral component of the golf course drainage system, and it is not expected to have an impact on flood flows and stages in Brown Deer Park Creek.

None of the elements of the County's proposed drainage and channel modifications would be expected to have a significant effect on downstream 100-year recurrence interval flows and stages. At the peak of the 100-year recurrence interval flood, the realigned portion of the drainage swale would be completely inundated by overbank flooding from the main channel of Brown Deer Park Creek; therefore, runoff from the west would combine with runoff in the main channel at approximately the same location as it would at the present time. Reconstruction of the main channel should not significantly alter the 100-year recurrence interval water surface profile through the golf course if narrower riffle sections are located at the inlets to all ponds, as well as being interspersed throughout each reach between ponds. During the 100-year recurrence interval flood, the spillways at the pond outlets would be completely submerged and would not significantly affect the water surface profile. Most of the 100-year recurrence interval flood flow is carried in the broad floodplain areas adjacent to the stream; therefore, the effect of the obstructions caused by the pedestrian and golf cart bridges located along the stream would be minor, and the proposed bridge removal and/or replacement would not be expected to have a significant hydraulic effect. Because none of the elements of the county proposal, as heretofore described, would be expected to significantly alter the 100-year recurrence interval flood profile through the golf course, the overbank volume available for floodwater storage should remain about the same under proposed conditions as under existing conditions.

In addition to asking the Commission staff to review the County's proposal for drainage modifications within the Brown Deer Park golf course, the County asked that the Commission review the impacts of removing accumulated sediment from the streambed of Brown Deer Park Creek in the reach within Brown Deer Park to the north of the golf course. As shown in Figure 61, the streambed elevation abruptly rises about one foot at Brown Deer Park Drive (south) at the north end of the golf course. According to the County, this sediment removal would be undertaken primarily to improve drainage of the golf course, rather than the remainder of the park. Increasing the hydraulic capacity of the channel through sediment removal downstream of the golf course would enable the golf course to drain more quickly following major storm events, when the capacity of the subsurface drains would be exceeded. As shown in Table 80, floodwater storage along the reach of the stream in Brown Deer Park between River Miles 0.88 and 1.44 causes a significant reduction in peak flood flows. Lowering the streambed would lower the flood profile, reducing the amount of available floodwater storage volume on parklands and thereby increasing downstream flows and stages.

The hydraulic and hydrologic simulation models developed for this study were used to estimate the amount of compensatory floodplain storage volume that would have to be provided to offset the loss of volume due to lowering the streambed, and to prevent an increase in flood flows and stages downstream of Brown Deer Park. Based upon discussions with county personnel, it was assumed that natural vegetation would be permitted to grow in the channel once the streambed had been lowered. It was determined that approximately 3.5 acre-feet of compensatory storage volume must be provided within the 100-year floodplain to offset the loss of volume due to lowered flood stages. The County indicated that the compensatory volume could best be provided within the golf course as part of the proposed regrading. Therefore, if the streambed is lowered in Brown Deer Park downstream of the golf course, a net additional floodplain storage volume of 3.5 acre-feet must be provided in Brown Deer golf course. If significant areas of the floodplain of the golf course are filled during landscaping operations, additional equivalent compensatory storage volume will have to be provided.

The County's proposal as described herein is preliminary. If the proposal is significantly revised, the revised plan will have to again be reviewed and the downstream impacts determined to assure consistency with the overall drainage and flood control system plan for Brown Deer Park Creek as presented in this report.

Recommended Flood Control System for Brown Deer Park Creek

Based upon consideration of the technical feasibility, economic viability, environmental impacts, potential public acceptance, and practicality of each of the alternatives considered, it is recommended that Alternative Plan 2—structure floodproofing—be adopted for Brown Deer Park Creek. It is also recommended that Milwaukee County's conceptual plan for drainage improvements and channel modifications for Brown Deer Park be adopted with the condition that about 3.5 acre-feet of compensatory floodplain storage be provided in the Brown Deer golf course if the stream channel is lowered downstream of the golf course.

The capital cost of the structure floodproofing element of the recommended drainage and flood control plan is estimated at $4,600 in 1986 dollars. The County has budgeted a total of $834,000 for the proposed drainage and channel modification measures. This amount does not include costs for an irrigation system and pond, which are not integral parts of the planned drainage improvements. The proposed drainage improvements should have no significant impact on flood flows and stages along Brown Deer Park Creek.

Implementation of the recommended plan, which consists of the floodproofing of one house and various drainage improvements and channel modifications in Brown Deer Park, would essentially eliminate all flood related damages to existing structures along the Brown Deer Park Creek channel for floods up to and including the 100-year recurrence interval event under planned land use conditions, and would resolve the stormwater drainage problems in the southwest corner of the Brown Deer Park golf course during more frequent events.

In addition to structure floodproofing, drainage improvements, and channel modification, it is recommended that new large-scale topographic maps be prepared for the southeast one-quarter of U. S. Public Land Survey Section 12, and the northeast and southeast one-quarters of Section 13, Township 8 North, Range 21 East. The southeast one-quarter of Section 12 includes the only developed area along the portion of Brown Deer Park Creek recommended for District jurisdiction in the policy plan, and it also includes a portion of Southbranch Creek, which is also being studied under this system plan. The maps in Section 13 cover an area of Brown Deer Park for which there is no large-scale mapping and an area for which there is no large-scale mapping referred to National Geodetic Vertical Datum. It is recommended that the maps be prepared upon completion of the proposed drainage improvements and channel modifications. Since the new maps would serve multiple purposes, none of the attendant costs have been assigned to the flood control plan.

It is recommended that when the bridges at N. Range Line Road and W. Bradley Road are replaced for transportation purposes, they be designed so as to accommodate the 50-year and 10-year recurrence interval flows, respectively, without overtopping the attendant roadways. With respect to W. Bradley Road, it may be necessary also to increase the hydraulic capacity of the N. Green Bay Road bridge in order to achieve this standard for roadway overtopping.

Flood Control and Related Drainage System Plan Implementation

The recommended drainage and flood control system plan for Brown Deer Park Creek consists of structure floodproofing of one house and drainage improvements and channel modifications on the Brown Deer Park golf course. Structure floodproofing measures would be undertaken by the property owner directly affected. It is recommended, however, that the professional services required to prepare plans for floodproofing of the house be made available, at no cost, to the property owner by the Village of Brown Deer through its engineering department. Also, it is recommended that the Village of Brown Deer review its building ordinance to ensure that appropriate floodproofing regulations are included, and that the Village explore, on behalf of the property owner involved, any available state and/or federal aids for such floodproofing measures.

It is further recommended that Milwaukee County implement the drainage improvement and channel modification alternative for Brown Deer Park. Finally, it is recommended that the Milwaukee Metropolitan Sewerage District prepare new large-scale topographic maps for the southeast one-quarter of Section 12, and the northeast and southeast one-quarters of Section 13, Township 8 North, Range 21 East.

SOUTHBRANCH CREEK SUBWATERSHED FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN

Southbranch Creek has not been studied under any previous Commission planning program. Detailed analyses of flood flows and stages were conducted by the Federal Emergency Management Agency (FEMA) along that portion of the creek through the Village of Brown Deer as part of the federal flood insurance study for that village. Those flood flows and stages were later revised under a flood control study conducted for the Village of Brown Deer.[3] The hydrologic and hydraulic analyses conducted under this system planning effort represent a further refinement of the earlier studies.

Overview of the Study Area

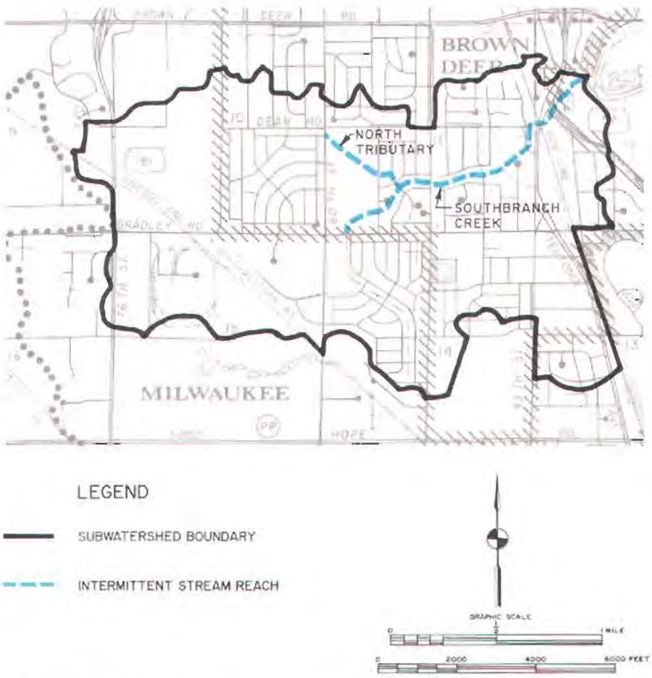
Southbranch Creek is a tributary of the Milwaukee River. The Southbranch Creek subwatershed is located largely within the City of Milwaukee and the Village of Brown Deer, with a small portion located within the Village of River Hills. From its origin at a storm sewer outfall along W. Bradley Road at about N. 59th Street, Southbranch Creek flows in a generally northeasterly direction for approximately 1.53 miles, and drains an area of about 2.95 square miles (see Map 139). Of this total drainage area, 1.30 square miles, or about 44 percent, lie within the City of Milwaukee; 1.59 square miles, or about 54 percent, lie within the Village of Brown Deer; and 0.06 square mile, or about 2 percent, lies within the Village of River Hills.

More specifically, from its origin at the storm sewer outfall located along W. Bradley Road at about N. 59th Street, Southbranch Creek flows in a generally northeasterly direction to W. Dean Road at N. Teutonia Avenue, thence northerly for a distance of about 0.10 mile to N. Teutonia Avenue, and thence northeasterly to its conflu-

[3] *Carl C. Crane, Inc., Southbranch Creek Drainage Study, Village of Brown Deer, Wisconsin, March 1983.*

Map 139

THE SOUTHBRANCH CREEK SUBWATERSHED



Source: SEWRPC.

ence with the Milwaukee River. The entire 1.53-mile reach described is classified as intermittent. This reach is recommended for District jurisdiction in the policy plan companion to this system plan.

Analyses were also made under this study of alternative stormwater drainage measures for a tributary of Southbranch Creek that drains the northwest portion of the subwatershed and enters the Southbranch Creek main channel at N. 54th Street. This tributary was not recommended for District jurisdiction. However, the drainage problems along this tributary warrant that remedial action be considered. Since such action could have an impact on flood flows and stages along Southbranch Creek, it was necessary to consider it in this analysis.

In 1985, the Southbranch Creek subwatershed was almost completely developed for urban use, including residential, commercial, industrial, institutional, and urban open space uses. All of the developed areas of this subwatershed except the residential areas in the Villages of Brown Deer and River Hills are provided with a full range of municipal street improvements, including paved streets with curbs and gutters and attendant storm sewers. The residential areas of Brown Deer and River Hills are served by roadside drainage ditches. Accordingly, surface runoff from much of the subwatershed is effectively conveyed from most individual sites to Southbranch Creek through storm sewers.

Information on pertinent characteristics of the subwatershed, such as hydrologic soil types, land slopes, and land use, is provided in Chapter II of this report. The planned land use conditions utilized in the system planning assume that the watershed will be fully urbanized by the design year of the system plan. However, some existing open space uses, primarily parks, will remain. Channel improvements have been made along the entire 1.37-mile stream length through the Village of Brown Deer. The channel has been physically altered by deepening and straightening, and in some reaches lining with concrete. Some minor channel modifications may have been made in the past along the 0.16-mile reach through the Village of River Hills, although the stream appears generally to be in a "natural" state.

Flooding and Related Drainage Problems

The investigations of historical flood problems along Southbranch Creek conducted under this system planning effort revealed flooding problems within the Village of Brown Deer along the reach between N. 47th Street and N. 55th Street, with the most severe problems occurring along the reach between N. 47th Street and N. 51st Street. As a result of the storms of September 10 and 11, 1986, about 25 complaints of flooding were reported to the Village Engineer. At least 10 of these incidents could be attributed to direct overland flooding from the creek. Four incidents of collapsed basement walls were reported in this area. In addition to the flooding problems along the main stem of Southbranch Creek, flooding and drainage problems have occurred along the north tributary due to overland flooding and storm sewer surcharging. Reported problems include flooding at Dean Elementary School at N. 55th Street and W. Dean Avenue, as well as overland flooding due to storm sewer surcharging in the area between N. 60th Street and the village limits at N. 68th Street. This overland flooding was responsible for a collapsed basement wall at a home located at 8387 N. Grandview Drive during the September 10 and 11, 1986, storm event. The locations of reported flooding and drainage problems within the Southbranch Creek subwatershed during 1986 are shown on Map 140.

It should be noted that the analyses conducted confirmed that these flooding problems are not related to stages on the Milwaukee River at the confluence with Southbranch Creek or to the operation of the Estabrook Park Dam. Flood stages in the areas along Southbranch Creek where damages occurred are generally over 30 vertical feet higher than flood stages on the Milwaukee River at its confluence with Southbranch Creek, and are 50 feet higher than the flood stages at the Milwaukee River at the Estabrook Park Dam. The impacts of the Milwaukee River on flood stages of Southbranch Creek are limited to the first 0.2-mile reach of the creek downstream of Green Bay Court.

The results of the hydrologic and hydraulic analyses indicate that the following number of existing residences may be expected to experience direct flooding along Southbranch Creek under existing and planned land use conditions and existing channel conditions:

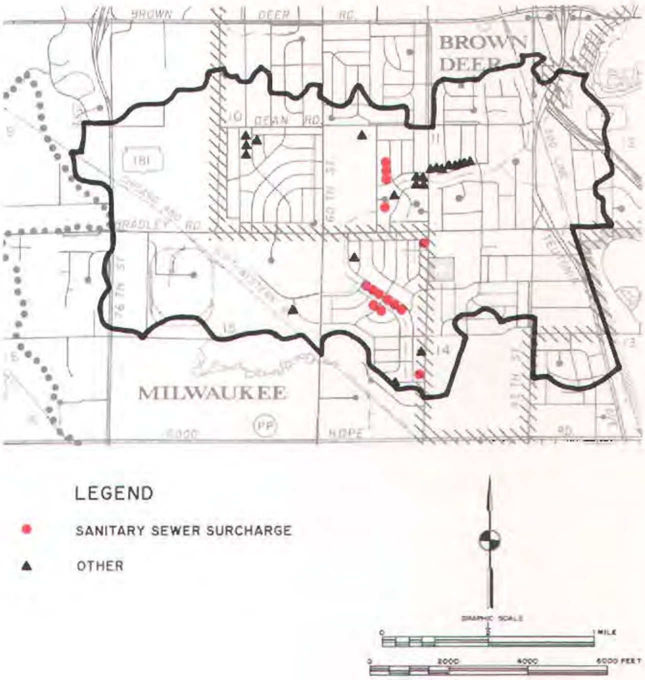
Flood Event Recurrence Interval	Approximate Number of Existing Homes Flooded Existing Land Use and Existing Channel Conditions	Approximate Number of Existing Homes Flooded Planned Land Use and Existing Channel Conditions
100	20	22
50	18	21
10	4	20

All of the homes that may be expected to incur direct flood damages are located within the Village of Brown Deer. No industrial or commercial properties are expected to experience direct damages for floods up to and including the 100-year recurrence interval event under either existing or planned land use conditions. Additional homes and industrial and commercial properties may, however, experience indirect flood damages through sanitary sewer backup. It should be noted that the flood control measures considered under this system plan are primarily intended to alleviate flood damages from direct overland flooding along the stream studied, as well as to provide an adequate outlet for local storm sewers. These measures may help to reduce flooding due to localized stormwater drainage problems or sanitary sewer backups.

The total average annual flood losses—damages—for Southbranch Creek are estimated at $12,200 under existing land use and channel

Map 140

AREAS WITH REPORTED FLOODING AND DRAINAGE PROBLEMS IN THE SOUTHBRANCH CREEK SUBWATERSHED: 1986



Source: SEWRPC.

conditions, and at $42,500 under planned land use and existing channel conditions. Flood losses from a 100-year recurrence interval event are estimated at $163,000 under existing land use and channel conditions, and at $191,000 under planned land use and existing channel conditions.

The drainage and flood control objectives and supporting principles and standards set forth in Chapter III specify the flood events which bridges shall accommodate without overtopping the related roadway. Based on those criteria, five bridges are considered hydraulically inadequate, as shown in Appendix C. These bridges are located at N. Green Bay Court, N. 47th Street, N. 51st Street, N. 54th Street, and N. 55th Street.

Flood Discharges and Stages

As noted in Chapter III of this report, the hydrologic model used for developing design flood discharges for Southbranch Creek uses design rainfall events as input. The design rainfall events were developed using 10-, 50-, and 100-year rainfall volumes obtained from the

Table 81

FLOOD DISCHARGES FOR SOUTHBRANCH CREEK FOR EXISTING AND YEAR 2000 LAND USE AND EXISTING CHANNEL CONDITIONS

Location	River Mile	Peak Flood Discharge (cfs) Existing Land Use, Existing Channel Conditions			Year 2000 Planned Land Use, Existing Channel Conditions		
		10-Year	50-Year	100-Year	10-Year	50-Year	100-Year
Mouth at Milwaukee River	0.00	900	1,290	1,430	1,050	1,570	1,690
Private Drive	0.11	900	1,290	1,430	1,050	1,570	1,690
N. Green Bay Court	0.16	900	1,290	1,430	1,050	1,570	1,690
N. Green Bay Road	0.23	900	1,290	1,430	1,050	1,570	1,690
N. Teutonia Avenue	0.35	890	1,220	1,340	1,130	1,500	1,580
550 Feet Downstream of W. Dean Road	0.37	830	1,140	1,250	1,060	1,420	1,490
W. Dean Road	0.47	640	950	1,050	870	1,170	1,240
650 Feet Downstream of N. 47th Street	0.63	640	920	1,020	870	1,140	1,200
600 Feet Downstream of N. 47th Street	0.64	630	900	980	870	1,120	1,170
N. 47th Street	0.75	630	900	980	870	1,120	1,170
300 Feet Downstream of N. 51st Street	0.95	630	870	950	870	1,090	1,130
250 Feet Downstream of N. 51st Street	0.96	540	760	830	770	970	1,010
N. 51st Street	1.01	540	760	830	770	970	1,010
Upstream of N. 51st Street	1.02	530	720	780	770	930	960
N. 54th Street	1.17	520	660	710	740	850	860
300 Feet Downstream of N. 55th Street	1.28	520	640	680	740	830	840
N. 55th Street	1.34	500	620	660	720	810	820

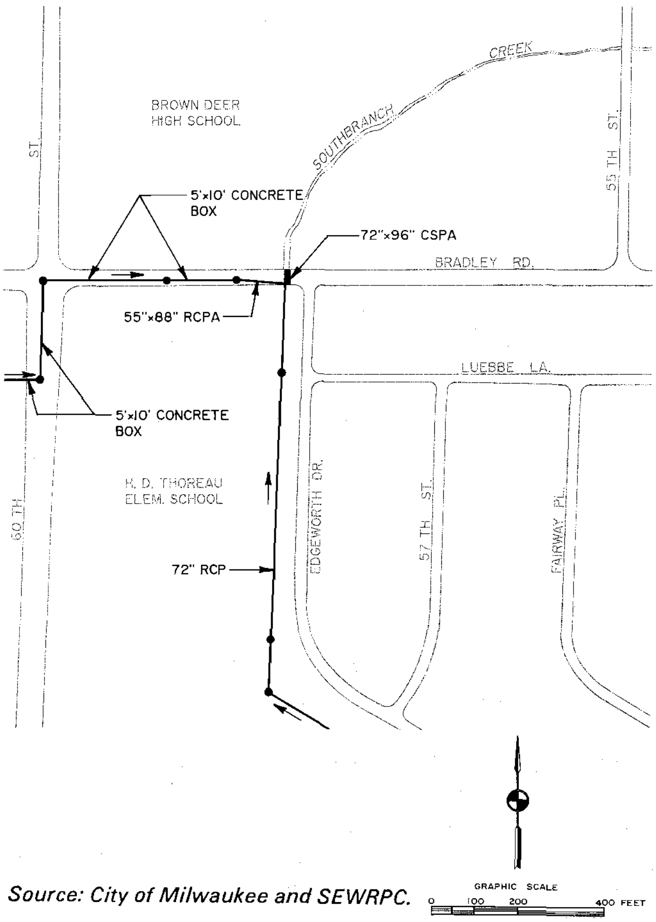
Source: SEWRPC.

updated point rainfall depth-duration-frequency relationships developed by the Commission as discussed in Chapter III. The rainfall distribution utilized for each design storm was the median distribution of a first-quartile storm, as shown in Chapter III. The design storm duration was determined for a given recurrence interval by simulating the peak discharge at a given location for a range of storm durations. The storm duration and associated rainfall volume which produced the largest peak discharge at a given location for a given recurrence interval was selected as the design storm for that location. This analysis was conducted for both existing and planned land use and channel conditions at 17 locations on the main stem of Southbranch Creek.

The estimated peak flood discharges under existing and planned, year 2000, land use conditions and existing channel conditions are set forth in Table 81. These flows were developed assuming two different outlet conditions at the City of Milwaukee storm sewer outfall located at W. Bradley Road at the upstream end of the study reach. At the present time, flow from a 10-foot-wide by 5-foot-high reinforced concrete box culvert storm sewer under W. Bradley Road is forced through an 88-inch by 55-inch reinforced concrete pipe arch outlet with an adverse slope. Flow through that pipe arch is combined with flow from a 72-inch reinforced concrete pipe from the south and passed through a 96-inch by 72-inch corrugated steel pipe arch under Bradley Road. The layout of those storm sewers is shown on Map 141. Detailed design plans prepared by the City of Milwaukee call for the replacement of the 96-inch by 72-inch outlet pipe with a 20-foot-wide by 9-foot-high reinforced concrete box culvert. This work is expected to proceed as part of the W. Bradley Road repaving scheduled for 1988. The invert of this box culvert would be

Map 141

TRUNK STORM SEWER SYSTEM UPSTREAM OF THE SOURCE OF SOUTHBRANCH CREEK



Source: City of Milwaukee and SEWRPC.

placed 4.0 feet below the existing invert of Southbranch Creek at this location, thus reducing the effective capacity of this culvert until such time that development conditions in the City of Milwaukee require the full use of the culvert capacity. Utilizing the full capacity of this culvert will require a lowering of the Southbranch Creek invert along the 0.19-mile-long reach between N. 55th Street and W. Bradley Road.

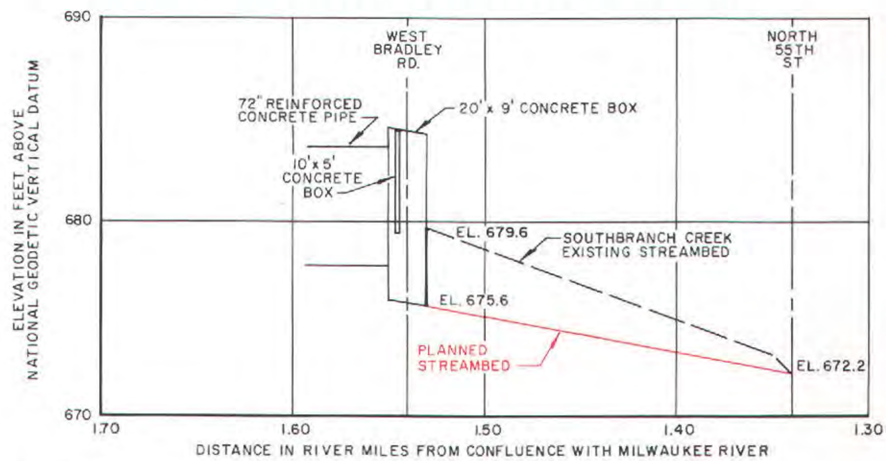
For purposes of this system planning effort, existing land use condition discharges were developed assuming the new box culvert under W. Bradley Road to be in place but with the outlet partially blocked by the existing downstream channel bed. Planned land use discharges were developed assuming the full capacity of this culvert would be available. Based upon review of the existing and planned storm sewer and utility systems in the vicinity of W. Bradley Road and Southbranch Creek, it was concluded that this lower invert elevation would be needed to provide proper drainage of the area and to avoid other utility conflicts. The details of this crossing under existing and planned channel conditions are shown in Figure 63. This culvert construction and the attendant channel modifications extending about 0.19 mile downstream of W. Bradley Road are necessary to resolve basement flooding and drainage problems and to accommodate utility conflicts upstream of W. Bradley Road, and do not serve to abate direct overland flooding from Southbranch Creek, as no structure flood damages are expected along this reach. Flood stage profiles were determined for the 10-, 50-, and 100-year recurrence interval runoff events under planned land use and existing channel conditions. These profiles, which encompass the full 1.53-mile-long reach of Southbranch Creek studied, constitute a graphic representation of the flood stages along Southbranch Creek under the specified recurrence interval flood discharges, and under planned land use and existing channel conditions. In addition to providing an overall representation of flood stages relative to familiar points of reference such as the channel bottom and bridge deck surfaces, the profiles, because they are continuous, permit the determination of flood stages at any point along the stream channel. The flood profiles are shown in Figure 64.

The accuracy of the hydrologic and hydraulic models were checked using high-water marks along Southbranch Creek from the flood of September 11, 1986. High-water marks were available for the two crest stage gages located on the head and end walls of the N. Green Bay Court bridge which are operated by the District, as well as from field observations made by the Brown Deer Village Engineer. That flood was caused by heavy rains, with a peak intensity of 2.90 inches in four hours falling on saturated ground which had received 2.31 inches of rain in the preceding 33 hours. The recorded rainfall amounts were input to the hydrologic model to simulate the resultant flood hydrographs along the stream. The peak flows from that simulation were then used as input to the hydraulic model and appropriate adjustments made to approximate the September 11, 1986, flood profile based on the recorded high-water marks.

The extent of the 100-year recurrence interval floodplain under planned land use conditions is shown on Map 142. The flood hazard area in the

Figure 63

DETAILS OF W. BRADLEY ROAD CROSSING
UNDER EXISTING AND PLANNED CHANNEL CONDITIONS



Source: SEWRPC.

reach from the mouth of the stream to N. Teutonia Avenue (River Mile 0.35) was delineated using large-scale topographic maps prepared in 1970. The remaining flood hazard area was delineated using large-scale topographic maps prepared in 1964.

Alternative Flood Control and Related Drainage System Plans for the North Tributary to Southbranch Creek

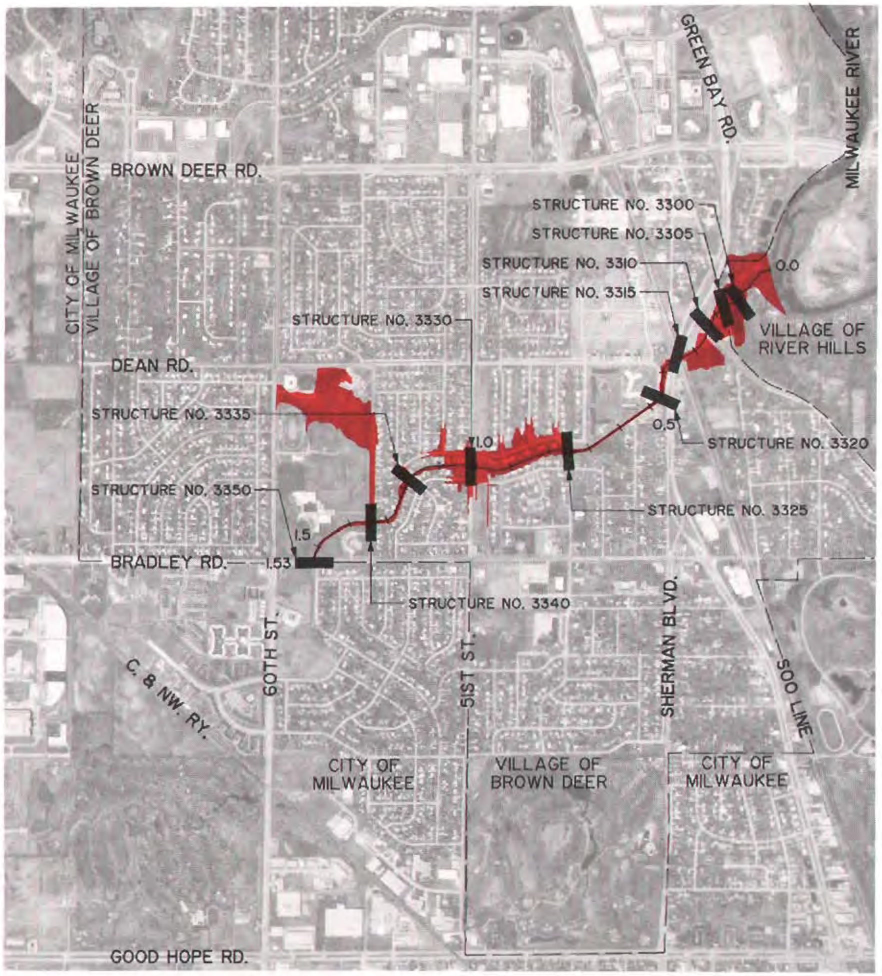
As previously noted, measures will need to be designed to alleviate the stormwater drainage and flooding problems along the north tributary to Southbranch Creek. Since those measures could have a significant impact on flood flows and stages along Southbranch Creek, analyses of alternative stormwater drainage and flood control measures for the north tributary were conducted under this system planning effort.

From its origin at a storm sewer outfall at W. Dean Road extended in the City of Milwaukee between N. 72nd Street extended and N. 68th Street extended, the north tributary flows in a generally easterly direction for approximately 1.35 miles, and drains an area of about 0.73 square mile. Of this total drainage area, 0.26 square mile, or about 36 percent, lies in the City of Milwaukee, and 0.47 square mile, or about 64 percent, lies in the Village of Brown Deer.

More specifically, from its origin at the storm sewer outfall at W. Dean Road extended, the north tributary flows for about 1,400 feet in a generally easterly direction to N. 68th Street extended. At that point, which is the boundary between the City of Milwaukee and the Village of Brown Deer, the stream enters a 60-inch-diameter storm sewer. The storm sewer runs about 2,300 feet in a generally easterly direction, changing in diameter from 60 inches to 42 inches, thence to 48 inches, and finally to a 65-inch by 40-inch pipe arch. The pipe arch runs south of, and parallel to, W. Dean Road, and terminates about 770 feet west of the intersection of W. Dean Road and N. 60th Street. From the outlet of the pipe arch, the stream flows in a 940-foot-long roadside ditch which runs to the east and then the south. At the end of the ditch, the stream passes under N. 60th Street through a series of corrugated steel pipe arch culverts which run in a generally southeasterly direction. The pipe arches consist of 60 feet of a double 50-inch by 31-inch culvert, followed by 60 feet of a double 58-inch by 36-inch culvert, followed by about 80 feet of parallel 64-inch by 43-inch and 71-inch by 47-inch culverts. From the culvert outlets, the stream enters an open channel which flows in a generally southeasterly direction for about 1,390 feet to N. 55th Street. At N. 55th Street, the stream enters a 900-foot-long, 48-inch-diameter culvert pipe which runs in a generally

Map 142

100-YEAR RECURRENCE INTERVAL FLOODPLAIN FOR SOUTHBRANCH CREEK UNDER YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

1.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR SOUTHBRANCH CREEK IS SHOWN IN APPENDIX H

GRAPHIC SCALE

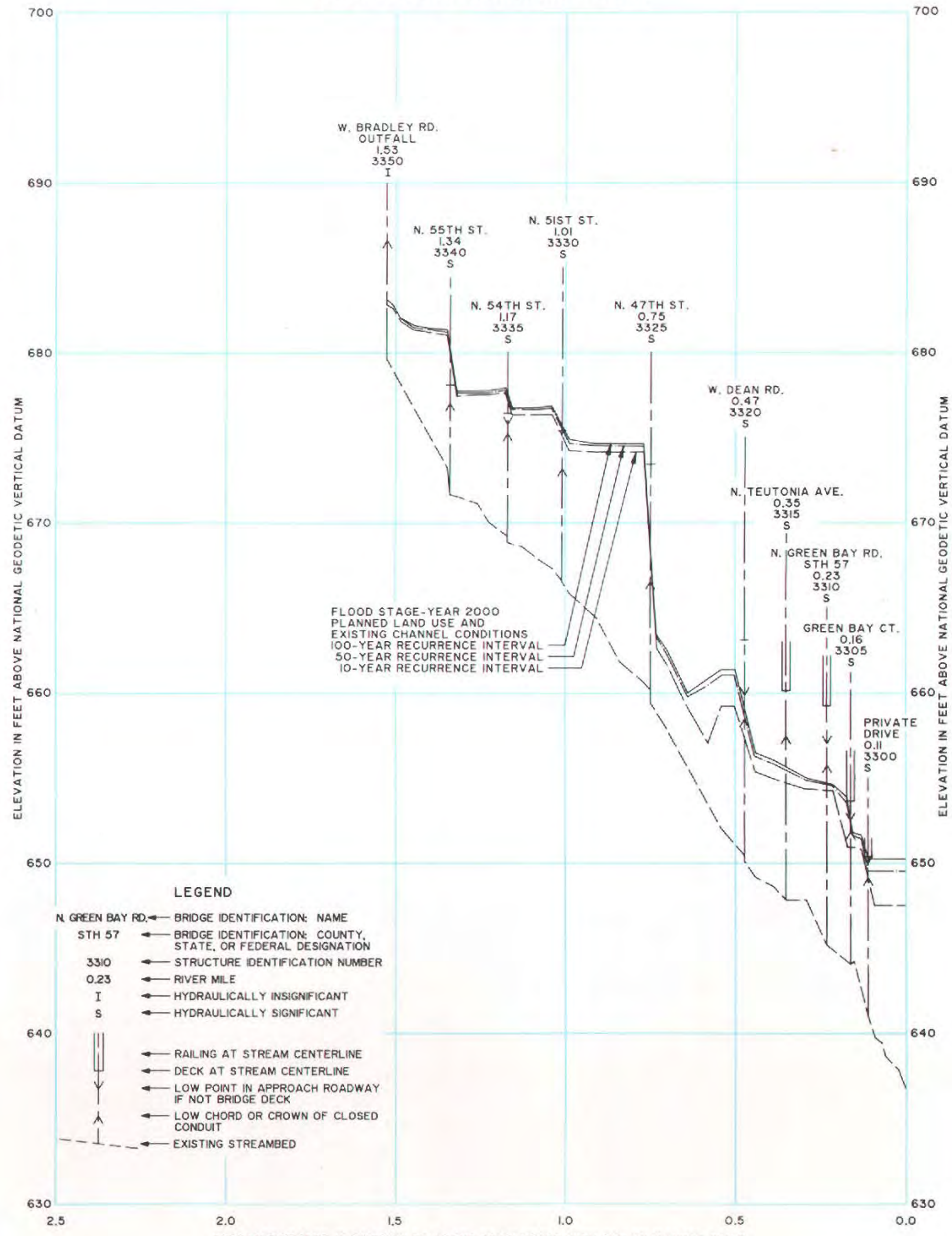
0 1/2 1 MILE

DATE OF PHOTOGRAPHY APRIL 1986

Source: SEWRPC.

Figure 64

FLOOD STAGE AND STREAMBED PROFILE FOR SOUTHBRANCH CREEK



southeasterly direction through a residential area. The culvert discharges to Southbranch Creek immediately downstream of N. 54th Street.

Under planned land use and existing storm sewer and stream channel conditions, the extent, severity, and frequency of overland flooding due to inadequate storm sewer and stream channel capacity could increase significantly over existing conditions, affecting additional residences, and the Dean Elementary School to a greater degree. Increased flooding would affect primarily homes located south of W. Dean Road between N. 68th Street extended and 60th Street, and between 55th Street and 53rd Street south of W. Nokomis Road and north of Southbranch Creek.

In preparation of this system plan, three alternative flood control plans were considered for alleviating the stormwater drainage and flooding problems along the north tributary to Southbranch Creek: 1) Alternative Plan A—channel modification, enclosure and construction, storm sewer replacement, and culvert replacement; 2) Alternative Plan B—channel modification, enclosure and construction, culvert replacement, storm sewer construction, and detention storage; and 3) Alternative Plan C—channel modification, enclosure and construction, culvert replacement, and storm sewer construction with maximum detention storage. Each alternative is described below. The economic costs attendant to each alternative are provided in Table 82. The 100-year recurrence interval flood discharges and stages along the north tributary under the alternative plans are compared in Table 83.

<u>Alternative Plan A—Channel Modification, Enclosure and Construction, Storm Sewer Replacement, and Culvert Replacement</u>: This alternative system for the resolution of the stormwater drainage and flooding problems along the north tributary is shown on Map 143, and consists of replacing the existing 3,440-foot-long trunk storm sewer-channel-culvert system between N. 68th Street extended and N. 60th Street with one 2,330-foot-long, 14-foot by 6-foot concrete box culvert followed by five parallel 91-inch by 58-inch elliptical concrete pipes running 1,110 feet to the east side of N. 60th Street. Elliptical pipes would be required to maintain adequate cover. The existing stormwater drainage system of roadside ditches in the Village of Brown Deer between N. 68th Street and N. 60th Street would be retained and connected to the new trunk sewer.

The existing streambed between N. 60th Street and N. 55th Street would be lowered by about 1.0 foot to 2.5 feet, the channel bottom would be widened to 20 feet, and the channel side slopes would be set at one vertical on three horizontal. A narrow low-flow channel would be provided to limit the width of the stream under normal flow conditions. The streambed and channel side slopes would remain grassed. A trapezoidal, concrete-lined channel with a 15-foot bottom width and one vertical on three horizontal side slopes would be constructed along the west side of N. 55th Street between the existing channel and Southbranch Creek. The channel would be lined with concrete to an elevation two feet above the 100-year recurrence interval flood level. A pedestrian bridge would be provided across the concrete-lined channel to maintain the existing access to Brown Deer High School from the east. The existing 900-foot-long, 48-inch-diameter storm sewer from N. 55th Street to Southbranch Creek would be retained to provide discharge capacity in excess of the new channel capacity.

Implementation of this alternative would essentially eliminate all damages attendant to floods up to and including the 100-year recurrence interval event.

Utilizing an annual interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this alternative is estimated at $303,000. This cost consists of the amortization of the $4,740,000 capital cost—$4,110,000 for storm sewer and culvert replacement and ditch enclosure, $40,000 for channel modification, $535,000 for channel construction, and $55,000 for pedestrian bridge construction—and $2,000 in annual operation and maintenance costs.

<u>Alternative Plan B—Channel Modification, Enclosure and Construction, Culvert Replacement, Storm Sewer Construction, and Detention Storage</u>: This drainage and flood control alternative is shown on Map 144, and calls for providing a 26.5-acre-foot detention pond on open land in the City of Milwaukee to the west of N. 68th Street extended. The existing trunk storm sewer in the Village of Brown Deer south of W. Dean

Table 82

COST ESTIMATES FOR FLOOD CONTROL ALTERNATIVE FOR THE NORTH TRIBUTARY TO SOUTHBRANCH CREEK IN THE VILLAGE OF BROWN DEER

Alternative	Description	Costs			
			Annual		
		Capital	Amortized Capital[a]	Operation and Maintenance	Total
A. Channel Modification, Enclosure, and Construction, and Storm Sewer and Culvert Replacement	Construct 1,100-foot-long concrete lined channel	$ 535,000			
	Modify existing grass-lined channel	40,000			
	Replace existing storm sewers and culverts and enclose roadside ditch	4,110,000			
	Construct one pedestrian bridge	55,000			
	Subtotal	$4,740,000	$301,000	$ 2,000	$303,000
B. Channel Modification, Enclosure, and Construction, Storm Sewer Construction, and Detention Storage	Construct 1,100-foot-long grass-lined channel	$ 135,000			
	Modify existing grass-lined channel	40,000			
	Replace culverts, enclose roadside ditch, and construct an additional trunk storm sewer	800,000			
	Provide one detention pond	520,000			
	Construct one pedestrian bridge	55,000			
	Subtotal	$1,550,000	$ 98,000	$13,000	$111,000
C. Channel Modification, Enclosure, and Culvert Replacement, and Storm Sewer Construction with Maximum Detention Storage	Construct 1,100-foot-long grass-lined channel	$ 110,000			
	Modify existing grass-lined channel	15,000			
	Replace culverts, enclose roadside ditch and construct an additional trunk storm sewer	800,000			
	Provide two detention ponds	980,000			
	Construct one pedestrian bridge	45,000			
	Subtotal	$1,950,000	$124,000	$25,000	$149,000

[a]Amortized capital cost is based on an interest rate of 6 percent and a project life of 50 years.

Source: SEWRPC.

Road between N. 68th Street extended and N. 60th Street would be retained and would serve as an outlet for the detention pond. An additional 2,140-foot-long, 42-inch-diameter reinforced concrete storm sewer would be constructed parallel to the existing storm sewer, beginning at N. Edge O' Woods Drive, which is the first street east of N. 68th Street extended. At the outlet of the existing trunk storm sewer line, that line and the new 42-inch sewer would enter a 780-foot-long, 8-foot by 4-foot concrete box which would be followed by a 370-foot-long, 10-foot by 4-foot concrete box, running to the east side of N. 60th Street. The existing streambed between N. 60th Street and N. 55th Street would be lowered by about 1.0 to 2.5 feet, the channel bottom would be widened to 20 feet, and the channel side slopes would be set at one vertical on three horizontal. A narrow, low-flow channel would be provided to limit the width of the stream under normal flow conditions. The streambed and channel side slopes would remain grassed. A

Table 83

IMPACT OF STORMWATER DRAINAGE AND FLOOD CONTROL PLANS FOR THE NORTH TRIBUTARY TO SOUTHBRANCH CREEK ON 100-YEAR RECURRENCE INTERVAL FLOOD DISCHARGES AND STAGES

		100-Year Recurrence Interval Flood Discharges Year 2000 Planned Land Use (cfs)							100-Year Recurrence Interval Stage (feet NGVD)						
Location	River Mile	Existing Channel Conditions	Alternative A	Percent Change from Existing	Alternative B	Percent Change from Existing	Alternative C	Percent Change from Existing	Existing North Tributary Channel Conditions	Alternative A	Change in Stage from Existing (feet)	Alternative B	Change in Stage from Existing (feet)	Alternative C	Change in Stage from Existing (feet)
Mouth at Southbranch Creek	0.00	70	720	1,030	330	470	170	240	676.8[b]	676.6[c,d]	--[b,c]	676.6[c,d]	--[b,c]	676.6[c,d]	--[b,c]
N. 55th Street	0.17	70	720	1,030	330	470	170	240	679.7	676.7	-3.0	676.9	-2.8	677.2	-2.5
N. 60th Street	0.43	720	720	0	260	-64	250	-65	679.7	678.0	-1.7	677.3	-2.4	677.3	-2.4
N. 68th Street Extended Outlet to Recommended Detention Pond	1.09	720	720	0	70	-90	70	-90	--	--	--	705.5	--	705.5	--
Inlet to Recommended 68th Street Detention Pond	--[a]	720	720	0	720	0	720	0	--	--	--	705.5	--	705.5	--

[a] *Exact location dependent on final design.*

[b] *Under existing conditions, the mouth of the north tributary is located immediately downstream of N. 54th Street.*

[c] *Under planned conditions with Alternative B or C, the mouth of the north tributary would be located immediately upstream of N. 55th Street.*

[d] *North tributary Alternatives B and C assume that measures are implemented along Southbranch Creek to limit peak stage at the confluence of the tributary and Southbranch Creek to approximate elevation of 676.6 feet above NGVD.*

Source: SEWRPC.

trapezoidal, grass-lined channel with a 10-foot bottom width and one vertical on three horizontal side slopes would be constructed along the west side of N. 55th Street between the existing channel and Southbranch Creek. A pedestrian bridge would be provided across the new channel to maintain the existing access to Brown Deer High School from the east.

Implementation of this alternative would essentially eliminate all damages attendant to floods up to and including the 100-year recurrence interval event.

Utilizing an annual interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this alternative is estimated at $111,000. This cost consists of the amortization of the $1,550,000 capital cost—$520,000 for the detention pond; $800,000 for storm sewer construction, culvert replacement, and ditch enclosure; $40,000 for channel modification; $135,000 for channel construction; and $55,000 for pedestrian bridge construction—and $13,000 in annual operation and maintenance costs.

Alternative Plan C—Channel Modification, Enclosure and Construction, Culvert Replacement, and Storm Sewer Construction with Maximum Detention Storage: This alternative is shown on Map 145, and consists of the same measures as called for under Alternative B through the 10-foot by 4-foot box culvert terminating on the east side of N. 60th Street. East of N. 60th Street, the first 500 feet of the existing streambed would be lowered by 1.5 to 2.5 feet, the channel bottom would be widened to 20 feet, and the channel side slopes would be set at one vertical on three horizontal. A narrow, low-flow channel would be provided to limit the width of the stream under normal flow conditions. The streambed and channel side slopes would remain grassed.

That channel would discharge to a 28-acre-foot detention pond located on village school property which is presently open land. The existing 48-inch pipe running from N. 55th Street to Southbranch Creek would be retained and would serve as the pond outlet. A backwater gate would be provided at the outlet of the 48-inch pipe to prevent backwater from Southbranch Creek

Map 143

ALTERNATIVE SYSTEM A: CHANNEL MODIFICATION, ENCLOSURE, AND CONSTRUCTION; STORM SEWER REPLACEMENT; AND CULVERT REPLACEMENT ALONG THE NORTH TRIBUTARY TO SOUTHBRANCH CREEK

Source: SEWRPC.

Map 144

ALTERNATIVE SYSTEM B: CHANNEL MODIFICATION, ENCLOSURE, AND CONSTRUCTION; CULVERT REPLACEMENT; STORM SEWER CONSTRUCTION; AND DETENTION STORAGE ALONG THE NORTH TRIBUTARY TO SOUTHBRANCH CREEK

Source: SEWRPC.

from partially filling the detention pond and reducing the available storage volume. In addition to the 48-inch outlet, the detention pond would have a grass-lined overflow spillway which would function during major flood events. The overflow spillway would discharge to a grassed channel with a five-foot bottom width and one horizontal to three vertical side slopes. The channel would be constructed along the west side of N. 55th Street between the detention pond and Southbranch Creek. A pedestrian bridge would be provided across the spillway discharge channel to maintain the existing access to Brown Deer High School from the east.

The detention pond at N. 55th Street would drain completely between major storms and would normally be dry, with base flows being passed by a low-flow channel running through the pond. The pond would not encroach on existing athletic fields or playgrounds, and, because the pond area would normally be dry, the existing open land between Brown Deer High School, Brown Deer Middle School, and Dean Elementary School would be preserved.

Implementation of this alternative would essentially eliminate all damages attendant to floods up to and including the 100-year recurrence interval event.

Utilizing an annual interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this alternative is estimated at $149,000. This cost consists of the amortization of the $1,950,000 capital cost—$980,000 for detention ponds; $800,000 for storm sewer construction, culvert replacement, and ditch enclosure; $15,000 for channel modification; $110,000 for channel construction; and $45,000 for pedestrian bridge construction—and $25,000 in annual operation and maintenance costs.

Map 145

ALTERNATIVE SYSTEM C: CHANNEL MODIFICATION, ENCLOSURE, AND CONSTRUCTION; CULVERT REPLACEMENT; AND STORM SEWER CONSTRUCTION WITH MAXIMUM DETENTION STORAGE ALONG THE NORTH TRIBUTARY TO SOUTHBRANCH CREEK

Source: SEWRPC.

Evaluation of Flood Control Alternatives for the North Tributary to Southbranch Creek

The principal features and costs associated with each of the stormwater and floodland management alternatives considered for the north tributary to Southbranch Creek are summarized in Table 82. All of the alternatives are technically feasible.

Because of the high cost of Alternative A—channel modification, enclosure and construction, storm sewer replacement, and culvert replacement—relative to Alternatives B and C, and because this alternative offers no beneficial reduction of flows entering Southbranch Creek, Alternative A was rejected.

The impacts of north tributary Alternatives B and C on 100-year recurrence interval flood discharges and stages in the existing Southbranch Creek channel are shown in Table 84. Implementation of the north tributary alternatives would require structural measures along Southbranch Creek to reduce the 100-year recurrence interval flood stage at the confluence of the tributary and Southbranch Creek upstream of N. 55th Street to an elevation at which there would be no flooding along the tributary due to backwater from Southbranch Creek. Such measures are included in the alternative plans for Southbranch Creek discussed in a later section of this chapter.

Under Alternative B, the 100-year recurrence interval flood-flows along Southbranch Creek downstream of the new mouth of the tributary at River Mile 1.35 would be less than the corresponding flows under Alternative A, but greater than the flows under existing channel conditions. This is because under existing channel conditions, the 48-inch-diameter outlet at N. 55th Street restricts the flow in the tributary, causing ponding upstream of N. 55th Street. The hydraulic restriction and resultant ponding reduce downstream flows, but also create upstream flooding problems. Under Alternative B, the channel modification measures along the north tributary in conjunction with upstream measures would decrease flood stages and essentially eliminate overland flooding along the tributary. However, because the implementation of Alternative B would increase 100-year recurrence interval flows along Southbranch Creek downstream of the existing mouth of the tributary, adoption of Alternative B would necessitate making appropriate legal arrangements with property owners in downstream areas where no structure flooding would be expected and no flood control measures would be required, but where increased flows would result in increased stages. Alternative B has the lowest average annual cost of the three alternatives considered.

As shown in Table 84, under Alternative C, the 100-year recurrence interval flood-flows along Southbranch Creek downstream of the existing mouth of the tributary at about River Mile 1.17 are somewhat less than the corresponding flows under existing channel and planned land use conditions. This flow reduction occurs because of the effects of the additional detention pond at N. 55th Street. Although the average annual cost of Alternative C is greater than that of Alternative B, the implementation of Alternative C would reduce the cost of the flood control measures

Table 84

IMPACT OF STORMWATER DRAINAGE AND FLOOD CONTROL PLANS FOR THE NORTH TRIBUTARY TO SOUTHBRANCH CREEK ON 100-YEAR RECURRENCE INTERVAL FLOOD DISCHARGES AND STAGES ALONG SOUTHBRANCH CREEK[a]

Location	River Mile	100-Year Recurrence Interval Flood Discharges Year 2000 Planned Land Use (cfs)					100-Year Recurrence Interval Stage (feet NGVD)				
		Existing North Tributary and Southbranch Creek Channel Conditions	North Tributary Alternative B Existing Southbranch Creek Channel	Percent Change from Existing	North Tributary Alternative C Existing Southbranch Creek Channel	Percent Change from Existing	Existing North Tributary and Southbranch Creek Channel Conditions	North Tributary Alternative B Existing Southbranch Creek Channel	Change in Stage from Existing (feet)	North Tributary Alternative C Existing Southbranch Creek Channel	Change in Stage from Existing (feet)
Mouth at Milwaukee River	0.00	1,690	1,890	12	1,640	-3	645.3	645.5	0.2	645.3	0.0
Private Drive	0.11	1,690	1,890	12	1,640	-3	649.2	649.6	0.4	649.1	-0.1
N. Green Bay Court	0.16	1,690	1,890	12	1,640	-3	651.9	652.2	0.3	651.9	0.0
N. Green Bay Road	0.23	1,690	1,890	12	1,640	-3	654.5	654.8	0.3	654.5	0.0
N. Teutonia Avenue	0.35	1,580	1,820	15	1,530	-3	655.5	655.8	0.3	655.5	0.0
550 Feet Downstream of W. Dean Road	0.37	1,490	1,740	17	1,440	-3	655.8	656.2	0.4	655.8	0.0
W. Dean Road	0.47	1,240	1,490	20	1,190	-4	657.0	657.7	0.7	656.8	-0.2
650 Feet Downstream of N. 47th Street	0.63	1,200	1,460	22	1,160	-3	659.6	660.0	0.4	659.5	-0.1
600 Feet Downstream of N. 47th Street	0.64	1,170	1,420	21	1,120	-4	659.9	860.4	0.5	659.8	-0.1
N. 47th Street	0.75	1,170	1,420	21	1,120	-4	668.7	667.4	0.7	666.6	-0.1
300 Feet Downstream of N. 51st Street	0.95	1,130	1,360	20	1,080	-4	674.7	675.0	0.3	674.6	-0.1
250 Feet Downstream of N. 51st Street	0.96	1,010	1,240	23	960	-5	674.7	675.1	0.4	674.6	-0.1
N. 51st Street	1.01	1,010	1,240	23	960	-5	674.7	675.1	0.4	674.6	-0.1
Upstream of N. 51st Street	1.02	960	1,210	26	920	-4	676.8	677.1	0.3	676.7	-0.1
N. 54th Street	1.17	860	1,180	37	860	0	676.8	677.1	0.3	676.7	-0.1
320 Feet Downstream of N. 55th Street	1.28	840	1,160	38	840	0	677.8	678.2	0.4	677.8	0.0
Southbranch Creek at N. 55th Street	1.34	820	1,140	39	820	0	680.0	680.3	0.3	680.0	0.0
320 Feet Upstream of N. 55th Street	1.41	820	820	0	820	0	681.4	682.0	0.6	681.4	0.0

[a]*This table illustrates the impact of the north tributary alternatives on flows and stages in the existing Southbranch Creek channel. Implementation of the north tributary alternatives would require structural measures along Southbranch Creek to reduce the 100-year recurrence interval flood stage at the confluence of the tributary and Southbranch Creek to an elevation at which there would be no flooding along the tributary due to backwater from Southbranch Creek.*

Source: SEWRPC.

required along the main stem of Southbranch Creek in comparison to Alternative B. Also, the implementation of Alternative C would not increase 100-year recurrence interval flood levels in downstream areas where there would be no structure flooding.

As already noted, the alternative implemented for the north tributary will affect flood flows and possible flood control measures along Southbranch Creek. Therefore, the recommended alternative for the north tributary can best be selected in conjunction with the selection of a flood control plan for Southbranch Creek. Therefore, the recommended alternative for the tributary was chosen as a component of the recommended plan for Southbranch Creek as presented below.

Alternative Flood Control and Related Drainage System Plans for Southbranch Creek

In preparation of this system plan, four alternative flood control plans were considered for alleviating the flood damage problems along Southbranch Creek: 1) Alternative Plan 1—no action; 2) Alternative Plan 2—combination of culvert replacement and channel modification; 3) Alternative Plan 3—combination of maximum storage, culvert replacement, and channel modification; and 4) Alternative Plan 4—combination of culvert replacement and channel modification with additional detention storage on the north tributary. Each alternative is described below. The economic benefits and costs attendant to each alternative are provided in Table 85.

Table 85

COST ESTIMATES FOR FLOOD CONTROL ALTERNATIVES FOR SOUTHBRANCH CREEK IN THE VILLAGE OF BROWN DEER

Alternative	Description	Costs					Benefit-Cost Analysis			
			Annual							
		Capital	Amortized Capital[a]	Operation and Maintenance	Other	Total	Annual Benefits	Annual Benefits Minus Annual Costs	Benefit-Cost Ratio	Ratio Greater than One
1. No Action	- -	$ 0	$ 0	$ 0	$27,250	$ 27,250	$ 0	$-27,250	- -	No
2. Combination of Culvert Replacement and Channel Modification	Replace culverts at five road crossings	$ 498,000								
	0.78 mile of channel modification	618,000								
	Subtotal	$1,116,000	$70,900	$ 1,600	$ 0	$ 72,500	$27,250	$-45,250	0.38	No
3. Combination of Maximum Storage, Culvert Replacement, and Channel Modification	Construct one detention basin	$ 260,000								
	Decentralized storage	179,000								
	Replace culverts at four road crossings	335,000								
	0.78 mile of channel modification	530,000								
	Subtotal	$1,304,000	$82,700	$22,800	$ 0	$105,500	$27,250	$-78,250	0.26	No
4. Combination of Culvert Replacement and Channel Modification, with Additional Storage on the North Tributary	Replace culverts at four road crossings	$ 335,000								
	0.78 mile of channel modification	530,000								
	Construct one detention basin	400,000[a]								
	Subtotal	$1,265,000	$80,300	$13,400	$ 0	$ 93,700	$27,250	$-66,450	0.29	No

[a]Incremental cost for providing 55th Street detention pond called for in north tributary Alternative C, as opposed to channelization called for in north tributary Alternative B.

Source: SEWRPC.

Each of the alternative flood control plans, with the exception of the no action alternative, was evaluated assuming that flood control measures would be implemented along the north tributary. In addition to solving the flood damage problems along Southbranch Creek, these flood control alternatives were designed to provide an adequate outlet for the north tributary. Under Alternative Plans 2 and 3, it was assumed that Alternative Plan B—channel modification, enclosure and construction, culvert replacement, storm sewer construction, and detention storage—would be implemented on the north tributary. Alternative Plan B was selected since it has the lowest cost of the three alternatives for the north tributary. Alternative Plan 4 was evaluated assuming that Alternative Plan C—channel modification, enclosure and construction, culvert replacement, storm sewer construction, and maximum detention storage—would be implemented along the north tributary. As previously noted, Alternative Plan C for the north tributary would cost about $38,000 per year more to implement than Alternative Plan B. Therefore, in order to have a consistent basis for comparing the costs of flood control alternatives on Southbranch Creek, this additional $38,000 per year was included in the cost of Alternative Plan 4.

<u>Alternative Plan 1—No Action</u>: One alternative course of action that could be taken in response to the flood problem along Southbranch Creek is to do nothing—that is, to recognize the inevitability of extensive flooding but to deliberately decide not to mount a collective, coordinated program to abate the flood damages. Under existing land use and existing channel conditions, the average annual flood damages along the creek would approximate $12,200. Under

Map 146

ALTERNATIVE PLAN 2: COMBINATION OF CULVERT REPLACEMENT AND CHANNEL MODIFICATION ALONG SOUTHBRANCH CREEK

Source: SEWRPC.

planned, year 2000, land use and existing channel conditions, the average annual flood damages along the creek would approximate \$42,500. There are no monetary benefits associated with this alternative, and the average annual cost would be equivalent to the average of the existing and planned land use average annual flood damage costs, or \$27,250.

Alternative Plan 2—Combination of Culvert Replacement and Channel Modification: This alternative system for the resolution of the flood problem along Southbranch Creek is shown on Map 146. The alternative would be implemented in conjunction with Alternative B for the north tributary. The plan consists of replacing the existing culverts at W. Dean Road, N. 47th

Street, N. 51st Street, N. 54th Street, and N. 55th Street. At W. Dean Road, the two existing eight-foot-diameter corrugated metal pipes would be replaced by two 8-foot-wide by 8-foot-high reinforced concrete box culverts. At N. 47th Street, the existing 10-foot-wide by 7-foot-high corrugated metal pipe arch would be replaced with two reinforced concrete box culverts, each being 10 feet wide by 8 feet high. At N. 51st Street, the existing 9-foot-wide by 6-foot-high corrugated metal pipe arch would be replaced with two 8-foot-wide by 8-foot-high reinforced concrete box culverts. At N. 54th Street, the existing six-foot-diameter corrugated metal pipe would also be replaced with two 8-foot-wide by 8-foot-high reinforced concrete box culverts. At N. 55th Street, the existing five-foot-diameter corrugated metal pipe would be replaced by three 10-foot-wide by 6-foot-high reinforced concrete box culverts. The relatively large concrete boxes at N. 55th Street are required to limit the 100-year flood stage immediately west of 55th Street to an elevation that will not cause flooding along the north tributary following construction of the bypass channel along the west side of 55th Street.

In addition to the culvert replacement, the existing streambed would need to be lowered from about 0.7 foot to 4.0 feet along the 0.78-mile reach between N. 47th Street and W. Bradley Road. Between N. 47th Street and N. 51st Street, the channel would have a bottom width of 10 feet and side slopes of one on three. The channel invert in that reach would be concrete lined, while the sides would have a turf lining. Along the 0.16-mile-long reach between N. 51st Street and N. 54th Street, the channel would have a 10-foot-wide, concrete-lined bottom and grassed sides with a slope of one vertical on two horizontal. The channel side slopes would be lined with rip-rap up to the 10-year recurrence interval flood level to protect against erosion due to excessive velocities.

For the first 480 feet upstream of N. 54th Street, the channel would have a 10-foot-wide, concrete-lined bottom and concrete-lined sides with a slope of one vertical on two horizontal. The concrete lining would extend to an elevation two feet above the 100-year recurrence interval flood stage under planned land use conditions. In the next 275 feet of channel length, the bottom would gradually widen to 31 feet and the side slopes would steepen to one on one. The channel invert and side slopes would be paved with concrete. The next 100 feet of channel up to N. 55th Street would have a 31-foot bottom width and one on one side slopes, and would be completely concrete-lined. The concrete channel lining is required to sufficiently lower the 100-year recurrence interval water surface profile to prevent flooding in the reach, to lower the tailwater elevation at N. 55th Street so the culverts under the street can pass the 100-year recurrence interval flood without causing flooding along the north tributary, and to prevent erosion due to excessive velocities.

The channel between W. Bradley Road and N. 55th Street would be lowered from one to four feet and would be rip-rap lined up to the 10-year recurrence interval flood level. These latter channel improvements are needed to accommodate the W. Bradley Road culvert modifications planned by the City of Milwaukee to resolve upstream drainage and utility conflict problems, and to accommodate the new culverts that would be installed at N. 55th Street under this alternative.

All channel modifications in the 0.78-mile-long reach between N. 47th Street and W. Bradley Road could be contained within the limits of the existing drainage easement.

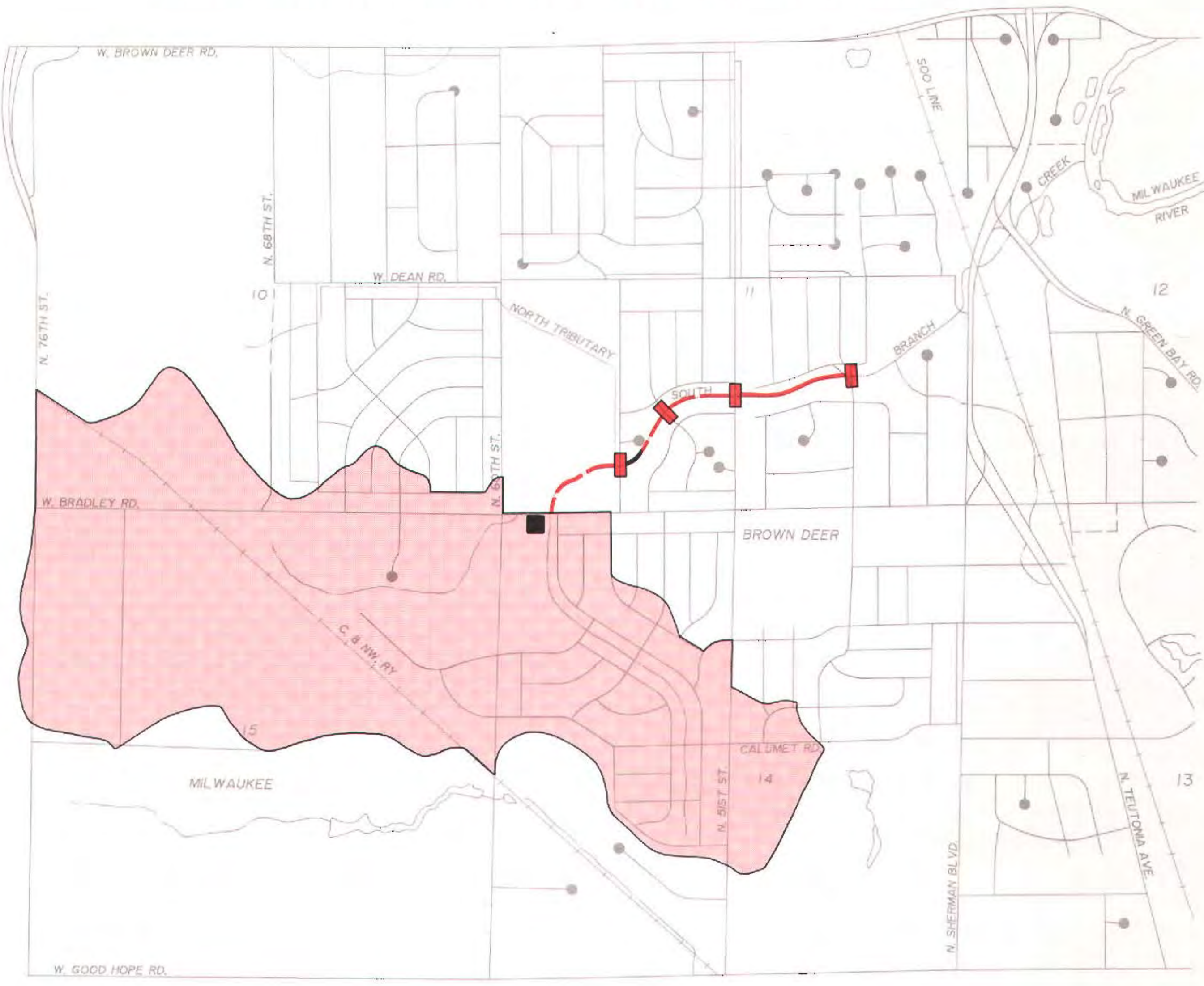
Implementation of this alternative would essentially eliminate all damages attendant to floods up to and including the 100-year recurrence interval event.

Utilizing an annual interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this alternative is estimated at $72,500. This cost consists of the amortization of the $1,116,000 capital cost—$498,000 for culvert replacement and $618,000 for channel modification—and $1,600 in annual operation and maintenance costs. The average annual flood abatement benefit is estimated at $27,250, resulting in a benefit-cost ratio of 0.38.

Alternative Plan 3—Combination of Maximum Storage, Culvert Replacement, and Channel Modification: This flood control alternative is shown on Map 147, and consists of providing both decentralized and centralized storage in that portion of the subwatershed located in the City of Milwaukee which drains to the storm sewer outfall located at W. Bradley Road at about N. 59th Street. Within this 0.94-square-

Map 147

ALTERNATIVE PLAN 3: COMBINATION OF MAXIMUM STORAGE, CULVERT REPLACEMENT, AND CHANNEL MODIFICATION ALONG SOUTHBRANCH CREEK



LEGEND

- CULVERT REPLACEMENT
- CHANNEL MODIFICATION-TURF LINING
- CHANNEL MODIFICATION-CONCRETE LINING
- CHANNEL MODIFICATION-RIPRAP LINING
- DETENTION STORAGE FACILITY
- AREA IN WHICH DECENTRALIZED STORAGE IS TO BE PROVIDED FOR ALL NEW DEVELOPMENT

GRAPHIC SCALE
0 1000 2000 FEET

Source: SEWRPC.

mile drainage basin, all new development would be required to have storage facilities that would limit the amount of stormwater runoff to existing land use levels. For cost-estimating purposes, it was assumed that three detention basins, each handling runoff from about 80 acres and being about two to three acres in size, would be required. In addition to the onsite, or decentralized, storage, this alternative calls for the construction of a 12-acre-foot detention reservoir on the Thoreau School property located on the southeast corner of the intersection of N. 60th Street and W. Bradley Road. The outlet for this reservoir would be the storm sewer outfall located under W. Bradley Road.

In order to minimize the 100-year recurrence interval outflow from the pond, it was assumed that the width of the planned 20-foot by 9-foot storm sewer outfall under W. Bradley Road would be reduced to 15 feet. Both the culvert invert and the 0.19-mile reach of channel downstream of W. Bradley Road would be lowered under planned conditions.

In addition to the provision of stormwater storage facilities, this alternative calls for the replacement of the existing culverts at N. 47th Street, N. 51st Street, N. 54th Street, and N. 55th Street. At N. 47th Street, the existing 10-foot-wide by 7-foot-high corrugated metal pipe arch would be replaced with two 8-foot-wide by 8-foot-high reinforced concrete box culverts. At N. 51st Street, the existing 9-foot-wide by 6-foot-high corrugated metal pipe arch would be replaced with two 8-foot-wide by 6-foot-high reinforced concrete box culverts. At N. 54th Street, the existing six-foot-diameter corrugated metal pipe would also be replaced with two 8-foot-wide by 6-foot-high reinforced concrete box culverts. At N. 55th Street, the existing five-foot-diameter corrugated metal pipe would be replaced with three 10-foot-wide by 6-foot-high reinforced concrete box culverts. The relatively large culverts at N. 55th Street are required to limit the 100-year recurrence interval flood stage immediately west of N. 55th Street to an elevation that will not cause flooding along the north tributary following construction of the bypass channel along the west side of 55th Street.

Finally, this alternative plan calls for channel modifications along the 0.78-mile reach between N. 47th Street and W. Bradley Road. Between N. 47th Street and N. 51st Street, the existing streambed would be lowered by about 0.7 foot to 2.9 feet, and the resulting channel would have a bottom width of 10 feet and side slopes of one on three. The channel invert would be concrete lined, and the side slopes would have a turf lining. Between N. 51st Street and a point 620 feet upstream of N. 54th Street, the existing streambed would be lowered by about 0.7 foot to 2.9 feet, and the resulting channel would have a 10-foot-wide bottom with a concrete lining and grassed sides with a slope of one on two. The channel side slopes would be lined with rip-rap up to the 10-year recurrence interval flood level to protect against erosion due to excessive velocities. The following 235 feet of channel extending up to N. 55th Street would be lowered 0.4 foot to 0.7 foot, and the resulting channel would have a 10-foot-wide concrete-lined bottom and concrete-lined side slopes of one on two. The concrete lining would extend to an elevation two feet above the 100-year recurrence interval flood stage under planned land use conditions. The concrete channel lining in this reach is required to sufficiently lower the tailwater elevation at N. 55th Street so the culverts under the street can pass the 100-year recurrence interval flood without causing flooding along the north tributary, and also to prevent erosion due to excessive velocities. Between N. 55th Street and W. Bradley Road, the existing streambed would be lowered by one to four feet, with the resulting channel having a four-foot bottom width and side slopes of one on two. The channel would be lined with rip-rap up to the 10-year recurrence interval flood level. The channel modifications in this last reach are needed to accommodate the W. Bradley Road storm sewer outlet modifications planned by the City of Milwaukee to resolve upstream drainage and utility conflict problems, and to accommodate the proposed culverts at N. 55th Street. All channel modifications along the 0.78-mile-long reach between N. 47th Street and W. Bradley Road would be contained within the limits of the existing drainage easement.

Implementation of this alternative would essentially eliminate all damages attendant to floods up to and including the 100-year recurrence interval event.

Utilizing an annual interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this alternative is estimated at $105,500. This cost consists of the amortization of the $1,304,000 capital cost—$260,000 for centralized storage, $179,000 for decentralized storage, $335,000 for culvert replacement, and $530,000 for channel modification—and $22,800 in annual operation and maintenance costs. The average annual flood abatement benefit is estimated at $27,250, resulting in a benefit-cost ratio of 0.26.

Alternative Plan 4—Combination of Culvert Replacement and Channel Modification with Additional Storage on the North Tributary: This alternative system for the resolution of the flood problem along Southbranch Creek is shown on Map 148. The alternative would be implemented in conjunction with Alternative C for the north tributary. The plan consists of replacing the existing culverts at N. 47th Street, N. 51st Street, N. 54th Street, and N. 55th Street. At N. 47th Street, the existing 10-foot-wide by 7-foot-high corrugated metal pipe arch would be replaced with two reinforced concrete box culverts, each being 8 feet wide by 8 feet high. At N. 51st Street, the existing 9-foot-wide by 6-foot-high corrugated metal pipe arch would be replaced with two 8-foot-wide by 6-foot-high reinforced concrete box culverts. At N. 54th Street, the existing six-foot-diameter corrugated metal pipe would also be replaced with two 8-foot-wide by 6-foot-high reinforced concrete box culverts. At N. 55th Street, the existing five-foot-diameter corrugated metal pipe would be replaced by three 10-foot-wide by 6-foot-high reinforced concrete box culverts. The relatively large concrete boxes at N. 55th Street are required to limit the 100-year flood stage immediately west of 55th Street to an elevation that will not cause flooding along the north tributary following construction of the channel along the west side of 55th Street.

Under this alternative, the existing streambed would need to be lowered by 0.7 foot to 4.5 feet along the 0.78-mile reach between N. 47th Street and W. Bradley Road. Between N. 47th Street and N. 51st Street, the channel would have a bottom width of 10 feet and side slopes of one on three. The channel invert in that reach would be concrete lined, and the sides would have a turf lining.

Along the 0.16-mile-long reach between N. 51st Street and N. 54th Street, the channel would have a 10-foot-wide bottom with a concrete lining and grassed sides with a slope of one vertical on two horizontal. The channel side slopes would be lined with rip-rap up to the 10-year recurrence interval flood level to protect against erosion due to excessive velocities. For the first 620 feet upstream of N. 54th Street, the channel would have a 10-foot-wide concrete-lined bottom and side slopes of one vertical on two horizontal, with rip-rap lining up to the 10-year recurrence interval flood level. The following 235 feet of channel up to N. 55th Street would have a 10-foot-wide, concrete-lined bottom and concrete-lined sides with slopes of one vertical on two horizontal. The concrete lining would extend to an elevation two feet above the 100-year recurrence interval flood stage under planned land use conditions. The concrete channel lining is required to sufficiently lower the tailwater elevation at N. 55th Street so the culverts under the street can pass the 100-year recurrence interval flood without causing flooding along the north tributary, and to prevent erosion due to excessive velocities.

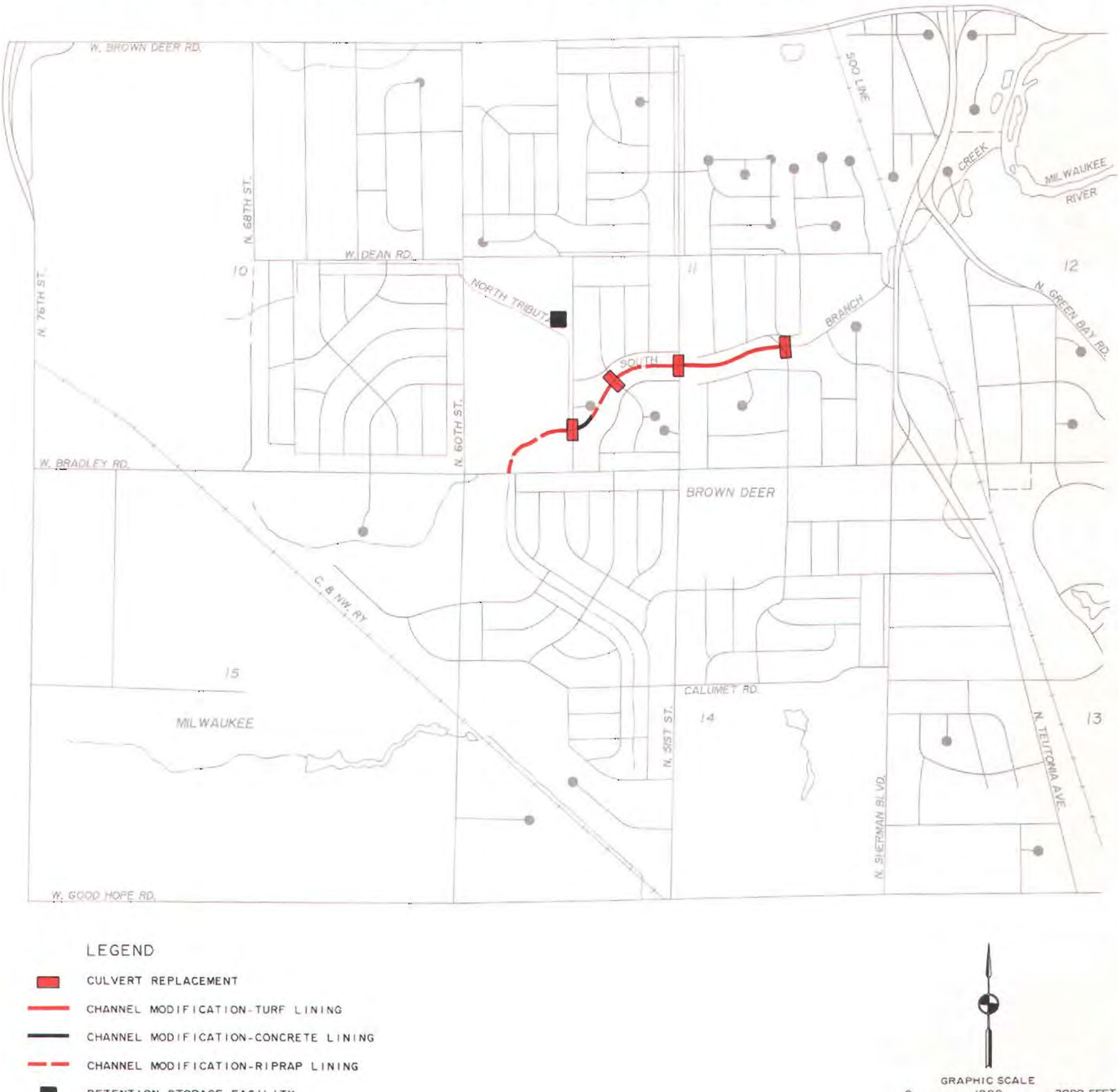
The channel between W. Bradley Road and N. 55th Street would be lowered by one to four feet and would be rip-rap lined up to the 10-year recurrence interval flood level. The resulting channel would have a four-foot bottom width and side slopes of one on two. These channel improvements are needed to accommodate the W. Bradley Road culvert modifications planned by the City of Milwaukee to resolve upstream drainage and utility conflict problems, and to accommodate the new culverts that would be installed at N. 55th Street under this alternative.

All channel modifications in the 0.78-mile-long reach between N. 47th Street and W. Bradley Road could be contained within the limits of the existing drainage easement.

Utilizing an annual interest rate of 6 percent and an amortization period and project life of 50 years, the average annual cost of this alternative is estimated at $93,700. This cost consists of the amortization of the $1,265,000 capital cost—$335,000 for culvert replacement, $530,000 for channel modification, and $400,000 for detention storage—and $13,400 in annual operation and maintenance costs. The detention storage cost is for provision of a detention basin on the north tributary at N. 55th Street. That basin is a component of north tributary Alternative C, but not north tributary Alternative B, which was assumed to be implemented for Southbranch Creek Alternatives 2 and 3. The incremental cost

Map 148

ALTERNATIVE PLAN 4: COMBINATION OF CULVERT REPLACEMENT AND CHANNEL MODIFICATION WITH ADDITIONAL STORAGE ON NORTH TRIBUTARY TO SOUTHBRANCH CREEK



Source: SEWRPC.

for the additional detention basin is assigned to Alternative 4 to provide a consistent basis for comparison with the other alternatives. The average annual flood abatement benefit is estimated at $27,250, resulting in a benefit-cost ratio of 0.29.

Evaluation of Flood Control
Alternatives for Southbranch Creek

The principal features of, and the cost and benefits associated with, each of the floodland management alternatives considered for Southbranch Creek are summarized in Table 85.

Excluding the no action alternative, all of the alternatives were found to be technically feasible. None of the alternatives produced a benefit-cost ratio of one or more. Part of the reason for the high costs of these alternatives is that they contain measures that are required to obtain an adequate outlet for the north tributary, in addition to measures required solely to eliminate flood damages along Southbranch Creek. The no action alternative, while offering the lowest cost, does nothing to alleviate the existing flood problem and does not represent a sound approach to flood control.

Implementation of Alternative Plan 2—combination of culvert replacement and channel modification—would serve to eliminate structure flooding up to a 100-year recurrence interval event while yielding the highest benefit-cost ratio of the alternatives considered. However, this alternative has two features that may not be desirable. First, the flood flows that would result are higher than those that would be experienced under existing channel conditions. This is primarily because the flood control measures to be carried out on the north tributary would result in a loss of floodwater storage. Under this alternative, these higher flood flows would produce stage increases of 0.2 foot to 0.5 foot downstream of W. Dean Road under a 100-year recurrence interval event. These increases would occur in both the Village of Brown Deer and the Village of River Hills. Accordingly, appropriate legal arrangements would have to be made with all property owners affected by the stage increase. The second less desirable feature is the more extensive concrete lining that would be required along the channel side slopes between N. 54th and N. 55th Streets. This would result in a less aesthetically appealing channel for the property owners along this reach.

Alternative Plan 3—combination of maximum storage, culvert replacement, and channel modification—has the highest cost of the flood control alternatives considered. This alternative would also produce higher flood flows than would be expected under existing channel conditions, although they would be less than under Alternative Plan 2. These higher flows would result in stage increases of up to 0.3 foot between W. Dean Road and N. 47th Street in the Village of Brown Deer. These higher stages should, however, be contained within the existing drainage easement. In addition, implementation of the decentralized storage element of this alternative may be a problem. A stable, long-term commitment to a decentralized storage policy by the local units of government is uncertain. It is also unlikely that a decentralized storage policy could, as a practical matter, be applied to every increment of urban land development within that subbasin. Finally, one desirable aspect of Alternative Plan 3 is that sediment removal within the detention basins may be expected to provide some water quality benefits.

Alternative Plan 4—combination of culvert replacement and channel modification with additional detention storage on the north tributary—while costing more than Alternative 2, has several benefits over that alternative. First, implementation of this alternative may be expected to result in a small decrease in flood flows in comparison to those that would be expected under existing channel conditions. Therefore, opposition to the plan by the Village of River Hills is less likely. Second, construction of a second detention basin on the north tributary would provide some added water quality benefits due to sediment removal. Finally, less concrete lining would be employed along the channel side slope than under Alternative Plan 2, thereby resulting in a more visually appealing channel for the adjoining property owners.

Recommended Flood Control System for Southbranch Creek

Based upon consideration of the technical feasibility, economic viability, environmental impacts, potential public acceptance, and practicality of each of the alternatives considered, it is recommended that Alternative Plan 4—combination of culvert replacement and channel modification with additional storage on the north tributary—be adopted for Southbranch Creek; and that Alternative Plan C—channel modification, enclosure, and construction, culvert replacement, storm sewer construction, and maximum detention storage—be adopted for the north tributary of Southbranch Creek.

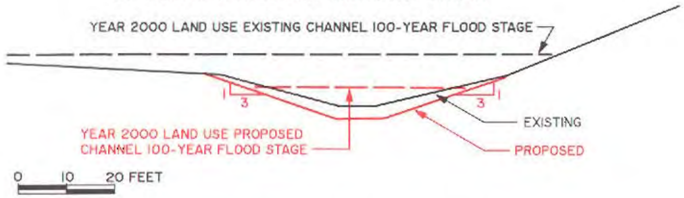
The total capital cost of the recommended flood control plan is estimated at $2,815,000 in 1986 dollars. Annual operation and maintenance costs are estimated at $27,000. For Southbranch Creek alone, the total capital cost is estimated at $1,265,000, while the annual operation and maintenance cost is estimated at $14,000, yielding a benefit-cost ratio of 0.29. The recommended plan is shown graphically on Map 149.

The peak flood profile that would be attendant to planned land use and planned channel conditions is shown in Figure 65.

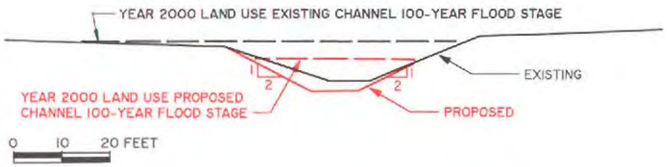
Map 149

RECOMMENDED FLOOD CONTROL SYSTEM PLAN FOR SOUTHBRANCH CREEK

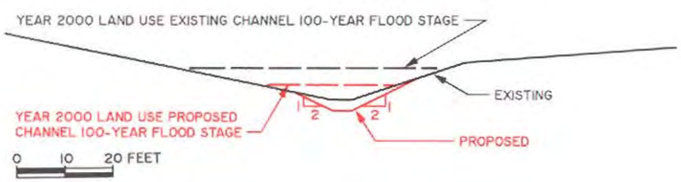
TYPICAL CROSS SECTION OF THE EXISTING AND PROPOSED CHANNEL ALONG SOUTHBRANCH CREEK BETWEEN N. 47TH STREET AND N. 51ST STREET



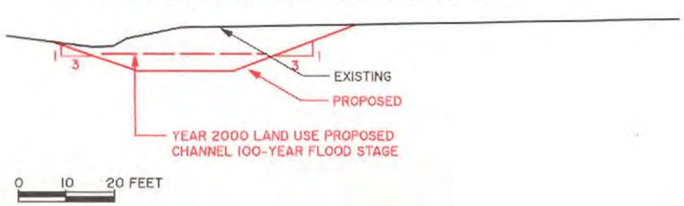
TYPICAL CROSS SECTION OF THE EXISTING AND PROPOSED CHANNEL ALONG SOUTHBRANCH CREEK BETWEEN N. 51ST STREET AND N. 55TH STREET



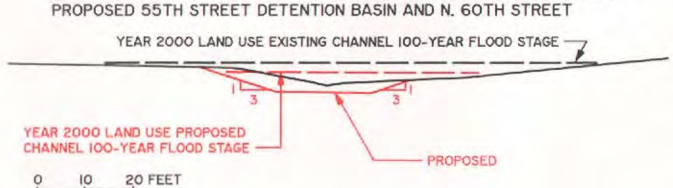
TYPICAL CROSS SECTION OF THE EXISTING AND PROPOSED CHANNEL ALONG SOUTHBRANCH CREEK BETWEEN N. 55TH STREET AND W. BRADLEY RD.

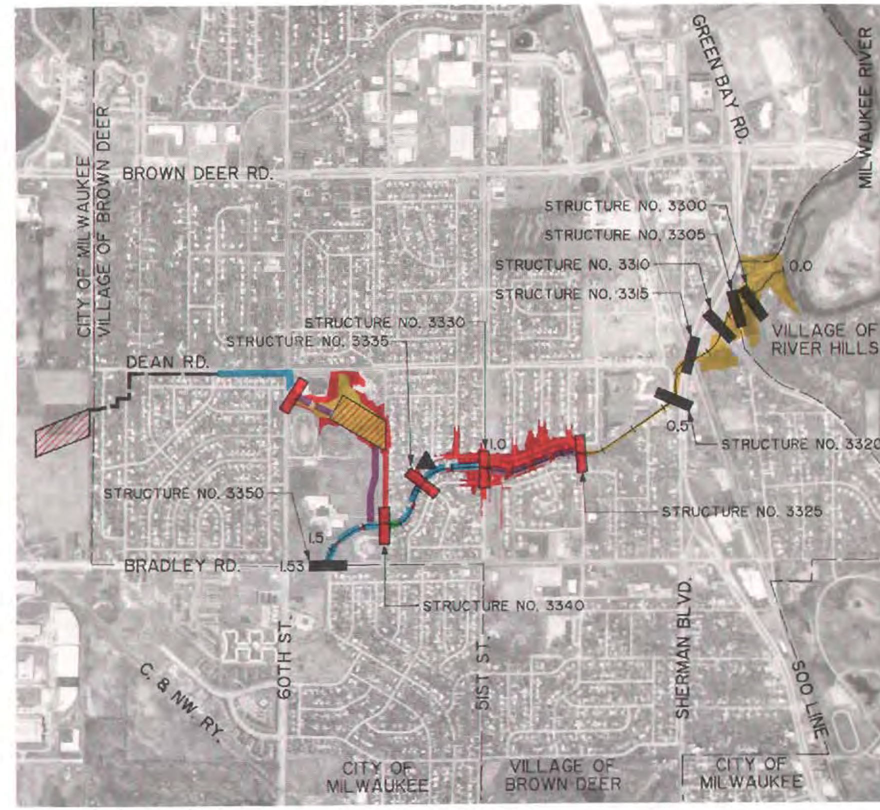


TYPICAL CROSS SECTION OF THE PROPOSED DETENTION BASIN OVERFLOW CHANNEL ALONG N. 55TH STREET



TYPICAL CROSS SECTION OF THE EXISTING AND PROPOSED CHANNEL ALONG THE NORTH TRIBUTARY TO SOUTHBRANCH CREEK BETWEEN THE PROPOSED 55TH STREET DETENTION BASIN AND N. 60TH STREET





LEGEND

- 100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS
- 100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS
- 1.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING
- CHANNEL MODIFICATION-TURF LINED
- CHANNEL MODIFICATION-CONCRETE LINED
- CHANNEL MODIFICATION-RIPRAP LINED
- NEW CHANNEL CONSTRUCTION
- CHANNEL ENCLOSURE
- STORM SEWER CONSTRUCTION
- CULVERT REPLACEMENT
- BACKWATER GATE
- DETENTION STORAGE FACILITY

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR SOUTHBRANCH CREEK IS SHOWN IN APPENDIX H

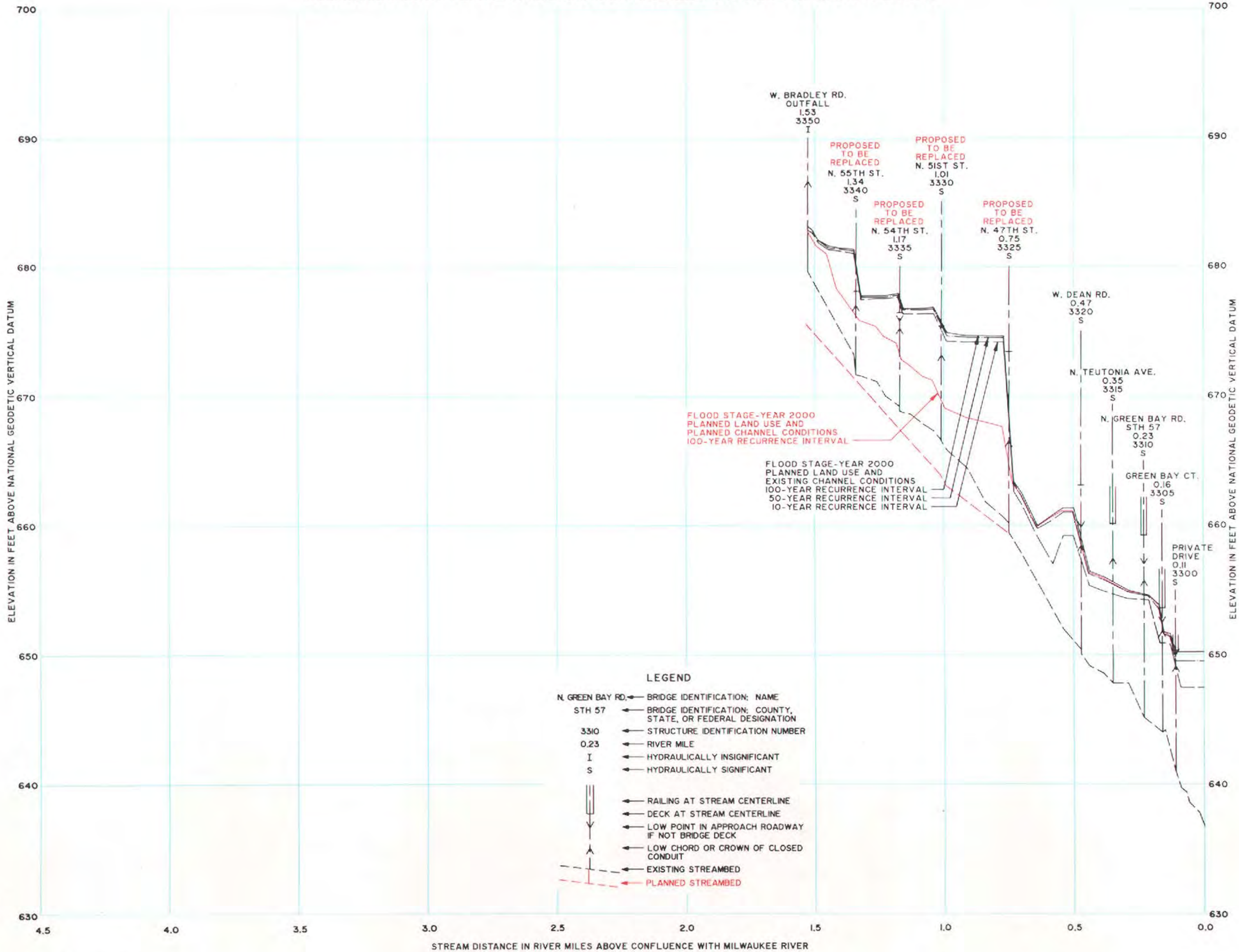
NOTE: DUE TO MAP SCALE LIMITATIONS, THE DIFFERENCE BETWEEN THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS, AND THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS, MAY NOT APPEAR ON THIS MAP. WHERE NO DIFFERENCE APPEARS REFERENCE SHOULD BE MADE TO THE FLOOD STAGE PROFILE SHOWN BELOW

DATE OF PHOTOGRAPHY: APRIL 1986

Source: SEWRPC.

