COMMUNITY ASSISTANCE
PLANNING REPORT NO. 121

A STORMWATER MANAGEMENT PLAN FOR THE VILLAGE OF HALES CORNERS

MILWAUKEE COUNTY WISCONSIN

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NUMBER 121

A STORMWATER MANAGEMENT PLAN FOR THE VILLAGE OF HALES CORNERS

Village of Hales Corners
Milwaukee County, Wisconsin

Prepared by the
Southeastern Wisconsin Regional Planning Commission
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March 1986

Inside Region $5.00
Outside Region $10.00

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SOUTHEASTERN WISCONSIN REGIONAL PLANNING COMMISSION

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March 21, 1986

Village President, Village Board,
and Village Public Works Commission
c/o Village Clerk
Village of Hales Corners
5635 S. New Berlin Road
Hales Corners, Wisconsin 53130

Ladies and Gentlemen:

On January 25, 1984, the Village of Hales Corners requested the Southeastern Wisconsin Regional Planning Commission to assist the Village in the preparation of a stormwater management plan. The Regional Planning Commission, working in cooperation with the firm of W. G. Nienow Engineering Associates, the firm which had historically served as Village Engineers, and the Village Public Works Commission and staff, has now completed the technical work required, and is pleased to herewith transmit a recommended stormwater management plan for consideration and adoption by the Village Public Works Commission, the Village Plan Commission, and the Village Board.

The stormwater management plan presented herein is consistent with regional as well as local land use development, water quality management, and flood control objectives, and is intended to serve as a guide to village officials in the making of sound decisions over time concerning the development of stormwater management facilities in the Village of Hales Corners.

The Regional Planning Commission is appreciative of the assistance offered by village officials and staff in the preparation of this report. The Commission staff stands ready to assist the Village in securing the adoption of the plan and in its implementation over time.

Sincerely,

Kurt W. Bauer
Executive Director

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TABLE OF CONTENTS

Page

LIST OF APPENDICES

Appendix A (continued)

LIST OF TABLES

Chapter II

LIST OF FIGURES

Figure | Page

Chapter VI

LIST OF MAPS

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Chapter I

INTRODUCTION

The Village of Hales Corners is located in southwestern Milwaukee County within the Root River watershed. The 1980 resident population of the Village was approximately 7,100 persons, with a projected year 2000 population of 8,500 persons. As of 1980, the areal extent of the Village was approximately 2,073 acres, or 3.2 square miles. Of this total area, about 1,670 acres, or 81 percent, was devoted to urban uses and the remaining 403 acres were in open lands. Of these open lands, approximately 205 acres, or 51 percent, were agricultural or unused lands generally suitable for urban development, and the remaining 198 acres constituted important natural resource features, including surface waters, wetlands, and woodlands.

The Village of Hales Corners has a history of drainage problems. The most persistent problems appear to be related to high groundwater levels which require the operation of building sump pumps over extended periods of time and contribute to ponding of stormwater in 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.

These drainage problems may be expected to be exacerbated by the further development of the approximately 205 acres of remaining agricultural or unused lands within the Village; and, importantly, by the further development of the approximately 528 acres of remaining open lands within the Cities of New Berlin and Greenfield from which stormwater drains into and through the Village of Hales Corners. In addition, consideration must be given to stormwater runoff from currently developed lands in the Cities of Muskego and Franklin which also drain into and through the Village. Recognizing the need to abate the existing stormwater drainage problems and avoid the creation of new problems as development proceeds in the area, the Village Board in February 1984 authorized the preparation of a stormwater management plan for the Village.

The purpose of this report is to present that management plan. The plan seeks to promote the development of an effective stormwater management system for the Village, adequate to serve the Village at least through the turn of the century. Ultimately, to the extent practicable, the system is designed to minimize damages attendant to poor drainage while reducing downstream flooding. More specifically, this report:

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;

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.

This report was prepared by the staff of the Southeastern Wisconsin Regional Planning Commission in cooperation with the staff of the Village of Hales Corners and the firm of W. G. Nienow Engineering Associates, which together with the predecessor firm of H.C. Webster & Son have served as the village engineers for many decades. The recommended stormwater management plan for the Village, as presented herein, is properly set within the context of broader flood control and water quality management plans for the Root River watershed.[1]

DISTINCTION BETWEEN STORMWATER DRAINAGE AND FLOOD CONTROL

Both stormwater drainage and flood control deal with the problems of disposal of unwanted water, and the distinction between these two issues is not always clear. For the purposes of this report, flood control is defined as the prevention of damage from the overflow of natural streams and watercourses. Drainage is defined as the disposal of excess stormwater on the land surface before such water has entered stream channels. This report focuses on the latter, and addresses flood control only as necessary to avoid the intensification of existing, or the creation of new, flood control problems along the natural streams and watercourses of the study area which must receive the discharge from the existing and proposed stormwater drainage facilities.

NEED FOR AND IMPORTANCE OF STORMWATER MANAGEMENT PLANNING

Stormwater drainage--the collection, transport, and disposal of excess stormwater--is one of the most important and costly requirements of sound urban

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[1]See SEWRPC Planning Report No. 9, A Comprehensive Plan for the Root River Watershed, July 1966; and SEWRPC Community Assistance Planning Report No. 37, A Nonpoint Source Water Pollution Control Plan for the Root River Watershed, March 1980. Also see SEWRPC Planning Report No. 30, A Regional Water Quality Management Plan for Southeastern Wisconsin: 2000, Volume One, Inventory Findings, September 1978; Volume Two, Alternative Plans, February 1979; and Volume Three, Recommended Plan, June 1979. The Root River watershed plan has been formally adopted by the Wisconsin Department of Natural Resources (DNR), Milwaukee County, Milwaukee Metropolitan Sewerage District, and City of Franklin, as well as by the Regional Planning Commission. The nonpoint source water pollution abatement plan for the Root River watershed has been adopted by the DNR, the City of Greenfield, and the Village of Hales Corners, as well as by the Commission. The regional water quality management plan has been adopted by the DNR, Milwaukee and Waukesha Counties, the Cities of Greenfield and New Berlin, and the Village of Hales Corners, as well as by the Commission.

development. Good stormwater drainage is essential to the provision of an attractive and efficient, as well as safe and healthful, environment for urban life.

Inadequate stormwater drainage, however, can be even more costly than the provision of adequate drainage. Inadequate stormwater drainage can disrupt the safe and efficient movement of people and goods essential to the proper functioning of an urban area; undermine the structural stability of pavements, utilities, and buildings, requiring costly maintenance and reconstruction; and depreciate and destroy the market value of real property with an attendant loss of tax base. Inadequate stormwater drainage can result in the excessive infiltration and inflow of clear water into sanitary sewerage systems with attendant surcharging of sanitary sewers, the backing of sanitary sewage into residential and commercial buildings, the bypassing of raw sewage to streams and watercourses through sanitary sewer system flow relief devices, and the creation of serious hazards to public health. In extreme situations, inadequate stormwater drainage can constitute a hazard to human life. Inadequate stormwater drainage can also cause serious and costly soil erosion and sedimentation, create unsightly depositions of debris, and promote the breeding of mosquitoes and other troublesome insects with attendant hazards to the health of humans and of domestic animals.

Municipal officials have long recognized the hazards to human health and safety, and the economic losses, caused by inadequate stormwater drainage. Such officials are increasingly recognizing the adverse ecological and environmental impacts of improperly managed stormwater runoff, including the pollution of surface waters, the reduction of groundwater recharge, and the adverse effects on desirable forms of plant and animal life.

Because of its important social, economic, and environmental impacts, stormwater drainage is a problem which requires sound resolution through fairly sophisticated planning and engineering. The factors which must be considered in the planning and design of stormwater drainage facilities are complex and highly interrelated. Perhaps the most important of these factors is the magnitude and frequency of the flows that must be accommodated. Yet, this variable cannot be determined with certainty since it is dependent on the occurrence of random meteorological events, as well as on topographic, pedologic, and land use conditions. Moreover, the factors determining the quantity and quality of the runoff to be accommodated by an urban stormwater drainage system are altered by urbanization itself, which particularly affects the overall imperviousness of the catchment areas concerned, reducing the infiltration capacity of soils, the amount of natural depression storage, and the flow times in the drainage system, thereby significantly increasing the rate and volume of stormwater runoff.

Careful application of the sciences of hydrology and hydraulics, as well as the art of urban engineering, is therefore important to the sound planning and design of urban stormwater drainage systems. Hydrology may be defined as the study of the physical behavior of the water resource from its occurrence as precipitation to its entry into streams and watercourses or its return to the atmosphere via evapotranspiration. The application of hydrology to the planning and design of urban stormwater drainage systems requires the collection and analyses of definitive information on precipitation, soils, and land uses, and on the volume and timing of that portion of precipitation which ultimately reaches the surface water system as runoff.

Hydraulics may be defined as the study of the physical behavior of water as it flows within pipes and natural and artificial channels; under and over bridges, culverts, and dams; and through lakes and impoundments. The application of hydraulics to the planning and design of stormwater drainage systems requires the collection and analysis of definitive information on the configuration of the natural and artificial stormwater drainage systems of the study area, including information on the shape and dimensions of the cross-sectional areas, on the longitudinal gradients, and on the roughness and attendant hydraulic performance of the collection, storage, and conveyance facilities involved.

Thus, stormwater management planning and design requires knowledge and understanding of the complex relationships existing among the many interrelated natural and man-made features that together comprise the hydrologic-hydraulic system of the study area, and of how these relationships may change over time. In addition, knowledge of the economic and environmental impacts of such systems, and of the public attitudes involved, is required.

BASIC CONCEPTS INVOLVED

The basic concept underlying urban stormwater management is undergoing reexamination. The old concept sought to remove excess surface water during and after a rainfall as quickly as possible through the provision of an efficient drainage system, a system usually consisting of enclosed conduits, although sometimes consisting of improved open channels. The problems created by application of this traditional approach to urban stormwater drainage were more or less acceptable when urban development was compact and confined to relatively small areas. These problems have become increasingly aggravating and unacceptable as the pattern of urban development has changed and urban land uses have diffused over ever-larger areas.

The new concept emphasizes storage of rainfall onsite, even at some localized inconvenience, thus reducing both the total and the peak rate of runoff, reducing the transport of sediment and other water pollutants to downstream surface waters, and protecting against increased downstream flooding. The new concept also looks to controlling the quality, as well as the quantity, of runoff, and seeks to manage stormwater as a potentially valuable resource rather than as a nuisance to be disposed of as quickly as possible.

Both the older concept and the newer concept were applied in the study in the development of alternative plans. Regardless of the concept, urban stormwater management systems are generally designed to fulfill three basic objectives: 1) to prevent significant damage to buildings, other structures, and other forms of real property from relatively infrequent major rainfall events; 2) to maintain reasonably convenient access to and egress from the various land uses of an urban area during relatively frequent minor rainfall events; and 3) to avoid undue hazards to public safety and health. Thus, the total stormwater management system of an urban area may be conceived of as consisting of a major element operating infrequently and a minor element operating frequently.

Both of these elements of the system can, under certain conditions, utilize stormwater retention or detention as a potential design solution. The benefits of stormwater storage are that it reduces the high kinetic energy of surface runoff, reduces peak discharges, provides multiple-use opportunities for recreational and aesthetic purposes, provides groundwater recharge, traps some pollutants, and reduces the adverse impacts of the remaining pollutants by controlled release.

For predominantly developed urban communities such as the Village of Hales Corners, the development of stormwater storage is constrained by the availability of open land on, or adjacent to, the drainage system. Some storage potential also exists within the developed areas such as on parking lots in commercial and industrial areas and on site in residential and recreational areas. Successful efforts have been made to integrate stormwater storage facilities into the existing urban environment; however, such efforts may be costly and difficult to implement because of the existing development pattern and public concerns. Nevertheless, the practice of detaining or retaining stormwater within the confines of an urban area to mitigate flooding, soil erosion, sedimentation, and pollutant contributions is increasingly being recognized as a sound and cost-effective stormwater management approach.

The recommended stormwater management plan for the Village of Hales Corners, as set forth herein, incorporates compatible multiple-use planning concepts and recognizes the constraints imposed by other community needs, such as park and open space, transportation, sanitary sewerage, and water supply. Drainage requirements under existing and plan year 2000 land use conditions are evaluated. Flood control and drainage problems are addressed as necessary. Finally, the plan encompasses not only the existing and planned future urban service area of the Village, but the entire upstream watersheds of the natural streams and watercourses flowing into and through the study area which must receive the discharge of the engineered urban drainage systems.

Review of Previous Studies

The first step in the preparation of the stormwater management plan was the review of the findings and recommendations of previous stormwater drainage studies made for the Village. These studies are documented in various letter reports and staff memoranda on file in the Village Hall and in the offices of the firm of W. G. Nienow Engineering Associates. The studies reviewed are listed below, and the salient findings and recommendations thereof summarized.

1. Drainage Study of a Part of the Northwest Quadrant of the Village of Hales Corners, Milwaukee County, Wisconsin, January 1962, prepared by H. C. Webster & Son.

   This study investigated alternative stormwater drainage systems to serve a drainage area tributary to a proposed culvert beneath STH 24 (W. Janesville Road) at S. 111th Street. The study, which considered stormwater drainage from an area of approximately 740 acres, compared the advantages, disadvantages, and costs of alternative open channel and closed conduit designs to convey stormwater runoff to the highway culvert. A map in the report showed proposed storm sewer horizontal alignments, inlet and manhole locations, and drainage areas. Design data reported for each stormwater catchment area were based upon a five-year recurrence interval storm event and included design peak flows, flow velocities, proposed sewer sizes and grades, sewer capacities, and estimated costs. In addition, channel cross-sections, grades, and the depth of flows were presented for an open channel alternative. A combination alternative consisting of both storm sewers and open channels was recommended to be included in the drainage system in this study area. As of 1984, none of the recommendations had been implemented.

2. Supplementary Report No. 1 for Part of the Northwest Quadrant of the Village of Hales Corners, June 1969, prepared by Nienow, Landry, Webster & Associates.

   This report supplemented and provided more detailed information on the drainage system evaluated in the January 1962 report described above. The supplementary report did not contain a plan map. However, design data were reported for culverts and for open channels. Data reported for each culvert included design peak flows for a five-year recurrence interval storm event, culvert sizes and grades, culvert capacities, and culvert types. Data reported for each open channel included design peak flows for a five-year recurrence interval storm event, channel cross-sections and length, grades, and depth of flow. The supplementary report recommended some changes to the 1962 plan. These changes were based on an analysis of horizontal alignment, grade requirements, easement widths, and costs. In addition, the supplementary report considered alternative plans and recommended a plan for providing stormwater drainage for an adjacent area located south of STH 24. As of 1984, none of these recommendations had been implemented.

3. System Plan Showing Proposed Storm Sewers in the Northeast One-Quarter of Section 31, Township 6 North, Range 21 East, in the Village of Hales Corners, January 1972, prepared by Nienow, Landry, Webster & Associates.

   This system plan showed existing and proposed storm sewers within a 12-acre area of the Village bordered by W. Denis Avenue, W. Parnell Avenue, S. 111th Street, and S. 110th Street. The plan map showed existing and proposed storm sewer horizontal alignments, inlet and outlet locations, invert elevations and grades, street grades, and drainage areas. Design data reported for each sewer segment included runoff amounts for a five-year recurrence interval storm event, design peak flows, flow velocities, and proposed sewer sizes and grades. As of 1984, few of the proposed storm sewers had been installed.

4. Storm Sewer System Plan of the Southwesterly Part of the Village of Hales Corners, Milwaukee County, Wisconsin, May 1975, prepared by W. G. Nienow Engineering Associates.

   This plan presented a recommended storm sewer system for U. S. Public Land Survey Township 6 North, Range 21 East, Section 31, which includes the area south of W. Liberty Avenue and west of 108th Street within the Village. The plan map showed existing and proposed storm sewer horizontal alignments, inlet and outlet locations, invert elevations and grades, street grades, and drainage areas. Reported design data for each stormwater catchment area were based upon a five-year recurrence interval storm event, and included design peak flows, flow velocities, and proposed sewer sizes and grades. As of 1984, few of the proposed storm sewers had been installed.

In addition to these studies prepared by the Village, a flood insurance study has been prepared for the Village by the Federal Emergency Management Agency as documented in Flood Insurance Study for the Village of Hales Corners, Milwaukee County, Wisconsin, June 1979. The study describes the existence and severity of flood hazards within the Village of Hales Corners. A hydrologic-hydraulic simulation model was used to determine the 10-, 100-, and 500-year recurrence interval flood discharges and associated stages under existing conditions. Flood insurance rate maps in the report show the flood insurance zones and the boundaries of the 100- and 500-year flood hazard areas. The results of the study enabled property owners within the Village to participate in the Federal Insurance Administration's flood insurance program.

SUMMARY

The Village of Hales Corners is located in southwestern Milwaukee County. The Village has a history of drainage problems which are related to high groundwater and flat topography. In 1980 there were approximately 205 acres of developable land within the Village and an additional 528 acres of developable land within the Cities of Greenfield and New Berlin that drain to watercourses that flow through the Village. Development of these lands is expected to aggravate existing drainage problems and pose new problems downstream. The need to resolve existing problems and to avoid the occurrence of new problems dictates the need to prepare a long-range stormwater management plan for the Village of Hales Corners and environs.

This report presents such a stormwater management plan. The plan seeks to promote the development of an effective stormwater system for the study area through the year 2000, a system that will minimize damages attendant to poor drainage while reducing downstream flooding.

More specifically, this report describes the existing stormwater drainage system and stormwater drainage problems of the Hales Corners area; describes proposed land use conditions and identifies related stormwater management requirements; provides a set of stormwater management objectives and supporting principles and standards to guide the development of an effective stormwater management system for the area; presents alternative stormwater management system plans and provides a comparative evaluation of the technical, economic, and environmental features of these plans; recommends a stormwater management plan for the Village and environs; and sets forth a plan implementation program.

The plan focuses on stormwater drainage as opposed to flood control problems, addressing the latter only as necessary to avoid the intensification of existing or the creation of new flood control problems along the natural streams and watercourses of the study area which must receive the discharge from the existing and proposed urban drainage facilities. The plan recognizes that good stormwater drainage is essential to the provision of an attractive and efficient, as well as safe and healthful, environment for urban life; and that inadequate stormwater drainage can be costly and disruptive, can create hazards to public health and safety, and can have adverse ecological and environmental impacts. Because of the technical complexity of the problem and the important social, economic, and environmental impacts involved, stormwater management planning must be based upon knowledge of the art of urban engineering and of the sciences of hydrology and hydraulics; an understanding of the social, economic, and environmental impacts involved; and information on the public attitudes toward stormwater drainage.

The recommended stormwater management plan presented herein also recognizes that the basic concept underlying urban stormwater management is undergoing reexamination. The old concept sought to eliminate excess surface water during and after a rainfall as quickly as possible through the provision of an efficient drainage system, a system consisting of enclosed conduits and improved open channels. The new concept emphasizes the storage of rainfall onsite, even at some localized inconvenience, thus reducing both the total volume and the peak rate of runoff and providing protection against increased downstream flooding. The new concept also looks to controlling the quality, as well as the quantity, of runoff, and seeks to manage stormwater as a potentially valuable resource rather than as a nuisance to be disposed of as quickly as possible.

Accordingly, the plan presented herein regards the stormwater runoff system of the area as consisting of a major element operating infrequently and a minor element operating frequently, with both of these elements incorporating, to the extent practicable, the storage of excess runoff. The recommended stormwater management plan set forth herein thus incorporates compatible multi-use planning concepts and recognizes the opportunities provided as well as the constraints imposed by other community needs, such as park and open space, transportation, and water supply. Drainage requirements are evaluated under existing and planned land use conditions, and flood control problems are addressed as necessary. Finally, the plan encompasses not only the existing and future urban service area of the Village but the entire upstream watersheds of the natural streams and watercourses flowing through the study area, which must constitute the outlets for the engineered urban drainage system of the area.

Chapter II

OVERVIEW OF THE STUDY AREA

INTRODUCTION

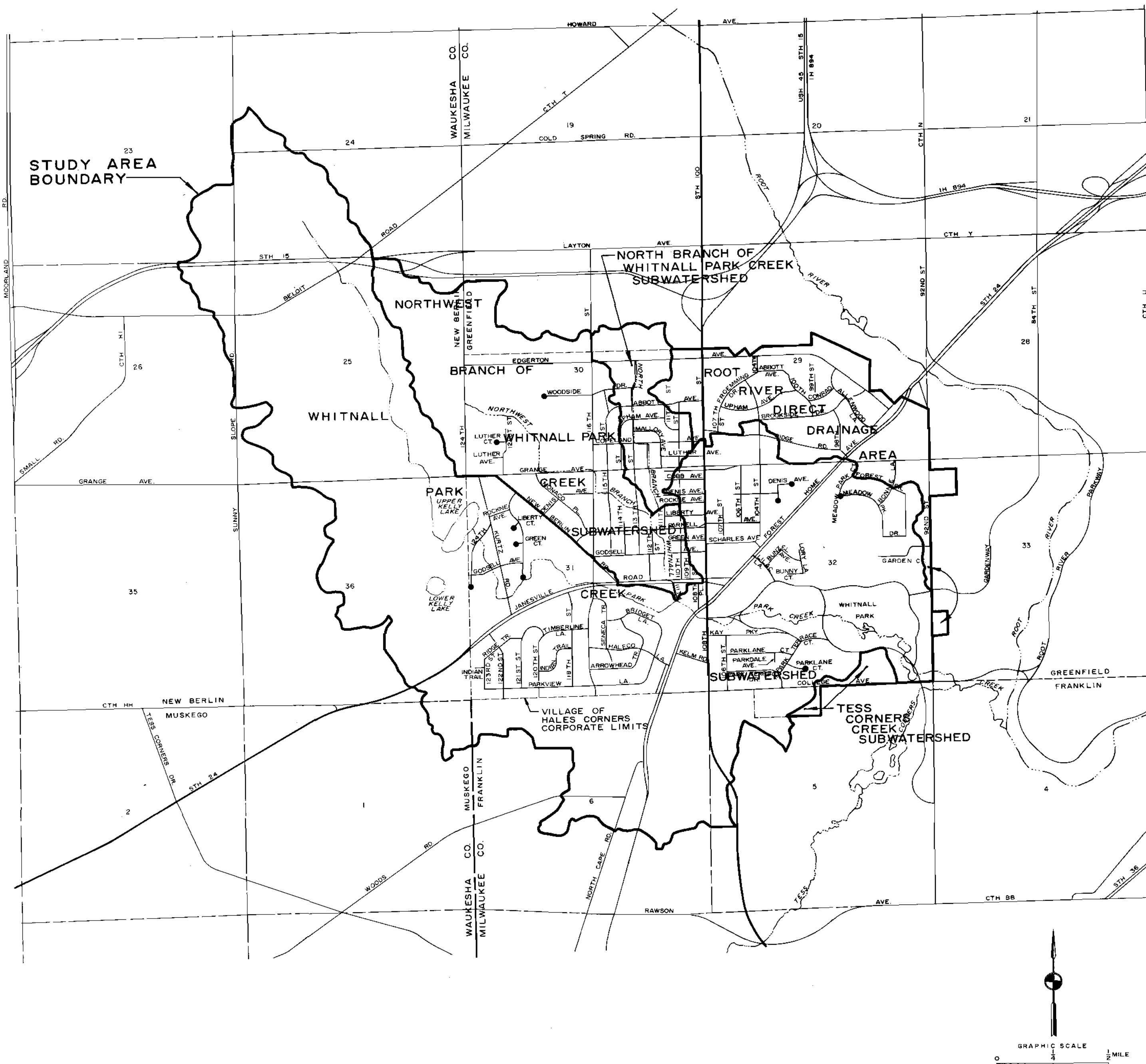
The primary focus of the stormwater management plan presented in this report is the 3.24-square-mile area contained within the corporate limits of the Village of Hales Corners. Stormwater runoff from the Village is drained to five separate surface water drainage systems--those systems being the intermittent and perennial streams of 1) the Whitnall Park Creek subwatershed; 2) the Northwest Branch of Whitnall Park Creek subwatershed; 3) the North Branch of Whitnall Park Creek subwatershed; 4) the Tess Corners Creek subwatershed; and 5) the Root River direct drainage area. In addition to serving as outlets for stormwater drainage from within the corporate limits of the Village, these surface water systems receive drainage from areas within the Cities of New Berlin, Muskego, Franklin, and Greenfield located upstream of the Village. These upstream tributary drainage areas must also be considered in the proper design of a stormwater management system for the Village. Thus, the total study area herein considered for stormwater management planning purposes, as shown on Map 1, includes the drainage subbasins of the natural subwatersheds which are upstream of and tributary to the natural surface water drainage system which lies within the Village of Hales Corners. These upstream areas total an additional 2.58 square miles in area. The study area boundary as well as the 1984 corporate limits of the Village of Hales Corners, the natural stream and watercourse system, and the subwatershed boundaries are shown on Map 1, and the drainage areas involved are quantified in Table 1.

STORMWATER MANAGEMENT STUDY AREA

The total areal extent of the study area is approximately 3,726 acres, of which 2,073 acres, or about 56 percent, lie within the corporate limits of the Village of Hales Corners, and 1,653 acres, or about 44 percent, lie outside the village corporate limits. As set forth in Table 1, about 2,609 acres, or 70 percent of the total study area, drain to Whitnall Park Creek; about 596 acres, or 16 percent, drain to the Northwest Branch of Whitnall Park Creek; about 136 acres, or 4 percent, drain to the North Branch of Whitnall Park Creek; about 26 acres, or less than 1 percent, drain to Tess Corners Creek; and about 359 acres, or 10 percent, drain directly to the Root River. About 1,172 acres, or 57 percent of the Village of Hales Corners, drain to Whitnall Park Creek; 404 acres, or 20 percent, drain to the Northwest Branch of Whitnall Park Creek; 112 acres, or 5 percent, drain to the North Branch of Whitnall Park Creek; 26 acres, or 1 percent, drain to Tess Corners Creek; and 359 acres, or 17 percent, drain directly to the Root River. Of the study area outside the village corporate limits, 1,437 acres, or 87 percent, drain to Whitnall Park Creek; 192 acres, or 12 percent, drain to the Northwest Branch of Whitnall Park Creek; and 24 acres, or 1 percent, drain to the North Branch of Whitnall Park Creek.

Map 1

VILLAGE OF HALES CORNERS STORMWATER MANAGEMENT STUDY AREA



Source: SEWRPC.

Table 1

AREA AND PROPORTION OF THE WHITNALL PARK CREEK, NORTHWEST BRANCH OF WHITNALL PARK CREEK, NORTH BRANCH OF WHITNALL PARK CREEK, TESS CORNERS CREEK, AND ROOT RIVER SUBWATERSHEDS WITHIN THE VILLAGE OF HALES CORNERS CORPORATE LIMITS AND THE STUDY AREA: 1984

Subwatershed	Village of Hales Corners Corporate Limits		Study Area Outside the Village of Hales Corners		Total Study Area	
	Area (acres)	Percent of Total	Area (acres)	Percent of Total	Area (acres)	Percent of Total
Whitnall Park Creek	1,172	56.5	1,437	86.9	2,609	70.0
Northwest Branch of Whitnall Park Creek	404	19.5	192	11.6	596	16.0
North Branch of Whitnall Park Creek	112	5.4	24	1.5	136	3.7
Tess Corners Creek	26	1.3	--	--	26	0.7
Root River	359	17.3	--	--	359	9.6
Total	2,073	100.0	1,653	100.0	3,726	100.0

Source: SEWRPC.

Table 2

EXISTING 1980 AND ANTICIPATED YEAR 2000 POPULATION WITHIN THE VILLAGE OF HALES CORNERS STORMWATER MANAGEMENT STUDY AREA

Area	1980		Anticipated Year 2000	
	Population	Percent of Total Study Area	Population	Percent of Total Study Area
Village of Hales Corners.....	7,110	65.8	8,500	57.8
Study Area Outside the Village of Hales Corners....	3,695	34.2	6,200	42.2
Total Study Area	10,805	100.0	14,700	100.0

Source: SEWRPC.

As set forth in Table 2, the total study area had a resident population of 10,805 persons in 1980. The resident population of the study area may be expected to increase to about 14,700 persons by the year 2000, an increase of about 3,900 persons, or almost 40 percent, over the 20-year period. In 1980 the Village of Hales Corners had a resident population of 7,110 persons. The resident population of the Village may be expected to increase to approximately 8,500 persons by the year 2000, an increase of about 1,400 persons, or almost 20 percent, over the 20-year period. That part of the study area outside the corporate limits of the Village had a resident population of 3,695 persons in 1980. The resident population of this area may be expected to increase to about 6,200 persons by the year 2000, an increase of about 2,500 persons, or about 70 percent, over the 20-year period.

Table 3

EXISTING 1980 AND PLAN YEAR 2000 URBAN AND RURAL LAND USE WITHIN THE VILLAGE OF HALES CORNERS STORMWATER MANAGEMENT STUDY AREA

Area	1980				Planned 2000				Total Area	
	Urban Land		Rural and Open Land		Urban Land		Rural and Open Land			
	Acres	Percent of Total	Acres	Percent of Total	Acres	Percent of Total	Acres	Percent of Total	Acres	Percent of Total
Village of Hales Corners	1,670	80.6	403	19.4	1,831	88.3	242	11.7	2,073	55.6
Study Area Outside the Village of Hales Corners	981	59.3	672	40.7	1,509	91.3	144	8.7	1,653	44.4
Total Study Area	2,651	71.1	1,075	28.9	3,340	89.6	386	10.4	3,726	100.0

Source: SEWRPC.

As set forth in Table 3, the amount of land devoted to urban use in 1980 within the total study area was about 2,651 acres, or about 71 percent of the total study area. Within the Village of Hales Corners, urban uses totaled about 1,670 acres, or 81 percent of the village area, in 1980. That portion of the study area lying outside the Village of Hales Corners had about 981 acres in urban use in 1980, or 59 percent of this area. By the year 2000, additional development may be expected to result in almost all of the study area being in urban use. As further indicated in Table 3, only 386 acres, or about 10 percent of the total study, would remain in open lands. Such lands are located primarily within public parks and private outdoor recreation and open space lands.

Table 4 sets forth the area and proportion of the study area located within various civil divisions as of 1984. Over half--56 percent--of the study area lies within the Village of Hales Corners. About 32 percent of the study area lies within the City of New Berlin and about 9 percent of the study area lies within the City of Franklin. The Cities of Greenfield and Muskego together contain less than 3 percent of the study area.

SURFACE WATER DRAINAGE IN THE STUDY AREA

Selected characteristics of the surface water drainage system of the study area, including certain related features, are shown on Map 2. More specifically, shown on this map are watershed boundaries, perennial and intermittent streams and watercourses, minor lakes and ponds, the 100-year recurrence interval flood hazard area under existing land use and channel conditions, and the area served by storm sewer systems.

Engineered stormwater drainage facilities within the study area as of 1984--defined as constructed channels, sewers, and appurtenances, as opposed to natural watercourses--had a combined service area of about 2,651 acres, or 71 percent of the total study area. About 274 acres, or 10 percent of the total area served by engineered stormwater drainage facilities, were tributary to drainage systems relying primarily on storm sewers for conveyance, while the remaining 2,377 acres, or 90 percent, were tributary to drainage systems relying primarily on open drainage channels and associated culverts.

Table 4

AREA AND PROPORTION OF THE WHITNALL PARK CREEK, NORTHWEST BRANCH OF WHITNALL PARK CREEK, NORTH BRANCH OF WHITNALL PARK CREEK, TESS CORNERS CREEK, AND ROOT RIVER SUBWATERSHEDS WITHIN THE VILLAGE OF HALES CORNERS CORPORATE LIMITS AND THE STUDY AREA: 1984

Subwatershed	1984 Civil Divisions	Area (acres)	Percent of Subwatershed
Whitnall Park Creek	City of Franklin City of Muskego City of New Berlin Village of Hales Corners	352 26 1,059 1,172	13.5 1.0 40.6 44.9
	Subtotal	2,609	100.0
Northwest Branch of Whitnall Park Creek	City of Greenfield City of New Berlin Village of Hales Corners	59 133 404	9.9 22.3 67.4
	Subtotal	596	100.0
North Branch of Whitnall Park Creek	City of Greenfield Village of Hales Corners	24 112	17.6 82.4
	Subtotal	136	100.0
Tess Corners Creek	Village of Hales Corners	26	100.0
Root River	Village of Hales Corners	359	100.0
Total Study Area	City of Franklin City of Greenfield City of Muskego City of New Berlin City of Hales Corners	352 83 26 1,192 2,073	9.5 2.2 0.7 32.0 55.6
	Total	3,726	100.0

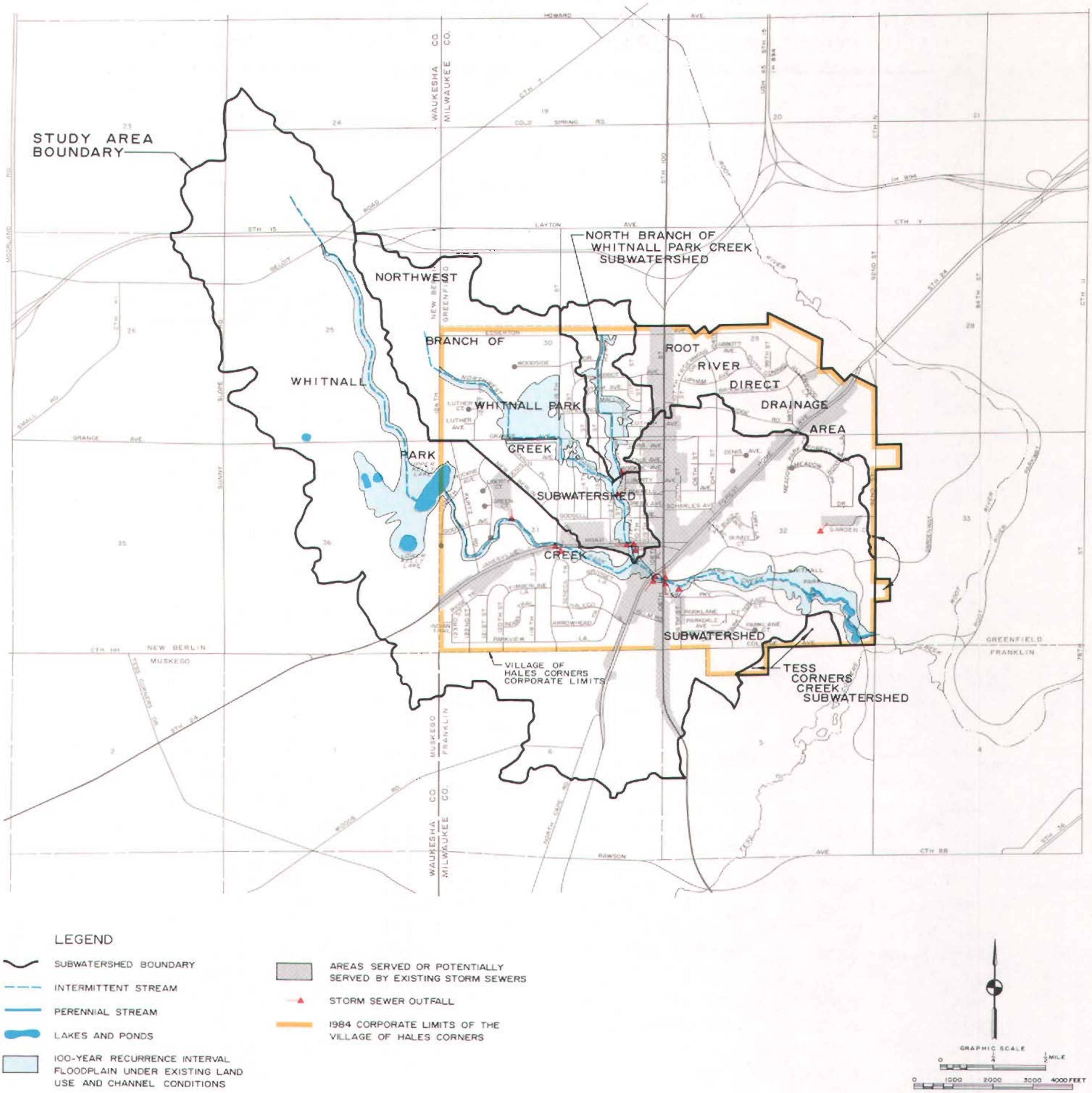
Source: SEWRPC

Within the Village of Hales Corners, about 252 acres, or 15 percent of the area served by engineered stormwater drainage systems, were tributary to the storm sewer system, while the remaining 1,418 acres of developed urban land, or 85 percent of the area served by engineered systems, were drained by the open channel drainage system. The existing storm sewer system for the Village of Hales Corners consists primarily of subsurface conduits discharging to drainage ditches and a full pipe system underlying the state and county trunk highways. This system contains no public stormwater storage or pumping facilities. The existing system actually consists of 16 subsystems, as shown on Map 2, discharging to 16 individual stormwater outfalls ranging in size from 12 inches to 66 inches in diameter. As shown on Map 2, 14 of the outfalls discharge to the North Branch, Northwest Branch, or Main Stem of Whitnall Park Creek, while one of the two remaining outfalls discharges to an intermittent tributary of the Root River, and one outfall discharges directly to the Root River.

Within the portion of the City of Franklin lying within the study area, about 22 acres, or 6 percent of the area served by engineered drainage systems, were tributary to the storm sewer system, while the remaining 330 acres were drained by the open channel drainage system. The existing engineered storm

Map 2

SELECTED CHARACTERISTICS OF THE VILLAGE OF HALES CORNERS STORMWATER MANAGEMENT STUDY AREA SURFACE WATER DRAINAGE SYSTEM: 1984



Source: Village of Hales Corners Public Works Department, W. G. Engineering Associates, and SEWRPC.

Table 5

LENGTH OF PERENNIAL AND INTERMITTENT STREAMS: 1984

Area	Perennial		Intermittent	
	Length of Stream (miles)	Percent of Total	Length of Stream (miles)	Percent of Total
Village of Hales Corners	0.52	100.0	4.47	69.4
Study Area Outside the Village of Hales Corners	--	--	1.97	30.6
Total Study Area	0.52	100.0	6.44	100.0

Source: SEWRPC.

sewer system consists of a full pipe system underlying the state and county trunk highways. This system contains no public stormwater storage or pumping facilities. The existing system consists of two subsystems, as shown on Map 2. One subsystem provides drainage for CTH OO, while the other subsystem provides drainage for STH 100 and USH 45. Both subsystems within the City of Franklin discharge into the Village of Hales Corners storm sewer system, with the stormwater being eventually discharged through that system into Whitnall Park Creek.

The urban portions of the Cities of Greenfield, Muskego, and New Berlin located within the study area are served by engineered open drainage channels. These urban areas have a total area of about 629 acres. These drainage systems contain no storm sewers or public stormwater storage or pumping facilities.

Perennial streams or watercourses which maintain a continuous flow throughout the year serve as the major drainage outlets for the storm sewers, drainage ditches, open channels, and intermittent streams of the study area. Intermittent streams are those watercourses which do not sustain continuous flow during dry periods. As set forth in Table 5, a 0.52-mile reach of Whitnall Park Creek is classified as perennial, all of which lies within the corporate limits of the Village. All of the remaining natural watercourses within the study area are intermittent streams. As shown in Table 5, there are 6.44 miles of intermittent streams within the study area, of which 4.47 miles, or 69 percent, lie within the corporate limits of the Village. This network of streams serves a vital function by providing natural drainage for those areas not drained by engineered stormwater drainage facilities, and by receiving the discharge of the engineered stormwater drainage facilities. Both perennial and intermittent streams constitute important components of the existing and planned stormwater management systems of the study area. The importance of these streams to future stormwater management is primarily due to two factors: 1) the streams accommodate surface runoff and provide outlets for engineered stormwater drainage systems, and 2) the streams carry flows from upstream areas into and through the urban service area, transmitting flows from both the upstream areas and the urban service area to downstream areas.

The floodlands of a stream are the normally wide, gently sloping areas contiguous with, and usually lying on both sides of, the stream channel. When stream discharges increase beyond the conveyance capacity of the existing stream channel, the stream rises and spreads laterally over the floodlands. Map 2 shows those areas within the study area subject to inundation by the 100-year recurrence interval flood event under existing land use and channel conditions. Floodlands within the corporate limits of the Village occupy an area of approximately 182 acres, or 9 percent of the total area of the Village. The floodlands shown occupy an area of approximately 266 acres, or 7 percent of the study area.

SUMMARY

The primary focus of the stormwater management plan presented in this report is the 3.24-square-mile area contained within the corporate limits of the Village of Hales Corners. The plan, however, considers, as may be necessary, drainage areas of the natural watersheds which lie upstream of, and are tributary to, the Village. About 70 percent of the study area is drained by Whitnall Park Creek; about 16 percent is drained by the Northwest Branch of Whitnall Park Creek; about 10 percent is drained directly by the Root River; about 4 percent is drained by the North Branch of Whitnall Park Creek; and less than 1 percent is drained by Tess Corners Creek.

As of 1980, the total resident population of the study area was 10,805 persons, of which 7,110 resided within the Village of Hales Corners. By the year 2000, the resident populations of the total study area and the Village are expected to increase to about 14,700 persons and 8,500 persons, or by almost 40 percent and 20 percent, respectively.

About 2,651 acres, or 71 percent of the study area, were devoted to urban land use in 1980. Within the Village of Hales Corners, about 1,670 acres, or 81 percent of the village area, were in urban use. By the year 2000, almost all of the study area is expected to be in urban use, with only 386 acres, or about 10 percent of the study area, remaining in open uses in the plan design year. Approximately 56 percent of the study area lies within the Village of Hales Corners; 32 percent lies within the City of New Berlin; 9 percent lies within the City of Franklin; and less than 3 percent lies within the Cities of Greenfield and Muskego.

Engineered stormwater drainage facilities within the study area in 1984 consisted of both storm sewer systems and open drainage channel systems. Of the 2,651 acres of land served by engineered drainage facilities in 1984 within the study area, about 274 acres, or 10 percent, were tributary to storm sewer systems, while the remaining 2,377 acres, or 90 percent, were tributary to open drainage channels. Storm sewer systems exist within the Village of Hales Corners and the City of Franklin.

The only perennial stream within the study area is a 0.52-mile reach of Whitnall Park Creek which lies entirely within the Village of Hales Corners. The remaining 6.44 miles of streams within the study area are classified as intermittent. The 100-year recurrence interval floodlands under existing land use and channel conditions occupy a total area of about 266 acres, of which about 182 acres, or 68 percent, lie within the corporate limits of the Village of Hales Corners.

Chapter III

INVENTORY AND ANALYSIS

INTRODUCTION

Information on certain pertinent natural and man-made features of the study area is essential to sound stormwater management planning. Accordingly, the collation and collection of definitive information on key hydrologic and hydraulic characteristics of the stormwater management planning area, on the existing stormwater drainage system of that area, and on the erosion and sedimentation characteristics of that area constitutes an important step in the stormwater management planning process. The resulting information is essential to the planning process, because alternative stormwater management plans cannot be formulated and evaluated without an in-depth knowledge of the pertinent conditions in the planning area. This is particularly true for stormwater management planning, which must address the complex interaction of natural meteorologic events, key hydrologic and hydraulic characteristics of the planning area, and certain man-made physical systems.

Accordingly, this chapter presents pertinent data on the location, configuration, and capacity of the existing stormwater drainage system of the Hales Corners area; on the magnitude of stormwater flows to be accommodated by that system; and on the hydrologic phenomena governing the magnitude and frequency of those stormwater flows. Also presented are data on existing stormwater drainage and flood control problems. The data pertinent to stormwater management planning are presented in this chapter under the following headings: land use, land use regulations, climate, soils, stormwater drainage systems, stormwater management and flood control problems, and erosion and sedimentation problems. Because water quality impacts are becoming increasingly of concern in stormwater management, this chapter also presents data on surface water quality conditions in the Hales Corners area and identifies those sources of pollution related to stormwater management.

LAND USE

The type, density, and spatial distribution of land uses are important determinants of the quantity and quality of stormwater runoff. The amount of impervious area, the type of stormwater drainage system, the level and characteristics of human activity, and the type and amount of water pollutant deposition all vary with land use. Pertinent data on the existing land use pattern in the Hales Corners study area are presented in Table 6, and that pattern is shown on Map 3.

The study area encompasses a total area of about 3,726 acres, or 5.82 square miles. As indicated in Table 6, in 1980 urban land uses accounted for about 2,651 acres, or about 71 percent of the total study area. Of these urban land uses, residential uses occupied about 1,701 acres, or 64 percent. The other urban land uses--governmental and institutional, commercial, industrial, transportation and utilities, and recreational--together covered about

Table 6

EXISTING LAND USE CONDITIONS IN THE VILLAGE OF HALES CORNERS STUDY AREA: 1980

Land Use Category	Village of Hales Corners		Study Area Outside the Village of Hales Corners		Total Study Area	
	Acres	Percent of Total	Acres	Percent of Total	Acres	Percent of Total
Urban						
Residential	1,024	50	616	37	1,640	44
Vacant Residential	21	1	40	2	61	1
Residential Subtotal	1,045	51	656	39	1,701	45
Governmental and Institutional	50	3	32	2	82	2
Commercial	60	3	21	1	81	2
Industrial	4	--[a]	15	1	19	1
Transportation and Utilities	381	18	240	15	621	17
Recreational	130	6	17	1	147	4
Nonresidential Subtotal	625	30	377	23	950	30
Urban Subtotal	1,670	81	981	59	2,651	71
Rural						
Woodlands	111	5	65	4	176	4
Wetlands	81	4	61	4	142	4
Agriculture and Other	205	10	528	32	733	20
Surface Water	6	--[a]	18	1	24	1
Rural Subtotal	403	19	672	41	1,075	29
Total	2,073	100	1,653	100	3,726	100

[a] Less than 0.5 percent.

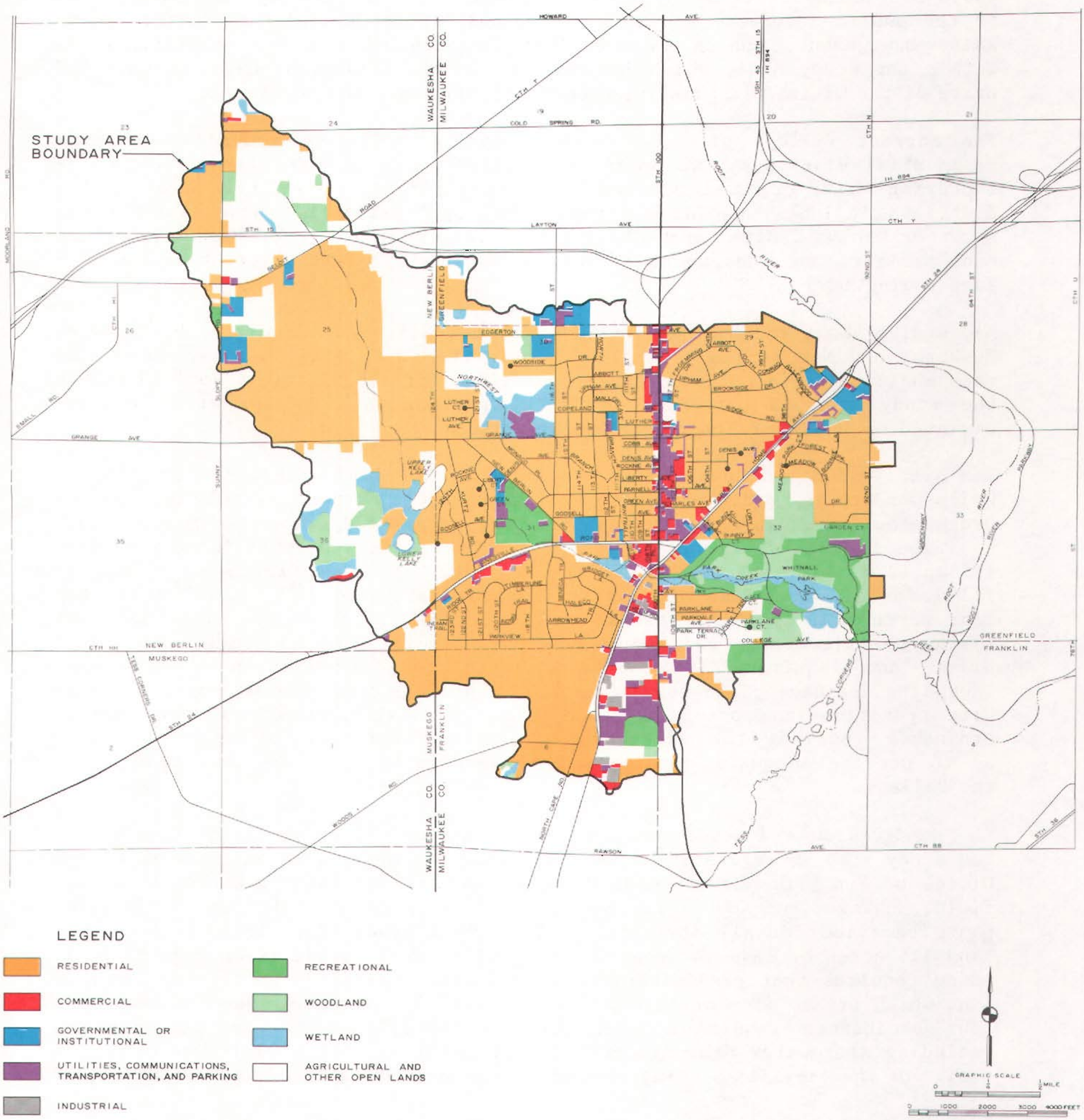
Source: SEWRPC.

950 acres, or the remaining 36 percent of the urban land uses. In 1980, rural land uses accounted for 1,075 acres, or 29 percent of the total study area. Agricultural and other open lands occupied about 733 acres, or about 68 percent of the rural land. Other rural land uses including wetlands, woodlands, and water, comprised the remaining 342 acres, or about 32 percent of the rural land uses.

The incorporated Village of Hales Corners encompasses approximately 2,073 acres, or 56 percent of the study area. In 1980, urban land uses within the Village accounted for 1,670 acres, or about 81 percent of the village area. The dominant urban land use was residential, covering 1,045 acres, or about 63 percent of the developed urban area. Rural land uses accounted for 403 acres, or about 19 percent of the total area of the Village, with the dominant use being agricultural and other open lands--which comprised 205 acres, or about 51 percent of the rural land uses. By the year 2000, additional urban development is expected to result in essentially all of the Village and the study area being in urban use.

Map 3

EXISTING LAND USE IN THE VILLAGE OF HALES CORNERS STUDY AREA: 1980



Source: SEWRPC.

LAND USE REGULATIONS

Pertinent land use regulations in the study area include zoning and land division ordinances. Comprehensive zoning represents one of the most important tools available to local units of government for controlling the use of land in the public interest, and such zoning has important implications for stormwater management. Zoning is exercised by each of the five municipalities within the study area, which includes all of the Village of Hales Corners and parts of the Cities of Franklin, Greenfield, Muskego, and New Berlin.

The current Village of Hales Corners zoning ordinance provides for four residential districts, two business districts, one commercial and light manufacturing district, and one wetland and floodplain district. Each of the districts includes adjoining streets. The application of these districts is shown on Map 4. Table 7 presents a brief summary of the regulations governing each district and the amount of acreage assigned to each district on the village zoning map.

Map 4 also shows the zoning districts for the Cities of Franklin, Greenfield, Muskego, and New Berlin as applied within the study area. A brief summary of each municipality's corresponding regulations for the districts applied within the study area is presented in Table 7, along with the amount of acreage assigned to each district within the study area.

The subdivision and development for urban use of land within the Village of Hales Corners is regulated by the Village of Hales Corners subdivision control ordinance. The ordinance requires that preliminary and final subdivision plats be filed for all divisions of land which create five or more parcels of land 1.5 acres or less in area. It further requires that a certified survey map be filed for all divisions of land which create at least two but not more than four parcels of land that are 1.5 acres or less in area. The ordinance sets forth specific design and improvement requirements for preliminary and final plats, and requires the subdivider to install subdivision improvements, including drainage channels, culverts, and other surface drainage facilities to city specifications prior to final plat approval. The subdivision control ordinance requires the stormwater drainage facilities to be designed so as to provide adequate surface water drainage subject to the approval of the Village.

The subdivision and development for urban use of the remainder of land within the study area is regulated by the land subdivision control ordinances of the Cities of Franklin, Greenfield, Muskego, and New Berlin. The Cities of Greenfield, Muskego, and New Berlin require that preliminary and final subdivision plats be filed for all divisions of land which create five or more parcels of land 1.5 acres or less in area. The City of Franklin subdivision control ordinance requires that preliminary and final plats be filed for all divisions of land which create five or more parcels of land 3.0 acres or less in area. All four ordinances require a subdivider to install subdivision improvements, including stormwater drainage facilities, prior to final plat approval, the design of the facilities being subject to the approval of the municipality.

The zoning and subdivision control ordinances exercised by each of the five municipalities within the study area serve to regulate the type, location, and intensity of the various land uses, and the improvements provided for new urban development. These ordinances regulate aspects of development which influence both the amount and rate of stormwater runoff, as well as the quality of that runoff. For example, the size of lots and the placement and size of structures on those lots, as regulated by the zoning ordinances, affect the proportion of the land surface covered by impervious surfaces. Generally, as imperviousness increases, the rate and amount of stormwater runoff increases and the water quality of the runoff decreases. The type and design of the stormwater drainage system, as regulated by the subdivision control ordinances, also affect the quantity and quality of stormwater runoff. For example, storm-sewered urban areas usually generate higher runoff rates and amounts, and a lower quality of runoff, than do areas drained by vegetated open channels.

CLIMATE AND HYDROLOGY

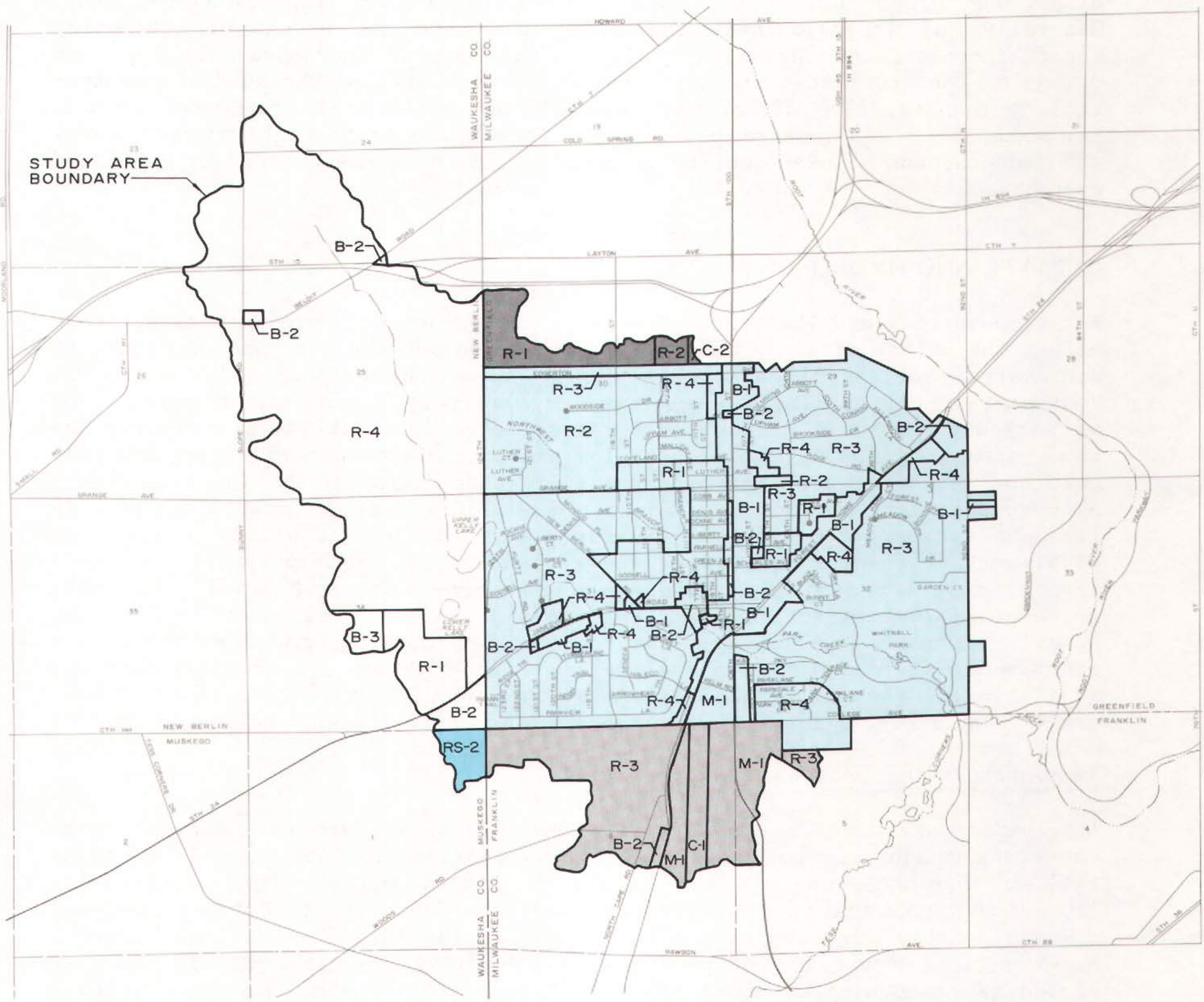
Air temperatures and the type, intensity, and duration of precipitation events affect the extent of areas subject to inundation and the type and magnitude of stormwater problems throughout the study area. The study area has a typical continental-type climate characterized primarily by a continuous progression of markedly different seasons and a wide 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 meteorologic events influence the rate and amount of stormwater runoff, the severity of storm drainage problems, and the required capacities of stormwater conveyance and storage facilities. Definitive, long-term meteorologic data are available for the Milwaukee National Weather Service Station, located at General Mitchell Field, in reasonable proximity to the Village of Hales Corners.

Temperature and Seasonal Considerations

Air temperatures, which exhibit a wide monthly range, are relevant to stormwater management planning and determine whether precipitation occurs as rainfall or snowfall; whether the ground is frozen and therefore essentially impervious; and the rate of snowmelt and attendant runoff. Table 8 presents average monthly air temperature variations at the Milwaukee National Weather Service Station for the 30-year period from 1951 through 1980. The 30-year period of meteorologic record of 1951 through 1980 corresponds to the World Meteorological Organization's normal climatic period. Summer temperatures, as measured by the monthly means for June, July, and August, average from 65°F to 70°F. Winter temperatures, as measured by the monthly means for December, January, and February, average from 19°F to 25°F. For the period 1871 through 1970 at Milwaukee, the maximum recorded temperature was 105°F in July 1934, and the lowest recorded temperature was -25°F in January 1875. The growing season, which is defined as the number of days between the last 32°F temperature reading in spring and the first in fall, averages about 180 days for the study area. The last frost in spring normally occurs near the end of April, whereas the first freeze in fall usually occurs during the latter half of October. Streams and lakes begin to freeze over in late November, and ice breakup usually occurs in late March or early April. Ice jams at bridges in spring can be a major cause of localized flooding. Such occurrences can be severe when combined with spring rainfall periods.

Map 4

EXISTING ZONING IN THE VILLAGE OF HALES CORNERS STUDY AREA: 1984



LEGEND

VILLAGE OF HALES CORNERS

R-1 RESIDENCE
R-2 RESIDENCE
R-3 RESIDENCE
R-4 RESIDENCE
B-1 BUSINESS
B-2 BUSINESS
M-1 COMMERCIAL AND LIGHT MANUFACTURING

CITY OF NEW BERLIN

R-1 RESIDENTIAL
R-2 RESIDENTIAL
B-2 LOCAL BUSINESS
B-3 GENERAL COMMERCIAL
FP-1 FLOODPLAIN

CITY OF FRANKLIN

R-3 SINGLE-FAMILY RESIDENCE
B-2 COMMERCIAL
M-1 LIMITED INDUSTRIAL
C-1 CONSERVANCY

CITY OF GREENFIELD

R-1 RESIDENCE
R-2 RESIDENCE
C-2 COMMERCIAL

CITY OF MUSKEGO

RS-2 SUBURBAN RESIDENCE

GRAPHIC SCALE
0 ½ MILE
0 1000 2000 3000 4000 FEET

Source: SEWRPC.

Table 7

SUMMARY OF EXISTING ZONING DISTRICTS IN THE VILLAGE OF HALES CORNERS, AND OF THE EXISTING ZONING DISTRICTS IN THE CITIES OF FRANKLIN, GREENFIELD, MUSKEGO, AND NEW BERLIN WITHIN THE STUDY AREA

Zoning District	Permitted Uses	Conditional Uses	Minimum Lot Area	Minimum Lot Width (feet)	Acres[a]	Percent of Civil Division	Percent of Study Area
Village of Hales Corners Zoning Ordinance							
R-1 Residence District	Churches, schools, universities, two-family detached dwellings, single-family detached and semi-detached dwellings, libraries, museums, community buildings, and public and private recreational uses and facilities	None	10,000 square feet	75	161	7.8	4.3
R-2 Residence District	Same as permitted in the R-1 Residence District	None	15,000 square feet	90	259	12.5	6.9
R-3 Residence District	Same as permitted in the R-2 Residence District excluding two-family dwellings	None	20,000 square feet	100	1,325	64.0	35.6
R-4 Residence District	Same as permitted in the R-3 Residence District, and two-family dwellings	Private clubs, fraternities, lodges, convalescent homes, rest homes, multiple-family dwellings, and planned residential development projects	10,000 square feet	80	86	4.1	2.3
B-1 Business District	Indoor amusement places, automobile agencies, motels, banks, vocational and business schools, substations, mechanical garages, hotels, offices, restaurants, theaters, utility offices, lodge halls, police or fire stations, post offices, and similar uses	Private clubs, fraternities and lodges, drive-in uses, parking and storage lots, mechanical garages, tire rebuilding or recapping plants, and undertaking establishments with attached living quarters	None	None	174	8.4	4.7
B-2 Business District	Same as permitted in the R-1 Residence District, telephone exchanges, parking lots, personal service shops, clinics, studios, tailor shops, clothes cleaning, beauty shops, professional offices, and financial institutions	Financial institutions with drive-in facilities	10,000 square feet for residential use. Buildings shall not exceed 40% of lot area	None	34	1.6	0.9
M-1 Commercial and Light Manufacturing District	Same as permitted in the B-2 Business District, exluding single-family, multi-family, and mixed residences, building supply yards, bakeries, laundries, cleaning and dyeing plants, storage warehouses, and similar uses	None	None Building shall not exceed 50% of lot area	None	34	1.6	0.9
Total	--	--	--	--	2,073	100.0	55.6
City of New Berlin Zoning Within the Study Area							
R-1 Residential District	One-family dwellings, public parks and recreation areas, keeping of poultry and domestic livestock, horticulture, accessory buildings, and home occupations	Airports, landing fields, dumps, disposal areas, transportation terminal facilities, cemeteries, churches, extraction of gravel, hospitals, public or private schools, outdoor recreation facilities, public buildings and uses, and public utility facilities	3 acres	200	45	3.8	1.2
R-4 Residential District	Same as permitted in the R-1 Residential District	Same as permitted in the R-1 Residential	20,000 square feet	100	1,051	88.2	28.2
B-2 Local Business District	Single-family residences in conjuntion with permitted business uses, boarding houses, delicatessens, florist shops, funeral homes, gift shops, interior decorators, professional offices, restaurants, tourist homes, appliance stores, barber and beauty shops, banks, clothing and drug stores, furniture stores, grocery and hardware stores, music and radio stores, photographers, shoe stores, theaters, automobile service stations, and similar uses	Same as permitted in the R-4 Residential District	10,000 square feet	None	23	1.9	0.6
B-3 General Commercial District	Any use permitted in the Local Business District, except that new residential uses shall not be permitted, wholesalers, distributors, garages, automobile showrooms, used car lots, storage yards, laundries, body shops, publishing houses, and similar uses	Same as permitted in the B-2 Local Business Districts	10,000 square feet	85	12	1.0	0.3
FP-1 Floodplain District	Uses not involving structures which are permitted in an underlying use district	Residential, commercial, industrial, and other nonresidential structures when fill requirements are met	None	None	61	5.1	1.7
Total	--	--	--	--	1,192	100.0	32.0

Table 7 (continued)

Zoning District	Permitted Uses	Conditional Uses	Minimum Lot Area	Minimum Lot Width (feet)	Acres[a]	Percent of Civil Division	Percent of Study Area
		City of Franklin Zoning Within the Study Area					
R-3 Single-Family Residence District	One-family detached dwellings, parks, forest preserves, recreational areas, home occupations, and accessory uses	Churches, public and private schools, universities, private parks, forest preserves, recreational areas, hospitals, rest homes, public service uses, airports, radio and television stations and towers, cemeteries and similar uses	20,000 square feet	110	195	55.4	5.3
B-2 Commercial District	Antique shops, banks, barber and beauty shops, clothing stores, clubs and lodges, department stores, food stores, garages, gift shops, hardware shops, laboratories, professional offices, restaurants, schools, theaters, and similar uses	Churches, hospitals, hotels, motels, parking lots, parks and recreational buildings, public utility and governmental service uses, radio and television towers, funeral homes, and similar uses	None	None	3	0.8	0.1
M-1 Limited Industrial District	Retail and service uses, any production, processing, cleaning, servicing, testing, repair, or storage of materials, goods, or products, wholesaling and warehousing, public and community service uses, and accessory uses	Airports, planned developments, motor freight terminals, sawmill operations, stadiums, auditoriums, and arenas	None	None	139	39.5	3.7
C-1 Conservancy District	Fishing, preservation of scenic, historic, and scientific areas, public fish hatcheries, soil and water conservation, sustained yield forestry, stream bank and and lakeshore protection, water retention, and wildlife preserves	Drainage, water measurement and water control, grazing, accessory structures, orchards, truck farming, utilities, and wildcrop harvesting	None	None	15	4.3	0.4
Total	--	--	--	--	352	100.0	9.5
		City of Greenfield Zoning Within the Study Area					
R-1 Residence District	Single-family detached dwellings and churches	None	20,000 square feet	120	65	78.3	1.7
R-2 Residence District	Single-family detached dwellings and churches	None	15,000 square feet	100	14	16.9	0.4
C-2 Commercial District	Barber shops and beauty parlors, clinics, drug stores, food stores, professional offices, restaurants, arenas, banks, clothing stores, department stores, radio and television stations, schools, indoor theaters, laboratories, and similar uses	None	5,000 square feet	50	4	4.8	0.1
Total	--	--	--	--	83	100.0	2.2
		City of Muskego Zoning Within the Study Area					
RS-2 Suburban Residence	Single-family detached dwellings; public parks; recreation areas; dairy, cattle and tree farming; horticulture; and accessories for public utilities	Outdoor recreational facilities, public and private schools, churches, public administrative offices, private lodges and clubs, rest homes, public utility offices, two-family residences, and commercial development of historic restorations	20,000 square feet	110	26	100.0	0.7
Total	--	--	--	--	26	100.0	0.7

[a]Rounded to the nearest acre.

Source: SEWRPC.

Table 8

AVERAGE MONTHLY AIR TEMPERATURE AT MILWAUKEE: 1951 THROUGH 1980

Month	Average Daily Maximum (°F)	Average Daily Minimum (°F)	Mean[a] (°F)
January	26.0	11.3	18.7
February	30.1	15.8	23.0
March	39.2	24.9	32.1
April	53.5	35.6	44.6
May	64.8	44.7	54.8
June	75.0	54.7	64.9
July	79.8	61.1	70.5
August	78.4	60.2	69.3
September	71.2	52.5	61.9
October	59.9	41.9	50.9
November	44.7	29.9	37.3
December	32.0	18.2	25.1
Annual	54.6	37.6	46.1

[a]The monthly mean temperature is the mean of the average daily maximum temperature and the average daily minimum temperature for each month.

Source: National Weather Service and SEWRPC.

Table 9

AVERAGE MONTHLY PRECIPITATION AND SNOW AND SLEET AT MILWAUKEE: 1951 THROUGH 1980

Month	Average Total Precipitation (inches)	Average Snow and Sleet (inches)
January	1.64	13.5
February	1.33	10.5
March	2.58	10.1
April	3.37	2.1
May	2.66	Trace
June	3.59	0.0
July	3.54	0.0
August	3.09	0.0
September	2.88	Trace
October	2.25	0.2
November	1.98	3.4
December	2.03	11.4
Year	30.94	51.2

Source: National Weather Service and SEWRPC.

Precipitation

Precipitation within the study area takes the form of rain, sleet, hail, and snow, and ranges from gentle showers of trace quantities to brief, but intense and potentially destructive, thunderstorms or major rainfall-snowmelt events causing property damage, inundation of poorly drained areas, stream flooding, street and basement flooding, and severe soil erosion and sedimentation. Average monthly and annual total precipitation and snowfall data from the Milwaukee National Weather Service station at General Mitchell Field for the period 1951 through 1980 are presented in Table 9. The average annual total precipitation in the Hales Corners study area based on the Milwaukee National Weather Service station is 30.94 inches, expressed as water equivalent, while the average annual snowfall and sleetfall measured as snow and sleet is 51.2 inches. Assuming that 10 inches of measured snowfall and sleetfall are equivalent to one inch of water, the average annual snowfall of 51.2 inches is equivalent to 5.12 inches of water and, therefore, only about 17 percent of the average annual total precipitation occurs as snowfall and sleet. Average total monthly precipitation for the Hales Corners study area ranges from 1.33 inches in February to 3.59 inches in June. The principal snowfall months are December, January, February, and March, during which 89 percent of the average annual snowfall may be expected to occur.

An important consideration in stormwater drainage is the seasonal nature of precipitation patterns. For larger streams such as the Root River, major flooding events occur during the late winter or early spring as a result of snowmelt and major storms when the ground is either frozen or saturated. However, based on Commission streamflow simulation studies for the period 1940 through 1980, major flooding along Whitnall Park Creek is likely to occur at any time throughout the year except during winter. This is because the drainage area of Whitnall Park Creek is relatively small and consists of predominantly developed urban land. Urban areas contain relatively large proportions of impervious surfaces which inhibit infiltration, thereby significantly increasing surface runoff during even minor rainfall events. Because the dampening effects of infiltration, including leaf interception during summer months, are diminished in urban areas, the annual distribution of flood events in urbanized watersheds is similar to the annual distribution of significant rainfall events, and significant flood events may be expected to occur during spring, summer, and fall.

Extreme precipitation data for southeastern Wisconsin, based on observations for stations located throughout the Region that have relatively long periods of record, are presented in Table 10. The minimum annual precipitation within southeastern Wisconsin, as determined from the tabulated data for the indicated observation period, occurred at Waukesha in 1901 when only 17.30 inches of precipitation occurred, or 55 percent of the average annual precipitation of 31.30 inches for southeastern Wisconsin. The maximum annual precipitation within southeastern Wisconsin occurred at Milwaukee in 1876, when 50.36 inches of precipitation was recorded, equivalent to 161 percent of the average annual precipitation. The maximum monthly precipitation measured in southeastern Wisconsin was 13.14 inches, which occurred at West Bend in August 1924. The maximum 24-hour precipitation recorded in southeastern Wisconsin also occurred in the West Bend area on August 4, 1924, when 7.58 inches of rain fell.

Based on a period of record from 1870 through 1980 at General Mitchell Field, the minimum annual precipitation was 18.69 inches reported in 1901, and the maximum annual precipitation was 50.36 inches reported in 1876. The maximum monthly precipitation was 10.03 inches recorded in June 1917, and the maximum 24-hour precipitation was 5.76 inches also recorded in June 1917. Based on a period of record from 1940 through 1980, the maximum and minimum annual snowfall amounts were 90.8 inches in 1951-52, and 12.1 inches in 1967-68.

Stormwater drainage system design must also consider the characteristics of rainfall events for periods of time substantially shorter than 24 hours. The characteristics of rainfall events over these shorter peak precipitation periods are discussed in the section on hydrology.

Snow Cover and Frost Depth

The likelihood of snow cover and the depth of snow on the ground are important precipitation-related factors that influence the planning, design, construction, and maintenance of stormwater management facilities. Snow cover in the Hales Corners study area is most likely during the months of December, January, and February, during which at least a 0.4 probability exists of having one inch or more of snow cover, as measured at the Milwaukee weather station. The amount of snow cover influences the severity of spring snowmelt-rainfall flood events, which usually occur during March.

The depth and duration of ground frost, or frozen ground, influences hydrologic processes, particularly the proportion of rainfall or snowmelt that will run off the land directly into storm sewerage systems and surface watercourses. The amount of snow cover is an important determinant of frost depth. Since the thermal conductivity of snow cover is less than one-fifth that of moist soil, heat loss from the soil to the colder atmosphere is greatly inhibited by the insulating snow cover. Frozen ground is likely to exist throughout the study area for approximately four months each winter season, extending

Table 10

EXTREME PRECIPITATION PERIODS IN SOUTHEASTERN WISCONSIN: SELECTED YEARS 1870 THROUGH 1981

Observation Station		Period of Precipitation Records Except Where Indicated Otherwise	Total Precipitation						
			Maximum Annual		Minimum Annual		Maximum Monthly		
Name	County		Amount	Year	Amount	Year	Amount	Month	Year
Mitchell Field.....	Milwaukee	1870-1980	50.36[a]	1876	18.69[a]	1901	10.03	June	1917
Racine.............	Racine	1895-1980	48.33	1954	17.75	1910	10.98	May	1933
Waukesha...........	Waukesha	1892-1980	43.57	1938	17.30	1901	11.41	July	1952
West Bend..........	Washington	1922-1980	40.52	1938	19.72	1901	13.14[b]	August	1924
West Allis.........	Milwaukee	1954-1981	42.85	1960	17.49	1963	9.63	June	1954
Mt. Mary College...	Milwaukee	1954-1981	41.25	1965	18.50	1963	10.17	June	1968

[a]Based on the period 1841-1980.

[b]Based on the period 1895-1959 in A Survey Report for Flood Control on the Milwaukee River and Tributaries, U. S. Army Engineer District, Chicago, Corps of Engineers, November 1964.

Source: U. S. Army Corps of Engineers, the National Weather Service, Wisconsin Statistical Reporting Service, and SEWRPC.

from late November through March, with frost penetration to a depth ranging from six inches to more than four feet occurring in January, February, and the first half of March.

Hydrology

Rainfall intensity-duration-frequency relationships are an important element in stormwater management data analysis and system design. Such relationships facilitate determination of the average rainfall intensity--normally expressed in inches per hour--which may be expected to be reached or exceeded for a particular duration at a given recurrence interval. Under its comprehensive water resources planning program, the Southeastern Wisconsin Regional Planning Commission has developed a set of rainfall intensity-duration-frequency relationships using both a graphic procedure and a mathematical curve fitting method executed by a digital computer program. The data, based upon the 64-year record from 1903 through 1966 collected by the National Weather Service at the General Mitchell Field National Weather Service station in Milwaukee, are shown in tabular form in Table 11 and in graphic form in Figure 1. The intensity-duration-frequency equations resulting from the analysis of the Milwaukee data are presented in Table 12. Analyses conducted by the Commission staff indicate that these data are valid for use not only for the Milwaukee area, but anywhere in southeastern Wisconsin.

The volume of rainfall and stormwater associated with a given storm is also useful in assessing the adequacy of stormwater drainage systems. The determination of annual maximum precipitation event volumes was based on about 37 years of hourly precipitation data--January 1, 1940 through October 31, 1976--as recorded at the General Mitchell Field National Weather Service station. These data had been previously obtained, verified, and placed in a computer file under the Commission water resources planning program.

A "discrete" precipitation event may be defined as a continuous or uninterrupted period of rainfall. The available historic records report precipitation on an hourly basis; therefore, in accordance with the above definition, a precipitation event would be defined as the period preceded by and followed by at least one hour during which no precipitation was recorded. The minimum

Table 11

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

Recurrence Interval (years)	Duration and Intensity[b]						
	5 Minutes	10 Minutes	15 Minutes	30 Minutes	1 Hour	2 Hours	24 Hours
2	4.32	3.40	2.89	1.93	1.16	0.70	0.098
5	5.55	4.55	3.79	2.57	1.57	0.95	0.135
10	6.37	5.31	4.38	3.00	1.84	1.12	0.160
25	7.40	6.27	5.13	3.54	2.19	1.33	0.191
50	8.17	6.98	5.69	3.94	2.44	1.48	0.215
100	8.93	7.68	6.23	4.34	2.70	1.54	0.238

[a] These data are based on a statistical analysis of Milwaukee rainfall data for the 64-year period of 1903 through 1966.

[b] Intensity expressed in inches per hour.

Source: SEWRPC.

Table 12

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

Recurrence Interval (years)	Equation[b]	
	Duration of Five Minutes or More But Less Than 60 Minutes	Duration of 60 Minutes or More Through 24 Hours
2	$I = \frac{87.5}{15.4 + T}$	$I = 28.9\ T^{-0.781}$
5	$I = \frac{120.2}{16.6 + T}$	$I = 38.2\ T^{-0.776}$
10	$I = \frac{141.8}{17.1 + T}$	$I = 44.2\ T^{-0.772}$
25	$I = \frac{170.1}{17.8 + T}$	$I = 52.3\ T^{-0.771}$
50	$I = \frac{190.1}{18.0 + T}$	$I = 57.3\ T^{-0.768}$
100	$I = \frac{211.4}{18.4 + T}$	$I = 63.5\ T^{-0.768}$

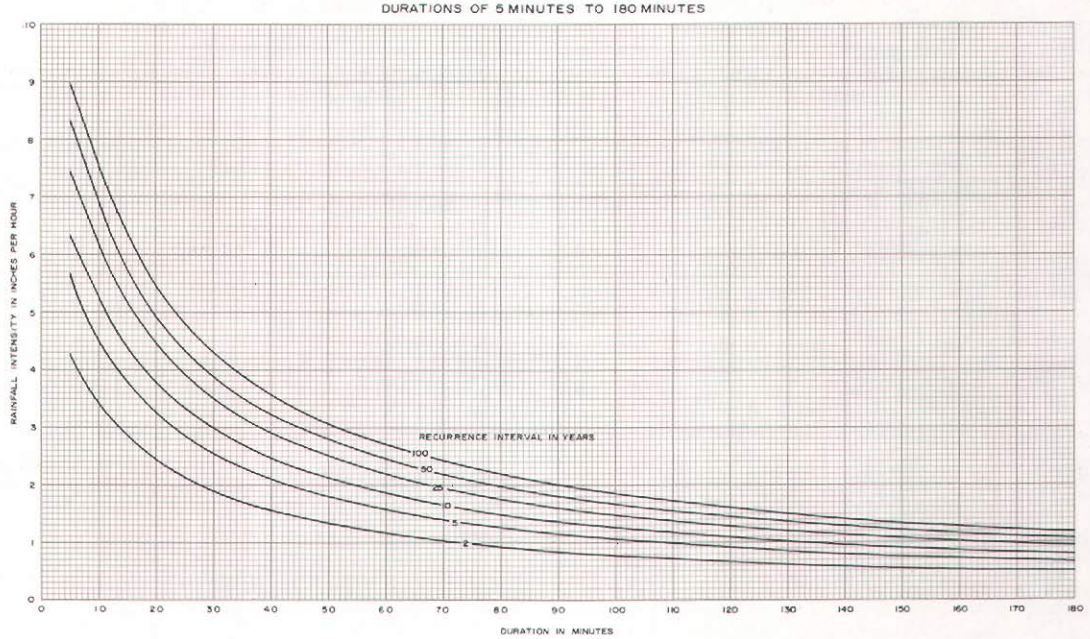
[a] The equations are based on Milwaukee rainfall data for the 64-year period of 1903 through 1966. These equations are applicable, within an accuracy of ±10 percent, to the entire Southeastern Wisconsin Region.

