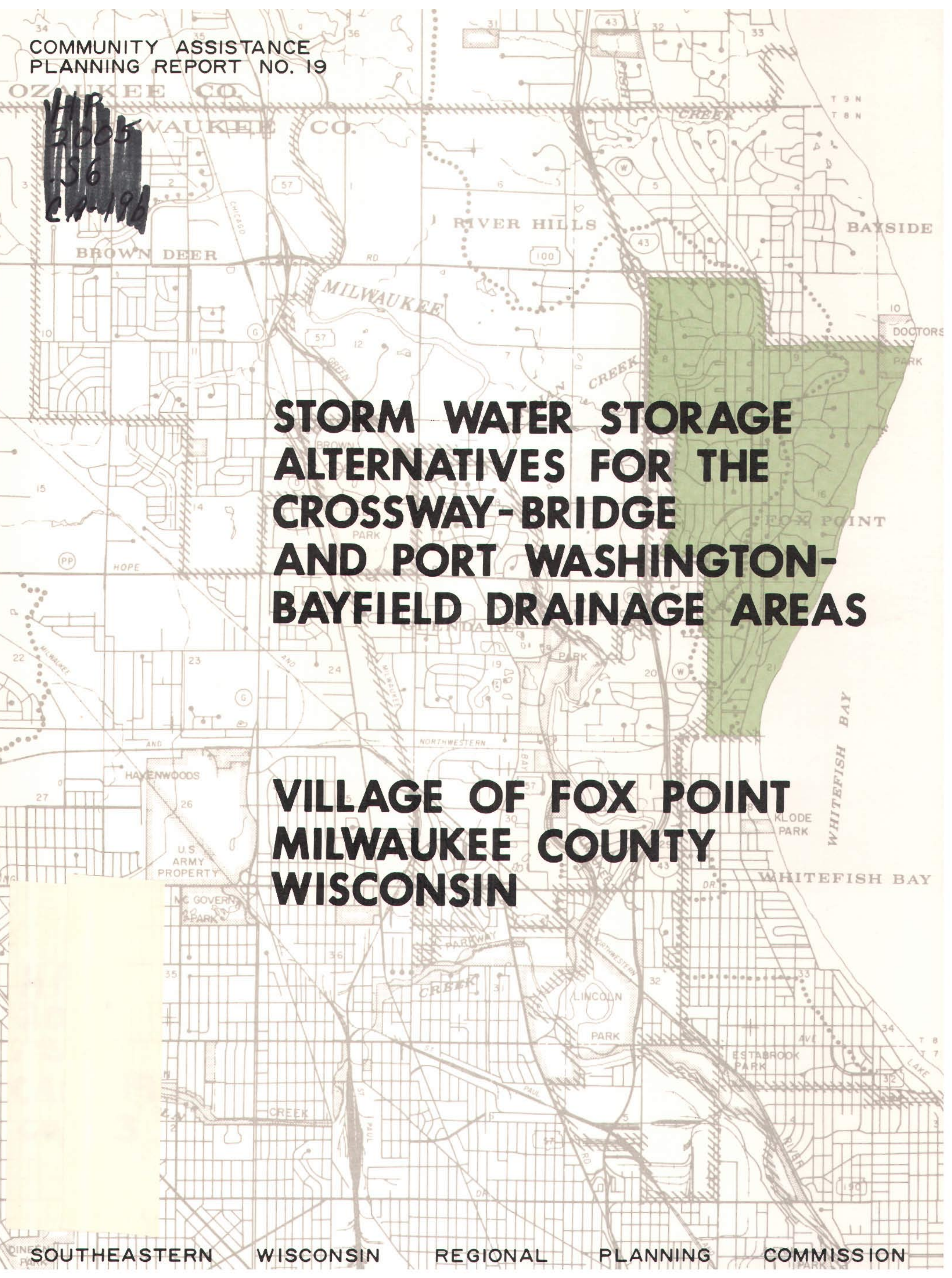
COMMUNITY ASSISTANCE
PLANNING REPORT NO. 19

STORM WATER STORAGE ALTERNATIVES FOR THE CROSSWAY-BRIDGE AND PORT WASHINGTON-BAYFIELD DRAINAGE AREAS

VILLAGE OF FOX POINT
MILWAUKEE COUNTY
WISCONSIN



SOUTHEASTERN WISCONSIN REGIONAL PLANNING COMMISSION

Special acknowledgement is due Mr. Stuart G. Walesh, P.E., SEWRPC Water Resources Engineer and Mr. Gary E. Raasch, SEWRPC Associate Engineer, for their efforts in the conduct of this study and in the preparation of this report.

COMMUNITY ASSISTANCE PLANNING REPORT
NUMBER 19

STORM WATER STORAGE ALTERNATIVES FOR THE CROSSWAY-BRIDGE AND PORT WASHINGTON-BAYFIELD DRAINAGE AREAS IN THE VILLAGE OF FOX POINT

Prepared for the
Village of Fox Point
Milwaukee County, Wisconsin

by the
Southeastern Wisconsin Regional Planning Commission
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August 1977

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SOUTHEASTERN WISCONSIN REGIONAL PLANNING COMMISSION

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August 31, 1977

Mr. W. J. Blong
Village Manager
Village of Fox Point
7200 N. Santa Monica Blvd.
Fox Point, Wisconsin 53217

Dear Mr. Blong:

On April 4, 1977, the Village of Fox Point requested the Regional Planning Commission staff to undertake a study of the use of storm water storage as one alternative to the storm water inundation problems currently being experienced in the Crossway-Bridge and Port Washington-Bayfield drainage areas lying in parts of the Village of Fox Point and the City of Glendale. The Regional Planning Commission staff has now completed that study and is pleased to hereby transmit its findings as documented in the enclosed report entitled "Storm Water Storage Alternative for the Crossway-Bridge and Port Washington-Bayfield Drainage Areas in the Village of Fox Point."

The storage-oriented storm water control alternatives presented in this report are intended to facilitate comparison with the three conveyance-oriented storm water control alternatives previously developed by the Village staff. These alternatives are: 1) a storm sewer to Lake Michigan plus supplemental local sewers, 2) a bored sewer to Glendale on the alignment of an existing sewer plus a tunnel and local sewers discharging to Lake Michigan, and 3) a sewer constructed in a trench to Glendale on the above existing alignment plus a tunnel and local sewers to Lake Michigan. The Village engineering staff concluded, and the Commission staff concurs, that the most favorable of the three conveyance-oriented alternatives on the basis of technical, economic, and nontechnical and noneconomic considerations is the sewer to Lake Michigan with supplemental local sewers.

The Commission staff completed an analysis of a storage-oriented solution that would consist of two principal components: 1) a 24 acre-foot underground concrete reservoir located beneath the Village ice rink and provided with a pump to safely empty the reservoir and 2) a supplemental system of concrete sewer intended to convey storm water runoff from that portion of the Crossway-Bridge drainage area west of approximately N. Santa Monica Blvd. to the underground reservoir. The reservoir would be empty before the beginning of the runoff event, would gradually fill during and immediately after such an event, and the pump would be used to evacuate the reservoir subsequent to the event.

Although the level of storm water control provided by these storage-oriented alternatives would be similar to that provided by the most favorable conveyance-oriented alternative, the latter is more cost-effective. Therefore, it is concluded that of the four basic alternatives available to the Village of Fox Point for controlling storm water inundation in the Crossway-Bridge and Port Washington-Bayfield drainage areas, the most favorable alternative is the sewer to Lake Michigan supplemented with local sewers.

The principal result of improved storm water control in the Crossway-Bridge and Port Washington-Bayfield drainage areas in the Village of Fox Point would be a reduction in the frequency, depth, and lateral extent of storm water inundation. While improved storm water control may also contribute to solving the sanitary sewer system surcharging and basement flooding problems, the ultimate solution to those problems is eliminating clear water in the sanitary sewer system during wet weather periods.

We trust that you will find the enclosed report useful in your consideration of alternative solutions to the storm water control problems of the Village. The Commission staff stands ready to assist the Village in interpreting the findings and recommendations contained in the enclosed report.

Sincerely,

Kurt W. Bauer
Executive Director

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INTRODUCTION

The purpose of this report, and the supporting inventories and analyses, is to develop and present storage-oriented storm water control alternatives for a portion of the Village of Fox Point, Milwaukee County, Wisconsin. These storage-oriented alternatives are intended to significantly reduce the threat of damaging storm water inundation within the Crossway-Bridge and Port Washington-Bayfield drainage areas in the Village of Fox Point.

More specifically, a systems level analysis—as opposed to a detailed design analysis—was conducted for the purpose of: 1) determining the required volume or volumes of storm water runoff storage; 2) identifying possible locations or sites for such a storage within the problem area; 3) estimating the likely cost of providing a storage-oriented solution to the storm water inundation problems; and 4) presenting the advantages and disadvantages of storm water storage. The analyses were made in sufficient depth to permit sound comparison of the technical, economic, and environmental features of storage-oriented alternatives to the conveyance-oriented alternatives previously developed by the Village engineering staff.

This report was prepared by the Southeastern Wisconsin Regional Planning Commission (SEWRPC) in response to a formal request made by the Village of Fox Point on April 4, 1977. The study was conducted by the Commission from April 1977 through July 1977. The requested assistance was provided at no cost to the Village on the condition that the work could be used as an example in SEWRPC Planning Guide No. 7, Urban Storm Water Management Guide, currently being prepared by the Commission under its areawide water quality management planning program.

Information and materials provided by the Village and used in completing the analyses and preparing this report include:

- A map showing the existing storm drainage system in the study area including storm sewer and drainageway alignments, manhole locations, and invert and flow line elevations, together with the location of drainage divides and spot ground elevations.

- The report entitled "Storm Sewer Feasibility Study for the Port Washington-Bayfield Drainage Area and the Crossway-Bridge Drainage Area" a copy of which is attached as Appendix A. This February 1977 report was prepared by the Village engineering staff and includes a description of three conveyance-oriented alternative solutions to the storm water inundation problems occurring in the Port Washington-Bayfield and Crossway-Bridge drainage areas. The report also includes basic data such as delineation of drainage areas, locations of areas inundated by storm water as a result of April 21, 1973 rainfall, and a description of the nature and capacity of the existing storm water control system.

- Sewer construction costs, as a function of sewer diameter, and manhole costs as developed by the Village in the preparation of the above-cited feasibility study. These data were used by the Commission to ensure comparability between the costs of the storage-oriented alternatives prepared by the Commission and the conveyance-oriented alternatives prepared by the Village.

- Rational method calculations used in the preparation of the above-cited feasibility study.

- Memorandum to the Village Board from the Village engineering staff dated May 23, 1975, entitled "Pollution Abatement and Clear Water Control Order No. 4B-70-5-4" and a July 7, 1975, addendum to that memorandum. The memorandum and addendum include a description of historic sanitary sewer backup problems in the Village and a discussion of measures undertaken by the Village since 1952 to reduce the severity of the problem. These remedial actions include: 1) installation of nine sanitary sewer system overflows, all of which have received Wisconsin Department of Natural Resources permits; 2) analysis of the severity and cause of sanitary sewer clearwater problems using television and photographic inspection techniques; 3) enactment of an ordinance by the Village Board prohibiting the connection of storm water drainage devices—for example, downspouts and swales—to the sanitary sewer systems; 4) inspection of sanitary sewer system manholes and sealing of selective manhole covers and frames to prevent inflow of ponded storm water runoff; 5) construction of storm drainage improvements, including installation of short segments of storm sewers, to relieve localized ponding of storm water and paving the inverts of some roadside drainage ditches to provide for conveyance of sump pump discharges; and 6) adoption of an occupancy permit ordinance requiring that, prior to sale of the structure, the structure be certified as being in compliance with Village ordinances and codes which include a stipulation that foundation drains be disconnected from the sanitary sewer system and that a sump pump be installed.

Information and materials provided by the Village were supplemented, as needed, with existing Commission data and information on the natural resource base and man-made features of the Fox Point area including:

- Large-scale (1" = 400') 1975 aerial photographs of the study area;

- Long-term (1940 through 1976) precipitation data from the Milwaukee National Weather Service office. Precipitation recorded at the Milwaukee National Weather Service office—currently located at General Mitchell Field—is not likely to be identical to that which simultaneously occurred in the Village of Fox Point—12 miles north of Mitchell Field. However, the long-term characteristics or the statistical features of precipitation in the Village are likely to be very similar to those of Mitchell Field and, therefore, any statistical analysis of the Milwaukee precipitation data may be considered directly applicable to the Fox Point area; and

- Soils data including interpretations of hydrologic and erosion characteristics of the soils.

Additional data obtained or collated and analyses conducted by the Commission especially for this project include:

- Cost versus volume data for underground concrete storm water storage tanks;

- Cost versus discharge rate data for pumps and controls;

- Statistical analyses of long-term precipitation data for the Milwaukee National Weather Service office; and

- Geologic and groundwater data obtained from U. S. Geological Survey files.

Construction and equipment costs used in this analysis represent 1977 levels. All costs include 10 percent for engineering and administration. Costs of the conveyance-oriented alternatives set forth in the Village report, "Storm Sewer Feasibility Study for the Port Washington-Bayfield Drainage Area and the Crossway-Bridge Drainage Area," reflect 1976 construction costs. These costs have been increased 10 percent to approximate 1977 construction costs and to permit direct comparison with costs of the storage-oriented alternatives described herein. All economic analyses assume an interest rate of 5 percent, which approximates the current cost of money to the Village, and an amortization period of 15 years.[1]

[1] *The Commission normally uses an interest rate of 6 percent and an amortization period of 50 years. The 5 percent interest rate and 15 year amortization period were used by the Commission in preparing this report at the request of the Village.*

TWO BASIC APPROACHES TO URBAN STORM WATER CONTROL

Two fundamentally different approaches to storm water control in urban and urbanizing areas may be identified. Each approach is described below to assist in a fuller understanding of the various storm water control alternatives available to the Village of Fox Point.

Conveyance-Oriented Approach

The first approach to preventing or mitigating urban storm water drainage problems is the traditional and time-tested "conveyance-oriented" storm water control system. The principal function of systems designed in accordance with this approach is to provide for the collection of storm water runoff in the service area followed by the immediate and rapid conveyance of the collected storm water from that area to a point of discharge usually on a natural watercourse 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. The principal components of storm water control systems employing the conveyance-oriented approach are improved open drainage channels and storm sewers. These components are supplemented with appurtenances such as inlets and catch basins, culverts, and energy dissipators.

Storage-Oriented Approach

The second—and to date far less common approach in terms of the extent to which it has been applied in urban storm water control—is the "storage-oriented" system. The function of a storm water control system planned and designed in accordance with this approach is to provide for the temporary storage of storm water runoff within or near the service area for subsequent slow release to downstream channels or storm sewers, thus minimizing disruption and damage both within and downstream of the service area and reducing the required size and therefore cost of any associated downstream conveyance facilities. The principal component of a storm water control system based on the storage-oriented concept is a storage facility. Such a facility may be either a detention or retention facility. A detention facility is normally dry, and is designed to fill only during runoff events. Examples of detention storage include: natural swales provided with crosswise earthern berms as control structures; natural or constructed surface depressions; subsurface storage tanks or reservoirs; and rooftop storage. A local example of a detention reservoir is that located within the area bounded by Parkside Drive, Burlawn Parkway, and Commons Drive in the City of Brookfield.

A retention facility normally contains a substantial volume of water at an established stage which serves recreational, aesthetic, water supply, or other functions, and storm water is stored only above this stage. Examples of retention storage reservoirs include permanent ponds in residential and commercial developments and in public park and open space areas. A local example of a retention reservoir is that located within the Northridge development near the intersection of W. Brown Deer Road and N. 76th Street in the City of Milwaukee. Secondary components and supplementary appurtenances associated with storm water control systems based on the storage-oriented concept include: open channels, storm sewers, inlets and catch basins, culverts, energy dissipators, inlet and outlet control works, and pumping facilities.

Comparison of Features

Table 1 summarizes selected characteristics of the two basic approaches to urban storm water control including: function, principal and secondary components, applicability to existing and newly developing urban areas, downstream impact in terms of both the quality and quantity of storm water runoff, multipurpose capability, operation and maintenance considerations, impact on sanitary sewers, hazards, and hydrologic and hydraulic analysis and design procedures. Advantages of the conveyance-oriented approach to urban storm water control include: applicability to both existing and newly developing urban areas, rapid removal of storm water runoff from the service area, minimal operation and maintenance requirements, and widely accepted analysis and design procedures. Advantages of the storage-oriented approach to urban storm water control include: possible cost reductions in newly developing urban areas, reduction in the downstream quantity and quality impacts of storm water runoff, and potential for multipurpose uses.

THE STORM WATER PROBLEM

The focus of this report is storage-oriented solutions to the storm water inundation problems within the Crossway-Bridge and Port Washington-Bayfield drainage areas in the Village of Fox Point. The location and extent of these two drainage areas are shown on Map 1. The existing storm water control system in these two drainage areas is conveyance-oriented consisting almost entirely of roadside ditches, some of which have paved concrete inverts. The extensive system of roadside ditches is supplemented with short segments of small diameter storm sewer installed to mitigate localized inundation problems.

These two drainage areas have experienced nuisance flooding of street intersections and other low-lying areas for at least a decade. On April 20 and 21, 1973, this area received approximately 4.75 inches of rainfall. As a result of this large quantity of rainfall, which occurred in conjunction with very wet ground conditions, serious storm water flooding occurred throughout the Village, as shown on Map 1, including four locations (areas H, J, K, and L on Map 1) within the Crossway-Bridge and Port Washington-Bayfield drainage areas. Storm water ponded to a depth of 2.5 feet in the intersection of E. Mall and N. Crossway Road (area J on Map 1), flowed onto the adjacent residential property, entered some of the residential structures, and caused substantial damage.

As indicated on Map 1, corrective actions have been taken or are underway for most of the areas prone to storm water inundation located within the Village but outside of the Crossway-Bridge and Port Washington-Bayfield drainage areas. Therefore, the previously completed feasibility study conducted by the Village of Fox Point engineering staff, as well as this supplemental report, are addressed only to the Port Washington-Bayfield and Crossway-Bridge drainage areas.