Figure 65

RECOMMENDED PLAN FLOOD STAGE PROFILE FOR SOUTHBRANCH CREEK



Source: SEWRPC.

Implementation of the recommended plan would essentially eliminate all flood-related damages to existing structures along Southbranch Creek and the north tributary of Southbranch Creek for floods up to and including the 100-year recurrence interval event under planned land use conditions.

The recommended plan for Southbranch Creek consists of replacing the existing culverts at N. 47th Street, N. 51st Street, N. 54th Street, and N. 55th Street. The number and sizes of the replacement culverts would be as follows: 1) two 8-foot-wide by 8-foot-high reinforced concrete box culverts at N. 47th Street; 2) two 8-foot-wide by 6-foot-high reinforced concrete box culverts at N. 51st Street; 3) two 8-foot-wide by 6-foot-high reinforced concrete box culverts at N. 54th Street; and 4) three 10-foot-wide by 6-foot-high reinforced concrete box culverts at N. 55th Street. Channel modifications would also be required along the 0.78-mile-long reach between N. 47th Street and W. Bradley Road. Within this reach, the existing streambed would be lowered by 0.7 foot to 4.0 feet. Between N. 47th Street and N. 51st Street, the channel would have a bottom width of 10 feet and side slopes of one on three. The channel invert would have a concrete lining, while the side slopes would be turf lined. Between N. 51st Street and a point located about 620 feet upstream of N. 54th Street, the channel would have a bottom width of 10 feet and side slopes of one on two. The channel invert would be concrete lined, and the side slopes would be lined with rip-rap up to the 10-year recurrence interval flood level and with turf above that. Along the next 235 feet up to N. 55th Street, the channel would have a bottom width of 10 feet and side slopes of one on two. Within this reach, the channel would be concrete lined to an elevation two feet above the 100-year recurrence interval flood stage. Finally, between N. 55th Street and W. Bradley Road, the channel would have a bottom width of about four feet and side slopes of one on two. The channel would be lined with rip-rap up to the 10-year recurrence interval flood level and with turf above that.

The recommended plan for the north tributary of Southbranch Creek consists of the construction of two stormwater detention basins. One basin would provide about 26.5 acre-feet of storage and would be located on presently open land in the City of Milwaukee to the west of N. 68th Street extended and south of W. Dean Road extended. The second basin would provide about 28.0 acre-feet of storage and would be located south of Dean Elementary School in the Village of Brown Deer. The outlet of the existing 48-inch-diameter culvert which carries the north tributary between N. 55th Street and Southbranch Creek would be provided with a backwater gate to prevent backwater from Southbranch Creek from partially filling this second detention basin. Also, an overflow spillway constructed at this second basin would discharge to a grassed channel constructed along the west side of N. 55th Street and discharging to Southbranch Creek. This channel would be turf lined, with a bottom width of five feet and side slopes of one on three. A pedestrian bridge would be provided across this channel to maintain the existing access to Brown Deer High School from the east. In addition to the two detention basins, the recommended plan for the north tributary includes the following measures: 1) construction of 2,140 feet of 42-inch-diameter reinforced concrete relief storm sewer along the route of an existing storm sewer between N. Edge O' Woods Drive and a point about 780 feet west of N. 60th Street; 2) enclosure of 780 feet of open roadside drainage ditch with an 8-foot-wide by 4-foot-high reinforced concrete box culvert along W. Dean Road between the outlet of the 42-inch storm sewer and N. 60th Street; 3) replacement of the existing culverts under N. 60th Street with 370 feet of 10-foot-wide by 4-foot-high reinforced concrete box culvert; and 4) modification of about 500 feet of open channel east of N. 60th Street, with the streambed being lowered 1.5 to 2.5 feet and the resulting channel being turf lined, with a bottom width of 20 feet and side slopes of one on three.

It is recommended that when the bridge at N. Green Bay Court is replaced for transportation purposes, it be designed so as to accommodate the 10-year recurrence interval flood flow without overtopping the attendant roadway.

In addition to the flood control measures recommended for Southbranch Creek and the north tributary to Southbranch Creek, it is recommended that updated large-scale topographic maps be prepared for Southbranch Creek. Large-scale topographic maps currently available from the Village of Brown Deer were prepared in 1964 and do not reflect the significant amount of development that has occurred since then, including channel modifications of Southbranch

Creek. Such maps will be extremely useful in the detailed design of the recommended improvements and in evaluating low-lying areas in the tributary areas where storm sewer surcharging or local stormwater ponding could be a problem. Since these new maps would serve multiple purposes, none of the attendant costs have been assigned to the flood control plan.

Flood Control and Related Drainage System Plan Implementation

It is recommended that the structural measures developed for the abatement of flood problems along Southbranch Creek and the north tributary to Southbranch Creek be implemented through the cooperative efforts of the Village of Brown Deer, the City of Milwaukee, and the Milwaukee Metropolitan Sewerage District. The division of responsibilities among these three implementing agencies raised an issue not explicitly covered in the policy plan for stormwater drainage and flood control. That issue was the allocation of costs for the storage facilities to be constructed at locations remote from the stream courses included under the District jurisdiction, but which would result in significant reductions in the magnitude and cost of the improvements required on those streams under District jurisdiction. This particular issue was raised in conjunction with the construction of the storage facilities on the north tributary to Southbranch Creek. The tributary is not under the District jurisdiction, while Southbranch Creek is.

Three alternative ways of dividing the costs entailed were considered at the request of the Technical Advisory Committee. Under the first alternative, the local communities would be allocated the full cost of the storage facilities located on the north tributary to Southbranch Creek. Under the second alternative, the District would be allocated all of the cost of the storage facilities. Under the third alternative, the District and the local communities would share the cost of the storage facilities. The distribution of costs under each of these three options is shown in Tables 86 and 87.

Under the third alternative, the cost allocation varied for the two storage facilities. In the case of the detention basin located in the City of Milwaukee, the cost to be allocated to the District was based upon the estimated savings in improvement costs on Southbranch Creek attributable to the construction of the basin. That is, the cost of the improvements on Southbranch Creek was estimated both with and without the basin, and the difference in costs between those two estimates—$100,000—was allocated to the District. The cost analysis assumed that the second basin located in the Village of Brown Deer would be in place. However, review of the impacts of the basin in the City of Milwaukee indicated that the cost savings to the District would be similar under another alternative which did not provide for the construction of a basin in the Village of Brown Deer. In the case of the detention basin located in the Village of Brown Deer, the cost allocated to the District was based upon the difference between the costs which could be expected to be incurred by the Village of Brown Deer without detention and the costs with detention. That difference—$400,000—was allocated to the District. In the case of both basins, the operation and maintenance costs were allocated to the District and the local municipalities in which the basins were located based on the same cost-sharing percentages as for the capital costs.

In reviewing the three alternatives considered, as summarized in Table 86, it was recommended by the Technical Advisory Committee that the capital cost of the storage facilities in this particular instance be allocated to both the District and the local municipalities, and that the operation and maintenance cost be borne by the local municipalities. In making that recommendation, the Committee noted that the construction of the basins would benefit not only the local municipalities that would be resolving local drainage and flooding problems on streams under their jurisdiction through the use of a storage alternative, but also the District, which would incur a lower cost to resolve drainage and flooding problems on the stream under its jurisdiction. Regarding operation and maintenance costs, the Committee concluded that the local communities should be responsible for the operation and maintenance of any storage facilities located on the streams under their jurisdiction, since those streams would be maintained by those communities and not the District. The District stream maintenance responsibilities would be remote from the storage facilities and could cause confusion in responsibility, as well as inefficiency in extending those responsibilities beyond the otherwise fixed jurisdiction.

Table 86

SUMMARY OF ALTERNATIVE PLAN CAPITAL COSTS FOR NORTH TRIBUTARY TO SOUTHBRANCH CREEK

Implementing Agency	Improvements	Estimated Capital Costs		
		Alternative 1 Local Storage Facility Cost	Alternative 2 District Storage Facility Cost	Alternative 3 Shared Storage Facility Cost
Milwaukee Metropolitan Sewerage District	28-acre-foot detention basin and overflow channel—proportional cost (including land acquisition)	$ --	$ 610,000	$ 400,000
	26.5-acre-foot detention basin—proportional cost (including land acquisition)	--	520,000	100,000
	Subtotal	$ --	$1,130,000	$ 500,000
City of Milwaukee	26.5-acre-foot detention basin—proportional cost (including land acquisition)	$ 520,000	$ --	$ 420,000
Village of Brown Deer	Channel modification	$ 20,000	$ 20,000	$ 20,000
	Culvert replacement, channel enclosure, additional truck storm sewer . . .	800,000	800,000	800,000
	28-acre-foot detention basin and overflow channel—proportional cost	610,000	--	210,000
	Subtotal	$1,430,000	$ 820,000	$1,030,000
Total		$1,950,000	$1,950,000	$1,950,000

Source: SEWRPC.

Based upon those determinations by the Technical Advisory Committee, it is recommended that the District design, construct, and maintain the major channel modifications recommended along the 0.78-mile reach of Southbranch Creek between N. 47th Street and W. Bradley Road. It is recommended that the District remove the culverts at N. 47th Street, N. 51st Street, N. 54th Street, and N. 55th Street. It is further recommended that the District work with the Village of Brown Deer on the design and construction of the 28-acre-foot stormwater detention basin and attendant overflow spillway channel to be located west of N. 55th Street, and work with the City of Milwaukee on the design and construction of the 26.5-acre-foot detention basin to be located west of N. 68th Street extended. The operation and maintenance costs of these two detention basins would be borne by the municipality in which each basin is located. The capital cost to be allocated to the District is based upon the estimated savings in improvement costs on Southbranch Creek attributable to the construction of the basin in the City of Milwaukee. In the case of the basin in the Village of Brown Deer, the difference between the lowest cost solution to problems on the northern tributary and the alternative providing for storage was allocated

Table 87

SUMMARY OF ALTERNATIVE PLAN OPERATION AND MAINTENANCE COSTS FOR NORTH TRIBUTARY TO SOUTHBRANCH CREEK

Implementing Agency	Improvements	Estimated Annual Operation and Maintenance Costs		
		Alternative 1 Local Storage Facility Cost	Alternative 2 District Storage Facility Cost	Alternative 3 Shared Storage Facility Cost
Milwaukee Metropolitan Sewerage District	28-acre-foot detention basin and overflow channel—proportional cost	$ --	$12,500	$ 8,200
	26.5-acre-foot detention basin—proportional cost . . .	--	11,700	2,200
	Subtotal	$ --	$24,200	$10,400
City of Milwaukee	26.5-acre-foot detention basin—proportional cost . . .	$11,700	$ --	$ 9,500
Village of Brown Deer	Channel improvements	$ 200	$ 200	$ 200
	Storm sewer	600	600	600
	28-acre-foot detention basin and overflow channel—proportional cost	12,500	--	4,300
	Subtotal	$13,300	$ 800	$ 5,100
Total		$25,000	$25,000	$25,000

Source: SEWRPC.

to the District. Also, it is recommended that the District prepare large-scale topographic maps for the areas along Southbranch Creek.

Finally, it is recommended that the Village of Brown Deer design, construct, and maintain the remaining flood control measures recommended for the north tributary of Southbranch Creek. It is further recommended that the Village design and construct four replacement roadway culverts over Southbranch Creek.

The capital and operation and maintenance costs associated with the various components of the recommended plan are summarized in Table 88 for Southbranch Creek and in Table 89 for the north tributary of Southbranch Creek.

Table 88

SUMMARY OF RECOMMENDED PLAN COSTS—SOUTHBRANCH CREEK

Implementing Agency	Improvements	Estimated Capital Cost	Estimated Annual Operation and Maintenance Cost
Milwaukee Metropolitan Sewerage District	Channel improvements including culvert removal	$564,000	$1,600
Village of Brown Deer	Road construction, culvert replacement	301,000	- -
	Total	$865,000	$1,600

Source: SEWRPC.

Table 89

SUMMARY OF RECOMMENDED PLAN COSTS—NORTH TRIBUTARY TO SOUTHBRANCH CREEK

Implementing Agency	Improvements	Estimated Capital Cost	Estimated Operation and Maintenance Cost
Milwaukee Metropolitan Sewerage District	28-acre-foot detention basin and overflow channel—proportional cost (including land acquisition)	$ 400,000	$ - -
	26.5-acre-foot detention basin—proportional cost (including land acquisition)	100,000	- -
	Subtotal	$ 500,000	$ - -
City of Milwaukee	26.5-acre-foot detention basin—proportional cost (including land acquisition)	$ 420,000	$11,700
Village of Brown Deer	Channel modification	$ 20,000	$ 200
	Culvert replacement, channel enclosure, additional trunk storm sewer	800,000	600
	28-acre-foot detention basin and overflow channel—proportional cost	210,000	12,500
	Subtotal	$1,030,000	$13,300
	Total	$1,950,000	$25,000

Source: SEWRPC.

Chapter IX

EVALUATION OF ALTERNATIVE AND SELECTION OF RECOMMENDED FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN—MENOMONEE RIVER WATERSHED

INTRODUCTION

The drainage and flood control policy plan companion to this system plan recommends that the Milwaukee Metropolitan Sewerage District assume jurisdiction for eight perennial streams in the Menomonee River watershed. These eight streams, totaling 56.9 miles in length, consist of the Menomonee River, Woods Creek, Honey Creek, Underwood Creek, the South Branch of Underwood Creek, Dousman Ditch, the Little Menomonee River, and Butler Ditch. Of these eight streams, all but one, Woods Creek, were studied under the comprehensive watershed planning program for the Menomonee River watershed completed by the Commission in 1976.[1] Hydrologic and hydraulic analyses were conducted for these streams, and alternative flood control measures evaluated, under the watershed study. The system plan herein presented represents a refinement of the watershed plan.

Each of the eight streams within the Menomonee River watershed for which the District has assumed jurisdiction by adopting the policy plan is considered in the following sections of this chapter. Data are presented on existing and probable future drainage and flood control problems; alternative and recommended flood control and related drainage improvement measures; and recommended implementation actions.

OVERVIEW OF THE STUDY AREA

The Menomonee River watershed is located largely within western Milwaukee County and eastern Waukesha County, with smaller portions extending into southeastern Washington County and southwestern Ozaukee County. The watershed includes all or portions of the Cities of Brookfield, Greenfield, Mequon, Milwaukee, New Berlin, Wauwatosa, and West Allis; the Villages of Butler, Elm Grove, Germantown, Greendale, Menomonee Falls, and West Milwaukee; and the Towns of Brookfield, Germantown, Lisbon, and Richfield. From its origin in a large woodland-wetland area located in the northeastern corner of Germantown, the Menomonee River flows in a generally southeasterly direction for a distance of about 29.4 miles to its confluence with the Milwaukee River in the City of Milwaukee. The Menomonee River drains an area of about 135.7 square miles, as shown on Map 150. The extent of the watershed area within each minor civil division involved is given in Table 90.

The planned land use conditions utilized in the system planning effort assume that the watershed will be about 60 percent urbanized by the design year of the system plan. The remaining rural lands would be located primarily in the northern portion of the watershed.

Specific information on certain pertinent characteristics of the watershed, such as hydrologic soil types, land slopes, and land use appears in Chapter II of this report. Data on the stream system and subwatersheds are provided, for each of the eight perennial streams studied, in later sections of this chapter, along with the alternative evaluation and recommended plan description.

FLOODING AND RELATED DRAINAGE PROBLEMS

As documented in the Menomonee River watershed study and as observed during floods occurring subsequent to the publication of that report, overland flooding of buildings has occurred periodically along the Menomonee River in the Cities of Milwaukee and Wauwatosa, along Honey Creek in the Cities of Wauwatosa and West Allis, and along Underwood Creek in the Village of Elm Grove and the City of Brookfield. In addition, secondary flooding of basements due to sanitary sewer backup or infiltration through

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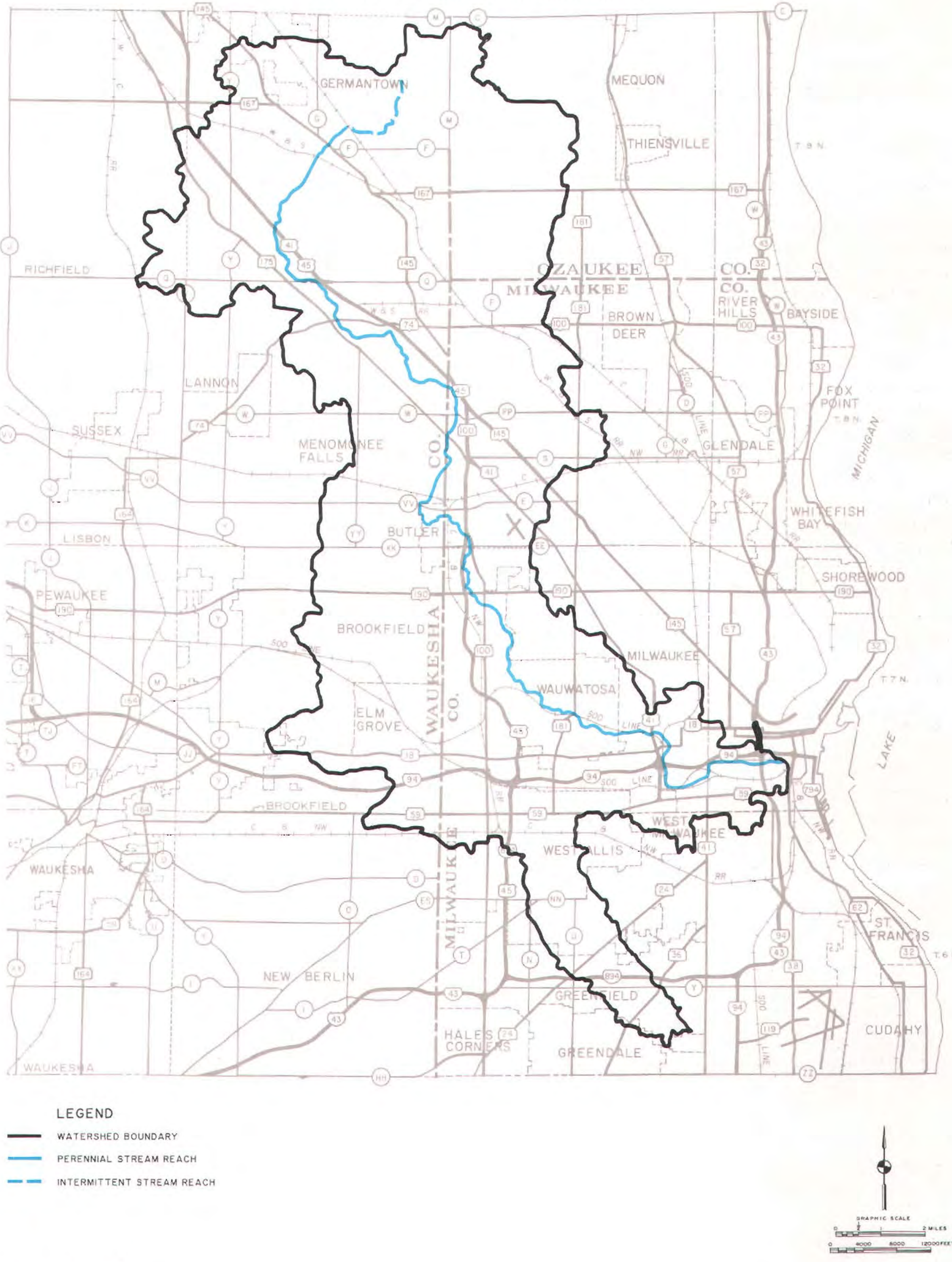
[1] *See SEWRPC Planning Report No. 26.* *A Comprehensive Plan for the Menomonee River Watershed, Volume One, Inventory Findings and Forecasts, and Volume Two, Alternative Plans and Recommended Plan, October 1976.*

Map 150

THE MENOMONEE RIVER WATERSHED



Source: SEWRPC.

Table 90

AREAL EXTENT OF CIVIL DIVISIONS IN THE MENOMONEE RIVER WATERSHED

County or Civil Division	County or Civil Division Area Included Within Watershed (square miles)	Percent of Subwatershed Area Within County or Civil Division
Milwaukee County		
City		
Greenfield	2.77	2.0
Milwaukee	31.32	23.1
Wauwatosa	13.35	9.8
West Allis	6.83	5.0
Village		
Greendale	0.10	0.1
West Milwaukee	0.61	0.5
Ozaukee County		
City		
Mequon	11.70	8.6
Washington County		
City		
Milwaukee	0.03	0.1
Village		
Germantown	29.33	21.6
Town		
Germantown	0.74	0.6
Richfield	1.55	1.1
Waukesha County		
City		
Brookfield	13.54	10.0
New Berlin	0.68	0.5
Village		
Butler	0.82	0.6
Elm Grove	3.27	2.4
Menomonee Falls	18.58	13.7
Town		
Brookfield	0.18	0.1
Lisbon	0.31	0.2
Total	135.71	100.0

Source: SEWRPC.

foundation walls has occurred along the Menomonee River in the Cities of Milwaukee and Wauwatosa and in the Villages of Germantown and Menomonee Falls, along Honey Creek in the Cities of Wauwatosa and West Allis, along Underwood Creek in the Cities of Wauwatosa and Brookfield and the Village of Elm Grove, and along Dousman Ditch in the City of Brookfield and the Village of Elm Grove.

Prior to the construction of channel modifications and the enclosure of a portion of the South Branch of Underwood Creek, street flooding occurred due to overflow of the South Branch and to storm sewer backups caused by high water levels in the South Branch.

The locations with the most severe existing flooding problems are areas of intensive residential, commercial, and industrial development along the Menomonee River in the Cities of Milwaukee and Wauwatosa from Hawley Road through Harmonee Avenue and along Underwood Creek in the Village of Elm Grove from the Milwaukee-Waukesha County line to Marcella Street.

Flood damages have been minimized in those floodland areas which have been kept in essentially natural open space uses compatible with occasional inundation. Prime examples of this are the Milwaukee County park land lying along portions of the Menomonee River, the Little Menomonee River, Underwood Creek, and Honey Creek and the Village of Menomonee Falls park land along the Menomonee River.

Structural flood control works which have been constructed over the past approximately sixty years have served to greatly alleviate overland flooding problems. Structural works constructed by the Milwaukee Metropolitan Sewerage District include channel modifications and sheet steel floodwalls along the 1.5-mile reach of the Menomonee River in the City of Milwaukee downstream from N. 45th Street; channel modifications and channel enclosures along a 6.6-mile reach of Honey Creek in the Cities of Greenfield, Milwaukee, Wauwatosa, and West Allis; channel modifications along the 2.5-mile reach of Underwood Creek in the City of Wauwatosa; and channel modifications and enclosure along a 1.6-mile reach of the South Branch of Underwood Creek in the Cities of Brookfield, Wauwatosa, and West Allis. Also, concrete and sheet steel floodwalls were constructed along the Menomonee River in the City of Milwaukee near S. 27th Street by the Falk Corporation in 1962.

There are areas in the Menomonee River watershed which experience localized stormwater drainage problems, as opposed to overland flooding problems. Such drainage problems may be related to overland flooding problems in cases where storm sewer outlets are constructed below streambed elevations or where high flood levels submerge storm sewer outlets, causing surcharging of storm sewers located off-stream. Although a comprehensive analysis of the storm sewer systems tributary to the streams studied here is beyond the scope of this planning program, the plan does address the interaction between the stream system and the stormwater drainage system to the extent that existing storm sewer outlet invert elevations are considered in establishing streambed grades in areas where channel modification is recommended and that submergence of storm sewer outfalls may be reduced or eliminated in stream reaches where channel modifications are recommended.

The flood of April 21, 1973, which was of a magnitude approximating the 100-year recurrence interval flood under planned land use and existing channel conditions at the U. S. Geological Survey stream gage at N. 70th Street in the City of Wauwatosa, caused overland flooding problems throughout much of the urban area within the watershed. That flood occurred after many of the major public and private flood control projects currently in existence within the watershed had been constructed; therefore, it offers a reasonable representation of the degree and type of overland flooding which would be experienced during a large flood under existing stream channel conditions.

The flood of August 6, 1986, while causing considerable damage and disruption within the watershed due to storm sewer backups and stormwater drainage problems, had a recurrence interval of less than 50 years at the Wauwatosa gage. Within the Menomonee River watershed, the overland flooding effects of that event were, therefore, not as severe as those of the April 1973 flood.

<u>Problem Areas Along the Menomonee River During the Flood of April 21, 1973</u>

Since the construction of the channel modifications and floodwalls, the most severe overland flooding along the Menomonee River in the Cities of Milwaukee and Wauwatosa occurred on April 21, 1973. Extensive flooding of residential, commercial, industrial, and governmental buildings occurred in the area bounded by N. Hawley Road in the City of Milwaukee through W. Harwood Avenue in the City of Wauwatosa. Flooding was concentrated in the area north of the river to W. State Street, with buildings on both sides of W. State Street being flooded. There were scattered areas of secondary basement flooding in the City of Wauwatosa from W. Harwood Avenue to W. Hampton Avenue, but costly damage from overland flooding was avoided due to the preservation in open uses of the Milwaukee County park lands along both sides of the river in that reach. In the Village of Menomonee Falls, secondary flooding of buildings located along the Menomonee River occurred near the northern boundary of the Village in the vicinity of N. Grand Avenue. Basement flooding northeast of the intersection of W. Fond du Lac Avenue and N. Lilly Road was attributable to stormwater drainage problems due in part to inadequate hydraulic capacity of a culvert beneath W. Fond du Lac Avenue.

<u>Problem Areas Along Woods Creek During the Flood of April 21, 1973</u>

Flooding which occurred at the Milwaukee County Stadium in April of 1973 may be attributed to stormwater drainage problems.

<u>Problem Areas Along Honey Creek During the Flood of April 21, 1973</u>

In April of 1973, secondary flood damage to basements of residential and institutional buildings occurred along the reach of Honey Creek in the City of Wauwatosa from W. St. Jude Court (extended), which is located just north of W. Wisconsin Avenue, to S. 84th Street just north of the East-West Freeway (IH 94). Basement flooding at St. Jude Church at the downstream end of that reach was probably due to a combination of overland flooding and sewer backup.

Secondary flooding of residential, commercial, and industrial buildings due to storm and sanitary sewer backups occurred in the area along the enclosed portion of the stream in the City of West Allis bounded by W. Lapham Street and W. National Avenue and S. 83rd Street and S. 85th Street. Additional secondary flooding of residential and commercial buildings was reported in the vicinity of the W. Oklahoma Avenue crossing of Honey Creek. At the time of the April 1973 flood, portions of the old Honey Creek channel adjacent to the enclosed reach had not yet been filled and the channel modifications in the upper portion of the subwatershed south of IH 894 were still under construction.

<u>Problem Areas Along Underwood Creek During the Flood of April 21, 1973</u>

Secondary flooding was experienced in the City of Wauwatosa along the north side of Underwood Creek immediately downstream of the Zoo

Freeway (USH 45) and along Underwood Creek Parkway Drive near the Waukesha-Milwaukee County line. The latter area is adjacent to the reach of Underwood Creek that was undergoing a major channelization project at the time of the April 1973 flood.

Extensive areas of basement and first floor inundation due to overland flooding involving residential, industrial, and institutional buildings, and scattered concentrations of secondary flooding were reported in April 1973 along the entire reach of Underwood Creek within the Village of Elm Grove. That reach extends from N. 124th Street (extended) to W. North Avenue. The Village carried out minor channel modifications in the reach near its eastern boundary in late 1973, but no major flood control projects have been undertaken along Underwood Creek in the Village since that time.

In the City of Brookfield, some overland and secondary flooding of basements occurred along N. Clearwater Drive and in the vicinity of W. Woodbridge Road, W. Indian Creek Parkway, and N. Kevenauer Drive. In addition, first floor flooding occurred at the northernmost building of the W. A. Krueger Company complex, which is located south of W. Blue Mound Road near the Waukesha-Milwaukee County line. That flooding was attributed to the southerly flow of floodwaters from Underwood Creek across W. Blue Mound Road.

Problem Areas Along Dousman Ditch During the Flood of April 21, 1973

In the City of Brookfield and Village of Elm Grove floodwaters from Dousman Ditch overtopped Pilgrim Parkway at several locations in the 1973 flood, and a few incidents of secondary basement flooding at scattered residences were reported.

Problem Areas Along the Little Menomonee River During the Flood of April 21, 1973

Primarily due to the preservation of floodlands in open space uses along the Little Menomonee River within the study area, there were no significant flood damages reported as a result of the flood of April, 21, 1973.

Problem Areas Along Butler Ditch During the Flood of April 21, 1973

No flooding of buildings was reported in the City of Brookfield or the Villages of Butler or Menomonee Falls during the 1973 flood. North Lilly Road in the City of Brookfield was, however, overtopped.

Problem Areas During the Flood of August 6, 1986

Within the watershed, the heaviest rainfall from the storm of August 6, 1986, was concentrated in Milwaukee County in a one- to four-mile-wide band extending from Mitchell International Airport through the northern portion of the City of Wauwatosa and the near northwest side of the City of Milwaukee just south of Timmerman Field. The 24-hour rainfall total within that band ranged from about 4.0 inches on the fringe to 6.84 inches at Mitchell International Airport. Because the most intense rainfall was concentrated over only a portion of the basin, flood flows in the streams under District jurisdiction were relatively low, while local stormwater runoff amounts in the areas of intense rainfall were high. As a result, damages within the watershed were primarily caused by the inadequacy of the major and minor stormwater drainage facilities to carry the runoff from the intense rainfall. In the most severely affected areas, the rainfall amounts were in excess of the 100-year recurrence interval event, which is the cost-effective design storm for the major drainage system. The nature of the flood problems is highlighted by the fact that residences located as far as two miles away from any major watercourse experienced flooded basements.

The damages to buildings within the subwatersheds considered in this report were primarily caused by the inadequacy of the combined major and minor storm sewer systems plus associated sanitary sewer backup into basements. Within the City of Wauwatosa, which is completely contained within the watershed, an estimated 600 homes were damaged and the average damage amount was estimated to be $2,500. Of the 600 damaged homes, approximately 100 incurred structural damage. The Milwaukee County Stadium and associated parking lots, located near its confluence of Woods Creek and the Menomonee River, were flooded. The Stadium Freeway (USH 41) near Woods Creek was closed from the Stadium Interchange to W. National Avenue.

Estimate of Damages for Design Flood Events

The costs of flooding in the Menomonee River watershed were estimated using damage cost curves prepared by the Regional Planning

Commission as described in Chapter III. The dollar amount of the flood damages is based upon the depth of inundation and the assessed valuation, required by law to approximate full market value, of the buildings involved. Damages to building contents are included in the total costs.

Flooding, as defined herein, includes basement flooding due to overland flow, yard inundation, and flooding above the first floor level. The number of existing residences that may be expected to experience direct flooding within the Menomonee River watershed is given in Table 91 by community.

Additional homes and commercial properties may, however, experience indirect flood damages through sanitary sewer backup. It should be noted that the flood control measures considered under this system plan are primarily intended to alleviate flood damages from direct overland flooding along the streams studied, as well as to provide an adequate outlet for local storm sewers and drainageways. These measures, although not specifically designed to do so, may be expected to reduce damages due to localized stormwater drainage problems or sanitary sewer backup.

The total average annual flood losses in the Menomonee River watershed, together with such losses anticipated under a 100-year recurrence interval flood event are listed in Table 92 by community.

FLOOD DISCHARGES AND STAGES

As noted in Chapter III of this report, the hydrologic model used for development of design discharges for the Menomonee River watershed simulates streamflow on a continuous basis, using recorded climatological data as input. Using this model, stream discharges were computed at 15-minute time intervals over a 49-year period from 1940 to 1988. Peak flood discharges were developed by performing discharge-frequency analyses of simulated annual peak discharges generated by the model using the log Pearson Type III method of analysis. This analysis was conducted for both existing and planned land use and existing channel conditions at a total of 89 locations throughout the watershed. The estimated peak flood discharges under existing (1985) and planned year 2000 land use and existing channel conditions are set forth in Table 93.

The hydrologic modeling conducted under this system planning effort represents a refinement of that conducted under the Commission's Menomonee River watershed study. As part of this system planning effort, a review was made of recorded streamflow records, some of which were not available at the time of the Menomonee River watershed study. These included records from two continuous recording streamflow gages operated by the U. S. Geological Survey in cooperation with the Commission and the Milwaukee Metropolitan Sewerage District. These two gages are located on the Menomonee River at Pilgrim Road in the Village of Menomonee Falls and on Underwood Creek at the Zoo Freeway (USH 45) in the City of Wauwatosa. These two gages were placed in operation in 1975, therefore records from these gages were not available for calibration purposes during the Menomonee River watershed study. Discharge-frequency analyses were made using the recorded data from these two gages and the results compared with the simulated flood flows from the watershed study. Based upon that comparison, a decision was made to recalibrate the hydrologic model by incorporating all available streamflow records through 1988. This recalibration required updating recorded weather data files for the years 1975 through 1988 for four stations. The results of the recalibration are represented by the discharge-frequency curves shown in Figures 66, 67, and 68 for the Menomonee River gage at N. 70th Street, the Menomonee River gage at Pilgrim Road and the Underwood Creek gage at the Zoo Freeway (USH 45), respectively. It should be noted that the N. 70th Street gage was the only continuous recording gage available for calibration purposes during the conduct of the Menomonee River watershed study. The discharges developed under that study are in close agreement with the recorded data for that gage.

A comparison of the flood discharges developed for the Menomonee River watershed under this system plan and those developed under the previous watershed study is provided in Table 94. As shown in this table, the revised flows are generally lower for the downstream portions of the watershed and generally higher for the upstream reaches, reflecting, in part, the

Table 91

STRUCTURE FLOODING ALONG THE MENOMONEE RIVER AND TRIBUTARIES

Stream	Community	Recurrence Interval (years)	Approximate Number of Existing Homes Flooded		Approximate Number of Existing Industrial, Commercial, or Institutional Properties Flooded	
			Existing Land Use, Existing Channel	Planned Land Use, Existing Channel	Existing Land Use, Existing Channel	Planned Land Use Existing Channel
Menomonee River	Milwaukee	10 50 100	0 3 57	0 21 91	0 2 24	0 11 37
	Wauwatosa	10 50 100	0 13 83	0 65 110	0 3 45	4 40 50
	Menomonee Falls	10 50 100	1 9 12	4 17 21	0 1 1	0 2 3
	Germantown	10 50 100	1 2 3	1 2 3	0 0 0	0 0 0
Honey Creek	Wauwatosa	10 50 100	0 0 0	0 0 0	0 0 0	0 0 1
	Greenfield	10 50 100	0 1 3	0 3 4	0 0 0	0 0 0
Underwood Creek	Elm Grove	10 50 100	0 3 5	1 6 27	4 10 14	10 15 25
	Brookfield	10 50 100	0 6 10	0 12 17	0 1 2	1 2 2
Little Menomonee River	Milwaukee	10 50 100	0 1 1	1 3 3	0 0 0	0 0 0

Source: SEWRPC.

Table 92

ESTIMATED FLOOD DAMAGES ALONG THE MENOMONEE RIVER AND TRIBUTARIES

Stream	Community	Average Annual Flood Damage		100-Year Recurrence Interval Flood Damage	
		Existing Land Use, Existing Channel	Planned Land Use, Existing Channel	Existing Land Use, Existing Channel	Planned Land Use, Existing Channel
Menomonee River	Milwaukee	$ 7,040	$ 16,200	$ 507,000	$ 928,000
	Wauwatosa	61,500	117,200	4,125,000	4,196,000
	Menomonee Falls	5,140	11,290	187,000	275,900
	Germantown	1,370	1,390	19,700	23,700
Honey Creek	Wauwatosa	0	2,200	0	150,000
	Greenfield	300	900	18,300	24,300
Underwood Creek	Elm Grove	21,300	146,400	547,000	1,844,000
	Brookfield	9,500	19,900	587,000	620,000
Little Menomonee River	Milwaukee	350	1,600	5,300	17,100

Source: SEWRPC.

results of the recalibration. In addition to recalibration, the differences between the discharges may be attributed to changes in existing land use conditions as well as physical changes made to the channel system since the watershed study was conducted. In addition, the flows developed under this system plan are based on an additional 14 years of simulated streamflow data. As noted above, this simulation encompassed a 49-year period from 1940 through 1988. The Menomonee River watershed study flows were based on 35 years of simulation, from 1940 through 1974. It can be noted that in most cases the differences in the estimated flows as developed under the two studies are 20 percent or less, with the exceptions generally being the small tributaries and the uppermost reach of major streams.

In the case of Butler Ditch, the hydrologic model used under this system planning effort was refined to better account for the natural floodplain storage and also to reflect a diversion of runoff from the Butler Ditch subwatershed to the Underwood Creek subwatershed. This diversion occurs under major storm events along a very flat area located along the South Branch of Butler Ditch south of W. Capitol Drive. In the case of Dousman Ditch, the natural and recently constructed storage which now exists upstream of W. Gebhardt Road was accounted for in the more recent modelling. In the case of Underwood Creek, the reduced flows from Dousman Ditch and a better accounting for existing storage upstream of the Soo Line Railroad Company (former Chicago, Milwaukee, St. Paul & Pacific Railroad) railway resulted in the significant change in estimated flows.

Flood stage profiles were developed for the 10-, 50-, and 100-year recurrence interval runoff events under planned land use and existing channel conditions. These profiles, which encompass the full 56.9 miles of streams studied, constitute a graphic representation of the flood stages under the specified recurrence interval discharges. In addition to providing an overall representation of flood stages relative to familiar points of reference such as the channel bottom and bridge deck surfaces, the profiles, because of their continuity, permit the convenient determination of flood stages at any point along the stream channel. Flood profiles for streams in the Menomonee River watershed are presented throughout this chapter, along with more detailed descriptions of the individual subwatersheds.

Table 93

FLOOD DISCHARGES FOR EXISTING AND YEAR 2000 LAND USE AND EXISTING CHANNEL CONDITIONS

Stream	Location	River Mile	Peak Flood Discharge (cfs)					
			Existing Land Use Existing Channel Condition			Year 2000 Planned Land Use Existing Channel Condition		
			10-Year	50-Year	100-year	10-Year	50-Year	100-Year
Menomonee River	Confluence with the Milwaukee River	0.00	8,560	13,400	15,900	9,330	14,300	16,800
	S. 32nd Street extended	2.45	8,380	13,100	15,600	9,130	14,000	16,400
	Upstream of confluence with Woods Creek	3.21	7,540	11,700	13,800	8,420	12,800	14,900
	Soo Line Railroad Company	4.24	6,850	10,700	12,700	7,800	11,700	13,700
	N. 70th Street	6.10	6,740	10,600	12,600	7,730	11,600	13,600
	Upstream of confluence with Honey Creek	6.24	5,010	7,700	9,080	5,800	8,710	10,200
	Upstream of confluence with Underwood Creek	8.39	2,900	3,960	4,450	3,480	4,780	5,390
	Upstream of W. Center Street extended	9.22	2,670	3,710	4,200	3,360	4,670	5,290
	Upstream of W. Burleigh Street	9.98	2,660	3,700	4,180	3,350	4,670	5,290
	Upstream of confluence with Grantosa Creek	10.66	2,480	3,520	3,990	3,220	4,510	5,130
	Upstream of W. Capitol Drive	11.38	2,460	3,480	3,970	3,200	4,470	5,070
	Upstream of confluence with Little Menomonee River	12.58	1,890	2,740	3,140	2,700	3,790	4,290
	Upstream of confluence with Butler Ditch	14.42	1,600	2,180	2,440	2,420	3,290	3,670
	W. Appleton Avenue	16.54	1,600	2,180	2,440	2,420	3,290	3,670
	Upstream of confluence with Dretzka Park Tributary	17.97	1,400	1,940	2,180	2,130	2,910	3,250
	Upstream of confluence with Lilly Creek	18.95	1,100	1,490	1,650	1,540	2,030	2,250
	Upstream of confluence with Nor-X-Way Channel	20.27	860	1,180	1,310	900	1,220	1,360
	Menomonee Falls Dam	21.89	700	1,010	1,150	730	1,040	1,180
	Woodlawn Avenue extended	22.71	670	1,000	1,150	700	1,000	1,150
	W. County Line Road	23.43	660	980	1,130	700	1,000	1,140
	Upstream of confluence with Willow Creek	24.67	630	840	930	890	1,110	1,220

Table 93 (continued)

Stream	Location	River Mile	Peak Flood Discharge (cfs)					
			Existing Land Use Existing Channel Condition			Year 2000 Planned Land Use Existing Channel Condition		
			10-Year	50-Year	100-year	10-Year	50-Year	100-Year
Menomonee River (continued)	Downstream of Lilac Lane	25.06	470	680	790	500	680	790
	Upstream of confluence with West Branch of Menomonee River	26.89	330	490	560	330	490	560
	Upstream of confluence with North Branch of Menomonee River	27.90	190	270	310	190	270	310
Little Menomonee River	Confluence with the Menomonee River	0.00	940	1,330	1,510	1,040	1,480	1,700
	W. Appleton Avenue	1.57	920	1,300	1,480	1,040	1,480	1,700
	W. Fond du Lac Avenue	2.56	960	1,360	1,550	1,140	1,610	1,820
	Upstream of confluence with Noyes Creek	3.07	520	730	820	740	990	1,100
	W. Good Hope Road	3.66	520	730	820	740	990	1,100
	W. Bradley Road	4.69	340	500	580	400	560	640
	W. County Line Road	6.95	340	490	550	470	650	730
	W. Donges Bay Road	8.01	270	390	440	300	440	500
	Upstream of confluence with Little Menomonee Creek	8.26	140	200	220	170	220	250
	W. Mequon Road	9.16	120	200	240	140	230	270
	Upstream of Sunnyvale Road extended	9.71	150	280	360	220	360	420
	W. Freistadt Road	10.22	100	190	250	100	190	250
Underwood Creek	Confluence with the Menomonee River	0.00	2,480	4,150	5,080	2,990	4,800	5,760
	USH 45	0.75	2,110	3,510	4,270	2,610	4,190	5,030
	Watertown Plank Road	1.50 1,950	3,150	3,790	2,290	3,620	4,310	
	Upstream of confluence with South Branch of Underwood Creek	2.54	600	1,050	1,300	860	1,370	1,640
	W. Juneau Boulevard	3.67	510	880	1,080	640	1,070	1,310
	W. North Avenue	4.82	460	820	1,030	620	1,050	1,280
	Upstream of confluence with North Branch of Underwood Creek	5.38	390	710	890	530	890	1,090
	Soo Line Railroad Company	6.32	300	520	650	420	680	820

Table 93 (continued)

Stream	Location	River Mile	Peak Flood Discharge (cfs)					
			Existing Land Use Existing Channel Condition			Year 2000 Planned Land Use Existing Channel Condition		
			10-Year	50-Year	100-year	10-Year	50-Year	100-Year
Underwood Creek (continued)	Upstream of confluence with Dousman Ditch	6.97	80	120	130	90	120	130
	Soo Line Railroad Company	7.68	66	72	74	66	72	74
South Branch of Underwood Creek	Confluence with Underwood Creek	0.00	1,400	1,990	2,260	1,520	2,030	2,260
	W. Schlinger Avenue tunnel outlet	1.08	930	1,220	1,340	980	1,300	1,430
	Upstream of branch in channel enclosure	1.65	480	570	610	540	650	690
Dousman Ditch	Confluence with Underwood Creek	0.00	230	390	480	310	510	620
	W. Juneau Boulevard extended	1.13	160	240	290	260	400	470
Honey Creek	Confluence with the Menomonee River	0.00	2,310	3,300	3,500	2,510	3,350	3,600
	W. Wisconsin Avenue	0.91	2,220	3,000	3,100	2,410	3,100	3,200
	IH 94	1.99	2,050	2,500	2,500	2,100	2,500	2,500
	W. Greenfield Avenue	3.10	1,760	2,200	2,200	1,810	2,200	2,280
	Footbridge	4.57	1,030	1,720	2,080	1,180	1,900	2,280
	W. Oklahoma Avenue	5.27	840	1,410	1,710	970	1,560	1,870
	Downstream of W. Howard Avenue	6.44	550	960	1,190	740	1,130	1,310
	IH 894	7.53	250	520	690	470	670	760
	Downstream side of W. Layton Avenue	7.80	220	470	620	420	600	680
	Upstream side of W. Layton Avenue	7.81	210	440	580	350	560	640
	Downstream side of Loomis Road	8.53	140	290	390	270	380	430
Butler Ditch	Confluence with the Menomonee River	0.00	340	640	830	470	780	950
	W. Hampton Road	1.02	310	570	740	320	590	760
	Downstream of Lilly Road	1.72	220	400	500	230	420	520
	Upstream of confluence with South Branch of Butler Ditch	2.49	190	350	450	190	350	450
	W. Lisbon Road	3.40	160	300	380	160	300	380

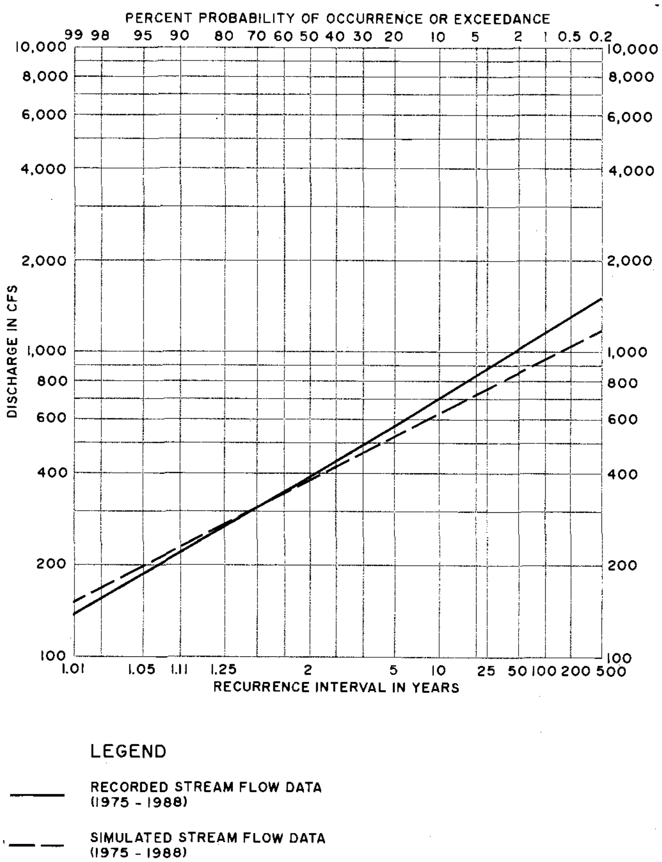
Table 93 (continued)

Stream	Location	River Mile	Peak Flood Discharge (cfs): Existing Land Use Existing Channel Condition 10-Year	50-Year	100-year	Year 2000 Planned Land Use Existing Channel Condition 10-Year	50-Year	100-Year
Woods Creek	Confluence with the Menomonee River	0.00	830	1,070	1,160	830	1,070	1,160
	Stadium Freeway enclosure	0.08	780	990	1,080	780	990	1,080
	Soo Line Railroad Company	0.265	790	1,020	1,120	790	1,020	1,120
	Outlet of 108-inch culvert	0.33	640	810	880	640	810	880
	Outlet of Veterans Administration Center tunnel	0.63	620	780	850	620	780	850
	Upstream of Veterans Administration Center tunnel	0.92	440	540	580	440	540	580

Source: SEWRPC.

Figure 66

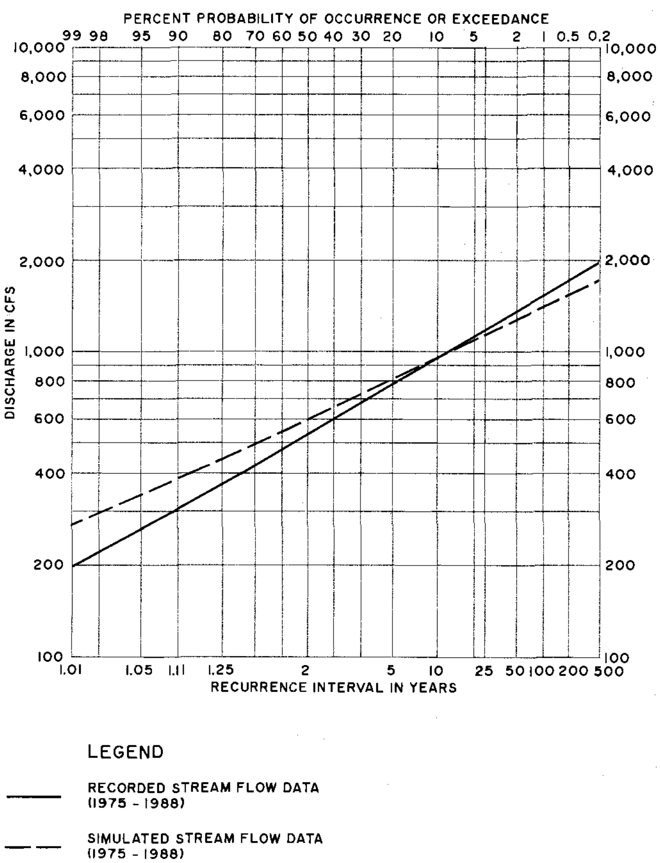
DISCHARGE FREQUENCY RELATIONSHIPS FOR THE MENOMONEE RIVER AT N. 70TH STREET



Source: SEWRPC.

Figure 67

DISCHARGE FREQUENCY RELATIONSHIPS FOR THE MENOMONEE RIVER AT PILGRIM ROAD



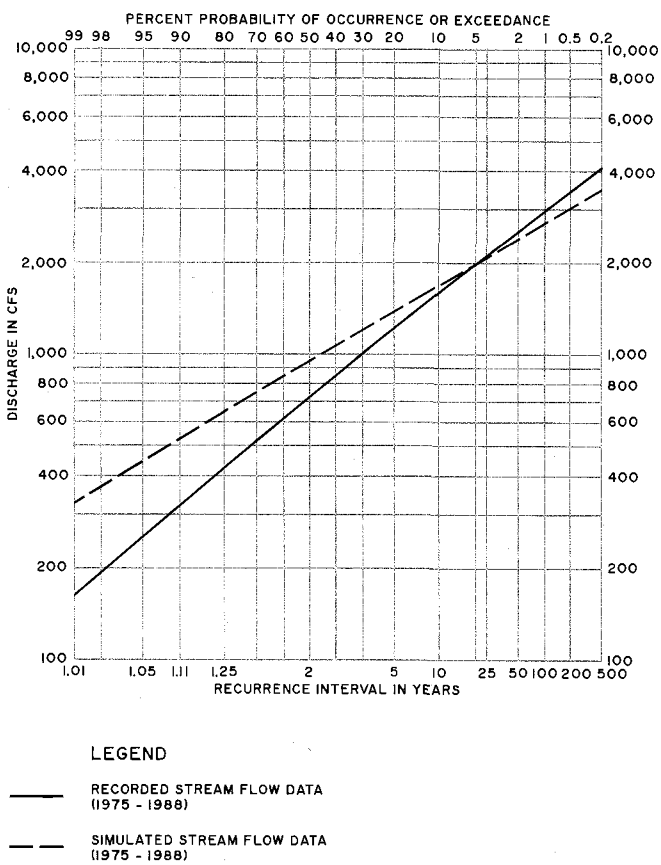
Source: SEWRPC.

OVERVIEW OF PREVIOUS COMMISSION-DEVELOPED ALTERNATIVE FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLANS FOR THE MENOMONEE RIVER WATERSHED

Flood control alternatives were previously evaluated for the Menomonee River and its major tributaries under the Commission's Menomonee River watershed study.[2] As already noted, with the exception of Woods Creek, all of the tributaries under District jurisdiction were also considered under the watershed study. The following six flood control measures were considered alone or in various combinations under the watershed study: 1) structure floodproofing and removal; 2) channel modification; 3) detention storage; 4) dikes and floodwalls; 5) bridge and culvert alteration or replacement; and 6) diversion of flood flows to Lake Michigan or to the District's deep tunnel inline storage system. In addition, a "No Action" alternative was considered. The evaluations indicated that the diversion alternative would be economically infeasible, and this alternative was eliminated from further consideration under the watershed plan. The recommended flood control system plan herein presented represents a refinement of the recommended plan formulated under the watershed study.

Figure 68

DISCHARGE FREQUENCY RELATIONSHIPS FOR UNDERWOOD CREEK AT USH 45



Source: SEWRPC.

Consideration of Detention Storage

The potential for a watershedwide system of detention storage sites was evaluated in the Menomonee River watershed study. The recommended plan included the provision of substantial storage, including the preservation of essentially all natural floodplain storage remaining in the watershed and the construction of one storage facility. In addition, under the watershed planning effort, the construction of storage was evaluated at 24 additional sites. Of these 24 additional sites, 10 were found to have some potential for reducing flood flows. An evaluation of these 10 additional sites is presented in the following paragraphs.

An initial screening identified 25 potential detention storage sites located in Milwaukee, Ozaukee, Washington, and Waukesha Counties. Those sites were located to protect areas of existing, as well as planned, urban development. Twenty-two of the sites were located directly on the watershed stream system whereas three sites, the Hartung Quarry in the City of Milwaukee, a gravel pit in the City of Wauwatosa, and an abandoned sewage treatment plant site in the Village of Menomonee Falls, were located "off-channel."

Based on the drainage area tributary to each site, the available storage volume at each site, and the proximity of each site to downstream flood-prone areas, the potential of each site to produce a significant reduction in downstream flood damages was evaluated. Twelve of the 25 potential sites were found to warrant further analysis. One of the remaining 12 sites, located in the City of Brookfield in U. S. Public Land Survey Section 2, Township 7 North, Range 20 East, at the confluence of Butler Ditch and the

[2]*Ibid.*

Table 94

COMPARISON OF PLANNED LAND USE AND EXISTING CHANNEL CONDITION FLOOD FLOWS: MENOMONEE RIVER WATERSHED STUDY AND MILWAUKEE METROPOLITAN SEWERAGE DISTRICT SYSTEM PLAN

Stream	Location	100-Year Recurrence Interval Flood Discharge (cfs)			
		River Mile	Menomonee River Watershed Study	Milwaukee Metopolitan Sewerage District System Plan	Percent Change
Menomonee River	Confluence with Milwaukee River	0.00	19,600	16,800	-14
	Soo Line Railroad	4.24	16,800	13,700	-18
	Upstream of confluence with Honey Creek	4.24	12,700	10,200	-20
	W. North Avenue	8.50	6,900	5,390	-22
	USH 45	12.88	4,730	4,290	-10
	W. Silver Spring Road	14.64	3,680	3,540	-4
	W. Appleton Avenue	16.54	3,640	3,670	+1
	Lilly Road	19.70	1,910	2,250	+18
	Menomonee Falls Dam	21.89	1,010	1,180	+17
Little Menomonee River	Confluence with Menomonee River	0.00	1,900	1,700	-10
	W. Good Hope Road	3.66	995	1,100	+10
	W. Bradley Road	4.69	605	640	+6
Underwood Creek	Confluence with Menomonee River	0.00	6,100	5,760	-6
	USH 45	0.75	6,100	5,030	-18
	N. 124th Street extended	2.54	1,940	1,640	-15
	W. North Avenue	4.82	1,940	1,280	-34
South Branch of Underwood Creek	Confluence with Underwood Creek	0.00	2,760	2,260	-18
Dousman Ditch	Confluence with Underwood Creek	0.00	1,310	620	-53
Honey Creek	Confluence with Menomonee River	0.00	3,490	3,600	+3
	W. Wisconsin Avenue	0.91	2,620	3,200	+22
	IH 94	2.04	2,620	2,500	-4
	W. Arthur Avenue	4.32	2,540	2,280	-10
	W. Howard Avenue	6.54	1,520	1,310	-14
Butler Ditch	Confluence with Menomonee River	0.00	1,550	950	-39

Source: SEWRPC.

South Branch of Butler Ditch just north of W. Capitol Drive, was eliminated as being economically infeasible.

The remaining 11 potential detention storage sites are listed in Table 95 and are shown on Map 151. Of those sites, the four which were judged to have the greatest potential positive impact on downstream flood damages were Site No. 1 on the Menomonee River in the Village of Germantown, Site No. 16 on the Little Menomonee River in the City of Mequon, Site No. 19, the Hartung Quarry, lying adjacent to the Menomonee River in the City of Milwaukee, and Site No. 22 on the Dousman Ditch in the City of Brookfield. The individual impacts of each of these four potential detention basins on downstream flood flows, stages, and damages, as well as the cumulative impact of all 11 potential detention basins, were evaluated under the watershed study. The findings of that evaluation are summarized below.

Site No. 1—On the Menomonee River in the Village of Germantown: This detention basin would be located at River Mile 23.43, upstream of CTH Q. The watershed study concluded that, under planned land use conditions, the provision of this basin could achieve a reduction in peak flood discharge of about 35 percent at Main Street in the Village of Menomonee Falls, about 1.6 miles downstream from the potential detention basin site, but a reduction in peak flood discharge of only about 3 percent could be anticipated at a location about 3.2 miles downstream from the basin. Since the completion of the watershed study, this site has been partially filled and currently would have even less impact than indicated in the watershed study. The 1.6-mile reach from the potential basin to Main Street includes an area of significant flood damages, but the next 1.6-mile reach downstream does not. The stream reach immediately downstream of the potential basin requires modification in order to provide for adequate storm sewer outlets. As shown in the subsequent section, the channel improvements required for stormwater drainage purposes would resolve the identified flooding problems in the reach immediately downstream of the basin. Thus, the provision of detention storage was not considered further at this site.

Site No. 16—On the Little Menomonee River in the City of Mequon: A detention basin could be created by an earthen embankment located on the Little Menomonee River at Freistadt Road in the City of Mequon at River Mile 10.22, about 3.22 miles upstream of the upper end of the reach under District jurisdiction. The potential basin would provide significant flood flow, flood stage, and flood damage reduction only within the City of Mequon along a stream reach outside of the District's jurisdiction. Therefore, this basin site was not considered further under this study.

Site No. 19—Hartung Quarry Adjacent to the Menomonee River in the City of Milwaukee: The Hartung Quarry, located off-stream, serves as a sanitary landfill site for the City of Milwaukee. Under conditions at the time of the conduct of the watershed study, it was estimated that utilization of this quarry for floodwater storage could reduce peak flood discharges under planned land use conditions by about 9 percent in the downstream flood-damage-prone reach in the City of Wauwatosa. Since that time, the available flood storage volume of the quarry has been reduced somewhat due to landfilling operations. The fact that the site has been used as a landfill and the fact that the quarry walls and bottom are a creviced stone which could provide direct pathways for polluted water to reach the groundwater reservoir, make this site unsuitable for floodwater storage use. In addition, pumping the stored water out of the deep quarry would be costly. Thus, this site is no longer considered to be viable as a storage facility location.

Site No. 22—Dousman Ditch in the City of Brookfield: As set forth in the watershed study, the outlet to this detention site would be located at W. Gebhardt Road in the City of Brookfield. The watershed study concluded that, under planned land use conditions, a detention basin at this location could substantially reduce peak flood flows, stages, and damages along downstream flood-prone channel reaches in the City of Brookfield and the Village of Elm Grove. Therefore, this detention storage site was included in this system plan.

System of Eleven Detention Basins: The four sites previously described, along with the additional seven warranting further consideration, were analyzed under the watershed study to determine the combined effects on flood flows, stages, and discharges on the Menomonee River

Table 95

SUMMARY OF POTENTIAL DETENTION AND RETENTION STORAGE SITES IDENTIFIED FOR THE MENOMONEE RIVER WATERSHED STUDY

Number[a]	Name	Location				Dam	Impoundment Data at Approximate Maximum Flood Stage			
		Stream	County	City, Village, or Town	River Mile	Street, Highway, or Other Description	Tributary Area (square miles)	Stage (feet NGVD)	Surface Area (acres)	Volume (acre-feet)
1	Germantown	Menomonee River	Washington	Village of Germantown	23.47	CTH Q	26.36	845.0	1,019	5,806
3	NXWC-Upstream	Nor-X-Way Channel	Washington	Village of Germantown	2.08	Wisconsin & Southern Railroad Company	2.58	790.0	213	1,470
6	Dretzka	Dretzka Park Tributary	Milwaukee and Waukesha	City of Milwaukee	0.48	Bradley Road	3.27	750.0	194	801
8	West Granville	Menomonee River	Milwaukee	City of Milwaukee	16.65	STH 175	49.40	740.0	145	635
9	Carmen	Menomonee River	Waukesha	Village of Menomonee Falls	15.00	Chicago & North Western Transportation Company	51.06	734.0	102	606
10	Butler Ditch-Downstream	Butler Ditch	Waukesha	Village of Menomonee Falls	0.00	Just upstream of the confluence with Menomonee River	5.06	780.0	242	1,793
13	Zoo Freeway	Menomonee River	Milwaukee and Waukesha	City of Milwaukee	12.88	USH 45	58.00	714.0	95	694
16	Mequon-Upstream	Little Menomonee River	Ozaukee	City of Mequon	10.18	Freistadt Road	1.40	755.0	381	5,861
19	Hartung Quarry	Menomonee River	Milwaukee	City of Milwaukee	10.15	Menomonee River Parkway	85.57	690.0	20	1,950
20	Tosa Gravel Pit	Underwood Creek	Milwaukee	City of Wauwatosa	1.35	Watertown Plank Road	17.18	- -	47	700
22	Gebhardt	Dousman Ditch	Waukesha	City of Brookfield	0.63	Gebhardt Road	3.78	830.0	376	1,366

[a]*Menomonee River Watershed Study numbering system.*

Source: SEWRPC.

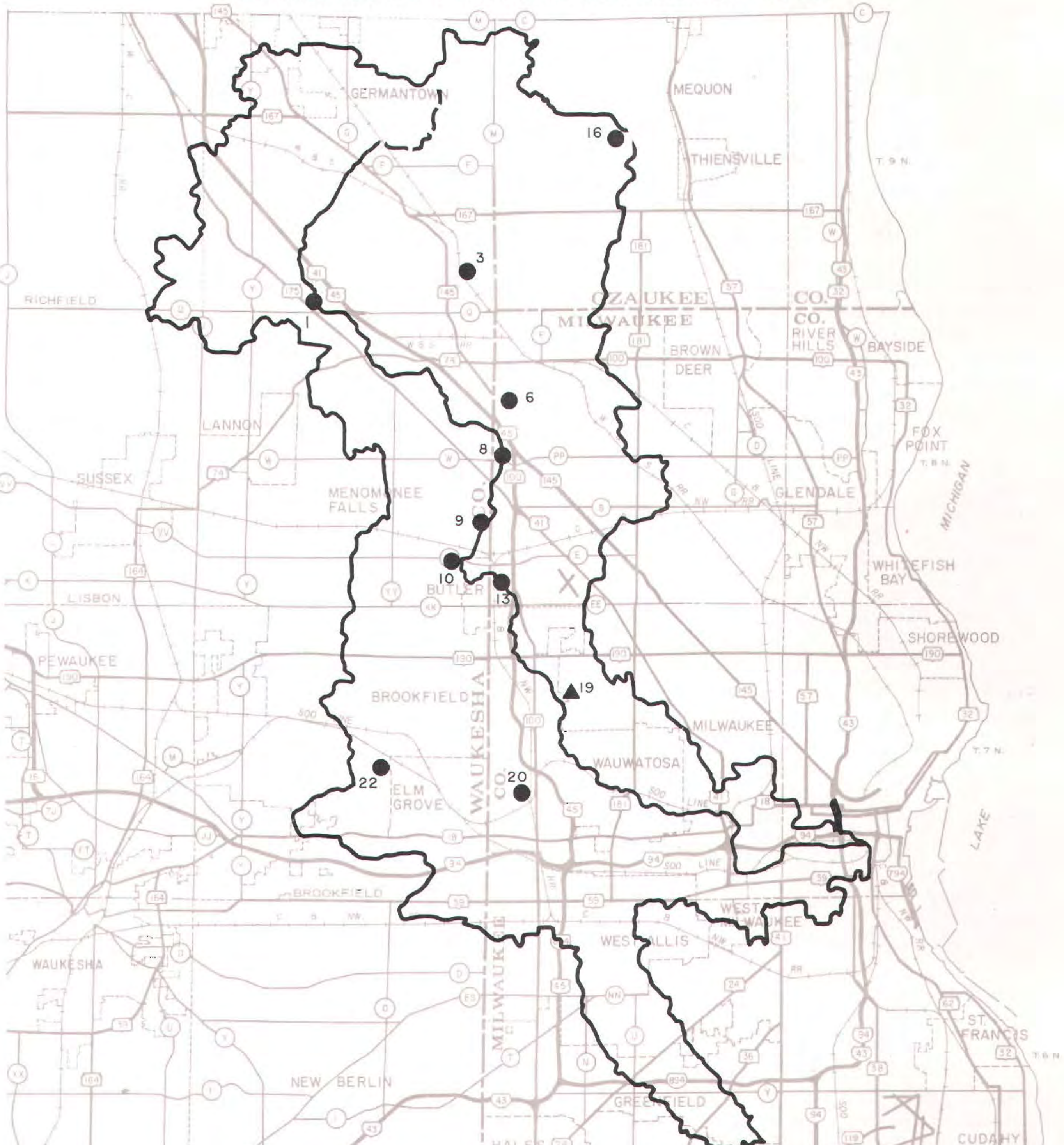
of all 11 basins acting as a system. The watershed study concluded that such a system of detention basins could achieve a peak flood flow reduction of about 30 percent along the flood-prone reach of the Menomonee River from about N. 70th Street downstream to N. Hawley Road under planned land use conditions, but that such a reduction in flow would not offer significant relief to critical flood-prone lands in the City of Wauwatosa and the City of Milwaukee. It was further concluded that the design of the improvements needed to fully resolve the identified problems would not be significantly reduced if the additional detention were constructed.

Reevaluation of the Menomonee River Watershed Study Conclusions Regarding Detention Storage

The Menomonee River watershed study concluded that storage in the watershed was important. However, the use of natural storage was considered the most cost effective and environmentally sensitive approach. The only location where the use of constructed detention storage was found to be technically feasible and economically justifiable was at Site No. 22 along the Dousman Ditch. That conclusion reached in the watershed study was found, upon reconsideration under this study, to still be sound.

Map 151

POTENTIAL DETENTION OR RETENTION STORAGE SITES
IDENTIFIED FOR THE MENOMONEE RIVER WATERSHED STUDY



LEGEND

● DETENTION STORAGE SITE LOCATION

▲ RETENTION STORAGE SITE LOCATION

20 IDENTIFICATION NUMBER FOR POTENTIAL SITE (SEE TABLE 95)

GRAPHIC SCALE

0 1 2 MILES

0 4000 8000 12000 FEET

Source: SEWRPC.

The evaluation of the effects of the provision of constructed detention storage on flood damage reduction as set forth in the watershed study was comprehensive. Although the study dealt primarily with centralized detention facilities, its conclusions would also be applicable to the "best case" scenario for decentralized detention basins located at, or upstream of, the sites considered for centralized detention. This situation exists because the larger central detention sites were strategically located to provide control of runoff from substantial areas of planned development, the same areas in which decentralized detention could be provided cost-effectively. The effects of decentralized detention on the timing and combination of flood hydrograph peaks from a subbasin can be approximated by the effects of a centralized detention basin situated at the outlet of the subbasin. In general, this overestimates, or presents the "best case" impact, of decentralized storage. Thus, although decentralized detention may be effective in resolving some local stormwater drainage and water quality problems within developing portions of the watershed along certain tributaries beyond the jurisdiction of the District, the results of the detention analyses conducted under the watershed study indicated that the impact of decentralized detention on reducing flood damages along the flood-damage-prone reaches of the major streams of the watershed would not be significant.

It should be noted that since the publication of the watershed study, several of the 11 detention storage sites identified in that study have had their available storage volume significantly reduced through filling. Such sites include No. 1 in the Village of Germantown, No. 10 in the Village of Menomonee Falls, and No. 19 in the City of Milwaukee. As already noted, the use of Site No. 19, the Hartung Quarry, is no longer considered feasible for floodwater storage, due to its use as a landfill, the nature of the creviced bedrock walls and bottom, and the high pumping costs associated with its use. In addition, Site No. 20 has been filled and thereby eliminated from consideration. As a result, the combined effect of the detention basins on reducing flood peaks as determined under the watershed study would be even less today, although the flow reductions determined under the watershed study can provide a "best case" representation of the effects of a similar amount of decentralized detention.

MENOMONEE RIVER MAIN STEM FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN

Hydrologic and hydraulic analyses of the main stem of the Menomonee River were previously conducted under the Commission's Menomonee River watershed study. That study also assessed existing and possible future flood problems along the stream, evaluated alternative measures to alleviate those problems, and included recommendations for the implementation of certain flood control measures. This system planning effort represents a refinement of that earlier study. Presented below are an overview of the subwatershed, a review of the previously considered flood control measures, and a refined recommended flood control plan for the Menomonee River.

Overview of the Watershed

As described previously, the Menomonee River drains an area of about 135.7 square miles and flows 29.4 miles in a generally southeasterly direction from its origin in the Village of Germantown to its mouth in the City of Milwaukee. More specifically, from its origin in the northeast corner of the Village of Germantown, the Menomonee River flows in a southerly direction for a distance of about 5.9 miles to W. County Line Road; thence in a southeasterly direction through the Village of Menomonee Falls to the Milwaukee-Waukesha County line at W. Calumet Road extended, a distance of about 5.4 miles; thence in a southerly direction to W. Silver Spring Drive in the Village of Butler, a distance of about 3.4 miles; thence in a southeasterly direction for about 8.6 miles to N. 70th Street in the City of Wauwatosa; and thence in an easterly direction for about 6.1 miles to its confluence with the Milwaukee River near N. 2nd Street and E. Chicago Street extended in the City of Milwaukee. Of the 29.4-mile reach described, 27.9 miles, or 95 percent, is classified as perennial, and 1.5 miles, or 5 percent, is classified as intermittent.

It is recommended in the policy plan companion to this system plan that 16.2 miles of perennial stream located within the current District limits be included under the District's jurisdiction. This 16.2-mile reach extends from the Falk Corporation dam in the City of Milwaukee, the upstream limit of the Menomonee River estuary, to the crossing of the Milwaukee-Waukesha County line just south of USH 45. The remainder of the

Menomonee River, which is located outside of the current District limits, but in an area defined in the policy plan as within possible future District limits, was found to meet the criteria for District jurisdiction. Moreover, any flood control measures carried out along the upper reaches of the Menomonee River may impact flood flows and stages and recommended flood control measures along the reach of the River under District jurisdiction. This additional reach of stream was accordingly included in the system planning effort.

In 1985, about 49 percent of the Menomonee River watershed was developed for urban use, including residential, commercial, institutional, and urban open space uses. Most of the developed land was concentrated in the southern portion of the watershed in Milwaukee and Waukesha Counties. The developed areas of the watershed in Milwaukee County are generally provided with a full range of municipal street improvements, including paved streets with curbs and gutters and attendant storm sewers. In Washington and Waukesha Counties some of the developed areas are provided with a full range of municipal street improvements, including paved streets with curbs and gutters and attendant storm sewers, while other developed areas are provided with paved streets with road ditches. In Ozaukee County drainage is generally provided with road ditches discharging to surface swales and natural watercourses.

The flood profile for the Menomonee River is shown as Figure 69. The extent of the 100-year recurrence interval flood hazard area under planned land use and existing channel conditions is shown on Map 152.

Evaluation of Alternative Flood Control and Related Drainage System Plans for the Menomonee River

The alternative flood control measures considered under the watershed study are presented below, together with their estimated costs. Based upon an evaluation of those alternatives, a final composite flood control plan was recommended for the Menomonee River in the watershed plan. That plan has been further refined as part of this system planning effort. The refined recommended plan as developed under this study is presented below by stream reach.

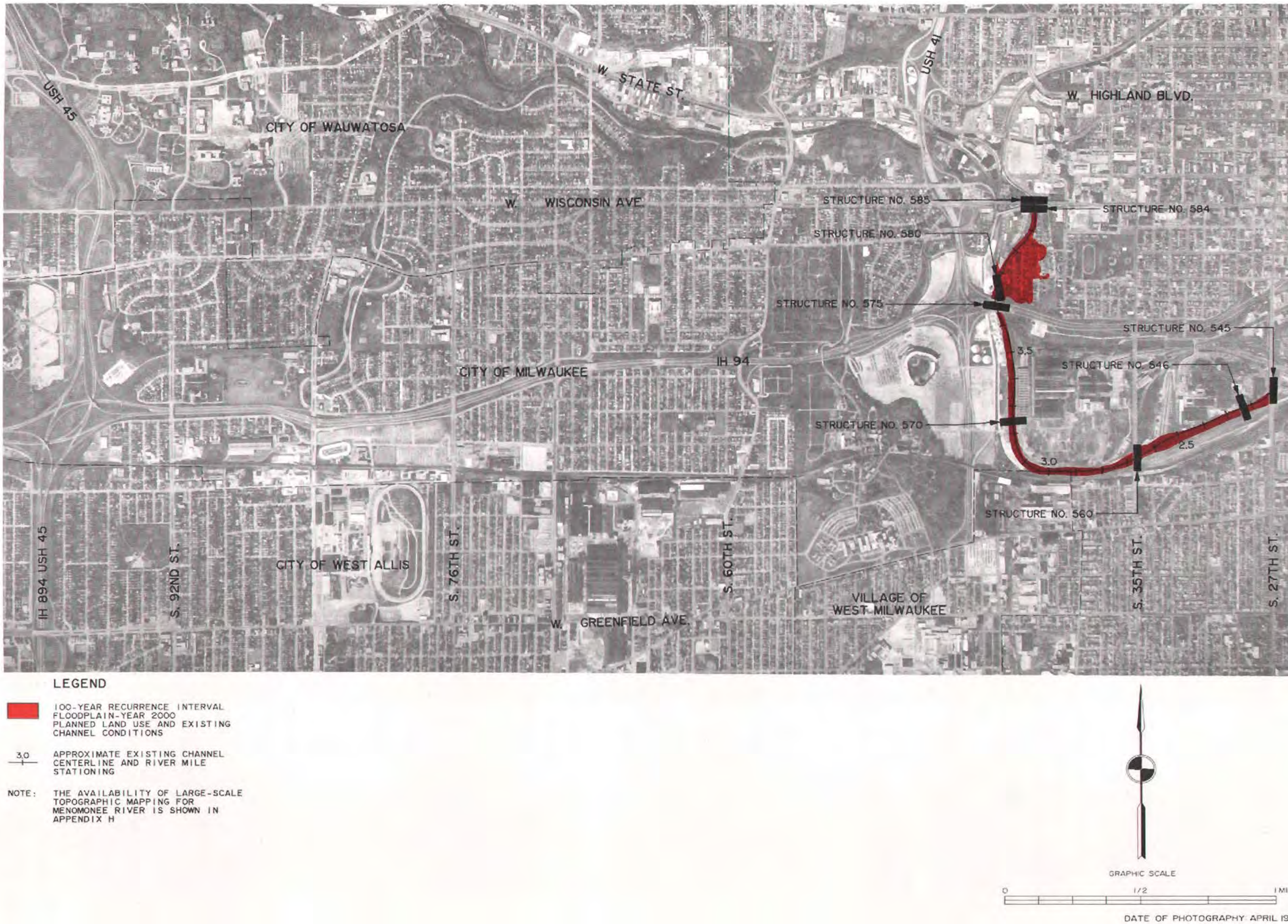
City of Milwaukee from the 27th Street Viaduct to IH 94: Because the extensive structural flood control measures which were constructed in this reach have significantly reduced the flood damage potential, it was not necessary that a full range of alternatives be examined under the watershed study. The watershed study concluded that the existing 0.50-mile-long floodwall at the Falk Corporation property near the 27th Street Viaduct and the existing 0.64-mile-long sheet pile floodwall on the east side of the river at the former Chicago, Milwaukee, St. Paul & Pacific Railroad Company yards had adequate heights to contain the 100-year recurrence interval flood stage under planned land use and channel conditions with a minimum of two feet of freeboard.[3] The watershed study also concluded that the 0.49-mile-long earthen dike along the south side of the former railway yards would be overtopped during a 100-year recurrence interval flood under planned land use and channel conditions.