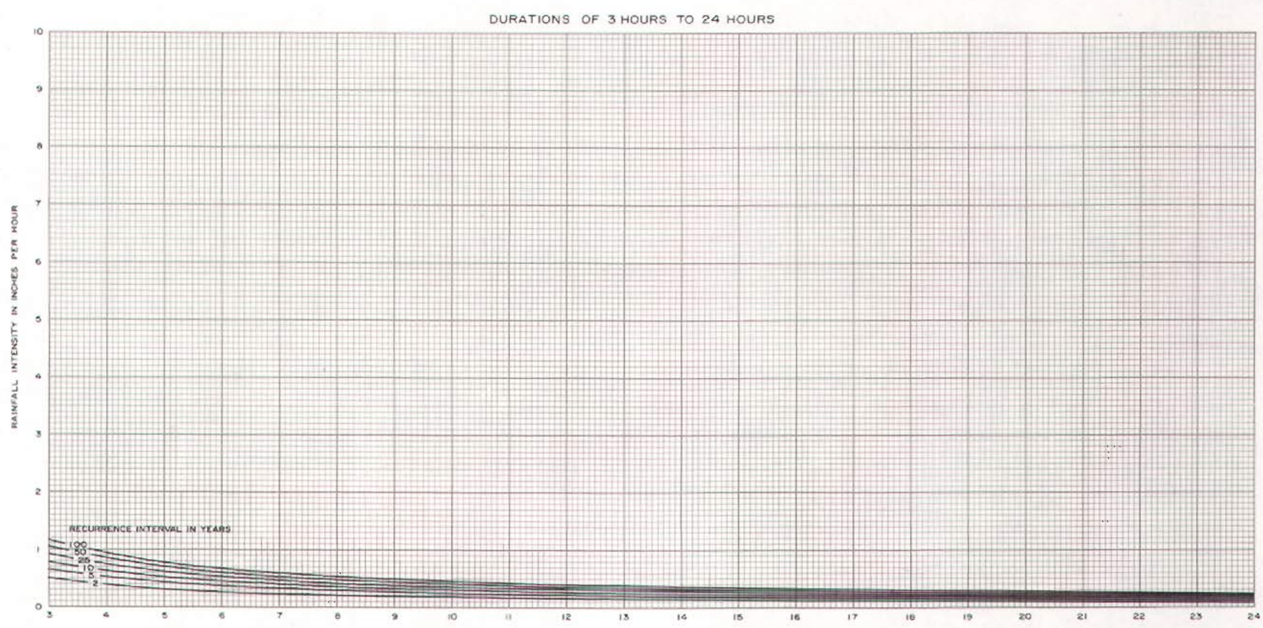
[b] I = Rainfall intensity in inches per hour.
T = Duration in minutes.

Source: SEWRPC.

Figure 1

POINT RAINFALL INTENSITY-DURATION-FREQUENCY RELATIONSHIPS FOR MILWAUKEE, WISCONSIN





Source: SEWRPC.

length of the antecedent and subsequent dry period used to define a precipitation event must be tailored to the intended use of the resulting data on rainfall volumes.

Because of the importance of the minimum length antecedent and subsequent dry period used to define precipitation events, the 37-year percipitation record was analyzed using a range of dry periods. Specifically, the number, time of

Table 13

SELECTED INFORMATION ABOUT PRECIPITATION EVENTS AS DEFINED USING MINIMUM ANTECEDENT AND SUBSEQUENT DRY PERIODS OF 1, 2, 3, 6, 12, AND 24 HOURS[a]

Minimum Antecedent and Subsequent Dry Period (hours)	Number of Precipitation Events		Smallest Event (inches)	Largest Event (inches)	Median Event (inches)
	In 37-Year Period	Average Per Year			
1	6,719	182	0.01	3.42	0.04
2	5,577	151	0.01	4.16	0.06
3	5,008	136	0.01	4.31	0.07
6	4,147	113	0.01	6.05	0.10
12	3,458	94	0.01	6.20	0.14
24	2,842	77	0.01	6.20	0.19

[a] Based on approximately 37 years of hourly precipitation data for the Milwaukee National Weather Service Station from January 1, 1940 through October 31, 1976.

Source: The National Weather Service and SEWRPC.

occurrence, and depth of precipitation events during that period were determined using minimum antecedent and subsequent dry periods of 1, 2, 3, 6, 12, and 24 hours.

Table 13 presents selected information on precipitation events identified for each of the six minimum lengths of antecedent and subsequent dry periods, including the number of events in the 37-year period, the average number of events per year, the depth of the largest and smallest events, and the depth of the median event. As would be expected, the total number of events in the 37-year period and the average number of events per year decreases as the minimum length of the antecedent and subsequent dry period increases. For example, using a minimum antecedent and subsequent dry period of one hour, 6,719 precipitation events occurred during a 37-year period for an average of 182 per year, with the largest event having a depth of 3.42 inches. When the minimum antecedent and subsequent dry period is increased to 24 hours, the number of precipitation events in the 37-year period decreases 58 percent to 2,842, or an average of 77 events per year, and the magnitude of the largest event increases by 81 percent to 6.20 inches.

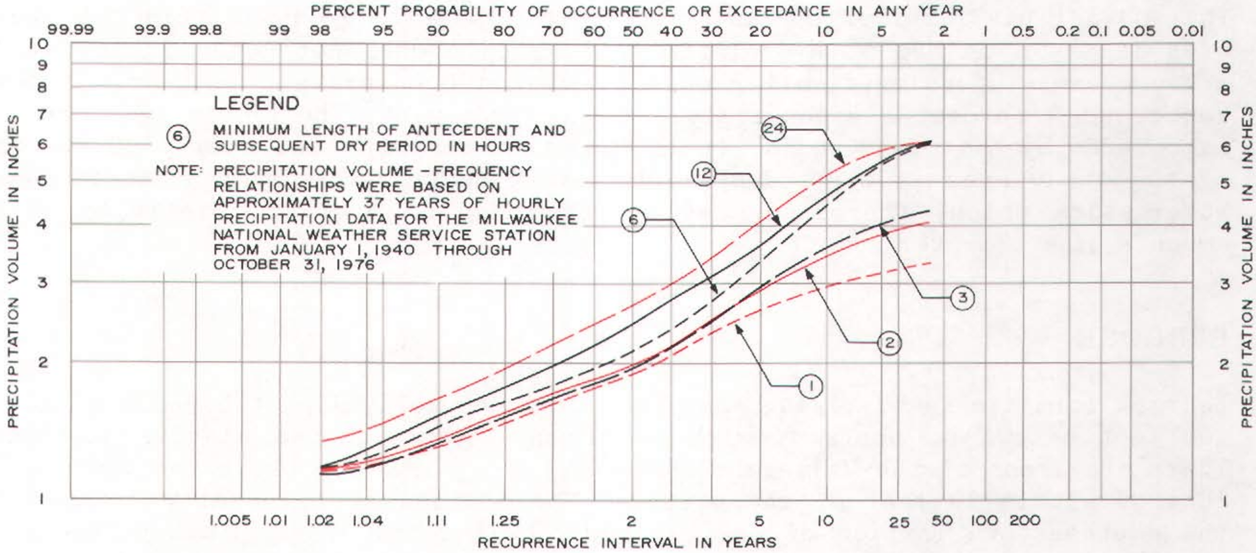
Figure 2 permits determination of a precipitation volume for a specified design frequency or recurrence interval and a specified minimum length antecedent and subsequent dry period. That design precipitation volume can then be converted to a design stormwater runoff volume. Rainfall-runoff relationships and calculations are discussed in more detail in Chapter VII of this report.

SOILS

Soil properties are an important factor influencing the rate and amount of stormwater runoff from land surfaces. The type of soil is also an important consideration in the evaluation of shallow groundwater aquifer recharge and

Figure 2

PRECIPITATION VOLUME-FREQUENCY RELATIONSHIPS FOR A STORM EVENT DEFINED BY MINIMUM ANTECEDENT AND SUBSEQUENT DRY PERIODS OF 1, 2, 3, 6, 12, AND 24 HOURS OVER THE PERIOD OF 1940 THROUGH 1976



Source: SEWRPC.

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.

In order 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 have been published in SEWRPC Planning Report No. 8, Soils of Southeastern Wisconsin. The regional soil surveys have resulted in the mapping of 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, and for underlying stormwater management purposes. Detailed soils maps are available of the study area for use in stormwater management planning.

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:

- Hydrolgic 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 capacity, low permeability, and extremely poor drainage.

The spatial distribution of the four hydrologic soil groups within the study area is shown on Map 5. Hydrologic soil group A does not occur in the study area, whereas hydrologic soil groups B, C, and D comprise 6 percent, 85 percent, and 4 percent, respectively, of the study area. The remaining 5 percent is covered by disturbed soils. It is important to note that nearly 90 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.

BEDROCK

Bedrock formations underlying the study area generally lie at a depth of 20 to 100 feet below the surface, with overlying unconsolidated glacial deposits. There are areas of the Village of Hales Corners, however, where the bedrock is located within 10 feet of the surface. Those areas are generally located in the southeastern portion of the Village. Shallow bedrock conditions have been encountered along S. 92nd Street and along S. 108th Street between W. Grange Avenue and W. College Avenue. This bedrock condition is an important consideration to be taken into account in the development of and cost estimating of alternative stormwater drainage plans.

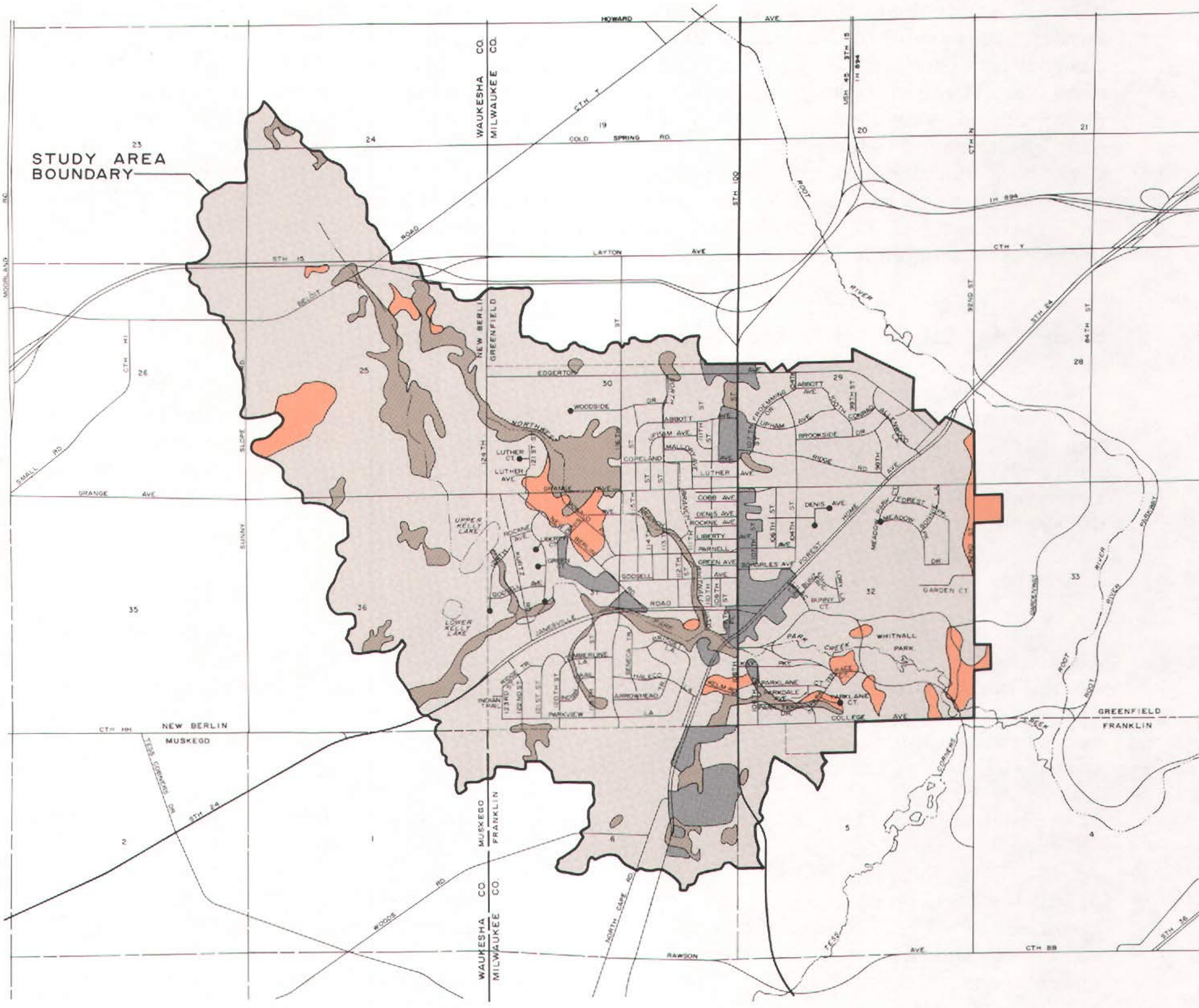
WATER QUALITY

The quality of the surface waters in the study area, primarily Whitnall Park Creek, the North Branch of Whitnall Park Creek, the Northwest Branch of Whitnall Park Creek, and a few lakes and ponds, is an important concern of this study. Improper stormwater management may result in pollutant contributions from the watershed to the streams and in high-flow velocities and volumes, causing erosion of stream banks and undermining of the root systems of trees and shrubs which stabilize these banks. Under these conditions, high pollutant loadings are contributed, some of which are deposited in downstream beds, thereby potentially influencing water quality conditions over a relatively long period of time. Erosion and the resulting sediment contributed to the stream systems also result in the discharge of other pollutants, such as nutrients, pesticides, and metals, which are transported in the stream system attached to sediment particles. High pollutant concentrations and excessive erosion and sedimentation in the streams and ponds also reduce the suitability of these surface waters for recreational uses, such as swimming, fishing, and boating, and limit the ability of the water body to support desirable forms of fish and other aquatic life. Stormwater runoff from urban lands, including lawns and pavements, can contain relatively high concentrations of water pollutants, such as organic substances, nutrients, fecal coliform organisms, metals, and sediment.

During the period 1942 through 1981, sanitary sewage from the Village of Hales Corners was conveyed to, and treated by, a village sewage treatment plant which discharged its treated effluent to Whitnall Park Creek at a location

Map 5

HYDROLOGIC SOIL GROUPS WITHIN THE VILLAGE OF HALES CORNERS STUDY AREA



LEGEND

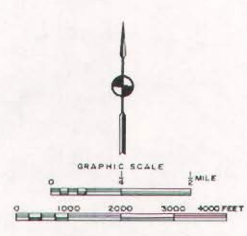
NONE GROUP A WELL DRAINED SOIL

GROUP B MODERATELY DRAINED SOIL

GROUP C POORLY DRAINED SOIL

GROUP D VERY POORLY DRAINED SOIL

MAN MADE FEATURES OR DISTURBED SOILS



Source: SEWRPC.

about 2.2 miles upstream of its confluence with the Root River. Since May 1981, sanitary sewage from the Village of Hales Corners has been collected and treated by the Milwaukee Metropolitan Sewerage District. An analysis of water quality in 1973, prior to the abandonment of the village sewage treatment plant, indicated that the dissolved oxygen concentrations and temperature levels measured in Whitnall Park Creek were generally suitable to support desirable forms of fish and other aquatic life. Measured fecal coliform, ammonia-nitrogen, and phosphorus levels, however, exceeded Commission-recommended standards. These pollutant levels are expected to have significantly improved since the abandonment of the village sewage treatment plant. With the abandonment of the plant and abatement of separate sewer overflows, the remaining sources of water pollution in the planning area are all nonpoint. The abatement of these remaining sources requires careful consideration in any stormwater management planning effort.

STORMWATER DRAINAGE SYSTEM

The existing stormwater drainage system serving the study area consists of the streams and watercourses of the area together with certain constructed drainage facilities. The performance of this system is influenced by, among other factors, the topography of the land surface and the location and extent of the tributary drainage areas, as well as by the characteristics of the streams and watercourses, and related man-made drainage facilities.

Topography

Topography, or the relative elevation of the land surface within the study area, is one of the most important considerations in the planning and design of a stormwater management system. The topography of the land surface defines drainage areas, influences the rate and magnitude of surface water runoff and soil erosion, and determines the uses to which the land can be put and, therefore, related stormwater management needs.

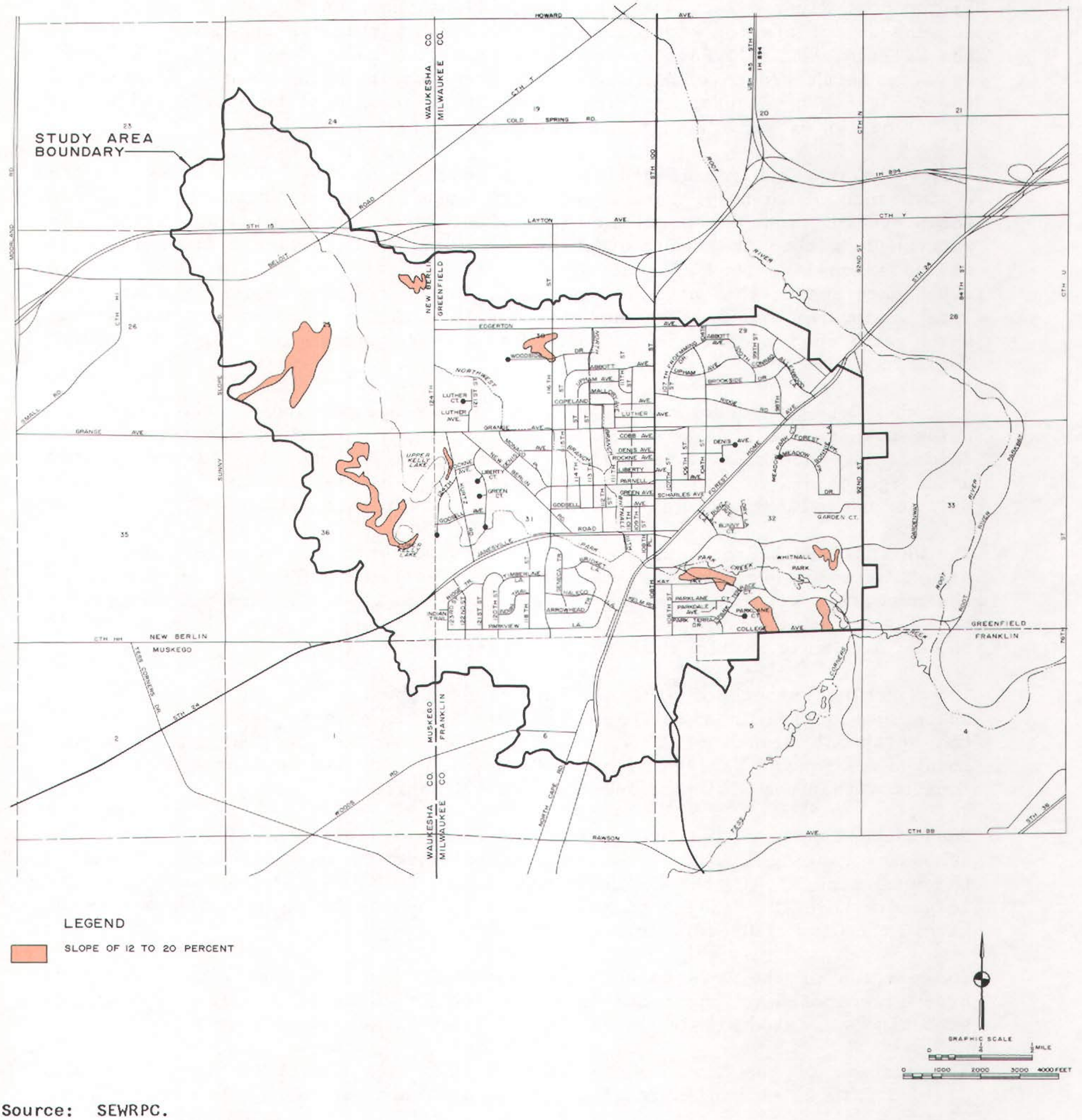
The elevation of the study area ranges from a low of about 710 feet above National Geodetic Vertical Datum (NGVD) in the southeast one-quarter of U.S. Public Land Survey Section 32, Township 6 North, Range 21 East, in the Village of Hales Corners, to a high of about 958 feet NGVD at the southwest one-quarter of U.S. Public Land Survey Section 25, Township 6 North, Range 20 East, in the City of New Berlin. Land surface slopes for small drainage areas range from a low of about 0.3 percent for a drainage area located in the southeast one-quarter of U.S. Public Land Survey Section 30, Township 6 North, Range 21 East, to a high of about 12.3 percent for a drainage area located in the southeast one-quarter of U.S. Public Land Survey Section 25, Township 6 North, Range 20 East. Areas with steep slopes within the study are shown on Map 6. About 96 acres, or about 3 percent of the study area, are marked by slopes ranging from 12 to 20 percent; and about one-third of such steeply sloped area lies within the Village of Hales Corners. The remainder of area with steep slopes is located within the City of New Berlin. In general, areas with slopes greater than 12 percent have severe limitations for urban residential development and, if developed, present serious potential drainage and erosion problems.

Watershed Subbasins

Stormwater from the entire study area, as delineated in Chapter II, is drained to five separate surface water systems--those systems being the intermittent

Map 6

AREA WITH STEEP SLOPES WITHIN THE VILLAGE OF HALES CORNERS STUDY AREA



Source: SEWRPC.

and perennial streams of 1) the Whitnall Park Creek subwatershed, 2) the Northwest Branch of Whitnall Park Creek subwatershed, 3) the North Branch of Whitnall Park Creek subwatershed, 4) the Tess Corners Creek subwatershed, and 5) the Root River direct drainage area, all as shown on Map 1. In addition to serving as outlets for stormwater drainage from within the corporate limits of the Village, Whitnall Park Creek, the Northwest Branch of Whitnall Park Creek, and the North Branch of Whitnall Park Creek drain areas located upstream of the Village. These upstream tributary drainage areas must be considered in the proper design of a stormwater management system for the Village.

For stormwater management planning purposes, the Whitnall Park Creek, Northwest Branch of Whitnall Park Creek, and North Branch of Whitnall Park Creek subwatersheds, and the portions of the Root River and Tess Corners Creek subwatersheds within the study area, were divided into smaller hydrologic units called subbasins. The delineation of these subbasins permits a more accurate representation of the watershed hydrology in the computer model used to simulate stormwater runoff. The subbasin was thus the basic inventory unit within which watershed hydrologic characteristics were quantified prior to hydrologic modeling.

A number of considerations entered into the delineation of the subbasins. Using the available large-scale topographic map prepared to Commission standards, the subbasins were delineated so as to provide desired areas above discharge points at confluences of tributaries and main stems; at, or near, bridges and culverts; and at selected storm sewer inlets and outlets.

The Whitnall Park Creek subwatershed was divided into 147 subbasins ranging in size from about one acre to about 83 acres, as shown on Map 7. Eighty-six of the subbasins within the Whitnall Park Creek subwatershed are located within the Village of Hales Corners. The remaining 61 subbasins are located within the Cities of New Berlin, Muskego, and Franklin.

The Northwest Branch of Whitnall Park Creek subwatershed was divided into 32 subbasins ranging in size from approximately one acre to about 49 acres. In the Northwest Branch of Whitnall Park Creek subwatershed, 23 subbasins are located within the Village of Hales Corners. The remaining nine subbasins are located within the Cities of New Berlin and Greenfield.

The North Branch of Whitnall Park Creek subwatershed was divided into nine subbasins ranging in size from about eight acres to about 31 acres. Eight of the subbasins within the North Branch of Whitnall Park Creek subwatershed are located within the Village of Hales Corners. The remaining subbasin is located within the City of Greenfield.

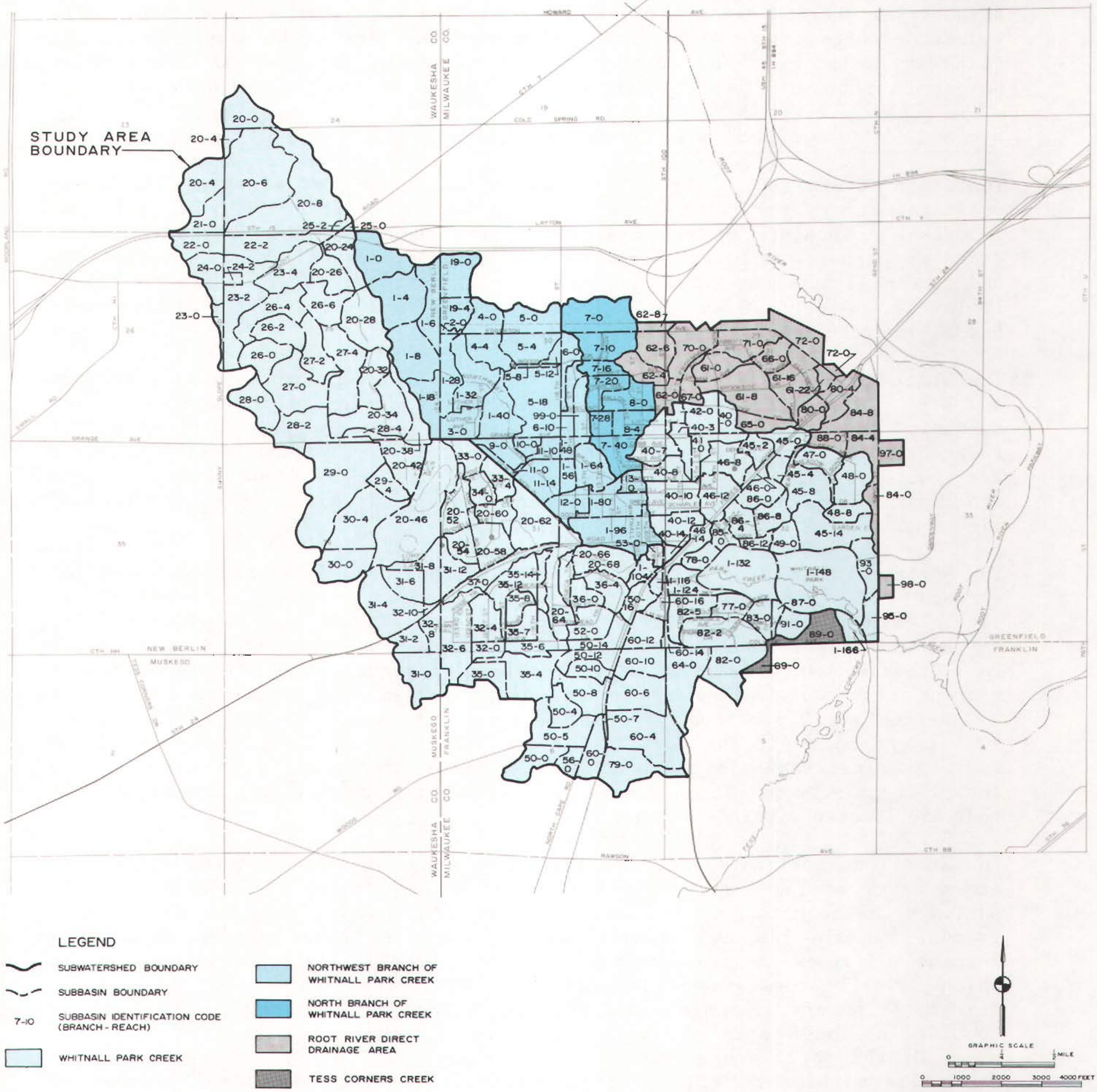
The portion of the Tess Corners Creek subwatershed that lies within the study area is composed of one subbasin approximately 26 acres in size. The entire subbasin is located within the Village of Hales Corners.

The portion of the Root River watershed that lies within the study area was divided into 27 subbasins ranging in size from approximately one acre to about 29 acres, all of which are located within the Village of Hales Corners.

Within the total study area there are 216 subbasins, of which 145, or 67 percent, are located within the Village of Hales Corners. The subbasins have an average size of about 17 acres; the smallest subbasin is about one acre in size, the largest is 83 acres. As shown on Map 7, the subbasins are designated

Map 7

WATERSHED SUBBASINS WITHIN THE VILLAGE OF HALES CORNERS STUDY AREA



Source: SEWRPC.

by a branch number and a reach number. The branch number identifies the individual branch or tributary to the main drainage channel of the subwatershed. The main drainageway is designated by a number, with the major branches or tributaries increasing in order downstream. The reach numbers designate individual segments of the main drainage channel and its tributaries increasing in order downstream. The reach numbering system is designed so that smaller numbered reaches of a branch drain to larger numbered reaches of that branch.

Streams, Drainage Channels, Ponds, and Lakes

The intermittent and perennial streams in the study area serve as the major drainage outlets for the storm sewers and drainage ditches. As such, they are important components of the drainage system which must be characterized in order to properly plan a stormwater management system. All known intermittent and perennial streams, lakes, and ponds in the study area are shown on Map 8. Tables 16, 17, and 28 set forth pertinent characteristics of the drainageways and major storm sewers within each subbasin.

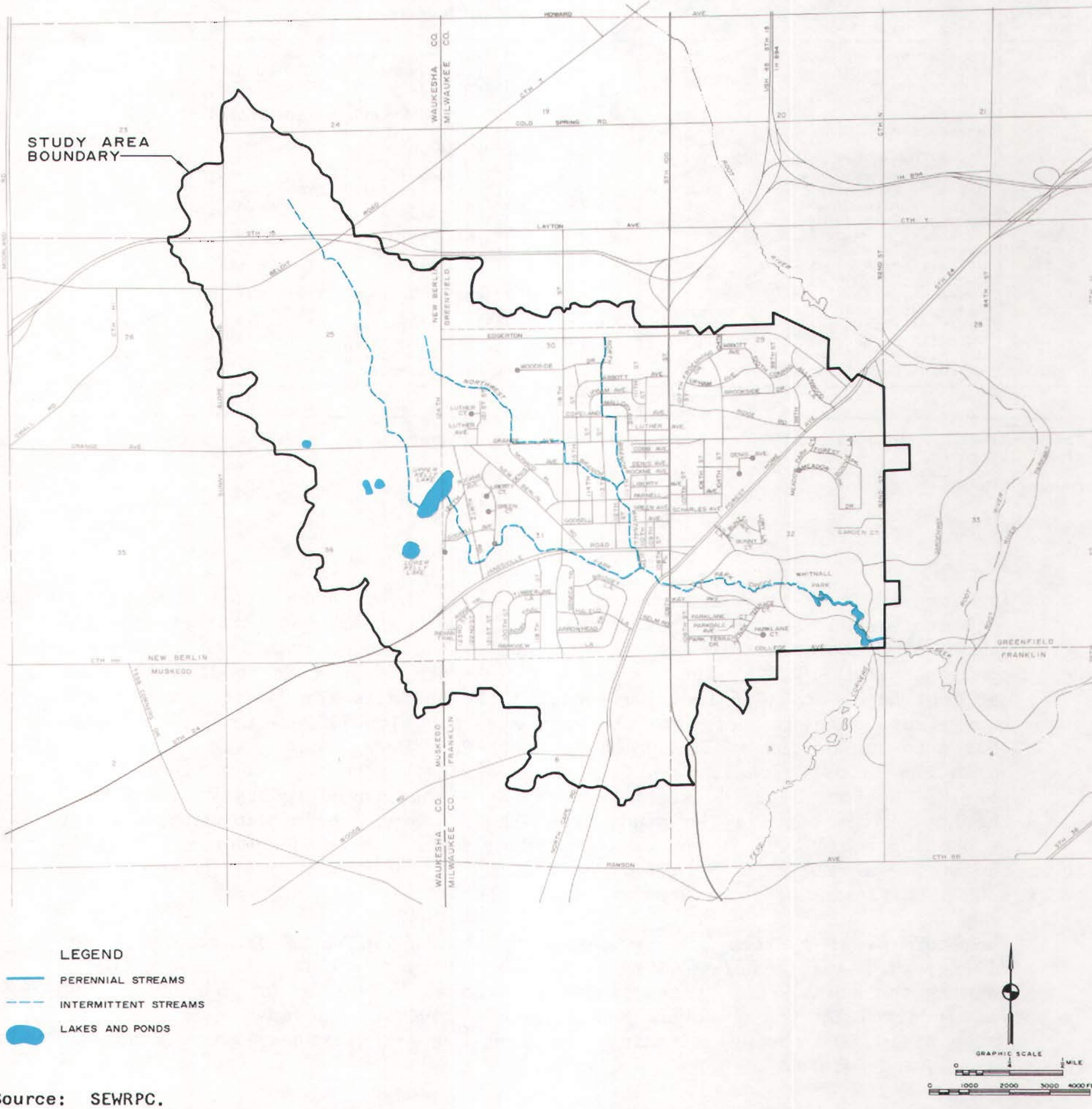
The Whitnall Park Creek subwatershed contains 0.52 mile of perennial streams and 3.92 miles of intermittent streams. Average streambed slopes within each subbasin range from 0.2 percent to 1.0 percent. Typical channel bottom widths range from almost zero to about 27 feet. Upper Kelly Lake and Lower Kelly Lake are located within the subwatershed and have surface areas of about 13.2 and 3.5 acres, respectively. There are also five ponds within the subwatershed with a total area of 12.1 acres.

The Northwest Branch of Whitnall Park Creek subwatershed contains no perennial streams and 1.73 miles of intermittent streams. Average streambed slopes within each subbasin range from 0.1 percent to 1.2 percent. Typical channel bottom widths vary from almost zero to 14 feet. The North Branch of Whitnall Park Creek subwatershed contains no perennial streams and 0.79 mile of intermittent streams. Average streambed slopes within each subbasin range from 0.2 percent to 0.8 percent. Typical channel bottom widths range from almost zero to eight feet. The portions of the Tess Corners Creek subwatershed and Root River watershed that lie within the study area contain no perennial or intermittent streams. There are also no lakes or ponds within these watersheds that are located within the study area.

The location, configuration, and tributary areas of the existing engineered storm sewer system serving the Village of Hales Corners is shown on the map enclosed in the pocket on the back cover of this report, together with street grades, manhole rim and sewer invert elevations, sewer grades, and sewer lengths and sizes. As presented in Table 14, the existing storm sewer system--which actually consists of 16 individual subsystems--serves, or potentially serves, tributary drainage areas ranging in size from 1 to 95 acres, with a combined drainage area of about 274 acres, or about 7 percent of the study area. Of the total storm-sewered drainage area, 252 acres, or about 92 percent, lie within the Village of Hales Corners. The remaining 22 acres, or about 8 percent, lie within the City of Franklin but discharge into the Village of Hales Corners storm sewer system. The length of each storm sewer system ranges from 164 to 13,207 feet with a combined total of about 36,637 lineal feet of sewers. The sewers range in size from 12 inches to 66 inches in diameter.

Map 8

INTERMITTENT AND PERENNIAL STREAMS WITHIN THE VILLAGE OF HALES CORNERS STUDY AREA



Source: SEWRPC.

Table 14

CHARACTERISTICS OF INDIVIDUAL STORM SEWER SYSTEMS WITHIN THE VILLAGE OF HALES CORNERS

Storm Sewer System Number	Receiving Stream	Tributary Area (acres)	Length of Storm Sewer (feet)	Range of Storm Sewer Sizes	Range of Storm Sewer Slopes (ft/ft)	Number of Storm Sewer Inlets and Catch Basins	Number of Manholes
1	Root River............	30	2,462	15"-48" diameter	.0019-.0187	34	14
2	Whitnall Park Creek....	10	1,485	15"-27" diameter	.0060-.0080	3	0
3	Root River............	18	1,610	15"-30" diameter	.0206-.0324	16	7
4	Northwest Branch of Whitnall Park Creek...	5	643	18"-24" diameter	.0036-.0058	6	5
5	Whitnall Park Creek....	5	1,092	18"-29" pipe arch	.0030	1	7
6	North Branch of Whitnall Park Creek...	2	164	12" diameter	.019	3	0
7	Whitnall Park Creek....	34	4,064	12"-48" diameter	.0024-.0440	49	4
8	Whitnall Park Creek....	18	3,360	12"-60" diameter	.0018-.0289	24	12
9	Northwest Branch of Whitnall Park Creek...	1	287	12" diameter	.0050-.050	0	2
10	Whitnall Park Creek....	3	545	18"-36" diameter	.0038-.0050	5	0
11	Whitnall Park Creek....	4	196	15" diameter	.0096-.0184	4	2
12	Northwest Branch of Whitnall Park Creek...	3	576	12"-24" diameter	.0035-.0113	11	4
13	Northwest Branch of Whitnall Park Creek...	16	1,713	12"-30" diameter	.0020-.0120	13	10
14	Whitnall Park Creek....	27	4,307	42"-48" diameter	.0030-.0261	18	8
15	Whitnall Park Creek....	95	13,207	12"-66" diameter	.0017-.0857	71	36
16	Whitnall Park Creek....	3	926	12"-48" diameter	.0060-.0083	6	9
Total		274[a]	36,637	--	--	263	124

[a]Of this total tributary area, 22 acres are located in the City of Franklin and 252 acres are located in the Village of Hales Corners.

Source: SEWRPC.

As shown in Table 14, the number of storm sewer inlets and catch basins within each storm sewer system ranges from 0 to 71, with a combined total of 257. The number of manholes within each storm sewer system ranges from 1 to 36, with a combined total of 124. The slopes of the sewers range from about 0.002 foot per foot to about 0.086 foot per foot. Of the outfalls for the 16 storm sewer subsystems, nine discharge to the main stem of Whitnall Park Creek; four discharge to the Northwest Branch of Whitnall Park Creek; one discharges to the North Branch of Whitnall Park Creek; one discharges to an intermittent tributary of the Root River outside the study area; and one discharges directly to the Root River outside the study area. There are no public stormwater storage or pumping facilities in the existing storm sewer system. Chapter VII of this report presents a more detailed description of the storm sewer system within each subbasin in the study area.

The storm sewer systems are maintained by the Public Works Department of the Village of Hales Corners and by Milwaukee County. In 1984, the cost of maintaining the storm sewer systems was estimated at $6,500, of which about $5,000 was incurred by the Village. Maintenance activities include sewer, culvert, catch basin, and channel cleaning; and minor repair work on sewers, manholes, basins, and inlets.

Estimates of the peak flows and average total annual flows discharged from the existing storm sewer system to receiving streams are set forth in Chapter VII. A description of the design rainfall recurrence interval used to estimate those flows is presented in Chapter V.

Wetlands

Wetlands are natural areas in which the groundwater table lies near, at, or above the surface of the ground, and which support certain types of vegetation. Wetlands are usually covered by organic soils, silts, and marl deposits. Wetlands provide valuable ecological habitats, enhance water quality conditions by trapping pollutants, and stabilize streamflows by storing peak discharges and releasing water during low flow conditions. Wetlands also have important recreational, educational, and aesthetic values.

A sound stormwater management plan should, to the extent practicable, utilize the stormwater storage capacity of any existing natural wetlands, incorporating this storage into the drainage system. Thus, wetland preservation should be an integral part of a stormwater management plan. Wetlands in the study area were identified in a special inventory conducted by the Commission using aerial photographic interpretation and field inspection supplemented by analysis of mapped soil data. The location, type, and extent of wetlands in the study area are shown on Map 9 and quantified in Table 15. In 1979, there were approximately 115 acres of wetlands in the study area, comprising about 3 percent of that area. Within the Village of Hales Corners, there were about 44 acres of wetlands, comprising about 2 percent of that area.

Bridges, Culverts, and Other Structures

Bridges and culverts significantly influence the hydraulic behavior of a stream system. Constrictions caused by inadequately designed bridges and culverts can, during storm events, result in large backwater effects, thereby creating a floodland area upstream of the structure that is significantly larger than that which would exist in the absence of the bridge or culvert.

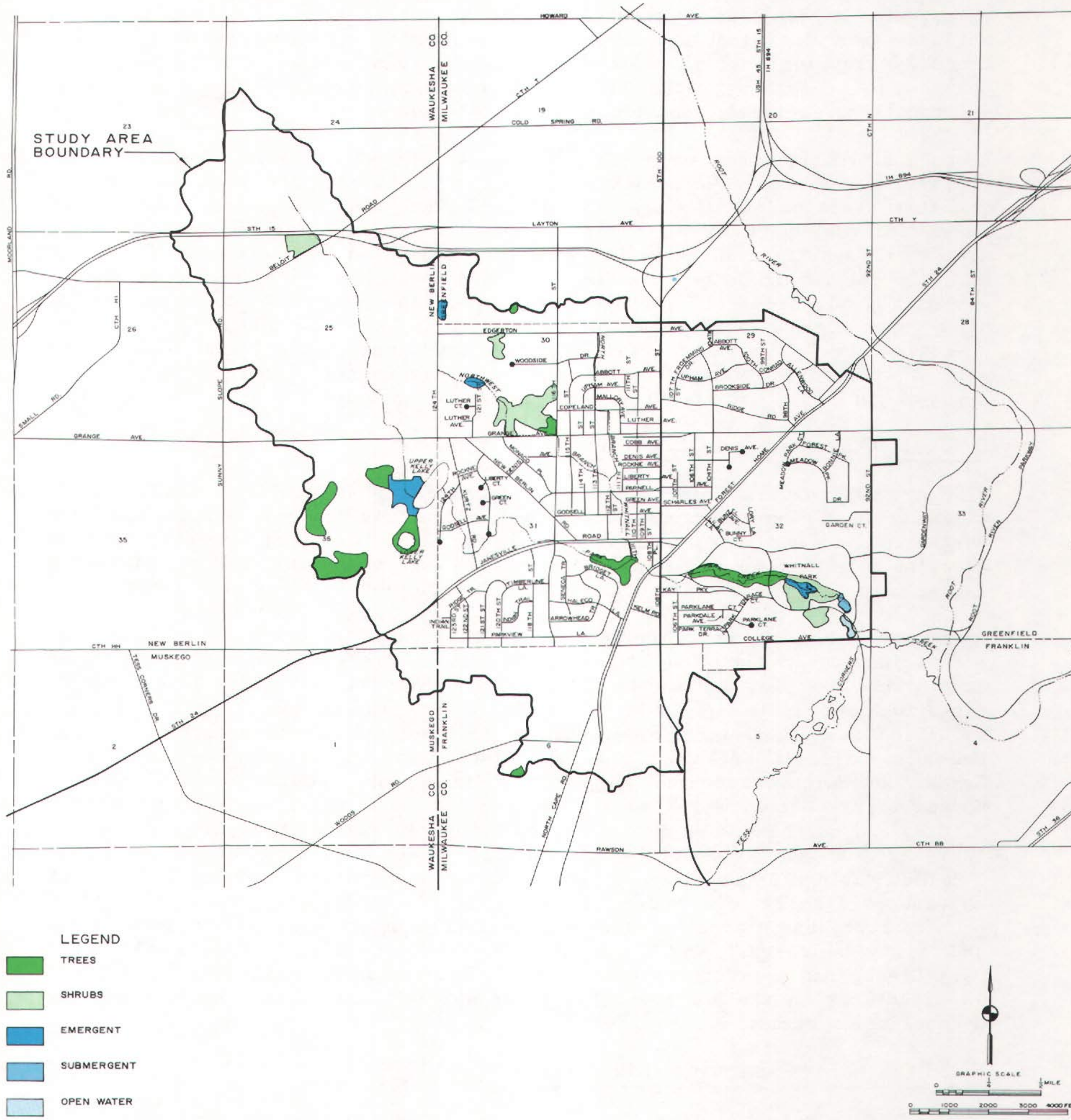
As shown on Map 10, Whitnall Park Creek is crossed 33 times by roadways and pedestrianways in the study area; the Northwest Branch of Whitnall Park Creek is crossed 10 times, and the North Branch of Whitnall Park Creek is crossed 17 times. As set forth in Table 16, a determination was made of the hydraulic significance of each existing structure; that is, whether or not the structure had a significant effect on the peak discharges and stages of Whitnall Park Creek, the Northwest Branch of Whitnall Park Creek, and the North Branch of Whitnall Park Creek. Based on a federal flood insurance study report, certain bridges and culverts were determined to be hydraulically insignificant because they were of such size or elevation as not to increase flood stages more than 0.1 foot during 10- to 100-year recurrence interval storm events. A bridge or culvert is likely to be hydraulically insignificant if it spans a stream from bank to bank, has approach roadways with little or no fill on the floodplain, and has a relatively small superstructure. Eighteen structures on Whitnall Park Creek, nine structures on the North Branch of Whitnall Park Creek, and 10 structures on the Northwest Branch of Whitnall Park Creek were found to be hydraulically significant.

Flood Discharges and Natural Floodlands

A flood insurance study for the Village of Hales Corners was prepared by the U.S. Geological Survey for the Federal Emergency Management Agency in 1977. The study estimated peak flood discharges at selected structure locations for the 10-, 50-, 100-, and 500-year recurrence interval flood events under existing (1977) conditions within the Village of Hales Corners as presented in

Map 9

WETLAND VEGETATIVE COVER TYPES WITHIN THE VILLAGE OF HALES CORNERS STUDY AREA: 1979



Source: SEWRPC.

Table 15

WETLAND VEGETATIVE COVER AND EXTENT WITHIN THE VILLAGE OF HALES CORNERS STUDY AREA: 1979

Dominant Wetland Vegetative Cover Type	Village of Hales Corners		Study Area Outside the Village of Hales Corners		Total Study Area	
	Acres	Percent of Total	Acres	Percent of Total	Acres	Percent of Total
Trees	21.0	29.5	22.2	50.8	43.2	37.6
Shrubs	40.5	57.0	3.2	7.3	43.7	38.1
Emergent	7.3	10.3	18.3	41.9	25.6	22.3
Submergent	0.9	1.2	--	--	0.9	0.8
Open Water	1.4	2.0	--	--	1.4	1.2
Total	71.1	100.0	43.7	100.0	114.8	100.0

Source: SEWRPC.

Table 17. The 100-year recurrence interval peak flood discharge of Whitnall Park Creek approximately one mile above the downstream limits of the study area was estimated at 1,800 cubic feet per second (cfs). The 100-year recurrence interval peak flood discharge of the Northwest Branch of Whitnall Park Creek just upstream from the confluence with the main stem of Whitnall Park Creek was estimated at 540 cfs. The 100-year recurrence interval peak flood discharge of the North Branch of Whitnall Park Creek at the confluence with the Northwest Branch of Whitnall Park Creek was estimated at 220 cfs. These flood flows were reviewed in conjunction with the preparation of estimated flows in the existing stormwater drainage system under this study. Chapter V presents refined estimates of the flood flows under planned land use and channel conditions.

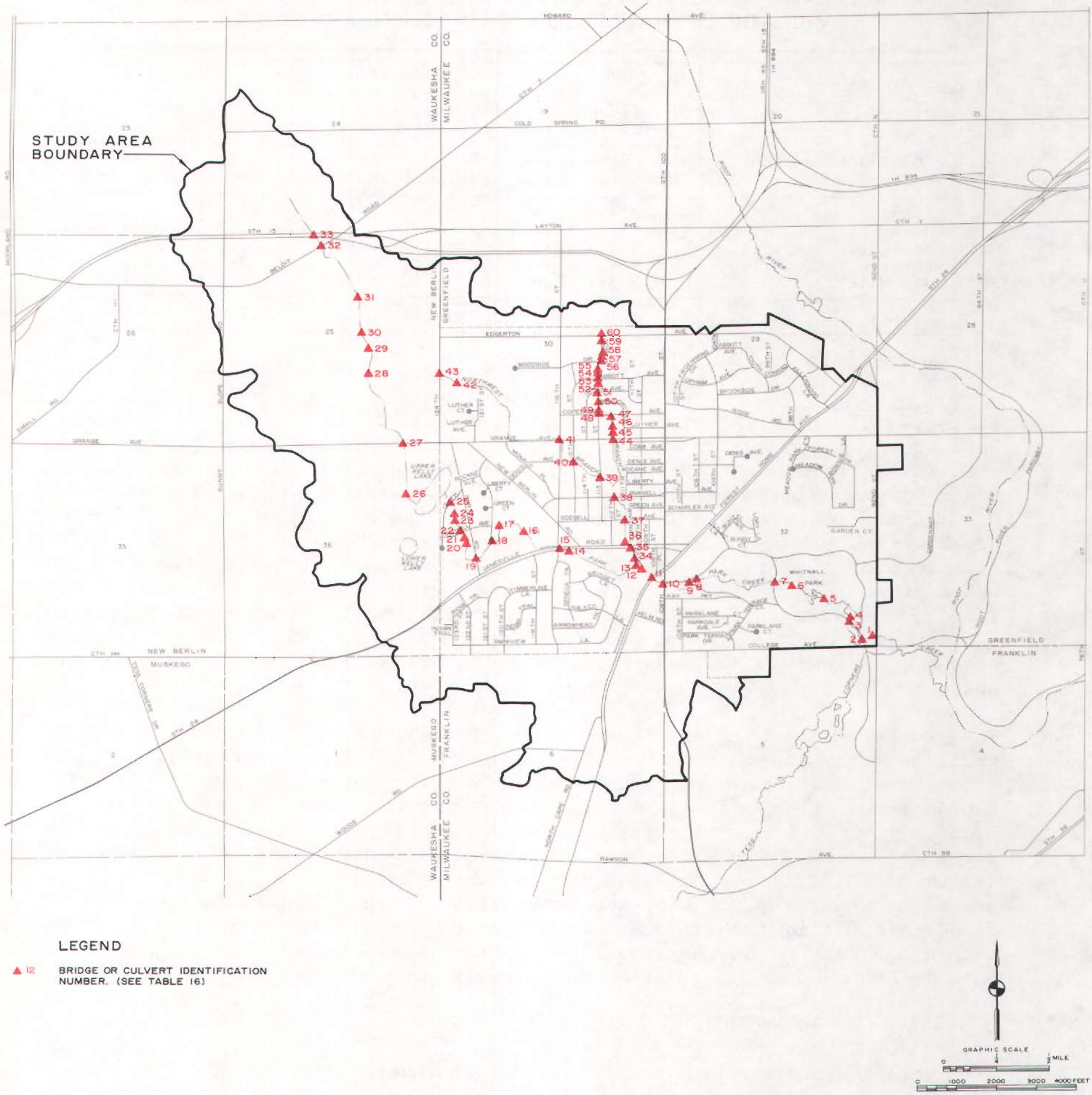
The federal flood insurance study report includes flood insurance rate maps which show the expected surface elevations of the base 100-year flood and the attendant flood hazard areas under existing land use and channel conditions. Map 11 shows the flood hazard areas as delineated in the federal flood study. Floodlands within the total study area occupy an area of approximately 266 acres, of about 7 percent of the study area. Floodlands within the corporate limits of the Village of Hales Corners occupy an area of approximately 182 acres, or about 9 percent of the total area of the Village. The 84 acres of floodlands within the study area but outside the Village are all located within the City of New Berlin. Floodland areas within the City of New Berlin were delineated in a federal flood insurance study prepared in 1975.

STORMWATER MANAGEMENT PROBLEMS

Stormwater management problems consist of stormwater drainage and flood control problems. Drainage problems may be defined as the accumulation of excess stormwater on the land surface before such water has entered stream channels. Such problems are caused by stormwater runoff attempting to reach the stream channels. Flood control problems may be defined as damage from the overflow of natural stream channels and watercourses. Such problems are caused by streamflow exceeding the bank's full capacity and moving away from the stream channels to inundate adjacent floodlands.

Map 10

LOCATION OF BRIDGES AND CULVERTS IN THE VILLAGE OF HALES CORNERS STUDY AREA: 1984



Source: SEWRPC.

Table 16

STRUCTURE INFORMATION FOR WHITNALL PARK CREEK, THE NORTHWEST BRANCH OF WHITNALL PARK CREEK, AND THE NORTH BRANCH OF WHITNALL PARK CREEK: 1984

Stream	Identification Number on Map 10	Structure Name	U. S. Public Land Survey Section	Structure Type		Estimated Hydraulically Significant
				Bridge	Culvert	
Whitnall Park Creek	1	S. 92nd Street	Southeast one-quarter, southeast one-quarter, Section 32, Town 6 North, Range 21 East	X	--	Yes
	2	Whitnall Park Road	Southeast one-quarter, southeast one-quarter, Section 32, Town 6 North, Range 21 East	X	--	No
	3	Whitnall Park Road	Southeast one-quarter, southeast one-quarter, Section 32, Town 6 North, Range 21 East	X	--	No
	4	Pedestrian Bridge	Southeast one-quarter, southeast one-quarter, Section 32, Town 6 North, Range 21 East	X	--	No
	5	Pedestrian Bridge	Southwest one-quarter, southeast one-quarter, Section 32, Town 6 North, Range 21 East	X	--	No
	6	Pedestrian Bridge	Northwest one-quarter, southeast one-quarter, Section 32, Town 6 North, Range 21 East	X	--	No
	7	Whitnall Park Drive	Northwest one-quarter, southeast one-quarter, Section 32, Town 6 North, Range 21 East	X	--	Yes
	8	Whitnall Park Drive	Northwest one-quarter, southwest one-quarter, Section 32, Town 6 North, Range 21 East	X	--	Yes
	9	Whitnall Park Drive	Northwest one-quarter, southwest one-quarter, Section 32, Town 6 North, Range 21 East	X	--	Yes
	10	STH 100 (S. 108th Street)	Northwest one-quarter, southwest one-quarter, Section 32, Town 6 North, Range 21 East	X	--	Yes
	11	CTH 00 (W. Forest Home)	Northeast one-quarter, southeast one-quarter, Section 31, Town 6 North, Range 21 East	X	--	Yes
	12	Parking Lot Bridge	Northeast one-quarter, southeast one-quarter, Section 31, Town 6 North, Range 21 East	X	--	Yes
	13	Parking Lot Bridge	Northeast one-quarter, southeast one-quarter, Section 31, Town 6 North, Range 21 East	X	--	Yes
	14	Pedestrian Bridge	Northwest one-quarter, southeast one-quarter, Section 31, Town 6 North, Range 21 East	X	--	No
	15	STH 24 (Janesville Road)	Northwest one-quarter, southeast one-quarter, Section 31, Town 6 North, Range 21 East	--	X	Yes
	16	Pedestrian Bridge	Southeast one-quarter, northwest one-quarter, Section 31, Town 7 North, Range 21 East	X	--	No
	17	Parking Lot Bridge	Southwest one-quarter, northwest one-quarter, Section 31, Town 6 North, Range 21 East	--	X	Yes
	18	Pedestrian Bridge	Southwest one-quarter, northwest one-quarter, Section 31, Town 6 North, Range 21 East	X	--	No

Table 16 (continued)

Stream	Identification Number on Map 10	Structure Name	U. S. Public Land Survey Section	Structure Type		Estimated Hydraulically Significant
				Bridge	Culvert	
Whitnall Park Creek (continued)	19	S. Kurtz Road	Northwest one-quarter, southwest one-quarter, Section 31, Town 6 North, Range 21 East	--	X	Yes
	20	Pedestrian Bridge	Northwest one-quarter, southwest one-quarter, Section 31, Town 6 North, Range 21 East	X	--	No
	21	Pedestrian Bridge	Southwest one-quarter, northwest one-quarter, Section 31, Town 6 North, Range 21 East	X	--	No
	22	Godsell Road	Southwest one-quarter, northwest one-quarter, Section 31, Town 7 North, Range 21 East	--	X	Yes
	23	Pedestrian Bridge	Southwest one-quarter, northwest one-quarter, Section 31, Town 6 North, Range 21 East	X	--	No
	24	Pedestrian Bridge	Southwest one-quarter, northwest one-quarter, Section 31, Town 6 North, Range 21 East	X	--	No
	25	S. 124th Street	Southwest one-quarter, northwest one-quarter, Section 31, Town 6 North, Range 21 East	X	--	Yes
	26	St. Mary's Drive	Northeast one-quarter, northeast one-quarter, Section 36, Town 6 North, Range 20 East	X	--	No
	27	W. Grange Avenue	Northeast one-quarter, northeast one-quarter, Section 36, Town 6 North, Range 20 East	--	X	Yes
	28	Marquette Drive	Northwest one-quarter, southwest one-quarter, Section 25, Town 6 North, Range 20 East	X	X	Yes
	29	Balboa Drive	Northwest one-quarter, southeast one-quarter, Section 25, Town 6 North, Range 20 East	X	X	Yes
	30	Cherrytree Lane	Southwest one-quarter, northeast one-quarter, Section 25, Town 6 Range, Range 20 East	--	X	No
	31	Radisson Drive	Southwest one-quarter, northeast one-quarter, Section 25, Town 6 North, Range 20 East	--	X	No
	32	Courtland Parkway-Beloit	Northwest one-quarter, northeast one-quarter, Section 25, Town 6 North, Range 20 East	--	X	Yes
	33	STH 75	Northeast one-quarter, northwest one-quarter, Section 25, Town 6 North, Range 20 East	--	X	Yes
Northwest Branch of Whitnall Park Creek	34	Driveway	Northeast one-quarter, southeast one-quarter, Section 31, Town 6 North, Range 20 East	X	--	Yes
	35	Janesville Road	Southeast one-quarter, northeast one-quarter, Section 31, Town 6 North, Range 21 East	--	X	Yes
	36	Driveway Culvert	Southeast one-quarter, northeast one-quarter, Section 31, Town 6 North, Range 21 East	--	X	Yes
	37	W. Godsell Avenue	Southeast one-quarter, northeast one-quarter, Section 31, Town 6 North, Range 21 East	--	X	Yes

Table 16 (continued)

Stream	Identification Number on Map 10	Structure Name	U. S. Public Land Survey Section	Structure Type		Estimated Hydraulically Significant
				Bridge	Culvert	
Northwest Branch of Whitnall Park Creek (continued)	38	W. Parnell Avenue	Southeast one-quarter, northeast one-quarter, Section 31, Town 6 North, Range 21 East	--	X	Yes
	39	S. 113th Street	Northwest one-quarter, northeast one-quarter, Section 31, Town 6 North, Range 21 East	--	X	Yes
	40	S. 115th Street	Northwest one-quarter, northeast one-quarter, Section 31, Town 6 North, Range 21 East	--	X	Yes
	41	W. Grange Avenue and S. 116th Street 4 culverts	Northwest one-quarter, northeast one-quarter, Section 31, Town 6 North, Range 21 East	--	X	Yes
	42	Robinwood Lane	Northwest one-quarter, southwest one-quarter, Section 30, Town 6 North, Range 21 East	--	X	Yes
	43	S. 124th Street	Northwest one-quarter, southwest one-quarter, Section 30, Town 6 North, Range 21 East	--	X	Yes
North Branch of Whitnall Park Creek	44	W. Grange Avenue	Northeast one-quarter, northeast one-quarter, Section 31, Town 6 North, Range 21 East	--	X	No
	45	Driveway Culvert	Southeast one-quarter, southeast one-quarter, Section 30, Town 6 North, Range 21 East	--	X	Yes
	46	Driveway Culvert	Southeast one-quarter, southeast one-quarter, Section 30, Town 6 North, Range 21 East	--	X	No
	47	S. 112th Street	Southeast one-quarter, southeast one-quarter, Section 30, Town 6 North, Range 21 East	--	X	No
	48	W. Copeland Avenue	Southwest one-quarter, southeast one-quarter, Section 30, Town 6 North, Range 21 East	--	X	No
	49	Driveway Culvert	Southwest one-quarter, southeast one-quarter, Section 30, Town 6 North, Range 21 East	--	X	No
	50	W. Mallory Avenue	Southwest one-quarter, southeast one-quarter, Section 30, Town 6 North, Range 21 East	--	X	Yes
	51	W. Upham Avenue	Southwest one-quarter, southeast one-quarter, Section 30, Town 6 North, Range 21 East	--	X	No
	52	Driveway Culvert	Northwest one-quarter, southeast one-quarter, Section 30, Town 6 North, Range 21 East	--	X	Yes
	53	W. Abbott Avenue	Northwest one-quarter, southeast one-quarter, Section 30, Town 6 North, Range 21 East	--	X	Yes
	54	Driveway Culvert	Northwest one-quarter, southeast one-quarter, Section 30, Town 6 North, Range 21 East	--	X	Yes
	55	Driveway Culvert	Northwest one-quarter, southeast one-quarter, Section 30, Town 6 North, Range 21 East	--	X	Yes

Table 16 (continued)

Stream	Identification Number on Map 10	Structure Name	U. S. Public Land Survey Section	Structure Type: Bridge	Structure Type: Culvert	Estimated Hydraulically Significant
North Branch of Whitnall Park Creek (continued)	56	Woodside Drive	Northwest one-quarter, southeast one-quarter, Section 30, Town 6 North, Range 21 East	--	X	Yes
	57	Driveway Culvert	Northwest one-quarter, southeast one-quarter, Section 30, Town 6 North, Range 21 East	--	X	Yes
	58	Driveway Culvert	Northwest one-quarter, southeast one-quarter, Section 30, Town 6 North, Range 21 East	--	X	No
	59	Driveway Culvert	Northwest one-quarter, southeast one-quarter, Section 30, Town 6 North, Range 21 East	--	X	Yes
	60	W. Edgerton Avenue	Northwest one-quarter, southeast one-quarter, Section 30, Town 6 North, Range 21 East	--	X	No

Source: SEWRPC.

Table 17

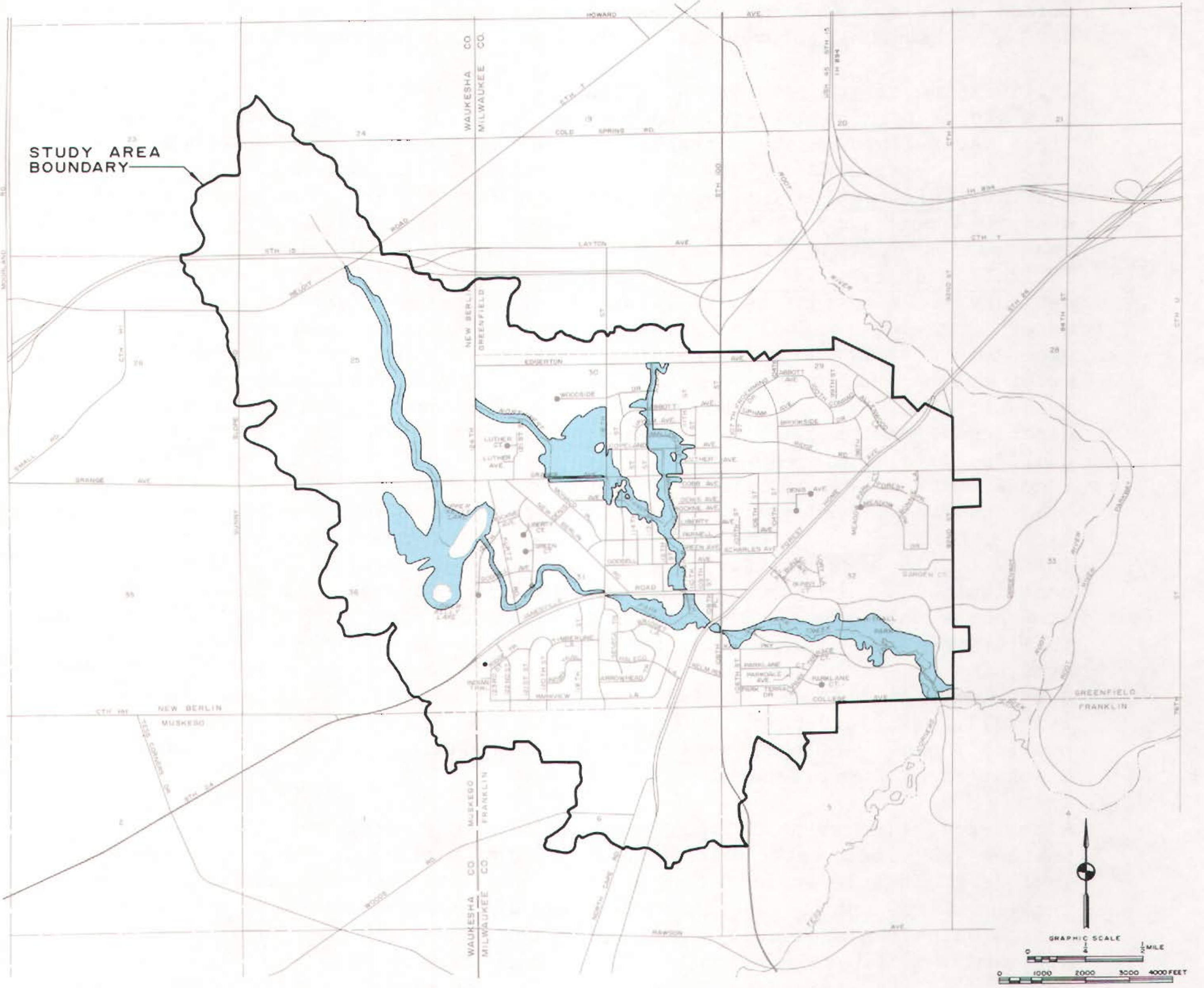
EXISTING FLOOD DISCHARGES FOR WHITNALL PARK CREEK, THE NORTHWEST BRANCH OF WHITNALL PARK CREEK, AND THE NORTH BRANCH OF WHITNALL PARK CREEK: 1977

Stream	U. S. Public Land Survey Section	Discharge Location	Peak Discharges (cubic feet per second)			
			10-Year Recurrence Interval Flood Event	50-Year Recurrence Interval Flood Event	100-Year Recurrence Interval Flood Event	500-Year Recurrence Interval Flood Event
Whitnall Park Creek	Northwest one-quarter, southeast one-quarer, Section 32, Town 6 North, Range 21 East	Whitnall Park Drive	1,000	1,500	1,800	2,300
Whitnall Park Creek	Northeast one-quarter, southeast one-quarter, Section 31, Town 6 North, Range 21 East	500 feet upstream of Forest Home Road	370	580	670	970
Northwest Branch of Whitnall Park Creek	Northeast one-quarter, southeast one-quarter, Section 31, Town 6 North, Range 21 East	Confluence with Whitnall Park Creek	320	470	540	710
Northwest Branch of Whitnall Park Creek	Northwest one-quarter, northeast one-quarter, Section 31, Town 6 North, Range 21 East	Confluence with North Branch of Whitnall Park Creek	210	310	360	470
North Branch of Whitnall Park Creek	Northeast one-quarter, northeast one-quarter, Section 31, Town 6 North, Range 21 East	Confluence with Northwest Branch of Whitnall Park Creek	130	190	220	280

Source: U. S. Geological Survey and the Federal Emergency Management Agency 1977 flood insurance study.

Map 11

100-YEAR RECURRENCE INTERVAL FLOODPLAIN WITHIN THE VILLAGE OF HALES CORNERS STUDY AREA UNDER EXISTING LAND USE AND CHANNEL CONDITIONS



Source: U. S. Geological Survey, the Federal Emergency Management Agency 1977 flood insurance study, and SEWRPC.

Within the study area most drainage problems are related to high groundwater levels which require the operation of building sump pumps over extended periods of time and which contribute to the ponding of stormwater in ditches and low areas during wet weather conditions. In some areas, these drainage problems are aggravated by the existence of drainage ditches with insufficient slopes. In May 1984, the Village of Hales Corners held a public meeting to record citizen knowledge of the historical and existing stormwater drainage problems within the Village. The problems reported at that meeting consisted chiefly of standing water in roadside ditches or backyard swales, perceived excessive operation of sump pumps, and minor flooding of yards. In addition, a few accounts of substantial flooding of roadways and properties were reported. This information was considered in the evaluation of alternative plans; and while improvements recommended to the village stormwater drainage system should help to resolve some of these problems, other problems which affect isolated, individual properties may not be the types of problems that can be fully solved through public drainage system improvements.

Infiltration of groundwater and inflow of stormwater into the sanitary sewers is a stormwater drainage-related problem. Because of the presence of excessive clear water flows in the metropolitan sanitary sewer system, as documented in the report prepared by the Milwaukee Metropolitan Sewerage District (MMSD) entitled Infiltration/Inflow Analysis, January 1979, a sewer system evaluation study was performed throughout the Village of Hales Corners by the MMSD, and the results of that study are set forth in the MMSD report, Sewer System Evaluation Survey, Village of Hales Corners, August 1981. Clear water enters sanitary sewer systems as infiltration or as inflow. The former is defined as water that leaks into a sanitary sewerage system through defective pipes, pipe joints, connections, or manhole walls. The latter is defined as clear water discharged into a sanitary sewerage system from such sources as roof leaders; cellar, yard, and area drains; foundation drains; cooling water discharges; drains from springs and swampy areas; manhole covers; cross connections from storm sewers; and catch basins. Inflow consists of stormwater runoff, street wash waters, and other forms of surface drainage. The evaluation study estimated that infiltration occurs at a maximum rate of 2.34 million gallons per day (mgd) in the village sanitary sewer system; and that inflow occurs at a maximum rate of 5.22 mgd. Of particular concern was the contribution of both inflow and infiltration through foundation drains and sump pumps when groundwater levels are high and during rainfall events. This infiltration and inflow into the sanitary sewer system has resulted in the bypassing of wastewater to prevent surcharging the sanitary sewer, and subsequent basement backups. In 1979 it was estimated that during periods of heavy rainfall, auxiliary-pump bypassing within the Village was required in three problem areas, and that an estimated 1.61 million gallons of wastewater was discharged to the surface waters of the area.

A cost-effective rehabilitation program was recommended by MMSD to eliminate sources of clear water entry from private property. These recommendations include the disconnection of all sump pumps which are presently connected to the sanitary sewer; the encouragement of the disconnection of foundation drains which are presently connected to the sanitary sewer; and the repair of all sanitary lateral defects in the public right-of-way, if such repair was found to be cost-effective. This rehabilitation program would reduce the maximum daily rate of infiltration and inflow in the Hales Corners sanitary sewer by approximately one million gallons. In addition, the construction of one local relief sewer was recommended to eliminate bypassing and auxiliary pumping. The implementation of these recommendations will divert the flows of sump pumps and foundation drains from the sanitary sewers to surface water drainage facilities and may thereby exacerbate ponding and other surface drainage problems. The provision of an efficient stormwater drainage system, however, may be expected to abate any such problems, as well as to further reduce infiltration and inflow problems by removing standing water which now tends to pond in selected areas and in some drainage ditches.

EROSION AND SEDIMENTATION PROBLEMS

Field surveys were conducted by the staffs of the Village of Hales Corners Public Works Department, the Regional Planning Commission, and the U. S. Soil Conservation Service in November 1984 to identify stormwater runoff-related soil erosion and sedimentation problems in the study area. The following types of soil erosion were identified: construction site erosion; erosion of cropland; and erosion of stream banks, gullies, and drainage ditches. Map 12 shows the locations of erosion problems identified within the study area during the field survey. It should be noted that many problems may be of a temporary nature, particularly when associated with construction projects. However, new urban development may be expected to create additional construction sites, and new erosion problems.

The largest construction site observed at the time of the survey, with an areal extent of approximately 35 acres, was located in the southwest one-quarter of U.S. Public Land Survey Section 30, Town 6 North, Range 21 East. The area contained large areas of exposed soils adjacent to the newly constructed roads. Another large area of construction site erosion was located in the southeast one-quarter of Section 30, and is an eastward extension of Woodside Drive. This five-acre parcel of land was poorly vegetated and severe erosion was observed over the entire area. Construction site erosion within the study area was occurring on approximately 42 acres.

The extent and severity of cropland erosion varies with the topography, hydrology, soils, slopes, specific crops grown, and conservation practices used. The field survey rated areas of cropland erosion as slight, moderate, and severe, based on the amount of sediment the crop field may be expected to contribute to surface waters. Row crops, such as corn, are grown on most of these fields. Most of the croplands are located within the City of New Berlin, as shown on Map 12. Approximately 10 acres of cropland were rated as slight erosion sites; about 50 acres of cropland were rated as moderate erosion sites; and about 130 acres of cropland were rated as severe erosion sites. Areas rated as severe cropland erosion sites generally had steep slopes. The total area of cropland identified as exhibiting slight, moderate, or severe erosion was about 190 acres, or about 5.1 percent of the total study area.

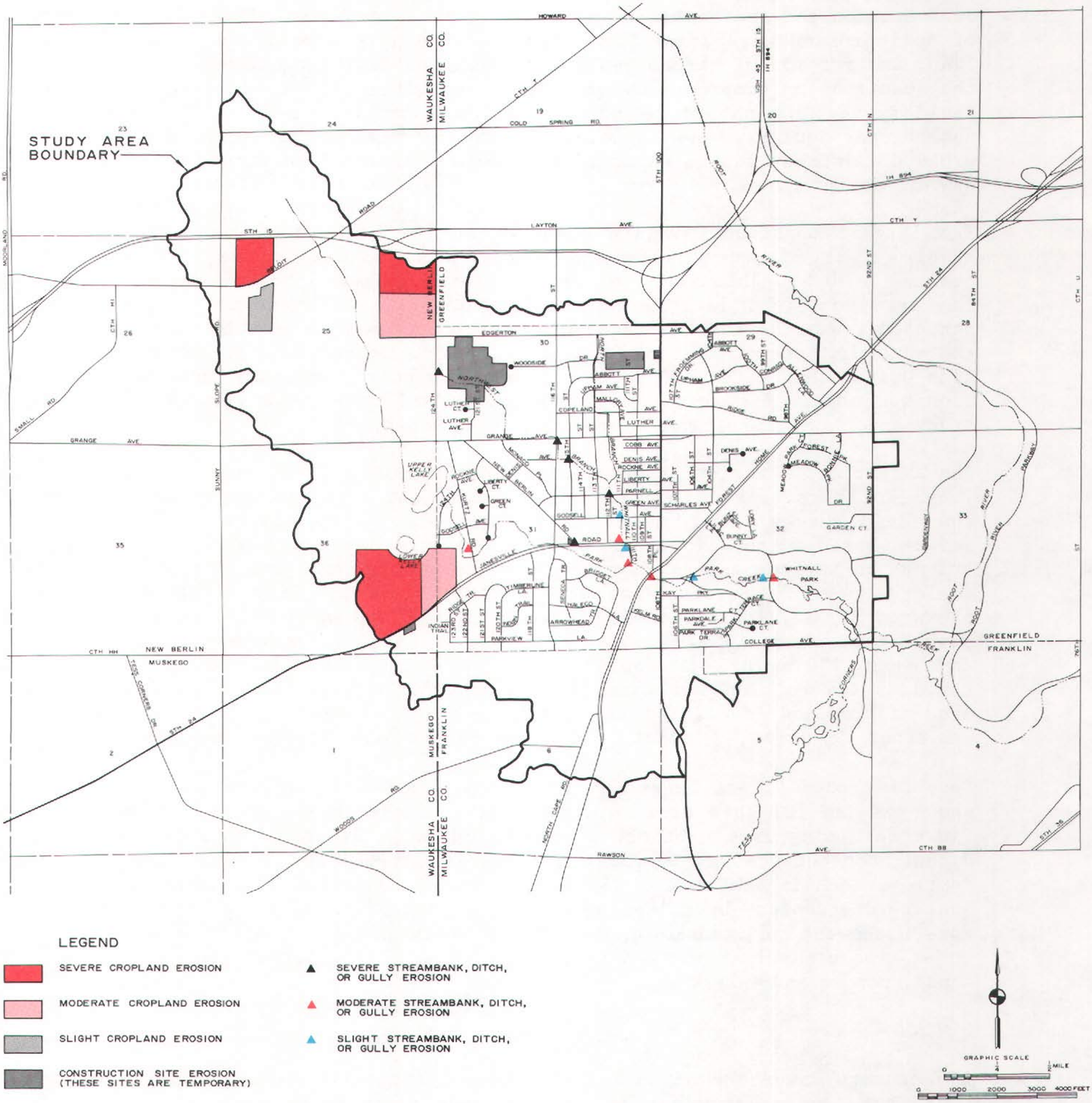
Several eroded stream banks and gullies were observed in the study area, as shown on Map 12. This erosion may be attributable to the increased peak storm flows resulting from urban development; hence, these erosion sites may be a direct consequence of improper stormwater management. Stream bank erosion destroys aquatic habitats at the erosion site, contributes to downstream water quality degradation by releasing sediments to the water, and provides material for subsequent sedimentation downstream which covers valuable benthic habitats, impedes navigation, and fills downstream stormwater storage basins, wetlands, ponds, and lakes.

SUMMARY

An accurate inventory of certain hydrologic-hydraulic characteristics of the study area and related natural and man-made features is an essential step in the stormwater management plan process. Data on the existing stormwater drainage system, stormwater flows, existing drainage and flooding problems, and erosion and sedimentation problems are accordingly presented in this chapter. Also presented are data on land use and land use regulations, climate, soils, hydrology, and water quality.

Map 12

EXISTING EROSION AND SEDIMENTATION PROBLEMS WITHIN THE VILLAGE OF HALES CORNERS STUDY AREA: 1984



Source: Village of Hales Corners Public Works Department, U. S. Soil Conservation Service, and SEWRPC.

Land use characteristics, including impervious area, the type of storm drainage system, the level and characteristics of human activity, and the type and amount of pollutants deposited on the land surface, greatly influence the quantity and quality of stormwater runoff. Urban land uses cover 75 percent of the total study area, and 86 percent of the total area in the Village of Hales Corners. Residential land uses comprise the singularly largest land use category, accounting for 45 percent of the total study area and 61 percent of the developed urban area.

Existing pertinent land use regulations include zoning ordinances and subdivision ordinances for the Village of Hales Corners and Cities of New Berlin, Franklin, Greenfield, and Muskego. These land use regulations, summarized in this chapter, represent important tools for local units of government in directing the use of land in the public interest. Such zoning has important implications for stormwater management.

Climatological factors affecting stormwater management include air temperature and the type and amount of precipitation. Air temperature affects whether precipitation occurs as rainfall or snowfall; whether the ground is frozen and, therefore, essentially impervious; and the rate of snowmelt and attendant runoff. The seasonal nature of precipitation patterns is an important consideration in stormwater drainage. Flooding along Whitnall Park Creek is likely to occur at any time throughout the year except during winter because of its relatively small drainage area and the predominance of developed urban land. The maximum monthly precipitation recorded in the area was 10.03 inches in June 1917 and the maximum 24-hour precipitation was 5.76 inches also recorded in June 1917. The amount of snow cover influences the severity of snowmelt flood events and the extent and depth of frozen soils. The maximum annual snowfall amount, based on a period of record from 1940 through 1980, was 90.8 inches in the winter of 1951-52.

The relationship between rainfall intensity, duration, and frequency is an important element in stormwater management analysis and system design. Rainfall intensity, duration, and frequency relationship equations and curves, based on 64 years of record at Milwaukee, are presented in this chapter. This information permits the estimate of peak flows and annual discharges from stormwater drainage systems.

Soil properties influence the rate and amount of stormwater runoff from land surfaces. About 3,371 acres of the study area, or 90 percent, are covered by soils which generate large or very large amounts of runoff.

The water quality impacts of stormwater management are of increasing concern. High surface runoff and erosion can result in high pollutant concentrations in surface waters, reducing the suitability of the waters for recreational use and limiting the ability of the water to support desired forms of fish and other aquatic life. Prior to 1981, when the Village of Hales Corners sewage treatment plant was in operation, levels of fecal coliform, ammonia nitrogen, and phosphorus exceeded Commission-recommended standards; however, dissolved oxygen concentrations and temperature levels were generally suitable to support desirable forms of fish and other aquatic life. With the abandonment of the village sewage treatment plant and abatement of separate sewer overflows, the remaining sources of water pollution in the planning area are all nonpoint. The abatement of these remaining sources requires careful consideration in any stormwater management planning effort.