EXPECTED RESULT OF IMPROVED STORM WATER CONTROL

Improved storm water control is intended first of all to reduce the frequency, depth, and lateral extent of storm water inundation. Such inundation has proven to be disruptive to vehicular and pedestrian traffic and, in some instances, has resulted in basement flooding when ponded storm water entered residential structures via basement windows and other openings. It is technically practicable to improve control over storm water using any of the conveyance-oriented alternatives developed by the Village or one of the storage alternatives described in this report.

A secondary objective of improved storm water control is to reduce the severity of sanitary sewer system surcharge and resulting basement flooding during wet weather periods. As already noted, the Village since 1952, has undertaken a series of studies, regulatory actions, and construction projects directed to reduce inflow and infiltration of clear water into the sanitary sewer system. Improved control of storm water by either a conveyance-oriented or a storage-oriented system may be expected to complement these efforts by further reducing the likelihood of storm water inflow and infiltration. However, improved storm water control should not be expected completely to eliminate the surcharging of sanitary sewers and the resulting backup of sanitary sewage into the basements of residential structures. While improved storm water control will contribute to solution of the surcharging of sanitary sewer systems and the basement flooding problem, the ultimate solution to that problem is elimination of clear water in the sanitary sewer system during wet weather periods regardless of how that clear water enters the sanitary sewers.

CONVEYANCE-ORIENTED ALTERNATIVES

The Village engineering staff completed a systems level analysis of three basic conveyance-oriented solutions to all or part of the storm water inundation problems existing in the Crossway-Bridge and Port Washington-Bayfield drainage areas. Each of these three basic conveyance-oriented solutions has two variations giving rise to a total of six conveyance-oriented alternatives for resolving the storm water inundation problem. The principal features and costs of these six measures are summarized in Table 2, and each of the six alternatives is briefly described below. A more detailed description of the six measures is contained in the February 1977 report prepared by the Village of Fox Point staff and titled, "Storm Sewer Feasibility Study for the Port Washington-Bayfield Drainage Area and the Crossway-Bridge Drainage Area," and included as Appendix A of this report.

Alternative 1A: Sewer (Trench and Tunnel) to Lake Michigan

The intended function of this alternative is to mitigate storm water inundation problems in the N. Crossway

Table 1

SELECTED CHARACTERISTICS OF THE TWO BASIC APPROACHES TO URBAN STORM WATER CONTROL

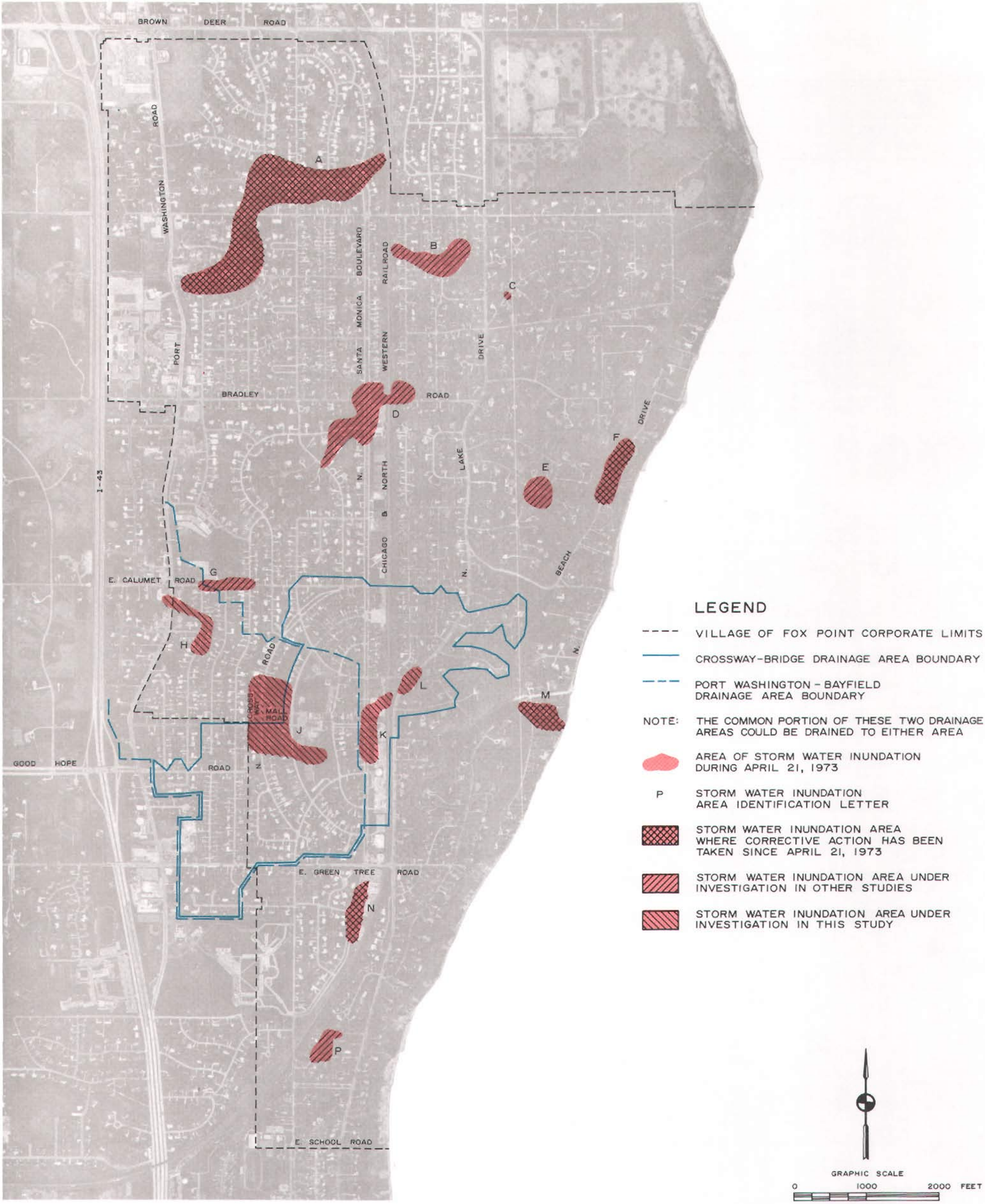
<table>
<tr><th colspan="2" rowspan="2">Characteristic or Feature</th><th colspan="2">Approach</th></tr>
<tr><th>Conveyance-Oriented</th><th>Storage-Oriented</th></tr>
<tr><td colspan="2">Function</td><td>● Provide for the collection of storm water runoff in the service area and the rapid conveyance of storm water from that 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</td><td>● Provide for the temporary storage of storm water 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</td></tr>
<tr><td rowspan="2">Components</td><td>Principal</td><td>● Improved open drainage channels and storm sewers</td><td>● Surface or subsurface detention storage facilities and surface retention storage facilities[a]</td></tr>
<tr><td>Secondary</td><td>● Inlets and catch basins
● Culverts
● Energy dissipators</td><td>● Open drainage channels and storm sewers
● Inlets and catch basins
● Culverts
● Energy dissipators
● Inlet and outlet works and/or pumping facilities</td></tr>
<tr><td colspan="2">Applicability</td><td>● Suitable for installation in existing and in newly developing urban areas</td><td>● Most suitable for incorporation in newly developing urban areas but may be used in existing urban areas if suitable surface or subsurface sites are available</td></tr>
<tr><td rowspan="2">Downstream Impact</td><td>Quantity</td><td>● Tends to significantly increase relative to predevelopment conditions, downstream discharges, stages, and areas of inundation</td><td>● May be designed so as 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.</td></tr>
<tr><td>Quality</td><td>● Transmits suspended material and other potential pollutants to downstream areas</td><td>● Provides for removal, by the natural settling process, of sediment and other suspended material thus reducing the pollution loading on receiving waters. Provides an opportunity for physical-chemical treatment such as disinfection, coagulation-flocculation, and swirl concentration</td></tr>
<tr><td colspan="2">Multipurpose Capability</td><td>● Storm sewers serve only a storm water collection and conveyance function
● Open drainage channels can provide a basis for development of linear park and open space areas</td><td>● Quantity control
● Quality control
● Recreation
● Aesthetic
● Water supply
● Groundwater recharge</td></tr>
<tr><td colspan="2">Operation and Maintenance</td><td>● Minimal—periodic cleaning of catch basins</td><td>● Pumping and/or inlet-outlet control operation and maintenance costs
● Sediment removal
● Weed and insect control</td></tr>
<tr><td colspan="2">Impact on Sanitary Sewer System</td><td>● Surcharging of storm sewers accompanied by inundation of streets and roadways may result in infiltration of storm water from storm sewers to adjacent sanitary sewers and inflow of storm water into sanitary sewers through manholes. Flow in excess of storm water channel capacity may also result in surface inundation and inflow to sanitary sewers</td><td>● 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 and roadways may result in infiltration of storm water from storm sewers to adjacent sanitary sewers and inflow of storm water into sanitary sewers through manholes</td></tr>
<tr><td colspan="2">Hazards</td><td>● Minimal hazard associated with storm sewers
● High velocities in improved open channels may pose safety hazard particularly to children</td><td>● Minimal hazard associated with subsurface and rooftop storage but surface storage, particularly retention basins, may pose a safety hazard particularly to children</td></tr>
<tr><td colspan="2">Hydrologic-Hydraulic Analysis and Design Procedure</td><td>● 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</td><td>● Requires determination of both a peak rate and a volume of inflow associated with a specified recurrence interval and an estimate of allowable outflow rate and design of pumps or control works to satisfy the discharge conditions. Proven and widely accepted analysis and design procedures are not available</td></tr>
</table>

[a] *A detention facility is normally dry, and is designed to fill only during runoff events. Examples of detention storage include: 1) natural swales provided with crosswise earthen berms as control structures; 2) natural or constructed surface depressions; 3) rooftop storage; and 4) constructed subsurface tanks or reservoirs. A retention facility normally contains a significant volume of water to serve recreational and aesthetic functions between runoff events. Examples of retention storage reservoirs include permanent ponds in residential and commercial developments and in public park and open space areas.*

Source: SEWRPC.

Map 1

STORM WATER INUNDATION IN THE VILLAGE OF FOX POINT



Source: Village of Fox Point and SEWRPC.

Table 2

PRINCIPAL FEATURES AND COST OF STORM WATER CONTROL ALTERNATIVES FOR THE CROSSWAY-BRIDGE AND PORT WASHINGTON-BAYFIELD DRAINAGE AREAS IN THE VILLAGE OF FOX POINT

Alternative				Cost (dollars)				Other Considerations		
					Annual					
Number	Name	Intended Storm Water Control Function	Components	Capital Cost	Amortization of Capital Cost	Operation and Maintenance	Total	Positive	Negative	Documentation
1A	Sewer (Trench and Tunnel) to Lake Michigan	Mitigate problem in vicinity of N. Crossway Road, E. Mall and N. Lombardy Road and in the vicinity of Community Place and N. Santa Monica Boulevard in Crossway-Bridge Drainage Area Mitigate Problem in vicinity of W. Bayfield Road and N. Mohawk Road in Port Washington-Bayfield Drainage Area	● 1,500 feet of 48 inch diameter concrete sewer ● 2,600 feet of 54 inch diameter tunnel	1,215,000	117,000	0	117,000	● Minimal disruption—construction on public land or in tunnel	● No assurance of complete drainage relief for areas not near facilities	"Storm Sewer Feasibility Study for the Port Washington-Bayfield Drainage Area and the Crossway-Bridge Drainage Area," Village of Fox Point, Wisconsin February 1977, 21 pp. (see Appendix A of this report)
1B	Sewer (Trench and Tunnel) to Lake Michigan plus Supplemental Local Sewers	Same as 1A plus: Mitigate problems in remainder of Crossway-Bridge Drainage Area Including Glendale portion	Same as 1A plus: ● 12,300 feet of 18 inch to 42 inch local sewer	1,835,000	177,000	0	177,000	Same as 1A plus: ● Provides relief to entire Crossway-Bridge Drainage Area	-	
2A	Sewer (Bored) to Glendale on Existing Alignment	Same as 1A except: No relief in vicinity of Community Place and N. Santa Monica Boulevard	● 2,550 feet of 60 inch bored steel pipe	708,000	68,000	0	68,000	--	● Difficult construction through residential areas ● No drainage relief for areas not near facilities or potential for such relief ● No drainage relief for Glendale or potential for such relief	
2B	Sewer (Bored) to Glendale on Existing Alignment Plus Tunnel and Local Sewers to Lake Michigan	Same as 2A plus: Mitigate problems in Crossway-Bridge Drainage Area east of Santa Monica Boulevard	Same as 2A plus: ● 1,700 feet of 54 inch diameter tunnel ● 5,800 feet of 18 inch to 30 inch local sewer	1,875,000	181,000	0	181,000	-	● Difficult construction through residential areas ● No drainage relief for areas not near facilities or potential for such relief ● No drainage relief for Glendale or potential for such relief	
3A	Sewer (Trench) to Glendale on Existing Alignment	Same as 2A	● 2,550 feet of 60 inch diameter concrete sewer	400,000	38,500	0	38,500	--	Same as 2A	
3B	Sewer (Trench) to Glendale on Existing Alignment Plus Tunnel and Local Sewers to Lake Michigan	Same as 2B	Same as 3A plus: ● 1,700 feet of 54 inch diameter tunnel ● 5,800 feet of 18 inch to 30 inch local sewer	1,570,000	151,000	0	151,000	--	Same as 2B	
4A	Detention Storage Beneath Ice Rink	Same as 1A	● 19 acre-foot underground concrete reservoir equipped with 6,000 gpm pump ● 2,440 feet of 36 inch to 48 inch diameter local sewer	1,725,000	166,000	7,000	173,000	Same as 1A	Same as 1A plus: ● Operation and maintenance requirements	This report
4B	Detention storage beneath ice rink plus supplemental local sewers	Same as 1B except: No relief in the N. Lake Drive-E. Wye Lane-Portage Road area	● 24 acre-foot underground concrete reservoir equipped with a 6,000 gpm pump ● 10,902 feet of 18 inch to 48 inch diameter local sewer	2,470,000	238,000	7,000	245,000	Same as 1A	● Operation and maintenance requirements	This report

Source: Village of Fox Point and SEWRPC.

Road-E. Mall-N. Lombardy Road area (location J on Map 1) and the E. Community Place-N. Santa Monica Boulevard area (location K on Map 1) in the Crossway-Bridge drainage area and in the vicinity of W. Bayfield Road and N. Mohawk Road (location H on Map 1) in the Port Washington-Bayfield drainage area. Principal components of this alternative are 1,500 feet of 48 inch diameter concrete sewer and 2,600 feet of 54 inch diameter tunnel. The estimated capital cost of this alternative is $1,215,000.

Alternative 1B: Sewer (Trench and Tunnel) to Lake Michigan Plus Supplemental Local Sewers

The intended function of this alternative is the same as Alternative 1A plus mitigation of storm water inundation problems in the remainder of the Crossway-Bridge drainage area (locations H, J, K, and L on Map 1) including the City of Glendale portion of the area located immediately southwest of the Village. Principal components of this alternative are 1,500 feet of 48 inch diameter concrete sewer, 2,600 feet of 54 inch diameter tunnel, and 12,300 feet of 18 inch to 42 inch local sewer. The estimated capital cost of this alternative is $1,835,000.

Alternative 2A: Sewer (Bored) to Glendale on Existing Alignment

The intended function of this alternative is the same as Alternative 1A except that no relief would be provided in the vicinity of E. Community Place and N. Santa Monica Boulevard. The principal component of this alternative is 2,550 feet of 60 inch diameter bored steel pipe. The estimated capital cost is $708,000.

Alternative 2B: Storm Sewer (Bored) to Glendale on Existing Alignment Plus Tunnel and Supplemental Local Sewers to Lake Michigan

The intended function of this alternative is the same as Alternative 2A plus mitigation of storm water inundation problems in that portion of the Crossway-Bridge drainage area east of N. Santa Monica Boulevard (locations K and L on Map 1). Principal components of this alternative are 2,550 feet of 60 inch diameter bored steep pipe, 1,700 feet of 54 inch diameter tunnel, and 5,800 feet of 18 inch to 30 inch local sewer. The estimated capital cost of this alternative is $1,875,000.

Alternative 3A: Sewer (Trench) to Glendale on Existing Alignment

The intended function of this alternative is the same as Alternative 2A. The principal component of this alternative is 2,550 feet of 60 inch diameter concrete sewer. The estimated capital cost is $400,000.

Alternative 3B: Storm Sewer (Trench) to Glendale on Existing Alignment Plus Tunnel and Local Sewers to Lake Michigan

The intended function of this alternative is the same as Alternative 2B. Principal components of this alternative are 2,550 feet of 60 inch diameter concrete sewer; 1,700 feet of 54 inch tunnel; and 5,800 feet of 18 inch to 30 inch local sewer. The estimated capital cost of this alternative is $1,570,000.

STORAGE-ORIENTED ALTERNATIVES

The Commission staff developed a basic storage-oriented solution to the storm water inundation problems existing in the Crossway-Bridge and Port Washington-Bayfield drainage areas. This basic storage-oriented solution has two variations which are intended to supplement the six conveyance-oriented alternatives developed by the Village, thereby expanding the type and range of alternatives available to the Village. The storage-oriented alternatives were developed so as to perform storm water control functions similar to those provided by the conveyance-oriented alternatives thereby facilitating a meaningful comparison.

The storage-oriented alternatives were sized to capture and control storm sewer runoff volumes up to and including the quantity that may be expected to be reached or exceeded once on the average of every five years. Stated differently, storage was sized to accommodate a runoff volume that may be expected to be reached or exceeded with a 20 percent probability in any given year. This design criterion was used at the request of the Village engineering staff in order to assure comparability with the five-year recurrence interval design criterion used by the Village staff to develop the conveyance-oriented alternatives. The likely consequences of experiencing rainfall events of such severity as to generate runoff volumes in excess of the design criterion are discussed in the report.