Refined Flood Control System Plan: Under the watershed study, the 27th Street Viaduct was considered to be hydraulically insignificant. Subsequent hydraulic analyses conducted during the design of a replacement viaduct indicated that the concrete bases of the then-existing viaduct piers did create backwater. That backwater caused submergence of the Falk dam, located 0.12 mile upstream, under flood flow conditions. The replacement viaduct, which was completed in 1980, was designed to create about the same backwater effect under planned 100-year recurrence interval flood conditions as the former viaduct. Therefore, the Falk dam does not create a significant backwater effect under flood conditions and its removal is not necessary for

[3]*At the time of the watershed study, Commission standards recommended that dikes and floodwalls have two feet of freeboard above the 100-year recurrence interval flood stage. Since that time, Chapter NR 116 of the Wisconsin Administrative Code, dealing with floodplain management, was revised to require three feet of freeboard above the 100-year recurrence interval stage. Therefore, to meet State regulations, dikes and floodwalls must now have three, rather than two, feet of freeboard above the 100-year recurrence interval stage.*

Map 152

100-YEAR RECURRENCE INTERVAL FLOODPLAIN FOR THE MENOMONEE RIVER
UNDER YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS



LEGEND

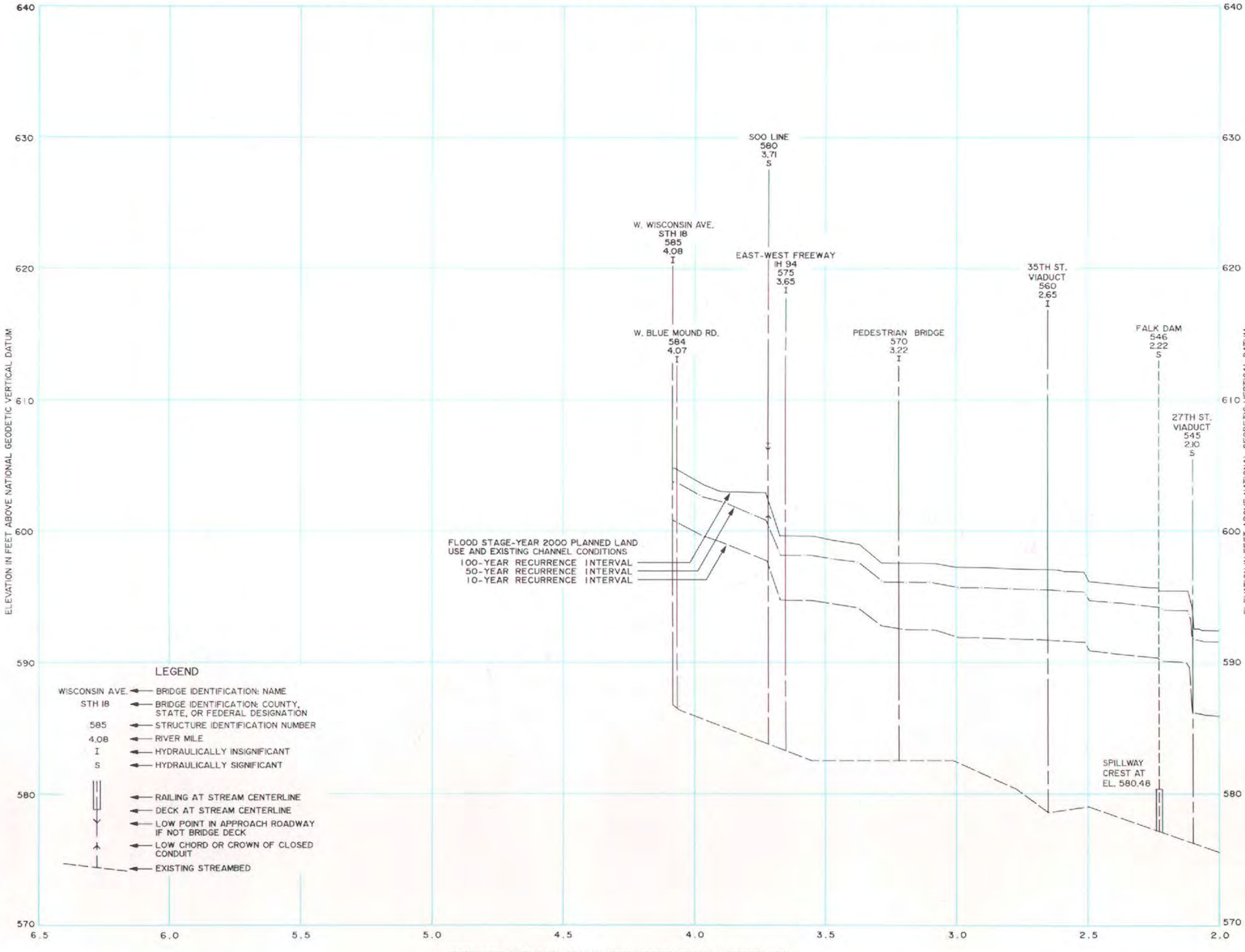
100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

3.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

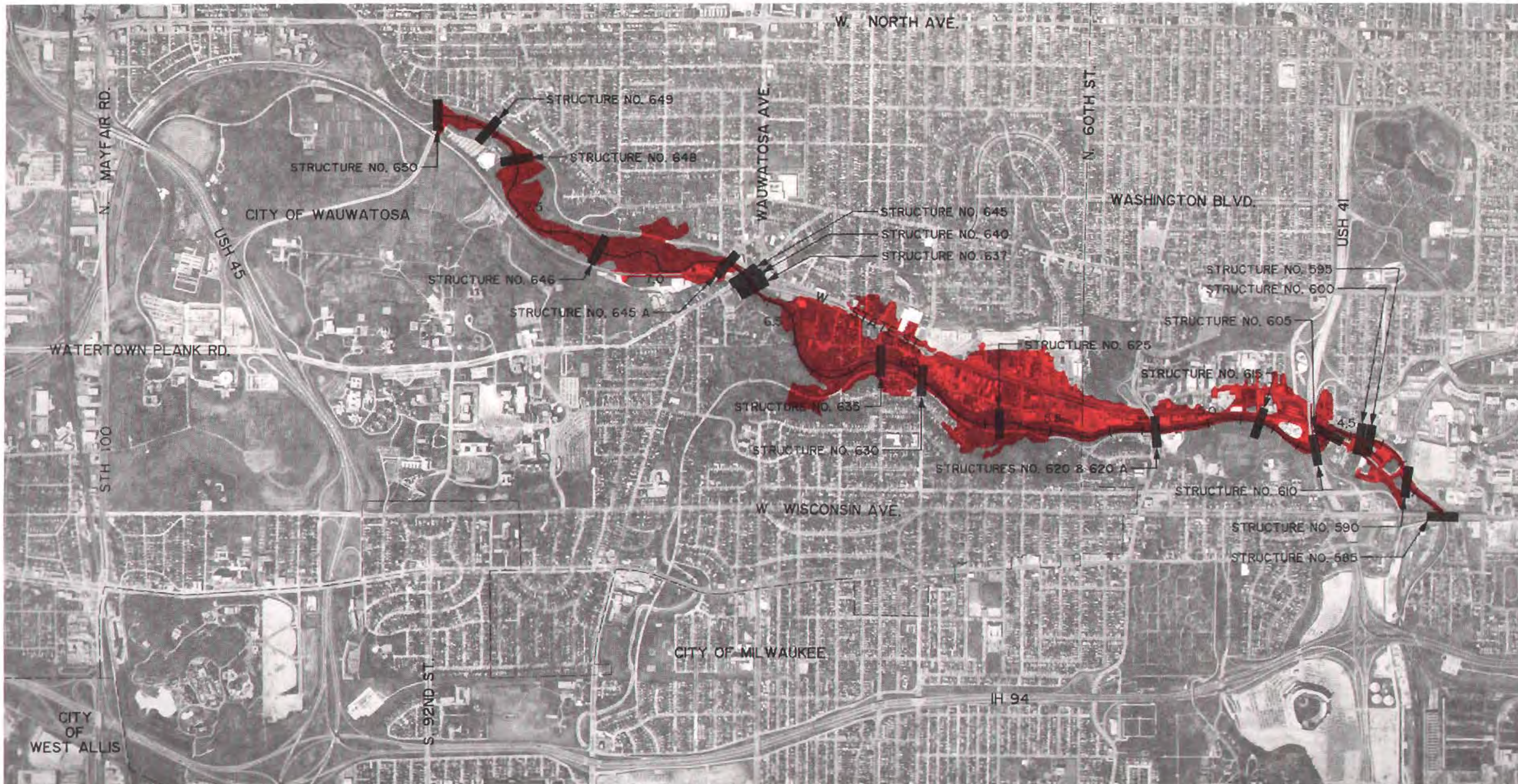
NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR MENOMONEE RIVER IS SHOWN IN APPENDIX H

Figure 69

FLOOD STAGE AND STREAMBED PROFILE FOR THE MENOMONEE RIVER



Map 152 (continued)



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

6.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

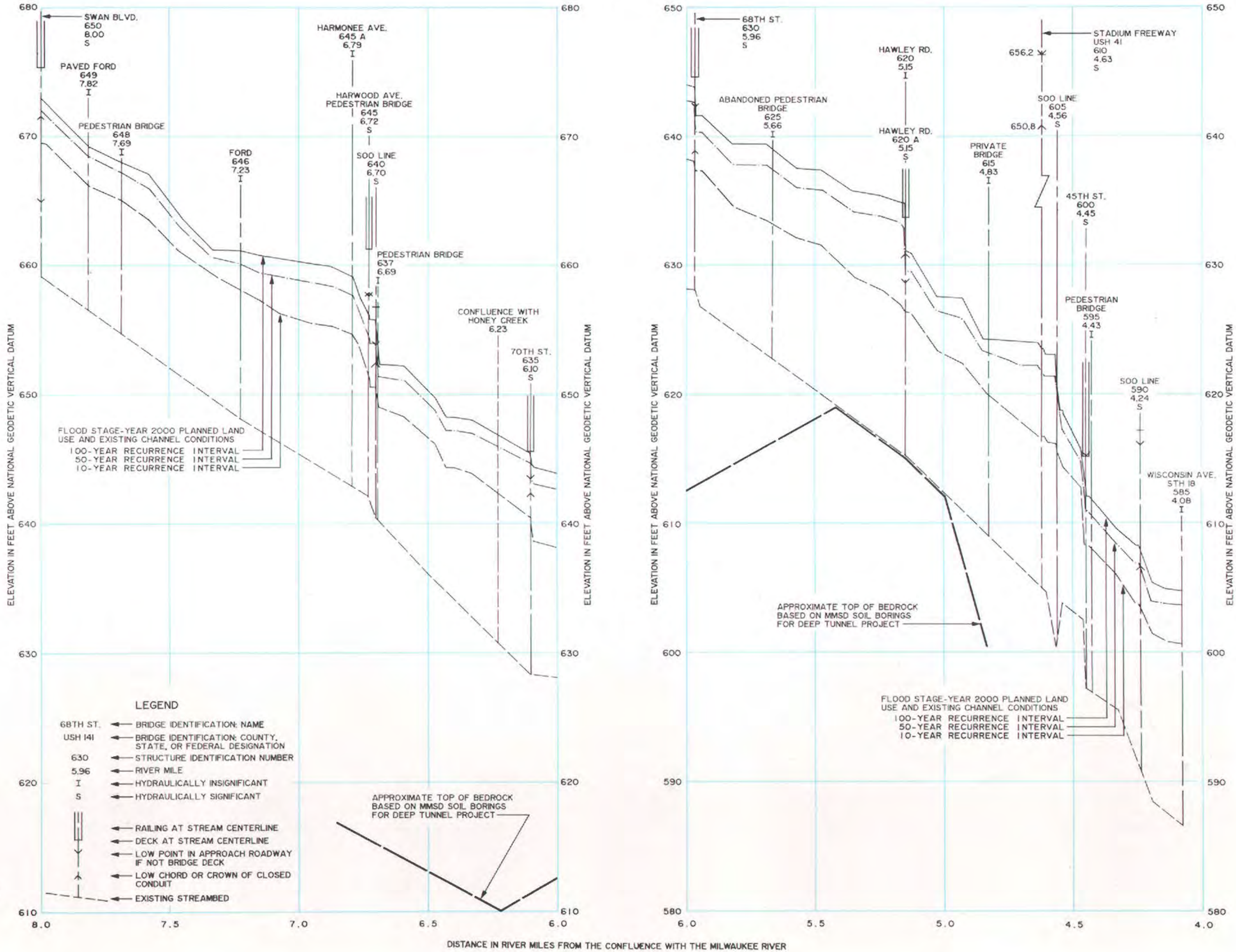
NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR MENOMONEE RIVER IS SHOWN IN APPENDIX H

GRAPHIC SCALE

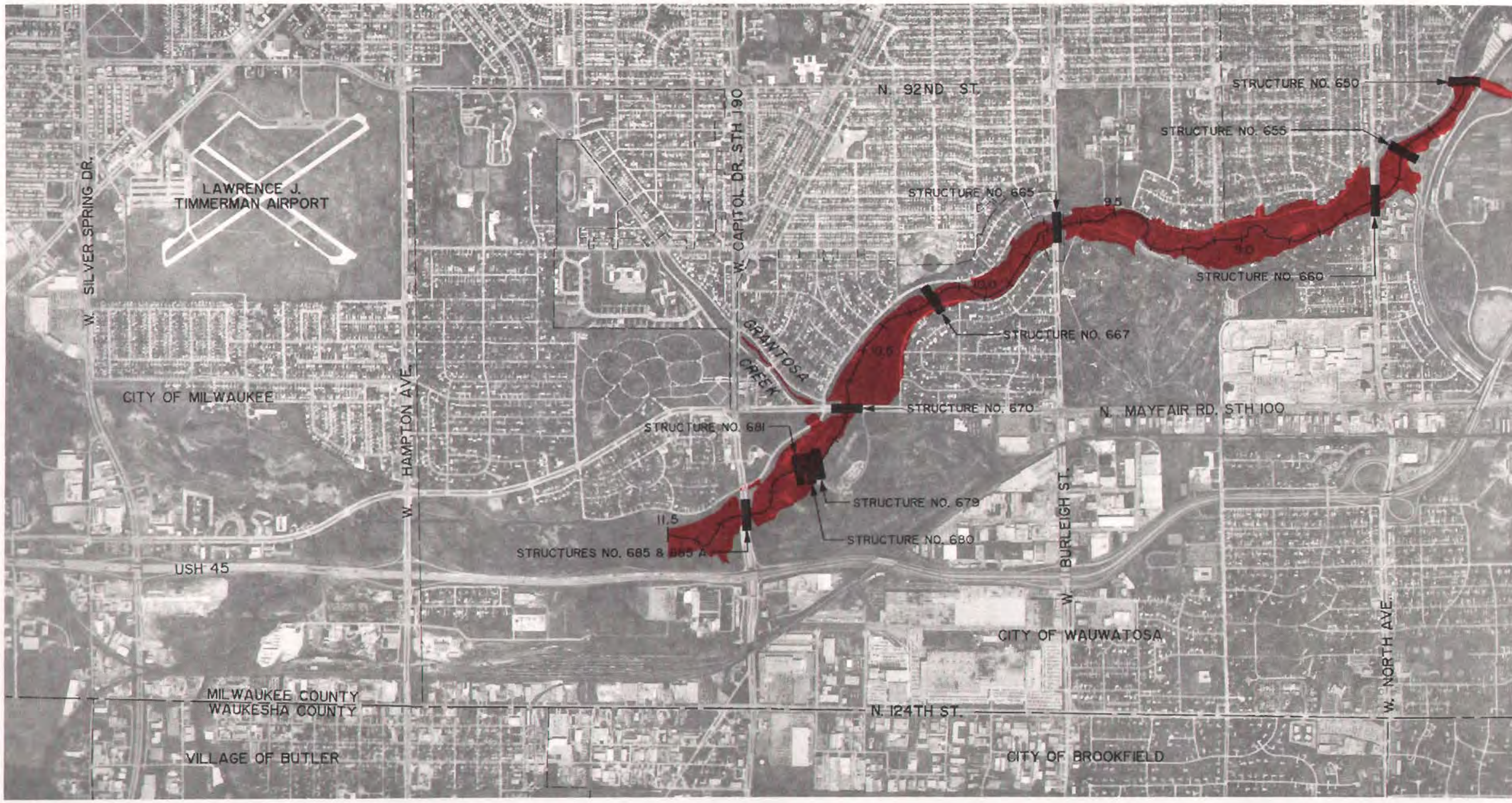
0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Figure 69 (continued)



Map 152 (continued)



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

9.5 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR MENOMONEE RIVER IS SHOWN IN APPENDIX H

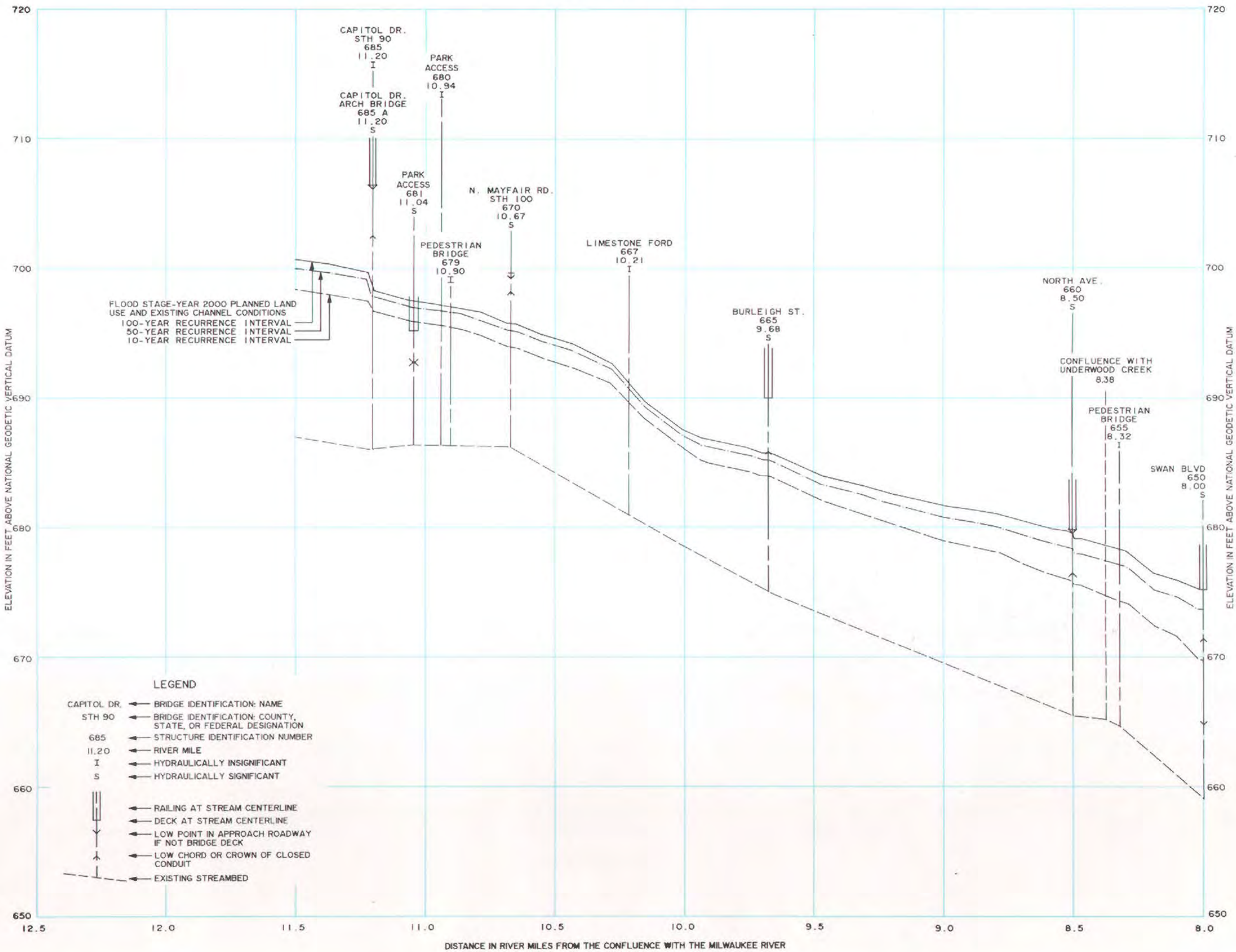
NOTE: THE FLOODLAND LIMITS SHOWN ALONG GRANTOSA CREEK ARE BASED ONLY ON BACKWATER FROM THE MENOMONEE RIVER DURING A 100-YEAR RECURRENCE INTERVAL FLOOD. AS SUCH, THEY DO NOT REPRESENT THE FLOODLAND LIMITS RESULTING FROM A 100-YEAR FLOOD OCCURRING ALONG GRANTOSA CREEK.

GRAPHIC SCALE

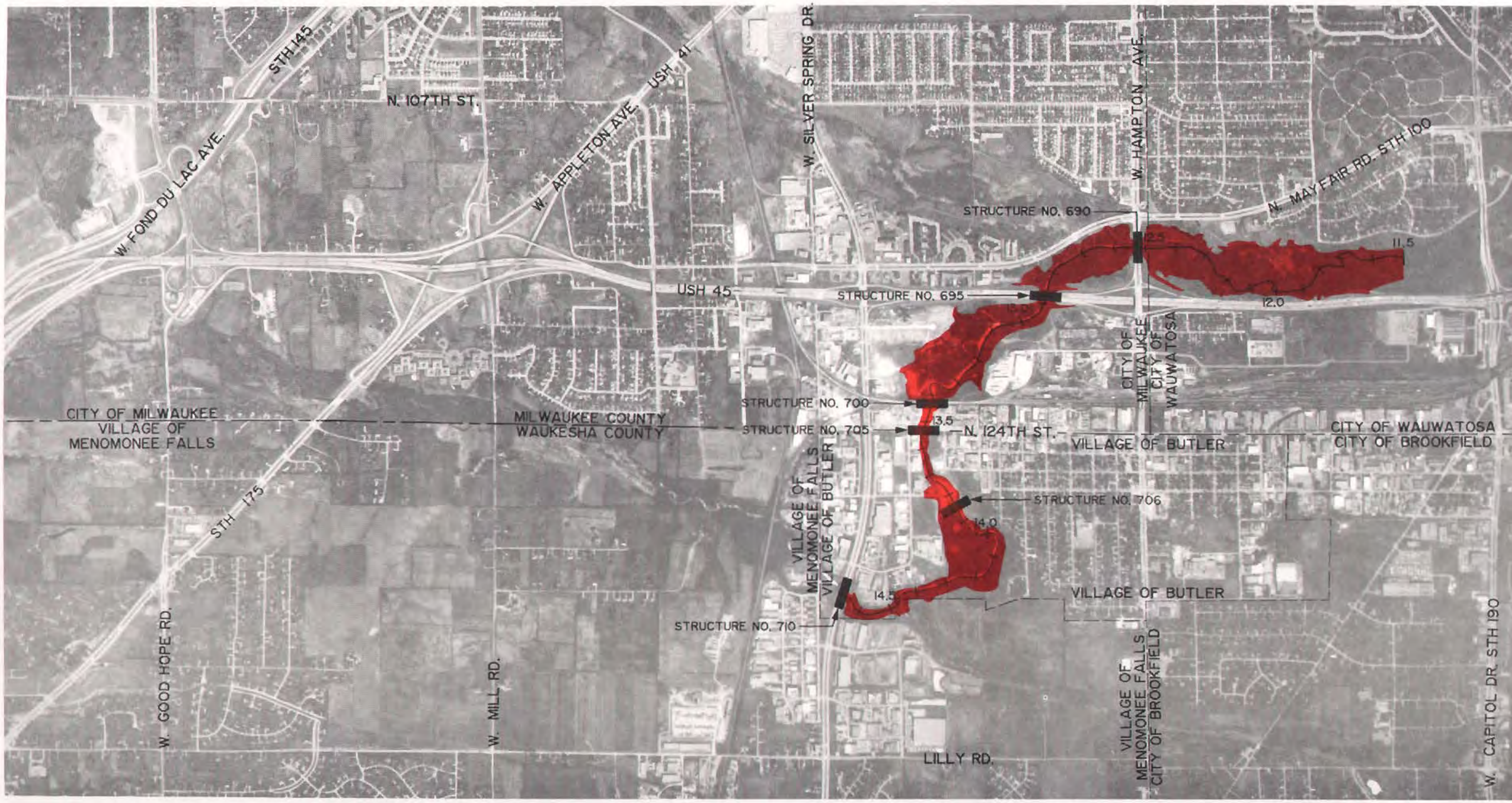
0 1/2 1 MILE

DATE OF PHOTOGRAPHY APRIL 1986

Figure 69 (continued)



Map 152 (continued)



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

13.5 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

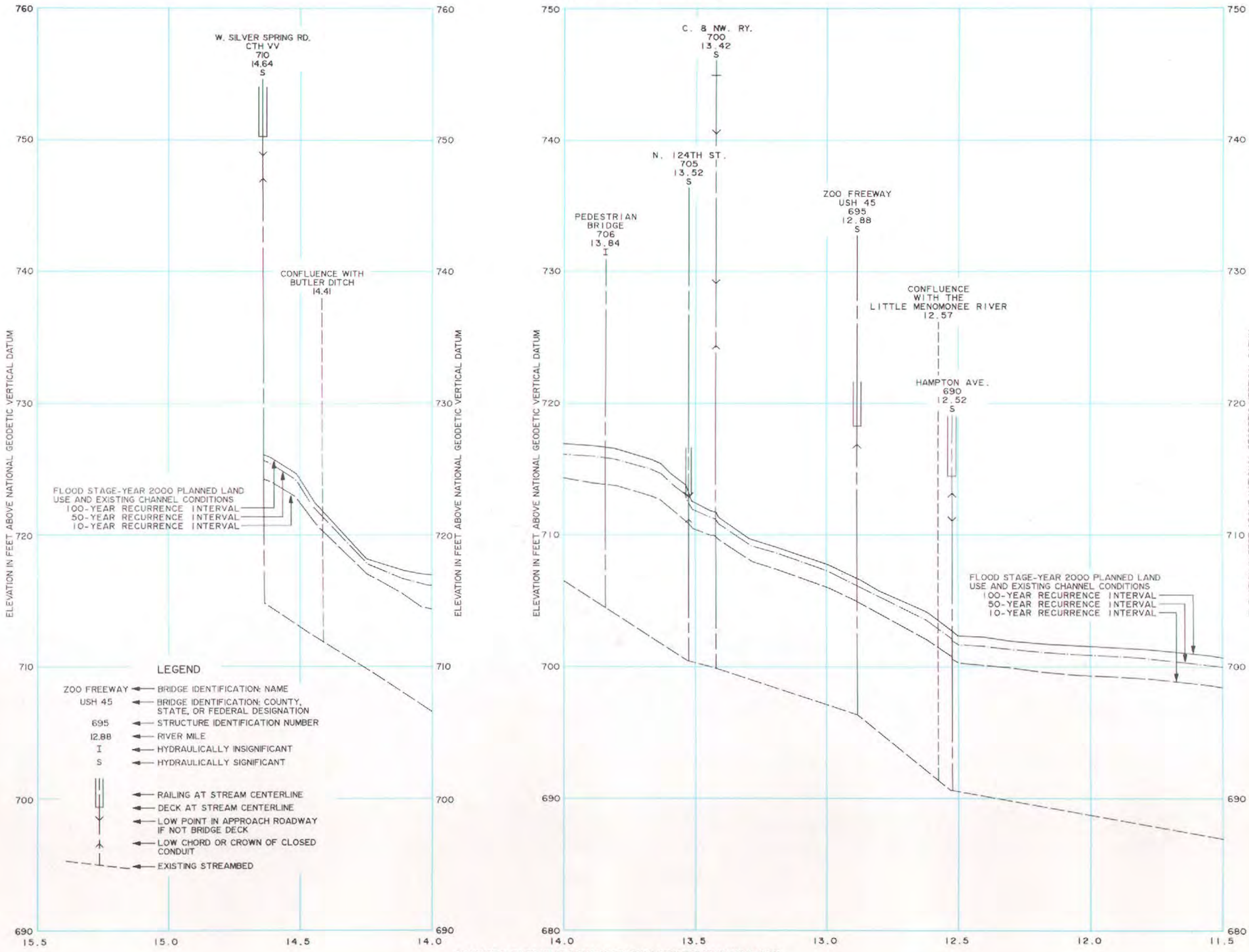
NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR MENOMONEE RIVER IS SHOWN IN APPENDIX H

GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Figure 69 (continued)



Map 152 (continued)

LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

17.5 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

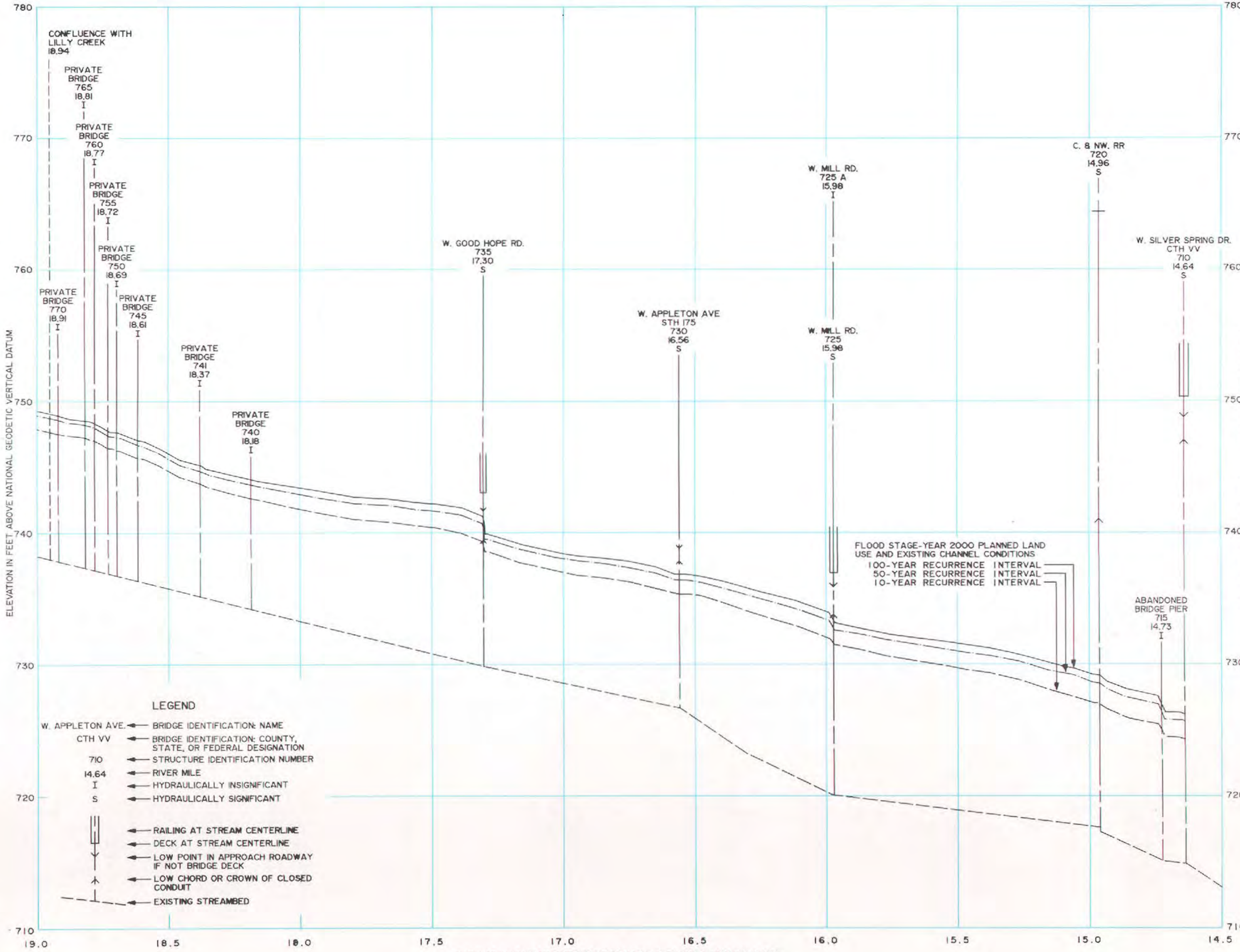
NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR MENOMONEE RIVER IS SHOWN IN APPENDIX H

GRAPHIC SCALE

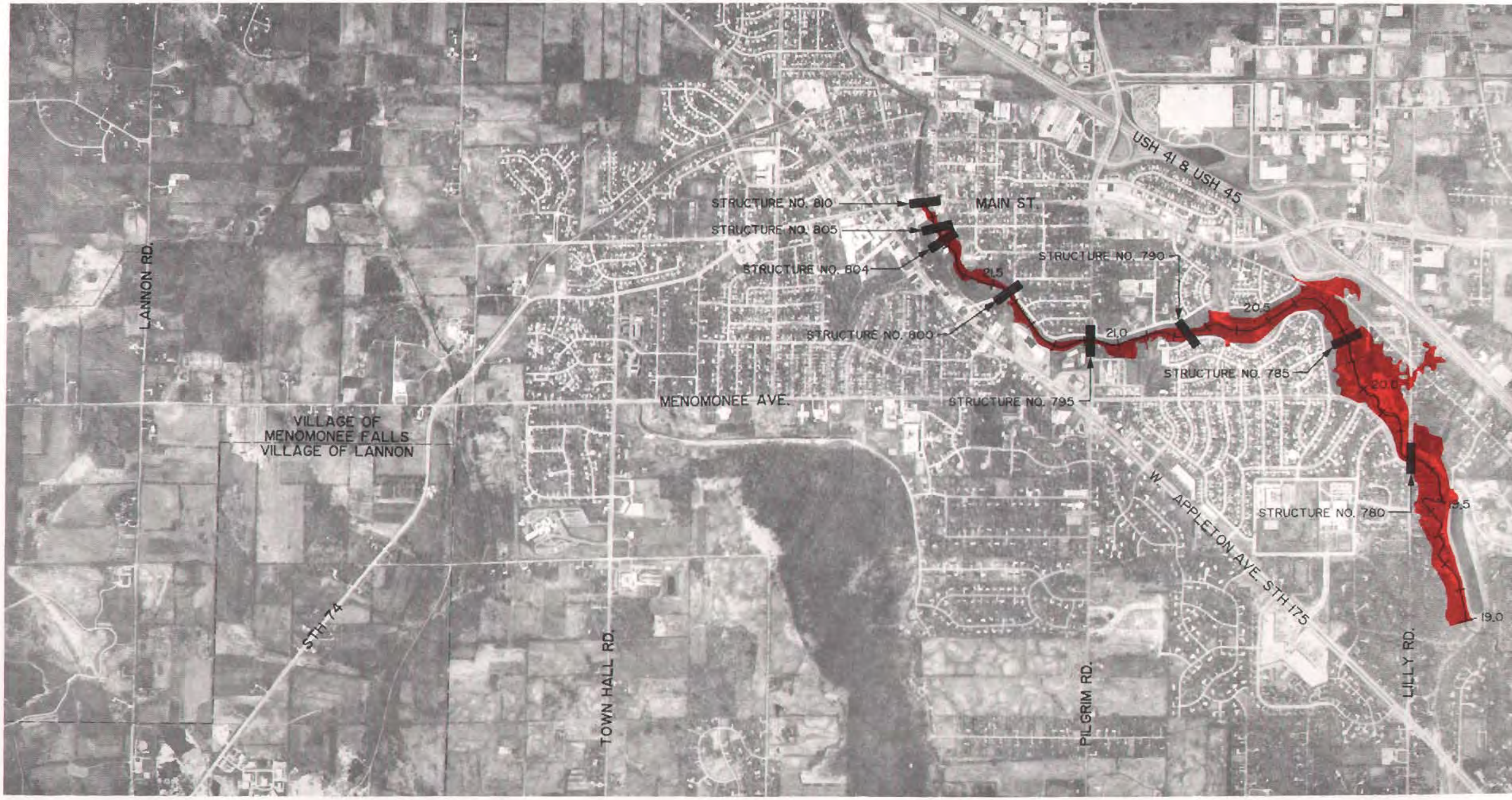
0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Figure 69 (continued)



Map 152 (continued)



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

21.5 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

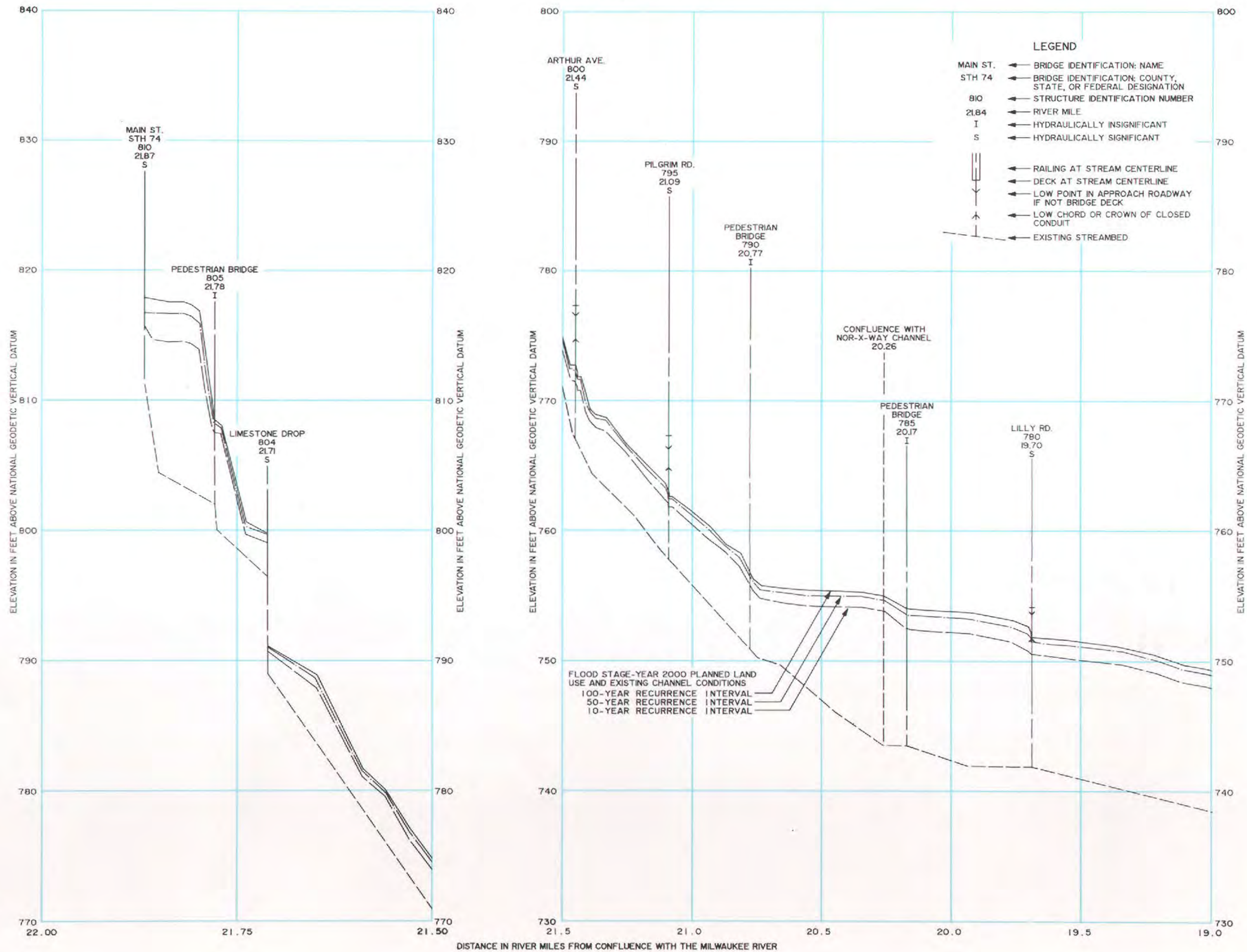
NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR MENOMONEE RIVER IS SHOWN IN APPENDIX H

GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Figure 69 (continued)



Map 152 (continued)

LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

22.5 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

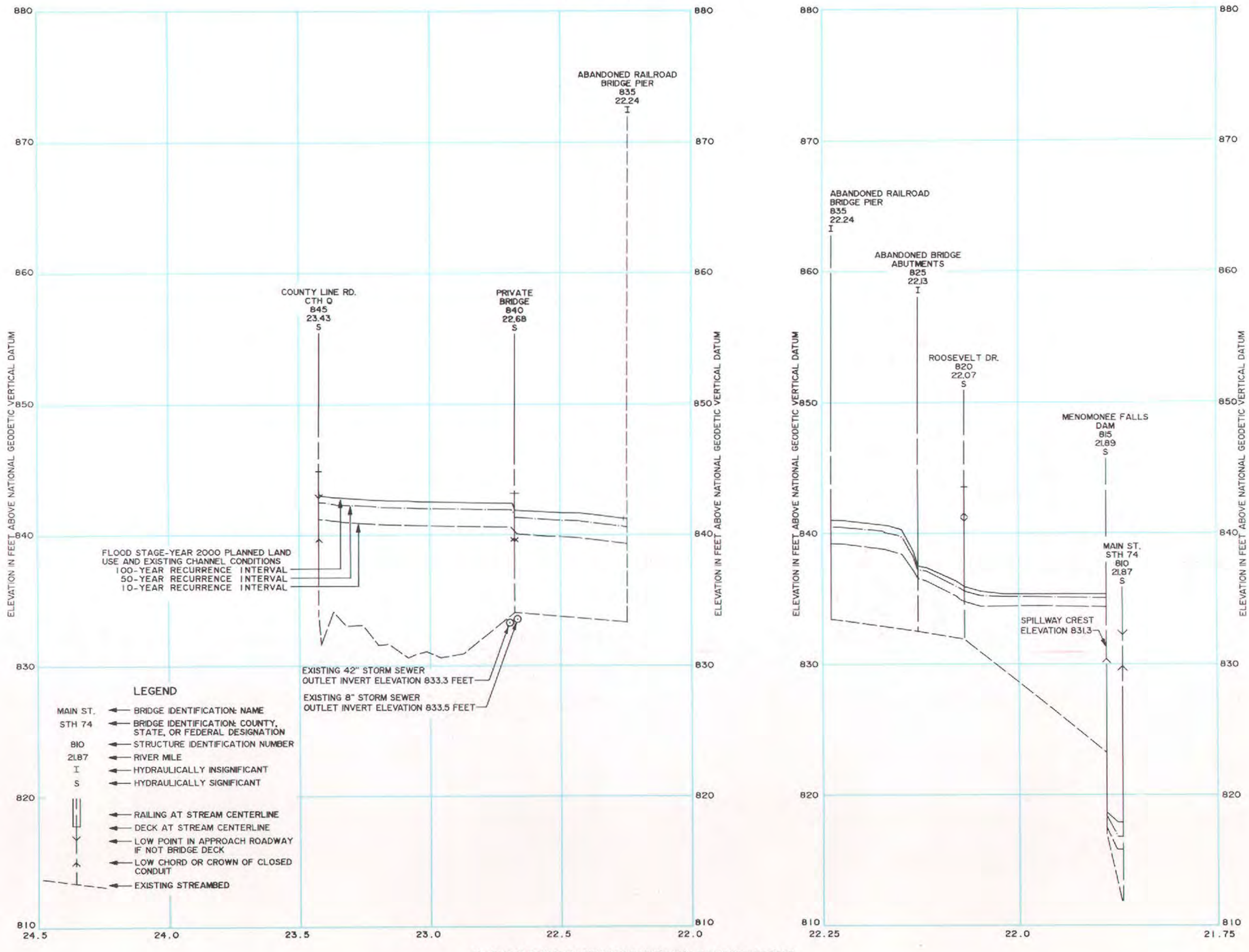
NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR MENOMONEE RIVER IS SHOWN IN APPENDIX H

GRAPHIC SCALE

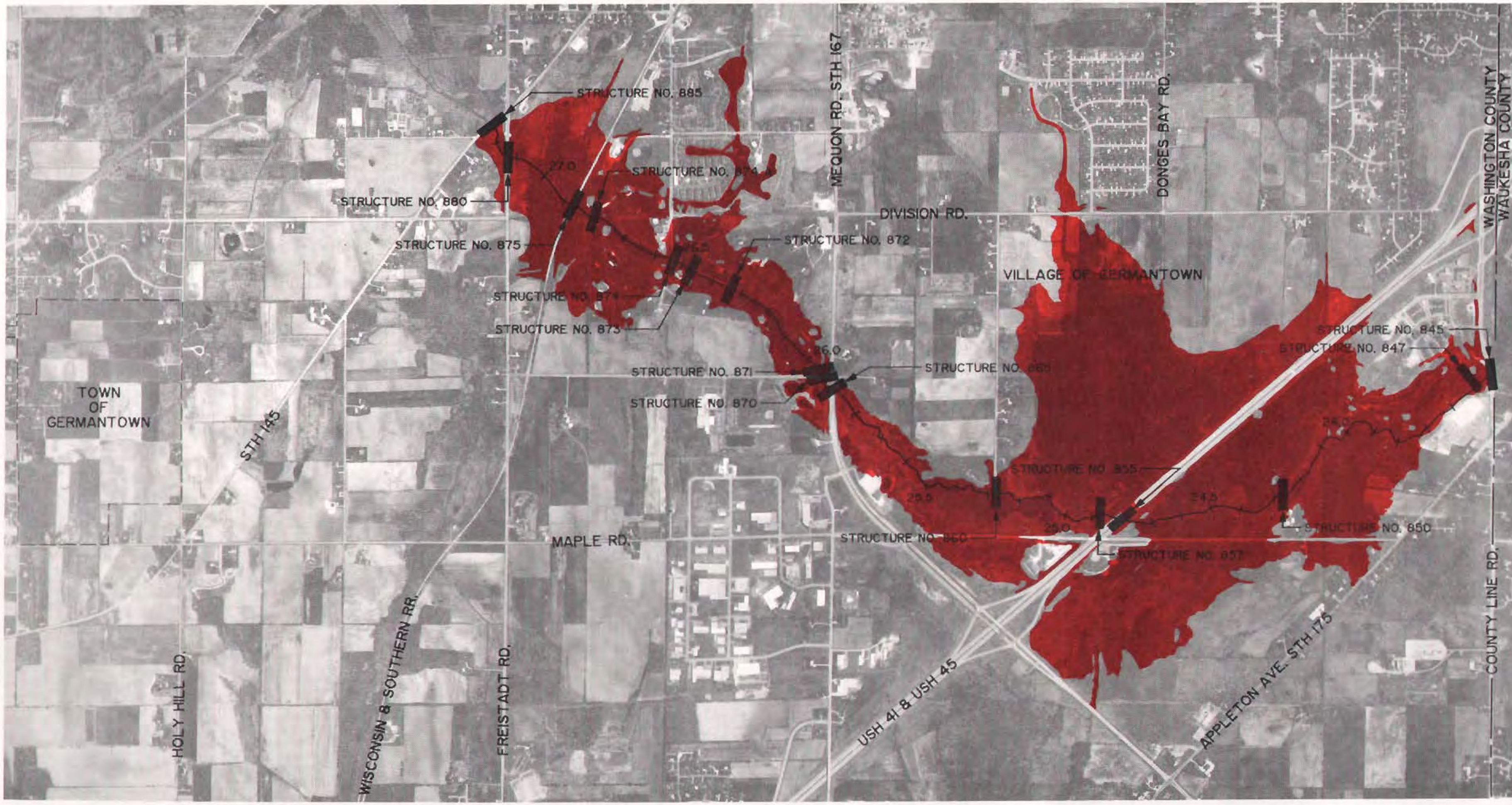
0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Figure 69 (continued)



Map 152 (continued)



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

25.5 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR MENOMONEE RIVER IS SHOWN IN APPENDIX H

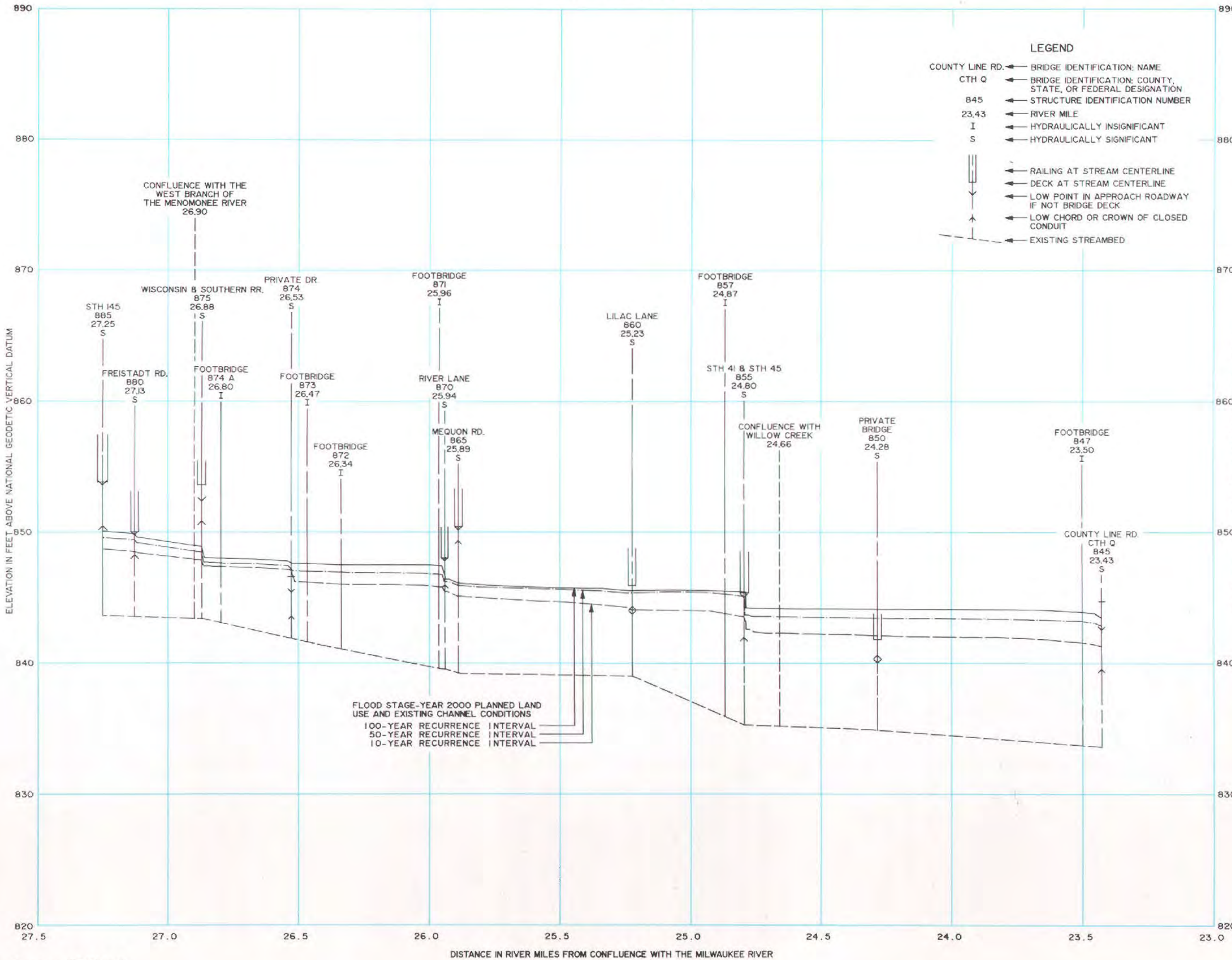
Source: SEWRPC.

GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Figure 69 (continued)



Source: SEWRPC.

flood control purposes, as shown of Figure 69. The pool above the dam is used by the Falk Corporation to provide cooling water for its operations.

A recent local development plan for the Menomonee River Valley prepared by the City of Milwaukee Department of City Development sets forth a goal of removal of the Falk dam to permit fish migration and small boat access to upstream reaches of the River.[4] While removal of the dam is not required for flood control purposes, such removal would not be inconsistent with this flood control system plan in that removal would create a relatively minor localized decrease in upstream flood stages and would have no significant effect on downstream flood stages.

The new 27th Street Viaduct configuration was included in the refined hydraulic simulation model used for this system planning effort. Based on the revised 100-year recurrence interval flood profile under planned land use and channel conditions, it was found that the floodwall at the Falk Corporation was adequate to contain the 100-year recurrence interval flood with from 0.0 to 2.8 feet of freeboard; the earthen dike along the former railway yards was adequate to contain the 100-year recurrence interval flood with from 0.0 to 3.0 feet of freeboard; and the floodwall along the former railway yards would contain the 100-year flood with 3.0 feet of freeboard. The watershed study recommendations were refined to call for the provision of 3.0 feet of freeboard above the 100-year stage at the Falk Corporation floodwall and at the dike along the south side of the former railway yard. The crest of the Falk floodwall would be raised to elevation 595.5 feet National Geodetic Vertical Datum (NGVD) at its downstream end and to elevation 599.1 feet NGVD at its upstream end. Throughout the length of the floodwall, the crest would be raised from 0.2 to 3.0 feet. The crest of the dike would be raised from 0.0 to 3.0 feet to establish a top elevation of about 600 feet NGVD throughout its length. No increase in height is required in the floodwall along the former railway yards. The cost of providing freeboard for the approximately 2,500-foot-long Falk Corporation floodwall is estimated at $150,000. The cost of raising the approximately 2,500-foot-long earthen dike along the former railway yard is estimated at $90,000. The sections of dike and floodwall to be raised are shown on Map 153. The peak flood profile attendant to planned land use and channel conditions is shown on Figure 70.

The City of Milwaukee is considering the re-creation of an approximately 30-acre wetland area along the south bank of the river between the 27th Street and the 35th Street Viaducts. It is not anticipated that the proposed wetland would have a significant impact on flood flows and stages along the river.

A part of the former railway yard property is now owned by the State of Wisconsin and a part is owned by the CMC Corporation. Those combined properties are one of several sites under consideration as a potential location for a new baseball stadium to be constructed by the Milwaukee Brewers Baseball Club or as a potential site for new commercial development. In the event of the construction of a baseball stadium or a commercial development, the flood control objectives of this plan could be met by raising the height of the existing earthen dikes to provide three feet of freeboard during a 100-year flood or by filling the property to an approximate elevation of 599 feet NGVD, which is two feet above the 100-year flood level. For cost-estimating purposes, it was assumed that the existing steel sheet floodwall would be retained and that the existing dikes would be raised.

Flood Control and Related Drainage System Plan Implementation: It is recommended that the District design, construct, and maintain the proposed dike and floodwall raises.

City of Milwaukee from IH 94 to W. Michigan Street Extended: In the 1960's, the channel in the 0.80-mile-long reach from IH 94 through N. 45th Street was deepened and paved with concrete as part of the District flood control program. A primarily residential area located along the east bank of this reach, extending from the Soo Line (former Chicago, Milwaukee, St. Paul & Pacific Railroad) railway bridge just upstream of IH 94 to W. Michigan Street extended, was identified in the watershed study as a 100-year flood hazard area under planned

[4]*City of Milwaukee Department of City Development, A Plan for the Menomonee Valley, prepared in cooperation with the City Department of Public Works, March 5, 1990.*

Table 96

COST ESTIMATES FOR FLOOD CONTROL ALTERNATIVES FOR THE MENOMONEE RIVER IN THE CITY OF MILWAUKEE FROM IH 94 TO W. MICHIGAN AVENUE EXTENDED

Alternative	Description	Costs: Capital	Costs: Annual: Amortized Capital[a]	Costs: Annual: Operation and Maintenance	Costs: Annual: Other	Costs: Annual: Total	Benefit-Cost Analysis: Annual Benefits	Benefit-Cost Analysis: Annual Benefits Minus Annual Costs	Benefit-Cost Analysis: Benefit-Cost Ratio	Benefit-Cost Analysis: Economic Ratio Greater than One
1. No Action	- -	$ 0	$ 0	$ 0	$3,100	$ 3,100	$ 0	$ -3,100	- -	No
2. Structure Floodproofing, Elevation, and Removal	Floodproof 74 structures	340,000	23,000	0	0	23,000	3,100	-19,900	0.13	No
	Elevate one structure	30,000								
	Subtotal	$ 370,000								
3. Dikes, Floodwalls, and Stormwater Pumping	Construct 200 feet of earthen dike	$ 28,000	$142,300	$22,700	$ - -	$165,000	$3,100	$-161,900	0.02	No
	Construct 1,200 feet of concrete floodwall	387,000								
	Construct two stormwater pumping stations	1,820,000								
	Subtotal	$2,235,000								

[a]*Amortized capital cost is based on an interest rate of 6 percent and a project life of 50 years.*

Source: SEWRPC.

land use and channel conditions. The hydrologic and hydraulic analyses conducted under this system planning effort reaffirmed this finding of the watershed study.

The watershed study recommended that the existing channel be supplemented with construction of an approximately 1,400-foot-long floodwall along the east bank of the reach and that three backwater gates and two stormwater pumping stations be provided. Under this system planning effort, it was determined to re-analyze three alternative plans for alleviating the flood damage problem in this reach: 1) Alternative Plan 1—No Action; 2) Alternative Plan 2—Structure Floodproofing and Removal; 3) Alternative Plan 3—Dikes, Floodwalls, and Stormwater Pumping Stations.

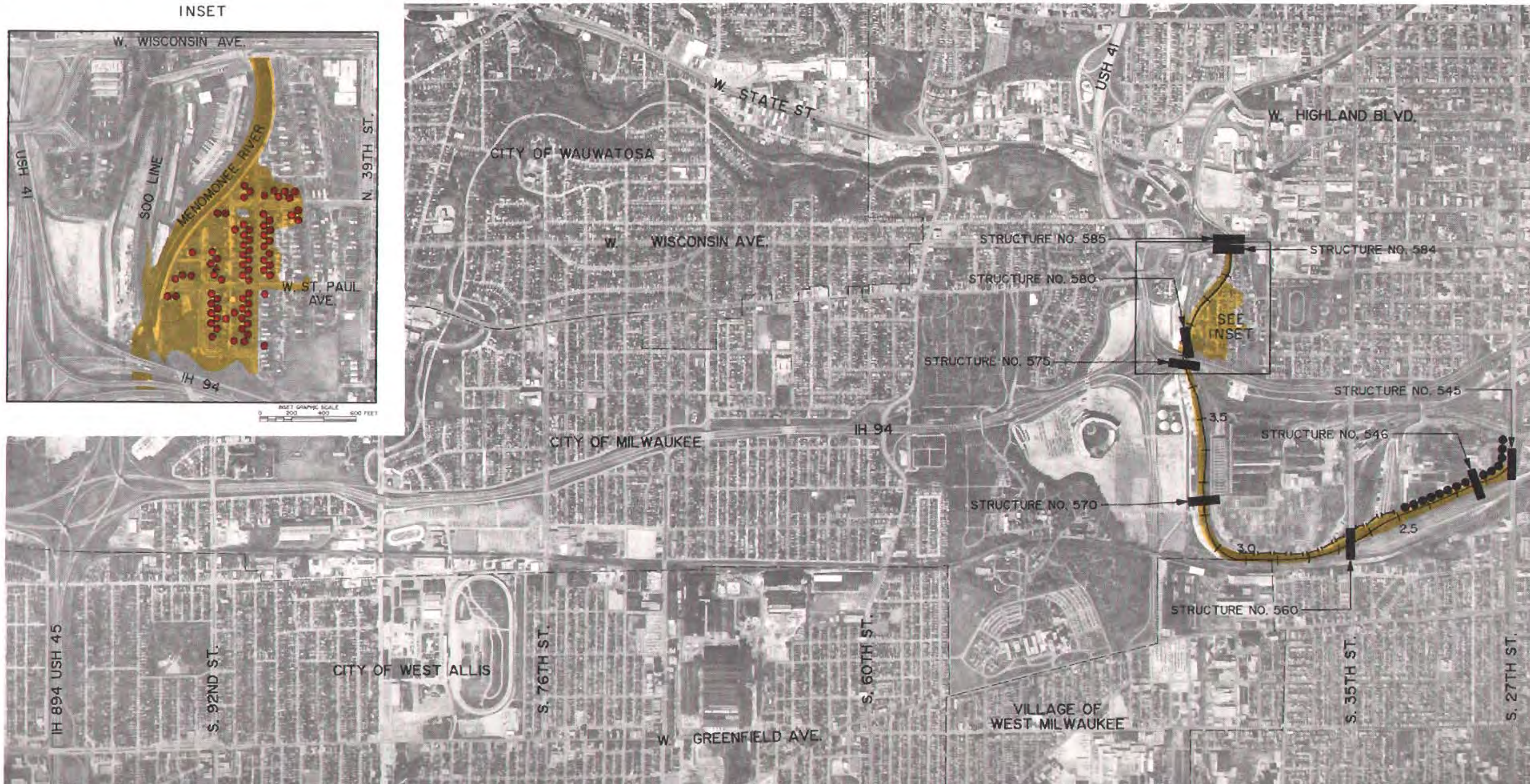
Each alternative system is described briefly below. The economic benefits and costs attendant to each alternative are provided in Table 96.

Alternative Plan 1—No Action: One alternative course of action with respect to the flood problem is to do nothing, that is, to recognize the inevitability of flooding but to deliberately decide not to mount a collective, coordinated program to abate the flood damages. Under planned year 2000 land use and existing channel conditions, the average annual flood damages would approximate $3,100. The damages from a 100-year recurrence interval flood may be expected to approximate $365,000. There are no monetary benefits associated with this alternative. The average annual cost would be equivalent to the annual flood damage costs under planned land use conditions, or about $3,100.

Alternative Plan 2—Structure Floodproofing, Elevation, and Removal: A structure floodproofing and elevation flood control system was analyzed to determine if such a structure-by-structure approach would be a technically feasible and economically viable solution to the flood problem. The 100-year recurrence interval

Map 153

RECOMMENDED FLOOD CONTROL SYSTEM PLAN FOR THE MENOMONEE RIVER FROM THE 27TH STREET VIADUCT TO W. WISCONSIN AVENUE



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS

3.5 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

INCREASE HEIGHT OF EARTHEN DIKE TO PROVIDE 3 FEET OF FREEBOARD UNDER 100-YEAR CONDITIONS

INCREASE HEIGHT OF STEEL SHEET PILE FLOODWALL TO PROVIDE 3 FEET OF FREEBOARD UNDER 100-YEAR CONDITIONS

STRUCTURE TO BE FLOODPROOFED

STRUCTURE TO BE ELEVATED

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR MENOMONEE RIVER IS SHOWN IN APPENDIX H

GRAPHIC SCALE

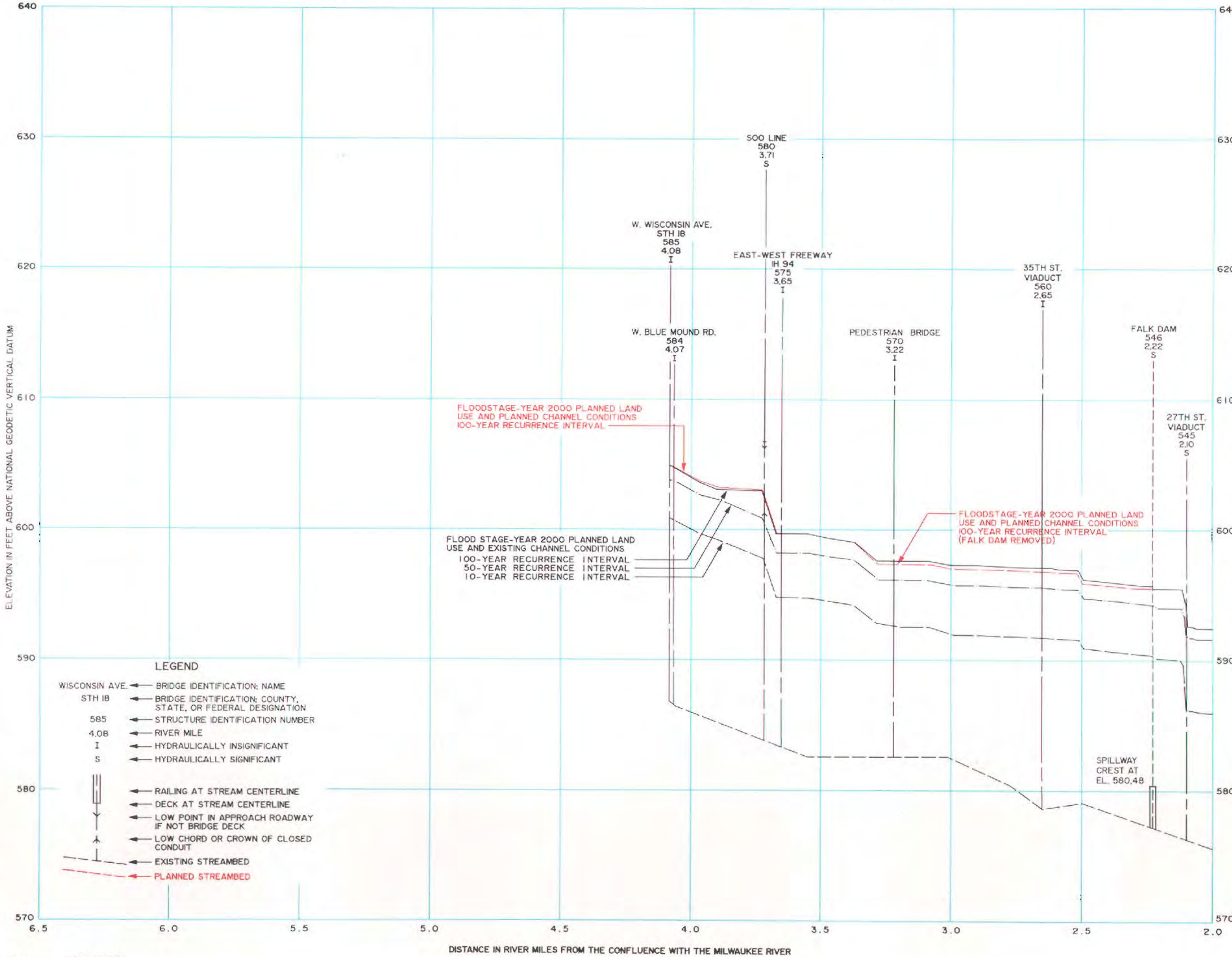
0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Source: SEWRPC.

Figure 70

RECOMMENDED PLAN FLOOD STAGE PROFILE FOR THE MENOMONEE RIVER FROM THE 27TH STREET VIADUCT TO W. WISCONSIN AVENUE



Source: SEWRPC.

flood stage under planned year 2000 land use and planned channel conditions was used to estimate the number of existing flood-prone structures to be floodproofed and the approximate costs involved.

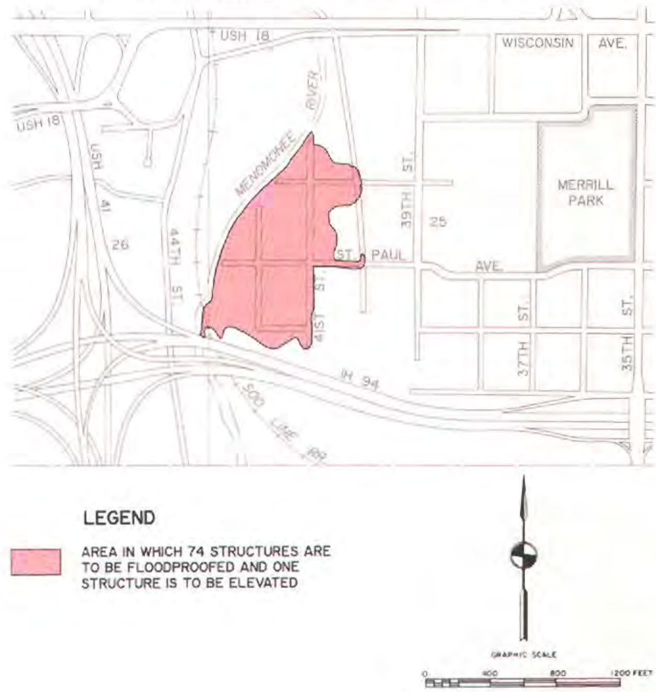
In the case of residential structures, floodproofing was assumed to be feasible if the design flood stage was below the first floor elevation. Structure elevation was considered feasible for residential structures with basements if the estimated cost of elevating the structure and floodproofing the basement was less than the estimated removal cost. Structures to be elevated were assumed to have the first floor raised to an elevation of at least two feet above the 100-year recurrence interval flood stage to provide adequate freeboard. For aesthetic reasons, structure elevation was limited to a maximum of four feet. Structures which would have to be elevated more than four feet were considered for removal.

The area for which structure floodproofing and elevation was considered is shown on Map 154. Of the 75 structures which may be expected to incur flood damage from a 100-year flood, 74 would have to be floodproofed and one would have to be elevated. Future damage from floods up to and including the 100-year flood would be virtually eliminated if this alternative were fully implemented.

Assuming that these structure floodproofing measures would be fully implemented, and utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of this alternative is estimated at $23,000. This cost consists of the amortization of the $370,000 capital cost, $340,000 for floodproofing and $30,000 for structure elevation. The average annual flood damage abatement is estimated at $3,100, yielding a benefit-cost ratio of 0.13.

Map 154

ALTERNATIVE PLAN 2: STRUCTURE FLOODPROOFING ALONG THE MENOMONEE RIVER IN THE CITY OF MILWAUKEE FROM IH 94 TO W. MICHIGAN STREET EXTENDED



Source: SEWRPC.

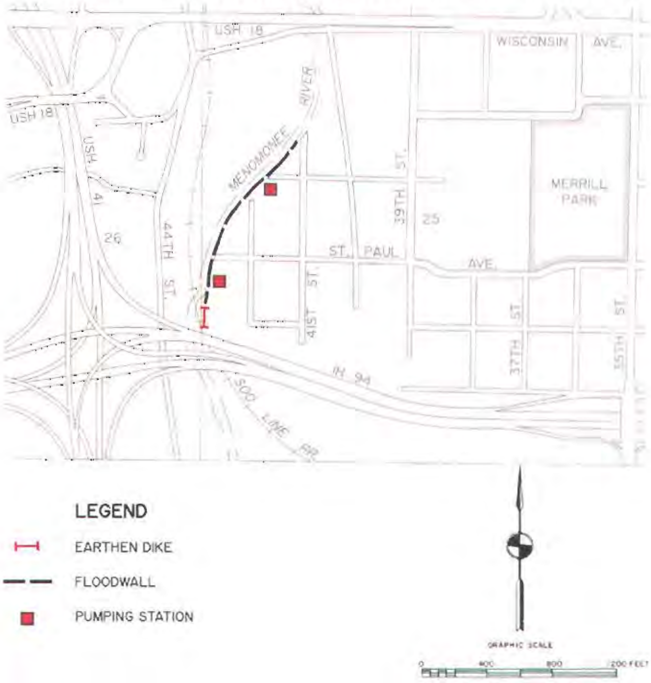
<u>*Alternative Plan 3—Dikes, Floodwalls, and Stormwater Pumping Stations*</u>: This alternative system plan consists of the construction of 200 feet of earthen dike, 1,200 feet of concrete floodwall, and two stormwater pumping stations, as shown on Map 155. The dikes and floodwalls would be designed to pass the 100-year recurrence interval flood with three feet of freeboard. The maximum height of the dike would be eight feet and the maximum height of the floodwall would be six feet. Implementation of this alternative would essentially eliminate all damages attendant to floods up to and including the 100-year recurrence interval flood.

Utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of this alternative is estimated at $165,000. This cost consists of the amortization of the $2,235,000 capital cost, including $28,000 for dikes, $387,000 for floodwalls, and $1,820,000 for pumping stations, and $22,700 in annual operation and maintenance costs. The average annual flood damage abatement is estimated at $3,100, yielding a benefit-cost ratio of 0.02.

<u>*Evaluation of Alternatives*</u>: All of the alternatives described above were found to be technically feasible. Although it offers the lowest cost, Alternative Plan 1, the "no action" alternative, does nothing to alleviate the existing flood problem and does not represent a sound

Map 155

ALTERNATIVE PLAN 3: DIKES, FLOODWALLS, AND STORMWATER PUMPING STATIONS ALONG THE MENOMONEE RIVER IN THE CITY OF MILWAUKEE FROM IH 94 TO W. MICHIGAN STREET EXTENDED



Source: SEWRPC.

approach to flood control. None of the alternatives was found to have a benefit-cost ratio greater than one.

Alternative Plan 2—Structure Floodproofing, Elevation, and Removal—has the lower cost of the two alternatives besides the "no action" alternative. Complete implementation of a voluntary structure floodproofing and elevation program is unlikely; however, because only basement floodproofing is required at 74 of the 75 affected structures, this alternative might be implemented with some success. Partial implementation would leave the City of Milwaukee with a residual problem whenever a major flood occurs. Also, yard damages and cleanup costs would remain under this alternative.

Alternative Plan 3—Dikes, Floodwalls, and Stormwater Pumping Stations—would abate structural flood damages, but would be extremely costly and would rely on the proper operation of stormwater pumps which may be rendered inoperable if their power source is cut during a major storm.

It is recommended that Alternative Plan 2—Structure Floodproofing, Elevation, and Removal—be implemented for this reach of the river. The recommended plan is shown graphically on Map 153. The peak flood profile attendant to planned land use and channel conditions is shown on Figure 70. Full implementation of this plan would serve to eliminate structural flood damages in this reach of the Menomonee River for floods up to and including the 100-year recurrence interval flood under planned land use and channel conditions.

Flood Control and Related Drainage System Plan Implementation: The recommended flood control plan would be implemented by the individual property owners within the 100-year floodplain. It is recommended that these private owners bear the cost of structure floodproofing or removal. It is further recommended that the professional services required to prepare plans for the floodproofing and elevation of individual buildings be made available to property owners, at no cost, by the City of Milwaukee engineering department. Also, it is recommended that the City of Milwaukee review its building ordinance to ensure that appropriate floodproofing regulations are included. It is recommended that the City explore, on behalf of the property owners involved, any available state and/or federal aids for such floodproofing measures.

City of Milwaukee from W. Michigan Street Extended at River Mile 3.97 to N. 43rd Street at River Mile 4.33: The watershed study identified potential flooding problems along the west bank of this reach during a 100-year recurrence interval flood under planned land use and channel conditions. The study recommended that the existing concrete-lined channel be supplemented with a floodwall and stormwater pumping stations. The refined analysis of this reach conducted under this study indicated that the 100-year recurrence interval flood flows may be expected to be lower than those developed under the watershed study. The reasons for this are discussed in the Flood Discharges and Stages section of this report. Accordingly, this system planning effort did not identify a flood-

Table 97

MENOMONEE RIVER WATERSHED STUDY FLOOD CONTROL ALTERNATIVES FOR THE MENOMONEE RIVER IN THE CITY OF MILWAUKEE BETWEEN N. 45TH STREET AND N. 60TH STREET EXTENDED

Alternative	Cost[a]					Benefit-Cost Ratio
	Capital	Amortized Capital[b]	Operation and Maintenance	Other	Total	
1. No Action	$ 0	$ 0	$ 0	$97,200	$ 97,200	0
2. Structure Floodproofing	640,000	40,600	0	0	40,600	2.39
3. Major Channel Modifications	5,519,600	350,200	800	0	351,000	0.28
4. Dikes and Floodwalls	4,984,400	316,400	9,600	0	326,800	0.30
5. Bridge Alteration or Replacement Floodproofing and Removal	- -[c]	- -[c]	- -[c]	- -[c]	- -[c]	- -

[a]*Costs are expressed in 1986 dollars.*

[b]*Amortized capital cost is based on an interest rate of 6 percent and a project life of 50 years.*

[c]*No costs were computed as this alternative was found to be technically infeasible.*

Source: SEWRPC.

ing problem under planned land use and channel conditions. Therefore, no recommended improvements are recommended for this reach.

N. 43rd Street Extended at River Mile 4.33 in the City of Milwaukee through Glenview Avenue Extended at River Mile 6.88 in the City of Wauwatosa: The watershed study identified potential flooding problems along both the north and south banks of the reach of the Menomonee River from River Mile 4.24 through 4.38 during a 100-year recurrence interval flood under planned land use and channel conditions. The watershed study recommended that the existing concrete-lined channel be supplemented with a floodwall and stormwater pumping stations. The refined analysis of the reach from River Mile 4.24 through 4.38 which was conducted under this study indicated that the 100-year recurrence interval flood flows may be expected to be lower than those developed under the watershed study. The reasons for this are discussed in the Flood Discharges and Stages section of this report. The refined analysis did identify the potential for damages along the south bank of that reach. That flooding would be primarily attributable to overflow of the bank upstream of River Mile 4.38; therefore, the recommended flood control measures between River Mile 4.24 and 4.38 were formulated in conjunction with the measures for the reach from River Mile 4.38 through River Mile 6.88.

The watershed study considered a total of five flood control alternatives for the reach from N. 45th Street through N. 60th Street in the City of Milwaukee. These alternatives included the following: 1) No Action; 2) Floodproofing of Structures; 3) Channel Modifications; 4) Dikes and Floodwalls; and 5) Bridge Alteration or Replacement. The estimated cost of each of these alternatives, as well as the attendant benefit-cost ratio, is presented in Table 97.

A total of five flood control alternatives were initially considered from N. 60th Street through Glenview Avenue extended in the City of Wauwatosa. These alternatives include the following:

Table 98

MENOMONEE RIVER WATERSHED STUDY FLOOD CONTROL ALTERNATIVES FOR THE MENOMONEE RIVER IN THE CITY OF WAUWATOSA DOWNSTREAM OF HARWOOD AVENUE

Alternative	Cost[a]					Benefit-Cost Ratio
	Capital	Amortized Capital[b]	Operation and Maintenance	Other	Total	
1. No Action	$ 0	$ 0	$ 0	$661,800	$661,800	0
2. Structure Floodproofing and Removal	6,800,000	433,600	0	0	433,600	1.53
3. Major Channel Modification	9,263,200	587,600	1,800	0	589,400	1.12
4. Dikes and Floodwalls	5,902,000	374,400	15,400	0	389,800	1.70
5. Combination of Channelization and Structure Floodproofing and Removal	11,998,800	761,200	1,200	0	762,400	0.87

[a]*Costs are expressed in 1986 dollars.*

[b]*Amortized capital cost is based on an interest rate of 6 percent and a project life of 50 years.*

Source: SEWRPC.

1) No Action; 2) Floodproofing and Removal of Structures; 3) Channel Modifications; and 4) Dikes and Floodwalls. The estimated cost of each of these alternatives, as well as the attendant benefit-cost ratio, is presented in Table 98.

The initial recommendation of commission staff called for channelization within the City of Wauwatosa and structure floodproofing in the City of Milwaukee. The channelization was recommended to extend 0.5 mile into the City of Milwaukee in order to substantially reduce flood stages near the eastern limits of Wauwatosa and to achieve an acceptable downstream transition with the existing channel. Following review of the channelization plan for Wauwatosa, the Menomonee River Watershed Committee requested that an additional alternative, combining channel modification, structure floodproofing, and structure removal be developed. That plan was finally recommended for adoption by the Committee because it would retain the "natural" character of the stream in critical areas; would provide for a locally proposed expansion of Hart Park through structure removal to the east of the Park; and would offer a long-range solution to the perceived problem of decreasing property values in the area bounded by Hart Park on the west, W. State Street on the north, N. 70th Street on the east, and the Menomonee River on the south. The estimated cost of that alternative and the attendant benefit-cost ratio are presented in Table 98.

Subsequent to a series of informational meetings and a public hearing on the plan, the Watershed Committee reconsidered and revised its recommendation for the reach of the River through the City of Wauwatosa. The plan for structure floodproofing and removal between W. Harwood Avenue and N. 70th Street, as initially recommended by the Watershed Committee, was opposed by some residents favoring channelization, others opposed channelization on environmental and aesthetic grounds, while still others favored structure removal. In addition, the Wauwatosa Common Council adopted two resolutions. The first of these indicated support of channel cleaning and minor channel modification and the second indicated opposition to any major channel modifications, not only within the City, but anywhere in the watershed.

After careful consideration of the public comments and the Common Council resolutions, the Watershed Committee reiterated its recommendation for structure floodproofing and removal between W. Harwood Avenue and N. 70th Street and recommended that a channelization-dike and floodwall subalternative be implemented downstream of N. 70th Street in the City of Wauwatosa. The subalternative for the reach downstream of N. 70th Street called for a rectangular concrete channel having a width approximately equal to the existing bank-to-bank width of the River in the reach with a depth sufficient to convey the 100-year recurrence interval flood flow under planned land use and channel conditions within the confines of dikes and floodwalls having a maximum height of about four feet above existing ground grade. Under the subalternative, the channel modification would extend 0.93 mile into the City of Milwaukee to N. 45th Street. A seven-foot-high drop structure would be provided on the downstream side of N. 70th Street and a three-foot-high drop structure at N. 45th Street. The option of substituting stepped channel sidewalls constructed of rock gabions was made available to provide a more natural appearance for the channel and to avoid potential safety problems associated with vertical sidewalls. It was noted that the use of gabions, which have a greater hydraulic roughness than concrete, would require deepening the channel by up to three feet. A substantial reduction in flood damages in the City of Milwaukee upstream of N. 45th Street would also be achieved through implementation of the subalternative. Therefore, the floodproofing recommendation for the City of Milwaukee was retained, but the scope and expense of the floodproofing was reduced.

The total capital cost of the recommended channelization-dike and floodwall plan from N. 45th Street through N. 70th Street and the structure floodproofing and removal plan from N. 70th Street through Harwood Avenue as estimated for the watershed study, expressed in 1986 dollars, would be $10,931,000. Utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of the plan is estimated to be $704,700, including $10,600 in annual operation and maintenance costs. The benefit-cost ratio of the plan is estimated to be 1.28.

Recommended Flood Control System Plan: The flood control plan developed as part of this system planning effort represents a refinement of that proposed under the watershed study. Incorporated into this recommendation are the results of the hydrologic and hydraulic analyses conducted as a part of this system plan, updated topographic information, and current recreation and redevelopment plans which impact on the recommended plan.

As shown on Map 156, the flood control plan for this reach of the Menomonee River consists of a combination of channel modification, dike construction, and structure floodproofing and elevation. Full implementation of this plan would serve to eliminate structural flood damages in this reach of the Menomonee River for floods up to and including the 100-year recurrence interval flood under planned land use and channel conditions. The peak flood profile attendant to planned land use and channel conditions is shown on Figure 71.

A structure floodproofing and elevation flood control system was found to be a technically feasible solution to the residual flood problem following channel modification and dike construction. The 100-year recurrence interval flood stage under planned land use and channel conditions was used to estimate the number of existing flood-prone structures to be floodproofed and the approximate costs involved.

In the case of residential structures, floodproofing was assumed to be feasible if the design flood stage was below the first floor elevation. Structure elevation was considered feasible for residential structures with basements if the estimated cost of elevating the structure and floodproofing the basement was less than the estimated removal cost. Structures to be elevated were assumed to have the first floor raised to an elevation of at least two feet above the 100-year recurrence interval flood stage to provide adequate freeboard. For aesthetic reasons, structure elevation was limited to a maximum of four feet. Structures which would have to be elevated more than four feet were considered for removal.

As set forth below, following channel modification and dike construction, residual structural damages in the reach from N. 43rd Street through Glenview Avenue extended could be eliminated during floods up to a 100-year recurrence interval under planned land use conditions by floodproofing 23 structures in the City of Milwaukee, and 11 structures in the City of Wauwatosa, and by elevating one structure in the City of Milwaukee.

Channel deepening is recommended for the 2.24-mile-long reach from the pedestrian bridge upstream of Hart Park in Wauwatosa through the existing drop structure at N. 45th Street in Milwaukee. The channel modifications call for lowering the existing streambed about nine feet at N. 70th Street and about five feet at N. 45th Street, as shown on Figure 71. Based on soil and rock borings taken for the Metropolitan Sewerage District's deep tunnel project,[5] the bedrock surface profile along the river channel alignment was estimated as shown on Figure 71. The profile indicates that the modified channel would have to be constructed partially in rock for about one mile of its total length of 2.24 miles.

The recommended modified channel would not cause significant disturbance of the existing streambanks and the existing trees and vegetation along the banks would essentially be retained. The modified channel would consist of a low-flow channel and a flood control channel as shown on Map 156. The three-foot-deep, riprap-lined trapezoidal low-flow channel would have a four-foot bottom width and one vertical on 2.5 horizontal side slopes. The two- to six-foot-deep flood control channel would have stepped sidewalls constructed of bedrock, rock gabions, or limestone blocks at an approximate average slope of 0.5 horizontal on one vertical. The channel bottom width, including the low-flow channel, would be about 51 feet. The existing cobbles, boulders, and rock slabs in the streambed would be saved during excavation and used to line the flood control channel bed in those reaches where the channel is constructed in alluvial material. Vegetation might be established in the interstices of the gabions, creating a more natural appearance for the channel sidewalls.

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[5]*<u>Contract Documents-Crosstown Interceptor and Inline Pump Station</u>, Volume III-Geotechnical Report with Supplements, September 1982.*

Upstream from N. 70th Street, the channel modifications would be limited to those necessary to provide an adequate transition from the deepened downstream channel. The low-flow and flood control channels in that reach would have the same shape and dimensions as in the downstream reach except that the flood control channel transition section would only have a 19-foot bottom width. That transition section would extend for about 0.3 mile upstream from N. 70th Street.

There are existing limestone retaining walls along the streambanks in several sections of the reach for which channel modifications are proposed. The modified channel sidewalls would be constructed adjacent to, and below, those existing walls. A structural analysis of the existing walls should be performed preceding implementation of the recommended improvements, and any necessary repairs should be made prior to, or as part of, channel modification.

The recommended channel work would necessitate the modification of the existing foundations of the N. 68th Street bridge. In addition, the N. 70th Street bridge and the private bridge at River Mile 4.84 would have to be replaced.[6]

The recommended channel deepening would require alteration of several sanitary sewers and water mains which cross beneath the River.

An approximately 1,450-foot-long earthen dike would be constructed along the north bank of the River adjacent to an industrial area located downstream from N. 68th Street, near Jacobus Park. With the exception of a low 150-foot-long section on private property at the downstream end, the dike would be located entirely on Milwaukee County park land. The dike would have a maximum height of about eight feet to provide three feet of freeboard and would improve riparian aesthetics by screening the industrial area from the view of an observer standing near stream level in Jacobus Park. The

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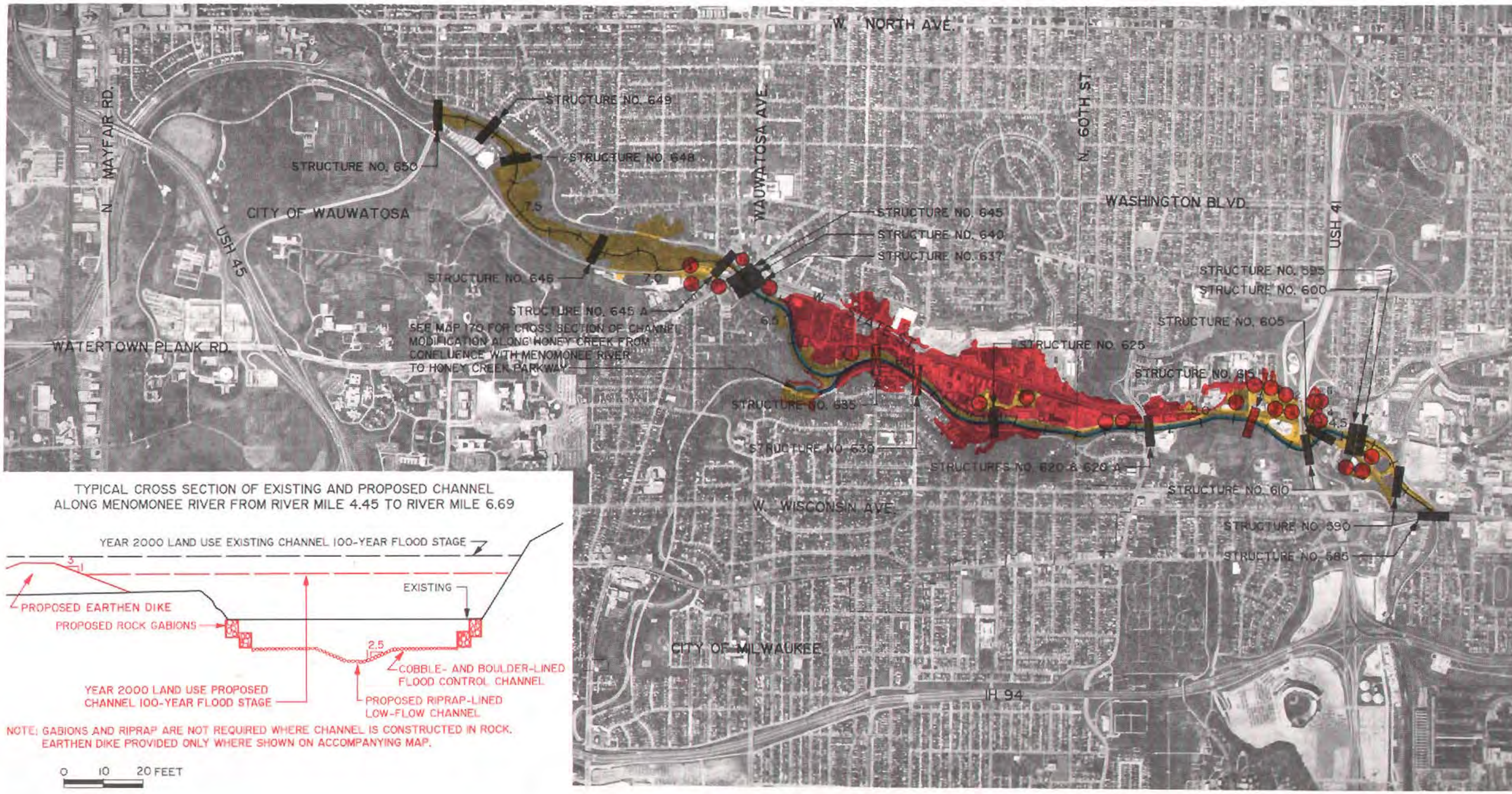
[6]*The channel modification as presented in this section of the report would require replacement of the N. 70th Street bridge; however, the refined recommended plan set forth in a subsequent section eliminates the need for that bridge replacement.*

Map 156

RECOMMENDED FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN FOR THE MENOMONEE RIVER FROM W. WISCONSIN AVENUE TO SWAN BOULEVARD



LEGEND

- 100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS
- 100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS
- 6.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING
- CHANNEL MODIFICATION
- EARTHEN DIKE
- 2 STRUCTURE TO BE FLOODPROOFED AND NUMBER OF STRUCTURES (IF MORE THAN ONE)
- STRUCTURE TO BE ELEVATED
- BRIDGE TO BE REPLACED
- BRIDGE REQUIRING ALTERATION OF FOUNDATION

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR MENOMONEE RIVER IS SHOWN IN APPENDIX H

DUE TO MAP SCALE LIMITATIONS, THE DIFFERENCE BETWEEN THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS, AND THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS, MAY NOT APPEAR ON THIS MAP. WHERE NO DIFFERENCE APPEARS REFERENCE SHOULD BE MADE TO THE FLOOD STAGE PROFILE SHOWN BELOW

GRAPHIC SCALE

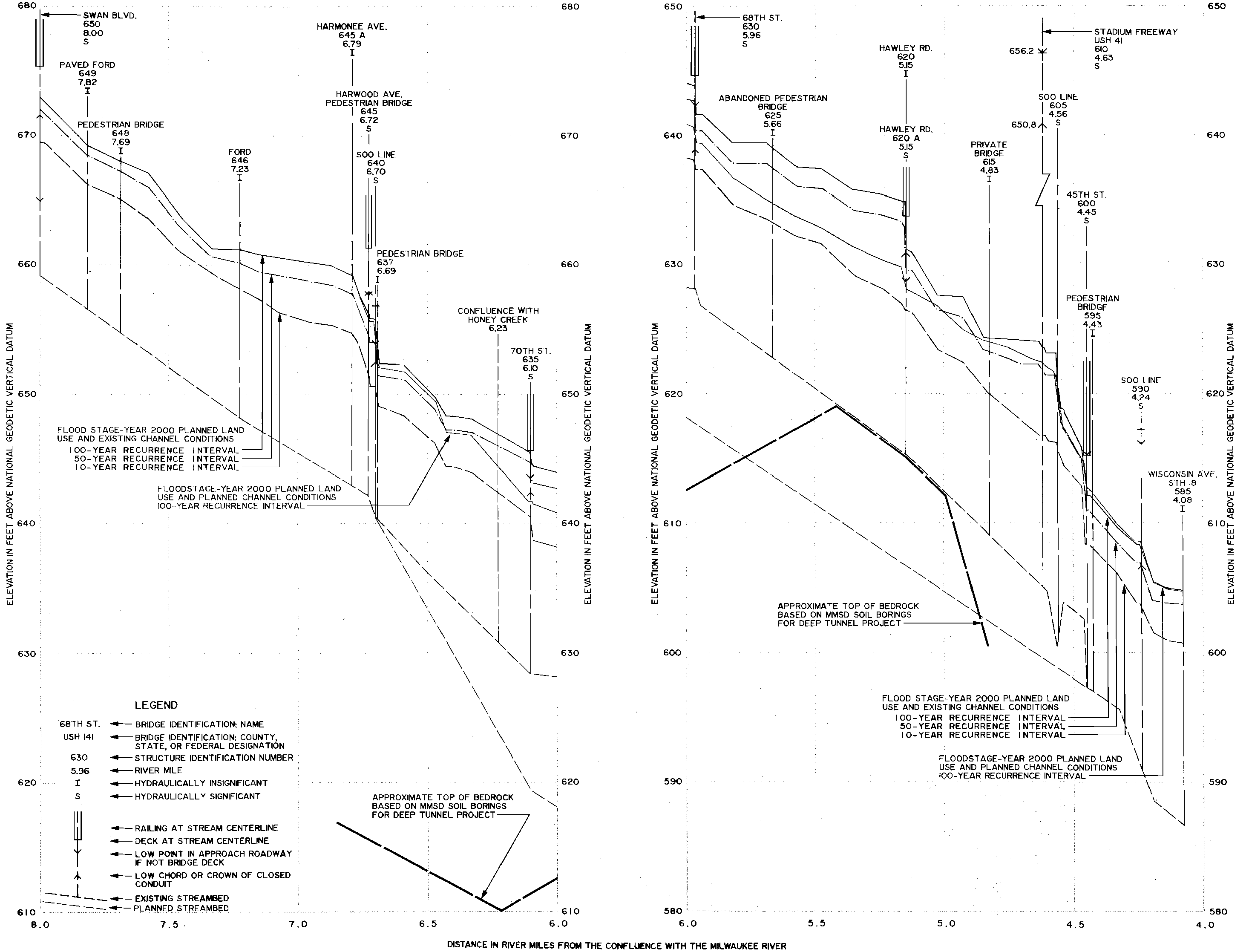
0 1/2 1 MILE

DATE OF PHOTOGRAPHY APRIL 1986

Source: SEWRPC.

Figure 71

RECOMMENDED PLAN FLOOD STAGE PROFILE FOR THE MENOMONEE RIVER FROM W. WISCONSIN AVENUE TO SWAN BOULEVARD



Source: SEWRPC.

dike crest elevation would range from about 637 feet NGVD at its downstream end to about 641 feet NGVD at its upstream end. In the facilities design stage, it is recommended that the dike be aligned so that mature trees along the bank are preserved to the extent possible. The dike and modified channel in the reach along Jacobus Park would be aligned to avoid any disturbance of the south streambank in order to preserve the rare and valuable plant species which have been identified along the bank. The recommended channel modifications would remove approximately 50 buildings from the 100-year floodplain in the Cities of Milwaukee and Wauwatosa along the reach from N. 68th Street to N. 43rd Street, but approximately 56 buildings would remain in the floodplain without construction of the recommended dike. Construction of the dike along this reach would remove 27 of those buildings from the floodplain, leaving only 23 to be floodproofed and one to be elevated in the City of Milwaukee, and five to be floodproofed in the City of Wauwatosa.

Local runoff from the area bounded by N. 68th Street on the west, the Soo Line (formerly Chicago, Milwaukee, St. Paul & Pacific Railroad) railway embankment on the north, N. 60th Street extended on the east, and the recommended dike on the south would collect on the landward side of the dike. In order to avoid the cost of constructing stormwater pumping stations, estimated at about $1,500,000, the storm sewer outlet in N. 63rd Street extended would not be provided with a backwater gate but additional storm sewer capacity would be provided to handle runoff blocked by the proposed dike. That runoff would have to be collected and conveyed to the location of the existing storm sewer outfall in N. 63rd Street extended. Because it would be necessary for the collection and conveyance facilities to have adequate hydraulic capacity to convey the peak flow of about 160 cubic feet per second (cfs) from a 100-year recurrence interval event, the existing 36-inch-diameter reinforced concrete storm sewer outlet in N. 63rd Street would have to be replaced with a 66-inch-diameter pipe. Five buildings north of the dike would remain in the 100-year floodplain. It is recommended that those buildings be floodproofed.

A 9-foot by 5.5-foot concrete box storm sewer discharges to the river just east of N. 62nd Street extended. That storm sewer drains a large area extending west beyond N. 76th Street and north to W. North Avenue, collecting runoff from Schoonmaker Creek. The reduced flood stages in the Menomonee River due to construction of the recommended channel modification would improve the hydraulic efficiency of that storm sewer under flood conditions. There would only be minor backup from the river through the sewer, producing localized, shallow street flooding along an approximately 400-foot section of W. State Street. The minor inconvenience due to such street flooding would not merit the provision of backwater gates and provision of a stormwater pumping station for the box sewer.

City staff has indicated that there are no longer any plans to expand Hart Park eastward to N. 70th Street. Therefore, the structure removal recommendation of the watershed study for the area bounded by Hart Park, W. State Street, N. 70th Street, and the River was reevaluated under this system planning effort. The watershed study recommended removal of all buildings in that area, whether in or out of the floodplain, in order to solve the flooding problem and provide usable park land. Once the locally proposed recreation objective no longer exists, there is no need to acquire all buildings in the area. The recommended channel modifications alone would remove approximately 80 buildings from the 100-year floodplain in the reach from N. 68th Street through Hart Park, but 21 buildings, including those in Hart Park, would remain in the floodplain. Construction of a dike along this reach would remove 20 of those buildings from the floodplain.

It is recommended that a 1,650-foot-long, two- to eight-foot-high dike be constructed along the north bank adjacent to Hart Park in Wauwatosa. The dike would terminate at N. 72nd Street to avoid having to provide expensive stormwater pumping facilities to handle runoff from the relatively large area draining to the river east of N. 72nd Street. This would leave one residence in the floodplain. It is recommended that the residence be floodproofed. Under the proposed dike alignment, one small pumping station would be provided to handle localized stormwater runoff from a relatively small drainage area in Hart Park. A storm sewer or swale would be provided along the landward side of the dike to convey runoff to the pumping station.[7]

To accommodate the recommended Menomonee River streambed elevation without providing drop structures along Honey Creek, it would be necessary to lower the Honey Creek streambed up to seven feet in the 0.17-mile-long reach between the Honey Creek Parkway bridge and its confluence with the Menomonee River. It is recommended that the Honey Creek streambed be lowered by constructing a channel below the existing streambed and within the existing banks. As shown on Map 170, that could be accomplished with a trapezoidal channel, having a four-foot-wide bottom and average side slopes of 0.7 horizontal to one vertical. The stepped channel sidewalls would be constructed of rock gabions or limestone block, in a manner similar to those recommended for the Menomonee River. Costs for the channel modification along this reach of Honey Creek are included in the Menomonee River costs because the modifications along Honey Creek are solely required because of the recommended modification along the Menomonee. The reach of Honey Creek near its confluence with the Menomonee River is experiencing severe erosion problems. Bank stabilization measures for that reach should be considered during the design of the recommended channel modifications.