For planning purposes, the study area was divided into 216 drainage subbasins. These subbasins range in size from about 1 to 83 acres, with an average size of 17 acres. These subbasins are drained by a total of 0.52 mile of perennial streams and 6.44 miles of intermittent streams.

The existing Village of Hales Corners storm sewer system--which actually consists of 15 individual subsystems--serves a combined drainage area of about 885 acres, or about 24 percent of the study area, of which 629 acres lie within the Village of Hales Corners. The remaining 256 acres lie within the City of Franklin but discharge into the Village of Hales Corners storm sewer system. The systems consist of a combined total of approximately 33,711 lineal feet of sewers which range in diameter from 12 inches to 66 inches. There is a combined total of 257 inlets and catch basins and 15 outfalls in the 15 storm sewer subsystems. Nine of the outfalls discharge to the main stem of Whitnall Park Creek, two discharge to the Northwest Branch of Whitnall Park Creek, two discharge to the North Branch of Whitnall Park Creek, one discharges to an intermittent tributary of the Root River, and one discharges directly to the Root River.

Bridges and culverts significantly influence the hydraulic behavior of a stream system. Whitnall Park Creek is crossed by bridges and culverts 33 times, of which 18 crossings were determined to be hydraulically significant. The Northwest Branch of Whitnall Park Creek is crossed 17 times, 10 times by structures determined to be hydraulically significant; and the North Branch of Whitnall Park Creek is crossed 10 times, 9 times by structures determined to be hydraulically significant.

Existing stormwater management problems consist of drainage problems and flood control problems. Most drainage problems within the study area are related to high groundwater levels which require excessive sump pump operation and which contribute to the ponding of stormwater in ditches and low areas. These drainage problems are aggravated by the existence of drainage ditches with insufficient slopes and conveyance capacities.

A field survey was conducted by the staffs of the Village of Hales Corners, the Regional Planning Commission, and the U. S. Soil Conservation Service in November 1984 to identify stormwater runoff-related erosion and sedimentation problems in the study area. The survey identified construction site erosion, cropland erosion, and stream bank and gully erosion as existing problems. Approximately 42 acres of land under construction contained large areas of exposed soils, contributing to severe erosion. About 190 acres of cropland, or about 5 percent of the study area, were identified as slight, moderate, or severe erosion sites. Several eroded stream banks and gullies observed in the study area were caused by high runoff rates generated by urban development.

Chapter IV

ANTICIPATED GROWTH AND CHANGE

INTRODUCTION

The Village of Hales Corners stormwater management plan is intended to identify the stormwater management needs of the Village of Hales Corners through the year 2000 and to propose the best means of meeting those needs. In the case of the Village, the year 2000 land use pattern may be considered to represent an ultimate development pattern, barring any significant changes in community development objectives and attendant major redevelopment. Accordingly, the system plan should serve the Village as an effective guide to storm management system development well beyond the design year. Land uses in the study area markedly influence the stormwater runoff process. The 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 runoff for a given rainfall event and decrease the time of runoff. Unless special stormwater management measures are taken, the typical net 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 different types and increased amounts of pollutants as compared to runoff from rural lands. Land use--and probable changes in such use over time--affects the stormwater runoff process, and therefore existing and probable future changes in the loadings on the stormwater management system, and that system must serve to support existing, and promote desirable, land use development in the planning area. Therefore, consideration of both the probable future and existing land use pattern of an area is necessary for the effective development of alternative stormwater management plans and for the selection of a recommended plan.

It should be noted that the land use changes occurring within the Village are, in part, the result of an aggressive village development program. This program includes the establishment of a tax incremental finance district to fund and support through public infrastructure development desired land use development and redevelopment; the imaginative reuse of remnant parcels of land; and the provision by the Village of services encouraging development. This village development program gives impetus to the need to develop a stormwater management system plan.

Accordingly, this chapter presents information on the anticipated type, density, and spatial distribution of land uses in the stormwater management study area, and on the impact of the anticipated changes in land use on the stormwater management needs of the study area.

LAND USE

As already noted, probable future, as well as existing, land use must be considered in any sound stormwater management planning effort. Accordingly, a design year 2000 land use pattern was developed for the stormwater management planning area. This pattern was based on the adopted year 2000 regional land use plan; the development objectives implied by the existing zoning ordinance of the Village of Hales Corners and of the other municipalities comprising the study; area discussions with officials of the Village of Hales Corners to identify development opportunities and constraints; and preliminary work completed on a land use plan for the City of New Berlin. Existing land uses and land use regulations governing development and redevelopment in the planning area are discussed in Chapter III of this report.

Probable future land use patterns are presented herein for two different geographic areas. First, a future land use pattern is presented for the Village of Hales Corners. Land use changes in this area are clearly of direct concern in the stormwater management planning effort. In addition, the probable future land use pattern in the drainage area upstream of, and tributary to, the natural surface water drainage channels within the Village must be considered in plan development. Therefore, a probable future land use pattern in that tributary drainage area is also presented.

The total area contained within the corporate limits of the Village of Hales Corners in 1980 was 2,073 acres, or about 3.2 square miles. The existing 1980 and design year 2000 areas associated with each of the various land uses in the Village are set forth in Table 18. The year 2000 land use pattern is shown on Map 13. As indicated in the table, about 161 acres of rural land, or about 8 percent of the total area of the Village, may be expected to be converted from rural to urban uses over the approximately 20-year plan design period. This conversion would increase the amount of land in urban use within the Village by about 10 percent. Of the total area to be converted, about 102 acres, or 63 percent, would be converted to residential use; about 13 acres, or 8 percent, to commercial use; and about 47 acres, or 29 percent, to other urban uses. Industrial use is expected to decrease by about one acre by the year 2000, through the anticipated conversion to commercial use of a one-acre parcel of industrial land located in the southwest corner of the intersection of STH 100 and W. Forest Home Avenue.

As indicated in Table 18, under year 2000 conditions, urban land uses would account for about 1,831 acres, or about 88 percent of the total area of the Village. Of these developed urban lands, residential uses would occupy about 1,147 acres, or about 63 percent; while the remaining urban land uses--commercial, industrial, transportation and utilities, governmental and institutional, and recreational--would occupy 684 acres, or the remaining 37 percent. Under year 2000 conditions, rural land uses would still be expected to account for about 242 acres, or about 12 percent of the total area of the Village. Woodlands would occupy about 110 acres of that total, or about 45 percent; agricultural and other open lands about 55 acres, or about 23 percent; and other rural land uses, including wetlands and open water, about 77 acres, or about 32 percent.

The entire stormwater management study area encompasses about 3,726 acres, or about 5.8 square miles. The existing 1980 and design year 2000 areas of land associated with each of the various land uses within the study area are set forth in Table 19. The year 2000 land uses within the study area are shown on Map 14. As indicated in the table, about 689 acres of rural land, or about 18 percent of the total study area, may be expected to be converted from rural to

Table 18

EXISTING AND PROBABLE FUTURE LAND USE IN THE VILLAGE OF HALES CORNERS: 1980 AND 2000

Land Use Category	Existing 1980		Planned Increment		Total 2000	
	Acres	Percent of Major Category	Acres	Percent Change	Acres	Percent of Major Category
Urban						
Residential.........	1,045	62.6	102	9.8	1,147	62.6
Commercial..........	60	3.6	13	21.7	73	4.0
Industrial..........	4	0.2	-1	-25.0	3	0.2
Governmental and Institutional......	50	3.0	1	2.0	51	2.8
Transportation and Utilities......	381	22.8	44	11.5	425	23.2
Recreational........	130	7.8	2	1.5	132	7.2
Urban Subtotal	1,670	100.0	161	9.6	1,831	100.0
Rural						
Woodlands...........	111	27.5	-1	-0.9	110	45.5
Wetlands............	81	20.1	-10	-12.3	71	29.3
Surface Water.......	6	1.5	--	--	6	2.5
Agricultural and Other Open Lands...	205	50.9	-150	-73.2	55	22.7
Rural Subtotal	403	100.0	-161	-40.0	242	100.0
Total	2,073	--	--	--	2,073	--

Source: SEWRPC.

Table 19

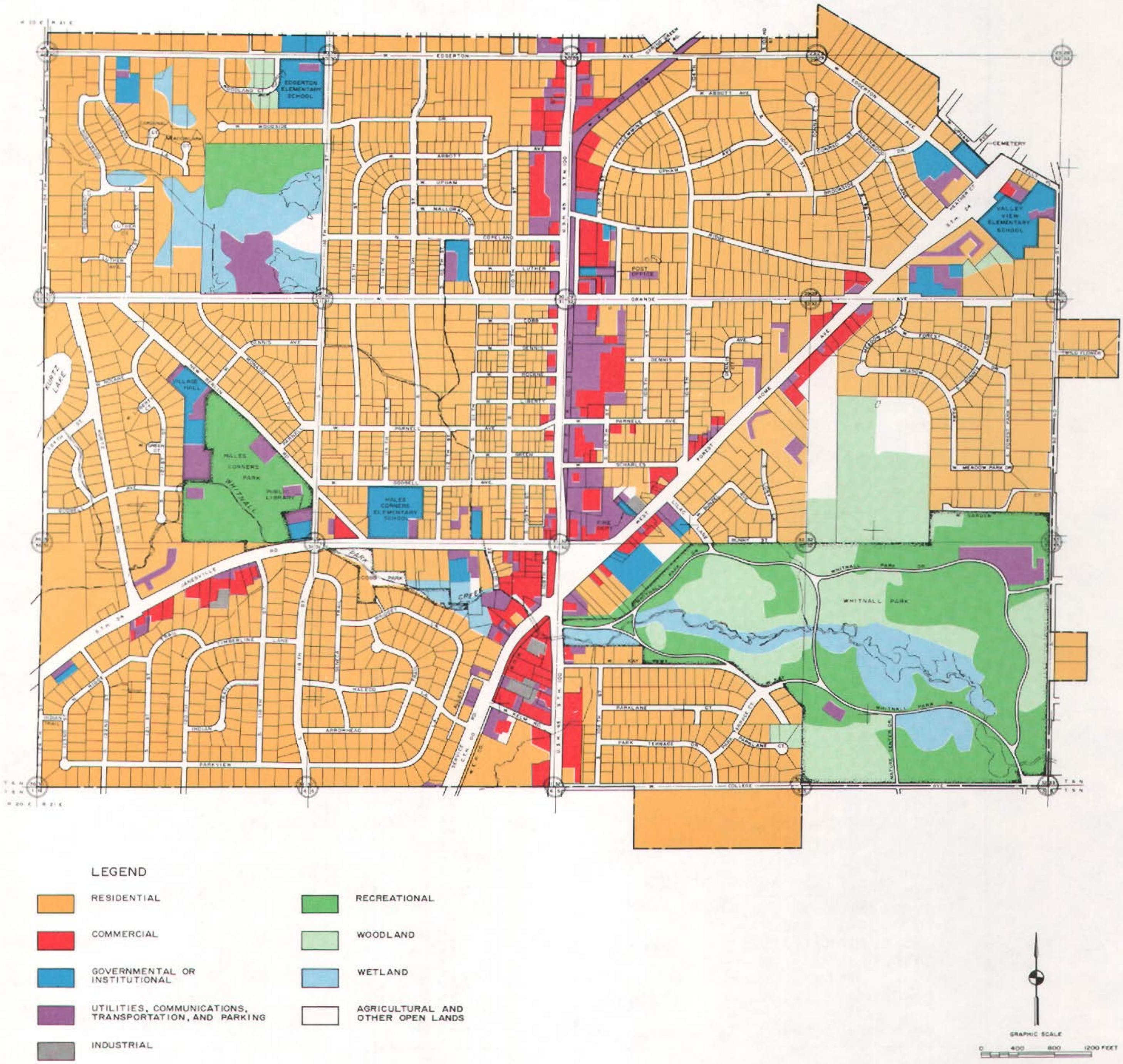
EXISTING AND PROBABLE FUTURE LAND USE IN THE VILLAGE OF HALES CORNERS STORMWATER MANAGEMENT STUDY AREA: 1980 AND 2000

Land Use Category	Existing 1980		Planned Increment		Total 2000	
	Acres	Percent of Major Category	Acres	Percent Change	Acres	Percent of Major Category
Urban						
Residential.........	1,701	64.2	444	26.1	2,145	64.2
Commercial..........	81	3.1	53	65.4	134	4.0
Industrial..........	19	0.7	16	84.2	35	1.1
Governmental and Institutional......	82	3.1	1	1.2	83	2.5
Transportation and Utilities......	621	23.4	171	27.5	792	23.7
Recreational........	147	5.5	4	2.7	151	4.5
Urban Subtotal	2,651	100.0	689	26.0	3,340	100.0
Rural						
Woodlands...........	176	16.4	-1	-0.6	175	45.3
Wetlands............	142	13.2	-10	-7.0	132	34.2
Surface Water.......	24	2.2	--	--	24	6.2
Agricultural and Other Open Lands...	733	68.2	-678	-92.5	55	14.3
Rural Subtotal	1,075	100.0	-689	-64.1	386	100.0
Total	3,726	--	--	--	3,726	--

Source: SEWRPC.

Map 13

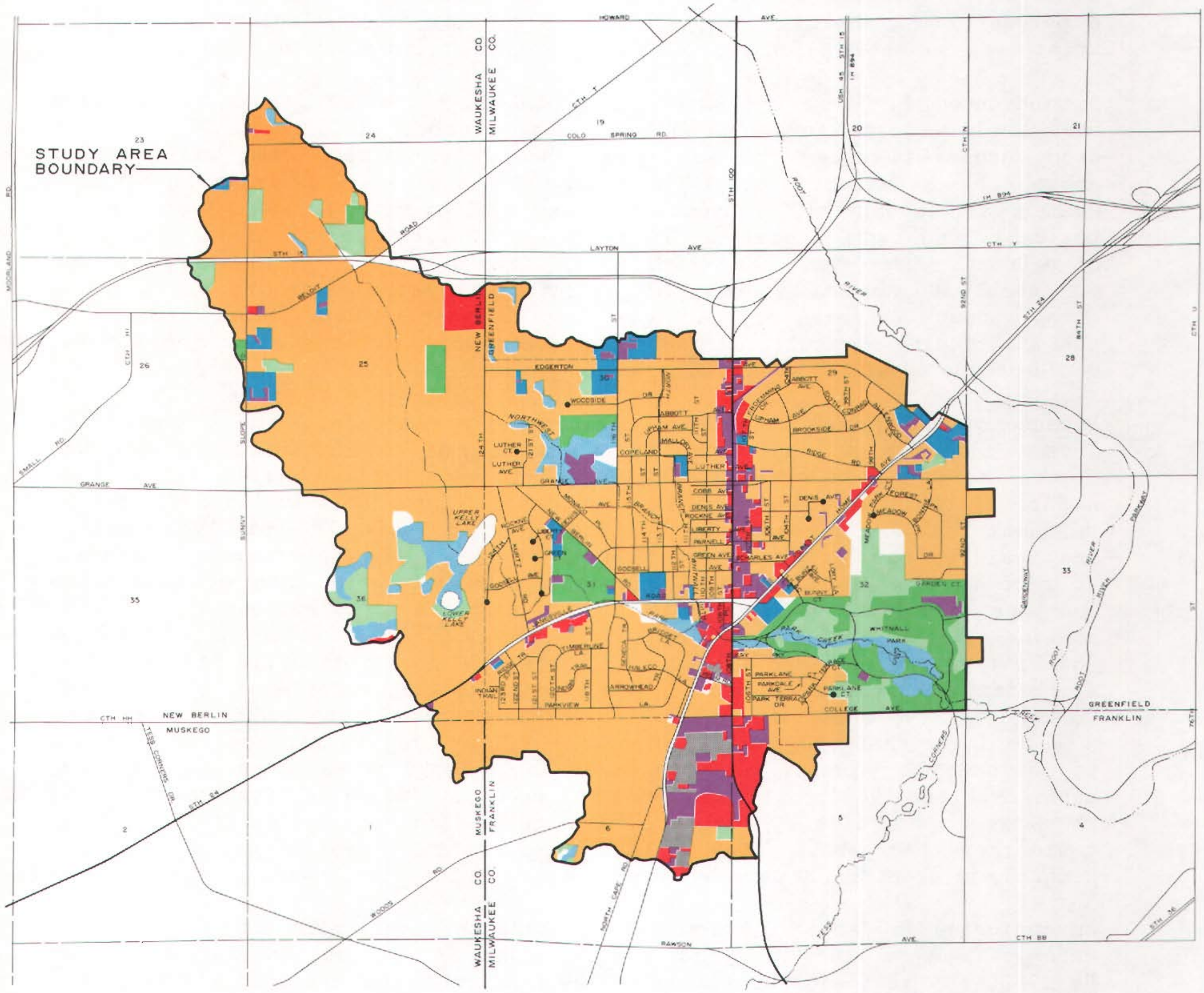
PROBABLE FUTURE LAND USE PATTERN FOR THE VILLAGE OF HALES CORNERS: 2000



Source: SEWRPC.

Map 14

PROBABLE FUTURE LAND USE PATTERN FOR THE STORMWATER MANAGEMENT STUDY AREA



Source: SEWRPC.

urban uses over the approximately 20-year plan design period. This conversion would increase the amount of land in urban use within the study area by about 26 percent. Of the total area to be converted, about 444 acres, or about 64 percent, would be converted to residential use; about 53 acres, or about 8 percent, to commercial use; about 16 acres, or about 2 percent, to industrial use; and about 176 acres, or about 26 percent, to other urban uses.

As indicated in Table 19, under year 2000 land use conditions, urban land uses would account for about 3,340 acres, or about 90 percent of the total study area. Of these developed urban lands, residential uses would occupy about 2,145 acres, or about 64 percent; while the remaining urban land uses--commercial, industrial, transportation and utilities, governmental and institutional, and recreational--would occupy about 1,195 acres, or the remaining 36 percent. Under year 2000 conditions, rural land uses would still account for about 386 acres, or about 10 percent of the study area. Woodlands would occupy about 175 acres of that total, or about 45 percent. Other rural land uses, including agricultural and other open lands, wetlands, and open waters, would occupy about 211 acres, or about 55 percent.

Because of the direct relationships which exist between resident population levels and land use patterns, an evaluation of the historic and probable future resident population levels in the Village of Hales Corners was made as a part of the stormwater management planning effort. This evaluation was used to check the land use analyses. As indicated in Table 20, from 1960 to 1970 the resident population of the Village of Hales Corners increased by about 40 percent, to 7,771 persons. This was a much higher rate of population increase than experienced by either Milwaukee County or the Southeastern Wisconsin Region over the same time period. By 1980, the resident population of the Village had declined slightly from the 1970 level, to 7,110 persons. This approximately 8 percent reduction was similar to that experienced by Milwaukee County over the same time period. Forecasts of population growth to the year 2000 indicate that the population of the Village may be expected to increase to about 8,500 persons, an increase of about 1,390 persons, or about 20 percent, over the 1980 population level. Alternative future analyses conducted by the Commission indicated, however, that the resident population of the Village could range from about 7,000 persons under the most pessimistic future considered, to about 9,100 persons under the most optimistic future.

As indicated in Table 20, from 1963 to 1970 the population within the stormwater management study area increased by about 36 percent, to 11,350 persons. By 1980, the resident population of the study area had declined slightly from the 1970 level, to 10,800 persons. Forecasts of population growth to the year 2000 indicate that the population of the study area may be expected to increase to about 14,700, an increase of about 3,900 persons, or about 36 percent, over the 1980 population level. Alternative future analyses conducted by the Commission indicate, however, that the resident population of the study area could range from about 10,600 persons under the most pessimistic future considered to about 16,450 persons under the most optimistic future. A graphic comparison of historical, existing, and forecast population levels for the Village of Hales Corners, the stormwater management study area, Milwaukee County, and the Southeastern Wisconsin Region is set forth in Figure 3. The anticipated increase in population within the Village, as well as within the entire stormwater management study area, can readily be accommodated by the increase in residential lands anticipated within the Village and study area over the 1980-2000 time period.

Table 20

HISTORIC AND PROBABLE FUTURE RESIDENT POPULATION LEVELS FOR THE SOUTHEASTERN WISCONSIN REGION, MILWAUKEE COUNTY, AND THE VILLAGE OF HALES CORNERS

Year	Southeastern Wisconsin Region		Milwaukee County		Village of Hales Corners		Stormwater Management Study Area	
	Population	Percent Change	Population	Percent Change	Population	Percent Change	Population	Percent Change
1900	501,808	--	330,017	--	--	--	--	--
1910	631,161	25.8	433,187	31.3	--	--	--	--
1920	783,681	24.2	539,449	24.5	--	--	--	--
1930	1,006,118	28.4	725,263	34.4	--	--	--	--
1940	1,067,699	6.1	766,885	5.7	--	--	--	--
1950	1,240,618	16.2	871,047	13.6	--	--	--	--
1960	1,573,614	26.8	1,036,041	18.9	5,549[a]	--	8,320[b]	--
1970	1,756,083	11.6	1,054,249	1.8	7,771	40.0	11,344	36.3
1980	1,764,919	0.5	964,988	-8.5	7,110	-8.5	10,805	-4.8
2000	2,219,300	25.7	1,049,600	8.8	8,500	19.5	14,700	36.0

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

[b]Represents 1963 population levels as determined under the 1963 SEWRPC origin-destination travel survey.

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

Within the Village of Hales Corners, the forecast year 2000 population level of 8,500 persons--assuming a household size of 2.7 persons per housing unit--would result in the need for approximately 3,150 housing units. Such housing units, if uniformly distributed over the 1,147 acres of residential land anticipated to be within the Village by the year 2000, would result in a density of approximately 2.8 housing units per net residential acre. Alternative futures population levels for the Village--ranging from a low of about 7,000 persons to a high of about 9,100 persons--however, would result in a housing unit density of from 2.3 to 2.9 housing units per net residential acre.

Within the entire stormwater management study area, the forecast year 2000 population level of 14,700 persons--assuming a household size of 2.8 persons per housing unit--would result in the need for approximately 5,250 housing units. Such housing units, if uniformly distributed over the 2,145 acres of residential land anticipated to be within the study area by the year 2000, would result in a density of approximately 2.5 housing units per net residential acre. Alternative futures population levels for the study area--ranging from a low of about 10,600 persons to a high of about 16,500 persons--however, would result in a housing unit density of from 1.8 to 2.7 housing units per net residential acre.

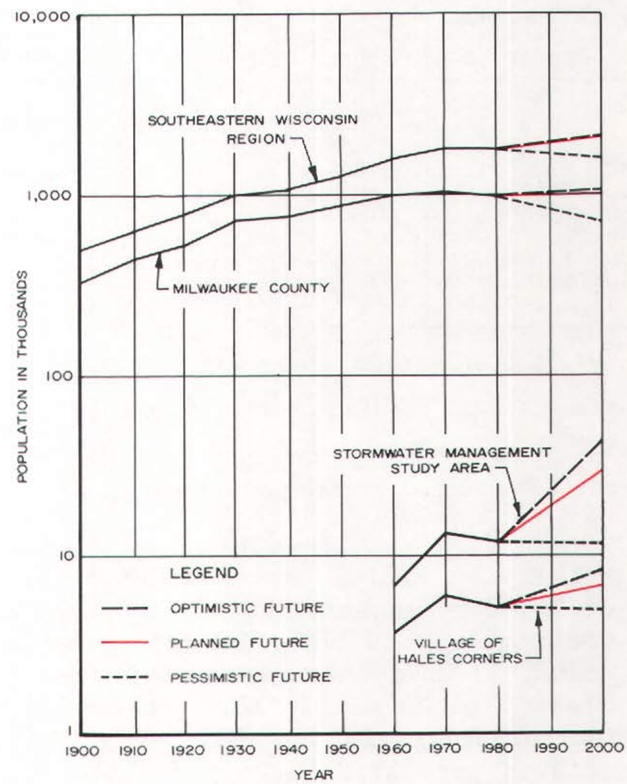
IMPACT OF CHANGED LAND USE ON STUDY AREA STORMWATER MANAGEMENT SYSTEMS

The conversion of 689 acres of rural land within the study area to urban uses would result in about 3,340 acres, or about 90 percent of the study area, being devoted to urban land uses by the year 2000. This compares to the 2,651 acres, or 70 percent of the study area, in urban land use under existing 1980 conditions and, as already noted, indicates an increase of approximately 26 percent in the amount of land in urban use. This change in land use will have a direct impact upon the quality, amount, and rate of stormwater runoff.

The combined land use and cover of an area is probably the single characteristic which best indicates the influence of urban development on the hydrologic processes. In an area like southeastern Wisconsin, both land use and land cover are largely the result of human activities. Land cover differs from land use in that it describes the types of surface--for example, roofed, paved, grassed, or wooded--whereas land use describes the function or activity served by the land--for example, residential, commercial, or recreational. The combination of land use and cover is an important determinant of the stormwater runoff characteristics of an area, and, as such, is used in the quantification of loadings on, and in the design of, stormwater management systems. Table 21 lists the imperviousness ranges defined for various land use and land cover conditions.

Figure 3

COMPARISON OF HISTORICAL, EXISTING, AND FORECAST POPULATION TRENDS FOR THE VILLAGE OF HALES CORNERS, THE STORMWATER MANAGEMENT STUDY AREA, MILWAUKEE COUNTY, AND THE SOUTHEASTERN WISCONSIN REGION



Source: SEWRPC.

The percent of impervious surface in a given area is an important factor in determining both the amount of stormwater runoff and the rate at which stormwater runoff is generated. More than 65 percent of the total area of industrial and commercial areas may be impervious surface, while from 10 to 65 percent of the total area of residential areas may be impervious surface , depending upon the density of the development. Generally, less than 10 percent of the total area of rural areas is impervious surface. The impact of the planned changes in land use on the volume and rate of stormwater runoff from each of the drainage subbasins established for this study is set forth in Chapter VII, which discusses the results of the stormwater drainage system hydrologic-hydraulic simulation modeling work.

Another important consideration in the stormwater management planning effort was the increased urban area within the Village, which must be provided with urban stormwater drainage facilities. As shown in Table 18, about 102 acres of new residential land, and about 60 acres of new commercial, governmental, institutional, and transportation lands will have to be accommodated. In addition to the facilities needed to serve these new urban land uses, including stormwater management facilities, the planning effort considered the degree of rehabilitation needed to properly maintain, improve, or extend the existing stormwater management system serving the 1,670 acres of already developed lands in the Village of Hales Corners.

Table 21

RANGE OF SURFACE IMPERVIOUSNESS FOR LAND USE AND LAND COVER CONDITIONS

Description	Range of Percent Imperviousness	Typical Corresponding Land Use/Cover Combinations
Rural..................	0-8	Agricultural lands, woodlands, wetlands, and unused lands
Low Imperviousness......	9-20	Low-density residential with supporting urban uses and associated land cover
Low to Medium Imperviousness........	21-33	Low- to medium-density residential with supporting urban uses and associated land cover
Medium Imperviousness...	34-45	Medium-density residential with supporting urban uses and associated land cover
High Imperviousness.....	46-65	High-density residential with supporting urban uses and associated land cover
Very High Imperviousness........	66-100	Commercial and industrial and associated land cover

Source: SEWRPC.

SUMMARY

The existing and probable future land use patterns of the stormwater management study area directly influence stormwater management needs. Thus, consideration of probable future land use conditions, as well as of existing conditions, is necessary for the sound development of alternative stormwater management plans, and for the selection of a recommended plan. Accordingly, this chapter presents information on the anticipated type, extent, and distribution of land uses in the Village of Hales Corners and in the study area for the plan design year 2000.

Urban land uses within the Village of Hales Corners are expected to increase from a total of 1,670 acres in 1980 to about 1,831 acres in the year 2000, about a 10 percent increase. Urban land uses are expected to occupy about 88 percent of the total area of the Village by the plan design year 2000, as opposed to about 81 percent in 1980. The residential land use category is expected to experience the largest absolute increase--about 102 acres--to a total in the plan design year of about 1,147 acres. Within the study area, urban land uses are expected to increase from a total of about 2,651 acres in 1980 to about 3,340 acres in the year 2000, about a 26 percent increase. Urban land uses are thus expected to occupy about 90 percent of the total study area by the design year 2000, as opposed to about 71 percent in 1980.

Attendant to this increase in urban land use is an anticipated increase in the resident population of the Village and the study area. The 1980 resident population of the Village of Hales Corners of 7,110 persons is expected to increase to about 8,500 persons by the year 2000. However, the population could range from a low of 7,050 persons to a high of 9,060 persons under alternative futures.

The 1980 resident population of the stormwater management study area of 10,805 persons is expected to increase to about 14,700 persons by the year 2000. However, the population could range from a low of 10,600 persons to a high of 16,450 persons under alternative futures. The anticipated increase in population within the Village, as well as within the entire stormwater management study area, can readily be accommodated by the increase in residential land anticipated within the Village and study area over the 1980-2000 time period.

The anticipated change in land use will directly impact the amount--particularly the rate--and quality of stormwater runoff. Increased rates of runoff result from the higher proportion of impervious areas--such as streets, parking lots, and rooftops. Impervious surfaces generally cover from 30 to more than 65 percent of urban areas, compared to less than 10 percent of rural areas. In addition to having impacts on stormwater quantity due to the creation of impervious areas, stormwater drainage systems constructed to serve urban areas are generally more efficient than the natural systems, and convey the runoff to the receiving watercourse more efficiently. Thus, urban stormwater drainage system development can increase flood flows and stages in downstream areas. Such system development can also increase the downstream surface-water pollutant loadings. Therefore, careful planning of such systems to meet sound water resource and related management objectives is essential.

Chapter V

STORMWATER MANAGEMENT OBJECTIVES, STANDARDS, AND DESIGN CRITERIA

INTRODUCTION

Planning may be defined as a rational process for formulating and meeting objectives. Consequently, the formulation of objectives is an essential task which must be undertaken before plans can be prepared. Accordingly, this chapter sets forth a set of stormwater management objectives and supporting standards for use in the design and evaluation of alternative stormwater management system plans for the Village of Hales Corners and environs, and in the selection of a recommended plan from among those alternatives.

In addition, this chapter sets forth certain engineering design criteria and describes certain analytical procedures which were used in the preparation and evaluation of the alternative stormwater management system plans. These criteria and procedures include the engineering techniques used to design the alternative plan elements, test the physical feasibility of those elements, and make necessary economic comparisons between the plan elements. This chapter thus documents the degree of detail and level of sophistication employed in the preparation of the recommended stormwater management plan, and thereby provides a better understanding by all concerned of the plan and of the need for refinement of some aspects of the plan prior to and during implementation.

STORMWATER MANAGEMENT OBJECTIVES AND STANDARDS

The following five stormwater management objectives were formulated to guide the design, test, and evaluation of alternative stormwater management plans for the Hales Corners stormwater management planning area and the selection of a recommended plan from among the alternatives considered:

1. The development of a stormwater management system which reduces the exposure of people to drainage-related inconvenience and to health and safety hazards, and which reduces the exposure of real and personal property to damage through inadequate stormwater drainage and inundation.

2. The development of a stormwater management system which will effectively serve existing and proposed future land uses.

3. The development of a stormwater management system which will minimize soil erosion, sedimentation, and attendant water pollution.

4. The development of a stormwater management system which will be flexible and readily adaptable to changing needs.

5. The development of a stormwater management system which will efficiently and effectively meet all of the other stated objectives at the lowest practicable cost.

Complementing each of the foregoing stormwater management system development objectives is a set of quantifiable standards which can be used to evaluate the relative or absolute ability of alternative stormwater management plan designs to meet the objective. These standards are set forth in Table 22. The planning standards 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. The absolute standards can be applied individually to each alternative plan proposal since they are expressed in terms of maximum, minimum, or desirable values.

OVERRIDING CONSIDERATIONS

In the application of the stormwater management development objectives and standards in the preparation, test, and evaluation of stormwater management system plans, several overriding considerations must be recognized. First, it must be recognized that any proposed stormwater management facilities must constitute integral parts of a total system. It is not possible from an application of the standards alone, however, to assure such system integration, since the standards cannot be used to determine the effect of individual facilities on the system as a whole, nor on the environment within which the system must operate. This requires the application of planning and engineering techniques developed for this purpose which can be used to quantitatively test the potential performance of proposed facilities as part of a total system. The use of mathematical simulation models facilitates such quantitative tests. Furthermore, by using these models, the configuration and capacity of the system can be adjusted to the existing and future runoff loadings. Second, it must be recognized that it is unlikely that any one plan proposal will fully meet all of the standards; and the extent to which each standard is met, exceeded, or violated must serve as the measure of the ability of each alternative plan proposal to achieve the objective which the given standard complements. Third, it must be recognized that certain objectives and standards may be in conflict and require resolution through compromise, such compromise being an essential part of any design effort.

ENGINEERING DESIGN CRITERIA AND ANALYTICAL PROCEDURES

Introduction

Certain engineering criteria and procedures were used in designing alternative stormwater management plan elements, and in making the necessary economic evaluations. While these criteria and procedures are widely accepted and firmly based in current engineering practice, it is, nevertheless, useful to briefly document them here. The criteria and procedures provide the means for quantitatively sizing and analyzing the performance of both the minor and major components of the total stormwater management system components considered in this stormwater management plan. In addition, these criteria and procedures can serve as a basis for the more detailed design of stormwater management system components which are related directly to the stormwater management facilities. These criteria and procedures thus constitute a reference for use in facility design, and as such are intended to be applied uniformly and consistently in all phases of the implementation of the stormwater management plan.

Table 22

STORMWATER MANAGEMENT OBJECTIVES AND STANDARDS
FOR THE VILLAGE OF HALES CORNERS

OBJECTIVE NO. 1

The development of a stormwater management system which reduces the exposure of people to drainage-related inconvenience and to health and safety hazards and which reduces the exposure of real and personal property to damage through inadequate stormwater drainage and inundation.

STANDARDS

1. In order to prevent significant property damage and safety hazards, the major components of the stormwater management system should be designed to accommodate runoff from a 100-year recurrence interval storm event.

2. In order to provide for an acceptable level of access to property and of traffic service, the minor components of the stormwater management system should be designed to accommodate runoff from a 10-year recurrence interval storm event.

3. In order to provide an acceptable level of access to property and of traffic service, the stormwater management system should be designed to provide two clear 10-foot lanes for moving traffic on arterial streets, and one clear 10-foot lane for moving traffic on collector and land access streets during storm events up to and including the 10-year recurrence interval event.

4. Curbs and gutters, inlets, sewers, roadside swales and culverts, and other elements of the minor stormwater drainage system should be located and sized so as to preclude the flow of stormwater along and across the pavements of arterial, collector, and land access streets during storm events up to and including the 10-year recurrence interval event.

5. Uncontrolled flow of stormwater along and across the full pavement width of collector and land access streets shall be acceptable; and controlled flow of stormwater along and across arterial streets shall be acceptable during storm events exceeding a 10-year recurrence interval when the streets are intended to constitute integral parts of the major stormwater drainage system. The degree of flow control shall be determined by the importance of the arterial as a traffic carrier.

OBJECTIVE NO. 2

The development of a stormwater management system which will effectively serve existing and proposed future land uses.

STANDARDS

1. Stormwater drainage systems should be designed assuming that the layout of collector and land access streets for all proposed urban development and redevelopment will be carefully adjusted to the topography in order to minimize grading and drainage problems, to utilize to the fullest extent practicable the natural drainage and storage capabilities of the site, and to provide the most economical installation of a gravity flow system. Generally, drainage systems should be designed to complement a street layout wherein collector streets follow valley lines and land access streets cross contour lines at right angles.

2. Stormwater drainage systems should be designed assuming that the layouts and grades of collector and land access streets can, during major storm events, serve as open runoff channels supplementary to the minor stormwater drainage system without flooding adjoining building sites. The stormwater drainage system design should avoid midblock sags in street grades, and street grades should generally parallel swale, channel, and storm sewer gradients.

3. Stormwater management systems shall utilize urban street cross-sections with curbs and gutters, inlets, and storm sewers in all areas identified in the system plan for the use of such sections. Stormwater management systems in all other areas of the Village shall utilize rural street cross-sections with roadside swales and culverts.

4. The stormwater management system shall be designed to minimize the creation of new drainage or flooding problems, or the intensification of existing problems, at both upstream and downstream locations.

OBJECTIVE NO. 3

The development of a stormwater management system which will minimize soil erosion, sedimentation, and attendant water pollution.

STANDARDS

1. Flow velocities which cause stream bank erosion and channel sediment scouring should be avoided.

2. Storm sewer outfalls should be located and designed so as to prevent stream bank erosion and channel sediment scouring.

3. Nonpoint source water pollution abatement measures such as stream bank protection and stormwater storage basins should be incorporated, wherever appropriate, into the stormwater management system.

OBJECTIVE NO. 4

The development of a stormwater management system which will be flexible and readily adaptable to changing needs.

STANDARDS

1. Larger, less frequent storm events should be used to design and size those site-specific elements of the stormwater drainage system for which it would not be economically feasible to provide flow relief and repairs during and following a major storm event.

2. Larger, less frequent storm events should be used to design and size special structures, such as roadway underpasses, requiring pumping stations.

3. Street elevations and grades, and appurtenant site elevations and grades, shall be set to provide overland, gravity drainage to natural watercourses so that positive drainage may be effected in the event of failure of piped stormwater drainage facilities.

OBJECTIVE NO. 5

The development of a stormwater management system which will efficiently and effectively meet all of the other stated objectives at the lowest practicable cost.

STANDARDS

1. The sum of stormwater management system capital investment and operation and maintenance costs should be minimized.

2. Maximum feasible use should be made of all existing stormwater management components, as well as the natural storm drainage system. The latter should be supplemented with engineered facilities only as necessary to serve the anticipated stormwater management needs generated by existing and proposed land use development and redevelopment.

3. Stormwater management facilities should be designed for staged, or phased, construction so as to limit the required investment in such facilities at any one time and to permit maximum flexibility to accommodate changes in urban development, in economic activity growth, in the objectives or standards, or in the technology of stormwater management.

4. To the maximum extent practicable, the location and alignment of new storm sewers and engineered channels and storage facilities should coincide with existing public rights-of-way to minimize land acquisition or easement costs.

5. Stormwater storage facilities--consisting of retention facilities and of both centralized and onsite detention facilities--should, where hydraulically feasible and economically sound, be considered as a means of reducing the size and resultant costs of the required stormwater conveyance facilities immediately downstream of these storage sites.

System Components and Associated Analytic Procedures

There are two distinct drainage systems to be considered in the development of a stormwater management plan for the Village of Hales Corners: 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, generally up to the 10-year recurrence interval storm event. 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.

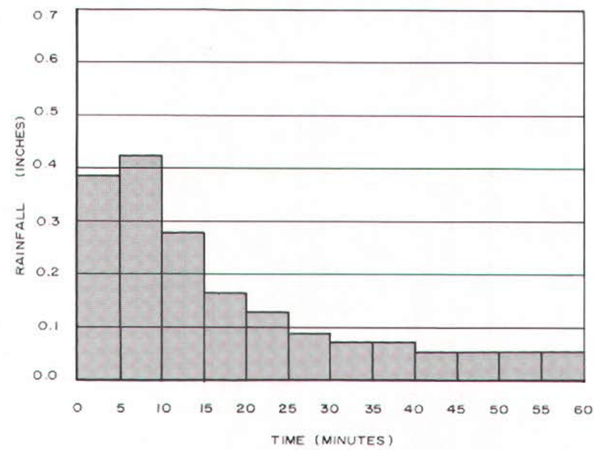
The major stormwater drainage system is designed for conveyance of stormwater runoff during major storm events--that is, generally, for storms exceeding the 10-year recurrence interval--when the capacity of the minor system is exceeded. The major stormwater drainage system consists of the entire street 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.

Three different procedures were used to analyze flows in, and design system components of, the minor stormwater drainage system. The first procedure involved the application of a mathematical simulation model known as the Illinois Urban Drainage Area Simulator (ILLUDAS). This model uses discrete rainfall patterns for the selected recurrence interval design storms. The rainfall patterns used for the 10- and 100-year recurrence interval storms are shown in Figures 4 and 5. In the application of this method, the study area is divided into catchment areas, and hydrographs are produced for the pervious and impervious portions of each catchment area by applying the rainfall pattern or hyetograph to the contributing areas. These hydrographs are combined and routed downstream from one critical location in the system to the next to provide system loadings in the form of peak flow rates and total flow volumes. This model was used in both of its two operational modes, the evaluation mode and the design mode. In the evaluation mode, the model routes hydrographs through a specified drainage system and is used to calculate needed hydraulic capacity at each critical location in the system. In this mode of operation, undersized components can be identified, and the effects of detention storage on peak flow rates and, therefore, on required hydraulic capacities can be analyzed. In the design mode the model may be used to calculate channel or sewer capacities needed to carry the hydraulic loadings at specified slopes. The simulation model application results are presented in Chapter VII.

The second procedure involved the application of another 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 intensity-duration-frequency analyses. Such analyses have been performed by the Regional Planning Commission on

Figure 4

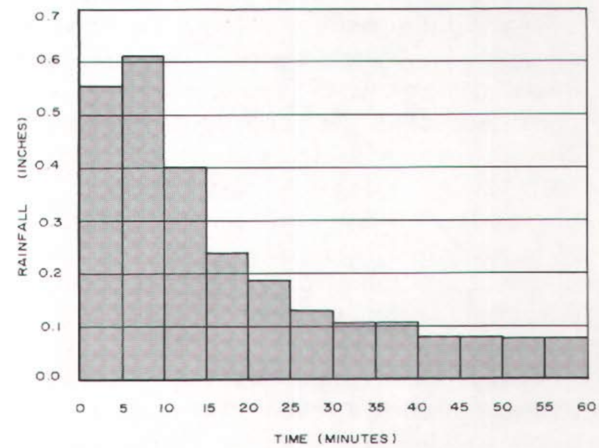
RAINFALL PATTERN FOR 10-YEAR RECURRENCE INTERVAL, ONE-HOUR STORM



Source: SEWRPC.

Figure 5

RAINFALL PATTERN FOR 100-YEAR RECURRENCE INTERVAL, ONE-HOUR STORM



Source: SEWRPC.

Milwaukee area meteorological data.[1] The rainfall function used for both the 10- and 100-year recurrence interval storms is shown in Figure 6 and is generated as an internal input in the model. 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 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 evaluation of multiple, sequential reservoir storage facilities and their effect on downstream peak flow rates.

The third procedure used involved the application of commonly used formulas and design criteria to check simulation modeling results and to provide supplementary information for system components not readily amenable to model application. Peak rates of flow for selected recurrence interval storms were calculated at critical locations in the minor stormwater drainage system using the Rational Method with Commission-developed rainfall intensity-duration-frequency data. Peak flows and total volumes were calculated using the U. S. Soil Conservation Service Technical Release No. (TR) 55 Method. The hydraulic capacities required to carry the peak flows were computed utilizing the Manning formula, and the cross-sectional areas and slopes of the pipes and channels concerned.

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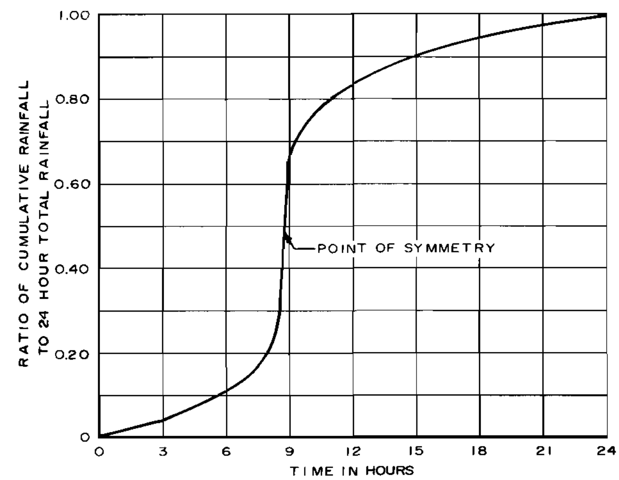
[1]See SEWRPC Technical Record, Vol. 3, No. 5, March 1973.

Stormwater Flow Rate and Volume

The quantification of the stormwater flow rates and volumes under both existing and probable future land use conditions allows sound, rational decisions to be made concerning stormwater management. Such quantification aids in determining the type, location, and configuration of stormwater management facilities and is essential to sizing facilities such as storm sewers, open channels, culverts and bridges, and storage and pumping facilities. The techniques used to quantify stormwater flow rate and volume in both the minor and major drainage systems have been briefly described above. These techniques provide the basic quantitative data needed to locate, configure, and size drainage facilities, and are needed to determine surface water flow rates, velocities, and volumes at the inlet and outlet points of each catchment area, and to determine the hydrologic and hydraulic characteristics of the catchment areas.

Figure 6

CUMULATIVE RAINFALL UNITGRAPH



Source: SEWRPC.

To ensure that the stormwater 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 ratioes 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 five-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 12 to 24 percent. However, the incremental cost increases on a systemwide basis may be expected to be on the order of about 12 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. Table 25 indicates that a five-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.

Based upon consideration of the costs and risks involved, a 10-year recurrence interval storm event was selected for use in the design of the minor elements of the stormwater management system for the Village of Hales Corners stormwater management study area, including the design of most conveyance and storage facilities. This recurrence interval is widely used to size storm drainage facilities within the Southeastern Wisconsin Region.

When designing the minor urban storage water system, the designer should be aware that exceeding capacity does not cause incipient catastrophy. On the contrary, it only means that the minor drainage system capacity has now been completely utilized and the unaccommodated portion of the stormwater flow will begin to cause inconvenience and/or disruption of activities as it courses through the major system. In this respect, the minor system differs substantially from the major system.

A 100-year recurrence interval storm event was selected for use in delineating areas of potential inundation along the stormwater management system, and to size major elements of the system. This recurrence interval--which is also used by the Regional Planning Commission in its flood control planning efforts, and by federal and state agencies for floodland regulation--was selected because the 100-year recurrence interval event approximates, in terms of the amount of land area inundated, the largest known flood levels that have actually occurred in the Region, thereby providing a conservatively safe level of protection against property damage and hazards to human health and safety from surcharge of the major, as opposed to the minor, stormwater management system.

Rainfall data, including rainfall intensity-duration-frequency relationships, were available from the files of the Regional Planning Commission as input to various methods used to compute stormwater runoff rates and volumes. These data are described in Chapter III. Data on the hydrologic and hydraulic characteristics of the study area were also available from the files of the Commission, including data on soils; topography; the drainage patterns of the natural streams and watercourses, the waterway openings of related bridges and culverts, and related flood hazard areas; wetlands; and areas with existing flood problems. Topographic maps prepared by the Village to Commission specifications at a scale of 1 inch equals 100 feet, with two-foot contour intervals, and Commission ratioed and rectified aerial photographs at a scale of 1 inch equals 400 feet, were used in the analyses. Storm drainage system maps, construction plans, as-built plans, and other pertinent information were obtained from the files of the Village and of a number of other governmental agencies having jurisdiction in the study area. These materials were evaluated and included in the body of resource materials drawn upon in the analytic and design phases of the work.

The data noted above were utilized to estimate hydraulic loadings--stormwater runoff rates and volumes--under existing and planned future land use conditions, and under existing and proposed stormwater management system configurations in the study area. The methods to quantify the runoff rates and volumes included, as already noted, the ILLUDAS mathematical simulation model, the HYDROUT mathematical simulation model, and two traditional methods--the Rational method and the U. S. Soil Conservation Service TR 55 method.

Criteria and Assumptions for Street Cross-Sections, Site Grading, Inlets, and Parallel Roadside Culverts

An important secondary function of all streets and highways is the collection and conveyance of stormwater runoff. The planning of stormwater drainage systems should therefore be done simultaneously with the planning of the location, configuration, and gradients of the street system. At the systems planning level, recommendations concerning the approximate center line elevations and gradients of existing and proposed streets are provided. Pertinent details of the curbs and gutters, roadside swales, and street crowns are assumed based upon typical cross-sections and must be further addressed in subsequent project development engineering.

The location and size of inlets and culverts, as a part of the minor stormwater drainage system, are dictated by the allowable stormwater spread and depth of flow in streets, and attendant interference with the safe movement of pedestrian and vehicular traffic. The commonly used street cross-section in the Village of Hales Corners has uniform pavement cross slopes of 0.02 foot per foot, and is drained with roadside swales and culverts. Grading beyond the right-of-way is at a slope of one foot vertical on five feet horizontal.

Given the standards formulated under the study, only two assumptions concerning site grading, and one assumption concerning culverts and inlets, were required for the systems planning. It was assumed that all new urban development and redevelopment will be designed to facilitate good drainage, with slopes away from all sides of buildings of at least one-quarter inch per foot to provide positive gravity drainage to streets or to interior drainage swales. It was assumed that interior drainage swales along side lot or back lot lines or site boundaries will have a minimum gradient of 0.01 foot per foot, and will provide positive gravity drainage to streets.

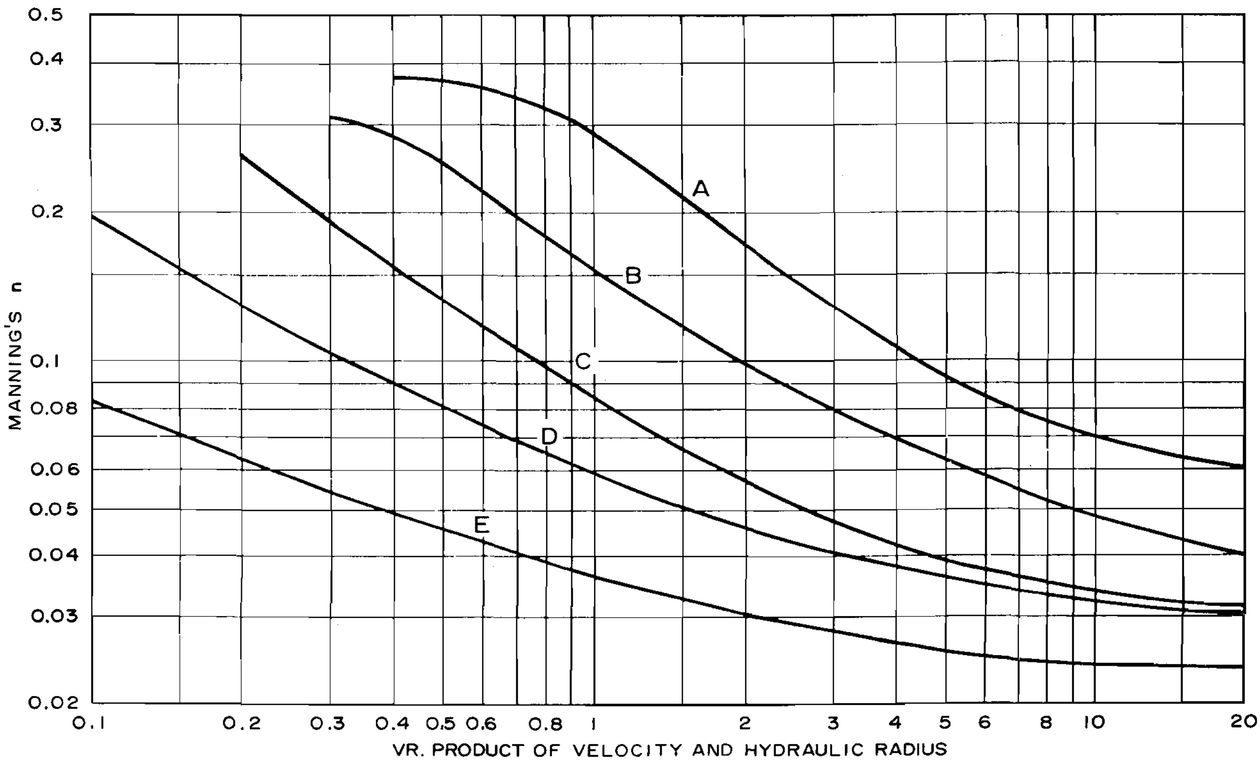
With regard to inlets and parallel roadside culverts, it was assumed that these system components will be designed to provide sufficient capacity to intake all flow in the tributary gutters or swales from storms up to and including the 10-year recurrence interval event. In the systems planning, critical locations were selected at which to check the specified overland and swale flow depths.

Criteria and Assumptions for Roadside Swales

At the systems planning level, only recommendations relating to the general configuration, size, approximate depth, slope, and type of roadside swales are provided. More detailed engineering at the project development level will be

Figure 7

MANNING'S "n" FOR VEGETAL-LINED CHANNELS FOR VARIOUS RETARDANCE LEVELS



Source: U. S. Soil Conservation Service.

needed to determine precise depth, location, and horizontal and vertical alignment of the swales, and the best response to constraints posed by structures and utilities.

In the systems planning, the Manning equation was used together with the cross-sectional area of flow to determine the required hydraulic capacity of swales. A Manning's "n" value corresponding to retardance level "D" in Figure 7 was assumed for well-constructed, properly maintained, frequently mowed, grass-lined roadside drainage swales, such as may be expected to exist adjacent to the frontyards in residential areas. A Manning's "n" value corresponding to retardance level "C" in Figure 7 was assumed for properly constructed, less frequently maintained (one- to two-month mowing cycle), grass-lined roadside drainage swales commonly found in rural areas. The retardance level for other vegetation is classified in Tables 23 and 24.

The following criteria and assumptions relating to the details of the grass-lined storm drainage swales and channels in and along street rights-of-way were used in the development of the stormwater management plan:

1. Swales were assumed generally to be located in public street rights-of-way and to follow the street alignments and gradients.

Table 23

CLASSIFICATION OF VEGETAL COVERS AS TO DEGREE OF RETARDANCE

Retardance	Cover	Condition
A	Weeping lovegrass................	Excellent stand, tall (average 30 inches)
	Yellow bluestem Ischaemum........	Excellent stand, tall (average 36 inches)
B	Kudzu............................	Very dense growth, uncut
	Bermuda grass....................	Good stand, tall (average 12 inches)
	Native grass mixture (little bluestem, blue grama, and other long and short mid-west grasses)..................	Good stand, unmowed
	Weeping lovegrass................	Good stand, tall (average 24 inches)
	Lespedeza sericea................	Good stand, not woody, tall (19 inches)
	Alfalfa..........................	Good stand, uncut (average 11 inches)
	Weeping lovegrass................	Good stand, mowed (average 13 inches)
	Kudzu............................	Dense growth, uncut
	Blue grama.......................	Good stand, uncut (average 13 inches)
C	Crabgrass........................	Fair stand, uncut (10 to 48 inches)
	Bermuda grass....................	Good stand, mowed (average 6 inches)
	Common lespedeza.................	Good stand, uncut (average 11 inches)
	Grass-legume mixture--summer (orchard grass, redtop, Italian ryegrass, and common lespedeza)..............	Good stand, uncut (6 to 8 inches)
	Centipedegrass...................	Very dense cover (average 6 inches)
	Kentucky bluegrass...............	Good stand, headed (6 to 12 inches)
D	Bermuda grass....................	Good stand, cut to 2.5-inch height
	Common lespedeza.................	Excellent stand, uncut (average 4.5 inches)
	Buffalograss.....................	Good stand, uncut (3 to 6 inches)
	Grass-legume mixture--fall, spring (orchard grass, red-top, Italian ryegrass, and common lespedeza)..............	Good stand, uncut (4 to 5 inches)
	Lespedeza sericea................	After cutting to 2-inch height. Very good stand before cutting
E	Bermuda grass....................	Good stand, cut to 1.5-inch height
	Bermuda grass....................	Burned stubble

NOTE: Covers classified have been tested in experimental channels. Covers were green and generally uniform.

Source: U.S. Soil Conservation Service.

Table 24

GUIDE TO SELECTION OF VEGETAL RETARDANCE

Stand	Average Length of Vegetation	Degree of Retardance	Stand	Average Length of Vegetation	Degree of Retardance
Good	Longer than 30 inches	A	Fair	Longer than 30 inches	B
	11 to 24 inches	B		11 to 24 inches	C
	6 to 10 inches	C		6 to 10 inches	D
	2 to 6 inches	D		2 to 6 inches	D
	Less than 2 inches	E		Less than 2 inches	E

Source: U.S. Soil Conservation Service.

2. All swales should be designed to accommodate the peak runoff expected from a minor--that is, a 10-year recurrence interval--storm when flowing full with no freeboard.

3. All swales should be designed to provide a minimum flow velocity of 2.5 feet per second when accommodating the design storm; while the maximum flow velocity during the design storm event should be five feet per second.

4. The minimum depth of swales below street shoulder should be one and one-half feet, while the maximum depth should not exceed three feet.

Criteria and Assumptions for Cross Culverts

Cross culverts, which are a common feature of open drainage systems, are used to convey stormwater under a street, highway, railroad, or embankment. At the systems planning level, recommendations concerning the location and size of cross culverts are provided. More detailed engineering at the project development level will be needed to determine precise depth, location, and horizontal and vertical alignment of the culverts; the type of material to be used; and the best response to constraints posed by structures and utilities. In the systems planning, the Manning equation was used to determine flow rates and headlosses of culverts. The hydraulic capacity of any culvert is affected by its cross-sectional area, shape, entrance geometry, length, slope, construction material, and depth of ponding at the inlet and outlet, details which must be addressed at the project development level. In planning the system, required culvert sizes were determined by evaluating multiple constraints and selecting an appropriate size which best met all requirements. Inlet control nomographs relating culvert headwater depth to flow rates for specific culvert entrances are shown in Figures 8 and 9. Culvert headloss nomographs for annular corrugated metal pipes flowing full are shown in Figures 10 and 11. Culvert capacity charts which relate culvert headwater depth, flow rates, pipe lengths and pipe gradients for annular corrugated metal pipes are shown in Figures 12 through 15. Similar design information is readily available in the literature for elliptical, or box, sections for other entrance conditions and for other materials such as precast concrete, corrugated aluminum, and structural plate corrugated metal.

Manning's "n" values as shown in Figure 16 were assumed for properly installed and maintained corrugated metal pipe and pipe arch culverts. A Manning's "n" value of 0.012 was assumed for well-constructed, precast, concrete pipe culverts flowing full. Where analyses indicated that pipes would flow less than full at design loading, the hydraulic element charts set forth in Figures 17 and 18 were used to determine critical characteristics required for solution of Manning's equation, or those characteristics were computed directly in the simulation model. Hydraulic conditions for major system components under major storm event conditions were evaluated on a case-by-case basis.

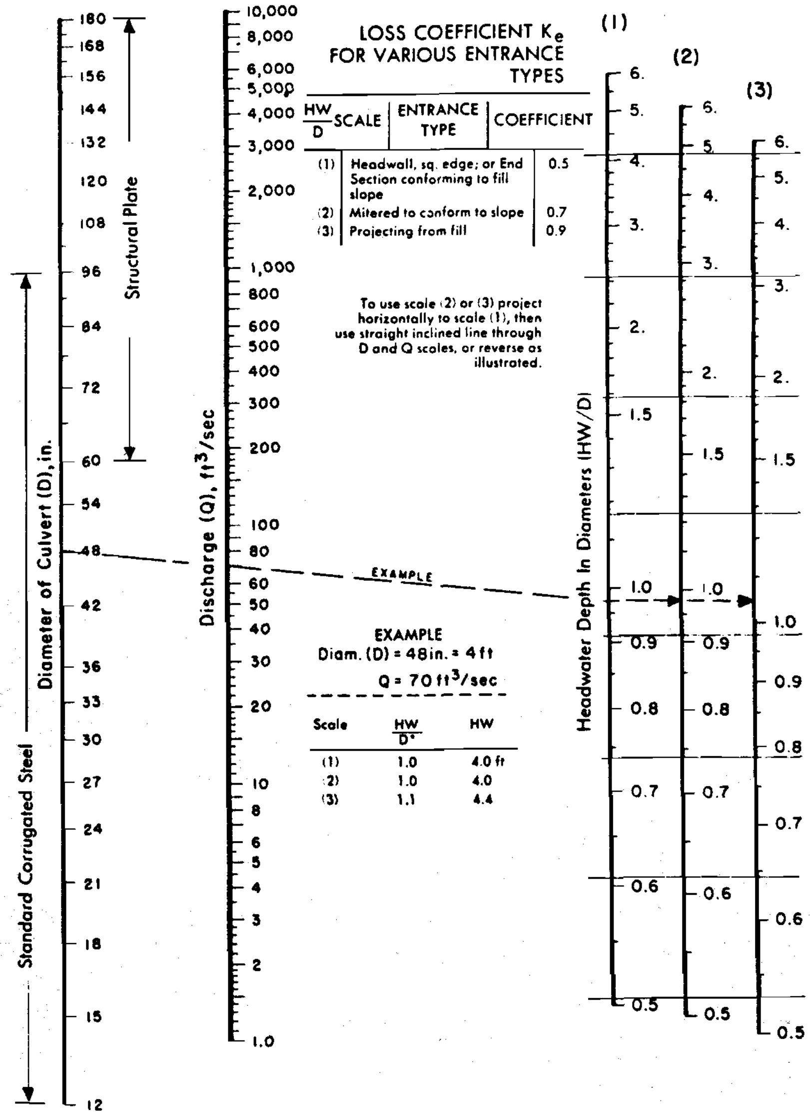
The following criteria and assumptions were used in the development of culvert sizes for the stormwater management system plan:

1. The culvert location should provide a direct exit, avoiding an abrupt change in direction at the outlet end and, preferably, at the inlet end.

2. The minimum culvert size would be 12 inches in diameter.

Figure 8

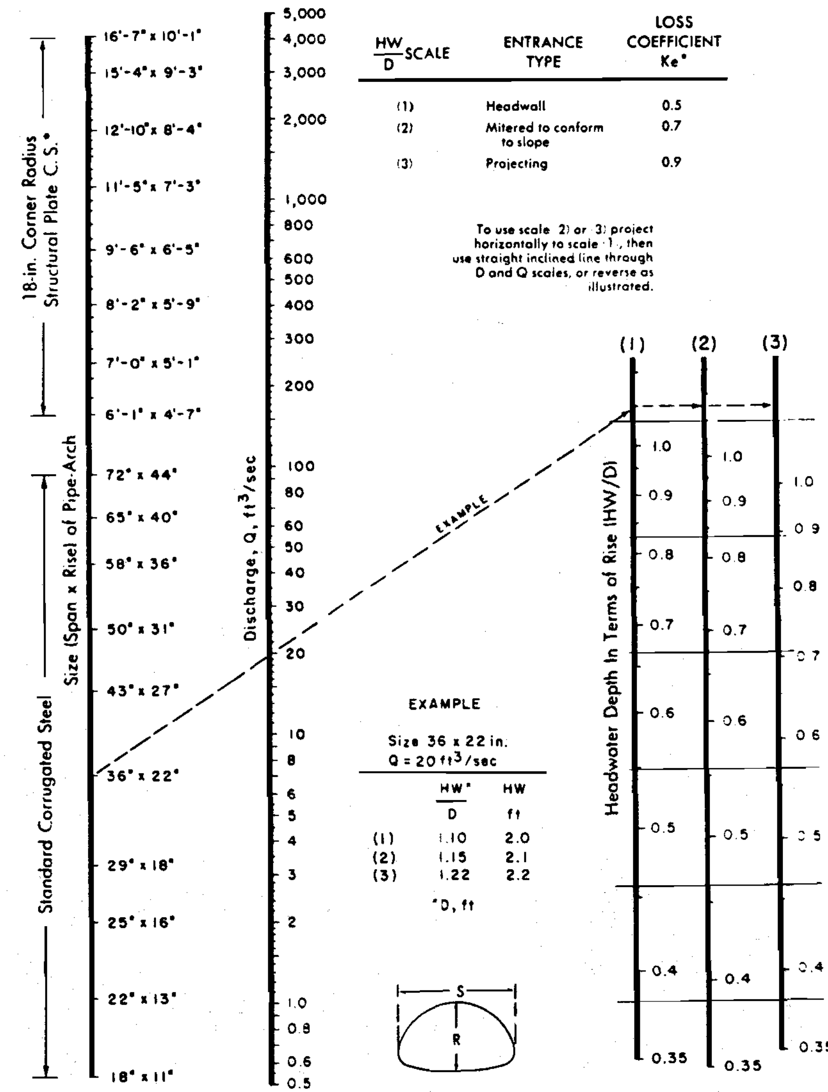
HEADWATER DEPTH FOR CORRUGATED METAL PIPE CULVERTS WITH INLET CONTROLS



Source: Federal Highway Administration; and American Iron & Steel Institute, Handbook of Steel Drainage and Highway Construction Products.

Figure 9

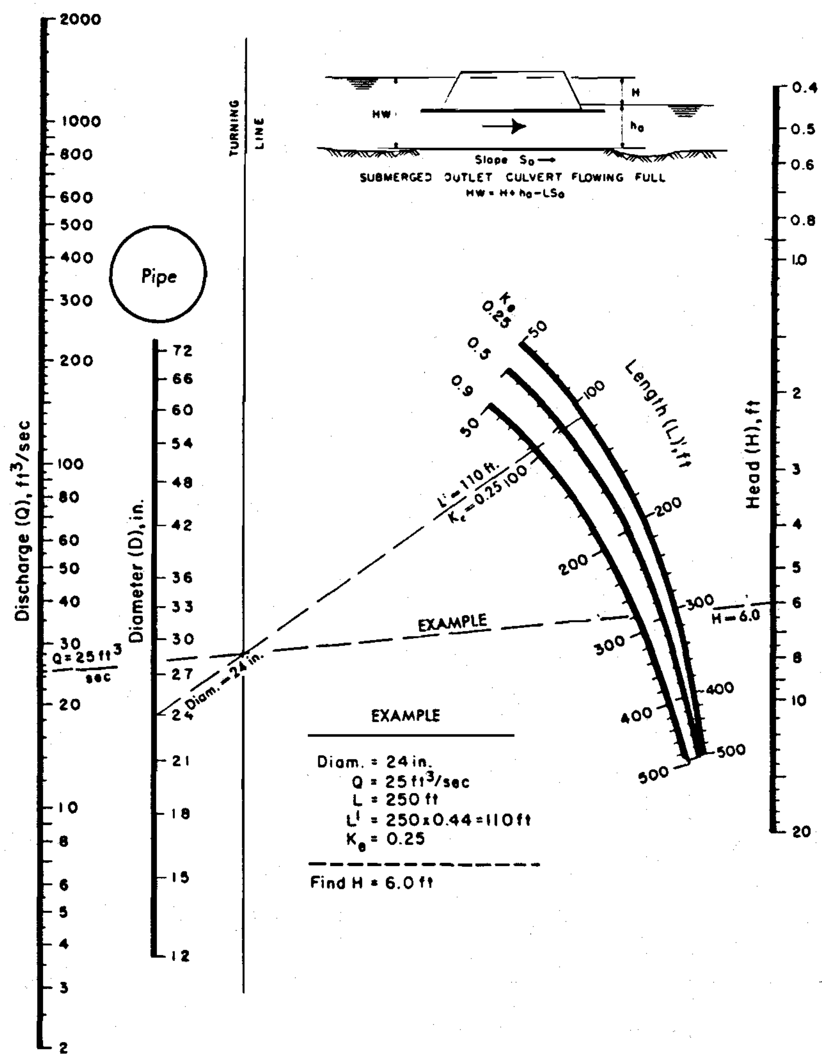
HEADWATER DEPTH FOR CORRUGATED METAL PIPE-ARCH CULVERTS WITH INLET CONTROLS



Source: Federal Highway Administration; and American Iron & Steel Institute, Handbook of Steel Drainage and Highway Construction Products.

Figure 10

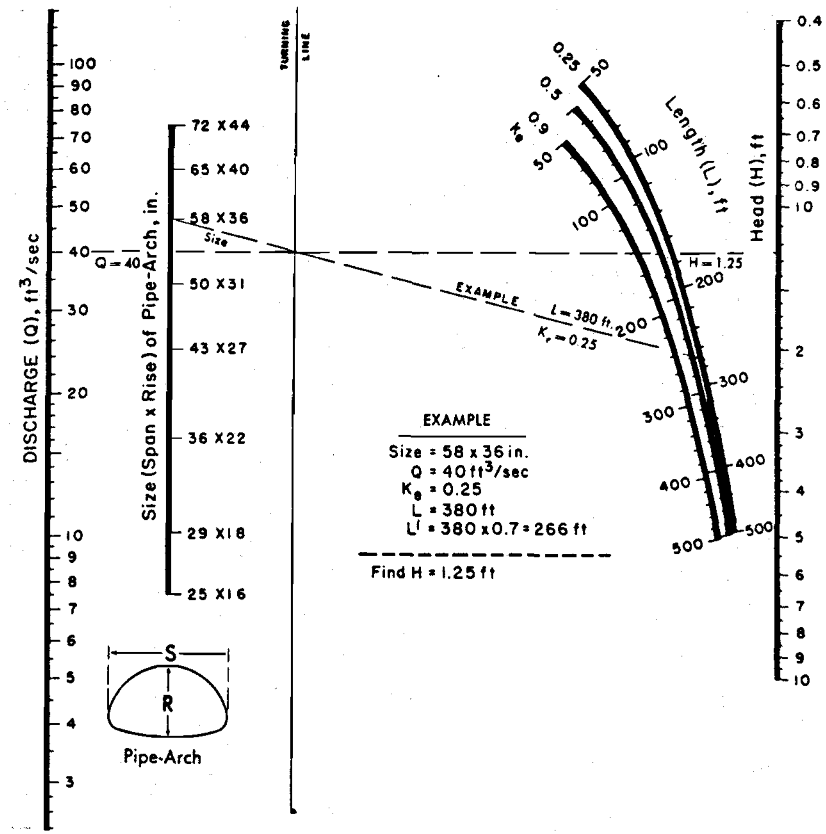
HEAD FOR STANDARD ANNULAR CORRUGATED METAL PIPE CULVERTS FLOWING FULL n = 0.024



Source: Federal Highway Administration; and American Iron & Steel Institute, Handbook of Steel Drainage and Highway Construction Products.

Figure 11

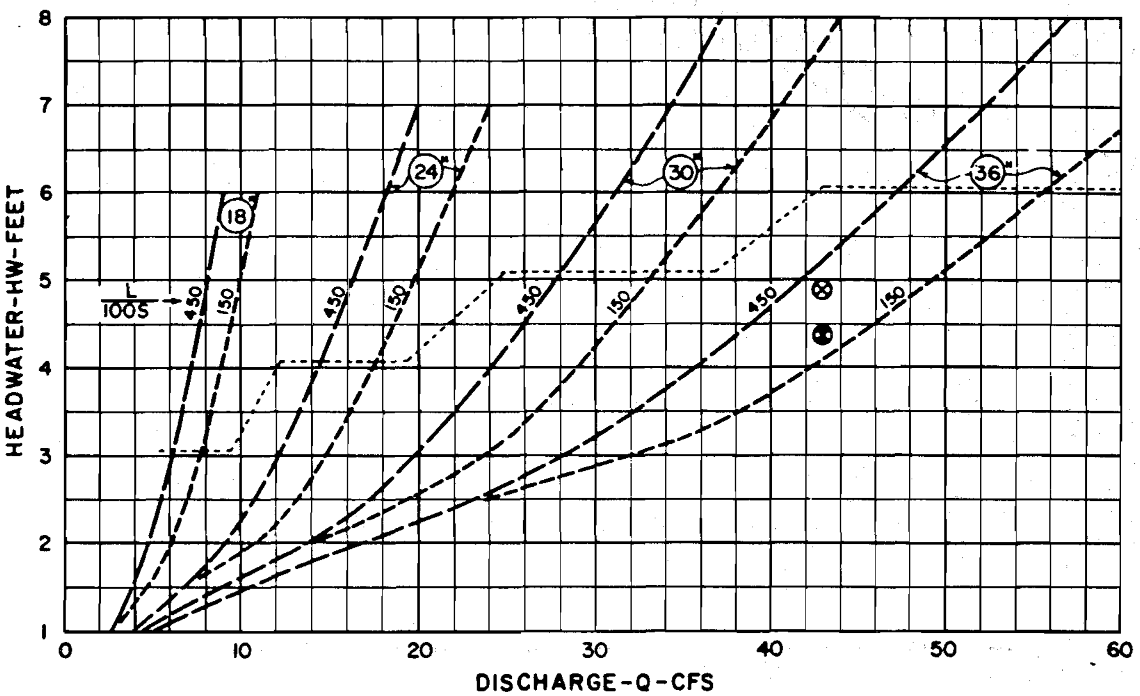
HEAD FOR STANDARD ANNULAR CORRUGATED METAL PIPE-ARCH CULVERTS FLOWING FULL n = 0.024

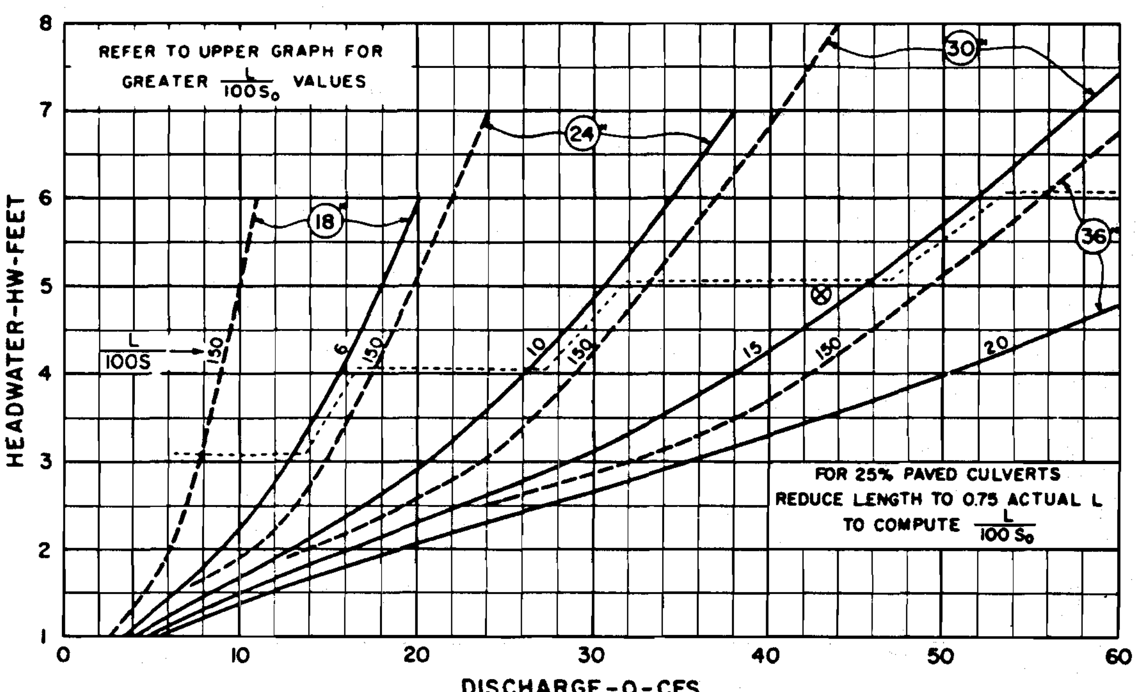


Source: Federal Highway Administration; and American Iron & Steel Institute, Handbook of Steel Drainage and Highway Construction Products.

Figure 12

CULVERT CAPACITY STANDARD CIRCULAR CORRUGATED METAL PIPE HEADWALL ENTRANCE: 18" TO 36"





From BPR
EXAMPLE

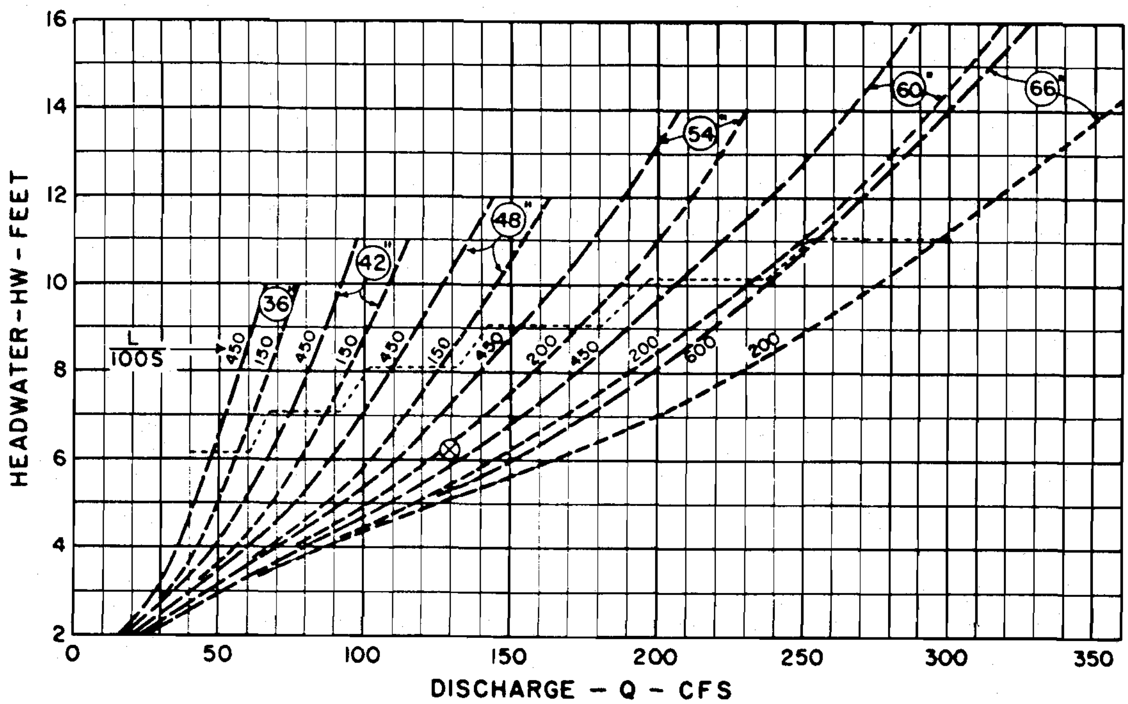
⊗ GIVEN:
43 CFS; AHW = 4.9 FT.
L = 72 FT.; S_o = 0.003

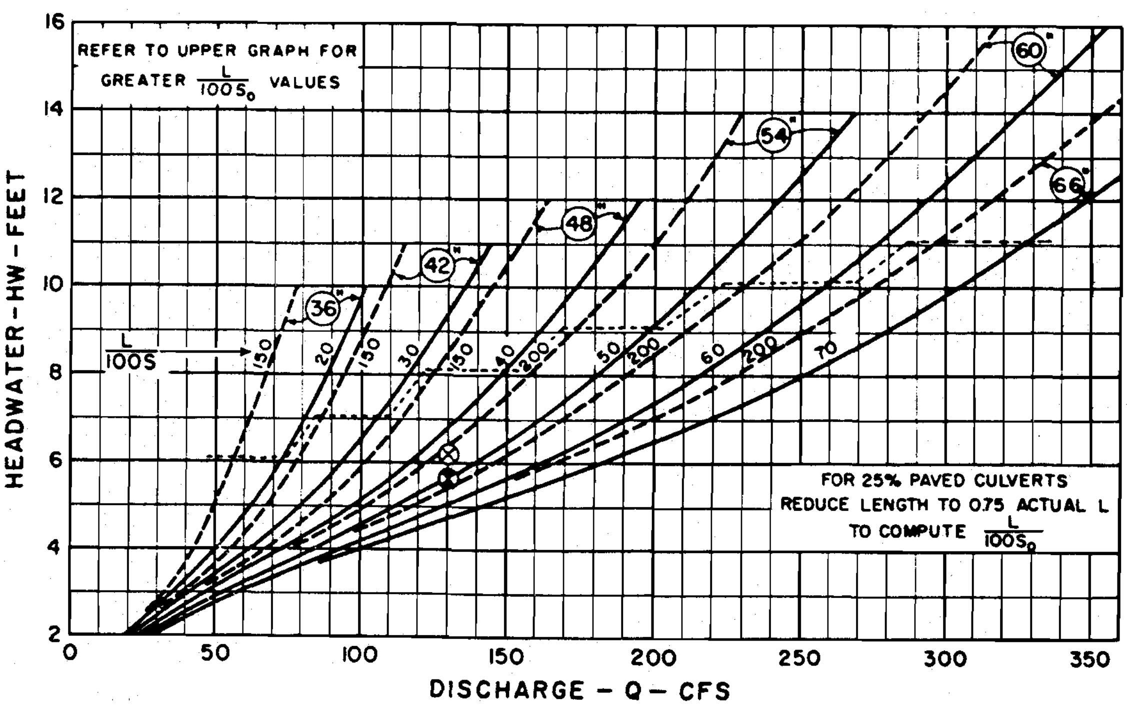
⊗ SELECT 36" UNPAVED
HW = 4.4 FT.