Alternative 4A: Detention Storage Beneath Ice Rink

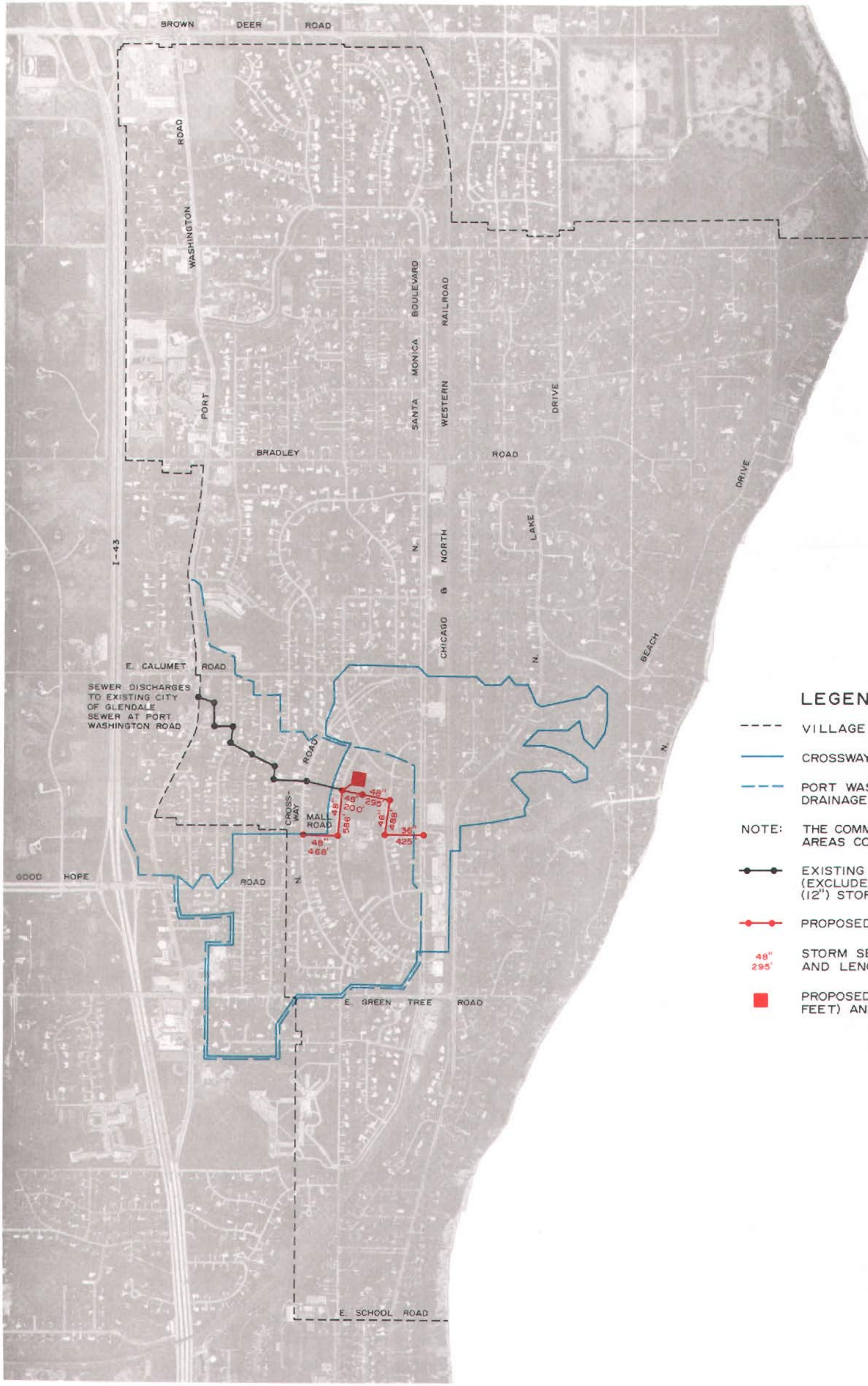
The first storage alternative developed envisions detention storage beneath the Village ice rink located immediately south of N. Bell Road between N. Lombardy Road and N. Longacre Road. This storage alternative is shown on Map 2 and a schedule of its physical features and costs is set forth in Table 2. A discussion of this alternative follows and addresses the following topics: principal components, intended storm water control function, the volume of storage required, location of storage facility, physical features of the storage reservoir, existing surface drainage and proposed storm sewers, operation of the surface drainage system during and after a runoff event, consequences of runoff volumes in excess of design capacity, opportunity for control of diffuse source pollution, and capital and operation and maintenance costs.

Principal Components: This alternative consists of two principal components: 1) a 19 acre-feet underground concrete reservoir located beneath the Village ice rink and provided with a 6,000 gallon per minute pump to evacuate the reservoir by pumping to an existing storm sewer after a storm water runoff event and 2) about 2,440 feet of 36 inch to 48 inch diameter concrete sewer intended to convey storm water runoff from the N. Crossway Road-E. Mall-N. Lombardy Road area to the underground reservoir and from the E. Community Place-N. Santa Monica Boulevard area to the underground reservoir.

Intended Storm Water Control Function: This storage-oriented alternative is intended to provide a level of storm water control identical to that provided by Con-

Map 2

ALTERNATIVE 4A: DETENTION STORAGE BENEATH ICE RINK



LEGEND

VILLAGE OF FOX POINT CORPORATE LIMITS

CROSSWAY-BRIDGE DRAINAGE AREA BOUNDARY

PORT WASHINGTON - BAYFIELD DRAINAGE AREA BOUNDARY

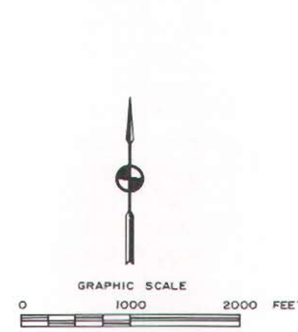
NOTE: THE COMMON PORTION OF THESE TWO DRAINAGE AREAS COULD BE DRAINED TO EITHER AREA

EXISTING MAJOR STORM SEWER AND MANHOLE (EXCLUDES SHORT SEGMENTS OF SMALL (12") STORM SEWERS)

PROPOSED STORM SEWER AND MANHOLE

48" 295' STORM SEWER DIAMETER IN INCHES AND LENGTH IN FEET

PROPOSED UNDERGROUND RESERVOIR (19 ACRE-FEET) AND PUMPING STATION



Source: Village of Fox Point and SEWRPC.

veyance Alternative 1A. More specifically, Alternative 4A is intended primarily to mitigate storm water inundation problems in the N. Crossway Road-E. Mall-N. Lombardy Road area (location J on Map 1) and the E. Community Place-N. Santa Monica Boulevard area (location K on Map 1) within the Crossway-Bridge drainage area, and in the vicinity of W. Bayfield Road and N. Mohawk Road (location H on Map 1) in the Port Washington-Bayfield drainage area. The detention reservoir would mitigate storm water inundation problems in the N. Crossway-E. Mall-N. Lombardy Road area and the E. Community Place-N. Santa Monica Boulevard area by providing for rapid conveyance of storm water runoff from that area and temporary storage in the underground reservoir. By temporarily storing runoff in the underground storage reservoir, relief would be provided to the existing storm sewer discharging from the vicinity of the detention reservoir site in a westerly direction to the City of Glendale, thereby providing storm water inundation relief in the W. Bayfield Road-N. Mohawk Road area.

The detention reservoir is sized to accommodate runoff from the City of Glendale portion of the Crossway-Bridge drainage area. While this may provide some relief to storm water problems in Glendale, the complete control of runoff from Glendale is likely to require construction of a storm sewer from Glendale to the detention reservoir.

Volume of Storage Required: The storage facility was sized to capture and contain the volume of surface runoff expected to be reached or exceeded once on the average of every five years. Stated differently, the storage facility was sized so that there is only 20 percent probability in any given year that runoff in excess of the capacity of the facility would occur and that, therefore, the capacity of the facility would be exceeded. The procedure used to determine the required storage volume is described in Appendix B of this report. The five-year recurrence interval direct runoff—surface runoff and interflow—from the 155 acre drainage area is estimated to be 1.5 inches for a total volume of 19.0 acre-feet or 6.2 million gallons. A storage facility of this size could capture and contain the direct runoff from about 99.5 percent of the precipitation events occurring on the tributary area. The 155 acre drainage area consists of that portion of the Crossway-Bridge drainage area lying west of N. Santa Monica Boulevard and southwest of N. Bell Road. About 101 acres, or 65 percent of the 155 acre tributary area, lies within the Village of Fox Point and the remaining 54 acres, or 35 percent, is contained within that portion of the City of Glendale lying immediately southwest of the Village.

Location of Storage Facility: Inasmuch as the Crossway-Bridge and Port Washington-Bayfield drainage areas in the Village of Fox Point are long established residential neighborhoods, very little open space remains that is suitable for construction of surface or subsurface storm water detention or retention facilities. The little space that is available consists of: 1) the school district property located between N. Lombardy Road and N. Longacre Road at E. Community Place and E. Mall Road extended; 2) the Village parkland immediately to the south containing four tennis courts and a parking lot; and 3) the Village parkland situated immediately to the north containing a recreation building and an outdoor ice rink site.

The third or "ice rink" site was selected for a detention reservoir for three reasons. First, this site is closest to the existing storm sewer that conveys storm water runoff from the upper portion of the Crossway-Bridge drainage area to the west, eventually discharging into the City of Glendale, and that could be used to safely evacuate storm water from the reservoir after a rainfall event at a rate equal to one-half of the hydraulic capacity of the storm sewer. Second, the site is in Village ownership, as opposed to school district or private ownership, thereby facilitating use of the site for a Village project. Finally, neither construction nor operation of an underground storage reservoir would interfere with use of the site assuming that such construction is not carried out during the December to February ice skating period. In contrast, construction of an underground storage facility on Village property south of the school district property could interfere with use of the tennis courts and require reconstruction of the courts, whereas construction in the school site could interfere with recreational and other activities.

A subsurface or underground storage facility was used rather than a surface detention or retention facility for two reasons. First, as described above, little open space remains within the Crossway-Bridge and Port Washington-Bayfield drainage areas and that which does exist and is within Village ownership is actively used for recreational purposes. Second, there is very little relief within the drainage areas; that is, the area is rather flat and without low-lying open areas that could be used for gravity fed storm water detention or retention reservoirs. The subsurface facility provides for the positive drainage of storm sewers and surface drainage swales.

Physical Features of the Storage Facility: The underground reservoir would be constructed of reinforced concrete and be either rectangular or circular in plan—a rectangular configuration was assumed for illustrative purposes on Map 2. The reservoir would have a plan area of about one acre—43,560 square feet—and a height or vertical dimension of about 19 feet. If constructed as a square in plan, the reservoir would have dimensions of 210 by 210 feet, whereas a circular reservoir would have a diameter of 235 feet. The roof of the reservoir would be located at an elevation of about 90 feet above Village of Fox Point Datum, or 670.6 feet above National Geodetic Vertical Datum (Mean Sea Level Datum). Thus the roof of the reservoir would be about 20 feet below existing ground grade, a differential intended to provide for gravity drainage from both roadside swales and storm sewers into the reservoir and to provide a low discharge point for a possible future gravity drainage from storm sewer extensions. Access to the reservoir would be provided by a five foot diameter, 20 foot long vertical shaft extending from the ground surface down to the roof of the reservoir.

The underground reservoir would be equipped with a 6,000 gpm (13 cfs) submersible electrically-driven pump. This pump would discharge to the existing storm sewer which flows to the west into the City of Glendale and could completely evacuate the full reservoir in 18 hours while discharging at 13 cfs which is about one-half the capacity of the existing storm sewer, thereby assuring that no storm sewer surcharge or surface inundation problems would occur along the route of the sewer and receiving stream either in the Village of Fox Point or the City of Glendale.

Geologic and groundwater conditions at the site do not appear to pose any serious problems for construction and operation of an underground storm water storage facility, based on information provided by the U. S. Geological Survey.[2] The dolomite bedrock is at least 100 feet below the land surface and the groundwater table is slightly above the bedrock surface. Both the bedrock surface and the normal water table would lie about 60 feet below the reservoir floor although seasonal groundwater levels may rise temporarily to within 30 feet of the ground surface. Therefore, the underground reservoir would be constructed entirely within easily excavated unconsolidated material—sand, clay, gravel, and rocks. Furthermore, because the groundwater table is normally well below the reservoir floor, minimal infiltration problems would be expected both during and after construction.

Existing Surface Drainage and Proposed Storm Sewers: Storm water runoff from the tributary area would be carried to the underground storage reservoir primarily by the existing drainage system consisting primarily of roadside ditches with concrete inverts. This existing surface drainage system would be supplemented with about 1,240 feet of 48 inch diameter concrete sewer which would be constructed from the intersection of N. Crossway Road and E. Mall Road and would terminate at the underground detention reservoir thereby providing for a positive and effective drainage of the problem area in the vicinity of N. Crossway Road, E. Mall Road, and N. Lombardy Road. About 1,200 feet of 36 inch to 48 inch diameter concrete sewer would be constructed from the intersection of N. Santa Monica Boulevard and E. Community Place and would terminate at the subsurface storage reservoir, thereby providing for positive and effective drainage of the problem area in the vicinity of N. Santa Monica Boulevard and E. Community Place. The rational method, with a runoff coefficient of 0.30, was used to size the storm sewers so that they would convey the discharge produced by a five-year recurrence interval rainfall event.

Operation of the Storage System During and After a Runoff Event: As precipitation begins to fall on a tributary area, surface water runoff would be conveyed by drainage swales and the new storm sewers to the underground detention reservoir. The reservoir, which would normally be empty before the beginning of the runoff event, would gradually fill during and immediately after the cessation of the rainfall. Subsequent to the rainfall event and after flow in the existing sewer to the west had subsided, the pump in the detention reservoir would be automatically activated and the pumpout process would begin with the flow being directed to the existing storm sewer for ultimate discharge in the City of Glendale. The full reservoir could be evacuated in 18 hours at the maximum design pumpout rate of 13 cfs which is one-half the capacity of the existing storm sewer. In the event that the flow into the underground reservoir should be of such magnitude as to exceed the capacity of the reservoir, the reservoir would surcharge, water would rise in the access shaft and overflow into the ice rink located immediately above the reservoir.

Consequences of Rainfall-Runoff Volumes in Excess of Design Capacity: The 19 acre-foot underground storage facility is sized to capture and control the volume of surface runoff expected to be reached or exceeded once on the average of every five years. There is, of course, the possibility that severe meteorological events could occur that would generate storm water runoff volumes in excess of those used to size the storage facility. While cost considerations prohibit construction of storm water control facilities to accommodate all possible storm water runoff volumes, it is important to assess the likely impact of storm water runoff volumes greatly in excess of those used to size the storage facility.

Accordingly, a gross analysis was conducted to determine the approximate disposition of storm water runoff generated by rainfall events ranging from a two-year through a 50-year recurrence interval. The analysis was made for existing conditions and for Alternative 4A in order to provide a relative assessment of the system performance capability. The analytic technique described in Appendix B was used to determine the rainfall and runoff volumes corresponding to the specified recurrence intervals.

It was assumed that all storm water runoff would remain within the service area during any of the runoff events; that is, runoff would not be pumped from or otherwise diverted from the service area until the rainfall event had ceased and sufficient capacity was available in existing contiguous storm sewers. The first increment of runoff in excess of the five-year recurrence interval volume of

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[2] *Earl L. Skinner and Ronald G. Borman, "Water Resources of Wisconsin-Lake Michigan Basin," U. S. Geological Survey, Hydrologic Investigations Atlas HA-432, 1973.*

F. C. Foley, W. C. Walton, and W. J. Drescher, "Ground-Water Conditions in the Milwaukee-Waukesha Area, Wisconsin," U. S. Geological Survey Water Supply Paper 1229, 1953.

Unpublished map of the Planning Region entitled "Depth to Seasonal High Water," prepared by the U. S. Geological Survey in January 1977 for the SEWRPC areawide water quality management planning program.

the detention facility—equivalent to 1.5 inches over the service area—would be pumped into the shallow area formed by the earthen berm that has been constructed around the ice rink site and located immediately above the subsurface detention facility. The volume of storage so available on the ice rink is equivalent to approximately 0.2 inch over the service area. Because of the relatively flat land slopes and street and roadside ditch grades in the service area, it was assumed that storm water runoff under existing conditions, as well as storm water runoff in excess of the volume stored in the subsurface detention reservoir and at the ice rink site under Alternative 4A, would be distributed throughout the service area.

Under Alternative 4A, the first increment of storage to be distributed throughout the service area would be that which would fill the short segment of storm sewers included in this alternative. These storm sewers, however, would have a capacity equivalent to only about 0.05 inch of runoff over the service area which is negligible compared to the capacity of other types of storage in the system.

Storm water runoff in excess of that which could be contained in the detention facility, the ice rink, and the proposed sewers would accumulate in roadside ditches. Although land slopes and street grades are relatively flat in the service area, slight topographic irregularities exist and, therefore, the full storage capacity of the ditches and roadways would not be realized under either existing conditions or assumed implementation of Alternative 4A; that is, water would move to low points in the ditches. Therefore, it was conservatively assumed that only half of the available capacity of the roadside ditches would be available for storm water storage. It is estimated that the roadside ditches could accommodate storm water runoff equivalent to 0.3 inch over the surface area under either existing or detention storage conditions.

Storm water runoff in excess of that stored in the detention facility, in the ice rink, in the storm sewers, and in the roadside ditches would accumulate on the road surface and above the roadside ditches all within the street rights-of-way. It was assumed that a typical street right-of-way could, under either existing conditions or detention storage conditions, accommodate up to one foot of water above the roadway surface without damage to contiguous private property. Again, because of slight topographic irregularities, the full storage capacity of the street rights-of-way would not be available; that is, water would move to low points along the street profile. Therefore, it was assumed that only half of the storage above street rights-of-way would be available and this was estimated to be equivalent to one inch runoff from the service area.

Table 3 sets forth the expected rainfall and runoff volumes, expressed in inches over the service area, for two-, five-, 10-, 25-, and 50-year recurrence interval rainfall-runoff events. Based on the above assumptions and analyses, Table 3 also indicates the disposition of storm water runoff in the service area under existing conditions and assuming implementation of Alternative 4A. Thus, for each recurrence interval, the table sets forth the estimated volume of storm water runoff that would accumulate in the proposed detention reservoir, in the proposed sewers, on the ice rink, in roadside ditches, and on street rights-of-way. Finally, Table 3 indicates the volume of storm water runoff in excess of the above storage capacities. That excess is an index to the severity of storm water inundation problems for existing and detention storage systems under a range of rainfall-runoff events.