Floodproofing of two commercial buildings and three industrial buildings located upstream of Hart Park in the vicinity of Harwood Avenue and the Harmonee Avenue Bridge is also recommended.

The changes in the flood discharges which may be expected along the Menomonee River as a result of the recommended channel modification are provided in Table 99. No increase in the 100-year flood discharge would be anticipated downstream from River Mile 3.21. In the 1.12-mile-long reach from River Mile 3.21 through River Mile 4.33, the increased flood flows may be expected to result in increases of from 0.05 to 0.22 foot in the 100-year recurrence interval flood stage under planned land use and channel conditions. With the exception of the 0.24-mile-long reach from River Mile 3.71 to 3.95 and the 0.07-mile-long reach from River Mile 4.26 to 4.33, those 100-year stage increases would essentially be contained within the modified channel. In the 600-foot reach from River Mile 4.33 through 4.45, the increased flood flows may result in localized 100-year flood stage increases of 0.22 to 0.55 foot. Upstream from River Mile 4.45, 100-year flood stages would be reduced by 0.4 to 6.7 feet due to the increased hydraulic capacity provided by the recommended channel modifications.

The potential increases in the 100-year flood stage under planned land use and channel conditions were considered in the development of the recommended plans for the entire reach downstream from the recommended channel modifications. Implementation of the recommended plan for each part of that downstream reach, as set forth above, would essentially eliminate all flood related damages to existing structures along the Menomonee River from River Mile 4.45 through the downstream end of the study reach at Falk Dam at River Mile 2.22. However, because the 100-year recurrence interval flood stage would be expected to increase by more than 0.01 foot downstream of the recommended channel modification, it may be necessary to make legal arrangements with affected downstream property owners prior to construction of the recommended channel modifications.

Utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of the recommended plan for this reach is estimated at $464,000. This cost consists of the amortization of the $7,116,000 capital cost, including $4,737,000 for channel modification, $185,000 for dikes, $792,000 for structure floodproofing or elevation, $485,000 for stormwater pumping stations and stormwater drainage facilities, $838,000 for bridge removal and replacement, and $79,000 for bridge foundation modification, plus $12,000 in annual operation and maintenance costs. The average annual flood damage abatement benefit is estimated at $129,000, yielding a benefit-cost ratio of 0.28.

[7] *Under the refined recommended plan presented in a subsequent section of this chapter, the need would be eliminated for the downstream 840 feet of the proposed dike and for the proposed stormwater pumping station.*

Table 99

IMPACT OF RECOMMENDED FLOOD CONTROL PLAN FOR THE MENOMONEE RIVER DOWNSTREAM OF GLENVIEW AVENUE EXTENDED AT RIVER MILE 6.88 ON 100-YEAR RECURRENCE INTERVAL FLOOD DISCHARGE

Stream	Location	River Mile	100-Year Recurrence Interval Flood Discharges (cfs) Year 2000 Planned Land Use		Percent Increase
			Existing Channel Condition	Recommended Plan Condition	
Menomonee River	At mouth	0.00	16,800	16,800	0
	S. 32nd Street extended	2.45	16,400	16,400	0
	Upstream of confluence with Woods Creek	3.21	14,900	15,100	1
	Soo Line Railway	4.24	13,700	14,000	2
	N. 70th Street	6.10	13,600	14,000	3
	Upstream of confluence with Honey Creek	6.24	10,200	10,200	0

Source: SEWRPC.

It should be noted that the economic analysis for the watershed study assumed benefits for elimination of secondary flooding due to the implementation of the recommended flood control measures, while the analyses herein presented do not. As a result, the total benefit amounts determined under the watershed study are higher than those determined under this study. It should be recognized that the flood control measures recommended here would indeed produce some secondary flooding benefits. Thus, the actual benefit cost ratios may be expected to be higher than those herein presented for the recommended plan.

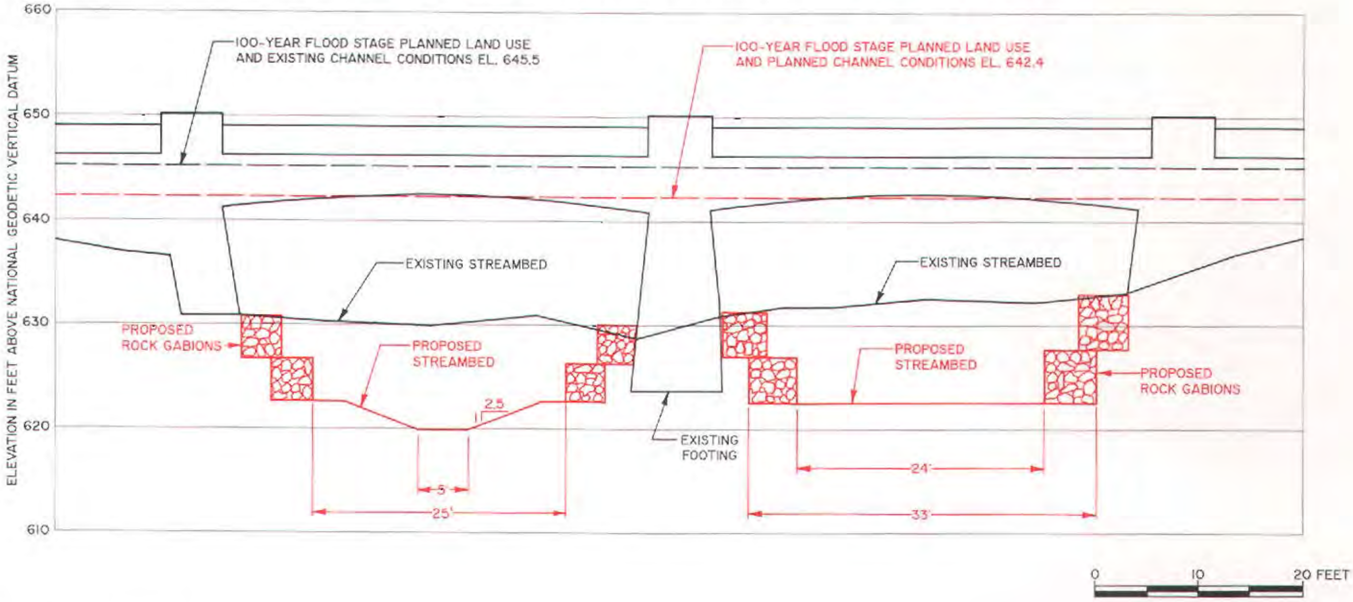
Refinement of Recommended Flood Control System Plan: During the June 14, 1990, meeting of the Technical Advisory Committee, the Engineer of the City of Wauwatosa requested that additional consideration be given to the possibility of maintaining the existing N. 70th Street bridge instead of replacing this bridge, as initially recommended. He noted that recent inspection had resulted in a good structural rating for the bridge, and that only resurfacing of the bridge deck was being proposed by the City. Bridge removal and replacement was initially recommended in the flood control system plan since it was believed that the City would be replacing the bridge for structural reasons, thus minimizing the need for channel modifications upstream of N. 70th Street, while achieving the desired degree of flood control. The minimization of channel modification was intended to be as consistent as practicable with the position of Common Council of the City of Wauwatosa when the Menomonee River watershed plan was completed in 1976.

In response to city staff request, the commission staff determined that additional channel modification in the vicinity of the bridge would permit the bridge to be maintained, while still accomplishing the flood control objectives of the plan. While the alternative approach would laterally expand the channel modification initially proposed, the modification could still be confined within the existing channel banks.

The changes to the initially-recommended plan would all occur in the approximately one-third-mile-long reach beginning at the downstream

Figure 72

REFINED RECOMMENDED MODIFIED CHANNEL CROSS SECTION FOR THE MENOMONEE RIVER AT THE N. 70TH STREET BRIDGE



Source: SEWRPC.

side of the N. 70th Street bridge and proceeding upstream. As shown in Figure 72, the existing bridge could be maintained by lowering the streambed on either side of the bridge pier. The channel modification would be designed to minimize disturbance to the bridge foundation; however, some foundation modification may be required. By itself, the proposed channel modification at the bridge would not sufficiently lower the upstream 100-year recurrence interval flood profile to eliminate the potential for flooding due to backwater through the N. 71st Street storm sewer outfall. Therefore, it is recommended that the modified flood control channel section for the first 400 feet upstream of the bridge be expanded from the originally-recommended 19-foot bottom width to a 51-foot width. The section would be similar to that shown on Map 156. In the next 770 feet of the River, the modified flood control channel width would transition from 51 feet to 19 feet. The 19-foot-wide flood control channel bottom would be maintained for 360 feet and would then transition to the existing river channel section over the next 1,480 feet. The modified streambed profile would remain as shown on Figure 71.

Upstream of N. 70th Street, the reduction in the 100-year recurrence interval flood profile due to the revised channel modifications would be sufficient to remove an additional 18 buildings from the existing floodplain, while enabling elimination of both the 840 feet of low dike downstream and the stormwater pumping station originally recommended for that reach. The stormwater pumping station would no longer be required because elimination of the downstream portion of the proposed dike would provide stormwater runoff with an unobstructed path to the River. The construction of the upstream 810 feet of proposed dike, which would remove the Muellner Building and the Park Administration and Athletic Building in Hart Park from the floodplain, would remain a part of the recommended plan. Floodproofing of one residence located on the west side of N. 71st Street at the Menomonee River would still be required under the refined recommended plan.

With respect to the entire 2.24-mile-long reach of the Menomonee River from N. 43rd Street in the City of Milwaukee through Glenview Avenue extended in the City of Wauwatosa it is recommended that, during final project design, consideration be given to providing erosion protection for the existing streambanks above the recommended modified channel. Such protection would need to be considered only in reaches where there are no existing limestone retaining walls, or where those walls are not structurally adequate.

Utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of the refined recommended plan for this reach is estimated at $386,000. This cost consists of the amortization of the $5,988,000 capital cost, including $4,797,000 for channel modification, $116,000 for dikes, $637,000 for structure floodproofing or elevation, $170,000 for storm sewers, $117,000 for removal and replacement of one privately-owned bridge, and $151,000 for bridge foundation modification, plus $6,000 in annual operation and maintenance costs. The average annual flood damage abatement benefit is estimated at $129,000, yielding a benefit-cost ratio of 0.33. The refined benefit-cost ratio is somewhat higher than the ratio of 0.28 for the initial plan.

Consideration of Potential Redevelopment of the Former Jacobus Quickflash Company Property: At the request of the City of Wauwatosa, consideration was given to the impact on flood flows and stages on the potential for redevelopment of the former Jacobus Company heating oil tank farm located along the south bank of the River between River Miles 6.79 and 6.88 near the Harmonee Avenue bridge. Any future redevelopment of that property for purposes other than appropriate open space uses would require filling within the floodway. Such filling may be expected to raise upstream 100-year recurrence interval flood stages under planned land use and existing channel conditions a maximum of 0.03 foot in the reach from River Mile 6.79 through River Mile 7.47. It is recommended that the property be kept in open space uses; however, if the City should decide to permit redevelopment for intensive urban use, appropriate legal arrangements with upstream property owners would be required. Although most of the upstream floodlands are owned by Milwaukee County, some private property would also be affected in the reach extending about 1,500 feet upstream of the redevelopment site.

Flood Control and Related Drainage System Plan Implementation: It is recommended that the refined recommended plan for the reach of the Menomonee River from N. 43rd Street through Glenview Avenue extended be implemented expeditiously through the cooperative efforts of the Cities of Milwaukee and Wauwatosa, Milwaukee County, private property owners, and the Milwaukee Metropolitan Sewerage District. More specifically, it is recommended that the District design, construct, and maintain: 1) the channel modifications recommended from N. 45th Street at River Mile 4.45 to the pedestrian bridge in Wauwatosa at River Mile 6.69; the associated bridge foundation modifications at N. 68th Street and N. 70th Street; and the private bridge removal and replacement at River Mile 4.84; 2) the channel modifications along Honey Creek from its confluence with the Menomonee River to the Honey Creek Parkway bridge at River Mile 0.17; and 3) the 2,260 feet of dikes proposed for the reach from near N. 62nd Street extended through Hart Park.

It is recommended that the City of Wauwatosa design, construct, and maintain the proposed stormwater conveyance facilities near N. 63rd Street. It is also recommended that the City of Wauwatosa cooperate in the channel modifications and dike construction through the provision of attendant construction easements and rights-of-way.

It is recommended that Milwaukee County cooperate in the channel modifications and dike construction through the provision of attendant construction easements and rights-of-way.

The recommended structure floodproofing or elevation would be implemented by the individual property owners. It is recommended that these private owners bear the cost of structure floodproofing or removal. It is further recommended that the professional services required to prepare plans for the floodproofing and elevation of individual buildings be made available to property owners, at no cost, by the engineering departments of the Cities of Milwaukee and Wauwatosa. Also, it is recommended that the Cities of Milwaukee and Wauwatosa review their building ordinances to ensure that appropriate floodproofing regulations are included. It is recommended that the communities concerned

Table 100

SUMMARY OF REFINED RECOMMENDED PLAN CAPITAL COSTS FOR THE MENOMONEE RIVER FROM N. 43RD STREET THROUGH GLENVIEW AVENUE

Municipality Where Flood Control Measures Are to be Located	Implementing Agency	Flood Control Measure	Estimated Capital Cost
City of Milwaukee	Milwaukee Metropolitan Sewerage District	Channel modifications	$2,176,000
		Private bridge removal and replacement	117,000
		Subtotal	$2,293,000
	Various private property owners	Structure floodproofing and elevation	$ 281,000
		Total in City of Milwaukee	$2,574,000
City of Wauwatosa	Milwaukee Metropolitan Sewerage District	Channel modifications	$2,621,000
		Bridge modification attendant to channel modifiction	151,000
		Dikes	116,000
		Subtotal	$2,888,000
	City of Wauwatosa	Storm sewer	$ 170,000
		Subtotal	$ 170,000
	Various private property owners	Structure floodproofing	$ 356,000
		Total in City of Wauwatosa	$3,414,000
		Total	$5,988,000

Source: SEWRPC.

explore, on behalf of the property owners involved, any available state and/or federal aids for such floodproofing measures.

The capital costs of the various components of the refined recommended plan are apportioned by agency in Table 100.

City of Wauwatosa from W. Harwood Avenue through W. Capitol Drive: The watershed study identified a total of 220 structures as being located in the primary and secondary flooding zones in the reach of the Menomonee River extending from W. Harwood Avenue through W. Capitol Drive. Of these 220 structures, approximately 190 were located in the secondary flooding zone.

Under the watershed study, a total of three flood control alternatives were analyzed from Harwood Avenue through W. Capitol Drive in the City of Wauwatosa. These alternatives included: 1) No Action; 2) Floodproofing and Removal of Structures; and 3) Bridge or Culvert Alteration or Replacement. The estimated cost of each of these alternatives, as well as the attendant benefit-cost ratio, is presented in Table 101.

Table 101

MENOMONEE RIVER WATERSHED STUDY FLOOD CONTROL ALTERNATIVES FOR THE MENOMONEE RIVER IN THE CITY OF WAUWATOSA FROM HARWOOD AVENUE THROUGH W. CAPITOL DRIVE

Alternative	Capital	Cost[a]			Total	Benefit-Cost Ratio
		Amortized Capital[b]	Operation and Maintenance	Other		
1. No Action	$ 0	$ 0	$0	$250,800	$250,800	0
2. Structure Floodproofing and Removal	1,885,400	119,800	0	0	119,800	2.09
3. Bridge Alteration or Removal	- -[c]	- -[c]	- -[c]	- -[c]	- -[c]	- -

[a]*Costs are expressed in 1986 dollars.*

[b]*Amortized capital cost is based on an interest rate of 6 percent and a project life of 50 years.*

[c]*No costs were computed, since this alternative was found to be technically infeasible.*

Source: SEWRPC.

The watershed study concluded that, while removal or modification of bridges producing backwater in excess of 1.0 foot under 100-year planned flood conditions would reduce local flood stages, monetary flood risks would not be significantly reduced within the flood-prone reaches. Thus, further technical and economic analyses of this alternative were not considered necessary.

Under the structure floodproofing and removal alternative, the watershed study called for nine structures to be removed and 211 structures to be floodproofed. That alternative was recommended because it had a benefit-cost ratio greater than one and because the bridge alteration or replacement alternative was judged to be technically infeasible. The total capital cost of the recommended structure floodproofing and removal plan as estimated for the watershed study and updated to 1986 economic conditions would be $1,885,400.

Refined Flood Control System Plan: The flood control plan developed as part of this system planning effort represents a refinement of that proposed under the watershed study. The refined analysis of this reach which was conducted for this study did not include consideration of secondary flooding for the reasons already noted. The re-analysis identified only eight structures which would be expected to experience overland flooding during a 100-year recurrence interval flood under planned land use and channel conditions. Four of those eight structures are located just upstream of the recommended channel modifications in the City of Wauwatosa. The flood control recommendations for those structures are, therefore, included in the preceding section addressing the reach from N. 43rd Street through Glenview Avenue extended. The lower number of buildings in the primary flooding zone as identified under this study is attributable to 1) lower 100-year flood discharges based on the simulation of flows using a 49-year period of record as opposed to a 35-year period used under the watershed study, 2) refinement of the hydraulic model used to calculate water surface profiles, and 3) the availability of updated topographic mapping.

A structure floodproofing and elevation flood control system was determined to be a technically feasible solution to the flood problem. The 100-year recurrence interval flood stage under planned year 2000 land use and planned channel conditions was use to estimate the number of existing flood-prone structures to be floodproofed and the approximate costs involved.

In the case of residential structures, floodproofing was assumed to be feasible if the design flood stage was below the first floor elevation. Structure elevation was considered feasible for residential structures with basements if the estimated cost of elevating the structure and floodproofing the basement was less than the estimated removal cost. Structures to be elevated were assumed to have the first floor raised to an elevation of at least two feet above the 100-year recurrence interval flood stage to provide adequate freeboard. For aesthetic reasons, structure elevation was limited to a maximum of four feet. Structures which would have to be elevated more than four feet were considered for removal.

As shown on Map 157, the refined flood control plan for this reach of the Menomonee River calls for floodproofing of four structures along the east bank of the River in the reach from N. 97th Street through W. Ridge Boulevard. The peak flood profile attendant to planned land use and channel conditions is shown on Figure 73. Full implementation of this plan would serve to eliminate structural flood damages in this reach of the Menomonee River for floods up to and including the 100-year recurrence interval flood under planned land use and channel conditions.

Assuming that the structure floodproofing measures would be fully implemented, the capital cost of the refined recommended plan for this reach is estimated to be $29,000. Utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of the refined recommended plan is estimated to be $1,800. The average annual flood damage abatement benefit under planned land use and existing channel conditions is estimated at $900, yielding a benefit-cost ratio of 0.50.

Flood Control and Related Drainage System Plan Implementation: The recommended structure floodproofing or elevation would be implemented by the individual property owners. It is recommended that these private owners bear the cost of structure floodproofing or removal. It is further recommended that the professional services required to prepare plans for the floodproofing and elevation of individual buildings be made available to property owners, at no cost, by the City of Wauwatosa engineering department. Also, it is recommended that the City of Wauwatosa review its building ordinance to ensure that appropriate floodproofing regulations are included. It is recommended that the City explore, on behalf of the property owners involved, any available state and/or federal aids for such floodproofing measures.

Menomonee River in the Village of Menomonee Falls: Under the watershed study, a total of three flood control alternatives were considered in the Village of Menomonee Falls. These alternatives included: 1) No Action; 2) Floodproofing and Removal of Structures; and 3) Major Channel Modification as proposed by the Village. The estimated cost of each of these alternatives, as well as the attendant benefit-cost ratio, are presented in Table 102. The major channel modification alternative proposed by the Village was intended to abate flood damages and to provide a lower channel bed which would provide adequate outfalls for existing and planned storm sewers along the river.

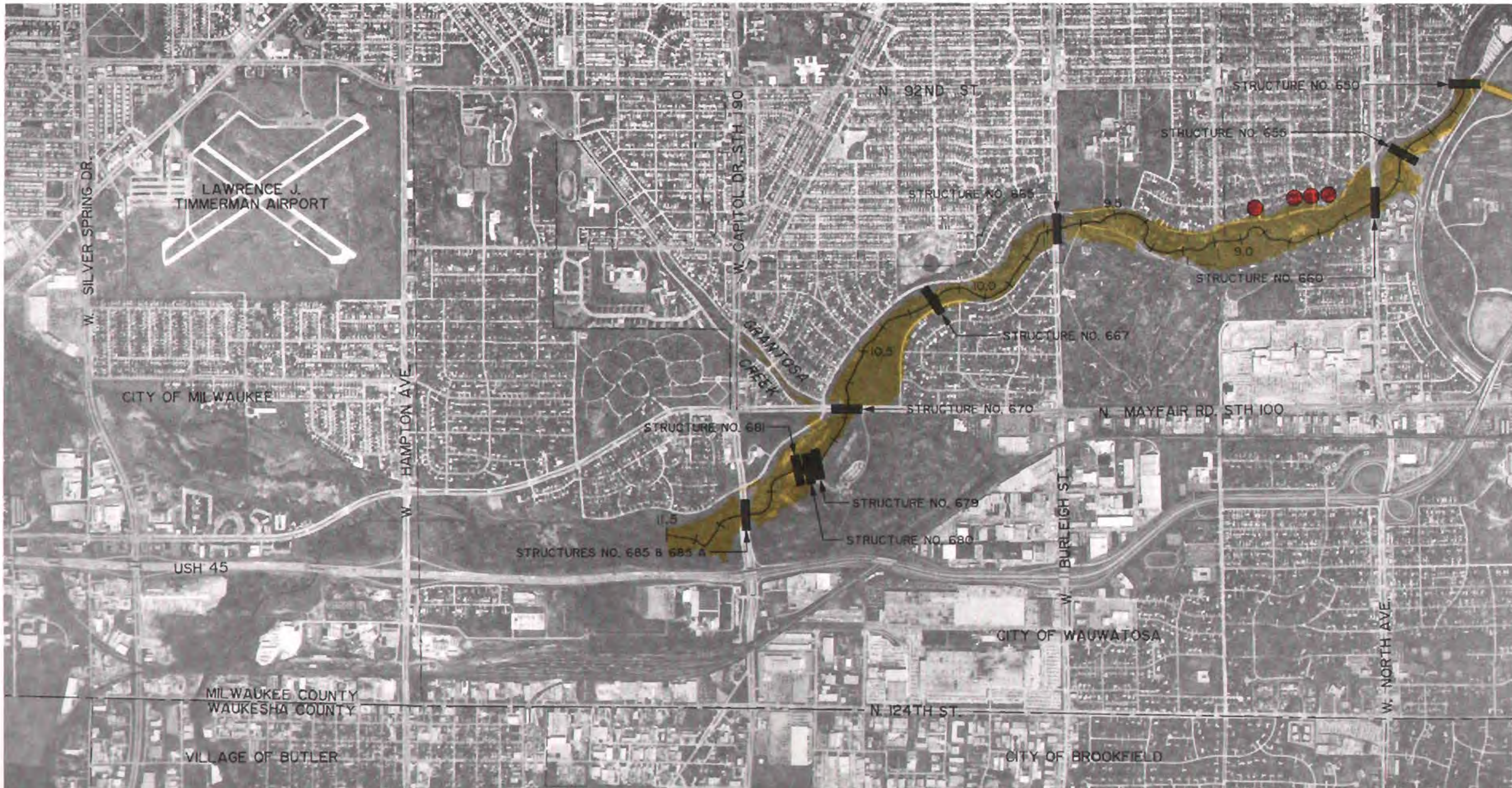
The commission staff recommended the use of structure floodproofing and removal to resolve existing and forecast flood problems; however, the Watershed Committee recommended implementation of the Village's channel modification plan in light of the Village's commitment to channelization as reflected by the location and size and grades of existing and proposed storm sewers and storm sewer outfalls.

The total capital cost of the recommended channel modification plan as estimated for the watershed study and updated to 1986 economic conditions would be $4,157,000. Utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of the plan is estimated to be $272,600, including $8,600 in annual operation and maintenance costs. The benefit-cost ratio of the plan was estimated to be 0.27.

Refined Flood Control System Plan: The flood control plan developed as part of this system planning effort represents a refinement of that proposed under the watershed study. The refined analysis of this reach which was conducted for this study did not include consideration of secondary flooding for reasons already noted. The watershed study identified a total of 120 structures in the primary and secondary flooding zones. This system planning effort identified

Map 157

RECOMMENDED FLOOD CONTROL PLAN FOR THE MENOMONEE RIVER FROM SWAN BOULEVARD TO W. SILVER SPRING DRIVE



LEGEND

- 100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS
- 9.5 — APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING
- STRUCTURE TO BE FLOODPROOFED

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR MENOMONEE RIVER IS SHOWN IN APPENDIX H

NOTE: THE FLOODLAND LIMITS SHOWN ALONG GRANTOSA CREEK ARE BASED ONLY ON BACKWATER FROM THE MENOMONEE RIVER DURING A 100-YEAR RECURRENCE INTERVAL FLOOD. AS SUCH, THEY DO NOT REPRESENT THE FLOODLAND LIMITS RESULTING FROM A 100-YEAR FLOOD OCCURRING ALONG GRANTOSA CREEK.

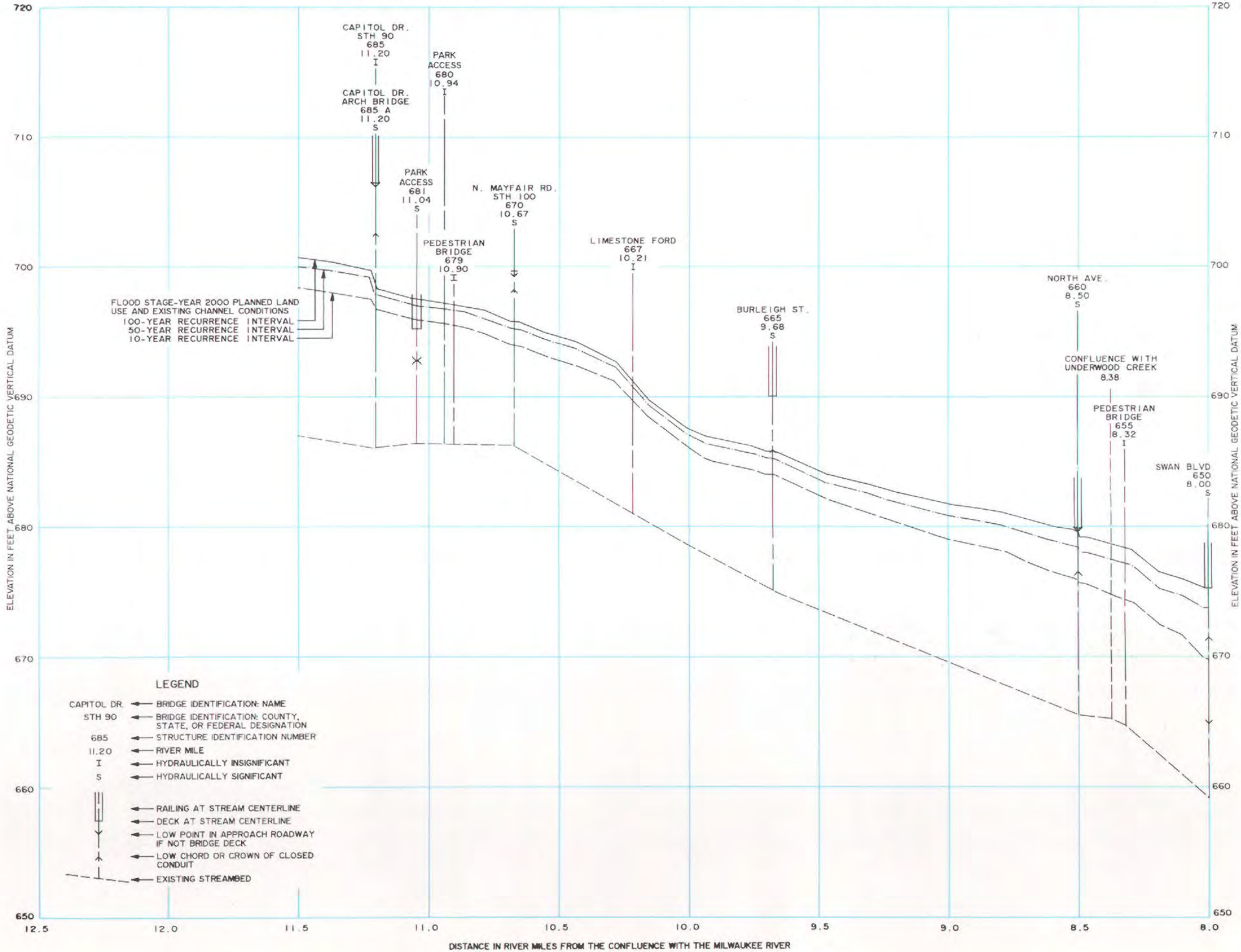
GRAPHIC SCALE

0 1/2 1 MILE

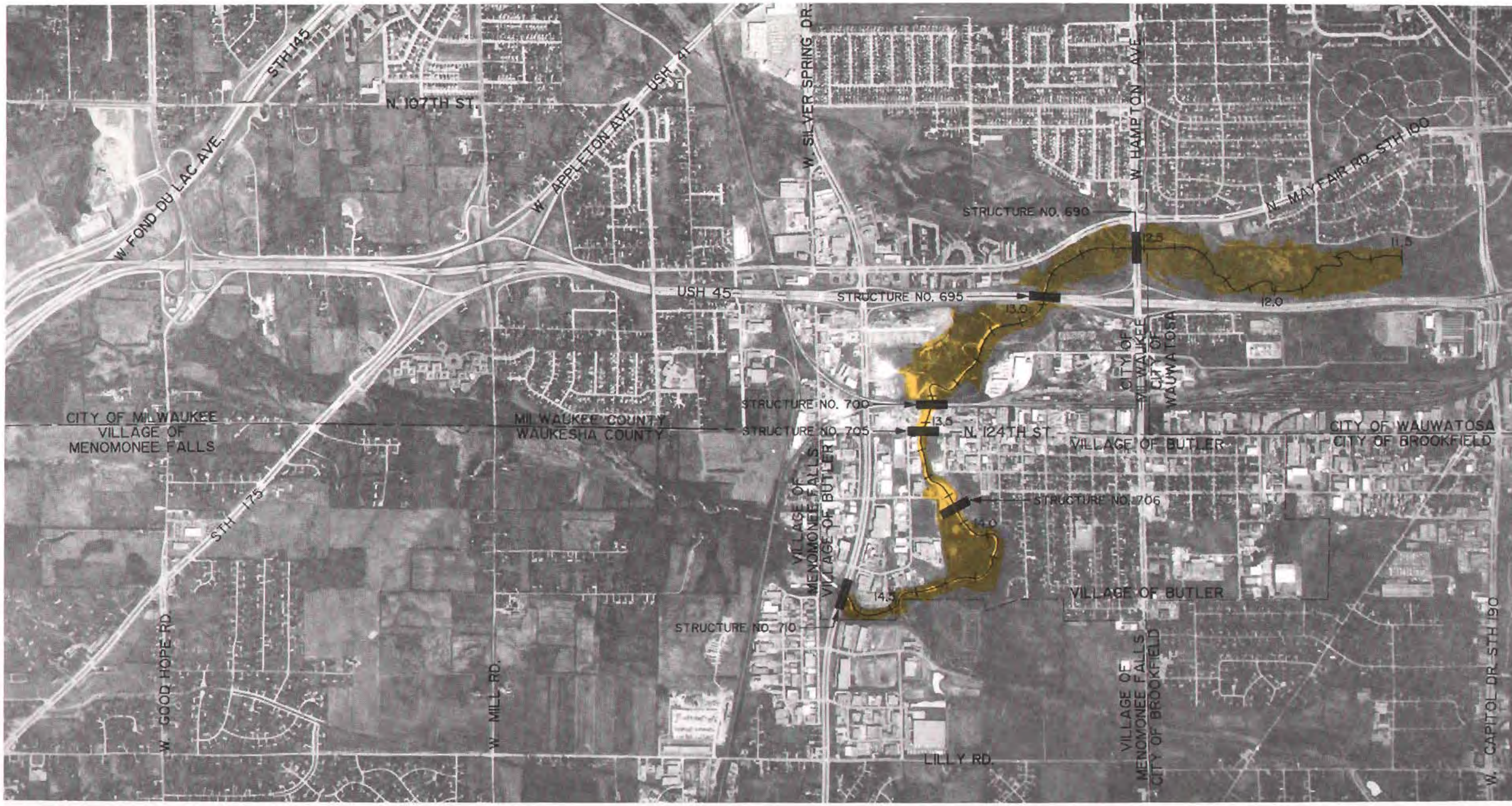
DATE OF PHOTOGRAPHY: APRIL 1986

Figure 73

RECOMMENDED PLAN FLOOD STAGE PROFILE FOR THE MENOMONEE RIVER FROM SWAN BOULEVARD TO W. SILVER SPRING DRIVE



Map 157 (continued)



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS

13.5 — APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR MENOMONEE RIVER IS SHOWN IN APPENDIX H

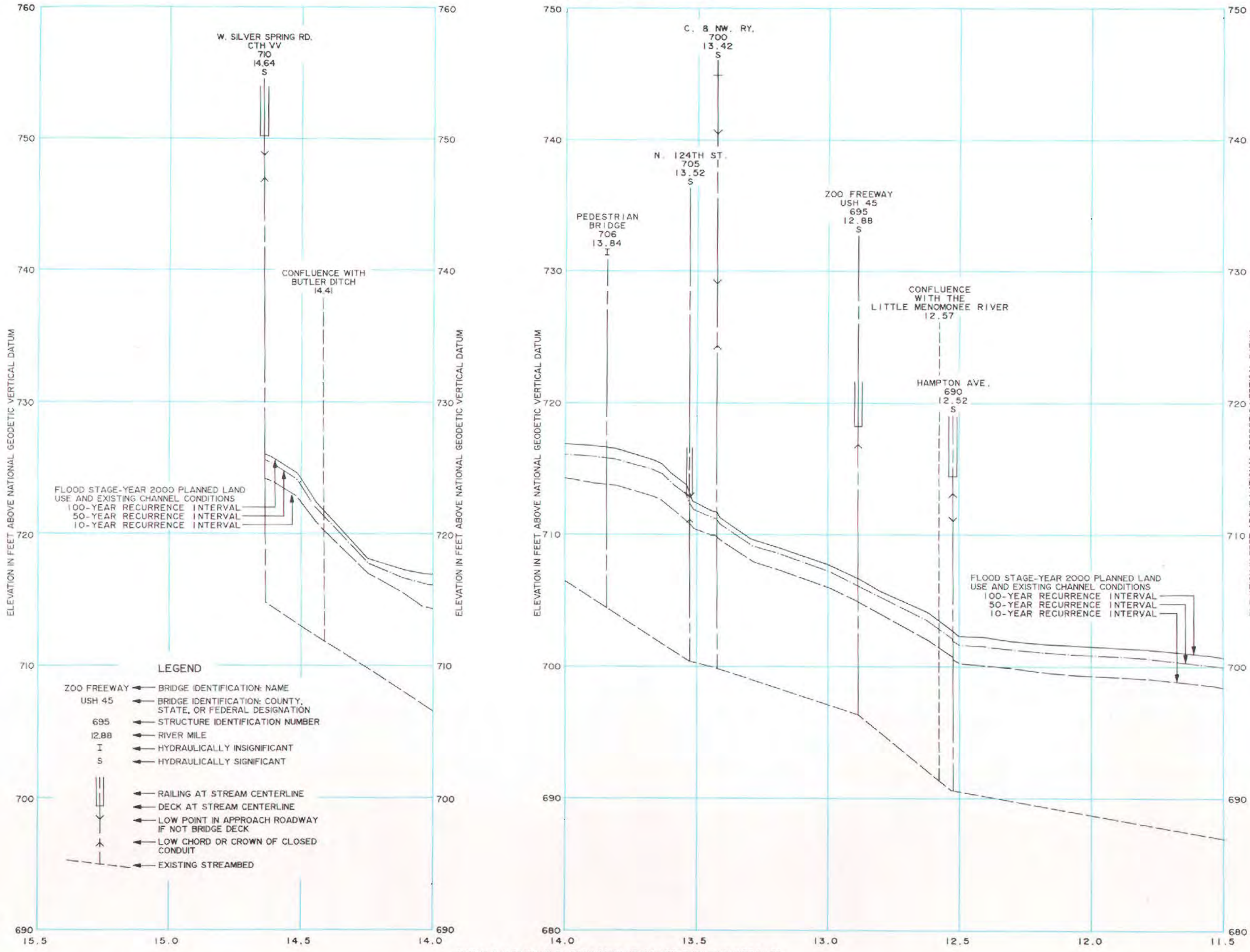
GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Source: SEWRPC.

Figure 73 (continued)



Source: SEWRPC.

Table 102

MENOMONEE RIVER WATERSHED STUDY FLOOD CONTROL ALTERNATIVES FOR THE MENOMONEE RIVER IN THE VILLAGE OF MENOMONEE FALLS

Alternative	Cost[a]					Benefit-Cost Ratio
	Capital	Amortized Capital[b]	Operation and Maintenance	Other	Annual Total	
1. No Action	$ 0	$ 0	$ 0	$72,600	$ 72,600	0
2. Structure Floodproofing and Removal						
Between Margaret Road Extended and Lilly Road	143,000	9,000	0	0	9,000	1.88
Between Jacobson Drive Extended and 700 Feet West of Pilgrim Road	21,800	1,400	0	0	1,400	13.60
Between Northern Village Limits and STH 74	311,000	19,600	0	0	19,600	1.87
Subtotal	$ 475,800	$30,000	$ 0	$ 0	$ 30,000	2.42
3. Major Channel Modification Between Northern Village Limits and STH 74; and Between Arthur Avenue and the Eastern Limits of the Village	$4,157,000	$264,000	$8,600	$ 0	$272,600	0.27

[a]*Costs are expressed in 1986 dollars.*

[b]*Amortized capital cost is based on an interest rate of 6 percent and a project life of 50 years.*

Source: SEWRPC.

24 buildings which may be expected to experience overland flooding during a 100-year recurrence interval flood under planned land use and existing channel conditions. The lower number of buildings in the primary flooding zone as identified under this study is attributable to 1) lower 100-year flood discharges in certain reaches based on the simulation of flows for a 49-year period of record as opposed to a 35-year period for the watershed study, 2) refinement of the hydraulic model used to calculate water surface profiles, and 3) the availability of updated topographic mapping.

As shown on Maps 158, 159, and 160, the refined flood control plan for the Menomonee River within the Village of Menomonee Falls consists of a combination of channel modification plus structure floodproofing and elevation. The peak flood profile attendant to planned land use and channel conditions is shown on Figures 74, 75, and 76. Full implementation of this plan would serve to eliminate structural flood damages in this reach of the Menomonee River for floods up to and including the 100-year recurrence interval flood under planned land use and channel conditions.

A structure floodproofing and elevation flood control system was analyzed to determine if such a structure-by-structure approach would be a technically feasible and economically viable solution to the flood problem in the reach from Silver Spring Road through Lilly Road. The 100-

year recurrence interval flood stage under planned year 2000 land use and planned channel conditions was use to estimate the number of existing flood-prone structures to be floodproofed and the approximate costs involved.

In the case of residential structures, floodproofing was assumed to be feasible if the design flood stage was below the first floor elevation. Structure elevation was considered feasible for residential structures with basements if the estimated cost of elevating the structure and floodproofing the basement was less than the estimated removal cost. Structures to be elevated were assumed to have the first floor raised to an elevation of at least two feet above the 100-year recurrence interval flood stage to provide adequate freeboard. For aesthetic reasons, structure elevation was limited to a maximum of four feet. Structures which would have to be elevated more than four feet were considered for removal.

As a result of the re-analysis, the initial commission staff recommendation for structure floodproofing, elevation, and removal to abate flood damage problems in the Village of Menomonee Falls is reinstated at the locations shown on Maps 158 and 159. Those maps show eight buildings that are recommended to be floodproofed and one building that is recommended to be elevated.

The final recommendation of the Menomonee River Watershed Committee called for channel modification in the vicinity of Lilly Road to abate flood damages and to provide an adequate storm sewer outlet for the area north of the river. Based on discussions with village officials and on preliminary consideration of possible stormwater drainage alternatives, it was concluded that local stormwater drainage problems could probably be resolved through the provision of a combination of upland detention storage and storm sewer conveyance without deepening the existing channel. Therefore, because channel deepening is not required to resolve local stormwater drainage problems and because channel modification to abate flooding is obviously more expensive than structure floodproofing and elevation, structure floodproofing and elevation is recommended.

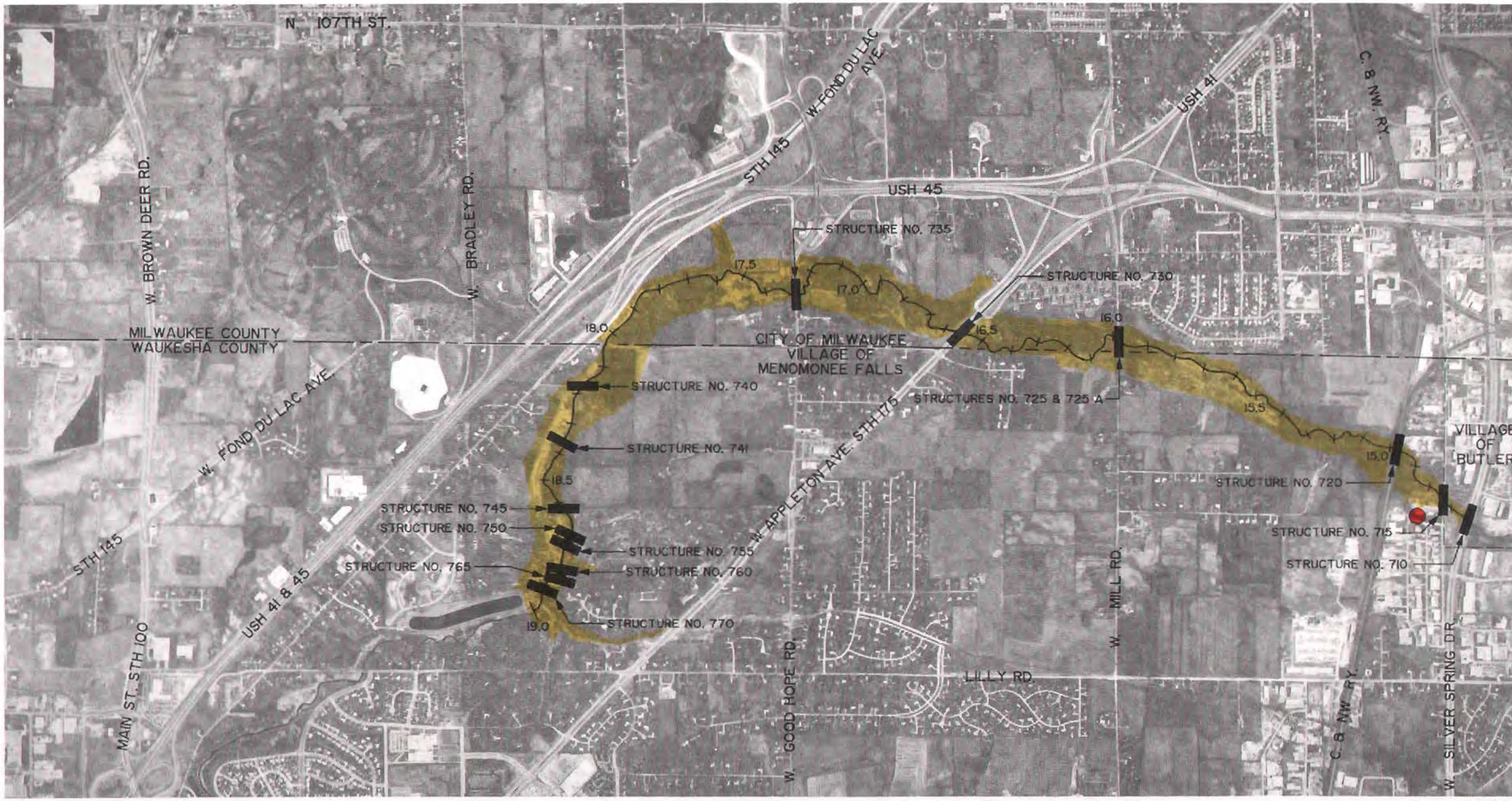
Assuming full implementation of these structure floodproofing measures, the capital cost of the refined recommended plan for the reach from Silver Spring Road through Lilly Road is estimated to be $74,000. Utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of the refined recommended plan is estimated to be $4,700. The average annual flood damage abatement benefit is estimated at $2,400, yielding a benefit-cost ratio of 0.51.

Channel widening and deepening is called for in a 0.94-mile-long reach from River Mile 22.02 just downstream of Roosevelt Drive to River Mile 22.96 at Erika Road extended. As shown on Map 161 and Figure 76, in that reach there are two existing storm sewer outfalls, the inverts of which are below the existing streambed, and there is one proposed outfall which would normally be submerged due to an adverse slope on the streambed which creates a ponded condition in the stream. There are also approximately 15 buildings in this reach which are located in the 100-year recurrence interval flood hazard area under planned land use and existing channel conditions. Those buildings would be removed from the flood hazard area following construction of the recommended channel modifications. Because the channel modifications are primarily designed to provide adequate storm sewer outlets and secondarily to abate flood damages, construction of a channel which would completely contain the 100-year recurrence interval flood under planned land use conditions with two feet of freeboard would be uneconomical; therefore, some overbank flooding into existing floodplain areas is permitted. It was also found that one upstream building at River Mile 23.48 in the Village of Germantown just north of the Washington-Waukesha County line would be removed from the 100-year flood hazard area and anticipated damages would be reduced at two other buildings, one at River Mile 24.19 and one at River Mile 24.33, due to reductions in the flood stage resulting from construction of the recommended channel modifications.

The channel modification recommended for the reach would accomplish the dual purpose of abating flood damages and providing adequate outlets for the existing and proposed storm sewers. As shown on Figure 76, it is recommended that the streambed be lowered a maximum of about four feet. As shown on Map 160, the modified channel section would have a 1.5-foot-deep, turf-lined low-flow channel with one vertical on two horizontal side slopes and a

Map 158

RECOMMENDED FLOOD CONTROL SYSTEM PLAN FOR THE MENOMONEE RIVER FROM W. SILVER SPRING DRIVE TO RIVER MILE 19.0



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

17.5 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

STRUCTURE TO BE FLOODPROOFED

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR MENOMONEE RIVER IS SHOWN IN APPENDIX H

GRAPHIC SCALE

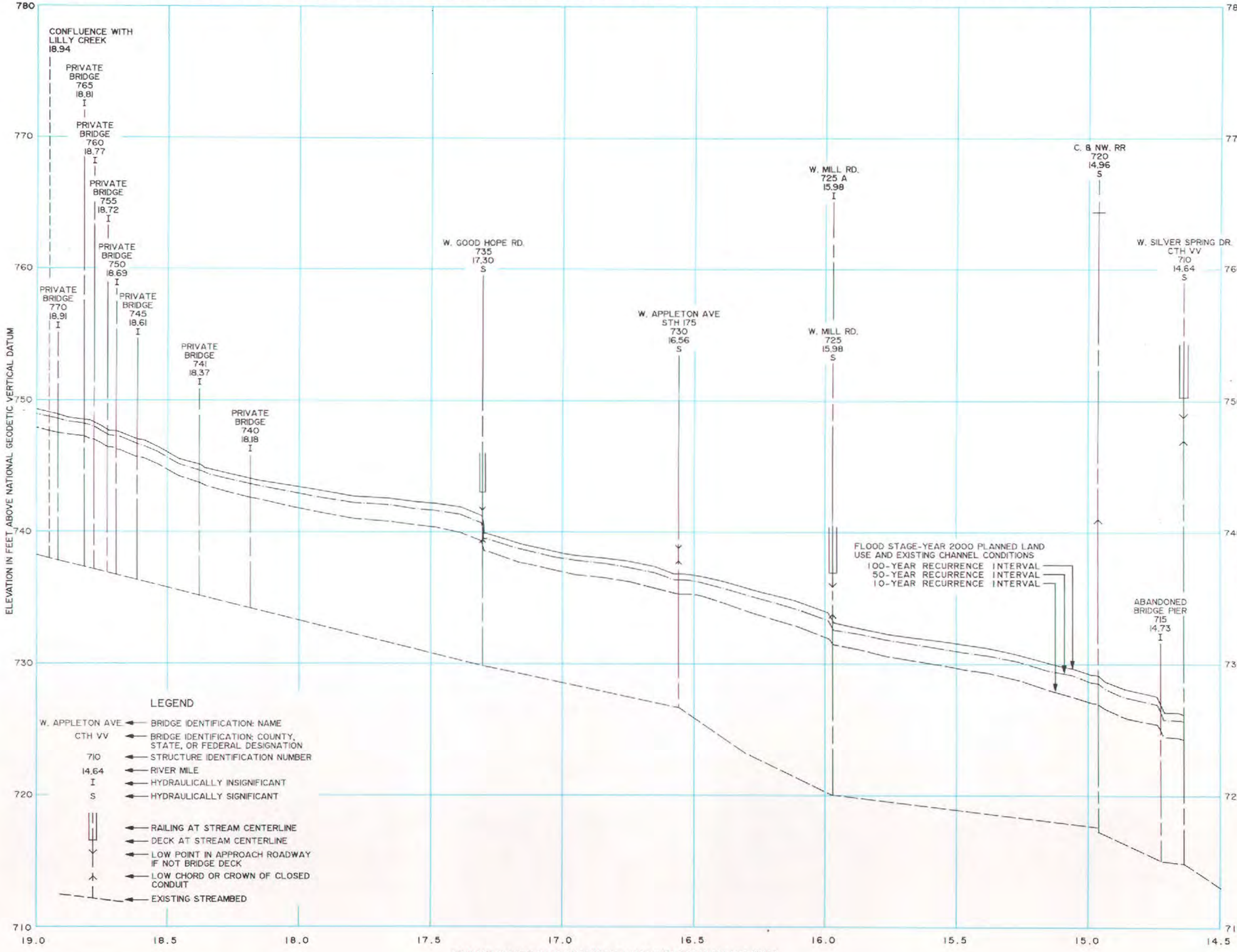
0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Source: SEWRPC.

Figure 74

RECOMMENDED PLAN FLOOD STAGE PROFILE FOR THE MENOMONEE RIVER FROM W. SILVER SPRING DRIVE TO RIVER MILE 19.0



Source: SEWRPC.

Map 159

RECOMMENDED FLOOD CONTROL SYSTEM PLAN FOR THE
MENOMONEE RIVER FROM RIVER MILE 19.0 TO STH 74 (MAIN STREET)

LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS

21.5 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

2 STRUCTURE TO BE FLOODPROOFED AND NUMBER OF STRUCTURES(IF MORE THAN ONE)

STRUCTURE TO BE ELEVATED

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR MENOMONEE RIVER IS SHOWN IN APPENDIX H

GRAPHIC SCALE

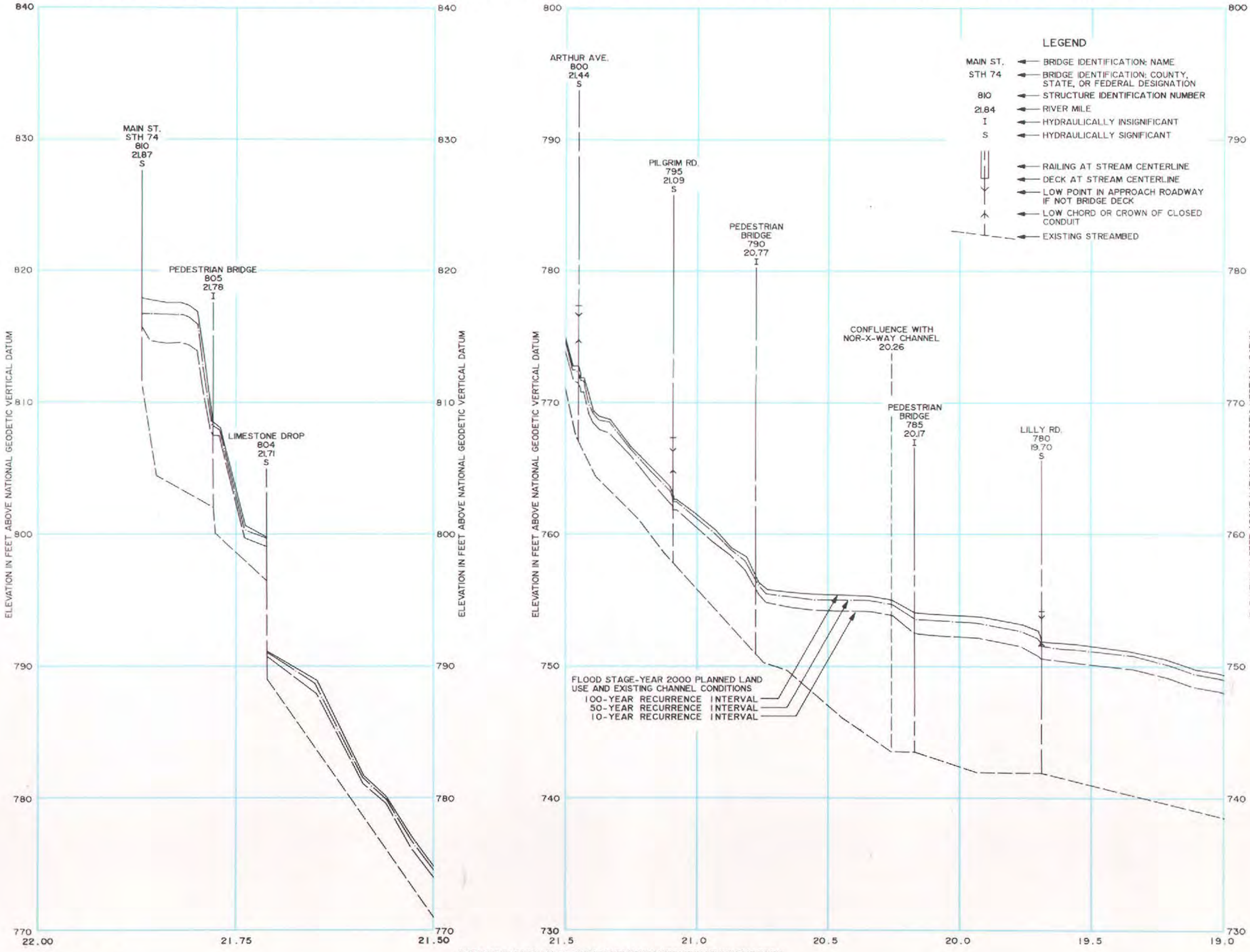
0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Source: SEWRPC.

Figure 75

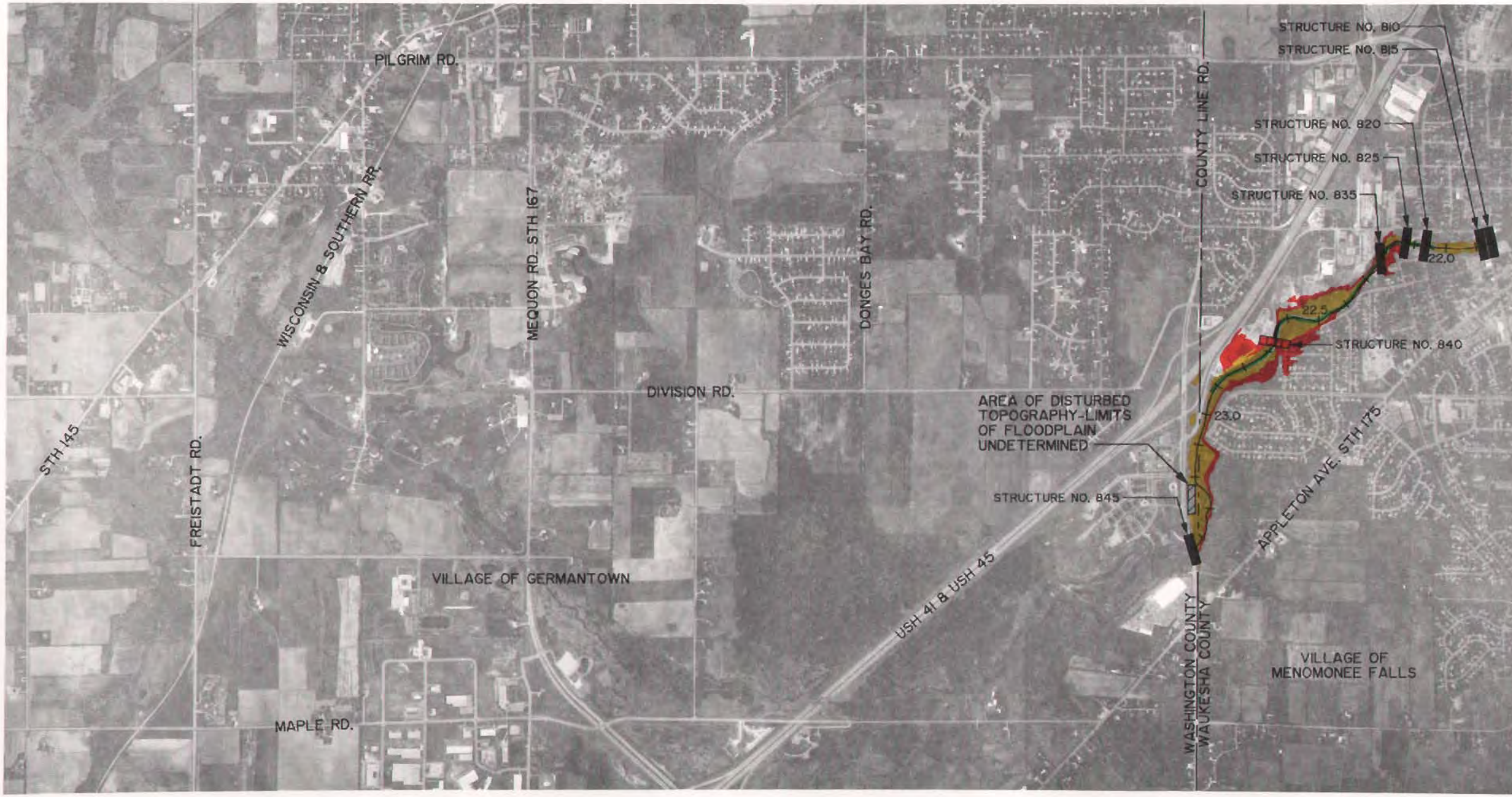
RECOMMENDED PLAN FLOOD STAGE PROFILE FOR THE MENOMONEE RIVER FROM RIVER MILE 19.0 TO STH 74 (MAIN STREET)



Source: SEWRPC.

Map 160

RECOMMENDED FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN FOR THE MENOMONEE RIVER FROM STH 74 (MAIN STREET) TO W. COUNTY LINE ROAD



LEGEND

- 100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS
- 100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS
- 22.5 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING
- SINGLE BANK CHANNEL MODIFICATION
- BRIDGE REPLACEMENT

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR MENOMONEE RIVER IS SHOWN IN APPENDIX H

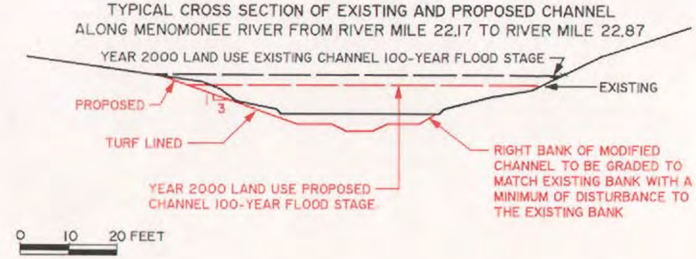
NOTE: DUE TO MAP SCALE LIMITATIONS, THE DIFFERENCE BETWEEN THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS, AND THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS, MAY NOT APPEAR ON THIS MAP. WHERE NO DIFFERENCE APPEARS REFERENCE SHOULD BE MADE TO THE FLOOD STAGE PROFILE SHOWN BELOW

GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

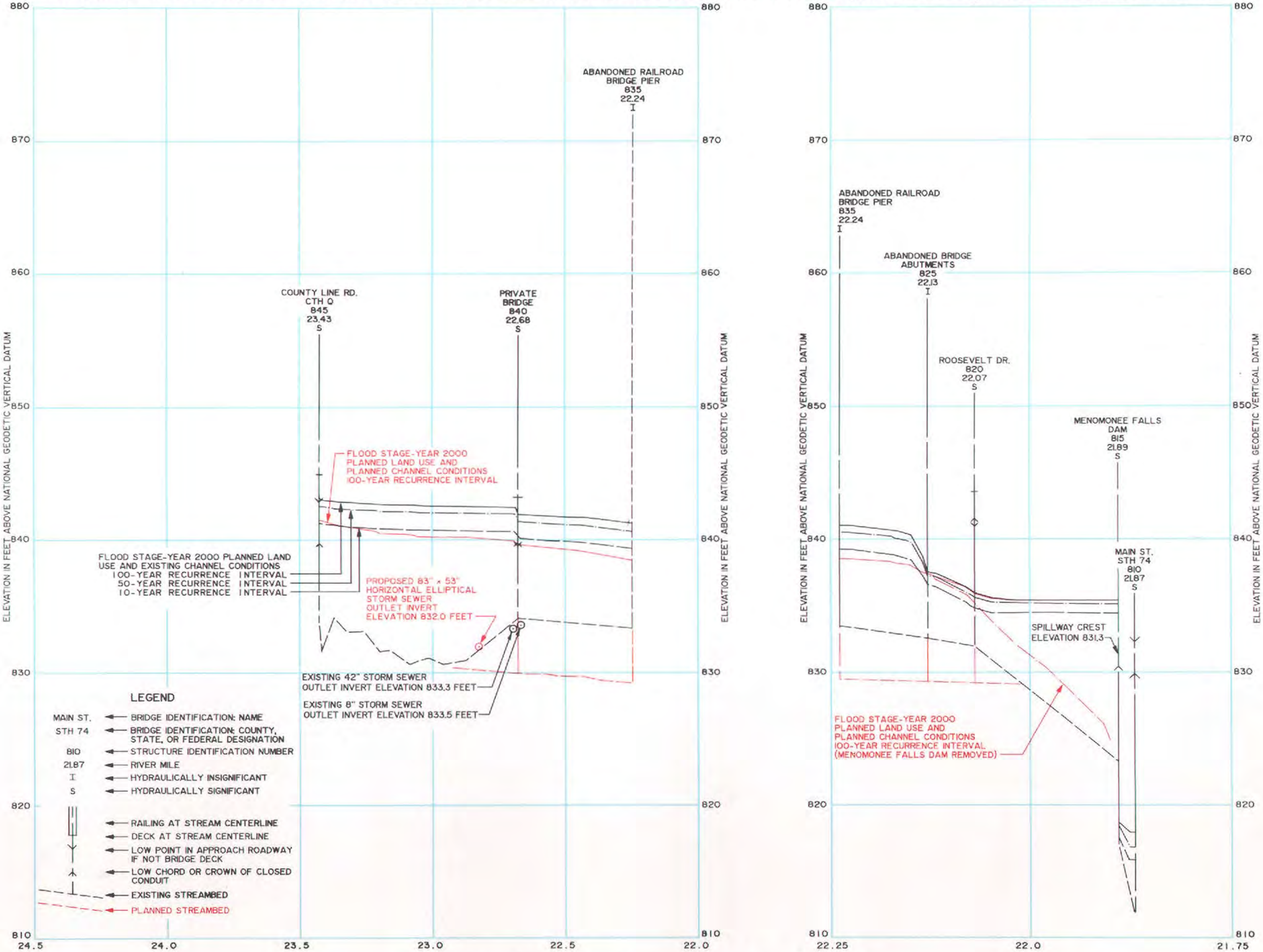
TYPICAL CROSS SECTION OF EXISTING AND PROPOSED CHANNEL ALONG MENOMONEE RIVER FROM RIVER MILE 22.17 TO RIVER MILE 22.87



Source: SEWRPC.

Figure 76

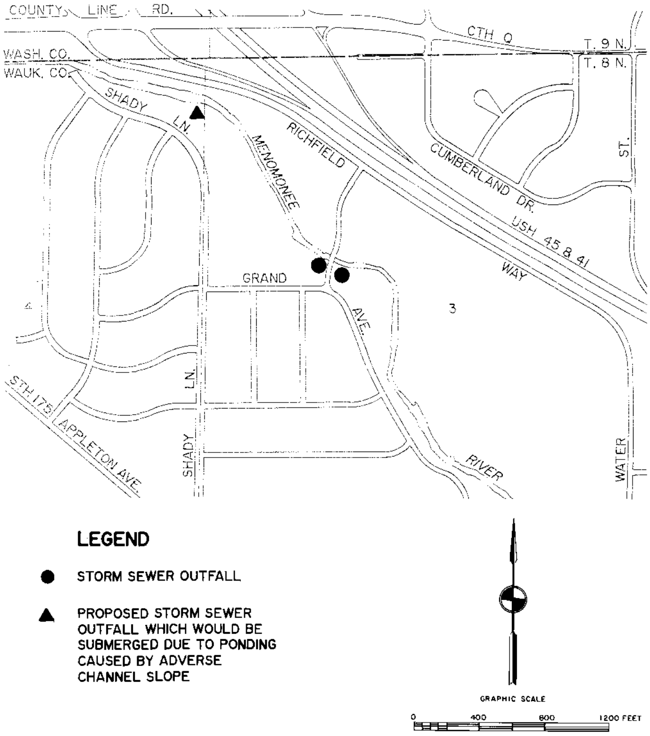
RECOMMENDED PLAN FLOOD STAGE PROFILE FOR THE MENOMONEE RIVER FROM STH 74 (MAIN STREET) TO W. COUNTY LINE ROAD



Source: SEWRPC.

Map 161

LOCATIONS OF STORM SEWER OUTFALLS ALONG THE MENOMONEE RIVER WITH INVERT ELEVATIONS BELOW EXISTING STREAMBED



Source: SEWRPC.

five-foot bottom width. The flood control channel would be constructed so as to disturb significantly only the left bank, viewed in the downstream direction. From River Mile 22.02 through the Roosevelt Drive bridge to River Mile 22.08, only minimal modification to the channel would be necessary to bring the bed to the desired grade. The Roosevelt Drive bridge would not require modification. Upstream from River Mile 22.08 through River Mile 22.37, the turf-lined flood control channel would have a 25-foot bottom width, the left bank slope would be one vertical on three horizontal, and the existing right bank would be retained, with some minor regrading required. Upstream from River Mile 22.37, the flood control channel would have the same characteristics, but the bottom width would be narrowed to 20 feet. According to information provided by the Village, there may be bedrock near the elevation of the existing streambed downstream from River Mile 22.44. Therefore, the modified channel in that reach may have to constructed at least partially in rock.

The existing private bridge providing access to the River Court Shopping Center would be replaced with a new structure spanning the modified channel and providing no significant impediment to the conveyance of flood flows.

The possibility of the removal of the Menomonee Falls dam at River Mile 21.90 has recently been considered by the Village in preliminary planning for the riverfront area in the vicinity of the dam. No decision on the whether or not to remove the dam has been made as yet; however, the possibility of removal was considered in analyzing the proposed channel modification. As shown on Figure 76, it was found that removal of the dam would reduce the 100-year recurrence interval flood stage under planned land use conditions by approximately 10.6 feet at the site of the dam. The stage reduction would be decreased to about 0.1 foot 1,500 feet upstream of the dam site. Because of the limited extent of the reduced stages, it is concluded that removal of the dam would have no significant impact on the recommended channel modifications.

Utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of the refined recommended plan for this reach is estimated at \$37,000. This cost consists of the amortization of the \$559,000 capital cost, \$384,000 for channel modification and \$175,000 for bridge removal and replacement, plus \$2,000 in annual operation and maintenance costs. The average annual flood damage abatement benefit is estimated at \$10,000, yielding a benefit-cost ratio of 0.27, including the benefits due to reduced flood stages at the three upstream buildings in the Village of Germantown.

Flood Control and Related Drainage System Plan Implementation: It is recommended that the refined recommended plan for the Menomonee River in the Village of Menomonee Falls be implemented through the cooperative efforts of the Village and private property owners. More specifically, it is recommended that the Village design, construct, and maintain the channel modifications and the bridge removal and replacement recommended in the reach from Roosevelt Drive through Erika Road extended.

The Village is currently outside the boundaries of the Metropolitan Sewerage District. If, at some future date prior to the construction of the proposed channel modification and bridge replacement, the District boundaries were expanded to include the area where those measures are proposed, it is recommended that the cost of channel modification and bridge removal and replacement be borne by the District.

The recommended structure floodproofing or elevation would be implemented by the individual property owners. It is recommended that these private owners bear the cost of structure floodproofing or elevation. It is further recommended that the professional services required to prepare plans for the floodproofing and elevation of individual buildings be made available to property owners, at no cost, by the Village of Menomonee Falls. Also, it is recommended that the Village of Menomonee Falls review its building ordinance to ensure that appropriate floodproofing regulations are included. It is recommended that the Village explore, on behalf of the property owners involved, any available state and/or federal aids for such floodproofing measures.

Menomonee River in the Village of Germantown: The watershed study did not identify any areas requiring flood control measures along the Menomonee River in the Village of Germantown. Under the system planning effort presented here, three buildings were identified in the 100-year recurrence interval floodplain under planned land use and existing channel conditions. The addition of these buildings to the 100-year floodplain under this study is attributable to 1) use of a somewhat higher 100-year flood discharges based on the simulation of flows for a 49-year period of record as opposed to a 35-year period for the watershed study; 2) refinement of the hydraulic model used to calculate water surface profiles; and 3) the availability of updated topographic mapping.

For the study presented here, it was decided to compare two alternative plans for alleviating the flood damage problem in this reach: 1) Alternative Plan 1—No Action and 2) Alternative Plan 2—Structure Floodproofing and Removal. Other, more costly, structural alternatives were eliminated from consideration because of the limited extent of the flood damages to be expected under planned conditions.

Each alternative system is described briefly below. The economic benefits and costs attendant to each alternative are provided in Table 103.

Alternative Plan 1—No Action: One alternative course of action with respect to the flood problem is to do nothing, that is, to recognize the inevitability of flooding but to deliberately decide not to mount a collective, coordinated program to abate the flood damages. Under planned year 2000 land use and existing channel conditions, the average annual flood damages would approximate $1,400. The damages from a 100-year recurrence interval flood may be expected to approximate $23,700. There are no monetary benefits associated with this alternative. The average annual cost would be equivalent to the average of the annual flood damage costs under planned land use conditions, or about $1,400. If the recommended channel modifications in the Village of Menomonee Falls were constructed, one of the three buildings would be removed from the floodplain in Germantown. Under those conditions the average annual flood damages would approximate $200 and the damages from a 100-year flood would approximate $20,300.

Alternative Plan 2—Structure Floodproofing, Elevation, and Removal: A structure floodproofing and elevation flood control system was analyzed to determine if such a structure-by-structure approach would be a technically feasible and economically viable solution to the flood problem. The 100-year recurrence interval flood stage under planned year 2000 land use and planned channel conditions was used to estimate the number of existing flood-prone structures to be floodproofed and the approximate costs involved.

In the case of residential structures, floodproofing was assumed to be feasible if the design flood stage was below the first floor elevation. Structure elevation was considered feasible for residential structures with basements if the estimated cost of elevating the structure and floodproofing the basement was less than the estimated removal cost. Structures to be elevated were assumed to have the first floor raised to an elevation of at least two feet above the 100-year recurrence interval flood stage to provide adequate freeboard. For aesthetic reasons, structure elevation was limited to a maximum of four feet. Structures which would have to be elevated more than four feet were considered for removal.

If the recommended channel modifications are implemented in Menomonee Falls, one structure in Germantown at River Mile 24.19 would

Table 103

COST ESTIMATES FOR FLOOD CONTROL ALTERNATIVES
FOR THE MENOMONEE RIVER IN THE VILLAGE OF GERMANTOWN

		Costs (1986 dollars)					Benefit-Cost Analysis			
			Annual							
Alternative	Description	Capital	Amortized Capital[a]	Operation and Maintenance	Other	Total	Annual Benefits	Annual Benefits Minus Annual Costs	Benefit-Cost Ratio	Economic Ratio Greater than One
1A. No Action	--	$ 0	$ 0	$0	$0	$ 0	$ 0	$ 0	--	No
2A. Structure Floodproofing, Elevation, and Removal[b]	Floodproof one structure	5,000	2,200	0	0	2,200	200	-2,000	0.09	No
	Elevate one structure	30,000								
	Subtotal	$35,000								
1B. No Action[c]	--	$ 0	$ 0	$0	$0	$ 0	$ 0	$ 0	--	No
2B. Structure Floodproofing, Elevation, and Removal[c]	Floodproof one structure	5,000	4,300	0	0	4,300	1,400	-2,900	0.33	No
	Elevate two structures	63,000								
	Subtotal	$68,000								

[a]*Amortized capital cost is based on an interest rate of 6 percent and a project life of 50 years.*

[b]*Assuming recommended channel modifications are implemented in the Village of Menomonee Falls.*

[c]*Assuming recommended channel modifications are not implemented in the Village of Menomonee Falls.*

Source: SEWRPC.

require floodproofing and one at River Mile 24.33 would require elevation, as shown on Map 162. Future damage from floods up to and including the 100-year flood would be virtually eliminated.

Assuming that these structure floodproofing measures would be fully implemented, and utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of this alternative is estimated at $2,000. This cost consists of the amortization of the $35,000 capital cost, $5,000 for floodproofing and $30,000 for structure elevation. The average annual flood damage abatement is estimated at $200, yielding a benefit-cost ratio of 0.09.

If the recommended channel modifications are not implemented in Menomonee Falls, one structure in Germantown at River Mile 23.48 would require floodproofing, and one at River Mile 24.19 and one at River Mile 24.33 would require elevation. Assuming that these structure floodproofing measures would be fully implemented, and utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of this alternative is estimated at $4,300. This cost consists of the amortization of the $67,200 capital cost, $4,600 for floodproofing and $62,600 for structure elevation. The average annual flood damage abatement is estimated at $1,400, yielding a benefit-cost ratio of 0.33.

Evaluation of Alternatives: The principal features of, and the costs and benefits associated with, each of the floodland management alternatives considered for this reach of the Menomonee River are summarized in Table 103. Each of the alternatives described above was found to be technically feasible. Although it offers the lowest cost, the "no action" alternative does nothing to alleviate the existing flood problem and does not represent a sound approach to flood control.

Complete implementation of a voluntary structure floodproofing and elevation program is unlikely; however, because only two to three structures are affected, this alternative might be implemented with some success. Partial implementation would leave the Village of Germantown with a residual problem whenever a major flood occurs. Also, yard damages and cleanup costs would remain under this alternative.

It is recommended that Alternative Plan 2—Structure Floodproofing, Elevation, and Removal—be implemented. Full implementation of this plan would serve to eliminate structural flood damages in this reach for floods up to and including the 100-year recurrence interval flood under planned land use and channel conditions. The recommended plan is shown graphically on Map 162. The peak flood profile attendant to planned land use and channel conditions is shown on Figure 77.

Flood Control and Related Drainage System Plan Implementation: The recommended flood control plan would be implemented by the individual property owners within the 100-year floodplain. It is recommended that these private owners bear the cost of structure floodproofing or removal. It is further recommended that the professional services required to prepare plans for the floodproofing and elevation of individual buildings be made available to property owners, at no cost, by the Village of Germantown engineering department. Also, it is recommended that the Village review its building ordinance to ensure that appropriate floodproofing regulations are included. It is recommended that the Village explore, on behalf of the property owners involved, any available state and/or federal aids for such floodproofing measures.

WOODS CREEK SUBWATERSHED FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN

Woods Creek was not studied under any previous Commission planning programs. Analyses of the hydrologic and hydraulic characteristics of the stream and of the tributary subwatershed were accordingly conducted under this system planning effort.

Overview of the Subwatershed

The Woods Creek subwatershed is located entirely in Milwaukee County. The subwatershed includes portions of the Cities of Milwaukee and West Allis and the Village of West Milwaukee. Woods Creek begins on the west side of the Veterans Administration Center, in West Allis, at a storm sewer outfall, and flows in a generally easterly and northeasterly direction for about 1.1 miles to its confluence with the Menomonee River. The Woods Creek subwatershed drains an area of about 2.13 square miles, as shown on Map 163. The extent of the subwatershed area within each minor civil division involved is given in Table 104.

More specifically, Woods Creek originates near the intersection of W. Walker Street and S. 56th Street at a storm sewer outfall in the City of West Allis, whence it flows easterly for about 0.2 mile as an open channel; continues easterly in an enclosure for about 0.3 mile; thence, flows southeasterly in an open channel for about 0.2 mile; thence northerly through an enclosure for about 0.1 mile; thence continues northerly in an open channel for about 0.1 mile; thence flows easterly through another enclosure for about 0.2 mile to its confluence with the Menomonee River. The entire 1.1-mile reach described is classified as perennial. All of Woods Creek is recommended for District jurisdiction in the policy plan companion to this system plan.

In 1985, the entire Woods Creek subwatershed was developed for urban use. The developed areas of the subwatershed are generally provided with a full range of municipal street improvements, including paved streets with curbs and gutters and attendant storm sewers.

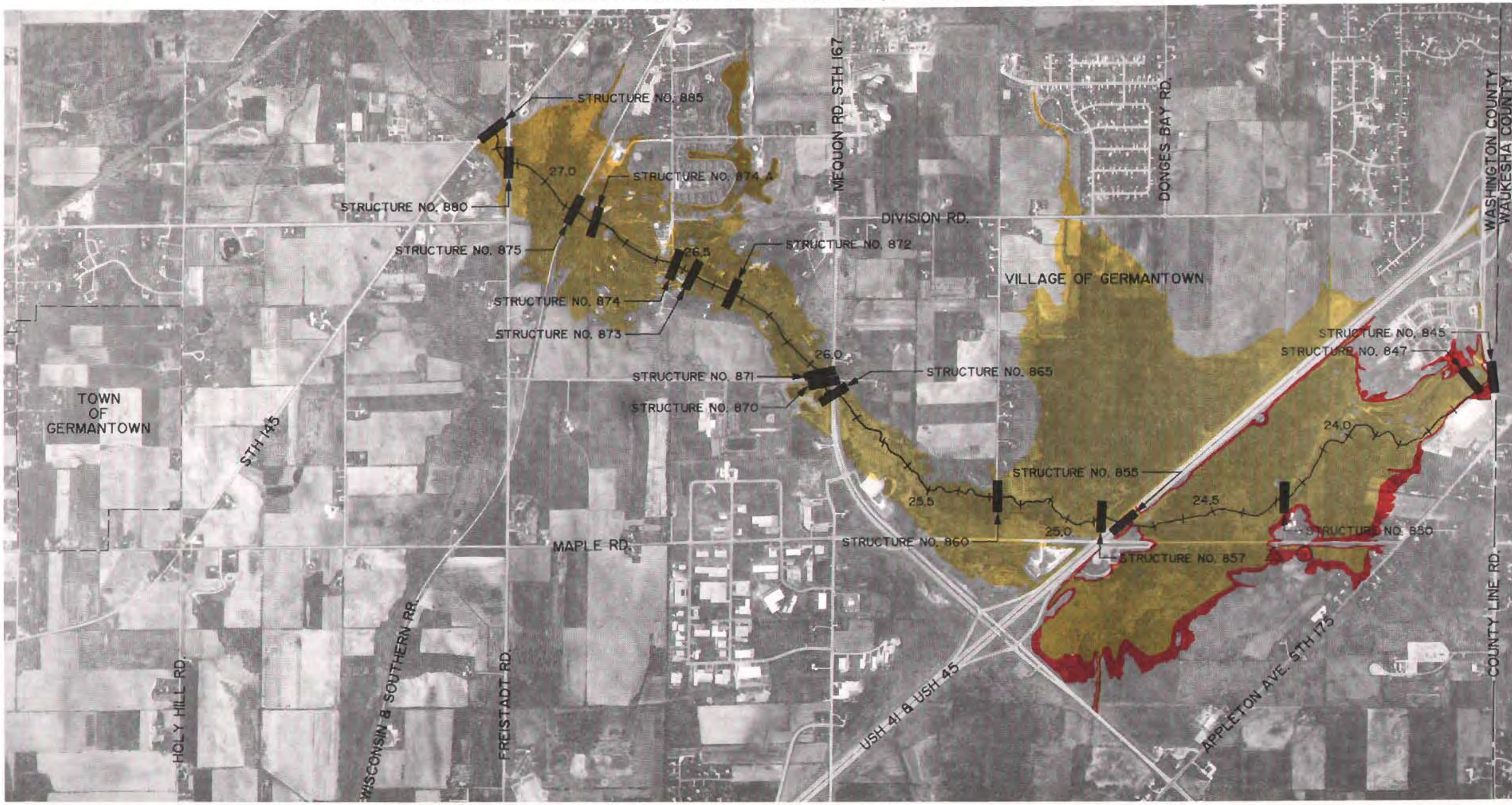
The flood profile for Woods Creek is shown in Figure 78. The extent of the 100-year recurrence interval flood hazard area under planned land use and existing channel conditions is shown on Map 164.

Evaluation of Alternative Flood Control and Related Drainage System Plans for Woods Creek

No structural flood damages are expected to be incurred due to flooding along Woods Creek for floods up to and including a 100-year recurrence interval event. Flooding of portions of the County Stadium parking lot and the Stadium Freeway (USH 41) may be expected due to the inadequate hydraulic capacity of a series of culverts which currently convey Woods Creek from the Soo Line (former Chicago, Milwaukee, St. Paul & Pacific Railroad) railway at River Mile 0.27 to the Menomonee River. This lack of capacity results in surcharging of storm sewers both within the parking lot and along the freeway, as well as causing flooding from direct

Map 162

RECOMMENDED FLOOD CONTROL SYSTEM PLAN FOR THE MENOMONEE RIVER FROM W. COUNTY LINE ROAD TO STH 145 (W. FOND DU LAC AVENUE)



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS

25.5 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

STRUCTURE TO BE FLOODPROOFED

STRUCTURE TO BE ELEVATED

BUILDING OUTSIDE FLOODPLAIN ASSUMING IMPLEMENTATION OF RECOMMENDED CHANNEL MODIFICATION IN VILLAGE OF MENOMONEE FALLS

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR MENOMONEE RIVER IS SHOWN IN APPENDIX H

DUE TO MAP SCALE LIMITATIONS, THE DIFFERENCE BETWEEN THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS, AND THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS, MAY NOT APPEAR ON THIS MAP. WHERE NO DIFFERENCE APPEARS REFERENCE SHOULD BE MADE TO THE FLOOD STAGE PROFILE SHOWN BELOW

GRAPHIC SCALE

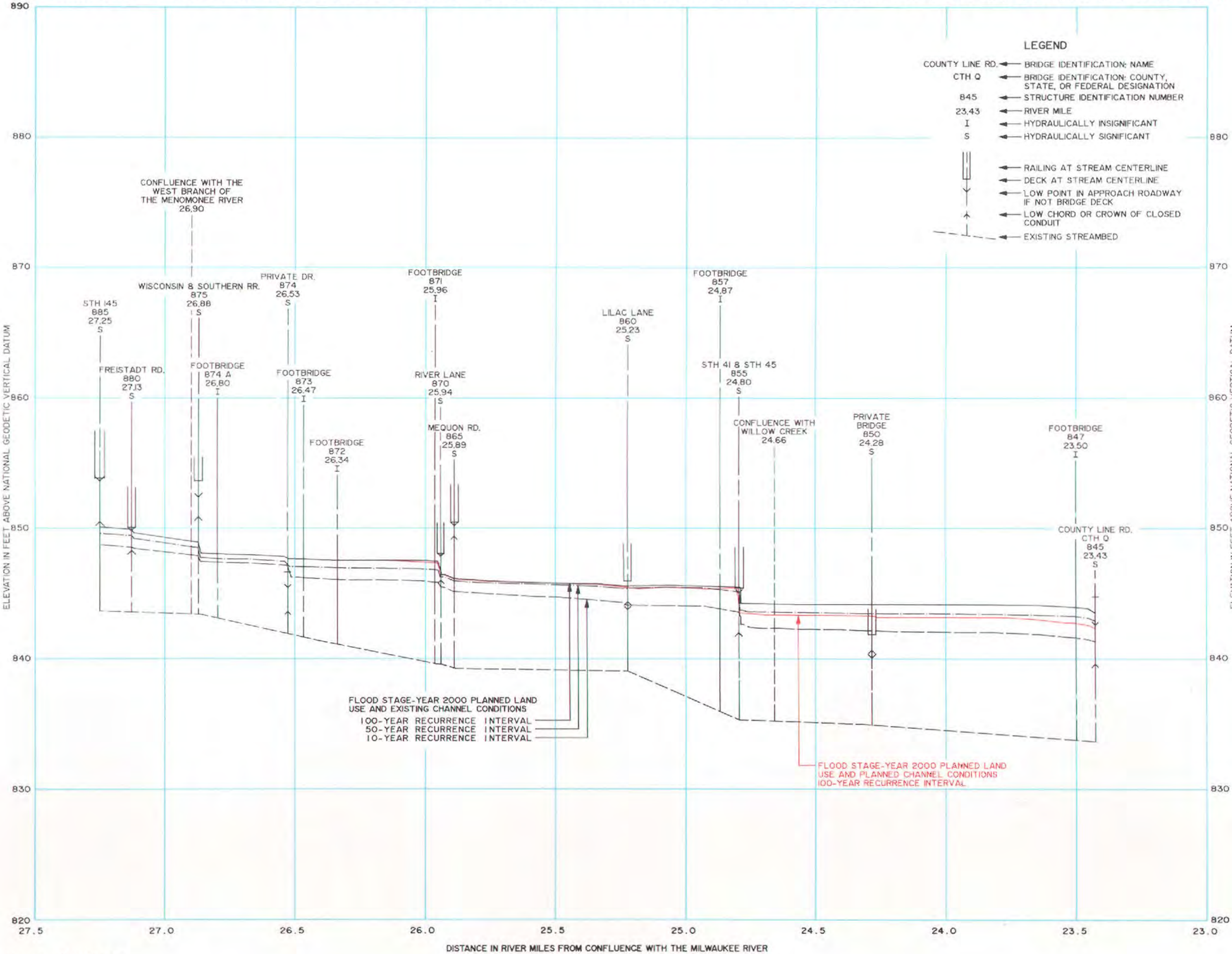
0 1/2 1 MILE

DATE OF PHOTOGRAPHY APRIL 1986

Source: SEWRPC.

Figure 77

RECOMMENDED PLAN FLOOD STAGE PROFILE FOR THE MENOMONEE RIVER FROM W. COUNTY LINE ROAD TO STH 145 (W. FOND DU LAC AVENUE)



Source: SEWRPC.

Map 163

THE WOODS CREEK SUBWATERSHED

Source: SEWRPC.

overland flow. This flooding does not include County Stadium. Flooding of the Stadium which occurred during the August 6, 1986, storm event was due to inability of the existing stadium drainage facilities to handle the unusually large amount of runoff associated with that storm, and was not due to direct flooding from Woods Creek. The total rainfall from that storm was in excess of a 100-year recurrence interval event.

Since efforts to alleviate flooding of the parking lot and freeway could impact on flood flows and stages on the Menomonee River, an evaluation of alternative flood control plans for Woods Creek was made as part of this system plan. A total of three alternative flood control plans were considered: 1) Alternative Plan 1—No Action; 2) Alternative Plan 2—Construction of Relief Culvert; and 3) Alternative Plan 3—Combination of Storm Sewer Diversion and Construction of Relief Culvert. Each alternative is described below.

Alternative Plan 1—No Action: One alternative course of action with respect to the flood problem along Woods Creek is to do nothing, that is, to recognize the inevitability of flooding but to decide not to mount a collective, coordinated program to abate the problem. There are no costs associated with this alternative as well as no benefits.

Table 104

AREAL EXTENT OF CIVIL DIVISIONS IN THE WOODS CREEK SUBWATERSHED

Civil Division	Civil Division Area Included Within Subwatershed (square miles)	Percent of Subwatershed Area Within Civil Division
City of Milwaukee	1.20	56.3
City of West Allis	0.48	22.5
Village of West Milwaukee	0.45	21.2
Total	2.13	100.0

Source: SEWRPC.

Alternative Plan 2—Construction of Relief Culvert: This alternative is shown on Map 165 and would consist of installing about 1,500 feet of 10-foot-wide by 5-foot-high reinforced concrete box culvert. This culvert would begin at the Soo Line railway crossing and would run parallel to the existing system of culverts to the Menomonee River. Installation of this culvert would eliminate flooding of the Stadium parking lot and of the Stadium Freeway (USH 41) due to the surcharging of Woods Creek, for floods up to and including a 100-year recurrence interval event.

The cost of this relief culvert is estimated at about $981,000. Utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of this alternative is estimated to be about $62,000. No benefit-cost ratio was calculated since the flooding problems do not include direct overland flood damage, but rather are problems which cause inconvenience and secondary-type damages; the corresponding benefits for their alleviation are not typically accounted for in benefit-cost ratio calculations.