Source: Federal Highway Administration and Denver Regional Council of Governments.

Figure 13

CULVERT CAPACITY STANDARD CIRCULAR CORRUGATED METAL PIPE HEADWALL ENTRANCE: 36" TO 66"





EXAMPLE

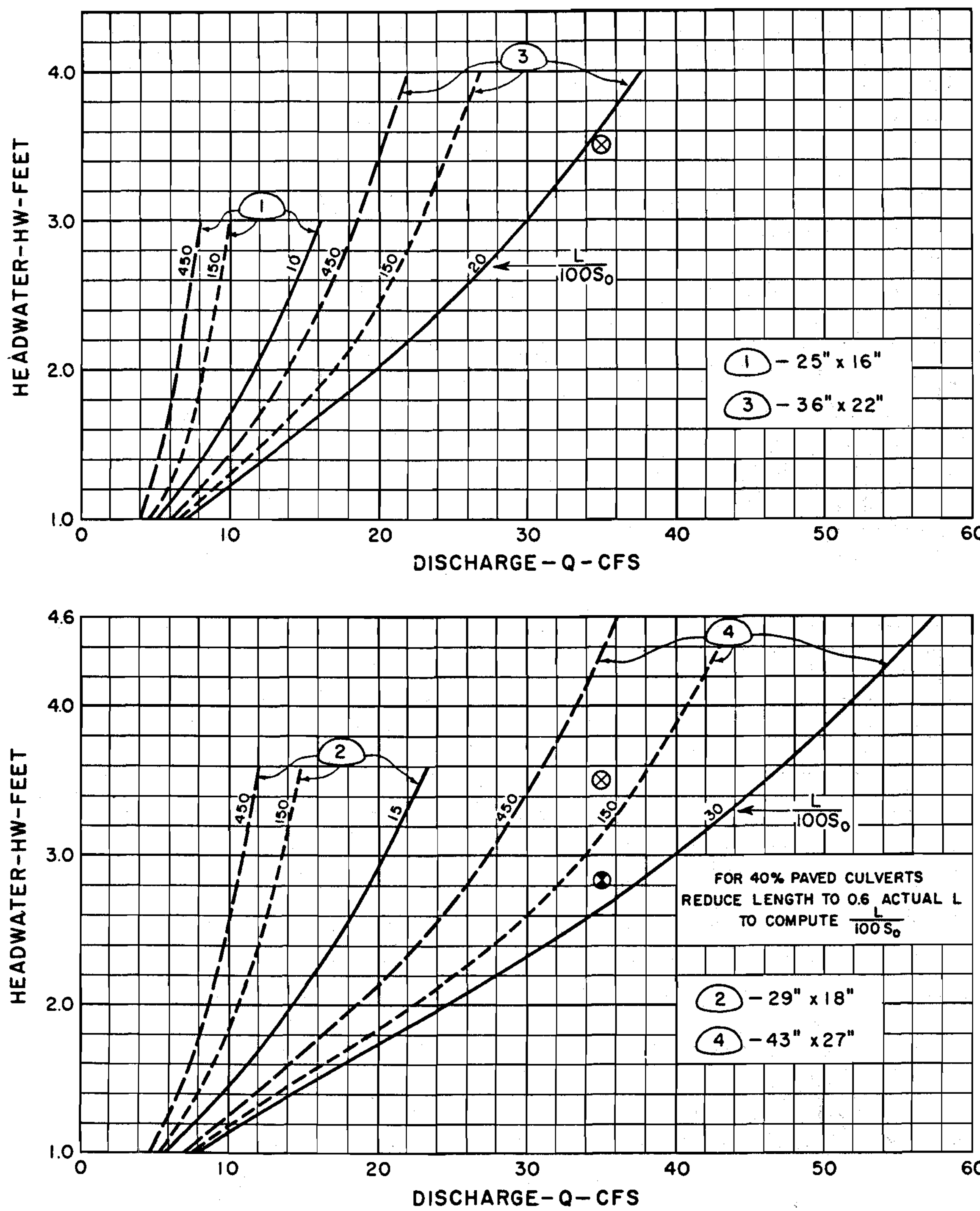
⊗ GIVEN:
130 CFS; AHW = 6.2 FT.
L = 120 FT.; S_0 = 0.025

⊗ SELECT 54" UNPAVED
HW = 5.6 FT.

Source: Federal Highway Administration and Denver Regional Council of Governments.

Figure 14

CULVERT CAPACITY STANDARD CORRUGATED METAL PIPE-ARCH HEADWALL ENTRANCE: 25" X 16" TO 43" X 27"



EXAMPLE

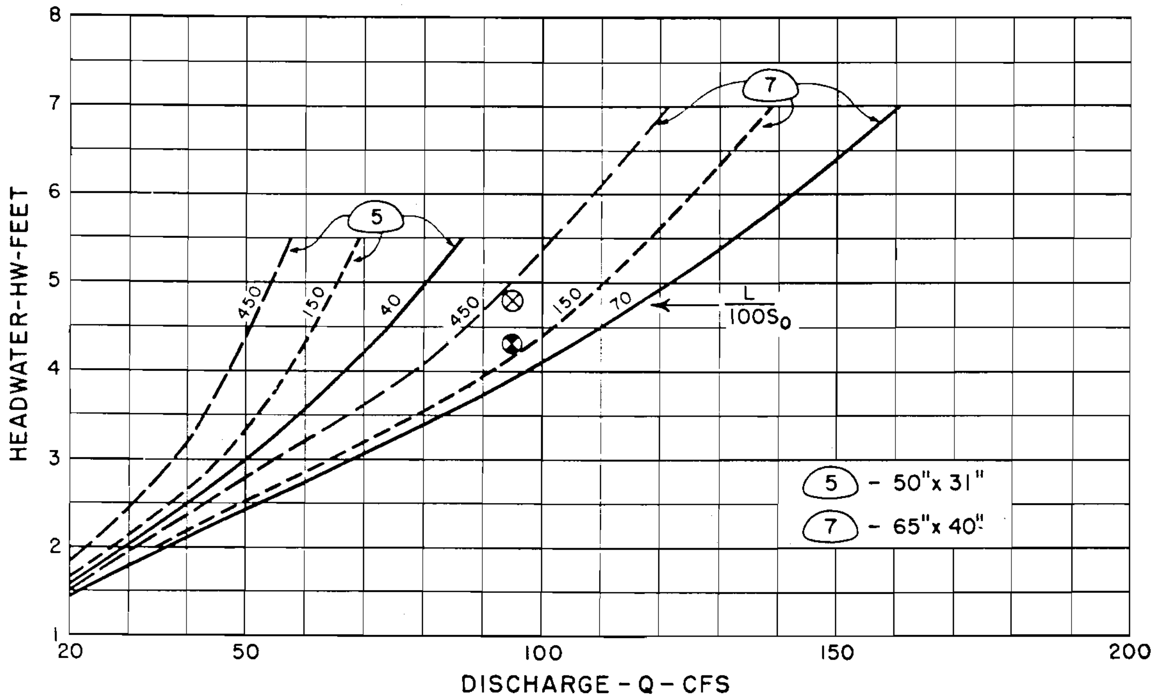
⊗ GIVEN:
35 CFS; AHW = 3.5 FT.
L = 145 FT.; S_0 = 0.020

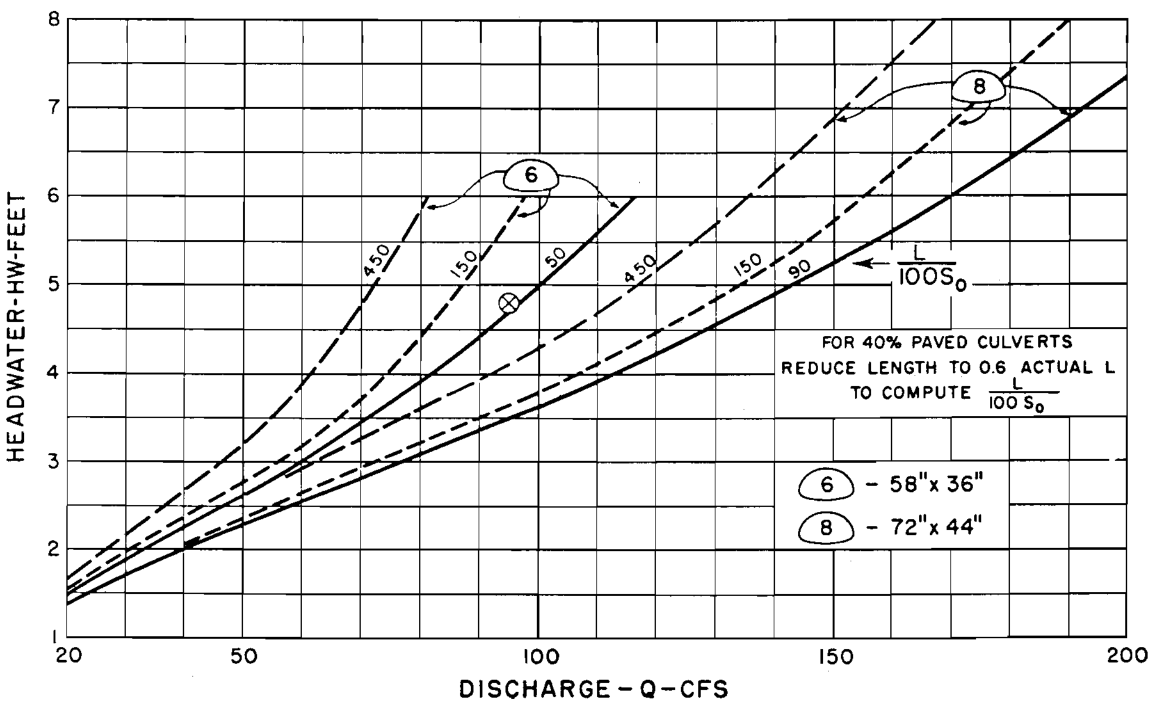
⊗ SELECT NO.4, 43" x 27"
HW = 2.8 FT.
UNPAVED INVERT

Source: Federal Highway Administration.

Figure 15

CULVERT CAPACITY STANDARD CORRUGATED METAL PIPE-ARCH HEADWALL ENTRANCE: 50" X 31" TO 72" X 44"





EXAMPLE

⊗ GIVEN:
95 CFS; AHW = 4.8 FT.
L = 240 FT.; S_0 = 0.012

SELECT NO. 7, 65"x 40"
HW = 4.3 FT.
UNPAVED INVERT

Source: Federal Highway Administration.

3. The culverts should be laid on a constant gradient.

4. Culverts were assumed to be circular or pipe arches, constructed of corrugated metal pipe.

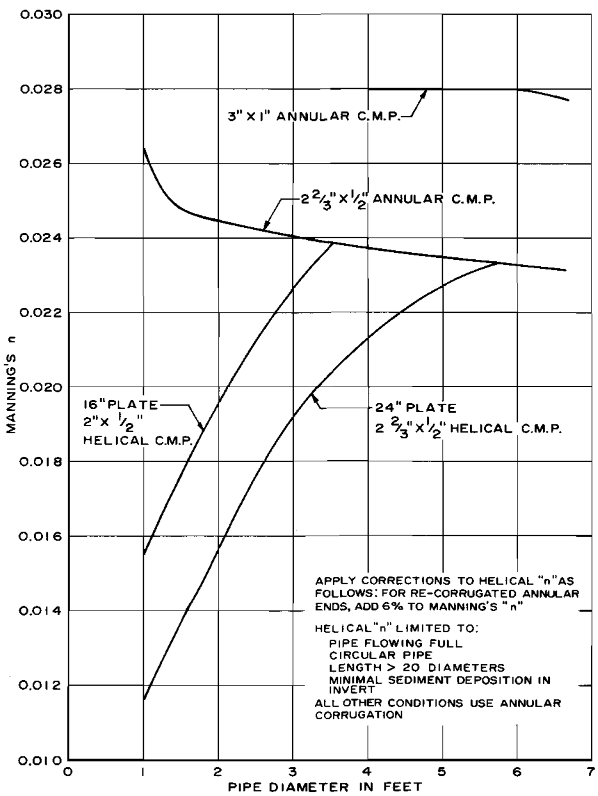
5. Culverts were assumed to have an unsubmerged outlet during a minor, that is, a 10-year recurrence interval, storm event.

Criteria and Assumptions for Open Drainage Channels

At the systems planning level, recommendations are provided for only the general location, cross-section bottom width and approximate bottom elevation depth, side slopes, area, gradient, and type of open drainage channels. More detailed engineering at the project development level will be needed to determine the precise location and horizontal and vertical alignment of the channels, the need for and type of channel lining, and the best response to constraints posed by structures, other utilities, and street layout.

Figure 16

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



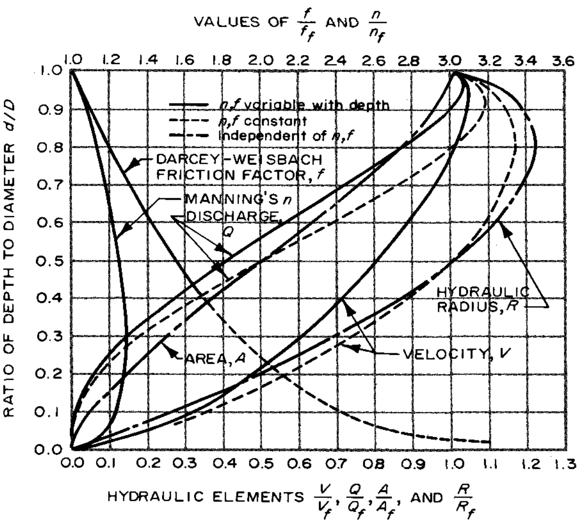
Source: U. S. Department of Transportation, Hydraulic Flow Resistance Factors for Corrugated Metal Conduit, and SEWRPC.

Open drainage channels in and along exclusive rights-of-way are a necessary and appropriate component of the total stormwater drainage system of the Village and environs. Such channels may, in certain areas, serve as part of the minor drainage system, as, for example, in parks and cemeteries, in some commercial and industrial areas, and in some low-density residential areas. Such channels inevitably form part of the major stormwater drainage system as well. In some areas of the stormwater management study area, open drainage channels, together with roadside swales, may serve as the sole component of the engineered stormwater drainage system which conveys surface runoff to the receiving natural stream system.

In the system planning, the Manning's equation was used together with the cross-sectional area of flow to determine the hydraulic capacity of open channels. A Manning's "n" value of 0.030 was assumed for all turf-lined channels, and a value of 0.015 for all concrete-lined channels. Composite channels with grass slopes and a concrete cunette or bottom pavement were analyzed by summation of flows in each vertical segment using the appropriate Manning's "n" value. Receiving natural stream channels were analyzed using the U. S. Army Corps of Engineers' HEC-2 step backwater simulation model. Depths and velocities of open channel flow for various channel cross-sections were determined from Figures 19 through 22, or were computed directly in the simulation model.

Figure 17

HYDRAULIC ELEMENTS GRAPH FOR CIRCULAR SEWERS

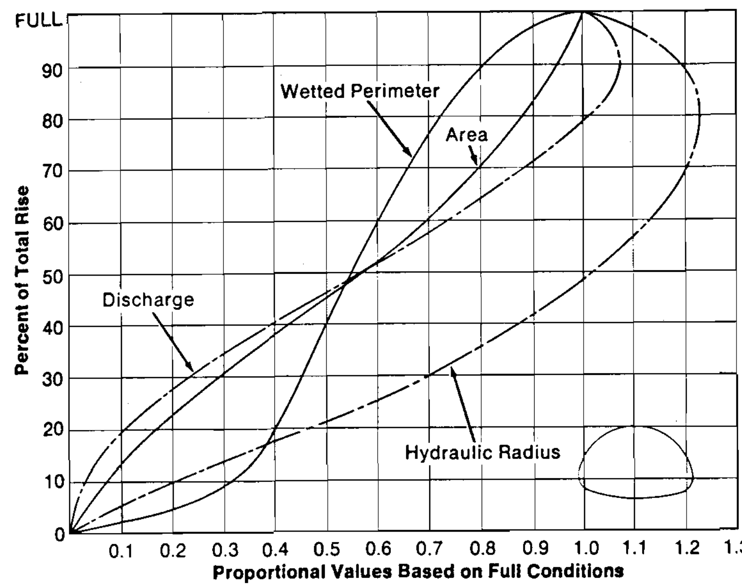


NOTE: $n, V, Q,$ AND R REPRESENT PARTIAL FLOW CONDITIONS. $n_f, V_f, Q_f,$ AND R_f REPRESENT FULL FLOW CONDITIONS.

Source: American Society of Civil Engineers.

Figure 18

HYDRAULIC PROPERTIES OF CORRUGATED STEEL AND STRUCTURAL PLATE PIPE-ARCHES



Source: American Iron & Steel Institute.

The following criteria relating to the details of the open drainage channels were used in the development of the stormwater management plan:

1. All open drainage channels should be designed to accommodate the peak runoff from a major, that is, a 100-year recurrence interval, storm when flowing full with no freeboard.

2. Turf-lined side slopes should not exceed one vertical on two and one-half horizontal, and where practical should be one vertical on four horizontal.

3. The minimum gradient of all turf-lined open channels should be 0.010 foot per foot.

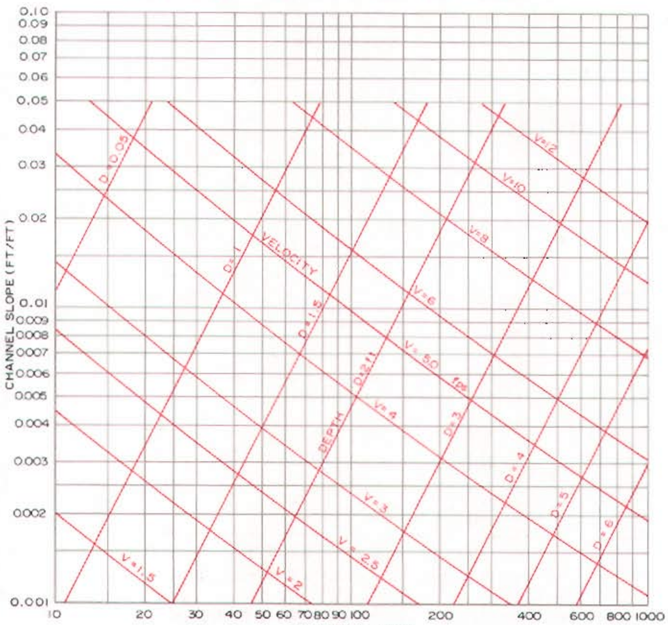
4. All concrete-lined and composite-lined channels should be designed to provide a minimum flow velocity of 2.5 feet per second when accommodating the peak runoff from a minor, that is, a 10-year recurrence interval, storm; while the maximum flow velocity during the design storm event should be five feet per second for turf linings.

Criteria and Assumptions for Storm Sewers

At the systems planning level, only recommendations for the general configuration, size, approximate invert elevation, slope, and type of storm sewer facilities are provided. More detailed engineering at the project development level will be needed to determine the precise invert elevation, location, and

Figure 19

SLOPE DISCHARGE RELATIONSHIP FOR 5-FOOT-WIDE BOTTOM OPEN CHANNEL

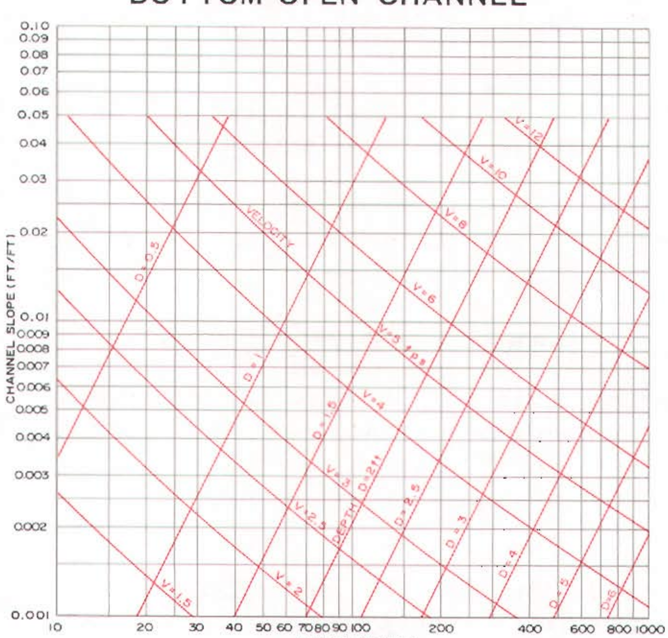


NOTE: BASED ON MANNING'S EQUATION
MANNING'S "n" OF 0.030
CHANNEL SIDE SLOPE OF ONE ON FOUR

Source: SEWRPC.

Figure 20

SLOPE DISCHARGE RELATIONSHIP FOR 10-FOOT-WIDE BOTTOM OPEN CHANNEL

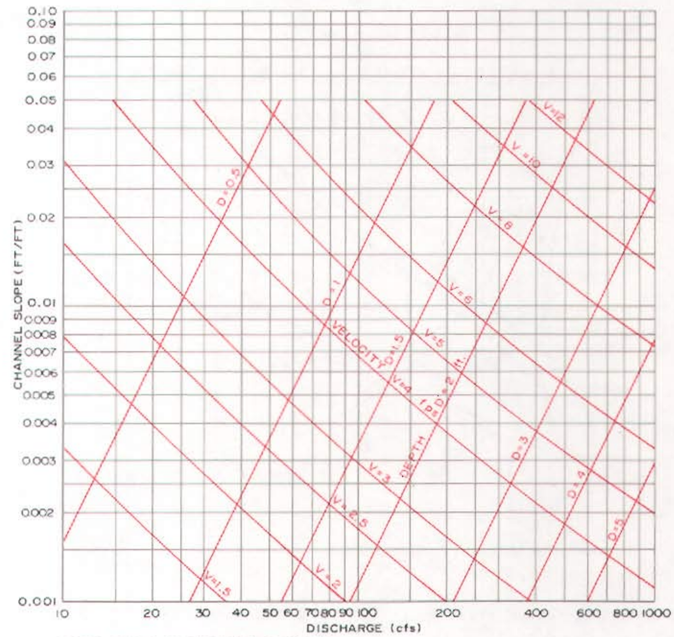


NOTE: BASED ON MANNING'S EQUATION
MANNING'S "n" OF 0.030
CHANNEL SIDE SLOPE OF ONE ON FOUR

Source: SEWRPC.

Figure 21

SLOPE DISCHARGE RELATIONSHIP FOR 15-FOOT-WIDE BOTTOM OPEN CHANNEL

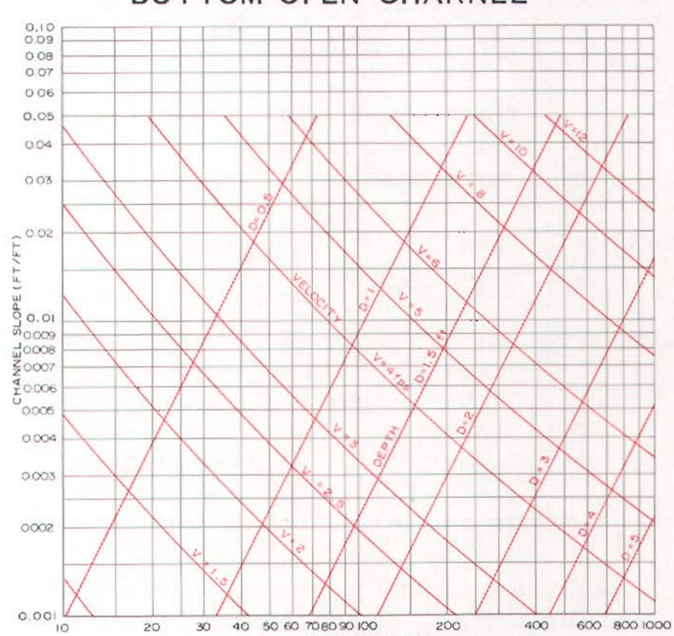


NOTE: BASED ON MANNING'S EQUATION
MANNING'S "n" OF 0.030
CHANNEL SIDE SLOPE OF ONE ON FOUR

Source: SEWRPC.

Figure 22

SLOPE DISCHARGE RELATIONSHIP FOR 20-FOOT-WIDE BOTTOM OPEN CHANNEL



NOTE: BASED ON MANNING'S EQUATION
MANNING'S "n" OF 0.030
CHANNEL SIDE SLOPE OF ONE ON FOUR

Source: SEWRPC.

horizontal and vertical alignment of the sewer, the type of material used for the sewer, and the best response to constraints posed by structures and other utilities.

In the system planning, Manning's equation was used together with the cross-sectional area of flow to determine the hydraulic capacity of sewers. Values for the Manning's roughness coefficient "n" vary with the type and conditions of the sewer, the depth of flow in the sewer, and the diameter of the sewer. A Manning's "n" value of 0.012 was assumed typical of well-constructed, precast, concrete pipe sewer lines. Accordingly, sewer capacities and flow velocities were either determined from the graph set forth in Figure 23, or calculated directly in the simulation model.

Where the analyses indicated the sewers would flow less than full at design loading, the hydraulic element chart set forth in Figure 17 was used to determine the critical characteristics; or those characteristics were computed directly in the simulation model.

The following criteria and assumption relating to the details of the storm sewers were used in the development of the stormwater management plan:

1. Storm sewers were assumed generally to be located in public street rights-of-way and to follow the street alignments and gradients.

2. All storm sewers should be designed to accommodate the peak runoff expected from a minor, that is, a 10-year recurrence interval, storm when flowing full.

3. The minimum pipe size should be 12 inches in diameter.

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

5. At all junctions and changes in pipe size, the 0.8 depth-diameter point of the pipes should be aligned.

6. At all changes in horizontal direction of 30° or more, a drop should be provided to compensate for associated energy losses. The drop shall equal:
   $K \times \frac{V^2}{2g}$, where K is determined from Figure 24.

7. The radius of the centerline of a bend should be at least one and one-half times the diameter of the sewer. Additional drop should be provided to the pipe to compensate for associated energy losses. The drop shall equal:
   $K \times \frac{V^2}{2g}$, where K is determined from Figure 24.

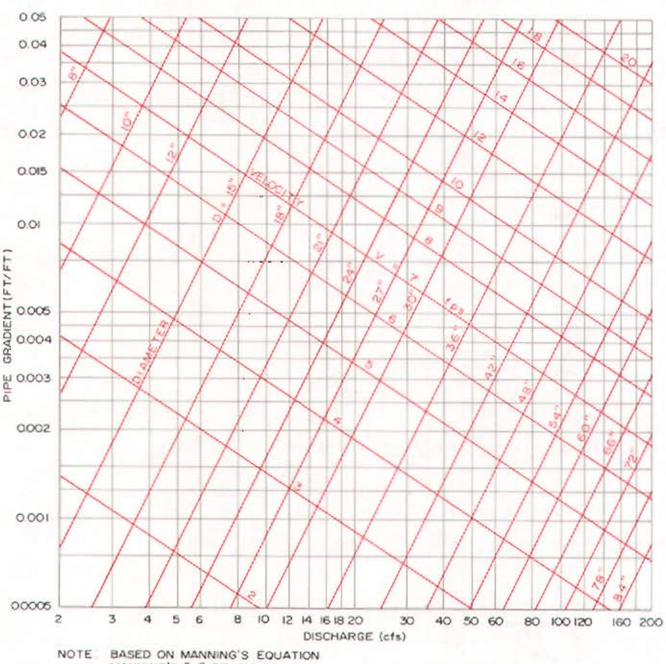
8. The minimum depth of cover over the top of the sewer should be three feet.

Criteria and Assumptions for Stormwater Storage Facilities

Natural storage of stormwater is provided during overland flow in surface depressions, vegetated areas, and pervious soils. Natural storage can be enhanced by preserving open areas, woodlands, wetlands, ponds, and areas with

Figure 23

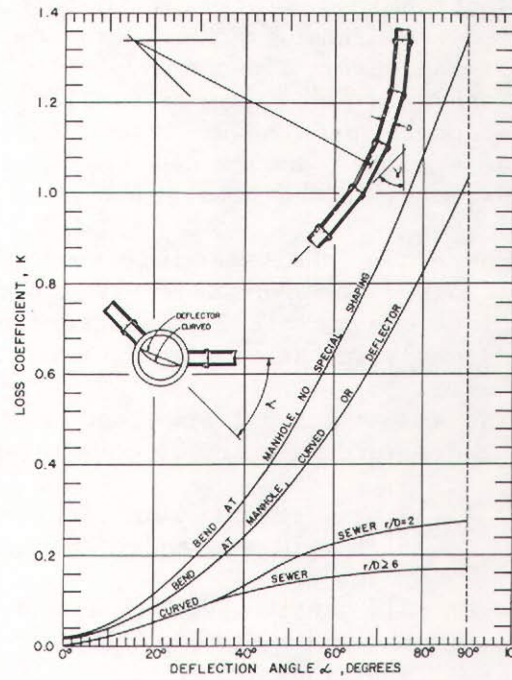
CAPACITY AND VELOCITY
OF FLOW IN CIRCULAR
STORM SEWERS FLOWING FULL



Source: SEWRPC.

Figure 24

SEWER BEND
LOSS COEFFICIENT



Source: Denver Regional Council of Governments, Urban Storm Drainage.

large infiltration capacities. These attributes can usually be incorporated into a stormwater management system at less cost than would be required for the incorporation of artificial storage facilities. Artificial storage facilities include constructed onsite swales, roadside swales, temporary storage facilities on parking lots and other open areas, and retention and detention basins.

At the systems planning level, recommendations concerning only the location, type, approximate size, and capacity of storage facilities and outlet flow constraints are provided. For systems planning and preliminary project planning purposes, the relationships between peak flow reduction, design storage volume, and runoff volume are indicated in Figure 25. Storage facilities with outlets of the overflow weir or unsubmerged pipe-end types tend to approach the performance indicated by curve 1 in Figure 25, while outlets of the submerged pipe-end type tend to approach the performance indicated by curve 2. More detailed engineering at the project development level will be needed to precisely locate, configure, and size storage facilities and to specify such details as the inlet and outlet control facilities.

In planning the system, required storage volumes were calculated using a modification of the Rational Method, the HYDROUT simulation model, or the ILLUDAS simulation model. The following criteria relating to storage facilities were used in the development of the stormwater management system plan:

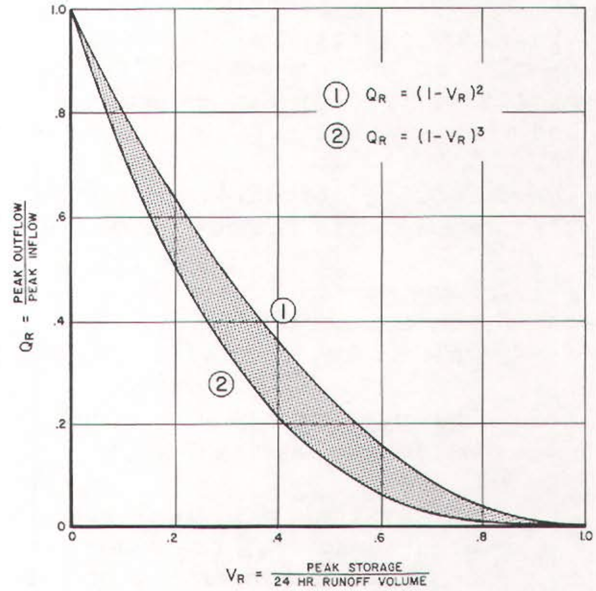
1. Storage facilities should be sized to accommodate a minor, that is, a 10-year recurrence interval, storm event. This criterion does not apply to storage facilities designed as components of the downstream floodland management system, which should be sized to accommodate a major, that is, a 100-year recurrence interval, storm event.

2. Storage facilities should be considered to achieve reductions in peak runoff rates to eliminate identified site-specific problems.

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

Figure 25

DETENTION OUTFLOW-STORAGE RELATIONSHIPS



Source: Bauman, Drainage Report for Oak Creek Southeast Branch, December 1982.

4. To effectively trap sediments, storm runoff should be stored when and where practical for at least 45 minutes during the design storm, thus allowing about 70 percent of the incoming sediments to settle out. Providing a retention pond volume in acre-feet equal to the tributary drainage area in acres divided by 150 generally should accomplish this reduction.[2]

5. Where practical, the length of the storage facility, as measured from the inlet to the outlet, should be at least twice the width. Such facilities should, where possible, be wedge-shaped, with the apex, or narrow end, containing the inlet, and have side slopes not exceeding one on three.

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

Stormwater Pumping

The purpose of stormwater pumping is to remove stormwater from low-lying areas that cannot be effectively drained by gravity. Stormwater pumping stations are also commonly associated with stormwater storage facilities that have limited

[2]Eugene D. Driscoll, "Performance of Detention Basins for Control of Urban Runoff Quality," presented at 1984 International Symposium on Urban Hydrology, Hydraulics and Sediment Control, University of Kentucky, Lexington, Kentucky, July 1983.

land surface available and are restricted to deep storage. Pumping should not be included as a component of the stormwater management plan when another alternative providing gravity drainage is practical.

At the systems planning level, only recommendations concerning the location, type, and capacity of the pumping facility are provided. More detailed engineering at the project development level 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 assumption relating to stormwater pumping facilities were used in the development of the stormwater management system plan:

1. Pumping stations should be designed with sufficient capacity to handle the estimated flows from a minor, that is, a 10-year recurrence interval, storm event with one pump out of service.

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

3. For systems planning purposes, it was assumed that the pumps would be high-capacity, low-head centrifugal pumps with constant speed motors designed for intermittent service.

Water Quality Management Measures

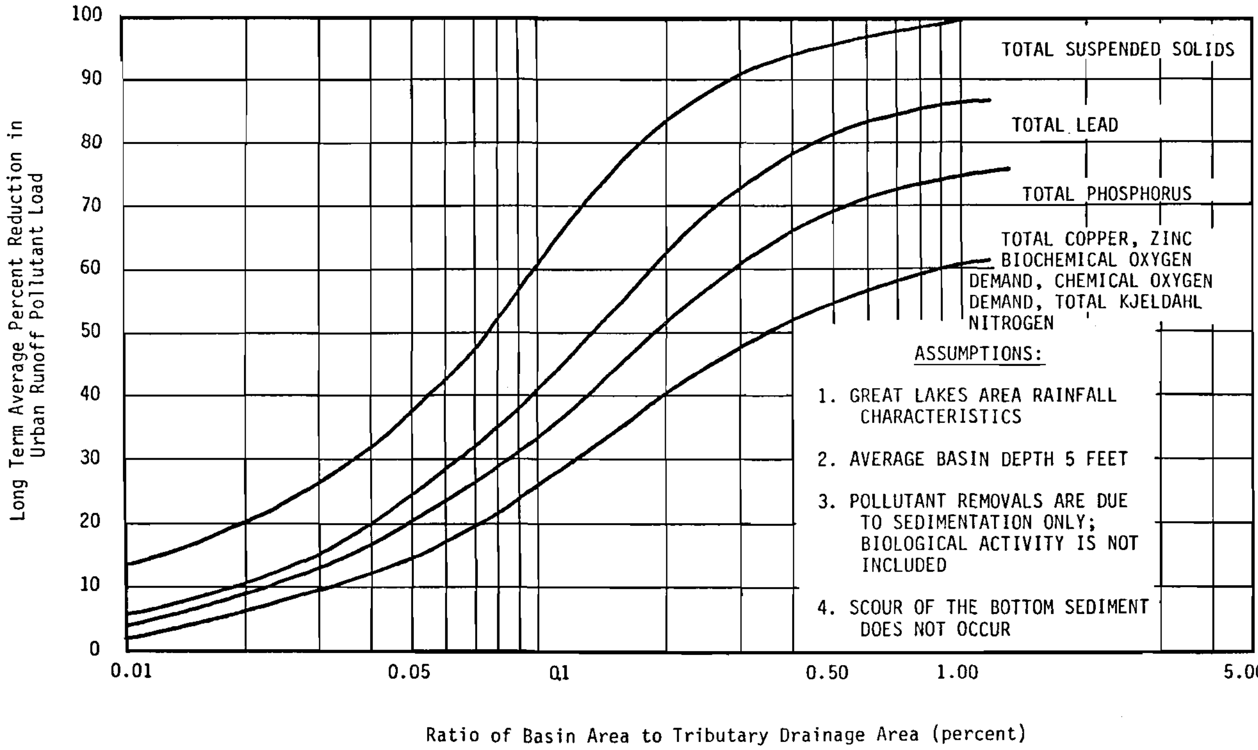
Stormwater quality management measures include stormwater storage measures and other nonpoint source pollution abatement measures. Stormwater storage measures remove pollutants in stormwater runoff by sedimentation, biological uptake, and chemical reactions. Pollutant removal rates as a function of basin volume are shown in Figure 26. Other nonpoint source pollution abatement measures help protect water quality by reducing the rate and volume of storm runoff which transports pollutants to a receiving stream and by controlling pollutants at their source before transport by runoff.

At the systems planning level, only the type, location, and general water quality benefits expected from urban nonpoint source pollution abatement measures are provided. The detailed design of a nonpoint source pollution abatement program will require a site-specific inventory of nonpoint pollution problems, the determination of the exact sizing and extent of application of measures, an identification of which measures are publicly acceptable and can be incorporated into the existing public works programs of the Village, and the physical detailed design of any structural measures.

Ideally, an acceptable level of risk should be determined for areas temporarily susceptible to erosion such as construction sites, taking into account the expected duration of vulnerability to excessive erosion, time of year of vulnerability, fraction of site vulnerable, erodibility of onsite soils, cost of construction and maintenance of control measures, and cost of damage and restoration if capacities are exceeded. The risk of design rainfall exceedance is shown in Figure 27, or can be calculated from the values provided in Table 25. The selection of an appropriate risk involves value judgments which should be made by the responsible local officials involved and applied consis-

Figure 26

THE POLLUTANT REMOVAL EFFECTIVENESS OF RETENTION BASINS IN THE GREAT LAKES AREA



Source: Driscoll, April 1983.

tently in both the public and private sectors. An acceptable risk level should be determined based on engineering judgment and experience with performance of erosion and sediment control facilities in similar circumstances.

The following criteria were used in the development of this stormwater management plan:

1. Where large amounts of settleable solids are generated, such as from construction sites, a combination of onsite source controls and sedimentation basins should be applied. Where pollutant contributions consist primarily of small clay-sized particles which resist settling or of dissolved pollutants, such as nitrates, onsite source controls should be emphasized.

2. Temporary erosion control and sedimentation measures, such as those which should be applied at construction sites during summer construction, should be designed to provide adequate protection at a 33 percent risk level--that is, on the average there is one chance in three that the structure capacity will be exceeded during its life. The following

Table 25

THEORETICAL RISK OF DESIGN STORM OCCURRENCE

Average Recurrence Interval Tr, Years	Probability That Interval Between Events Will Not Be Exceeded in Period of N Years						
	5%	10%	25%	50%	75%	90%	95%
10	29.957 yr	23.026 yr	13.863 yr	6.931 yr	2.877 yr	1.054 yr	0.513 yr
5	14.979	11.513	6.931	3.466	1.438	0.575	0.256
2	5.991	4.605	2.773	1.386	0.575	0.211	0.103
1	2.996	2.303	1.386	0.693	0.288	0.105	0.051
0.5	1.498	1.151	0.693	0.347	0.144	0.053	0.026
0.25	0.749	0.576	0.347	0.173	0.072	0.026	0.013

Based on:

$$Pn = e^{-N/Tr}$$

$$N = Tr \times LOG_e \frac{1}{Pn}$$

$$Tr = \frac{N}{LOG_e \frac{1}{Pn}}$$

Where:

Pn = Probability of nonoccurrence
N = Number of years of interest
Tr = Recurrence interval, years

Source: SEWRPC.

recurrence interval storms should be used in the design of structural construction site erosion control measures:

Construction Period Including June, July, or August

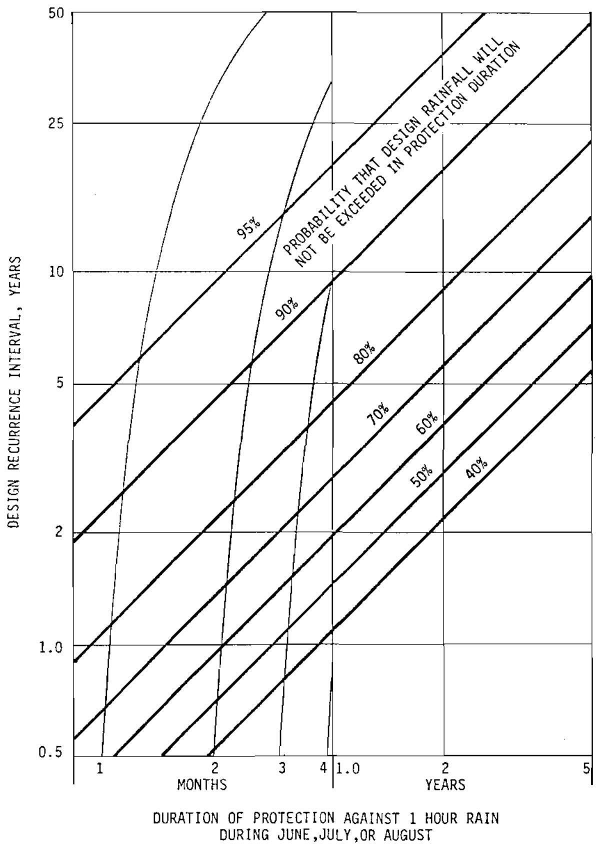
Duration of Construction During Summer	Design Recurrence Interval Storm Event With a 67 Percent Chance of Nonexceedance	One Hour Design Storm Depth (inches)
1 month	0.6 year	0.84
2 months	1.3 years	1.12
3 months	2.0 years	1.28
4 months-1 year	2.5 years	1.37

Construction during May or September, but not June, July, or August, should be designed for a 0.3-year recurrence interval event which has a corresponding one-hour depth of 0.58 inch. Construction during but not exceeding the period of October to April in general does not require structural measures for erosion and sediment control.

3. Vegetative cover should be restored as soon as possible on land disturbed for construction activity, agricultural production, and industrial uses.

Figure 27

CALCULATED RISK DIAGRAM



Source: SEWRPC.

ECONOMIC EVALUATION

It is customary to evaluate plans for water resource development projects on the basis of benefits and costs. This is particularly appropriate if the prospective development represents opportunities for investments to provide economic return to the public and if a comparison of alternative investments is desirable. In the case of stormwater management systems, however, it is assumed that such systems must be provided to fulfill a fundamental need of the community, and, consequently, they do not compete with alternatives of investment in other economic sectors. Accordingly, it is assumed that the least costly alternative system that meets the stormwater management objectives set forth in this chapter will be the most desirable alternative economically.

The economic evaluations conducted under this stormwater management planning program include capital cost estimates and annual operation and maintenance cost estimates. Capital costs include construction contract costs plus engineering, inspection, and contract administration costs. Costs for storm sewers, culverts, manholes, inlets, open channels, surface storage basins, and pumping stations are presented in Table 26.

The unit costs presented in Table 26 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, and labor costs. Any necessary land and acquisition costs were estimated utilizing the latest available state equalized assessed valuations.

SUMMARY

The process of formulating objectives and standards for stormwater management is an essential part of the planning process. To reflect the basic needs and values of the community, it is necessary that these stormwater management objectives and standards be prepared within the context of, and be fully consistent with, proposed land use conditions and broad community development objectives.

The following five stormwater management objectives were established to guide the design and evaluation of alternative stormwater management plans:

Table 26

UNIT COSTS FOR SELECTED STORMWATER MANAGEMENT COMPONENTS

Component	Description	Unit Cost
Corrugated Metal Culverts	12-inch diameter	$ 16 per lineal foot
	15-inch diameter	18 per lineal foot
	18-inch diameter	20 per lineal foot
	24-inch diameter	28 per lineal foot
	30-inch diameter	36 per lineal foot
	36-inch diameter	50 per lineal foot
	42-inch diameter	60 per lineal foot
	48-inch diameter	70 per lineal foot
	60-inch diameter	110 per lineal foot
	78-inch diameter	150 per lineal foot
Reinforced Concrete Storm Sewers--Four- to Seven-foot Cover	12-inch diameter	$ 28 per lineal foot
	15-inch diameter	32 per lineal foot
	18-inch diameter	38 per lineal foot
	24-inch diameter	50 per lineal foot
	30-inch diameter	70 per lineal foot
	36-inch diameter	85 per lineal foot
	42-inch diameter	100 per lineal foot
	48-inch diameter	120 per lineal foot
	60-inch diameter	150 per lineal foot
	72-inch diameter	200 per lineal foot
	84-inch diameter	280 per lineal foot
Manholes Five to Eight Feet Deep	For 12- to 30-inch pipe	$ 850 each
	36-inch diameter	1,050 each
	48-inch diameter	1,400 each
	60-inch diameter	1,800 each
	72-inch diameter	2,500 each
	84-inch diameter	3,400 each
Street Inlets	Standard Inlet	$ 600 each
	Inlet bowl	500 each
Open Channels	Grass-lined:	
	0 foot bottom x 5 feet deep	$ 29 per lineal foot
	6 feet bottom x 5 feet deep	36 per lineal foot
	12 feet bottom x 6 feet deep	55 per lineal foot
	18 feet bottom x 7 feet deep	75 per lineal foot
	Concrete- and grass-lined:	
	10 feet bottom x 6 feet deep	110 per lineal foot
	15 feet bottom x 7 feet deep	160 per lineal foot
	20 feet bottom x 8 feet deep	210 per lineal foot
Surface Storage Basins	Storage volume:	
	5 million gallons	$ 190,000 each
	10 million gallons	340,000 each
	20 million gallons	620,000 each
	100 million gallons	2,800,000 each
Pumping Stations	1 million gallons per day	$ 220,000 each
	5 million gallons per day	340,000 each
	10 million gallons per day	500,000 each
	25 million gallons per day	820,000 each
Maintenance	Storm sewer maintenance	$1,000 per mile per year
	Open channel maintenance	2,000 per mile per year

Source: W. G. Nienow Engineering Associates; and SEWRPC, 1984.

1. The development of a stormwater management system which reduces the exposure of people to drainage-related inconvenience and to health and safety hazards, and which reduces the exposure of real and personal property to damage through inadequate stormwater drainage and inundation.

2. The development of a stormwater management system which will effectively serve existing and proposed future land uses.

3. The development of a stormwater management system which will minimize soil erosion, sedimentation, and attendant water pollution.

4. The development of a stormwater management system which will be flexible and readily adaptable to changing needs.

5. The development of a stormwater management system which will efficiently and effectively meet all of the other stated objectives at the lowest practicable cost.

Complementing each of the foregoing stormwater management development objectives is a set of quantifiable standards which can be used to evaluate the relative or absolute ability of alternative stormwater management plan designs to meet the objective.

In addition to presenting and discussing the objectives and standards established for the Hales Corners stormwater management plan, this chapter presents the engineering design criteria and analytic procedures which were used to design and size the alternative plan elements and which will serve as a basis for the more detailed design of stormwater management system components. Criteria and procedures were developed for estimating stormwater flow rate and volume and for designing street cross-sections, swales, culverts, storm sewer inlets, storm sewers, open channels, storage facilities, pumping facilities, and water quality management measures.

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Chapter VI

STORMWATER MANAGEMENT SYSTEM COMPONENTS

INTRODUCTION

A stormwater management system plan seeks to combine certain drainage system components in an efficient manner which will effectively meet the system objectives. This chapter describes, to the extent required for system planning purposes, six stormwater management system components, and the function of these components within a stormwater management system. Each component or element is described, its purpose identified, and its relationship to the overall stormwater management system discussed.

SYSTEM COMPONENTS

Urban stormwater management systems may be thought of as consisting of three basic components: collection, conveyance, and storage. Such systems may include two additional components--treatment and nonpoint source water pollution control. In addition, overland flow must be considered in the design of the system, as such flow will affect the amount and quality of the runoff reaching the system proper. Accordingly, overland flow is herein considered as a sixth basic component of the overall stormwater management system.

Overland Flow

When precipitation and snowmelt occur in amounts that exceed the capacity of the ground surface to absorb it, the stormwater first accumulates on the ground surface, filling the depression storage, and then begins to flow down slope. In an area served by a traditional urban stormwater management system, this overland flow carries the stormwater runoff to a collection facility. Thus, overland flow serves to concentrate stormwater from its initially more diffuse form. In an urban area, the pattern of overland flow is determined by the siting of buildings and the grading of the surrounding sites, so that such siting and grading becomes an important part of the design of the stormwater management system. Proper siting and grading of buildings is important in order to provide proper drainage and to provide access to and from buildings during and after foreseeable rainstorm and snowmelt events.

Overland flow may develop relatively high velocities if it occurs over smooth surfaces such as rooftops or paved driveways or parking lots, or relatively low velocities if it occurs over rough surfaces such as vegetated areas. In addition, stormwater may either accumulate pollutants as overland flow occurs, such as in flow across a paved parking lot; or actually lose pollutants, such as in flow over a vegetated area where sediment may be precipitated.

Urbanization generally entails a conversion of rough vegetated surfaces with water- and pollutant-absorbing and energy-dissipating characteristics to smooth paved surfaces with significantly reduced water-absorbing and energy-dissipating characteristics. This change in surface configuration will produce a greater quantity and generally a lower quality of stormwater at higher velocities for a given storm. Thus, following urbanization it is necessary to significantly improve natural drainage systems by providing artificial stormwater collection and conveyance facilities.

Overland flow is an important component of the overall stormwater management system, and has a direct and significant relationship to several of the overall system objectives. Overland flow patterns in urbanizing areas should be designed to maximize the inlet time of stormwater runoff without adversely affecting urban structures or interrupting human activities. Thus, while providing adequate urban drainage, overland flow patterns should be designed to minimize the total volume of stormwater runoff by allowing maximum infiltration of the stormwater; to reduce the peak rate of discharge of stormwater to the collection and conveyance facilities; and to reduce the velocity of overland flow, thereby reducing the energy level of flowing stormwater and its ability to disturb sediment particles and surface pollutants.

The velocity of overland flow can be controlled by minimizing the amounts of paved surfaces and, where possible, draining paved surfaces to pervious grassed areas rather than directly to paved gutters. Various detention and retention storage techniques are also effective in reducing the velocity of overland flow. Such systems are discussed later in this chapter. These management techniques can also reduce the overall volume of stormwater runoff by increasing infiltration and thereby reducing downstream stormwater management requirements.

Because overland flow has a broad impact on the overall system objectives, it was considered to be an important and essential component of the stormwater management system for the Hales Corners area. Specific arrangements for overland flow, however, cannot be addressed at the systems level of planning. The design of such arrangements must be done on a site-specific basis as urban development or redevelopment takes place. Overland flow was considered in the systems planning process, however, through the development of the general guidelines set forth in Chapter V, which includes a description of practical techniques for minimizing the rate and volume of runoff. In the evaluation of alternative stormwater management systems, it was assumed that these general guidelines will be followed to the extent practicable.

Collection

Stormwater collection is the process of further concentrating stormwater flowing overland and transmitting it to conveyance facilities. Stormwater collection facilities may include drainage swales, roadside swales, roadway gutters, stormwater inlets, and inlet leads in which stormwater is collected and then transmitted to surface or subsurface conveyance systems.

The stormwater collection system may also provide some conveyance and storage functions in the stormwater management system. For minor precipitation events, drainage swales, roadside swales, and roadway gutters collect and transmit stormwater to the stormwater conveyance facilities. Subsurface conveyance facilities--storm sewers--are designed to accommodate minor runoff events only, constituting the minor conveyance system referred to in Chapter V. During major runoff events, the stormwater collected will, by design, exceed the capacity of the subsurface conveyance facilities, with the excess stormwater being temporarily stored on and conveyed over collector and land access roadways, and interconnected surface drainageways--the major conveyance system, also referred to in Chapter V.

Drainage Swale: A stormwater drainage swale is a small depression or valley in the land surface. The purpose of a drainage swale is to collect overland flow from areas such as front-, side-, and backyards and transmit it to larger, open stormwater drainage channels or to subsurface conveyance facilities. Drainage swales are generally grass lined, but may be paved to prevent erosion on steep slopes, or to avoid standing water on flat slopes. A typical drainage swale is shown in Figure 28.

Drainage swales cannot be specifically addressed at the systems level of planning. The design of such components must be done on a site-specific basis as urban development or redevelopment takes place. The design of swales, then, like the design of overland flow, is considered in the systems planning process using as guidelines the criteria provided in Chapter V for detailed design.

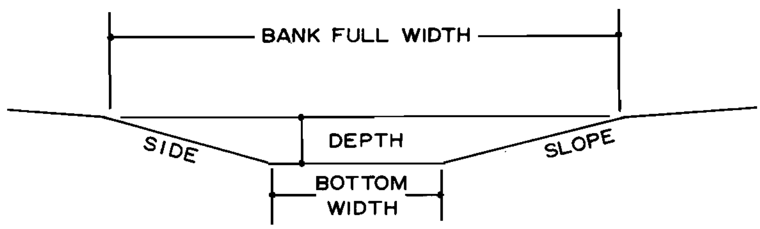
Roadside Swale: A roadside swale is a long, narrow, shallow depression or valley running parallel and adjacent to a roadway providing longitudinal drainage. Roadside swales in urban areas are generally grass lined, but also may be paved to prevent erosion on steep slopes, or to avoid standing water on flat slopes. The roadside swale can serve as either a collection component or a conveyance component of the stormwater management system. A typical residential roadway and swale combination is shown in Figure 28. The swale collects stormwater runoff from the roadway surface and the tributary overland flow areas of abutting lands. The collected stormwater is then transmitted to open channel or subsurface conveyance facilities. Roadside swales are generally less expensive than curb-and-gutter collection systems. They also provide lower runoff velocities and can provide for stormwater infiltration and for storage capacity. Nonpoint source water pollution loadings carried by stormwater are generally reduced as flows are collected in swales. More importantly, through the use of roadside swales, stormwater runoff can be managed entirely in a surface drainage system, and the construction of storm sewers can be avoided. Such surface drainage systems are most practical in areas developed at relatively low densities since each intersecting private driveway, as well as public roadway, must be provided with a culvert pipe to carry the drainage. As densities increase, lot areas and widths decrease and front yard setbacks decrease, and a point is reached where the provision of a storm sewer becomes more economical, desirable, and maintainable than the provision of roadside swales and culverts. The use of roadside swales provides a "rural," "suburban," or "estate" appearance and is desired by some communities for this reason.

Recommendations for the typical shape, alignment, and type of roadside swale are provided in this stormwater management system plan. Additional details and refinement must be addressed on a site-specific basis during the detailed design phase preceding construction. Criteria are provided in Chapter V as guidelines for the detailed design of all drainage swales which are to be an integral part of the stormwater drainage system. Typically, these roadside swales are designed using open channel flow hydraulic equations such as Manning's equation and consider such variables as: an allowable depth of flow in

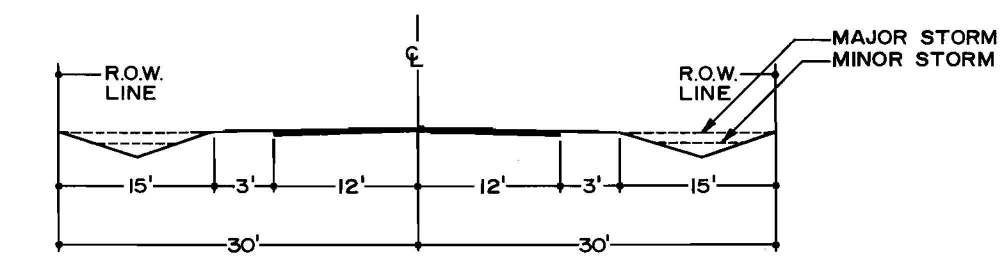
Figure 28

TYPICAL SWALE AND ROADWAY CROSS-SECTIONS
SHOWING WATER COLLECTION AREAS

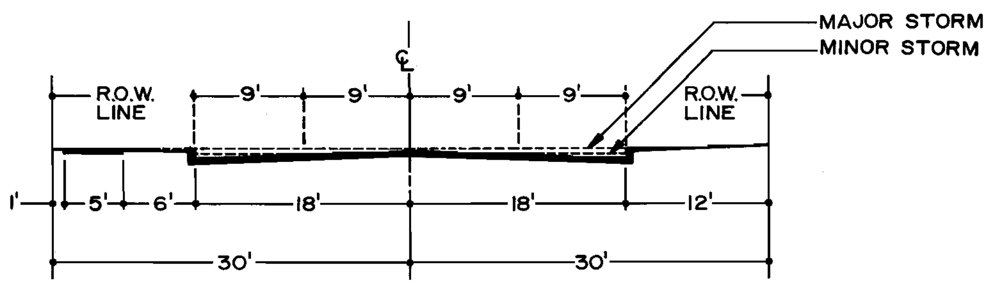
DRAINAGE SWALE



ROADWAY WITH ROADSIDE SWALE



ROADWAY WITH CURB AND GUTTER



Source: Village of Hales Corners and SEWRPC.

each area to prevent unacceptable velocities and damage to facilities and adjacent land uses; available slope; and available right-of-way. Under some conditions, as, for example, very close driveway culvert spacing or minimum longitudinal gradient, culvert headwater elevations and entrance losses may dictate the design. In areas with limited right-of-way, a rectangular, concrete-lined channel may be required. In other reaches, the channel can more typically be triangular or trapezoidal in shape with grassed bottom and side slopes. In areas of minimum longitudinal gradient, a paved channel bottom may be necessary. The stormwater management plan assumes the use of roadside swales with a cross-section similar to that shown in Figure 28 in certain areas of the Village.

Roadway Gutters: A roadway gutter is a depression in the roadway surface adjacent to the curb line. A typical residential roadway configuration with curb and gutter is shown in Figure 28. The roadway gutter collects stormwater from the roadway surface and from the tributary overland flow areas of abutting lands. The collected stormwater is typically discharged from the roadway gutters into stormwater inlets or catch basins that transmit the stormwater to subsurface conveyance facilities. Curbs and gutters are required in higher density urban areas where the use of roadside swales and culverts becomes impractical. Curbs and gutters reduce the potential for stormwater infiltration, increase stormwater runoff flow velocity, and limit the removal of non-point source water pollution loadings.

The design of roadway gutters, however, cannot be specifically addressed at the systems level of planning. Such design must be addressed during the detailed design phase preceding construction. The stormwater management plan assumes the use of a typical roadway cross-section with curb and gutter similar to that shown in Figure 28 in certain areas of the Village.

Stormwater Inlets: The stormwater inlet is a device through which stormwater is transmitted from the surface collection facilities to subsurface conveyance facilities. Stormwater inlets are placed at strategic locations along drainage swales, roadside swales, and gutters for the purpose of transmitting collected stormwater into subsurface conveyance facilities. Typical stormwater inlet structures are shown in Figure 29. The inlet structure includes a stormwater grate, drop structure, and connection to the underground conveyance facility.

The three basic types of inlets commonly used in stormwater management systems are:

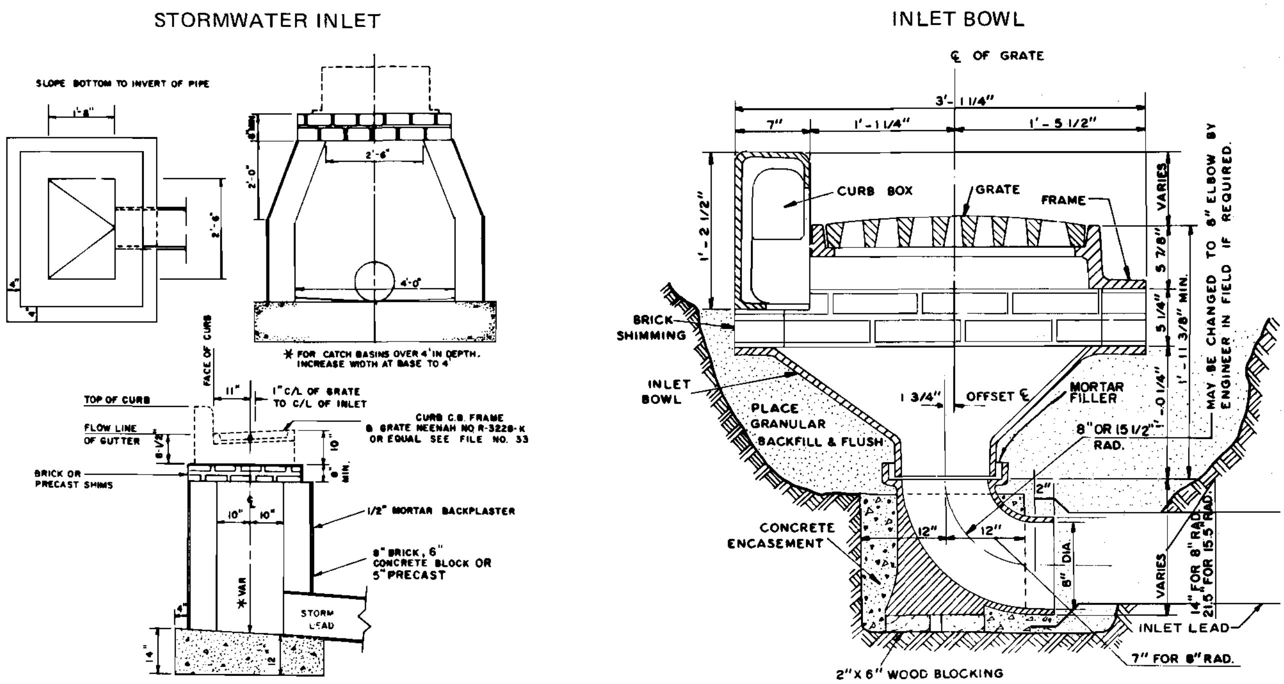
1. The curb inlet, which consists of a relatively large, vertical opening in the curb face extending up from the base of the curb face or gutter line through which stormwater can flow (see Figure 30).

2. The gutter inlet, which consists of an opening in the roadway gutter that is covered by a cast iron grate (see Figure 30). Stormwater is allowed to flow into the gutter inlet while sticks and large debris are trapped by the iron grate, which also prevents pedestrian, cycle, and vehicular traffic from dropping into the inlet.

3. The combined curb inlet and gutter inlet, which is referred to as a combination inlet (see Figure 30).

Figure 29

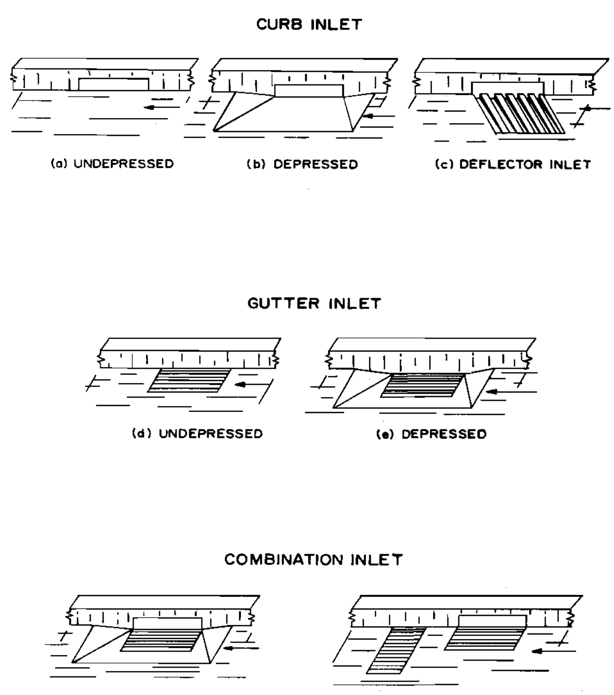
TYPICAL STORMWATER INLET STRUCTURES



Source: Standard Specifications for Sewer and Water Construction in Wisconsin, Second and Third Editions.

Figure 30

TYPICAL STORMWATER INLET DESIGNS



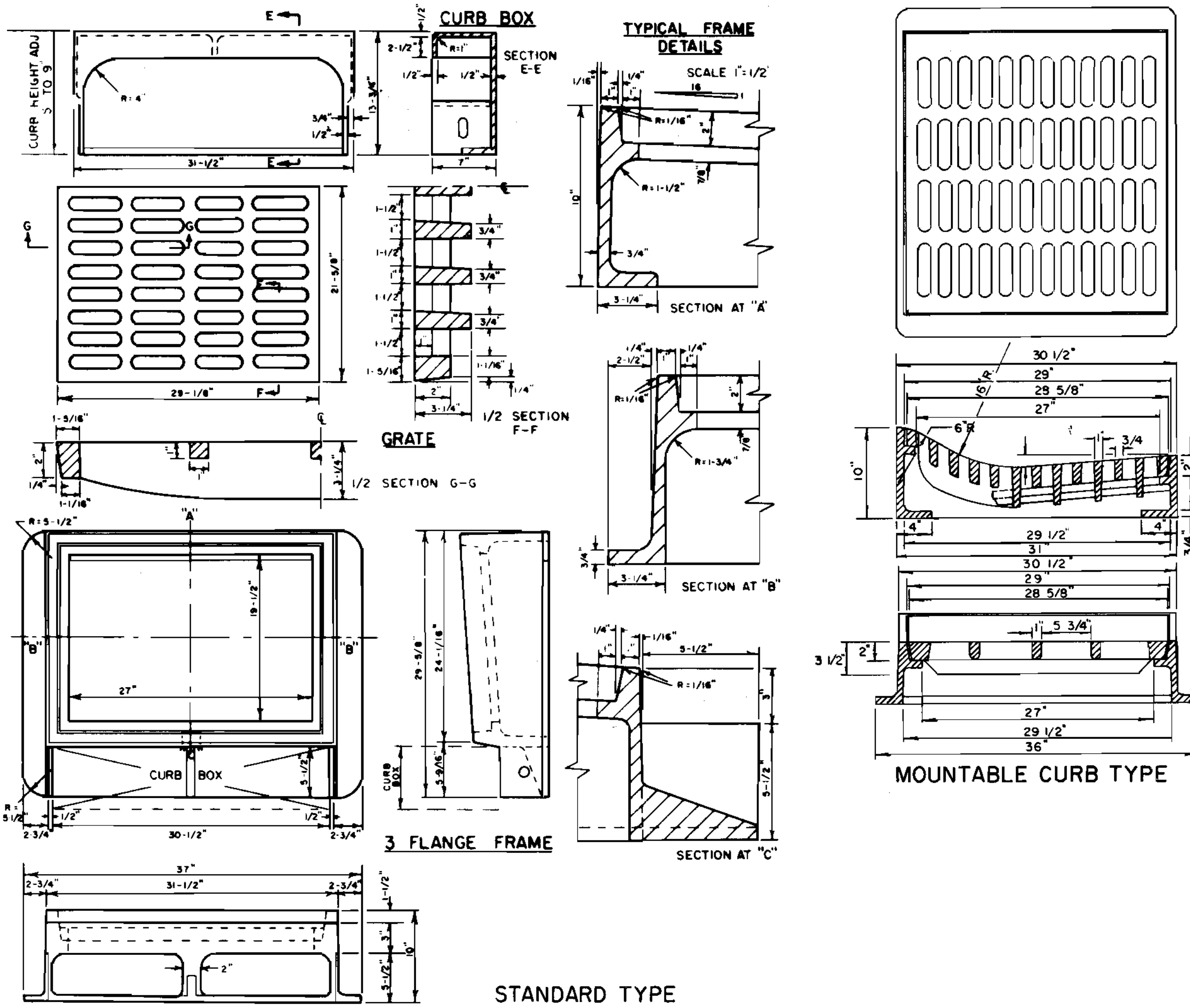
Source: American Society of Civil Engineers.

Many variations of these basic inlet designs are used in stormwater management systems. For example, the three basic inlet types may be either set at grade in the gutter line (undepressed inlet) or set slightly below grade in the gutter line (depressed inlet), which improves hydraulic efficiency and gutter flow capture. Inlet grate types are shown in Figures 31 and 32.

Catch Basin: A catch basin is defined as a stormwater inlet equipped with a small sedimentation basin or grit chamber. The purpose of a catch basin is to remove sediment and debris from stormwater before it is transmitted to the subsurface conveyance facilities. A typical catch basin is shown in Figure 33. Stormwater enters through the surface inlet and drops to the lower basin area. Heavy sediment particles and other debris are collected in the basin area. This debris is then removed during maintenance operations. The catch basin is designed to reduce the mainte-

Figure 31

TYPICAL INLET FRAME AND GRATE FOR A STANDARD OR MOUNTABLE CURB SECTION



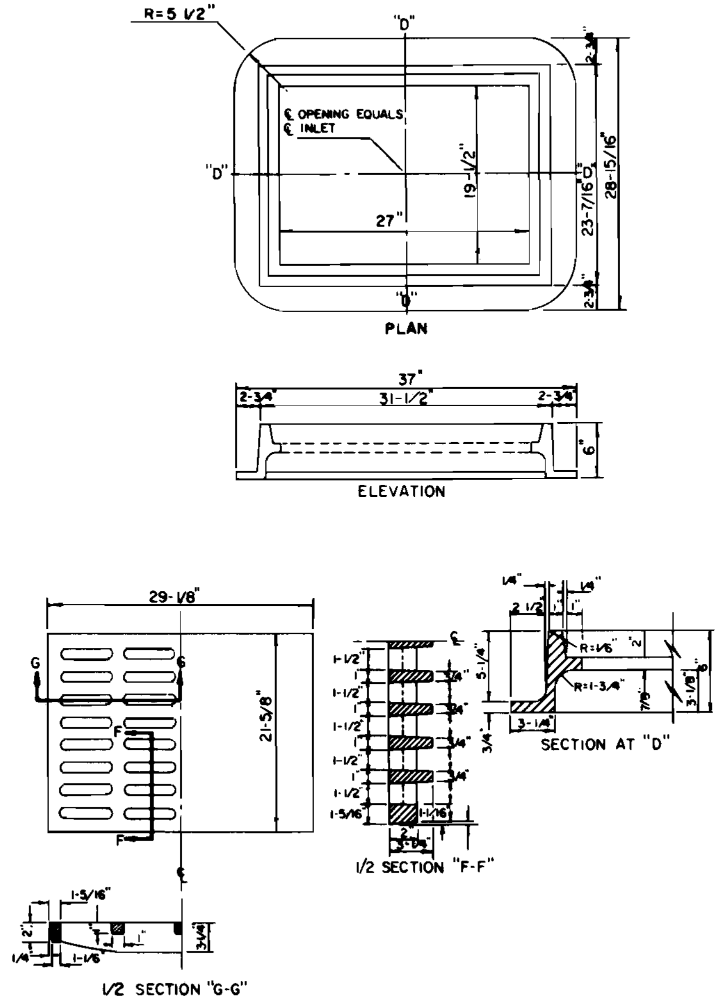
Source: Standard Specifications for Sewer and Water Construction in Wisconsin, Fourth Edition.

nance requirements for the underground conveyance system, particularly in areas where heavy sediment loads may otherwise be carried into the conveyance system. Catch basins also provided a form of nonpoint source water pollution abatement in the period before the automobile, when large quantities of horse manure were deposited on street surfaces. The use of catch basins fell into disfavor because of the cost of the periodic cleaning required. Nonpoint source pollution abatement, however, may warrant the reintroduction of the catch basin in urban areas.

If properly maintained, the catch basin has been known to be an effective sediment trap. Improperly or inadequately cleaned catch basins may have a negative impact on receiving water quality. Decaying organic material trapped in the basin may produce noxious odors or the basin water may become rich in organic material and nutrients and low in dissolved oxygen content. This basin water becomes a part of the first flush of stormwater from subsequent storm

Figure 32

TYPICAL INLET FRAME AND GRATE
FOR FLAT SURFACE APPLICATION



Source: Standard Specifications for Sewer and Water Construction in Wisconsin, Fourth Edition.

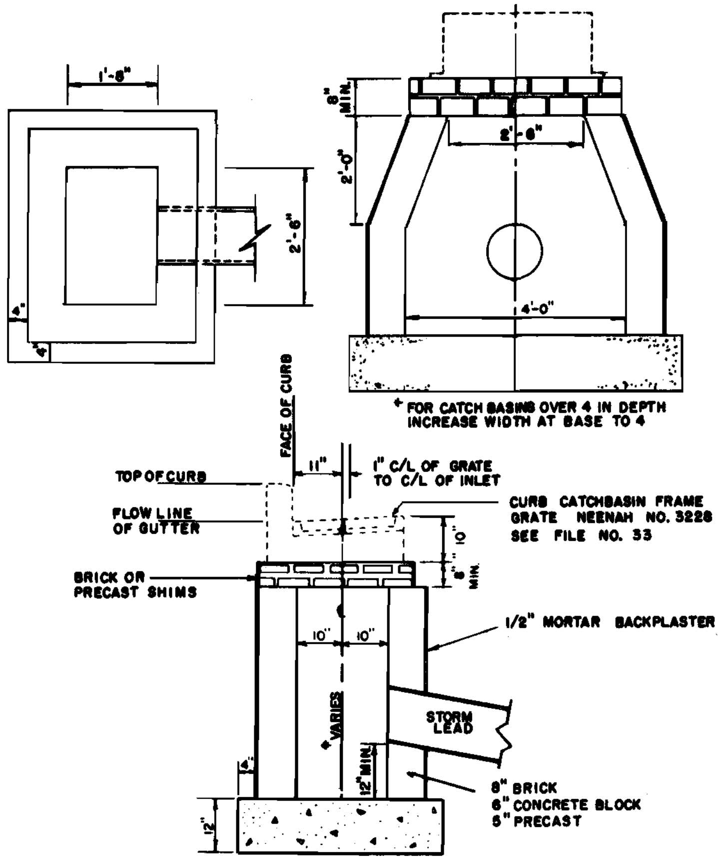
events. Basin waters may also provide a place for mosquitoes to breed. Accordingly, under most circumstances, catch basins are not considered to be beneficial components of the overall stormwater management system.

Recommendations for the location of inlets and catch basins are provided in the stormwater management plan. The inlet type, flow capacity, related street grades, types of street crowns, and expected depth of flow must be addressed in subsequent engineering for project development.

Collection Elements Applicable to the Village of Hales Corners Stormwater Management System: The general policy of the Village of Hales Corners is to provide roadside swales for the collection of stormwater in residential areas. There are also some streets with an "urban" cross-section, including full curb and gutter and storm sewers, within the Village--chiefly arterial highways through commercial areas. These include W. Janesville Road (STH 24), S. 108th Street (STH 45), portions of W. Forest Home Avenue (STH 24 and CTH OO), and

Figure 33

TYPICAL CATCH BASIN



Source: <u>Standard Specifications for Sewer and Water Construction in Wisconsin</u>, Fourth Edition.

portions of streets adjacent to these major highways. In the preparation of the stormwater management plan, continued use of street cross-sections similar to those which currently exist in the various areas of the Village was assumed except where specific plans were in place which provide for a change in the type of street cross-section. In such cases, the planned street cross-section will be used in the design of alternative stormwater management plans. In addition, consideration was given in the system plan development to potential changes in the type of street cross-section if such changes were found to be necessary in order to solve existing and probable future drainage problems. Finally, the impact on the stormwater drainage system of a change from a "rural" street cross-section to a "suburban" cross-section--such a section utilizing mountable curbs, gutters, inlets, and storm sewers within certain sections of the Village developed to higher densities--was evaluated. This evaluation was conducted to identify the system components needed to provide flexibility to change the type of street cross-section in certain areas of the Village in the future. It was assumed in the planning effort, however, that except in clearly identified and isolated cases, rural street cross-sections with roadside swales would be used for stormwater collection, along with swales for stormwater conveyance.

Conveyance

Conveyance facilities are normally the most costly component of the stormwater management system. The conveyance components of a stormwater management system may include both open channels and subsurface conduits--storm sewers--designed to receive and transport stormwater runoff from or through urban areas to a receiving stream or watercourse. Stormwater conveyance facilities may also be used to transport nonpolluted wastewaters, such as spent industrial cooling waters.

In most urban settings it is not possible to maintain the natural stormwater conveyance system because of the increase in the volume and rate of stormwater runoff attendant to the conversion of land from rural to urban use. In addition, land filling and drainageway excavation are frequently required to facilitate the use of land and roadways unencumbered by stormwater. Therefore, significant modifications are usually made to the natural drainage system to meet the increased stormwater conveyance and increased vertical separation requirements.

Open Channel Conveyance: Open channel conveyance facilities generally follow the natural surface drainage pattern. In some instances, the natural channel configuration can be maintained with only minor modifications such as removal of obstructions and reducing the overall channel roughness. In certain areas it may be necessary to "improve" the existing channel by widening, deepening, and realigning, or to construct an entirely new channel, in order to provide the required conveyance capacity. Man-made open channel conveyance facilities may be grass lined, concrete lined, or composite lined, depending on the need to prevent erosion or avoid standing water. Typical open channel cross-sections are shown in Figure 34.

When compared to subsurface storm sewer conveyance facilities, open channel surface conveyance facilities are generally less costly for high flow rates; provide a greater degree of nonpoint source water pollutant removal; and are more adaptable to providing inline storage. Grass-lined conveyance facilities reduce the overall velocity of stormwater runoff, reduce the peak discharge rate from the drainage basin, and allow stormwater to recharge the groundwater reservoir. Open channel conveyance facilities, if poorly designed, may be aesthetically less desirable; may constitute a safety hazard; and may have higher maintenance requirements than storm sewer conveyance facilities.

Recommendations relating to the shape, alignment, and type of open channel conveyance facilities are included in the stormwater management plan. Refinement of these details must be addressed in the detailed design phase prior to construction. Criteria for design of open channels are provided in Chapter V. Typically, the channels are designed using appropriate open channel flow hydraulic formulas, such as the Manning's equation, with careful consideration given to allowable grades and depths of flow to prevent unacceptable velocities and damage to the facilities and adjacent land uses. In areas with limited right-of-way, a trapezoidal or rectangular concrete-lined channel may be required. In more open areas the channel is more typically trapezoidal, and either turf lined or composite turf and concrete lined.

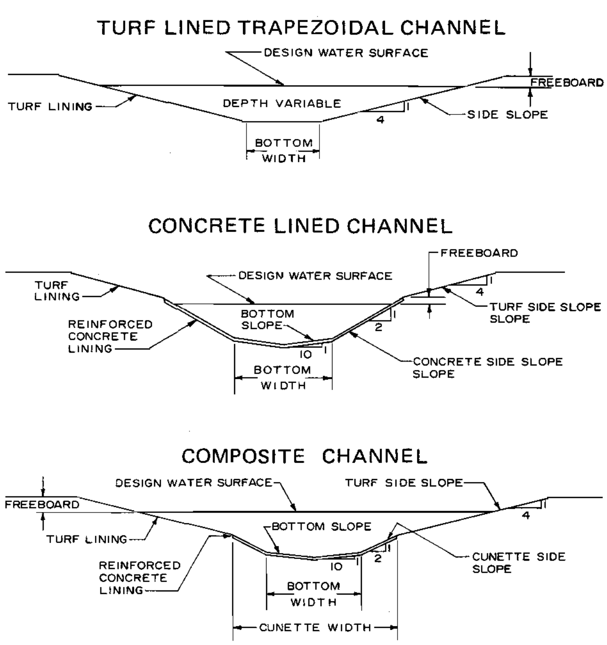
Culverts: A culvert is a closed conduit used to convey stormwater under a street, highway, railway, or other embankment. Culverts are a common and hydraulically important feature of open channel drainage systems.