Several conclusions may be drawn concerning the performance of Alternative 4A under existing conditions. First, it is apparent that the volume of storage available in the subsurface facility is large compared to each of the other storage components in the system and that the subsurface detention reservoir volume approximates the combined storage of all other storage locations in the system. That is, the addition of the detention facility approximately doubles the available storm water runoff storage capacity in the service area.

Second, under the existing system, street right-of-way inundation may be expected to occur for rainfall-runoff events as small as the two-year recurrence interval whereas, under Alternative 4A, street inundation would occur only for rainfall-runoff events having a recurrence interval of 10 years or more.

Third, with the existing system, a 50-year recurrence interval rainfall-runoff event would result in a volume exceeding that which could be safely accommodated in roadside ditches and in street rights-of-way of 2.2 inches over the service area. However, with Alternative 4A, this excess volume would be reduced to about 0.5 inches.

Opportunity for Control of Diffuse Source Pollution: Section 208 of the Federal Water Pollution Control Act (P.L. 92-500), as amended by the U. S. Congress in 1972, provides for the development and implementation of areawide water quality planning and management programs within all of the nation's major metropolitan areas. A specific requirement of this Act is assessment of surface water pollution problems attributable to diffuse sources, identification of the nature of those sources, preparation of alternative means of controlling diffuse sources, and development of a strategy for implementing the resulting recommendations. The seven-county Southeastern Wisconsin Region has been designated as an area requiring an areawide water quality planning and management program under the Federal Water Pollution Control Act, and the Regional Planning Commission has been designated as the water quality management planning agency for the area. The Commission undertook the areawide water quality planning and management program on July 1, 1975, and the work is scheduled for completion by the end of 1977. It is anticipated that the resulting water quality management plan will contain an element for eliminating pollution from nonpoint sources, primarily rainfall-runoff from urban and rural lands.

As noted earlier, a positive aspect of the detention-retention storage of storm water is the opportunity to provide treatment of storm water runoff through plain

Table 3

APPROXIMATE DISPOSITION OF STORM WATER RUNOFF IN THE AREAS SERVED BY ALTERNATIVES 4A AND 4B UNDER TWO- TO 50-YEAR RECURRENCE INTERVAL RAINFALL-RUNOFF EVENTS

Recurrence Interval (years)	Water Volume Expressed in Inches Over the Service Area															
	Rainfall	Runoff	Volume Stored in Proposed Detention Reservoir (1.5 inches maximum)		Volume Stored in Proposed Sewers (Negligible for Alternative 4A, 0.1 inch maximum for Alternative 4B)		Volume Stored in Ice Rink (0.2 inch maximum)		Volume Stored in Roadside Ditches (0.3 inch maximum)		Volume Stored on Street Right-of-Way (1.0 inch maximum)		Excess Volume	
			Existing System	With Storage Alternative	Existing System	With Storage Alternative	Existing System	With Storage Alternative	Existing System	With Storage Alternative	Existing System	With Storage Alternative	Existing System	With Storage Alternative
Alternative 4A—Detention Storage Beneath Ice Rink																
2	2.6	0.7	0.0	0.7	0.0	0.0	0.0	0.0	0.3	0.0	0.4	0.0	0.0	0.0
5	3.8	1.5	0.0	1.5	0.0	0.0	0.0	0.0	0.3	0.0	1.0	0.0	0.2	0.0
10	4.9	2.4	0.0	1.5	0.0	0.0	0.0	0.2	0.3	0.3	1.0	0.4	1.1	0.0
25	5.9	3.2	0.0	1.5	0.0	0.0	0.0	0.2	0.3	0.3	1.0	1.0	1.9	0.2
50	6.2	3.5	0.0	1.5	0.0	0.0	0.0	0.2	0.3	0.3	1.0	1.0	2.2	0.5
Alternative 4B—Detention Storage Beneath Ice Rink Plus Supplemental Local Sewers																
2	2.6	0.7	0.0	0.7	0.0	0.0	0.0	0.0	0.3	0.0	0.4	0.0	0.0	0.0
5	3.8	1.5	0.0	1.5	0.0	0.0	0.0	0.0	0.3	0.0	1.0	0.0	0.2	0.0
10	4.9	2.4	0.0	1.5	0.0	0.1	0.0	0.2	0.3	0.3	1.0	0.3	1.1	0.0
25	5.9	3.2	0.0	1.5	0.0	0.1	0.0	0.2	0.3	0.3	1.0	1.0	1.9	0.1
50	6.2	3.5	0.0	1.5	0.0	0.1	0.0	0.2	0.3	0.3	1.0	1.0	2.2	0.4

[a] *Assumes that storm water runoff remains in the service area during each specified rainfall-runoff event.*

Source: SEWRPC.

sedimentation or through other physical or biochemical means prior to its discharge from the storage facility to the surface water system. For example, the temporary storage of storm water runoff in the detention or retention reservoir provides an opportunity for suspended material to settle out carrying with it some of the potential pollutants. A recently completed research investigation concluded that only 15 minutes of quiescent settling of urban land runoff could remove 50 percent of the turbidity, 60 percent of the chemical oxygen demand, and 77 percent of the suspended solids.[3] Storage times in the proposed detention reservoir could exceed this time period for essentially all runoff events. Because of the expected sedimentation effectiveness of the subsurface detention facilities being considered for the Village of Fox Point, cost estimates include allowance for annual removal of accumulated sediment and other solids.

A storage-oriented facility like that envisioned under this alternative is more likely than a conveyance-oriented system to be adaptable to treatment of storm water runoff if and when diffuse source pollution controls are recommended in the areawide water quality planning and management program and subsequently required by the federal or state government. At the present stage of the areawide water quality planning and management program, it appears as though some form of diffuse source pollution control will be required in urban areas like the Village of Fox Point. However, the degree of control required and the best means of achieving such control have not yet been determined.

Capital and Operation and Maintenance Costs: The estimated capital cost of Alternative 4A is $1,725,000, consisting of $1,470,000 for the underground concrete reservoir—which includes $20,000 for the pump—and $255,000 for new storm sewers leading to the reservoir. Using an annual interest rate of 5 percent and amortization period of 15 years, the average annual cost of amortizing the capital expenditure is $166,000.

Automatic operation of the pump used to evacuate the subsurface storage facility after runoff events would require an estimated average annual cost of $2,000 for electrical energy and pump maintenance. Successful

[3] *N. V. Colston, "Characterization and Treatment of Urban Land Runoff," U. S. Environmental Protection Agency, Publication No. 670/2-74-096, December 1974, pp. 65-87.*

functioning of the storage system would depend on periodic, careful inspection and maintenance of the facilities, particularly the pump and automatic controls.

In addition, suspended sediment and other solids carried into and deposited within the reservoir would have to be periodically—probably yearly—removed. An average of about 100 cubic yards of sediment and other suspended material may be expected to be deposited in the storage facility each year. The estimated annual cost for removing and disposing of this sediment, and of inspecting the concrete structure and making minor repairs, is $5,000, bringing the total annual operation and maintenance cost to $7,000.

The total average annual cost of Alternative 4A, computed using an annual interest rate of 5 percent and an amortization period of 15 years, is estimated at $173,000. This cost consists of $166,000 per year for amortization of the $1,725,000 capital cost of the reservoir and storm sewers, $2,000 per year pump operation and maintenance costs, and $5,000 per year for sediment removal and disposal and for structure inspection and repair.

Alternative 4B: Detention Storage Beneath Ice Rink Plus Supplemental Local Sewers

The second storage alternative also envisions detention storage beneath the Village ice rink located immediately south of N. Bell Road between N. Lombardy Road and N. Longacre Road. This storage alternative is shown on Map 3 and a schedule of physical features and costs is set forth in Table 2. A discussion of this alternative follows and addresses the following topics: principal components, intended storm water control function, the volume of storage required, location of storage facility, physical features of the storage reservoir, existing surface drainage and proposed storm sewers, operation of the surface drainage system during and after a runoff event, consequences of runoff volumes in excess of design capacity, opportunity for control of diffuse source pollution, and capital and operation and maintenance costs.

Principal Components: This alternative consists of two principal components: 1) a 24-acre-foot underground concrete reservoir located beneath the Village ice rink and provided with a 6,000 gallon per minute pump to evacuate the reservoir by pumping to an existing storm sewer after a storm water runoff event and 2) about 10,900 feet of 18 inch to 48 inch diameter concrete sewer intended to convey storm water runoff from that portion of the Crossway-Bridge drainage area west of approximately N. Santa Monica Boulevard to the underground reservoir.

Intended Storm Water Control Function: This storage-oriented alternative is intended to provide a level of storm water control identical to that provided by conveyance Alternative 1B except that no storm sewers would be provided in the N. Bridge Lane-E. Coleman Lane-N. Lake Drive-E. Portage Road-N. Boyd Way area. The detention reservoir would mitigate storm water inundation problems in that portion of the Crossway-Bridge drainage area west of approximately N. Santa Monica Boulevard, including the City of Glendale portion, by providing for rapid conveyance of storm water runoff from that area and temporary storage in the underground reservoir. By temporarily storing runoff in the underground reservoir, relief would be provided to the existing storm sewer discharging from the vicinity of the detention reservoir site in a westerly direction to the City of Glendale, thereby providing storm water inundation relief in the W. Bayfield Road-N. Mohawk Road portion of the Port Washington-Bayfield drainage area.

Volume of Storage Required: The storage facility was sized to capture and contain the volume of surface runoff expected to be reached or exceeded once on the average of every five years. The procedure used to determine the required volume is described in Appendix B of this report. The five-year recurrence interval direct runoff—surface runoff and interflow—from the 192-acre drainage area is estimated to be 1.5 inches for a total volume of 24 acre-feet or 7.8 million gallons. A storage facility of this size could capture and contain the direct runoff from about 99.5 percent of the precipitation events occurring on the tributary area. The 192-acre drainage area consists of that portion of the Crossway-Bridge drainage area lying west of approximately N. Santa Monica Boulevard. About 138 acres, or 72 percent of the 192-acre tributary area, lies within the Village of Fox Point and the remaining 54 acres, or 28 percent, is contained within that portion of the City of Glendale lying immediately southwest of the Village.

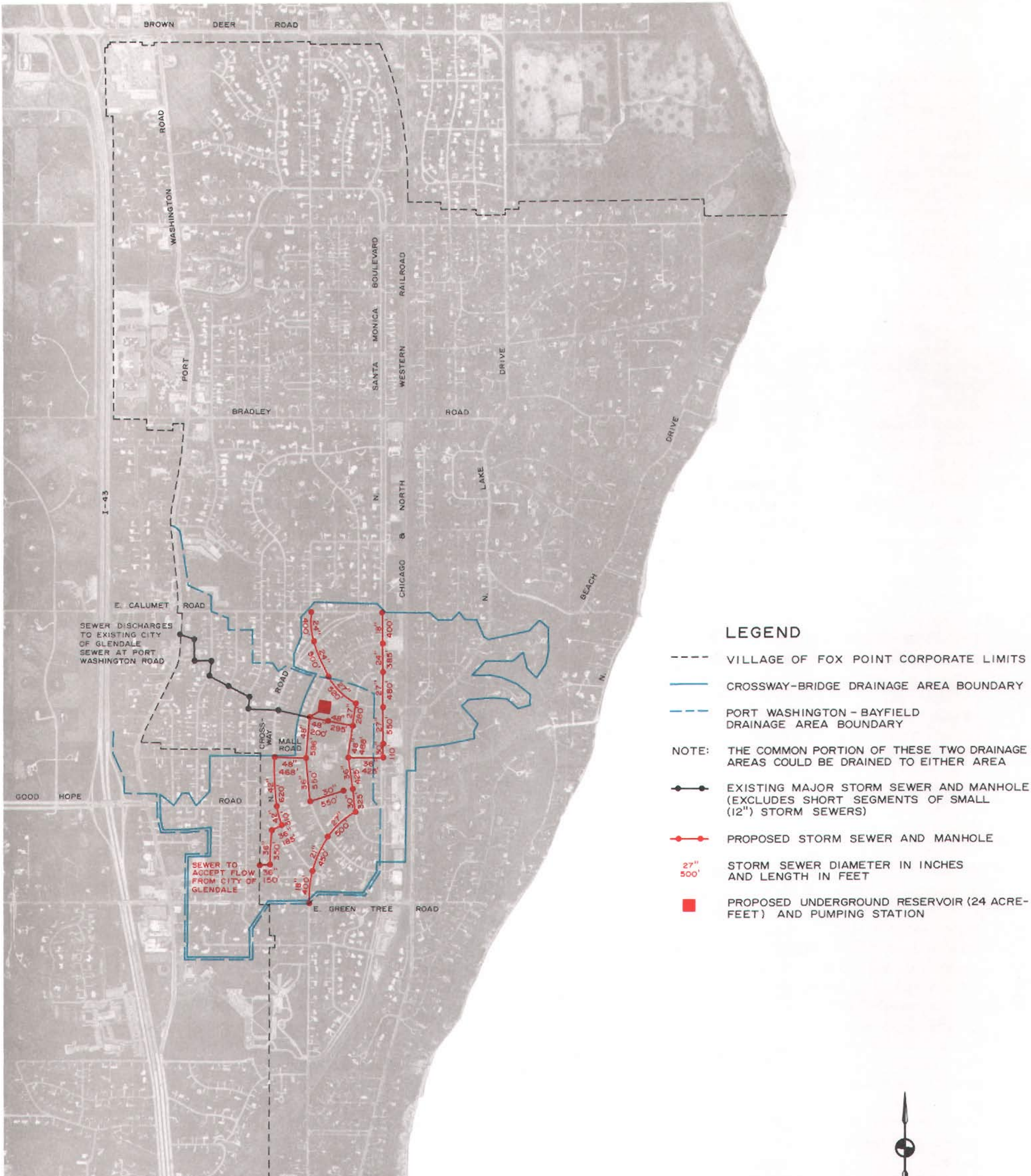
Location of Storage Facility: The Village-owned ice rink site was selected for the detention reservoir for reasons cited in the above discussion of Alternative 4A. Similarly, a subsurface or underground storage facility was selected.

Physical Features of the Storage Facility: The underground reservoir would be constructed of reinforced concrete and be either rectangular or circular in plan—a rectangular configuration was assumed for illustrative purposes on Map 3. The reservoir would have a plan area of about one acre—43,560 square feet—and a height or vertical dimension of about 24 feet. If constructed as a square in plan, the reservoir would have dimensions of 210 by 210 feet whereas a circular reservoir would have a diameter of 235 feet. The roof of the reservoir would be located at an elevation of about 90 feet above Village of Fox Point Datum, or 670.6 feet above National Geodetic Vertical Datum (Mean Sea Level Datum). Thus, the roof of the reservoir would be about 20 feet below existing ground grade, a differential intended to provide for gravity drainage from both roadside swales and storm sewers into the reservoir and to provide a low discharge point for a possible future gravity drainage and storm sewer extension. Access to the reservoir would be provided by a five foot diameter, 20 foot long vertical shaft extending from the ground surface to the roof of the reservoir.

The underground reservoir would be equipped with a 6,000 gallon per minute (13 cfs) submersible electrically-driven pump. This pump would discharge to the

Map 3

ALTERNATIVE 4B: DETENTION STORAGE BENEATH ICE RINK PLUS SUPPLEMENTAL LOCAL SEWERS



Source: Village of Fox Point and SEWRPC.

existing storm sewer which flows to the west into the City of Glendale and could completely evacuate the full reservoir in 22 hours while discharging at 13 cfs, which is about one-half the capacity of the existing storm sewer, thereby assuring that no storm sewer surcharge or surface inundation problems would occur along the route of the sewer and receiving stream either in the Village of Fox Point or the City of Glendale.

Geologic and groundwater conditions at the site do not appear to pose any serious problems for construction and operation of the underground storm water storage facility. Both the bedrock surface and the normal water table would lie about 60 feet below the reservoir floor although seasonal groundwater levels may rise temporarily to within 30 feet of the ground surface. Therefore, the underground reservoir would be constructed entirely within easily excavated unconsolidated material, and infiltration problems would be minimal both during and after construction.

Existing Surface Drainage and Proposed Storm Sewers: Storm water runoff in the tributary area would be carried to the underground storage reservoir partly by the existing drainage system consisting primarily of roadside ditches with concrete inverts. This existing surface drainage system would be heavily supplemented with about 10,900 feet of 18 inch to 48 inch diameter concrete sewer which would be constructed beneath most roadways within that portion of the Crossway-Bridge drainage area lying west of approximately N. Santa Monica Boulevard. The rational method, with a runoff coefficiently 0.30, was used to size the storm sewers so that they would convey the discharge produced by five-year recurrence interval rainfall event.

Operation of the Storage System During and After a Runoff Event: The reservoir would operate in a manner identical to that expected for the reservoir included in Alternative 4A. The reservoir would be empty prior to the beginning of a runoff event, would gradually fill during and immediately after such an event and, after flow in the existing storm sewer to the west had subsided, the pump would be used to safely evacuate the reservoir through that sewer.

Consequences of Rainfall-Runoff Volumes in Excess of Design Capacity: The 24-acre-foot underground storage facility is sized to capture and control the volume of surface runoff expected to be reached or exceeded once on the average of every five years. There is, of course, the possibility that severe meteorological events could occur that would generate storm water runoff volumes in excess of those used to size the storage facility. While cost considerations prohibit construction of storm water control facilities to accommodate all possible storm water runoff volumes, it is important to assess the likely impact of storm water runoff volumes greatly in excess of those used to size the storage facility.

Accordingly, a gross analysis was conducted to determine the approximate disposition of storm water runoff generated by rainfall events ranging from a two-year through 50-year recurrence interval. The analysis was made for existing conditions and for Alternative 4B in order to provide a relative assessment of the system performance capability. The analysis was conducted in a manner identical to that described above for Alternative 4A.

The results are set forth in Table 3 and are seen to be essentially the same as those obtained for Alternative 4A. The only difference between the results of the two analyses is that a small amount of storage—relative to the volume of the storm water detention facility—is available in the storm sewers that would be constructed under Alternative 4B whereas that type of storage is negligible for Alternative 4A.

Opportunity for Control of Diffuse Source Pollution: As noted earlier, a positive aspect of the detention-retention storage of storm water is the opportunity to provide treatment of storm water runoff through plain sedimentation or through other physical or biochemical means, prior to its discharge from the storage facility to the surface water system.