Alternative Plan 3—Combination of Storm Sewer Diversion and Relief Culvert: This alternative is shown on Map 166 and consists of diverting directly to the Menomonee River a 66-inch-diameter storm sewer which currently discharges to Woods Creek at a point about 300 feet upstream of the Soo Line railway crossing. This sewer conveys storm runoff from a portion of the Village of West Milwaukee. The proposed diversion sewer would run north for a distance of about 1,400 feet along Harnischfeger Avenue and S. 44th Street to the existing Woods Creek culverts, and thence about 300 feet east to the Menomonee River.

In addition to the storm sewer diversion, this alternative would require the installation of a six-foot-diameter reinforced concrete relief culvert. This culvert would begin at the Soo Line railway crossing and would run parallel to the existing Woods Creek culvert system to the Menomonee River, a distance of about 1,500 feet.

Implementation of this alternative would eliminate flooding of both the Stadium parking lot and the Stadium Freeway due to surcharging from Woods Creek for floods up to and including the 100-year recurrence interval event.

The cost of this storm sewer diversion and relief culvert is estimated at about $1,110,000. Utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of this alternative is estimated to be about $70,000. As noted above, no benefit-cost ratio was calculated for this alternative since the damages are of an inconvenient and secondary nature.

Channel Enclosure as Proposed by the U. S. Veterans Administration: From its origin near W. Walker Street and S. 56th Street to the Soo Line railway, Woods Creek runs through property owned by the U. S. Veterans Administration. In 1978, the Veterans Administration prepared plans for the enclosure and partial relocation of the stream in that reach. As set forth in the overview of the subwatershed, a portion of the channel enclosure, but none of the relocation, has been constructed. The hydrologic and hydraulic analyses conducted for this system plan identified no need to provide further channel enclosure for the purpose of flood damage abatement. Therefore, that alternative was eliminated from further consideration.

Evaluation of Alternatives: The two action alternatives considered were found to be technically feasible. Those alternatives are similar in concept and would both solve the existing flood problem for a similar cost. Also, neither of these two alternatives would have a significant impact on peak flood discharges along the Menomonee River. This is due to the fact that the flow from Woods Creek is relatively small compared to that on the Menomonee River (1,160 cfs versus 16,400 cfs) and also due to the fact that the peak discharge from Woods Creek occurs sooner than that for the Menomonee River.

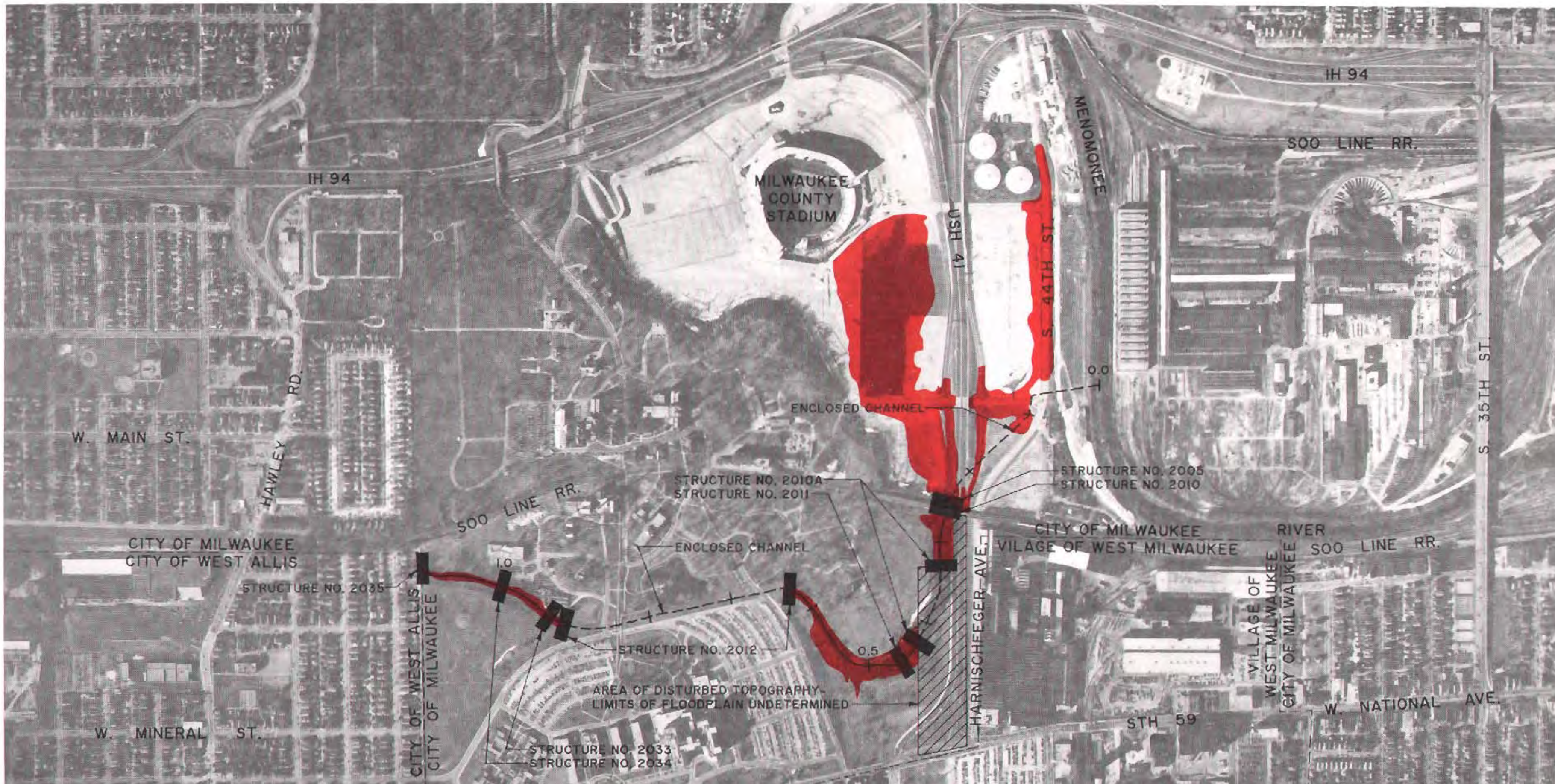
Since it offers a solution to the flood problems along Woods Creek at the lowest capital cost, it is recommended that Alternative Plan 2—Construction of Relief Culvert—be adopted as part of this system plan. The recommended plan is shown graphically on Map 167. The peak flood profile attendant to planned land use and channel conditions is shown on Figure 79.

Flood Control and Related Drainage System Plan Implementation

The recommended flood control plan for Woods Creek consists solely of the construction of a relief culvert downstream of the Soo Line railway

Map 164

100-YEAR RECURRENCE INTERVAL FLOODPLAIN FOR WOODS CREEK UNDER YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

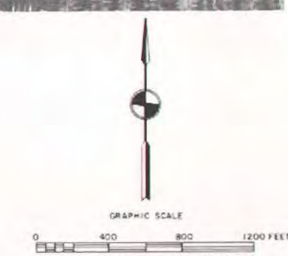


LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

0.5 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

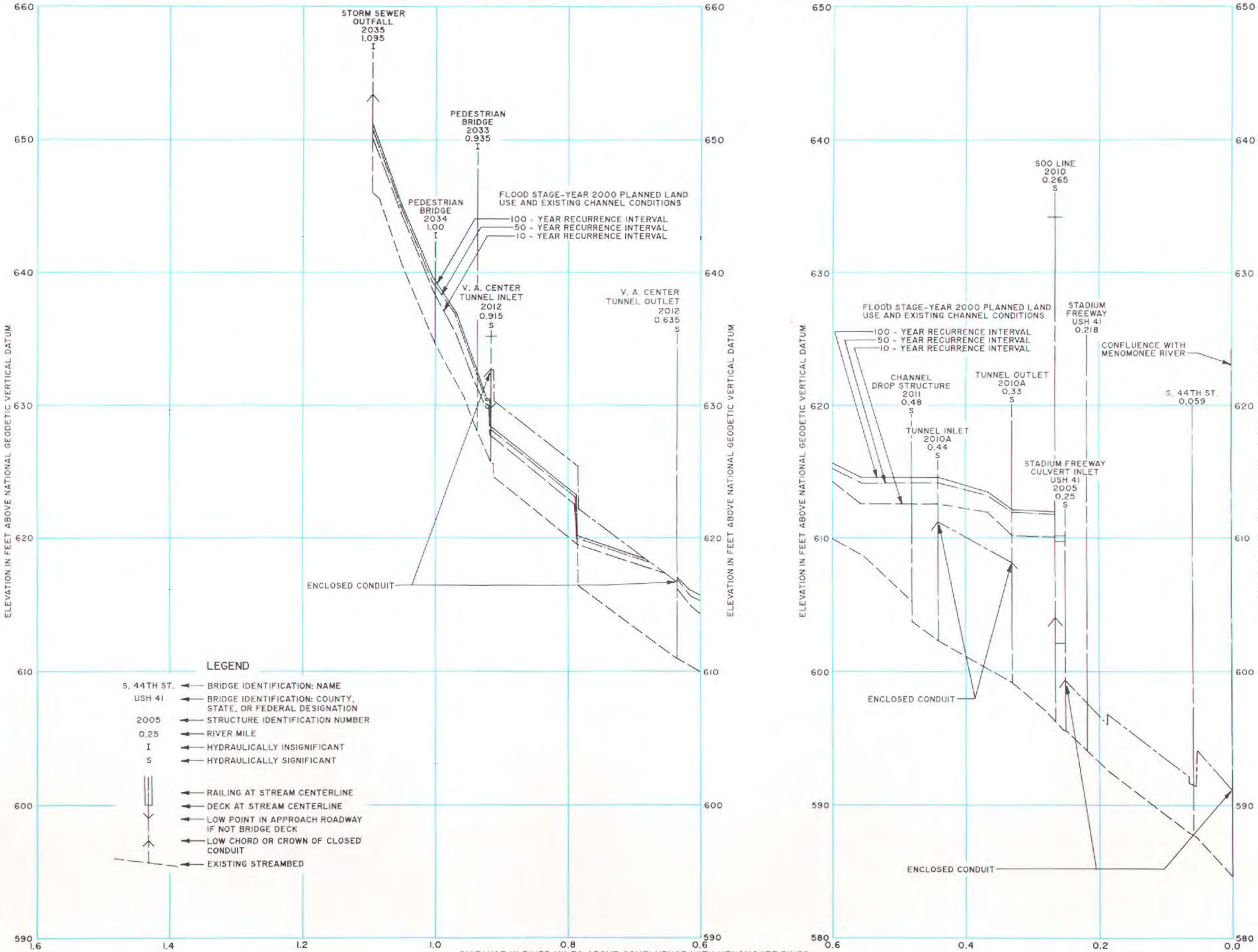
NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR WOODS CREEK IS SHOWN IN APPENDIX H



Source: SEWRPC.

Figure 78

FLOOD STAGE AND STREAMBED PROFILE FOR WOODS CREEK



Source: SEWRPC.

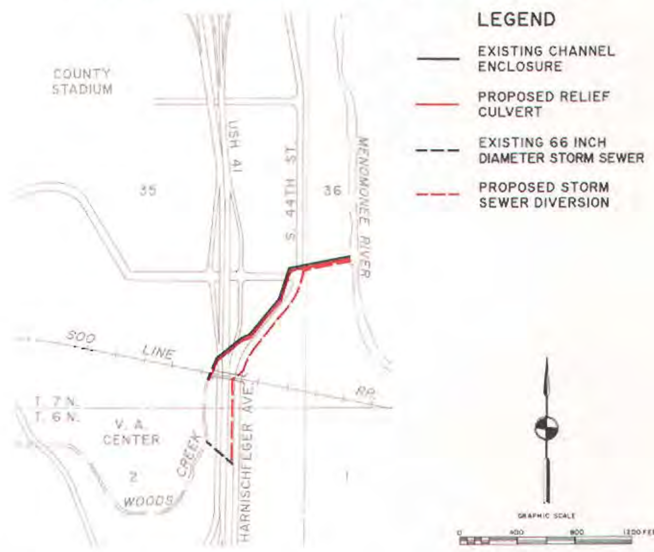
Map 165

ALTERNATIVE PLAN 2: CONSTRUCTION OF RELIEF CULVERT ALONG WOODS CREEK

Source: SEWRPC.

Map 166

ALTERNATIVE PLAN 3: COMBINATION OF STORM SEWER DIVERSION AND RELIEF CULVERT ALONG WOODS CREEK



Source: SEWRPC.

crossing. It is recommended that this culvert be designed, constructed, and maintained by the Milwaukee Metropolitan Sewerage District.

HONEY CREEK SUBWATERSHED FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN

Hydrologic and hydraulic analyses of the Honey Creek subwatershed were previously conducted under the Commission's Menomonee River watershed study. That study also assessed existing and possible future flood problems along the stream from its mouth to IH 894 at River Mile 7.53, evaluated alternative measures to alleviate those problems, and included a recommendation for the implementation of certain flood control measures. This system planning effort represents a refinement of that earlier study and an expansion of that study to include consideration of the 1.30-mile-long reach from IH 894 to the S. 43rd Street storm sewer outfall. Presented below are an overview of the subwatershed, a review of the previously considered flood control measures, and a refined and expanded recommended flood control plan for Honey Creek.

Overview of the Subwatershed

The Honey Creek subwatershed is located entirely in south central Milwaukee County. The subwatershed includes portions of the Cities of Greenfield, Milwaukee, Wauwatosa, and West Allis. From its origin at the S. 43rd Street storm sewer outfall in the City of Greenfield, Honey Creek flows generally northerly for a distance of about 8.8 miles to its confluence with the Menomonee River. Honey Creek drains an area of about 10.78 square miles, as shown on Map 168. The extent of the subwatershed area within each minor civil division involved is given in Table 105.

More specifically, from its origin at a storm sewer outfall at S. 43rd Street, Honey Creek flows northwesterly for about 1.0 mile to W. Layton Avenue; thence, northerly for about 0.3 mile to IH 894; thence, northwesterly for about 3.2 miles to W. Arthur Avenue in the City of West Allis; thence, northerly in an enclosure for about 2.3 miles to IH 94; continues northerly in an open channel for about 1.5 miles to Portland Avenue in the City of Wauwatosa; and thence easterly for about 0.5 mile to its confluence with the Menomonee River. The entire 8.8-mile reach described is classified as a perennial stream and is recommended for District jurisdiction in the policy plan companion to this system plan.

In 1985, about 98 percent of the Honey Creek subwatershed was developed for urban use. About 64 percent of the developed area was in residential use. Other uses included institutional, recreational, and commercial. The developed areas of the subwatershed are generally provided with a full range of municipal street improvements, including paved streets with curbs and gutters and attendant storm sewers. The planned land use conditions utilized in the system planning effort assume that the subwatershed will be entirely urbanized by the design year of the system plan.

The flood profile for Honey Creek is shown as Figure 80. The extent of the 100-year recurrence interval flood hazard area under planned land use and existing channel conditions is shown on Map 169.

<u>Evaluation of Alternative Flood Control and Related Drainage System Plans for Honey Creek</u>
The alternative flood control measures considered under the watershed study are presented below, together with their estimated costs. Based upon an evaluation of those alternatives, a final flood control plan was recommended in the watershed plan. That plan has been further refined and expanded as part of this system planning effort. Some reaches are not expected to incur any structural damages due to overland flooding during the 100-year recurrence interval flood under planned land use and channel conditions. Those reaches are: 1) mouth of Honey Creek to River Mile 0.84 and 2) River Mile 0.86 through IH 894 at River Mile 7.55. Together, these reaches total 7.53 miles in length, or 85 percent of the total stream length under District jurisdiction.

<u>City of Wauwatosa from the Mouth of Honey Creek through W. Wisconsin Avenue</u>: The watershed study identified a total of 13 structures in the secondary flooding zone in this reach.

Under the watershed study, a total of three flood control alternatives were analyzed for this reach. These alternatives include the following: 1) No Action; 2) Floodproofing and Removal of Structures; and 3) Bridge or Culvert Alteration or Replacement. The estimated cost of each of these alternatives as well as the attendant benefit-cost ratio, is presented in Table 106.

The watershed study concluded that removal or modification of bridges producing backwater in excess of 1.0 foot under 100-year planned flood conditions would reduce local flood stages; however, because monetary flood risks in the reach are low, bridge replacement would not be an economically feasible solution to the flooding problem.

Under the structure floodproofing and removal alternative, the watershed study called for all 13 structures in hazard to be floodproofed. That alternative was recommended because it had a benefit-cost ratio greater than one, and because the bridge alteration or replacement alternative was not economically feasible.

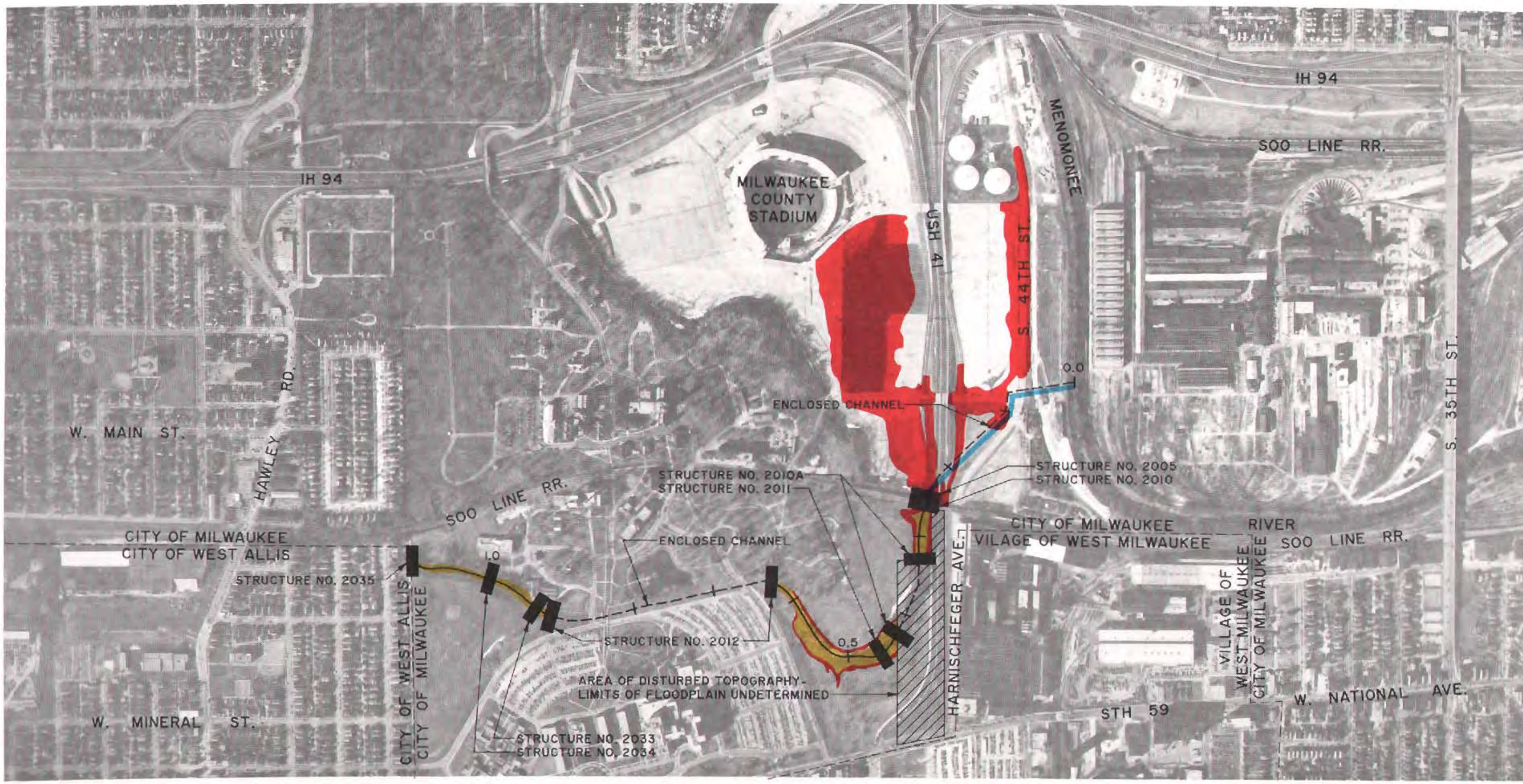
<u>Refined Flood Control System Plan</u>: The flood control plan developed as part of this system planning effort represents a refinement of that proposed under the watershed study. The refined analysis of this reach did not include consideration of secondary flooding for reasons already noted. The refined analysis concluded that only one structure in the reach may be expected to experience overland flooding during a 100-year recurrence interval flood under planned land use and channel conditions.

A structure floodproofing and elevation flood control system was determined to be a technically feasible solution to the flood problem. The 100-year recurrence interval flood stage under planned year 2000 land use and planned channel conditions was used to estimate the number of existing flood-prone structures to be floodproofed and the approximate costs involved. In the case of residential structures, floodproofing was assumed to be feasible if the design flood stage was below the first floor elevation, which is the case for the single structure located in the floodplain in this reach.

As shown on Map 170, the refined flood control plan for this reach of Honey Creek calls for floodproofing of one structure, a church building, along the west bank of the stream at St. Jude Court. The peak flood profile attendant to planned land use and channel conditions is shown on Figure 81. Full implementation of this plan would serve to eliminate structural damages due to overland flooding in this reach for

Map 167

RECOMMENDED FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN FOR WOODS CREEK



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS

PROPOSED RELIEF CULVERT

0.5 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

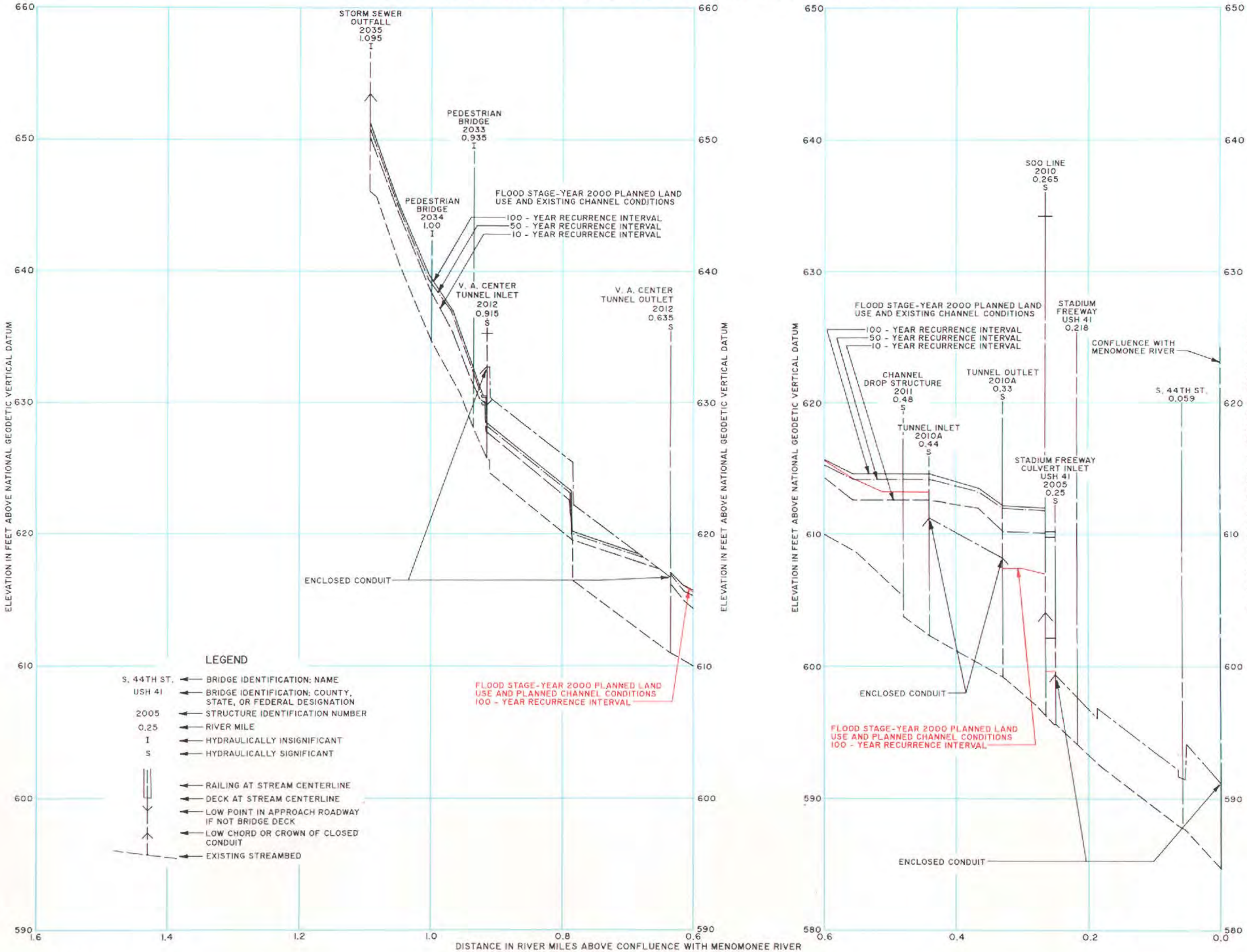
NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR WOODS CREEK IS SHOWN IN APPENDIX H.

DUE TO MAP SCALE LIMITATIONS, THE DIFFERENCE BETWEEN THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS, AND THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS, MAY NOT APPEAR ON THIS MAP. WHERE NO DIFFERENCE APPEARS REFERENCE SHOULD BE MADE TO THE FLOOD STAGE PROFILE SHOWN BELOW

Source: SEWRPC.

Figure 79

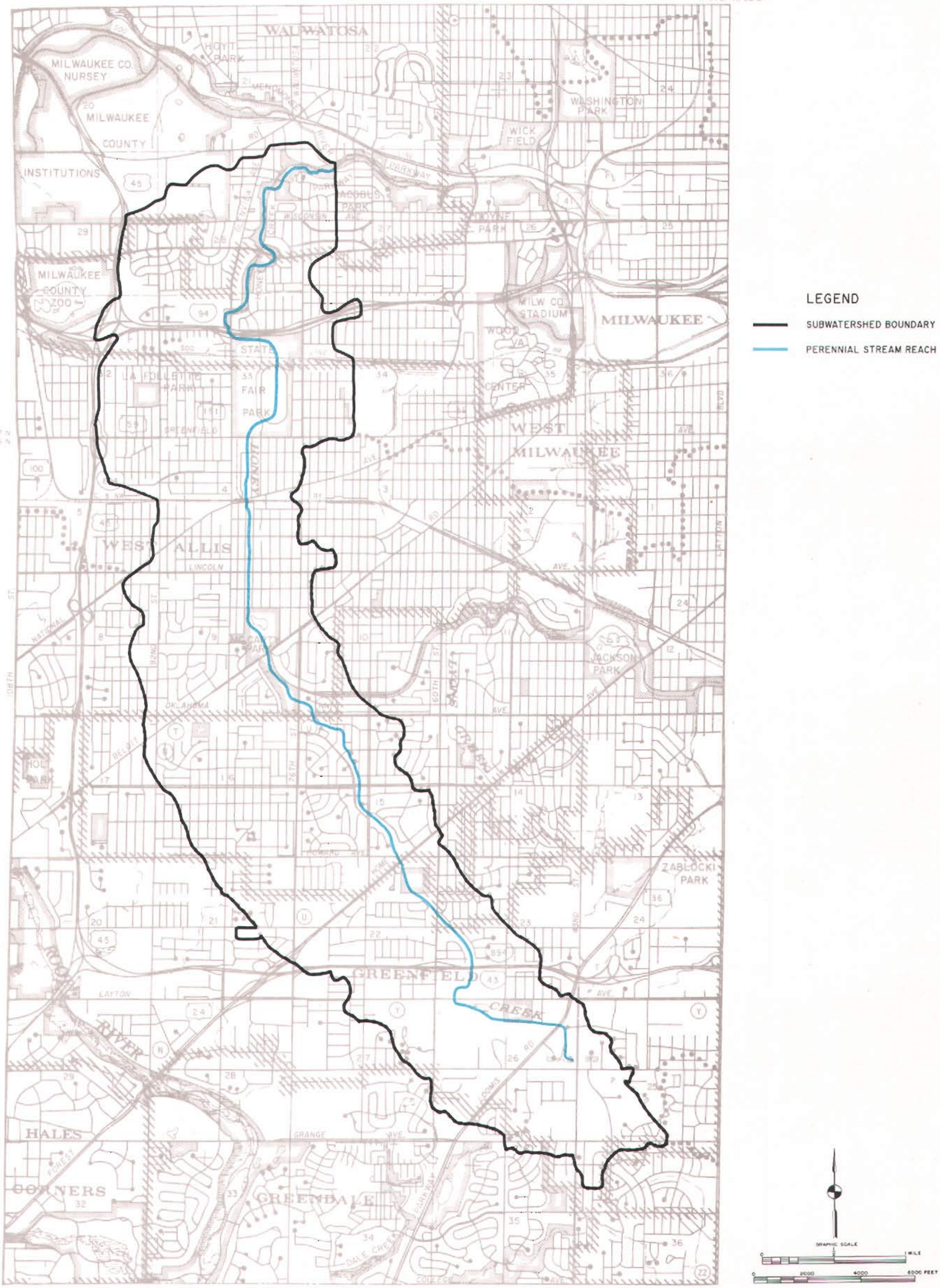
RECOMMENDED PLAN FLOOD STAGE PROFILE FOR WOODS CREEK



Source: SEWRPC.

Map 168

THE HONEY CREEK SUBWATERSHED



Source: SEWRPC.

Table 105

AREAL EXTENT OF CIVIL DIVISIONS IN THE HONEY CREEK SUBWATERSHED

Civil Division	Civil Division Area Included Within Subwatershed (square miles)	Percent of Subwatershed Area Within Civil Division
City of Greenfield	2.87	26.6
City of Milwaukee	3.46	32.1
City of Wauwatosa	0.95	8.8
City of West Allis	3.50	32.5
Total	10.78	100.0

Source: SEWRPC.

Table 106

MENOMONEE RIVER WATERSHED STUDY FLOOD CONTROL ALTERNATIVES FOR HONEY CREEK BETWEEN THE MENOMONEE RIVER AND W. WISCONSIN AVENUE

Alternative	Cost[a]					Benefit-Cost Ratio
	Capital	Amortized Capital[b]	Operation and Maintenance	Other	Annual Total	
1. No Action	$ - -	$ - -	$- -	$1,400	$1,400	0
2. Structure Floodproofing	6,600	400	0	0	400	3.50
3. Bridge Alteration or Replacement	- -[c]	- -[c]	- -[c]	- -[c]	- -[c]	- -

[a]*Costs are expressed in 1986 dollars.*

[b]*Amortized capital cost is based on an interest rate of 6 percent and a project life of 50 years.*

[c]*No costs were computed since this alternative was found to be technically infeasible.*

Source: SEWRPC.

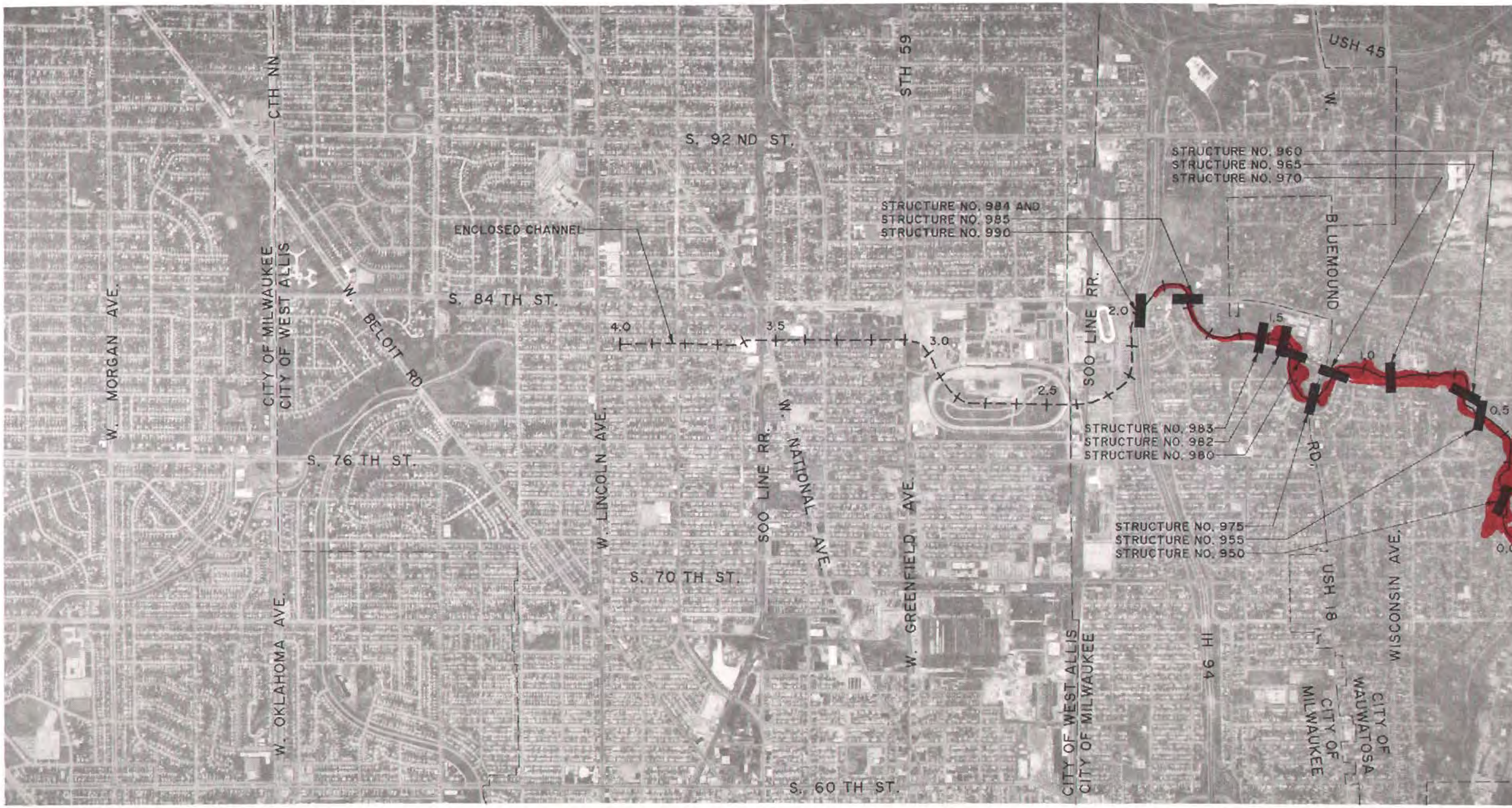
floods up to and including the 100-year recurrence interval flood under planned land use and channel conditions.

Assuming that the structure floodproofing measures would be fully implemented, the capital cost of the refined recommended plan for this reach is estimated to be about $50,000. Utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of the refined recommended plan is estimated to be $3,200. The average annual flood damage abatement benefit is estimated at $2,200, yielding a benefit-cost ratio of 0.7.

As already noted in the section describing the recommended flood control plan for the main stem of the Menomonee River, channel modifications along Honey Creek from its mouth to the Honey Creek Parkway bridge at River Mile 0.17 would be required in order for the Honey Creek streambed grade to match the lowered Menomonee River streambed grade. The cost of those

Map 169

100-YEAR RECURRENCE INTERVAL FLOODPLAIN FOR HONEY CREEK UNDER YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

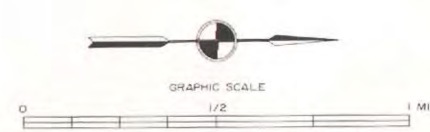


LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

4.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

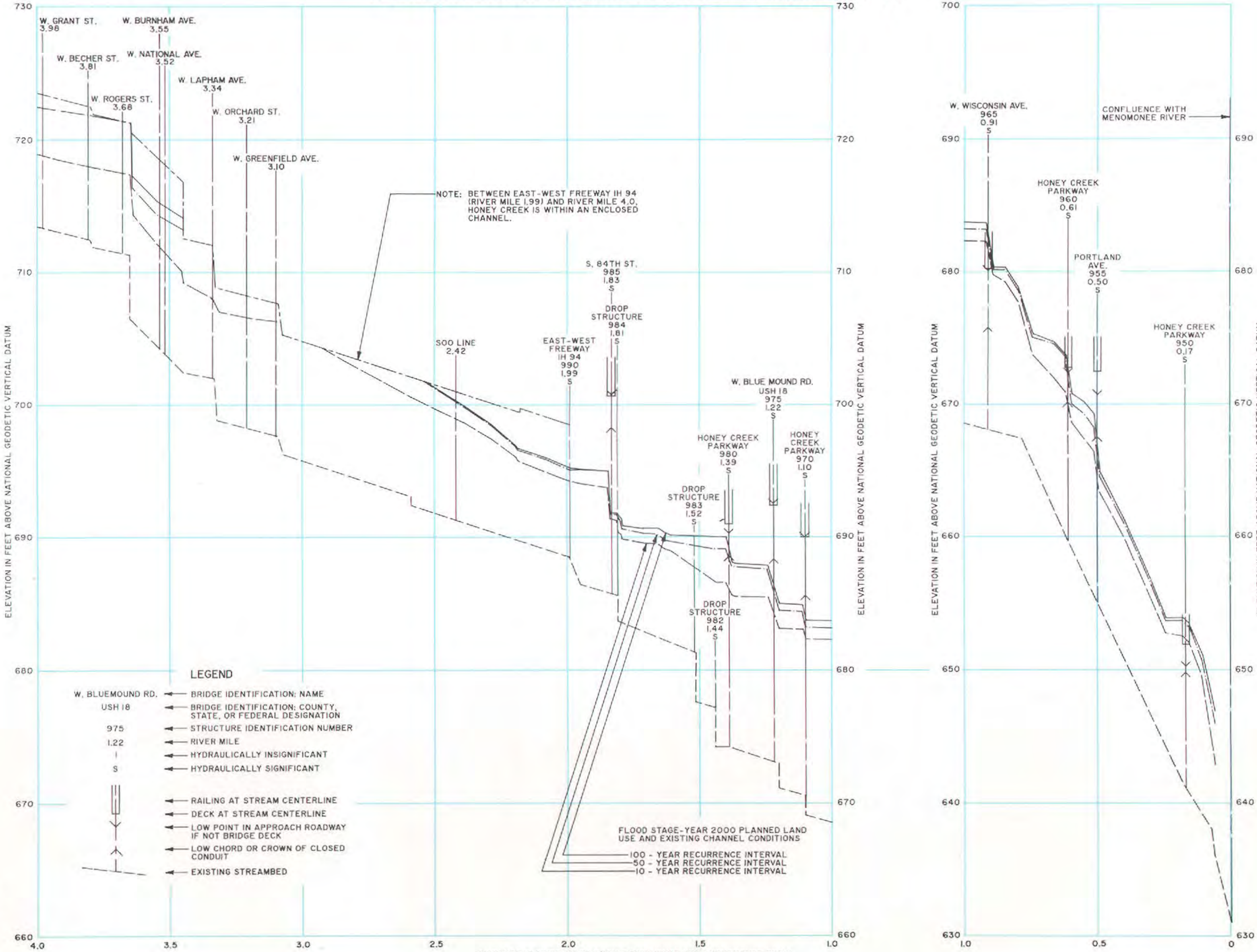
NOTE: THE AVAILABILITY OF LARGE-SCALE TOPGRAPHIC MAPPING FOR HONEY CREEK IS SHOWN IN APPENDIX H



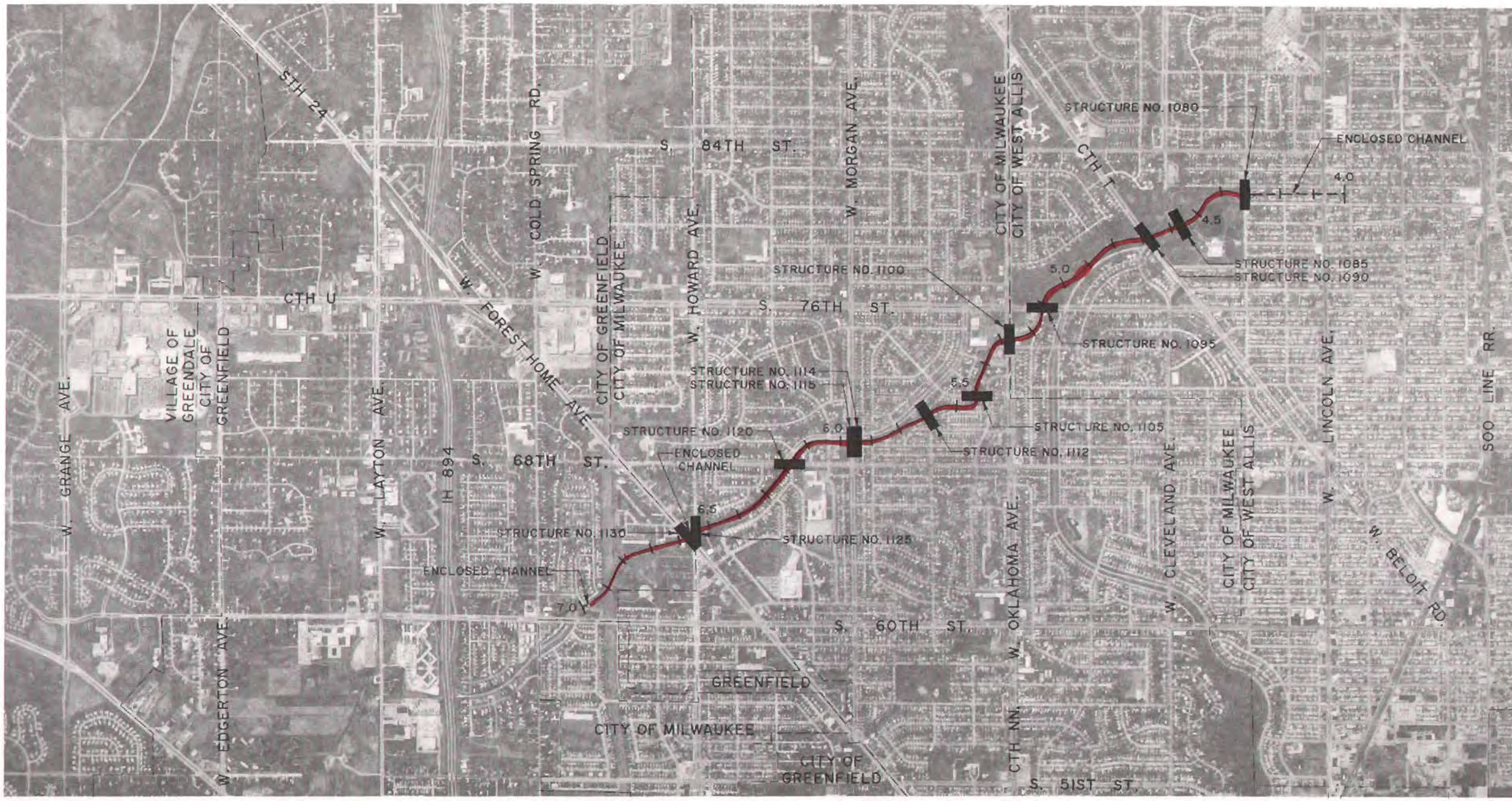
DATE OF PHOTOGRAPHY: APRIL 1986

Figure 80

FLOOD STAGE AND STREAMBED PROFILE FOR HONEY CREEK



Map 169 (continued)



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

7.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

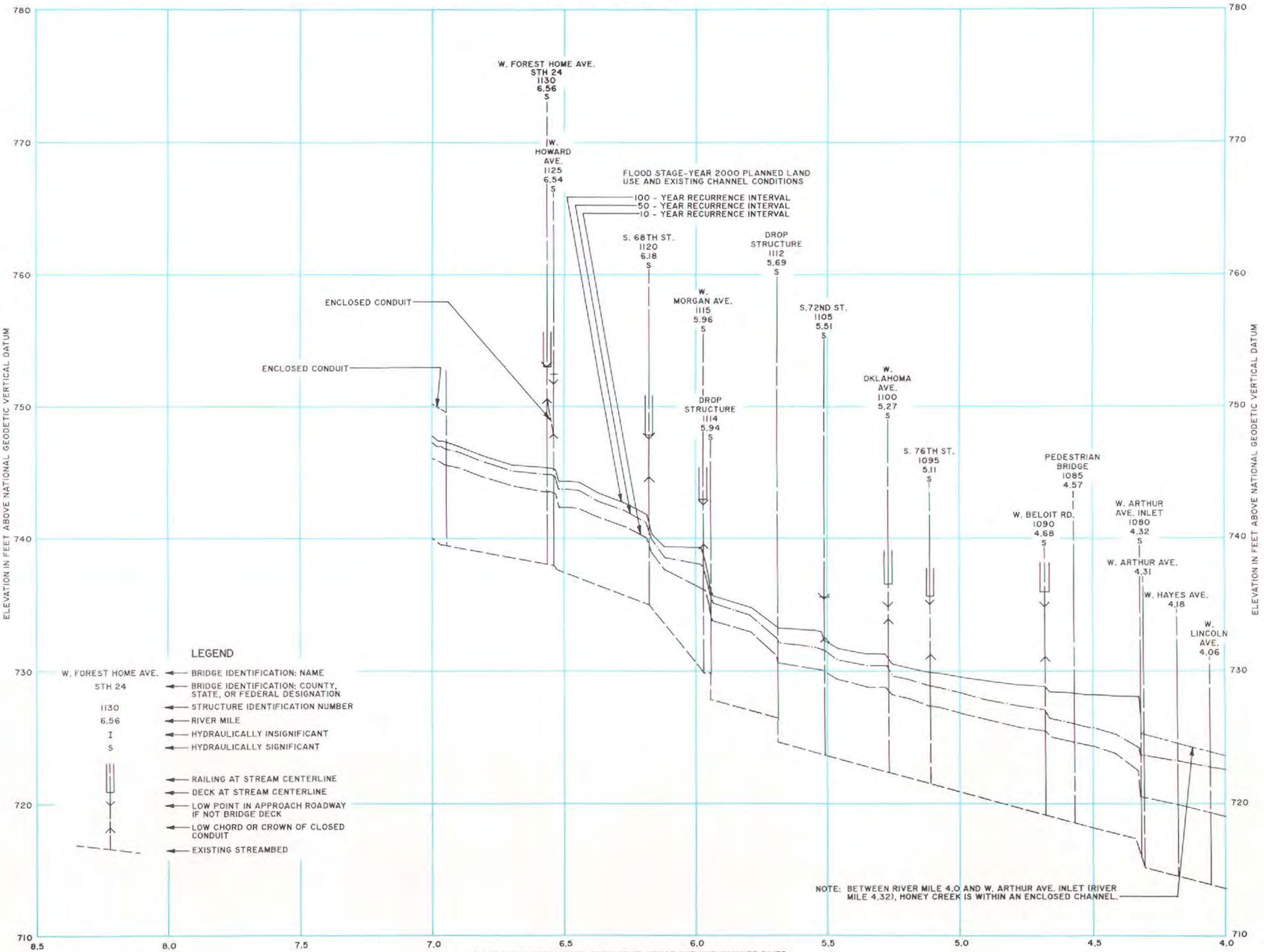
NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR HONEY CREEK IS SHOWN IN APPENDIX H

GRAPHIC SCALE

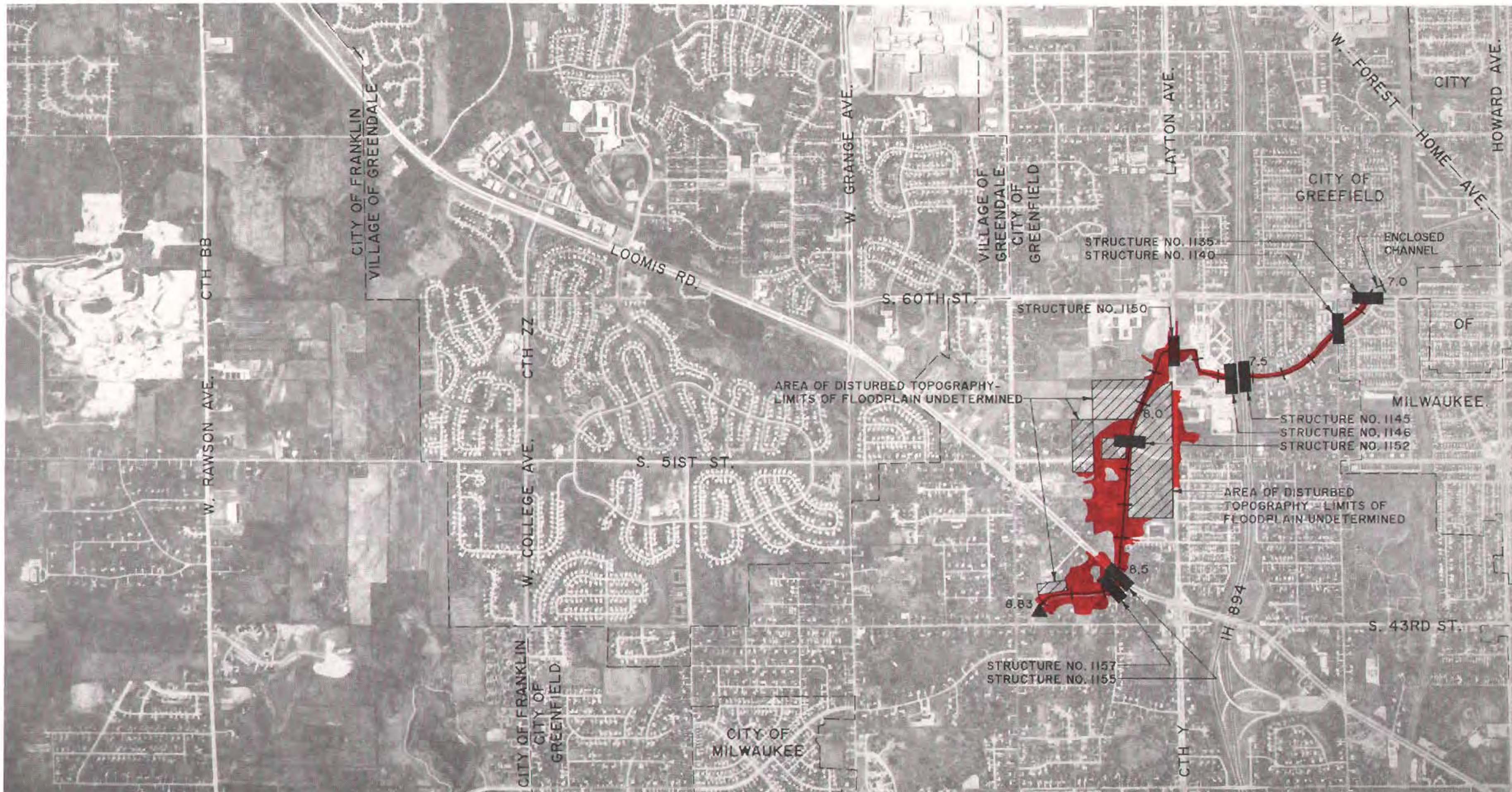
0 1/2 1 MILE

DATE OF PHOTOGRAPHY APRIL 1986

Figure 80 (continued)



Map 169 (continued)



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

EXISTING STORM SEWER OUTFALL

8.5 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR HONEY CREEK IS SHOWN IN APPENDIX H

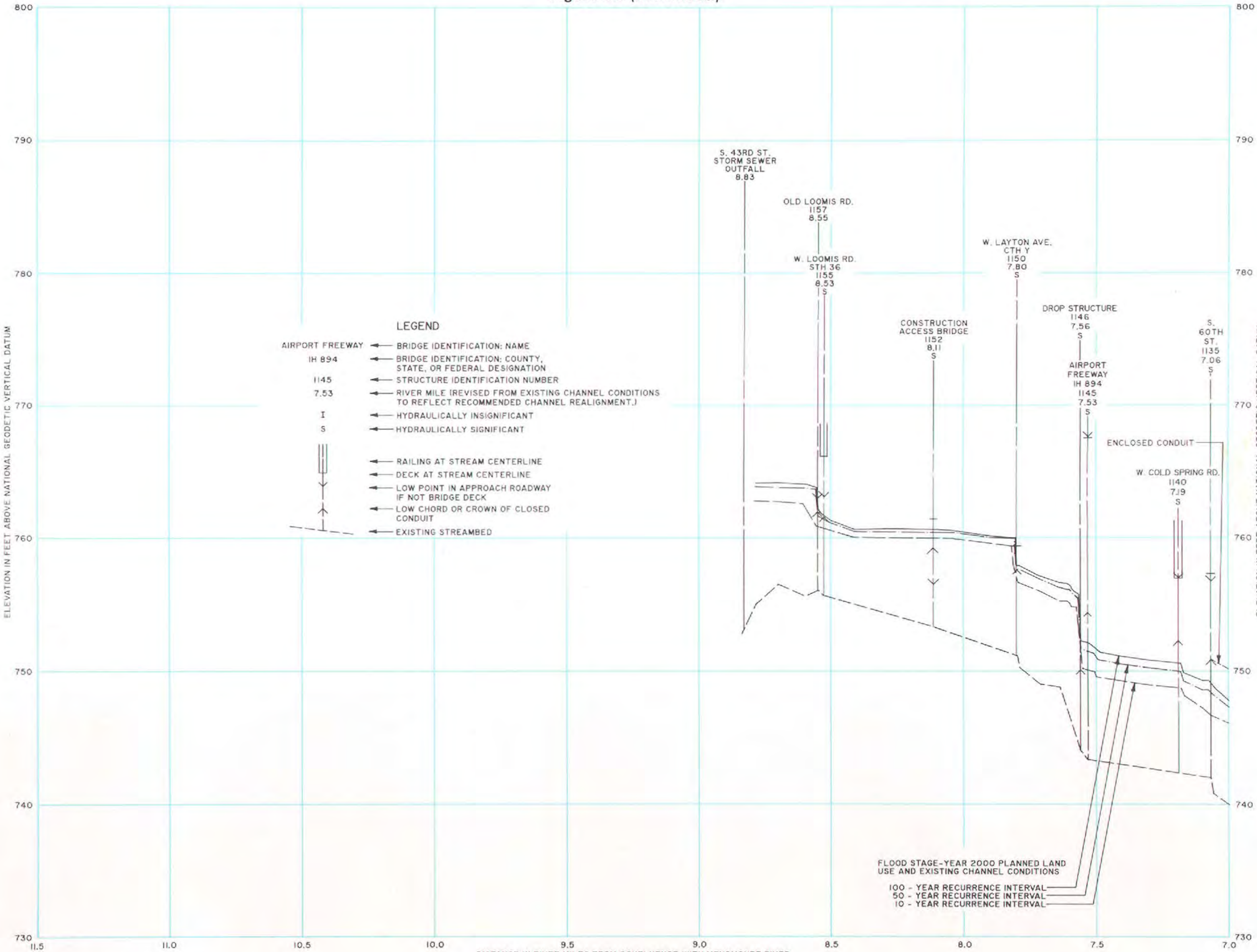
GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Source: SEWRPC.

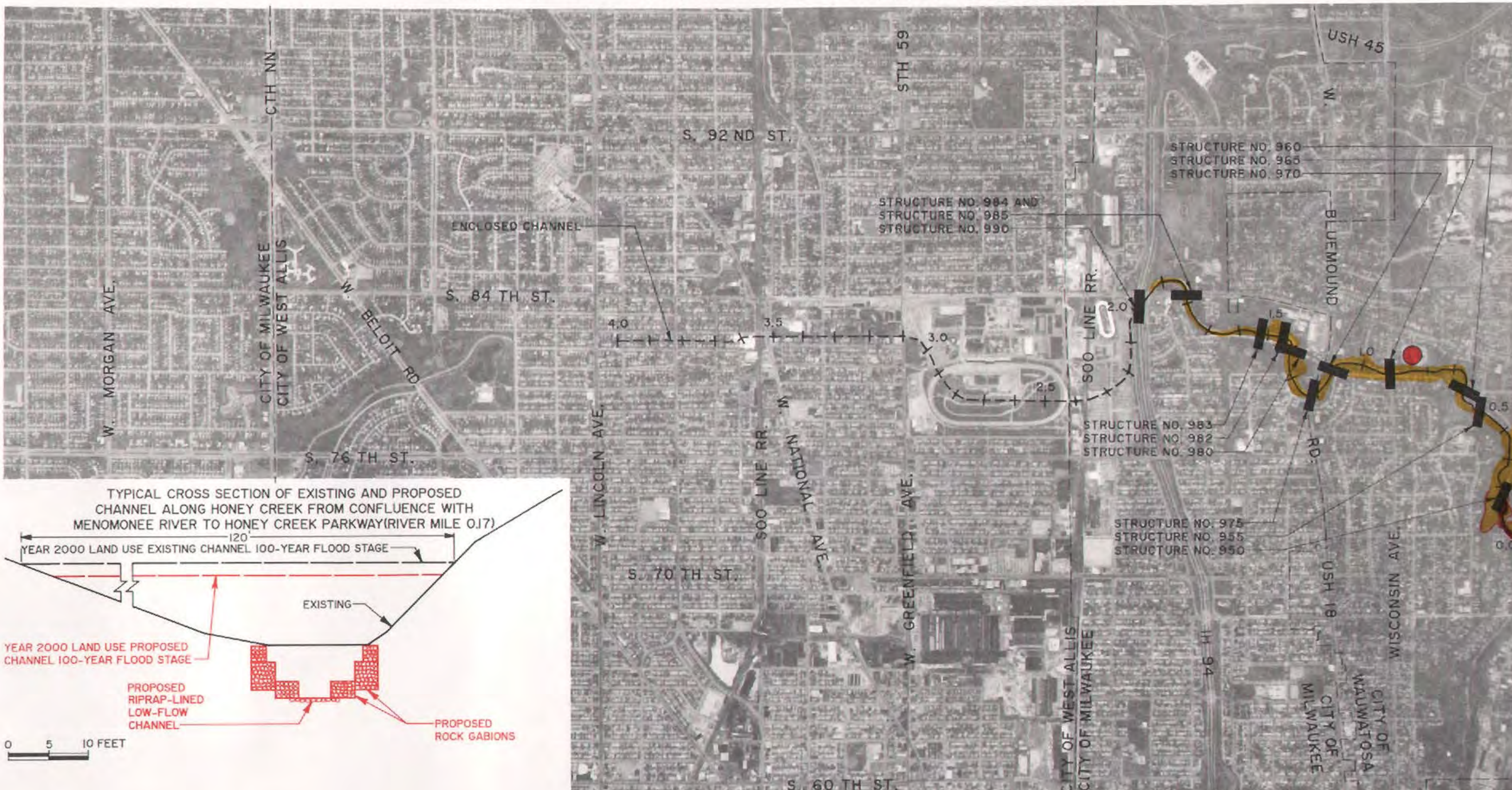
Figure 80 (continued)



Source: SEWRPC.

Map 170

RECOMMENDED FLOOD CONTROL SYSTEM PLAN HONEY CREEK FROM CONFLUENCE WITH MENOMONEE RIVER TO RIVER MILE 7.0



LEGEND

- 100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS
- 100-YEAR RECURRENCE INTERVAL FLOODPLAIN - YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS
- CHANNEL MODIFICATION
- STRUCTURE FLOODPROOFING
- 4.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPGRAPHIC MAPPING FOR HONEY CREEK IS SHOWN IN APPENDIX H

FUTURE STORM SEWER IMPROVEMENTS IN THE AREA TRIBUTARY TO THE HONEY CREEK CHANNEL ENCLOSURE SHOULD BE DESIGNED SO THAT THE EXISTING 2,650 CFS CAPACITY OF THE ENCLOSURE IS NOT EXCEEDED UNDER 100-YEAR RECURRENCE INTERVAL CONDITIONS.

DUE TO MAP SCALE LIMITATIONS, THE DIFFERENCE BETWEEN THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS, AND THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS, MAY NOT APPEAR ON THIS MAP. WHERE NO DIFFERENCE APPEARS REFERENCE SHOULD BE MADE TO THE FLOOD STAGE PROFILE SHOWN BELOW

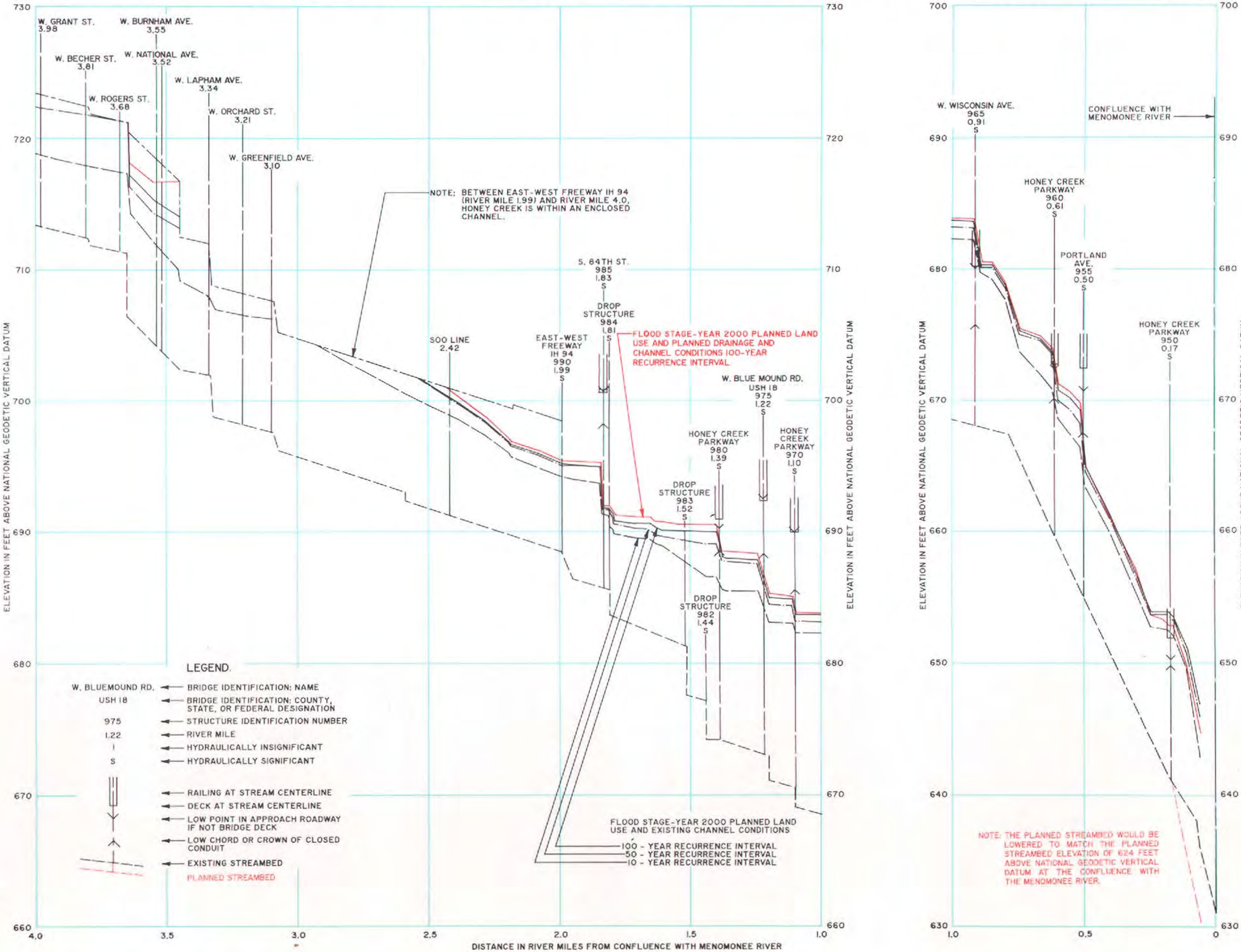
GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Figure 81

RECOMMENDED PLAN FLOOD STAGE PROFILE FOR HONEY CREEK FROM CONFLUENCE WITH MENOMONEE RIVER TO RIVER MILE 7.0



Map 170 (continued)

LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

100-YEAR RECURRENCE INTERVAL FLOODPLAIN - YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS

7.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR HONEY CREEK IS SHOWN IN APPENDIX H

NOTE: FUTURE STORM SEWER IMPROVEMENTS IN THE AREA TRIBUTARY TO THE HONEY CREEK CHANNEL ENCLOSURE DOWNSTREAM OF W. ARTHUR AVENUE SHOULD BE DESIGNED SO THAT THE EXISTING 2,650 CFS CAPACITY OF THE ENCLOSURE IS NOT EXCEEDED UNDER 100-YEAR RECURRENCE INTERVAL CONDITIONS.

DUE TO MAP SCALE LIMITATIONS, THE DIFFERENCE BETWEEN THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS, AND THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS, MAY NOT APPEAR ON THIS MAP. WHERE NO DIFFERENCE APPEARS REFERENCE SHOULD BE MADE TO THE FLOOD STAGE PROFILE SHOWN BELOW

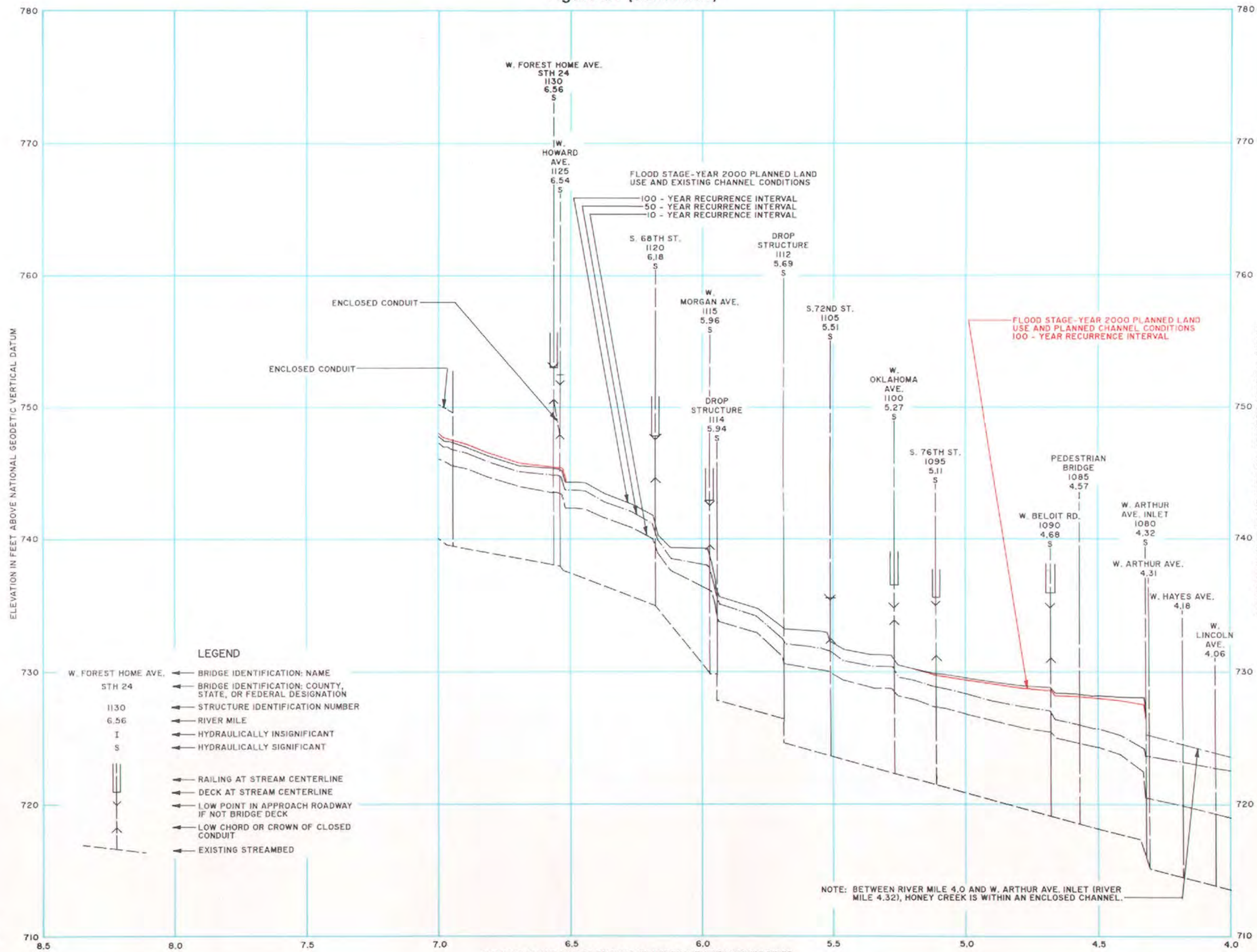
GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Source: SEWRPC.

Figure 81 (continued)



Source: SEWRPC.

modifications was assigned to the Menomonee River portion of the plan since the recommended measures were required solely due to the Menomonee River modifications.

Flood Control and Related Drainage System Plan Implementation: The recommended structure floodproofing or elevation would be implemented by the individual property owner concerned. It is recommended that the private owner bear the cost of structure floodproofing or removal. It is further recommended that the professional services required to prepare plans for the floodproofing and elevation of the building be made available to the property owner, at no cost, by the City of Wauwatosa engineering department. Also, it is recommended that the City of Wauwatosa review its building ordinance to ensure that appropriate floodproofing regulations are included. It is recommended that the City explore, on behalf of the property owner, any available state and/or federal aids for such floodproofing measures.

It is further recommended that the Milwaukee Metropolitan Sewerage District prepare large-scale topographic maps for the northeast and northwest one-quarters of U. S. Public Land Survey Section 33, Township 7 North, Range 21 East, Cities of Milwaukee and West Allis. Since these topographic maps would serve multiple purposes, no cost has been assigned to the flood control plan.

Cities of Milwaukee and West Allis Along the Channel Enclosure from River Mile 1.99 through 4.32 in the City of West Allis: The watershed study identified no flood hazard along this enclosed reach of the stream. That conclusion was reaffirmed under this study; however, further analyses were conducted of the hydraulic capacity of the channel enclosure and of the interrelationship between the enclosure, its tributary storm sewers, and the stream channel downstream of the enclosure.

A comprehensive, detailed analysis of the hydraulic characteristics of the entire storm sewer system tributary to the channel enclosure was not possible, due to the lack of large-scale topographic mapping of the tributary area. However, it was possible to evaluate the adequacy of the existing hydraulic capacity of the channel enclosure and of the tributary storm sewer system using the hydrologic simulation model developed for the watershed study and refined under this system planning effort, the hydraulic simulation model of the enclosure developed under this study, and historic flood stage data collected at the four Metropolitan Sewerage District crest stage gages located along the enclosure.

The largest recorded flood in the subwatershed was the flood of April 21, 1973. The peak discharge at the enclosure outlet, located just downstream of IH 94, the East-West Freeway, during that flood is estimated to be about 2,500 cfs, based on crest stage gage data and application of the hydraulic model. Application of the hydrologic simulation model indicated that the total peak flood flow at the outlet would have been greater if it were possible for all runoff to reach the stream enclosure without limitation by the hydraulic capacity of the tributary storm sewers. The box culvert forming the channel enclosure has a capacity of 2,650 cfs when flowing full.

With the exception of the area in the vicinity of S. 74th Street and W. Walker Street, where storm sewer surcharging has created localized flooding problems, there is apparently sufficient storage capacity available in streets and other open areas to detain the excess runoff during events exceeding the capacity of the storm sewer-channel enclosure system. Exceedance of the 2,650 cfs capacity of the enclosure would cause surcharging of the enclosure and attendant backflow out of storm sewer manholes and inlets of the connected storm sewers, with attendant flooding.

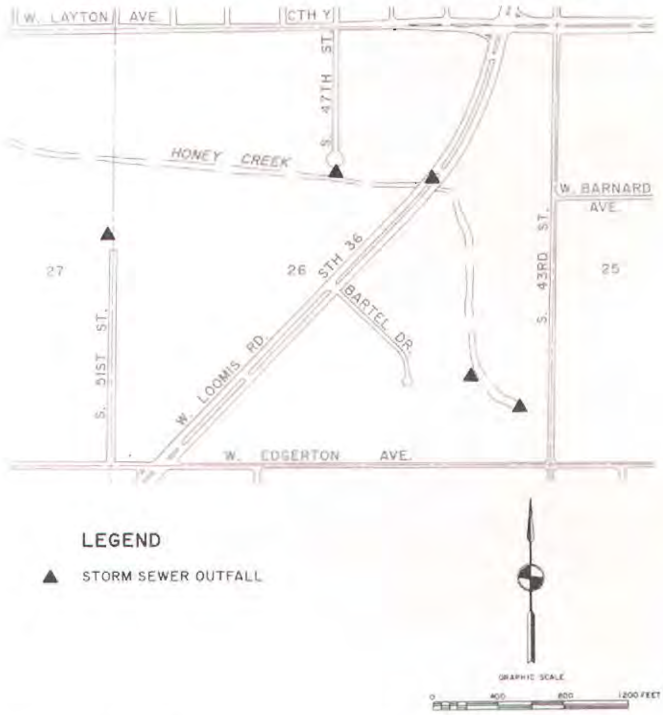
It is recommended that any future storm sewer improvements undertaken by the City of West Allis for the purpose of alleviating local stormwater drainage problems in the Honey Creek subwatershed be designed so as to limit the increase in the total peak discharge in the channel enclosure under 100-year recurrence interval conditions to about 150 cfs. This limitation may allow for an aggregate increase in individual storm sewer capacities of more than 150 cfs if it is determined that the timing of the peak flow from the individual storm sewers relative to the timing of the peak flow in the enclosure is such that the storm sewer peak would occur before or after the peak in the enclosure.

It is also recommended that the Milwaukee Metropolitan Sewerage District prepare large-scale topographic maps for the following U. S.

Public Land Survey one-quarter sections in Township 7 North, Range 21 East: the northeast and southeast of Section 32, the southwest and southeast of Section 33, and the northwest and southwest of Section 34; and for the following one-quarter sections in Township 6 North, Range 21 East: the southwest of Section 3, all of Section 4, the northeast and southeast of Section 5, the northeast of Section 8, the northeast and northwest of Section 9, and the northwest of Section 10. This topographic mapping would greatly assist in the future analysis of the stormwater drainage system tributary to the channel enclosure. Since these topographic maps would serve multiple purposes, no cost has been assigned to the flood control plan.

Map 171

LOCATIONS OF STORM SEWER OUTFALLS ALONG HONEY CREEK WITH INVERT ELEVATIONS BELOW EXISTING STREAMBED



Source: SEWRPC.

<u>City of Greenfield from IH 894 at River Mile 7.53 through the S. 43rd Street Storm Sewer Outfall at River Mile 8.83</u>: The watershed study did not include an analysis of this reach. The policy plan attendant to this system plan identified this reach for District jurisdiction; therefore, it was analyzed under this system planning effort.

The analysis considered the proposed reconstruction of the W. Layton Avenue crossing by Milwaukee County in 1990 and 1991 and the five existing storm sewer outfalls which are located below the existing streambed as shown on Map 171 and Figure 82.

<u>Channel Modification as Proposed by the City of Greenfield</u>: About two decades ago, the City of Greenfield developed plans to lower the Honey Creek streambed and enlarge the channel from IH 894 to S. 43rd Street. The design and construction of storm sewers installed since that time were based on the assumption that the City's proposed channel modification would be implemented. To that end, the City has purchased the right-of-way for the entire channel modification project.

Milwaukee County's recent design for the reconstruction and widening of W. Layton Avenue, including the crossing of Honey Creek, was done within the context of the City's proposed modifications. Milwaukee County's W. Layton Avenue reconstruction project calls for the removal of the 4.8-foot-high drop structure located just upstream of IH 894 and the construction of a realigned, deepened, and widened turf-lined channel between IH 894 and W. Layton Avenue. The channel would have an eight-foot-wide, 0.4-foot-deep, low-flow channel with side slopes of one vertical on 10 horizontal; and a flood control channel with side slopes of one vertical on three horizontal for the first 5.5 feet of depth, transitioning to one vertical on about 3.5 horizontal up to the existing grade. Realignment of the channel would eliminate three channel bends of about 90 degrees and would shorten the channel in that reach by approximately 0.1 mile. The County's proposal calls for the installation of a 10-foot by 10-foot reinforced concrete box culvert under W. Layton Avenue.[8] The County project terminates on the upstream side of W. Layton Avenue where another drop structure is proposed.

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[8] *The initial County design called for a 10-foot by 10-foot reinforced concrete box culvert. Because the Layton Avenue project was being designed at the same time this system plan was being prepared, the County modified the design to incorporate a 10-foot-wide by 8-foot-high reinforced concrete box culvert as recommended under this plan.*

The modified streambed profile as planned by the County would permit construction of the channel proposed by the City between W. Layton Avenue and the S. 43rd Street storm sewer outfall. The proposed drop structure on the upstream side of W. Layton Avenue would be removed during construction of the deepened and widened channel from IH 894 to S. 43rd Street.

With the exception of a short transition reach near W. Layton Avenue, the City's proposed modifications upstream of W. Layton Avenue would deepen and widen the channel following the approximate existing channel alignment. The City's channel design calls for a turf- and concrete-lined channel. The channel would have an eight-foot-wide, 0.4-foot-deep, concrete-lined low-flow channel with side slopes of one vertical on 10 horizontal and a concrete-lined flood control channel with side slopes of one vertical on two horizontal for the first 3.5 feet of depth, with a transition to a turf-lined channel with side slopes of one vertical on four horizontal.

Refined Flood Control System Plan: The flood control and related drainage system plan herein presented represents a refinement of that proposed by the City of Greenfield and Milwaukee County. The refined modified channel follows the same alignment and has approximately the same streambed profile and channel cross-sections as the city and county proposals.

The analyses which were conducted under this study did not include consideration of secondary flooding for the reasons already noted. The analysis identified a total of four structures in the 100-year recurrence interval flood hazard area under planned land use and existing channel conditions. One of these structures is located between W. Layton Avenue and IH 894 and the other three are located along the upper reach of Honey Creek near W. Loomis Road.

As shown on Map 172, the refined flood control and related drainage system plan for Honey Creek along the reach from IH 894 to S. 43rd Street consists of modification, realignment, and straightening of about 1.28 miles of stream, resulting in a total reach length of 1.17 miles under planned conditions. The peak flood profile attendant to planned land use and channel conditions is shown on Figure 82. Full implementation of this plan would serve to eliminate structural flood damages in this reach for floods up to and including the 100-year recurrence interval flood under planned land use and channel conditions. Implementation of the plan would also provide adequate outlets for five existing storm sewers. Because the channel modifications are primarily designed to provide adequate storm sewer outlets and secondarily to abate flood damages, construction of a channel which would completely contain the 100-year recurrence interval flood under planned land use conditions with two feet of freeboard would be uneconomical. Therefore, some overbank flooding into existing floodplain areas is permitted.

As shown on Figure 82, it is recommended that the streambed be lowered a maximum of about seven feet. As shown on Map 172, downstream of W. Layton Avenue the modified channel section would have a 1.0-foot-deep, three-foot-wide, turf-lined low-flow channel and a turf-lined flood control channel with an eight-foot bottom width, side slopes of one vertical on three horizontal for the lower 5.4 feet, then side slopes of one vertical on about 3.5 horizontal up to the existing grade. Upstream of W. Layton Avenue the modified channel section would have a 1.0-foot-deep, three-foot-wide, turf-lined low-flow channel and a turf-lined flood control channel with an eight-foot bottom width, side slopes of one vertical on two horizontal for the lower three feet, then side slopes of one vertical on about 3.5 horizontal up to the existing grade. The modified channel cross-section in this reach is shown on Map 172.

In the reach upstream of W. Layton Avenue, the channel is located either in city or county park land or in a 120-foot-wide drainage easement. In order to improve the appearance of this reach and to complement the park and residential setting through which the Creek flows, it is recommended that the detailed design for this reach consider the provision of some meandering and variability in the low-flow section of the proposed channel. Due to right-of-way restrictions along the north bank on the downstream side of W. Loomis Road, it would be necessary to provide a vertical retaining wall along that bank for about 80 feet downstream of the road.

Hydrologic simulation modeling conducted for this system planning effort indicates that the channelization project would create an unacceptable increase in downstream 100-year recurrence interval flood flows without the provision of detention storage in the reach containing the modification. Therefore, it is recommended that 48 acre-feet of detention storage be provided

Table 107

IMPACT OF RECOMMENDED FLOOD CONTROL PLAN FOR HONEY CREEK FROM IH 894 TO S. 43RD STREET ON 100-YEAR RECURRENCE INTERVAL FLOOD DISCHARGE

Stream	Location	River Mile	100-Year Recurrence Interval Flood Discharges (cfs) Year 2000 Planned Land Use: Existing Channel Condition	Recommended Plan Condition	Percent Increase
Honey Creek	Upstream of Arthur Avenue enclosure inlet	4.32	2,280	2,270	-0.4
	W. Oklahoma Avenue	5.27	1,870	1,860	-0.5
	Downstream of W. Howard Avenue	6.44	1,310	1,370	+4
	IH 894	7.53	760	970	+28
	W. Layton Avenue	7.80	640	760	+19
	W. Loomis Road	8.53	430	430	0

Source: SEWRPC.

upstream of W. Layton Avenue. That total amount of storage would be provided in the modified channel and a supplementary 12.5-acre-foot detention basin in the northeast one-quarter of U. S. Public Land Survey Section 26, Township 6 North, Range 21 East, at the location shown on Map 172. The total detention basin area, including a buffer strip around its perimeter, would be about 7.5 acres. The maximum flood control pond elevation would be about 755.6 feet NGVD during a 100-year recurrence interval flood under planned land use and channel conditions.

The existing W. Layton Avenue bridge would be replaced with a new 10-foot-wide by 8-foot-high reinforced concrete box culvert, rather than the 10-foot-wide by 10-foot-high box culvert initially proposed by the County. This reduction in culvert size is recommended to provide storage in the upstream channel and detention basin. The proposed detention basin could be constructed with a permanent pond which would trap pollutants carried in stormwater runoff, providing water quality benefits as well as flood control benefits along Honey Creek. In addition, the pond could be designed for other recreational uses, such as ice skating.

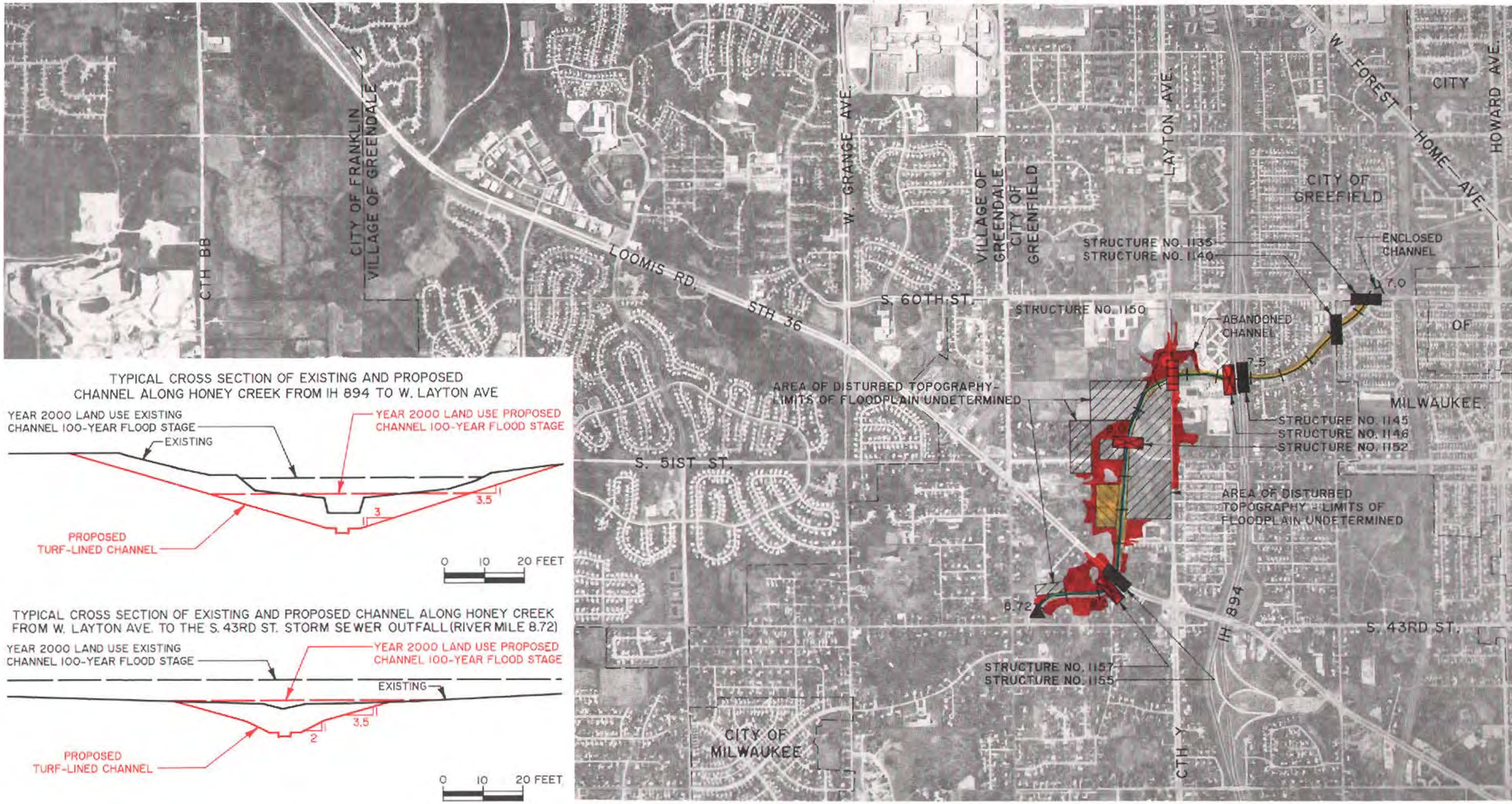
It is recommended that the existing construction access bridge at River Mile 8.11 and the old Loomis Road bridge at River Mile 8.55 be removed and not replaced.

The changes in the flood discharges which may be expected along Honey Creek as a result of the recommended channel modification are displayed in Table 107. The channel modification should cause no increase in the 100-year flood discharge downstream from River Mile 4.32. From the Arthur Avenue enclosure inlet at River Mile 4.32 through River Mile 6.52, a decrease of up to 0.2 foot may be expected in the 100-year recurrence interval flood stage under planned land use and channel conditions. From River Mile 6.53 through 7.55, increased flood flood flows may result in 100-year flood stage increases of up to 0.3 foot. Those stage increases would be contained within the existing drainage easement for Honey Creek. Upstream from River Mile 7.55, 100-year flood stages would be reduced by 4.0 to 6.5 feet due to the increased hydraulic capacity provided by the recommended channel modifications.

Utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of the refined

Map 172

RECOMMENDED FLOOD CONTROL PLAN FOR HONEY CREEK FROM RIVER MILE 7.0 TO THE S. 43RD STREET STORM SEWER OUTFALL



LEGEND

- 100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS
- 100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS
- CHANNEL MODIFICATION
- CHANNEL REALIGNMENT
- STRUCTURE REPLACEMENT
- STRUCTURE REMOVAL
- PROPOSED DETENTION BASIN
- EXISTING STORM SEWER OUTFALL
- 8.5 APPROXIMATE CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR HONEY CREEK IS SHOWN IN APPENDIX H

DUE TO MAP SCALE LIMITATIONS, THE DIFFERENCE BETWEEN THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS, AND THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS, MAY NOT APPEAR ON THIS MAP WHERE NO DIFFERENCE APPEARS REFERENCE SHOULD BE MADE TO THE FLOOD STAGE PROFILE SHOWN BELOW

GRAPHIC SCALE

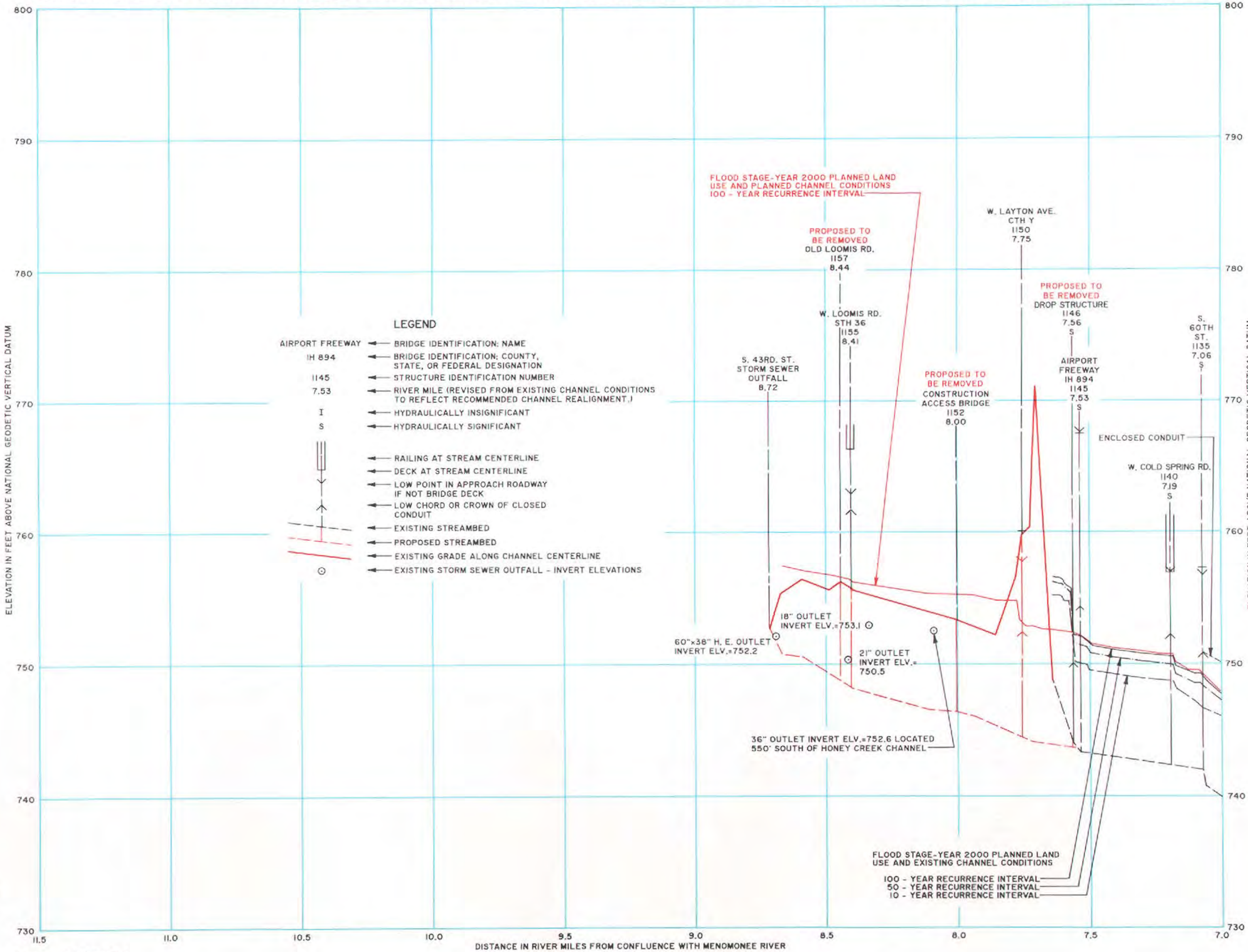
0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Source: SEWRPC.