The locations and sizes of existing and proposed culverts in the Village of Hales Corners and environs are set forth in the stormwater management system plan. The hydraulic capacity of any culvert is determined by its cross-sectional area, shape, entrance geometry, length, slope, and construction material, and the depth of ponding at the inlet to (headwater) and outlet from (tailwater) the structure. Culvert flows are classified as having either inlet or outlet control--that is, according to whether the discharge capacity is controlled by the inlet or outlet characteristics. Typical inlet control and outlet control culvert conditions are shown in Figure 35. Under inlet control conditions, the discharge capacity of a culvert is controlled at its entrance by the depth of headwater, the entrance shape and cross-sectional area, and the type of entrance edge. Under outlet control conditions, the discharge capacity of a culvert is influenced by the headwater depth, tailwater depth, entrance shape and cross-sectional area, and type of entrance edge, and by the cross-sectional area, shape, slope, length, and roughness of the culvert barrel.

Storm Sewer Conveyance: A storm sewer is defined as an underground conduit that transports stormwater runoff from collection facilities to an ultimate point of disposal. The purpose of a storm sewer is to receive stormwater runoff from stormwater inlets and catch basins, and convey that runoff to surface water drainage facilities. The storm sewer provides a rapid conveyance

Figure 34

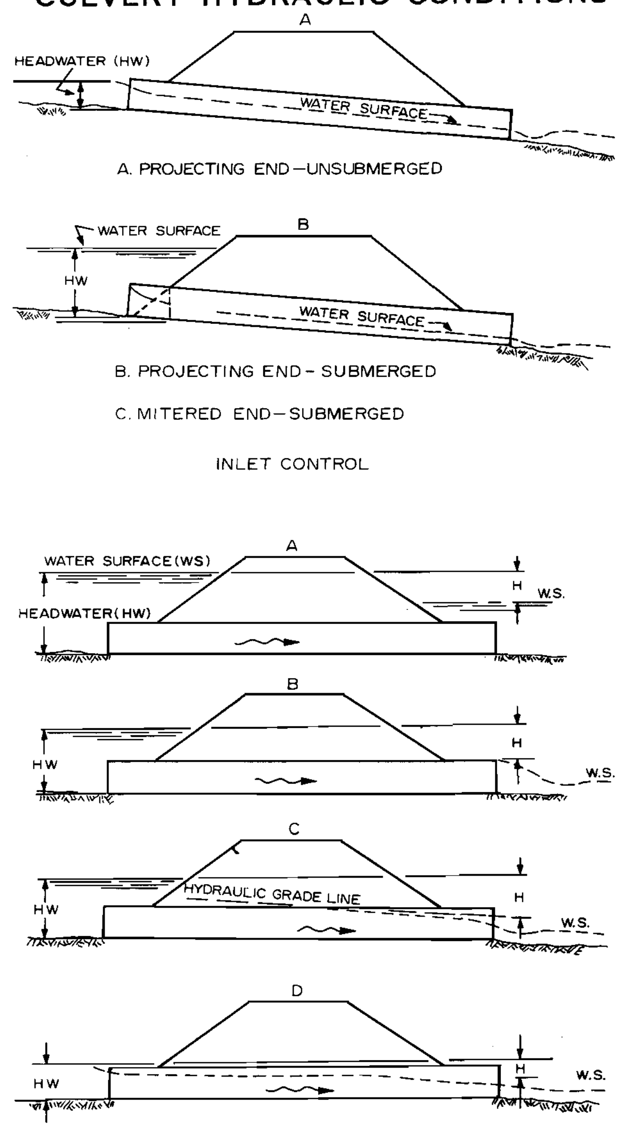
TYPICAL OPEN CHANNELS



Source: SEWRPC.

Figure 35

CULVERT HYDRAULIC CONDITIONS



Source: American Iron & Steel Institute, Handbook of Steel Drainage and Highway Construction Products.

route for stormwater to a point of disposal on a receiving surface watercourse. Subsurface storm sewer systems are generally more costly to construct than surface conveyance facilities; however, they are often required in order to meet overall stormwater management system objectives.

Prefabricated Portland cement concrete pipe is the most commonly used material for the construction of storm sewers in the Region. Concrete pipe is available in standard lengths ranging from four feet to eight feet and in circular, elliptical, and arch pipe sections, with circular sections ranging from six inches to 108 inches in diameter. Nonreinforced concrete pipe is available in diameters ranging from six inches to 18 inches, while reinforced concrete pipe is available in diameters ranging from 12 inches to 108 inches. Fittings for concrete pipe such as wyes, tees, and manholes are readily available. Concrete provides a high-strength, widely used and accepted storm sewer pipe. Prefabricated galvanized steel pipe such as corrugated metal pipe and corrugated metal pipe arch is also commonly used in stormwater management systems. The most common application of these materials is in culvert installations, but in some cases corrugated metal pipe is used for storm sewer construction. Corrugated metal is light weight, strong, and flexible and is manufactured in generally longer lengths than is concrete pipe. It is more difficult to connect inlets to corrugated metal pipe.

Other pipe materials such as asbestos-cement pipe, vitrified clay pipe, ductile iron pipe, welded steel pipe, and plastic pipe are also available. These materials are not commonly used for gravity flow storm sewers in the Region. There are limited applications for asbestos-cement pipe, ductile iron pipe, and plastic pipe as pressure stormwater conveyance facilities.

Recommendations for the alignment, depth, size, slope, and type of storm sewer facilities are provided in the stormwater management system plan. The recommendations will require refinement in the detailed design phase prior to construction, as will determination of such details as the relative location of stormwater management facilities in relation to other underground utilities. It is recommended, however, to the extent practicable, that stormwater management facilities be located generally as shown in Figure 36. Because the storm sewers may be provided within the Village either as replacements or as supplements to the existing roadside swales, they may require special locational consideration. Criteria for the hydraulic design of storm sewers are provided in Chapter V.

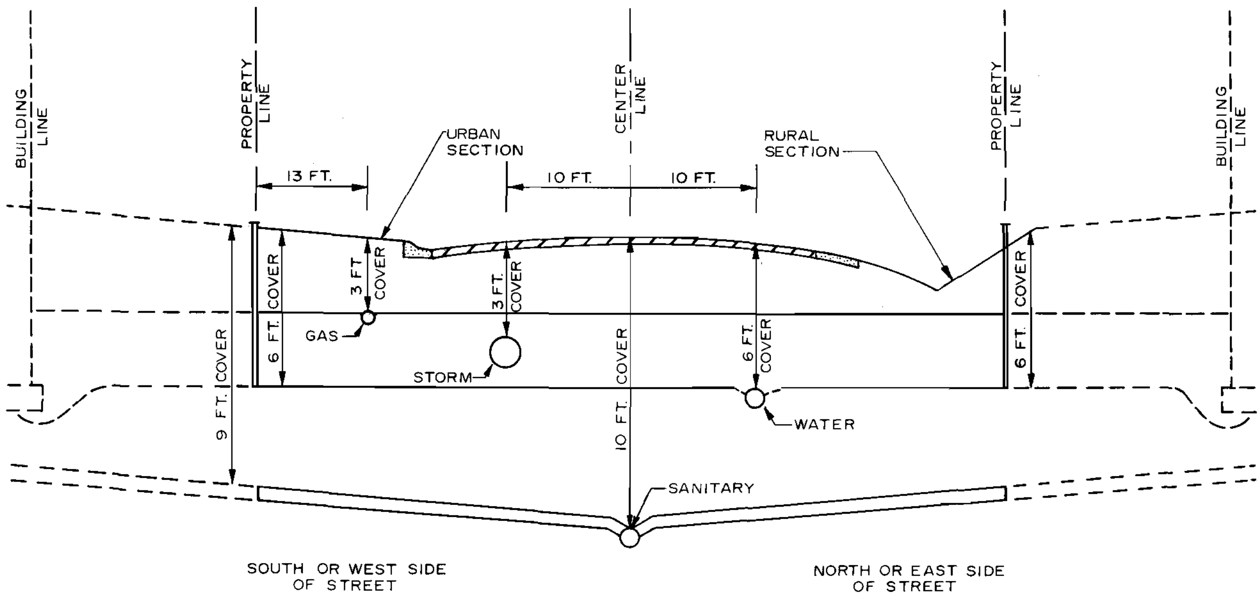
Typically, storm sewers are designed to flow full under gravity conditions using hydraulic formulas such as Manning's equation and considering the available elevation differential at control points within the system. A minimum storm sewer size of 12 inches in diameter was assumed.

Stormwater Pumping Stations: A stormwater pumping station is a mechanical device that lifts and transports stormwater under pressure. The purpose of a stormwater pumping facility is to remove stormwater from a low-lying area that cannot be effectively drained by gravity. Stormwater pumping stations are commonly associated with stormwater storage facilities that have limited land surface available and therefore require deep storage. This type of storage design requires the use of mechanical pumping to fully evacuate storage areas.

Pumping stormwater from storage areas is less dependable and more costly than gravity drainage. Electrical service can suffer service interruptions, especially during thunderstorm activity. Maintenance of stormwater pumping

Figure 36

SUGGESTED UTILITY LOCATIONS IN THE VILLAGE OF HALES CORNERS



Source: Village of Hales Corners and SEWRPC.

facilities is a significant concern since these facilities require periodic inspection and maintenance. Where deep storage is required, or where there is not sufficient grade to provide adequate gravity drainage, pumped discharge is necessary.

Pumping was not included in the recommendations for the Village of Hales Corners stormwater management plan.

Manholes: A manhole is a structure which provides an access way to underground sewers. The purpose of a storm sewer manhole is to provide access to the storm sewer system for observation and maintenance purposes. Manholes are typically placed at all junctions in the sewer system and from 300 to 600 feet apart along the sewers. Smaller size sewers are normally laid in straight lines between manholes; larger sewers may be laid on curves. Greater spacing distances are allowable for sewers large enough to allow entrance by maintenance personnel. Junctions for smaller size storm sewers can be accommodated within ordinary manholes. Larger sewers, however, may require special junction chambers to provide a connection. A typical storm sewer manhole design is shown in Figure 37.

Recommendations for the locations and spacing of manholes are provided in the stormwater management plan. The type of manhole is a local design consideration which does not significantly affect the system plan.

Junction Chambers: A junction chamber is a structure which both provides access to an underground sewer and accommodates major changes in the size, and junctions of, storm sewers. Junction chambers are intended to provide a large

Figure 37

TYPICAL STORM SEWER MANHOLE

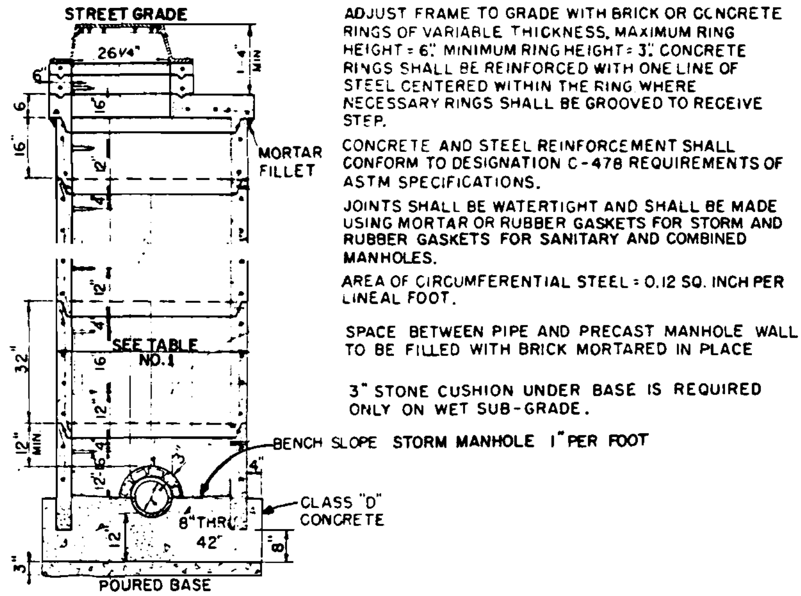


TABLE NO. 1

PIPE DIA.	MANHOLE DIA.	WALL THICKNESS
8" THUR 27"	3'-6"	4 1/2"
30"	4'-0"	5"
36"	5'-0"	6"
42"	6'-0"	7"

Source: Standard Specifications for Sewer and Water Construction in Wisconsin, Fourth Edition.

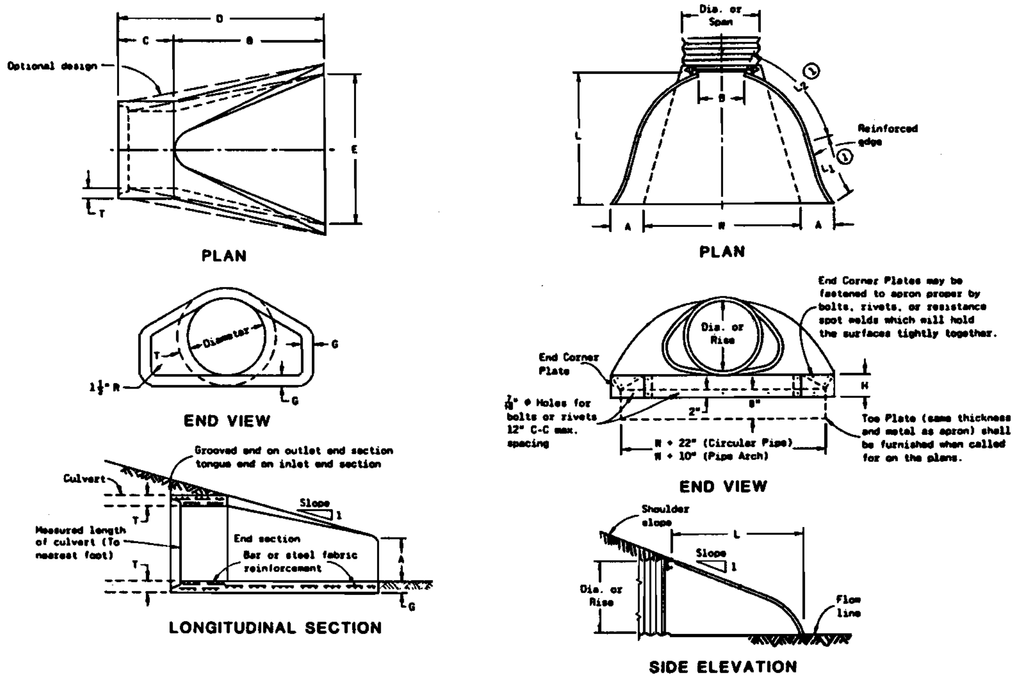
underground structure to accommodate conduit size and direction changes. Typically, they are unique cast-in-place, reinforced concrete vaults.

The approximate locations of junction chambers in the Village of Hales Corners and environs are set forth in the stormwater management plan. The type of junction chamber is dependent upon the sewer sizes and alignment conditions at each point in the system. Accordingly, the details of any proposed junction chamber must be determined in the detailed design phase preceding construction. Design criteria for junction chambers are set forth in Chapter V.

Conduit End Structures: A conduit end structure is a structure used to make the transition between a culvert or storm sewer and a swale, channel, or other surface watercourse. The primary purpose of an end structure is hydraulic control and efficiency. This includes preventing scour before the pipe inlet and scour and undermining beyond the pipe outlet, and providing a hydraulically efficient pipe entrance. Conduit end structures also provide structural support for the pipe end and stabilization and protection of the embankment slope. The end structure provides protection from and dissipation of the excess energy due to the velocity change and turbulence associated with these flow transitions. Typical end structures, also called apron endwalls for smaller size culverts and storm sewers, are shown in Figure 38. Larger size pipe, multiple pipe, or more complex installations may require an end structure such as shown in Figure 39.

Figure 38

APRON ENDWALLS



Source: Wisconsin Department of Transportation Facilities Development Manual.

The approximate locations and types of end structure in the Village of Hales Corners planned urban area are set forth in the stormwater management plan. The details of any end structure must be determined on a site-specific basis in the detailed design phase preceding construction. Design criteria are set forth in Chapter V.

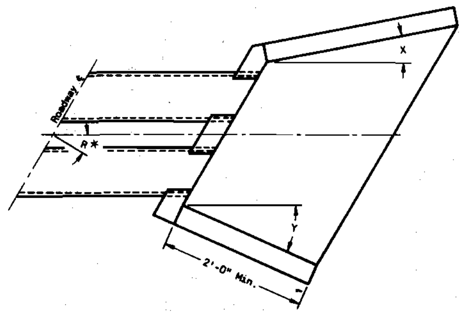
Storage

Stormwater storage can be defined as both the temporary detention and the long-term retention of stormwater within the system. The primary purpose of stormwater storage is to reduce the peak stormwater discharge rates both within the stormwater management system itself and in the receiving waterways. Stormwater storage also allows greater infiltration of stormwater, recharging the groundwater reservoir, reduces flow velocity and thus the potential for stream erosion, enhances the removal of sediment and nutrients suspended in stormwater, and usually reduces the cost of downstream stormwater conveyance and flood control facilities.

In order to reduce the cost of conveyance facilities, full advantage must be taken of means to reduce peak flow in the overland flow and collection system components. Larger cost savings are usually associated with reducing the size of new underground storm sewers, which negates the need to re-lay and increase the size of existing storm sewers, and the need to lay parallel relief sewers,

Figure 39

END STRUCTURES



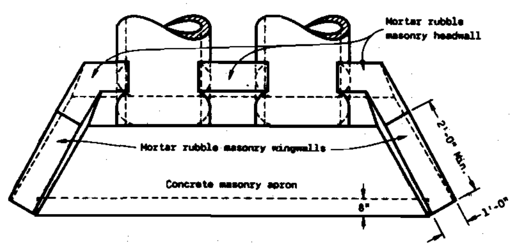
WINGWALL ANGLE DETAILS

INLET			OUTLET		
R*	X	Y	R*	X	Y
0 - 7°	30°	30°	0 - 15°	15°	15°
8 - 22°	25°	"	16 - 45°	10°	"
23 - 37°	20°	"	46 - 75°	5°	"
38 - 52°	15°	"	over 75°	0°	"
53 - 67°	10°	"			
68 - 82°	5°	"			
over 82°	0°	"			

*R = Number of degrees right or left hand forward.

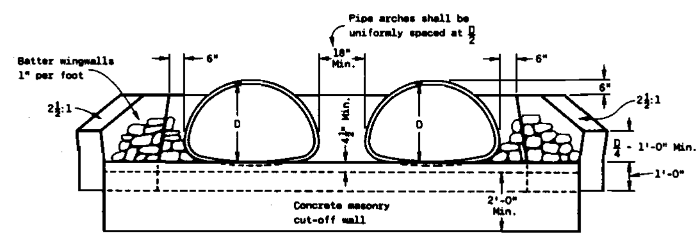
PIPE SIZE (D)	FRONT FACE ("F")	BACK FACE ("B")	"W"
24" - 48"	6" Per ft.	6" Per ft.	1.0'
60" C.M.P. 60" R.C.C.P.	6" Per ft.	3" Per ft.	2.2' 2.4'
72" C.M.P. 72" R.C.C.P.	6" Per ft.	3" Per ft.	2.4' 2.7'

All endwalls for pipe arches shall have a 6" per ft. slope on front and back face.



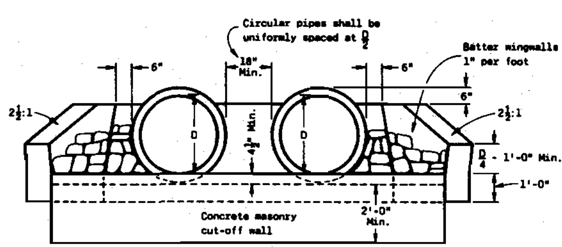
PLAN VIEW

CIRCULAR PIPE AND PIPE ARCH



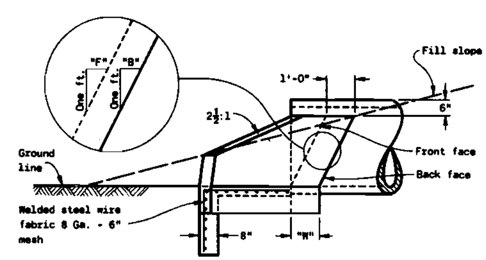
END ELEVATION

PIPE ARCH



END ELEVATION

CIRCULAR PIPE



SIDE ELEVATION

CIRCULAR PIPE AND PIPE ARCH

Source: Wisconsin Department of Transportation Facilities Development Manual.

and avoids the need to enlarge downstream bridges and culverts spanning waterways. Smaller cost savings are usually associated with the construction of smaller capacity open channels. When related to larger bridges, culverts, and storm sewers, the costs of conveyance facilities are largely site-independent, since they relate to the market prices of concrete, steel, and labor. However, the construction costs normally associated with stormwater storage are primarily for excavation and surface restoration, both very site-dependent.

Stormwater storage may be either natural or man-made. In an undisturbed setting, natural stormwater storage areas normally exist. Stormwater is stored in natural surface depressions, in wetlands, on floodplains, and in soils. These natural storage areas dispersed throughout a drainage area serve to significantly reduce the volume and rate of stormwater runoff, and to increase the removal of stormwater from the surface water system by evaporation and infiltration.

In an urban area, the storage capacity of the natural terrain is significantly reduced by grading to provide smooth, free-draining surfaces; by the filling of wetlands; and by the construction of impervious surfaces such as rooftops, driveways, and streets. These changes result in a significant reduction in stormwater storage capacity. In order to compensate for the loss of natural stormwater storage areas and to reduce the size and cost of conveyance facilities, it may be necessary or desirable to provide man-made storage in the stormwater management system. Such storage may be less costly than higher capacity conveyance facilities and may reduce the impact of stormwater runoff on downstream areas.

Storage facilities can be further classed as detention or retention facilities. Detention basins normally drain completely between spaced runoff events. In contrast, detention ponds maintain a relatively fixed minimum water elevation between runoff events. Retention facilities can be a basin or pond which has no positive outlet but relies on infiltration and evaporation as the only means of removing stormwater.

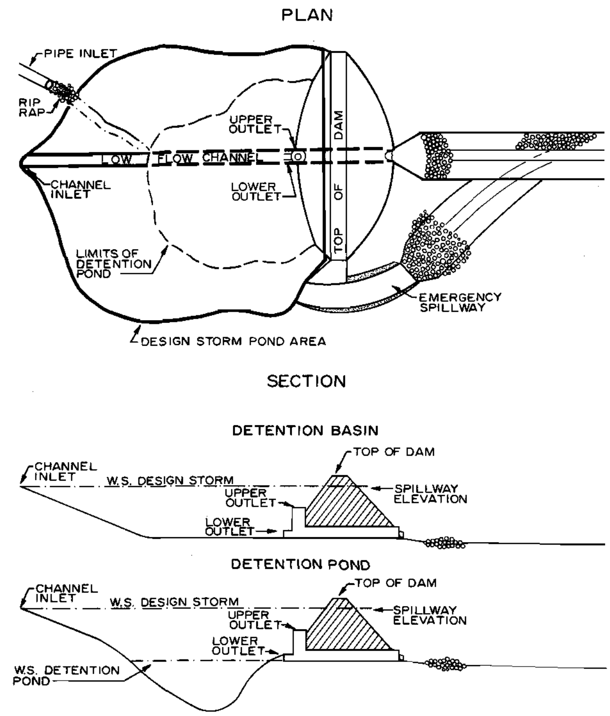
The stormwater management planning effort included an evaluation of available sites for stormwater storage facility use. The evaluation of each site was based on site topography and specific storage volume-outlet discharge relationships.

A word of caution is in order regarding the use of detention facilities. It has been shown that the indiscriminate location and/or phasing of construction of detention facilities within a watershed can actually increase downstream peak flows. Therefore, it is imperative that such facilities be planned, designed, and evaluated on a watershedwide basis and within the context of a system plan by competent engineers experienced in this field, and not by ordinance requirements based upon broad "policy" plans. It is not always desirable or feasible to provide storage in a stormwater management system. In most developed urban areas, suitable parcels of land are not readily available for the construction of stormwater retention or detention basins. Other, more subtle methods of onsite storage and collection system storage may be feasible in such cases, but may cause objectionable disruption of urban activity.

Recommendations for the location, size, and capacity of storage facilities are provided in the stormwater management plan. Additional details and refinement must be addressed in the detailed design phase preceding construction. Criteria for design are provided in Chapter V.

Figure 40

TYPICAL DETENTION STORMWATER STORAGE STRUCTURES



Source: SEWRPC.

Detention Storage: Detention storage is the temporary storage of stormwater accompanied by controlled release. The purpose of detention storage is to hold back or delay stormwater runoff temporarily, increasing the overall time of concentration for the drainage area and thereby reducing the peak rate of stormwater runoff.

There are a wide variety of passive stormwater detention measures that can be used in an urban setting at little or no cost. These measures consist of grassed stormwater collection swales designed to flow at low velocities, thereby providing "in line" storage; stormwater conveyance swales designed to include check dams to reduce flow velocities, thereby providing storage; and berms, also used to provide increased storage volume. Stormwater storage can also be provided on flat rooftops, in parking lots, and in specially designed and constructed stormwater storage facilities. These storage measures generally detain stormwater for short periods of time, in some cases allowing increased infiltration, evaporation, and transpiration, and can significantly reduce downstream peak stormwater discharges. A typical stormwater detention basin and detention pond are shown in Figure 40.

Retention Storage: Retention storage is the long-term storage of stormwater without release to the surface water drainage system. The purpose of retention storage is not only to detain but to remove stormwater from the surface drainage system and allow stormwater to infiltrate or evaporate, reducing the overall volume of stormwater that reaches the outfall of the drainage basin.

Stormwater retention basins are often relatively shallow basins, either natural or man-made, with substantial bottom area to allow infiltration into the groundwater reservoir. Stormwater retention ponds with normal water level at the water table elevation may serve as water supply and fire protection reservoirs, and may capture stormwater for industrial or municipal uses. Retention basins and ponds can also serve as recreational facilities and as aesthetic focal points in desirable "green" open spaces. Stormwater retention ponds can be designed in series to include connecting open "green" areas that further enhance the overall stormwater management system effectiveness.

Stormwater Treatment

Stormwater treatment is the deliberate removal of pollutants from stormwater. The purpose of stormwater treatment is to reduce the undesirable environmental impact of stormwater discharges on the quality of downstream surface waters.

The natural environment contains many control mechanisms that prevent pollutants from entering the stormwater drainage system. Urban development can remove or disrupt these mechanisms and thereby cause adverse water quality impacts. In addition, new urban sources of surface pollutants are exposed to the surface water drainage system. The result is generally a significant increase in pollutants transported to the surface water system. Stormwater quality from urban areas may be controlled by providing comprehensive nonpoint source pollution control, or by removing pollutants from the stormwater after collection from the urban drainage basin. Typically, a stormwater treatment facility would consist of a stormwater detention facility to provide a more constant flow rate followed by a physical treatment facility. Stormwater treatment processes include screens, microstrainers, dissolved air flotation, swirl concentrators, high rate filtration, and disinfection or ozonation. A reduction of from 10 to 50 percent in released pollutants may be achieved by stormwater treatment processes.

Stormwater treatment methods are costly. Less costly urban nonpoint source control measures may be a more attractive alternative in many cases. For this reason, and because there are few motivating legal requirements regarding the quality of stormwater discharged to the surface water system, municipalities have not normally pursued this component of the stormwater management system. Limited application of stormwater treatment has been effected for certain types of stormwater runoff from industrial areas.

A second level planning report, SEWRPC Community Assistance Planning Report No. 37, A Nonpoint Source Water Pollution Control Plan for the Root River Watershed, published in March 1980, has specifically determined that stormwater treatment is not required to meet established water use objectives and supporting water quality standards in the study area. Accordingly, such treatment measures were not further considered in this stormwater management planning effort.

Other Nonpoint Source Pollution Control Measures

Nonpoint source water pollution control is the management of urban and rural land uses to reduce pollutants discharged to surface waters. For the purposes of this report, such control measures will be considered only with respect to urban nonpoint sources of pollution. Table 27 presents various nonpoint source control measures. Each of the measures listed may be utilized in both existing and newly developing urban settings. The last two measures--parking lot storage and treatment and onsite storage--while probably more applicable to new urban development, do have limited application in existing urban areas. As already noted, nonpoint source control is usually a considerably less costly method than treatment for controlling pollution from stormwater runoff. In addition, nonpoint source control measures such as parking lot storage and onsite storage provide an additional benefit in peak stormwater runoff and volume reductions.

Table 27

GENERALIZED SUMMARY OF METHODS AND EFFECTIVENESS OF NONPOINT SOURCE WATER POLLUTION ABATEMENT MEASURES

Control Measures	Summary Description	Approximate Percent Reduction of Released Pollutants
Litter and pet waste control ordinance	Prevent the accumulation of litter and pet wastes on streets and residential, commercial, industrial, and recreational areas	2-5
Improved timing and efficiency of street sweeping, leaf collection and disposal, and catch basin cleaning	Improve the scheduling of these public works activities, modify work habits of personnel, and select equipment to maximize the effectiveness of these existing pollution control measures	2-5
Management of onsite sewage treatment systems	Regulate septic system installation, monitoring, location, and performance; replace failing systems with new septic systems or alternative treatment facilities; develop alternatives to septic systems; eliminate direct connections to drain tiles or ditches; dispose of septage at sewage treatment facility	10-30
Increased street sweeping	On the average, sweep all streets in urban areas an equivalent of once or twice a week with vacuum street sweepers; require parking restrictions to permit access to curb areas; sweep all streets at least eight months per year; sweep commercial and industrial areas with greater frequency than residential areas	30-50
Increased leaf and clippings collection and disposal	Increase the frequency and efficiency of leaf collection procedures in fall; use vacuum cleaners to collect leaves; implement ordinances for leaves, clippings, and other organic debris to be mulched, composted, or bagged for pickup	2-5
Increased catch basin cleaning	Increase frequency and efficiency of catch basin cleaning; clean at least twice per year using vacuum cleaners; catch basin installation in new urban development not recommended as a cost-effective practice for water quality improvement	2-5
Reduced use of deicing salt	Reduce use of deicing salt on streets; salt only intersections and problem areas; prevent excessive use of sand and other abrasives	Negligible for pollutants addressed in this chapter but helpful for reducing chlorides and associated damage to vegetation
Improved street maintenance and refuse collection and disposal	Increase street maintenance and repairs; increase provision of trash receptacles in public areas; improve trash collection schedules; increase cleanup of parks and commercial centers	2-5
Parking lot stormwater temporary storage and treatment measures	Construct gravel-filled trenches, sediment basins, or similar measures to store temporarily the runoff from parking lots, rooftops, and other large impervious areas; if treatment is necessary, use a physical-chemical treatment measure such as screens, dissolved air flotation, or a swirl concentrator	5-10
Onsite storage--residential	Remove connections to sewer systems; construct onsite stormwater storage measures for subdivisions	5-10

Source: SEWRPC.

SEWRPC Planning Report No. 37 identified commercial stormwater runoff, roadside erosion, construction site erosion, and agricultural stormwater runoff as being significant problems in the study area. In addition, stream bank erosion was found by the U. S. Department of Agriculture, Soil Conservation Service staff during implementation of the plan. The planning report recommends a 50 percent reduction in nonpoint source pollutant loads in urban stormwater runoff in the study area. Accordingly, nonpoint source control is consistent with the water quality and hydraulic objectives of the Hales Corners area stormwater management plan. The specific measures to be utilized within the Village to achieve the needed reductions in pollutant runoff must be determined in a more detailed planning effort, and those control measures requiring construction must be designed on a site-specific basis as urban development and redevelopment proceeds within the Village.

SUMMARY

This chapter has described the characteristics and functions of six stormwater management system components. The three basic components of overland flow, collection, and conveyance have been traditionally considered in stormwater management system planning, and, as such, were considered in the stormwater management planning effort for the Village of Hales Corners.

With respect to overland flow, the system plan provides general guidelines and a description of practical techniques for minimizing the rate and volume of runoff. The plan assumes that these general guidelines will be followed to the extent practicable as community development and redevelopment proceeds and the siting of buildings and the grading and improvement of surrounding sites take place. Specific measures for overland flow, however, must be designed on a site-specific basis as urban development or redevelopment takes place; and, therefore, overland flow cannot be addressed in any detail in the system plan.

With respect to collection, the system plan contains recommendations concerning the typical shape, general horizontal and vertical alignment, and type of roadside swales and of roadway gutters, and the type and general location of inlets and catch basins. In addition, the system plan provides general guidelines and criteria for the more detailed design of the collection facilities included in the plan. The plan recognizes that such details of the collection system as driveway culvert spacing and sizing; longitudinal gradients; provision of paved swale bottoms; gutter types, locations, and configurations; and inlet and catch basin types and locations must be determined on a site-specific basis in the design phase of system development preceding construction.

With respect to conveyance facilities, the system plan contains recommendations concerning the general horizontal and vertical alignment, shape, and type of open channel conveyance facilities; the general locations and sizes of culverts; and the general alignment, depth, size, slope, and type of storm sewer facilities. The system plan also indicates the general locations of manholes and junction chambers. No stormwater pumping or lift stations were recommended as components of the Village's stormwater management plan.

The three remaining system components of storage, treatment, and nonpoint source water pollution control were presented in this chapter as newer components that may be required within stormwater management systems to meet overall system development and performance objectives. Stormwater storage was specifically considered in the systems plan, which contains recommendations concerning the general location, area, volume, and storage volume-outlet discharge relationships. Additional details of such storage facilities must be addressed on a site-specific basis in the detailed design phase preceding construction. Criteria for such design are provided in the plan.

Stormwater treatment was not considered in the system planning effort since a second level planning report, SEWRPC Community Assistance Planning Report No. 37, A Nonpoint Source Water Pollution Control Plan for the Root River Watershed, published in March 1980, specifically determined that such treatment is not required to meet the established water use objectives and supporting water quality standards in the study area.

Urban nonpoint source water pollution control measures, other than treatment, were considered in the plan. The control measures available for use in both existing and newly developing urban settings were identified, along with the percentage reduction in released pollutants that can be achieved by each measure and the needed levels of reduction. The specific measures to be utilized within the Village to achieve the needed reductions in pollutant runoff must, however, be determined in a more detailed planning effort; and measures requiring construction must be designed on a site-specific basis as urban development and redevelopment proceeds within the Village.

The system plan recognizes that the selection of street cross-sections, including appurtenant drainage details, is a decision which, to a considerable extent, must be based upon the preferences of local residents. Within the Village, residents have generally favored the use of rural street cross-sections with roadside swales instead of the use of urban street cross-sections with gutters, inlets, and storm sewers. Accordingly, the system plan was based on the assumption that, except in certain isolated cases, stormwater collection and conveyance in areas of new single-family residential development, as well as in most areas of existing single-family residential development, will be achieved through the use of roadside swales, along with open swales for stormwater conveyance. In certain areas of the Village, including areas of higher density residential development and commercial and industrial development, the system plan evaluated the use of an alternative suburban street cross-section having mountable curbs, gutters, inlets, and storm sewers in order to identify the system component types and sizes needed to provide flexibility to change the type of street cross-section in the future.

Chapter VII

EVALUATION OF EXISTING AND ALTERNATIVE FUTURE STORMWATER MANAGEMENT SYSTEMS

INTRODUCTION

This chapter presents the findings of an evaluation of the existing stormwater management system serving the Village of Hales Corners and environs, together with a description and evaluation of alternative stormwater management plans designed to serve this area through the design year 2000. In order to evaluate alternative stormwater management plans, it was first necessary to characterize the existing stormwater drainage system of the planning area. This required the collection and collation of definitive data on the locations and configurations and the sizes, elevations, and grades of the various components of that system; the computation of the hydraulic capacity of those components; and a comparison of those capacities to anticipated rates and volumes of stormwater runoff under both existing and planned future land use conditions. As indicated in Chapter V of this report, a 10-year recurrence interval storm event was used to evaluate and design the minor system components consisting of backyard and sideyard swales, roadside swales, curbs and gutters, inlets, storm sewers, storage facilities, and related appurtenances. The major system components, including the entire street cross-section and interconnected drainage swales and watercourses, were evaluated and designed using a 100-year recurrence interval storm event.

Following a description of the findings of the evaluation of the existing system, this chapter describes and evaluates alternative conceptual approaches to stormwater management which could be applied in the planning area to mitigate existing stormwater management problems and accommodate runoff from planned development to the design year 2000. Descriptions and evaluations of three specific alternative stormwater management system plans for the planning area follow the general description and evaluation.

EVALUATION OF THE EXISTING STORMWATER MANAGEMENT SYSTEM

In order to characterize the existing stormwater management system, the major components of that system need to be described. Such a description permits the hydraulic capacities of the existing conveyance and storage facilities to be calculated, along with the required capacities under the design storms and under planned future and existing land use development conditions in the tributary catchment areas. Those system components which are unable to accommodate the runoff expected from the design storms under either existing or future land use conditions, or both, are thus identified, and these components then can be addressed in the design of alternative stormwater management system plans.

The evaluation of the existing stormwater management system was directed toward the storm sewers, storage facilities, open channels, roadside swales, and culverts of the minor system, as defined in Chapter V of this report; and toward the open watercourses and related bridges and culverts of the major system. In the evaluation it was assumed that the backyard and sideyard drainage swales, the roadside swales and curbs and gutters, and the inlets would have adequate capacity to convey the stormwater flows generated by storms up to and including the 10-year recurrence interval event to the receiving conveyance and storage facilities of the minor system. In addition, it was assumed that the street cross-sections and interconnecting drainage swales of the major system would have adequate capacity to convey the stormwater flows generated by storms in excess of the 10-year recurrence interval event and up to the 100-year recurrence interval event to the watercourses of the major system. The system components assumed to be adequate in this chapter for the purpose of designing and evaluating alternative system plans were, however, subject to quantitative analysis in the development of the recommended plan as set forth in Chapter VIII of this report.

Physical Characteristics

As described in Chapter III of this report, the total planning area was divided into 208 subbasins for analytical purposes, as shown on Map 7 of Chapter III. Of the total of 208 subbasins, 140 were located within the Village of Hales Corners. The pertinent characteristics of the stormwater drainage system of each subbasin, together with the pertinent characteristics of the subbasin itself, are presented in Table 28. Data are provided on subbasin size, existing and planned land use, the type and capacity of the stormwater drainage component comprising the outlet of the subbasin, and the peak stormwater flow rates expected to be generated from the subbasin. The existing stormwater drainage systems are primarily comprised of roadside swales, roadway curbs and gutters, storm sewer inlets, storm sewers, and open channels and associated culverts, together with the streams to which the outlets of the engineered and constructed system components discharge. The existing stormwater management systems are described in Chapter III of this report.

Hydraulic Capacities of Conveyance Systems and Storm Flows

The hydraulic capacity of conveyance facilities--storm sewers, culverts, and open channels--is determined by the shape and dimensions of the cross-section of the facility and the facility's composition and lining, its elevation and gradient, and the roughness of the surface--as represented by Manning's "n" value. The methods used to determine the hydraulic capacity of the system components are described in Chapter V of this report. The hydraulic capacity of the conveyance facilities at the outlet of each subbasin is presented in Table 28. In addition to the capacity at the outlet of each subbasin, the capacities of all storm sewers, storage facilities, and open channels and culverts in the minor stormwater management system and of selected watercourses of the major stormwater management system were calculated.

Peak rates of stormwater runoff, as determined by the hydrologic and hydraulic characteristics of each catchment area, were estimated utilizing the methods described in Chapter V of this report. The estimated peak rates of stormwater runoff at the outlets of each subbasin for the 10-year and 100-year recurrence interval storm events, as appropriate, are also set forth in Table 28. Peak rates of flow were also estimated for catchment areas within subbasins in order to determine the hydraulic loadings, as appropriate, on each segment of

Table 28

SELECTED CHARACTERISTICS OF THE EXISTING STORMWATER DRAINAGE SYSTEM IN THE HALES CORNERS PLANNING AREA UNDER EXISTING AND PLANNED LAND USE CONDITIONS

Subwatershed and Subbasin or Special Component	Area of Subbasin (acres)	Subbasin Downstream Conveyance Component		Principal Land Use in Subbasin		Peak Stormwater Flow (cfs)			
						10-Year Recurrence Interval Storm Event		100-Year Recurrence Interval Storm Event	
		Description	Hydraulic Capacity (cfs)	Existing Conditions	Planned Conditions	Existing Land Use	Planned Land Use	Existing Land Use	Planned Land Use
Northwest Branch Whitnall Park Creek 1-18	18.0	Swale-grass Depth = 4 feet Bottom width = 2 feet Side slopes = 4:1 and 2 No. 36 corrugated metal pipe arches	108	Single-family residential	Single-family residential	116	164	240	315
2-0	1.3	Swale-grass Depth = 1.5 feet Bottom width = 0 Side slopes = 5:1	62	Single-family residential	Single-family residential	2	2	4	4
1-28	18.7	Swale-grass Depth = 4 feet Bottom width = 2 feet Side slopes = 20:1 and 2 No. 48 corrugated metal pipe arches	136	Single-family residential	Single-family residential	128	183	273	352
1-32	28.7	Swale-grass Depth = 4 feet Bottom width = 3 feet Side slopes = 20:1	1,584	Open	Single-family residential	157	222	322	424
3-0	16.6	Pipe-concrete Diameter = 21 inches	15	Single-family residential	Single-family residential	14	14	35	35
3-2	0.0	Pipe-concrete Diameter = 21 inches	15	--	--	14	14	35	35
3-4	0.0	Swale-grass Depth = 2.5 feet Bottom width = 0 Side slopes = 5:1	150	--	--	14	14	35	35
9-0	11.3	--	--	Single-family residential	Single-family residential	10	10	25	25
1-40	25.7	Swale-grass Depth = 2.5 feet Bottom width = 4 feet Side slopes = 20:1	71	Open, wetland	Single-family residential	176	235	351	448
10-0	5.3	--	--	Single-family residential	Single-family residential	5	5	12	12
1-44	0.0	Swale-grass Depth = 2 feet Bottom width = 4 feet Side slopes = 10:1	71	--	--	176	236	348	450

Table 28 (continued)

Subwatershed and Subbasin or Special Component	Area of Subbasin (acres)	Subbasin Downstream Conveyance Component		Principal Land Use in Subbasin		Peak Stormwater Flow (cfs)			
						10-Year Recurrence Interval Storm Event		100-Year Recurrence Interval Storm Event	
		Description	Hydraulic Capacity (cfs)	Existing Conditions	Planned Conditions	Existing Land Use	Planned Land Use	Existing Land Use	Planned Land Use
Northwest Branch Whitnall Park Creek (continued)									
5-0	21.4	Swale-grass Depth = 2.5 feet Bottom width = 0 Side slopes = 5:1	230	Open, institutional	Single-family residential, institutional	21	42	40	72
5-4	21.5	Pipe-corrugated metal Depth = 36 inches	19	Institutional, woodland	Single-family residential	44	70	88	123
5-8	2.8	Swale-grass Depth = 4 feet Bottom width = 1 foot Side slopes = 3:1	200	Single-family residential	Single-family residential	48	73	94	130
5-12	3.9	Swale-grass Depth = 4 feet Bottom width = 4 feet Side slopes = 3:1	469	Residential	Single-family residential	53	79	105	140
5-14	0.0	Swale-grass and water Depth = 3.5 feet Bottom width = 300 feet Side slopes = 50:1	4,200	--	--	57	77	103	138
5-16	0.0	Swale-grass and water Depth = 2.5 feet Bottom width = 300 feet Side slopes = 50:1	2,200	--	--	52	77	103	138
4-0	15.0	Swale-grass Depth = 2 feet Bottom width = 0 Side slopes = 10:1	223	Open, residential	Single-family residential	17	23	34	44
4-4	17.3	Swale-grass Depth = 3 feet Bottom width = 0 Side slopes = 10:1	353	Open	Single-family residential	26	48	67	87
5-18	48.9	Swale-grass and water Depth = 3 feet Bottom width = 300 feet Side slopes = 50:1	3,100	Wetland, utility, open	Wetland, recreational	124	130	233	243
6-0	12.4	Swale-grass Depth = 2 feet Bottom width = 0 Side slopes = 4:1	45	Single-family residential	Single-family residential	12	12	24	24

Table 28 (continued)

Subwatershed and Subbasin or Special Component	Area of Subbasin (acres)	Subbasin Downstream Conveyance Component: Description	Subbasin Downstream Conveyance Component: Hydraulic Capacity (cfs)	Principal Land Use in Subbasin: Existing Conditions	Principal Land Use in Subbasin: Planned Conditions	Peak Stormwater Flow (cfs): 10-Year Recurrence Interval Storm Event: Existing Land Use	Peak Stormwater Flow (cfs): 10-Year Recurrence Interval Storm Event: Planned Land Use	Peak Stormwater Flow (cfs): 100-Year Recurrence Interval Storm Event: Existing Land Use	Peak Stormwater Flow (cfs): 100-Year Recurrence Interval Storm Event: Planned Land Use
Northwest Branch Whitnall Park Creek (continued)									
6-10	13.0	--	--	Single-family residential	Single-family residential	21	21	40	40
99-0	12.0	Swale-grass Depth = 2 feet Bottom width = 0 Side slopes = 4:1	35	Single-family residential	Single-family residential	13	13	24	24
1-46	0.0	Box-concrete Width = 7 feet Depth = 3 feet	120	--	--	112	112	257	257
1-48	5.6	Swale-grass Depth = 4 feet Bottom width = 2 feet Side slopes = 5:1 and No. 48 corrugated metal pipe arch	36	Single-family residential	Single-family residential	112	112	257	257
11-0	4.1	Swale-grass Depth = 1.5 feet Bottom width = 0 Side slopes = 5:0	22	Single-family residential	Single-family residential	3	3	7	7
11-10	7.1	--	--	Single-family residential	Single-family residential	9	9	18	18
11-14	20.5	Pipe-corrugated metal No. 36 corrugated metal arch	33	Single-family residential	Single-family residential	20	20	44	44
1-56	8.4	Swale-grass Depth = 4 feet Bottom width = 10 feet Side slopes = 3:1 and No. 54 corrugated metal pipe arch	95	Single-family residential	Single-family residential	112	112	260	258
1-64	19.0	Swale-grass Depth = 6 feet Bottom width = 10 feet Side slopes = 3:1 and No. 54 corrugated metal pipe arch	111	Single-family residential	Single-family residential	115	115	260	258
1-72	0.0	Swale-grass Depth = 5 feet Bottom width = 10 feet Side slopes = 3:1 and 12-foot x 6-foot concrete box culvert	500	--	--	196	196	312	313

Table 28 (continued)

Subwatershed and Subbasin or Special Component	Area of Subbasin (acres)	Subbasin Downstream Conveyance Component		Principal Land Use in Subbasin		Peak Stormwater Flow (cfs)			
						10-Year Recurrence Interval Storm Event		100-Year Recurrence Interval Storm Event	
		Description	Hydraulic Capacity (cfs)	Existing Conditions	Planned Conditions	Existing Land Use	Planned Land Use	Existing Land Use	Planned Land Use
Northwest Branch Whitnall Park Creek (continued)									
12-0	16.4	Swale-grass Depth = 2 feet Bottom width = 0 Side slopes = 4:1	99	Single-family residential	Single-family residential	26	26	48	48
13-0	7.1	Pipe-concrete Depth = 24 feet	16	Single-family residential	Single-family residential	13	13	23	23
1-80	23.3	Swale-grass Depth = 4 feet Bottom width = 10 feet Side slopes = 3:1 and 12-foot x 6-foot concrete box culvert	560	Single-family residential	Single-family residential	201	201	312	312
1-88	0.0	Swale-grass Depth = 2.2 feet Bottom width = 10 feet Side slopes = 2:1 and a 72-inch driveway culvert	115	--	--	201	200	312	311
1-96	40.0	Swale-grass Depth = 3.5 feet Bottom width = 10 feet Side slopes = 3:1 and a 4.8-foot x 5.0-foot driveway box culvert	616	Single-family residential, commercial, institutional	Single-family residential, commercial institutional	207	204	394	398
North Branch Whitnall Park Creek									
7-0	31.2	Swale-grass Depth = 4 feet Bottom width = 6 feet Side slopes = 3:1	464	Open, governmental	Governmental, single-family residential	62	59	102	95
7-10	21.4	Swale-grass Depth = 4 feet Bottom width = 3 feet Side slopes = 2.5:1 and No. 42 corrugated metal pipe arch	37	Residential	Single-family residential	91	97	158	163
7-16	9.0	Swale-grass Depth = 4.5 feet Bottom width = 3 feet Side slopes = 2.5:1 and No. 48 corrugated metal pipe arch	53	Single-family residential	Single-family residential	103	109	178	182

Table 28 (continued)

Subwatershed and Subbasin or Special Component	Area of Subbasin (acres)	Subbasin Downstream Conveyance Component: Description	Subbasin Downstream Conveyance Component: Hydraulic Capacity (cfs)	Principal Land Use in Subbasin: Existing Conditions	Principal Land Use in Subbasin: Planned Conditions	Peak Stormwater Flow (cfs), 10-Year Recurrence Interval Storm Event: Existing Land Use	Peak Stormwater Flow (cfs), 10-Year Recurrence Interval Storm Event: Planned Land Use	Peak Stormwater Flow (cfs), 100-Year Recurrence Interval Storm Event: Existing Land Use	Peak Stormwater Flow (cfs), 100-Year Recurrence Interval Storm Event: Planned Land Use
North Branch Whitnall Park Creek (continued)									
7-20	13.4	Swale-grass Depth 5 feet Bottom width = 3 feet Side slopes = 2.5:1 and No. 48 corrugated metal pipe arch	53	Single-family residential	Single-family residential	122	128	212	216
7-24	0.0	Pipe-corrugated metal No. 48 pipe arch	50	--	--	121	126	212	212
8-0	18.5	Swale-grass Depth = 2 feet Bottom width = 0 Side slopes = 5:1	--	Single-family residential	Single-family residential	35	35	62	62
8-4	0.0	Swale-grass Depth = 2 feet Bottom width = 0 Side slopes = 5:1	--	--	--	35	35	62	62
7-28	0.0	Swale-grass Depth = 4.5 feet Bottom width = 3 feet Side slopes = 2.5:1 and No. 48 corrugated metal pipe arch	29	--	--	145	156	266	270
7-30	22.3	Swale-grass Depth = 4:5 feet Bottom width = 10 feet Side slopes = 4:1 and No. 48 corrugated metal pipe arch	29	Single-family residential, institutional	Single-family residential, institutional	173	185	316	321
7-40	20.6	Swale-grass Depth = 4.5 feet Bottom width = 10 feet Side slopes = 4:1	798	Single-family residential	Single-family residential	80	86	202	202
Whitnall Park Creek									
20-49	60.4	Swale-grass Depth = 6 feet Bottom width = 10 feet Side slopes = 2:1	643	Single-family residential, water	--	120	120	231	231
20-50	0.0	Two corrugated metal pipes Diameter = 72 inches	180	--	--	120	120	231	231

Table 28 (continued)

Subwatershed and Subbasin or Special Component	Area of Subbasin (acres)	Subbasin Downstream Conveyance Component: Description	Subbasin Downstream Conveyance Component: Hydraulic Capacity (cfs)	Principal Land Use in Subbasin: Existing Conditions	Principal Land Use in Subbasin: Planned Conditions	Peak Stormwater Flow (cfs), 10-Year Recurrence Interval Storm Event: Existing Land Use	10-Year: Planned Land Use	100-Year Recurrence Interval Storm Event: Existing Land Use	100-Year: Planned Land Use
Whitnall Park Creek (continued)									
20-51	0.0	Swale-grass Depth = 6 feet Bottom width = 3 feet Side slopes = 5:1 and 2 72-inch corrugated metal pipes	180	--	--	120	120	231	231
20-52	19.5	Swale-grass Depth = 3.5 feet Bottom width = 8 feet Side slopes = 9:1	524	Single-family residential	Single-family residential	120	120	231	231
20-54	11.8	Swale-grass Depth = 3 feet Bottom width = 3 feet Side slopes = 5:1 and 2 84-inch corrugated metal pipes	311	Single-family residential	Single-family residential	80	142	208	271
32-0	8.1	Swale-grass Depth = 2 feet Bottom width = 1 foot Side slopes = 5:1	75	Single-family residential	Single-family residential	17	17	28	28
32-4	18.1	Swale-grass Depth = 3 feet Bottom width = 0 Side slopes = 4:1 and No. 24 corrugated metal pipe arch	12	Single-family residential	Single-family residential	29	29	66	66
32-6	34.2	Swale-grass Depth = 2 feet Bottom width = 0 Side slopes = 10:1 and 2 No. 24 corrugated metal pipe arches	24	Single-family residential	Single-family residential	70	70	142	142
31-8	19.2	Swale-grass Depth = 3 feet Bottom width = 8 feet Side slopes = 20:1	967	Open land	Single-family residential	106	142	223	304
31-12	16.1	Swale-grass Depth = 2.5 feet Bottom width = 10 feet Side slopes = 5:1 and No. 60 corrugated metal pipe arch	78	Single-family residential, open land	Single-family residential	107	145	229	317

Table 28 (continued)

Subwatershed and Subbasin or Special Component	Area of Subbasin (acres)	Subbasin Downstream Conveyance Component: Description	Subbasin Downstream Conveyance Component: Hydraulic Capacity (cfs)	Principal Land Use in Subbasin: Existing Conditions	Principal Land Use in Subbasin: Planned Conditions	Peak Stormwater Flow (cfs): 10-Year Recurrence Interval Storm Event: Existing Land Use	Peak Stormwater Flow (cfs): 10-Year Recurrence Interval Storm Event: Planned Land Use	Peak Stormwater Flow (cfs): 100-Year Recurrence Interval Storm Event: Existing Land Use	Peak Stormwater Flow (cfs): 100-Year Recurrence Interval Storm Event: Planned Land Use
Whitnall Park Creek (continued)									
20-56	0.0	Swale-grass Depth = 5 feet Bottom width = 10 feet Side slopes = 5:1	1,113	--	--	140	201	307	430
20-58	22.8	Swale-grass Depth = 6 feet Bottom width = 3 feet Side slopes = 5:1	1,469	Single-family residential	Single-family residential	164	219	347	472
33-0	12.3	Swale-grass Depth = 12 feet Bottom width = 0 Side slopes = 10:1	131	Single-family residential	Single-family residential	19	19	36	36
34-0	10.6	Pipe-concrete Diameter = 24 inches	19	Single-family residential	Single-family residential	16	16	31	31
33-4	11.6	Pipe-concrete Diameter = 27 inches	26	Single-family residential, utilities, governmental	Single-family residential	54	54	99	99
20-60	12.5	Swale-grass Depth = 5 feet Bottom width = 10 feet Side slopes = 4:1	845	Single-family residential	Single-family residential	207	263	428	551
20-62	33.1	Box-culvert Depth = 4.7 feet Bottom width = 22 feet	1,210	Recreational	Parking	209	266	446	569
35-4	25.6	Pipe-corrugated metal Diameter = 24 inches	15	Single-family residential	Single-family residential	64	64	120	120
35-6	26.4	Swale-grass Depth = 2 feet Bottom width = 2 feet Side slopes = 3.5:1 and No. 24 corrugated metal pipe arch and 18-inch corrugated metal pipe	21	Single-family residential	Single-family residential	41	41	65	65
35-8	21.9	Swale-grass Depth = 2.5 feet Bottom width = 2 feet Side slopes = 3.5:1 and No. 36 corrugated metal pipe arch	32	Single-family residential	Single-family residential	69	67	114	114

Table 28 (continued)

Subwatershed and Subbasin or Special Component	Area of Subbasin (acres)	Subbasin Downstream Conveyance Component: Description	Subbasin Downstream Conveyance Component: Hydraulic Capacity (cfs)	Principal Land Use in Subbasin: Existing Conditions	Principal Land Use in Subbasin: Planned Conditions	Peak Stormwater Flow (cfs): 10-Year Recurrence Interval Storm Event: Existing Land Use	Peak Stormwater Flow (cfs): 10-Year Recurrence Interval Storm Event: Planned Land Use	Peak Stormwater Flow (cfs): 100-Year Recurrence Interval Storm Event: Existing Land Use	Peak Stormwater Flow (cfs): 100-Year Recurrence Interval Storm Event: Planned Land Use
Whitnall Park Creek (continued)									
35-12	24.4	Swale-grass Depth = 5 feet Bottom width = 1 foot Side slopes = 5:1 and No. 42 corrugated metal pipe arch	77	Single-family residential	Single-family residential	101	101	175	175
35-14	7.9	Swale-grass Depth = 7 feet Bottom width = 1 foot Side slopes = 5:1	3,300	Single-family residential	Single-family residential	108	110	192	192
35-16	0.0	Pipe-concrete Diameter = 48 inches	144	--	--	106	108	191	191
37-0	18.4	Pipe-concrete Diameter = 36 inches	127	Single-family residential	Single-family residential	27	27	53	53
35-18	0.0	Pipe-concrete Diameter = 60 inches	212	--	--	128	130	235	235
20-64	28.7	Swale-grass Depth = 8 feet Bottom width = 10 feet Side slopes = 5:1	2,685	Single-family residential	Single-family residential	361	410	730	853
20-66	8.0	Swale-grass Depth = 8 feet Bottom width = 10 feet Side slopes = 5:1	3,044	Single-family residential	Single-family residential	363	410	739	860
36-0	11.3	Swale-grass Depth = 2 feet Bottom width = 0 Side slopes = 10:1	242	Single-family residential	Single-family residential	16	16	29	29
36-4	17.4	Pipe-corrugated metal Diameter = 24 inches Swale-grass Depth = 1 foot Bottom width = 0 Side slopes = 6:1	31	Single-family residential	Single-family residential	37	37	68	68
20-68	22.5	Swale-grass Depth = 4 feet Bottom width = 10 feet Side slopes = 30:1	3,393	Single-family residential	Single-family residential	395	442	793	916

Table 28 (continued)

Subwatershed and Subbasin or Special Component	Area of Subbasin (acres)	Subbasin Downstream Conveyance Component: Description	Subbasin Downstream Conveyance Component: Hydraulic Capacity (cfs)	Principal Land Use in Subbasin: Existing Conditions	Principal Land Use in Subbasin: Planned Conditions	Peak Stormwater Flow (cfs): 10-Year Recurrence Interval Storm Event: Existing Land Use	Peak Stormwater Flow (cfs): 10-Year Recurrence Interval Storm Event: Planned Land Use	Peak Stormwater Flow (cfs): 100-Year Recurrence Interval Storm Event: Existing Land Use	Peak Stormwater Flow (cfs): 100-Year Recurrence Interval Storm Event: Planned Land Use
Whitnall Park Creek (continued)									
1-104	7.7	Swale-grass Depth = 9 feet Bottom width = 15 feet Side slopes = 8:1 and 30-foot-span bridge	2,200	Commercial	Commercial	546	592	1,116	1,207
53-0	4.3	Pipe-concrete Diameter = 36 inches	24	Commercial	Commercial	17	17	27	27
50-12	11.7	Pipe-concrete Diameter = 48 inches	125	Single-family residential	Single-family residential	123	123	81	81
52-0	18.3	Swale-grass Depth = 1.75 feet Bottom width = 0 Side slopes = 5:1	94	Single-family residential	Single-family residential	26	26	49	49
50-14	14.4	Pipe-concrete Diameter = 48 inches	150	Single-family residential	Single-family residential	164	164	165	165
50-16	11.3	Pipe-concrete Diameter = 48 inches	170	Single-family residential, commercial	Single-family residential	169	169	189	189
1-116	3.8	Swale-grass Depth = 9 feet Bottom width = 10 feet Side slopes = 2.5:1 and 30-foot x 6-foot concrete box culvert	2,000	Utilities, communication, transportation	Commercial	697	734	1,273	1,373
60-10	17.5	Swale-grass Depth = 2 feet Bottom width = 0 Side slopes = 75:1	1,096	Utility, open, industrial	Commercial, utility	186	273	489	594
60-12	16.0	Two 54-inch reinforced concrete pipes	245	Open	Residential, commercial	191	284	498	606
64-0	34.3	Pipe-concrete Diameter = 54 inches	182	Open, utility	Commercial	30	88	59	142
60-14	3.3	Pipe-concrete Diameter = 66 inches	380	Transportation, single-family residential, open	Transportation, commercial	216	366	546	736

Table 28 (continued)

Subwatershed and Subbasin or Special Component	Area of Subbasin (acres)	Subbasin Downstream Conveyance Component		Principal Land Use in Subbasin		Peak Stormwater Flow (cfs)			
						10-Year Recurrence Interval Storm Event		100-Year Recurrence Interval Storm Event	
		Description	Hydraulic Capacity (cfs)	Existing Conditions	Planned Conditions	Existing Land Use	Planned Land Use	Existing Land Use	Planned Land Use
Whitnall Park Creek (continued) 60-16	8.7	Pipe-concrete Diameter = 66 inches	360	Transportation, single-family residential, commercial, open	Transportation, commercial, single-family residential	225	377	557	754
40-0	9.7	Swale-grass Depth = 1.75 feet Bottom width = 0 Side slopes = 5:1	96	Single-family residential	Single-family residential	13	13	25	25
40-2	17.5	Swale-grass Depth = 1.75 feet Bottom width = 0 Side slopes = 5:1	49	Single-family residential	Single-family residential	37	37	68	68
41-0	13.0	Swale-grass Depth = 1.75 feet Bottom width = 0 Side slopes = 5:1	43	Single-family residential	Single-family residential	15	15	28	28
40-3	10.0	Pipe-corrugated metal Diameter = 18 inches	7	Utility, commercial	Utility, commercial	67	67	124	124
40-4	0.0	Pipe-corrugated metal Diameter = 24 inches	12	--	--	67	67	124	124
42-0	5.8	Pipe-concrete Diameter = 18 inches	10	Open, utility, transportation	Commercial, industrial	17	17	28	28
40-6	0.0	Pipe-concrete Diameter = 48 inches	103	--	--	82	82	150	149
40-7	20.6	Pipe-concrete Diameter = 54 inches	119	Single-family residential	Single-family residential	125	125	224	224
40-8	13.0	Pipe-concrete Diameter = 60 inches	185	Utility, single-family residential	Utility, single-family residential	151	155	275	275
40-10	25.5	Pipe-concrete Diameter = 60 inches to 66 inches	240	Single-family residential, commercial	Single-family residential, commercial	403	211	369	369
40-12	17.6	Pipe-concrete Diameter = 66 inches	365	Single-family residential, utility, commercial	Single-family residential, utility, commercial	250	250	437	437

Table 28 (continued)

Subwatershed and Subbasin or Special Component	Area of Subbasin (acres)	Subbasin Downstream Conveyance Component		Principal Land Use in Subbasin		Peak Stormwater Flow (cfs)			
						10-Year Recurrence Interval Storm Event		100-Year Recurrence Interval Storm Event	
		Description	Hydraulic Capacity (cfs)	Existing Conditions	Planned Conditions	Existing Land Use	Planned Land Use	Existing Land Use	Planned Land Use
Whitnall Park Creek (continued)									
45-0	12.6	Pipe-concrete Diameter = 24 inches	12	Commercial, Single-family residential, transportation	Commercial, Single-family residential, transportation	37	37	60	60
45-2	9.8	--	--	Single-family residential	Single-family residential	53	53	89	89
45-4	6.7	--	--	Open land	Open land	58	58	100	100
45-6	5.7	Pipe-concrete Diameter = 30 inches	32	Transportation, single-family residential	Transportation, single-family residential	72	72	124	124
45-8	20.2	--	--	Single-family residential	Single-family residential	94	94	163	163
45-10	0.0	Pipe-concrete Diameter = 48 inches	88	--	--	94	94	163	163
45-12	20.1	Pipe-concrete Diameter = 48 inches	97	Single-family residential	Single-family residential	125	125	222	222
45-14	11.5	Pipe-concrete Diameter = 48 inches	175	Transportation commercial, governmental	Transportation, commercial, governmental	151	153	270	270
40-14	3.7	Pipe-concrete Diameter = 66 inches	380	Transportation, open	Transportation, open	400	402	708	709
82-0	29.6	Swale-grass Depth = 3 feet Bottom width = 0 Side slopes = 30:1	1,343	Open, recreational	Single-family residential	30	30	57	57
82-2	31.6	Swale-grass Depth = 3 feet Bottom width = 0 Side slopes = 15:1	981	Open residential	Single-family residential	59	59	121	121
82-4	20.5	Swale-grass Depth = 4 feet Bottom width = 3 feet Side slopes = 3:1	381	Single-family residential, open	Single-family residential	67	67	148	148
82-6	0.0	Pipe-corrugated metal Diameter = 30 inches	43	--	--	66	66	146	146

Table 28 (continued)

Subwatershed and Subbasin or Special Component	Area of Subbasin (acres)	Subbasin Downstream Conveyance Component: Description	Subbasin Downstream Conveyance Component: Hydraulic Capacity (cfs)	Principal Land Use in Subbasin: Existing Conditions	Principal Land Use in Subbasin: Planned Conditions	Peak Stormwater Flow (cfs): 10-Year Recurrence Interval Storm Event: Existing Land Use	10-Year: Planned Land Use	100-Year Recurrence Interval Storm Event: Existing Land Use	100-Year: Planned Land Use
Whitnall Park Creek (continued)									
86-0	15.7	Swale-grass Depth = 1.75 feet Bottom width = 0 Side slopes = 5:1	92	Single-family residential, open	Single-family residential	27	27	50	50
86-4	14.1	Swale-grass Depth = 2 feet Bottom width = 3 feet Side slopes = 4:1	134	Single-family residential	Single-family residential	49	49	90	90
47-0	16.8	Pipe-corrugated metal Diameter = 15 inches	2	Single-family residential	Single-family residential	23	23	42	42
47-4	0.0	Swale-grass Depth = 1.75 feet Bottom width = 0 Side slopes = 5:1	57	--	--	22	22	42	42
47-8	28.1	Swale-grass Depth = 1.5 feet Bottom width = 0 Side slopes = 25:1	285	Woodland, open	Woodland, open	35	35	72	72
48-0	21.0	Swale-grass Depth = 1.75 feet Bottom width = 0 Side slopes = 4:1	53	Single-family residential	Single-family residential	25	25	48	48
48-4	0.0	Pipe-corrugated metal arch No. 24	4	--	--	25	25	48	48
48-8	9.8	Pipe-corrugated metal arch No. 24	4	Single-family residential	Single-family residential	37	37	68	68
Root River									
62-0	10.2	Pipe-concrete Diameter = 36 inches	33	Transportation, utility, open	Transportation commercial, utility	22	31	34	46
67-0	9.7	Swale-grass Depth = 4 feet Bottom width = 2 feet Side slopes = 2:1	305	Single-family residential	Single-family residential	6	6	11	11
67-2	0.0	Pipe-concrete Diameter = 30 inches and 30-inch corrugated metal pipe	33	--	--	6	6	11	11
62-2	0.0	Pipe-concrete Diameter = 42 inches	75	--	--	26	35	43	55

Table 28 (continued)

Subwatershed and Subbasin or Special Component	Area of Subbasin (acres)	Subbasin Downstream Conveyance Component: Description	Subbasin Downstream Conveyance Component: Hydraulic Capacity (cfs)	Principal Land Use in Subbasin: Existing Conditions	Principal Land Use in Subbasin: Planned Conditions	Peak Stormwater Flow (cfs): 10-Year Recurrence Interval Storm Event: Existing Land Use	Peak Stormwater Flow (cfs): 10-Year Recurrence Interval Storm Event: Planned Land Use	Peak Stormwater Flow (cfs): 100-Year Recurrence Interval Storm Event: Existing Land Use	Peak Stormwater Flow (cfs): 100-Year Recurrence Interval Storm Event: Planned Land Use
Root River (continued)									
62-4	14.9	Pipe-concrete Diameter = 48 inches	108	Single-family residential	Single-family residential, commercial, utility	38	46	68	80
62-6	18.4	Pipe-concrete Diameter = 48 inches	115	Single-family residential	Single-family residential, commercial, utility	67	94	113	153
62-8	4.0	Pipe-concrete Diameter = 48 inches	145	Single-family residential, utility	Single-family residential, utility	71	97	120	160
70-0	25.7	Swale-grass Depth 1.5 feet Bottom width = 0 Side slopes = 3:1	44	Single-family residential	Single-family residential, utility	29	29	51	51
71-0	21.7	Swale-grass Depth = 1.5 feet Bottom width = 0 Side slope = 20:1	443	Single-family residential	Single-family residential	17	17	35	35
61-0	15.5	Swale-grass Depth = 1 foot Bottom width = 2 feet Side slopes = 3.5:1	15	Single-family residential	Single-family residential	11	11	23	23
65-0	13.5	Swale-grass Depth = 1 foot Bottom width = 0 Side slopes = 20:1	60	Single-family residential	Single-family residential	13	13	26	26
61-8	38.5	Swale-grass Depth = 3 feet Bottom width = 3 feet Side slopes = 3.5:1	298	Single-family residential	Single-family residential	47	47	88	88
61-10	0.0	Pipe-corrugated metal arch No. 30	32	--	--	46	46	86	86
66-0	13.1	Swale-grass Depth = 1.5 feet Bottom width = 0 Side slopes = 5:1	61	Single-family residential	Single-family residential	11	11	22	22
61-16	33.8	Pipe-corrugated metal Diameter = 36 inches	65	Single-family residential	Single-family residential	77	77	149	149

Table 28 (continued)

Subwatershed and Subbasin or Special Component	Area of Subbasin (acres)	Subbasin Downstream Conveyance Component		Principal Land Use in Subbasin		Peak Stormwater Flow (cfs)			
						10-Year Recurrence Interval Storm Event		100-Year Recurrence Interval Storm Event	
		Description	Hydraulic Capacity (cfs)	Existing Conditions	Planned Conditions	Existing Land Use	Planned Land Use	Existing Land Use	Planned Land Use
Root River (continued)									
61-18	0.0	Pipe-corrugated metal Diameter = 36 inches	58	--	--	77	77	149	149
61-22	4.7	Pipe-corrugated metal Diameter = 36 inches	70	Governmental, utility	Governmental, utility	82	82	156	156
61-24	0.0	Swale-concrete Depth = 1.5 feet Bottom width = 0 Side slopes = 3.5:1	89	--	--	82	82	156	156
80-0	10.8	Pipe-concrete Diameter = 18 inches	21	Single-family residential	Single-family residential, government	18	18	31	31
80-2	0.0	Pipe-concrete Diameter = 30 inches	80	--	--	18	18	31	31
80-4	16.0	Pipe-concrete Diameter = 36 inches	136	Single-family residential, transportation utility	Single-family residential, transportation	41	41	68	68
84-0	8.3	Swale-grass Depth = 1.5 feet Bottom width = 0 Side slopes = 5:1	16	Single-family residential	Single-family residential	6	6	10	10
88-0	10.2	Swale-grass Depth = 1.5 feet Bottom width = 0 Side slopes = 5:	42	Single-family residential	Single-family residential	9	9	14	14
84-4	13.3	Swale-grass Depth = 1.5 feet Bottom width = 0 Side slopes = 3.5:1	43	Single-family residential	Single-family residential	21	21	34	34
84-8	27.3	Swale-grass Depth 2.5 feet Bottom width = 0 Side slopes = 3.5	144	Single-family residential, governmental	Single-family residential, governmental	55	55	86	86

Source: W. G. Nienow Engineering Associates and SEWRPC.

the storm sewer and drainage channel. Where these stormwater flows exceed the capacities of the conveyance facilities, surface ponding, flooding, and surcharging of upstream or downstream drainage facilities may be expected to occur.

Identified Problem Areas

The calculated capacities of each of the components of the existing drainage system were compared to the anticipated stormwater flow rates to identify those areas where problems may be expected under design storm conditions. As already noted, the evaluation considered the capacity of the minor system components in relation to the stormwater flows and volumes generated by a 10-year recurrence interval rainfall event; and the capacity of the major system components in relation to the stormwater flows and volumes generated by a 100-year recurrence interval rainfall event. In identifying existing and potential problems in the existing system, consideration was given to the potential impact of excessive flows. In some cases, problems were not created even though the capacity of the system component was exceeded--for example in inundated areas that were undeveloped and in which no buildings, transportation facilities, or other damage-prone improvements were affected.

Map 15 shows the locations of those existing system components which have inadequate hydraulic capacity and the attendant problems under existing and planned land use conditions. A brief description of these problems is provided in Table 29. The identified problems can be grouped into one of the following two general types:

- The hydraulic capacity of a culvert, storm sewer, or open channel is exceeded under both existing and planned land use conditions and may be expected to result in the inundation of adjacent streets and associated urban development.

- The hydraulic capacity of a culvert, storm sewer, or channel is not exceeded under existing land use conditions but is expected to be exceeded under planned land use conditions and may be expected to result in the inundation of adjacent streets and associated urban development.

In addition, areas of significant erosion and sedimentation related to stormwater drainage were also identified, as set forth in Chapter III of this report.

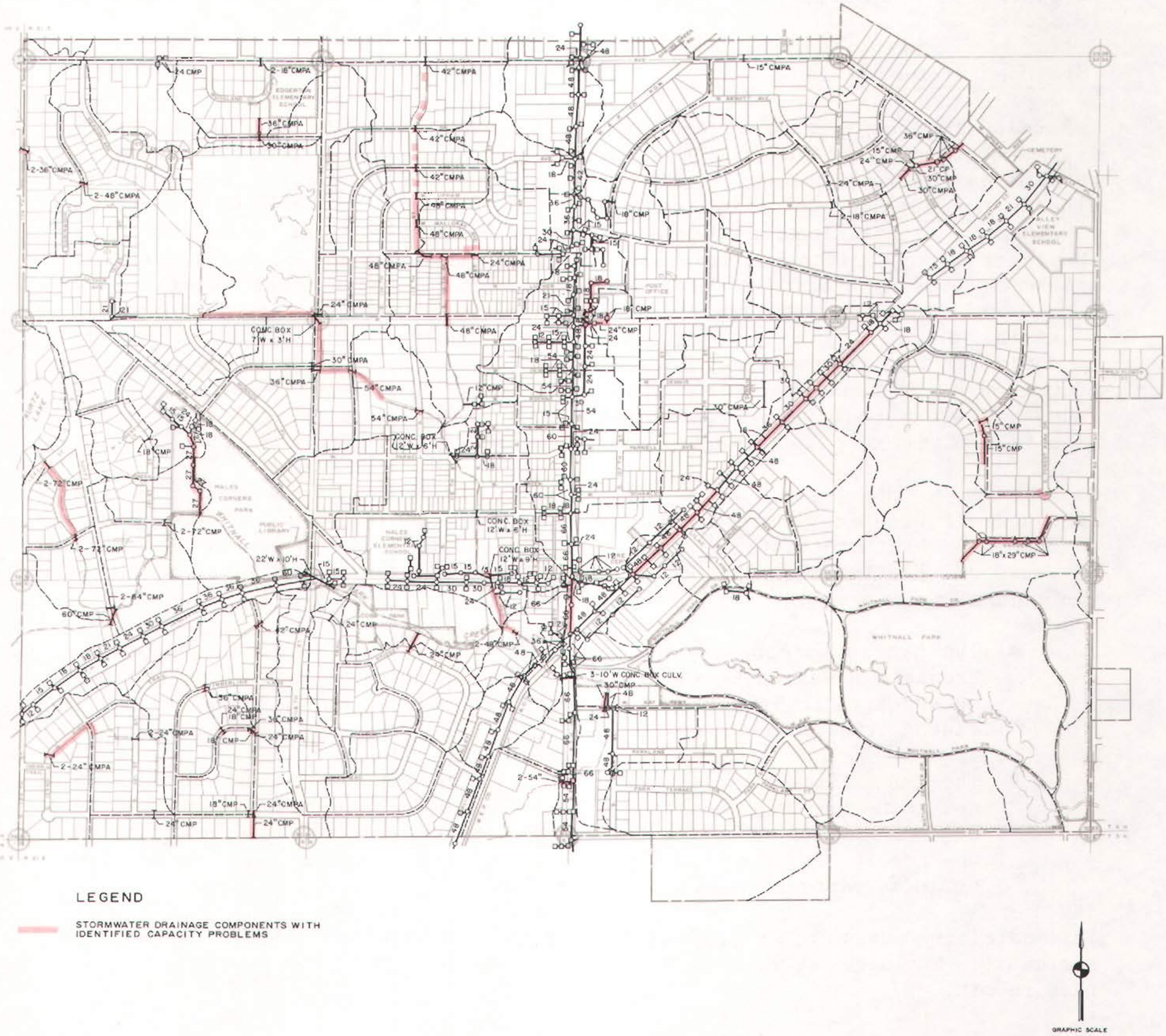
DESCRIPTION AND EVALUATION OF ALTERNATIVE STORMWATER MANAGEMENT APPROACHES

Introduction

As indicated in Chapter IV of this report, urban land use within the planning area may be expected to increase by about 25 percent between 1980 and the year 2000. This urbanization may be expected to produce an increase in the peak rate of stormwater runoff and in the volume of runoff for a given storm event. Stormwater runoff from urban land also contains different types--and, in some cases, increased amounts--of pollutants compared to stormwater runoff from

Map 15

IDENTIFIED PROBLEM AREAS IN THE EXISTING VILLAGE OF HALES CORNERS STORMWATER DRAINAGE SYSTEM UNDER PLANNED LAND USE CONDITIONS



Source: W. G. Nienow Engineering Associates and SEWRPC.