A storage-oriented facility like that envisioned under this alternative is more likely than a conveyance-oriented system to be adaptable to treatment of storm water runoff if and when diffuse source pollution controls are recommended in the areawide water quality planning and management program currently being conducted by the Commission and subsequently to be required by the federal or state government. At the present stage of the areawide water quality planning and management program, it appears as though some form of diffuse source pollution control will be required in urban areas like the Village of Fox Point. However, the degree of control required and the best means of achieving such control have not yet been determined.

Capital and Operation and Maintenance Costs: The estimated capital cost of Alternative 4B is $2,470,000, consisting of $1,720,000 for the underground concrete reservoir—which includes $20,000 for the pump—and $750,000 for new storm sewers leading to the reservoir. Using an annual interest rate of 5 percent and an amortization period of 15 years, the average annual cost of amortizing the capital expenditure is $238,000.

Automatic operation of the pump used to evacuate the subsurface storage facility after runoff events would require an estimated average annual cost of $2,000 for electrical energy and pump maintenance. The pump and controls would require periodic inspection and maintenance.

In addition, suspended sediment and other solids carried and deposited within the reservoir would have to be periodically—probably yearly—removed. An average of about 100 cubic yards of sediment and other suspended material may be expected to be deposited in the storage facility each year. The estimated annual cost for removing and disposing of this sediment, and of inspecting the concrete structure including minor repairs, is $5,000, bringing the total annual operation and maintenance cost to $7,000.

The total average annual cost of Alternative 4B, computed using an annual interest rate of 5 percent and an amortization period of 15 years, is estimated at $245,000. This cost consists of $238,000 per year for amortization of the $2,470,000 capital cost of the reservoir and storm sewers, $2,000 per year pump operation costs, and $5,000 per year for sediment removal and disposal and for structure inspection and repair.

COMPARISON OF ALTERNATIVES AND RECOMMENDED ACTION

The Village of Fox Point engineering staff completed a systems level analysis of three basic conveyance-oriented solutions to all or part of the storm water inundation problems existing in the Crossway-Bridge and Port Washington-Bayfield drainage areas. The February 1977 report prepared by the Village staff and titled, "Storm Sewer Feasibility Study for the Port Washington-Bayfield Drainage Area and the Crossway-Bridge Drainage Area," concludes that the most desirable of the three basic conveyance-oriented solutions is Alternative 1, the storm sewer (trench and tunnel) to Lake Michigan. If this alternative were to be implemented, it would probably be constructed in two phases. The first phase, which is identified as Alternative 1A in this report, would be the construction of the sewer and tunnel to Lake Michigan intended primarily to provide relief in the vicinity of N. Crossway Road, E. Mall, and N. Lombardy Road and in the vicinity of E. Community Place and N. Santa Monica Boulevard in the Crossway-Bridge drainage area and also to provide relief in the vicinity of W. Bayfield Road and N. Mohawk Road in the Port Washington-Bayfield drainage area. The second phase involves the construction of additional local sewers within the Crossway-Bridge drainage area, would result in the system identified as Alternative 1B in this report, and would provide relief from storm water inundation in the remainder of the Crossway-Bridge drainage area including that portion lying within the City of Glendale.

The Commission staff concurs in the findings of the Village engineering staff: that is, that the most favorable of the three available conveyance-oriented solutions to the storm water inundation problems in the Crossway-Bridge and Port Washington-Bayfield drainage areas in the Village of Fox Point is the sewer to Lake Michigan with supplemental local sewers. Relative to the other two basic conveyance-oriented alternatives, the sewer to Lake Michigan alternative has three principal advantages.

- First, this alternative, upon completion of the above two phases, would serve the entire Crossway-Bridge drainage area including the Glendale portion. While the other two alternatives would provide some relief from storm water inundation problems within the Crossway-Bridge drainage area, neither could be readily expanded to provide storm water drainage to the entire portion of the Crossway-Bridge drainage area west of N. Santa Monica Boulevard which includes the City of Glendale portion of the drainage area.

- Second, the sewer to Lake Michigan alternative would result in minimal disruption to established residential areas since all of the construction would be in trenches on public land or public right-of-way or in deep tunnel.

- Third, the sewer to Lake Michigan alternative is more cost-effective than either of the other two basic alternatives. The annualized cost of Alternative 1B, the sewer to Lake Michigan plus supplemental local sewers, approximates the annualized cost of Alternative 2B, the sewer to Glendale on existing alignment plus tunnel and local sewers to Lake Michigan, and the former would provide more extensive storm water inundation relief than the latter. Alternative 1B is more costly—by 17 percent—than Alternative 3B, the sewer to Glendale on existing alignment plus tunnel and local sewers to Lake Michigan, but would provide considerably more storm water inundation relief and, therefore, is more cost-effective.

This report provides one additional alternative—a storage-oriented system—for comparison to the most favorable of the three conveyance-oriented alternatives. Like Alternative 1, this detention storage alternative would probably be implemented in two phases. The first phase (Alternative 4A) would entail the construction of the detention storage reservoir beneath the ice rink supplemented with short segments of storm sewers intended to serve the N. Crossway Road-E. Mall Road and N. Lombardy Road area and the E. Community Place-N. Santa Monica Boulevard area. The second phase would be the construction of additional storm sewers intended to serve the entire portion of the Crossway-Bridge drainage area west of N. Santa Monica Boulevard including the section lying within the City of Glendale. The first phase of Alternative 1 and the first phase of the storage-oriented system would have identical storm water control functions, and both share the positive features of resulting in minimal disturbance to established residential areas during construction. The principal difference between these two alternatives is cost in that the first phase of Alternative 1—construction of sewer and tunnel to Lake Michigan—has an annualized cost of $117,000, which is 68 percent of the $173,000 annualized cost of the first phase of the storage-oriented system.

Upon completion of the second phase of each alternative, Alternative 1 and the storage-oriented system would each have similar storm water control functions except that the former would provide additional service in the form of storm water control to the N. Lake Drive-E. Wye Lane-E. Portage Road area of the Crossway-Bridge drainage area. Furthermore, there is a significant cost differential favoring the former since the annualized cost of the completed Alternative 1 is estimated at $177,000, which is 72 percent of the $245,000 estimated annualized cost of the completed storage-oriented system.

Therefore, the Commission staff concludes that the most effective of the four basic available solutions to the storm water inundation problem in the Crossway-Bridge and Port Washington-Bayfield drainage areas is Alternative 1— the sewer to Lake Michigan with supplemental local sewers.

SUMMARY

The purpose of this report is to present conveyance-oriented storm water control alternatives for mitigating storm water inundation problems in the Crossway-Bridge and Port Washington-Bayfield drainage areas in the Village of Fox Point, Milwaukee County, Wisconsin, so that Village officials and concerned citizens can make decisions for resolving storm water inundation problems in that portion of the Village. The storage-oriented storm water control alternatives are presented in this report to facilitate comparison with the conveyance-oriented storm water control alternatives previously developed by the Village staff. This report was prepared by the Southeastern Wisconsin Regional Planning Commission at the request of the Village of Fox Point.

Two fundamentally different approaches to urban storm water control may be identified—a conveyance-oriented approach and a storage-oriented approach. The principal function of systems designed with the conveyance-oriented approach is to provide for the collection of storm water runoff in the service area followed by the immediate and rapid conveyance of storm water from that area so as to minimize disruption and possibly damaging surface ponding in streets, parking lots, and other low-lying areas. The principal function of systems designed with the storage-oriented approach is to provide for the temporary storage of storm water runoff within or near the service area for subsequent slow release to downstream channels or storm sewers, thus minimizing disruption and damage both within and downstream of the service area.

The principal result of improved storm water control in the Crossway-Bridge and Port Washington-Bayfield drainage areas in the Village of Fox Point would be a reduction in the frequency, depth, and lateral extent of storm water inundation. Such inundation has proven to be disruptive to the vehicular and pedestrian traffic and at times has resulted in basement flooding when ponded storm water entered residential structures through basement windows and other openings. While improved storm water control may also contribute to solving the sanitary sewer system surcharging and basement flooding problems, the ultimate solution to those problems is eliminating clear water in the sanitary sewer system during wet weather periods.

The Village engineering staff completed a systems level analysis of three basic conveyance-oriented solutions to all or part of the storm water inundation problems in the Crossway-Bridge and Port Washington-Bayfield drainage areas. Alternative 1 ultimately would consist of a sewer to Lake Michigan plus supplemental local sewers. Alternative 2 ultimately would consist of a bored sewer to Glendale on the alignment of an existing storm sewer, plus a tunnel and local sewers discharging to Lake Michigan. Alternative 3 ultimately would consist of a sewer constructed in a trench to Glendale on the above existing alignment plus a tunnel and local sewers to Lake Michigan. The Village engineering staff concluded, and the Commission staff concurs, that the most favorable of the three basic conveyance-oriented alternatives on the basis of technical, economic, and nontechnical and noneconomic considerations is Alternative 1—the sewer to Lake Michigan plus supplemental local sewers.

The Commission staff completed a systems level analysis of a storage-oriented solution to the storm water inundation problems existing in the Crossway-Bridge and Port Washington-Bayfield drainage areas. The storage-oriented solution was designed to provide a level of storm water control similar to that provided by the Village's Alternative 1, thus facilitating a sound comparison between these two fundamentally different approaches to storm water control.

Assuming implementation as a two-phase project, the basic storage-oriented alternative developed by the Commission and described in this report would consist of two principal components: 1) a 24-acre-foot underground concrete reservoir located beneath the Village ice rink and provided with a 6,000 gallon per minute pump to safely empty the reservoir and 2) about 10,900 feet of 18 to 48 inch diameter concrete sewer intended to convey storm water runoff from that portion of the Crossway-Bridge drainage area west of approximately N. Santa Monica Boulevard to the underground reservoir. The reservoir would be empty before the beginning of the runoff event, would gradually fill during and immediately after such an event, and pumps would be used to evacuate the reservoir through that sewer after flow had subsided in the existing storm sewer that flows to the west through the Port Washington-Bayfield drainage area into the City of Glendale.

The total average annual cost of the storage-oriented solution, computed using an annual interest rate of 5 percent and an amortization period of 15 years, is estimated at $245,000. This cost consists of $238,000 per year for amortization of the $2,470,000 capital cost of the reservoir and storm sewers, $2,000 per year pump operation and maintenance, and $5,000 per year for sediment removal and disposal and for structure inspection and repair.

The level of storm water control provided by the most favorable conveyance-oriented alternative—sewer to Lake Michigan plus supplemental local sewers—and the storage-oriented alternative are similar although the former would provide storm water relief in the N. Lake Drive-E. Wye Lane-E. Portage Road area whereas the latter would not provide relief to that area. The conveyance-oriented alternative, however, is more cost-effective in that it will provide somewhat better storm water control at an annual cost that is 72 percent of that of the storage-oriented alternative. Therefore, it is concluded that, of the four basic alternatives available to the Village of Fox

Point for controlling storm water inundation in the Crossway-Bridge and Port Washington-Bayfield drainage areas, the most favorable alternative based on technical, economic, and nontechnical and noneconomic factors is Alternative 1—the sewer to Lake Michigan supplemented with local sewers.

APPENDICES

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Appendix A

VILLAGE OF FOX POINT
WISCONSIN

STORM SEWER FEASIBILITY STUDY

for

The Port Washington-Bayfield
Drainage Area

and the

Crossway-Bridge
Drainage Area

February, 1977

February 18, 1977

STORM SEWER FEASIBILITY STUDY

The following report is the basis for consideration by the Village Board of a flooding problem that occurs in the vicinity of Mall and Crossway. Over the past 10 to 15 years, flooding of the intersection has occurred. No private property damage was evident until the flood of 1973.

On April 21, 1973 the intersection was totally inundated to a depth of $2\frac{1}{2}$ feet. The rainfall in the 1973 storm amounted to 4.75 inches, which is referred to as a rainfall that would occur once in 50 to 75 years. This caused substantial damage to many of the residences in the area. Other areas throughout the Village experienced flooding as shown on the enclosed Village map on Page 14. Concern over this situation is what prompted Mr. Stegeman, Mr. Rosendahl, and Capt. Killoran, residents of the Mall & Crossway area, to actively seek action by the Village to rectify the problem. (Refer to the Village Board minutes of June 8 and June 22, 1976.)

The existing system, which starts on Longacre Road at the Ice Skating Pavillion, runs westerly through easements between residences to Port Washington Road just north of Bayfield Road, where it is intercepted by a storm sewer in the City of Glendale. The system is shown on the map on Page 18, labeled Alt. No. 2 which will be discussed later. It should be pointed out that the present storm sewer is capable of handling approximately 26 cfs (cubic feet per second) plus a considerable retention volume of water in the ditches of the drainage area. In accord with present design criteria, the present corrugated storm sewer is under-designed. The storm sewer should be required to handle approximately 74 cfs for a 5 year storm. A storm which occurs once in 5 years is a design criteria based on the theory of probability which states that statistically after all the storm water data has been collected, a certain amount of rain should only occur once every five years. It seems evident then that some revision of the present system should be made.

In order to have a more complete perspective of this study, we believe a review of some of the past Village storm sewer projects should be presented. The present system of underground storm sewers within the Village is local in character in that they were originally constructed to take care of specific problem areas. No comprehensive system was considered due to limited capital funding. In attempting to maintain a rural character in the Village, the traditional sidewalks and curbs have not seriously been considered. Storm water drainage relies almost entirely on our roadside ditches. Only in certain instances where it has been found to be the only solution to a drainage problem or necessary because of traffic and pedestrian movement, has the drainage system been placed underground. This is not to say that capital expenditures have not been made for storm water drainage. Since 1960, $463,000.00 has been spent for the construction and maintenance of our roadside ditches, and $175,000.00 has been spent for constructing concrete inverts in problem ditches. Within the same period, $129,300 was spent for the installation of culverts and storm sewers. A few of the major projects that have been constructed are: in 1949 a storm sewer and outfall at the base of the Beach Drive hill at a cost of $56,000.00; in 1953 on Green Tree Road, Yates, and Foxdale Roads, a project which cost $12,754.00; in 1961 on Dean Road and Fox Lane $36,635.00 was spent; in 1966 on Lake Drive $15,771.00 was used to enclose an existing ditch system in the 7100 block.

In 1974, a 60" storm sewer was constructed in the 7300 block of Beach Road at a cost of $152,587.00. This project being the first phase of Alternate No. 1, as shown on Page No. 17 , was constructed to correct an existing drainage problem along Beach Road and Willets Lane and was designed to operate in conjunction with Phase II, Alternate No. 1.

Prior to 1950 all storm sewer construction was paid for by special assessment. In July of 1930 the Village Board passed an ordinance creating what was referred to as "Storm Sewer Districts". This ordinance was revised in May of 1941 when the total number of districts reached twelve. (A map of these original drainage districts is on page 15 .) You will note that these drainage districts generally coincide with the natural drainage patterns. Special assessments for capital storm water improvements were based on contributing acreage in relation to the cost of the project in the drainage district. The storm sewer and outfall constructed along Beach Road in 1949 had a project cost of $56,000. This storm sewer fell within the boundaries of Storm Sewer Districts #10 and #11: The cost of the project was shared by all property owners in the two districts at an assessment of $54.00 per acre. Since 1950 storm sewers constructed in the Village have been financed through the general fund. Some of these projects were listed above.

At present the drainage areas of the Village and the surrounding area comprise six general areas as compared to the original twelve drainage districts. (A map of these areas is found on page 16 .) It is interesting to note that in every drainage area except the Crossway to Bridge area, there has been some major storm sewer construction. In the Port Washington to Bayfield area, there is the storm sewer in question. In the South ravine area there is the sewer in Green Tree Road, Foxdale, and Yates Roads; the sewer in Lake Drive in the 7100 block; and a system that was constructed back in 1928 and 1929 on Belmont and Barnett Lanes. In the North ravine - railroad - and Lake Michigan areas there have been a number of small and large projects including those on Fox Lane, Dean Road, Beach Drive, Thorn Lane, Lake Drive, and Berkeley Boulevard. Then in the Indian Creek area, major work was done to improve the drainage on Santa Monica Boulevard to Indian Creek. Indian Creek to date has cost $165,610.00 for excavating, bridges, culverts, concrete invert, and landscaping.

The map on page 14 shows the areas that were flooded in the April 1973 storm. As was mentioned this storm was one that is predicted to occur only once in 50 to 75 years, and is one that no storm sewer system is designed to handle. The Village has experienced many other heavy rainfalls in the past and since the 1973 storm some of these same areas have been affected. As recent as March 4, 1976 and April 24, 1976 heavy rains caused local flooding in the areas marked B, G, H, J, and K on the map. However, in some of the areas, corrective measures have taken place since the 1973 flood. Along Indian Creek the possibility of local flooding has been reduced with completion of the concrete invert project from Port Washington Road to I-43. The County has also improved the situation by cleaning and restoring the invert of the box culvert under I-43. The latest U. S. Department of Housing and Urban Development's "Flood Hazard Boundary Map" however, still identifies this area as one eligible for flood insurance when referring to a 100 year storm. At Beach Road and Willets Lane, the 60" storm sewer mentioned above has removed the threat of flooding in Area M. The flooding which occurred in Area F on the Beach Road was caused by excessive debris flowing down the ravine clogging the inlet, which is designed as a self-cleaning structure, leading to the sewer which empties into Lake Michigan. In addition, the catchbasins along Santa Monica Boulevard at Foxdale Road have been rebuilt which should improve the situation in Area N.

The intersection of Greenvale Road and Spooner Road, in Area B which has experienced frequent flooding is served by underground storm sewer. This flooding situation is currently being investigated by the Engineering Department. Areas D, G, H, J, K, and L which all can be relieved in some way by this proposed storm sewer project, Alternate No. 1, will now be discussed.

The solution to the flooding problem at the intersection of Crossway and Mall Roads is to provide a larger capacity sewer or compliment the present sewer with an additional sewer to serve the intersection. There appears to be, after study by the Engineering Department, three alternate solutions.