Figure 82

RECOMMENDED PLAN FLOOD STAGE PROFILE FOR HONEY CREEK FROM RIVER MILE 7.0 TO THE S. 43RD STREET STORM SEWER OUTFALL



Source: SEWRPC.

recommended plan for this reach is estimated at $93,000. This cost consists of the amortization of the $1,282,000 capital cost, $800,000 for channel modification, $172,000 for bridge removal and replacement, and $310,000 for detention basin construction, plus $12,000 in annual operation and maintenance costs. The average annual flood damage abatement benefit is estimated at $900, yielding a benefit-cost ratio of less than 0.1. The actual benefit-cost ratio of the recommended plan would be higher than that based solely on the abatement of primary, overland flood damages. The plan would also abate stormwater drainage problems and secondary flood damages.

Flood Control and Related
Drainage System Plan Implementation

It is recommended that the refined recommended plan for Honey Creek in the City of Greenfield be implemented through the cooperative efforts of the City of Greenfield, Milwaukee County, and the Milwaukee Metropolitan Sewerage District. More specifically, it is recommended that the District design and construct the recommended detention basin and channel modifications from the upstream end of the proposed Milwaukee County bridge project at W. Layton Avenue through the S. 43rd Street storm sewer outlet. It is recommended that the District, with financial assistance from the Wisconsin Department of Natural Resources, develop the detention facility component to provide water quality and recreational benefits. It is recommended that the District maintain the modified channel from IH 894 to W. Layton Avenue. The City would maintain the recommended detention basin. It is also recommended that the District bear the cost of the removal of the private bridge at existing River Mile 8.11 and the old Loomis Road bridge at River Mile 8.55. The City has acquired the rights-of-way needed for the channel improvements.

It is recommended that Milwaukee County design and construct the recommended modified channel from the existing drop structure just upstream of IH 894 through W. Layton Avenue, including the recommended box culvert at W. Layton Avenue. The design of those components was in progress as of the publication date of this report.

Due to significant development along Honey Creek since the preparation of the most recent large-scale topographic maps in 1975, it is recommended that the Milwaukee Metropolitan Sewerage District prepare large-scale topographic maps for the southwest one-quarter of U. S. Public Land Survey Section 23 and the northeast and northwest one-quarters of Section 26, all in Township 6 North, Range 21 East, City of Greenfield. It is suggested that those maps be prepared following the W. Layton Avenue reconstruction and the construction of the recommended channel modifications. Since the topographic maps would serve multiple purposes, no cost has been assigned to the flood control plan.

The capital cost of the various components of the recommended plan is apportioned by agency in Table 108.

UNDERWOOD CREEK AND DOUSMAN DITCH SUBWATERSHED FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN

Hydrologic and hydraulic analyses of Underwood Creek and Dousman Ditch had previously been conducted under the Commission's Menomonee River watershed study. That study also assessed existing and possible future flood problems along these streams, evaluated alternative measures to alleviate those problems, and included a recommendation for the implementation of certain flood control measures. This system planning effort represents a refinement of that earlier study. Presented below are an overview of the subwatershed, a review of the previously considered flood control measures, and a refined recommended flood control plan for Underwood Creek and Dousman Ditch.

Overview of the Study Area

The Underwood Creek subwatershed, which includes Dousman Ditch and its tributary drainage area, is located largely in east-central Waukesha County, with a significant portion, however, extending into Milwaukee County. The subwatershed includes portions of the Cities of Brookfield and Wauwatosa, and the entire Village of Elm Grove. From its origin near Franklin Wirth Park in the City of Brookfield, Underwood Creek flows in a generally easterly direction for a length of about 7.7 miles to its confluence with the Menomonee River in the City of Wauwatosa. From its origin at Wisconsin Avenue in the City of Brookfield, Dousman Ditch flows in a northerly direction for a distance of about 2.5 miles to its confluence with

Table 108

SUMMARY OF RECOMMENDED PLAN CAPITAL COSTS FOR HONEY CREEK FROM IH 894 THROUGH THE S. 43RD STREET STORM SEWER OUTFALL

Implementing Agency	Flood Control Measures	Estimated Capital Cost[a,b]
Milwaukee Metropolitan Sewerage District	Channel modifications Bridge removal Detention basin	$ 590,000 15,000 310,000
	Subtotal	$ 915,000
Milwaukee County	Channel modifications Bridge replacement	$ 210,000 157,000
	Subtotal	$ 367,000
	Total	$1,282,000

[a]*Costs do not include specific consideration of added facility requirements associated with water quality and recreation benefits. A portion of those costs may be funded under the Wisconsin Department of Natural Resources nonpoint source priority watershed program.*

[b]*Costs are expressed in 1986 dollars.*

Source: SEWRPC.

Underwood Creek. Underwood Creek drains an area of about 11.08 square miles, as shown on Map 173. Of this 11.08 square miles, 3.68 square miles are tributary to Dousman Ditch. The Dousman Ditch subwatershed is shown on Map 174. The extent of the subwatershed area within each minor civil division involved is given in Table 109.

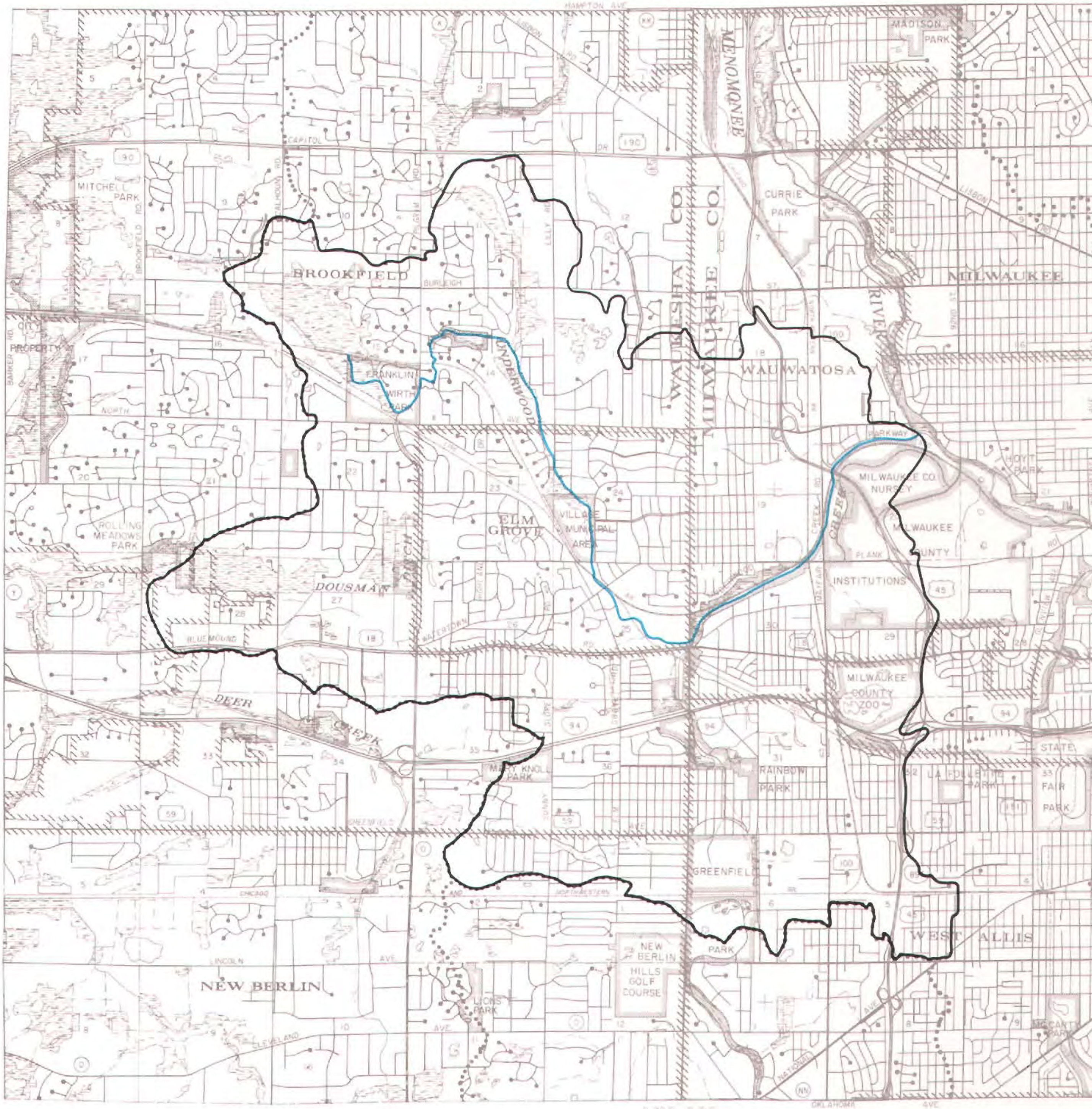
More specifically, from its origin near the Soo Line Railroad Company railway at Franklin Wirth Park in the City of Brookfield, Underwood Creek flows in a southeasterly direction to its confluence with Dousman Ditch, a distance of about 0.7 mile; thence, northerly for about 0.9 mile to Woodbridge Road; thence easterly for about 0.5 mile to Clearwater Drive; thence southerly for about 3.1 miles to its confluence with the South Branch of Underwood Creek; thence northeasterly for about 1.0 mile to W. Watertown Plank Road in the City of Wauwatosa; thence northerly for about 0.75 mile to USH 45; thence easterly for about 0.75 mile to its confluence with the Menomonee River. The entire 7.7-mile reach described is classified as perennial.

From its origin at Wisconsin Avenue, just north of Blue Mound Road, Dousman Ditch flows easterly for about 1.1 miles; thence in a northerly direction to Gebhardt Road, a distance of about 0.8 mile; and continues northerly for about 0.6 mile through North Avenue to its confluence with Underwood Creek. Of the 2.5-mile reach described, the entire stream is classified as perennial.

The lower 2.6-mile reach of Underwood Creek is located within the current District limits and is recommended for District jurisdiction in the policy plan companion to this system plan. The remainder of Underwood Creek and all of Dousman Ditch, which are located outside of the current District limits but within an area defined in the policy plan as within possible future District limits, were found to meet the criteria for

Map 173

THE UNDERWOOD CREEK SUBWATERSHED



LEGEND

SUBWATERSHED BOUNDARY

PERENNIAL STREAM REACH

Source: SEWRPC.

Table 109

AREAL EXTENT OF CIVIL DIVISIONS IN THE UNDERWOOD CREEK SUBWATERSHED

Civil Division	Civil Division Area Included Within Subwatershed (square miles)	Percent of Subwatershed Area Within Civil Division
City of Brookfield	4.44	40.1
City of Wauwatosa	3.99	36.0
Village of Elm Grove	2.65	23.9
Total	11.08	100.0

Source: SEWRPC.

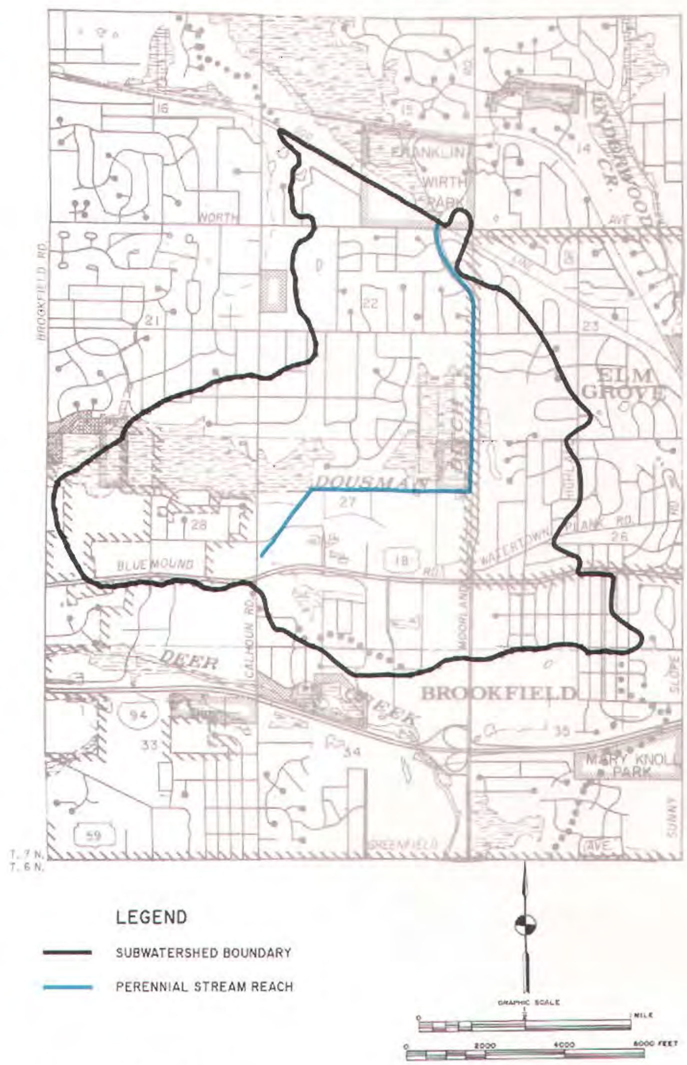
District jurisdiction. Moreover, any flood control measures carried out along the upper reaches of Underwood Creek and Dousman Ditch could impact on flood flows and stages and recommended flood control measures along the reach of Underwood Creek under District jurisdiction. These additional stream reaches were accordingly included in the system planning effort.

In 1985, about 84 percent of the Underwood Creek subwatershed, including Dousman Ditch, was developed for urban use. Nearly 50 percent of the urban area was developed for residential use. Other uses included governmental and institutional, commercial, and recreational. The developed areas of the subwatershed in Milwaukee County are generally provided with a full range of municipal street improvements, including paved streets with curbs and gutters and attendant storm sewers. In Waukesha County, some of the developed areas are provided with paved streets with road ditches, while other development areas are provided with paved streets with curbs and gutters and attendant storm sewers. The planned land use conditions utilized in the system planning effort assume that the subwatershed will be almost entirely urbanized by the design year of the system plan.

The flood profiles for Underwood Creek and Dousman Ditch for planned land use and existing channel conditions are shown as Figures 83 and 84, respectively. The extent of the 100-year recurrence interval flood hazard area under planned land use and existing channel conditions is shown on Map 175 for Underwood Creek and on Map 176 for Dousman Ditch.

Map 174

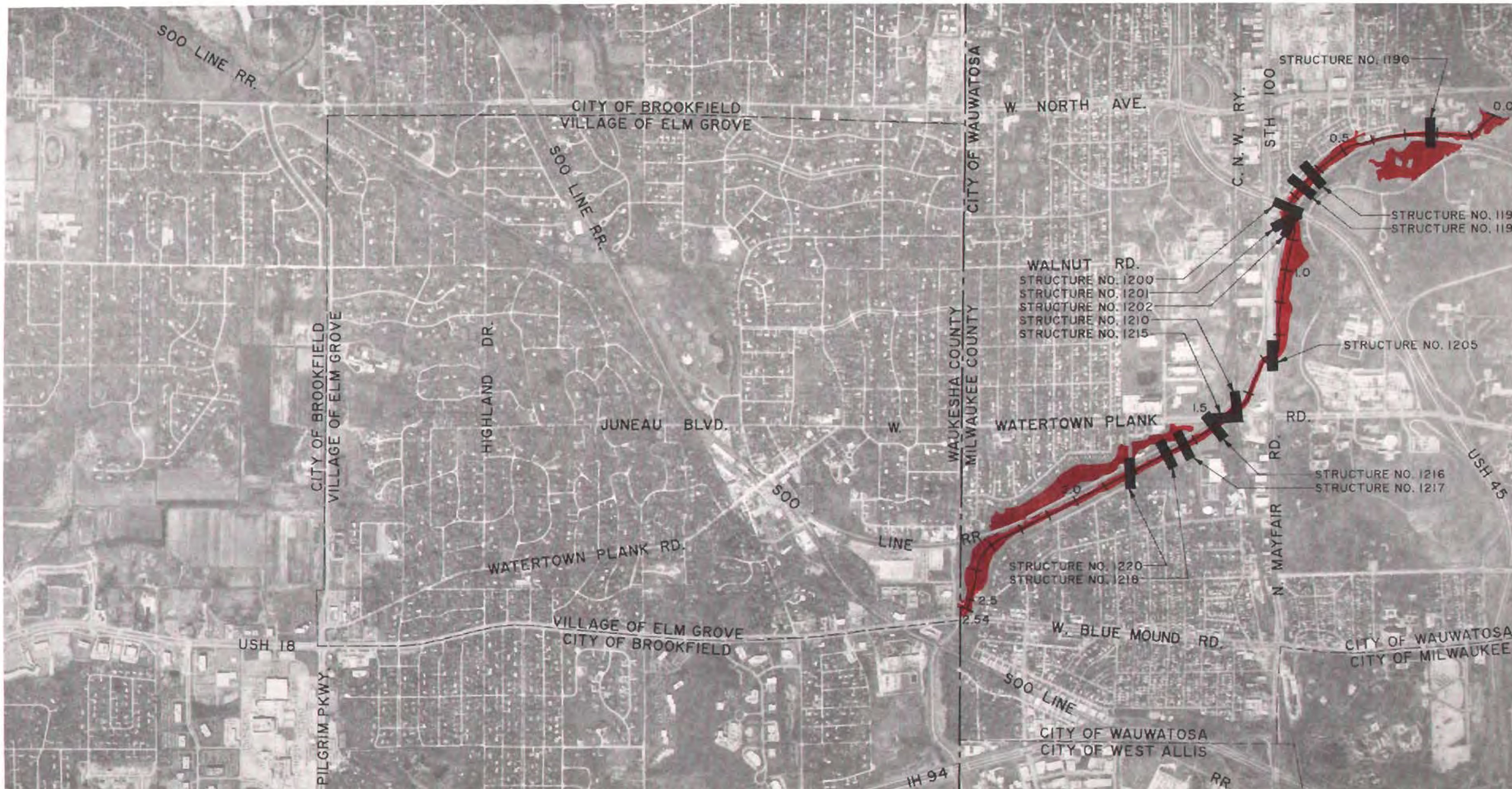
THE DOUSMAN DITCH SUBWATERSHED



Source: SEWRPC.

Map 175

100-YEAR RECURRENCE INTERVAL FLOODPLAIN FOR UNDERWOOD CREEK UNDER YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

2.5 — APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPGRAPHIC MAPPING FOR UNDERWOOD CREEK IS SHOWN IN APPENDIX H

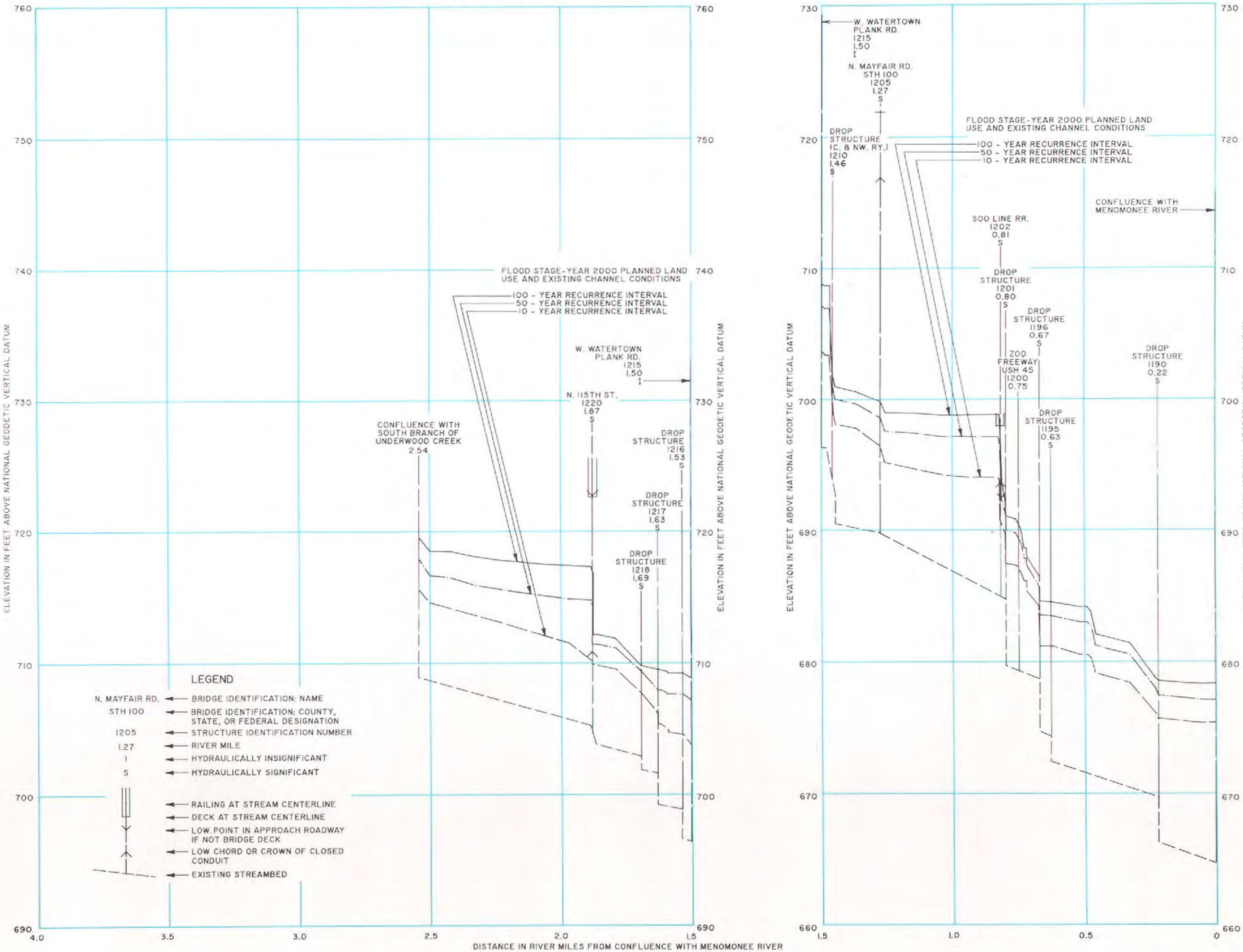
GRAPHIC SCALE

0 1/2 1 MILE

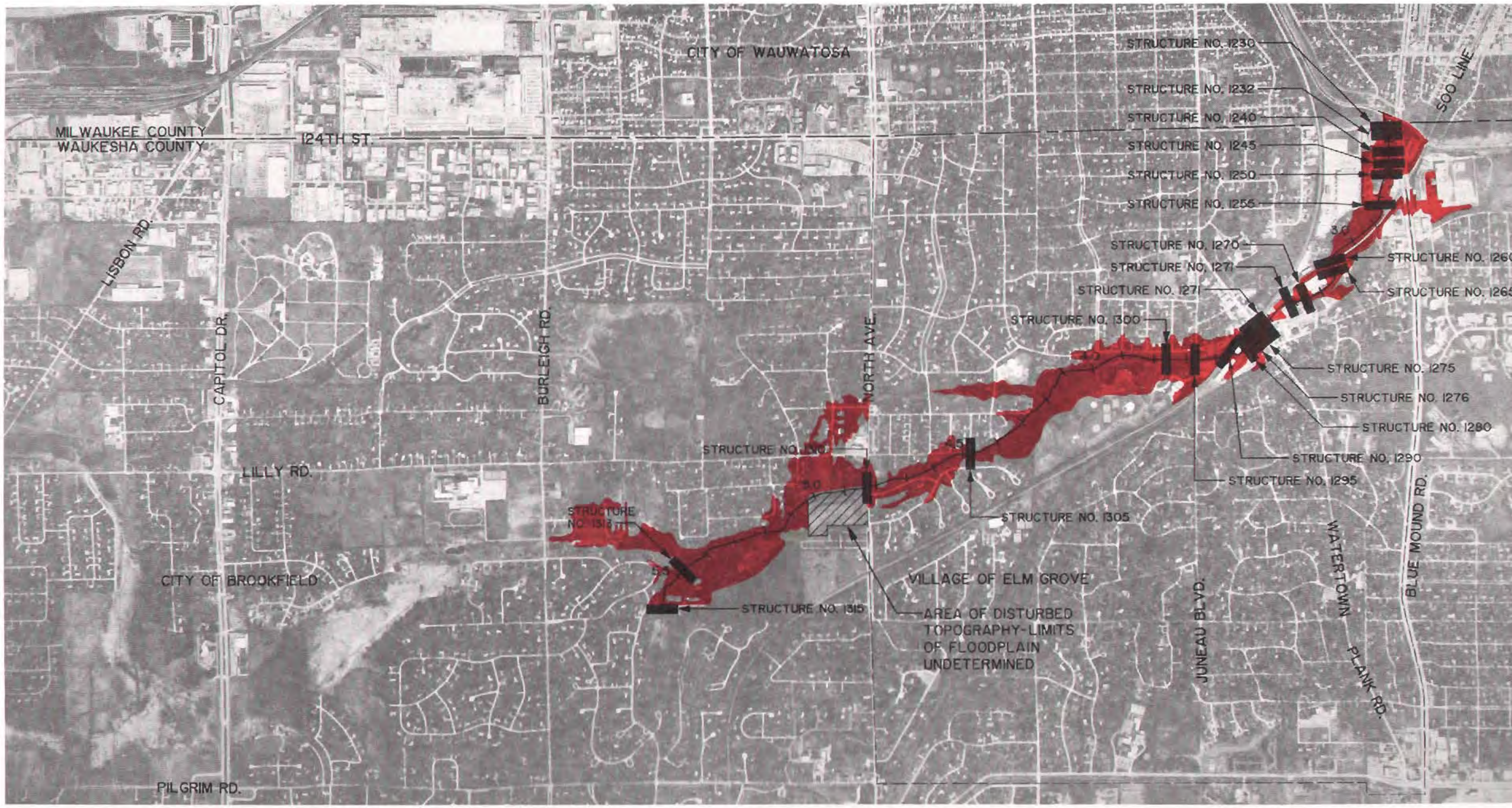
DATE OF PHOTOGRAPHY: APRIL 1986

Figure 83

FLOOD STAGE AND STREAMBED PROFILE FOR UNDERWOOD CREEK



Map 175 (continued)



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

4.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

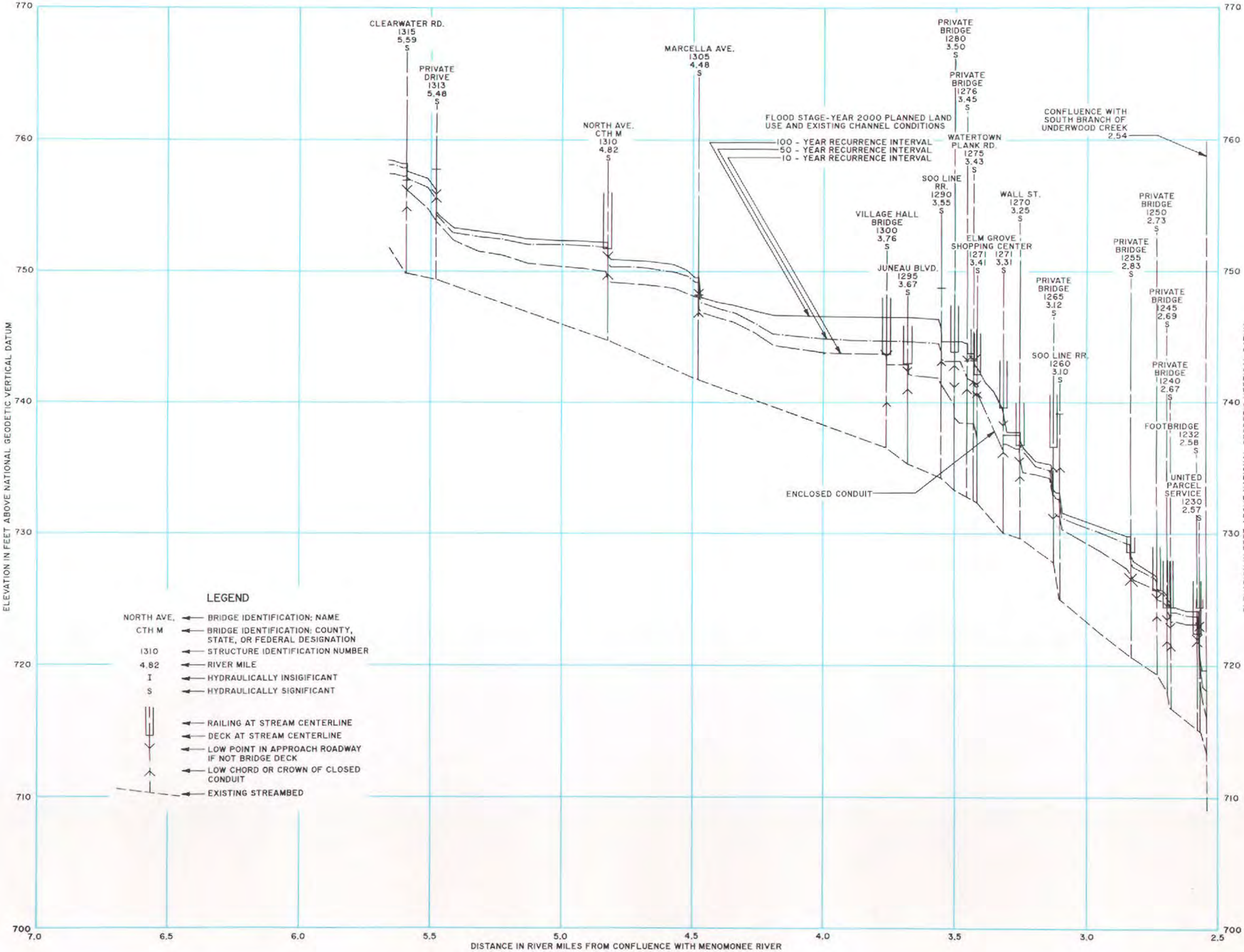
NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR UNDERWOOD CREEK IS SHOWN IN APPENDIX H

GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Figure 83 (continued)



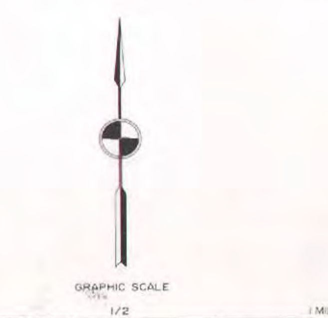
Map 175 (continued)

LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

7.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

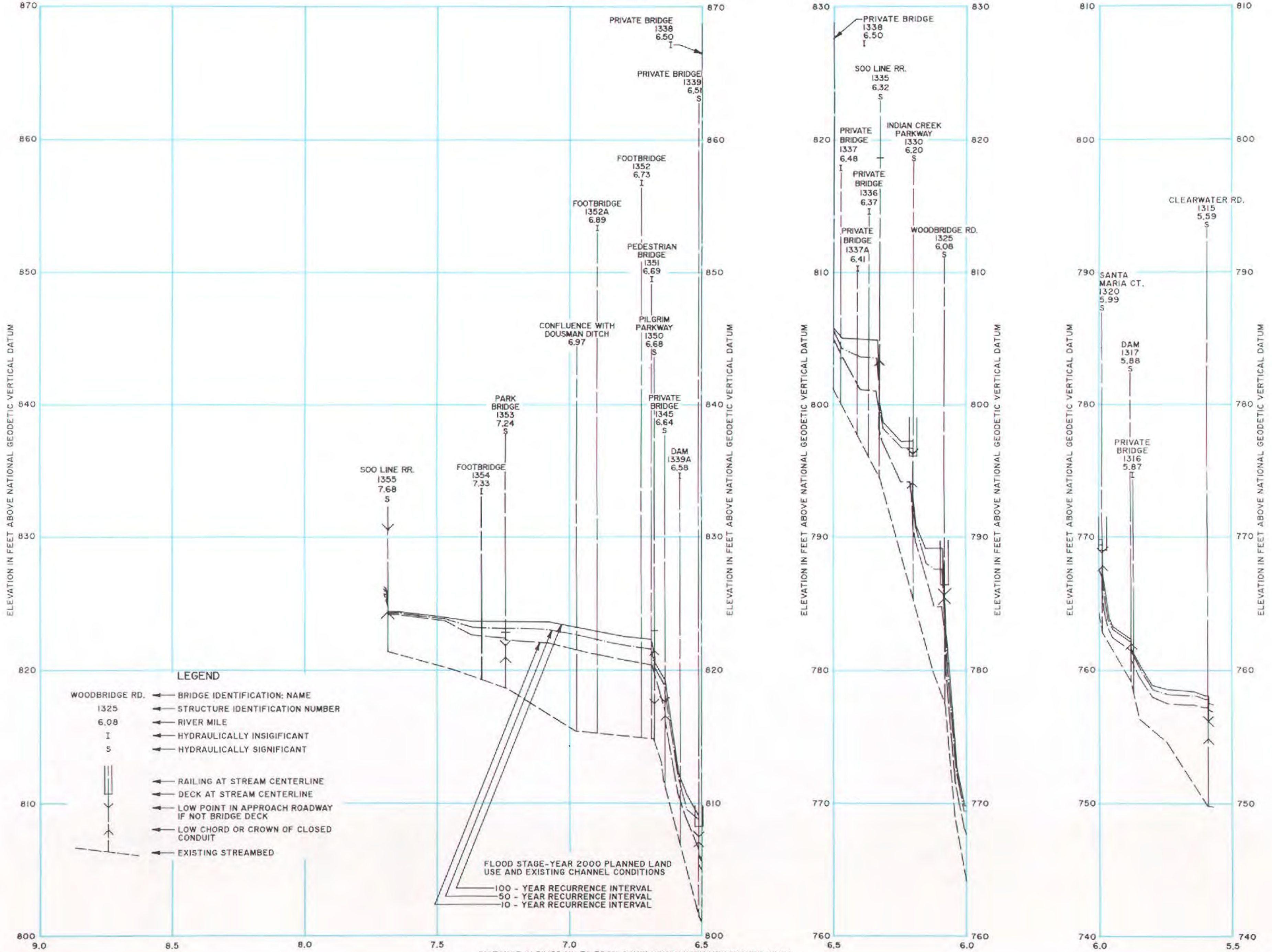
NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR UNDERWOOD CREEK IS SHOWN IN APPENDIX H



DATE OF PHOTOGRAPHY: APRIL 1986

Source: SEWRPC.

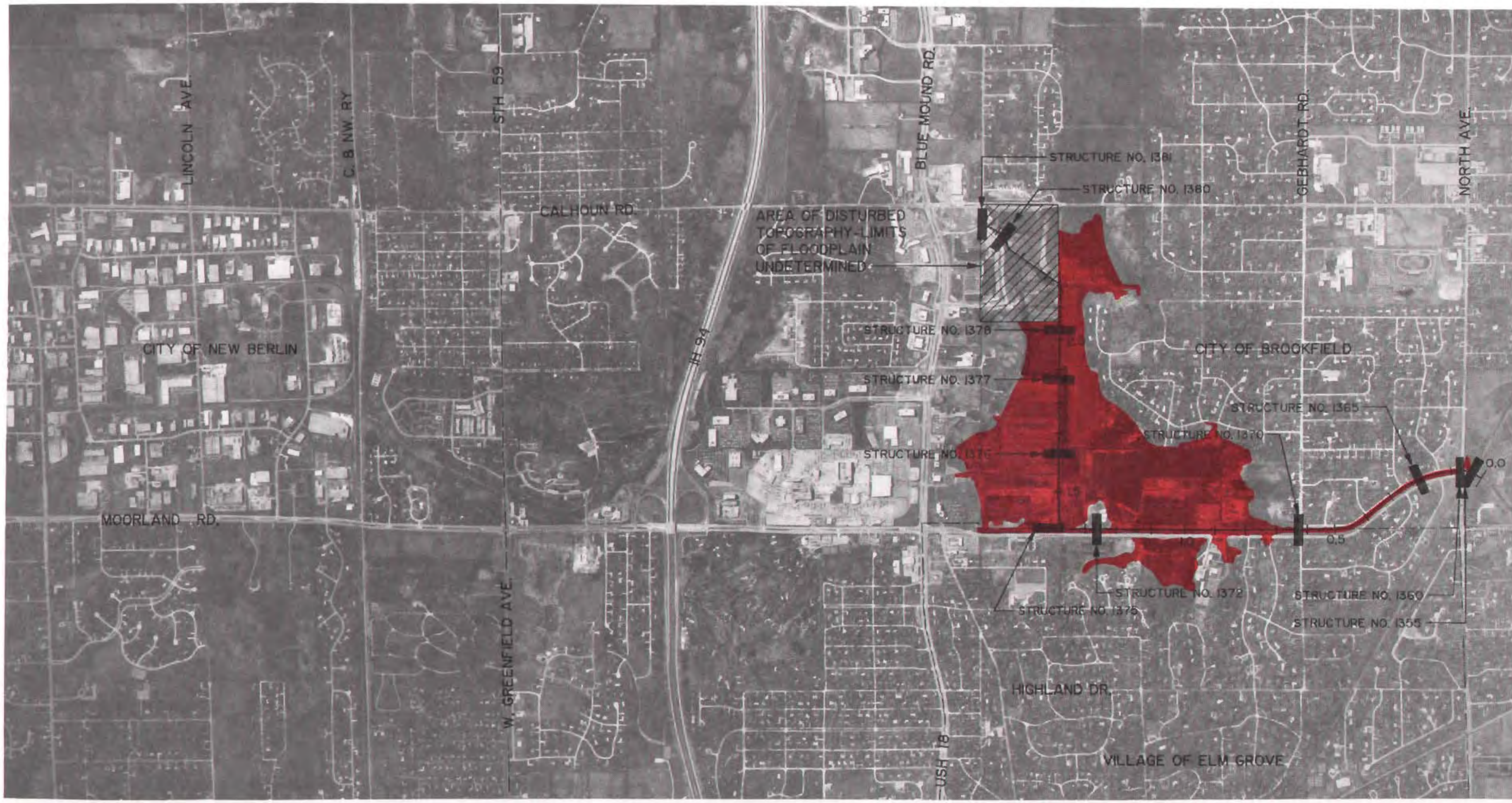
Figure 83 (continued)



Source: SEWRPC.

Map 176

100-YEAR RECURRENCE INTERVAL FLOODPLAIN FOR DOUSMAN DITCH UNDER YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

1.5 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR DOUSMAN DITCH IS SHOWN IN APPENDIX H

Source: SEWRPC.

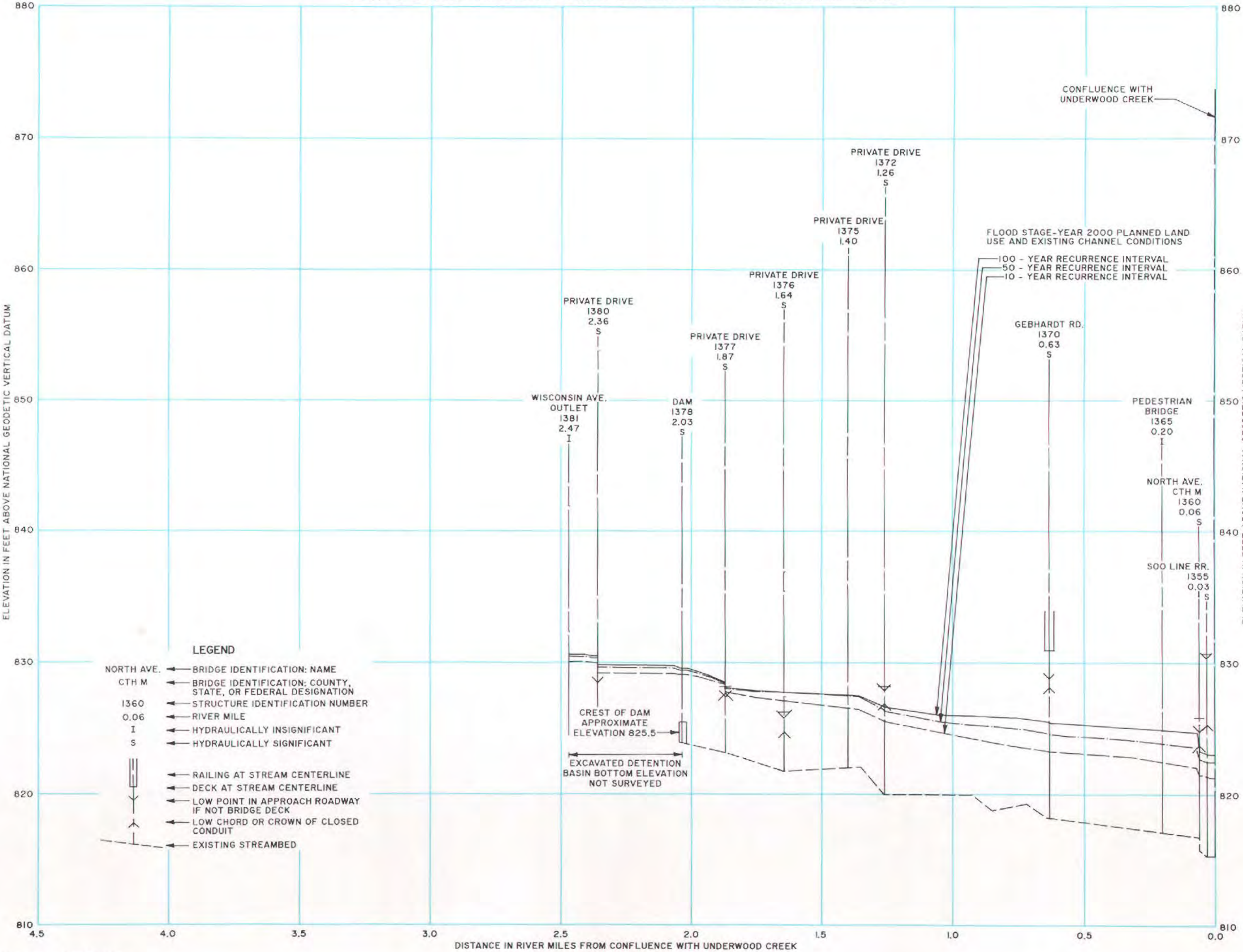
GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Figure 84

FLOOD STAGE AND STREAMBED PROFILE FOR DOUSMAN DITCH



Source: SEWRPC.

Evaluation of Alternative Flood Control and Related Drainage System Plans for Underwood Creek

A number of alternative flood control measures were considered for Underwood Creek and Dousman Ditch under the Commission's Menomonee River watershed study. These alternatives are presented below along with their estimated costs. From these alternatives, a final composite flood control plan was recommended under the watershed study for Underwood Creek and Dousman Ditch. This plan has been further refined as part of this system planning effort.

Some reaches are not expected to incur any structural damages due to overland flooding during the 100-year recurrence interval flood under planned land use and channel conditions. Those reaches are: 1) Underwood Creek from its confluence with the Menomonee River to the Milwaukee-Waukesha County line and 2) Underwood Creek upstream of its confluence with Dousman Ditch. Together, these reaches total 3.29 miles in length, or 43 percent of the total stream length under District jurisdiction.

Menomonee River Watershed Study: Under the Menomonee River watershed study, a total of seven flood control alternatives were considered for the City of Brookfield. These alternatives included: 1) No Action; 2) Detention Storage; 3) Structure Floodproofing and Removal; 4) a Combination of Channelization, Structure Floodproofing and Removal, and Bridge Alteration; 5) a Combination of Dikes and Floodwalls, Structure Floodproofing and Removal, and Bridge Alteration; 6) Bridge Alteration or Removal; and 7) a Combination of Detention Storage, Bridge Alteration, and Structure Floodproofing and Removal. The estimated cost of each of these alternatives and the attendant benefit-cost ratio are presented in Table 110.

A total of 10 flood control alternatives were considered under the watershed study for the Village of Elm Grove. These alternatives included: 1) No Action; 2) Detention Storage; 3) Structure Floodproofing and Removal; 4) Major Channel Modification; 5) Minor Channel Modification; 6) Dikes and Floodwalls; 7) Bridge and Culvert Alteration or Replacement; 8) a Combination of Detention Storage and Channelization; 9) a Combination of Detention Storage, Major and Intermediate Channelization, and Structure Floodproofing; and 10) a Combination of Detention Storage and Structure Floodproofing and Removal. The estimated cost of each of these alternatives and the attendant benefit-cost ratio are provided in Table 111.

The initial recommendation made by the Menomonee River Watershed Advisory Committee for flood control on Underwood Creek and Dousman Ditch included the combination of detention storage, bridge alteration, and structure floodproofing and removal for the City of Brookfield; and a combination of detention storage, major and intermediate channel modification, and structure floodproofing for the Village of Elm Grove. The detention storage for both communities would be achieved by the construction of a 215-acre-foot capacity dry detention basin located along Dousman Ditch immediately upstream of of W. Gebhardt Road in the City of Brookfield.

Subsequent to a series of informational meetings and a public hearing on the watershed plan, the Watershed Committee revised its recommendation for the Village of Elm Grove. The revision was made in response to opposition voiced by village officials to the initially recommended plan. That opposition was based on both aesthetic and financial considerations. The revised plan called for a combination of detention storage with structure floodproofing and removal.

The flood control plan for Underwood Creek and Dousman Ditch as recommended in the Menomonee River watershed study would have an estimated capital cost of $3,633,400. Assuming an annual interest rate of 6 percent and a project life of 50 years, the average annual cost of the plan would be $242,800, including $12,400 in annual operation and maintenance costs. The benefit-cost ratio of the plan was found to be 3.60.

Refined Flood Control System Plan: The flood control plan for Underwood Creek and Dousman Ditch developed as part of this system planning effort represents a refinement of that proposed under the Commission's Menomonee River watershed study. Incorporated into this refinement are the results of the hydrologic and hydraulic analyses conducted as a part of this system planning effort, updated topographic information provided by large-scale topographic maps produced since completion of the watershed study, and proposed development plans which may impact on the recommended plan.

Table 110

MENOMONEE RIVER WATERSHED STUDY FLOOD CONTROL ALTERNATIVES FOR UNDERWOOD CREEK AND DOUSMAN DITCH IN THE CITY OF BROOKFIELD

Alternative	Cost[a]					Benefit-Cost Ratio
	Capital	Amortized Capital[b]	Operation and Maintenance	Other	Annual Total	
1. No Action	$ 0	$ 0	$ 0	$148,800	$148,800	0
2. Detention Storage	231,800[c]	14,800	2,200	0	17,000	4.44
3. Structure Floodproofing and Removal	2,072,600	131,400	0	0	131,400	1.12
4. Combination of Channelization, Structure Floodproofing and Removal, and Bridge Alteration	1,996,600	126,600	600	0	127,200	1.16
5. Combination of Dikes and Floodwalls, Structure Floodproofing and Removal, and Bridge Alteration	2,529,400	160,400	14,000	0	174,400	0.84
6. Bridge Alteration or Removal	- -[d]	- -[d]	- -[d]	- -[d]	- -[d]	- -
7. Combination of Detention Storage, Bridge Alteration, and Structure Floodproofing and Removal	1,439,000[c]	91,200	2,200	0	93,400	1.57

[a] *Costs are expressed in 1986 dollars.*

[b] *Amortized capital cost is based on an interest rate of 6 percent and a project life of 50 years.*

[c] *Based on the assumption that the cost of the detention basin would be shared by the Village of Elm Grove and the City of Brookfield in proportion to the flood hazard mitigation derived by the community.*

[d] *No costs were computed since this alternative was found to be technically infeasible.*

Source: SEWRPC.

As part of the flood control plan refinement, two alternative floodwater storage measures were considered. The first alternative consists only of maintaining the existing natural floodwater storage along Dousman Ditch between W. Gebhardt Road and W. Wisconsin Avenue. Under a 100-year recurrence interval event and assuming planned land use and existing channel conditions, about 230 acre-feet of floodplain storage is presently available along this reach. This storage would be maintained by not allowing any new development to occur within the 100-

Table 111

MENOMONEE RIVER WATERSHED STUDY FLOOD CONTROL ALTERNATIVES FOR UNDERWOOD CREEK AND DOUSMAN DITCH IN THE VILLAGE OF ELM GROVE

Alternative	Cost[a]					Benefit-Cost Ratio
	Capital	Amortized Capital[b]	Operation and Maintenance	Other	Total	
1. No Action	$ 0	$ 0	$ 0	$725,600	$725,600	0
2. Detention Storage	1,028,400[c]	65,200	10,200[c]	0	75,400	4.24
3. Structure Floodproofing and Removal	3,740,000	237,600	0	0	237,600	3.05
4. Major Channel Modification	7,322,400	464,600	2,000	0	466,600	1.56
5. Minor Channel Modifications	- -[d]	- -[d]	- -[d]	- -[d]	- -[d]	- -
6. Dikes and Floodwalls	9,560,200	606,600	22,400	0	629,000	1.15
7. Bridge and Culvert Replacement	- -[d]	- -[d]	- -[d]	- -[d]	- -[d]	- -
8. Combination of Detention Storage and Channelization	7,513,000	476,400	12,200	0	488,600	1.49
9. Combination of Detention Storage, Major and Intermediate Channelization, and Structure Floodproofing	6,544,400	415,200	13,200	0	428,400	1.69
10. Combination of Detention Storage and Structure Floodproofing and Removal	2,194,400[c]	139,200	10,200[c]	0	149,400	4.86

[a]*Costs are expressed in 1986 dollars.*

[b]*Amortized capital cost is based on an interest rate of 6 percent and a project life of 50 years.*

[c]*Based on the assumption that the cost of the detention basin would be shared by the Village of Elm Grove and the City of Brookfield in proportion to the flood hazard mitigation derived by the community.*

[d]*No costs were computed since this alternative was found to be technically infeasible.*

Source: SEWRPC.

year floodplain. This alternative is similar to that recommended under the watershed study in that it would provide a like amount of floodwater storage while allowing some discharge to downstream reaches.

The second alternative considered includes the construction of two stormwater detention basins along Dousman Ditch upstream of Gebhardt Road. This two-basin approach represents a revision of the original recommended plan and is based in part on a 1979 study conducted by the engineering consulting firm of Donohue and Associates, Inc. That study, which was prepared for the City of Brookfield, recommended that the proposed basins be designed to hold the entire 100-year recurrence interval runoff from the area tributary to the basins with no significant discharge to downstream reaches. This would require the provision of about 330 acre-feet of storage.

Under the first floodwater storage alternative, a total of 76 structures would continue to incur flood damages along Underwood Creek from a 100-year recurrence interval flood under planned land use conditions. Under the second alternative, this number would be reduced to 41 structures. Therefore, it was decided that the second alternative, calling for the construction of two detention basins, would be included in the refined recommended flood control plan.

The refined flood control plan for Underwood Creek and Dousman Ditch is shown on Map 177 and includes the construction of two stormwater detention basins along Dousman Ditch upstream of Gebhardt Road as well as structure floodproofing and elevation along Underwood Creek in the City of Brookfield and the Village of Elm Grove. The peak flood profile attendant to planned land use and channel conditions in the Underwood Creek Subwatershed is shown on Figure 85. The peak flood profile for Dousman Ditch is similarly shown on Figure 86.

The lower, or northern, detention basin would be located in the southeast one-quarter of U. S. Public Land Survey Section 22, Township 7 North, Range 20 East, with its outlet located immediately upstream of the proposed extension of W. Choctaw Trail, about 600 feet upstream of N. Gebhardt Road. This basin would cover an area of about 54 acres and would have a design capacity of about 50 acre-feet at a pool elevation of 825.2 feet above NGVD. The outlet control structure would be created by constructing about 330 feet of earthen dike beginning at N. Pilgrim Parkway and extending west across the channel. This dike would range in height from one to five feet and have side slopes of one vertical on three horizontal. Outflow from the basin would be handled by an 18-inch-diameter reinforced concrete pipe placed in this dike at the existing channel invert. This pipe would restrict outflow from the basin during larger storm events but would allow for subsequent drainage of the basin as well as the conveyance of flows along Dousman Ditch during periods of low flow. The top of the dike would be at the design pool elevation of 825.2 feet NGVD and would act as an emergency spillway for stormwater runoff in excess of 50 acre-feet. Although no inundation of N. Pilgrim Parkway is expected at the design pool elevation, there are two low points along the roadway where there would be only about 0.3 foot of freeboard. Thus, it may be desirable to raise those portions of N. Pilgrim Parkway in order to provide for greater freeboard.

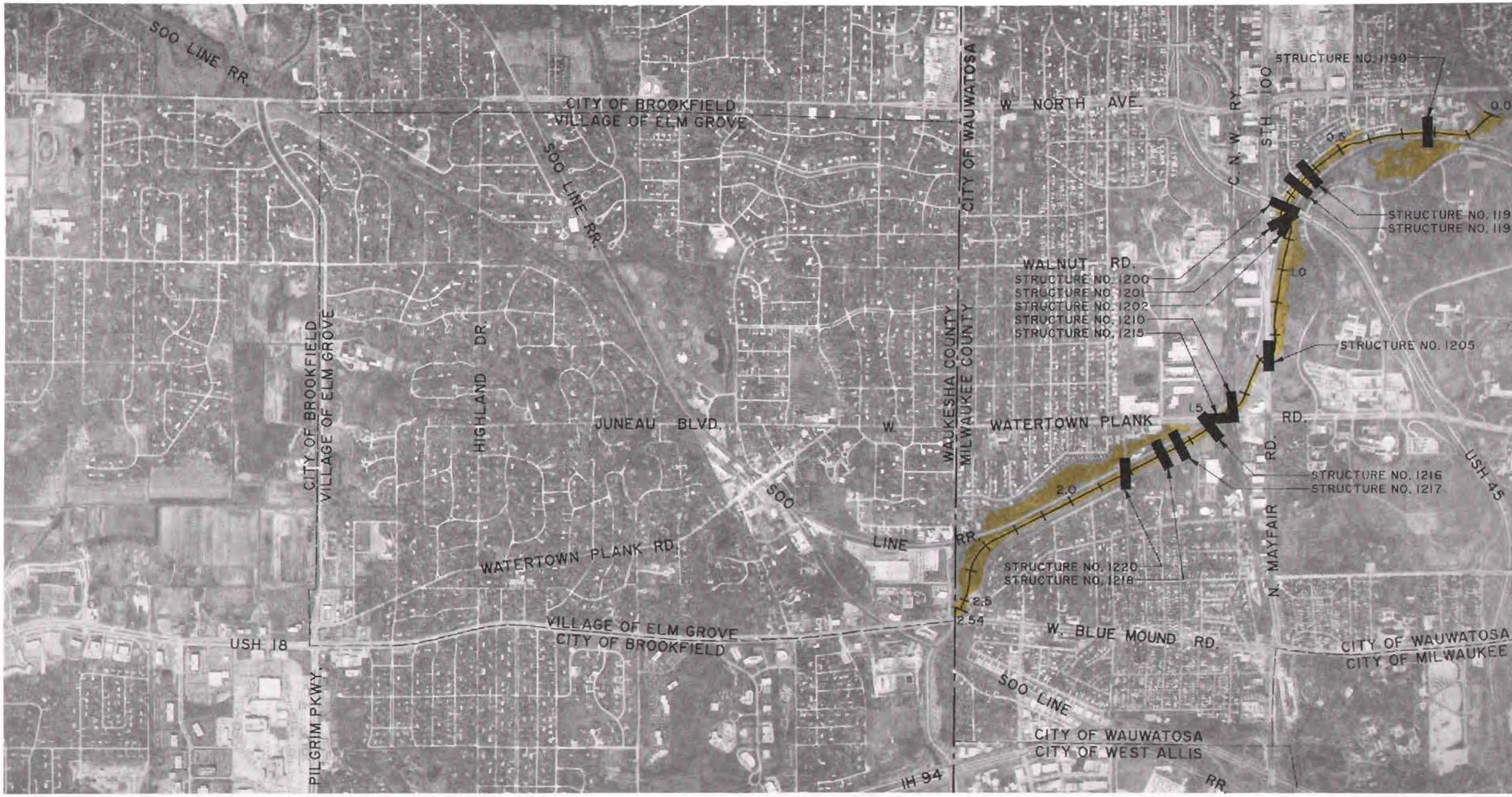
The upper, or southern, detention basin would be located in the northern one-half of U. S. Public Land Survey Section 27, Township 7 North, Range 20 East, between N. Pilgrim Parkway and N. Calhoun Road. This basin would cover an area of about 110 acres and would have a design capacity of about 280 acre-feet at a pool elevation of 830.2 feet above NGVD. It would be created by constructing about 3,700 feet of earthen dikes ranging in height from one to seven feet with side slopes of one vertical on three horizontal. The outlet from the basin would consist of an 18-inch-diameter reinforced concrete pipe located at River Mile 1.36, or about 500 feet upstream of the private drive at the Dousman-Dunkel Inn. An emergency spillway at the design pool elevation of 830.2 feet NGVD would be constructed at this location to handle stormwater runoff in excess of 280 acre-feet. About 1,300 feet of N. Pilgrim Parkway would be raised an average of 1.5 feet in order to prevent overtopping at the design pool elevation and to provide a minimum freeboard of two feet.

No stormwater pumping facilities would be required for either of these two basins. Drainage to the basins would be accomplished by gravity flow along existing drainageways. By leaving these drainageways open, some inundation of currently developed land, but no buildings, would occur. A total of 10 properties would be

Map 177

REFINED RECOMMENDED FLOOD CONTROL SYSTEM PLAN FOR UNDERWOOD CREEK AND DOUSMAN DITCH

UNDERWOOD CREEK



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN - YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS

2.5 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPGRAPHIC MAPPING FOR UNDERWOOD CREEK IS SHOWN IN APPENDIX H

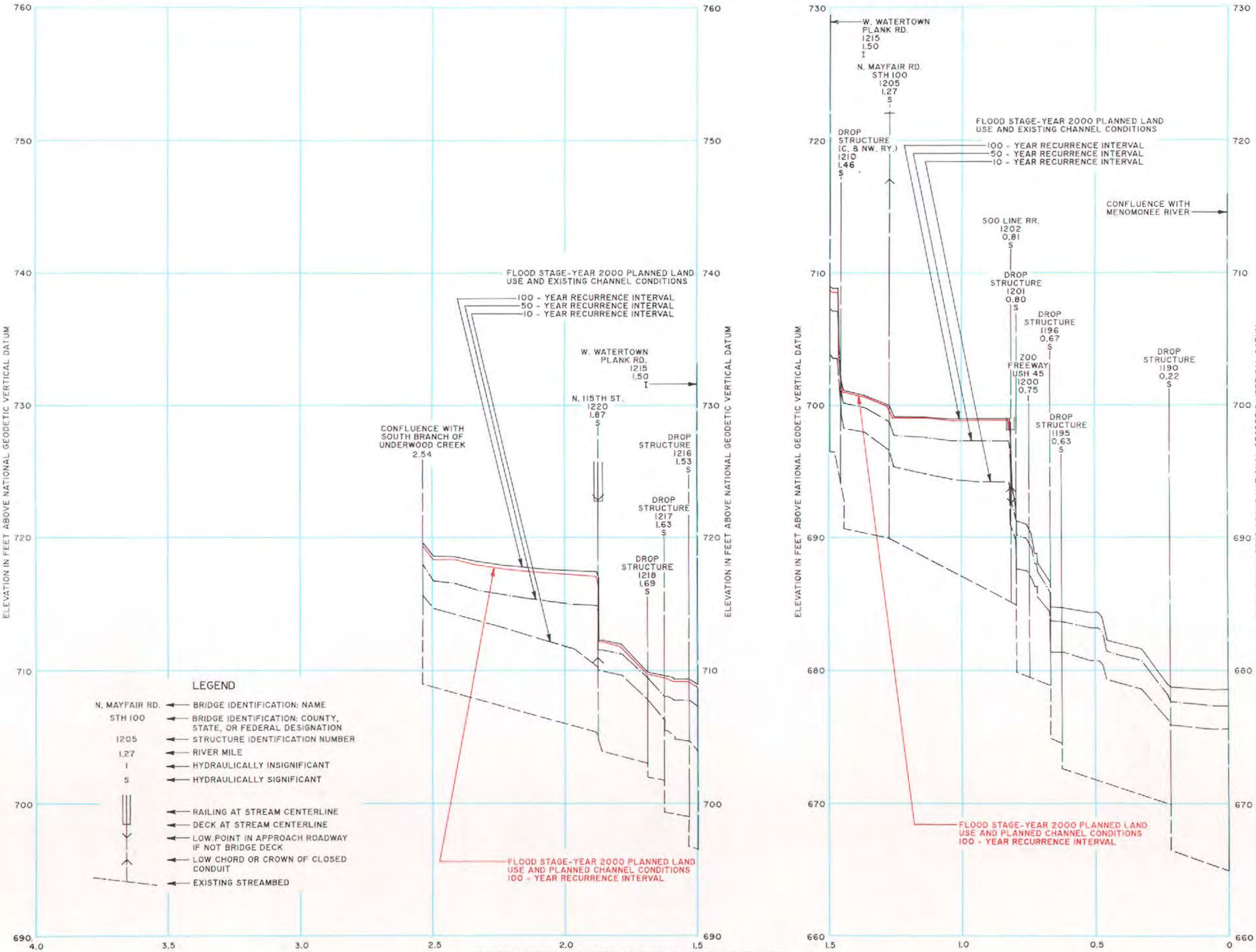
GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY APRIL 1986

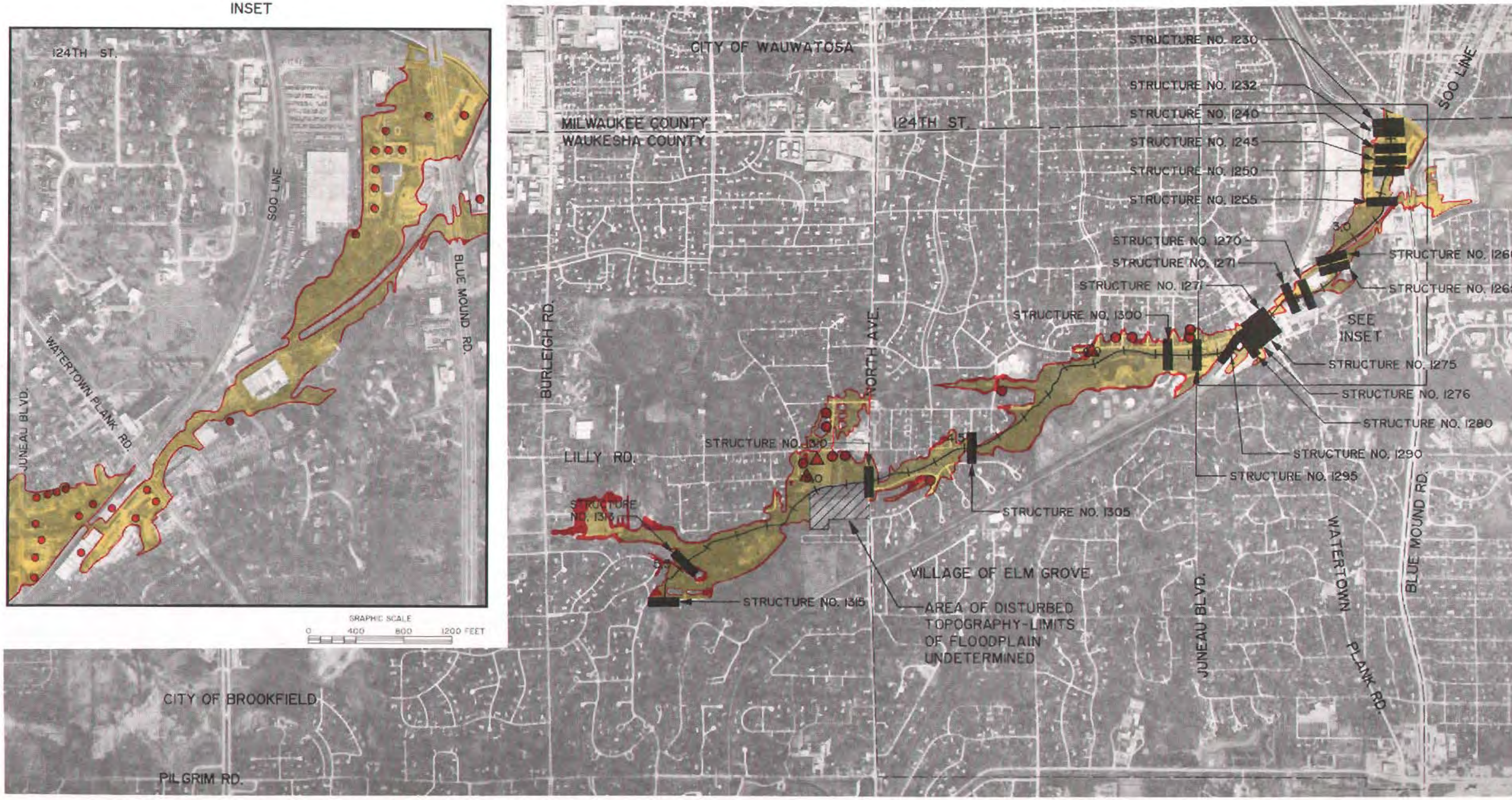
Figure 85

RECOMMENDED PLAN FLOOD STAGE PROFILE FOR UNDERWOOD CREEK



Map 177 (continued)

UNDERWOOD CREEK (continued)



LEGEND

- 100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS
- 100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS
- 4.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING
- STRUCTURE TO BE FLOODPROOFED
- STRUCTURE TO BE ELEVATED

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR UNDERWOOD CREEK IS SHOWN IN APPENDIX H

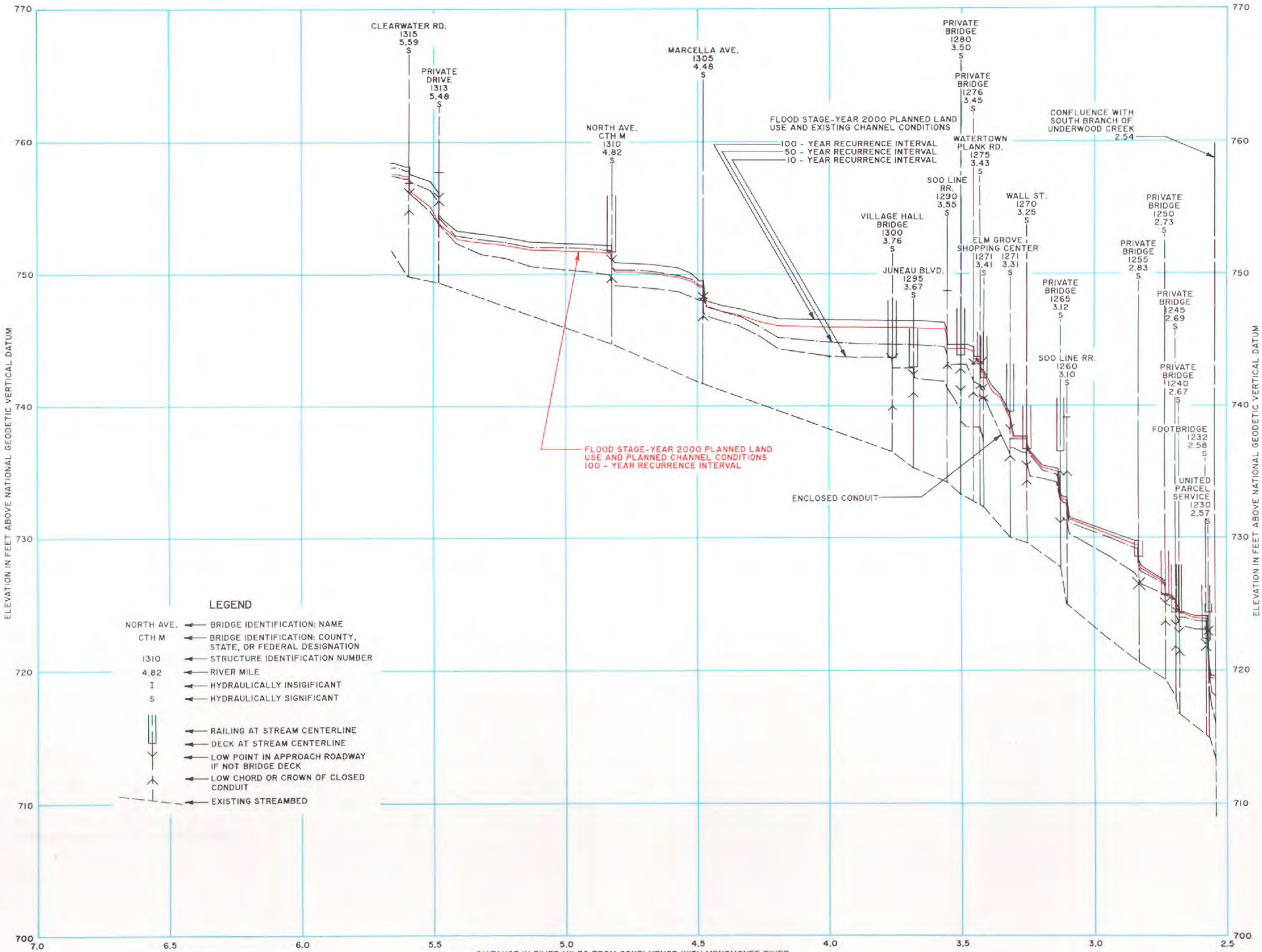
DUE TO MAP SCALE LIMITATIONS, THE DIFFERENCE BETWEEN THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS, AND THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS, MAY NOT APPEAR ON THIS MAP. WHERE NO DIFFERENCE APPEARS REFERENCE SHOULD BE MADE TO THE FLOOD STAGE PROFILE SHOWN BELOW

GRAPHIC SCALE

0 1/2 1 MILE

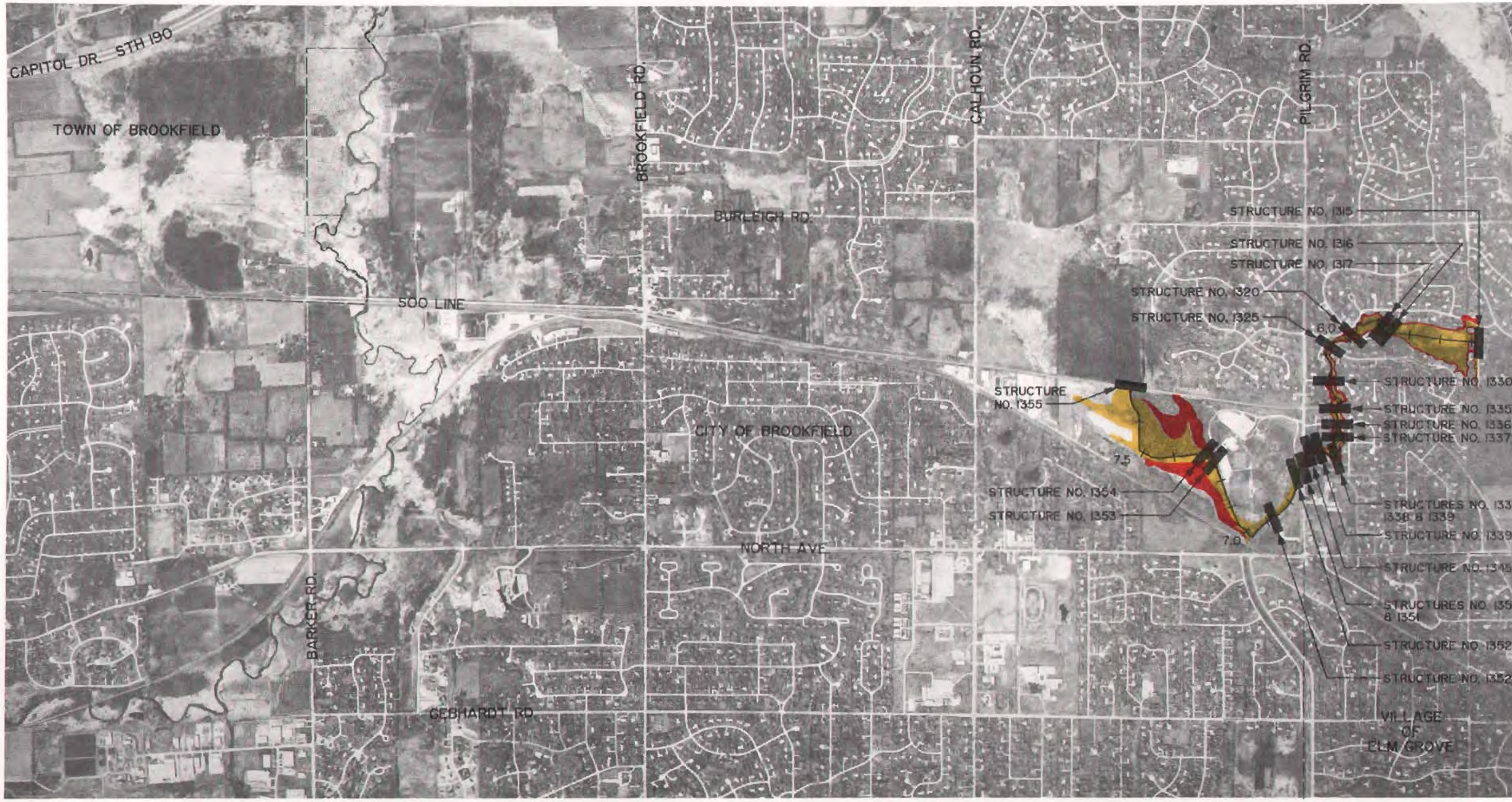
DATE OF PHOTOGRAPHY: APRIL 1986

Figure 85 (continued)



Map 177 (continued)

UNDERWOOD CREEK (continued)



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS

7.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

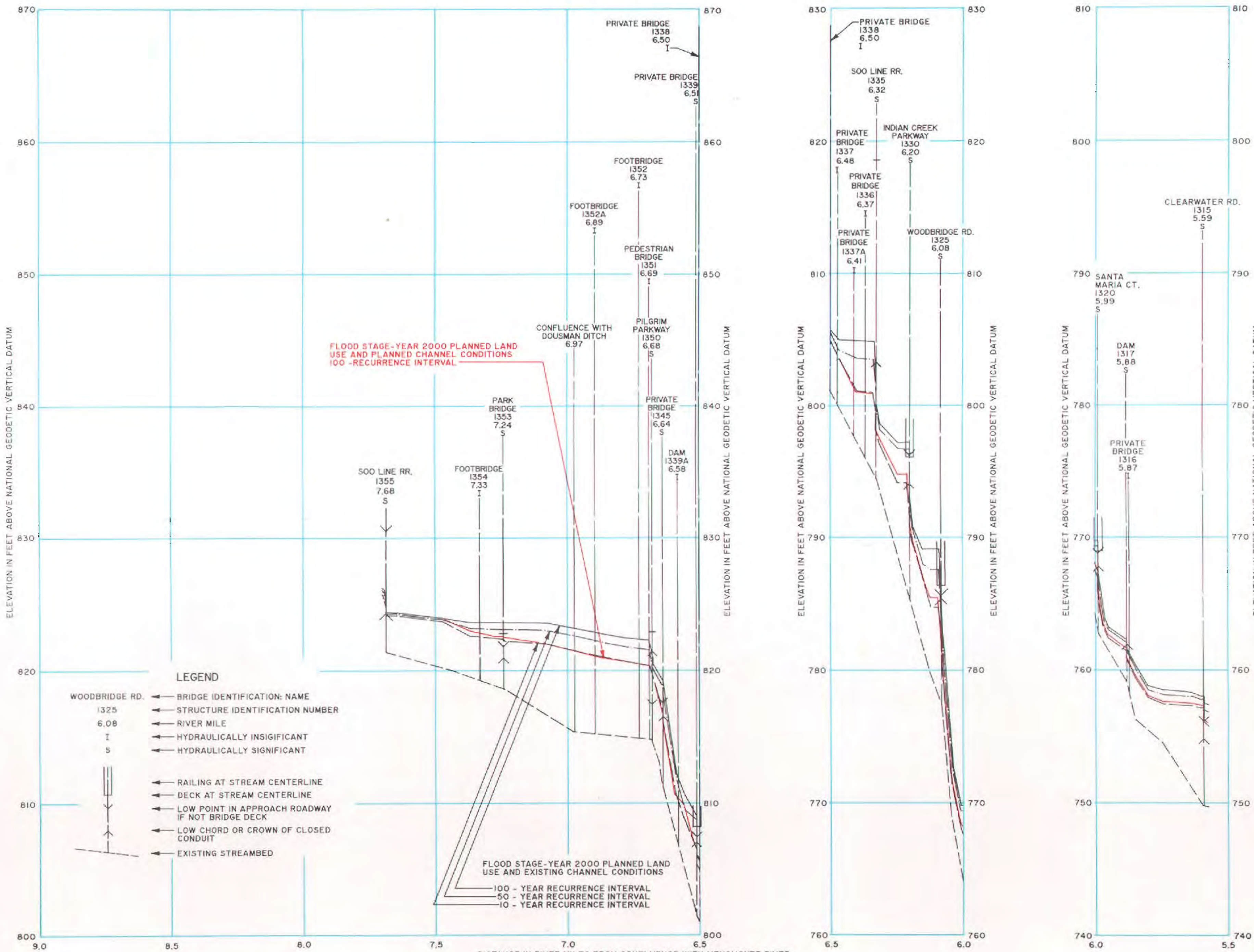
NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR UNDERWOOD CREEK IS SHOWN IN APPENDIX H

GRAPHIC SCALE

0 1/2 1 MILE

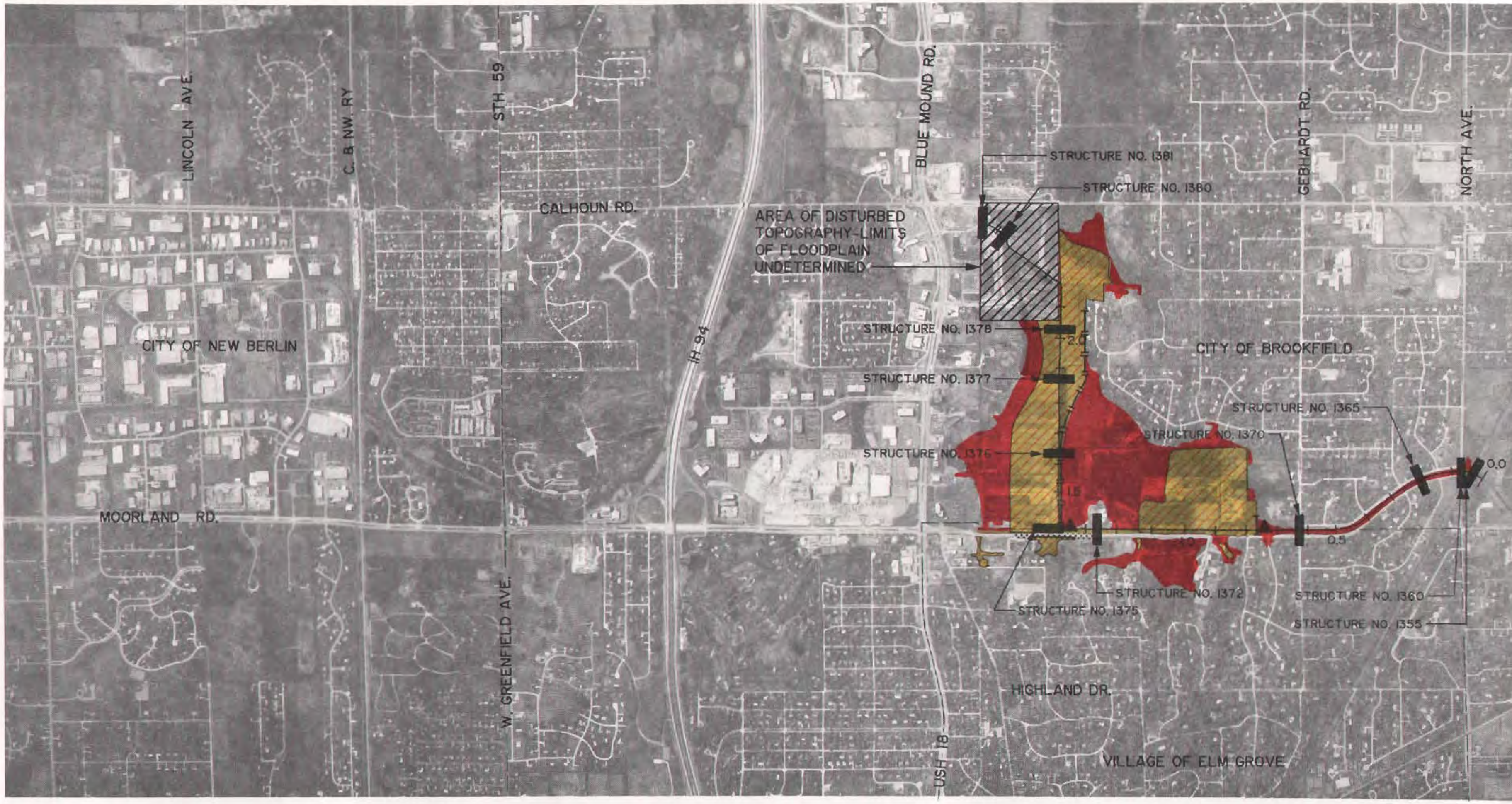
DATE OF PHOTOGRAPHY: APRIL 1986

Figure 85 (continued)



Map 177 (continued)

DOUSMAN DITCH



LEGEND

- 100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS
- 100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS
- 1.5 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING
- DETENTION BASIN
- EARTHEN DIKE
- OUTLET STRUCTURE
- ROADWAY ELEVATION

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR DOUSMAN DITCH IS SHOWN IN APPENDIX H

THE DIFFERENCE BETWEEN THE EXISTING AND PLANNED CHANNEL FLOODPLAIN IN THOSE AREAS WHICH ARE NOT PROTECTED BY DIKES AND ARE ADJACENT TO THE RECOMMENDED UPSTREAM DETENTION BASIN IS NOT DUE TO A REDUCTION IN FLOOD STAGES. THE DIFFERENCE REFLECTS LOCAL LAND USE PLANS WHICH CALL FOR DEVELOPMENT IN THOSE AREAS SHOWN AS BEING REMOVED FROM THE FLOODPLAIN UNDER PLANNED CONDITIONS. THIS SYSTEM PLAN ASSUMES THAT WHEN THOSE AREAS ARE DEVELOPED, THEY WILL BE FILLED TO A MINIMUM LEVEL OF 2 FEET ABOVE THE 100-YEAR RECURRENCE INTERVAL FLOOD STAGE UNDER PLANNED LAND USE AND CHANNEL CONDITIONS.

GRAPHIC SCALE

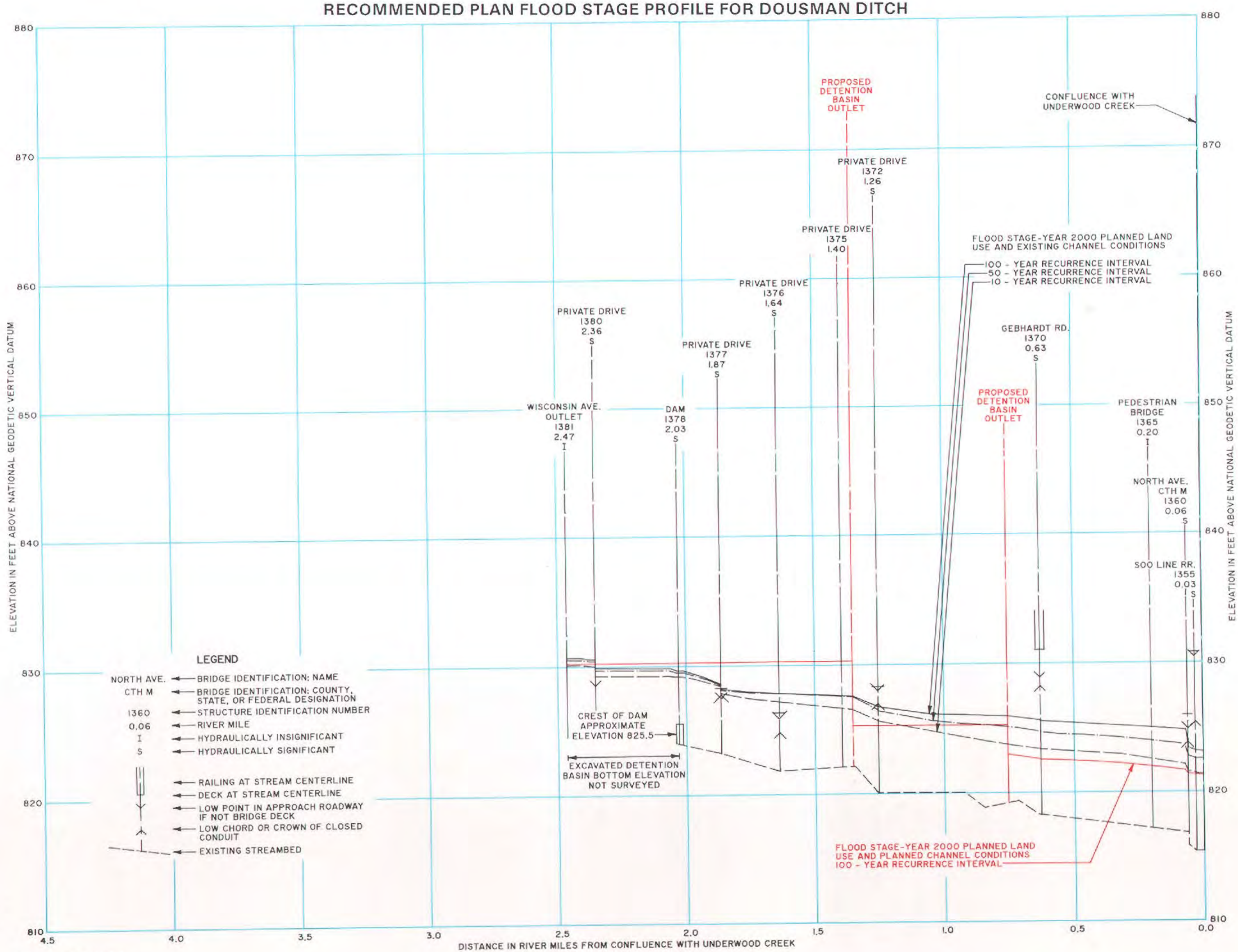
0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Source: SEWRPC.

Figure 86

RECOMMENDED PLAN FLOOD STAGE PROFILE FOR DOUSMAN DITCH



Source: SEWRPC.

affected along the lower basin, although the design pool elevation is about 0.8 foot below the 100-year recurrence interval flood level under planned land use and existing channel conditions along these properties. A total of four properties would be affected along the upper detention basin. The design pool elevation would be about 2.5 feet above the 100-year flood level under planned land use and existing channel conditions at these properties. At those locations where the 100-year flood level would be increased due to construction of the detention basin, it may be necessary to obtain flood easements from the property owners affected. As the nature of these easements can vary on an individual basis, no cost has been assigned to such easements in the recommended plan.

Construction of the two detention basins would reduce the number of structures expected to incur direct flood damages under a 100-year recurrence interval flood along Underwood Creek from 76 to 41. Under this refined flood control plan, damages to these remaining structures would be eliminated by a combination of floodproofing and elevation. In the case of residential structures, floodproofing was assumed to be feasible if the design flood stage was below the first floor elevation. Structure elevation was considered feasible for residential structures with basements if the estimated cost of elevating the structure and floodproofing the basement was less than the estimated removal cost. Structures to be elevated were assumed to have the first floor raised to an elevation of at least two feet above the 100-year recurrence interval flood stage to provide adequate freeboard. For aesthetic reasons, structure elevation was limited to a maximum of four feet. Structures which would have to be raised more than four feet were considered for removal. Floodproofing was considered feasible for all nonresidential structures provided the flood stage was not more than seven feet above the first floor elevation. Floodproofing costs for nonresidential structures were assumed to be a function of the depth of water over the first floor.