Table 29

IDENTIFIED PROBLEM AREAS IN THE EXISTING HALES CORNERS STORMWATER DRAINAGE SYSTEM UNDER EXISTING AND PLANNED LAND USE CONDITIONS

Subwatershed and Subbasin	System Component[a]	Location	Component Description	Problem Description
1-18	Minor	124th Street at Marquette Drive	2 No. 36 corrugated metal pipe arches	Inadequate capacity of culvert. Inundation of arterial street results
1-28	Minor	Robinwood Lane at Robinwood Court	2 No. 48 corrugated metal pipe arches	Inadequate capacity of culvert. Inundation of residential land and street results
1-40	Minor	Grange Avenue at Monaco Lane	Grass swale 2.5 feet deep, 4-foot bottom width, 20:1 side slope	Hydraulic capacity of swale is exceeded. Storm flows inundate arterial street and adjacent residential and commercial land
1-44	Minor	Grange Avenue at 116th Street	Grass swale 2.5 feet deep, 4-foot bottom width, 10:1 side slope and driveway culvert No. 24 corrugated metal arches	Hydraulic capacity of swale is exceeded. Storm flows inundate arterial street and adjacent residential and commercial land
5-4	Minor	Woodside Drive at 118th Street extended	No. 30 and 36 corrugated metal pipe arches	Hydraulic capacity of pipe is exceeded. Storm flows inundate adjacent residential land and street
1-48	Major	116th Street from Grange Avenue to Denis Avenue	Grass swale 6 feet deep, 2-foot bottom width, 5:1 side slopes and No. 48 corrugated metal pipe arch	Capacity of swale and driveway culverts is exceeded. Storm flows inundate residential land and street
1-56	Major	115th Street at Denis Avenue extended	No. 54 corrugated metal pipe arch and grass swale	Inadequate capacity of culvert. Inundation of residential land and street results
1-64	Minor	113th Street at Rockney Avenue extended	No. 54 corrugated metal pipe arch	Inadequate capacity of culvert. Inundation of residential land and street results
7-10	Minor	113th Street at Woodside Drive	Grass swale and No. 42 corrugated metal pipe arch	Hydraulic capacity of pipes is exceeded. Storm flows inundate adjacent residential land and street
7-16	Minor	113th Street at Abbot Avenue	Grass swale and No. 48 corrugated metal pipe arch	Hydraulic capacity of pipes is exceeded. Storm flows inundate adjacent residential land and street
7-20	Minor	113th Street at Mallory Avenue	Grass swale and No. 48 corrugated metal pipe arch	Hydraulic capacity of pipes and swale is exceeded. Storm flows inundate adjacent residential land and cause diversion
7-24	Minor	Copeland Avenue at 113th Street	No. 48 corrugated metal pipe arch	Hydraulic capacity of pipe is exceeded. Storm flows inundate adjacent residential land and street
8-0	Major	Copeland Avenue at 111th Street	Grass swale and culvert	Hydraulic capacity of culvert is exceeded. Inundation of residential land and street results

Table 29 (continued)

Subwatershed and Subbasin	System Component[a]	Location	Component Description	Problem Description
8-4	Major	Copeland Avenue from 111th Street to 112th Street	Grass swale and culvert	Hydraulic capacity of culvert and swale is exceeded. Inundation of residential and institutional land and street results
7-28	Minor	112th Street north of Grange Avenue	Grass swale and No. 48 corrugated metal pipe arch	Hydraulic capacity of pipes and swale is exceeded. Storm flow backwater inundates upstream streets, residential and institutional buildings and lands
7-30	Minor	112th Street at Grange Avenue	No. 48 corrugated metal pipe arch	Inadequate capacity of culvert. Inundation of upstream residential and institutional land and buildings and street results
1-88	Major	111th Street extended from Godsell Road to Janesville Road	Open channel and 72-inch-diameter driveway culvert	Hydraulic capacity of culvert and channel is exceeded. Inundation of residential buildings results
1-96	Minor	111th Street from Janesville Road to south	Open channel and 4.8 foot x 5.0 foot driveway box culvert	Hydraulic capacity of culvert and channel is exceeded. Inundation of residential building results
20-50	Major	124th Street west of Kurtz Road	2 72-inch-diameter corrugated metal pipes	Inadequate capacity of culvert. Inundation of residential land and buildings and street results
20-51	Major	Goodsell Road west of Kurtz Road	Grass swale 6 feet deep, 3-foot bottom width, 5:1 side slopes 72-inch-diameter corrugated metal pipe	Capacity of swale and pipes is exceeded. Storm flows inundate residential land and buildings and street
32-4	Minor	Ridge Trail from 122nd Street to 123rd Street	Swale and culverts	Hydraulic capacity of swale and cross-culverts is exceeded. Inundation of residential land and street results
32-6	Minor	Ridge Trail extended from 123rd Street to west	Cross-culvert	Hydraulic capacity of cross-culverts is exceeded. Inundation of resi-dential land and street results
31-12	Major	Kurtz Road at Janesville Road	No. 60 corrugated metal pipe arch	Inadequate inlet capacity of pipe. Storm flows inundate adjacent residential and agricultural land
33-4	Minor	Village Hall site	27-inch-diameter concrete pipe	Hydraulic capacity of pipe is exceeded. Storm flows inundate institutional land
35-4	Minor	118th Street at Parkview Lane	24-inch-diameter corrugated metal pipe	Hydraulic capacity of pipe is exceeded. Storm flows inundate streets and adjacent residential land
35-6	Minor	118th Street at Indian Trail	No. 24 corrugated metal pipe arch and 18-inch-diameter corrugated metal pipe	Hydraulic capacity of pipe is exceeded. Storm flows inundate streets and adjacent residential land

Table 29 (continued)

Subwatershed and Subbasin	System Component[a]	Location	Component Description	Problem Description
35-8	Minor	Indian Trail west of 118th Street	No. 36 corrugated metal pipe arch	Hydraulic capacity of pipes is exceeded. Storm flows inundate streets and adjacent residential land
35-12	Minor	Indian Trail and Timberline Lane west of 118th Street	No. 24 corrugated metal pipe arch and 18-inch-diameter corrugated metal pipe No. 36 corrugated metal pipe arch	Inadequate capacity of culvert. Inundation of residential land and street results
36-4	Minor	Bridget Lane at Arrowhead Trail	24-inch-diameter corrugated metal pipe and grass swale 1 foot deep, 0-foot bottom width, 6:1 side slopes	Hydraulic capacity of pipe is exceeded. Storm flows exit via grass swale
1-104	Minor	Whitnall Park Creek from 111th Street to Forest Home Avenue	Driveway cross-culverts	Hydraulic capacity of cross-culverts is exceeded. Inundation of Commercial and residential land and street results
60-12	Minor	Kelm Road at 108th Street	2 54-inch-diameter concrete pipes	Capacity of pipe is exceeded. Arterial street is inundated. Caused by upstream diversion into this basin by inadequate capacity in Forest Home Avenue sewer. This is a problem only under planned land use conditions
40-3	Minor	Grange Avenue at 108th Street	18-inch-diameter corrugated metal pipe	Inadequate capacity in cross-culvert causes inundation of arterial street and commercial lands
40-4	Minor	Grange Avenue at 108th Street	24-inch-diameter corrugated metal pipe	Capacity of sewer is exceeded. Storm flows inundate arterial street and commercial lands
42-0	Minor	108th service street at Grange Avenue	18-inch-diameter concrete pipe	Inadequate capacity of pipe. Inundation of service road and commercial lands
45-0	Minor	Forest Home Avenue from Grange Avenue to Denis Avenue extended	24-inch-diameter concrete pipe	Inadequate capacity of sewer. Inundation of residential and commercial land and arterial street results
45-6	Minor	Forest Home Avenue from Denis Avenue to Parnell Avenue	30-inch to 36-inch-diameter concrete pipes	Inadequate capacity of sewer. Inundation of residential and commercial land and arterial street results
45-10	Minor	Forest Home Avenue from Scharles Avenue to Janesville Road	48-inch-diameter concrete pipe	Capacity of sewer is exceeded. Storm flows inundate arterial street, commercial and residential lands
82-6	Major	Kay Parkway at 106th Street	30-inch-diameter corrugated metal pipe	Inadequate capacity of pipe. Inundation of street and residential lands results

Table 29 (continued)

Subwatershed and Subbasin	System Component[a]	Location	Component Description	Problem Description
47-0	Minor	Meadow Park Drive at Bonnie Lane	15-inch-diameter corrugated metal pipe	Inadequate capacity of pipe. Inundation of street and residential lands results
48-0	Minor	Forest Park and Meadow Park	Cross 12-inch-diameter corrugated metal pipe culvert	Hydraulic capacity of cross-culvert is exceeded. Inundation of residential land and street results
48-4	Minor	Garden Court at Forest Park extended	No. 24 18-inch x 29-inch corrugated metal pipe arch	Inadequate capacity of pipe. Inundation of street and residential lands results
48-8	Minor	Garden Court at Forest Park	No. 24 18-inch x 29-inch corrugated metal pipe arch	Inadequate capacity of pipe. Inundation of street and residential lands results
61-10	Minor	Brookside Drive at Allenwood Lane	No. 30 corrugated metal pipe arch	Hydraulic capacity of pipe is exceeded. Storm flows inundate street
61-16	Minor	Brookside Drive at Allenwood Lane	36-inch-diameter corrugated metal pipe	Hydraulic capacity of pipe is exceeded. Storm flows inundate street
61-18	Minor	Brookside Drive west of Edgerton Avenue	36-inch-diameter corrugated metal pipe	Hydraulic capacity of pipe is exceeded. Storm flows inundate street
61-22	Minor	Brookside Drive at Edgerton Avenue	36-inch-diameter corrugated metal pipe	Hydraulic capacity of pipe is exceeded. Storm flows inundate street

[a] *Anticipated exceedance of the hydraulic capacity of the system structures is based on calculated stormwater flows during a 10-year recurrence storm event for the minor system components and a 100-year recurrence interval storm event for the major system components.*

Source: W. G. Nienow Engineering Associates and SEWRPC.

undeveloped land. Increased urbanization, accordingly, may be expected to place increased demands on the existing stormwater management system, requiring additional engineered drainage facilities to accommodate the increased loadings. These facilities are designed to minimize the occurrence of stormwater management problems and the associated disruption of the urban environment and adverse water quality impacts.

To accommodate these increased loadings and to abate existing, as well as future, stormwater management problems, several stormwater management approaches were considered. These alternative approaches to stormwater management were first evaluated on a conceptual basis, considering the technical feasibility, applicability, and advantages and disadvantages of each approach. Elements of the most feasible approaches were then incorporated into three systems-level alternative stormwater management plans for the Village of Hales Corners area as described later in this chapter.

Alternative Stormwater Management Approaches

Alternative approaches to stormwater management which were considered for application in the Hales Corners area included conventional conveyance, centralized detention, onsite detention, centralized retention, onsite retention, "blue-green" system, and nonstructural measures. Pertinent characteristics of each of these alternative approaches are set forth in Table 30. Based upon consideration of these characteristics, the general feasibility and applicability of each approach to the Hales Corners area were determined.

Conveyance: The conveyance approach would utilize storm sewers and concrete-lined or composite channels and related appurtenances to provide for the collection and rapid conveyance of stormwater runoff to the receiving streams within the urban service area. The major advantages of this type of system are the minimization of onsite inconvenience because the water is rapidly collected and conveyed downstream; and ready applicability to both existing and newly developing urban areas. Nonpoint source pollution abatement measures appropriate under this approach would be those that do not involve storage or infiltration of stormwater such as: increased street and parking lot sweeping, improved leaf collection, construction site erosion and pet waste control, and public education programs. Properly designed, constructed, and maintained storm sewers present no hazard to the public health and safety; and the hydraulic design procedures, as well as the construction techniques, are simple, well developed, and commonly used. The disadvantages of the conveyance approach are that downstream peak flows and stages and areas of inundation are usually increased; pollutants are not removed from the runoff; there is little potential for multipurpose uses of the system; and this approach usually has a high capital cost.

Since most of the developed portion of the Village of Hales Corners currently relies on an engineered stormwater drainage system, further application of the conveyance approach would represent a continuation of the existing practices and policies. Hence, this approach would likely be understood and well accepted by local public officials and citizens alike. Technically, the existing stormwater problems experienced by the Village, as well as probable future problems, could be abated using the conveyance approach. However, there would be some concern about the downstream impacts of the conveyance system. Given the advantages of the conveyance approach, it was utilized in the development of alternative stormwater management plans for the Hales Corners area.

Centralized Detention: A centralized detention approach would utilize major surface or subsurface detention facilities to provide for the temporary storage of stormwater runoff for subsequent slow release to downstream channels or storm sewers. The centralized detention facilities would be located on a few strategic sites to maximize benefits, and not all areas would drain to a centralized facility. The centralized detention facilities can be supplemented by improved conveyance facilities as may be necessary. Nonpoint source pollution control can be provided by various types of centralized detention facilities, along with measures such as construction site erosion control and pet waste control.

The major advantages of a centralized detention approach are that if properly applied, the facilities can limit the effects of urban development on downstream discharges and areas of inundation; a substantial amount of sediment and other particulate pollutants are removed; the size and resultant cost of

Table 30

CHARACTERISTICS OF ALTERNATIVE STORMWATER MANAGEMENT APPROACHES

Characteristic	Conveyance	Centralized Detention	Onsite Detention	Centralized Retention	Onsite Retention	"Blue-Green" System	Nonstructural
Function	Provide for the collection of stormwater runoff and the rapid conveyance of stormwater from the area so as to minimize disruptive and possibly damaging surface ponding in streets and low-lying areas and possible inundation of residential and other sites and structures	Provide for the temporary storage of stormwater runoff in the service area for subsequent slow release to downstream channels or storm sewers, thus minimizing disruption and damage within and downstream of the service area and reducing the required size and therefore cost of any constructed downstream conveyance facilities	Provide for the temporary storage of stormwater runoff at small sites located close to the source of generation of the runoff to be controlled	Provide for the storage of stormwater runoff for subsequent evaporation and infiltration to groundwater, thus removing the area runoff from the surface drainage system and reducing the required size and therefore cost of downstream conveyance facilities	Provide for the storage of stormwater runoff for subsequent evaporation and infiltration to groundwater at small sites located close to the source of generation of the runoff to be retained	Provide for the temporary storage and/or conveyance of stormwater runoff using natural or vegetated channels which slow the runoff rate and allow a portion of the runoff to infiltrate into the soil	Primarily to reduce damages from excessive stormwater runoff and flooding, rather than controlling the runoff rates or flood levels themselves
Components Principal	Improved open drainage channels and storm sewers	Surface or subsurface detention facilities	Parking lot storage facilities Rooftop storage facilities Relatively small detention facilities Swales, over-sized channels, and diversions	Surface retention facilities Construction site erosion and pet waste control	Relatively small surface retention facilities Subsurface infiltration systems (drywells, etc.)	Open vegetated channels Swales Natural surface depressions and wetlands Over-sized channels Ponds and lakes Construction site erosion and pet waste control	Floodproofing of structures Relocation of structures Land use regulations Open space and floodland preservation Increased street and parking lot sweeping Improved leaf collection Construction site erosion and pet waste control
Secondary	Storm inlets Culverts Outfalls Manholes Increased street and parking lot sweeping Improved leaf collection Construction site erosion and pet waste control	Open drainage channels Storm inlets Culverts Outfalls Manholes Inlet and outlet works and/or pumping facilities Construction site erosion and pet waste control	Same as centralized detention	Open drainage channels Storm inlets Culverts Outfalls Manholes	Same as centralized retention	A "blue-green" system may be supplemented with storm sewers, storm inlets, outfalls, manholes, and culverts	Can be used with other stormwater management facilities
Applicability	Suitable for installation in existing and newly developing urban areas	Most suitable for incorporation in newly developing urban areas if suitable surface or subsurface sites are available	Suitable for installation in existing and newly developing urban areas. May be more suitable than centralized detention in many existing urban areas because of reduced site requirements	Most suitable for incorporation in newly developing urban areas with permeable soils but may be used in existing urban areas if suitable sites are available	Same as centralized retention	Suitable for incorporation in developing urban areas. A "blue-green" system may be undesirable in moderate- or high-density urban development and it may be difficult to develop an economically feasible open channel system which can accommodate the high peak flows from developed urban areas	Suitable for implementation in existing and and newly developing urban areas
Downstream Impact Quantity	Tends to significantly increase--relative to predevelopment conditions--downstream discharges, stages, and areas of inundation	May be designed to cause no significant increase, relative to predevelopment conditions, in downstream discharges, stages, and areas of inundation. Decreased discharges, stages, and areas of inundation are possible	Same as centralized detention, although onsite detention facilities are designed for smaller storms and shorter detention times than are centralized detention facilities	Same as centralized detention	Same as onsite detention	May be designed to allow storm runoff to be temporarily stored in a low gradient channel, reducing downstream peak discharge	Minimal impact, although preservation of open space lands may maintain higher levels of natural storage and infiltration than if these lands were developed
Quality	A relatively low level of removal of pollutants from nonpoint sources would be achieved	Provides for removal, by the natural settling process, of sediment and other suspended material, thus reducing the pollutant loading on receiving waters. Provides an opportunity for physical-chemical treatment such as disinfection, coagulation-flocculation, and swirl concentration	Provides some pollutant removal, but may be less than by centralized detention if detention time is shorter. Less opportunity for physical-chemical treatment than with centralized facilities	Provides removal of suspended and settleable pollutants but dissolved pollutants may percolate to the water table without reduction	Same as centralized retention	Provides for removal of pollutants in storm runoff by infiltration into the soil, settling of solids, and filtration by vegetation	Minimal impact
Multipurpose Capability	Storm sewers serve only a stormwater collection and conveyance function Open drainage channels can provide a focus for development of linear park and open space areas	Quantity control Quality control Can provide park and open space areas	Same as centralized detention	Quantity control Quality control Recreation benefits Aesthetic benefits Groundwater recharge Wildlife habitat	Same as centralized retention	Quantity control Quality control Park and open space areas Aesthetic benefits Wildlife habitat	Park and open space areas
Operation and Maintenance Requirements	Periodic cleaning and repair of storm inlets, channels, and storm sewers required Maintenance of open channel lining material required Increased street and parking lot sweeping Improved leaf collection	Pumping and/or inlet-outlet control operation and maintenance required Insect and odor control may be required Periodic cleaning and maintenance of facility lining required Dam maintenance may be required	Same as centralized detention except that maintenance of onsite facilities may be less intensive but required at a larger number of sites	Operation and maintenance required Sediment removal required Insect control may be required Weed and algae control and water pollution control may be required Bank maintenance required	Same as centralized retention except that maintenance of onsite facilities may be less intensive but required at a larger number of sites	Periodic cleaning of channels and inlets required Maintenance of open channel vegetative cover required	Increased street and parking lot sweeping Improved leaf collection
Impact on Sanitary Sewer System	Surcharging of storm sewers accompanied by inundation of streets may result in infiltration of stormwater from storm sewers to adjacent sanitary sewers and inflow of stormwater into sanitary sewers through manholes. Flow in excess of stormwater channel capacity may also result in surface inundation and inflow to sanitary sewers	Runoff volumes in excess of available storage volume, and runoff rates in excess of the capacity of tributary storm sewers and channels, accompanied by inundation of streets may result in infiltration of stormwater from storm sewers to adjacent sanitary sewers and inflow of stormwater into sanitary sewers through manholes	Same as centralized detention	Percolation waters may result in excessive infiltration of stormwater into sanitary sewers	Same as centralized retention	Exceedance of channel capacity accompanied by inundation of streets may result in infiltration of stormwater into ajacent sanitary sewers and inflow of stormwater into sanitary sewers through manholes	Minimal
Hazards	Minimal hazard associated with storm sewers High velocities in improved open channels may pose a safety hazard, particularly to children	Minimal hazard associated with subsurface storage, but surface storage may pose a health and safety hazard, particularly to children	Ponded water in parking lots, small detention facilities, and swales may pose a health and safety hazard, particularly to children, though the size and depth of onsite facilities are frequently minimal	Ponded water may pose a health and safety hazard, particularly to children	Ponded water may pose a health and safety hazard, particularly to children, though the size and depth of onsite facilities are frequently minimal	Flowing channels may pose a safety hazard, particularly to children	Minimal
Hydrologic-Hydraulic Analysis	Requires determination only of the peak rate of flow associated with a specified recurrence interval. This is normally obtained with the relatively simple and widely accepted Rational method	Requires determination of both a peak rate and a volume of inflow associated with a specified recurrence interval, an estimate of allowable outflow rate and storage, and design of pumps or control works to satisfy the discharge conditions. A hydrograph-developing technique must be used to simulate peak flow and volume conditions	Same as centralized detention	Requires determination of both a peak rate and a volume of inflow associated with a specified recurrence interval and an estimate of percolation rate and storage to satisfy conditions. A hydrograph-developing technique must be used to simulate peak flow and volume conditions	Same as centralized retention	Requires determination of peak rate of flow, flow volumes, velocity, and flow depths. This can be obtained by using the hydrograph-developing technique	Requires delineation of areas affected by flooding and poor stormwater drainage. The Hydrologic Engineering Center (HEC-2) model may be used to determine flood stages under various recurrence interval storm events
Ability to Meet Stormwater Management Objectives and Supporting Standards	All objectives and supporting standards can be met	All objectives and supporting standards can be met	All objectives and supporting standards can be met	All objectives and supporting standards can be met	All objectives and supporting standards can be met	Some objectives and supporting standards would probably not be met because of the difficulty in accommodating the design flows efficiently and economically using this approach	This alternative would not satisfy the recommended objectives and supporting standards by itself, and must be combined with other alternatives

Source: SEWRPC.

downstream conveyance facilities can be reduced; and the facilities can be combined with recreation and open space to provide multipurpose-use areas. The disadvantages of a centralized detention approach are that large, flat, open areas are usually required, thereby reducing the availability of adequate potential sites; the facility may not be cost-effective if the site costs cannot be offset by providing smaller conveyance facilities downstream; the operation and maintenance requirements may be substantial; for a permanent pool facility, the ponded water may be perceived as a public health and safety hazard; odor and insect problems may be produced; and the hydraulic design techniques and analytic procedures are more involved than those for conventional storm sewerage systems. While readily applicable as an integral part of large-scale urban development proposals, the approach is more difficult to apply to areas of existing urban development.

Within the Hales Corners Village area, centralized detention facilities could be used to abate some of the existing and potential stormwater runoff problems. Higher maintenance requirements and an opposition to ponds or dry basins in urban areas by some citizens for aesthetic or health and safety reasons may make this approach unacceptable in the service area. However, because of its potential benefits, the centralized detention approach was utilized in the development of alternative stormwater management plans for the Hales Corners area.

Onsite Detention: Like centralized detention, onsite detention provides for the temporary storage of stormwater runoff, but the storage sites are located close to, or at, the source of runoff generation. Hence, these detention sites tend to be smaller than centralized detention facilities. Onsite detention measures include parking lot storage, swales, and large channels with gentle slopes. To a limited extent, onsite detention is included in all alternative approaches to stormwater management in the Hales Corners area, since the Commission recommends the preservation of all remaining floodlands, wetlands, and other natural open areas, all of which effectively serve as onsite detention areas. The onsite detention systems, like the centralized detention systems, can also be supplemented by improved conveyance facilities. Nonpoint source control can be achieved by various types of onsite detention measures, along with measures such as construction site erosion control and pet waste control.

The advantages of the onsite detention approach are similar to those of the centralized detention approach with regard to downstream water quantity and quality control and to the potential for reducing the size requirements of downstream conveyance systems. Onsite facilities, however, have smaller unit site requirements than do centralized facilities and are therefore more readily applicable--although not without difficulty--in existing as well as newly developing urban areas. Onsite facilities may be less suitable for multipurpose uses such as recreation and open space, but more suitable for uses such as parking or yard space in residential areas. Disadvantages of the onsite detention approach are that maintenance requirements may be substantial, although probably less intensive than for centralized facilities; the ponded water in a detention pond may cause localized inconvenience and represent a health and safety hazard; odor and insect problems may be produced; hydraulic design techniques are more involved than for conveyance systems; and the costs may be high if not offset by smaller downstream conveyance systems. While readily applicable as an integral part of large-scale urban development proposals, the concept is difficult to effectively implement with small-scale, piecemeal development proposals and in areas of existing urban development.

The onsite detention approach could be used to abate the existing and potential stormwater runoff problems in the planning area. Although there may be some citizen opposition to ponded water in urban areas, the smaller affected sites and greater availability of potential sites may make this approach more acceptable than the centralized approach. Because of its potential benefits, the onsite detention approach was utilized in the development of alternative stormwater management plans for the Hales Corners area.

Centralized Retention: Retention facilities provide for the storage of stormwater runoff for subsequent evaporation and/or infiltration. This approach can also be supplemented by improved conveyance facilities. Nonpoint source control can be achieved by various types of centralized retention facilities, along with measures such as construction site erosion control and pet waste control.

The major advantages of the centralized retention approach are that if properly applied, the facilities can limit the effects of urban development on downstream peak discharges and areas of inundation; sediment and other particulate pollutants are removed; the size and resultant cost of downstream conveyance facilities can be reduced and the need for reconstruction sometimes avoided; the facilities can be combined with recreation and open space to provide multipurpose-use areas; and the facilities can provide groundwater recharge. The disadvantages of the retention approach are that the facilities require large, flat, open areas; may be more expensive than detention facilities; less permeable soils require larger facilities; maintenance requirements are substantial; and the water quality of a permanent pool may be poor because of the generally higher pollutant levels of urban runoff. The effects on groundwater levels may create problems such as wet basements, costly operation of sump pumps, and excessive infiltration of clear water into sanitary sewers. Because of the large site requirements, this approach is generally suitable only in newly developing urban areas. Any permanently ponded water may present a potential health and safety hazard, and the hydraulic design and construction techniques are more involved than for conveyance systems.

While centralized retention facilities could be utilized to abate some of the existing and potential stormwater management problems in the Hales Corners area, there has been no demonstrated need or desire for the additional multipurpose use benefits which a retention facility provides. Accordingly, given the generally higher cost and maintenance requirements of a retention facility compared to those of a detention facility, centralized retention facilities were not considered further in the development of alternative stormwater management plans for the Hales Corners area.

Onsite Retention: Like centralized retention, onsite retention provides for the temporary storage and subsequent infiltration and/or evaporation of stormwater runoff, but the storage sites are located close to, or at, the source of runoff generation. Hence, these sites tend to be smaller than centralized retention facilities. Onsite retention measures include aboveground and subsurface infiltration systems. Nonpoint source control measures appropriate under the onsite retention approach may include various types of onsite retention facilities, construction site erosion control, and pet waste control.

The advantages of the onsite retention approach are similar to those of the centralized retention approach with regard to water quantity and quality control downstream, and to the potential for reducing the size requirements for downstream conveyance systems. However, onsite facilities have smaller unit site requirements, thereby being more readily applicable--although not without difficulty--in existing as well as newly developing urban areas. Onsite facilities may be less suitable for multipurpose uses such as recreation and open space, but more suitable for uses such as parking or yard space in residential areas. Disadvantages of the onsite retention approach are that maintenance requirements may be substantial; the ponded water may cause localized inconvenience and represent a health and safety hazard; odor and insect problems may be produced; hydraulic design techniques are more involved than for conveyance systems; and the costs may be high if not offset by smaller downstream conveyance systems. The effects on groundwater levels may create severe problems such as wet basements, costly operation of sump pumps, and excessive infiltration of clear water into sanitary sewers. While readily applicable as an integral part of large-scale urban development proposals, the concept is more difficult to implement effectively and dependably with small-scale, piecemeal development proposals and in areas of existing urban development.

While the onsite retention approach could be used to abate some of the existing and potential stormwater runoff problems in the Village, because of the general lack of soils conducive to infiltration in the area, the potential for increased nuisance by facilities, and the potential adverse effects on groundwater levels, onsite retention facilities were not considered further in the development of alternative plans for the Hales Corners area.

"Blue-Green" System: The "blue-green" stormwater management system consists of vegetation-lined channels, preferably "free-form" as opposed to geometrically shaped and interconnected natural surface depressions, and wetlands. Such a system provides for the temporary storage and conveyance of stormwater runoff in the vegetation-lined channels and associated depression and wetland areas, which slow the runoff and allow ponding and infiltration. The drainage system of an area may consist almost entirely of "blue-green" channels, or it may be supplemented by other management measures including storm sewers. Nonpoint source control measures appropriate under the "blue-green" approach may include certain types of stormwater detention and retention facilities, turf-lined open channels, construction site erosion control, and pet waste control.

The advantages of the "blue-green" approach are that downstream peak flows may be reduced; pollutants in storm runoff may be removed by filtration through the soil and vegetation and by sedimentation; the "free-form" open channels and related drainage areas can serve as part of park and open space sites following the multi-use concept; construction costs may be lower; and the aesthetic qualities of a "natural" drainage system may be attractive to some citizens. The disadvantages of the "blue-green" approach are that it becomes increasingly uneconomical to develop an open channel system which can effectively accommodate the high peak flows generated from medium- to high-density urban areas; the channels generally are difficult to incorporate into developed urban areas served by storm sewers; the flowing channels may be perceived as a safety hazard; such systems often are not properly cleaned and maintained by the responsible authorities; and some citizens and local public officials may not desire open channel flow in urban areas.

Within the Hales Corners Village area, "blue-green" system facilities could be used to abate existing and potential stormwater runoff problems. Although there may be some citizen opposition to the short-term standing and flowing water, and to the more extensive land areas required, this approach was utilized in the development of alternative stormwater management plans for the Hales Corners area.

Nonstructural Measures: The nonstructural approach to stormwater management primarily involves reducing damages from unusually high stormwater runoff and inundation rather than controlling the runoff rates or inundation levels themselves. Nonstructural measures include structure floodproofing, relocation of structures, land use regulations, and open space and floodland preservation. Appropriate nonstructural nonpoint source abatement measures may include increased street and parking lot sweeping, improved leaf collection, construction site erosion control, and pet waste control. The nonstructural approach is not in itself an alternative in that in medium- to high-density urban areas the existing and potential stormwater management problems usually cannot be abated by nonstructural measures alone, although the impact of these problems may be reduced. Hence, nonstructural measures are usually considered only in combination with the alternative approaches described above.

The advantages of the nonstructural approach are that the measures are suitable for use in existing as well as newly developing urban areas; the measures are highly flexible and adaptable to different situations; the cost of nonstructural measures is generally low; the measures can often be used to create needed park and open space; and there are few hazards associated with nonstructural measures. The disadvantages of the nonstructural approach are that downstream water quantity and quality is generally not controlled; most stormwater problems are not abated; land condemnation may be necessary; and some measures may benefit relatively few individuals.

Because of its applicability under a wide array of situations, the nonstructural approach was utilized in the design of alternative stormwater management plans for the Hales Corners area, but only in conjunction with other alternative approaches.

ALTERNATIVE STORMWATER MANAGEMENT PLANS

Utilizing the alternative stormwater management approaches, as described above, three alternative stormwater management plans were developed for the Hales Corners area: 1) a conveyance plan; 2) a centralized detention plan; and 3) a decentralized detention plan.

During the alternative plan development and evaluation stage, components of the minor drainage system, such as storm sewers and off-channel detention facilities, were considered, as were such components of the major drainage system as major engineered drainage channels, natural watercourses, and on-channel detention facilities. In areas with existing or planned urban street patterns, the alternative plans included a complete system of minor system components. In areas planned to be developed for urban use but for which no street layout had been established, only certain components of the minor system such as trunk storm sewers, important open drainage channels, and centralized detention facilities could be explicitly considered. Smaller collector storm sewers and some onsite storage systems could be only implicitly considered through the simulation modeling. Roadside swales and attendant culverts, curbs and gutters, and inlets were considered only in a general manner in the development and evaluation of the alternative system plans. However, these details of the minor system, together with the major system, were specifically considered in the design and evaluation of the recommended plan.

Each alternative stormwater management plan also included nonpoint source pollution abatement measures which would be consistent and compatible with the proposed stormwater conveyance and storage facilities. This chapter describes the types and approximate cost of those pollution abatement measures which would be cost-effective and which would, at a minimum, prevent any increases in pollutant loadings under future development conditions. Since some stormwater conveyance and storage facilities are also effective in removing pollutant loadings, the pollutant removal effectiveness of these facilities was also estimated. More detailed designs and cost estimates for the recommended pollution abatement measures are provided in Chapter VIII. Construction site erosion controls are also addressed as part of the recommended plan.

In order to compare and evaluate the alternative plans, the Hales Corners Village area was divided into 14 hydrologic units. Each unit was comprised of two or more subbasins tributary to the same conveyance system component, or to a detention facility and its associated downstream conveyance system. A description of individual components and the estimated costs are presented for each hydrologic unit under each alternative plan. The hydrologic unit boundaries are shown on Maps 16, 17, and 18.

The three alternative plans were all designed to serve the Hales Corners Village proper. Stormwater management facilities for areas outside the Village but within the study area were not specifically designed, although the peak flow rates to be generated under each alternative at the locations where stormwater flows enter or leave the Village were considered in the design and evaluation of the alternative plans. An analysis of the impacts of the recommended stormwater management plan for the Hales Corners Village area on areas outside the Village but within the study area is specifically addressed in the recommended plan, and recommendations are made as appropriate.

Conveyance Alternative Plan

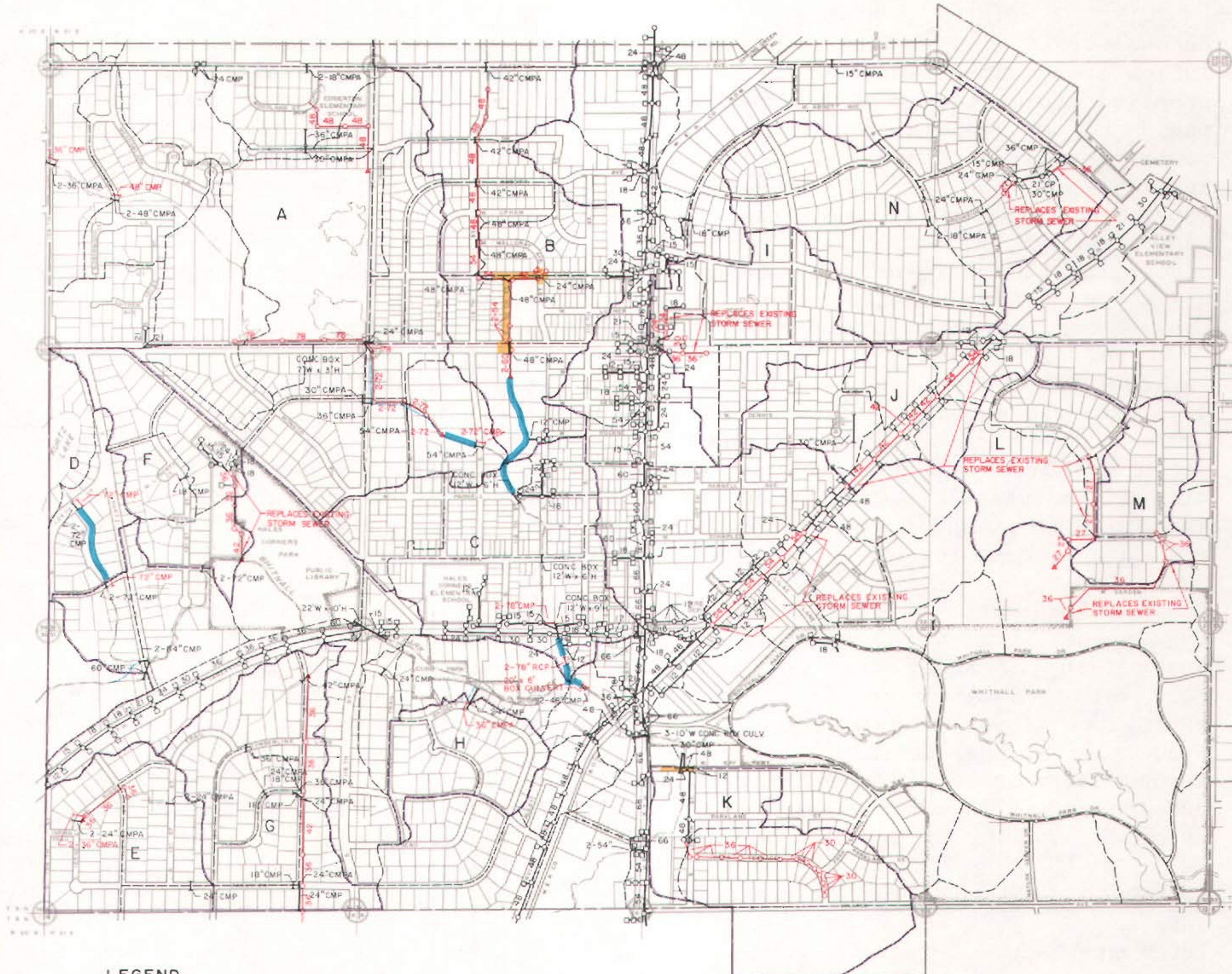
The conveyance alternative plan primarily involves the provision of new storm sewers and engineered open channels and attendant culverts to abate existing stormwater runoff problems and to effectively serve planned new urban development within the Village. Map 16 shows the location and alignment of new storm sewers and engineered open channels and attendant culverts proposed under the conveyance alternative. Table 31 presents the salient characteristics of the new storm sewers, channels, and attendant culverts comprising this alternative plan.

The conveyance alternative consists of 20,570 lineal feet of new storm sewers ranging in size from 24 to 78 inches in diameter. All new storm sewers are assumed to be constructed of reinforced concrete pipe. New sewer segments would discharge to surface streams or open channels from seven new outfalls, while three new sewer segments would discharge to existing storm sewers.

About 3,460 lineal feet of new engineered open channels would be provided under this alternative. The new engineered channels would be concrete lined, or combination concrete bottom and grass side slopes. The plan also consists of nine new culvert installations ranging in size from a 36-inch corrugated metal pipe to a 20-foot-wide by 6-foot-deep concrete box culvert.

Map 16

CONVEYANCE ALTERNATIVE PLAN FOR STORMWATER MANAGEMENT IN THE VILLAGE OF HALES CORNERS



LEGEND

- 24 — EXISTING STORM SEWER OR CULVERT AND SIZE IN INCHES
- EXISTING MANHOLE
- EXISTING CATCHBASIN OR INLET
- 24 — PROPOSED STORM SEWER OR CULVERT AND SIZE IN INCHES
- PROPOSED MANHOLE
- PROPOSED STORM SEWER OUTFALL
- PROPOSED OPEN CHANNEL OR SWALE
- PROPOSED OPEN CHANNEL OR SWALE IMPROVEMENT
- PROPOSED ROAD RECONSTRUCTION
- SUBBASIN BOUNDARY
- HYDROLOGIC UNIT BOUNDARY
- A — HYDROLOGIC UNIT IDENTIFICATION LETTER
- VILLAGE OF HALES CORNERS CORPORATE LIMITS

GRAPHIC SCALE
0 400 800 1200 FEET

Source: W. G. Nienow Engineering Associates; Graef, Anhalt, Schloemer & Associates, Inc.;and SEWRPC.

Table 31

SELECTED CHARACTERISTICS AND COSTS OF THE CONVEYANCE ALTERNATIVE HALES CORNERS STORMWATER MANAGEMENT PLAN

Project and Component Description	Estimated Cost: Capital	Estimated Cost: Annual Operation and Maintenance[a]
Western Portion of Village		
A. Northwest Branch-Whitnall Park Creek Improvements		
1. Add 36-inch culvert under 124th Street at Woodside Drive extended	$ 3,000	$ 0
2. Add 48-inch culvert under Robinwood Drive west of 121st Street extended	4,000	0
3. 1,140 feet of 78-inch storm sewer in Grange Avenue from east of Monaco Lane to 116th Street	270,000	-200
4. 380 feet of 48-inch storm sewer in 116th Street at Woodside Drive	50,000	-100
5. 610 feet of 48-inch storm sewer north of Woodside Drive from 116th Street to 118th Street extended	70,000	100
6. 1,200 feet of twin 72-inch storm sewer at Grange Avenue and 116th Street south to Denis Avenue and 116th Street, and east from Denis Avenue and 116th Street across 115th Street to 114th Street extended	480,000	0
7. Twin 72-inch culvert under 113th Street north of Parnell Avenue	15,000	0
8. Regrade 650 feet of channel west of 113th Street north of Parnell Avenue	30,000	0
9. Miscellaneous roadside swale and culvert improvements	60,000	0
10. Miscellaneous and contingencies	148,000	0
Subtotal	$1,130,000	$ -200
B. North Branch-Whitnall Park Creek Improvements		
1. 1,430 lineal feet of 48-inch storm sewer in 113th Street from south of Edgerton Avenue to Upham Avenue	$ 170,000	$ -300
2. 940 lineal feet of 54-inch storm sewer in 113th Street from Upham Avenue to Copeland Avenue and in Copeland Avenue from 113th Street to 112th Street	130,000	-200
3. 330 lineal feet of 42-inch storm sewer in Copeland Avenue from 111th Street to 112th Street	30,000	-100
4. 590 lineal feet of two 54-inch storm sewers in 112th Street from Copeland Avenue to Grange Avenue	160,000	0
5. 200 lineal feet of two 60-inch storm sewers in 112th Street extended from Grange Avenue to south	60,000	0
6. Regrade 1,100 feet of open channel in 112th Street extended from south of Grange Avenue to Parnell Avenue	40,000	0
7. Roadway reconstruction	80,000	0
8. Miscellaneous and contingencies	100,000	-100
Subtotal	$ 750,000	$ -700

Table 31 (continued)

Project and Component Description	Estimated Cost	
	Capital	Annual Operation and Maintenance[a]
C. North Branch-Whitnall Park Creek Junction Area Improvements		
1. Twin 78-inch culvert under driveway at Janesville Road and 111th Street extended.....	$ 25,000	$ 0
2. 400 feet of channel improvement in 111th Street from Janesville Road to south..........	20,000	0
3. 150 feet of channel improvement east of 111th Street................................	10,000	0
4. Twin 78-inch culvert under driveway at 111th Street one block south of Janesville Road.....	30,000	0
5. 20-foot x 6-foot box culvert under driveway at 111th Street south of Janesville Road......	40,000	0
6. Miscellaneous and contingencies..............	15,000	0
Subtotal	$ 140,000	$ 0
D. Upper Kelly Lake Discharge Channel Improvements		
1. Culvert under 124th Street...................	$ 10,000	$ 0
2. 760-foot channel improvement from 124th Street to Godsell Road......................	40,000	0
3. Culvert under Godsell Road...................	10,000	0
4. Miscellaneous and contingencies..............	10,000	0
Subtotal	$ 70,000	$ 0
E. Hale Park West Improvements		
1. 650 feet of 36-inch storm sewer in Ridge Trail from 122nd Street to 123rd Street.......	$ 60,000	$ -100
2. Culvert under 123rd Street at Ridge Trail.....	10,000	0
3. 250 feet of channel improvement at Kurtz Road	10,000	0
4. Miscellaneous and contingencies..............	10,000	0
Subtotal	$ 90,000	$ -100
F. Village Hall Area Improvements		
1. 540 feet of 36-inch storm sewer south of New Berlin Road at 120th Street extended..........	$ 50,000	$ 0
2. 350 feet of 42-inch storm sewer south of New Berlin Road at 120th Street extended..........	30,000	0
3. Miscellaneous and contingencies..............	10,000	0
Subtotal	$ 90,000	$ 0
G. Hale Park Central (118th Street) Improvements		
1. 250 feet of 36-inch storm sewer in 118th Street extended from village boundary to Parkview Lane................................	$ 20,000	$ 0
2. 890 feet of 42-inch storm sewer in 118th Street from Parkview Lane to Indian Trail.....	90,000	-200
3. 1,020 feet of 36-inch storm sewer in 118th Street from Indian Trail to north............	90,000	-200
4. Miscellaneous and contingencies..............	30,000	-100
Subtotal	$ 230,000	$ -500

Table 31 (continued)

Project and Component Description	Estimated Cost	
	Capital	Annual Operation and Maintenance[a]
H. Hale Park East Improvements		
1. Culvert under Bridget Lane....................	$ 2,000	$ 0
2. 150 feet of swale in 113th Street extended from Bridget Lane north.....................	3,000	0
3. Miscellaneous and contingencies...............	1,000	--
Subtotal	$ 6,000	$ 0
Eastern Portion of Village		
I. Grange Avenue and S. 108th Street Area Improvements		
1. 500 feet of 24-inch storm sewer from Grange Avenue north 500 feet........................	$ 30,000	$ 0
2. 700 feet of 30-inch and 36-inch storm sewers in and adjacent to Grange Avenue from S. 107th Street to 108th Street...............	60,000	-100
3. Miscellaneous and contingencies...............	10,000	--
Subtotal	$ 100,000	$ -100
J. Forest Home Area Improvements		
1. 720 feet of 36-inch storm sewer relay in Forest Home Avenue south of Grange Avenue.....	$ 60,000	$ 0
2. 1,070 feet of 42-inch storm sewer relay in Forest Home Avenue south of Grange Avenue.....	110,000	0
3. 1,190 feet of 54-inch storm sewer relay in Forest Home Avenue north of Janesville Road...	150,000	0
4. Miscellaneous and contingencies...............	50,000	--
Subtotal	$ 370,000	$ 0
K. Kay Parkway Area Improvements		
1. 400 feet of 27-inch storm sewer from College Avenue north on Park Terrace Drive............	$ 20,000	$ 100
2. 500 feet of 30-inch storm sewer in Park Terrace Drive................................	40,000	100
3. 730 feet of 36-inch storm sewer in Park Terrace Drive to 106th Street.................	60,000	100
4. 190 feet of 36-inch storm sewer from Park Terrace Drive north in 106th Street...........	20,000	0
5. 180 feet of 36-inch storm sewer in 106th Street from Parklane Court south..............	10,000	0
6. 450 feet of 36-inch storm sewer in 106th Street from Parklane Court to Kay Parkway.....	40,000	-100
7. 100 feet of 36-inch storm sewer in Kay Parkway from 106th Street to the west 100 feet....................................	10,000	0

Table 31 (continued)

Project and Component Description	Estimated Cost: Capital	Estimated Cost: Annual Operation and Maintenance[a]
K. Kay Parkway Area Improvements (continued)		
8. 200 feet of 36-inch storm sewer from Kay Parkway to the north 100 feet west of 106th Street	20,000	0
9. Road reconstruction of 250 feet of Kay Parkway west of 106th Street	13,000	0
10. Miscellaneous and contingencies	37,000	--
Subtotal	$ 270,000	$ 200
L. Meadow Park Area Improvements		
1. 1,300 feet of 27-inch storm sewer at Meadow Park Lane south of S. Bonnie Lane	$ 80,000	$ -200
2. Miscellaneous and contingencies	10,000	--
Subtotal	$ 90,000	$ -200
M. Garden Court Area Improvements		
1. 300 feet of 27-inch storm sewer from Meadow Park Drive to Garden Parkway	$ 20,000	$ -100
2. 1,160 feet of 36-inch storm sewer at Garden Parkway	100,000	0
3. Miscellaneous and contingencies	20,000	--
Subtotal	$ 140,000	$ -100
N. Brookside Drive Area Improvements		
1. 156 feet of 27-inch storm sewer in Brookside Drive near Allenwood Lane	$ 7,000	$ 0
2. 205 feet of 36-inch storm sewer in Brookside Drive west of Edgerton Avenue	10,000	0
3. Miscellaneous and contingencies	3,000	--
Subtotal	$ 20,000	$ 0
Incremental Nonpoint Source Abatement Measures[b]	$ 0	$9,300
Total	$3,496,000	$7,600

[a]Costs were noted to be zero when the project proposed replacement of a component with a component which has similar operation and maintenance costs. Negative costs were noted when the replacement component was estimated to have a lesser operation and maintenance cost, i.e., a storm sewer replacing an open channel.

[b]Includes costs for increased spring street and parking lot sweeping for county and state trunk highways, improved leaf collection, and a public education program. Costs for implementation of construction site erosion and pet waste controls are not included.

Source: SEWRPC.

Under the conveyance alternative plan, abatement of pollutants from nonpoint sources would be achieved primarily by certain public works activities, and by the use of roadside swales. Along 10 miles of highways which have curb and gutter--STH 24, STH 100, and CTH 00--street sweeping would be increased during spring and fall from the existing frequency of about once per month to once per week. Sweeping of commercial and industrial parking lots would also be increased. Leaf and vegetative debris collection during fall would be increased throughout the Village. Ordinances would be implemented to regulate construction site erosion and the placement and disposal of pet waste. Public education programs would be developed to encourage good urban "housekeeping" practices and to promote the acceptance and understanding of the proposed abatement measures and the importance of water quality protection.

Abatement of urban nonpoint source pollution would also result from the use of roadside swales which, under all alternative plans, would serve over 80 percent of the total area of the Village, and over 85 percent of the total study area. The filtering effects of grasses lining the swales, along with infiltration, would reduce particulate pollutant loadings from tributary land areas.

Centralized Detention Alternative Plan

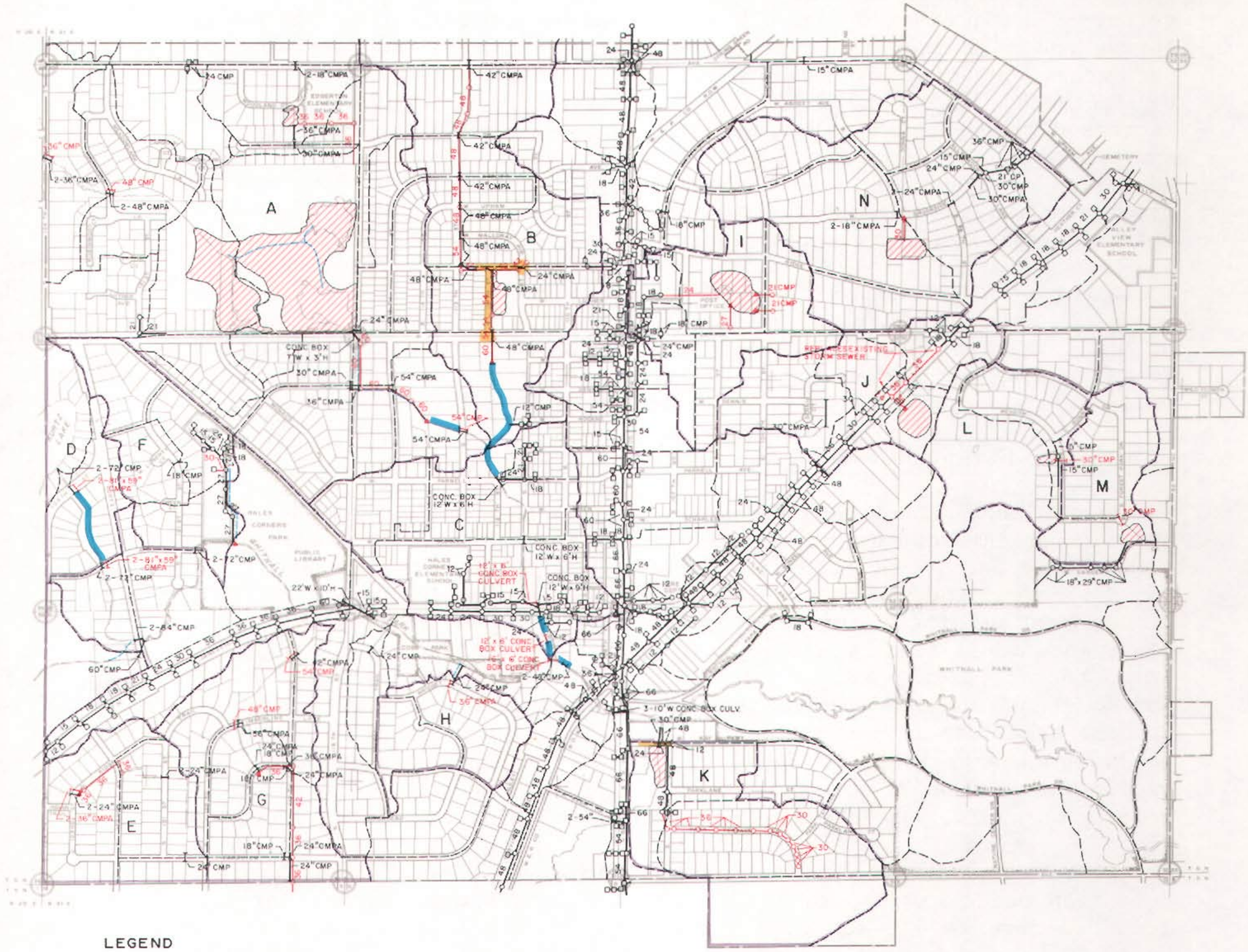
The centralized detention alternative plan would provide eight detention ponds and one parking lot detention facility strategically located within the study area. These detention facilities would reduce downstream discharges, allowing, in some cases, the use of smaller conveyance facilities downstream. The detention ponds and the parking lot detention facility, along with supplementary conveyance facilities, would serve to abate existing stormwater drainage problems, to effectively accommodate increased runoff from new urban development, and to reduce nonpoint source pollutant loadings within the Village of Hales Corners area. Map 17 shows the locations of the proposed centralized detention ponds and the parking lot facility, and of the major supplementary conveyance facilities. Table 32 presents the salient characteristics of the new storm sewers, channels, and detention ponds and facilities comprising this plan.

The centralized detention alternative consists of a total of eight centralized detention ponds having, under dry weather conditions, surface areas ranging from 0.2 acre to 7.0 acres, and a minimum permanent pool depth of four feet. Under 10-year recurrence interval runoff conditions, the eight detention ponds would have surface areas ranging from 0.5 acre to 18.4 acres, with storage volumes ranging from 1.5 acre-feet to 18.3 acre-feet and totaling 34.2 acre-feet. The parking lot detention facility 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.

The supplementary conveyance facilities include 7,500 lineal feet of new storm sewer ranging in diameter from 18 to 60 inches. Ten new culvert installations are also proposed, ranging in size from a 21-inch corrugated metal pipe to a 12-foot-wide by 6-foot-deep concrete box culvert. All new storm sewers are assumed to be constructed of reinforced concrete pipe. New sewer segments would discharge into surface streams, open channels, or detention ponds from six new outfalls, while three new sewer segments would discharge into existing storm sewers. About 3,800 feet of new engineered open channels would be provided under this alternative, as shown on Map 17. All of the new engineered channels would be turf lined.

Map 17

CENTRALIZED DETENTION ALTERNATIVE PLAN FOR STORMWATER MANAGEMENT IN THE VILLAGE OF HALES CORNERS



LEGEND

- 24 EXISTING STORM SEWER OR CULVERT AND SIZE IN INCHES
- EXISTING MANHOLE
- EXISTING CATCHBASIN OR INLET
- 24 PROPOSED STORM SEWER OR CULVERT AND SIZE IN INCHES
- PROPOSED MANHOLE
- PROPOSED STORM SEWER OUTFALL
- PROPOSED OPEN CHANNEL OR SWALE
- PROPOSED OPEN CHANNEL OR SWALE IMPROVEMENT
- PROPOSED DETENTION FACILITY
- PROPOSED ROAD RECONSTRUCTION
- SUBBASIN BOUNDARY
- HYDROLOGIC UNIT BOUNDARY
- A HYDROLOGIC UNIT IDENTIFICATION LETTER
- VILLAGE OF HALES CORNERS CORPORATE LIMITS

GRAPHIC SCALE

0 400 800 1200 FEET

Source: W. G. Nienow Engineering Associates; Graef, Anhalt, Schloemer & Associates, Inc.;and SEWRPC.

Table 32

SELECTED CHARACTERISTICS AND COSTS OF THE CENTRALIZED DETENTION ALTERNATIVE HALES CORNERS STORMWATER MANAGEMENT PLAN

Project and Component Description	Estimated Cost: Capital	Estimated Cost: Annual Operation and Maintenance[a]
Western Portion of Village		
A. Northwest Branch-Whitnall Park Creek Improvements		
1. Detention facility at 116th Street and Grange Avenue including 1,350-foot-interval open channel, 18.3 acre-feet	$ 70,000	$ 9,900
15 acres of land to be used for stormwater detention	60,000	0
2. 1,200 feet of 60-inch storm sewer from detention facility to 114th Street extended	180,000	-200
3. 48-inch culvert under Robinwood Drive	4,000	0
4. 36-inch culvert under 124th Street	3,000	0
5. Detention facility north of Woodside Drive at 118th Street extended, 1.6 acre-feet	30,000	1,400
6. 1,000 feet of 36-inch storm sewer from detention facility north of Woodside Drive to 180 feet south of Woodside Drive in 116th Street	80,000	200
7. 540 feet of channel improvement from 114th Street extended to 113th Street	20,000	0
8. 54-inch culvert in 113th Street north of Parnell Avenue	4,000	0
9. Miscellaneous roadside swale and culvert improvements	60,000	0
10. Miscellaneous and contingencies	69,000	1,700
Subtotal	$ 580,000	$13,000
B. North Branch-Whitnall Park Creek Improvements		
1. 1,430 feet of 48-inch storm sewer in 113th Street from Edgerton Avenue to Upham Avenue	$ 170,000	$ -300
2. 940 feet of 54-inch storm sewer in 113th Street from Upham Avenue to Copeland Avenue and in Copeland Avenue from 113th Street to 112th Street	130,000	-200
3. Road regrading in W. Copeland Avenue from 11th Street to west of 112th Street, in S. 112th Street from Copeland Avenue to Grange Avenue and in Grange Avenue to east and west of 112th Street	80,000	0
4. Detention facility at Hales Corners Lutheran School, 5.3 acre-feet	80,000	3,600
5. 150 feet of 54-inch storm sewer from detention facility to Grange Avenue	20,000	0
6. 210 feet of 60-inch storm sewer from Grange Avenue to 180 feet south of Grange Avenue	30,000	0
7. 1,100 feet of channel reconstruction south of Grange Avenue	40,000	0
8. 330 feet of 42-inch storm sewer in Copeland Avenue from 111th Street to 112th Street	30,000	-100
9. Miscellaneous and contingencies	90,000	500
Subtotal	$ 670,000	$ 3,500

Table 32 (continued)

Project and Component Description	Estimated Cost: Capital	Estimated Cost: Annual Operation and Maintenance[a]
C. North Branch-Whitnall Park Creek Junction Area Improvements		
1. Culvert under driveway at Janesville Road and 111th Street extended....................	$ 25,000	$ 0
2. 400 feet of channel improvement in 111th Street from Janesville Road to south.................	20,000	0
3. Culvert under driveway at 111th Street south of Janesville Road.....................	25,000	0
4. Culvert under driveway at 111th Street one block south of Janesville Road................	30,000	0
5. 150 feet of channel improvement..............	10,000	0
6. Miscellaneous and contingencies..............	20,000	0
Subtotal	$ 130,000	$ 0
D. Upper Kelly Lake Discharge Channel Improvements		
1. Culvert under 124th Street...................	$ 10,000	$ 0
2. 760 feet channel improvement from 124th Street to Godsell Road.......................	40,000	0
3. Culvert under Godsell Road....................	10,000	0
4. Miscellaneous and contingencies..............	10,000	0
Subtotal	$ 70,000	$ 0
E. Hale Park West Improvements		
1. 650 feet of 36-inch storm sewer in Ridge Trail from 122nd Street to 123rd Street.............	$ 60,000	$ -100
2. Culvert under 123rd Street at Ridge Trail.....	10,000	0
3. 250 feet of channel improvement at Kurtz Road.	10,000	0
4. Miscellaneous and contingencies..............	10,000	0
Subtotal	$ 90,000	$ -100
F. Village Hall Area Improvements		
1. 150 feet of 30-inch storm sewer connections...	$ 10,000	$ 0
2. 750 feet of grass swale......................	15,000	300
3. Miscellaneous and contingencies..............	5,000	0
Subtotal	$ 30,000	$ 300
G. Hale Park Central (118th Street) Improvements		
1. 250 feet of 36-inch storm sewer in 118th Street extended from village boundary to Parkview Lane..............................	$ 20,000	$ 0
2. 890 feet of 42-inch storm sewer in 118th Street from Parkview Lane to Indian Trail.....	90,000	-200
3. 360 feet of 36-inch storm sewer in Indian Trail from 118th Street to west...............	30,000	-100
4. Add culvert under Timberline Lane............	4,000	0
5. Add culvert under 118th Street...............	6,000	0
6. Miscellaneous and contingencies..............	20,000	0
Subtotal	$ 170,000	$ -300

Table 32 (continued)

Project and Component Description	Estimated Cost	
	Capital	Annual Operation and Maintenance[a]
H. Hale Park East Improvements		
1. Culvert under Bridget Lane	$ 2,000	$ 0
2. 150 feet of swale in 113th Street extended from Bridget Lane north	3,000	0
3. Miscellaneous and contingencies	1,000	0
Subtotal	$ 6,000	$ 0
Eastern Portion of Village		
I. Grange Avenue and 108th Street Area Improvements		
1. Detention facility north of Grange Avenue and west of 104th Street, 5.0 acre-feet	$ 80,000	$ 3,500
2.3 acres of land to be used for stormwater detention	20,000	0
2. Culvert under 104th Street	1,000	0
3. Culvert under 104th Street	1,000	0
4. 250 feet of 27-inch diameter storm sewer from Grange Avenue to detention facility	15,000	100
5. 550 feet of 24-inch diameter storm sewer outlet from the detention facility to the west	30,000	100
6. Miscellaneous and contingencies	23,000	600
Subtotal	$ 170,000	$ 4,300
J. Forest Home Avenue Area Improvements		
1. Detention facility south of Forest Home Avenue, 2.0 acre-feet	$ 40,000	$ 1,700
2. 780 feet of 36-inch storm sewer relay in Forest Home Avenue from south of Grange Avenue to detention facility	65,000	0
3. 60 feet of 24-inch storm sewer north of Forest Home Avenue	3,000	0
4. 100 feet of 18-inch storm sewer south of Forest Home Avenue to detention pond	4,000	0
5. Miscellaneous and contingencies	13,000	200
Subtotal	$ 125,000	$ 1,900
K. Kay Parkway Area Improvements		
1. 400 feet of 27-inch storm sewer from College Avenue north on Park Terrace Drive	$ 20,000	$ 100
2. 500 feet of 30-inch storm sewer in Park Terrace Drive	40,000	100
3. 730 feet of 36-inch storm sewer in Park Terrace Drive to 106th Street	60,000	100

Table 32 (continued)

Project and Component Description	Estimated Cost	
	Capital	Annual Operation and Maintenance[a]
K. Kay Parkway Area Improvements (continued)		
4. 190 feet of 36-inch storm sewer from Park Terrace Drive north in 106th Street..........	20,000	0
5. Road reconstruction of 250 feet of Kay Parkway west of 106th Street.................	13,000	0
6. Detention facility south of Kay Parkway and west of 106th Street, 2.1 acre-feet..........	40,000	1,700
1.6 acres of land to be used for stormwater detention..................................	20,000	0
7. Miscellaneous and contingencies..............	27,000	300
Subtotal	$ 240,000	$ 2,300
L. Meadow Park Area Improvements		
1. Detention facility west of Meadow Park Drive and south of S. Bonnie Lane, 1.65 acre-feet...	$ 30,000	$ 1,400
2. 30-inch culvert under Meadow Park Drive to detention facility..........................	2,000	0
3. Miscellaneous and contingencies..............	8,000	200
Subtotal	$ 40,000	$ 1,600
M. Garden Court Area Improvements		
1. Detention facility north of Garden Court and south of Forest Park Drive, 2.0 acre-feet.....	$ 40,000	$ 1,700
2. 30-inch culvert under Meadow Park Drive and Forest Park Drive intersection...............	2,000	0
3. Miscellaneous and contingencies..............	8,000	300
Subtotal	$ 50,000	$ 2,000
N. Brookside Drive Area Improvements		
1. Detention facility south of Brookside Drive at 100th Street extended, 1.5 acre-feet.......	$ 30,000	$ 1,300
1.5 acres of land to be used for stormwater detention..................................	15,000	0
2. 200 feet of 30-inch storm sewer north from detention facility to Brookside Drive.........	14,000	0
3. Miscellaneous and contingencies..............	6,000	200
Subtotal	$ 65,000	$ 1,500
Incremental Nonpoint Source Abatement Measures[b]	$ 0	$ 2,000
Total	$2,436,000	$32,200

[a]Costs were noted to be zero when the project proposed replacement of a component with a component which has similar operation and maintenance costs. Negative costs were noted when the replacement component was estimated to have a lesser operation and maintenance cost, i.e., a storm sewer replacing an open channel.

[b]Includes costs for a public education program. Costs for implementation of construction site erosion and pet waste controls are not included.

Source: SEWRPC.

Properly designed and maintained, the detention ponds can be effective in removing nonpoint source pollutant loadings, primarily through the sedimentation of particulate pollutants and the biological uptake of nutrients. The eight detention ponds would have a total tributary drainage area of about 505 acres in the Village, or about 24 percent of the total area of the Village, and about 705 acres in the planning area, or about 19 percent of the total planning area. The engineered open channels would provide additional infiltration and removal of pollutant loadings, especially since the channels would be turf lined rather than concrete lined as under the conveyance alternative plan. Through the control of construction site erosion and pet waste, through the use of roadside swales, and by implementation of a public education program, this alternative plan would achieve about the same level of abatement of nonpoint source pollutants achieved by the conveyance alternative plan.

The parking lot detention facility would be a dry detention basin in that a permanent pool of water would not be provided. A relatively small amount of particulate pollutants could be deposited during storm events on the parking lot surface and removed by subsequent sweeping of the parking lot. However, the overall pollutant removal effectiveness of this facility would be expected to be insignificant.

Infiltration basins and trenches and onsite infiltration facilities can, in some areas, effectively reduce stormwater runoff flows and associated pollutant loadings. Because the village site is generally covered by clay soils with a low permeability, because depths to groundwater are generally shallow, and because citizens have complained about ponded surface water and poor drainage, infiltration facilities were not included in this alternative plan.

Decentralized Detention Alternative Plan

The decentralized detention alternative plan incorporates certain natural features into the stormwater drainage system for storage, conveyance, and treatment, supplemented, as necessary, by man-made facilities and onsite rooftop and parking lot detention. The plan advocates joint use of areas for stormwater management and park and open space or other compatible uses. The preservation and use of natural swales, creeks, and watercourses for conveyance of stormwater is proposed. Natural ponds, lakes, wetlands, and floodplains are proposed to be preserved for storage purposes and integrated with the conveyance facilities.

Stormwater runoff from both existing paved parking lots, where practical, and new paved parking lots having an area greater than 20,000 square feet would be detained by parking lot storage and outflow facilities to a maximum depth of six inches. Also, stormwater runoff from both existing flat roof buildings, where practical, and new flat roof buildings in excess of 20,000 square feet in area would be detained by rooftop drain facilities to a maximum depth of five inches. In addition, some onsite detention would be provided by the preservation of floodlands, wetlands, and other open natural areas which serve to store stormwater runoff.

This alternative was developed assuming the use of rooftop storage in order to test the value of this means of reducing the peak flow rates in areas developed with buildings having large rooftop areas. Structural analyses to determine whether the attendant stormwater loads could, indeed, be accommodated were not conducted for the existing buildings. Such analyses would have to be done as part of plan implementation should these components be recommended.

The natural elements of the decentralized detention plan would be supplemented as necessary by constructed storage and conveyance facilities. The combination of natural and man-made features of this plan would serve to abate existing stormwater runoff problems and accommodate increased runoff from new urban development within the study area.

Map 18 shows the location of the proposed natural and constructed facilities comprising the decentralized storage alternative plan. Table 33 presents the salient characteristics of the facilities comprising this alternative plan.

Six potential parking lot storage sites would function as onsite detention facilities, ranging in size from 0.2 acre to 0.5 acre with a total area of 1.7 acres, and would have volumes of less than 0.1 acre-foot each at a maximum depth of six inches during a 10-year recurrence interval runoff event. The maximum amount of storage provided by all the onsite parking lot storage sites combined would be 0.3 acre-foot. One additional parking lot storage facility would be provided as a centralized detention facility, accommodating runoff from areas outside, and tributary to, the parking lot.

Under a 10-year recurrence interval runoff event, this centralized parking lot and playground detention facility would have a surface area of 1.2 acre and a storage volume of about 5.3 acre-feet at a maximum depth of 5.7 feet. Twelve rooftop storage sites would be provided ranging in size from 0.5 acre to 2.0 acres, with storage volumes ranging from 0.2 acre-foot to 0.8 acre-foot, at a maximum depth of five inches during a 10-year recurrence interval runoff event. The maximum amount of storage provided by all the potential rooftop storage sites together would be 3.4 acre-feet.

In all but four cases, the parking lot and rooftop detention volumes were determined to be too minimal or the potential storage sites were not located within tributary areas where the storage could solve conveyance deficiences. Thus, only four such detention sites were analyzed in this alternative.

The supplementary conveyance facilities under this alternative plan would include 8,350 lineal feet of new storm sewer ranging in diameter from 24 to 60 inches. The plan would also include 16 new culvert installations ranging in size from a 21-inch corrugated metal pipe to a 16-foot-wide by 6-foot-deep concrete box culvert. All new storm sewers are assumed to be constructed of reinforced concrete. Some of the new sewer segments would discharge into surface streams or open channels from four new outfalls, while others would discharge into existing storm sewers.

Five centralized detention ponds would also be provided. During dry weather conditions, the five detention ponds would range in surface area from 0.2 acre to 2.0 acres, with a minimum pool depth of three feet. During a 10-year recurrence interval runoff event, the ponds would range in surface area from 0.5 acre to 18.4 acres and have total storage volumes ranging from 1.5 acre-feet to 18.3 acre-feet. The combined maximum storage capacity of the ponds would be 37.6 acre-feet. About 8,700 feet of new engineered open channels would also be provided under this alternative, as shown on Map 18. All the new engineered channels would be turf lined.

Properly designed and maintained, the five detention ponds, along with preserved wetland areas, would provide substantial removal of nonpoint source pollutant loadings. The five detention ponds would have a total tributary drainage area of about 433 acres in the Village, or about 21 percent of the village area, and about 633 acres in the planning area, or 17 percent of the total planning area. The 8,700 lineal feet of turf-lined, engineered open channels would provide additional infiltration and removal of pollutant loadings. Through the control of construction site erosion and pet waste, through the use of roadside swales, and by implementation of a public education program, this alternative plan would achieve the same level of abatement of nonpoint source pollutant loadings achieved by the conveyance alternative plan. The parking lot and rooftop detention facilities would not be expected to remove a significant amount of pollutant loadings.

EVALUATION OF ALTERNATIVE STORMWATER MANAGEMENT PLANS

The preceding section described the three alternative stormwater management system plans considered for the Village of Hales Corners study area. The information presented was intended to provide a basis for a comparative evaluation of the three alternative plans. Each alternative was designed to resolve the identified existing drainage problems as well as to serve planned development within the Village of Hales Corners study area, and to accommodate stormwater flows entering the village proper from surrounding areas under planned land use conditions. Thus, the principal criteria for the comparative evaluation were reduced to cost and nonpoint source pollutant removal effectiveness.

Each alternative has certain advantages and disadvantages. These are summarized in Table 34.

For each hydrologic unit within the planning area, Table 35 compares the capital costs, the annual operation and maintenance costs, and the present value of the costs of each alternative. A comparison of the ability of each alternative plan to meet the recommended stormwater management objectives and supporting standards is provided in Table 36 for those objectives and standards which differ in level of achievement between the plans.

A review of the alternative plan maps and cost information presented indicates that three hydrologic units--D, E, and H--have essentially the same components and costs under each alternative plan. Accordingly, it was not considered necessary to further consider these hydrologic units in the following discussion. The remaining 11 hydrologic units are considered in the discussion of each alternative plan.

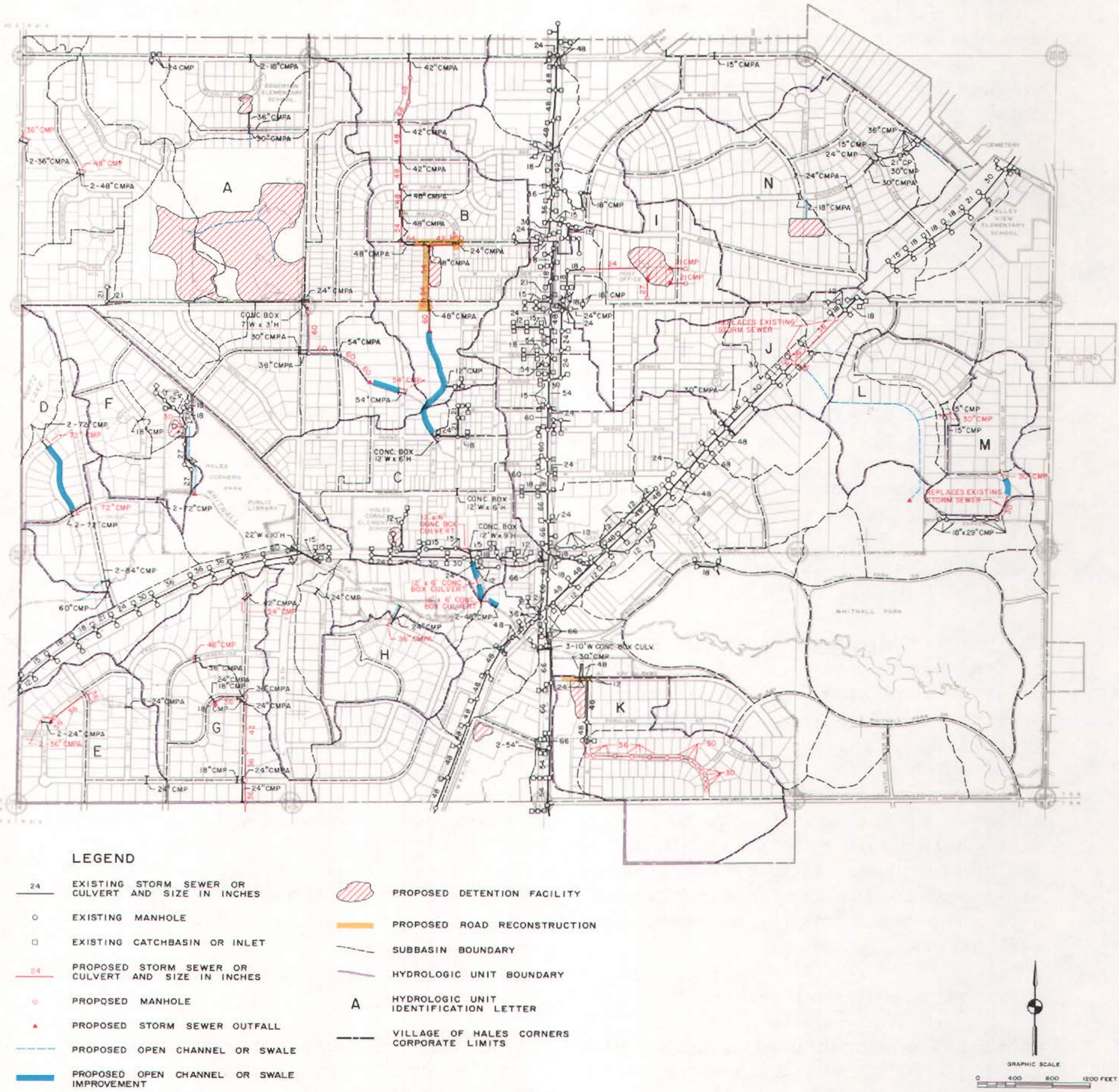
Conveyance Alternative Plan

Under the conveyance alternative plan, the Village of Hales Corners would continue to rely on storm sewers, roadside swales, and open channels to convey stormwater runoff as quickly and directly as practicable to receiving surface watercourses. The alternative would entail a capital cost of about $3.50 million and an average annual operation and maintenance cost increase of about $7,600, and would have a present value cost of $3.62 million.

Nonpoint source abatement would be achieved through increased sweeping of state and county trunk highways and adjoining parking lots during spring, improved leaf collection, control of construction site erosion and pet waste,

Map 18

DECENTRALIZED DETENTION ALTERNATIVE PLAN FOR STORMWATER MANAGEMENT IN THE VILLAGE OF HALES CORNERS



Source: W. G. Nienow Engineering Associates; Graef, Anhalt, Schloemer & Associates, Inc.; and SEWRPC.

Table 33

SELECTED CHARACTERISTICS AND COSTS OF THE DECENTRALIZED DETENTION ALTERNATIVE HALES CORNERS STORMWATER MANAGEMENT PLAN

Project and Component Description	Estimated Cost: Capital	Estimated Cost: Annual Operation and Maintenance[a]
Western Portion of Village		
A. Northwest Branch-Whitnall Park Creek Improvements		
1. Culvert under 124th Street	$ 3,000	$ 0
2. Culvert under Robinwood Drive	4,000	0
3. 18.3-acre-foot detention facility at 116th Street and Grange Avenue, including 1,350 feet internal open channel	70,000	12,100
4. 3.2-acre-foot detention facility at 118th Street extended north at Woodside Drive	50,000	2,400
15 acres of land to be used for stormwater detention	60,000	0
5. 1,200 feet of 60-inch storm sewer from detention facility to 114th Street extended	180,000	-200
6. 540-foot channel improvement from 114th Street extended to 113th Street	20,000	0
7. Culvert under 113th Street	4,000	0
8. Miscellaneous roadside swale and culvert improvements	60,000	0
9. Miscellaneous and contingencies	59,000	2,100
Subtotal	$ 510,000	$16,400
B. North Branch-Whitnall Park Creek Improvements		
1. 1,430 lineal feet of 48-inch storm sewer in 113th Street from Edgerton Avenue to Upham Avenue	$ 170,000	$ -300
2. 940 lineal feet of 54-inch storm sewer in 113th Street from Upham Avenue to Copeland Avenue and in Copeland Avenue from 113th Street to 112th Street	130,000	-200
3. 330 lineal feet of 42-inch storm sewer in Copeland Avenue from 111th Street to 112th Street	30,000	-100
4. 5.3-acre-foot detention facility at Hales Corners Lutheran School playground and parking lot	80,000	3,600
5. 200 lineal feet of 54-inch storm sewer in 112th Street south of Copeland Avenue to Grange Avenue	30,000	0
6. 200 lineal feet of 60-inch storm sewer in 112th Street extended from Grange Avenue to south	30,000	0
7. Regrade 1,100 feet of open channel in 112th Street extended from south of Grange Avenue to Parnell Avenue	40,000	0
8. Roadway reconstruction	80,000	0
9. Miscellaneous and contingencies	90,000	500
Subtotal	$ 680,000	$ 3,500

Table 33 (continued)

Project and Component Description	Estimated Cost	
	Capital	Annual Operation and Maintenance[a]
C. North Branch-Whitnall Park Creek Junction Area Improvements		
1. Culvert under driveway at Janesville Road and 111th Street extended....................	$ 25,000	$ 0
2. 400 feet of channel improvement in 111th Street from Janesville Road to south..........	20,000	0
3. Culvert under driveway at 111th Street south of Janesville Road.....................	25,000	0
4. Culvert under driveway at 111th Street one block south of Janesville Road................	30,000	0
5. 150 feet of channel improvement..............	10,000	0
6. Miscellaneous and contingencies..............	20,000	0
Subtotal	$ 130,000	$ 0
D. Upper Kelly Lake Discharge Channel Improvements		
1. Culvert under 124th Street...................	$ 10,000	$ 0
2. 760-foot channel improvement from 124th Street to Godsell Road	40,000	0
3. Culvert under Godsell Road...................	10,000	0
4. Miscellaneous and contingencies..............	10,000	0
Subtotal	$ 70,000	$ 0
E. Hale Park West Improvements		
1. 650 lineal feet of 36-inch storm sewer in Ridge Trail from 122nd Street to 123rd Street................................	$ 60,000	$ -100
2. Culvert under 123rd Street at Ridge Trail	10,000	0
3. 250 feet of channel improvement at Kurtz Road................................	10,000	0
4. Miscellaneous and contingencies..............	10,000	0
Subtotal	$ 90,000	$ -100
F. Village Hall Area Improvements		
1. 150 feet of 30-inch storm sewer connections...	$ 10,000	$ 0
2. 750 feet of grass swale......................	15,000	300
3. Miscellaneous and contingencies..............	5,000	0
Subtotal	$ 30,000	$ 300
G. Hale Park Central (118th Street) Improvements		
1. 250 feet of 36-inch storm sewer in 118th Street extended from village boundary to Parkview Lane..............................	$ 20,000	$ 0
2. 890 feet of 42-inch storm sewer in 118th Street from Parkview Lane to Indian Trail.....	90,000	-200
3. 360 feet of 36-inch storm sewer in Indian Trail from 118th Street to west..............	30,000	-100
4. Add culvert under Timberline Lane............	4,000	0

Table 33 (continued)

Project and Component Description	Estimated Cost	
	Capital	Annual Operation and Maintenance[a]
G. Hale Park Central (118th Street) Improvements (continued)		
5. Add culvert under 118th Street	6,000	0
6. Miscellaneous and contingencies	20,000	0
Subtotal	$ 170,000	-300
H. Hale Park East Improvements		
1. Culvert under Bridget Lane	$ 2,000	$ 0
2. 150 feet of swale in 113th Street extended from Bridget Lane north	3,000	0
3. Miscellaneous and contingencies	1,000	0
Subtotal	$ 6,000	$ 0
Eastern Portion of Village		
I. Grange Avenue and S. 108th Street Area Improvements		
1. Detention facility north of Grange Avenue and west of 104th Street, 5.0 acre-feet	$ 80,000	$ 3,500
2.3 acres of land to be used for stormwater detention	20,000	0
2. 21-inch culvert under 104th Street	1,000	0
3. 21-inch culvert under 104th Street	1,000	0
4. 250 feet of 27-inch storm sewer from Grange Avenue to detention facility	15,000	0
5. 550 feet of 24-inch storm sewer from detention facility to the west	30,000	100
6. Miscellaneous and contingencies	23,000	500
Subtotal	$ 170,000	$ 4,100
J. Forest Home Area Improvements		
1. 780 feet of 36-inch storm sewer relay in Forest Home Avenue south of Grange Avenue	$ 65,000	$ 0
2. 60 feet of 24-inch storm sewer in Forest Home Avenue	3,000	0
3. 1,550 feet of open channel from Forest Home Avenue south and west of Meadow Park Drive	45,000	600
4. Miscellaneous and contingencies	17,000	100
Subtotal	$ 130,000	$ 700
K. Kay Parkway Area Improvements		
1. 400 feet of 27-inch storm sewer from College Avenue north on Park Terrace Drive	$ 20,000	$ 100
2. 500 feet of 30-inch storm sewer in Park Terrace Drive	40,000	100
3. 730 feet of 36-inch storm sewer in Park Terrace Drive to 106th Street	60,000	100
4. 190 feet of 36-inch storm sewer from Park Terrace Drive north in 106th Street	20,000	0

Table 33 (continued)

Project and Component Description	Estimated Cost	
	Capital	Annual Operation and Maintenance[a]
K. Kay Parkway Area Improvements (continued)		
5. Detention facility south of Kay Parkway and west of 106th Street, 9.6 acre-feet.......	130,000	5,800
1.6 acres of land to be used for stormwater detention..................................	20,000	0
6. Miscellaneous and contingencies..............	40,000	900
Subtotal	$ 330,000	$ 7,000
L. Meadow Park Area Improvements		
1. 280 feet of channel improvements from Meadow Park Drive to the west.................	$ 8,000	$ 100
2. 30-inch culvert under Meadow Park Drive south of S. Bonnie Lane......................	2,000	0
3. 1,000 feet of open channel west of Meadow Park Drive..................................	30,000	400
4. Miscellaneous and contingencies..............	5,000	100
Subtotal	$ 45,000	$ 600
M. Garden Court Area Improvements		
1. Culvert under Forest Park Drive at Meadow Park Drive..................................	$ 2,000	$ 0
2. 200 feet of channel improvements from Meadow Park Drive south to Garden Court.............	5,000	0
3. 260 feet of 30-inch storm sewer along Garden Court to low area in parkland................	20,000	0
4. Miscellaneous and contingencies..............	3,000	0
Subtotal	$ 30,000	$ 0
N. Brookside Drive Area Improvements		
1. Detention facility south of Brookside Drive at 100th Street extended, 1.5 acre-feet.......	$ 30,000	$ 1,300
2. 200 feet of channel improvements north from detention facility to Brookside Drive.........	6,000	100
1.5 acres of land to be used for stormwater detention..................................	15,000	0
3. Miscellaneous and contingencies..............	4,000	200
Subtotal	$ 55,000	$ 1,600
Incremental Nonpoint Source Abatement Measures[b]	$ 0	$ 2,000
Total	$2,446,000	$35,800

[a]Costs were noted to be zero when the project proposed replacement of a component with a component which has similar operation and maintenance costs. Negative costs were noted when the replacement component was estimated to have a lesser operation and maintenance cost, i.e., a storm sewer replacing an open channel.