The first alternative is to construct a 54" storm sewer which will discharge into the ravine at the south end of Bridge Lane and flow down the ravine into the storm sewer constructed in 1974 on Beach Drive. The proposed sewer will then extend to and along Daisy Lane and across the railroad tracks to Community Place; to Longacre; to the Village Skating Rink property; to Lombardy Road; to Mall Road; and then to Crossway. The route of this sewer is shown on the map labeled Alternate No. 1 on Page 17 . This particular project would be done in two parts. The first section would be a 54" sewer constructed in a deep tunnel to Longacre Road and the Village Skating Shelter property. The second part of the project is a 48" reinforced concrete sewer to be constructed in an open trench to Crossway and Mall. The cost of this project in terms of 1976 money is estimated at $1,103,000.00.

As shown on the map this alternate provides for future extensions which should be constructed at a later date to alleviate other problems (See Page 15) within the drainage area. The cost to construct these future extensions is estimated to be $564,000.00. This brings the total cost of Alternate No. 1 to $1,667,000.00.

The drainage area for Alternate No. 1, which encompasses some 253 acres, is in actuality the Crossway to Bridge drainage area described on the map of the Village drainage areas. You will note from that drainage map that this area includes portions of the Port Washington - Bayfield area and the Indian Creek area. The capability of being able to take some of the load from these other drainage areas is a distinct advantage of this alternate.

Alternate No. 2 as shown on the map on Page 18 follows the route of the existing sewer to Port Washington and Bayfield Roads. This project calls for boring a 60" steel pipe along side of the existing sewer. However, this system is constructed through easements between private residences; thereby making construction difficult. The cost of this method is $644,000.00. The sewer would discharge into the recently completed 72" elliptical storm sewer in the City of Glendale west of Port Washington Road. Presently the County has a small box culvert under Port Washington Road which would have to be relayed. Regardless of what decision the Village Board will make, this box culvert should be relayed and such a request will be made to the County. The proposed sewer is necessarily very shallow and due to this, it is impossible to complete any of the other extensions as found in Alternate No. 1, most notably the extension to Fairchild Circle to accept the flow coming from the City of Glendale on Seneca Road. Attached to this report is a letter from the City of Glendale requesting that we construct an outlet for their proposed storm sewer system in that area. In addition to the proposal of Alternate No. 2, there could be another system built to serve the areas of Lake Drive and Santa Monica Boulevard. Although this system is outside of the drainage area, it should be considered for a total comparison of the three alternates. The additional system would be a deep tunnel, similar to that of Alternate No. 1, constructed to Santa Monica Boulevard and Community Place, and then various extensions from that. The cost of this additional system is approximately $1,373,000.00 in terms of 1976 money, giving us a total cost of $1,705,000.00 for Alternate No. 2.

The third proposal is similar to that of Alternate No. 2 in that it would follow essentially the same route of the existing sewer. The difference is that this proposal calls for constructing a 60" reinforced concrete pipe through new easements which would have to be obtained. The construction would be in open trenches, close to existing dwellings in some situations and would make construction most difficult. Inconvenience to the residents, landscaping, and possible damage to existing dwellings could cause problems. Despite the difficult construction, the cost would be less than the cost of Alternate No. 2. The estimated cost for this phase of the project would be $363,000.00.

This method however, as in Alternate No. 2, does not provide for the extensions able to serve other sections within the drainage area. Flow from these areas would have to remain in the existing roadside ditches whose capacities are limited.

Also as part of Alternate No. 3, an additional system could be constructed to compliment the project as in Alternate No. 2. This would bring the total cost of Alternate No. 3 to $1,425,000.00.

In October of 1976, the Village made application for a federal assistance grant for the construction of Alternate No. 1 under the Local Public Works Capital Development and Investment Act of 1976 (L.P.W.) through the U. S. Department of Commerce. This grant was for 100% funding of the project. However, on December 23, 1976 we were notified that our proposal was not among those selected for approval. When the U. S. Congress reconvened in January some Democratic members of the House Public Works Committee urged that the L.P.W. program be refunded and continued. The House Committee will be holding hearings in late January or early February on the L.P.W. program.

The National League of Cities is in support of continuing the L.P.W. program and will be testifying to that effect at the above mentioned hearings. The NLC has said it will urge an amendment to the act, not to create the need for further applications, but that the more than 20,000 applications still on file with the Economic Development Administration be used as the basis for further funding.

In 1966 the Village started a Storm Sewer Capital Improvement Fund with an initial appropriation of $75,000.00. The intent of the fund was for projects such as this. Money has been appropriated each year and the only funds that have been expended were $152,587.00 for the Beach Road sewer in the 7200 block to Lake Michigan. The balance of this fund in 1976 amounted to $386,078.95.

A brief review of the three alternate projects with comparable costs are as follows:

1.) Drainage area map Alternate No. 1, serves the Crossway to Bridge Area. This alternate project has the capability of being extended to serve all the sections of the drainage area and alleviate some of the loads from existing systems.

2.) Drainage area map Alternate No. 2 serves the Port Washington - Bayfield drainage area. Because of design limitations, it would not be possible to add additional underground storm sewer extensions to serve all the sections of the drainage area. The roadside ditches especially along Crossway south of Mall and Fairchild Circle are severely taxed during heavy rains. This is mainly caused by the drainage from the City of Glendale at Fairchild Circle. One must consider under this alternate, the inconvenience during construction to the residents; the problems of re-establishing the landscaping including trees and shrubs; and the possible damage during construction to existing buildings.

3.) Drainage area map Alternate No. 3 serves the same area as Alternate No. 2 and has like limitations and possibilities.

To correct the problem at Crossway and Mall Road, which any of the three alternates will accomplish, the cost is estimated to be:

Alternate No. 1	$1,103,000.00
Alternate No. 2	$644,000.00
Alternate No. 3	$363,000.00

As has been explained, Alternate No. 1 would be able to serve a broader area. To serve some of the other problem areas, additional systems would have to be built to compliment Alternates No. 2 and No. 3. Thus a more equitable comparison of the three projects would be to include the cost of all possible future extensions:

Alternate No. 1	$1,667,000.00
Alternate No. 2	$1,705,000.00
Alternate No. 3	$1,425,000.00

The financing of a project of this scope is very difficult to consider. The imposed state levy limits is causing us some difficulty in budgeting annually, even though we have reduced our expenditures by increasing our fees; eliminating some personnel; and increased productivity in all departments.

Since 1971 monies have been transferred from the Tax Stabilization Account to augment our decrease in state shared taxes; thereby decreasing the amount of property tax required. In 1973 monies were transferred from the water utility accrued tax account to the village for the same purpose as mentioned above. At the suggestion of the Public Service Commission and our accountants, it was advisable to decrease this accrued tax account of the water utility. The 1976 budget was the first budget prepared under the State tax levy limit law.

As of December 31, 1976 it was estimated that the General Fund balance of the Village amounted to $456,353.00. The Village will have a total fund balance of approximately $1,408,348.00, which includes monies from the income stabilization fund; money still owed the Village by the water utility from the accrued tax account; and capital improvement funds amounting to $507,807.00. In projecting our revenues and expenditures through 1981, using a 5% inflation factor and a 12% increase in tax levies, we find that the total funds mentioned would be practically depleted. Unfortunately we now find ourselves in a position of whether we should borrow all of the necessary monies required to proceed with the storm sewer project; or use the funds allocated to the storm sewer capital improvement fund and borrow the balance. If the total capital improvement reserve accounts would not be used as revenues in the succeeding years, the Village will not have sufficient funds to operate after 1979. According to our projection, we would have a revenue gap in 1980 of $204,000 and in 1981 of $217,000. In 1982 we would have a revenue gap of $64,200 with levy limits increasing at a rate of 12% and inflation of 4%. We feel that by 1982 we would be able to budget without reducing our services within the amount of tax levy allowed.

We understand from the financial people that we are presently experiencing an ideal market for municipal bonding. The interest rates should be between 4.5% and 5.0%. The table below will show the annual payment including principal and interest on $400,000; $500,000; $700,000; and $1,000,000 for 10 and 15 year bonds. It also projects the assessed valuation, the village tax rate, and the additional tax rate required to pay the annual bond issue for the next five years. The projected Village tax rate is based on the allowable tax levy limits of 12% per year to 1983. It should be noted that in 1979 the rate will exceed the 20 mil statutory limit. This will require the Village to reassess prior to 1979.

TABLE I

Annual Debt Service

10 YEARS	4.5%	4.75%	5.0%
$400,000	$49632	$50330	$51000
$500,000	62040	62912	63750
$700,000	86856	88077	89250
$1,000,000	124080	125825	127500

TABLE II

Annual Debt Service

15 YEARS	4.5%	4.75%	5.0%
$400,000	$36267	$36800	$37333
$500,000	45333	46000	46667
$700,000	63467	64400	65333
$1,000,000	90667	92000	93333

TABLE III

	1978	1979	1980	1981	1982
Projected Assessed Value	49,500,000	50,000,000	50,100,000	50,200,000	50,300,000
Projected Tax Rate-Village	$17.98	$20.17	$22.65	$24.82	$28.31
Projected Additional Tax Rate for borrowing 10 years @ 4.5%					
$400,000	$1.00	$0.99	$0.99	$0.99	$0.99
$500,000	$1.25	$1.24	$1.24	$1.24	$1.23
$700,000	$1.75	$1.74	$1.74	$1.74	$1.73
$1,000,000	$2.51	$2.50	$2.48	$2.47	$2.47
Projected Additional Tax Rate for borrowing 15 years @ 4.5%					
$400,000	$0.73	$0.72	$0.72	$0.72	$0.72
$500,000	$0.92	$0.91	$0.91	$0.90	$0.90
$700,000	$1.28	$1.27	$1.27	$1.26	$1.26
$1,000,000	$1.83	$1.82	$1.82	$1.81	$1.80

Another possibly way to finance the project could be a return to the method of special assessments. Even here many variables could be applied. Taking for example the cost of Alternate No. 1, $1,103,000.00 and having the project be 100% assessable against every lot that the project would benefit, the cost would be $1,973.16 per lot, based on a district of 559 lots. Another way would be to view the district in terms that 76 acres of the 253 within the drainage area is Village property and in the City of Glendale. That portion would be the entire Village's responsibility and would reduce the assessable cost of the project by 30% to $772,100.00 or $1,381.22 per lot. This assessment could possibly be spread over a 10 year period. Using a 6% interest rate, the yearly payment amounts to $187.66. If special assessments are considered, future extensions of the storm sewer should be taken into consideration. It should also be noted that the special assessment could be from 0 to 100% of the total cost of the project. The only restriction the Village has on the amount of assessment is that the benefit derived must equal or exceed the assessment cost.

We believe we have submitted for your consideration sufficient alternates in engineering and methods of financing for the construction of a storm sewer system. Also included are its ramifications on the Village's operational budget for the succeeding years. There is no question from an engineering viewpoint that Alternate No. 1 should be recommended. The problem is that with a comparatively static tax base coupled with inflationary governmental operational costs, the Village tax rate is increasing disproportionately. State imposed levy limits must be considered in our financing decision. The actual number of residences which are presently adversely affected by flooding is small in comparison to the total number of residences in the drainage or benefit area.

We believe this report will stimulate considerable discussion and careful study by the Village Board. It would be our feeling that a subcommittee of the Budget Committee be appointed to study the proposed project and make their recommendations. to the Village Board. This Committee is suggested because budgetary matters are of considerable importance in the deliberation of this storm sewer project. In 1976, this Committee was formulated to study the budgetary format of the Village and make its recommendations to the Village Board.

FROM AWWA JOURNAL -

Weather researchers now tell us, it is only the beginning of a new round of roaring floods, earth-cracking droughts, and smothering snow storms. No less an expert than J. Murry Mitchell, senior research climatologist at the US Environmental Data Service in Washington, D. C. points out, "From the early 1950s to the early 1970s, we've had benign weather, the variability of the weather has been uncommonly small and it has lulled us into a false sense of security. Now we are going back to an earlier pattern of greater variability and we are getting worried about it. But, to put it into perspective, we are kind of getting back to normal. Purely on the basis of probability, it is highly unlikely that we will experience the weather of the '50s and '60s again any time soon. We should be banking on greater droughts, more floods."

Submitted by:

W. J. Blong

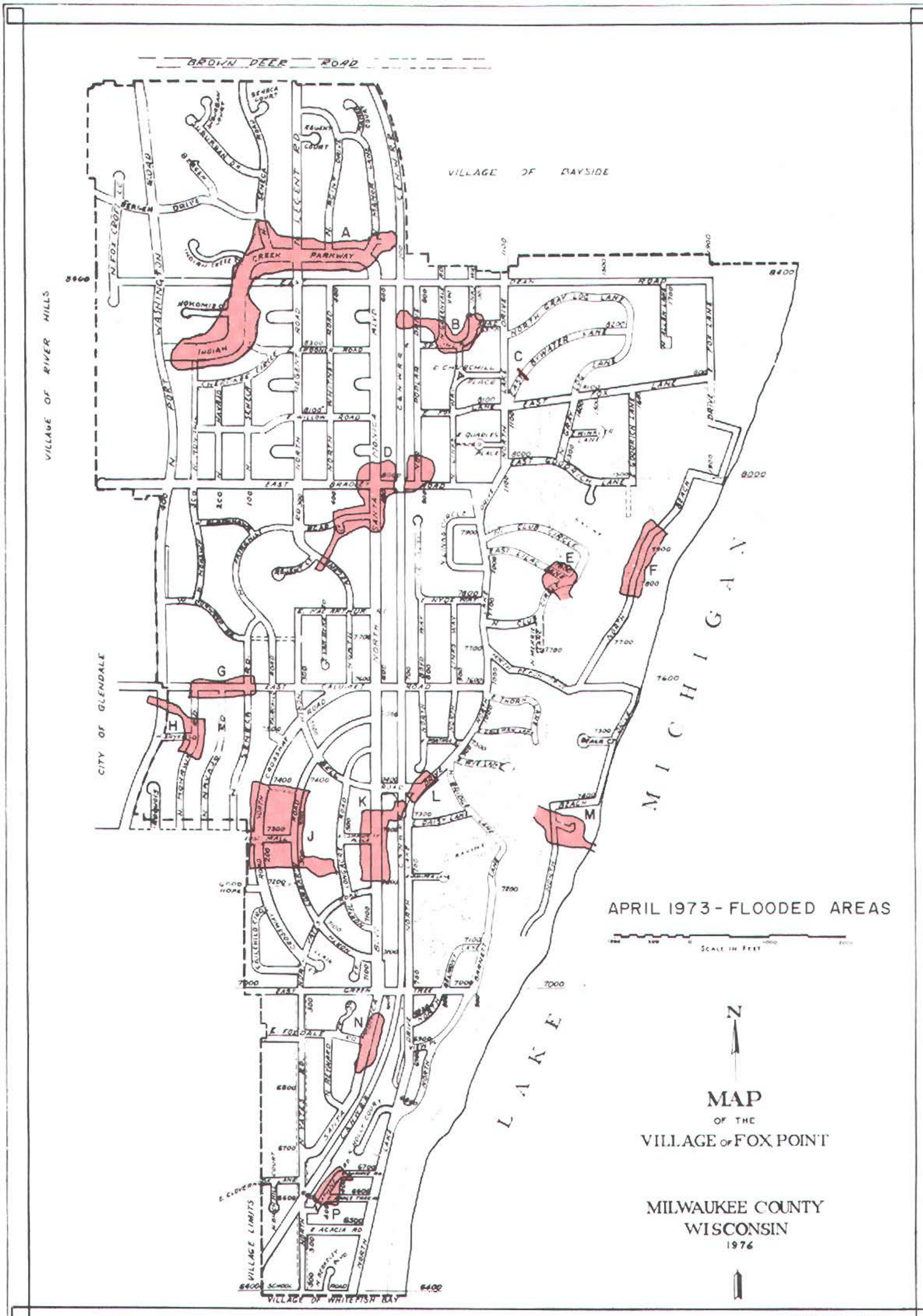
W. J. Blong, Village Manager
& Engineer

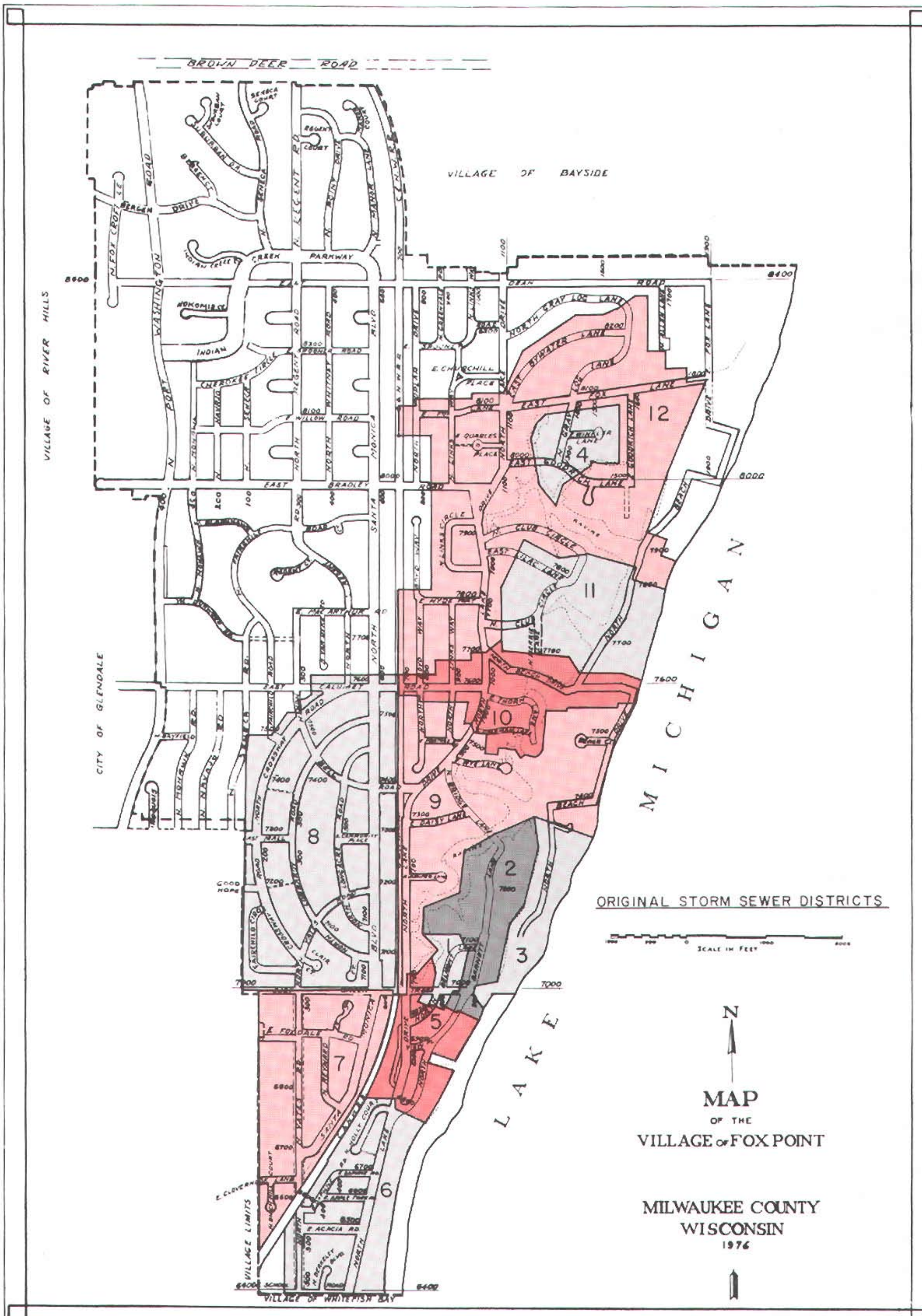
William R. Weltin

William R. Weltin, Assistant Village
Engineer

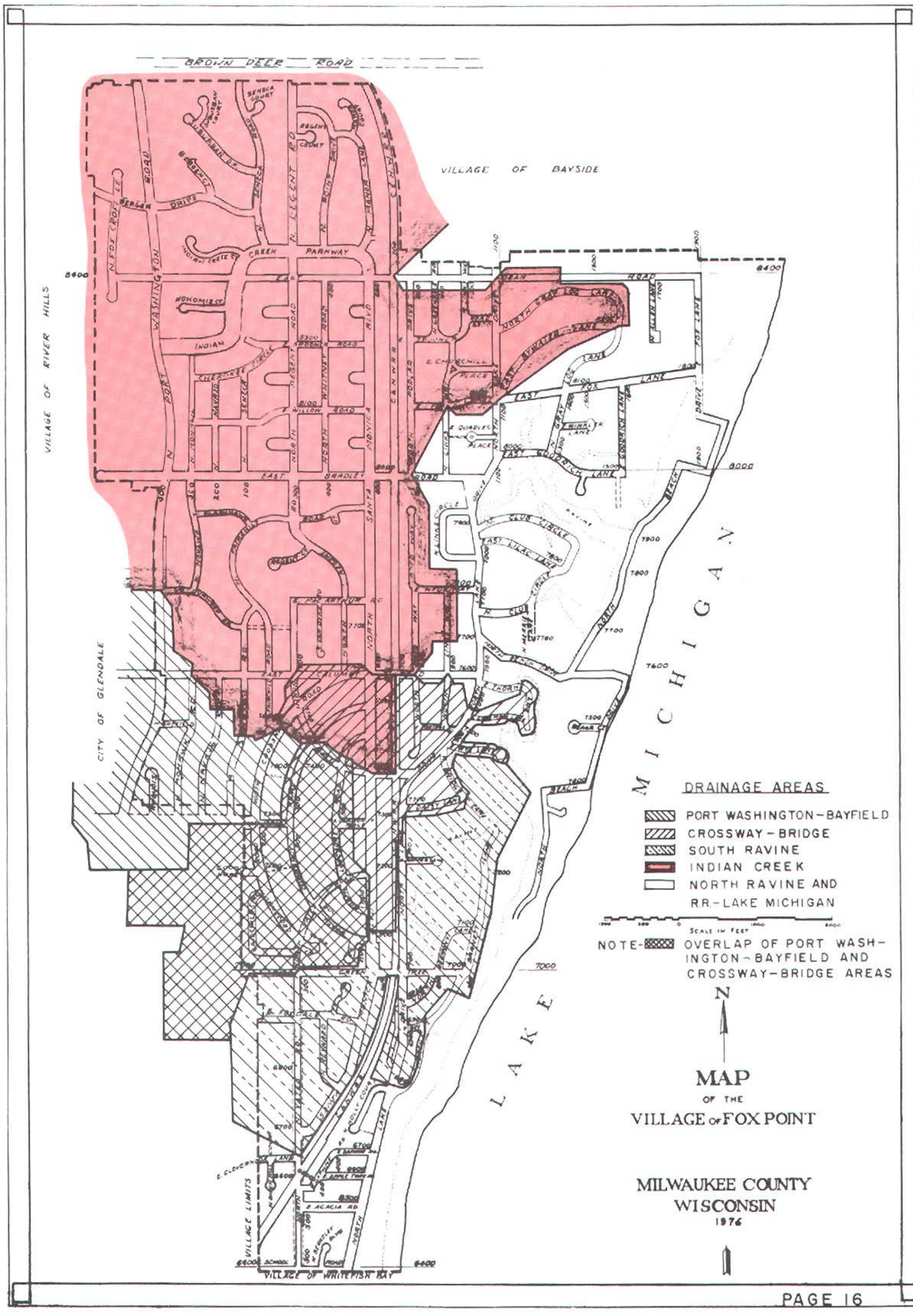
Appendix A-1

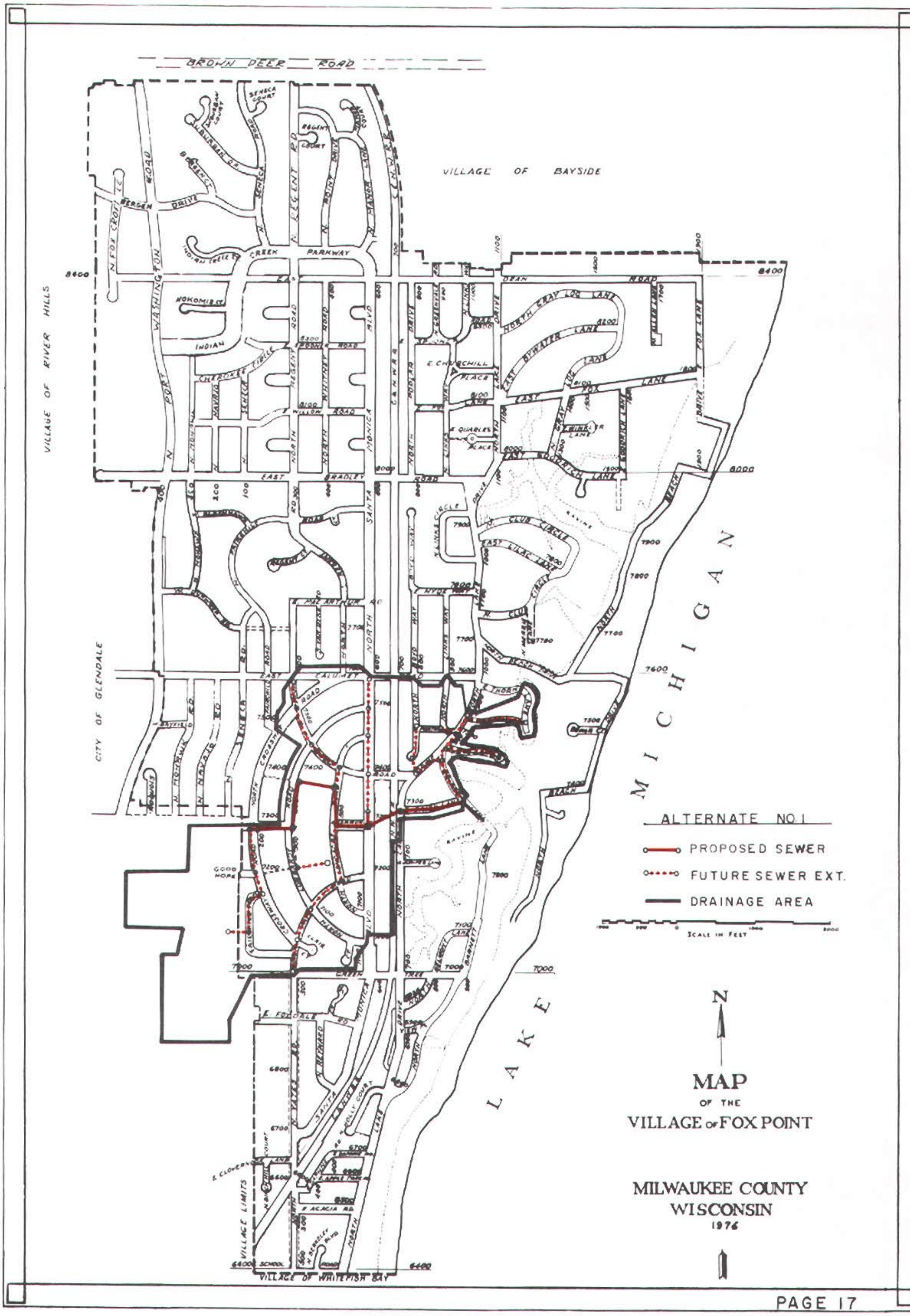
MAPS OF THE VILLAGE OF FOX POINT

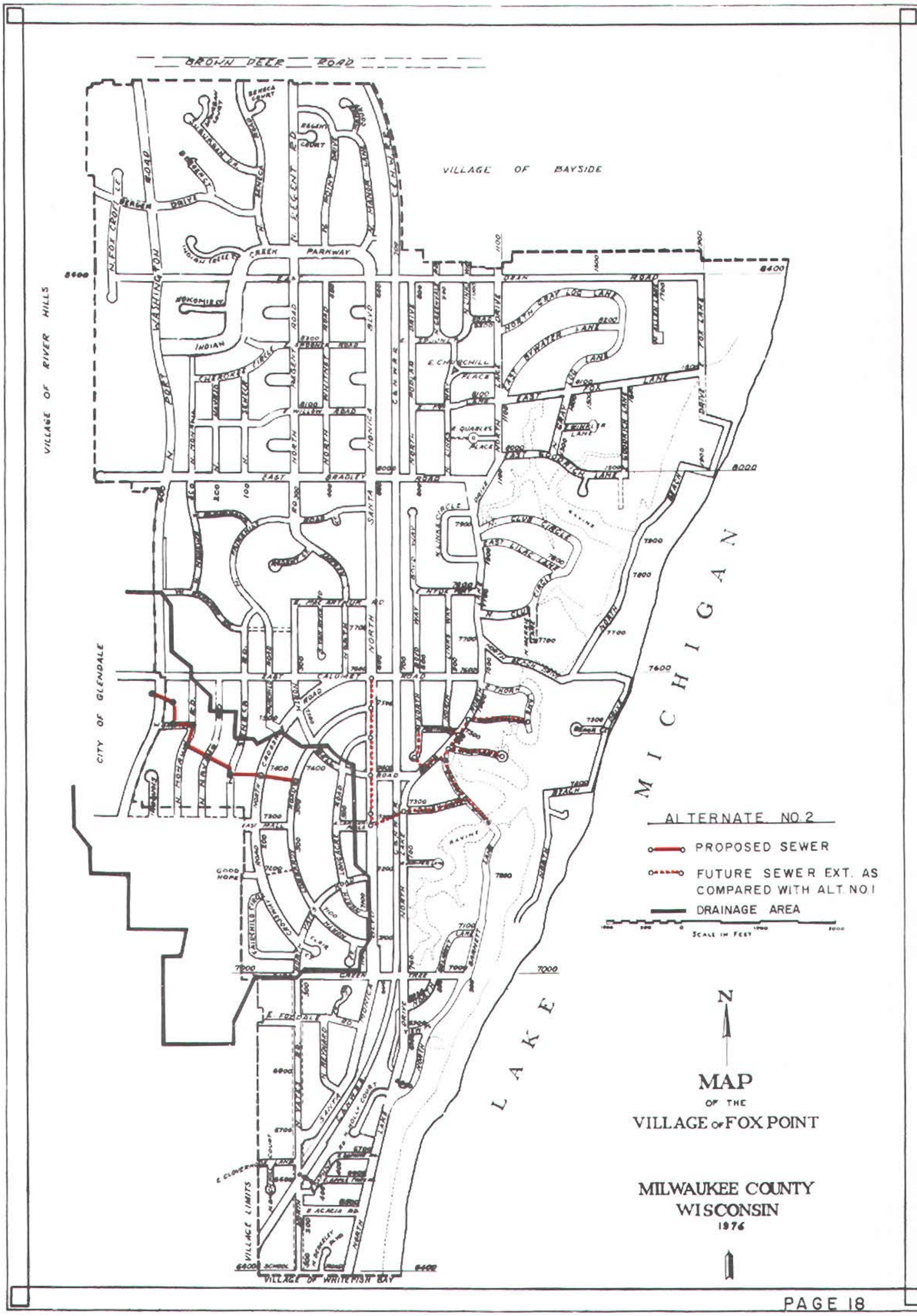




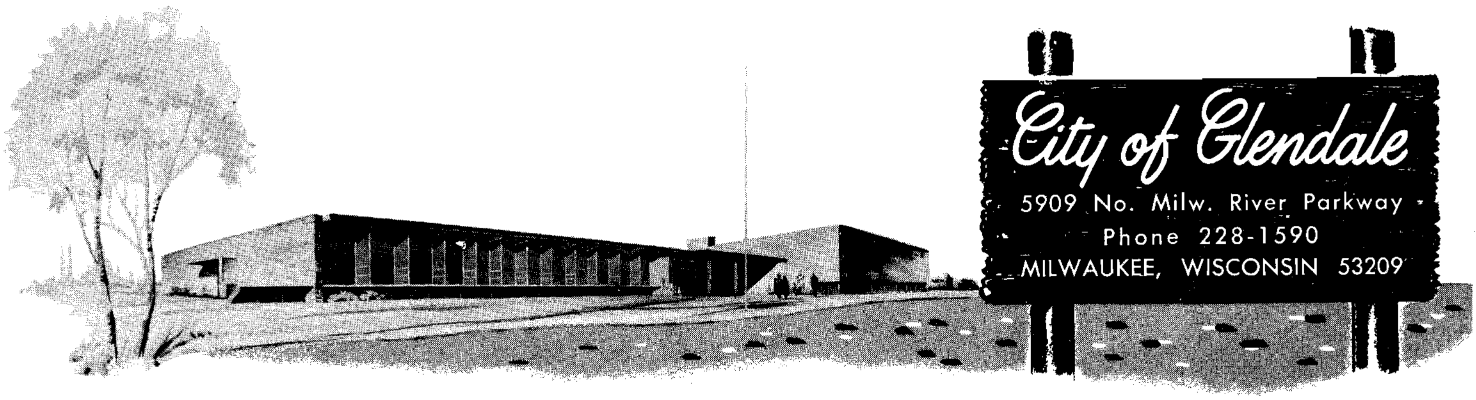
ORIGINAL STORM SEWER DISTRICTS
MAP
OF THE
VILLAGE OF FOX POINT
MILWAUKEE COUNTY
WISCONSIN
1976
PAGE 15
BROWN DEER ROAD
VILLAGE OF BAYSIDE
VILLAGE OF RIVER HILLS
CITY OF GLENDALE
VILLAGE OF WHITEFISH BAY
LAKE MICHIGAN
VILLAGE LIMITS
N


BROWN DEER ROAD
VILLAGE OF BAYSIDE
VILLAGE OF RIVER HILLS
CITY OF GLENDALE
LAKE MICHIGAN
VILLAGE OF WHITEFISH BAY
DRAINAGE AREAS
PORT WASHINGTON-BAYFIELD
CROSSWAY-BRIDGE
SOUTH RAVINE
INDIAN CREEK
NORTH RAVINE AND
R.R.-LAKE MICHIGAN
SCALE IN FEET
NOTE- OVERLAP OF PORT WASH-
INGTON-BAYFIELD AND
CROSSWAY-BRIDGE AREAS
N
MAP
OF THE
VILLAGE OF FOX POINT
MILWAUKEE COUNTY
WISCONSIN
1976
PAGE 16


ALTERNATE NO.1
PROPOSED SEWER
FUTURE SEWER EXT.
DRAINAGE AREA
SCALE IN FEET
MAP
OF THE
VILLAGE OF FOX POINT
MILWAUKEE COUNTY
WISCONSIN
1976
VILLAGE OF BAYSIDE
VILLAGE OF RIVER HILLS
CITY OF GLENDALE
VILLAGE OF WHITEFISH BAY
BROWN DEER ROAD
LAKE MICHIGAN
PAGE 17


BROWN DEER ROAD
VILLAGE OF BAYSIDE
VILLAGE OF RIVER HILLS
CITY OF GLENDALE
VILLAGE OF WHITEFISH BAY
LAKE MICHIGAN
ALTERNATE NO. 2
PROPOSED SEWER
FUTURE SEWER EXT. AS COMPARED WITH ALT. NO. I
DRAINAGE AREA
SCALE IN FEET
N
MAP
OF THE
VILLAGE OF FOX POINT
MILWAUKEE COUNTY
WISCONSIN
1976
PAGE 18

Appendix A-2



Office of City Engineer

October 26, 1976

Mr. W. J. Blong, Village Manager
Village of Fox Point
7200 North Santa Monica Boulevard
Milwaukee, Wisconsin 53217

RE: Drainage Storm Sewer

Dear Mr. Blong:

The City of Glendale has been troubled for years with drainage problems in the area of North Seneca Road and West Green Tree Road. Only last May we were threatened with legal action if we did not take steps to correct these problems.

Since our drainage system enters into the Village of Fox Point on Seneca, several hundred feet north of West Green Tree, our plans to solve our problem necessarily hinge on your plans to build the downstream system.

I respectfully request, therefore, that the Village of Fox Point consider the construction of the storm sewer system from North Seneca Avenue downstream so that we could plan for our own sewers.

Yours truly,

Peter J. Peters

PETER J. PETERS, P.E.
City Engineer

PJP:rlr

cc:Mr.Mikulich
City Adm.