Forty-one structures would still be expected to incur flood damages under a 100-year recurrence interval flood; however, the flood damage would be significantly less than would be expected under existing conditions. Thirty-eight of the 41 structures would have to be floodproofed and three would have to be elevated. No structures would have to be removed.

No bridge or culvert replacement is included in the refined recommended plan. The Menomonee River watershed study included a recommendation for replacement of the Soo Line (former Chicago, Milwaukee, St. Paul & Pacific Railroad) railway bridge located at River Mile 6.32. The refined floodplain analysis conducted under this system plan indicates that only two houses would be expected to incur damages under a 100-year recurrence interval flood due to the backwater effect from this bridge. Furthermore, construction of the two detention basins would remove these two houses from the 100-year floodplain, thus eliminating the need to replace this railway bridge.

Assuming complete implementation of these flood control measures, and utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of this flood control plan is estimated at $130,000. This cost consists of the amortization of the $672,000 capital cost for the detention basins, including land acquisition; the $970,000 cost for structure floodproofing; the $94,000 cost for structure elevation; and $20,000 in annual operation and maintenance costs. The average annual flood abatement benefit is estimated at $166,000. The resulting benefit-cost ratio is 1.28. As noted earlier in this chapter, these flood abatement benefits do not account for a reduction in potential secondary flooding. If these secondary flood damages were included, the resulting benefit-cost ratio would be higher.

Flood Control and Related Drainage System Plan Implementation

The recommended flood control plan for Underwood Creek is largely nonstructural, in that it emphasizes structure floodproofing and elevation as a means of alleviating flood damages. The structure floodproofing and elevation measures would be undertaken by the property owners directly affected. It is further recommended that the professional services required to prepare plans for the floodproofing and elevation of individual buildings be made available, at no cost, to the property owners by the City of Brookfield and the Village of Elm Grove. Also, it is recommended that these communities review their building ordinances to ensure that appropriate floodproofing regulations are included. Finally, it is recommended that these communities explore, on behalf of the property owners involved, any available state and/or federal aids for such floodproofing measures.

Table 112

SUMMARY OF RECOMMENDED PLAN CAPITAL COSTS FOR UNDERWOOD CREEK AND DOUSMAN DITCH

Implementing Agency	Flood Control Measures	Estimated Capital Cost
City of Brookfield	Detention basin	$ 168,000[a]
Village of Elm Grove	Detention basin	504,000[a]
Various Private Property Owners	Structure floodproofing Structure elevation	970,000 94,000
	Total	$1,736,000

[a] *Cost is recommended to be borne by the Milwaukee Metropolitan Sewerage District if District boundaries are expanded to include these areas.*

Source: SEWRPC.

It is recommended that the structural flood control measures recommended for Underwood Creek be implemented through the cooperative efforts of the City of Brookfield and the Village of Elm Grove. More specifically, it is recommended that the City of Brookfield and the Village of Elm Grove share the cost for the design, construction, and maintenance of the two detention basins recommended along Dousman Ditch upstream from Gebhardt Road. Although these basins would be located entirely within the City of Brookfield, flood damage abatement benefits due to these basins would be realized in both communities. It is further recommended that the costs associated with these basins be assigned in proportion to the flood damage mitigation benefits derived by each community. If, however, District jurisdiction is extended to include the sewer service areas, it is recommended that the costs for the design, construction, and maintenance of these detention basins be borne by the District.

A residential development which incorporates a portion of the recommended upstream detention storage area is currently under construction. It is recommended that, following completion of that construction, the City of Brookfield prepare a large-scale topographic map for the northwest one-quarter of U. S. Public Land Survey Section 27, Township 7 North, Range 20 East, City of Brookfield.

The capital costs associated with the recommended plan are apportioned by agency in Table 112.

SOUTH BRANCH OF UNDERWOOD CREEK SUBWATERSHED FLOOD CONTROL AND RELATED DRAINAGE PLAN

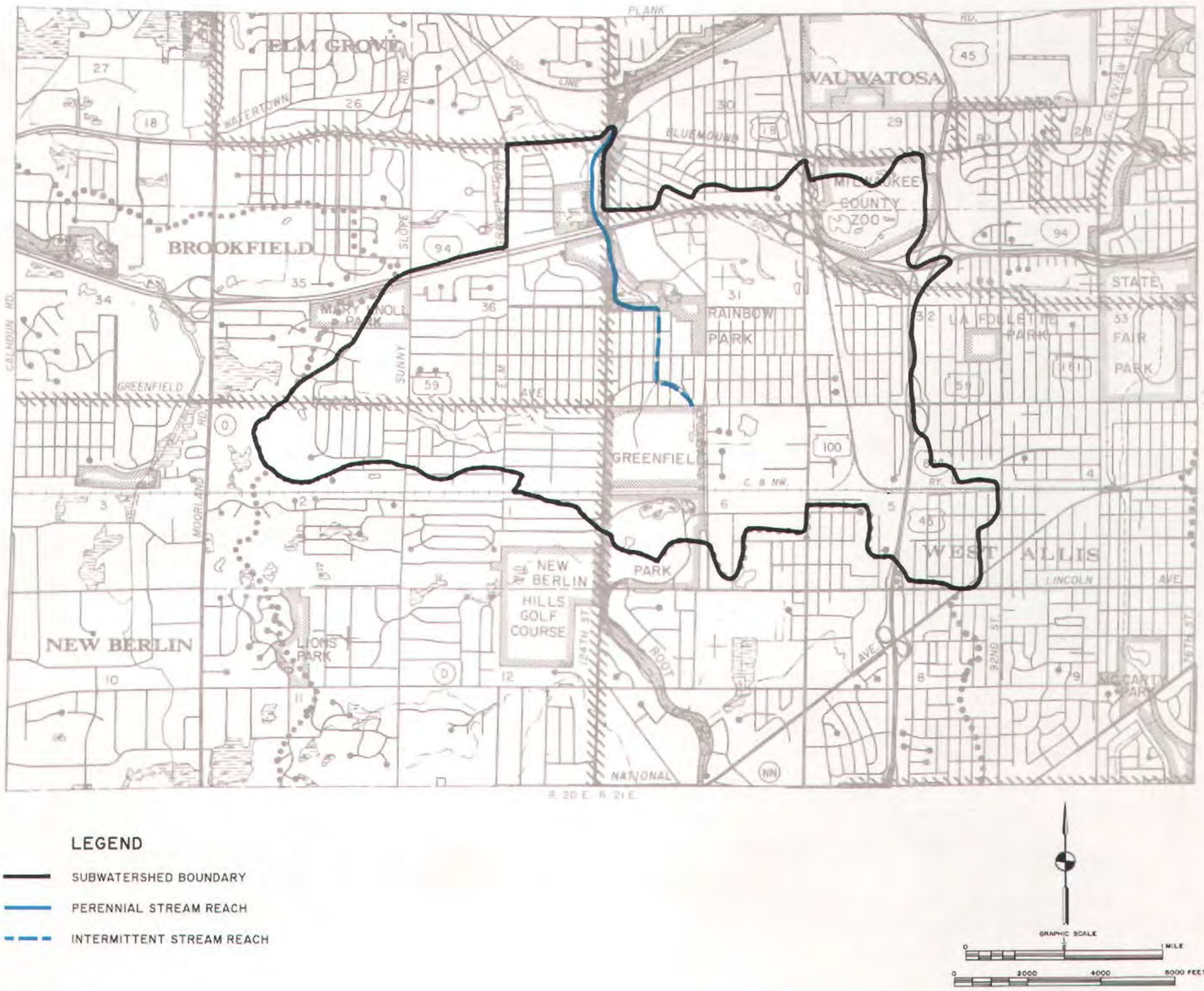
Hydrologic and hydraulic analyses of the South Branch of Underwood Creek were previously conducted under the Commission's Menomonee River watershed study. This system planning effort represents a refinement of the analyses conducted under that earlier study.

Overview of the Subwatershed

The South Branch of Underwood Creek subwatershed is located largely within west-central Milwaukee County, with portions extending into Waukesha County. The subwatershed includes portions of the Cities of Milwaukee, West Allis, Brookfield, and New Berlin. From its origin at the W. Greenfield Avenue enclosure inlet in the City of West Allis, the South Branch of Underwood Creek flows in a northerly direction for a distance of about 1.6 miles to its confluence with Underwood Creek in the City of Brookfield. The South Branch of Underwood Creek drains an area of about 5.18 square miles, as shown on Map 178. The extent of the subwatershed area within each minor civil division involved is given in Table 113.

Map 178

THE SOUTH BRANCH OF UNDERWOOD CREEK SUBWATERSHED



Source: SEWRPC.

More specifically, from its origin at Greenfield Avenue, the South Branch of Underwood Creek flows in a northerly direction in an enclosure for a distance of about 0.5 mile to Theodore Trecker Way; thence northerly in an open channel for about 0.5 mile to IH 94; continuing northerly in an open channel for about 0.6 mile across W. Blue Mound Road to its confluence with Underwood Creek. Of the 1.6-mile reach described, 1.1 miles, or about 69 percent, are classified as perennial; while the remaining 0.5 mile is classified as intermittent. The entire stream reach is recommended for District jurisdiction in the policy plan companion to this system plan.

In 1985, about 95 percent of the South Branch of Underwood Creek subwatershed was developed for urban use. About 43 percent of the urban land was in residential uses. Other urban uses included recreational, industrial, and commercial lands. The developed areas of the

Table 113

AREAL EXTENT OF CIVIL DIVISIONS IN THE SOUTH BRANCH OF UNDERWOOD CREEK SUBWATERSHED

Civil Division	Civil Division Area Included Within Subwatershed (square miles)	Percent of Subwatershed Area Within Civil Division
City of Brookfield	1.35	26.1
City of Milwaukee	0.32	6.2
City of New Berlin	0.68	13.1
City of West Allis	2.83	54.6
Total	5.18	100.0

Source: SEWRPC.

subwatershed in Milwaukee County are generally provided with a full range of municipal street improvements, including paved streets with curbs and gutters and attendant storm sewers. In Waukesha County, some of the developed lands are provided with a full range of municipal street improvements, including paved streets with curbs and gutters and attendant storm sewers, while other developed lands are provided with paved streets and road ditches, with drainage accomplished through a combination of these road ditches, storm sewers, and surface swales and watercourses. The planned land use conditions utilized in the system planning effort assume that the subwatershed will be almost entirely urbanized by the design year of the system plan.

The flood profile for the South Branch of Underwood Creek under planned land use and existing channel conditions is shown as Figure 87. The extent of the 100-year recurrence interval floodplain under planned land use and existing channel conditions is shown on Map 179.

Evaluation of Alternative Flood Control and Related Drainage System Plans for the South Branch of Underwood Creek

As previously noted in this chapter, no structure flood damages are expected to be incurred along the South Branch of Underwood Creek for floods up to and including the 100-year recurrence interval event. All flow from a 100-year event under planned land use and existing channel conditions would be contained within the present channel system. Thus, no flood control alternatives were evaluated for this stream under this system plan.

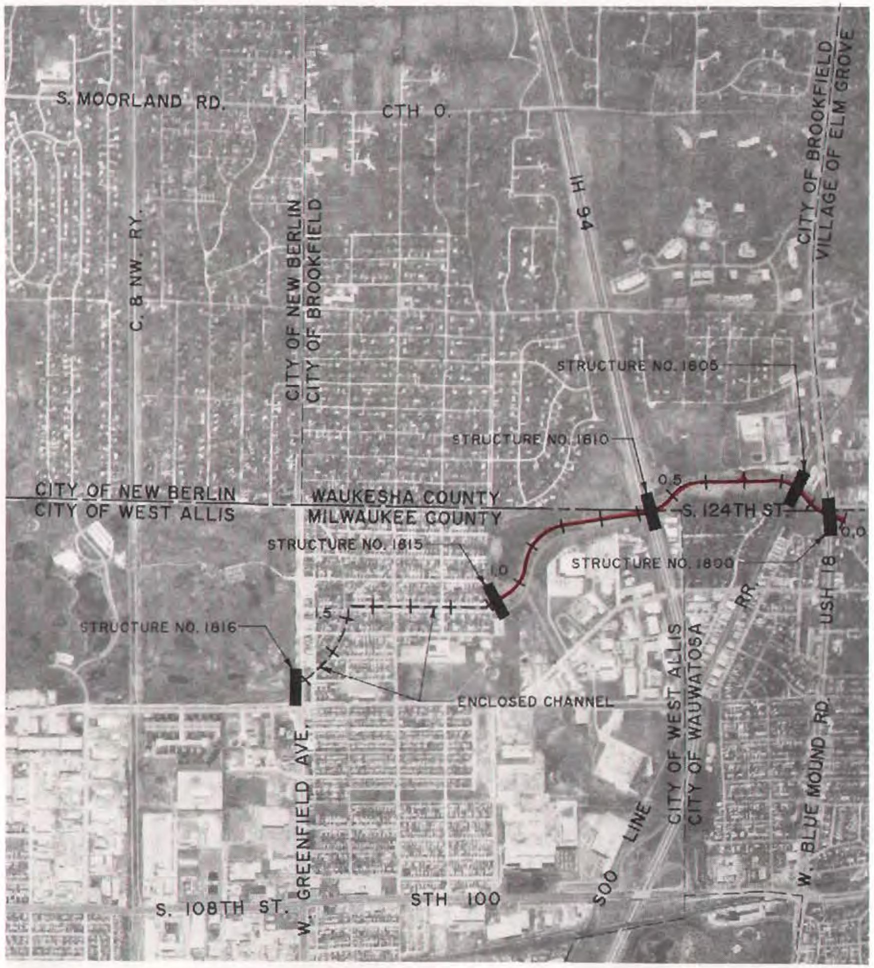
Implementation of the recommended drainage and flood control system plan for Underwood Creek and Dousman Ditch would result in a slight reduction in the 100-year recurrence interval flood flow and stage at the confluence of Underwood Creek and the South Branch of Underwood Creek under planned land use conditions. Thus, the 100-year flood profile along the lower reach of the South Branch of Underwood Creek would be lowered due to a reduction in backwater from Underwood Creek. The extent of the 100-year recurrence interval floodplain along the South Branch of Underwood Creek under planned land use and channel conditions is shown on Map 180. The peak flood profile attendant to planned land use and channel conditions is shown on Figure 88.

LITTLE MENOMONEE RIVER SUBWATERSHED FLOOD CONTROL AND RELATED DRAINAGE PLAN

Hydrologic and hydraulic analyses of the Little Menomonee River were previously conducted under the Commission's Menomonee River watershed study. This system planning effort represents a refinement of the analyses conducted under that earlier study.

Map 179

100-YEAR RECURRENCE INTERVAL FLOODPLAIN FOR THE SOUTH BRANCH OF UNDERWOOD CREEK UNDER YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

0.5 — APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPGRAPHIC MAPPING FOR THE SOUTH BRANCH OF UNDERWOOD CREEK IS SHOWN IN APPENDIX H

GRAPHIC SCALE

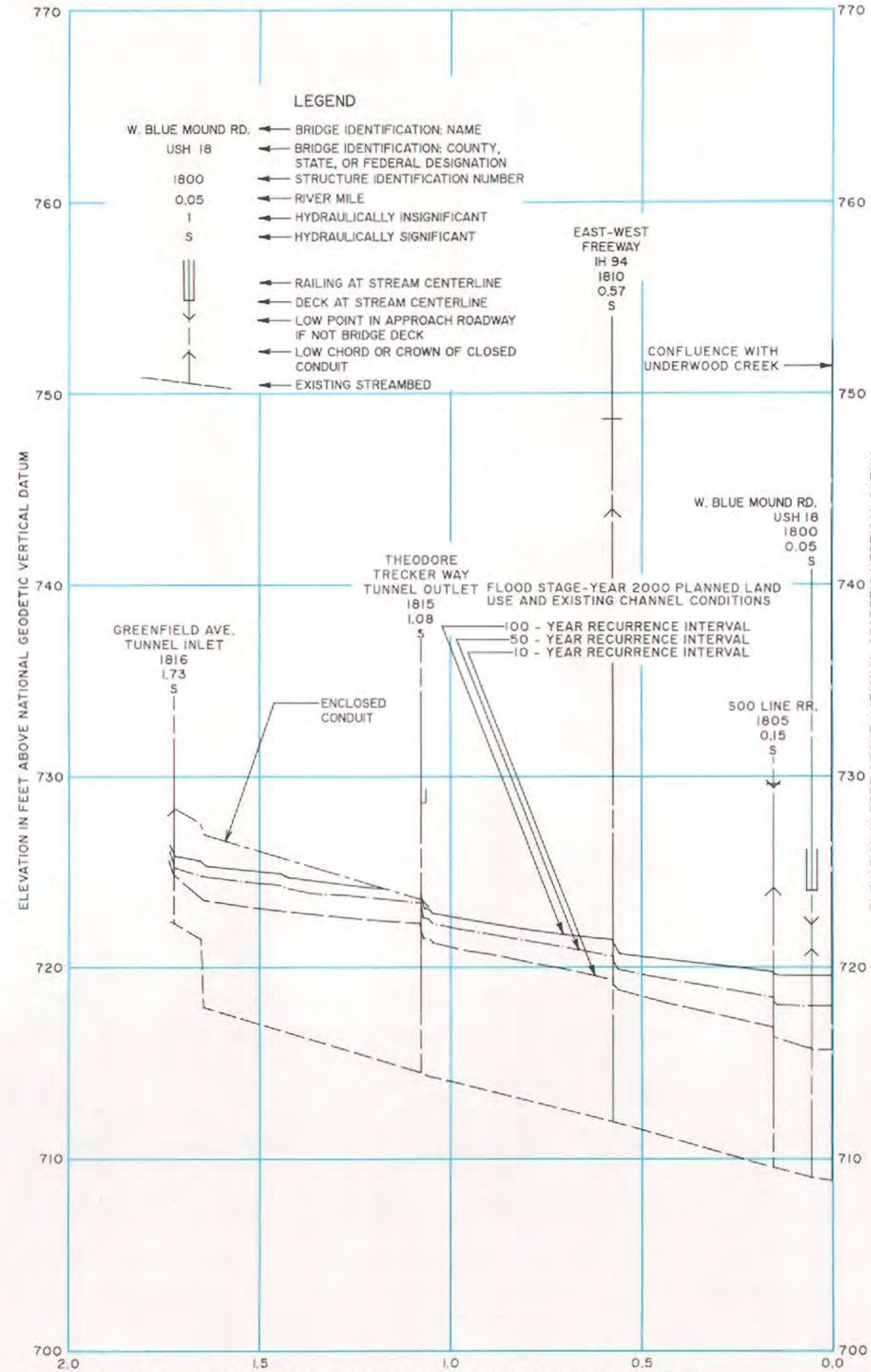
0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Source: SEWRPC.

Figure 87

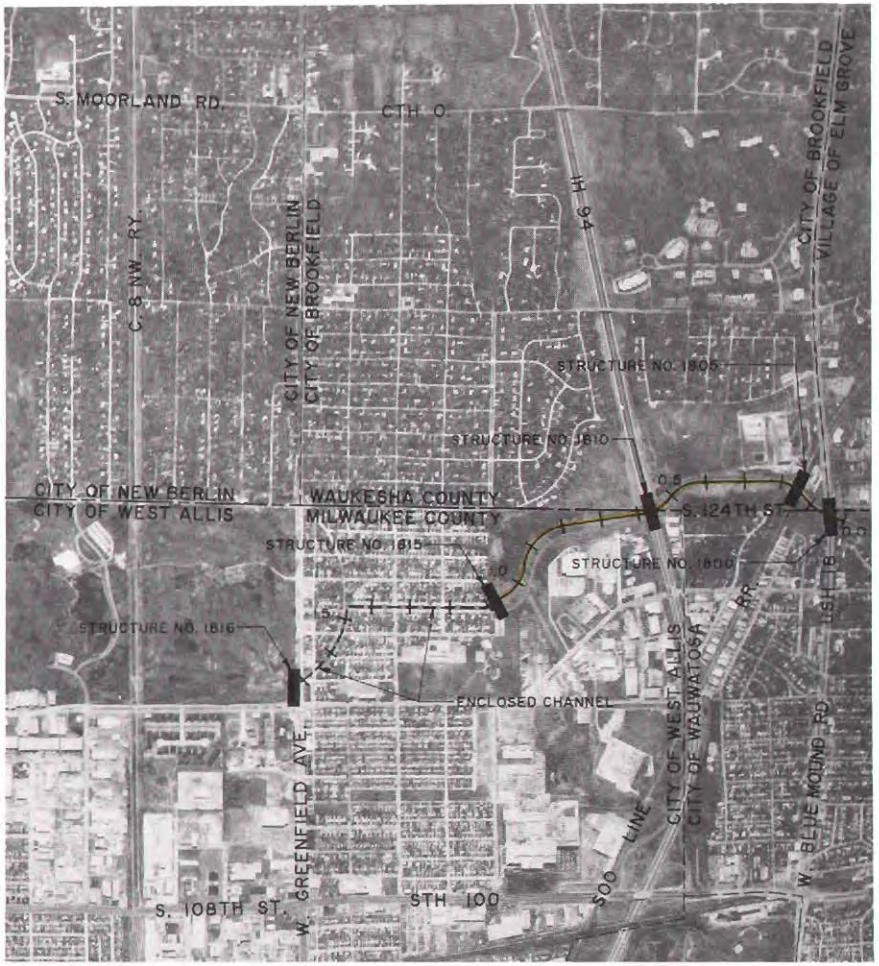
FLOOD STAGE AND STREAMBED PROFILE FOR THE SOUTH BRANCH OF UNDERWOOD CREEK



Source: SEWRPC.

Map 180

RECOMMENDED FLOOD CONTROL SYSTEM PLAN FOR THE SOUTH BRANCH OF UNDERWOOD CREEK



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN - YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS

0.5 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPGRAPHIC MAPPING FOR THE SOUTH BRANCH OF UNDERWOOD CREEK IS SHOWN IN APPENDIX H

DUE TO MAP SCALE LIMITATIONS, THE DIFFERENCE BETWEEN THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS, AND THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS, MAY NOT APPEAR ON THIS MAP. WHERE NO DIFFERENCE APPEARS REFERENCE SHOULD BE MADE TO THE FLOOD STAGE PROFILE SHOWN BELOW

GRAPHIC SCALE

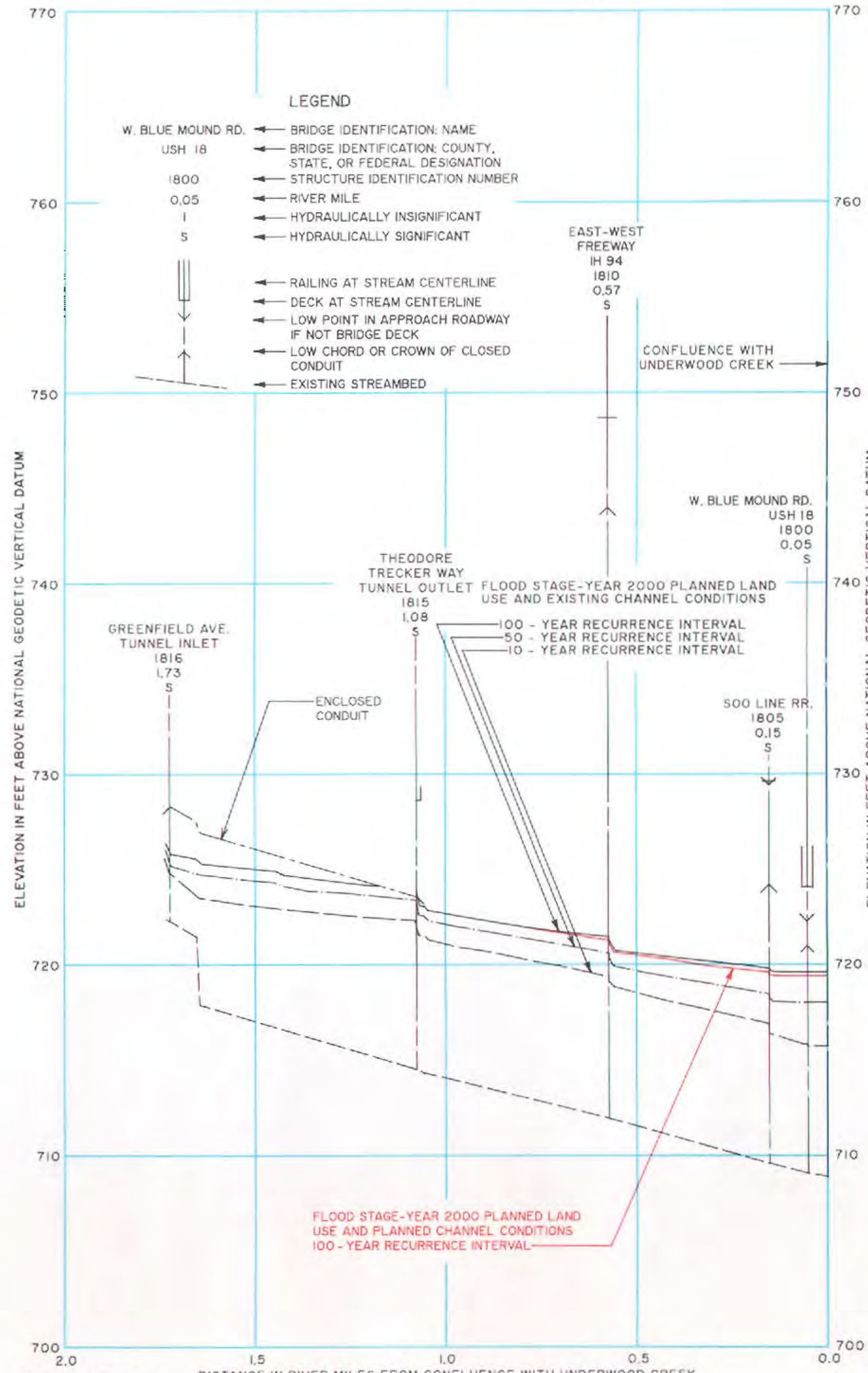
0 1/2 1 MILE

DATE OF PHOTOGRAPHY APRIL 1986

Source: SEWRPC.

Figure 88

RECOMMENDED PLAN FLOOD STAGE PROFILE FOR THE SOUTH BRANCH OF UNDERWOOD CREEK



Source: SEWRPC.

Table 114

AREAL EXTENT OF CIVIL DIVISIONS IN THE LITTLE MENOMONEE RIVER SUBWATERSHED

Civil Division	Civil Division Area Included Within Subwatershed (square miles)	Percent of Subwatershed Area Within Civil Division
City of Mequon	10.06	46.1
City of Milwaukee	11.37	52.1
Village of Germantown	0.41	1.8
Total	21.84	100.0

Source: SEWRPC.

Overview of the Subwatershed

The Little Menomonee River subwatershed is located largely within northwestern Milwaukee County and southwestern Ozaukee County, with small portions extending into Washington County. The subwatershed includes portions of the Cities of Mequon and Milwaukee and the Village of Germantown. From its origin in southwestern Ozaukee County in the City of Mequon, the Little Menomonee River flows in a generally southerly direction for a distance of 10.2 miles to its confluence with the Menomonee River in the City of Milwaukee. The Little Menomonee River drains an area of about 21.84 square miles, as shown on Map 181. The extent of the subwatershed area within each minor civil division involved is given in Table 114.

More specifically, from it origin at Freistadt Road in the City of Mequon, the Little Menomonee River flows in a southerly direction for about 3.2 miles to County Line Road at the Ozaukee-Milwaukee County line; continues southerly for about 1.1 miles to W. Brown Deer Road in the City of Milwaukee; thence southeasterly for about 1.2 miles to W. Bradley Road; thence southerly for about 1.0 mile to W. Good Hope Road; thence southwesterly for about 3.7 miles to its confluence with the Menomonee River. Of the 10.2-mile reach described, 9.7 miles are classified as perennial. The lower 7.0 miles of the stream are recommended for District jurisdiction in the policy plan companion to this system plan. The remainder of the stream lies in an area not within the current District limits or planned future District limits.

In 1985, about 43 percent of the Little Menomonee River subwatershed was developed for urban uses, while the other 57 percent remained in rural uses. Of the developed urban land, about 50 percent was used for residential purposes. Other urban uses included industrial, institutional, and open space uses. The developed areas of the subwatershed are generally provided with a full range of municipal street improvements, including paved streets with curbs and gutters and attendant storm sewers. Accordingly, surface runoff is generally conveyed quickly from most individual sites through storm sewers to the Little Menomonee River. The planned land use conditions utilized in the system planning effort assume that the watershed will be about 60 percent urbanized by the design year of the system plan.

The flood profile for the Little Menomonee River is shown as Figure 89. The extent of the 100-year recurrence interval floodplain under planned land use and existing channel conditions is shown on Map 182.

Evaluation of Alternative Flood Control and Related Drainage System Plans for the Little Menomonee River

As previously noted in this chapter, structure flood damages along the Little Menomonee River are limited to three houses in the City of Milwaukee. Additional flood damages were identified in the Menomonee River watershed study for a reach in the City of Mequon which is not recommended for District jurisdiction. That study had recommended that those flood problems be resolved through structure flood-

Map 181

THE LITTLE MENOMONEE RIVER SUBWATERSHED

Source: SEWRPC.

Map 182

100-YEAR RECURRENCE INTERVAL FLOODPLAIN FOR THE LITTLE MENOMONEE RIVER UNDER YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

2.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR LITTLE MENOMONEE RIVER IS SHOWN IN APPENDIX H

NOTE: THE 100-YEAR RECURRENCE INTERVAL FLOODPLAIN IS NOT DELINEATED IN THE AREA NORTH OF W. MILL RD. AND WEST OF N. 99TH ST. EXTENDED DUE TO THE LACK OF LARGE-SCALE TOPOGRAPHIC MAPPING.

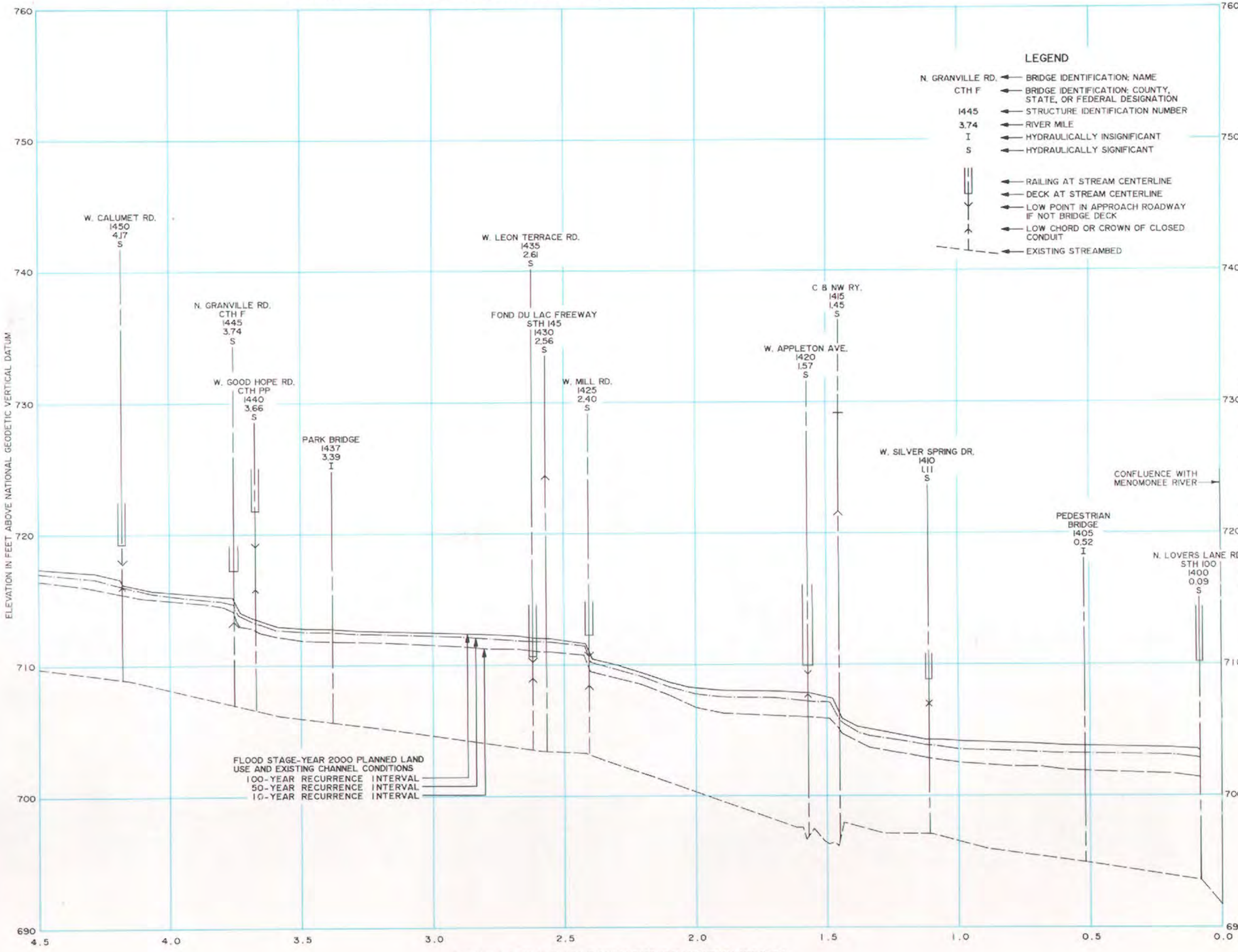
GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Figure 89

FLOOD STAGE AND STREAMBED PROFILE FOR THE LITTLE MENOMONEE RIVER



Map 182 (continued)

LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

5.5 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR LITTLE MENOMONEE RIVER IS SHOWN IN APPENDIX H

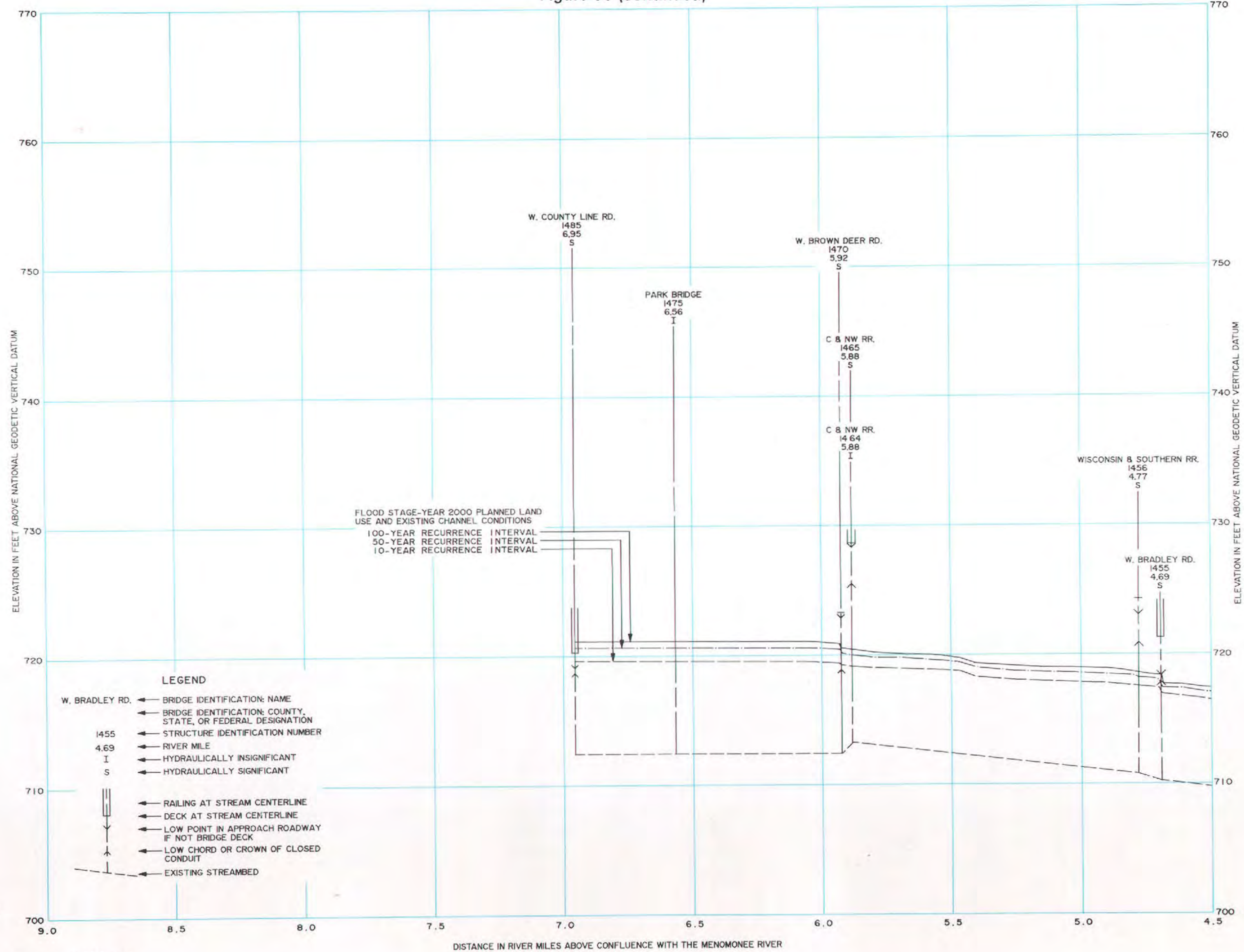
GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Source: SEWRPC.

Figure 89 (continued)



Source: SEWRPC.

proofing. Since those measures would not impact on flood flows and stages for the reach under District jurisdiction, no further analysis of flooding in the City of Mequon was made under this system plan.

Two alternative flood control plans were considered to alleviate flood problems in the City of Milwaukee: 1) Alternative Plan 1—No Action and 2) Alternative Plan 2—Structure Floodproofing, Elevation, and Removal. Each alternative is described below.

Alternative Plan 1—No Action: One alternative course of action with respect to the flood problem along the Little Menomonee River is to do nothing, that is, to recognize the inevitability of flooding but to decide not to mount a collective, coordinated program to abate the problem. Under planned year 2000 land use and existing channel conditions, the average annual flood damages would approximate $1,600. The damages from a 100-year recurrence interval event may be expected to approximate $17,077. There are no monetary benefits associated with this alternative.

Alternative Plan 2—Structure Floodproofing, Elevation, and Removal: A structure floodproofing, elevation, and removal alternative was evaluated to determine if such a structure-by-structure approach would be a technically feasible and economically viable solution to the flood problem along the Little Menomonee River in the City of Milwaukee. The 100-year recurrence interval flood stage under planned year 2000 land use and existing channel conditions was used to estimate the number of existing flood-prone structures to be floodproofed, elevated, or removed.

In the case of residential structures, floodproofing was assumed to be feasible if the design flood stage was below the first floor elevation. Structure elevation was considered feasible for residential structures with basements if the estimated cost of elevating the structure and floodproofing the basement was less than the estimated removal cost. Structures to be elevated were assumed to have the first floor raised to an elevation of at least two feet above the 100-year recurrence interval flood stage to provide adequate freeboard. For aesthetic reasons, structure elevation was limited to a maximum of four feet. Structures which would have to be raised more than four feet were considered for removal.

Floodproofing was considered feasible for all nonresidential structures provided the flood stage was not more than seven feet above the first floor elevation. Floodproofing costs for nonresidential structures were assumed to be a function of the depth of water over the first floor.

As indicated on Map 183, all three of the structures which would be expected to incur flood damages under a 100-year recurrence interval flood would have to be floodproofed. No structures would be elevated or removed.

Assuming that these structure floodproofing measures would be fully implemented, and utilizing an annual interest rate of 6 percent and a project life of 50 years, the average annual cost of this alternative is estimated at $900. This cost represents the amortization of the $14,000 capital cost of floodproofing three houses. The average annual flood damage abatement benefit was estimated at about $1,600 per year, yielding a benefit-cost ratio of 1.78.

Evaluation of Alternatives: Although it offers the lowest cost, the "no action" alternative does nothing to alleviate the existing flood problem and does not represent a sound approach to flood control.

Alternative Plan 2—Structure Floodproofing, Elevation, and Removal—presents several problems in implementation. First, complete implementation of a voluntary structure floodproofing program is unlikely, and with partial implementation, the City of Milwaukee would be left with a residual problem whenever a major flood event occurs. Also yard damages and cleanup costs would remain under this alternative. However, because the number of houses affected is small and because they are not all located together, structure floodproofing provides the only reasonable and cost effective solution to the flooding problem. Therefore, it is recommended that structure floodproofing be adopted as part of this system plan for solving the flood problem along the Little Menomonee River in the City of Milwaukee. The recommended flood control plan is shown graphically on Map 183.

As set forth in the Menomonee River watershed study, field reconnaissance sampling and laboratory analyses conducted in 1971 concluded that the Little Menomonee River bottom sediments were contaminated with creosote in the 3.75-mile reach from W. Brown Deer Road to a point about 2,000 feet downstream of the Fond du Lac Freeway (USH 145). The 1971 studies were limited to that reach; therefore, no information was available on the possible extent of creosote in the bottom sediments farther downstream.

The creosote in the streambed of the upper 0.75 mile of the contaminated reach was removed under a 1973 demonstration project. The watershed study recommended that the residual creosote pollution problem downstream of the 0.75-mile-long upper reach be resolved by excavating a new 3.46-mile-long parallel channel of similar size approximately 20 feet from the existing channel, filling the existing channel, and restoring the site.

Since the watershed study was issued, additional investigations of the extent of the creosote pollution and of alternatives to resolve the in-place pollutant problem have been conducted under the U. S. Environmental Protection Agency (EPA) "Superfund" program. Those investigations have identified creosote pollution in the entire reach of the Little Menomonee River from W. Brown Deer Road to its mouth. The U. S. Environmental Protection Agency, in cooperation with the Wisconsin Department of Natural Resources, has completed a two-part study of contamination at the Moss-American site. The remedial investigation was completed in January 1990, and the feasibility study report in May 1990. The remedial investigation identified the nature and extent of site contamination by collecting and analyzing soil, river sediment, surface water, and groundwater samples. Site geology and groundwater flow patterns were also examined. The remedial investigation confirmed that previous site activities have heavily contaminated soil and groundwater at the site, as well as sediment in the Little Menomonee River. The risk assessment element of the remedial investigation concluded that the public health risks were unacceptable. Based on the results of the remedial investigation, a feasibility study was then conducted to identify and evaluate remedial alternatives that would minimize or eliminate the health risks caused by site-related contaminants.

Six remedial action alternatives were evaluated in detail. The recommended remedial action plan for the Moss-American site is shown in graphic summary form on Map 184 and provides for the following:

- Rerouting of the Little Menomonee River from the Moss-American site to its mouth.
- Removal and biological treatment of highly contaminated soil and river sediment using an onsite treatment system.
- Burial of remaining sediments in the current streambed with soil excavated from the new channel.
- Burial of the untreated soil and the treated material from the treatment system onsite under a soil cover.
- Collection and treatment of contaminated groundwater with discharge to the sanitary sewerage system.
- Treatment of the landfilled soil onsite and disposal of it onsite in a specially designed landfill.

The recommended remedial action plan is consistent with recommendations contained in the adopted Menomonee River watershed plan which addressed this problem.[9] The remedial action plan is estimated to cost $26 million. Its annual operation and maintenance cost is $130,000, and it is expected to take three to four years to complete.

Refinement of Recommended Flood Control System Plan: During the June 14, 1990, meeting of the Technical Advisory Committee, the City Engineer of the City of Milwaukee requested that additional consideration be given to the possibility of lowering the Little Menomonee River streambed to provide adequate outlets for 13 existing storm sewers. Those storm sewer outfalls are located along a 6.5-mile-long reach of the stream beginning at a point 0.45 mile upstream of the confluence with the Menomonee River and extending upstream to W. County Line Road. The storm sewers were designed and constructed under the assumption that the Little

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[9]*See SEWRPC Planning Report No. 26, A Comprehensive Plan for the Menomonee River Watershed, Volume Two, Alternative Plans and Recommended Plan, October 1976, pp. 205-213 and p. 263.*

Map 183

RECOMMENDED FLOOD CONTROL PLAN FOR THE LITTLE MENOMONEE RIVER

LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS

2.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

STRUCTURE TO BE FLOODPROOFED

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR LITTLE MENOMONEE RIVER IS SHOWN IN APPENDIX H

NOTE: THERE IS NO CHANGE BETWEEN THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS, AND THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS. SEE FIGURE 89 ON PAGE 603 FOR THE FLOODSTAGE AND STREAMBED PROFILES FOR LITTLE MENOMONEE RIVER.

THE 100-YEAR RECURRENCE INTERVAL FLOODPLAIN IS NOT DELINEATED IN THE AREA NORTH OF W. MILL RD. AND WEST OF N. 99TH ST. EXTENDED DUE TO THE LACK OF LARGE-SCALE TOPOGRAPHIC MAPPING.

GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Map 183 (continued)

LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS

5.5 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

STRUCTURE TO BE FLOODPROOFED

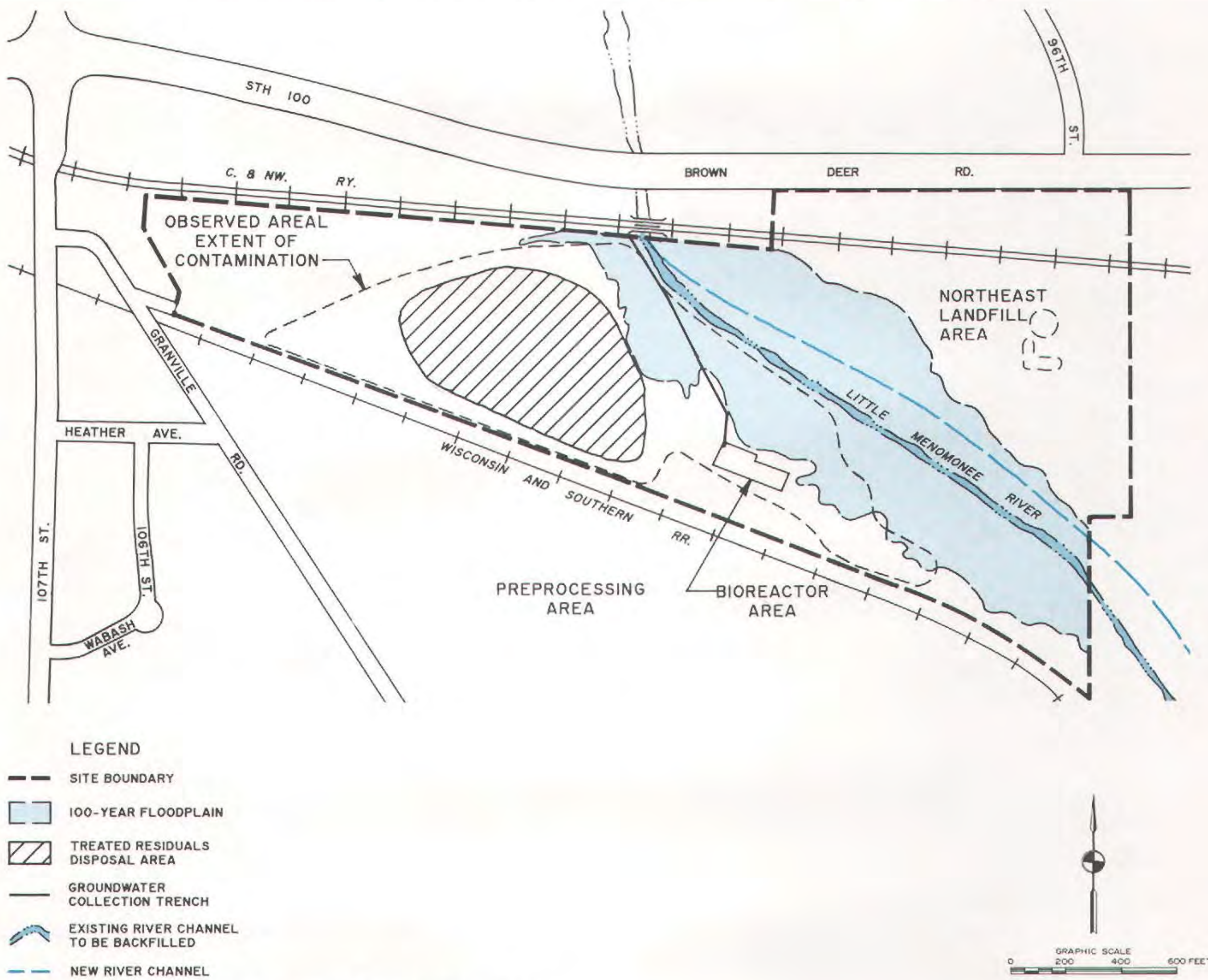
NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR LITTLE MENOMONEE RIVER IS SHOWN IN APPENDIX H

NOTE: THERE IS NO CHANGE BETWEEN THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS, AND THE 100-YEAR RECURRENCE INTERVAL FLOODLANDS UNDER PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS. SEE FIGURE 89 ON PAGE 605 FOR THE FLOODSTAGE AND STREAMBED PROFILES FOR LITTLE MENOMONEE RIVER.

Source: SEWRPC.

Map 184

RECOMMENDED REMEDIAL ACTION PLAN FOR THE MOSS-AMERICAN SITE



Source: U. S. Environmental Protection Agency, and SEWRPC.

Menomonee River channel would be widened and deepened at a future date in accordance with a preliminary design prepared in 1961 by the predecessor agency of the Milwaukee Metropolitan Sewerage District. Because most of the floodplain has been preserved in park and open space uses, modification of the stream channel is not now required for flood control purposes under planned land use conditions, and the initial recommendation to floodproof three buildings is retained. However, to insure adequate functioning of existing City storm sewers, which serve areas of existing as well as planned urban development, it is recommended that the Little Menomonee River streambed be lowered to accommodate those storm sewer outfalls.

The streambed lowering could readily be accomplished as an addition to the U. S. Environmental Protection Agency remedial action plan, which calls for realignment of the stream from the Moss-American site just south of Brown Deer Road to its confluence with the Menomonee River. The approximate streambed profile required to provide adequate storm sewer outlets is shown in Figure 90. The final profile would have to be determined during the design phase of the channel realignment project recommended by the U. S. Environmental Protection Agency. The profile is intended to provide adequate outlets for storm sewers discharging directly to the Little Menomonee River and also to enable the provision of hydraulically adequate outlet channels between certain outfalls and the River. As indicated on Figure 90, the distance between off-channel outfalls and the existing river channel ranges from about 425 feet to about 1,900 feet. Many of the existing channels conveying flow from those outfalls to the River are presently constructed without a bed slope, or a with a very flat bed slope, causing sediment to collect and restrict the outfalls. In addition, because the pipe invert elevations at the outfalls are below grade, the outfall pipes are constructed with a reverse slope, which causes sediment to accumulate in the storm sewer, reducing the hydraulic capacity of the sewer.

The combined channel realignment recommended by the U. S. Environmental Protection Agency and the channel deepening recommended by this system plan should be accomplished in such a manner that 100-year recurrence interval flood flows in the stream, and downstream in the Menomonee River as well, will not be increased. This would require that the existing hydraulic capacity and floodplain storage characteristics of the Little Menomonee River be maintained. The most direct means of assuring that flood flows are not significantly increased would be to design the realignment and deepening project to maintain the existing 100-year recurrence interval flood profile under planned land use and channel conditions. If such a constraint cannot be achieved in certain reaches of the stream, compensatory flood storage volume should be provided to replace the floodplain storage lost due to a lowered water surface profile.

Flood Control and Related Drainage System Plan Implementation: The recommended flood control system plan for the Little Menomonee River consists of floodproofing three houses. It is recommended that these floodproofing measures be undertaken by the property owners directly affected. It is further recommended that the professional services required to prepare plans for the floodproofing be made available, at no cost, to the property owners by the City of Milwaukee through its Engineering Department. Also, it is recommended that the City of Milwaukee review its building ordinance to ensure that appropriate floodproofing regulations are included. Finally, it is recommended that the City explore, on behalf of the property owners involved, any available state and/or federal aids for such floodproofing measures.

BUTLER DITCH SUBWATERSHED FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN

Hydrologic and hydraulic analyses of Butler Ditch were previously conducted under the Commission's Menomonee River watershed study and under the federal flood insurance studies for the City of Brookfield and the Village of Menomonee Falls. This system planning effort represents a refinement of those earlier analyses.

Overview of the Subwatershed

The Butler Ditch subwatershed is located entirely within east-central Waukesha County. The subwatershed includes portions of the City of Brookfield and the Village of Menomonee Falls. From its origin in southern Menomonee Falls, the Butler Ditch flows in a southerly direction into the City of Brookfield, where it changes course to flow in a northeasterly direction for a total distance of about 4.0 miles to its confluence with the Menomonee River. The Butler Ditch subwatershed drains an area of about 5.40 square miles, as shown on Map 185. The extent of the subwatershed area within each minor civil division involved is given in Table 115.

More specifically, from its origin in southern Menomonee Falls, the Butler Ditch flows in a southerly direction for a distance of about 1.5 miles to its confluence with the South Branch of Butler Ditch; thence easterly for about 0.7 mile to Lilly Road; thence northerly for about 1.8 miles to its confluence with the Menomonee River. Of the 4.0-mile reach described, 3.7 miles are classified as perennial, while 0.3 mile is classified as intermittent. The entire perennial length of Butler Ditch, which is located outside of the current District limits but within an area identified within the District policy plan as within possible future District limits, was found to meet the criteria for District jurisdiction. Moreover, any flood control measures carried out

Figure 90

RECOMMENDED APPROXIMATE STREAMBED PROFILE TO ACCOMMODATE EXISTING STORM SEWER OUTLETS ALONG THE LITTLE MENOMONEE RIVER IN THE CITY OF MILWAUKEE

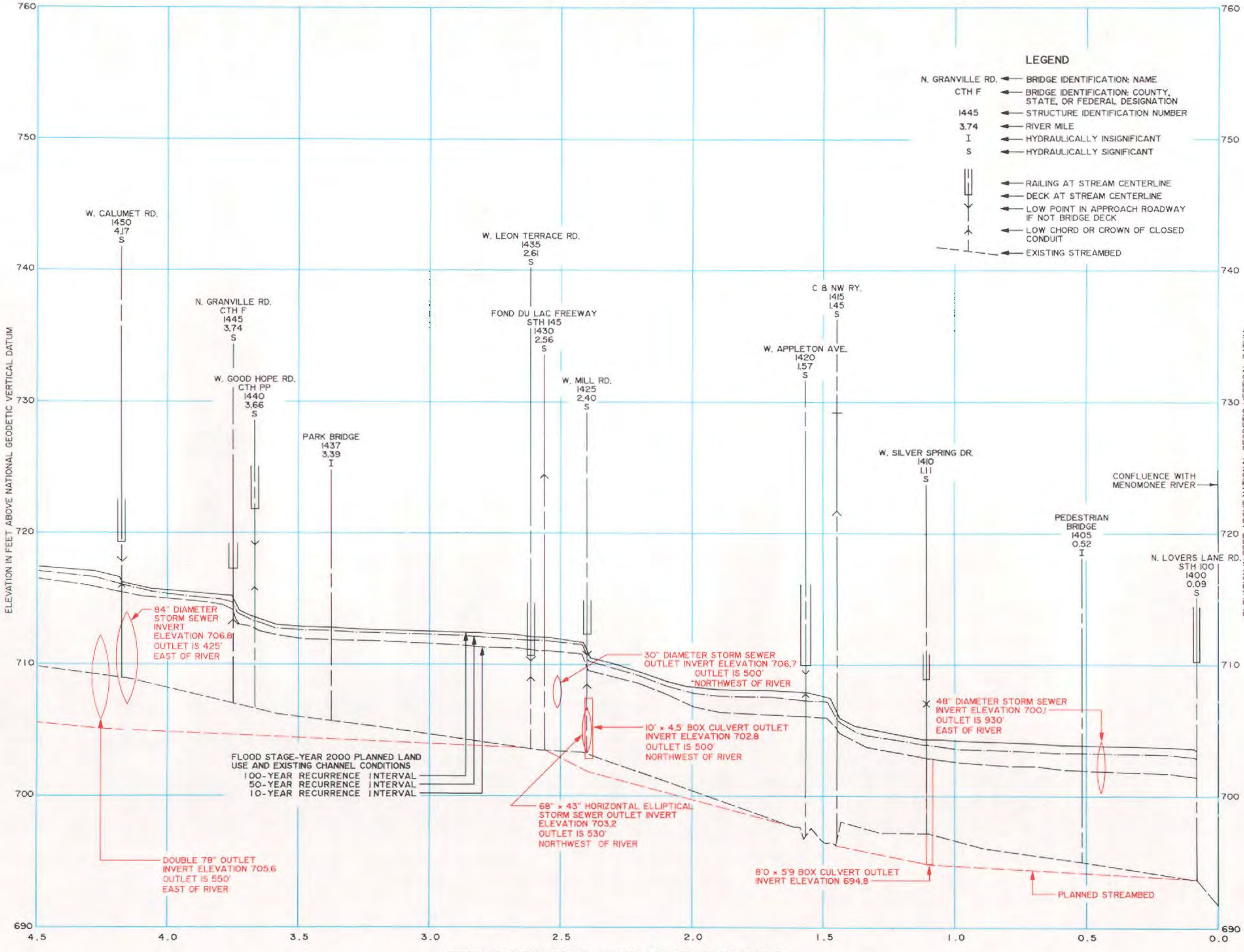
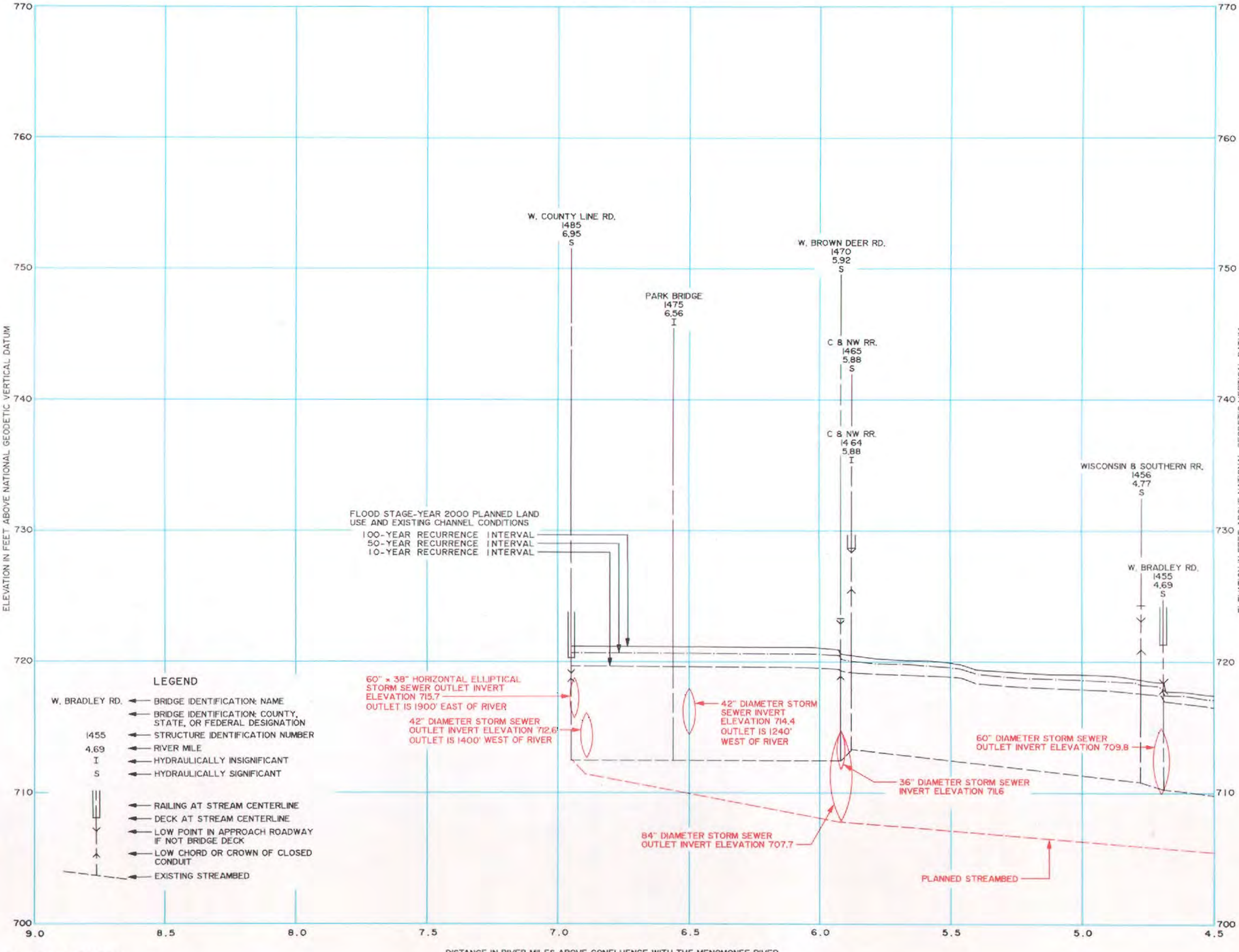


Figure 90 (continued)



Source: SEWRPC.

Table 115

AREAL EXTENT OF CIVIL DIVISIONS IN THE BUTLER DITCH SUBWATERSHED

Civil Division	Civil Division Area Included Within Subwatershed (square miles)	Percent of Subwatershed Area Within Civil Division
City of Brookfield	3.97	73.5
Village of Menomonee Falls	1.43	26.5
Total	5.40	100.0

Source: SEWRPC.

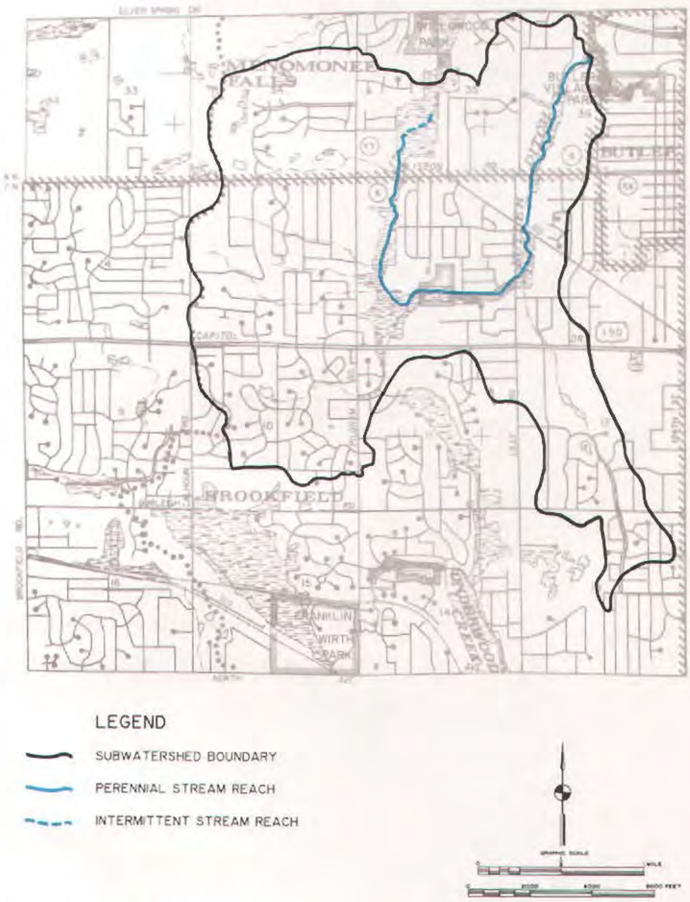
along Butler Ditch may impact flood flows and stages and recommended flood control measures along that portion of the Menomonee River under District jurisdiction. This stream was accordingly included in the system planning effort.

In 1985, about 70 percent of the Butler Ditch subwatershed was developed for urban uses, while the other 30 percent remained in rural uses. Of the urban uses, residential was predominant, comprising about 80 percent of the total urban uses. Other urban land uses included commercial, institutional, and open space uses. In Waukesha County, some of the developed areas are provided with a full range of municipal street improvements, including paved streets with curbs and gutters and attendant storm sewers, while other developed areas are provided with paved streets and road ditches. The planned land use conditions utilized in the system planning effort assume that the watershed will be about 92 percent urbanized by the design year of the system plan.

The flood profile for Butler Ditch is shown as Figure 91. The extent of the 100-year recurrence interval flood hazard area under planned land use and existing channel conditions is shown on Map 186.

Map 185

THE BUTLER DITCH SUBWATERSHED



Source: SEWRPC.

<u>Evaluation of Alternative Flood Control and Related Drainage System Plans for Butler Ditch</u>

As noted previously in this chapter, no structural flood damages are expected to occur along Butler Ditch for floods up to and including a 100-year recurrence interval event. Stormwater drainage problems, however, exist near the upstream end of Butler Ditch in the Village of Menomonee Falls. One problem involves an existing 36-inch-diameter storm sewer located in Dolphin Drive, the invert of which is located at elevation 773.5 feet NGVD, or about 2.5 feet below the existing invert of Butler Ditch, thus reducing the effective capacity of the sewer. The location of this storm sewer outfall is shown on Map 187. Another problem results from inadequate outlet grades on grassed swales in the vicinity of El Rio Drive. In order to relieve these problems, it is the intention of the Village of Menomonee Falls to lower the invert of Butler Ditch. Lowering the Butler Ditch streambed would provide an adequate outfall for the Dolphin Drive storm sewer and would provide the opportunity to improve the swale outlets near El Rio Drive through the construction of additional connecting ditches. Although the sewer outfall and swales are located along a reach of Butler Ditch which is beyond that recommended for future District jurisdiction, the proposed channel modification may impact downstream flood flows and stages. Also, it is the intent of this system plan to identify those known areas where conflicts exist between existing stormwater drainage systems and their receiving streams, and to provide a solution to those situations. Therefore, an evaluation of this channel deepening was included as part of this system planning effort.

The Butler Ditch channel modifications considered under this system plan are intended to provide the required outlets with a minimum disturbance of the existing channel. As evaluated under this study, the proposed modification along Butler Ditch would consist of lowering the existing channel bottom from 0.1 to 2.8 feet along the 0.6-mile-long reach of the Ditch between W. Lisbon Road and the existing Dolphin Drive storm sewer outfall as shown on Map 188. The resulting widened and deepened channel would have a bottom width of three feet and side slopes of one vertical on three horizontal. For purposes of this analysis it was assumed that following widening and deepening, the channel would be allowed to revert to natural vegetative cover since it lies largely within an isolated wooded area. The peak flood profile attendant to planned land use and channel conditions is shown on Figure 92.

As shown on Table 116, the recommended channel widening and deepening would not change the 100-year recurrence interval flood flows along the entire reach of Butler Ditch downstream of the Dolphin Drive storm sewer outfall. The 100-year recurrence interval flood stage would be reduced by a maximum of about 0.8 foot in the 0.43-mile-long reach of widened and deepened channel between the storm sewer outfall and W. Lisbon Road. Downstream of W. Lisbon Road the 100-year flood stage would be unchanged.

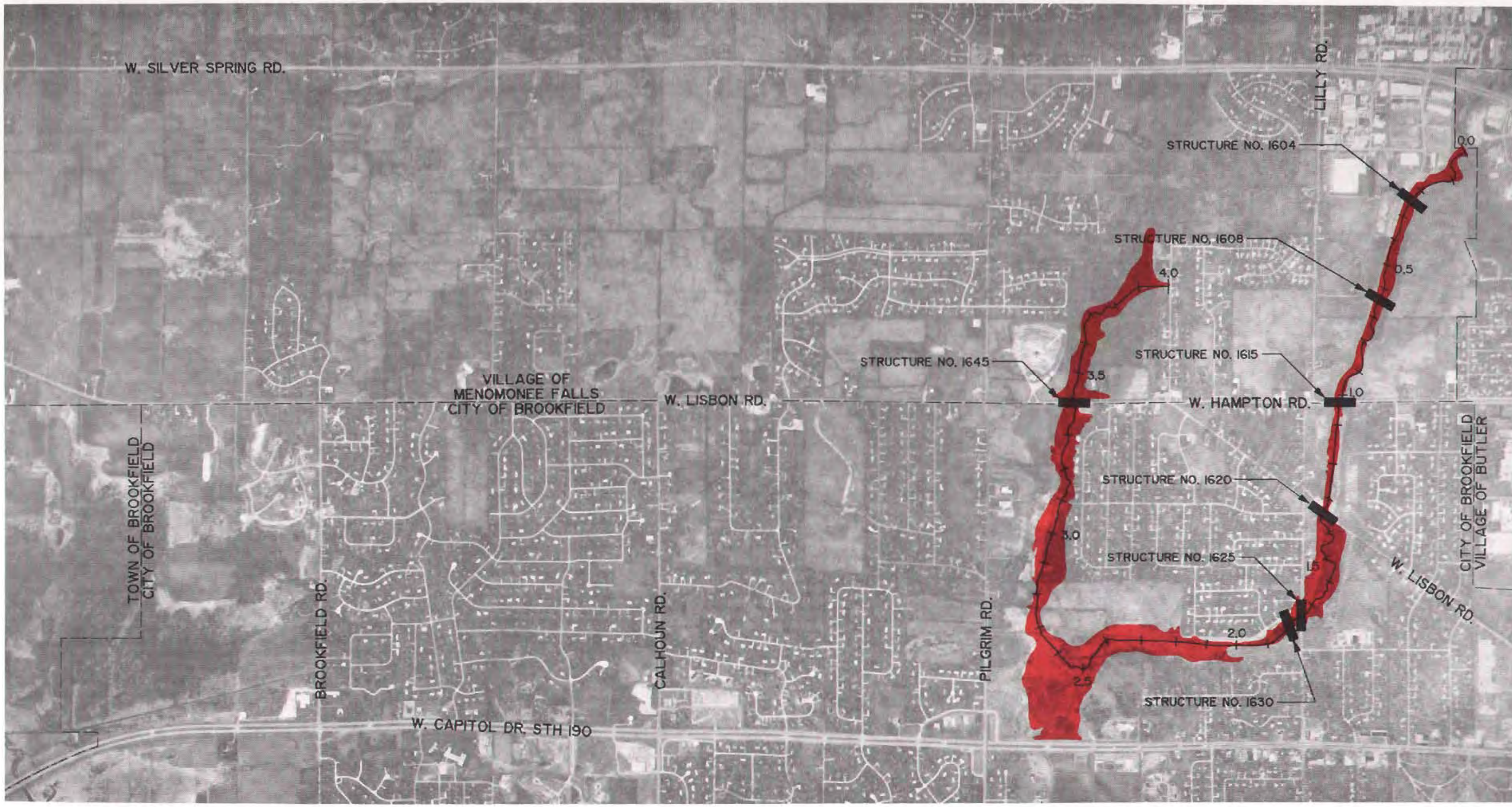
The total capital cost of this channel widening and deepening is estimated at $60,000. Utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of this project is estimated at $3,900. No benefit-cost ratio was calculated since flooding problems do not include direct overland flood damage but rather are problems which cause inconvenience and secondary damages. Such damages and the corresponding benefits for their alleviation are not typically accounted for in benefit-cost ratio calculations.

In December of 1986, the Wisconsin Department of Natural Resources granted the Village of Menomonee Falls a permit to construct a "connected enlargement" to Butler Ditch. That project would have accomplished the same stormwater drainage objectives as the channel widening and deepening recommended in this system plan. The "connected enlargement" proposal called for construction of a widened and deepened channel which would begin at the Dolphin Drive storm sewer outfall, would extend to the west along the approximate existing alignment of Butler Ditch for about 650 feet, would then leave the existing Butler Ditch alignment and continue in a westerly direction for about 1,000 feet, would then connect with a proposed 1,200-foot-long ditch coming from the north and draining the El Rio Drive area, would then run to the south for 800 feet until it again intersected the existing alignment of Butler Ditch, and finally would run 600 feet to W. Lisbon Road.

The project was never constructed by the Village and the permit granted by the Department expired in December of 1988. According to an account in the May 24, 1988 edition of the Milwaukee Journal, the Department approved the "connected enlargement" because it would minimize negative impacts on fish and aquatic life associated with modification of the existing Butler Ditch channel. The Village's reasons for not constructing the "connected enlargement" included concern over the impacts of the project on the woodland through which the enlargement would be located and concern that the project would lower groundwater levels and dry up the existing Butler Ditch channel. On the basis of the Village objections to the "connected enlargement" and because of the feasibility of providing habitat mitigation measures to offset the impacts of channel modification on fish and aquatic life, this system plan recommends

Map 186

100-YEAR RECURRENCE INTERVAL FLOODPLAIN FOR BUTLER DITCH UNDER YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

2.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR BUTLER DITCH IS SHOWN IN APPENDIX H

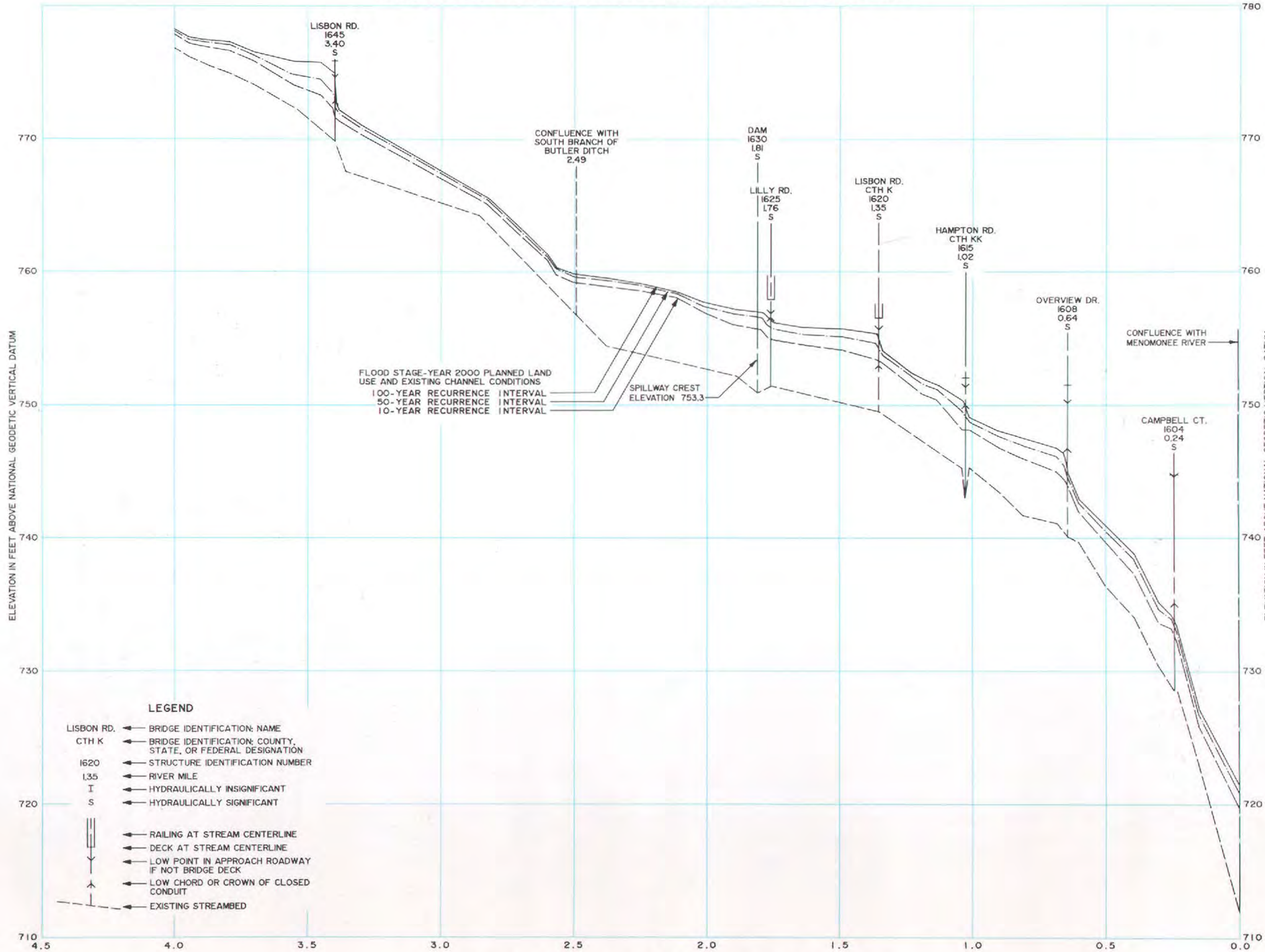
GRAPHIC SCALE

0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Source: SEWRPC.

Figure 91
FLOOD STAGE AND STREAMBED PROFILE FOR BUTLER DITCH



Source: SEWRPC.

Map 187

LOCATIONS OF STORM SEWER OUTFALL ALONG BUTLER DITCH WITH INVERT ELEVATION BELOW EXISTING STREAMBED

Source: SEWRPC.

minimal widening and deepening of the existing Butler Ditch channel as a means of resolving the stormwater drainage problems concerned.

Flood Control and Related Drainage System Plan Implementation

The recommended system plan for Butler Ditch consists solely of channel deepening measures intended to alleviate an existing stormwater drainage problem in the Village of Menomonee Falls. It is recommended that this channel deepening be implemented by the Village of Menomonee Falls. More specifically, it is recommended that the Village design, construct and maintain the channel modifications along the 0.6-mile reach of Butler Ditch between Lisbon Road and the Dolphin Drive storm sewer outfall. If, however, District jurisdiction for channel maintenance is extended to include the sewer service areas, it is recommended that that portion of the channel deepening cost attendant to the reach recommended for District jurisdiction be borne by the District.

IMPACT OF THE RECOMMENDED FLOOD CONTROL SYSTEM PLAN ON FLOOD FLOWS AND STAGES

Structural flood control measures herein recommended for streams in the Menomonee River watershed include channel modification, bridge replacement and modification, and detention storage. In some instances, these measures are intended to improve the hydraulic efficiency of the channel system and increase, to some degree, downstream flood flows and stages. Other measures will serve to decrease downstream flood flows and stages. Hydrologic and hydraulic analyses were conducted as part of this system planning effort to determine the impact of the recommended flood control measures on downstream flood flows and stages. A comparison of the 100-year recurrence interval flood flows and stages under planned land use and existing and planned channel conditions is shown in Table 116.

As already noted, implementation of flood control measures for the Menomonee River watershed may be expected to increase the 100-year recurrence interval flood flow on the Menomonee River by 1 to 3 percent downstream of its confluence with Honey Creek. This increase in flow would result in stage increases of 0.05 to 0.55 foot in the flood profile between River Miles 3.21 and 4.45. These stage increases have been incorporated into the flood control measures recommended for this reach. No significant increase in flood flows or stages is expected downstream of River Mile 3.21. Flood stages upstream of River Mile 4.45 may be expected actually to be lower than under existing channel conditions due to the recommended channel modifications. It should be noted that the increase in flood flows along the Menomonee River is due not only to the channelization along that stream, but also to the stormwater drainage improvements expected to be carried out on an ad hoc basis in the Honey Creek subwatershed.

Flood control and stormwater drainage measures recommended for Honey Creek are expected to result in an increase of up to 28 percent in the 100-year recurrence interval flood flows along that stream, with the greatest increases occurring along that reach in the City of Greenfield recommended to be channelized. This increase in discharge would result in stage increases of up to 0.3 foot along Honey Creek between IH 894 and W. Arthur Avenue. These stage increases would be contained within the existing drainage easement for Honey Creek. Increases in flood flows and stages downstream of W. Arthur Avenue may be attributed to stormwater drainage measures expected to be carried out on an ad hoc basis in the City of West Allis stormwater drainage system tributary to the enclosed portion of the stream. These improvements would reduce the temporary stormwater "detention" caused by the inability of the existing drainage system to adequately convey runoff from large storm events to the enclosure. However, this system plan recommends that such stormwater drainage measures be designed so that the existing 2,650 cfs capacity of the enclosure is not exceeded under 100-year recurrence interval conditions.

The detention storage measures recommended for Dousman Ditch would serve to reduce downstream flooding. For a 100-year recurrence interval flood event under planned land use conditions, flows along Dousman Ditch are expected to be reduced by up to 68 percent and along Underwood Creek by up to 50 percent. This reduction in flow may be expected to result in decreases of 0.01 to 4.46 feet in the 100-year recurrence interval flood stage along Underwood Creek, and 1.83 to 3.17 feet along Dousman Ditch. The 100-year recurrence interval flood profile along the lower reach of the South Branch of Underwood Creek would be lowered slightly due to a reduction in backwater from Underwood Creek. No significant reduction in flood flows is expected along the downstream reaches of the Menomonee River due to construction of the recommended detention basins.

The remaining structural flood control measures recommended for the Menomonee River watershed under this system plan are not expected to have a significant impact on downstream flood flows and stages. Those measures include channel modifications along the Menomonee River upstream of the Menomonee Falls dam and along Butler Ditch upstream of Lisbon Road, and the construction of a relief culvert along the downstream reach of Woods Creek.

MENOMONEE RIVER WATERSHED RECOMMENDED PLAN SUMMARY

The preceding sections of this report presented evaluations of alternative flood control and related drainage system plans for the Menomonee River, Woods Creek, Honey Creek, Underwood Creek and Dousman Ditch, the South Branch of Underwood Creek, the Little Menomonee River, and Butler Ditch. With the exception of Woods Creek, all of these streams were studied under the comprehensive watershed planning program for the Menomonee River watershed completed by the Commission in 1976. The recommended plan for each stream as set forth in the watershed study has been refined for this system plan, using updated hydrologic and hydraulic models and accounting for changes in local land development and redevelopment plans. An original recommended plan has been developed for Woods Creek. The refined plans are recommended based upon consideration of their technical feasibility, economic viability, environmental impacts, potential public acceptance, and practicality.

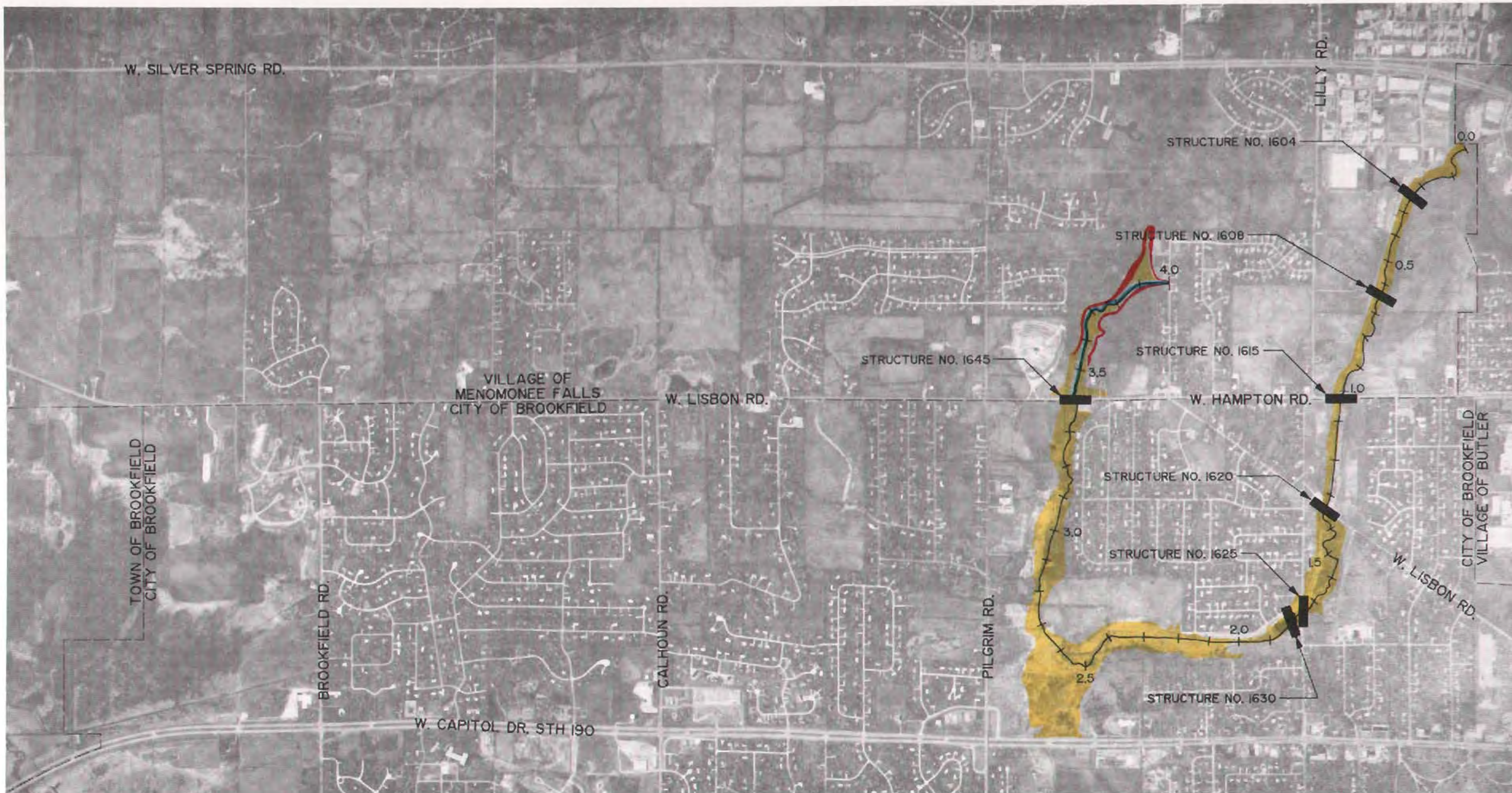
The total capital cost of the recommended combined flood control plan for the Menomonee River and tributaries is estimated at $11,418,000. This includes costs for measures designed to abate flood damages and to provide adequate outlets for existing and proposed storm sewers. The costs of the recommended plans for each reach of stream are given in Table 117. The apportionment of the recommended plan costs among the various public agencies and private owners concerned are set forth in Table 118.

The recommended plan would essentially eliminate all flood-related damages during floods up to and including a 100-year recurrence event under planned land use and channel conditions along the Menomonee River and the seven tributaries. The flood control measures considered under this system plan are primarily intended to alleviate flood damages from direct overland flooding along the streams studied, as well as to provide an adequate outlet for local storm sewers and drainageways. These measures, although not specifically designed to do so, may also be expected to reduce damages due to localized stormwater drainage problems or sanitary sewer backup.

The recommended plan includes the provision of substantial floodwater storage through the preservation of essentially totally natural floodplain storage remaining in the watershed and through the construction of two storage facilities. The two storage facilities which are recommended for construction include a 110-acre, 280-acre-foot detention basin in series with a

Map 188

RECOMMENDED FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN FOR BUTLER DITCH



LEGEND

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 PLANNED LAND USE AND PLANNED CHANNEL CONDITIONS

2.0 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

PROPOSED CHANNEL DEEPENING

NOTE: THE AVAILABILITY OF LARGE-SCALE TOPOGRAPHIC MAPPING FOR BUTLER DITCH IS SHOWN IN APPENDIX H

GRAPHIC SCALE

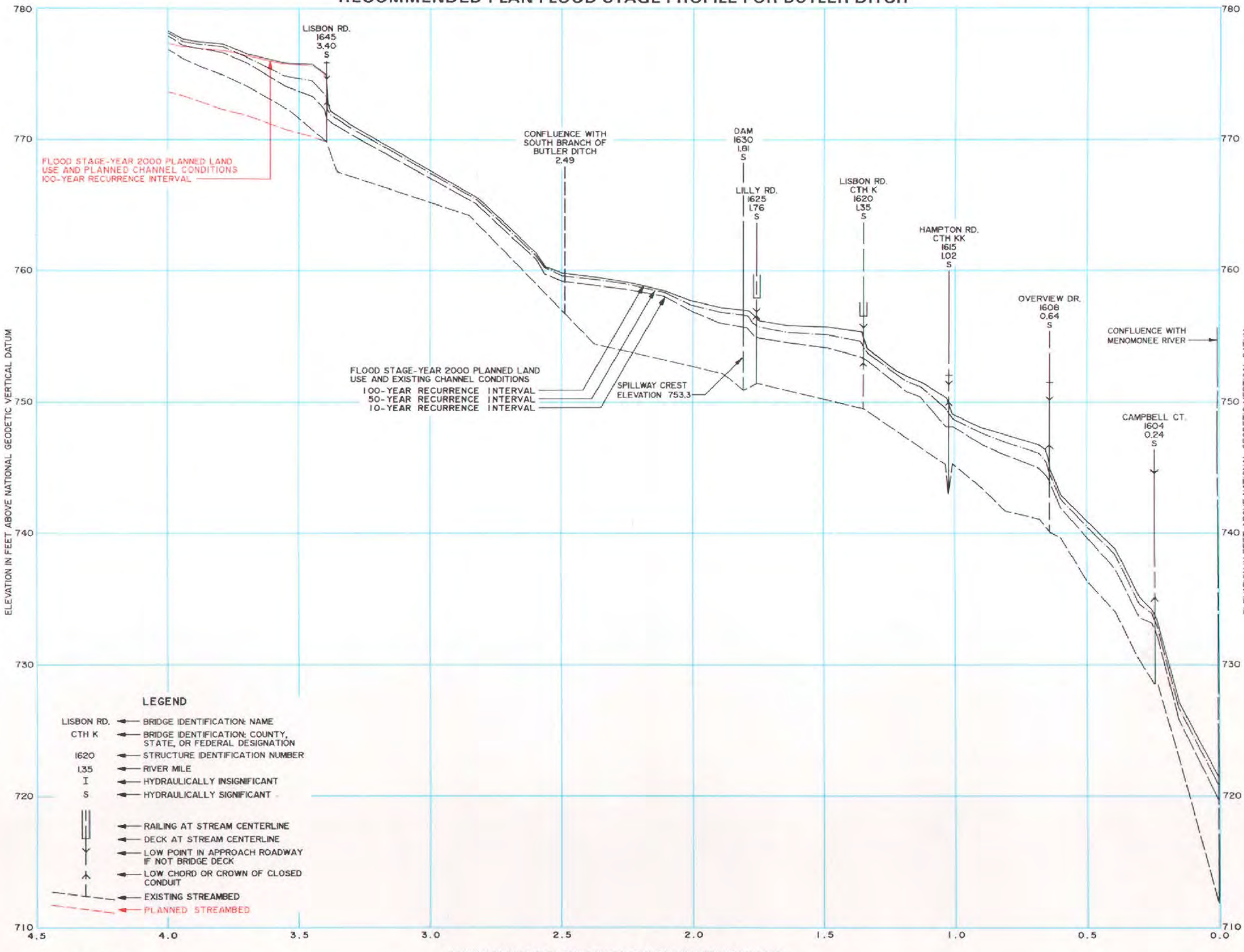
0 1/2 1 MILE

DATE OF PHOTOGRAPHY: APRIL 1986

Source: SEWRPC.