[b]Includes costs for a public education program. Costs for implementation of construction site erosion and pet waste controls are not included.

Source: SEWRPC.

Table 34

SUMMARY OF PRINCIPAL COMPONENTS AND ADVANTAGES AND DISADVANTAGES OF ALTERNATIVE STORMWATER MANAGEMENT PLANS FOR THE HALES CORNERS VILLAGE AREA

Alternative	Principal New Components	Advantages	Disadvantages
Conveyance	20,570 feet of storm sewer 3,460 feet of engineered open channels or channel improvements Increased spring sweeping of state and county trunk highways and adjoining parking lots Increased cleaning of catch basins Improved leaf collection Construction site erosion and pet waste control Public education program	Stormwater drainage components are acceptable and well known to the public; minimal operation and maintenance is required	Downstream peak discharges and flow volumes are increased; some public officials and citizens may oppose high capital cost; relatively low level of reduction in pollutant loadings from nonpoint sources is achieved; higher incremental cost for nonpoint source abatement
Centralized Detention	Eight centralized detention ponds One centralized parking lot detention facility 7,500 feet of storm sewer 3,800 feet of engineered open channels or channel improvements Construction site erosion and pet waste control Public education program	Minimizes future increases in peak discharges and areas of inundation; reduces the required size and resultant cost of some downstream conveyance systems; relatively high level of reduction in pollutant loadings from nonpoint sources	Maintenance requirements are substantial; land requirements are considerably greater than under the conveyance alternative; some public officials and citizens may oppose ponded water in urban areas
Decentralized Detention	8,350 feet of storm sewer 8,700 feet of engineered open channels or channel improvements Five centralized detention ponds One centralized parking lot detention facility Six onsite parking lot detention facilities 12 commercial and industrial rooftop detention facilities Construction site erosion and pet waste control Public education program	Minimizes future increases in peak discharges and areas of inundation; reduces the required size and resultant cost of some downstream conveyance systems; relatively high level of reduction in pollutant loadings from nonpoint sources	Maintenance requirements are substantial; land requirements are considerably greater than under the conveyance alternative; some components are necessarily located on private property, so implementation may be difficult; some local opposition to onsite detention facilities may occur; some public officials and citizens may oppose ponded water in urban areas

Source: SEWRPC.

Table 35

COSTS OF THE ALTERNATIVE STORMWATER MANAGEMENT PLANS FOR THE VILLAGE OF HALES CORNERS PLANNED URBAN SERVICE AREA

Hydrologic Unit Designation	Estimated Cost--Plan Year Land Use Conditions								
	Conveyance Alternative			Centralized Detention Alternative			Decentralized Detention Alternative		
	Capital	Annual Operation and Maintenance	Present Value[a]	Capital	Annual Operation and Maintenance	Present Value[a]	Capital	Annual Operation and Maintenance	Present Value[a]
A	$1,130,000	$ -200	$1,127,000	$ 580,000	$13,000	$ 785,000	$ 510,000	$16,400	$ 768,000
B	750,000	-700	740,000	670,000	3,500	725,000	680,000	3,500	735,000
C	140,000	0	140,000	130,000	0	130,000	130,000	0	130,000
D	70,000	0	70,000	70,000	0	70,000	70,000	0	70,000
E	90,000	-100	88,000	90,000	-100	88,000	90,000	-100	88,000
F	90,000	0	90,000	30,000	300	35,000	30,000	300	35,000
G	230,000	-500	222,000	170,000	-300	165,000	170,000	-300	165,000
H	6,000	0	6,000	6,000	0	6,000	6,000	0	6,000
I	100,000	-100	98,000	170,000	4,300	238,000	170,000	4,100	235,000
J	370,000	0	370,000	125,000	1,900	155,000	130,000	700	141,000
K	270,000	200	273,000	240,000	2,300	276,000	330,000	7,000	440,000
L	90,000	-200	87,000	40,000	1,600	65,000	45,000	600	54,000
M	140,000	-100	138,000	50,000	2,000	82,000	30,000	0	30,000
N	20,000	0	20,000	65,000	1,500	89,000	55,000	1,600	80,000
Incremental Nonpoint Source Abatement Costs	0	9,300[b]	147,000	0	2,000[c]	32,000	0	2,000[c]	32,000
Total	$3,496,000	$ 7,600	$3,616,000	$2,436,000	$32,000	$2,940,000	$2,446,000	$35,800	$3,011,000

[a]Present value computations assume 50-year life and 6 percent annual interest.

[b]Includes costs for increased spring street and parking lot sweeping for state trunk highways, improved leaf collection, and a public education program. Costs for implementation of construction site erosion and pet waste controls are not included.

[c]Includes costs for a public education program. Costs for implementation of construction site erosion and pet waste controls are not included.

Source: SEWRPC.

and a public education program. The construction site erosion and pet waste controls would be relatively effective in reducing sediment and fecal coliform loadings, respectively, from urban nonpoint sources. The remaining practices, however, would be able to achieve only a modest reduction--about 10 percent--in pollutant loadings.

Increased sweeping of state and county trunk highways and adjoining parking lots during spring could reduce pollutant loadings from these mostly commercial areas by 5 to 10 percent. Street sweeping is more cost-effective during the spring because spring street surface loadings of sediment, lead, and phosphorus are two to five times higher than the average annual loadings. Leaf collection in fall could be improved by requiring all leaves to be placed in plastic bags for collection rather than allowing selective burning. The selective burning of leaves in areas where the ash may be washed off adds nutrients to surface waters. Construction site erosion controls, such as sediment trapping, maintenance of vegetative cover, and runoff controls, can reduce sediment loadings from active construction sites by 75 to 90 percent. Effective pet waste controls can reduce nonpoint source fecal coliform loadings by up to 35 percent.

For the planning area as a whole, the conveyance alternative has a higher capital cost but is considerably lower in operation and maintenance costs than the other two alternatives. Significantly, the annual operation and maintenance cost is $24,400 less than under the centralized detention alternative and $28,200 less than under the decentralized detention alternative. However, there are certain subareas of the Village of Hales Corners study area where components of the conveyance alternative would be less costly than components of the centralized or decentralized detention alternatives needed to serve the same hydrologic units. Specifically, in Hydrologic Units I and N, the capital cost of the conveyance alternative plan would be lower than the cost of the centralized or decentralized detention alternatives. In addition, the conveyance alternative in Hydrologic Unit K has a capital cost of only $30,000, or 14 percent, more than the lowest cost centralized detention alternative. Operation and maintenance costs would be $2,100 higher for the centralized detention alternative for Hydrologic Unit K, however, and thus the conveyance alternative is lower in total cost than the centralized detention alternative.

When compared to the other two alternative system plans, the advantages of the conveyance alternative plan, in addition to low operation and maintenance costs, are that the proposed system would be readily implementable and likely to be more acceptable to local officials and citizens. Importantly, few health and safety hazards or aesthetic nuisances would be created.

The major disadvantage of the conveyance alternative plan is the high capital cost. Another significant disadvantage is that downstream peak discharges may be expected to be higher than existing discharges, and to be higher than discharges under the centralized and decentralized detention alternatives. Other disadvantages include a relatively low level of nonpoint source pollution removal, and the lack of any multipurpose-use benefits.

Most of the agreed-upon stormwater management objectives could be met by the conveyance alternative plan, although a lower level of nonpoint source pollution reduction would be provided than under the other plans considered. Stormwater storage was not, by design, incorporated into this alternative. Based on the cost analyses and other considerations, conveyance plan facility components should be considered further for Hydrologic Units I, K, and N in the preparation of a recommended plan.

Table 36

ABILITY OF THE STORMWATER MANAGEMENT ALTERNATIVE PLANS TO MEET THE RECOMMENDED STORMWATER MANAGEMENT OBJECTIVES AND SUPPORTING STANDARDS

Stormwater Management Objective[a]	Supporting Standards	Conveyance	Centralized Detention	Decentralized Detention
The development of a stormwater management system which will minimize soil erosion, sedimentation, and attendant water pollution	1. Flow velocities which increase stream bank erosion and channel sediment scouring should be avoided	Partially met; flow velocities may increase because of higher streamflows	Can be met	Can be met
	2. Nonpoint source pollution abatement measures should be incorporated, wherever appropriate, into the stormwater management plan	Partially met, since a relatively low level of reduction in pollutant loadings would be achieved	Can be met; the detention ponds and turf-lined open channels will reduce nonpoint source loadings by from less than 5 percent to 95 percent from those drainage areas tributary to the ponds and open channels	Can be met; the detention ponds and turf-lined open channels will reduce nonpoint source loadings by from less than 5 percent to 95 percent from those drainage areas tributary to the ponds and open channels
The development of a stormwater management system which will be flexible and readily adaptable to changing needs	1. Larger, less frequent storm events should be used to design and size those site-specific elements of the stormwater drainage system for which it would not be economically feasible to provide flow relief during and following a major storm event	Can be met	Can be met	Can be met
The development of a stormwater management system which will efficiently and effectively meet all of the other stated objectives at the lowest practicable cost	1. The sum of storm drainage system capital investment and the operation and maintenance costs should be minimized	Partially met; this alternative has the lowest capital cost for 5 of the 14 hydrologic units within the Village. The operation and maintenance cost is the lowest for the Village	Partially met; this alternative has the lowest capital cost for 5 of the 14 hydrologic units within the Village. The operation and maintenance costs are intermediate	Partially met; this alternative has the lowest capital cost for 4 of the 14 hydrologic units within the Village. The operation and maintenance cost is the highest
	2. Maximum feasible use should be made of all existing stormwater management components, as well as the natural storm drainage system. The latter should be supplemented with engineered facilities only as necessary to serve the anticipated stormwater management needs generated by existing and proposed land use development and redevelopment	Partially met; would not use existing floodlands and wetlands	Can be met	Can be met

Table 36 (continued)

Stormwater Management Objective[a]	Supporting Standards	Conveyance	Centralized Detention	Decentralized Detention
The development of a stormwater management system which will efficiently and effectively meet all of the other stated objectives at the lowest practicable cost (continued)	3. To the maximum extent practicable, the location and alignment of new storm sewers and engineered channels and storage facilities should coincide with existing public rights-of-way to minimize land acquisition or easement costs	Can be met	Can be met	Partially met; most of the onsite detention facilities would be located on private property
	4. Stormwater storage facilities--consisting of retention facilities and of both centralized and onsite detention facilities--should, where hydraulically feasible and economically sound, be considered as a means of reducing the size and resultant costs of the required stormwater conveyance facilities immediately downstream of these storage sites	Not met; by design, stormwater storage facilities were not included in the conveyance alternative	Met	Met

[a]The stormwater management objectives and supporting standards are set forth in Table 22 of Chapter V. This table compares only those objectives and supporting standards which differ in the degree to which they are met by the alternatives.

Source: SEWRPC.

Centralized Detention Alternative Plan

The centralized detention alternative plan would provide eight centralized surface detention ponds and one centralized parking lot storage facility to store temporarily a portion of the stormwater runoff generated from the urban service area for subsequent slow release to the drainage system. The alternative would entail a capital cost of about $2.40 million and an annual operation and maintenance cost increase of about $32,000, and would have a present value cost of $2.90 million.

The eight centralized detention ponds would provide a substantial removal of nonpoint source pollutant loadings, thereby protecting the water quality of Whitnall Park Creek and its tributaries. The estimated reductions in pollutant loadings achieved by the eight ponds would be about 90 percent for total solids, 80 percent for lead, and about 50 percent for total phosphorus. Similar levels of reduction would be achieved for other pollutants. The detention ponds would receive runoff from 505 acres, or 24 percent, of the village area, and from 705 acres, or 19 percent, of the total planning area.

The stormwater discharge and pollutant removal rates estimated for each of four new or improved turf-lined open channels are set forth in Table 37. The removal rates are provided for a mean storm event, a one-year recurrence interval storm event, and a 10-year recurrence interval storm event. The estimated rates range from 5 to 50 percent for a mean storm event, from 5 percent to 15 percent for a one-year recurrence interval storm event, and from 5 percent to 10 percent for a 10-year recurrence interval storm event. These reductions would be achieved for the 392 acres, or 19 percent, of the village area and 583 acres, or 15 percent, of the total planning area tributary to these open channels. Reduction in pollutant loadings would also be achieved by construction site erosion and pet waste controls.

For the planning area as a whole, the capital cost of the centralized detention alternative is about the same as that of the decentralized detention alternative and considerably less than that of the conveyance alternative. However, for certain hydrologic units--specifically, units B, J, K, and L, as shown on Map 18--the capital cost would be lower for the centralized detention alternative than for either the conveyance alternative or the decentralized detention alternative. Present value cost analyses indicated that for hydrologic units J and L, the total cost of the centralized detention alternative would be higher than for the decentralized detention alternative. Likewise, present worth cost analyses indicated that for Hydrologic Unit K, the total cost of the centralized detention alternative would be higher than the cost of the conveyance alternative. Also, for Hydrologic Units C, F, and G, as shown on Map 18, the capital cost of the centralized detention alternative would be the same as the capital cost of the decentralized detention alternative and lower than the capital cost of the conveyance alternative. The annual operation and maintenance costs of the conveyance alternative and decentralized detention alternative would be, respectively, $24,400 less than and $3,800 greater than the operation and maintenance costs of the centralized detention alternative. In addition to the water quality benefits and cost advantages in certain areas, the centralized detention alternative would reduce the peak rate of stormwater flow downstream of the proposed detention facilities.

Table 37

REDUCTIONS IN STORM RUNOFF DISCHARGES AND POLLUTANT LOADINGS PROVIDED BY TURF-LINED OPEN CHANNELS UNDER THE CENTRALIZED DETENTION AND DECENTRALIZED DETENTION ALTERNATIVE PLANS

Centralized Detention Alternative Plan	Decentralized Detention Alternative Plan	Hydrologic Unit	Reduction in Storm Runoff Discharge and Pollutant Loadings Tributary to the Channel (percent)		
			Mean Storm Event	1-Year Recurrence Interval Storm Event	10-Year Recurrence Interval Storm Event
X	X	A	5	5	5
X	X	E	10	5	5
X	X	F	50	15	10
X	X	H	25	10	5
--	X	J	60	25	15
--	X	L	85	25	20
--	X	N	10	5	5

Source: SEWRPC.

Disadvantages of the centralized detention alternative include the increased land area required for the proposed detention facilities, and, in some cases, higher costs in comparison to the conveyance alternative.

Most stormwater management objectives could be met by the centralized detention alternative plan. However, for four of the 11 hydrologic units being considered in this discussion, the centralized detention alternative plan would have a higher capital cost than either the conveyance or the decentralized detention alternatives.

Based on the cost analyses and other considerations, centralized detention plan facility components should be considered further for Hydrologic Units A and F in the preparation of a recommended plan. Combined centralized detention and decentralized detention facility components should be further considered for Hydrologic Units B, C, and G.

Decentralized Detention Alternative Plan

The decentralized detention alternative plan utilizes certain natural features for stormwater storage, conveyance, and treatment, supplemented by man-made facilities as needed. Compared to the centralized detention alternative, the decentralized detention alternative contains a greater number and variety of detention facilities, although the hydraulic capacity of each facility may be smaller.

Significant amounts of nonpoint source pollutants would be removed by the proposed five detention ponds and by the 8,700 feet of turf-lined open channels included in this alternative plan. For the detention ponds, the removal rates would be expected to be the same as for the centralized detention alternative: about 90 percent for total solids, about 80 percent for lead, and about

50 percent for total phosphorus. These pollutant loading reductions would be achieved for the 433 acres, or 21 percent, of the village area and 633 acres, or 17 percent, of the total planning area tributary to the five ponds.

The stormwater discharge and pollutant removal rates estimated for each of the turf-lined open channels are presented in Table 37. The estimated rates range from 5 to 85 percent for a mean storm event, from 5 percent to 25 percent for a one-year recurrence interval storm event, and from 5 percent to 20 percent for a 10-year recurrence interval storm event. These pollutant loading reductions would be achieved for the 611 acres, or 29 percent, of the village area and 802 acres, or 22 percent, of the total planning area tributary to these open channels. The loading reductions achieved by control of construction site erosion and pet wastes would be the same as under the conveyance alternative plan. On an overall basis, the pollutant loading reductions achieved under the decentralized detention alternative plan would be similar to the reductions achieved under the centralized detention alternative plan, and substantially greater than those achieved under the conveyance alternative plan.

The decentralized detention alternative would entail a capital cost of about \$2.45 million and an annual operation and maintenance cost increase of about \$35,800, and would have a present value cost of \$3.01 million. The cost of the decentralized detention alternative is slightly higher than the cost of the detention alternative and substantially less than the cost of the conveyance alternative. The annual operation and maintenance costs of the conveyance alternative and the centralized detention alternative are \$28,200 and \$3,800 less, respectively, than such costs for the decentralized detention alternative. Only in Hydrologic Units A and M does the decentralized detention alternative have the lowest capital cost. When present value costs are considered, Hydrologic Units J, L, and M have a lower total cost under the decentralized alternative than under the centralized detention alternative.

The most significant advantage of the decentralized detention alternative is that peak rates of discharge would be considerably less than under the conveyance alternative. Another advantage is that significant reductions would be achieved in downstream pollutant loadings.

The primary disadvantages of the decentralized detention alternative include high maintenance costs and the required location of the onsite detention facilities on what is now private property, which could make implementation and funding of this alternative difficult.

Most stormwater management objectives could be met by the decentralized detention alternative plan. However, for 6 of the 11 hydrologic units considered, the decentralized detention alternative plan would have a higher capital cost than either the conveyance or centralized detention alternatives.

Based on the cost analyses and other considerations, decentralized detention plan components should be considered further for Hydrologic Units J, L, and M. In addition, joint centralized detention and decentralized detention plan facility components should be considered for Hydrologic Units B, C, and G.

SUMMARY

The comparative evaluation of three alternative stormwater management system plans for the Village of Hales Corners study area indicated that the capital cost of such plans may be expected to range from $2.40 million to $3.50 million, while the annual operation and maintenance incremental costs may be expected to range from $7,600 to $35,800.

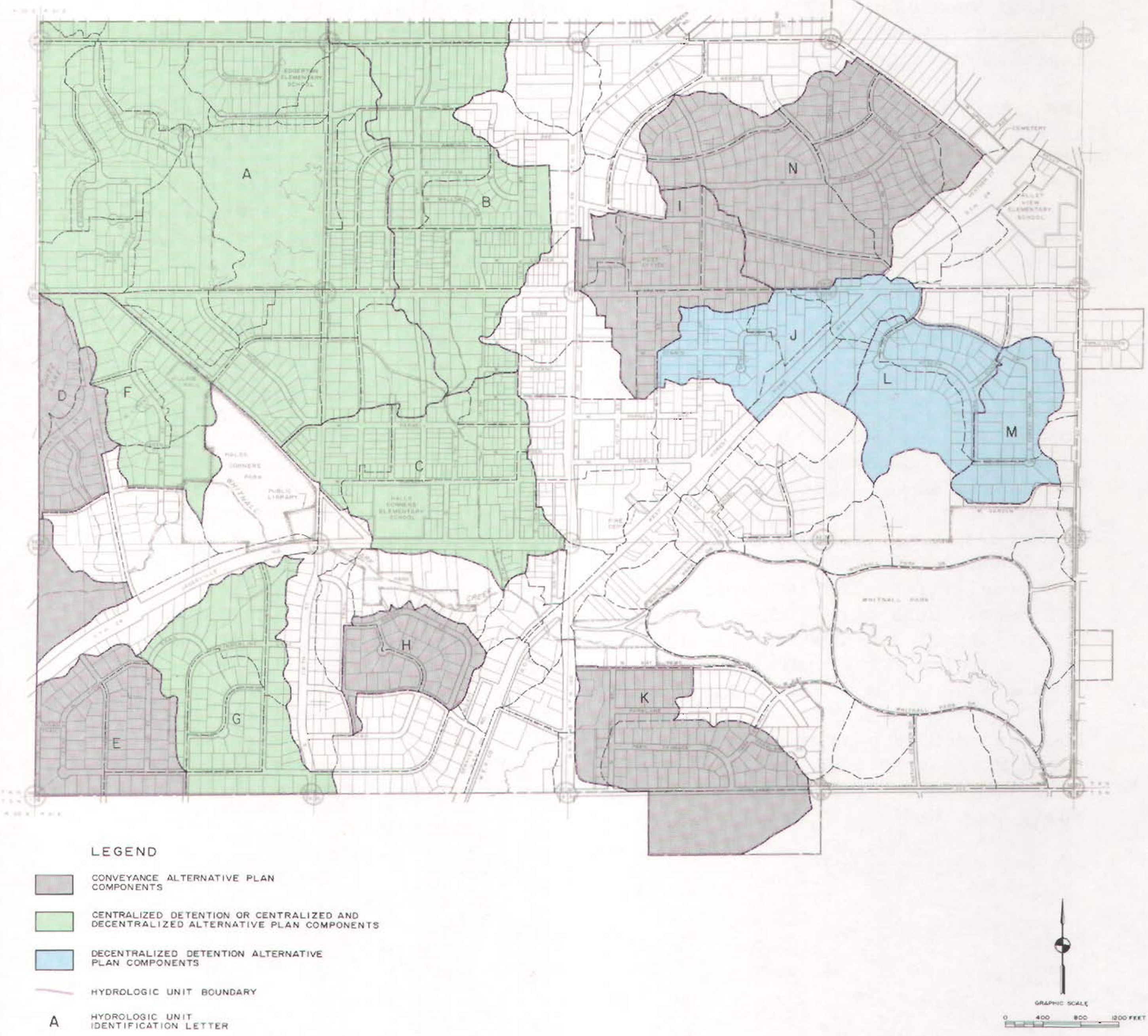
The comparative evaluation also indicated that a combination of the conveyance, centralized detention, and decentralized detention alternative plan components should be considered in the synthesis of a recommended plan--incorporating in that plan for each hydrologic unit the most cost-effective elements of each plan. Such a combined plan should provide beneficial water quantity and quality control at the least cost, be implementable, and fully satisfy the stormwater management objectives and standards formulated under the study.

The Hales Corners Village area has been divided for plan preparation purposes into 14 hydrologic units. Based upon the evaluation of the components of each of the three alternative plans considered, it was concluded that the alternative plan components shown on Map 19 should be further considered for application to each hydrologic unit. For 6 of these 14 units, the conveyance alternative components were judged to be the best. This includes the three hydrologic units that were not considered for centralized or decentralized detention facilities because one unit contains the outlet from upper Kelly Lake, a natural detention pond, and the other two units contain completely developed residential neighborhoods with no area readily available for detention facilities. For two hydrologic units, the centralized detention alternative components were judged to be the best. For three hydrologic units, the decentralized detention alternative components were judged to be the best. For the remaining three hydrologic units, components common to both the decentralized and centralized detention alternatives were judged to be the best.

The recommended plan presented in Chapter VIII accordingly represents, for the planning area as a whole, a judicious combination of the conveyance, centralized detention, and decentralized detention alternatives. Chapter VIII more fully describes the recommended plan.

Map 19

SELECTED COMBINATION OF ALTERNATIVE PLANS FOR THE HALES CORNERS VILLAGE AREA



Source: SEWRPC.

Chapter VIII

RECOMMENDED STORMWATER MANAGEMENT PLAN

INTRODUCTION

In order to design a recommended stormwater management plan for the Village of Hales Corners, one of the three alternative plans considered for each hydrologic unit in the Village was selected for refinement and detailing and integration into a system plan for the Village as a whole. These three alternative plans, as presented in Chapter VII of this report, were, in each case, a conveyance alternative, a centralized detention alternative, and a decentralized detention alternative. The comparative evaluation of these plans, as described in Chapter VII, was focused primarily on the cost of the minor stormwater management system components of the plans. The hydraulic capacities of the minor system components were all designed to accommodate flows from storm events up to and including the 10-year recurrence interval event. The impacts of the alternative plans on the peak rates of flow in the receiving watercourses and the effects of stormwater detention on surface water quality were also considered in the comparative evaluation. The evaluation of the three alternatives indicated that different alternatives should be selected for each of the various hydrologic units. Of the 14 hydrologic units delineated in the village area, the pure conveyance alternative was found to be best for six of the units; the centralized detention alternative was found to be best for two of the units; the decentralized detention alternative was found to be best for three of the units; and a combination of the centralized and decentralized plan components was found to be best for three units.

This chapter presents the recommended stormwater management system plan for the Village. The minor system components are described in some detail, including the approximate locations, lengths, sizes, and slopes of storm sewers; the approximate locations, lengths, sizes, and slopes of open channels and grass swales; and the approximate locations, site areas, sizes, storage capacities, water depths, detention times, and outlet capacities of centralized and decentralized detention facilities. The ability of the partial roadway cross-sections to serve the required stormwater collection system effectively during minor storms while providing for adequate traffic movement was also determined. The capacities of the minor system components were sized to accommodate flows resulting from storm events up to and including the 10-year recurrence interval event.

This chapter also describes and evaluates the performance of the major stormwater management system components--the full street cross-sections, major open channel drainageways, and receiving natural watercourses. Street pavement crown elevations are recommended for all intersections and for all locations of recommended changes in street grade. The capacity of the major system components is evaluated on the basis of flows resulting from the 100-year recurrence interval storm event.

The design of the recommended plan is thus based upon careful consideration of many factors, with primary emphasis upon the degree to which the recommended stormwater management objectives and supporting standards are satisfied. Most important among the considerations were those relating to cost and to the ability of the system components to accommodate flows resulting from the design storm events without exacerbating downstream drainage and flooding problems.

PLAN RECOMMENDATIONS

Based upon the comparative evaluation of the various alternative plans considered, as set forth in Chapter VII, the minor and major stormwater management system components recommended for inclusion in the stormwater management system plan are set forth in Tables 38, 39, and 40 by hydrologic unit. The recommended plan is summarized in graphic form on a one inch equals 400 feet system plan map, Map 20, located in the pocket attached to the inside back cover of this report.

The minor stormwater management system includes conveyance, centralized detention, and decentralized detention system components which have been designed to contain flows for storm events up to and including the 10-year recurrence interval storm. Onsite detention and nonstructural components were utilized in conjunction with other alternative approaches. The conveyance components include storm sewers and related inlets, manholes, and outfalls, along with open channels. The centralized detention components include surface detention basins and ponds with associated facility inlets and outlets. The decentralized detention components include surface detention basins and ponds with associated inlets and outlets, along with natural and man-made turf-lined open channels and swales. The ability of yard swales and roadway cross-sections to collect and convey drainage to the minor conveyance system was considered in the design of the system. Table 38 presents a description of individual minor system components, along with the costs of the recommended plan.

The major stormwater management system includes conveyance components which have been designed to accommodate flows from a 100-year recurrence interval storm. Conveyance components include street cross-sections, major open channel drainageways, and receiving watercourses. The major stormwater management system consists of those minor stormwater management system components necessary to meet drainage requirements, together with certain components recommended to offset adverse impacts of the recommended minor system facilities on downstream flood flows. The major drainage system plan does not include facilities for comprehensive flood control. A description of the recommended major system components, along with their costs, is presented in Table 40.

The recommended stormwater management plan envisions that the full street cross-section will be utilized to convey flows in excess of those generated by a 10-year recurrence interval storm event and up to the flows generated by a 100-year recurrence interval storm event. In areas with existing urban street patterns, or in areas where street pattern plans were available, the capacity of the streets to convey the stormwater was calculated and evaluated. In other areas it was assumed that street patterns and grades will be developed which will be compatible with stormwater drainage needs. Recommended typical street cross-sections for arterial, collector, and minor land access streets are provided in Chapter VI of this report.

Table 38

SELECTED CHARACTERISTICS AND COSTS OF THE MINOR SYSTEM COMPONENTS OF THE RECOMMENDED HALES CORNERS STORMWATER MANAGEMENT PLAN

Project and Component Description	Estimated Cost	
	Capital	Annual Operation and Maintenance[a]
Western Portion of Village		
A. Northwest Branch-Whitnall Park Creek Improvements		
1. Add one 60-foot-long No. 36 corrugated metal pipe arch culvert under 124th Street..........	$ 3,000	$ 0
2. Add one 70-foot-long No. 48 corrugated metal pipe arch culvert under Robinwood Drive.......	4,000	0
3. 18.3-acre-foot detention facility at 116th Street and Grange Avenue, including 1,350-foot internal open channel..................	70,000	12,100
4. 1.6 acre-foot detention facility at 118th Street extended north of Woodside Drive.......	30,000	1,400
5. 15 acres of land to be used for stormwater detention..................................	60,000	0
6. 1,200 feet of 60-inch storm sewer from detention facility to 114th Street extended...	180,000	-200
7. 540-foot channel improvement from 114th Street extended to 113th Street..............	20,000	0
8. Add one 50-foot-long 54-inch corrugated metal pipe culvert under 113th Street.........	4,000	0
9. Miscellaneous roadside swale and culvert improvements................................	40,000	0
10. Miscellaneous and contingencies..............	69,000	2,100
Subtotal	$ 480,000	$15,400
B. North Branch-Whitnall Park Creek Improvements		
1. 1,420 feet of 48-inch storm sewer in 113th Street from Edgerton Avenue to Upham Avenue...	$ 170,000	$ -200
2. 940 feet of 54-inch storm sewer in 113th Street from Upham Avenue to Copeland Avenue and in Copeland Avenue from 113th Street to 112th Street.............................	130,000	-200
3. 330 lineal feet of 42-inch storm sewer in Copeland Avenue from 111th Street to 112th Street...............................	30,000	-100
4. Detention facility at Hales Corners Lutheran School, 5.3 acre-feet.......................	70,000	3,200
5. 580 feet of 60-inch storm sewer from detention facility to Grange Avenue....................	90,000	-100
6. 210 feet of 66-inch storm sewer from Grange Avenue to 180 feet south of Grange...........	40,000	0
7. 1,360 feet of channel reconstruction south of Grange Avenue............................	50,000	0
8. Miscellaneous and contingencies..............	90,000	400
Subtotal	$ 670,000	$ 3,000

Table 38 (continued)

Project and Component Description	Estimated Cost: Capital	Estimated Cost: Annual Operation and Maintenance[a]
C. North Branch-Whitnall Park Creek Junction Area Improvements		
1. Install one 30-foot-long by 12-foot-wide by 6-foot-deep concrete box culvert under driveway at Janesville Road and 111th Street extended	$ 25,000	$ 0
2. Install one 30-foot-long by 12-foot-wide by 6-foot-deep concrete box culvert under driveway at 111th Street one block south of Janesville Road	25,000	0
3. 150 feet of channel improvement	10,000	0
4. Miscellaneous and contingencies	10,000	0
Subtotal	$ 70,000	$ 0
E. Hale Park West Improvements		
1. 650-feet of 36-inch storm sewer in Ridge Trail from 122nd Street to 123rd Street	$ 60,000	$ -100
2. Install two 50-foot-long No. 36 corrugated metal pipe arch culverts under 123rd Street at Ridge Trail	10,000	0
3. 250 feet of new open channel at Kurtz Road	10,000	0
4. Miscellaneous sideyard and backyard swale improvements	15,000	0
5. Miscellaneous and contingencies	15,000	0
Subtotal	$ 110,000	$ -100
F. Village Hall Area Improvements		
1. 150 feet of 30-inch storm sewer connections	$ 10,000	$ 0
2. 750 feet of grass swale	15,000	300
3. Miscellaneous and contingencies	5,000	0
Subtotal	$ 30,000	$ 300
G. Hale Park Central (118th Street) Improvements		
1. 250 feet of 36-inch storm sewer in 118th Street extended from village boundary to Parkview Lane	$ 20,000	$ 0
2. 890 feet of 42-inch storm sewer in 118th Street from Parkview Lane to Indian Trail	90,000	-200
3. 360 feet of 36-inch storm sewer in Indian Trail from 118th Street to west	30,000	-100
4. Add one 50-foot-long 48-inch corrugated metal pipe culvert under Timberline Lane	4,000	0
5. Add one 60-foot-long 54-inch corrugated metal pipe culvert under 118th Street	6,000	0
6. Miscellaneous sideyard swale improvements	5,000	0
7. Miscellaneous and contingencies	25,000	0
Subtotal	$ 180,000	$ -300

Table 38 (continued)

Project and Component Description	Estimated Cost: Capital	Estimated Cost: Annual Operation and Maintenance[a]
H. Hale Park East Improvements		
1. Install one 50-foot-long No. 36 corrugated metal pipe arch culvert under Bridget Lane....	$ 2,000	$ 0
2. 150 feet of swale in 113th Street extended from Bridget Lane north......................	3,000	0
3. Miscellaneous and contingencies..............	1,000	0
Subtotal	$ 6,000	$ 0
Eastern Portion of Village		
I. Grange Avenue and S. 108th Street Area Improvements		
1. 500 feet of 24-inch storm sewer from Grange Avenue north 500 feet........................	$ 30,000	$ 0
2. 700 feet of 30-inch and 36-inch storm sewers in and adjacent to Grange Avenue from S. 107th Street to 108th Street......................	60,000	-100
3. Miscellaneous roadside swale and backyard swale improvements..........................	20,000	0
4. Miscellaneous and contingencies..............	20,000	0
Subtotal	$ 130,000	$ -100
J. Forest Home Area Improvements		
1. 780 feet of 36-inch storm sewer relay in Forest Home Avenue south of Grange Avenue.....	$ 65,000	$ 0
2. 60 feet of 24-inch storm sewer in Forest Home Avenue.................................	3,000	0
3. 1,550 feet of new open channel from Forest Home Avenue south and west of Meadow Park Drive.................................	45,000	600
4. Miscellaneous and contingencies..............	17,000	100
Subtotal	$ 130,000	$ 700
K. Kay Parkway Area Improvements		
1. 400 feet of 27-inch storm sewer from College Avenue north on Park Terrace Drive...........	$ 20,000	$ 100
2. 500 feet of 30-inch storm sewer in Park Terrace Drive..............................	40,000	100
3. 730 feet of 36-inch storm sewer in Park Terrace Drive to 106th Street................	60,000	100
4. 190 feet of 36-inch storm sewer from Park Terrace Drive north in 106th Street..........	20,000	0
5. 180 feet of 36-inch storm sewer in 106th Street from Parkland Court to Kay Parkway.....	10,000	0
6. 450 feet of 36-inch storm sewer in Kay Parkway from Parkland Court to Kay Parkway...........	40,000	-100
7. 100 feet of 36-inch storm sewer in Kay Parkway from 106th Street to the west 100 feet.......	10,000	0

Table 38 (continued)

Project and Component Description	Estimated Cost	
	Capital	Annual Operation and Maintenance[a]
K. Kay Parkway Area Improvements (continued)		
8. 200 feet of 36-inch storm sewer from Kay Parkway to the north 100 feet west of 106th Street....	20,000	0
9. Miscellaneous and contingencies..............	30,000	0
Subtotal	$ 250,000	$ 200
L. Meadow Park Area Improvements		
1. 280 feet of new open channel improvements from Meadow Park Drive to the west...........	$ 8,000	$ 100
2. Install one 50-foot-long 30-inch corrugated metal pipe culvert under Meadow Park Drive south of S. Bonnie Lane......................	2,000	0
3. 1,000 feet of new open channel west of Meadow Park Drive............................	30,000	400
4. Miscellaneous and contingencies..............	4,000	100
Subtotal	$ 44,000	$ 600
M. Garden Court Area Improvements		
1. Install one 60-foot-long 30-inch corrugated metal pipe culvert under Forest Park Drive at Meadow Park Drive.........................	$ 2,000	$ 0
2. 200 feet of channel improvements from Meadow Park Drive south to Garden Court.............	5,000	0
3. 260 feet of 30-inch storm sewer along Garden Court to low area in parkland................	20,000	0
4. Miscellaneous and contingencies..............	6,000	0
Subtotal	$ 33,000	$ 0
N. Brookside Drive Area Improvements		
1. 156 feet of 27-inch storm sewer in Brookside Drive near Allenwood Lane....................	$ 7,000	$ 0
2. 205 feet of 36-inch storm sewer in Brookside Drive west of Edgerton Avenue................	10,000	0
3. Miscellaneous roadside and sideyard swale improvements................................	10,000	0
4. Miscellaneous and contingencies..............	3,000	0
Subtotal	$ 30,000	$ 0
Total	$2,163,000	$19,700

[a] Costs were noted to be zero when the project proposed replacement of a component with a component which has similar operation and maintenance costs. Negative costs were noted when the replacement component was estimated to have a lower operation and maintenance cost than the cost of the existing facility.

Source: SEWRPC.

Table 39

PERTINENT CHARACTERISTICS OF PROPOSED DETENTION FACILITIES OF THE MINOR AND MAJOR STORMWATER MANAGEMENT SYSTEM

Hydrologic Unit Designation	Component Designation	Recurrence Interval Design (years)	Tributary Area (acres)	Storage Volume Provided (acre-feet)	Basin Size (acres)	Maximum Water Depth (feet)	Maximum Outlet Capacity (cfs)
A	A-3	10	444	18.3	18.4	4	119
A	A-4	10	43	1.6	1.1	2	38
B	B-4	10	109	5.3	0.9	6	130

Source: SEWRPC.

The hydraulic pathways for stormwater under major storm event conditions are shown on Map 20, which also indicates the location of those areas where the capacity of the street cross-section will likely be exceeded, and where adjacent land may be expected to be inundated during a major storm event. In such areas it has been determined that inundation of land outside the street cross-section will not cause major property damage or endanger human health or safety. Accordingly, no major drainage system improvements were recommended for these areas. 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, as well as to facilitate the design of the minor system; and are intended to be used as guides in the establishment of street grades throughout the village area as required by law.

Discussion of the Recommended Stormwater Management System by Hydrologic Unit

A brief summary of the stormwater drainage needs and the recommended plan components for each of the 14 hydrologic units in the planned urban service area is provided below.

Hydrologic Unit A: Approximately 60 percent of Hydrologic Unit A was developed as of 1980, with the remainder being in open space. In the plan design, it was assumed that an additional 30 percent would be developed for residential uses and the other 10 percent would remain as park and open space. Six minor stormwater drainage system problems were identified in this hydrologic unit: inadequate culvert capacity under 124th Street at Marquette Drive; inadequate culvert capacity under Robinwood Lane at Robinwood Court; inadequate pipe capacity at Woodside Drive near 118th Street extended; inadequate swale capacity along Grange Avenue at Monaco Lane; inadequate culvert capacity under 113th Street at Rockney Avenue extended; and inadequate swale capacity along Grange Avenue at 116th Street. Two minor and major system problems were identified: inadequate swale and driveway culvert capacity along 116th Street from Grange Avenue to Denis Avenue; and inadequate culvert capacity under 115th Street at Denis Avenue extended. An anticipated stormwater management problem is the need to accommodate the increased stormwater runoff by improving the existing minor drainage system and by anticipating increases in impervious surfaces due to more complete development of existing urban areas.

Table 40

SELECTED CHARACTERISTICS AND COSTS OF THE MAJOR SYSTEM COMPONENTS OF THE RECOMMENDED HALES CORNERS STORMWATER MANAGEMENT PLAN

Hydrologic Unit	Project and Component Description	Estimated Cost: Capital	Estimated Cost: Annual Operation and Maintenance[a]
B	North Branch-Whitnall Park Creek Improvements		
	Road regrading in W. Copeland Avenue from 111th Street to west of 112th, in S. 112th Street from Copeland Avenue to Grange Avenue, and in Grange Avenue to east and west of 112th Street	$ 80,000	$ 0
	Miscellaneous and contingencies	10,000	0
	Subtotal	$ 90,000	$ 0
C	North Branch-Whitnall Park Creek Junction Area Improvements		
	400 feet of channel improvement in 111th Street from Janesville Road to south	$ 20,000	$ 0
	Install one 25-foot-long by 16-feet-wide by 6-foot-deep concrete box culvert under driveway at 111th Street south of Janesville Road	25,000	0
	Miscellaneous and contingencies	10,000	0
	Subtotal	$ 55,000	$ 0
D	Upper Kelly Lake Discharge Channel Improvements		
	Add two 60-foot-long, 81-inch by 59-inch corrugated metal pipe arch culverts under 124th Street	$ 20,000	$ 0
	760-foot channel improvement from 124th Street to Godsell Road	40,000	0
	Add two 60-foot-long, 81-inch by 59-inch corrugated metal pipe arch culverts under Godsell Road	20,000	0
	Miscellaneous and contingencies	10,000	0
	Subtotal	$ 90,000	$ 0
K	Kay Parkway Area Improvements		
	Road reconstruction of 250 feet of Kay Parkway west of 106th Street	$ 13,000	$ 0
	Miscellaneous and contingencies	2,000	0
	Subtotal	$ 15,000	$ 0
	Total	$250,000	$ 0

[a]Costs were noted to be zero when the project proposed replacement of a component with a component which has similar operation and maintenance costs. Negative costs were noted when the replacement component was estimated to have a lesser operation and maintenance cost than the cost of the existing facility.

Source: SEWRPC.

To improve the existing conditions in the problem areas and to accommodate anticipated runoff conditions, 1,200 lineal feet of 60-inch-diameter storm sewer and 1,200 lineal feet of swale, 540 lineal feet of channel improvements, and a new 54-inch-diameter culvert are recommended as new components of the major and minor drainage systems. Recommended components of the minor system include a new 36-inch and new 48-inch corrugated metal culvert pipe, and a 18.3-acre-foot and a 1.6-acre-foot detention pond. In addition, limited local drainage improvements are recommended. The street system required to support future urban development in the unit should be carefully laid out to provide the necessary major drainage system conveyance capacity.

Hydrologic Unit B: Approximately 80 percent of Hydrologic Unit B was urbanized as of 1980, with the remainder being in open space. In the plan design it was assumed that the remaining 20 percent would be developed for residential uses. Four minor system problems were identified and four major and minor system problems were identified. The four minor system problems consist of inadequate culvert capacity at 113th Street and Woodside Drive; inadequate culvert capacity at 113th Street and Abbot Avenue; inadequate culvert capacity at 113th Street and Copeland Avenue; and inadequate culvert and swale capacity at 113th Street and Mallory Avenue. The four major and minor system problems that were identified consist of inadequate pipe capacity at 111th Street and Copeland Avenue; inadequate swale and pipe capacity at Copeland Avenue from 111th Street to 112th Street; inadequate pipe and swale capacity at 112th Street north of Grange Avenue; and inadequate culvert capacity at 112th Street and Grange Avenue.

To improve existing conditions in the problem areas and to accommodate anticipated runoff conditions, approximately 2,580 lineal feet of new storm sewer, ranging in size from 48 inches to 60 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 112th Street. The following streets are proposed to be lowered and reconstructed: 112th Street between Copeland Avenue and Grange Avenue from 2.1 feet to 1.2 feet; Copeland Avenue in the vicinity of 112th Street from 0 foot to 2.1 feet; and Grange Avenue in the vicinity of 112th Street from 0 foot to 1.2 feet. The Copeland Avenue and 111th Street intersection is proposed to be raised about 0.5 foot. Approximately 1,420 lineal feet of turf-lined channel is recommended to be constructed from 200 feet south of Grange Avenue to Parnell Avenue.

Hydrologic Unit C: Approximately 90 percent of Hydrologic Unit C was urbanized as of 1980, with the remainder being in open space. In the plan design it was assumed that the remaining 10 percent would be developed for residential uses. One minor system problem was identified and one major and minor system problem was identified in this unit. An anticipated stormwater management problem is the need to accommodate the increased stormwater runoff from possible new urban development over the remaining 10 percent of the hydrologic unit. To accommodate this increase in runoff and to improve existing conditions, three concrete box culverts, ranging in size from 12 feet wide by 6 feet high to 16 feet wide by 6 feet high, are proposed to be constructed under a driveway at Janesville Road and 111th Street extended, under a driveway at 111th Street south of Janesville Road, and under a driveway at 111th Street one block south of Janesville Road. In addition, approximately 550 lineal feet of open channel from Janesville Road south is proposed to be improved. The street system required to support future urban development in the unit should be carefully laid out to provide the necessary major drainage system capacity.

Hydrologic Unit D: Hydrologic Unit D was fully urbanized as of 1980. Three major system problems were identified: inadequate culvert capacity under 124th Street downstream from the Upper Kelly Lake outlet; inadequate culvert capacity under W. Godsell Road 200 feet west of Kurtz Road; and inadequate ditch capacity between these culverts. To improve existing conditions and to accommodate anticipated stormwater management problems, two 81-inch by 59-inch corrugated metal pipe arch culverts are proposed to be installed under 124th Street adjacent to the existing 72-inch-diameter culverts downstream from the Upper Kelly Lake outlet; two 81-inch by 59-inch corrugated metal pipe arch culverts are proposed to be installed under W. Godsell Road adjacent to the existing 72-inch-diameter culverts 200 feet west of Kurtz Road; and the grass-lined open channel between the culverts is proposed to be regraded.

Hydrologic Unit E: Hydrologic Unit E was fully urbanized as of 1980. The problems encountered in this minor system are inadequate swale and cross-culvert capacities. To improve the existing conditions in the problem areas and to accommodate anticipated runoff conditions, approximately 650 lineal feet of 36-inch-diameter storm sewer in Ridge Trail from 122nd to 123rd Street is proposed to be installed as a part of the minor drainage system. In addition, 250 lineal feet of channel construction at Kurtz Road is recommended. A limited number of local drainage improvements are also recommended.

Hydrologic Unit F: Hydrologic Unit F was fully urbanized as of 1980. One problem is anticipated in this minor system: inadequate capacity in the storm sewer draining the paved area between the two village public works equipment garages. To improve the existing conditions in the problem area and to accommodate anticipated runoff conditions, approximately 150 lineal feet of 30-inch-diameter storm sewer is proposed to be installed as part of the minor drainage system. In addition, approximately 750 lineal feet of grass swale is proposed to be constructed along this paved area, ending at the main branch of Whitnall Park Creek, as a part of the minor drainage system.

Hydrologic Unit G: Hydrologic Unit G was fully urbanized as of 1980. The anticipated problems in this minor system are inadequate storm sewer and culvert capacities. To improve the existing conditions in the problem areas and to accommodate runoff conditions, approximately 1,500 lineal feet of new storm sewer ranging in size from 36 inches to 42 inches in diameter is proposed along 118th Street extended from the village boundary north and along Indian Trail. In addition, 48-inch-diameter and 54-inch-diameter culverts are proposed to be installed under Timberline Drive and 118th Street, respectively, as a part of the minor drainage system. A limited number of local drainage improvements are also recommended.

Hydrologic Unit H: Hydrologic Unit H was fully urbanized as of 1980. One minor system problem is insufficient culvert capacity. To improve the existing conditions in the problem area and to accommodate anticipated runoff conditions, approximately 150 lineal feet of grass swale is proposed to be constructed as part of the major and minor drainage systems for the unit. In addition, a 36-inch-diameter culvert is proposed to be installed under Bridget Lane as a part of the minor drainage system.

Hydrologic Unit I: Hydrologic Unit I was fully urbanized as of 1980. Two minor system problems are inadequate capacity of a storm sewer along Grange Avenue and 108th Street and inadequate cross-culvert capacity in the same area. To improve the existing conditions in the problem area and to accommodate anticipated runoff conditions, approximately 1,200 lineal feet of new storm sewer ranging in size from 24 inches to 36 inches in diameter is proposed to be installed as part of the minor drainage system.

Hydrologic Unit J: Approximately 50 percent of Hydrologic Unit J was urbanized as of 1980, with the remainder being in open space. In the plan design it was assumed that an additional 30 percent will be developed into high-density residential uses and the remaining 20 percent will remain as county parkland. One problem is anticipated in this minor system: inadequate storm sewer capacity along Forest Home Avenue. To improve the existing conditions in the problem area and to accommodate anticipated runoff conditions, approximately 780 lineal feet of 36-inch-diameter storm sewer relay in Forest Home Avenue south of Grange Avenue and 60 lineal feet of 24-inch-diameter storm sewer in Forest Home Avenue are proposed to be installed. In addition, 1,550 lineal feet of open channel is proposed to be constructed from Forest Home Avenue south and west of Meadow Park Drive as a part of the minor drainage system.

Hydrologic Unit K: Approximately 40 percent of Hydrologic Unit K was urbanized as of 1980, with the remainder being in open space. In the plan design it was assumed that the remaining 60 percent will be developed into residential uses. One minor and major system problem was identified: inadequate storm sewer capacity at Kay Parkway and 106th Street extended. An anticipated stormwater management problem is the need to accommodate the increased stormwater runoff that will result from current proposals for an additional 20 percent of new urban development over an additional 40 percent of the hydrologic unit. To accommodate this anticipated increase in runoff and to improve existing conditions, approximately 2,750 lineal feet of new storm sewer ranging in size from 30 inches to 36 inches in diameter is proposed to be installed as a part of the minor drainage system. The street system required to support future urban development should be carefully laid out to provide the necessary major drainage system conveyance capacity.

Hydrologic Unit L: Approximately 50 percent of Hydrologic Unit L was urbanized as of 1980, with the remainder being in open space. In the plan design it was assumed that the remaining 50 percent will be developed into residential uses. An anticipated stormwater management problem is the need to accommodate the increased stormwater runoff from possible new urban development over an additional 50 percent of the hydrologic unit. To accommodate this increase in runoff, 1,000 lineal feet of turf-lined open channel is recommended to be constructed as a part of the minor drainage system. In addition, 280 lineal feet of channel improvement is recommended from Meadow Park Drive to the west. Also, a 30-inch-diameter culvert is proposed to be installed under Meadow Park Drive south of S. Bonnie Lane. The street system required to support future urban development should be carefully laid out to provide the necessary major drainage system conveyance capacity.

Hydrologic Unit M: Approximately 80 percent of Hydrologic Unit M was urbanized as of 1980, with the remainder being in open space. In the plan design it was assumed that the remaining 20 percent will be developed into residential uses. Three minor system problems were identified: insufficient culvert, storm sewer, and open swale capacities. To accommodate the increased runoff of future development and to improve existing conditions, approximately 60 lineal feet of new storm sewer 30 inches in diameter is proposed to be installed as a part of the minor drainage system for the unit. In addition, a 30-inch-diameter culvert is proposed to be installed under Forest Park Drive. Approximately 200 lineal feet of turf-lined open channel improvement is also recommended to be constructed as a part of the minor drainage system. The street system required to support future urban development should be carefully laid out to provide the necessary major drainage system conveyance capacity.

Hydrologic Unit N: Approximately 90 percent of Hydrologic Unit N was urbanized in 1980, with the remainder being in open space. In the plan design it was assumed that the remaining 10 percent will be developed into residential uses. Four minor system problems were identified: Two storm sewer segments at Brookside Drive and Allenwood Lane, along with two storm sewer segments at Brookside Drive at Edgerton Drive, have insufficient capacity to accommodate the flow of stormwater runoff. To accommodate the increased runoff of future development and to improve existing conditions, 361 lineal feet of new storm sewer ranging in size from 27 inches to 36 inches in diameter is proposed to be installed as a part of the minor drainage system for the unit. In addition, a limited amount of local drainage improvements are recommended. The street system required to support future urban development should be carefully laid out to provide the necessary major drainage system conveyance capacity.

Stream Channel Modifications and Associated Detention Facility Components

As already noted, the recommended major stormwater management system includes certain components recommended to offset any adverse impacts of the recommended minor stormwater management facilities on downstream flows. Table 41 presents estimated 10- and 100-year recurrence interval flood flows at pertinent locations throughout the study area under both existing and future land use and drainage system conditions. In addition, Table 41 presents estimated 10- and 100-year recurrence interval flood flows under future land use and recommended minor and major drainage system conditions.

Channel modifications along two stream segments are recommended as components of the major drainage system. The first modification involves channel profile adjustments along the North Branch of Whitnall Park Creek from Janesville Road to approximately 200 feet beyond the confluence with Whitnall Park Creek. The existing channel bottom slope varies from approximately five feet per mile (0.001 foot per foot) to approximately 80 feet per mile. The channel bottom would be lowered approximately two feet at the outfall from Hydrologic Unit C. The implementation of these two plan recommendations may be expected to permit the effective operation of the minor stormwater drainage system in Hydrologic Unit M.

The second modification involves channel profile adjustments in the main branch of Whitnall Park Creek in Hydrologic Unit D. The stream segment has been documented as a problem area because of a history of local flooding. In addition, urbanization of the tributary area over the planning period is expected to cause 10- and 100-year recurrence interval flood stages to increase by about 0.8 foot on Upper Kelly Lake. Review of hydraulic conditions in this area indicates that channel improvements are necessary in order to provide sufficient capacity for expected channel discharges. The proposed channel profile for this segment of Hydrologic Unit D is shown in Appendix A.

Table 41

COMPARISON OF 10-YEAR AND 100-YEAR RECURRENCE INTERVAL FLOOD FLOWS FOR WHITNALL PARK CREEK, NORTH BRANCH OF WHITNALL PARK CREEK, AND NORTHWEST BRANCH OF WHITNALL PARK CREEK UNDER EXISTING AND FUTURE CONDITIONS

Location	Existing Land Use and Drainage System Conditions (cfs)		Future Land Use and Existing Drainage System Conditions (cfs)		Future Land Use and Recommended Drainage System Conditions (cfs)	
	10 Year	100 Year	10 Year	100 Year	10 Year	100 Year
North Branch-Whitnall Park Creek						
at Copeland Avenue and 113th Street..........	120	212	126	216	134	230
at Grange Avenue.......	173	316	185	321	120	260
Northwest Branch-Whitnall Park Creek						
at 124th Street........	116	240	164	315	164	315
at Grange Avenue.......	112	257	112	257	112	279
at Janesville Road.....	207	399	204	398	305	545
Whitnall Park Creek at Kurtz Road						
above confluence......	120	231	120	231	247	471
below confluence......	140	307	201	430	251	505
at 116th Street........	209	446	270	569	270	569
at Forest Home Avenue..	546	1,116	592	1,207	760	1,398

Source: SEWRPC.

In addition to the channel improvements described above, there are two areas where roadway reconstruction is necessary in order for the system to convey stormwater properly from the 10-year and 100-year recurrence interval storm events. The first area is in the Hales Corners Lutheran School area of Hydrologic Unit B located at 112th Street between Copeland Avenue and Grange Avenue. The two intersections are recommended to be reconstructed and lowered about 2.1 and 1.2 feet, respectively. This will allow stormwater runoff from a 100-year recurrence interval storm event to flow over the road, as opposed to backing up and flooding nearby houses. In addition, Copeland Avenue is recommended to be reconstructed from 112th Street to 111th Street, with the 111th Street and Copeland Avenue intersection being raised about 0.5 foot. Raising the intersection will provide enough grade to more effectively convey stormwater runoff from the adjacent area into the North Branch of Whitnall Park Creek.

The second area of recommended roadway reconstruction is in the Kay Parkway area of Hydrologic Unit K. Kay Parkway is recommended to be lowered by about one foot at a point about 350 feet east of Janesville Road, where an unnamed tributary of Whitnall Park Creek crosses under the road. This should prevent stormwater runoff from a 100-year recurrence interval storm event from flooding nearby houses.

The final three components of the major stormwater system in the recommended plan are culverts. Two are in Hydrologic Unit D, below Upper Kelly Lake. Two 81-inch-span by 59-inch-rise corrugated metal pipe arch culverts are proposed to be installed under 124th Street and another two 81-inch-span by 59-inch-rise corrugated metal pipe arch culverts are proposed to be installed under Godsell Road, through which Whitnall Park Creek passes.

The third culvert is in Hydrologic Unit C directly below the confluence of Whitnall Park Creek and the North Branch of Whitnall Park Creek. A 16-foot-wide by 6-foot-high concrete box culvert is proposed to be installed under the driveway to the Holtz Motors outside storage area off 111th Street.

One Hundred-Year Recurrence Interval Flood Flows and Floodplain

Major drainage system flood flows and stages, and attendant flood hazard areas, have been determined by others under the Federal Flood Insurance Study for Whitnall Park Creek and its major tributaries. The Regional Planning Commission used the "ILLUDAS" hydrologic-hydraulic simulation model to develop the data presented in this report. This ILLUDAS model was used to simulate 10- and 100-year recurrence interval flood discharges under existing 1980 and future (2000) land use conditions and existing channel conditions. The resulting flood discharges were then used in a uniform flow channel hydraulic model and a culvert hydraulic model to determine flood stages and the corresponding flood hazard areas. Where complex and nonuniform conditions existed, the resulting flood discharges were then used in the U. S. Army Corps of Engineers' HEC-2 hydraulic backwater model to determine flood stages and flood hazard areas. The results of these simulation model analyses were used as a basis for the comparative evaluation of the effects of the recommended major drainage system improvements. The impacts of the recommended major drainage system improvements on the 100-year recurrence interval flood flows and stages along stream reaches within the village urban area were considered in detail.

The recommended stormwater management plan includes major drainage system components providing both improved conveyance and increased stormwater detention capacity. Recommended conveyance improvements consist of channel modifications and channel profile adjustments that would increase the hydraulic capacity of the open channels concerned by either increasing the cross-sectional area of the channels or increasing the velocity of the waters being transported. Both of these types of adjustment tend to increase peak downstream flows. Recommended storage improvements consist of both increased storage capacity and an improved distribution of storage in the major drainage system. These recommendations are designed to offset the effects of the improved channel conveyance capacity and attendant reduction in floodplain storage. The detention facilities reduce the overall volume of stormwater runoff by allowing some of the detained stormwater to percolate into the groundwater system, and increase the time required to transport surface waters out of the watershed. The impacts of these stormwater management plan recommendations are considered below by subwatershed.

Main Branch-Whitnall Park Creek: Estimated 10-year and 100-year recurrence interval flood flows for selected locations and land use conditions along the Main Branch of Whitnall Park Creek are set forth in Table 41. The 100-year recurrence interval flood flow at Forest Home Avenue is estimated to be 1,116 cubic feet per second (cfs) under existing land use and channel conditions. Under planned land use and existing channel conditions, the flow is estimated to be 1,207 cfs. Under planned land use and recommended stormwater drainage system conditions within the Hales Corners Village area, the 100-year recurrence interval flood flow at the same location is estimated to be 1,398 cfs.

Even though there is a significant increase in the flow rate, the major and minor stormwater flow criteria as defined in Chapter IV of this report would be met. In some instances, the flood flows will increase because of a recommended increase in channel or culvert capacity. This increased capacity was selected as the most cost-effective method of meeting major and/or minor drainage criteria. Downstream locations were then analyzed to determine if additional capacity was needed. In this manner, downstream reaches were reviewed to verify that the recommended plan components would accommodate flows compatible with plan components recommended upstream.

North Branch-Whitnall Park Creek: Estimated 10-year and 100-year recurrence interval flood flows for various locations and land use conditions along the North Branch of Whitnall Park Creek are set forth in Table 41. The 100-year recurrence interval flood flow at Copeland Avenue at 113th Street is estimated to be 212 cfs under existing land use and channel conditions. Under planned land use and existing channel conditions, the 100-year recurrence interval flood flow at the same location is estimated to be 216 cfs, or about the same flow as under existing conditions. Under planned land use and recommended stormwater drainage system conditions within the Hales Corners urban service area, the 100-year recurrence interval flood flow at the same location is estimated to be 230 cfs. The plan recommends the lowering of 112th Street from Copeland Avenue to Grange Avenue and the raising of Copeland Avenue at 111th Street.

Northwest Branch-Whitnall Park Creek: Estimated 10-year and 100-year recurrence interval flood flows for various locations and land use conditions along the Northwest Branch of Whitnall Park Creek are set forth in Table 41. The 100-year recurrence interval flood flow at Janesville Road is estimated to be 399 cfs under existing land use and channel conditions. Under planned land use and existing channel conditions, the 100-year recurrence interval flood flow at the same location is estimated to be 398 cfs, or the same flow as under existing conditions. Under planned land use and recommended stormwater drainage system conditions within the Hales Corners Village area, the 100-year recurrence interval flood flow at Janesville Road is estimated to be 545 cfs. Portions of land areas in Franklin presently draining into the Northwest Branch of Whitnall Park Creek will drain away from this creek under proposed development. This diversion of flow, however, may be expected to only partially offset the increases in flow resulting from further urbanization in the area.

Nonpoint Source Pollution Abatement

Some of the recommended minor and major stormwater management system components would provide substantial reductions in nonpoint source pollutant loadings. Construction site erosion control measures are also recommended.

The recommended detention ponds and turf-lined open channels would remove a substantial portion of the pollutant loadings discharging into these facilities. The pollutant removal rates for the individual ponds and channels are provided in Chapter VII. About 244 acres in the Village, or approximately 12 percent of the Village, and about 444 acres in the study area, or 12 percent of the study area, would drain to at least one or both of the recommended detention ponds. Of the 444 acres, 43 acres, or 10 percent, would drain to both detention ponds since the Woodside Drive pond would lie within the drainage area of the WEMP Radio Station pond. On an annual basis, the ponds may be expected to remove about 90 percent of the total solids, about 80 percent of the lead, and about 50 percent of the total phosphorus carried by the runoff discharged into the ponds.

About 563 acres, or about 27 percent of the Village and 15 percent of the study area, would drain to the turf-lined open channels. These channels would allow stormwater to infiltrate the soil and remove associated pollutants by filtration and settling. The channels would be most effective in removing pollutants during smaller storm events and least effective during larger storm events. Pollutant removal rates would range from 5 to 85 percent during a mean storm event, and from less than 5 to 20 percent during a 10-year recurrence interval storm event. Since the stormwater would infiltrate into the soil, the pollutant removal rates would be the same for all pollutants and would be equal to the resulting reduction in storm runoff volume. In addition to open channels, roadside swales would allow pollutants to settle out.

Construction Site Erosion Control: It is recommended that erosion associated with construction and development activities be controlled through the implementation of an erosion control ordinance. Upon request, the Commission staff would assist the Village in drafting a construction erosion control ordinance. The ordinance would include the definition of land disturbance activities subject to control, set forth standards and criteria for erosion control, describe permit application and administration procedures, identify enforcement and appeal procedures, and define pertinent terms used in the ordinance.

Auxiliary Plan Recommendations

The foregoing recommendations primarily address stormwater drainage system improvements. To provide a comprehensive stormwater management plan, however, these drainage system recommendations must be supplemented by plan elements relating to natural resource and open space protection, and by the continual proper maintenance of the stormwater drainage system.

Natural Resource and Open Space Preservation: A land use plan should be adopted for the Village that provides for the preservation of the primary environmental corridors, including associated floodlands and wetlands, in essentially natural, open uses. The protection of floodlands and wetlands from intrusion by urban land uses has important implications for stormwater management since these lands can provide needed capacity for the storage, infiltration, and transport of stormwater runoff. As presented in Table 19 of Chapter IV, the probable future land use pattern used in the plan design and evaluation process envisions the preservation of about 55 acres of agricultural and other open lands within the Village, or about 3 percent of the total village area; and of about 187 acres of surface water, wetlands, and woodlands, or about 9 percent of the total village area.

Floodplain Map Revisions: It is recommended that the Village request revision of the FEMA Flood Hazard Boundary Maps from the Federal Insurance Administration in two stages. Numerous citizens whose homes can be removed from the floodplain will benefit. In the conduct of the flood flow and stage analyses under the stormwater management study, the Commission staff utilized two-foot contour interval, one inch equals 100 feet scale topographic maps, supplemented by field survey measurements and elevations. Tributary areas were delineated into approximately 20-acre increments, and drainageway reaches of modest length were utilized so as to fully meet federal standards for hydrologic and hydraulic analyses attendant to floodplain delineation. The Village should immediately upon adoption of this plan and implementation of significant improvements submit a "Partial Appeal to More Accurately Reflect Boundaries" to the Chicago FEMA office based on the revised hydrologic and hydraulic data contained in this report. Further subsequent revisions in the floodplain boundaries can be requested by the Village at such time as the drainage improvements herein recommended have been constructed and are operational.

Maintenance of Stormwater Management Facilities: The effectiveness of the stormwater conveyance and detention facilities, once developed, can be maintained only if proper operation, repair, and maintenance procedures are carefully followed. Important maintenance activities include the periodic inspection and repair of storm sewers, clearing of sewer obstructions, maintenance of open channel vegetative lining, clearing of debris and sediment from open channels, maintenance of detention facility inlets and outlets, maintenance of detention basin vegetative cover, periodic removal of sediment accumulated in detention basins, and sweeping of parking lots used as detention facilities. These maintenance activities are recommended to be carried out on a continuing basis to maximize the effectiveness of the stormwater management facilities and measures, and to protect the capital investment in the facilities. Cost estimates of the recommended maintenance activities are included in the total plan costs.

Stormwater Management System Costs

The capital and operation and maintenance costs of the recommended stormwater management plan are presented by hydrologic unit and component in Tables 38 and 40. Table 38 presents those costs required for implementation of the minor drainage system and Table 40 presents those costs required for implementation of the major drainage system.

The capital cost of the recommended stormwater management plan is estimated to be $2.43 million, of which $2.2 million, or 90 percent, is attributed to minor system costs, and $0.25 million, or 10 percent, is attributed to major system costs. The annual operation and maintenance cost increase of the recommended stormwater management plan is estimated to be $19,700, all attributable to the minor system. These costs are based upon full development of the urban service area and do not include the cost of minimum-diameter collector sewers and road culverts that may be required to drain collector and land access roadways, the alignment of which has not as yet been determined, or the cost of roadway sections in newly developing areas that have been designated to function as a component of the major drainage system.

IMPACTS OF RECOMMENDED STORMWATER MANAGEMENT PLAN

Hydraulic Impacts

The primary impact of the recommended stormwater management plan is that storm flows from a 10-year recurrence interval storm event or smaller will be safely and efficiently conveyed by the minor drainage system to major drainage channels with only minimal inconvenience to residents. Also, storm flows from a 10-year to a 100-year recurrence interval storm event will not be significantly increased along the main stem of and major tributaries to Whitnall Park Creek, and in some instances will be effectively reduced as a result of the stormwater management plan recommendations.

Water Quality Improvement

The recommended plan will provide water quality benefits in that it will result in the detention of some stormwater runoff, with subsequent settling of particulate pollutants within the detention facilities. The attendant reductions in such pollutants as biochemical oxygen-demanding organic materials, nutrients, and toxic metals such as lead are consistent with, and serve to advance, the regional water quality management plan prepared and adopted by the Regional Planning Commission, and will help in achieving the recommended water quality standards in the stream system.

SUMMARY

Based on the best alternative for each of 14 hydrologic units in the Hales Corners urban service area, a recommended stormwater management system plan was developed which includes minor system components and major system components. The minor system components are designed for a 10-year recurrence interval peak flow, and the major system components are designed for a 100-year recurrence interval peak flow.

The recommended minor system components consist of 12,410 lineal feet of new storm sewers with associated appurtenances, and three detention facilities. The major system components include 4,430 lineal feet of engineered open channels. The total capital cost of the recommended plan is $2.43 million, and the average annual operation and maintenance cost is about $20,700. The plan recommends the most cost-effective means of resolving existing and probable future drainage and flooding problems in the Village, thereby reducing the public costs attributable to improperly functioning drainage facilities. Implementation of the recommended plan would provide protection against substantial inconvenience to residents during minor storm events, and against major property damage or a significant hazard to human health and safety during major storm events. It would support the continued sound land use development and redevelopment of the Village, enhancing the quality of life within the Village.

Chapter IX

PLAN IMPLEMENTATION

INTRODUCTION

The recommended stormwater management plan described in Chapter VIII is designed to attain, to the maximum extent practicable, the stormwater management objectives and standards set forth in Chapter V of this report. In a practical sense, however, the plan is not complete until the steps to implement it--that is, to convert the plan into action policies and programs--have been specified. Following formal adoption of this plan by the Village of Hales Corners, realization of the plan will require a long-term commitment to the objectives of the plan and a high degree of coordination and cooperation among village officials and staff, land developers, and concerned citizens in undertaking the substantial investments and series of actions needed to provide both existing and future urban development in the Hales Corners area with an efficient and effective stormwater drainage system. The plan should be used as a guide for the development of the stormwater drainage system and related stormwater management measures in the Village.

The first section of this chapter describes the relation of the future village land use development to the effectiveness of the planned stormwater management measures. The second section discusses the importance of more detailed engineering to implementation of the plan. The third section sets forth the actions required to implement the plan. A preliminary plan schedule of implementation and financing is set forth in the fourth section. The fifth section discusses the need for periodic reevaluation and updating of the plan itself.

RELATION TO FUTURE LAND USE DEVELOPMENT

Fundamental to implementation of a sound stormwater management plan is coordination with future land use development. The design year 2000 land use for the stormwater management planning area was summarized in Chapter IV of this report. To a large extent, the effectiveness of the recommended stormwater management measures will depend upon the degree to which future land use development and the stormwater management plan properly supplement and complement each other.

Implementation of the stormwater management plan will assure that the Village of Hales Corners will be served by a stormwater drainage system that is economical and effective; that has the capacity to accommodate stormwater runoff from not only existing development but planned future development; and that will not exacerbate existing or create new downstream flooding problems. The plan also provides an estimate of the capital investment required to meet the stormwater management needs, allowing the public officials and developers concerned to fairly allocate immediate and future capital cost requirements, as well as to determine the operation and maintenance costs to be imposed upon the Village.

Importantly, the stormwater management plan identifies those areas of the Village which should be preserved in open, natural uses. Such preservation will provide economies in stormwater management--maximizing the use of natural stormwater conveyance and storage, and permitting such conveyance and storage to be incorporated into the stormwater management plan and system. If the preservation of these open areas is greatly compromised, stormwater management problems, such as localized flooding, poor drainage, and water pollution, may be expected to result.

RELATION OF DETAILED ENGINEERING DESIGN TO SYSTEM PLANNING

The systems-level stormwater management plan presented in this report is intended to serve as a guide to the future design and construction of stormwater management facilities. The detailed engineering phase begins where the systems planning phase ends. The detailed engineering design should examine in greater depth and detail the variations in the technical, economic, and environmental features of the recommended solutions to problems identified in the system plan in order to determine the best means of carrying out the plan. The resulting facility development plans should be fully consistent with the stormwater collection, conveyance, and detention facility recommendations presented in Chapter VIII of this report.

Chapter V of this report presented engineering design criteria and analytic procedures used in the preparation and evaluation of the alternative stormwater management system plans. These criteria and procedures, firmly based in current engineering practice, provided the means for quantitatively sizing and analyzing the performance of both the minor and major stormwater drainage system components. These criteria and procedures should also serve as a basis for the more detailed design of stormwater management system components in the implementation of the recommended plan. It is important that such criteria and procedures be applied uniformly and consistently in all phases of implementation of the plan if the resulting system is to perform as envisioned in the plan. Accordingly, Table 42 sets forth the design criteria and analytic procedures recommended to be followed in the detailed engineering design of the recommended plan components. Criteria and procedures are presented in the table for estimating stormwater flows, calculating hydraulic capacities of conveyance facilities, designing street cross-sections and related site grading, locating and designing storm sewer inlets, designing storm sewers, designing roadside swales, open channels, and culverts, and designing detention facilities. In this respect, it is recognized that over time new design techniques may be developed and become available for use in the design of stormwater management system components. Such techniques should, however, be carefully reviewed before adoption for consistency with the criteria and procedures set forth in the plan.

PLAN IMPLEMENTATION

Plan Adoption

An important first step in plan implementation is the formal adoption of the recommended stormwater management plan, as documented herein, by the Village of Hales Corners Public Works and Plan Commissions and by the Village Board.

Table 42

DESIGN CRITERIA AND PROCEDURES RECOMMENDED TO BE FOLLOWED IN THE DETAILED ENGINEERING DESIGN OF THE RECOMMENDED STORMWATER MANAGEMENT SYSTEM COMPONENTS

Design Function	Recommended Criteria and Procedure
Storm Runoff Flows	Minor system components should be designed to accommodate flows expected from a 10-year recurrence interval storm event. Major system components should be designed to accommodate flows expected from a 100-year recurrence interval storm event. To determine peak rates of flow for the design of pure conveyance facilities with no significant upstream storage, the Rational Method should be used as described in SEWRPC <u>Technical Record</u>, Vol. 2, No. 4, April-May 1965, "Determination of Runoff for Urban Storm Water Drainage System Design." The rainfall intensity, duration, and frequency curves suitable for use with the Rational Method are provided in Figure 1 of Chapter III. When storage is to be included in the facilities and estimates of runoff volumes as well as peak rates of discharge are required, the modified Rational Method or a suitable hydrologic-hydraulic simulation model should be used.
Conveyance Facilities	Manning's formula should be used to determine hydraulic capacities of conveyance facilities where flow conditions approximate uniform conditions. The use of Kutter's formula is also acceptable for uniform pipe flow computations. Storm sewers should be designed to flow full during the design storm event. Flow velocities should not be less than 2.5 feet per second in storm sewers. The chart set forth in Figure 17 of Chapter V should be used to determine the hydraulic elements of storm sewers. A chart relating open channel cross-section slopes and capacity is provided in Figures 19 through 22 of Chapter V. Flow velocities should not exceed five feet per second in turf-lined channels. Where flow conditions do not approach uniform conditions, backwater, drawdown, or inlet control conditions should be determined mathematically or by use of appropriate nomographs.
Street Cross-Sections and Related Site Grading	Except in special cases, streets should be designed with rural cross-sections providing roadside swales. Typical street cross-sections are shown in Figure 28 of Chapter VI of this report. Slopes away from all buildings, as well as the slopes of interior drainage swales, should be at least one-quarter inch per foot to provide positive drainage.
Storm Sewer Inlets	Storm sewer inlet location and capacity should be dictated by the allowable stormwater spread and depth of flow in streets. Combination inlets should be used in most instances. Uncontrolled flow across streets should not be allowed when the streets are functioning as a part of the minor stormwater drainage system.
Culverts	The length and size of recommended culverts are set forth in Tables 38 and 40 of Chapter VIII. Culvert capacities should be determined by using the charts set forth in Figures 8 through 15 in Chapter V.
Detention Facilities	The recurrence interval design, size, capacity, and discharge rate of recommended centralized detention facilities are set forth in Table 39 of Chapter VIII. Storage volumes should be calculated using a modification of the Rational Method, or using a hydrologic-hydraulic simulation model. It is recommended that the design storm pattern include a rising limb so that the peak intensity does not occur at the beginning of the storm, and that the rainfall duration extend beyond the outflow hydrograph peak.

NOTE: For a more detailed discussion of these design criteria see Chapter V of this report.

Source: SEWRPC.

adoption, the stormwater management plan becomes the official guide to the making of stormwater management decisions by village officials. Such formal adoption serves to signify agreement with, and official support of, the recommendations contained in the plan, and enables the village staff to begin integrating the plan recommendations into the ongoing public works development planning and programming and subdivision plat review processes of the Village.

Implementation Procedures

Following formal plan adoption, the Village can draw upon a number of legal and administrative tools to assist in plan implementation. These tools include development proposal review; a capital improvements program; conformance with the village zoning ordinance; an appeal of FEMA floodplain boundary maps; a maintenance program; and coordination with stormwater management programs in adjacent communities.

In reviewing subdivision plats, the Village Plan Commission should determine the compatibility of the plats with the land use assumptions set forth in the stormwater management plan. Any proposed departures from those assumptions should be carefully considered in light of the stormwater management needs of the proposed development and the impacts on upstream and downstream areas. It should be noted that development within the Village will be limited and that the future development most likely to have an impact on stormwater drainage will take place in adjacent communities. Thus, it will be important to attempt to achieve agreement with the Cities of New Berlin and Greenfield in order to coordinate development in those communities with the plan recommendations.

Capital improvements programming can also be an important tool for implementing the recommended stormwater management plan. Typically, a capital improvements program is a five-year program for the timing and financing of priority capital improvement projects. Such a program is based upon the projected financial capability of the community and is formulated from a detailed analysis of municipal revenues, debt service obligations, financing procedures, and external funding potentials. Once formulated, the program should be reevaluated, refined, and extended on an annual basis. The Village has a well-developed procedure for capital improvement financing, and it is recommended that the stormwater management plan components be incorporated into the program in a manner consistent with the construction schedule set forth below.

Implementation of the zoning map and ordinance will ensure that the identified stormwater management needs and problems are in balance with the layout and capacity of the recommended stormwater management system components. In addition, unlike subdivision control which operates on a plat-by-plat basis, this zoning ordinance operates over the entire Village in advance of development proposals, serving to increase public acceptance of the plan recommendations and improving coordination between upstream development and downstream stormwater management.