Appendix A-3

March 1, 1977

TO: Village Board Members

FROM: W. J. Blong
Village Manager

SUBJECT: Addendum #1 to Storm Sewer Feasibility Study

It should be brought to your attention that after the possible completion of the recommended storm sewer with all its extensions, sanitary sewer back-up in basements during heavy rainfalls will still persist. Sanitary sewer back-up will continue until the clear water entering the sanitary sewer is greatly reduced or eliminated. The nine sanitary sewer overflows and lift stations and portable pumps have up to now kept the surcharging in basements to a minimum; the 1973 storm being an exception. The use of overflows or lift stations in the near future will be required to be discontinued by orders of the D.N.R. (Ref. Report - Pollution Abatement and Clear Water Control - May 23, 1975)

There still exists a great deal of misunderstanding regarding the functions of a sanitary sewer and a storm sewer. A sanitary sewer's function is to remove refuse liquids and matter. Sanitary sewer back-up in basements always occurs during a heavy rainfall. The main source of clear water entering the sanitary sewer causing surcharging, is from foundation drains. Infiltration through manhole covers, house laterals, and the main sewers are the other contributing sources.

The completion of the storm sewer, whose function is to collect and remove surface water, should help reduce surcharging by reducing infiltration thru sanitary sewer manholes and covers; infiltration into house laterals and main sewers; and in some cases, flooding or inflow of clear water into basements, which flows into the floor drains connected to the sanitary sewer compounding the surcharging problem.

WJB:bn

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Appendix B

TECHNIQUE FOR DETERMINATION OF STORAGE VOLUME

INTRODUCTION

The development of storage-oriented alternatives for control of storm water runoff in portions of the Village of Fox Point required development of a technique for estimating the volume of runoff to be stored as a function of a specified design recurrence interval. The development of the technique and the manner in which it was applied by the Commission staff are described in this Appendix.

OVERALL APPROACH USED TO DEVELOP THE TECHNIQUE

The following three-step procedure was used to develop the technique for determining the volume of storm water runoff as a function of a specified design recurrence interval:

- Determine annual maximum rainfall event volumes by analyzing long-term rainfall records.
- Perform a rainfall volume-frequency (volume-recurrence interval) analysis on the annual maximum rainfall event volumes.
- Convert rainfall volumes, obtained from the rainfall volume-frequency relations for a specified design frequency or recurrence interval, to runoff volumes.

Each of the above three steps is described in detail below followed by an example of the use of the resulting runoff volume-frequency relationship.

DETERMINATION OF ANNUAL MAXIMUM RAINFALL VOLUMES

Source of Precipitation Data

The determination of annual maximum precipitation event volumes were based on about 37 years of hourly precipitation data—January 1, 1940 through October 31, 1976—as recorded at the Milwaukee National Weather Service station currently located at General Mitchell Field. This data had been previously obtained, verified, and placed in a computer file under the Commission's water resources planning program.

Milwaukee station precipitation data was used because that station is the only one in southeastern Wisconsin at which long-term hourly precipitation records are available. Precipitation recorded at the Milwaukee National Weather Service office is not likely to be identical to that which simultaneously occurred in the Village of Fox Point which is located 12 miles north of the station location at General Mitchell Field. However, the long-term characteristics or the statistical features of precipitation in the Village of Fox Point are likely to be very similar to those of Mitchell Field and, therefore, any statistical analysis of the Milwaukee precipitation data may be considered directly applicable to Fox Point.

All precipitation events—both rainfall and snowfall events—were analyzed. The largest annual precipitation events used in the analysis described below were assumed to be rainfall events, that is, events that would, relative to snowfall events, be more likely to cause immediate storm water runoff. The assumption that all largest annual precipitation events were rainfall events is not likely to introduce a significant error in the resulting statistical analyses of precipitation event volumes since less than 10 percent of the annual maximum precipitation event volumes used in the analysis occurred during the principal snowfall months of December, January, February, and March and some of these precipitation events did occur as rainfall. Furthermore, the magnitude of those precipitation events occurring during December, January, February, and March was such that they were not ranked near the largest events, thereby minimizing their potential effect on the higher recurrence interval events of interest, the five-, 10-, and 25-year events.

Definition of Precipitation Event

In a strict sense, a discrete precipitation event may be defined as a continuous or uninterrupted period of rainfall. The available historic precipitation 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. A schematic representation of rainfall events using a minimum one-hour antecedent and subsequent dry period is shown on Figure B-1.

The above strict definition of a precipitation event may not be appropriate for the purpose of analyzing runoff volumes from urban catchments or for planning detention or retention storage facilities. The rainfall-runoff response of a catchment and the functioning of the detention or retention storage facility being considered for a catchment may be such that a one-hour cessation of precipitation is of no practical concern. The minimum length of the antecedent and subsequent dry period used to define a precipitation event must be tailored to the intended use of the resulting rainfall volumes.

For example, assume that the time of concentration for an urban catchment is two hours; that is, two hours are required for surface water to move from the hydraulically most remote point of the catchment to the catchment outlet. Assume further that intermittent rainfall occurs over a five-hour period with one-hour rainfall lapses occurring during the second and fourth hours. Even though rainfall ceased during the two separate one-hour periods, runoff would continue without interruption during the entire period—and for two hours afterward—because of the two-hour time of concentration of the basin. Therefore, from the perspective of analyzing runoff, the minimum antecedent and subsequent time period used to define the precipitation events should be in excess of one hour in order that the number of runoff events might approximate the number of precipitation events.

Another example of the importance of tailoring the minimum length of antecedent and subsequent dry periods used to define rainfall events to the intended use of the resulting rainfall volumes is the relationship between the length of precipitation event and the period of time available to empty a detention or retention storage facility. Assume, for example, that a detention or retention facility intended to receive storm water runoff from an urban catchment is located so that it must be slowly evacuated after a rainfall event requiring approximately 24 hours to empty the facility. In this case, the minimum length of the antecedent and subsequent dry period used to define a precipitation event should be about 24 hours to assure that the design storm water runoff volume and the resulting storage facility are of sufficient magnitude so that the facility may be safely emptied prior to the occurrence of additional precipitation. Use of a much shorter minimum antecedent and subsequent dry period, such as one hour, would result in a smaller facility but one that would be much more likely to contain storm water prior to initiation of the next rainfall event.

Because of the apparent importance of the minimum length antecedent and subsequent dry period used to define precipitation events, the 37-year precipitation record was analyzed using a range of dry periods. More specifically, the number, time of occurrence, and volume of precipitation events during that period were determined using minimum antecedent and subsequent dry periods of 1, 2, 3, 6, 12, and 24 hours. This range was selected to encompass periods of time likely to be of interest either in analyzing catchment response or in considering the evacuation time of detention or retention storage facilities. The influence of minimum length of the antecedent and subsequent dry period on the number of precipitation events, is illustrated in Figure B-1.

Results

Table B-1 presents selected information about the precipitation events identified for each of the six minimum lengths of antecedent and subsequent dry periods including the number of events in 37 years, the average number of events per year, the volume of the largest and smallest events, and the volume of the median event. As would be expected, the total number of events in 37 years and the average number of events per year decreases as the minimum length of the antecedent and subsequent dry period increases. Also, the volume of the largest event during the 37-year period increases as the minimum 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 and the largest event had a volume of 3.42 inches. When the minimum antecedent and subsequent dry period is increased to 24 hours, the number of precipitation events in 37 years decreases 58 percent to 2,842, or an average of 77 per year, and the magnitude of the largest event increases by 81 percent to 6.20 inches.

RAINFALL VOLUME-FREQUENCY ANALYSIS

The 37 annual maximum rainfall volumes for each of the six minimum antecedent and subsequent dry periods were ranked in descending order, and the recurrence interval of each precipitation volume was calculated using the formula $(N + 1)/m$ where N equals the number of values in the series, that is, 37, and m equals the rank of each precipitation volume. The resulting recurrence intervals are presented graphically in Figure B-2 as precipitation volume versus recurrence interval in years or, alternately, versus probability of occurrence or exceedance in any year (the reciprocal of the recurrence interval) for each of the six minimum length antecedent and subsequent dry periods.

For a given recurrence interval, Figure B-2 indicates that the volume of precipitation associated with a given recurrence interval increases as the minimum antecedent and subsequent dry period increases. For example, for a recurrence interval of five years, the precipitation volume associated with a minimum antecedent and subsequent dry period of one hour is about 2.7 inches whereas the precipitation volume associated with a minimum antecedent and dry period of 24 hours is about 4.1 inches.

The variation in precipitation volume-recurrence interval relationships with the minimum length of the antecedent and subsequent dry period, as is evident on Figure B-2, could be generalized by preparing a "short duration dry period" curve—1 to 3 hours—and a "long duration dry period" curve—6 to 24 hours. This would be feasible since the precipitation volume-recurrence interval curves for minimum length antecedent and subsequent dry periods of 1, 2, and 3 hours tend to be grouped together and the curves for 6, 12, and 24 hours tend to be grouped together.

CONVERSION OF RAINFALL VOLUME TO RUNOFF VOLUME

The above curves permit 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 must be converted to a design direct storm water runoff volume for use in sizing a detention or retention storage facility. The conversion from precipitation volume to runoff volume was accomplished with an existing U. S. Department of Agriculture, Soil Conservation Service (SCS) procedure.[1] The SCS procedure converts a specified rainfall depth to direct runoff and incorporates average antecedent soil moisture conditions, hydrologic soil type, land use, and percent imperviousness.

For application of the SCS method to the Village of Fox Point study area, the percent imperviousness of the medium-density residential development in the Crossway-Bridge drainage area was determined to be about 40 percent and the dominant hydrologic soil group was determined from Commission detailed soil survey data to be B. Under this combination of land use and soil type and assuming average antecedent soil moisture conditions, precipitation volumes of 2, 3, 4, 5, and 6 inches may be expected to yield runoff volumes of 0.4, 1.0, 1.7, 2.5, and 3.3 inches.

USE OF THE RUNOFF VOLUME DETERMINATION METHOD

As indicated in the text, the critical minimum length of the antecedent and subsequent dry period was in the 12- to 24-hour range for the storm water detention facility alternative considered for the Crossway-Bridge drainage area in the Village of Fox Point. That critical duration was necessitated by the long evacuation time imposed on the subsurface storage facility as the result of the limited capacity of the existing storm sewer intended to receive and convey discharge pumped from the detention facility. Entering Figure B-2 with the specified design recurrence interval of five years and intersecting the volume-recurrence interval relationships about midway between the 12- and 24-hour relationships yields a design precipitation volume of about 3.8 inches. Using the aforementioned SCS procedure and land use and soils data for the Village of Fox Point and assuming average antecedent soil moisture conditions, a rainfall volume of 3.8 inches may be expected to yield a runoff volume of about 1.5 inches. Therefore, the subsurface detention facility was sized to accommodate 1.5 inches of rainfall. Once on the average of every five years, or with a 20 percent probability in any given year, a rainfall event may be expected that is preceded and followed by a dry period of at least 24-hours duration and during which the direct runoff volume will reach or exceed 1.5 inches.

Table B-1 indicates that for a minimum length antecedent and subsequent dry period in the 12- to 24-hour range, there have been approximately 3,100 precipitation events during the 37-year period for which hourly precipitation data were available. Approximately 99.5 percent of those precipitation events were less than 3.8 inches, the precipitation volume used to size the subsurface detention facility. Therefore, the subsurface detention facility may be expected to capture and contain the direct runoff from about 99.5 percent of the precipitation events occurring on the service area.

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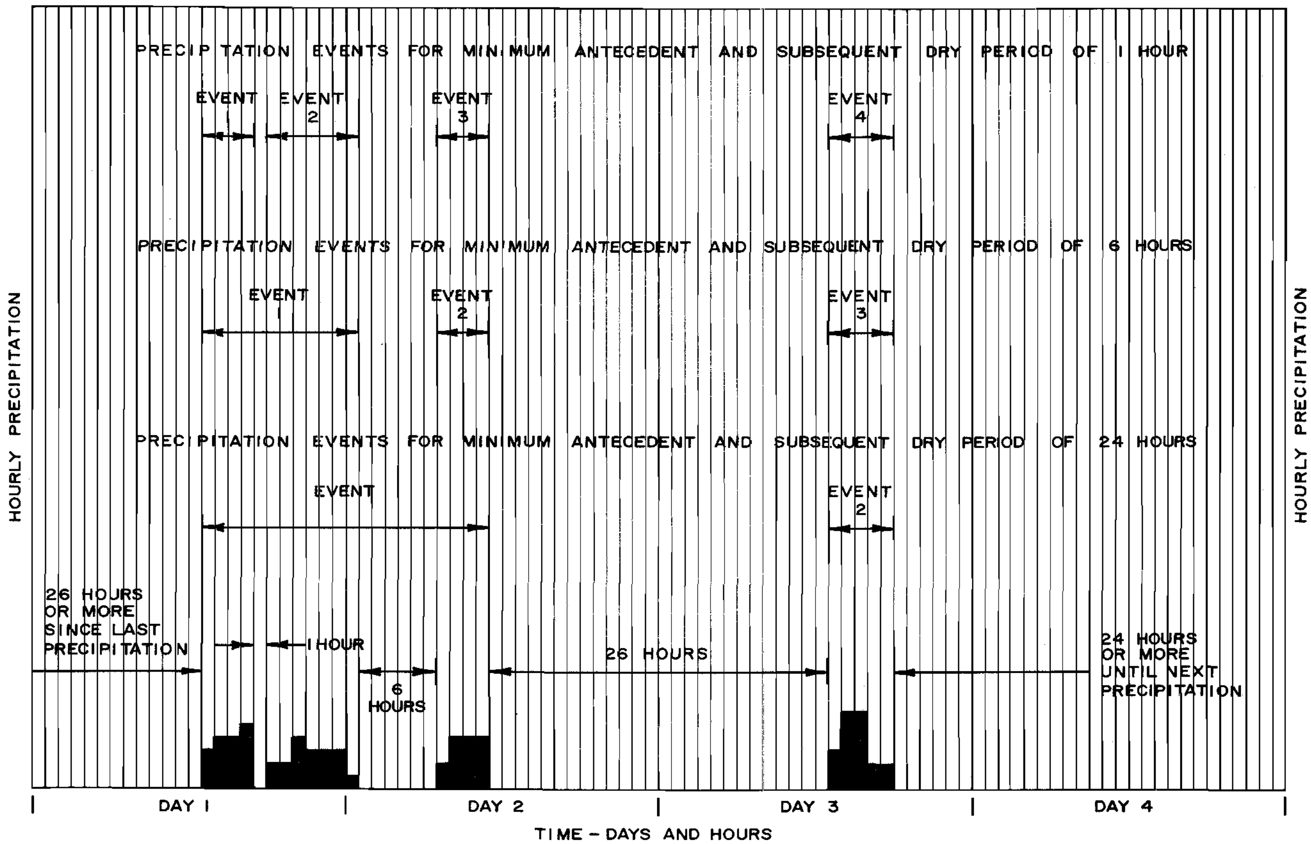
[1] *U. S. Department of Agriculture, Soil Conservation Service, Urban Hydrology for Small Watersheds, Technical Release No. 55, Chapter II, "Estimating Runoff from Urban Areas," January 1975.*

Figure B-1

SCHEMATIC REPRESENTATION OF THE DURATION OF A PRECIPITATION EVENT



Source: SEWRPC.

Table B-1

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

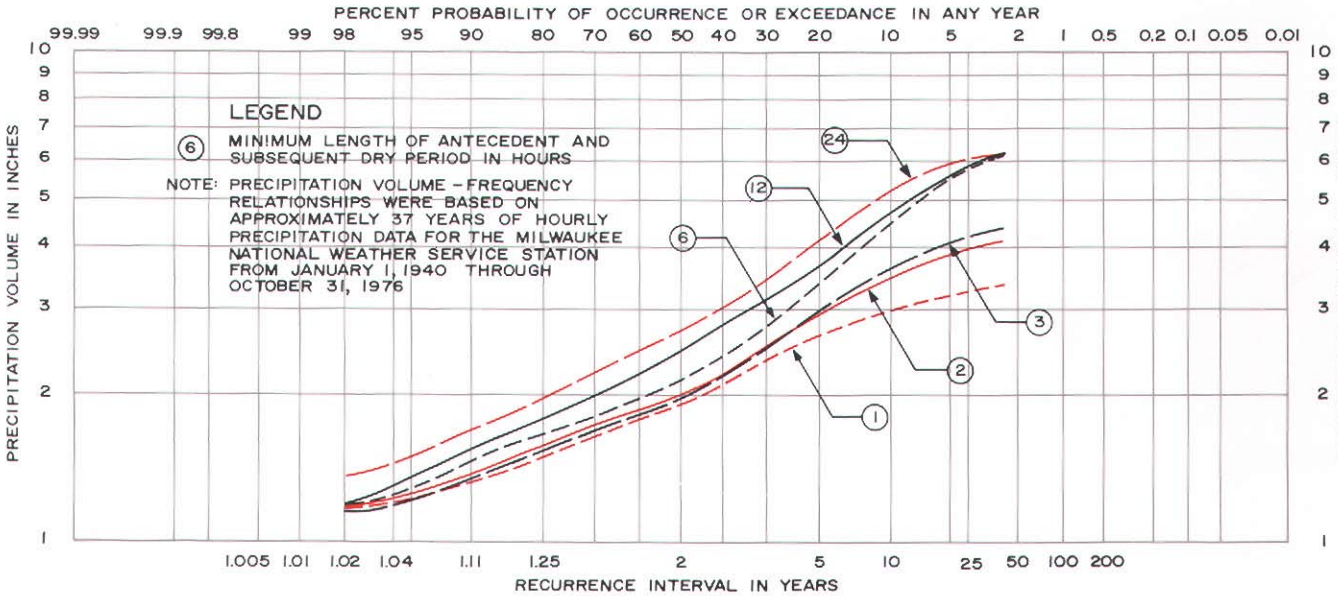
Minimum Antecedent and Subsequent Dry Period (hours)	Number of Precipitation Events		Smallest Event (inches)	Largest Event (inches)	Median Event (inches)
	In 37 Years	Average Per Year			
1	6,719	182	0.01	3.42	0.04
2	5,577	151	0.01	4.16	0.06
3	5,008	136	0.01	4.31	0.07
6	4,147	113	0.01	6.05	0.10
12	3,458	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: National Weather Service and SEWRPC.

Figure B-2

PRECIPITATION VOLUME–FREQUENCY RELATIONSHIPS FOR MINIMUM ANTECEDENT AND SUBSEQUENT DRY PERIODS OF 1, 2, 3, 6, 12, AND 24 HOURS



Source: SEWRPC.