Figure 92

RECOMMENDED PLAN FLOOD STAGE PROFILE FOR BUTLER DITCH



Source: SEWRPC.

Table 116

IMPACT OF RECOMMENDED FLOOD CONTROL AND RELATED DRAINAGE SYSTEM PLAN ON MENOMONEE RIVER WATERSHED FLOOD DISCHARGES AND STAGES

Stream	Location	River Mile	100-Year Recurrence Interval Flood Discharge Planned Land Use (cfs) Existing Channel Conditions	Recommended Plan Conditions	Percent Change	100-Year Recurrence Interval Flood Stage Planned Land Use[a] (feet NGVD) Existing Channel Conditions	Recommended Plan Conditions	Change in Flood Stage (feet)
Menomonee River	At Falk Corporation dam	2.22	16,800	16,800	0	595.7	595.7	0
	S. 32nd Street extended	2.45	16,400	16,400	0	596.1	596.1	0
	Upstream of confluence with Woods Creek	3.21	14,900	15,100	1	597.6	597.6	0
	Soo Line Railroad (former Chicago, Milwaukee, St. Paul & Pacific Railroad)	3.71	14,900	15,100	1	602.9	603.0	+0.1
	Soo Line Railroad	4.24	13,700	14,000	2	608.4	608.7	+0.3
	Downstream from N. 45th Street	4.44	13,700	14,000	2	612.2	612.7	+0.5
	Upstream from N. 45th Street	4.46	13,700	14,000	2	614.4	613.8	-0.6
	N. Hawley Road	5.15	13,700	14,000	2	634.7	628.1	-6.6
	N. 70th Street	6.10	13,600	14,000	3	645.5	642.4	-3.1
	Upstream of confluence with Honey Creek	6.24	10,200	10,200	0	647.1	643.7	-3.4
	W. Harmonee Avenue	6.79	10,200	10,200	0	659.1	659.0	-0.1
	W. North Avenue	8.50	5,390	5,390	0	679.5	679.5	0
	W. Burleigh Street	9.68	5,290	5,290	0	685.7	685.7	0
	N. Mayfair Road (STH 100)	10.67	5,130	5,130	0	695.8	695.8	0
	Upstream from W. Capitol Drive	11.38	5,070	5,070	0	700.5	700.5	0
	Zoo Freeway (USH 45)	12.88	4,290	4,290	0	706.6	706.6	0
	W. Silver Spring Road	14.64	3,670	3,670	0	726.1	726.1	0
	Upstream of confluence with Dretzka Park Tributary	17.97	3,250	3,250	0	743.3	743.3	0
	Lilly Road	19.74	2,250	2,250	0	752.3	752.3	0
	Upstream of confluence with Nor-X-Way Channel	20.27	1,360	1,360	0	755.0	755.0	0
	Menomonee Falls Dam	21.89	1,180	1,180	0	835.3	835.3	0
	Woodlawn Avenue extended	22.71	1,150	1,150	0	842.2	840.0	-2.2

Table 116 (continued)

Stream	Location	River Mile	100-Year Recurrence Interval Flood Discharge Planned Land Use (cfs)		Percent Change	100-Year Recurrence Interval Flood Stage Planned Land Use[a] (feet NGVD)		Change in Flood Stage (feet)
			Existing Channel Conditions	Recommended Plan Conditions		Existing Channel Conditions	Recommended Plan Conditions	
Menomonee River (continued)	W. County Line Road	23.43	1,140	1,140	0	843.5	842.2	-1.3
	STH's 41 and 45	24.80	1,220	1,220	0	845.4	845.2	-0.2
	Lilac Lane	25.23	790	790	0	845.4	845.3	-0.1
	STH 145	27.25	560	560	0	850.1	850.1	0
Little Menomonee River	Confluence with the Menomonee River	0.00	1,700	1,700	0	703.5[b]	703.5[b]	0
	W. Appleton Avenue	1.57	1,700	1,700	0	707.8	707.8	0
	W. Fond du Lac Avenue	2.56	1,820	1,820	0	712.0	712.0	0
	Upstream of confluence with Noyes Creek	3.07	1,100	1,100	0	712.5	712.5	0
	W. Bradley Road	4.69	640	640	0	718.3	718.3	0
	W. County Line Road	6.95	730	730	0	721.1	721.1	0
Underwood Creek	Confluence with the Menomonee River	0.00	5,760	5,730	-0.5	678.6[b]	678.6[b]	0
	Zoo Freeway (USH 45)	0.75	5,030	4,930	-2	691.1	691.1	0
	W. Watertown Plank Road	1.50	4,310	4,230	-2	709.0	708.8	-0.2
	United Parcel Service Drive	2.57	1,640	1,520	-7	723.6	723.6	0
	Soo Line Railroad	3.10	1,640	1,520	-7	733.1	732.9	-0.2
	Watertown Plank Road	3.43	1,640	1,520	-7	743.7	743.4	-0.3
	W. Juneau Boulevard	3.67	1,310	990	-24	746.4	745.9	-0.5
	W. North Avenue	4.82	1,280	920	-28	752.1	751.4	-0.7
	Clearwater Drive	5.59	1,090	600	-45	758.1	757.4	-0.7
	Indian Creek Parkway	6.20	1,090	600	-45	797.2	792.7	-4.5
	Upstream from Soo Line Railroad	6.33	820	410	-50	804.8	800.9	-3.9
	Upstream of confluence with Dousman Ditch	6.97	130	130	0	823.1	821.4	-1.7
	Wirth Park footbridge	7.24	130	130	0	823.6	822.5	-1.1
	Soo Line Railroad	7.28	74	74	0	825.4	825.4	0
South Branch of Underwood Creek	Confluence with Underwood Creek	0.00	2,260	2,260	0	719.6[c]	719.4[c]	-0.2
	Theodore Trecker Way enclosure outlet	1.08	1,430	1,430	0	723.9	723.9	0

Table 116 (continued)

Stream	Location	River Mile	100-Year Recurrence Interval Flood Discharge Planned Land Use (cfs): Existing Channel Conditions	100-Year Recurrence Interval Flood Discharge Planned Land Use (cfs): Recommended Plan Conditions	Percent Change	100-Year Recurrence Interval Flood Stage Planned Land Use[a] (feet NGVD): Existing Channel Conditions	100-Year Recurrence Interval Flood Stage Planned Land Use[a] (feet NGVD): Recommended Plan Conditions	Change in Flood Stage (feet)
Dousman Ditch	Confluence with Underwood Creek	0.00	620	200	-68	823.1[c]	821.3[c]	-1.8
	Upstream from W. North Avenue	0.07	620	200	-68	824.8	821.6	-3.2
Honey Creek	Confluence with the Menomonee River	0.00	3,600	3,700	+3	646.9[b]	643.9[b]	-3.0
	Honey Creek Parkway	0.17	3,600	3,700	+3	653.8	653.3	-0.5
	W. Portland Avenue	0.50	3,600	3,700	+3	668.5	668.9	+0.4
	W. Wisconsin Avenue	0.91	3,200	3,350	+5	683.0	683.1	+0.1
	Drop structure	1.52	2,850	3,000	+5	690.0	690.6	+0.6
	IH 94	1.99	2,500	2,650	+6	695.3	695.5	+0.2
	W. Greenfield Avenue	3.10	2,280	2,450	+7	- -[d]	- -[d]	- -[d]
	Upstream of W. Arthur Avenue enclosure inlet	4.32	2,280	2,270	-0.4	727.9	727.7	-0.2
	W. Oklahoma Avenue	5.27	1,870	1,860	-0.5	731.2	731.1	-0.1
	Downstream of W. Howard Avenue	6.44	1,310	1,370	+4	744.2	744.2	0
	IH 894	7.53	760	970	+28	752.2	752.3	+0.1
	W. Layton Avenue	7.80e	640	760	+19	759.9	753.4	-6.5
	W. Loomis Road	8.53e	430	430	0	761.9	756.5	-5.4
Butler Ditch	Confluence with the Menomonee River	0.00	950	950	0	721.9[b]	721.9[b]	0
	W. Hampton Road	1.02	760	760	0	750.2	750.2	0
	Downstream of Lilly Road	1.72	520	520	0	756.0	756.0	0
	Upstream of confluence with South Branch of Butler Ditch	2.49	450	450	0	759.7	759.7	0
	W. Lisbon Road	3.40	380	380	0	774.8	774.8	0
	Upstream of W. Lisbon Road	3.56	380	380	0	775.7	775.7	0
	Downstream of Dolphin Drive storm sewer outfall	3.99	95	95	0	777.9	777.1	-0.8
Woods Creek	Confluence with the Menomonee River	0.00	1,160	1,160	0	- -[d]	- -[d]	- -
	Stadium Freeway culvert	0.08	1,080	1,120	+4	610.2	599.6	-10.6

Table 116 (continued)

Stream	Location	River Mile	100-Year Recurrence Interval Flood Discharge Planned Land Use (cfs) Existing Channel Conditions	Recommended Plan Conditions	Percent Change	100-Year Recurrence Interval Flood Stage Planned Land Use[a] (feet NGVD) Existing Channel Conditions	Recommended Plan Conditions	Change in Flood Stage (feet)
Woods Creek (continued)	Soo Line Railroad	0.265	1,120	1,120	0	612.0	607.0	-5.0
	Outlet of 108-inch culvert	0.33	880	880	0	612.2	607.4	-4.8
	Outlet of Veterans Administration Center enclosure	0.63	850	850	0	617.6	617.6	0
	Upstream of Veterans Administration Center enclosure	0.92	580	580	0	630.5	630.5	0

[a] *Flood stages at road crossings are for upstream side of the bridge or culvert.*

[b] *Menomonee River flood stage.*

[c] *Underwood Creek flood stage.*

[d] *Flow is contained within culvert.*

[e] *Measured along existing, as opposed to planned, channel alignment.*

Source: SEWRPC.

54-acre, 50-acre-foot basin along Dousman Ditch in the City of Brookfield, and a 7.5-acre, 12.5-acre-foot basin in conjunction with 35.5 acre-feet of storage in a deepened and widened channel along Honey Creek in the City of Greenfield.

The recommended flood control plan is best understood by dividing the Menomonee River and its tributary streams into several distinct reaches. The recommended plan for each of those reaches is described below.

Lower Menomonee River

The Lower Menomonee River reach extends from the mouth of the River at its confluence with the Milwaukee River to its confluence with the Little Menomonee River at River Mile 12.52. The portion of the Lower Menomonee River recommended for District jurisdiction begins at the Falk dam at River Mile 2.22. The plan recommendations for each subreach are as follows:

1. City of Milwaukee from the 27th Street Viaduct to IH 94: It is recommended that the existing steel sheet floodwall along the Falk Corporation property be raised from 0.2 to 3.0 feet to provide three feet of freeboard during a 100-year recurrence interval flood under planned land use and channel conditions. The crest of the Falk floodwall would be raised to elevation 595.5 feet NGVD at its downstream end and to 599.1 feet NGVD at its upstream end. It is also recommended that the existing earth dike along the former Chicago, Milwaukee, St. Paul & Pacific railway yards upstream of the Falk property be raised from 0 to three feet to elevation 600 feet NGVD to provide three feet of freeboard under 100-year flood conditions. The recommended plan for this reach is shown on Map 153.

2. City of Milwaukee from IH 94 to W. Michigan Street Extended: As shown on Map 153, it is recommended that 74 structures be floodproofed and one structure be elevated along the east bank in this reach.

Table 117

COST ESTIMATE SUMMARY FOR RECOMMENDED PLANS FOR THE MENOMONEE RIVER AND TRIBUTARIES

Stream	Reach	Recommended Plan: Name	Recommended Plan: Description	Costs: Capital	Costs: Annual: Total Amortized Capital[a]	Costs: Annual: Operation and Maintenance	Costs: Total	Benefit-Cost Analysis: Annual Benefits	Benefit-Cost Analysis: Annual Benefits Minus Annual Costs	Benefit-Cost Analysis: Economic Benefit-Cost Ratio	Benefit-Cost Analysis: Ratio Greater than One
Lower Menomonee River	City of Milwaukee from 27th Street Viaduct to IH 94	Dike and floodwall raises	Raise floodwall	$ 150,000	$ 15,000	$ --[b]	$ 15,000	$ --[c]	$ --	--	--
			Raise dike	90,000							
			Subtotal	$ 240,000							
	City of Milwaukee from IH 94 to W. Michigan Street extended	Structure floodproofing and elevation	Floodproof 74 structures	$ 340,000	$ 23,000	$ 0	$ 23,000	$ 3,100	$ -19,900	0.13	No
			Elevate one structure	30,000							
			Subtotal	$ 370,000							
	N. 43rd Street extended in the City of Milwaukee through Glenview Avenue extended in the City of Wauwatosa	Channel modification, dikes, structure floodproofing and elevation	2.31 miles of channel deepening including, bridge foundation modificaion	$ 4,948,000	$380,000	$ 6,000	$386,000	$129,000	$-257,000	0.33	No
			2,260 feet of earthen dike	116,000							
			Floodproof 23 structures in Milwaukee and 11 structures in Wauwatosa. Elevate one structure in Milwaukee	637,000							
			Storm sewers	170,000							
			Bridge removal and replacement	117,000							
			Subtotal	$ 5,988,000							
	City of Wauwatosa from Harwood Avenue through W. Capitol Drive	Structure floodproofing	Floodproof four structures	$ 29,000	$ 1,800	$ 0	$ 1,800	$ 900	$ -900	0.50	No
			Subtotal	$ 29,000							
Upper Menomonee River	Village of Menomonee Falls at River Mile 14.74 and from River Mile 19.42 to 19.94	Structure floodproofing	Floodproof eight structures	$ 42,000	$ 4,700	$ 0	$ 4,700	$ 2,400	$ -2,300	0.51	No
			Elevate one structure	32,000							
			Subtotal	$ 74,000							
	Village of Menomonee Falls from Roosevelt Drive through Erika Road extended	Channel modification	Channel widening and deepening	$ 384,000	$ 35,000	$ 2,000	$ 37,000	$ 10,000[d]	$ -27,000	0.27	No
			Bridge removal and replacement	175,000							
			Subtotal	$ 559,000							
	Village of Germantown at River Miles 23.48, 24.19, and 24.33	Structure floodproofing and elevation[e]	Floodproof one structure	$ 5,000	$ 2,200	$ 0	$ 2,200	$ 200	$ -2,000	0.09	No
			Elevate one structure	30,000							
			Subtotal	$ 35,000							
		Structure floodproofing and elevation[f]	Floodproof one structure	$ 5,000	$ 4,300	$ 0	$ 4,300	$ 1,400	$ -2,900	0.33	No
			Elevate two structures	63,000							
			Subtotal	$ 68,000							

Table 117 (continued)

Stream	Reach	Recommended Plan: Name	Recommended Plan: Description	Costs: Capital	Costs: Annual: Total Amortized Capital[a]	Costs: Annual: Operation and Maintenance	Costs: Annual: Total	Benefit-Cost Analysis: Annual Benefits	Annual Benefits Minus Annual Costs	Economic Benefit-Cost Ratio	Ratio Greater than One
Woods Creek	- -	Construction of relief culvert	1,500-foot-long, 10-foot-wide by 5-foot-high concrete box culvert	$ 981,000	$ 62,000	$ 0	$ 62,000	$ - -[g]	$ - -[g]	- -[g]	- -[g]
			Subtotal	$ 981,000							
Honey Creek	City of Wauwatosa at River Mile 0.85	Structure floodproofing	Floodproof one structure	$ 50,000	$ 3,200	$ 0	$ 3,200	$ 2,200	$ -1,000	0.69	No
			Subtotal	$ 50,000							
	City of Greenfield from IH 894 through the S. 43rd Street storm sewer outfall	Channel modification, detention storage, and bridge removal and replacement	1.17 miles of channel widening, deepening, and realignment	$ 800,000	$ 81,000	$12,000	$ 93,000	$ 900	$ -92,100	0.01[h]	No[h]
			Bridge removal and replacement	172,000							
			12.5-acre-foot detention basin construction	310,000							
			Subtotal	$ 1,282,000							
Underwood Creek and Dousman Ditch	Underwood Creek from Milwaukee-Waukesha County line to its confluence with Dousman Ditch and Dousman Ditch from its mouth to Wisconsin Avenue storm sewer outlet	Stormwater detention, structure floodproofing and elevation	50-acre-foot detention basin and 280-acre-foot detention basin	$ 672,000	$110,000	$20,000	$130,000	$166,000	$ 36,000	1.28	Yes
			Floodproof 38 structures	970,000							
			Elevate three structures	94,000							
			Subtotal	$ 1,736,000							
South Branch of Underwood Creek[i]	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
Little Menomonee River	River Miles 3.80, 6.90, and 6.93	Structure floodproofing	Floodproof three structures	$ 14,000	$ 900	$ 0	$ 900	$ 1,600	$ 700	1.78	Yes
			Subtotal	$ 14,000							
Butler Ditch	W. Lisbon Road to Dolphin Drive storm sewer outlet	Minor channel modification	0.6 mile of minor channel widening and deepening	$ 60,000	$ 4,000	$ 0[j]	$ 4,000	$ - -[g]	$ - -[g]	- -[g]	- -[g]
			Subtotal	$ 60,000							
			Total	$11,418,000[e]	$722,800[e]	$40,000	$762,800[e]	$ - -	$ - -	- -	- -

[a]Amortized capital cost is based on an interest rate of 6 percent and a project life of 50 years.

[b]No increase over existing operation and maintenance cost.

[c]Same approximate level of flood protection. Structures raised to provide three feet of freeboard.

[d]Includes flood damage reduction at three upstream buildings in the Village of Germantown.

[e]Assuming recommended channel modifications are implemented in the Village of Menomonee Falls.

[f]Assuming recommended channel modifications are not implemented in the Village of Menomonee Falls.

[g]Benefits due to abatement of secondary flooding and street or highway flooding. Such benefits are not typically accounted for in benefit-cost ratio calculations.

[h]Benefits primarily due to abatement of secondary flooding through the provision of adequate storm sewer outlets. Secondary flooding benefits are not typically accounted for in benefit-cost ratio calculations.

[i]No structure flood damages expected.

[j]Natural channel permitted to develop.

Source: SEWRPC.

Table 118

SUMMARY OF RECOMMENDED PLAN CAPITAL COSTS
BY AGENCY FOR THE MENOMONEE RIVER AND TRIBUTARIES

Implementing Agency	Stream	Reach	Flood Control Measures	Estimated Capital Cost
Milwaukee Metropolitan Sewerage District	Lower Menomonee River	City of Milwaukee from 27th Street Viaduct to IH 94	Raise floodwall Raise dike	$ 150,000 90,000
			Subtotal	$ 240,000
		N. 43rd Street extended in the City of Milwaukee through Glenview Avenue extended in the City of Wauwatosa	2.31 miles of channel deepening including bridge foundation modification	$ 4,948,000
			2,260 feet of earthen dike	116,000
			Private bridge removal and replacement	117,000
			Subtotal	$ 5,181,000
	Woods Creek	- -	1,500-foot-long, 10-foot-wide by 5-foot-high concrete box culvert	$ 981,000
			Subtotal	$ 981,000
	Honey Creek	City of Greenfield from W. Layton Avenue through S. 43rd Street storm sewer outfall	0.95 mile of channel widening, deepening, and realignment	$ 590,000
			Bridge removal	15,000
			Detention basin construction	310,000[a]
			Subtotal	$ 915,000
Milwaukee Metropolitan Sewerage District Total Capital Cost				$ 7,317,000[b,c]
Milwaukee County	Honey Creek	City of Greenfield from IH 894 through W. Layton Avenue	0.22 mile of channel widening, deepening, and realignment	$ 210,000
			Bridge replacement	157,000
Milwaukee County Total Capital Cost				$ 367,000
City of Wauwatosa	Lower Menomonee River	N. 60th Street extended through Glenview Avenue extended	Stormwater pumping station and stormwater drainage facilities	$ 170,000
City of Wauwatosa Total Capital Cost				$ 170,000

Table 118 (continued)

Implementing Agency	Stream	Reach	Flood Control Measures	Estimated Capital Cost
City of Brookfield	Underwood Creek and Dousman Ditch	Underwood Creek from Milwaukee-Waukesha County line to its confluence with Dousman Ditch and Dousman Ditch from its mouth to Wisconsin Avenue storm sewer outlet	50-acre-foot detention basin and 280-acre-feet detention basin	$ 168,000[d]
City of Brookfield Total Capital Cost				$ 168,000[e]
Village of Elm Grove	Underwood Creek and Dousman Ditch	Underwood Creek from Milwaukee-Waukesha County line to its confluence with Dousman Ditch and Dousman Ditch from its mouth to Wisconsin Avenue storm sewer outlet	50-acre-foot detention basin and 280-acre-feet detention basin	$ 504,000[d]
Village of Elm Grove Total Capital Cost				$ 504,000[e]
Village of Menomonee Falls	Upper Menomonee River	Roosevelt Drive through Erika Road extended	Channel widening and deepening	$ 384,000
			Bridge removal and replacement	175,000
			Subtotal	$ 559,000
	Butler Ditch	W. Lisbon Road to Dolphin Drive storm sewer outlet	0.6 mile of minor channel widening and deepening	$ 60,000
			Subtotal	$ 60,000
Village of Menomonee Falls Total Capital Cost				$ 619,000[e]
Various Private Property Owners	Lower Menomonee River	City of Milwaukee from IH 94 to W. Michigan Street extended	Floodproof 74 structures	$ 340,000
			Elevate one structure	30,000
			Subtotal	$ 370,000
		N. 43rd Street extended in the City of Milwuakee through Glenview Avenue extended in the City of Wauwatosa	Floodproof 23 structures in the City of Milwaukee and 11 structures in Wauwatosa. Elevate one structure in Milwaukee	$ 637,000
			Subtotal	$ 637,000

Table 118 (continued)

Implementing Agency	Stream	Reach	Flood Control Measures	Estimated Capital Cost
Various Private Property Owners (continued)		City of Wauwatosa from Harwood Avenue through W. Capitol Drive	Floodproof four structures	$ 29,000
			Subtotal	$ 29,000
	Honey Creek	City of Wauwatosa at River Mile 0.85	Floodproof one structure	$ 50,000
			Subtotal	$ 50,000
	Underwood Creek	Milwaukee-Waukesha County line to the confluence with Dousman Ditch	Floodproof 38 structures	$ 970,000
			Elevate three structures	94,000
			Subtotal	$ 1,064,000
	Little Menomonee River	City of Milwaukee at River Miles 3.80, 6.90, and 6.93	Floodproof three structures	$ 14,000
			Subtotal	$ 14,000
	Upper Menomonee River	Village of Menomonee Falls at River Mile 14.74 and and from River Mile 19.42 to 19.94	Floodproof eight structures	$42,000
			Elevate one structure	32,000
			Subtotal	$ 74,000
		Village of Germantown at River Miles 23.48, 24.19, and 24.33	Floodproof one structure[f]	$ 5,000
			Elevate one structure[f]	30,000
			Subtotal	$ 35,000
			Floodproof one structure[g]	$ 5,000
			Elevate two structures[g]	63,000
			Subtotal	$ 68,000
Private Property Owners Total Capital Cost				$ 2,273,000[f]
Total Project Cost				$11,418,000[f]

Table 118 (continued)

[a]*Cost does not include specific consideration of added facility requirements associated with water quality and recreation benefits.*

[b]*Cost would be increased by approximately $672,000 if District boundaries were extended into affected areas of the City of Brookfield and the Village of Elm Grove.*

[c]*Cost would be increased by approximately $619,000 if District boundaries were extended into affected areas of the Village of Menomonee Falls.*

[d]*Cost of recommended detention basin assigned in proportion to the flood damage mitigation benefits derived by the City of Brookfield and the Village of Elm Grove.*

[e]*Cost is recommended to be borne by the Milwaukee Metropolitan Sewerage District if District boundaries are expanded to include these areas.*

[f]*Assuming recommended channel modifications are implemented in the Village of Menomonee Falls.*

[g]*Assuming recommended channel modifications are not implemented in the Village of Menomonee Falls.*

Source: SEWRPC.

3. City of Milwaukee from W. Michigan Street Extended at River Mile 3.97 through the Soo Line Railroad Bridge at River Mile 4.24: No flood control improvements are recommended for this reach.

4. N. 43rd Street at River Mile 4.33 in the City of Milwaukee through Glenview Avenue Extended at River Mile 6.88 in the City of Wauwatosa: As shown on Map 156, the recommended flood control plan for this reach consists of a combination of channel modification, dike construction, and structure floodproofing and elevation. Channel deepening is recommended for the 2.24-mile-long reach from the pedestrian bridge upstream of Hart Park in Wauwatosa through the existing drop structure at N. 45th Street in Milwaukee. The channel modifications call for lowering the existing streambed about nine feet at N. 70th Street and about five feet at N. 45th Street. Based on soil and rock borings taken for the Metropolitan Sewerage District's deep tunnel project, it is expected the modified channel would have to be constructed partially in rock for about one mile of its total length.

   The recommended modified channel would not cause significant disturbance of the existing streambanks and the existing trees and vegetation along the banks would essentially be retained. The modified channel would consist of a low-flow channel and a flood control channel as shown on Map 156. The three-foot-deep, riprap-lined trapezoidal low-flow channel would have a four-foot bottom width and one vertical on 2.5 horizontal side slopes. The two- to six-foot-deep flood control channel would have stepped sidewalls constructed of bedrock, rock gabions, or limestone blocks at an approximate average side slope of 0.5 horizontal on one vertical. The channel bottom width, including the low-flow channel, would be about 51 feet. The existing cobbles, boulders, and rock slabs in the streambed would be saved during excavation and used to line the flood control channel bed in those reaches where the channel is constructed in alluvial material.

   Beginning about 350 feet upstream from N. 70th Street, and extending for 770 feet, the flood control channel bottom width would narrow from 51 feet to 19 feet. The low-flow channel in that reach would have the same shape and dimensions as in the downstream reach. The 19-foot-wide flood control channel bottom would be maintained for the next 360 feet of river channel and would then transition to the existing river channel section over the next 1,480 feet.

There are existing limestone retaining walls along the streambanks in several sections of the reach for which channel modifications are proposed. The modified channel sidewalls would be constructed adjacent to, and below, those existing walls. A structural analysis of the existing walls should be performed preceding implementation of the recommended improvements, and any necessary repairs should be made prior to or as part of the channel modification. Also, during the final design, consideration should be given to providing erosion protection for the existing streambanks above the recommended modified channel. Such protection would only need to be considered in reaches where there are no existing limestone walls or where these walls are not structurally adequate.

The recommended channel work would necessitate the modification of the existing foundations of the N. 68th Street bridge. Some modification of the N. 70th Street bridge foundation might also be required. The private bridge at River Mile 4.84 would have to be replaced.

An approximately 1,450-foot-long earthen dike would be constructed along the north bank of the River adjacent to an industrial area located downstream from N. 68th Street at Jacobus Park. The dike would have a maximum height of about eight feet to provide three feet of freeboard and would improve riparian aesthetics by screening the industrial area from the view of an observer standing near stream level in Jacobus Park. The dike crest elevation would range from about 637 feet NGVD at its downstream end to about 641 feet NGVD at its upstream end. In the facilities design stage, it is recommended that the dike be aligned so that mature trees along the bank are preserved to the maximum extent possible. The dike and modified channel in the reach along Jacobus Park would be aligned to avoid any disturbance of the south streambank in order to preserve the rare and valuable plant species which have been identified along the bank. The recommended channel modifications would remove approximately 50 buildings from the 100-year floodplain in the Cities of Milwaukee and Wauwatosa along the reach from N. 68th Street to N. 45th Street. However, approximately 56 buildings would remain in the floodplain without construction of the recommended dike. Construction of the dike along this reach would remove 27 of those buildings, leaving 23 to be floodproofed and one to be elevated in the City of Milwaukee, and five to be floodproofed in the City of Wauwatosa.

Local runoff from the area bounded by N. 68th Street on the west, the Soo Line (formerly Chicago, Milwaukee, St. Paul & Pacific Railroad) railway embankment on the north, N. 60th Street extended on the east, and the recommended dike on the south would be collected on the landward side of the dike, and conveyed to the existing storm sewer in N. 63rd Street extended. The storm sewer outlet would be replaced with a larger 66-inch-diameter pipe. Five buildings north of the dike would remain in the 100-year floodplain. It is recommended that those buildings be floodproofed.

The recommended channel modifications would remove approximately 98 buildings from the 100-year floodplain in the reach from N. 68th Street through Hart Park. However, three buildings, including the Park Administration and Athletic Building and the Muellner Building in Hart Park and one residence would remain in the floodplain.

It is recommended that an 810-foot-long, two- to five-foot-high dike be constructed along the east bank adjacent to Hart Park in Wauwatosa. Construction of the dike would remove the Park Administration and Athletic Building and the Muellner Building from the floodplain, leaving only one residence to be floodproofed. The dike crest elevation would range from about 649 feet NGVD at its downstream end to 655 feet NGVD at its upstream end.

To accommodate the recommended Menomonee River streambed elevation without providing drop structures along Honey Creek, it would be necessary to lower the Honey Creek streambed by up to seven feet in the 0.17-mile-long reach between the Honey Creek Parkway bridge and its confluence with the Menomonee River. It is recommended that the Honey Creek streambed be lowered by constructing a channel below the existing streambed and within the existing banks. As shown on Map 170, that could be accomplished with a trapezoidal channel, having a four-foot-wide bottom and average side slopes of 0.7 horizontal to one vertical. The stepped channel sidewalls would be constructed of rock gabions or limestone block, in a manner similar to those recommended for the Menomonee River.

Floodproofing of two commercial buildings and three industrial buildings located upstream of Hart Park in the vicinity of Harwood Avenue and the Harmonee Avenue Bridge is also recommended.

In summary, following channel modification and dike construction, residual structural damages could be eliminated during floods up to a 100-year recurrence interval under planned land use conditions by floodproofing 23 structures in the City of Milwaukee and 11 structures in the City of Wauwatosa, and by elevating one structure in the City of Milwaukee.

5. City of Wauwatosa from Glenview Avenue Extended through W. Capitol Drive: As shown on Map 157, the recommended flood control plan consists of the floodproofing of four structures along the east bank of the River from N. 97th Street through W. Ridge Boulevard.

6. City of Wauwatosa and the City of Milwaukee from W. Capitol Drive to the Confluence with the Little Menomonee River at River Mile 12.52: No flood control improvements are recommended for this reach.

Upper Menomonee River

The Upper Menomonee River reach extends from its confluence with the Little Menomonee River at River Mile 12.52 to the upstream end of the stream at River Mile 29.37. The policy plan companion to this system plan recommends that 5.9 miles of the Upper Menomonee River which are located within the current district limits be included under district jurisdiction. This reach extends from the confluence with the Little Menomonee River to the crossing of the Milwaukee-Waukesha County line just south of USH 45. An additional upstream 8.8 miles of the Menomonee River, extending from the Milwaukee-Waukesha County line through the Washington-Waukesha County line to STH 145 at River Mile 27.3, is located outside of the current district limits, but in an area identified in the policy plan as within possible future district limits. That reach was also found to meet the criteria for district jurisdiction. The plan recommendations for each subreach are a follows:

1. Menomonee River from the Confluence with the Little Menomonee River to River Mile through River Mile 14.73: No flood control improvements are recommended for this reach.

2. Menomonee River at River Mile 14.74: As shown on Map 158, it is recommended that one building at this location be floodproofed.

3. Menomonee River from River Mile 14.75 through River Mile 19.41: No flood control improvements are recommended for this reach.

4. Menomonee River from River Mile 19.42 through 19.94: As shown on Map 159, it is recommended that seven buildings in this reach be floodproofed and that one building be elevated.

5. Menomonee River from River Mile 19.95 through River Mile 22.01: No flood control improvements are recommended for this reach.

6. Menomonee River from River Mile 22.02 Just Downstream of Roosevelt Drive through River Mile 22.96 at Erika Road Extended: Channel modification is called for in a 0.94-mile-long reach from River Mile 22.02, just downstream of Roosevelt Drive, to River Mile 22.96 at Erika Road extended. As shown on Map 161, in that reach there are two existing storm sewer outfalls, the inverts of which are located below the existing streambed; and there is one proposed outfall which would be submerged due to an adverse slope on the streambed which creates a ponded condition in this stream reach. There are also approximately 15 buildings along this reach which are located in the 100-year recurrence interval flood hazard area under planned land use and existing chan-

nel conditions. Those buildings would be removed from the flood hazard area following construction of the recommended channel modifications. One building at River Mile 23.48 in the Village of Germantown just north of the Washington-Waukesha County line would also be removed from the 100-year flood hazard area due to reductions in the flood stage resulting from construction of the recommended channel modifications.

The channel modification recommended for the reach would accomplish the dual purpose of abating flood damages and providing adequate outlets for the existing and proposed storm sewers. As shown on Figure 73, it is recommended that the streambed be lowered a maximum of about four feet. As shown on Map 160, the widened and deepened channel section would have a 1.5-foot-deep, turf-lined low-flow channel with one vertical on two horizontal side slopes and a five-foot bottom width. The flood control channel would be constructed so as to only significantly disturb the left bank, viewed in the downstream direction. From River Mile 22.02 through the Roosevelt Drive bridge to River Mile 22.08, only minimal modification to the channel would be necessary to bring the bed to the desired grade. The Roosevelt Drive bridge would not require modification. Upstream from River Mile 22.08 through River Mile 22.37, the turf-lined flood control channel would have a 25-foot bottom width, the left bank side slope would be one vertical on three horizontal, and the existing right bank side slope would be retained with some relatively minor regrading required. Upstream from River Mile 22.37, the flood control channel would have the same characteristics, but the bottom width would be narrowed to 20 feet. According to information provided by the Village, there may be bedrock near the elevation of the existing streambed downstream from River Mile 22.44. Therefore, the modified channel in that reach may have to be at least partially constructed in rock.

The existing private bridge providing access to the River Court Shopping Center would be replaced with a new structure spanning the modified channel and providing no significant impediment to the conveyance of flood flows.

The possibility of the removal of the Menomonee Falls dam at River Mile 21.90 has been considered by the Village in preliminary planning for the riverfront area in the vicinity of the dam. No decision on whether or not to remove the dam has been made as yet, however, the possibility of removal was considered in analyzing the proposed channel modification. It was found that removal of the dam would reduce the 100-year recurrence interval flood stage under planned land use conditions by approximately 10.6 feet at the site of the dam. The stage reduction would be decreased to about 0.1 foot 1,500 feet upstream of the dam site. Because of the limited upstream extent of the reduced stages, it is concluded that removal of the dam would have no significant impact on the recommended channel modifications.

7. <u>Menomonee River from River Mile 22.97 through River Mile 23.43 at the Washington-Waukesha County Line</u>: No flood control improvements are recommended for this reach.

8. <u>Menomonee River In the Village of Germantown Upstream of River Mile 23.43</u>: As shown on Map 162, if the recommended channel modifications are implemented in the Village of Menomonee Falls, it is recommended that one structure in the Village of Germantown at River Mile 24.19 be floodproofed and one at River Mile 24.33 be elevated. If the recommended channel modifications in Menomonee Falls are not implemented, it is recommended that one structure in Germantown at River Mile 23.48 be floodproofed and that two structures, one at River Mile 24.19 and one at 24.33, be elevated.

<u>Woods Creek</u>

The entire 1.1-mile perennial stream length of Woods Creek is recommended for District jurisdiction in the policy plan companion to this system plan. No structural flood damages are expected to be incurred due to flooding along Woods Creek for floods up to and including the 100-year recurrence interval event. Flooding of portions of the County Stadium parking lot and the Stadium Freeway (USH 41), however, may be expected due to the inadequate hydraulic capacity of a series of culverts which currently convey Woods Creek from the Soo Line (former Chicago, Milwaukee, St. Paul & Pacific Railroad) railway at River Mile 0.27 to the Menomonee River. This lack of capacity results in surcharging of storm sewers both within the parking lot and along the Stadium Freeway, as well as flooding from direct overland flow.

As shown on Map 164, in order to alleviate flooding of the Freeway and the Stadium parking lot, it is recommended that a 1,500-foot-long, 10-foot-wide by 5-foot-high reinforced concrete box relief culvert be installed parallel to the existing culverts running from the Soo Line railway to the Menomonee River.

Honey Creek

The entire 8.8-mile-long reach of Honey Creek is recommended for District jurisdiction. The plan recommendations for each subreach are as follows:

1. City of Wauwatosa from the Mouth of Honey Creek through W. Wisconsin Avenue: As shown on Map 170, it is recommended that one building located at River Mile 0.85 be floodproofed.

   As recommended in the flood control plan for the main stem of the Menomonee River, channel modifications along Honey Creek from its mouth to the Honey Creek Parkway bridge at River Mile 0.17 would be required for the Honey Creek streambed grade to match the lowered Menomonee River streambed grade. Those modifications are discussed in the preceding section setting forth the recommendations for the Menomonee River.

2. Cities of Milwaukee and Wauwatosa from W. Wisconsin Avenue to the Channel Enclosure Outlet at the East-West Freeway (IH 94) at River Mile 1.99: No flood control improvements are recommended in this reach.

3. Cities of Milwaukee and West Allis Along the Channel Enclosure from River Mile 1.99 through 4.32 in the City of West Allis: No flood control improvements are recommended in this reach.

   It is recommended that any future storm sewer improvements undertaken by the City of West Allis for the purpose of alleviating local stormwater drainage problems in the Honey Creek subwatershed be designed so as to limit the increase in the total peak discharge in the channel enclosure, in the aggregate, to about 150 cfs. This limitation may allow for an increase in individual storm sewer capacities of more than 150 cfs if it is determined that the timing of the peak flow from an individual storm sewer relative to the timing of the peak flow in the enclosure is such that the storm sewer peak would occur before or after the peak in the enclosure.

4. Cities of Greenfield, Milwaukee, and West Allis from River Mile 4.32 through IH 894 at River Mile 7.55: The entire length of the stream channel in this reach has been modified under the earlier District flood control program. No additional flood control improvements are recommended in this reach.

5. City of Greenfield from IH 894 at River Mile 7.55 through the S. 43rd Street Storm Sewer Outfall at River Mile 8.83: As shown on Map 172, the refined flood control and related drainage system plan for Honey Creek along the reach from IH 894 to S. 43rd Street consists of modification and realignment of about 1.28 miles of stream, resulting in a total reach length of 1.17 miles under planned conditions.

   As shown on Figure 82, it is recommended that the streambed be lowered a maximum of about seven feet. As shown on Map 172, downstream of W. Layton Avenue the widened and deepened channel section would have a one-foot-deep, three-foot-wide, turf-lined low-flow channel; and a turf-lined flood control channel with an eight-foot bottom width, side slopes of one vertical on three horizontal for the lower five feet and side slopes of one vertical on about 3.5 horizontal up to the existing grade. Upstream of W. Layton Avenue the modified channel section would have a one-foot-deep, three-foot-wide, turf-lined low-flow channel; and a turf-lined flood control channel with an eight-foot bottom width, side slopes of one vertical on two horizontal for the lower three feet and side slopes of one vertical on about 3.5 horizontal up to the existing grade. The modified channel cross-section in this reach is shown on Map 172.

Due to right-of-way restrictions along the north bank on the downstream side of S. Loomis Road, it will be necessary to provide a vertical wall along that bank for about 80 feet downstream of the roadway. In the reaches upstream of W. Layton Avenue, the channel is located either in city or county park land or in a 120-foot-wide drainage easement. In order to improve the appearance of this reach and complement the park and residential setting through which the Creek flows, it is recommended that the detailed design for this reach consider the provision of some meander and variability in the low-flow section of the proposed channel.

Hydrologic simulation modeling conducted under this system planning effort indicates that the channelization project would create unacceptably high increases in downstream 100-year recurrence interval flood flows without the provision of detention storage in the reach containing the modification. Therefore, the plan recommends the provision of 48 acre-feet of detention storage upstream of W. Layton Avenue. That total amount of storage would be provided in the modified channel and a supplementary 12.5-acre-foot detention basin in the northeast one-quarter of U. S. Public Land Survey Section 26, Township 6 North, Range 21 East at the location shown on Map 172. The total detention basin area, including a buffer strip around its perimeter, would be about 7.5 acres. The maximum flood control pool elevation would be about 755.6 feet NGVD during a 100-year recurrence interval flood under planned land use and channel conditions. The existing W. Layton Avenue bridge would be replaced with a new 10-foot-wide by 8-foot-high reinforced concrete box culvert, rather than the 10-foot-wide by 10-foot-high box culvert initially proposed by the County. This reduction in culvert size is recommended to provide storage in the upstream channel and detention basin. The proposed detention basin could be constructed with a permanent pond which would trap pollutants in stormwater runoff, providing water quality benefits as well as flood control benefits along Honey Creek. In addition, the pond could be designed for other recreational uses, such as ice skating.

It is recommended that the existing construction access bridge at River Mile 8.11 and the old Loomis Road bridge at River Mile 8.55 be removed and not replaced.

Underwood Creek and Dousman Ditch

Underwood Creek has a total length of 8.1 miles, 2.6 of which are located in the City of Wauwatosa within the current District limits and are recommended for District jurisdiction in the policy plan companion to this system plan. Dousman Ditch is a tributary to Underwood Creek, with a total length of about 2.5 miles. The portion of Underwood Creek in the City of Brookfield and the Village of Elm Grove and all of Dousman Ditch, both of which are located outside the current District limits but within an area defined in the policy plan as within possible future District limits, were found to meet the criteria for District jurisdiction. These additional reaches were accordingly included in the system planning effort.

1. Underwood Creek in the City of Wauwatosa from Its Confluence with the Menomonee River to the Milwaukee-Waukesha County Line: The entire length of the stream channel in this reach has been modified under the District flood control program. No additional flood control improvements are recommended in this reach.

2. Underwood Creek in the City of Brookfield and the Village of Elm Grove from the Milwaukee-Waukesha County Line to Its Confluence with Dousman Ditch; Dousman Ditch from Its Mouth to the Wisconsin Avenue Storm Sewer Outlet: The recommended flood control plan for Underwood Creek and Dousman Ditch is shown on Map 177 and includes the construction of two stormwater detention basins along Dousman Ditch upstream of N. Gebhardt Road as well as structure floodproofing

and elevation along Underwood Creek in the City of Brookfield and the Village of Elm Grove.

The lower, or northern, detention basin would be located in the southeast one-quarter of U. S. Public Land Survey Section 22, Township 7 North, Range 20 East, with its outlet located immediately upstream of the proposed extension of W. Choctaw Trail, about 600 feet upstream of N. Gebhardt Road. This basin would cover an area of about 54 acres and would have a design capacity of about 50 acre-feet at a pool elevation of 825.2 feet above NGVD. The outlet control structure would be created by constructing about 330 feet of earthen dike beginning at N. Pilgrim Parkway and extending west across the channel. This dike would range in height from one to five feet and have side slopes of one vertical on three horizontal.

Outflow from the basin would be handled by an 18-inch-diameter reinforced concrete pipe placed in this dike at the existing channel invert. This pipe would restrict outflow from the basin during larger storm events but would allow for subsequent drainage of the basin as well as the conveyance of flows along Dousman Ditch during periods of low flow. The top of the dike would be at the design pool elevation of 825.2 feet NGVD and would act as an emergency spillway for stormwater runoff in excess of 50 acre-feet. Although no inundation of N. Pilgrim Parkway is expected at the design pool elevation, there are two low points along the roadway where there would be only about 0.3 foot of freeboard. Thus, it may be desirable to raise those portions of N. Pilgrim Parkway in order to provide for greater freeboard.

The upper, or southern, detention basin would be located in the north one-half of U. S. Public Land Survey Section 27, Township 7 North, Range 20 East, between N. Pilgrim Parkway and N. Calhoun Road. This basin would cover an area of about 110 acres and would have a design capacity of about 280 acre-feet at a pool elevation of 830.2 feet above NGVD. It would be created by constructing about 3,700 feet of earthen dikes ranging in height from one to seven feet with side slopes of one vertical on three horizontal. The outlet from the basin would consist of an 18-inch-diameter reinforced concrete pipe located at River Mile 1.36, or about 500 feet upstream of the private drive at the Dousman-Dunkel Inn. An emergency spillway at the design pool elevation of 830.2 feet NGVD would be constructed at this location to handle stormwater runoff in excess of 280 acre-feet. About 1,300 feet of N. Pilgrim Parkway would be raised an average of 1.5 feet in order to prevent overtopping at the design pool elevation and to provide a minimum freeboard of two feet.

No stormwater pumping facilities would be required for either of these two basins. Drainage to the basins would be accomplished by gravity flow along existing drainageways. By leaving these drainageways open, some inundation of currently developed land, but not buildings, would occur. A total of 10 properties would be affected along the lower basin, although the design pool elevation is about 0.8 foot below the 100-year recurrence interval flood level under planned land use and existing channel conditions along these properties. A total of four properties would be affected along the upper detention basin. The design pool elevation would be about 2.5 feet above the 100-year flood level under planned land use and existing channel conditions at these properties. At those locations where the 100-year flood level would be increased due to construction of the detention basin, it may be necessary to obtain flood easements from the property owners affected.

Construction of the two detention basins would reduce from about 76 to about 41 the number of structures expected to incur direct flood damages under a 100-year recurrence interval flood along Underwood Creek. Although 41 structures would still be expected to incur flood damages under a 100-year recurrence interval flood, the flood damages would be significantly less than would be expected under existing conditions. Thirty-eight of these 41 structures would have to be floodproofed and three would have to be elevated. No structures would have to be removed.

3. Underwood Creek Upstream of Its Confluence with Dousman Ditch: No flood control improvements are recommended in this reach.

Little Menomonee River

The lower 7.0 miles of the total 10.2-mile length of the Little Menomonee River are recommended for District jurisdiction in the policy plan companion to this system plan. That lower reach lies entirely within the City of Milwaukee. The remainder of the stream lies in an area not within the current District limits or planned future District limits.

To insure adequate functioning of the existing storm sewers which serve areas of existing as well as planned development in the City of Milwaukee, it is recommended that the Little Menomonee River streambed be lowered to accommodate 13 existing storm sewer outfalls. Many of the existing channels conveying flow from those outfalls to the River are presently constructed without a bed slope, or with a very flat bed slope, causing sediment to collect and restrict the outfalls. In addition, because the pipe invert elevations at the outfalls are below grade, the outfall pipes are constructed with a reverse slope, which causes sediment to accumulate in the storm sewers, reducing the hydraulic capacity of the sewers.

The flood control plan recommendations for each reach of the Little Menomonee River are as follows:

1. Confluence with the Menomonee River to River Mile 3.79 Just Upstream of N. Granville Road: No flood control improvements are recommended in this reach.

2. River Mile 3.80: As shown on Map 183, it is recommended that one building at this location be floodproofed.

3. River Mile 3.81 through River Mile 6.89 Just Downstream of the Milwaukee-Ozaukee County Line: No flood control improvements are recommended in this reach.

4. River Mile 6.90 through 6.93: As shown on Map 183, it is recommended that two buildings in this reach be floodproofed.

As set forth in the Menomonee River watershed study, field reconnaissance sampling and laboratory analyses conducted in 1971 concluded that the Little Menomonee River bottom sediments were contaminated with creosote in the 3.75-mile reach from W. Brown Deer Road to a point about 2,000 feet downstream of the Fond du Lac Freeway (USH 145). The 1971 studies were limited to that reach; therefore, no information was available on the possible extent of creosote in the bottom sediments farther downstream.

The creosote in the streambed of the upper 0.75 mile of the contaminated reach was removed under a 1973 demonstration project. The watershed study recommended that the residual creosote pollution problem downstream of the 0.75-mile-long upper reach be resolved by excavating a new 3.46-mile-long parallel channel of similar size to that of the existing channel, but approximately 20 feet from the existing channel, filling the existing channel, and restoring the site.

Since the watershed study was issued, additional investigations of the extent of the creosote pollution and of alternatives to resolve the in-place pollutant problem have been conducted under the U. S. Environmental Protection Agency (EPA) "Superfund" program. Those investigations have identified creosote pollution in the entire reach of the Little Menomonee River from W. Brown Deer Road to its mouth. Based on the results of a remedial investigation, conducted by the U. S. Environmental Protection Agency, in cooperation with the Wisconsin Department of Natural Resources, completed in January 1990, a feasibility study was conducted by the Agency in cooperation with the Department to identify and evaluate remedial alternatives that would minimize or eliminate the health risks caused by site-related contaminants.

Six remedial action alternatives were evaluated in detail. The recommended plan for the Moss-American site provides for the following:

- Rerouting of the Little Menomonee River from the Moss-American site to its mouth.

- Removal and biological treatment of highly contaminated soil and river sediment, using an onsite treatment system.

- Burial of remaining sediments in the current streambed with soil excavated from the new channel.

- Burial of the untreated soil and the treated material from the treatment system onsite under a soil cover.

- Collection and treatment of contaminated groundwater with discharge to the sanitary sewerage system.

- Treatment of the landfilled soil onsite and its disposal onsite in a specially designed landfill.

The recommended remedial action plan is estimated to cost $26 million. Its annual operation and maintenance cost is $130,000. It is expected to take three to four years to complete.

The selected plan for the river channel is similar to the watershed study recommendation with the project extended downstream based on the findings of additional downstream contamination.

Butler Ditch

Butler Ditch has a total length of 4.0 miles. The 3.7-mile-long perennial stream reach of Butler Ditch Creek in the City of Brookfield and the Village of Menomonee Falls, which reach is located outside the current District limits but within an area defined in the policy plan as within possible future District limits, was found to meet the criteria for District jurisdiction. This reach was accordingly included in the system planning effort. The plan recommendations for each reach of Butler Ditch are as follows:

1. City of Brookfield and Village of Menomonee Falls from the Confluence with the Menomonee River to Lisbon Road at River Mile 3.40: No flood control improvements are recommended for this reach.

2. Village of Menomonee Falls from Lisbon Road to the Dolphin Drive Storm Sewer Outfall at River Mile 4.00: The Dolphin Drive storm sewer outlet is located below the existing streambed elevation at River Mile 4.00. In order to provide adequate stormwater drainage for the area tributary to the outlet, the Village has proposed deepening the Butler Ditch stream channel along its existing alignment between Dolphin Drive and Lisbon Road. Minimal channel modifications are recommended to achieve the necessary streambed grade to accommodate the Dolphin Drive outlet.

   The recommended modification would consist of lowering the existing channel bottom by from 0.1 to 2.8 feet along the 0.6-mile-long reach of Butler Ditch between Lisbon Road and the existing Dolphin Drive storm sewer outfall, as shown on Map 188. The resulting widened channel would have a bottom width of three feet and side slopes of one vertical on three horizontal. For purposes of this analysis, it was assumed that following deepening the channel would be allowed to revert to natural vegetative cover since it lies largely within an isolated wooded area.

Hydraulic Adequacy of Bridges

The drainage and flood control objectives and supporting principles and standards set forth in Chapter III specify the flood events which bridges shall accommodate without overtopping the related roadway. Based on those criteria, a number of bridges on the streams studied are considered hydraulically inadequate as shown in Appendix G.

It is recommended that when those bridges are replaced for transportation purposes, they be designed so as to accommodate the appropriate flood flow as specified in the drainage and flood control standards without overtopping the attendant roadway.

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Chapter X

SUMMARY OF MILWAUKEE METROPOLITAN SEWERAGE DISTRICT STORMWATER DRAINAGE AND FLOOD CONTROL SYSTEM PLAN

INTRODUCTION

The preceding chapters of this report presented alternative and recommended flood control and related drainage system plans for the streams proposed in the District policy plan to potentially be under the jurisdiction of the Milwaukee Metropolitan Sewerage District for flood control purposes.[1] Recommended plans are presented for the streams concerned in the Kinnickinnic River, Lake Michigan Direct Drainage Area, Oak Creek, Root River, Milwaukee River, and Menomonee River watersheds.

Specific information on certain pertinent characteristics of each watershed, such as hydrologic soil types and land slopes, and on existing and planned land use, appears in Chapter II of this report. The flood flows and stages for regulation purposes and for use in the design of all structural flood control works are based upon the planned land use conditions. System objectives and supporting standards and design criteria are set forth in Chapter III. Data on the stream flows and stages, and on areas subject to flooding and attendant damages are provided, together with the alternative and recommended plans by watershed, in Chapters IV through IX.

Flood control plans for all of the watersheds concerned, with the exception of the Lake Michigan Direct Drainage Area watershed, were previously studied under the comprehensive watershed planning program carried out by the Commission in cooperation with the local units of government in southeastern Wisconsin over the past 25 years. The recommended plan for each watershed as set forth in the earlier watershed studies has been refined under this system planning effort, using updated hydrologic and hydraulic models and accounting for changes in actual land use development and in land development and redevelopment plans. An original system plan was developed for the Lake Michigan Direct Drainage Area watershed. The system plans are recommended based upon careful consideration of technical feasibility, economic viability, environmental impacts, potential public acceptance, and practicality.

Streams having a total of 147 miles were evaluated under the system planning effort. Of these 147 miles, 114 stream miles are located within the current District limits and are recommended for District jurisdiction in the policy plan companion to this system plan. Of these 114 miles of stream currently under the District jurisdiction, flooding and stormwater drainage problems were identified along streams having about 99 stream miles, as shown in Table 119. In addition to the 114 stream miles currently under District jurisdiction, about 29 stream miles were studied which are located outside of the current District limits, but within an area identified in the policy plan as within possible future District limits and found to meet criteria for District jurisdiction. In most cases, these stream reaches were located in the upper reaches of the watersheds and any changes could impact on flood flows and stages and recommended flood control measures along the reaches within the District jurisdiction. Of these 29 stream miles, streams having a length of about 24 miles were found to have identified problems and are listed in Table 119. All or part of five streams, totaling about four miles in length, were not recommended for District jurisdiction: Hale Creek, the northern tributary of Southbranch Creek, a 0.3-mile reach of Butler Ditch, a 0.3-mile reach of Beaver Creek, and a 0.6-mile reach of the Upper Menomonee River. These were included in the system planning effort because these reaches were upstream of reaches under District jurisdiction and because flooding and stormwater problems potentially existed along those streams. Those problems may be expected to require flood control measures which may impact on flood flows and stages on, as well as flood control recommendations for, the downstream reaches under District jurisdiction. Thus,

[1] *The policy plan is documented in SEWRPC Community Assistance Planning Report No. 130, A Stormwater Drainage and Flood Control Policy Plan for the Milwaukee Metropolitan Sewerage District, March 1986.*

Table 119

SUMMARY OF SELECTED CHARACTERISTICS AND ECONOMIC ANALYSIS DATA FOR THE MILWAUKEE METROPOLITAN SEWERAGE DISTRICT STORMWATER DRAINAGE AND FLOOD CONTROL SYSTEM PLAN

Stream	Reach	Reach Length (miles)	Recommended Plan	Capital[a]	Cost: Total Amortized Capital[b]	Cost: Annual Operation and Maintenance	Cost: Total	Benefit-Cost Analysis: Annual Benefits	Benefit-Cost Analysis: Economic Benefit-Cost Ratio
1. Edgerton Channel	- -	0.8[c]	Channel modification and enclosure, bridge and road replacements	$ 2,276,000[d]	$ 144,300	$ 1,900	$ 146,200	$ 212,000	1.45
Edgerton Channel Subtotal		0.8	- -	$ 2,276,000[d]	$ 144,300	$ 1,900	$ 146,200	$ 212,000	1.45
2. Underwood Creek and Dousman Ditch	Underwood Creek from Milwaukee-Waukesha County line to its confluence with Dousman Ditch	2.6[c] 5.1[e]	Stormwater detention, structure floodproofing and elevation	$ 1,736,000	$ 110,000	$ 20,000	$ 130,000	$ 166,000	1.28
	Dousman Ditch from its mouth to Wisconsin Avenue storm sewer outlet	2.5[e]							
Underwood Creek and Dousman Dousman Ditch Subtotal		10.2	- -	$ 1,736,000	$ 110,000	$ 20,000	$ 130,000	$ 166,000	1.28
3. Oak Creek	Oak Creek	8.4[c] 4.7[e]	Channel modification, floodproofing and elevation, bridge replacement	$ 1,193,000	$ 76,000	$ 2,000	$ 78,000	$ 88,000	1.13
	Mitchell Field Drainage Ditch	3.3[c]							
	North Branch of Oak Creek	5.7[c]							
Oak Creek Subtotal		22.1	- -	$ 1,193,000	$ 76,000	$ 2,000	$ 78,000	$ 88,000	1.13
4. Wilson Park Creek	- -	5.3[c]	Major channelization, bridge modification	$ 520,000	$ 33,000	$ 2,600	$ 35,600	$ 39,000	1.10
Wilson Park Creek Subtotal		5.3	- -	$ 520,000	$ 33,000	$ 2,600	$ 35,600	$ 39,000	1.10
5. Indian Creek		1.9[c]	Culvert replacement	$ 290,000	$ 18,400	$ - -	$ 18,400	$ 18,500	1.01
Indian Creek Subtotal	- -	1.9	- -	$ 290,000	$ 18,400	$ - -	$ 18,400	$ 18,500	1.01
6. Lincoln Creek	Upper Lincoln Creek	2.9[c]	Channel modification, dikes, bridge replacement, channel cleaning	$ 1,525,000[f]	$ 45,100[f]	$ 1,100	$ 46,200	$ 42,000	0.91
	Lower Lincoln Creek	5.6[c]	Channel modification, dikes, floodwalls, bridge replacement	20,340,000[g]	1,301,900	27,100	1,329,000	802,000	0.60
Lincoln Creek Subtotal		8.5	- -	$21,865,000	$1,347,000	$ 28,200	$1,375,200	$ 844,000	0.61
7. East Branch of Root River	Franklin	4.9[c]	Structure floodproofing, elevation, removal	$ 153,000	$ 9,700	$ - -	$ 9,700	$ 6,600	0.68
East Branch of Root River Subtotal		4.9	- -	$ 153,000	$ 9,700	$ - -	$ 9,700	$ 6,600	0.68
8. North Branch Whitnall Park Creek, Northwest Branch Whitnall Park Creek, Whitnall Park Creek	- -	3.0[c]	Channel modification, culvert installation, floodproofing	$ 1,433,200	$ 91,000	$ 6,200	$ 97,200	$ 50,500	0.52
North Branch and Northwest Branch Whitnall Park Creek and Whitnall Park Creek Subtotal		3.0	- -	$ 1,433,200	$ 91,000	$ 6,200	$ 97,200	$ 50,500	0.52

Table 119 (continued)

Stream	Reach	Reach Length (miles)	Recommended Plan	Capital[a]	Cost: Total Amortized Capital[b]	Cost: Annual Operation and Maintenance	Cost: Total	Benefit-Cost Analysis: Annual Benefits	Benefit-Cost Analysis: Economic Benefit-Cost Ratio
9. North Branch of Root River and Hale Creek	- -	14.3[c] 2.2[e] 1.0[h]							
	West Allis	- -	Structure floodproofing, elevation, removal, minor channel deepening	$ 1,483,000[i]	$ 94,000	$ 5,100	$ 99,100	$ 28,250	0.28
	Greenfield	- -	Structure floodproofing, elevation, removal	1,676,000	106,300	- -	106,300	35,000	0.33
	Franklin	- -	Structure floodproofing, elevation, removal	329,000	20,900	- -	20,900	8,600	0.41
North Branch of Root River and Hale Creek Subtotal		17.5	- -	$ 3,488,000	$ 221,200	$ 5,100	$ 226,300	$ 71,850	0.32
10. Brown Deer Park Creek	- -	2.0[c]	Structure floodproofing	$ 4,600	$ 290	$ - -	$ 290	$ 90	0.31
Brown Deer Park Creek Subtotal		2.0	- -	$ 4,600	$ 290	$ - -	$ 290	$ 90	0.31
11. Southbranch Creek	Southbranch Creek	1.5[c]	Channel modification, culvert replacement, detention storage	$ 1,265,000	$ 80,300	$ 14,000	$ 94,300	$ 27,250	0.29
	North Tributary to Southbranch Creek	1.4[h]	Channel modification, culvert replacement, detention facilities	1,550,000	98,300	12,600	110,900	- -[j]	- -[j]
Southbranch Creek Subtotal		2.9	- -	$ 2,815,000	$ 178,600	$ 26,600	$ 205,200	$ 27,250	0.29[k]
12. Upper Menomonee River and Little Menomonee River	Upper Menomonee River in Village of Menomonee Falls at River Mile 14.74 and from River Mile 19.42 to 19.94	5.5[c] 9.3[e] 0.6[h]	Structure floodproofing	$ 74,000	$ 4,700	$ - -	$ 4,700	$ 2,400	0.51
	Upper Menomonee River in Village of Menomonee Falls from Roosevelt Drive through Erika Road extended		Channel modification	559,000	35,000	2,000	37,000	10,000[l]	0.27
	Upper Menomonee River in Village of Germantown at River Miles 23.48, 24.19, and 24.33		Structure floodproofing and elevation	35,000	2,200	- -	2,200	200	0.09
	Little Menomonee River at River Miles 3.80, 6.90, and 6.93	7.0[c]	Structure floodproofing	14,000	900	- -	900	1,600	1.78
Upper Menomonee River and Little Menomonee River Subtotal		22.4	- -	$ 682,000	$ 42,800	$ 2,000	$ 44,800	$ 14,200	0.32
13. Lower Menomonee River	City of Milwaukee from 27th Street Viaduct to IH 94		Dike and floodwall raises	$ 240,000	$ 15,000	$ - -	$ 15,000	$ - -[m]	- -[m]
	City of Milwaukee from IH 94 to W. Michigan Street extended		Structure floodproofing and elevation	370,000	23,000	- -	23,000	3,100	0.13
	N. 43rd Street extended in the City of Milwaukee through Glenview Avenue extended in the City of Wauwatosa		Channel modification, dikes, structure floodproofing and elevation	5,988,000	380,000	6,000	386,000	129,000	0.33

Table 119 (continued)

Stream	Reach	Reach Length (miles)	Recommended Plan	Capital[a]	Cost: Total Amortized Capital[b]	Cost: Annual Operation and Maintenance	Cost: Total	Benefit-Cost Analysis: Annual Benefits	Benefit-Cost Analysis: Economic Benefit-Cost Ratio
13. Lower Menomonee River (continued)	City of Wauwatosa from W. Harwood Avenue through W. Capitol Drive		Structure floodproofing	$ 29,000	$ 1,800	$ - -	$ 1,800	$ 900	0.50
Lower Menomonee River Subtotal		10.3[c]	- -	$ 6,627,000	$ 419,800	$ 6,000	$ 425,800	$ 133,000	0.31
14. Beaver Creek	- -	1.9[c] 0.3[h]	Channel modification, culvert replacement	$ 202,300	$ 13,000	$ 600	$ 13,600	$ 750	0.06
Beaver Creek Subtotal		2.2	- -	$ 202,300	$ 13,000	$ 600	$ 13,600	$ 750	0.06
15. Honey Creek	City of Wauwatosa at River Mile 0.85		Structure floodproofing	$ 50,000	$ 3,200	$ - -	$ 3,200	$ 2,200	0.69
	City of Greenfield from IH 894 through the S. 43rd Street storm sewer outfall		Channel modification, detention storage, and bridge removal and replacement	1,282,000	81,000	12,000	93,000	900[j]	0.01[j]
Honey Creek Subtotal		8.8[c]	- -	$ 1,332,000	$ 84,200	$ 12,000	$ 96,200	$ 3,100	0.03[j]
16. Butler Ditch	W. Lisbon Road to Dolphin Drive storm sewer outlet	0.3[h]	Minor channel modification	$ 60,000	$ 4,000	$ - -	$ 4,000	$ - -[j]	- -[j]
Butler Ditch Subtotal		0.3	- -	$ 60,000	$ 4,000	$ - -	$ 4,000	$ - -[j]	- -[j]
17. Crayfish Creek and Caledonia Branch of Crayfish Creek	- -	1.0[c]	Channel modification, debrushing, berm and culvert construction	$ 399,000	$ 25,300	$ 10,900	$ 36,200	$ - -[j]	- -[j]
Crayfish Creek and Caledonia Branch of Crayfish Creek Subtotal		1.0	- -	$ 399,000	$ 35,300	$ 10,900	$ 46,200	$ - -[j]	- -[j]
18. Woods Creek	- -	1.1[c]	Construction of relief culvert	$ 981,000	$ 62,000	$ - -	$ 62,000	$ - -[j]	- -[j]
Woods Creek Subtotal		1.1	- -	$ 981,000	$ 62,000	$ - -	$ 62,000	$ - -[j]	- -[j]
19. S. 43rd Street Ditch[n]	- -	1.1[c]	Dike construction and channel debrushing	$ 135,000	$ 8,600	$ 8,500	$ 17,100	$ - -[j]	- -[j]
S. 43rd Street Ditch Subtotal		1.1	- -	$ 135,000	$ 8,600	$ 8,500	$ 17,100	$ - -[j]	- -[j]
Total		126.3	- -	$46,192,100	$2,889,190	$132,600	$3,021,790	$1,674,840	0.62[o]

[a]*Capital cost is expressed in 1986 dollars.*

[b]*Amortized capital cost is based on an interest rate of 6 percent and a project life of 50 years.*

[c]*Stream reach under current District jurisdiction (98.9 miles).*

[d]*Capital cost was refined as part of a separate analysis conducted after Chapter IV was completed. Includes cost for reconstructing S. Nicholson Road and an existing frontage road over the proposed culvert, and building new bridges at the Chicago & North Western Railway line and at the utility road just upstream of the railway. Costs are not included for the replacement of the S. Pennsylvania Avenue bridge, since the bridge is being replaced for transportation purposes.*

[e]*Stream reach under recommended for potential District jurisdiction (23.8 miles).*

[f]*Amortized capital cost is based upon $712,000 and does not include $813,000 for channel modifications required for adequate storm sewer outlet improvements, which would not result in direct flood abatement benefits.*

[g]*Amortized capital cost includes the replacement of pumps after 25 years of operation.*

[h]*Stream reach not recommended for District jurisdiction (3.6 miles).*

[i]*Costs for bridges at W. Cleveland Avenue on the North Branch of the Root River and W. Cleveland Avenue on Hale Creek were previously assigned under the Commission's adopted regional transportation system plan.*

[j]*Benefits primarily due to provision of adequate outlets for storm sewers are not normally quantified. Benefits to stormwater drainage system, if included, would result in a higher benefit-cost ratio.*

[k]*Benefit-cost ratio calculated only for Southbranch Creek.*

[l]*Includes flood damage reduction at three buildings located upstream in the Village of Germantown.*

[m]*Same approximate level of flood protection. Dike and floodwall raised to provide three feet of freeboard.*

[n]*Need for implementation dependent upon construction of major storm sewer in the City of West Allis.*

[o]*Benefit-cost ratio calculated using the total annual cost, excluding costs for stream reaches Nos. 11b, 13a, 15b, 16, 17, 18, and 19, which costs are primarily associated with stormwater drainage system improvements or other purposes which do not result in direct flooding relief benefits.*

Source: SEWRPC.

these five stream reaches were also considered in the system planning effort.

Recommended System Plan

The recommended system plan for the Milwaukee Metropolitan Sewerage District consists of a carefully selected combination of structural and nonstructural measures. The basic nonstructural plan measures consist of carrying out the regional land use plan. By guiding future land use development in accordance with the plan, the intensification of existing, and the creation of new, drainage and flood control problems can be avoided. The importance of the land use recommendations cannot be overemphasized. Not only will the location, extent, and density or intensity of urban land use development affect the flood flows and stages upon which the system plan is based, but the preservation of the environmental corridors will preserve a high volume of floodwater storage capacity along the stream systems.

The major structural measures include a selected combination of storage; channel modification; structure floodproofing, elevation, and removal; diking; stormwater pumping; bridge and culvert replacement; and new channel construction. As already noted, the plan emphasizes the provision of substantial floodwater storage through the preservation of essentially all natural floodplain storage remaining in the study area and the construction of selected storage facilities. These measures are listed in Table 119 and are shown on Map 189. Those stream reaches for which no significant stormwater drainage and flooding problems were identified are listed in Table 120.

The recommended plan would essentially eliminate all flood-related damages during flood events up to and including a 100-year recurrence interval event under planned land use and channel conditions along the identified stream reaches set forth in Table 119. The flood control measures recommended are designed primarily to alleviate flood damages from direct overland flooding along the stream reaches studied, as well as to provide an adequate outlet for local storm sewers and drainageways. These measures, although not specifically designed to do so, may also be expected to materially reduce damages due to localized stormwater drainage problems or sanitary sewer backup.

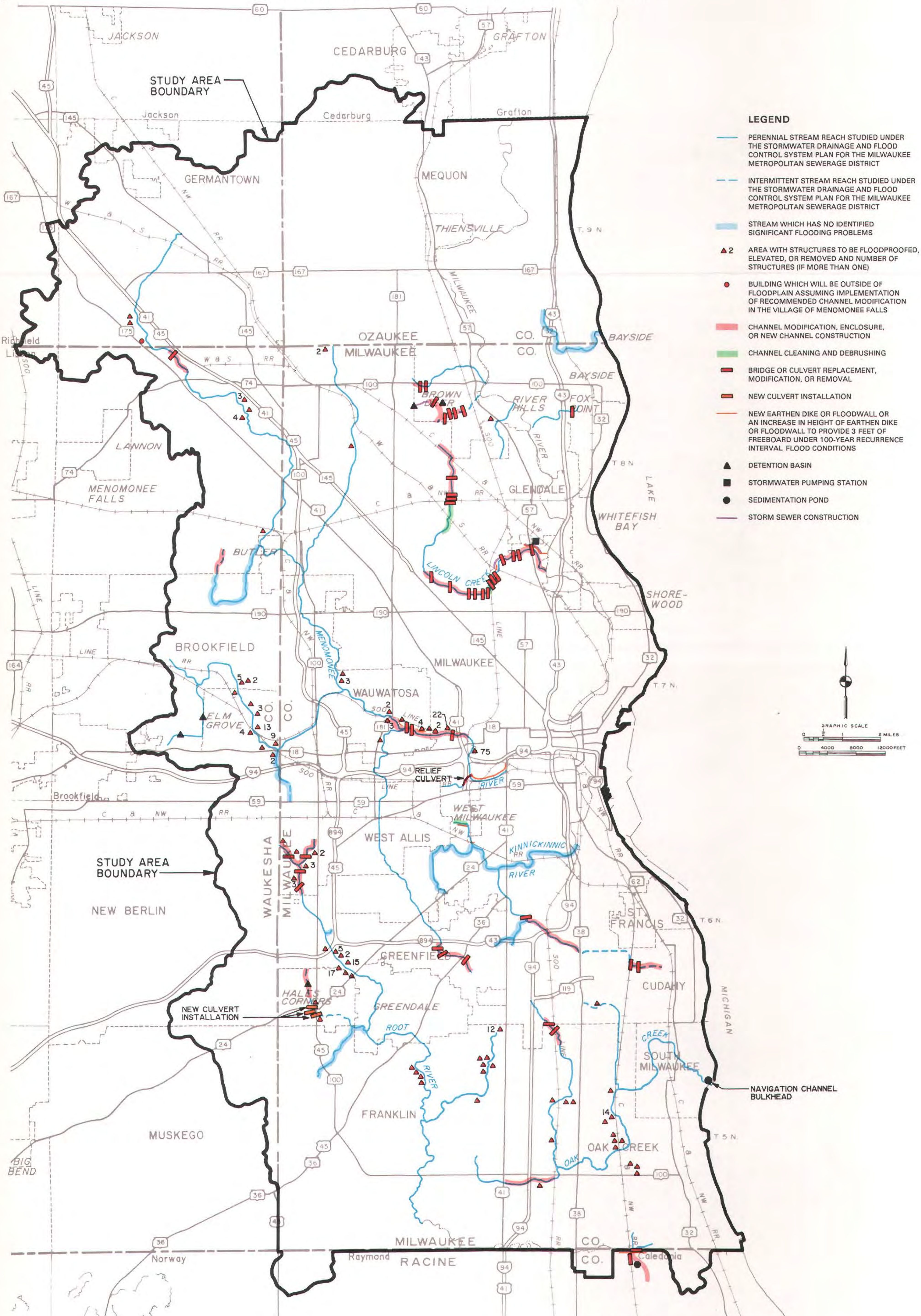
As shown in Table 119, the total capital cost of the recommended stormwater drainage and flood control improvements is estimated to be about $46.2 million, with a total annual operating and maintenance cost of $133,000. The total annual cost of the amortized capital and operation and maintenance costs is estimated to be $3.0 million. The total annual direct benefits expected are about $1.7 million. In addition to the direct benefits due to the abatement of direct overland flooding conditions, there are unquantified benefits relating to the provision of adequate outlets for stormwater drainage systems and a substantial reduction in the amount of annual flood insurance premiums required to be paid within the service area. This insurance premium cost is estimated to total approximately $600,000 per year in Milwaukee County. In addition, there are substantial indirect benefits due to the alleviation of mental stress, closure of roads, lost travel time, and lost time at work, losses experienced during flooding conditions but are not generally quantified. Most importantly, there are indirect benefits from the abatement of the severe public health and public safety hazards associated with flooding in urban areas.

In addition to the costs and improvements set forth in Table 119, there are a number of bridges on the streams evaluated in this system plan which do not currently meet the objective and supporting principals and standards set forth in Chapter III with regard to overtopping of the related roadways. Based on those criteria, a number of bridges on the streams studied are considered hydraulically inadequate as shown in Appendices B through G. It is recommended that when those bridges are replaced for transportation purposes, they be designed so as to accommodate the appropriate flood flow as specified in the drainage and flood control standards without overtopping the attendant roadway.

Finally, but importantly, the plan recommends that each of the units of government in the watersheds located within the study area review its floodland zoning regulations to ensure that such regulations complement the recommended watershed land use plan element and are coordinated with the structural flood control measures recommended in the plan. In general, those floodlands lying within the 100-year recurrence interval flood hazard lines under year 2000 planned land use conditions that are presently neither developed for urban use nor committed to such use by the recordation of land subdivi-

Map 189

RECOMMENDED MILWAUKEE METROPOLITAN SEWERAGE DISTRICT STORMWATER DRAINAGE AND FLOOD CONTROL SYSTEM PLAN



Source: SEWRPC.

Table 120

SELECTED STREAM REACHES STUDIED UNDER THE MILWAUKEE METROPOLITAN SEWERAGE DISTRICT STORMWATER DRAINAGE AND FLOOD CONTROL SYSTEM PLAN WHICH HAVE NO IDENTIFIED, SIGNIFICANT FLOODING PROBLEMS

Streams Recommended for Milwaukee Metropolitan Sewerage District Jurisdiction	
Stream and Reach	Length (miles)
Kinnickinnic River Watershed	
Kinnickinnic River Watershed	5.7[a]
Lyons Park Creek	1.3[a]
Villa Mann Creek	1.7[a]
Lake Michigan Direct Drainage Area	
Fish Creek	3.4 (2.1)[a] (1.3)[b]
Menomonee River Watershed	
South Branch of Underwood Creek	1.6[a]
Butler Ditch (south of Lisbon Road)	3.7[b]
Root River Watershed	
Tess Corners Creek	2.6[a]
104th Street Branch	0.4[a]
Total	20.4

[a] *Stream reach under current District jurisdiction (15.4 miles).*

[b] *Stream reach recommended for potential future District jurisdiction (5.0 miles).*

Source: SEWRPC.

sion plats and the installation of municipal improvements should be zoned so as to prohibit further urban development. Those existing urban land uses in the floodland recommended to be floodproofed, elevated, or protected through structural flood control measures should be placed in a flood hazard district until implementation of the recommended flood control measures, at which time the lands should be appropriately rezoned.

In selected reaches of the streams studied there is a lack of suitable large-scale topographic mapping needed to provide an accurate delineation of the floodplains and, in some cases, to carry out the detailed design of selected structural measures. Thus, it is recommended that the Milwaukee Metropolitan Sewerage District prepare large-scale topographic maps for the areas shown on Map 190, except for the Northwest one-quarter of U. S. Public Land Survey Section 27, Township 7 North, Range 20 East, where it is recommended that the City of Brookfield prepare a large-scale topographic map. New mapping is recommended to cover about 51.75 square miles along the stream reaches concerned.

PROJECT PRIORITIZATION

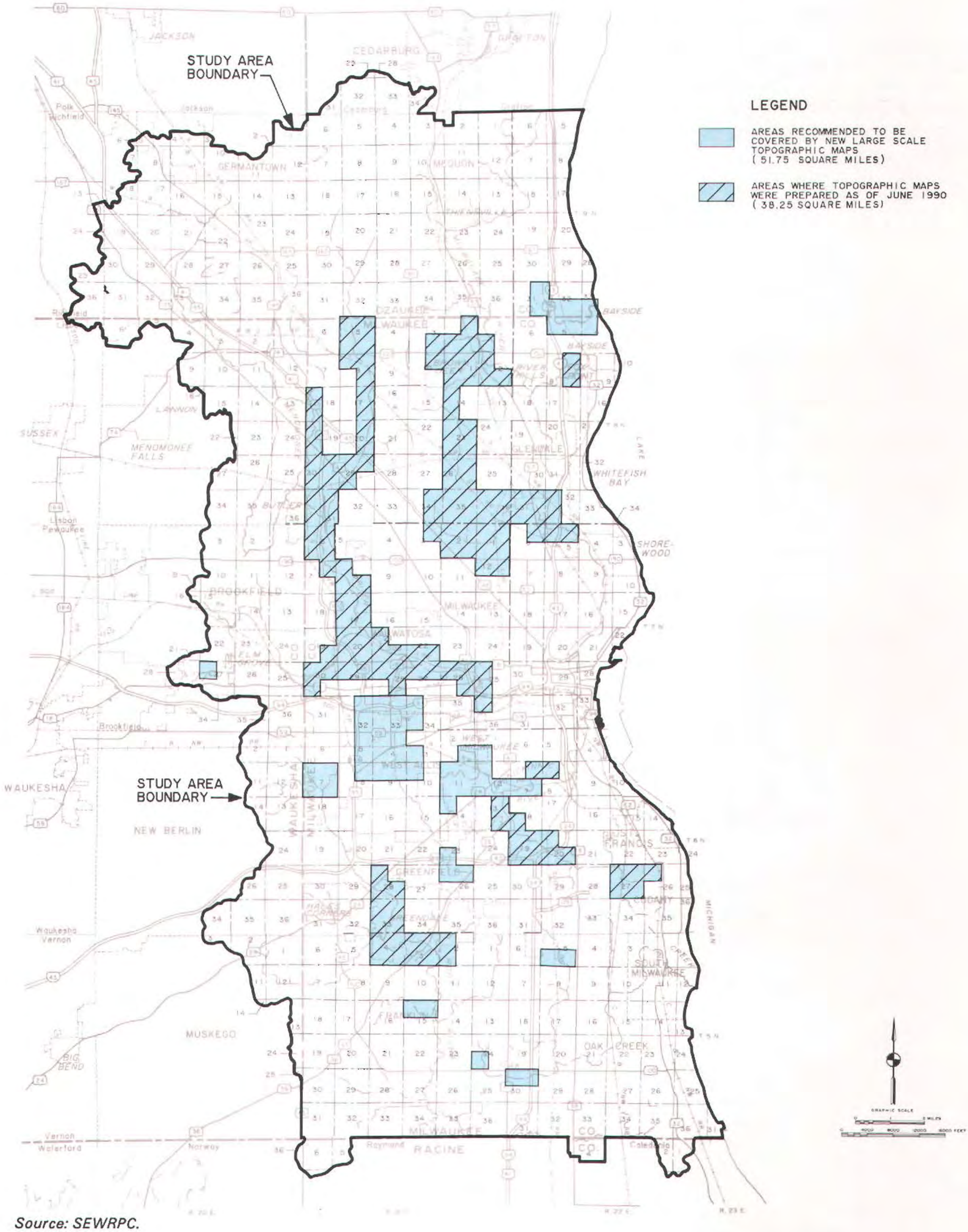
The recommended system plan has been listed by project reach in Table 119 in order of the calculated benefit-cost ratio. It is recommended that the plan be implemented generally on the basis of the benefit-cost ratios from highest to lowest. However, it is also recommended that selected projects be given special consideration due to the need for the stormwater drainage system outlet improvements the benefits of which are not generally quantified in the benefit-cost ratio. Such consideration would apply to the reach of Honey Creek in the City of Greenfield. In addition, the timing of related projects should be considered in the final project priority schedules selected by the Milwaukee Metropolitan Sewerage District. For example, if construction of a new stadium is implemented, consideration should be given to the need to implement the Woods Creek improvements as other construction in the area takes place. In addition, where safety hazards which could result in the potential loss of life are known to exist, which could be reduced by the proposed improvements, such consideration should be considered overriding.

PROJECT COST ALLOCATION

The apportionment of the recommended plan costs among the various public agencies and private owners concerned are set forth in Table 121. This apportionment is based upon the recommended policies as set forth in the aforementioned policy plan. As shown in Table 121, of the total capital costs of $46.2 million estimated for carrying out the system plan imple-

Map 190

RECOMMENDED LARGE-SCALE TOPOGRAPHIC MAPPING



Source: SEWRPC.

mentation, approximately \$30.5 million, or 66 percent, would be borne by the Milwaukee Metropolitan Sewerage District. Of the remaining 15.7 million, \$12.0 million would be borne by the 12 local and by the State units of government affected by the current flooding problems, and \$3.7 million would be borne by private property owners.

The cost allocation set forth in Table 121 assumes that the private property owners concerned will be responsible for costs associated with structure floodproofing and elevation when such elements are included as part of the recommended plan. This assumption is consistent with the adopted District policy plan as documented in SEWRPC Community Assistance Planning Report No. 130, A Stormwater Drainage and Flood Control Plan for the Milwaukee Metropolitan Sewerage District. During development of the system plan, requests were made to change the policy plan to provide District funding of requested floodproofing and elevation measures. Such requests were made by members of the Technical Advisory Committee, two local units of government, and citizens at public meetings on the proposed system plan. In view of these requests, it is recommended that the District reconsider this provision of the policy plan.

IMPACT OF RECOMMENDED FLOOD CONTROL SYSTEM PLAN ON FLOOD FLOWS AND STAGES

Structural flood control measures recommended for streams studied under this system plan will in some cases improve the hydraulic efficiency of the channel system and may be expected to increase, to the degree quantified in the system analyses, downstream flood flows and stages. These measures include channel modification, channel enclosure, bridge and culvert replacement, and channel cleaning and debrushing. Hydrologic and hydraulic analyses were conducted as part of this system plan to determine the impact of the recommended flood control measures on downstream flood flows and stages. These analyses considered the impact along the stream reaches studied as well as along any downstream reaches potentially impacted. A comparison of the 100-year recurrence interval flood flows and flood stages under planned land use with existing and planned channel conditions is provided for each watershed in the pertinent previous chapters of this report. In most cases, flood control and stormwater drainage measures are not expected to increase downstream flood flows and stages to any extent. In a few cases, increases of 0.1 to 0.6 foot are expected. However, no significant increase in downstream flooding problems is expected.

CONCLUSION

Adoption and implementation of the recommended system plan for the Milwaukee Metropolitan Sewerage District may be expected to result in the abatement of the most serious and costly flooding problems in Milwaukee County and in attendant drainage improvements. Importantly, such adoption and implementation may be expected to minimize the intensification of existing and the development of new problems of this kind. Consequently, implementation of the plan may be expected to provide a safer, more healthful, and more pleasant, as well as more orderly and efficient, environment for life in the watersheds concerned. Failure to implement the system plan may be expected to result in the further intensification of flooding and stormwater drainage problems and in the creation of new problems which will be even more expensive to resolve.

Table 121

SUMMARY OF CAPITAL COSTS FOR MILWAUKEE METROPOLITAN SEWERAGE DISTRICT STORMWATER DRAINAGE AND FLOOD CONTROL SYSTEM PLAN BY IMPLEMENTING AGENCY

Stream	Reach	Milwaukee Metropolitan Sewerage District	Milwaukee County	Wisconsin Department of Natural Resources	City of Brookfield	City of Cudahy	City of Franklin	City of Milwaukee	City of Wauwatosa	City of West Allis	Village of Brown Deer	Village of Elm Grove	Village of Hales Corners	Village of Menomonee Falls	Private Property Owners	Total[a]
1. Wilson Park Creek	--	$ 490,000	$ 30,000	$--	$--	$--	$--	$--	$--	$--	$--	$--	$--	$--	$--	$ 520,000
2. Edgerton Channel	--	2,265,000[b]	--	--	--	11,000[b]	--	--	--	--	--	--	--	--	--	2,276,000
3. Underwood Creek and Dousman Ditch	Underwood Creek from Milwaukee-Waukesha County line to confluence with Dousman Ditch and Dousman Ditch from its mouth to Wisconsin Avenue storm sewer outlet	--	--	--	168,000[c]	--	--	--	--	--	--	504,000[c]	--	--	1,064,000	1,736,000
4. Oak Creek	--	548,000	--	--	--	--	--	--	--	--	--	--	--	--	645,000	1,193,000
5. Indian Creek	Village of Fox Point	29,000	261,000	--	--	--	--	--	--	--	--	--	--	--	--	290,000
6. Lincoln Creek	Upper and Lower Lincoln Creek	15,021,000	--	10,000	--	--	--	6,834,000[b]	--	--	--	--	--	--	--	21,865,000
7. Whitnall Park Creek, North Branch of Whitnall Park Creek, and Northwest Branch of Whitnall Park Creek	--	1,037,600	--	--	--	--	--	--	--	--	--	--	293,200	--	102,400	1,433,200
8. East Branch of Root River	--	--	--	--	--	--	84,000	--	--	--	--	--	--	--	69,000	153,000
9. North Branch of Root River and Hale Creek	--	2,242,000	18,000	--	--	--	--	--	--	634,000	--	--	--	--	594,000	3,488,000
10. Brown Deer Park Creek	--	--	--	--	--	--	--	--	--	--	--	--	--	--	4,600	4,600
11. Southbranch Creek	Southbranch Creek and north tributary to Southbranch Creek	1,064,000	--	--	--	--	--	420,000	--	--	1,331,000	--	--	--	--	2,815,000

Table 121 (continued)

Stream	Reach	Milwaukee Metropolitan Sewerage District	Milwaukee County	Wisconsin Department of Natural Resources	City of Brookfield	City of Cudahy	City of Franklin	City of Milwaukee	City of Wauwatosa	City of West Allis	Village of Brown Deer	Village of Elm Grove	Village of Hales Corners	Village of Menomonee Falls	Private Property Owners	Total[a]
12. Upper Menomonee	River Mile 14.74 to River Mile 24.33	$- -	$- -	$- -	$- -	$- -	$- -	$- -	$- -	$- -	$- -	$- -	$- -	$559,000	$ 109,000	$ 668,000
12a. Little Menomonee	River Miles 3.80, 6.90, and 6.93	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	14,000	14,000
13. Lower Menomonee	27th Street Viaduct through W. Capitol Drive	5,421,000	- -	- -	- -	- -	- -	- -	170,000	- -	- -	- -	- -	- -	1,036,000	6,627,000
14. Beaver Creek	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	202,300	- -	- -	- -	- -	202,300
15. Honey Creek	- -	915,000	367,000	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	50,000	1,332,000
16. Butler Ditch	W. Lisbon Road to Dolphin Drive storm sewer outlet	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	60,000	- -	60,000
17. Crayfish Creek	Crayfish Creek and Caledonia Branch	399,000	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	399,000
18. Woods Creek	- -	981,000	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	981,000
19. S. 43rd Street Ditch	- -	135,000	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	135,000
Total	- -	$30,547,600	$676,000	$10,000	$168,000[c]	$11,000[b]	$84,000	$7,254,000	$170,000	$634,000	$1,633,300	$504,000[c]	$293,200	$619,000	$3,688,000	$46,192,100

[a]*All costs given in 1986 dollars.*

[b]*Portions of this cost may be allocated to the Wisconsin Department of Transportation.*

[c]*Cost of recommended detention basin assigned in proportion to the flood damage mitigation benefits derived by the City of Brookfield and the Village of Elm Grove.*

Source: SEWRPC.

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Special acknowledgement is due Messrs. Michael G. Hahn and Ronald J. Printz, SEWRPC Principal Engineers, for their contribution to the preparation of this report.