Implementation of the plan will allow the Federal Emergency Management Agency, upon the request of the Village, to revise the floodplain boundary maps following submittal of substantiating information. Such revisions, however, cannot be made until the stormwater management and flood control measures concerned are actually in place. Revision will then eliminate the need for property owners in the Village to purchase flood insurance, since the revised floodplain boundaries will not include any structures.

A common stormwater management problem facing municipalities is a lack of a continuing maintenance program for stormwater facilities, including periodic inspection and routine preventive maintenance. This problem is caused by the absence of an assured, continuous source of funding, and incomplete records to justify budgeting for this funding. Stormwater facility maintenance can be easily ignored for a limited period of time, and many officials and citizens alike incorrectly perceive that certain components, such as open channels or sewers, are self-maintaining, or that no hazards will result if such facilities become defective. However, a sound, continuing, preventive maintenance program must be given a high priority, particularly for a stormwater management system which includes various types of components such as storm sewers, roadside swales, culverts, open channels, and onsite and centralized detention facilities that are interrelated and interconnected. The Village does have a maintenance program for drainage facilities. It is therefore recommended that the public works program of the Village continue to provide for the maintenance, as well as construction, of the stormwater management facilities--including periodic inspection of conveyance and detention facilities; timely repair of facilities; cleaning of storm sewers, open channels, and detention facility inlets and outlets; maintenance of open channel and detention facility lining materials; and periodic removal of accumulated sediment from conveyance, detention, and sediment control facilities.

In addition to the land development coordination noted above, it will be necessary to coordinate specific stormwater management facilities with adjacent communities. In the case of the City of Franklin, it will be necessary to install a segment of reinforced concrete pipe storm sewer in Franklin south of the Hale Park Central area in order for the Franklin tributary area to meet the stated plan objectives. In order for the Hale Park Central area to be served most effectively by the proposed 36-inch-diameter storm sewer along 118th Street extended south of Parkview Lane, it is recommended that this sewer be extended 300 feet south of the Hales Corners-Franklin municipal border to the south side of Bel Mar Drive. Installation of this segment of storm sewer will ensure that the maximum floodwater elevations, as established in Chapter IV of this report, will not be exceeded. Installation of this segment will entail a capital cost for Franklin of approximately $3,100. This amount is not included in the cost tables in the report since they include the costs of items only in the Village of Hales Corners. It is assumed that the City of Franklin will pay for implementation of this part of the project, since both the project location and the area to which benefits will accrue lie entirely within the City of Franklin.

Plan formulation for the City of Greenfield was based upon the stormwater drainage plans for the City obtained from the City. These plans indicate a future reduction in tributary area due to planned construction and land regrading. With regard to the City of New Berlin, plan formulation was based upon the land use changes and channel improvements for the City set forth in that City's 1974 stormwater drainage master plan.

PLAN SCHEDULE OF IMPLEMENTATION AND FINANCING

Upon adoption of the recommended stormwater management plan by the Village Board, full implementation of the plan will require that the system development costs be allocated equitably between the public sector and the private sector, that the means of financing the plan components be identified, and that a schedule of capital and operation and maintenance costs be prepared. Public sector costs would primarily be borne by the Village of Hales Corners, although state or county funds could be used to construct and maintain certain stormwater drainage systems associated with state or county trunk highways. Private sector costs would, in most cases, be borne by land developers, and these costs would generally be passed on to individual land parcel purchasers.

Total plan implementation costs would include land acquisition, construction, operation and maintenance, facility replacement, and administrative costs. The plan costs presented herein, however, include only the construction, or capital, costs and operation and maintenance costs. The schedule of capital and operation and maintenance costs would result in total plan implementation over the 12-year period of 1985 through 1996. Land acquisition, facility replacement, and administrative costs are not included in the plan costs, with the exception of the cost of the land required for the large detention basin located north of W. Grange Avenue and west of 116th Street--$60,000. Most of the recommended stormwater management facilities can be placed in public street rights-of-way. Nevertheless, land acquisition costs may be significant for some types of facilities, particularly in existing, developed areas. However, the acquisition of land by dedication during land development and the joint use of some facilities, such as the joint use of detention facilities for recreational activities, can minimize acquisition costs. The new facilities recommended in the plan are not expected to require replacement prior to the year 2000, and administrative costs, such as the cost of reviewing the stormwater management elements of a subdivision plat by the village staff, are considered part of the normal village government expenditures.

Schedule of Public Sector and Private Sector Costs

The development of a plan implementation schedule requires that a construction completion date be designated for each recommended stormwater management component, and that it be determined whether each component will be funded by the public sector or the private sector. It is recommended that the highest priority for construction be given to those components which resolve the most serious existing stormwater problems which generally result in the flooding of structures; and that the second level of priority be given to those remaining components which would resolve less severe urban problems, such as street and yard flooding. Consideration was also given to the need to coordinate the drainage projects with other village projects which have specific construction timetables, and to the structural condition of the facilities being replaced. In general, capital costs were assumed to be borne by the public sector if the components were designed to serve public property, or if the general public--not simply the owners of the new or adjacent existing development--would benefit from the component. Capital costs were assumed to be borne by the private sector if the primary benefit of the component would accrue to the new development. The following criteria were applied to allocate capital costs to the public sector and private sector:

1. Upgraded, existing drainage system components intended to resolve existing stormwater problems for more than an isolated area, and components designed to serve public property, are assumed to be funded by the public sector.

2. Components, or portions of components, designed to serve specific, new, private urban development, or to solve an isolated problem, are assumed to be funded by the private sector.

3. Components intended to serve specific, new, private urban development which must be oversized to provide capacity for additional upstream urban development in the future are assumed to be funded by both the public sector and the private sector. The portion of the total capital cost allocated to each sector is based upon the percentage of the total component service area covered by the specific new urban development. The private sector is assumed to finance the costs of serving the specific new urban development; the public sector is assumed to finance the costs of the oversizing required to serve the additional urban development upstream.

All operation and maintenance costs for conveyance facilities--storm sewers and open channels--were assumed to be financed by the public sector, regardless of whether public sector or private sector funds were used to construct the facilities. It was assumed that all conveyance facilities constructed with private sector funds would be dedicated to the Village following construction. Public sector and private sector expenditures are listed in Table 43.

The recommended stormwater management program provides for the distribution of the necessary capital and operation and maintenance costs over the 12-year plan implementation period. This expenditure schedule is described graphically on Map 21 and is set forth in Table 44. Capital expenditures are described as public sector or private sector costs. The ultimate adoption of schedules of capital and operation and maintenance costs will require a determination by village officials of the timing of implementation of individual plan elements, and of the best means of financing.

Public Sector Financing

Several means of financing stormwater management components are available to local governmental agencies that are not available to the private sector. However, although these means offer flexibility, certain constraints and limitations are imposed on these financing methods by state law and, especially, by the approvals required of the electorate. Therefore, successful public financing of the recommended plan will require a thorough study of costs and available revenues, careful financial planning, public information programs, and a timely approach for securing public support and approvals.

In addition to using current tax revenue sources, such as property taxes, the Village of Hales Corners may make use of such revenue sources as user fees or special assessments, reserve funds, borrowing, tax incremental financing district funds, and gifts.

As of 1985, one tax incremental financing district had been created in the Village of Hales Corners. When such a district is created, a "tax incremental base" is established; this base is the aggregate value of all taxable property in the district as of the date of creation as equalized by the Wisconsin Department of Revenue. Any subsequent growth in the tax incremental district base is then "captured" so that as property value increases, levies on this growth represent positive dollar increments used for financing redevelopment.

Table 43

ASSIGNMENT OF PUBLIC SECTOR AND PRIVATE SECTOR COSTS FOR SYSTEM COMPONENTS OF THE RECOMMENDED STORMWATER MANAGEMENT PLAN

Hydrologic Unit Designation	Component Designation	Public Sector		Private Sector		Total	
		Capital	Annual Operation and Maintenance	Capital	Annual Operation and Maintenance	Capital	Annual Operation and Maintenance
A	1	$ 3,000	$ --	$ --	$ --	$ 3,000	$ 0
A	2	4,000	--	--	--	4,000	0
A	3	70,000	12,100	--	--	70,000	12,100
A	4	30,000	1,400	--	--	30,000	1,400
A	5	60,000	--	--	--	60,000	0
A	6	180,000	-200	--	--	180,000	-200
A	7	20,000	--	--	--	20,000	0
A	8	4,000	--	--	--	4,000	0
A	9	40,000	--	--	--	40,000	0
A	M&C	69,000	2,100	--	--	69,000	2,100
B	1	170,000	-200	--	--	170,000	-200
B	2	130,000	-200	--	--	130,000	-200
B	3	30,000	-100	--	--	30,000	-100
B	4	70,000	3,200	--	--	70,000	3,200
B	5	90,000	-100	--	--	90,000	-100
B	6	40,000	--	--	--	40,000	0
B	7	50,000	--	--	--	50,000	0
B	8	80,000	--	--	--	80,000	0
B	M&C	100,000	400	--	--	100,000	400
C	1	25,000	--	--	--	25,000	0
C	2	25,000	--	--	--	25,000	0
C	3	10,000	--	--	--	10,000	0
C	4	20,000	--	--	--	20,000	0
C	5	25,000	--	--	--	25,000	0
C	M&C	20,000	--	--	--	20,000	0
D	1	20,000	--	--	--	20,000	0
D	2	40,000	--	--	--	40,000	0
D	3	20,000	--	--	--	20,000	0
D	M&C	10,000	--	--	--	10,000	0
E	1	60,000	-100	--	--	60,000	-100
E	2	10,000	--	--	--	10,000	0
E	3	10,000	--	--	--	10,000	0
E	4	15,000	--	--	--	15,000	0
E	M&C	15,000	--	--	--	15,000	0
F	1	10,000	--	--	--	10,000	0
F	2	15,000	300	--	--	15,000	300
F	M&C	5,000	--	--	--	5,000	0
G	1	20,000	--	--	--	20,000	0
G	2	90,000	-200	--	--	90,000	-200
G	3	30,000	-100	--	--	30,000	-100
G	4	4,000	--	--	--	4,000	0
G	5	6,000	--	--	--	6,000	0
G	6	5,000	--	--	--	5,000	0
G	M&C	25,000	--	--	--	25,000	0
H	1	2,000	--	--	--	2,000	0
H	2	3,000	--	--	--	3,000	0
H	M&C	1,000	--	--	--	1,000	0
I	1	30,000	--	--	--	30,000	0
I	2	60,000	-100	--	--	60,000	-100
I	3	20,000	--	--	--	20,000	0
I	M&C	20,000	--	--	--	20,000	0
J	1	65,000	--	--	--	65,000	0
J	2	3,000	--	--	--	3,000	0
J	3	45,000	600	--	--	45,000	600
J	M&C	17,000	100	--	--	17,000	100
K	1	4,800	100	15,200	--	20,000	100
K	2	10,000	100	30,000	--	40,000	100
K	3	16,200	100	43,800	--	60,000	100
K	4	3,000	--	17,000	--	20,000	0
K	5	5,000	--	5,000	--	10,000	0
K	6	8,500	-100	31,500	--	40,000	-100
K	7	3,000	--	7,000	--	10,000	0
K	8	6,000	--	14,000	--	20,000	0
K	9	13,000	--	--	--	13,000	0
K	M&C	32,000	--	--	--	32,000	0
L	1	8,000	100	--	--	8,000	100
L	2	2,000	--	--	--	2,000	0
L	3	30,000	400	--	--	30,000	400
L	M&C	4,000	100	--	--	4,000	100
M	1	2,000	--	--	--	2,000	0
M	2	5,000	--	--	--	5,000	0
M	3	20,000	--	--	--	20,000	0
M	M&C	6,000	--	--	--	6,000	0
N	1	7,000	--	--	--	7,000	0
N	2	10,000	--	--	--	10,000	0
N	3	10,000	--	--	--	10,000	0
N	M&C	3,000	--	--	--	3,000	0
Total	--	$2,249,500	$19,700	$163,500	$ --	$2,413,000	$19,700

NOTE: M&C denotes miscellaneous and contingencies.

Source: SEWRPC.

The effect of the tax incremental law, then, is to delay the availability to general government of the revenues that result from the increase in values due to improvements in the tax incremental district until the public costs entailed in generating the development have been paid for. Tax incremental financing could be an attractive means of financing some of the recommended stormwater management system components. The Village has used this program to finance other public works projects.

Borrowing, with the use of general obligation bonds, combined with property tax revenues may also be an effective and acceptable means of financing plan components. User fees, special assessment districts, and utility assessments, while being equitable and dependable means of financing stormwater management, have not been widely used in southeastern Wisconsin, and, accordingly, may not be politically acceptable in the Village of Hales Corners.

Other than Wisconsin Department of Natural Resources (DNR) nonpoint source pollution abatement program funds, state and federal grants are generally not available to finance stormwater management measures. However, the Village may be able to obtain some financial assistance from the DNR Wisconsin Fund Nonpoint Source Pollution Abatement Program for the construction of the major recommended detention basin to be located at 116th Street and Grange Avenue since that basin would provide water quality benefits. It is recommended that the Village, in consideration of the costs and revenues involved, legal issues, equity concerns, and political and public acceptance, evaluate potential financing programs and develop a program which assures a sufficient, reliable funding source. Furthermore, as described above, incorporating expenditures for stormwater management facilities into a sound overall capital improvements program is an important means of prioritizing and scheduling the financing of the plan. The operation and maintenance costs attendant to implementation of the plan should be funded out of the village general fund as part of the ongoing public works program. The expected increase in operation and maintenance costs of $19,700 per year upon full plan implementation may be compared to the portion of the village public works budget allocated to drainage, streets, landscaping, winter operation, and lighting--about $390,000 in 1985. The increase may be expected to be phased in over a 12-year period as new facilities are constructed.

Private Sector Financing

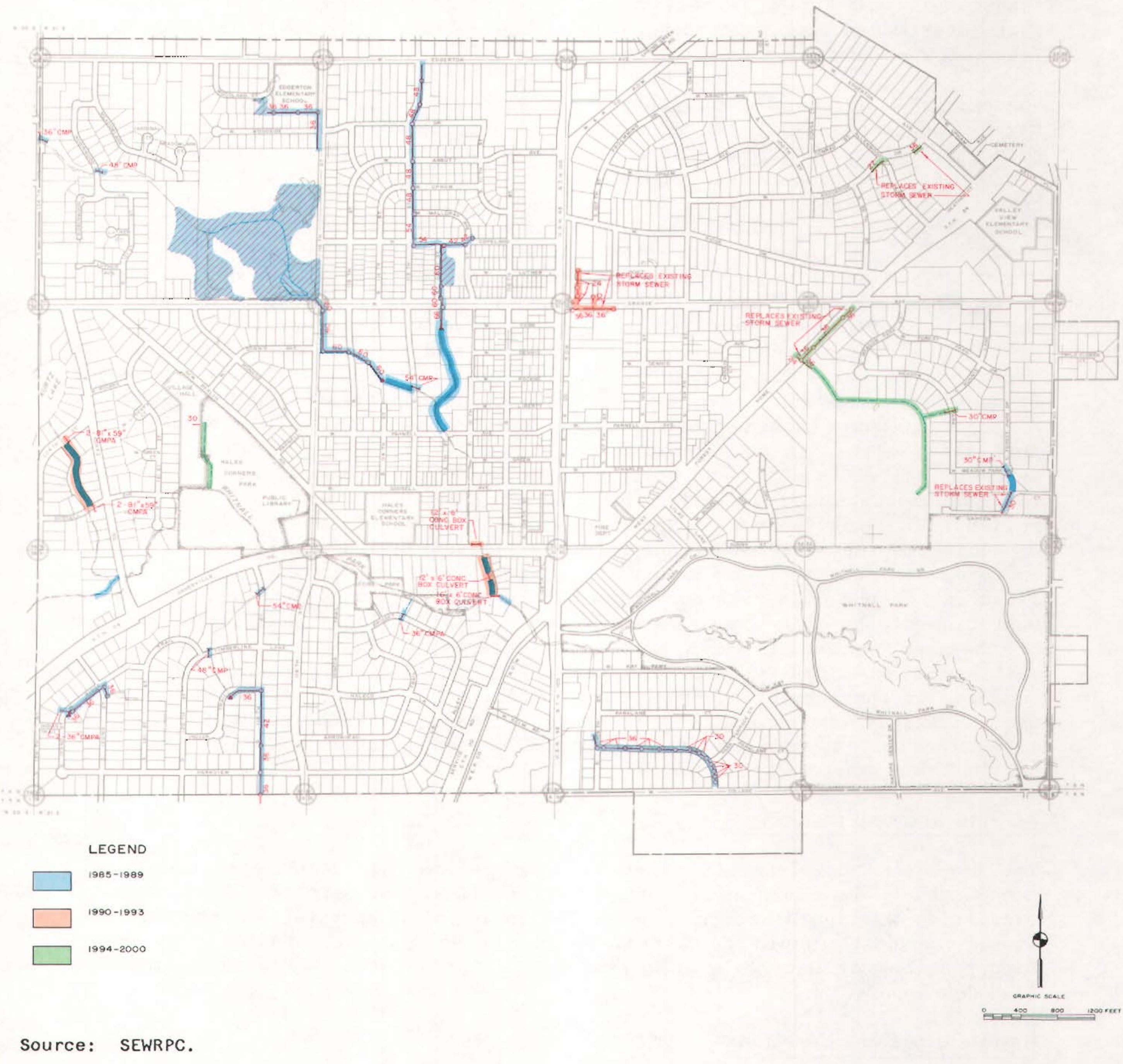
For new urban developments which contain recommended stormwater management components to be financed by the private sector, provision of the recommended facilities would ordinarily be a condition of plat approval by the Village. Thus, the costs would be ultimately borne by the land parcel purchasers. Contributions of materials and services to the Village may also be made by land developers.

Regulatory Considerations

Implementation of some of the drainage improvements recommended in this system plan may require the prior approval of certain regulatory agencies other than the Village, including the Milwaukee Metropolitan Sewerage District, 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 dispute between the staffs of at least two of the regulatory agencies

Map 21

ESTIMATED CONSTRUCTION COMPLETION DATES FOR THE RECOMMENDED STORMWATER MANAGEMENT PLAN: 1985-2000



Source: SEWRPC.

Table 44

SCHEDULE OF CAPITAL AND OPERATION AND MAINTENANCE EXPENDITURES FROM PUBLIC AND PRIVATE SOURCES FOR THE RECOMMENDED STORMWATER MANAGEMENT PLAN: 1985-1996

Time Interval	Hydrologic Units to be Completed	Public Sector		Private Sector		Total	
		Capital	Annual Operation and Maintenance	Capital	Annual Operation and Maintenance	Capital	Annual Operation and Maintenance
1985-1989	A,B,E,H,K,M,G	$1,670,500	$18,500	$163,500	--	$1,834,000	$18,500
1990-1993	C,D,I	345,000	-100	--	--	345,000	-100
1994-1996	F,J,L,N	234,000	1,300	--	--	234,000	1,300
Total	--	$2,249,500	$19,700	$163,500	--	$2,413,000	$19,700

Source: SEWRPC.

concerned. Accordingly, the Village should seek legal counsel prior to proceeding with any drainage improvements that involve the construction or improvement of artificial waterways connecting to navigable waters; the alteration or enclosure of navigable watercourses; the removal of material from the beds of navigable watercourses; or the filling of wetlands.

The federal regulatory authority relates to the filling of wetlands and is granted under Section 404 of the Federal Water Pollution Control Act of 1972 as amended. The administering agency is the U. S. Army Corps of Engineers.

The state regulatory authority relates to the construction or improvement of artificial waterways connecting to, or located within, a prescribed distance of a navigable waterway; the alteration of navigable waterways; the placement of deposits or structures in the bed of navigable waterways or the enclosure of navigable waterways; and the removal of material from navigable waters. The authority is contained in Sections 30.12, 30.19, 30.195, 30.196, and 30.20 of the Wisconsin Statutes. The administering agency is the Wisconsin Department of Natural Resources. Some of the authority granted to that Department under these sections of the Statutes may not apply within counties having a population of 500,000 or more, Milwaukee County being the only such county in the State at this time.

Finally, under Section 66.894 of the Wisconsin Statutes, the Milwaukee Metropolitan Sewerage District has authority to improve watercourses within the District, such improvement including the deepening, widening, or otherwise changing of watercourses, including navigable watercourses, where such change is deemed necessary to carry off surface or drainage waters. This District authority has been disputed in some instances by the staff of the Wisconsin Department of Natural Resources. Under the cited and related authority, the District has promulgated rules requiring municipalities to obtain the prior approval of the District for the construction of certain types of drainage improvements. Accordingly, because the Village of Hales Corners lies within Milwaukee County and within the Milwaukee Metropolitan Sewerage District, certain drainage improvements within the Village may be subject to approval by the Milwaukee Metropolitan Sewerage District.

PLAN REEVALUATION AND UPDATING

The recommended stormwater management components, as well as the forecasts and assumptions used as a basis for plan development, should be reevaluated at 10-year intervals, in light of changes in actual village development. The plan components, including the need for certain facilities and the location, size, and capacity of facilities, should be revised as necessary to reflect changing development patterns and stormwater management needs. In addition, in the initial plan development it was necessary to limit the analysis and recommendations to major conveyance and detention facilities, since the layout of collector and land access streets had not been determined. A major effort in plan updating should be directed toward developing recommendations and updating inventories for these smaller-size conveyance elements both in the Village and upstream areas as development plans are prepared, and incorporating this information into the master stormwater management plan.

SUMMARY

This chapter has presented recommendations for implementing the stormwater management plan for the Hales Corners Village area through the year 2005. This plan should be used as a guide for stormwater drainage system development and other stormwater management measures within the village area. The chapter discusses the relation of future land use development to the plan and the essential role of detailed engineering design activities in implementing the plan.

The initial step in plan implementation is formal adoption of the plan by the Village Public Works and Plan Commissions and by the Village Board. The recommended plan should be integrated into the Village's public works program to initiate construction of the recommended facilities, as well as to ensure reliable and stable operation and maintenance of both the existing and new facilities. In order to implement the plan, the Village should review subdivision plats to determine conformance between future land uses and the recommended plan, and incorporate public expenditures for stormwater management into a sound overall capital improvements program for the Village.

The plan is recommended to be implemented over the 12-year period extending from 1985 through 1996. About $2.25 million, or about 93 percent of the total plan capital cost of about $2.41 million, is recommended to be borne by the public sector, primarily the Village. The remaining $0.16 million, or about 7 percent of the capital cost, would be financed by the private sector, primarily land developers and land parcel purchasers. All of the approximately $19,700 average annual operation and maintenance cost increase over the 12-year implementation period would be financed by the public sector. This estimated increase in operation and maintenance cost may be compared to the general portion of the village public works annual budget allocated to drainage, streets, landscaping, winter operation, and lighting--about $390,000 in 1985. The increase may be expected to be phased in over a 12-year period as new facilities are constructed.

The total average annual cost of the recommended plan is about $207,000, or about $26 per person per year, based on a projected year 1991 resident village population of 7,875 persons. If the project costs are paid for over a longer period, such as 20 years, the average annual cost would be reduced to $132,000, or $16 per person per year (based on a 1995 population of 8,150), plus any financing cost associated with the longer pay period. The means of financing the public sector costs are recommended to be determined by village officials, but likely sources of funding include property tax revenues, general obligation bonds, and tax incremental financing district funds.

The recommended stormwater management plan provides the Village of Hales Corners with important guidelines for coordinating land use development and stormwater drainage and control. The stormwater management plan will assist village officials in guiding the physical development of the Village. In this respect, implementation of the plan will contribute toward enhancing the overall quality of the environment within the village area, and thereby contribute toward making the Village of Hales Corners a safer, more attractive and healthful, and more efficient and economical area in which to live and work.

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Chapter X

SUMMARY OF STORMWATER MANAGEMENT PLAN FOR THE VILLAGE OF HALES CORNERS

Stormwater drainage, or, as it has been more recently called, stormwater management, consists of the collection, temporary storage, transport, and disposal of excess stormwater. Stormwater drainage is thus one of the most important requirements of sound urban development, and is essential to the provision of an attractive, efficient, safe, and healthful urban environment. Inadequate stormwater management can be costly and disruptive, can create health and safety hazards, and can have adverse effects on the overall quality of the environment. Good stormwater management planning involves the art of urban engineering, the sciences of hydrology and hydraulics, and economic and environmental impact assessment, and takes into account public perceptions of, and attitudes toward, stormwater drainage problems.

Substantial urban development is anticipated in drainage areas lying upstream of the Village of Hales Corners over the next two decades. In the absence of adequate planning and engineering, this development may be expected to exacerbate existing, and create new, stormwater management problems within the Village. Therefore, the Village requested the Regional Planning Commission to assist the Village in the preparation of a long-range stormwater management system plan for the Hales Corners area. The plan is intended to facilitate the development of an effective stormwater management system for the Village that will minimize the damages attendant to poor drainage, as well as the costs of stormwater management facilities. The recommended stormwater management plan for the Village of Hales Corners focuses on stormwater drainage, addressing flood control problems only as necessary to avoid the intensification of existing problems or the creation of new problems along the natural streams and watercourses of the study area which must receive the discharges from urban drainage facilities.

The plan recognizes that the basic concepts underlying urban stormwater management are undergoing revision. The new concepts are aimed at controlling the quality, as well as quantity, of runoff, and seek to manage stormwater as a resource rather than to treat it as a nuisance. These new concepts envision the stormwater management system of an urban area as consisting of two elements: a minor element to manage the runoff from the smaller, more frequent rainfall events; and a major element to manage the runoff from the larger, less frequent rainfall and snowmelt events. The former is intended to avoid the nuisances attendant to minor ponding of stormwater runoff in yards and streets, and consists of curbs and gutters, or road ditches, and storm sewer inlets and sewers. The latter is intended to avoid the much more serious flooding of basements and even first floors of buildings, and consists of the full cross-section of the public streets and ways, discharging to either engineered or natural streams and watercourses. The major system is designed, insofar as practicable, to utilize storage as well as conveyance to minimize costs and water pollution. As part of the planning process, criteria and procedures were developed for use by the Village in estimating stormwater flows and for designing street cross-sections, storm sewer inlets, storm sewers, open drainage channels, storage facilities, pumping facilities, culverts, and water quality management measures.

PLANNING AREA

The planning area consisted of the incorporated area of the Village of Hales Corners, together with the drainage basins lying upstream of, and tributary to, the drainage system of the village proper. The planning area is drained by Whitnall Park Creek, the Northwest Branch of Whitnall Park Creek, the North Branch of Whitnall Park Creek, Tess Corners Creek, and the Root River. The study area is about 3,726 acres, or about 5.8 square miles, in areal extent, of which about 56 percent lies within the 1985 corporate limits of the Village. The boundaries of the planning area and the corporate limits of the Village, together with the drainage basin boundaries, are shown on Map 1 of Chapter II. The resident population of the Village was 7,110 persons in 1980, and is expected to increase to about 8,500 persons by the year 2000. The resident population of the study area was 10,800 persons in 1980, and is expected to increase to about 14,700 persons by the year 2000.

INVENTORY AND ANALYSIS

Data were collated and collected on the existing land use, climate, soils, natural and man-made stormwater drainage systems, drainage and flooding problems, and erosion and sedimentation control problems of the planning area. Data were also collated on existing water quality conditions in the area, and on those sources of water pollution related to stormwater management.

Urban land uses in 1980 occupied about 71 percent of the total planning area, with residential uses making up the largest urban land use category. Agricultural and other open uses still accounted for about 20 percent of the planning area, with other open uses, including woodlands, wetlands, and surface water, constituting about 9 percent of the planning area.

Because the relationships between rainfall intensity, duration, and frequency are important considerations in stormwater management planning, the Regional Planning Commission has developed a set of rainfall intensity-duration-frequency relationships for use in estimating peak rates of runoff anywhere in southeastern Wisconsin, including the Hales Corners area, together with data for use in estimating the volume of rainfall and stormwater runoff associated with a given frequency and duration storm event.

Soil properties are an important factor influencing the rate and amount of stormwater runoff from land surfaces. Accordingly, the soils of the area were categorized into four hydrologic soil groups, and the location and extent of the areas covered by each group mapped and quantified.

In 1985, the village storm sewer system serviced a tributary drainage area of about 274 acres, or about 7 percent of the total planning area. The system consisted chiefly of storm sewers and culverts flowing to natural drainage outlets. A total of 16 storm sewer subsystems existed in the Village, each discharging through an outlet to the natural drainage outlets. Five minor lakes and ponds also existed within the planning area at the time of inventory.

A field survey was conducted to identify stormwater runoff-related erosion and sedimentation problems in the study area. Construction site erosion, cropland erosion, and eroded gullies and stream banks were identified as the major existing problems.

ANTICIPATED GROWTH

The stormwater drainage plan was intended to identify the stormwater management needs of the Hales Corners area through the year 2000. Accordingly, information was collected on the anticipated type, density, and spatial distribution of land uses in the planning area, and on the impact of anticipated changes in land use on the stormwater management needs of the planning area. Under future land use conditions, about 689 acres of land, or an additional 18 percent of the planning area, may be expected to be converted from rural to urban land uses, resulting in about 90 percent of the total planning area being in urban land uses by the plan design year.

DESIGN CRITERIA AND OBJECTIVES

Early in the planning process, stormwater management design criteria, as well as objectives, were established and agreed upon. The plan was developed considering two basic objectives: 1) to prevent significant monetary damage from any reasonably foreseeable major storm event--defined as a 100-year recurrence interval event; and 2) to provide convenient access to the various land uses of the urban area following minor, more frequent rainfall events--defined as events up to and including the 10-year recurrence interval event. The plan was thus designed to consider both major--operating infrequently--and minor--operating frequently--stormwater management facilities.

The minor stormwater drainage system is intended to minimize the inconveniences attendant to inundation from more frequent storms and consists of sideyard and backyard drainage swales, street curbs and gutters, roadside swales, storm sewers, and some stormwater storage facilities. It is composed of the engineered paths provided for stormwater runoff to reach the receiving streams and watercourses during the more frequent, but minor, storm events.

The major stormwater drainage system is designed for conveyance and/or storage 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.

ALTERNATIVE PLANS

Prior to designing and evaluating alternative stormwater management plans, the existing stormwater drainage system was evaluated. The hydraulic capacities of the major components of the existing system were determined and compared to estimated design flows. Those system components which were found to be unable to accommodate the runoff expected from the design storms under either existing or future land use conditions, or both, were thus identified, and the deficiencies of these components were then addressed in the design of alternative stormwater management plans. Problem components were identified under both existing and design year development conditions. The locations of the inadequate components so identified are shown on Map 15 of Chapter VII.

The following major deficiencies of the existing stormwater management system of the Village were identified:

- Inadequate drainage ditch and culvert conveyance capacity was found along Grange Avenue from Monaco Lane to 116th Street, along 116th Street from Grange Avenue to Denis Avenue, and through private yards from 116th Street and Denis Avenue to 114th Street and Rockne Avenue extended.

- Inadequate drainage ditch and culvert conveyance capacity and a low pocket formed by adverse gradient streets were found along 113th Street from Edgerton Avenue to Copeland Avenue, in Copeland Avenue from 113th Street to 111th Street, and in 112th Street from Copeland Avenue to Grange Avenue.

- Inadequate drainage ditch and culvert conveyance capacity was found in the outlet channel from Upper Kelly Lake between 124th Street and Godsell Road.

- Inadequate drainage ditch and culvert conveyance capacity and an insufficient street gradient were found in Ridge Trail from 122nd Street to Indian Trail.

- Inadequate drainage ditch and culvert conveyance capacity was found along and one-half block west of 118th Street from College Avenue extended to immediately south of Janesville Road.

- Inadequate culvert conveyance capacity was found in the North Branch of Whitnall Park Creek from Godsell Road to the confluence with Whitnall Park Creek.

- Inadequate conveyance capacity was found in the storm sewer in Forest Home Avenue from the south village boundary to Whitnall Park Creek.

- Inadequate ditch, culvert, and storm sewer conveyance capacities were found in the vicinity of Grange Avenue and 108th Street.

- Inadequate storm sewer conveyance capacities were found in Forest Home Avenue between Grange Avenue and Janesville Road.

- Inadequate ditch enclosure conveyance capacities were found in Brookside Drive from 97th Street extended to Edgerton Avenue.

Three alternative stormwater management system plans were evaluated for the Hales Corners area: 1) a conveyance alternative, 2) a centralized detention alternative, and 3) a decentralized detention alternative. The conveyance alternative proposed new storm sewers and engineered open channels to abate existing stormwater runoff problems and to effectively serve planned new urban development within the Village of Hales Corners. This alternative is shown on Map 16 in Chapter VII.

The centralized detention alternative proposed eight relatively large centralized surface detention facilities, and one centralized parking lot storage facility, to store temporarily a portion of the stormwater runoff generated from the planning area for subsequent slow release to the drainage system. Storage would also be provided by the preservation of certain floodlands, wetlands, and other natural, open areas. These storage facilities were designed to reduce downstream discharges, allowing, in some cases, the use of smaller conveyance facilities downstream. This alternative would also require some new conveyance facilities similar to, but less expensive than, those required under the conveyance alternative. The centralized detention alternative is shown on Map 17 in Chapter VII.

The decentralized detention alternative considered 24 relatively small detention basins, but found that only nine were effective in reducing downstream conveyance needs. Accordingly, the plan proposed five relatively small decentralized detention facilities supplemented by numerous rooftop and parking lot detention facilities. The plan would also require the reconstruction of some existing storm sewers, the construction of some new storm sewers, and some engineered open channels to serve planned development within the Village. The decentralized detention alternative would also require significant new conveyance facilities similar to, but less expensive than, those required under the conveyance alternative. Storage would also be provided by the preservation of certain floodlands, wetlands, and other natural, open areas. This alternative is shown on Map 18 in Chapter VII.

Plan Evaluation

A comparative evaluation of the three alternative stormwater management system plans indicated that the capital cost of the plans may be expected to range from about $2.44 million for the centralized detention alternative to about $3.50 million for the conveyance alternative; while the attendant annual operation and maintenance costs may be expected to range from about $7,600 for the conveyance alternative to about $35,800 for the decentralized detention plan. Assuming the capital costs entailed would be spread over a 20-year plan implementation period, plan implementation costs, including the operation and maintenance costs, may be expected to range from $134,000 per year for the centralized detention alternative to $179,000 per year for the conveyance alternative.

Based upon the evaluation of the three alternative plans, a recommended system plan was designed. This plan represents a judicious combination of the most efficient features of the conveyance, centralized detention, and decentralized detention alternatives. The recommended plan combines three detention basins, two decentralized surface detention ponds, new and reconstructed storm sewers, and engineered open channels to serve the existing and planned development within the Village effectively and economically. The plan also utilizes to the extent practicable the storage capacity of certain floodlands and wetlands. The components of the major stormwater management system are fully detailed in the plan, with additional consideration being given to those components both upstream and downstream of the corporate limits of the Village of Hales Corners. Map 20, located in the back pocket of this report, graphically summarizes the recommended plan.

WATER QUALITY IMPROVEMENT

The recommended plan may be expected to have water quality benefits as a result of the detention of stormwater runoff due to the settling of particulate pollutants such as biochemical oxygen-demanding organic materials, nutrients, and toxic metals, including lead. Thus, the inclusion of detention facilities in the recommended stormwater management plan is consistent with, and serves to advance implementation of, the regional water quality management plan prepared and adopted by the Regional Planning Commission, and will help in achieving adopted water use objectives and supporting water quality standards in the stream system. In addition, implementation of a village erosion control program would further assist in improving water quality conditions. Upon request, the Commission would assist the Village in the draft of a construction erosion control ordinance.

COSTS

The capital and annual operation and maintenance costs of the minor and major system components of the recommended plan are set forth in Table 45. The total capital cost of the recommended improvements is approximately $2.41 million, with a total annual increase in operation and maintenance costs of about $19,700.

This estimated increase in operation and maintenance cost may be compared to the portion of the village public works annual budget allocated to stormwater drainage, as well as to streets, landscaping, winter operations, and lighting, of about $390,000 in 1985. The increase may be expected to be phased in over the plan implementation period as new facilities are constructed. Assuming the capital costs entailed would be spread over a 20-year plan implementation period, the plan implementation cost, including operation and maintenance costs, may be expected to approximate $132,000 per year.

IMPACTS OF RECOMMENDED STORMWATER MANAGEMENT PLAN

Under the recommended stormwater management plan, stormwater runoff from a 10-year recurrence interval storm event, or smaller, will be safely and efficiently stored and conveyed by the minor drainage system to major natural drainage channels with minimal inconvenience to residents. Storm flows from larger events up to and including the 100-year recurrence interval event will be transported by the major drainage system without substantial property damage or danger to human health or safety. In some localized areas, ponding and flooding may occur during a major storm event. However, it was determined that the expected ponding and flooding should not cause major property damage, nor should it endanger human health or safety. Careful consideration was given in the plan to the impacts of the recommended plan downstream of the village area, and implementation of the plan would not exacerbate downstream problems.

Table 45

VILLAGE OF HALES CORNERS STORMWATER MANAGEMENT PLAN COST SUMMARY

System	Total Capital Cost		Cost per Acre		Cost Per Capita	
	Public Sector	Private Sector	Public Sector	Private Sector	Public Sector	Private Sector
Minor	$1,999,500	$163,500	$ 964	$ 79	$ 245	$ 20
Major	250,000	0	120	0	31	0
Total	$2,249,500	$163,500	$1,084	$ 79	$ 276	$ 20

Source: SEWRPC.

Implementation of the recommended stormwater management plan would provide protection against substantial inconvenience to residents during minor storm events, and against major property damage or a significant hazard to human health and safety during major storm events. It would support the continued sound land use development and redevelopment of the Village, enhancing the quality of life within the Village.

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APPENDICES

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Appendix A

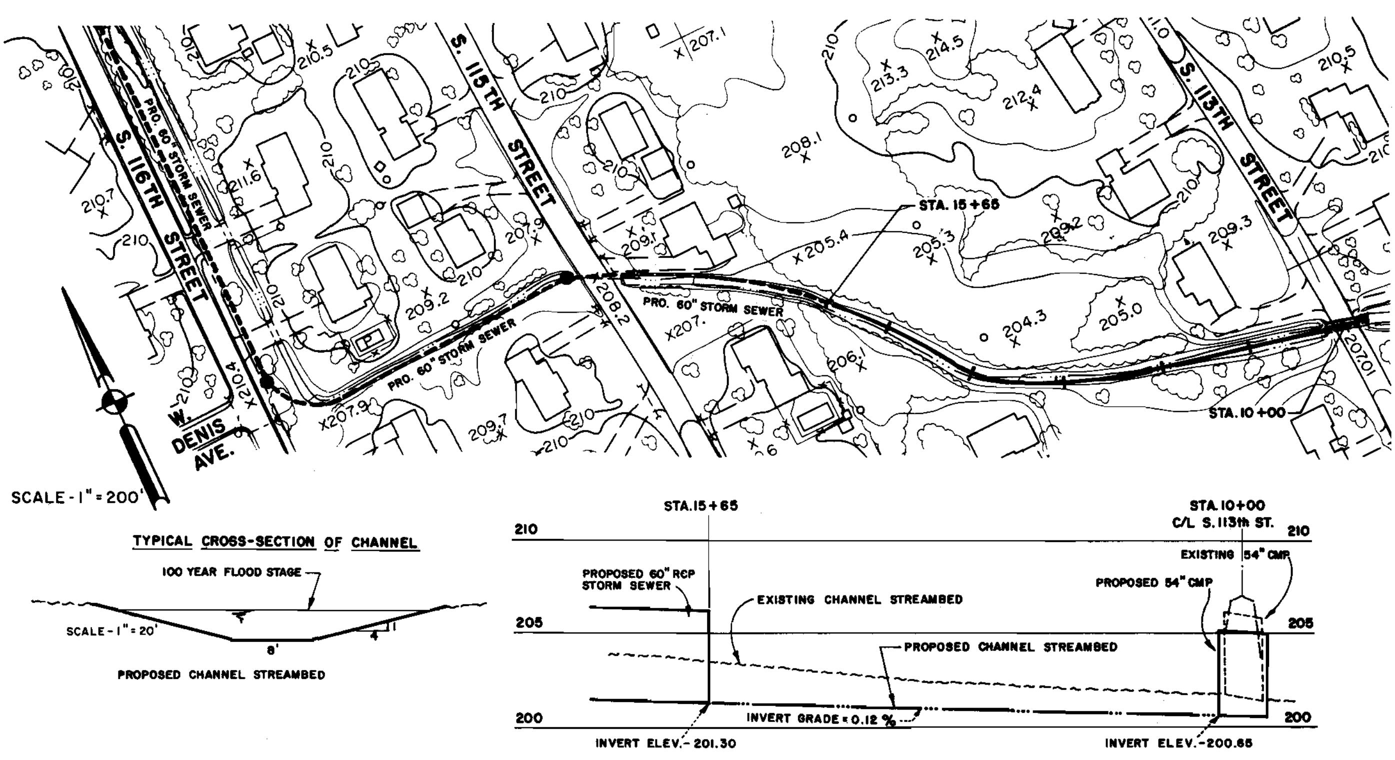
PLANS AND PROFILES OF DRAINAGEWAY AND OPEN CHANNEL COMPONENTS OF THE VILLAGE OF HALES CORNERS STORMWATER MANAGEMENT SYSTEM

This appendix includes a location plan and attendant profile for each of the nine drainageways and open channels which are recommended as components of the stormwater management system for the Village of Hales Corners. The profiles include information on the existing channel bottom, the proposed channel grade and invert elevations, and connecting sewers and culverts. A typical cross-section of the channel is also included. All of the channels included in the plan are recommended to be turf-lined.

Figure A-1

PLAN AND PROFILE OF PROPOSED CHANNEL IMPROVEMENT OF A PORTION OF THE NORTHWEST BRANCH OF WHITNALL PARK CREEK FROM PROPOSED STORM SEWER TO S. 113TH STREET

Hydrologic Unit A, Component A-7

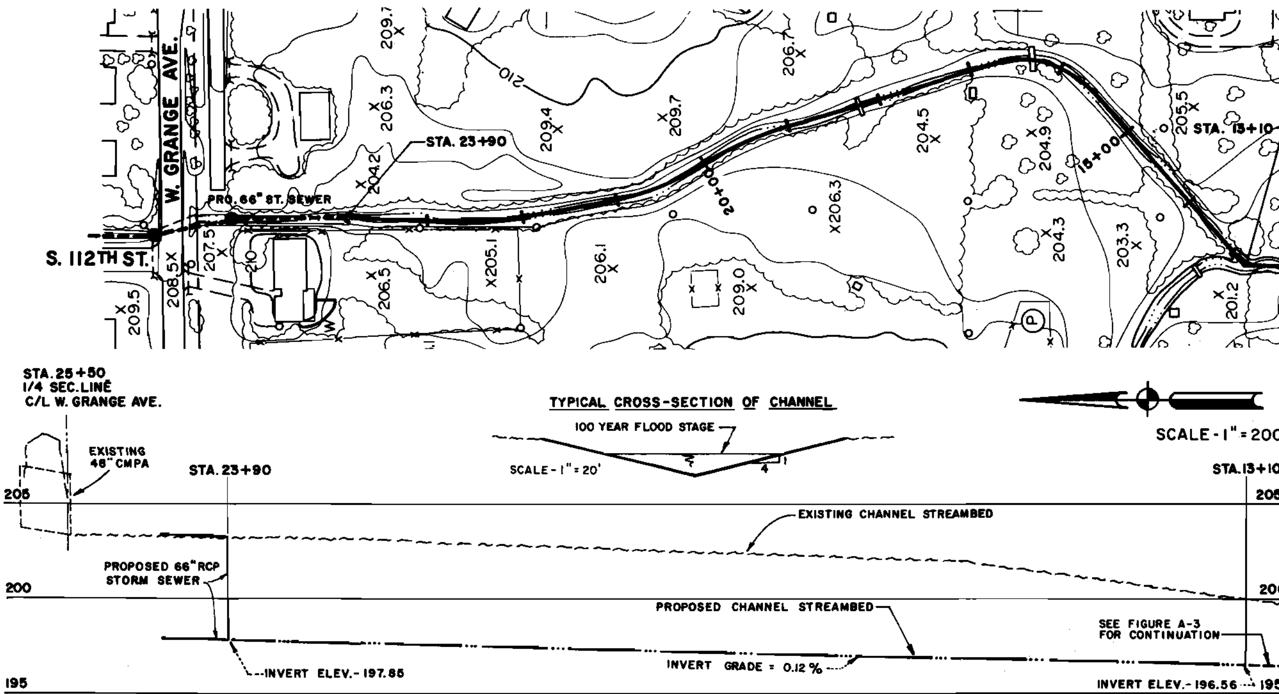


Source: W. G. Nienow Engineering Associates and SEWRPC.

Figure A-2

PLAN AND PROFILE OF PROPOSED CHANNEL IMPROVEMENT OF A PORTION OF THE NORTH BRANCH OF WHITNALL PARK CREEK FROM W. GRANGE AVENUE TO THE CONFLUENCE WITH THE NORTHWEST BRANCH OF WHITNALL PARK CREEK

Hydrologic Unit B, Component B-7A

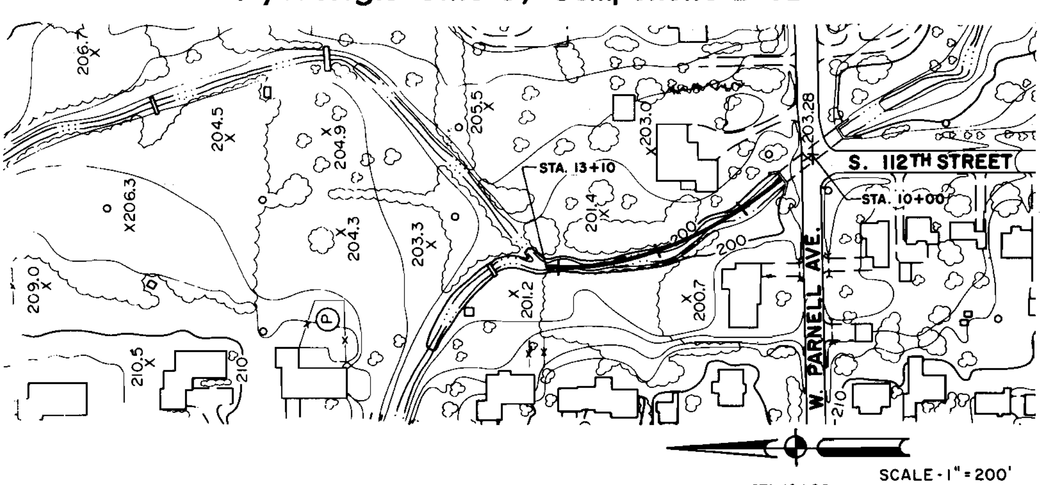


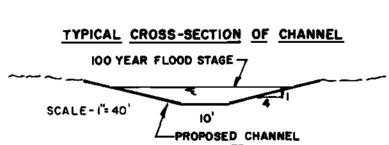
Source: W. G. Nienow Engineering Associates and SEWRPC.

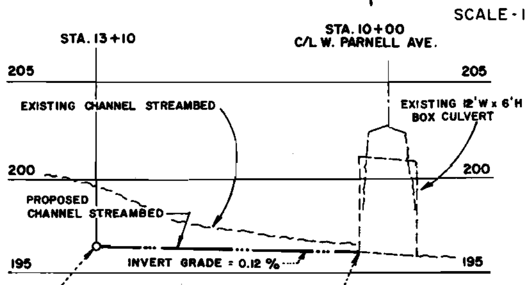
Figure A-3

PLAN AND PROFILE OF PROPOSED CHANNEL IMPROVEMENT OF A PORTION OF THE NORTHWEST BRANCH OF WHITNALL PARK CREEK FROM THE CONFLUENCE WITH THE NORTH BRANCH OF WHITNALL PARK CREEK TO W. PARNELL AVENUE

Hydrologic Unit C, Component B-7B





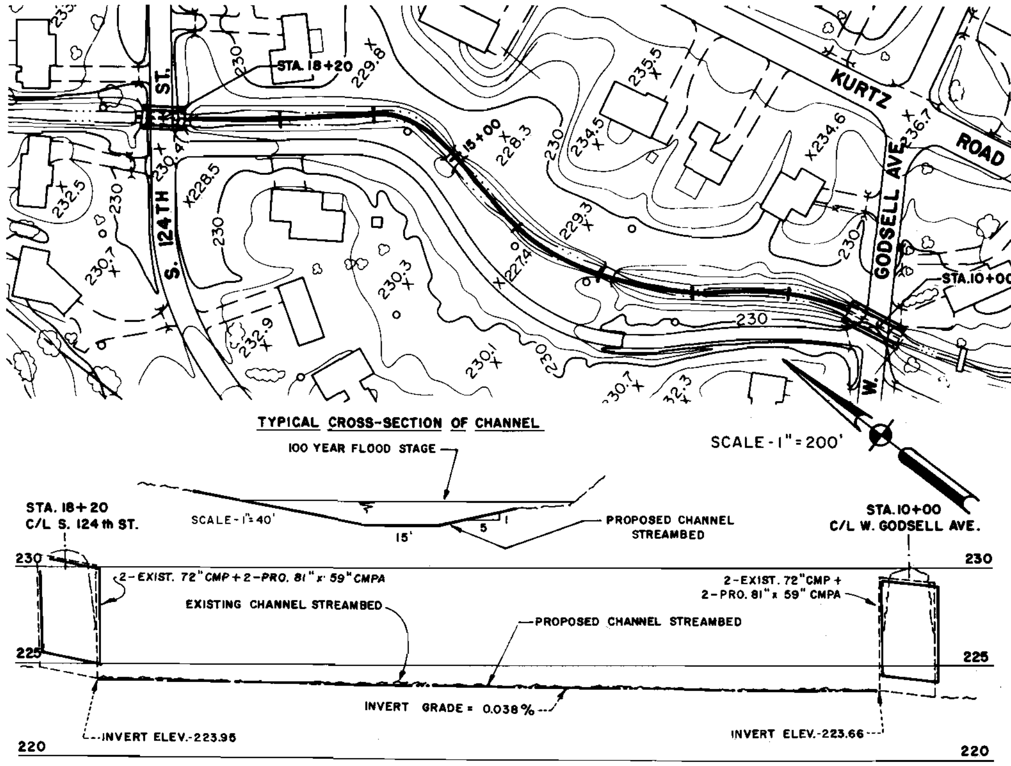


Source: W. G. Nienow Engineering Associates and SEWRPC.

Figure A-4

PLAN AND PROFILE OF PROPOSED CHANNEL IMPROVEMENT OF A PORTION OF WHITNALL PARK CREEK FROM S. 124TH STREET TO W. GODSELL AVENUE

Hydrologic Unit D, Component D-2

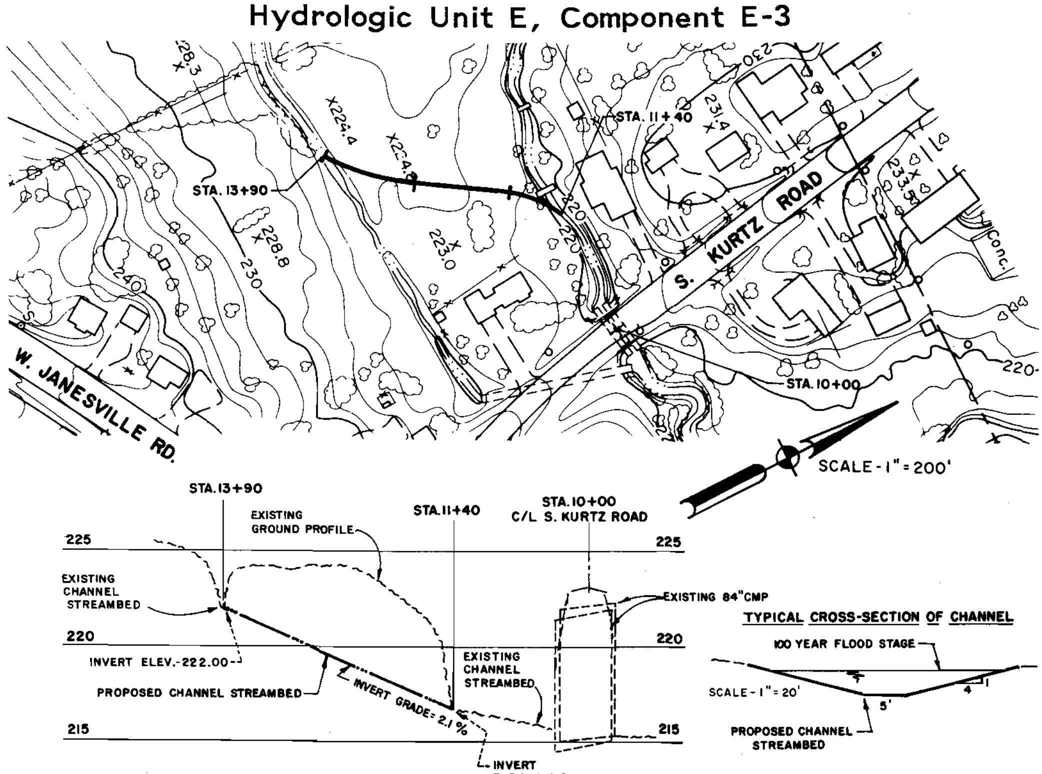


Source: W. G. Nienow Engineering Associates and SEWRPC.

Figure A-5

PLAN AND PROFILE OF PROPOSED CHANNEL FROM AN UNNAMED TRIBUTARY OF WHITNALL PARK CREEK TO WHITNALL PARK CREEK

Hydrologic Unit E, Component E-3

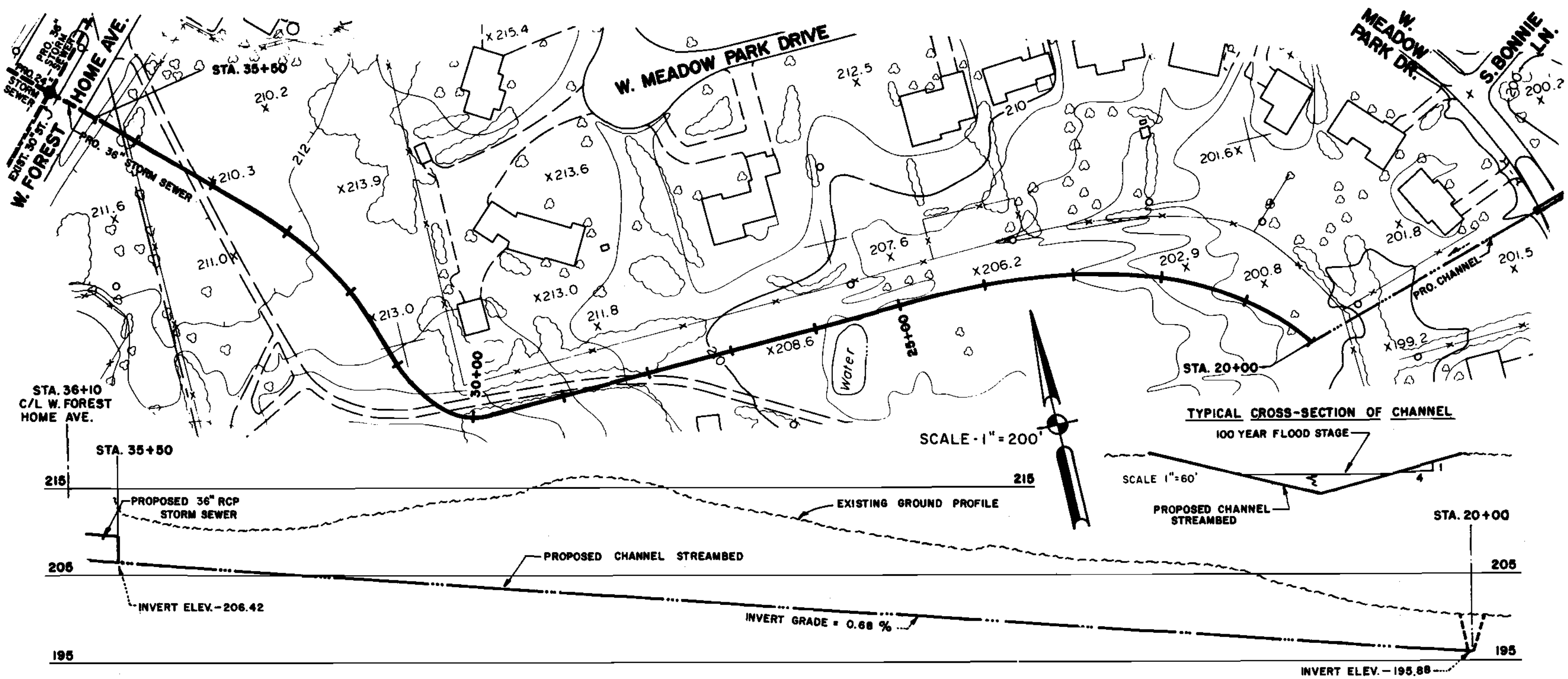


Source: W. G. Nienow Engineering Associates and SEWRPC.

Figure A-6

PLAN AND PROFILE OF PROPOSED CHANNEL FROM W. FOREST HOME AVENUE TO PROPOSED CHANNEL WEST OF W. MEADOW PARK DRIVE

Hydrologic Unit J, Component J-3

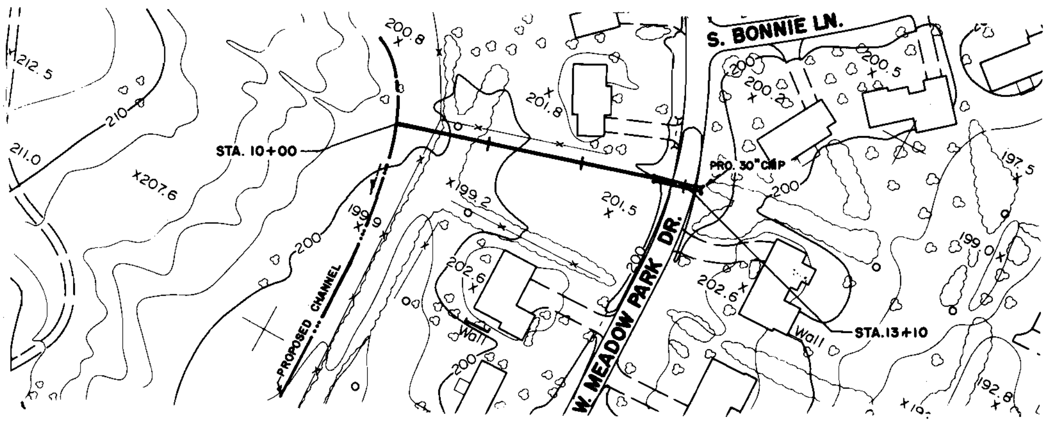


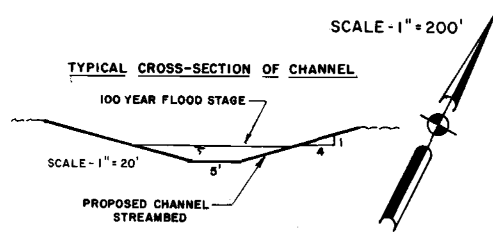
Source: W. G. Nienow Engineering Associates and SEWRPC.

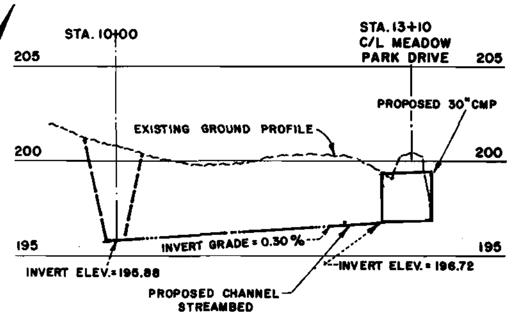
Figure A-7

PLAN AND PROFILE OF PROPOSED CHANNEL IMPROVEMENT FOR A PORTION OF AN UNNAMED TRIBUTARY OF WHITNALL PARK CREEK FROM W. MEADOW PARK DRIVE WEST 300 FEET

Hydrologic Unit L, Component L-1





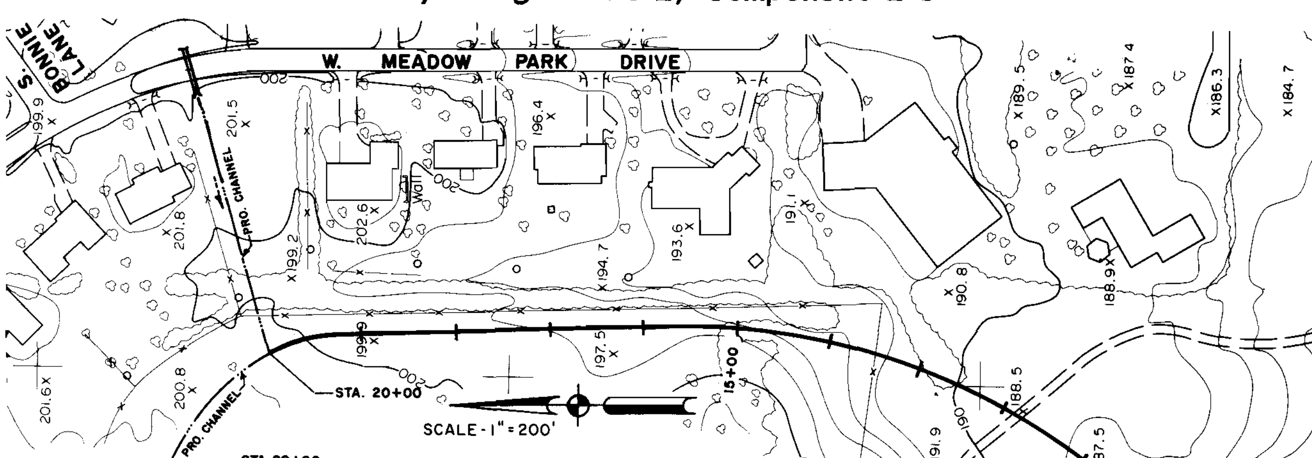


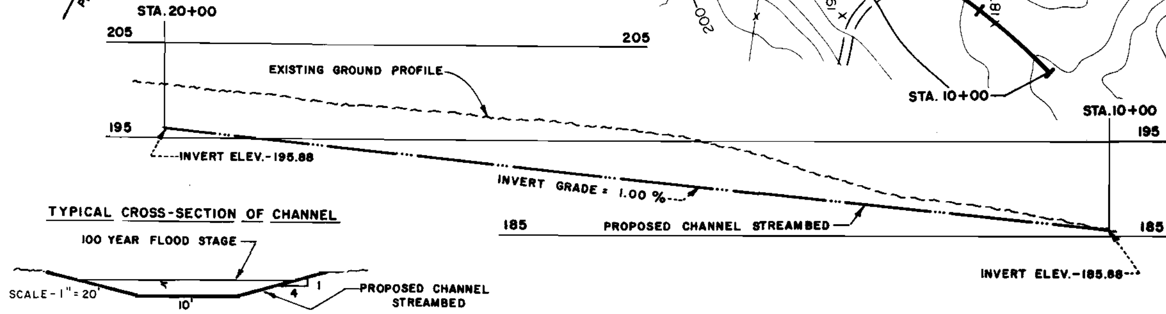
Source: W. G. Nienow Engineering Associates and SEWRPC.

Figure A-8

PLAN AND PROFILE OF PROPOSED CHANNEL FROM PROPOSED CHANNEL WEST OF W. MEADOW PARK DRIVE TO AN UNNAMED TRIBUTARY OF WHITNALL PARK CREEK

Hydrologic Unit L, Component L-3



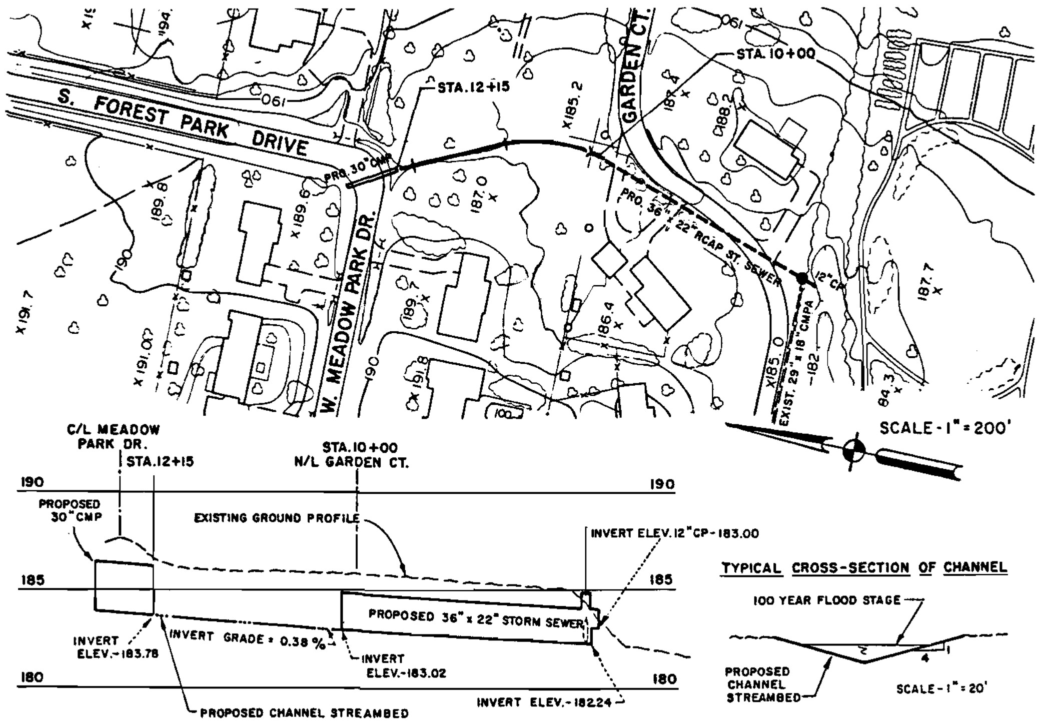


Source: W. G. Nienow Engineering Associates and SEWRPC.

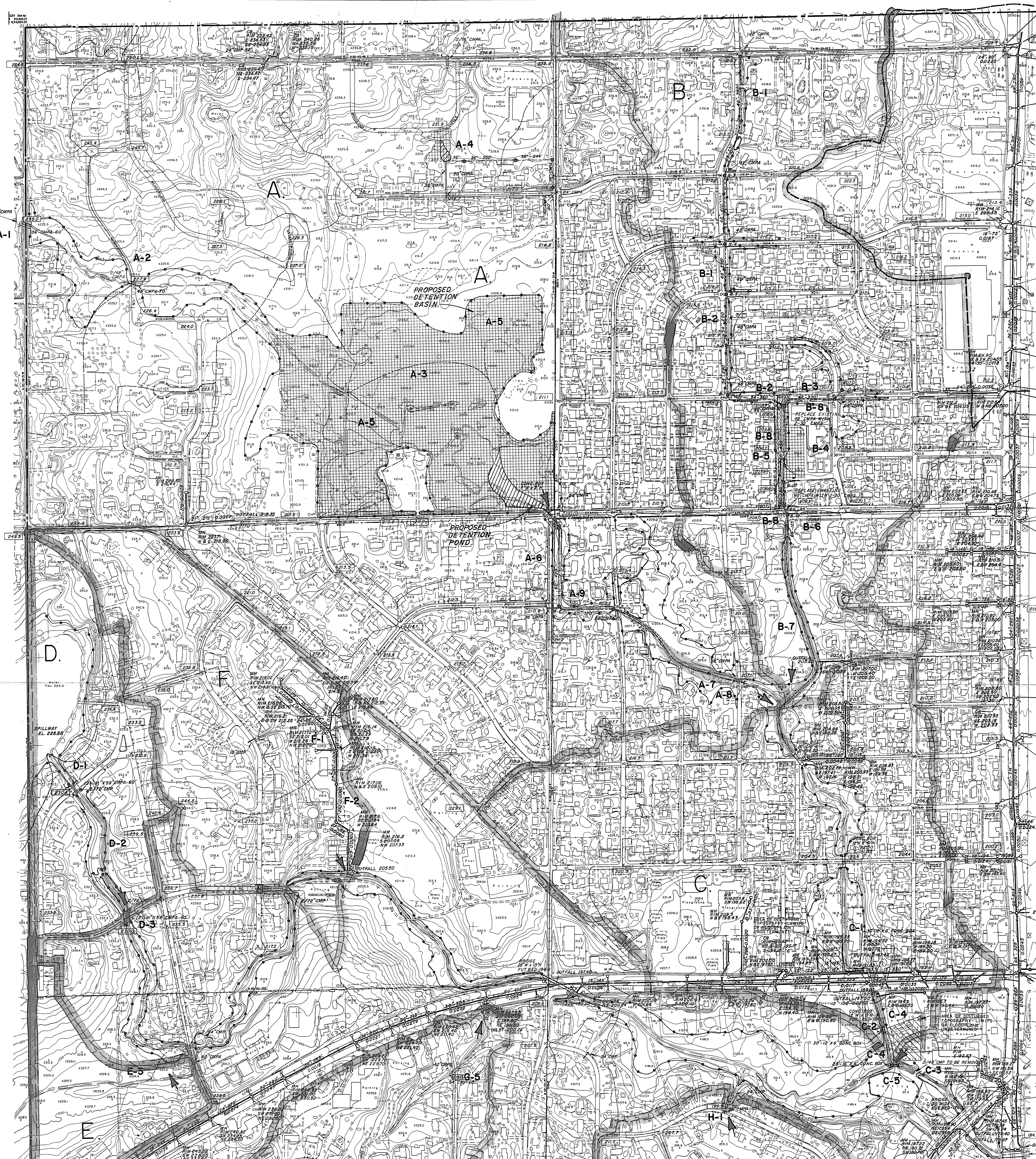
Figure A-9

PLAN AND PROFILE OF PROPOSED CHANNEL FROM W. MEADOW PARK DRIVE AT S. FOREST PARK DRIVE TO A PROPOSED STORM SEWER AT GARDEN COURT

Hydrologic Unit M, Component M-2



Source: W. G. Nienow Engineering Associates and SEWRPC.



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REGIONAL PLANNING COMMISSION
PLANNING LIBRARY

CAPR 121

LEGEND

EXISTING STORM SEWER
PROPOSED STORM SEWER
EXISTING STORM SEWER TO BE REPLACED
EXISTING MANHOLE
PROPOSED MANHOLE
EXISTING CATCH BASIN OR INLET
EXISTING CULVERT
PROPOSED OPEN CHANNEL OR SWALE
PROPOSED CHANNEL IMPROVEMENTS

PROPOSED DETENTION BASIN (DRY)
PROPOSED DETENTION POND (WET)
SUBBASIN BOUNDARY--EXISTING CONDITIONS
HYDROLOGIC UNIT BOUNDARY
A HYDROLOGIC UNIT IDENTIFICATION LETTER
A-3 COMPONENT IDENTIFICATION NUMBER
210.0 PROPOSED CENTERLINE STREET GRADES
PROPOSED ROAD RECONSTRUCTION
PROPOSED CULVERT
100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 LAND USE PLANNED CHANNEL CONDITIONS

HYDROLOGIC UNIT AND MAJOR SUBBASIN DISCHARGE POINT
VILLAGE OF HALES CORNERS CORPORATE LIMITS

A-1	MANHOLE NUMBER
RIM 215.60	MANHOLE ELEVATION
N- 212.89	NORTH INVERT ELEVATION
S- 212.60	SOUTH INVERT ELEVATION

24"- 450'- 0.0020 — SLOPE, SEWER LENGTH, SEWER SIZE.

W. G. NIENOW
ENGINEERING ASSOCIATES

SOUTHEASTERN WISCONSIN REGIONAL PLANNING COMMISSION

MAP 20 A
RECOMMENDED STORMWATER MANAGEMENT SYSTEM PLAN FOR THE VILLAGE OF HALES CORNERS, WESTERN HALF OF VILLAGE
MILWAUKEE COUNTY, WISCONSIN

GEOD NORTH GRID NORTH +01°20'03"
SCALE 1" = 200'
CONTOUR INTERVAL = 2'
200 0 200 400 600
SCALE IN FEET

COMBINATION SCALE AND SEAL LEVEL REDUCTION FACTOR: 0.9999263

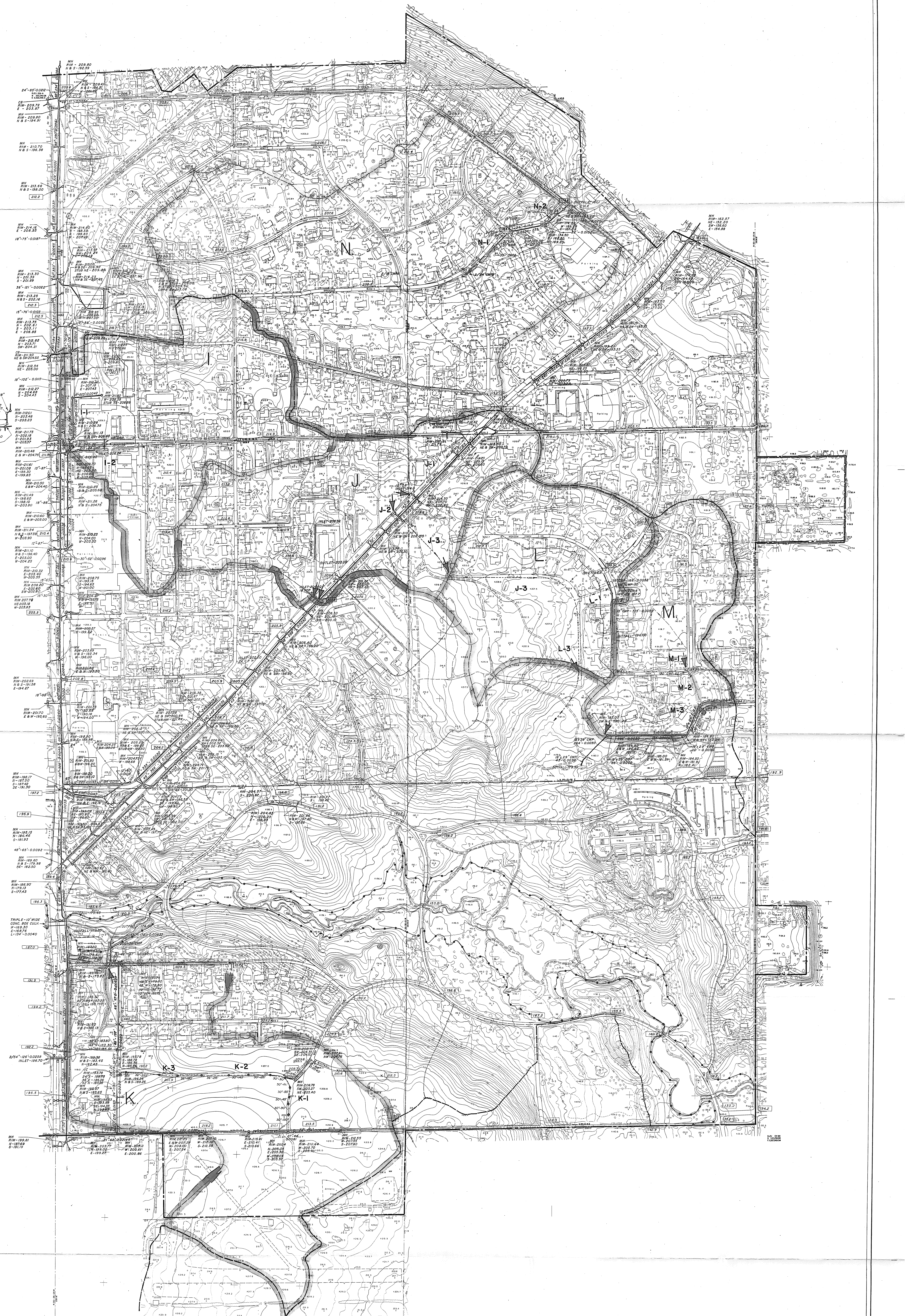
HORIZONTAL DATA IS BASED ON THE METROPOLITAN SEWERAGE COMMISSION DATUM, CONVERTED FROM MEAN SEA LEVEL DATUM, 1929 ADJUSTMENT.

VERTICAL DATUM IS BASED ON THE METROPOLITAN SEWERAGE COMMISSION DATUM, CONVERTED FROM MEAN SEA. LEVEL DATUM, 1929 ADJUSTMENT.

TO CONVERT FROM METROPOLITAN SEWERAGE COMMISSION DATUM TO MEAN SEA LEVEL DATUM ADD 580.603.

COMPILED TO NATIONAL MAP ACCURACY STANDARDS UTILIZING STEREOPHOTOGRAMMETRIC METHODS.

DATE OF PHOTOGRAPHY: APRIL 26, 1975.
DATE OF MAPPING: SPRING 1976.



LEGEND

- EXISTING STORM SEWER
- PROPOSED STORM SEWER
- EXISTING STORM SEWER TO BE REPLACED
- EXISTING MANHOLE
- PROPOSED MANHOLE
- EXISTING CATCH BASIN OR INLET
- EXISTING CULVERT
- PROPOSED OPEN CHANNEL OR SWALE
- PROPOSED CHANNEL IMPROVEMENTS
- PROPOSED DETENTION BASIN (DRY)
- PROPOSED DETENTION POND (WET)
- SUBBASIN BOUNDARY--EXISTING CONDITIONS
- HYDROLOGIC UNIT BOUNDARY
- A — HYDROLOGIC UNIT IDENTIFICATION LETTER
- A-3 — COMPONENT IDENTIFICATION NUMBER
- 210.0 — PROPOSED CENTERLINE STREET GRADES
- ////// PROPOSED ROAD RECONSTRUCTION
- PROPOSED CULVERT
- 100-YEAR RECURRENCE INTERVAL FLOODPLAIN-YEAR 2000 LAND USE PLANNED CHANNEL CONDITIONS
- HYDROLOGIC UNIT AND MAJOR SUBBASIN DISCHARGE POINT
- VILLAGE OF HALES CORNERS CORPORATE LIMITS

A-1	MANHOLE NUMBER
RIM 215.60	MANHOLE ELEVATION
N- 212.89	NORTH INVERT ELEVATION
S- 212.60	SOUTH INVERT ELEVATION

24"-450'-0.0020 — SLOPE; SEWER LENGTH; SEWER SIZE.

W. G. NIENOW
ENGINEERING ASSOCIATES

SOUTHEASTERN WISCONSIN REGIONAL PLANNING COMMISSION

MAP 20 B
RECOMMENDED STORMWATER MANAGEMENT SYSTEM PLAN FOR THE VILLAGE OF HALES CORNERS, EASTERN HALF OF VILLAGE
MILWAUKEE COUNTY, WISCONSIN

GEOD NORTH GRID NORTH
01°20'52"
SCALE 1" = 200'
CONTOUR INTERVAL = 2'
200 0 200 400 600
SCALE IN FEET

TO CONVERT FROM METROPOLITAN SEWERAGE COMMISSION DATUM TO MEAN SEA LEVEL DATUM ADD 580.603.

COMPILED TO NATIONAL MAP ACCURACY STANDARDS UTILIZING STEREOPHOTOGRAMMETRIC METHODS.

DATE OF PHOTOGRAPHY: APRIL 26, 1975.
DATE OF MAPPING: SPRING 1976.

COMBINATION SCALE AND SEAL LEVEL REDUCTION FACTOR: 0.9999271

HORIZONTAL DATA IS BASED ON THE METROPOLITAN SEWERAGE COMMISSION DATUM, CONVERTED FROM MEAN SEA LEVEL DATUM, 1929 ADJUSTMENT.

VERTICAL DATUM IS BASED ON THE METROPOLITAN SEWERAGE COMMISSION DATUM, CONVERTED FROM MEAN SEA LEVEL DATUM, 1929 ADJUSTMENT.